FLOOD ROUTING ON THE ILLINOIS RIVER IN OKLAHOMA

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I_1	Channel inflow on first day of routing period in cubic feet per second
I_2	Channel outflow on second day of routing period, in cubic feet per second
I'	Local inflow from intervening reach, in cubic feet per second
K_2	Storage factor, in second-foot-days
O_1	Channel outflow on first day of routing period in cubic feet per second
O_2	Channel outflow on second day of routing period in cubic feet per second
P or C	Dimensionless constant indicating the relative proportion of inflow or outflow in the determination of storage
S_1	Channel storage on first day of routing period, in second-foot-days
S_2	Channel storage on second day of routing period, in second-foot-days
ΔT	Time interval, in fractions or multiples of one day
X	Storage constant, in units of time

CHAPTER I

INTRODUCTION

A. General

Floods in the United States cause extensive property damage, probably amounting to as much as $100,000,000 annually (according to recent estimates from the Department of the Interior), in addition to a toll of human lives on which no monetary value can be placed. A full realization of the magnitude of the stakes involved in the solution of existing flood control problems should impress those in charge of these studies with a deep sense of responsibility and should imbue them with a determination to spare no efforts to obtain the most reliable results possible. Nevertheless, with very few exceptions, it is not uncommon for engineers to devote an incredibly short time to determine the magnitude of the flood for which the structure should be designed or to calculate the flood dampening efficiency for the reservoir. Far too frequently all that is done is to apply a few convenient formulas, or perhaps to determine from the records what the maximum flood has been in the past and then add 25 or 30 per cent as a factor of safety. On the other hand, months are spent on structural design. This situation, perhaps, explains why dams rarely fail because of structural defects; it is a matter of record that a far greater number of such failures are the direct result of faulty determinations of expected peaks and peak reductions on which these structures should be designed.

The proper solution to this problem lies in making the best possible use of all available data. In addition to utilizing the flood records in the past, the records pertaining to the factors that affected and determined the magnitudes of those floods are just as important. One of these factors is the expected relief from flood damage due to the construction of protective works to reduce the flood flows by providing additional channel storage. This is the primary objective of any flood-control reservoir. However, the failure to establish a method for evaluating the ability of the reservoir to control floods will often result in a condition as serious as that when the river has no flood control at all.

B. Justification for This Research

Development of a dependable technique for predicting the individual effect of a reservoir on floods would enable hydrologists to gain much needed insight into some of the other aspects of flood control design. The process whereby the hydrograph of a flood as it occurred at an upstream station is transferred to some point downstream is called "flood routing." Without this procedure, no intelligent planning of flood relief is possible.

Furthermore, as the water resources problem becomes more critical due to expanding industrial activity and increased population, the necessity for understanding the flow characteristics of a stream becomes extremely important. The increasing use of multi-purpose reservoir networks indicates that engineers should strive to gain a better understanding of the behavior and the conditions affecting the flow of water in a stream. However, without a practical technique for evaluating the capacity of these reservoirs to control floods, the operation of such a system will be handicapped.

It is the opinion of this author that to date many reservoirs designed for flood control are not evaluated on the basis of their efficiency to dampen flood peaks but rather on their ability to withstand a hypothetical design flood. Without the knowledge of a reservoir's flood dampening ability, no intelligent operational scheme for a network of flood control reservoirs is possible.

A flood routing method is presented in this report which will enable engineers concerned with hydrologic design to evaluate the ability of a reservoir to withstand a variety of flood peaks. In addition, this technique will provide those concerned with reservoir performance an operational scheme for the effective control of floods. This method is described in Chapter III and applied to an actual reservoir in Chapter IV.

C. Objectives

The primary objective of this study was to develop rationally an acceptable prediction technique which may be used to estimate the dampening efficiency that a reservoir exhibits on floods as they occur in a river basin. A further objective was to check the developed theory, using data gathered at gaging stations within this basin, and to test its applicability to previously published field data taken at these stations.

The secondary objective of this study was to utilize this technique to predict expected reservoir dampening efficiencies on future floods in this basin in order to gain insight into the resulting reservoir responses to a variety of flood magnitudes.

As a consequence of meeting the primary objective, a third objective was to develop a suitable flood routing method so that the resulting basin flow characteristics for a wide range of flood magnitudes can be accurately evaluated.

Finally, the fourth objective of this study was to provide an insight into an assessment of the allowable conservation storage not only within this river basin, but also the basin to which this river is a tributary. Hence, this information may be used to determine, predict, and/or recommend the allowable conservation storage which may be equitably apportioned within this basin and adjacent basins in light of present and future water requirements.

D. Organization of the Research Report

In the course of conducting this investigation, three very important hydrologic analyses were studied: (a) synthesis of missing data, (b) development of a flood routing method, and (c) prediction of reservoir flood-dampening efficiencies. The succeeding chapters of this report present each of these aspects.

CHAPTER II

LITERATURE REVIEW

A. Hydraulic Flood Routing Methods

A strict hydraulic method of flood routing has been found to be extremely complicated and difficult to handle. However, various simplified methods have been developed for practical purposes. Many of these methods belong to the general form known as the method of characteristics. Derived in 1900 by Massau (15) and further developed by Lin (11), this method is based upon the solution of characteristic equations of unsteady flow. Numerical solution by this method is generally very tedious and an electronic computer is often employed to speed up the computations.

Assuming that the vertical acceleration of the water particles in an infinitesimal element of channel length is negligible and that a resistance coefficient for the channel is the same for a given depth and velocity regardless of whether the flow is uniform or non-uniform, steady or unsteady, this method is based on the principle of the conservation of energy.

Stoker (24) made direct use of this method in studying floods on the Ohio and Mississippi Rivers. He also showed how the confluence of the two rivers produced a wave traveling upstream as well as downstream, a phenomenon which is not considered by other methods.

A second hydraulic approach has been developed using the statistical theory of flow diffusion. This concept, known as the method of diffusion analogy, applies the assumption that the diffusion of the disturbances of flow caused by channel irregularities is similar to the diffusion of the particles of flow in the channel. In 1951, Hayami (6) developed what is known as "the basic differential equation for flood flow in natural streams." His propagation equation, involving velocity, depth, time, and diffusivity parameters, has been found to predict theoretical hydrographs in good agreement with those observed.

Another approach, known as the method of successive approximations, is probably the best suited hydraulic analogy for flood routing problems of those methods discussed thus far. Based on more practical, but yet still valid, assumptions, this method is applicable to streams where the data are average daily flows rather than slope, stage, and velocity measurements. Rippl (20), in 1883, was one of the earliest engineers to apply this principle. In working on reservoir capacity problems, he developed what is now known as the continuity equation. This equation, presently the most practical and applicable equation used in flood routing, may be expressed as

$$\tfrac{1}{2}(I_1+I_2)\,\Delta T - \tfrac{1}{2}(O_1+O_2)\,\Delta T = S_2-S_1 \qquad (1)$$

in which I represents inflow into a given reach, O represents outflow from that reach, ΔT represents the time period for the flow to travel through that reach, S_2-S_1 represents the change in storage during that time period in the reach, and the subscripts 1 and 2 represent conditions at the beginning and end of the routing period.

A fourth hydraulic approach to flood routing was developed when, in 1950, Stoker (24) introduced a concept known as the method of coefficients. By this method he re-wrote the continuity equation to read

$$O_2-O_1 = C_1(I_1-O_1) + C_2(I_2-I_1) \qquad (2)$$

where C_1 and C_2 represent factors involving the proportionality between storage and outflow.

This equation, therefore, presents the rate of change of outflow in terms of instantaneous changes in inflow and outflow and in terms of the rate of change of inflow in the time increment ΔT. According to Rouse (21), the two outstanding features of this method are that it can be applied to conditions in which tributary inflows occur within a reach and also to conditions in which reservoir effects are to be determined.

Rouse (21) proposed a graphical solution to flood routing problems in short reaches in which the discharge at the lower end of the reach can be approximated in terms of the rise or fall in water surface through the reach. Although he approached the problem in a somewhat different manner than previous researchers, the stage-discharge working curves he developed have been found to be reasonably accurate for unsteady-flow conditions.

A number of other hydraulic approaches are available but the methods presented herein represent the trend of general hydraulic flood routing methods used by hydrologists and engineers today.

B. Hydrologic Flood Routing Methods

The hydrologic method of flood routing may be distinguished from the hydraulic method by the fact that the hydraulic approach is based upon the solution of the basic differential equations of unsteady flow in open channels whereas the hydrologic concept makes no direct use of these equations but approximates their solutions. It cannot be said that the hydrologic methods are inaccurate, although they do not approach the accuracy of the hydraulic methods, but they are more often used because the data required for the hydraulic approaches are often not available.

One hydrologic approach, suitably called the hydrograph method, is probably the most widely used of the hydrologic methods. From the continuity equation (Eq. 1) it can be seen that two unknowns exist: O_2 and S_2. It must then be necessary to find a second equation involving O_2 and S_2 to solve for their values simultaneously.

Puls (18) and Gustafson, according to Chow (2), working independently, derived a method for finding the relationship between storage and outflow graphically. In 1928, Puls published his findings in which he established a curve of relation between inflow plus outflow versus storage for a variety of floods on the Tennessee River. Gustafson's work, which was developed in Minnesota at the same time, agreed with that of Puls and this approach has been widely used hence.

In 1931, Wisler and Brater (27) presented a graphical approach which was the first method to use computed inflow hydrographs from tributaries and unmeasured areas for which the discharge records are not available. This procedure is based upon the continuity equation which was expanded to include the local inflow, I', as follows:

$$\tfrac{1}{2}(I_1+I_2)\Delta T + I'\Delta T - \tfrac{1}{2}(O_1+O_2)\Delta T = S_2-S_1 \qquad (3)$$

Then, solving for I' using $\Delta T = 1$ day, the equation reads

$$I' = \tfrac{1}{2}(2S_2+O_1+O_2) - \tfrac{1}{2}(2S_1+I_1+I_2) \qquad (4)$$

In the above equation, all of the terms on the right-hand side are known except S_1 and S_2, and these values may be determined from a curve of relation between S and (I+O).

After solving for the values of I' for all of the time intervals in the study, the hydrograph for I' may be drawn. Therefore, the hydrographs for the channel inflow at the upstream gaging station and for the flow from the intervening area may then be combined to produce the outflow hydrograph at the downstream station. The agreement of this graph with the actual hydrograph at the downstream station was reported to be quite accurate, especially the higher stages during which the greatest need exists for dependable results.

Wisler and Brater also described a procedure for determining the effectiveness of a flood peak reduction device, such as a reservoir, in lowering the flood flow within a channel reach (27). The effectiveness with which the storage thus provided may be expected to reduce the flood peak at the downstream station or at any other downstream point depends on the rules that are formulated governing the operation of that storage. The reservoir outflows resulting from any such proposed set of rules can be computed and this revised flow can then be routed and the reduction benefits determined at points downstream.

