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DIGITAL SIMULATION IN HYDROLOGY: STANFORD WATERSHED MODEL IV

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Prepared with the partial support of
National Science Foundation
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LIST OF SYMBOLS*

SYMBOL	
a	Constant in relation for overland flow discharge
b	Maximum rate of infiltration with linear cumulative distribution of infiltration capacity (overland flow parameter, Appendix A)
c	Interflow volume parameter
C	Overland flow parameter
D	Surface detention in overland flow
D_s	Snow depth
E_p	Potential evapotranspiration
e	Subscript - equilibrium
^{o}F	Degrees Fahrenheit
GWF	Groundwater outflow
g	Gravitational constant
Hm	Net radiation heat exchange
I	Inflow
i	Supply rate to overland flow (inches/hour)
INTF	Interflow discharge
K	Constant relating storage to discharge (overland flow parameter, Appendix A)
LZS	Lower zone storage
LW	Long-wave radiation
L	Length of overland flow plane
LIRC4	Component of interflow storage that enters interflow discharge
M_c	Convection melt

LIST OF SYMBOLS (continued)

SYMBOLS	
M_p	Melt caused by rainfall
n	Manning's n
NEGMELT	Heat required to increase snowpack temperature to 32^{o} F
MEGMELTM	Maximum NEGMELT (function of atmospheric temperature)
O	Discharge
PACK	Water equivalent of snow in a snowpack
P_g	Percentage of infiltration entering groundwater
P_x	Hourly rainfall
q	Overland flow discharge
Q	Discharge
r	Evapotranspiration opportunity index
^{o}R	Degrees Rankin
S	Slope
SDEN	Snow Density
SRGX	Transient Interflow Storage
SGW	Groundwater storage
t	Time
T	Temperature
T_r	Temperature in degrees Rankin
y	Depth of overland flow
$\bar{x}$	Mean mositure supply
UZI1 UZI2	Variables in upper zone response

LIST OF SYMBOLS (continued)

SYMBOLS	
W	Width
η	Supply rate ($ft^3/sec/ft^2$)
ν	Kinematic viscosity
β	Fraction of inflow that leaves the overland flow plane between $t = 0$ and $t = t_e$

*Does not contain symbols defined in Tables 5.1, 5.2 or 5.10.

I. INTRODUCTION

The hydrologic regimes of streams and rivers provide the basic information used for the design of hydraulic engineering projects. The development of methods to better estimate the hydrologic regime and the application of these methods in practice is the task of hydrology. Interest in hydrology has increased greatly in recent years as expanding populations have made greater demands on limited natural resources.

To accomplish their purposes, hydrologists have developed techniques for the collection of basic data, and for the analysis, correlation, and extension of these data. A considerable volume of basic data about hydrologic regimes has been collected in this century but few data are available prior to 1900. Measured against the diversity of hydrologic regimes even current levels of data collection are often inadequate. Projections and correlations based on limited data are necessary and are commonly used. Adequate projections and correlations of data involve considerable numerical analysis, and many of the methods originally developed for manual solution can be profitably programmed for digital computers. However, a technique developed for manual solution is generally trivial for a large-scale digital computer. Methods that are greatly expanded in scope are possible when the traditional limitation of calculating speed is removed. This report describes hydrologic techniques specifically developed for large-scale digital computers.

In hydrology physical data on rainfall, evapotranspiration, and runoff are collected at thousands of stations in the United States and abroad. The goal of hydrology is to establish an orderly discipline to account for changes in the observed hydrologic variables and to predict the hydrologic behavior of watersheds where observed data are incomplete or are not available.

The hydrologic cycle is fairly easy to describe in qualitative terms. The principal components of the cycle are reasonably easy to identify and the interactions between the major components are well known. The extension of this qualitative knowledge about the hydrologic cycle to obtain quantitative results is much more difficult. Few basic quantitative concepts exist in hydrology, compared to other fields. The physical dimensions and construction of a steel frame, for example, can be used to predict the dynamic response of the frame to earthquake motion. The parallel hydrologic problem may be to predict the outflow hydrograph response of a watershed to storm rainfall. The development of hydrology to date does not in general permit accurate estimates of hydrograph response from physical information. It may never be possible to develop hydrology into a mathematically precise science, nonetheless, the ability to accurately predict behavior is a severe test of the adequacy of knowledge in any subject.

For the past two years graduate students in hydrology at Stanford have been given a short quiz that attempts to measure their ability to make quantitative estimates of hydrologic behavior. The complexity of quantitative reasoning in hydrology is evident from the results of the quiz. A typical question is, "Given the hydrograph of a historic storm that occurred in a watershed, how would the hydrograph differ if the storm had occurred on a different date with much different (specified) initial conditions?" A problem of this type is difficult for two reasons. First, the processes of the hydrologic cycle that occur at any point in time are complex and interrelated. The instantaneous rate of infiltration, for example, is dependent on soil permeability and on the distribution of moisture in the soil profile. Second, most hydrologic processes are strongly time dependent. Thus, a typical quantitative estimate, such as the quantity of runoff to be expected from a storm, involves both estimates for processes at a point in time and the projection of these estimates forward in time.

Research in digital models of the hydrologic cycle began at Stanford in 1959[1,2,3,4]. The object of the research is to develop a general system of quantitative analysis for hydrologic regimes. The most effective means for doing this has been to establish continuous mathematical relationships between elements of the hydrologic cycle. The operation of these mathematical relationships is observed and improved by using digital computers to carry the calculations forward in time. Computer storage and calculating speed compensates for the complex effects of time dependence in hydrologic relationships. As mathematical relationships are developed, every attempt is made to realistically reproduce physical processes in the model. Experimental results and analytic studies are used wherever possible to assist in defining the necessary relationships.

Precipitation and potential evapotranspiration are the basic inputs for these digital models and actual evapotranspiration, streamflow, and soil moisture levels are generally obtained as output. Digital computer calculation is inherently flexible and many other items of input and output are often used. Calculations are made on selected time intervals and are carried continuously, whether or not precipitation is occurring, to simulate the entire spectrum of watershed behavior.

Comprehensive digital simulation models of the hydrologic cycle that generate streamflow, actual evapotranspiration, and related data directly from meteorological inputs are products of the computer revolution and as such have a short history. Professor Linsley at Stanford was responsible for one of the first studies that developed a simplified digital computer model of the hydrologic cycle[1]. This study encouraged further work and led to the publication of the series of technical reports previously cited. A digital model programmed by Dawdy and O'Donnell to investigate the feasibility and efficiency of the automatic evaluation of model parameters has been reported,[5] and work is now in progress on digital models at several universities.

Prior to the introduction of stored-program computers, a manual procedure developed by Professor Sugawara[6] of the Institute of Statistical Mathematics at Kyoto University was perhaps the only comprehensive model in use. Professor Sygawara used linear storages to produce continuous streamflow hydrographs from rainfall.

Analog simulation models that parallel digital models are also under development, notably by Professor Bagley and others at Utah State University.[7] Many useful analog and digital simulation methods for major components of the hydrologic cycle have been developed. D. R. Rockwood of the Corps of Engineers developed digital simulation methods to monitor flow in the lower Columbia River,[8] and Professor Harder of the University of California at Berkeley has developed techniques for simulation of the movement of flood flows in rivers.[9] An interesting application of hybrid computation to the analysis of the movement of flood flows in the Kitakami River in Japan was recently published.[10]

While comprehensive simulation models are a recent development, they are broad in scope and are highly dependent on previous work in hydrology. Many authors have contributed ideas and methods of calculation that influenced the development of the current Stanford Watershed Model system, and it is hoped that these authors have been given proper credit through references in this report.

A definition of digital simulation and a description of the development of simulation models in hydrology is given in Chapter II. A summary of the major quantitative concepts and calculation techniques used by the watershed model is found in Chapter III. Chapter IV contains the mathematical descriptions of all model components and Chapter V describes operation and optimization techniques for the model. Chapter VI contains some examples of results and statistical comparisons of results to recorded streamflows. Chapter VII is a summary of some current applications of the general method in hydrology, and finally Chapter VIII reviews the progress of simulation techniques to date and discusses future prospects.

II. SIMULATION METHODS AND HYDROLOGIC MODELS

In this chapter digital simulation is defined and compared with analog and physical model methods of simulation. The development of hydrologic simulation models is then described together with the philosophy, advantages, and limitations of these models for hydrologic studies.

Simulation Methods

Simulation is the indirect investigation of the response or behavior of a system. Three general types of simulation are used; physical models, analog models, and digital models.

Physical models have been used for investigation of hydraulic and hydrologic phenomena for many years. Analog models have also been extensively used. An analog model is a mechanical or electrical device that is constructed to have characteristics that are equivalent to the characteristics of the system under study. An analog component generally gives an exact representation of some mathematical relationship, and the process of developing an analog begins with the mathematical representation of a system. Analog components that will reproduce the relationships are then devised.

Digital simulation is a relatively new method for the investigation of the behavior of systems. Its major advantages are convenient high speed input and output and a lack of dependence on hardware. Digital simulation is based on digital computer programs, and these may seem at first glance to be rather far removed from physical or analog models. However, digital programs are mathematical representations, and when these reproduce components of a physical system the entire program becomes a model of the system. In digital models the mathematical representations may contain parameters which can be altered to represent any desired conditions in the system.

All simulation methods share certain characteristics and can often be used for similar purposes. They are more simply operated or observed than the physical systems they model. Time scales are most often compressed. Reproduction of years of observations in the physical system may require only a few minutes of simulation.

In digital simulation a physical system is analyzed and expressed as a collection of mathematical terms and parameters, and the mathematical representations are improved and verified by simulating system behavior with known input and output. This is continued until the simulation model is judged to be an adequate representation of the physical system.

The process of trial, error, and improvement has been widely used to develop concepts in the physical sciences where complex natural phenomena are studied, but digital computers have introduced an important new dimension through enormous changes in the time and cost of computations. In hydrology masses of data on temperature, precipitation, evapotranspiration, and streamflow are available to the analyst. Numerous theories can be developed relating processes in the hydrologic cycle. In the past the logistics of calculation have largely prevented evolutionary development of these theories. Well conceived ideas were tested only on a very limited scale and local or regional procedures were the result. The necessity for verification retarded general theoretical development since adequate tests of detailed quantitative relationships were impossible with manual calculations.

Simulation of any physical system is dependent on the accuracy of data about the system. Simulation of streamflow is difficult or impossible if some minimum amounts of reliable data are not available. Digital computers augment but do not substitute for analysis, laboratory experimentation, or intelligent judgement in the formulation of mathematical representations of systems.

With proper program design digital computers will indicate the adequacy or inadequacy of system parameters, and iterative loops may be used to find the optimum values of these parameters. However, finding optimum parameters for a given mathematical representation does not imply that the mathematical representation is itself optimum. Digital simulation cannot be expected to automatically develop optimum mathematical representations; this remains the work of the analyst. The analyst is free to devote much more time to the improvement of mathematical representations and can determine the relative merits of different representations of system components. Thus, in time adequate mathematical representations can be developed, even for complex systems.

The Development of Hydrologic Simulation Models

A quantitative hydrologic model must continuously simulate the major processes and interactions in the system that it represents. Rainfall, evapotranspiration, streamflow, and groundwater accretion must all be accounted for or evaluated. A workable structure is developed from the analysis of published research reports, analytical study of components, and repeated computer trials with physical data. The model develops by defining and improving quantitative relationships between hydrologic parameters and variables.* At any stage in this development the following questions may be asked:

"Is element A actually related to element B as the structure shows?"

"Could element C be eliminated without affecting the results?"

"Is the structure unique, and if not, what purpose is served by developing the model?"

A hydrologic model is nothing more than a collection of quantitative hydrologic concepts that are given mathematical representations. If each of these concepts is a well established physical law that has an exact mathematical representation, and if every physical component of the watershed is present in the model, the entire model structure would be unique and all physical processes in the watershed could be accurately simulated. Such a model would be most instructive, implying absolute knowledge of the hydrologic cycle, but would have one important disadvantage. Prohibitive amounts of input data would be required, far beyond practical limitations even for small experimental plots.

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* A parameter as used in this report is a constant value or collection of constant values that are used as indices in a mathematical representation in the model. A variable is used to represent physical quantities whose values are determined from operation of the model.

A useful quantitative description of the hydrologic cycle parallels a good qualitative description. The principal components and relationships in the hydrologic cycle must be selected to reduce a qualitative description to an acceptable level of complexity. This selection of principal components and relationships is also necessary for a quantitative description even though the selection may infringe upon both accuracy and uniqueness. The selection does not, however, preclude development of a hydrologic model that operates using physically relevant components and moderate quantities of input data, and accurately represents a broad range hydrologic behavior. A practical hydrologic model that is a skeleton of the hypothetical "absolute knowledge" model is the goal of digital hydrologic simulation.

Some criteria or requirements for a hydrologic model can be listed.

(1) The model should represent the hydrologic regimes of a wide variety of streams and rivers with a high order of accuracy.

(2) It should be easily applied to different watersheds with existing hydrologic data.

(3) The model should be physically relevant so that estimates of other useful data in addition to streamflow, such as overland flow or actual evapotranspiration, can be obtained.

The purpose of requirement (1) is obvious. Since the model is not strictly unique the adequacy and relevance of the concepts that it contains are supported only to the degree that simulation results indicate. To satisfy requirement (2) a dimensionless structure with a minimum number of independent parameters is necessary. This type of structure allows parameters to be developed rapidly, and is also best suited for selecting parameters to extend simulation into ungaged areas. Requirement (3) is necessary if improved hydrologic concepts are sought. One would expect that all models that satisfy requirements (1) and (3) would be based on similar quantitative concepts since in operation they would generate nearly identical physical data such as continuous watershed infiltration. Reliable and general quantitative methods of analysis are clearly desirable, particularly since digital computers now permit the wide use of these methods in practice.

When a model accurately reproduces the hydrologic cycle, the data input or individual model parameters can be varied, and the effect of these variations on the output can be examined. For example, the shape of the overland flow hydrographs can be altered by assuming increased roughness during overland flow, and the effects of these changes can be traced in the outflow hydrographs of the watershed. Similarly, the proportion of impervious area in a model can be changed and the changes in flow frequency that result can be studied. Output from runs that illustrate these applications of simulation are included in later chapters.

III. THE HYDROLOGIC CYCLE

The hydrograph of streamflow is the end product of the variable time and areal distributions of precipitation, evapotranspiration, physical watershed characteristics, and soil moisture conditions that are shown schematically in Figure 3.1.

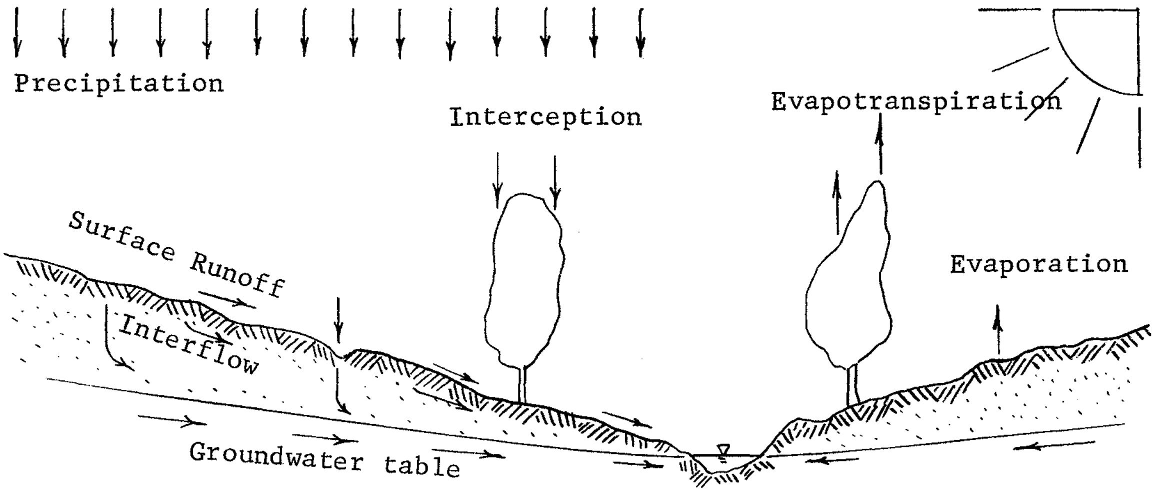


Figure 3.1

In this chapter the main quantitative concepts used in the watershed model are developed and discussed. Explanations of some related processes and mathematical representations for all processes are found in Chapter IV.

It is useful to separate land surface effects such as infiltration, interflow, and surface runoff from the effects of storage and flow in the channel system. When this is done the roles of the land surface and channel systems in watershed behavior can be examined in detail.

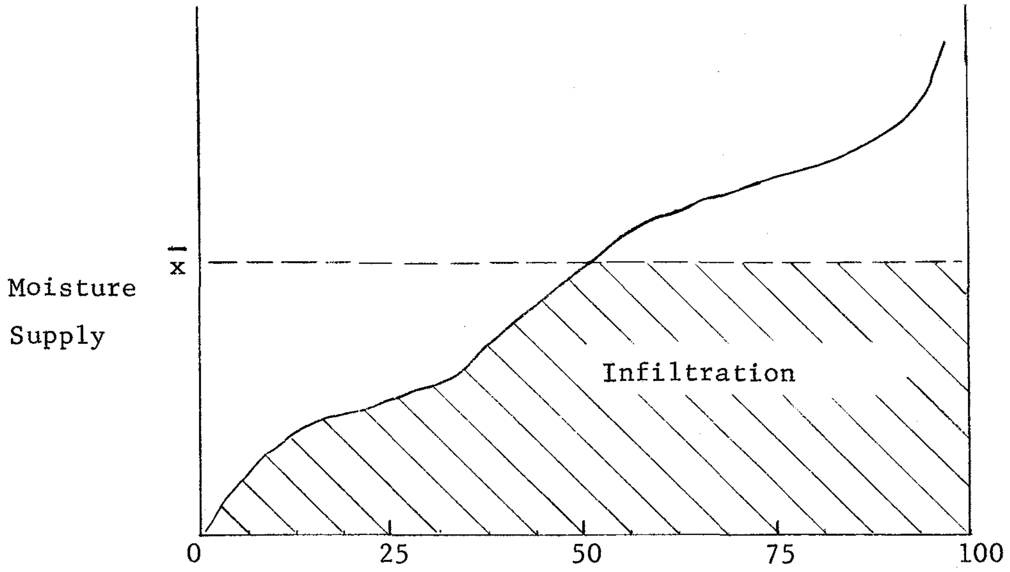
The Land Surface

The reaction of the land surface to rainfall will generally determine the volume of streamflow. Infiltration, storage in surface depressions, overland flow, and interflow all interact at the land surface. These processes are responsible for the complexity of hydrologic behavior and the widespread reliance on indirect or empirical methods for determining runoff volumes.

Infiltration Infiltration is the key process at the land surface. Water may infiltrate immediately from rainfall into the soil profile, or it may flow into temporary storages and infiltrate later. Storage in the soil profile is large but direct infiltration into this storage occurs at relatively low rates. Delayed infiltration complements direct infiltration and occurs when water flows into temporary storages of limited capacity, such as surface depressions and soil fissures. This water will later infiltrate or evaporate.

The interaction of the direct and delayed processes of infiltration during rainfall is of major importance. As rainfall begins, flow enters soil fissures, loosely packed surface soil, and surface depressions. Inflow to these elements can occur at very high rates, and their operation is effectively independent of rainfall rates. If heavy rainfall continues, the temporary storages fill and become much less effective, and overland flow, subject to direct infiltration begins to occur. When this condition is reached, direct infiltration throughout the watershed controls runoff volumes, and areal variations in infiltration capacity in the watershed will strongly influence watershed behavior.

Areal variations in infiltration capacities are illustrated in Figure 3.2 by plotting a cumulative frequency distribution of infiltration capacity. This curve would result if a large number of simultaneous infiltrometer measurements were made and plotted to show the percentage of watershed area with an infiltration capacity equal to or less than the measured values. Since infiltration capacity changes with time, this curve is time dependent and will be applicable only at a point in time, or for some short time interval.



Percent of Area with an Infiltration Capacity
Equal to or less than the Indicated Value.

Figure 3.2

The disposition of rainfall at any point in the watershed depends on the local infiltration rates. For example, assuming the mean moisture supply at the surface is $\bar{x}$ inches during some time interval, and the cumulative frequency distribution of infiltration capacity is as shown in Figure 3.2, the total volume of infiltration will be proportional to the shaded area. Note that the total volume of infiltration will increase as the moisture supply increases. The remaining area, between the moisture supply line and the infiltration capacity curve, represents the volume of water that is free to move toward stream channels as overland flow. This volume may not reach a stream channel since it may be diverted to temporary storages, and it remains subject to infiltration.

The shape of the cumulative distribution in Figure 3.2, assuming that a predominant shape exists, has not been established. Some indications

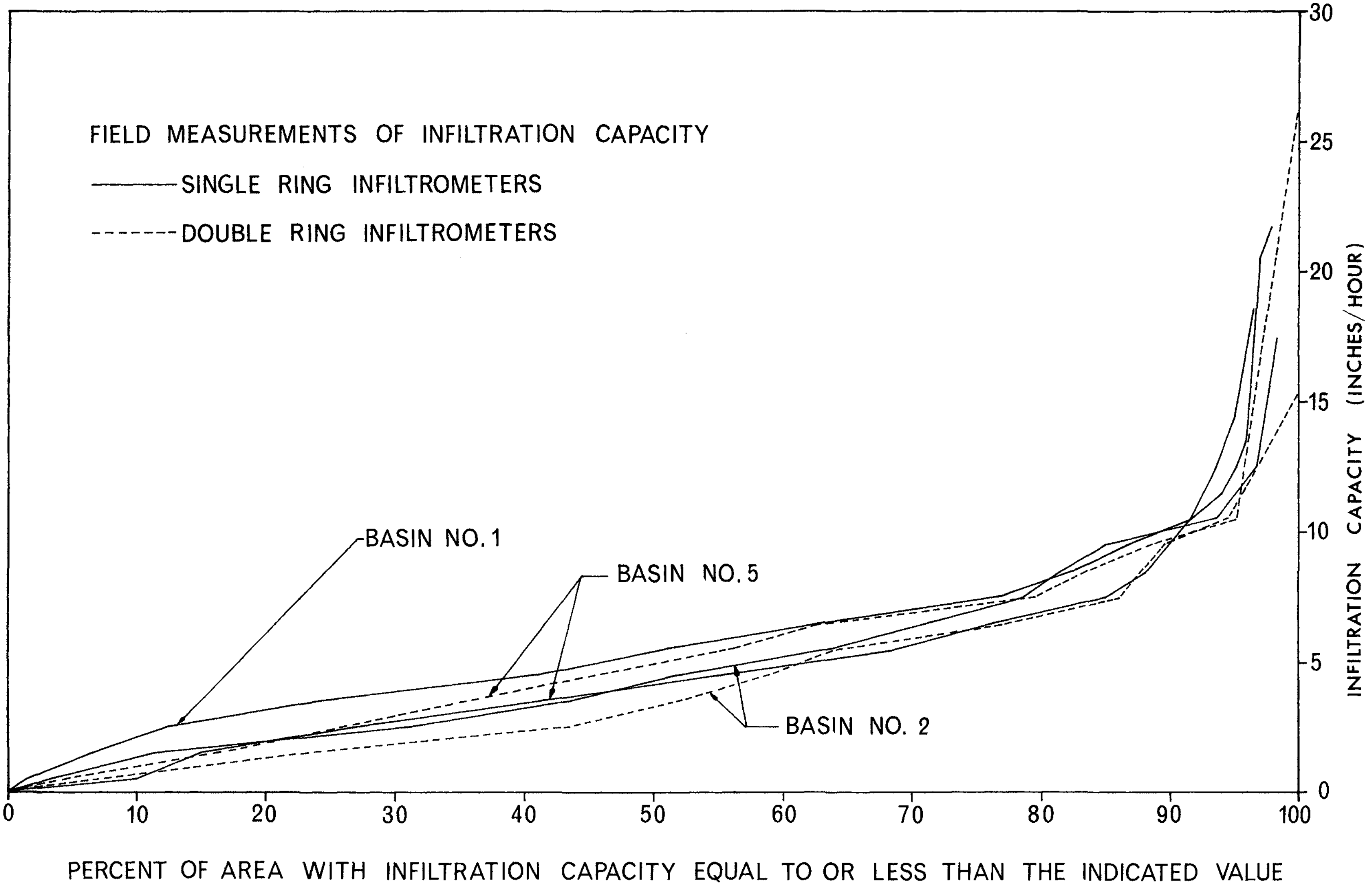

FIELD MEASUREMENTS OF INFILTRATION CAPACITY
SINGLE RING INFILTROMETERS
DOUBLE RING INFILTROMETERS
BASIN NO. 1
BASIN NO. 5
BASIN NO. 2
0
10
20
30
40
50
60
70
80
90
100
PERCENT OF AREA WITH INFILTRATION CAPACITY EQUAL TO OR LESS THAN THE INDICATED VALUE
0
5
10
15
20
25
30
INFILTRATION CAPACITY (INCHES/HOUR)

Figure 3.3

are provided by infiltrometer tests although applicable data is limited. Figure 3.3 shows cumulative frequency plots of infiltration capacities derived from data reported by R. H. Burgy and J. N. Luthin[11] for three small plots each only 40 by 20 feet. A large variation in the measured infiltration capacities was found even though the soil type was reported to be a uniform Yolo silt loam, measurements were taken at a spacing of only five feet, and each of the plots was flooded prior to the tests in an attempt to establish uniform soil moisture conditions. Considerable areal variations in measured infiltration capacities have also been reported from tests using the Geological Survey Rainfall-Simulator Infiltrometer.[12] These results were not plotted in Figure 3.3 since the relative locations of the measurements were not known.

Statistically derived evidence that infiltration rates during natural storms do increase as rainfall intensity increases has been reported.[13] Similiar evidence indicates that runoff often originates from only a small portion of the total watershed area.[14] The effects of variations in infiltration, as represented in Figure 3.2 or 3.3, provide an explanation for these observations. An extensive series of measurements would be needed to experimentally document the changes in shape of a cumulative frequency distribution of infiltration capacity for any natural watershed. For watersheds in heavily forested areas representative measurements would be difficult to obtain. However, it is possible to infer watershed infiltration characteristics from simulation.

The commonly used "infiltration capacity curve" represents the change in point or average watershed infiltration rate with time, if the supply rate is greater than the capacity. An infiltration capacity selected from such a curve is generally subtracted from the current rate of rainfall. Subtracting a single value from rainfall for the current average rate of infiltration, is equivalent to an assumption of uniform infiltration capacity throughout the watershed, or an assumption that the proper shape of the curve in Figure 3.2 or 3.3 is a horizontal line. The time dependence of an "infiltration capacity curve" considered in terms of Figure 3.2 or 3.3 is equivalent to a change in only the position of this horizontal line. The calculation of infiltration should be continuously accurate to avoid introducing errors in subsequent calculations. Thus, a relatively complex calculation based on the cumulative infiltration capacity is used in the watershed model to simulate natural behavior as closely as possible.

Overland Flow The movement of water in surface or overland flow is another important land-surface process. Interactions between overland flow and infiltration need to be considered since both processes occur simultaneously. The variations in rates of infiltration described above allow overland flow in areas with low infiltration while preventing overland flow in other areas. During overland flow water held in detention storage remains available for infiltration. Surface conditions such as heavy turf or very mild slopes that restrict the velocity of overland flow tend to reduce the total quantity of runoff by allowing more time for infiltration. Short high intensity rainfall bursts are attenuated by surface detention storage reducing the maximum outflow rate from overland flow. Thus, simulation of the infiltration-overland flow processes requires continuous estimates of detention storage as well as the continuous outflow rates from overland flow. The calculation methods used for overland flow should yield results that can be compared with the well known investigations of C. F. Izzard[15,16], and with other experimental and analytic results.

A wide range of methods for the calculation of unsteady overland flow were considered. The only rigorous general methods for simulating unsteady overland flow are finite difference techniques for the numerical solution of the governing partial differential equations, the continuity and momentum equations[17, 18, 19]. However, these methods have a major disadvantage for continuous simulation since substantial amounts of computer time are needed. In a natural watershed there are areal variations in the amount of runoff moving in overland flow due to areal variations in infiltration rates. Average values are used in calculations for the major surface parameters such as the length and slope of overland flow. Thus, while watersheds are broken up into segments and reasonably accurate methods are justified, the accuracy to be gained by using finite difference methods for overland flow is still subject to question because of the limited accuracy of the basic data.

Approximations to simulate unsteady overland flows are difficult to devise since the basic nature of the flows is not well established[20]. For convenience, the flows can be described as laminar or turbulent based on undisturbed flow criteria, even though turbulence from raindrop impact clouds this distinction[21]. Undisturbed flow criteria indicate that transitions from laminar to turbulent flow could occur in overland flow in typical natural watersheds.

Adaptation for simulation of both laminar and turbulent range empirical equations was considered. The turbulent range equations were finally selected for adaptation since experimental measurements of surface detention show a marked change in regime as turbulence becomes dominant[22], and high intensity rainfalls often yield Reynolds numbers that indicate turbulent flows. Overland flows in natural watersheds tend to collect and move along preferred paths, and a turbulent range approximation can be more logically adjusted to account for this effect.

Continuous surface detention storage is calculated in the model. Since the volume of surface detention was successfully used as a parameter for the rate of discharge for overland flows in the laminar range[16], the volume of surface detention was selected as the logical parameter to relate to discharge in the turbulent range. Of course, no fixed relation exists between detention storage and discharge from overland flow when the flow is unsteady, but some useful approximations to natural behavior can be made.

The Chezy-Manning equation is used in Appendix A to derive a relation between surface detention storage at equilibrium, the supply rate to overland flow, Manning's n, and the length and slope of the flow plane. The amount of surface detention at equilibrium from Equation A24 is

$$D_e = \frac{0.000818 \quad i^{0.6} \; n^{0.6} \; L^{1.6}}{S^{0.3}} \qquad (3.1)$$

where D_e is surface detention in ft^3/ft, i is the supply rate in inches per hour, S is slope in ft/ft, and L is the length of overland flow in feet. The rate of discharge from overland flow based on the Chezy-Manning equation is

$$q = \frac{1.486}{n} \quad y^{5/3} \quad S^{1/2} \tag{3.2}$$

for q in $ft^3/sec/ft$, and where y is the depth in feet at the lower edge of the flow plane. Therefore, to calculate continuous overland flow from total surface detention storage the depth y must be related to surface detention storage. This is easily done at equilibrium where

$$y = \frac{8}{5} \quad \frac{D_e}{L} \tag{3.3}$$

but for other conditions some approximations are needed. In the unsteady overland flow sketched in Figure 3.4 three general conditions will occur. Initially, as rain begins, the depth of overland flow will be uniform along the flow plane. Therefore, at time (a) a transition from a uniform depth to an equilibrium profile is taking place. If rainfall continues, the equilibrium profile is reached at time (b), and when rainfall stops recession flow occurs (c) from water in storage.

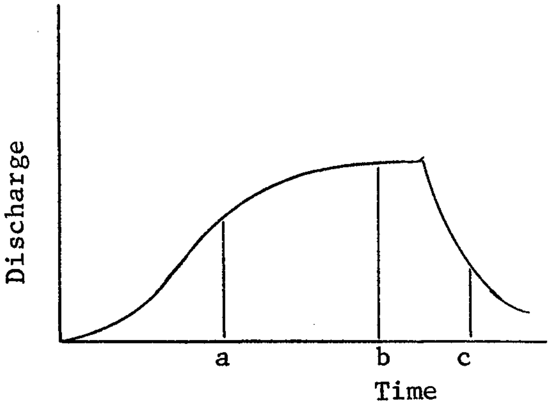


Figure 3.4

The minimum value of y must equal the mean depth D/L where D is the current surface detention storage in ft^3/ft. Therefore, y must be in the range

$$\frac{D}{L} \leq y \leq \frac{8}{5} \quad \frac{D_e}{L} \tag{3.4}$$

The current detention storage D, divided by the detention storage required at equilibrium D_e for the current rate of inflow, is used as an index to the distribution of water in the overland flow plane. The most satisfactory empirical relationship found between outflow depth and detention storage for reproducing experimental hydrographs is

$$y = \frac{D}{L} \left(1.0 + 0.6 \left(\frac{D}{D_e}\right)^3\right) \qquad (3.5)$$

Substituting Equation 3.5 in Equation 3.2 the rate of discharge from overland flow in ft^3/sec/ft is

$$q = \frac{1.486}{n} S^{1/2} \left(\frac{D}{L}\right)^{5/3} \left(1.0 + 0.6 \left(\frac{D}{D_e}\right)^3\right)^{5/3} \qquad (3.6)$$

where D_e is a function of the current supply rate to overland flow and is calculated from Equation 3.1. During recession flow when D_e is less than D the ratio D/D_e is assumed to be one.

Discharge from overland flow in inches per hour per unit area is often of interest for direct comparison to rainfall rates. For q in inches/hour/unit area Equation 3.6 can be modified to:

$$q = \frac{64200 \, S^{1/2}}{nL} \left(\frac{D}{L}\right)^{5/3} \left(1.0 + 0.6 \left(\frac{D}{D_e}\right)^3\right)^{5/3} \qquad (3.7)$$

A discussion of the response from the approximations just described is found in Appendix A, but two examples of output are included here. Comparisons between Izzard's experimental results, the watershed model subroutine, and finite difference methods are shown in Figures 3.5 and 3.6. The finite difference response in Figure 3.5 is taken from a report by Morgali[17], and the response in Figure 3.6 is taken from a recent report by Schaake[19].

In the watershed model the overland flow hydrographs are added to interflow and ground water flow hydrographs to form the total channel inflow hydrograph. This channel inflow hydrograph is, therefore, a function of land surface and rainfall characteristics and has the advantage, compared to ordinary streamflow hydrographs, of being independent of the channel system. This is a characteristic that can sometimes be used to advantage.

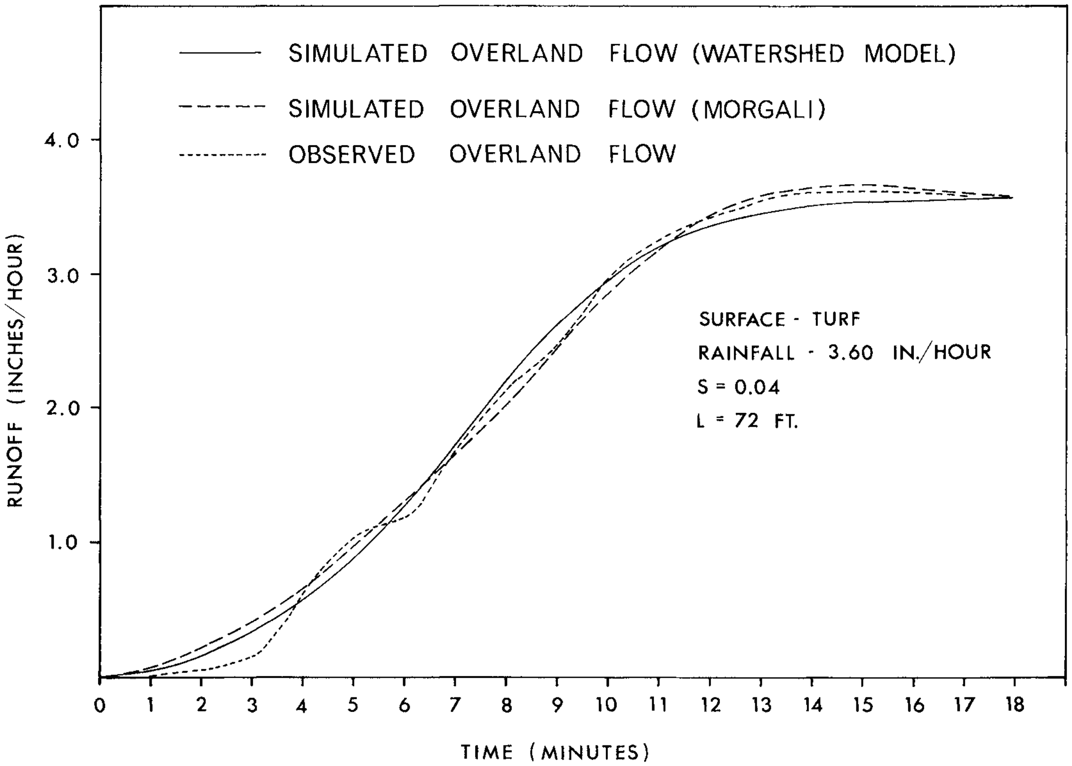


Figure 3.5

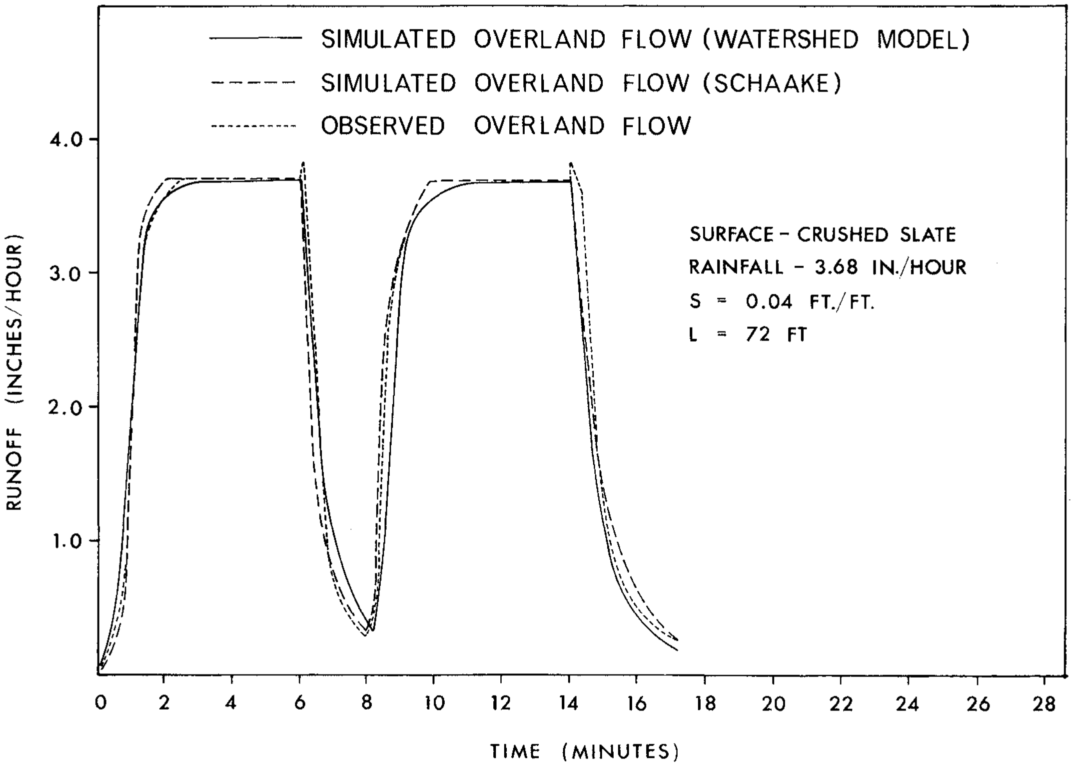


Figure 3.6

Evapotranspiration The volume of water that leaves a watershed as evaporation and transpiration exceeds the total volume of streamflow in most hydrologic regimes. Continuous estimates of actual evapotranspiration must therefore be found by the model. There are two separable issues involved in estimating actual evapotranspiration. Potential evapotranspiration must be selected, and actual evapotranspiration must be calculated as a function of moisture conditions and the potential evapotranspiration.

Potential evapotranspiration is assumed to be equal to lake evaporation estimated from U. S. Weather Bureau Class A pan records.[23] This procedure is more convenient than a theoretical approach[24] since input requirements are less stringent. A single variable, adjusted pan evaporation data, serves a purpose that would otherwise require input of several meteorologic variables. Provision is made in the standard input to modify the assumption that potential evapotranspiration and lake evaporation are equal.

The relationship of actual evapotranspiration to potential evapotranspiration over large areas should logically be a function of moisture conditions. Even if transpiration from vegetation is independent of soil moisture until the wilting point is reached, variable soil moisture will cause wilting in some parts of a watershed but not in others. Evaporation from soil, a component of the total process, is dependent on moisture conditions.

When near surface storage is depleted, the concept of evapotrans-transpiration opportunity is used to calculate actual evapotranspiration. Evapotranspiration opportunity is defined as the maximum quantity of water accessible for evapotranspiration in a time interval at a point in the watershed. It is analogous to infiltration capacity and would have a cumulative distribution similar to that in Figure 3.2. The cumulative evapotranspiration opportunity curve will be a function of watershed soil moisture conditions, and will give estimates of actual evapotranspiration for any quantity of potential evapotranspiration, just as the cumulative infiltration capacity curve estimates net infiltration for any moisture supply. A sketch of this concept and the functions used are given in Chapter IV.

The Channel System

The measured outflow hydrograph from a watershed reflects the importance of the land surface effects relative to the time delay and attenuation in the stream channel system. This is illustrated by the following two figures.

Figure 3.7 shows overland flow and watershed outflow hydrographs for a small turfed watershed near Stanford with a total area of 166 acres and a main channel length of 7500 ft. Figure 3.8 shows overland flow and watershed outflow hydrographs for Beargrass Creek near Cannons Lane, Kentucky, an area of 18.5 square miles.

These figures illustrate the purpose of dividing the flow calculations into land surface and channel system phases. In very small natural watersheds, hydrograph shape is primarily dependent on the shape of the channel inflow hydrograph, and therefore on land surface characteristics. The outflow hydrograph results from channel inflow distributed throughout the watershed, but in small undeveloped watersheds (Figure 3.7) the outflow hydrograph remains nearly equal to the hydrograph of total channel inflow. As watershed area increases, storage and flow times in the channel system become large compared to those in overland flow, and the channel system becomes the dominant factor in the shape of the outflow hydrograph. The separation of overland and channel flow helps to identify the relative importance of the various parameters in a given watershed. Channel system parameters, for example, could not be expected to indicate the time distribution of runoff from a small turfed watershed.

Wave movement in natural channels is a complex phenomena, but stream channel response to a given pattern or sequence of channel inflow is relatively stable. The same outflow hydrograph is observed each time a given channel inflow sequence recurs. This stable behavior is a welcome contrast to that found in many hydrologic processes.

Three general approaches to the problem of routing in natural streams have been investigated. An analytic approach replaces the natural channel system with an idealized channel system and seeks mathematical solutions of the continuity and momentum equations for

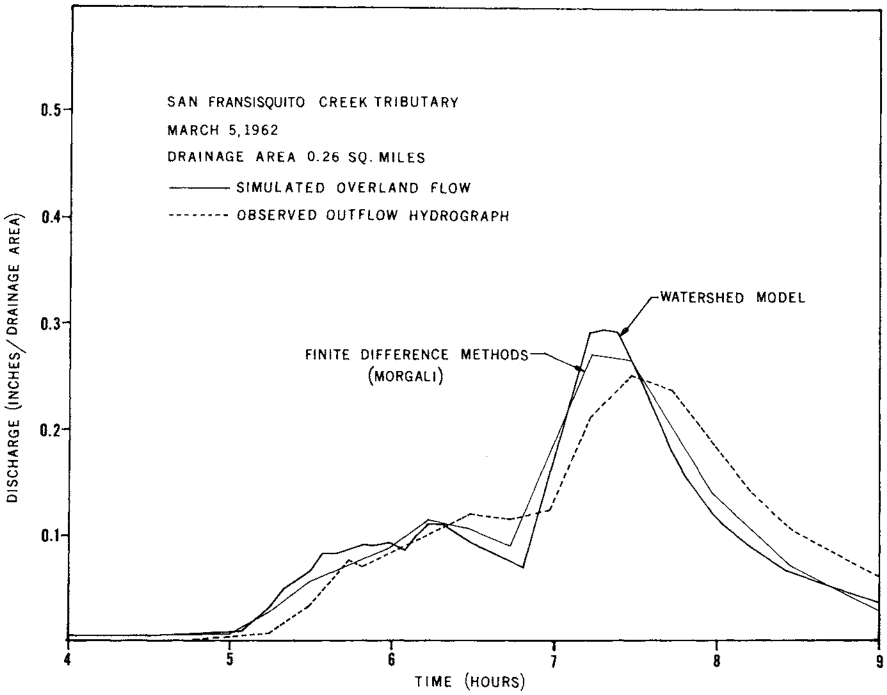


Figure 3.7

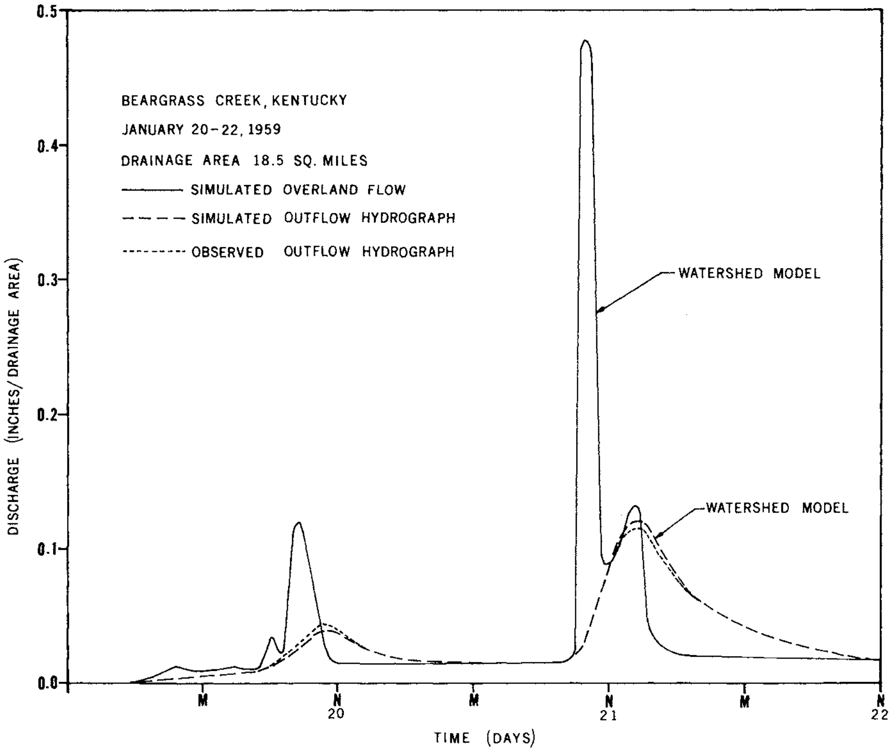


Figure 3.8

specific patterns of channel inflow[25]. This type of approach is necessarily limited in generality, but has provided some controls for the more general numerical and empirical methods.

The numerical or finite difference methods can also be applied for channel flow[17,18]. Actually, these methods were first adopted to follow the movement of flood waves in sections of large rivers[26], and were later used in smaller channels and for overland flow. Finite difference methods for channel routing are not used in the watershed model even though they are the most general physically based methods available for simulating unsteady open channel flows. Input requirements and calculating time are the major obstacles to the use of finite difference methods for channel system routing, and more rapid empirical methods are substituted. However, the finite difference methods do provide useful controls for empirical methods and may in the future be adopted more directly into general simulation models.

There are many empirical routing methods in common use. The well known unitgraph, or unit hydrograph method relates the outflow hydrograph shape to "rain excess", or rainfall less infiltration at the land surface. A unitgraph combines the overland flow and channel flow systems although this is of no consequence in larger watersheds. The Muskingum routing method[27], and the modified storage routing methods proposed by Kalinin and Milyukov[28], are typical of empirical and semi-empirical calculations of the movement of flow through a short reach of a channel system where lateral inflow is relatively small. Applications of storage routing in successive reaches of the channel system could be used to calculate the outflow hydrograph from channel inflow hydrographs, but a less cumbersome method is desirable.

A simple empirical method, that has been modified and adapted in the watershed model, was devised by C. O. Clark[29]. Clark assumed that the time-area curve* for a watershed must represent an outflow hydrograph

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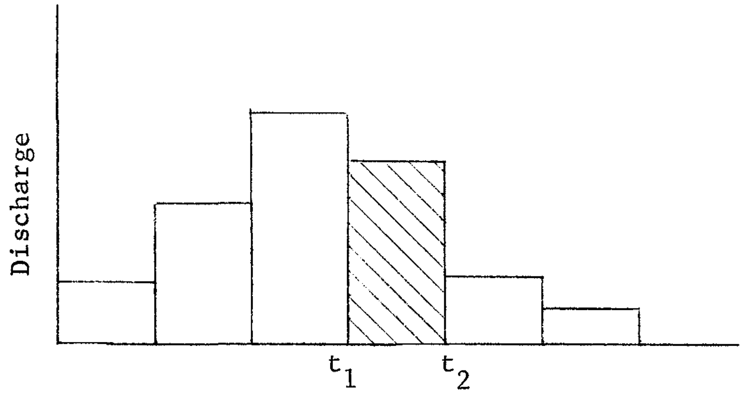
*A time-area curve is a plot of time of flow from any point in a watershed to the outlet, neglecting attenuation due to land surface and channel storage.

from a short (instantaneous) rainfall neglecting all attenuation due to storage, and proposed routing the time-area curve through a single reservoir storage to form an outflow hydrograph. This is the outflow hydrograph for a storm of zero length and using unitgraph terminology would be called an instantaneous unit hydrograph.

Three modifications are made in this procedure. First, since the land surface and the channel systems are separately modelled, the time-area curve is redefined to represent the time of flow in the channel system only. To avoid confusion since the curve no longer represents area, it is called a channel time-delay histogram. This curve may be found by planimetering contributing areas, estimating channel flows at successive points in the stream channel system, and calculating the time of flow to the outlet of the watershed. The channel time-delay curve calculated above would represent the response to an instantaneous surge of inflow to the channel system, and needs to be modified to represent inflow of finite duration. The channel time-delay curve shown in Figure 3.9 as a histogram, represents the response of the watershed in Figure 3.10 to an instantaneous channel inflow. Runoff from the cross-hatched channel segments in Figure 3.10 produces the element of the total



Channel Flow Time to Outlet

Figure 3.9

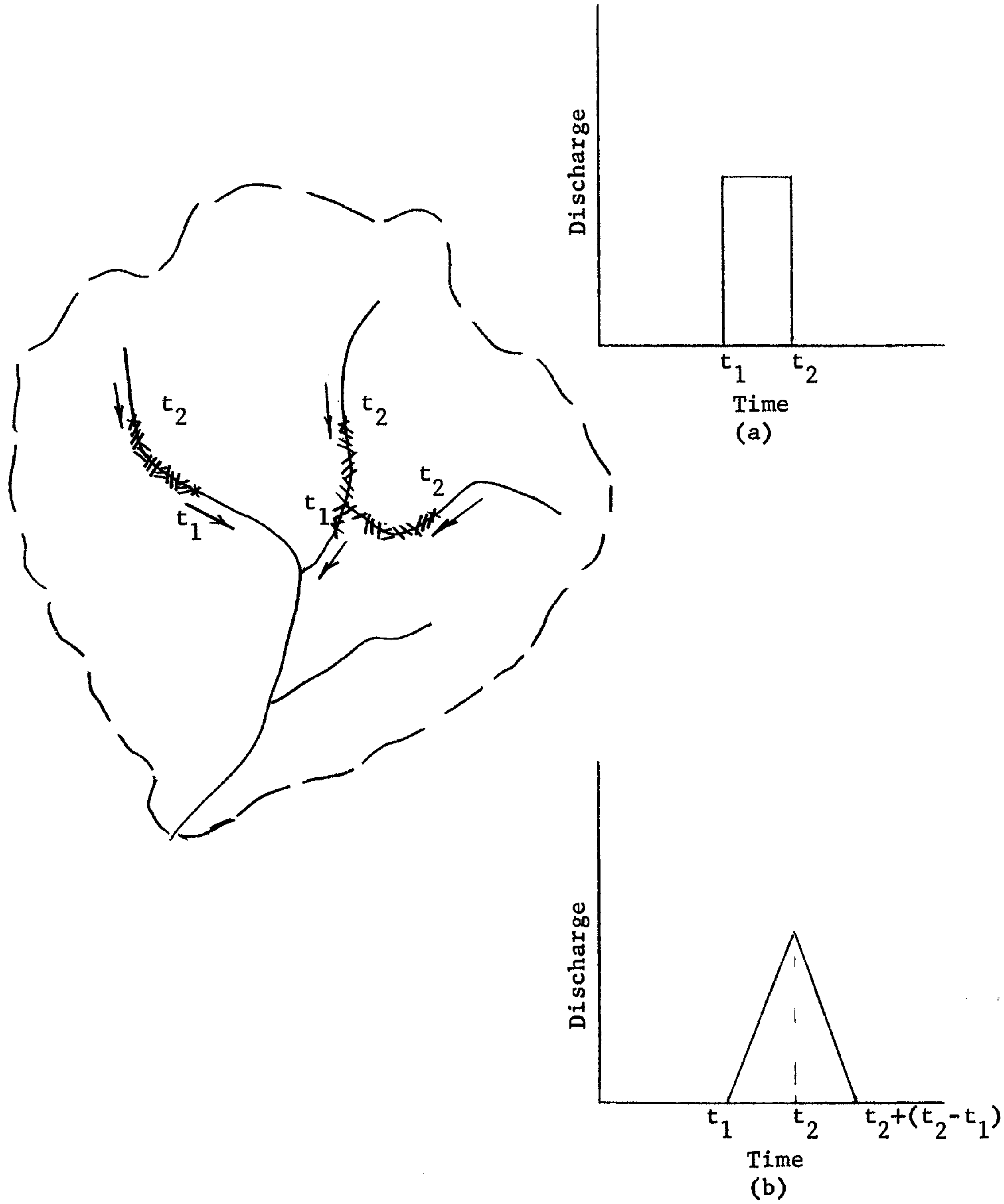

t2
t1
t2
t1
t2
Discharge
t1
t2
Time
(a)
Discharge
t1
t2
t2+(t2-t1)
Time
(b)

Figure 3.10

histogram between times t_1 and t_2 that is repeated in Figure 3.10(a). If the instantaneous channel inflow is replaced by a uniform inflow with a duration of $t_2 - t_1$ the response from the cross-hatched channels becomes as shown in Figure 3.10(b). The response starts at time t_1 and builds to a maximum at t_2, when flow entering all of the cross-hatched channels is observed at the outlet. The response decreases to zero at time t_2 plus t_2-t_1, the time of inflow. Applying the same procedure to the other elements in the histogram in Figure 3.9, Figure 3.11 becomes the channel time-delay histogram modified to represent response to an inflow with a duration equal to the time increment used to plot the histogram.

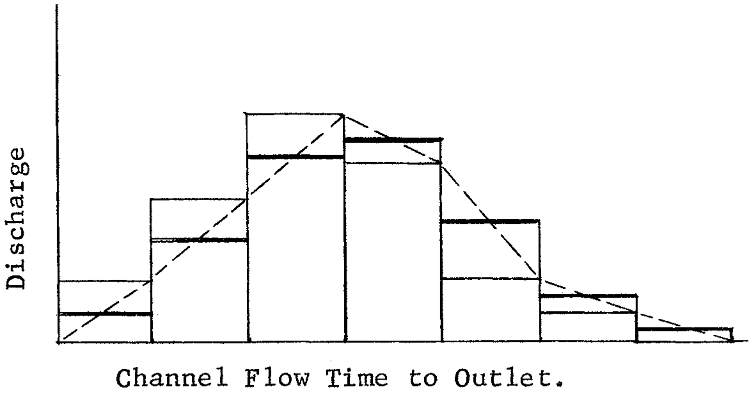


Figure 3.11

Finally, in Clark's procedure reservoir routing was used to represent the attenuation due to storage in the stream channels. The same method is used in the watershed model but this calculation is delayed. A histogram, or a collection of histograms similar to Figure 3.11, are used to delay and add the continuous channel inflows from each watershed segment prior to channel attenuation. This allows the opportunity where necessary to vary the amount of channel attenuation, most commonly as a function of total discharge. The time-delay histograms can also be modified as a function of discharge when necessary. A numeric example of the derivation of a time-delay histogram is shown in Chapter V.

The major advantage of the empirical routing method above is efficient program coding that allows simultaneous output hydrographs at several points in a watershed. Channel inflows can be simulated from an unlimited number of recording and storage rain gages.

IV. A GENERAL SIMULATION MODEL

This chapter develops the mathematical representations of the elements required to represent the hydrologic cycle. The mathematical functions used must be considered experimental, but they are the result of a continuing research program to develop a rational and detailed quantitative model of the hydrologic cycle, and they are based on extensive tests in more than forty diverse watersheds.

The first section of the chapter contains a flow chart and a brief description of the overall model. The second section contains the mathematical representations used for the land surface processes; and the third section contains mathematical representations for channel system processes. The fourth section contains a brief summary of a subroutine used for snowmelt simulation.

Model Structure

The major elements of the watershed model are shown in Figure 4.1. The calculations represented in Figure 4.1 may be carried for any number of watersheds or segments of watersheds from any number of input stations. This brief description is concerned only with the operation of the model in a small watershed or watershed segment. An explanation of model operations for single and multiple gage input and output is included in Chapter V.

Precipitation and potential evapotransportation are the major data inputs. Additional meteorologic data are used if snowfall is significant. Calculations begin from known or assumed moisture conditions, and are continued until the input data is exhausted. Precipitation is stored in the snowpack and in three soil moisture storages.

The upper and lower zone storages, together with the groundwater storage, combine to represent variable soil moisture profiles and groundwater conditions. The upper- and lower-zone storages control overland flow, infiltration, interflow, and inflow to the groundwater storage. The upper zone simulates the initial watershed response to rainfall and is of major importance for smaller storms, and for the first few hours of larger storms. The lower zone controls watershed response to major storms

Watersheds usually contain rock outcrops and buildings or roads that are isolated and do not contribute directly to the channel system. Runoff from these impervious or barely impervious areas must flow over pervious land before reaching a stream channel and, therefore, remains a function of watershed soil moisture conditions. The reaction of these areas is represented by the direct infiltration functions in the model.

Infiltration The complex processes in infiltration at any instant are approximated by two interlocking calculations that continuously determine the direct infiltration into the soil profile, and the increases in temporary storages that result in delayed infiltration.

Available moisture is first subject to the operation of the cumulative watershed infiltration capacity functions that model overall watershed reaction, and govern direct flows into the long-term lower zone and groundwater storages. Water that remains in surface detention after direct infiltration is calculated, becomes subject to the operation of the upper zone storages. The upper zone is designed to simulate the diversion of overland flows into depression storage, soil fissures, and disturbed or dry surface soil. None of the soil moisture storages have fixed capacities. Additions to and losses from storages are determined from continuous dimensionless storage ratios to avoid discontinuous model response.

Direct Infiltration (Lower Zone and Groundwater Storage): Rainfall at the land surface will either move toward stream channels as surface runoff, or will infiltrate into the soil profile. The moisture supply available for infiltration in any time interval includes water in transient storage in overland flow. Infiltration capacity will vary throughout the watershed, and in Chapter III the cumulative distribution of infiltration capacity was introduced to simulate the effects of these variations on runoff and infiltration.

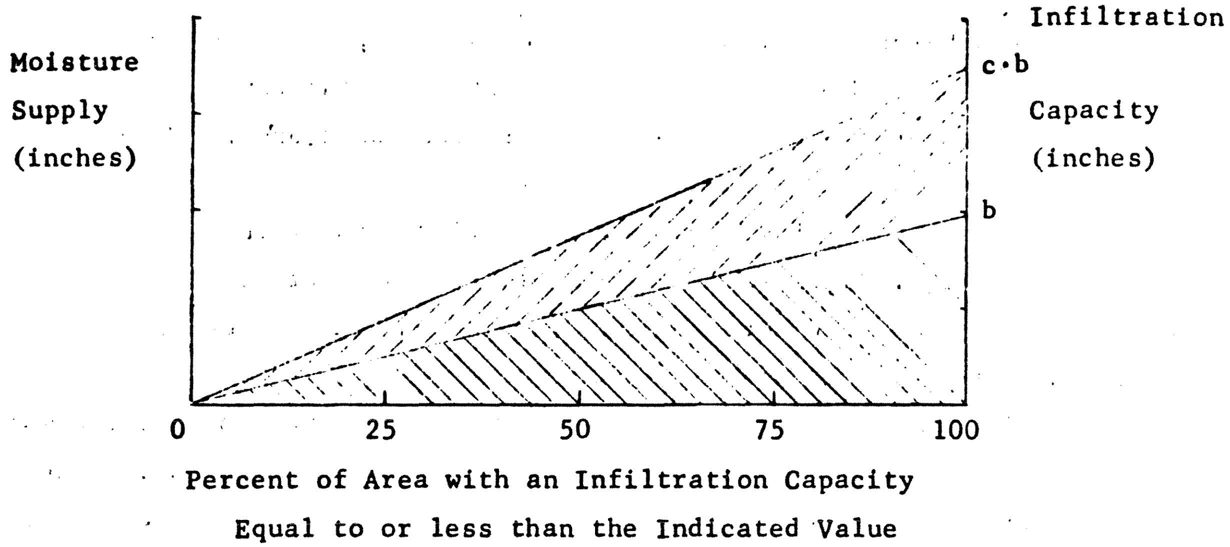


Figure 4.2

The cumulative frequency distribution of infiltration capacity in Figure 3.2 is assumed to be linear from zero to a maximum value as shown in Figure 4.2. Figure 4.2 also divides the infiltration capacity into two regions. In the region that is shaded with solid lines all infiltrated water is assumed to move into the lower zone and groundwater storages. In the region shaded with broken lines infiltration is assumed to contribute to interflow. Thus, the tendency for infiltrating water to become interflow is assumed to be proportional to the local infiltration capacity. The median direct lower zone and groundwater infiltration capacity is represented by (b/2), and the median total infiltration capacity is (c.b/2) where both c and b are nonlinear functions of moisture conditions in the lower zone storage.

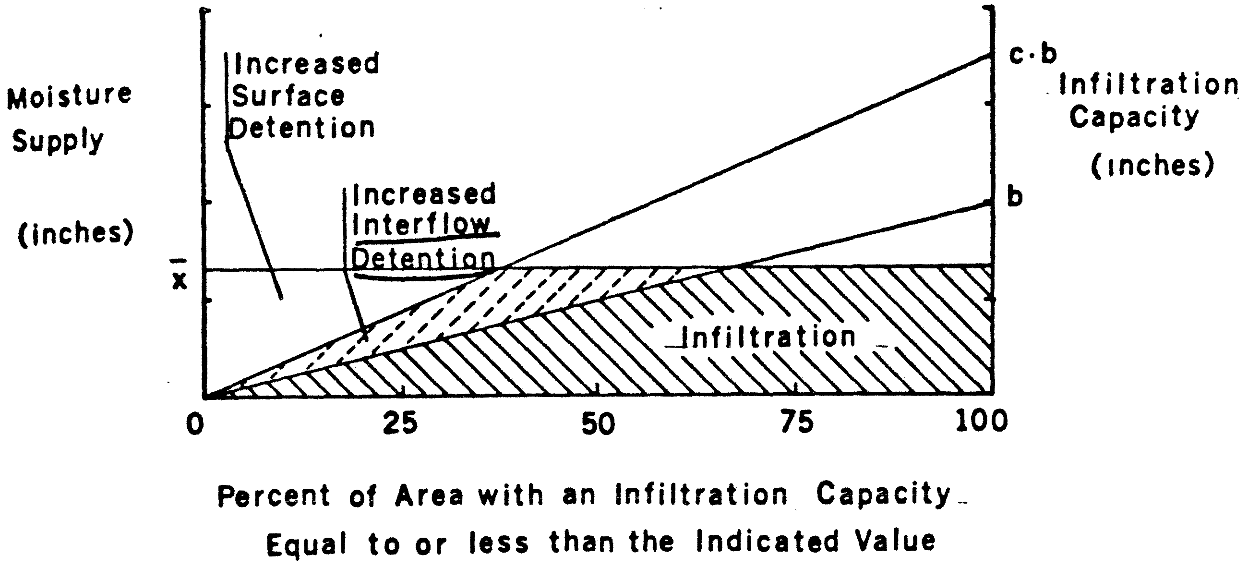


Figure 4.3

With these assumptions the reaction of a watershed to a moisture supply of $\bar{x}$ inches is shown in Figure 4.3. The quantity of infiltration and the increases in interflow and surface detention are shown in the figure. Functional relationships for land surface response for all values $\bar{x}$, c, and b can easily be found and a summary is given in Table 4.1.

