DEVELOPMENT AND EVALUATION OF A GENERALIZED UNIT HYDROGRAPH

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PREFACE

This study is concerned with the development of an unit hydrograph technique based on the Clark Unit Hydrograph Model. The main objectives were to develop a procedure that better estimate the storage coefficient, K for ungaged watersheds. The storage coefficient was determined for approximately 200 rainfall-runoff events throughout the United States. Using these values, a predictive procedure for the storage coefficient and the development of an unit hydrograph is outlined. The method was evaluated by testing the procedure on approximately 50 rainfall-runoff events throughout the United States.

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CHAPTER I

INTRODUCTION

Runoff is the flow of water over the land's surface. Runoff originates from many sources, but is a result of precipitation. As precipitation falls, a portion of it is lost to abstractions such as interception by plants, depression storage and infiltration, with the remaining portion becoming surface runoff. The amount of water contributing to runoff is also dependent upon the rainfall intensity and duration. During light rainfall, a significant portion of the rainfall may be lost to abstractions, especially if the rainfall intensity is close to the infiltration rate of the soil. The runoff process is very complex combining temporally and spatially varying precipitation and characteristics of a drainage basin.

Runoff is typically depicted by the time distribution of flow at a point in the flow path. This distribution is called a hydrograph and contains peak flow rate and runoff volume information. Hydrographs can describe one storm or a series of storms.

The hydrograph is of fundamental importance in predicting surface runoff and in designing hydrologic structures. Because of its importance, there have been several methods used to predict runoff hydrographs. Two methods in wide use are (1) the unit hydrograph and (2) the synthetic unit hydrograph. Unit hydrographs are preferred but require observed rainfall and streamflow data (gaged watersheds).

Synthetic unit hydrographs can be used for watersheds without recorded or measured flows (ungaged watersheds). These hydrographs are models that indirectly predict runoff from watershed characteristics, rainfall characteristics, and other parameters. The accuracy of this approach is often questionable because of uncertainty in estimating runoff parameters; thus, there is a potential for improving these methods with better parameter estimates.

One commonly used synthetic unit hydrograph method is Clark's method (1943). Clark (1943) used a technique similar to the Muskingum flood routing method to develop a unit hydrograph. A major obstacle in using Clark's method is the determination of a storage coefficient, K. The storage coefficient for a watershed has been estimated from observed hydrographs using the ratio of discharge reduction to total discharge (Clark, 1943). This approach can not be used for ungaged watersheds, because it requires observed runoff data.

The specific objectives of this study are to develop a better estimate of synthetic unit hydrographs for ungaged watersheds by:

1. Developing a dimensionless instantaneous unit hydrograph (IUH), based on Clark's method, that is a function of a dimensionless storage constant K^*,
2. Optimizing K^* values from a large data set of observed rainfall-runoff events,
3. Developing a predictive equation for K^* using this large data set, and
4. Evaluating the accuracy of the dimensionless IUH, coupled with the predictive relationship for K^*, using an independent rainfall-runoff data set and comparing results to those obtained by a widely-used SCS method.

CHAPTER II

LITERATURE REVIEW

Introduction

In this chapter, an overview of hydrograph theory will be given to support the direction and objectives of this project. Several hydrograph methods will be discussed from a historical perspective. The discussion of these methods will focus on the general theory, methodology, parameters, and weaknesses of each.

Linear Theory of Hydrologic Systems

Most hydrograph methods assume that the relationship between rainfall and runoff can be modeled as a linear system. There are three functions that are typically used to describe the response of linear systems depending on the type of input. These are the impulse, step, and pulse response functions. A brief description of these response functions is given below. Further details are explained by Chow el al. (1988, pp. 202-213).

The impulse response function describes the system response from a unit input, applied instantaneously. The function for a unit impulse at time t′ is represented by the notation $\mu(t-t')$. The total response for a continuous input can then be represented as

$$Q(t) = \int_0^t I(t') \mu(t-t') dt' \tag{1}$$

This expression is the convolution integral and is the solution of a linear system for a continuous input. $I(t')$ is the input into the system, which would typically be the rainfall excess rate, and $Q(t)$ is the resultant flow from the system input. The terms t' and t are the time of the input and the time of response, respectively.

The unit step function describes the response of a system where the input rate goes from 0 to 1 at time zero and continues at that rate indefinitely. This response can be described by Eq. 1 for $I(t')=1$ as

$$Q(t) = \int_0^t \mu(l) dl \tag{2}$$

The variable $Q(t)$ is the unit step response function and l is the lag time (i.e., $t-t'$) of the system. The term $\mu(l)$ is again the unit impulse function as given in Eq. 1.

Since rainfall data are usually recorded in discrete intervals, the convolution solution also needs to be evaluate for discrete inputs. In order to handle discrete input, unit pulse functions are needed. The resulting discrete convolution equation for a linear system is the pulse function response

$$Q_n = \sum_{m=1}^{n<M} P_m \; U_{n-m+1} \tag{3}$$

The value Q_n is the instantaneous output at the end of the discrete time interval and P_m is the depth of rainfall falling during the time interval. The term U, is a sample data function. The subscript M is the number of pulse inputs while n and m are counter variables.

It should be pointed out that there are some studies that indicate the rainfall-runoff process actually acts as a non-linear system (Muftuoglu, 1984; Wang et al., 1988). The assumption of linearity is generally used however, because linear methods are easier to use than non-linear methods and often provide acceptable results.

Unit Hydrograph

Theory

L.K. Sherman (1932) first proposed the unit hydrograph (originally called the unit-graph). Sherman's unit hydrograph is an unit pulse response function of a linear hydrologic system, where the resulting runoff hydrograph is the result of 1 inch of excess rainfall (runoff) generated uniformly over a watershed at a uniform rate for a specified duration.

The unit hydrograph can be derived from any amount of excess rainfall assuming the following principles are inherent to the model (Chow et al., 1988):

1. The excess rainfall has a constant intensity within the effective duration;
2. The excess rainfall is uniformly distributed throughout the whole drainage area;
3. The base time of the DRH, Direct Runoff Hydrograph, (the duration of direct runoff) resulting from an excess rainfall of given duration is constant;
4. The ordinates of all DRH's of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph; and
5. For a given watershed, the hydrograph resulting from a given excess rainfall reflects the unchanging characteristics of the watershed.

It is nearly impossible to meet all of these requirements. The rainfall intensity within an effective duration can vary widely with time and space. To more closely meet the first two requirements, storms must be of short duration and must occur over a relatively small area. The true length of the time base is difficult to determine because available methods of base flow separation are approximate. In addition, the time base probably varies because of changing watershed characteristics. The proportionality of the ordinates is also questionable because hydrologic system are not truly linear. The use of the unit hydrograph is considered acceptable for many practical uses, but the results should be checked for erroneous predictions.

Derivation

There are many methods to derive a unit hydrographs including the ordinate, matrix, and linear programming methods. For problems where different rainfall durations are needed, the S-hydrograph and lagging method and the instantaneous unit hydrograph are useful concepts.

The ordinate method of developing a unit hydrographs consists of first obtaining the runoff data for a storm, subtracting the base flow from the data, then determining the direct runoff hydrograph (DRH) (Viessman, 1977). The depth of excess rainfall (runoff) is determined by dividing the volume of runoff by the area of the watershed. The volume of runoff is the area under the DRH. The ordinates of the unit hydrograph are obtained by dividing the DRH ordinates by the excess rainfall depth. The duration of the unit hydrograph is determined as the time from the beginning of runoff to the end of the rainfall, assuming the rainfall excess was uniformly distributed and of uniform intensity.

Since it is assumed that the unit hydrograph is a linear system, the discrete convolution equation (3) can be used to determine the unit hydrograph. The direct runoff ordinates are given as Q_n and P_m is the excess rainfall. By a reverse process called deconvolution, the unit hydrograph ordinates, U_{n-m+1} can be calculated (Chow et al., 1988). If there are N pulses of direct runoff and M pulses of excess rainfall, then N equations can be written for Q_n in terms of N-M+1 unknown values of the unit hydrograph. The derived hydrograph may display erratic differences in the ordinates with some negative ordinates possibly occurring. These irregularities are a result of the data not being truly linear or the storm used not meeting the unit hydrograph theory requirements. This method can be used for complex multi-peaked storms, but the possibility of errors increases with storm complexity.

Another method used to derive the unit hydrograph is through the use of least-square fitting or linear programming. To facilitate these methods, the discrete convolution equation 3 may be expressed in matrix form

$$\begin{bmatrix} P_1 & 0 & 0 & \cdots & 0 & 0 & \cdots & 0 & 0 \\ P_2 & P_1 & 0 & \cdots & 0 & 0 & \cdots & 0 & 0 \\ P_3 & P_2 & P_1 & \cdots & 0 & 0 & \cdots & 0 & 0 \\ \cdot & & & & \cdot & & & & \cdot \\ \cdot & & & & \cdot & & & & \cdot \\ \cdot & & & & \cdot & & & & \cdot \\ P_M & P_{M-1} & P_{M-2} & \cdots & P_1 & 0 & \cdots & 0 & 0 \\ 0 & P_M & P_{M-1} & \cdots & P_2 & P_1 & \cdots & 0 & 0 \\ \cdot & & & & \cdot & & & & \cdot \\ \cdot & & & & \cdot & & & & \cdot \\ \cdot & & & & \cdot & & & & \cdot \\ 0 & 0 & 0 & \cdots & 0 & 0 & \cdots & P_M & P_{M-1} \\ 0 & 0 & 0 & \cdots & 0 & 0 & \cdots & 0 & P_M \end{bmatrix} \cdot \begin{bmatrix} U_1 \\ U_2 \\ U_3 \\ \cdot \\ \cdot \\ \cdot \\ U_{N-M+1} \end{bmatrix} = \begin{bmatrix} Q_1 \\ Q_2 \\ Q_3 \\ \cdot \\ \cdot \\ \cdot \\ Q_M \\ Q_{M+1} \\ \cdot \\ \cdot \\ \cdot \\ Q_{N-1} \\ Q_N \end{bmatrix} \quad (4)$$

or

$$[P][U] = [Q] \tag{5}$$

Chow et al.(1988) present a method for solving for [U]. First the matrix [P] is reduced to a square matrix [Z] where

$$[Z] = [P]^T[P] \tag{6}$$

and both sides of Eq. 5 are multiplied by the inverse of the [Z] matrix. Solving for the matrix [U] yields

$$[U] = [Z]^{-1}[P]^T[Q] \tag{7}$$

The solution to this is difficult to solve because of the number of repeated and blank entries in the [P] matrix, making it difficult to invert [Z].

Generally there is no solution for [U] that will satisfy all N equations. Typically a solution [U] is assumed that will yield an estimate of the DRH and satisfy all N equations. The new DRH is defined as

$$[P][U] = [Q'] \tag{8}$$

where the solution minimizes the error between the actual and estimated DRH's, [Q] and [Q'] respectively.

The method of least square minimizes the sums of squares between [Q] and [Q']. Singh (1976) presented a method of solution by least squares. This method may yield negative ordinates in the unit hydrograph. These discrepancies were attributed to the nonlinearity of the system, sampling and observation errors of the rainfall and runoff data.

A linear programming model determines the optimal unit hydrograph, given a set of constraints. Linear programming minimizes the sum of the absolute differences between the actual and estimated DRH's and insures that all the values of the [U] matrix are non-negative (Singh, 1976; Mays and Coles, 1980).

A unit hydrograph with a different duration can be derived from an existing unit hydrograph. The unit hydrographs are derived by applying the principles of proportionality and superposition. This method is called the S-hydrograph and lagging method (Chow et al., 1988).

The S-hydrograph is a hydrograph produced by a continuous excess rainfall rate for an infinite duration. This is the unit step function response as described by Eq. 2. The S-hydrograph is generated by repeating an infinite number of unit hydrographs, each lagging the other by the unit hydrograph duration (D), and then summing the ordinates of all the hydrographs at the corresponding times.

To construct a unit hydrograph with a different duration (D'), the S-hydrograph is plotted. The same S-hydrograph is plotted again, lagging the first by D'. The ordinates of the new hydrograph are obtained by determining the differences between the two S-hydrographs (i.e., subtract the discharge values of the S-hydrograph from the shifted S-hydrograph). The new ordinates are then multiplied by D/D′ yielding the unit hydrograph of duration D′(Chow et al., 1988).

As in other hydrograph methods, the S-hydrograph method assumes that the system has a linear relationship between the rainfall and runoff. Due to the fact that the system is not truly linear, fluctuations in the derived unit hydrograph ordinates are usually produced and can sometimes be very considerable. Also, inaccuracies in the runoff and rainfall data, and the duration may complicate the amplitude of the fluctuations.

A purely theoretical approach to the unit hydrograph is the instantaneous unit hydrograph (IUH). As the duration become infinitesimal, the resulting hydrograph is the impulse response function (Eq. 1) called the IUH. The IUH characterizes the watershed response to rainfall without reference to rainfall duration. The convolution integral can be written as

$$Q(t) = \int_0^t \mu(t - \tau)\ I(\tau)\ d\tau \tag{10}$$

where $Q(t)$ is the runoff at time t, $I(\tau)$ is the IUH ordinate at time τ, and $\mu(t-\tau)$ is the rainfall excess intensity at $t-\tau$. The IUH has the following properties:

$$0 \le \mu(t-\tau) \le \text{positive peak value} \quad \text{for } (t-\tau) > 0$$

$$\mu(t-\tau) = 0 \qquad \text{for } (t-\tau) \leq 0$$

$$\mu(t-\tau) \rightarrow 0 \qquad \text{as } (t-\tau) \rightarrow \infty$$

A useful time parameter in defining the IUH is defined as

$$t_l = \int_0^\infty \mu(t-\tau)\ (t-\tau)\ d(t-\tau) \tag{11}$$

where t_l is the lag time of the IUH. The time lag gives the time interval between the center of mass of an excess rainfall hyetograph and the center of mass of the corresponding direct runoff hydrograph. The ideal shape of the IUH resembles a single-peaked direct-runoff hydrograph (Chow et al., 1988).

An IUH ordinate at time t is equal to the slope at time t of a S-hydrograph constructed for an excess rainfall intensity of unit depth per unit time. This procedure is based on the fact that the S-hydrograph is an integral curve of the IUH. The IUH obtained from this method is an approximation because the slope of the S-hydrograph is difficult to measure accurately.

Synthetic Unit Hydrographs

Introduction

Since most small watersheds do not have observed rainfall and runoff data, various procedures have been developed to yield a unit hydrograph for these watersheds. Each of these methods differs in methodology or parametric considerations. The following discussion is limited to the more prevalent synthetic techniques.

Empirical Methods

The Time-Area-Method was introduced in the 1920's as a method for predicting discharge from a watershed (Dooge, 1973). The method consists of dividing the watershed into areas of equal travel time to the outlet. A time-area histogram is then constructed plotting the areas versus the travel time. The rainfall excess hyetograph is obtained and each storm burst is routed through the watershed with the aid of a time-area diagram. The hydrograph ordinates can be obtained from the following equation (Viessman et al., 1972)

$$Q_i = P_i A_1 + P_{i-1} A_2 + \ldots + P_1 A_i \tag{12}$$

where Q is the hydrograph ordinate, P is the excess rainfall hyetograph ordinate, A is the time-area histogram ordinate, and i is a counter variable. The method is a very crude approximation and does not account for surface storage, thus over-predicting the discharge from the watershed.

The first major synthetic method developed in the U.S. was Snyder's Method (Snyder, 1938). The procedure is an empirical method based on relationships of watershed geometry to hydrographs. The equations developed were obtained from studies conducted on watersheds ranging in size from 10 to 10,000 square miles in the Appalachia Mountains. Predictive equations were developed for the lag (lag as defined by Snyder), peak flow, hydrograph base time, effective storm duration, and duration lag adjustment.

Snyder's equation to predict his "lag" time, which was defined as the time between the center of mass of rainfall excess to peak discharge, is as follows

$$t_l = C_t \left(L_{CA} \ L \right)^{0.3} \tag{13}$$

where t_l is the "lag" time for a uniformly distributed storm in hours, C_t is a dimensionless coefficient ranging from 1.8 to 2.2, L is a watershed length in miles, and L_{CA} is the distance from the outlet to the center of the watershed in miles. The coefficient C_t accounts for storage and slope and is only valid for the Appalachian highlands.

The peak flow rate of the unit hydrograph is defined as

$$Q_P = \frac{640 \ C_P \ A}{t_l} \tag{14}$$

where Q_P is the peak discharge per square mile, A is the drainage area in square miles, and C_p is a coefficient ranging from 0.56 to 0.69.

The base length of Snyder's hydrograph is defined as

$$T = [\ 3 + (\ t_l\ /\ 8\ )\ ] \tag{15}$$

where T is the base length of the unit hydrograph in days. The values of t_P, Q_P, and T define points for an unit hydrograph of duration

$$D = t_l\ /\ 5.5 \tag{16}$$

For storms of different durations, the "lag" is adjusted by replacing the following term in place of t_l in equations (14) and (15)

$$t_{LP} = t_l + (D' - D)/4 \tag{17}$$

where D' is the effective storm duration of interest in hours. With the known points of the unit hydrograph, the hydrograph is sketched so that the area under the curve is equal to one inch of direct runoff.

Snyder (1938) identified several limitations to his proposed method. These included a potential for large discrepancies between actual and synthetic unit hydrographs when the watershed shape varied greatly from a fan shape, when the predicted "lag" values for small floods tended to be to large, and when the application was for flat areas, the coefficients needed to be adjusted.

Another popular empirical method was developed by the Soil Conservation Service (SCS NEH Sec. 4, 1972). The SCS hydrograph is a dimensionless curvilinear unit hydrograph that was derived from natural unit hydrographs from several watersheds varying in both size and geographic locations. The hydrograph ordinates are dimensionless values expressed by the ratio Q/Q_p and the abscissa by t/t_p. The method requires the prediction of the peak discharge (Q_p) and the time to peak (t_p) to obtain the unit hydrograph. Once these values are obtained, the dimensionless unit hydrograph ratios are multiplied by the estimated time to peak and peak discharge to obtain the unit hydrograph (i.e., the Q/Q_p ratios are multiplied by the predicted Q_p to yield the discharges Q).

The values of t_p and Q_p are obtained through a series of simple equations that account for watershed size, vegetative characteristics, slope, amount of rainfall, and rainfall duration. The time to peak is calculated by the following equation (SCS, 1972)

$$t_p = \frac{D_r}{2} + t_l \tag{18}$$

where t_p is the time from the beginning of rainfall to the peak discharge in hours, D_r is the duration of rainfall in hours, and t_l is the lag time from the center of mass of rainfall to the peak discharge in hours (t_l is the same as Snyder's "lag" time). The SCS (1972) used the following method to calculate the lag time

$$t_l = \frac{l^{0.8} \; (S+1)^{0.7}}{1900 \; Y^{0.5}} \tag{19}$$

where l is the hydraulic length of the watershed in feet, Y is the average watershed land slope in percent, and S is the potential maximum retention in inches. The potential maximum retention S is calculated by the equation

$$S = (1000 \, / \, CN) - 10 \tag{20}$$

where CN is a hydrologic soil cover complex number. This method for obtaining the lag time is only valid for watersheds of areas less than 2000 acres.

The duration and lag time have been linked to the time of concentration (t_c) of the watershed. The SCS (1972) uses a duration of rainfall of D_r=0.133t_c and the lag time of t_l=0.6t_c for average natural watershed conditions with uniformly distributed runoff. The time of concentration can be obtained from many different procedures. SCS (1972) has developed a nomograph for equations 16 and 17 where the time of concentration can be obtained directly. Typical methods include Kirpich's formula (Schwab et al., 1981), which accounts for watershed slope and length, and the SCS Upland Method

(SCS, 1972), which consists of dividing the main watershed channel into a series of reaches and summing the travel time for each reach to obtain the time of concentration.

The peak discharge is obtained by using the following equation (SCS, 1972)

$$Q_p = \frac{K\ A\ P}{t_p} \tag{21}$$

where K is a storage and unit constant whose value ranges from 600 in steep terrain to 300 in very flat, swampy country (typically is assumed to equal 484), A is the drainage area in square miles, and P is the depth of runoff in inches. For unit hydrographs, P equals one inch. For other hydrographs, SCS (1972) calculates P as

$$P = \frac{(\ I - 0.2S\ )^2}{I + 0.8S} \qquad \text{for } I > 0.2S \tag{22}$$

where I is the depth of rainfall in inches.

The SCS method is a very useful unit hydrograph method but is limited to watersheds of areas less than 20 square miles. The watershed shape should be uniform with a homogeneous drainage pattern. Also the accuracy of the peak flow estimate is dependent on the constant K in the prediction equation. The SCS Unit Hydrograph can be approximated by a triangle where the base time is equal to 2.67 times the time to peak for K=484.

Storage Model Methods

In 1943, Clark suggested that a unit hydrograph could be derived by routing a time-area-concentration curve through a single, linear reservoir (Clark, 1943). The routing procedure yielded an instantaneous unit hydrograph (IUH). A unit hydrograph for any rainfall duration can be obtained by dividing the instantaneous graph into periods of the desired duration and averaging the ordinates over the preceding periods of time (Clark, 1943).

Clark (1943) used a technique similar to the Muskingum routing method. The basic equations used by Clark included the following

$$I - O = dS/dt \tag{23a}$$

$$S = K\,Q \tag{23b}$$

$$Q = xI + (1 - x)O \tag{23c}$$

where I is the total inflow rate, O is the outflow rate, S is the total volume storage in the reach, K is the storage constant, Q is the weighted average of inflow and outflow, and x is the dimensionless weighting factor.

Clark (1943) assumed that the storage of the reach was constant throughout the entire range of discharge and that the storage constant K was dependent only on outflow rate, thus the weighting factor x is equal to zero. By combining equations 23a, 23b, and 23c, the simplified form can be expressed as

$$I - O = K\ (dO/dt) \tag{24}$$

The storage constant K for the watershed can be obtained from an observed hydrograph and corresponds to the ratio of discharge reduction to total discharge (Clark, 1943). The minimum K value can be obtained directly from Eq. 24 as

$$K = -\ O\ /\ (dO/dt) \tag{25}$$

where K is the storage constant for the watershed. An empirical estimation of the storage constant K is (Clark, 1943)

$$K = C\ L\ /\ (S)^{0.5} \tag{26}$$

where C is a coefficient ranging from 0.8 to 3.5, L is the length of the stream in miles, and S is the mean channel slope. Linsley (Clark, 1943), in discussion of Clark's paper, suggested the addition of drainage area into the equation to yield a prediction equation in the form

$$K = b\ L\ (A)^{0.5}\ /\ (S)^{0.5} \tag{27}$$

where b is the new coefficient, and A is the drainage area.

Clark (1943) defined the time of concentration for this method as the time between the cessation of runoff-producing rainfall and the storage constant K. Some of the indicated limitations of Clark's method include the variability in the estimate of the storage constant K and the size of the watershed the method is applied to. For large watersheds, the hydrograph may rise to slowly and fall to rapidly. Also, although the method exhibits the effect of watershed shape to produce high peaks, some of the predicted values may be exaggerated.

In a study conducted by the Irish Office of Public Works for determining unit hydrographs for ten Irish arterial drainage channels, a modified Clark's method was proposed (O'Kelley, 1955). The study presented a different method of estimating the time of concentration and storage constant. An isosceles triangle was substituted for the time-area diagram and routed through a single linear storage element without a significant loss in accuracy. A series of curves, considered to be the IUH, were produced by routing the triangular time-area diagram for different ratios of the storage constant to time of concentration (K/T_c). A unit hydrograph of duration N could then be derived from the IUH. The ordinate of the N-hour unit hydrograph at any time is the average ordinates of the IUH during a period N hours before that time. The peak is the largest average ordinate of the N-hour blocks of the IUH.

The main features of this method include the elimination of the effect of duration by the use of the IUH. The effect of watershed size is also eliminated by expressing the flow rate as flow rate per area. This method utilized the theories of similitude to eliminate the effect of watershed size on the time distribution of flow by modifying time and flow scales of the hydrograph so as to correspond to a model catchment of 100 square miles. The peak discharge, storage constant, and time of concentration for the model were plotted versus the statistical slope where the statistical slope is equal to the median slope. The storage constant K is derived from the falling leg of the hydrograph. O'Kelly (1955) used a method based on the equation

$$K = A / (Q_1 - Q_2) \tag{28}$$

where A is the area under the falling leg between the ordinates Q_1 and Q_2. From the derived K value, the time of concentration T_C is determined by using the K/T_C ratio appropriate to the slope.

There is great difficulty in accurately estimating the parameters for Clark's method. The K/T_c ratio is a function of the slope of the watershed and the slope is very difficult of define. O'Kelley (1955) used the statistical slope for a lack of a better method. The hydrograph peak and shape are sensitive to the values of K and T_c. Also, there is an indication that the assumption of an isosceles triangle may not be adequate (O'Kelley, 1955).

The previous two methods discussed were single storage element models. Nash (1957) introduced a two parameter synthetic method based on the storage constant and the concept an infinite number of successive reservoirs. An IUH was derived by routing instantaneous rainfall through a series of successive linear reservoirs of equal time delay. The watershed is replaced by a series of n reservoirs, each with the storage characteristics of equation (23b). The method ignores variation in the translation time over the watershed (i.e., all points have the same translation time).

The instantaneous inflow of volume V at the first reservoir will raise the level to accommodate for increased storage. The discharge rises instantly from zero to V/K and diminishes with time by the term:

$$Q_1 = \frac{V}{K} e^{-(t/K)} \tag{29}$$

where Q_1 is the discharge from the first reservoir, V is the volume of inflow, K is the storage constant, t is time, and e is the base of the natural logarithm. The discharge from the first reservoir becomes the inflow for the second reservoir and so on. With successive routing of the reservoirs, the discharge equation becomes

$$Q_n = \frac{V}{\Gamma(n)} K^{-n} e^{-t/K} t^{n-1} \tag{30}$$

where Γ is a gamma function and n is the number of reservoirs.

Nash (Gray, 1962) ignores the variation in translation time over the catchment, assuming all points have the same translation time. Thus the dimensionless time-area-concentration curve is a dirac delta function which when routed will yield Nash's solution for the IUH. An advantage to Nash's model is that the S-hydrograph can be described as the ratio of the incomplete to the complete gamma function. The ordinates of a finite duration unit hydrograph are obtained by lagging the S-hydrographs and determining the difference between the two tabulated values. Once again, the storage constant is difficult to estimate.

Dooge (1959) presented a general equation for a unit hydrograph that is derived from the physical assumption that the reservoir action can be separated from the translatory action and lumped in a number of reservoirs unrestricted in size, number, and distribution. By idealizing the reservoirs in the watershed, the complexity of the method was reduced. The ordinates of the instantaneous unit hydrograph are obtained by integrating the product of an ordinate of the time-area-concentration curve and an ordinate of the Poisson probability function. The ordinates of the IUH can be explained by the following equation:

$$U(0,t) = \frac{V}{T} \int_0^{t \leq T} P(m,n-1)\ w(\tau')\ dm \tag{31}$$

where U(0,t) is the ordinate of the instantaneous unit hydrograph, V is the volume of excess rainfall, T is the maximum translation time of the catchment, P(m,n-1) is the Poisson probability function, m is a dimensionless time variable equal to $(t-\tau')/K$, n is the number of linear reservoirs downstream at τ', $w(\tau')$ is an ordinate of the dimensionless time-area-concentration curve, t is the time elapsed since occurrence of rainfall excess, and τ' is the translation time.

Unlike the previous storage methods, where only a few of the parameters were variable, Dooge's method allows for variability in five parameters. These are the time of concentration, the storage constant K, the total number of reservoirs within the catchment, an adjusted time-area-concentration curve adjusted for variation in rainfall intensity, and the distribution of reservoirs in the catchment. Clark's, O'Kelly's, and Nash's models are all special cases of Dooge's model.

Similar to the previous methods, the parameters in Dooge's model are difficult to estimate, which is of greater concern because it has more parameters. Also, the method assumes that the response is linear, which may not be totally true.

Geomorphic Models

Geomorphic Concepts. Before discussing the different geomorphic models, a review of geomorphology will be presented. Geomorphology is the study of the formation of the landscape. Geomorphic models are based on the theory that the basin formation and hydrologic characteristics are related. One of the major drawbacks in the use of geomorphic parameters is the different definitions for the same parameters.

Horton (1945) suggested a method of classifying the branching of the stream within the basin. A first order stream is a small, unbranched tributary, a second order stream has only first order tributaries, a third order stream has first and second order tributaries, and so on. The higher-ordered streams are considered to extend headward to the tip of the longest tributary it drains. Strahler (1957) suggested an ordering scheme slightly different where he restricts the designation of stream order to stream segments. Streams of any given order include only segments formed by the merger of two channels of the next lower order and end when the segment merges with channels of equal or higher order (Figure 1.).

The order of the basin is determined by the order of the principal stream at the outlet of the watershed. This information is obtained from maps and photographs. A major problem is that maps are not constant in the delineation of streams, so different scale maps show different stream orders. Aerial photographs give a truer representation of stream order.

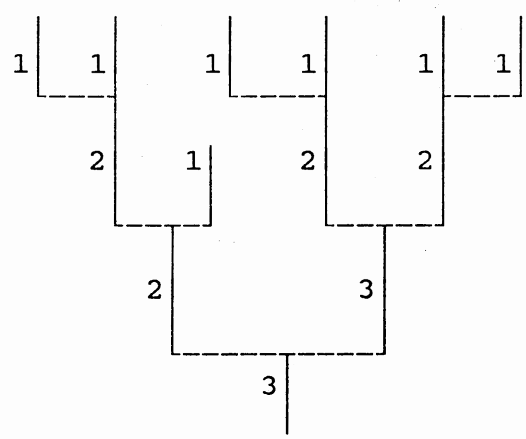


Figure 1. Definition of Strahler's Stream Order

Horton (1945) also introduced some physical descriptors using the stream ordering system. The bifurcation ratio was introduced to describe the ratio of the number of streams of any order to the number in the next lowest order. This observation led to the Law of Stream Numbers

$$R_B = \frac{N_w}{N_{w+1}} \tag{32}$$

where N_w and N_{w+1} are the number of streams of order w and w+1 respectively and R_B is the bifurcation ratio. Similarly, Horton suggested the Law of Stream Lengths

$$R_L = \frac{L_w}{L_{w+1}} \tag{33}$$

where L is the average length of streams of order w and w+1 respectively and R_L is the length ratio. An equivalent equation can be obtained for a Law of Stream Areas

$$R_A = \frac{A_w}{A_{w+1}} \tag{34}$$

where A is the average area contributing runoff to a stream of order w and w+1 respectively, and R_A is the area ratio.

Several other parameters are presented by Linsley et al.(1982). Some of these include the drainage density which is the total length of streams within the watershed divided by the drainage area. The average length of overland flow can be approximated by (Linsley et al., 1982)

$$L = 1 / 2D \tag{35}$$

where D is the drainage density. Horton suggested that the denominator be multiplied by $[1-(S_c/S_g)]^{0.5}$ where S_c is the slope of the channel and S_g is the slope of the ground.

The watershed shape plays an important part in the hydrologic response. Several indexes have been developed to account for the watershed shape. Horton (1945) suggested a dimensionless shape index defined as

$$R_f = A / L_b^2 \tag{36}$$

where R_f is the shape index, A is the area of the watershed, and L_b is the length of the watershed measured from the outlet to divide near the head of the longest stream along a straight line. Several other methods have been proposed using a circle or lemniscate as a reference shape (Linsley et al., 1982). These reference shapes are substituted into equation 36 for the L_b term.

Along with the watershed shape, the topography plays an important part in hydrologic response. Several descriptors have been developed. Some of the more typical and useful include the channel slope and land slope. Linsley et al. (1982) suggest dividing the channel into N segments, summing the square roots of the slope of each segment, dividing the summation by the number of segments and then squaring the value to obtain a channel slope index. The method for determining the slope of the land typically consisted of establishing a grid over the watershed and determining the mean and median slope (Linsley et al., 1982).

<u>Watershed Bounded Network Model.</u> In recent years,

a large number of methods have been developed for deriving a unit hydrograph based on geomorphic parameters. One such model is the Watershed Bounded Network Model (WBNM) which is a storage routing model based on geomorphology (Boyd et al., 1979). The model was developed from 241 rainfall events on ten watersheds with areas ranging from 0.4 to 251 square kilometers in eastern New South Wales, Australia. The storage routing model develops a runoff hydrograph from a rainfall excess hyetograph. The model divides the watershed into storage elements, connected in the same arrangement as the stream network. Each storage element has storage parameters based on geomorphic and hydrologic characteristics of the individual element. The flows are routed through each storage element using the continuity and storage equations. The computed outflows at each confluence of sub-areas are added together.

The watershed is divided into sub-areas and these areas are determined to be either ordered basins or inter-basin areas. The ordered basins have no inflow across their boundaries, thus only rainfall contributes to the runoff. The inter-basin areas are sub-areas with a stream flowing through it, thus having upstream inflow as well as rainfall contributing. Each sub-basin type is modeled individually and differently.

The catchment is modeled by representing each sub-area (ordered or inter-basin) by a lumped storage element and connecting the storage elements in the same topology network as the catchment stream network. There are two storage constants for the model. The first storage constant K_B is the lag time for transformation of the excess rainfall to direct runoff. This storage constant is found in both the ordered and inter-basin areas. The other storage constant is K_I. It is the lag time associated with the flow through the sub-area and is associated only with the inter-basin areas.

The storage constants are evaluated for each element and remain constant throughout the element. The storage constants relation to the geomorphology is obtained by using the area of the element.

The storage constant K_B is defined in hours as

$$K_B = 2.5 A^{0.38} \tag{37}$$

where A is the area of the element in square kilometers. Similarly, the storage constant K_I is defined as

$$K_I = 1.5 A^{0.38} \tag{38}$$

There are a large number of parameters for this model. For N number of storage elements, there are N values of K_B and N_i values of K_I for N_i inter-basin segments. The number, type, and arrangement of the storage elements is based on the catchment structure and relationships between the catchment geomorphology and hydrology. As the number of elements increases, better estimates are obtained, but the determination of whether the element is an ordered basin or inter-basin area is more critical than the number of elements. The estimates for the storage constants were developed for watersheds in Australia and may require modification for application to watersheds with very different topography and climates.