Originally proposed in 1931 by Goodrich (4), and subsequently used by Rutter, Graves, and Snyder (22), a semi-graphical approach involving the use of routing curves and tabular computation was developed in which the continuity equation was modified to read

$$I_1+I_2 + 2S_1/\Delta T - O_1 = 2S_2/\Delta T + O_2 \qquad (5)$$

Routing curves are required showing $(2S/\Delta T \pm O)$ as a function of O. Hence, the procedure for this method from this point on is similar to that of the hydrograph method. Goodrich reported that errors of less than one per cent are found with this method and that the accuracy of the results is, of course, largely dependent upon the length of the time intervals used.

Developed by McCarthy (16), and used extensively by the U. S. Army Corps of Engineers, the Muskingum method has been found to be a popular and satisfactory approach to flood routing. Similar to the hydraulic method of coefficients, this method makes use of the proportionality between storage and outflow. McCarthy modified the continuity equation to give the following working formula:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2\ O_1 \tag{6}$$

where

$$C_0 + C_1 + C_2 = 1.0 \tag{7}$$

in which

$$C_0 = \frac{-\ PX + 0.5\,\Delta T}{X-PX + 0.5\,\Delta T} \tag{8}$$

$$C_1 = \frac{PX + 0.5\,\Delta T}{X-PX + 0.5\,\Delta T} \tag{9}$$

and

$$C_2 = \frac{X-PX - 0.5\,\Delta T}{X-PX + 0.5\,\Delta T} \tag{10}$$

in which P is a dimensionless constant indicating the relative importance of I and O in determining storage and X is a storage constant with the dimension of time. The value of X approximates the time of travel of the flood crest through the reach, and Δ T represents the time lag between gauging station recordings.

With values for P, X, and Δ T established, O_2 can be found from Eq. 6. The value for O_2 for the first routing period becomes the value of O_1 for the second, and so on, and this procedure can be indefinitely repeated to compute successive values of outflow at time invervals Δ T.

Another variation of the solution of flood routing problems is the method proposed in 1938 by Steinberg (23), who presented a semi-graphical approach involving not only a series of storage rating curves but also a family of curves for a factor K, which he denoted as the storage factor. He modified the continuity equation to include this factor so that it became

$$\tfrac{1}{2}(I_1+I_2-O_1)\,\Delta T + S_1 = \tfrac{1}{2}O_2\,\Delta T + S_2 = K_2 \qquad (11)$$

Once a value of ΔT is established, the family of K curves is constructed since $K = \tfrac{1}{2}O\,\Delta T + S$. Using these curves, for each value of K and I, corresponding values of O and S may be found. As in the previous methods, the final values of discharge and storage for the first time interval become the initial values of these parameters for the second time interval, and so on.

For flood routing through streams and reservoirs, Wilson (26) devised a graphical solution in which he related storage and discharge by a single curve. In doing so, he introduced a time conversion factor, T, to show a ratio between the storage in a reach and the rate of change of discharge from that reach. In reservoir routing, T may be found by relating the stage-storage curve for the reservoir to the stage discharge curve of the spillway, thereby relating storage and discharge for all ranges of flow. For streams, T may be taken as the travel time of a flood wave through the reach. Wilson's method is valid for ideal reservoirs, but when it is applied to natural streams it has no advantages save that of expediency.

Linsley (12) in 1944 presented a nomograph method for solving flood routing problems in which he modified the continuity equation to read

$$(S_1/\Delta T - O_1/2) + \bar{I} = S_2/\Delta T + O_2/2 \tag{12}$$

in which $\bar{I}$ represents the average inflow during the time interval, ΔT. Scales of $S - O/2$ and $\bar{I}$ are constructed with arbitrary graduations. From Eq. 12 it can be seen that the sum of these two values is equal to $S + O_2$. Linsley then constructed and calibrated an axis of $S + O/2$ so that its intersection with a straight line connecting the values of $S - O/2$ and $\bar{I}$ was equal to their sum. At this point it is necessary to enter a nomograph with the value of $S + O/2$ to find the corresponding value of outflow during that time. For further discussion of this method the reader is referred to Linsley's article. The advantage of this method is that the outflow can be determined without continuous tabulation; however, the construction of the rating curves and the nomograph introduce inherent errors into the method.

Cheng (1) and Chow (2) also used graphical approaches using Linsley's modification of the continuity equation to construct an outflow hydrograph without continuous tabulation. In addition, Chow (2) introduced a second graphical solution in which the continuity equation was modified to read

$$S_1/\Delta T - O_1 + I_1 + I_2 = S_2/\Delta T + O_2 \tag{13}$$

which is similar to Goodrich's equation, except that storage is expressed in acre-feet instead of second-foot-days. By this method, characteristic curves for storage versus storage plus outflow are constructed for floods of record and outflows may be found by a step-by-step graphical solution for known values of inflow. Using this approach, Chow reported excellent results in cases where the downstream gaging station had been discontinued.

In flood forecasting or control and operation of multiple-purpose river projects, the stage of the flood is often of major concern, and a procedure for stage routing is required. For this purpose a method involving the use of multiple-line charts was proposed by Lane (10). An improved procedure was later developed by Kohler (9) which requires one chart for determining the normal relationships between gages and flows in the main channel, and auxiliary charts for each tributary.

The procedure then follows three steps: First, from the stage curve for the main channel, the stage at the downstream station is forecast by prorating the stage at the upstream station a length of time equal to the crest travel time; second, corrections are made for the effects of the tributaries by the same procedure; and third, the algebraic sum of these values is taken to forecast the unknown stage at the downstream station.

Although other methods (13)(22) are available, the Kohler method of stage routing is often used and is easily adaptable. This is especially true of large river systems which maintain continuous monitoring of stages in addition to discharge flows.

Another method of stage routing is that proposed by Ray and Mondschein (19) for forecasting stages on very flat rivers. Based on inadequacies in the Muskingum and Kohler methods for adaptation to flat rivers, they devised a new approach which not only forecasts downstream stages but also predicts intermediate stages and backwater effects due to tributaries.

Working at the junction of the flat Illinois River with the Mississippi River, they used a downstream stage parameter as an index of the slope and channel storage in order to plot stage-discharge routing curves. In addition, using upstream and downstream stages as an index to the slope, they were able to estimate stages within the reach. In comparing predicted hydrographs with and without corrections for backwater, they found that the corrections did not greatly change the hydrographs except at very high flows, which is to be expected.

Although the literature reports many more hydrologic flood routing methods, both analytical as well as graphical, it is felt that the methods presented herein represent a reasonable array of this approach.

C. Flood Routing Aids

The discharge integrator was a precision instrument originally designed about 1914 by E. A. Fuller (3). Its use was confined almost exclusively to the U. S. Geological Survey and its purpose was to translate mechanically a continuous gage-height graph into a record of mean daily discharge. The integrator gave results within two per cent when carefully operated.

Since that time, numerous integrating machines have been devised. Among them, the device developed by Tarpley (25) in 1937, is probably the most widely used instrument of its kind today. Based on historical flood records, the instrument draws elevation and outflow curves for any other flood. Accuracy within 0.5 per cent when compared with results of analytical and graphical methods have been shown (25).

In 1935, Posey (17) invented a sliding device for flood routing through storage reservoirs and lakes. Applying the working values of $S+\frac{1}{2}O\,\Delta T$ and $S-\frac{1}{2}O\,\Delta T$ in a manner which makes it convenient to calibrate this instrument, the slide rule has been shown to be adaptable to direct solution when controlled releases from a reservoir are involved.

Harkness (7), in 1945, introduced a rolling device for direct construction of an outflow hydrograph from a level-pool reservoir. Relationships between fixed points on the instrument may be set so that the mathematical relationships of flood routing hold true. This device has been found to be a great time-saver inasmuch as only the inflow hydrograph is required.

The U. S. Weather Bureau (13)(14) has developed an electronic analog which produces an outflow hydrograph while the operator traces an inflow hydrograph with a stylus. Resistances in the electric current can be adjusted to simulate the conditions for each reach of the river. The use of this machine has a decided advantage over analytical methods in that it solves the continuity equation in differential rather than incremental form. Furthermore, the entire hydrograph can be routed more rapidly than by analytical computation.

Graves (5), as recently as 1967, presented a flood routing example for digital computer analysis. In applying this method to the Mississippi River, he divided the long reaches into subreaches in order to account for backwater and levee effects. When tested against a previous model of the same reach, the method proved almost flawlessly accurate at various flood peaks.

CHAPTER III

METHODS AND MATERIALS

A. Description of the Basin

The river basin studied, Fig. 1, was the Illinois River, a tributary of the Arkansas River. The Illinois River originates in northwestern Arkansas as Osage Creek, and flows westward until it meets with Muddy Fork, which in turn drains Clear and Goose Creeks. The Muddy Fork system drains the southern portion of the tributary area of the Illinois River in the State of Arkansas, while Osage Creek and the upper reaches of Flint Creek drain the northern portion of the tributary area. The Illinois River then crosses the Oklahoma-Arkansas state line and continues westward, draining tributaries such as Wedington Creek and Ballard Creek. After Flint Creek joins the Illinois River, the river flows in a southerly direction into Tenkiller Ferry Reservoir. The major tributaries joining the river in this reach are Barren Fork and Caney Creek. After leaving Tenkiller Ferry Reservoir, the Illinois River flows southward for a distance of approximately seven miles and drains into the Arkansas River just upstream of the Robert S. Kerr lock and dam. The entire drainage area of the basin is 1,660 sq. mi.

