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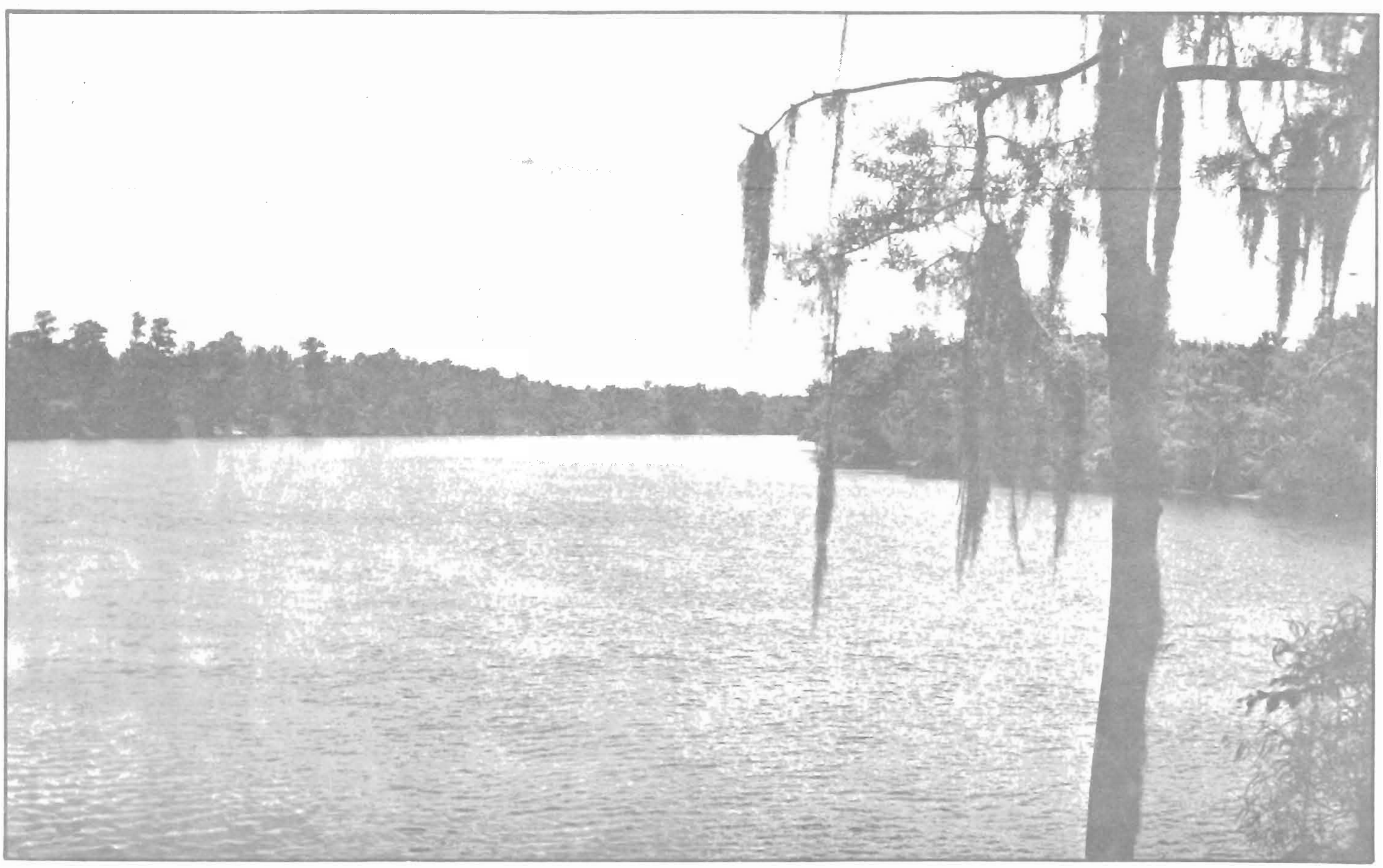
ECONOMIC AND PREDICTIVE RELIABILITY IMPLICATIONS OF STORMWATER DESIGN METHODOLOGIES

by

BRETT A. CUNNINGHAM
and
WAYNE C. HUBER

UNIVERSITY OF FLORIDA
Gainesville, Florida 32611

1987



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FLORIDA WATER RESOURCES RESEARCH CENTER
424 Black Hall
UNIVERSITY OF FLORIDA
Gainesville, Florida 32611

1987

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Abstract of Thesis Presented to the Graduate School
of the University of Florida in Partial Fullfillment of the
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ECONOMIC AND PREDICTIVE RELIABILITY IMPLICATIONS
OF STORMWATER DESIGN METHODOLOGIES

By

Brett Alan Cunningham

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Chairman: Dr. Wayne Huber
Major Department: Environmental Engineering Sciences

Conventional methodologies associated with a relatively low degree of predictive reliability are commonly used in stormwater runoff studies, even though more accurate methodologies exist. One of the reasons that they are persistently used over more accurate methodologies is because of the perceived difficulty and increased design costs associated with the more accurate methodologies.

Several conventional stormwater design methodologies and an alternative methodology based on continuous simulation are applied to five different areas that have been previously studied in order to determine tradeoffs in design costs, construction costs, and predictive reliability. The conventional methodologies give highly variable peak flow and total flow design values compared to the results from the alternative methodology, thus yielding large differences in construction costs. Shorter increment rainfall data would have allowed for more conclusive comparisons to be made between the design values from the conventional methodologies and the alternative methodology. Design costs for the alternative methodology are determined to be more than double those for most of the conventional methodologies. The

cost associated with the risk of over- or underdesign must ultimately be considered with the other costs to determine the most cost-effective methodology to apply to a given area.

CHAPTER 1

INTRODUCTION

Years ago, the only concern about stormwater was that it be quickly removed from an area so that it would not create a nuisance or cause flooding. Gradually, more concerns became associated with stormwater runoff. For example, the role of stormwater runoff in non-point source pollution is an issue that has received serious attention only in recent times. Since the number of important issues involving stormwater runoff has increased, the number of methods ordinarily used in design has also increased. These newer methods often require more time and detail than the more traditional methods that only addressed simpler issues. For example, the methodologies used to design detention basins are often more complex than the ones used to determine pipe sizes. Unfortunately, some of the methods commonly used to address the newer concerns and design the associated facilities have not been adequately developed to provide the degree of reliability and flexibility that is now required by many stormwater runoff studies.

Reliability, as used in this thesis, refers to predictive reliability, which in turn refers to the accuracy and consistency of the results from a given method. Risk, as used in this thesis, refers to the cost associated with an under- or over-designed system, not the traditional probabilistic definition of 1- reliability. Predictive reliability and risk can only be quantified if some standard of comparison is available.

Many newer methods also retain some of the undesirable aspects of the older methods like the rational method. Since the older and some of the newer methods are not associated with a high degree of reliability, a common practice has been to add a large safety factor to the design value. The engineer assumes that this safety factor will insure adequate system performance, and in some cases it might. However, two other undesirable possibilities exist: 1) the design values from the simple method multiplied by an arbitrary safety factor could yield an overdesigned system that is safe but not cost effective, and 2) the simple method could produce a value low enough that the system is inadequately sized even with the safety factor. The result of the second possibility could mean costly flooding. In some cases, the money saved in design by using a simple method may not be worth the risk of incurring one of the two possibilities mentioned above. The design of a dam that is upstream of a populated area is a fine example of one of these cases. An underdesign could result in the loss of lives, and an over-design could result in unnecessary land purchase and dam construction costs. Although methods that are more reliable than conventional methods may be more cost effective and desirable for some areas, they are generally not used. The perceived level of difficulty of some methods and the requirement by regulatory agencies to use a conventional method in order to ease their substantial administrative burdens often account for a more reliable, desirable method not being used.

Due in large part to the advent and advancement of the digital computer and its ability to manipulate large quantities of data, feasible methodologies with a degree of flexibility and predictive reliability that did not exist in years past are now available to the practicing engineer. The advent of comprehensive hydrologic/hydraulic computer models that accurately simulate hydrologic and hydraulic processes also account for the existence of these methodologies. In fact, using long periods of historical rainfall data as input to a hydrologic/hydraulic model (continuous simulation) represents what may be the state of the art in stormwater-study methodologies (McPherson, 1978). Continuous simulation offers advantages that no other method can offer. One might presume that a long-term streamflow record provides values that are more accurate than those determined from continuous simulation, and in some cases it might. However, very few areas have been gaged for a long period of time, and most of the ones that have been gaged for a long period of time have undergone gradual urbanization, which makes interpretation of the record very difficult. On the other hand, many long-term rainfall records exist. If the rainfall records are coupled with a model that can accurately translate them to runoff, an accurate runoff record for any level of urbanization can be created. This runoff record can then be analyzed to produce a reliable system design. One of the main advantages that this method offers over simpler methods is that it is based on accurate historical rainfall, not generalized rainfall characteristics. Continuous simulation with a comprehensive model is also much more flexible than simpler methods because of the greater number of processes that it can accurately simulate. Along with the tremendous amount of flexibility and reliability, however, comes an increase in design costs. In addition, more data are required than for the simpler methods, and the method is best suited to areas that have rainfall-runoff data. Unfortunately, very few areas have rainfall-runoff data. Also, the skills necessary to use an appropriate model are required. Clearly, no single method provides the optimal balance between reliability and design costs for every study area. This thesis attempts to quantify the tradeoffs involved between using continuous simulation to select design storms for more detailed analysis as opposed to some of the more conventional methods. The tradeoffs examined are in terms of reliability, design costs, and construction costs.

In order to examine the tradeoffs, several conventional methodologies and an alternative approach are applied to five areas that have been previously studied and for which large data bases exist. The first area is the Megginnis Arm Catchment in Tallahassee, FL. The other four sites are located in south Florida and shown in Figure 1. Table 1 lists the titles that are used to refer to the five areas throughout the thesis, and Table 2 gives some of the characteristics of the five basins. More information is given for each area in Chapter 3. Table 3 lists the various methods and the abbreviations used for them in this thesis.

Chapter 2 gives a description of the methods used along with some of the advantages, drawbacks, and other important aspects of each method. Some idea as to each method's level of reliability can be ascertained in Chapter 2.

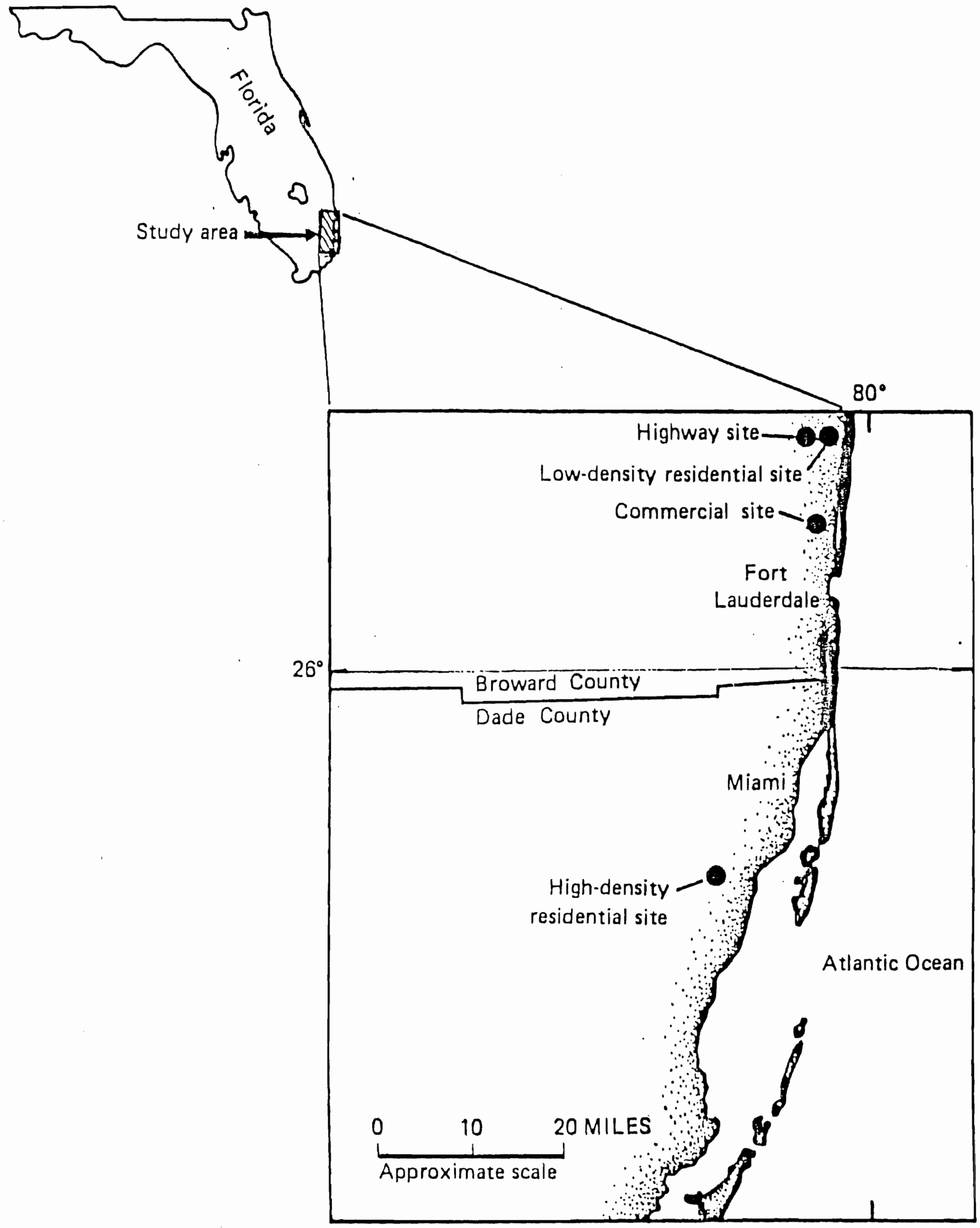


Figure 1. Map Showing the Location of the Four Study Areas in South Florida (Miller, 1979).

Table 1. Abbreviations Used for the Five Study Areas.

Location of Study Area	Abbreviation
Tallahassee, FL	MEG Basin
Pompano Beach, FL	LDR Basin
Broward County, FL	HWY Basin
Ft. Lauderdale, FL	COM Basin
Miami, FL	HDR Basin

Table 2. Some Basin Characteristics of the Five Study Areas.

	Basin				
	MEG	LDR	HWY	COM	HDR
Area, ac	1995	40.8	58.3	20.4	14.7
Imperviousness, %	28.3	5.9	18	98	44
Basin Slope, ft/ft	0.0216	0.0015	0.003	0.001	0.002
Hydrologic Soil Group	A	A	A	NA	D
Land Use	Mixed	Low Density Residential	Highway	Commercial	High Density Residential

Table 3. Abbreviations Used for the Various Methods.

Method	Abbreviation Peak Flow	Abbreviation Total Flow
Rational Method	RM	--
Synthetic Unit Hydrograph with		
Chicago Design Storm	SUH/CH	--
24-Hour SCS Type II Design Storm	SUH/SCS	--
SCS Peak Discharge Method	PDM	SCS
SCS Tabular Hydrograph Method	THM	SCS
Regression Equations	RE	--
Calibrated SWMM with		
Chicago Design Storm	CM/CH	CM/CH
24-Hour SCS Type II Design Storm	CM/SCS	CM/SCS
Alternative Method Based on Continuous Simulation	CS	CS

In Chapter 3 the results from the various methods are presented. Also, the parameters used for each method and a description how each method was applied is given. An examination of the results concludes Chapter 3.

The design costs in terms of man-hours associated with each method are given in Chapter 4. Differences in construction costs based on the 5-year return period peak flow and existing sewer data are also determined in Chapter 4.

Chapter 5 contains a summary of the thesis. Conclusions based on the information in this thesis are also presented in Chapter 5.

CHAPTER 2

DISCUSSION OF METHODOLOGIES

In order to compare the results based on continuous simulation modeling to those using conventional methodologies, several of the most commonly used rainfall-runoff methodologies and a methodology based on continuous simulation were applied to five study areas. A discussion of the methodologies used in this thesis is given in this chapter. The discussion includes a description, drawbacks, advantages, and other pertinent aspects of each methodology. The advantages and drawbacks are summarized for all of the methodologies in Tables 4 and 5. Refer to Table 3 for the list of abbreviations. Note that in Tables 4 and 5 the SUH is used for both SUH/CH and SUH/SCS, CM is used for both CM/CH and CM/SCS, and RE is not included.

Synthetic Design Storms

A design storm is a rainfall event which is developed for the design of particular hydrologic structures, and the return period of the design parameter of interest is assumed to be equal to the return period of the storm (Arnell, 1982). For example, the return period for the peak flow obtained by using a 10-year return period design storm developed for pipe sizing is assumed to be 10 years.

The design storms discussed in this section are synthetic design storms, which are storms based on rainfall (not runoff) statistics.

Although the storms are not a method by themselves, they are used or implied in the conventional methods used in this thesis and therefore need to be discussed.

Table 4. Advantages Offered by the Various Methods.

Advantage	Method(s) Containing Advantage
Easy, quick to use	RM, SUH, PDM, THM
Yields a hydrograph	SUH, THM, CM, CS
Easy to administer	RM, PDM, THM
Has been used for many years by a large number of people	RM, PDM, THM
Can easily evaluate various conditions and control alternatives	PDM, THM, CM, CS
Determines peak flow and total runoff volume	PDM, THM, CM, CS
All the necessary data can be obtained in a short period of time	RM, SUH, PDM, THM
Can account for a wide range of hydrologic processes and occurrences (e.g., snowmelt, high water table, reservoir routing, etc.)	CM, CS
Capable of handling a problem of almost any size	THM, CM, CS
Antecedent conditions are known	CS
Return period of parameter of interest based on frequency analysis of that parameter	CS
Uses real rainfall data	SUH,CS

Table 5. Drawbacks of the Various Methods.

Drawback	Method(s) Containing Drawback
Parameter of interest based on rainfall statistics	RM, SUH, PDM, THM, CM
Rainfall duration, temporal distribution assumed	SUH[a], PDM, THM, CM
Antecedent conditions assumed	RM, SUH, PDM, THM, CM
Practical limit on size of area that can be studied	RM, PDM, SUH
Difficult to evaluate control alternatives and levels of development	RM, SUH
Volume of storm obtained from IDF curve	RM, SUH, PDM, THM, CM
Best suited to area with rainfall-runoff data	CS
Computer required	THM, CS
Knowledge of computer model required	CS
More difficult and time consuming than other methods	CM, CS

[a]Only when synthetic rainfall is used.

A duration of 24 hours was used for the storms. This value was selected because it is commonly recommended by regulatory agencies. There is, however, no real scientific significance for using that particular duration.

Intensity-Duration-Frequency Relations

To understand why designs based on intensity-duration-frequency (IDF) curves are not necessarily associated with a high degree of reliability, it is important to discuss the derivation of the curves. First, the maximum average intensity for a wide range of durations is taken out of a number of independent storm events; the number of events depends on the period of record. If an actual storm has a duration that is less than the duration of interest, then the maximum average intensity is calculated as the total storm depth divided by the duration of interest, even if it is longer than the actual storm duration. This practice introduces "extended duration" values, or dummy values, for the larger durations, which distort the tails of the IDF curves. Second, a frequency analysis is then used to determine the return period associated with each depth for a given duration. Finally, the average intensities (or depths) for the different durations are plotted, and smoothed curves representing fixed return intervals are fitted through the points (McPherson, 1978; Arnell, 1982).

Several limitations can be seen upon examination of the above method. First, each smoothed curve can represent data from different storms that could have occurred many years apart from one another. Therefore, calling rainfall from an IDF curve an "n-year storm" is misleading and incorrect. Second, an IDF curve gives no indication of the timing of the maximum average intensity of the storm--a factor that can drastically affect the peak flow rate and sizing of detention basins. Third, the volume of rainfall during the maximum-average-intensity portion may only be a small fraction of the total storm volume. Fourth, any information about the storm dynamics is completely obscured. Lastly, for areas where rainfall data are not available, the IDF relationships are often derived from the isopluvial maps in the U.S. Weather Bureau Technical Paper Number 40 (Hershfield, 1961). Since these maps were developed from processed data including smoothing, translation, and interpolation, the IDF curves for these areas include an even greater amount of uncertainty (Tung, 1987).

Importance of Hyetograph Discretization

Hyetograph discretization, or the time interval used for rainfall intensities, can be an important design parameter to consider. For this thesis hourly rainfall was used for all applications. It was apparent after the methods had been applied to the five areas that a shorter time interval would have been more appropriate for several situations, especially for the synthetic unit hydrograph method and the COM basin. Ordinarily, an estimate of the time of concentration should be made in order to determine an appropriate hyetograph time interval. The rational method, when properly used with the kinematic wave eqution and wave travel times, is one possible way that an estimate of the time of concentration for a given basin and return period can be obtained. In general, time of concentration decreases with increasing imperviousness, rainfall intensity, and slope, and decreasing surface roughness, area, and soil permeability.

The topic of hyetograph discretization exposes another drawback of synthetic design storms. Many synthetic design storms were not developed using short time increment (15 min. or less) rainfall data (see discussion of SCS 24-hour type II below), so the practice of using a synthetic design storm with a short hyetograph time interval may be questionable.

Chicago Design Storm

The Chicago design storm was developed from IDF curves and an assumed temporal distibution in 1957 (Keifer and Chu, 1957). The storm is to be used as rainfall input to determine peak flow rates. Keifer and Chu determined that the three most important rainfall characteristics affecting peak flow are the volume of rainfall within the period having the maximum intensity, the amount of antecedent rainfall, and the location in the storm of the peak rainfall intensity.

Several studies were performed to determine the ratio of the time prior to peak intensity to the total duration (Keifer and Chu, 1957; Preul and Papadakis, 1973; Sifalda, 1973), and the value was consistently determined to be around 3/8. Once this value was known, the storm could be developed to meet the three important characteristics mentioned above. Figure 2 shows an example hyetograph of the Chicago synthetic design storm. Appendix A contains the dimensionless hourly intensities of the Chicago storm for a 24 hour duration.

Chicago design storms with return periods of 2, 5, 10, and 25 years were developed for this thesis by assuming a 24-hour duration, determining the total depths from IDF curves for that duration and the above return periods, and scaling the storms to match those depths.

SCS 24-Hour Type II Design Storm

The SCS Type II 24-hour design storm (which will be referred to from now on as the SCS design storm) is a dimensionless storm developed by using the Weather Bureau's Rainfall Frequency Atlas. To construct the design storm, 30-minute incremental depths were determined from generalized depth-duration-frequency curves. Those depths were

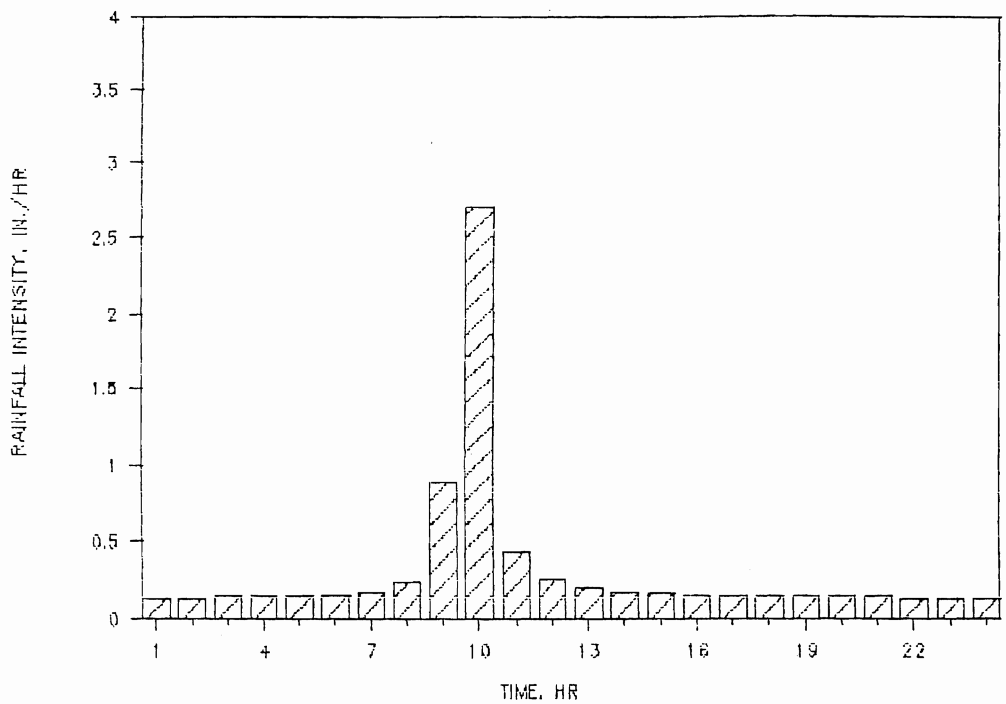


Figure 2. Example Hyetograph of the Chicago Design Storm.

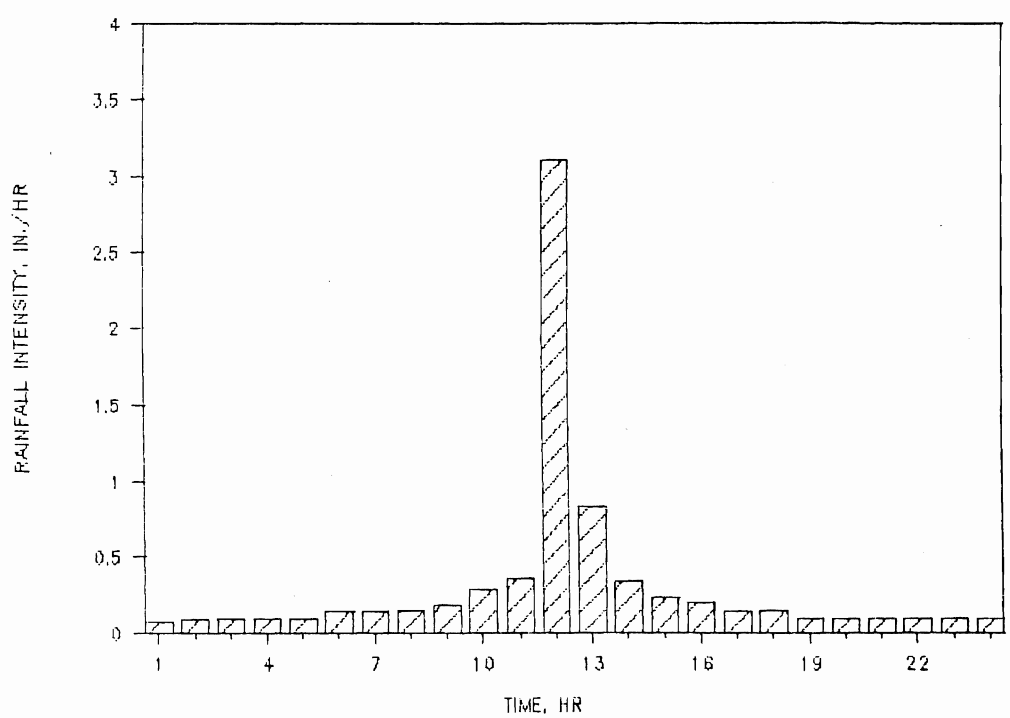


Figure 3. Example Hyetograph of the 24-Hour SCS Type II Design Storm.

then arbitrarily arranged so that the greatest depth occurs in the middle and the smaller depths decrease in magnitude on either side of the greatest depth (Soil Conservation Service, 1972). The resulting hyetograph from the SCS design storm has a very sharp peak in the middle with the intensities trailing off rapidly on either side of the peak, as shown in Figure 3. Appendix B contains the dimensionless hourly intensities for the storm. The SCS design storm was developed for this study in the same manner and for the same return periods as the Chicago design storm.

Rational Method

The rational method is a rainfall-runoff method that was introduced in the United States in 1889 (Kuichling, 1889) and has since become the most common method in this country for determining stormwater runoff peak flows (Clark et al., 1977). The rational method is based on the following equation:

$$Q = cia \tag{1}$$

where Q = peak runoff rate, cfs,
c = runoff coefficient, equal to the ratio of the peak runoff rate to average rainfall intensity over the time of concentration,
i = average rainfall intensity, in./hr, over the time of concentration,
a = drainage area, acres

The conversion factor of 1.008 ac-in./hr/cfs is usually neglected, and it is because of these units the formula was termed "rational".

The basis of the rational method lies in the assumption that the peak flow at any given point in the system is a direct function of the contributing drainage area and the maximum average intensity during the time of concentration for the most remote part of the contributing area. The last part of this assumption is based on the rationale that the maximum flow rate will occur when all parts of the drainage area are contributing to the outflow. Although no successful attempts to verify the basis of the rational method have been made to date using natural rainfall, experiments using simulated rainfall on small, impervious areas coupled with the kinematic wave equation (Eagleson, 1970) have yielded evidence that the basis is correct under certain conditions.

One of the most common problems with the rational method is the selection of the time of concentration. Most references tell how to estimate the time of concentration by first calculating the overland flow velocity and the channel or pipe velocity. The time of concentration is then determined by dividing the length of the greatest overland flow distance by the overland flow velocity and adding that time to the pipe or channel travel time. The calculation for overland flow velocity is normally made independently of the rainfall intensity. Two mistakes are being made by using this method. First, the velocities should depend on the rainfall intensity. It makes sense that the overland flow time should decrease as the rainfall intensity increases because the depth of flowing water will be greater. The same reasoning holds for the channel or pipe velocity. Second, the time of concentration should be based on wave speeds, not velocities, because the passage of waves determines how long it will take for the most hydraulically remote point in the catchment to contribute to runoff (Eagleson, 1970).

Unit Hydrograph Method

The basic theory of the unit hydrograph method of describing direct runoff was first suggested by Folse (1929), and the basic concept behind the method was introduced by Sherman (1932). Those concepts are reviewed in the following section. A unit hydrograph is the direct runoff hydrograph resulting from a direct runoff volume of 1 in. over a specified drainage area and for a specified duration of rainfall excess.

Unit hydrographs can be developed by two means. The first way involves using actual rainfall-runoff records from a gaged watershed to determine the size and shape of the hydrograph. The second way employs certain watershed characteristics to develop a synthetic unit hydrograph. The latter method is not generally as accurate as the first, but has the advantage of applicability to ungaged watersheds. A number of synthetic unit hydrographs are available (U.S. Army Corps of Engineers, 1959; Snyder, 1938; Taylor and Schwartz, 1952; Mockus, 1957; Gray, 1961). The one chosen for this study is a ten-minute unit hydrograph that was developed by Espey et al. (1977) for urban watersheds. A ten-minute unit hydrograph was chosen because of the small times of concentration for four of the five watersheds studied. A longer duration unit hydrograph would be more appropriate for the Megginnis Arm Catchment, so the results of this method for that site must be viewed with that in mind. In addition, hourly rainfall in the form of the two synthetic design storms discussed earlier (Chicago and SCS) was used for rainfall input. Judging from the results presented in the next chapter, a much shorter time increment rainfall needed to be used in order to adequately assess the results from this method. As stated previously in this chapter, however, the practice of using certain synthetic design storms with short-time-increment intensities may be questionable.

Three basic assumptions are essential to the concept of the unit hydrograph. First, the durations of direct-runoff hydrographs resulting from similar storms of equal duration, regardless of the intensity of the rainfall, are assumed to be equal for a given watershed. Second, the ordinates of the direct-runoff hydrographs from similar storms of equal duration are assumed to be proportional to the volume of direct runoff for a given watershed. Third, for a given watershed, the time distribution of direct runoff from a particular storm is assumed to be independent of that produced by any other storm period (Morgan and Johnson, 1962). In order to obtain a hydrograph for a storm whose duration is longer than that of the derived unit hydrograph, superposition can be used. By using superposition, one assumes that the linear response of the watershed is not influenced by previous storms. This assumption has been shown not to be entirely true, but the method provides a means of quickly producing a hydrograph for a longer duration storm once the rainfall excess has been determined. Some of the advantages and drawbacks of this method are noted in Tables 4 and 5 by the abbreviation SUH.

Regression Equations

A nationwide study of flood magnitude and return intervals in urban areas was undertaken by the United States Geological Survey, among other reasons, to develop methods of estimating urban flood characteristics based on basin characteristics (Sauer et al., 1983). Three sets of regression equations resulted: one set based on three independent parameters, and the other two based on seven independent parameters. All three sets of equations require an independent estimate of the equivalent rural discharge for the ungaged basin. Another set of regression equations designed for estimating flood magnitude and return interval for natural-flow streams in Florida was used to determine the equivalent rural discharge (Bridges, 1982). One of the seven-parameter equations was used to determine the urban discharge.

Since this methodology is not widely practiced and its inclusion in this study is only for comparison, a description of the assumptions, limitations, and advantages will not be given.

Although the south Florida sites used in this thesis lie slightly south of the southernmost area of applicability of the equations developed by Bridges, his equations were applied to these areas because of the lack of any other suitable regression equations. The results from the regression equations for the four south Florida areas should be interpreted with that in mind.

SCS Peak Discharge Method

The peak discharge method is based on SCS Technical Release No. 55 (1972, 1986) and has been modified for Florida use (Livingston et al., 1984). Because the method directly accounts for more factors than the rational method, engineers generally assume it to be more accurate than the rational method. The stated principal applications of the method are estimating peak runoff rates and total volumes.

Although a complete discussion of all of the assumptions involved in this empirical method will not be given, it is necessary to point out a couple of the more interesting ones. The method assumes that if everything but storm duration or intensity is equal for two storms then the estimate of runoff for the two storms is equal. The method ignores the time distribution, which can have a significant impact on the magnitude of the peak flow and total volume. Also, the method uses the SCS 24-hr Type II design storm, which was developed for the determination of peak flows, to determine the total runoff volume. A quick review of the development of that particular design storm suggests that the return period of the calculated runoff volume may be substantially different from that of the rainfall.

For this thesis, average antecedent moisture conditions (AMC II) were assumed for all cases.

SCS Tabular Hydrograph Method

The tabular hydrograph method is very similar to the peak discharge method. The main difference is that the former yields a hydrograph and the latter does not. The tabular hydrograph method used for this thesis is a computerized version, but it is not modified for Florida use (Soil Conseration Service, 1986).

Alternative Methodology Using Continuous Simulation

Continuous simulation can be used as the basis of an arguably superior alternative design methodology. Many models are capable of being used for continuous simulation. The earliest model used for such purposes was the Stanford Watershed Model (Crawford and Linsley, 1966). The HSPF Model (Johanson et. al, 1980) and STORM (Hydrologic

Engineering Center, 1977; Roesner et. al, 1974) are two widely used models capable of performing continuous simulations. The EPA Storm Water Management Model (SWMM) (Huber et al., 1981; Roesner et al., 1981) was chosen over the above models to perform the simulations for a variety of reasons (Huber et al., 1986). The following list describes the steps involved in using continuous simulation in stormwater design.

1. Calibrate and verify a model on the catchment.

2. Perform a continuous simulation using as long a historical rainfall record as possible.

3. Perform frequency analyses on parameters of interest, e.g., peak flow and total volume.

4. Select historical rainfall events with accompanying antecedent rainfall and run through model again using a more detailed simulation.

None of the questionable assumptions inherent to the methods previously discussed are necessary when using this alternative design methodology. The main assumption that must be made is that the calibration is robust, that is, that it produces a good fit, on the average, for many storms (Maalel and Huber, 1984). The only other assumptions that are involved are those that are inherent in the model itself.

In addition to the advantages listed in Table 4, this methodology offers several other advantages. It is known that real storms can have high spatial variability and that runoff can be very sensitive to storm movement (James and Shtifter, 1981; Surkan, 1974). Unlike the other methodologies which also rely on point rainfall, it is possible with this methodology to study the effects of storm dynamics on basin response (James and Scheckenberger, 1984). Also, local governments can explain and defend a design based on a real storm much easier than one based on a synthesized formulation (McPherson, 1978).

Although this methodology is best suited to areas where rainfall-runoff data are available, it can be applied to areas where those data are not available, but some decrease in reliability can be expected. In general the most sensitive parameters in a SWMM calibration are the imperviousness and the subcatchment width term. (Increasing the width term in SWMM is equivalent to increasing the slope or decreasing the surface roughness). The lack of rainfall-runoff data should not usually affect the estimate of any parameter, with the exception of the subcatchment width. Without the aid of rainfall-runoff data for calibration, the estimate used for the subcatchment width has generally been found to yield peak flows that are within 50 percent of the actual value. One way to offset the reduction in reliability when rainfall-runoff data are not available is to use a finer schematization. Although this involves more work and computer time, it usually introduces greater accuracy.

In the next chapter the results from the methodologies discussed in this chapter and applied to the five sites mentioned in Chapter 1 are presented. Also, a description of how the methodologies were applied is given. A discussion of the results concludes the chapter.

CHAPTER 3

CASE STUDIES

The five areas chosen for this study were selected for several reasons. First, all of the areas have been previously studied, so fairly extensive data bases exist on each one of them, especially the four in south Florida. Second, rainfall-runoff data exist for all of the areas, so model calibration can be readily accomplished. Third, a long-term precipitation record from a nearby weather station is available for all five areas.

Peak flow values were obtained for each of the five areas by seven methods. Two of the seven methods give two sets of peak flow values, meaning a total of nine sets of peak flow values were determined for each area. Values of total flow volume were determined by four of the nine methods. Table 3 in Chapter 1 contains a list of the abbreviations that are used for the methods.

