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HYDROGRAPH SIMULATION MODELS OF THE HILLSBOROUGH AND ALAFIA RIVERS, FLORIDA: A PRELIMINARY REPORT

By

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ABSTRACT

Mathematical (digital) models that simulate flood hydrographs from rainfall records have been developed for the following gaging stations in the Hillsborough and Alafia River basins of west-central Florida: Hillsborough River near Tampa, Alafia River at Lithia, and North Prong Alafia River near Keysville. These models, which were developed from historical streamflow and rainfall records, are based on rainfall-runoff and unit-hydrograph procedures involving an arbitrary separation of the flood hydrograph. These models assume the flood hydrograph to be composed of only two flow components, direct (storm) runoff, and base flow. Expressions describing these two flow components are derived from streamflow and rainfall records and are combined analytically to form algorithms (models), which are programmed for processing on a digital computing system.

Most Hillsborough and Alafia River flood discharges can be simulated with expected relative errors less than or equal to 30 percent and flood peaks can be simulated with average relative errors less than 15 percent.

Because of the inadequate rainfall network that is used in obtaining input data for the North Prong Alafia River model, simulated peaks are frequently in error by more than 40 percent, particularly for storms having highly variable areal rainfall distribution.

Simulation errors are the result of rainfall sample errors and, to a lesser extent, model inadequacy. Data errors associated with the determination of mean basin precipitation are the result of the small number and poor areal distribution of rainfall stations available for use in the study. Model inadequacy, however, is attributed to the basic underlying theory, particularly the rainfall-runoff relation.

These models broaden and enhance existing water-management capabilities within these basins by allowing the establishment and implementation of programs providing for continued development in these areas. Specifically, the models serve not only as a basis for forecasting floods, but also for simulating hydrologic information needed in flood-plain mapping and delineating and evaluating alternative flood control and abatement plans.

INTRODUCTION

Serious water-management problems are being caused by flood-plain development in the Hillsborough and Alafia River basins. Encroachment by urban development of the flood-prone areas has greatly increased in recent years.

Large-scale flooding in these areas has not occurred since 1960. Consequently, construction of waterfront homes on the flood plain has become commonplace, particularly along the lower reach of the Hillsborough River in the large urban area of northeast Tampa and Temple Terrace. Large-scale encroachment of urban developments on low-lying areas of the Alafia River basin have been minimal, although numerous agricultural developments and small residential sub-divisions and trailer parks are appearing on the flood plain. Prevention of floods in these areas would allow development to continue safely. Interim measures include flood-plain zoning and early flood warning. The Southwest Florida Water Management District has long recognized the need to be able to predict reliably the flow of these streams under various hydrologic conditions and in 1968 entered into a cooperative study with the U. S. Geological Survey to develop flow simulation models for these basins.

Flood-control measures for the Hillsborough River have been proposed by the U. S. Corps of Engineers and are described in their report, Comprehensive report on four river basins, Florida, 1961. They propose the construction of numerous flood-retention reservoirs on many headwater tributaries, and a by-pass canal. The by-pass canal would divert a significant part of flood water from the Hillsborough River to Tampa Bay, thereby preventing flooding of the Tampa-Temple Terrace area except under most extreme conditions. Additional drainage improvements are also proposed for the upper reaches of the basin.

To aid in the evaluation of the effects of these proposed flood-control structures, reliable streamflow information is needed, especially the magnitude and frequency of floods. Because streamflow records are not available for most of the proposed reservoir sites, flow simulation methods would be useful in deriving streamflow information that could be used in preconstruction evaluations.

Flooding of the Alafia River basin has not caused the same kinds of problems that exist in the Hillsborough River basin. Nonetheless, reliable streamflow information for the Alafia River basin will assist in the orderly development of that basin.

The objective of the investigation is to develop mathematical models that simulate segments of the streamflow hydrograph. Simulated streamflow discharges will be used in evaluating and implementing flood control and abatement programs within these basins. Simulated streamflow discharges will also be useful in flood-frequency and flood-routing studies associated with the development of flood maps in both basins.

Model development covers two distinct phases in the project. During the first phase, computer models that simulate just the flood hydrograph were developed for 1 gaging station in the Hillsborough River basin and 2 gaging stations in the Alafia River basin. These preliminary flood prediction models, which are based on unit-hydrograph and rainfall-runoff procedures, are described in the following sections of this report.

To expand and improve the flow simulation (prediction) capability on the Hillsborough River, the second phase of the investigation primarily involves development of a hydrologic model of the entire basin and application of this basin-wide model in: (1) flood forecasting; (2) evaluating alternative flood-control measures; and (3) flood plain delineation and flood-mapping studies. This basin-wide hydrologic model will incorporate subbasin models developed at selected gaged points on all main-stem tributaries. Subbasin flow will be simulated at each of these points, routed through flood-control structures, and then downstream to the main channel where the flow will be accumulated and routed downstream until discharged from the basin. In addition to flood simulation, the basin model will also include a provision for simulating fair-weather flows from known ground-water conditions. The basin model also will provide information needed in maintaining desired water levels in the lower Hillsborough River for both dry and flood periods after flood-control measures become effective. Because the basin-wide model will be used in simulating streamflow under an extensive range of prevailing hydrologic conditions, methods and techniques more sophisticated than those described in this report are being considered as a basis for continued model development.

Data networks are being realigned and expanded to provide the additional information required in developing the basin model. Detailed hourly rainfall data within each subbasin will be obtained along with some information on ground-water conditions in the areas. Recording-stream gages will be operated at one or two critical points on the Hillsborough River main stem.

Continuation of model studies in the Alafia River basin will primarily involve application of the hydrograph models developed in the first phase in flood forecasting and in simulating streamflow information that will be used in developing flood maps of selected stream reaches. Additional raingage coverage is also being obtained in the Alafia River Basin to improve predictive capability of the models.

This study represents the first attempt by the Geological Survey to simulate floods on Florida streams by the application of unit-hydrograph and rainfall-runoff procedures.

Purpose and Scope

The purpose of this report is to document progress on the initial phase of the investigation, including a description of the flood-hydrograph models developed for the following gaging stations: Hillsborough River near Tampa, Alafia River at Lithia, and North Prong Alafia River near Keysville. Although work has been done on the development of hydrograph models at other gaging stations in the Hillsborough River basin, results of these analyses are at various stages of completion, and therefore are not included in this report.

The models developed in this investigation consider the flood hydrograph to be composed of only two components--direct (storm) runoff and base flow. Derivation of the analytical expressions describing direct runoff and base flow is based on unit-hydrograph theory with an arbitrary base-flow separation of the flood hydrograph.

The relationships that form the actual basis of the models are determined from observed streamflow and precipitation records. These relationships include: (1) a characteristic (or average) unit hydrograph; (2) a rainfall-runoff relationship; and (3) equations describing the recession characteristics of a stream during flood periods.

The simulation models described in this report were derived by linking these basic relationships in forming composite computational routines. These routines are then programmed for processing on a digital computer. Mathematical formulation of the computational processes is given in matrix notation to simplify descriptions of the actual arithmetic involved in flow simulation. Computational routines are given in analytical form in lieu of abstract flow charts that usually accompany computer programs.

Reliability of the models is judged on the basis of a comparison of simulated and observed floods used in model development. Calculated relative errors between observed and simulated flood discharge are used as indicators of the expected errors that could exist in flood discharges simulated from rainfall.

Acknowledgments

This study is supported through a cooperative program between the U. S. Geological Survey and the Southwest Florida Water Management District.

J. A. Mann, formerly of the Geological Survey, worked on the initial phase of the investigation. D. O. Moore of the Geological Survey's Nevada District served as consultant to Mr. Mann in developing the first phase hydrograph programs. R. N. Cherry and W. R. Murphy aided in computer programming of the original simulation model for the Hillsborough River Basin.

The work on the investigation was under the immediate supervision of J. S. Rosenshein of the Tampa Subdistrict, whose guidance strongly influenced the direction of the investigation and its aims and results. The work was performed under the general supervision of C. S. Conover, District Chief for Florida.

Location and Extent of Area

The area covered by this report consists of the Hillsborough and the Alafia River basins in west-central Florida (fig. 1). The study area extends nort' irom Tampa to the Withlachoochee River basin, and as far east as Lakeland to the Peace River basin. It is bordered on the south by the Little Manatee River basin, on the west by Hillsborough Bay, and farther north by the Anclote and Pithlachascotee River basins. The Hillsborough and Alafia basins, which have a combined area of about 1,100 square miles, lie in parts of Hillsborough, Pasco, Hernando, and Polk counties. The area is outlined on the map shown in figure 1.

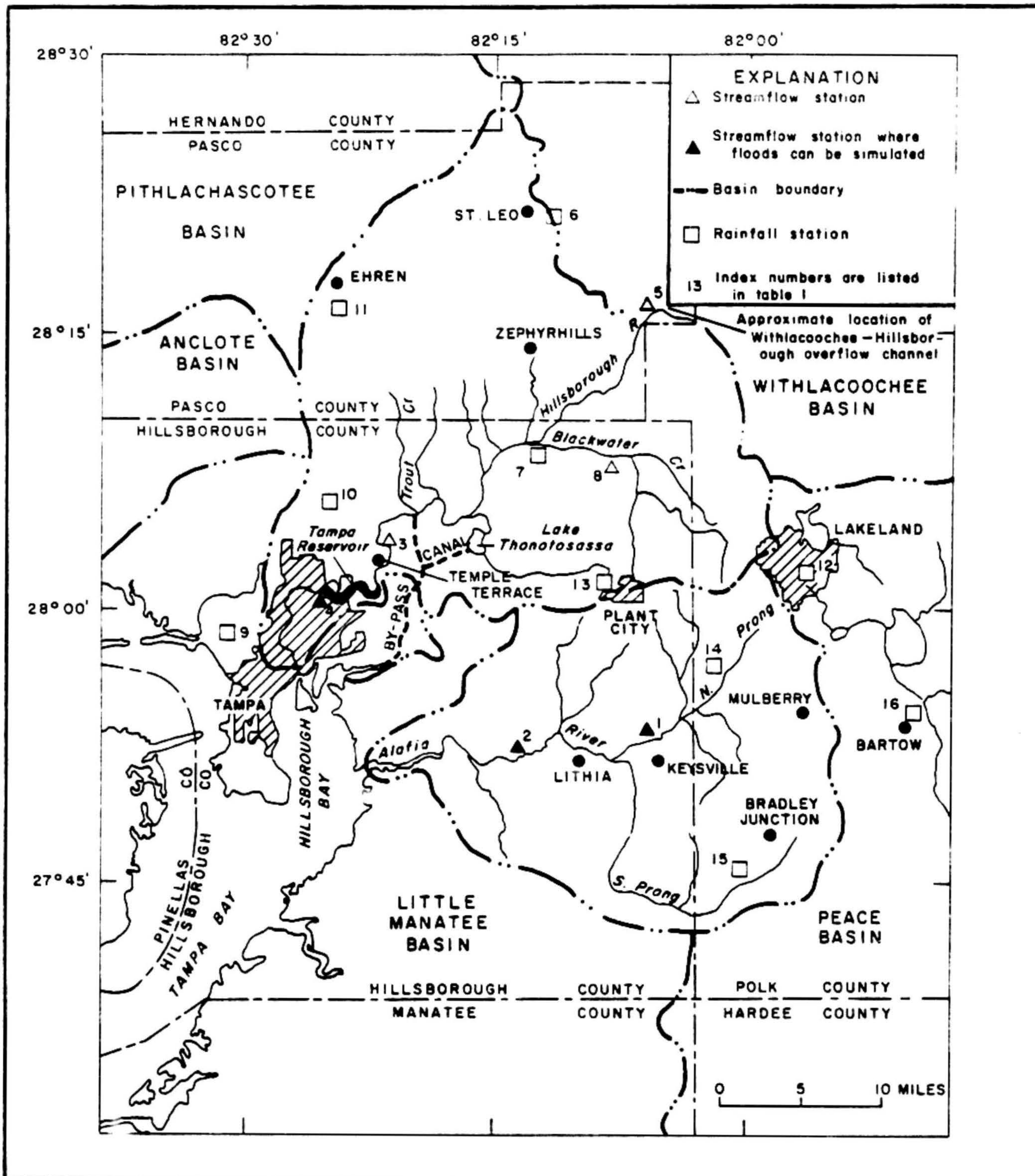


Figure 1.--Map of study area showing location of rainfall and streamflow instruments.

Climate and Physiography

The climate of both basins is about the same. The mean annual temperature for both basins is about 72°F (22.2°C) and mean annual precipitation ranges from 53 to 55 inches. The western sector of the area of investigation probably experiences more severe thunderstorms than any other area in the United States. A large tropical disturbance or regional storm, however, is expected only once every 2 years on the average, and a hurricane is expected only once every 5 years on the average.

Although annual rainfall over both basins is about the same, annual runoff (in inches) from the Alafia River basin is 13 percent greater than runoff from the Hillsborough basin. This difference is attributed principally to differences in topographic characteristics of the two basins and to some extent to differences in geology. Topographic relief of these basins is significantly different. The mean channel slope of the Hillsborough River is about 1.5 feet per mile; the slope of the Alafia River is more than 3.0 feet per mile.

The Hillsborough River basin contains many natural lakes, sinkholes, and swamps (6 to 7 percent of total basin area). It also receives flood water via an overflow channel from the Withlacoochee River (see fig. 1). The Alafia basin is characterized by numerous manmade features such as open-pit phosphate mines (5 to 10 percent of the basin).

DATA AVAILABLE

The development of hydrograph simulation models depends on the availability of streamflow and rainfall records. Streamflow data used in model development include records collected at the following gaging stations: Hillsborough River near Tampa, Alafia River at Lithia, and North Prong Alafia River near Keysville. Precipitation data include rainfall records collected by National Oceanic and Atmospheric Administration (NOAA) near Tampa, St. Leo, Lakeland, Bartow, and Plant City. Locations of these and other data sites related to the study are shown in figure 1. Specific locations and other pertinent information regarding records are summarized in table 1.

Table 1.--Summary of streamflow and rainfall networks related to model development

Map Index Number[f]	Station Name	Location	Instrument	Data Obtained	Period of Record
1	Alafia River, North Prong near Keysville, Fla. (Drainage area, 135 sq mi).	1.2 miles north of Keysville, Hillsborough County	Water-level recorder	Stage and discharge	May 1950-present
2	Alafia River at Lithia, Fla. (Drainage area, 335 sq mi).	1.1 miles northwest of Lithia, Hillsborough County	-do-	-do-	Oct. 1932-present
3	Hillsborough River at Fowler Ave., near Tampa, Fla. (Drainage area, 630 sq mi).	At Fowler Avenue, Hillsborough County	Staff gage[a]	Stage (read once daily) and discharge	Oct. 1933-Dec. 1939, Jan. 1961-present
4	Hillsborough River near Tampa, Fla. (Drainage area, 650 sq mi).	At Tampa Reservoir Dam, Hillsborough County[b]	Water-level recorder	Stage and discharge	Oct. 1938-present
5	Withlacoochee-Hillsborough overflow near Richland, Fla.	2.9 miles east of Richland, Pasco County	-do-	Stage and discharge[e]	Feb. 1930-Sep. 1931, Sep. 1950, July 1958-March 1960, April 1960-present
6	Saint Leo[c]	At St. Leo, Pasco County	Rain gage[a]	Rainfall	Sep. 1944-present
7	Hillsborough River State Park, Fla.[d]	At Hillsborough River State Park	-do-	-do-	Sep. 1943-present

Table 1.--Summary of streamflow and rainfall networks related to model development (continued)

8	Blackwater Creek near Knights, Fla.	4.4 miles northwest of Knights, Hillsborough County	Rain gage[a]	Rainfall	June 1970-present
9	Tampa WSO, Fla.[c]	At Tampa Internat'l Airport, Hillsborough County	Rain gage	-do-	June 1946-present
10	Pebble Creek near Tampa, Fla.	At Pebble Creek Golf and Country Club, approximately 11 miles northeast of Tampa, Hillsborough County	Rain gage[a]	-do-	Aug. 1970-present
11	Big Cypress Creek near Ehren, Fla.	1.0 mile northeast of Land O' Lakes, Hillsborough County	-do-	-do-	June 1970-present
12	Lakeland WSO, Fla.[c]	At Lakeland City Hall, Polk County	Rain gage	-do-	May 1915-present
13	Plant City, Fla.[c]	At Plant City, Hillsborough County	Rain gage[a]	-do-	Oct. 1892-present
14	Circle - X Airport near Mulberry, Fla.	At Circle - X Airport, 5.0 miles northwest of Mulberry, Polk County	-do-	-do-	June 1970-present
15	South Prong Alafia River near Bradley Junction, Fla.	3.8 miles southwest of Bradley Junction, Polk County	-do-	-do-	Aug. 1970-present
16	Bartow, Fla.	At Bartow, Polk County	-do-	-do-	Aug. 1895-present

a/ Non-recording
b/ Prior to Oct. 1945 gage was operated at site 2.1 miles upstream.
c/ Operated by the National Oceanic and Atmospheric Administration (Environmental Data Services).
d/ Operated by Florida Park Service
e/ Record intermittent
f/ Map index numbers refer to site locations shown on map in figure 1

Streamflow Records

Many factors affect the accuracy of streamflow records because of the manner in which flow is determined. Discharge is normally obtained by developing a rating curve that shows graphically the relation between periodic flow measurements and simultaneously observed stream stages. Estimates of continuous flow can be made from a continuous record of stage by using this rating curve. Ratings for many streams remain extremely stable; others change constantly or shift in response to physical changes in the stream channel. As these rating changes occur, new or modified rating curves must be developed to insure accurate flow records.

The errors associated with the discharge records collected at the Hillsborough River near Tampa gaging station are primarily the result of the computational technique used in determining flow. Discharge at this station is affected by stage of the Tampa Reservoir and setting of radial gates in the spillway. Obtaining data needed to compute daily discharge is difficult because of frequent non-scheduled changes in gate openings.

Errors associated with the discharge records collected at the Alafia River gaging stations result primarily from unstable ratings associated with scour and deposition occurring in the stream channels. Accuracy of the streamflow records collected at the gaging stations listed in table 1 is generally fair, and the errors associated with the flood data probably do not exceed 10 to 15 percent.

Physical changes occurring in both basins could alter their flow regime. One change in the Hillsborough basin that may affect streamflow is construction and operation of the Withlacoochee-Hillsborough overflow channel. This channel permits flood waters from the Withlacoochee River to be diverted into the headwaters of the Hillsborough River (fig. 1). Streamflow records collected at this site indicate that the effect of this diversion on the Hillsborough River is probably negligible during moderate-sized floods. Open-pit phosphate mines in the Alafia River basin alter the natural flow regime of the basin by increasing surface retention. This effect probably has varied over the entire period for which streamflow records have been collected in that basin because the number of abandoned mines has constantly increased.

Rainfall Records

Because of the erratic movement of small storms over a large area such as the Hillsborough and Alafia River basins, the task of adequately sampling rainfall is difficult without a relatively dense network of rain gages. For this reason, the Geological Survey operates five nonrecording rainfall stations in this area to supplement data from the NOAA rainfall stations (table 1). The Geological Survey rainfall stations, which are read daily by local observers in accordance with standard NOAA procedure, provide data needed in computing reliable mean basin rainfall for storms having highly variable rainfall rates. When large regional storms occur, a reliable estimate of areal distribution of rainfall may be derived from a smaller number of rain gages. In spite of these problems, the accuracy of all the rainfall data used in this study is considered good.

DEVELOPMENT OF HYDROGRAPH SIMULATION MODELS

In this investigation, development of hydrograph simulation models involves three phases: (1) determining the primary or physical components of the flood hydrograph; (2) expressing these components analytically; and (3) determining a procedure to combine these expressions in a hydrograph model. Models developed in this investigation assume the flood hydrograph to be composed of only two components--direct runoff and base flow. These flow components are defined in an arbitrary separation of the flood hydrograph, and corresponding analytical expressions are derived on the basis of rainfall-runoff and unit-hydrograph theories and observed streamflow and rainfall records.

The data used in model development are selected in a survey of concurrent streamflow and rainfall records to isolate simple independent storms. Flood hydrographs corresponding to these selected storm periods are empirically separated into two distinct hydrographs --direct (storm) runoff and base flow. Observed rainfall and the direct runoff part of the separated hydrographs are used jointly in developing a rainfall-runoff relationship. A characteristic or average unit hydrograph is computed from the direct runoff part of the separated hydrograph.

In this report, a unit hydrograph refers to the successive flow rates, for equal intervals of time, at which 1 inch of direct runoff covering an entire area is discharged from that area. The rainfall-runoff relationship and the unit hydrograph are used as the basis for simulating the direct runoff part of the flood hydrograph.

Relations that are used in this study to approximate the base-flow part of the flood hydrograph include an average base-flow recession curve and base-flow increase. The base-flow recession curve is based on data taken from the recession limb of the selected flood hydrographs. Base-flow increase, however, is computed from a relationship involving the maximum ordinate of the direct-runoff and base-flow recession segments of the separated flood hydrographs.

Digital simulation models are formed by combining the rainfall-runoff relation, unit hydrograph, average base-flow recession curve and base-flow increase relation to obtain composite computational routines which are subsequently programmed for processing on a digital computer.

This report cites, in addition to mathematical descriptions of the models, a numerical example concerning the simulation of a flood with the Hillsborough River model. This example demonstrates the computational steps involved in actual simulation.

Symbols used in the analytical expressions are given in table 2.

To show the relative magnitude of error that could exist in computed floods, storms that are used in developing the models are simulated and compared with observed hydrographs.

In this report, direct runoff refers only to rainfall that appears promptly in the stream channel; all other flow components are referred to as base flow.

Table 2. List of symbols used in analytical expressions in this report.

