AN ABSTRACT OF THE THESIS OF

Mark A. Fedora for the degree of Master of Science in Forest Engineering presented on May 19, 1987.

Title: Simulation of Storm Runoff in the Oregon Coast Range

Abstract approved: Robert L. Beschta
Robert L. Beschta

Simulation of storm hydrographs in the Oregon Coast Range was explored using the Soil Conservation Service (SCS) curve number methodology, and by developing and testing an antecedent precipitation index (API) method. Standard SCS procedures over-estimated peak discharge by about a factor of two (i.e., average over-prediction of 118 percent). When an average curve number was derived for Deer Creek (an Oregon Coast Range stream), errors in predicted peak flows averaged 26.8 percent. Even with adjustment of SCS parameters (watershed lag, shape of the unit hydrograph, and curve number), the simulated hydrograph shape and timing of predicted peak flows did not match with observed hydrographs. The assumed rainfall-runoff relationships of the SCS method are unable to account for changing runoff responses related to the time distribution of precipitation, and therefore provides

an unrealistic approach to storm runoff simulation. The SCS runoff curve number method is not recommended for estimation of peak discharge nor simulation of storm hydrographs in Oregon's Coast Range.

A simple rainfall-runoff model was developed, which requires only precipitation and watershed area as inputs. An antecedent precipitation index (API) was developed by decaying the residual effects of precipitation observations through time. Coefficients used to decay API values were derived from recession analyses of storm hydrographs during periods of no rainfall. Linear regression was used to correlate API and discharge values for five Coast Range watersheds. Model coefficients for the five watersheds were used to predict the API-discharge relation for a sixth coastal watershed. Errors in peak flow estimates for Deer Creek and the independent test watershed averaged 10.7 and 17.8 percent, respectively. Storm runoff volume errors for all watersheds averaged 15.9 percent, and storm hydrograph shape was accurately simulated. Errors in peak discharge and volume estimates may be attributed to differences in timing between observed and simulated hydrographs, seasonal variation in antecedent moisture, and effects of snowmelt during rainfall. Temporal and spatial variability in precipitation observations were also evaluated. API methods may be useful in frequency analyses (in areas where rainfall records are longer than runoff records), estimation of missing data, slope stability research, and suspended sediment modeling.

Simulation of Storm Runoff in the Oregon Coast Range

by

Mark A. Fedora

A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Master of Science

Completed May 19, 1987

Commencement June 1988

ACKNOWLEDGEMENTS

My gratitude is extended to the USDI Bureau of Land Management for providing the financial assistance for this study.

I wish to express my thanks to Robert Beschta and Marvin Pyles for their invaluable support and counsel throughout this project.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SIMULATION OF STORM RUNOFF IN THE OREGON COAST RANGE

INTRODUCTION

Problem Statement

Timber and fisheries resources account for much of the economic development of Oregon's coastal region and both industries are influenced by the quantity and timing of runoff from storms. For example, high flow events can be very destructive to forest road systems and the downstream aquatic resources. While there are methods available to estimate the magnitude and frequency of floods for culvert design (Campbell and Sidle, 1984), many culvert installations in the Oregon Coast Range appear to be under-designed for the passage of floods having a 25-year return period (Piehl, 1987).

A real-time model to simulate individual storm hydrographs and conditions that contribute to hill-slope failures would be a useful tool for forest land managers in Oregon's Coast Range. An event-based storm hydrograph model could also be used to generate peak flows for frequency analysis in areas where streamflow data is not available. The model could also drive a supply-based suspended sediment model (eg. VanSickle and Beschta, 1983). In addition, historic events could be reconstructed for use in fisheries, stream morphology, and slope stability research.

Objective

The objective of this study was to evaluate an existing method and/or develop an alternative method for simulating individual storm hydrographs. The chosen method should meet the following criteria:

<u>Practicality</u> Data required to use the method must be readily available to forest managers.

<u>Applicability</u> The method must be applicable to small forested drainage basins in the Oregon Coast Range.

<u>Reproducibility</u> The results obtained should be consistently repeatable by professionals using the method.

<u>Accuracy</u> The model should accurately simulate the actual hydrograph shape (subjective), peak discharge (within 10 percent), volume (within 10 percent), and timing of the peak discharge (within four hours) for events or basins not included in the calibration of the model.

Procedure

A review of the literature was undertaken to identify potentially useful streamflow simulation models that might be adaptable to the Oregon Coast Range. The Soil Conservation Service unit hydrograph procedure was examined and tested using actual rainfall-runoff data from a coast range watershed. This procedure was eventually abandoned in favor of developing a method that relates antecedent precipitation to streamflow.

An antecedent precipitation index (API) model was developed and calibrated using 44 station-years of rainfall-runoff records from five Oregon Coast Range watersheds. The method was further tested by using eight years of data from a sixth watershed. Procedures used to develop and test the API model, as well as recommendations for use are discussed.

LITERATURE REVIEW

Mathematical models used to describe streamflow characteristics abound in the literature:

> The essence of hydrology is modeling. As a physical science, hydrology is concerned with numbers--quantitative answers are desired. A model is a mathematical statement of the response of a system which takes system inputs and transforms them into outputs (Dawdy, 1982 p. 24).

Hydrologic models can be generally classified as (1) physical or (2) black-box. Black-box models have little or no regard for the hydrologic processes involved in generating streamflow, and can be further sub-divided into (1) empirical equations and (2) unit hydrograph techniques. The advantages and disadvantages of these modeling approaches are discussed in this chapter. Examples of each method and comparisons between methods are presented with an emphasis toward models used in forest environments.

Physical Models

Physical models are those designed with an understanding of the hydrologic cycle and are based directly or indirectly upon the laws of physics. These models commonly simulate streamflow continuously through time and are able to simulate the effects of changes (natural or man-induced) in the catchment. Physical models are typically complex and are often used to gain an understanding of the hydrologic system by quantifying all water-movement pathways and processes.

Moore, et al., (1983) have developed a physically based model for small forested watersheds in the Appalachian mountains. Daily precipitation and daily potential evapotranspiration are the two basic meteorological inputs required to estimate daily runoff, once the model is calibrated. Values for sixteen coefficients and parameters are required for calibration of the model:

- Maximum interception capacity
- Area of stream surface
- Two expanding area source area coefficients
- Soil zone thickness
- Three soil water movement coefficients
- Wilting point
- Three groundwater zone coefficients
- Actual groundwater volume
- Actual interception capacity
- Actual soil water volume
- Fraction of water contributing to direct runoff

Results from their research watershed show "good agreement between observed and predicted daily discharges."

Moore, et al., (1986) have since increased the complexity of the Moore, et al., 1983 streamflow model by adding a steady-state saturation zone routine (O'Loughlin, 1986) to predict the variable source areas contributing to storm runoff. The saturation zone model incorporates hillslope geometry, land slope, and the spatial variability of soil properties. Added complexity also requires increased knowledge of the basin in question, and substantially increases the input data required. For both the calibration and "test" events, the new streamflow model incorporating the saturation zone routine was reported to have "very good" agreement between observed and predicted hydrographs.

Other physical models applied to forested basins include the variable source area simulator (VSAS2) (Bernier, 1985), and a new version of TOPMODEL used in the Shenandoha Watershed Study (Hornberger, et. al., 1985). VSAS2 requires knowledge of the basin topography, soil mantle geometry, soil hydrological characteristics, and rainfall. Bernier (1985) reports a poor performance of the model for large winter storms and small summer storms on a Georgia Piedmont basin watershed. TOPMODEL requires values for thirteen parameters and Hornberger, et. al., (1985) report that "the model reproduced observed flows reasonably well throughout the calibration period."

Physical models are usually developed by a large research effort on a particular basin. The technique involves quantifying and tracking all moisture as it enters in and travels through the system. Invariably the resulting models are complex and empirical coefficients and relationships are developed for various components.

Many of the empirical coefficients will be applicable regionally, while others may vary considerably from basin to basin. Complexity may make the model difficult to use; calibration on a single watershed may make the model difficult to apply elsewhere. Physical models require a rigorous knowledge of system processes to develop, and an intensive data collection and calibration effort to implement once developed. Assumptions made within the model may not be readily detectable by the model user. Furthermore, because of interactions between various hydrologic processes, model parameters and coefficients may lose their physical significance. These models are best suited to larger river systems where comprehensive evaluations are required, and where high value resources justify the great expense in development and calibration. They are also an excellent tool for research purposes on both large and small basins.

Black-Box Models

A black-box model uses mathematical relationships between inputs and outputs with little or no regard for the processes involved. Types of black-box models include: empirical equations (derived from experience, observation, or statistical fitting) and unit hydrograph techniques. Many black-box models enjoy common advantages and suffer from common disadvantages.

Black-box models are widely used because they simplify real-world processes and are subsequently less data intensive. Since data requirements are greatly limited as compared to physical models, a rigorous knowledge of the system processes is not required. Accuracy of model output may be sacrificed as much of the variability within natural systems is not accounted for. Little or no knowledge of system processes may be gained by use of these models, and parameters fitted to a particular system are often not transferable to another region.

Empirical Equations

Empirical equations are the simplest of black-box models. They use mathematical relationships between inputs (i.e. rainfall volume, rainfall intensity, basin characteristics) and outputs (i.e. peak flow, volume of storm runoff). Historically, these equations were derived and refined through observation and experience, while today, statistical fitting is used to accomplish the same goals.

Rational equation

An example of an empirical equation that has been widely used for sizing culverts in municipal areas is the rational equation:

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

Q = peak discharge (cfs)
C = runoff coefficient
I = average rainfall intensity over the duration of the "time of concentration" of the basin (inches/hour)
A = watershed area (acres)

This equation was proposed in 1889, and was based on eleven years of rainfall/runoff data from watersheds in a built-up area (Hiemstra and Reich, 1967). It can quickly provide an estimate of peak flow at a given location for a given rainfall intensity. However, one needs to estimate the value of the runoff coefficient (C) for the watershed of interest. The value of C may change seasonally, storm to storm, and with changing land use. The equation is limited to a specific region for use on a specific type of problem (i.e. drainage structure sizing in municipal areas). Hiemstra and Reich (1967) intentionally violated the stipulations above and tested the equation on 45 agricultural research watersheds. They found that the method over-predicted peak flows by a factor of two. Equations of this type are often dimensionally incorrect and usually require some judgment on the part of the user before they can be employed. Hiemstra and Reich (1967) present a thorough review of five empirical equations commonly used to estimate peak flows.

Least squares regression

Statistical fitting through a least squares regression procedure makes use of actual data (eg. rainfall, runoff) to predict future values within the range of the fitted data. In the Pacific Northwest, these techniques have been used to predict peak flows for various return intervals using basin characteristics as independent variables; and to predict peak flows for specific storm events with rainfall and antecedent conditions as independent variables.

Flow frequency from basin characteristics

Harris, et al., (1979) derived separate flood frequency equations for differing climatic regions of Oregon. Using least squares regression, they found that watershed area, area of lakes and ponds, and 2-year, 24-hour precipitation intensity were the best predictors of flood magnitude and frequency for the coast region. Watersheds included in their study ranged from 0.27 to 667 square miles in size; while standard errors of the estimates for predicted peak flows ranged from 32 to 37 percent.

In a similar study, Campbell and Sidel (1984) focused on small (0.27 to 2.58 square miles) forested watersheds of Oregon to predict peak flows of various return intervals for use in culvert design. In the coast region, watershed area and elevation were significant predictors of peak flows with standard errors of the estimates from 33 to 38 percent.

Peak flows from antecedent moisture and precipitation

Peak discharge for any given event depends upon rainfall volume, time distribution of that rainfall, and the antecedent condition of the watershed prior to the event. Researchers have tried to explain the variability in peak flows by quantifying these factors.

Lyons and Beschta (1983) used cumulative storm precipitation to predict peak flows for a 258 square mile watershed in the western Cascades of Oregon. Storm precipitation was determined by adding precipitation on the day of the peak to that of the previous two days. Their equation explained 38 percent of the variation in peak flows greater than 13.6 cubic feet per second per square mile (csm).

Jackson and Van Haveren (1984) related peak flows on three Oregon Coast Range watersheds to the 24-hour rainfall and mean daily streamflow one day prior to the peak. Depending upon the watershed, 79 to 85 percent of the variation in peak flows greater than 50 csm was explained by the independent variables. Since mean daily flow was used as a predictor, equations of this form could not be used on ungaged basins.

Istok and Boersma (1986) used cumulative rainfall (of various durations) to predict the occurrence and magnitude of runoff on five small (0.0018 to 1.10 square miles) agricultural watersheds in western Oregon. Occurrence of overland flow was best predicted by 12 and 120-hour cumulative rainfall prior to the event, and the cumulative rainfall since the first of October of that water year. The magnitude of the events themselves were less significant predictors of the occurrence of overland flow. Runoff volumes were best predicted by several measures of antecedent rainfall (12, 48, or 120-hour cumulative rainfall prior to the event). The investigators concluded that in regions where long duration, low intensity rainfall events are common, some measure of antecedent rainfall would be important to the accurate prediction of runoff.

Regression techniques can be used to identify and quantify the relative importance of basin and meteorological characteristics in relation to streamflow characteristics. Development of these equations is relatively easy and they are based on actual data. Future use of the prediction equations is also easy, results are consistent among users, and the errors associated with their use are known. Unfortunately, the equations are site specific, purpose specific, and easily misused. Not only may the coefficients of the equations be inappropriate for use in areas outside the area where the data was collected, but the variables themselves may be inappropriate. Sometimes the variables may add statistically significant predictive capability to an equation, but the sign of the coefficients may not make physical sense. Misuse of the equations occurs when they are used for a purpose that was unintended by the original investigator, predictions are made outside the range of the originally fitted data, and/or the equation is used outside the region of study. Regression analysis can predict specific components of hydrographs but the technique cannot be used to simulate entire storm hydrographs.

Unit Hydrographs

Unlike the other empirical techniques described thus far, unit hydrograph techniques can simulate an entire storm hydrograph. A unit hydrograph depicts the average response of a watershed to a storm of a specified magnitude and duration. Since the physical characteristics of a watershed--size, shape, slope, etc.-

-are constant, the shape of storm hydrographs from similar rainfall events are expected to be consistent. The unit hydrograph is defined as "the hydrograph of one centimeter, millimeter or inch of direct runoff from a storm of specified duration" (Linsley, Kohler and Paulhus, 1982). Unit hydrographs for a particular basin can be developed from a limited data set.

Once the unit hydrograph is developed, runoff from an actual rainfall event can be simulated by summing the ordinates of the unit hydrographs through time. A general description of the technique is given by Dunne and Leopold (1978), and by Linsley, Kohler and Paulhus (1982).

On ungaged watersheds, the shape of a hydrograph from a given amount of rainfall over a specified duration is unknown. To apply the unit hydrograph technique to an ungaged basin, an average shape must be assumed. Since the shape can vary from basin to basin, depending on physical characteristics of the watershed, one can either use a unit hydrograph shape from a similar watershed, or derive a characteristic shape synthetically. Because it is usually difficult to locate a "similar" watershed, several methods have been employed to derive the shape of unit hydrographs for ungaged basins. The U.S. Army Corps of Engineers has used the Snyder method (Snyder, 1938) to simulate runoff events on large basins. On smaller watersheds, the USDA Soil Conservation Service (SCS) unit hydrograph technique has been used extensively.

Originally developed for agricultural watersheds, the SCS method has since been applied to basins of all types around the world. The inputs required are readily available to land managers, the technique is relatively simple, and yet it includes site specific information about antecedent conditions, infiltration rates, and land use and associated management practices. Since the SCS method can simulate a storm hydrograph, is widely known, and has been applied to forested watersheds, the method is examined and evaluated on an Oregon Coast Range watershed in a following chapter.

Comparison of Modeling Techniques

Objective evaluations of modeling techniques and specific models within techniques can be carried out by direct comparisons of model performance. Comparisons can provide a potential model user with information about a model's versatility, and ultimately which modeling technique or specific model is appropriate for use in a given area for a given situation.

Weeks and Hebbert (1980) compared the performance of four physically based models and one black-box model on three watersheds of Western Australia. Mean monthly discharge and a statistical examination of systematic error provided a basis for comparison of the models. The investigators recommended both a sophisticated physically based model (the Sacramento Model) and the black-box model for use in the south-western region of Western Australia.

Loague and Freeze (1985) compared a physically based model, a unit hydrograph model and a regression model on three small experimental watersheds in the eastern United States. The watersheds differed in climate (sub-humid and humid), size (0.04 to 2.77 square miles), land use (range, pasture and cultivated, and forest), and slope (gentle to steep). Dominant streamflow generation mechanisms varied considerably among the watersheds as well, and none of the models could completely accommodate the variability. The investigators were surprised by the poor performance of all the models and concluded that the simpler regression and unit hydrograph models provided as good or better predictions than the complex, physically based model.

Variations in estimations of streamflow charactristics occur not only as a function of modeling technique or specific model used, but also among practicing professionals using the models. Newton and Herrin (1982) studied accuracy and consistency in the estimation of flood peaks by 200 hydrologists. Seven black-box models (including the rational equation and three regression based procedures) and two physically based models (uncalibrated to study sites) were among the nine estimation techniques used in the study. Increased model sophistication had little effect on the accuracy and consistency of flood frequency predictions. Predictions using the rational method proved to be the least accurate and least consistent of all methods tested, while regression procedures proved to be the most accurate and consistent procedures tested. Estimations of flood frequencies based on modeling the rainfall-runoff process suffered from a lack of calibration and design storm assumptions. The researchers recommended that factors within models requiring user judgements should be avoided, and where possible, techniques used in the estimation of flood peaks should be based upon actual data from the region in question.

SOIL CONSERVATION SERVICE METHOD

The Soil Conservation Service (SCS) method of streamflow simulation is a rainfall driven, event based, unit hydrograph procedure. Often referred to as the "curve number method," it was originally designed to predict storm runoff volumes for various land use treatments. It has since been used for solving a wide range of hydrologic problems and adapted for use within a unit hydrograph procedure (Rallison and Miller, 1982). The basic concepts of the method have remained largely unchanged since its introduction in 1964 (Richardson and Cronshey, 1985). The popularity of the method for use on ungaged watersheds is maintained by its minimal input requirements; yet it incorporates general information about antecedent conditions, soil properties, land use, and associated management practices.

When used to predict peak flows, Hewlett (1982) has observed that the SCS method over-predicts large peak flows on forested watersheds by a factor of two or more, while it under-predicts small flow events. Settergren, et al., (1985) compared synthetic unit hydrographs derived from SCS methods to observed unit hydrographs from two forested watersheds in southeast Missouri. They found that the coefficient used in deriving the peak of the unit hydrograph caused an over-prediction of the same magnitude that Hewlett (1982) described. These results may indicate that the standard SCS procedures over-predict peak flows from forested watersheds in a consistent and predictable manner.

Hawkins (1979) observed, "despite widespread usage, curve numbers are infrequent topics in hydrology literature, and...most readings on the topic are authoritative rather than developmental, innovative, or critical." These observations are especially true with regard to forested basins. This chapter examines the SCS method for use on forested watersheds in the Oregon Coast Range as a single-event streamflow simulation model, and a peak flow prediction method. Coefficients used within the procedure were compared with those derived from an actual unit hydrograph from a Coast Range watershed. In addition, the method was tested using actual rainfall/runoff data to compare predicted hydrograph characteristics (peak flow, timing of peak, and hydrograph shape) to observed characteristics. The test was conducted using standard SCS procedures, and a slightly modified version based on the coefficients derived from an observed unit hydrograph for a Coast Range watershed.

Important Components of the SCS Method

To generate a storm hydrograph from rainfall, the SCS method requires information about the watershed (area, average land slope, length of the longest channel) and an additional coefficient (curve number). This information is used to calculate the moisture storage capacity, the time delay or response of the watershed to rainfall inputs, and the conversion of rainfall to a rate of streamflow. Watershed characteristics can be easily obtained from topographic maps and/or field surveys. Parameters that are important to the use of the method include (1) curve numbers, (2) hydrograph shape, (3) watershed lag and time of concentration, and (4) relationship between cumulative rainfall and total runoff volume.

Curve Number

Curve numbers are used to index soil moisture storage capacity, which ultimately determines the proportion of rainfall that will become runoff. Changes in the value of a curve number assigned to a given area will result in changes in the predicted total storm runoff volume and peak flow. Curve numbers are dependent upon watershed characteristics including: land use, soil type, and initial soil moisture content. The SCS has published tables of these values for Oregon (USDA Soil Conservation Service, 1979).

A watershed with a curve number value of 100 represents an area where all rainfall is converted into runoff. An impermeable parking lot would be an example of an area where the curve number approaches one hundred. An undisturbed forested watershed has a relatively low curve number--indicative of an area with a large moisture storage capacity. Land use alters the curve number assigned to the area. Tables published by the SCS for use in Oregon indicate that as management intensity increases, the value of the curve number becomes greater. For example, a change from an undisturbed forest to a low density residential area increases the curve number value by 74 percent (Table 1). Management practices within a given land use category also influence the curve number assigned to an area. According to the SCS (USDA Soil Conservation Service, 1979), the harvest of a previously undisturbed forest and subsequent establishment of a second growth stand, results in a 31 percent increase in the curve number value (Table 2). Apparently, this change attempts to account for an assumed road network, landings, and the increased efficiency of water drainage as a result of management activities.

Curve numbers within a land use class and management regime can also vary between watersheds depending on site

Table 1. Runoff curve numbers for selected land uses, Soil Group A. (From USDA Soil Conservation Service, 1979.)

Land Use	Curve Number
Fir forest	42
Residential	73
Orchards	81
Perennial row crops	88

Table 2. Runoff curve numbers for management practices within selected land use categories, Soil Group A. (From USDA Soil Conservation Service, 1979.)

Land Use	Management Practice	Curve Number
Fir Forest	Undisturbed condition	42
	Young, 2nd growth	55
Residential	Low density	73
	High density	78

specific soil properties. The SCS has identified four hydrologic soil groups (A, B, C, and D). Supposedly, any soil series can be categorized into one of the four groups. Hydrologic soil groups are distinguished by their relative infiltration capacity (high, moderate, low and very low) and texture (coarse, moderate, fine and very fine).

The classification of an area within one of these soil groups has a profound effect on the resultant curve number assigned to a watershed. For example, an undisturbed forest that has deep, well drained soils with a high infiltration capacity, would have a curve number of forty-two. However, if the same area was thought to have "moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well drained to well drained soils" (USDA Soil Conservation Service, 1972), the curve number would be sixty-four (USDA Soil Conservation Service, 1979). This represents a 52 percent difference in the curve number value by simply placing a soil series in group "B" instead of group "A" (Table 3). Thus, selection of hydrologic soil group has a major effect on predicted runoff.

In an attempt to remove the burden of subjectivity from the user of the method, the SCS has classified over 4000 soils in the United States into one of the four hydrologic soil groups (USDA Soil Conservation Service,

Table 3. Runoff curve numbers for hydrologic soil groups within land use and management practice categories. (From USDA Soil Conservation Service, 1979.)

Land use	Management Practice	Curve Number			
		Soil Group			
		A	B	C	D
Fir forest	Undisturbed condition	42	64	76	81
Residential	Low density	73	83	89	91

1972). The basis for classification of soils and the assumptions made are of particular interest when applied to forested watersheds of western Oregon:

> The majority (of classifications) are based on the judgments of soil scientists.... They assumed that the soil surfaces were bare, maximum swelling had taken place, and rainfall rates exceeded surface intake rates (USDA Soil Conservation Service, 1972, p. 7.2).

Forested watersheds have a very small percentage of bare ground. For example, on undisturbed forest sites in the western Cascades of Oregon, Johnson and Beschta (1980) reported that only one percent of the area had no vegetative or litter cover. They also reported infiltration capacities of 2-4 inches/hour on both harvested and undisturbed sites. In the coastal areas of Oregon, a rainfall intensity of 1.8 inches/hour lasting for 20 minutes has a recurrence interval of 100 years (USDC Weather Bureau, 1956). Therefore, infiltration rates are rarely exceeded by rainfall rates for appreciable lengths of time. Since the assumptions upon which soils have been classified by the SCS are not representative of forested watersheds, the published hydrologic soil groups are probably not applicable to these areas.

The predominant soil series' in the Oregon Coast Range (USDA Soil Conservation Service, 1975) are categorized in the "B" and "C" hydrologic soil groups (USDA Soil Conservation Service, 1979). These groups are defined as having "moderate" to "slow infiltration rates when thoroughly wetted" (USDA Soil Conservation Service, 1972). Moderate and slow infiltration rates were defined as 0.64-2.0 inches/hour and 0.06 to 0.63 inches/hour respectively, by the Western Regional Technical Service Center, SCS, Portland, Oregon (Froehlich, personal communication, February, 1987, Oregon State University, Corvallis). The implication that Coast Range forest soils have relatively low infiltration capacities is troublesome when compared to the high rates measured by Johnson and Beschta in the western Cascades (1980).

The contradiction between assumed infiltration capacities and those derived from field measurements leads to confusion when one is faced with categorizing a soil series into a hydrologic soil group. Placement of an undisturbed forested watershed in an erroneous soil group (eg. "C" instead of "A") can have a greater effect on the curve number value than converting the watershed to a low density residential area (Table 3)!

Curve numbers for a given land use, management practice, and soil group, can also vary by the antecedent moisture content of the soil. The SCS has three classifications for antecedent moisture. Condition I exists when the watershed is dry--all moisture in storage has been depleted. Condition II represents the "average" moisture content of the soil, and Condition III exists when the soil moisture capacity has been filled. This implies that curve numbers can vary on a given watershed from season to season and throughout a storm. Because curve numbers directly influence the peak flow rates and flow volumes predicted by the SCS method, the use of erroneous or inappropriate curve numbers could result in a serious over- or under-prediction of these hydrograph properties.

If curve numbers are indeed a function of land use, management practices, soil group classification, and antecedent moisture condition, then a single representative curve number for a watershed (or a region) cannot exist through time, since the variables influencing curve numbers are not constant. Thus, choosing or predicting a curve number for simulating streamflow from a given rainfall event is highly subjective. Hawkins (1986), has used rainfall/runoff data to empirically derive curve numbers for individual storm events. For the each watershed, curve numbers were estimated using land use, soil, and vegetation descriptions. Calculated and estimated curve numbers were then compared. Hawkins (1986) concluded that "... curve numbers estimated for forested watersheds were almost entirely unrelated to observed reality."

Hawkins (1975) reported that storm runoff volume predicted by the SCS method is more sensitive to errors in curve number estimates than precipitation errors for a considerable range of precipitation volumes (up to nine inches). Runoff volume estimates were most sensitive to errors in curve number estimates for watersheds with a high moisture storage capacity. Bondelid, et al., (1982) examined the sensitivity of predicted peak flows to errors in curve number estimation. Their results indicate that peak discharge estimates are most sensitive to curve number errors for low volume storms on areas with a high moisture storage capacity. The importance of accurate curve number selection for forested watersheds is therefore paramount.

Shape of the Unit Hydrograph

The curvilinear unit hydrograph used by the Soil Conservation Service is commonly simplified to a triangular unit hydrograph. Simplification allows for using the geometry of triangles to solve for the peak discharge rate of the unit hydrograph. The assumed shape of the unit hydrograph directly affects the calculated peak flow rate. The shape is described as the ratio of the time duration of the recession limb (Tr) relative to the time to the peak (Tp) of the unit hydrograph. The suggested ratio of Tr/Tp is 1.67. In special cases the SCS contends that it may be necessary to vary this ratio from 0.86 for steep terrain, to 3.30 for very flat and swampy country (USDA Soil Conservation Service, 1972).

Land slopes of Coast Range watersheds commonly range from 50 to 100 percent (USDA Soil Conservation Service, 1975). While steep slopes usually carry water more quickly to stream channels than gentle slopes, streamflow response to a rainfall event also depends upon the pathways taken by the water to the channel. Where overland flow is the dominant mechanism for water to reach a channel, streamflow response will be quicker then a similar area where subsurface flow dominates. While Coast Range watersheds have steep slopes (indicating a small Tr/Tp ratio may be appropriate), the dominant flow pathway is subsurface (indicating a large Tr/Tp ratio may be appropriate).

Determination of an actual Tr/Tp ratio is somewhat arbitrary. When developing a unit hydrograph using standard techniques (Linsley, Kohler and Paulhus, 1982) the method of baseflow separation used will greatly influence the Tr/Tp ratio. By visually separating the baseflow (Figure 1), and drawing a triangular hydrograph (Figure 2), a Tr/Tp ratio of 2.40 is obtained for Deer Creek in the Coast Range. This ratio is similar to the ratio recommended for very flat and swampy areas--quite unlike the Oregon Coast Range!

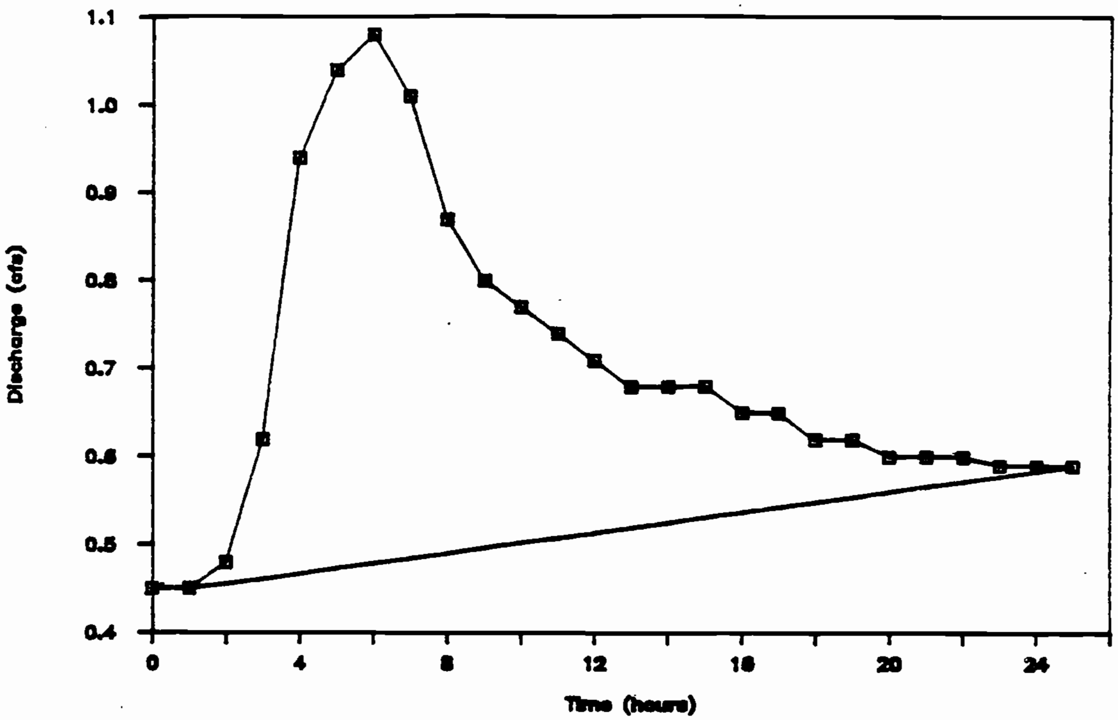


Figure 1. Hydrograph with visually separated baseflow for derivation of a unit hydrograph and determination of Tr/Tp ratio for Deer Creek, Oregon Coast Range.

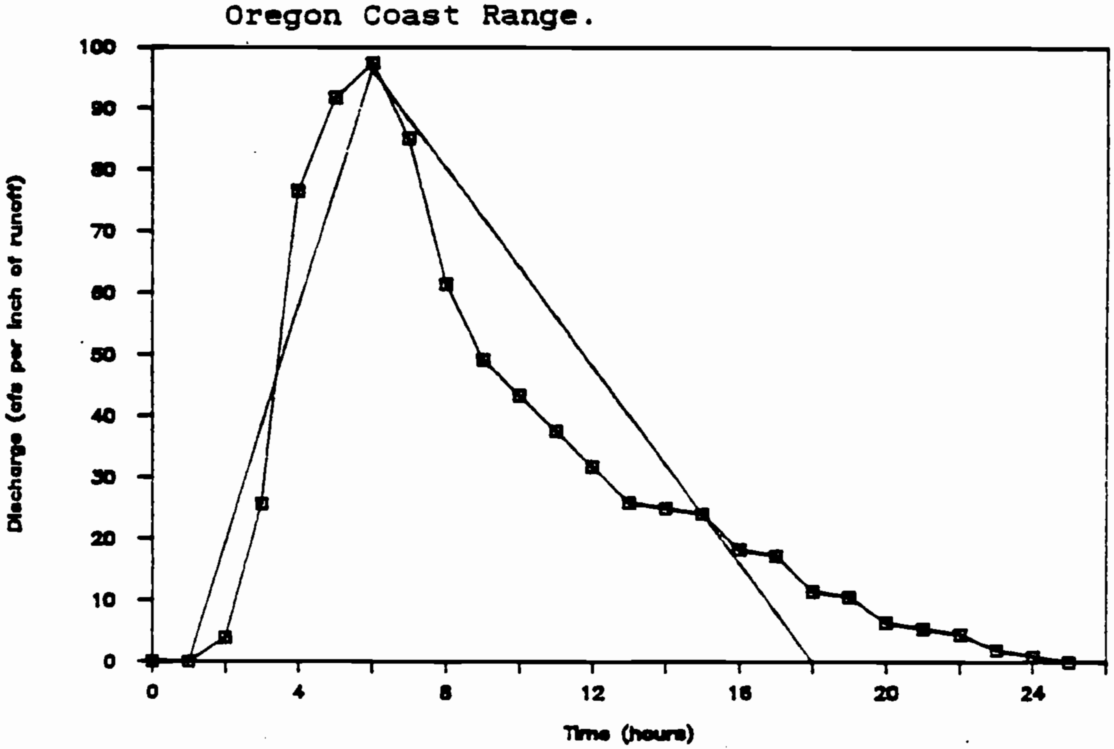


Figure 2. Unit hydrograph (2.5 hour effective storm duration) and approximated triangular unit hydrograph (Tr/Tp = 2.40) for Deer Creek, Oregon Coast Range.

A change from the recommended 1.67 Tr/Tp ratio to 2.4 will reduce the calculated peak of the unit hydrograph by changing the value of the dimensionless "constant" (K) in the peak flow equation:

$$q = \frac{645.33 * K * A * Q}{Tp} \qquad (2)$$

q = Peak flow of the triangular unit hydrograph (cfs)
K = Constant (dimensionless)
= 2/(1 + (Tr/Tp))
= 0.749
A = Watershed area (square miles)
Q = One inch of runoff
Tp= Time to peak of the unit hydrograph (hours)
Tr= Time of recession of the unit hydrograph (hours)

Figure 3 illustrates the change in the peak of the synthesized unit hydrograph with a change in the Tr/Tp ratio from 1.67 to 2.40. This change was also observed by Settergren, et al., (1985) when they compared observed unit hydrographs from forested watersheds in southeast Missouri, to unit hydrographs synthesized by SCS methods. A "flattening" of the unit hydrograph is expected to reduce the peak flows predicted for a given curve number.

Time of Concentration, Watershed Lag

Time of concentration (Tc) is defined in two ways by the Soil Conservation Service:

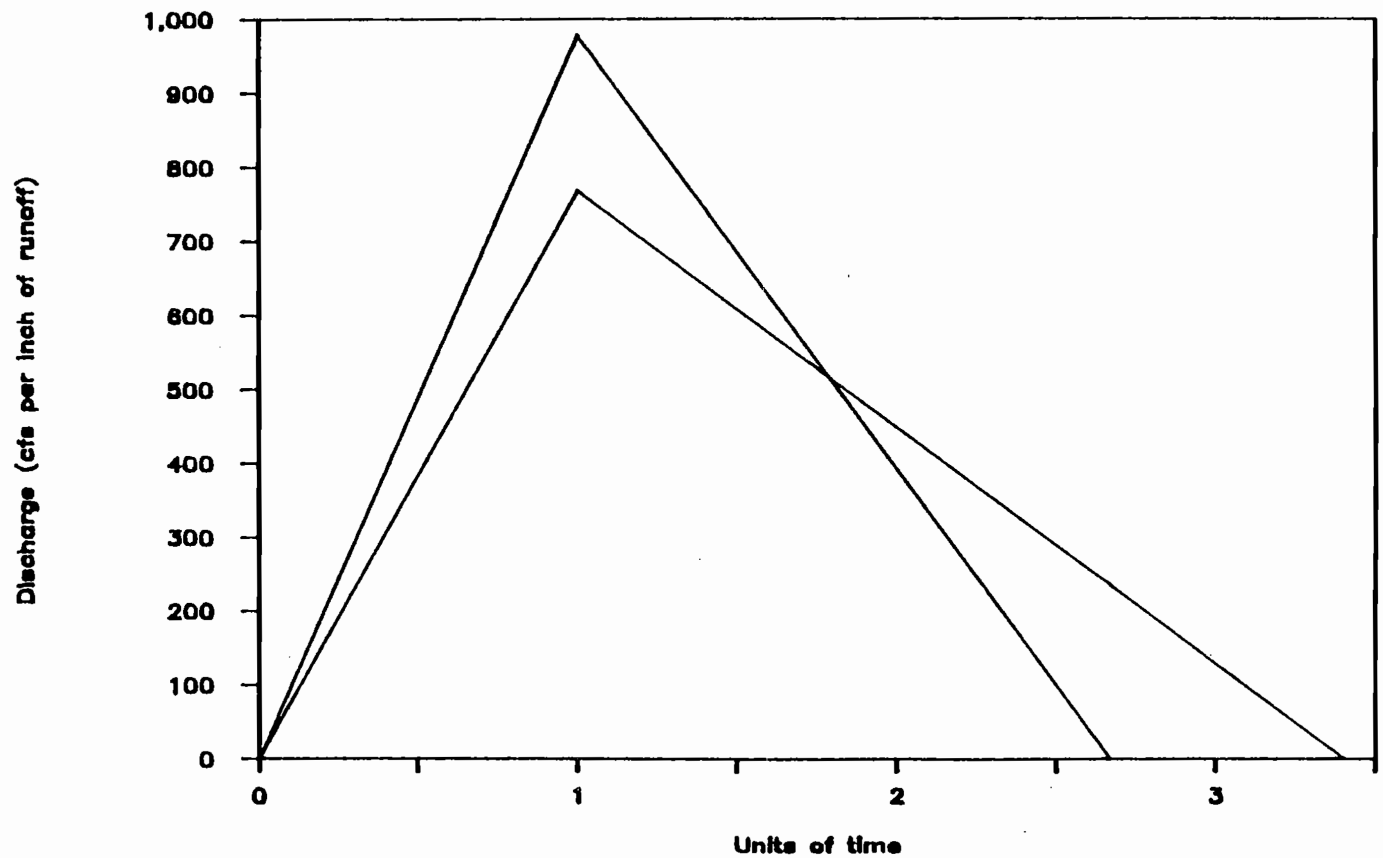


Figure 3. Change in the peak of the SCS synthesized triangular unit hydrograph with change in the Tr/Tp.ratio.