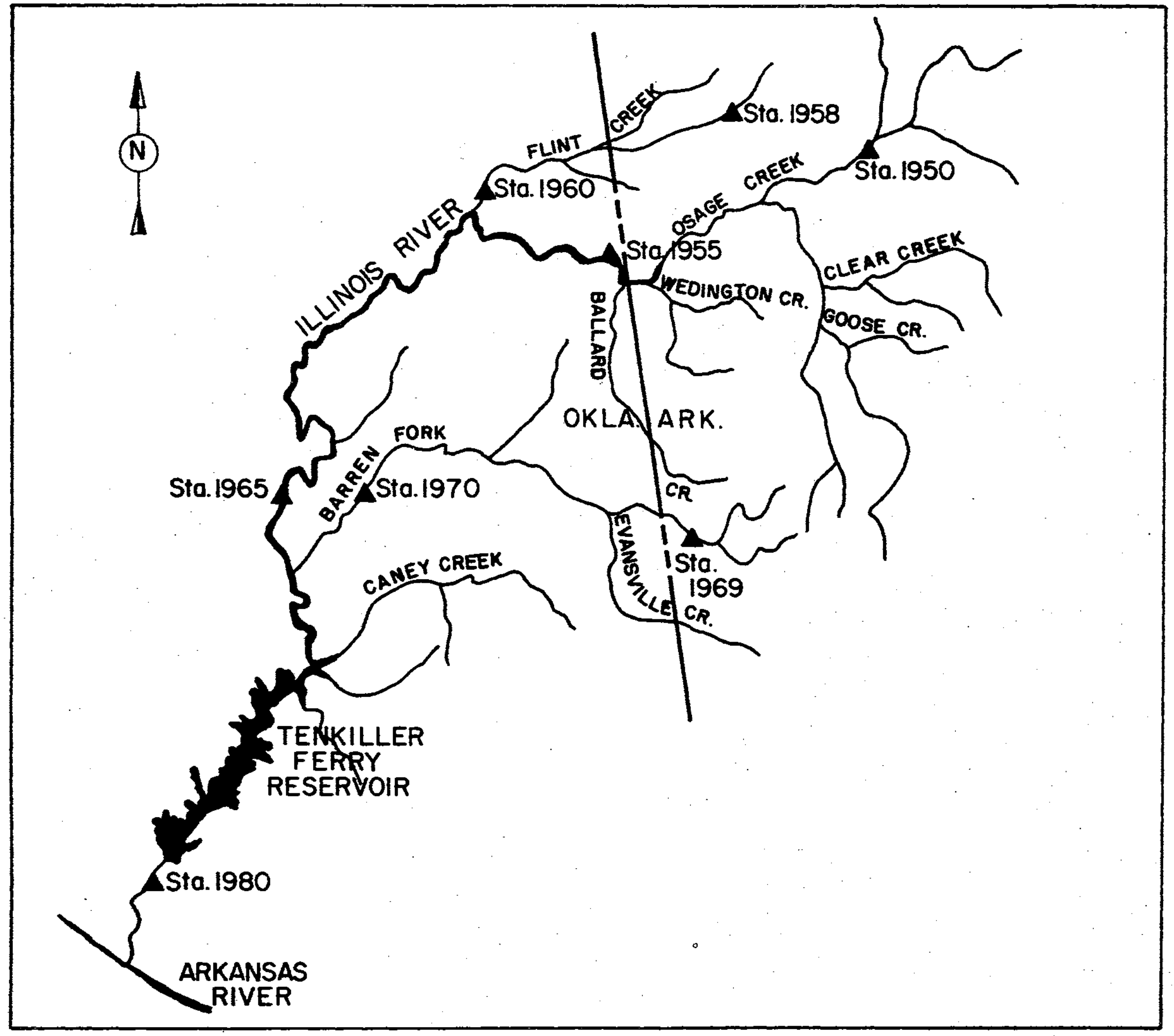

Fig. 1 - Map of the Illinois River Basin.

B. Synthesis of Missing Data

1. Mean Monthly Flows

Station 1965 covers thirty years of record on the Illinois River (1937 to present). The base period selected for this study covers the water years 1938 to 1965. Stations 1955, 1960, and 1970 have records covering only portions of the selected base period, and thus a method of synthesizing the missing data for these stations was required.

Hence, in order to synthesize the missing records, a method of cross-correlation between the flows at these stations and station 1965 during the overlapping period of record was used. Therefore, by establishing an average mean monthly ratio for each station compared to the base station, as well as their respective standard deviations (σ), the average mean monthly flows for stations 1955, 1960, and 1970 were established. The average percentage contribution and the standard deviation for each month at each of these three stations may be found in Table I.

In these studies, station 1965 was selected as the base station because it is on the main channel, it covers the entire base period, and there are no upstream reservoirs or ponds to give an unnatural flow record.

2. Computer Approach to Data Synthesis

To facilitate the handling of the mean monthly flow data, a computer program was developed. Each input data card was used to record the mean monthly flows for six

TABLE I

CROSS-CORRELATION OF MONTHLY RECORDS, STATIONS 1955, 1960, AND 1970 TO STATION 1965

	1955/1965			1960/1965			1970/1965		
Month	Ave. %	σ	σ as %	Ave. %	σ	σ as %	Ave. %	σ	σ as %
Oct.	72.9	7.15	9.8	12.9	1.83	14.2	23.7	5.79	25.0
Nov.	77.6	9.40	12.1	12.6	2.22	17.6	24.6	7.29	29.6
Dec.	69.4	6.03	8.7	13.8	2.51	18.2	29.6	8.54	28.8
Jan.	74.6	6.00	8.1	11.9	2.36	19.9	31.4	7.51	23.9
Feb.	71.2	5.30	7.4	11.4	2.09	18.3	33.2	6.54	19.7
Mar.	67.1	5.63	8.4	11.3	2.12	18.8	36.3	7.06	19.5
Apr.	59.4	7.54	12.7	13.5	3.35	24.8	35.5	4.87	13.8
May	65.8	5.82	8.9	11.6	1.98	17.0	41.0	9.70	23.6
June	63.0	6.04	9.6	12.7	2.56	20.2	29.9	9.37	31.3
July	69.5	11.30	16.3	12.6	1.86	14.8	31.0	7.07	22.8
Aug.	75.5	6.78	9.0	13.5	3.15	23.3	23.5	7.96	33.8
Sept.	73.2	7.74	10.6	15.2	3.71	24.4	24.0	9.32	39.0

months of each water year. Ratios of mean monthly flows derived from the overlapping years of record are used in conjunction with flows at station 1965 during the missing period to synthesize the mean monthly flows at stations 1955, 1960, and 1970. Fig. 2 shows the flow diagram for the synthesis of the mean monthly flows. The program for an IBM 1620 digital computer for the synthesis of the missing data may be found in the appendix of this report.

3. <u>Hydrograph Plotting</u>

After the missing data were synthesized, another program was devised to read the data and plot the mean monthly flow hydrographs for the gaging stations on the Illinois River. A flow diagram of the program is shown in Fig. 3, and the actual program is illustrated in the appendix.

The flow synthesis program and the hydrograph plotting program were loaded on a computer memory disk to be recalled at will to synthesize and plot flows of other gaging stations.

C. <u>Flood Synthesis</u>

To investigate the floods on the Illinois River, station 1965 was again used as a base station. Using the annual maximum series of floods occurring at this station it was found that the mean annual flood was 22,280 cfs. Also, the magnitudes of floods corresponding to different return periods were determined, and these floods are shown in Table II.

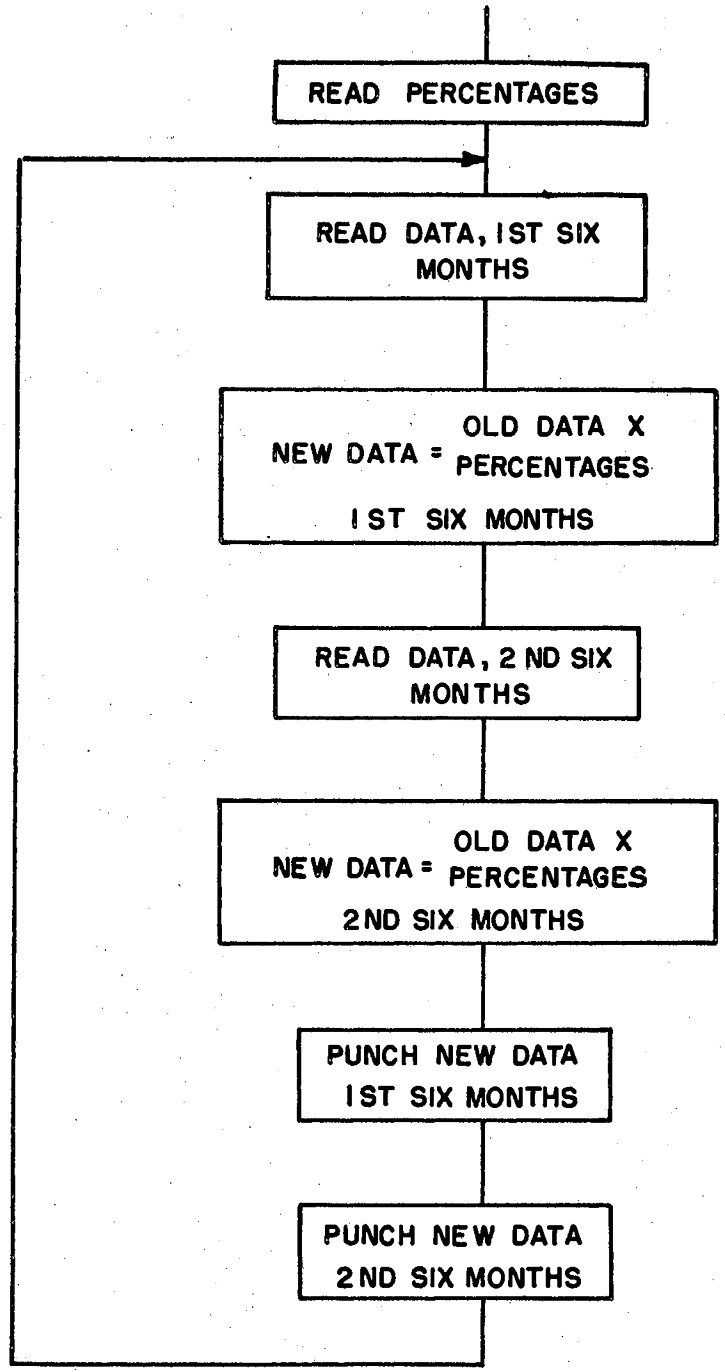

Fig. 2 - Computer Flow Diagram for Synthesis of Missing Mean Monthly Flows.

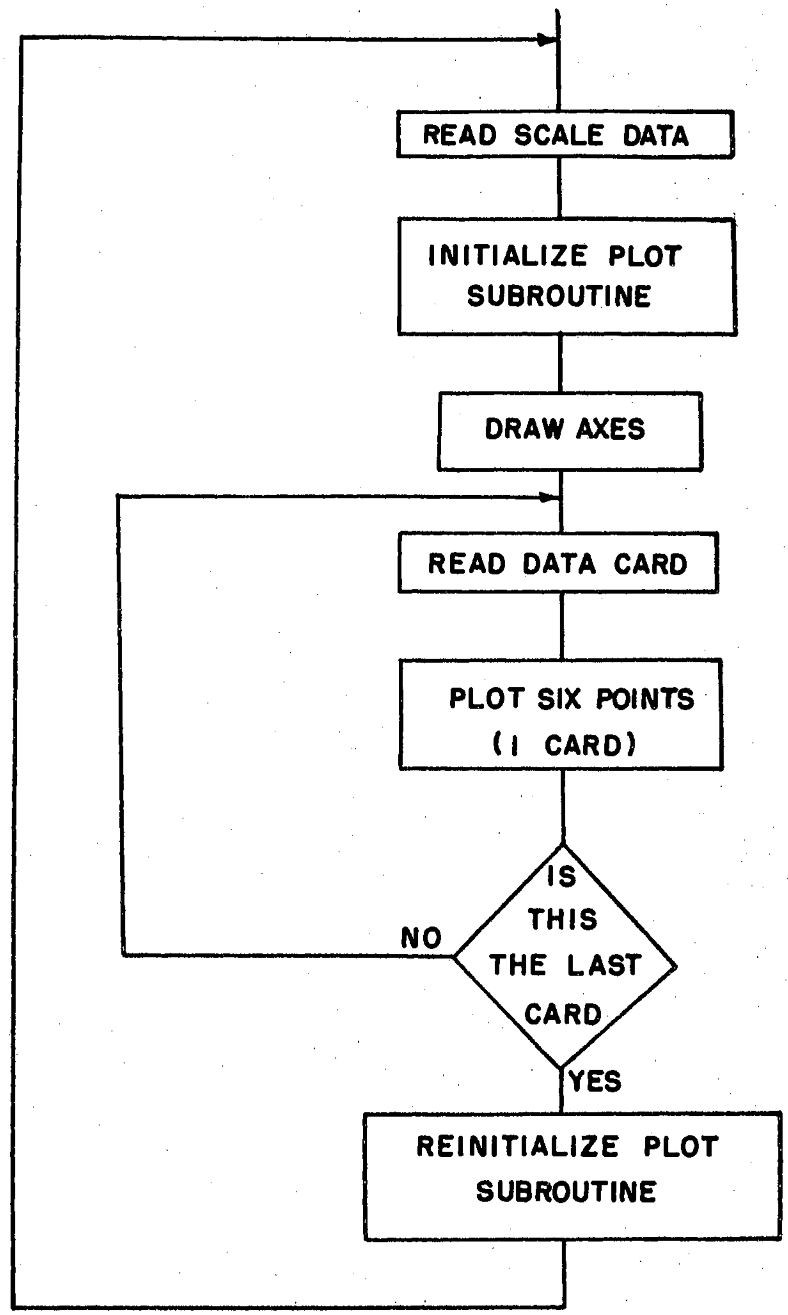

Fig. 3 - Computer Flow Diagram for Plotting River Hydrographs.