B	Daily rainfall observed at Bartow, in inches.
c_d	Storm duration coefficient.
d	Base-flow increase, in cubic feet per second.
k	Number of direct runoff values, n, plus the number of unit hydrograph ordinates, m, minus 1.
L	Daily rainfall observed at Lakeland, in inches.
m	Number of unit hydrograph ordinates.
n	Number of direct runoff values.
$\bar{p}$	Mean basin rainfall, in inches.
PC	Daily rainfall observed at Plant City, in inches.
Q_{bf}	Column matrix of simulated base-flow hydrograph ordinates (k-rows by 1-col.).
$(q_{bf})_i$	The i^{th} element of the simulated base-flow hydrograph matrix, Q_{bf}, in cubic feet per second.
q_i	Recession discharge, corresponding to the i^{th} time interval on the base-flow recession curve, in cubic feet per second.
Q_{sr}	Column matrix of simulated direct runoff hydrograph ordinates (k-rows by 1-col.).
Q_t	Column matrix of simulated flood hydrograph ordinates (k-rows by 1-col.).
R	Column matrix of direct runoff values (n-rows by 1-col.).
r	Direct runoff (for specified time intervals), in inches.

Table 2.--List of symbols used in analytical expressions in this report (continued)

r'	Adjusted direct runoff (for specified time intervals), in inches.
R_{adj}	Column matrix of adjusted direct runoff values (n-rows by 1-col.).
S	Daily rainfall observed at St. Leo, in inches.
S_d	Storm duration, in hours.
s	Direct peak runoff, in cubic feet per second.
T	Daily rainfall observed at Tampa, in inches.
t	Time index (referring to the number of time intervals past the highest recession curve discharge).
Δt	Duration of the unit hydrograph or the time increment used in hydrograph simulation; values used in this study were computed as 20 percent of the time lag between the center of mass of rainfall and the center of mass of direct runoff.
UH	Matrix developed from unit hydrograph ordinates (k-rows by n-cols.).

Hydrograph Theory

Hydrograph Separation

In this study relations that are used in model development require an arbitrary separation of the flood hydrograph. The separation method used is illustrated by the sketch shown in figure 2. Separation of flood hydrographs involves estimating the position of the interface between direct runoff and base-flow. The interface consists of three segments. The first extends from the beginning of the storm to the time of the peak, and the second extends backward from a point on the hydrograph where nearly all direct runoff has ceased, to one time interval (t of fig. 2) after the flood peak. The two recession curve segments are then connected by a straight line (segment three), and the difference in discharge is referred to in this report as base-flow increase (see fig. 2).

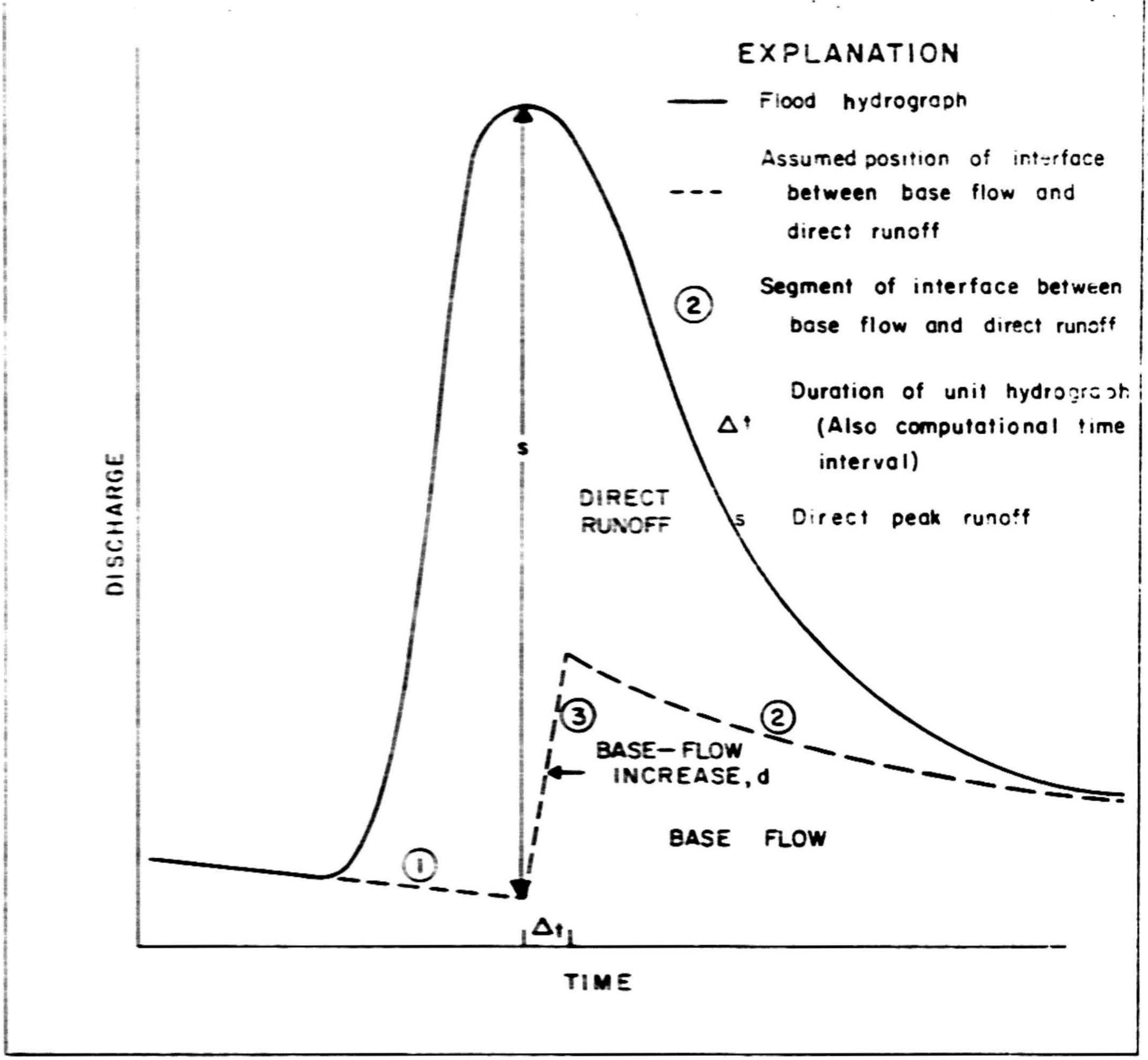


Figure 2.--Sketch showing components of idealized flood hydrograph used in study.

Unit Hydrograph

Two assumptions are used in developing average unit hydrographs in this investigation: (1) the flood hydrograph is composed only of two components, base-flow and direct runoff (see fig. 2); and (2) that direct runoff can be estimated from observed flood hydrographs using the arbitrary separation technique illustrated by figure 2. Using these assumptions, a set of direct-runoff hydrograph ordinates is computed for each of the selected storm periods by subtracting consecutive base-flow discharges from corresponding flood discharges. This calculation is made at equal time intervals throughout the entire flood hydrograph. A set of unit hydrograph ordinates is computed for each storm by dividing each of the computed direct-runoff ordinates by the accumulated or total direct runoff, in inches. An average unit hydrograph for the stream is obtained by finding the arithmetic mean of corresponding ordinates using the maximum ordinate as a base of reference.

The time increment, Δt, or unit-hydrograph duration, used in unit hydrograph computation (the Δt of fig. 2) should be sufficiently small to preserve the shape of the flood hydrograph, particularly the peak. In this study, duration of the unit hydrograph is calculated as 20 percent of the time lag between center of mass of runoff and center of mass of rainfall.

A more complete discussion of unit-hydrograph theory is given by Chow (1964).

Rainfall-runoff Relationships

A rainfall-runoff relation is based on mean basin rainfall and total direct runoff for various storms. In developing this relation, the selected flood hydrographs are partitioned into their two components (fig. 2), and total direct runoff for each flood period is computed in inches. Mean basin rainfall is calculated from observed rainfall by the application of areal weighting factors derived from Theissen polygons, as described in Chow (1964, p. 9-28). These weighting factors are referred to in this report as Theissen-weighting coefficients. A graphical relation is then determined by plotting total direct runoff as the dependent variable and mean basin rainfall (total observed during the storm) as the single independent variable. An equation that best describes the relation between these two variables is determined. Because incremental values of direct runoff are required in the simulation process, it is assumed in this study that these values can be calculated directly from the rainfall-runoff relations using incremental values of mean basin rainfall. The unit of time associated with these incremental values is equal to the duration of the unit hydrograph (Δt of fig. 2).

Direct-runoff Hydrograph

The direct-runoff hydrograph, or that part of the flood hydrograph which excludes base flow, is simulated using the rainfall-runoff relation and average unit hydrograph. The manner in which these two relations are used in simulating the runoff hydrograph initially involves the determination of mean basin rainfall. Rainfall is obtained for time intervals equal to the time increment or duration of the unit hydrograph (Δt on fig. 2) at selected gages located at points in and near the basins. Mean basin rainfall is determined by the application of Thiessen-weighting coefficients. Mean basin rainfall is used as an input to the rainfall-runoff relation to determine an array of runoff values for an entire storm at intervals of time equal to the duration of the unit hydrograph. Each of these runoff values is multiplied by each of the ordinates of the unit hydrograph to determine the time-discharge distribution from the basin. An individual runoff hydrograph is generated for each runoff value that is applied to the set of unit-hydrograph ordinates. Since the hydrographs are computed from runoff, the beginning point of each hydrograph is spaced (with regard to time) in the same sequential pattern as the runoff values. The direct-runoff hydrograph is determined from these sequentially spaced hydrographs by the addition of ordinates.

Where possible, adjustment to runoff is made to compensate for erratic rainfall patterns and storm duration. These adjustments are made prior to application of the unit hydrograph.

Base-flow Hydrograph

The base-flow part of the flood hydrograph represents ground-water discharge entering the stream. It is computed using two relations describing assumed base-flow characteristics of the stream during flood periods, namely :(1) an average base-flow recession curve ; and (2) a base-flow increase relation.

An average base-flow recession curve is developed by averaging discharges taken from the recession limbs of observed flood hydrographs. A simple exponential equation is fitted to the average recession curve and used in the simulation process for calculating base-flow recession discharges.

A base-flow increase relation is derived in a regression analysis of direct peak runoff and base-flow increase values estimated graphically from observed flood hydrographs separated according to the sketch shown in figure 2. An equation of the average base-flow increase relation is used with the average base-flow recession equation in simulating base-flow hydrograph discharges. Use of the base-flow increase relation, like the rainfall-runoff relation, assumes application on an incremental time basis.

Base-flow hydrograph computation begins with stream discharge prior to the storm. From this initial flow condition, sequential base-flow discharges are computed down the average recession curve until the first base-flow increase occurs. At this point, a new (higher) position on the recession curve is determined by adding the base-flow increase value to the last base-flow value computed. Base-flow increase is obtained from the base-flow increase relation using the peak of the first simulated runoff hydrograph. Computation of base-flow discharges resumes until the second base-flow increase occurs. At this point, the stair-stepping procedure is repeated, and computation continues until the base-flow hydrograph is completed.

For the Hillsborough River model, daily runoff values are lagged one day (added to the next runoff value) when runoff on a previous day is zero. In effect, therefore, two runoff peaks may occur on the same day. When this occurs, the sum of these two peaks is used in determining base-flow increase for that particular day. However, in actual computation, lags are achieved in a prior runoff adjustment process that is discussed in later sections of this report. Time lags are not used in the Alafia River models.

Flood Hydrograph

A generalized diagram of the primary elements associated with the development and linkage of the base-flow and runoff hydrographs is shown in figure 3. As indicated in this diagram, successive values of mean basin rainfall are used as an input to the rainfall-runoff relation in obtaining an array of direct-runoff values for a storm. Each direct-runoff value is then applied to the unit hydrograph in simulating the direct-runoff hydrograph. The ground-water contribution or base-flow hydrograph is simulated by means of the derived base-flow recession and base-flow increase relations using flow conditions prior to the storm as initial input. The direct-runoff hydrograph and the base-flow hydrograph are combined to form the storm hydrograph.

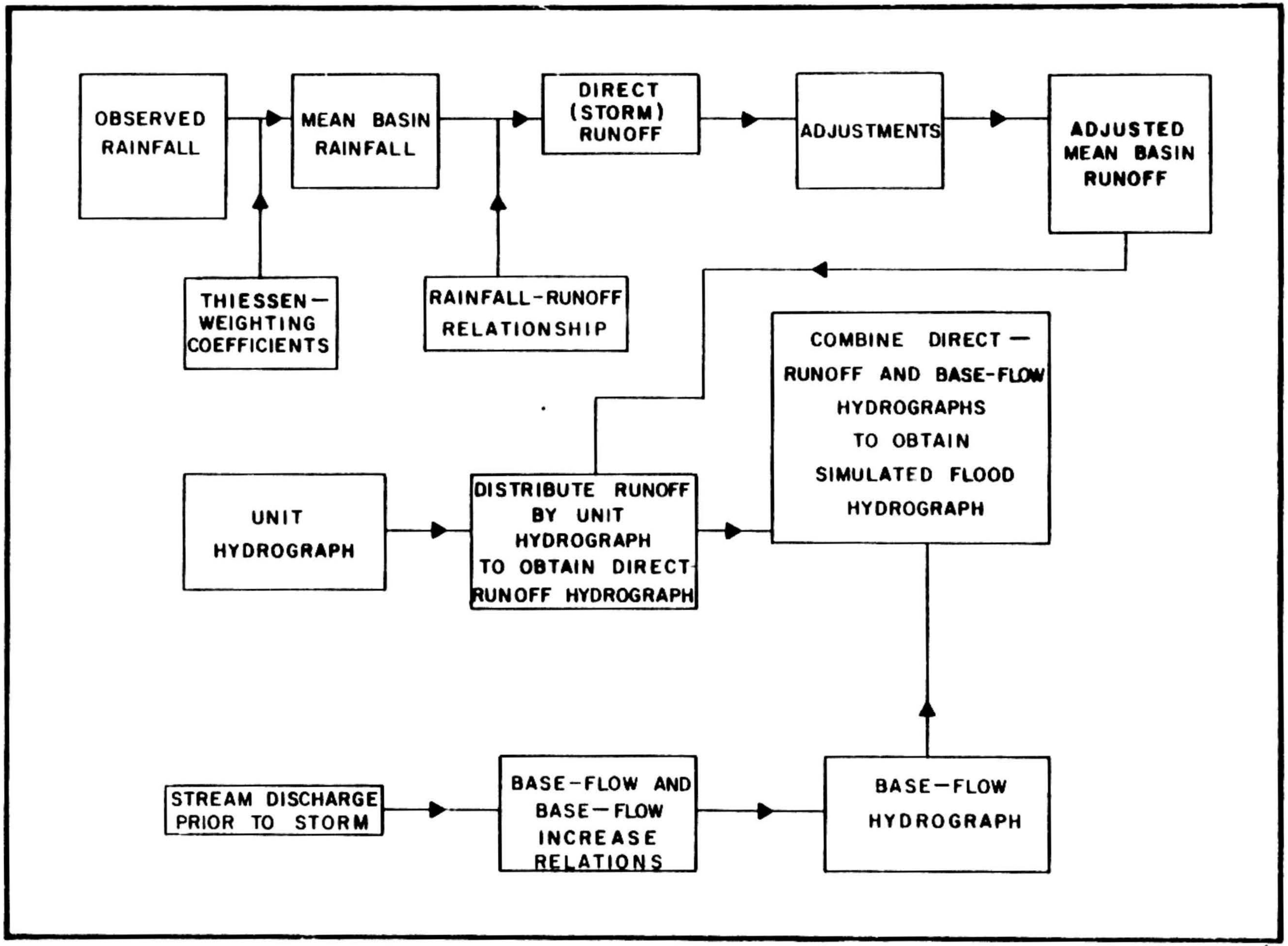


Figure 3.--Generalized flow chart showing primary elements of the hydrograph simulation model.

General Mathematical Statement of Models Developed in this Study

In order to conveniently develop computer programs that will do the arithmetic in hydrograph simulation, the actual computational routines are expressed where possible in analytical form. Because of the nature of the basic format and large volume of data involved in hydrograph simulation, the routines are expressed as a combination of matrices and simple analytical expressions. In general, the basic underlying relations describing the simulation process will be used in generating matrices (data arrays) that describe segments of the computational routines; subsequent manipulation of these matrices allows considerable simplification in describing the actual computational routines.

The general procedure that applies to the simulation of flood hydrographs in this study can be expressed by the following matrix relation, where the matrix of the simulated flood-hydrograph ordinates, Q_t, is given by

$$Q_t = Q_{sr} + Q_{bf} \qquad (1)$$

where Q_{sr} = Matrix of simulated direct-runoff hydrograph ordinates; and

Q_{bf} = Matrix of simulated base-flow hydrograph ordinates.

Direct-Runoff Hydrograph

The storm-runoff matrix, Q_{sr}, is defined by

$$Q_{sr} = UH \times R \qquad (2)$$

where UH = Matrix of the unit-hydrograph ordinates; and

R = Matrix of direct-runoff values.

Although the structure and dimension of the set of matrices given by equations 1 and 2 may vary slightly for each model, the general forms are nearly identical and are combined in the following manner to yield a general statement of the models:

$$Q_t = UH \times R + Q_{bf} \qquad (3)$$

The direct-runoff vector, R, is defined as

$$R = \begin{bmatrix} r_1 \\ r_2 \\ r_3 \\ \cdot \\ \cdot \\ \cdot \\ r_n \end{bmatrix} \qquad (4)$$

where the direct-runoff elements are computed by use of the derived rainfall-runoff relations and subsequent adjustment coefficients.

The storm-runoff matrix, Q_{sr}, is determined by multiplying the unit-hydrograph matrix, UH, by the direct-runoff vector, R, in the following manner:

$$\underset{(UH)}{\begin{bmatrix} U_1 & 0 & 0 & 0 \\ U_2 & U_1 & 0 & . \\ U_3 & U_2 & U_1 & . \\ . & U_3 & U_2 & U_1 \\ . & . & U_3 & U_2 \\ . & . & . & U_3 \\ . & . & . & . \\ U_m & . & . & . \\ 0 & U_m & . & . \\ . & 0 & U_m & . \\ . & . & 0 & . \\ . & . & . & . \\ 0 & 0 & 0 & . \\ 0 & 0 & 0 & U_m \end{bmatrix}} \times \underset{(R)}{\begin{bmatrix} r_1 \\ r_2 \\ r_3 \\ . \\ . \\ . \\ r_n \end{bmatrix}} = \underset{(Q_{sr})}{\begin{bmatrix} (q_{sr})_1 \\ (q_{sr})_2 \\ (q_{sr})_3 \\ . \\ . \\ . \\ (q_{sr})_k \end{bmatrix}}$$

The dimensions of these matrices are as follows:

Matrix	Dimension
UH	kxn
R	nxl
Q_{sr}	kxl

where k is the number of direct-runoff values, n, plus the number of unit-hydrograph ordinates, m, minus 1.

Base-flow Hydrograph

The base-flow vector, Q_{bf}, is defined by

$$Q_{bf} = \begin{bmatrix} (q_{bf})_1 \\ (q_{bf})_2 \\ \cdot \\ \cdot \\ \cdot \\ (q_{bf})_k \end{bmatrix} \qquad (6)$$

where base-flow discharge, $(q_{bf})_i$, is computed by means of an analytical expression based on the derived base-flow increase and recession relationships, and by conditional linkage routines. Because of the variation of base-flow routines between the models, each will be discussed in detail in later sections of the report.

Flood Hydrograph

The total flood-hydrograph vector, Q_t, is formed by the following summation:

$$Q_t = \begin{bmatrix} (q_{sr})_1 \\ (q_{sr})_2 \\ \cdot \\ \cdot \\ \cdot \\ (q_{sr})_k \end{bmatrix} + \begin{bmatrix} (q_{bf})_1 \\ (q_{bf})_2 \\ \cdot \\ \cdot \\ \cdot \\ (q_{bf})_k \end{bmatrix} = \begin{bmatrix} (q_t)_1 \\ (q_t)_2 \\ \cdot \\ \cdot \\ \cdot \\ (q_t)_k \end{bmatrix} \tag{7}$$

Since the models involve the multiplication and addition of two matrices, brief numerical examples are given below to demonstrate the arithmetic procedure.

<u>Multiplication of matrices</u>:

$$\begin{bmatrix} 3 & 4 \\ 1 & 0 \end{bmatrix} \times \begin{bmatrix} 1 \\ 2 \end{bmatrix} = \begin{bmatrix} (3 \times 1 & + & 4 \times 2) \\ (1 \times 1 & + & 0 \times 2) \end{bmatrix} = \begin{bmatrix} 11 \\ 1 \end{bmatrix} \tag{8}$$

<u>Addition of matrices</u>:

$$\begin{bmatrix} 1 \\ 5 \end{bmatrix} + \begin{bmatrix} 7 \\ 0 \end{bmatrix} = \begin{bmatrix} (7 + 1) \\ (5 + 0) \end{bmatrix} = \begin{bmatrix} 8 \\ 5 \end{bmatrix} \tag{9}$$

HILLSBOROUGH RIVER MODEL

The simulation model of the Hillsborough River is nearly the same as the generalized model given by equation 3 except for a modification for runoff adjustment. The matrix of simulated daily flood discharges, Q_t, is given by

$$Q_t = UH \times R_{adj} + Q_{bf} \qquad (10)$$

where UH = Matrix of daily unit-hydrograph ordinates;

R_{adj} = Matrix of adjusted daily direct runoff; and

Q_{bf} = Matrix of daily base-flow discharges.

Runoff hydrograph.--As indicated by equation 10, the runoff hydrograph matrix is formed by multiplying the unit-hydrograph matrix and the adjusted runoff matrix.

The 24-hour unit-hydrograph ordinates, used in forming the unit-hydrograph matrix shown in equation 5, are in the following list and also are shown graphically in figure 4. These ordinates indicate successive daily average rates at which 1 inch of direct runoff (covering the entire drainage area) would be discharged from the Hillsborough River basin at the gaging station.