Tallahassee, FL (MEG Basin)

The Megginnis Arm Catchment (henceforth referred to as the MEG basin) in Tallahassee, FL consists of approximately 2000 flat to moderately-sloping acres, as can be seen in Figure 4. Most of the soils in the catchment have relatively high hydraulic conductivities and are well drained. Land uses include commercial, residential, and undeveloped zones. Runoff from the area drains into Lake Jackson. A detention basin and artificial marsh exist at the lower portion of the

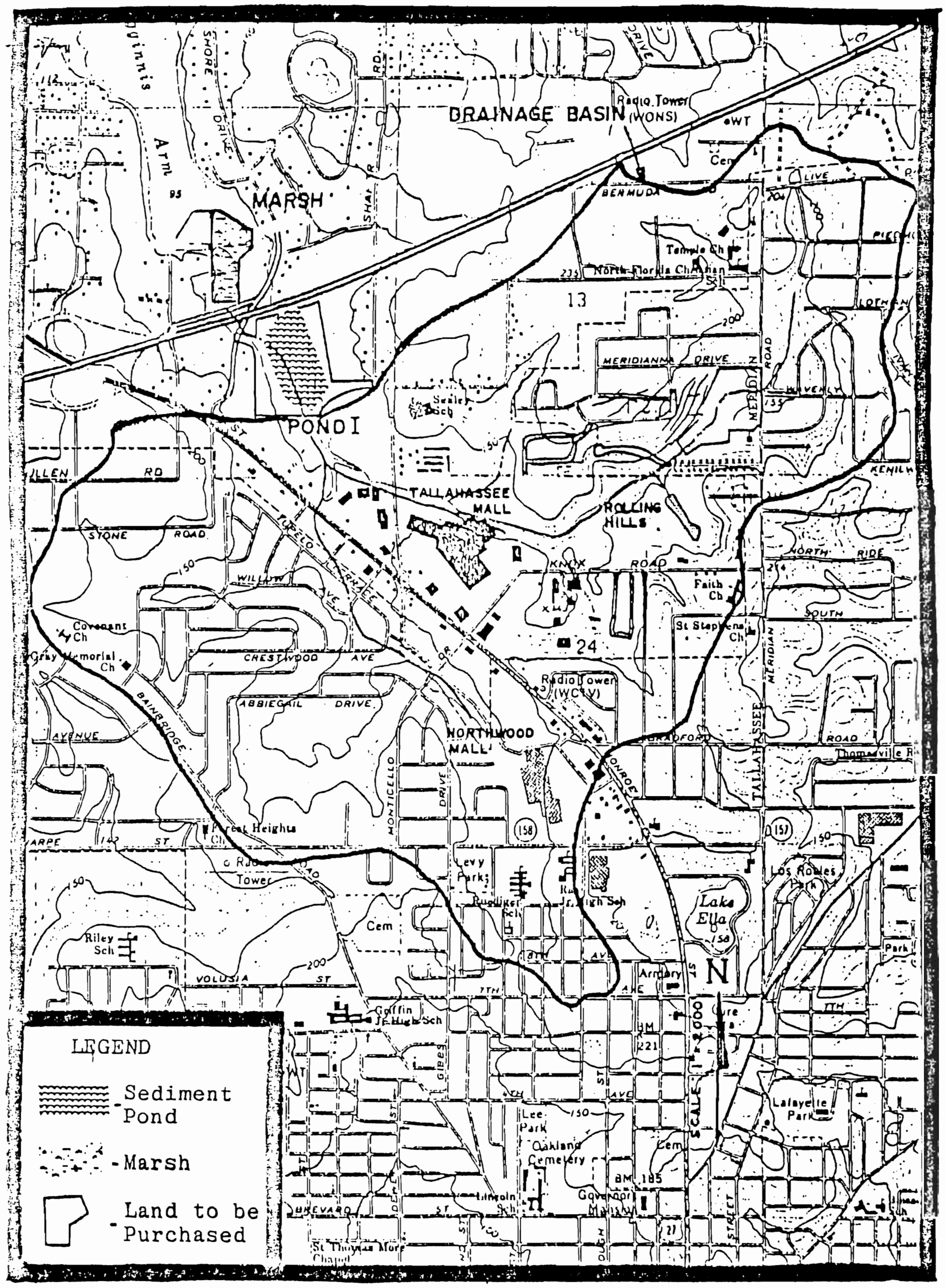


Figure 4. Topographic Map of the MEG Basin (Esry and Bowman, 1984).

catchment for the purpose of water quality control (Esry and Bowman, 1984).

Results

The results from the seven different methodologies applied to the MEG basin are described below. In addition, results from previous USGS flood frequency studies are included in this section for comparative purposes. Parameters used for the seven methodologies are given in Table 6. Some of the parameters came from previous studies (Esry and Bowman, 1984; Franklin and Losey, 1984). A summary of the results from all of the methods is contained in Tables 11 and 12 at the end of this section.

Rational method

The time of concentration was found by adding the overland flow time of a wave based on the kinematic wave equation (Eagleson, 1970) to travel time based on wave speed in the channels. Depths were adjusted in the channels for each return period in order to realistically represent the amount of flow in the channels. The runoff coefficient, c, was selected on the basis of hydraulically effective impervious area, soil type, and return period (Clark et. al, 1977). The coefficient was arbitrarily increased for increasing return periods in order to reflect the reduced soil storage capacity. Other factors such as surface cover and slope were accounted for in the equations used to calculate the time of concentration. Lotus 1-2-3 (Lotus Development Corporation, 1986), an electronic spreadsheet, was used to expedite the iterative process involving intensity and time of concentration. IDF curves from the region were used in the time of concentration calculations (Weldon, 1985). The values used in the rational method calculations are listed in Table 7.

Table 6. Parameters Used for the Methods Applied to the MEG Basin.*

Watershed Drainage Area		1995 ac
		3.12 sq mi
Length of Main Channel		11000 ft
Main Channel Slope		0.01 ft/ft
Hydraulically Effective Impervious Area		28.3 %
Average Basin Slope		0.0216 ft/ft
Soil Group		A
Curve Number		70
Conveyance Efficiency		1.0
Adjustment Factors		
Impervious Area		1.18
Hydraulic Length		1.06
Slope		1.31
Ponding		
Return Period yr	Ponding Adjustment Factor	
2	0.69	
5	0.70	
10	0.71	
25	0.74	
Lake Area		1 %
2 Year, 2 Hour Rainfall Intensity		2.7 in.
Basin Development Factor		7
Basin Storage		.11 %
Subcatchment Width		6300 ft
Manning's n		
Pervious Area		0.35
Impervious Area		0.015
Green-Ampt Infiltration Parameters		
Hydraulic Conductivity		5.76 in./hr
Capillary Suction		18.13 in.
Initial Moisture Deficit		0.15

* Generally not SWMM input values. See Appendix C.

Table 7. Parameters from the Rational Method--MEG Basin.

Return Period yr	Wave Pipes min	Travel Time Overland Flow min	Time of Concentration min	Rainfall Intensity in./hr	c
2	28.3	71.7	100	1.62	.28
5	26.6	61.4	88	2.19	.31
10	25.4	56.6	82	2.57	.33
25	23.9	51.1	75	3.16	.35

Synthetic unit hydrograph method

Five watershed parameters were required to determine the six given points on the hydrograph and are included in Table 6. The conveyance efficiency term is a measure of the drainage conditions and is expressed as a value on a scale of 0.6 to 1.3. Once the six points were known, the hydrograph was developed by adjusting the rest of the points via Lotus 1-2-3 until a depth of one inch and a reasonable shape were obtained. Rainfall input consisted of the two synthetic design storms scaled down by the fraction of hydraulically effective impervious area.

SCS peak discharge method

The six watershed parameters required for the peak discharge method are included in Table 6. Once those parameters were determined, the 24-hour rainfall depths for the given return periods were found (Hershfield, 1961). These depths were then translated into a runoff depth by the curve number equation. The above values are listed in Table 8. The peak discharge rates for the four return periods were calculated by multiplying the runoff depths by various factors found in tables and charts that are supplied with the method (Livingston, 1984).

Table 8. Rainfall and Runoff Depths from the SCS Peak Discharge Method--MEG Basin.

Return Period yr	Rainfall Depth in.	Runoff Depth in.
2	4.75	1.85
5	6.5	3.00
10	7.5	4.04
25	8.5	4.90

SCS tabular hydrograph method

Most of the values necessary for the tabular hydrograph method are identical to those used in the PDM. The only additional information needed was the total time of concentration, and that was calculated by using formulas supplied by the method (Soil Conservation Service, 1986).

Regression equations

The regression equation used to determine predevelopment peak flow requires three parameters: 1) drainage area, 2) channel slope, defined as the average slope of the main channel between points 10 and 85 percent of the distance upstream from the inlet to the basin divide, and 3) lake area. Four more parameters are required to determine the urban peak flow. They are 1) rainfall intensity for the 2-hour 2-year occurrence , 2) basin storage, 3)basin development factor on a scale of 0-12, and 4) impervious area.

Calibrated SWMM with synthetic design storms

Both the SCS and Chicago design storms were used as rainfall input to the calibrated SWMM. The calibration and verification are discussed in the next section. Average antecedent moisture conditions were used.

Design storms from continuous simulation

Parameters for the SWMM simulations came from various sources. Monthly evaporation values were gathered from National Weather Service values (Farnsworth and Thompson, 1982), from which evapotranspiration values were estimated by multiplying by a pan coefficient of 0.7. The soil types, which are primarily sand, were identified from a county soil survey map. The Green-Ampt infiltration parameters of hydraulic conductivity and capillary suction were estimated from data published on Florida soils (Carlisle et al., 1981). A weighted average over the soil types was used to determine the final infiltration parameters since a single subcatchment schematization was used. A five subcatchment schematization was also calibrated and verified, but since the results were only marginally better, the only results reported in this paper are for the single subcatchment schematization. The percent imperviousness used was the same as that found in an earlier study on the area (Franklin and Losey, 1984). Manning's n values were selected by determining the average surface cover and finding the appropriate value on a chart (Crawford and Linsley, 1966). Average catchment slope was determined by choosing eight points along the edge of the catchment, calculating the path length of each point to the inlet, dividing the path lengths by the change in elevation, and taking a weighted average of the eight slopes.

Rainfall-runoff data for calibration and verification were obtained from USGS records for the catchment. A total of 10 of the largest storms recorded from 1979-81 was used for calibration and verification. Calibration was carried out by simultaneously running five storms using identical catchment parameters (Maalel and Huber, 1984). Sufficient results for both total flow and peak flow were obtained by adjusting only the width parameter in SWMM. Results of the peak flows and total flows from the calibration runs are shown in Figures 5 and 6, respectively. Hydrographs of predicted and measured flows from the calibration storms with the best fit and worst fit are displayed in Figures 7 and 8, respectively.

Verification was accomplished by running the remaining five of ten storms using the same parameters as used in the final calibration runs. Results from the verification runs were comparable to those from the calibration runs, as can be seen in Figures 5 and 6. Hydrographs of predicted and measured flows from the verification storms with the best fit and worst fit are displayed in Figures 9 and 10, respectively.

A rainfall record of 21.6 years of hourly data from the Tallahassee Airport was used for the continuous simulation. The airport lies approximately 7 miles southwest of the catchment. Statistical analysis of the predicted peak flows was done using the Statistical Block of SWMM. The time series was broken into 1485 independent storm events by varying the minimum interevent time (MIT) until the

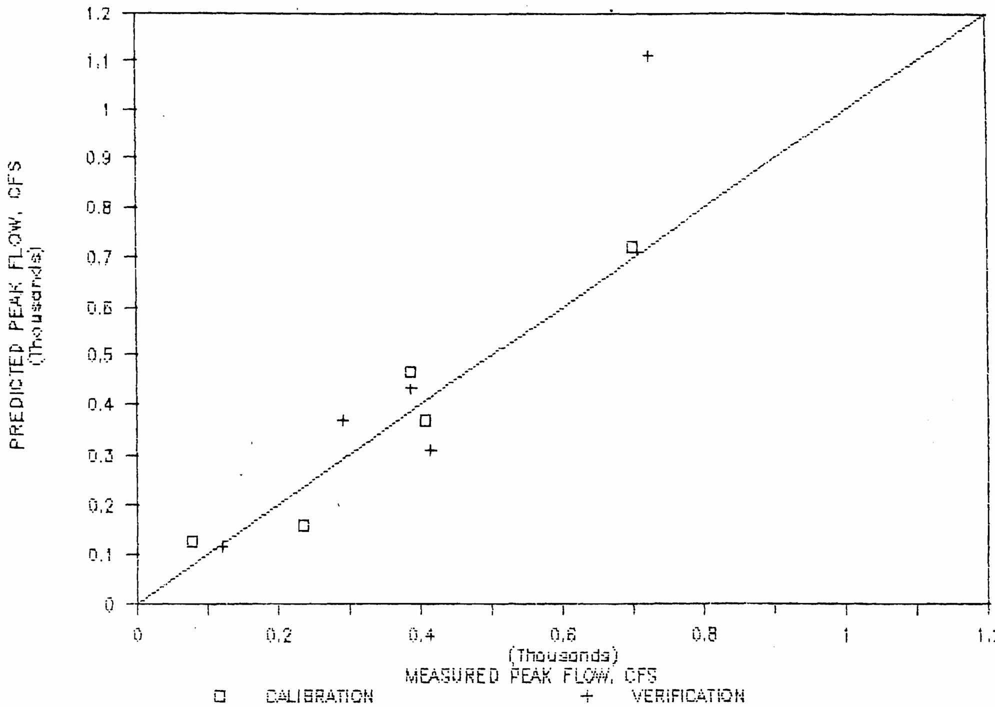


Figure 5. Peak Flow Results from the Calibration and Verification Runs--MEG Basin.

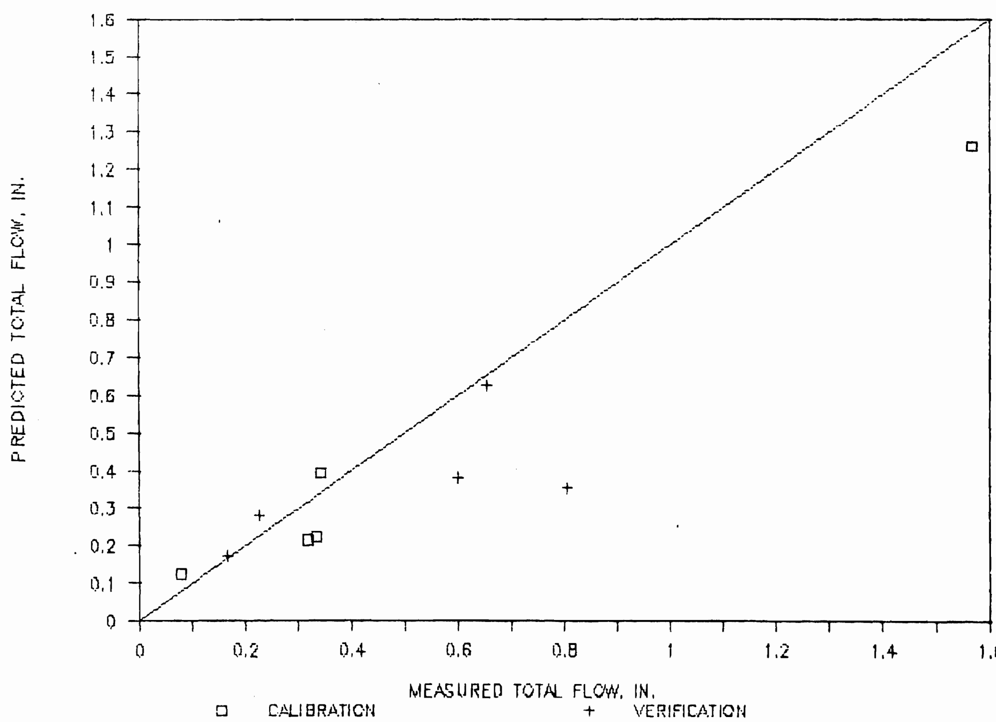
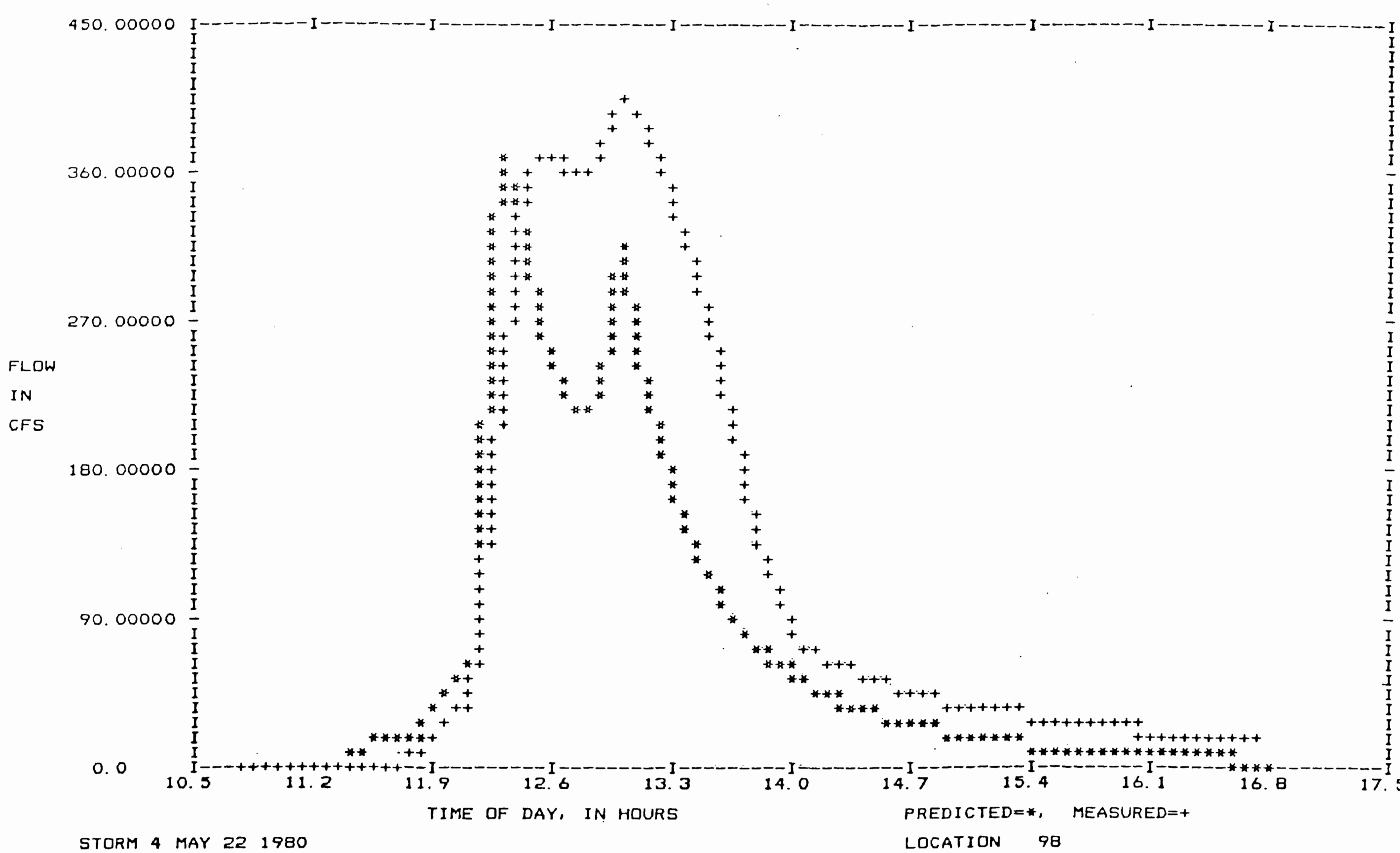


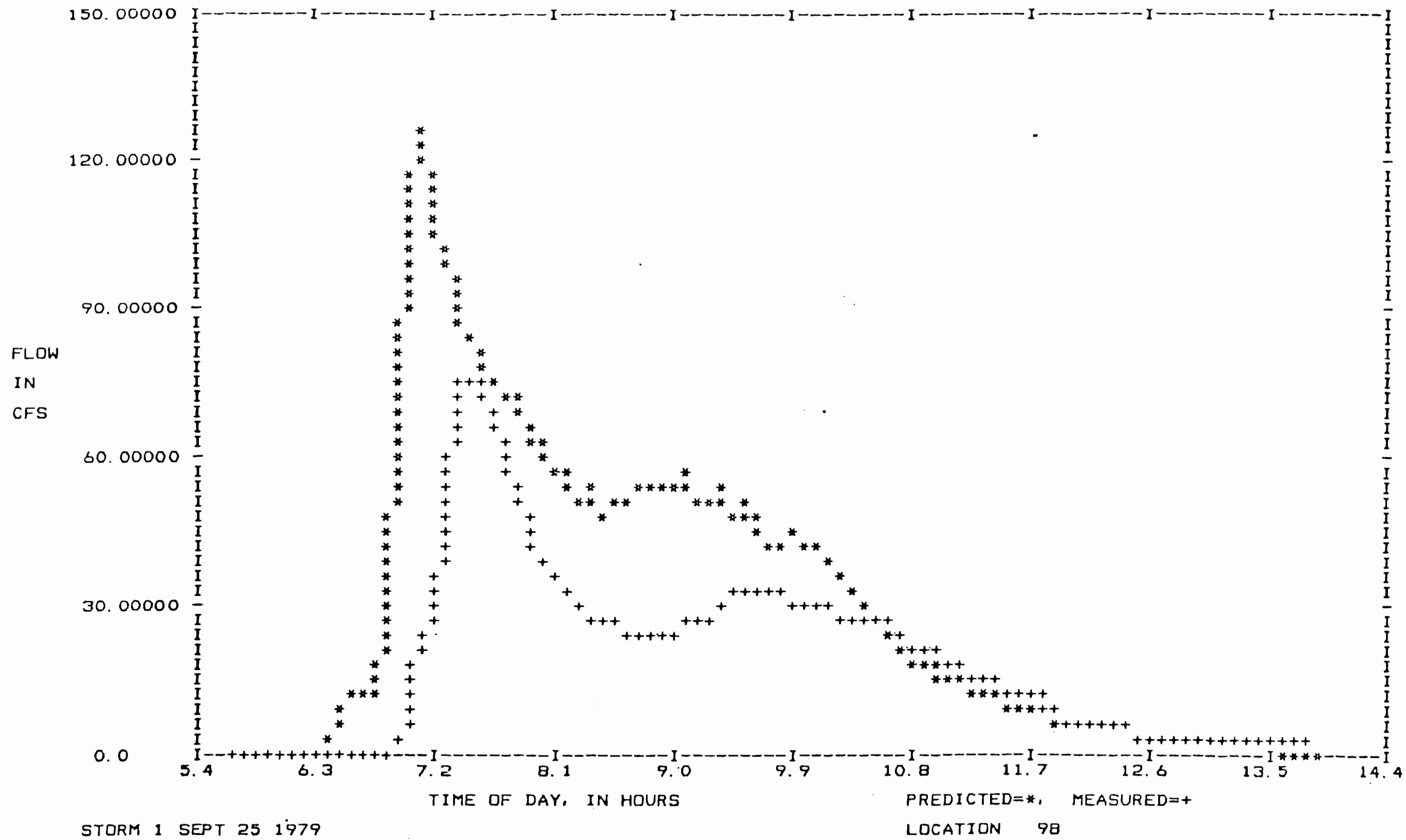
Figure 6. Total Flow Results from the Calibration and Verification Runs--MEG Basin.



HYDROGRAPH STATISTICS FOR LOCATION 98

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME, HR	FLOW, CFS	START, HR	END, HR	LENGTH, HR	POINTS
PREDICTED, TOTAL TIME	0.15637E+07	0.216	12.333	366.538	10.750	16.833	6.083	74
MEASURED, TOTAL TIME	0.23059E+07	0.318	13.000	406.000	10.750	16.750	6.000	73

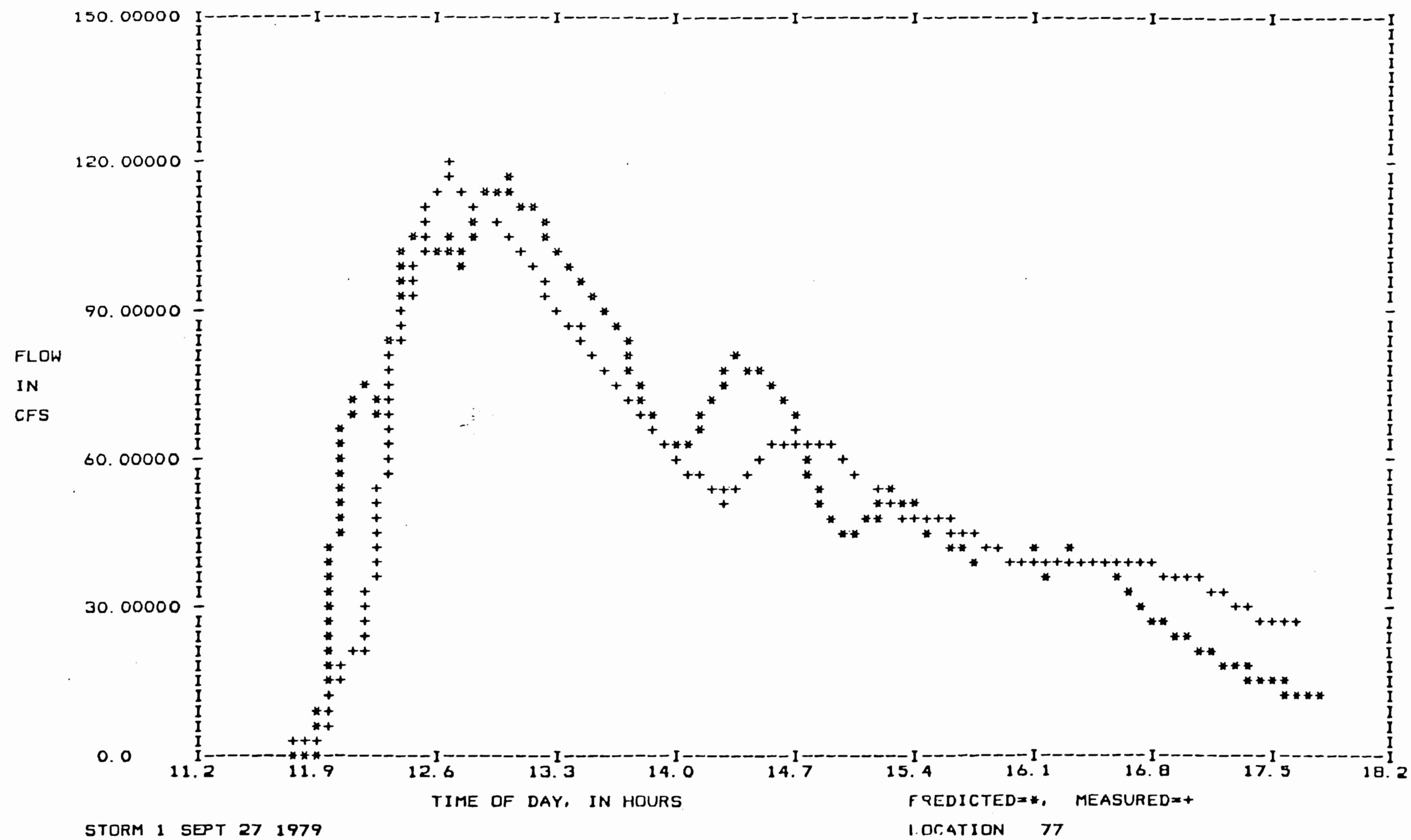
Figure 7. Predicted and Measured Hydrographs from the Best-Fitting Calibration Run on the MEG Basin.



HYDROGRAPH STATISTICS FOR LOCATION 98

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME, HR	FLOW, CFS	START, HR	END, HR	LENGTH, HR	POINTS
PREDICTED, TOTAL TIME	0.90132E+06	0.124	7.083	125.420	5.667	13.833	8.167	99
MEASURED, TOTAL TIME	0.56677E+06	0.078	7.500	76.000	5.667	13.750	8.083	98

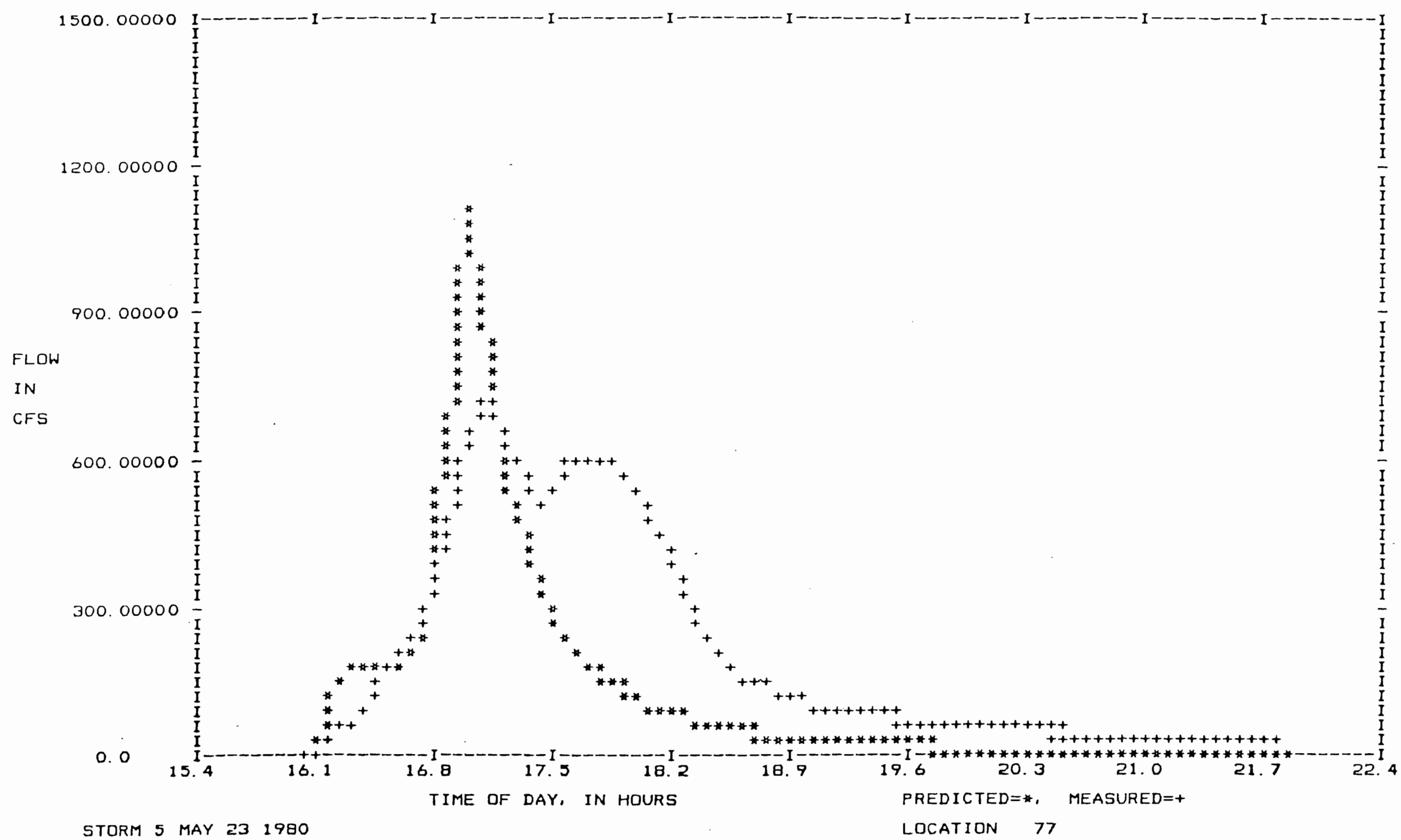
Figure 8. Predicted and Measured Hydrographs from the Worst-Fitting Calibration Run on the MEG Basin.



HYDROGRAPH STATISTICS FOR LOCATION 77

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME, HR	FLOW, CFS	START, HR	END, HR	LENGTH, HR	POINTS
PREDICTED, TOTAL TIME	0.12448E+07	0.172	13.000	115.920	11.750	17.750	6.000	73
MEASURED, TOTAL TIME	0.11976E+07	0.165	12.667	119.000	11.750	17.667	5.917	72

Figure 9. Predicted and Measured Hydrographs from the Best-Fitting Verification Run on the MEG Basin.



HYDROGRAPH STATISTICS FOR LOCATION 77

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME, HR	FLOW, CFS	START, HR	END, HR	LENGTH, HR	POINTS
PREDICTED, TOTAL TIME	0.27578E+07	0.381	17.000	1113.679	16.000	21.833	5.833	71
MEASURED, TOTAL TIME	0.43464E+07	0.600	17.083	724.000	16.000	21.750	5.750	70

Figure 10. Predicted and Measured Hydrographs from the Worst-Fitting Verification Run on the MEG Basin.

coefficient of variation of interevent times was as close to 1.0 as possible, using hourly increments. The resulting MIT was 19 hours. This method of determining independent events (Hydroscience, 1979; Restrepo-Posado and Eagleson, 1980) is based on the fact that the exponential distribution is often fit to interevent times, and the coefficient of variation of the exponential distribution is equal to 1.0. The storm events were then sorted and ranked by magnitude based on both peak flow and total flow, and assigned an empirical return period according to the Weibull formula (T = [n+1]/m, where T is the return period in months, n is the total number of months, and m is the rank of the event).

The next step of the analysis involved running the highest ranking 11 storms (return periods from 1.97 to 21.67 years) based on total flow and peak flow through single event simulation with a 5-min time step. Several days of antecedent rainfall for each storm were also used to accurately simulate antecedent conditions. Since the ranking of some of the storms changed when run with a shorter time step, the return periods of the 11 storms were rearranged based on the results determined from single-event simulation. The final results for the events based on peak flow are shown in Table 9; the results based on total flow are shown in Table 10.

USGS flood frequency analysis

The USGS has performed flood frequency analyses on 15 basins in the Tallahassee area using a combination of continuous simulation and regression analysis (Franklin and Losey, 1984; Franklin, 1984). The results from those studies for the MEG basin are included in Tables 11 and 12.

Table 9. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Peak Flow--MEG Basin.

Return Period yr	Date	Maximum Intensity In./hr	Runoff Duration hr	Runoff Volume in.	Peak Flow cfs
1.97	6/30/64	1.80	50	1.25	933
2.17	4/13/79	1.99	19	0.85	1068
2.41	7/09/65	2.16	86	1.59	1144
2.71	12/03/64	2.15	67	2.74	1205
3.10	9/20/69	2.18	88	3.69	1218
3.61	7/21/69	2.27	63	1.61	1241
4.33	11/25/72	2.80	14	1.04	1541
5.42	9/03/65	2.87	15	1.19	1548
7.22	7/16/64	3.23	75	2.69	1788
10.83	7/21/70	3.46	36	2.24	1949
21.67	9/08/68	4.83	11	1.81	2720

Table 10. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Total Flow--MEG Basin.

Return Period yr	Date	Runoff Duration hr	Runoff Volume in.
1.97	10/06/59	93	1.95
2.17	3/31/62	30	1.96
2.41	7/10/79	57	1.99
2.71	10/06/76	65	2.04
3.10	8/02/66	171	2.09
3.61	7/21/70	36	2.24
4.33	7/26/75	98	2.39
5.42	7/16/64	75	2.69
7.22	12/03/64	67	2.74
10.83	3/28/73	114	2.86
21.67	9/20/69	88	3.69

Summary of Results for MEG Basin

Results of the various methods used to determine peak flow are listed in Table 11 and shown in Figure 11, which does not show the THM result in order to allow more detail to be shown between the other methods. The THM yielded the highest flows at all return periods, especially at the three largest return periods. Perhaps, more reasonable results would have been obtained if the catchment had been modeled as several subcatchments, but the use of a single subcatchment was common to all methods. The CM/SCS gave the next highest results at all return periods except 25 years. The values from the USGS studies (labeled USGS on Figure 10) and CS are similar at the lowest three return periods and are the next highest in rank. Close to the 25 year return period, CS produced the second highest ranking peak flow. The return period for this value, however, may well deviate from that assigned to it by the Weibull formula. The RM gave values the next highest in ranking for all return periods except 2 years. The CM/CH gave the sixth largest peak flows for all return periods except 2 years. The RE and PDM produced the next highest peak flows for the three largest return periods. The SUH/SCS and SUH/CH yielded the lowest peak flows for all but the lowest return period.

Table 12 and Figure 12 give the results of the five methods used to determine total volume. The SCS methods yielded much larger volumes than the other methods for the three largest return periods. CS gave the next largest, followed by the USGS studies and then the CM/SCS and the CM/CH. Note that a 24-hour duration was used for all of the methods, except CS.

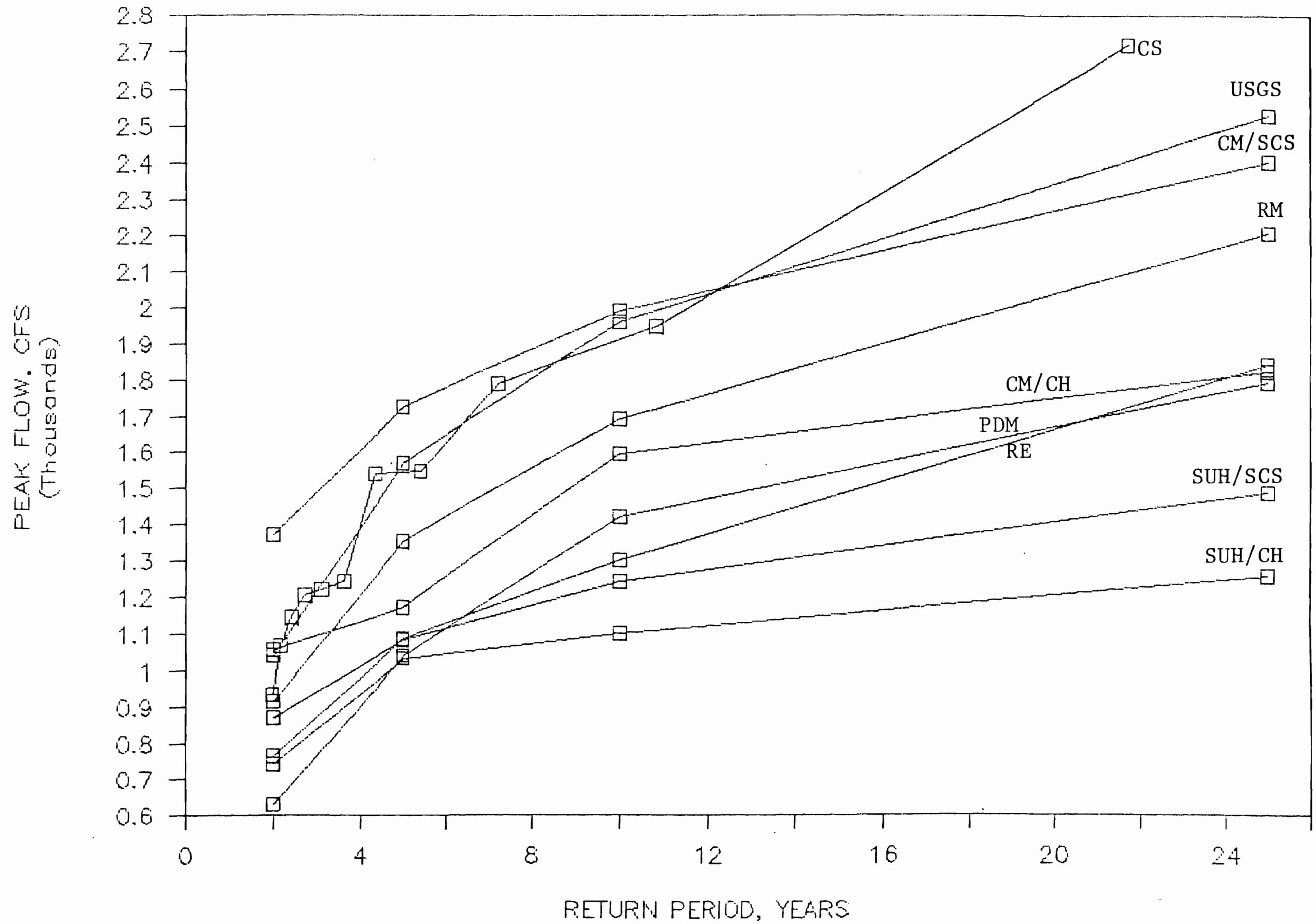


Figure 11. Peak Flow Results for the MEG Basin.