Probability-Based Geomorphic Models. A frequently cited IUH geomorphic model was developed by Rodriguez-Iturbe and Valdes (1979). In this model, a probabilistic approach is used to predict when a rainfall drop, chosen at random, will reach the outlet at time t. The rain drop movement is described by a Semi-Markovian process where the transition states are separated by the stream orders for the watershed and outlet. The successive state occupancies are governed by transition probabilities of a Markov process, but the time of stay by the drop in any state is described by a random variable that is dependent upon the present state and the next transition state. The resulting IUH is a function of R_B, R_A, R_L, velocity (v) and a scale factor L, where R_B, R_A, and R_L are reorganized forms of Horton's bifurcation ratio, stream area ratios and length ratios, and the scale factor L is the length of the highest order stream.

The transition probabilities are related to the geomorphic Horton numbers. Each probability incorporates geomorphic parameters and the waiting time of a drop in a state. The waiting time is the mean time spent in a state as both overland and stream flow. The waiting time of a drop in a state order is assumed to be a random variable that is exponentially distributed. This assumption is equivalent to that of a linear reservoir. The waiting time includes the size effect and dynamic component of response. Thus the waiting time is the ratio of the average stream velocity to the mean length of the stream of that order.

Rodriguez-Iturbe and Valdes (1979) state that the most important characteristics of the IUH are the peak (q_p) and the time to peak (t_p). If these values are correct, the IUH can be approximated with a triangle. The following equations were presented by Rodriguez-Iturbe and Valdes to predict q_p and t_p.

$$q_p = \frac{1.31 \; v \; R_L^{0.43}}{L} \tag{39}$$

and

$$t_p = \frac{0.44 \; L}{v} \left(\frac{R_B}{R_A} \right)^{0.55} R_L^{0.38} \tag{40}$$

The variables of the equation are as previously defined.

The product of q_p and t_p is independent of v or L. This dimensionless product IR is defined as

$$IR = q_p \cdot t_p = 0.58 \left(\frac{R_B}{R_A} \right)^{0.55} R_L^{0.38} \tag{41}$$

The IR factor is constant for each basin and is independent of storm characteristics. The factor is linked to the geomorphic structure and hydrologic response. This relationship also allows the derivation of the IUH from one parameter, either q_p or t_p.

Several methods for determining the time scale have been introduced. These methods attempt to improve the determination of the dynamic component of response by solving for velocity using linearized continuity and momentum equations for specified boundary conditions (Kirshen and Bras, 1983) or by relating the velocity to hydraulic and geometric characteristics of the highest order stream (Agnese et al., 1988).

In an attempt to validate the usefulness of the geomorphic IUH, a study was conducted on watersheds in Venezuela and Puerto Rico (Valdes et al., 1979). The watersheds varied in both geomorphic and physiographic characteristics. In controlled numerical experiments, the geomorphic IUH derived from the method proposed by Rodriguez-Iturbe and Valdes (1977) was compared to an IUH derived from discharge hydrographs produced by a physically based rainfall-runoff model. The derived geomorphic and rainfall-runoff model IUHs were very similar.

In another study of this method, the effect of the velocity on the q_p and t_p was analyzed (Rodriguez-Iturbe et al., 1979). Rodriguez-Iturbe and Valdes (1979) stated that the two most important parameters of the IUH were q_p and t_p, which vary from storm to storm and also during a given storm as a function of velocity (v). Storms of different intensity and duration were convoluted for sets of IUHs to obtain a q_p and t_p. These values were then compared to equivalent values obtained from the same storms that were simulated with an impervious rainfall-runoff model of the watershed. Each set of IUHs was developed from a single storm where a series of velocities were used. A sensitivity analysis was conducted on the predicted q_p and t_p values to determine the effect of the velocity on the accuracy of the estimates. For velocities greater than two meters per second, the errors in the velocity do not cause large errors in the estimates of q_p and t_p. This indicates that for small floods, there will be a greater chance of error in the estimates.

Gupta et al. (1980) proposed another probabilistic model for determining the functional form and parameters of an IUH based on geomorphic characteristics of a watershed that relaxes the Markovian requirements and provides a method for direct evaluation. In this model, it is assumed that the water particles are injected randomly over the watershed and follow a certain path overland and in a channel before reaching the outlet. Path function probabilities are based of Strahler's scheme of ordering channel networks. Each path has its own random holding time. The probability density function (pdf) of the watershed holding time (the IUH) is obtained by first determining the probability that a particle follows a certain path, by multiplying that probability by the pdf of the individual path's holding time and then summing these products over all possible paths.

In Gupta et al.'s (1980) approach the path probability functions are completely based upon the geomorphology of the watershed. The pdf of the random holding times is also based upon the geomorphology of the watershed and a dynamic component. This former pdf is assumed to be exponentially distributed.

In the application of this procedure by Gupta et al.(1980), the validity of the linear assumption was questioned. The method was evaluated by comparing model results with observed runoff values. The comparison for large basins was good but underestimated the peak for a smaller basin. The authors assume this was caused by nonlinearities in the rainfall-runoff transformations.

Troutman and Karlinger (1985) proposed a method of deriving an IUH assuming linear flow through topologically random channel networks. The linear routing method requires the knowledge of the topological configuration and individual channel segment lengths, along with the hydraulic parameters of the segment. The IUH for the watershed is derived from three parameters, the link (channel segment) length distribution, the vector of hydraulic parameters, and one of three topological properties. These topological properties include the magnitude (number of first order streams), diameter (mainstream length), and the stream order. The links are assumed to be independent and identically distributed random variables. Each network is a member of a topologically random population.

Three linear routing schemes, translation, diffusion, and general linear routing were tested assuming a constant drainage density. All of the schemes resulted in essentially the same IUH. From the watershed parameters, the derived IUH had a Weibull probability density function and the time to peak was a function of the magnitude, mean link length, and a scaler hydraulic parameter. The scaler hydraulic parameter is defined by celerity.

An assessment study of the method was conducted on drainage basins in the southeastern U.S.(Karlinger and Troutman, 1985). The translation and diffusion routing methods were used in the study. The study showed that the average celerity of the internal links was the critical parameter in the determination of the shape of the IUH. The results also indicated that the number of sources did not need to be large to approximate the topological IUH with the Weibull probability density function.

Non-Linear Models

The apparent nonlinear response over a range of watershed sizes has led to the development of nonlinear rainfall runoff models. Many of these models account for the nonlinearity in different parameters. Some place the nonlinear component in the storage and transformation effects (Muftuoglu, 1984; Boyd et al., 1978) while others account for it in the rainfall intensity (Wang et al., 1981).

In an expansion of his original theory, Rodriguez-Iturbe et al. (1981) proposed the development of an instantaneous unit hydrograph from geomorphic parameters along with the characteristic of rainfall excess. The probability density function of the peak and time to peak of the IUH are derived as functions of rainfall characteristics and the basin geomorphic parameters. With the introduction of the dependency of the IUH on the rainfall characteristics (intensity and duration) the model follows a nonlinear framework.

CHAPTER III

FORMULATION OF THE RUNOFF HYDROGRAPH

Introduction

This chapter contains the derivation of a dimensionless instantaneous unit hydrograph (IUH) based on Clark's unit hydrograph method (1943). First, a dimensionless continuity equation is developed assuming surface storage effects can be represented by a linear reservoir. A dimensionless inflow hydrograph is then routed through this reservoir to obtain a dimensionless IUH. A convolution technique for unsteady rainfall excess patterns is also briefly discussed.

A synthetic time-area curve is used to obtain a dimensionless inflow hydrograph. The time-area curve is a dimensionless, triangular curve (Wilson, 1983).

Dimensionless Unit Hydrograph Development

Dimensionless Continuity Equation

A general form of the continuity equation is described by Eq. 23a. Clark's method assumes a linear reservoir where the storage in the basin is described by Eq. 23b. Substituting Eq. 23b into Eq. 23a produces

$$I - O = \frac{d(KO)}{dt} = K \frac{dO}{dt} \qquad (42)$$

where K is the storage coefficient of the linear reservoir, I is the inflow, and O is the outflow. Multiplying Eq. 42 by $t_c/(A_T\ z_e)$, where t_c is the time of concentration, A_T is the total watershed area, and z_e is the depth of excess rainfall, will produce dimensionless inflow and outflow terms defined as

$$I^* = \frac{I\ t_c}{A_T\ z_e} \qquad (43)$$

and

$$O^* = \frac{O\ t_c}{A_T\ z_e} \qquad (44)$$

where I^* is the dimensionless inflow and O^* is the dimensionless outflow.

The storage constant K in the continuity equation has the dimension of time and is made dimensionless as

$$I^* - O^* = \frac{K}{t_c} \frac{dO^*}{d(t/t_c)} \qquad (45)$$

A dimensionless time and storage coefficient can then be defined as

$$K^* = K / t_c \qquad (46)$$

and

$$t^* = t / t_c \tag{47}$$

which when substituted into Eq. 45 yields the dimensionless continuity equation

$$I^* - O^* = K^* \frac{dO^*}{dt^*} \tag{48}$$

For a known I^* and K^*, the above equation can be integrated to obtain the ordinates of the dimensionless unit hydrograph.

Time-Area Response

In Clark's method, the inflow values in Eq. 42 are determined by the time-area curve for given watershed. A dimensionless time-area curve is more suitable for the solution of Eq. 48. Dimensionless inflow is defined here as

$$I^* = z_e{}^* \frac{dA^*}{dt^*} = \frac{dA^*}{dt^*} \tag{49}$$

where $A^*=A/A_T$ is a dimensionless cumulative area, t^* is the dimensionless time as defined by Eq. 47, and $z_e{}^*$ is dimensionless excess rainfall which is equal to one for a unit hydrograph.

Four dimensionless time area curves were evaluated using a subset of the calibration data given in Chapter IV. The four curves were the symmetrical U.S. Corp of

Engineer curve (HEC 1, 1971), a symmetrical triangular curve where the peak I^* occurred at $t^*=0.5$, an oblique triangular curve where the peak I^* occurred at $t^*=0.75$, and an oblique triangular curve where the peak I^* occurred at $t^*=0.25$. For simulations where $K^*>0.75$, the resulting unit hydrographs were very similar. However if $K^*\leq 0.75$, the oblique triangular curve with the earlier peak appeared to better represent the data. Therefore, this shape was used to obtain the dimensionless time-area response. Its ordinates are defined as

$$I_1^* = 8\ t^* \qquad \text{for } t^*<0.25 \qquad (50)$$

$$I_2^* = 2.667(1-t^*) \qquad \text{for } 0.25<t^*<1.0 \qquad (51)$$

$$I_3^* = 0 \qquad \text{for } t^*>1.0 \qquad (52)$$

where the subscripts are used to identify I^* for each time interval.

Dimensionless Unit Hydrograph

A dimensionless unit hydrograph is obtained by solving the continuity equation given by Eq. 48 using the time-area response given by Eqs. 50, 51, and 52. An analytical solution is possible because of the simple expressions used to define I^*. Eq. 48 can be rearranged into the following form using an integration constant of $\exp(t^*/K^*)$

$$d[\ O^* \exp(t^*/K^*)\ ] = [\ I^* \exp(t^*/K^*)\ dt^*\ ]/K^* \quad (53)$$

where variables are as defined previously. Eq. 53 can now be integrated directly for each of the relationships for I^* given by Eqs. 50, 51, 52, resulting in three separate equations.

The first equation is used to define the unit hydrograph for $0 \le t^* \le 0.25$. I^* in this equation is given by Eq. 50. Therefore Eq. 53 can be written as

$$\int_0^{t^*} d(O^* \exp(t^*/K^*)) = \int_0^{t^*} (I_1^*/K^*) \exp(t^*/K^*)\ dt^* \quad (54)$$

for $0 \le t^* \le 0.25$ and integrates to

$$O^* = 8t^* - 8K^* + 8K^* \exp(-t^*/K^*) \quad (55)$$

for $0 \le t^* \le 0.25$, where O^* is the ordinate of the dimensionless hydrograph at time t^*.

The second equation defines the unit hydrograph for $0.25 \le t^* \le 1.0$, where I^* is given by Eq. 51. For this range, Eq. 53 is written as

$$\int_0^{t^*} d(O^* \exp(t^*/K^*)) = \int_0^{0.25} (I_1^*/K^*) \exp(t^*/K^*) dt^* + \int_{0.25}^{t^*} (I_2^*/K^*) \exp(t^*/K^*) dt^* \quad (56)$$

for $0.25 \le t^* \le 1.0$ and integrates to

$$O^* = -10.67K^*\exp\left(\frac{.25}{K^*}\right)\exp\left(\frac{-t^*}{K^*}\right) + 8K^*\exp\left(\frac{-t^*}{K^*}\right) + \frac{2+2K^*-2t^*}{0.75} \qquad (57)$$

for $0.25 \le t^* \le 1.0$, where O^* is the ordinate of the dimensionless hydrograph at time t^*.

The third equation defines the unit hydrograph for $t^* \ge 1.0$, where I^* is given by Eq. 52. Eq. 53 is now written as

$$\int_0^{t^*} d(O^*\exp(t^*/K^*)) = \int_0^{0.25} (I_1^*/K^*)\exp(t^*/K^*)\,dt^* + \int_{0.25}^{1} (I_2^*/K^*)\exp(t^*/K^*)\,dt^* + \int_1^{t^*} I_3^*\,dt^* \qquad (58)$$

for $t^* \ge 1.0$ and integrates to

$$O^* = -10.67K^*\exp\left(\frac{.25}{K^*}\right)\exp\left(\frac{-t^*}{K^*}\right) + 8K^*\exp\left(\frac{-t^*}{K^*}\right) + 2.67K^*\exp\left(\frac{1}{K^*}\right)\exp\left(\frac{-t^*}{K^*}\right) \qquad (59)$$

for $t^* \ge 1.0$. Eq. 59 is used until the O^* ordinate reaches zero.

The effects of K^* on the dimensionless IUH are shown in Figure 2. Here IUH's are plotted for $K^*=0$, $K^*=0.5$, $K^*=1.0$, $K^*=2.0$, and $K^*=5.0$. The dimensionless time-area curve corresponds to the unit hydrograph curve for $K^*=0$. As shown by Figure 2, the IUH is sensitive to

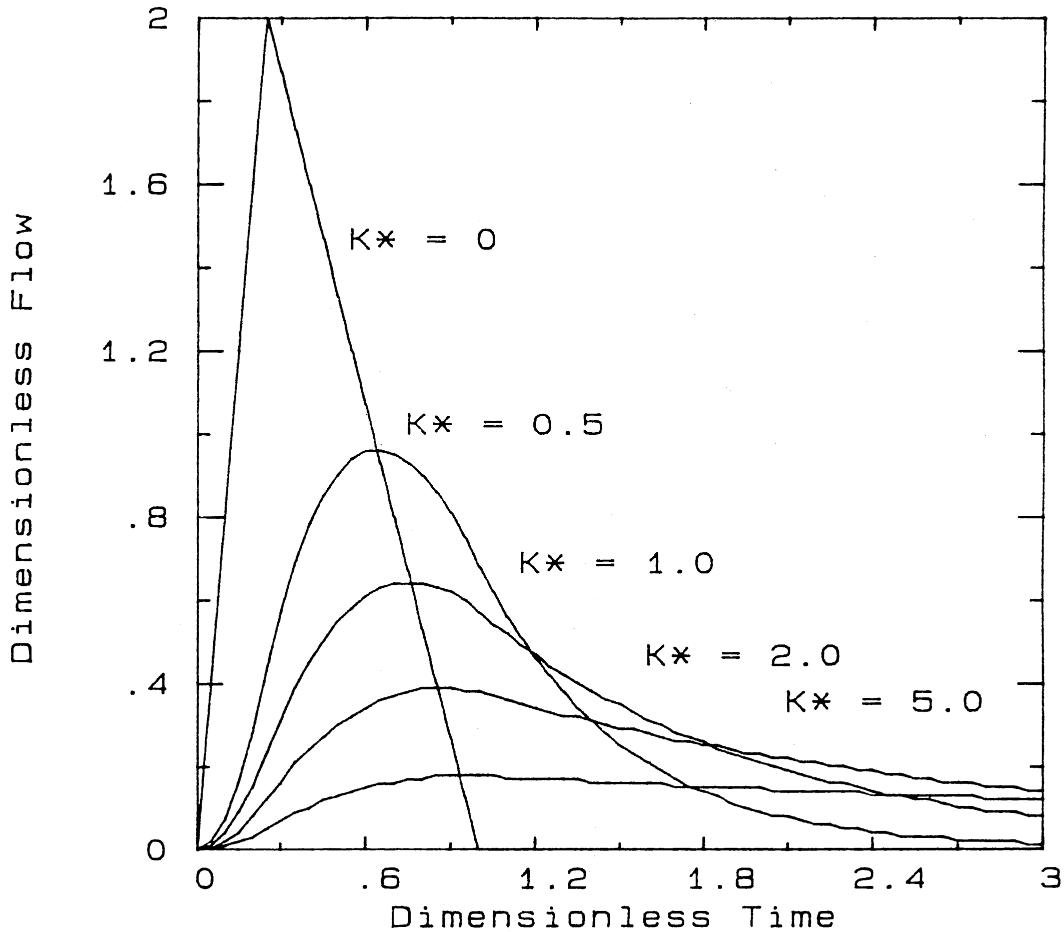


Figure 2. Effect of K^* on the Dimensionless Instantaneous Unit Hydrograph

K^* values less than 1.0. The change in the unit hydrograph from K^*=2.0 and K^*=5.0 is relatively small.

A dimensionless IUH can be calculated from Eq.'s 55, 57, and 59 for and estimated K^*. Developing procedures to estimate K^* from watershed and rainfall characteristics is a major thrust of this study. These procedures are discussed in subsequent chapters. An IUH can be used to approximate the unit hydrograph for discrete rainfall burst if the burst duration is relatively small. Work by O'Kelley (1955) indicates that an acceptable burst duration is $t_c/5$ or smaller. A burst duration of $t_c/20$ was used in this study.

Runoff Hydrograph Development

A summary of the computational steps is given below.

1. For an estimated K^*, the dimensionless hydrograph can be calculated directly form Eqs. 55, 57, and 59.

2. Site-specific unit hydrographs are obtained by multiplying O^* by $(1)A_T/t_c$ and t^* by t_c. The total watershed area and time of concentration are determined from map data as discussed in the following chapter. Site-specific ordinates are tabulated at constant time increments. The time increment is determined as $0.05t_c$.

3. The observed rainfall pattern is entered as cumulative depths with time. These values have been interpolated to the constant time increment values.

4. Cumulative and incremental rainfall excess depths for each time increment are estimated

using the SCS curve number model. The curve number is calibrated for each storm event using observed and rainfall data as discussed in the following chapter.

5. An incremental hydrograph is calculated for each rainfall excess depth.

6. The incremental hydrographs are lagged and summed to determine the total runoff hydrograph.

CHAPTER IV

CALIBRATION AND VALIDATION PROCEDURES

Introduction

A large rainfall-runoff data base was obtained to develop and test the dimensionless unit hydrograph. This data base was divided into events for calibration and events for validation. Characteristics of the rainfall-runoff data base are discussed in this chapter. Calibration and validation procedures are also presented.

Watershed and Storm Data

Watersheds for this study were selected from information supplied by the United States Department of Agriculture, Agricultural Research Service for experimental agricultural watersheds for the years 1958 through 1977 (Hydrologic Data, 1958-1977). Watersheds were selected on the basis of geographical location, size, length of record, and the availability of both rainfall and runoff information.

The watersheds selected vary in geographical location to allow for regional variability within the model. Sometimes two through five watersheds were selected of varying sizes in a given region. A limitation of 10,000 acres was placed on all watersheds selected, since the model is primarily for use on small agricultural watersheds. Also, a record of rainfall-runoff events for at least five years was required. Table I presents the location of the watersheds used and selection criteria. The fifth column in Table I identifies whether the watershed will be used in the calibration (C) or validation (V) procedures of the study. The test watersheds were selected randomly by first identifying geographic locations with more than one watershed. The identification number of each watershed was written on a piece of paper, placed in a can, and drawn out randomly. All storms for a given test watershed were used. A map showing the location of the watersheds is given in Figure 3.

Watershed and Rainfall Parameters

Geomorphic Parameter Estimation

The geomorphic parameters for each of the watersheds analyzed were taken from maps supplied by the U.S. Department of Agriculture, Agricultural Research Service. The parameters were measured using a Summagraphics Digitizer and an IBM portable computer. Watershed parameters were digitized from the maps, and a computer program calculated several geomorphic

TABLE I

WATERSHED SELECTION INFORMATION

Watershed Location	I.D.	Number of Events	Area Acres	Use
Blacksburg, VA	W13002	6	19.3	C
Blacksburg, VA	W13011	11	555.0	C
Klingerstown, PA	W16006	10	1,772.8	C
Iowa City, IA	W21001	6	1,930.0	C
McCredie, MO	W25001	17	154.0	C
Coshocton, OH	W26027	4	29.0	V
Coshocton, OH	W26030	16	303.0	C
Coshocton, OH	W26033	6	920.0	C
Coshocton, OH	W26036	6	4,580.0	C
Fennimore, WI	W31003	6	52.5	V
Fennimore, WI	W31004	7	171.0	C
Riesel, TX	W42004	6	579.0	C
Hastings, NE	W44001	5	481.0	C
Monticello, IL	W61001	8	45.5	C
Oxford, MS	W62001	7	2000.0	V
Oxford, MS	W62007	6	511.0	C
Reynolds, ID	W68003	12	7,846.0	C
Reynolds, ID	W68011	9	306.0	C
Chickasha, OK	W69008	15	4,846.0	C
Chickasha, OK	W69009	16	563.0	C
Chickasha, OK	W69028	3	1620.0	V
Chickasha, OK	W69032	11	44.3	C
Chickasha, OK	W69042	10	23.7	C
Treynor, IA	W71001	11	74.5	C
Tifton, GA	W74004	9	3936.0	V
Tifton, GA	W74009	4	646.0	C
Ahoskie, NC	W75003	7	2,368.0	C
Ahoskie, NC	W75004	8	1664.0	V

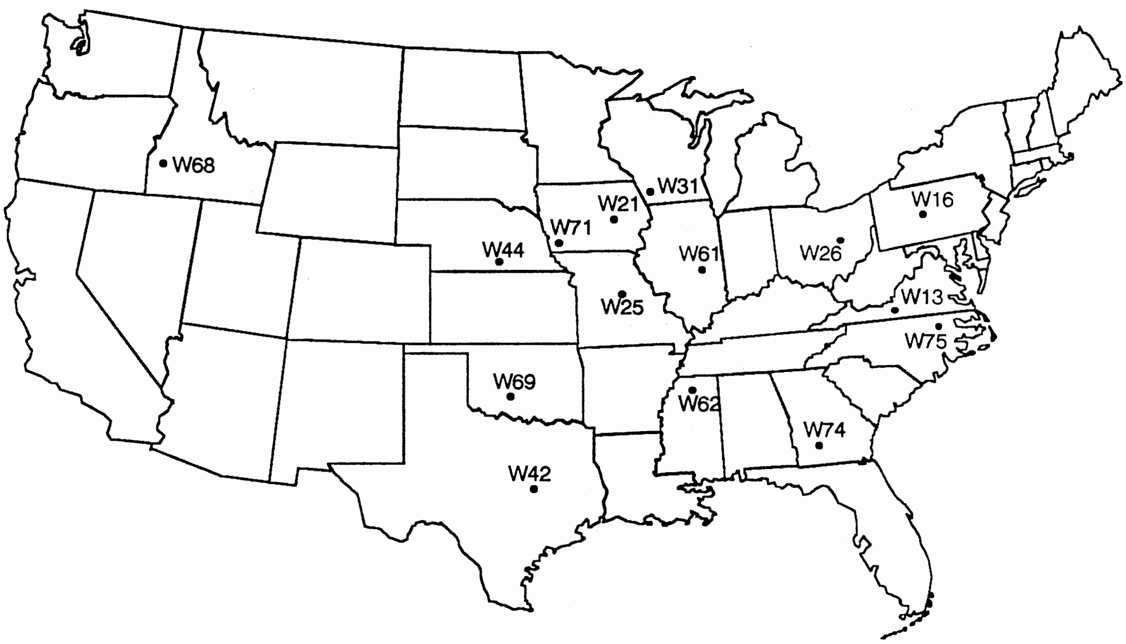


Figure 3. Geographic Location of the Test
and Validation Watersheds

TABLE II

GEOMORPHIC PARAMETERS

Area
Perimeter
Time of Concentration
Main Channel Length
Maximum Basin Length
Maximum Basin Width
Maximum Elevation Difference
Overland Slope
Channel Slope
Stream Order
Average Bifurcation Ratio
Relative Relief
Relief Ratio
Ruggedness Number
Elongation Ratio
Circularity Ratio

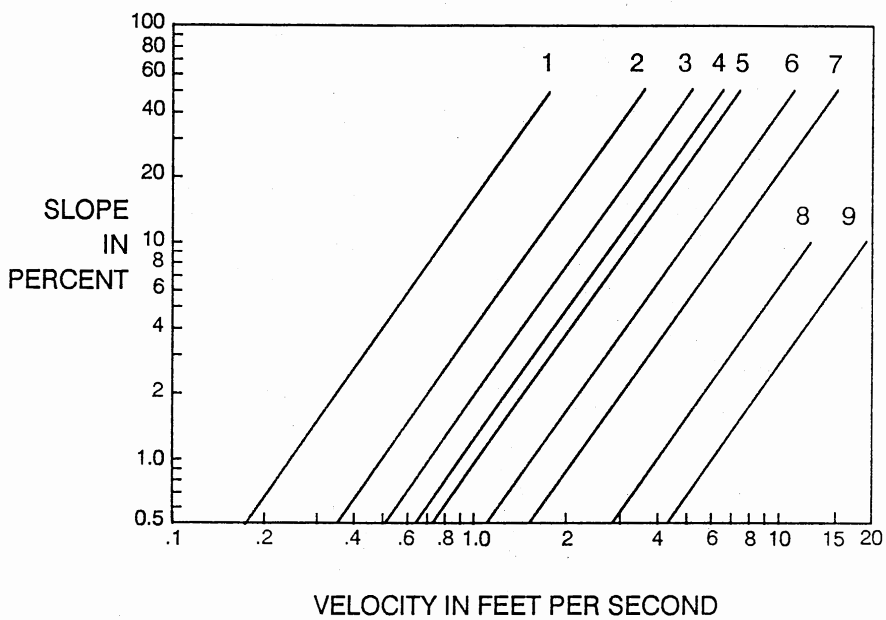
parameters. These geomorphic parameters estimated are summarized in Table II.

The area and perimeter of the basin were directly measured by the digitizer. The main channel length is defined as the length from the outlet to the beginning of the uppermost stream. The maximum watershed length is the longest length measured from the outlet. The maximum watershed width is the longest length of the watershed perpendicular to the maximum length.

The time of concentration was calculated in the digitizing program. The calculations are based on a modified SCS (1972) upland method that has been extended to account for flows in larger upland channels. Simple, but rough estimates of velocity in these channels are shown by Curves 8 and 9 in Figure 4. These curves were estimated using Mannings equation for flow depths of two and four feet for small and large upland channels, respectively. A Manning coefficient of 0.04 and a hydraulic radius of 1/2 the flow depth were assumed. Decisions on the type of ground cover and transitions between conditions were subjective. The time of concentration was obtained by summing the individual travel times for each flow segment.

Two slopes were calculated, the average channel slope and the average overland slope. The average channel slope is calculated by dividing the elevation difference between the uppermost stream and the outlet by the main channel length. The average overland slope was measured by averaging the slopes from five arbitrary locations. The slopes were calculated by drawing a line perpendicular to several contour lines and dividing the elevation difference by the length of the perpendicular lines.

The stream order, based on Strahler's ordering scheme, and the bifurcation ratio were estimated using



1 -	Forest with Heavy Ground Litter & Hay Meadow (Overland Flow)	$V=0.25\ S^{.5}$
2 -	Trash Fallow, Minimum Tillage, Contour or Strip Crop, & Woodland	$V=0.50\ S^{.5}$
3 -	Short Grass Pasture (Overland Flow)	$V=0.80\ S^{.5}$
4 -	Cultivated, Straight Row (Overland Flow)	$V=0.90\ S^{.5}$
5 -	Bare, Untilled, & Alluvial, Western Mountain Regions	$V=1.00\ S^{.5}$
6 -	Grassed Waterway	$V=1.50\ S^{.5}$
7 -	Paved Area, Small Upland Gullies	$V=2.00\ S^{.5}$
8 -	Small Upland Channels	$V=4.00\ S^{.5}$
9 -	Large Upland Channels	$V=6.00\ S^{.5}$

Figure 4. Chart for Channel Velocity

methods discussed in Chapter II. The drainage density was calculated by measuring the length of all the streams within the basin using the digitizer and dividing the summation of the stream lengths by the areaof the basin. The relief ratio was estimated by the ratio of the maximum elevation difference to the maximum length of the watershed. The relative relief was defined as the ratio of the maximum elevation difference to the perimeter of the basin (Brakensiek et al., 1979). The ruggedness number was defined as the drainage density multiplied by the maximum elevation difference within the basin (Brakensiek et al., 1979).

The elongation ratio is the ratio between the diameter of a circle with the same area as the basin to the maximum length of the basin. This ratio will approach one as the shape of the basin approaches a circle. The circularity ratio is very similar to the elongation ratio. It is defined as the ratio of the circumference of a circle of the same area as the basin to the basin perimeter (Brakensiek et al., 1979).

Rainfall Parameters

A summary of the estimated rainfall parameters is given in Table III. The computer code to estimate these parameters is given in Appendix B.

The total rainfall depth and total depth of runoff

TABLE III.

RAINFALL PARAMETERS

Cumulative Rainfall Depth
Cumulative Rainfall Excess Depth
Duration of Rainfall
Duration of Rainfall Excess
Peak Rainfall Intensity
Peak Excess Intensity
Duration of Peak Rainfall Intensity
Duration of Peak Excess Intensity
Rainfall Standard Deviation
Excess Rainfall Standard Deviation
Rainfall Coefficient of Skew
Excess Rainfall Coefficient of Skew

total depth of runoff. Once the potential maximum retention for the storm is calculated, the depth of the rainfall excess can be calculated for each rainfall breakpoint from Eq. 22. If the cumulative excess depth is less than 0.2S, the incremental excess depth was considered to be zero. The excess depth is considered for all the discrete time increments. This is done by determining the cumulative depth at the beginning and end of the time increment. From these values, the excess depth is determined for the increment.

The standard deviation and coefficient of skew for

for each event are known. Knowing these two values, the potential maximum retention S can be calculated from the equation (Wilson et al., 1984)

$$S = 5 \; [\; I + 2z_e \; - \; (\; 4z_e^{\;2} \; + 5Iz_e \;)^{1/2}] \tag{60}$$

where I is the total depth of rainfall and z_e is the the rainfall and excess rainfall are calculated using the second and third moments around the mean (Haan, 1977). The standard deviation is determined by estimating the variance of the rainfall event and taking the square root to the variance. The variance is determined by (Haan, 1977)

$$\text{VARIANCE} = \frac{\int (\; t - t' \;)^2 dA}{\int \; dA} \tag{61}$$

where dA is depth of the rainfall or rainfall excess burst, t is the average real time within the burst, and t' is the center of mass of the rainfall.

The skew coefficient is estimated by calculating the third moment of the rainfall event and dividing by the standard deviation raised to the third power. The third moment is estimated by (Haan, 1977)

$$\text{THIRD MOMENT} = \frac{\int (t - t')^3 dA}{\int dA} \tag{62}$$

where the terms are as previously defined. The statistical characteristics are calculated for both the rainfall and the excess rainfall.

Storage Coefficient Calibration

A dimensionless storage coefficient is needed to use the dimensionless IUH developed in Chapter III. The objective of this study is to estimate K^* with one or more geomorphic and/or rainfall parameters that are given in Tables II and III. This objective requires that K^* be first determined for the calibration storms. Procedures to complete this step are discussed here. Procedures to obtain a predictive relationship are discussed in the next section.

Before the calibration procedure of each rainfall-runoff event, the base flow was subtracted from the observed hydrograph. For lack of a better method and information concerning baseflow in the watersheds, a straightline approximation was used to estimate the base flow. Usually surface runoff models are only mildly sensitive to the base flow separation method (Bates and Davies, 1988).

As discussed in Chapter II, a number of different techniques have been proposed to determine observed unit hydrographs. None of these techniques are ideally suited for estimating IUHs from complex storm patterns. Therefore, the observed IUHs for the calibration events were not calculated directly. Instead K^* was calibrated using the general optimization procedure of Brent (Press et al., 1986). Different K^* values were selected using Brent's algorithm. For each K^*, an IUH was determined using Eqs. 55, 57, and 59. The predicted runoff hydrograph was then determined by convoluting this IUH with the observed storm pattern as discussed in Chapter III. The squared deviation between the observed an predicted peak flow rate was determined. The process was continued until the minimum square deviation was reached, thus providing the optimum K^* for the event.

Predictive Relationship

Multiple linear regression, multivariate regression analysis, and nonlinear regression were used to develop a predictive relationship for K^* as a function of geomorphic and rainfall parameters. These analyses were performed using the statistical package SYSTAT.

Multiple Linear Regression

Multiple linear regression was used to examine the relationship between K^* and geomorphic and rainfall parameters. The model was initially tested using all the parameters as independent variables. A stepwise regression was then performed in order to identify significant independent variables.

Multivariate Regression Analysis

A multivariate analysis was used to group parameters into physically significant groups. Combinations of parameters from each group can be used as independent variables in developing a prediction equation.

In multivariate analysis, the correlation matrix of geomorphic and rainfall parameters is manipulated using principal component analysis and varimax rotation (Haan, 1977). The interpretation of the principal components is done with the factor loading matrix. A high factor loading indicates a high correlation with the variable or a strong linear similarity between the component and the variable. From the factor loading matrix, the components and the variables within the components are chosen.