(1) The time for runoff to travel from the furthest point in the watershed to one point in question,
(2) The time from the end of excess rainfall to the point of inflection of the unit hydrograph (USDA Soil Conservation Service, 1972, p. 16.7).

Watershed lag (L) is related to Tc by the empirical equation:

$$L = 0.6 * Tc \tag{3}$$

L = Watershed Lag (hours)
Tc= Time of Concentration (hours)

Watershed lag is also defined as the time from the centroid of the excess rainfall to the peak of the unit hydrograph.

The SCS relates watershed lag to the hydraulic length of the watershed, average land slope, and maximum watershed storage (based on the watershed curve number) by the empirical equation:

$$L = l^{0.8} * (S + 1)^{0.7}/(1900 * Y^{0.5}) \tag{4}$$

L = Watershed Lag (hours)
l = Hydraulic Length of the watershed (feet)
S = (1000/CN) - 10
= Maximum watershed Storage (inches)
CN= Curve Number
Y = Average Land Slope (percent)

This relationship was developed using watershed research data for areas less than 2000 acres (USDA Soil Conservation Service, 1972).

The above definitions and equations allow for a comparison of calculated and observed values of L and Tc. Assuming a curve number of 64 for the Deer Creek watershed (fir forest, undisturbed condition, antecedent moisture condition II, and soil group B), watershed lag from equation 4 is 0.52 hours (average watershed slope is 35.3% by the contour method (USDA Soil Conservation Service, 1979), and hydraulic length is 9770 feet from topographic map of the Deer Creek watershed). Equation 3 can be solved to obtain a Tc of 0.86 hours. These values are less than those obtained using the derived unit hydrograph from Deer Creek (Figure 4), where L is 2.8 hours and Tc is 3.5 hours. The absolute magnitude of L and Tc for Deer Creek are expected to be greater than the values predicted by equation 3 since subsurface flow mechanisms dominate on forested watersheds. The empirically derived relationship between L and Tc for Deer Creek becomes:

$$L = 0.8 * Tc \tag{5}$$

L = Watershed Lag (hours)
Tc= Time of Concentration (hours)

Presumably, the coefficients in equations 3 and 4 could be adjusted and better defined for forested watersheds if a

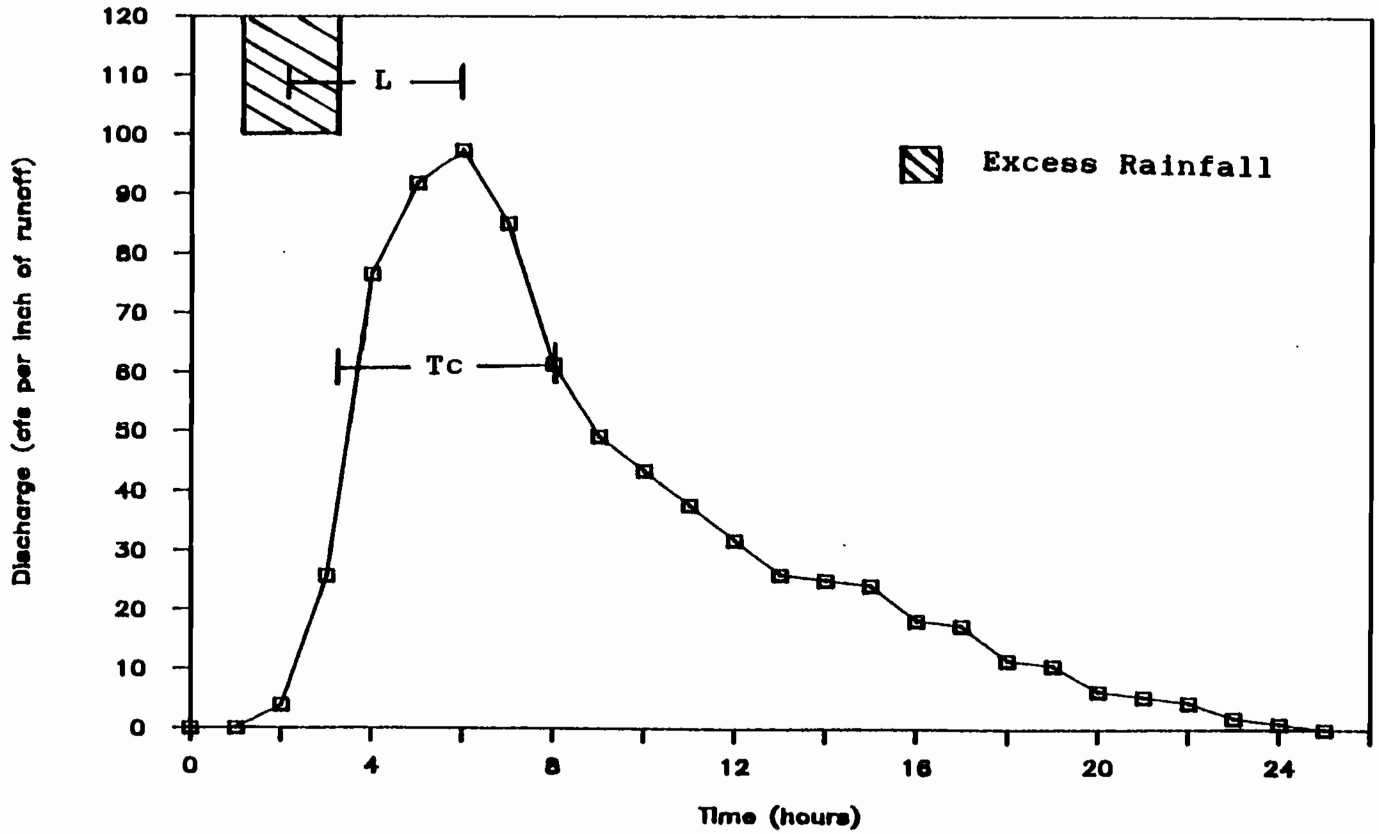


Figure 4. Relationships between Time of Concentration (Tc), Watershed Lag (L), Excess Rainfall, and the derived unit hydrograph for Deer Creek, Oregon Coast Range. Tc is 3.5 hours and L is 2.8 hours.

large number of unit hydrographs were analyzed and if hydraulic length, land slope, and curve numbers, are indeed related to watershed lag.

Watershed lag and time of concentration influence the duration of the synthesized unit hydrograph and subsequent timing of peak flow predictions, and the slope of the recession limb following a hydrograph peak. L and Tc are also used to derive the time-to-peak (Tp) of the unit hydrograph:

$$Tp = D/2 + L \tag{6}$$

Tp= Time-to-peak of the unit hydrograph (hours)
D = Duration of unit excess rainfall (hours)
  = 0.133 * Tc
Tc= Time of concentration (hours)
L = Watershed Lag (hours)

The time-to-peak of the unit hydrograph is ultimately used to derive the peak flow (q) of the unit hydrograph (equation 2). Thus, larger values of L and Tc will reduce the peak of the unit hydrograph. For the Deer Creek example, the peak flow of the unit hydrograph using standard SCS procedures (equations 2, 3, 4, and 6) is reduced from 979 cfs/inch of runoff, to 187 cfs/inch of runoff when the observed values of L, Tc, and Tr/Tp are substituted into equations 6 and 2. Again, it is unclear whether these adjustments must be compensated by increased curve numbers (thereby increasing the volume of runoff for a given rainfall input) or if these changes will offset the peak flow over-predictions observed by Hewlett (1982).

Rainfall and Runoff Volumes

A mechanism to convert precipitation inputs into runoff volume is common of all streamflow prediction models. The Soil Conservation Service procedure for this conversion assumes that the total runoff volume for a given rainfall volume will be constant--regardless of the rainfall distribution within the storm. Total runoff volume is based on the cumulative precipitation and the curve number:

$$Q = \frac{(P - 0.2*S)^2}{P + 0.8*S} \tag{7}$$

Q = Total Runoff Volume (inches)
P = Cumulative Precipitation (inches)
S = Maximum Watershed Storage (inches)
  = (1000/CN) - 10
CN= Curve Number

The coefficients in equation 7 (0.2 and 0.8), represent an "initial abstraction" of precipitation before streamflow begins (USDA Soil Conservation Service, 1972). Since the coefficients were derived from observations on agricultural watersheds, their values may not be appropriate for forested watersheds. However the "abstraction" may simulate the processes of interception, and detention and retention storage observed on forest watersheds.

Figure 5 illustrates the relationships defined by equation 7. Again, the percentage of rainfall that becomes runoff for a given rainfall amount is solely dependant upon curve number. As cumulative rainfall for an event becomes greater, the efficiency of a watershed to convert rainfall into runoff increases at an increasing rate. This effect simulates the way a watershed may react to rainfall as pathways for water travel become less tortuous, retention and detention storage become satisfied, and source areas for quickflow volume expand. As a storm passes, the watershed drains and source areas contract. Similarly, detention storage will drain and retention storage will become depleted by evapotranspiration or subsurface drainage. There is no mechanism for simulation of these "post storm" processes within the SCS method. For this reason, continuous streamflow simulation over long periods of time is not possible with the SCS method.

For single storm events with rainfall intensities that gradually increase, and then rapidly taper off--the SCS method may provide a reasonable approach to simulating a storm hydrograph. However for complex storms, with multiple bursts of heavy rainfall and periods of no rain, the SCS method would not be expected to accurately

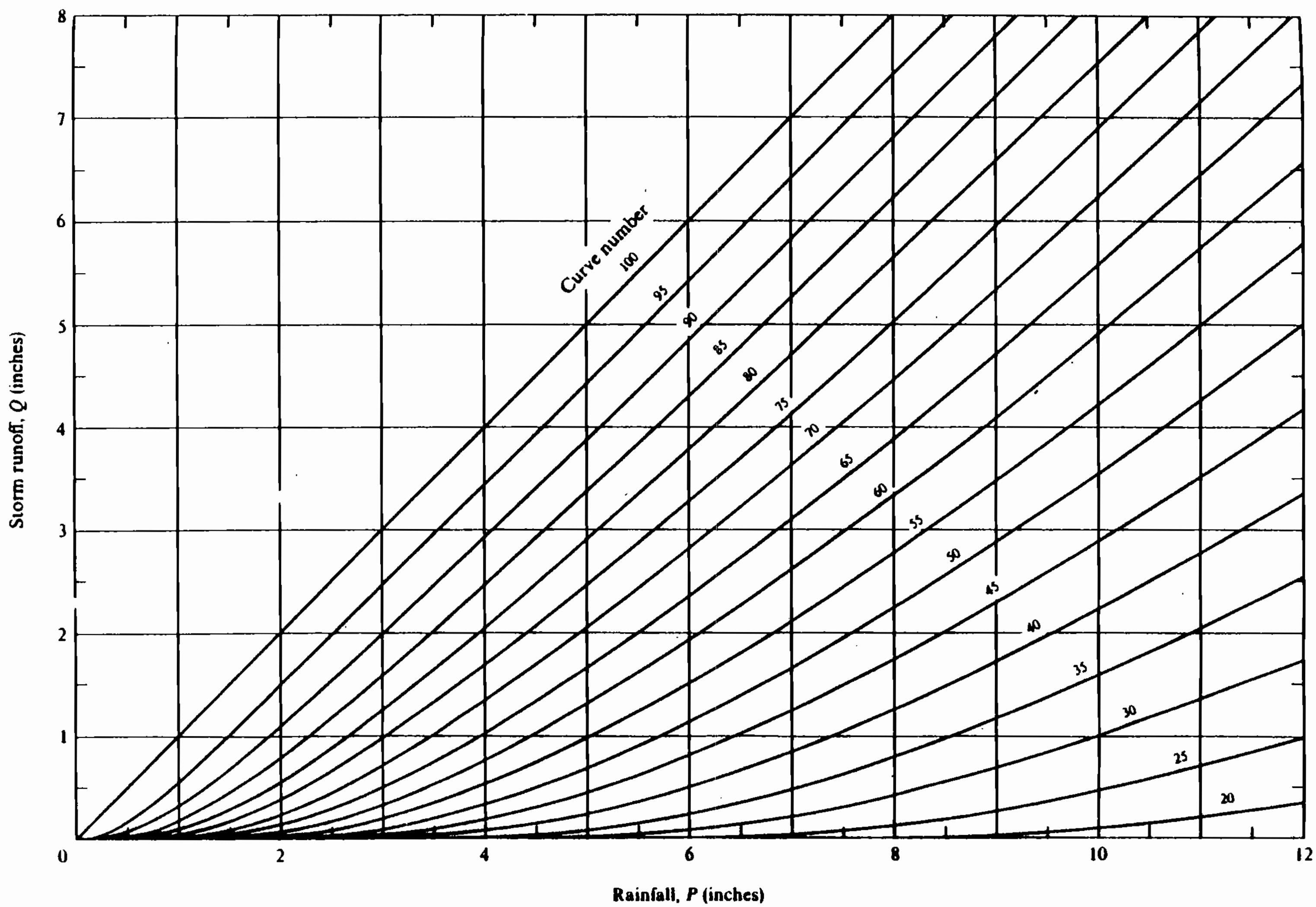


Figure 5. Relationship between cumulative precipitation and cumulative runoff for various curve numbers. (From Dunne and Leopold, 1978, pp. 293.)

simulate a storm hydrograph. Throughout western Oregon and the Pacific Northwest, many hours can pass between "pulses" of relatively high rainfall intensities. These interludes allow watersheds to drain slightly before the next pulse of precipitation. Therefore, streamflow will not rise as quickly with these latter rainfall inputs as it would have had the rain fallen in a contiguous manner. While it is conceivable that the SCS method might accurately depict the overall shape and peak of a storm hydrograph for relatively simple rainfall distribution patterns (Figure 6), the method greatly exaggerates the effects of rainfall near the end of a complex storm event (Figure 7). For complex storms, the consequences of this error can cause a gross over-prediction of the actual peak flow rate (Figure 7). (In these examples--for purposes of illustration and simplification--simulated hydrographs were adjusted to meet the observed peak flow values by adjusting the curve number.)

Rallison and Miller (1982) have described the limits of application of equation 7. Citing a 1964 letter written by V. Mockus (one of the original authors of the SCS runoff procedure), Rallison and Miller explain:

> For a continuous storm--one with no breaks in the rainfall--(equation 7) can be used to calculate the accumulated runoff. For a discontinuous storm, which has intervals of no rain, there is some recovery of infiltration rates during the intervals. If the period does not exceed an hour of so, it can be ignored and the estimate will be reasonably accurate. When

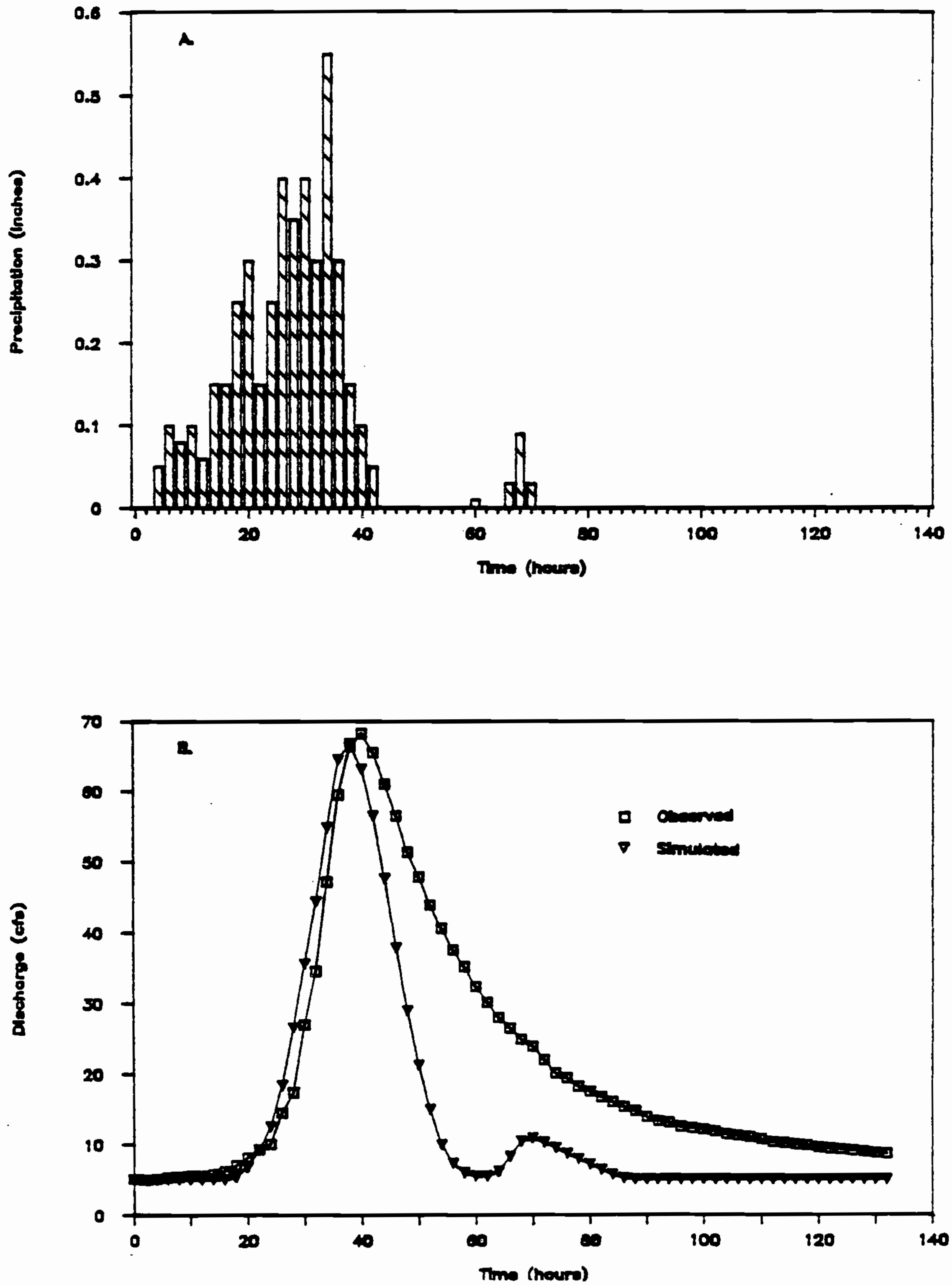


Figure 6. (A) Hyetograph, and (B) simulated and observed hydrographs for a "simple" rainfall event on Deer Creek, Oregon Coast Range.

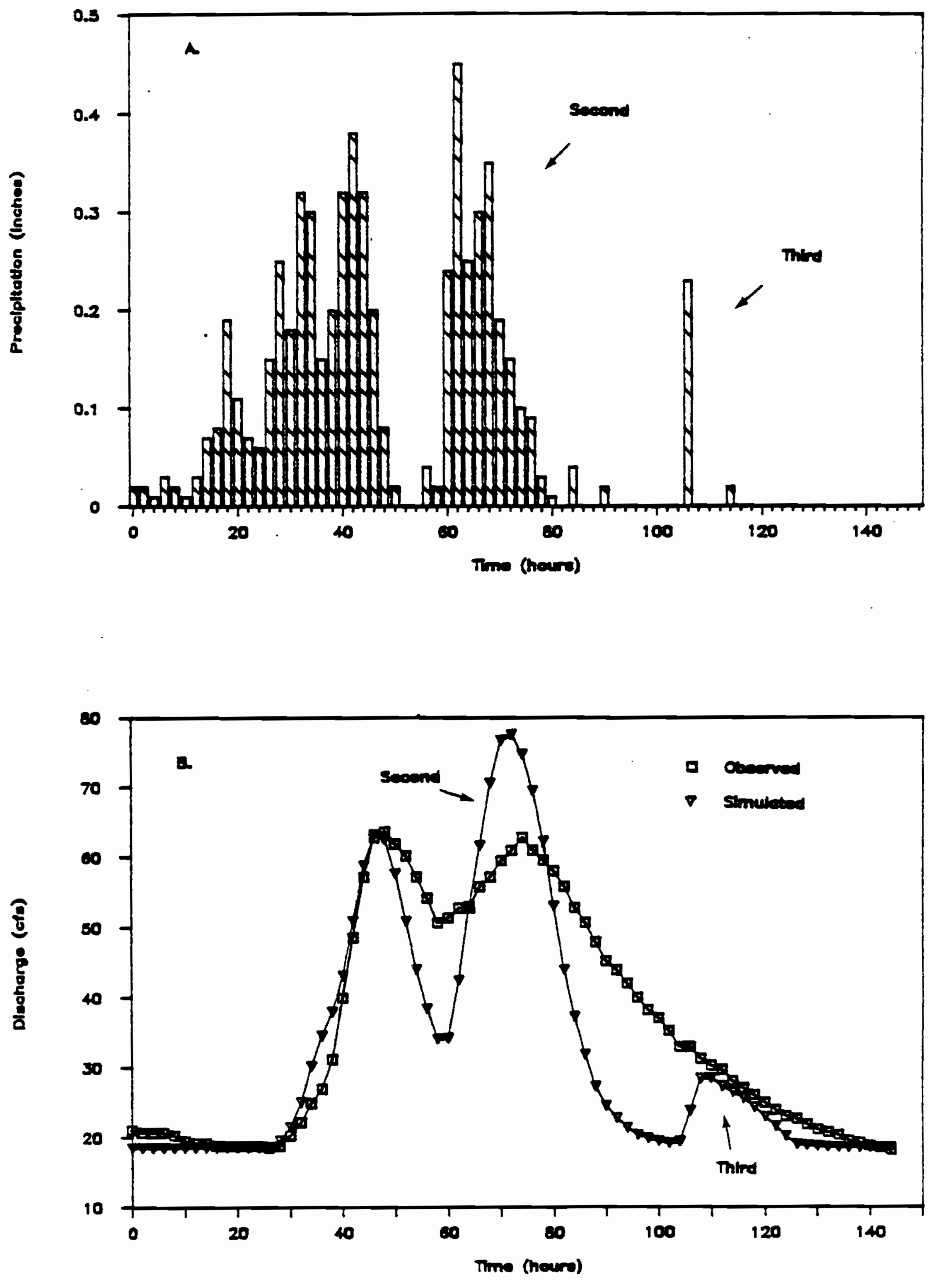


Figure 7. (A) Hyetograph, and (B) simulated and observed hydrographs for a "complex" rainfall event on Deer Creek, Oregon Coast Range. The SCS method over-emphasizes the effects of a second and a third "pulse" of precipitation.

> the rainless periods are over an hour, a new higher curve number is usually selected on the basis of the change in antecedent moisture for the next period of rain (p. 359).

No guidance is given within the standard SCS procedures for adjusting curve numbers with changing antecedent conditions. In addition, an increase in the curve number following a brief period of no rain implies that calculation of runoff from cumulative precipitation (equation 7) must begin again at zero, and a new initial abstraction be satisfied before runoff begins. With the initial abstraction satisfied, the effects of additional precipitation may be over-emphasized more strongly than depicted in Figure 7.

The discussion above suggests that streamflow simulation using SCS procedures may not be an objective means of simulating storm runoff, but rather a hydrologic form of art. While there have been efforts to modify the SCS method to allow curve numbers to vary with changes in soil moisture (Williams and LeSeur, 1976) or precipitation volume (Hawkins, 1979), it is the purpose of this chapter to evaluate the accuracy of the SCS method (using a single curve number) to simulate streamflow responses of an Oregon Coast Range watershed to rainfall events.

Testing the SCS Method

The SCS method was tested using rainfall/runoff data from the Deer Creek watershed in the Oregon Coast Range. Eleven events were selected for this analysis based on (1) rainfall/runoff data availability, and (2) runoff events exceeding the USDI Geological Survey base for which peak flows are reported (60 cubic feet/second). Coefficients that were derived from unit hydrograph analysis of Deer Creek and explained above were substituted for those in the original model and will be referred to as the "adjusted" model. Adjustments made are summarized below:

Watershed Lag = 2..8 hours
Time of concentration = 3.5 hours
Time of recession/time to peak ratio = 2.40
"Constant" (K) from equation 1, was adjusted accordingly
$K = 2/(1 + (Tr/Tp))$
$K = 0.588$

Streamflow simulated by the original and adjusted models was compared to observed streamflow for eleven separate rainfall events. Streamflow characteristics evaluated for this comparison were (1) peak flow, (2) timing of peak flows, and (3) overall hydrograph shape. A rainfall event that occurred on Deer Creek February 6-17, 1961 (Figure 8), will be used as an example for the comparisons of observed and simulated hydrograph shape. The return interval of the peak discharge resulting from this rainfall event was approximately three years.

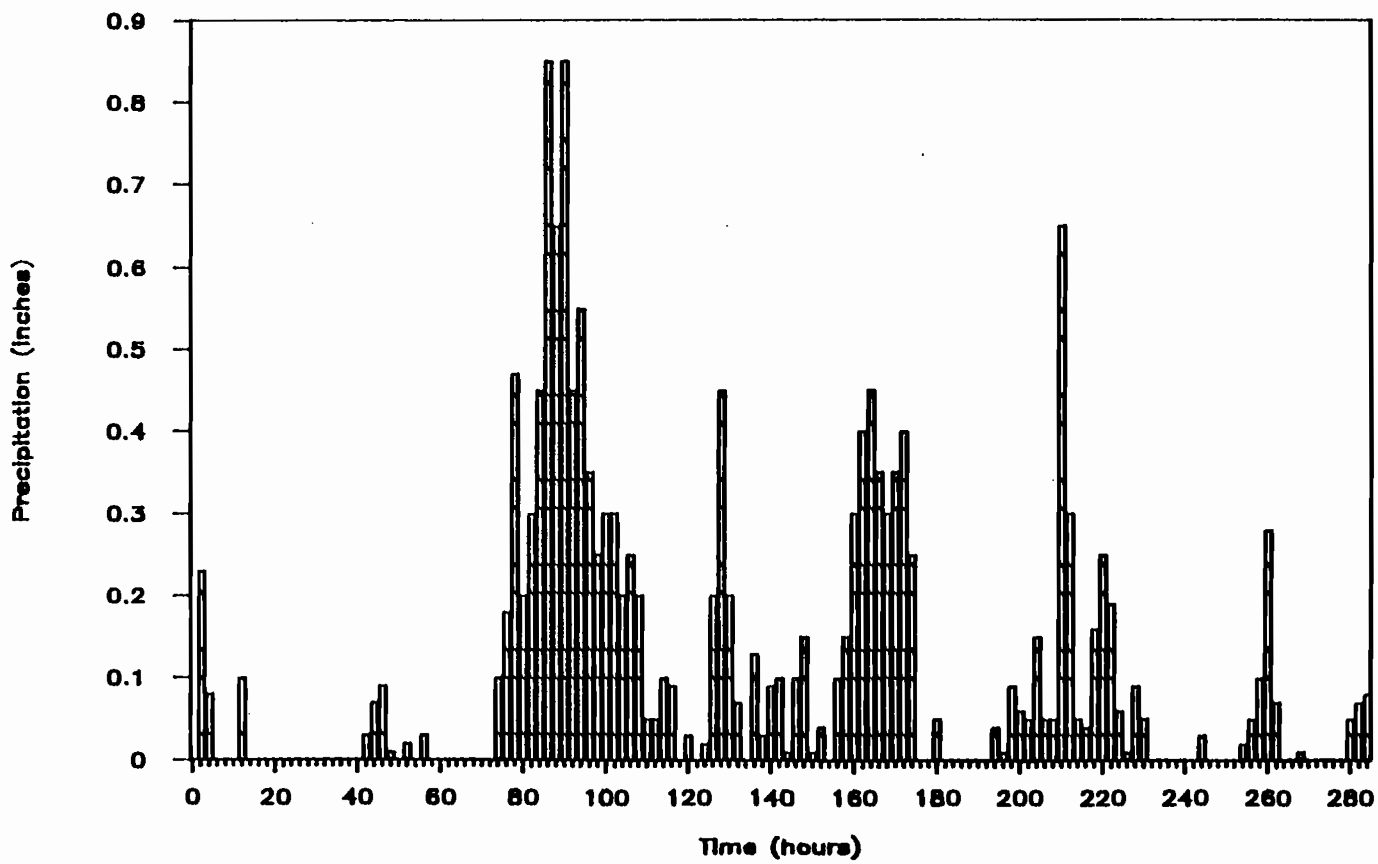


Figure 8. Hyetograph for rainfall event February 6-17, 1961, Deer Creek, Oregon Coast Range. This event is used for a comparison of observed and simulated hydrographs (Figures 11, 14, and 17).

The Deer Creek watershed had an undisturbed forest canopy from 1959-1966. Harr, et al., (1975) detected no significant changes in peak flows after the watershed was 29% patch-cut in 1966. For this reason, the watershed will be assumed to remain in an "undisturbed condition" throughout the study period (1959-1972). The Slickrock, Knappa, and Bohannon soil series' which underlie the watershed are categorized in the hydrologic soils groups "B," "B," and "C," respectively (USDA Soil Conservation Service, 1979). Using a weighted average of the area within each soils group and antecedent moisture condition II, the curve number chosen for use in the original model was 71 (USDA Soil Conservation Service, 1979). Harr, et al., (1975) used a base flow of 3.5 cubic feet/second/square mile (csm) to distinguished between autumn and winter events in the Oregon Coast Range. Since the base flow for an ungaged watershed would not be known, and most large runoff events occur in the winter, an assumed constant base flow of 3.5 csm was used in this analysis.

A plot of observed and SCS predicted peak flows (Figure 9) show a close correlation (r^2=0.745), with the standard error of the estimate (S_y) 22.4 cubic feet per second (cfs). The slope of the line falls far short of a 1:1 ratio, supporting Hewletts' contention that the SCS method over-predicts peak flows. The timing of the

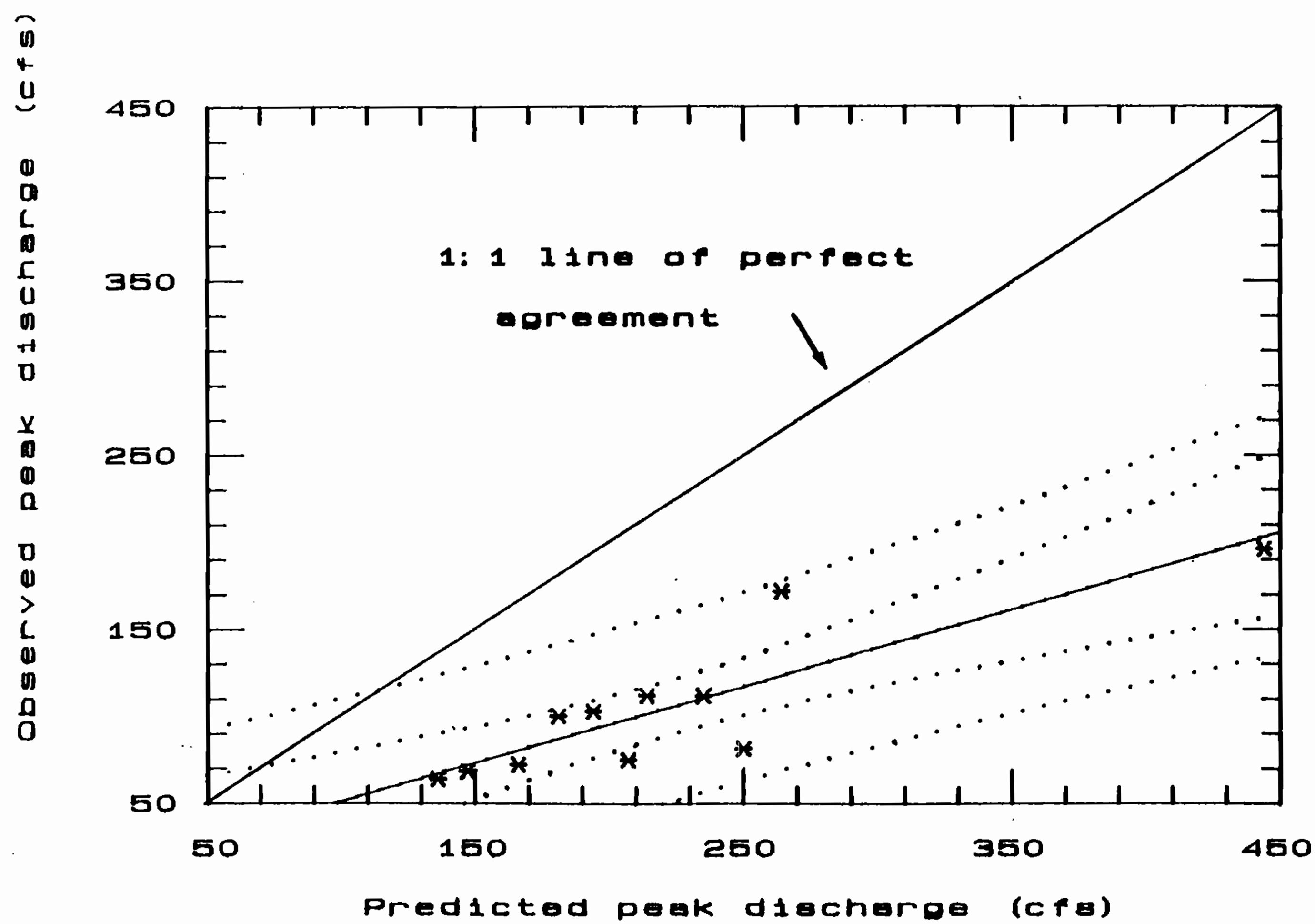


Figure 9. Observed and predicted peak flows with 95% confidence intervals for significance of regression and prediction limits (r^2=0.745). Predicted values from standard SCS unit hydrograph procedures, curve number 71.

predicted peaks is evaluated by observing a frequency/departure relationship. For approximately 60 percent of the storms, the SCS method predicted peak flows within 10 hours of the observed peak (Figure 10). The simulated hydrograph shape (Figure 11) was highly sensitive to precipitation intensity.

Since standard procedures for arriving at curve numbers are arbitrary at best, an average curve number for Deer Creek was sought. Curve numbers were adjusted for each storm until the simulated hydrograph peak met the observed value. Curve numbers were averaged to arrive at a value of 41.1. Using this number as a representative value for the watershed, the preceding analysis was repeated. Observed and predicted peak flows show more scatter (r^2=0.663, S_y=26.3 cfs) than the original analysis, however the slope of the regression line does not differ significantly from a 1:1 line at the 95 percent level (Figure 12). The predicted timing of the peak flows were generally much later than the observed peak flows. The effects of precipitation falling late in the storm were greatly over-emphasized, generating peak flows 20-110 hours after the observed peaks for 55 percent of the storms (Figure 13). Observed and simulated hydrographs show improvement in the magnitude of the peak flow values, but little improvement in the over-all shape of the simulated hydrograph (Figure 14).