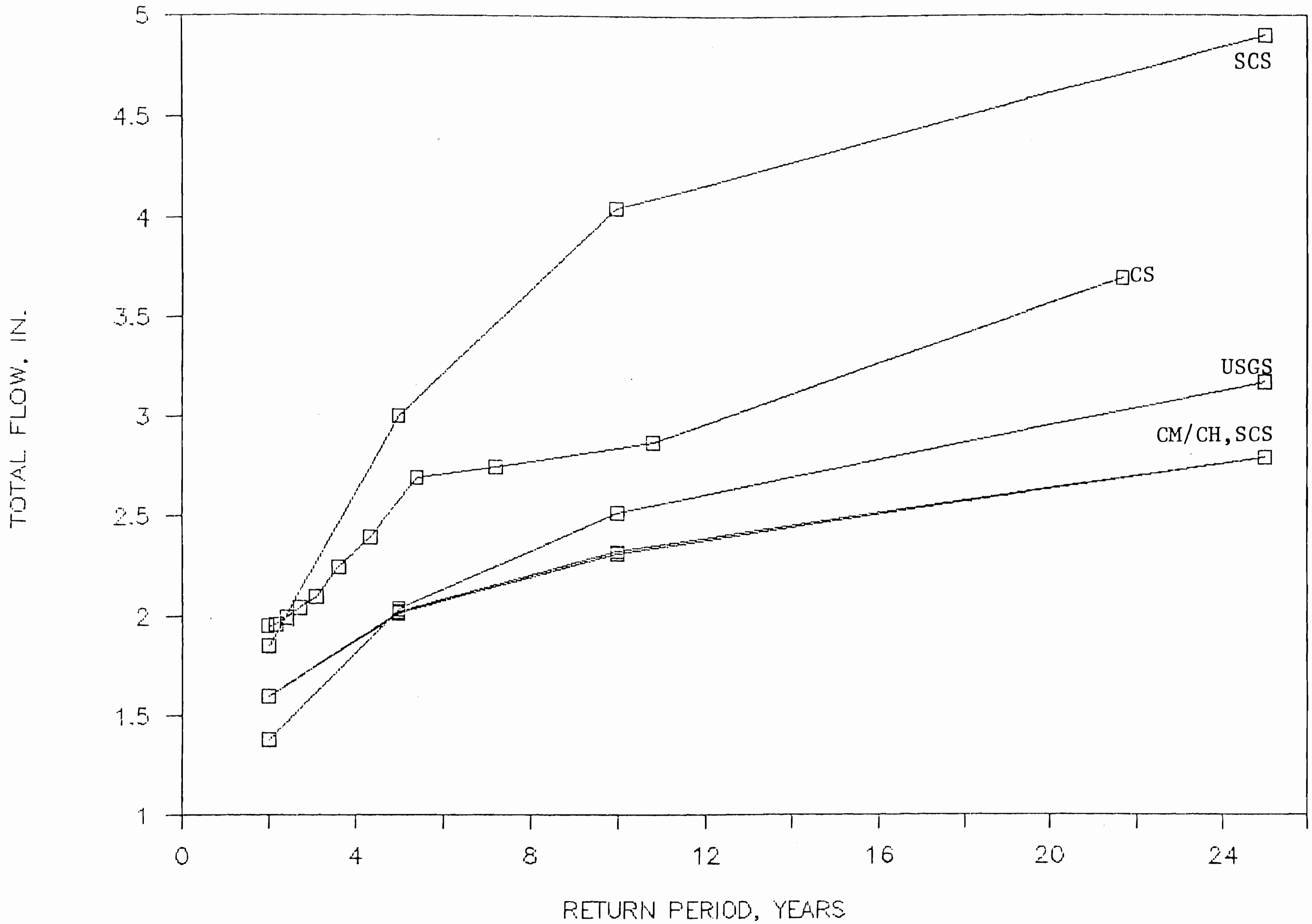


Figure 12. Total Flow Results for the MEG Basin.

Table 11. Results of Peak Flow from Eight Methodologies--MEG Basin.

Return Period yr	CS	USGS Study	PDM	THM	RM	SUH/CH	SUH/SCS	CM/CH	CM/SCS	RE
1.97	933									
2		1040	632	1474	905	743	868	1056	1371	758
2.17	1068									
2.41	1144									
2.71	1205									
3.10	1218									
3.61	1241									
4.33	1541									
5		1570	1039	2949	1354	1031	1081	1171	1726	1078
5.42	1548									
7.22	1788									
10		1960	1419	3414	1692	1100	1241	1596	1991	1299
10.83	1949									
21.67	2720									
25		2530	1794	4202	2206	1255	1489	1822	2403	1841

Table 12. Results of Total Flow from Various Methodologies--MEG Basin.

Return Period yr	CS	USGS Study	SCS	CM/CH	CM/SCS
1.97	1.95				
2		1.38	1.85	1.60	1.60
2.17	1.96				
2.41	1.99				
2.71	2.04				
3.10	2.09				
3.61	2.24				
4.33	2.39				
5		2.03	3.00	2.02	2.01
5.42	2.69				
7.22	2.74				
10		2.51	4.04	2.32	2.31
10.83	2.86				
21.67	3.69				
25		3.17	4.90	2.79	2.79

Pompano Beach, FL (LDR Basin)

The study area in Pompano Beach (henceforth referred to as the LDR basin) consists of 40.8 acres of flat, low-density residential land, as shown in Figure 13. The soil is a fine, loose sand with high permeability. Downspouts from the houses drain onto the lawns, and water that drains off the lawns is carried by shallow swales to an inlet (Miller, 1979; Mattraw et al., 1975).

Results

The seven methodologies discussed in Chapter 2 and applied to the MEG basin were applied to the LDR basin. Since the methodologies are the same, many of the details discussed in the previous section on the MEG basin will not be repeated in this section nor any of the remaining sections. Most of the parameters for this area came from a USGS Open-File Report (Miller, 1979). The remainder of the parameters came from an earlier study done on the site (Maalel, 1983). Parameters used for the seven methodologies are given in Table 13. A summary of the results for the LDR basin is given in Tables 18 and 19 at the end of this section.

Rational method

The parameters used in the rational method are listed in Table 14. A total of six pipes and one swale were used in the calculation of pipe/channel travel time. Regional IDF curves were used for the calculations (Weldon, 1985).

Synthetic unit hydrograph method

The five watershed parameters required for the development of the unit hydrograph are included in Table 13. Rainfall excess was

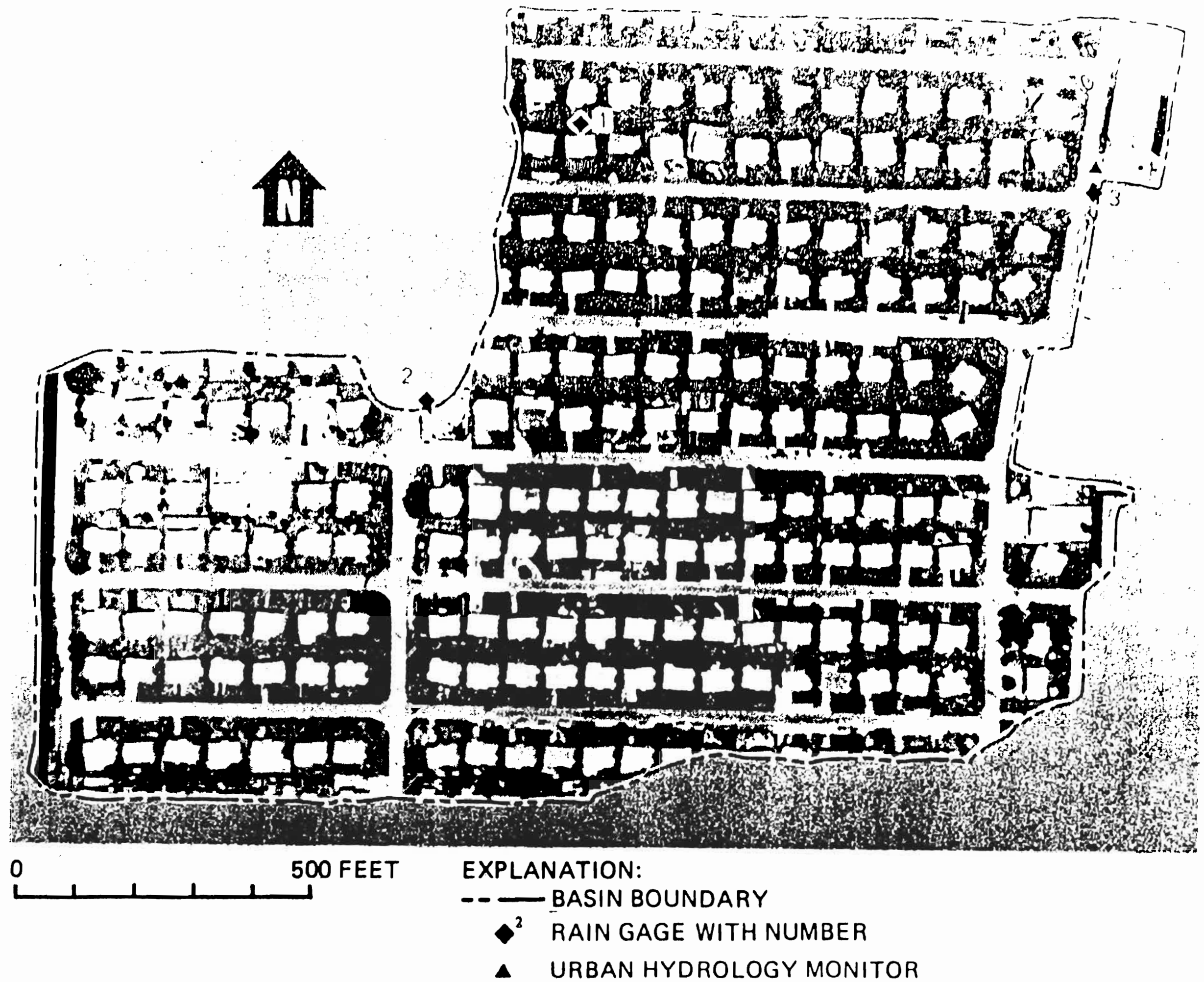


Figure 13. Photomosaic Map of the LDR Basin (Miller, 1979).

calculated by multiplying the design storm hyetograph values by the fraction of impervious area.

Table 13. Parameters Used for the Methods Applied to the LDR Basin. *

Watershed Drainage Area	40.8 ac 0.06375 sq mi	
Length of Main Channel	2700 ft	
Main Channel Slope	0.0015 ft/ft	
Hydraulically Effective Impervious Area	5.9 %	
Average Basin Slope	0.0015 ft/ft	
Soil Group	A	
Curve Number	65	
Conveyance Efficiency	1.0	
Adjustment Factors		
Impervious Area	1.05	
Hydraulic Length	1.18	
Slope	0.50	
Ponding		
Return Period yr		Ponding Adjustment Factor
2		0.78
5		0.79
10		0.81
25		0.83
Lake Area	1 %	
2 Year, 2 Hour Rainfall Intensity	3.2 in.	
Basin Development Factor	6	
Basin Storage	1.0 %	
Subcatchment Width	6400 ft	
Manning's n		
Pervious Area	0.25	
Impervious Area	0.015	
Green-Ampt Infiltration Parameters		
Hydraulic Conductivity	0.40 in./hr	
Capillary Suction	15.00 in.	
Initial Moisture Deficit	0.10	

* Generally not SWMM input values. See Appendix C.

SCS peak discharge method

The six watershed parameters required for this method are shown in Table 13, as are the adjustment factors for peak flow calculation.

Rainfall (Hershfield, 1961) and calculated runoff depths are listed in Table 15.

Table 14. Parameters from the Rational Method--LDR Basin.

Return Period yr	Wave Pipes min	Travel Time Overland Flow min	Time of Concentration min	Rainfall Intensity in./hr	c
2	0.96	57.0	58.0	2.67	.06
5	0.95	48.7	49.6	3.67	.09
10	0.93	46.0	46.9	4.10	.13
25	0.91	41.7	42.6	5.00	.18

Table 15. Rainfall and Runoff Depths from the SCS Peak Discharge Method--LDR Basin.

Return Period yr	Rainfall Depth in.	Runoff Depth in.
2	5.8	2.24
5	8.0	3.89
10	9.0	4.72
25	11.0	6.43

SCS tabular hydrograph method

Most of the values needed for this method are given in Table 13. Time of concentration, the only additional parameter needed, was calculated using formulas supplied with method.

Regression equations

Table 13 contains the three parameters needed to determine pre-development peak flows and the four additional parameters needed to calculate the urbanized peak flows. Table 18 contains the post-development peak flows for four return periods.

Calibrated SWMM with synthetic design storms

The SCS and Chicago synthetic design storms were used as rainfall input to the calibrated model. The calibration is described in the next section. Average antecedent moisture conditions were assumed.

Design storms from continuous simulation

Rainfall-runoff data for calibration and verification came from a USGS Open-File Report (Mattraw et al., 1975) consisting of more than one year of data. The calibration and verification were carried out in the same manner as for the MEG basin. Parameters from the final calibration runs are listed in Table 13. Results from the calibration and verification runs for peak flow and total flow can be seen in Figures 14 and 15. Hydrographs of measured and predicted flows from the calibration storms with the best and worst fits are shown in Figures 16 and 17, respectively; similar hydrographs from the verification storms are shown in Figures 18 and 19.

A 38-year record of hourly rainfall data from a National Weather Service station in West Palm Beach (station # 089525) was used for the continuous simulation. A minimum interevent time of 21 hours was used to delineate events, yielding 2858 independent storm events. Twelve of the 20 highest-ranking storms (return periods from 1.90 to 38.08 years) were run in single-event mode with a 5-min time step and several days of antecedent moisture. The return periods of the 12 events were rearranged based on the results from single-event simulation. Table 16 lists the 12 events based on peak flow, and Table 17 lists the 12 based on total flow.

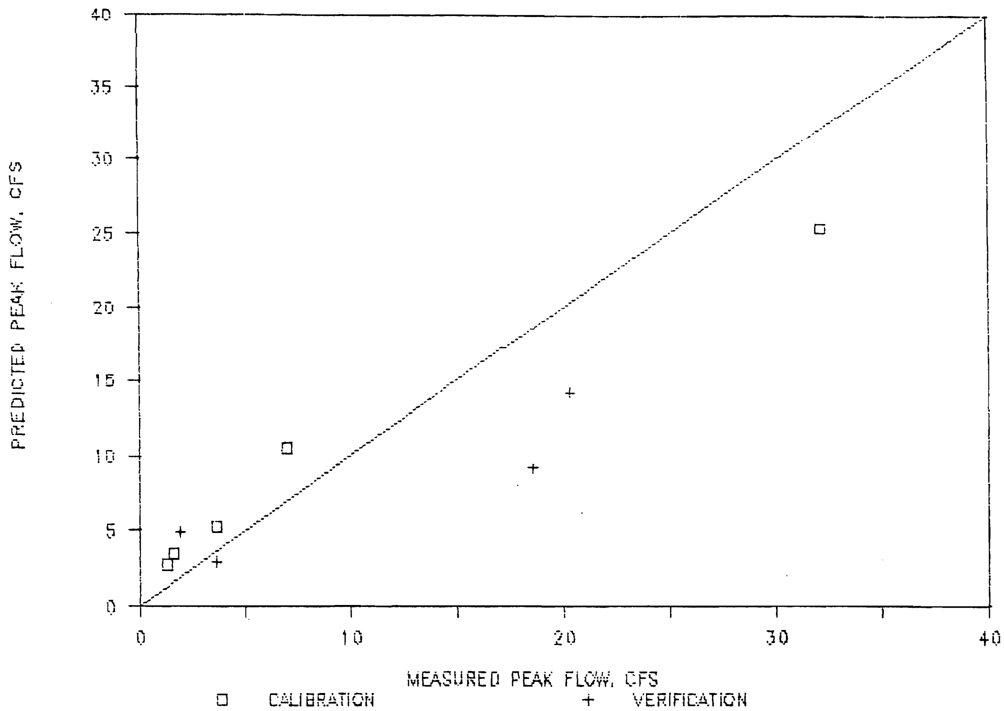


Figure 14. Peak Flow Results from the Calibration and Verification Runs--LDR Basin.

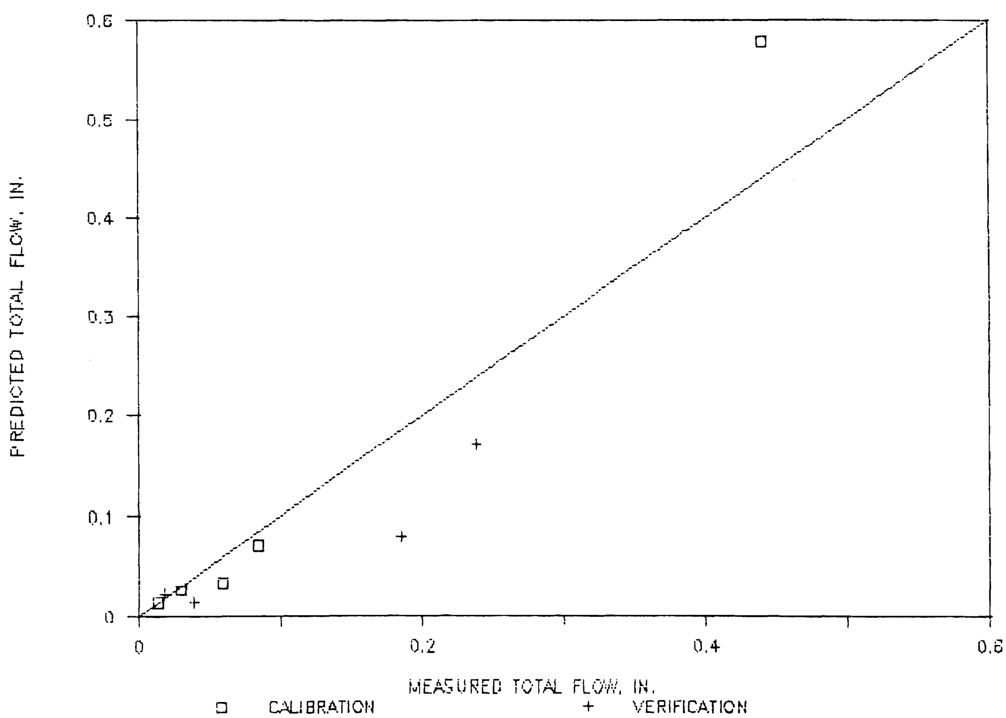
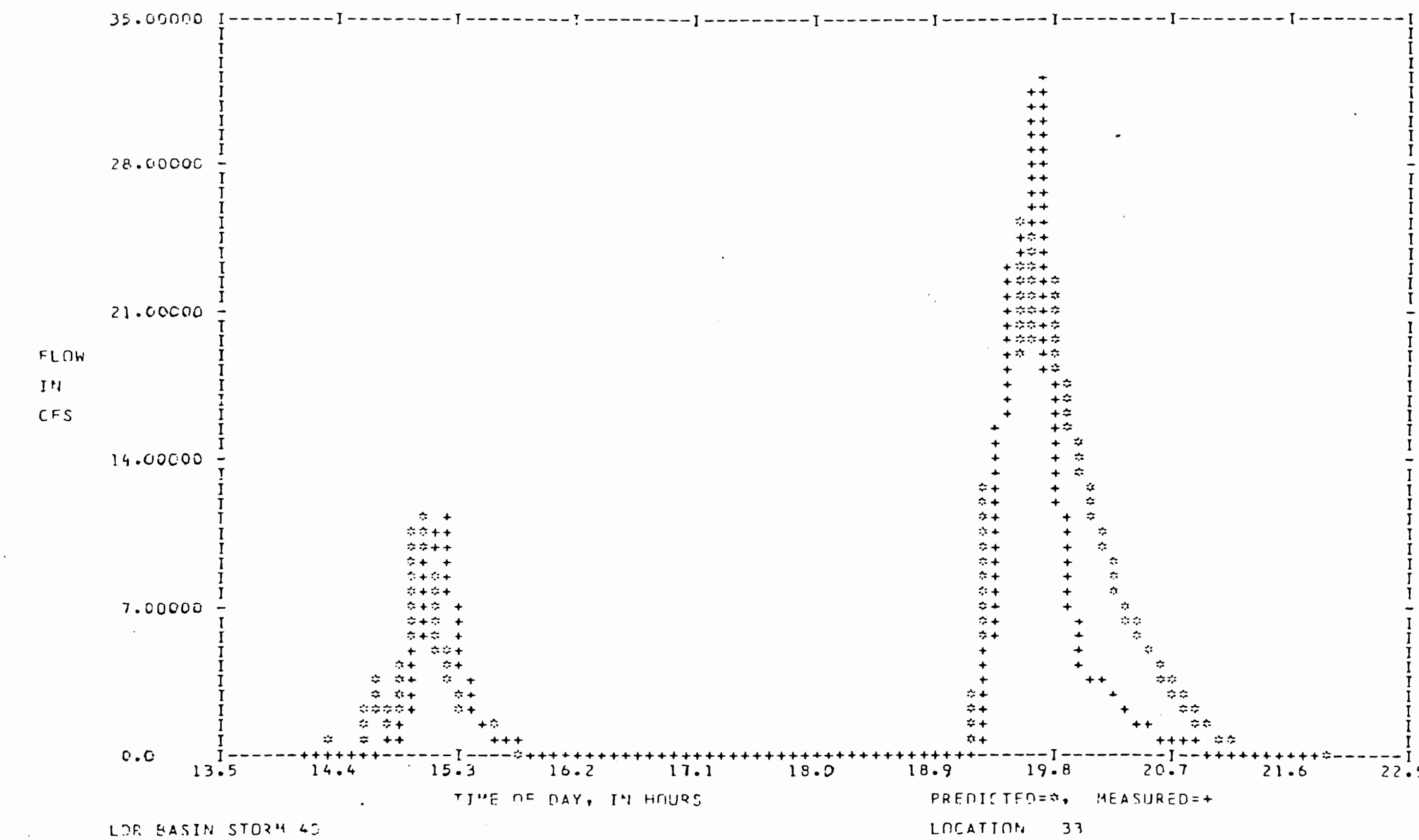


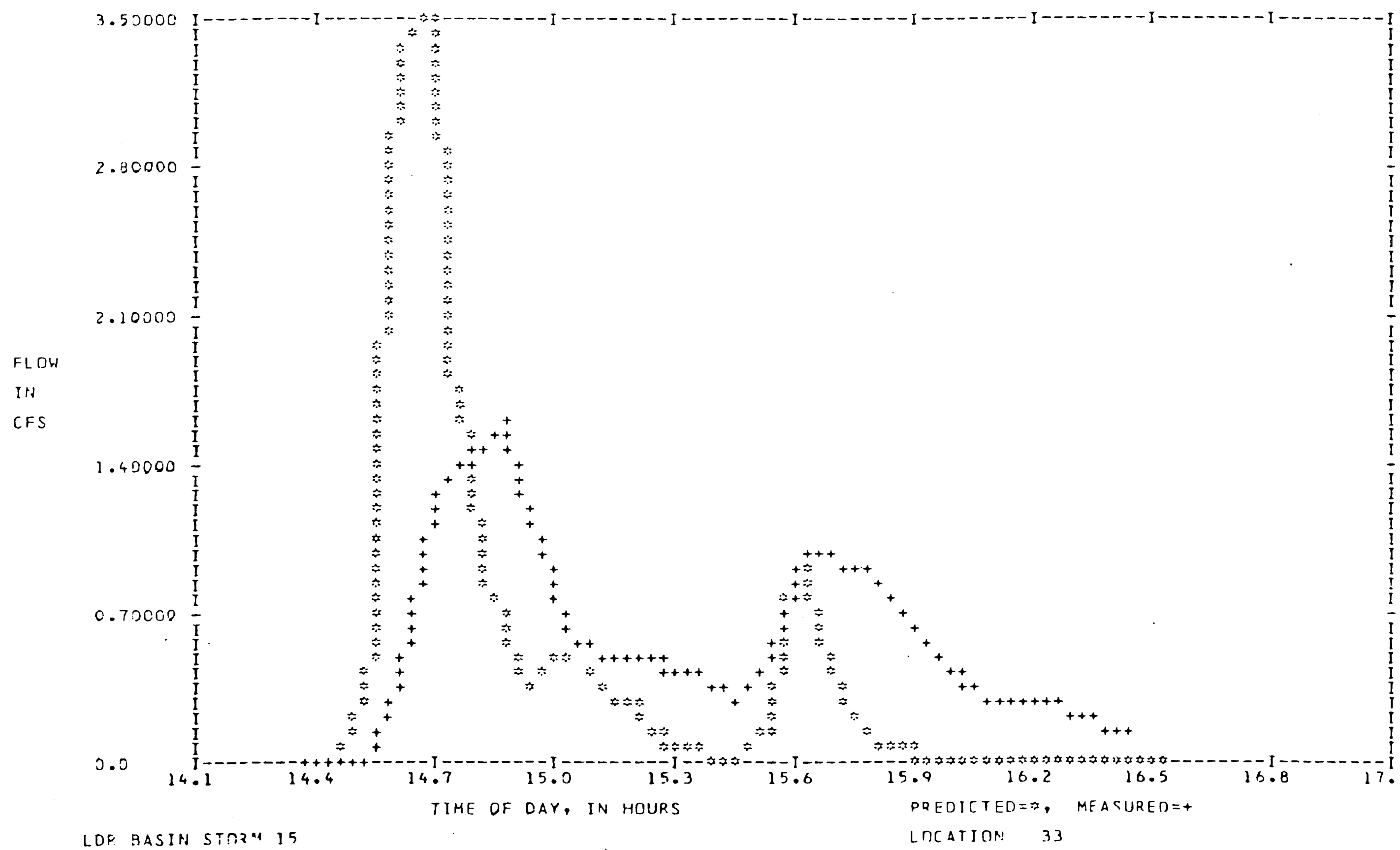
Figure 15. Total Flow Results from the Calibration and Verification Runs--LDR Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME,HR	FLOW,CFS	START,HR	END,HR	LENGTH,HR	POINTS
PREDICTED, TOTAL TIME	0.85670E+05	0.578	19.667	25.306	14.167	21.833	7.667	185
MEASURED, TOTAL TIME	0.65167E+05	0.440	19.667	32.080	14.167	21.750	7.583	92

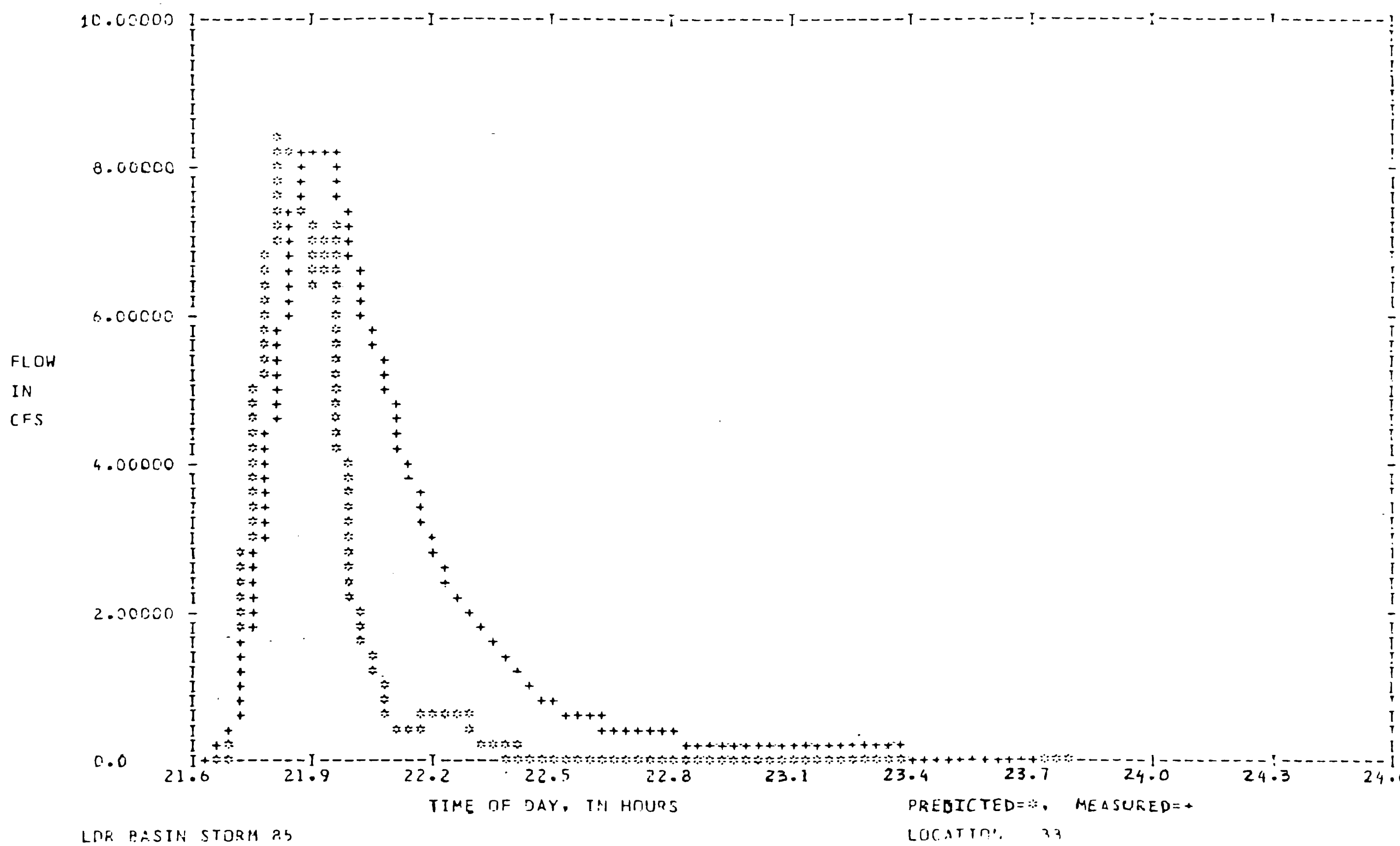
Figure 16. Predicted and Measured Hydrographs from the Best-Fitting Calibration Run on the LDR Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME,HR	PEAK FLOW FLOW,CFS	DURATION START,HR	DURATION END,HR	DURATION LENGTH,HR	NO. POINTS
PREDICTED, TOTAL TIME	0.37954E+04	0.026	14.700	3.490	14.367	16.533	2.167	53
MEASURED, TOTAL TIME	0.43575E+04	0.029	14.867	1.600	14.367	16.450	2.083	26

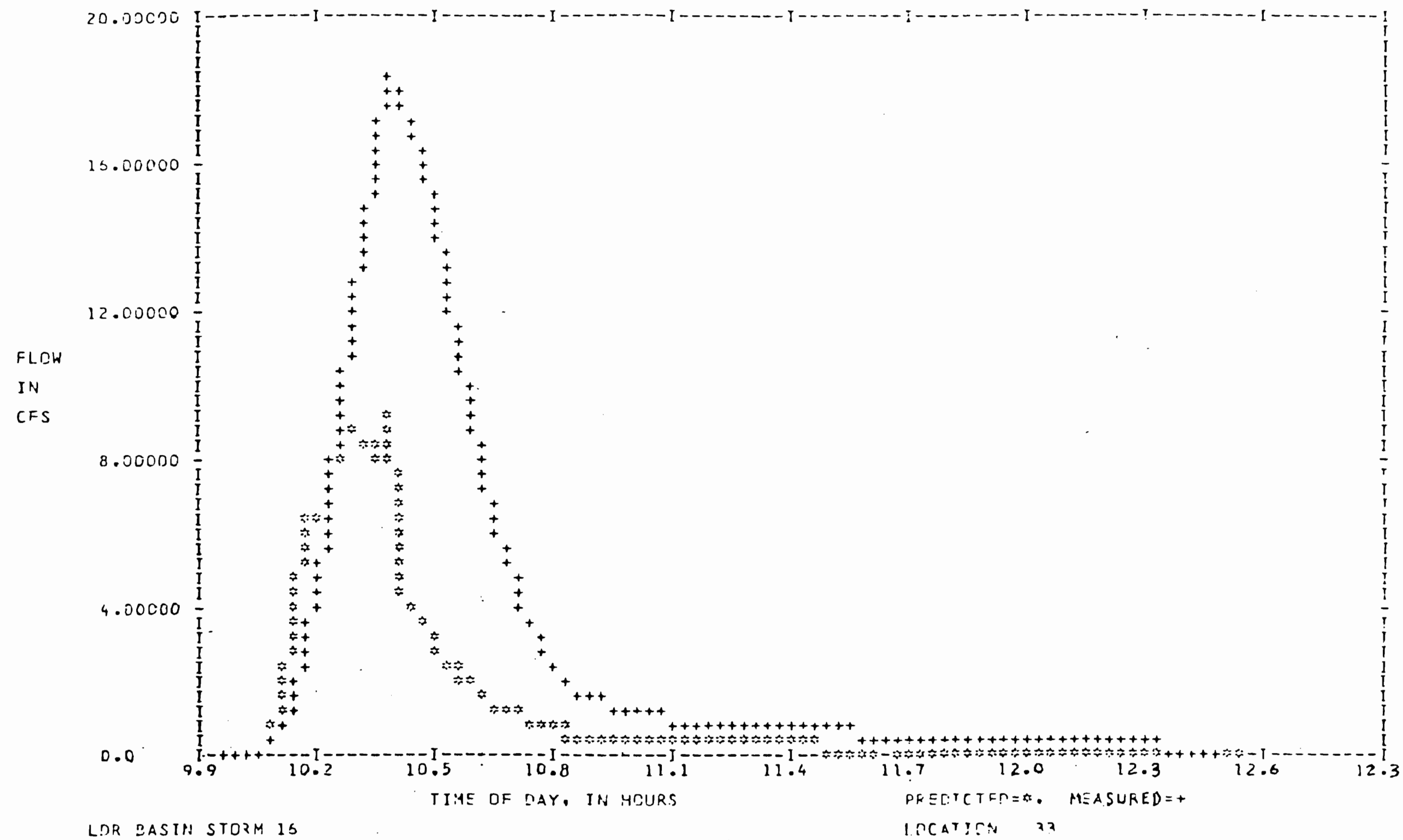
Figure 17. Predicted and Measured Hydrographs from the Worst-Fitting Calibration Run on the LDR Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME,HR	PEAK FLOW FLOW,CFS	DURATION START,HR	DURATION END,HR	DURATION LENGTH,HR	NO. POINTS
PREDICTED, TOTAL TIME	0.71411E+04	0.048	21.825	8.303	21.617	23.783	2.167	53
MEASURED, TOTAL TIME	0.12352E+05	0.083	21.867	8.100	21.617	23.700	2.083	26

Figure 18. Predicted and Measured Hydrographs from the Best-Fitting Verification Run on the LDR Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME,HR	FLOW,CFS	START,HR	END,HR	LENGTH,HR	POINTS
PREDICTED, TOTAL TIME	0.11716E+05	0.079	10.383	9.125	9.967	12.550	2.583	63
MEASURED, TOTAL TIME	0.27340E+05	0.185	10.383	18.560	9.967	12.467	2.500	31

Figure 19. Predicted and Measured Hydrographs from the Worst-Fitting Verification Run on the LDR Basin.

Table 16. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Peak Flow--LDR Basin.

Return Period yr	Date	Maximum Intensity in./hr	Runoff Duration hr	Runoff Volume in.	Peak Flow cfs
1.90	5/25/77	1.77	49	1.21	23
2.00	9/10/49	2.22	9	1.34	26
2.72	8/09/68	1.96	6	1.37	28
3.81	5/02/58	1.76	53	3.37	40
4.76	6/21/45	2.49	40	4.34	43
5.44	5/24/68	3.10	48	1.69	58
6.35	10/13/51	2.00	33	3.65	60
7.62	4/23/79	2.55	49	3.11	66
9.52	5/28/76	3.13	52	2.88	68
12.69	10/01/63	3.18	61	2.87	70
19.04	6/19/69	3.55	53	2.50	90
38.08	4/14/42	5.26	78	10.50	158

Table 17. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Total Flow--LDR Basin.

Return Period yr	Date	Runoff Duration hr	Runoff Volume in.
1.90	7/16/72	99	1.67
2.00	5/24/68	48	1.69
2.72	8/31/77	95	1.88
3.81	6/19/69	53	2.50
4.76	9/17/60	26	2.55
5.44	10/01/63	61	2.87
6.35	5/28/76	52	2.88
7.62	4/23/79	49	3.11
9.52	5/02/58	53	3.37
12.69	10/13/51	33	3.65
19.04	6/21/45	40	4.34
38.08	4/14/42	78	10.50

<u>Summary of Results for the LDR Basin</u>

The results of the seven methodologies used to determine peak flows for the LDR basin are listed in Table 18 and shown in Figure 20. The CM/SCS gave the highest peak flows for all return periods. CS, THM, and CM/CH yielded similar results for the three lowest return periods and their values were next in rank, followed by the RE, the PDM, and the RM. The two synthetic unit hydrographs gave much lower results than all of the other methods, probably because of the rainfall interval used.

Table 18. Results of Peak Flow from Eight Methodologies--LDR Basin.

Return Period yr	CS	PDM	THM	RM	SUH/CH	SUH/SCS	CM/CH	CM/SCS	RE
	Peak Flow, cfs								
1.90	23								
2.00	26	13	31	6.5	1.6	1.6	33	46	25
2.72	28								
3.81	40								
4.76	43								
5		23	57	14	2.1	2.0	38	69	37
5.44	58								
6.35	60								
7.62	66								
9.52	68								
10		28	70	22	2.3	2.3	73	88	45
12.69	70								
19.04	90								
25		40	98	37	2.7	2.8	87	113	64
38.08	158								

Table 19 and Figure 21 show the values of total flow found by the four different methods. The SCS methods produced the largest volumes for all return periods. CS and the CM/SCS gave similar results for the three smallest return periods, but the value with the largest

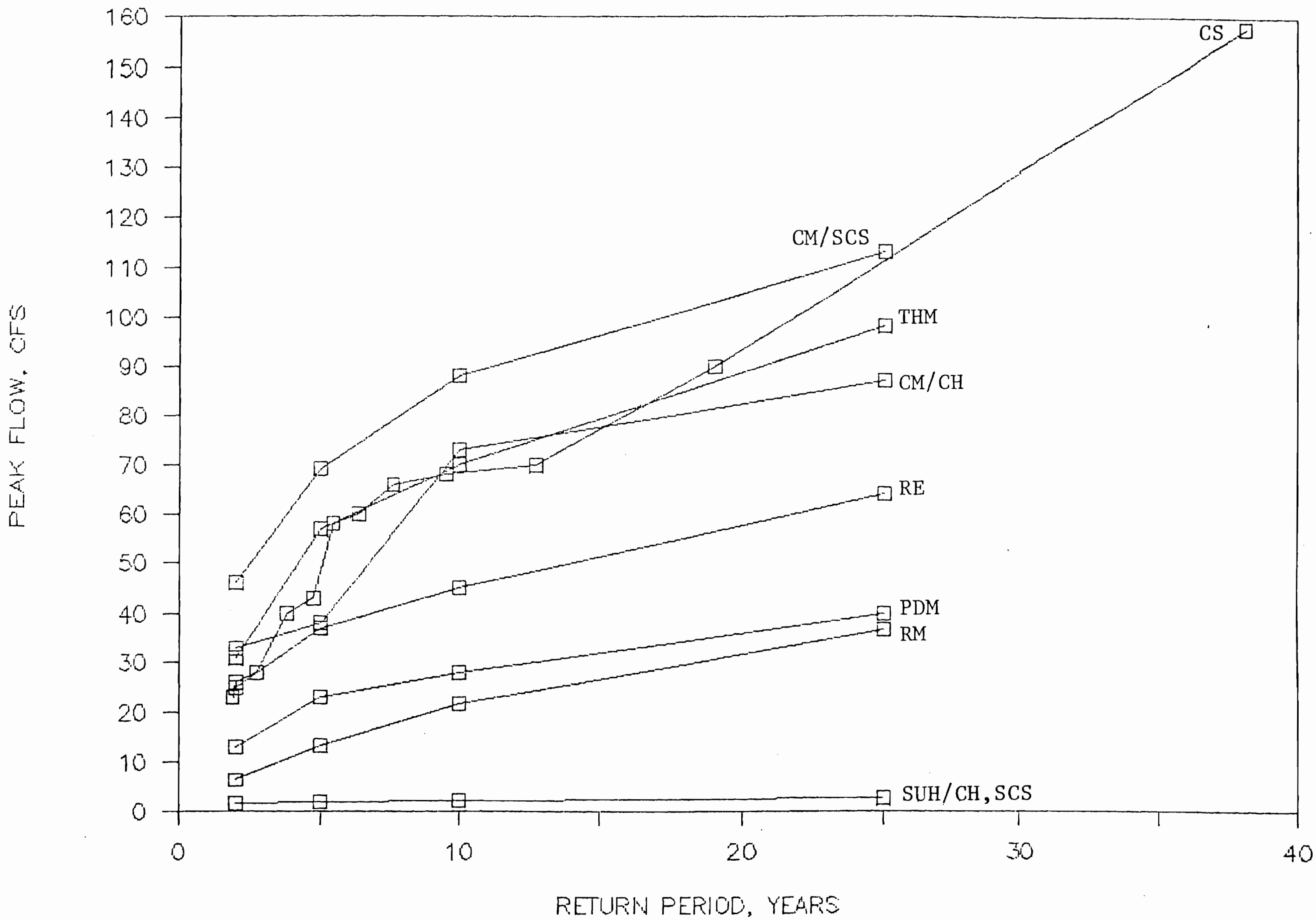


Figure 20. Peak Flow Results for the LDR Basin.