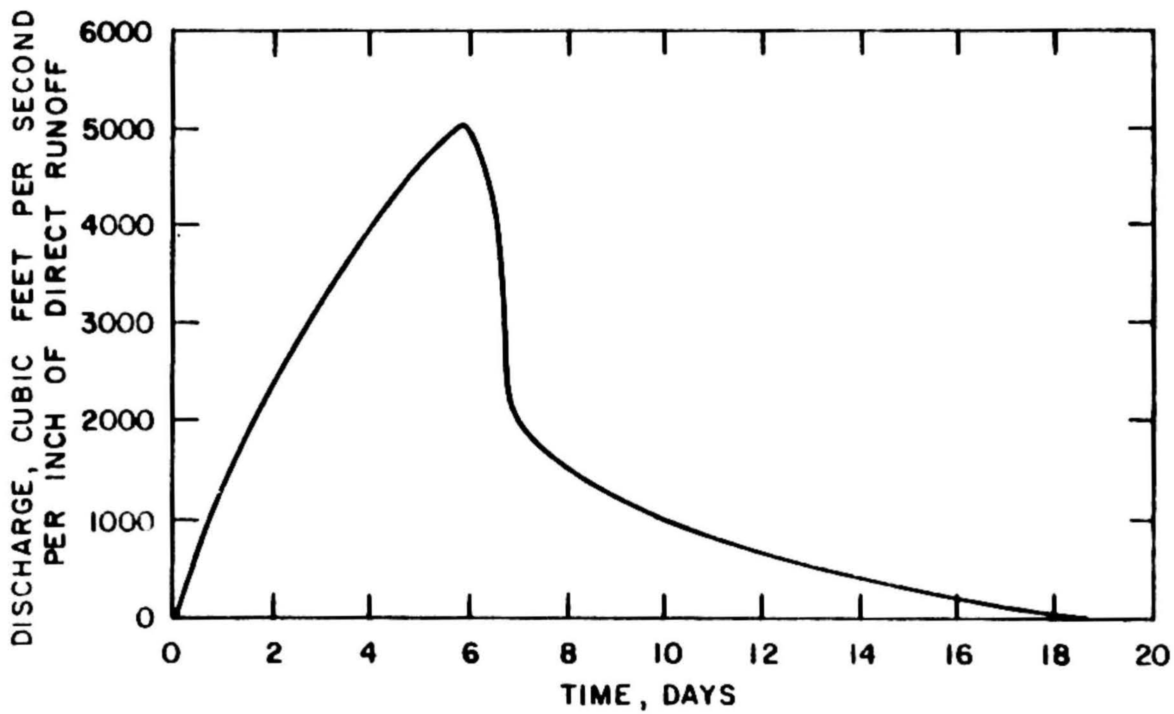


Figure 4.--Graph showing average 24-hour unit hydrograph for the Hillsborough River near Tampa.

Time, in days	Discharge, in cubic feet per second per inch of direct runoff
1	0
2	1,000
3	2,500
4	3,000
5	4,000
6	4,500
7	5,000
8	1,800
9	1,600
10	1,300
11	1,000
12	800
13	600
14	500
15	400
16	300
17	200
18	100
19	0

Daily direct runoff (in inches), which is applied to the unit hydrograph in computing the direct-runoff hydrograph, is determined by use of the rainfall-runoff relation shown in figure 5. The equation for this relation, which is based on 13 storms, is as follows:

$$r = -0.03 + 0.155(\bar{p}) + 0.01(\bar{p})^2; \; (r = 0 \text{ for } \bar{p} \leq 0.2) \qquad (11)$$

where $\bar{p}$ = Mean basin rainfall, in inches.

Mean basin rainfall is computed from daily rainfall observed at Tampa, St. Leo, and Lakeland, by use of the following equation:

$$\bar{p} = 0.17T + 0.32L + 0.51S \qquad (12)$$

where T = Daily rainfall observed at Tampa, in inches;

L = Daily rainfall observed at Lakeland in inches; and

S = Daily rainfall observed at St. Leo, in inches.

The constants shown in equation 12 are Thiessen-weighting coefficients.

To minimize simulation errors that occur as a result of unusual rainfall patterns, daily direct runoff, r, is adjusted by multiplying each value by the appropriate empirical weighting coefficient listed in table 3. These coefficients were derived by trial and error adjustment of runoff from several storms having extremely variable rainfall. For these erratic storms, calibration errors were significantly reduced by application of the adjustment factors.

Additional runoff adjustment that is made prior to actual simulation involves lagging daily direct-runoff values that follow a day on which no direct runoff occurs. Under this condition, a runoff value that is preceeded by a zero value is added to the next daily runoff value and assumes a value of zero in the runoff sequence. This adjustment scheme is used because of the lag in basin response time, or the time between the onset of rainfall and the appearance of direct runoff in the stream.

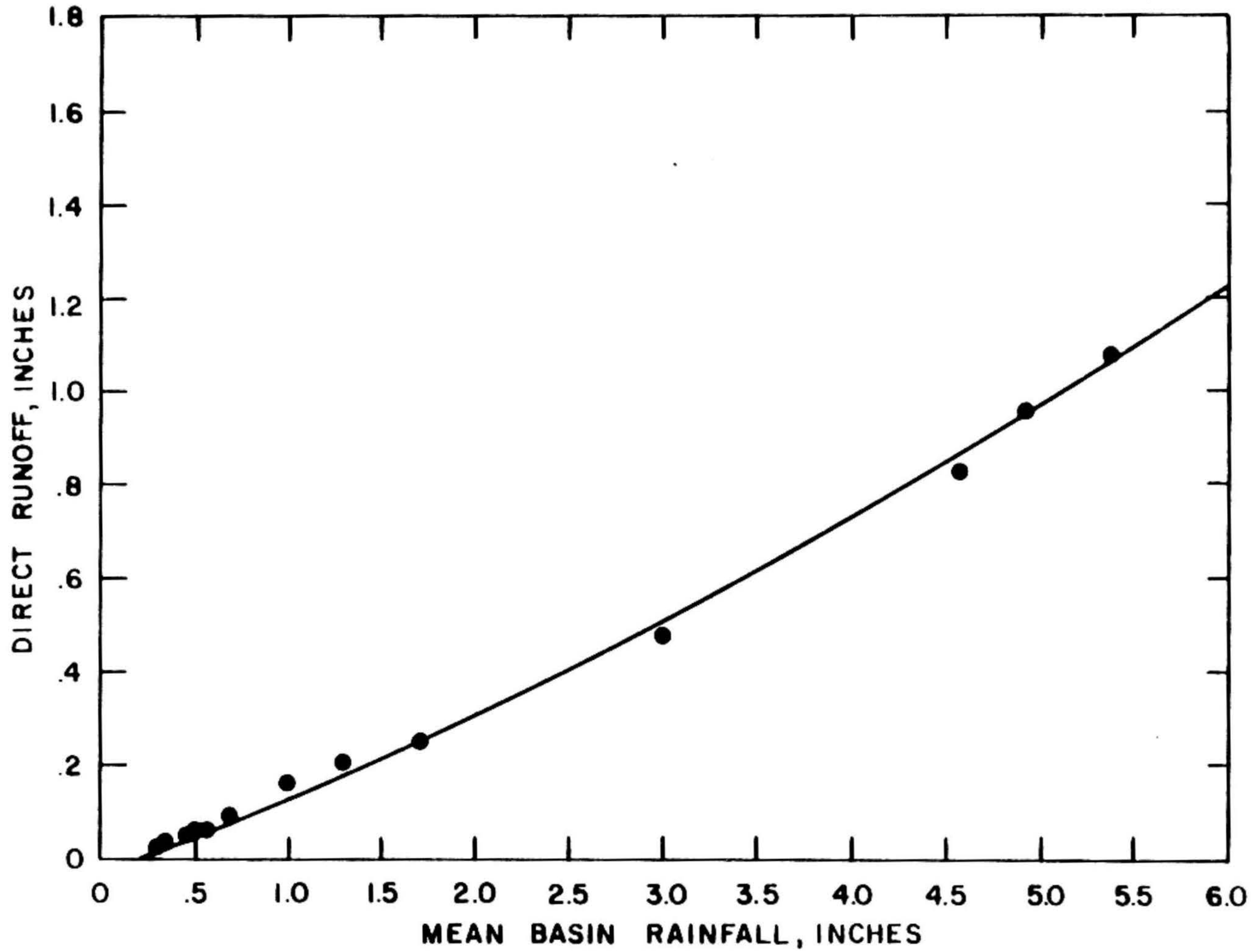


Figure 5.--Graph showing rainfall-runoff relation for the Hillsborough River near Tampa.

Table 3.--Schedule of direct-runoff coefficients for the Hillsborough River near Tampa, Fla.

Variation in basin rainfall	Coefficient
T+L > 6 inches, and S < 1.5 inches	0.49
T+S > 6 inches, and L < 1.5 inches	.68
L+S > 6 inches, and T < 1.5 inches	.83
T+L = 0, and S > 3 inches	.51
T+S = 0, and L > 3 inches	.32
L+S = 0, and T > 3 inches	.17

T = daily rainfall observed at Tampa;

L = daily rainfall observed at Lakeland;

S = daily rainfall observed at St. Leo.

Base-flow hydrograph.--The simulated base-flow hydrograph is computed using a multiple relation involving the recession characteristics of the stream. This multiple relation is based on an average recession curve and base-flow increase.

The average recession curve developed for the Hillsborough River is shown in figure 6. The equation of this relation, which gives recession discharge, q_i, in cubic feet per second, is as follows:

$$q_i = \left[85 + 7900(10)^{-0.036t}\right] \qquad (13)$$

where t = Time, in days.

Base-flow increase is obtained from the graph of base-flow increase versus peak runoff, shown in figure 7. The equation of this relation, which gives daily base-flow increase, d, in cubic feet per second, is as follows:

$$d = 15 \left[1 - (10)^{0.00019s}\right] + 0.561s \qquad (14)$$

where s = Direct peak runoff, in cubic feet per second.

However, in the simulation process, direct peak runoff, s (see fig. 1), is computed by multiplying the maximum unit-hydrograph ordinate (5,000) by each daily direct-runoff value (see col. 15, table 4); therefore, equation 14 can be reduced to

$$d = 15 \left[1 - (10)^{0.95r'}\right] + 2{,}800r' \qquad (15)$$

where r' = Adjusted daily direct runoff, in inches.

By combining equations 13 and 15, this multiple relation can now be expressed as,

$$(q_{bf})_i = \underbrace{\left[85 + 7900(10)^{-0.036t}\right]}_{\text{(recession)}} + \underbrace{\left\{15 \left[1-(10)^{0.95r'}\right] + 2800r'\right\}}_{\text{(increase)}} \qquad (16)$$

where t = Time in days; and

r' = Adjusted daily direct runoff, in inches. For $i < 8$, $r' = 0$.

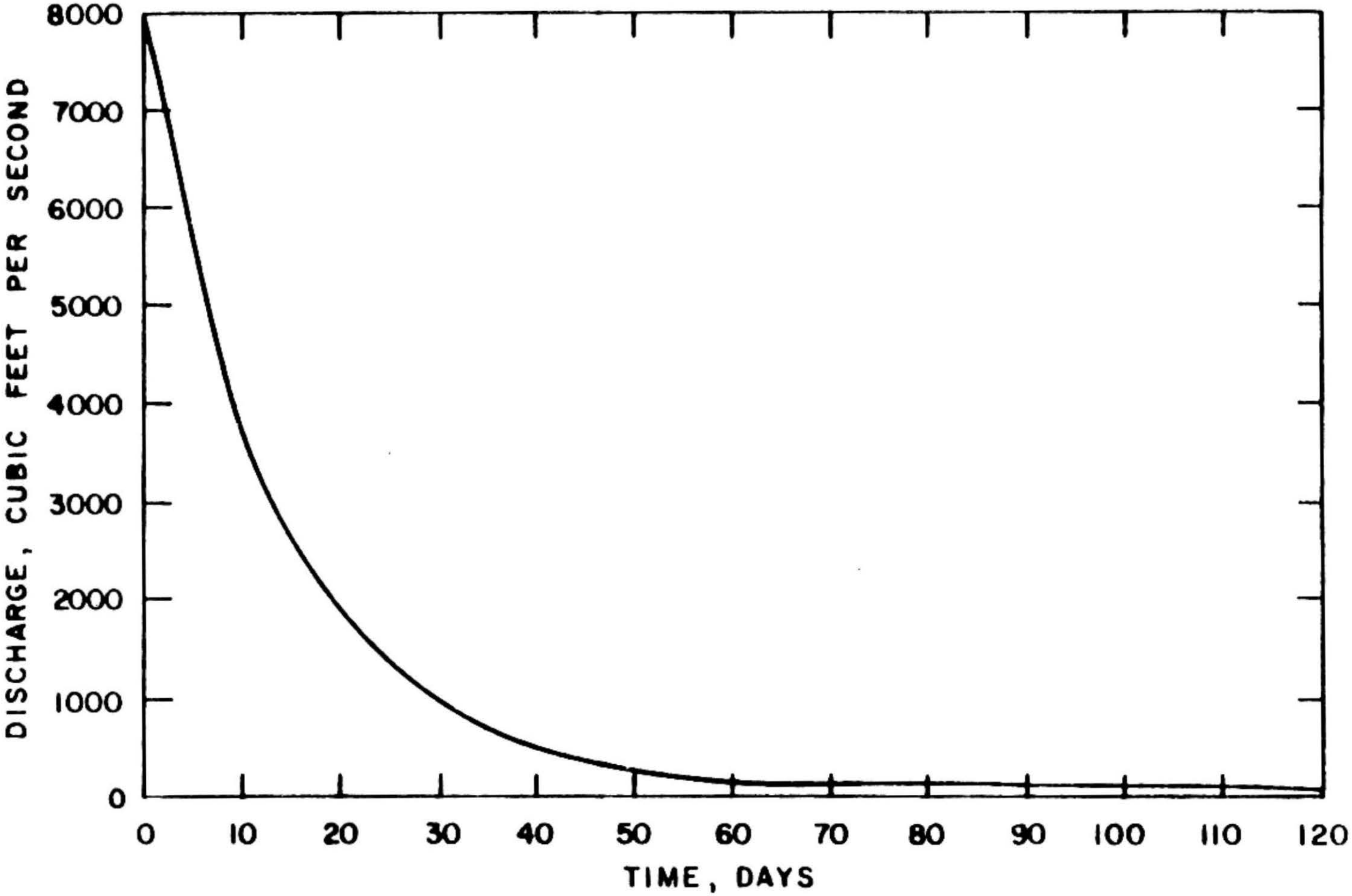


Figure 6.--Graph showing average base-flow recession curve for the Hillsborough River near Tampa.

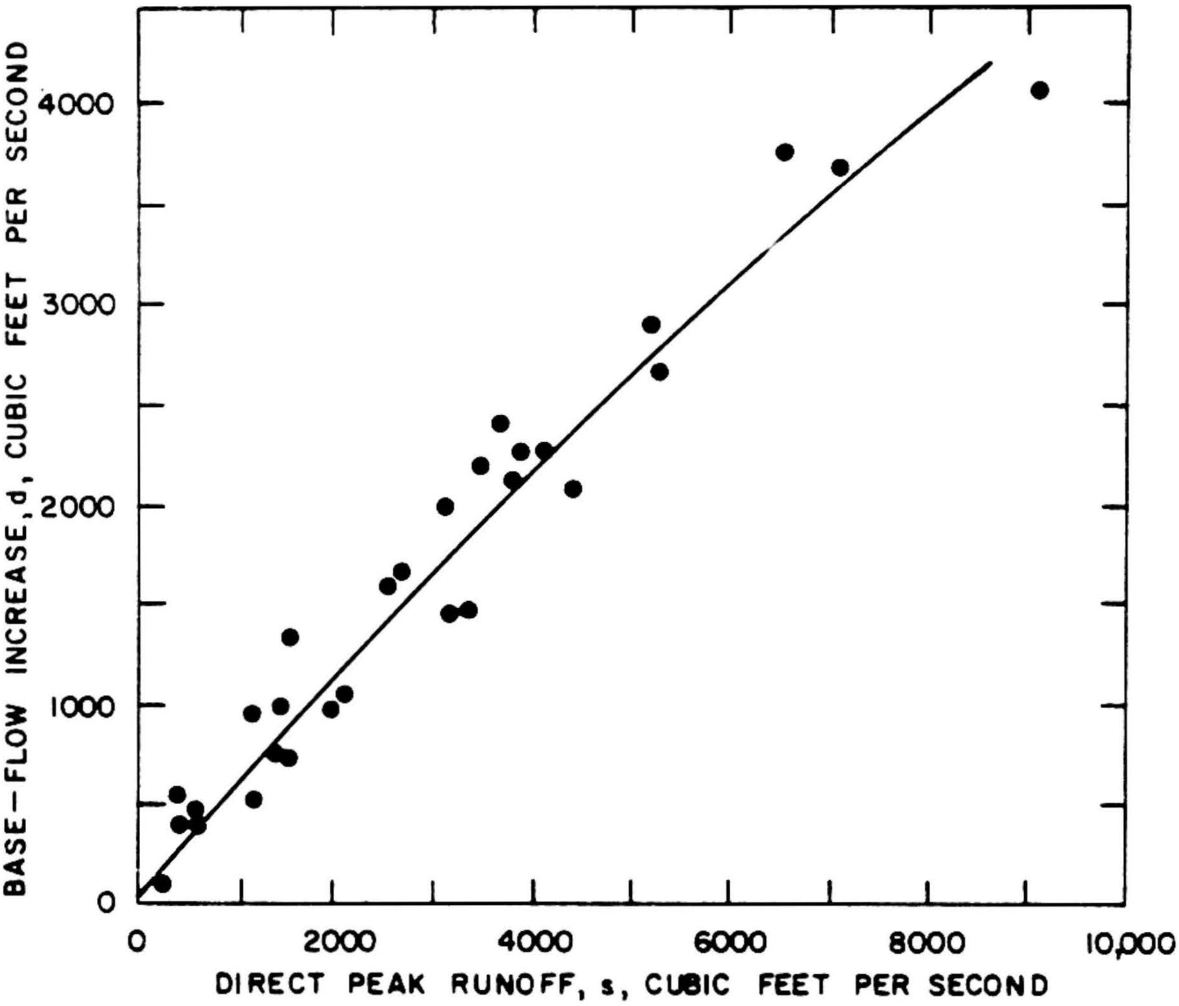


Figure 7.--Graph showing base-flow increase relation for Hillsborough River near Tampa.

As indicated by equation 16, daily base-flow discharges are computed by the recession part of equation 16 for the first seven days after precipitation began. On the eighth day (and on the eighth day following each of the days during which runoff occurred) daily base-flow discharge is computed by use of both parts of equation 16.

The entire base-flow generation process is time dependent. Therefore, whenever base-flow increases occur, entry to the recession curve is determined by a new time index computed by

$$t = -27.77 \log_{10} \left[\frac{(q_{bf})_i - 85}{7900} \right] \qquad (17)$$

where $(q_{bf})_i$ = Base-flow discharge, in cubic feet per second.

The initial time index is also determined by use of equation 17, substituting initial base flow for $(q_{bf})_i$. Equation 17 is the inverse of equation 16, when adjusted daily direct runoff is equal to zero.

Numerical example.--A simplified numerical example of the computational procedure that is used in synthesizing the flood hydrograph (January 13 through February 12, 1948) for the Hillsborough River near Tampa is shown in table 4. Daily rainfall (columns 1, 2, and 3 of table 4) is used with equation 12 to compute daily mean rainfall (column 4) for the Hillsborough River basin. Daily mean rainfall is then used with equation 11 to compute daily direct runoff and is adjusted according to the schedule of direct-runoff coefficients listed in table 3. Each daily direct-runoff value that is preceded by a day on which no runoff occurs is added to the next daily runoff value. Adjusted daily direct runoff is listed in column 5.

Each adjusted daily direct runoff is then applied to each ordinate of the 24-hour unit hydrograph (column 6), and their products listed (columns 7-13). Positioning of the first non-zero product of these columns corresponds to the date on which runoff begins. For example, daily direct runoff is zero for January 13 and 0.19 inch for January 14. Each ordinate of the unit hydrograph listed in column 6 is multiplied by 0.19 inch and listed in column 7 beginning on January 13. Columns 8-13 are computed in a similar manner.

The ordinates of the direct-runoff hydrograph (column 14) are row-by-row summations of the values listed in columns 7-15. For example, direct runoff for January 24 (column 14) is computed in the following manner:

$$(152 + 342 + 50 + 135 + 120 + 420) = 1,219 \text{ cfs} \tag{18}$$

Daily direct peak runoff (column 15) is the maximum value in each of the columns 7-13. These peak values are used in computing base-flow increase (column 16) by use of equation 15 or the Hillsborough base-flow increase relation shown in figure 7.

Table 4. Numerical example showing computational procedure for synthesizing a flood hydrograph for the Hillsborough River near Tampa, Fla.