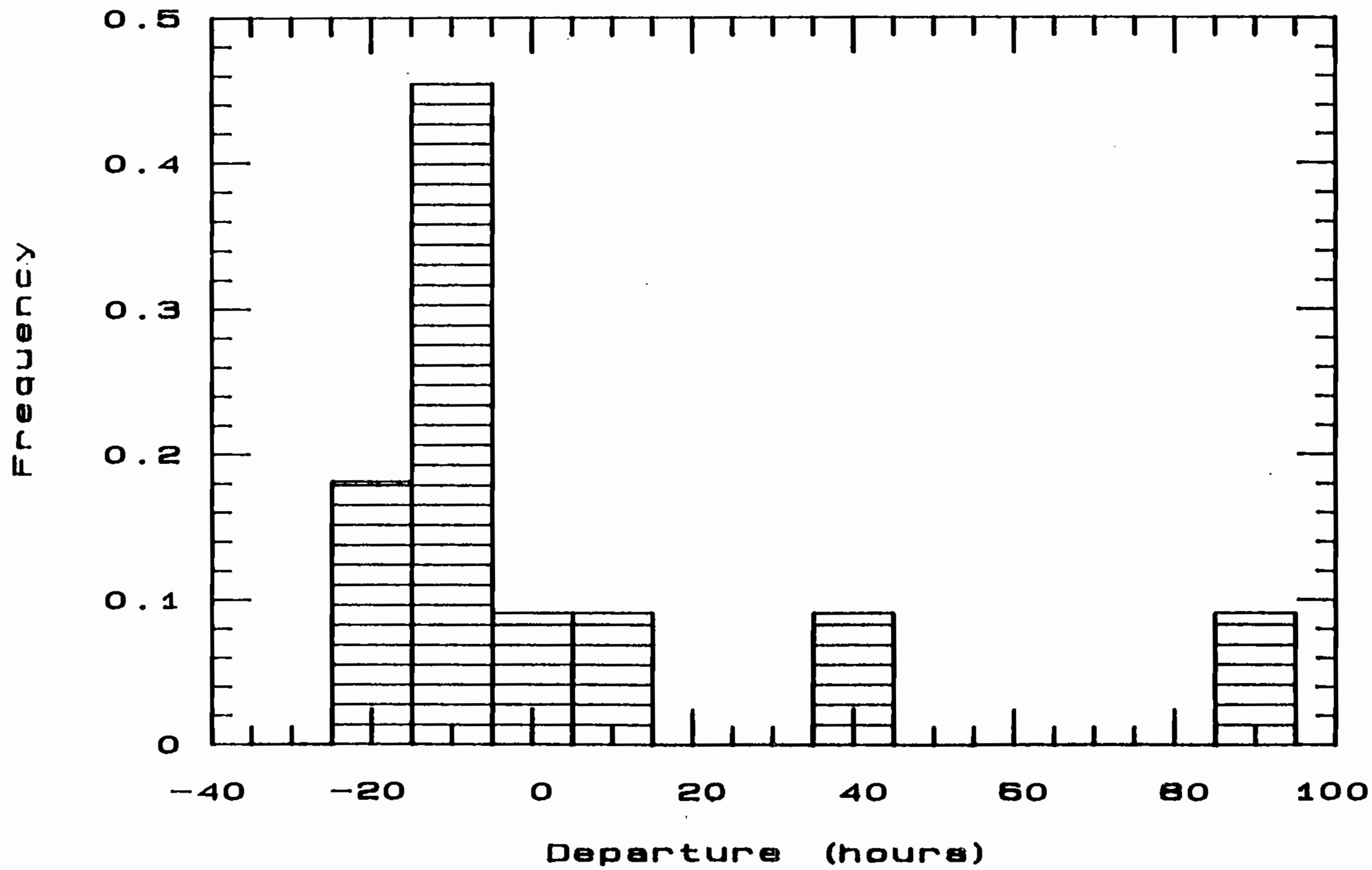


Figure 10. Relative frequency and departure of the timing (predicted-observed) of predicted peak flows using standard SCS procedures, curve number 71.

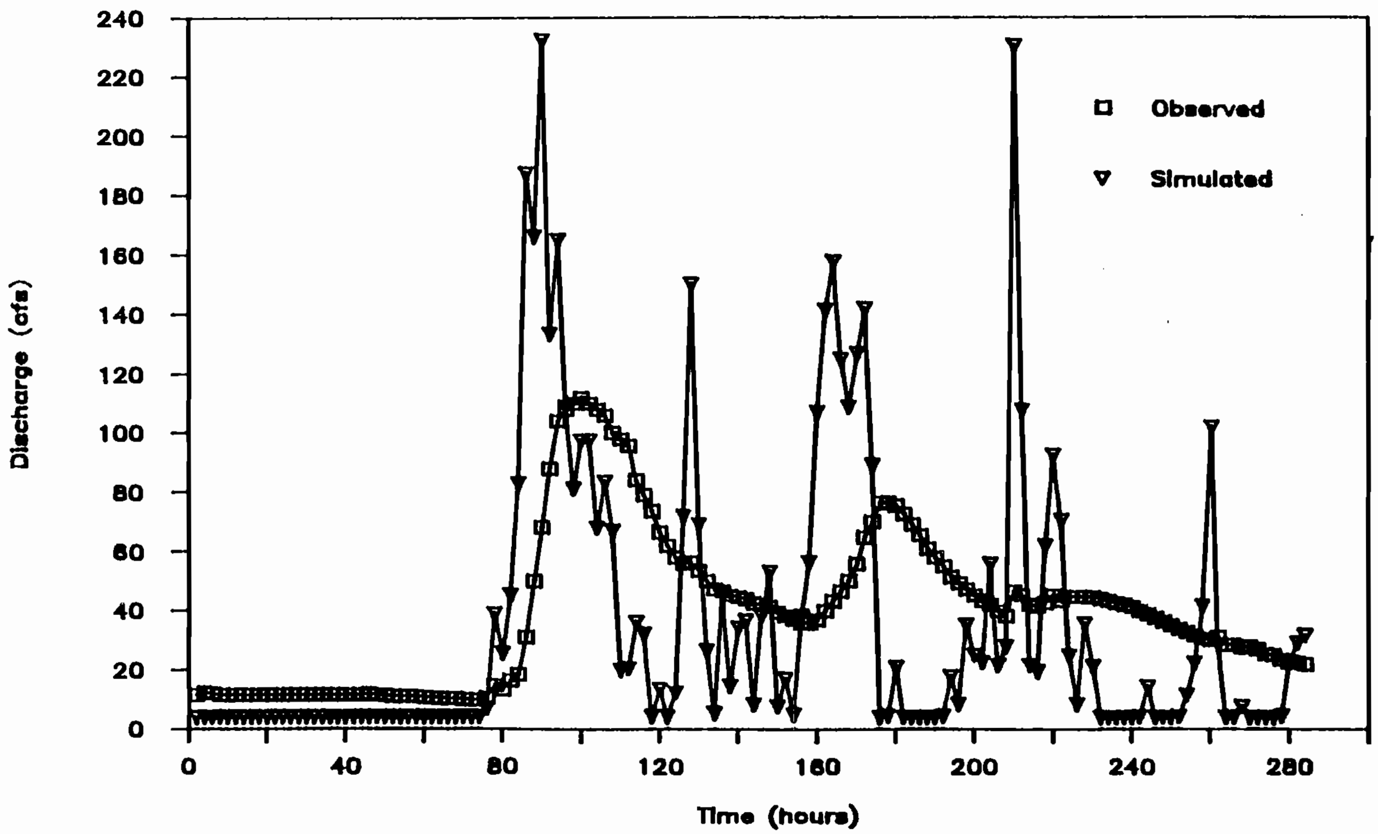


Figure 11. Observed and simulated hydrographs for the storm February 6-17, 1961, Deer Creek, Oregon Coast Range. Simulated runoff from standard SCS procedures, curve number 71.

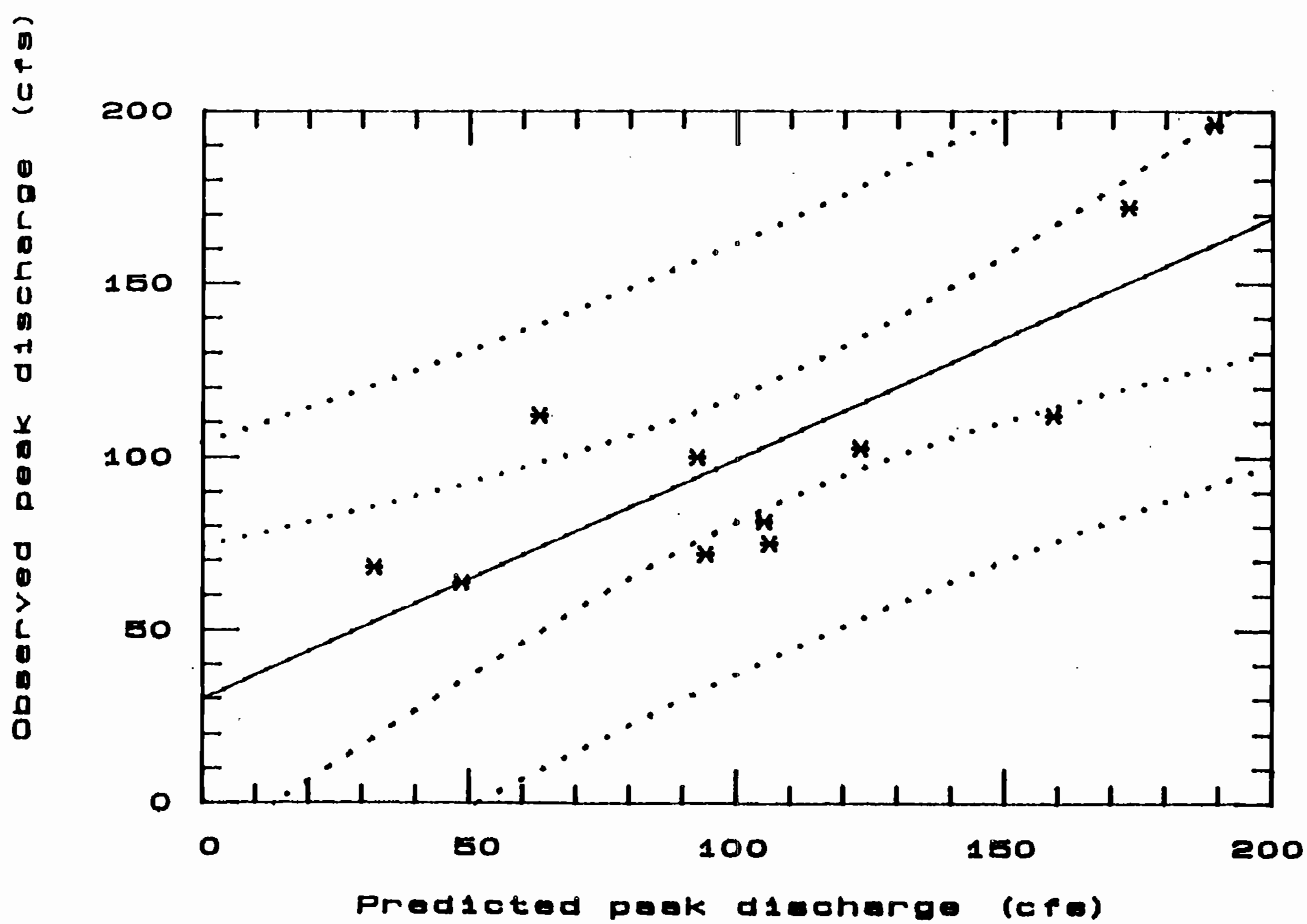


Figure 12. Observed and predicted peak flows with 95% confidence intervals for significance of regression and prediction limits (r^2=0.663). Predicted values from standard SCS unit hydrograph procedures, curve number 41.1.

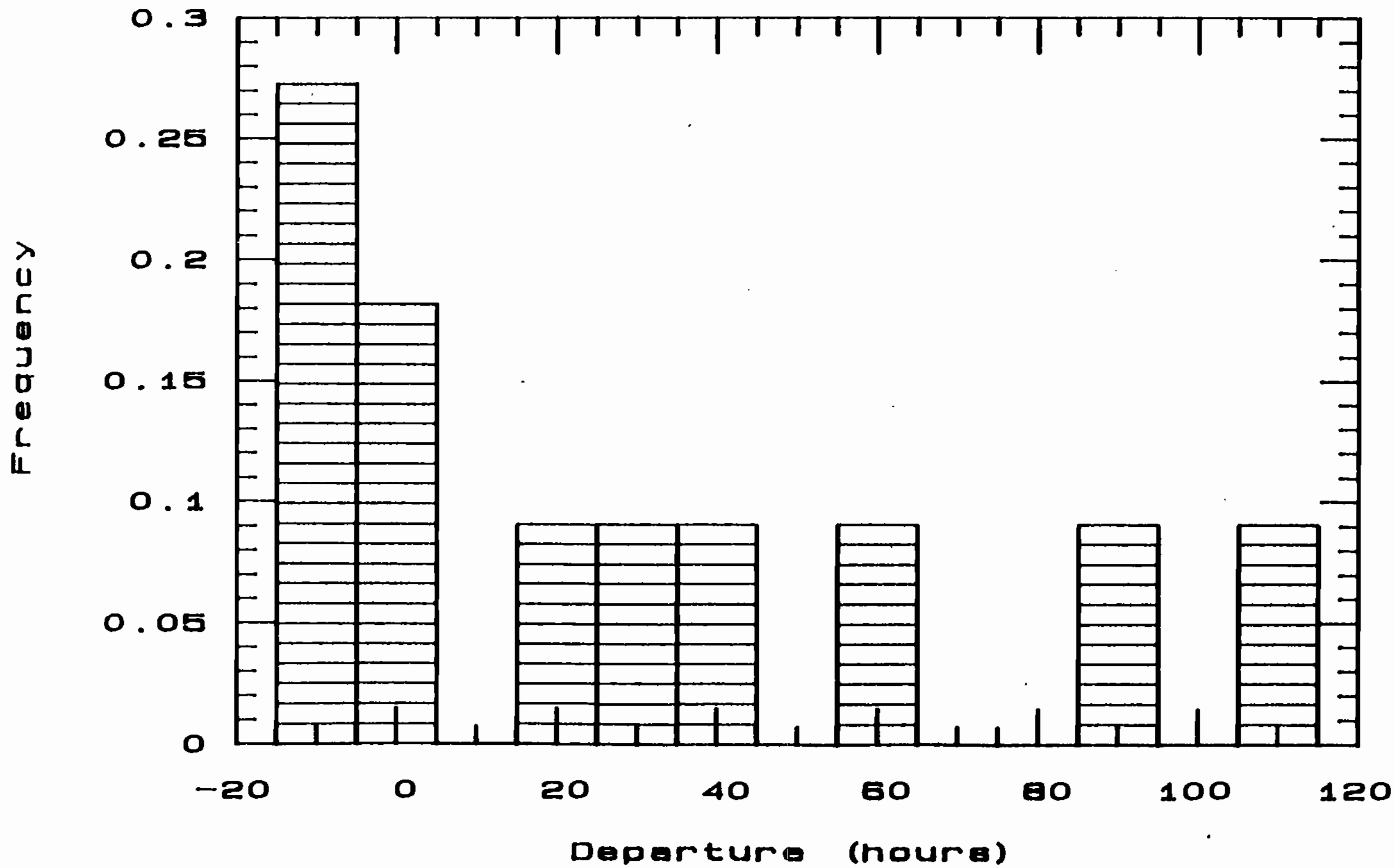


Figure 13. Relative frequency and departure of the timing (predicted-observed) of predicted peak flows using standard SCS procedures, curve number 41.1.

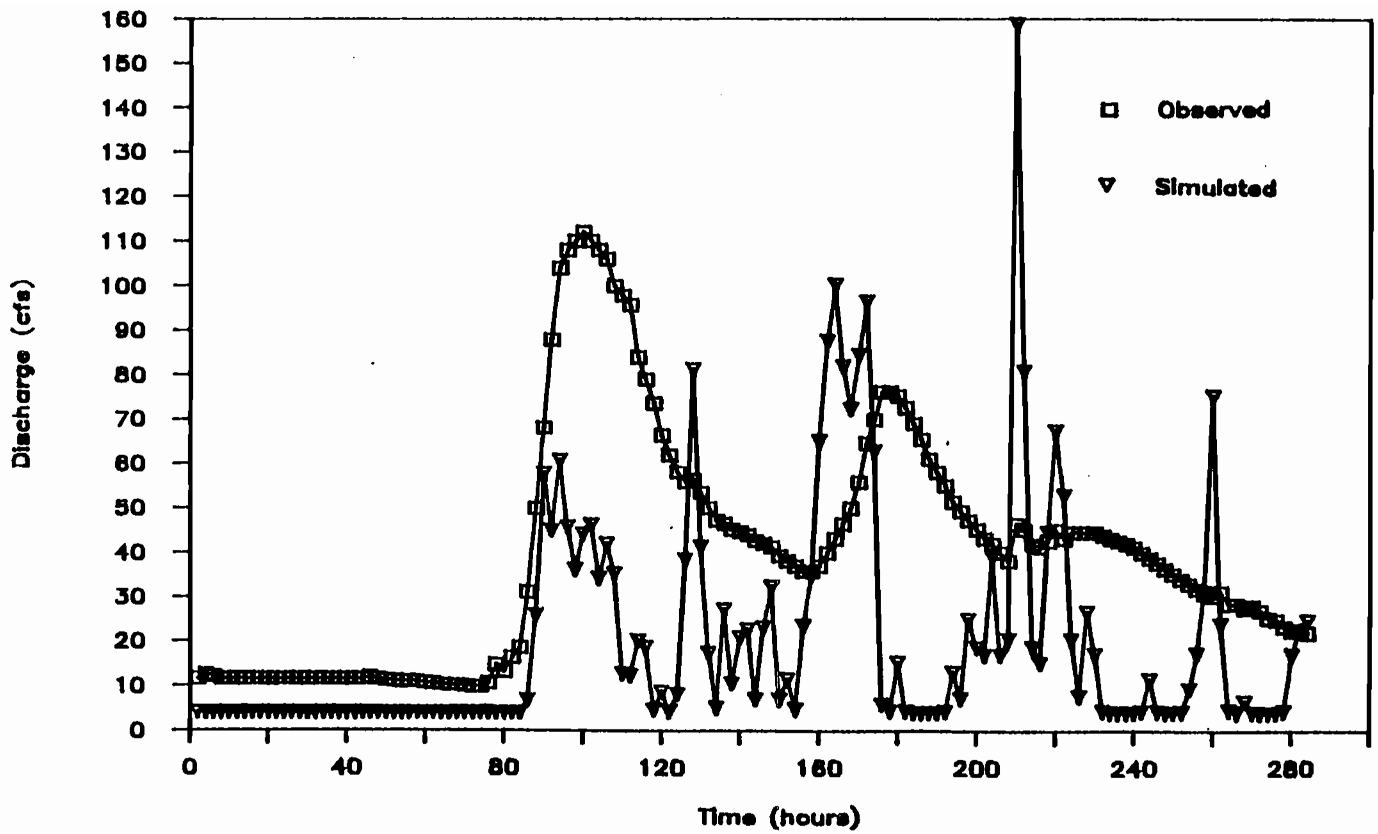


Figure 14. Observed and simulated hydrographs for the storm February 6-17, 1961, Deer Creek, Oregon Coast Range. Simulated runoff from standard SCS procedures, curve number 41.1.

For the "adjusted" model, the shape and peak of the unit hydrograph had been changed, and therefore an average curve number was determined using the same procedure as above. In this case, the average curve number for the adjusted model (49.8) was higher than the average curve number determined for the original model (41.1). This increase is an apparent compensation for the reduced peak of the unit hydrograph. The regression equation relating predicted to observed peak flows shows both a high correlation (r^2=0.887, S_y=15.2 cfs), and a slope that does not significantly differ from a 1:1 line at the 95 percent level (Figure 15). The timing of the predicted peak flows (Figure 16) and the shape of the simulated hydrograph (Figure 17) show very little improvement despite the adjustments made.

Use of SCS Methods on Oregon Coast Range Watersheds

In the case presented above for Deer Creek, standard SCS unit hydrograph procedures over-estimated peak flows by a factor of two or more. These errors are of the same magnitude as those observed by Hewlett (1982) and Settergren, et al., (1985) for forested watersheds in eastern United States. The principal cause for the over-prediction appears to lie in the standard procedures used

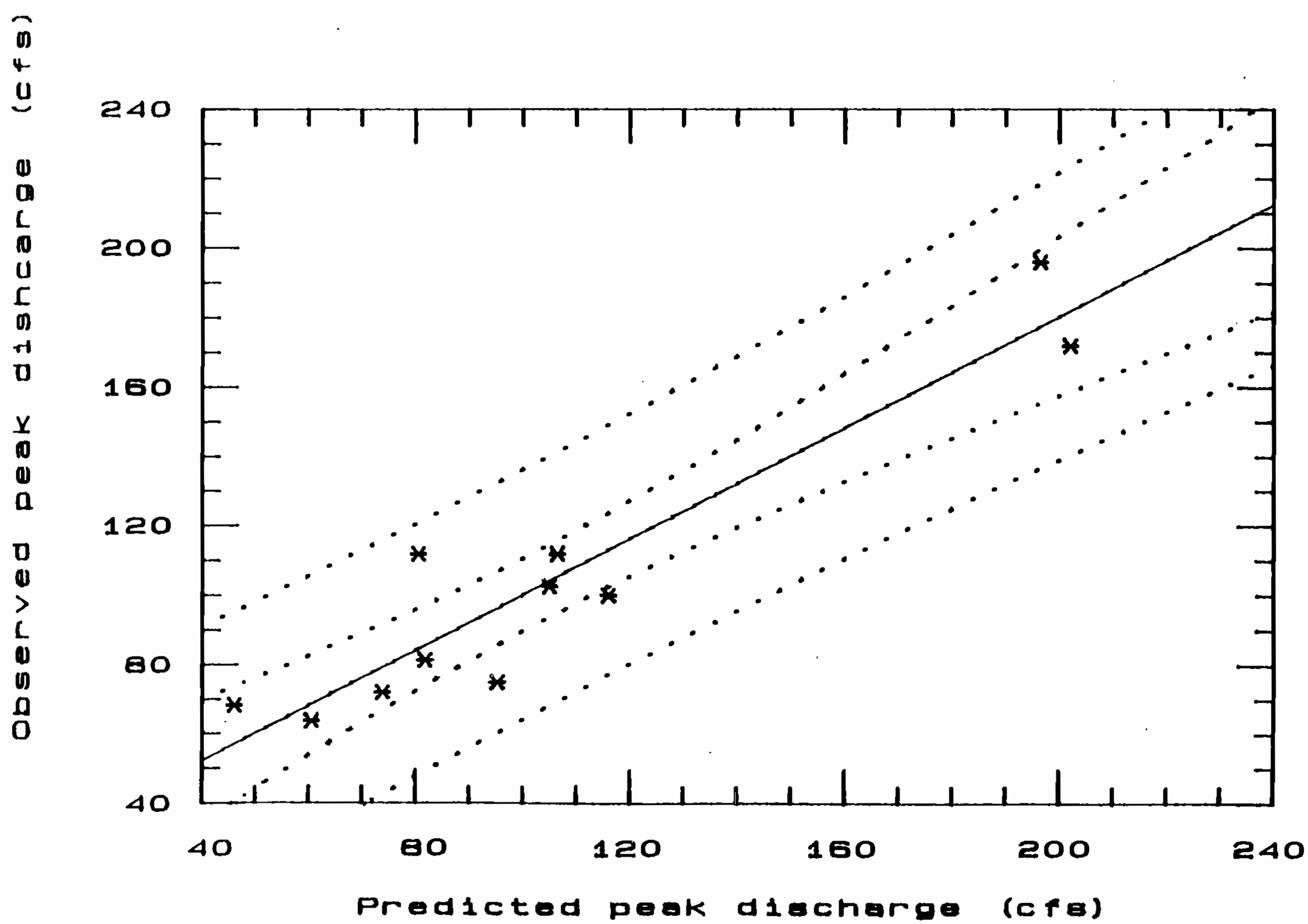


Figure 15. Observed and predicted peak flows with 95% confidence intervals for significance of regression and prediction limits (r^2=0.663). Predicted values from "adjusted" model, curve number 49.8.

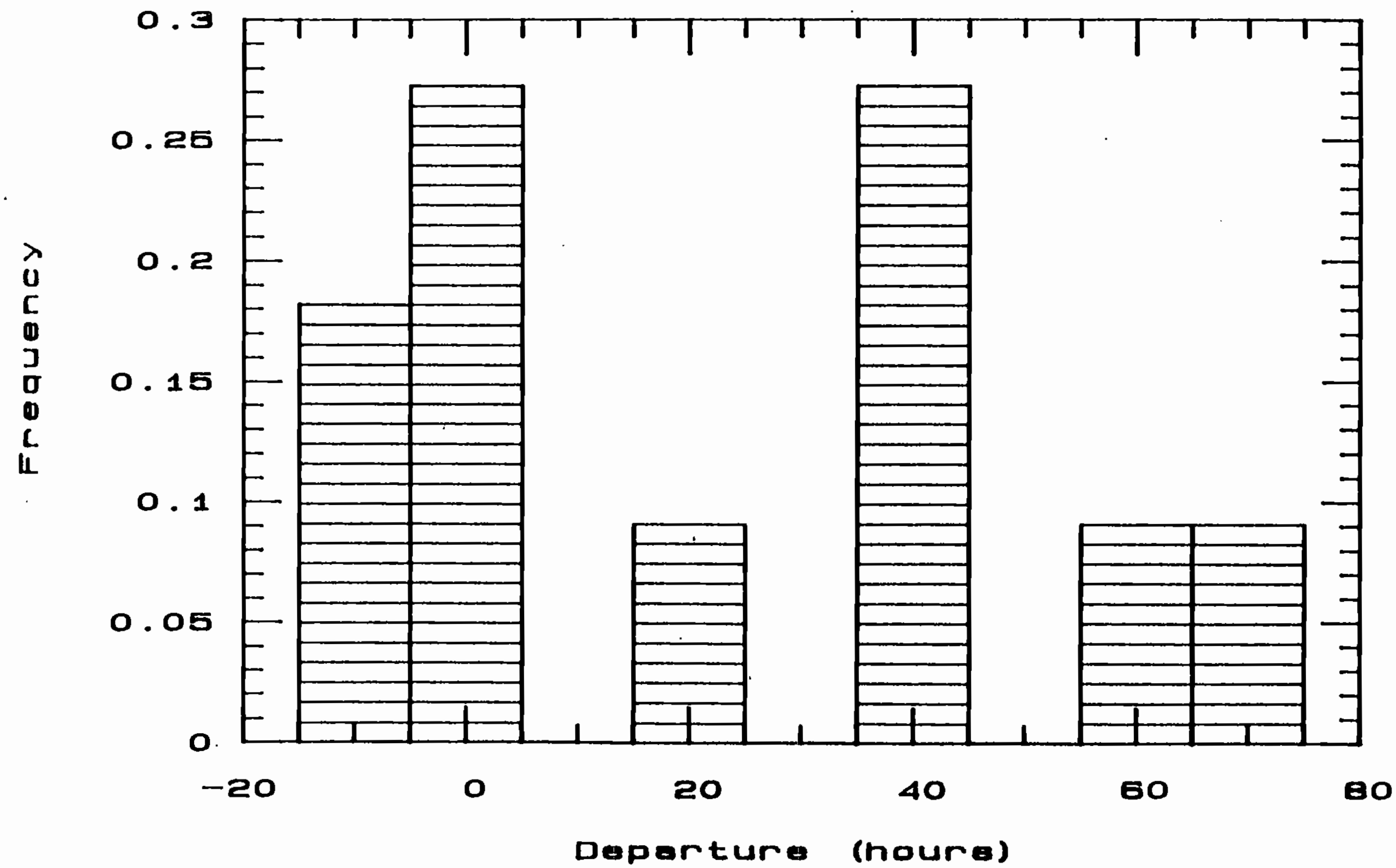


Figure 16. Relative frequency and departure of the timing (predicted-observed) of predicted peak flows using "adjusted" model, curve number 49.8.

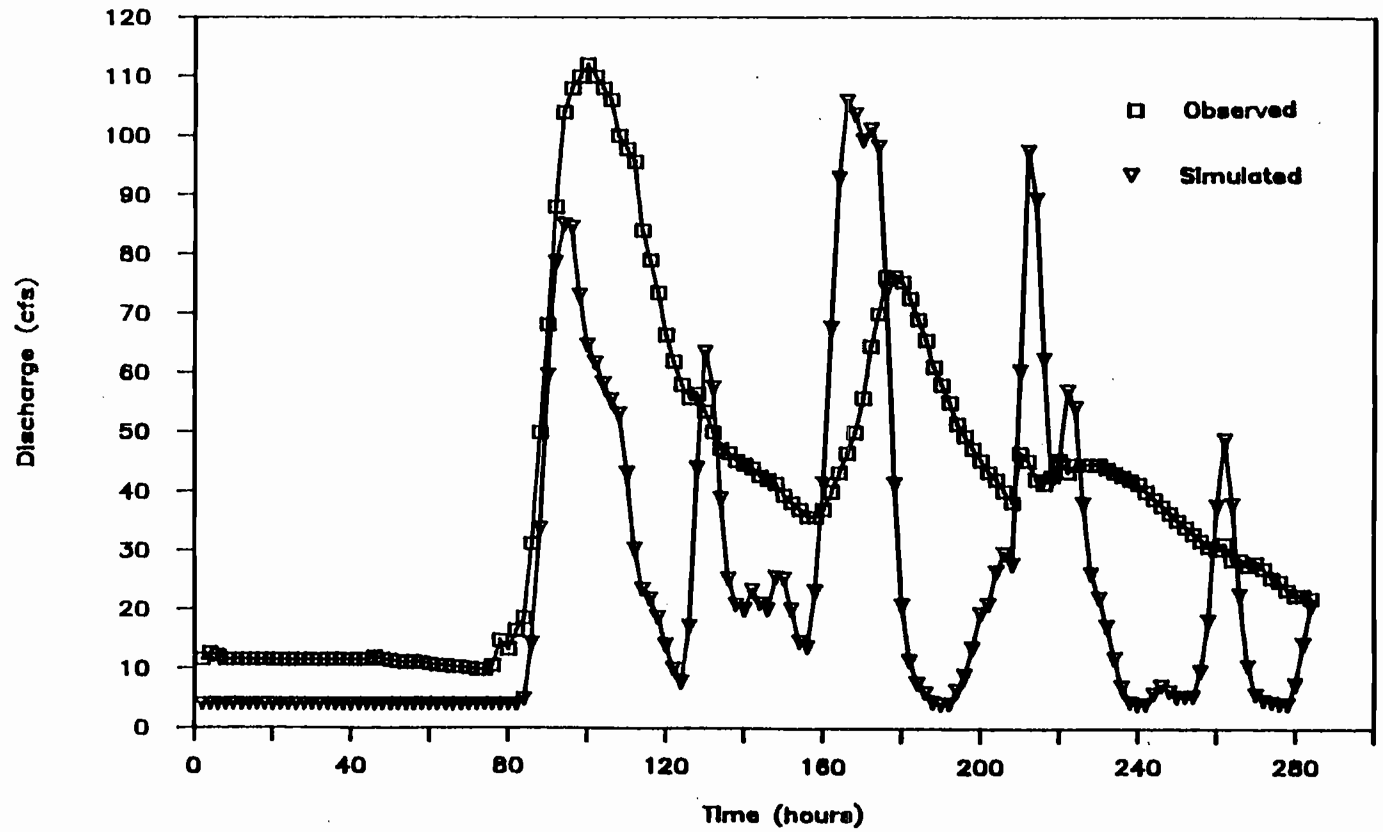


Figure 17. Observed and simulated hydrographs for the storm February 6-17, 1961, Deer Creek, Oregon Coast Range. Simulated runoff from "adjusted" model, curve number 49.8.

to derive the curve number. Hydrologic soil groups (and associated descriptions of runoff processes) do not match with field evidence nor our understanding of water movement on forest mountain watersheds. Antecedent moisture conditions are arbitrarily determined. Furthermore, the influence of management practices upon changes in runoff volumes and peak flows are not supported by watershed research studies.

When the average curve number (41.8) was derived for the "original" model, the tendency of the model to greatly over-predict peak flows (using curve number 71) was removed as a result of the fitting procedure. However, the standard error of the peak flow estimates using curve number 41.8 were greater than the standard error of the estimates using curve number 71. In both cases, the timing of the predictions was highly influenced by high intensity "pulses" of precipitation late in the storm. When the standard SCS coefficients and relationships were adjusted and/or fitted (watershed lag, time of concentration, shape of the unit hydrograph, and curve number), the peak flow predictions and the shape of the storm hydrographs were improved somewhat, but the timing of the peak flow predictions was not.

Rapid rises in the simulated storm hydrographs occur as a result of the increasing proportion of rainfall that becomes runoff. The time distribution of rainfall is not accounted for by equation 7, and therefore the simulated hydrographs are greatly influenced by changes in precipitation intensity. The rapid fall of the simulated hydrographs occur as a result of the duration of each unit hydrograph. For Deer Creek, the recession limb of the unit hydrograph using standard SCS procedures is only 0.80 hours (curve number 71, equations 3, 4, and 6, and Tr/Tp ratio 1.67). Therefore, as a storm passes, and rainfall stops, the simulated discharge necessarily falls to zero 0.80 hours later. A lower curve number and an increased Tr/Tp ratio increases the duration of the unit hydrograph, however, in this study, simulated recessions fell much more quickly than the observed recessions despite these adjustments.

Errors in the timing of the peak stem largely from the assumed rainfall/runoff relationship. The effects of individual bursts of rainfall occurring late in the event are greatly over-emphasized, causing simulated peak flows to occur well after the observed peak. Most rainfall events that produce high flow events on watersheds of the Oregon Coast Range, have a long, complex rainfall distribution pattern. The increasing proportion of rainfall that is converted to streamflow is an unrealistic approach to simulating storm runoff, except for the simplest of rainfall events.

Prediction of peak flows on forested Coast Range watersheds using SCS methods hinges upon the appropriate choice of a curve number. While standard procedures clearly resulted in an over-prediction of peak flows in the Deer Creek example, appropriate curve numbers for other watersheds remain unknown. It is not recommended that the derived curve number for Deer Creek be applied to other Coast Range watersheds. Use of the SCS method as a single event simulation model is confined to the limits of application of the rainfall/runoff equation (equation 7). No amount of adjustment of coefficients will compensate for the limits of equation 7. Artificially adjusting curve numbers following periods of no rain and satisfying a new initial abstraction is a truly unrealistic approach to simulating streamflow. The ambiguity of curve numbers and the limits of application of the rainfall/runoff equation preclude the use of SCS procedures for use as a peak flow prediction model and/or a streamflow simulation model for forested watersheds of the Oregon Coast Range.

ANTECEDENT PRECIPITATION INDEX METHOD

An antecedent precipitation index (API) method of storm runoff simulation was developed when existing methods were found impractical or theoretically inappropriate for use in the Oregon Coast Range. Soil Conservation Service unit hydrograph procedures proved too responsive to precipitation intensity, and results are strongly dependent upon the subjectively derived curve number. Extensive and detailed watershed data for calibration and testing of a sophisticated physically based method of hydrograph generation was unavailable. Hence, an API method was developed using precipitation/streamflow records from five Oregon Coast Range watersheds, and was tested using records from a sixth watershed.

Watershed Selection, Sources of Data

Four criteria were used to select watersheds for use in this study:

(1) Forested watershed in the Oregon Coast Range.
(2) Corresponding rainfall-runoff records of at least five years.
(3) Recording precipitation gage less than five miles from the centroid of the watershed.
(4) No diversion or regulation of streamflow above the gaging station.

Six watersheds were found to meet these criteria. Data from Needle Branch, Flynn Creek, Deer Creek, North Yamhill, and the North Fork of the Siuslaw watersheds were used to formulate the API model and will be referred to as the "calibration watersheds." The Nestucca watershed data was used as an independent test of the API method.

Deer Creek, Flynn Creek, and Needle Branch watersheds were experimental watersheds in the Alsea Watershed Study (1959-1972). These watersheds remained in an undisturbed condition from 1959-1966. In 1966, the Deer Creek watershed was patch-cut (29 percent), Needle Branch was clear-cut and burned (89 percent) and Flynn Creek remained undisturbed. Changes in peak flows and storm runoff volumes following these logging activities were studied by Harr, et al. (1975). They found that peak flows and storm runoff volumes increased significantly following clear-cutting and burning, but did not change significantly following patch-cutting. For this reason, data from the Needle Branch watershed was used from 1959-1966, while data from Deer Creek and Flynn Creek were used throughout their respective periods of record (Table 4). Road-

Table 4. Summary of watershed characteristics.

Watershed	Stream gage*	Precip. gage	Distance separating watershed and precipitation gage (miles)	Period of record (years)**	Drainage area (mi^2)	Mean watershed elevation (feet)	Precip. gage elevation (feet)
Needle Branch Creek	306700	***	0.5	8 (9)	0.27	815	480
Flynn Creek	306800	***	1	14(20)	0.78	1090	550
Deer Creek	306810	***	1	11(19)	1.17	1100	690
Nestucca River	302900	Haskins Dam	3	8 (8)	6.18	2040	840
N. Yamhill River	194300	Haskins Dam	5	6 (6)	9.03	1190	840
N. Fk. Siuslaw River	307645	Mapleton 2NNW	5	5 (7)	41.2	1170	40

* USDI Geological Survey Station Number.

** Number of years of coinciding precipitation and runoff data; number of runoff events evaluated in parentheses.

*** Unpublished records from Alsea Watershed Study.

building and logging activities that may have taken place on the North Yamhill, North Fork of the Siuslaw, and Nestucca watersheds were not taken into consideration.

Similar formations of bedded sediments underlie the six watersheds (Burroughs, et al., 1973). Because of the similar geologic nature of the watersheds, runoff processes are expected to be similar as well.