TABLE II

FLOODS OF VARIOUS RETURN PERIODS AS DERIVED FROM THE MAXIMUM ANNUAL SERIES FOR STATION 7-1965

Return Period Years	Flood Magnitude cfs	Return Period Years	Flood Magnitude cfs
5	40,330	30	75,680
10	54,480	50	85,280
15	62,280	60	88,780
20	67,880	75	92,980
25	72,180	100	98,480

TABLE III

FLOODS OF VARIOUS RETURN PERIODS AS DERIVED FROM THE PARTIAL DURATION SERIES FOR STATION 1965

Year	Month	Day	Discharge (cfs)	Partial Order	Return Period (tp, Years)
1941	Apr.	20	30,600	10	3.00
1943	May	11	65,000	2	15.00
1945	Mar.	20	37,200	5	6.00
	Apr.	15	58,300	3	10.00
1950	May	11	90,400	1	30.00
1957	Apr.	4	36,500	6	5.00
	June	3	12,700	30	1.00
1958	July	13	22,000	15	2.00

The floods at station 1965 were further investigated using the partial duration series analysis. Since the channel capacity at station 1965 is approximately 8000 cfs, and any flows exceeding this value will therefore overtop the banks to cause a flood, the periods of high flows less than 8,000 cfs were ignored. The partial duration series for the floods of records at station 1965 is shown in Table III. It is of interest to note that the flood of one year return period, called the marginal flood, has a magnitude of 12,700 cfs.

1. Flood Flows at Stations 1955 and 1960 Synthesized from Station 1965

The overlapping period of record between stations 1955 and 1960 and station 1965 is ten years. When cross-correlating the flood flows at these stations, it is necessary to consider the travel time from station 1955 and station 1960 to station 1965, since daily flows are utilized. It was found that the channel distance from station 1955 and station 1960 to station 1965 was 43 and 36 miles, respectively. If a velocity for floods of 3 to 4 feet per second is assumed, the travel time is between 22 and 27 hours, or approximately one day. In other words, flows recorded at stations 1955 and 1960 on a given day will be recorded at station 1965 on the next day.

The floods used for comparison, their duration, and the magnitude of their peaks are shown in Table IV.

TABLE IV

FLOODS AT STATIONS 1955, 1960, AND 1965

Duration, Days	Peak Flow, cfs		
	1955	1960	1965
April 1-16, 1957	22,800	2,830	36,500
May 16-21, 1957	11,300	2,630	18,000
May 21-June 1, 1957	20,100	4,290	31,200
June 3-June 9, 1957	8,110	908	12,700
June 9-June 18, 1957	6,570	1,350	10,200
July 6-19, 1958	13,600	3,960	22,000
May 4-10, 1960	14,400	4,000	18,600
May 18-31, 1960	5,430	1,690	9,450
July 24-31, 1960	23,700	460	23,200
May 4-15, 1961	33,400	4,320	42,000

The pattern of response of stations 1955 and 1960 as compared to station 1965 in cases of high flows and floods was included in a computer program. The flow diagram of that program is illustrated in Fig. 4 and the actual program is shown in the appendix. Using this diagram, it was possible to synthesize the daily flows at stations 1955 and 1960 from the daily flows at station 1965 over the period of missing records. Again, the program was loaded on a computer memory disk for recall to synthesize the floods during the years of missing records at stations 1955 and 1960.

2. Flood Flows at Station 1970 Synthesized from Station 1965

Station 1970 was put in operation in 1948, and there exist 17 years of overlapping records between this station on Barren Fork and station 1965 on the Illinois River. The tributary area of station 1970 is 307 square miles, or 32 per cent of the tributary area of station 1965. With a channel length of 150 miles, station 1970 yields a channel length to tributary area ratio of 0.49. The morphology of the basin is such that it contains numerous short tributaries draining into the main stream at almost equal intervals forming a physical shape similar to that of the backbone of a fish--a situation that invites extremely fast response to flood flows.

When determining the flow time lag between station 1970 and station 1965 based on an average flood flow veloc-

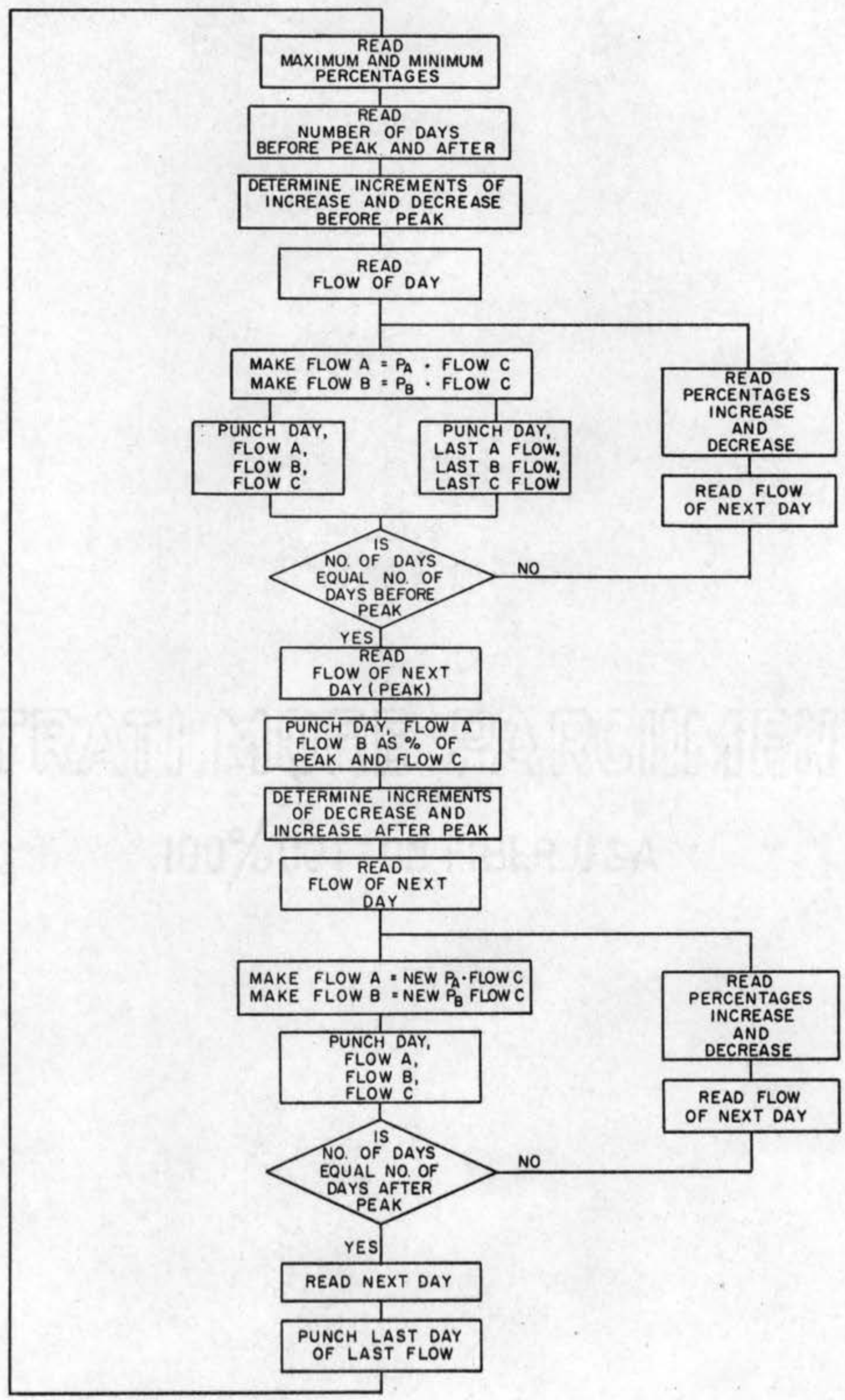

Fig. 4 - Computer Flow Diagram for Synthesis of Floods at Stations 1955 and 1960 from Station 1965.

ity of 3-4 f p s , it was found that flows at the farthest point on the basin contributing to station 1965 take 20-27 hours longer to reach station 1965 than flows from the farthest point on each basin tributary to station 1970. In other words, during a flood, the flows recorded at station 1970 on any day corresponded to flows recorded at station 1965 on the following day.

The floods used for comparison during the period of overlapping record, their magnitudes, durations, and ratios of peaks are shown in Table V. These findings have been transferred to a computer program and memory disk for the synthesis of missing flood flows at station 1970. The flow diagram of this program is shown in Fig. 5, and the actual program may be found in the appendix.