Table 4.1 Land Surface Response as a Function of Mean Moisture Supply ($\bar{x}$)

Component	$\bar{x} < b$	$b < x < c . b$	$\bar{x} > c.b$
Net Infiltration	$x - \frac{\bar{x}^2}{2b}$	$\frac{b}{2}$	$\frac{b}{2}$
Increase in Interflow Detention	$\frac{\bar{x}^2}{2b}(1 - \frac{1}{c})$	$\bar{x} - \frac{b}{2} - \frac{\bar{x}^2}{2\ c.b}$	$\frac{b}{2}(c - 1)$
Increase in Surface Detention	$\frac{\bar{x}^2}{2\ c.b}$	$\frac{\bar{x}^2}{2\ c.b}$	$\bar{x} - \frac{c.b}{2}$
Percentage of Increased Detention Assigned to Interflow	$100(1 - \frac{1}{c})$	$100\left(1 - \frac{\bar{x}^2}{2\ c.b(\bar{x} - \frac{b}{2})}\right)$	$100\frac{(c - 1)}{(\frac{2\bar{x}}{b} - 1)}$

The functions in Table 4.1 give a smooth variation in model response as the amount of moisture supply varies. Figure 4.4 is an example of the variation in the components of land surface response as the moisture supply is increased.

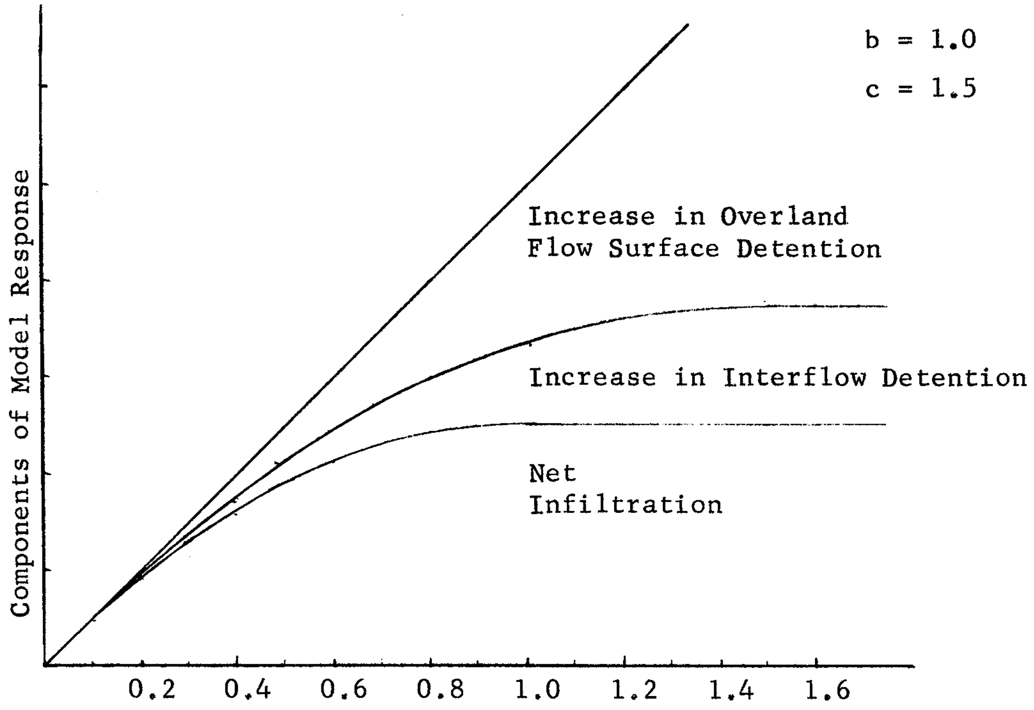


Figure 4.4

The quantity of net and lower zone or groundwater infiltration is determined by the current value of b. The current value of c alters outflow hydrograph shape or the time distribution of runoff by controlling the ratio of increments to surface detention leading to overland flow, and interflow detention. The magnitudes of the variables b and c in any time interval are functions of the current dimensionless lower zone storage ratio LZS/LZSN, and input parameters CB and CC. The quantity LZS is the current soil moisture storage in the lower zone. LZSN is a nominal storage level assigned by an input parameter that is approximately equal to the median value of lower zone storage. The quantity CB is an input parameter that assigns the overall level of net infiltration, while CC is an input parameter that assigns the level of interflow relative to overland flow.

Numerous computer trials in many different watersheds in the last few years resulted in adoption of the following relationships.

$$b = CB/2^{(4 \cdot LZS/LZSN)} \qquad (4.1)$$

when LZS/LZSN is less than one, and

$$b = CB/2^{(4.0 + 2 \cdot ((LZS/LZSN) - 1.0))} \qquad (4.2)$$

when LZS/LZSN is greater than one. The minimum value of b is 1/64 of CB and is reached when LZS/LZSN is 2.0.
The value of c is

$$c = CC \cdot 2^{(LZS/LZSN)} \qquad (4.3)$$

Sketches of these relationships when CB and CC equal one are shown in Figures 4.5 and 4.6.

Delayed Infiltration (Upper Zone Storage): Moisture that is not infiltrated directly will increase surface detention storage. The increment to surface detention calculated from Figure 4.3 will either contribute to overland flow and interflow or enter upper zone storage. Depression storage, and storage in highly permeable surface soils are modelled by the upper zone. Hence, the upper zone inflow rates are independent of rainfall intensity but upper zone storage capacity is low. Moisture is lost from the upper zone by evaporation and percolation to the lower zone and groundwater storages.

The following expressions are used to calculate the response of the upper zone storage. The upper zone has a nominal capacity given by the input parameter UZSN. The percentage P_r of a potential addition to overland flow surface detention that is held in the upper zone is a function of the upper zone storage (UZS) and the nominal capacity (UZSN), when the ratio (UZS/UZSN) is less than two.

$$P_r = 100 \left(1.0 - \left(\frac{UZS}{2 \cdot UZSN}\right) \cdot \left(\frac{1.0}{1.0 + UZI1}\right)^{UZI1} \right) \qquad (4.4)$$

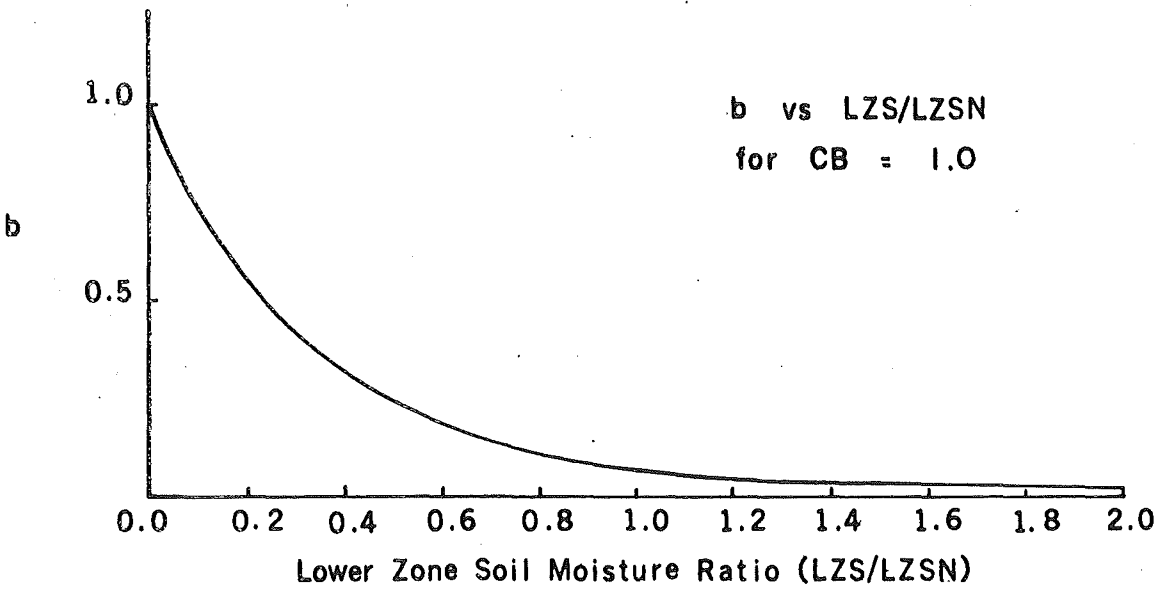


Figure 4.5

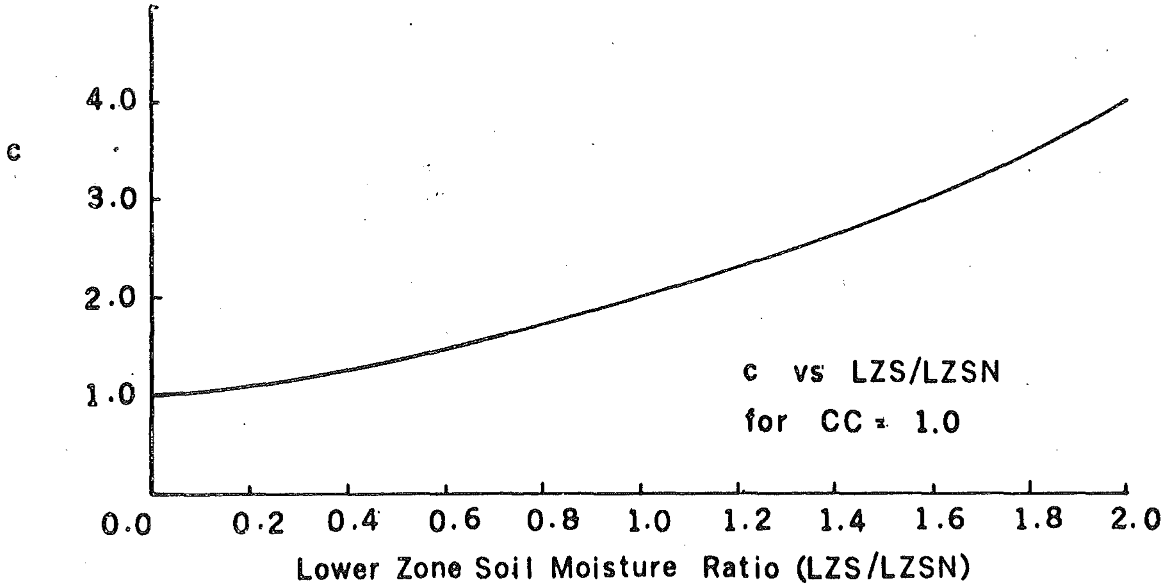


Figure 4.6

where UZI1 is

$$\mathrm{UZI1} = 2.0 \left| \left(\frac{\mathrm{UZS}}{2\cdot\mathrm{UZSN}}\right) - 1.0 \right| + 1.0 \qquad (4.5)$$

When UZS/UZSN is greater than two the percentage is given by

$$P_r = 100 \left(1.0 - \left(\frac{1.0}{1.0 + \mathrm{UZI2}}\right)^{\mathrm{UZI2}}\right) \qquad (4.6)$$

where UZI2 is

$$\mathrm{UZI2} = 2.0 \left| (\mathrm{UZS/UZSN}) - 2.0 \right| + 1.0 \qquad (4.7)$$

These relationships are plotted in Figure 4.7.

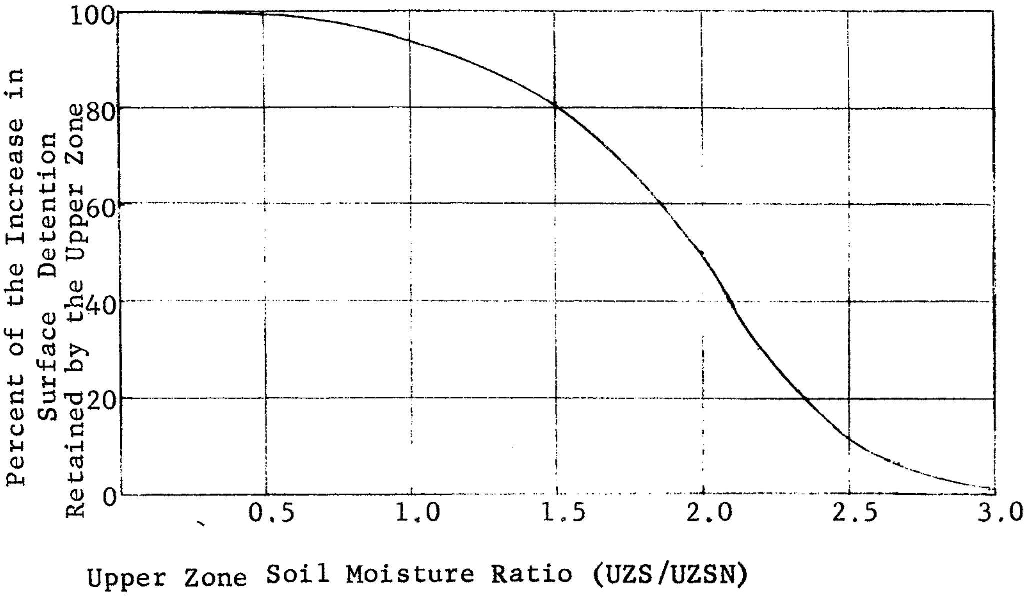


Figure 4.7

The upper zone storage prevents overland flow from a portion of the watershed depending on the value of the ratio UZS/UZSN, but since the nominal capacity assigned by the input parameter UZSN is low, the upper zone retention percentage decreases rapidly with early increments of accretion. Evapotranspiration and percolation remove water from the upper zone storage. Delayed infiltration or percolation occurs from the upper zone to the groundwater and lower zone storages when the upper zone storage ratio UZS/UZSN exceeds the lower zone storage ratio LZS/LZSN. This is calculated as

$$\text{PERC} = 0.003 \cdot \text{CB} \cdot \text{UZSN} \cdot \left((\text{UZS}/\text{UZSN}) - (\text{LZS}/\text{LZSN}) \right)^3 \qquad (4.8)$$

where CB is the infiltration level input parameter and PERC is the percolation rate in inches/hour. Evapotranspiration occurs from the upper zone storage at the potential rate.

Overland Flow The method used for the simulation of overland flow was outlined in Chapter III and in Appendix A. The basic relationships are Equations 3.1 and 3.6. The model continuously solves a continuity equation

$$D_2 = D_1 + \Delta D - \bar{q} \Delta t \qquad (4.9)$$

where Δt is the time interval used, D_2 is the surface detention at the end of the current time interval, D_1 is the surface detention at the end of the previous time interval, ΔD is the increment added to surface detention in the time interval, and $\bar{q}$ is the overland flow into the stream channel during the time interval. The discharge $\bar{q}$ is a function of the moisture supply rate and of $(D_1 + D_2)/2$, the average detention storage during the time interval (D in Equation 3.6).

The system of equations can be solved numerically with good accuracy if the time interval of the calculation is sufficiently small so that the value of discharge in any time interval remains a small fraction of the volume of surface detention. Calculations of discharge from overland flow in the model are made on a 15-minute time interval in the general model, but shorter time intervals can be used if required by the characteristics of the flow plane, or if justified by the input data. The increment to overland flow surface detention (ΔD) is found from equations based on Figure 4.3. The length, slope, and estimated roughness of an overland flow plane are used as input in the watershed.

Interflow The calculation of inflow to interflow detention storage was illustrated in Figure 4.3. Outflow from this storage is calculated on a

15-minute time interval and is

$$\mathrm{INTF} = \mathrm{LIRC4} \cdot \mathrm{SRGX} \tag{4.10}$$

where

$$\mathrm{LIRC4} = 1.0 - (\mathrm{IRC})^{1/96} \tag{4.11}$$

The input parameter IRC is a daily recession or depletion constant for the interflow component; the ratio of the interflow discharge at any time to the interflow discharge twenty-four hours earlier.

Groundwater The inflow to groundwater storage is a portion of the net infiltration shown in Figure 4.3 and a portion of the delayed infiltration from the upper zone storage. The balance of the infiltrating water is held in the lower zone storage. The percentage of either direct or delayed infiltration that enters the groundwater storage is a function of the dimensionless storage ratio LZS/LZSN, where LZS is the quantity of moisture in the lower zone storage, and LZSN is the storage level at which fifty per cent of all incoming moisture moves to groundwater storage. The percentage of infiltration that enters the groundwater storage is given by

$$P_g = 100 \left(\frac{\mathrm{LZS}}{\mathrm{LZSN}} \left(\frac{1.0}{1.0 + \mathrm{LZI}} \right)^{\mathrm{LZI}} \right) \tag{4.12}$$

when LZS/LZSN is less than one and by

$$P_g = 100 \left(1.0 - \left(\frac{1.0}{1.0 + \mathrm{LZI}} \right)^{\mathrm{LZI}} \right) \tag{4.13}$$

when LZS/LZSN is greater than one. LZI is defined by

$$\mathrm{LZI} = 1.5 \left| \frac{\mathrm{LZS}}{\mathrm{LZSN}} - 1.0 \right| + 1.0 \tag{4.14}$$

These relationships are plotted in Figure 4.8.

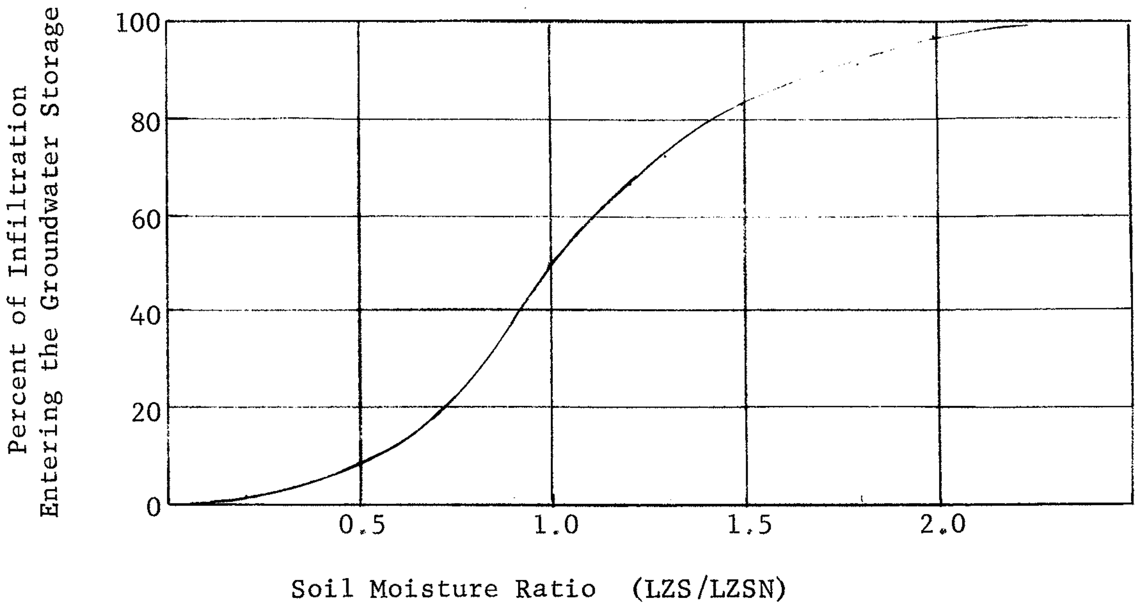


Figure 4.8

The outflow from groundwater storage at any time is based on the simplified model in Figure 4.9. The discharge of an aquifer is proportional to the product of the cross-sectional area and the energy gradient of the flow. A representative cross-sectional area of flow is assumed proportional to the groundwater storage level in the model. The energy gradient is estimated as a base gradient plus a variable gradient that depends on groundwater accretion. The groundwater outflow GWF at any time is given by

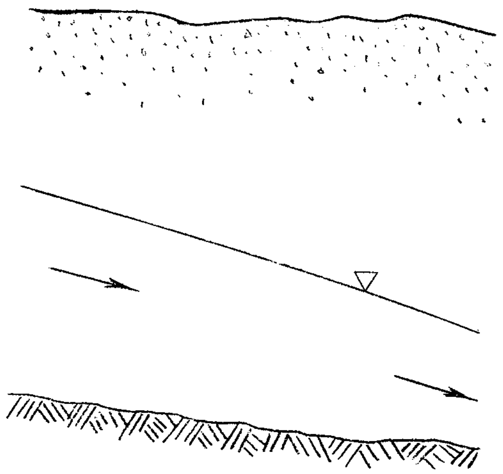


Figure 4.9

$$GWF = LKK4 \cdot (1.0 + KV \cdot GWS) \cdot SGW \qquad (4.15)$$

The variable GWS is an antecedent index based on inflow to groundwater storage and is calculated daily as

$$GWS = 0.97 \ (GWS + \text{inflow to groundwater storage}) \qquad (4.16)$$

Groundwater outflow is calculated on 15-minute intervals. The parameter LKK4 is defined as

$$LKK4 = 1.0 - (KK24)^{1/96} \qquad (4.17)$$

where KK24 is the minimum observed daily recession constant of groundwater flow, the ratio of current groundwater discharge to the groundwater discharge twenty-four hours earlier. When the parameter KV is zero and inflow to groundwater storage is zero, Equation 4.15 reproduces the commonly used logarithmic depletion curve, i.e., the flow after a period of n days decreases by $(KK24)^n$, and a semi-logarithmic plot of discharge vs time is a straight line.

KV is introduced to allow variable groundwater recession rates. When KV is non-zero a semi-log plot of discharge vs time is not linear. For example, if the typical daily dry season recession rate in a stream is 0.99 and a recession of 0.98 is more typical when groundwater storages are being recharged, the value of KK24 can be set to 0.99 and the value of the parameter KV can be adjusted so that $1.0 + KV \cdot GWS$ will reduce the effective recession rate to 0.98 during recharge periods. This added flexibility in groundwater outflow simulation, introduced at the cost of an additional input parameter, is useful in many watersheds.

Percolation to deep or inactive groundwater storage is modelled by allowing a fixed portion of the inflow to groundwater to bypass the active storage that contributes to streamflow and percolate to deep or inactive groundwater storages. This portion is assigned by the input parameter K24L.

Evapotranspiration Evapotranspiration occurs from interception storage, and from the upper zone storage at the potential rate. Evapotranspiration opportunity controls evapotranspiration from the lower zone storage. Minor amounts of evaporation from stream surfaces, and evapotranspiration from groundwater storages are also simulated. Daily lake evaporation or potential evapotranspiration data, or semi-monthly data from which daily data are estimated, are used as input. Hourly values are found from the daily totals.

Potential evapotranspiration will result in a water loss or actual evapotranspiration only if water is available. The program first attempts to satisfy the potential from interception storage and from the upper zone in that order. Any remaining potential enters as E_p in Figure 4.10. Since evapotranspiration opportunity in a watershed on a given day may be expected to vary through a considerable range, a cumulative frequency distribution similar to those found for infiltration capacity in Figure 3.3 might be reasonable. Following the assumption made for infiltration capacity the cumulative frequency distribution of evapotranspiration opportunity is assumed to be linear as shown in Figure 4.10.

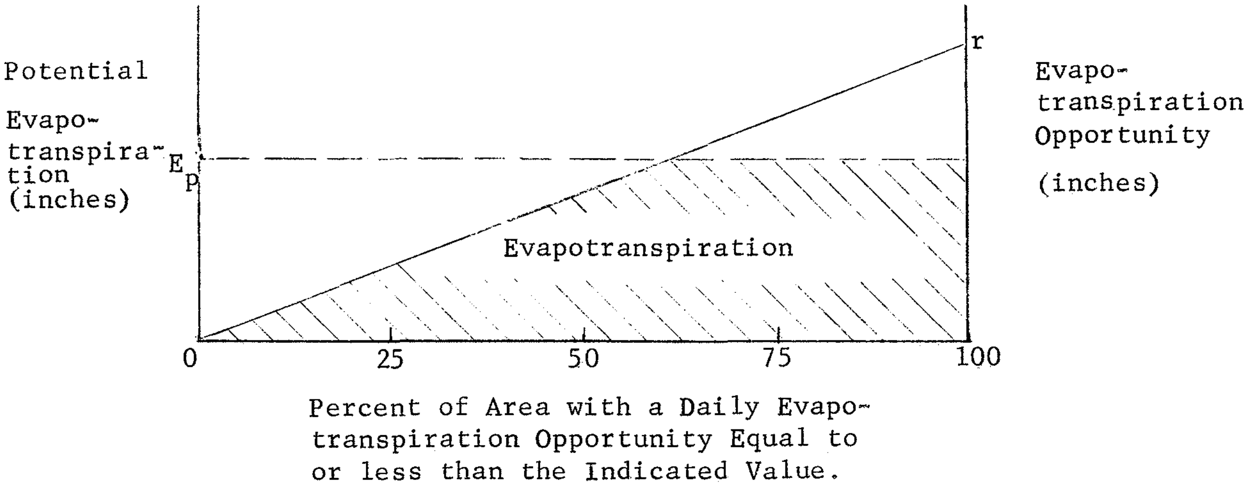


Figure 4.10

The quantity of water lost by evapotranspiration from the lower zone when E_p is less than r can be found from Figure 4.10 and is:

$$E = E_p - \frac{E_p^2}{2r} \qquad (4.18)$$

where E is actual evapotranspiration and E_p is potential evapotranspiration in inches per day. The variable r is an index given by:

$$r = K3 \left(\frac{LZS}{LZSN} \right) \qquad (4.19)$$

where K3 is an input parameter.

Maximum actual evapotranspiration for a given lower zone storage level occurs when the potential evapotranspiration is greater than r and equals r/2 inches over the watershed. Numerical values of the parameter K3, and of other model parameters, are discussed in the next chapter.

Two additional forms of evapotranspiration are simulated. Evaporation at the potential rate from stream surfaces is governed by the parameter ETL which is the ratio of the total stream area in the watershed to the total watershed area. Evapotranspiration from groundwater storage is governed by K24EL; a parameter that represents the fraction of the total watershed area in which evapotranspiration from groundwater storage is assumed to occur at the potential rate.

The Channel System

The operation of the land surface components of the model produces continuous overland flow, interflow, and groundwater flows that enter the stream channel system. The following sections contain a description of the mathematical representations used to simulate the time delay and attenuation of channel inflow as it moves in the channel system.

Channel Translation The use of a time-delay curve to represent the flow time in channels neglecting storage attenuation was described in Chapter III. To construct a time-delay curve or histogram, estimates of flow time in channels are needed. Some approximate but useful estimates can be found from empirical equations for steady open channel flow. From the Manning equation, for example, the flow time in hours for steady flow in a reach of wide channel with a length L and slope S is

$$t = \frac{n \quad L}{5370 \quad y2/3 \qquad S1/2} \tag{4.20}$$

or

$$t = \frac{n^{3/5} \quad L \quad W^{2/5}}{4560 \quad S^{3/10} \quad Q^{2/5}} \tag{4.21}$$

where the hyraulic radius is assumed equal to the flow depth y, n is Manning's n, W is channel width, and Q is discharge. Equations 4.20 or 4.21 can be used to find an estimate of the time of flow from any point in the channel system to the outlet of a watershed for any assumed discharge level. When parameters for a watershed are being developed, a short calculation program based on Equation 4.21 is used that calculates ordinates of the time-delay histogram. Input and output for this program are outlined in Chapter V.

The volume of channel inflow in any time interval is multiplied by successive elements of the time-delay histogram to give a watershed outflow hydrograph that neglects storage attenuation. For each time interval, discharge neglecting storage attenuation is calculated as

$$I_t = \sum_{x = o}^{x = z - 1} R_{t - x} \ C_{x + 1} \tag{4.22}$$

Where I_t is the inflow in the current time interval to a hypothetical reservoir storage used to represent storage attenuation, $R_{t - x}$ is the channel inflow x time intervals ago, and $C_{x + 1}$ is an element of the time-delay histogram.

The sum

$$\sum_{x = o}^{x = z - 1} C_{x + 1} = 1.0 \tag{4.23}$$

where z is the total number of elements in the time-delay histogram. The program allows the use of any time interval for the time-delay histogram so that suitable calculation interval can be selected for watersheds of different size.

Channel Routing The outflow hydrograph produced by channel translation calculations in Equation 4.22 is routed through a storage system to simulate attenuation in the channel system. It is assumed that outflow level O is proportional to storage,

$$0 = k \cdot S \tag{4.24}$$

and

$$\frac{d0}{dt} = k \frac{dS}{dt} \tag{4.25}$$

but

$$\frac{dS}{dt} = I - 0 \tag{4.26}$$

or

$$\frac{dO}{dt} = k(I-O) \tag{4.27}$$

An equivalent numeric form of this equation can be found from Figure 4.11.

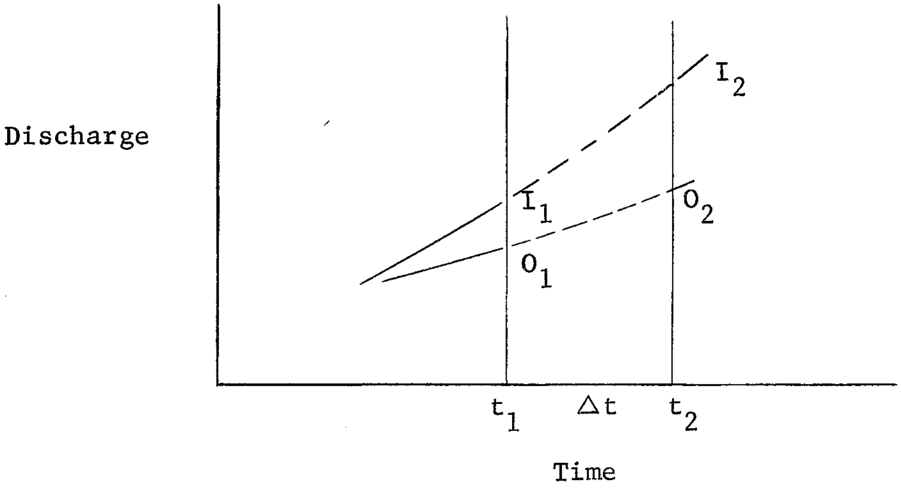


Figure 4.11

Using the notation in Figure 4.11 and time interval Δt Equation 4.27 becomes

$$\frac{O_2 - O_1}{\Delta t} = k \left(\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \right) \tag{4.28}$$

Equation 4.28 can be more conveniently expressed as

$$O_2 = \frac{I_1 + I_2}{2} - \frac{(1/k - \Delta t/2)}{(1/k + \Delta t/2)} \left(\frac{I_1 + I_2}{2} - O_1 \right) \tag{4.29}$$

or

$$O_2 \doteq \bar{I} - KS1 \; (\bar{I} - O_1) \tag{4.30}$$

where $\bar{I}$ is the average inflow during the time interval and KS1 is

$$KS1 = \frac{(1/k - \Delta t/2)}{(1/k + \Delta t/2)} \tag{4.31}$$

The watershed model uses Equation 4.30 which is a very fast and simple form. The parameter KS1 can be varied* if necessary as a function of discharge. When inflow is zero, KS1 becomes a recession constant for the water in channel storage.

Snowmelt

The storage of precipitation in a snowpack, followed by the release of water as snowmelt is an important hydrologic process in many watersheds. The continuous heat exchange between the atmosphere and the snowpack must be simulated to correctly reproduce the quantities and timing of melt water reaching the land surface. A study of simulation techniques to monitor snowpack behavior was undertaken by E. A. Anderson and a general report on this topic has been published.[4]

The main processes that cause melt at the snow surface are radiation, convection, condensation, and rainfall. Ground melt may occur at the land surface due to heat transfer from the earth. In previous studies[4], a subroutine that calculated each of the components of melt using observed data for radiation, wind velocity, dewpoint, and temperature was developed.[4] The input requirements for this subroutine are stringent, and its application is limited to well instrumented experimental watersheds.

An alternate subroutine that could use the sparse data that is typically available was also developed. This subroutine used temperature data only, with the assumption that melt could be calculated from the temperature differential between the atmosphere and the snow surface. Radiation melt is the principal source of heat for snowmelt and the relatively poor correlation between the atmosphere/snow surface temperature differential and incoming radiation appeared to be largely responsible for the reduction in accuracy when output from this subroutine was compared

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*When a stream rises beyond its normal channel and flows over a flood plain, storage increases more rapidly than discharge. The value of k in Equation 4.24 decreases and KS1 increases. Functions that decrease KS1 at high stages have been used for a few watersheds, but are not included in the model since they lack generality and are seldom needed.

to that from more detailed simulation.[4]

Since the sparse data situation is of major practical importance, procedures were developed to represent components of melt more adequately for this case. The subroutine described in the following two sections was designed specifically for sparse data and is recommended for watersheds where the basic data are temperature or temperature and radiation measurements. If wind velocity and dewpoint temperature measurements are also available,more precise snowmelt equations should be substituted. Detailed discussions of snowmelt simulation and of general snowmelt processes are found in "The Synthesis of Continuous Snowmelt Runoff Hydrographs"[4] and "Snow Hydrology"[30] respectively.

Subroutine Structure The snowmelt subroutine included in the model in Appendix C uses daily maximum and minimum temperature, measured or estimated short-wave radiation, snow evaporation, and precipitation data. Calculations are made hourly as incoming precipitation is added to the snowpack or to liquid water storage. Temperature and radiation data are used to find the net heat exchange in the hour. If the net heat exchange is negative, heat is being lost from the snowpack and the temperature of the snowpack decreases from 32° F. This process is modelled by increasing a "negative heat" storage.

When the net heat exchange becomes positive, the temperature of the snowpack increases and the negative heat storage is reduced. When negative heat storage becomes zero, snowmelt will begin. Melt enters liquid water storage until a limiting storage is reached. Additional melt or rainfall is then discharged from the snowpack.

Snow evaporation is calculated from estimated potential snow evaporation if the air temperature is less than 32° F. The limiting negative heat and liquid water storages are functions of the current water equivalent of the snowpack. Liquid water storage tends to freeze into the snowpack as negative heat storage increases. Allowance is made for variations in the portion of short-wave radiation absorbed by the snowpack by calculating continuous estimates of snow surface albedo. Continuous estimates of the surface area of snowpack in each watershed segment are also calculated. A schematic flowchart of the subroutine is shown in Figure 4.12.

SNOWMELT SUBROUTINE IV
FLOWCHART

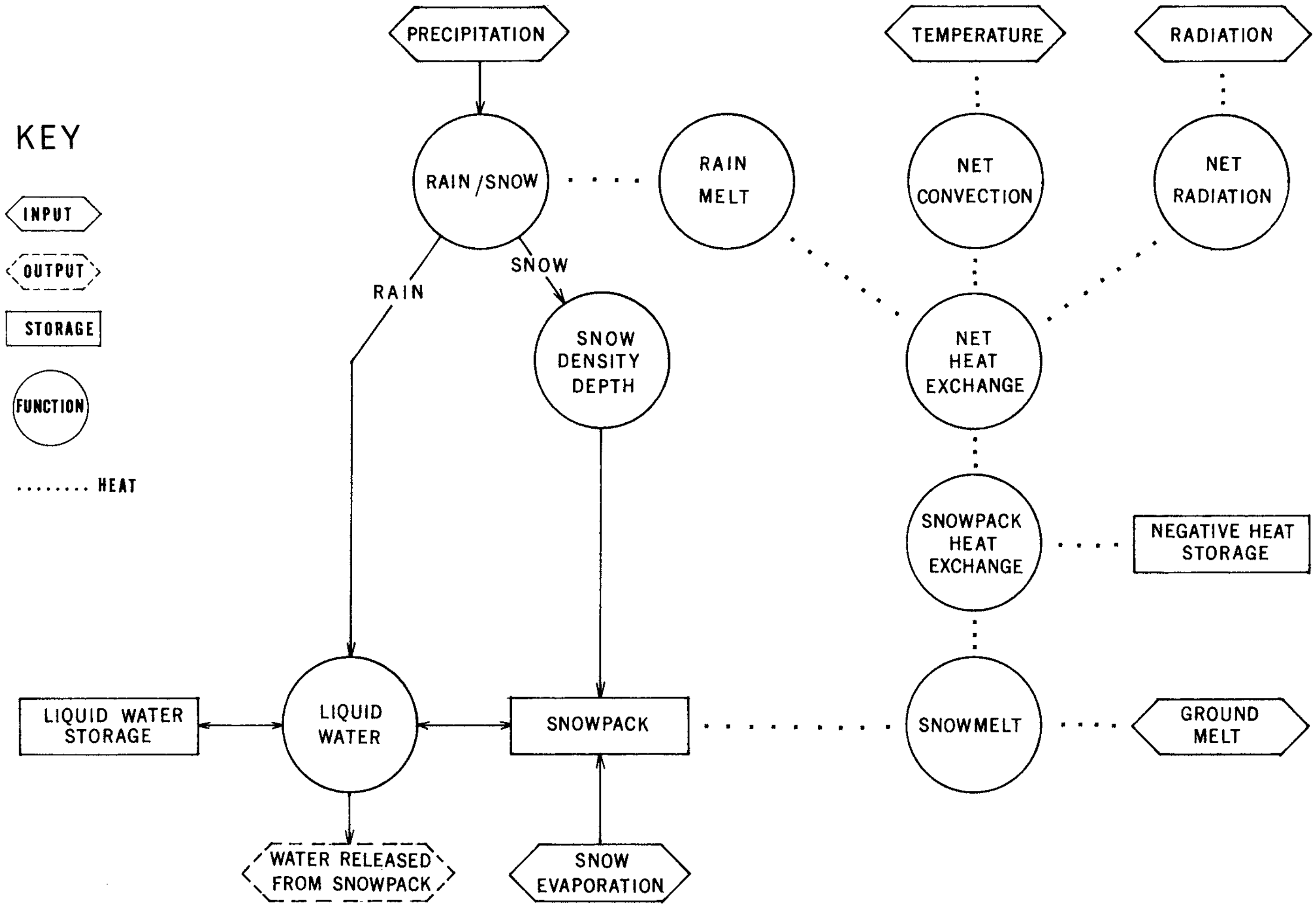

Figure 4.12

Mathematical Functions The following functions are used to calculate the components of melt and the continuous variables needed to represent snowpack conditions. Snowpack calculations are made for each of a series of watershed segments based on elevation and exposure.

Hourly temperatures are calculated for each watershed segment from the maximum and minimum daily temperatures at base stations. Temperature variation with elevation is modelled using lapse rates that allow for diurnal variation and typical dry weather or storm conditions. Storage of liquid water in the snowpack is limited by an input parameter WC; where WC is the maximum liquid water storage as a fraction of storage in the snowpack. The parameter WC is assumed to be constant in the normal range of snow densities.

Snow surface albedo varies between 0.75 and 0.65, a relatively narrow range developed from simulation runs. Albedo increases when snow occurs and decreases as surface melt occurs.

The density of new snow for temperatures above 0° F is estimated as a function of temperature as

$$DNS = INDS + (T/100)^2 \qquad (4.32)$$

where DNS is the density of new snow, T is temperature, and INDS is the snow density at or below 0° F. Snow depth is decreased hourly, if snow density SDEN is less than 0.6, by multiplying the current depth D_S by $(1.0 - 0.00002\,(D_S(0.6 - SDEN)))$.

When meteorologic records do not indicate whether the precipitation occurred as rain or snow some criteria must be used to estimate the form of precipitation. It is assumed that snow occurs in a watershed segment if the temperature 750 ft. above the segment is less than 32° F. Variable meteorologic conditions and the relatively imprecise estimates of hourly temperature found from maximum and minimum daily temperatures cause some discrepancies, and no criteria of this type is entirely satisfactory. In watersheds where large storms are common at temperatures near freezing, the form of precipitation simulated should be checked against snow depth measurements, and the criteria above overruled where necessary to produce the proper form of precipitation.

Measured incoming short-wave radiation in langleys per day can be used as input. The amount of radiation in each hour is calculated from the daily total using an average distribution. If measured radiation is not available, bi-monthly theoretical or clear sky radiation at the watershed is used as input. The portion of the clear sky radiation that reaches the watershed each day is estimated from the temperature data by assuming the ratio of actual to theoretical radiation varies linearly with the diurnal temperature increase, and maximum radiation is assumed if the temperature increase exceeds 27° F.

Incoming short-wave radiation is intercepted by forest cover on a portion of each watershed segment given by input parameter F, and is also multiplied by the input parameter RADCON to allow for variation in slope and exposure in the watershed segments. A large portion of the incoming radiation, given by the current value of albedo, is reflected from the snow surface, and the remainder is the net short-wave radiation. Long-wave radiation is emitted by the snowpack at a known rate. Clouds, trees, and other objects absorb short-wave radiation and emit long-wave radiation that may reach the snow surface, and long-wave radiation exchange is complex. The subroutine uses estimates developed by the Corps of Engineers[31] for net long-wave radiation exchange for open areas and for forested areas. Net long-wave radiation in langleys per hour is

$$LW = 27.5(1.0-F)(0.76\cdot(T_r/273.0)^4-1.0)+27.5(F)((T_r/273.0)^4-1.0) \quad (4.33)$$

where F is the portion of the watershed with forest cover and T_r is absolute temperature in $^{\circ}R$. Long-wave and short-wave radiation are combined each hour to find the net total radiation Hm. Total radiation melt is:

$$M_r = \frac{Hm}{203.2} \quad (4.34)$$

where M_r is melt from radiation in inches.

Heat exchange due to convection is a function of wind velocity and the temperature differential between the atmosphere and the snow surface. Assuming uniform wind velocities convection melt is expressed as:

$$M_c = CONMELT\cdot(T-32) \quad (4.35)$$

where M_c is melt from convection in inches and T is current atmospheric temperature.

Condensation melt is erratic, occurring only when atmospheric vapor pressure exceeds the vapor pressure at the snow surface. Since dewpoint data are not normally available, a separate estimate of condensation melt cannot be calculated. Condensation melt does require that atmospheric temperature be above 32° F and will occur only when M_c above is positive. When the parameter CONMELT is estimated from simulation runs condensation melt will influence its value.

The radiation melt and convection melt equations may give negative values or a net heat loss. A net heat loss due to radiation from Equation 4.34, or due to convection from Equation 4.35, is multiplied by (1.0-(NEGMELT/NEGMELTM)) and added to negative heat storage if the negative heat storage does not exceed a calculated maximum value NEGMELTM. The maximum negative heat storage that can exist at any time is found by assuming a linear temperature distribution from atmospheric temperature at the snow surface to 32° F at the bottom of the snowpack. Thus NEGMELTM is a function of current air temperature and PACK, the water equivalent of the snowpack.

Melt due to rainfall is calculated by assuming that the temperature of rain equals atmospheric temperature and is:

$$M_p = \frac{(T - 32.0)\cdot PX}{144} \qquad (4.36)$$

where M_p is melt in inches, T is atmospheric temperature, and PX is rainfall in inches. Ground melt is set equal to an input parameter DGM that represents the daily water equivalent of melt in each watershed segment.

The input parameter ELDIF is the elevation difference in thousands of feet between the base temperature station and any watershed segment. It is positive if the elevation of the segment exceeds the elevation of the base station. The areal coverage of the snowpack in each watershed segment is modelled by assuming that water-equivalent must exceed the variable IPACK for complete areal coverage. IPACK equals the maximum value of PACK to date in each water year up to MPACK, in input parameter. The ratio PACK/IPACK is assumed to be the portion of segment area with snow cover if PACK is less than IPACK.

Examples of output from this subroutine are included in Chapter IV. The functions used above for snowmelt simulation are not, of course, the most accurate representations since they were designed for specific data limitations, and more general information and approaches to snowmelt analysis are found in the references previously cited.

V. OPERATION OF THE MODEL

The basic simulation model is designed to accept input from any number of recording gages and to produce streamflow at a series of points in the stream channel system. In this chapter typical input and output and the major input and output options are described. Initial estimates of watershed parameters and a description of the sequence of computer runs used to improve watershed parameters are included.

Model Operation

A definition sketch of the watershed components used in the model is shown in Figure 5.1. Streamflow may be calculated at several locations in the stream channel called flowpoints. These flowpoints are usually at existing stream gages, but may be placed at other points in the channel system. The area above each flowpoint is divided into segments so that there are one or more segments for each recording rain gage. The segments are selected from topographical considerations or by constructing a Thiessen network. Each of the segments may contain one daily or storage rain gage. The average of several storage gages can be used in a segment if all gages have the same observation time.

The general model will continuously calculate the streamflow at each flowpoint from rainfall in each successive watershed segment, and from flows measured or calculated at upstream flowpoints. All calculations are carried independently for each watershed segment so that areal variations in rainfall and topographic features are represented. There is no limit to the number of segments or flowpoints that can be used.

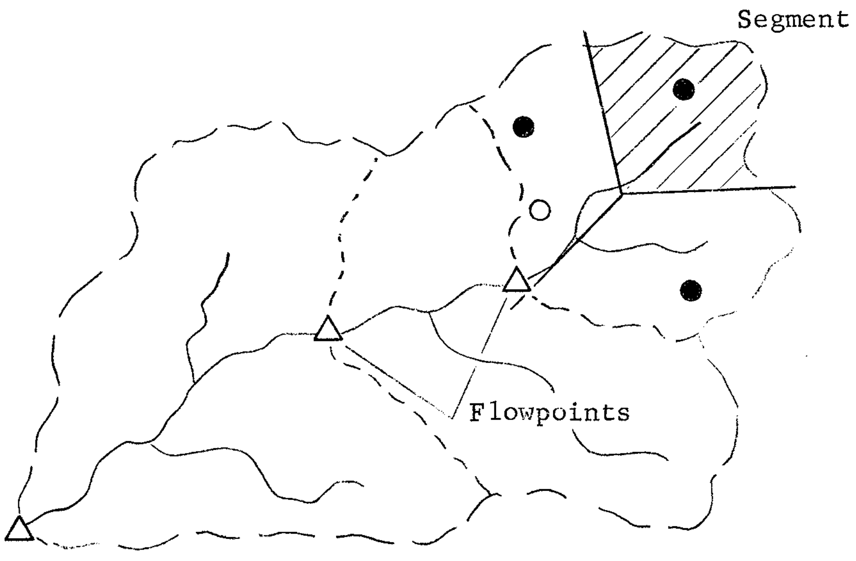

Figure 5.1

Input The general model is a detailed simulation program that monitors watershed conditions and produces a wide variety of output. Included in the general model is a data tape section that reads data cards and stores precipitation data on magnetic tape for use in simulation. This conserves both computer time and core storage since the magnetic tapes of rainfall records can be stored and used many times. The tapes are read in small sections during simulation runs.

The general model input can be divided into two categories: Input used to create magnetic tapes of precipitation data, and the additional input needed for streamflow simulation. The runs that prepare precipitation data tapes and the actual simulation runs are not usually made at the same time. The first card that the watershed model reads for any run contains two Boolean, or true-false variables, TAPES and RUN. If TAPES is non-zero the input for the precipitation tape option in Table 5.1 is read. If RUN is non-zero the simulation input in Table 5.2 is read, and if both variables are non-zero, input from Tables 5.1 and 5.2 is read in order.

Table 5.1 outlines the input used for successive station years of recording and storage gage data. A sample recording gage data card referred to by the DATA Identifier in Table 5.1 is shown in Figure 5.2.

Table 5.1 Precipitation Data Input

SUBALGOL Identifiers	Elements	TYPE I, Integer R, Real	Comments
STATYR	YRS	I	The number of station years to be processed
	FILX	I	The number of tape files spaced forward.
	SG	I	Storage gage in use if non-zero
WATYR	I1	I	First year of water-year
	I2	I	Second year of water-year
RR	STT	I	Storage gage station number
	YR	I	Year
	MO	I	Month of the year (9th month must be used)
	WSG	R	Storage gage weight
	OBS	I	Observation time (24 hour clock)
	PREC ( )	R	Daily rainfall for the month.
DATA	ST	I	Recording gage station number
	YR	I	Year (98 for the last day of the water-year)
	MO	I	Month of the year
	DAY	I	Day of the month
	CN	I	Index (1 is A.M., 2 is P.M.)
	P1 ( )	R	Hourly rainfall for 12 hours.

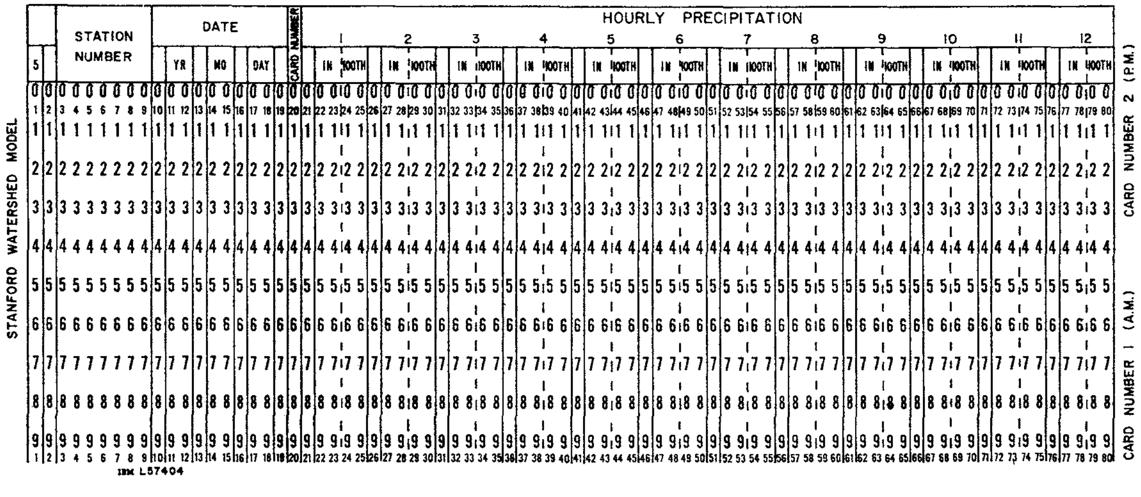

Figure 5.2

When the RUN variable indicates simulation output is to be obtained, the input required is of four types:

i) Control cards that key the program to input and output only what is required for the run.
ii) Physical data for each flowpoint in the run. This includes evapotranspiration data, streamflow and diversion data, and the initial soil moisture and groundwater storages.
iii) Physical parameters for each watershed segment. Each segment has a channel time-delay histogram and data cards that contain general physical parameters, land surface parameters, and channel system parameters.
iv) Magnetic tape precipitation input previously prepared under the TAPES option.

The model will consecutively run any number of water years for any number of flowpoints with any number of watershed segments or recording rain gages per flowpoint. All watershed conditions including flow in

STANFORD WATERSHED MODEL IV

Simulation Input Sequence

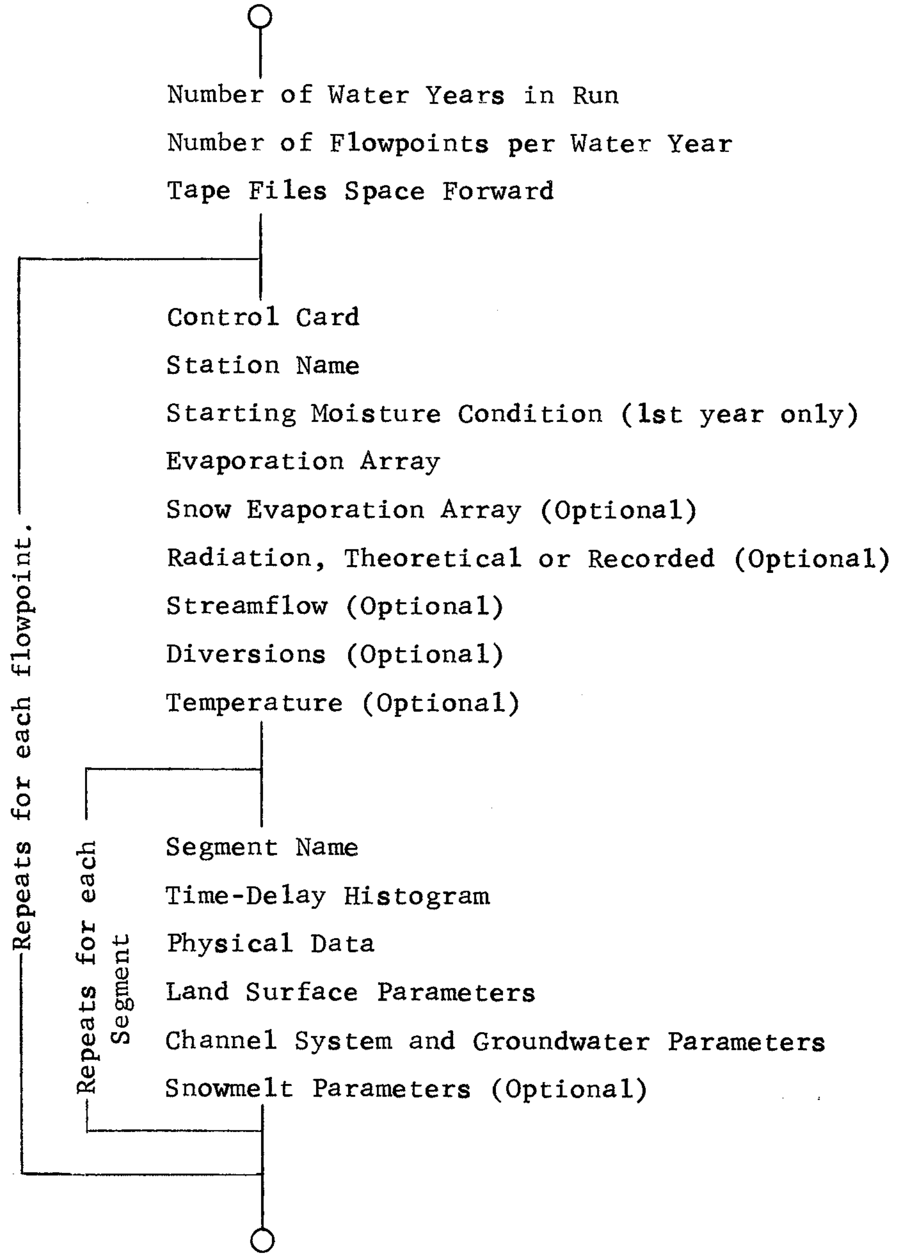

Figure 5.3

transit in the stream channels are carried over from one calculation period or water year to the next. The model will, also, read actual streamgage data on an upstream gage, augment this through simulation of additional flows, and produce the continuous hydrograph that would occur downstream at some ungaged site. Figure 5.3 shows the input sequence used for a typical simulation run. A summary of the simulation input data is given in Table 5.2.

Table 5.2 General Simulation Input

SUBALGOL Identifiers	Elements	Type I Integer R Real B Boolean	Comments
RUNDAT	FPYR	I	Flowpoint years in run.
	NXTSEG	I	Flowpoints per water-year.
	MFILX	I	Tape files space forward
CONTROL	NX	I	Number of DCS options
	DCS (1)	B	Detailed storm analysis
	DCS (2)	B	and parameter optimization output
	DCS (3)	B	Input bimonthly evaporation data
	DCS (4)	B	Input streamflow
	DCS (5)	B	Input diversions
	DCS (6)	B	Output flow duration & error table
	DCS (7)	B	Output maximum rainfall & runoff
	DCS (8)	B	Plot mean daily flows
	DCS (9)	B	Input daily max & min temperature
	DCS (10)	B	Input daily radiation
	DCS (11)	B	Input 15-min rainfall
	FLOWPOINT	I	Flowpoint number
	SEG	I	Segments at flowpoint
	TAREA	R	Total watershed area at flowpoint (square miles)
	MAXCFS	R	Maximum flow plotted (c.f.s.-days)
	TAPEMOVE	I	Shift records on input data tape
	SHIFT	I	Flow time from upstream flowpoint (hours)

Table 5.2 continued

SUBALGOL Identifiers	Elements	Type I Integer R Real B Boolean	Comments
	MINH	R	Minimum hourly discharge printed.
NEWY	DDYR1 DDYR2 YEAR QQO ( )	I I R I	First year of water-year. Second year of water-year. Annual recorded runoff in acre-ft. Sixty Alpha-numeric spaces Flowpoint name and location.
START[1]	SGWS ( ) UZSS ( ) LZSS ( ) GWSS ( )	R R R R	Initial groundwater storage Initial upper zone storage Initial lower zone storage Initial groundwater slope index
EVAPM or EVAP	E ( ) E ( )	R R	Daily potential evapotranspiration for semi-monthly intervals (1st value 15 days, 2nd remainder of month) Daily potential evapotranspiration. (Optional-DCS (3)).
EVC	EVCR ( )	R	Monthly correction to potential evapotranspiration.
EVAPS	EES ( )	R	Snow evaporation on semi-monthly intervals. (Optional - DCS (9))
RADIATION or MRAD	RAD ( ) MAXRAD ( )	R R	Observed incoming daily short-wave radiation. (Optional-DCS (10)) Theoretical maximum daily incoming short-wave radiation.
TEM	T ( )	R	Daily maximum and minimum temperature °F. (Optional DCS (9))

[1] Reads for the first water year of a run at each flowpoint.

Table 5.2 continued

SUBALGOL Indentifiers	Elements	Type I Integer R Real B Boolean	Comments
FLOWS	FLO ( )	R	Mean daily flow (Optional-DCS (4))
DIVER	SDIV ( )	R	Mean daily diversion into or out of the watershed (Optional-DCS (5))
TRI	QQQ ( )	I	Alphanumeric input: Date, Name, Parameter Set, etc.
ARRA1	RINT	I	Routing interval in hours
	Z	I	Number of time-delay elements
	C ( )	R	Elements of time-delay histogram
CL1			Physical Parameters
	K1	R	Ratio of average segment rainfall to average gage rainfall.
	AREA	R	Segment area
	A	R	Impervious area
CL2			Land Surface Parameters
	EPXM	R	Interception storage: Maximum value
	UZSN	R	Nominal upper zone storage
	LZSN	R	Nominal lower zone storage
	K3	R	Actual evaporation loss index
	K24L	R	Portion of groundwater recharge assigned to deep percolation
	K24EL	R	Evapotranspiration from groundwater
	CB	R	Infiltration Index
	CC	R	Interflow index
	L	R	Overland flow length
	SS	R	Overland flow slope
	NN	R	Manning's n for overland flow
CL3			Channel System and Groundwater Parameters

Table 5.2 continued

SUBALGOL Indentifiers	Elements	Type I Integer R Real B Boolean	Comments
	KS1	R	Stream channel storage recession parameter
	IRC	R	Interflow runoff recession
	KV	R	Groundwater recession: variable component
	KK24	R	Groundwater recession: basic rate
	ETL	R	Evaporation from stream surfaces
CL4			<u>Snowmelt Parameters</u>
	RADCON	R	Radiation melt parameter
	CONMELT	R	Convection-condensation melt parameter
	SCF	R	Snow correction factor
	ELDIF	R	Elevation difference in thousands of feet
	IDNS	R	Index density of new snow
	F	R	Forest cover index
	DGM	R	Daily ground melt
	WC	R	Water content of snow at saturation
	MPACK	R	Water equivalent of the snowpack for complete areal coverage
	NXTAPM	R	Shifts records on precipitation input tape.

<u>Output</u> The watershed model will produce basic output and a variety of optional output on demand. The basic output that is printed for each water year, is shown in Appendix B. This output consists of:

i) A summary table of the end of the month values such as soil moisture conditions for each segment.

ii) Monthly summaries of processes such as total interflow discharge and actual evapotranspiration.

iii) Complete hyrographs for all storms that produced flows greater than some preselected base flow.
iv) Summary tables of mean daily flows for each flowpoint.

Optional output includes:

i) Maximum clock hour rainfall and channel inflow values.
ii) Statistical comparisons of mean daily simulated and recorded streamflow.
iii) Graphical plots of simulated and recorded mean daily flows at the flowpoints.
iv) Daily snowpack water equivalent, depth, density, and liquid water storage.
v) Detailed storm analysis with 15-minute rainfall, interception, infiltration, and overland flows.
vi) Storm period summaries with indicated or assigned parameter or variable changes and data consistency output.

Many other items of output data can be printed where necessary by adding output statements to the general program. This is often done for specialized studies. Several examples of standard and optional output in different watersheds are included in Chapter VI, and listings of typical input and output sequences are shown in Appendix B.

Parameter Optimization

The parameters used in the model to represent any watershed are given in the input cards CL1, CL2, CL3, and CL4 in Table 5.2,and by the channel time-delay histograms. Since only a few watersheds with snowmelt have been run,no attempt has been made to develop expected values for the snowmelt parameters on CL4. The following discussion is limited to the time-delay histograms and the parameters on input cards CL1, CL2, and CL3.

Although these lists of parameters are lengthy, most of the parameters can be readily found from hydrologic or meterorologic records and topographic maps. The exceptions, parameters that are not easily derived, will be identified and examined in detail. It is hoped that with guidelines for initial estimates and specified methods of analysis, the parameters used by Model IV can be consistently derived by investigators at different computation centers. Some examples of watersheds for which the values of all parameters can be closely defined will be examined, as will some cases for which interactions between parameters persist. A sequence of computer runs that operate on selected watershed parameters will be explained.

Estimates and Guidelines

The majority of the simulation input data in Table 5.2 consists of program control options and fixed parameters that depend on easily derived watershed characteristics, such as mean rainfall or watershed area.

The channel time-delay histogram on ARRA1, and the physical, land surface, and channel system parameters in inputs CL1, CL2, and CL3 were developed to represent watershed response using a minimum number of degrees of freedom. Each element or parameter in these input lists is defined to represent a physical process in a watershed. The range of each parameter, and sketches or tables of expected values based on experience to date are given in this section.

The first item of input listed above is the channel time-delay histogram. This histogram can be derived from measurements or estimates of channel characteristics. A very simple program based on Equation 4.21 is used to calculate approximate ordinates of the time-delay histogram for steady flow. A typical channel system is shown in Figure 5.4. The channels are divided into a series of reaches using stations numbered starting at the outlet. Mean values of channel width and Manning's n for each reach are entered at the upstream station number. The input is listed in Table 5.3.

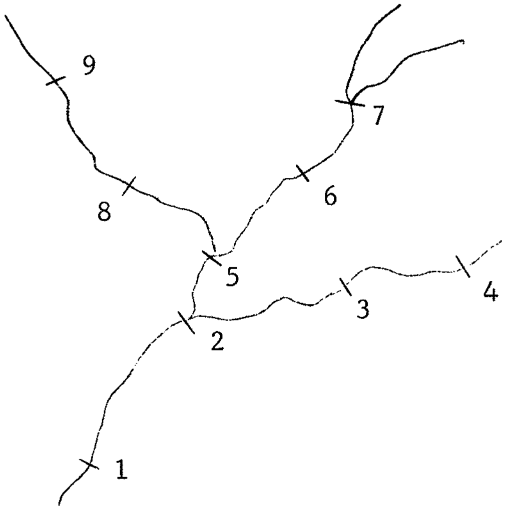

Figure 5.4

SUBALGOL Identifer	Element	Type I Integer R Real B Boolean	Remarks
STA	ST Q ELEV(1)	I R R	Number of Stations Total discharge at outlet Elevation at Station 1
DATTA	STUP STDOWN N L W Q DELQ ELEV( )	I I R R R R R R	Upstream station number Downstream station number Manning's n for channel Length of reach Mean width of flow Discharge in reach Incremental discharge between stations Elevation at the upstream station

Table 5.3

Flow must always move from higher to lower station numbers, and calculations must proceed upstream. For example, in Figure 5.4 the flow from 2 to 1 is calculated first, followed by the flow from 5 to 2 or 3 to 2.