Date	Observed rainfall in inches: Tampa	Observed rainfall in inches: Lakeland	Observed rainfall in inches: St. Leo	Mean basin rainfall, inches per day	Adjusted daily direct runoff, inches per day	Ordinates of 24-hour unit hydrograph, cfs per inch of runoff	Computation of the direct runoff hydrograph (Each column is the product of adjusted daily direct runoff (> 0) and ordinates of the 24-hour unit hydrograph. Units are in cfs.)							Daily direct runoff, cfs	Computation of the base-flow hydrograph: Direct peak runoff, cfs	Base-flow increase, cfs	Daily base-flow, cfs	Simulated flood hydrograph: Daily discharge, cfs (Column 14 plus column 17)
Col. No.	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
Jan. 1948																		
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	(229) +	229
13	1.18	2.23	0.87	1.35	0	1000	0	0	0	0	0	0	0	0	0	0	219	219
14	0	0	0	0	0.19	2500	190	0	0	0	0	0	0	190	0	0	209	399
15	0	0	0	0	0	3000	475	0	0	0	0	0	0	475	0	0	199	[illegible]
16	0	0	0	0	0	4000	570	0	0	0	0	0	0	570	0	0	189	[illegible]
17	.00	1.35	.85	1.20	0	4500	760	0	0	0	0	0	0	760	0	0	184	[illegible]
18	0	.03	.78	.41	.19	5000	855	190	0	0	0	0	0	1045	0	0	179	1224
19	0	0	.54	.28	.01	1800	950*	475	10	0	0	0	0	1435	956	520	174	1609
20	1.12	.67	0	.40	.03	1600	342	570	25	30	0	0	0	967	0	0	694	1661
21	.09	1.14	0	.38	.03	1300	304	760	30	75	30	0	0	1199	0	0	654	1853
22	0	0	0	0	0	1000	247	855	40	90	75	0	0	1307	0	0	614	1921
23	.85	.41	0	.27	0	800	190	950*	45	120	90	0	0	1395	950	520	584	1979
24	2.10	2.10	2.75	2.43	.42	600	152	342	50*	135	120	420	0	1219	50	30	1104	2323
25	0	0	0	0	0	500	114	304	18	150*	135	1050	0	1771	150	80	1134	2905
26	0	0	0	0	0	400	95	247	16	54	150*	1260	0	1822	150	80	1214	3036
27	0	0	0	0	0	300	76	190	13	48	54	1680	0	2061	0	0	1294	3355
28	0	.01	0	0	0	200	57	152	10	39	48	1890	0	2196	0	0	1194	3390
29	0	0	0	0	0	100	38	114	8	30	39	2100*	0	2325	2100	1150	1104	3429
30	.04	0	0	0	0	0	19	95	6	24	30	756	0	930	0	0	2254	3184
31	.30	.57	.18	.32	0	----	0	76	5	18	24	672	0	795	0	0	2104	2899
Feb. 1	----	----	----	----	.01	----	0	57	4	15	18	546	10	650	0	0	1974	2624
2	----	----	----	----	----	----	0	38	3	12	15	420	25	513	0	0	1844	2357
3	----	----	----	----	----	----	0	19	2	9	12	316	30	408	0	0	1724	2132
4	----	----	----	----	----	----	0	0	1	6	9	252	40	308	0	0	1614	1922
5	----	----	----	----	----	----	0	0	0	3	6	210	45	264	0	0	1504	1768
6	----	----	----	----	----	----	0	0	0	0	3	168	50*	221	50	30	1394	1615
7	----	----	----	----	----	----	0	0	0	0	0	126	18	144	0	0	1424	1568
8	----	----	----	----	----	----	0	0	0	0	0	84	16	100	0	0	1324	1424
9	----	----	----	----	----	----	0	0	0	0	0	42	13	55	0	0	1224	1279
10	----	----	----	----	----	----	0	0	0	0	0	0	10	10	0	0	1134	1144
11	----	----	----	----	----	----	0	0	0	0	0	0	8	8	0	0	1057	1065
12	----	----	----	----	----	----	0	0	0	0	0	0	6	6	0	0	984	990

+ Initial Baseflow

* peak runoff value used in computing column (17)

Ordinates of the base-flow hydrograph (column 17) are computed by use of the Hillsborough base-flow recession curve shown in figure 6, an initial base flow of 229 cfs, and the base-flow increase values (column 16). The reason 229 cfs is used as the initial base flow is that several days prior to the storm the discharge of the Hillsborough River averaged about 229 cfs. This initial value is shown for January 12 in column 17. The next daily base-flow value, 219 cfs, is determined by entering the curve shown in figure 6 for a time value 24 hours greater than the time value corresponding to initial base flow. Subsequent base-flow values are obtained in the same manner, provided no adjustment is required for base-flow increase. When a base-flow increase occurs, it is added to the base-flow on the same day to form the base flow for the next day.

Ordinates of the simulated flood hydrograph (column 18 of table 4) are computed by adding the direct-runoff hydrograph ordinates (column 14) to the base-flow hydrograph ordinates (column 17).

Computer Processing

The Hillsborough River model has been programmed in FORTRAN IV-G and in basic machine language for processing on a WANG system owned by the U. S. Geological Survey (Tampa). Both programs have been completed and are operational. A complete listing of the FORTRAN program is given in the following section. However, because processing of the WANG program is done locally, a listing of that program is not given, except for output format. Except for convenience of processing the WANG program, no preference is given for using either the FOR .AN or WANG program.

Format and sequence of data input cards.--Three different types of data cards are required as input to the program. The first shows the number of days of rainfall used in simulation; the second shows base flow on day prior to the storm; and the third shows the date and daily rainfall amounts observed at Tampa, Lakeland, and St. Leo.

The program will only accept storms of at least 10 days in duration and base-flow discharges less than 1,000 cfs. However, flood peaks resulting from storms of less than 10 days in duration can be computed by adjusting the data in the following manner. Card 1 should indicate 10 days, and additional rainfall data cards (needed to compute 10 days of record) should be punched showing zero rainfall for all rainfall stations.

When discharge (prior to the storm) at Tampa Reservoir cannot be determined, an arbitrary value of 850 cfs should be used. An initial discharge value that exceeds 1,000 cfs must be punched on card 3 as having a value of 999 cfs.

```
0001          INTEGER IARY(4)
0002          DIMENSION UH(19),AUH(30,50),MODAY(30,6),TAM(30),ALAK(30),STL(30),
             1WP(30),RUNOF(30),ARUNO(30),BASE(50),CBASE(50)
0003          DATA IARY/'B@','E@','G@','I@'/
0004        10 FORMAT(6A1,3F6.0)
0005        14 FORMAT(1X,6A1,25X,F5.2,42X,I6)
0006       144 FORMAT(1X,6H------,26X,4H----,42X,I6)
0007       300 FORMAT(1H0,17X,99H SIMULATED FLOOD HYDROGRAPH FOR THE HILLSBOROUGH
             1 RIVER (AT TAMPA RESERVOIR DAM) NEAR TAMPA, FLORIDA)
0008       301 FORMAT(1H ,43X,55H (AT US GEOLOGICAL SURVEY STREAM GAGING STATION
             12-3045)////)
0009       302 FORMAT(1H ,100HDATE             MEAN BASIN RAINFALL, INCHES PER DAY
             1                      SIMULATED DAILY DISCHARGE, CFS//)
        C
        C
        C     STL # ST LEO
        C     ALAK # LAKELAND
        C     TAM # TAMPA
        C     MODAY # MONTH DAY YEAR
              BASEX # FLOW ON DAY PREVIOUS TO INITIAL RAINFALL DATA
        C     CBASE # CHANGE IN BASE
        C     BASE # BASE FLOW AFTER THE INITIAL BASE
        C     N # NUMBER OF RAINFALL DAYS
        C     NU # NUMBER OF DAY FOR WHICH FLOW IS
        C     DETERMINED - 19 DAYS PAST LAST RAINFALL ENTRY
        C     UH # UNIT HYDROGRAPH
        C     AUH # ADJUSTED UNIT HYDROGRAPH
        C     NAUH # NUMBER OF THE SUBSCRIPTED ARRAY AT WHICH AUH STARTS
        C     RUNOF # FACTOR DETERMINED BASED ON AMOUNT OF RAINFALL
        C     ARUNOF # RUNOFF ADJUSTED BASED ON RAINFALL DISTRIBUTION
        C     TRUNO # SUMMATION OF AUH'S FOR A GIVEN DAY
        C     FROM THE FAMILY OF AUH CURVES
        C     TOTQ # TOTAL FLOW EQUAL TO TRUNO & BASED ON A GIVEN DAY
        C
        C
        C
        C     THIS IS THE UNIT HYDROGRAPH FOR HILLSBOROUGH RIVER
        C
        C
        C     DATSW 1 ON  - TAPE INPUT
        C             OFF - CARD INPUT
        C
0010          IRE=1
0011          IWR=3
0012        3 DO 797 I=1,6
0013          DO 797 J=1,30
0014      797 MODAY(J,I)=16448
0015          UH(1) = 0.0
0016          UH(2) = 1000.0
0017          UH(3) = 2500.0
0018          UH(4) = 3000.0
0019          UH(5) = 4000.0
0020          UH(6) = 4500.0
0021          UH(7) = 5000.0
0022          UH(8) = 1800.0
0023          UH(9) = 1600.0
0024          UH(10) = 1300.0
0025          UH(11) = 1000.0
```

```
              UH(12) = 800.0
              UH(13) = 600.0
              UH(14) = 500.0
              UH(15) = 400.0
              UH(16) = 300.0
              UH(17) = 200.0
              UH(18) = 100.0
              UH(19) = 0.0
              READ(IRE,8)N
            8 FORMAT(I3)
              READ(IRE,5656)BASEX
         5656 FORMAT(F10.0)
              NU = N+19
      C
      C       THIS IS THE READ DATA STATEMENT AND AND
      C       COMPUTATION OF WEIGTHED PRECIPITATION
      C       RUNOFF FACTOR DETERMINATION
      C
              DO 50 I=1,N
              READ(IRE,10)(MODAY(I,J),J=1,6),TAM(I),ALAK(I),STL(I)
              WP(I) = .51*STL(I)+.32*ALAK(I)+.17*TAM(I)
              IF(WP(I)-.2)48,209,209
          209 RUNOF(I) = ((WP(I)/5.08)**1.2)
      C
      C       THE FOLLOWING SELECTS THE ADJUSTMENT OF THE RUNOFF
      C       DEPENDING ON THE RAINFALL DISTRIBUTION
      C
              S1 = STL(I)+TAM(I)
              S2 = STL(I)+ALAK(I)
              S3 = TAM(I)+ALAK(I)
              IF(S1-6.)199,30,30
          199 IF(S2-6.)201,31,31
          201 IF(S3-6.)202,32,32
          202 IF(STL(I)-3.)203,33,33
          203 IF(ALAK(I)-3.)204,35,35
          204 IF(TAM(I)-3.)40,36,36
           30 IF(ALAK(I)-1.5)46,40,40
           31 IF(TAM(I)-1.5)41,40,40
           32 IF(STL(I)-1.5)42,40,40
           33 IF(ALAK(I)-1.5)43,34,34
           34 IF(TAM(I)-1.5)43,40,40
           35 IF(TAM(I)-1.5)44,40,40
           36 IF(ALAK(I)-1.5)45,38,38
           38 IF(STL(I)-1.5)45,40,40
           40 ARUNO (I) = RUNOF(I)
              GO TO 50
           41 ARUNO (I) = RUNOF(I)*.83
              GO TO 50
           42 ARUNO (I) = RUNOF(I)*.49
              GO TO 50
           43 ARUNO (I) = RUNOF(I)*.51
              GO TO 50
           44 ARUNO (I) = RUNOF(I)*.32
              GO TO 50
           45 ARUNO (I) = RUNOF(I)*.17
              GO TO 50
           46 ARUNO (I) = RUNOF(I)*.68
              GO TO 50
```

```
      48 ARUNO (I) = 0.0
         RUNOF(I) = 0.0
      50 CONTINUE
C
C        THE FOLLOWING IS THE INITIAL BASE ADJUSTMENT FROM THE INITIAL
C        BASE FLOW ENTRY
C
         DO 71 K=1,NU
         IF(BASEX-700.)61,67,67
      61 IF(BASEX-246.)62,68,68
      62 IF(BASEX-160.)65,65,69
      65 BASE(K) = BASEX*.985
         GO TO 70
      67 BASE(K) = BASEX*.935
         GO TO 70
      68 BASE(K) = BASEX*.950
         GO TO 70
      69 BASE(K) = BASEX*.970
      70 CBASE(K) = 0.0
         BASEX = BASE(K)
      71 CONTINUE
C
C        THE FOLLOWING IS THE ADJUSTMENT OF THE UNIT HYDROGRAPH
C        DEPENDING ON THE WEIGHTED PRECIPITATION
C
         DO 225 J=1,N
         NAUH = 0
         NUH = 0
         DO 224 I=1,NU
         IF(J-3)205,90,90
     205 IF(J-2)206,93,93
     206 IF(J-1)207,95,95
     207 DUMMY = 1.0
      90 IF(WP(J)-.2)220,91,91
      91 IF(WP(J-1)-.2)100,98,98
      93 IF(WP(J)-.2)220,94,94
      94 IF(WP(J-1)-.2)100,98,98
      95 IF(WP(J)-.2)220,100,100
      98 IF(I-2)99,100,100
      99 NAUH = NAUH+2
         GO TO 101
     100 NAUH = NAUH+1
     101 IF(NAUH-J)220,103,103
     103 NUH = NUH+1
         IF(NUH-19)208,208,220
     208 AUH(J,I) = UH(NUH)*ARUNO (J)
C
C        THE FOLLOWING ADJUSTS THE INITIAL BASE FLOW
C        BASED ON A PEAK OCCURING 8 DAYS AFTER RAINFALL
C
         IF(NUH-8)224,150,224
     150 KO = I
         M = I
         DO 200 K=KO,NU
         IF(K-M)151,150,151
     151 M = K
         IF(BASE(M-1)-700.)152,158,158
     152 IF(BASE(M-1)-246.)153,159,159
```

```
      153 IF(BASE(M-1)-160.)154,160,160
      154 BASE(K) = BASE(M-1)*.985
          GO TO 165
      158 BASE(K) = BASE(M-1)*.935
          GO TO 165
      159 BASE(K) = BASE(M-1)*.950
          GO TO 165
      160 BASE(K) = BASE(M-1)*.970
      165 CBASE(K) = 0.0
          GO TO 200
      190 M = M-1
          IF(AUH(J,M)-4500.)191,193,193
      191 CBASE(K) = CBASE(K)+.55*AUH(J,M)
          GO TO 195
      193 CBASE(K) = CBASE(K)+26.4*((AUH(J,M)-1870.)**.576)
      195 BASE(K) = BASE(M)+CBASE(K)
      200 CONTINUE
          GO TO 224
      220 AUH(J,I) = 0.0
      224 CONTINUE
      225 CONTINUE
          WRITE(IWR,300)
          WRITE(IWR,301)
          WRITE(IWR,302)
C
C
C         THE FOLLOWING CALCULATES THE TOTAL FLOW WITH ALL BASE ADJUSTMENTS
C         AND PRINTS OUT THE INITIAL RAINFALL DATA
C
          J = 1
     5065 TRUNO = 0.0
          TOTQ = 0.0
          DO 500 I=1,N
          TRUNO = AUH(I,J)+TRUNO
      500 CONTINUE
          TOTQ = BASE(J)+TRUNO
          ITOTQ = TOTQ
          IF(J-N)800,800,801
      800 WRITE(IWR,14)(MODAY(J,I),I=1,6),WP(J),ITOTQ
          GO TO 810
      801 WRITE(IWR,144)ITOTQ
      810 J = J+1
          IF(J-NU)5065,5065,805
      805 CALL EXIT
          END
```

Data-card formats are shown in the following list:

Data Card	Cols.	Data Punched
1	2-3	Total number of days of rainfall;
2	1-3	Daily mean discharge observed at Tampa Dam on day prior to day of initial rainfall, in cfs;
3	1-2	Month (must show 2 digits);
	3-4	Day (must show 2 digits);
	5-6	Year (must show 2 digits);
	9-12	24-hour rainfall observed at Tampa, in inches;
	15-18	24-hour rainfall observed at Lakeland, in inches;
	21-24	24-hour rainfall observed at St. Leo, in inches.

Program output.--Rainfall data for August 1-30, 1967 were processed through the Hillsborough River program, and the computer output follows:

(FORTRAN PROGRAM OUTPUT)

SIMULATED FLOOD HYDROGRAPH FOR THE HILLSBOROUGH RIVER (AT TAMPA RESERVOIR DAM) NEAR TAMPA, FLORIDA
(AT US GEOLOGICAL SURVEY STREAM GAGING STATION 2-3045)

DATE	MEAN BASIN RAINFALL, INCHES PER DAY	SIMULATED DAILY DISCHARGE, CFS
080167	0.07	315
080267	0.0	299
080367	0.94	284
080467	0.0	402
080567	0.11	587
080667	0.25	640
080767	0.83	905
080867	0.80	1284
080967	0.34	1615
081067	0.01	1806
081167	0.12	1951
081267	1.52	2047
081367	1.82	2349
081467	0.67	2851
081567	0.51	3105
081667	0.32	3431
081767	0.40	3651
081867	0.54	3931
081967	0.03	3961
082067	0.59	3939
082167	0.84	4051
082267	0.33	4249
082367	0.21	4288
082467	0.04	4349
082567	0.04	4194
082667	0.76	4059
082767	0.22	4045
082867	0.26	4144
082967	0.14	4105
083067	0.0	3905
------	----	3718
------	----	3543
------	----	3454
------	----	3377
------	----	3106
------	----	2851
------	----	2621
------	----	2405
------	----	2232
------	----	2074
------	----	1928
------	----	1790
------	----	1661
------	----	1538
------	----	1436
------	----	1342
------	----	1255
------	----	1173
------	----	1097

Output from WANG program.--Although the input data are identical, program output formats for the WANG and FORTRAN programs are different. A copy of the WANG program output shows not only simulated discharge at Tampa Reservoir Dam for the period March 15 to April 11, 1960 but also expected stage at Fowler Avenue (data point 3, fig. 1; see also table 1). Even though the stage-discharge rating curve for the Fowler Avenue gage is provisional, the projected stage data serve as a general guide to expected flood elevations in this reach of the Hillsborough River. For a projected daily discharge of 5,000 cfs or less, error in stage should be 1 foot or less. Errors in projected stage would be larger for discharges less than 5,000 cfs, and smaller for discharges greater than 5,000 cfs. Additional errors could result from unknown rating shifts that may occur.

(WANG PROGRAM OUTPUT)

SIMULATED FLOOD HYDROGRAPH FOR HILLSBOROUGH RIVER NEAR TAMPA FLA

STORM BEGINNING ON 3-15-60

DAYS	RUNOFF (INCHES)	DISCHARGE (CFS)	STAGE (FT,MSL)
1	.00	710	19.33
2	1.87	2539	24.59
3	.44	5754	28.71
4	.06	7372	30.08
5	.00	9524	31.57
6	.00	10898	32.39
7	.00	12086	33.03
8	.00	10684	32.27
9	.00	10137	31.95
10	.00	9459	31.53
11	.00	8264	30.74
12	.00	7290	30.02
13	.00	6396	29.29
14	.00	5728	28.68
15	.00	5135	28.10
16	.00	4575	27.49
17	.00	4041	26.85
18	.00	3531	26.18
19	.00	3144	25.61
20	.00	2785	25.03
21	.00	2546	24.61
22	.00	2347	24.23
23	.00	2167	23.87
24	.00	2001	23.51
25	.00	1849	23.16
26	.00	1709	22.82
27	.00	1579	22.48

Note: (a) Days are numbered consecutively, beginning with the first day of precipitation;

(b) Stage refers to the expected water elevation at the Fowler Avenue gage.

Simulation Errors

Errors inherent in simulated flood flows obtained from the Hillsborough River model consist primarily of model errors and sample errors associated with rainfall data. Since errors in the rainfall data are not practical to assess on the basis of available information, these two error components cannot be conveniently separated and will be considered jointly in a discussion of simulation errors.

In demonstrating the range of errors to be expected in data simulated by means of the Hillsborough River model, many storms (other than those used in developing the model) should be processed through the model and compared with actual flood data. However, since independent storms are so few, 13 storms used in developing the model were reconstructed and compared with observed discharges during these storm periods. Even though a comparison of simulated and observed storms used in developing the model may be biased, the comparison of these storms serves as a general indication of errors to be expected in other floods simulated by the model.

Flood hydrographs of all the storms used in developing the Hillsborough River model have been simulated and are shown along with the actual flood hydrographs in figures 8, 9, 10 and 11.

To illustrate the difference between simulated and observed hydrographs, a relative error was computed for each simulated flood discharge. Relative errors were calculated at each day on the hydrographs by taking the difference in observed and simulated discharge and dividing this difference by observed discharge. These calculated relative errors

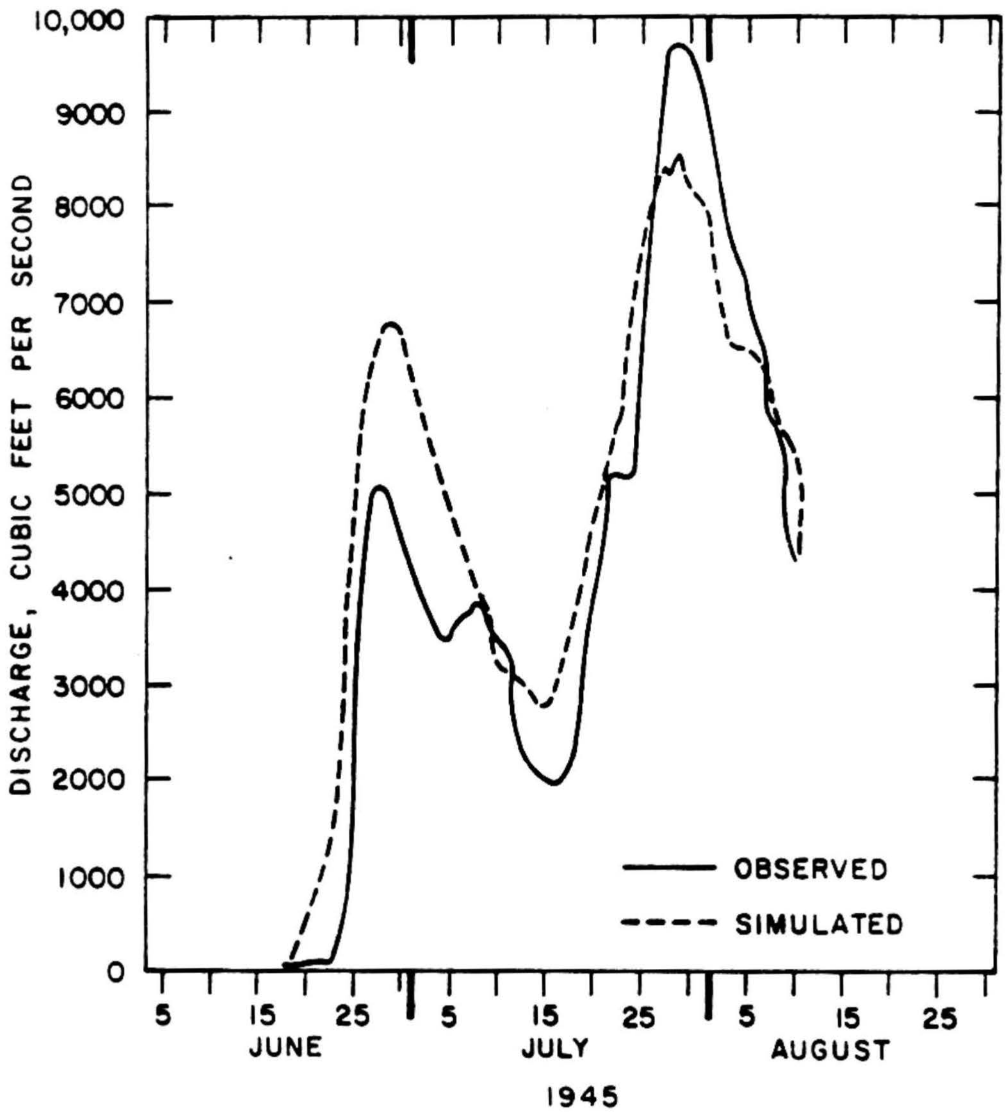


Figure 8.--Graph showing simulated and observed flood hydrographs for the Hillsborough River near Tampa.