Streamflow records for all watersheds were available from the USDI Geological Survey. Most of the records available were the original gage-height charts, and more recent bi-hourly stage or discharge data was available from computer files. Original precipitation charts for Needle Branch, Flynn and Deer creeks were available from gages near each of the three watersheds (Table 4). Precipitation data for the North Yamhill, North Fork of the Siuslaw, and Nestucca watersheds was gathered from published records of the Mapleton 2NNW and Haskins Dam gages (USDC National Oceanic and Atmospheric Administration, 1960-1986). Bi-hourly observations of precipitation and streamflow were the smallest time intervals consistently available for all watersheds in this study.

Method Description and Development

Rainfall-Runoff Correlation, Derivation of API

Much progress has been gained in understanding the importance of specific rainfall characteristics that contribute to storm flow volumes and peak flows (eg. Hewlett, et al., 1977, 1984; Bren et al., 1987). However, generation of entire storm hydrographs using rainfall inputs alone has not been accomplished.

Streamflow occurring at any point in time can be thought of as a function of the volume and temporal distribution of precipitation preceding that point in time. Cumulative storm precipitation volume and cumulative storm runoff volume have been shown to be strongly correlated (Hewlett, et al., 1977, 1984; Bren et al., 1987). Cumulative rainfall would not be a good predictor of stream discharge throughout a storm since cumulative precipitation can only increase or stay constant while discharge rises and falls through time.

Cumulative precipitation within a specific time interval (eg. 24-hours) may be positively correlated with stream discharge at the end of that time period. For example, if precipitation amounts are recorded hourly, cumulative precipitation during any 24-hour interval could be correlated with hourly stream discharge at the end of the 24-hour period. Values of 24-hour cumulative rainfall will rise and fall as a storm approaches and passes a given watershed (just as streamflow would). With this approach, all observations of hourly precipitation within the specified duration of a "moving window" of cumulative precipitation would be weighted equally. That is, precipitation occurring early in the interval would contribute to the cumulative interval precipitation as fully as precipitation occurring at the end of the interval. A system that responds in this manner to precipitation inputs would have complete "memory" of rain falling within the interval, and have no "memory" of rain falling prior to it. Correlation between precipitation volumes within a "moving window" of time and stream discharge at the end of the time interval are plausible, but perhaps unrealistic. For example, results of this method would depend greatly upon the length of window chosen.

Precipitation falling prior to a specific point in time of interest (antecedent precipitation) would be better correlated with stream discharge if it was not weighted as fully as precipitation occurring nearer the time of interest. A system responding to precipitation in this manner would have a complete "memory" of rain falling at the time of interest, a partial "memory" of rain that fell a short time ago, and only a vague "memory" of rain that fell a long time ago. Thus, the influence of a given precipitation observation on stream discharge observations would "decay" through time. This is the premise of the Antecedent Precipitation Index (API) method.

In this study, the influence of antecedent precipitation on stream discharge was assumed to "decay" at the same rate as the recession limb of a hydrograph during periods of no rain. Recession analysis was carried out in the manner described by Garstka, et al., (1958) to determine the rate of "decay." While Garstka, et al., (1958) used daily observations of streamflow to derive "recession factors" for snowmelt runoff, two-hour observations were used in this study. The recession coefficient was determined by deriving the slope of the line formed by plotting stream discharge during periods of no rain, with the discharge 2-hours prior to those observations. For Deer Creek, the slope of the line was 0.929, that is, the discharge at any time during periods of no rain is 92.9 percent of the discharge two hours ago (Figure 18). Similarly, the discharge two hours in the future is expected to be 92.9 percent of the discharge now, assuming no rain falls in the next two hours.

The recession coefficient (C) was used to "decay" the importance of individual rainfall observations through time to formulate an antecedent precipitation index at any time (API_t):

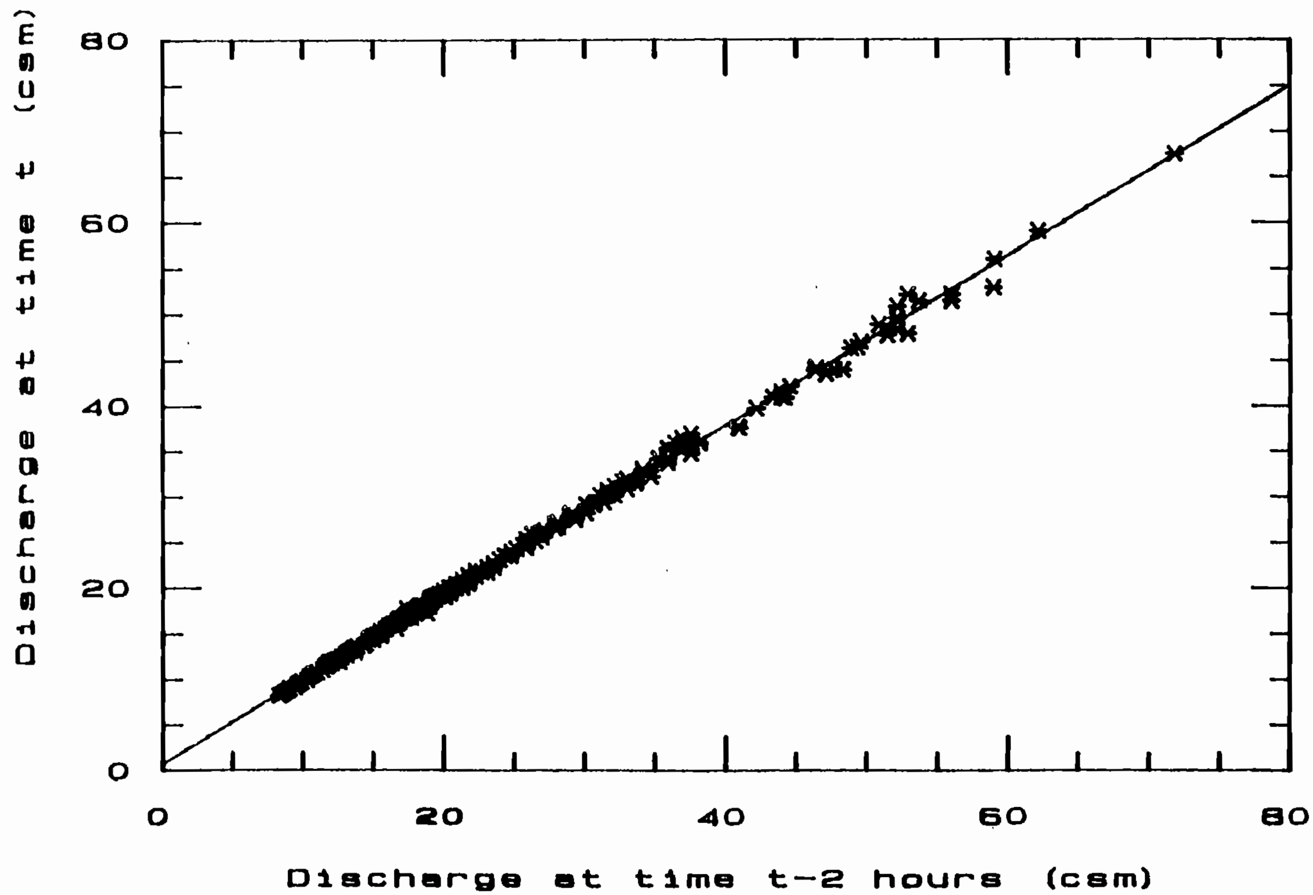


Figure 18. Recession limb data for Deer Creek, Orégon Coast Range. Recession coefficient (C) is 0.929.

$$API_t = API_{t-\Delta t} * C + P_t \tag{8}$$

API_t = Antecedent precipitation index at time t (inches)
Δt = Time interval of precipitation observations (hours)
C = Recession coefficient (dimensionless)
P_t = Precipitation volume during one Δt ending at time t (inches)

Values of API at any time t are dependent upon all precipitation occurring prior to that time. New observations of precipitation during a time interval (Δt) contribute fully to a new value of API, while previously fallen precipitation is decayed through time. API at any time has a complete "memory" of precipitation that has fallen during the most recent time interval, a partial "memory" of rain that fell a short time ago, and only a vague "memory" of rain that fell a long time ago.

Equation 8 can only be used when the time interval of precipitation observations and the time interval used to derive the recession coefficient are equal. However, the equation can be easily adjusted for any time interval of precipitation observations or any time interval of streamflow observations used to derive C by the following relation:

$$C(a) = C(b)^{(\Delta t(a)/\Delta t(b))} \tag{9}$$

C(a) = Recession coefficient based on time interval $\Delta t(a)$
C(b) = Recession coefficient based on time interval $\Delta t(b)$
t(a)= Time interval of precipitation observations
t(b)= Time interval used to derive recession coefficient C(b)

Equation 8 becomes:

$$API_t = API_{t-\Delta t} * C(a) + P_t \tag{10}$$

Recently, Ziemer and Albright (1987) have concurrently developed a similar equation for use in the prediction of peak flows through subsurface soil pipes in the north-coastal region of California. Depending on pipe size, 60 to 66 percent of the variation in peak flows through soil pipes was explained by peak values of API.

API and Discharge Correlation

Runoff events used in this analysis were defined to begin and end using the baseflow separation technique described by Hewlett and Hibbert (1967). During the formulation stages of model development, all events with peak discharge above the USDI Geological Survey base level for reporting peak flows (USDI Geological Survey, 1959-1972) were included in the analysis. This criteria (used for Deer Creek, Flynn Creek, Needle Branch, and N. Fork

Siuslaw River) resulted in not using data or the inclusion of more than one large event from a single water year. Annual peak flows were analyzed for N. Yamhill, and Nestucca rivers.

Istok and Boersma (1986), and Lyons and Beschta (1983) have demonstrated the importance of quantifying antecedent precipitation at least several days prior to a discharge event in western Oregon. For this reason, the calculation of API values began 72 hours before the runoff events were defined to begin. Seventy-two hours is somewhat arbitrary, however, the relationship between API and discharge on Flynn Creek was not improved when API values were calculated beginning seven days before the runoff events began. The optimum amount of time to begin calculating API values before a runoff event was not explored.

Corresponding two-hour values of API were correlated with the two-hour discharge values (cubic feet per second per square mile, csm). It was found that a linear function best described the relation between API and the square root of discharge (Figure 19). Although values of discharge and API are highly auto-correlated, a least squares procedure provided an objective means to fit a line to the data.

The slope of the line in Figure 19 can be thought of as the rate of response of the watershed to precipitation

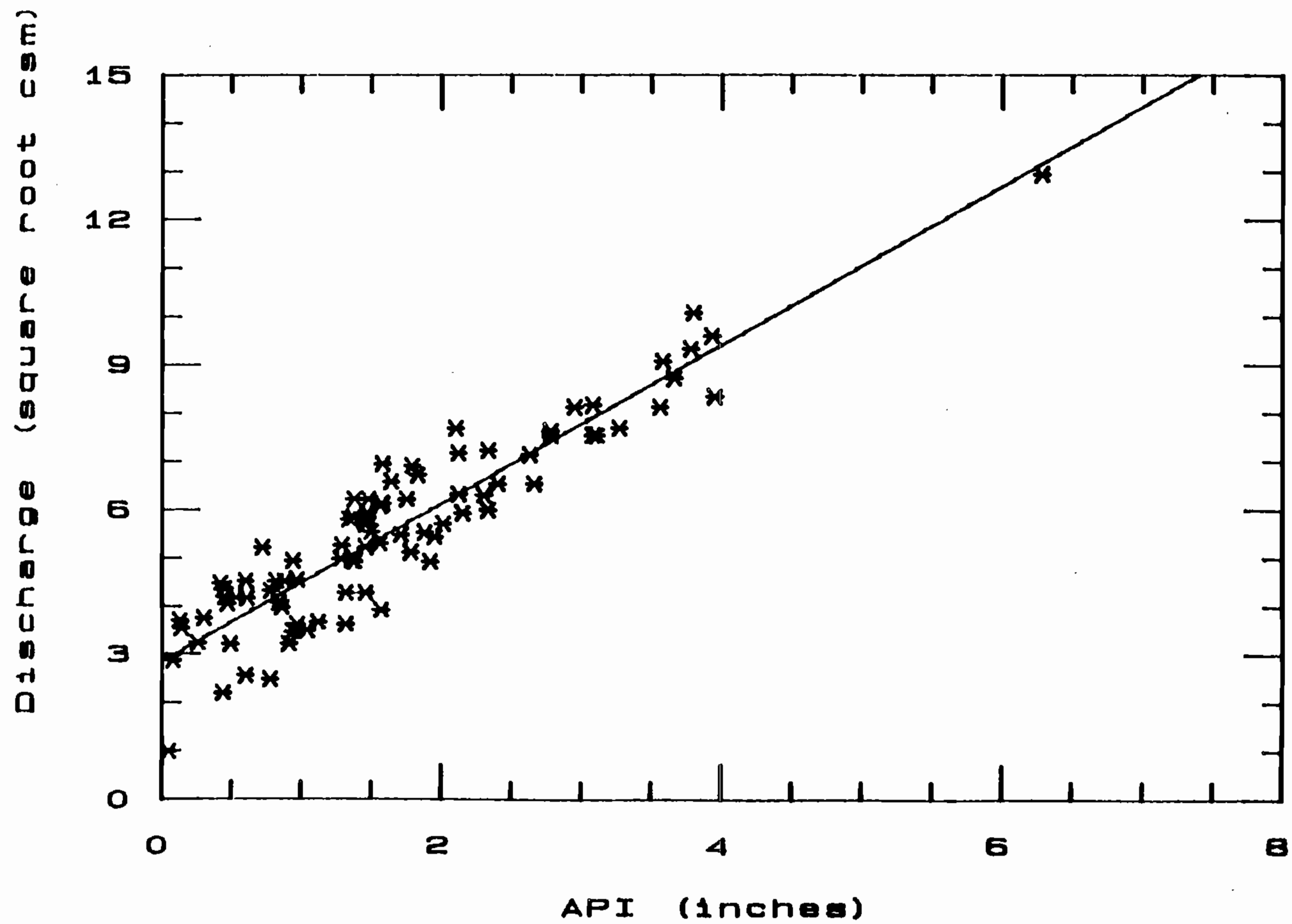


Figure 19. API and discharge values from Deer Creek, Oregon Coast Range.

inputs or changes in API. The y intercept can be thought of as the average winter base flow prior to and following high flow events. By using the precipitation record for any rainfall event and equation 8, storm hydrographs can be simulated using the relationship between API and stream discharge for a specified watershed (Figure 20). Relationships between API and the square root of stream discharge were developed for each of the five calibration watersheds.

Correlation of Coefficients with Basin Characteristics

Three coefficients are necessary to calculate the streamflow from rainfall on a given watershed using the API method: (1) a recession coefficient (C), and the (2) intercept (I) and (3) slope (S) of the line relating API to the square root of discharge (eg. Figure 19). By evaluating the variability of these coefficients among watersheds using watershed characteristics as independent variables, a predictive model was developed for use on ungaged watersheds.

Recession coefficient

The recession coefficients (C) derived for the calibration watersheds ranged from 0.907 on Needle Branch Creek, to 0.949 on the N. Fork of the Siuslaw River (Table 5). Variability among recession coefficients may be

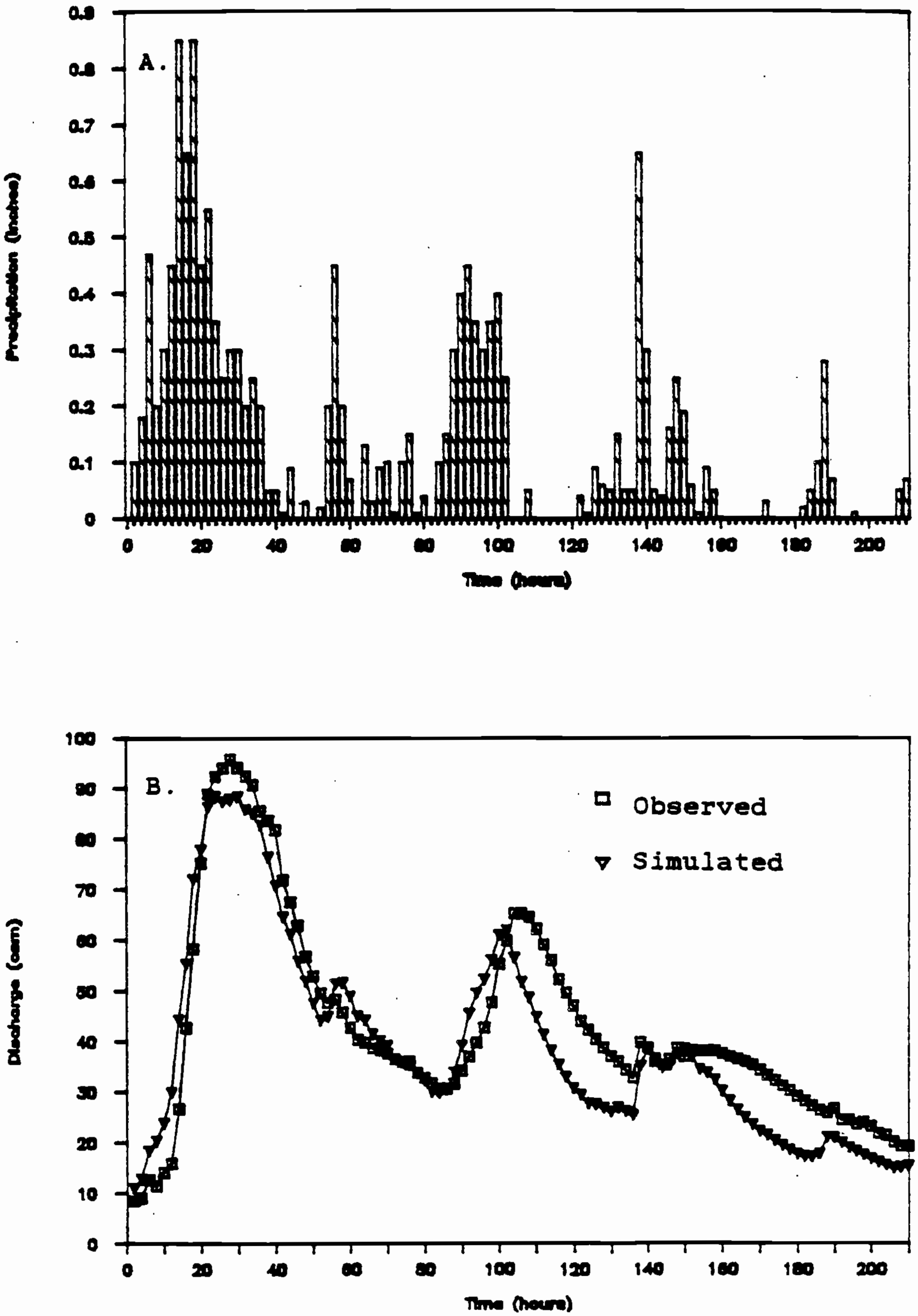


Figure 20. (A) Hyetograph, and (B) simulated and observed hydrographs for the storm February 9-17, 1961, Deer Creek, Oregon Coast Range.

Table 5. Values of C, S, and I for the five calibration watersheds; original and normalized models.

	Original Model			Normalized Model *		
Watershed	C	S	I	C	S	I
Needle Branch Creek	0.907	2.10	2.85	0.907	2.10	2.85
Flynn Creek	0.913	1.48	3.86	0.936	1.44	3.22
Deer Creek	0.929	1.61	2.94	0.928	1.61	3.06
N. Yamhill River	0.888	2.14	3.35	0.930	1.74	2.83
N. Fk. Siuslaw River	0.949	1.42	3.24	0.960	1.42	3.18

* Used only flows $\leq$60 csm for derivation of "C".

explained by watershed size, and factors affecting average response of a watershed to rainfall inputs: soil depth to bedrock, drainage density, soil conductivity, vegetation type or stage of development, side-slope gradient, channel gradient, channel roughness, and basin shape and, perhaps, land use.

Errors in the estimate of C also arise as a result of the methods used to gather the data. Periods of no rainfall at the precipitation gage do not necessarily indicate that rain is not falling on the watershed--particularly when the precipitation gage and the watershed are separated by a considerable distance. Artificially high recession coefficients would result from data used when rain was actually falling on the watershed. Periods of no rain following extremely large peak flow events were not observed on each watershed. Since the slope of the line which defines C is highly influenced by these observations, the data sets were restricted to the range of flows observed on all watersheds (less than or equal to 60 csm) during periods of no rain. This procedure "normalized" the values and removed the variability in C caused by the data gathering procedure (Table 5). Additional variability associated with basin characteristics could then be analyzed.

The model used to describe streamflow recession may ultimately influence the predictive ability of the API method. Garstka (1958) described a two-slope model of snowmelt recession limb analysis in which the slope varies with stream discharge. The slope of the recession data was greater for low discharge than for high discharge. Boughton (1986) observed the occurrence of non-linear recessions in small, wet catchments of eastern Australia. Non-linear recessions were also observed in the watersheds used in this study. A two-slope piecewise linear regression model was fitted to the Flynn Creek recession data as a close approximation of a non-linear recession. New API values were derived and a new model was developed for streamflow simulation. A comparison of peak flows predicted by the new model (based on a two-slope recession model) and the original model (based on a single-slope linear recession model) revealed no improvement. The more complex two-slope recession model was abandoned in favor of a simple linear model.

When used to calculate API, C can be thought of as the relative "memory" a basin has regarding previously fallen precipitation. Small basins drain quickly and "remember" very little of past rainfall (low values of C), while very large basins drain more slowly and "remember" rainfall for a longer period of time. Recession coefficients derived from extremely large basins should, in theory, approach an upper limit of 1.0.

Watershed areas were correlated with recession coefficients; a non-linear regression equation that approaches an expected upper limit of 1.0 for large watersheds was developed:

$$C = 1.0 - 0.0773e^{-0.0158a} \tag{11}$$

C = Recession coefficient (dimensionless)
a = Watershed area (square miles)

Equation 11 explains 81.2 percent of the variability of the five recession coefficients used in its formulation. This non-linear relationship provides a conceptually pleasing model, in that values of C for very large watersheds can only approach--but never exceed--1.0. However, for a small sample size, the least squares estimators for non-linear regression are not normally distributed and unbiased. Therefore the standard error of the estimates remain unknown.

For the range of watershed sizes used in this study, a linear approximation of the relationship between the natural logarithm of watershed size was obtained (P=0.072; Figure 21):

$$C = 0.925 + 7.93E{-}3 * Ln(a) \tag{12}$$

Watersheds that experienced extremely large peak flows (greater than 60 csm) and were followed by periods of no

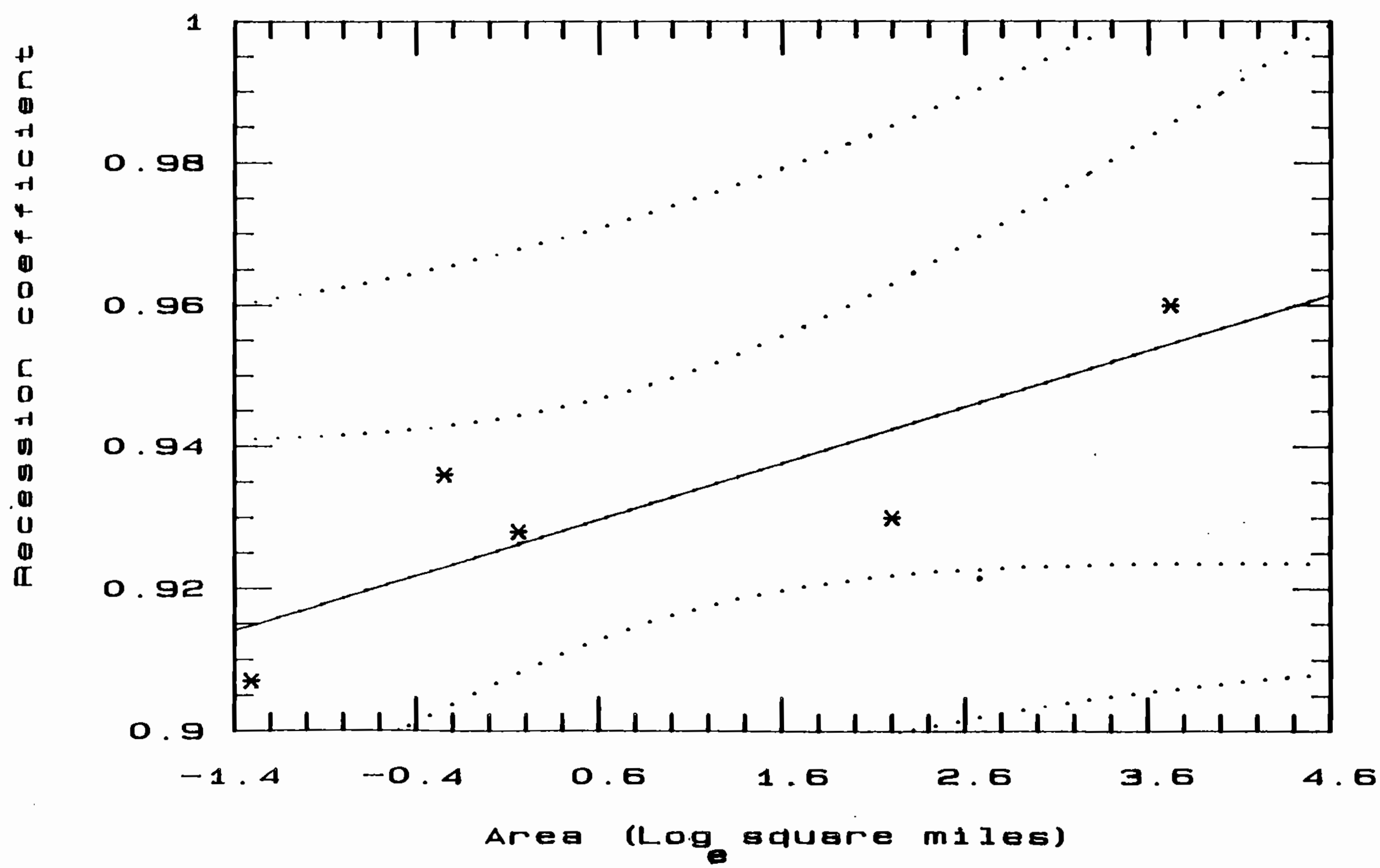


Figure 21. Relationship between watershed area and recession coefficient (C) (r^2=0.713, S_y=0.012).

rain, had recession coefficients that averaged 0.025 less than the normalized coefficients. Therefore the best approximation of a "true" recession coefficient throughout the range of flows that an ungaged watershed may experience is estimated as 0.025 less than the that from equation 12. Equation 13 was used to estimate C for the test watershed in this study:

$$C = 0.900 + 7.93E-3 * Ln(a) \tag{13}$$

<u>Slope</u>

The slope (S) of the line relating API to discharge (square root of csm) for the five calibration watersheds ranged from 1.42 (N. Fk. Siuslaw River) to 2.14 (N. Yamhill River). Models were also derived for each watershed using the normalized recession coefficients to calculate API values. Values of S for the new models ranged from 1.42 (N. Fk. Siuslaw River) to 2.10 (Needle Branch Creek) (Table 5).

The slope of the line fitted to API and discharge represents the rate change in discharge with the rate change in API. Discharge from smaller basins is likely to respond more quickly to precipitation than discharge from larger basins. Since watershed area is also related to the recession coefficient (C), larger watersheds have a greater "memory" of previously fallen precipitation, and individual 2-hour precipitation amounts have a smaller relative influence on API values. Therefore, the rate change in discharge to the rate change in API values (S) is expected to be lower for larger watersheds with high values of C.

A preliminary examination of the relationship between C and S was carried out using data from Deer Creek. Recession coefficients were artificially adjusted upward and downward from the mean value of C (i.e., 0.928) derived for Deer Creek. New models relating API and discharge were formulated for each value of C; S was found to be inversely related to C. Small changes in C had a strong influence on S.

For the five calibration watersheds, recession coefficients were used as an independent variable for the prediction of S. A simple linear relationship was fitted to the data (P=0.049; Figure 22):

$$S = 13.6 - 12.8 * C \tag{14}$$

Equation 14 was used for prediction of S on the test watershed in this study.

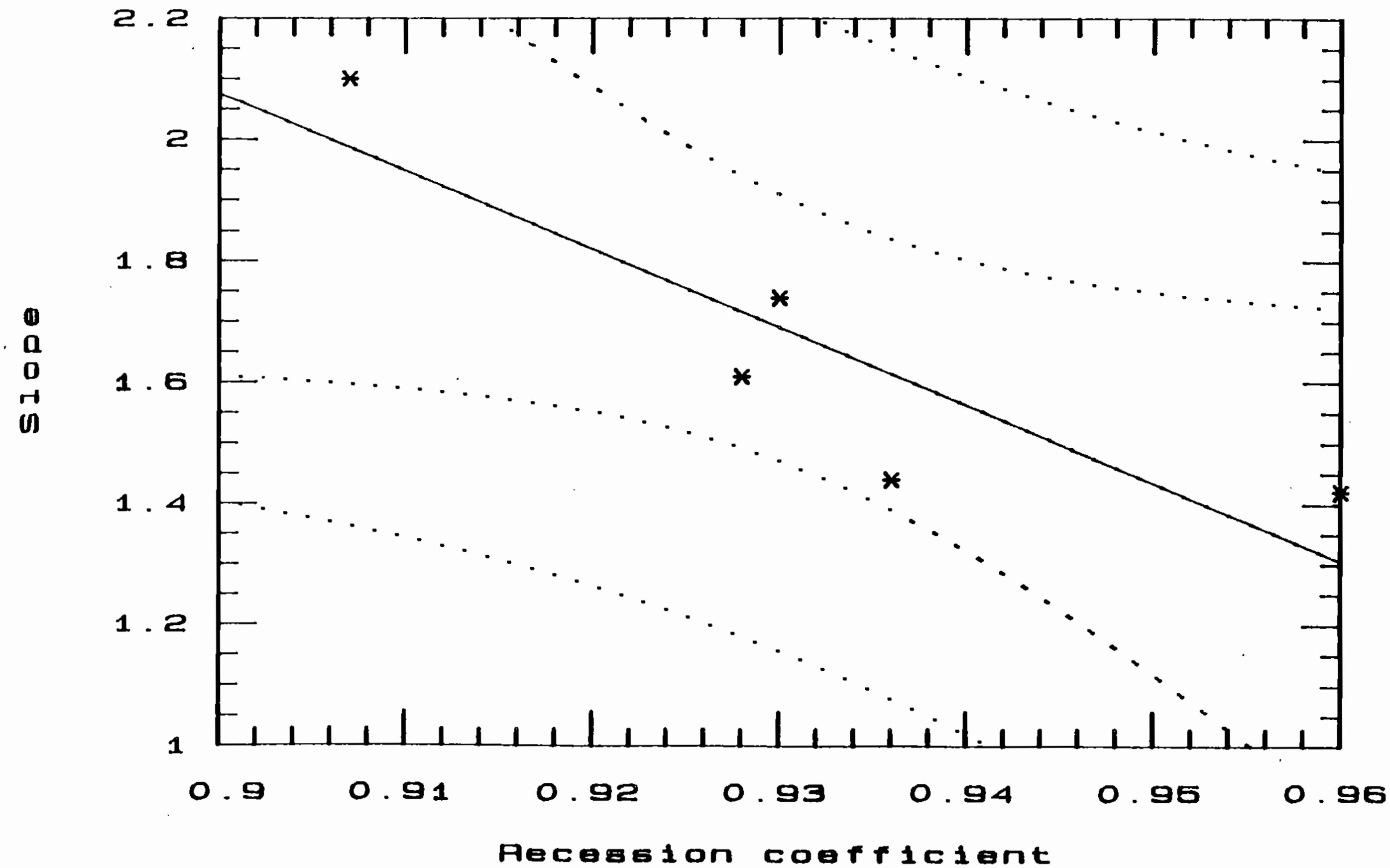


Figure 22. Relationship between recession coefficient (C) and slope (S) (r^2=0.733, S_y=0.15).

Intercept

The y intercept (I) of the line relating API to discharge, can be thought of as the average base flow prior to and following high flow events. Values of I from the five calibration watersheds ranged from 2.85 (Needle Branch Creek) to 3.86 (Flynn Creek). The range of variation was somewhat narrower for the normalized models (Table 5). Since all values of discharge (expressed in csm units) have been divided by watershed area, I is not expected to be related to watershed size. Variability in I might be explained by data gathering and model fitting procedures, as well as physical watershed characteristics that influence the water yield of a basin.

High flow events included in the data sets were chosen without regard to the base flow prior to the event. While peak discharge is correlated with streamflow prior to the peak (Jackson and Van Haveren, 1984), the volume and temporal distribution of large rainfall events may overcome dry antecedent conditions to produce a discharge of sufficient magnitude for inclusion in this study. Some variability in the y intercept among watersheds may be explained by the presence or absence of these events.

For all watersheds, it was noted that discharge on the recession limb of the hydrograph was typically higher than that on the rising limb for equal values of API. Since it generally takes a longer period of time for peak flows to fall than to rise, recession limb data are disproportionately represented when regression models were developed. Variability in recession limb duration among storms may account for additional variation in the intercept.

Since a least squares regression procedure was used to fit lines to observations of discharge and API, any factors that influence the slope of the line also influence the intercept, and vise versa. Physical characteristics of the watersheds that influence the amount of water the basin receives (eg. aspect, elevation, latitude) and/or influence the percent of rainfall that becomes runoff (eg. vegetation characteristics, soil depth, land slope, land use) would also contribute to the variability of I.

For the five calibration watersheds, the slope (S) of the linear relationship between discharge and API was used to predict the y intercept (I) ($P=0.020$; Figure 23):

$$I = 3.95 - 0.545 * S \tag{15}$$

Equation 15 was used to estimate the y intercept for the test watershed.

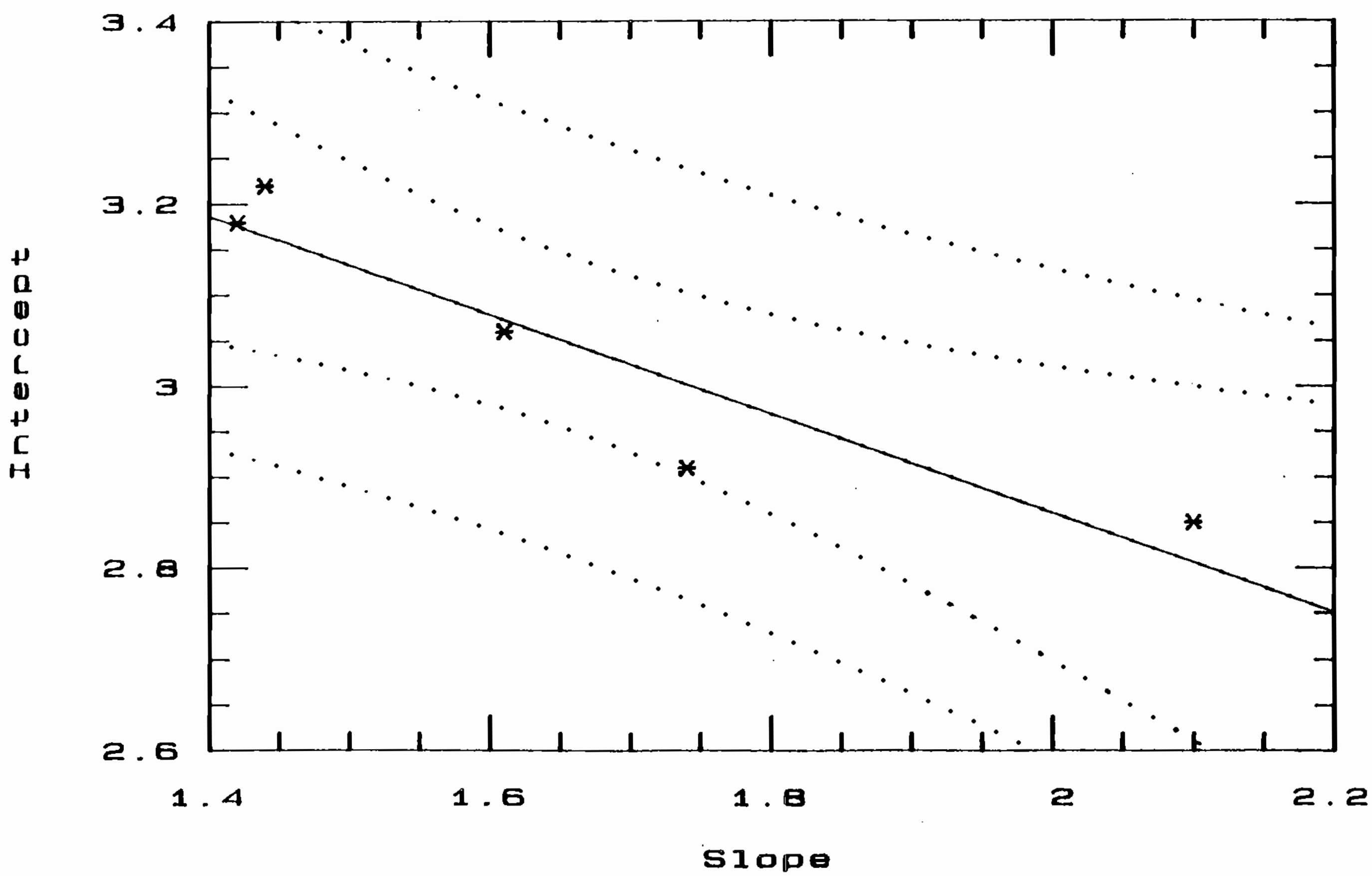


Figure 23. Relationship between slope (S) and intercept (I) (r^2=0.871, S_y=0.11).