FLOW FROM STATION	DISCHARGE	INCREMENTAL TIME	TOTAL TIME
2 to 1	9700.0	13 MIN 0.22 HRS	13 MIN 0.22 HRS
3 to 2	9400.0	8 MIN 0.13 HRS	21 MIN 0.35 HRS

TIME DELAY	HISTOGRAM
0 - 15	0.02
15 - 30	0.13
30 - 45	0.21
45 - 60	0.23
60 - 75	0.24
75 - 90	0.12
90 - 105	0.05

Table 5.4

The output from the program shown in Table 5.4 contains a listing of station numbers with the steady flow time delay to the watershed outlet,and ordinates of a time-delay histogram for 15-minute intervals. Generally, a relatively high discharge level is selected for the time-delay histogram used in simulation, corresponding to a flow of once in ten or twenty years frequency. The reproduction of peak flows is of major interest in most simulation runs, and minor errors in timing of small peaks are not objectionable.

This simple, steady flow calculation is an imprecise approximation to the time delay for unsteady flood flows for many reasons. For example, wave velocities always exceed the velocities of steady flow and flow velocities vary depending on the assumed discharge level. Despite the shortcomings of assuming that a single time-delay histogram applies, it is rarely necessary to alter the time-delay histogram found, using the program described above, in order to reproduce flood flows. The calculation of hydrograph shape is usually well within the accuracy of the streamflow data when flow volumes are correctly simulated.

Turning now to the input lists, the physical parameters on input list CL1 are straightforward with the possible exception of the impervious area parameter A. In undeveloped watersheds this parameter is usually zero.

In typical urbanized watersheds Figure 5.5 can be used to approximately relate effective impervious area for the model, to the total impervious area estimated or measured from areal photographs.

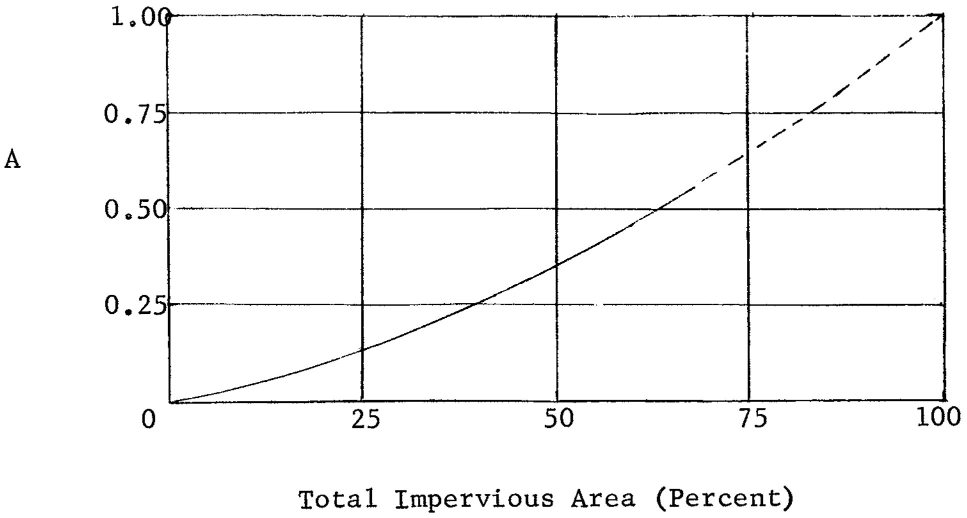

Figure 5.5

Input list CL2 contains the land surface parameters that are responsible for runoff volumes. The list begins with the interception storage parameter EPXM, which can be estimated from Table 5.5.

Watershed Cover	EPXM
Grassland	0.10
Moderate forest cover	0.15
Heavy forest cover	0.2

Table 5.5

The upper zone storage parameter UZSN and the lower zone storage and groundwater parameter LZSN are major runoff volume parameters. A sequence of runs used in their derivation and examples of response sensitivity are given in the next two sections.

The parameter K3 controls the actual evapotranspiration loss rate and was defined in Figure 4.10 and Equation 4.19 in Chapter IV. K3 is a moderately important parameter for long term runoff volumes and can be estimated from Table 5.6.

Watershed Cover	K3
Open Land	0.2
Grassland	0.23
Light Forest	0.28
Heavy Forest	0.3

Table 5.6

The parameters K24L and K24EL control the loss of moisture from the active groundwater storage and can often be assumed zero. These losses are usually small compared to rainfall, evapotranspiration, and runoff. When percolation to deep groundwater is suspected, K24L must be estimated from observed changes in deep groundwater levels or estimates of subsurface outflow from the basin. Alternatively, K24L may be approximated by trial. K24EL is approximately the percent of basin area with shallow groundwater within reach of vegetation.

The parameter CB defined in Equations 4.1 and 4.2, governs the level of infiltration capacity and is moderately effective as a runoff volume parameter in some hydrologic regimes. The interflow parameter CC defined in Equation 4.3, governs only the time distribution of runoff. The derivation of both of these parameters is discussed in the next two sections. The average overland flow length and slope, parameters L and SS, can be found from topographic maps. Manning's n for overland flows can be estimated from the Table 5.7, or from other more detailed sources.[32]

Watershed Cover	Manning's n for Overland Flow
Smooth Asphalt	0.012
Asphalt or Concrete Paving	0.014
Packed Clay	0.03
Light Turf	0.20
Dense Turf	0.35
Dense Shrubbery and Forest Litter	0.4

Table 5.7

Input list CL3 contains channel system and groundwater parameters. KS1 is a time distribution parameter, and is the hourly recession rate for surface runoff in channels when the inflow $\bar{I}$ in Equation 4.3 is zero.

$$\mathrm{KS1} = \frac{\text{Discharge in hour (1)}}{\text{Discharge in hour } (t + 1)} \tag{5.1}$$

The parameters KS1 for the surface runoff recession and the parameters IRC and KK24 for the interflow and groundwater recessions respectively, can be estimated from hydrographs using graphical techniques suggested by Barnes[33].

The parameter KV, described in Chapter IV, is used to allow a variable recession rate for groundwater discharge. For example, if KV is 1.0, Table 5.8 shows the effective recession rate for different levels of KK24 and GWS.

KK24	GWS			
	0.0	0.5	1.0	2.0
0.99	0.99	0.985	0.98	0.97
0.98	0.98	0.97	0.96	0.94
0.97	0.97	0.955	0.94	0.91
0.96	0.96	0.94	0.92	0.88

Table 5.8

Watershed Cover	Manning's n for Overland Flow
Smooth Asphalt	0.012
Asphalt or Concrete Paving	0.014
Packed Clay	0.03
Light Turf	0.20
Dense Turf	0.35
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0.99	0.99	0.985	0.98	0.97
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0.97	0.97	0.955	0.94	0.91
0.96	0.96	0.94	0.92	0.88

Table 5.8

The derivation of the three runoff volume parameters can be somewhat more difficult. Logically in nature and by definition in the watershed model, temporary storage at or near the surface in the upper zone, storage in the remainder of the soil profile or lower zone, and the rate of infiltration into the soil profile from the surface, will all interact in hydrologic response. The parameters UZSN, LZSN, and CB (fortunately there are only three) cannot be independent. However, they are not necessarily strongly dependent in all regimes, or in all circumstances in a particular regime. Since the parameters are physically defined, an examination of the physical effects of storage and infiltration rate interactions in watersheds provides some guidance.

One important feature of the hydrologic regime in a watershed is the proportion of overland flow relative to total runoff. In some regimes overland flow is extremely rare, while in others overland flow occurs in moderate showers. Although no single infiltration rate is applicable in a watershed, significant overland flow will occur only when the general levels of infiltration are low compared to rainfall intensities.

Infiltration rates are highly dependent on the moisture content of the soil profile, but the moisture content of the soil profile near the surface becomes high if vertical drainage or storage in the soil profile is restricted. Therefore, to explain the occurrence of overland flow, it could be said that infiltration rates were low (CB) or that storage capacity in the soil profile (LZSN) was insufficient. Many examples of storage and infiltration rate interactions can be found.

In Figure 5.7 the percentage of runoff for maximum rainfall events is plotted against rainfall intensity in four watersheds. Two of the watersheds, the Russian River and Arroyo Seco, have seasonal rainfall and often develop moisture storage conditions that severely limit infiltration rates. Beargrass Creek and Wollombi Brook have more uniform rainfall and develop limiting moisture storages much less frequently. Therefore, in Figure 5.7, the percentage of runoff in the Russian River and Arroyo Seco is relatively independent of rainfall intensity, since infiltration rates in these streams are frequently very small compared to rainfall intensity. In Beargrass Creek and Wollombi Brook, infiltration rates are typically quite significant and the percentage of runoff is

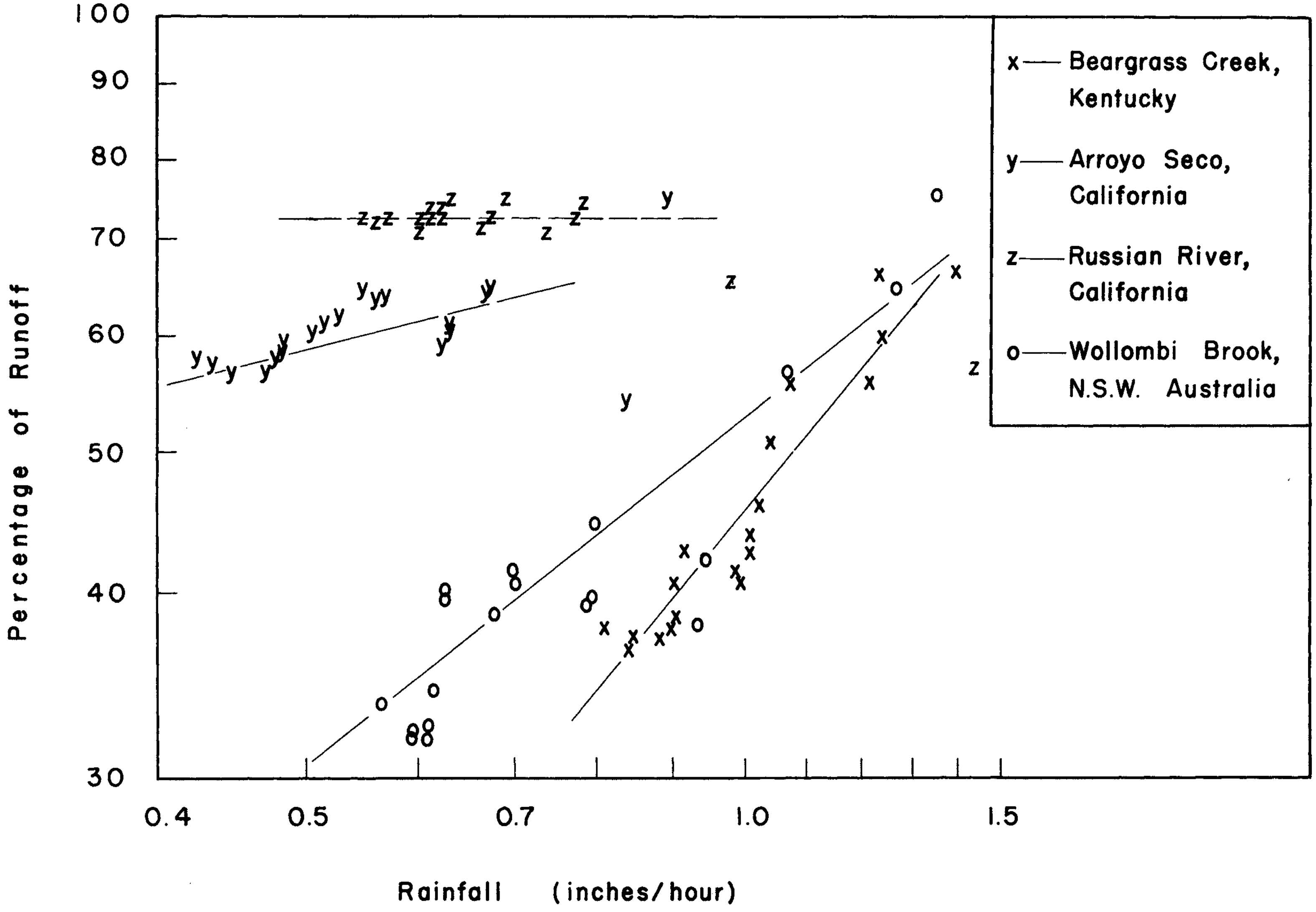

100
90
80
70
60
50
40
30
Percentage of Runoff
0.4
0.5
0.7
1.0
1.5
Rainfall (inches/hour)
x—— Beargrass Creek, Kentucky
y—— Arroyo Seco, California
z—— Russian River, California
o—— Wollombi Brook, N.S.W. Australia

Figure 5.7

more dependent on rainfall intensities.

All watersheds experience storms in which infiltration rates dominate overland flow. In an regime such as the Russian River these are storms that occur after several weeks of dry weather. Watersheds do, however, show this interesting variation in the degree to which storage capacities become limiting.

If a watershed frequently develops storage conditions that greatly restrict infiltration rates, the value of LZSN, an index to the effective storage in the soil profile, can be found fairly easily. For these watersheds the long term volume of runoff is primarily dependent on LZSN,and is effectively independent of the infiltration levels represented by CB. An example of this behavior is shown in Figures 5.8 and 5.9. In Figure 5.8 watershed infiltration rates in the Russian River watershed at Hopland were doubled,and the simulated flow that resulted was plotted against simulated flow at the original infiltration rates. The reduction in total runoff due to doubling infiltration rates was less than three percent. The small storms and early season storms were modified much more than storms at high soil moisture conditions. Note that although total runoff volumes were almost unchanged, a marked increase in simulated groundwater flows resulted from the increase in infiltration rates. In Figure 5.9 the effects of a change in storage capacities for the same water year are illustrated. When the effective storage was increased about thirty percent, the volume of runoff was reduced by fifteen percent. Storms that were modified most by the infiltration rate increase in Figure 5.8,have recovered and are close to their original response. Groundwater outflow has been reduced sharply as the soil profile is now able to retain more water for eventual evapotranspiration.

Ideally,the opposite to a regime with frequent storage limitations would be a regime in which runoff is dependent only on rainfall intensities and a stable infiltration level. This may be approximately true in some watersheds, but in general both basic infiltration rates and the effects on infiltration rates of storage levels will determine regime. For watersheds without storage limitation,computer trials using several values of nominal storage LZSN and infiltration level CB may be

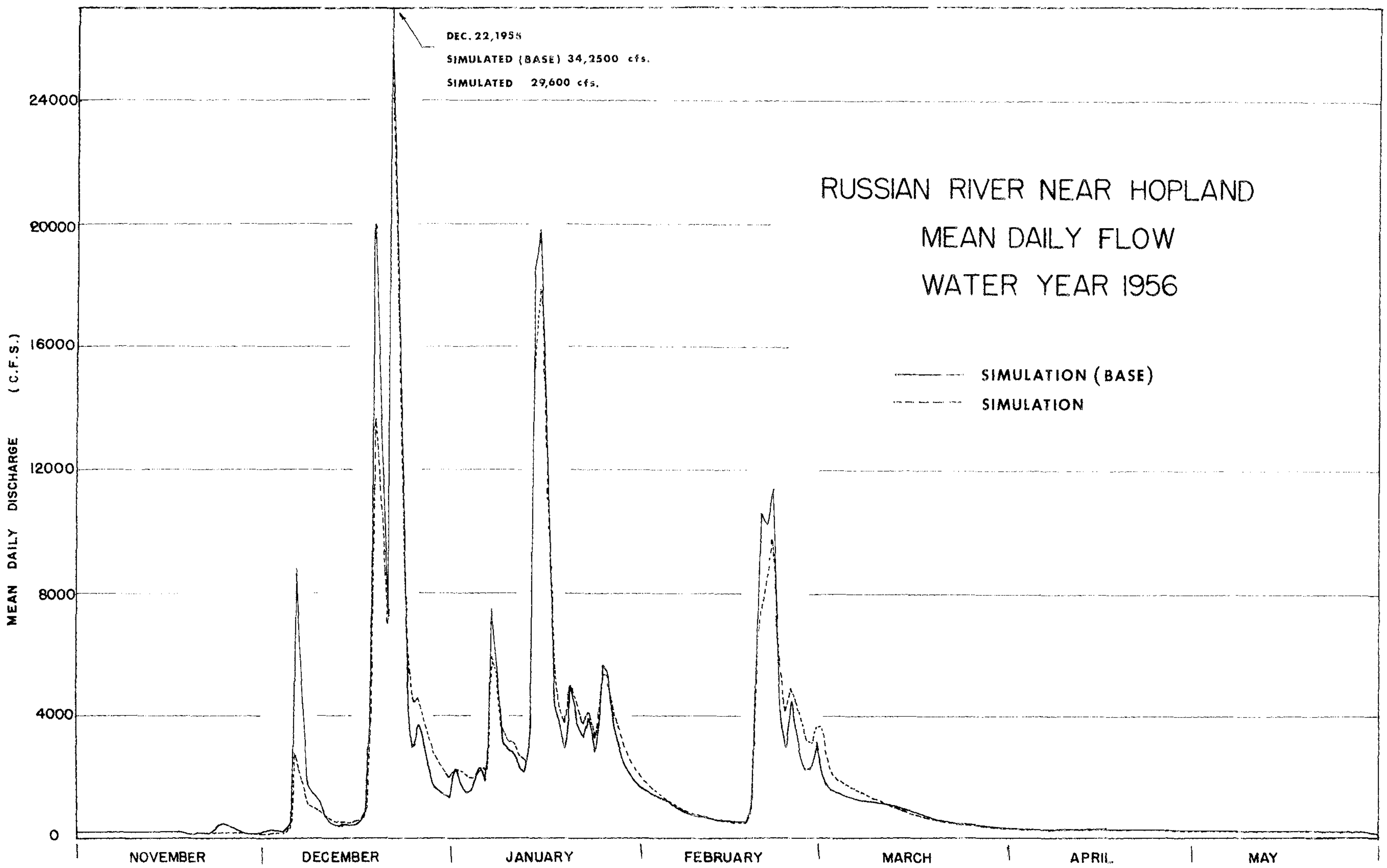

RUSSIAN RIVER NEAR HOPLAND
MEAN DAILY FLOW
WATER YEAR 1956
DEC. 22,1955
SIMULATED (BASE) 34,2500 cfs.
SIMULATED 29,600 cfs.
SIMULATION (BASE)
SIMULATION
MEAN DAILY DISCHARGE (C.F.S.)
24000
20000
16000
12000
8000
4000
0
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY

Figure 5.8

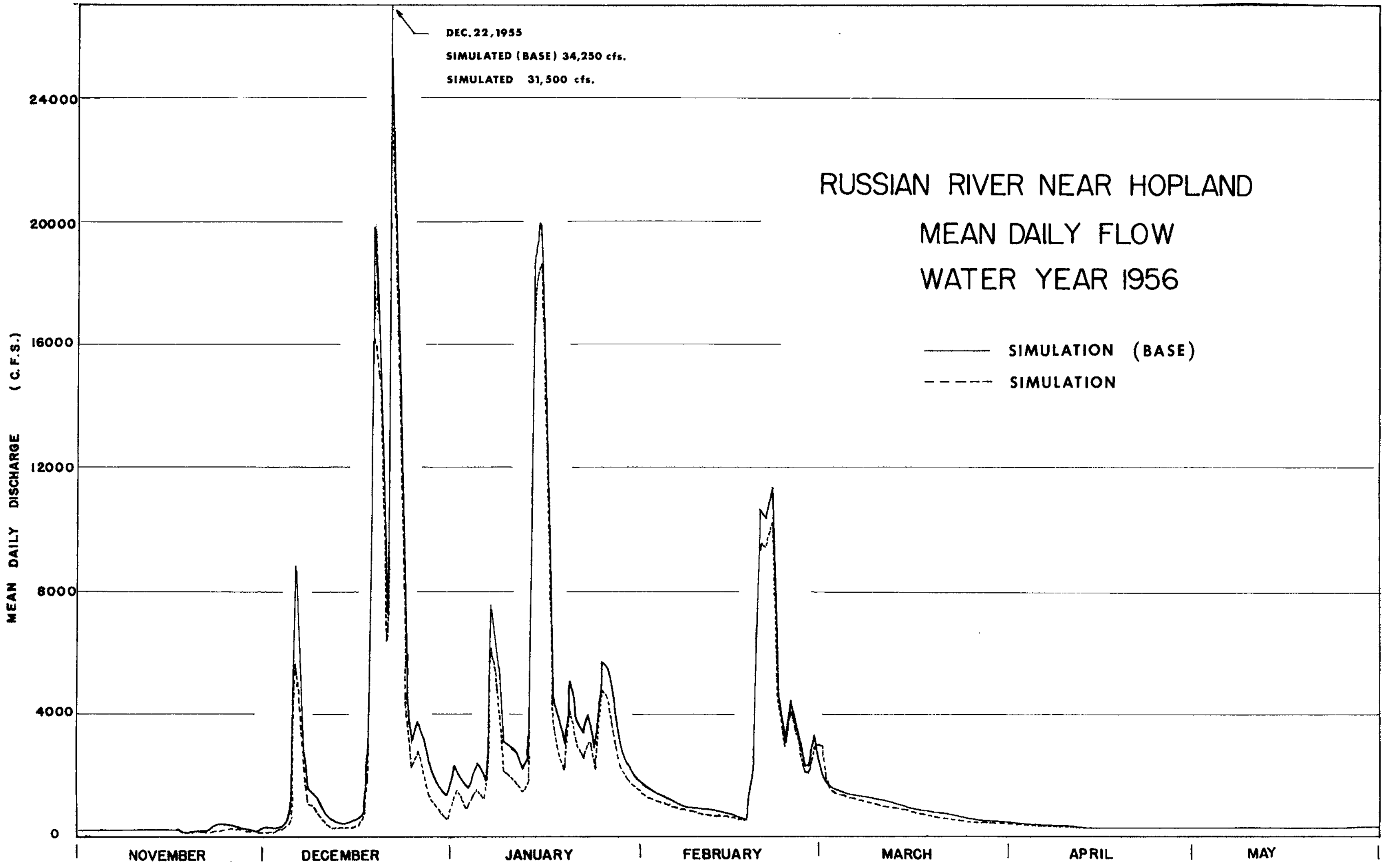

DEC. 22, 1955
SIMULATED (BASE) 34,250 cfs.
SIMULATED 31,500 cfs.
RUSSIAN RIVER NEAR HOPLAND
MEAN DAILY FLOW
WATER YEAR 1956
SIMULATION (BASE)
SIMULATION
MEAN DAILY DISCHARGE (C.F.S.)
24000
20000
16000
12000
8000
4000
0
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY

Figure 5.9

necessary to find an optimum combination of storage and infiltration levels.

The remaining volume parameter, the upper zone parameter UZSN, governs depression storage and storage in the soil profile near the land surface. Physically, these are temporary storages with high or infinite inflow rates and low capacities. They act to retain or delay water for later infiltration, that might otherwise reach a stream channel. These elements will completely dominate showers and small storms and are important in the early stages of larger storms. Interaction between the upper zone (UZSN) and the general infiltration level (CB) occurs in all regimes, and is particularly effective when infiltration rates remain significant. The temporary storages still operate but tend to become less important in sequences of storms governed by storage capacity limitation conditions, since they may not have sufficient opportunity to decrease their storage between storms without moderately high infiltration rates.

The value of UZSN relative to LZSN can be approximately defined as a function of watershed topography and cover. An estimate of UZSN relative to LZSN can be found from Table 5.9. Table 5.9 can be used for an initial estimate of the relationship between two of the three unspecified volume parameters.

Watershed	UZSN
Steep slopes limited vegetation, low depression storage.	0.06 LZSN
Moderate slopes, moderate vegetation, moderate depression storage.	0.08 LZSN
Heavy vegetal or forest cover, soils subject to cracking, high depression storage, very mild slopes.	0.14 LZSN

Table 5.9

In summary, for the volume parameters UZSN, LZSN, and CB the following procedure is recommended:

i) Assume a value for LZSN. An initial estimate in inches of 4 + ¼ (Mean Annual Rainfall) can be used where rainfall is seasonal, and 4 + 1/8 (Mean Annual Rainfall) can be used where rainfall is reasonably uniform throughout the year. These initial estimates will generally need revision.

ii) Select a value for UZSN from Table 5.9.

iii) Assume a value for CB. This value is normally in the range from 0.3 to 1.2, depending on the characteristics of watershed soil profiles.

With the parameters obtained simulate a period of recorded streamflow data. If the stream reaches storage limitation, the parameters LZSN and CB will be relatively independent (Figures 5.8 and 5.9). LZSN can be adjusted to reproduce observed annual yields, and CB can be adjusted to obtain the best fit to surface and groundwater flows in individual storms in each water year.

If the stream generally retains moderate to high infiltration rates, LZSN and CB become more dependent but are still separable to a degree. The combination of LZSN and CB that will most satisfactorily reproduce long term groundwater and surface runoff volumes, and short term response in individual storms, must be found.

Programmed Guides to Optimization Subroutines are used in the general model to assist in establishing infiltration and moisture storage parameters, when records of runoff from individual watershed segments are available. For watersheds that avoid storage limitation, various combinations of LZSN and CB need to be examined. The following subroutines in the general model analyse, storage and infiltration relationships and check the consistency of data.

The water balance equation must apply over any time period with or without precipitation. Thus in the general model

$$\begin{aligned} P = E + \text{UZS inflow} - \text{UZS outflow} + \text{LZS inflow} \\ - \text{LZS outflow} + \text{SGW inflow} - \text{SGW outflow} + R \end{aligned} \qquad (5.2)$$

where P is precipitation, E is actual evapotranspiration, R is runoff, UZS is upper zone storage, LZS is lower zone storage, and SGW is groundwater storage.

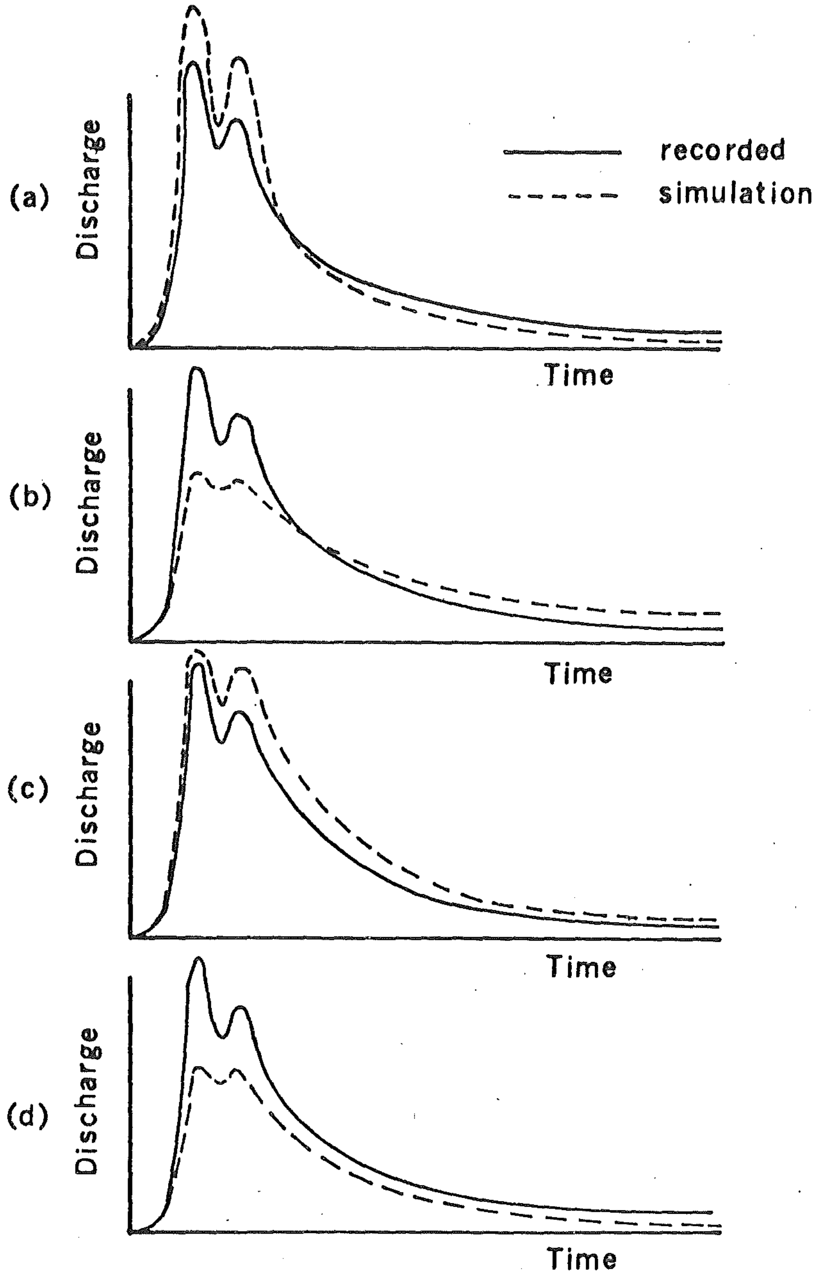
A storm period is defined as a time interval that starts when UZS exceeds $(1.5)\cdot\text{UZSN}$ and ends when UZS returns to $(1.5)\cdot\text{UZSN}$. From the general model structure the outflow from the lower zone during this period is zero. The inflow and outflow from the upper zone will, of course, cancel and Equation 5.2 reduces to:

$$P = E + \text{SGW inflow} - \text{SGW outflow} + \text{LZS inflow} + R \qquad (5.3)$$

for any storm period.

A subroutine based on Equation 5.3 is used to check the consistency of the data and to optimize infiltration levels. The subroutine operates on intervals of severaldays and avoids the problems caused by errors in instantaneous rates or rainfall and runoff. The calculations represented in Equation 5.3 are checked against two relatively stable indices; runoff volumes during an extended period, and groundwater outflow at the end of the period. The total volume of storage added during a storm to the groundwater and lower zone storages varies directly with infiltration rates. The subroutine eliminates consideration of minor storms by requiring that SGW inflow exceed SGW outflow for all storms that are analyzed.

For moderate and large storms, the groundwater outflow at the end of the storm period is an index to groundwater inflow during the storm; which in turn is an index to total infiltration. For example, if calculated groundwater outflow at the end of the storm period exceeds the recorded flow, the infiltration during the storm was too great. The volume of surface runoff varies inversely with infiltration rates. Therefore, from each storm period two indications about infiltration levels are obtained. Strongly conflicting indications imply that the data input was inconsistent during the storm period. Figure 5.10 is a collection of hydrographs that summarize possible response.



Indicated Correction in Infiltration:	(a)	(b)	(c)	(d)
from surface runoff volume	increase	decrease	increase	decrease
from groundwater flow	increase	decrease	decrease	increase
Remarks on results	consistent		conflicting	

Figure 5.10

The subroutines used in the general model are controlled by DCS(1) and DCS(2) on the CONTROL input card. DCS(1) will print the output shown in Table 5.10, for all storm periods in which inflow to groundwater storage exceeds outflow from groundwater storage.

Heading	Sample of Output	Remarks
STORM PERIOD	2/15/16 to 3/1/13	Month, day, and hour of storm
PRECIP	2.12	Precipitation
EVAP	0.54	Actual evapotranspiration
SGWIN	0.15	Groundwater inflow
SGWOUT	0.02	Groundwater outflow
LZSIN	1.31	Increase in lower zone storage
SURF-RO	0.20	Overland flow
INTER-RO	0.06	Interflow
TOTAL-RO	0.28	Total runoff
INFILT-UP	0.65	Infiltration through upper zone
INFILT-DIR	0.80	Direct infiltration
CB/GW	0.42	Indicated CB from groundwater discharge
CB/RO	0.28	Indicated CB from total runoff
SGWCOR	0.01	Groundwater storage correction (Optional-DCS(2))
CB	0.30	Altered CB (Optional-DCS(2))
LZRAT	0.62	Lower zone storage ratio (LZS/LZSN)

Table 5.10

DCS(2) corrects groundwater storage to reproduce measured streamflow at the end of each storm period. When consistent indications for a change in infiltration levels are obtained, DCS(2) alters the value of CB. A summary of overall data consistency is printed for each water year. Therefore, each run indicates values of CB and the results of several runs will establish the best value of CB for a given value of LZSN.

Seasonal changes in potential evapotranspiration and rainfall cause seasonal variations in moisture storage levels. Simulation of total infiltration, groundwater accretion, and runoff in storms at all times of the year, may require experimenting with the volume parameters and the DCS (2) output in watersheds where LZSN and CB are dependent. It is sometimes useful to test the sensitivity of the watershed under study to the volume parameters, as was done for the Russian River in Figures 5.8 and 5.9. When the separate effects on output of the storage and infiltration parameters are known, it becomes fairly easy to interpret the indications from the subroutines and select parameters that will closely reproduce recorded streamflows.

Illustrations of Model Response

The operation of processes in the general model can be studied by printing detailed output from natural storms, or by constructing hypothetical storms. Several examples of model behavior are included in this section.

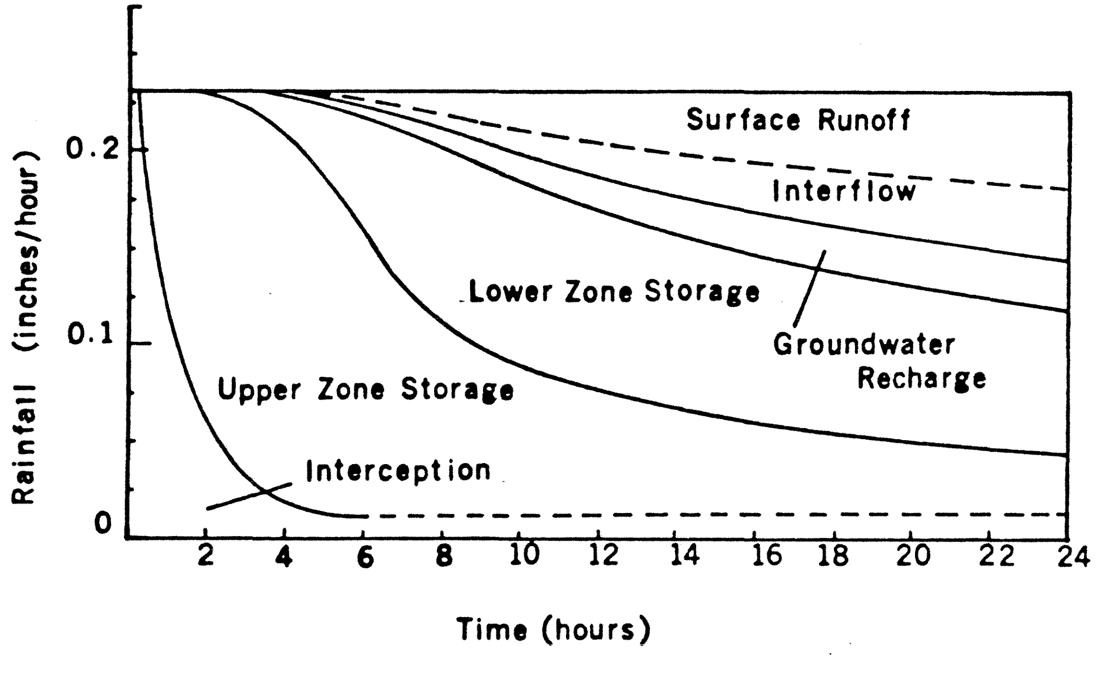


Figure 5.11

Figure 5.11 is a sketch of model response to a uniform rainfall of 0.23 inches per hour for twenty-four hours. Note that interception and the upper zone storage, control the response for the first two or three hours, but they control less than twenty percent of the rainfall after one day. Interception is shown as a dotted line after the first few hours since it depends on the diurnal variation in potential evapotranspiration.

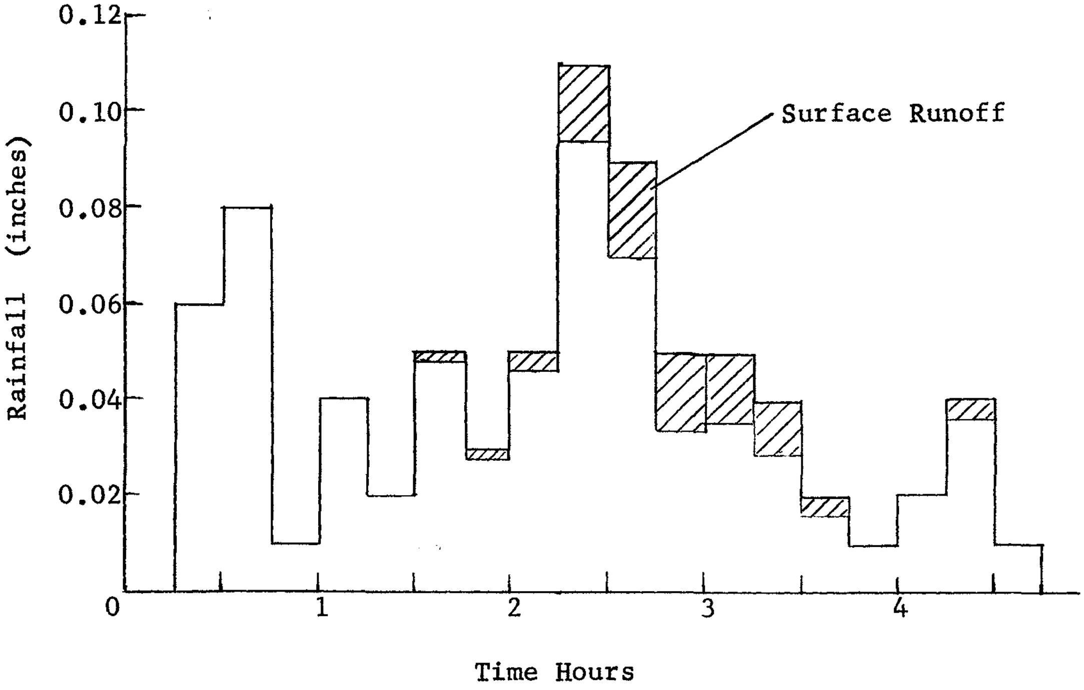


Figure 5.12

Simulation for a sequence of natural rainfall is shown in Figure 5.12. Overland flow for 15-minute intervals is plotted on the rainfall histogram. Infiltration, part of which causes interflow, accounts for the remainder of the rainfall histogram, when changes in overland flow surface detention are neglected. The increase in infiltration as rainfall intensity increases can be seen.

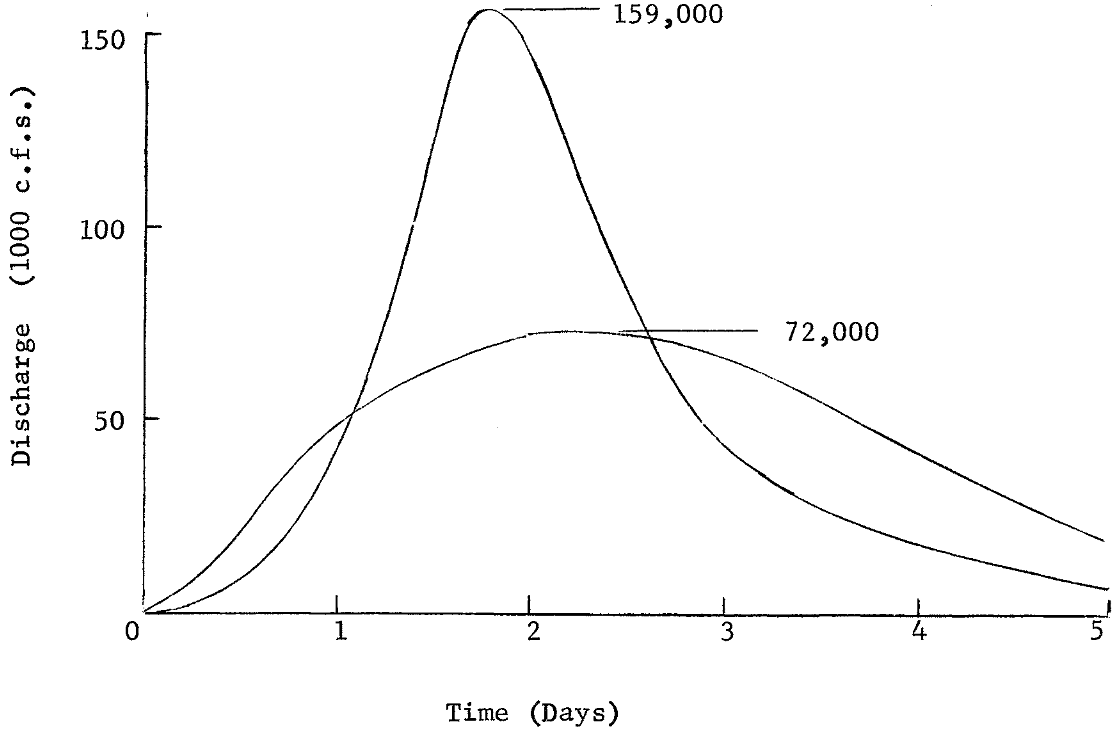


Figure 5.13

Finally, hydrographs for storms with different areal rainfall distributions are plotted in Figure 5.13. Two hypothetical storms, one designed to produce a maximum peak flow, and the other designed to produce a minimum peak flow, resulted in the two hydrographs shown for the Russian River. It is very unlikely that the hydrographs of two natural storms would approach the variation shown in Figure 5.13, but areal rainfall distributions can strongly influence hydrograph shapes in large rivers. The duration of both of the storms used to produce Figure 5.13 was thirty hours, and both storms had the same total volume of runoff.

VI. SIMULATION RESULTS

In this chapter summaries of the output obtained from simulation runs in five watersheds are given. These watersheds were selected from the forty watersheds that have been analyzed to date, to obtain a broad range in climate and in physical characteristics. Simulation output for several other watersheds has been included in previous reports.[4,36]

Measurements of accuracy are considered in the first section of this chapter. Summaries of output are given in the second section, and the concluding section contains some general remarks about results.

Accuracy and Statistical Comparisons

An analysis of the accuracy of streamflow is an attempt to measure the inherent ability of the simulation model to respond as a natural stream would respond. Two factors bear on this analysis. First, the adequacy of the rainfall network to correctly represent the rainfall on the watershed, and second, the accuracy of the stream gage recording watershed response. In almost all cases the adequacy of rainfall input is the major factor. Streamflow at a single point is less troublesome to measure than rainfall over an extended area. Nonetheless, stream gaging accuracy alone does provide a limiting criteria. There is no basis for attempting to improve the accuracy of simulation beyond the accuracy of stream gaging.

In addition to the tables and plots of recorded and simulated flows, two statistical measures of accuracy are used. Table 6.1 is called a Flow Duration and Error Table; it gives for each water year a summary of the number of days when the recorded mean daily flow was within an assigned interval, and the average error, average absolute error, and standard error in the simulated mean daily flows for these days. Also, the last line of Table 6.1 gives the correlation coefficient obtained by matching the simulated and recorded flows each day.

Daily Flow Duration and Errors: Recorded/Simulation Comparison
Russian River at Hopland, 1955-56

Flow Interval	Cases	Average Error (c.f.s.)	Average Absolute Error (c.f.s.)	Standard Error (c.f.s.)
90-148	3	-31	31	1
148-245	169	0	14	20
245-403	43	-46	57	46
403-665	46	-81	88	49
665-1097	28	32	110	202
1097-1808	22	173	344	551
1808-2981	18	-88	308	344
2981-4914	20	-125	462	557
4914-8103	7	423	1304	1598
8103-13360	3	1101	1701	2230
13360-22026	6	-598	2050	2376
22026-361315	1	448		

Correlation Coefficient (Daily) 0.9902

Table 6.1

Daily Flow Duration and Errors: Recorded/Modified Flow Comparison
Russian River at Hopland, 1955-56

Flow Interval (c.f.s.)	Cases	Average Error (c.f.s.)	Average Absolute Errors (c.f.s.)	Standard Errors (c.f.s.)
90-148	3	10	12	20
148-245	169	1	24	30
245-403	43	3	37	46
403-665	46	-3	58	69
665-1097	28	3	81	109
1097-1808	22	-65	169	191
1808-2981	18	27	336	401
2981-4914	20	-109	427	515
4914-8103	7	-130	634	789
8103-13360	3	-53	347	520
13360-22026	6	-192	1614	2313
22026-361315	1	-5445		

Correlation Coefficient (Daily) 0.9905

Table 6.2

In Table 6.1 recorded and simulated mean daily stream flow is compared for water year 1955-56 for the Russian River near Hopland, California. There were 20 days during the year when the recorded mean daily flow was between 2981 and 4914 cfs. The synthesized flow for these twenty days averaged 125 cfs lower than the recorded flow. The average absolute error (462 c.f.s.) and the standard error (557 c.f.s.) comparing recorded and synthesized flows in the interval are also given. The low flows in Table 6.1 (500 cfs and below) are influenced by a diversion into the watershed and should not be considered.

The recorded flow can be regarded as an estimate of the actual flow and differs from this flow by some amount. The U. S. Geological Survey rates the Hopland gage as a good record, for which the error in mean daily flows is said to be generally less than ten percent. Interpreting the ten percent figure as a standard error, a random program was developed that statistically generates a modified mean daily flow that is assumed to be as valid a sample as the recorded mean daily flow. The modified mean daily flow could, for example, have been recorded if a second independent gaging station were operated at the same point on the river. Table 6.2 is a comparison between these hypothetical modified mean daily flows and the recorded mean daily flows, again, for water year 1955-56 on the Russian River at Hopland.

The numerical values in Table 6.2 will, of course, change in successive trials. The purpose of this recorded/modified flow comparison is simply to provide a rough guide for interpreting the recorded/simulation comparison. The recorded/simulation comparison can become, at best, comparable to the recorded/modified flow comparison. For example, the recorded/modified flow comparison for the flow interval between 2981 and 4914 c.f.s. shows the modified mean daily flows averaging 109 c.f.s. less than the recorded mean daily flows. Another run of the random subroutine that calculates modified flows could equally well result in flows averaging 100 or 150 cfs higher than the recorded flows. Only the general magnitudes of these numbers are of interest. The recorded/simulation comparison in Table 6.1 does show slightly more variation than the recorded/modified flow comparison in Table 6.2. However, the differences are small and could easily be caused by non-representative rainfall input or rainfall and potential evapotranspiration input errors. When this point is reached in simulation, the variations in the basic data used prevent further attempts to improve model parameters.

Examples of Simulation

Figure 6.1 is a map showing the locations of all of the watersheds included in this chapter. The following information is given under a heading for each watershed:

i) A summary of watershed characteristics and special features.
ii) A map of the watershed with the locations of rain gages, streamflow recorders, and evaporation stations.
iii) Tables of monthly and annual recorded and simulated streamflows for each flowpoint, and correlation coefficients for mean daily recorded and simulated flows, for each water year in the simulation period.
iv) A graphical plot of recorded and simulated mean daily streamflow for one or more water years.
v) A plot of simulated and recorded peak flow frequency.
vi) General comments about the results for the watershed.

Hydrographs of major flood peaks and flow duration data are included for some of the watersheds. An example of the extension of streamflow records based on parameters developed in the Russian River, and an example of simulation on a stream that was assumed to be ungaged, are included in the section titled Hydrometeorological Networks in Chapter VII.

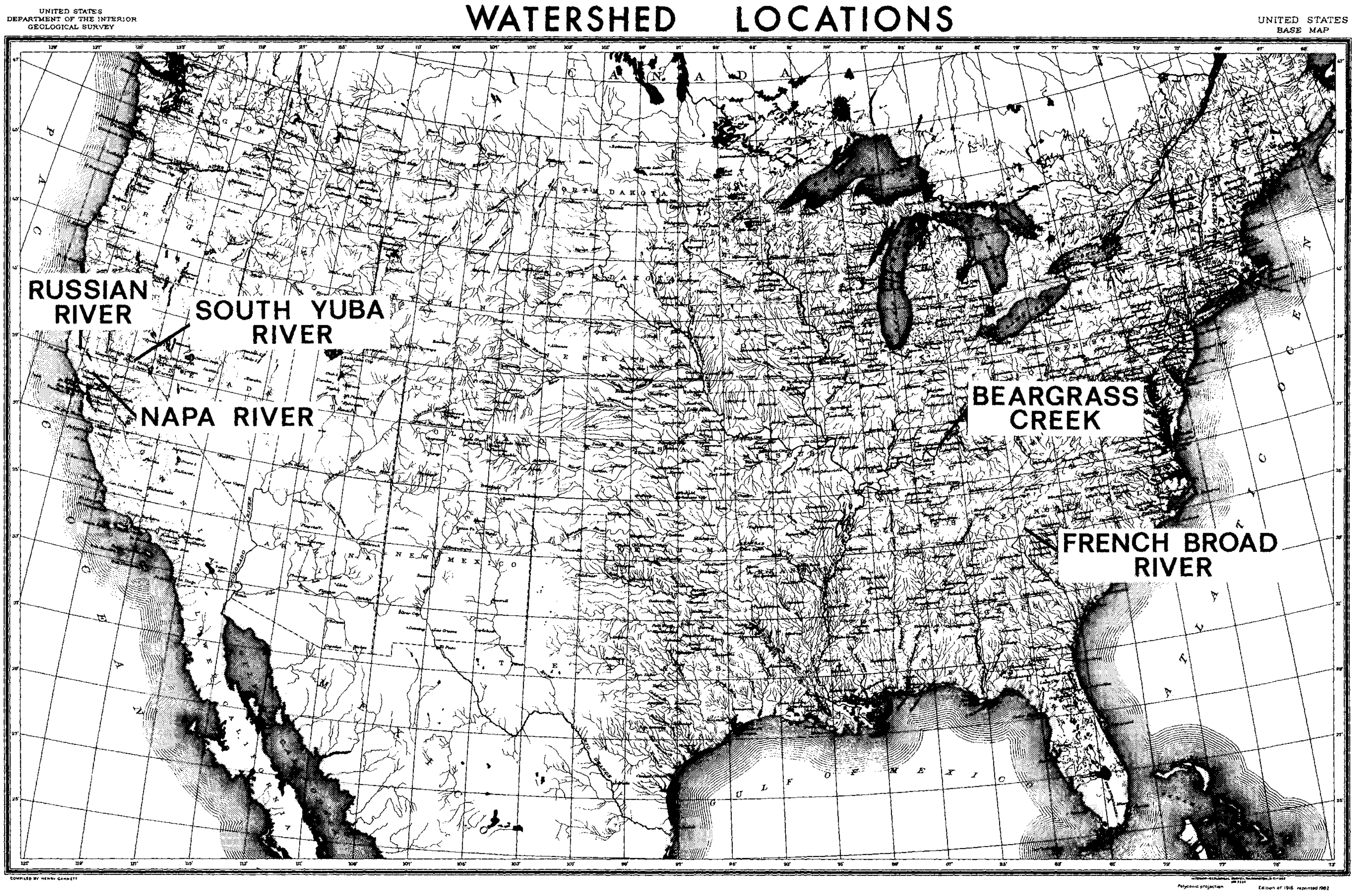

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATERSHED LOCATIONS
UNITED STATES
BASE MAP
RUSSIAN RIVER
SOUTH YUBA RIVER
NAPA RIVER
BEARGRASS CREEK
FRENCH BROAD RIVER
C A N A D A
NORTH DAKOTA
ARIZONA
NEW MEXICO
TEXAS
GULF OF MEXICO
PACIFIC OCEAN
ATLANTIC OCEAN
Polyconic projection
Edition of 1916 reprinted 1962

Figure 6.1

Russian River The Russian River flows into the Pacific about sixty miles north of San Francisco. The watershed contains the steeply folded and faulted sedimentary rocks of the Coast Range Mountains. Land slopes are moderate to steep but the main channel slopes are mild. The climate is mild with summer drought and winter rain. Mixed broadleaf deciduous and needleleaf evergreen trees are the natural vegetation. Land use is primarily ranching, with some irrigated farm lands on the valley floor.

Streamflow was simulated at three flowpoints that correspond to the streamgages at Hopland, Healdsburg, and Guerneville, California. A summary of watershed characteristics follows:

Flowpoint 1: Hopland
Stream Gage: Russian River near Hopland, California.
Area: 362 square miles
Elevation Range: 467 feet to 3300 feet
Average Annual Precipitation: 44 inches
Average Annual Runoff: 18 inches
Average Annual Potential Evapotranspiration: 38 inches
Segments: 1 - 232 square miles; recorder Redwood Valley; storage gage none.
2 - 87 square miles; recorder Potter Valley; storage gage none.
3 - 43 square miles; recorder Hopland 8NE; storage gage Hopland Largo St.

Flowpoint 2: Healdsburg
Stream Gage: Russian River near Healdsburg, California
Area: 791 square miles
Elevation Range: 77 feet to 4300 feet
Average Annual Precipitation: 45 inches
Average Annual Runoff: 18 inches
Average Annual Potential Evaportranspiration: 40 inches
Segments: 1 - 95 square miles; recorder Hopland 8NE; storage gage Hopland Largo St.

2 - 78 square miles; recorder Yorkville;
storage gage Hopland Largo St.
3 - 258 square miles; recorder The Geysers;
storage gages Cloverdale, Healdsburg.

Flowpoint 3: Guerneville
Stream Gage: Russian River near Guerneville, California
Area: 1342 square miles
Elevation Range: 70 feet to 4300 feet
Average Annual Precipitation: 43 inches
Average Annual Runoff: 17 inches
Average Annual Potential Evapotranspiration: 42 inches
Segments: 1 - 99 square miles; recorder Cloverdale 11W;
storage gage Cloverdale 3SE.
2 - 187 square miles; recorder Venada;
storage gages Cloverdale, Healdsburg
3 - 93 square miles; recorder St. Helena;
storage gage none
4 - 170 square miles; recorder Sebastopol;
storage gage Santa Rosa R.S.

Water is diverted into the Russian River above the Hopland gage from the Eel River, through the Potter Valley powerhouse. This diversion, and estimated irrigation diversions are used as input data. The maximum diversion from the Eel River is about 340 cubic feet per second. Dry Creek near Cloverdale, a Russian River tributary, was one of the first streams simulated by digital computer methods.[2]

Figure 6.2 is a map of the Russian River showing gage locations. Tables 6.3, 6.4, and 6.5 give the monthly and annual recorded and simulated runoff for the Hopland, Healdsburg, and Guerneville gages. Figures 6.3, 6.4, and 6.5 are plots of recorded and simulated mean daily flows for the Hopland, Healdsburg, and Guerneville gages for water year, 1956. Figure 6.6 shows recorded and simulated mean daily flows for the Healdsburg gage for water year, 1953. Figures 6.7, 6.8, and 6.9 show peak flow frequency based on the record and simulated flows during the

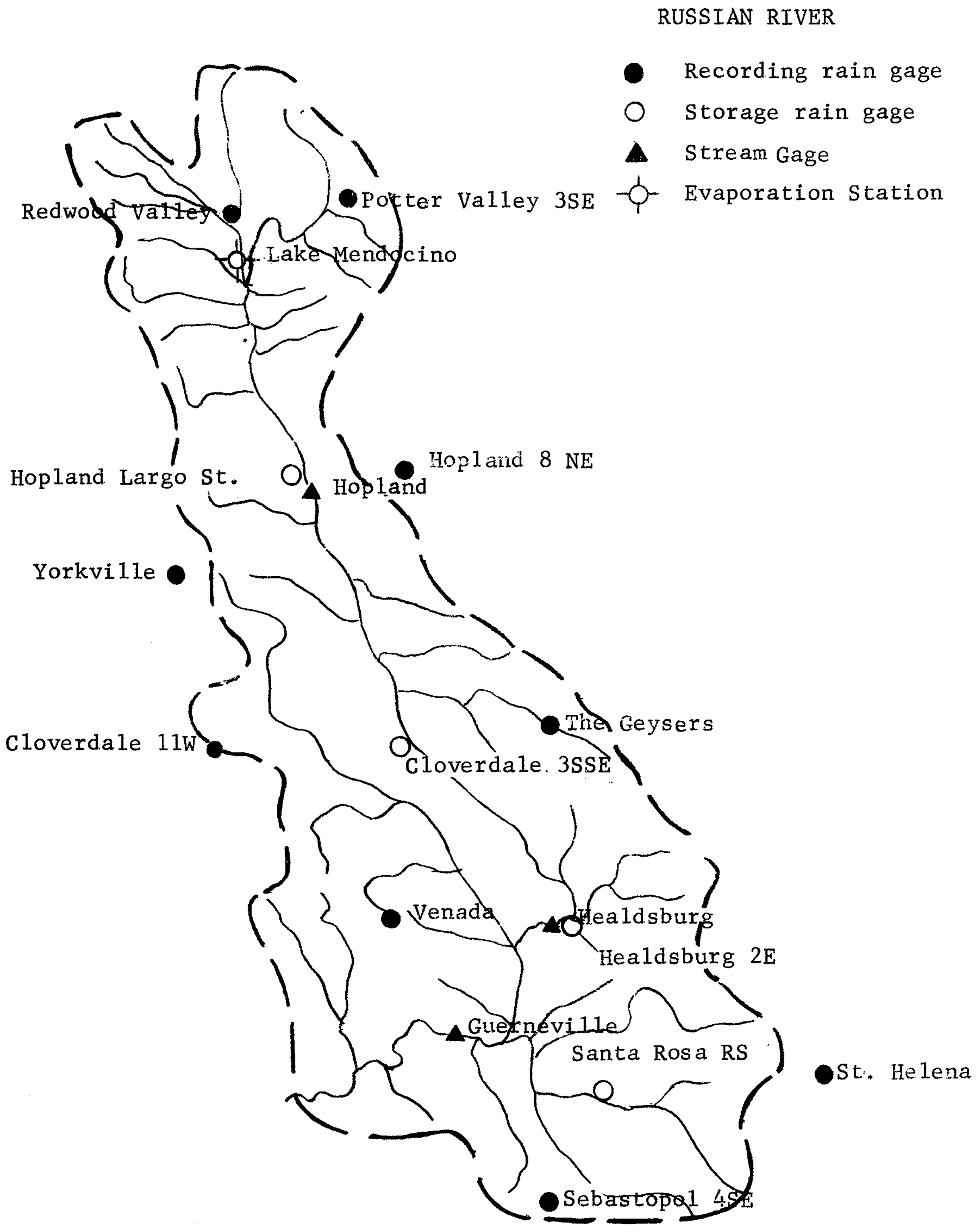

RUSSIAN RIVER
● Recording rain gage
○ Storage rain gage
▲ Stream Gage
Evaporation Station
Redwood Valley
Potter Valley 3SE
Lake Mendocino
Hopland 8 NE
Hopland Largo St.
Hopland
Yorkville
The Geysers
Cloverdale 11W
Cloverdale 3SSE
Venada
Healdsburg
Healdsburg 2E
Guerneville
Santa Rosa RS
St. Helena
Sebastopol 4SE

Figure 6.2

FLOWPOINT 1: HOPLAND

Water	Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Flow)
51	Rec	1.12	2.22	7.64	9.25	6.07	3.40	1.35	1.28	0.45	0.39	0.41	0.44	34.0	0.9788
	Sim	1.20	1.98	8.68	9.69	6.85	3.55	1.28	1.43	0.57	0.45	0.42	0.47	36.6	
52	Rec	0.63	1.41	8.58	10.2	7.78	5.44	1.72	1.20	0.84	0.56	0.80	0.67	39.9	0.9689
	Sim	0.67	1.24	10.4	10.6	6.91	5.47	1.54	1.06	0.81	0.55	0.81	0.67	40.9	
53	Rec	0.66	0.54	6.14	11.9	1.37	3.21	1.95	1.66	1.17	0.78	0.80	0.72	31.0	0.9841
	Sim	0.70	0.49	7.77	13.7	1.80	3.03	1.65	1.29	1.04	0.89	0.87	0.79	34.1	
54	Rec	0.84	1.44	1.74	7.06	4.47	4.31	4.02	1.30	0.65	0.51	0.58	0.80	27.7	0.9653
	Sim	0.91	1.12	1.49	6.74	5.50	4.19	3.80	1.30	0.67	0.53	0.55	0.83	27.7	
55	Rec	0.96	0.93	2.95	3.50	1.69	1.22	1.61	1.23	0.65	0.59	0.57	0.60	16.5	0.9441
	Sim	1.01	0.90	2.45	3.30	1.77	1.28	0.86	1.20	0.66	0.64	0.61	0.63	15.3	
56	Rec	0.70	0.78	13.6	13.5	9.00	3.53	1.42	1.15	0.58	0.57	0.61	0.61	46.2	0.9899
	Sim	0.76	0.72	13.6	15.2	8.87	2.94	1.19	0.98	0.54	0.58	0.61	0.67	46.8	
57	Rec	0.81	0.83	0.51	2.28	4.38	4.94	1.87	2.27	0.95	0.56	0.55	0.68	20.6	0.9723
	Sim	0.74	0.70	0.40	1.89	3.80	5.46	1.64	1.75	0.85	0.60	0.56	0.64	19.1	
58	Rec	1.76	1.94	3.44	7.29	19.5	5.62	6.62	1.05	0.79	0.62	0.60	0.69	50.0	0.9637
	Sim	2.12	1.38	2.86	8.57	16.8	5.75	5.61	1.03	0.71	0.54	0.51	0.70	46.7	