D. Description of the Floods to be Routed

After the flows from Barren Fork enter the mainstream of the Illinois River, the river then flows into Tenkiller Ferry Reservoir. Put in operation in 1952, this reservoir is a multipurpose project with flood control as the primary objective, and power development as the secondary objective. With capacities of 1,230,000 acre ft. at elevation 667.0 ft. (flood control pool), 791,900 acre ft. at elevation 642.0 ft. (spillway crest), 628,700 acre ft. at elevation 630.0 ft. (maximum power pool), and 283,100 acre ft. at elevation 594.5 ft. (conservation and minimum power pool), this reservoir was designed for a maximum release of 16,000 cfs (channel capacity at station 1980). In addition, this

TABLE V

CROSS-CORRELATION OF FLOODS AT STATIONS 1970 AND 1965

Water Year	Date	Peak Flow Station 1970 cfs	Peak Flow Station 1965 cfs	Ratio of Peak Flows
49-50	May 10, 11	22,500	90,400	24.8%
50-51	Feb. 20, 21	13,800	31,400	44.0%
53-54	May 2, 3	9,730	13,000	74.8%
54-55	Mar. 20, 21	4,780	6,930	68.9%
55-56	May 15, 16	3,050	5,700	53.5%
56-67	Apr. 3, 4	16,100	36,500	44.1%
57-58	July 13, 14	7,970	22,000	36.1%
58-59	July 23, 24	4,700	8,900	52.8%
60-61	May 7, 8	12,500	42,000	29.8%
64-65	Apr. 6, 7	4,780	11,000	43.4%

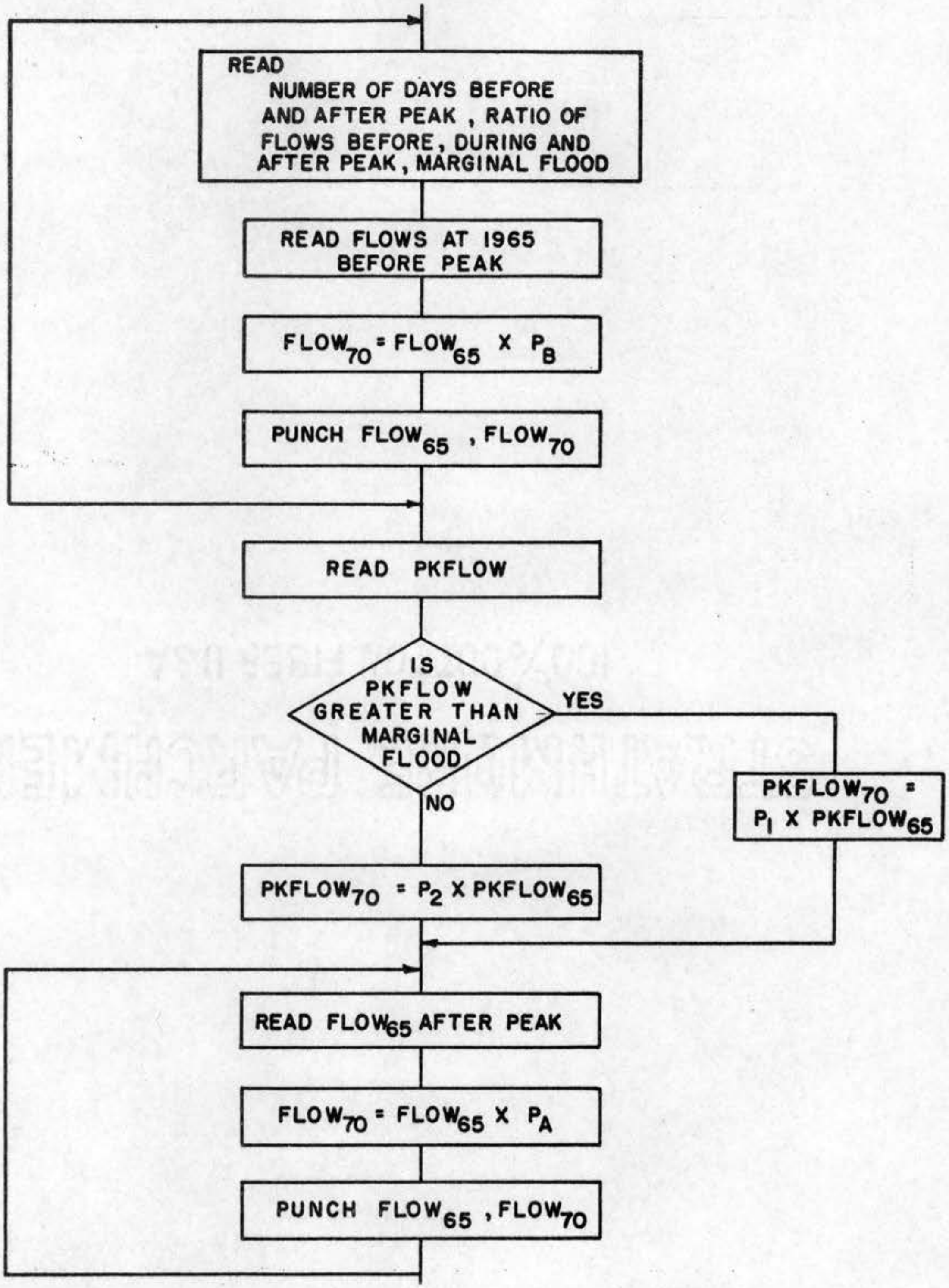

Fig. 5 - Computer Flow Diagram for Synthesis of Floods at Station 1970 from Station 1965.

reservoir was co-ordinated with Keystone, Wister, Oolagah, Ft. Gibson, and Eufaula reservoirs so that the gaging station at Ft. Smith, Arkansas, on the Arkansas River, would not record flows in excess of the channel capacity at that point (150,000 cfs). Minimum releases for nagivation, agriculture, recreation, and other downstream requirements are also co-ordinated.

With the use of 15 years of synthesized records before the reservoir was constructed and 14 years of actual records after it was put in operation, it was then possible to determine the dampening effect of the reservoir on flood peaks of varying magnitude and duration. By routing floods down the Illinois River prior to the year 1952 to station 1980 and then routing floods of similar size after 1952 to station 1980, and comparing the actual peaks with the routed peaks at station 1980, the dampening effect of the reservoir was determined.

The floods routed prior to 1952, their magnitudes, and their peaks compared to the channel capacity at station 1965 are shown in Table VI. It may be noted that the routed flood peaks are approximately ½, 1, 1½ and 3 times the channel capacity at station 1965. The maximum flood of record (90,400 cfs) was also routed.

There were two reasons for routing floods prior to the year 1952. First, it was necessary to determine the expected inflow from the intervening area between stations 1965, 1970, and 1980 for various stages of flow at station

TABLE VI

FLOODS ROUTED ON THE ILLINOIS RIVER
PRIOR TO YEAR 1952

Date of Peak	Peak at Station 1965	Ratio of Peak to Channel Capacity at Station 1965
April 26, 1947	3,840 cfs	3,840/7,900 = 0.49
February 14, 1950	7,980 cfs	7,980/7,900 = 1.01
February 16, 1949	13,300 cfs	13,300/7,900 = 1.68
November 1, 1941	25,000 cfs	25,000/7,900 = 3.16
May 11, 1950	90,400 cfs	90,400/7,900 = 11.44

1965. Second, these floods were routed as a check on the selected flood routing method.

In addition to routing these floods through the reach of the Illinois River that now contains the reservoir, the same floods were routed from stations 1955 and 1960 through station 1965. Therefore, by measuring the flows at stations 1955 and 1960 it was possible to predict peaks occurring about two days later at station 1980.

Finally, to determine the dampening effect of the reservoir on flood peaks occurring after the reservoir was put in operation, floods, having magnitudes similar to those in Table VI, were routed. These are reported in Table VII. Using the relatively high predicted flow values in comparison with the relatively low observed flow values at station 1980 it was then possible to determine the effect of the reservoir on a variety of floods on the Illinois River.

E. Routing Method

After a critical review of the literature on flood routing presented earlier, it was concluded that the method proposed by C. O. Wisler and E. F. Brater (27) was best suited for the routing of floods on the Illinois River because it was one of the few methods that developed a hydrograph for the local inflow. Based on daily mean flows and using flow hydrographs at both the upstream and the downstream ends of a reach, it was found that this method needed little modification prior to its application to the floods on the Illinois River.

TABLE VII

FLOODS ROUTED ON THE ILLINOIS RIVER
AFTER YEAR 1952

Date of Peak	Peak at Station 1965	Ratio of Peak to Channel Capacity at Station 1965
June 15, 1961	3,820 cfs	3,820/7,900 = 0.48
May 4, 1958	7,980 cfs	7,980/7,900 = 1.01
May 3, 1953	13,000 cfs	13,000/7,900 = 1.65
July 26, 1960	23,200 cfs	23,200/7,900 = 2.94

According to this approach, the difference in the areas under inflow and outflow hydrographs for a given reach will give the volume of storage within the intervening reach on any given day. The continuity equation was used to determine the daily inflow from the intervening reach. This equation may be written as:

$$S_1 + \frac{(I_1+I_2)}{2} \Delta T + I' \Delta T - \frac{(O_1+O_2)}{2} \Delta T = S_2 \qquad (14)$$

in which

S_1 = storage on first day, in sfd

S_2 = storage on second day, in sfd

I_1 = inflow on first day, in cfs

I_2 = inflow on second day, in cfs

I' = inflow from intervening area, in cfs

O_1 = outflow on first day, in cfs

O_2 = outflow on second day, in cfs

ΔT = time interval, taken as one day

After the hydrographs for inflow, outflow, and inflow from the intervening reach were plotted, it was found that, in most cases, the inflow peak occurred a day before the outflow peak. This was primarily due to the time lapse of one day between the recording of flows at the upstream gaging station and at the downstream gaging station. Also, the peak of inflow from the intervening reach was found to coincide with the outflow peak. This was due to the fact that the inflow from the intervening reach was not taken as the average inflow over a two-day period, as the inflow and outflow in the above equation are taken, but rather it was determined as an inflow that would be recorded only at the downstream gaging station on the second day. This will be further illustrated by an example in the next section of this chapter.

Hence, with these three hydrographs (I, I', and O) available, the predicted outflow hydrograph (O') was plotted by adding the inflow on the first day (I) to the inflow from the intervening reach (I') on the second day and plotting this value as a calculated flow that would be expected on the second day.

This procedure was repeated for the floods shown in Table VII to determine the expected inflow from the intervening reach for various magnitudes of flood peak. Using these expected I' inflow values, the floods of unknown outflow at station 1980 (see Table VIII) were predicted, and the efficiency of Tenkiller Ferry Reservoir in flood peak dampening was found by comparing the routed peaks with the observed peaks at that station.

F. Sample of Flood Routing Technique

In the previous section, the method selected for the routing of the floods in the Illinois River Basin was discussed briefly. To further explain this method, one of the floods chosen in Table VI will be routed as an example. First, the flood will be routed from stations 1955 and 1960 to station 1965, and then from stations 1965 and 1970 to station 1980. Then a flood of similar size will be selected from Table VII. This flood will be routed from stations 1965 and 1970 to station 1980, using the data obtained from the first flood. To observe the reservoir dampening effect on flood peaks, the routed peak will be compared to the observed peak.

1. Routing Floods from Stations 1955 and 1960 to Station 1965 (before Year 1952).

The flood of February 16, 1949, with a peak at Station 1965 of 13,300 cfs and a duration of 21 days, was selected as an example. The ratio of the peak to the channel capacity at station 1965 (7,900 cfs) is 1.68. The combined inflow from stations 1955 and 1960, as shown in Fig. 6, is plotted as the inflow hydrograph (I). The flow recorded at station 1965 is plotted as the outflow hydrograph (O). Table VIII shows the values of inflow and outflow during the flood.