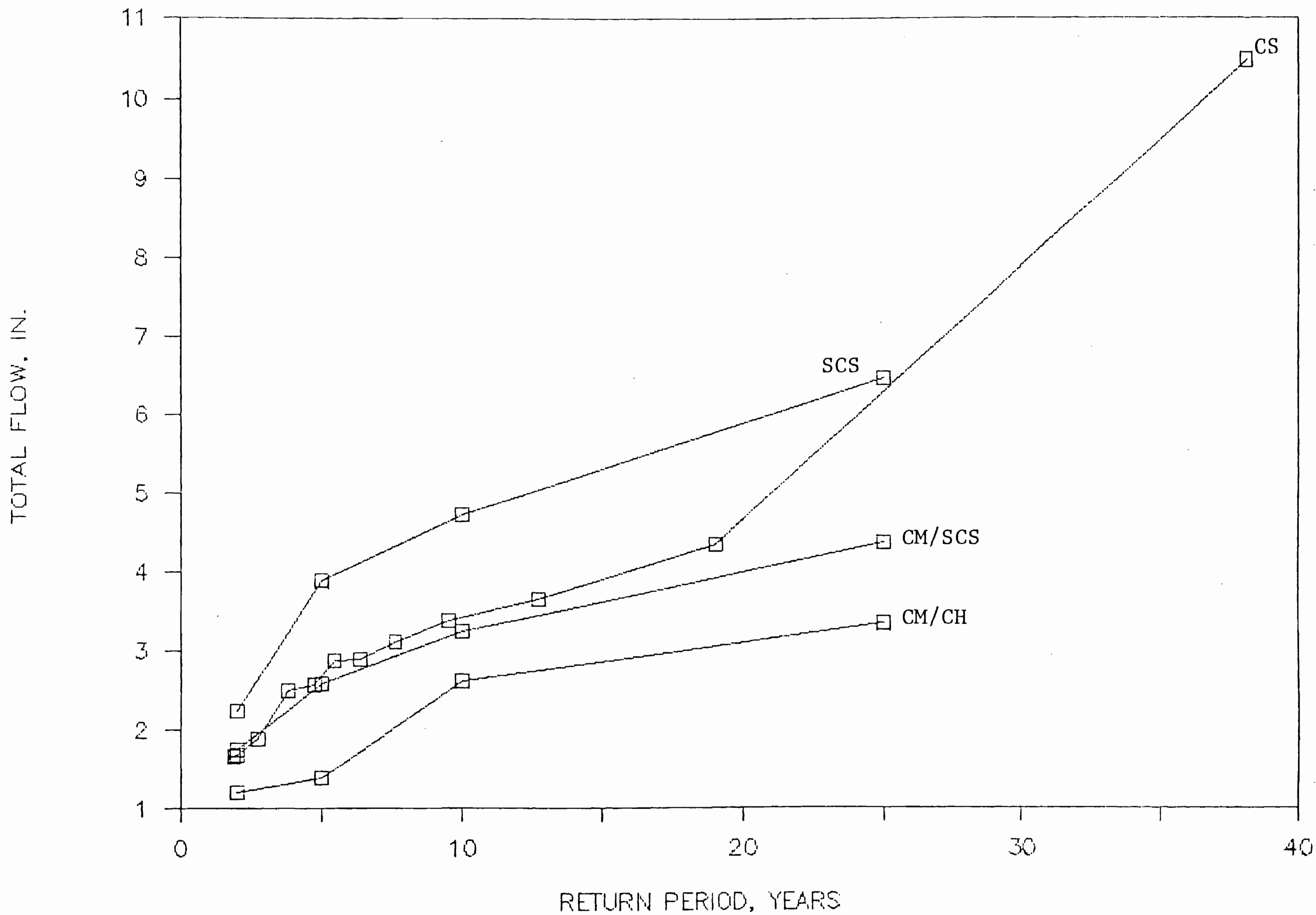


Figure 21. Total Flow Results for the LDR Basin.

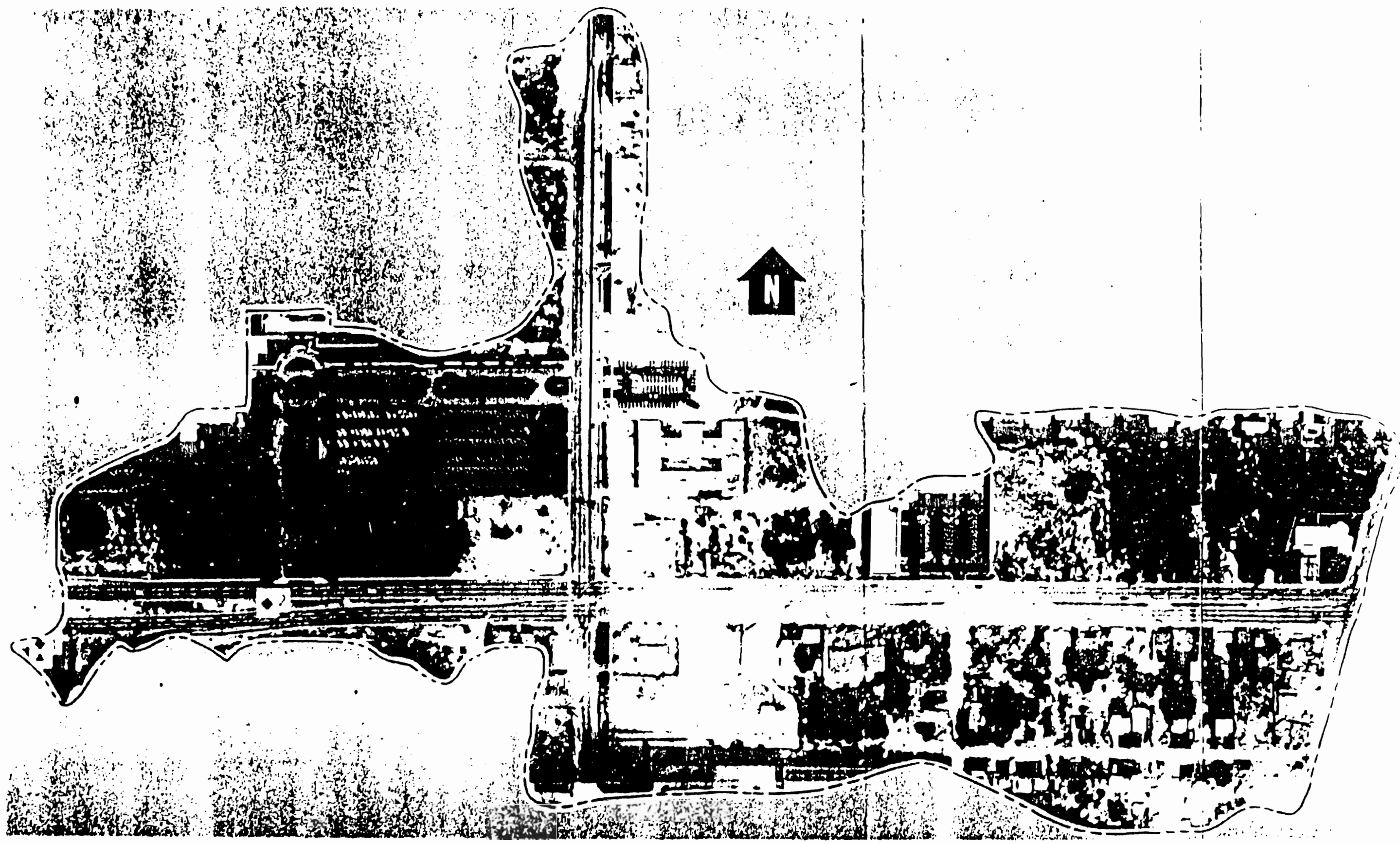
return period from CS was large enough to make the CS curve diverge from the CM/SCS at the largest return period. The CM/CH gave the lowest volumes at all of the return periods.

Table 19. Results of Total Flow from Various Methodologies--LDR Basin.

Return Period yr	Total Flow, in. CS	SCS	CM/CH	CM/SCS
1.90	1.67			
2.00	1.69	2.24	1.20	1.75
2.72	1.88			
3.81	2.50			
4.76	2.55			
5		3.89	1.40	2.58
5.44	2.87			
6.35	2.88			
7.62	3.11			
9.52	3.37			
10		4.72	2.62	3.24
12.69	3.65			
19.04	4.34			
25		6.43	3.35	4.36
38.08	10.50			

Broward County, FL (HWY Basin)

The study area in Broward County (henceforth referred to as the HWY basin) is 58.3 acres and contains a large highway with adjacent businesses and open lots and some small residential sections, as shown in Figure 22. The soil is a fine sand with high permeability, and the slope is generally very small. The area has a fairly extensive storm sewer system consisting of circular, concrete pipes (Miller, 1979; Hardee et al., 1978).



EXPLANATION: --—BASIN BOUNDARY ◆[2] RAIN GAGE WITH NUMBER ▲ URBAN HYDROLOGY MONITOR 0 500 FEET

Figure 22. Photomosaic Map of the HWY Basin (Miller, 1979).

Results

The seven methodologies applied to the LDR basin were applied to the HWY basin. A more detailed description of the analyses is given in MEG basin section. The sources of data for this area came from same sources as those for the LDR basin, with the exception of the rainfall-runoff data. Parameters used for the various methods are listed in Table 20. A summary of the results for the HWY basin is given at the end of this section in Tables 25 and 26.

Rational method

Table 21 contains the parameters used for the rational method. A total of 13 pipes was used in the calculation of pipe travel time. Regional IDF curves were used for the time of concentration calculations (Weldon, 1985).

Synthetic unit hydrograph method

The five watershed parameters used for the development of the synthetic unit hydrograph are included in Table 20. Rainfall excess was calculated by multiplying the hyetograph values by the fraction of hydraulically effective impervious area.

SCS peak discharge method

The six watershed parameters required for the PDM are included in Table 20, as are the adjustment factors. Rainfall (Hershfield, 1961) and calculated runoff depths from this method are listed in Table 22.

SCS tabular hydrograph method

Most of the values required for this method are shown in Table 20. Time of concentration, the only other parameter needed, was calculated from supplied equations (Soil Conservation Service, 1986).

Table 20. Parameters Used for the Methods Applied to the HWY Basin.*

Watershed Drainage Area	58.3 ac
	0.09109 sq mi
Length of Main Channel	2800 ft
Main Channel Slope	0.003 ft/ft
Hydraulically Effective Impervious Area	18 %
Average Basin Slope	0.003 ft/ft
Soil Group	A
Curve Number	65
Conveyance Efficiency	0.8
Adjustment Factors	
Impervious Area	1.07
Hydraulic Length	1.80
Slope	0.65
Ponding	

Return Period yr	Ponding Adjustment Factor
2	0.82
5	0.83
10	0.84
25	0.86

Lake Area	1 %
2 Year, 2 Hour Rainfall Intensity	3.2 in.
Basin Development Factor	10
Basin Storage	1.0 %
Subcatchment Width	1650 ft
Manning's n	
Pervious Area	0.25
Impervious Area	0.02
Green-Ampt Infiltration Parameters	
Hydraulic Conductivity	0.20 in./hr
Capillary Suction	15.00 in.
Initial Moisture Deficit	0.10

* Generally not SWMM input values. See Appendix C.

Regression equations

Table 20 includes the three parameters needed to determine pre-development peak flows and the four additional parameters needed to calculate the urbanized peak flows. The urbanized peak flows are given in Table 25 at the end of this section.

Table 21. Parameters from the Rational Method--HWY Basin.

Return Period yr	Wave Travel Time Pipes min	Wave Travel Time Overland Flow min	Time of Concentration min	Rainfall Intensity in./hr	c
2	5.5	29.0	34.5	3.62	.18
5	5.3	25.7	31.0	4.68	.20
10	5.1	24.7	29.8	5.14	.23
25	4.8	23.2	28.0	6.01	.25

Table 22. Rainfall and Runoff Depths from the SCS Peak Discharge Method--HWY Basin.

Return Period yr	Rainfall Depth in.	Runoff Depth in.
2	5.8	2.21
5	8.0	3.89
10	9.0	4.72
25	11.0	6.43

Calibrated SWMM with synthetic design storms

The SCS and Chicago design storms were used as rainfall input to the calibrated SWMM model. Average antecedent moisture conditions were assumed for all runs. The calibration is discussed in the next section.

Design storms from continuous simulation

Rainfall-runoff data for calibration and verification came from a USGS Open-File Report (Hardee et al., 1978) consisting of slightly more that two years worth of storms. The calibration and verification were carried out in the same manner as for the MEG basin. Parameters from the final calibration runs are shown in Table 20. Results from the calibration and verification runs for peak flow and total flow are shown in Figures 23 and 24, respectively. Hydrographs of measured and predicted flows from the calibrations runs producing the best and worst fits are shown in Figures 25 and 26, respectively; similar hydrographs from the verification runs are shown in Figures 27 and 28.

The same rainfall record used for the LDR basin was used for the HWY basin because that station was closest to both basins of all long-term rainfall-recording stations. A MIT of 20 hours was used to delineate the storm events, resulting in 2867 independent events. The same process as was carried out for the LDR basin of running 12 of the 20 highest ranking storms based on peak flow and total flow in single-event mode was repeated for the HWY basin. Results based on the peak flow rankings and total flow rankings of single-event runs are shown in Tables 23 and 24, respectively.

Table 23. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Peak Flow--HWY Basin.

Return Period yr	Date	Maximum Intensity in./hr	Runoff Duration hr	Runoff Volume in.	Peak Flow cfs
1.90	3/25/70	1.85	43	1.79	33
2.00	7/16/72	1.57	100	2.56	48
2.93	5/02/58	1.76	90	4.55	64
3.81	5/24/68	3.10	48	2.02	66
4.76	4/23/79	2.55	49	3.85	68
5.44	5/28/76	3.13	53	3.60	68
6.35	9/17/60	2.31	30	3.46	69
7.62	10/01/63	3.18	61	3.53	73
9.52	10/13/51	2.00	35	4.73	80
12.69	6/21/45	2.49	41	5.83	81
19.04	6/19/69	3.55	54	2.95	95
38.08	4/14/42	5.26	79	11.78	181

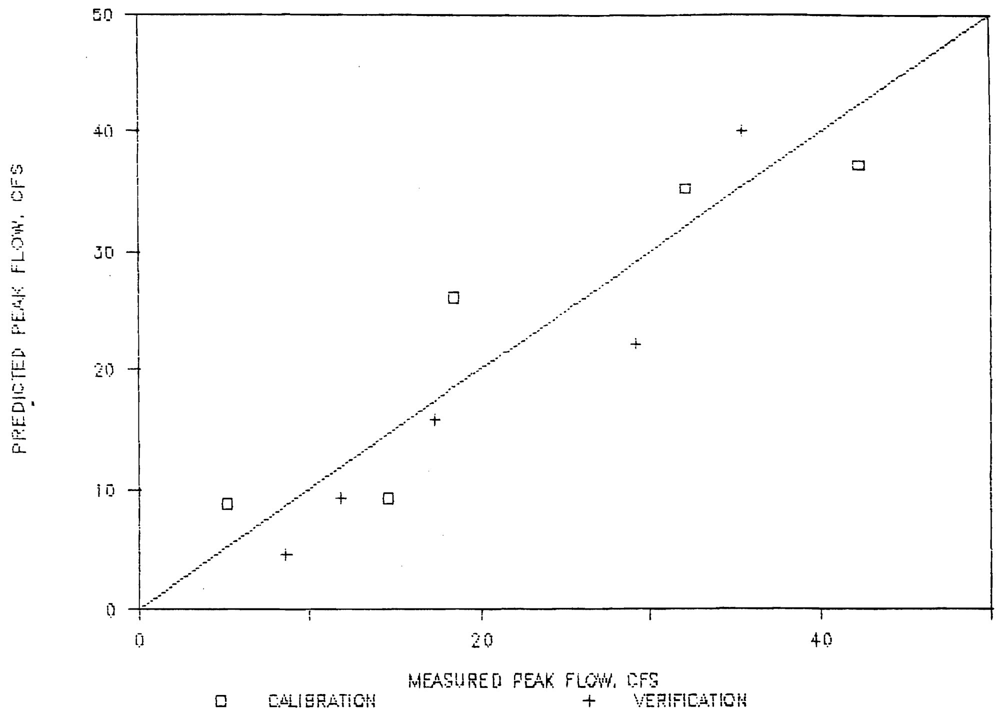


Figure 23. Peak Flow Results from the Calibration and Verification Runs--HWY Basin.

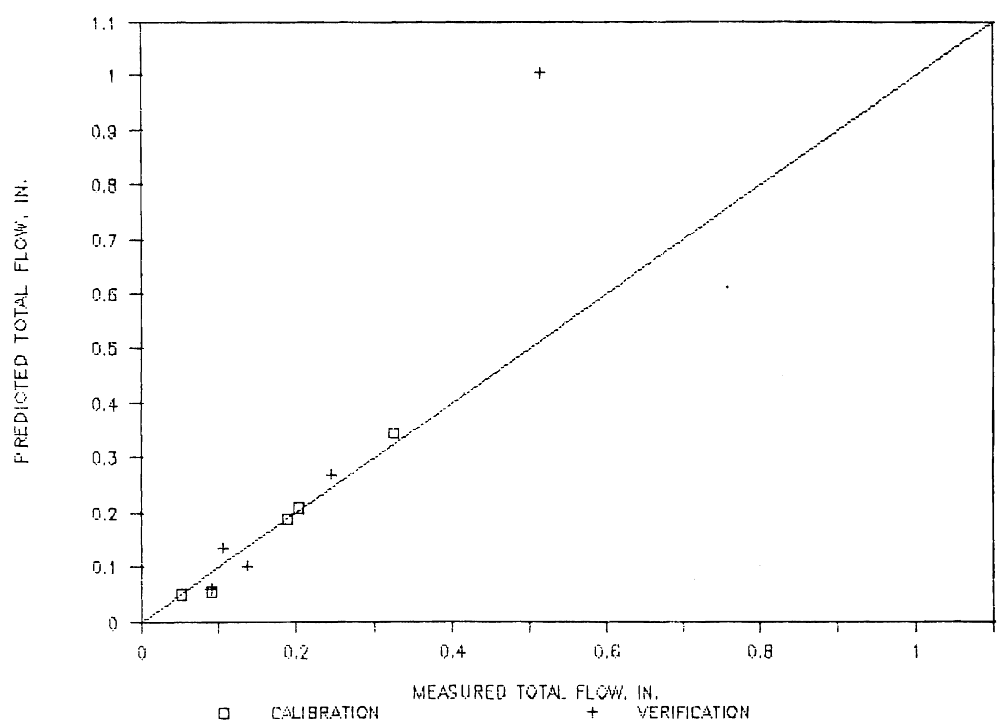
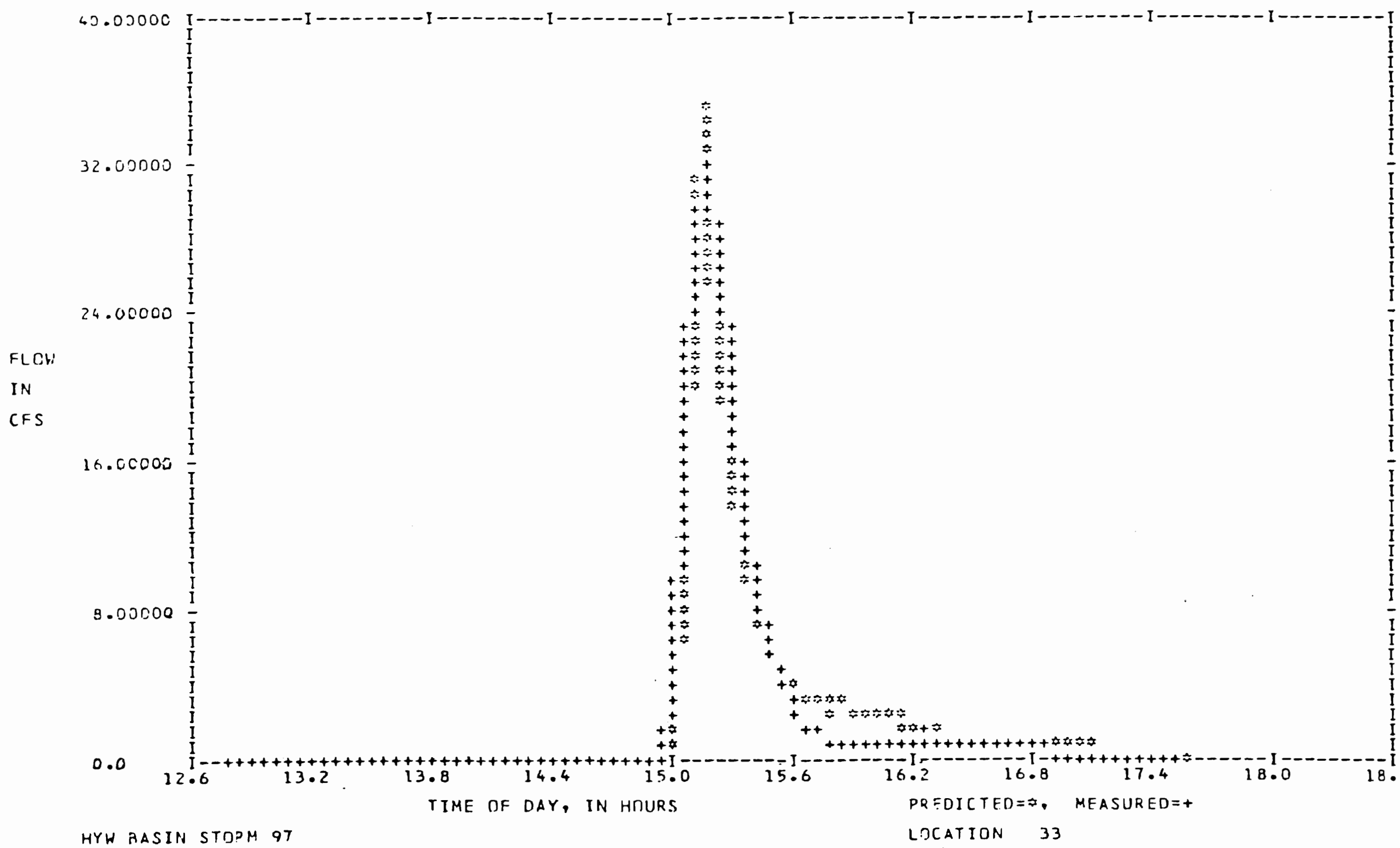


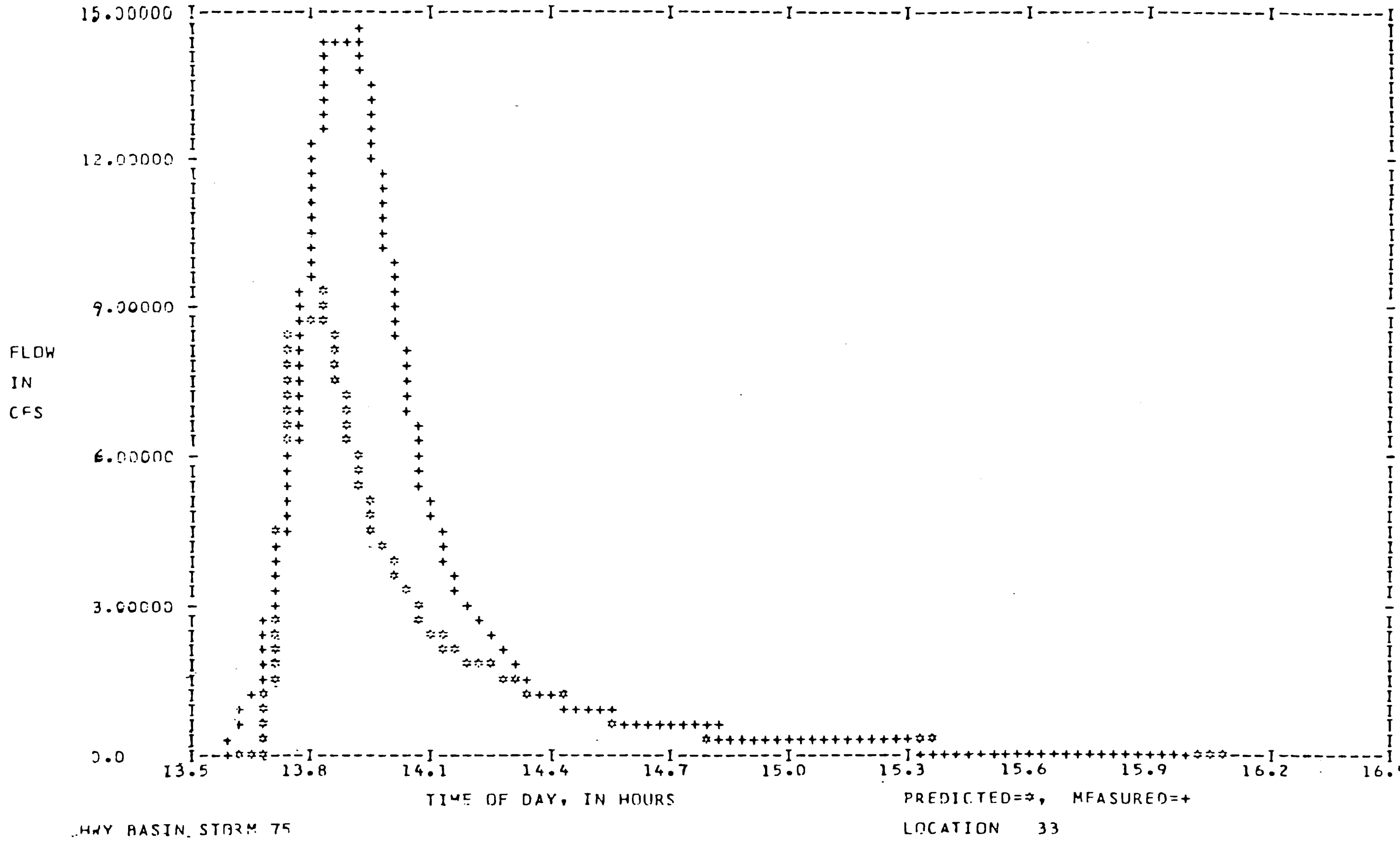
Figure 24. Total Flow Results from the Calibration and Verification Runs--HWY Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME,HR	PEAK FLOW FLOW,CFS	DUPATION START,HR	DUPATION END,HR	DUPATION LENGTH,HR	NO. POINTS
PREDICTED, TOTAL TIMF	0.39442E+05	0.187	15.167	35.206	12.750	17.583	4.833	117
MEASURED, TOTAL TIME	0.39952E+05	0.189	15.167	32.030	12.750	17.500	4.750	58

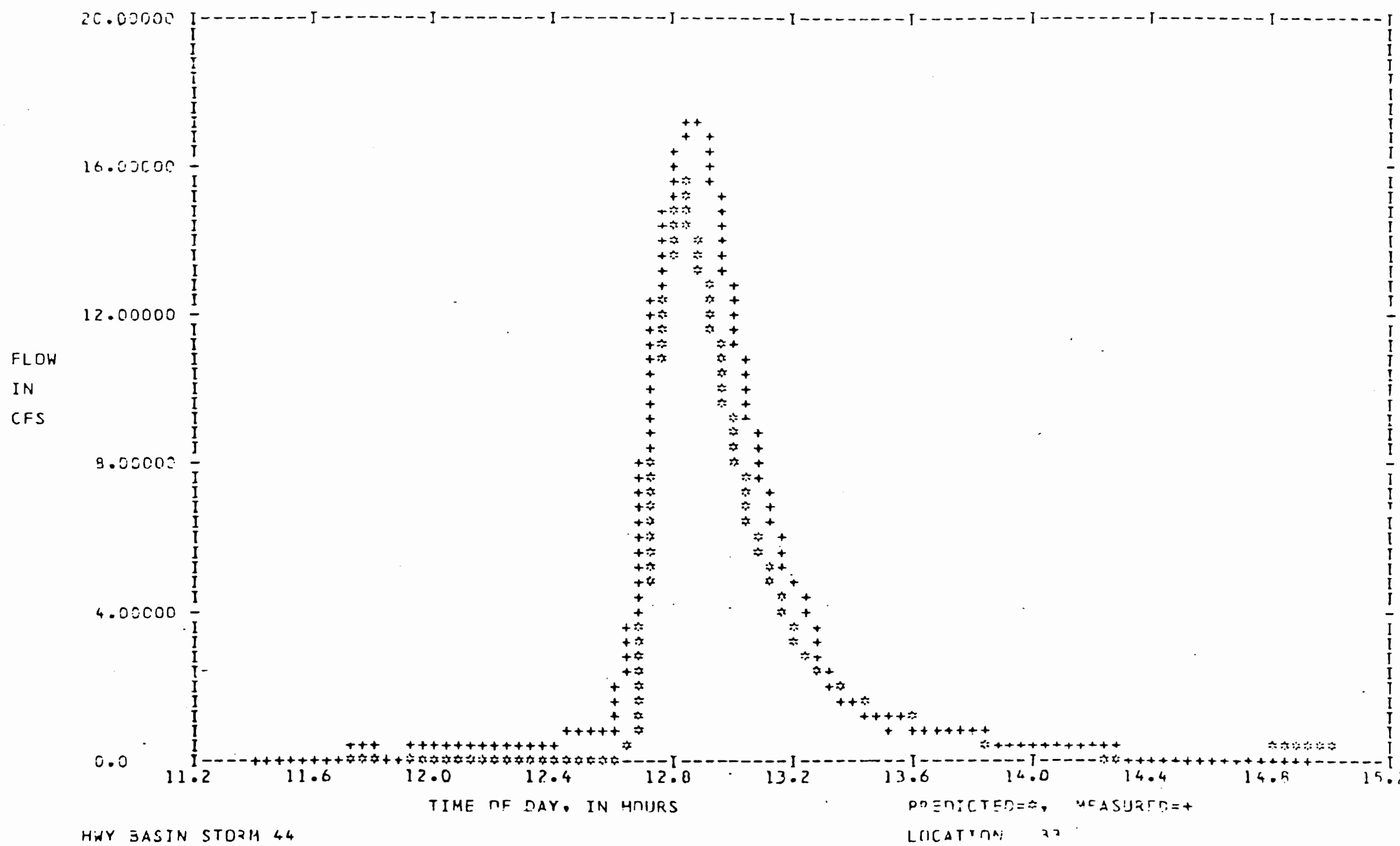
Figure 25. Predicted and Measured Hydrographs from the Best-Fitting Calibration Run on the HWY Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME,HR	FLOW,CFS	START,HR	END,HR	LENGTH,HR	POINTS
PREDICTED, TOTAL TIME	0.11923E+05	0.056	13.833	9.213	13.583	16.083	2.500	61
MEASURED, TOTAL TIME	0.19189E+05	0.091	13.917	14.570	13.583	16.000	2.417	30

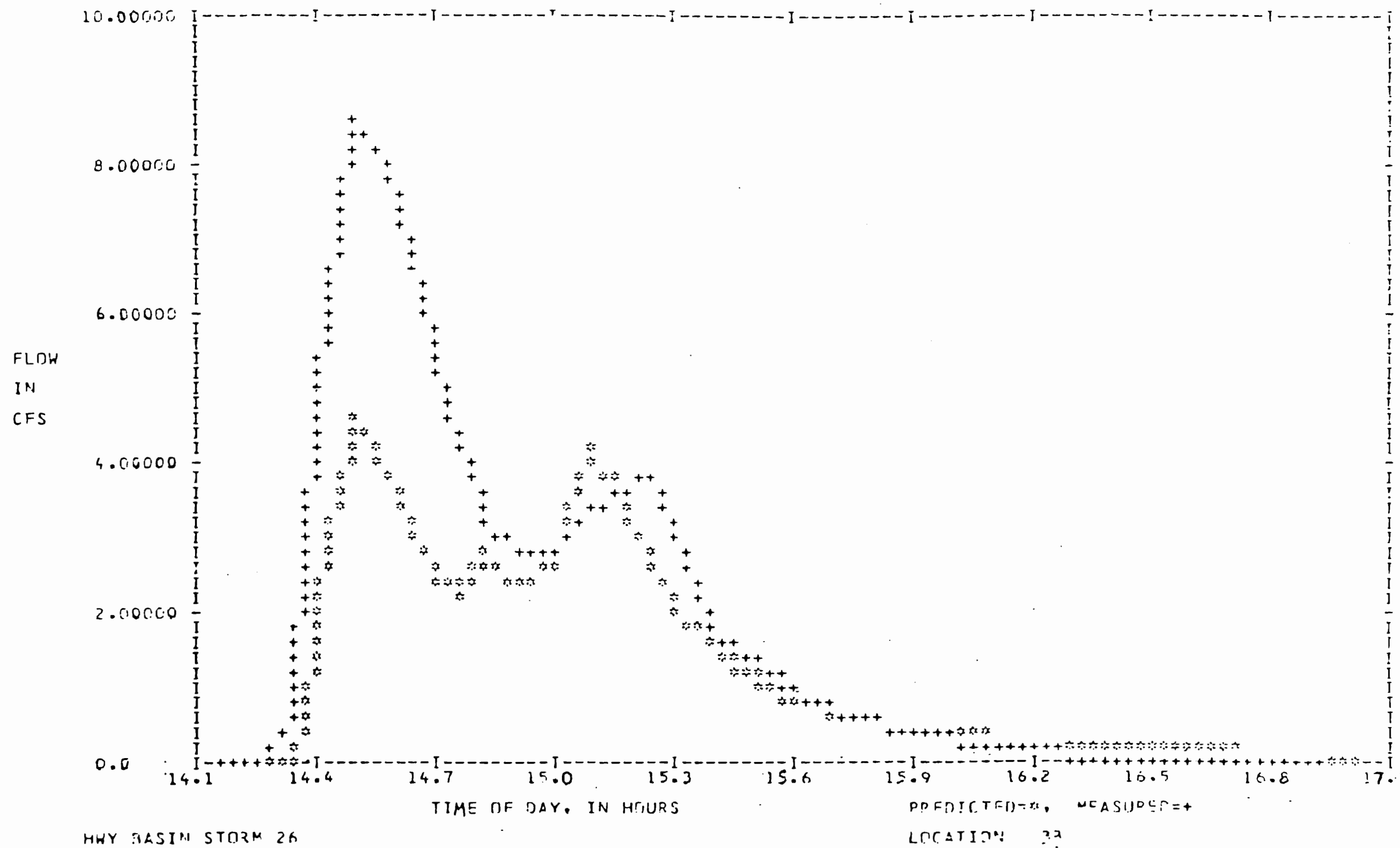
Figure 26. Predicted and Measured Hydrographs from the Worst-Fitting Calibration Run on the HWY Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME,HR	PEAK FLOW FLOW,CFS	DURATION START,HR	DURATION END,HR	DURATION LENGTH,HR	NO. POINTS
PREDICTED, TOTAL TIME	0.22167E+05	0.105	12.833	15.714	11.417	15.000	3.583	87
MEASURED, TOTAL TIME	0.28720E+05	0.136	12.833	17.300	11.417	14.917	3.500	43

Figure 27. Predicted and Measured Hydrographs from the Best-Fitting Verification Run on the HWY Basin.



HYDROGRAPH STATISTICS FOR LOCATION -33

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME,HR	FLOW,CFS	START,HR	END,HR	LENGTH,HR	POINTS
PREDICTED, TOTAL TIME	0.13001E+05	0.061	14.500	4.524	14.167	17.000	2.833	69
MEASURED, TOTAL TIME	0.19020E+05	0.090	14.500	8.640	14.167	16.917	2.750	34

Figure 28. Predicted and Measured Hydrographs from the Worst-Fitting Verification Run on the HWY Basin.

Table 24. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Total Flow--HWY Basin.

Return Period yr	Date	Runoff Duration hr	Runoff Volume in.
1.90	9/06/52	99	2.81
2.00	1/21/57	27	2.81
2.72	10/16/59	119	3.22
3.81	9/17/60	30	3.46
4.76	5/28/76	53	3.60
5.44	10/13/65	47	3.72
6.35	4/23/79	49	3.85
7.62	5/02/58	90	4.55
9.52	10/13/51	35	4.73
12.69	10/11/47	25	4.87
19.04	6/21/45	41	5.83
38.08	4/14/42	79	11.78

Summary of Results for HWY Basin

The peak flows produced by the seven methodologies are listed in Table 25 and shown Figure 29, which does not show the results from the THM or the largest value from CS in order that more detail may be seen among the curves from the other methodologies. Once again, the THM calculated much higher peak flows than all of the other methods. The general order of methods producing the next highest peak flows is as follows: CH/SCS, PDM, RE, CS, CM/CH, and RM. As was the case for the results of the two basins previously discussed, the value of largest return period peak from CS seemed somewhat high compared to the rest of the values from that method, suggesting that the true return period for that value may be somewhat higher than that assigned to it by the Weibull formula. The SUH/CH and the SUH/SCS produced unreasonably low peak flow values. Again, the low values are probably due to the large rainfall interval used.