Table 6.3

FLOWPOINT 2: HEALDSBURG

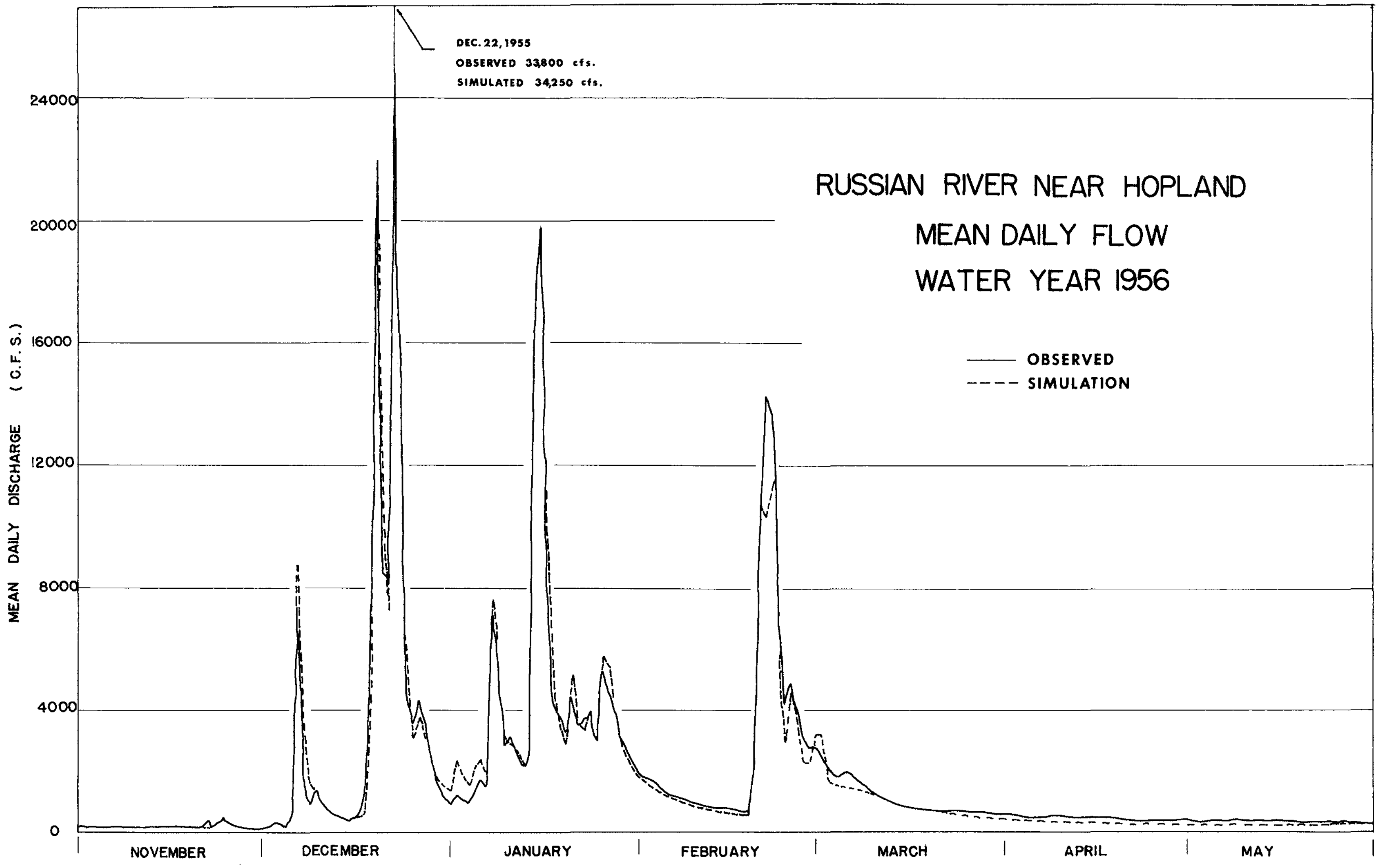
Water Year		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Discharge)
51	Rec	0.82	2.88	7.65	7.41	4.95	2.61	0.90	1.11	0.30	0.18	0.17	0.18	29.2	0.9650
	Sim	0.62	1.59	8.17	7.57	5.35	2.66	0.79	1.28	0.37	0.26	0.22	0.23	29.1	
52	Rec	0.30	0.96	9.33	11.1	5.90	4.98	1.34	0.80	0.46	0.29	0.35	0.28	36.2	0.9842
	Sim	0.34	0.68	10.0	10.7	5.81	5.16	1.08	0.62	0.44	0.31	0.40	0.32	35.9	
53	Rec	0.29	0.28	7.58	12.9	1.23	2.79	1.50	1.15	0.68	0.37	0.37	0.34	29.5	0.9791
	Sim	0.34	0.25	7.94	12.5	1.47	2.62	1.13	0.80	0.57	0.46	0.43	0.38	29.0	
54	Rec	0.41	1.10	1.06	7.18	4.66	3.95	3.56	0.95	0.32	0.22	0.28	0.39	24.1	0.9757
	Sim	0.45	0.90	0.92	7.25	5.06	4.26	3.26	0.87	0.42	0.30	0.29	0.43	24.4	
55	Rec	0.45	1.08	2.96	2.60	1.15	0.95	1.65	1.04	0.34	0.24	0.22	0.25	12.9	0.9771
	Sim	0.51	0.93	2.84	2.58	1.32	1.04	1.50	0.99	0.44	0.36	0.32	0.31	13.1	
56	Rec	0.33	0.43	13.0	14.1	9.68	2.70	0.99	0.80	0.30	0.22	0.21	0.24	43.0	0.9854
	Sim	0.37	0.36	13.8	14.3	10.1	2.46	0.74	0.57	0.30	0.29	0.29	0.31	44.0	
57	Rec	0.39	0.43	0.27	1.73	4.59	4.47	1.50	2.11	0.69	0.28	0.20	0.34	17.0	0.9762
	Sim	0.37	0.37	0.22	1.36	4.28	4.64	1.48	1.49	0.62	0.37	0.30	0.51	16.0	
58	Rec	2.33	1.13	2.93	6.29	19.1	6.78	7.71	0.95	0.57	0.31	0.26	0.28	48.7	0.9823
	Sim	1.97	0.87	2.31	7.44	19.8	7.76	7.09	0.71	0.40	0.29	0.25	0.32	49.3	

Table 6.4

FLOWPOINT 3: GUERNEVILLE

Water Year		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Discharge)
51	Rec	0.63	2.63	7.82	6.66	4.28	2.16	0.68	0.88	0.21	0.11	0.10	0.12	26.3	0.9754
	Sim	0.38	1.94	8.04	6.77	4.76	2.44	0.64	0.91	0.27	0.19	0.15	0.14	26.6	
52	Rec	0.19	0.74	9.05	11.4	5.36	4.62	1.12	0.57	0.29	0.19	0.20	0.17	34.0	0.9841
	Sim	0.23	0.52	9.28	10.5	5.35	4.89	0.91	0.46	0.30	0.21	0.26	0.20	33.2	
53	Rec	0.17	0.17	7.37	11.5	0.94	2.22	1.28	0.91	0.45	0.25	0.22	0.21	25.8	0.9848
	Sim	0.22	0.17	7.51	11.5	1.34	2.26	1.04	0.67	0.42	0.32	0.28	0.24	26.0	
54	Rec	0.25	0.81	0.72	6.22	4.38	3.76	3.30	0.74	0.28	0.13	0.14	0.21	21.0	0.9821
	Sim	0.29	0.69	0.68	6.73	4.53	4.05	3.22	0.75	0.33	0.22	0.19	0.28	22.0	
55	Rec	0.28	0.99	2.84	2.36	0.94	0.72	1.48	0.81	0.21	0.15	0.13	0.14	11.1	0.9730
	Sim	0.34	0.92	2.55	2.19	1.09	0.89	1.30	0.84	0.34	0.26	0.21	0.20	11.1	
56	Rec	0.18	0.25	14.9	14.2	9.41	2.28	0.85	0.68	0.24	0.14	0.14	0.16	43.6	0.9939
	Sim	0.24	0.23	14.1	13.8	9.52	2.20	0.57	0.45	0.22	0.20	0.19	0.19	42.0	
57	Rec	0.21	0.27	0.16	1.34	4.36	3.93	1.25	1.78	0.51	0.18	0.12	0.20	14.3	0.9639
	Sim	0.25	0.25	0.15	1.06	3.59	4.03	1.29	1.26	0.53	0.28	0.21	0.34	13.3	
58	Rec	1.67	0.80	2.55	5.62	20.1	7.30	8.22	0.74	0.40	0.19	0.15	0.17	48.0	0.9899
	Sim	1.89	0.68	1.80	6.62	18.9	7.42	7.45	0.60	0.30	0.20	0.16	0.20	46.3	

Table 6.5



DEC. 22, 1955
OBSERVED 33,800 cfs.
SIMULATED 34,250 cfs.
RUSSIAN RIVER NEAR HOPLAND
MEAN DAILY FLOW
WATER YEAR 1956
OBSERVED
SIMULATION
MEAN DAILY DISCHARGE (C.F.S.)
24000
20000
16000
12000
8000
4000
0
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY

Figure 6.3

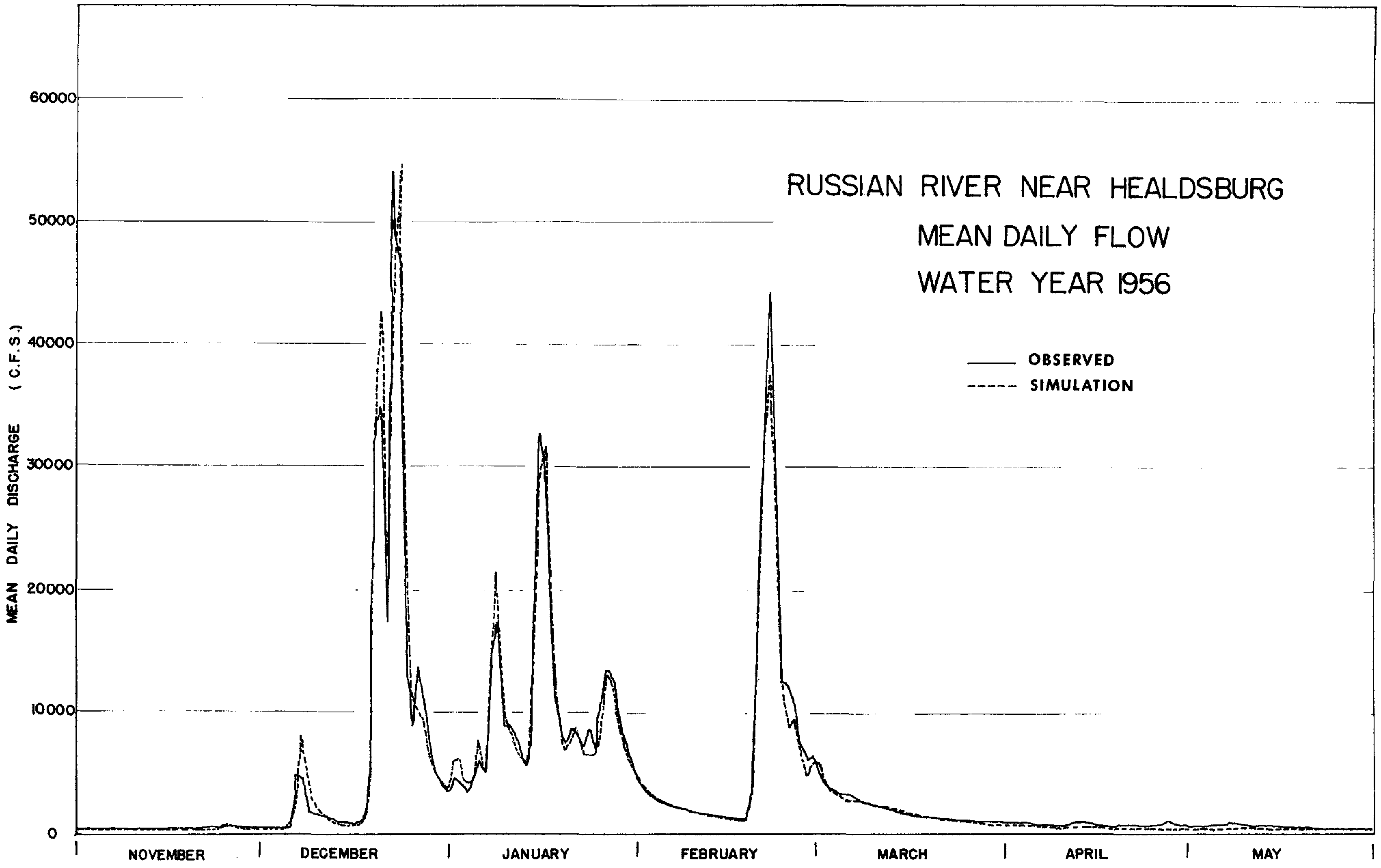

RUSSIAN RIVER NEAR HEALDSBURG
MEAN DAILY FLOW
WATER YEAR 1956
OBSERVED
SIMULATION
MEAN DAILY DISCHARGE (C.F.S.)
60000
50000
40000
30000
20000
10000
0
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY

Figure 6.4

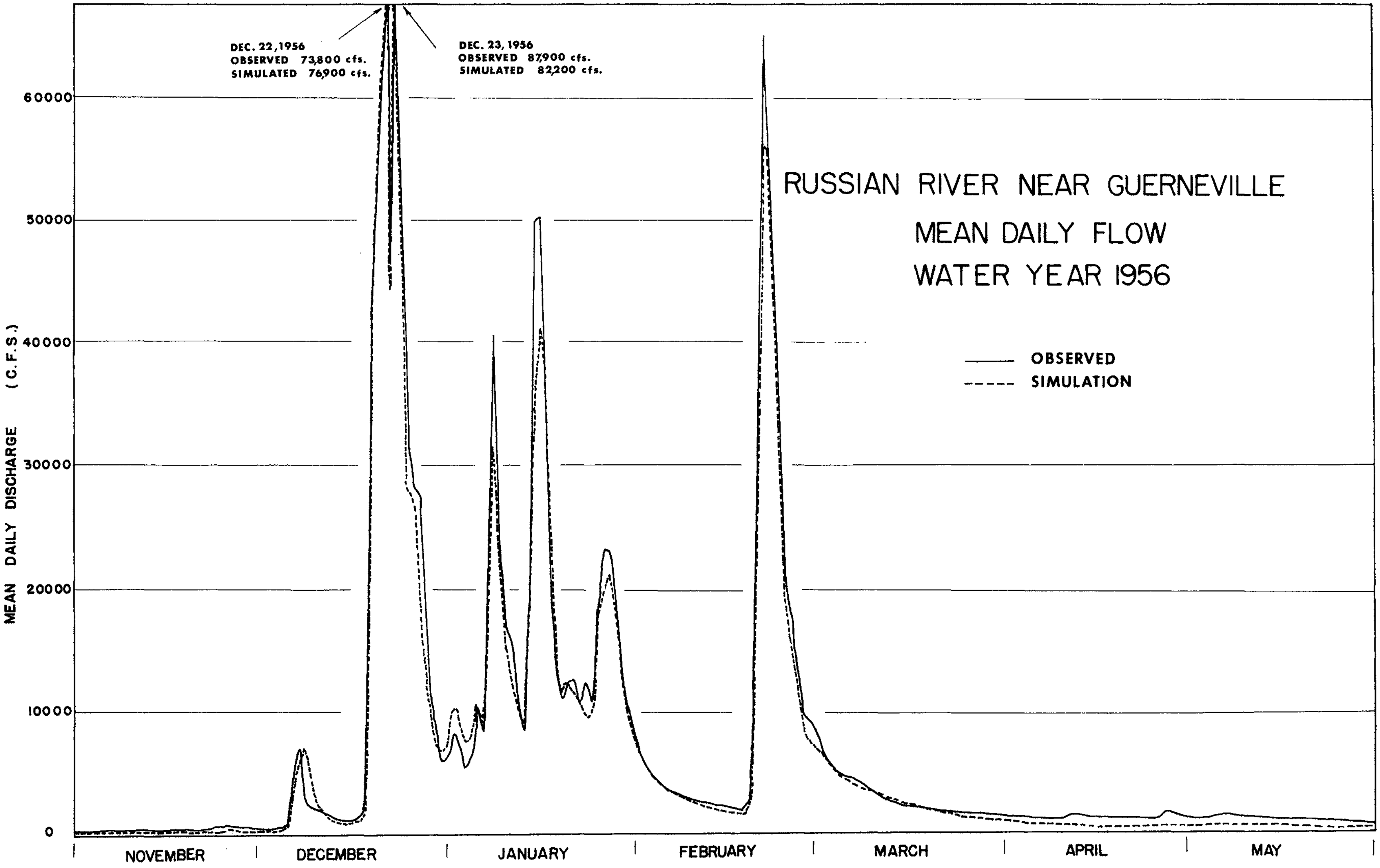

DEC. 22, 1956
OBSERVED 73,800 cfs.
SIMULATED 76,900 cfs.
DEC. 23, 1956
OBSERVED 87,900 cfs.
SIMULATED 82,200 cfs.
RUSSIAN RIVER NEAR GUERNEVILLE
MEAN DAILY FLOW
WATER YEAR 1956
OBSERVED
SIMULATION
MEAN DAILY DISCHARGE (C.F.S.)
60000
50000
40000
30000
20000
10000
0
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY

Figure 6.5

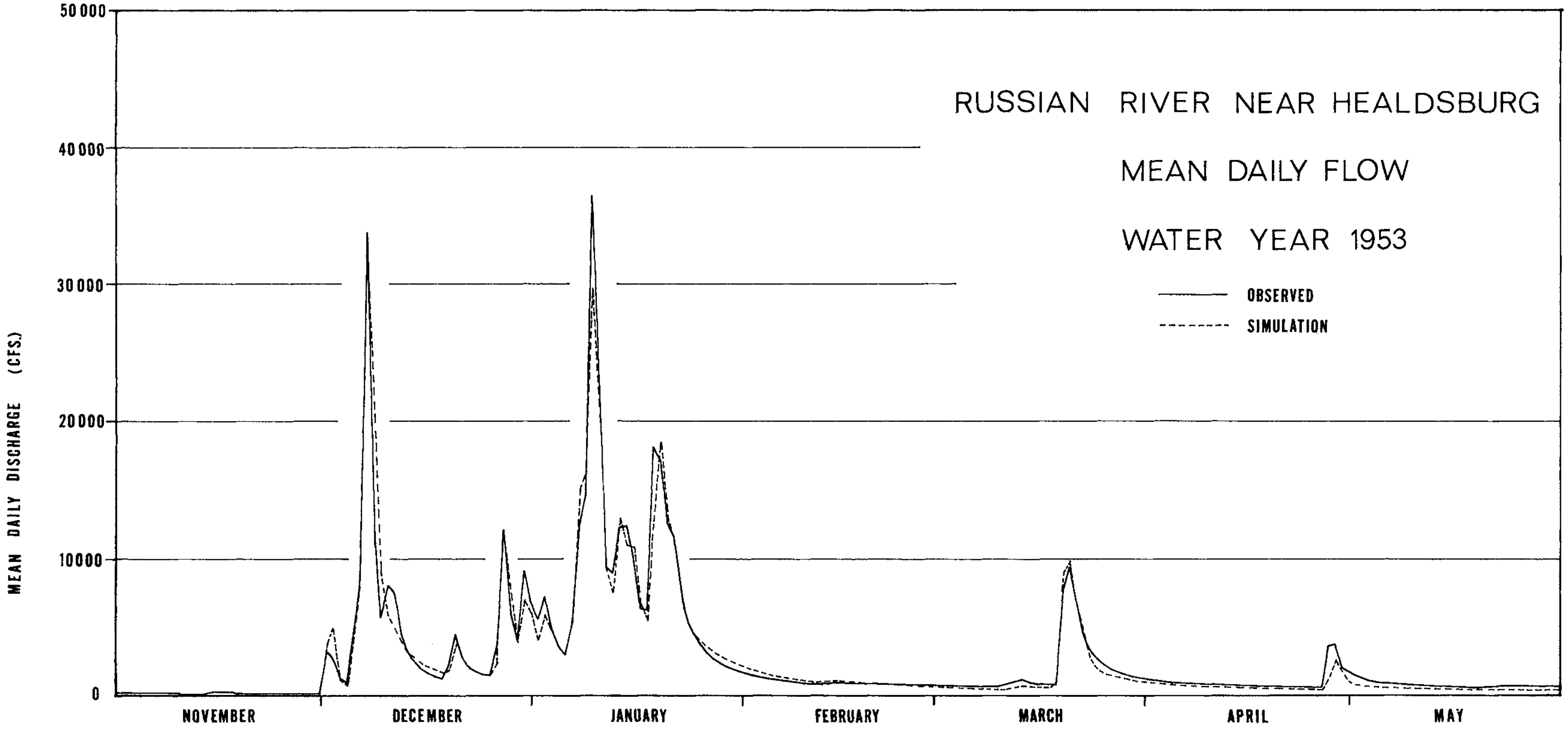

RUSSIAN RIVER NEAR HEALDSBURG
MEAN DAILY FLOW
WATER YEAR 1953
OBSERVED
SIMULATION
MEAN DAILY DISCHARGE (CFS.)
50000
40000
30000
20000
10000
0
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY

Figure 6.6

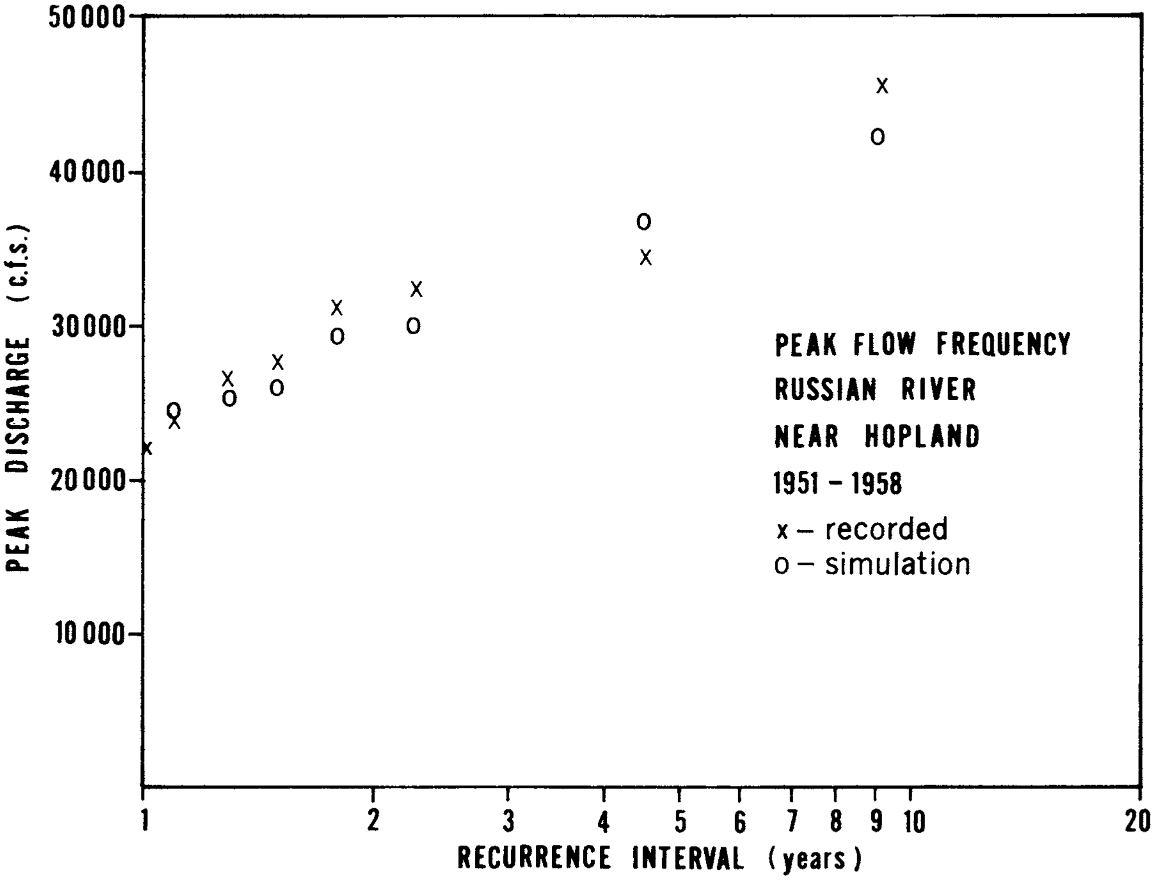

PEAK DISCHARGE (c.f.s.)
50000
40000
30000
20000
10000
1
2
3
4
5
6
7
8
9
10
20
RECURRENCE INTERVAL (years)
PEAK FLOW FREQUENCY
RUSSIAN RIVER
NEAR HOPLAND
1951 - 1958
x – recorded
o – simulation

Figure 6.7

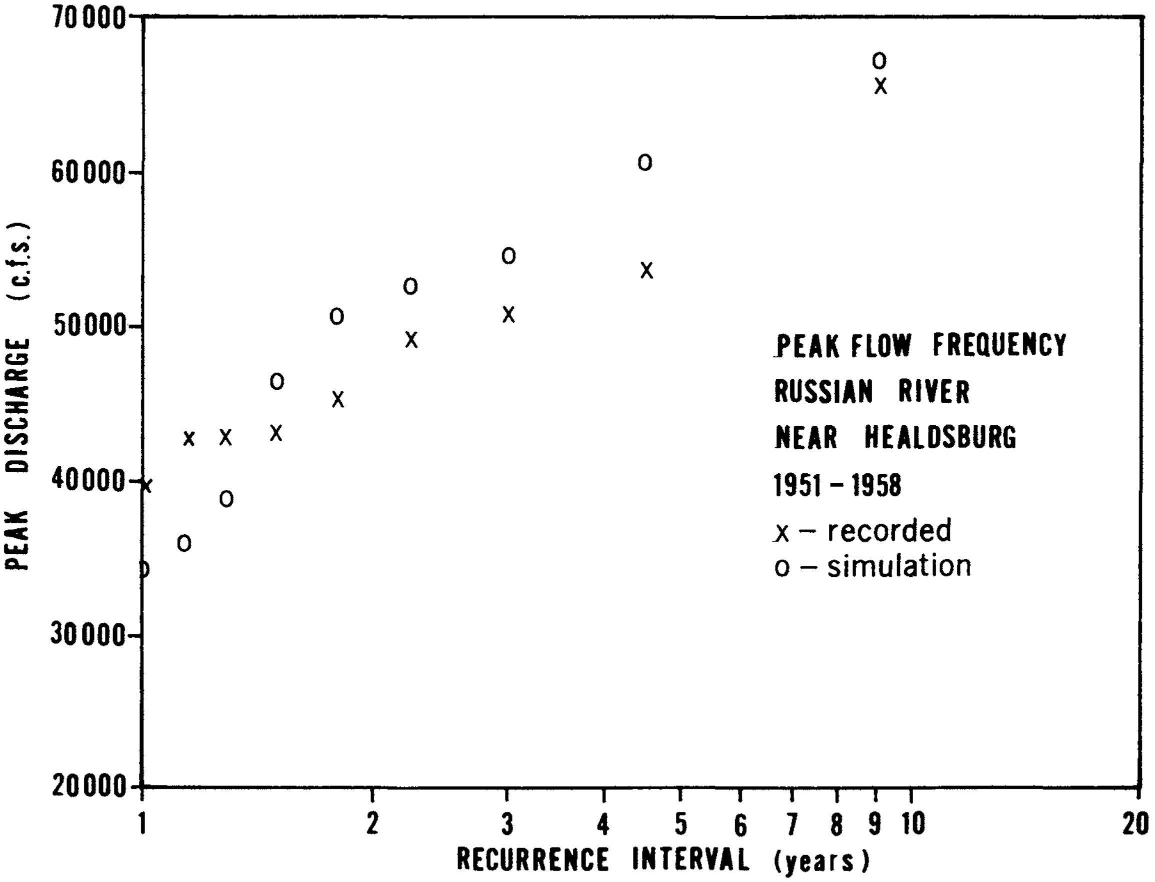

70000
60000
50000
40000
30000
20000
PEAK DISCHARGE (c.f.s.)
1
2
3
4
5
6
7
8
9
10
20
RECURRENCE INTERVAL (years)
PEAK FLOW FREQUENCY
RUSSIAN RIVER
NEAR HEALDSBURG
1951 - 1958
x – recorded
o – simulation

Figure 6.8

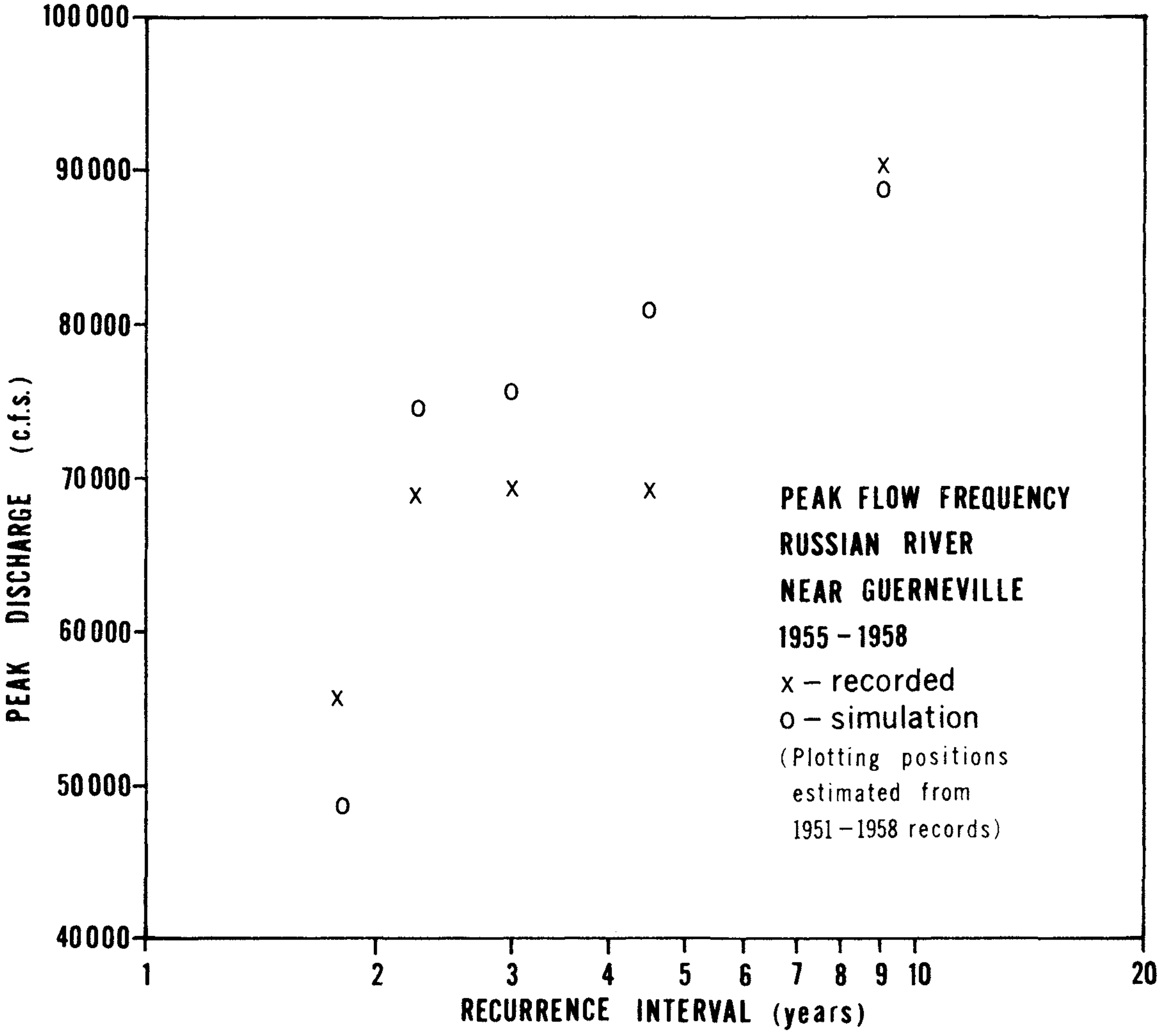

100000
90000
80000
70000
60000
50000
40000
PEAK DISCHARGE (c.f.s.)
1
2
3
4
5
6
7
8
9
10
20
RECURRENCE INTERVAL (years)
PEAK FLOW FREQUENCY
RUSSIAN RIVER
NEAR GUERNEVILLE
1955 - 1958
x – recorded
o – simulation
(Plotting positions
estimated from
1951 – 1958 records)

Figure 6.9

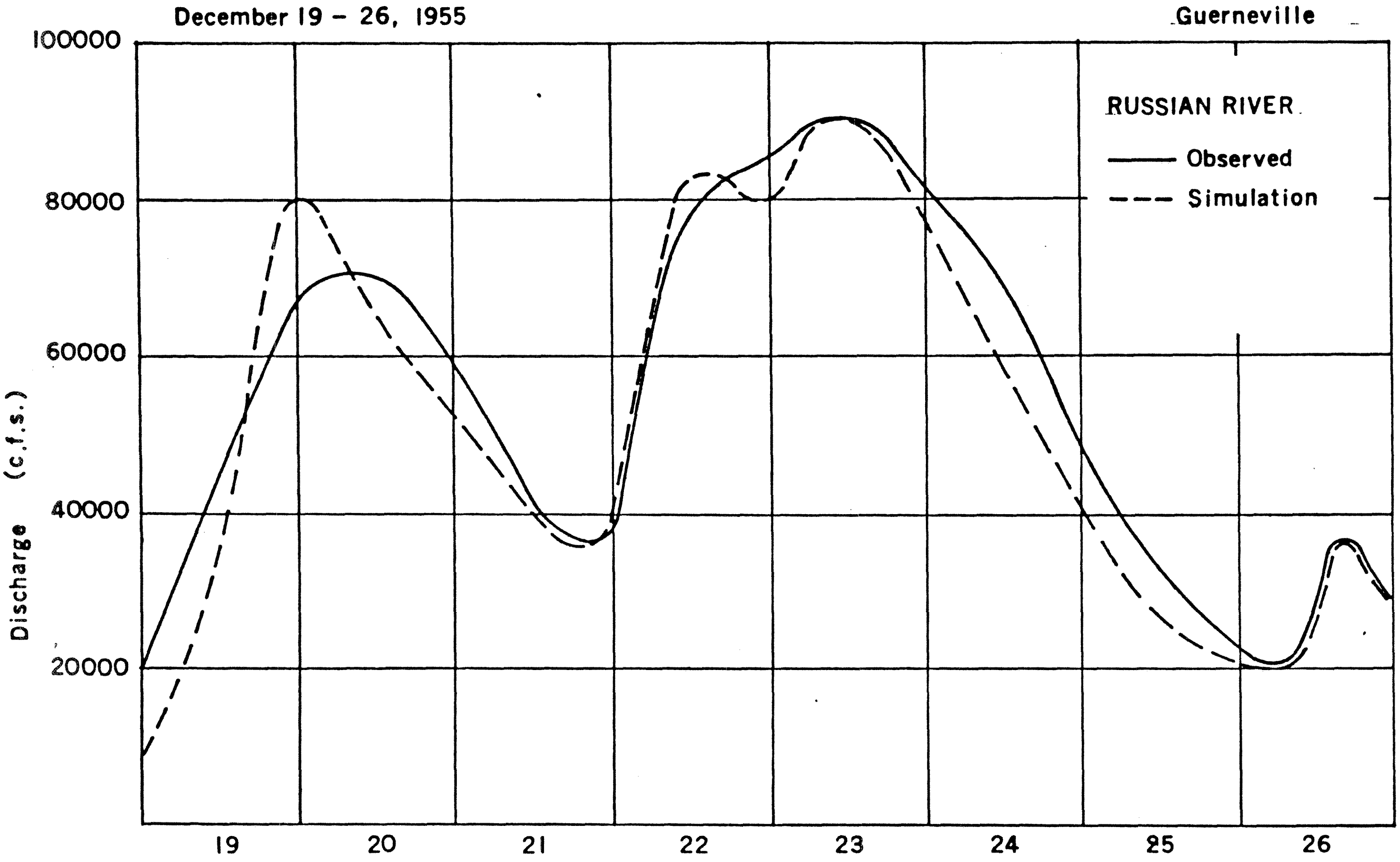

December 19 - 26, 1955
Guerneville
RUSSIAN RIVER
Observed
Simulation
Discharge (c.f.s.)
100000
80000
60000
40000
20000
19
20
21
22
23
24
25
26

Figure 6.10

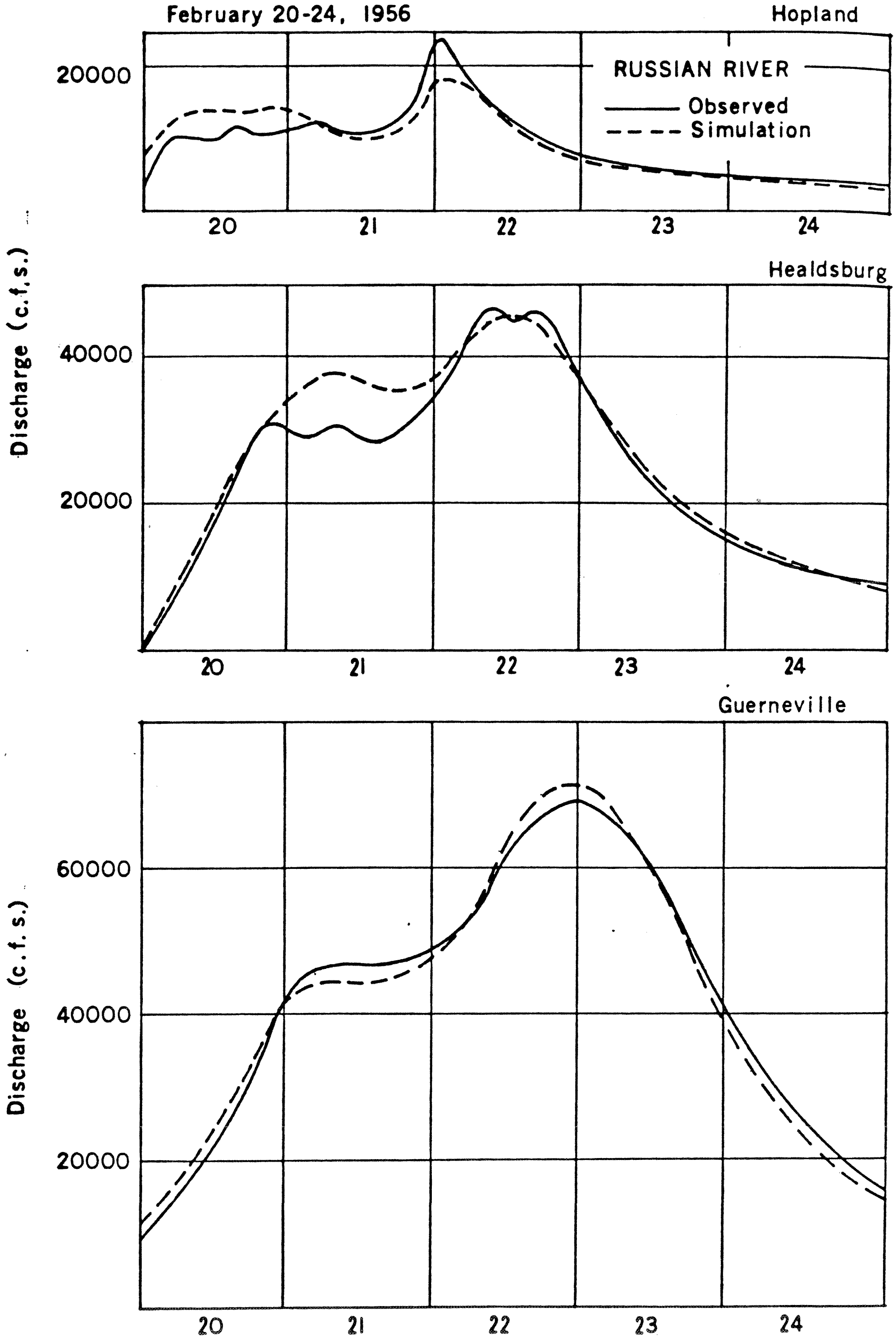

February 20-24, 1956
Hopland
RUSSIAN RIVER
Observed
Simulation
20000
20
21
22
23
24
Healdsburg
Discharge (c.f.s.)
40000
20000
20
21
22
23
24
Guerneville
Discharge (c.f.s.)
60000
40000
20000
20
21
22
23
24

Figure 6.11

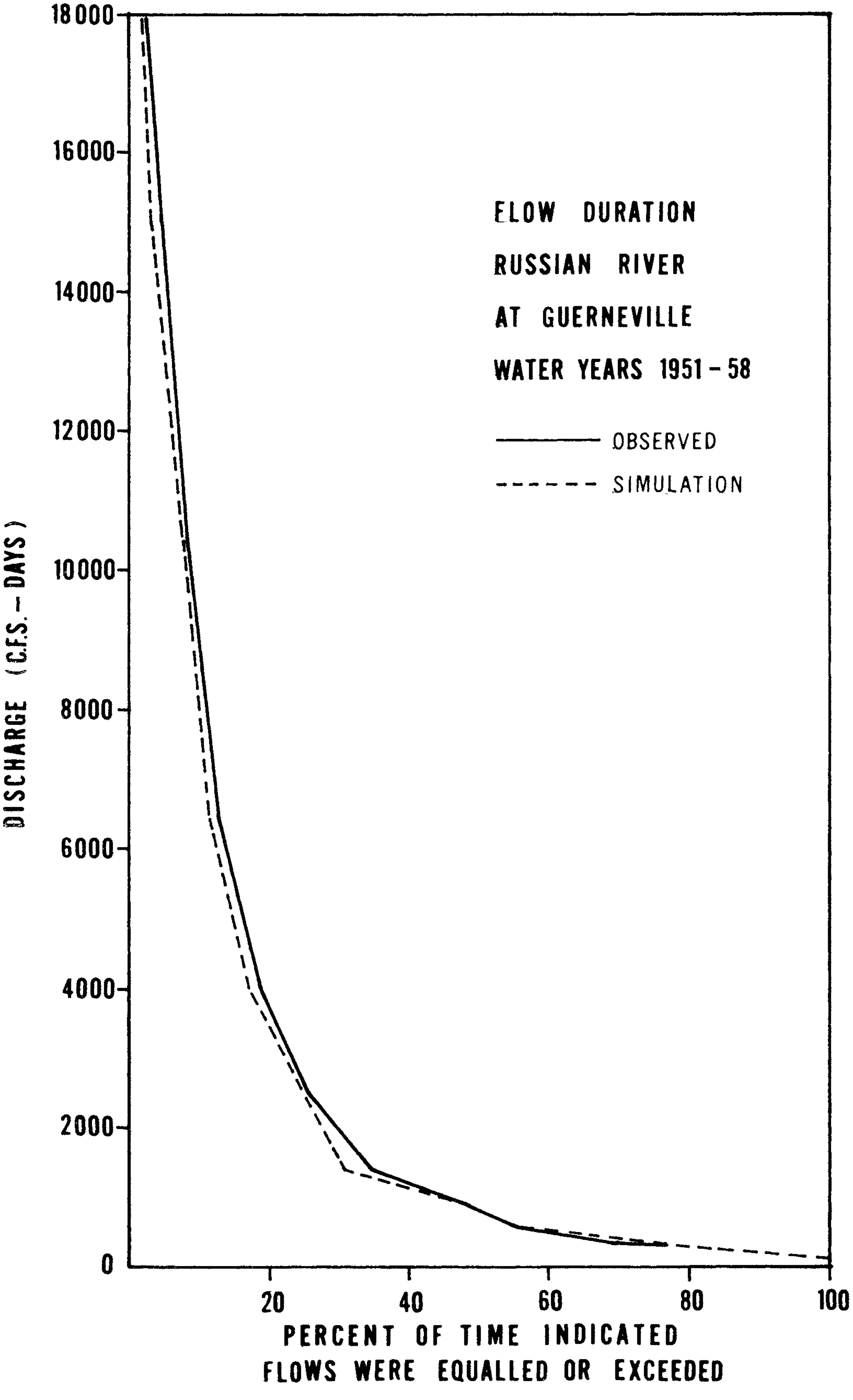

18000
16000
14000
12000
10000
8000
6000
4000
2000
0
DISCHARGE (C.F.S. - DAYS)
20
40
60
80
100
PERCENT OF TIME INDICATED
FLOWS WERE EQUALLED OR EXCEEDED
FLOW DURATION
RUSSIAN RIVER
AT GUERNEVILLE
WATER YEARS 1951 - 58
OBSERVED
SIMULATION

Figure 6.12

simulation period. Figures 6.10 and 6.11 are plots of hourly flows for several storms at each of the gages. Finally, Figure 6.12 is a flow duration plot of mean daily flows for the Russian River at Guerneville.

The simulation output on this river is good at all of the gaging stations. Data from the Hopland gage for 1956 was used in the discussion of accuracy in the first section of this chapter. There is very little variation in the quality of output on this river, and almost all of the daily flow duration and error tables (Table 6.1) closely approach the limiting condition of stream gage accuracy (Table 6.2). Peak flow reproduction is equally good; nine storms in ten give maximum flows within fifteen percent of the recorded value. The simulation clearly benefits from the number of rain gages used, since errors in individual gages tend to cancel. Precipitation regime is another favorable factor; typical storms are large extratropical cyclones, and thermal storms are rare. The summer low flows are strongly affected by the diversions.

French Broad River The French Broad River is a tributary to the Tennessee River above Knoxville, Tennessee. The upper watershed is in the Blue Ridge Mountains of North Carolina, an area of metamorphic and intrusive igneous rocks. Land slopes are moderate to steep and channel slopes are moderate. The area has a moderate continental forest climate, with mild winters. The natural vegetation is broadleaf deciduous forest. Most of the watershed above Blantrye is ungrazed woodland but there is some pasture and cropland in the valleys.

Streamflow was simulated at one flowpoint using the three segments as outlined below:

Flowpoint: Blantrye
Streamgage: French Broad River at Blantrye, North Carolina
Area: 296.0 square miles
Elevation Range: 2060 feet to 4800 feet
Average Annual Precipitation: 75 inches
Average Annual Runoff: 43 inches
Average Annual Potential Evapotranspiration: 32 inches
Segments:
1 - 99.0 square miles; recorder Quebec, storage gage Rosman
2 - 99.0 square miles; recorder Cedar Mountain, storage gage Pisgah Forest
3 - 98.0 square miles; recorder Glouchester Gap, storage gage Brevard

There are no major diversions or storage reservoirs in the watershed, but there is some diurnal fluctuation at low flows caused by a power plant.

Figure 6.13 is a map of the French Broad River showing gage locations. Table 6.6 gives monthly and annual recorded and simulated runoff for the Blantrye gage, and Figure 6.14 is an example of recorded and simulated mean daily flow. Figure 6.15 shows recorded and simulated peak flow frequency for the simulation period.

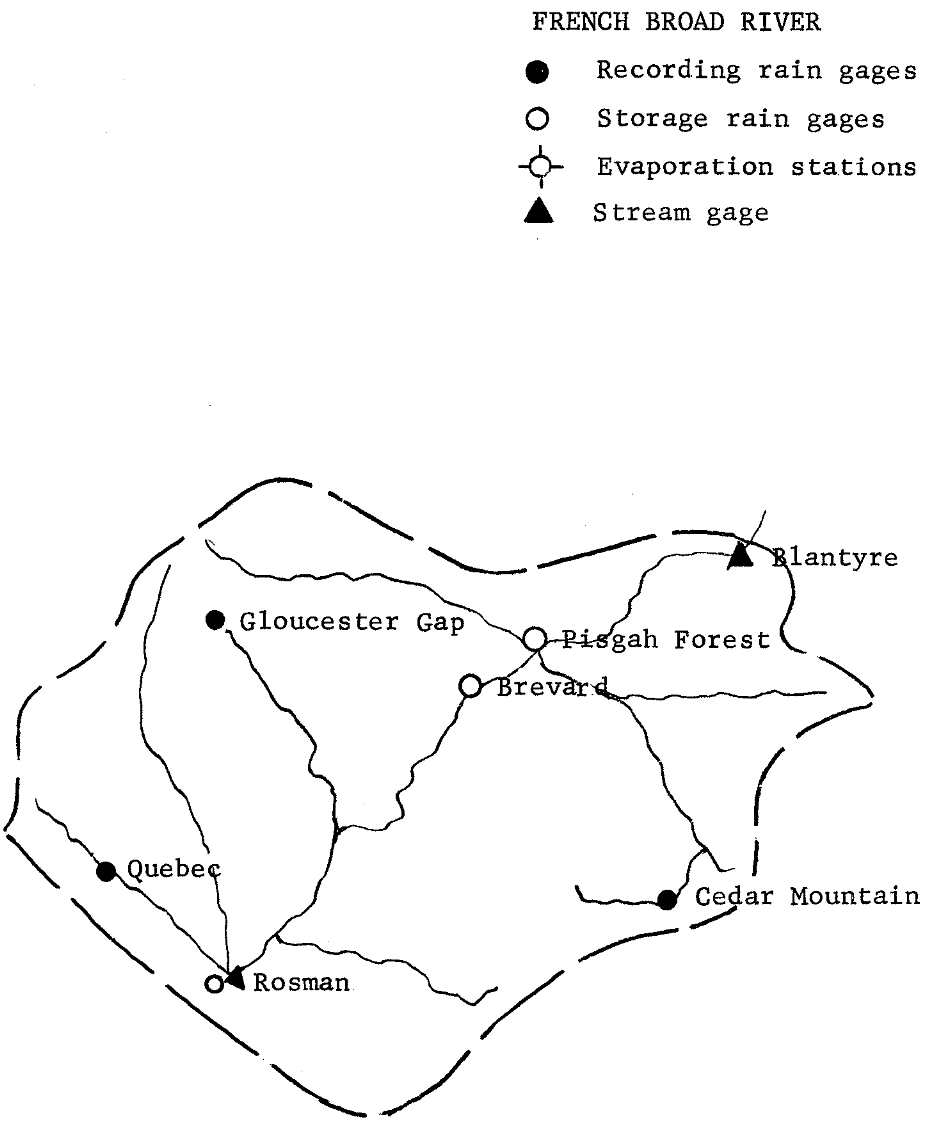

FRENCH BROAD RIVER
● Recording rain gages
○ Storage rain gages
⌖ Evaporation stations
▲ Stream gage
Blantyre
Gloucester Gap
Pisgah Forest
Brevard
Quebec
Cedar Mountain
Rosman

Figure 6.13

FLOWPOINT 1: BLANTYRE

Water Year		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Discharge)
49	Rec	1.95	8.20	6.48	6.16	5.73	4.78	5.87	6.07	5.29	8.62	7.12	5.45	71.7	0.9709
	Sim	2.32	9.57	6.57	6.30	5.91	5.05	4.93	5.16	3.70	9.02	7.76	5.18	71.5	
50	Rec	5.28	4.94	4.67	4.70	4.14	5.32	3.58	2.58	4.07	3.55	2.27	6.77	51.9	0.9787
	Sim	5.27	5.26	4.50	4.41	3.78	5.27	2.65	1.63	3.93	3.89	2.56	8.03	51.2	
51	Rec	3.51	2.41	4.93	2.89	3.25	4.53	4.73	3.22	2.72	2.12	1.81	1.30	37.4	0.9708
	Sim	4.37	2.69	5.45	2.58	3.37	3.98	4.38	2.79	2.30	1.93	1.71	1.30	36.9	
52	Rec	1.19	2.63	5.49	4.02	4.54	10.5	5.90	3.78	2.44	1.64	2.17	1.55	45.8	0.9440
	Sim	1.22	3.10	6.65	4.21	3.05	9.99	4.34	3.08	2.07	1.46	2.54	2.16	43.9	
53	Rec	1.11	1.86	2.35	4.59	6.29	6.48	3.51	4.15	2.89	2.30	1.58	1.69	38.8	0.9722
	Sim	1.60	1.75	2.73	5.72	7.15	5.98	3.49	4.28	2.78	2.56	1.96	1.98	42.0	
54	Rec	1.23	1.41	3.90	5.54	3.76	4.93	4.29	3.15	1.90	1.39	1.04	0.64	33.2	0.9812
	Sim	1.69	1.38	4.83	7.06	4.47	5.26	4.05	2.59	1.64	1.24	1.10	0.78	36.1	

Table 6.6

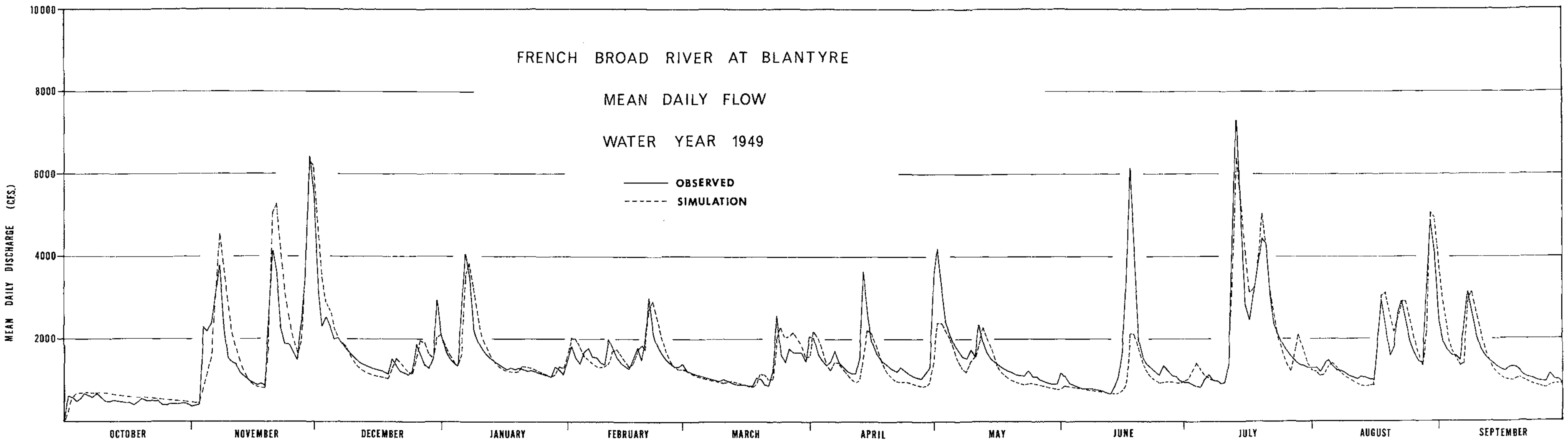

FRENCH BROAD RIVER AT BLANTYRE
MEAN DAILY FLOW
WATER YEAR 1949
OBSERVED
SIMULATION
MEAN DAILY DISCHARGE (CFS.)
10000
8000
6000
4000
2000
OCTOBER
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER

Figure 6.14

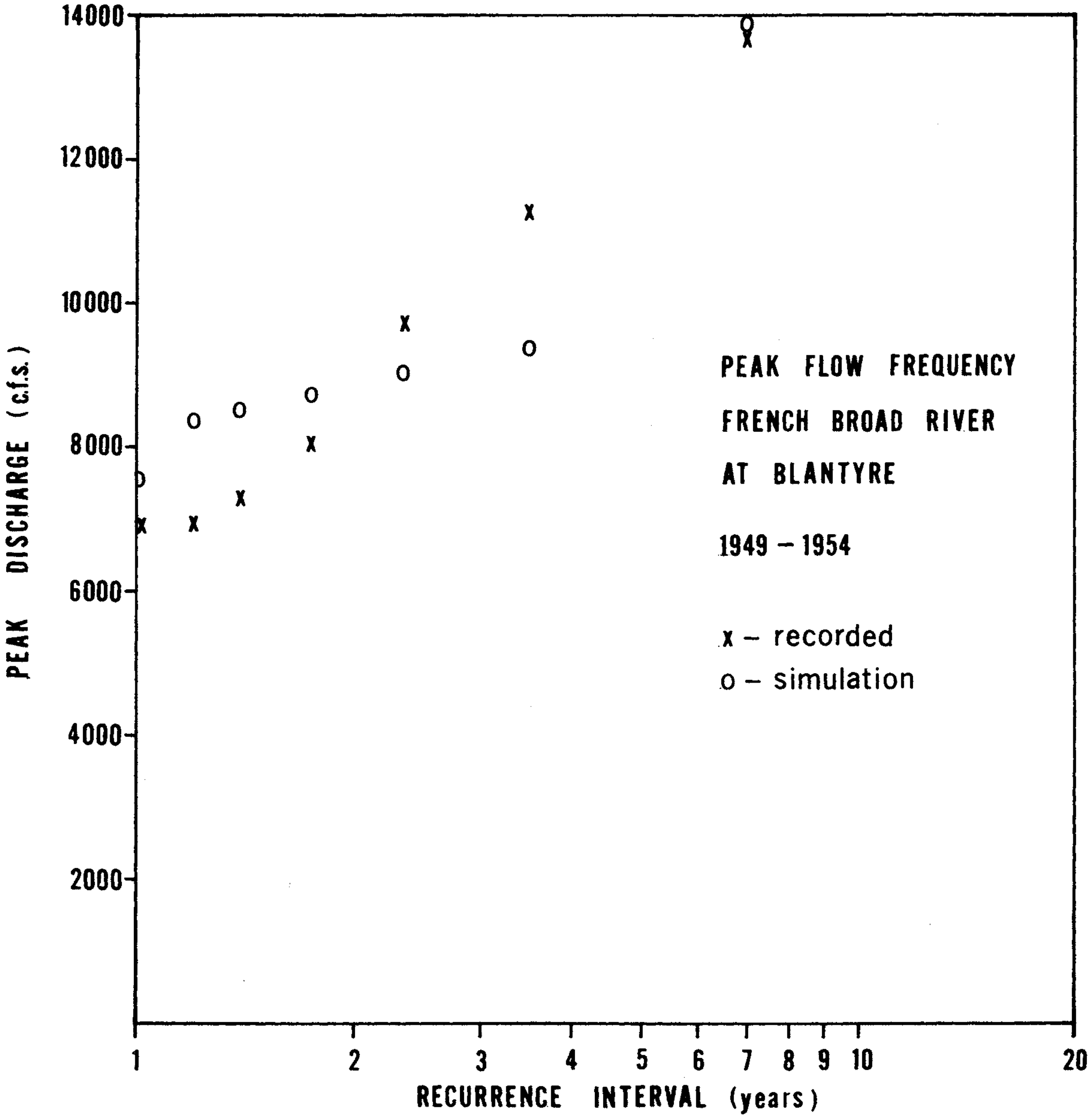

PEAK FLOW FREQUENCY
FRENCH BROAD RIVER
AT BLANTYRE
1949 – 1954
x – recorded
o – simulation
PEAK DISCHARGE (c.f.s.)
14000
12000
10000
8000
6000
4000
2000
1
2
3
4
5
6
7
8
9
10
20
RECURRENCE INTERVAL (years)

Figure 6.15

The output for this watershed is fairly good, but it is not equivalent to output on the West Coast for the same rain gage density. Precipitation patterns are more varied in the French Broad River, and thermal storms occur. Annual rainfall on the watershed segments differs by as much as twelve inches in some water years. Snow occurs occasionally in winter storms, but the snowmelt subroutine was not used in these runs.

The watershed has very limited surface runoff. For example, in segment 1 for 1952-53, only 1.5 inches of the 50.2 inches of runoff was surface runoff, and the maximum runoff rate from the land surface was 0.08 inches from a rainfall intensity of 1.23 inches per hour. In a more typical regime surface runoff would account for twenty or thirty percent of the total runoff, since rainfall intensities and annual rainfall amounts are both substantial. If the natural regime of high infiltration was altered, this watershed would react quite strongly.

South Yuba River* This watershed is on the western slope of the Sierra Nevada at Donner Pass near Lake Tahoe. The geological structure is complex, and consists of metamorphic and intrusive igneous rocks. Land slopes and channel slopes are steep. Cool dry summers, and heavy winter precipitation with eight to twelve feet of snow in the snowpack, are typical in this area. Needle leaf evergreen trees are the natural vegetation, and about fifty percent of the watershed has forest cover. Land use is limited to grazing in the lower elevation valleys. Streamflow was simulated at Lake Van Norden and at the gage at Cisco. A summary of watershed characteristics at the flowpoints follows:

Flowpoint 1: Lake Van Norden
Streamgage: none
Area: 10.81
Elevation Range: 6850 feet to 9100 feet
Average Annual Precipitation: 76 inches
Average Annual Runoff: 46 inches
Average Annual Potential Evapotranspiration: 30 inches
Segments: 1 - 8.07 square miles; recorder Soda Springs 1E, storage Big Bend R.S.(1/10), elevation 6850 to 7500 feet
2 - 2.35 square miles; recorder Soda Springs 1E, storage Big Bend R.S. (1/10), elevation 7500 to 8000 feet
3 - 0.39 square miles; recorder Soda Springs 1E, storage Big Bend R.S. (1/10), elevation 8000 to 9100 feet.

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*Data for this river was supplied through the courtesy of the Pacific Gas and Electric Company.

Flowpoint 2: Cisco

Streamgage: South Yuba River near Cisco

Area: 55.29 square miles

Elevation Range: 5520 feet to 9100 feet

Average Annual Precipitation: 76 inches

Average Annual Runoff: 46 inches

Average Annual Potential Evapotranspiration: 30 inches

Segments:

1 - 3.60 square miles; recorder Soda Springs 1E, storage Big Bend R.S. (7/8), elevation 5520 to 6000 feet

2 - 13.50 square miles; recorder Soda Springs 1E, storage Big Bend R.S. (7/8), elevation 6000 to 6500 feet.

3 - 15.90 square miles, recorder Soda Springs 1E, storage Big Bend R.S. (1/2), elevation 6500 to 7000 feet

4 - 9.94 square miles, recorder Soda Springs 1E, storage Big Bend R.S. (1/10), elevation 7000 to 7500 feet

5 - 4.44 square miles, recorder Soda Springs 1E, storage Big Bend R.S. (1/10), elevation 7500 to 8000 feet

6 - 0.70 square miles, recorder Soda Springs 1E, storage Big Bend R.S. (1/10, elevation 8000 to 9100 feet.

Lake Van Norden has a capacity of 5300 acre feet, which is less than four percent of the mean annual runoff. Summer low flows are altered by release from storage. A special purpose subroutine was added to simulate the operation of the reservoir. Upper Castle Creek,[4] the site of the Central Sierra Snow Laboratory, is a tributary to the South Yuba above Lake Van Norden.