Storm Runoff Simulation

To simulate storm runoff from an Oregon Coast Range watershed using an API method, three coefficients are necessary--recession coefficient, slope and intercept of the line relating API and discharge.

If streamflow data is available, standard procedures to derive a recession coefficient should be used (Garstka, et al., 1958). Equation 13 may be used if runoff data is not available. Values of API should be calculated (equation 10) beginning at least three days before runoff simulation begins. Simple linear regression can be used to fit a line to values of API and the square root of discharge; or, equations 14 and 15 can be used to estimate values of S and I if streamflow data is not available. Storm runoff is estimated by equation 16:

$$Q_t = (I + S * API_t)^2 \tag{16}$$

Q_t = Discharge at time t (csm)
I = Intercept of the line relating API and the square root of discharge
S = Slope of the line relating API and the square root of discharge
API_t = Antecedent precipitation index at time t (inches)

Testing the API Method

The API method was tested by comparing observed and predicted values of storm runoff volume, peak discharge, and the timing of the peak discharge. The shape of the storm hydrographs were compared visually. Regression equations developed for each comparison of observed and predicted storm runoff volume and peak discharge were evaluated by examining:

(1) Degree of linear association between the observations, measured by the coefficient of determination (r^2).
(2) Errors about the estimate, measured by the standard error of the estimate (S_y) and average percent error (Green and Stephenson, 1986).
(3) Bias of the predictions, measured by the confidence in the estimate of the regression intercept and slope with respect to a 1:1 line of perfect fit passing through the origin.

Timing of the peak discharge was evaluated by examining a plot of departure from the observed peak (predicted time - observed time) versus the cumulative frequency of those observations. These tests were conducted separately for both the calibration watersheds and the independent test watershed. For the test watershed, a sensitivity analysis was also undertaken.

Calibration Watersheds

For the five calibration watersheds, rainfall-runoff events used to derive the API-discharge models were also used to test the models. The tests that follow provide an indication of how well the API method can work--given the quality of the data used. The "original" fitted models were used throughout the tests (Table 5). Results from the entire data set are presented as well as those from individual watersheds within the calibration data set. A discussion of possible sources of errors follows.

A plot of observed and predicted peak flows for the 61 events in the calibration watershed data set is shown in Figure 24. Predicted peak flows explain 78.0 percent of the variability in observed peaks. The slope and intercept of the regression equation are not significantly different than a 1:1 line passing through the origin at the 95 percent confidence level. Errors in peak flow estimates averaged 14.8 percent, with 75 percent of the estimates falling within 20 percent of the observed values (Figure 25).

Storm runoff volume estimates were closely correlated with observed values (Figure 26). However, both the slope and intercept of the regression equation were significantly different than a 1:1 line at the 95% level. Predictions of low volume storms were often over-estimated, while predictions of high volume storms were consistently under-estimated. Errors in storm runoff volume estimates averaged 14.2 percent with 82 percent of predicted volumes within 20 percent of observed volumes

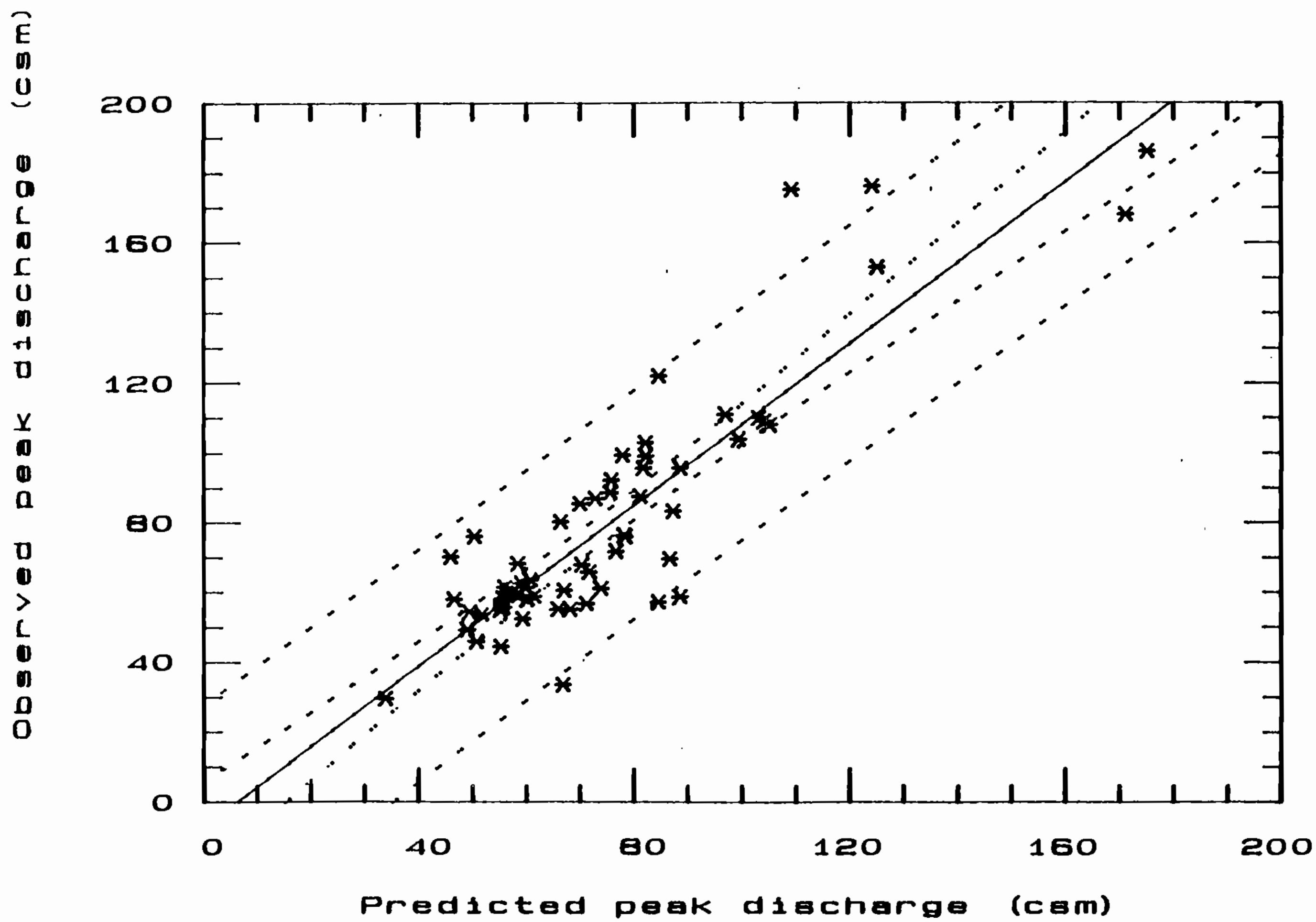


Figure 24. Observed and predicted peak flows with 95 percent confidence intervals for significance of regression and prediction limits (r^2=0.780, S_y=16.3 csm). Predicted values from calibration watersheds (n=61).

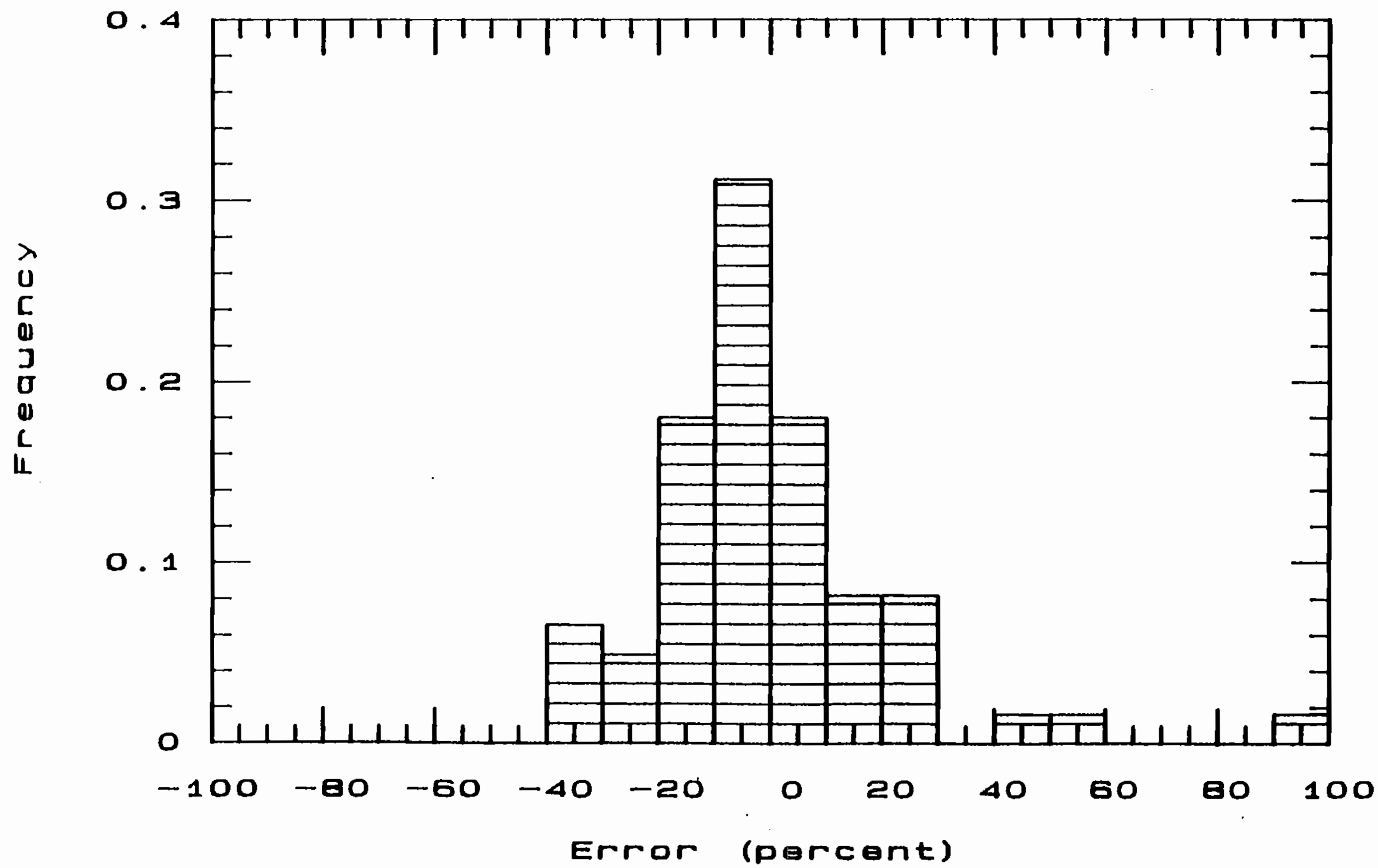


Figure 25. Relative frequency and distribution of errors in peak flow estimates. Predicted values from calibration watersheds (n=61).

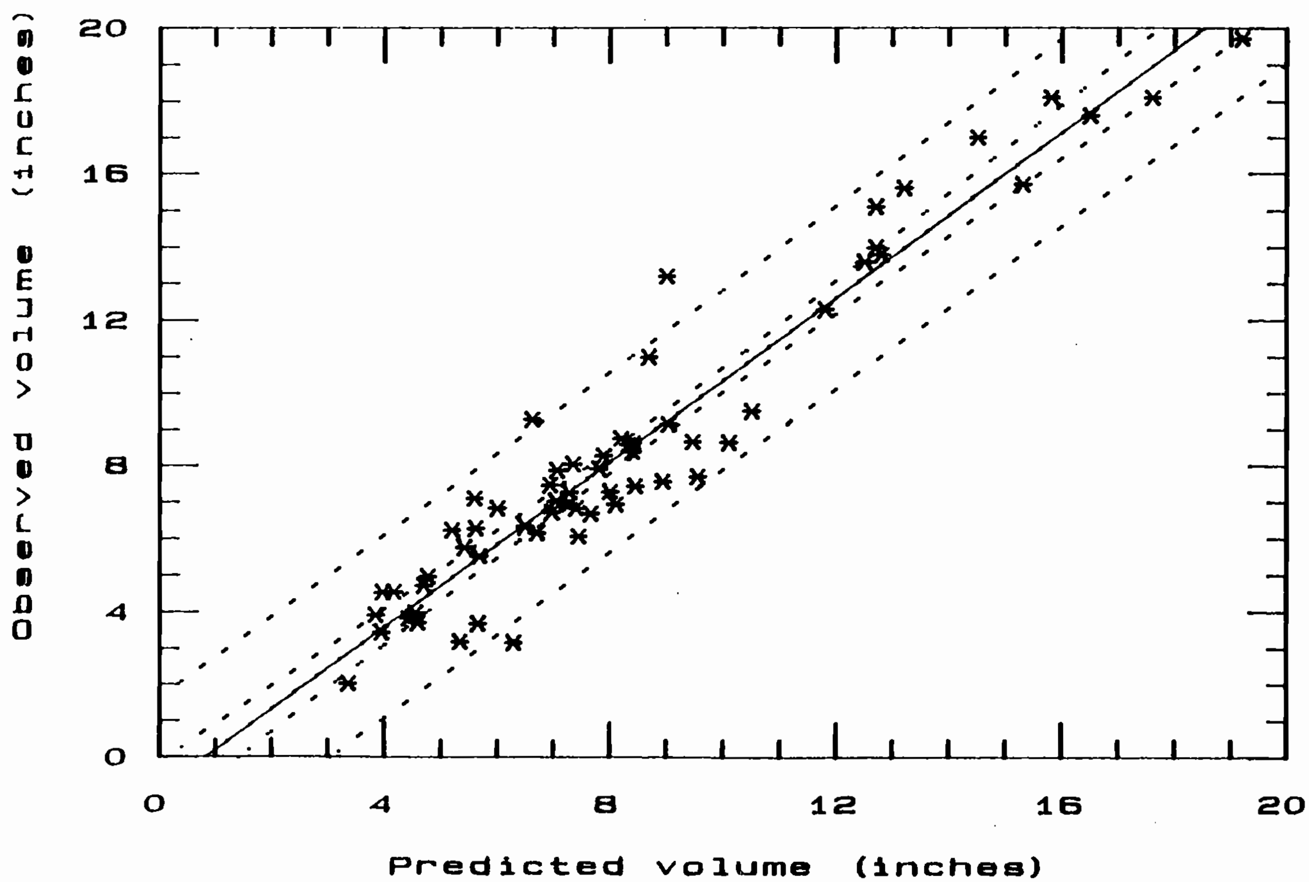


Figure 26. Observed and predicted storm runoff volume with 95 percent confidence intervals for significance of regression and prediction limits (r^2=0.920, S_y=1.22 inches). Predicted values from calibration watersheds (n=61).

(Figure 27). A statistical summary of peak flow and storm runoff results is presented in Table 6.

Sixty-six percent of the predicted peak flows fall within two hours of the observed peaks (Figure 28). Fifty-one percent of the 61 peak flows are predicted to occur before the observed peak, while 16 percent are predicted to occur after the observed peak. The remaining 33 percent of the peak flows were predicted to occur at the same time the observed peak flows occurred. The average error in the timing of the peak (predicted - observed) is -1.80 hours; which is significantly different than zero at the 95% level (Table 7).

Observed and predicted peak flows, storm runoff volume and timing of peak flow estimates were also examined for each calibration watershed individually. Average errors in the estimates of peak flows for individual watersheds ranged from 10.4 to 30.4 percent (N. Fork Siuslaw and N. Yamhill rivers, respectively). The slope of the line relating observed and predicted storm runoff volume for Needle Branch Creek (slope=1.15) was the only regression estimate that differed from a 1:1 line at the 95% confidence level (Table 6). Peak flows were generally predicted to occur before the observed peak, and the average deviation tended to increase with increasing watershed size (Table 7).

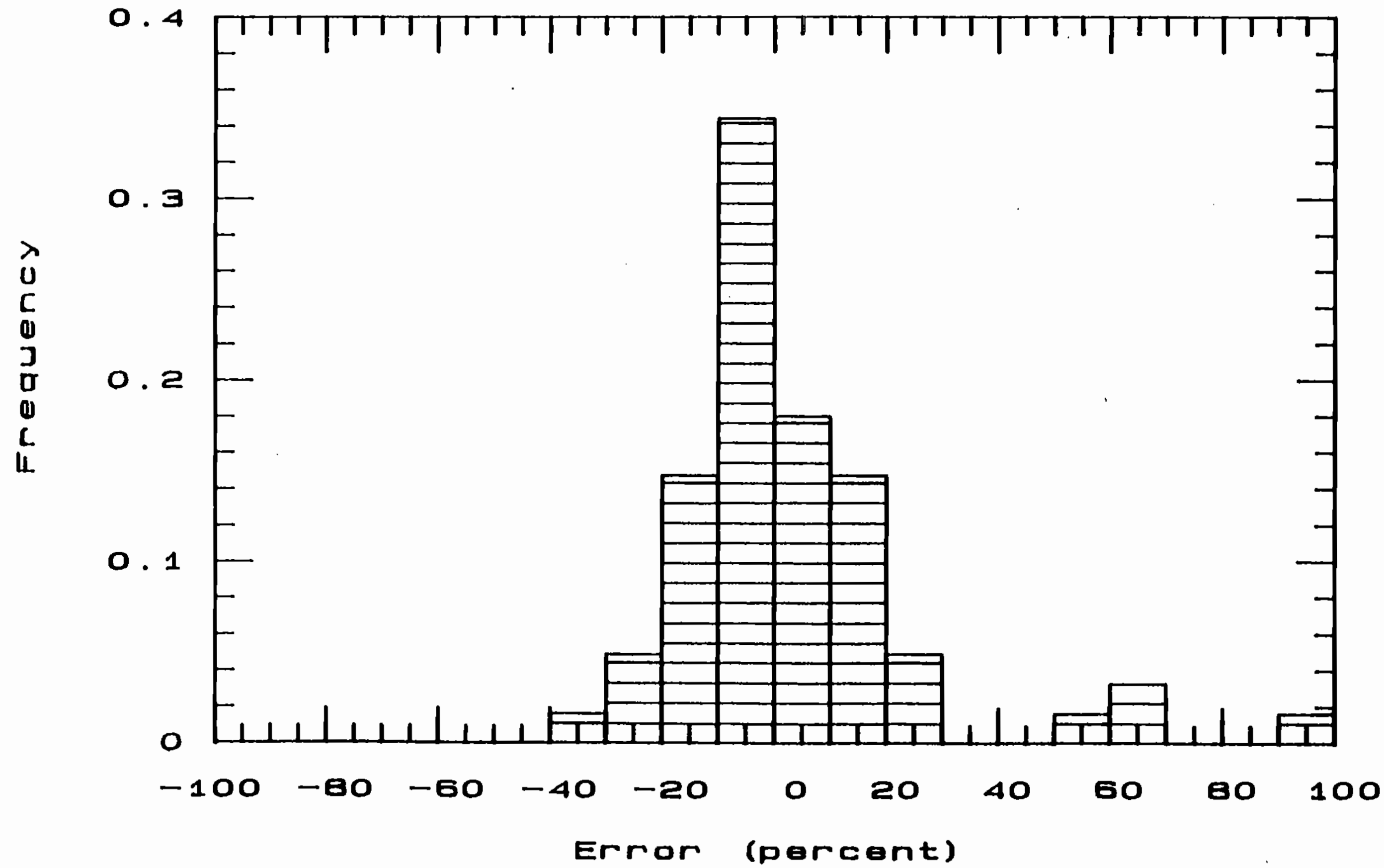


Figure 27. Relative frequency and distribution of errors in storm runoff volume estimates. Predicted values from calibration watersheds (n=61).

Table 6. Summary statistics for regression equations fitted to observed and predicted (API method) peak discharges and storm runoff volumes; calibration watersheds.

Watershed	n	Slope, S	Intercept, I	r^2	S_y	Ave. Error (percent)
PEAK DISCHARGE						
					(csm)	
Needle Branch	9	1.09	-1.09	0.875	16.0	12.9
Flynn	20	1.26	-0.78	0.722	17.8	16.4
Deer	19	0.94	-0.46	0.858	10.6	10.7
N. Yamhill	6	1.91	-3.10	0.809	26.6	30.4
N. Fk. Siuslaw	7	1.22	-2.15	0.953	6.32	10.4
All	61	1.15	-7.14	0.780	16.3	14.8
STORM RUNOFF VOLUME						
					(inches)	
Needle Branch	9	1.15*	-2.4	0.981	0.86	16.1
Flynn	20	1.10	-12.3	0.888	1.31	14.7
Deer	19	1.08	10.3	0.932	1.16	11.4
N. Yamhill	6	1.45	-59.9	0.601	2.17	15.8
N. Fk. Siuslaw	7	1.26	-19.2	0.965	1.05	16.5
All	61	1.13*	-0.9*	0.920	1.22	14.2

* Significantly different from a 1:1 line passing through the origin at the 95% confidence level.

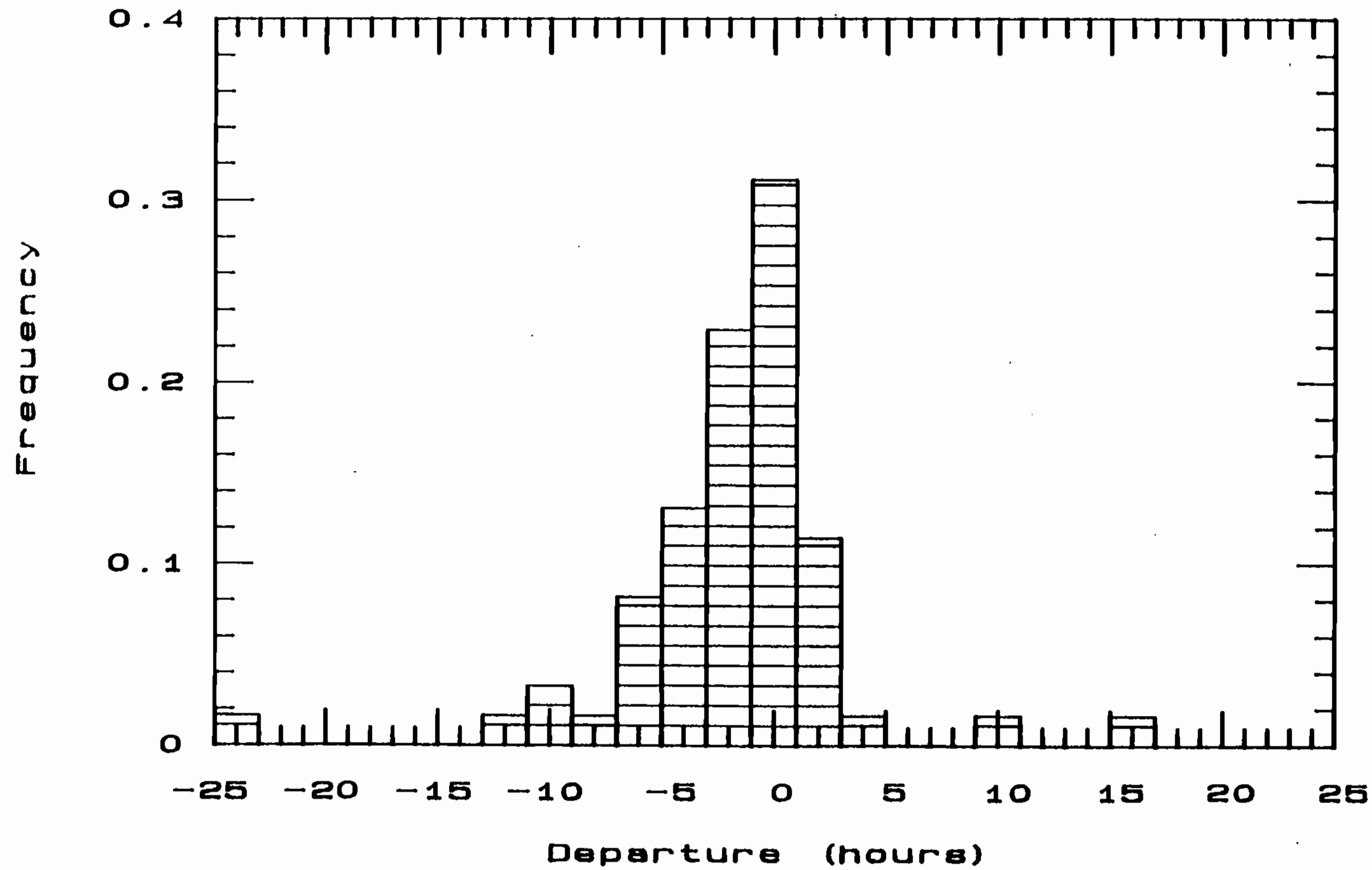


Figure 28. Relative frequency and departure of the timing (predicted-observed) of predicted peak flows from calibration watersheds (n=61).

Table 7. Average errors in the timing of peak flows (predicted-observed) for calibration watersheds.

Watershed	Ave. Departure (hours)	n
Needle Branch Creek	0.0	9
Flynn Creek	-1.0	20
Deer Creek	-0.2	19
N. Yamhill River	-7.3	6
N. Fk. Siuslaw River	-6.0*	7
All	-1.8**	61

* Significantly different than zero at the 99% confidence level.

**Significantly different than zero at the 95% confidence level.

Two examples of hyetographs and corresponding observed and simulated hydrographs are shown in Figures 29 and 30. The examples portray two typical features of API simulated hydrographs: over-prediction of streamflow early in the event, and under-prediction late in the event. For most rainfall/runoff events, a plot of API and the square root of discharge would reveal a hysteresis loop. For any specified value of API, two values of discharge would be observed; the lower value on the rising limb of the hydrograph, and the higher value on the falling limb of the hydrograph. Figure 31 illustrates the hysteresis loop using data from the event depicted in Figure 29. Figures 29 and 30 also show the capability of the API method to accurately simulate the shape of storm hydrographs resulting from simple and complex rainfall patterns. Seventeen additional examples of observed and simulated hydrographs from Flynn Creek are presented in Appendix A.

Test Watershed

Eight rainfall-runoff events occurring on the Nestucca River watershed were included in an independent test of the API method. Observed and predicted peak discharge, storm runoff volume and timing of peak flows were evaluated in the same manner as the calibration watersheds. A sensitivity analysis was conducted by manipulating the recession coefficient (plus and minus one

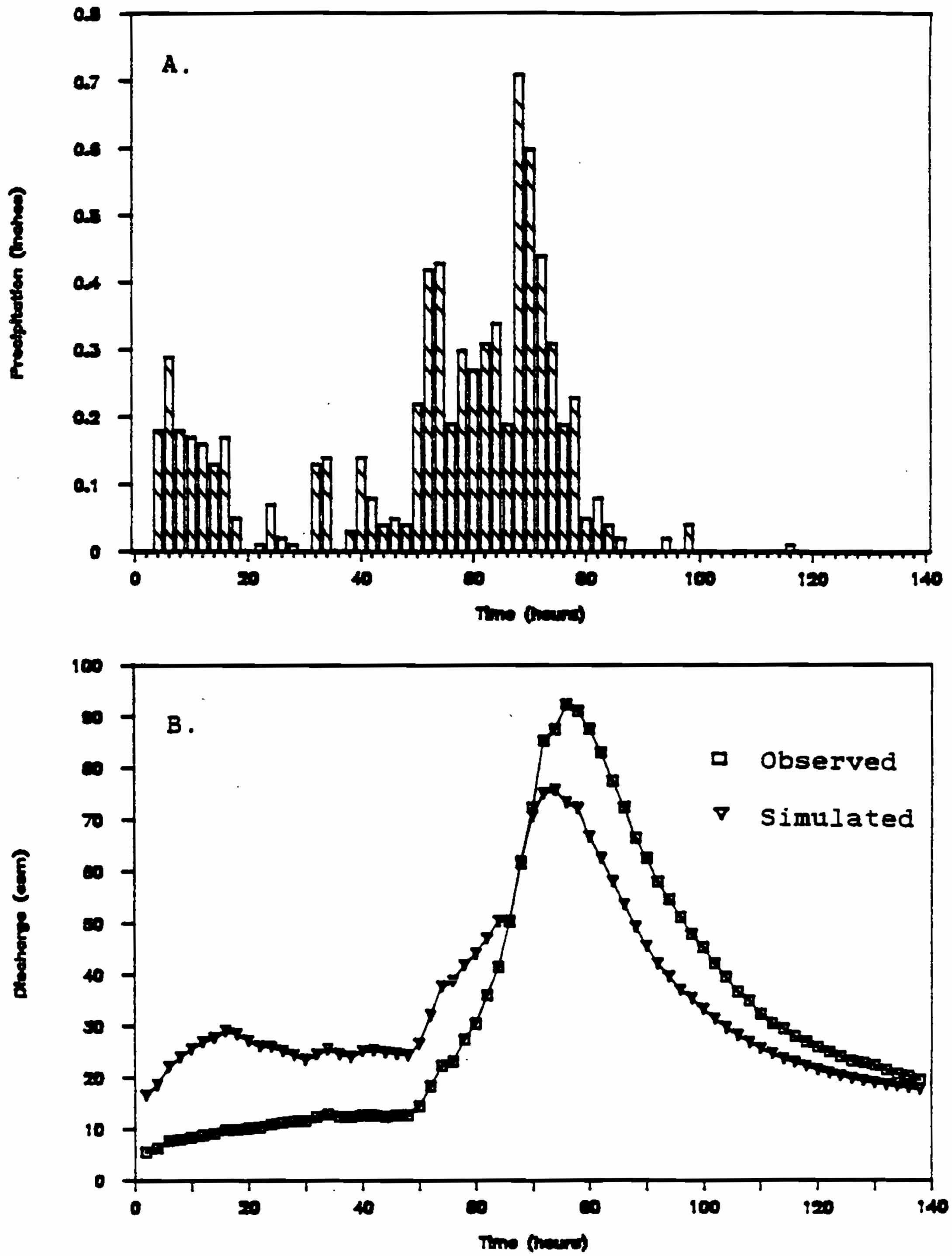


Figure 29. (A) Hyetograph, and (B) simulated and observed hydrographs for a "simple" rainfall event on Flynn Creek, Oregon Coast Range (March 6-12, 1966). The API method tends to over-estimate rising limb runoff and under-estimate falling limb runoff.

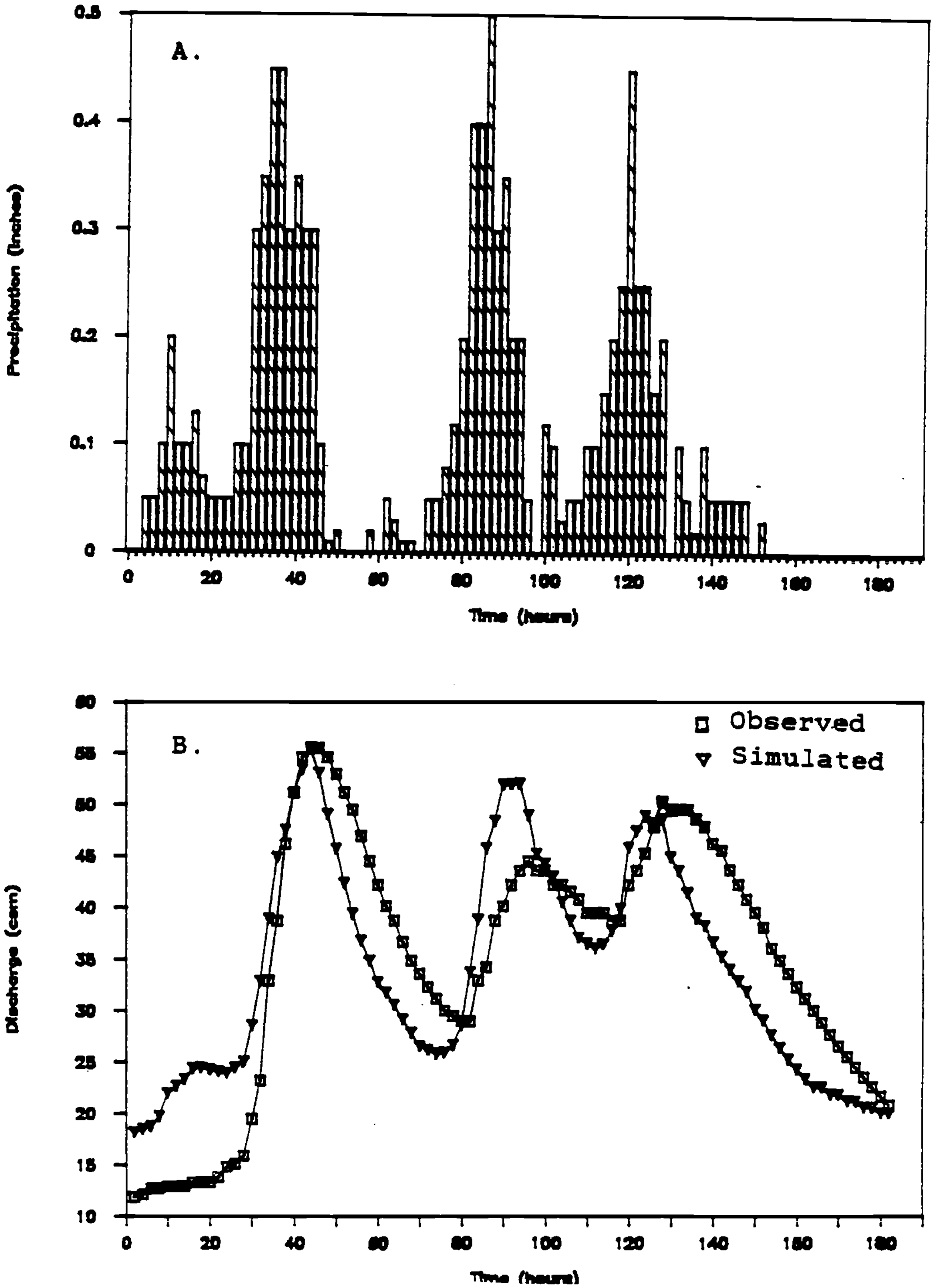


Figure 30. (A) Hyetograph, and (B) simulated and observed hydrographs for a "complex" rainfall event on Flynn Creek, Oregon Coast Range (January 5-13, 1969). The API method closely simulates the shape of a hydrograph resulting from a complex rainfall event.

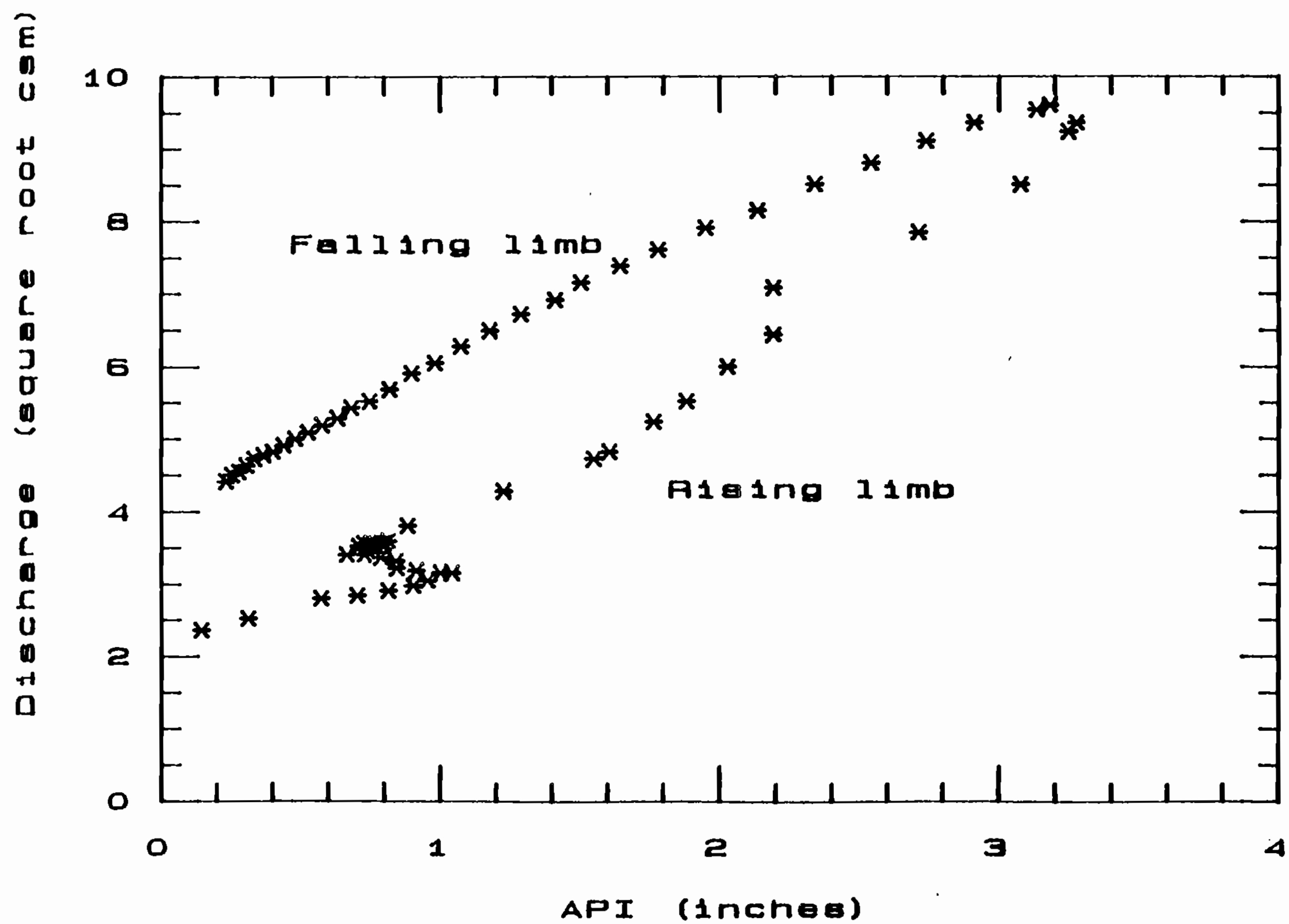


Figure 31. Hysteresis loop resulting from the rainfall-runoff event March 6-12, 1966, Flynn Creek, Oregon Coast Range.

standard error of the estimate) and examining the observed and predicted hydrograph properties again. Two examples of observed and simulated hydrographs and their respective hyetographs are presented.