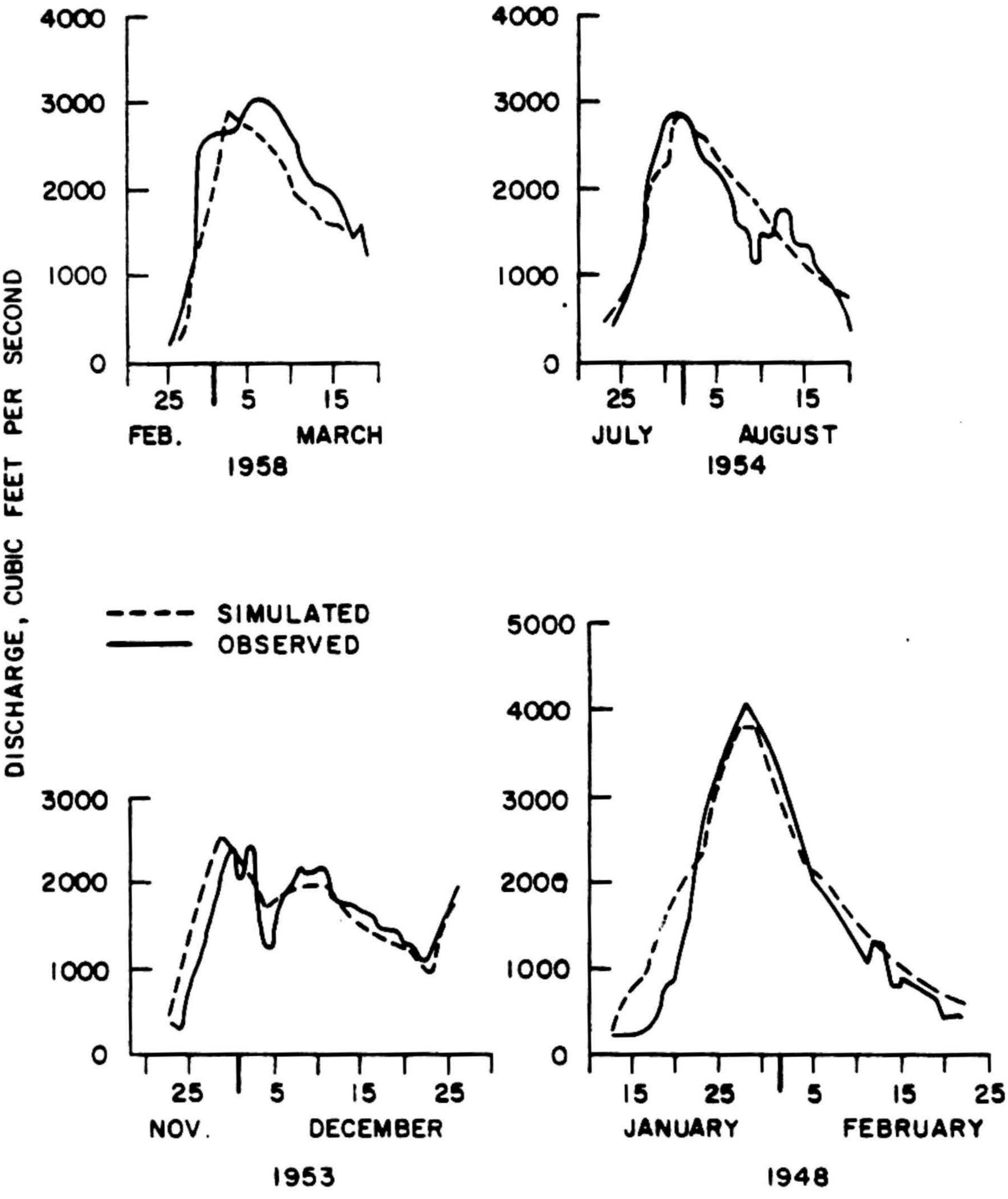


Figure 9.--Graphs showing simulated and observed flood hydrographs for the Hillsborough River near Tampa.

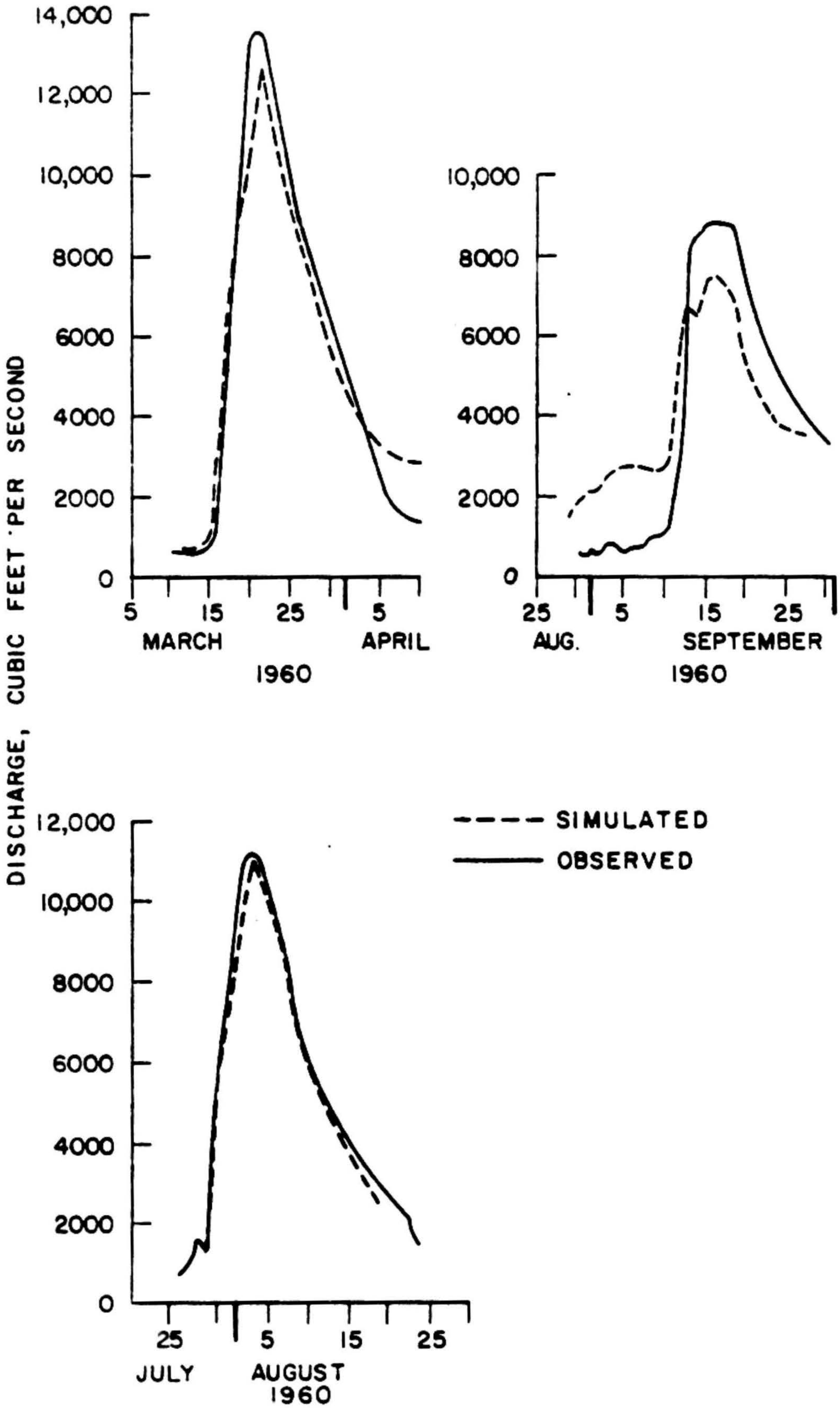


Figure 10.--Graphs showing simulated and observed flood hydrographs for the Hillsborough River near Tampa.

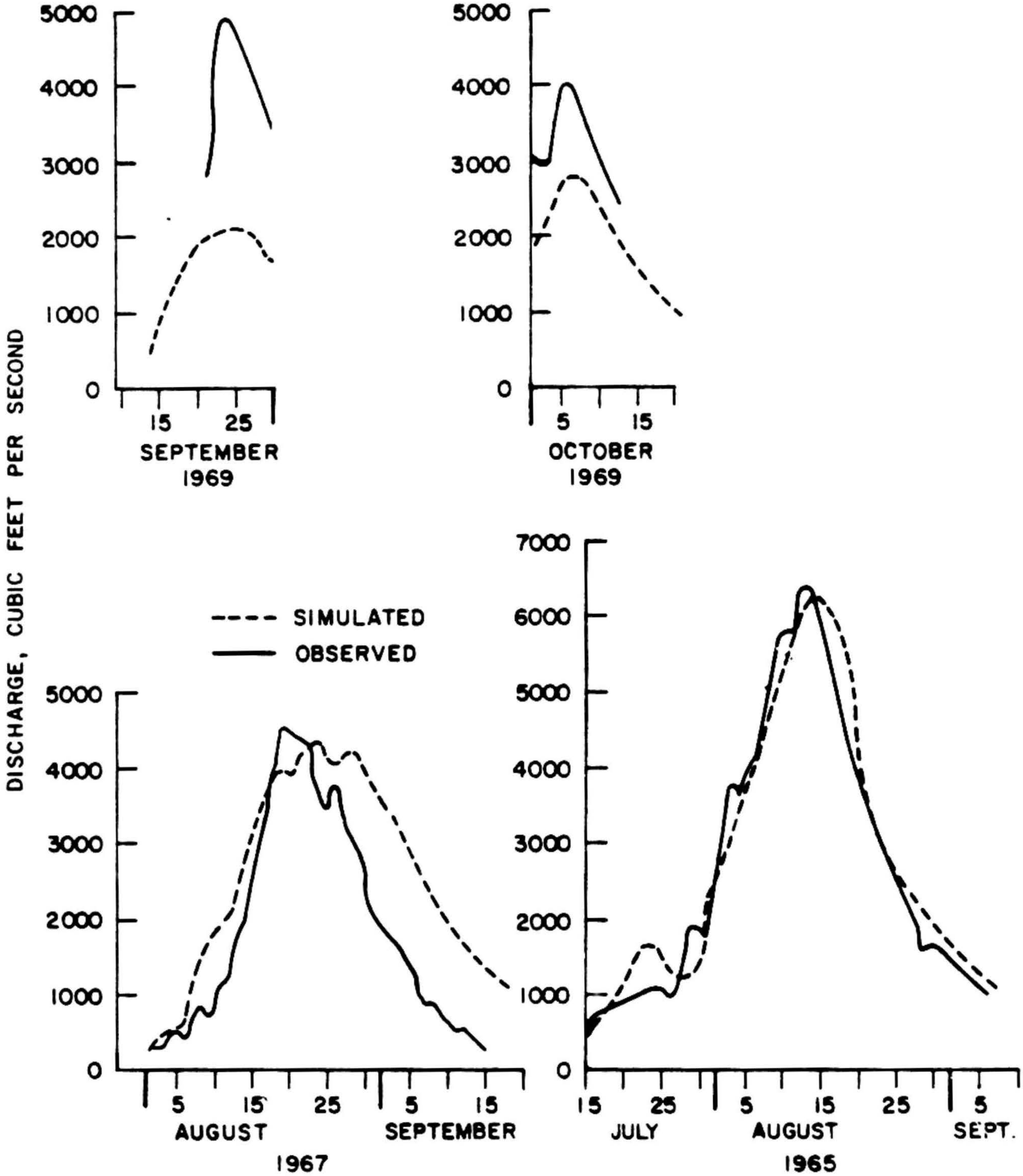
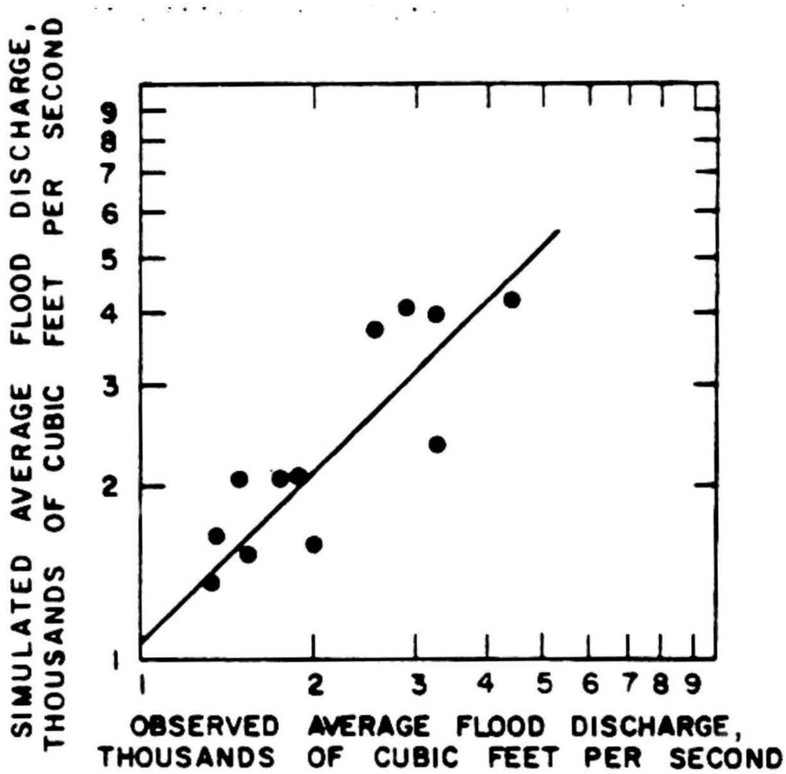


Figure 11.-Graphs showing simulated and observed flood hydrographs for the Hillsborough River near Tampa.

(neglecting signs) were analyzed collectively to obtain a frequency distribution. Sixty-seven percent of the simulated discharges have errors less than or equal to 30 percent. Fifty-five percent of the simulated discharges have errors of 20 percent or less, and 30 percent of the simulated discharges have errors of 10 percent or less. The average relative error between observed and simulated flood peaks is 14 percent.

Simulated and observed flood discharges (averaged over the entire storm) are shown as a plot in figure 12. Data shown in this plot indicate that, on the average, simulated flood volumes tend to be just slightly (6 percent) larger than actual flood volumes. This difference is not considered significant because of the small sample of data used in the analysis. Comparison of the simulated and observed peak discharges, however, indicates that, on the average, observed peaks are 5 to 11 percent larger than simulated peaks. A plot of simulated and observed peaks is also shown in figure 12.

A comparison of the time of occurrence of peaks indicates that, on the average, simulated peaks tend to lag no more than about 1.25 days behind observed peaks.



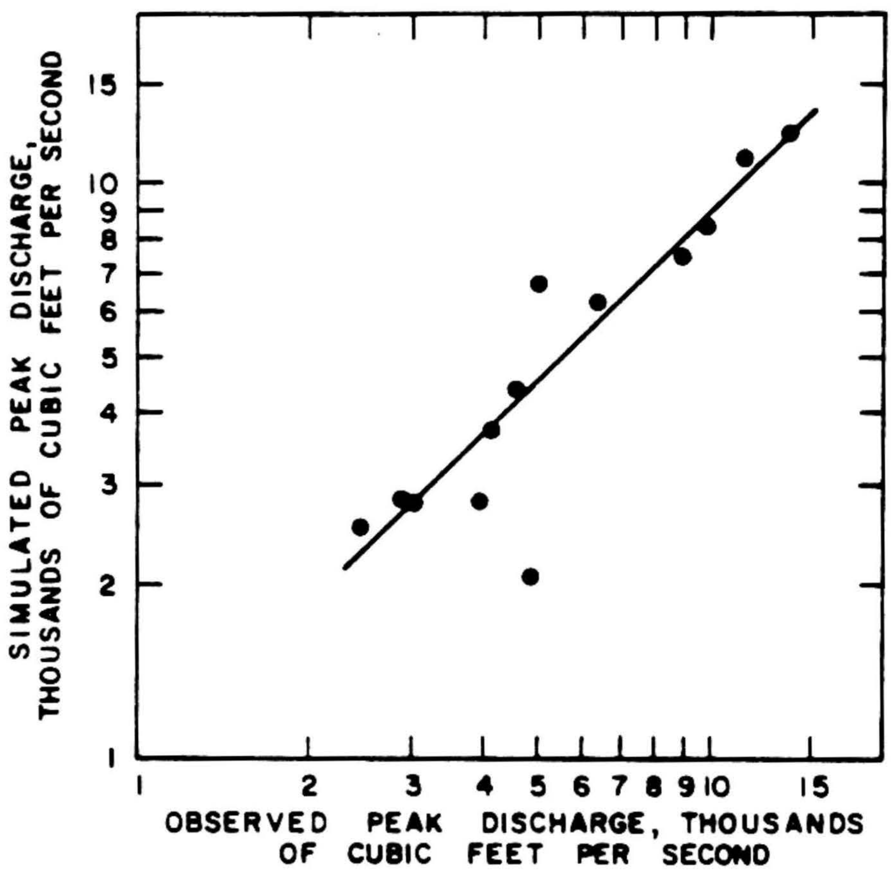


Figure 12.--Graphs showing relations between simulated and observed discharge (peak and average flood) for the Hillsborough River near Tampa.

The model developed for the Alafia River at Lithia is the same as the general model given by equation 3 except for the adjustment that is made to compensate for variation in storm duration. The equation of the Alafia River model, which has an 8-hour computational period, is as follows:

$$Q_t = UH \times R_{adj} + Q_{bf} \tag{19}$$

where UH = Matrix of 8-hour unit-hydrograph ordinates;

R_{adj} = Matrix of adjusted 8-hour direct-runoff values; and

Q_{bf} = Matrix of simulated 8-hour base flow discharges.

Runoff hydrograph.--As indicated by equation 19, elements (discharges) of the runoff hydrograph are formed by multiplying the unit-hydrograph matrix and the adjusted direct-runoff matrix. The adjusted direct-runoff matrix reflects adjustment for variation in storm duration. Ordinates of the average unit hydrograph are tabulated below and shown graphically in figure 13.

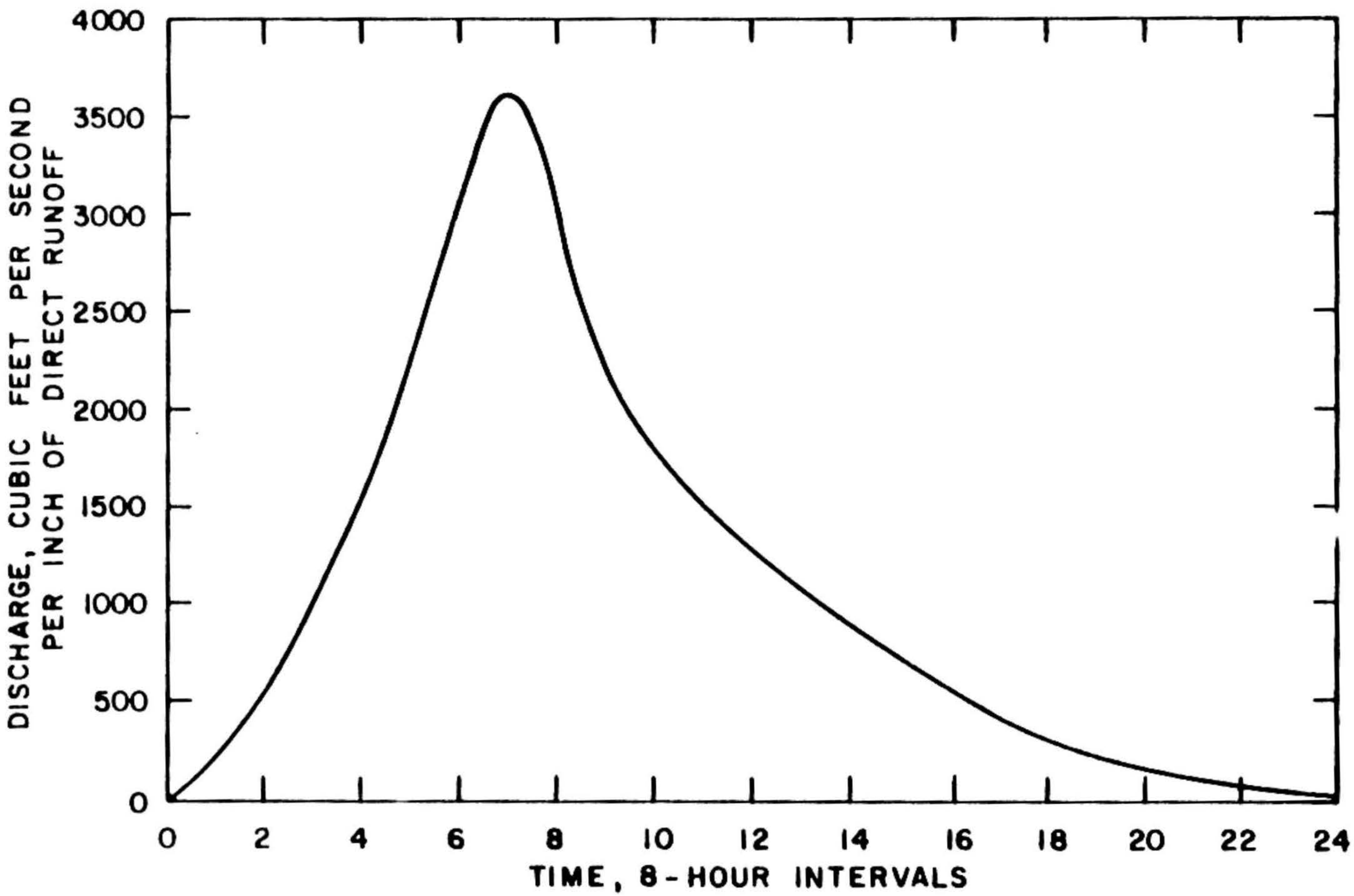


Figure 13.--Graph showing average 8-hour unit hydrograph for the Alafia River at Lithia.