Figure 6.16 is a map of the South Yuba River showing gage locations, and Table 6.7 gives the monthly and annual recorded and simulated runoff

SOUTH YUBA RIVER

● Recording rain gage and temperature station

○ Storage rain gage

△ Stream gage

⌖ Evaporation station

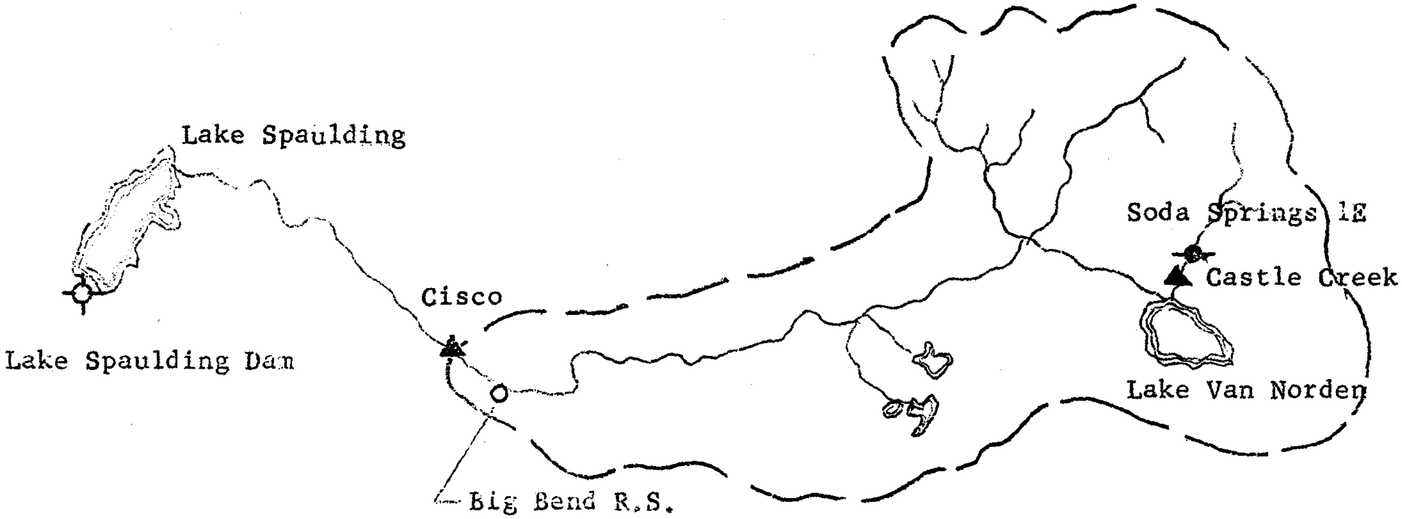


Figure 6.16

FLOWPOINT 2: CISCO

Water Year		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Discharge)
59	Rec	0.27	0.10	0.10	2.18	1.88	3.39	8.40	7.53	2.22	0.68	0.28	0.48	27.5	0.9065
	Sim	0.02	0.21	0.08	0.83	0.23	3.48	10.2	7.30	1.77	0.58	0.46	1.18	26.4	
60	Rec	0.36	0.29	0.09	0.15	3.00	6.95	10.0	10.3	3.55	1.05	0.44	0.32	36.5	0.9508
	Sim	0.50	0.15	0.08	0.08	1.64	6.20	8.73	11.8	4.17	0.67	0.47	0.45	35.0	
61	Rec	0.30	0.37	0.42	0.40	1.92	1.96	5.94	9.67	3.35	0.89	0.28	0.25	25.8	0.9460
	Sim	0.53	0.26	0.24	0.50	1.03	0.98	6.76	10.2	4.27	0.69	0.57	0.52	26.8	
62	Rec	0.31	0.58	0.54	0.56	1.59	1.38	12.9	14.1	8.73	0.73	0.19	0.96	42.6	0.9699
	Sim	0.60	0.72	0.66	1.15	2.02	0.48	11.0	13.6	7.76	0.86	0.53	0.43	39.7	
63	Rec	8.67	1.56	4.90	7.23	9.41	1.82	4.69	20.0	6.37	1.42	0.62	0.65	67.4	0.9504
	Sim	14.1	1.65	4.65	6.81	11.3	2.63	2.66	17.3	5.75	0.79	0.49	0.51	68.8	

Table 6.7

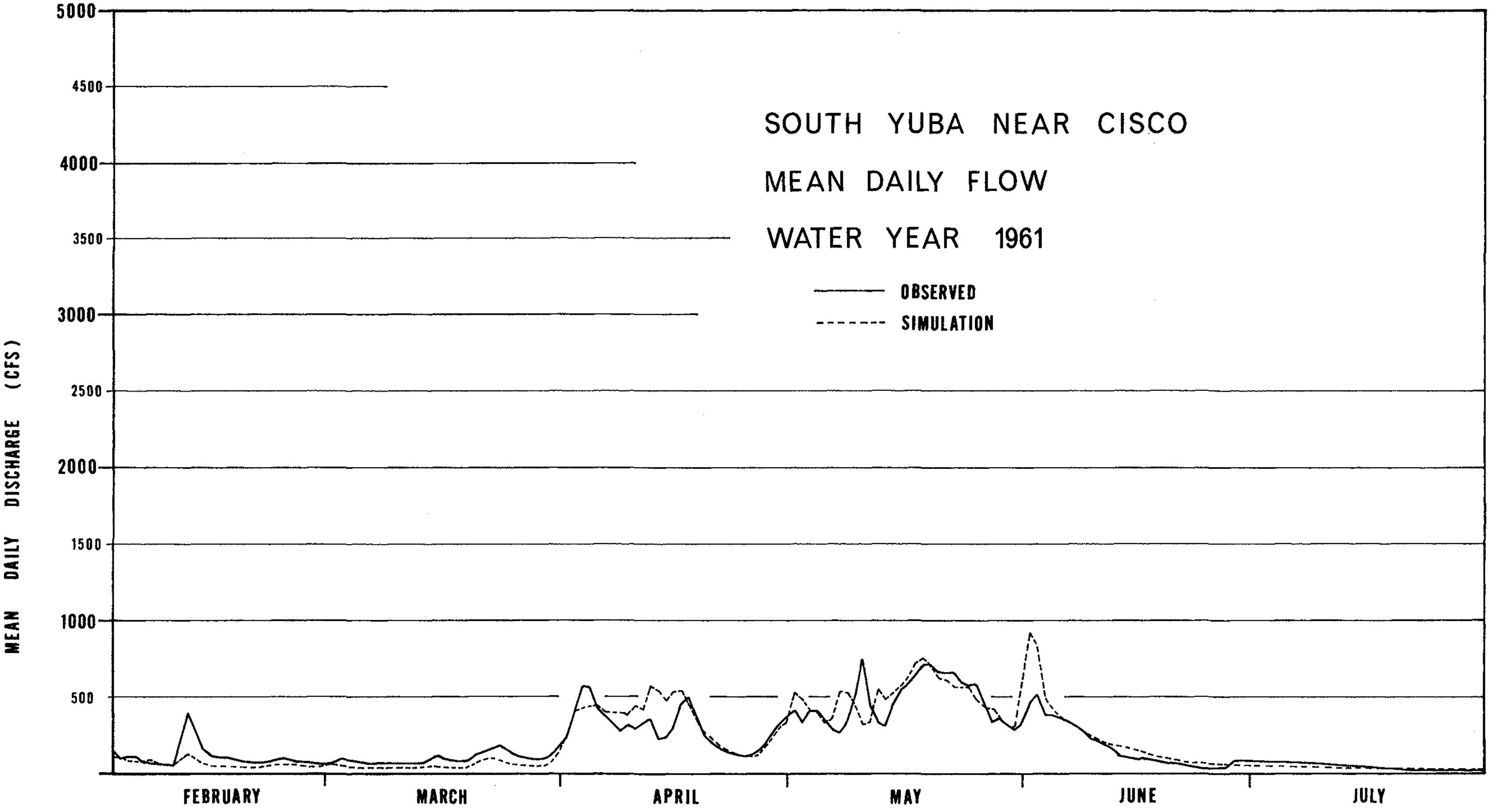

SOUTH YUBA NEAR CISCO
MEAN DAILY FLOW
WATER YEAR 1961
OBSERVED
SIMULATION
MEAN DAILY DISCHARGE (CFS)
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY

Figure 6.17

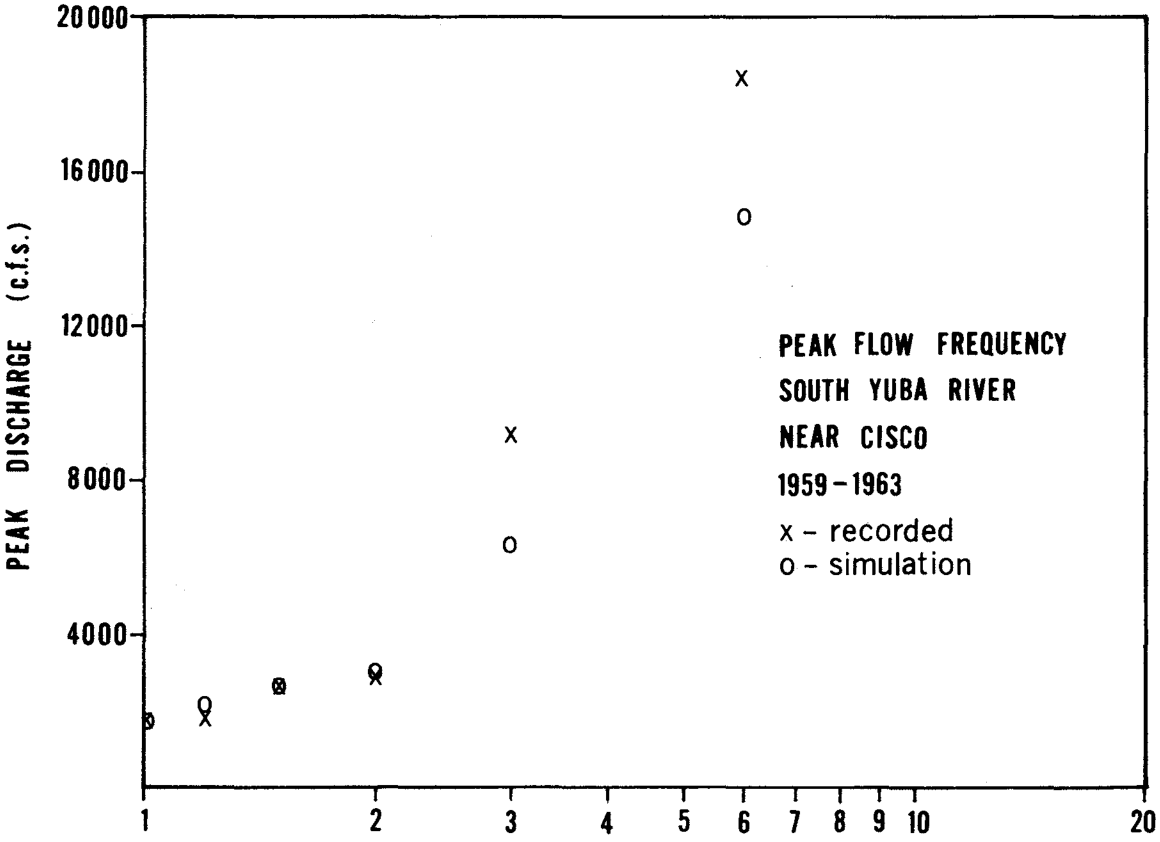

20000
16000
12000
8000
4000
PEAK DISCHARGE (c.f.s.)
PEAK FLOW FREQUENCY
SOUTH YUBA RIVER
NEAR CISCO
1959-1963
x - recorded
o - simulation
1
2
3
4
5
6
7
8
9
10
20
RECURRENCE INTERVAL (years)

Figure 6.18

May 27-30, 1963

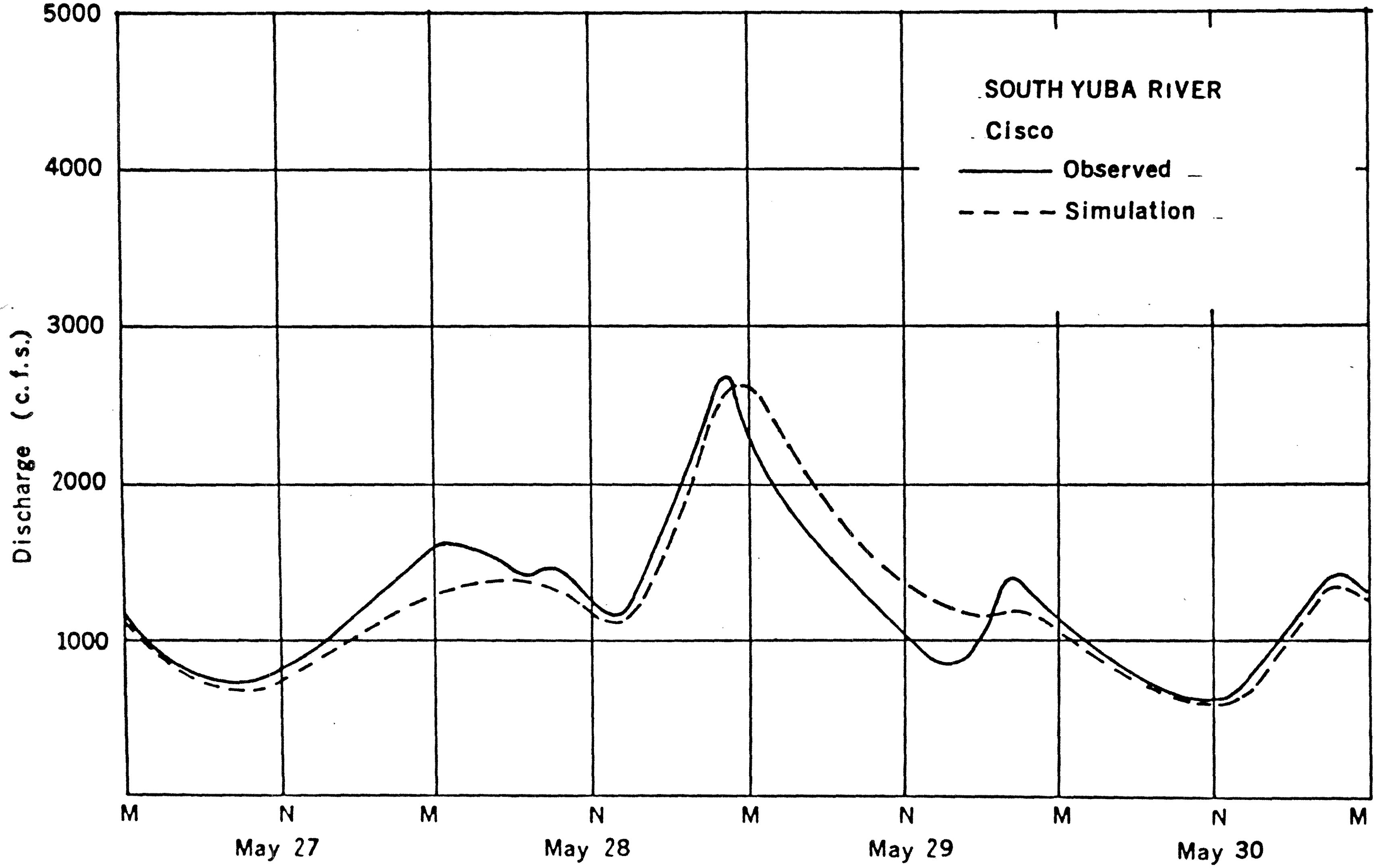


Figure 6.19

in the simulation period. Figure 6.17 is a plot of recorded and simulated mean daily flows, and Figure 6.18 shows flow frequency for the simulation period. Hourly flows are plotted in Figure 6.19.

Overall results on this stream are satisfactory for the basic data used. There are three factors that contribute to variations in the output. Radiation data was not available, and snowmelt calculations were based on an imperfect relationship between diurnal temperature change and expected short-wave radiation. The results should improve if radiation data were used as input. Precipitation input was limited, and in some storms (i.e. Oct. 11-14, 1962) there were large variations in the total amount of rainfall at Soda Springs 1E and Big Bend R.S. Another factor that influenced simulation quite strongly on this watershed was the rain/snow selection discussed in Chapter IV. Heavy rainfall at lower elevations with snow above 6500 to 7000 feet occurs frequently.

Streamflow was simulated for the Upper Castle Creek tributary so that parameters for segments above 6500 feet could be more closely established.

Napa River The Napa River enters San Pablo Bay about 40 miles north of San Francisco. The watershed above St. Helena is similar to the Russian River and consists of steeply folded and faulted sedimentary rocks, with steep land slopes and moderate to steep main channel slopes. The climate is mild with summer drought and winter rain, and the native vegetation is mixed broad deciduous and needleleaf evergreen trees. The mild year-round temperatures are suitable for grapes, and irrigated vineyards with some ranching, are the major land uses. Watershed characteristics are summarized below.

Flowpoint 1: St. Helena
Streamgage: Napa River near St. Helena
Area: 81.4 square miles
Elevation Range: 200 feet to 4344 feet
Average Annual Precipitation: 36 inches
Average Annual Runoff: 14 inches
Average Annual Potential Evapotranspiration: 42 inches
Segments: 1 - 81.4 square miles; recorder St. Helena,
storage gage Calistoga

There are no major diversions or reservoirs in the watershed, but some small irrigation diversions, that were not included in the simulation, do alter summer flows.

Figure 6.20 is a map of the watershed showing gage locations. Table 6.8 gives monthly and annual recorded and simulated runoff, and Figure 6.21 is a plot of mean daily recorded and simulated flows. Figure 6.22 is a plot of flow frequency for the simulation period and Figure 6.23 shows the output for some detailed hydrographs.

The output for the Napa River is generally good even though only one recording rain gage was used. As in the Russian River, the climate allows reasonable results when a sparse rain gage network is used to represent continuous rainfall over large areas.

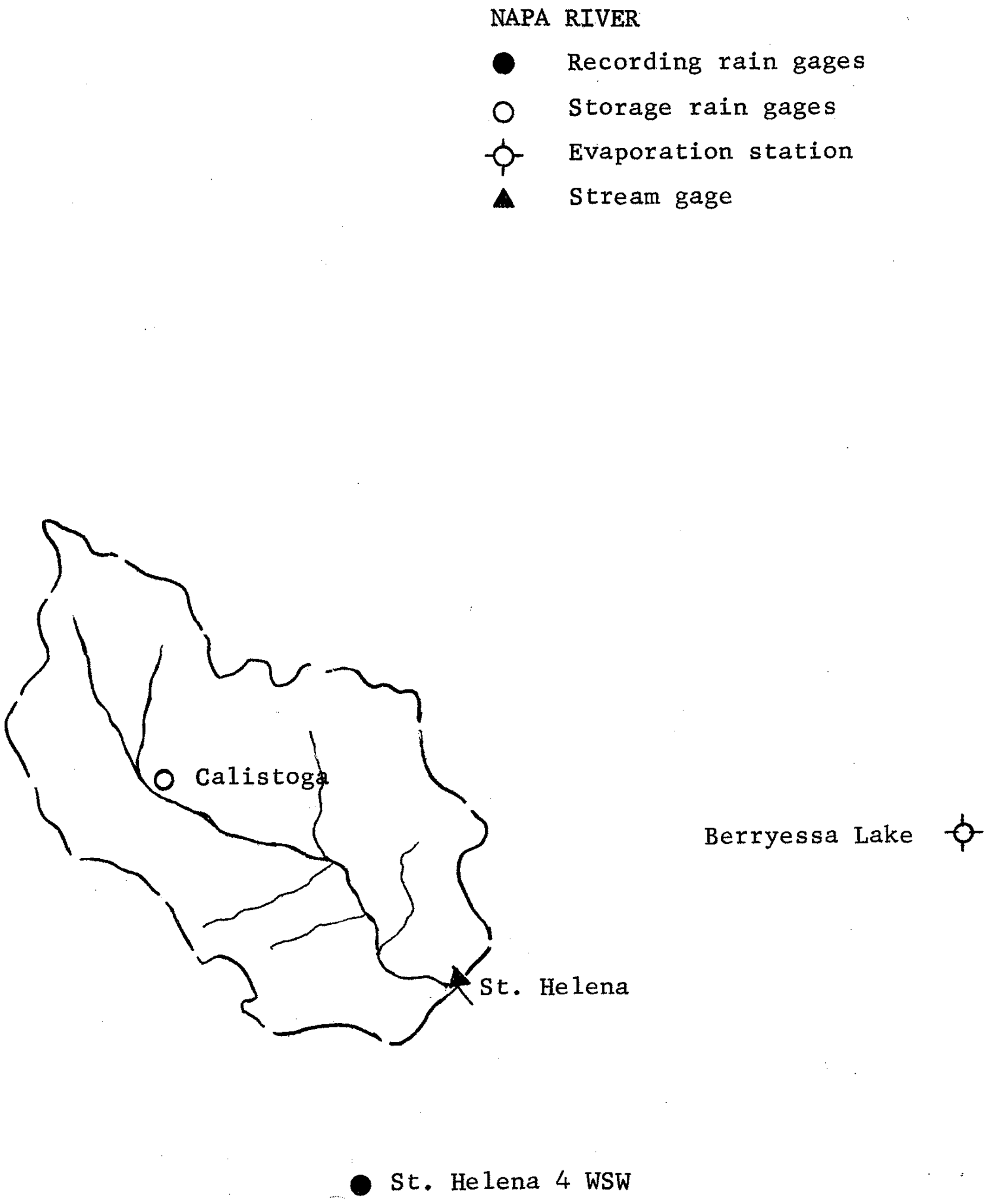

NAPA RIVER
● Recording rain gages
○ Storage rain gages
⌖ Evaporation station
▲ Stream gage
Calistoga
Berryessa Lake
St. Helena
St. Helena 4 WSW

Figure 6.20

FLOWPOINT 1: ST. HELENA

Water Year		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Discharge)
58	Rec	0.32	0.20	1.17	4.16	14.1	5.36	6.73	0.42	0.15	0.06	0.03	0.01	32.8	0.9859
	Sim	0.56	0.23	0.74	3.50	12.3	5.76	7.03	0.42	0.13	0.05	0.02	0.01	30.8	
59	Rec	0.02	0.02	0.04	1.09	3.51	0.56	0.21	0.08	0.01	0.00	0.00	0.02	5.6	0.9909
	Sim	0.00	0.01	0.06	1.10	3.70	0.59	0.24	0.10	0.04	0.02	0.01	0.17	6.0	
60	Rec	0.01	0.01	0.03	0.40	6.05	2.64	0.52	0.19	0.04	0.01	0.00	0.01	9.9	0.9928
	Sim	0.02	0.01	0.07	0.28	6.52	2.09	0.48	0.23	0.09	0.04	0.02	0.01	9.9	
61	Rec	0.01	0.04	0.41	1.01	2.34	1.65	0.57	0.18	0.05	0.01	0.01	0.01	6.3	0.9733
	Sim	0.02	0.16	0.53	0.88	2.12	1.52	0.54	0.22	0.08	0.04	0.02	0.02	6.1	
62	Rec	0.03	0.06	0.49	0.44	6.51	3.25	0.44	0.15	0.04	0.02	0.01	0.00	11.4	0.9530
	Sim	0.01	0.11	0.34	0.31	6.79	3.95	0.46	0.17	0.06	0.03	0.01	0.00	12.2	

Table 6.8

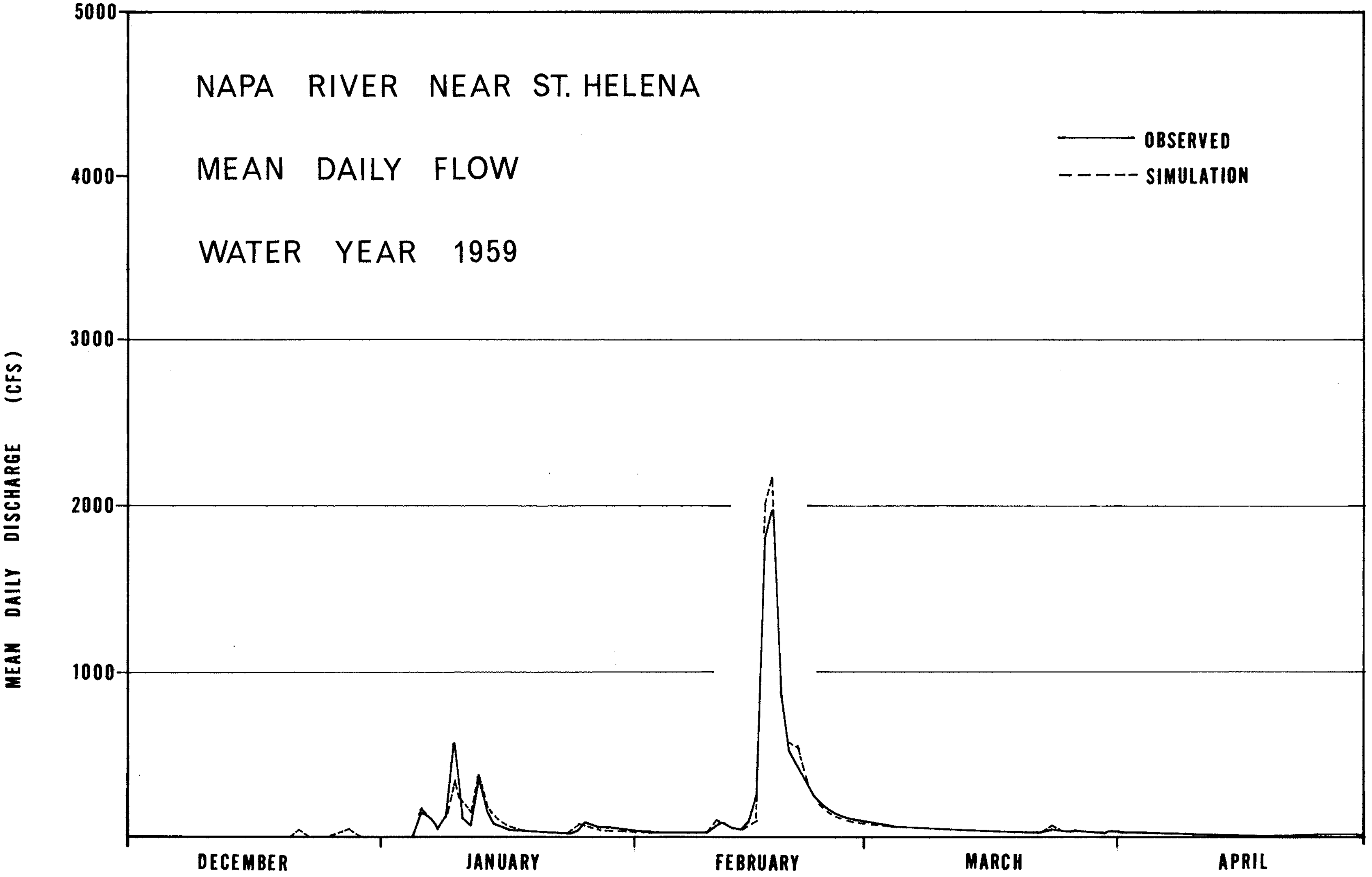

NAPA RIVER NEAR ST. HELENA
MEAN DAILY FLOW
WATER YEAR 1959
OBSERVED
SIMULATION
5000
4000
3000
2000
1000
MEAN DAILY DISCHARGE (CFS)
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL

Figure 6.21

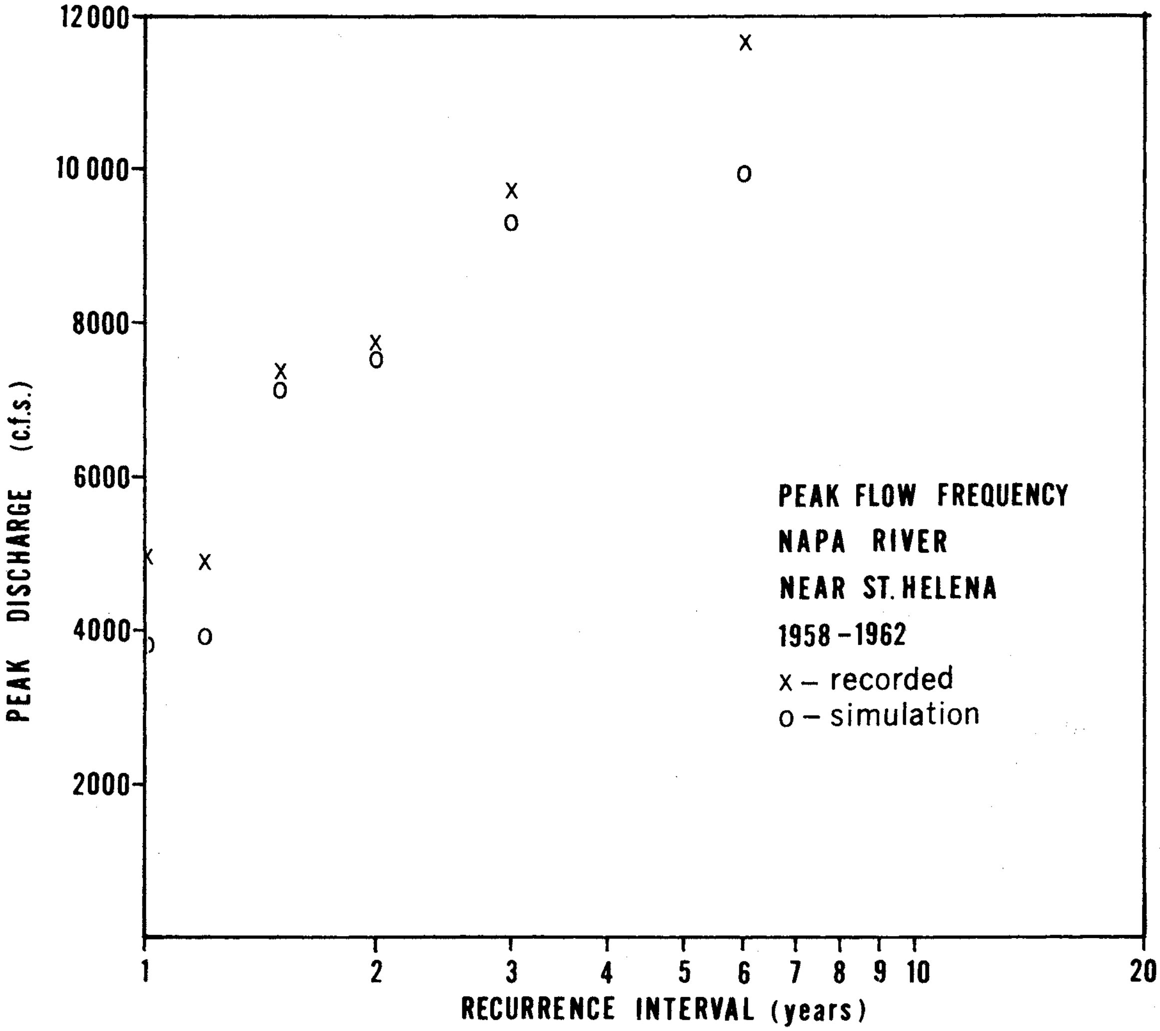
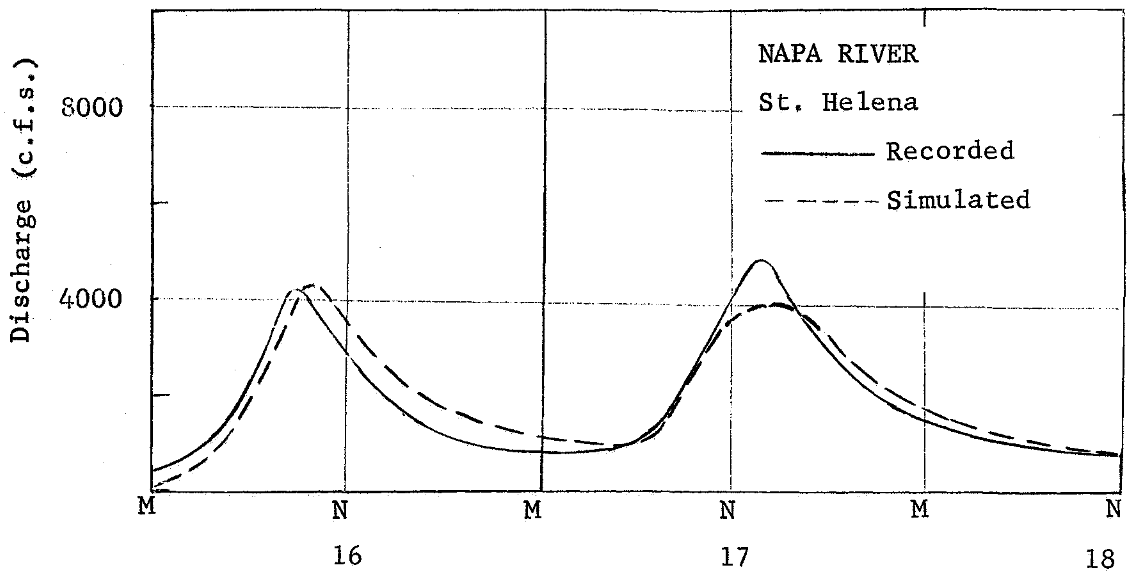

12000
10 000
8000
6000
4000
2000
PEAK DISCHARGE (c.f.s.)
1
2
3
4
5
6
7
8
9
10
20
RECURRENCE INTERVAL (years)
PEAK FLOW FREQUENCY
NAPA RIVER
NEAR ST. HELENA
1958 - 1962
x – recorded
o – simulation

Figure 6.22

February 16-19, 1960



February 7-10, 1960

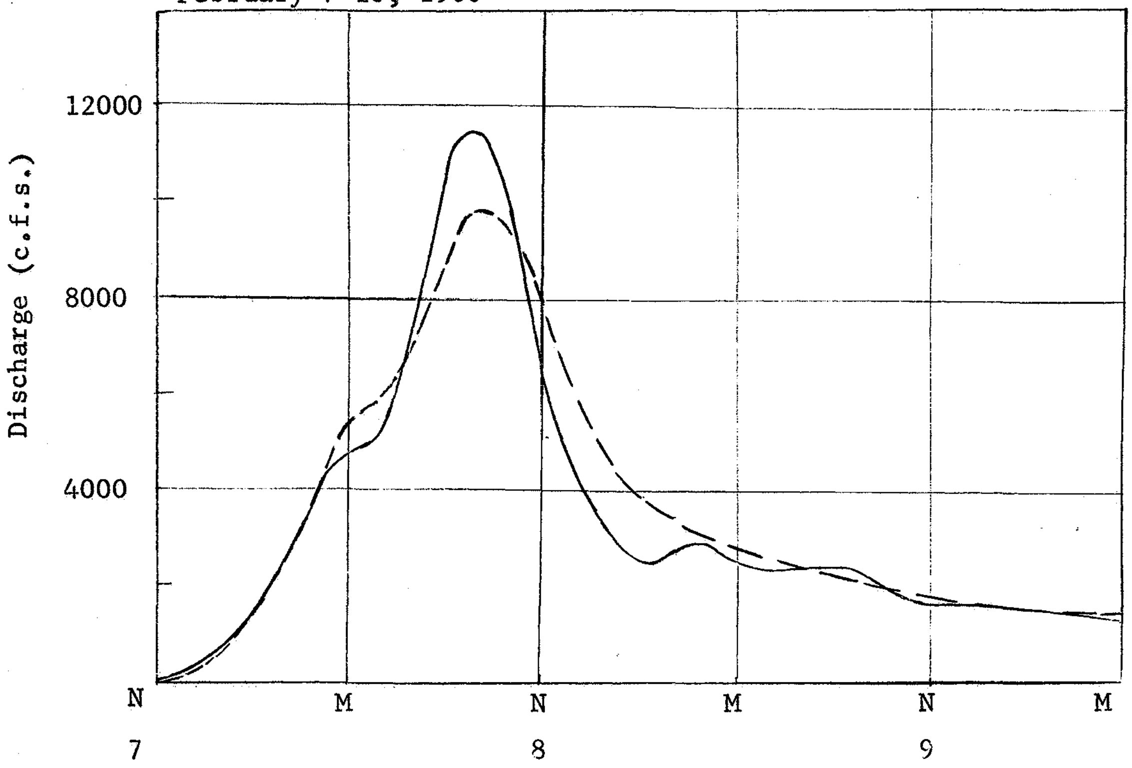


Figure 6.23

Beargrass Creek Beargrass Creek is a small tributary to the Ohio River at Louisville, Kentucky. The watershed is on the Bluegrass Plain, an area of slightly folded sedimentary rocks. Land and channel slopes are moderate to mild. The area has the humid continental climate with year-round precipitation, that is common to north-eastern United States. The natural vegetation is grass, with some broadleaf deciduous trees. In the simulation period, cropland and pasture land were the major land uses.

A summary of watershed characteristics follows:

Flowpoint 1: Cannons Lane
Streamgage: Middle Fork Beargrass Creek at Cannons Lane, Louisville, Kentucky
Area: 18.5 square miles
Elevation Range: 478 feet to 700 feet
Average Annual Precipitation: 43 inches
Average Annual Runoff: 14 inches
Average Annual Potential Evapotranspiration: 30 inches
Segments: 1 - 18.5 square miles; recorder Bowman Field, storage gage and temperature data Anchorage.

This watershed is one of several selected for continuing detailed study of simulation of urbanization effects. The water years included here are prior to urban development, and therefore serve as an example of simulation for this climatic region.

Figure 6.24 is a map of Beargrass Creek showing gage locations, and Table 6.9 gives the monthly and annual recorded and simulated runoff for the simulation period. Figure 6.25 is a plot of recorded and simulated mean daily flows, and Figure 6.26 shows peak flow frequency. Detailed hydrographs are shown in Figure 6.27.

The output shows the effects of a varied rainfall regime. Precipitation on the recording and storage gages often differs considerably in summer storms. The snowmelt parameters were checked against data on snow depth since the more detailed snowpack data is not observed.

BEARGRASS CREEK

- ● Recording rain gage and temperature
- ○ Storage rain gage and temperature
- ▲ Stream gage

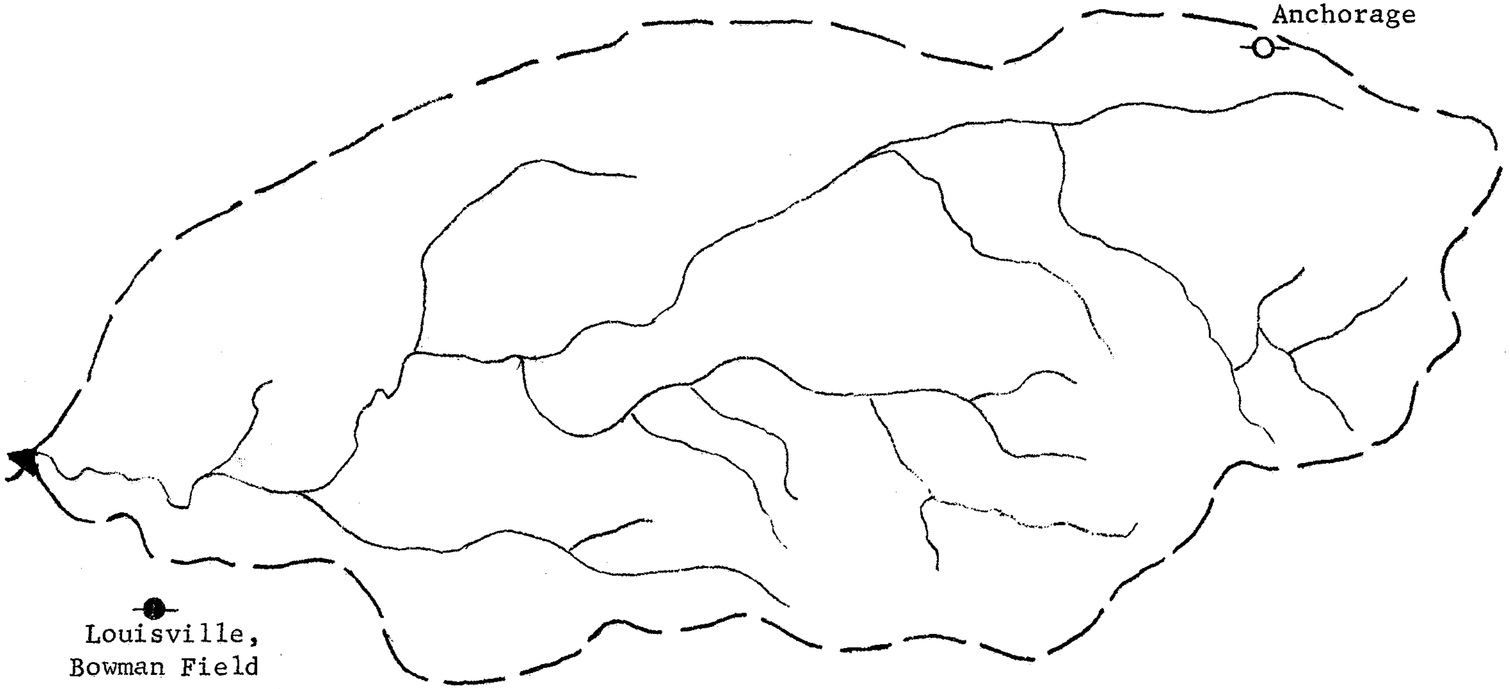


Figure 6.24

FLOWPOINT 1: CANNONS LANE

Water Year		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Discharge)
50	Rec	0.10	0.07	1.30	9.19	6.01	1.56	2.56	4.40	5.03	0.85	0.92	1.00	33.0	0.9559
	Sim	0.19	0.11	0.87	7.56	5.64	2.04	2.34	3.86	5.12	1.10	0.46	2.29	31.6	
51	Rec	0.24	2.12	3.08	5.56	4.09	4.33	2.03	0.59	0.11	0.14	0.08	0.09	22.5	0.9521
	Sim	0.56	2.98	2.85	5.68	3.38	4.07	1.47	0.42	0.20	0.19	0.22	0.32	22.4	
52	Rec	0.27	0.86	4.77	4.58	2.14	4.03	0.86	0.41	0.49	0.10	0.05	0.04	18.6	0.9372
	Sim	1.15	2.35	5.50	4.23	2.28	3.99	0.76	0.44	0.52	0.17	0.07	0.07	21.6	
53	Rec	0.05	0.10	0.53	1.88	0.52	2.84	1.62	2.17	0.29	0.30	0.04	0.00	10.4	0.9730
	Sim	0.09	0.09	0.83	1.77	0.74	2.28	1.15	1.66	0.37	0.41	0.16	0.05	9.6	

Table 6.9

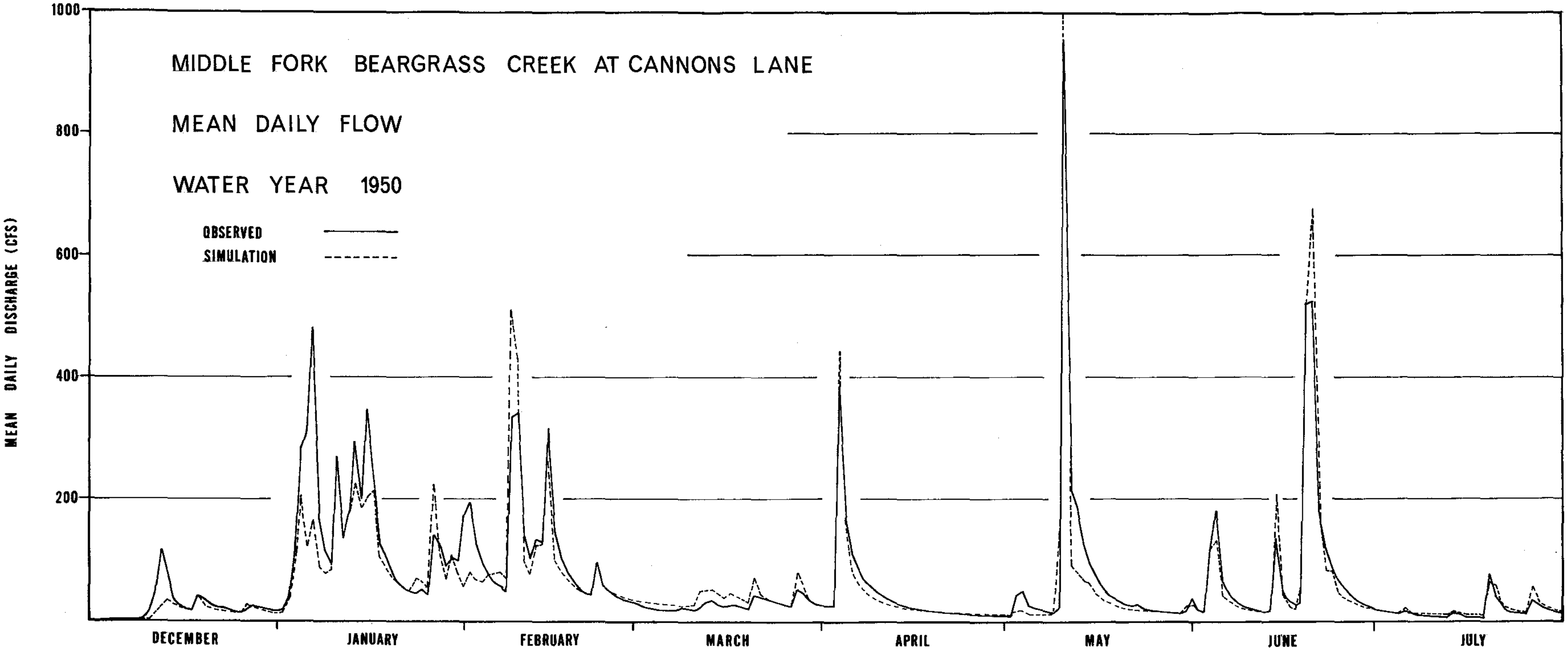

MIDDLE FORK BEARGRASS CREEK AT CANNONS LANE
MEAN DAILY FLOW
WATER YEAR 1950
OBSERVED
SIMULATION
MEAN DAILY DISCHARGE (CFS)
1000
800
600
400
200
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY

Figure 6.25

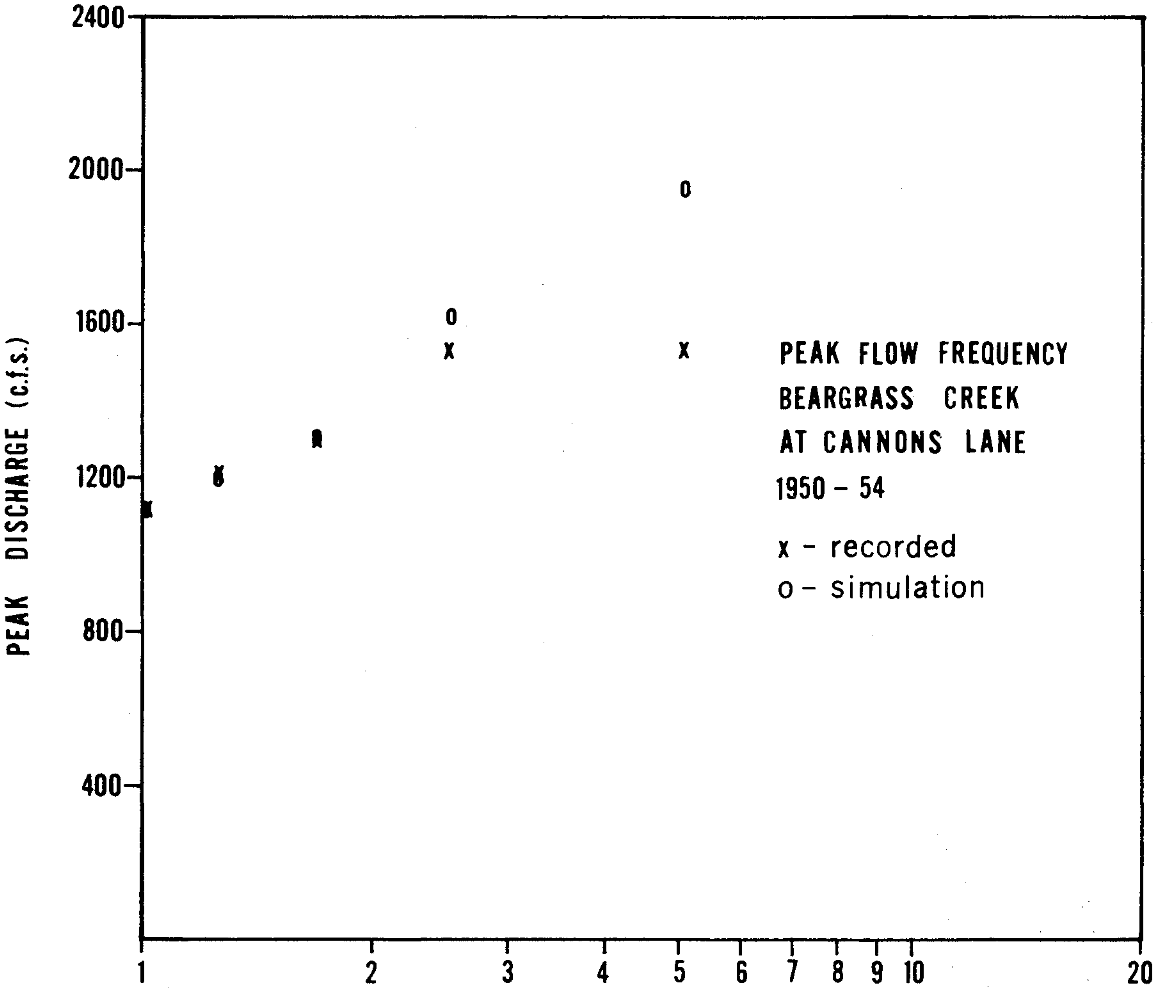
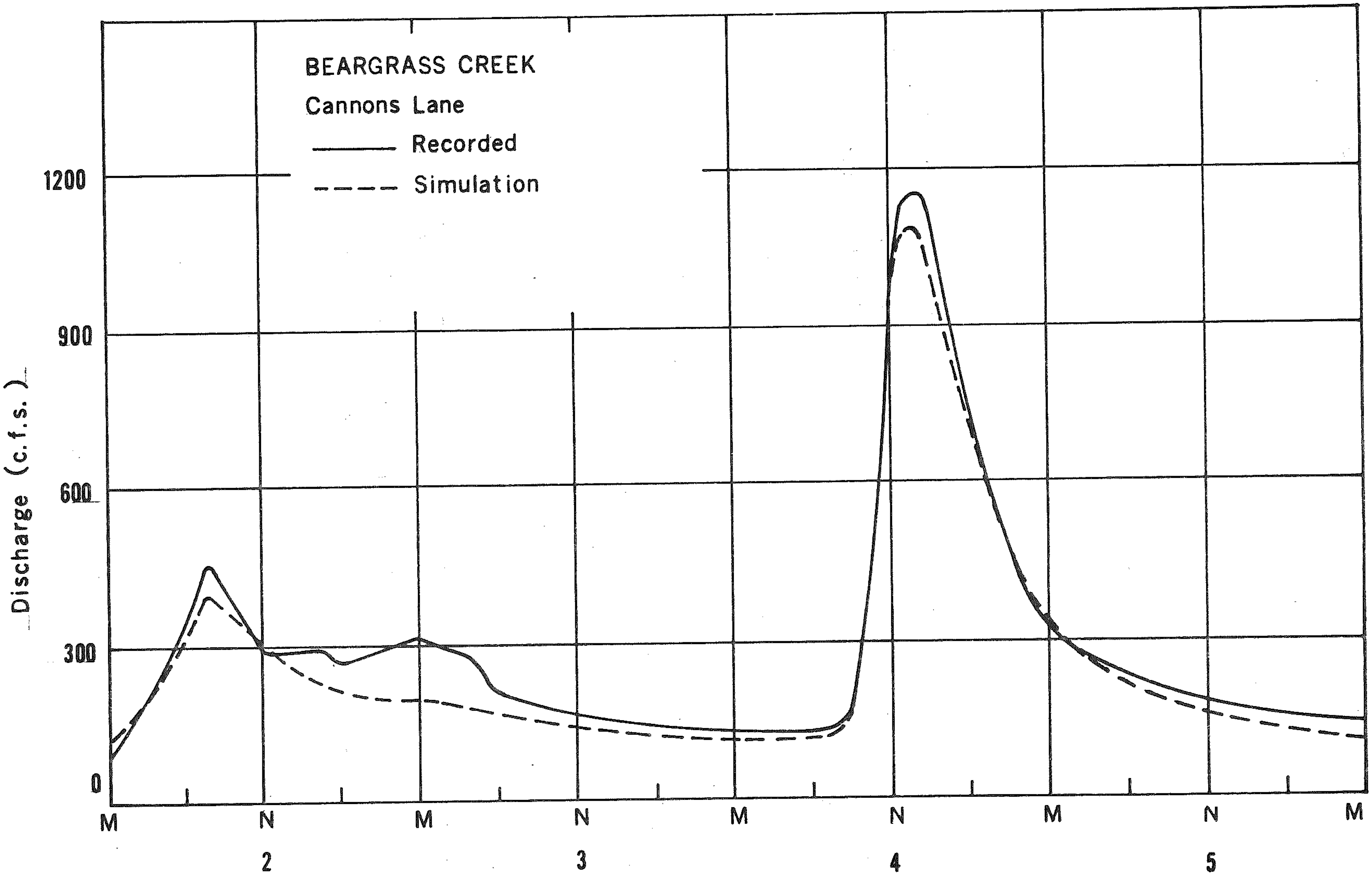

2400
2000
1600
1200
800
400
PEAK DISCHARGE (c.f.s.)
1
2
3
4
5
6
7
8
9
10
20
RECURRENCE INTERVAL (years)
PEAK FLOW FREQUENCY
BEARGRASS CREEK
AT CANNONS LANE
1950 - 54
x - recorded
o - simulation

Figure 6.26



January 2 - 5, 1952

Figure 6.27

Comments on Simulation Results

The five watersheds included in this chapter are in four of the six major climatic zones in the United States. The data used is routinely collected, and none of the watersheds would be classified as experimental. The accuracy of output for other watersheds in a given climatic zone should be similar to that for the watersheds included here, when data from the established hydrometeorological network is used. Experimental watersheds with high rain gage densities will give more accurate results.

No uniform criteria for required rain gage density, or for lengths of record for simulation, can be given since these depend on the overall hydrologic regime of individual watersheds. The most critical single factor in successful simulation is the raingage network. It is easy to demonstrate the extreme sensitivity of hydrographs to rainfall amounts, especially in arid regimes. More than one rain gage is a definite advantage, even in small watersheds.

It is felt that the goal of developing a general system for simulation based on basic processes has proved workable. Simulation allows convenient comparison and analysis of the basic processes that interact in the hydrologic regime. For example, two nearby watersheds may have different peak discharge characteristics. Is this due to their channel geometry, infiltration characteristics, or a small change in precipitation characteristics? It is very difficult to establish an answer from a single variable such as streamflow, and simulation of all watershed processes is necessary.

Several special situations have not yet been programmed into the model. No attempt is made to model runoff from permafrost areas or to include specific calculations for channel seepage. As more experience with the general model is acquired, additions will inevitably be made, but the representation of basic processes common to the great majority of watersheds will continue to be emphasized.

VII. APPLICATIONS OF SIMULATION

In this chapter some applications of digital simulation are discussed. The applications included are not exhaustive, and are intended only to illustrate some of the many possible uses of simulation in hydrology.

In the section titled Weather Modification, the analysis of watershed response for assumed weather modifications is described. The section titled Urbanization explores the unique problems of urban hydrology, and design methods for urban drainage. Finally, the section on Hydrometeorological Networks is a summary of applications of simulation to the classical problems of predicting regimes from limited data, and of evaluating requirements for data networks.

Weather Modification

The capricious behavior of weather has prompted many technical studies of the possibilities for weather modification. Most of these studies are directed toward techniques for modification, or toward establishing whether or not a modification has or has not been achieved. It is also of interest to discover, in advance, the hydrologic consequences of selected weather changes in different geographical areas, assuming that these changes were to take place.

The land phase of the hydrologic cycle was defined as runoff into streams from rainfall, and evaporation or transpiration of moisture into the atmosphere. This system has only two major long term constraints; the rates or volumes of rainfall and potential evapotranspiration. In weather modification there are many possible detailed modifications to hydrologic regimes that could be investigated, but some useful basic results can be found using uniform variations in precipitation and potential evapotranspiration.

If the energy exchange process governing evapotranspiration could be altered so that evapotranspiration rates were changed, how would rivers and streams react? How would the water yield from a river change if rainfall were increased? These questions were investigated in a recent paper on the hydrologic consequences of weather modification,[34] and a brief summary of typical response will be given here.

Figures 7.1 and 7.2 show some of the results that were obtained by using models to simulate watershed behavior as input elements were changed. Figure 7.1 shows a summary of the results for three watersheds when rainfall rates were uniformly increased by ten percent. The figure shows the increase in runoff, in percent of unmodified annual yield, for each water year of a five year trial period, due to the ten percent increase in rainfall. For example, in the first water year of the trial period on Beargrass Creek runoff increased by 19 percent.

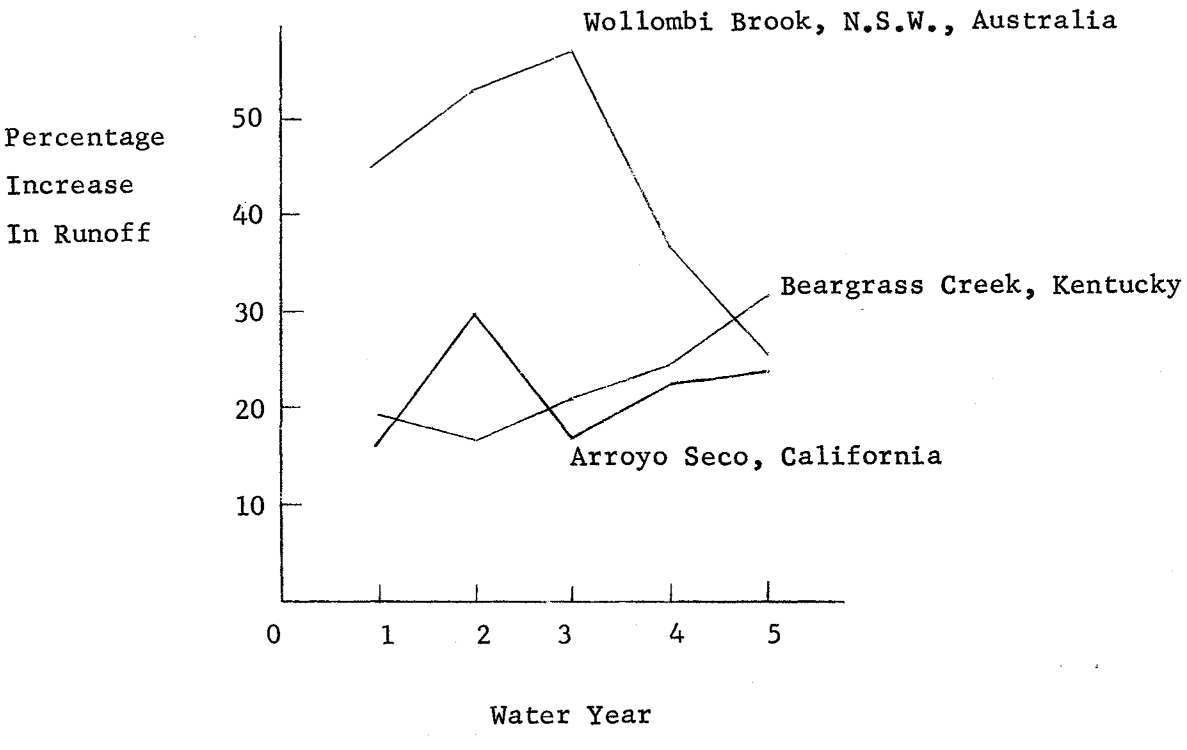


Figure 7.1

Figure 7.2 shows the results of runs in which potential evapotranspiration was increased uniformly by ten percent. For each watershed the percentage reduction in runoff each year due to the increase in potential evapotranspiration is shown.

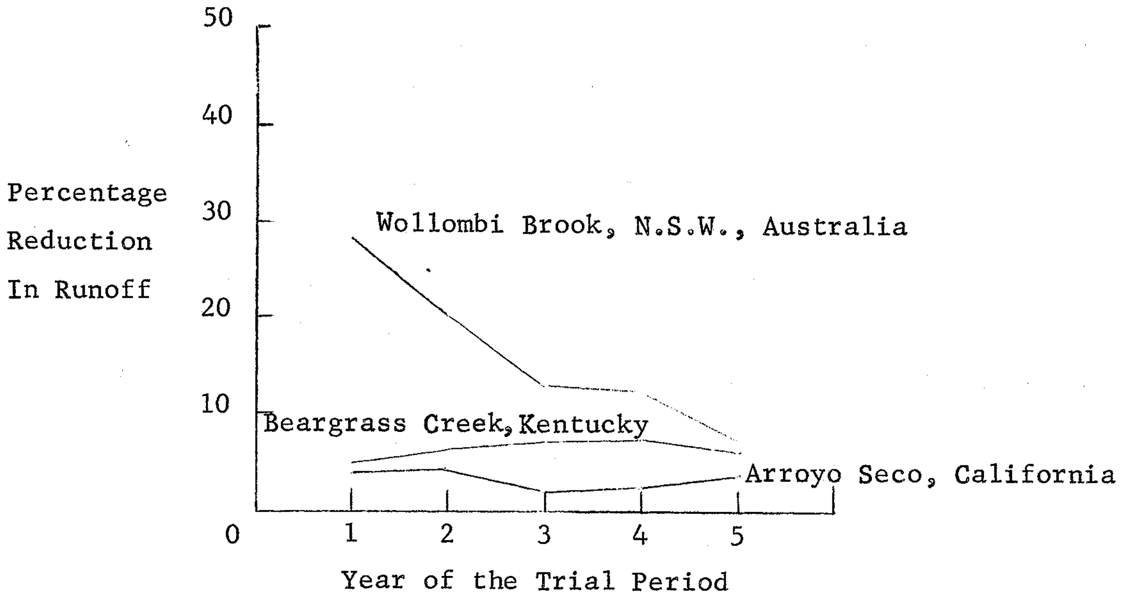


Figure 7.2

These trials show the striking sensitivity of streamflow to relatively small changes in regime, and underscore the role of streamflow as a residual, or the difference between rainfall input and evapotranspiration losses. One logical and interesting trend was observed in all watersheds. On a given stream the percentage changes away from natural conditions are greatest in years of minimum streamflow. However, in these years the effectiveness or efficiency of weather modifications for changing streamflow is also a minimum. Thus, the effect of weather modifications when compared to undisturbed conditions will be most spectacular in arid regions, but the efficiency of modifications in these areas will be low.

Urbanization

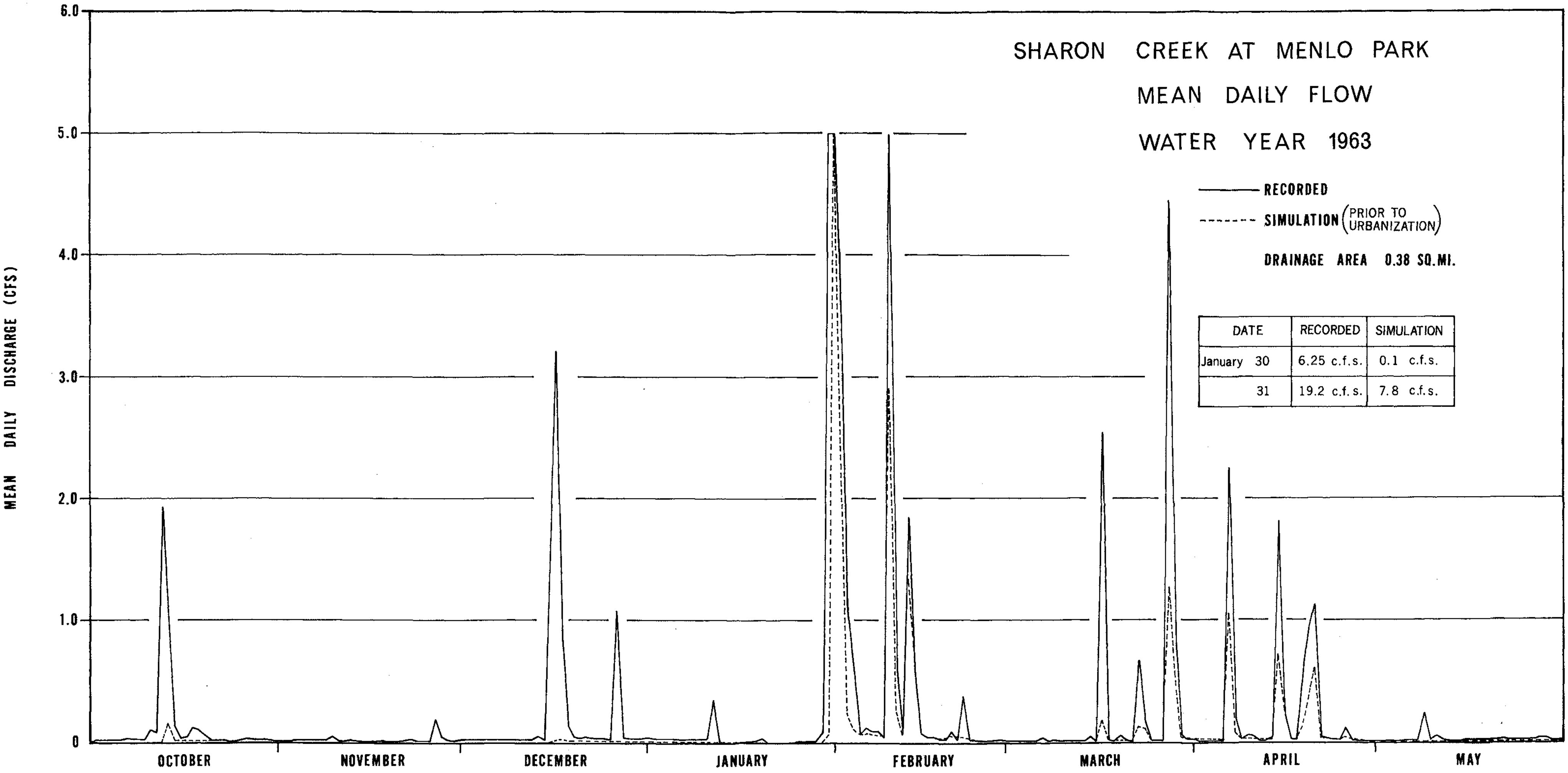
Hydrology in urban areas is the study of watersheds of limited size that are modified by a variety of physical changes. This subject is of interest because of its economic importance for the design of storm drainage and small hydraulic structures. The basic processes in hydrologic response that were described in Chapters III and IV are applicable in urban areas. Impervious area alters runoff volumes, and channel lining or storm sewers may influence the time distribution of runoff, but the land surface processes of infiltration and evapotranspiration still take place on the portion of the watershed that remains pervious, and cannot normally be neglected.

A brief description and some examples of the changes in hydrologic response that accompany urbanization will be given, followed by a summary of the use of simulation for urban drainage design.

Hydrologic Effects of Urbanization Urban development in watersheds alters component runoff processes in a manner that is reasonably predictable. The net effect of these changes in basic processes on the critical gross measures of response, such as flood frequency, is much more difficult to predict. Simulation methods follow the basic processes and output data on the overall regime, and can be readily adapted to the analysis of the trends in hydrologic response that accompany urbanization.