Nonlinear Regression Analysis

A nonlinear regression analysis was performed on the data. The predictive equation was based on a modified form of the Weibull cumulative distribution function. This equation was selected because of its simple form, variety of possible shapes, and well behaved nature. The Weibull distribution function can be written as (Haan, 1977)

$$f(X) = \frac{\beta}{\eta}\left(\frac{X}{\eta}\right)^{\beta-1}\exp\left[-\left(\frac{X}{\eta}\right)^{\beta}\right] \qquad \text{for } X>0 \qquad (63)$$

where β and η are determined by nonlinear regression analysis. Both β and η must be greater than zero.

The X term in the Eq. 63 will incorporate the geomorphic and/or rainfall parameters. The parameters used in the nonlinear regression will be the parameters identified in the stepwise regression and multivariate analysis as being significant. The parameters will be developed into dimensionless variables and tested in the nonlinear equation. Several forms of X using the dimensionless variables will be tried including additive, exponential, and power functions.

The regression equation with the best statistical fit and that makes physical sense will be chosen. It should be emphasized that the equation must make sense physically.

Validation Procedure

The dimensionless IUH developed in Chapter III, used in conjunction with the predictive relationship for K^*, will be tested for accuracy using thirty-five, independent rainfall-runoff events previously identified in Table I. Absolute accuracy will be evaluated by visually comparing predicted and observed hydrographs and by statistically comparing predicted and observed peak flow rates. A convenient statistic for evaluating accuracy is the absolute value of fractional error defined as

$$Err = ABS(\ Obs - Pre)/Obs \qquad (64)$$

where Err is the fractional error, Obs is the observed peak flow rate, and Pre is the predicted value.

Relative accuracy will also be evaluated by comparing predicted values to those obtained by the SCS dimensionless hydrograph. The SCS method was selected for examining relative accuracy because it is widely used and because Howard and Meadows (1981) found that it was the most accurate of four techniques considered in predicting peak flows for 270 rainfall-runoff events on 38 watersheds.

Runoff hydrographs with the proposed IUH method were predicted using the convolution steps previously outlined in Chapter III. The storage coefficient, K^*, was predicted using the predictive relationship developed with procedures discussed in the previous section. All simulations were based on the observed rainfall pattern and calibrated curve number.

Runoff hydrographs with the SCS method were predicted with similar convolutions steps using the SCS dimensionless unit hydrograph instead of the proposed IUH. The SCS dimensionless unit hydrograph was estimated by the equation (Barfield et al., 1981)

$$\frac{q(t)}{q_p} = \left[\frac{t}{t_p} \exp^{1-t/tp}\right]^{3.75} \tag{65}$$

where $q(t)$ is the discharge at time t, t_p is the time to peak, and q_p is the peak discharge (Eq. 21). The time to peak was estimated using the SCS relationship

$$t_p = 0.6\ t_c + \Delta t/2 \tag{66}$$

where t_c is the time of concentration and Δt is the convolution time step which corresponds to the duration of incremental rainfall excess bursts.

CHAPTER V

RESULTS AND DISCUSSION

Introduction

This chapter is divided into two sections. In the first section, the storage coefficient calibration and model development are discussed. Model validation is discussed in the second section.

Model Development

Storage Coefficient Calibration

The storage coefficient, K^*, was calibrated using the method discussed in Chapter IV. Initially there were 205 rainfall-runoff events in the calibration data set. There were indications that some of this data may be incorrect, probably due to instrumentation errors. As an example, some storms had some runoff occurring before rainfall had begun. For seven events, the depth of excess rainfall exceeded the depth of rainfall. This could be attributed to either subsurface flow separation techniques or instrumentation problems. These seven events were removed from the data set.

Using calibration procedures, a storage coefficient was estimated for the remaining rainfall-runoff event. A plot of the observed peak flow rates versus predicted peak flow rates using calibrated K^* values is shown in Figure 5. A perfect fit between predicted and observed values would plot as a 45° line. As shown by Figure 5, calibration procedures estimated storage coefficients that produced peak flow rates that were very similar to those observed.

Although calibration procedures worked very well, some of the storage coefficients were greater than five. As shown by Figure 2 in Chapter III, the dimensionless IUH was insensitive to storage coefficients greater than five. Therefore, a large change in K^* is required to significantly affect the predicted peak flow rate. Conversely, a small change in observed peak flow rate due to base flow separation could cause a significant change in K^*. Visual inspection of the observed hydrograph for events with large K^* indicated that subsurface flow was probably significant and more rigorous base flow separation techniques were needed. Thus, watersheds with significant subsurface flows were removed from the data set. In proposed IUH method, subsurface flow is assumed negligible in comparison to surface flow.

With the removal of questionable data, the calibration data set was reduced to 164 events having

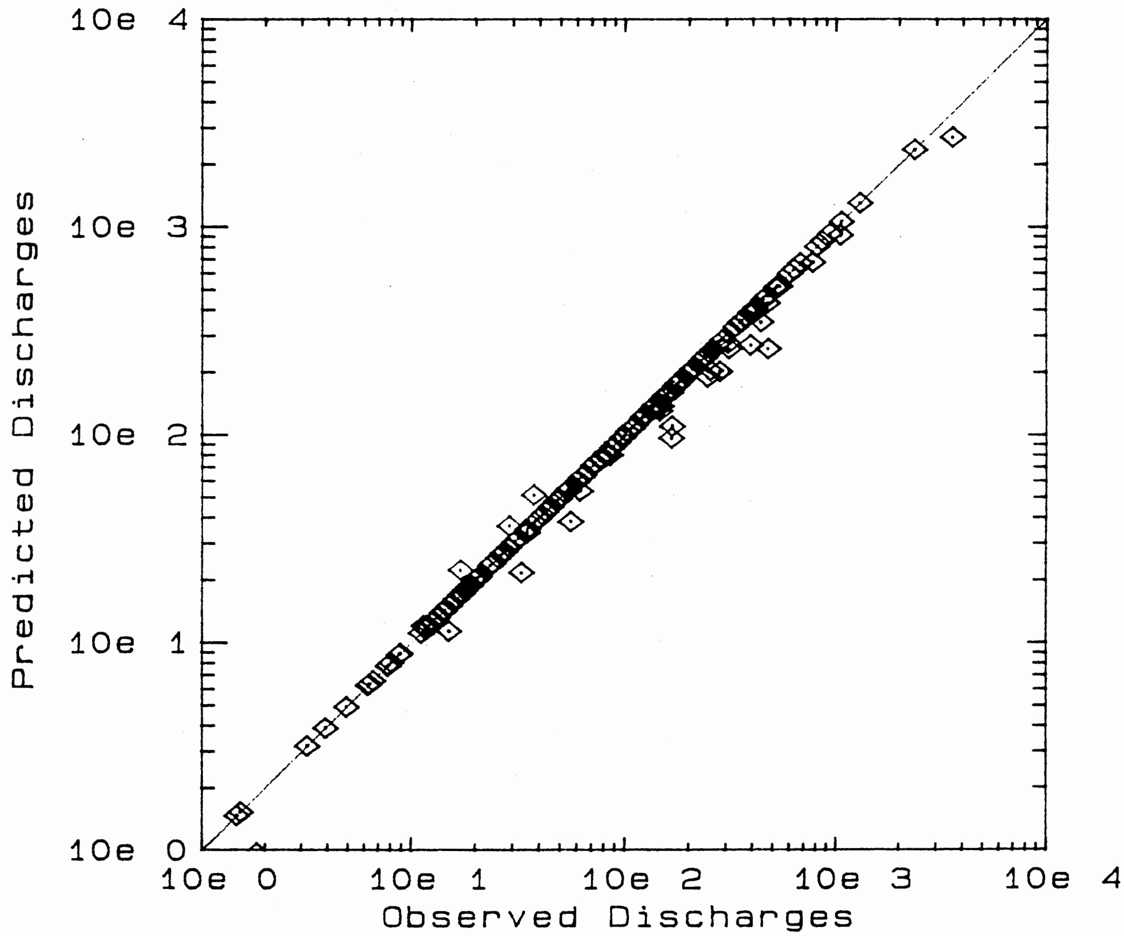


Figure 5. Observed Discharges Versus Discharges Predicted Through the K* Calibration Technique

storage coefficients between 0.02 and 4.66. In Table I, watersheds W13002 in Virginia and W68011 in Idaho were removed because of significant subsurface flow components.

For the reduced data set, the mean storage coefficient was determined to be 1.16 with a standard deviation of 1.19. The median storage coefficient value was 0.81. Thus, 50 percent of the calibration watersheds had a storage coefficient value greater than 0.81 and 50 percent of the watersheds had a value less than 0.81.

Four probability distributions were applied to the data set: normal, lognormal, log pearson type III, and extreme value type I. Of these four distributions, the extreme value type I distribution appeared to fit the data the best. Figure 6 shows a probability plot of storage coefficients where exceedance probabilities were calculated using standard procedures (Haan, 1977). A curve representing the extreme value type I distribution is plotted along with the observed data.

The results shown in Figure 6 were not used in developing a predictive equation. They are presented for possible future studies.

Storage Coefficient Estimation

A predictive equation for the storage coefficient

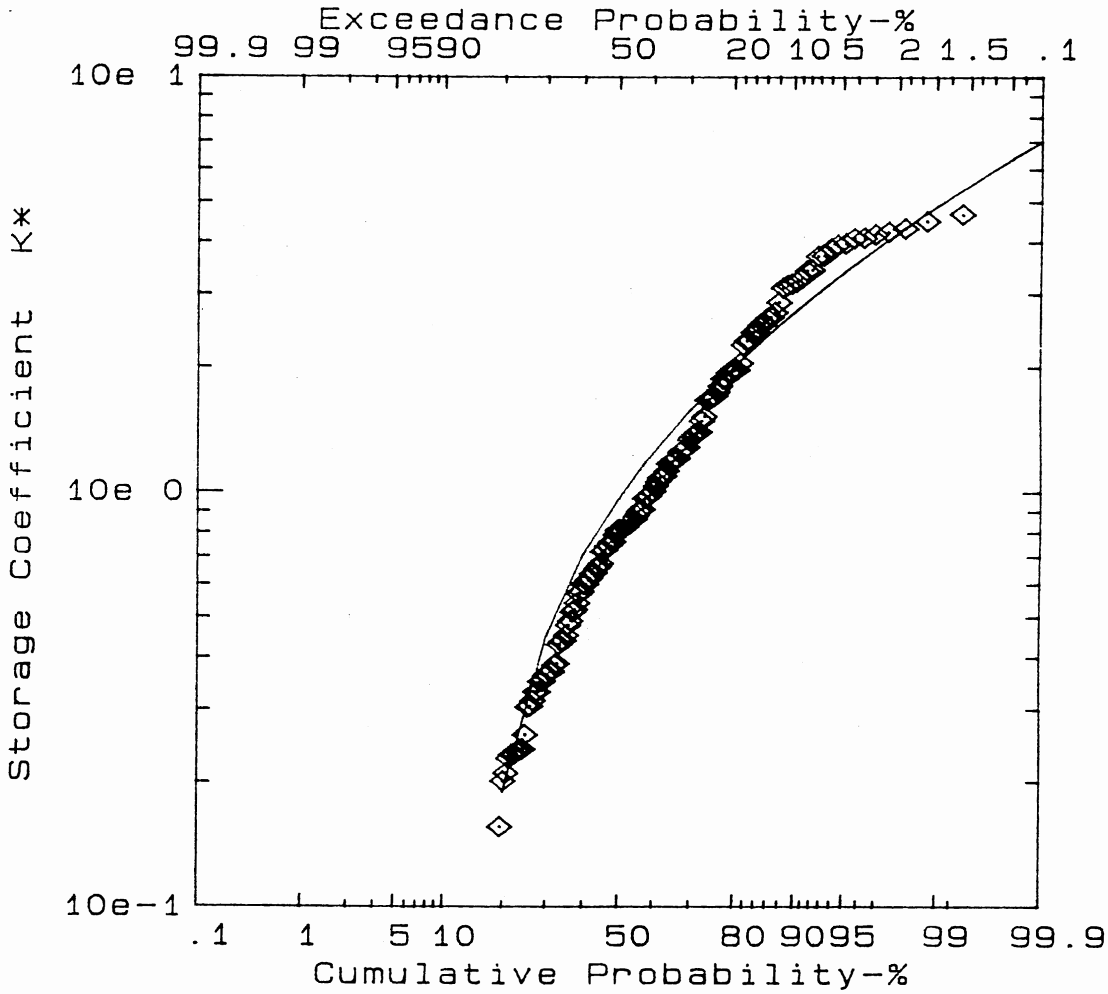


Figure 6. Extreme Value Type I Distribution for the Calibrated K^* Values

was developed from the calibrated data. A correlation matrix of the data set was obtained prior to regression analysis. In Table IV the correlation between storage coefficients and geomorphic and rainfall parameters are presented. It can be seen that none of parameters are highly correlated to the storage coefficient. A plot of K^* versus channel slope is shown in Figure 7.

Some of the variability in the storage coefficients might be caused by spatially varied rainfall depths. The assumption of constant rainfall depth is probably invalid for some of the events. Storm movement across the watershed could also affect observed storage coefficients. In a long, large watershed, the observed K^* values might vary significantly among storms that move either away from or toward the watershed outlet.

Linear regression failed to provide an acceptable predictive equation for estimating the storage coefficient. A regression model including thirty variables was tested and yielded an R^2 of 0.47. A stepwise analysis was also applied, yielding an R^2 of 0.41. The stepwise model identified the following variables to be included in a regression model; excess duration, rainfall skew coefficient, depth rainfall, watershed area, main channel length, maximum watershed length, maximum elevation difference, channel slope, relief ratio, relative relief, ruggedness number,

TABLE IV.

PARAMETER CORRELATIONS TO K^*

Parameter	Correlation
Storage Coefficient, K^*	1.000
Potential Retention, S	-0.161
Rainfall Duration	0.201
Peak Intensity Duration	0.209
Rainfall Excess Duration	0.227
Peak Excess Intensity Duration	0.203
Peak Rainfall Intensity	-0.159
Peak Excess Intensity	-0.111
Standard Deviation Rainfall	0.187
Standard Deviation Excess	0.189
Skew Coefficient Rainfall	-0.149
Skew Coefficient Excess	-0.172
Depth Rainfall	-0.093
Depth Excess	0.043
Watershed Perimeter	-0.044
Watershed Area	-0.028
Main Channel Length	-0.069
Maximum Length	-0.064
Maximum Width	-0.016
Maximum Elevation Difference	0.071
Overland Slope	0.087
Channel Slope	0.192
Stream Order	0.080
Bifurcation Ratio	0.098
Drainage Density	0.030
Relief Ratio	0.145
Relative Relief	0.113
Ruggedness Number	0.122
Elongation Ratio	0.110
Circularity Ratio	0.104
Time of Concentration	0.044

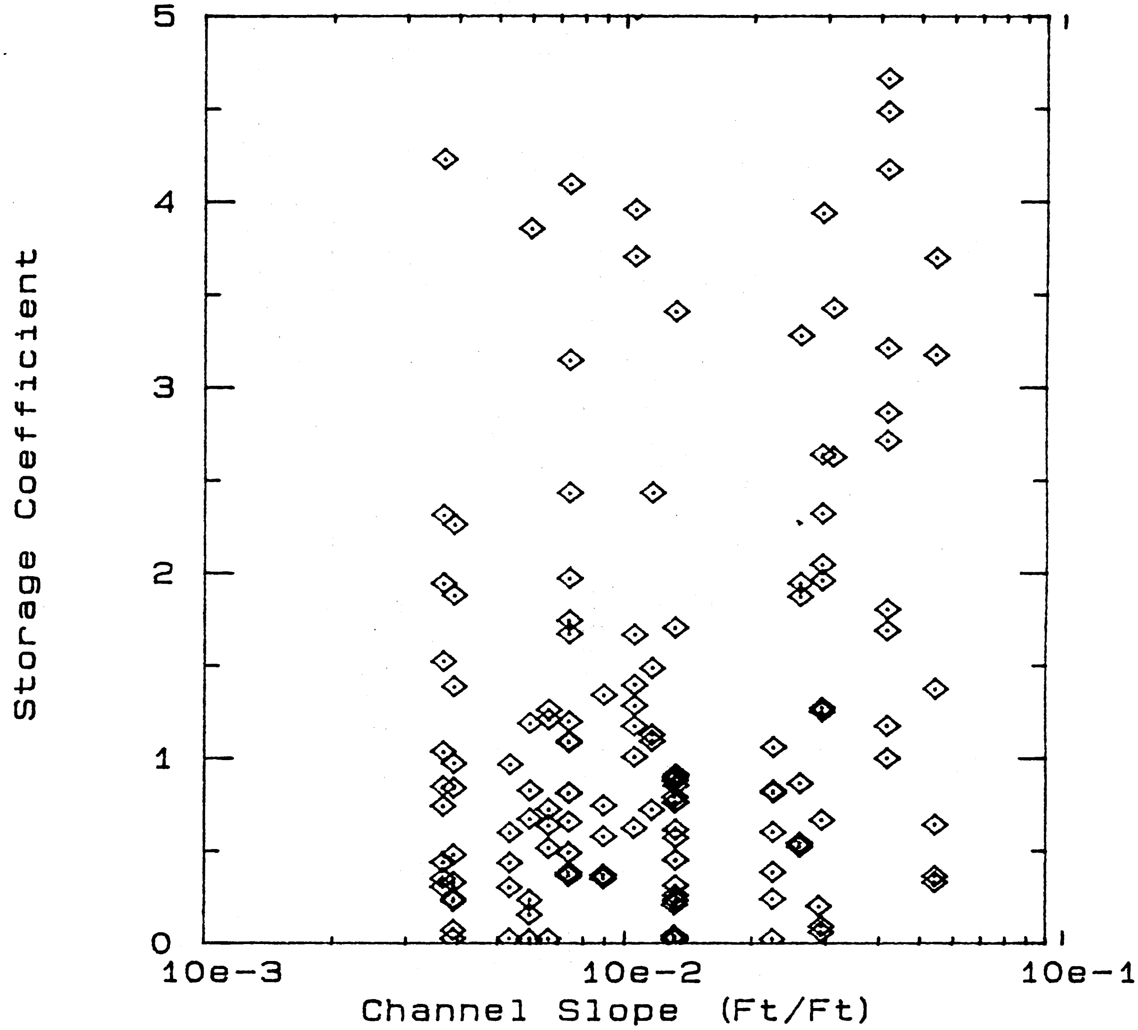


Figure 7. Calibrated K* Versus Measured Channel Slope

elongation ratio, and time of concentration.

Multivariate analysis was applied to the data set in an effort to identify significant components. From this analysis three components were identified. The first component was a size component. Watershed area, perimeter, main channel length, maximum watershed length, maximum watershed width were identified as having high correlations. The second component was a gradient component. It was highly correlated with channel slope, relief ratio, and relative relief. Component three was highly correlated with rainfall duration, excess duration, rainfall duration's standard deviation, and excess duration's standard deviation. Models using various combinations of variables in these three components were tried. None produced a model that adequately predicted the storage coefficient.

Since none of the linear models provided adequate results, nonlinear models were tried. The nonlinear models were based on the Weibull distribution discussed in Chapter IV. The modified form of the cumulative Weibull function used to predict K^* is

$$K^* = 5 * \left\{ 1 - \exp\left[- \left(\frac{X}{\eta} \right)^{\beta} \right] \right\} \tag{67}$$

where β and η are constant determined by regression and X is the function incorporating the independent variables. Values predicted by Eq. 67 are bounded between zero and five.

Several formats of the function X were tried, additive, exponential, and power functions. Initially, independent variables comprising the function X were those identified as being significant in the stepwise regression and multivariate analysis. Many different combinations of function and independent variables were examined. A model with a relatively good fit was developed using a dimensionless size component, defined as Area/(Main Channel Length*Maximum Width), and a channel slope component. This model, however, predicted a trend that K^* decreased with an increase in watershed area, which was contrary to expected results. Additional models were tried using either the size component or the gradient component. The gradient component was a better model.

The summation of the residuals squared referred to here as LOSS, is an indicator of the accuracy of a nonlinear model. The LOSS value for the predictive equation using channel slope as the independent variable was 342.3. The storage coefficient is predicted as

$$K^* = 5*[1-\exp(-W)] \tag{68}$$

where

$$W = [\exp(-15.426*CS)]^{14} \tag{69}$$

where CS is the channel slope.

Attempts were made to incorporate another component into Eq. 69. This was done by holding W constant in Eq. 68 and applying regression analysis, using other components as the independent variables. A rainfall component was found to improve the estimate of the storage coefficient. Eq. 68 is rewritten as

$$K^* = 5*[1-\exp(-W)+R] \qquad \text{for } K^*>0.1 \qquad (70)$$

where

$$R = 0.092*ln\left(\frac{0.447\ \text{Dur}}{T_c} \right) \qquad (71)$$

where Dur is the storm duration and T_c is the time of concentration. This predictive equation has a LOSS of 266.6, compared to the LOSS of 342.3 for Eq. 68. This equation has the potential to predict a negative K^*. In this case, the value of K^* is set equal to 0.1, or

$$K^*=0.1 \qquad \text{for } K^*<0.1. \qquad (72)$$

A plot of predicted versus observed in shown in Figure 8.

The predictive model given by Eq. 70 appears to be rational. Predicted K^* is inversely proportional to slope, that is, storage effects become larger for milder channel slopes. Interpretation of the rainfall parameter is not as straightforward. For a short

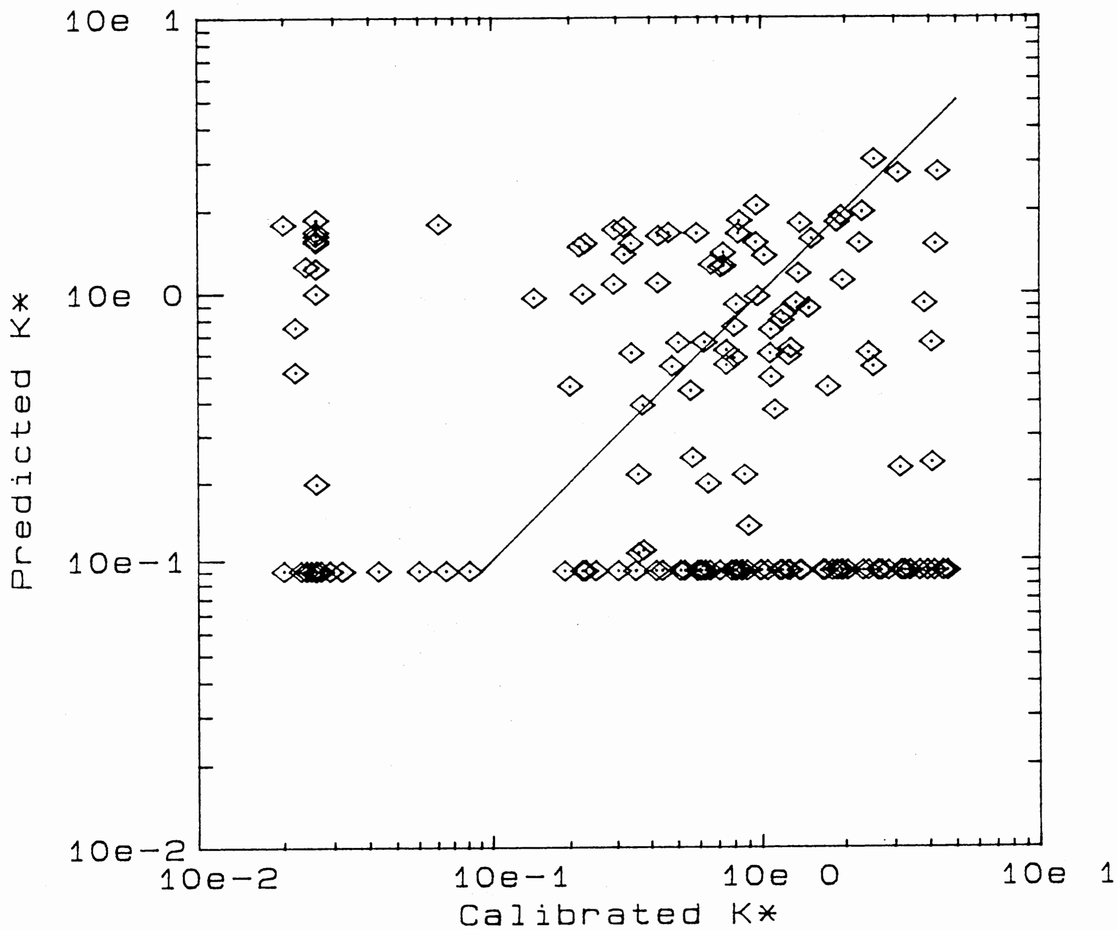


Figure 8. Predicted Versus Calibrated K^* Values

duration storm, where $DUR<0.447t_c$, R defined by Eq. 71 will be negative and hence decrease the value of K^*. Otherwise R will increase storage effects. In general, the ratio of DUR/t_c can be conceptually related to channel storage. For a small ratio, runoff from areas near the watershed outlet will exit the channel before it can be combined with runoff from other upslope areas. For a larger ratio, however, it is possible for a channel to include runoff from downslope and upslope areas and thereby increase flow depth and channel storage.

Model Testing

Seven watersheds initially were identified to test the model but two were removed because of the presence of significant subsurface flow within the watershed. Watersheds W68014 (Idaho) and W13013 (Virginia) were removed leaving five watersheds with 35 rainfall-runoff events for the validation of the model. These watersheds were removed before the testing procedure because of their similarity to the watershed that were removed from the calibration data set.

The peak discharge for both the proposed IUH and SCS methods are shown in Figure 9. Table V shows a comparison between the fractional error produced by the proposed IUH and SCS method in estimating the peak discharge.

TABLE V.

FRACTIONAL ERROR

	Mean Error	First Quartile	Median Error	Third Quartile	Maximum Error
IUH	0.36	0.11	0.32	0.52	1.06
SCS	1.19	0.27	0.96	1.96	3.24

The proposed IUH method typically predicted a better estimate of the peak discharge than the SCS method. Table V shows that the median fractional error for the proposed IUH method was 0.32 compared to 0.96 for the SCS method. Thus 50 percent of the validation watersheds had a fractional error less than 0.32 in the prediction of the peak discharge.

Figure 10 shows a typical fit of the proposed IUH method and the SCS method to observed values. It can be seen that the generalized method take into account the storage of the watershed, lagging the hydrograph to fit the observed data more closely. Appendix C contains plots of the 35 validation test events. Based on visual inspection of the plots in Appendix C, the generalized model matched the peaks and shape of the observed

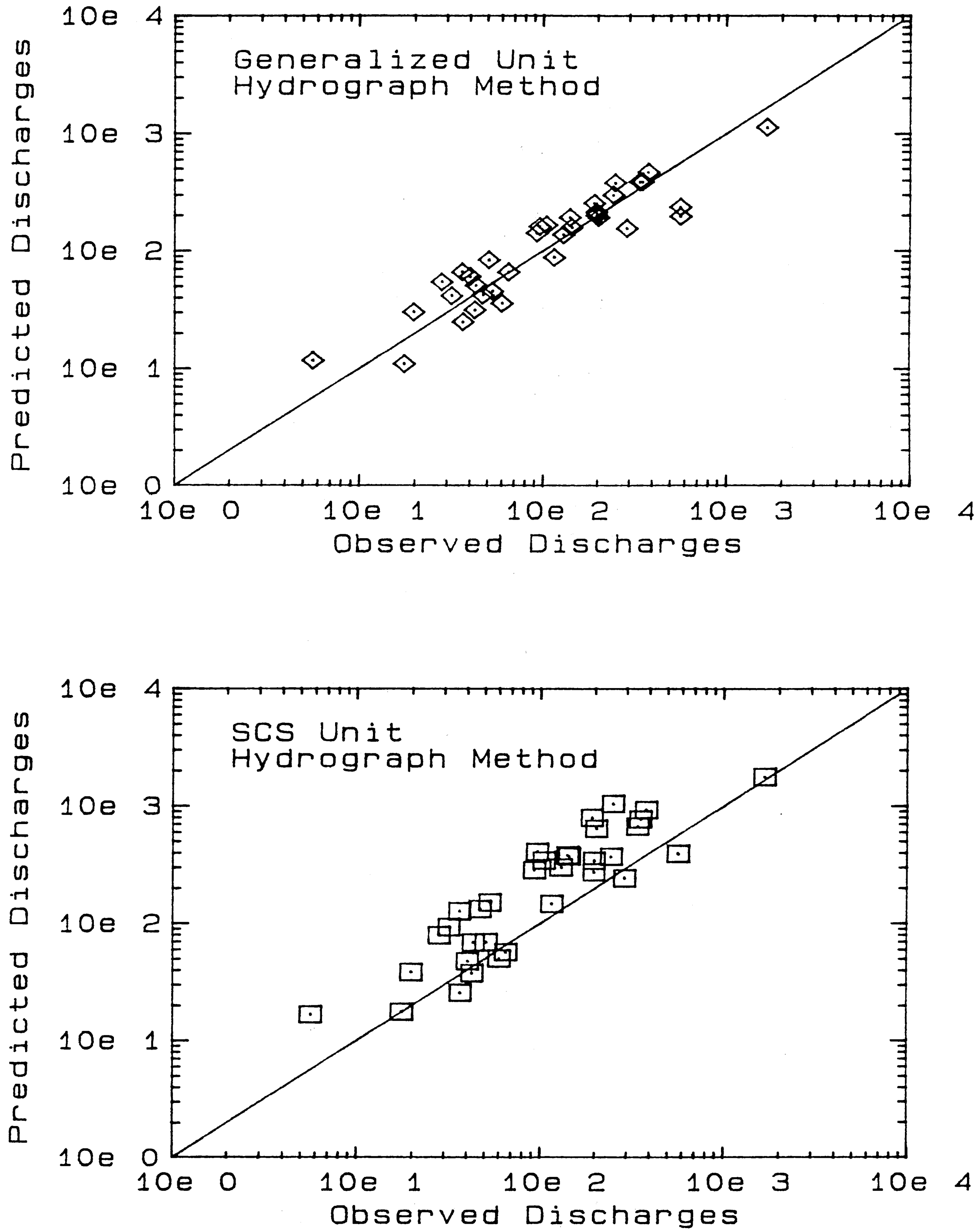


Figure 9. Comparison of Predicted Peak Flows to Observed Peak Flows for the Proposed IUH and the SCS Unit Hydrograph Methods

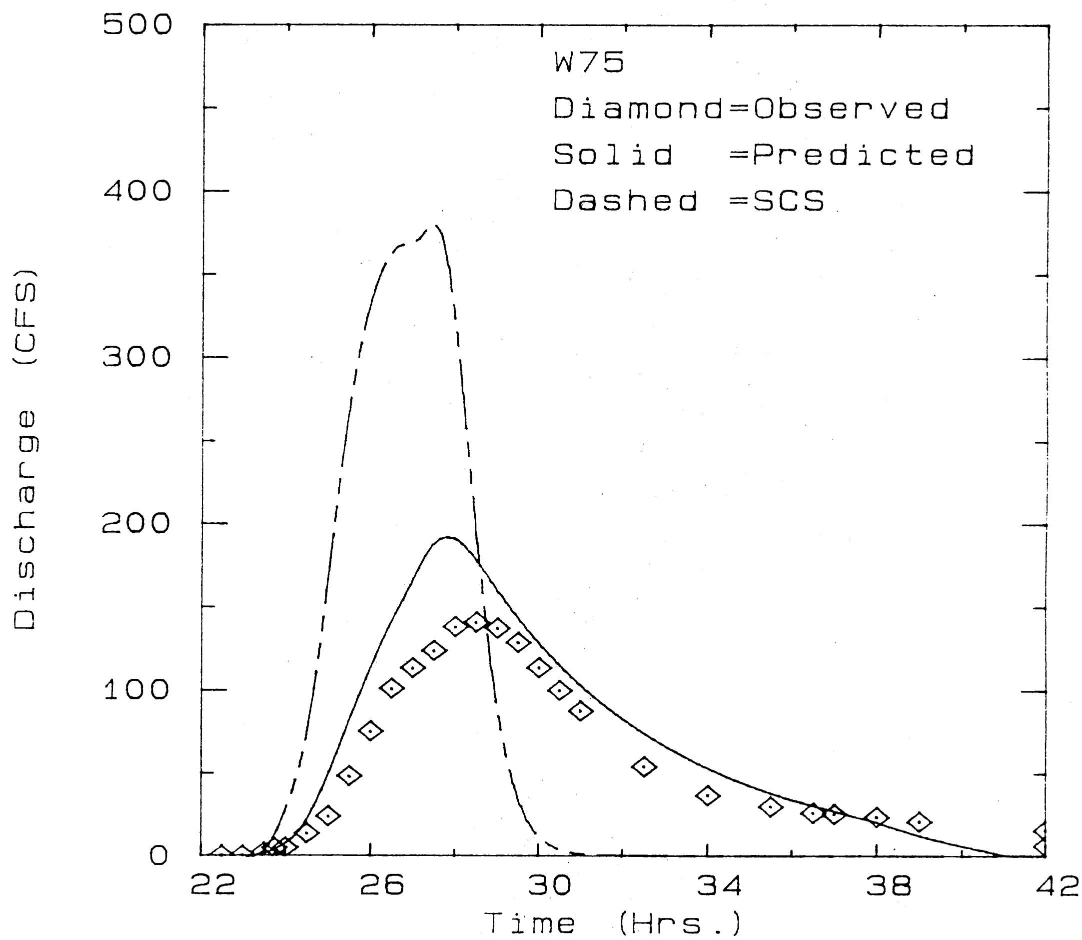


Figure 10. Typical Fit of the Proposed IUH and the SCS Unit Hydrograph

hydrograph better than the SCS method.

CHAPTER VI

SUMMARY AND CONCLUSIONS

The goal of this study was to develop and evaluate a unit hydrograph method for predicting runoff hydrographs. A generalized IUH was first developed using Clark's concepts (1943). A key component in this theory is the value of dimensionless storage coefficient K^*. The dimensionless storage coefficient K^* was estimated through the use of an optimization scheme based on Brent's Method where K^* was minimized based on the square deviation of the predicted and observed peak discharges. A predictive equation for K^* was developed using the optimal K^* values and a nonlinear regression model. Predicted K^* values are used in conjunction with the proposed IUH to define the unit hydrograph, which can be convoluted to produce an outflow hydrograph for a given rainfall event.