Assuming a base flow of 700 cfs, the inflow storage (S_I) is computed. To illustrate, the storage above base flow for the last day of the flood is determined by finding the area under the inflow hydrograph for that day. Using a straight line between flow values, a triangle is formed, the area of which is one-half the base times the height. In this case, the base is one day and the height is 725 cfs - 700 cfs = 25 cfs. Therefore, the inflow storage for the twentieth day is 12.5 sfd. On the nineteenth day, the storage is (762 cfs - 725 cfs) x $\frac{1}{2}$ x 1 = 18.5 plus (725 cfs - 700 cfs) x 1 = 25.0 sfd, or 43.5 sfd. This procedure is

TABLE VIII

FLOOD OF FEBRUARY 16, 1949, AT STATION 1965 FROM STATIONS 1955 AND 1960

(1) Day	(2) I	(3) O	(4) (I+O)	(5) S_I	(6) S_O	(7) S_I'	(8) I'	(9) O'
0	725	700	1,425	-	-	7.0	77.5	777.5
1	1,164	907	2,071	-	-	67.0	263.0	988.0
2	5,854	1,420	7,272	-	-	1,170.0	500.0	1,664.0
3	11,438	6,970	18,408	7,244.0	18,723.0	11,499.0	1,400.0	7,254.0
4	4,410	13,300	17,710	2,954.5	8,318.0	5,363.5	1,697.0	13,135.0
5	2,899	5,150	8,049	1,878.5	3,368.0	1,489.5	997.0	5,407.0
6	2,258	3,400	5,658	1,333.5	2,123.0	789.5	684.0	3,583.0
7	1,809	2,660	4,469	1,016.5	1,493.0	476.5	608.0	2,417.0
8	1,624	2,140	3,764	827.5	1,128.0	300.5	406.0	2,215.0
9	1,441	1,930	3,371	713.5	918.0	204.5	378.0	2,002.0
10	1,386	1,720	3,106	612.5	783.0	171.0	355.0	1,796.0
11	1,238	1,660	2,898	510.5	668.0	157.0	287.0	1,673.0
12	1,183	1,490	2,673	472.5	553.0	80.5	114.0	1,352.0
13	1,162	1,430	2,592	406.5	313.0	-93.0	530.0	1,713.0
14	1,050	1,410	2,460	315.5	438.0	123.0	782.0	1,944.0
15	980	1,280	2,260	258.0	333.0	75.0	274.0	1,324.0
16	936	1,200	2,136	201.0	268.0	67.0	274.0	1,254.0
17	866	1,150	2,016	136.0	203.0	67.0	251.0	1,187.0
18	806	1,070	1,876	84.0	128.0	44.0	231.0	1,097.0
19	762	1,000	1,762	43.5	67.5	24.0	216.0	1,022.0
20	725	949	1,674	12.5	21.0	8.5	207.0	969.0
21	700	907	1,607	0.0	0.0	0.0	200.0	925.0

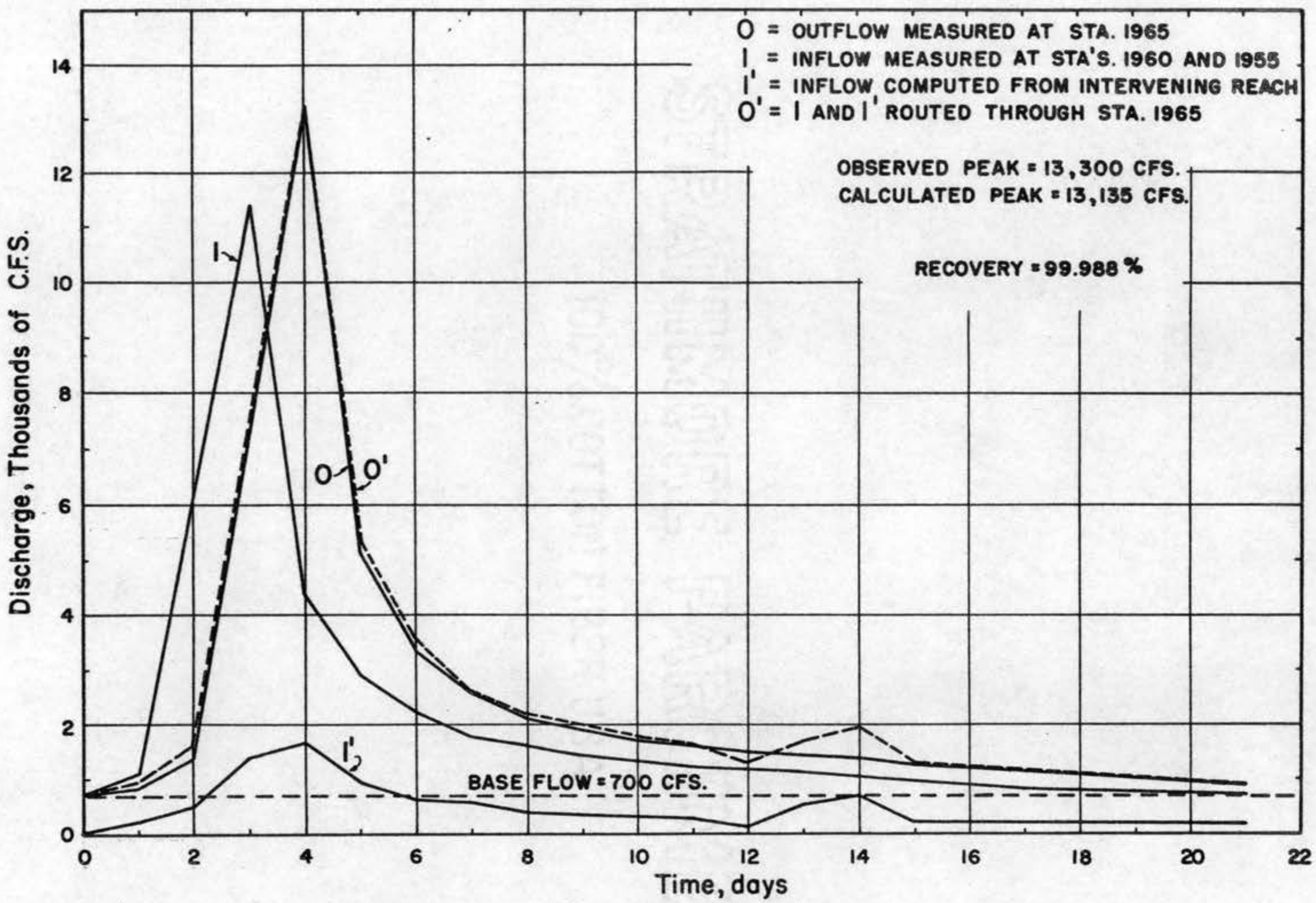

Fig. 6 - Flood of February 16, 1949, routed from Stations 1955 and 1960 to Station 1965.

then repeated back to the day the flood peak occurred.

To determine the outflow storage (S_O), a base flow of 907 cfs is assumed. The values of outflow storage are computed in a similar manner as those of inflow storage. For example, on the twentieth day, the outflow storage is (949 cfs - 907 cfs) x $\frac{1}{2}$ x 1 = 21.0 cfs, and on the nineteenth day the storage is (1000 cfs - 949 cfs) $\frac{1}{2}$ x 1 = 25.5 sfd, plus (949 cfs - 907 cfs) x 1 = 42.0 sfd, or a total of 67.5 sfd. This procedure is again repeated back to the day of the flood peak.

The storage in the intervening reach (S_I') is then found by subtracting the value of inflow storage from outflow storage for each day back to the flood peak. This gives an estimate of the storage in the channel for each day during the flood. In essence, the difference in the areas under the inflow and outflow hydrographs, above the base flow, was determined.

A relationship was then established between values of S_I' and corresponding values of (I+O). From this curve (Fig. 7), the S_I' values for the flood period prior to the peak are determined. These values, in addition to the calculated S_I' values after the peak, are shown in column (7) of Table VIII.

Using the continuity equation, the daily I' values may be found. To solve for I', the continuity equation was modified to read

$$I' = \frac{(2S_2+O_1+O_2) - (2S_1+I+I_2)}{2} \tag{15}$$

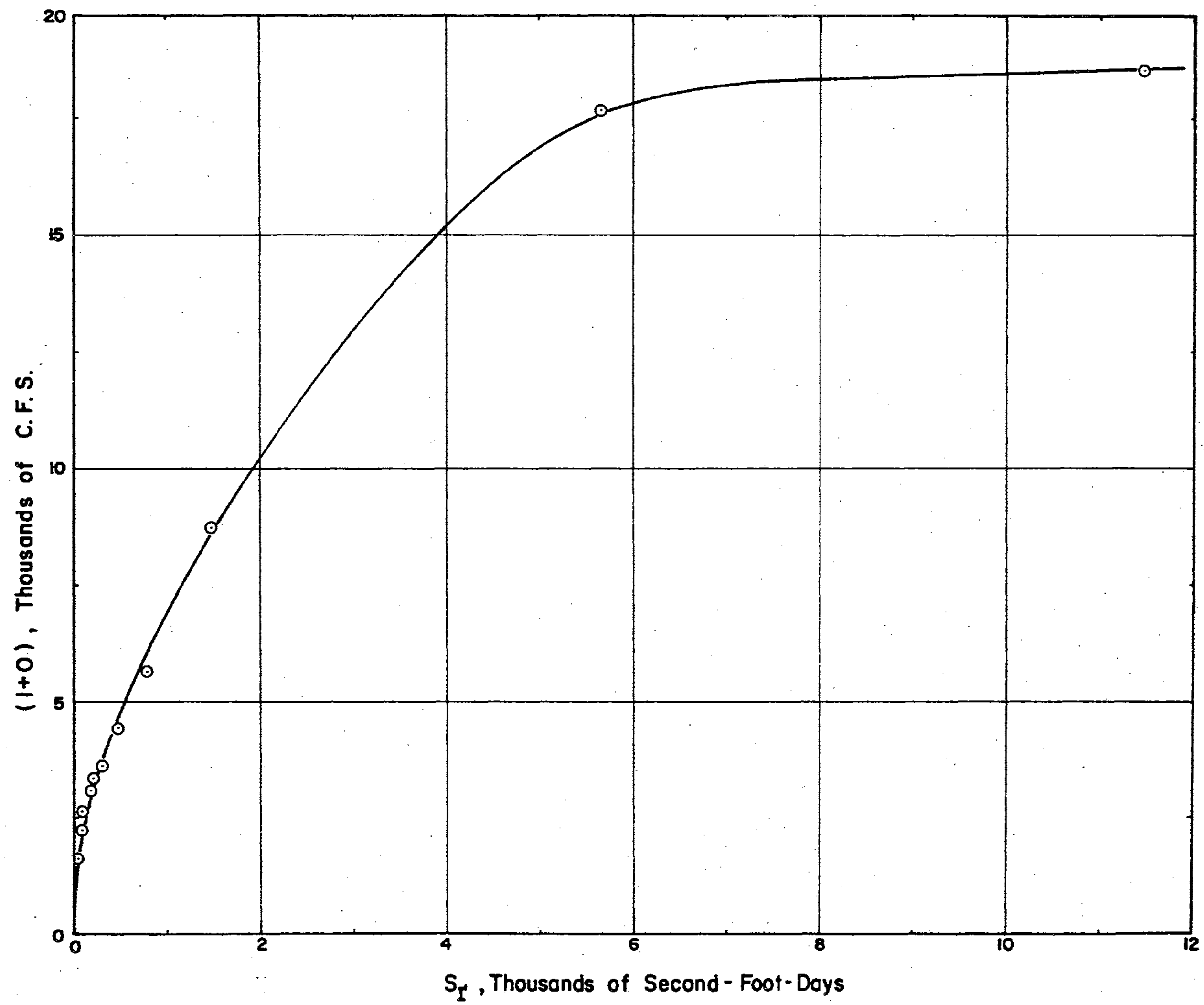

Fig. 7 - Storage-Discharge Relationship for Flood of February 16, 1949.