The Nestucca River watershed is 6.18 square miles in size. Using equations 13, 14, and 15, C, S, and I were estimated as 0.914, 1.89, and 2.92, respectively. Values of API were calculated (equation 8) for each rainfall event beginning 72 hours before the runoff event was defined to begin. Equation 17 was used to calculate discharge:

$$Q_t = (2.92 + 1.89 * API_t)^2 \tag{17}$$

Q_t = Discharge at time t (csm)
API_t = Antecedent precipitation index at time t (inches)

Two examples of observed and simulated storm hydrographs and their corresponding hyetographs are presented in Figures 32 and 33. For the relatively simple rainfall-runoff event (Figure 32), the peak discharge, timing of the peak, storm runoff volume, and hydrograph shape appear to be well simulated. However, rising limb runoff is over-predicted. For the relatively complex rainfall-runoff event (Figure 33), the peak and shape of the hydrograph appear well simulated, while the volume is

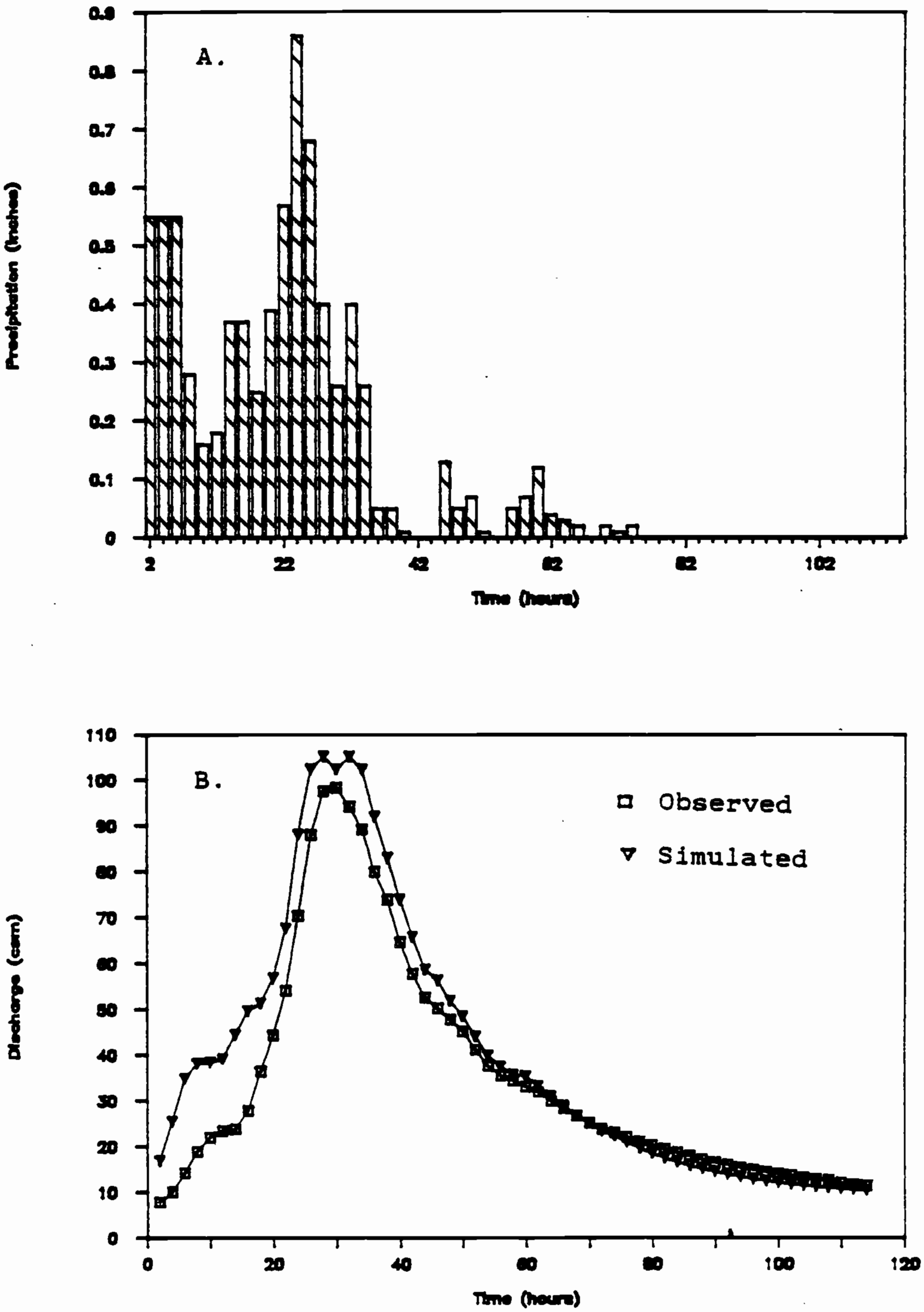
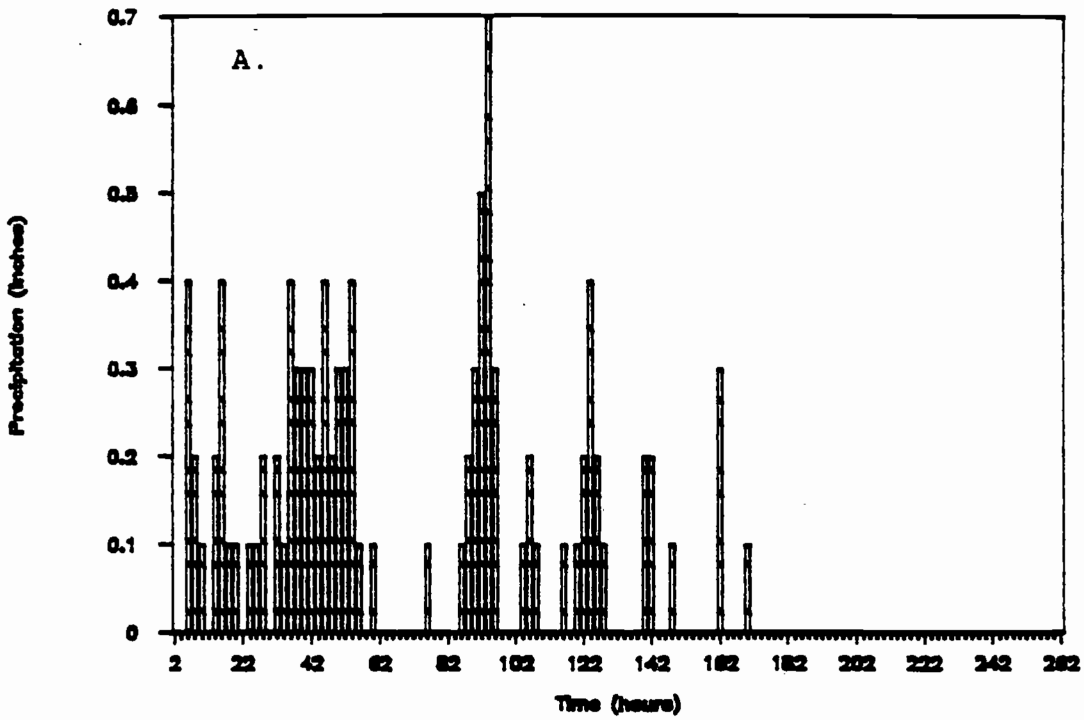


Figure 32. (A) Hyetograph, and (B) simulated and observed hydrographs for a "simple" rainfall event on the Nestucca River, Oregon Coast Range (November 21-29, 1962).



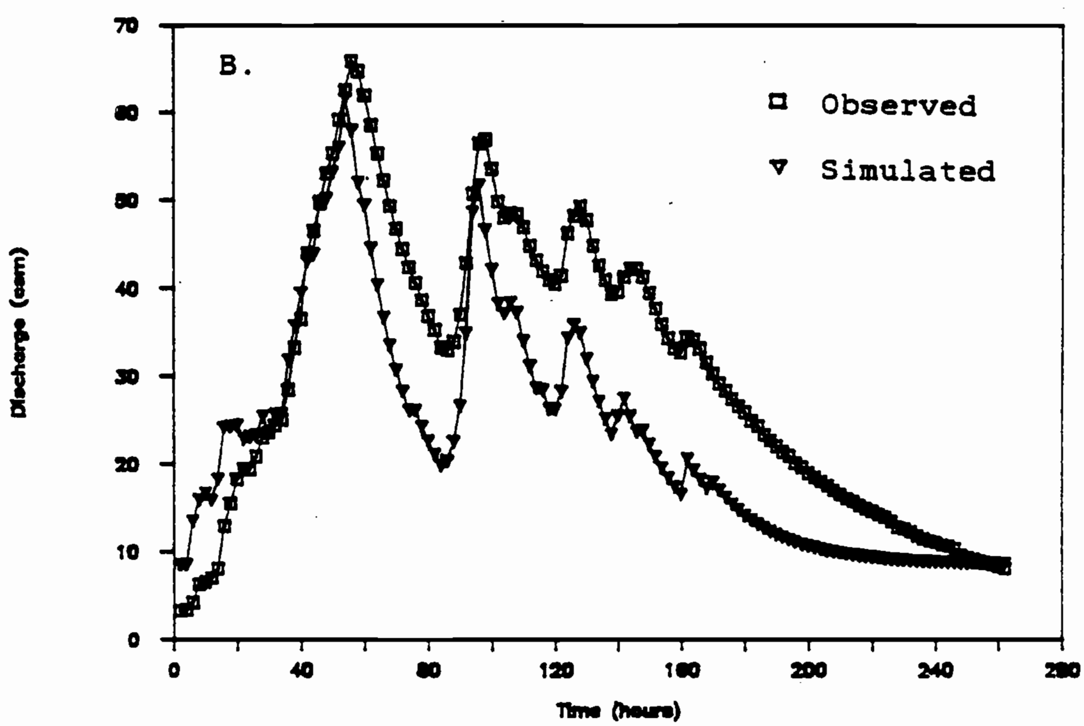


Figure 33. (A) Hyetograph, and (B) simulated and observed hydrographs for a "complex" rainfall event on the Nestucca River, Oregon Coast Range (February 14-28, 1968).

under-estimated. Early storm runoff is also over-estimated in this example.

A plot of observed and predicted peak discharge for the eight test storms is shown in Figure 34. Observed and predicted values of peak discharge were not as highly correlated (r^2=0.580) as those from the calibration watersheds which averaged 0.843 (Table 6). The regression estimates of slope and intercept did not differ from those of a 1:1 line passing through the origin at the 95 percent confidence level. Errors in the estimates of peak flows averaged 17.8 percent, and the standard error of the estimate was 17.9 csm; both values are slightly higher than the values for the calibration watersheds (Table 6). Sixty-three percent of the predicted peaks fall within 20 percent of the observed values (Figure 35).

Observed and predicted values of storm runoff volume are highly correlated (Figure 36). A 1:1 line falls within the errors of the regression estimates. Errors in storm runoff volume averaged 20.8 percent, with 63 percent of the estimates falling within 20 percent of the observed values (Figure 37).

Sixty-three percent of predicted peak flows fall within two hours of the observed peaks (Figure 38). Sixty-three percent are also predicted to occur before the observed peak, while 25 percent are predicted to occur after the observed peak. On the average, peak flows are

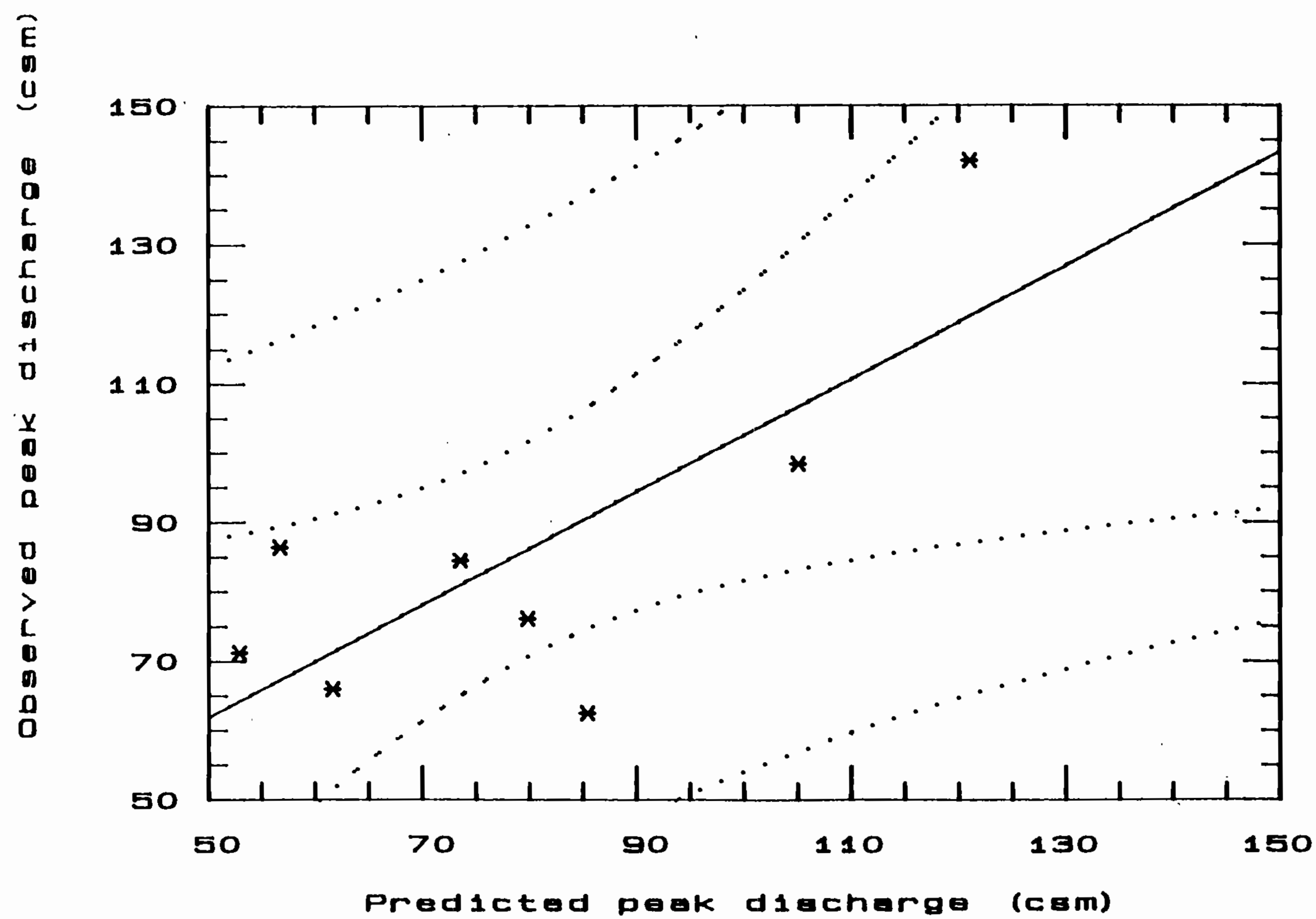


Figure 34. Observed and predicted peak flows with 95 percent confidence intervals for significance of regression and prediction limits (r^2=0.580, S_y=17.9 csm). Predicted values from the Nestucca watershed (n=8).

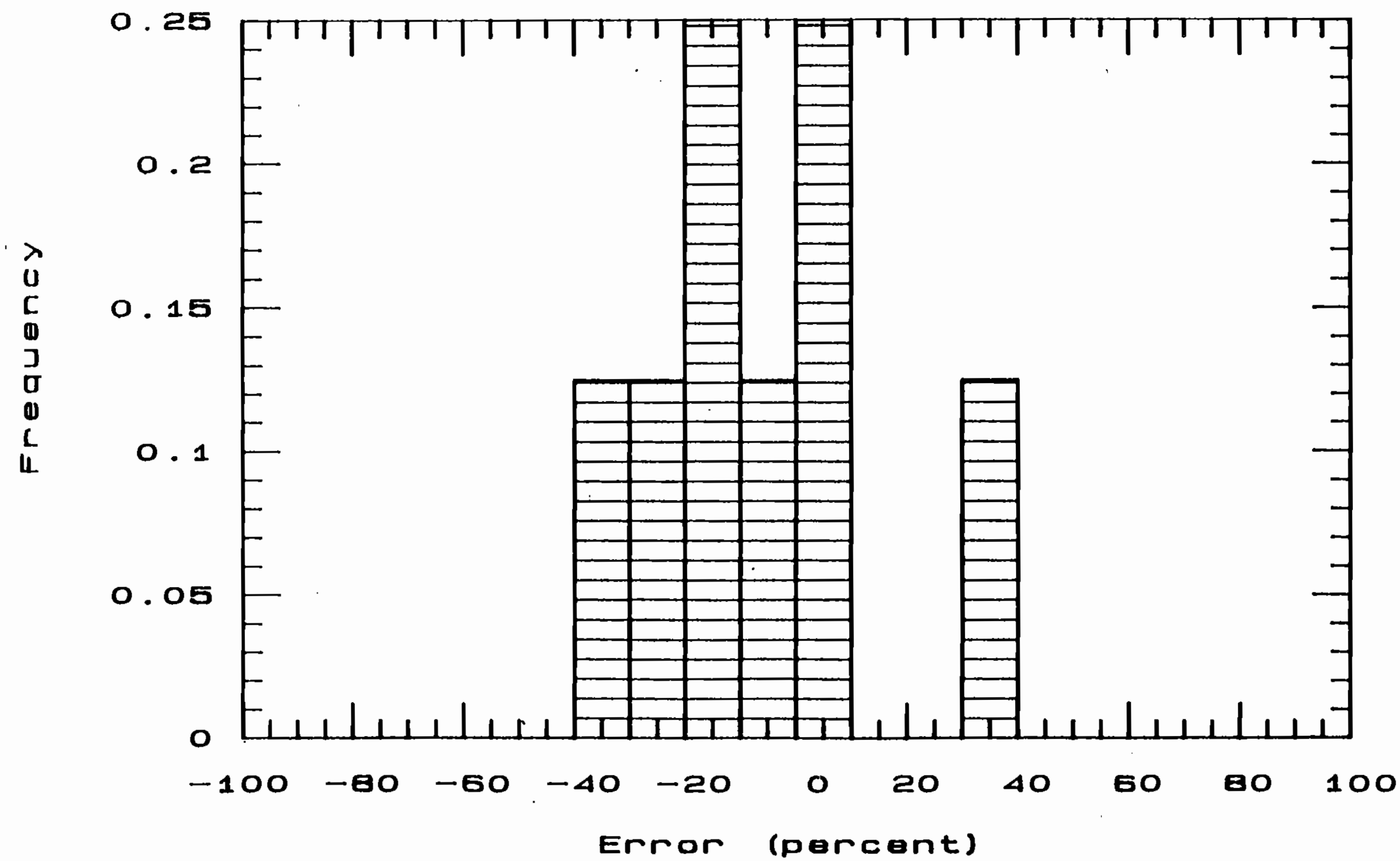


Figure 35. Relative frequency and distribution of errors in peak flow estimates. Predicted values from the Nestucca watershed (n=8).

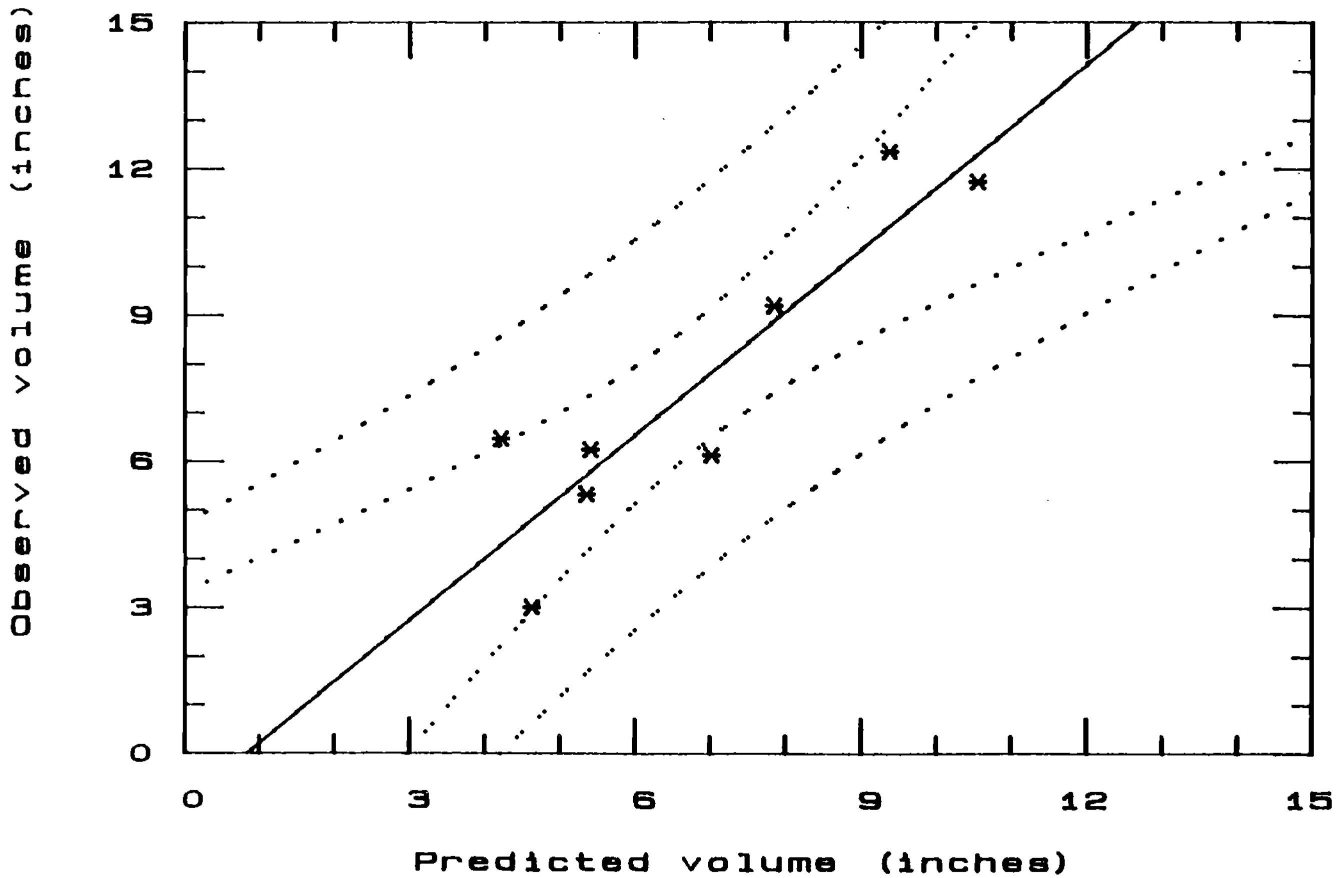


Figure 36. Observed and predicted storm runoff volume with 95 percent confidence intervals for significance of regression and prediction limits (r^2=0.801, S_y=1.53 inches). Predicted values from the Nestucca Watershed (n=8).

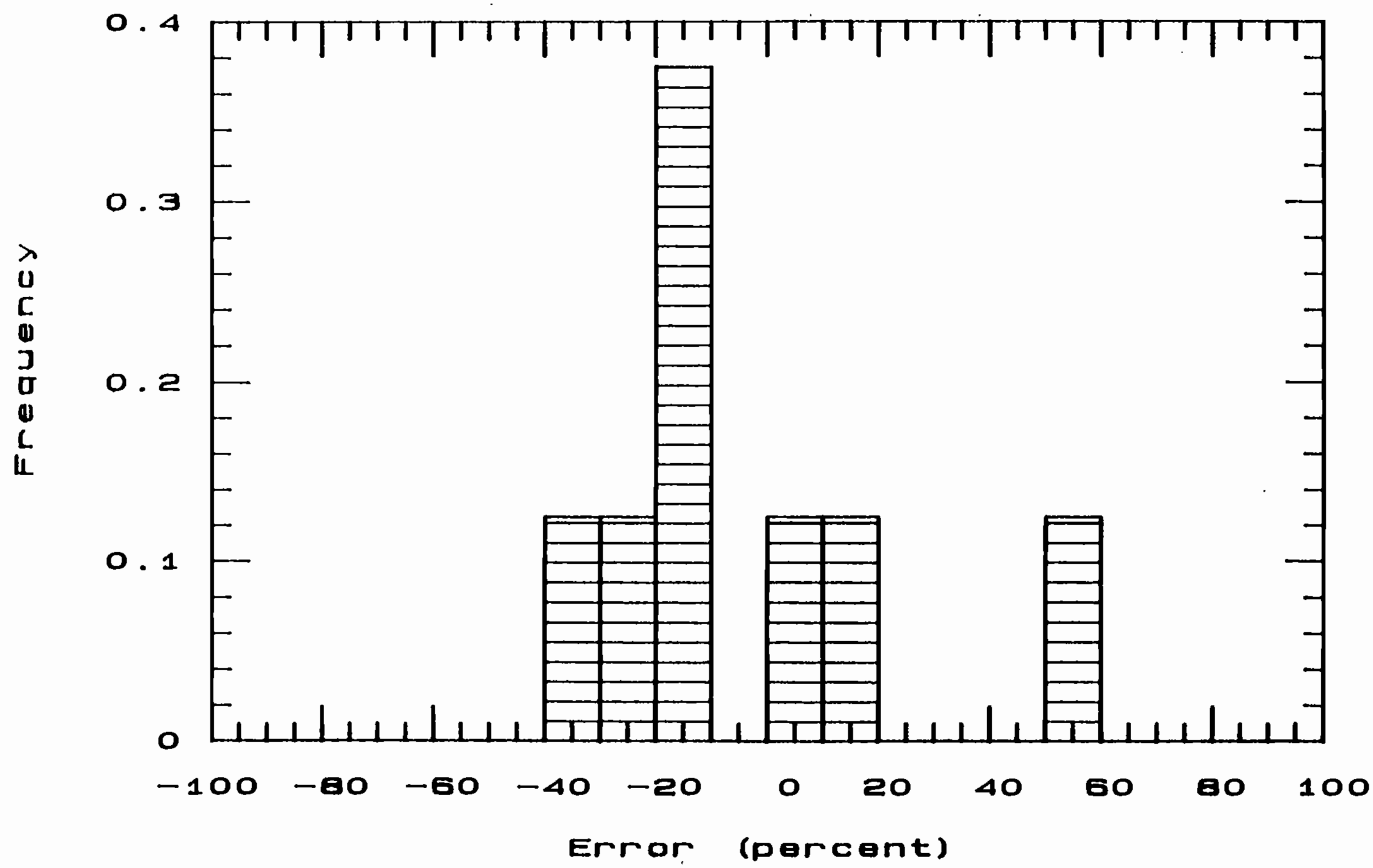


Figure 37. Relative frequency and distribution of errors in storm runoff volume estimates. Predicted values from the Nestucca watershed (n=8).

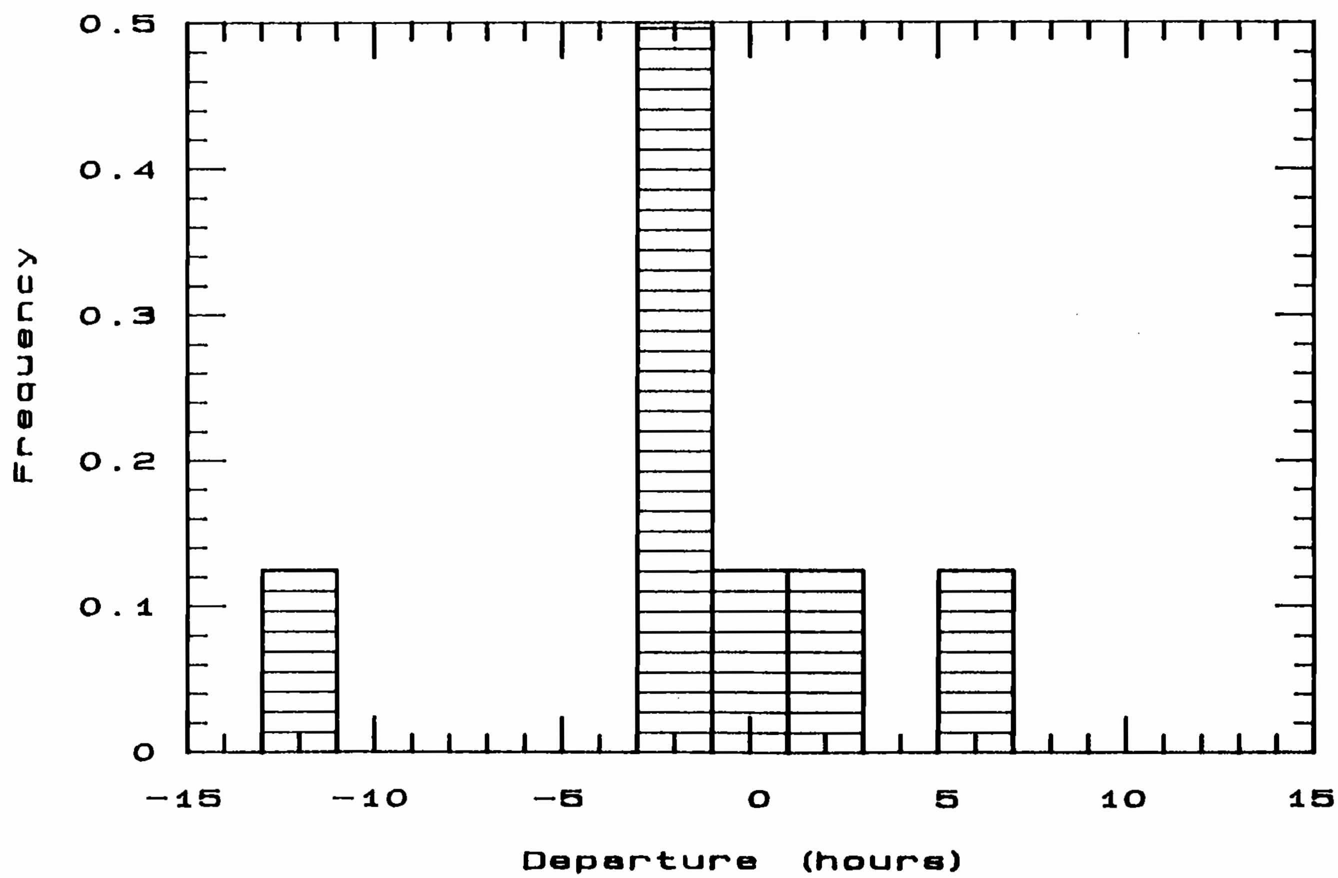


Figure 38. Relative frequency and departure of the timing (predicted-observed) of predicted peak flows from Nestucca watershed (n=8).

predicted to occur 1.5 hours before the observed peaks, which is not statistically significantly different from zero at the 95 percent confidence level.

A second analysis was conducted to determine the sensitivity of the results to changes in the recession coefficient. The estimated recession coefficient (c=0.914) was adjusted upward and downward by one standard error of the estimated value of C (S_y=0.012). New values of S and I were calculated for each value of C using equations 14 and 15. This analysis revealed that the API model is relatively insensitive to changes in C because of compensating changes in values of S and I. Results are summarized in Table 8.

Sources of Error

Errors in predicted streamflow characteristics and simulated hydrographs arise from errors within the API methodology as well as errors common of all streamflow modeling techniques. Some of the errors in observed and predicted peak flows and volumes, may be attributed to timing differences between peak discharge and peak API values. Errors in the timing of peak flows increased with watershed size. For large watersheds, peak values of API were paired with rising limb values of discharge,

Table 8. Summary statistics for regression equations fitted to observed and predicted (API method) peak discharges and storm runoff volumes, Nestucca watershed. Sensitivity analysis conducted by adjusting values of C (+ and - 1 S_y), and re-calculating S and I (n = 8).

Recession coefficient, C	Slope, S	Intercept, I	r^2	S_y	Ave. Error (percent)
PEAK DISCHARGE					
				(csm)	
0.902	0.764	25.3	0.566	18.2	21.7
0.914	0.815	21.1	0.580	17.9	17.8
0.926	0.864	17.0	0.596	17.5	17.0
STORM RUNOFF VOLUME					
				(inches)	
0.902	1.35	-1.08	0.801	1.57	22.9
0.914	1.26	-1.05	0.801	1.53	20.8
0.926	1.17	-0.98	0.818	1.50	20.0

while peak discharge values were paired with decaying API values. Therefore, when models were fit to the data, the effects of peak API values were under-estimated, resulting in a built-in bias in the model.

A cross-correlation between discharge and API was conducted using data from the N. Fork of the Siuslaw River. Results indicated that API at any time (t) was most strongly correlated with discharge occurring four hours later (t + 4 hours). If the timing differences between discharge and API was a large component of the errors observed in the API methodology, one would expect increasing bias in peak flow estimates with increasing watershed size. A simple sign test (Aitken, 1973) did not detect any bias in peak flow or storm runoff volume estimates (90 percent confidence level) for any of the calibration watersheds.

Antecedent conditions prior to an event were quantified by calculating values of API beginning three days before the runoff event was defined to begin. This technique does not appear to be an adequate measure of long-term antecedent conditions. Since models were fit to data without regard to base flow prior to the event, the models represent an average response of streamflow to precipitation. Rainfall events occurring on soils with a large moisture deficit, will not cause the same response in stream discharge as a similar rainfall event later in the winter season when moisture deficits have been satisfied. Incorporating some measure of seasonal changes in antecedent conditions might help reduce the variability in peak flow estimates by the API method.

The hysteresis loop remains largely unexplained. The API method does not attempt to quantify any components of the hydrologic cycle occurring throughout a rainfall event--interception, detention and retention storage, evapotranspiration, and the increasing efficiency of water movement through soil throughout an event--are all ignored. Any of these processes may account for some of the hysteresis effect.

Results produced from hydrologic models can only be as accurate as the data used for inputs and calibration. Variability in the accuracy of streamflow and precipitation data used in this study may account for additional variability in the results obtained. Streamflow data used in this study was classified as "good" by the USDI Geological Survey (1962-1984). "Good" is defined as "about 95 percent of the daily discharges is within 10 percent." Errors are likely to be greater during high flow events, and no mention is made regarding the accuracy of the reported instantaneous peak flows. The majority of the data used in this study was removed from original stage strip-charts. Temporary shifts in the rating curves may not have been apparent.

Information was not available to determine if runoff events included in this study were influenced by snowmelt during rainfall. All watersheds used in this study are at least partially within the transient snow zone (1100-3600 feet) identified by Harr (1986). A hand-written comment on an original gage-height record from the Nestucca watershed indicated that 1.5 feet of snow was present at the gaging station preceding one of the runoff events included in this study. Antecedent snow conditions would certainly introduce additional variability in peak flows, timing of peaks, and storm runoff volume.

Precipitation gages were as far as five miles from the center of the watershed and were assumed to be accurate and representative of the entire basin. Larson and Peck (1974) report that gage catch deficiencies average 20 percent with wind speeds of 20 miles/hour. Biases in runoff prediction can also result as a function of spatial variability of precipitation (Troutman, 1983). The magnitude of precipitation variability over the basins used in this study is unknown, although it is expected to increase with increasing watershed size.

Errors associated with distance and elevation differences between a rain gage and a watershed are typically unknown. However, the magnitude and direction of these errors were examined using streamflow and precipitation data from Deer Creek, precipitation data from four other gages, and the equation derived for Deer Creek to predict storm runoff. Since the Deer Creek precipitation gage was used to derive the streamflow prediction equation for Deer Creek, changes in elevation were measured from the Deer Creek precipitation gage (rather than the mean watershed elevation). For gages within 19 miles of the Deer Creek watershed, the change in elevation separating the precipitation gages was the best indicator of errors in predicted peak flows and storm runoff volumes (Figures 39 and 40, respectively).

For the Nestucca watershed, the precipitation gage was located 1200 feet below the mean watershed elevation. Using Figure 39 (and extrapolating beyond the range of the data), one might expect peak discharge errors to average over 100 percent. Errors of this magnitude were not observed, and may be partially explained by the elevation change between calibration watersheds and their respective precipitation gages. For calibration watersheds, storm runoff was correlated with precipitation gage catch at elevations from 335 to 1150 feet below the watersheds. Therefore, errors in the estimates of peak discharge for the Nestucca watershed are expected to be of the same magnitude as the calibration watersheds.