Thirty-five rainfall-runoff events were used to test the validity and accuracy of the generalized model. The hydrograph produced by the generalized model were compared to the observed values and values estimated by the SCS method. Based on the results in Chapter V the following conclusions were drawn:

1. The K^* calibration procedure did a good job of determining the K^* value of the rainfall-runoff events.

2. The predictive equation suggests that K^* is controlled by the channel slope and storm characteristics.

3. The accuracy of the K^* prediction equation is marginal.

4. The proposed IUH, however, still provides a better estimate of peak discharge and hydrograph shape than does the SCS hydrograph method.

5. The generalized model is limited to watersheds where the subsurface flow is negligible in comparison to the surface flow.

Recommendations for Future Research

1. Test the model over a larger range of watersheds, varying in size and location.

2. Develop a component to be included in the K^* prediction equation that will account for rainfall pattern and movement across the watershed.

3. Develop a term to be included in the K^* prediction equation to account for subsurface flow within the basin.

REFERENCES

Agnese, C., F. D'Asaro and G. Giordano. 1988. Estimation of the Time Scale of the Geomorphologic Instantaneous Unit Hydrograph From Effective Streamflow Velocity. Water Resources Research. 24(7):969-978.

Barfield, B. J., R. C. Warner and C. T. Haan. 1981. Applied Hydrology and Sedimentology for Disturbed Areas. Oklahoma Technical Press. Stillwater, Oklahoma.

Bates, B. C., P. K. Davies. 1988. Effect of Baseflow Separation Procedures on Surface Runoff Models. Journal of Hydrology. 103:309-322.

Boyd, M. J., D. H. Pilgrim and I. Cordery. 1979. A Storage Routing Model Based On Catchment Geomorphology. Journal of Hydrology. 42:209-230.

Brakensiek, D. L., H. B. Osborn and W. J. Rawls, coordinators. 1979. Field Manual for Research in Agricultural Hydrology. U. S. Department of Agriculture, Agriculture Handbook 224. Washington, D.C.

Chow V. T., V. C. Kulandaiswamy. 1982. The IUH of General Hydrologic System Model. Journal of the Hydraulics Division, American Society of Civil Engineers. 108(HY7):830-845.

Chow, V. T., D. R. Maidment and L. W. Mays, 1988. Applied Hydrology. McGraw-Hill, Inc. New York.

Clark, C. O. 1943. Storage and The Unit Hydrograph. Proceedings of American Society of Civil Engineers. 69:1419-1447.

Dooge, J. C. I. 1959. A General Theory of the Unit Hydrograph. Journal of Geophysical Research. 64(2):241-256.

Dooge, J. C. I. 1973. Linear Theory of Hydrologic Systems. U. S. Department of Agriculture, Technical Bulletin 1468. Washington, D.C.

Gray, D. M. 1962. Derivation of Hydrographs for Small Watersheds From Measurable Physical Characteristics, Research Bulletin 506. Agricultural and Home Economics Experiment Station, Iowa State University of Science and Technology. Ames, Iowa.

Gupta, V. K., E. Waymire and C. T. Wang. 1980. A Representation of an Instantaneous Unit Hydrograph From Geomorphology. American Geophysical Union. 16(5):855-862.

Haan, C. T. 1977. Statistical Methods in Hydrology. Iowa State University Press. Ames, Iowa.

HEC 1: Hydrologic Engineering Methods For Water Resources Development, Volume 1. 1971. The Hydrologic Engineering Center, Corps of Engineers, U. S. Army Center. Davis, California.

Horton, R. E. 1945. Erosional Development of Streams and Their Drainage Basins; Hydrophysical Approach To Quantitative Morphology. Geological Society of America Bulletin. 56:275-370.

Howard, K. M., M. E. Meadows. 1981. A Comparison of Four Synthetic Unit Hydrograph Techniques. Proceeding of the 1981 Symposium on Surface Mining Hydrology Sedimentology, and Reclamation. University of Kentucky. Lexington, Kentucky.

Hromadka, T. V., J. J. Devries. 1988. Kinematic Wave Routing and Computational Error. Journal of Hydraulic Engineering. 114(2):207-217.

Hydrologic Data for Experimental Agricultural Watersheds in the United States, 1956-1977. U. S. Department of Agriculture. Miscellaneous Publication Number 945, 994, 1070, 1164, 1194, 1216, 1226, 1262, 1330,1370, 380, 1383, 1412, 1420, 1437, 1446, 1451,1454.

Introduction To Hydrosystems Engineering, CE 255 Class Notes. 1985. Hydrosystems Laboratory, Department of Civil Engineering, University of Illinois. Urbana-Champaign, Illinois.

Kirshen, D. M., R. L. Bras. 1983. The Linear Channel and Its Effect on the Geomorphologic IUH. Journal of Hydrology. 65:175-208.

Leopold, L. B., M. G. Wolman and J. P. Miller. 1964. Fluvial Processes in Geomorphology. W. H. Freeman and Company. San Francisco, California.

Linsley, R. K., Jr., M. A. Kohler and J. L. H. Paulhus. 1982. Hydrology For Engineers. 3rd Ed. McGraw-Hill Company Inc. New York.

Mays, L. W., L. Coles. 1980. Optimization of Unit Hydrograph Determination. Journal of the Hydraulics Division. 106(HY1):85-97.

Misra, N., C. T. Haan and T. Satyanarayana. 1988. Regional Frequency Analysis of Annual Runoff For A River Basin In India. Unpublished Paper.

Muftuoglu, R. F. 1984. New Models For Nonlinear Catchment Analysis. Journal of Hydrology. 73:335-357.

O'Kelly, J. J. 1955. The Employment of Unit Hydrographs To Determine The Flows of Irish Arterial Drainage Channels. Institute of Civil Engineering Proceedings. 4(3):365-412.

Overton, D. E. 1971. Estimation of Surface Water Lag Time From The Kinematic Wave Equations. Water Resources Bulletin. 7(3):428-440.

Overton, D. E. 1966. Hydraulics in Synthesis of Upland Runoff Hydrographs. Transactions of the ASAE. 9(4):543- 549.

Press, W. H., B. P. Flannery, S. A. Teukolsky and W. T. Vetterling. 1986. Numerical Recipes The Art of Scientific Computing. Cambridge University Press. Cambridge, Massachusetts.

Rodriguez-Iturbe, I., G. Devoto and J. B. Valdes. 1979. Discharge Response Analysis and Hydrologic Similarity: The Interrelation Between the Geomorphologic IUH and the Storm Characteristics. Water Resources Research. 15(6):1435-1444.

Rodriguez-Iturbe, I., M. Gonzalez-Sanabria. 1982. A Geomorphoclimatic Theory of the Instantaneous Unit Hydrograph. Water Resources Research. 18(4):877-886.

Rodriguez-Iturbe, I., M. Gonzalez-Sanabria and G. Caamano. 1982. On the Climatic Dependence of the IUH: A Rainfall-Runoff Analysis of the Nash Model and the Geomorphoclimatic Theory. Water Resources Research. 18(4):887-903.

Rodriguez-Iturbe, I., J. B. Valdes. 1979. The Geomorphologic Structure of Hydrologic Response. Water Resources Research. 15(6):1409-1420.

Rosso, R. 1984. Nash Model Relation to Horton Order Ratios. Water Resources Research. 20(7):914-920.

Sabol, G. V. 1988. Clark Unit Hydrograph and R-Parameter Estimation. Journal of Hydraulic Engineering. 114(1):103-111.

Schwab, G. O., R. K. Frevert, T. W. Edminster and K. K. Barnes. 1981. Soil and Water Conservation Engineering. 3rd Ed. John Wiley & Sons, Inc. New York.

SCS National Engineering Handbook, Section 4, Hydrology. 1972. Soil Conservation Service, U. S. Department of Agriculture. Washington, D.C.

Selected Runoff Events for Small Agricultural Watersheds in the United States. 1960. Soil and Water Conservation Research Division. Washington D.C.

Shreve, R. L. 1966. Statistical Law of Stream Numbers. Journal of Geology. 74:17-37.

Singh, K. P. 1976. Unit Hydrographs- A Comparative Study. Water Resources Bulletin. 12(2):381-392.

Snyder, F. F. 1938. Synthetic Unit-Graphs. Transactions of the American Geophysical Union. 19:447-454.

Valdes, J. B., Y. Fiallo and I. Rodriguez-Iturbe. 1979. A Rainfall-Runoff Analysis of the Geomorphologic IUH. Water Resources Research. 15(6):1421-1434.

Viessman, W., J. W. Knapp, G. L. Lewis, and T. E. Harbough. 1977. Introduction of Hydrology. Harper & Row, Publishers. New York.

Wang, C. T., V. K. Gupta. 1981. A Geomorphologic Synthesis of Nonlinearity in Surface Runoff. Water Resources Research. 17(3):545-554.

Wilson, B. N., B. J. Barfield, A. D. Ward and I. D. Moore. 1984. A Hydrology and Sedimentology Watershed Model. Part I: Operational Format and Hydrologic Component. American Society of Agricultural Engineers 27(5):1370-1377.

Wilson, B. N. 1983. Unpublished Notes. Department of Agricultural Engineering, Oklahoma State University. Stillwater, Oklahoma.

APPENDIX A

CALIBRATION PROGRAM

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20 '**************************************************
30 '***** Bill Brown    M.S. Thesis Program # 2 ****
40 '**************************************************
50 '**************************************************
60 '***** STORAGE COEFFICIENT OPTIMIZATION      *****
90 '**************************************************
95 KEY OFF
100 DIM PTIME(200),PRATE(200),PACCU(200)
110 DIM RTIME(300),RRATE(300),RACCU(300),CFS(300)
113 DIM P(500),CUMZ(500)
116 DIM FLOW(2000),UH(2000),OUTF(2000)
120 CLS
130 INPUT "What is the watershed I.D. "; ID$
133 INPUT "Which drive contains the rainfall-runoff data
";DRIVER$
135 INPUT "Which drive contains the watershed data
";DRIVEW$
137 INPUT "Which drive contains the rainfall statictical
data   ";DRIVES$
139 INPUT "Which drive do you wish to results to be
written to  ";DRIVEF$
140 INPUT "What is the first storm  ";STORM1
145 INPUT "What is the last storm ";STORM2
150 FOR J = STORM1 TO STORM2
160   G$ = STR$(J)
170   Q$ = MID$(G$,2)
180   IDP$ = DRIVER$ + ID$ + ".P" + Q$  ' Assign the
file names for the
190   IDR$ = DRIVER$ + ID$ + ".R" + Q$  ' precipitation
and runoff data files.
195   'Read in the rainfall and runoff data for the
watershed.
200   OPEN IDP$ FOR INPUT AS #1
210   INPUT #1, ROWP
220   FOR K = 1 TO ROWP  'Input the precipitation data.
230     INPUT #1, PTIME(K),PRATE(K),PACCU(K)
240   NEXT K
250   CLOSE #1
260   OPEN IDR$ FOR INPUT AS #1
265   QPEAK = 0
270   INPUT #1, ROWR
280   FOR K = 1 TO ROWR  'Input the runoff data.
290     INPUT #1, RTIME(K),RRATE(K),RACCU(K),CFS(K)
300   NEXT K
310   CLOSE #1
320 'Read in the geomorphic data for the watershed.
330   IDG$ = DRIVEW$ + ID$ + ".dat"
340   OPEN IDG$ FOR INPUT AS #1
350   INPUT #1,
PERIM,AREA,MCHANL,MAXL,MAXWID,MAXELE,OVERSLP,CHANSLP
360   INPUT #1,
STRMORD,BIFUR,DRAIND,RELIEF,RELTIV,RUGGED,ELONG,CIRC,TC
```

```
370   CLOSE #1
380 'Read in the Rainfall Statistical data for the
storm.
390   IDS$ = DRIVES$ + ID$ + ".s" + Q$
400   OPEN IDS$ FOR INPUT AS #1
410   INPUT #1,
RAINDUR,PKDUR,XRAINDUR,PKEXDUR,PKRAIN,PKEXRAIN
420   INPUT #1, STDRAIN,STDXRAIN,CSRAIN,CSXRAIN
430   CLOSE #1
440 '
450 TPR = .25 '
460 DEVQPK=1000000!
470 KSTARMIN=100
480 CLS
485 '
490 GOSUB 1000 'Convert to decimal time.
500 GOSUB 3300 'Subtract out the base flow of the
hydrograph.
505 IA = 1!
506 CLS: PRINT "    Kstar     Dev. Sqr.            Qpeak"
510 GOSUB 2000 'Calculate the CN, S, and depth of excess
rainfall.
520 GOSUB 3500 'Calculate the lag time and time of
concentration.
530 GOSUB 3600 'Determine the time step increment.
600 GOSUB 5000 'Subroutine to calculate the optimum
Kstar for the storm.
610 '
630 DEVQPK=DEVMIN
640 KSTARMIN=KMIN
650 RUNDEPTH = ( SUM * DT * 12 * 3600 ) / ( AREA )
660 PRINT:PRINT
670 PRINT "The watershed ID is ";ID$
680 PRINT USING "The observed depth of excess rainfall
was ##.#### inches.";QEND
690 PRINT USING "The predicted depth of excess rainfall
was ##.#### inches.";RUNDEPTH
700 PRINT USING "Kstar =##.####    DEVIATION SQUARED
=######.####";KSTARMIN,DEVQPK
710 '
750 IDOUT$ = DRIVEF$ + ID$ + ".fnl"
790 OPEN IDOUT$ FOR APPEND AS #1
800 WRITE #1,
ID$,J,KSTARMIN,S,RAINDUR,PKDUR,XRAINDUR,PKEXDUR,PKRAIN
810 WRITE #1,
PKEXRAIN,STDRAIN,STDXRAIN,CSRAIN,CSXRAIN,PEND,QEND,PERIM
820 WRITE #1,
AREA,MCHANL,MAXL,MAXWID,MAXELE,OVERSLP,CHANSLP,STRMORD,
BIFUR
830 WRITE #1, DRAIND,RELIEF,RELTIV,RUGGED,ELONG,CIRC,TC
840 CLOSE #1
850 '
```

```
860 IDOUTQ$ = DRIVEF$ + ID$ + ".q"
870 OPEN IDOUTQ$ FOR APPEND AS #1
880 WRITE #1, QPEAK,QPMAX1
890 CLOSE #1
900 '
950 ERASE
CUMZ,UH,CFS,RTIME,OUTF,P,PACCU,PTIME,FLOW,RTIME,PRATE,
RRATE
960 NEXT J
990 END
1000 '************************************************
1010 'Subroutine to convert the times from hours and
1020 'minutes to decimal time in hours.
1030 '
1040 FOR K = 1 TO ROWP  'Convert the Precipitation Data
times.
1050   IF PTIME(K) < 100 THEN 1060 ELSE 1080
1060     PTIME(K) = PTIME(K)/60
1070   GOTO 1280
1080   IF PTIME(K) < 1000 THEN 1100 ELSE 1170
1100     T$ = STR$(PTIME(K))
1110     T0$ = MID$(T$,3,2)
1120     T1$ = MID$(T$,1,2)
1160   GOTO 1250
1170   IF PTIME(K) < 10000 THEN 1180 ELSE 1220
1180     T$ = STR$(PTIME(K))
1190     T0$ = MID$(T$,4,2)
1200     T1$ = MID$(T$,1,3)
1210   GOTO 1250
1220     T$ = STR$(PTIME(K))
1230     T0$ = MID$(T$,5,2)
1240     T1$ = MID$(T$,1,4)
1250   T0 = VAL(T0$)/60
1260   T1 = VAL(T1$)
1270   PTIME(K) = T0 + T1 'Store the decimal time value.
1280 NEXT K
1290 'Convert the Runoff Data Times.
1300 FOR K = 1 TO ROWR
1310   IF RTIME(K) < 100 THEN 1320 ELSE 1340
1320     RTIME(K) = RTIME(K)/60
1330   GOTO 1495
1340   IF RTIME(K) < 1000 THEN 1350 ELSE 1390
1350     T$ = STR$(RTIME(K))
1360     T0$ = MID$(T$,3,2)
1370     T1$ = MID$(T$,1,2)
1380   GOTO 1470
1390   IF RTIME(K) < 10000 THEN 1400 ELSE 1440
1400     T$ = STR$(RTIME(K))
1410     T0$ = MID$(T$,4,2)
1420     T1$ = MID$(T$,1,3)
1430   GOTO 1470
1440     T$ = STR$(RTIME(K))
```

```
1450     T0$ = MID$(T$,5,2)
1460     T1$ = MID$(T$,1,4)
1470   T0 = VAL(T0$)/60
1480   T1 = VAL(T1$)
1490   RTIME(K) = T0 + T1 'Store the decimal time value.
1495 '
1500 NEXT K
1510 RETURN
1520 '*************************************************
2000 '*************************************************
2010 'Subroutine to calculate the potential maximum
retention
2020 '(S) and the curve number (CN).
2030 '
2040 PEND = PACCU(ROWP)  'Input the depth of rainfall.
2050 QEND = RACCU(ROWR)  'Input the depth of runoff.
2060 '
2070 RADICAL = SQR(4*PEND*QEND*.2 + QEND^2*(1-.2)^2)
2080 S = ( 2*.2*PEND + QEND*(1-.2) - RADICAL ) / (
2*.2^2)
2100 CN = 1000 / ( 10 + S ) 'Calculate the Curve Number
(CN).
2112 P(1)=PACCU(1)
2115 P(1)=PACCU(1)
2118 RETEN = IA*.2
2120 RADICAL = SQR(4*PEND*QEND*RETEN +
QEND^2*(1-RETEN)^2)
2125 S = ( 2*RETEN*PEND + QEND*(1-RETEN) - RADICAL ) / (
2*RETEN^2 )
2128 PRINT S
2130 FOR K = 1 TO ROWP 'Loop to calculate the depth of
excess rainfall.
2140   IF PACCU(K) < RETEN*S THEN 2150 ELSE 2190
2150   P(K) = 0
2160   TMEXSTRT = PTIME(K)
2165   ESTART = K
2170   GOTO 2210
2180 '
2190   P(K) = (PACCU(K) - RETEN*S)^2 / (PACCU(K) + S -
RETEN*S)
2200   TMEXEND = PTIME(K)
2205   EEND = K
2210   IF P(K)<0! THEN P(K)=0!
2220 NEXT K
2230 RETURN
2400 '*************************************************
3300 '*************************************************
3310 'Subtract the base flow from the observed
hydrograph.
3320 '
3325 CFSBGIN = CFS(1)
3330 FOR K = 1 TO ROWR
```

```
3340   TM = (RTIME(K) - RTIME(1)) / (RTIME(ROWR) -
RTIME(1))
3350   BASEFLOW = CFSBGIN + (CFS(ROWR) - CFSBGIN)*TM
3360   CFS(K) = CFS(K) - BASEFLOW
3370   IF CFS(K)>QPEAK THEN QPEAK=CFS(K):
TMPEAK=RTIME(K)
3375   IF CFS(K) < 0! THEN CFS(K) = 0
3380 NEXT K
3390 '
3395 RETURN
3400 '*********************************************
3500 '*********************************************
3510 'CACULATE THE LAG TIME AND THE TIME OF
CONCENTRATION
3520 '
3530 'INPUT "What is the lag time (hrs.) ";LAG
3560 '
3565 '
3570 'TC = LAG / (.5)
3580 RETURN
3590 '************************************************
3600 '************************************************
3610 '     Determine the delta time step
3620 '
3630 EXTIME = TMEXEND - TMEXSTRT 'Length of time of
excess rainfall.
3640 '
3660  DT=TC*.05
3690 '
3700 NUH = ( TC / DT ) * 4
3710 '
3720 RETURN
3730 '********************************************
4000 '************************************************
4010 'DEFINE THE DIMENSIONLESS TRIANGULAR HYDROGRAPH
4020 '
4030 T = 0:  IT = 0
4040 M = KSTAR '
4050 FOR IT = 1 TO 3*NUH
4055   T = ((T*TC) + DT)/TC
4060   IF T > 1 THEN 4065 ELSE 4075
4065   H(IT)=2-(2*M/TPR)-2/(1-TPR)+2*TPR/(1-TPR)
-2*M/(1-TPR))*EXP(TPR/M)*EXP(-T/M) + (2*M/TPR)*EXP(-T/M)
+ (2*M/(1-TPR))*EXP(1/M)*EXP(-T/M)
4070   GOTO 4120
4075   IF T<TPR THEN 4080 ELSE 4100
4080     UH(IT) = 2*T/TPR-2*M/TPR+(2*M/TPR)*EXP(-T/M)
4090   GOTO 4110
4100     UH(IT) = (2-(2*M/TPR)-2/(1-TPR)+2*TPR/(1-TPR)
-2*M/(1-TPR))*EXP(TPR/M)*EXP(-T/M) + (2*M/TPR)*EXP(-T/M)
+ 2*(1+M-T)/(1-TPR)
4110 'PRINT USING "  ###.####    #######.#  ";T,UH(IT)
```

```
4120 NEXT IT
4130 '
4140 RETURN
4150 '*************************************************
5000 '*************************************************
5010 'BRENT'S METHOD TO DETERMINE THE OPTIMUM Kstar
5020 '
5025 CGOLD = .381966:  ZEPS = 1E-10: TOL = .001: QPMAX =
-1000!
5030 A = 0!
5040 B = 10!
5045 KSTAR = 2
5050 V = 2
5060 W = V
5070 X = V
5080 E = 0!
5090 GOSUB 7000
5100 FX = DEVSQR
5110 FV = FX
5120 FW = FX
5130 FOR ITER = 1 TO 100
5135 PRINT USING " #.######    ###.######
#######.###";KSTAR,DEVMIN,QPMAX
5136 'IF ITER > 1 THEN LOCATE 13,10: PRINT ITER,KSTAR
5140   XM = .5 * ( A + B )
5150   TOL1 = TOL * ABS(X) + ZEPS
5160   TOL2 = TOL1 * 2!
5170   IF ABS(X-XM) =< TOL2-.5*(B-A) THEN GOTO 5640
5180   IF (ABS(E) > TOL1) THEN 5190 ELSE 5360
5190    R=(X-W)*(FX-FV)
5200    Q=(X-V)*(FX-FW)
5210    P=(X-V)*Q-(X-W)*R
5220    Q=2*(Q-R)
5230    IF (Q>0) THEN P=-P
5240    Q=ABS(Q)
5250    ETEMP=E
5260    E=D
5270    IF ABS(P)>=ABS(.5*Q*ETEMP) THEN GOTO 5360
5280    IF P<=(Q*(A-X)) THEN GOTO 5360
5290    IF P>=(Q*(B-X)) THEN GOTO 5360
5300    D = P/Q
5310    U = X+D
5320    IF (U-A)<TOL2 OR (B-U)<TOL2 THEN 5330 ELSE 5350
5330     IF XM-X<0! THEN D=-TOL1 ELSE D=TOL1
5350    GOTO 5380
5360   IF X>=XM THEN E=A-X ELSE E=B-X
5370   D = CGOLD * E
5380   IF ABS(D)>=TOL1 THEN 5390 ELSE 5410
5390    U = X + D
5400    GOTO 5420
5410    IF D<0 THEN U=X-TOL1 ELSE U=X+TOL1
5420  KSTAR = U
```

```
5430  IF KSTAR<.02 THEN 5640
5435  GOSUB 7000
5440  FU = DEVSQR
5450  IF FU<=FX THEN 5460 ELSE 5530
5460    IF U>=X THEN A=X ELSE B=X
5470    V=W
5480    FV= FW
5490    W=X
5495    FW=FX
5500    X=U
5510    FX=FU
5520  GOTO 5630
5530  IF U<X THEN A=U ELSE B=U
5540  IF FU<=FW OR W=X THEN 5550 ELSE 5600
5550    V=W
5560    FV=FW
5570    W=U
5580    FW=FU
5590  GOTO 5630
5600 IF FU<=FV OR V=X OR V=W THEN 5610 ELSE 5630
5610   V=U
5620   FV=FU
5630 NEXT ITER
5640 '
5650 KMIN = X
5660 DEVMIN = FX
5690 '
5700 '********************************************
7000 '********************************************
7010 'RUNOFF SUBROUTINE TO K* VALUE FOR BRENT'S
ITERATION
7020 '
7080 SUM = 0!
7090 DEVMIN=100000!
7100 QPMAX = -100
7110   DTSTAR = DT/TC
7160   FLOW(1) = 0!
7250   UH(1) = 0!
7270   KCOEF = KSTAR * TC
7280 JEND = NUH
7300 GOSUB 4000 ' Subroutine to calculate dimensionless
triangular hydrograph.
7455 EXSTEP=(3*JEND)+10
7460 TMSTEP = PTIME(ESTART) '
7470 TEMPZ = 0: BASEZ = 0: ST = 1
7480 ES = ESTART
7485 CUMZ(ES)=0
7490 FOR Z = ESTART TO EXSTEP
7500   TMSTEP = TMSTEP + DT: ST=1
7520   IF TMSTEP >= PTIME(EEND) THEN 7525 ELSE 7535
7525   CUMZ(Z) = P(EEND)
7530   GOTO 7600
```

```
7535    IF PTIME(ST)<TMSTEP THEN 7540 ELSE 7560
7540      ST = ST + 1
7550      GOTO 7535
7560    CUMZ(Z)=P(ST-1)+((TMSTEP-PTIME(ST-1))/(PTIME(ST)
-PTIME(ST-1)))*(P(ST)-P(ST-1))
7600    ZEX = CUMZ(Z) - CUMZ(Z-1)
7630    'PRINT USING "##.######  ####.####
##.######";CUMZ(Z),TMSTEP,ZEX
7640    FOR K = 1 TO 3*JEND
7650      ISUB = ( Z - ESTART) + K
7660      FLOW(ISUB) = FLOW(ISUB) +
(ZEX/12)*AREA*UH(K)/(TC*3600)
7665      IF FLOW(ISUB) > QPMAX THEN QPMAX = FLOW(ISUB):
TMPK=TMSTEP+K*DT
7668        OUTF(ISUB)=FLOW(ISUB)
7670    NEXT K
7675   '
7680   SUM = SUM + OUTF(Z-ESTART+1) '
7685   '
7690 NEXT Z
7695 ERASE FLOW, CUMZ, UH
7700 DEVSQR=((QPEAK-QPMAX)/QPEAK)^2
7760 IF DEVSQR<DEVMIN THEN DEVMIN=DEVSQR: QPMAX1=QPMAX
7780 RETURN
7800 '*************************************************
```

APPENDIX B

RAINFALL STATISTICS PROGRAM

```
20 '*************************************************
30 '*****Bill Brown    M.S. Thesis Program # 1   *****
40 '*************************************************
50 '*************************************************
60 '*****12-07-88  RAINFALL STATISTICS           *****
90 '*************************************************
95 KEY OFF
100 DIM PTIME(300),PRATE(300),PACCU(300)
110 DIM RTIME(300),RRATE(300),RACCU(300),CFS(300)
115 DIM EIT(300),EI(300),P(300)
120 CLS
130 INPUT "What is the watershed I.D. "; ID$
140 INPUT "What is the first storm number   ";STORM1
145 INPUT "What is the final storm number   ";STORM2
150 FOR J = STORM1 TO STORM2
160   G$ = STR$(J)
170   Q$ = MID$(G$,2)
180   IDP$=ID$+".P"+Q$ 'Assign the file names for the
190   IDR$=ID$+".R"+Q$ 'precipitation and runoff data
files.
200   OPEN IDP$ FOR INPUT AS #1
210   INPUT #1, ROWP
220   FOR K = 1 TO ROWP  'Input the precipitation data.
230     INPUT #1, PTIME(K),PRATE(K),PACCU(K)
240   NEXT K
250   CLOSE #1
260  OPEN IDR$ FOR INPUT AS #1
270   INPUT #1, ROWR
280   FOR K = 1 TO ROWR  'Input the runoff data.
290     INPUT #1, RTIME(K),RRATE(K),RACCU(K),CFS(K)
300   NEXT K
310   CLOSE #1
490 GOSUB 1000 'Convert to decimal time.
500 GOSUB 3300 'Subtract out the base flow of the
hydrograph.
510 GOSUB 2000 'Calculate the CN and S.
520 GOSUB 4000 'Calculate the center of mass of the
rainfall.
530 GOSUB 4200 'Calculate the variance of the rainfall.
540 GOSUB 2200 'Calculate the center of mass of the
rainfall excess.
550 GOSUB 4400 'Calculate the variance of the rainfall
excess.
570 GOSUB 4600 'Determine the peak rainfall and excess
intensity and duration.
650 GOSUB 5000 'Write the data to a file.
700 GOSUB 3000 'Print out the data for the storm.
890 NEXT J
900 END
998 '***********************************************
999 '*********  SUBROUTINES **************************
1000 '**********************************************
```

```
1010 'Subroutine to convert the times from hours and
1020 'minutes to decimal time in hours.
1030 '
1040 FOR K = 1 TO ROWP 'Convert the Precipitation Data
times.
1050   IF PTIME(K) < 100 THEN 1060 ELSE 1080
1060     PTIME(K) = PTIME(K)/60
1070   GOTO 1280
1080   IF PTIME(K) < 1000 THEN 1100 ELSE 1170
1100     T$ = STR$(PTIME(K))
1110     T0$ = MID$(T$,3,2)
1120     T1$ = MID$(T$,1,2)
1160   GOTO 1250
1170   IF PTIME(K) < 10000 THEN 1180 ELSE 1220
1180     T$ = STR$(PTIME(K))
1190     T0$ = MID$(T$,4,2)
1200     T1$ = MID$(T$,1,3)
1210   GOTO 1250
1220     T$ = STR$(PTIME(K))
1230     T0$ = MID$(T$,5,2)
1240     T1$ = MID$(T$,1,4)
1250   T0 = VAL(T0$)/60
1260   T1 = VAL(T1$)
1270   PTIME(K) = T0 + T1 'Store the decimal time value
1280 NEXT K
1290 'Convert the Runoff Data Times.
1300 FOR K = 1 TO ROWR
1310   IF RTIME(K) < 100 THEN 1320 ELSE 1340
1320     RTIME(K) = RTIME(K)/60
1330   GOTO 1495
1340   IF RTIME(K) < 1000 THEN 1350 ELSE 1390
1350     T$ = STR$(RTIME(K))
1360     T0$ = MID$(T$,3,2)
1370     T1$ = MID$(T$,1,2)
1380   GOTO 1470
1390   IF RTIME(K) < 10000 THEN 1400 ELSE 1440
1400     T$ = STR$(RTIME(K))
1410     T0$ = MID$(T$,4,2)
1420     T1$ = MID$(T$,1,3)
1430   GOTO 1470
1440     T$ = STR$(RTIME(K))
1450     T0$ = MID$(T$,5,2)
1460     T1$ = MID$(T$,1,4)
1470   T0 = VAL(T0$)/60
1480   T1 = VAL(T1$)
1490   RTIME(K) = T0 + T1 'Store the decimal time value
1495 '
1500 NEXT K
1510 RETURN
1520 '***********************************************
2000 '***********************************************
2010 'Subroutine to calculate the potential maximum
```