Hence, for each daily value of inflow, outflow, and storage the daily local inflows can be calculated. These flows are recorded in Table VIII (col. 8) as flows which would be registered at station 1965 on the same day as the outflow. The I' values on one day are then added to the inflow values of the preceding day. For example, on the fourth day the local inflow was 1,697 cfs, and the inflow on the third day was 11,438 cfs. This adds up to an outflow on the fourth day of 13,135 cfs.

The hydrograph for the local inflow (I') as well as that for the calculated outflow (O') are both shown in Fig. 6. It can be seen that the actual outflow hydrograph and the calculated outflow hydrograph compare very favorably.

2. <u>Routing Floods from Stations 1965 and 1970 to Station 1980 (before Year 1952)</u>

Again, the flood of February 16, 1949, is used as an example. An observed peak of 16,200 cfs at station 1980 due to peaks at stations 1965 and 1970 of 13,300 cfs and 3,325 cfs, respectively, was predicted as a peak of 16,258 cfs by the same procedure described earlier. Fig. 8 and Table IX are shown to illustrate the hydrographs and the calculations for the flood through this reach.

Since there is a time lag through the reach of approximately one day, the inflow hydrograph values for each day are again added to the I' hydrograph values on the succeeding day to plot the routed outflow hydrograph.

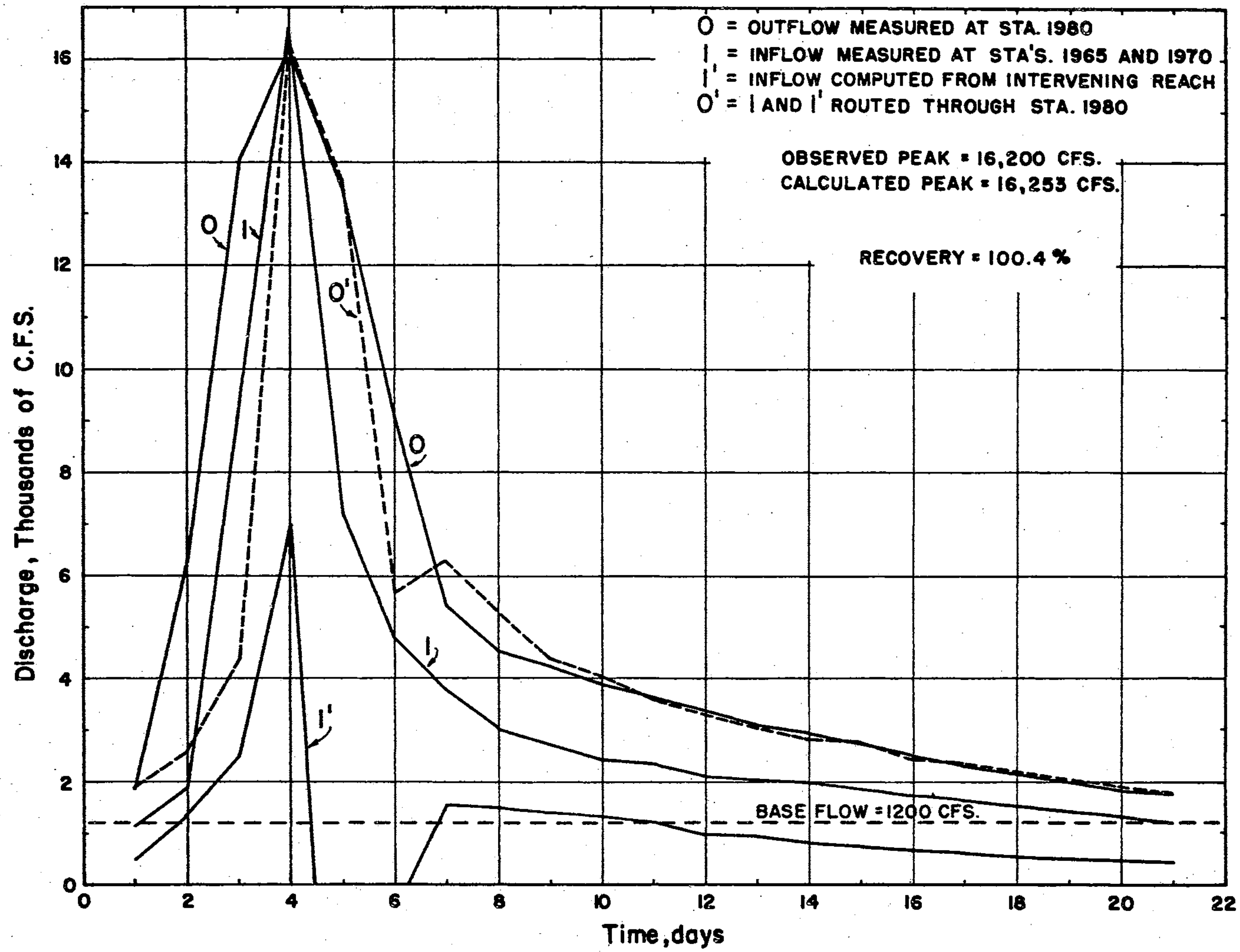

Fig. 8 - Flood of February 16, 1949, routed from Stations 1965 and 1970 to Station 1980.

TABLE IX

FLOOD OF FEBRUARY 16, 1949, AT STATION 1980 FROM STATIONS 1965 AND 1970

(1) Day	(2) I	(3) O	(4) (I+O)	(5) S_I	(6) S_O	(7) $S_{I'}$	(8) I'	(9) O'
1	1,206	1,880	3,086	-	-	0	500	
2	1,888	6,410	8,298	-	-	1,000	1,380	2,586
3	9,270	14,000	23,270	-	-	3,100	2,521	4,409
4	16,625	16,200	32,825	11,750	10,450	-1,300	6,988	16,253
5	7,261	13,500	20,761	4,828	7,600	2,772	-2,998	13,629
6	4,794	7,100	11,894	3,072	4,570	1,498	1,597	5,664
7	3,750	5,440	9,190	2,183	3,280	1,097	1,515	6,309
8	3,017	4,520	7,537	1,669	2,690	1,021	1,500	5,250
9	2,721	4,260	6,981	1,373	2,370	997	1,377	4,394
10	2,425	3,880	6,305	1,183	2,060	877	1,300	4,021
11	2,340	3,640	5,980	1,020	1,810	790	1,172	3,597
12	2,100	3,380	5,480	858	1,530	672	973	3,313
13	2,016	3,080	5,096	802	1,275	473	929	3,029
14	1,988	2,910	4,898	696	1,105	409	792	2,808
15	1,804	2,700	4,504	548	890	342	729	2,717
16	1,692	2,480	4,172	456	685	229	654	2,458
17	1,621	2,290	3,911	365	520	155	627	2,319
18	1,508	2,150	3,658	259	385	126	534	2,155
19	1,410	2,020	3,430	174	210	34	502	2,010
20	1,338	1,800	3,138	69	50	0	472	1,882
21	1,278	1,760	3,038	0	0	0	450	1,788

By routing this flood through this reach prior to 1952, it is possible to predict the values of local inflow due to a flood of similar magnitude after year 1952.

3. <u>Routing Floods from Stations 1965 and 1970 to Station 1980 (after Year 1952)</u>

Since Tenkiller Ferry Reservoir was put in operation in 1952, the flows recorded at station 1980 will not be the same as those due to floods of similar size prior to 1952. However, it is possible to predict what these flows could have been had the reservoir not been in operation. By comparing the actual flows with the flows that could have occurred, it is possible to estimate the dampening effect of the reservoir on floods.

As an example, the flood of May 3, 1954, at station 1980 was selected. Prior to entering the reservoir, peaks of 13,000 cfs at station 1965 and 9,730 cfs at station 1970 were observed. Since the flood is similar to that of February 16, 1949, the values of I' from the 1949 flood may be used. Table X shows the inflows, both channel and local, and the predicted outflow at station 1980. Also shown is the observed outflow. Fig. 9 depicts these values graphically.

This method was then repeated for floods of various magnitudes. The dampening effect of the reservoir on floods was determined using this method, and is discussed in the next chapter.

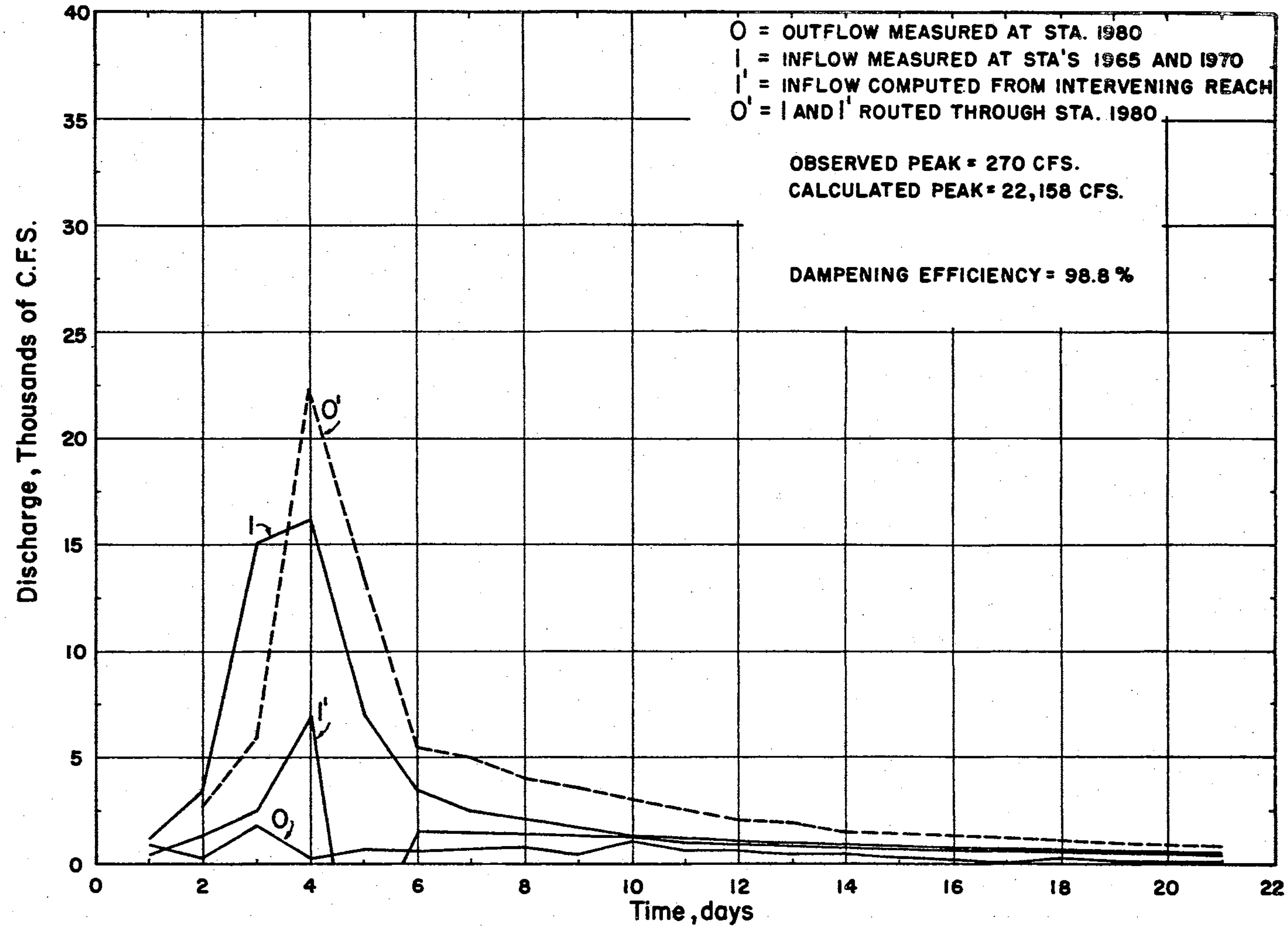

Fig. 9 - Flood of May 3, 1954, showing Difference between Actual and Calculated Outflow at Station 1980.