Time, in 8-hour intervals	Discharge, in cubic feet per second per inch of direct runoff
1	0
2	200
3	520
4	1,000
5	1,600
6	2,300
7	3,000
8	3,600
9	3,000
10	2,200
11	1,800
12	1,540
13	1,300
14	1,100
15	900
16	730
17	580
18	430
19	310
20	220
21	140
22	90
23	50
24	20
25	10
26	5
27	0

These ordinates indicate, on the average, successive 8-hour rates at which 1 inch of direct runoff will discharge from the Alafia River basin at the gaging station. Because 8 hours is the time basis, or duration of the unit hydrograph for this station, all computations are made accordingly, including the derivation of 8-hour values of direct runoff from the rainfall-runoff relation shown in figure 14. The equation of this relation is

$$r = 0.28\bar{p} \tag{20}$$

where r = Direct runoff, in inches; and

$\bar{p}$ = Mean basin rainfall, in inches.

Mean basin rainfall is computed from daily rainfall records collected at Bartow and Plant City and from hourly records collected at Lakeland. The process involves the following application of Thiessen-weighting coefficients:

$$\bar{p} = 0.40B + 0.50PC + 0.10L \tag{21}$$

where B = Daily rainfall observed at Bartow, in inches;

PC = Daily rainfall observed at Plant City, in inches; and

L = Daily rainfall observed at Lakeland, in inches.

Because 8-hour mean basin rainfall is required in actual simulation, daily values of mean basin rainfall are calculated by use of equation 21 and subdivided according to the 8-hour rainfall distribution at Lakeland. These subdivided (8-hour) values are used as direct input to the model. Adjustment for variation in storm duration is made on the basis of the relation between storm duration and an average ratio involving ordinates of the average unit hydrograph and ordinates of the individual unit hydro-

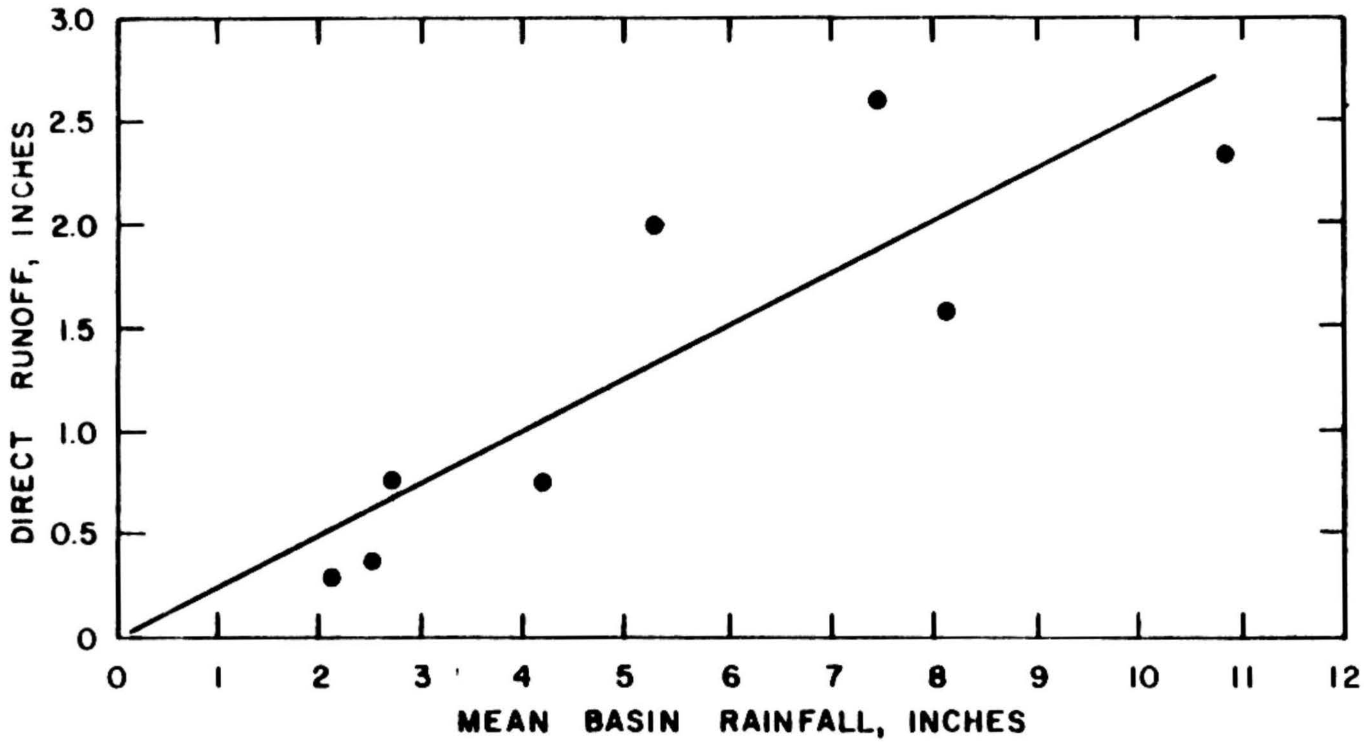


Figure 14.--Graph showing rainfall-runoff relation for the Alafia River at Lithia.

graphs used in the analysis. The equation of this relation, which is shown graphically in figure 15 is as follows:

$$c_d = 1.60 - 0.0073S_d \tag{22}$$

where c_d = Average ratio of the average unit hydrograph to the individual storm unit hydrographs; and

S_d = Storm duration, in hours.

Because the direct-runoff hydrograph is formed by a process involving multiplication of the average unit-hydrograph ordinates and direct runoff, the unit-hydrograph adjustment coefficients can be applied directly to runoff. Therefore, equation 22 is combined with the rainfall-runoff relation to yield

$$r' = \left[\frac{0.26\bar{p}}{1.60 - 0.0073S_d}\right] \tag{23}$$

where $\bar{p}$ = Mean basin rainfall, in inches;

S_d = Storm duration in hours.

The multiple relation given by equation 23 is used in computing 8-hour direct runoff (adjusted for storm duration) that is used in simulation.

<u>Base-flow hydrograph</u>.--Discharges that make up the Alafia River base-flow hydrograph are computed by use of a relation based on an average recession curve and a base-flow increase curve. The equation of the Alafia River recession curve, shown in figure 16, is as follows:

$$q_i = 5{,}000(0.96)^t \tag{24}$$

where q_i = Recession discharge, in cubic feet per second; and

t = Time, in 8-hour intervals.

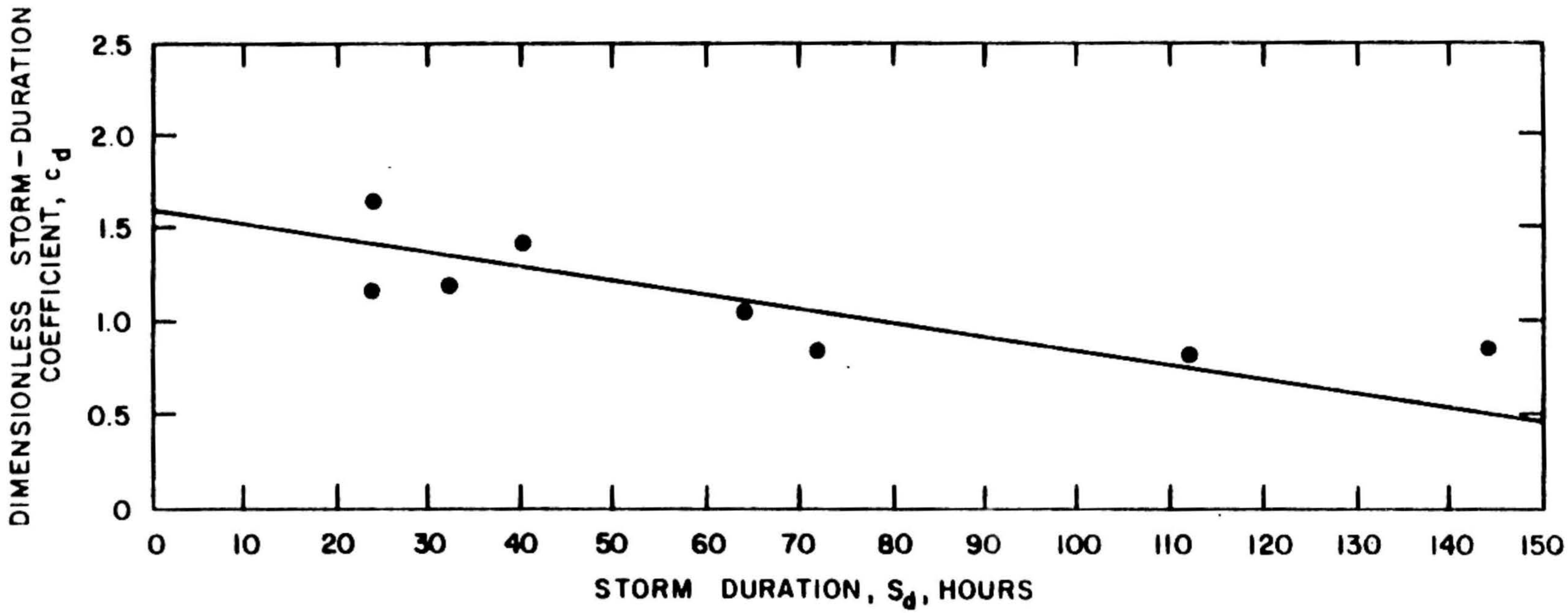


Figure 15.--Graph showing relation between storm-duration coefficient, c_d, and storm duration, S_d, for the Alafia River at Lithia.

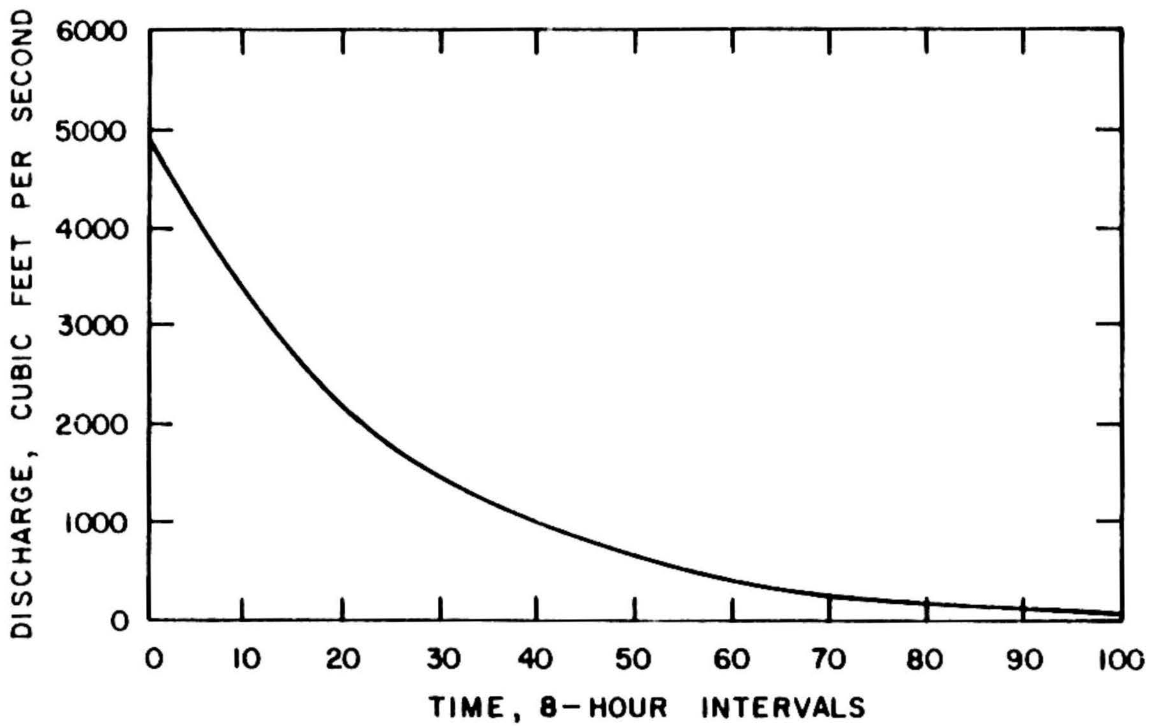


Figure 16.--Graph showing average base-flow recession curve for the Alafia River at Lithia.

The equation for the base-flow increase relation (fig. 17) is as follows:

$$d = 0.238s \qquad (25)$$

where d = Base-flow increase, in cubic feet per second; and

s = Direct peak runoff, in cubic feet per second.

However, because direct peak runoff, s, is considered a function of the maximum unit-hydrograph ordinate (3600 cfs) and adjusted 8-hour direct runoff, equation 25 can be re-written as

$$d = 858r' \qquad (26)$$

where r' = Adjusted 8-hour direct runoff, in inches.

Equations 24 and 26 are combined to form the following multiple relation that is used in computing base-flow discharges:

$$(q_{bf})_i = 5000(0.96)^t + 858r'; \; (r' = 0, \text{ for } i < 8). \qquad (27)$$

where t = Time, in 8-hour intervals; and

r' = Adjusted 8-hour direct runoff, in inches.

Actual calculation of the base-flow hydrograph begins by considering the flow conditions preceding direct runoff from a given storm. The initial time step, as well as all subsequent time steps that must be found after a base-flow increase, is calculated by

$$t = \left[\frac{\log_{10}(q_{bf})_i - \log_{10}(5000)}{\log_{10}(0.96)} \right] \qquad (28)$$

where $(q_{bf})_i$ = base-flow discharge, in cubic feet per second. Equation 28 is the inverse of equation 27 when adjusted 8-hour direct runoff is equal to zero.

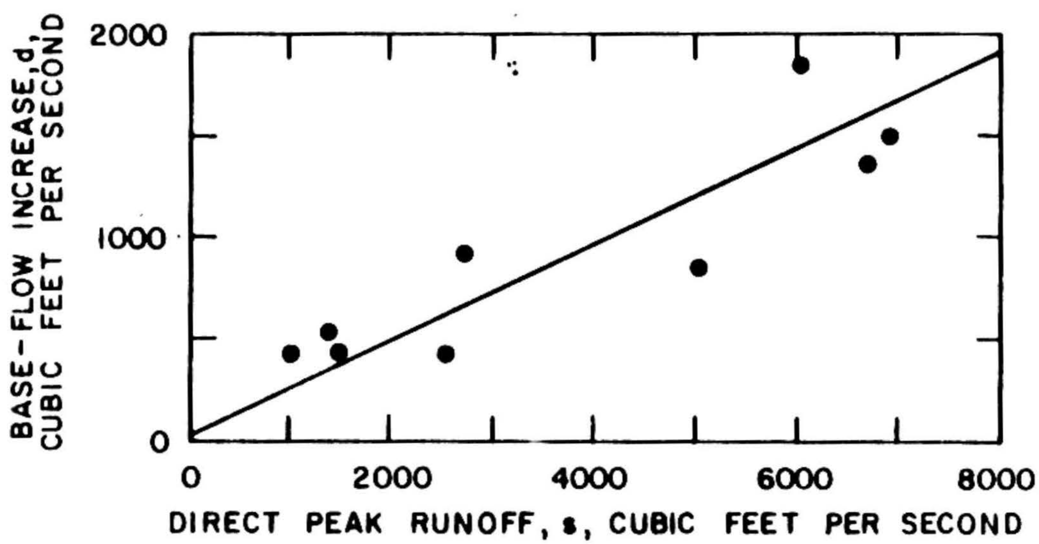


Figure 17.--Graph showing base-flow increase relation for the Alafia River at Lithia.

The Alafia River model operates in basically the same manner as the Hillsborough River model (table 4). The primary differences between these two models are:

(1) The Alafia River model time basis is 8 hours, rather than 24 hours;

(2) Runoff values are not lagged one time unit when preceded by a zero value; and

(3) Runoff adjustments are made for storm duration rather than for variation in storm pattern.

The Alafia River model has not been programmed in FORTRAN for processing on a large digital computer, but it has been programmed for processing on the WANG system.

Simulation Errors

To illustrate the relative accuracy of flood discharges simulated by use of the Alafia River model, flood hydrographs were obtained for each of the storms used in the analysis. These simulated and observed hydrographs are shown in figures 18 and 19.

Relative errors were computed for the simulated 8-hour flood discharges plotted in figures 18 and 19. Relative error is computed as the ratio of the difference in observed and simulated discharge to observed discharge. The computed relative errors (neglecting signs) were analyzed collectively to obtain a frequency distribution. Sixty-six percent of the simulated (8-hour) discharges have relative errors equal to or less than 30 percent. Fifty-four percent of the simulated discharges have relative errors equal to or less than 20 percent, and 34 percent of the simulated discharges have relative errors less than or equal to 10 percent. The average relative error between observed and simulated flood peaks is 12 percent.

Times of occurrence of the simulated peaks are randomly distributed about times of the observed peaks. The average difference in time of occurrence is about 16 hours.

A plot of simulated and observed average flood discharges is shown in figure 20. These data indicate that simulated average flood discharges could be as much as 8 percent greater than observed flood volumes for large storms, and 4 percent less than observed flood volumes for small storms.

A plot of simulated and observed flood peaks is also shown in

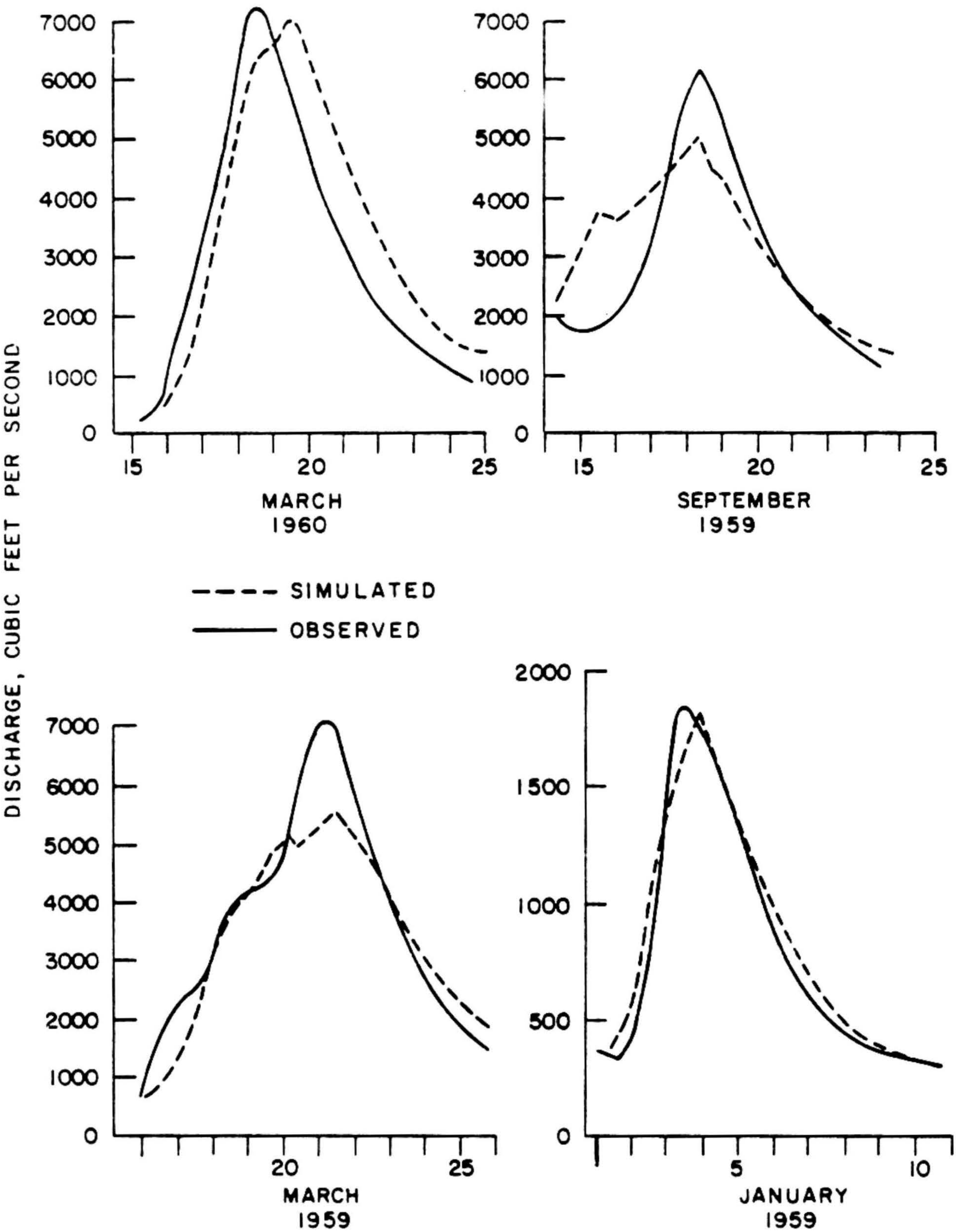


Figure 18.--Graphs showing simulated and observed flood hydrographs for the Alafia River at Lithia.