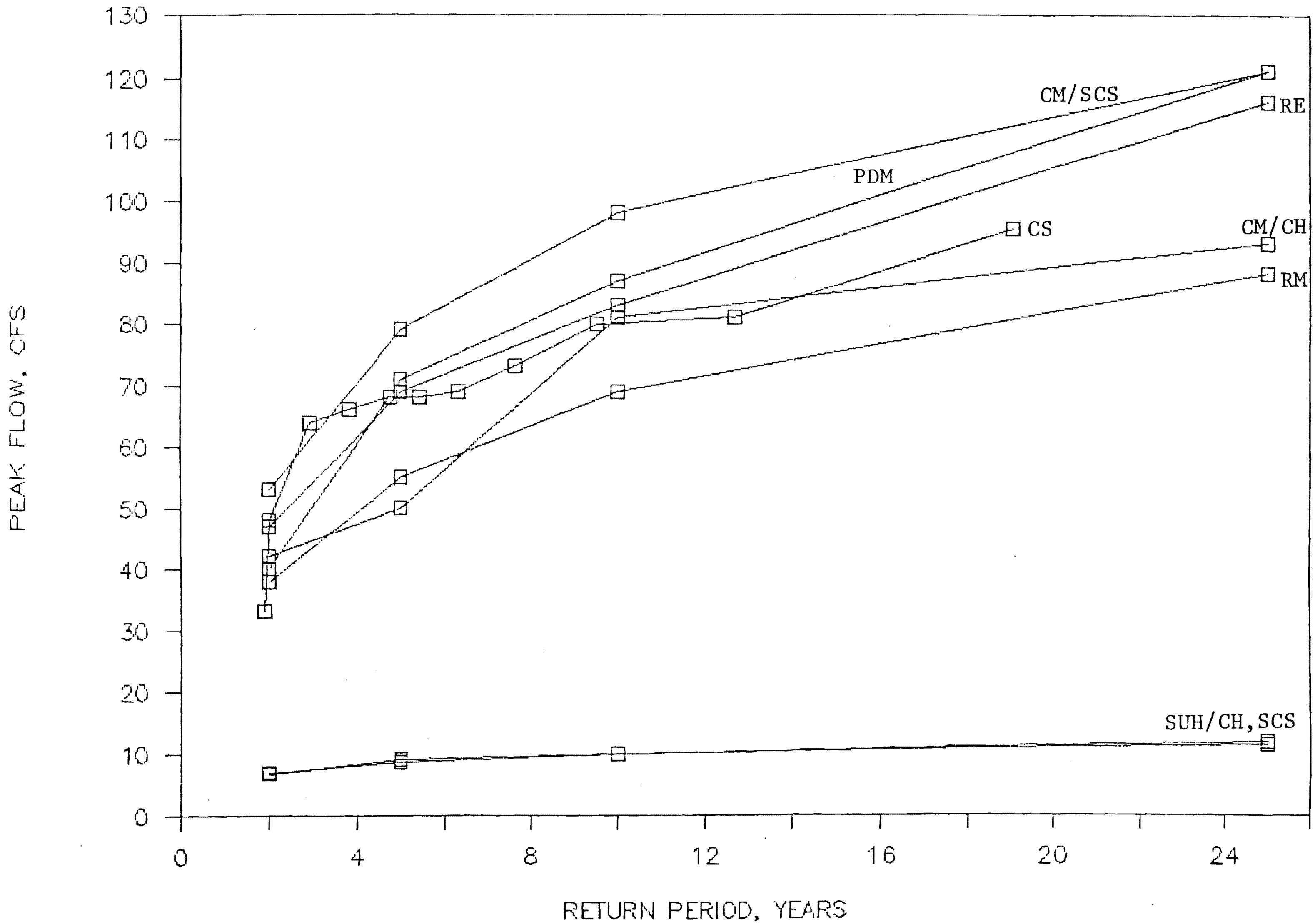


Figure 29. Peak Flow Results for the HWY Basin.

Table 25. Results of Peak Flow from Eight Methodologies--HWY Basin.

Return Period yr	CS	Peak Flow, cfs PDM	THM	RM	SUH/CH	SUH/SCS	CM/CH	CM/SCS	RE
1.90	33								
2.00	48	40	77	38	6.8	7.0	42	53	47
2.93	64								
3.81	66								
4.76	68								
5		71	142	55	9.1	8.7	50	79	69
5.44	68								
6.35	69								
7.62	73								
9.52	80								
10		87	174	69	10	10	81	98	83
12.69	81								
19.04	95								
25		121	241	88	12	12	93	121	116
38.08	181								

Table 26 and Figure 30 contain the results from the methods that give a total flow. CS, the SCS methods, and the CM/SCS gave similar values for all return periods except the 25 year return period, where the value of the largest return period from CS made the CS-curve diverge from the other two. The CM/CH gave the lowest volumes for all four return periods.

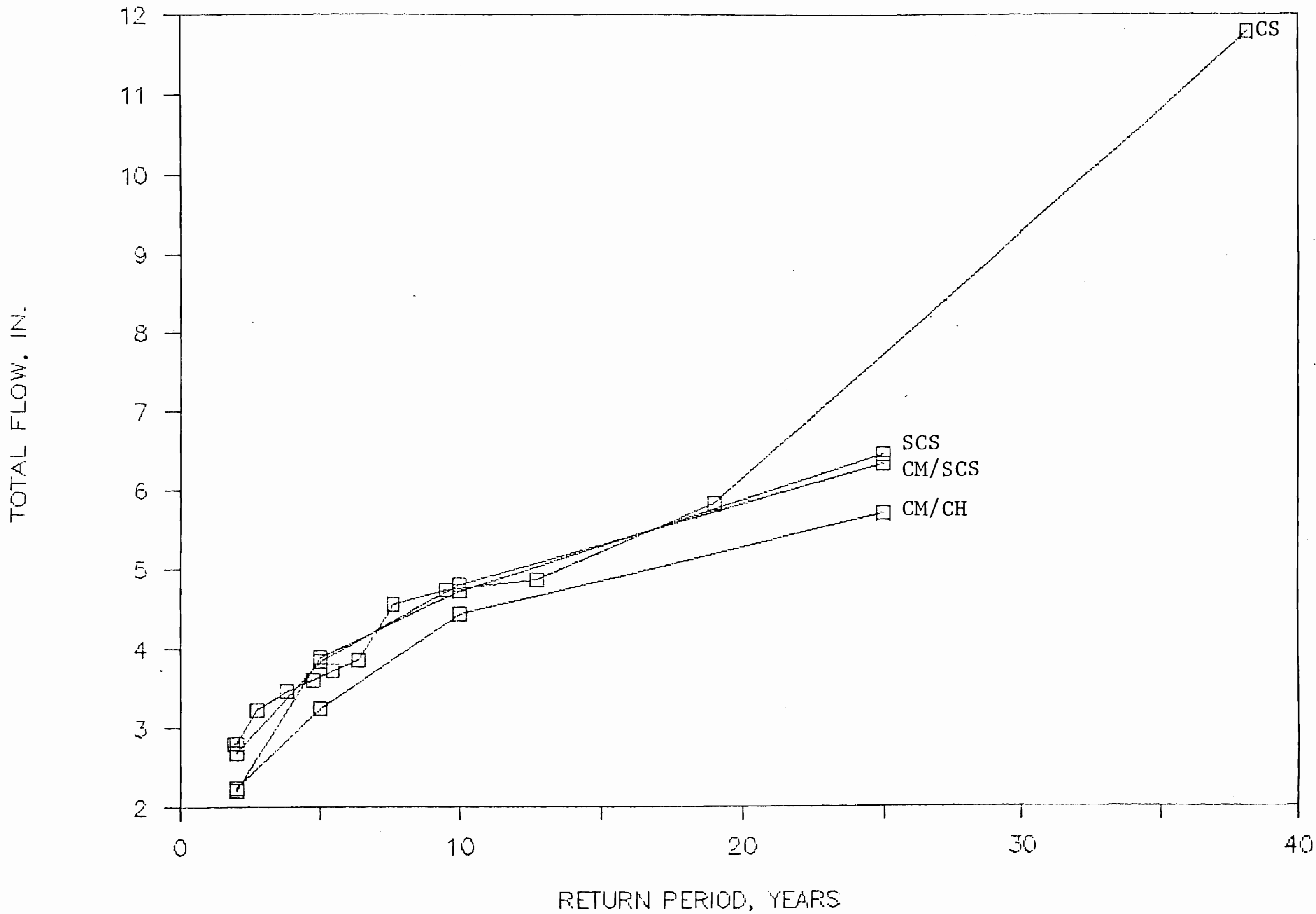


Figure 30. Total Flow Results for the HWY Basin.

Table 26. Results of Total Flow from Various Methodologies--HWY Basin.

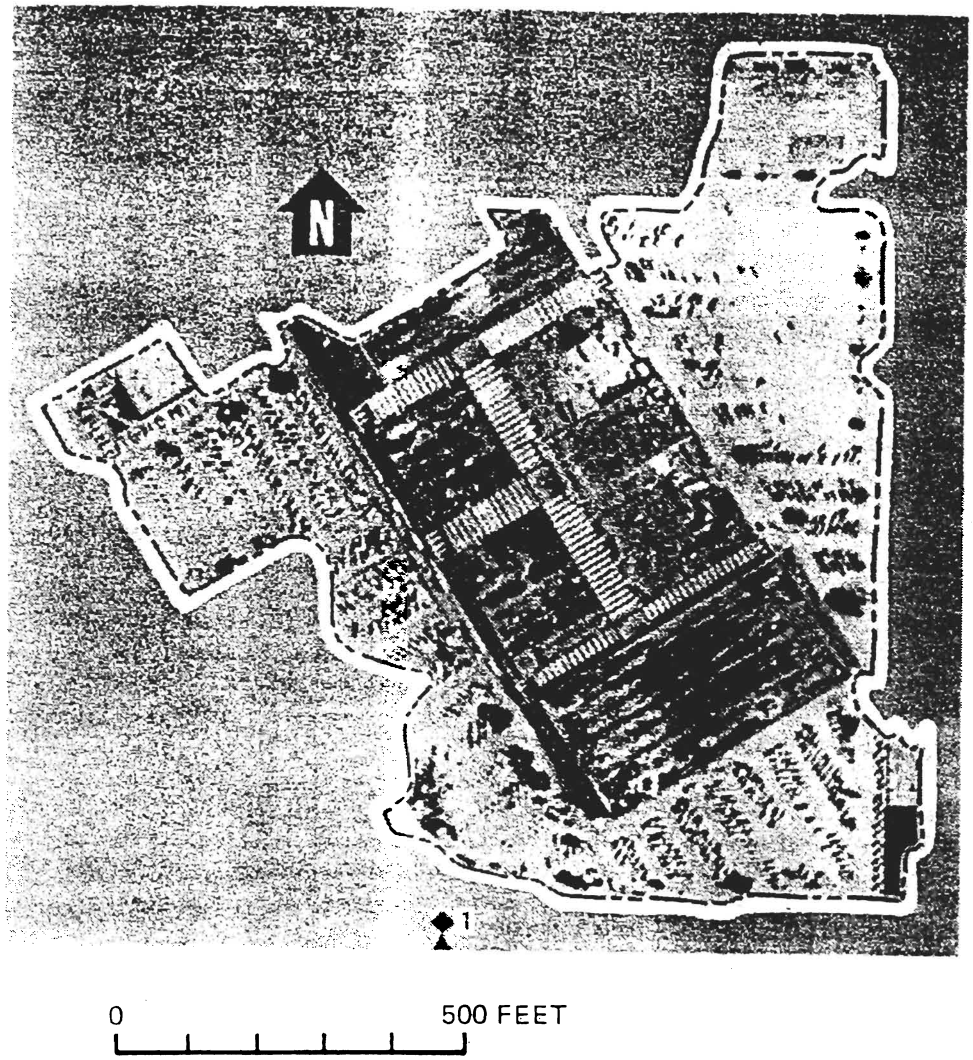
Return Period yr	Total Flow, in. CS	SCS	CM/CH	CM/SCS
1.90	2.81			
2.00	2.81	2.21	2.25	2.68
2.72	3.22			
3.81	3.46			
4.76	3.60			
5		3.89	3.24	3.83
5.44	3.72			
6.35	3.85			
7.62	4.55			
9.52	4.73			
10		4.72	4.42	4.79
12.69	4.87			
19.04	5.83			
25		6.43	5.70	6.32
38.08	11.78			

Ft. Lauderdale, FL (COM Basin)

The study area in Ft. Lauderdale (henceforth referred to as the COM basin) consists of a shopping center and its associated parking lots all sitting on 20.4 acres, as shown in Figure 31. The area is almost totally impervious and is very flat. An extensive storm sewer system consisting of circular concrete pipes drains the runoff from the area (Miller, 1979; Miller et al., 1979).

Results

The same seven methodologies that were applied to the HWY and LDR basins were applied to the COM basin. Parameters for the different methodologies came from the same sources as for the HWY and LDR basins and are shown in Table 27. The results from all of the methods are summarized in Tables 32 and 33 at the end of this section.



EXPLANATION:

--- BASIN BOUNDARY
◆[2] RAIN GAGE WITH NUMBER
▲ URBAN HYDROLOGY MONITOR

Figure 31. Photomosaic Map of the COM Basin (Miller, 1979).

Table 27. Parameters Used for the Methods Applied to the COM Basin.*

Watershed Drainage Area	20.4 ac
	0.03188 sq mi
Length of Main Channel	1200 ft
Main Channel Slope	0.0015 ft/ft
Hydraulically Effective Impervious Area	98 %
Average Basin Slope	0.001 ft/ft
Soil Group	NA
Curve Number	98
Conveyance Efficiency	0.62
Adjustment Factors	
Impervious Area	1.0
Hydraulic Length	1.0
Slope	0.47
Ponding	

Return Period yr	Ponding Adjustment Factor
2	1.0
5	1.0
10	1.0
25	1.0

Lake Area	1 %
2 Year, 2 Hour Rainfall Intensity	3.2 in.
Basin Development Factor	11
Basin Storage	0.1 %
Subcatchment Width	6325 ft
Manning's n	
Pervious Area	0.25
Impervious Area	0.015
Green-Ampt Infiltration Parameters	
Hydraulic Conductivity	0.15 in./hr
Capillary Suction	8.00 in.
Initial Moisture Deficit	0.05

* Generally not SWMM input values. See Appendix C.

Rational method

The parameters used the RM are listed in Table 28. Nine pipes were used in the calculation of the pipe travel time. Regional IDF curves were used for the calculations (Weldon, 1985).

Table 28. Parameters from the Rational Method--COM Basin.

Return Period yr	Wave Pipes min	Travel Time Overland Flow min	Time of Concentration min	Rainfall Intensity in./hr	c
2	4.4	12.9	17.3	5.10	.98
5	4.3	11.9	16.2	6.08	.98
10	4.3	11.5	15.8	6.57	.99
25	4.1	10.9	15.0	7.59	.99

Synthetic unit hydrograph method

The five watershed parameters required for the development of the unit hydrograph are listed in Table 27. Rainfall excess was determined by multiplying the design storm hyetograph values by the fraction of impervious area.

SCS peak discharge method

The six watershed parameters required the PDM are listed in Table 27, as are the adjustment factors. Rainfall (Hershfield, 1961) and calculated runoff depths are listed in Table 29.

Table 29. Rainfall and Runoff Depths from the SCS Peak Discharge Method--COM Basin.

Return Period yr	Rainfall Depth in.	Runoff Depth in.
2	5.9	5.66
5	8.0	7.76
10	9.0	8.76
25	11.0	10.76

SCS tabular hydrograph method

Most of the values needed for the THM are listed in Table 27. Time of concentration was calculated in the same manner as for the other basins.

Regression equations

Table 27 lists the three parameters required to calculate predevelopment peak flows and the four additional parameters required to determine the urbanized peak flows. Postdevelopment peak flows are given in Table 32.

Calibrated SWMM with synthetic design storms

The SCS and Chicago design storms were used as rainfall input to the calibrated model. Average antecedent moisture conditions were assumed. The calibration is described below.

Design storms from continuous simulation

Rainfall-runoff data for calibration and verification came from a USGS Open-File Report (Miller et al., 1979). The calibration and verification done in the same manner as for the MEG basin. Parameters from the final calibration runs are listed in Table 27. Results for peak flow and total flow from the calibration and verification runs can be seen in Figures 32 and 33. Hydrographs of measured and predicted flows from the calibration storms producing the best and worst fits are shown in Figures 34 and 35, respectively; similar hydrographs from the verification runs are displayed in Figures 36 and 37.

A 29.25-year record of hourly rainfall data from the Miami Airport (station # 085663) was used for the long-term rainfall record for continuous simulation. The record from that station was chosen

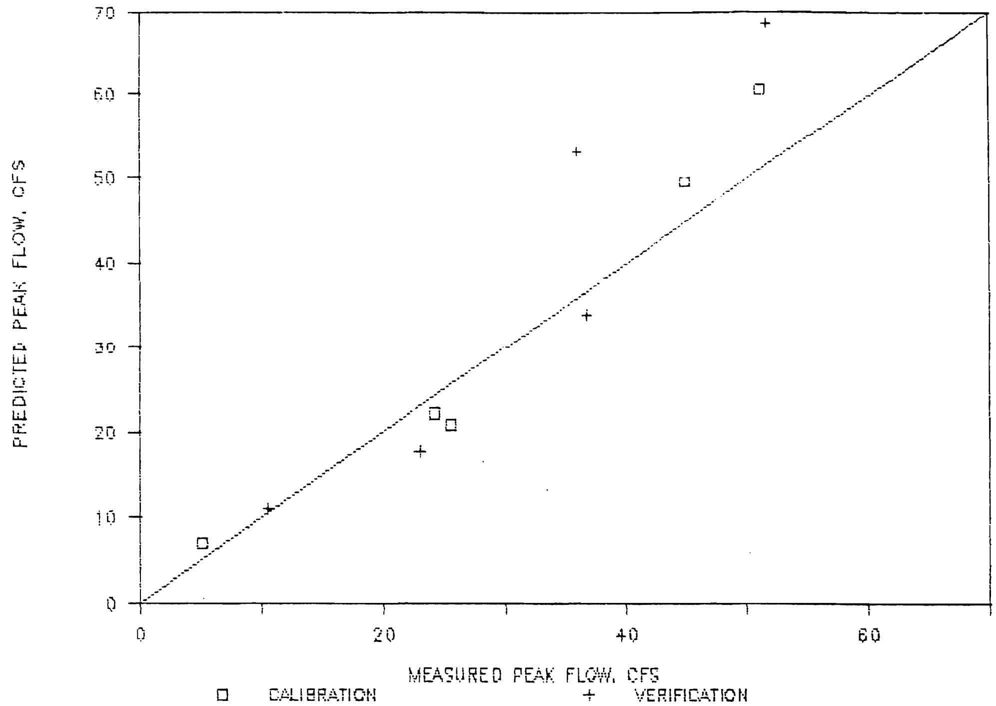


Figure 32. Peak Flow Results from the Calibration and Verification Runs--COM Basin.

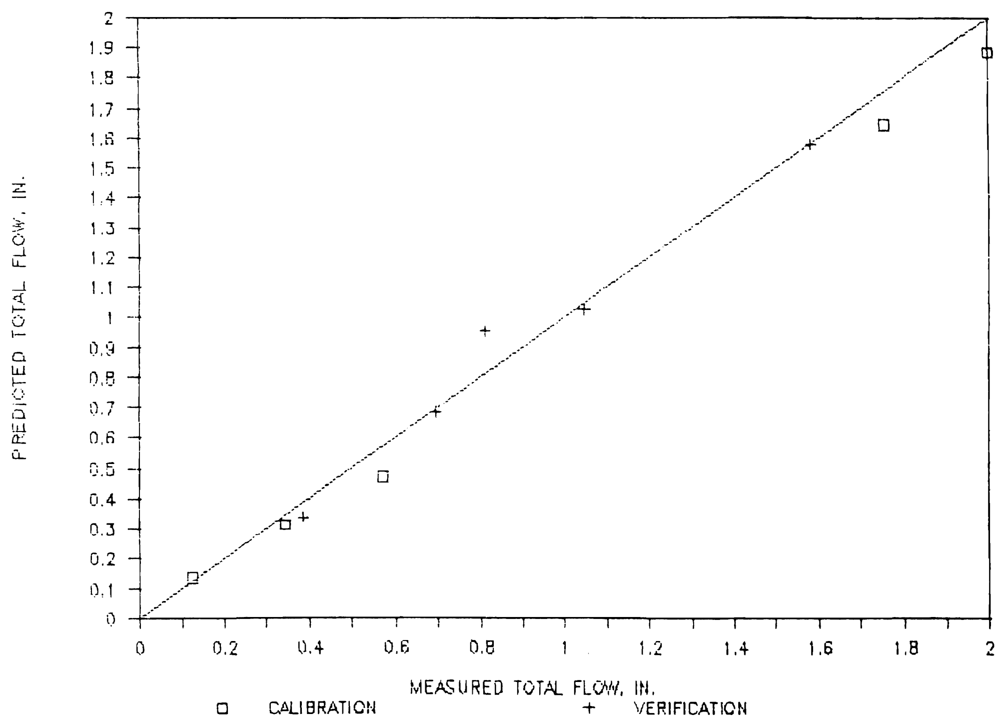
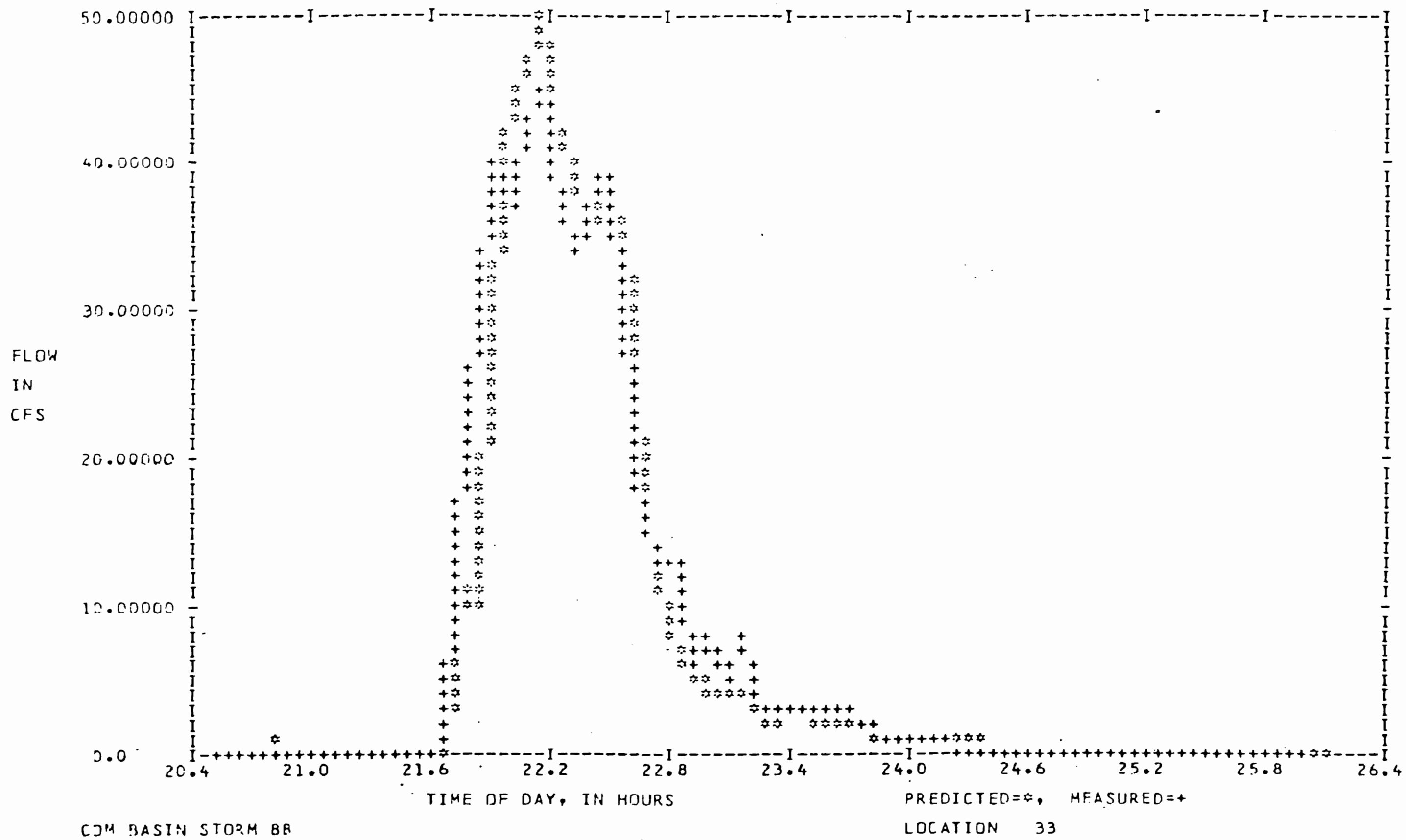


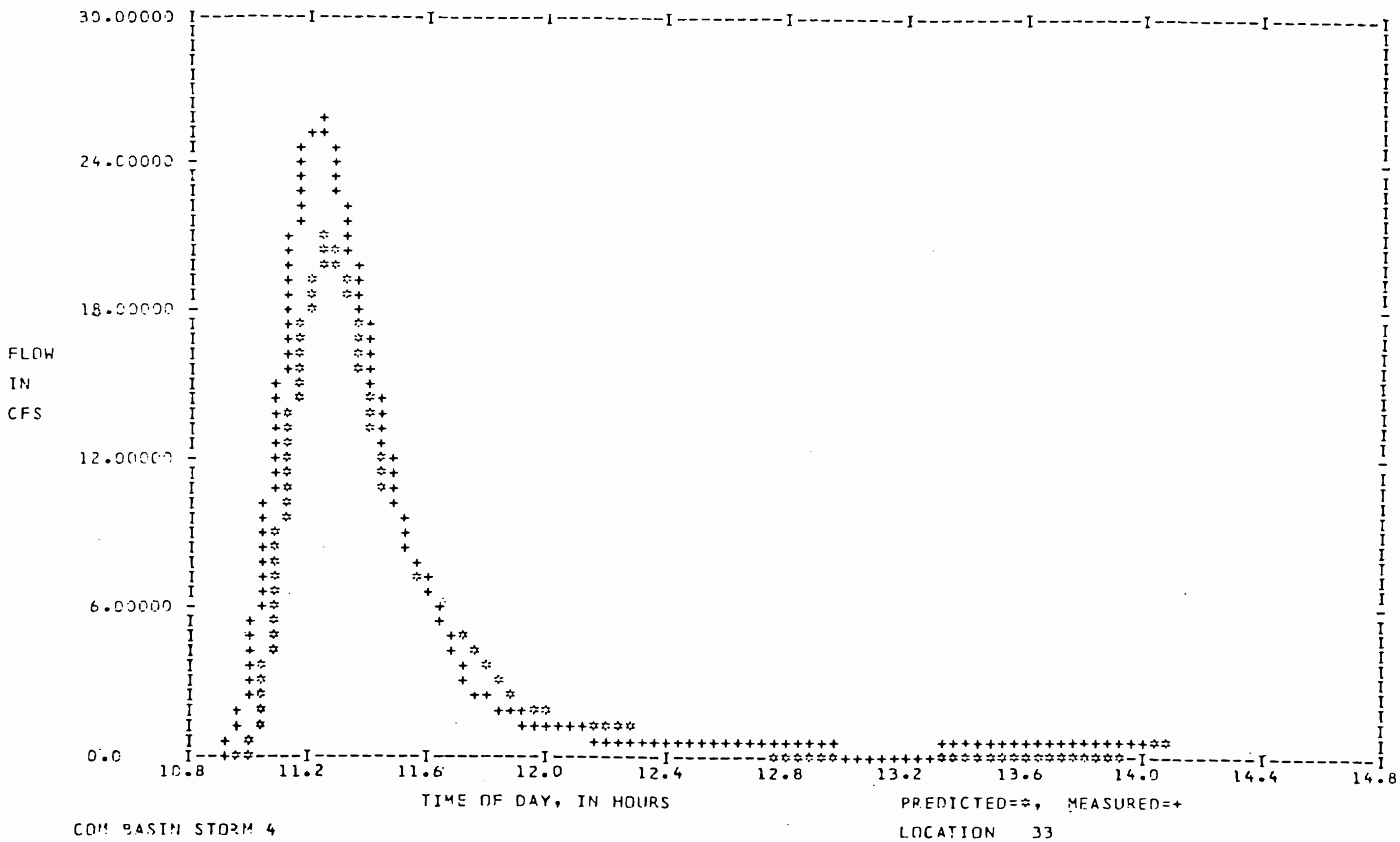
Figure 33. Total Flow Results from the Calibration and Verification Runs--COM Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME,HR	FLOW,CFS	START,HR	END,HR	LENGTH,HR	POINTS
PREDICTED, TOTAL TIME	0.13952E+06	1.884	22.167	49.579	20.500	26.083	5.583	135
MEASURED, TOTAL TIME	0.14800E+06	1.999	22.167	44.910	20.500	26.000	5.500	67

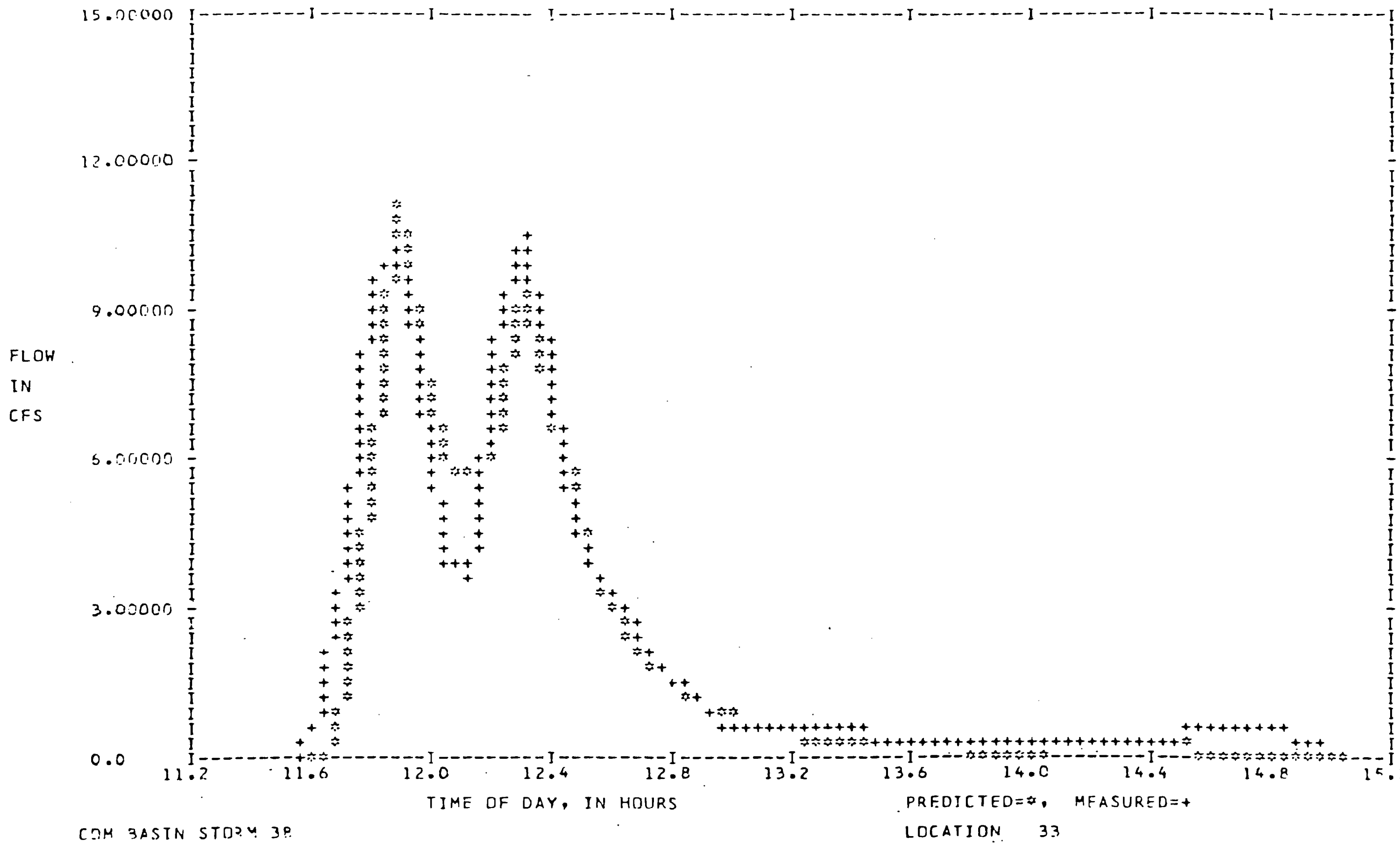
Figure 34. Predicted and Measured Hydrographs from the Best-Fitting Calibration Run on the COM Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME,HR	FLOW,CFS	START,HR	END,HR	LENGTH,HR	POINTS
PREDICTED, TOTAL TIME	0.34971E+05	0.472	11.250	20.784	10.917	14.083	3.167	77
MEASURED, TOTAL TIME	0.42222E+05	0.570	11.250	25.600	10.917	14.000	3.083	38

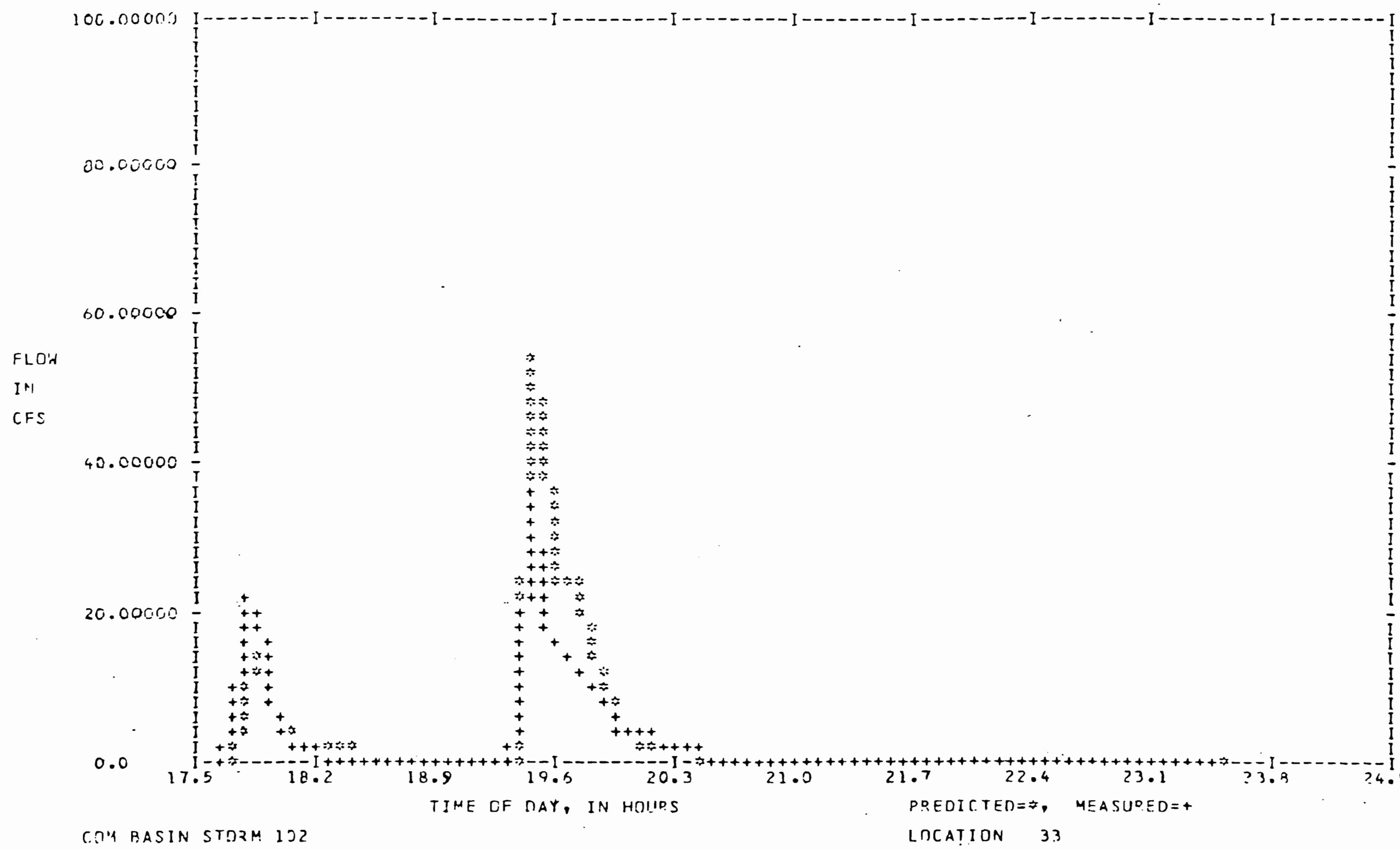
Figure 35. Predicted and Measured Hydrographs from the Worst-Fitting Calibration Run on the COM Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME,HR	PEAK FLOW FLOW,CFS	DURATION START,HR	DURATION END,HR	DURATION LENGTH,HR	NO. POINTS.
PREDICTED, TOTAL TIME	0.25048E+05	0.338	11.883	10.963	11.550	15.050	3.500	85
MEASURED, TOTAL TIME	0.28518E+05	0.385	12.300	10.560	11.550	14.967	3.417	42

Figure 36. Predicted and Measured Hydrographs from the Best-Fitting Verification Run on the COM Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME,HR	PEAK FLOW FLOW,CFS	DURATION START,HR	DURATION END,HR	DURATION LENGTH,HR	NO. POINTS
PREDICTED, TOTAL TIME	0.70723E+05	0.955	19.467	53.257	17.633	23.550	5.917	143
MEASURED, TOTAL TIME	0.60184E+05	0.813	19.467	36.090	17.633	23.467	5.833	71

Figure 37. Predicted and Measured Hydrographs from the Worst-Fitting Verification Run on the COM Basin.

because that station is the closest station to the COM basin with a long time series of data. An MIT of 23 hours was used to define independent events, yielding 2118 of them. Twelve of the 20 highest ranking events from continuous simulation (return periods from 1.96 to 29.3 years) based on peak flow and total flow were run in single-event mode with a 5-min time step and several days of antecedent rainfall, as was done for the previously discussed study areas. The return periods of the 12 events based on peak flow and the 12 events based on total flow were rearranged according the ranking based on the single-event simulation results. Tables 30 and 31 list the results from the 12 events based on peak flow and total flow, respectively.

Table 30. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Peak Flow--COM Basin.

Return Period yr	Date	Maximum Intensity in./hr	Runoff Duration hr	Runoff Volume in.	Peak Flow cfs
1.96	9/06/60	2.25	30	3.16	46
2.10	5/13/58	2.26	48	3.73	46
2.44	7/02/52	2.33	60	5.30	48
2.93	7/07/51	2.55	72	4.98	52
3.67	5/19/68	2.62	182	8.38	54
4.19	5/04/77	2.79	39	11.45	57
4.89	4/29/57	2.95	65	8.10	60
5.87	6/16/59	3.14	133	11.87	64
7.33	9/03/59	3.29	28	3.71	67
9.78	6/03/74	3.33	52	5.27	68
14.67	8/26/64	3.65	27	6.75	75
29.33	4/24/79	4.51	28	16.00	126

Table 31. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Total Flow--COM Basin.