```
2020 'retention (S) and the curve number (CN).
2030 '
2040 P = PACCU(ROWP)  'Input the depth of rainfall.
2050 Q = RACCU(ROWR)  'Input the depth of runoff.
2060 '
2070 S = 5 * ( P + 2*Q - SQR( 4*Q^2 + 5*P*Q))
2080 '
2090 CN = 1000 / (10+S) 'Calculate the Curve Number
2100 '
2110 RETURN
2120 '*********************************************
2200 '*********************************************
2210 'Subroutine to calculate the center of mass of
2220 'Rainfall Excess
2230 P(1) = PACCU(1)
2240 EIT(1) = 0
2245 EI(1) = 0
2250 '
2260 FOR K = 2 TO ROWP 'Account for potential retention
storage.
2270   IF PACCU(K) < .2*S THEN 2300 ELSE 2280
2280   P(K) = (PACCU(K) - 0.2*S)^2 / (PACCU(K) + .8*S)
2290   GOTO 2310
2300   P(K) = 0
2305   TIME = PTIME(K)
2310 '
2320   DT = PTIME(K) - PTIME(K-1)
2330   IE = ( P(K) -  P(K-1) )
2340   TBASE = PTIME(K-1) + DT/2
2350   EIT(K) = TBASE*IE + EIT(K-1)
2360   EI(K) = IE + EI(K-1)
2370 '
2380 NEXT K
2390 '
2400 CMRAINEX = EIT(ROWP)/EI(ROWP) 'Calculate the
centroid.
2410 '
2420 RETURN
2430 '********************************************
3000 '********************************************
3005 '         Print out the data for the storm.
3008 '
3010 PRINT:PRINT
3015 PRINT "The watershed I.D. is ";ID$;"."
3020 PRINT "The storm number is";J;"."
3025 PRINT "The potential retention S is =";S;"."
3030 PRINT "The curve number CN is =";CN;"."
3035 PRINT "The rainfall duration is =";RAINDUR;"
hours."
3040 PRINT "The rainfall excess duration is
=";XRAINDUR;" hours."
3045 PRINT "The peak intensity is =";PKRAIN;" in/hr."
```

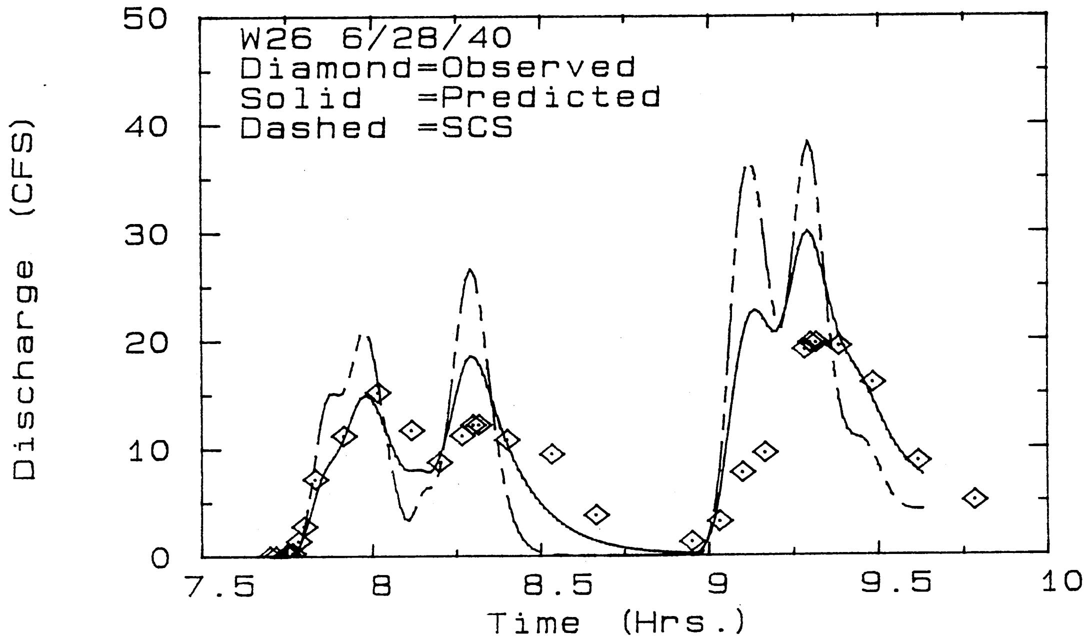
```
3050 PRINT "The peak intensity duration is =";PKDUR;"
hours."
3055 PRINT "The peak excess intensity is =";PKEXRAIN;"
in/hr."
3065 PRINT "The peak excess intensity duration is
=";PKEXDUR;" hours."
3070 PRINT "The center of mass of the rainfall is
=";CMRAIN;" hours."
3075 PRINT "The center of mass of the rainfall excess is
=";CMRAINEX;" hours."
3085 PRINT "The standard deviation of the rainfall is
=";STDRAIN;"."
3087 PRINT "The skewness coefficient of the rainfall is
=";CSRAIN;"."
3090 PRINT "The standard deviation of the rainfall
excess is =";STDXRAIN;"."
3095 PRINT "The skewness coefficient of the excess
rainfall is =";CSXRAIN;"."
3190 RETURN
3200 '*************************************************
3300 '*************************************************
3310 'Subtract the base flow from the observed
hydrograph.
3320 '
3325 CFSBGIN = CFS(1)
3330 FOR K = 1 TO ROWR
3340   TM = (RTIME(K) - RTIME(1)) / (RTIME(ROWR) -
RTIME(1))
3350   BASEFLOW = CFSBGIN + (CFS(ROWR) - CFSBGIN)*TM
3360   CFS(K) = CFS(K) - BASEFLOW
3380 NEXT K
3390 '
3395 RETURN
3400 '*************************************************
4000 '*************************************************
4010 'Subroutine to calculate the center of mass of the
4015 'rainfall.
4020 EIT(1) = 0
4030 EI(1) = 0
4040 '
4050 FOR K = 2 TO ROWP '
4060   DT = PTIME(K) - PTIME(K-1)
4070   IE = ( PACCU(K) -  PACCU(K-1) )
4080   TBASE = PTIME(K-1) + DT/2
4090   EIT(K) = TBASE*IE + EIT(K-1)
4100   EI(K) = IE + EI(K-1)
4120 NEXT K
4130 '
4135 CMRAIN = EIT(ROWP)/EI(ROWP) 'Calculate the center
of mass.
4140 RETURN
4150 '***********************************************
```

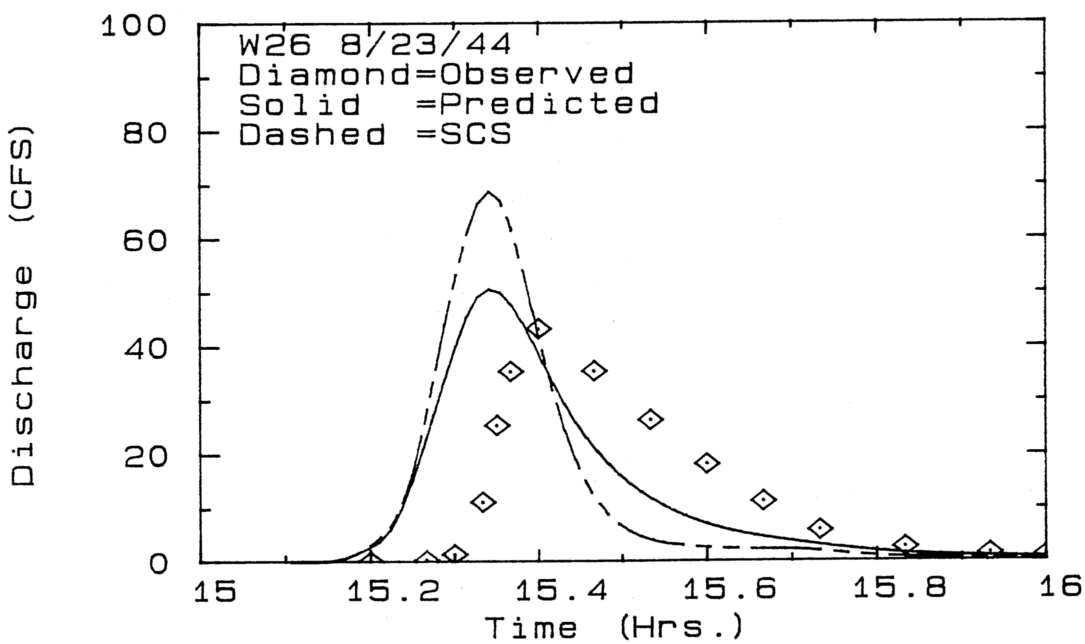
```
4200 '**************************************************
4210 'Subroutine to calculate the variance of the
rainfall.
4220 '
4225 SUMVARD = 0: THIRDD = 0
4230 SUMVARN = 0: THIRDN = 0
4235 '
4240 FOR K = 2 TO ROWP
4250   DT = (PTIME(K) + PTIME(K-1))/2
4260   SUMVARN = SUMVARN + (DT -
CMRAIN)^2*(PACCU(K)-PACCU(K-1))
4265   THIRDN = THIRDN + (DT -
CMRAIN)^3*(PACCU(K)-PACCU(K-1))
4270   SUMVARD = SUMVARD + (PACCU(K)-PACCU(K-1))
4275   THIRDD = THIRDD + (PACCU(K)-PACCU(K-1))
4280 NEXT K
4290 '
4300 STDRAIN =(SUMVARN / SUMVARD)^(1/2)  ' Standard
Deviation of the rainfall.
4305 CSRAIN = (THIRDN/THIRDD)/(STDRAIN)^3 ' Skew
Coefficient of the rainfall.
4310 '
4320 RETURN
4330 '**************************************************
4400 '**************************************************
4410 'Subroutine to calculate the variance of the
4420 ' rainfall excess.
4430 SUMVARD = 0: THIRDD = 0
4440 SUMVARN = 0: THIRDN = 0
4450 '
4460 FOR K = 2 TO ROWP
4470   DT = (PTIME(K) + PTIME(K-1))/2
4480   SUMVARN = SUMVARN + ( DT -
CMRAINEX)^2*(P(K)-P(K-1))
4485   THIRDN = THIRDN + ( DT -
CMRAINEX)^3*(P(K)-P(K-1))
4490   SUMVARD = SUMVARD + (P(K) - P(K-1))
4495   THIRDD = THIRDD + (P(K) - P(K-1))
4500 NEXT K
4510 '
4520 STDXRAIN=(SUMVARN/SUMVARD)^(.5)'  Std. Deviation of
the excess rainfall.
4525 CSXRAIN=(THIRDN/THIRDD)/(STDXRAIN)^3 ' Skew Coef.
of the excess rainfall.
4530 '
4540 RETURN
4550 '**************************************************
4600 '**************************************************
4610 'Determine the peak rainfall and excess rainfall
4620 'intensities and duration.
4630 '
4640 PKRAIN = 0:  PKDUR = 0
```

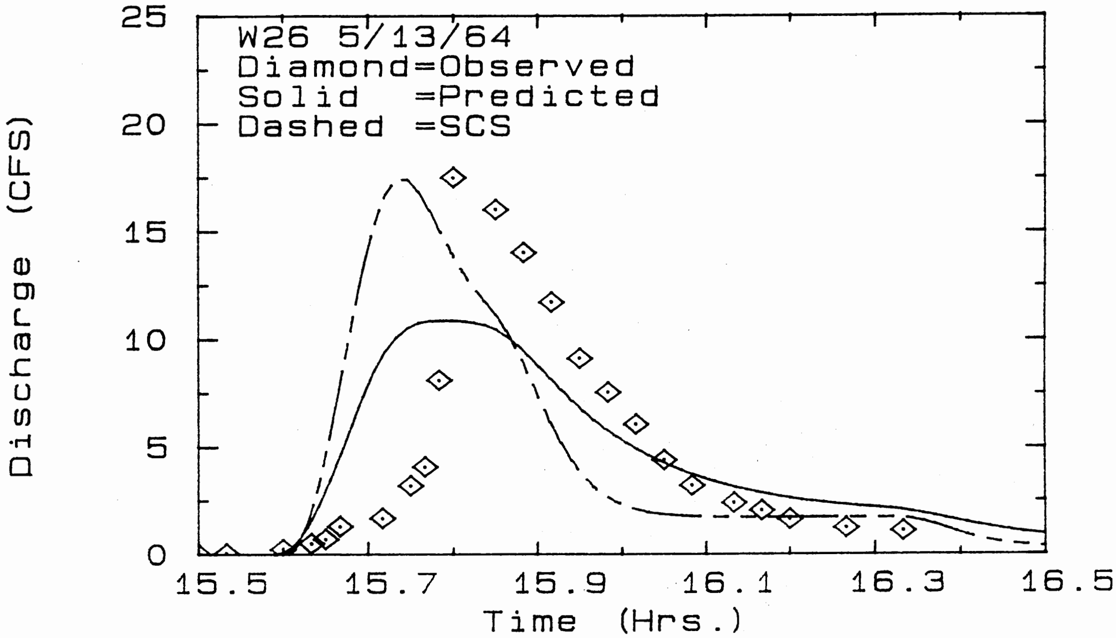
```
4650 PKEXRAIN = 0:  PKEXDUR = 0
4660 '
4670 FOR K = 2 TO ROWP
4680   IF PRATE(K) > PKRAIN THEN 4690 ELSE 4710
4690     PKRAIN = PRATE(K)
4700     PKDUR = (PTIME(K) - PTIME(K-1))
4710   EXPRATE = (P(K)-P(K-1))/(PTIME(K)-PTIME(K-1))
4720   IF EXPRATE > PKEXRAIN THEN 4730 ELSE 4750
4730     PKEXRAIN = EXPRATE
4740     PKEXDUR = (PTIME(K)-PTIME(K-1))
4750 NEXT K
4760 '
4770 RAINDUR = PTIME(ROWP) - PTIME(1)
4780 XRAINDUR = PTIME(ROWP) - TIME
4790 '
4795 RETURN
4800 '************************************************
5000 '************************************************
5010 '          Store the data for the storm.
5020 '
5030 IDZ$ = ID$ + ".S" + Q$ 'Assign storage file name.
5040 OPEN IDZ$ FOR OUTPUT AS #1
5050 PRINT #1, USING "####.######"; RAINDUR;PKDUR '
Rainfall & Intensity Dur.
5060 PRINT #1, USING "####.######"; XRAINDUR;PKEXDUR
'Excess Rain & Int. Dur.
5070 PRINT #1, USING "####.######"; PKRAIN;PKEXRAIN '
Peak Rain & Intensity.
5090 PRINT #1, USING "####.######"; STDRAIN;STDXRAIN '
Stand. Dev. of Rain.
5110 PRINT #1, USING "####.######"; CSRAIN,CSXRAIN '
Coef. of Skew. of Rain.
5120 CLOSE #1
5130 '
5140 RETURN
5150 '************************************************
```

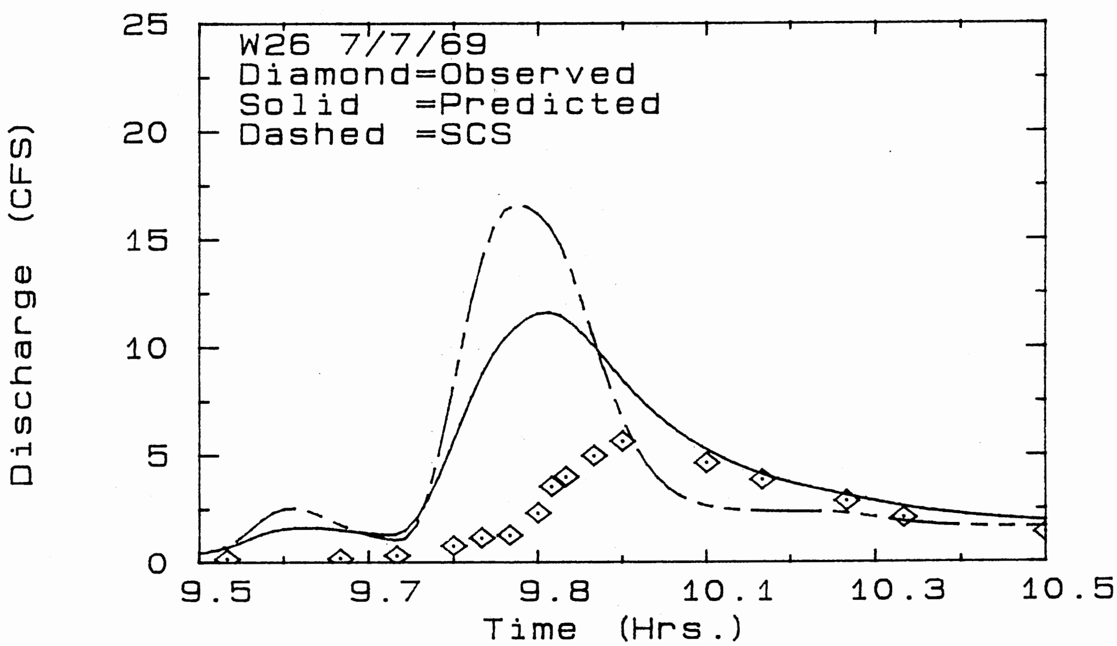
APPENDIX C

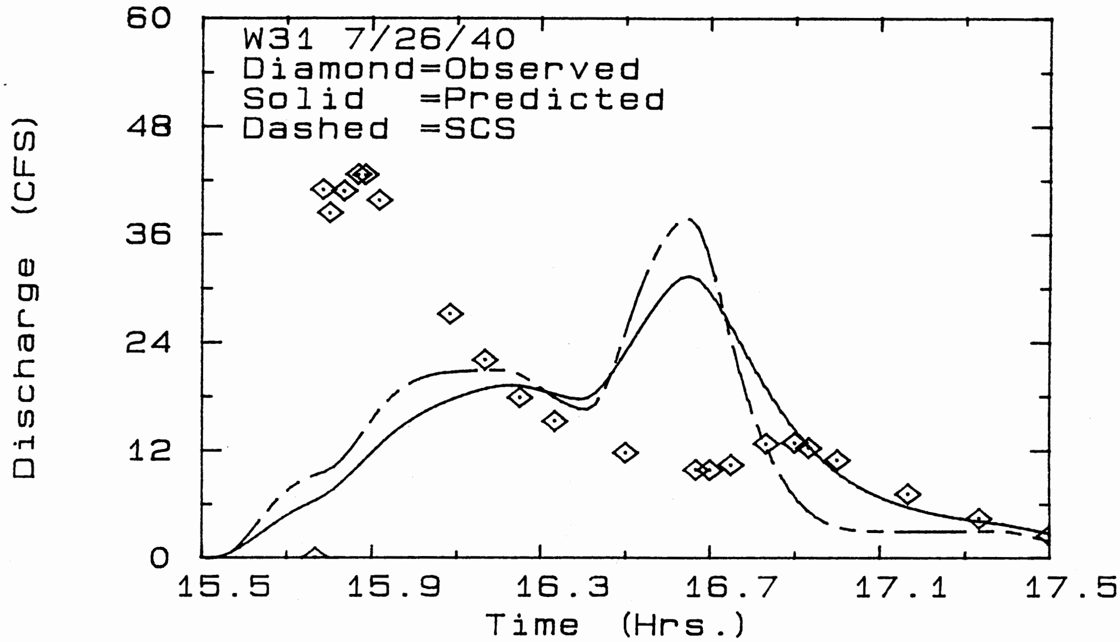
MODEL TEST PLOTS

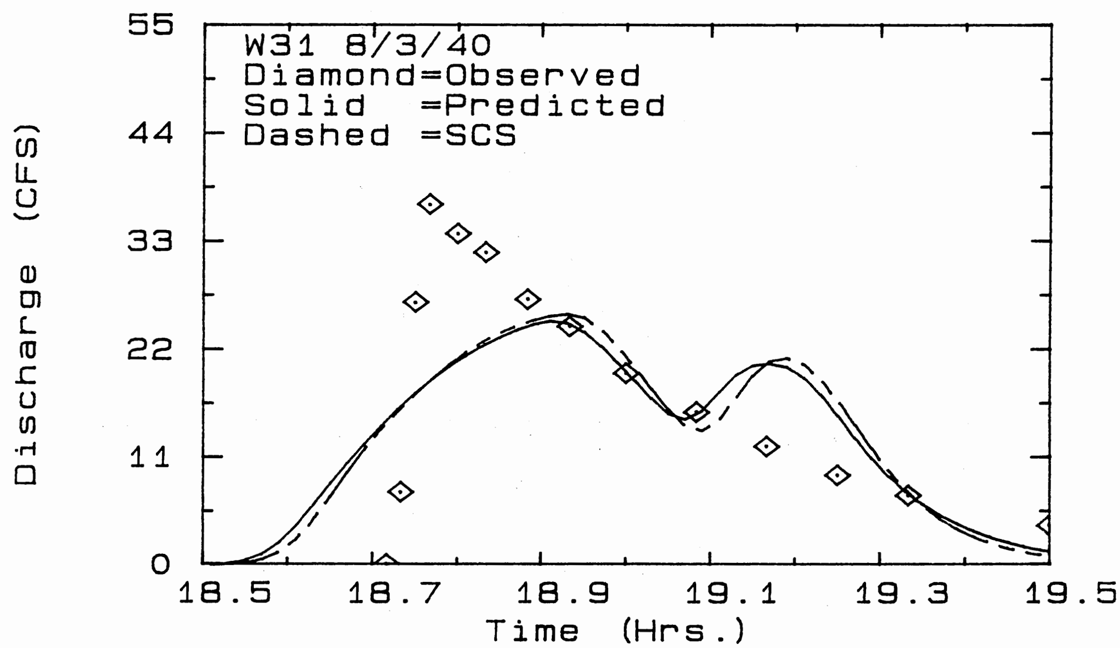

W26 6/28/40
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
10
20
30
40
50
7.5
8
8.5
9
9.5
10
Time (Hrs.)

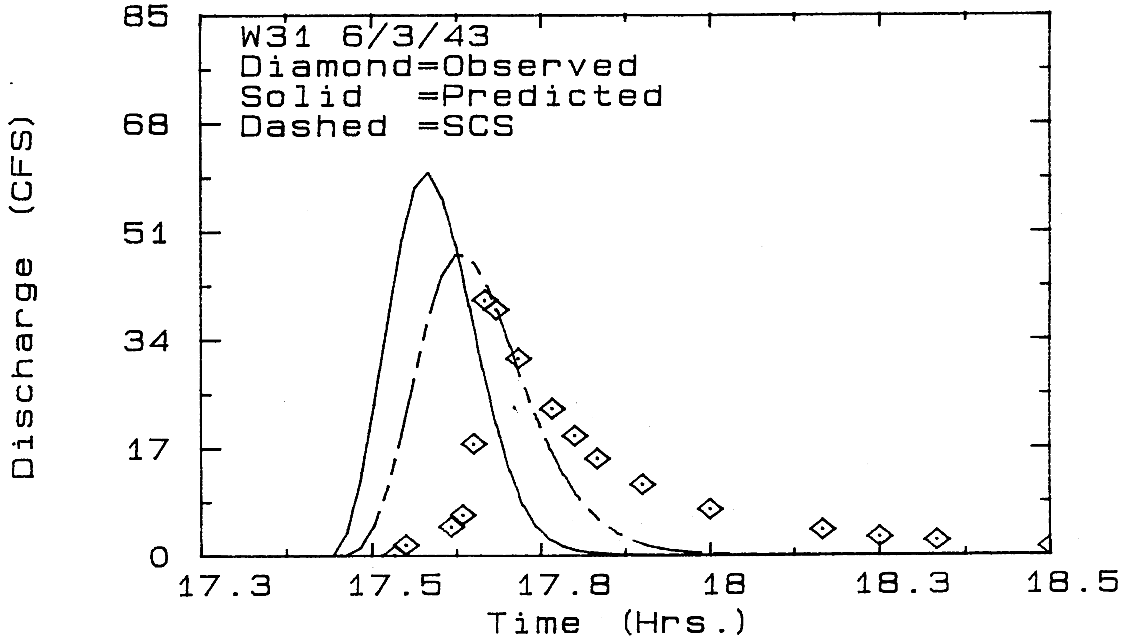

W26 8/23/44
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
20
40
60
80
100
15
15.2
15.4
15.6
15.8
16
Time (Hrs.)

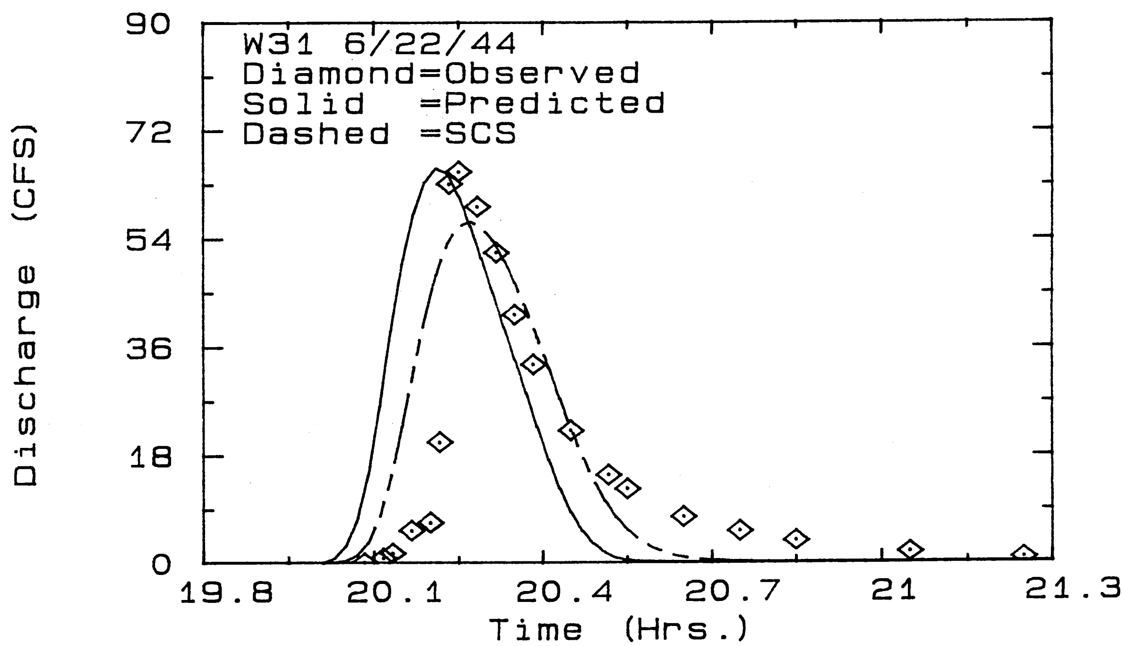

W26 5/13/64
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
5
10
15
20
25
15.5
15.7
15.9
16.1
16.3
16.5
Time (Hrs.)

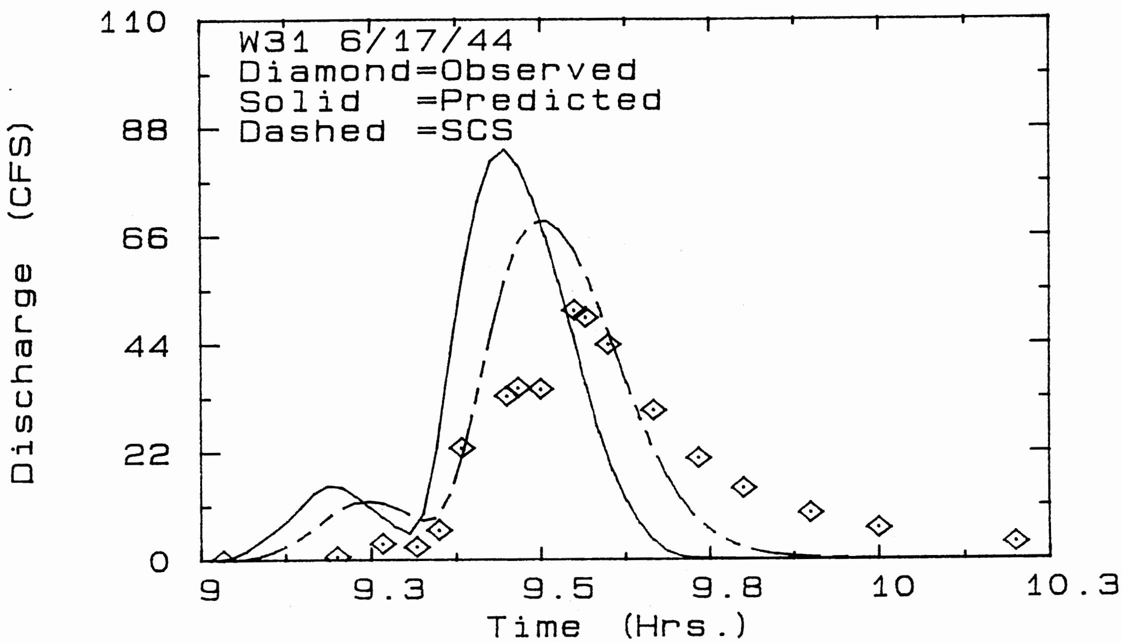

W26 7/7/69
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
5
10
15
20
25
9.5
9.7
9.8
10.1
10.3
10.5
Time (Hrs.)

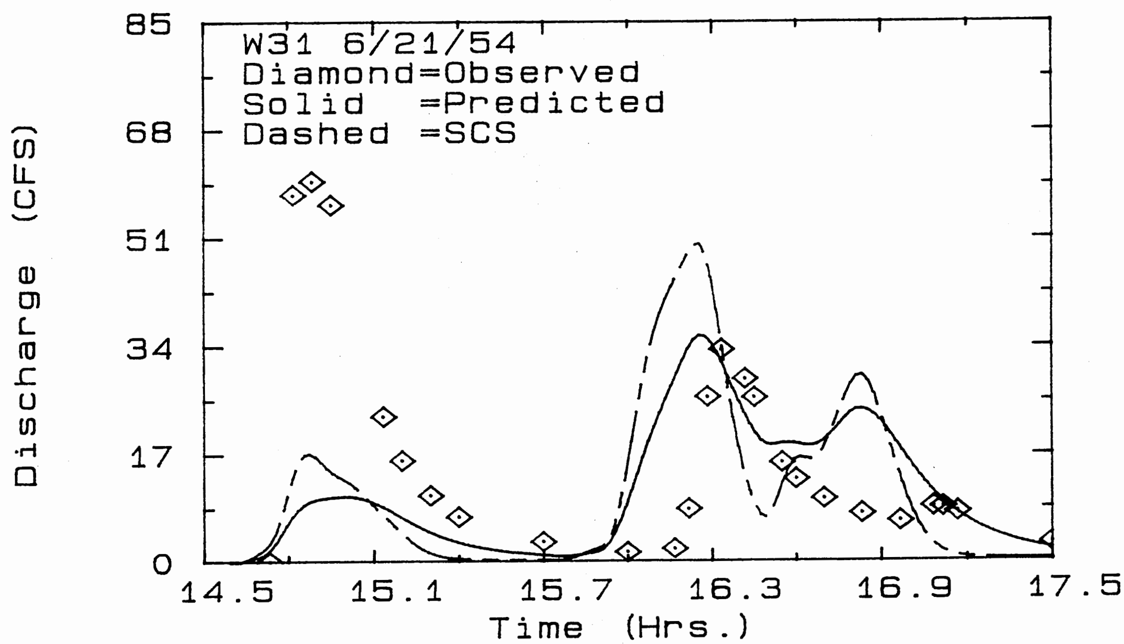

W31 7/26/40
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
12
24
36
48
60
15.5
15.9
16.3
16.7
17.1
17.5
Time (Hrs.)

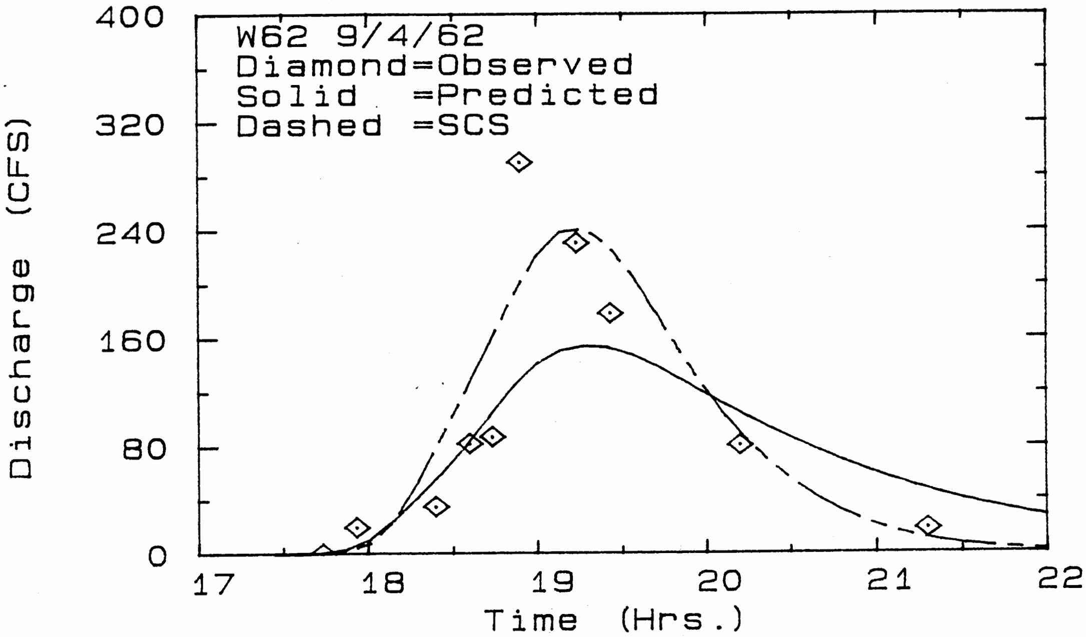

W31 8/3/40
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
11
22
33
44
55
18.5
18.7
18.9
19.1
19.3
19.5
Time (Hrs.)

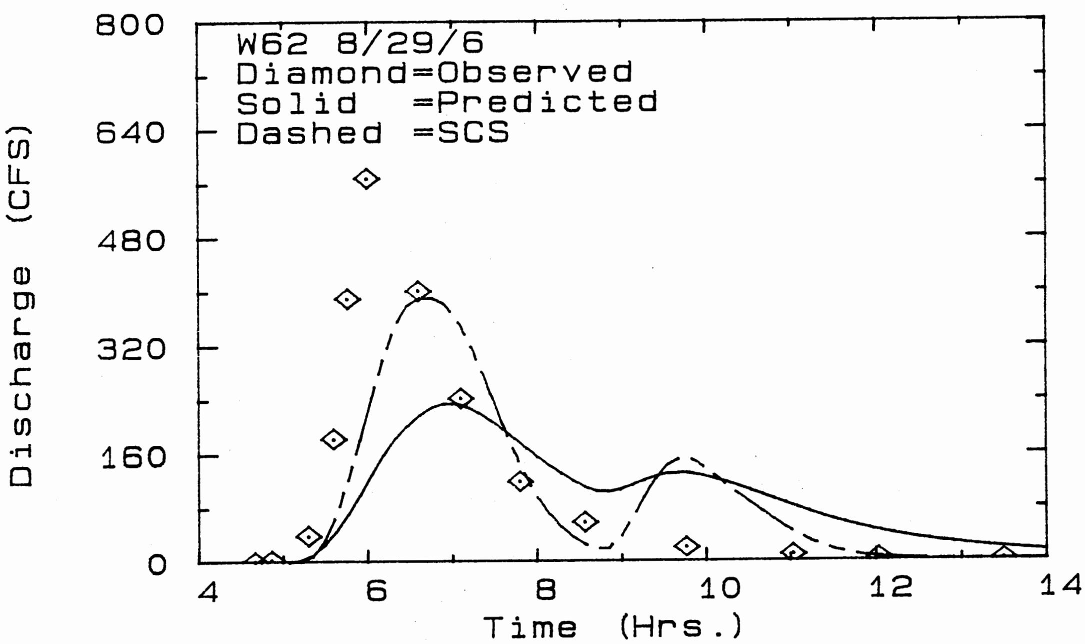

W31 6/3/43
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
85
68
51
34
17
0
17.3
17.5
17.8
18
18.3
18.5
Time (Hrs.)

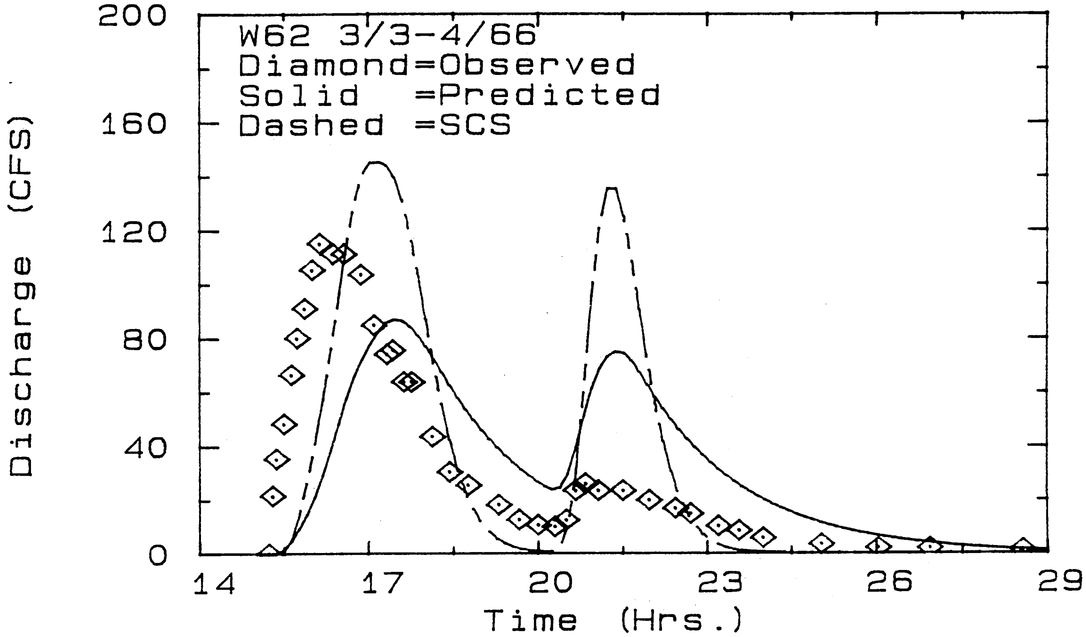

W31 6/22/44
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
90
72
54
36
18
0
19.8
20.1
20.4
20.7
21
21.3
Time (Hrs.)

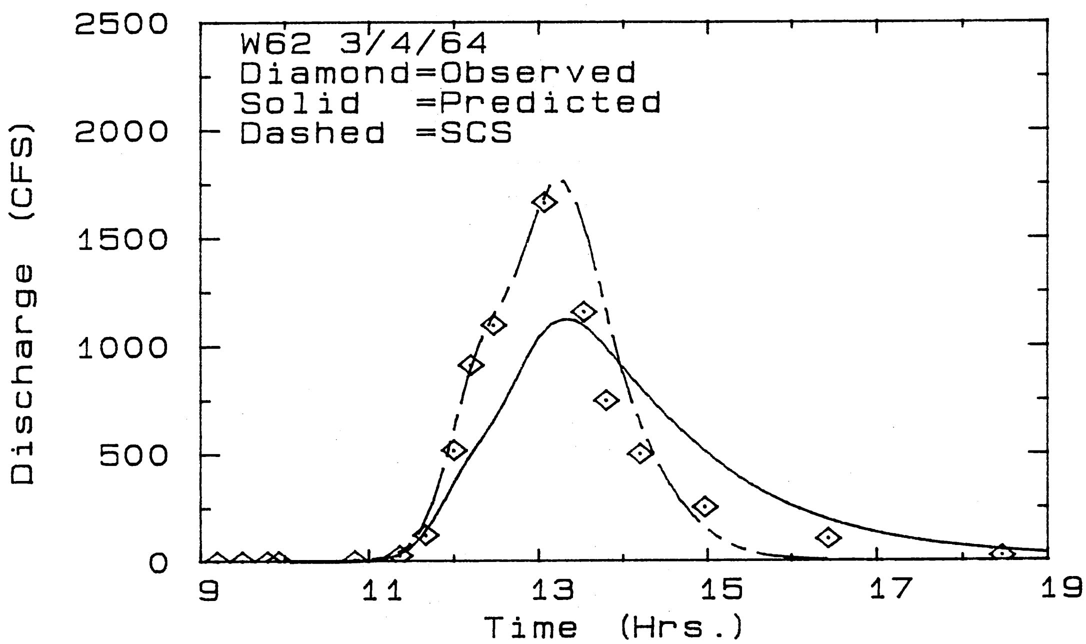

W31 6/17/44
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
110
88
66
44
22
0
9
9.3
9.5
9.8
10
10.3
Time (Hrs.)

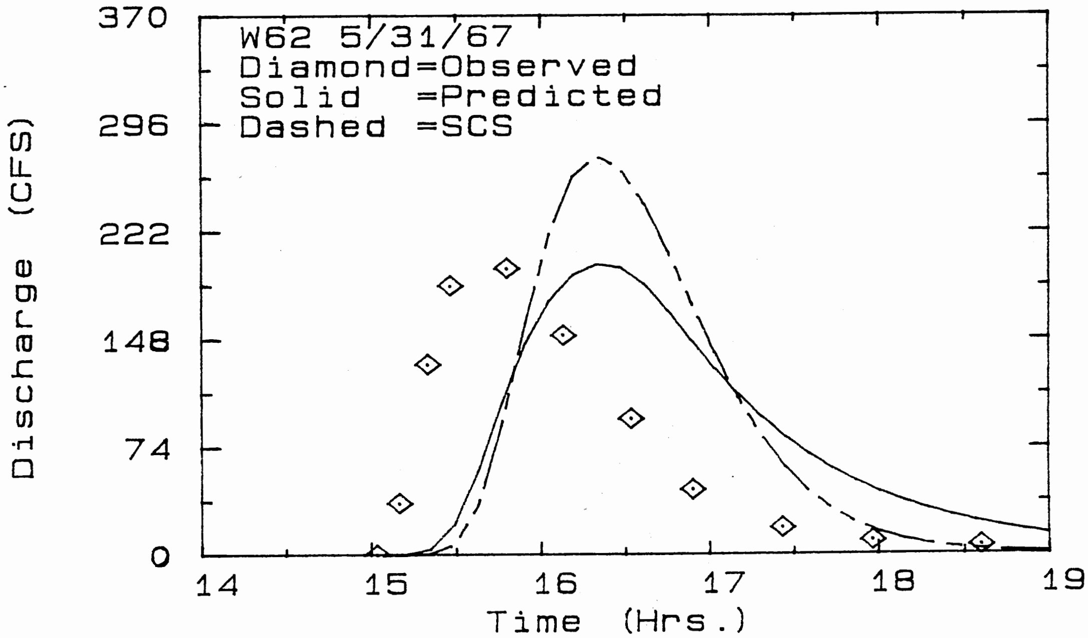

W31 6/21/54
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
85
68
51
34
17
0
14.5
15.1
15.7
16.3
16.9
17.5
Time (Hrs.)

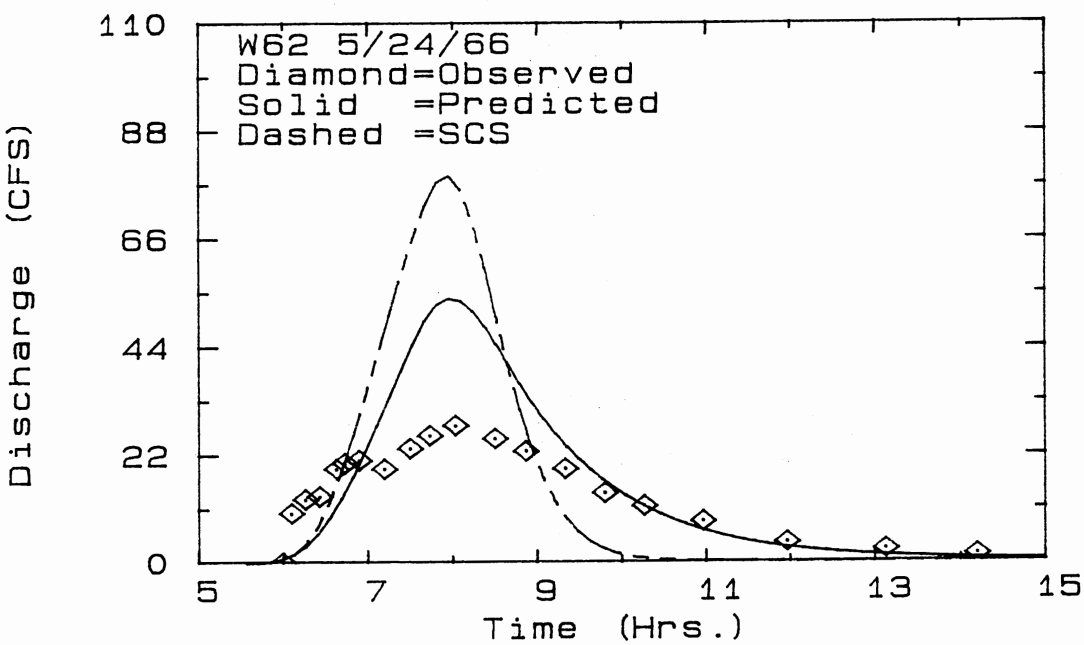

W62 9/4/62
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
80
160
240
320
400
17
18
19
20
21
22
Time (Hrs.)

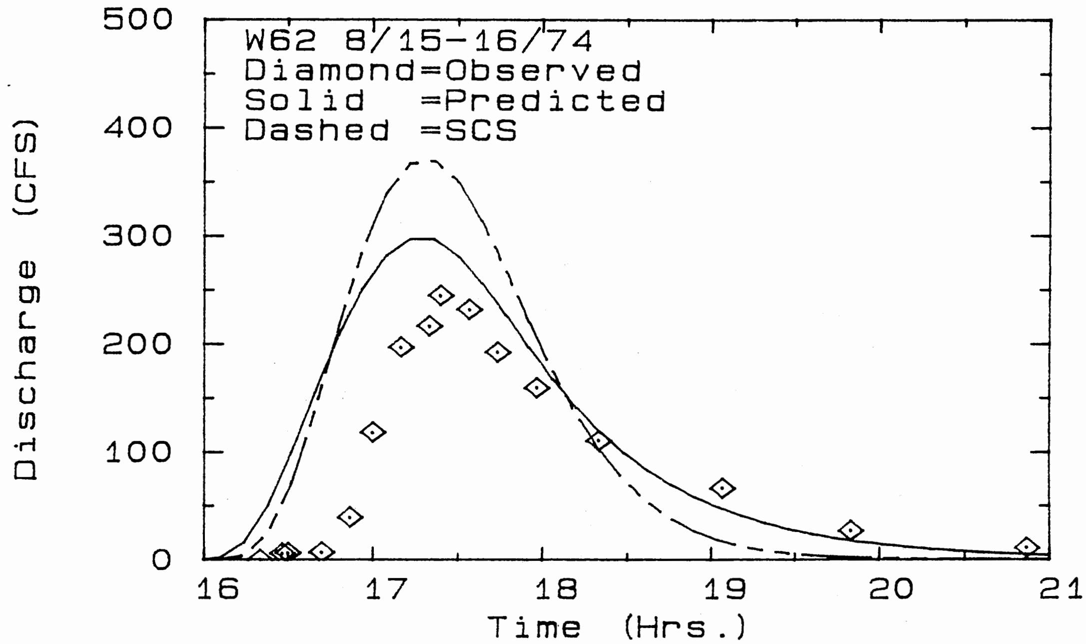

W62 8/29/6
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
160
320
480
640
800
4
6
8
10
12
14
Time (Hrs.)

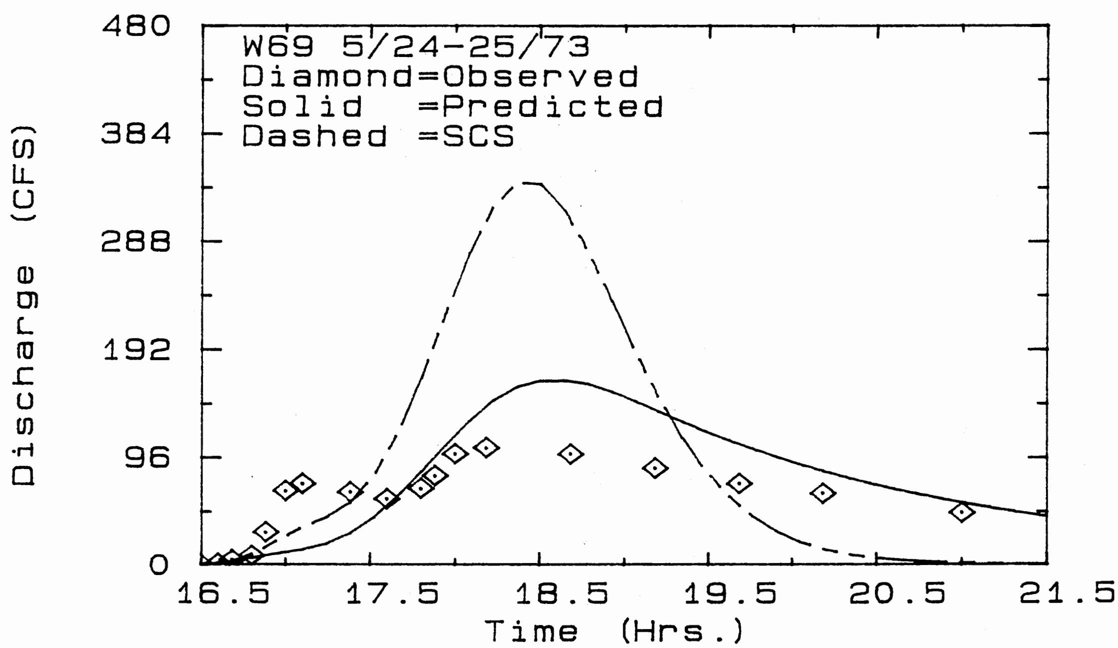

W62 3/3-4/66
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
40
80
120
160
200
14
17
20
23
26
29
Time (Hrs.)

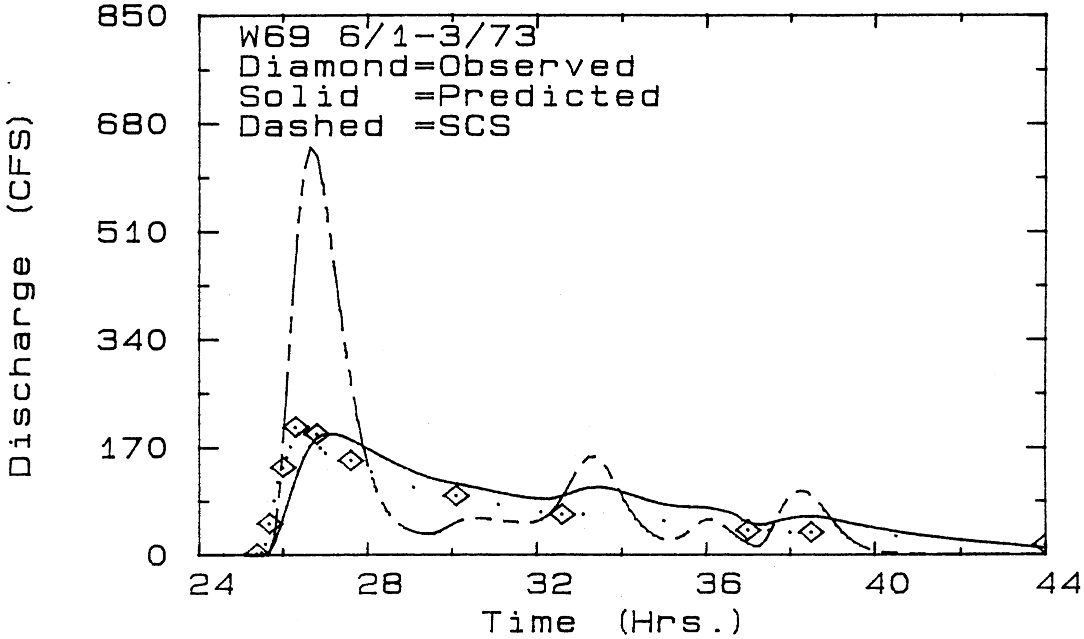

W62 3/4/64
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
500
1000
1500
2000
2500
9
11
13
15
17
19
Time (Hrs.)

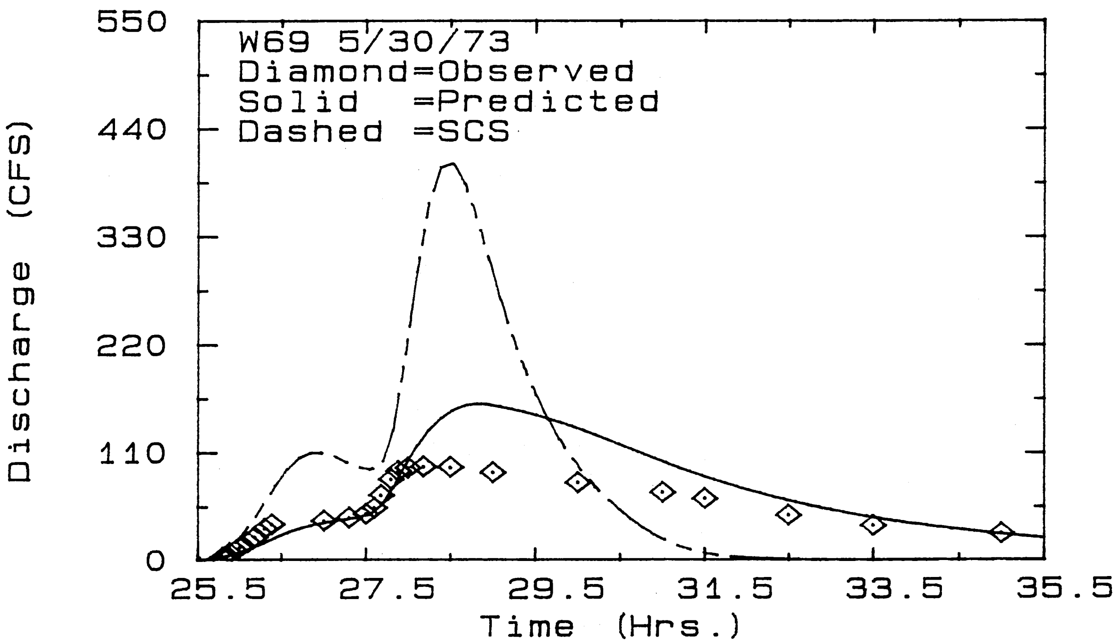

W62 5/31/67
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
370
296
222
148
74
0
14
15
16
17
18
19
Time (Hrs.)

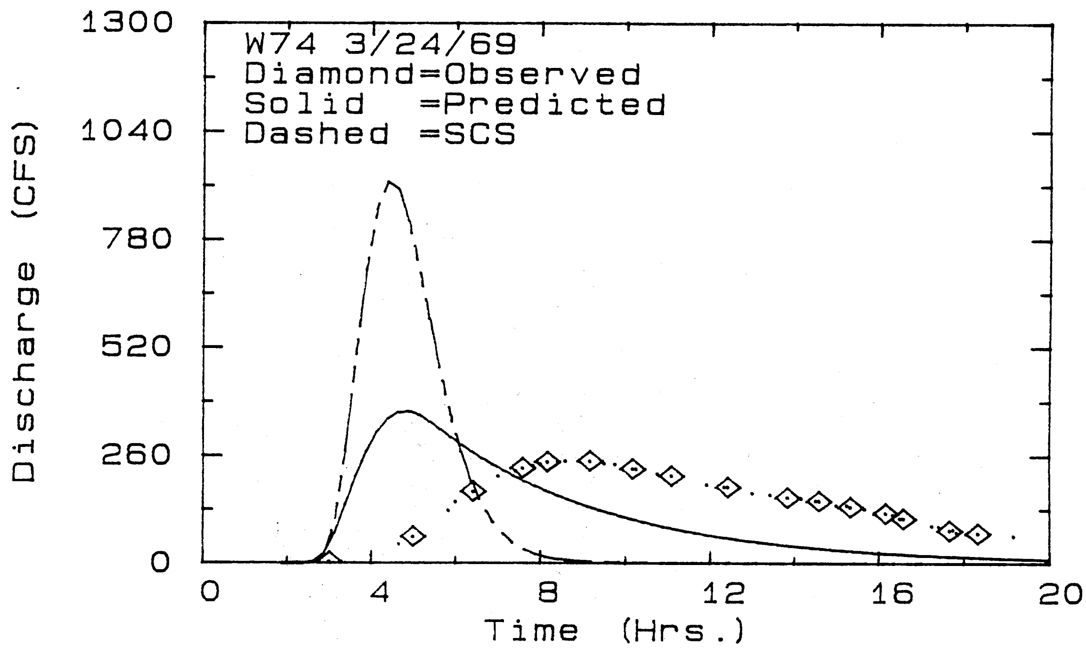

W62 5/24/66
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
110
88
66
44
22
0
5
7
9
11
13
15
Time (Hrs.)


W62 8/15-16/74
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
500
400
300
200
100
0
16
17
18
19
20
21
Time (Hrs.)

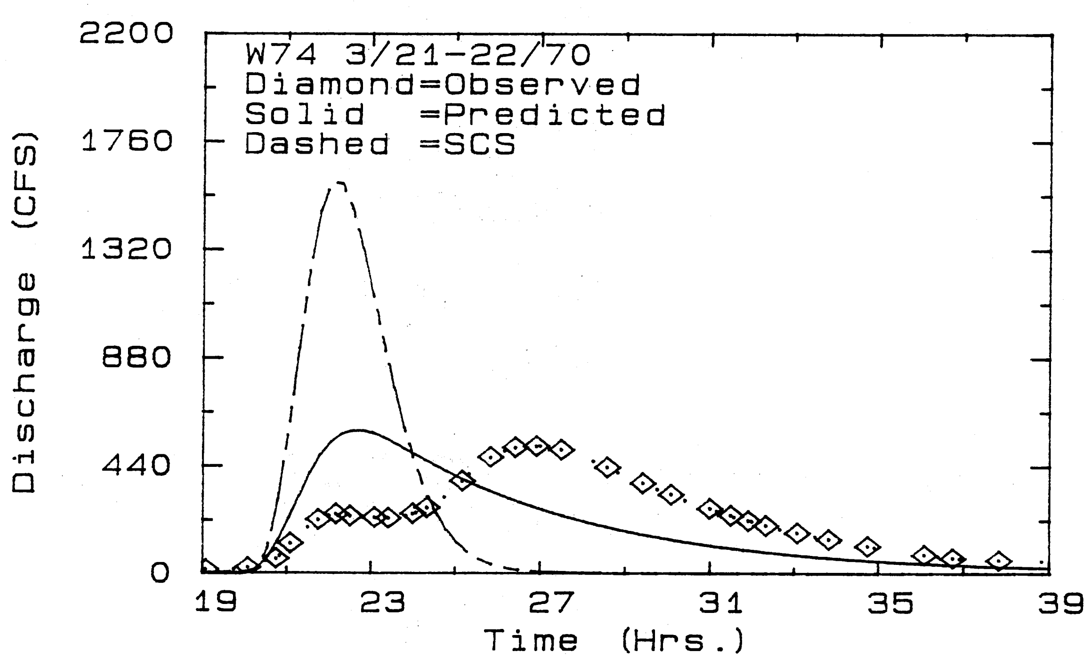

W69 5/24-25/73
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
480
384
288
192
96
0
16.5
17.5
18.5
19.5
20.5
21.5
Time (Hrs.)


W69 6/1-3/73
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
850
680
510
340
170
0
24
28
32
36
40
44
Time (Hrs.)

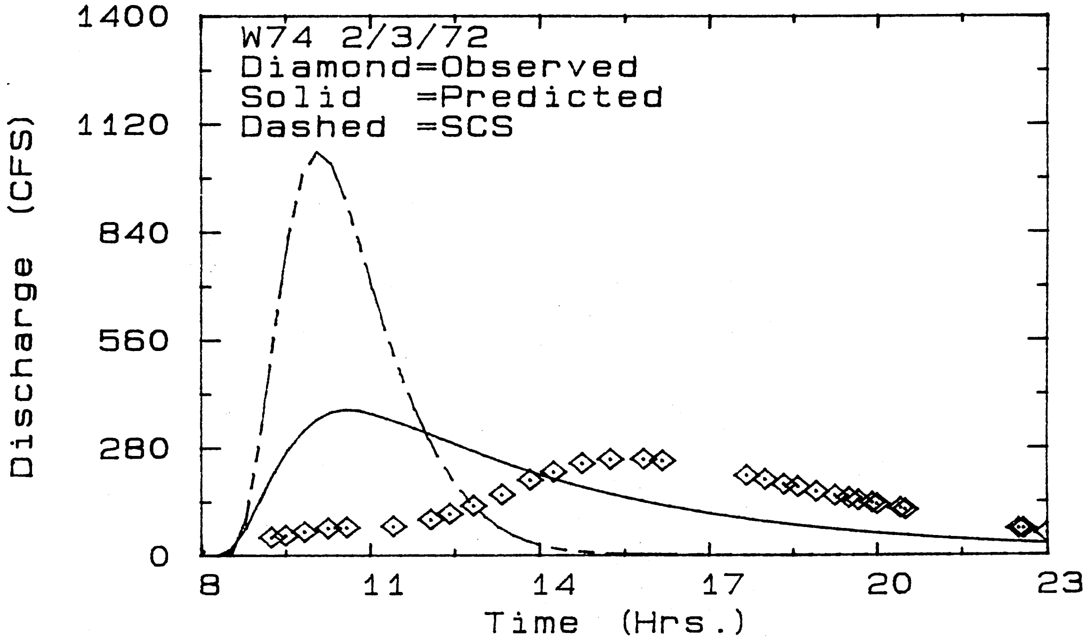

W69 5/30/73
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
550
440
330
220
110
0
25.5
27.5
29.5
31.5
33.5
35.5
Time (Hrs.)

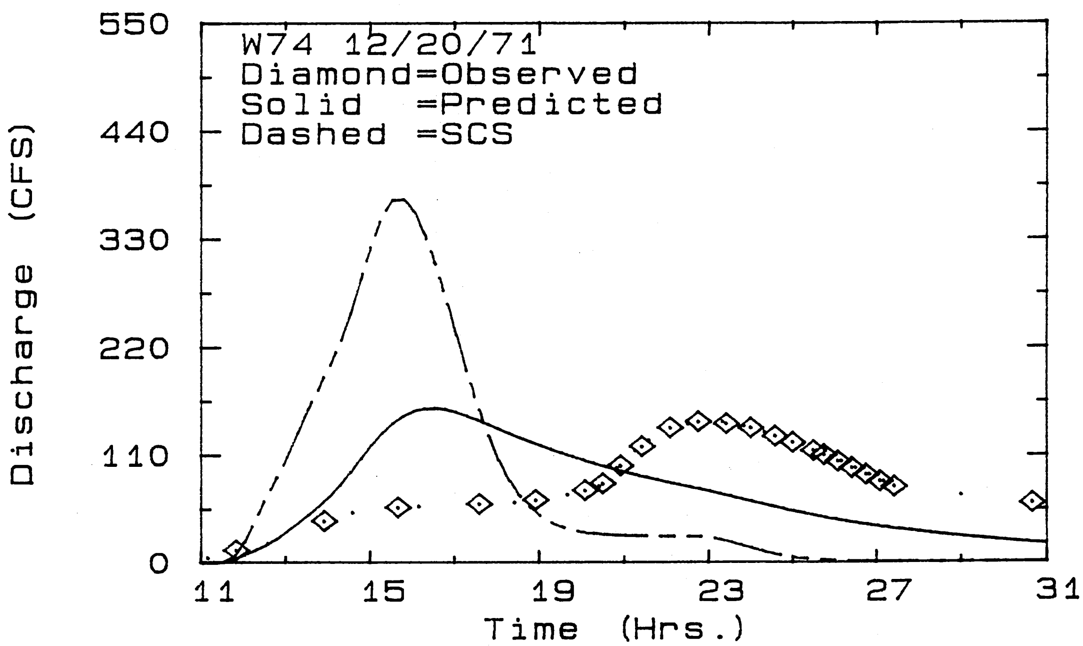


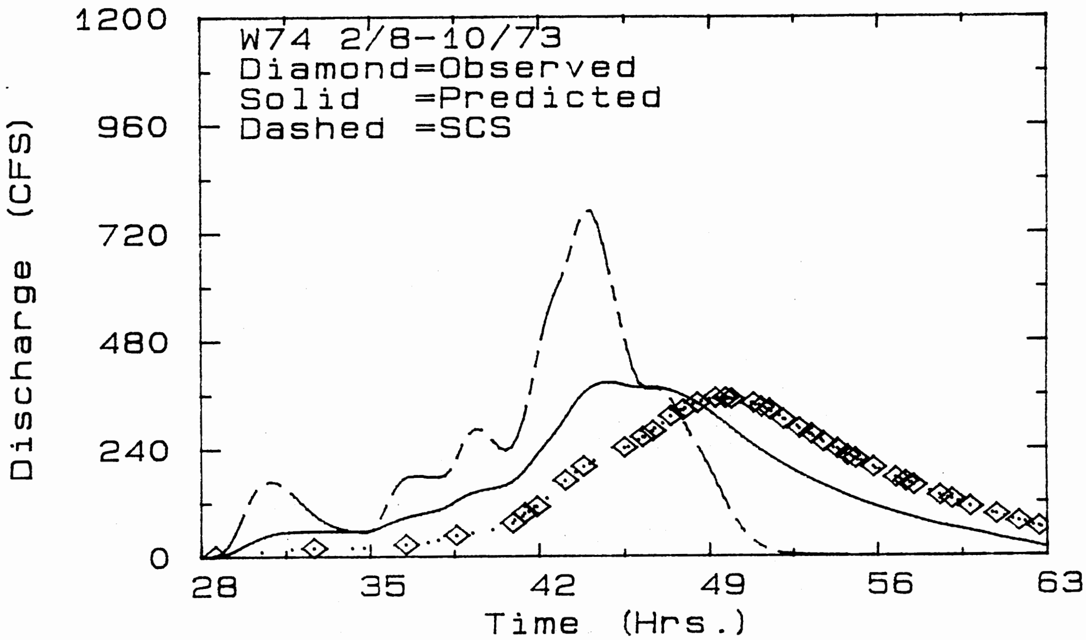
W74 3/24/69
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
1300
1040
780
520
260
0
0
4
8
12
16
20
Time (Hrs.)

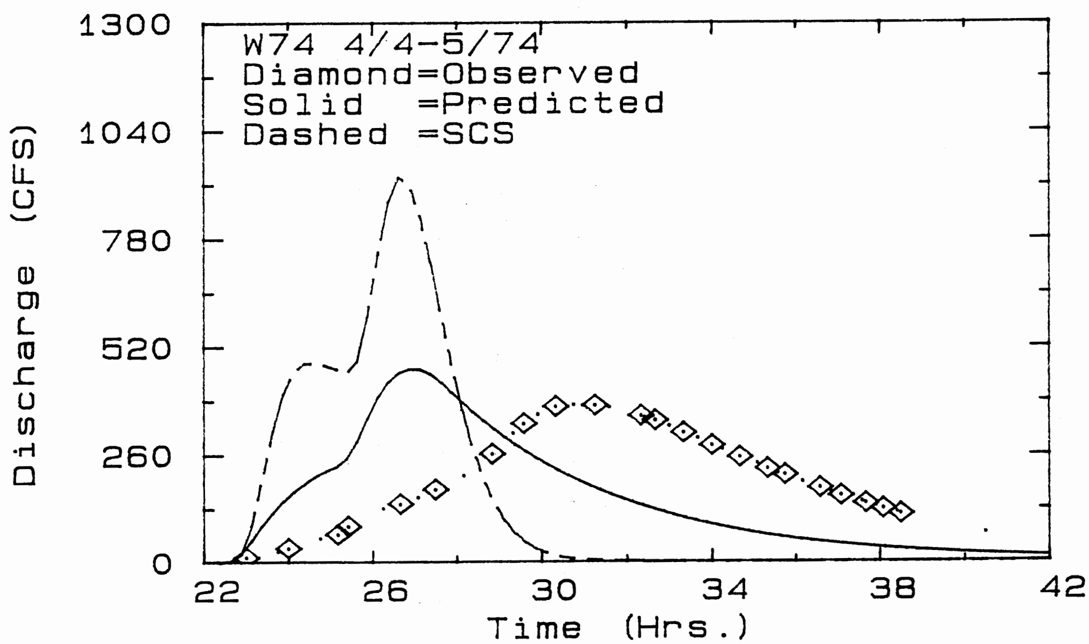


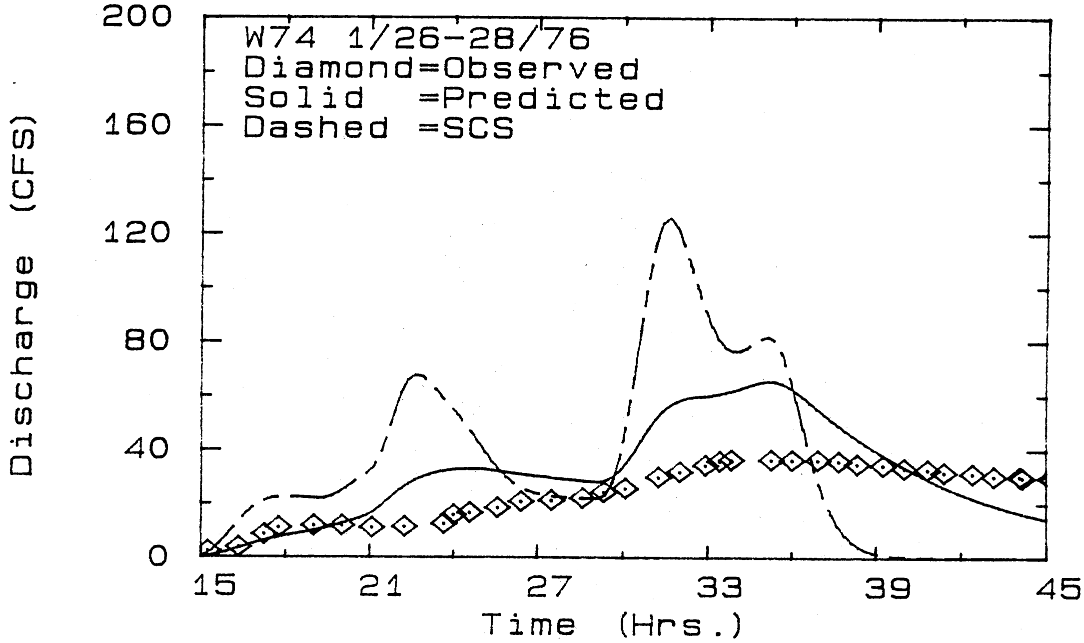
W74 3/21-22/70
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
2200
1760
1320
880
440
0
19
23
27
31
35
39
Time (Hrs.)


W74 2/3/72
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
1400
1120
840
560
280
0
8
11
14
17
20
23
Time (Hrs.)

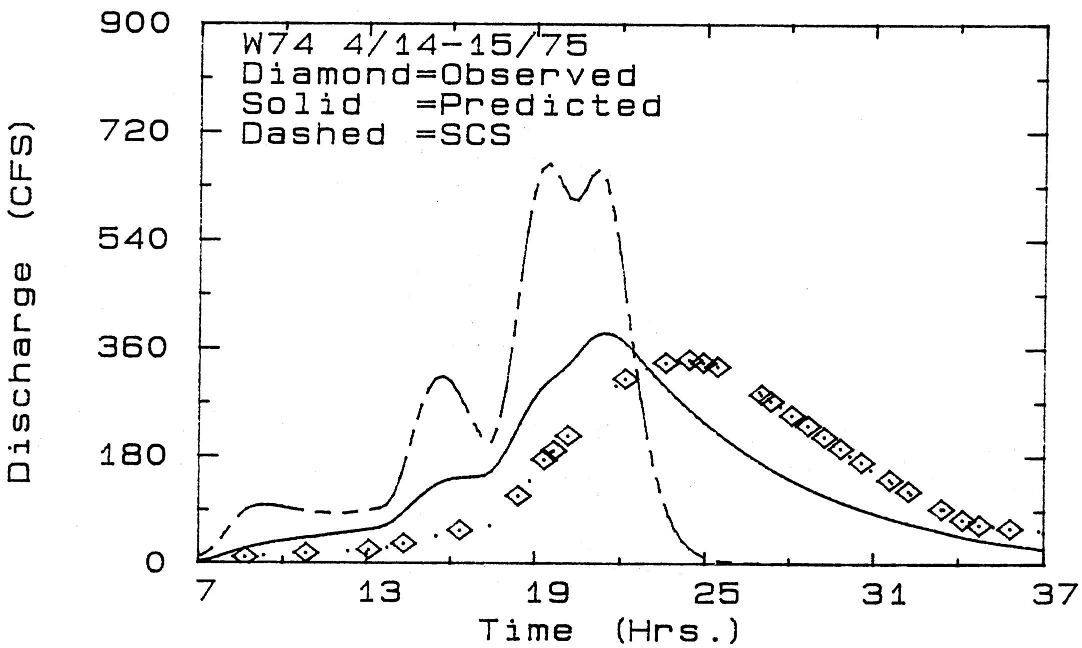

W74 12/20/71
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
550
440
330
220
110
0
11
15
19
23
27
31
Time (Hrs.)


W74 2/8-10/73
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
1200
960
720
480
240
0
28
35
42
49
56
63
Time (Hrs.)

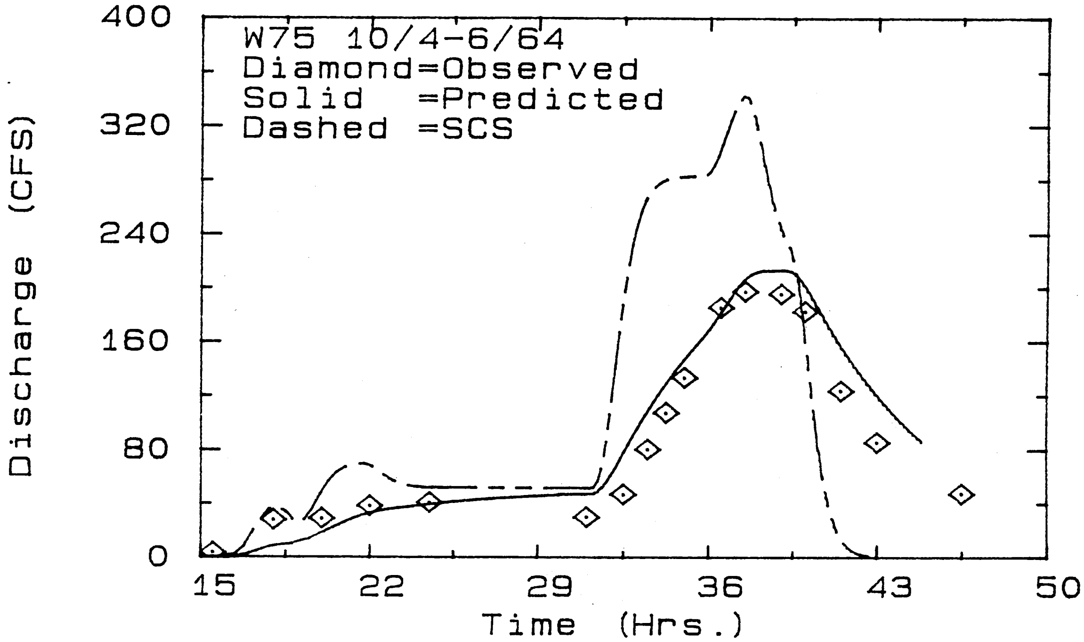

W74 4/4-5/74
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
1300
1040
780
520
260
0
22
26
30
34
38
42
Time (Hrs.)

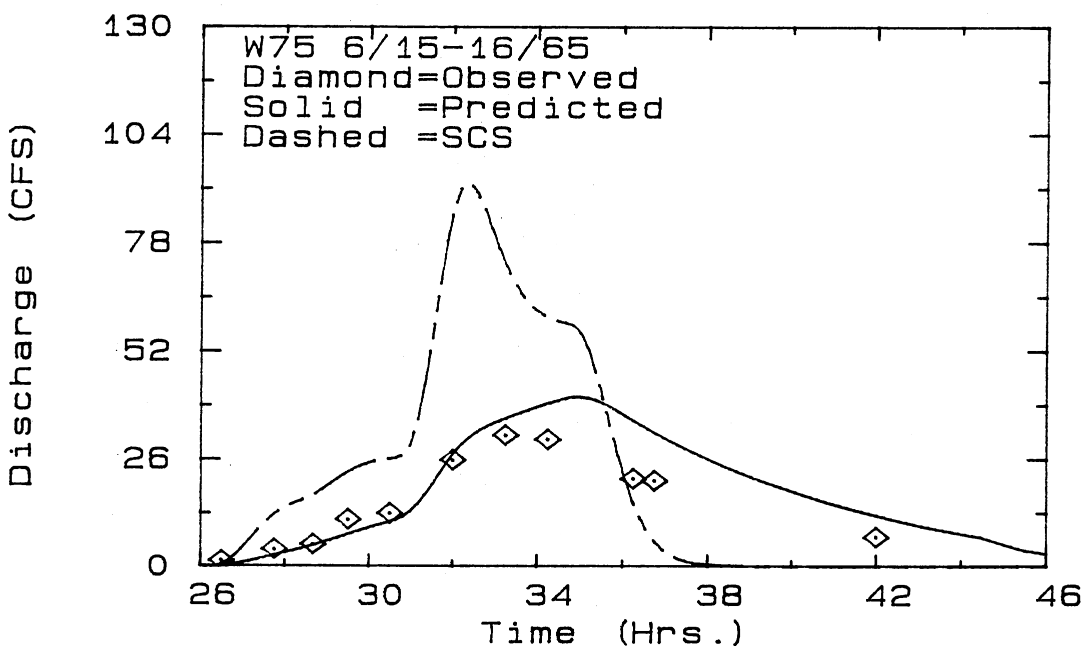


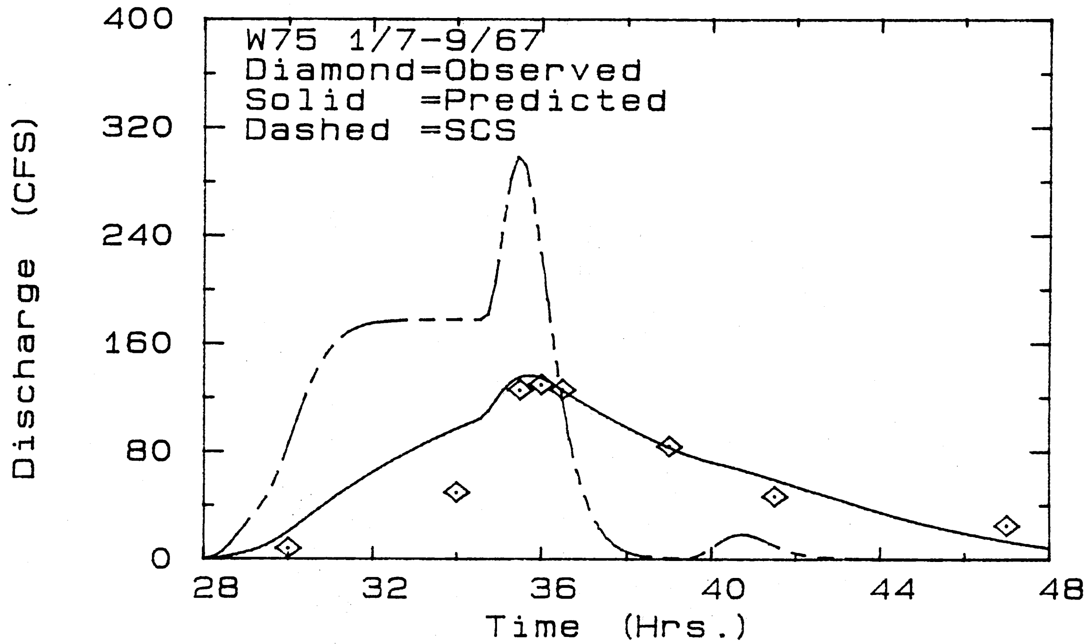
W74 1/26-28/76
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
200
160
120
80
40
0
15
21
27
33
39
45
Time (Hrs.)

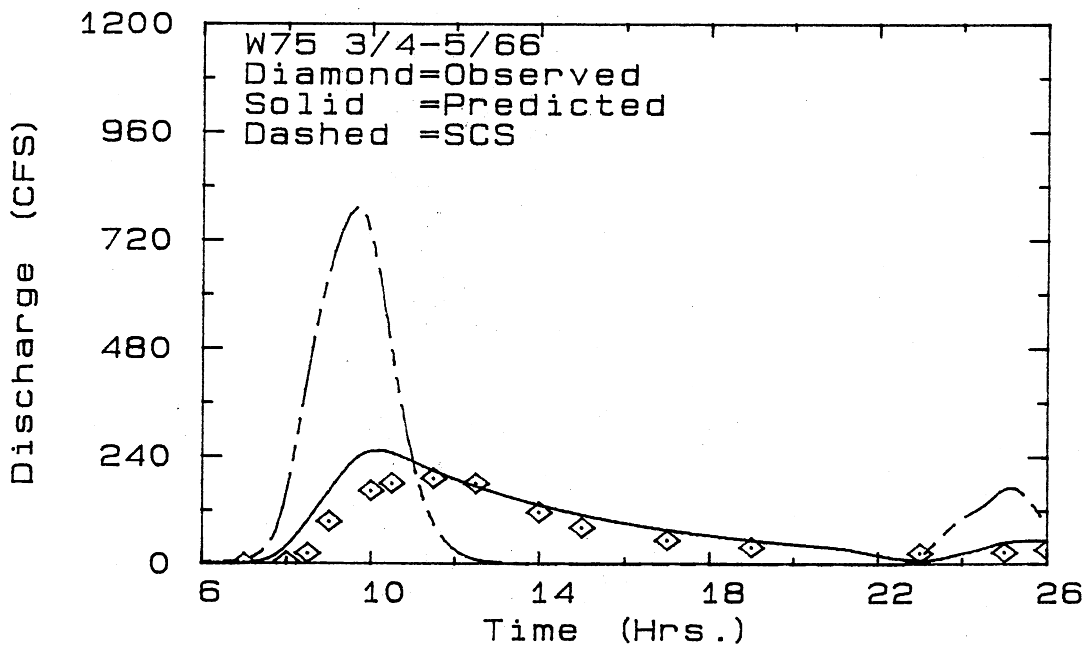


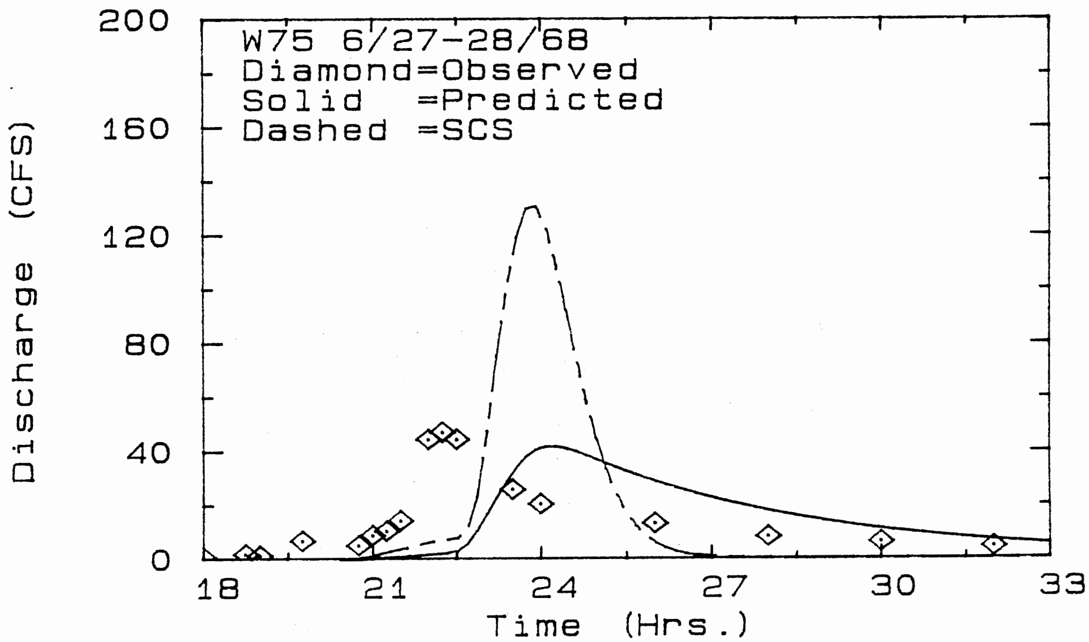