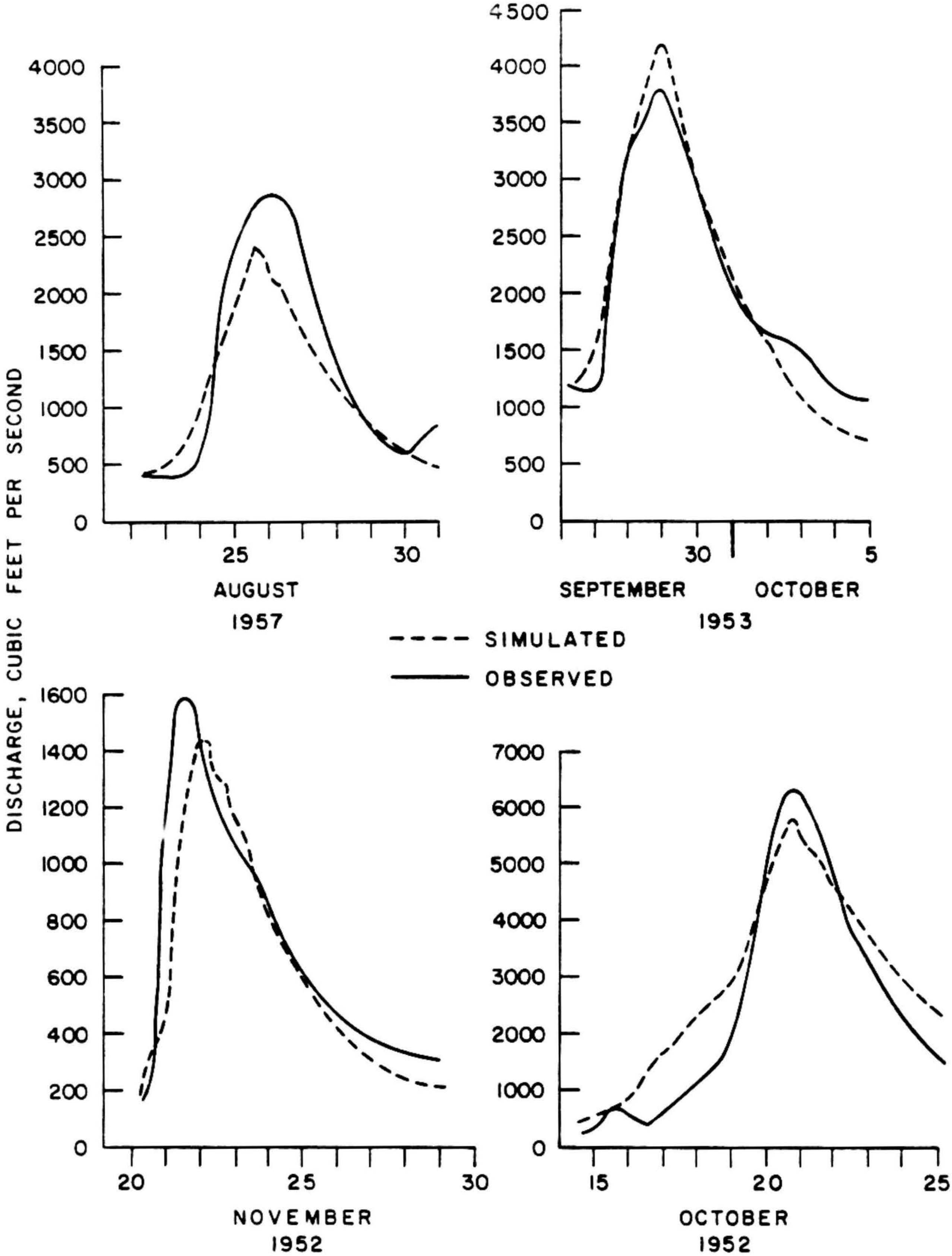


Figure 19.--Graphs showing simulated and observed flood hydrographs for the Alafia River at Lithia.

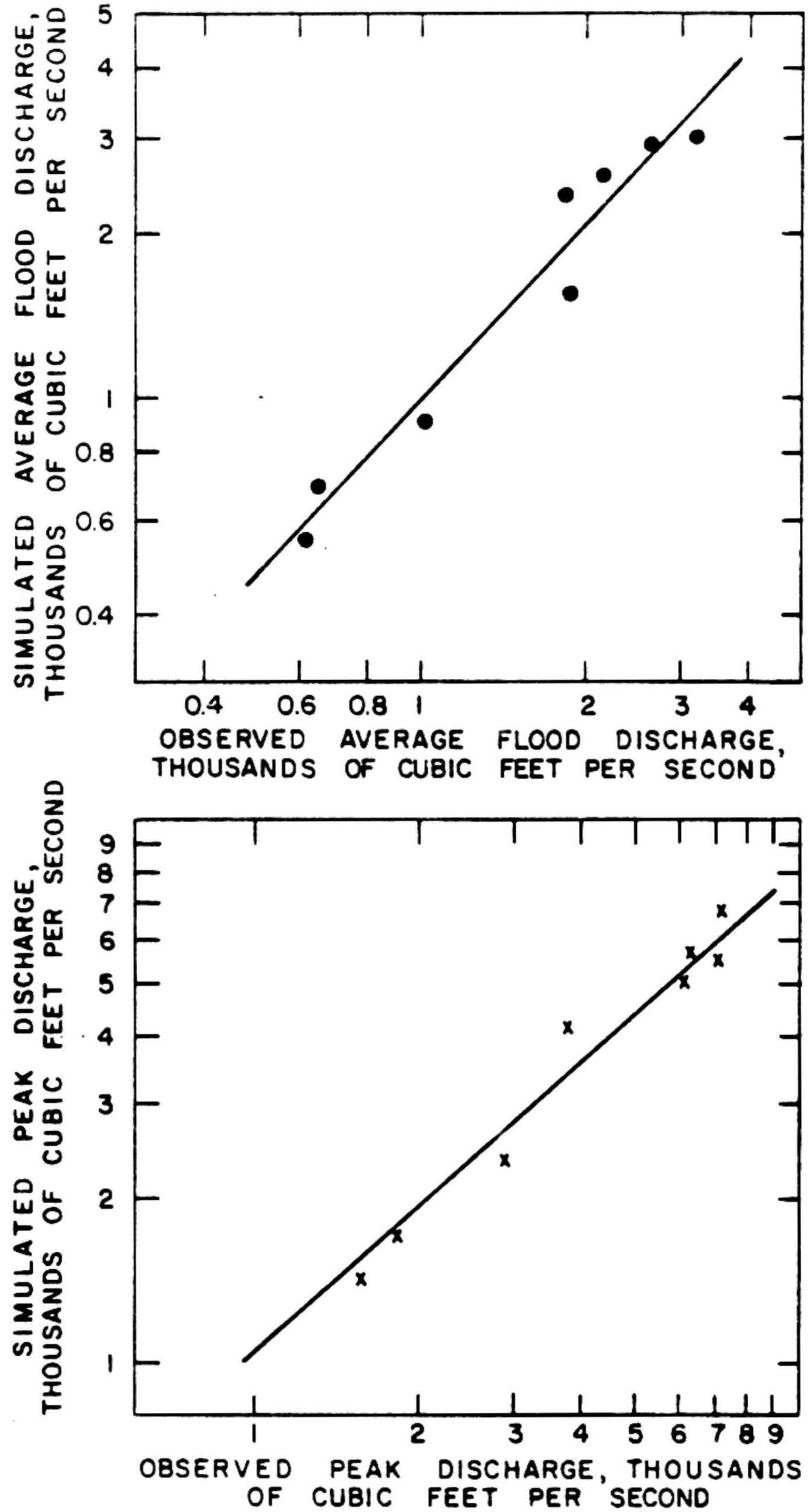


Figure 20.--Graphs showing relations between simulated and observed discharge (peak and average flood) for the Alafia River at Lithia.

figure 20. The average relation between the peaks indicates that simulated peaks are overestimated 3 percent for small storms and underestimated 20 percent for large storms. Even though the data shown in figure 20 indicate that some bias may exist in a flood simulated by the Alafia model, this bias is believed to result primarily from rainfall coverage rather than model error. In any case, a statistical adjustment is not merited for the apparent bias on the basis of available data used in this analysis.

The model developed for the North Pring Alafia River near Keysville is similar in form to the model developed for the Alafia River at Lithia. The matrix of the simulated 4-hour flood-hydrograph ordinates, Q_t, is given by

$$Q_t = UH \times R + Q_{bf} \tag{3}$$

where UH = Matrix of 4-hour unit-hydrograph ordinates;

R = Matrix of 4-hour direct runoff; and

Q_{bf} = Matrix of simulated 4-hour base-flow discharges.

Runoff hydrograph.--The ordinates used in forming the unit-hydrograph matrix, UH, represent successive 4-hour average flow rates at which 1-inch of direct runoff would be discharged from the North Prong Alafia basin (at the gaging station). These ordinates, which were derived by averaging unit-hydrograph ordinates of 12 independent storms of varying intensity and duration, are listed below and shown graphically in figure 21.

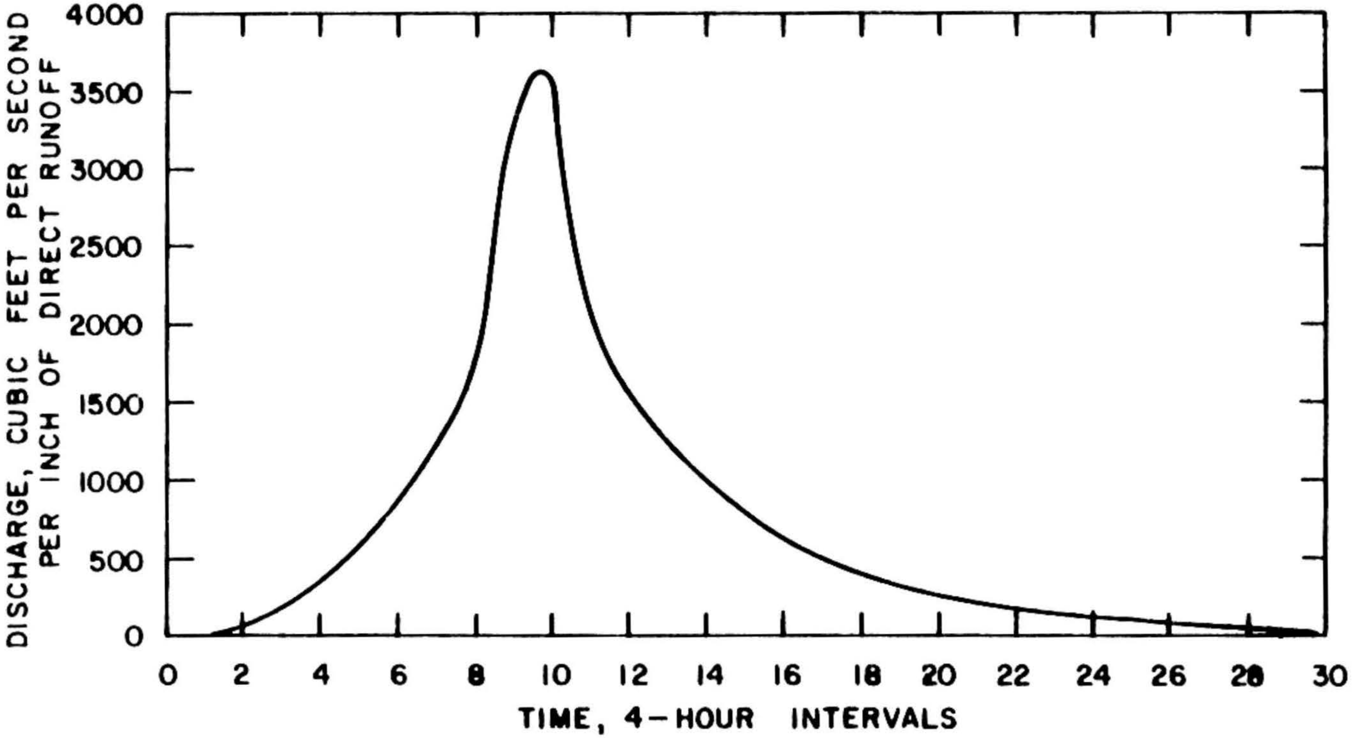


Figure 21.--Graph showing average 4-hour unit hydrograph for the North Prong Alafia River near Keysville.

Time, in 4-hour intervals	Discharge, in cubic feet per second per inch of direct runoff
1	0
2	30
3	75
4	170
5	370
6	570
7	925
8	1,260
9	1,810
10	3,220
11	3,630
12	1,930
13	1,540
14	1,240
15	980
16	780
17	630
18	500
19	410
20	335
21	280
22	230
23	200
24	170
25	145
26	120
27	100
28	80
29	60
30	0

Four-hour direct runoff used in simulation is computed by use of the relation shown in figure 22. This graph was obtained by plotting mean basin rainfall and storm runoff from 12 storms. The equation that best fits these data is

$$r = 0.15(\bar{p})^{1.2} \qquad (29)$$

where r = Direct runoff, in inches; and

$\bar{p}$ = Mean basin rainfall, in inches.

Mean basin rainfall, $\bar{p}$, is computed from daily rainfall observed at Bartow, Plant City, and Lakeland, using the following relation involving Thiessen-weighting coefficients:

$$\bar{p} = 0.27B + 0.33L + 0.40PC \qquad (30)$$

where B = Daily rainfall observed at Bartow, in inches;

L = Daily rainfall observed at Lakeland, in inches; and

PC = Daily rainfall observed at Plant City, in inches.

In flood simulation, mean basin rainfall for 4-hour intervals is required. Mean basin precipitation can be derived from daily rainfall records from Plant City, Lakeland, and Bartow by use of equation 30. Daily values are then subdivided into 4-hour values using the Lakeland record.

Base-flow hydrograph.--Four-hour base-flow discharges are computed by use of a composite relation on the basis of an average recession curve and a base-flow increase relation. The average recession curve developed for the North Prong Alafia River is shown in a plot in figure 23 and the base-flow increase relation is shown in figure 24. The equation of the recession curve, which gives discharge, q_1, in cubic feet per second, is

$$q_1 = 3050(0.946)^t \qquad (31)$$

where t = Time, in 4-hour intervals.

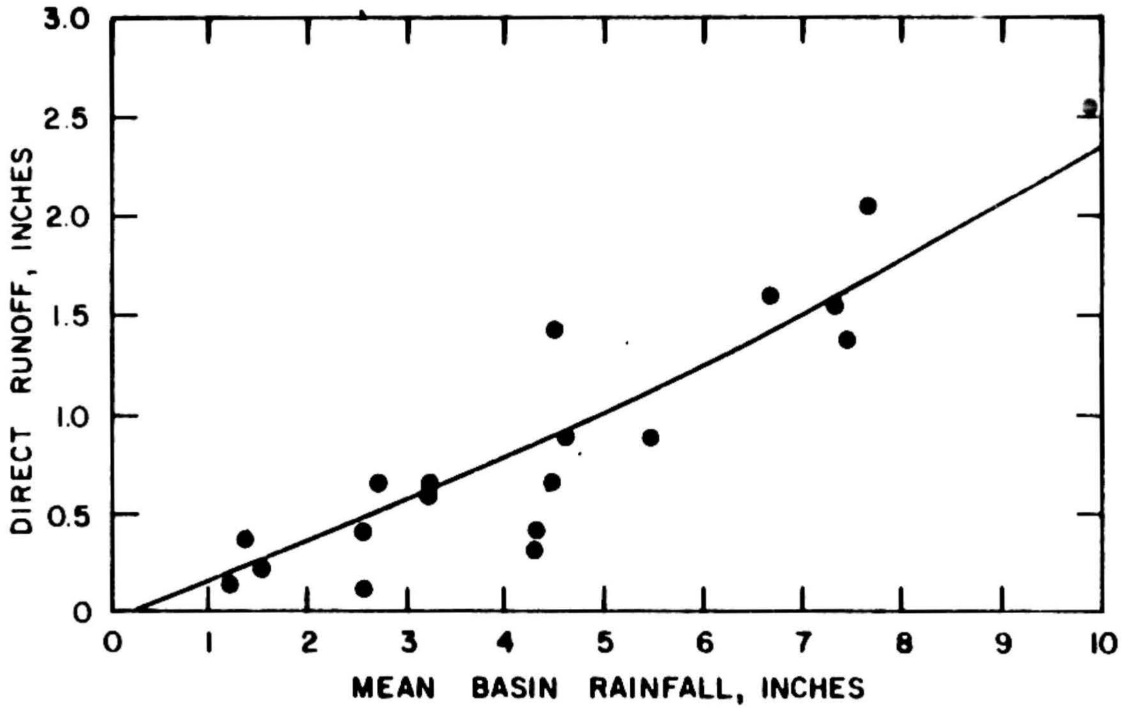


Figure 22.--Graph showing rainfall-runoff relation for the North Prong Alafia River near Keysville.

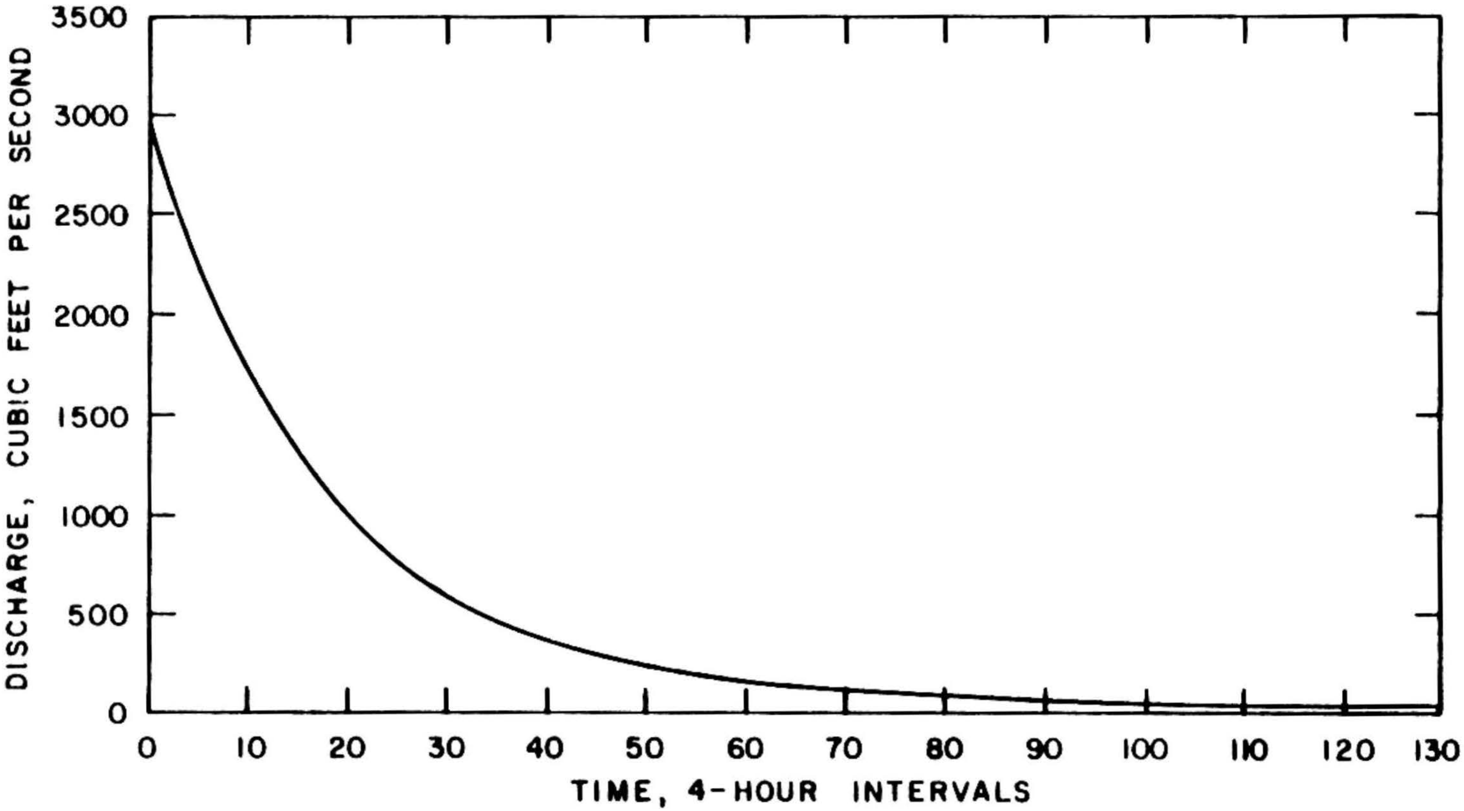


Figure 23.--Graph showing average base-flow recession curve for the North Prong Alafia River near Keysville.

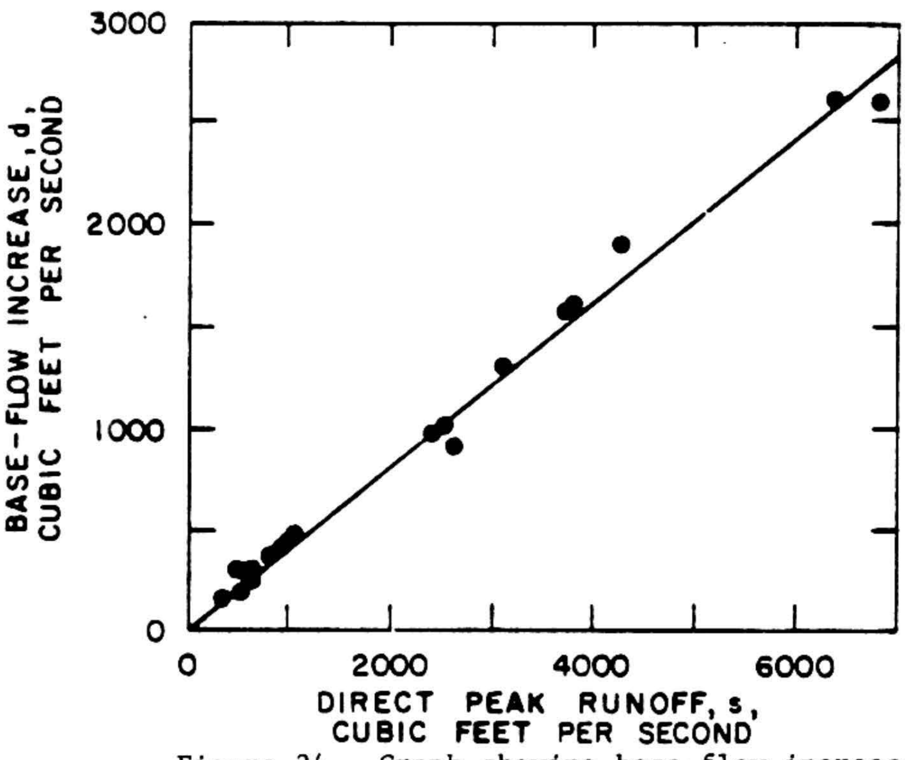


Figure 24.--Graph showing base-flow increase relation for the North Prong Alafia River near Keysville.

The equation of the base-flow increase relation, which gives base-flow increase, d, in cubic feet per second, is

$$d = 0.408s \tag{32}$$

where s = Direct peak runoff, in cubic feet per second.