Return Period yr	Date	Runoff Duration hr	Runoff Volume in.
1.96	4/29/57	65	8.10
2.10	9/08/60	42	8.11
2.44	5/19/68	182	8.38
2.93	6/01/64	220	8.57
3.67	10/24/52	85	9.15
4.19	5/31/77	118	9.73
4.89	6/01/66	196	9.86
5.87	11/18/59	78	9.96
7.33	5/22/58	52	10.24
9.78	5/04/77	39	11.45
14.67	6/16/59	133	11.87
29.33	4/24/79	28	16.00

Summary of Results for the COM Basin

Table 32 and Figure 38 display the results of the seven methodologies used to determine peak flow rates. In order to show more detail among the variuos curves, the THM and RM curves are omitted from Figure 38. The THM and the RM gave much higher peak flows than the other methods, with the THM being the higher of the two. The next highest values came from, in general, the CM/SCS, CS, and the PDM. CS produced values lower than the CM/SCS and higher than the PDM for the three lowest return periods, but once again the largest value from CS made the curve for that method diverge up from curves that it was similar to at smaller return periods. The CM/CH gave the next highest flows, followed by, in general, the RE, the SUH/SCS, and the SUH/CH. The SUH/CH produced the lowest peak flows at all return periods except 25 years, where it gave the same value as the RE. Once again, the

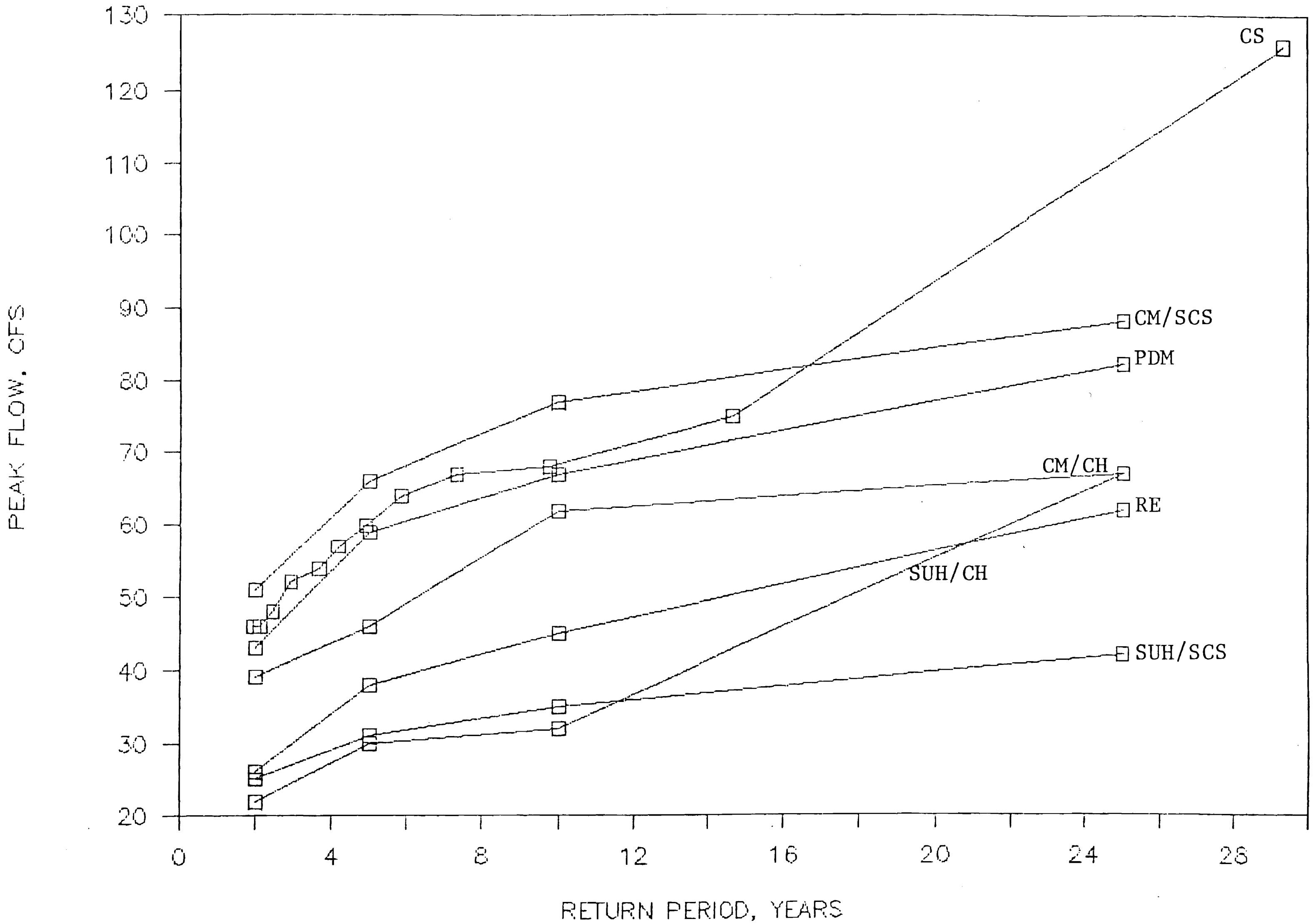


Figure 38. Peak Flow Results for the COM Basin.

SUH/CH and SUH/SCS results were low due to the large rainfall interval used.

Table 32. Results of Peak Flow from Eight Methodologies--COM Basin.

Return Period yr	CS	PDM	THM	RM	SUH/CH	SUH/SCS	CM/CH	CM/SCS	RE
1.96	46								
2		43	120	102	22	25	39	51	26
2.10	46								
2.44	48								
2.93	52								
3.67	54								
4.19	57								
4.89	60								
5		59	164	122	30	31	46	66	38
5.87	64								
7.33	67								
9.78	68								
10		67	185	132	32	35	62	77	45
14.67	75								
25		82	227	153	67	42	67	88	62
29.33	126								

Table 33 and Figure 39 give the results from the four methodologies that calculate total flows. CS yielded the largest volumes, by far, for all return periods. The SCS methods gave the next largest volumes, followed by the CM/SCS and CM/CH.

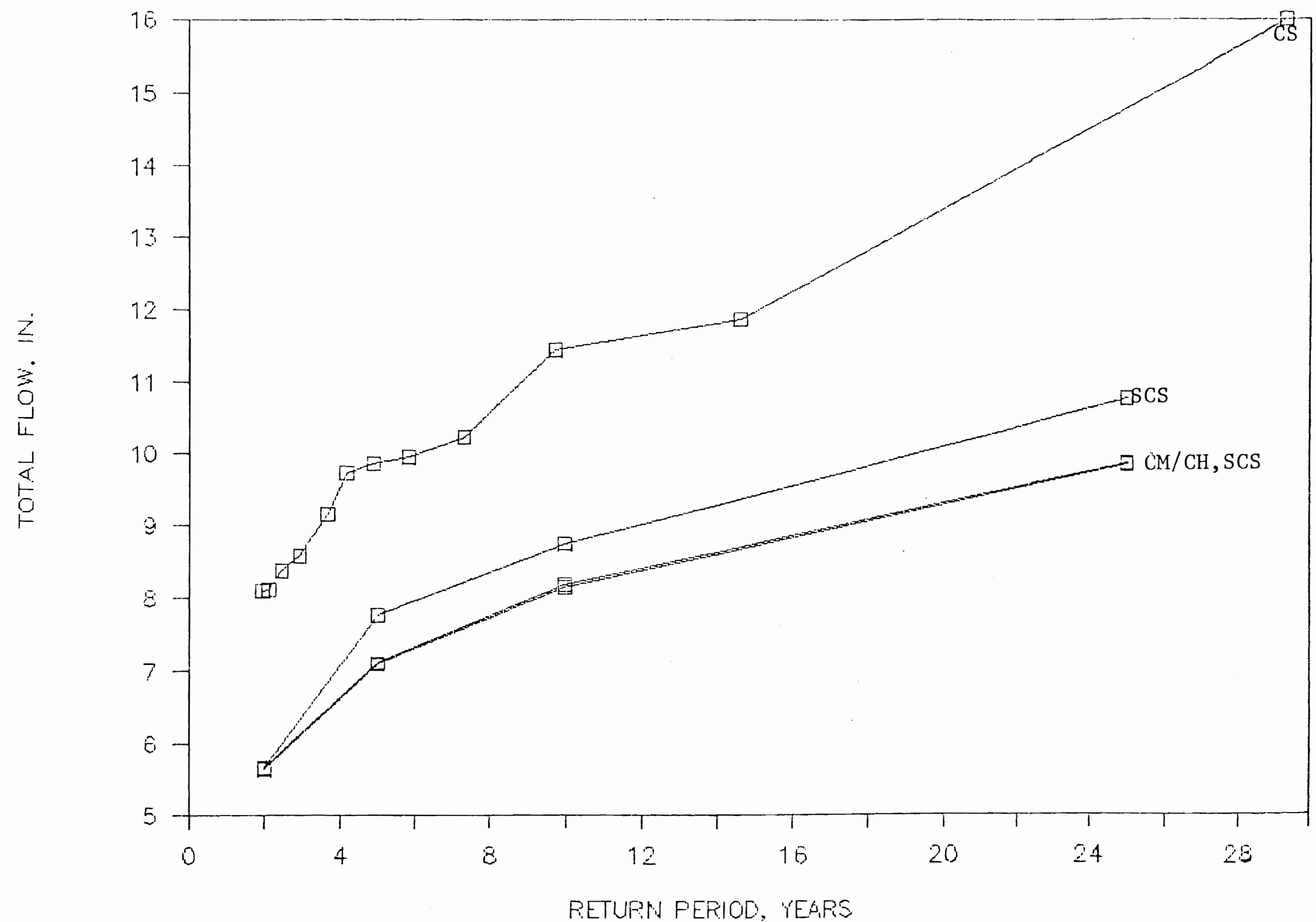


Figure 39. Total Flow Results for the COM Basin.

Table 33. Results of Total Flow from Various Methodologies--COM Basin.

Return Period yr	Total Flow, in.			
	CS	SCS	CM/CH	CM/SCS
1.96	8.10			
2		5.66	5.64	5.66
2.10	8.11			
2.44	8.38			
2.93	8.57			
3.67	9.15			
4.19	9.73			
4.87	9.86			
5		7.76	7.08	7.10
5.87	9,96			
7.33	10.24			
9.78	11.45			
10		8.76	8.16	8.18
14.67	11.87			
25		10.76	9.84	9.86
29.33	16.00			

Miami, FL (HDR Basin)

The site in Miami (henceforth referred to as the HDR basin) is a high-density residential area consisting of a large apartment complex that sits on 14.7 acres, as shown in Figure 40. The soil has a fairly low hydraulic conductivity. The streets drain the runoff to a corrugated metal sewer (Miller, 1979; Hardee et al., 1979).

Results

The seven methodologies applied to the COM basin were also applied to the HDR basin, and the parameters for the HDR basin came from the same sources as for the COM basin. The parameters used for the methodologies are listed in Table 34. Results are shown at the end of this section.

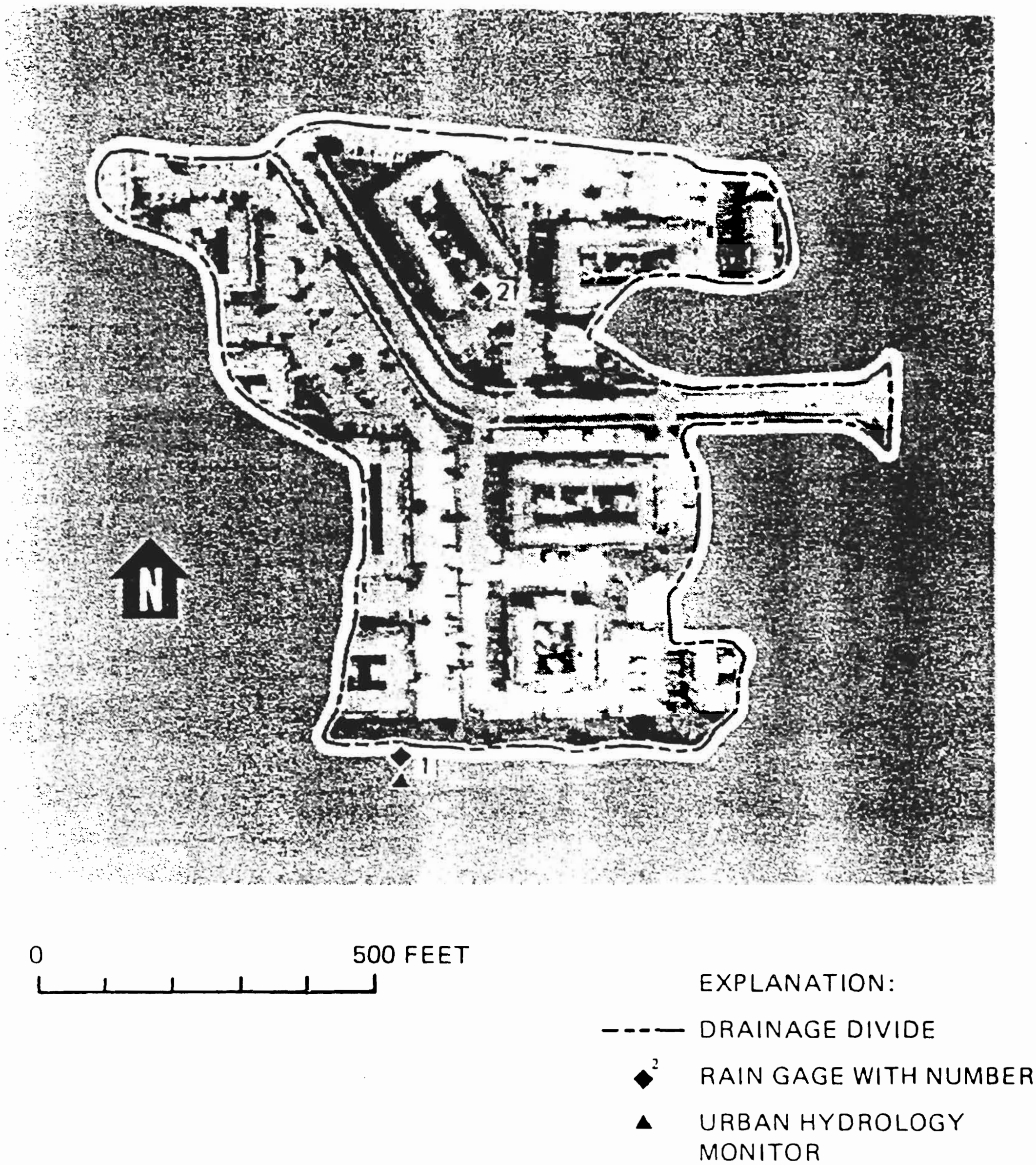


Figure 40. Photomosaic Map of the HDR Basin (Miller, 1979).

Table 34. Parameters Used for the Methods Applied to the HDR Basin.*

Watershed Drainage Area	14.7 ac
	0.02986 sq mi
Length of Main Channel	1100 ft
Main Channel Slope	0.002 ft/ft
Hydraulically Effective Impervious Area	44 %
Average Basin Slope	0.002 ft/ft
Soil Group	NA
Curve Number	95
Conveyance Efficiency	0.7
Adjustment Factors	
Impervious Area	1.08
Hydraulic Length	1.18
Slope	0.60
Ponding	

Return Period yr	Ponding Adjustment Factor
2	0.94
5	0.95
10	0.96
25	0.97

Lake Area	1 %
2 Year, 2 Hour Rainfall Intensity	3.2 in.
Basin Development Factor	9
Basin Storage	0.2 %
Subcatchment Width	2325 ft
Manning's n	
Pervious Area	0.25
Impervious Area	0.015
Green-Ampt Infiltration Parameters	
Hydraulic Conductivity	0.40 in./hr
Capillary Suction	10.00 in.
Initial Moisture Deficit	0.10

* Generally not SWMM input values. See Appendix C.

Rational method

Table 35 gives the parameters used for the RM calculations. Five pipes were used in the calculation of pipe travel time. Regional IDF curves were used in the calculations (Weldon, 1985).

Table 35. Parameters from the Rational Method--HDR Basin.

Return Period yr	Wave Travel Time Pipes min	Wave Travel Time Overland Flow min	Time of Concentration min	Rainfall Intensity in./hr	c
2	0.98	25.5	26.5	4.22	.44
5	0.97	23.0	24.0	5.20	.48
10	0.96	21.8	22.8	5.78	.51
25	0.95	20.1	21.1	6.77	.55

Synthetic unit hydrograph method

The five watershed parameters needed for the development of the synthetic unit hydrograph are listed in Table 34. Rainfall excess was determined by multiplying the intensities from the synthetic storm hyetographs by the fraction of hydraulically effective impervious area.

SCS peak discharge method

The six parameters needed for the PDM are given in Table 34, along with the adjustment factors for peak flow determination. Rainfall (Hershfield, 1961) and calculated runoff depths are listed in Table 36.

Table 36. Rainfall and Runoff Depths from the SCS Peak Discharge Method--HDR Basin.

Return Period yr	Rainfall Depth in.	Runoff Depth in.
2	6.0	5.41
5	8.0	7.40
10	9.0	8.40
25	10.9	10.29

SCS tabular hydrograph method

Most of the values required for this method are listed in Table 34. Time of concentration, the only other parameter needed, was determined from formulas supplied by the method (Livingston, 1984).

Regression equations

Table 34 lists the three parameters required to calculate predevelopment peak flows and the four additional parameters needed to determine the postdevelopment peak flows by use of the RE. The postdevelopment peak flows are given in Table 39 at the end of this section.

Calibrated SWMM with synthetic design storms

The SCS and Chicago design storms were used as rainfall input to the calibrated model. Average antecedent moisture conditions were assumed. The calibration is described below.

Design storms from continuous simulation

A USGS Open-File Report (Hardee et al., 1979) provided the rainfall-runoff data used for calibration and verification. The same method of calibration and verification that was used for the MEG basin was also used for the HDR basin. Parameters from the final calibration runs are listed in Table 34. Predicted and measured peak flows and total flows from the calibration and verification runs are shown in Figures 41 and 42. Hydrographs of predicted and measured flows from the calibration runs having the best and worst fits are displayed in Figures 43 and 44; similar hydrographs from the verification runs are displayed in Figures 45 and 46.

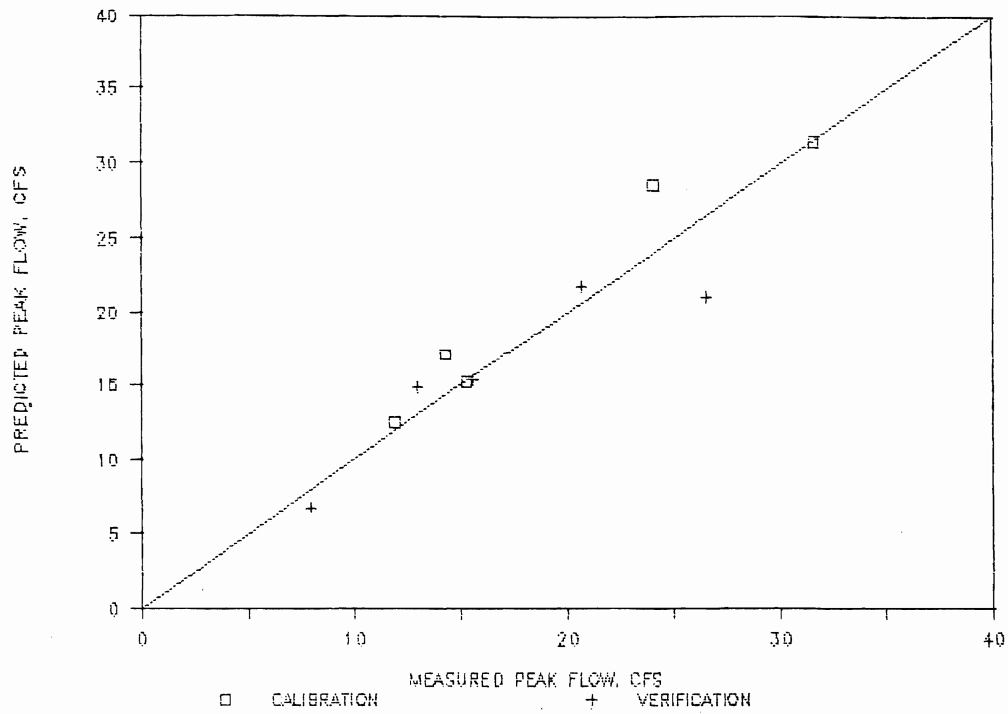


Figure 41. Peak Flow Results from the Calibration and Verification Runs--HDR Basin.

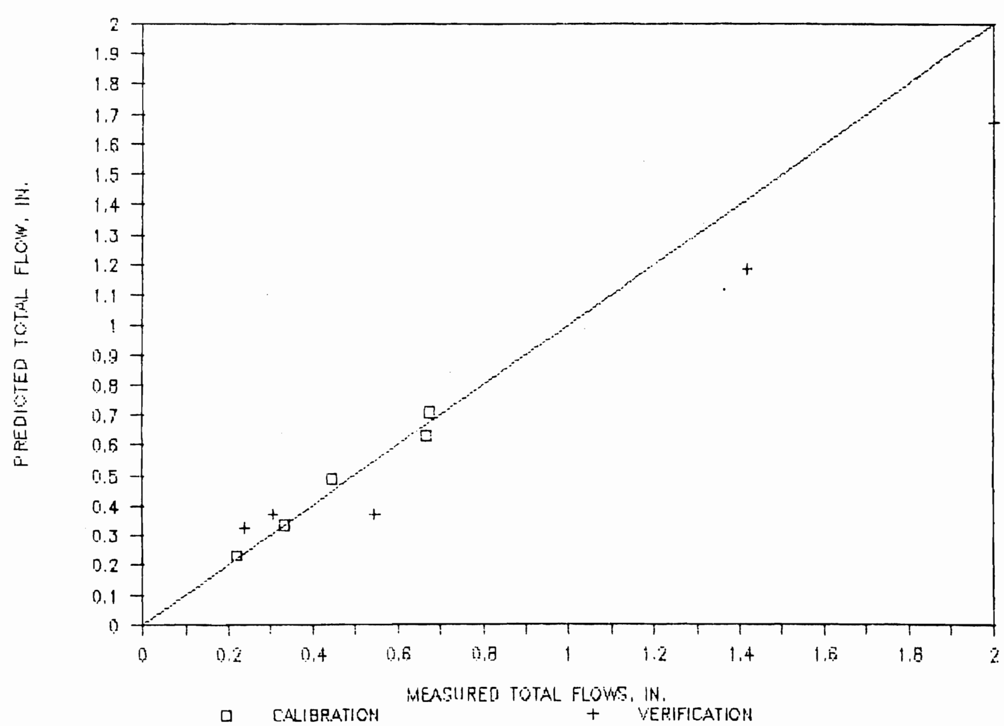
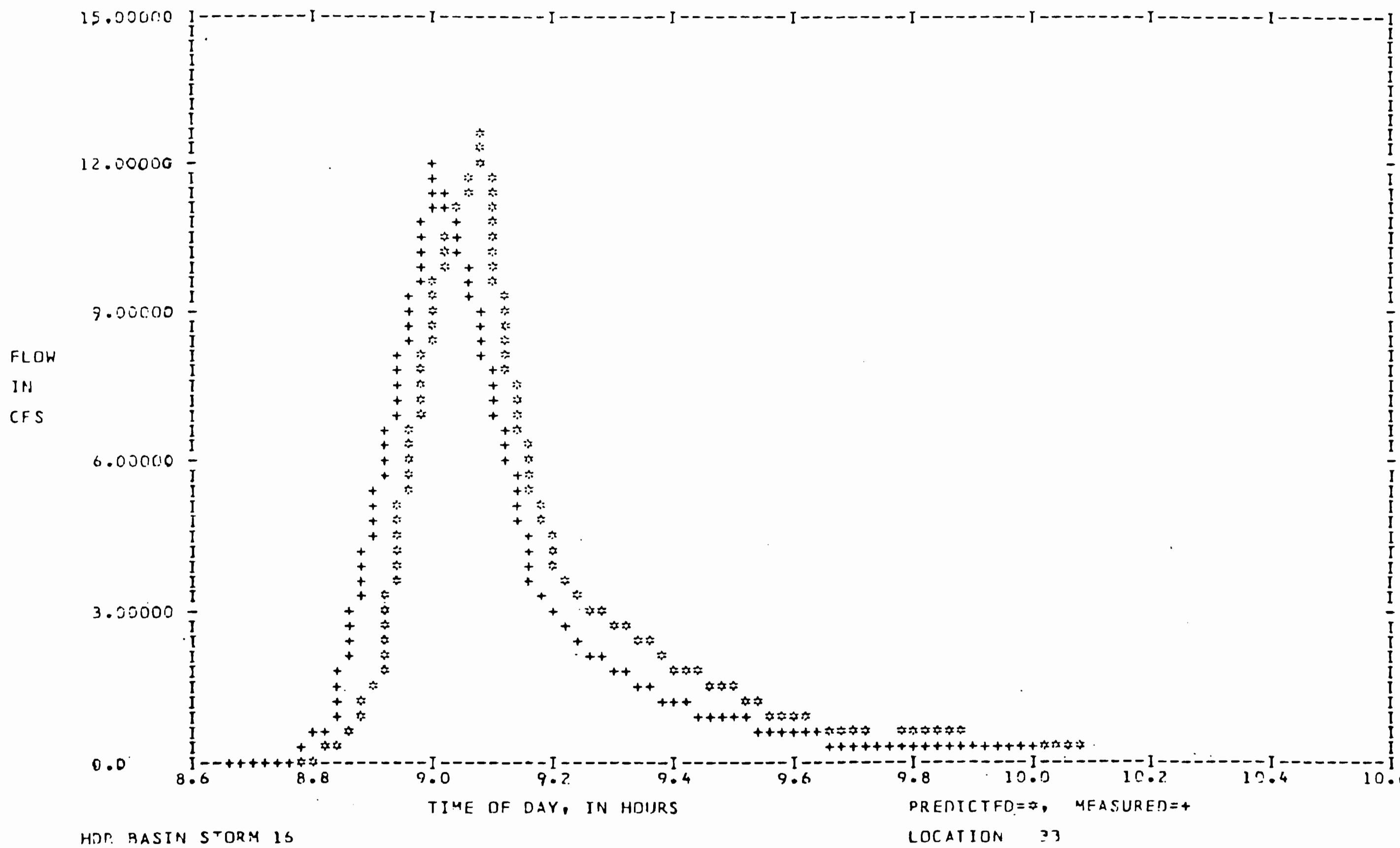


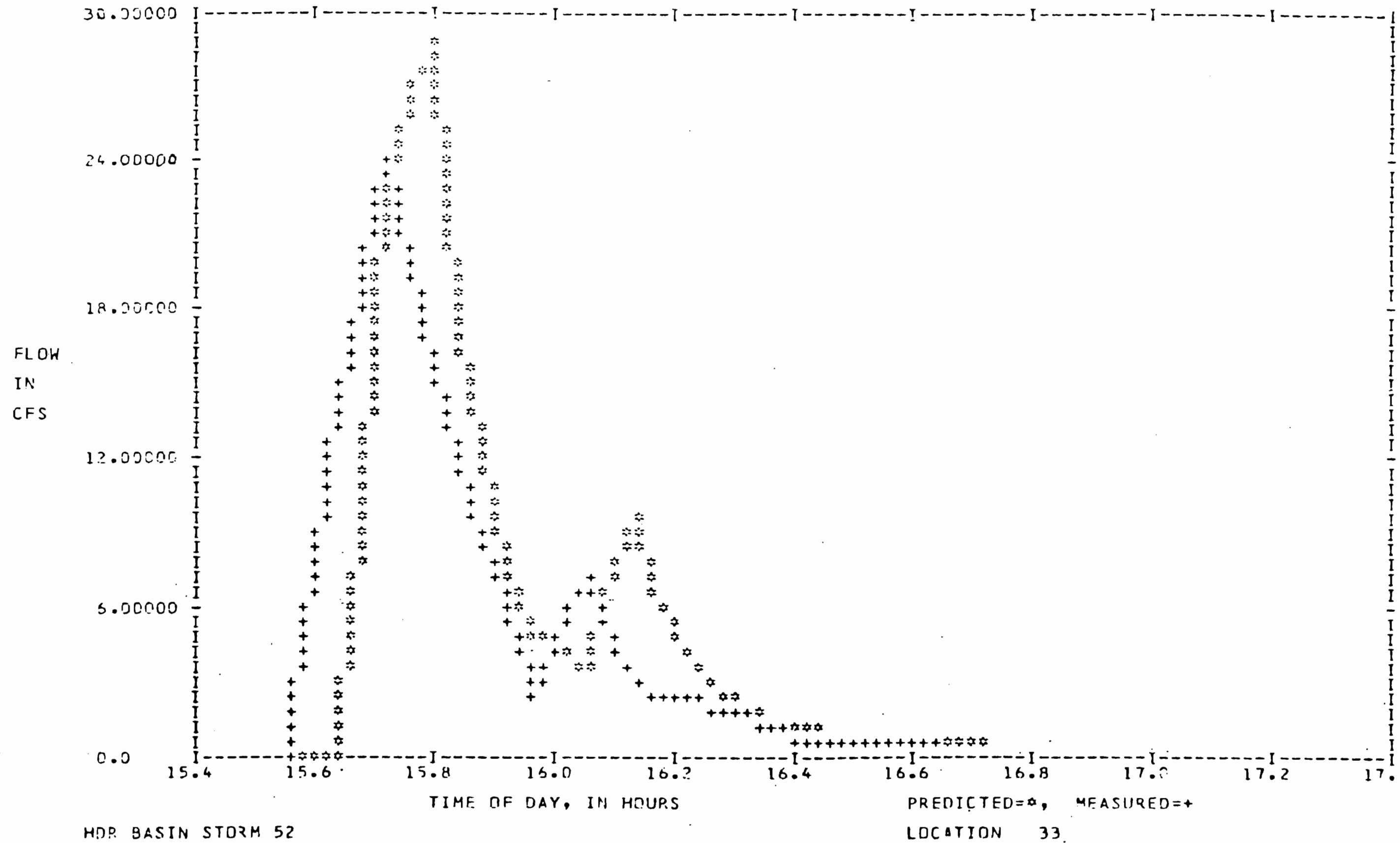
Figure 42. Total Flow Results from the Calibration and Verification Runs--HDR Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME,HR	FLOW,CFS	START,HR	END,HR	LENGTH,HR	POINTS
PREDICTED, TOTAL TIME	0.12277E+05	0.230	9.083	12.476	8.667	10.083	1.417	35
MEASURED, TOTAL TIME	0.11608E+05	0.218	9.000	11.910	8.667	10.000	1.333	17

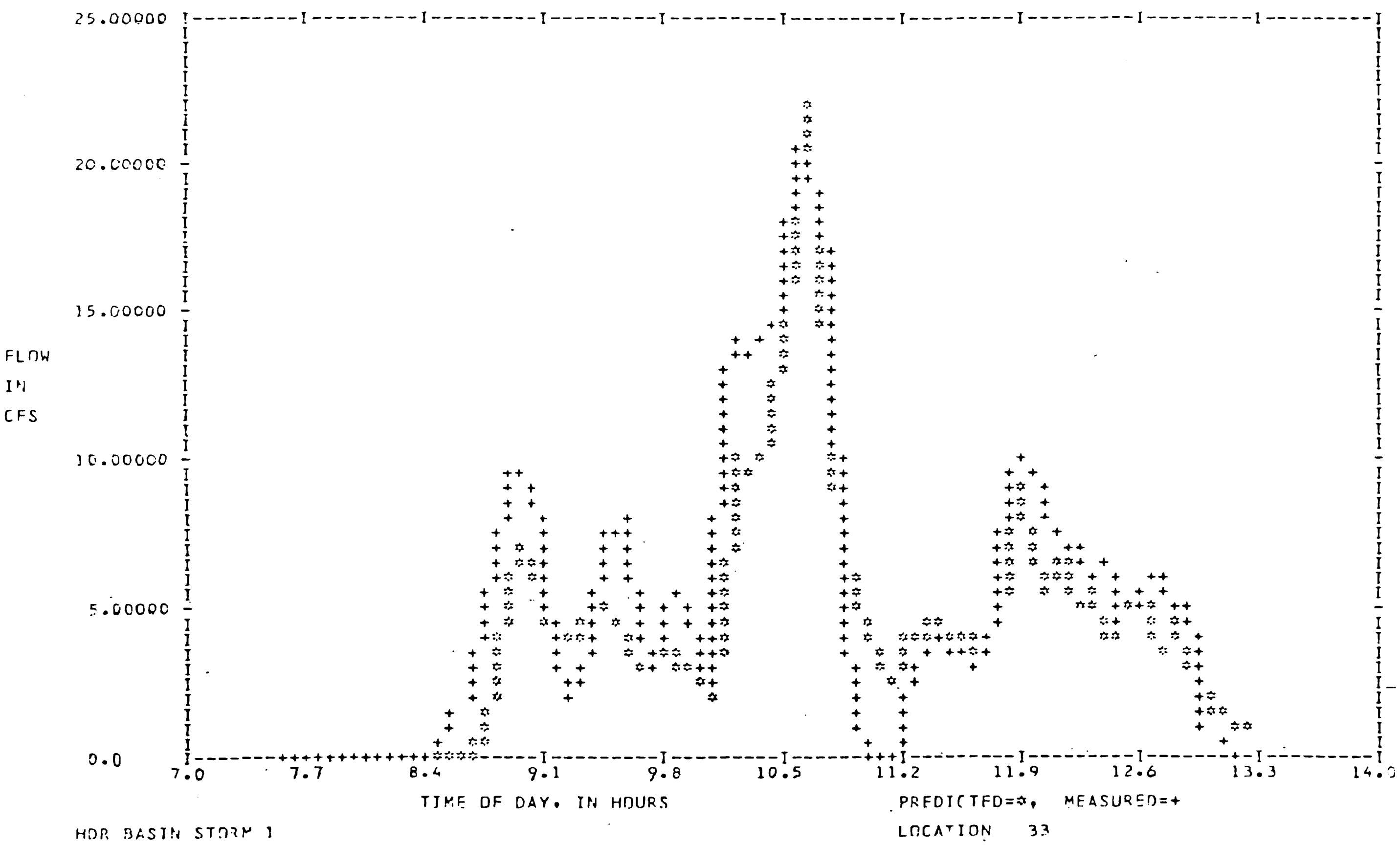
Figure 43. Predicted and Measured Hydrographs from the Best-Fitting Calibration Run on the HDR Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME,HR	FLOW,CFS	START,HR	END,HR	LENGTH,HR	POINTS
PREDICTED, TOTAL TIME	0.26054E+05	0.488	15.800	28.560	15.550	16.717	1.167	29
MEASURED, TOTAL TIME	0.23842E+05	0.447	15.717	24.030	15.550	16.633	1.083	14

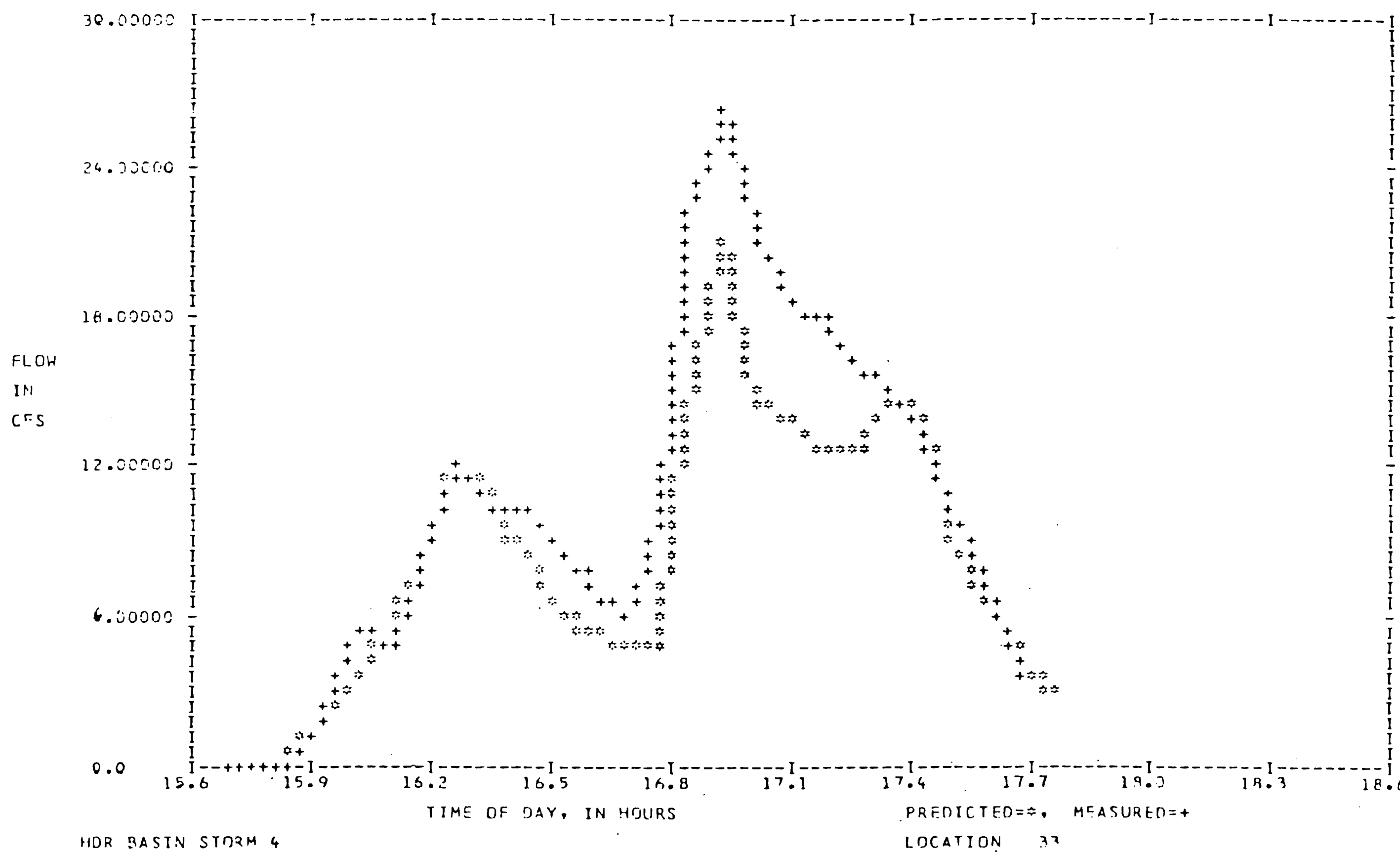
Figure 44. Predicted and Measured Hydrographs from the Worst-Fitting Calibration Run on the HDR Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME,HR	PEAK FLOW FLOW,CFS	DURATION START,HR	DURATION END,HR	DURATION LENGTH,HR	NO. POINTS
PREDICTED, TOTAL TIME	0.89497E+05	1.677	10.650	21.851	7.567	13.233	5.667	137
MEASURED, TOTAL TIME	0.10699E+06	2.005	10.567	20.680	7.567	13.150	5.583	68

Figure 45. Predicted and Measured Hydrographs from the Best-Fitting Verification Run on the HDR Basin.



HYDROGRAPH STATISTICS FOR LOCATION 33

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME,HR	PEAK FLOW FLOW,CFS	DURATION START,HR	DURATION END,HR	DURATION LENGTH,HR	NO. POINTS
PREDICTED, TOTAL TIME	0.63247E+05	1.185	16.933	21.128	15.683	17.767	2.083	51
MEASURED, TOTAL TIME	0.75814E+05	1.421	16.933	26.470	15.683	17.683	2.000	25

Figure 46. Predicted and Measured Hydrographs from the Worst-Fitting Verification Run on the HDR Basin.

The same rainfall record used for the continuous simulation of the COM basin was used for the HDR basin. An MIT of 23 hours was used to delineate events, yielding 2129 independent storm events. Twelve of the 20 highest ranking storms based on both peak flow and total flow from continuous simulation were run in single-event mode with a 5-min time step and several days of antecedent rainfall. The return periods of the 12 events based on peak flow and the 12 events based on total were rearranged according to the ranking from the single-event runs. Tables 37 and 38 list the results of the design storms based on peak flow and total flow, respectively.

Table 37. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Peak Flow--HDR Basin.