W74 4/14-15/75
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
900
720
540
360
180
0
7
13
19
25
31
37
Time (Hrs.)

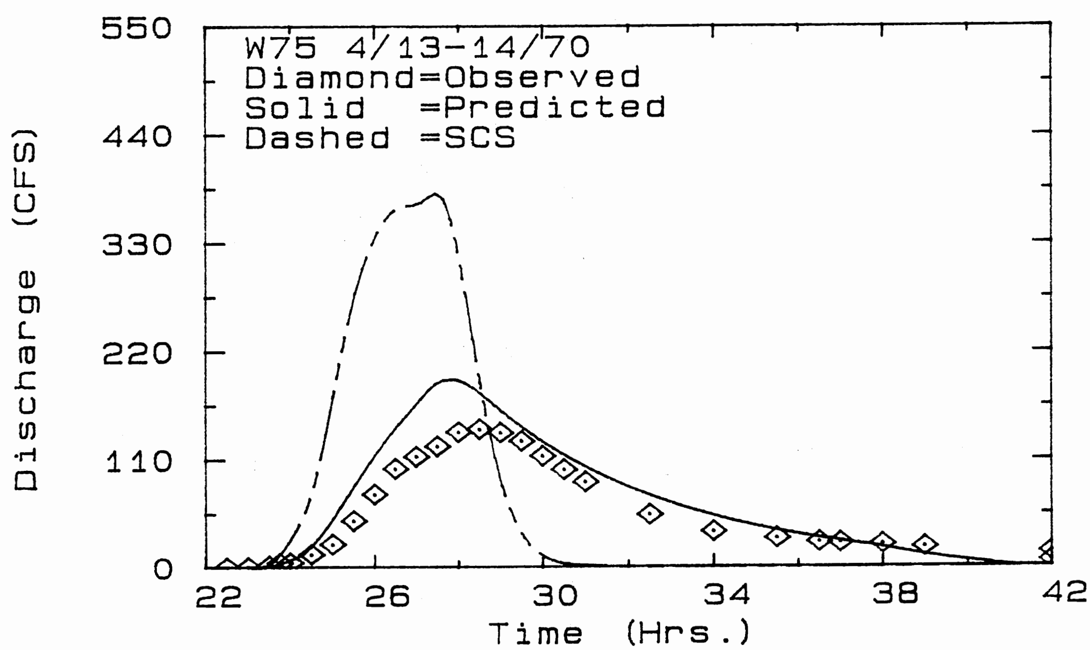

W75 10/4-6/64
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
80
160
240
320
400
15
22
29
36
43
50
Time (Hrs.)

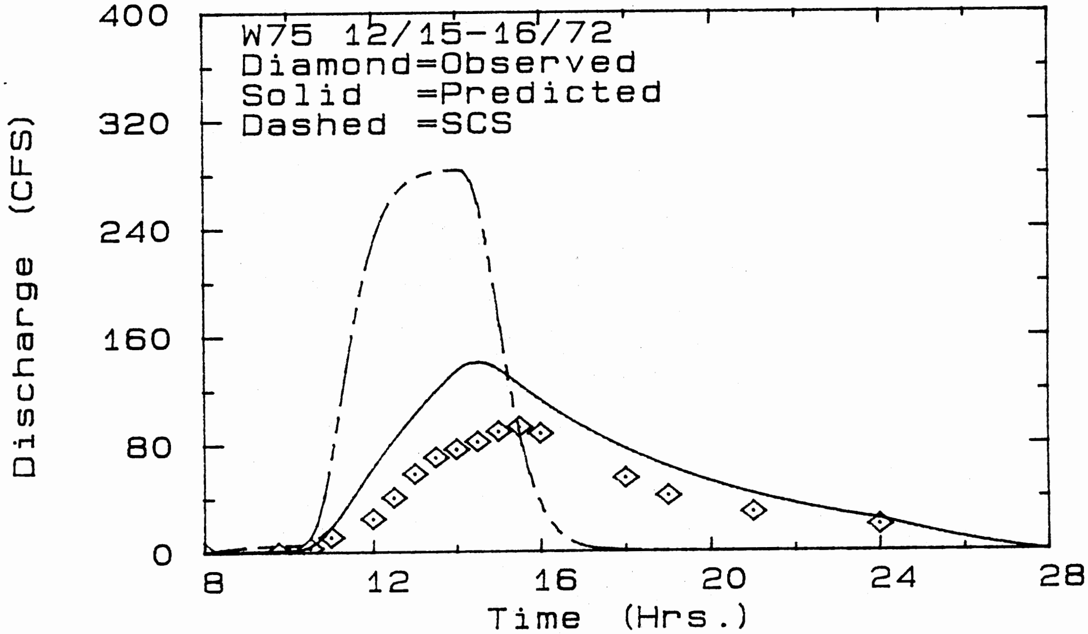

W75 6/15-16/65
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
0
26
52
78
104
130
26
30
34
38
42
46
Time (Hrs.)

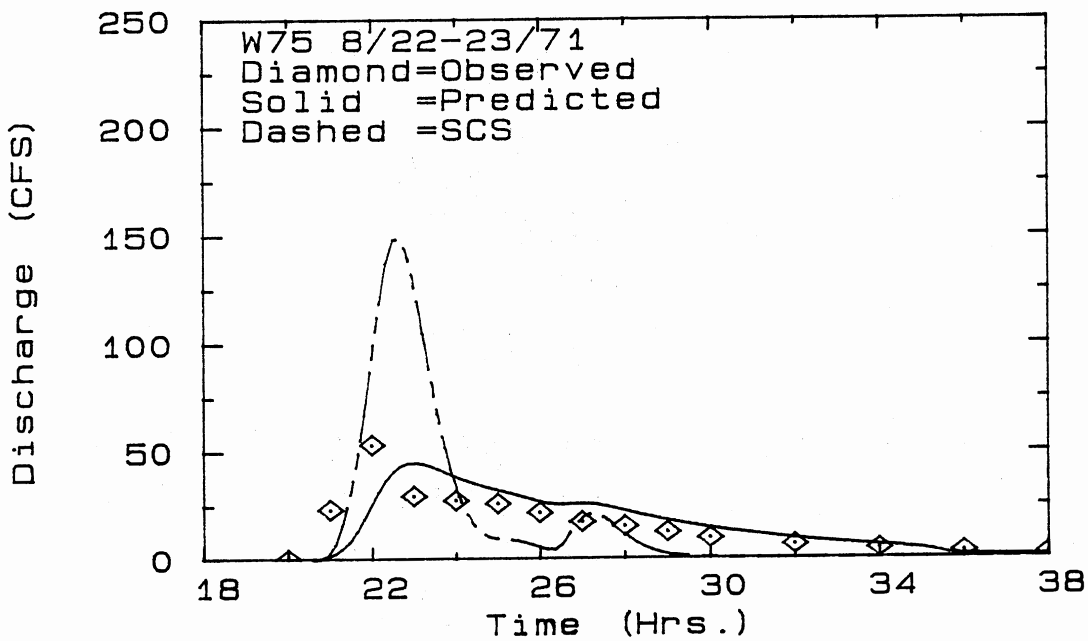

W75 1/7-9/67
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
Time (Hrs.)
0
80
160
240
320
400
28
32
36
40
44
48


W75 3/4-5/66
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
Time (Hrs.)
0
240
480
720
960
1200
6
10
14
18
22
26


W75 6/27-28/68
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
200
160
120
80
40
0
18
21
24
27
30
33
Time (Hrs.)


W75 4/13-14/70
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
550
440
330
220
110
0
22
26
30
34
38
42
Time (Hrs.)


W75 12/15-16/72
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
400
320
240
160
80
0
8
12
16
20
24
28
Time (Hrs.)


W75 8/22-23/71
Diamond=Observed
Solid =Predicted
Dashed =SCS
Discharge (CFS)
250
200
150
100
50
0
18
22
26
30
34
38
Time (Hrs.)

APPENDIX D

CALIBRATION DATA SET

Watershed ID	Storm No.	K*	Maximum Potential Retention S	Rainfall Duration	Peak Intensity Duration
w13011	1	1.3951	3.9631	9.833	0.050
w13011	2	3.9580	1.2736	13.517	0.067
w13011	3	3.7020	4.8701	10.550	0.117
w13011	4	0.6215	4.2302	5.667	0.067
w13011	5	1.2846	1.4539	0.867	0.050
w13011	6	1.0063	2.2271	4.467	0.067
w13011	7	1.1742	5.0494	5.833	0.100
w13011	9	1.6646	3.3199	6.000	0.067
w16006	1	0.7896	8.1164	15.583	0.083
w16006	2	1.7028	4.7423	18.417	0.083
w16006	3	3.4068	4.6432	7.583	0.167
w16006	4	0.7598	3.2042	2.083	0.083
w16006	5	0.0224	5.5808	2.250	0.083
w16006	6	0.8789	3.8758	4.583	0.083
w16006	7	0.2098	4.1833	2.583	0.083
w16006	8	0.8993	3.2875	17.083	0.083
w16006	9	0.0263	4.0128	4.750	0.083
w16006	10	0.0426	0.6583	8.833	0.083
w21001	1	0.4364	4.7402	10.283	0.100
w21001	2	0.0263	3.9708	6.917	0.083
w21001	3	0.5990	1.7563	1.883	0.100
w21001	4	0.9663	0.6363	2.833	0.083
w21001	5	0.3027	3.2554	10.717	0.050
w21001	6	0.0263	4.7664	13.333	0.050
w25001	1	4.0930	1.7274	6.517	0.100
w25001	2	0.4885	1.4109	3.183	0.100
w25001	3	0.3835	0.5188	4.583	0.067
w25001	4	0.3631	0.3595	28.583	0.117
w25001	5	0.8137	0.1343	2.883	0.133
w25001	6	0.8114	0.2604	1.867	0.083
w25001	7	1.0932	3.7585	1.967	0.067
w25001	8	1.7422	0.5643	3.933	0.083
w25001	9	1.9699	0.2816	0.717	0.083
w25001	10	1.0830	0.3605	2.750	0.050
w25001	12	0.3705	1.2990	8.750	0.067
w25001	13	1.6710	0.9881	9.600	0.050
w25001	14	2.4314	0.2125	35.500	0.067
w25001	15	0.6554	0.4362	7.133	0.067
w25001	16	1.1969	0.0149	18.417	0.083
w25001	17	3.1454	1.0981	6.667	0.083
w26030	1	1.6866	1.4314	4.283	0.033
w26030	3	1.1724	1.2110	3.650	0.100
w26030	5	3.2074	0.0237	5.267	0.033

Watershed ID	Storm No.	K*	Maximum Potential Retention S	Rainfall Duration	Peak Intensity Duration
w26030	6	0.9973	0.8418	7.083	0.133
w26030	8	2.8606	4.8209	2.783	0.050
w26030	9	4.6591	1.1349	4.300	0.033
w26030	10	4.1689	4.6113	2.833	0.017
w26030	11	4.4816	0.5143	11.333	0.050
w26030	14	1.8017	3.7452	1.767	0.017
w26030	16	2.7104	6.3996	4.000	0.100
w26033	4	3.4223	0.5275	3.000	0.217
w26033	6	2.6223	1.1886	9.883	0.200
w26036	1	1.4842	1.0123	1.317	0.033
w26036	2	1.0910	1.0569	3.517	0.033
w26036	4	2.4320	0.3752	2.667	0.083
w26036	5	1.1254	0.2023	4.667	0.333
w26036	6	0.7193	0.8507	9.583	0.167
w31004	1	0.0230	3.7307	1.167	0.167
w31004	2	0.2402	3.3386	3.283	0.133
w31004	3	0.3861	1.3574	0.500	0.050
w31004	4	0.8136	1.9543	0.567	0.083
w31004	5	0.8243	4.3663	5.333	0.050
w31004	6	1.0601	1.1960	0.567	0.050
w31004	7	0.5990	13.2124	2.583	0.167
w42002	1	0.0242	1.4372	1.367	0.067
w42002	2	0.7250	0.3116	1.417	0.050
w42002	3	0.6358	1.7866	6.800	0.033
w42002	4	0.5140	1.1068	6.833	0.083
w42002	5	1.2614	0.3504	8.100	0.117
w42002	6	1.2087	0.1793	4.333	0.033
w44001	1	1.3406	2.3359	0.850	0.183
w44001	2	0.3499	1.2786	1.933	0.050
w44001	3	0.7441	0.3900	0.333	0.083
w44001	4	0.5765	1.4025	4.517	0.167
w44001	5	0.3694	5.1705	4.867	0.250
w61002	1	0.2337	1.3552	1.817	0.350
w61002	2	0.6729	2.1636	0.833	0.150
w61002	3	0.0200	2.4263	0.167	0.167
w61002	4	0.1563	1.3469	2.000	0.100
w61002	5	3.8541	1.5534	2.250	0.083
w61002	6	0.0218	5.1498	3.450	0.050
w61002	7	1.1884	0.0359	3.083	0.333
w61002	8	0.8241	0.7778	2.283	0.150
w62007	1	0.5396	4.1017	3.350	0.317
w62007	2	0.5208	6.3285	4.750	0.250
w62007	3	3.2776	1.5603	5.000	0.250

Watershed ID	Storm No.	K*	Maximum Potential Retention S	Rainfall Duration	Peak Intensity Duration
w62007	4	0.8611	3.3892	5.250	0.250
w62007	5	1.9443	2.1300	2.250	0.250
w62007	6	1.8725	1.9839	9.250	0.250
w68003	2	0.6391	2.5412	66.550	0.317
w68003	3	1.3737	1.9632	20.317	1.383
w68003	6	0.3620	8.4390	45.217	0.267
w68003	10	3.6937	0.6766	17.700	0.633
w68003	11	3.1722	1.5461	30.117	2.783
w68003	12	0.3295	1.1467	0.167	0.167
w69008	1	0.0263	6.6943	8.717	0.100
w69008	2	0.0712	2.0999	3.800	0.083
w69008	3	0.0263	6.1906	6.650	0.733
w69008	4	0.0263	3.2809	3.000	0.400
w69008	5	0.0263	5.2689	8.083	0.050
w69008	6	0.3304	3.5677	4.500	0.167
w69008	7	1.3855	2.5902	3.750	0.067
w69008	8	0.2392	3.8691	9.200	0.083
w69008	9	0.2281	4.1916	10.517	0.133
w69008	10	0.0263	9.0538	5.583	0.117
w69008	11	0.8383	3.4187	6.117	0.033
w69008	12	2.2607	4.6774	9.283	0.100
w69008	13	0.9703	2.4188	1.350	0.017
w69008	14	0.4770	4.8249	5.833	0.017
w69008	15	1.8789	1.5137	3.567	0.033
w69009	1	0.7602	2.5060	0.583	0.083
w69009	2	0.5681	2.2981	0.900	0.133
w69009	3	0.0201	1.6531	5.667	0.150
w69009	4	0.9094	6.1440	1.817	0.050
w69009	5	0.0269	1.6563	5.667	0.150
w69009	6	0.9094	6.1440	1.817	0.067
w69009	7	0.0263	2.6413	3.217	0.067
w69009	8	0.8527	4.0413	4.183	0.183
w69009	9	0.0263	6.8061	5.483	0.100
w69009	10	0.3135	2.7201	6.150	0.050
w69009	11	0.2603	9.0362	7.750	0.050
w69009	12	0.0318	13.9224	16.650	0.167
w69009	13	0.2328	6.3072	8.950	0.083
w69009	14	0.4507	2.1488	2.467	0.050
w69009	15	0.0263	5.3496	4.200	0.083
w69009	16	0.6135	2.2545	3.733	0.017
w69032	1	0.8458	1.0176	2.983	0.133
w69032	3	0.3485	1.1296	8.517	0.100
w69032	4	0.3054	2.2900	4.683	0.133

Watershed ID	Storm No.	K*	Maximum Potential Retention S	Rainfall Duration	Peak Intensity Duration
w69032	5	0.4379	1.1863	6.200	0.117
w69032	6	0.7406	4.1690	12.067	0.067
w69032	7	1.0353	2.4550	13.433	0.017
w69032	8	2.3122	0.7891	1.817	0.017
w69032	9	1.5197	1.8376	7.117	0.033
w69032	10	1.9418	2.1631	2.333	0.017
w69032	11	4.2262	4.1269	8.883	0.017
w69042	1	0.6629	2.8702	8.650	0.333
w69042	2	2.0459	2.1647	6.333	0.183
w69042	3	0.0909	10.9385	11.983	0.083
w69042	4	1.2482	2.3239	1.450	0.017
w69042	5	3.9359	0.8103	1.917	0.017
w69042	6	1.2687	2.8147	5.917	0.033
w69042	7	2.6362	3.1255	6.250	0.033
w69042	8	2.3178	1.3447	2.950	0.017
w69042	9	1.9589	3.2014	2.917	0.067
w69042	10	0.0604	4.9612	5.650	0.017
w71001	1	0.2014	0.6688	0.950	0.083
w71001	2	0.2369	2.0399	1.833	0.067
w71001	3	0.0285	0.1987	1.933	0.050
w71001	4	0.0263	0.9701	1.500	0.033
w71001	5	0.0248	0.7754	0.967	0.050
w71001	6	0.0753	0.3197	1.683	0.033
w71001	7	0.0263	2.0461	2.900	0.150
w71001	8	0.0239	0.4229	1.383	0.067
w71001	9	0.0426	2.7453	1.783	0.050
w71001	10	0.4289	0.3382	3.167	0.050
w71001	11	0.0239	1.8292	1.833	0.067
w74009	2	4.0792	5.8379	20.183	0.083
w74009	3	1.3638	4.0463	5.017	0.083
w74009	4	0.9776	7.6614	8.933	0.083
w74009	5	2.5289	7.4787	27.383	0.083
w75003	1	4.3060	0.6706	32.500	1.250
w75003	4	3.1044	3.6030	39.000	2.250
w75003	5	2.5426	4.7768	6.750	0.083

Watershed ID	Storm No.	Excess Rainfall Duration	Peak Excess Intensity Duration	Peak Rainfall Intensity	Peak Excess Rainfall Intensity
w13011	1	7.367	0.117	3.6000	1.6388
w13011	2	10.950	0.067	1.0500	0.8532
w13011	3	5.750	0.117	2.7400	1.5068
w13011	4	5.167	0.067	6.1500	2.8361
w13011	5	0.733	0.050	2.4000	1.3047
w13011	6	4.300	0.067	4.0500	1.6435
w13011	7	2.933	0.100	0.9999	0.5165
w13011	9	4.050	0.067	4.5001	2.9137
w16006	1	6.917	0.083	2.4000	4.2329
w16006	2	4.000	0.083	4.7999	2.0253
w16006	3	2.833	0.250	0.5999	0.8484
w16006	4	1.500	0.083	1.2001	1.8018
w16006	5	2.000	0.083	6.0004	3.0505
w16006	6	3.333	0.083	2.4000	1.9689
w16006	7	1.750	0.083	2.4000	2.7607
w16006	8	11.667	0.083	2.4001	1.4717
w16006	9	1.667	0.083	4.8001	2.8502
w16006	10	8.417	0.083	1.2000	0.8502
w21001	1	9.400	0.100	5.0000	3.5483
w21001	2	6.567	0.083	5.4000	2.9812
w21001	3	1.583	0.100	4.0000	2.8295
w21001	4	2.300	0.083	3.9600	3.6116
w21001	5	7.083	0.250	2.0000	1.2072
w21001	6	9.950	0.050	6.8000	4.9040
w25001	1	5.950	0.100	2.6000	1.8590
w25001	2	2.783	0.100	3.7000	2.4144
w25001	3	3.700	0.067	3.3000	2.8700
w25001	4	28.200	0.117	3.4300	3.1317
w25001	5	2.883	0.133	1.6500	1.5784
w25001	6	1.867	0.083	2.5200	2.3476
w25001	7	1.233	0.067	9.3000	6.1964
w25001	8	3.667	0.083	3.0000	2.2582
w25001	9	0.717	0.083	5.1600	4.6224
w25001	10	2.450	0.050	6.2000	5.5340
w25001	12	8.200	0.067	5.2501	4.9391
w25001	13	9.600	0.050	4.6000	4.6000
w25001	14	35.500	0.067	3.0000	2.9088
w25001	15	7.033	0.067	6.0000	5.0027
w25001	16	18.417	0.083	2.1597	2.1599
w25001	17	6.667	0.083	3.5999	2.7052
w26030	1	4.217	0.050	5.1000	2.9039
w26030	3	3.517	0.100	4.0000	1.9917
w26030	5	5.267	0.033	1.8000	1.7982

Watershed ID	Storm No.	Excess Rainfall Duration	Peak Excess Intensity Duration	Peak Rainfall Intensity	Peak Excess Rainfall Intensity
w26030	6	6.367	0.133	1.2000	1.0360
w26030	8	1.617	0.050	3.8000	4.2669
w26030	9	4.300	0.033	1.4997	0.9736
w26030	10	2.533	0.033	13.8000	7.5799
w26030	11	11.333	0.050	2.2000	1.4273
w26030	14	1.417	0.017	10.2025	3.1971
w26030	16	1.983	0.067	2.4000	4.3382
w26033	4	2.833	0.217	1.7077	1.2513
w26033	6	8.000	0.200	1.3000	1.0426
w26036	1	1.250	0.033	5.4000	3.3882
w26036	2	3.450	0.033	4.8000	3.7906
w26036	4	2.667	0.133	1.1999	1.1962
w26036	5	4.667	0.333	0.7496	0.7213
w26036	6	9.583	0.167	1.1400	0.9911
w31004	1	1.000	0.167	4.9200	2.9106
w31004	2	3.150	0.033	4.5000	2.0596
w31004	3	0.350	0.100	5.2000	3.3106
w31004	4	0.383	0.083	5.4000	2.8323
w31004	5	4.700	0.050	7.0000	5.0660
w31004	6	0.567	0.050	4.8000	3.0885
w31004	7	1.133	0.167	4.5600	3.5861
w42002	1	1.283	0.067	5.2500	3.9244
w42002	2	1.417	0.050	4.0000	3.7721
w42002	3	6.300	0.033	5.4000	4.1616
w42002	4	4.667	0.083	7.7300	6.2760
w42002	5	8.100	0.117	1.1100	1.0567
w42002	6	4.000	0.033	2.7000	2.6547
w44001	1	0.550	0.083	4.3100	1.8813
w44001	2	1.833	0.050	5.4000	4.1819
w44001	3	0.333	0.083	4.9200	2.7940
w44001	4	4.517	0.167	4.2000	3.3023
w44001	5	3.833	0.250	2.6800	1.5600
w61002	1	0.500	0.350	1.4857	0.9514
w61002	2	0.683	0.167	1.7334	0.6640
w61002	3	0.167	0.167	3.6600	0.8752
w61002	4	1.833	0.117	2.0999	1.0345
w61002	5	1.750	0.083	3.6000	1.7215
w61002	6	3.233	0.050	10.0000	6.9253
w61002	7	3.083	0.333	0.4500	0.3778
w61002	8	1.767	0.150	4.0667	3.0763