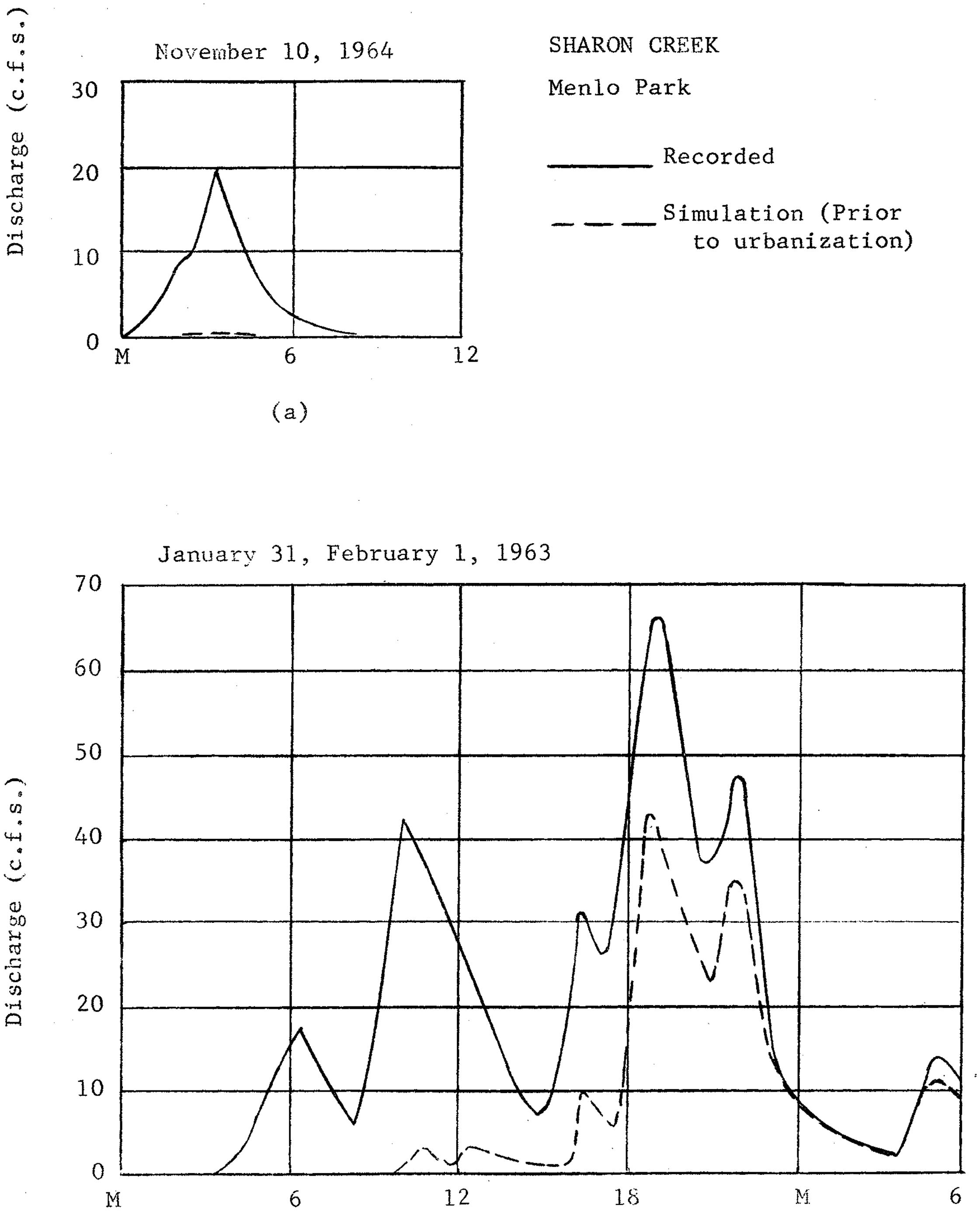
The main land surface processes are infiltration, overland flow, interflow, and evapotranspiration. Infiltration was classified in Chapter IV as direct or delayed. Direct infiltration rates are governed by characteristics of the soil profile, and for the portion of the watershed that remains pervious, they should not be affected by urbanization. The volume of direct infiltration could change without any change in basic infiltration levels, if, for example, grading decreased land slopes. Delayed infiltration is a function of surface characteristics and depression storage and could be expected to change. The amount of impervious area directly connected to stream channels will increase, particularly when storm sewers are installed.

A reduction in the mean length of overland flow on pervious surfaces would be expected. Replacing overland flow on pervious surfaces by overland flow on paved or impervious surfaces causes a major reduction in time delay for the portion of overland flow involved. The quantity of interflow may also vary, although in this case the direction and extent of the change is not clear. Changes in actual evapotranspiration for watershed area that remains pervious should be minor, except when irrigation is introduced.



DATE	RECORDED	SIMULATION
January 30	6.25 c.f.s.	0.1 c.f.s.
31	19.2 c.f.s.	7.8 c.f.s.

Figure 7.3


November 10, 1964
SHARON CREEK
Menlo Park
Recorded
Simulation (Prior to urbanization)
Discharge (c.f.s.)
30
20
10
0
M
6
12
January 31, February 1, 1963
Discharge (c.f.s.)
70
60
50
40
30
20
10
0
M
6
12
18
M
6

(a)

(b)

Figure 7.4

The channel system in typical undeveloped watersheds of 200 acres or less has an almost negligible effect on hydrograph shape. Lining and straightening channels reduces transient storage and resistance to flow, and will tend to further reduce channel attenuation.

To summarize the above, the factors affecting runoff volumes in their most common order of importance are:

i) The increase in impervious area. This effect is highly dependent on the undeveloped regime. In an arid watershed with typical runoff coefficients in the range from 0.0 to 0.2, introduction of twenty percent impervious area will give overall runoff coefficients from about 0.2 to 0.36. In a humid watershed where runoff coefficients are generally 0.5 to 0.8, twenty percent impervious area will give overall runoff coefficients of 0.6 to 0.84. Note that the change is greater if the watershed is more arid, and is greater for the smaller, more frequent storms.

ii) Reductions or increases in surface retention and major changes in vegetation during development.

iii) Changes in overland flow lengths, and irrigation.

The factors that affect the time distribution of runoff are:

i) Reduction in the mean length and roughness for overland flows, particulary the degree or extent of replacement of overland flow over grassland, by flow on asphalt or in storm drains.

ii) Changes in the channel system characteristics; although these usually have negligible effect for small urban areas unless significant changes under (i) above take place. When land surface time delay is greatly reduced, changes in channel system time delay and attenuation may become more important.

The effect of urbanization on runoff volumes can be seen in Figure 7.3, which shows the mean daily discharge in Sharon Creek, California, for 1962-63. Until 1961 this watershed was undeveloped pasture land. A suburban residential development and a golf course were installed in 1961 and 1962. Figure 7.3 is a plot of recorded mean daily discharge, and a simulation of the mean daily discharge that would have occurred if the watershed had not been developed. Small storms throughout the year show the greatest increase, and at the start of the water year runoff was measured when none would have occurred if the watershed was undisturbed. The large storms change less dramatically.

The continuous hydrographs in Figure 7.4 show typical runoff volume response. Figure 7.4(a) is a small early season storm that undergoes a marked change. Figure 7.4(b) is a storm that occurred at higher soil moisture storage levels. The rising limb of this hydrograph is altered, but the change in peak discharge rate is small compared to 7.4(a).

Flow frequency is used for design purposes in small watersheds, and changes in flow frequency that occur during urbanization need to be considered. In a series of simulation runs on small streams made for an analysis of urban drainage requirements*, altering undeveloped conditions by increasing impervious area and reducing overland flow lengths, invariably produced the change in flow frequency sketched in Figure 7.5.

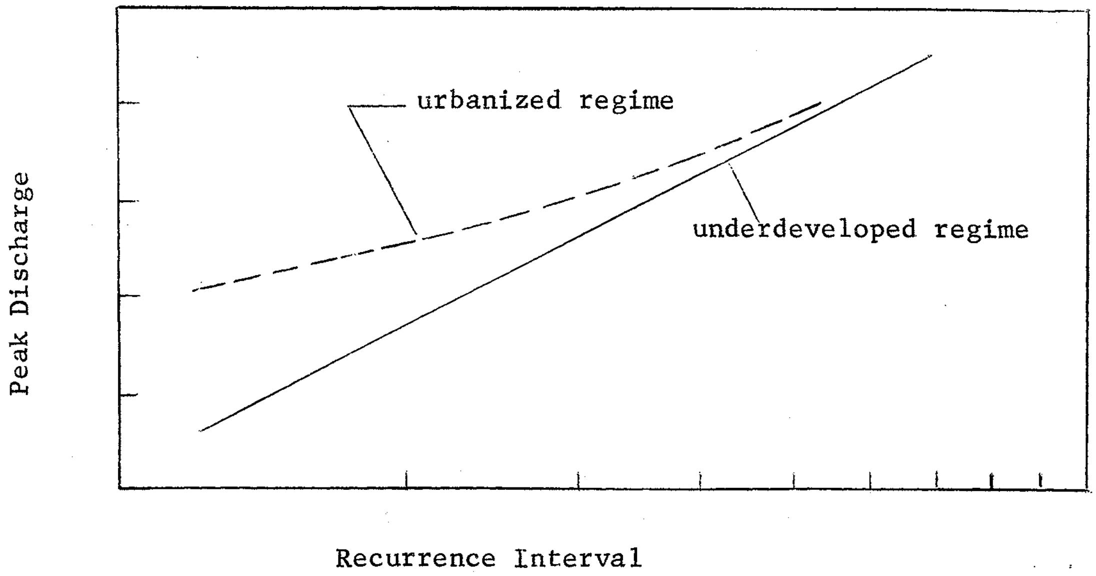


Figure 7.5

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*A summary of output from these studies is contained in the Master Storm Drainage Plan for Santa Clara County, California 1965.

This behaviour is logical and could be anticipated without simulation output, nonetheless, it was reassuring to discover in recent conversations with John Crippen**, that an independent analysis found the same trend in flow frequency for data collected on Sharon Creek.

Urban Drainage Design A brief experiment was carried out recently with a class of graduate students at Stanford. They were asked to estimate the peak flows to be expected at specified frequencies from three drainage areas near the campus. Detailed land use and topographic maps with intensity-frequency-duration plots of rainfall for each area were supplied. The students worked independently and their results are summarized in Table 7.1.

Point	Area	Design Return Period	Average	Design Flow Minimum	Maximum	Standard Deviation
	(acres)	(yrs)	(cfs)	(cfs)	(cfs)	(cfs)
1	98	3	39	6	77	18
2	329	10	101	22	212	49
3	1241	25	359	89	1300	251

Table 7.1

Almost all of the students used the "Rational Formula" method. The wide range in results was due to two main factors: First, the arbitrary choice of a runoff coefficient; and second, considerable variations in the calculated time to equilibrium required to select a rainfall intensity for the rational formula. The errors introduced by variations in the choice of a runoff coefficient were anticipated, but the range in answers found for time to equilibrium from various formulas

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**Sharon Creek is an experimental watershed established by the U. S. Geological Survey to investigate urbanization effects. Data for simulation of this stream was provided through the courtesy of John Crippen of the Geological Survey in Menlo Park, California.

was unexpected.

Application of simulation in design has two major advantages. Continuous short time interval simulation avoids empirical constants. For example, the frequency of flows that enter stream channels is established from historic rainfall and streamflow records without using a "runoff coefficient," an inherent difficulty of conventional design methods[35]. Discharges to be expected at given frequency levels are closely estimated, and the application of engineering economy to drainage design becomes possible.

Simulation of channel system attenuation evaluates another subtle and often neglected process. In conventional design, channel system attenuation is introduced through the concept of a "time of concentration" for urban drainage. A time of concentration is selected and a rainfall rate corresponding to this time period is used, with the assumption that the highest peak will occur when the whole area is contributing uniformly. This assumption is imprecise, and does not need to be used in simulation.

In urban drainage design both the land surface and channel system processes outlined in Chapter III may be important. The designs usually involve very small watersheds whose hydrographs are similar to that shown in Figure 3.7, but larger watersheds or watershed in which overland flow time delay is not predominant may be encountered.

In typical simulation runs, the watershed model is used to produce a long record of continuous hydrographs from which flow frequency data can be directly obtained. It would be possible to use the model in this way for urban drainage design, but to do so would be costly in computer time. Drainage design requires flow frequency for a very large number of small areas, and for these conditions the detailed standard model procedures can be modified to give flow frequency data more efficiently.

The flow from the land surface into stream channels is a function of regional land surface and rainfall characteristics and does not, except in unusual and easily recognized circumstances, depend on the characteristics of a specific small drainage area. Therefore, the frequency characteristics of flow into stream channels can be established on a regional basis by simulating the land surface processes of infiltration, interflow, evapotranspiration, and overland flow, from regional rainfall, topography, and physical characteristics. In the watershed model continuous flow into stream channels is immediately used by the channel system functions to simulate channel outflow. However, in urban drainage runs this need not be done. Instead the maximum sequences of inflow to stream channels are selected from continuous simulation for each water-year. These sequences are then analyzed to determine their frequency characteristics.

At selected frequency levels the land surface runoff or channel inflow for key durations, such as the 15-minute channel inflow with a once in ten year frequency, are found and mapped over the entire area where design flows are needed. Additional computer runs are used wherever there are changes in rainfall and land surface characteristics. Isolines of surface runoff characteristics at known frequency levels are the result.

The channel inflows for known frequency levels are, therefore, available for all of the study area. These data, at the design frequency level selected, can be used in a short program with estimates of main channel length, width, slope, and roughness for each small watershed. Output from the program is the discharge that will be equalled or exceeded from the watershed at the design frequency level.

The key element in this program was adapted from a dissertation by Larson[36], and is shown in Figure 7.6. In Figure 7.6 the peak discharge Q, that results from a channel inflow of Q_t for a duration t, can be found. The program selects t and the corresponding Q_t from channel inflow data, and calculates t_e from Equation A26 in Appendix A. When t/t_e is known, Q/Q_t and Q are found. The program considers all possible durations by incrementing t, and continues until a maximum value of Q is found for the design frequency level.

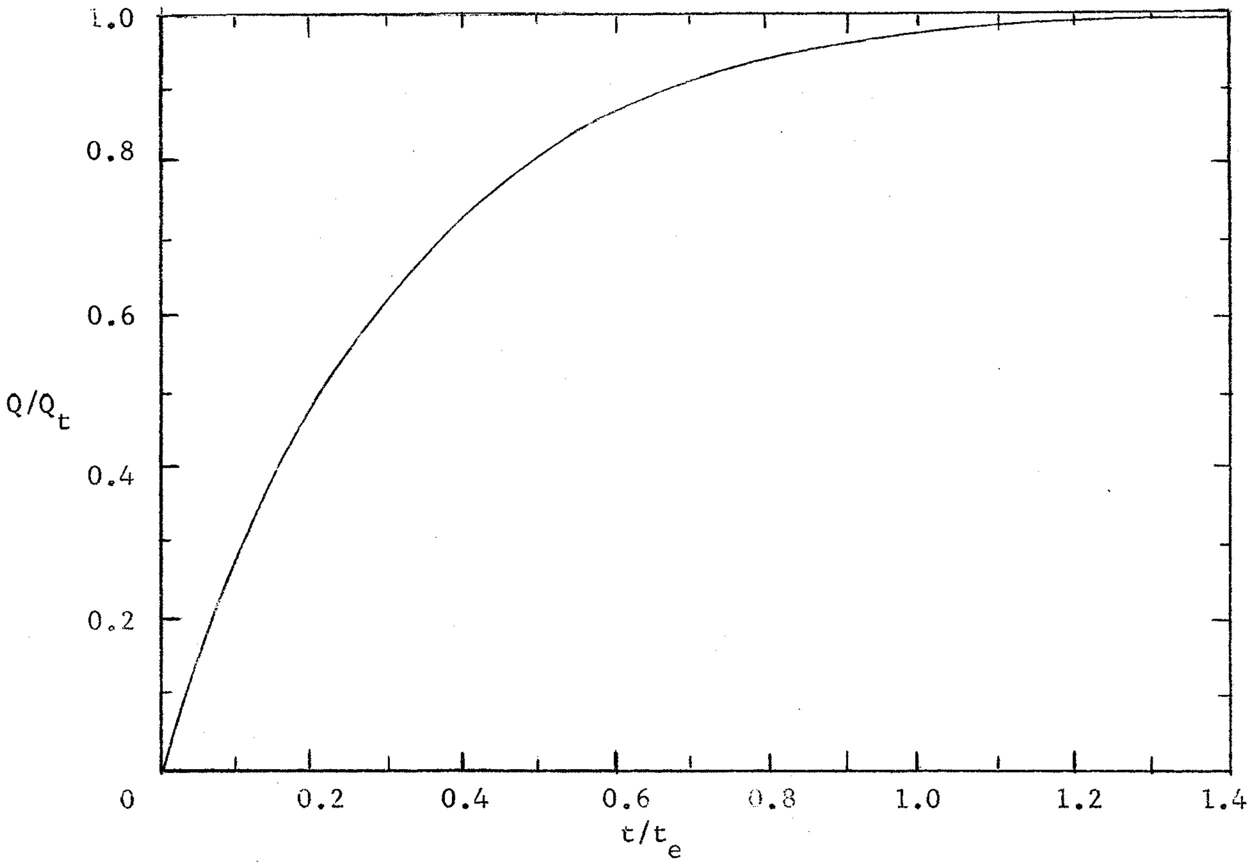

Figure 7.6

Adjustments are made in main channel time to equilibrium to allow for initial flow in the channel system. Since all possible inflow durations are investigated, and since channel inflow rates decrease as the duration t of inflow increases, the peak discharge is usually found when the duration of channel inflow is sixty to eighty percent of the corresponding time to equilibrium.

Digital simulation would not be considered to design an isolated storm drain. Considerable background information is required for this approach, primarily for the development of regional infiltration and moisture storage parameters. However, it is the thorough computer analysis of streamflow and precipitation data that replaces subjective estimates of runoff, and allows realistic derivation of flows at selected frequencies. If several small watersheds are involved, the cost of urban drainage provides ample justification for advanced design methods.

The methods outlined above are for flows with a free surface and would not apply if storm drains surcharge and flow under pressure. They do not apply to large areas since areal variations in rainfall intensity and channel inflow rates were not allowed. For large urban areas or complex storm drainage systems, unmodified continuous simulation is necessary to properly account for the effects of time and areal variations in precipitation and channel or storm drain inflow.

Hydrometeorological Networks

Data networks of precipitation, streamflow, or evaporation stations, have usually been planned separately with network density for each element determined from arbitrary standards, or by available funds. The end-product, of primary importance in most studies of surface hydrology, is an estimate of streamflow probability. Could data networks be planned to permit determination of these probabilities within established limits of accuracy?

When related variables are being measured, maximum information can be extracted from the measurements by correlating the variables. Simulation models relating precipitation to streamflow and other variables, may aid in network design by indicating the density of precipitation gages required to simulate streamflow data within specified tolerances. An example of the extension of streamflow records from precipitation data, and an example of simulation on a stream that was assumed to be ungaged are included in this section.

Network Planning Networks for the collection of hydrometeorological data are established in response to, or in anticipation of, a need for information. Hydrometeorological data is used to describe, in general terms, the hydrometeorological regime of an area; to design water control or water conservation works; or to plan land use and land management.

For a general description of the hydrometeorological regime a sample of relatively low density is adequate, but for specific design problems a network suitable as a descriptive data source may be quite inadequate. Considering the variety of design problems encountered in the utilization and control of water, data on the probability distribution of floods and runoff volumes on all streams of economic importance should be available.

In the highly populated areas of the world, streams draining five square miles or more will have some economic importance. The requirement for "data on the probability distribution of floods and runoff volumes" would be satisfied by a long period of records from a well maintained stream gage, but other less precise means of estimating these probability distributions may need to be substituted.*

The quality or accuracy of estimates of the probability distributions of floods and runoff volume varies, depending on the method of analysis and on available data. A logical approach to network design would complement methods of analysis, by supplying data of maximum utility for improving estimates of these probability distributions. The type of data most needed, and optimum gage locations, can be indicated through analysis of simulation output. For example, simulation might show that stream gages are sparse in a given region, and that two or three added stream gages would sharply improve the minimum accuracy of simulation for twenty or thirty nearby streams.

Experiments in Simulation The following two figures were taken from a recent paper[37] on the coordination of precipitation and streamflow networks. They illustrate two important considerations in the collection of hydrometeorologic records; the density of gaging, and the time interval on which gages are operated.

In Figure 7.7 the reproduction of streamflow frequency using raingage networks of different densities was investigated.

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*In the 3 million square mile area of the conterminous United States, approximately 500,000 streamgaging stations might be needed if recorded flows were the only acceptable means of estimating probability distributions. Only about 10,000 gaging stations are now operating, a two percent sample.

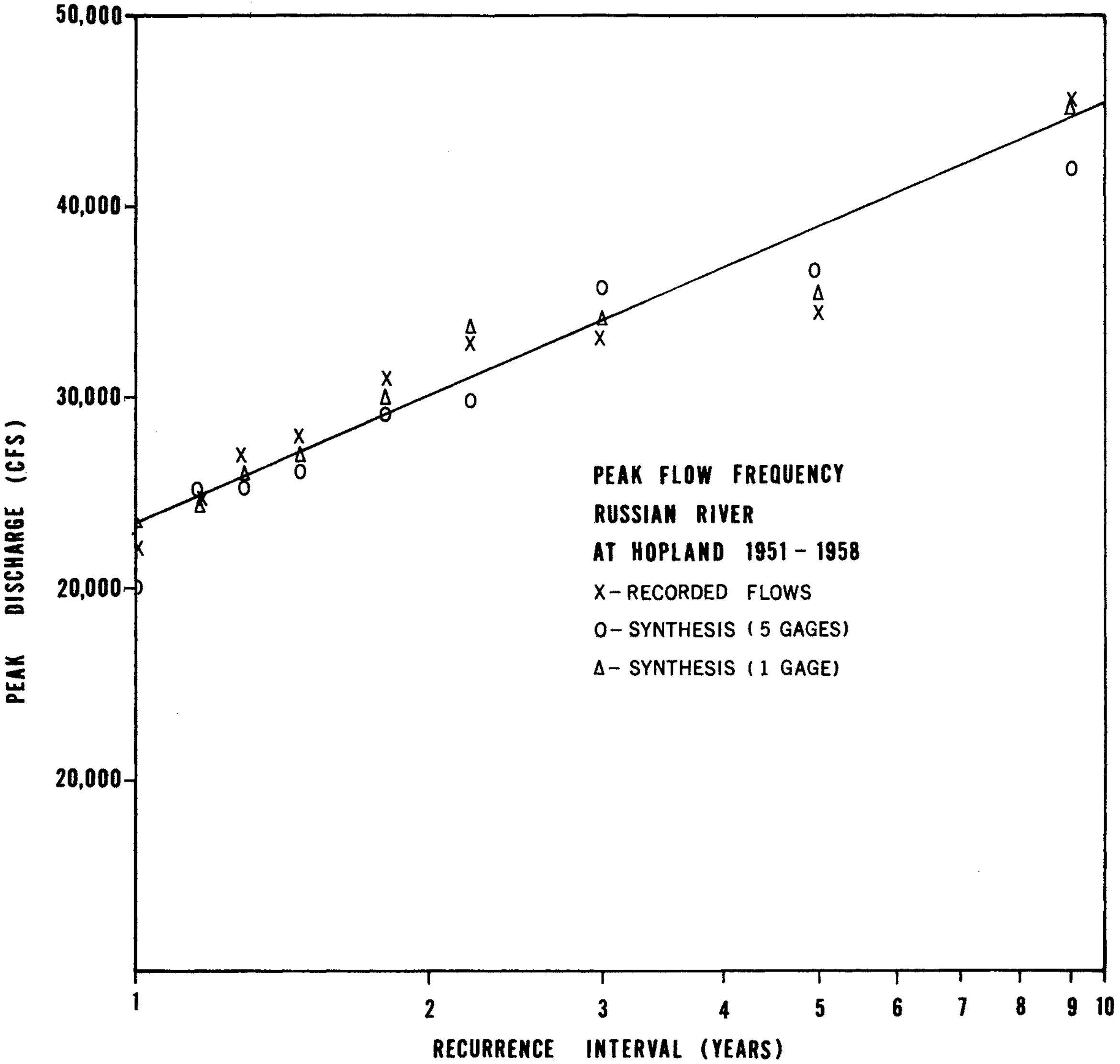
50,000
40,000
30,000
20,000
20,000
PEAK DISCHARGE (CFS)
1
2
3
4
5
6
7
8
9
10
RECURRENCE INTERVAL (YEARS)
PEAK FLOW FREQUENCY
RUSSIAN RIVER
AT HOPLAND 1951 - 1958
X - RECORDED FLOWS
O - SYNTHESIS (5 GAGES)
Δ - SYNTHESIS (1 GAGE)

Figure 7.7

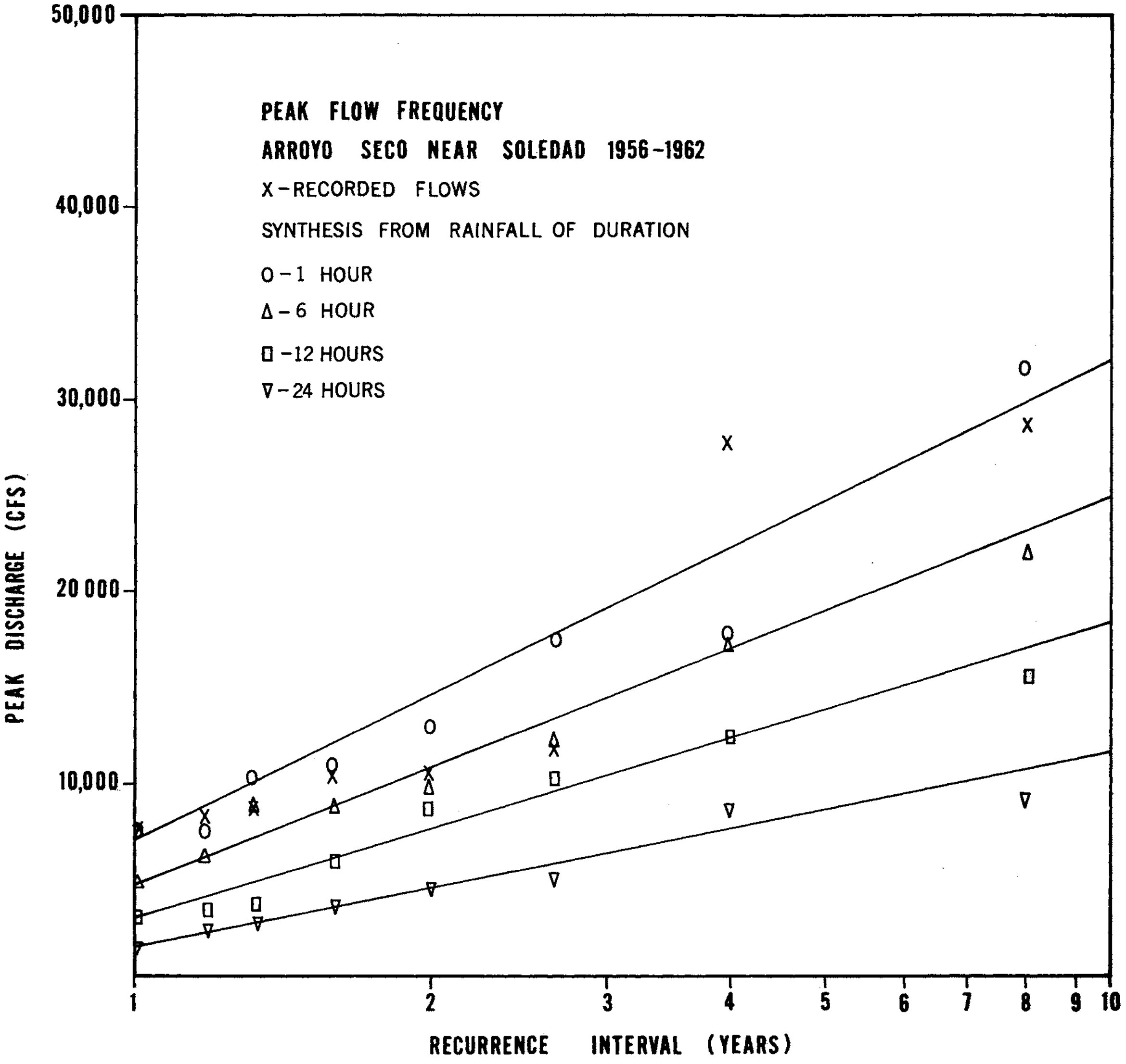
PEAK FLOW FREQUENCY
ARROYO SECO NEAR SOLEDAD 1956-1962
X - RECORDED FLOWS
SYNTHESIS FROM RAINFALL OF DURATION
O - 1 HOUR
Δ - 6 HOUR
□ - 12 HOURS
∇ - 24 HOURS
PEAK DISCHARGE (CFS)
50,000
40,000
30,000
20 000
10,000
1
2
3
4
5
6
7
8
9
10
RECURRENCE INTERVAL (YEARS)

Figure 7.8

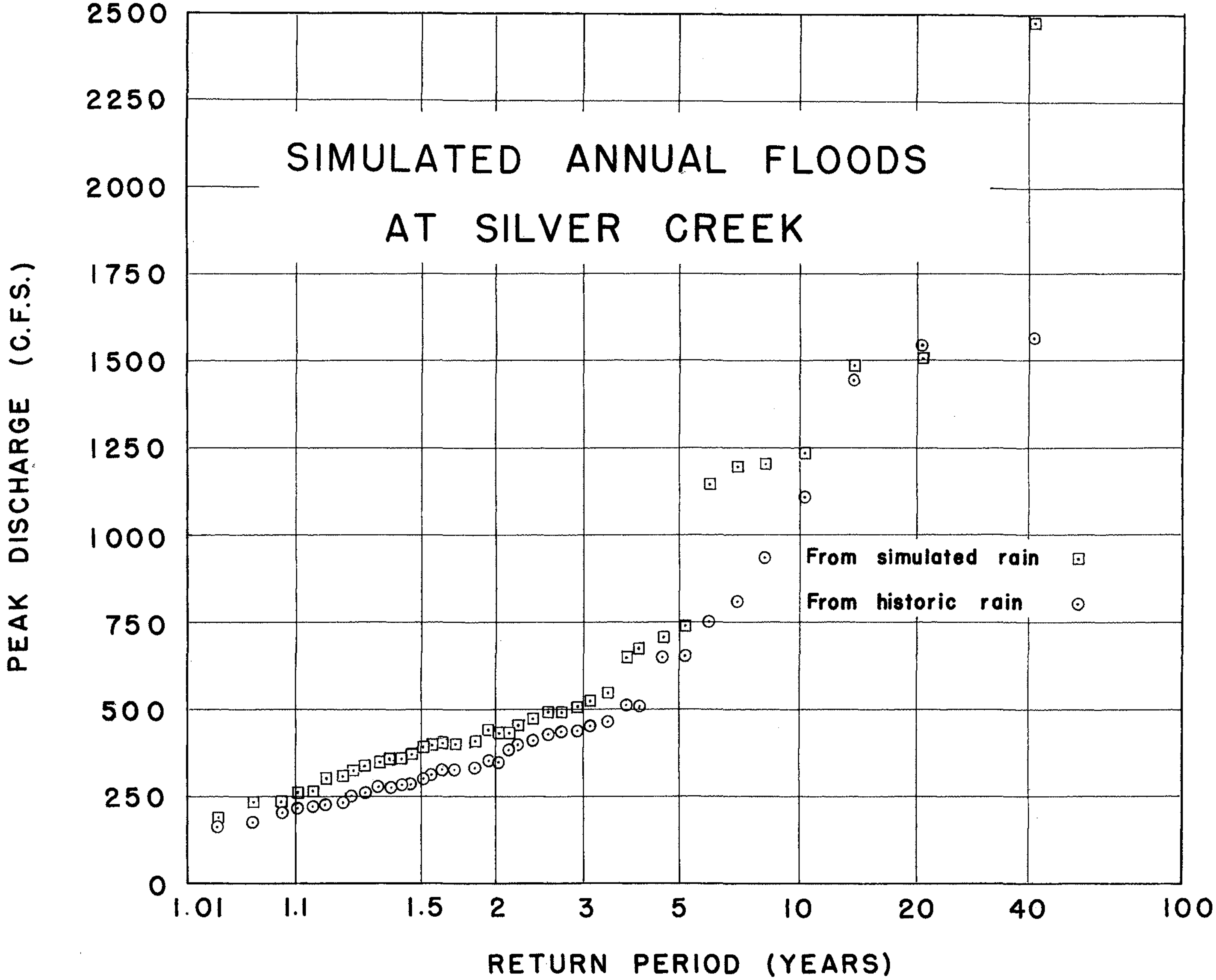
SIMULATED ANNUAL FLOODS
AT SILVER CREEK
PEAK DISCHARGE (C.F.S.)
2500
2250
2000
1750
1500
1250
1000
750
500
250
0
1.01
1.1
1.5
2
3
5
10
20
40
100
RETURN PERIOD (YEARS)
From simulated rain ⊡
From historic rain ⊙

Figure 7.9

The figure shows a comparison between synthesized and recorded partial duration series peak flow frequency for the Russian River at Hopland for water-years 1951 to 1958. The accuracy of peak flow frequencies calculated from simulated flows, changes only slightly as the rain gage network is reduced, from five gages in or very close to the watershed, to only one gage at St. Helena, 75 miles south of the watershed.

In Figure 7.8, simulation is used to investigate the effect of the minimum observed time increment of rainfall on predicted streamflow frequencies. The peak flow frequency for Arroyo Seco near Soledad, California, an area of 241 square miles, is shown using time increments of one, six, and twenty-four hours. Rainfall is assumed to be evenly distributed in each time interval. Recording gage data giving short interval rainfall is clearly necessary for adequate simulation.

Still another experiment is shown in Figure 7.9, which was taken from a dissertation by Pattison[38]. This figure compares the peak flow frequencies simulated from a forty-year record of historic rainfall, with peak flow frequencies simulated from a forty-year record of statistically synthesized rainfall.

The examples above suggest the possibility of establishing mean annual precipitation patterns from non-recording rain gages, perhaps using roving gages, and relying on a few well maintained recording gages to provide a basis for simulating precipitation and streamflow data for design purposes. They do not imply, of course, that such procedures would always be satisfactory. Nonetheless, the burgeoning requirements for hydrologic design data for both current or future projects severely tax the capabilities of the hydrometeorological network, and simulation techniques allow maximum use of data from the full hydrometeorological network.

Ungaged Watersheds The accurate prediction of streamflow regimes in the absence of recorded streamflow data has always been a major goal in hydrology. The use of simulation for this application depends on the quality and quantity of related data, mainly precipitation data, that is available. It also depends on successful projection to estimate watershed parameters for the ungaged streams.

RUSSIAN RIVER NEAR HEALDSBERG

Water Year		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Flow)
45	Rec	0.23	1.32	2.41	1.43	5.39	3.40	1.39	0.67	0.35	0.21	0.20	0.24	17.3	0.9886
	Sim	0.25	1.51	2.78	1.39	5.69	3.63	1.32	0.68	0.44	0.31	0.26	0.24	18.5	
46	Rec	0.60	1.85	10.9	4.34	1.75	1.53	1.10	0.48	0.25	0.17	0.16	0.17	23.3	0.9803
	Sim	0.99	2.16	10.2	4.07	1.91	1.62	1.16	0.52	0.33	0.21	0.16	0.22	23.6	
47	Rec	0.21	0.68	1.17	0.43	2.38	3.54	1.24	0.31	0.24	0.10	0.12	0.17	10.6	0.9794
	Sim	0.18	1.11	1.11	0.52	2.42	4.30	1.38	0.40	0.30	0.18	0.22	0.23	12.4	
48	Rec	0.49	0.49	0.52	2.15	0.97	2.87	5.90	1.79	0.58	0.20	0.20	0.24	16.4	0.9776
	Sim	0.45	0.54	0.46	2.07	0.98	2.75	5.99	1.77	0.50	0.23	0.22	0.19	16.2	
49	Rec	0.27	0.31	1.00	1.06	2.88	8.91	1.07	0.52	0.14	0.12	0.17	0.15	16.6	0.9859
	Sim	0.34	0.34	0.59	0.90	2.37	9.90	1.11	0.66	0.25	0.24	0.21	0.18	17.2	

Table 7.2

DRY CREEK NEAR NAPA

Water Year		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Annual	Correlation Coefficient (Mean Daily Discharge)
58	Rec	0.33	0.16	1.13	2.82	13.7	6.48	9.82	0.55	0.21	0.07	0.01	0.00	35.3	0.9556
	Sim	0.56	0.23	0.73	3.49	12.3	5.80	6.99	0.42	0.13	0.05	0.02	0.00	30.8	
59	Rec	0.00	0.01	0.04	1.31	3.74	0.60	0.18	0.08	0.01	0.00	0.00	0.06	6.0	0.9787
	Sim	0.00	0.01	0.06	1.19	3.77	0.60	0.24	0.10	0.04	0.02	0.01	0.18	6.2	
60	Rec	0.01	0.01	0.03	0.41	4.48	1.91	0.44	0.18	0.04	0.00	0.00	0.00	7.5	0.9769
	Sim	0.02	0.01	0.07	0.29	6.60	2.13	0.48	0.22	0.09	0.04	0.02	0.01	10.0	
61	Rec	0.00	0.05	0.36	0.73	1.28	1.56	0.60	0.16	0.03	0.00	0.00	0.00	4.8	0.9278
	Sim	0.02	0.17	0.58	0.95	2.14	1.54	0.55	0.22	0.08	0.04	0.02	0.02	6.3	
62	Rec	0.00	0.06	0.79	0.70	7.23	3.69	0.51	0.17	0.03	0.00	0.00	0.00	13.2	0.9018
	Sim	0.01	0.12	0.36	0.33	6.88	3.94	0.47	0.17	0.06	0.03	0.01	0.01	12.4	

Table 7.3

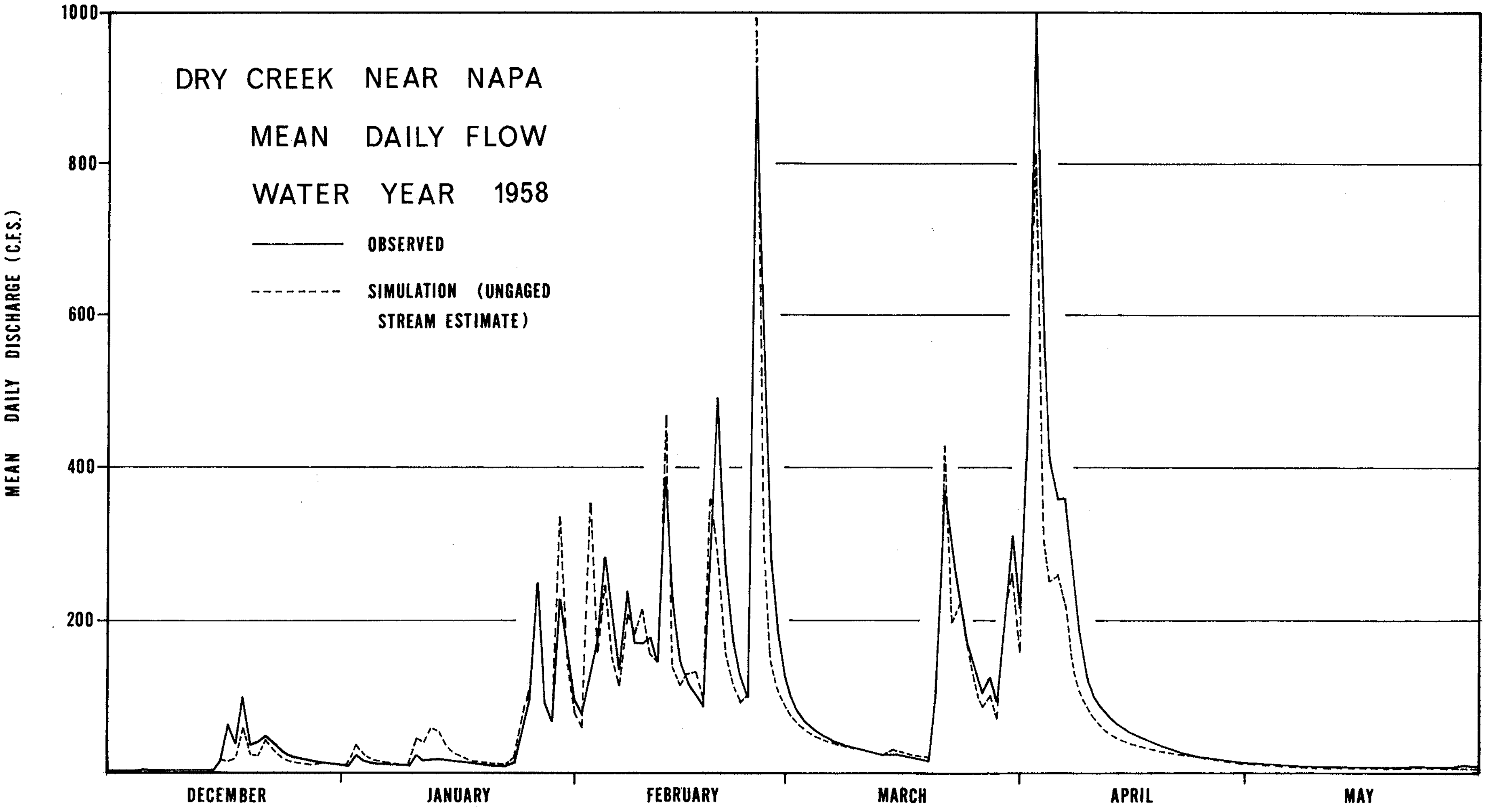
DRY CREEK NEAR NAPA
MEAN DAILY FLOW
WATER YEAR 1958
OBSERVED
SIMULATION (UNGAGED STREAM ESTIMATE)
MEAN DAILY DISCHARGE (C.F.S.)
1000
800
600
400
200
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
MAY

Figure 7.10

A similar problem, the extension of streamflow records in gaged streams, works well when watershed parameters have been developed from the period when a stream gage was in operation. Table 7.2 shows an experimental extension of records for the Russian River at Healdsburg for water years, 1945 to 1949. This extension was based on parameters developed using stream gage records from the period, 1951 to 1958. Compared to the recorded flows for 1945 to 1949, the accuracy of the simulated flows was not significantly different than that achieved in Chapter VI for the calibration period.

For streams that have never been gaged the problem is more difficult since some watershed parameters must be estimated. From Chapter V, Model IV has four major parameters that must be estimated. There are two possible approaches to deriving estimates. The parameters might be correlated with physical features such as soil permeability and depth, or they might be mapped or correlated on a regional basis from parameters in nearby gaged streams.

An example of simulation that used regional estimates for the unspecified parameters is shown in Figure 7.10, and Table 7.3. Figure 7.10, shows mean daily simulated and recorded flows for Dry Creek near Napa, California. This watershed was run only once, and the parameters used were based on a projection from the Russian River and Napa River parameters.

A large scale analysis of the same type has been made for Santa Clara County in California. Streamflows were simulated for a thirty-year period on ten ungaged watersheds using parameters mapped from six gaged watersheds in the same area. The records on the gaged streams were extended to obtain thirty-years continuous records at all stream gages.

The accuracy obtained for the streams that have never been gaged, is of course unknown, but it is probably similar to that found in the gaged streams in same geographical area. The storage and infiltration parameters are quite stable and usually vary only slightly for adjacent watersheds.

The accuracy of simulation output in arid and semi-arid climates is strongly dependent on the rainfall inputs, as would be expected from the discussion of sensitivity to weather modification. Streamflow volumes and flood frequencies react quite strongly to small differences in annual precipitation, and the most serious source of error in simulation of streamflow in an ungaged arid watershed is likely to be a poorly defined rainfall regime.

VIII. CONCLUSIONS

Digital simulation methods for the hydrologic cycle have been described in detail in this report, and some of the current applications of these techniques have been outlined.

Digital simulation could be called controlled experimentation in mathematical representations. A digital computer is an admirable calculating machine, but if large scale computers are considered to be calculating machines in the organization of research, a great deal of their potential is lost. Computers can be of great value in the process of determining what sort of calculations should be done, and many mathematical and logical representations can be investigated and optimized. It was pointed out in Chapter II that parameter optimization does not imply optimum mathematical representations, but the systematic development of improved mathematical representations through rapid interchange with digital computers is a very effective procedure.

The practical problems of design and operation of water resource systems are steadily becoming more numerous and complex. Estimates or predictions of peak flow frequency and runoff volumes are necessary for almost all hydraulic projects. Typically, the hydrologist has little or no streamflow record and limited rainfall data to use as a basis for predicting behavior at the project site. Despite meager data, progress in hydrology requires that techniques be developed to improve the general level of accuracy of numeric predictions.

Consider, for example, an engineer or hydrologist with the problem of estimating peak flow frequencies and runoff volumes on a small ungaged stream. He would likely use one or more of the following techniques.

1) Records of nearby gaged streams might be transposed, perhaps with some modifications, to the project site.
2) Records of rainfall might be used together with regional rainfall-runoff relationships and a synthetic unit-hydrograph to reconstruct streamflows.
3) Regional graphical correlations might be developed to relate peak flow and runoff volume with watershed parameters.

4) Flood peak or flow volume formulas might be applied.

All of the techniques above are sometimes useful but all have limitations. Perhaps the most important limitation is that the most sophisticated methods are seldom used in practice. To a designer, the relative merits of different methods for a particular application may be difficult to establish, and some of the methods are time consuming while others are simple. The amount of time that can be used for hydrologic analysis for the design of a small or moderate hydraulic structure is limited, and relatively simple methods that do not involve detailed correlations or data searches are very appealing.

One of the fascinations of digital computers is that complex and advanced methods can be used in practice at moderate cost. Digital simulation, for example, offers the opportunity to establish computation services that could assist hydrologists by making available simulated flows for many streams. The designer could request information at a project site and receive after a few minutes of computation any or all of the following:

1) Historic recorded flows at the project site and statistical estimates of gaging accuracy.
2) Simulated streamflows and statistical estimates of simulation accuracy from data based on:
    a) Recorded flows at other sites on the stream augmented by simulation.
    b) Recorded data from precipitation stations.
    c) Statistically generated precipitation data.

The designer could also request flow frequency estimates based on the data from any of the several commonly used statistical projections. The advantages to be gained from such a system result partly from the well known calculating speed, and efficient information storage and retrieval characteristics of large-scale computers, and partly from the increased efficiency that specialization allows. It becomes feasible, for example, to search out and evaluate all of the hydrometeorologic records that exist in a region, if it is known that the data will be used for the design of several hundred structures each year. Refining and improving the general simulation procedures used by these computation services for streamflow or precipitation data would be a major concern of hydrologists. These studies should be more productive than the piecemeal approach of data searches and calculations for each individual project.

Large projects and interconnected systems can also be analyzed using simulation methods. Enormous investments in water resources and close inventory control of water supplies are indicated in the near future in many parts of the world. Simulation models give immediate and accurate calculations of the water balance and water requirements over large areas. Ultimately, probabilistic models based on continuous simulation will very likely govern interbasin transfers of water supplies.

Simulation is ideally suited to studies of hydrologic sensitivity. These may involve the changes in weather or urban development discussed in Chapter VII, or purely hypothetical changes to enable a student to discover the effect on regime of a physical parameter. Thus, applications of simulation are anticipated in research programs and education, as well as in the design and control of engineering projects.

Digital hydrologic simulation has been actively developed at Stanford for about six years. In that time some progress has been made in the development of digital models, nonetheless, the potential of digital simulation methods remains largely unexplored and rapid developments in the application of these methods may now be imminent.

Comments on this report or requests of information about the programs used are invited.

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APPENDIX A

Approximations for Two-Dimensional Flow

The movement of overland flow toward stream channels is generally assumed to be two-dimensional or to take place in a thin sheet of infinite width. The flow in open channels is also often assumed to be two-dimensional. This assumption is reasonably close for flow in broad channels of shallow depth. Flow in channels is almost always turbulent while overland sheet flow is considered to be initially laminar, becoming turbulent only if the depth and velocity of the flow increase sufficiently before a stream channel is reached.

In this appendix some simple analytic approximations for two-dimensional flows are outlined. These analytic approximations are then compared to experimental results. Finally, the algorithms used in the general watershed model are described and typical output is illustrated. The output plotted in the figures at the end of this section was calculated using the general model equations with time intervals of 0.5 to 2 minutes.

Analytic Approximations The features of overland flow that are of primary interest are those that govern the response to various patterns of uniform rainfall. For both laminar and turbulent flows* the discharge is related to flow depth in the form

$$q = a\, y^{b} \tag{A1}$$

where y is flow depth and a and b are constants. From the properties of laminar flow;

$$q = \frac{g\, S\, y^{3}}{3\nu} \tag{A2}$$

---

*For convenience two-dimensional flows are described as in the laminar or turbulent range based on undisturbed flow criteria.

where g is the gravitational constant, y is depth in ft., S is slope in ft/ft, and ν is the kinematic viscosity of the flow. For turbulent flows Manning's equation indicates

$$q = \frac{1.486}{n} \quad y^{5/3} \quad S^{1/2} \tag{A3}$$

where n is Manning's n. In Equations A2 and A3 the hydraulic radius of the flow is assumed equal to the flow depth, and the energy gradient is assumed equal to the gradient of the flow plane.

The continuity equation for two dimensional flow is

$$\frac{\partial q}{\partial x} = \eta - \frac{\partial y}{\partial t} \tag{A4}$$

where η is the inflow or supply rate in $ft^3/sec/ft^2$. Two useful conclusions can be drawn from Equation A4. At equilibrium $\frac{\partial y}{\partial t}$ is zero and

$$q_e = \eta x \tag{A5}$$

where q_e is discharge in $ft^3/sec/ft$ at any section x on the flow plane. As Wolf[40] points out, the change in discharge as a function of x on a uniformly sloping plane must be zero, before local equilibrium is reached. Hence, the depth at any point on the plane is

$$y = \eta t \tag{A6}$$

prior to local equilibrium.

For overland flow on the plane in Figure A1, with a depth y at a distance x along the flow plane, the general shape of overland flow hydrograph between t = o and t = t_e, is shown in Figure A2.

The outflow at equilibrium is $q_e = \eta L$, and the total inflow

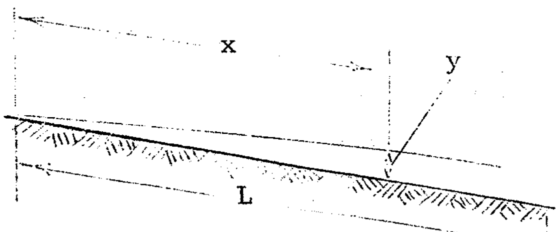


Figure A1

between time $t = o$ and time $t = t_e$ is $t_e \eta L$.

The volume of surface detention at $t = t_e$, is

$$D_e = \int_0^L y dx \tag{A7}$$

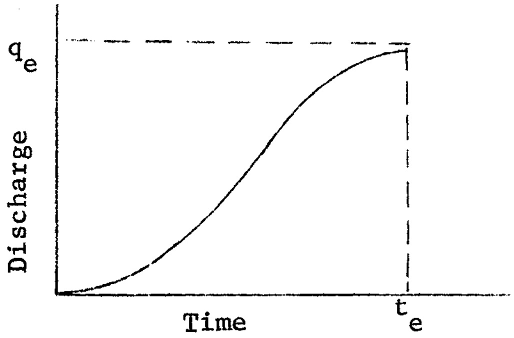


Figure A2

The total runoff from time $t = 0$ to $t = t_e$ is some fraction β, of the total inflow during the period. Hence

$$\beta t_e \eta L = \int_0^{t_e} q dt \tag{A8}$$

The total inflow is

$$t_e \eta L = D_e + \beta t_e \eta L \tag{A9}$$

or the time to equilibrium is

$$t_e = \frac{D_e}{\eta L (1.0 - \beta)} \tag{A10}$$

From Equation A6 the depth near the lower edge of the flow plane should be

$$y = \left(\frac{t}{t_e} \right) y_e \tag{A11}$$

from $t = 0$ to $t = t_e$.

From Equation A1 and A11

$$q = a y^b = a \left(\frac{t}{t_e} \right)^b y_e^b \tag{A12}$$

Hence from Equation A8

$$\beta \, t_e \eta L = \frac{a \, y_e^{\,b}}{t_e^{\,b}} \int_0^{t_e} t^b \, dt \tag{A13}$$

and

$$\beta = \frac{1}{b + 1} \tag{A14}$$

The detention at equilibrium is*

$$\int_0^L y \, dx$$

where y from Equation A1 is

$$y = \left(\frac{q}{a} \right)^{1/b}$$

and $q = \eta x$ at equilibrium. Hence

$$D_e = \frac{\eta^{1/b}}{a^{1/b}} \int_0^L x^{1/b} \, dx \tag{A15}$$

or

$$D_e = \frac{b \, \eta^{1/b} \, L^{(1 + 1/b)}}{a^{1/b} \, (b + 1)} \tag{A16}$$

Substituting Equation A16 in Equation A10, the general expression for time to equilibrium is

$$t_e = \frac{\eta^{(1/b - 1)} \, L^{1/b}}{a^{1/b}} \tag{A17}$$

---

*It can be shown[25] that the depth at x = 0 is not zero, but the extra storage due to this effect is negligible.

Substituting the values of the parameters a and b from Equations A2 and A3, into Equations A14, A16, and A17, the following expressions are found.

For laminar flows on smooth surfaces:

$$b = 3$$

$$\beta = 1/4$$

$$D_e = \frac{0.0078\, \eta^{1/3}\, L^{4/3}}{S^{1/3}} \qquad \text{(A18)}$$

in ft^3/ft

or

$$D_e = 0.000222\ \frac{i^{1/3}\, L^{4/3}}{S^{1/3}} \qquad \text{(A19)}$$

in inches depth per unit area, where i is supply rate in inches/hour.

The time to equilibrium in minutes is

$$t_e = \frac{1.33\, D_e}{60\, L} \qquad \text{(A20)}$$

hence

$$t_e = \frac{0.0104\, L^{1/3}}{\eta^{2/3}\, S^{1/3}} \qquad \text{(A21)}$$

or

$$t_e = \frac{16.4\, L^{1/3}}{i^{2/3}\, S^{1/3}} \qquad \text{(A22)}$$

For turbulent flows:

$$b = 1.67$$

$$\beta = 3/8$$

$$D_e = \frac{0.492\, \eta^{3/5}\, n^{3/5}\, L^{8/5}}{S^{3/10}} \qquad \text{(A23)}$$

or

$$D_e = \frac{0.000818\, i^{3/5}\, n^{3/5}\, L^{8/5}}{S^{0.3}} \qquad \text{(A24)}$$

for surface detention in ft^3/ft.

Time to equilibrium in minutes is

$$t_e = \frac{1.6\ D_e}{60\ \eta\ L} \qquad (A25)$$

$$t_e = \frac{0.0132\ \ L^{3/5}\ n^{3/5}}{\eta^{2/5}\ \ S^{3/10}} \qquad (A26)$$

or

$$t_e = \frac{0.94\ \ L^{3/5}\ n^{3/5}}{i^{2/5}\ \ S^{3/10}} \qquad (A27)$$

In the equations above, the surface detention D_e is in ft^3/ft, where η is in $ft^3/sec/ft^2$. Equations for rainfall or supply rate in inches per hour are obtained by substituting $i = 43200\eta$. In these equations surface detention is in inches depth per unit area. The estimates of time to equilbrium t_e are in minutes.

Equations A2, A3, and A12 also result in the idealized shapes of overland flow response shown in Figure A3.

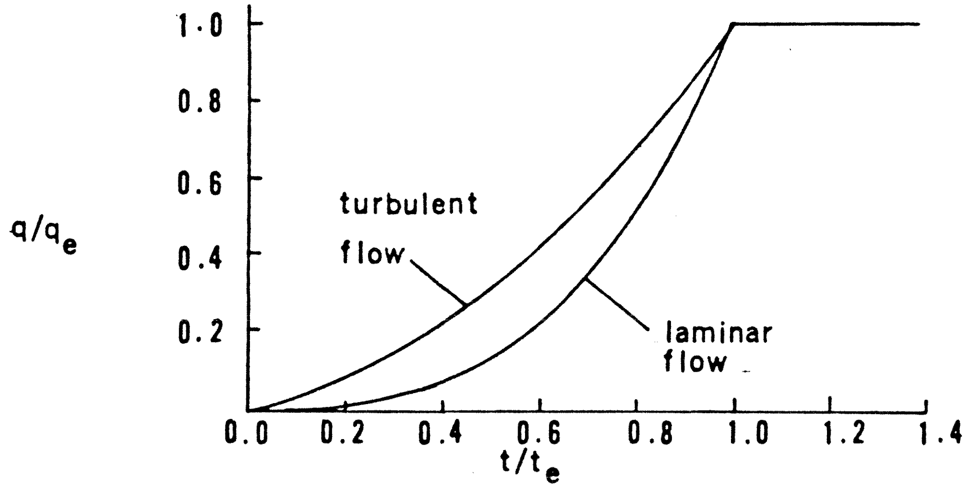


Figure A3

Comparisons to Experimental Results. The analytic approximations outlined above serve only as a guide to the behavior of actual flows. In actual overland flow the lower critical Reynolds number, defining the limit below which turbulence entering the flow will damp out, is about 500 fo sheet flow.[39] From the properties of laminar flow over a smooth surface where the hydraulic radius of the flow is equal to the depth, the discharge at equilibrium at any distance L from the top of the flow plane is:

$$q = \frac{iL}{43200} \tag{A28}$$

where q is discharge in $ft^3/sec/ft$ and i is the supply rate in inches/hour. Equilibrium conditions are reached when the inflow along the flow plane is equal to the volume of discharge from the flow plane. For a Reynolds number of 500 for a flow with a mean velocity V and a depth y:

$$500 = \frac{Vy}{\nu} = \frac{q}{\nu} = \frac{iL}{43200\,\nu} \tag{A29}$$

This corresponds to a value of iL of approximately 260, or a product of flow velocity and depth of 0.00605 $ft^3/sec/ft$ using a kinematic viscosity ν of 1.21×10^{-5} ft^2/sec for water at 60° F. Overland sheet flow during rainfall is subject to major turbulence as rain strikes the surface of the flow and should be turbulent at the lower critical Reynolds number. In the laminar flow range the most extensive series of experimental measurements were carried out by Izzard[15,16]. Izzard used the total detention at equilibrium as a parameter to define the response of an overland flow plane to a uniform rainfall. The time to equilibrium, or the time required in minutes for outflow from the overland flow plane to equal the rate of rainfall on the overland flow plane, was defined as:

$$t_e{}^{*} = \frac{2\,D_e}{60\,q_e} \tag{A30}$$

where q_e is the equilibrium discharge in ft^3/sec, and D_e is the surface detention in ft^3/ft at equilibrium. Izzard found that the detention at equilibrium could be empirically estimated as

$$D_e = \frac{KL^{4/3}\ i^{1/3}}{35.1} \tag{A31}$$

---

*Since equilibrium is approached asymptotically this definition of time to equilibrium corresponds to a value of $q/_{q_e}$ of 0.97.

where K is a parameter given by,

$$K = \frac{(0.0007i + C)}{S^{1/3}} \qquad \text{(A32)}$$

C is a coefficient that varies with the roughness of the overland flow plane, and S is the slope of the flow plane. The use of a roughness parameter for a laminar flow appears to reflect energy loss from raindrop impact, and the influence of roughness projections in the flow boundary on hydraulic radius. These projections can be large compared to typical depths of overland flow.[21]

Equation A19 for surface detention at equilibrium during overland flow on a smooth surface can be compared to Equations A31 and A32 using the minimum value found for the parameter C. Substituting C of 0.007 in equations A31 and A32 gives

$$D_e = (0.0002 + 0.00002i) \frac{i^{i/3} L^{4/3}}{S^{1/3}} \qquad \text{(A33)}$$

Hence, equation A19 is a good estimate of surface detention for smooth surfaces.

Equation A20, however, does not agree with Izzard's experimentally derived estimate in Equation A30. This is to be expected since the approximate dimensionless response in Figure A3 is also not found in experimental trials.

In the turbulent range the surface detention at equilibrium varies as a function of the resistance parameter n. In order to compare laminar and turbulent range surface detention Figure A4 is used to relate Manning's n and Izzard's C. Laminar and turbulent range surface detentions were assumed to be equal at the limit of the laminar range, where iL is about 260. From the ratio of Equations A23 and A31,

$$\frac{D_e \text{ turb}}{D_e \text{ lam}} = \frac{0.0287 \; n^{0.6} \; (iL)^{0.27} \; S^{0.03}}{(0.0007i + C)} \qquad \text{(A34)}$$

the ratio of turbulent to laminar range surface detention will be one for all combinations of iL of 260 if

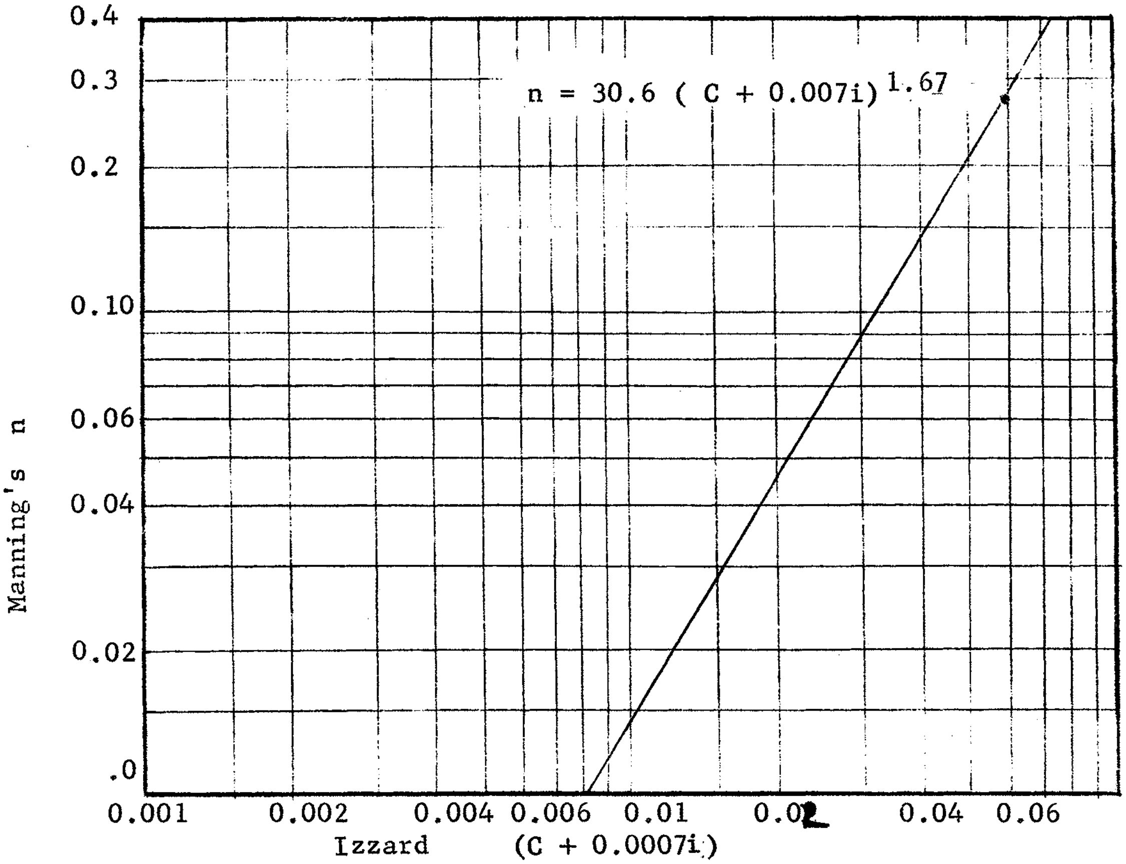


Figure A4

$$n = 30.6 \quad (C + 0.0007i)^{1.67} \tag{A35}$$

assuming that $S^{0.03} \approx 1.0$. Thus, Figure A4 is based on the assumption that laminar and turbulent range surface detentions should be equivalent at the assumed lower boundary of the turbulent range. Figure A4 thus gives the values of Manning's n that could be expected for the surfaces used in Izzard's experiments.