Return Period yr	Date	Maximum Intensity in./hr	Runoff Duration hr	Runoff Volume in.	Peak Flow cfs
1.96	5/31/77	1.65	117	6.67	21
2.10	8/12/74	2.60	21	2.10	27
2.44	9/06/60	2.25	30	2.23	27
2.93	7/02/52	2.33	60	3.45	28
3.67	5/19/68	2.62	182	5.63	31
4.19	4/29/57	2.95	65	5.73	38
4.89	5/04/77	2.79	38	9.31	38
5.87	9/03/59	3.29	28	2.90	40
7.33	6/16/59	3.14	133	8.16	40
9.78	6/03/74	3.33	52	3.70	43
14.67	8/26/64	3.65	27	5.33	50
29.33	4/24/79	4.51	28	13.60	86

Table 38. Results from Single Event Runs of Design Storms Selected by Continuous Simulation and Based on Total Flow--HDR Basin.

Return Period yr	Date	Runoff Duration hr	Runoff Volume in.
1.96	10/13/65	54	4.75
2.10	9/08/60	42	4.76
2.44	6/11/67	84	5.52
2.93	9/30/52	122	5.63
3.67	5/22/58	52	5.71
4.19	4/29/57	65	5.73
4.89	11/18/59	77	5.77
5.87	10/24/52	85	5.91
7.33	5/31/77	117	6.67
9.78	6/16/59	133	8.16
14.67	5/04/77	38	9.31
29.33	4/24/79	28	13.60

Summary of Results for the HDR Basin

The results of the seven methodologies used to determine peak flows for the HDR basin are listed in Table 39 and shown in Figure 47. The PDM and the SUH/SCS gave the highest peak flows, followed by the SUH/CH and the THM. The CM/SCS and CS produced the next highest peak flows, with the CS results being larger only at the 25 year return period. Once again, the value of the largest return period for the CS method seems to be somewhat high when compared to the rest of the values found by that method. The RM, the CM/CH, and the RE produced the lowest peak flows of all the methods.

Table 40 and Figure 48 show the results from the methods that calculate a total flow. The SCS methods and CS produced the largest volumes for all return periods, followed by the CM/SCS. The CM/CH gave much smaller volumes than any of the other methods.

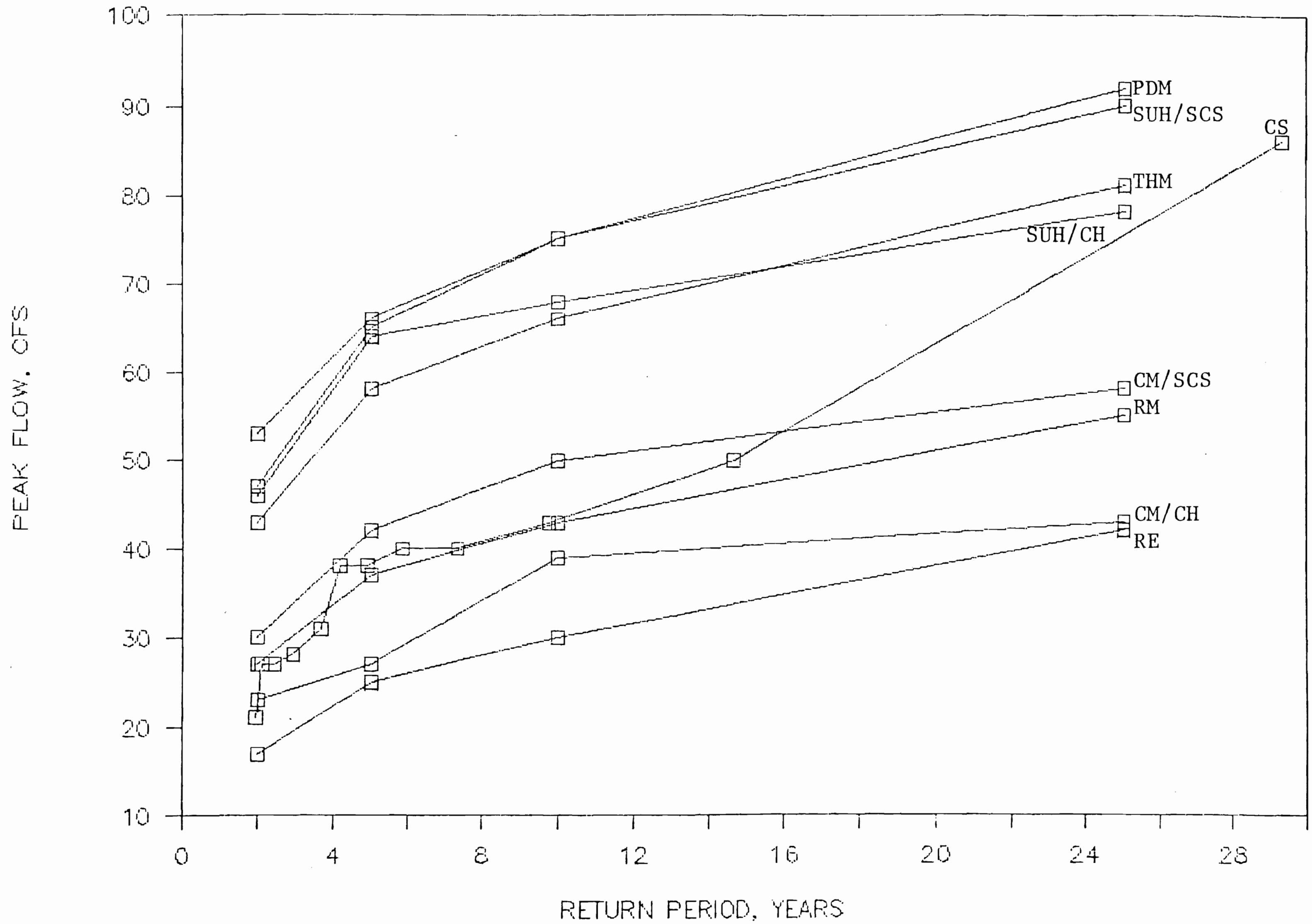


Figure 47. Peak Flow Results for the HDR Basin.

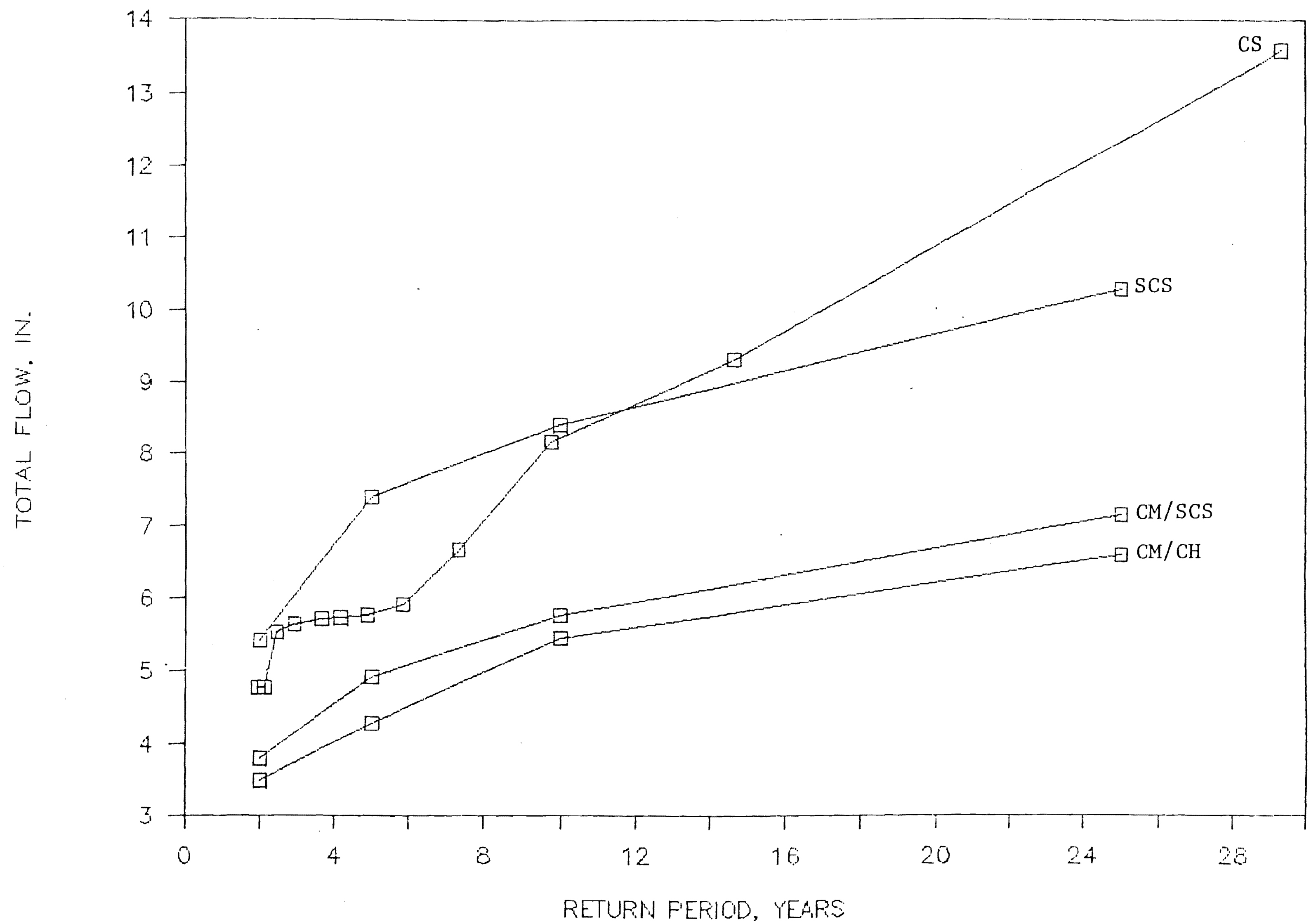


Figure 48. Total Flow Results for the HDR Basin.

Table 39. Results of Peak Flow from Eight Methodologies--HDR Basin.

Return Period yr	CS	PDM	THM	RM	SUH/CH	SUH/SCS	CM/CH	CM/SCS	RE
		Peak Flow, cfs							
1.96	21								
2		47	43	27	46	53	23	30	17
2.10	27								
2.44	27								
2.93	28								
3.67	31								
4.19	38								
4.89	38								
5		65	58	37	64	66	27	42	25
5.87	40								
7.33	40								
9.78	43								
10		75	66	43	68	75	39	50	30
14.67	50								
25		92	81	55	78	90	43	58	42
29.33	86								

Table 40. Results of Total Flow from Various Methodologies--HDR Basin.

Return Period yr	CS	SCS	CM/CH	CM/SCS
	Total Flow, in.			
1.96	4.75			
2		5.41	3.49	3.79
2.10	4.76			
2.44	5.52			
2.93	5.63			
3.67	5.71			
4.19	5.73			
4.87	5.77			
5		7.40	4.27	4.91
5.87	5.91			
7.33	6.67			
9.78	8.16			
10		8.40	5.44	5.77
14.67	9.31			
25		10.29	6.60	7.15
29.33	13.60			

Discussion of Results from Case Studies

The CS results are used as a reference to which the other methodologies are compared in this section, but this is not meant to imply that the CS results are the most accurate for all of the areas. If short-time-increment rainfall data had been used for all of the areas, the CS results could have probably been considered the most accurate results. However, since hourly rainfall was used, the results from the methods using hourly rainfall for the basins with relatively short times of concentration are probably much lower than they should be. Also, due to the fact that the true return period of the largest one or two values found by CS for some of the areas appears to be somewhat different than the return period found by the Weibull plotting formula, results past the 10-year return period are not given as much weight as those at or below the 10-year return period.

As discussed earlier, the synthetic design storms could have been used with a short hyetograph time interval, but the use of any interval less than 30 minutes (which would be necessary for several of the study areas) may be questionable. Short-time-increment rainfall data can be obtained for historical storms, albeit this takes a considerable amount of time and patience, by obtaining the strip charts of the storms from the National Weather Service and discretizing the storms into the desired intervals. This process was done for the MEG Basin in an unpublished study. The resulting peak flow from the 15-minute-interval historical storms were generally 25 percent higher than the hourly storms, but the ranking of the storms stayed approximately the same. Higher peak flows would have resulted for the basins in south

Florida if this same process had been applied to them. However, the ranking of the storms may have changed considerably because of the sensitivity of these small basins to short-time-increment rainfall.

All of the conventional methodologies produced widely varying peak flows when compared to CS. Table 41 contains the standard deviations of the peak flows at the 10-year return period (the RE results were omitted from the calculations). Table 42, which also compares peak flow results, was composed by calculating the percent difference between the CS values and the values from the conventional methodologies at the 5- and 10-year return periods, finding the average of those two differences, and counting the number of occurrences for each of the five categories in Table 42. (The 5- and 10-year return period values for CS were estimated by linear interpolation). Note that the use of hourly rainfall for the SUH/CH and SUH/SCS was probably inadequate. If a 10-minute rainfall interval had been used, the results from this method would have probably much more consistent with the other methods.

Table 41. Standard Deviations of Peak Flows at the 10-Year Return Period.

	Basin				
	MEG	LDR	HWY	COM	HDR
Std. Dev. (cfs)	617.9	23.1	32.8	45.9	14.6

Although CS is arbitrarily used as a reference for comparison for all of the basins, the results from the rational method may represent the most accurate results from COM basins for two reasons: 1) the basis for the rational method is especially valid on small, impervious ares, and 2) the rational method is the only method that explicitly used short-time-increment rainfall. Taking into account the fact that the RM generally used a much shorter rainfall interval than CS, the results from the RM for the rest of the basins may be considered low. Similarly, the results from the PDM generally tend to be low, with the exception of the results from the HDR basin. The results from the THM tend to be very high, but they may have appeared to be more reasonable if short-time-increment rainfall data had been used for all of the basins. The results from the RE appear to be consistently low. It is difficult to generalize on the results from the CM/CH and CM/SCS because of the use of hourly rainfall.

Table 42. Comparison of Peak Flows from Conventional Methodologies to Peak Flows from CS.

	Number of Occurences Based on Avg. Percent Difference from 5- and 10-Year Return Period CS Values				
Method	Less Than -25%	Between -25% and -10%	Within +or-10%	Between 25% and 10%	Greater Than 25%
RM	1	2	1		1
PDM	2		2		1
THM				1	4
SUH/CH	4				1
SUH/SCS	4				1
CM/CH		4	1		
CM/SCS			1	3	1
RE	4		1		

The conventional methodologies that calculate a total flow also gave highly variable total flow results when compared to CS. Table 43 contains the standard deviations of the total flows at the 10-year return period. Table 44, which also compares total flow results, was composed in the same manner as Table 42. Note that the CS storm volumes are generally based on rainfall of a much longer duration (a maximum of 220 hours for the 2.93 return period storm for the COM basin) than that used for the conventional methodologies (24 hours). Although these durations may seem somewhat large, a detention basin would "see" the storm lasting that long because during the course of the storm the basin would continue to drain and fill without ever completely draining. Also note that the CM/SCS and the SCS give very different total flow results. This difference is due to two reasons: 1) the storm depths for the CM/SCS came from a regional IDF curve, whereas the storm depths for the SCS came from a rainfall atlas (Hershfield, 1961), and 2) the losses for the CM/SCS were calculated by the calibrated SWMM model, whereas the losses for the SCS were calculated by the curve number equation.

Table 43. Standard Deviations of Total Flows at the 10-Year Return Period.

	Basin				
	MEG	LDR	HWY	COM	HDR
Std. Dev. (in.)	.647	.765	.147	1.365	1.357

Table 44. Comparison of Total Flows from Conventional Methodologies to Total Flows from CS.

	Number of Occurences Based on Avg. Percent Difference from 5- and 10-Year Return Period CS Values				
Method	Less Than -25%	Between -25% and -10%	Within +or-10%	Between 25% and 10%	Greater Than 25%
CM/CH		4	1		
CM/SCS			1	3	1
SCS	4		1		

There are several other interesting contrasts between the different methodologies that are worth noting. One has to do with a comparison of synthetic and historical design storm characteristics. The second deals with a comparison of hydrographs from several of the methodologies. The third deals with the problem of using the frequency analysis of one parameter to determine the return period of another parameter.

Synthetic design storms were used with or are implicit in several of the methodologies used in this thesis. The durations for the synthetic design storms were either given or assumed to be 24 hours. An examination of Tables 16 and 17 and similar tables shows that the duration for real storms varies tremendously and is often much greater than 24 hours. (Rainfall durations are within a couple of hours of the runoff durations listed in Tables 16 and 17 and similar tables). Another discrepancy between synthetic and historical storms is hyetograph shape. Figures 49 and 50 show the 5-year return period Chicago and SCS design storm hyetographs for the MEG basin. Figures 51, 52, 53, and 54 show the hyetographs from the corresponding historical storms based on peak flow and total flow. Clearly, there is little similarity between the shape of the historical and synthetic storm hyetographs.

As can be expected from a comparison of the hyetographs, the hydrographs from the conventional methodologies bear little resemblance to the hydrographs from historical design storms. Figures 55, 56, 57, and 58 display hydrographs from the conventional methodologies for the HDR basin for the 5-year return period. Figures 59, 60, 61, and 62 display the corresponding hydrographs from the historical

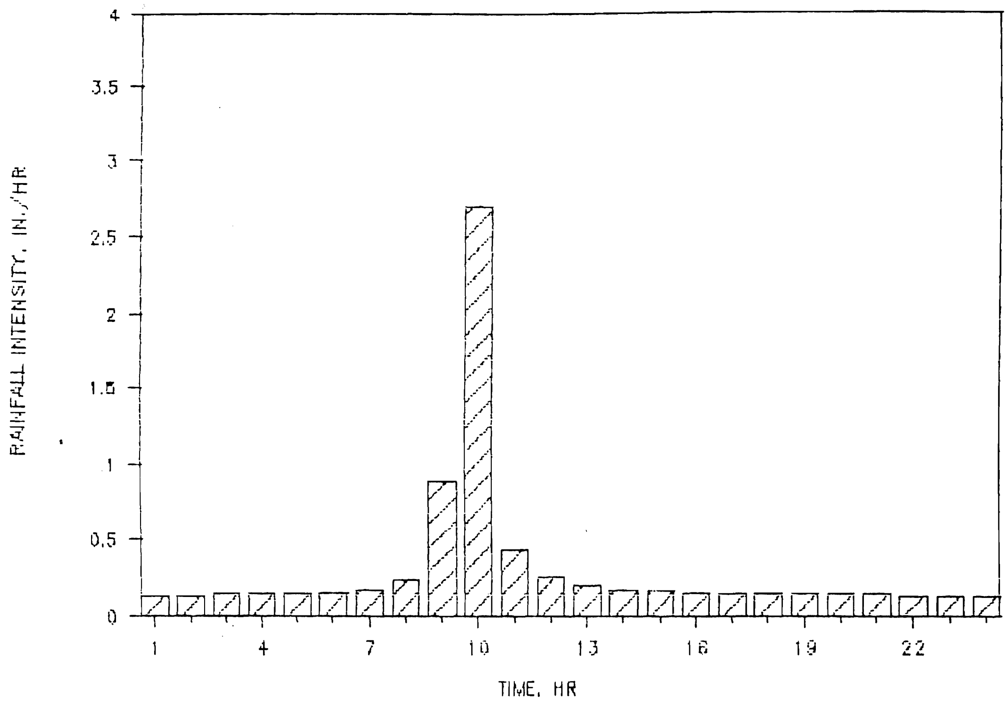


Figure 49. Five-Year Return Period Chicago Design Storm for the MEG Basin.

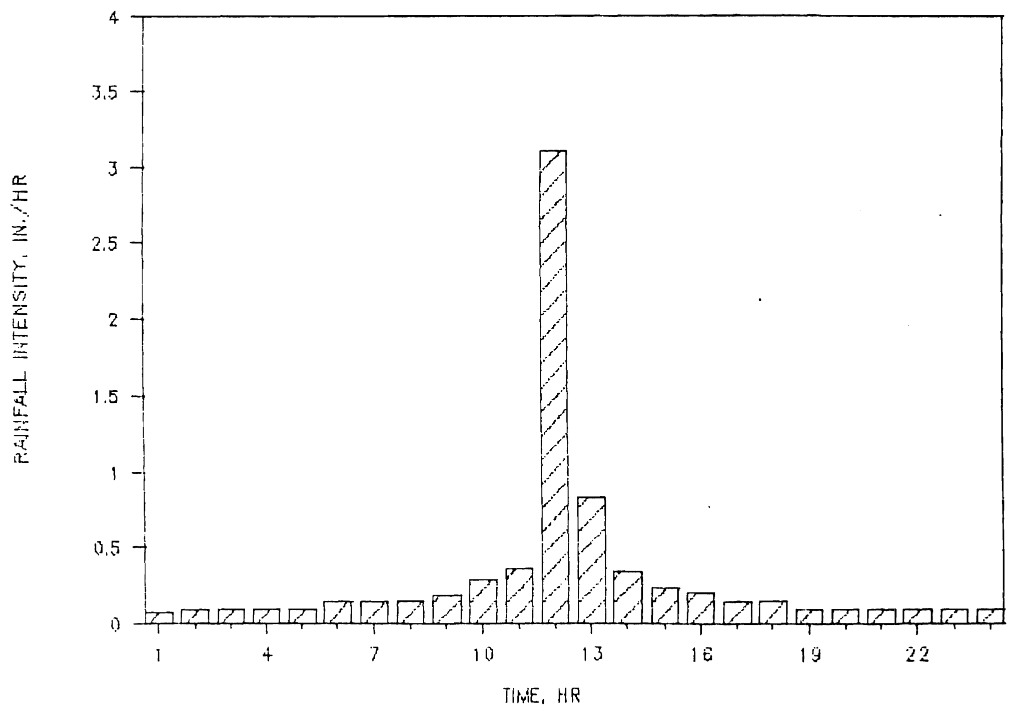


Figure 50. Five-Year Return Period 24-Hour SCS Type II Design Storm for the MEG Basin.

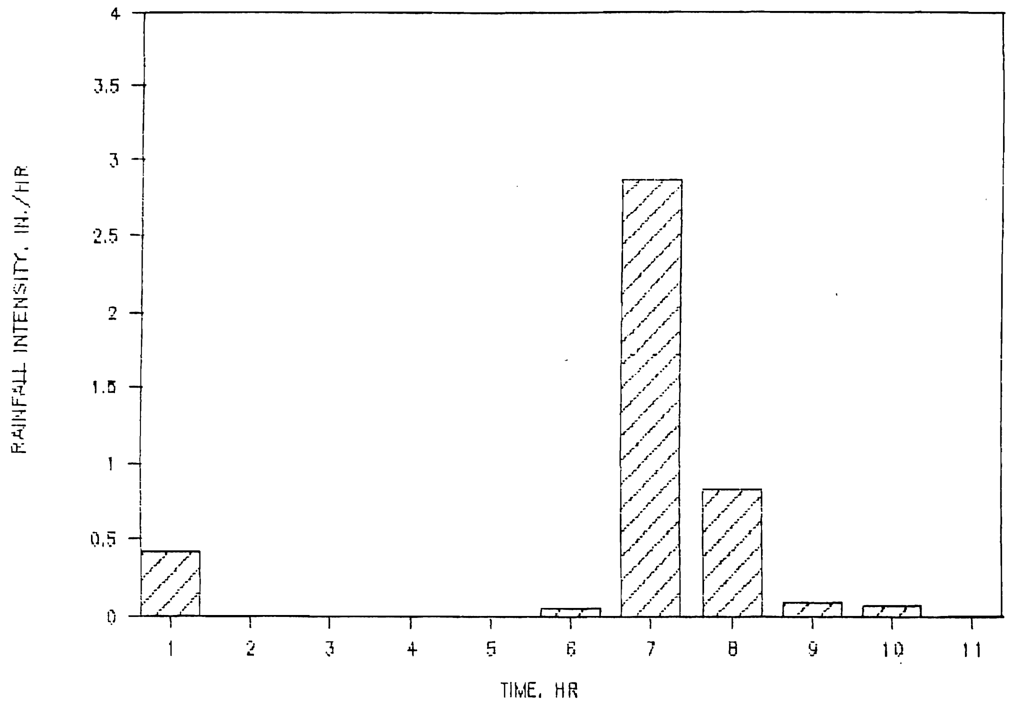


Figure 51. Historical Design Storm with a 4.33-Year Return Period Based on Peak Flow for the MEG Basin.

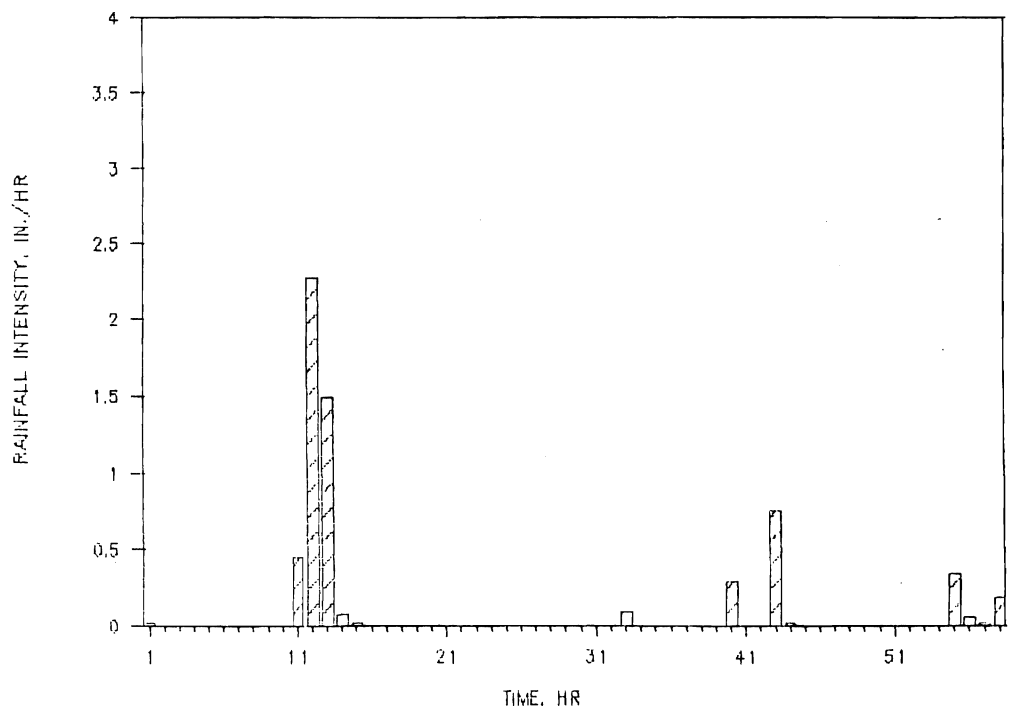


Figure 52. Historical Design Storm with a 5.42-Year Return Period Based on Peak Flow for the MEG Basin.

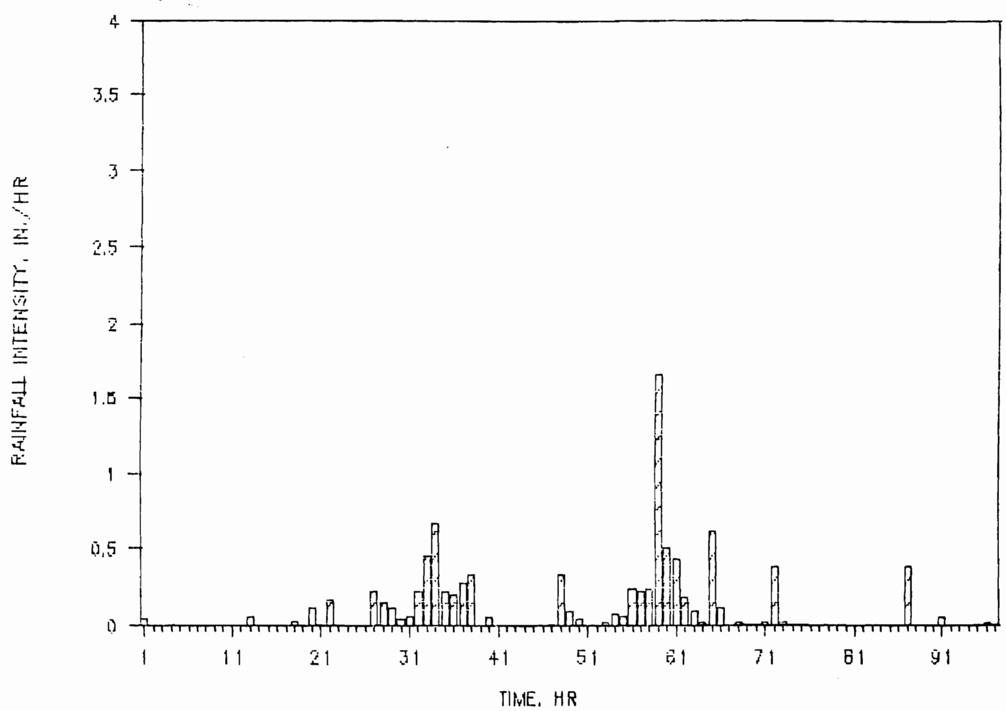


Figure 53. Historical Design Storm with a 4.33-Year Return Period Based on Total Flow for the MEG Basin.

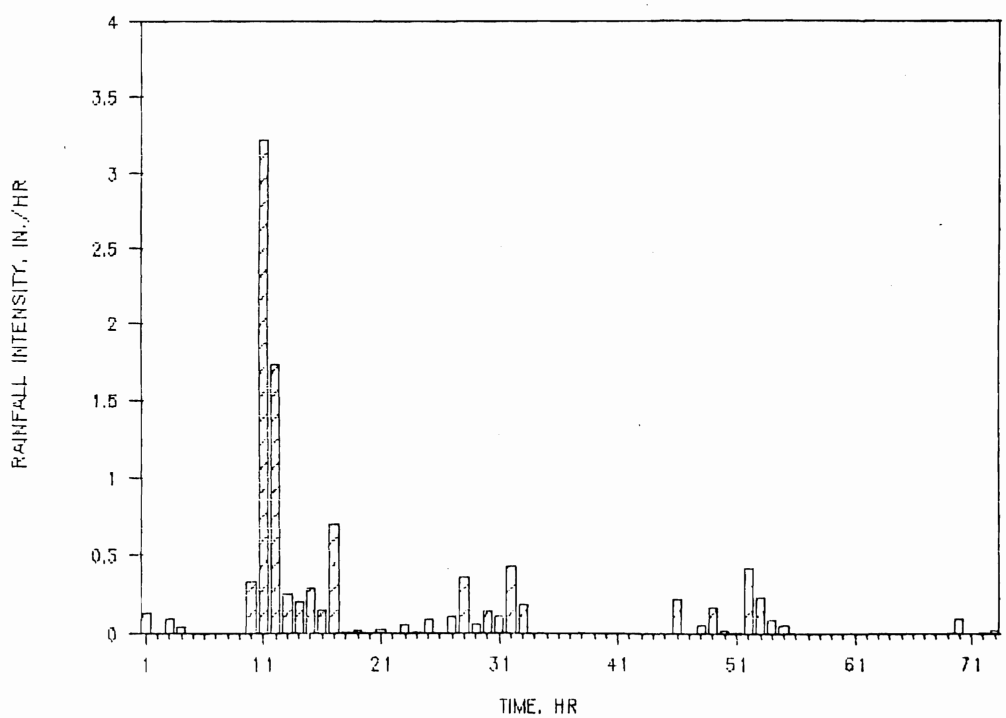


Figure 54. Historical Design Storm with a 5.42-Year Return Period Based on Total Flow for the MEG Basin.

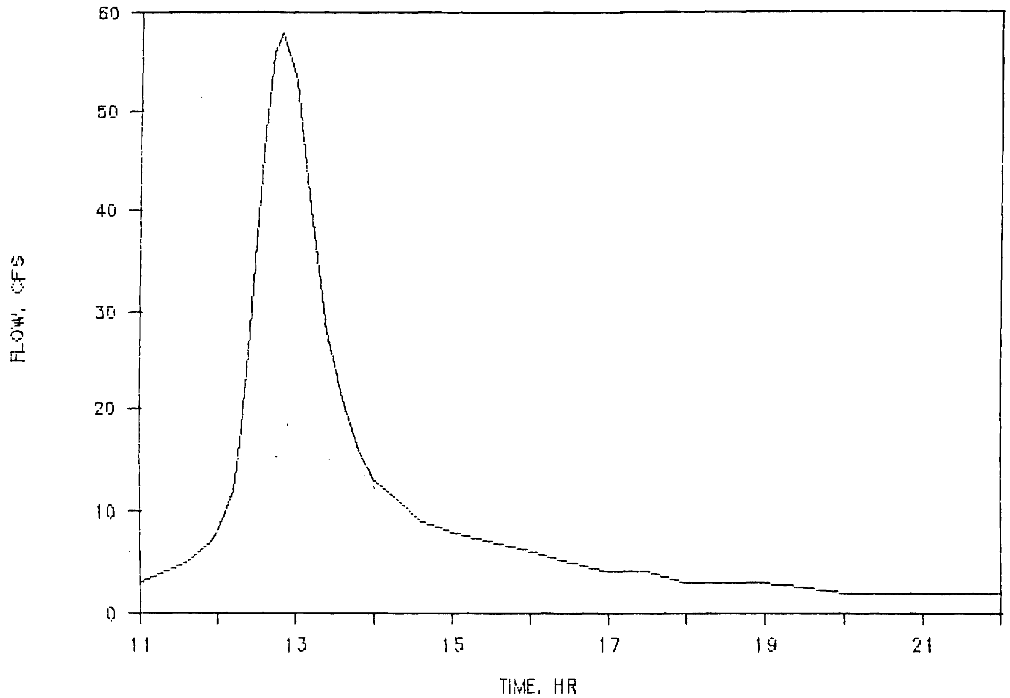


Figure 55. Hydrograph from the THM with a 5-Year Return Period for the HDR Basin.

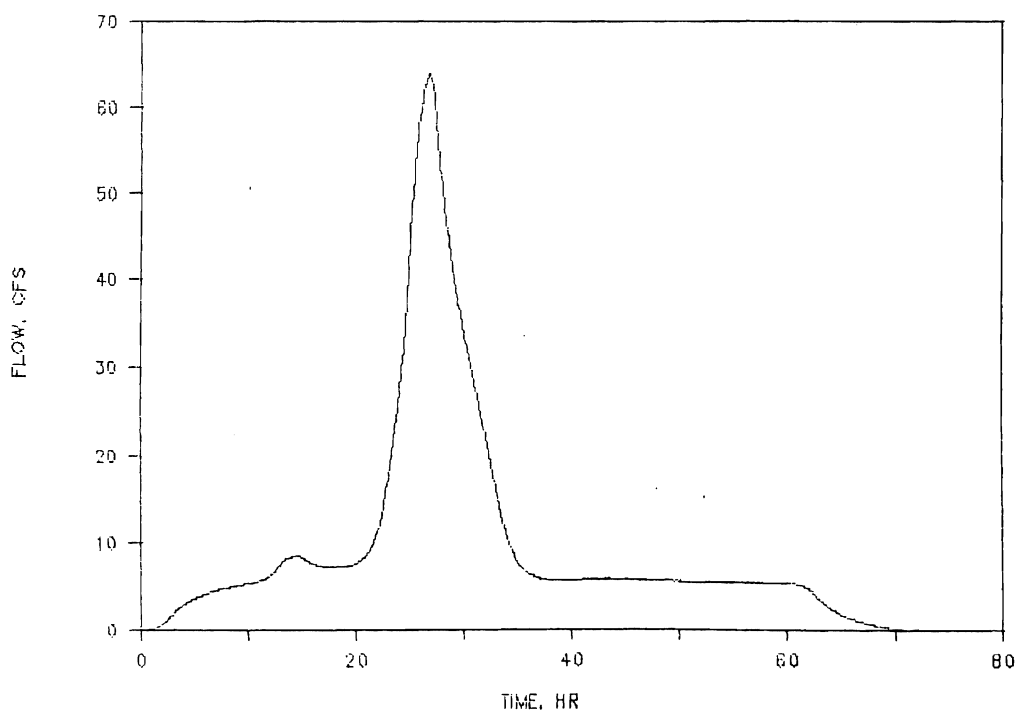


Figure 56. Hydrograph from the SUH/CH with a 5-Year Return Period for the HDR Basin.

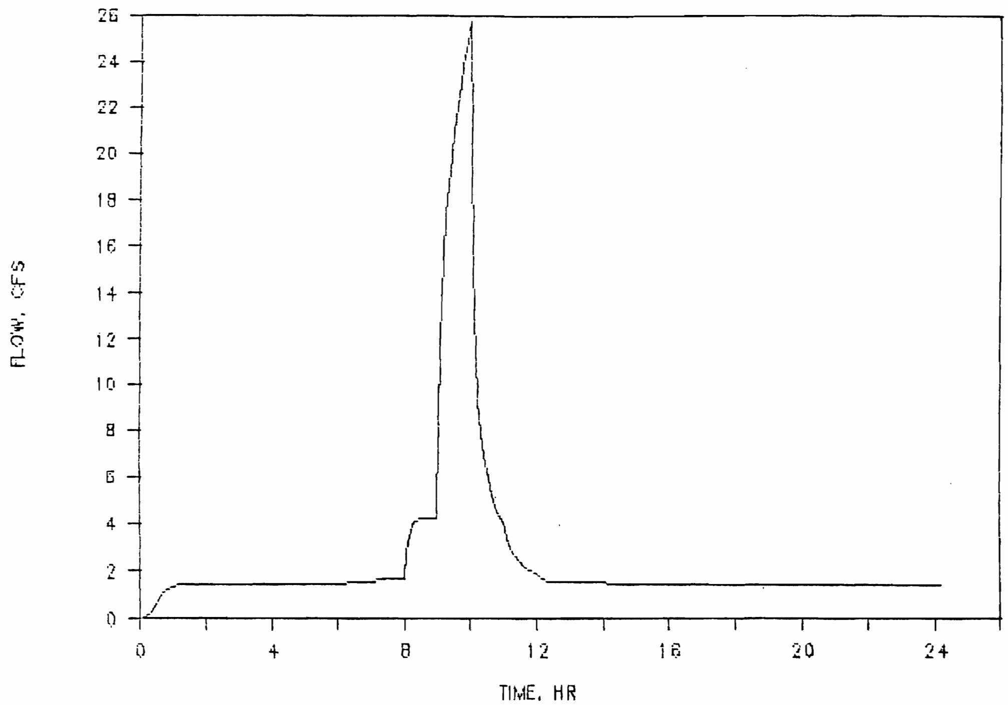


Figure 57. Hydrograph from the CM/CH with a 5-Year Return Period for the HDR Basin.