Hourly or bi-hourly precipitation data may not be available to all users of a rainfall-runoff model. To assess the possible errors in peak discharge and storm

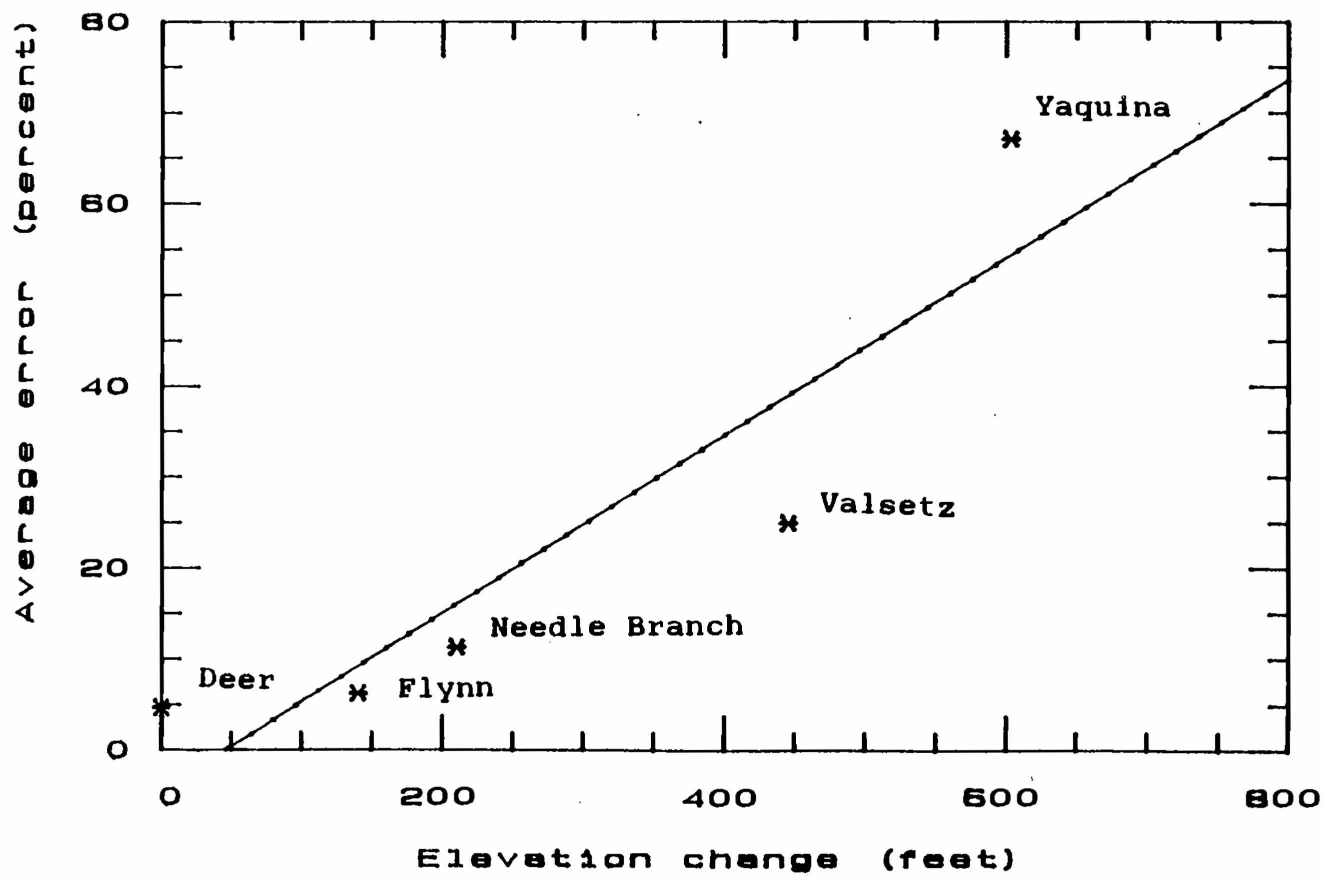


Figure 39. Average error in peak flow estimates with change in precipitation gage elevation from the Deer Creek precipitation gage.

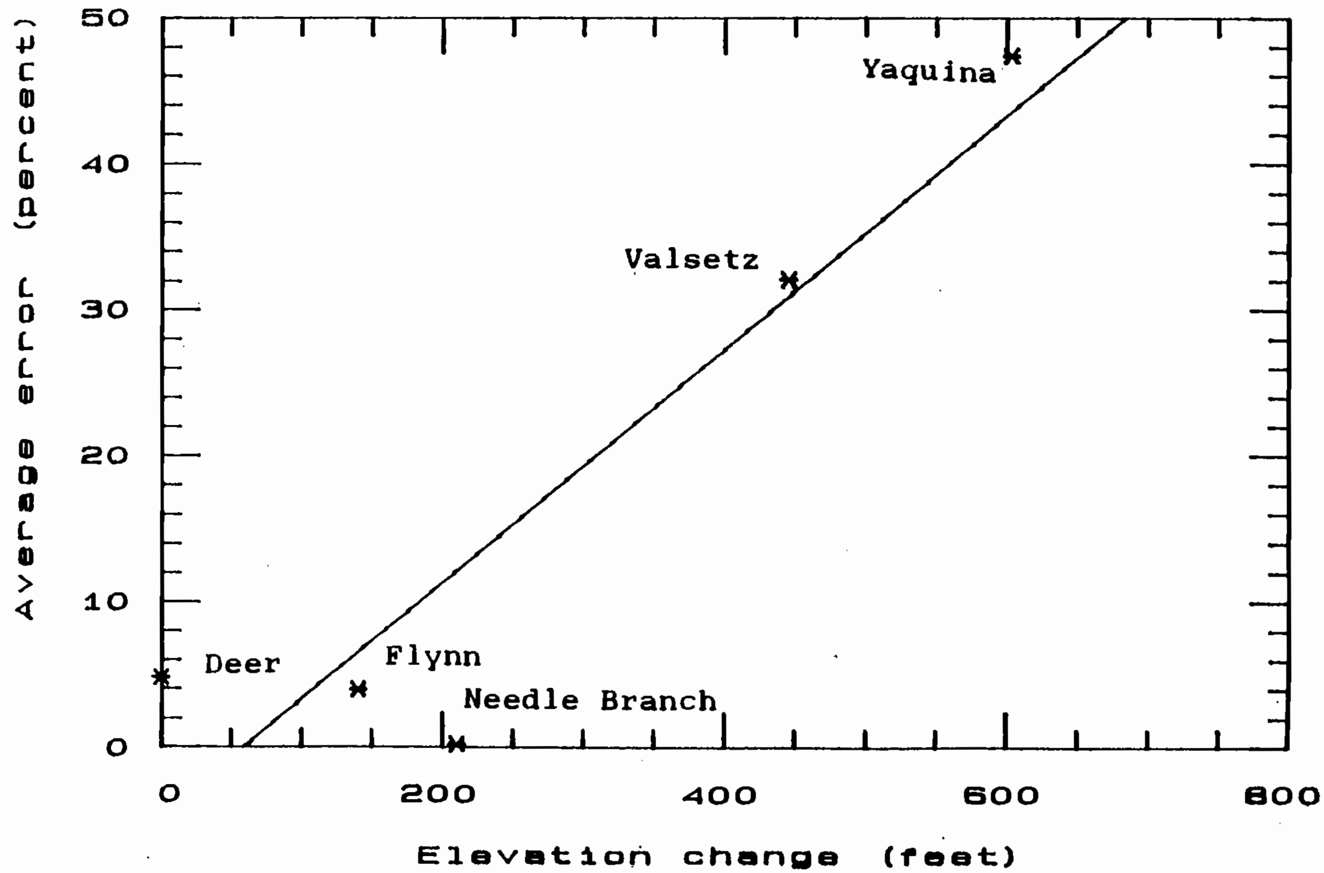


Figure 40. Average error in storm runoff volume estimates with change in precipitation gage elevation from the Deer Creek precipitation gage.

runoff volume estimates associated with the time interval (Δt) of precipitation observations, precipitation data from Deer Creek was summed into 4-, 6-, 12- and 24-hour periods. Precipitation was averaged over each time interval to form equal two-hour values of precipitation. Nineteen precipitation events were simulated for each interval, using the equation derived to predict storm runoff for Deer Creek. When daily (24-hour) observations of precipitation were used in the API method, peak flows were under-estimated by an average of 17 percent (Figure 41). This is approximately a 3-fold change from the average error of 6 percent associated with two-hour observations. A similar relationship was not detected for errors in storm runoff volume.

The number of years of data necessary for the accurate estimation of C, S and I, was explored using eleven years of record from Deer Creek. Values of C, S and I were derived for the annual peak flow, as though only one year of record was available. Values of C, S, and I were also derived for data sets of 2, 3, and 5 years. The standard deviation in values of S with increasing years of data is shown in Figure 42. Deviation in all three coefficients declined markedly within a period of record of about five years, indicating that five years of data is probably necessary to obtain reasonable estimates of C, S and I.

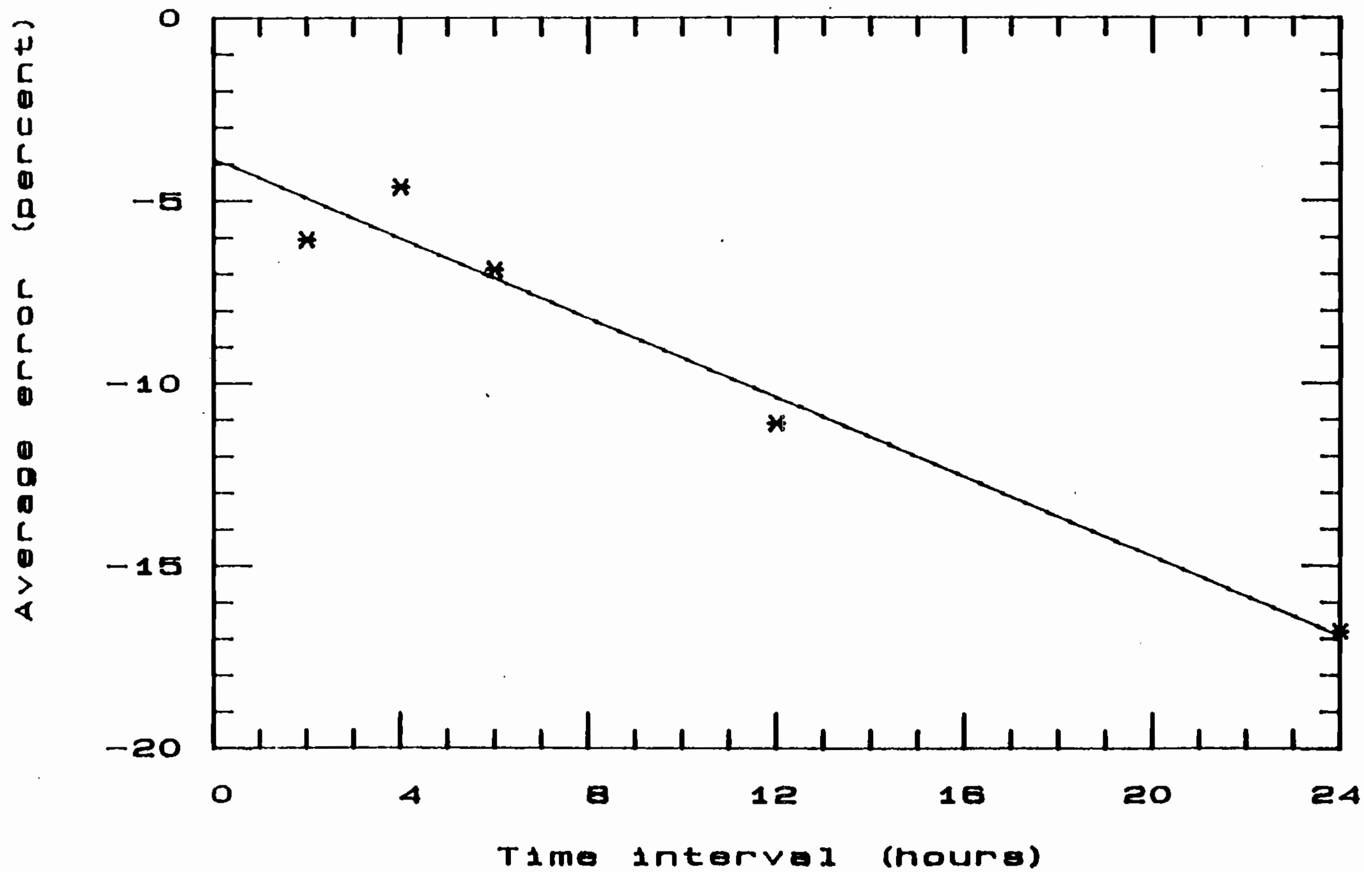


Figure 41. Time interval of precipitation observations (Δt) and associated average errors in peak flow estimates, Deer Creek, Oregon Coast Range. Nineteen events are included in each average error estimate for the five intervals of precipitation observations.

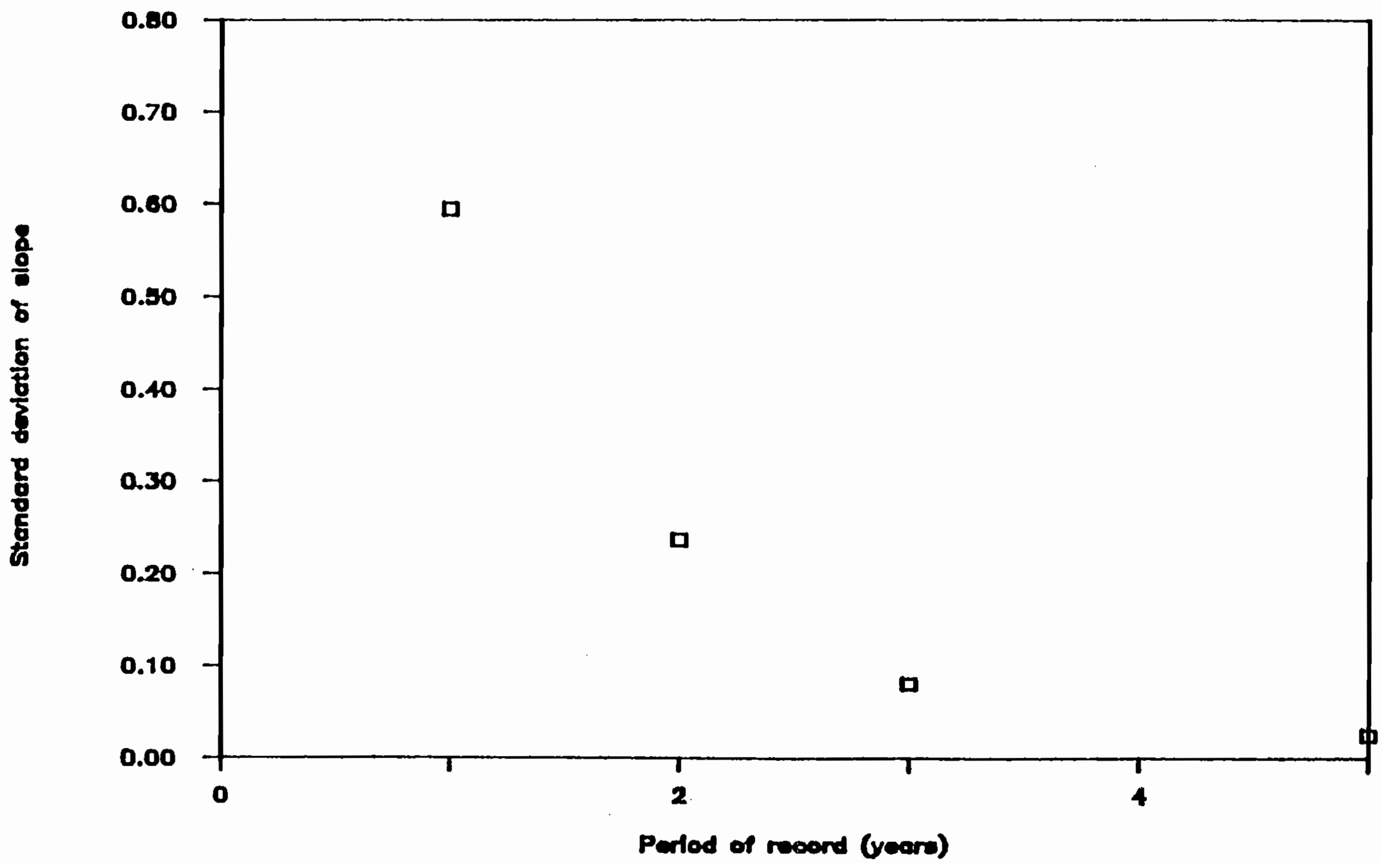


Figure 42. Standard deviation in values of slope (S) with changing period of record.

Use of the API Method on Oregon Coast Range Watersheds

The characteristics of rainfall that contribute to storm flow volumes and peak flows have been discussed in detail by Hewlett, et al., (1977, 1984), and Bren, et al., (1987). While these authors have found that peak rainfall intensity (as an independent variable) does not contribute significantly (statistically speaking) to the prediction of peak flows, this study points out that peak flows are strongly correlated with the volume and temporal distribution of rainfall. API values, based on recession flow analysis, were found to be strongly correlated with peak flows and discharge throughout entire rainfall events.

Errors in the estimate of peak flows averaged 16.2 percent for the watersheds used to calibrate the API model, and 17.8 percent for the independent test watershed. Errors in storm runoff volume estimates averaged 14.9 percent for calibration watersheds and 20.8 percent for the test watershed. On the average, predicted peak flows occurred before the observed peaks, and the errors increased with increasing watershed size. The model was found to be insensitive to small changes in the rate at which precipitation was decayed (recession coefficient) because of compensating adjustments in S and I. Errors in peak flows and storm runoff volumes on Deer

Creek increased when the elevation of precipitation observations, in comparison to the elevation of the Deer Creek precipitation gage, became increasingly greater. Errors in peak flow estimates decreased as the time interval (Δt) for precipitation observations became shorter. In general, the API method accurately simulates storm hydrograph shapes regardless of the complexity of rainfall distribution patterns.

Errors in the estimates of hydrograph characteristics may be partially mitigated by cross-correlating values of discharge and API. Linear regression models fit to the cross-correlated data could be used to simulate storm hydrographs. The hydrographs, and subsequent predictions of the timing of peak flows, would then be adjusted by the amount of the cross-correlation. A cross-correlation timing adjustment was not conducted in this study, however, it is expected to improve predictions of hydrograph characteristics particularly on large watersheds.

A seasonal index of antecedent moisture might improve the prediction of peak flows using an API method. Cumulative or "decayed" precipitation occurring before a runoff event (eg. 30, 60, 120 days) could be used as a second independent variable (with API_t), in a multiple regression model with storm runoff as the dependant variable. A model of this type was not explored in this study.

Errors introduced by snowmelt during rainfall were not evaluated in this study and may be substantial. Some measure of antecedent snowpack water equivalent and snowmelt rates during a rainfall event are essential for accurate predictions of storm hydrographs during rain-on-snow events. The API model could be linked to a snowmelt prediction model to account for the additional moisture.

The API method provides a simple and objective methodology for simulating individual storm hydrographs. Assuming the model has been previously calibrated for a region, drainage area is the only characteristic of a watershed required for use of the API model. Records from a recording rain gage are also necessary.

CONCLUSIONS AND RECOMMENDATIONS

The SCS runoff curve number technique was originally developed for predicting changes in storm runoff volume with changing land management practices. It has since been applied to problems well outside the original intentions of the authors. As a single event hydrograph model, the simulated runoff was found to be highly sensitive to the assumed curve number and precipitation intensity. Curve numbers require a considerable amount of user judgment to derive. When standard SCS procedures were used to estimate the curve number for Deer Creek, peak flows were over-estimated by about a factor of two. When unit hydrograph shape, watershed lag, and curve number were adjusted using data from Deer Creek, simulated hydrograph shape and timing of predicted peak flows were not improved. Furthermore, the increasing proportion of rainfall that becomes runoff throughout a storm--regardless of the time distribution of the rainfall--is a major source of error. Because of these limitations, the SCS curve number procedure is not recommended for use as a peak flow prediction technique, nor as a single event simulation model in the Oregon Coast Range. Thus, the average curve number derived for Deer Creek in this study, using a fitting procedure, is not recommended for use elsewhere in the Oregon Coast Range.

An index of antecedent conditions, using precipitation that is "decayed" through time, was found to be highly correlated with peak flows and discharge throughout high flow events. As a single event simulation model the API method works well and requires no user judgment of parameters. Relationships between API and discharge can be developed from a relatively brief period of record (about five years). Differences between observed and simulated flows and timing of runoff increase with increasing separation of the precipitation gage and the watershed, and with increasing time intervals (Δt) between precipitation observation.

The API method was developed from Coast Range watersheds which are underlain by similar bedded sediments. Hydrologic processes and runoff characteristics are expected to differ in other areas. Further research is necessary to determine the basin characteristics that contribute to recession coefficient variability. Further exploration of the API method is recommended with regard to timing differences between observed and predicted flows, time intervals of precipitation observations, seasonal indices of antecedent moisture, and the effects of snowmelt during rainfall. Because of model sensitivity to precipitation gage elevation, continuously recording precipitation gages should be located throughout a range of elevations where simulation of streamflow is likely to take place.

Although a direct comparison of the SCS and API methods was beyond the scope of this study, a simple comparison of the two procedures can be made by examining Figures 17 and 20. Figure 17 shows an example of observed and SCS simulated storm hydrographs after fitting SCS parameters to the Deer Creek data. Figure 20 shows the observed and API simulated hydrograph for the same event on Deer Creek, using the API discharge equation derived for Deer Creek. These examples represent the best fit of each model. Clearly, the API method of streamflow simulation represents a much improved method for simulating storm hydrographs on small, forested watersheds in the Oregon Coast Range.

The API methodology has potential applications beyond a storm hydrograph simulation model. Although the subject was briefly explored here, spatial distribution of precipitation gage locations and time intervals of observation necessary for estimation of streamflow on watersheds of various sizes, could be studied using an API model. An antecedent precipitation index may prove to be a good measure of groundwater fluctuations caused by precipitation, and aid in the prediction of mass failures. An API and discharge relationship can be used to estimate missing discharge or missing precipitation data. In locations were the precipitation record is longer than the runoff record, the runoff record could be extended with an API model for use in frequency analyses. In addition, API models could be linked to snowmelt simulation and suspended sediment models. The applicability of API methods may carry far beyond the limited geographic area examined in this study.

LITERATURE CITED

Aitken, A. P., 1973. Assessing Systematic Errors in Rainfall-Runoff Models. Journal of Hydrology, 20: 121-136.

Bernier, P. Y., 1985. Variable Source Areas and Storm-flow Generation: An Update of the Concept and a Simulation Effort. Journal of Hydrology, 79: 195-213.

Bondelid, T. R., R. H. McCuen, and T. J. Jackson, 1982. Sensitivity of SCS Models to Curve Number Variation. Water Resources Bulletin, 18(1): 111-116.

Boughton, W. C., 1986. Linear and Curvilinear Baseflow Recessions. Journal of Hydrology (N.Z.), 25(1): 41-48.

Bren, L. J., P. W. Farrell, and C. J. Leitch, 1987. Use of Weighted Integral Variables to Determine the Relation Between Rainfall Intensity and Storm Flow and Peak Flow Generation. Water Resources Research, (in press).

Burroughs, E. R. Jr., G. R. Chalfant, M. A. Townsend, 1973. Guide to Reduce Road Failures in Western Oregon. USDI Bureau of Land Management, Portland, Oregon, 111 pp.

Campbell, A. J. and R. C. Sidle, 1984. Prediction of Peak Flows on Small Watersheds in Oregon for use in Culvert Design. Water Resources Bulletin, 20(1): 9-14.

Dawdy, D. R., 1982. A Review of Deterministic Surface Water Routing in Rainfall-Runoff Models. pp. 23-36. <u>In</u>: Rainfall-Runoff Relationship, Singh, V. P. (editor), Water Resources Publications, Littleton, Colorado, 582 pp.

Dunne, T. and L. B. Leopold, 1978. Water in Environmental Planning. W. H. Freeman and Company, pp. 287, 288, 329-335.

Garstka, W. V., L. D. Love, B. C. Goodel, and F. A. Bertle, 1958. Section 7--Recession Analyses. pp. 63-70. In: Factors Affecting Snowmelt and Streamflow, USDA Forest Service, Rocky Mountain Forest and Range Experiment Station, Ft. Collins, Colorado, 189 pp.

Green, I. R. A. and D. Stephenson, 1986. Criteria for Comparison of Single Event Models. Journal of Hydrological Sciences, 31(3): 395-411.

Harr, R. D., 1986. Effects of Clearcutting on Rain-on-Snow Runoff in Western Oregon: A New Look at Old Studies. Water Resources Research, 22(7): 1095-1100.

Harr, R. D., W. C. Harper, J. T. Krygier, and F. S. Hsieh, 1975. Changes in Storm Hydrographs after Roadbuilding and Clear-cutting in the Oregon Coast Range. Water Resources Research, 15(1): 90-94.

Harris, D. D., L. L. Hubbard, and L. E. Hubbard, 1979. Magnitude and Frequency of Floods in Western Oregon. U.S.D.I. Geological Survey, Open-file Report 79-553, Portland, Oregon, 35 pp.

Hawkins, R. H., 1975. The Importance of Accurate Curve Numbers in the Estimation of Storm Runoff. Water Resources Bulletin, 11(5): 887-891.

Hawkins, R. H., 1979. Runoff Curve Numbers from Partial Area Watersheds. Journal of Irrigation and Drainage Division, ASCE, Vol. 105, No. IR4, pp. 375-389.

Hawkins, R. H., 1986. A Comparison of Predicted and Observed Runoff Curve Numbers. Journal No. 2944, Utah Agricultural Experiment Station, Project 696, 8 pp.

Hewlett, J. D., 1982. Principles of Forest Hydrology. The University of Georgia Press, Athens, Georgia, 183 pp.

Hewlett, J. D., J. C. Forston, and G. B. Cunningham, 1977. The Effect of Rainfall Intensity on Storm Flow and Peak Discharge from Forest Land. Water Resources Research, 13(2): 259-266.

Hewlett, J. D., J. C. Forston, and G. B. Cunningham, 1984. Additional Tests on the Effect of Rainfall Intensity on Storm Flow and Peak Flow from Wild Land Basins. Water Resources Research, 20(7): 985-989.

Hewlett, J. D. and A. R. Hibbert, 1967. Factors Affecting the Response of Small Watersheds to Precipitation in Humid Areas. pp. 275-290. In: International Symposium on Forest Hydrology, Sopper, W. E. and H. W. Lull (editors), Pergamon Press, New York, 813 pp.

Hiemstra, L. A. V. and B. M. Reich, 1967. Engineering Judgment and Small Area Flood Peaks. Colorado State University, Hydrology Paper No. 19, 29pp.

Hornberger, G. M., K. J. Beven, B. J. Cosby, and D. E. Sappington, 1985. Shenandoah Watershed Study: Calibration of a Topography-Based, Variable Contributing Area Hydrological Model to a Small Forested Catchment. Water Resources Research, 21(12): 1841-1850.

Istok, J. D. and L. Boersma, 1986. Effect of Antecedent Rainfall on Runoff During Low-Intensity Rainfall. Journal of Hydrology, 88: 329-342.

Jackson, W. L. and B. P. Van Haveren, 1984. Rainfall-Runoff Prediction and the Effects of Logging: The Oregon Coast Range. U.S.D.I. Bureau of Land Management, Denver Service Center, Denver, Colorado, 36pp.

Johnson, M. G. and R. L. Beschta, 1980. Logging, Infiltration Capacity, and Surface Erodibility in Western Oregon. Journal of Forestry, 78(6): 334-337.

Larson, L. W. and E. L. Peck, 1974. Accuracy of Precipitation Measurements for Hydrologic Modeling. Water Resources Research, 10(4): 857-863.

Linsley, R. K., M. A. Kohler, and J. L. H. Paulhus, 1982. Hydrology for Engineers. McGraw-Hill, pp. 214-229.

Loague, K. M. and R. A. Freeze, 1985. A Comparison of Rainfall-Runoff Modeling Techniques on Small Upland Catchments. Water Resources Research, 21(2): 229-248.

Lyons, J. K. and R. L. Beschta, 1983. Land Use, Floods, and Channel Changes: Upper Middle Fork Willamette River, Oregon (1936-1980). Water Resources Research, 19(2): 463-471.

Moore, I. D., G. B. Coltharp, and P. G. Sloan, 1983. Predicting Runoff from Small Appalachian Watersheds. Trans. Ky. Acad. Sci., 44(3-4): 135-145.

Moore, I. D., S. M. Mackay, P. J. Wallbrink, G. J. Burch, and E. M. O'Loughlin, 1986. Hydrologic Characteristics and Modelling of a Small Forested Catchment in Southeastern New South Wales, Pre-Logging Condition. Journal of Hydrology, 83: 307-385.

Newton, D. W., and J. C. Herrin, 1982. Assessment of Commonly Used Methods of Estimating Flood Frequency. Transportation Research Record, TRR896: 10-30.

O'Loughlin, E. M., 1986. Prediction of Surface Saturation Zones in Natural Catchments by Topographic Analysis. Water Resources Research, 22(5): 794-804.

Piehl, B. T., 1987. An Evaluation of Culverts on Low Volume Forest Roads in the Oregon Coast Range. MS Thesis, Oregon State Univ., Corvallis, Oregon, 78 pp.

Rallison, R. E. and N. Miller, 1982. Past, Present, and Future SCS Runoff Procedure. pp. 353-364. In: Rainfall-Runoff Relationship, Singh, V. P. (editor). Water Resources Publications, 582 pp.

Richardson, H. H. and R. G. Cronshey, 1985. The Improved SCS TR-20 Network Watershed Model. pp. 92-98. In: Watershed Management in the Eighties, Jones, E. B. and T. J. Ward (editors), 319 pp.

Settergren, C., M. Morris, A. Hjelmfelt, and G. Henderson, 1985. Synthetic Unit Hydrographs for Ozark Watersheds. pp. 140-145. In: Watershed Management in the Eighties, Jones, E. B. and T. J. Ward (editors), 319 pp.

Snyder, F. F., 1938. Synthetic Unit Hydrographs. Trans. Am. Geophysical Union, 19(1): 447-454.

Troutman, B. M., 1983. Runoff Prediction Errors and Bias in Parameter Estimation Induced by Spatial Variability of Precipitation. Water Resources Research, 19(3): 791-810.

USDA Soil Conservation Service (in cooperation with Oregon Agricultural Experiment Station), 1975. Soil Survey of Benton County Area, Oregon. Washington, D.C. 119 pp.

USDA Soil Conservation Service, 1972. Hydrology. National Engineering Handbook, Section 4, Washington, D.C., (various pagings).

USDA Soil Conservation Service, 1979. Chapter 2--Estimating Runoff. pp. 1-10. Engineering Field Manual--Oregon, Western Regional Technical Service Center, Portland, Oregon, (various pagings).

USDC National Oceanic and Atmospheric Administration, 1960-1986. Hourly Precipitation Data--Oregon. National Climatic Data Center, Asheville, N. Carolina.

USDC Weather Bureau, 1956. Rainfall Intensities for Local Drainage Design in Western United States. Technical Paper 28, 46 pp.

USDI Geological Survey, 1962-1984. Water Resources Data--Oregon. Portland, Oregon.

VanSickle, J. and R. L. Beschta, 1983. Supply-Based Models of Suspended Sediment Transport in Streams. Water Resources Research, 19(3): 768-778.

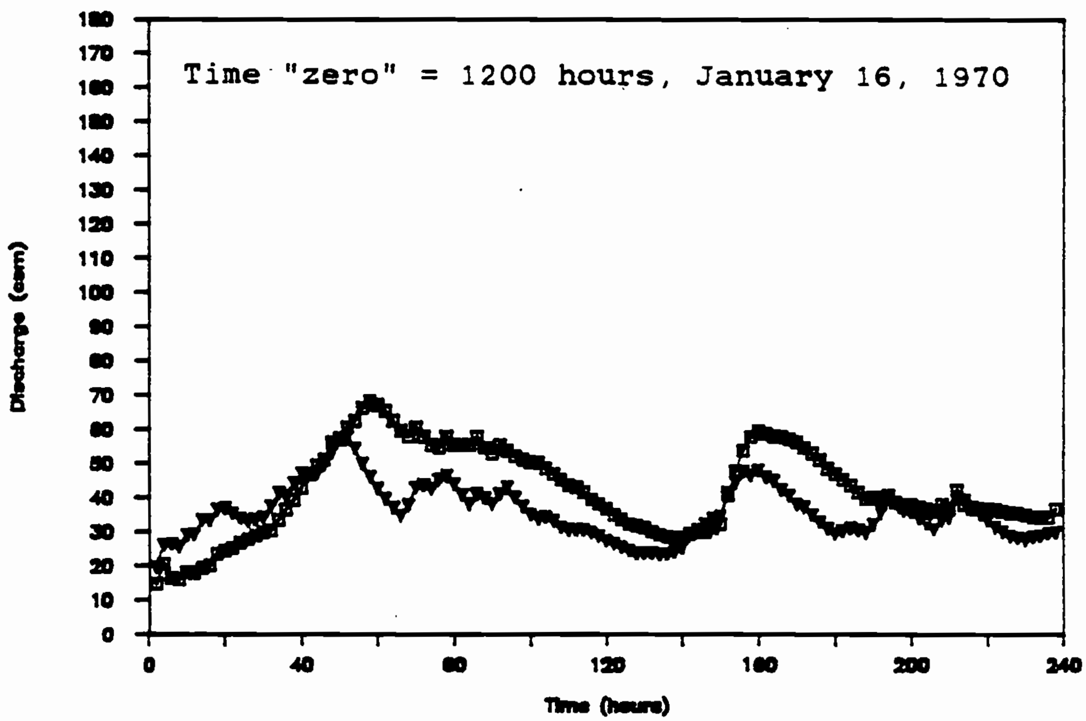
Weeks, W. D. and R. H. B. Hebbert, 1980. A Comparison of Rainfall-Runoff Models. Nordic Hydrology, 11: 7-24.

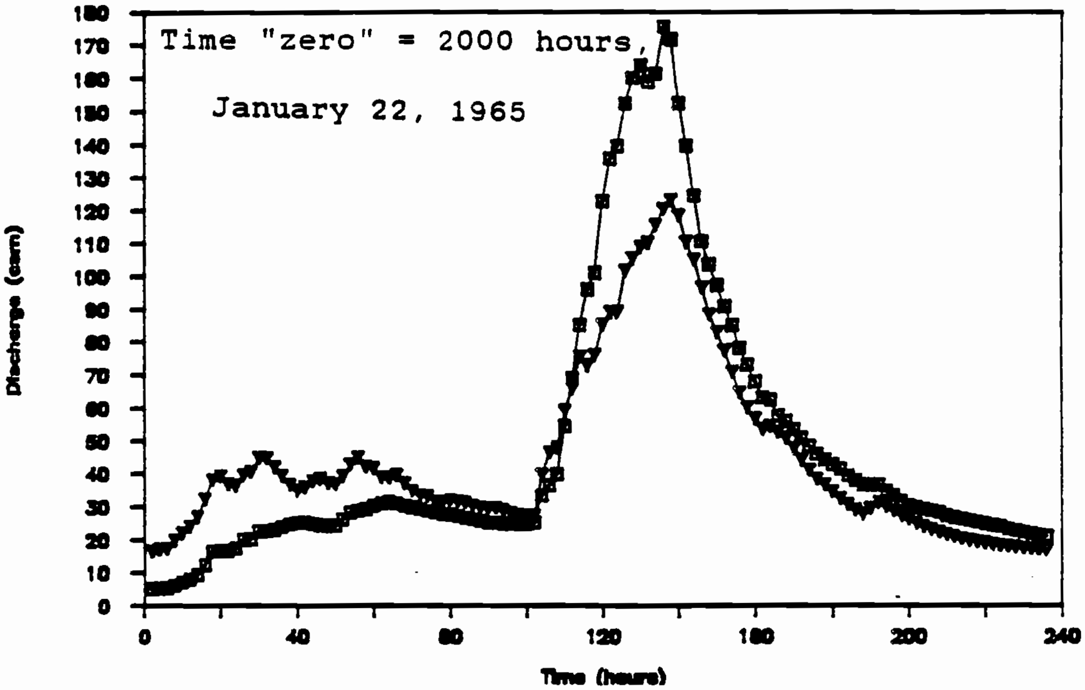
Williams, J. R. and W. V. LaSeur, 1976. Water Yield Model Using SCS Curve Numbers. Journal of the Hydraulics Division, ASCE, Vol. 102, No. HY9, pp. 1241-1253.