The general trend of relative detention storage is shown in Figure A5. Figure A5 shows the change in the ratio of the volume of detention for turbulent flow to the volume of detention for laminar flow as the value of iL increases, assuming the values of n and C are related by Figure A4.

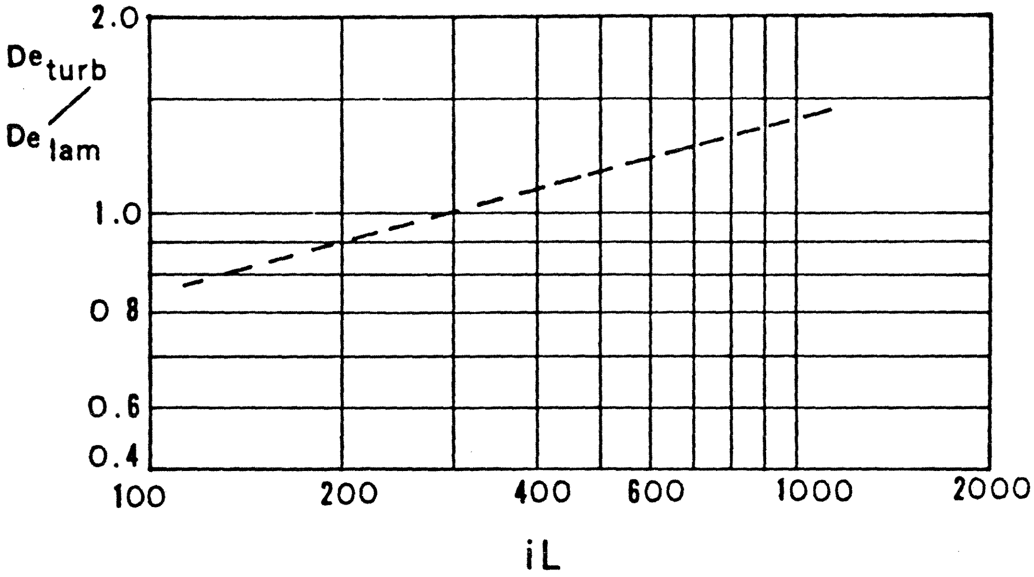


Figure A 5

Subject, of course, to the limitations of the assumptions used, Figure A5 indicates that as the value of iL increases beyond 260, the laminar range equations will tend to underestimate t_e in the turbulent range, as Izzard observed.[16] However, the overall response appears to be relatively insensitive, and either the laminar or the turbulent range equations could be applied for surface detention without appreciable error over a reasonably broad range of values of iL or Reynolds numbers.

The estimates of time to equilibrium in the turbulent range (Equation A26) used by Wolf,[40] appear to be reasonable. In a series of computer runs with finite difference equations that used Manning's n as a resistance coefficient, Morgali[17] found that the time to equilibrium could be estimated as;

$$t_e = \frac{0.993 \quad L^{0.593} \quad n^{0.605}}{i^{0.388} \quad S^{0.38}} \tag{A36}$$

a result that is very similar to Equation A27. A brief discussion of

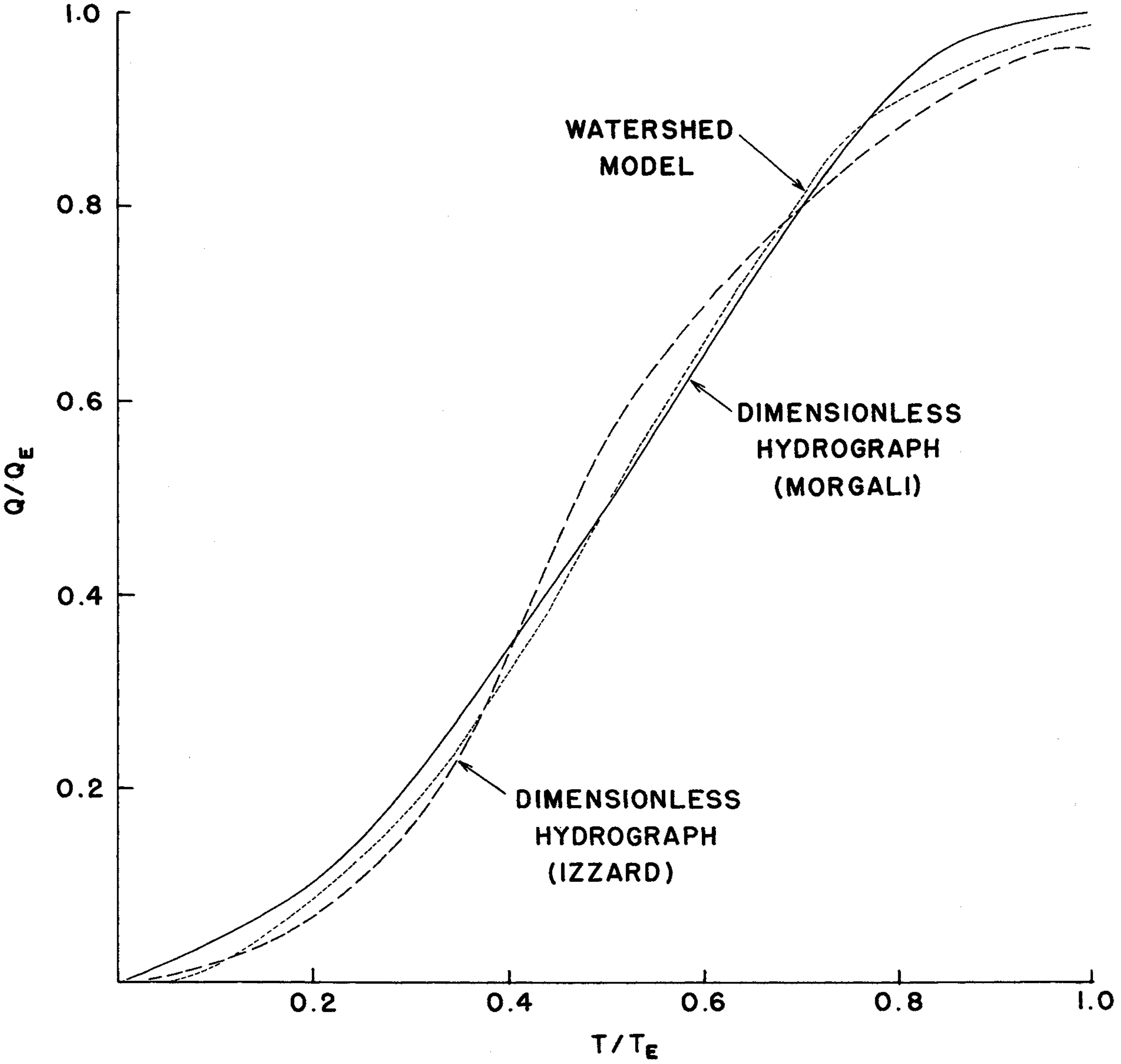

1.0
0.8
0.6
0.4
0.2
Q/Q_E
0.2
0.4
0.6
0.8
1.0
T/T_E
WATERSHED MODEL
DIMENSIONLESS HYDROGRAPH (MORGALI)
DIMENSIONLESS HYDROGRAPH (IZZARD)

Figure A6

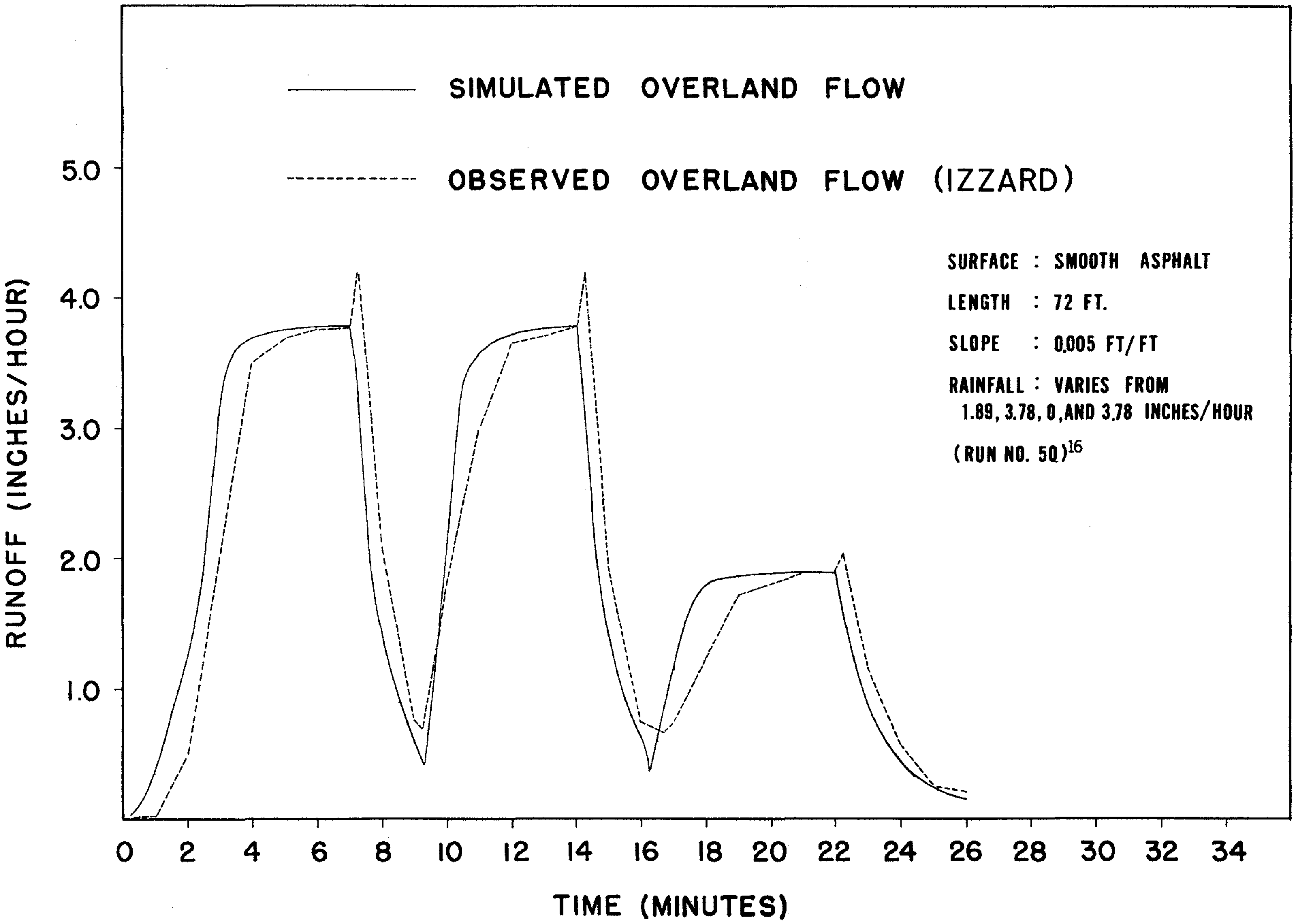

SIMULATED OVERLAND FLOW
OBSERVED OVERLAND FLOW (IZZARD)
SURFACE : SMOOTH ASPHALT
LENGTH : 72 FT.
SLOPE : 0.005 FT/FT
RAINFALL : VARIES FROM
1.89, 3.78, 0, AND 3.78 INCHES/HOUR
(RUN NO. 5Q)16
RUNOFF (INCHES/HOUR)
5.0
4.0
3.0
2.0
1.0
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
TIME (MINUTES)

Figure A7

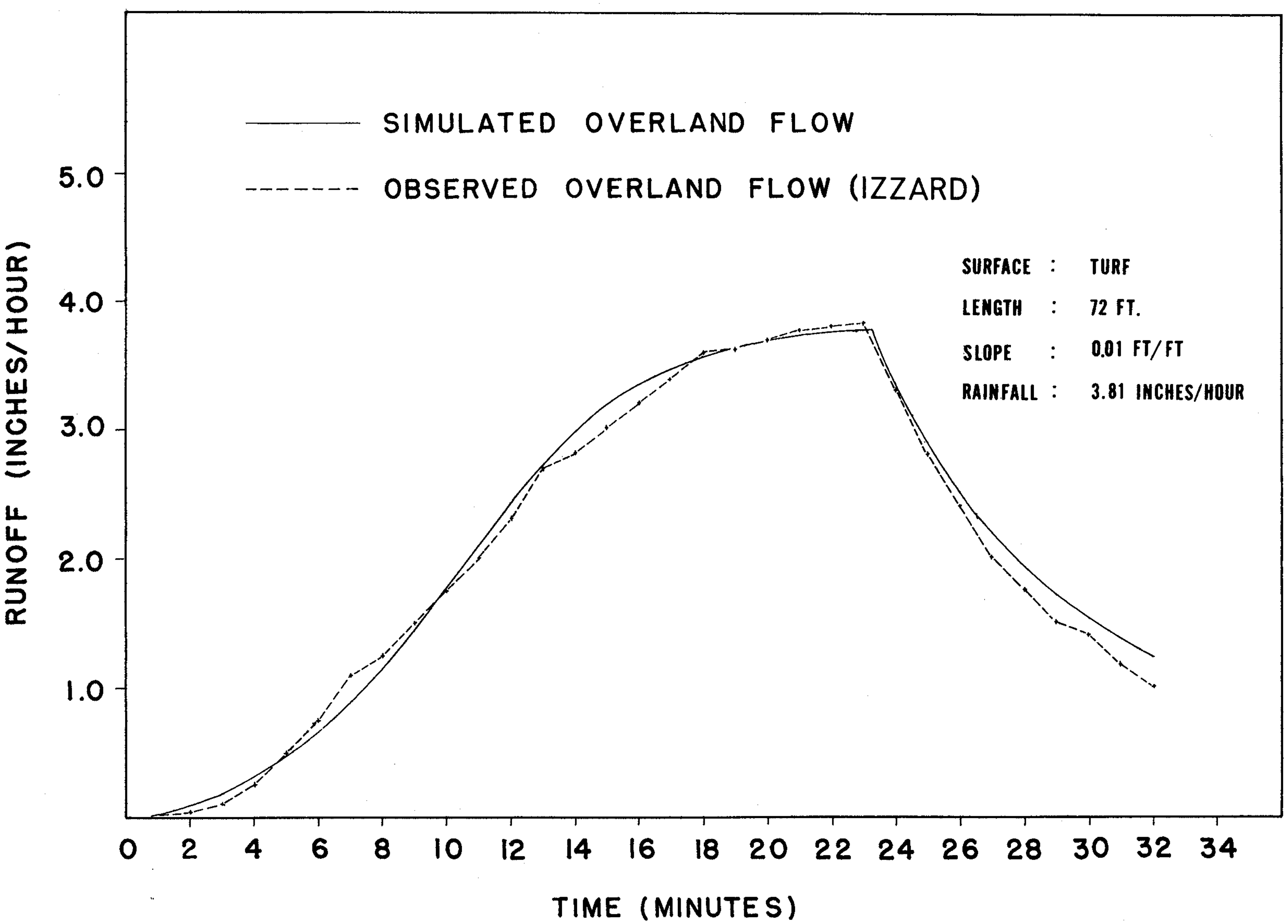

SIMULATED OVERLAND FLOW
OBSERVED OVERLAND FLOW (IZZARD)
SURFACE : TURF
LENGTH : 72 FT.
SLOPE : 0.01 FT/FT
RAINFALL : 3.81 INCHES/HOUR
RUNOFF (INCHES/HOUR)
5.0
4.0
3.0
2.0
1.0
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
TIME (MINUTES)

Figure A8

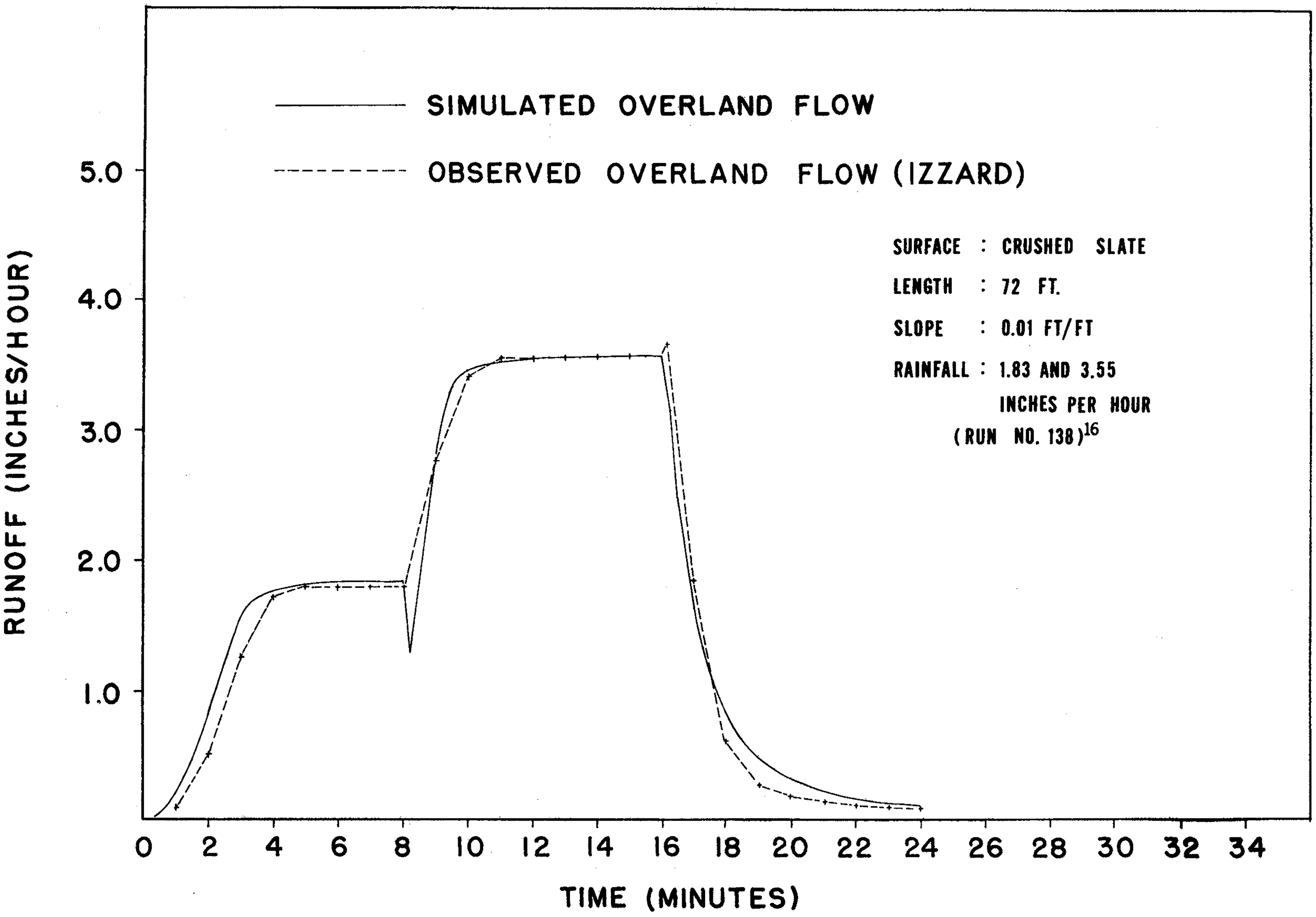
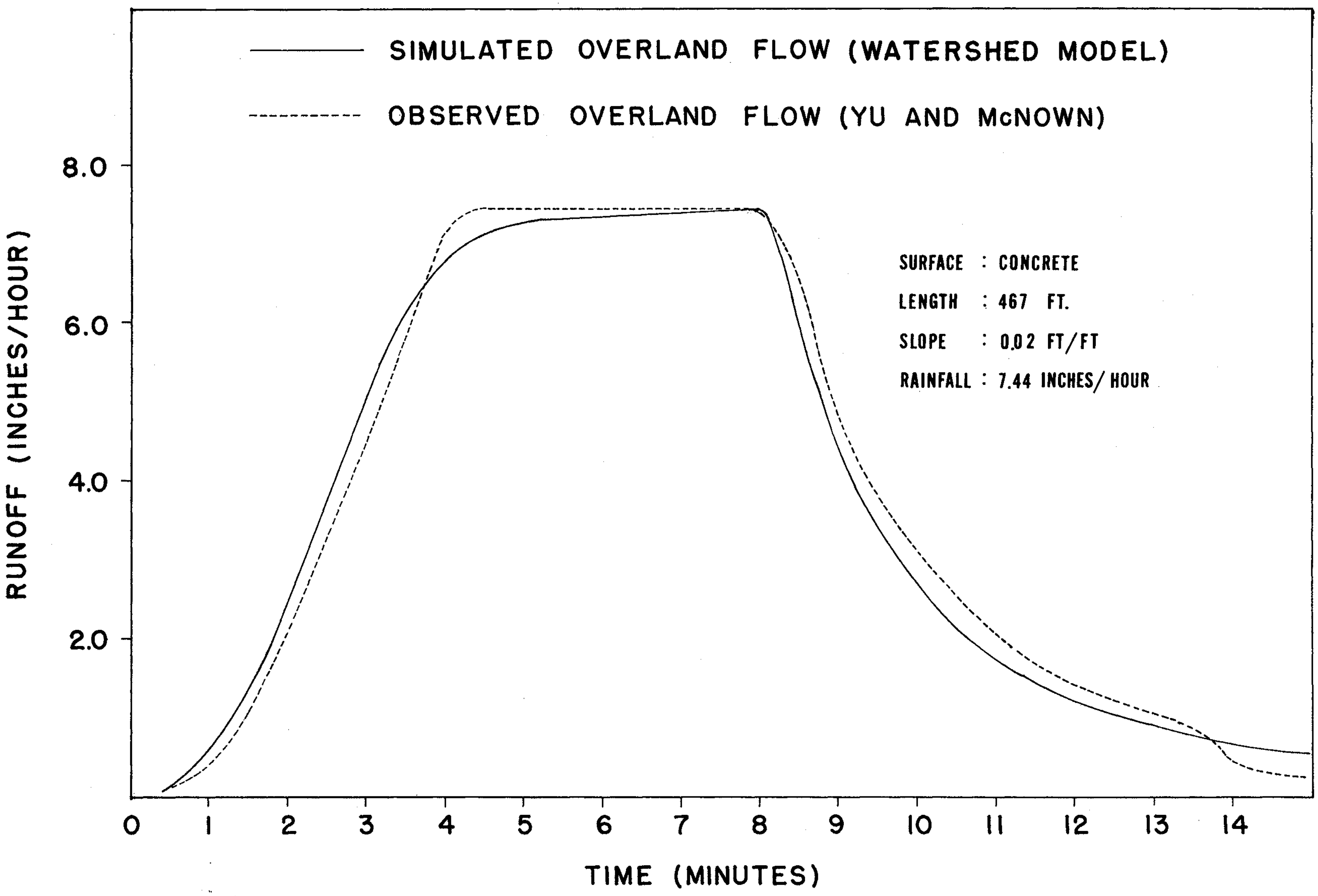

SIMULATED OVERLAND FLOW
OBSERVED OVERLAND FLOW (IZZARD)
SURFACE : CRUSHED SLATE
LENGTH : 72 FT.
SLOPE : 0.01 FT/FT
RAINFALL : 1.83 AND 3.55 INCHES PER HOUR
(RUN NO. 138)16
RUNOFF (INCHES/HOUR)
5.0
4.0
3.0
2.0
1.0
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
TIME (MINUTES)

Figure A9



SIMULATED OVERLAND FLOW (WATERSHED MODEL)
OBSERVED OVERLAND FLOW (YU AND McNOWN)
SURFACE : CONCRETE
LENGTH : 467 FT.
SLOPE : 0.02 FT/FT
RAINFALL : 7.44 INCHES/ HOUR
RUNOFF (INCHES/HOUR)
8.0
6.0
4.0
2.0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
TIME (MINUTES)

Figure A10

Equation A26 is also found in a recent technical report[41] by Grace and Eagleson. Equation A26 is, however, the result of the dimensionless response in Figure A3. Izzard's experimental value of 2.0, instead of 1.6 in Equation A25, may be a better approximation; at least at the lower limit of the turbulent range.

General Model Simulation The simulation of overland flows described in Chapter III required that surface detention be used as a parameter for the rate of discharge. Thus, surface detention based on Manning's equation, Equation A24, and the empirical approximation for discharge in Equation 3.6 determine overland flow response. When equation 3.6 is run with uniform rainfall, the dimensionless response in Figure A6 is the result. Finite difference and experimental dimensionless response are shown for comparison. Equation 3.6 reaches a discharge $q/q_e = 0.97$ when 60 q_e equals $2D_e$, or in effect follows Izzard's definition in Equation A30 rather than Equation A25. This insures compatibility with Izzard's results for values of iL of 260 at the lower limit of the turbulent range. Figures A7, A8, and A9 show typical comparisons between the general model simulation and experimental results for various low to moderate Reynolds numbers. In Figure A10, general model simulation is compared to experimental results obtained by Yu and McKnown[21] at relatively high Reynolds numbers. A slight tendency to underestimate high Reynolds number response was noted, perhaps indicating that Equation A25 becomes a better approximation as Reynolds number increases. The difference is not significant for practical purposes.

The urban drainage simulation in Chapter VII required estimates of time to equilibrium in channels with lateral inflow. Equation A26 was used directly in these studies.

The relationships developed here are particularly tailored for the purposes intended; the rapid simulation of two-dimensional overland flows for a general watershed model, and estimation of time to equilibrium in channels for urban drainage studies. They are not intended to compete with more general or precise solutions for steady-state turbulent two-dimensional flows, or with the more exact finite-difference methods that are available for unsteady two-dimensional flows.[17,18,19] The relationships developed were substituted for the more exact methods to gain simplicity and calculating speed, while attempting to maintain a reasonable approximation to physical behavior. Consistent theoretical resistance parameters for overland flow with rainfall have not yet been developed, and even the division between the laminar and turbulent ranges is difficult to establish.[20,21] Manning's equation was used although there is considerable experimental evidence[18] that Manning's n is not constant with depth, and increases quite rapidly as depth decreases and Reynold's numbers approach the lower limit of the turbulent range. Despite the limitations of these assumptions, particularly the use of constant Manning's n, the relationships used appear to give a satisfactory approximation to physical behavior.

APPENDIX B   SAMPLE OF PRECIPITATION SUBROUTINE INPUT

```
5 13  0  1
5 49 50
51 49  10   .60  19
5  .00  .00  .17 1.30  .61  .17  .00  .00  .00  .00
5  .00  .18  .00  .00  .00  .06  .04  .00  .00  .00
5  .18  .07  .00  .00  .00  .00  .00  .00  .17  .26
5  .19
51 49  11   .60  19
5  .00  .03  .09  .00  .00  .00  .00  .00  .00  .00
5  .00  .01  .55  .00  .00  .00  .11  .16  .00  .00
5  .03  .00  .00  .07  .15  .02  .00  .35  .00  .00
51 49  12   .60  19
5  .00  .00  .00  .15  .00  .14  .06  .00  .00  .17
5 1.21  .92  .23  .00  .00  .00  .00  .72  .00  .00
5  .00  .11  .00  .00  .00  .64  .00  .00  .00  .00
5  .00
                      .  .  .  .  .  .
                      .  .  .  .  .  .
                      .  .  .  .  .  .
                      .  .  .  .  .  .
51 50   8   .60  19
5  .07  .00  .03  .00  .02  .00  .00  .00  .69  .04
5  .00  .00  .00  .00  .23  .00  .00  .15  .00  .00
5  .00  .00  .00  .00  .00  .00  .15 1.09 1.39  .33
5  .23
51 50   9   .60  19
5  .00  .40  .71  .00  .00  .00  .14  .41  .27  .23
5  .31 1.23  .00  .00  .00  .00  .00  .00  .42  .52
5  .13  .45  .00  .00  .00  .00  .00  .00  .00  .00
5   *  OCT 1949 TO SEPT 1950  RAINFALL DATA, BOWMAN FIELD
5     4951 49 10  3 1 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.03 0.06
5     4951 49 10  3 2 0.07 0.04 0.00 0.00 0.00 0.00 0.01 0.01 0.00 0.00 0.00 0.01
5     4951 49 10  4 1 0.01 0.00 0.00 0.00 0.11 0.05 0.02 0.21 0.16 0.23 0.22 0.11
5     4951 49 10  4 2 0.15 0.05 0.01 0.01 0.01 0.01 0.01 0.01 0.01 0.07 0.05 0.09
5     4951 49 10  5 1 0.05 0.01 0.01 0.00 0.00 0.00 0.00 0.00 0.01 0.01 0.00 0.00
5     4951 49 10  5 2 0.00 0.00 0.00 0.04 0.08 0.05 0.00 0.00 0.00 0.00 0.00 0.00
5     4951 49 10  6 1 0.00 0.00 0.03 0.01 0.01 0.04 0.08 0.00 0.00 0.00 0.00 0.00
5     4951 49 10 12 1 0.00 0.00 0.00 0.00 0.04 0.01 0.01 0.01 0.00 0.01 0.00 0.00
5     4951 49 10 16 2 0.00 0.00 0.00 0.00 0.00 0.00 0.08 0.05 0.00 0.00 0.00 0.00
                      .  .  .  .  .  .
                      .  .  .  .  .  .
                      .  .  .  .  .  .
                      .  .  .  .  .  .
5     4951 53  7 21 2 0.00 0.10 0.14 0.00 0.00 0.02 0.00 0.00 0.30 0.05 0.13 0.14
5     4951 53  7 22 1 0.13 0.00 0.00 0.00 0.00 0.03 0.00 0.00 0.00 0.00 0.03 0.00
5     4951 53  7 31 2 0.00 0.00 0.00 0.00 0.48 0.00 0.00 0.00 0.00 0.00 0.00 0.00
5     4951 53  8  4 1 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.01 0.06 0.00
5     4951 53  8  7 2 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.25 0.11
5     4951 53  9  4 2 0.05 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
5     4951 53  9 19 2 0.05 0.07 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
5     4951 98  9 30 2 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00
```

APPENDIX B

SAMPLE OF PRECIPITAION SUBROUTINE OUTPUT

```
TAPE  578      RECORD   1  GAGE
STATION        4951  WATER  YEAR  49 50  MONTH  10  TAPE PRECIP   3.28  RECORDER   3.11  STORAGE GAGE   3.40

STATION        4951  WATER  YEAR  49 50  MONTH  11  TAPE PRECIP   1.24  RECORDER    .75  STORAGE GAGE   1.57

STATION        4951  WATER  YEAR  49 50  MONTH  12  TAPE PRECIP   4.59  RECORDER   4.96  STORAGE GAGE   4.35

STATION        4951  WATER  YEAR  49 50  MONTH   1  TAPE PRECIP  12.03  RECORDER  11.54  STORAGE GAGE  12.35

STATION        4951  WATER  YEAR  49 50  MONTH   2  TAPE PRECIP   5.65  RECORDER   4.74  STORAGE GAGE   6.50

STATION        4951  WATER  YEAR  49 50  MONTH   3  TAPE PRECIP   2.87  RECORDER   2.71  STORAGE GAGE   2.98

STATION        4951  WATER  YEAR  49 50  MONTH   4  TAPE PRECIP   3.33  RECORDER   3.14  STORAGE GAGE   3.45

STATION        4951  WATER  YEAR  49 50  MONTH   5  TAPE PRECIP   8.95  RECORDER   7.61  STORAGE GAGE   9.84

STATION        4951  WATER  YEAR  49 50  MONTH   6  TAPE PRECIP   8.96  RECORDER   8.41  STORAGE GAGE   9.33

STATION        4951  WATER  YEAR  49 50  MONTH   7  TAPE PRECIP   5.62  RECORDER   5.42  STORAGE GAGE   5.75

STATION        4951  WATER  YEAR  49 50  MONTH   8  TAPE PRECIP   3.76  RECORDER   2.77  STORAGE GAGE   4.42

STATION        4951  WATER  YEAR  49 50  MONTH   9  TAPE PRECIP   4.42  RECORDER   3.23  STORAGE GAGE   5.22
```

APPENDIX B

SAMPLE OF WATERSHED MODEL INPUT: ONE FLOWPOINT, ONE SEGMENT, WITHOUT SNOWMELT INPUT

```
5 5 1 0 *NAPA RIVER FILE 0 - 5 TAPE 323
5 10 0 0 1 1 0 1 1 1 0 0  1 1 81.3 5000.0 0 0 200.0       *CONTROL
5 57 58 142300.0  $NAPA RIVER NR ST HELENA, CALIFORNIA                        $
5  0.2 1.0 6.0 0.0                                            *START
5 *EVAPRORATION
5    .124  .086  .079  .055  .050  .027  .032  .026  .035  .032  .061  .072
5    .061  .110  .138  .140  .174  .183  .200  .198  .188  .203  .181  .192
5 0.95 0.95 0.95 0.95 0.95 0.95 0.95 0.95 0.95 0.95 0.95 0.95
5* NAPA RIVER STREAMFLOW
5      3.1    2.8    2.5    2.5    2.6    2.6    2.8    2.8    5.8   50.0   18.0
5      8.1  159.0   52.0   20.0   14.0   10.0    8.6    7.6    7.1    6.6    6.2
                 .  .  .  .  .  .
                 .  .  .  .  .  .
                 .  .  .  .  .  .
                 .  .  .  .  .  .
5      1.0    1.6    1.4     .7     .7    1.1    1.0     .5     .5     .5    1.2
5      1.0     .8     .6     .4     .2     .5    1.1     .8
5      $NEGEV JAN 30/66 CONST C SEG 1 RG ST HELENA SG CALISTOGA               $
5 1 6 0.10 0.15 0.25 0.25 0.15 0.10                             *TIME DELAY
5 0.92 81.3 0.03                                                *CL1
5 0.15 0.8 13.5 0.28 0.0 0.0 0.6 1.5 300.0 0.07 0.4             *CL2
5 0.85 0.6 1.0 0.975 0.0                                        *CL3
```

APPENDIX B

SAMPLE WATERSHED MODEL INPUT: TWO FLOWPOINTS WITH THREE AND SIX SEGMENTS, WITH SNOWMELT INPUT

```
5    10     2    0
5 12 0 0 1 0 0 0 1 0 1 0 0 1      1  3  10.81 5000.0 0 0 1000.0
5 58 59 0.0         $SOUTH YUBA RIVER AT LAKE VAN NORDEN, CALIFORNIA             $
5 0.0 0.0 2.0 0.0
5 0.0 0.0 1.0 0.0
5 0.0 0.0 1.0 0.0
5 0.104 0.084 0.06 0.02 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.044 0.084 0.149 0.189 0.19
5 0.197 0.217 0.249 0.249 0.226 0.206 0.185 0.145
5 0.7 0.7 0.7 0.7 0.7 0.7 0.7 0.7  0.7 0.7 0.7 0.7
5        * ESTIMATE SNOW EVAPORATION
5 0.0 0.002 0.011 0.016 0.014 0.012 0.011 0.009 0.008 0.008 0.009 0.012
5 0.015 0.019 0.011 0.004 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0
5     * MAXIMUM SHORT WAVE RADIATION
5 480. 400. 360. 310. 260. 240. 270. 310. 360. 420. 500. 580.
5 630. 700. 750. 800. 820. 820. 810. 800. 750. 700. 610. 570.
5    * MAX AND MIN TEMPERATURE
5 74. 50. 73. 42. 74. 38. 74. 38. 70. 34. 67. 30.
5 78. 34. 72. 42. 68. 38. 70. 34. 66. 32. 37. 34. 68. 36.
5 67. 35. 70. 37. 70. 37. 66. 38. 50. 37. 41. 27. 47. 25.
                      .   .   .   .   .   .
                      .   .   .   .   .   .
                      .   .   .   .   .   .
                      .   .   .   .   .   .
5 44. 28. 52. 30. 48. 28. 50. 34. 45. 37. 45. 35. 52. 32.
5 60. 32. 60. 32. 59. 32. 63. 34. 64. 36. 61. 32. 62. 27.
5 56. 28. 54. 27. 48. 30.
5     $CRAWFORD JUN 27/66   CONST D  SEG 1  6800 7500                        $
5     1  3  0.3 0.5 0.2
5  1.03   8.07  0.1
50.1 0.5 11.5 0.3 0.0 0.0 0.6 1.4 400.0 0.1 0.4
5 0.8 0.6 0.5 0.96 0.0
5 0.65 0.0012 1.01 0.4 0.1 0.4 0.025 0.04 12.0 2
5     $CRAWFORD JUN 27/66 CONST D SEG 2  7500 8000                           $
5 1  4     0.2 0.3 0.3 0.2
5  1.09 2.35 0.1
50.07 0.45 10.5 0.3 0.0 0.0 0.55 1.4 300.0 0.15 0.35
5 0.82 0.5 0.5 0.96 0.0
5 0.65 0.0012 1.01 0.9 0.12 0.3 0.02 0.04 12.0 -1
5     $CRAWFORD JUN 27/66 CONST D SEG 3  8000 9000                           $
5  1  4 0.1 0.2 0.4 0.3
5 1.13 0.39 0.15
50.05 0.4 8.5 0.3 0.0 0.0 0.5 1.4 300.0 0.2 0.3
5 0.82 0.4 0.5 0.96 0.0
5 0.5 0.0016 1.01 1.65 0.14 0.0 0.01 0.04 10.0 -1
5 12 0 0 1 1 0 1 1 1 1 0 0 0 2  6 55.29 5000.0 0 5 1500.0
5  58 59 80030.0  $SOUTH YUBA NR CISCO,  CALIFORNIA 58 59                    $
5 0.0 0.0 3.0 0.0
5 0.0 0.0 3.0 0.0
5 0.0 0.0 2.0 0.0
5 0.0 0.0 2.0 0.0
5 0.0 0.0 1.0 0.0
```

```
5 0.0 0.0 1.0 0.0
5 0.104 0.084 0.06 0.02 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.044 0.084 0.149 0.189 0.19
5 0.197 0,217 0.249 0.249 0.226 0.206 0.185 0.145
5 0.7 0.7 0.7 0,7 0.7 0.7 0.7 0.7  0.7 0.7 0.7 0.7
5        * ESTIMATE SNOW EVAPORATION
5 0.0 0.002 0.011 0.016 0.014 0.012 0.011 0.009 0.008 0.008 0.009 0.012
5 0.015 0.019 0.011 0.004 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0
5     * MAXIMUM SHORT WAVE RADIATION
5 480. 400. 360. 310. 260. 240. 270. 310. 360. 420. 500. 580.
5 630. 700. 750. 800. 820. 820. 810. 800. 750. 700. 610. 570.
5    * MAX AND MIN TEMPERATURE
5 74. 50. 73. 42. 74. 38. 74. 38. 70. 34. 67. 30.
5 78. 34. 72. 42. 66. 38. 70. 34. 66. 32. 37. 34. 68. 36.
5 67. 35. 70. 37. 70. 37. 66. 38. 50. 37. 41. 27. 47. 25.
                  .  .  .  .  .  .
                  .  .  .  .  .  .
                  .  .  .  .  .  .
                  .  .  .  .  .  .
5 60. 32. 60. 32. 59. 32. 63. 34. 64. 36. 61. 32. 62. 27.
5 56. 28. 54. 27. 48. 30.
5      *      STREAMFLOW
521. 21. 21. 20. 20. 20. 19. 19. 19. 19. 18. 19. 19. 18. 17. 17.      *CISCO 1058
516. 14.0 12.0 9.8 8.1 6.6 5.5 4.4 3.7 3.1 2.6 2.2 2.0 1.8 1.8        *CISCO 1058
51.8 2.0 1.8 1.6 1.5 1.5 1.5 1.4 1.4 2.4 2.2 2.1 2.4 11. 8.9 5.2 4.0 *CISCO 1158
                  .  .  .  .  .  .
                  .  .  .  .  .  .
                  .  .  .  .  .  .
                  .  .  .  .  .  .
519. 19. 19. 19. 19. 19. 20. 20. 19. 19. 19. 20. 21. 21. 21. 20. 20. *CISCO 0959
552. 48. 30. 23. 28. 27. 27. 26. 26. 26. 23. 21. 21.                  *CISCO 0959
5      $CRAWFORD JUN 27/66 CONST D  SEG 1 5520 6000                            $
5 2  2 0.5 0.5
5 1.03 3.6 0.05
50.12 0.55 12.0 0.3 0.0 0.0 0.65 1.4 400.0 0.1 0.4
50.85 0.6 0.5 0.97 0.0
5 0.7 0.0012 1.01 -1.1 0.1 0.5 0.025 0.05 13.0 -3
                  .  .  .  .  .  .
                  .  .  .  .  .  .
                  .  .  .  .  .  .
                  .  .  .  .  .  .


5      $CRAWFORD JUN 27/66 CONST D SEG 6  8000 9000                            $
5 2 4 0.0 0.2 0.4 0.4
5 1.13 0.7 0.15
50.05 0.4 8.5 0.3 0.0 0.0 0.5 1.4 300.0 0.2 0.3
50.88 0.4 0.5 0.96 0.0
5 0.5 0.0016 1.01 1.65 0.14 0.0 0.01 0.04 10.0 -1
```

APPENDIX B SAMPLE OF BASIC WATERSHED MODEL OUTPUT

```
          NEGEV JAN 30/66 CONST C SEG 1 RG ST HELENA SG CALISTOGA
NAPA RIVER NR ST HELENA, CALIFORNIA                            WATER YEAR  1959-60        STANFORD WATERSHED MODEL  IV

        DAY      OCT     NOV     DEC     JAN     FEB     MAR     APR     MAY     JUN     JUL     AUG     SEPT         ANNUAL

TOTAL           .017    .007    .067    .286    6.51    2.10    .474    .224    .090    .039    .018    .008      9.85  INCHES
INTERFLOW       .000    .000    .000    .036    2.47    .869    .014    .001    .000    .000    .000    .000      3.39  INCHES
BASE            .017    .007    .008    .077    .761    .968    .429    .202    .088    .038    .016    .007      2.61  INCHES
PRECIP           .03     .05    2.27    6.79   11.61    6.40    1.83    1.15     .00     .00     .00     .01     30.14  INCHES

EVP/TRAN-NET    1.08    .788    .823    1.00    1.19    2.27    3.21    3.53    2.72    2.06    1.49    1.06      21.2  INCHES
  -POTENTIAL    3.79    2.05    1.42    1.01    1.20    2.31    3.50    4.68    5.81    6.47    5.98    5.45      43.7  INCHES

STORAGES-UZS    .000    .000    .134    1.12    .859    1.26    .289    .000    .000    .000    .000    .000              INCHES
         LZS    2.84    2.08    3.35    7.68    11.7    13.3    12.7    10.5    7.71    5.58    4.04    2.95              INCHES
         SGW    .014    .006    .022    .212    .555    .632    .313    .156    .067    .030    .013    .006              INCHES

INDICES- GWS    .009    .004    .019    .189    .385    .544    .307    .154    .062    .024    .009    .004
BALANCE          .0066  INCHES
          TWENTY HIGHEST CLOCKHOUR RAINFALL EVENTS IN THE WATER YEAR

       .589  .424  .393  .346  .338  .330  .322  .298  .283  .275  .275  .275  .270  .266  .259  .251  .243  .243  .243  .236

          TWENTY HIGHEST CLOCKHOUR OVERLAND FLOW RUNOFF EVENTS IN THE WATER YEAR

       .399  .335  .271  .189  .189  .178  .173  .167  .160  .152  .142  .138  .114  .111  .094  .083  .081  .076  .075  .072
```

```
NAPA RIVER NR ST HELENA, CALIFORNIA                                    WATER YEAR  1959-60        STANFORD WATERSHED MODEL  IV
OCTOBER

NOVEMBER

DECEMBER

JANUARY

FEBRUARY

   1  AM      54.1     53.5     50.7     46.0     42.0     40.9        58.4     71.5     76.1    105.0    139.7    194.1
      PM     241.6    293.8    331.4    396.3    393.4    370.6       374.6    361.7    344.3    331.8    316.6    301.6     207.9

   2  AM     293.9    303.6    291.8    281.2    271.6    262.7       254.6    247.2    240.3    233.8    227.8    222.1
      PM     216.8    211.7    206.9    202.3    198.0    193.8       189.8    185.9    182.1    178.5    175.0    171.6     226.8

   3  AM     168.3    165.1    162.0    158.9    156.0    153.1       150.3    147.5    144.8    142.2    139.7    142.7
      PM     170.6    185.2    209.6    230.0    243.9    270.7       263.6    290.0    292.9    281.3    270.9    261.4     200.0

   4  AM     252.8    244.9    237.6    230.9    224.7    218.9       213.5    208.4    203.6    199.0    194.6    190.5
      PM     186.5    182.7    179.1    175.6    172.2    168.9       165.7    162.6    159.6    156.7    173.7    247.6     197.9

   5  AM     378.0    520.6    638.6    679.2    708.1    664.7       610.6    558.9    514.2    475.5    443.8    415.3
      PM     389.6    367.0    347.5    329.8    314.2    300.3       287.9    277.9    268.9    259.8    251.3    243.5     426.9

   6  AM     236.3    229.7    223.6    217.8    212.5    207.5       202.8    198.3    194.1    190.0    186.2    182.5
      PM     178.9    175.5    172.2    169.0    165.9    162.9       160.0    157.2    154.5    151.8    149.2    146.7     184.4

   7  AM     144.2    141.7    139.4    141.5    146.3    181.2       177.4    181.1    176.9    173.0    177.5    223.9
      PM     347.4    543.0    867.3   1277.8   1695.5   2098.1      2551.2   3177.1   3903.8   4619.1   5111.4   5299.3    1395.6

   8  AM    5365.6   5446.0   5672.0   6198.5   6934.5   7848.2      8783.9   9429.3   9707.3   9535.8   9006.1   8257.4
      PM    7398.4   6540.8   5773.2   5128.5   4581.6   4103.2      3727.3   3444.4   3265.8   3151.8   3026.1   2874.3    6050.0

   9  AM    2683.0   2484.7   2310.7   2176.0   2077.5   2011.8      1958.2   1898.6   1832.7   1757.7   1685.0   1617.6
      PM    1555.6   1499.0   1447.5   1400.4   1357.0   1316.8      1280.0   1248.2   1241.2   1256.7   1295.7   1337.8    1697.1

  10  AM    1353.1   1344.6   1311.0   1270.4   1232.5   1198.0      1165.8   1137.0   1110.1   1083.9   1058.8   1033.8
      PM    1010.0    987.3    965.6    944.7    924.7    905.4       886.7    868.7    851.3    834.3    817.8    801.8    1045.7

  11  AM     786.1    770.9    756.0    741.5    727.4    713.7       700.2    687.1    674.3    661.8    649.6    637.7
      PM     626.0    614.6    595.7    566.7    521.6    540.7       543.6    536.8    529.8    522.5    515.1    507.6     630.3

  12  AM     499.9    492.1    484.4    476.7    469.0    461.4       453.9    446.5    439.2    432.0    425.0    418.0
      PM     411.1    404.4    397.8    391.3    384.9    378.6       372.5    366.5    360.6    354.8    349.1    343.6     417.2

  13  AM     338.0    332.5    327.2    321.9    316.8    311.9       307.0    302.2    297.5    293.0    288.5    284.1
      PM     279.9    275.7    271.6    267.6    263.6    259.8       256.0    252.3    248.7    245.2    241.7    238.3     284.2

  14  AM     234.9    231.5    228.2    225.0    221.8    218.8       215.8    212.9    210.0    207.2    204.5    201.8
      PM     199.2    196.7    194.2    191.7    189.3    187.0       184.7    182.4    180.2    178.1    175.9    173.9     201.9

MARCH

   5  AM     136.4    128.7    123.7    128.2    139.7    154.2       174.3    197.6    235.9    224.6    258.5    276.0
      PM     310.4    390.5    404.7    422.1    429.4    411.1       434.3    435.8    428.7    417.4    407.0    397.3     294.4

   6  AM     388.0    380.0    373.2    365.2    357.6    351.8       346.3    342.4    338.5    346.6    377.9    425.1

      PM     416.3    408.4    389.1    413.3    420.3    417.0       413.1    408.8    404.2    399.3    394.3    389.1     386.1

   7  AM     383.6    378.1    372.6    367.0    370.2    403.8       500.6    583.3    717.0    857.8    956.8    996.5
      PM     979.1    924.4    851.0    764.1    747.2    719.5       686.7    657.6    631.5    608.1    587.0    567.9     650.5

   8  AM     550.3    534.2    519.4    505.8    493.1    481.2       470.2    459.7    449.9    440.6    431.7    423.2
      PM     415.1    407.4    399.9    392.7    385.7    379.0       372.5    366.2    360.1    354.1    348.3    342.7     428.5

   9  AM     337.0    331.5    326.1    320.9    315.8    310.9       306.0    301.3    296.7    292.2    287.9    283.6
      PM     279.4    275.3    271.3    267.3    263.5    259.8       256.1    252.5    249.0    245.5    242.1    238.8     283.8

  10  AM     235.4    232.1    228.9    225.8    222.7    219.7       216.8    213.9    211.1    208.4    205.8    203.2
      PM     200.6    198.1    195.7    193.3    190.9    188.7       186.4    184.2    182.1    179.9    177.9    175.9     203.2

  12  AM     190.7    196.9    200.6    204.1    205.2    206.1       204.7    203.3    199.7    196.4    193.2    190.3
      PM     198.6    210.4    229.5    220.4    248.1    266.5       297.7    293.7    286.8    280.4    274.5    269.0     227.8

  13  AM     263.7    258.7    254.1    249.7    245.6    241.6       237.9    234.3    230.9    227.6    224.4    221.4
      PM     218.4    215.6    212.8    210.1    207.5    204.9       202.4    200.0    197.7    195.4    193.1    190.9     222.5

APRIL

MAY

JUNE

JULY

AUGUST

SEPTEMBER
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APPENDIX B   OUTPUT (continued)

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NAPA RIVER NR ST HELENA, CALIFORNIA                            WATER YEAR  1959-60       STANFORD WATERSHED MODEL  IV
       DAY       OCT     NOV     DEC     JAN     FEB     MAR     APR     MAY     JUN     JUL     AUG    SEPT        ANNUAL

        1        1.8      .8      .4     1.2   207.9    42.6    66.1    23.0    10.0     4.1     1.8      .9
        2        1.8      .7      .3     1.2   226.8    40.8    58.8    21.9     9.7     4.0     1.7      .8
        3        1.7      .7      .3     1.2   200.0    39.4    53.5    21.0     9.4     3.8     1.7      .8
        4        1.7      .7      .3     1.1   197.9    97.6    49.5    20.1     9.1     3.7     1.7      .8
        5        1.7      .7      .3     1.1   426.9   294.4    46.3    19.3     8.8     3.6     1.6      .8

        6        1.8      .7      .3     1.1   184.4   386.1    43.6    18.5     8.5     3.5     1.6      .8
        7        1.6      .7      .3    13.5  1395.6   650.5    41.2    17.9     8.3     3.4     1.5      .7
        8        1.5      .6      .3    39.1  6050.0   428.5    39.2    17.2     8.0     3.4     1.5      .7
        9        1.4      .6      .3    18.0  1697.1   283.8    37.3    16.6     7.8     3.3     1.5      .7
       10        1.4      .6      .3    19.4  1045.7   203.2    35.6    15.9     7.5     3.2     1.4      .7

       11        1.3      .6      .3    61.3   630.3   160.8    34.0    15.4     7.3     3.1     1.4      .7
       12        1.3      .6      .3    14.9   417.2   227.8    32.5    14.8     7.1     3.0     1.4      .7
       13        1.2      .6      .3     4.7   284.2   222.5    31.1    14.3     6.9     2.9     1.3      .7
       14        1.2      .5      .3    17.9   201.9   167.1    29.8    13.8     6.7     2.9     1.3      .6
       15        1.2      .5      .2     9.3   151.3   134.0    28.5    13.3     6.5     2.8     1.3      .6

       16        1.2      .5      .2     4.7   119.8   112.7    27.3    12.8     6.3     2.7     1.2      .6
       17        1.1      .5      .2     4.4    99.7    98.3    26.2    12.4     6.1     2.6     1.2      .6
       18        1.1      .5      .2     4.2    86.5    88.1    25.2    12.0     5.9     2.6     1.2      .6
       19        1.1      .5      .2     4.0    77.4    80.5    24.2    11.6     5.7     2.5     1.2      .6
       20        1.0      .5      .2     3.9    70.9    74.5    23.2    11.2     5.6     2.4     1.1      .6

       21        1.0      .5      .2    36.9    66.0    69.6    22.3    10.8     5.4     2.4     1.1      .6
       22        1.0      .4      .2    42.5    62.2    65.4    25.1    10.4     5.3     2.3     1.1      .5
       23        1.0      .4    69.4    13.6    58.9    61.8    28.4    51.7     5.1     2.3     1.1      .5
       24         .9      .4    55.4    33.4    55.9    58.5    24.5    17.3     5.0     2.2     1.0      .5
       25         .9      .4     6.9    71.8    53.3    55.5    21.6    13.0     4.8     2.1     1.0      .5

       26         .9      .4     1.5    63.0    50.9    52.8    32.8    12.4     4.7     2.1     1.0      .5
       27         .9      .4     1.4    39.2    48.6    69.6    67.2    11.9     4.5     2.0     1.0      .4
       28         .8      .4     1.4    26.7    46.4    78.1    29.0    11.5     4.4     2.0      .9      .4
       29         .8      .4     1.3    19.9    44.5    58.1    25.6    11.1     4.3     1.9      .9      .4
       30         .8      .4     1.3    16.9           105.5    24.2    10.7     4.2     1.9      .9      .4
       31         .8             1.3    17.5            78.7            10.4             1.8      .9

SYNTHESIS        38.     16.    146.    608.  14258.   4587.   1054.    494.    199.     87.     40.     19.    21543.  CFSD
                .017    .007    .067    .278    6.52    2.09    .482    .226    .091    .040    .018    .009       9.9  INCHES
                                                                                                                 42656.  ACFT

RECORDED         18.     28.     65.    877.  13232.   5778.   1129.    409.     89.     20.      6.     14.    21665.  CFSD
                .008    .013    .030    .401    6.05    2.64    .516    .187    .041    .009    .003    .006       9.9  INCHES
                                                                                            (   42980.  )        42896.  ACFT
```

```
DAILY FLOW DURATION AND ERROR TABLE
FLOW INTERVAL      CASES   AV.ERROR   AVR. ABS. ERROR   STANDARD ERROR

          .0-      123.0        1.0          1.07             .84
         1.0-       51.0         .0          1.13            1.52
         1.6-       28.0         .5          2.12            2.98
         2.7-       10.0        6.3          6.65           10.17
         4.5-       19.0        6.6          7.06           14.45
         7.4-       17.0        8.1          8.92           14.55
        12.2-       16.0         .5           .92            1.15
        20.1-       21.0         .2          2.75            4.07
        33.1-       22.0        -.2         10.54           13.38
        54.6-       25.0       -4.7         10.96           14.45
        90.0-        7.0        6.4         23.89           29.37
       148.4-       11.0      -14.7         44.58           59.38
       244.7-        6.0       53.2         96.88          138.35
       403.4-        5.0     -185.8        185.85          145.62
       665.1-        2.0       -6.4        294.12          415.95
      1096.6-        2.0       16.3         29.28           41.41
      1808.0-         .0
      2981.0-         .0
      4914.8-        1.0      230.0
      8103.1-         .0
     13359.7-         .0
     22026.5-         .0
     36315.5-         .0
     59874.1-         .0
     98715.8-         .0
                   366.0     -121.3       4224.32          950.77

CORRELATION COEFFICIENT (DAILY)        .9928

 .01      .00     2.96      .00  FLOW      .40  TEMP      .00
```