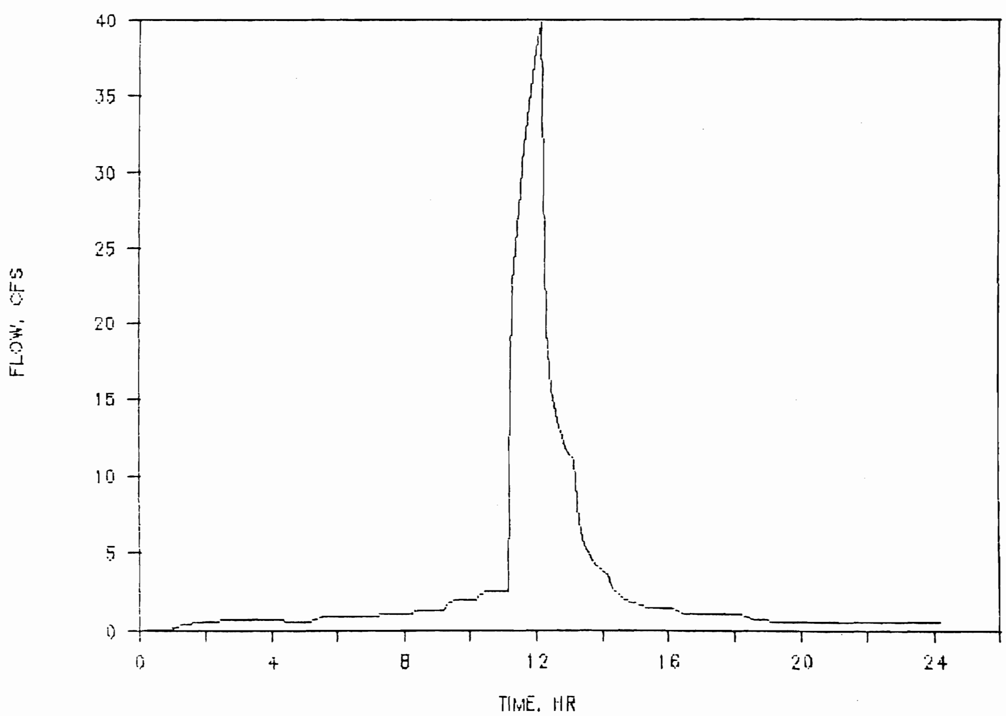


Figure 58. Hydrograph from the CM/SCS with a 5-Year Return Period for the HDR Basin.

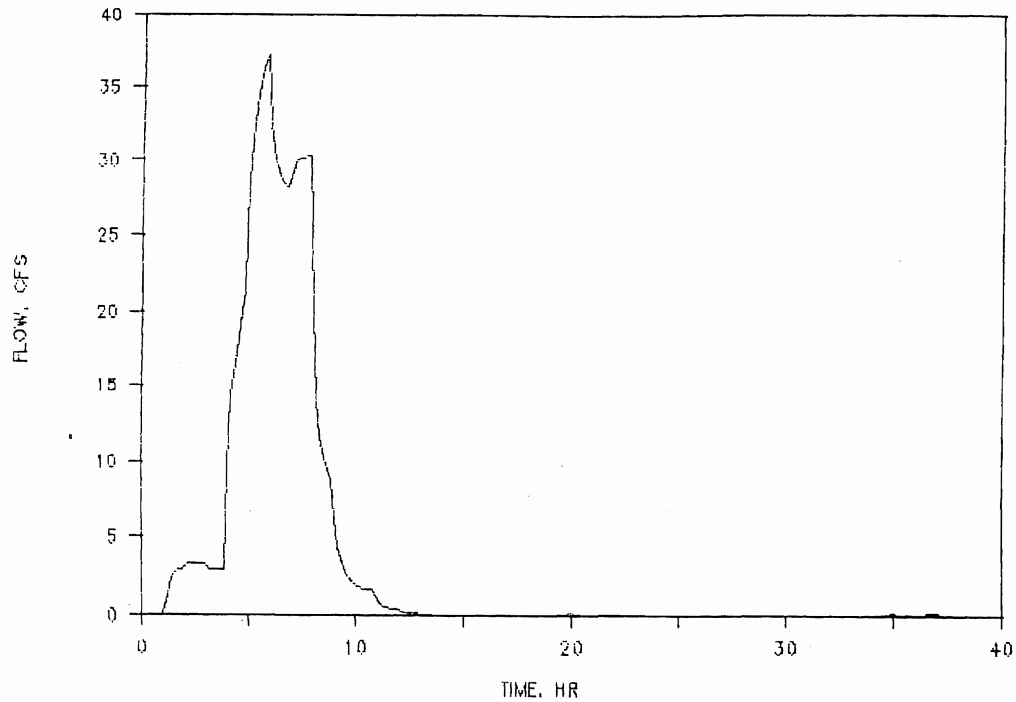


Figure 59. Hydrograph from CS with a 4.89-Year Return Period Based on Peak Flow for the HDR Basin.

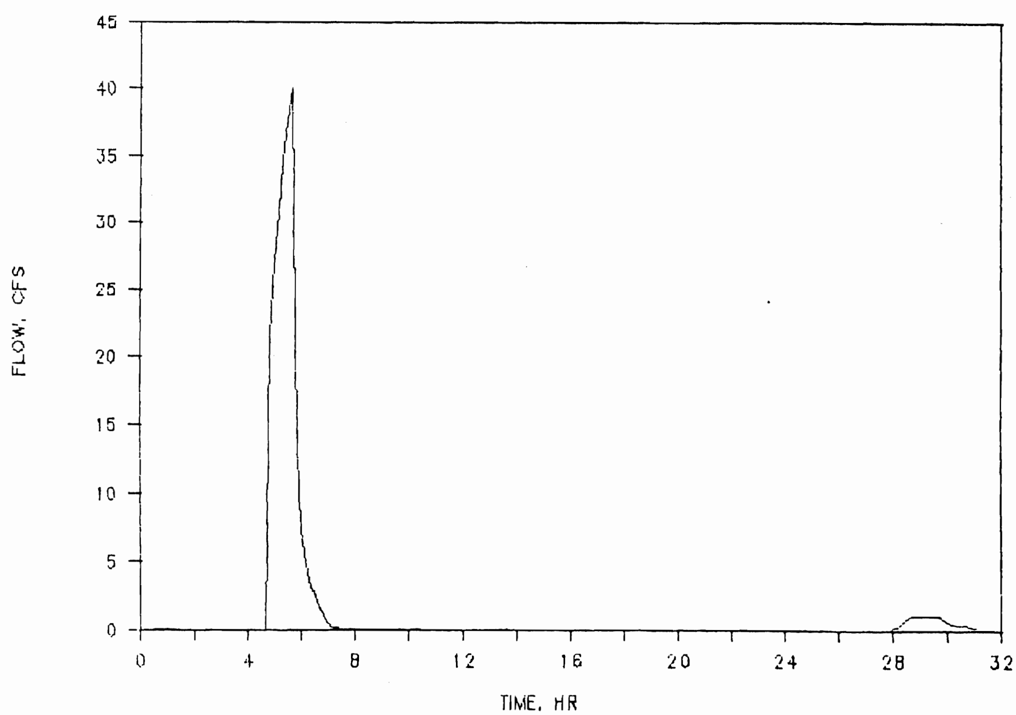


Figure 60. Hydrograph from CS with a 5.87-Year Return Period Based on Total Flow for the HDR Basin.

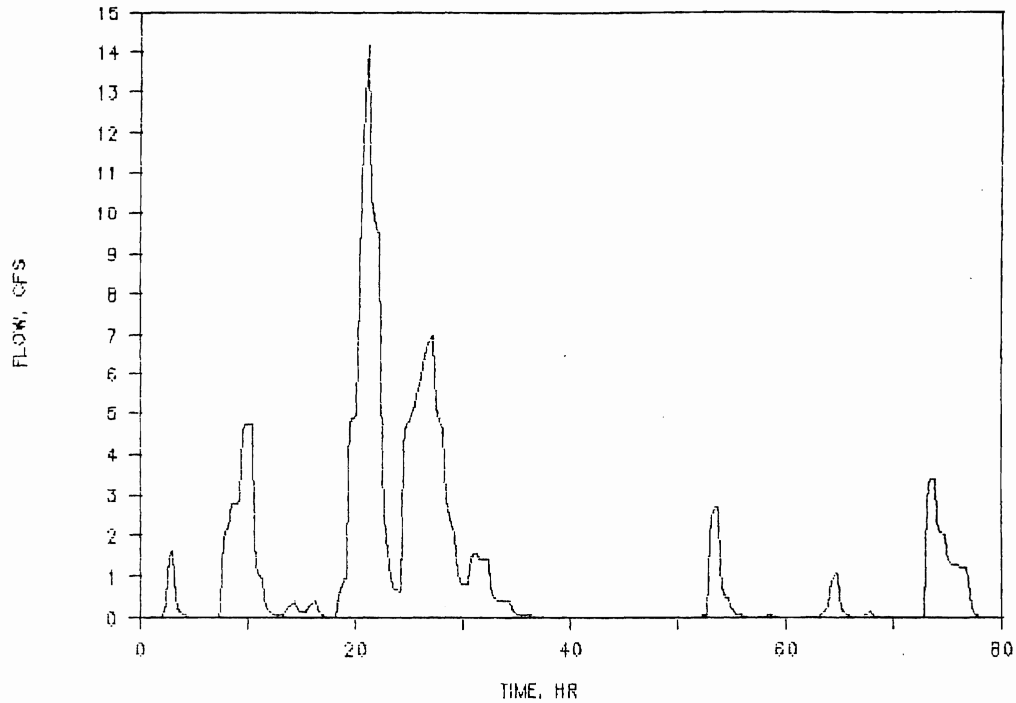


Figure 61. Hydrograph from CS with a 4.89-Year Return Period Based on Total Flow for the HDR Basin.

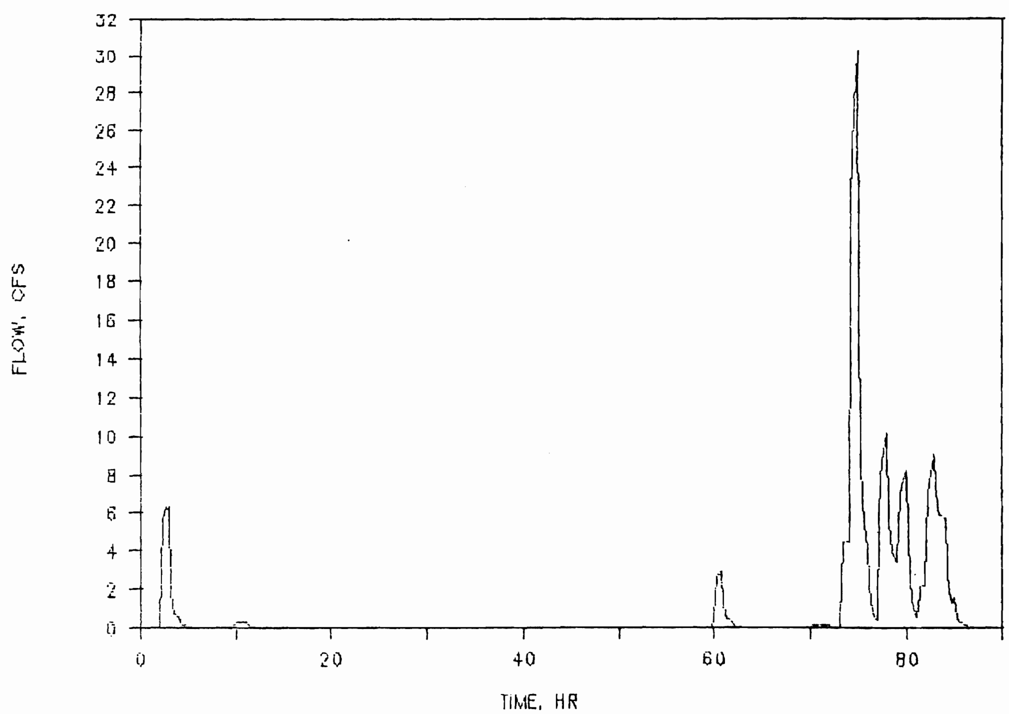


Figure 62. Hydrograph from CS with a 5.87-Year Return Period Based on Total Flow for the HDR Basin.

design storms run through the calibrated SWMM. The basin and return period were chosen at random, but hydrographs are fairly representative of those from other return periods and basins. The multiple peaks present in some of the historical design storm hydrographs are not present in any of the ones from synthetic design storms.

Tables 16 and 23 illustrate the problem with using the frequency analysis of one parameter to determine the return period of another parameter. If the storms in Tables 16 and 23 had been ranked by maximum intensity (which is what is done, in one form or another, for most conventional methodologies), total rainfall volume, or total runoff volume, the return periods would be very different.

CHAPTER 4

COST IMPLICATIONS

It is clear from reading Chapters 2 and 3 that there is a substantial difference in the time required to gather the data and perform the analysis for each of the methodologies. This difference, of course, translates into a difference in design cost. It is also clear from Chapter 3 that the various methodologies can produce widely varying results for both peak flow and total volume. These variations translate into large differences in construction costs and levels of safety. This chapter attempts to quantify the differences in both design and construction costs between the different methodologies. The regression equation methodology is excluded from the analysis in this chapter because its inclusion was simply for comparison purposes.

Design Costs

The design costs are broken into two categories: 1) data collection and parameter estimation, and 2) actual analysis using the various methodologies. The costs for both categories are expressed in units of man-hours since the time required for each methodology is the predominate design cost and the amount charged for an engineer's time varies. The values for each methodology assume that the study area is not broken up into sub-areas and the parameter of interest at the downstream end of the area is the only value desired.

Data Collection and Parameter Estimation

The number of man-hours involved for data collection and parameter estimation was determined by breaking each methodology down into tasks and assigning either an estimate or an actual value of the time required for that task; estimates had to be made for tasks that had been performed for another study by another person. All of task times are based on the assumption that the user is experienced with the methodology. Table 45 contains tasks that must be performed for every methodology. The remainder of the tables in this section contain tasks that are not universal to all of the methodologies.

Table 45. Cost of Data Collection and Parameter Estimation Required for all of the Methodologies.

Task	Time Required, man-hours
Obtain contour map of the area	.75
Obtain soils map of the area	.5
Determine basin boundary and size	2.5
Determine imperviousness and hydraulically effective imperviousness	4.0
Total	7.75

Rational method

The time required to collect the data and estimate the parameters for the rational method is given in Table 46. The total time was calculated by adding total time in Table 45 to the sum of the times in Table 46.

Table 46. Cost of Data Collection and Parameter Estimation for the Rational Method.

Task	Time Required, man-hours
Obtain IDF curves for the area	--
Determine average surface cover	.25
Determine soil characteristics	.5
Find the most remote point for time of concentration calculations and determine the characteristics of its flow path to the outlet	1.5
Determine runoff coefficient as a function of watershed characteristics and return period	.5
Total	10.5

Synthetic unit hydrograph method

The number of man-hours needed for data collection and parameter estimation is shown in Table 47, which is based on the assumption that a synthetic design storm is used for rainfall. The total in Table 47 includes the total in Table 45.

Table 47. Cost of Data Collection and Parameter Estimation for the Synthetic Unit Hydrograph Method.

Task	Time Required, man-hours
Determine characteristics of conveyance system and estimate conveyance efficiency, main channel slope and length	1.0
Determine rainfall and rainfall excess	.5
Total	9.25

SCS peak discharge method

Table 48 gives the time required for data collection and parameter estimation for the PDM. The total in Table 48 includes the total time in Table 45.

Table 48. Cost of Data Collection and Parameter Estimation for the SCS Peak Discharge Method.

Task	Time Required, man-hours
Obtain maps with design rainfall for area	--
Determine hydraulic length	.75
Determine average slope	.5
Determine hydrologic soil group	.75
Determine curve number	1.0
Total	10.75

SCS tabular hydrograph method

The amount of time needed for data collection and parameter estimation for the THM is shown in Table 49, which includes the total time in Table 45.

Table 49. Cost of Data Collection and Parameter Estimation for the SCS Tabular Hydrograph Method.

Task	Time Required, man-hours
Same as Table 48 plus,	
Determine characteristics of flow path having largest time of concentration	.75
Total	11.5

Calibrated model with synthetic design storm

Table 50 gives the time required for data collection and parameter estimation for the CM/CH and CM/SCS. Implicit in the tasks and task times is that SWMM is used. The total time in Table 50 includes the total time from Table 45.

Table 50. Cost of Data Collection and Parameter Estimation for the Calibrated Model with Synthetic Design Storms.

Task	Time Required, man-hours
Obtain rainfall-runoff data and select calibration and verification events	.5[a]
Obtain IDF curves for the area	--
Determine average slope	.5
Determine average surface cover for pervious and impervious areas and select appropriate Manning's n values	.75
Determine depression storage for impervious and pervious areas	.5
Estimate subcatchment width parameter	.5
Determine infiltration parameters	3.0
Determine channel-routing parameters (No channels used for any of the areas)	--
Determine evaporation values	.5
Determine rainfall hyetograph	.25
Determine miscellaneous SWMM parameters	.25
Total	14.5

[a]This estimate is based on the data having already been collected. If cost of collecting the rainfall-runoff data is included, then this value is much greater. It was not included in this thesis since the data already existed for all of the sites.

Design storms from continuous simulation

Table 51 lists the time needed for data collection and parameter estimation for CS. Tasks and task times are given under the assumption that SWMM is the model used for the methodology. The total time in Table 51 includes the total time in Table 45.

Table 51. Cost of Data Collection and Parameter Estimation for Design Storms from Continuous Simulation.

Task	Time Required, man-hours
Same as Table 50 minus,	
Determine rainfall hyetograph	
plus,	
Obtain long-term precipitation record from station closest to site with adequate data	2.0
Total	16.25

Analysis Costs

As was done in the previous section, the number of man-hours involved with the actual analysis for each methodology was determined by breaking the analyses up into a number of tasks and determining the time required for each task. Also, each estimate of task time is based on the assumption that the user is experienced with the methodology. Estimates of analysis costs only include the time required to determine the parameter of interest for several return periods at the outlet end of the study area; they do not include the time required for the design of structures, e.g., pipe sizing and detention basin sizing. Presumably, the cost of designing structures will be fairly constant for all of the methodologies.

Rational method

Table 52 lists the tasks and task times required to use the RM. Values listed in Table 52 are based on the assumption that the method is performed on a spreadsheet.

Table 52. Cost of Analysis Using the Rational Method.

Task		Time Required, man-hours
Set up pipe/channel information		.5
Set up overland flow information		.25
Input runoff coefficients and miscellaneous information		.1
Perform iterations to find time of concentration and corresponding rainfall intensities		.75
Calculate peak flows		.1
	Total	1.7

Synthetic unit hydrograph method

The analysis time necessary for the synthetic unit hydrograph methodology is given in Table 53. The estimates are based on the assumption that a synthetic design storm is used for rainfall input and a simple method is used to determine rainfall excess.

SCS peak discharge method

Table 54 gives the analysis time required for the PDM. Tasks and task times are based on the assumption that a spreadsheet is used to aid in calculations. If the method is performed by hand the total time will be only slightly greater since few calculations have to made.

Table 53. Cost of Analysis for Synthetic Unit Hydrograph Method.

Task		Time Required, man-hours
Determine the six given points on the hydrograph using the five watershed parameters and accompanying equations		.25
Determine remaining points on hydrograph		1.25
Input design rainfall excess and determine peak flows		.75
	Total	2.25

Table 54. Cost of Analysis for SCS Peak Discharge Method.

Task		Time Required, man-hours
Determine runoff depth		.1
Determine equivalent drainage area, peak discharge rate per inch of runoff, preliminary peak discharge rates, and actual discharge rates		.25
Determine peak flow adjustment factors and final peak flow rates		.5
	Total	.85

SCS tabular hydrograph method

The time needed for analysis using the THM is given in Table 55. It is assumed that the computerized version of the methodology is used for the analysis.

Table 55. Cost of Analysis for SCS Tabular Hydrograph Method.

Task	Time Required, man-hours
Determine rainfall type and input parameters	.15
Calculate time of concentration and peak flows	.5
Total	0.65

Calibrated model with synthetic design storms

Table 56 lists the analysis time required for the CM/CH and the CM/SCS. The tasks and task times are based on the assumption that SWMM is the model used.

Table 56. Cost of Analysis for Calibrated Model with Synthetic Design Storms.

Task	Time Required, man-hours
Set up data file for calibration	1.0
Change parameters for calibration until satisfactory fit is achieved	3.0
Set up data file for verification	1.0
Input design storm hyetographs into a data file	.25
Total	5.25

Design storms from continuous simulation

The analysis time necessary for CS is listed in Table 57. It is assumed that SWMM is the model used for the methodology.

Table 57. Cost of Analysis for Design Storms from Continuous Simulation.

Task	Time Required, man-hours
Same as Table 87 minus,	
Input design storm hyetographs into a data file	
plus,	
Process rainfall data and perform continuous simulation	.5
Run STATS Block of SWMM to determine MIT	.5
Run STATS Block of SWMM to determine historical design storms	.1
Input and run historical design storms in single-event mode with 5-min time step and several days of antecedent rainfall	3.5
Total	9.7

Summary of Design Costs

Table 58 contains the combined design costs for the different methodologies. The RM, SUH/SCS, SUH/CH, PDM, and THM require about the same amount of design time. The CM/SCS and CM/CH require about 60 percent more design time than the first five, and CS requires sligthly more than twice the design time of the first five.

Table 58. Summary of Design Costs for the Various Methodologies.

Methodology	Costs, man-hours Data Collection and Parameter Estimation	Analysis	Total
RM	10.5	1.7	12.2
SUH/SCS & SUH/CM	9.25	2.25	11.5
PDM	10.75	.85	11.6
THM	11.5	.65	12.15
CM/SCS & CM/CH	14.5	5.25	19.75
CS	16.25	9.7	25.95

Construction Costs

In order to estimate construction costs of the drainage systems, a number of assumptions had to be made. First, only major pipes in the drainage network were considered, meaning that the calculated costs do not represent the total cost of the networks. The cost of inlets, minor pipes, etc. were assumed to be fixed costs and therefore not as important to calculate. Second, the 5-year return period peak flows were used as the design values. Although a larger return period would normally be used in design for some of the areas, the 5-year return period peak flow was chosen because of uncertainty involved with some of the larger values from CS. If the uncertainty had not existed and a larger return period had been used, the relative differences would probably have remained the same, but the actual differences would have been greater. Third, published estimates of existing slopes were used for the calculation of all pipe sizes. Fourth, cost data from a recent publication (McMahon, 1986) were used to determine installed pipe costs. Fifth, the flows capable of being handled by the existing system relative to the flow capable of being handled by the existing pipe at the outlet end of the basin were assumed to be valid, regardless of the design flow determined for the outlet end of the basin.

The construction costs were estimated in the following manner:

1. The pipe size at the outlet end of the basin was calculated for each methodology. All pipe sizes were rounded up to the nearest available pipe size.

2. The ratio of full flow for the calculated pipe to full flow for existing pipe at the outlet end of the basin was determined.

3. The flows for the rest of the major pipes in the system were determined by multiplying the full flow in the existing pipes by the ratio found in Step 2.

4. Pipe sizes for the rest of the major pipes were found by using the flows from Step 3.

5. The cost of each calculated network was determined by multiplying each pipe length by the appropriate published cost estimate and calculating the sum of all of those products.

Because of the technique used and the asssumptions made, the reader should understand that the costs are only rough estimates and that the difference between the costs is much more significant than the actual costs.

MEG Basin

Construction costs for the MEG basin were not calculated for a number of reasons--the lack of existing drainage system data being one of them. Instead, the pipe size at the outlet end of the basin was calculated for each methodology so that some idea of construction costs could be inferred. A somewhat unrealistic slope of 0.1 ft/ft, which is much larger than the existing slope at the outlet end of the basin (0.0027 ft/ft), was used in for calculation of pipe sizes so that only pipe need be listed for each of the methodologies. The pipes were assumed to be concrete with a Manning's n of 0.013. Table 59 lists the pipe sizes as determined from the peak flow values of the various methodologies. The peak flow value for CS at the 5-year return period was interpolated from the two values that bracket that return period.

Table 59. Pipe Sizes for 5-Year Return Period Flows for the MEG Basin.

Metholology	5-Year Return Period Peak Flow cfs	Pipe Diameter Needed to Accomodate Peak Flow in.
RM	1354	78
SUH/SCS	1081	72
SUH/CH	1031	66
PDM	1039	66
THM	2494	96
CM/SCS	1726	84
CM/CH	1171	72
CS	1545	78

LDR Basin

Concrete pipes with Manning's n values of 0.013 were used for pipe sizing. The peak flow value for CS at the 5-year return period was interpolated from the two values that bracket that return period. Table 60 lists construction costs estimates determined for the various methodologies.

HWY Basin

Concrete pipes with Manning's n values of 0.013 were used to determine the pipe sizes that will accomodate the 5-year return period peak flow values found by the various methodologies. The 5-year return period peak flow value for CS was calculated by interpolating between the values of the two return periods that bracket the 5-year return period. Table 61 lists estimates of construction costs for the major parts of the drainage networks calculated for to handle the peak flows from the various methodologies.

Table 60. Comparison of Calculated Construction Costs for the Main Drainage Network--LDR Basin.

Metholology	5-Year Return Period Peak Flow cfs	Pipe Diameter Needed to Accomodate Peak Flow in.	Calculated Cost of Main Drainage Network dollars	Difference in Cost from Calculated CS Network dollars
RM	14	24	38,750	-27,000
SUH/SCS	2	15	24,000	-41,750
SUH/CH	2	15	24,000	-41,750
PDM	23	30	46,400	-19,350
THM	57	42	65,750	0
CM/SCS	69	48	76,500	10,750
CM/CH	38	36	55,000	-10,750
CS	48	42	65,750	-

Table 61. Comparison of Calculated Construction Costs for the Main Drainage Network--HWY Basin.

Metholology	5-Year Return Period Peak Flow cfs	Pipe Diameter Needed to Accomodate Peak Flow in.	Calculated Cost of Main Drainage Network dollars	Difference in Cost from Calculated CS Network dollars
RM	55	42	217,750	-42,700
SUH/SCS	7	24	131,750	-128,700
SUH/CH	7	24	131,750	-128,700
PDM	71	48	260,450	0
THM	142	60	378,250	117,800
CM/SCS	79	48	260,450	0
CM/CH	50	42	217,750	-42,700
CS	68	48	260,450	-

COM Basin

Concrete pipes with Manning's n values of 0.013 were used for the determination of pipe sizes. The 5-year return period value for CS was determined by interpolating between the two values that bracket that return period. Table 62 gives the construction cost estimates calculated for the COM basin.

Table 62. Comparison of Calculated Construction Costs for the Main Drainage Network--COM Basin.

Metholology	5-Year Return Period Peak Flow cfs	Pipe Diameter Needed to Accomodate Peak Flow in.	Calculated Cost of Main Drainage Network dollars	Difference in Cost from Calculated CS Network dollars
RM	122	54	161,300	33,600
SUH/SCS	31	30	90,150	-37,550
SUH/CH	30	30	90,150	-37,550
PDM	59	42	127,700	0
THM	164	54	161,300	33,600
CM/SCS	66	42	127,700	0
CM/CH	46	36	102,400	-25,300
CS	60	42	127,700	-

HDR Basin

Corrugated pipes with Manning's n values of 0.024 were used to determine pipe sizes that would accomodate the 5-year return period peak flows. The 5-year return period peak flow for CS was calculated by interpolating between the two values that bracket that return period. Table 63 lists the construction costs determined for the HDR basin.

Table 63. Comparison of Calculated Construction Costs for the Main Drainage Network--HDR Basin.

Metholology	5-Year Return Period Peak Flow cfs	Pipe Diameter Needed to Accomodate Peak Flow in.	Calculated Cost of Main Drainage Network dollars	Difference in Cost from Calculated CS Network dollars
RM	37	42	99,550	0
SUH/SCS	66	54	127,250	27,700
SUH/CH	64	54	127.250	27,700
PDM	65	54	127,550	27,700
THM	58	54	127,550	27,700
CM/SCS	42	48	104,100	4,550
CM/CH	27	42	99,550	0
CS	38	42	99,550	-

Differences in calculated construction costs based on the 5-year return period peak flows from the four south Florida basins are as variable from method to method as the peak flow results. No conclusions can be made as to any trend in the costs. What can be seen, however, is that differences in peak flow results can translate into substantial differnces in construction costs.

The next chapter (Chapter 5) draws some conclusions based on the material presented in this thesis. Chapter 5 also contains a brief summary of the thesis.

CHAPTER 5

SUMMARY AND CONCLUSIONS

In order to study the predictive reliability and cost implications of several convetional stormwater-study methodologies and an alternative methodology based on continuous simulation (CS), the methodologies were first examined on the basis of their advantages, disadvantages, and underlying assumptions. The conventional methodologies are generally based on synthetic rainfall, which means that the return period of the parameter of interest is based on rainfall statistics and antecedent conditions have to be assumed. However, the conventional methodologies are usually easy to use and widely accepted. Table 64 lists the conventional methodologies and the abbreviations used for them. CS does not have the drawbacks mentioned above, but it is more difficult to use.

Table 64. Abbreviation Used for the Conventional Methodologies.

Method	Abbreviation Peak Flow	Abbreviation Total Flow
Rational Method	RM	--
Synthetic Unit Hydrograph with		
Chicago Design Storm	SUH/CH	--
24-Hour SCS Type II Design Storm	SUH/SCS	--
SCS Peak Discharge Method	PDM	SCS
SCS Tabular Hydrograph Method	THM	SCS
Regression Equations	RE	
Calibrated SWMM with		
Chicago Design Storm	CM/CH	CM/CH
24-Hour SCS Type II Design Storm	CM/SCS	CM/SCS

The methodologies were then applied to five study areas in Florida (listed in Table 65) so that peak flow and total flow results could be compared. The five study areas have been studied extensively in the past, so a large data base is available on each of them. Although hourly rainfall data was used for all of the methodologies that explicitly use a design storm, it was apparent afterwards that short-time-increment rainfall data - had it been available - should have been used on several of the areas to better judge the performance of the different methods. Such data were available for the MEG basin from a previous unpublished study. The peak flows from the 15-minute-interval storms were generally 25 percent greater than the corresponding hourly storms, but the ranking of the storms remained the same. Greater peak flows could have also been expected for the south Florida basins, but the ranking may have been more likely to change due to the greater sensitivity of small basins to rainfall time step.

Table 65. Abbreviations Used for the Five Study Areas.

Location of Study Area	Abbreviation
Tallahassee, FL	MEG Basin
Pompano Beach, FL	LDR Basin
Broward County, FL	HWY Basin
Ft. Lauderdale, FL	COM Basin
Miami, FL	HDR Basin

In general, the RM yielded peak flow results that seem somewhat low, when considering the fact that the rainfall interval for this method was usually fairly short. For the COM basin, however, the results from the RM may be the most accurate of all the methods for two reasons: 1) it is the only method that explicitly used a short rainfall interval, and 2) it is especially valid for small, impervious basins such as the COM basin. The PDM also appeared to give low results, although it gave fairly high results for the HWY basin. The RE gave consistently low results in light the fact that hourly rainfall was used for several of the methods. The THM results were generally much higher than the results from the other methods, but part of this may have been because of the use of hourly rainfall for some of the other methods. The SUH/CH and the SUH/SCS, which used a 10-min synthetic unit hydrograph, gave low results for four of the five basins, but the low results are probably due to the use of hourly rainfall instead of 10-min rainfall, which itself may have been questionable. The CM/CH generally gave slightly low results, and the CM/SCS generally produced slightly high results.

All of the conventional methodologies used to determine total flow gave inconsistent, unreliable results when compared to the CS results. The difference in durations used for the different methodologies (24 hours for the conventional methodologies, usually much greater than 24 hours for the alternative methodology) may account for some of the difference in results, but certainly not all of it. It is apparent that these conventional methodologies were designed for and are more useful for determining peak flows. Detention basins sized by these methodologies stand a good chance of being either oversized or too small to hold the difference between pre- and postdevelopment flows for higher return period storms based on total flow.

The next part of the study involved making a determination of the time required to apply each methodology. This step was carried out by breaking the methodologies down into a number of tasks and determining the time required to carry out those tasks. Design cost implications could then be inferred from the total time required to apply the various methodologies. As can be expected, the estimated design costs for CS are much greater than the other methodologies. Design costs for the PDM, the SUH/SCS, and the SUH/CH were calculated as being the lowest.

Differences in sewer construction costs implied by the 5-year return period peak flow results were then calculated based on the existing sewer data and published cost-estimation data. This section of the study allowed for the significance of the differences in the peak flow results to be examined by being able to determine when a difference in peak flow would result in a different pipe size (and therefore a cheaper or more expensive system). The results from this part of the study are mixed. With the exception of the HDR basin, the CS values do not always yield the least expensive system, but they never yield the most expensive system. The one point that is clear is that differences in peak flows can translate into substantial differences in construction costs.

In to order make conclusions on total costs, the somewhat intangible cost of risk (as defined in Chapter 1) must be considered. Since the reliability of CS (when properly used) is greater than the other methodologies, the cost associated with risk for CS is lower than the other methodologies. Therefore, the total cost of using CS may often be lower than that of conventional methodologies.

Although CS with SWMM is usually more reliable and flexible than the conventional methodologies, it is not free from drawbacks and caveats. To begin with, the return periods for the largest one or two values are questionable because of the inherent variability in estimation of large return periods. (In this study the Weibull plotting position was used). A conventional form of frequency analysis could be used to recalculate the return period of the highest values. In addition, not all areas require such a detailed form of analysis. Applying CS to small areas that are not subject to flooding might not be cost effective. For example, suppose that there was an area that was not prone to flooding and had a drainage network that was only a fraction of the size of LDR basin's network. Since the cost of the drainage network and the risk associated with flooding would be relatively small, using CS would be difficult to justify because of the larger design costs associated with that methodology, especially the cost of obtaining short-time-increment historic rainfall data. (The use of a model with a varible rainfall time step would reduce this cost because only a portion of the storm would have to be in short increments). For an area like this one, using a simpler methodology coupled with a sizeable safety factor would probably be the most cost-effective means of designing the system. Lastly, there is the issue of needing rainfall-runoff data for calibration and verification. Most areas do not have such data available, and the cost and time involved in obtaining them is an important factor to consider. Parameters for SWMM and similar models can be estimated fairly accurately, but without rainfall-runoff data the reliability of the results is reduced.

For some areas using CS can be easily justified. An area like the MEG basin is one example. Since the construction costs appear substantially greater than design costs for the MEG basin, the methodology that has the greatest probability of insuring that the expensive drainage system is optimally sized is the logical methodology to use. Design costs are of minor consequence for this area.

Obviously, CS is not the final answer in stormwater design methodolgies. It is, however, a superior alternative to conventional methodologies for a number of applications.

APPENDIX A

DIMENSIONLESS HOURLY INTENSITIES FOR THE CHICAGO DESIGN STORM

Source: Keifer and Chu, 1957.

Hour	Dimensionless Intensity
1	0.020
2	0.021
3	0.021
4	0.021
5	0.022
6	0.023
7	0.025
8	0.033
9	0.115
10	0.340
11	0.060
12	0.035
13	0.028
14	0.025
15	0.023
16	0.022
17	0.022
18	0.021
19	0.021
20	0.021
21	0.021
22	0.021
23	0.020
24	0.020

	1.000

APPENDIX B

DIMENSIONLESS HOURLY INTENSITIES FOR THE 24-HOUR SCS TYPE II DESIGN STORM

Source: Soil Conservation Service, 1972.

Hour	Dimensionless Intensity
1	0.010
2	0.012
3	0.013
4	0.014
5	0.013
6	0.019
7	0.019
8	0.021
9	0.026
10	0.039
11	0.049
12	0.425
13	0.114
14	0.047
15	0.032
16	0.028
17	0.020
18	0.021
19	0.013
20	0.013
21	0.013
22	0.013
23	0.013
24	0.013

	1.000

APPENDIX C

ADDITIONAL INFORMATION ON SWMM RUNS

Calibrated Runoff Block input parameters for the five basins are shown in Table C.1. These parameters do not generally correspond to the values listed in Tables 6, 13, 20, 27 and 34 for the various catchments for a variety of reasons. The width parameter in the tables is just the area divided by a characteristic length and not the calibrated value given in Table C.1. Slopes, roughnesses and Green-Ampt hydraulic conductivity and capillary suction parameters (Table C.1) were based on Maalel's (1983) SWMM calibrations for the four South Florida sites and were not recalibrated. Thus, the calibrated SWMM input slopes and hydraulic conductivities are not representative of the actual site conditions for the South Florida basins (the SWMM slopes are generally higher, and the hydraulic conductivities are at least a factor of ten lower than would be expected for sandy soils). Values of the initial moisture deficit for all real storms (calibration, verification and historic design storms) were adjusted within the ranges shown (Table C.1) in a subjective manner (lower initial moisture deficit for higher antecedent rainfall); a constant value was used only for the SCS and Chicago design storms.

Single-event runs are relatively insensitive to evaporation values. However, evaporation values used for the five basins are also listed in this appendix in Table C.2. Finally, the storm events used for calibration and verification at each of the five sites are listed in Table C.3. The numbers for the USGS storms correspond to the storm numbers assigned them in the USGS source decuments: Mattraw et al. (1977) for LDR, Hardee et al. (1978) for HWY, Miller et al. (1979) for COM and Hardee et al. (1979) for HDR.

Table C.1. Calibrated SWMM Input Parameters

(Runoff Block Parameters, Data Group H1)

Parameter	MEG	LDR	Site HWY	COM	HDR
Width (ft)	5500	1500	525	2000	600
Area (ac)	1995	40.8	58.26	20.4	14.7
Imperviousness (%)	28.3	5.92	18.1	97.9	49.92
Slope (dimensionless)	0.0216	0.027	0.030	0.010	0.030
n - impervious	0.015	0.015	0.020	0.015	0.015
n - pervious	0.35	0.25	0.25	0.25	0.25
Depr. storage (in.)					
impervious	0.02	0.06	0.06	0.02	0.06
pervious	0.50	0.35	0.20	0.20	0.20
Green-Ampt parameters					
Suction (in.)	18.13	15	15	8	10
Hyd. cond. (in./hr)	5.76	0.40	0.20	0.15	0.40
Init. moist. deficit					
Cal./verif., typical	0.15	0.05	0.07	0.01	0.02
SCS, Chicago	0.32	0.01	0.01	0.01	0.02
Historic, range	0.32	0.05-0.15	0.001-0.15	0.01-0.14	0.01=0.14

Table C.2. Daily Evaporation Values Used in SWMM Runs

Month	Evaporation (in./day) for basin: MEG	LDR-HWY-COM	HDR	Month	Evaporation (in./day) for basin: MEG	LDR-HWY-COM	HDR
Jan	0.08	0.09	0.09	Jul	0.24	0.16	0.17
Feb	0.12	0.11	0.11	Aug	0.22	0.16	0.16
Mar	0.17	0.14	0.14	Sep	0.22	0.14	0.14
Apr	0.22	0.18	0.17	Oct	0.17	0.12	0.13
May	0.23	0.18	0.18	Nov	0.11	0.10	0.11
Jun	0.22	0.16	0.17	Dec	0.88	0.09	0.09

Table C.3. Storm Events Used for Calibration and Verification

(Numbers correspond to numbers in USGS reports)

LDR		HWY		COM		HDR	
Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.
3	16	22	26	4	43	3	1
15	18	34	44	6	55	9	4
35	23	66	51	16	38	16	17
40	50	75	79	64	77	44	33
57	85	97	83	88	102	52	48

MEG	
Calib.	Verif.
9/25/79	9/27/79
3/9/80	9/21/79
6/6/79	9/26/79
5/22/80	3/10/80
2/10/81	5/23/80

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BIOGRAPHICAL SKETCH

Brett Alan Cunningham was born on November 3, 1962, in Dunedin, Florida, to Mr. and Mrs. Donald W. Cunningham. He has one brother, Jeff. He was raised in Dunedin, attending public schools and graduating from Dunedin High School in June, 1980. He received an AA degree from St. Petersburg Junior College in May, 1982, and a BS degree from the University of Florida in May, 1985. He continued his studies at the University of Florida after graduation, concentrating on water resources in the Department of Environmental Engineering Sciences, and was awarded an ME degree in August, 1987. His plans after graduation include moving to Annandale, Virginia to begin his professional career.