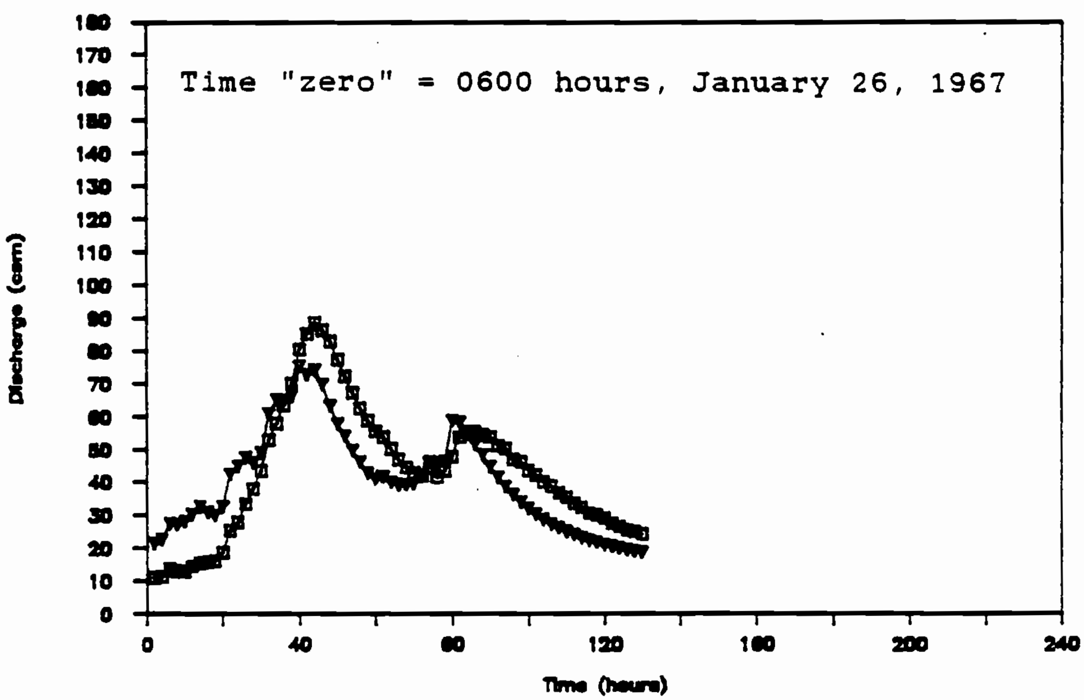
Ziemer, R. R. and J. S. Albright, 1987. Subsurface Pipeflow Dynamics of North-Coastal California Swale Systems. Proceedings of: Erosion and Sedimentation Pacific Rim, Corvallis, Oregon, August 3-7, (in press).

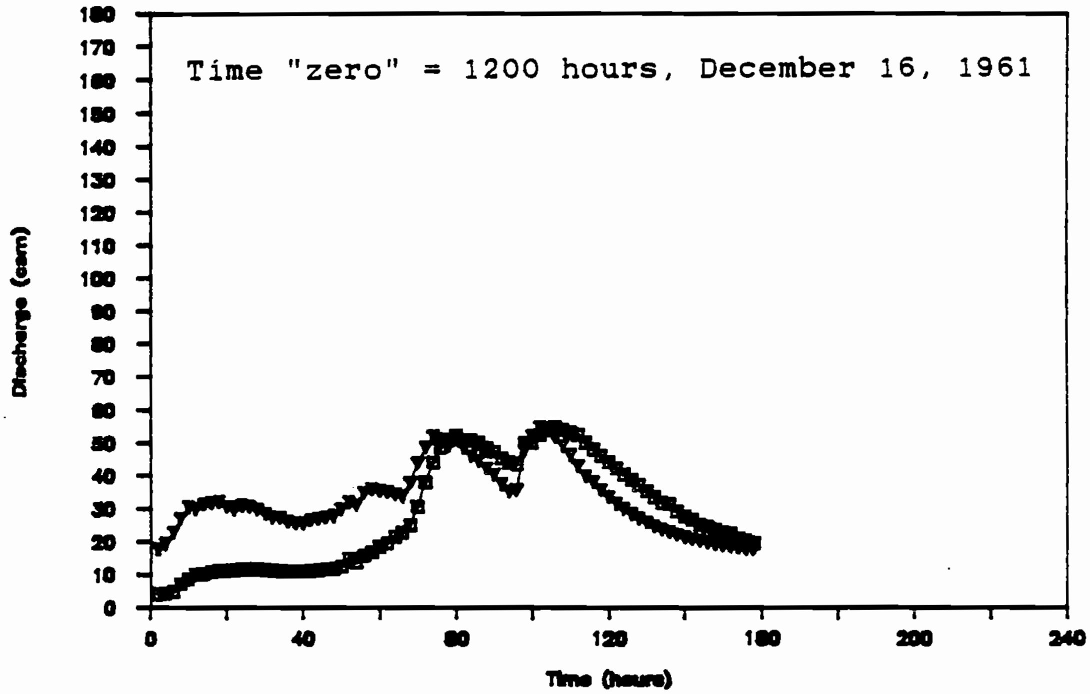
APPENDIX

Appendix A. Observed (□) and simulated (▽) hydrographs (API method), Flynn Creek, Oregon Coast Range.

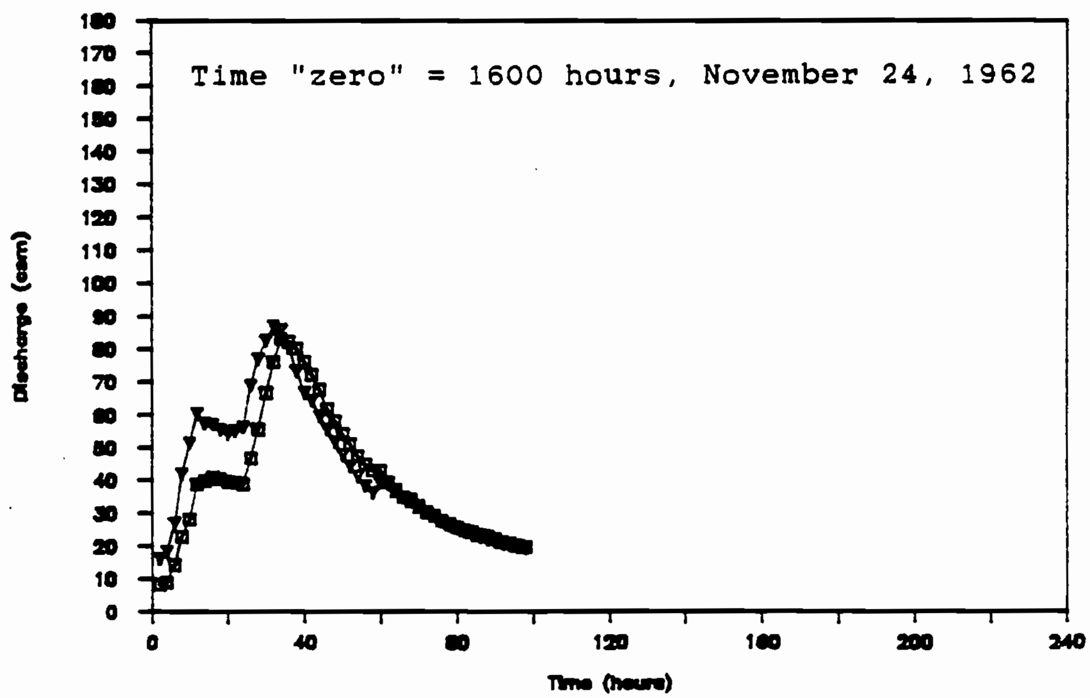



Time "zero" = 2000 hours,
January 22, 1965
Discharge (csm)
Time (hours)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
0
40
80
120
160
200
240

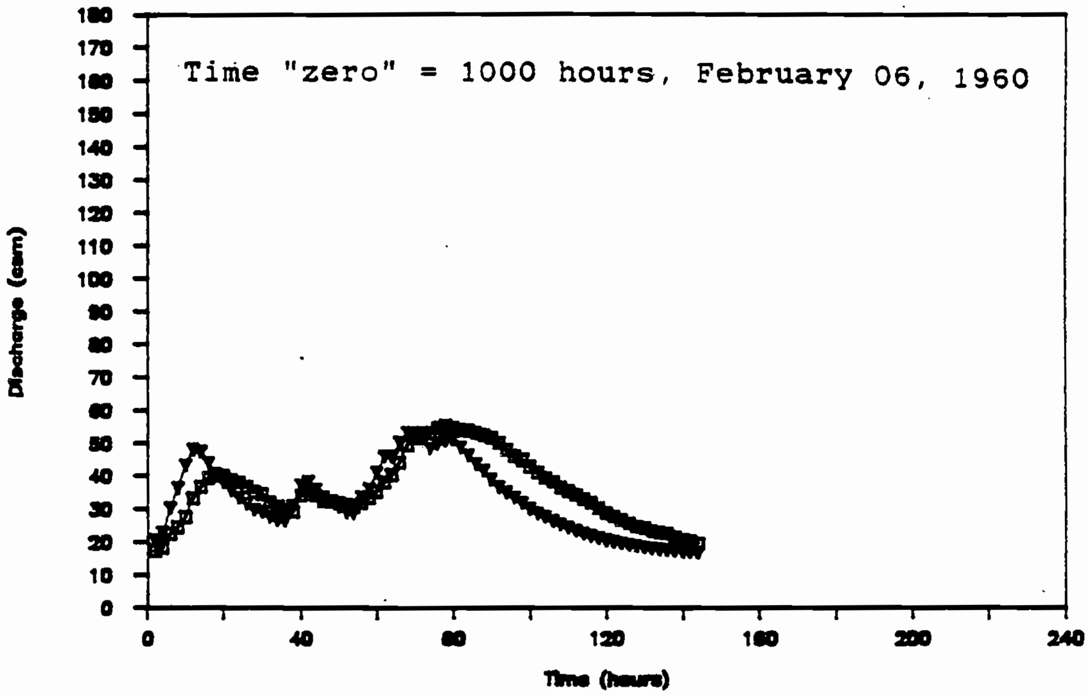

Time "zero" = 0600 hours, January 26, 1967
Discharge (csm)
Time (hours)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
0
40
80
120
160
200
240

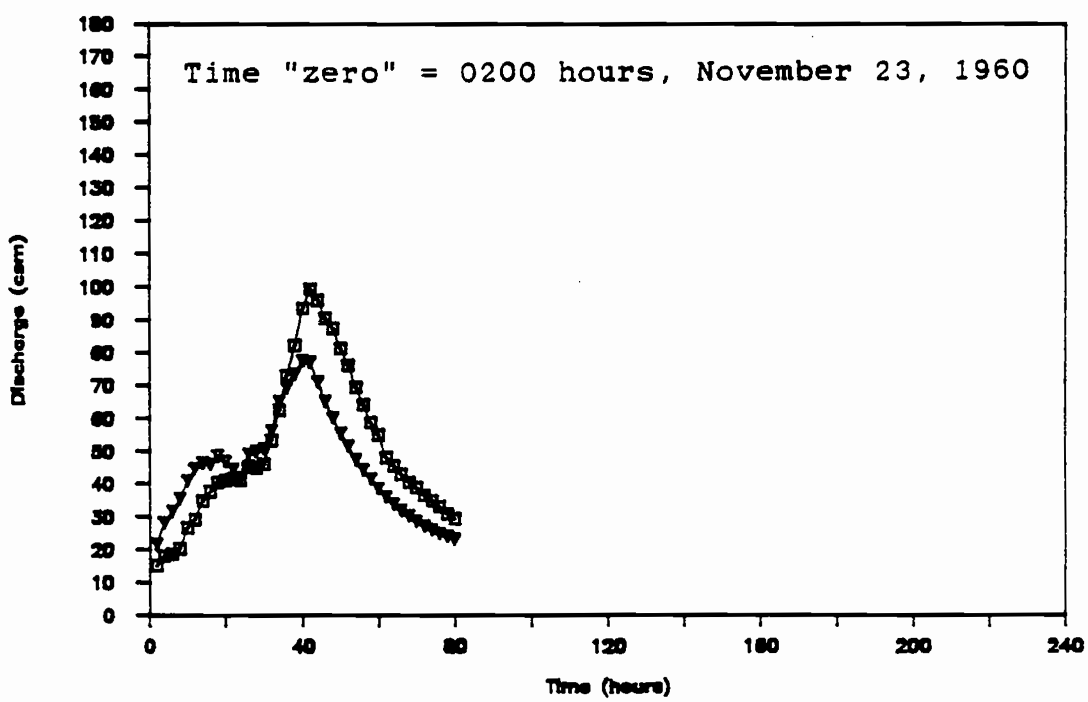


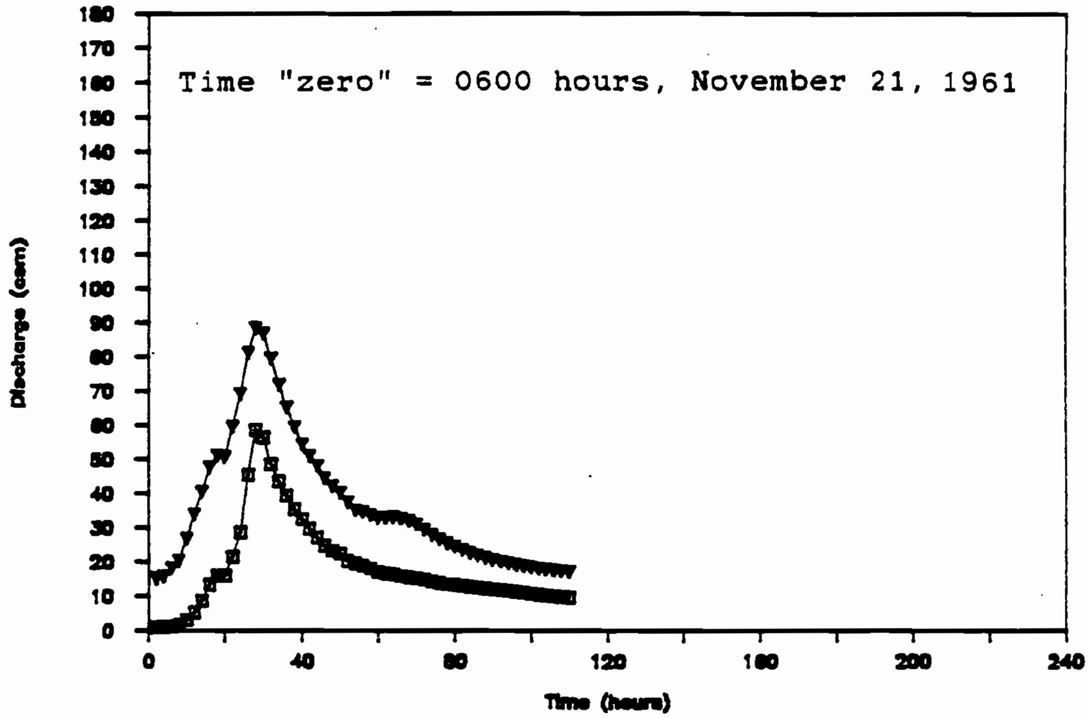
Time "zero" = 1200 hours, December 16, 1961
Discharge (csm)
Time (hours)

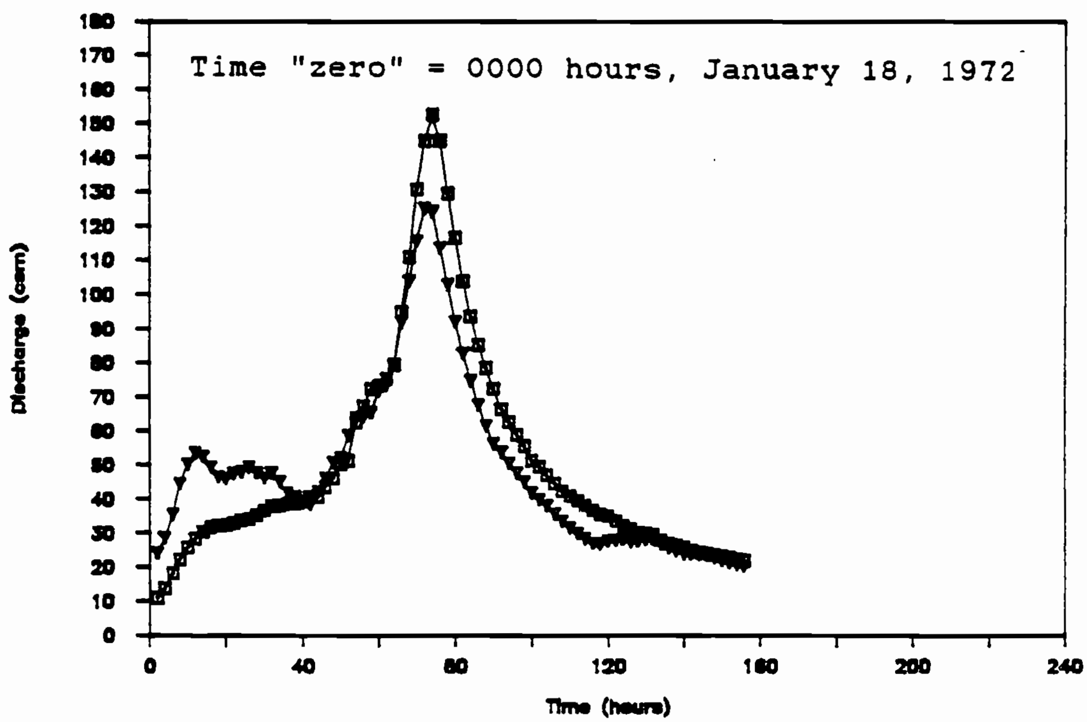


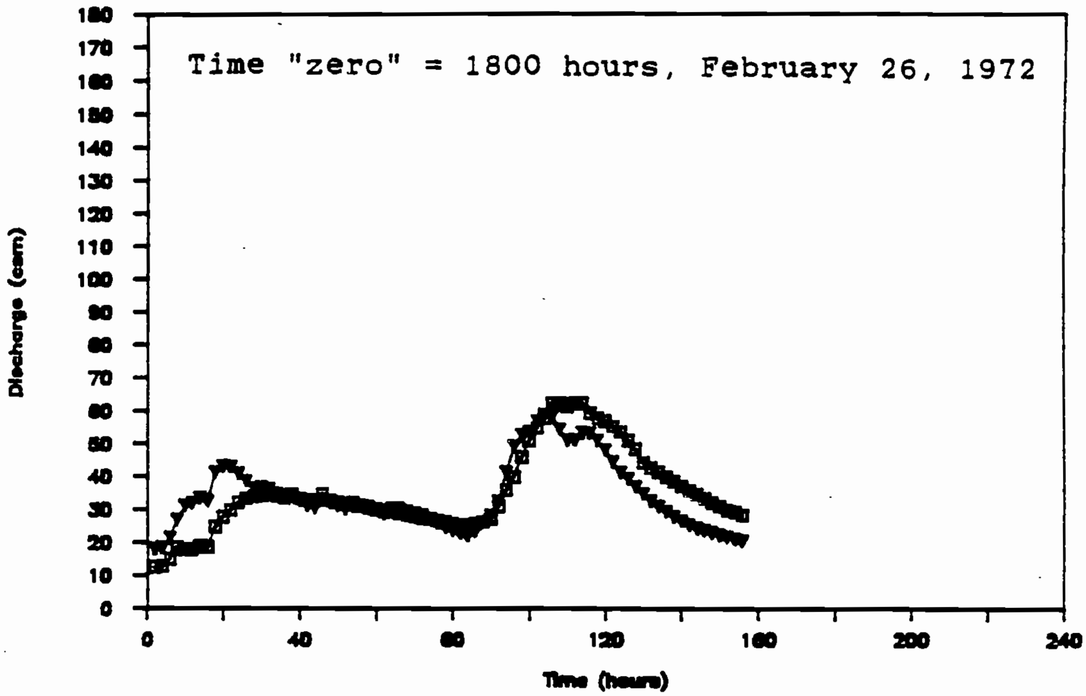
Time "zero" = 1600 hours, November 24, 1962
Discharge (csm)
Time (hours)

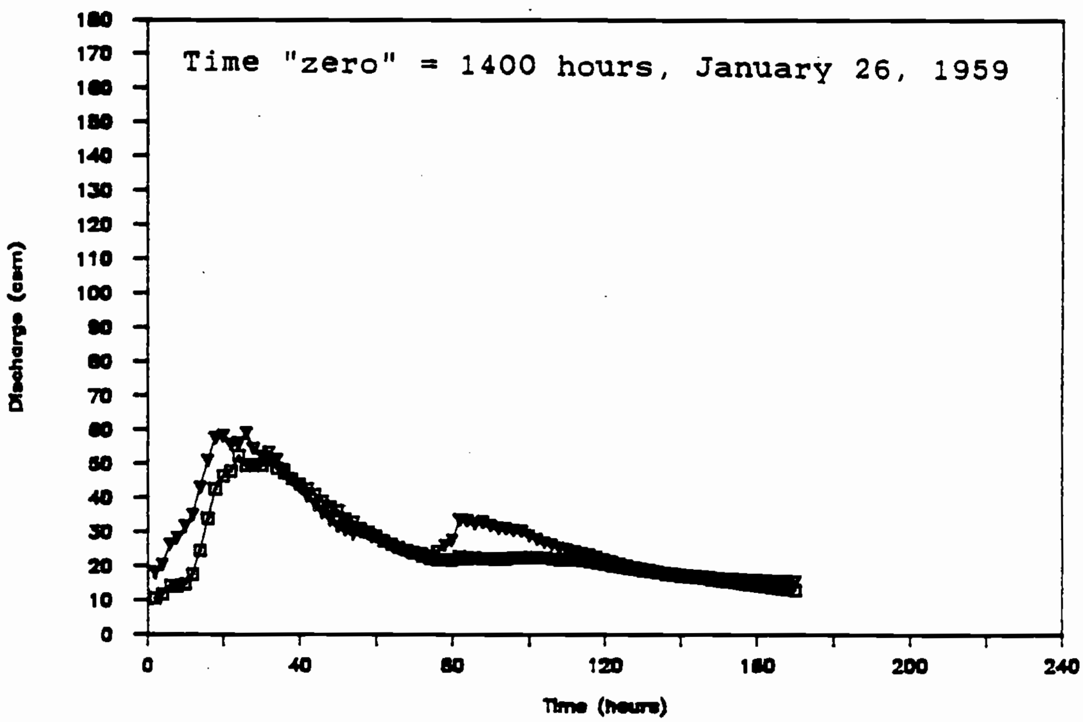

Time "zero" = 1000 hours, February 06, 1960
Discharge (cms)
Time (hours)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
0
40
80
120
160
200
240

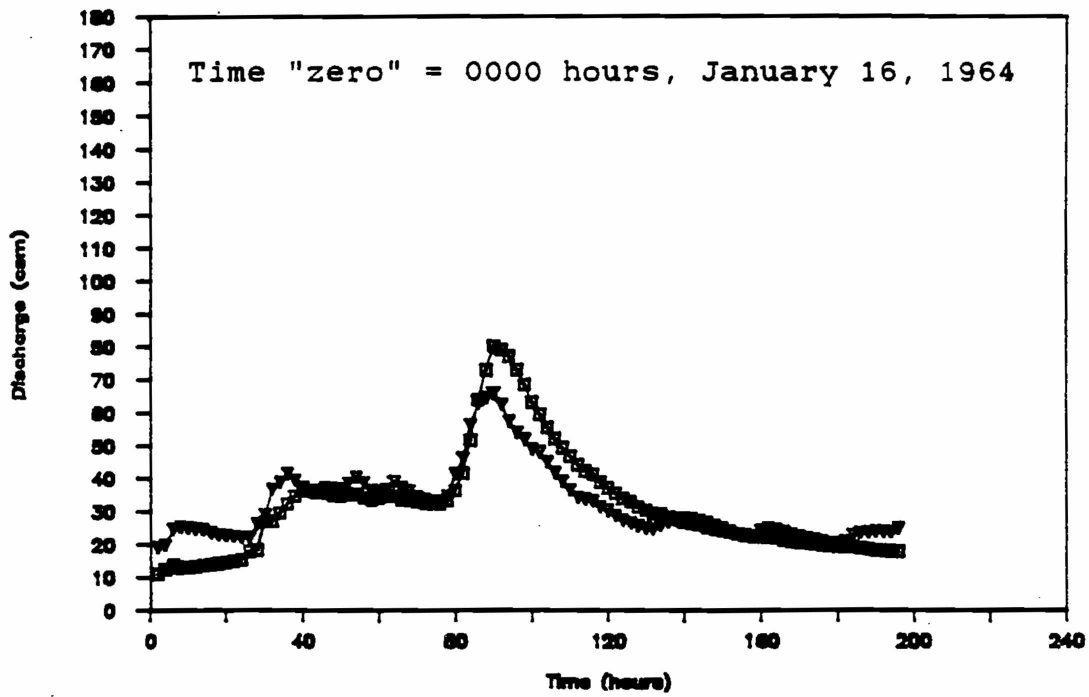

Time "zero" = 0200 hours, November 23, 1960
Discharge (cms)
Time (hours)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
0
40
80
120
160
200
240


Time "zero" = 0600 hours, November 21, 1961
Discharge (cms)
Time (hours)

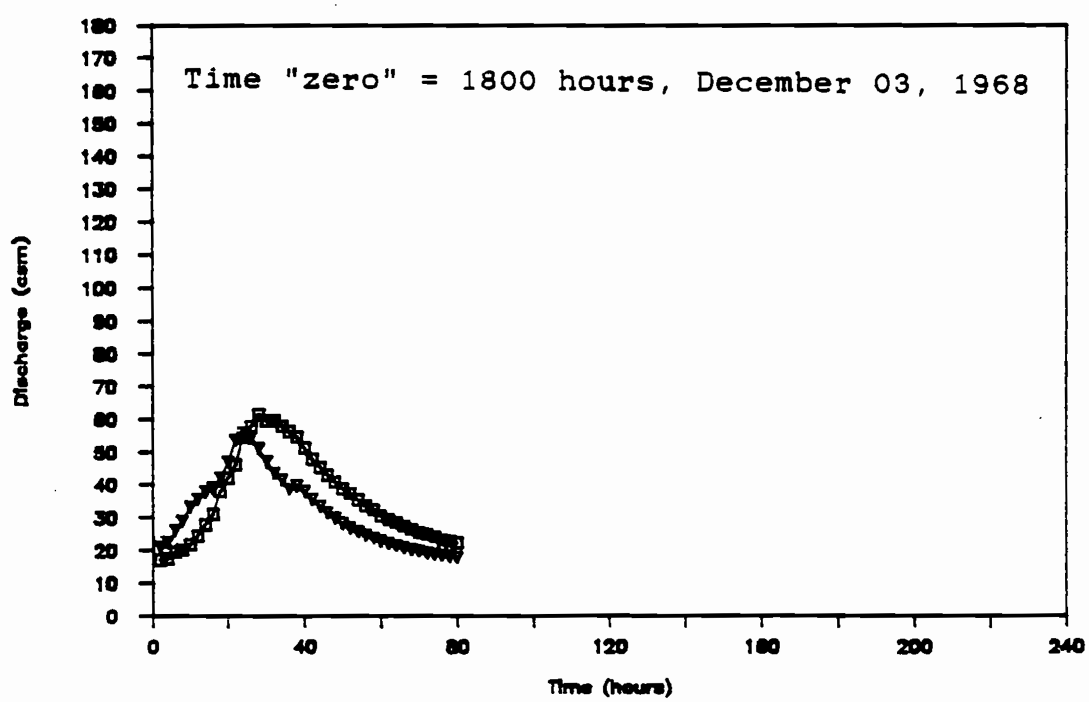

Time "zero" = 0000 hours, January 18, 1972
Discharge (cms)
Time (hours)


Time "zero" = 1800 hours, February 26, 1972
Discharge (csm)
Time (hours)
0
40
80
120
160
200
240
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0

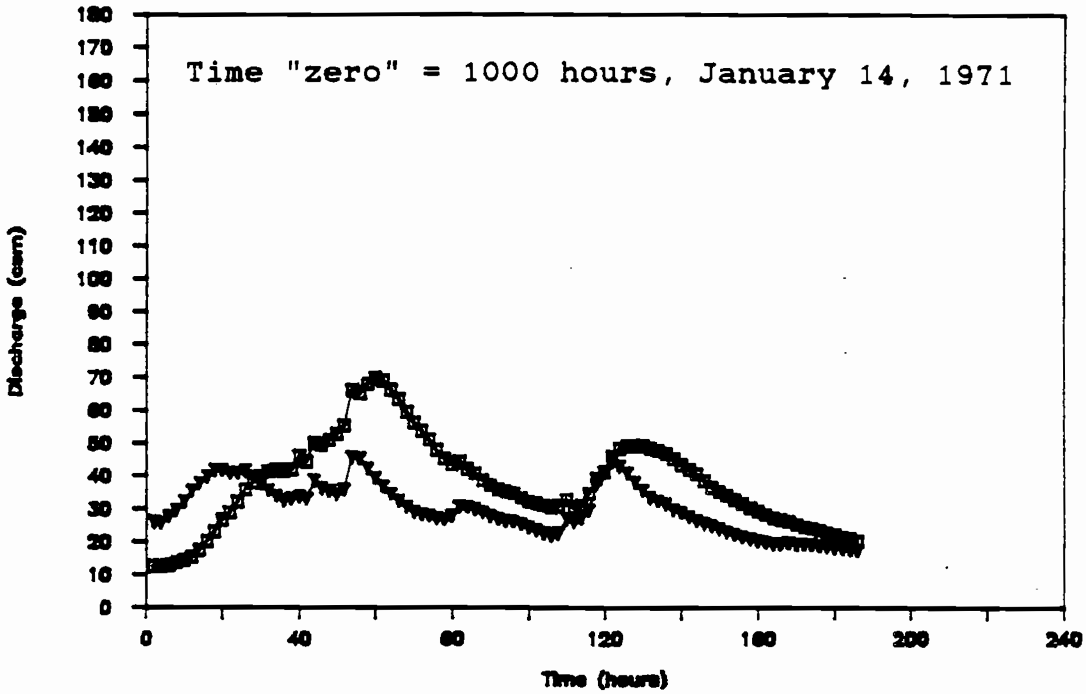

Time "zero" = 1400 hours, January 26, 1959
Discharge (csm)
Time (hours)
0
40
80
120
160
200
240
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0

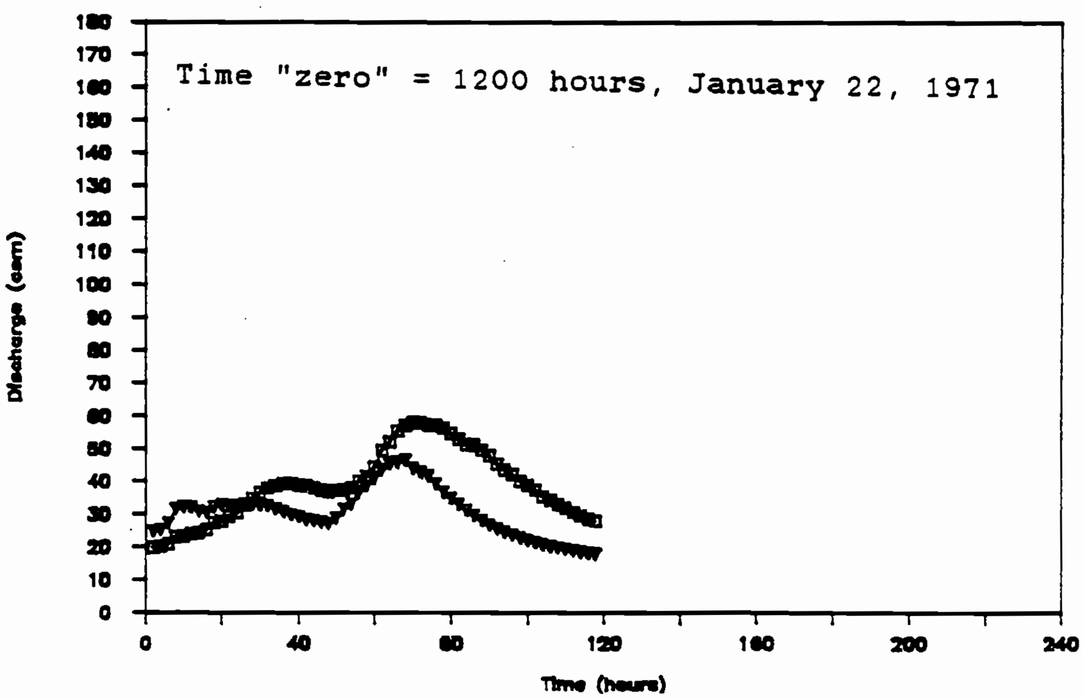


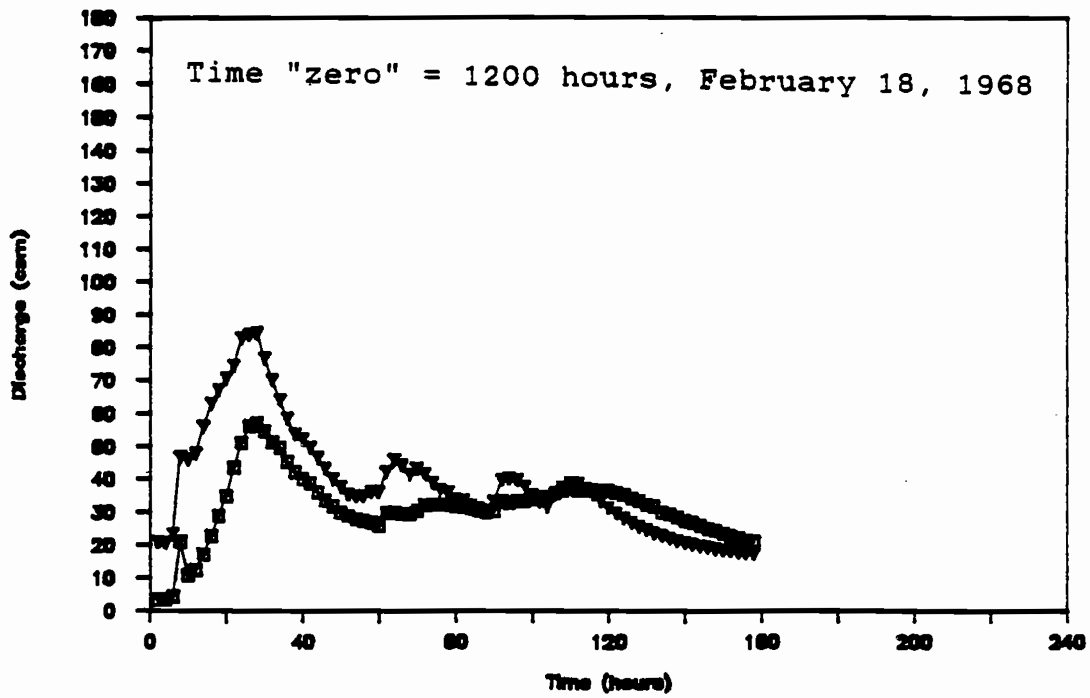
Time "zero" = 0000 hours, January 16, 1964
Discharge (cms)
Time (hours)

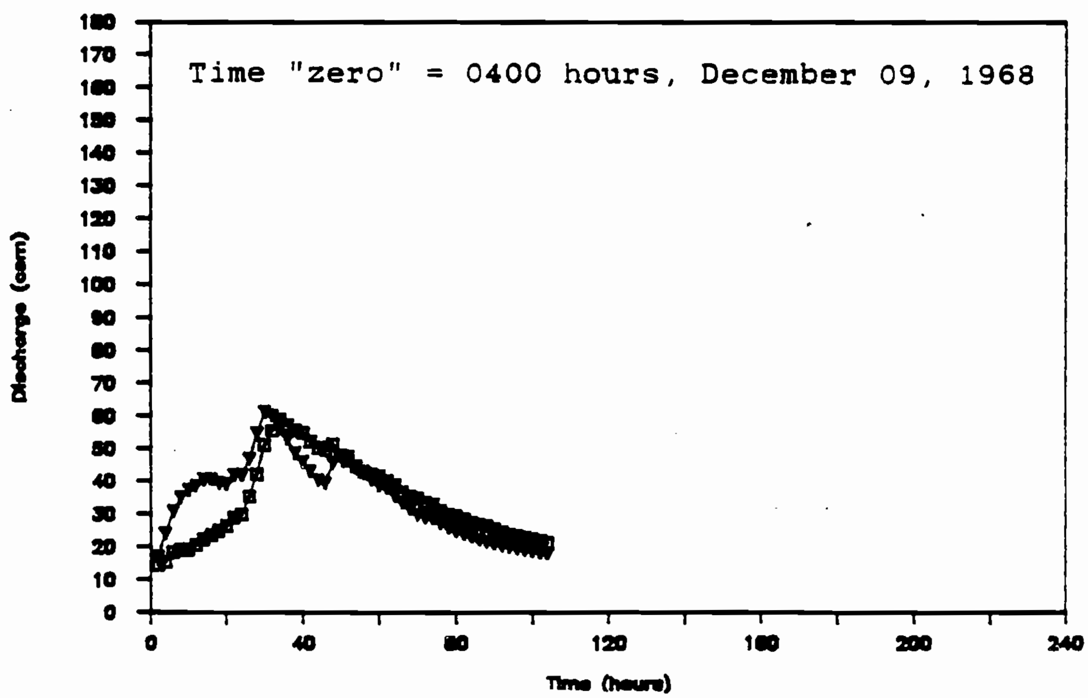


Time "zero" = 1800 hours, December 03, 1968
Discharge (cms)
Time (hours)


Time "zero" = 1000 hours, January 14, 1971
Discharge (cms)
Time (hours)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
0
40
80
120
160
200
240


Time "zero" = 1200 hours, January 22, 1971
Discharge (cms)
Time (hours)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
0
40
80
120
160
200
240


Time "zero" = 1200 hours, February 18, 1968
Discharge (cm)
Time (hours)


Time "zero" = 0400 hours, December 09, 1968
Discharge (cm)
Time (hours)