Because direct peak runoff is computed from the maximum unit-hydrograph ordinate (3630 cfs) and 4-hour direct runoff, equation 32 can be reduced to

$$d = 1481r \tag{33}$$

where r = Four-hour direct runoff, in inches.

Equations 31 and 33 are combined to form a composite analytical expression that is used to compute discharges of the base-flow hydrograph. This equation, which gives 4-hour base-flow discharge, $(q_{bf})_i$, in cubic feet per second, is

$$(q_{bf})_i = 3050(0.946)^t + 1481r; \; (r = 0, \text{ for } i < 11) \tag{34}$$

where t = Time, in 4-hour intervals; and

r = Four-hour direct runoff, in inches.

As indicated by equation 34, base-flow increases are scheduled to occur during the first time interval following direct peak runoff. Accordingly, 11 base-flow discharges are calculated prior to the first base-flow increase.

The manner in which equation 34 is used to compute base-flow discharge involves the determination of an initial time index, t, from base-flow conditions that exist prior to the appearance ofdirect runoff. Under these conditions, 4-hour direct runoff is equal to zero, and equation 34 can be reduced to,

$$t = \left[\frac{\log_{10}(q_{bf})_i - \log_{10}(3050)}{\log_{10}(0.946)} \right] \qquad (35)$$

where $(q_{bf})_i$ = Base-flow discharge, in cubic feet per second.

Equation 35 not only enables determination of the initial time step on the recession curve at which simulation of the base-flow recession begins, but also allows computation of all subsequent time steps or reentry points on the recession curve following increases in base flow. After the base-flow hydrograph has been computed by use of equations 34 and 35, these discharges are combined with corresponding direct runoff discharges to form the flood hydrograph.

Simulation Errors

Hydrographs of observed and simulated floods are shown in figures 25, 26, and 27. These hydrographs were simulated from hourly rainfall records collected at Lakeland because of the proximity of this gage to the basin. Rainfall records collected at Plant City and Bartow were not used in simulating these storms because of the large errors associated with the determination of 4-hour mean basin rainfall from daily rainfall record. The simulated hydrographs had an unsatisfactory level of accuracy for most storms. Therefore, it is concluded that the North Prong Alafia River model cannot be used with confidence in simulating floods from existing rainfall records. Even though most of the simulated hydrographs do not compare well with observed hydrographs, the simulated hydrographs of November 25 and December 5, 1953, shown in figure 25, and January 12, 1964, shown in figure 27, are similar to the observed floods. Simulated peaks for these three storms are in error by no more than 20 percent and lie within one time unit (4 hours) of the actual peak.

Open pit phosphate mines may affect the flood peaks in this basin, but the extent of this effect cannot be determined from the data shown in figures 25, 26, and 27. The number of phosphate mines in the basin has increased since 1953, but simulated hydrographs do not indicate that earlier storms have less error than later storms.

Data on simulated and observed flood peaks plotted in figure 28 indicate a significant bias for discharges greater than about 2,500 cfs. Within this range, simulated peaks are significantly smaller than observed peaks. For discharges less than 2,000 cfs, no bias is detected in the data.

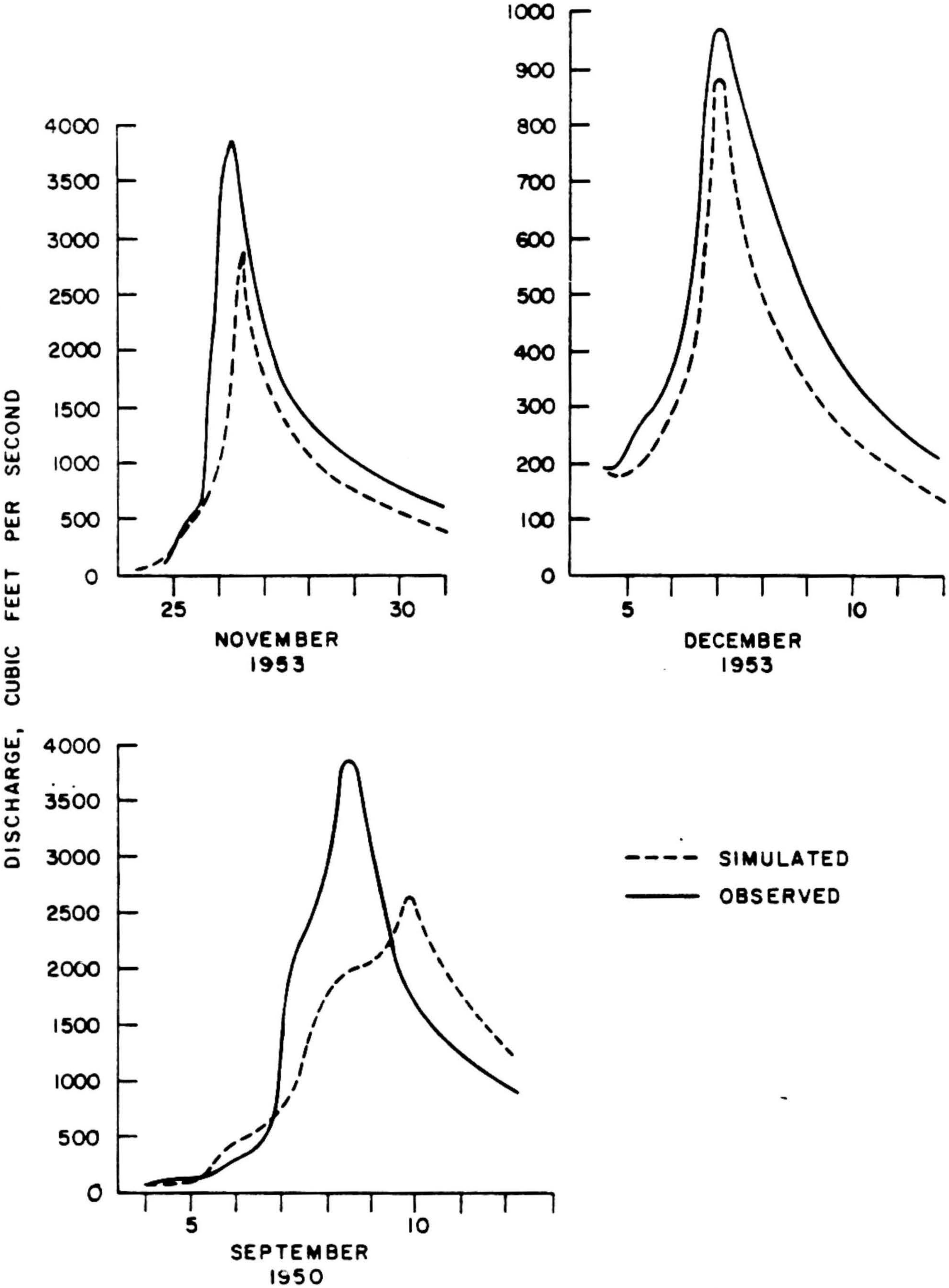


Figure 25.--Graphs showing simulated and observed flood hydrographs for the North Prong Alafia River near Keysville.

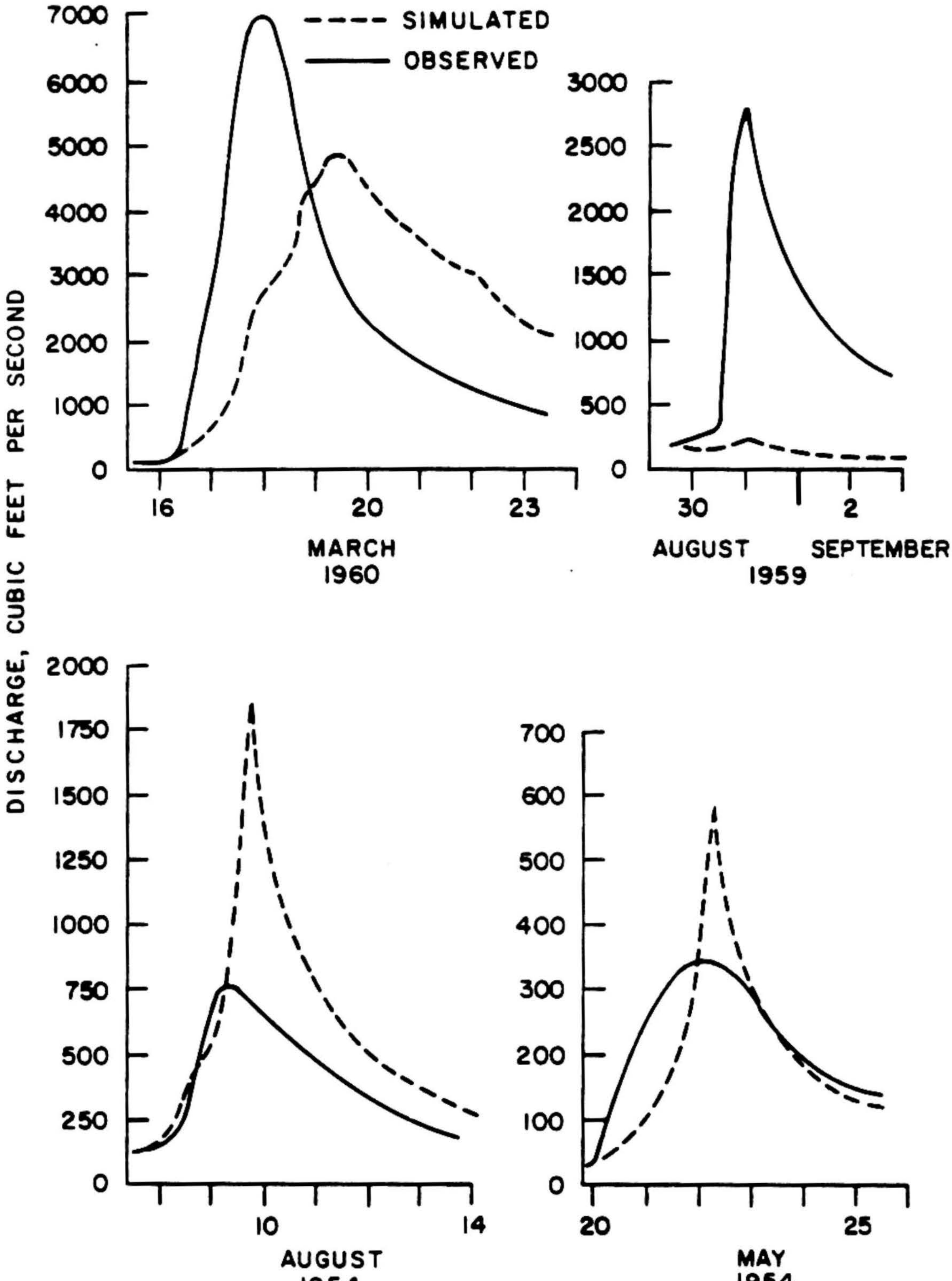


Figure 26.--Graphs showing simulated and observed flood hydrographs for the North Prong Alafia River near Keysville.

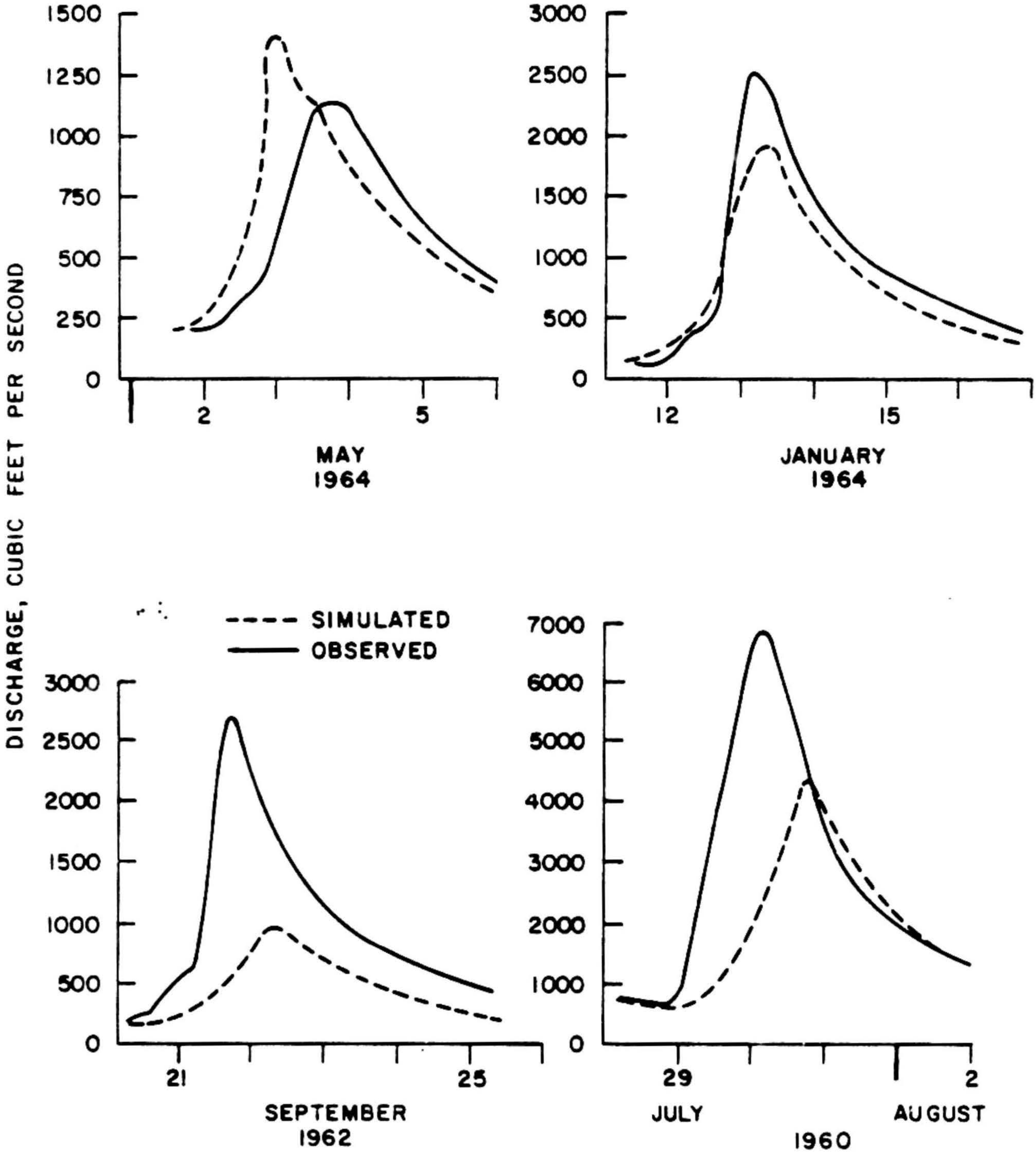


Figure 27.--Graphs showing simulated and observed flood hydrographs for the North Prong Alafia River near Keysville.

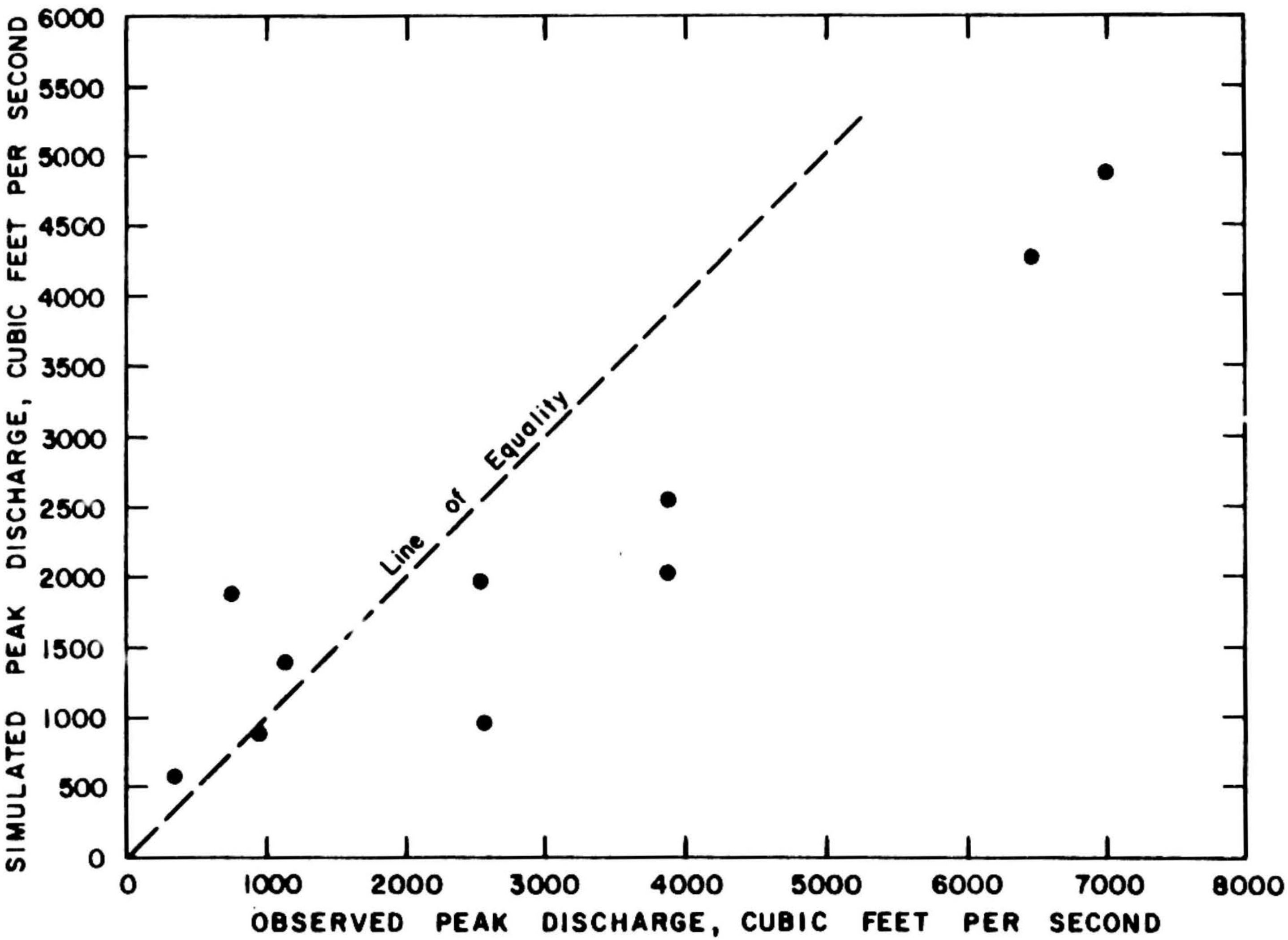


Figure 28.--Graph showing relation between simulated and observed peak discharge for the North Prong Alafia River near Keysville.

SUMMARY AND CONCLUSIONS

Flood hydrograph models based on modified unit-hydrograph and rainfall-runoff theories were developed for the following stream gaging stations in west-central Florida: Hillsborough River near Tampa, Alafia River at Lithia, and North Prong Alafia River near Keysville. These models broaden and enhance existing water-management capabilities within these basins especially with respect to implementation of flood control measures. The models serve primarily as a means for forecasting floods.

Simulation errors associated with the models are believed to result primarily from rainfall sample errors and to a lesser extent from model inadequacy. Rainfall sample errors result from the small number and poor areal distribution of rainfall stations available for use in the study. Model inadequacies result from insufficiences in the basic underlying theory of the models as well as errors in the relations (rainfall-runoff relationships, for example) used in developing the models. Even though an in-depth discussion of model inadequacy reaches far beyond the scope of this report, the manner in which the hydrograph is separated affects derivation of subsequent relations that are actually used in model development.

Relative errors between observed and simulated flood discharges were computed for each storm used in model development. A relative error was computed for each simulated flood discharge and each flood peak. Relative errors were analyzed collectively to obtain a frequency distirbution. Statistics comput in this analysis and relative errors for simulated flood peaks are listed below for the Hillsborough and Alafia River models.

Model	Number of storms	Percent of simulated discharges having relative errors equal to or less than percent indicated			Average relative error of simulated flood peaks in percent
		Percent			
		10	20	30	
Hillsborough River	12	30	55	67	14
Alafia River	8	34	54	66	12
North Prong Alafia River	12	(Errors greater than 40 percent)			

The data for the Hillsborough and Alafia River models indicate a high degree of correspondence between simulated and observed floods and flood peaks. On the basis of these data, it is concluded that these two models can be used to predict floods and flood peaks associated with large regional storms having homogeneous rainfall patterns within reasonable limits of accuracy.

Relative errors for the North Prong Alafia River model generally exceed 40 percent. However, with adequate rainfall input, simulation errors probably would be lowered to the same order of magnitude as for the Hillsborough and Alafia River models.

Because only a small number of storms was used in the analyses, the data listed above should be considered only as general guides to the magnitude of errors that could possibly be expected in simulated floods. A larger number of storms, other than those used in model development, would be required to establish statistically acceptable prediction errors.

Use of the models described in this report to forecast floods is limited to some extent by the availability of current rainfall data. From a cursory examination of these models, it could be surmised that total storm rainfall and associated time distribution would be needed to forecast a particular flood hydrograph. However, it should be noted that in an actual application of the models to forecast an impending flood, total storm rainfall may not be available. Under this condition, predictions would be made on the basis of current rainfall amounts with subsequent forecast as future rainfall information becomes available. This procedure can be used on a daily or shorter time interval basis; however, the limitation which is imposed by rainfall data availability is characteristic of most predictive models employing rainfall-runoff methods and is not just characteristic of the Hillsborough, Alafia, and North Prong Alafia River models.

Models described in this report only simulate with acceptable accuracy floods associated with large regional storms having homogeneous rainfall over the entire basin. However, the Hillsborough River model will be improved and expanded to allow simulation of reliable flood hydrographs associated with non-regional storms having heterogeneous rainfall amounts and areal coverage by including application of more sophisticated simulation techniques on subbasin areas. Subbasin floods would be simulated for each principal tributary and then routed through existing control structures downstream to the main-stem. Tributary discharge would be accumulated with the main-channel flow and routed down the main channel and subsequently out of the basin. Subbasin flood simulation by this approach would require better rain-gage coverage, involving a considerable expansion of the present networks. Measurement of rainfall in each of the subbasins would be required in order to significantly reduce simulation errors resulting from highly variable rainfall patterns.

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