APPENDIX C

STANFORD WATERSHED MODEL IV

```
2COMMENT STANFORD WATERSHED MODEL IV, COMBINED VERSION OF JULY 12,1966$
2COMMENT TAPES PRECIP ON LOGICAL 10 - UNIT B5$
2COMMENT READS PRECIP FROM LOGICAL TAPE 10- UNIT B5 $
2LIBRARY PROCEDURE PLOT($X,Y,IC)$
2LIBRARY PROCEDURE SCALE($A(),N,SMAX,SACT,XMIN,DX)$
2LIBRARY PROCEDURE AXIS($X,Y,BCD,N,S,THETA ,XMIN,DX)$
2LIBRARY PROCEDURE PLOTWRITE(X,Y,H,THETA$$LIST,FMT)$
2GLOBAL LIBRARY SUBROUTINE PLOT10$
2BOOLEAN DCS(),TAB,TABU,TAPES,RUN$
2INTEGER DD...,DPY,DLY,DAYMO,DAYS,DAY,HOUR,HARP,HAAP,MO,M1,NX...,
2QQQ...,YR,STYR,FPYR,SEG,FLOWPOINT,SHFT,RINT,DPM(),TAPEMOVE, Z,
2MFILX,ST,CN,MOCOUNT,SG,STT,FILX,OBS,I1,I2,YRS,
2IC,BCD,N,LABLE$
2ARRAY  C(99),C22(110),CAS(25),           AVER(25),AVAR(25),
2EE(24),
2DCS(14),DR(367),E(375),        EVCR(12),
2FLO(367),GWSA(12),
2MXRO(21),MXRA(21),
2QQQ(12),QQL(12),QQO(12),
2REC2(750),K(367),PREC(367),SUMPREC(12),SUMPRECR(12),
2          LZSA(12),REC1(768),SERR(25),
2SERA(25),SQER(25),       SPRA(12),SPRMA(12),SGWFA(12),SGWA(12),
2SQ(25),
2SDIV(367),SGWS(5,10),UZSS(5,10),LZSS(5,10),GWSS(5,10),
2SPETA(12),SAETA(12),SINTA(12),
2TRS(8884),TRSH(5,200),TR(24),TONE(12),TOND(12),TONN(12),
2INFA(12),UZSA(12)$
2COMMENT SNOWMELT IV ARRAYS$
2ARRAY                       MAXRAD(367),EES(24),SE(367),RAD(367),T(735),
2       RAS(24),      RNMM(12),MSURE(12),RADMES(12),SCOMELTS(12)$
2ARRAY
2RADDIST(24)=(0.0,0.0,0.0,0.0,0.0,0.0,0.019,0.041,0.067,0.088,
2       0.102,0.11,0.11,0.11,0.105,0.095,0.081,0.055,0.017,
2     0.0,0.0,0.0,0.0,0.0)$
2ARRAY
2LAPSE(24)=(0.6,0.0,0.0,0.0,0.0,0.0,1.0,2.0,3.0,4.0,4.0,4.0,4.0,4.0,
2       4.0,4.0,4.0,4.0,4.0,4.0,3.2,2.6,2.0,1.2)$
2ARRAY
2GRAD(24)=(0.04,0.04,0.03,0.02,0.02,0.02,0.02,0.06,0.14,0.18,0.20,
2   0.17,0.13,0.06,0.03,0.01,0.05,0.07,0.10,0.13,0.15,0.13,0.12,0.08)$
2ARRAY DPM(12)=(31,28,31,30,31,30,31,31,30,31,30,31)$
2ARRAY DDPM(12)=(31,30,31,31,28,31,30,31,30,31,31,30)$
2ARRAY HARP(12)=(304,334,  0,31,59,90,120,151,181,212,243,273)$
2ARRAY HAAP(12)=(0,31,59,90,120,151,181,212,243,273,304,334)$
2ARRAY LABLE(3)=("DISCHARGE C.F.S.D.")$
2READ ($$COMBO)$
2INPUT COMBO(TAPES,RUN)$
2IF TAPES$
2BEGIN
2COMMENT SECTION READS STORAGE AND HOURLY PRECIPITATION DATA AND
```

```
2OUTPUTS WEIGHTED HOURLY RAINFALL ON TAPE 85$
2COMMENT OBS ON 24 HOUR DAY -  MIDNIGHT IS 24$
2COMMENT REWIND TAPES
2REWIND(10)$
2DDRECR=1$
2P1SUM=0.0$
2STT=MO=0$
2READ ($$STATYR)$
2INPUT STATYR (YRS,FILX,SG)$
2IF FILX GTR 0$ MOVEFILE(10,FILX)$
2FOR DDD=(1,1,YRS)$
2   BEGIN
2   READ ($$WATYR)$
2   INPUT WATYR (I1,I2)$
2    FOR DD92=(1,1,12)$ (SUMPREC(DD92)=0$ SUMPRECR(DD92)=0$)$
2   YR=0$
2   MO=0$
2   DPY=365$ DDPM(5)=28$
2 DPM(2)=28$
2   IF MOD (I2,4) EQL 0$
2      BEGIN
2      DPY=366$
2    DPM(2)=29$
2     DDPM(5)=29$
2      END$
2   FOR DD10=(1,1,367)$PREC(DD10)=0.0$
2   IF SG GTR 0$ UNTIL MO EQL 9$
2      BEGIN
2      READ($$RR)$
2      INPUT RR(STT,YR,MO,WSG,OBS,FOR DD14=(HAAP(MO)+1,1,HAAP(MO)+
2       DPM(MO))$ PREC(DD14))$
2    FOR DD15 = ( HAAP(MO) + 1, 1, HAAP(MO)+ DPM(MO))$
2    SUMPREC(MO) = SUMPREC(MO) + PREC(DD15)$
2      IF MO EQL 2$ IF  DPM(2) EQL 29$
2         BEGIN
2         PREC(366)=PREC(60)$
2         PREC(60)=0.0$
2         END$
2      END$
2   FOR DD85=(1,1,8884)$ TRS(DD85)=0.0$
2   UNTIL YR GEQ 98$
2      BEGIN
2      READ($$DATA)$
2    FOR DD93 = (1 + 12.(CN-1),1,12+12.(CN-1))$
2     SUMPRECR(MO) = SUMPRECR(MO) + TRS(24.(HAAP(MO)+DAY-1)+DD93)$
2      IF MO EQL 2$ IF DAY EQL 29$
2      FOR NX91=(1+12.(CN-1),1,12+12.(CN-1))$
2         BEGIN
2         TRS(24.(366-1)+NX91)=TRS(24.(60-1)+NX91)$
2         TRS(24.(60-1)+NX91)=0.0$
2         END$
2      END$
2   INPUT DATA(ST,YR,MO,DAY,CN,FOR HOUR=(1+12.(CN-1),1,12+12.(CN-1))$
2   TRS(24.(HAAP(MO)+DAY-1)+HOUR))$
2   COMMENT STORAGE GAGE ADJUSTMENT$
```

```
2   IF SG GTR 0$
2      BEGIN
2      DDL=273$ KK=1.0$
2      FOR DDY=(274,1,365),(1,1,59),(366,1,DPY),(60,1,273)$
2         BEGIN
2         FOR NXH=(1,1,24)$
2            BEGIN
2            P1SUM=P1SUM+TRS(24.(DDY-1)+NXH)$
2            IF NXH EQL OBS$
2            EITHER IF P1SUM GTR 0.0$
2               BEGIN
2               K(DDL)=(PREC(DDY).WSG+P1SUM.(1.0-WSG))/P1SUM$
2               P1SUM=0.0$
2               END$
2            OTHERWISE$
2
2               BEGIN
2               EITHER IF OBS NEQ 1$
2               TRS(24.(DDY-1)+OBS)=TRS(24.(DDY-1)+OBS-1)=
2               0.5.WSG.PREC(DDY)$
2               OTHERWISE$
2               TRS(24.(DDY-1)+OBS)=WSG.PREC(DDY)$
2               K(DDL)=1.0$
2               END$
2            END$
2         DDL=DDY$
2         END$
2      KK=K(273)$
2      FOR DDQ=(274,1,365),(1,1,59),(366,1,DPY),(60,1,273)$
2      FOR DDHR=(1,1,24)$
2         BEGIN
2         TRS(24.(DDQ-1)+DDHR)=KK.TRS(24.(DDQ-1)+DDHR)$
2         IF DDHR EQL OBS$(KK=K(DDQ)$ IF DDQ EQL 273$ KK=1.0$)$
2         END$
2      END$
2   MOCOUNT=1$ DDCOUNT=1$ NX=1$  RECSUM=0.0$
2   FOR DDA=(274,1,365),(1,1,59),(366,1,DPY),(60,1,273)$
2      BEGIN
2      FOR DDR=(1,1,24)$
2         BEGIN
2         REC1(NX)=TRS(24.(DDA-1)+DDR)$
2         RECSUM=RECSUM+REC1(NX )$
2         NX=NX+1$
2         END$
2      DDCOUNT=DDCOUNT+1$
2      IF DDCOUNT EQL DDPM(MOCOUNT)+1$
2         BEGIN
2         NX=NX-1$
2         WAIT.. UNTIL CHECKM(10) NEQ 0$ GO TO WAIT$
2         FOR NX5=(1,1,NX)$
2         REC2(NX5)=REC1(NX5)$
2         WRITEM (10,NX$REC2(1))$
2         EITHER IF MOCOUNT LEQ 3$ DDMOC=MOCOUNT+9$
2         OTHERWISE$DDMOC=MOCOUNT-3$
2         IF DDMOC EQL 10$
```

```
2               BEGIN
2               WRITE($$TIIL,TITLF)$
2               OUTPUT TITL(DDRECR)$
2               FORMAT TITLF(*TAPE*,B12,*RECORD*,B1,I3,B2,*GAGE*,W3)$
2               DDRECR=DDRECR+1$
2               END$
2            WRITE($$ANS,ANSF)$
2            OUTPUT ANS(ST,I1,I2,DDMOC,RECSUM,SUMPRECR(DDMOC),
2      SUMPREC(DDMOC))$
2            FORMAT ANSF (*STATION*,B2,I8,B2,*WATER  YEAR*,B2,I2,B1,I2,B2,
2            *MONTH*,B2,I2,B2,*TAPE PRECIP*,X8.2,B2,*RECORDER*,X8.2, B2,
2            *STORAGE GAGE*,X8.2,W2)$
2            RECSUM=0.0$
2            NX=1$
2            DDCOUNT=1$
2            MOCOUNT=MOCOUNT+1$
2            END$
2         END$
2    END$
2    RZ..UNTIL CHECKM(10) NEQ 0$ GO TO RZ$
2    ENDFILE (10)$
2 IF RUN$ REWIND(10)$
2END$
2IF NOT RUN$   GO TO LFIN$
2ETL=1.0$ YR=0$ NXSEG=0$
2FOR DD28=(1,1,8884)$TRS(DD28)=0.0$
2READ($$RUNDAT)$
2INPUT RUNDAT (FPYR,NXTSEG,MFILX)$
2REWIND(10)$
2    WAI5..UNTIL CHECKM(10) NEQ 0$ GO TO WAI5$
2IF MFILX GTR 0$ MOVEFILE (10,MFILX)$
2LINY..
2YR=YR+1$
2NXSEG=NXSEG+1$
2IF YR GTR FPYR$ GO TO LFIN$
2FOR DD46=(1,1,25)$  CAS(DD46)=SERR(DD46)=SERA(DD46)=SQER(DD46)=
2AVER(DD46)=AVAR(DD46)=0$
2FOR DD107=(1,1,367)$SDIV(DD107)=DR(DD107)=0.0$
2READ ($$CONTROL)$
2INPUT CONTROL(DD2,FOR DD1=(1,1,DD2)$DCS(DD1),FLOWPOINT,
2SEG,TAREA,MAXCFS,TAPEMOVE,SHFT,MINH)$
2TCFSD=26.9.TAREA$
2    WAI6..UNTIL CHECKM(10) NEQ 0$ GO TO WAI6$
2IF ABS(TAPEMOVE) GTR 0$
2MOVEM(10,12.TAPEMOVE)$
2    READ ($$NEWY)$
2    INPUT NEWY (DDYR1,DDYR2,YEAR,FOR DD44=(1,1,10)$QQO(DD44))$
2DPY=DLY=365$         EXD=0.0$              DPM(2)=28$
2IF MOD (DDYR1,4) EQL 0$DLY=366$
2IF MOD (DDYR2,4) EQL 0$ (DPY=366$ DPM(2)=29$ EXD=1.0$)$
2COMMENT CARRYOVER FLOWS
2COMMENT BASIC TIME SHIFTS
2EITHER IF SHFT GTR 0$
2FOR DD4=(24.DPY,-1,1)$TRS(DD4+SHFT)=TRS(DD4)$
2OTHERWISE$FOR DD29=(1,1,8884)$TRS(DD29)=0.0$
```

```
2FOR DD3=(1,1,200)$TRS(DD3)=TRS(DD3)+TRSH(FLOWPOINT,DD3)$
2
2IF NXSEG LEQ NXTSEG$
2READ ($$START)$
2INPUT START (    FOR DD9=(1,1,SEG)$(SGWS(FLOWPOINT,DD9),UZSS(FLOWPOINT,
2DD9),LZSS(FLOWPOINT,DD9),GWSS(FLOWPOINT,DD9)))$
2   EITHER IF DCS(3)$
2      BEGIN
2      READ ($$EVAPM)$
2      INPUT EVAPM (FOR DD11=(1,1,24)$ EE(DD11))$
2      DDE3=1$
2      FOR DDE1=(10,1,12),(1,1,9)$
2         BEGIN
2         FOR DDE2=(1,1,DPM(DDE1))$
2            BEGIN
2            E(HAAP(DDE1)+DDE2)=EE(DDE3)$
2            IF DDE2 EQL 15$ DDE3=DDE3+1$
2            END$
2         DDE3=DDE3+1$
2         END$
2      END$
2   OTHERWISE$       READ ($$EVAP)$
2   INPUT EVAP (FOR DD9=(274,1,365),(1,1,59),(366,1,DPY),(60,1,273)$
2   E(DD9))$
2   READ ($$EVC)$
2   INPUT EVC(FOR DD78=(10,1,12),(1,1,9)$EVCR(DD78))$
2   IF DCS(9)$
2      BEGIN
2      READ($$EVAPS)$
2      INPUT EVAPS(FOR DD85=(1,1,24)$EES(DD85))$
2      DDS3=1$
2      FOR DDS1=(10,1,12),(1,1,9)$
2         BEGIN
2         FOR DDS2=(1,1,DPM(DDS1))$
2            BEGIN
2            SE(HAAP(DDS1)+DDS2)=EES(DDS3)$
2            IF DDS2 EQL 15$DDS3=DDS3+1$
2            END$
2         DDS3=DDS3+1$
2         END$
2      EITHER IF DCS(10)$ READ ($$RADIATION)$
2      INPUT RADIATION (FOR DD76=(274,1,365),(1,1,59),(366,1,DPY),
2      (60,1,273)$ RAD(DD76))$
2      OTHERWISE$
2      BEGIN
2      READ ($$MRAD)$
2   INPUT      MRAD(FOR DDS5=(1,1,24)$RAS(DDS5))$
2   DDX3=1$
2   FOR DDX1=(10,1,12),(1,1,9)$
2      BEGIN
2      FOR DDX2=(1,1,DPM(DDX1))$
2         BEGIN
2         MAXRAD(HAAP(DDX1)+DDX2)=RAS(DDX3)$
2         IF DDX2 EQL 15$DDX3=DDX3+1$
2         END$
```

```
2       DDX3=DDX3+1$
2       END$
2   END$
2READ($$TEM)$
2INPUT TEM (FOR DD19=(1,1,730+2(DPY-365))$T(DD19))$
2       END$
2   FOR DD74=(1,1,366)$FLO(DD74)=0.0$
2   IF DCS(4)$  READ($$FLOWS)$
2   INPUT FLOWS (FOR DD17=(274,1,365),(1,1,59),(366,1,DPY),(60,1,273)$
2   FLO(DD17))$
2   IF DCS(5)$ READ($$DIVER)$
2INPUT DIVER (FOR DD53=(274,1,365),(1,1,59),(366,1,DPY),(60,1,273)$
2SDIV(DD53))$
2
2
2FOR DDSEG=(1,1,SEG)$
2BEGIN
2
2SABC=0.0$
2   READ ($$TRI)$
2   INPUT TRI (               FOR DD31=(1,1,12)$ QQQ(DD31))$
2   READ ($$ARRA1)$
2   INPUT ARRA1 (RINT,Z,FOR DD3=(1,1,Z)$C(DD3))$
2   READ ($$CL1)$
2   INPUT CL1(K1,AREA,A)$
2   READ ($$CL2)$
2INPUT CL2(EPXM,UZSN ,LZSN,K3,K24L,K24EL,CB,CC,L,SS,NN)$
2   READ ($$CL3)$
2   INPUT CL3(KS1,IRC,KV,KK24,ETL)$
2   IF DCS(9)$ READ ($$CL4)$
2   INPUT CL4(RADCON,CONMELT,SCF,ELDIF,IDNS,F,DGM,WC,MPACK,NXTAPM)$
2   WAI7..UNTIL CHECKM(10) NEQ 0$ GO TO WAI7$
2   IF DCS(9)$
2   MOVEM(10,12.NXTAPM)$
2   SGW1=SGW=SGWS(FLOWPOINT,DDSEG)$
2   UZS1=UZS=UZSS(FLOWPOINT,DDSEG)$
2   LZS1=LZS=LZSS(FLOWPOINT,DDSEG)$
2   GWS=GWSS(FLOWPOINT,DDSEG)$
2   SRC=1020.SQRT(SS)/(NN.L)$
2   DEC=0.00982.((NN.L/SQRT(SS))*0.6)$
2   CFSD=26.9.AREA$
2   CFS=24.CFSD$
2   ALBEDO=0.75$
2TOTELH=0.0$ NXSTORMS=0$ NXFHI=0$ NXFLOW=0$
2KK4=KK24*(1.0/96.0)$  LKK4=1.0-KK4$
2IRC4=IRC*(1.0/96.0)$  LIRC4=1.0-IRC4$
2SSGWF=SPR=SPRM=0.0$
2 TEMP=50.0$ NXTF=2$ SPX1=0$ COMMENT START EACH YEAR$
2PA=1.0-A$  SABC=SABD=SABM=0.0$
2EPX=EPXM$
2NXB=1$
2           WRITE($$CHECOUT,CHKF)$
2    OUTPUT CHECOUT(SGW,UZS,LZS,GWS,FLO(273),T(730+2.(DPY-365)))$
2      FORMAT CHKF(4(X8.2),B2,*FLOW*,X8.2,B2,*TEMP*,X8.2,W2)$
2   FOR MO=(10,1,12),(1,1,9)$
```

```
2         BEGIN
2         WAZZ..UNTIL CHECKM(10) NEQ 0$ GO TO WAZZ$
2         READM(10,24.DPM(MO)$REC1(1))$
2   SSF=0.0$
2         WAT..UNTIL CHECKM(10) NEQ 0$ GO TO WAT$
2         CHECKM(10$DDSTAT)$
2      IF DDSTAT EQL 1$ WRITE($$EOF)$
2      FORMAT EOF(*END OF FILE READ ON LOGICAL 10(PRECIP INPUT)*,W2)$
2      IF DDSTAT GTR 1$
2      WRITE ($$TRB)$
2      FORMAT TRB(*POSSIBLE ERROR ON LOGICAL 10 CHECK PRECIP TOTALS*,
2W2)$
2         FOR DAYMO=(1,1,DPM(MO))$
2            BEGIN
2            DAY=HAAP(MO)+DAYMO$
2   EP=EVCR(MO).E(DAY)$
2   SFM=0.0$
2   REP=0.0$
2   SCHGWF=PRPR=SFSF=0.0$
2   FOR HOUR=(1,1,24)$
2      BEGIN
2      PR=0.0$
2      NX=24.(DAYMO-1)+HOUR$
2      EPHRLI=0.0$
2      IF (HOUR GTR 8) AND (HOUR LSS 21)$ EPHRLI=0.08333333.EP$
2      ELH=EPHRLI.ETL$
2      PX=K1.REC1(NX)$
2      IF NOT DCS(11)$
2         SPR=SPR+PX$
2   IF DCS(9)$ ENTER SNOWMELTIV$
2      IF PX GTR 0.0$ RNM=RNM+PX$
2      COMMENT 15 MIN ACCOUNTING AND ROUTING LOOP$
2      SF=0.0$
2      FOR DD23=(1,1,4)$
2         BEGIN
2         EITHER IF DCS(11)$
2            BEGIN
2            PR=0.0$
2             IF DD23 EQL 1$
2            (XX=PX+0.000000001$  PX=0.0$)$
2            IF XX GTR 0.0$
2               BEGIN
2               PR=0.01.FIX(100.XX)$
2               PX=PX+PR$
2               SPR=SPR+PR$
2               XX=100.(XX-PR)$
2               END$
2            END$
2         OTHERWISE$
2         PR=0.25.PX$
2         EITHER IF PR GTR 0.0$
2         (PRPR=PRPR+PR$  GO L1$ )$
2         OR IF RES GTR 0.0$
2            BEGIN
2            P3=0.0$ GO TO L2$
```

```
2            END$
2         OR IF SRGX GTR 0.0$
2            BEGIN
2            P3=0.0$
2            ROS=0.0$ GO X2
2            END$
2            OTHERWISE$ (P3=0.0$ GO TO L4$)$
2            L1..
2         IF TAB$CUMPREC=CUMPREC+PR$
2            COMMENT INTERCEPTION$
2            EPX=EPXM-SCEP$
2            IF EPX LSS 0.0$ EPX=0.0$
2            EITHER IF PR LSS EPX$
2            BEGIN
2            SCEP=SCEP+PR$
2               P3=0.0$
2            END$
2         OTHERWISE$
2            BEGIN
2            P3=PR-EPX$
2            SCEP=SCEP+EPX$
2            END$
2         COMMENT P3 IS RAIN REACHING THE GROUND SURFACE$
2         LNRAT=LZS/LZSN$
2         COMMENT LOWER ZONE AND GW INFILTRATION$
2            L2..P4=P3+RES$
2            EITHER IF LNRAT LSS 1.0$ LNRATM=4.LNRAT$
2                 OR IF LNRAT LSS 2.0$ LNRATM=4.0+2(LNRAT-1.0)$
2            OTHERWISE$ LNRATM=6.0$
2            D3FV=CB/(2.0*LNRATM)$
2         D4F=0.25.D3FV$
2            RATIO=CC.(2.0*LNRAT)$
2         IF RATIO LSS 1.0$ RATIO =1.0$
2         EITHER IF P4 LSS D4F$
2            SHRD=P4.P4/(2.0.D4F)$
2         OTHERWISE$  SHRD=P4-0.5.D4F$
2         EITHER IF P4 LSS D4F.RATIO$
2            RXX=P4.P4/(2.0.D4F.RATIO)$
2         OTHERWISE$  RXX=P4-0.5.D4F.RATIO$
2            EITHER IF UZS LSS 2.0.UZSN$
2            BEGIN
2               UZI=2.0.ABS(0.5(UZS/UZSN)-1.0)+1.0$
2               PRE=(0.5.UZS/UZSN).(1.0/(1.0+UZI))*UZI$
2            END$
2            OTHERWISE$
2            BEGIN
2               UZI=2.0.ABS(((UZS/UZSN)-1.0)-1.0)+1.0$
2               PRE=1.0-(1.0/(1.0+UZI))*UZI$
2            END$
2         RGXX=SHRD-RXX$
2         RGX=RGXX.PRE$
2         COMMENT RGX IS THE VOLUME TO INTERFLOW DETENTION STORAGE$
2         RX=RXX.PRE$
2         COMMENT RX IS THE VOLUME TO OVERLAND FLOW SURFACE DETENTION$
2         UZS=UZS+SHRD-RGX-RX$
```

```
2            IF DCS(1)$ IF UZS GTR 1.5(UZSN)$ IF NOT TAB$
2            BEGIN
2            DDIM=MO$ DDID=DAYMO$ DDIH=HOUR$
2            CUMPREC=CUMPREC+0.25.PX$
2            SCEPS=SCEP$
2            TAB=1$ TABU=1$
2            END$
2         EITHER IF RX-RES GTR 0$ DE=DEC.((RX-RES)*0.6)$
2         OTHERWISE$DE=(RES+RX)/2.0$
2         IF (RES+RX) GTR 2.0.DE$ DE=(RES+RX)/2.0$
2         EITHER IF (RES+RX) GTR 0.01$
2         ROS=0.25.SRC.(((RES+RX)/2.0)*1.67). ((1.0+0.6((RES+RX)
2         /2.0.DE)*3.0)*1.67)$
2         OTHERWISE$ ROS=0.0$
2         IF ROS GTR 0.75.RX$ ROS=0.75.RX$
2         SROS=SROS+ROS$
2         IF TAB$ SURRO=SURRO+ROS$
2         RES=RX-ROS$
2         IF RES LSS 0.001$
2            BEGIN
2            LZS=LZS+RES$
2         IF TAB$ LZSIN=LZSIN+RES$
2            RES=0.0
2            END$
2         LZI=(1.5).ABS((LZS/LZSN)-1.0)+1.0$
2         PRE=(1.0/(1.0+LZI))*LZI$
2      IF LZS LSS LZSN$ PRE=1.0-PRE.(LZS/LZSN)$
2         COMMENT F3 HELD IN LOWER$
2         F3=PRE.(P4-SHRD)$
2         F1A=(1.0-PRE).(P4-SHRD)$
2         F1=F1A(1.0-K24L).PA$
2         IF TAB$ INFDIR=INFDIR+F1+F3$
2         SGW=SGW+F1$
2         IF TAB$CUMSGWIN=CUMSGWIN+F1$
2         GWS=GWS+F1$
2         LZS=LZS+F3$
2         IF TAB$ LZSIN=LZSIN+F3$
2         COMMENT INTERFLOW STORAGE IS SRGX$
2         SRGX=SRGX+RGX$
2         X2..INTF=LIRC4.SRGX$
2         IF TAB$ INTRO=INTRO+INTF$
2         COMMENT SUM OF INTERFLOW IS SINT$
2         SINT=SINT+INTF$
2         SRGX=SRGX-INTF$
2         IF SRGX LSS 0.0001$
2            BEGIN
2            LZS=LZS+SRGX$
2            IF TAB$ LZSIN=LZSIN+SGRX$
2            SRGX=0.0
2            END$
2         COMMENT GROUNDWATER FLOW CALC$
2         L4..
2         EITHER IF SGW GTR 0.00001$
2            GWF=SGW.LKK4.(1.0+KV.GWS)$
2         OTHERWISE$ GWF=0.0$
```

```
2             SCHGWF=SCHGWF+GWF$
2             SGW=SGW-GWF$
2             IF TAB$ CUMSGWOUT=CUMSGWOUT+GWF$
2             SSGWF=SSGWF+GWF$
2                R=(PA.ROS + P3.A+PA.INTF+GWF-0.25.ELH).CFS$
2             IF R LSS 0.0$ R=0.0$
2             SF=SF+R$
2             IF TAB$ IF R GTR 0.0$ CUMEVAP=CUMEVAP+0.25.ELH$
2                IF TAB$TOTRO=TOTRO+(R/CFS)$
2                IF R GTR 0.0$ TOTELH=TOTELH+0.25.ELH$
2             COMMENT ENTER STATSUB HERE$
2             IF DCS(1)$
2                BEGIN
2                IF DAY EQL 274$IF HOUR EQL 1$ IF DD23 EQL 1$
2                WRITE($$TITLE,TITLEG)$
2                FORMAT TLESS(B3,*MO*,B5,*DAY*,B5,*TIME*,B6,
2                *RAIN*,B8,*INTERCEPT*,B3,*INFILT*,B6,*INTERFLOW*,B3,
2                *SURFACE*,B5,*GROUND*,B6,*TOTAL*,B5,*TOTAL-CFS*,W2)$
2                EITHER IF 4.R LSS MINH$     DDEX=1$
2                OTHERWISE$
2                   BEGIN
2                    IF DDEX EQL 1$ (WRITE($$TLESS)$ DDEX=0.0$)$
2                DDTIME=100.(HOUR-1)+15.DD23$
2                IF MOD(DDTIME,100) EQL 60$ DDTIME=DDTIME+40$
2                WRITE($$DETAIL,DETAILF)$
2                   END$
2                OUTPUT DETAIL(MO,DAYMO,DDTIME,PR,
2                PR-P3,F1+F3,INTF,ROS,GWF,R/CFS,4.R)$
2                FORMAT DETAILF(I5,I7,I9,7(X12.3),X10.1,W0)$
2                F1=F3=INTF=0.0$
2             END$
2             END$
2          COMMENT END OF 15 MIN LOOP$
2          SFSF=SFSF+SF$
2          DDI=NXB$
2          COMMENT TRANSLATION IN TIME$
2       SC=SF/CFS$
2             EITHER IF SC GTR 0.01$
2             BEGIN
2             FOR DD=(1,1,Z)$
2                BEGIN
2             DDX=DDI+RINT(DD-1)$
2             TRS(DDX)=TRS(DDX)+SF.C(DD)$
2                END$
2             END$
2          OTHERWISE$TRS(DDI+(Z.RINT/2))=TRS(DDI+(Z.RINT/2))+SF$
2             SSF=SSF+SF$
2          IF DCS(7)$
2          BEGIN
2             IF SC GTR 0.0$
2             BEGIN
2                FOR DD33=(20,-1,1)$
2                EITHER IF SC GTR MXRO(DD33)$MXRO(DD33+1)=MXRO(DD33)$
2                OTHERWISE$
2                   BEGIN
```

```
2                  MXRO(DD33+1)=SC$
2               GO TO Z100$
2               END$
2            MXRO(1)=SC$
2         END$
2         Z100..
2         IF PX GTR 0.0$
2         BEGIN
2            FOR DD34=(20,-1,1)$
2            EITHER IF PX GTR MXRA(DD34)$ MXRA(DD34+1)=MXRA(DD34)$
2            OTHERWISE$
2               BEGIN
2                MXRA(DD34+1)=PX$
2                GO TO Z200$
2                END$
2            MXRA(1)=PX$
2         END$
2         Z200..
2      END$
2      IF EPHRLI EQL 0.0$ GO TO PASS$
2     IF SCEP GTR 0.0$
2      EITHER IF SCEP GTR EPHRLI$
2         BEGIN
2         SCEP=SCEP-EPHRLI$
2         SAET=SAET+EPHRLI$
2         IF TAB$CUMEVAP=CUMEVAP+EPHRLI$
2         EPHRLI=0.0$
2         GO TO PASS$
2         END$
2      OTHERWISE$
2         BEGIN
2         EPHRLI=EPHRLI-SCEP$
2         SAET=SAET+SCEP$
2         IF TAB$CUMEVAP=CUMEVAP+SCEP$
2         SCEP=0.0$
2         END$
2      IF UZS GTR 0.0$
2      EITHER IF UZS GTR EPHRLI$
2         BEGIN
2         UZS=UZS-EPHRLI$
2         IF UZS LSS 1.5,UZSN$ TAB=0$
2         SAET=SAET+PA.EPHRLI$
2         IF TAB$CUMEVAP=CUMEVAP+PA.EPHRLI$
2         EPHRLI=0.0$
2         GO TO PASS$
2         END$
2       OTHERWISE$
2         BEGIN
2         EPHRLI=EPHRLI-UZS$
2         SAET=SAET+PA.UZS$
2         IF TAB$CUMEVAP=CUMEVAP+PA.UZS$
2         UZS=0.0$
2         END$
2   REP=REP+EPHRLI$
2      PASS..
```

```
2      DEEPL =(UZS/UZSN)- (LZS/LZSN)$
2      IF DEEPL GTR 0.0$
2         BEGIN
2         LNRAT=LZS/LZSN$
2         RECE=0.003.CB.UZSN.(DEEPL*3)$
2         UZS=UZS-RECE$
2         IF UZS LSS 1.5(UZSN)$ TAB=0$
2         IF TAB$ INFUP=INFUP+RECE$
2         LZI=(1.5).ABS(LNRAT-1.0)+1.0$
2         PRE=(1.0/(1.0+LZI))*LZI$
2         IF LZS LSS LZSN$ PRE=1.0-PRE.LNRAT$
2         F3=PRE.RECE$
2         F1A=(1.0-PRE).RECE$
2         F1=F1A.(1.0-K24L).PA$
2         LZS=LZS+F3$
2         IF TAB$ LZSIN=LZSIN+F3$
2         SGW=SGW+F1$
2         IF TAB$ CUMSGWIN=CUMSGWIN+F1$
2         F1=F3=0.0$
2         GWS=GWS+F1
2         END$
2      COMMENT EVAPORATION 7PM$
2      IF HOUR EQL 21$
2         BEGIN
2         SPET=SPET+EP$
2         EP=REP$
2         IF GWS GTR 0.00001$
2         GWS=0.97.GWS$
2         LUS=SGW.K24EL.EP.PA$
2         CUMMENT EVAP-TRANS LOSS FROM GROUNDWATER$
2         SGW=SGW-LOS$
2         GWS=GWS-LOS$
2   TOTELH=IOTELH+LOS$
2         IF GWS LSS 0.0$ GWS=0.0$
2         IF EP NEQ 0.0$
2            BEGIN
2               LNRAT=LZS/LZSN$
2               EITHER IF EP LSS K3.LNRAT$
2                  BEGIN
2                  AETR=EP.(1.0-(EP/(2.0.K3.LNRAT)))$
2                   LZS=LZS-AETR$
2                  SAET=SAET+PA.AETR$
2                   END$
2               OTHERWISE$
2                  BEGIN
2                   AETR=0.5.(K3.LNRAT)$
2                   LZS=LZS-AETR$
2                  SAET=SAET+PA.AETR$
2                   END
2            END
2         END$
2        NXB=NXB+1$
2   IF DCS(1)$
2      BEGIN
2     FORMAT SAND(B5,*STORM PERIOD*,B5,*PRECIP*,B1,*EVAP*,B1,*SGWIN*,B1,
```

```
2         *SGWOUT*,B1,*LZSIN*,B1,*SURF-RO*,    B1,*INTER-RO*,B1,
2     *TOTAL-RO*,B1,*INFILT-UP*,B1,*INFILT-DIR*,B2,*CB/GW*,B2,*CB/RO*,
2           B2,
2         *SGWCOR*,B2,*CB*,B2,*LZRAT*,W2)$
2     IF TAB$ GO TO LEND$ COMMENT CONTINUING STORMS$
2     IF NOT TABU$ GO TO LEND$ COMMENT STORM ENDED AND PROCESSED$
2      IF CUMSGWIN LSS CUMSGWOUT$ GO TO RESET$
2     COMMENT SMALL STORMS$
2         BALNC=CUMPREC-CUMEVAP-CUMSGWIN+CUMSGWOUT-LZSIN.PA-TOTRO+SCEPS
2         -SCEP$
2     IF ABS(BALNC) GTR 0.01$
2     WRITE($$BALN1,BALN1F)$
2     OUTPUT BALN1 (BALNC)$
2     FORMAT BALN1F (B10,*STORM BALANCE IS*,B2,X7.2,W2)$
2     IF CUMRECFLOW GTR 0.0$
2     SFRAT=CUMSIMFLOW/CUMRECFLOW$
2     IF FLO(DAY) GTR 0.0$
2         BEGIN
2     GFRAT=SF/FLO(DAY)$
2     IF GFRAT GTR 0.0$
2     CBGW=CB/GFRAT$
2     CBRO=CB.SFRAT$
2         NXSTORMS=NXSTORMS+1$
2         IF (CBGW LSS 0.9.CB) AND (CBRO GTR 1.1.CB)$ NXFHI=NXFHI+1$
2         IF (CBGW GTR 1.1.CB) AND (CBRO LSS 0.9.CB)$ NXFLOW=NXFLOW+1$
2             END$
2     SGWCUR=0.0$
2     IF DCS(2)$
2         BEGIN
2             IF SF GTR 0.0$
2             BEGIN
2         NEWSGW=SGW.(FLO(DAY))/SF$
2         SGWCOR=NEWSGW-SGW$
2         SGW=NEWSGW$
2         END$
2             IF INFDIR GTR 0.33.INFUP$
2             BEGIN
2         IF (CBGW LSS 0.9.CB) AND (CBRO LSS 0.9.CB)$ CB=0.8.CB$
2         IF (CBGW GTR 1.1.CB) AND (CBRO GTR 1.1.CB)$ CB=1.2.CB$
2             END$
2         END$
2         WRITE ($$SAND)$
2     WRITE($$EZQ,EZQF)$
2     OUTPUT EZQ(DDIM,DDID,DDIH,MO,DAYMO,HOUR,
2     CUMPREC,CUMEVAP,CUMSGWIN,CUMSGWOUT,LZSIN,
2     SURRO,INTRO,TOTRO,INFUP,INFDIR,CBGW,CBRO,
2     SGWCOR,CB,LZS/LZSN)$
2   FORMAT EZQF(I2,*/*,I2,*/*,I2,B1,*TO*,B1,
2     I2,*/*,I2,*/*,I2,B2,X5.2,B1,X4.2,B2,X4.2,
2     B2,X4.2,B3,X4.2,B2,X4.2,B4,X4.2,B6,X4.2,
2     B5,X4.2,B7,X4.2,B5,X5.2,B2,X5.2,B3,X5.2,B1,X5.2,
2     B2,X4.2,W2)$
2     RESET.. CUMPREC=CUMEVAP=CUMSGWIN=CUMSGWOUT=0.0$
2         LZSIN=SURRO=INTRO=TOTRO=INFUP=INFDIR=0.0$
2     CBGW=SGWCOR=CBRO=0.0$
```

```
2      CUMSIMFLOW=CUMRECFLOW=0.0$
2      TABU=0$
2      LEND.,
2      END$
2      END$
2   COMMENT END OF HOUR LOOP$
2   IF TAB$
2      BEGIN
2      CUMSIMFLOW=CUMSIMFLOW+SFSF$
2         CUMRECFLOW=CUMRECFLOW+24.FLO(DAY)$
2      END$
2  END$
2   COMMENT END OF DAY LOOP$
2   TONE(MO)=SSF/CFS$
2     GWSA(MO)=GWS$
2      SPRA(MO)=SPR$    SPR=0.0$
2      SPRMA(MO)=SPRM$    SPRM=0.0$
2      SGWFA(MO)=SSGWF$   SSGWF=0.0$
2      SINTA(MO)=SINT$   SINT=0.0$
2      SPETA(MO)=SPET$   SPET=0.0$
2      SAETA(MO)=SAET$   SAET=0.0$
2      RNMM(MO)=RNM$  RNM=0.0$
2      MSURE(MO)=MSUREVAP$  MSUREVAP=0.0$
2      RADMES(MO)=RADME$   RADME=0.0$
2      SCOMELTS(MO)=SCOMELT$  SCOMELT=0.0$
2      SGWA(MO)=SGW$
2      UZSA(MO)=UZS$
2      LZSA(MO)=LZS$
2END$
2COMMENT END OF MONTH LOOP$
2WRITE ($$TRIAL,TRIALF)$
2OUTPUT TRIAL (FOR DD32=(1,1,12)$QQQ(DD32))$
2FORMAT TRIALF(B10,A72,W3)$
2WRITE ($$TITLE,TITLEF)$
2OUTPUT TITLE(FOR DD45=(1,1,10)$QQO(DD45),DDYR1,DDYR2)$
2FORMAT TITLEF    (A60,B3,*WATER YEAR  19*,I2,*-*,I2,B7,
2*STANFORD WATERSHED MODEL  IV *,W2)$
2SABC=RNA=TZN=RNB=SSAET=SSPET=SSINT=0.0$
2SRADME=SRNM=SMS=SCO=0.0$
2FOR DD25=(1,1,12)$
2   BEGIN
2   SABC=SABC+TONE(DD25)$
2   RNA=RNA+SPRA(DD25)$
2   TZN=TZN+SGWFA(DD25)$
2   RNB=RNB+SPRMA(DD25)$
2   SSAET=SSAET+SAETA(DD25)$
2   SSPET=SSPET+SPETA(DD25)$
2   SSINT=SSINT+SINTA(DD25)$
2   SRADME=SRADME+RADMES(DD25)$
2      SCO=SCO+SCOMELTS(DD25)$
2      SMS=SMS+MSURE(DD25)$
2   SRNM=SRNM+RNMM(DD25)$
2   END$
2WRITE ($$HSUM)$
2FORMAT HSUM(B8,*DAY*,B6,*OCT*,B5,*NOV*,B5,*DEC*,B5,*JAN*,B5,*FEB*,B5,
```

```
2*MAR*,B5,*APR*,B5,*MAY*,B5,*JUN*,B5,*JUL*,B5,*AUG*,B5,*SEPT*,B9,
2*ANNUAL*,W2)$
2WRITE ($$YY2,YY2F)$
2OUTPUT YY2(FOR NX0=(10,1,12),(1,1,9)$TONE(NX0),SABC)$
2FORMAT YY2F (*TOTAL*,B7,12(S8,3),B3,X7.2,B2,*INCHES*   ,W)$
2WRITE ($$YY40,YY40F)$
2OUTPUT YY40(FOR NX40=(10,1,12),(1,1,9)$SINTA(NX40),SSINT)$
2FORMAT YY40F(*INTERFLOW*,B3,12(S8.3),B3,S7.3,B2,*INCHES*   ,W)$
2WRITE ($$YN5,YN5F)$
2OUTPUT YN5(FOR DD30=(10,1,12),(1,1,9)$SGWFA(DD30),TZN)$
2FORMAT YN5F(*BASE*,B8,12(S8,3),B3,S7,3,B2,*INCHES*   ,W)$
2WRITE ($$YY15,YY15F)$
2OUTPUT YY15(FOR NX15=(10,1,12),(1,1,9)$SPRA(NX15),RNA)$
2FORMAT YY15F(*PRECIP*,B6,12X8.2,B2,X8.2,B2,*INCHES*    ,W2)$
2  IF DCS(9)$
2    BEGIN
2   WRITE($$FR1,FR1F)$
2    OUTPUT FR1(FOR NX1=(10,1,12),(1,1,9)$RNMM(NX1),SRNM)$
2    FORMAT FR1F(*RAIN+EF MELT*,12X8.2,B2,X8.2,B2,*INCHES*,W)$
2   WRITE($$FR2,FR2F)$
2    OUTPUT FR2(FOR NX2=(10,1,12),(1,1,9)$RADMES(NX2),SRADME)$
2    FORMAT FR2F(*RAD MELT*,B4,12X8.2,B2,X8.2,B2,*INCHES*,W)$
2   WRITE($$FR3,FR3F)$
2    OUTPUT FR3(FOR NX3=(10,1,12),(1,1,9)$SCOMELTS(NX3),SCO)$
2    FORMAT FR3F(*CONV MELT*,B3,12X8.2,B2,X8.2,B2,*INCHES*,W)$
2   WRITE($$FR4,FR4F)$
2    OUTPUT FR4(FOR NX4=(10,1,12),(1,1,9)$MSURE(NX4),SMS)$
2    FORMAT FR4F(*EVAP-SNOW*,B3,12X8.2,B2,X8.2,B2,*INCHES*,W2)$
2    END$
2WRITE($$YY17,YY17F)$
2OUTPUT YY17(FOR NX27=(10,1,12),(1,1,9)$SAETA(NX27),SSAET)$
2FORMAT YY17F(*EVP/TRAN-NET*,12(S8.3),B3,S7.3,B2,*INCHES*   ,W)$
2WRITE ($$YY18,YY18F)$
2OUTPUT YY18(FOR NX28=(10,1,12),(1,1,9)$SPETA(NX28),SSPET)$
2FORMAT YY18F(B2,*-POTENTIAL*,12(S8.3),B3,S7.3,B2,*INCHES*   ,W2)$
2WRITE ($$YY31,YY31F)$
2OUTPUT YY31(FOR NX21=(10,1,12),(1,1,9)$UZSA(NX21))$
2FORMAT YY31F(*STORAGES-UZS*,12(S8,3),B12,*INCHES*   ,W)$
2WRITE ($$YY8,YY8F)$
2OUTPUT YY8(FOR NX8=(10,1,12),(1,1,9)$LZSA(NX8))$
2FORMAT YY8F(B9,*LZS*,12(S8,3),B12,*INCHES*   ,W)$
2WRITE ($$YY22,YY22F)$
2OUTPUT YY22(FOR NX17=(10,1,12),(1,1,9)$SGWA(NX17))$
2FORMAT YY22F(B9,*SGW*,12(S8,3),B12,*INCHES*   ,W2)$
2WRITE ($$YY30,YY30F)$
2OUTPUT YY30(FOR NX20=(10,1,12),(1,1,9)$ GWSA(NX20))$
2FORMAT YY30F(*INDICES* GWS*,12(S8,3),W)$
2BAL=(LZS+UZS-LZS1-UZS1).PA+SGW-SGW1+SABC+TOTELH+SSAET   -RNA$
2WRITE($$BALOUT,BALOUTF)$
2OUTPUT BALOUT(BAL)$
2FORMAT BALOUTF(*BALANCE*,B5,X10.4,B2,*INCHES*,W2)$
2IF DCS(1)$         IF DCS(2)$
2WRITE($$HHH1,HHH1F)$
2OUTPUT HHH1(1.0-
2         (NXFHI+NXFLOW)/FLOAT(NXSTORMS),NXFHI,NXFLOW,NXSTORMS)$
```

```
2FORMAT HHH1F (*DATA CONSISTENCY INDEX*, B2,X7.2,B2,*HIGH INPUT INDICATI
2ONS*,B2,I5,B2,*LOW INPUT INDICATIONS*,B2,I5,B2,*TOTAL STORMS*,B2,I5,W2)
2 $
2IF NOT DCS(7)$ GO LBQ$
2 COMMENT OUTPUT  MAX. RUNOFF,PRECIP. AT END OF YEAR$
2   WRITE ($$HEADX)$
2   FORMAT HEADX(B10,*TWENTY HIGHEST CLOCKHOUR RAINFALL EVENTS IN THE WA
2TER YEAR*,W2)$
2   WRITE($$RAMAX,RAMAXF)$
2   OUTPUT RAMAX(FOR DD36=(1,1,20)$MXRA(DD36))$
2WRITE ($$HEADY)$
2   FORMAT HEADY(B10,*TWENTY HIGHEST CLOCKHOUR OVERLAND FLOW RUNOFF EVEN
2TS IN THE WATER YEAR*,W2)$
2   OUTPUT ROMAX(FOR DD37=(1,1,20)$MXRO(DD37))$
2   WRITE ($$ROMAX,RAMAXF)$
2   FORMAT RAMAXF (B5,20(X6.3),W2)$
2   FOR DD35=(1,1,20)$ MXRA(DD35)=MXRO(DD35)=0.0$
2LBQ..DDCOM=0$
2UZSS(FLOWPOINT,DDSEG)=UZS$
2LZSS(FLOWPOINT,DDSEG)=LZS$
2SGWS(FLOWPOINT,DDSEG)=SGW$
2GWSS(FLOWPOINT,DDSEG)=GWS$
2END$
2
2COMMENT END OF SEGMENT LOOP$
2NXC=0$ SSABD=0.0$
2SSABM=0.0$
2WRITE ($$TITLE,TITLEG)$
2FORMAT TITLEG    (A60,B3,*WATER YEAR  19*,I2,*-*,I2,B7,
2*STANFORD WATERSHED MODEL  IV *,W3)$
2
2FOR M1 =(10,1,12),(1,1,9)$
2   BEGIN
2SWITCH M1,(1,2,3,4,5,6,7,8,9,10,11,12)$
21..WRITE($$HJAN)$GO W$2..WRITE($$HFEB)$GO W$3..WRITE($$HMAR)$ GO W$
24..WRITE($$HAPR)$GO W$5..WRITE($$HMAY)$GO W$6..WRITE($$HJUN)$ GO W$
27..WRITE($$HJUL)$GO W$8..WRITE($$HAUG)$GO W$9.. WRITE($$HSEPT)$GO W$
210..WRITE($$HOCT)$GO W$11..WRITE($$HNOV)$GO W$12..WRITE($$HDEC)$GO W$
2FORMAT HNOV(*NOVEMBER*,W2)$ FORMAT HDEC(*DECEMBER*,W2)$
2FORMAT HJAN(*JANUARY*,W2)$ FORMAT HFEB(*FEBRUARY*,W2)$
2FORMAT HMAR(*MARCH*,W2)$ FORMAT HAPR(*APRIL*,W2)$  FORMAT HMAY(*MAY*,
2W2)$  FORMAT HJUN(*JUNE*,W2)$  FORMAT HJUL(*JULY*,W2)$
2FORMAT HAUG(*AUGUST*,W2)$ FORMAT HSEPT(*SEPTEMBER*,W2)$
2FORMAT HOCT(*OCTOBER*,W2)$
2W..
2SABD=SABM=0.0$
2FOR DAYS=(1,1,DPM(M1))$
2   BEGIN
2   SUMSF=0.0$
2   NXPRINT=0$
2   DAY=HAAP(M1)+DAYS$
2   IF (DAYS EQL 29) AND (M1 EQL 2)$ DAY=366$
2   FOR HOUR=(1,1,24)$
2      BEGIN
2      NXC=NXC+1$
```

```
2      IN=TRS(NXC)+SDIV(DAY)$
2      EITHER IF IN LSS 0.0000001$
2      SF=IN$
2   OTHERWISE$
2      SF=IN-KS1.(IN-LSF)$
2      IF SF GTR MINH$ NXPRINT=1$
2      LSF=SF$
2      TR(HOUR)=SF$
2      TRS(NXC)=SF$
2      SUMSF=SUMSF+SF$
2      END$
2   DRDAY=DR(DAY)=SUMSF/24.0$
2   IF NXPRINT GTR 0$
2      BEGIN
2      WRITE($$HOURFLOW1,HRF1)$
2      WRITE($$HOURFLOW2,HRF2)$
2      END$
2   OUTPUT HOURFLOW1(DAYS          ,FOR HOUR=(1,1,12)$TR(HOUR))$
2   OUTPUT HOURFLOW2(FOR HOUR=(13,1,24)$TR(HOUR),DRDAY)$
2   FORMAT HRF1(I4,B2,*AM*,B1,6X8.1,B3,6X8.1,W)$
2   FORMAT HRF2(B6,*PM*,B1,6X8.1,B3,7X8.1,W2)$
2   SABD=SABD+FLO(DAY)$
2   SABM=SABM+DR(DAY)$
2   COMMENT STORE ERRORS AND FLOW DURATION$
2   IF NOT DCS(6)$ GO TO LBY$
2   ERR=DR(DAY)-FLO(DAY)$
2   IF ABS(ERR) LSS 0.00001$ ERR=0.0$
2   EITHER IF FLO(DAY) LSS 1.0$ IND=1.0$
2   OTHERWISE$ IND=2.LOG(FLO(DAY))+2.0$
2   CAS(IND)=CAS(IND)+1.0$
2   SERR(IND)=SERR(IND) +ERR$
2   SERA(IND)=SERA(IND) +ABS(ERR)$
2   SQER(IND)=SQER(IND)+ERR.ERR$
2   AVER(IND)=SERR(IND)/CAS(IND)$
2   AVAR(IND)=SERA(IND)/CAS(IND)$
2   EITHER IF CAS(IND) GTR 1$
2     SQ(IND)=SQRT((SQER(IND)-((SERR(IND))*2)/CAS(IND))/(CAS(IND)-1.0))$
2   OTHERWISE$
2     SQ(IND)=0.0$
2   LBY..
2   END$
2COMMENT END OF DAYS LOUP$
2TONN(M1)=SABM$
2SSABM=SSABM+SABM$
2TOND(M1)=SABD$
2SSABD=SSABD+SABD$
2END$
2COMMENT END OF M1 LOUP$
2FOR NX57=(1,1,200)$
2TRSH(FLOWPOINT,NX57)=TRS(24.DPY+NX57)$
2
2WRITE ($$TITLE,TITLEG)$
2WRITE($$HSUM)$
2FOR DD27=(1,1,28)$
2   BEGIN
```

```
2   EITHER IF MOD(DD27,5) EQL 0$WRITE ($$FLTAB,FLTAB2)$
2   OTHERWISE$WRITE ($$FLTAB,FLTAB1)
2   END$
2EITHER IF DPY EQL 366$
2   BEGIN
2   DD27=29$    DR(60)=DR(366)$
2   WRITE($$FLTAB,FLTAB1)$
2   END$
2OTHERWISE$
2WRITE ($$DA29,DA29F)$
2OUTPUT DA29(DR(302),DR(333),DR(363),DR(29),DR(88),DR(119),DR(149),
2DR(180),DR(210),DR(241),DR(272))$
2FORMAT DA29F(B7,*29*,B3,4(X8.1),B3,*---*,B2,7(X8.1),W)$
2WRITE ($$DA30,DA30F)$
2OUTPUT DA30(DR(303),DR(334),DR(364),DR(30),DR(89),DR(120),DR(150),
2DR(181),DR(211),DR(242),DR(273))$
2FORMAT DA30F(B7,*30*,B3,4(X8.1),B8,7(X8.1),W)$
2WRITE ($$DA31,DA31F)$
2OUTPUT DA31(DR(304),DR(365),DR(31),DR(90),DR(151),DR(212),DR(243))$
2FORMAT DA31F(B7,*31*,B3,X8.1,B8,2X8.1,B8,X8.1,B8,X8.1,B8,2X8.1,W2)$
2OUTPUT FLTAB (DD27,FOR DD26=12,(1,1,11)$DR(HARP(DD26)+DD27))$
2FORMAT FLTAB1(B3,I6,B3,12(X8.1),W)$
2FORMAT FLTAB2(B3,I6,B3,12(X8.1),W2)$
2WRITE ($$YY1,YY1F)$
2OUTPUT YY1(FOR NX58=(10,1,12),(1,1,9)$TONN(NX58),SSABM)$
2FORMAT YY1F(*SYNTHESIS*,B3,12(X8.0),B1,X9.0,B2,*CFSD*,W)$
2WRITE ($$YZ1,YZ1F)$
2OUTPUT YZ1(FOR NX59=(10,1,12),(1,1,9)$TONN(NX59)/TCFSD,SSABM/TCFSD)$
2FORMAT YZ1F(B12,12(S8.3),B2,X8.1,B2,*INCHES*,W )$
2WRITE ($$YY23,YY23F)$
2OUTPUT YY23(1.98.SSABM)$
2FORMAT YY23F(B109,X9.0,B2,*ACFT*,W2)$
2IF NOT DCS(4)$ GO TO OMBY$
2WRITE ($$YY10,YY10F)$
2OUTPUT YY10(FOR NX59=(10,1,12),(1,1,9)$TOND(NX59),SSABD)$
2FORMAT YY10F(*RECORDED*,B4,12(X8.0),B1,X9.0,B2,*CFSD*,W)$
2WRITE ($$YY24,YY24F)$
2OUTPUT YY24(FOR NX60=(10,1,12),(1,1,9)$TOND(NX60)/TCFSD,SSABD/TCFSD)$
2FORMAT YY24F(B12,12(S8.3),B2,X8.1,B2,*INCHES*,W)$
2WRITE ($$YY25,YY25F)$
2OUTPUT YY25(YEAR,1.98.SSABD)$
2FORMAT YY25F(B92,*(*,X9.0,B2,*)*,B4,X9.0,B2,*ACFT*,W2)$
2OMBY..
2IF NOT DCS(6)$ GO TO DJ$
2WRITE ($$ERS)$
2FORMAT ERS(B10,*DAILY FLOW DURATION AND ERROR TABLE*,W3)$
2FORMAT ERT (B10,*FLOW INTERVAL*,B5,*CASES*,B3,*AV.ERROR*,B3,*AVR. ABS.
2ERROR*,B3,*STANDARD ERROR*,W2)$
2WRITE ($$ERT)$
2 SCASE=SSERR=SSERA=SSTER=0.0$
2FOR DD30=(1,1,25)$
2   BEGIN
2   EITHER IF DD30 EQL 1$ FLOO=0.0$
2      OR IF DD30 EQL 2$ FLOO=1.0$
2   OTHERWISE$ FLOO=EXP((DD30/2.0)-1.0)$
```

```
2   CAAS=CAS(DD30)$
2   OUTPUT ERRS1(FLOO,CAAS,SERR(DD30)/CAAS)$
2   OUTPUT ERRS0 (FLOO,CAAS)$
2   OUTPUT ERRS(FLOO,CAAS,SERR(DD30)/CAAS,SERA(DD30)/CAAS, SQ(DD30))$
2   FORMAT ERRSF( B13,X8.1,*-*,X9.1,X12.1,B5,X8.2,B5,X8.2,W)$
2   EITHER IF CAAS EQL 0.0$ WRITE ($$ERRS0,ERRSF)$
2   OR IF CAAS EQL 1.0$ WRITE ($$ERRS1,ERRSF)$
2   OTHERWISE$WRITE($$ERRS,ERRSF)$
2   SCASE=SCASE+CAS(DD30)$
2   SSERR=SSERR+SERR(DD30)$
2   SSERA=SSERA+SERA(DD30)$
2   SSTER=SSTER+SQ(DD30)$
2   END$
2OUTPUT ERRSUM (SCASE,SSERR,SSERA,SSTER)$
2FORMAT ERRSUMF(B22,X9.1,X12.1,B3,X10.2,B3,X10.2,W2)$
2WRITE ($$ERRSUM,ERRSUMF)$
2MEANSY=SABC/DPY$  MEANAC=SABD/DPY$
2ZACDIF=ZSYDIF=PRODIF=0.0$
2FOR DD38=(1,1,DPY)$
2BEGIN
2   ACDIF=FLO(DD38)-MEANAC$
2SYDIF=DR(DD38)-MEANSY$
2ZACDIF=ZACDIF+ACDIF.ACDIF$
2ZSYDIF=ZSYDIF+SYDIF.SYDIF$
2   PRODIF=PRODIF+ACDIF.SYDIF$
2END$
2CORCO=PRODIF/SQRT(ZACDIF.ZSYDIF)$
2OUTPUT COR(CORCO)$
2WRITE ($$COR,CORF)$
2FORMAT CORF(B10,*CORRELATION COEFFICIENT (DAILY)*,B3,X10.4,W2)$
2DJ..
2IF DCS(8)$
2   BEGIN
2   ENTER PLOT10$
2COMMENT DRAW AXIS$
2   COMMENT HORIZONTAL AXIS$
2   FOR X=31.0,61.0,92.0,123.0,151.0+EXD,182.0+EXD,212.0+EXD,243.0+EXD,
2   273.0+EXD,304.0+EXD,335.0+EXD,365.0+EXD$
2      BEGIN
2      PLOT($X/10.0,0.0,2)$
2      PLOT($X/10.0,0.2,2)$
2      PLOT($X/10.0,0.0,2)$
2      END$
2   DR(367)=FLO(367)=MAXCFS$
2   FOR DD82=(1,1,366)$
2      BEGIN
2      IF DR(DD82) GTR MAXCFS$DR(DD82)=MAXCFS$
2      IF FLO(DD82) GTR MAXCFS$FLO(DD82)=MAXCFS$
2      END$
2   SCALE($DR(),367,10.0,SACT,YMIN,DY)$
2   SCALE($FLO(),367,10.0,SSACT,YYMIN,DYY)$
2   AXIS($0.0,0.0,LABLE(1),18,SACT,90.0,YMIN,DY)$
2   PLOTWRITE (20.0,8.57,0.14,0.0$$FMT2)$
2   FORMAT FMT2(*STANFORD WATERSHED MODEL IV*,E)$
2   PLOTWRITE (20.0,8.07,0.14,0.0$$PLTX,PLTXF)$
```

```
2   OUTPUT PLTX(FOR DD76=(1,1,10)$QQQ(DD76),DDYR1,DDYR2)$
2   FORMAT PLTXF(A60,B3,*19*,I2,*-*,I2,E)$
2   PLOTWRITE(20.0,7.57,0.14,0.0$$PLTI,PLTIF)$
2   OUTPUT PLTI(FOR DD75=(1,1,12)$QQQ(DD75))$
2FORMAT PLTIF(A72,E)$
2       PLOT ($1.5,0.0,3)$
2   PLOTWRITE  (1.5,-0.2,0.14,0.0$$FMT)$
2   FORMAT FMT(*OCTOBER*,B18,*NOVEMBER*,B17,*DECEMBER*,B17,
2  *JANUARY*,B18,*FEBRUARY*,B17,*MARCH*,E)$
2   PLOTWRITE(19.5,-0.2,0.14,0.0$$FMTA)$
2FORMAT FMTA(*APRIL*,B20,*MAY*,B21,*JUNE*,B20,*JULY*,B20,*AUGUST*,
2   B19,*SEPTEMBER*,E)$
2       PLOT($0.0,-0.2,3)$
2       PLOT($0.0,0.0,3)$
2   COMMENT PLOT RECORDED FLOW$
2   PLOT($0.0,0.0,3)$
2   X=0.0$
2   FOR DDAY1=(274,1,365),(1,1,59),(366,1,DPY),(60,1,273)$
2       BEGIN
2       X=X+1.0$
2       PLOT($X/10.0,FLO(DDAY1),2)$
2       END$
2   PLOT($0.0,0.0,-3)$
2   COMMENT PLOT SYNTHESIS/ CHANGE TO RED$
2   X=0.0$
2   FOR DDAY2=(274,1,365),(1,1,59),(366,1,DPY),(60,1,273)$
2       BEGIN
2       X=X+1.0$
2       PLOT($X/10.0,DR(DDAY2),2)$
2       END$
2   PLOT($(DPY/10.0)+6.0,0.0,-3)$
2   COMMENT CLOSE OUT PLOT$
2END$
2COMMENT END OF PLOT LOOP$
2 GO TO L1NY$
2SUBROUTINE SNOWMELTIV$
2   BEGIN
2   M=0.0$
2   IF DAY EQL 274$IF HOUR EQL 1$ WRITE ($$TRIAL,TRIALF)$
2   IF DAY EQL 274$ IPACK=0.1.MPACK$
2   COMMENT CALCULATE CONTIN TEMPERATURE$
2   TEMPGRAD=GRAD(HOUR).CHANGE$
2   TEMP=TEMP+TEMPGRAD$
2   IF (HOUR EQL 6) OR (HOUR EQL 16)$
2   BEGIN
2       EITHER IF DCS(13)$
2       BEGIN
2       EITHER IF MOD(NXTF,2) EQL 0$NXTF=NXTF-1$
2       OTHERWISE$NXTF=NXTF+3$
2       COMMENT INPUT SERIES IS TMAX, TMIN EACH DAY, TMAX OCCURS
2       BEFORE OBSERVATION TIME$
2       END$
2       OTHERWISE$
2       NXTF=NXTF+1$
2       CHANGE=T(NXTF)-TEMP$
```

```
2      IF HOUR EQL 6$
2         BEGIN
2         CHAN=CHANGE/27,0$
2         IF CHAN GTR 1.0$ CHAN =1,0$
2         RADFAC=RADCON.CHAN,MAXRAD(DAY)$
2         END$
2      END$
2   COMMENT TEMPX IN ZONE$
2   LAPS=LAPSE(HOUR)$
2   IF PX GTR 0.05$ LAPS=0.75.LAPS$
2   TEMPX=TEMP-LAPS.ELDIF$    TEMPXR=0.557(TEMPX-32.0)+273.0$
2   IF PX +PACK EQL 0.0$ RETURN$
2   IF PX GTR 0.0$
2      BEGIN
2      COMMENT SNOW/RAIN CONTROL IS TEMP AT 750 FT$
2      IF (TEMPX - 0.75.LAPS) LEQ 32,0$
2      COMMENT SNOW$
2         BEGIN
2             PX=SCF.PX$
2            SPR=SPR+(SCF-1.0).PX$
2            EITHER IF TEMPX GTR 0.0$
2            DNS=IDNS+((TEMPX/100,0)*2.0)$
2            OTHERWISE$  DNS=IDNS $
2             PACK=PACK+PX $
2            IF PACK GTR IPACK$ IPACK=PACK$
2         IF IPACK GTR MPACK$ IPACK=MPACK$
2               ALBEDO=ALBEDO+0.04.PX$
2               IF ALBEDO GTR 0.75$ ALBEDO=0,75$
2             DEPTH=DEPTH+(PX/DNS)$
2            SUMSNOW=SUMSNOW+PX$
2             PX=0.0 $
2             END $
2         END $
2         CUMMENT FOR TEMPX GTR 32 PX IS UNCHANGED$
2   IF PACK EQL 0.0$ RETURN$
2   IF SDEN LSS 0.6$ DEPTH=DEPTH(1.0-0.00002(DEPTH(0,6-SDEN)))$
2   IF NEGMELT GTR 0.01.PACK$ IF LIQW GTR 0.2.WC.PACK$
2      BEGIN
2      NEGMELT=NEGMELT-0,01.LIQW$
2      PACK=PACK+0.01.LIQW$
2      LIQW=0;99.LIQW$
2      END$
2   COMMENT SNOW EVAPORATION$
2   IF TEMPX LSS 32.0$
2      BEGIN
2      EITHER IF PACK GTR IPACK$
2      SEVAP=SE(DAY)$
2      OTHERWISE$SEVAP=(PACK/IPACK).SE(DAY)$
2      COMMENT ASSUME DAILY SEVAP OCCURS IN 12 HOUR PERIOD$
2      SEVAP=0,0832.SEVAP$
2      MSUREVAP=MSUREVAP+SEVAP$
2      SAET=SAET+SEVAP$
2      IF PACK GTR SEVAP$
2      PACK=PACK-SEVAP$
2      IF SDEN GTR 0.0$
```

```
2      DEPTH=DEPTH-(SEVAP/SDEN)$
2      END$
2      COMMENT REDUCE REGULAR ET$
2      E1E=E2E=EPHRLI$
2      EITHER IF PACK GTR IPACK$ E1E=0.0$
2      OTHERWISE$E1E=(1.0-(PACK/IPACK)).E1E$
2      IF TEMPX LSS 38.0$ E2E=0.0$
2      EPHRLI=(1.0-F).E1E+F.E2E$
2      COMMENT FIND INCOMING SHORTWAVE$
2      EITHER IF DCS(10)$
2      RA=RADCON.RAD(DAY).RADDIST(HOUR)$
2      OTHERWISE$ RA=RADFAC.RADDIST(HOUR)$
2      COMMENT NET SHORTWAVE$
2      RA=(1.0-F).(1.0-ALBEDO).RA$
2      COMMENT CLEAR SKY LONG-WAVE RADIATION EXCHANGE
2      AND FOREST LONG-WAVE EXCHANGE$
2         LW=-27.5(1.0-F)(0.76.(TEMPXR/273.0)*4.0-1.0)
2      -27.5(F)((TEMPXR/273.0)*4.0-1.0)$
2      IF LW GTR 28.0$LW=28.0$
2      HM=RA-LW$
2   EITHER IF TEMPX LSS 32.0$
2         NEGMELTM=  ((32.0-TEMPX)/288.0).PACK$
2   OTHERWISE$ NEGMELTM=0.0$
2   EITHER IF HM LSS 0.0$
2         BEGIN
2         CHNEGM=-(HM/203.2)$
2         IF PACK LSS IPACK$CHNEGM=(PACK/IPACK).CHNEGM$
2      IF NEGMELT LSS NEGMELTM$
2         NEGMELT=NEGMELT+(1.0-(NEGMELT/NEGMELTM)).CHNEGM$
2         END$
2      OTHERWISE$
2      BEGIN
2      M=HM/203.2$
2      RADME=RADME+M$
2      END$
2        M=M+CONMELT.(TEMPX-32.0)$
2      SCOMELT=SCOMELT+CONMELT.(TEMPX-32.0)$
2      IF TEMPX -0.75.LAPSE(HOUR) GTR 32.0$
2      IF PX GTR 0.0$ IF TEMPX GTR 32.0$
2      M=M+((TEMPX-32.0).(PX/144.0))$
2      IF PACK LSS IPACK$
2      M=(PACK/IPACK).M$
2      IF M LSS 0.0$
2          BEGIN
2         IF NEGMELT LSS NEGMELTM$
2         NEGMELT=NEGMELT-(1.0-(NEGMELT/NEGMELTM)).M$
2         M=0.0$
2         END$
2IF M+PX GTR 0.0$
2   BEGIN
2      EITHER IF M LSS NEGMELT$
2      BEGIN
2      NEGMELT=NEGMELT-M$
2      M=0.0$
2      END$
```

```
2      OTHERWISE$
2      BEGIN
2      M=M-NEGMELT$
2      NEGMELT=0.0$
2      END$
2      EITHER IF ALBEDO GTR 0.7$ ALBEDO=ALBEDO-0.04.M$
2      OTHERWISE$ ALBEDO=ALBEDO-0.02.M$
2      IF ALBEDO LSS 0.65$ ALBEDO=0.65$
2      EITHER IF PX LSS NEGMELT$
2      BEGIN
2      NEGMELT=NEGMELT-PX$
2      PACK=PACK+PX$  PX=0.0$
2      END$
2      OTHERWISE$
2      BEGIN
2      PX=PX-NEGMELT$
2      PACK=PACK+NEGMELT$  NEGMELT=0.0$
2      END$
2      IF PX +M EQL 0.0$ GO TO LUPO$
2      EITHER IF M GEQ PACK$
2         BEGIN
2      M=PACK+LIQW$
2         DEPTH=PACK=LIQW=0.0$
2         ALBEDO=0.75$
2         END$
2      OTHERWISE$
2         BEGIN
2         PACK=PACK-M$
2         IF SDEN GTR 0.0$
2            DEPTH=DEPTH-(M/SDEN)$
2         IF PACK GEQ 0.9.DEPTH$ DEPTH=1.11.PACK$
2      IF PACK LSS 0.001$ PACK=0.0$
2      LIQS=WC.PACK$
2         IF SDEN GTR 0.6$ LIQS=WC.(3.0-(3.33).SDEN).PACK$
2         IF LIQS LSS 0.0$ LIQS=0.0$
2         END$
2EITHER IF (LIQW+M+PX) GTR LIQS$
2      BEGIN
2      PX=M+PX+LIQW-LIQS$
2      LIQW=LIQS  $
2   GO TO LUPO$
2      END $
2OTHERWISE $
2      BEGIN
2      LIQW=LIQW+M+PX$
2      PX=0.0 $
2   GO TO LUPO$
2      END $
2   END$
2   LUPO..
2      IF PACK GTR 0.0$ IF DEPTH NEQ 0.0$
2   IF PACK GTR 0.0$ IF PACK LSS DEPTH$
2      SDEN=PACK/DEPTH$
2   COMMENT GROUND MELT$
2   IF HOUR EQL 16$
```

```
2      BEGIN
2         DGMM=DGM$
2         IF NEGMELT GTR 0.01.PACK$
2         BEGIN
2         DGMM=DGM(1.0-(NEGMELT/0.08.PACK))$
2         NEGMELT=NEGMELT-DGM+DGMM$
2         END$
2         EITHER IF PACK GTR DGMM$
2      BEGIN
2         PX=PX+DGMM$
2         PACK=PACK-DGMM$
2            DEPTH=DEPTH-(DGMM/SDEN)$
2         IF PACK GEQ 0.9.DEPTH$ DEPTH=1.11.PACK$
2      END$
2   OTHERWISE$
2      BEGIN
2      PX=PACK+PX+LIQW$
2      PACK=DEPTH=LIQW=NEGMELT=0.0$
2         END$
2   END$
2   IF MO NEQ DDMOTO$
2      BEGIN
2      WRITE ($$FORMSP)$
2      FORMAT FORMSP (W)$
2      DDMOTO=MO$
2      END$
2      IF HOUR EQL 24$ IF PACK GTR 0.0$
2   WRITE($$SNOWA,SNOWF)$
2      OUTPUT SNOWA(MO,DAYMO,PACK,DEPTH,SDEN,ALBEDO,NEGMELT,LIQW)$
2   FORMAT SNOWF(B2,I4,B2,I4,B2,*PACK=*,X8.2,B2,
2                               *DEPTH=*,X8.2,B2,*DENS=*,X6.2,B2,
2   *ALBEDO=*,X6.2,B2,*NEGMELT=*,X6.2,B2,*LIQW=*,X6.2,W)$
2 RETURN
2 END$
2LFIN..UNLOAD(10)$
2FINISH$
```