w62007	1	2.683	0.317	2.4600	0.6117
w62007	2	4.250	0.250	2.5600	1.3231
w62007	3	3.000	0.250	1.3600	0.9902

Watershed ID	Storm No.	Excess Rainfall Duration	Peak Excess Intensity Duration	Peak Rainfall Intensity	Peak Excess Rainfall Intensity
w62007	4	4.250	0.250	1.6000	0.7947
w62007	5	1.250	0.250	1.3600	0.7179
w62007	6	2.500	0.250	0.9644	1.6751
w68003	2	48.067	0.467	0.9870	0.2349
w68003	3	12.283	1.383	0.1518	0.0944
w68003	6	24.417	0.300	1.7625	1.3009
w68003	10	14.233	0.633	0.1421	0.1040
w68003	11	29.100	2.783	0.1114	0.0236
w68003	12	0.167	0.167	1.8600	0.4698
w69008	1	3.883	0.183	3.4998	2.3275
w69008	2	3.067	0.083	4.8003	3.4205
w69008	3	5.917	0.200	1.6364	1.4078
w69008	4	2.483	0.400	1.9000	0.4569
w69008	5	4.983	0.883	3.1998	0.2535
w69008	6	3.017	0.417	1.9201	0.7002
w69008	7	2.833	0.067	3.7500	1.8445
w69008	8	8.283	0.550	2.0400	0.3123
w69008	9	10.133	0.133	2.6250	1.4000
w69008	10	4.783	0.300	5.5714	0.3407
w69008	11	1.383	0.050	7.4984	3.2282
w69008	12	5.217	0.100	2.2999	2.4588
w69008	13	1.150	0.017	7.2000	6.6084
w69008	14	4.850	0.017	10.2025	5.4801
w69008	15	0.300	0.033	7.5000	4.8885
w69009	1	0.333	0.083	5.5200	2.8557
w69009	2	0.767	0.100	3.0800	1.3161
w69009	3	5.450	0.083	2.1300	0.5545
w69009	4	1.583	0.050	4.2000	5.4381
w69009	5	5.450	0.083	2.1300	0.5538
w69009	6	1.583	0.050	5.8500	5.4381
w69009	7	2.133	0.067	3.7500	1.6071
w69009	8	0.817	0.067	2.4545	2.7105
w69009	9	4.800	0.117	6.4000	4.0594
w69009	10	1.367	0.067	5.3997	3.0188
w69009	11	2.433	0.050	5.5997	8.6685
w69009	12	9.433	0.217	2.8200	2.9732
w69009	13	5.133	0.083	2.6402	3.1518
w69009	14	2.400	0.050	4.4000	2.0182
w69009	15	2.900	0.083	4.2000	1.7919
w69009	16	0.350	0.017	10.8000	6.2938
w69032	1	2.683	0.150	2.2500	1.8054
w69032	3	7.950	0.100	3.6000	2.8311
w69032	4	4.450	0.150	3.3000	1.6953

Watershed ID	Storm No.	Excess Rainfall Duration	Peak Excess Intensity Duration	Peak Rainfall Intensity	Peak Excess Rainfall Intensity
w69032	5	6.200	0.167	2.6571	1.1626
w69032	6	11.550	0.117	4.0500	1.5505
w69032	7	8.350	0.017	1.8004	1.5422
w69032	8	1.750	0.017	11.4028	8.5509
w69032	9	5.517	0.033	5.3988	4.0301
w69032	10	1.767	0.017	6.0014	4.9261
w69032	11	7.833	0.083	2.4000	2.1773
w69042	1	8.067	0.333	2.1300	1.3459
w69042	2	3.800	0.183	3.3273	1.9682
w69042	3	7.667	0.183	4.4400	2.4305
w69042	4	1.350	0.017	10.8026	8.2753
w69042	5	0.500	0.017	12.6031	9.8961
w69042	6	5.033	0.033	7.1985	3.9070
w69042	7	4.467	0.050	3.0000	2.5357
w69042	8	2.733	0.017	4.2000	4.4019
w69042	9	2.017	0.467	1.5000	0.2977
w69042	10	5.300	0.017	7.8000	13.8606
w71001	1	0.517	0.083	4.4400	3.7895
w71001	2	0.967	0.067	5.2500	2.8834
w71001	3	1.667	0.050	2.4000	1.8868
w71001	4	1.167	0.050	6.9000	4.8009
w71001	5	0.967	0.050	4.8000	2.8887
w71001	6	1.550	0.033	6.6000	5.8065
w71001	7	2.617	0.100	6.5300	5.0859
w71001	8	1.383	0.067	6.9001	6.4593
w71001	9	1.683	0.050	11.5993	6.8275
w71001	10	3.167	0.050	5.5997	4.8027
w71001	11	1.633	0.067	4.0491	2.5500
w74009	2	12.250	0.333	1.2000	0.7359
w74009	3	2.417	0.083	2.4000	2.3495
w74009	4	5.167	0.333	1.2001	1.4762
w74009	5	23.583	0.167	2.4000	1.8041
w75003	1	32.500	1.250	0.3900	0.3113
w75003	4	30.250	3.500	0.4700	0.2657
w75003	5	5.917	0.083	4.9203	2.4003

Watershed ID	Storm No.	Stand. Dev. Rain	Stand. Dev. Rain Excess	Coef. Skew Rain	Coef. Skew Rain Excess
w13011	1	1.3241	1.1533	1.2683	3.1501
w13011	2	2.9112	2.7624	0.7783	0.8361
w13011	3	2.3946	1.2141	-0.8394	-0.3125
w13011	4	0.6419	0.7319	4.4067	3.8805
w13011	5	0.0963	0.0796	1.4131	3.2289
w13011	6	1.1055	1.2506	1.6195	0.9460
w13011	7	1.8382	0.9118	-0.3357	0.1248
w13011	9	0.9588	0.8002	0.2774	1.6795
w16006	1	3.1810	1.4112	-0.4438	2.6208
w16006	2	7.2139	0.6074	0.2548	2.2988
w16006	3	1.7024	0.8651	-0.1488	1.2955
w16006	4	0.2910	0.1336	0.0070	0.6077
w16006	5	0.1911	0.1839	1.9850	2.0747
w16006	6	0.7868	0.7426	0.9924	1.1390
w16006	7	0.6272	0.5493	1.5321	1.9642
w16006	8	3.5597	2.6950	-0.4093	0.1552
w16006	9	0.5380	0.3384	-1.1442	1.9874
w16006	10	1.4625	1.4479	0.8747	0.6409
w21001	1	1.9692	1.7921	0.4744	0.4924
w21001	2	1.8713	1.8215	0.7488	0.4602
w21001	3	0.4467	0.3117	-0.6362	-0.8224
w21001	4	0.6704	0.6269	0.1910	0.0615
w21001	5	2.2114	1.8071	0.3540	1.3821
w21001	6	3.2888	2.7557	0.1389	0.1904
w25001	1	2.5354	2.3617	0.1062	-0.4194
w25001	2	0.4896	0.3921	0.1305	1.2605
w25001	3	0.7310	0.7646	2.6515	2.5238
w25001	4	8.1960	7.8515	-0.2014	-0.2864
w25001	5	0.7119	0.6975	0.6659	0.6326
w25001	6	0.4944	0.4622	-0.2373	-0.4689
w25001	7	0.2137	0.1799	1.2107	3.0890
w25001	8	0.9455	0.9911	1.9327	1.7140
w25001	9	0.1309	0.1237	0.8096	1.0010
w25001	10	0.6494	0.6710	1.6197	1.4458
w25001	12	1.7202	1.5781	0.3237	0.4962
w25001	13	2.1230	2.1230	0.3893	0.3893
w25001	14	7.5941	7.5051	0.6050	0.6076
w25001	15	1.2096	1.2224	1.9697	1.9197
w25001	16	5.7276	5.7122	-0.0002	-0.0053
w25001	17	1.6760	1.5301	0.1472	0.0722
w26030	1	0.7535	0.7376	0.8310	0.6138
w26030	3	0.3835	0.4436	5.9265	5.1814
w26030	5	1.1169	1.1173	1.6956	1.6893

Watershed ID	Storm No.	Stand. Dev. Rain	Stand. Dev. Rain Excess	Coef. Skew Rain	Coef. Skew Rain Excess
w26030	6	1.7429	1.4539	-0.2879	-0.2730
w26030	8	0.5202	0.2743	0.0550	1.8189
w26030	9	1.1086	0.9614	-0.2383	-0.8904
w26030	10	0.6273	0.7483	2.8421	2.0371
w26030	11	2.3019	2.4730	2.0630	1.7305
w26030	14	0.3999	0.3374	0.3823	0.2826
w26030	16	0.6696	0.3477	-0.9365	1.9950
w26033	4	0.6325	0.4755	-0.6493	-0.7517
w26033	6	1.7435	1.4039	0.0208	0.6895
w26036	1	0.1672	0.1649	2.1881	2.7372
w26036	2	0.6787	0.6548	0.4069	0.0585
w26036	4	0.5567	0.4433	-0.6808	-0.6041
w26036	5	1.1687	1.1817	1.2757	1.1638
w26036	6	1.8086	1.6266	0.3330	0.5190
w31004	1	0.2143	0.1878	0.8461	1.1131
w31004	2	0.5167	0.5103	1.3029	1.2853
w31004	3	0.0767	0.0598	0.3574	1.4447
w31004	4	0.1148	0.0904	0.1173	0.5031
w31004	5	1.4614	1.4901	1.0220	0.6645
w31004	6	0.1622	0.1517	0.1893	-0.3342
w31004	7	0.5751	0.1835	-0.2295	2.0612
w42002	1	0.3450	0.3089	0.3767	0.5113
w42002	2	0.2640	0.2428	0.2851	0.4499
w42002	3	0.8466	0.9764	3.3722	2.7798
w42002	4	0.8743	0.8859	2.1597	2.3429
w42002	5	1.8211	1.7515	0.6642	0.6779
w42002	6	0.7845	0.7648	1.7349	1.9452
w44001	1	0.1472	0.1286	0.1719	0.4380
w44001	2	0.3316	0.3290	1.3809	1.3923
w44001	3	0.0674	0.0697	0.9648	0.6569
w44001	4	1.2357	1.1139	-0.2605	-0.8536
w44001	5	0.9352	0.8089	1.2550	1.7156
w61002	1	0.5755	0.0346	-1.1060	6.9235
w61002	2	0.1467	0.0999	0.2755	0.6761
w61002	3	0.0000	0.0000	0.0000	0.0000
w61002	4	0.2846	0.3003	2.0898	2.0559
w61002	5	0.4733	0.4606	1.4444	1.5468
w61002	6	0.7618	0.8643	2.0716	1.4763
w61002	7	0.5839	0.5821	0.1938	0.0740
w61002	8	0.4441	0.3817	0.3015	1.2485
w62007	1	0.7907	0.8211	0.9460	0.4394
w62007	2	1.0622	1.1563	2.2539	1.9155
w62007	3	0.9587	0.6478	-0.5489	0.3439

Watershed ID	Storm No.	Stand. Dev. Rain	Stand. Dev. Rain Excess	Coef. Skew Rain	Coef. Skew Rain Excess
w62007	4	1.178397	1.188503	1.205578	0.948040
w62007	5	0.349227	0.241692	-0.124153	0.967921
w62007	6	1.569451	0.366107	-3.008945	0.240370
w68003	2	18.636590	16.184720	0.283204	0.146573
w68003	3	4.396132	2.793491	-0.480756	-0.009431
w68003	6	10.298630	6.019603	-0.178464	1.627225
w68003	10	4.329544	3.296980	-0.353686	0.025677
w68003	11	10.363750	10.866860	0.761956	0.093058
w68003	12	0.000000	0.000000	0.000000	0.000000
w69008	1	2.246770	0.941161	-0.504847	1.575571
w69008	2	0.346730	0.344526	3.569429	5.000025
w69008	3	1.580765	1.753043	2.231960	1.526798
w69008	4	0.513155	0.578744	1.718799	1.181019
w69008	5	2.162902	1.325385	1.088189	0.907930
w69008	6	0.822052	0.775623	0.941546	1.114086
w69008	7	0.818537	0.570410	0.358029	0.565004
w69008	8	2.079692	2.017898	1.106102	0.693772
w69008	9	4.039051	4.077992	0.568678	-0.099337
w69008	10	3.885082	4.522555	-0.223258	0.457246
w69008	11	1.626890	0.242383	-1.569942	2.207170
w69008	12	2.462794	1.637894	-0.161474	0.726665
w69008	13	0.245476	0.256580	2.052115	1.984663
w69008	14	1.379283	1.506782	2.076424	1.694485
w69008	15	0.540995	0.069998	-3.125340	0.707896
w69009	1	0.097370	0.062575	-0.300957	0.482335
w69009	2	0.089596	0.084091	1.248921	2.358252
w69009	3	0.621099	0.737172	3.796989	3.048924
w69009	4	0.607353	0.614653	1.048178	0.430416
w69009	5	0.621099	0.737246	3.796989	3.048495
w69009	6	0.607353	0.614653	1.048178	0.430416
w69009	7	0.437306	0.458708	1.582974	1.521991
w69009	8	1.091064	0.261871	-2.066251	-0.255473
w69009	9	1.151209	1.302100	1.965587	1.363739
w69009	10	1.703925	0.267670	-1.393156	2.128246
w69009	11	1.131613	0.364208	-2.407725	2.151376
w69009	12	4.159630	3.229546	0.242423	0.271839
w69009	13	1.987738	1.660213	0.593082	1.117629
w69009	14	0.271997	0.302020	2.381763	1.960681
w69009	15	0.771156	0.713053	1.126177	1.618874
w69009	16	0.837129	0.095787	-2.532551	-0.199566
w69032	1	0.623049	0.593231	0.652351	0.635950
w69032	3	1.590397	1.684921	1.730153	1.436084
w69032	4	1.635301	1.588915	-1.026189	-0.952739

Watershed ID	Storm No.	Stand. Dev. Rain	Stand. Dev. Rain Excess	Coef. Skew Rain	Coef. Skew Rain Excess
w69032	5	2.369450	2.214988	-0.040888	-0.567099
w69032	6	3.736600	3.587302	0.464967	0.067624
w69032	7	3.310665	2.138620	-1.146850	-0.779818
w69032	8	0.493779	0.505654	0.690598	0.475266
w69032	9	1.163521	1.059003	1.750453	3.069452
w69032	10	0.506940	0.391366	-0.268245	-0.057918
w69032	11	2.789969	2.771586	0.736652	0.208633
w69042	1	1.633786	1.774973	1.867797	1.463676
w69042	2	1.777027	0.831495	-1.119478	-1.548116
w69042	3	3.632921	2.532673	0.376867	0.544390
w69042	4	0.255959	0.256656	1.519799	1.501403
w69042	5	0.477462	0.095385	-2.336080	0.013158
w69042	6	0.809648	0.871424	2.526650	2.394318
w69042	7	1.058549	0.865482	0.928653	2.993003
w69042	8	0.498185	0.542343	2.636307	2.346272
w69042	9	0.760557	0.485489	1.163988	1.199206
w69042	10	1.527633	1.710018	1.680547	1.089834
w71001	1	0.145738	0.086252	-1.350044	0.110356
w71001	2	0.314928	0.252017	0.114341	1.091927
w71001	3	0.332297	0.289679	0.372557	1.187146
w71001	4	0.221523	0.198177	0.725719	1.533913
w71001	5	0.168131	0.180128	2.013019	1.708927
w71001	6	0.251249	0.261430	2.877044	2.832861
w71001	7	0.543743	0.517096	0.553034	0.513119
w71001	8	0.213481	0.220236	3.000478	2.933649
w71001	9	0.283022	0.313495	3.089186	2.724642
w71001	10	0.752032	0.647581	-0.693463	-0.803598
w71001	11	0.428041	0.408874	0.820126	0.750280
w74009	2	3.850302	2.736852	0.202767	1.367951
w74009	3	1.298527	0.798441	-0.323746	-0.135472
w74009	4	2.436940	2.085713	0.345689	-0.636382
w74009	5	4.597273	4.196461	1.528925	2.047751
w75003	1	9.113202	8.454604	-0.369481	-0.539495
w75003	4	8.200917	8.484084	1.618032	1.543460
w75003	5	1.136177	1.280208	3.242573	2.823395

Watershed ID	Storm No.	Depth Rainfall	Depth Rainfall Excess	Basin Perimeter	Basin Area
w13011	1	1.8500	0.2227	23643.2	24066880
w13011	2	2.0300	1.0337	23643.2	24066880
w13011	3	3.1100	0.6512	23643.2	24066880
w13011	4	1.1400	0.0191	23643.2	24066880
w13011	5	0.5300	0.0338	23643.2	24066880
w13011	6	0.6900	0.0242	23643.2	24066880
w13011	7	1.8400	0.1172	23643.2	24066880
w13011	9	3.7900	1.5160	23643.2	24066880
w16006	1	2.8000	0.1490	38245.9	76931380
w16006	2	1.6000	0.0787	38245.9	76931380
w16006	3	1.3000	0.0275	38245.9	76931380
w16006	4	1.0000	0.0362	38245.9	76931380
w16006	5	1.5000	0.0247	38245.9	76931380
w16006	6	2.2000	0.3830	38245.9	76931380
w16006	7	1.3000	0.0462	38245.9	76931380
w16006	8	4.2000	1.8374	38245.9	76931380
w16006	9	2.1000	0.3170	38245.9	76931380
w16006	10	0.9000	0.4138	38245.9	76931380
w21001	1	6.0300	2.6294	47455.2	83764160
w21001	2	3.2000	0.9077	47455.2	83764160
w21001	3	1.7200	0.5995	47455.2	83764160
w21001	4	1.9500	1.3511	47455.2	83764160
w21001	5	2.3000	0.5544	47455.2	83764160
w21001	6	4.7400	1.6765	47455.2	83764160
w25001	1	1.8900	0.7291	12695.7	6708000
w25001	2	1.9100	0.8720	12695.7	6708000
w25001	3	1.3600	0.8891	12695.7	6708000
w25001	4	3.9800	3.5789	12695.7	6708000
w25001	5	1.0900	0.9439	12695.7	6708000
w25001	6	1.1900	0.9260	12695.7	6708000
w25001	7	1.7800	0.2209	12695.7	6708000
w25001	8	1.2400	0.7511	12695.7	6708000
w25001	9	1.0900	0.8124	12695.7	6708000
w25001	10	1.5500	1.1881	12695.7	6708000
w25001	12	5.6600	4.3531	12695.7	6708000
w25001	13	2.4000	1.5203	12695.7	6708000
w25001	14	6.2800	6.0320	12695.7	6708000
w25001	15	3.0900	2.6219	12695.7	6708000
w25001	16	2.6500	2.6322	12695.7	6708000
w25001	17	2.3600	1.4146	12695.7	6708000
w26030	1	1.5200	0.5711	16545.6	13008970
w26030	3	1.0700	0.3361	16545.6	13008970
w26030	5	1.1400	1.1120	16545.6	13008970

Watershed ID	Storm No.	Depth Rainfall	Depth Rainfall Excess	Basin Perimeter	Basin Area
w26030	6	1.7200	1.0059	16545.6	13008970
w26030	8	1.6300	0.0808	16545.6	13008970
w26030	9	0.8400	0.2150	16545.6	13008970
w26030	10	1.3400	0.0347	16545.6	13008970
w26030	11	1.0100	0.5789	16545.6	13008970
w26030	14	1.9200	0.2789	16545.6	13008970
w26030	16	2.4300	0.1752	16545.6	13008970
w26033	4	0.7600	0.3624	27311.7	40461860
w26033	6	1.7700	0.8629	27311.7	40461860
w26036	1	1.1400	0.4508	65625.1	211097900
w26036	2	1.5400	0.7400	65625.1	211097900
w26036	4	0.7500	0.4338	65625.1	211097900
w26036	5	1.0700	0.8605	65625.1	211097900
w26036	6	1.8400	1.1063	65625.1	211097900
w31004	1	2.3500	0.4822	10829.4	7333378
w31004	2	3.0000	0.9592	10829.4	7333378
w31004	3	1.0900	0.3079	10829.4	7333378
w31004	4	1.3900	0.3380	10829.4	7333378
w31004	5	3.4700	0.9684	10829.4	7333378
w31004	6	1.0000	0.2958	10829.4	7333378
w31004	7	3.4400	0.0454	10829.4	7333378
w42002	1	1.7800	0.7604	23323.9	25406920
w42002	2	1.6600	1.3369	23323.9	25406920
w42002	3	1.2600	0.3030	23323.9	25406920
w42002	4	2.9700	1.9596	23323.9	25406920
w42002	5	1.7200	1.3609	23323.9	25406920
w42002	6	1.3600	1.1662	23323.9	25406920
w44001	1	1.6600	0.4032	21901.6	20628180
w44001	2	1.6900	0.7583	21901.6	20628180
w44001	3	0.7200	0.3994	21901.6	20628180
w44001	4	1.9300	0.8915	21901.6	20628180
w44001	5	2.0700	0.1729	21901.6	20628180
w61002	1	0.8000	0.1485	6393.6	1947935
w61002	2	0.7600	0.0430	6393.6	1947935
w61002	3	0.6100	0.0061	6393.6	1947935
w61002	4	0.7500	0.1264	6393.6	1947935
w61002	5	0.7800	0.1089	6393.6	1947935
w61002	6	2.6800	0.4004	6393.6	1947935
w61002	7	0.3700	0.3302	6393.6	1947935
w61002	8	1.1700	0.5742	6393.6	1947935
w62007	1	1.5900	0.1216	21448.5	22656790
w62007	2	2.7700	0.2889	21448.5	22656790
w62007	3	1.7000	0.6534	21448.5	22656790

Watershed ID	Storm No.	Depth Rainfall	Depth Rainfall Excess	Basin Perimeter	Basin Area
w62007	4	1.8100	0.2835	21448.5	22656790
w62007	5	1.1700	0.1926	21448.5	22656790
w62007	6	1.3700	0.3203	21448.5	22656790
w68003	2	2.2300	0.6954	91579.7	344132900
w68003	3	1.2200	0.2453	91579.7	344132900
w68003	6	2.4300	0.0600	91579.7	344132900
w68003	10	0.7500	0.2926	91579.7	344132900
w68003	11	0.6400	0.0583	91579.7	344132900
w68003	12	0.3100	0.0053	91579.7	344132900
w69008	1	2.6200	0.2058	72159.2	214787700
w69008	2	1.8800	0.5988	72159.2	214787700
w69008	3	1.6400	0.0245	72159.2	214787700
w69008	4	1.1900	0.0747	72159.2	214787700
w69008	5	1.4200	0.0238	72159.2	214787700
w69008	6	1.9000	0.2961	72159.2	214787700
w69008	7	0.9200	0.0540	72159.2	214787700
w69008	8	1.3700	0.0796	72159.2	214787700
w69008	9	1.9800	0.2444	72159.2	214787700
w69008	10	3.2100	0.1873	72159.2	214787700
w69008	11	1.4500	0.1403	72159.2	214787700
w69008	12	2.1300	0.2430	72159.2	214787700
w69008	13	1.1900	0.1596	72159.2	214787700
w69008	14	2.5300	0.3833	72159.2	214787700
w69008	15	1.2100	0.3400	72159.2	214787700
w69009	1	1.2300	0.1642	21475.0	23416470
w69009	2	0.7800	0.0392	21475.0	23416470
w69009	3	0.7900	0.0999	21475.0	23416470
w69009	4	2.0700	0.1013	21475.0	23416470
w69009	5	0.7900	0.0995	21475.0	23416470
w69009	6	2.0700	0.1013	21475.0	23416470
w69009	7	1.2300	0.1473	21475.0	23416470
w69009	8	1.7100	0.1645	21475.0	23416470
w69009	9	3.2500	0.4103	21475.0	23416470
w69009	10	1.3700	0.1924	21475.0	23416470
w69009	11	3.6800	0.3215	21475.0	23416470
w69009	12	5.5700	0.4644	21475.0	23416470
w69009	13	2.5400	0.2155	21475.0	23416470
w69009	14	1.3100	0.2558	21475.0	23416470
w69009	15	2.6000	0.3403	21475.0	23416470
w69009	16	1.2900	0.2276	21475.0	23416470
w69032	1	2.0000	1.1469	6351.3	1909859
w69032	3	1.8600	0.9662	6351.3	1909859
w69032	4	2.3300	0.8420	6351.3	1909859

Watershed ID	Storm No.	Depth Rainfall	Depth Rainfall Excess	Basin Perimeter	Basin Area
w69032	5	1.7400	0.8398	6351.3	1909859
w69032	6	3.8000	1.2331	6351.3	1909859
w69032	7	3.2700	1.4755	6351.3	1909859
w69032	8	2.6000	1.8458	6351.3	1909859
w69032	9	2.0200	0.7824	6351.3	1909859
w69032	10	2.7100	1.1680	6351.3	1909859
w69032	11	2.1900	0.3391	6351.3	1909859
w69042	1	2.5000	0.7734	4466.3	1068209
w69042	2	1.7200	0.4799	4466.3	1068209
w69042	3	4.1200	0.2901	4466.3	1068209
w69042	4	3.6000	1.8006	4466.3	1068209
w69042	5	1.2900	0.6564	4466.3	1068209
w69042	6	2.1900	0.5960	4466.3	1068209
w69042	7	1.9600	0.3995	4466.3	1068209
w69042	8	1.2700	0.4272	4466.3	1068209
w69042	9	0.8800	0.0167	4466.3	1068209
w69042	10	2.6700	0.4240	4466.3	1068209
w71001	1	1.1000	0.5710	8288.4	3267475
w71001	2	1.5400	0.4040	8288.4	3267475
w71001	3	0.5800	0.3950	8288.4	3267475
w71001	4	1.6300	0.8570	8288.4	3267475
w71001	5	0.9000	0.3650	8288.4	3267475
w71001	6	0.7700	0.4860	8288.4	3267475
w71001	7	6.1400	4.2230	8288.4	3267475
w71001	8	1.6500	1.2325	8288.4	3267475
w71001	9	2.6400	0.9040	8288.4	3267475
w71001	10	1.5100	1.1684	8288.4	3267475
w71001	11	1.7700	0.6098	8288.4	3267475
w74009	2	2.5000	0.2476	24136.2	28402380
w74009	3	1.9000	0.2316	24136.2	28402380
w74009	4	2.6000	0.1306	24136.2	28402380
w74009	5	3.0000	0.2519	24136.2	28402380
w75003	1	4.1600	3.4510	41164.6	105701700
w75003	4	3.0900	0.9400	41164.6	105701700
w75003	5	2.1500	0.2390	41164.6	105701700

Watershed ID	Main Channel Length	Maximum Length	Maximum Width	Maximum Elevation Difference
w13011	8561.0	7964.7	4519.3	99.0
w16006	8386.9	11637.5	8971.3	775.0
w21001	20594.3	16669.5	6887.4	120.0
w25001	3518.1	4576.9	2228.3	38.0
w26030	3478.1	4551.4	4795.4	300.0
w26033	7586.9	8044.8	6886.6	275.0
w26036	23772.6	25937.5	12130.7	360.0
w31004	2233.0	3126.5	3301.7	87.0
w42002	7102.3	7316.9	5447.3	55.0
w44001	7850.2	7420.3	3924.9	75.0
w61002	1677.5	2408.8	1034.5	19.0
w62007	6951.3	6666.7	5240.8	190.0
w68003	35045.7	29582.5	16286.1	2350.0
w69008	28212.5	29313.1	13520.3	180.0
w69009	5304.8	6071.4	6149.0	105.0
w69032	544.1	2392.6	1576.9	5.0
w69042	719.5	1467.5	878.9	46.0
w71001	2780.9	2995.4	1601.7	115.0
w74009	8648.7	9690.6	3913.0	85.0
w75003	11042.8	13436.2	11185.0	5.0

Watershed ID	Overland Slope	Channel Slope	Stream Order	Bifurcation Ratio
w13011	0.05471	0.01051	4	3.88
w16006	0.15717	0.01312	3	2.83
w21001	0.04791	0.00534	4	3.06
w25001	0.03326	0.00739	2	3.00
w26030	0.12317	0.04169	3	2.75
w26033	0.10717	0.03097	3	2.50
w26036	0.12068	0.01157	2	13.00
w31004	0.05411	0.02239	2	3.00
w42002	0.02787	0.00662	3	2.25
w44001	0.04595	0.00892	4	3.00
w61002	0.01623	0.00596	1	1.00
w62007	0.07052	0.02589	3	3.50
w68003	0.27475	0.05421	3	3.75
w69008	0.05594	0.00390	2	3.00
w69009	0.04924	0.01320	1	1.00
w69032	0.00214	0.00368	2	2.00
w69042	0.03681	0.02919	1	1.00
w71001	0.07895	0.02877	3	2.00
w74009	0.03172	0.00520	2	3.00
w75003	0.00245	0.00036	2	2.00

Watershed ID	Drainage Density	Relief Ratio	Relative Relief	Ruggedness Number
w13011	0.0019291	0.0124298	0.0041872	0.19098
w16006	0.0002912	0.0665948	0.0202636	0.22572
w21001	0.0008191	0.0071987	0.0025287	0.09830
w25001	0.0007116	0.0083025	0.0029931	0.02704
w26030	0.0006377	0.0659145	0.0181316	0.19132
w26033	0.0004317	0.0341837	0.0100689	0.11874
w26036	0.0002438	0.0138795	0.0054857	0.08778
w31004	0.0005178	0.0278270	0.0080337	0.04506
w42002	0.0005543	0.0075168	0.0023580	0.03049
w44001	0.0015602	0.0101074	0.0034244	0.11702
w61002	0.0008731	0.0078876	0.0029717	0.01659
w62007	0.0010210	0.0284997	0.0088584	0.19401
w68003	0.0003355	0.0794388	0.0256607	0.78857
w69008	0.0001857	0.0061406	0.0024944	0.03343
w69009	0.0002193	0.0172941	0.0048894	0.02303
w69032	0.0004796	0.0020898	0.0007872	0.00240
w69042	0.0006781	0.0313462	0.0102992	0.03120
w71001	0.0012371	0.0383917	0.0138748	0.14227
w74009	0.0005161	0.0087713	0.0035216	0.04387
w75003	0.0001428	0.0003721	0.0001214	0.00071

Watershed ID	Storm No.	Elongation Ratio	Circularity Ratio	Time of Concentration
w13011		0.69502	0.73554	0.5541
w16006		0.85044	0.81297	1.7719
w21001		0.61953	0.68368	2.2124
w25001		0.67078	0.75970	0.5573
w26030		0.89420	0.77276	0.2802
w26033		0.89221	0.82562	0.4787
w26036		0.63207	0.78483	2.0229
w31004		0.97736	0.88644	0.2665
w42002		0.77733	0.76609	1.0733
w44001		0.69066	0.73512	0.7324
w61002		0.65379	0.77383	0.4596
w62007		0.80564	0.78670	0.4305
w68003		0.70759	0.71807	2.6074
w69008		0.56415	0.71998	1.9910
w69009		0.89935	0.79879	0.5956
w69032		0.65176	0.77134	1.4490
w69042		0.79471	0.82032	0.1707
w71001		0.68093	0.77311	0.1371
w74009		0.62056	0.78273	1.2527
w75003		0.86342	0.88536	3.9185

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VITA

James William Brown

Candidate for the Degree of

Master of Science

Thesis: DEVELOPMENT AND EVALUATION OF A GENERALIZED UNIT HYDROGRAPH

Major Field: Agricultural Engineering

Biographical:

Personal Data: Born in Jacksonville, Illinois, February 8, 1964, the son of Richard D. and Frances J. Brown. Married to Judianne Allen on August 1, 1988.

Education: Graduated from Winchester Community High School, Winchester, Illinois, in May, 1982; received Bachelor of Science in Agricultural Engineering and Agricultural Sciences from the University of Illinois at Urbana-Champaign in May, 1987; completed requirements for the Master of Science Degree at Oklahoma State University in May, 1989.

Professional Experience: Research Assistant, Department of Agricultural Engineering, Oklahoma State University, September, 1987 to February, 1989.

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