TABLE X

FLOOD OF MAY 3, 1954, AT STATION 1980 FROM STATIONS 1965 and 1970

Day	I_{1965}	I_{1970}	I_{Total}	I'	O'	O
1	179	1,190	1,369	500		904
2	1,810	1,720	3,550	1,380	2,749	305
3	5,440	9,730	15,170	2,521	6,071	1,770
4	13,000	3,230	16,230	6,988	22,158	270
5	5,470	1,600	7,070	-2,998	13,232	731
6	2,600	984	3,584	-1,597	5,473	710
7	1,870	686	2,556	1,515	5,099	776
8	1,520	766	2,286	1,500	4,056	785
9	1,240	505	1,745	1,377	3,663	515
10	1,050	382	1,432	1,300	3,045	132
11	859	320	1,179	1,172	2,604	740
12	746	276	1,026	973	2,152	751
13	650	245	895	929	1,954	248
14	554	223	777	792	1,687	264
15	481	195	676	729	1,506	380
16	432	175	607	654	1,330	236
17	404	155	559	627	1,234	67
18	376	153	529	534	1,093	380
19	341	143	484	502	1,031	56
20	327	127	454	472	956	17
21	314	112	426	450	904	10

CHAPTER IV

RESULTS

A. Synthesis of Flood Flows

1. Floods at Stations 1955 and 1960 Synthesized from Station 1965

When the contributions of stations 1955 and 1960 were compared as a percent of the flow registered at station 1965, it was found that the ratios (Fig. 10) were invariably the same for all floods at station 1965 which were in excess of the marginal flood (12,700 cfs). Fig. 10 illustrates that the percent of flow at station 1965 due to station 1960 varies from 6 per cent to 16 per cent, and the percent of flow at station 1965 from station 1955 varies from 65 per cent to 80 per cent. However, the peak flow at station 1955 did not occur at the same time as that at station 1960. It was felt that, at the peak, station 1955 contributed most of the flow, since it drains a larger drainage area than does station 1960. This large volume of flow took up the greatest portion of the channel capacity, causing a rise in the water level in the channel, and resulting in a retardation of backwater effect on the flow from station 1960. This, in turn, caused station 1960 to contribute a lower percent-

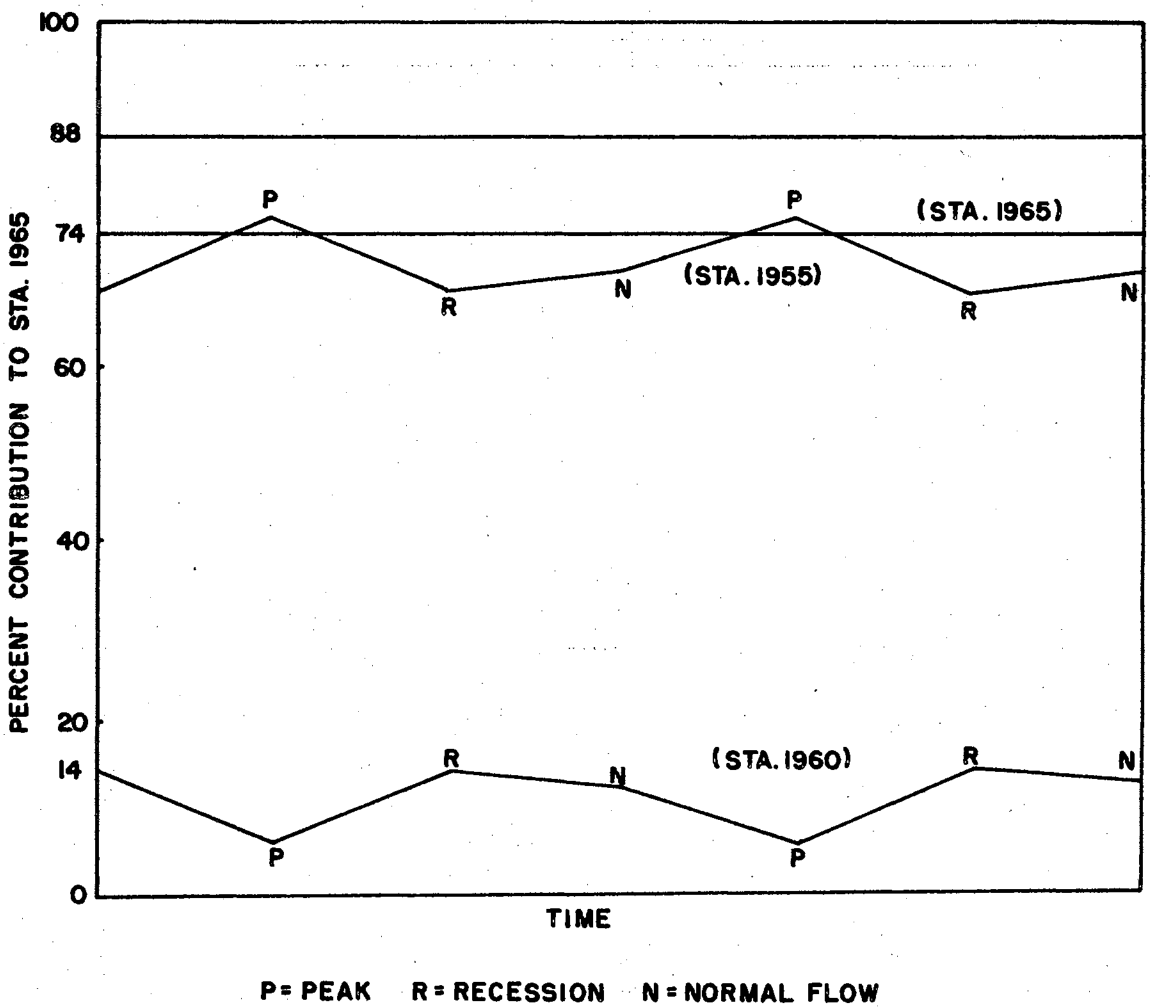

Fig. 10 - Ratios of Contribution during Floods between Stations 1955 and 1960 and Station 1965.

age of flow at the time that the peak from station 1955 was passing the confluence of the tributary with the main stream.

However, after the peak had passed, the flows from station 1960 began to increase due to the then available channel capacity and also due to the extra flow that was kept in channel storage in the tributary while the greater portion of flow was passing through the channel.

It was found that the morphology of the basin affects the response to high flows in such a fashion that the contribution from a certain gaging station as compared to another gaging station is not only proportional to the ratio of drainage areas, but also to the ratio of the length of channel per square mile of tributary area as well as to the physical characteristics of the basin and its tributaries.

The sum of flood flows at stations 1955 and 1960 constituted about 88 per cent of the peak flows at station 1965. The combined tributary areas of both station 1955 and station 1960 is 745 square miles, and that of station 1965 is 959 square miles, yielding a ratio of areas of 0.78. The length of channel draining stations 1955 and 1960 is 345 miles, yielding a channel length to tributary area ratio of 0.46. Also, the length of channel draining station 1965 is 390 miles, yielding a ratio of 0.41. Thus, the ratio of flood contribution by both stations 1955 and 1960 to station 1965 by this method was found to be

$$0.78 \times \frac{0.46}{0.41} = 0.88$$

However, when these ratios were broken down into ratios for individual stations, it was found that the ratios of channel length to tributary area for stations 1955, 1960, and 1965 were 0.46, 0.50, and 0.41, respectively. Also, the ratios of tributary areas of stations 1955 and 1960 to that of station 1965 were found to be 0.66 and 0.11, respectively. Thus, in comparing flood flows at station 1955 to those at station 1965, the expected ratio of flows was found to be

$$0.66 \times \frac{0.46}{0.41} = 0.74$$

Also, for the flood flows of station 1960 compared to station 1965, the expected ratio was found to be

$$0.11 \times \frac{0.50}{0.41} = 0.14$$

It may be noted that the values of 0.74 for station 1955 and 0.14 for station 1960 agree with the percentage peaks for these stations compared with that at station 1965 as shown in Fig. 6. As a result, in synthesizing the missing flood flows at stations 1955 and 1960 from available flood flows at station 1965, the maximum flow at both stations was taken as approximately 0.88 of the flow at station 1965. The minimum flow at both stations was taken as approximately 0.74 of the flow at station 1965, and the flows were divided among the stations according to the magnitude of the flood peak, the duration of the flood, and the principals discussed above.

For floods under 12,700 cfs in magnitude it was found that the contribution of flows from station 1960 were high, approximately 17 per cent, and those from station 1960 were low, approximately 58 per cent. The reason for such behavior lies in the fact that a fast response to high flows from small tributary areas is expected, while large tributary areas tend to attenuate the peaks of such floods. In addition, the smaller cross-section of the tributary on which station 1960 was placed as compared to the cross-section of the main channel would cause higher flow velocities and immediate drainage of floods of relatively smaller magnitude.

2. Floods at Station 1970 Synthesized from Station 1965

It was noted (Fig. 11) that there existed a marked difference in the response of floods lower than the marginal flood (12,700 cfs) and floods higher than the marginal flood. The ratio of flows at station 1970 to station 1965 before the peak was noted to be approximately 0.32--the same as the ratio of tributary areas. However, the flow ratios after the peak were found to vary from 0.38 to 0.45. This reflects the effects of the morphology of both basins and the faster response of a small basin to the higher flows.

It may be pointed out that the average ratio of flow after the peak of 41 per cent is approximately equal to the basin tributary areas ratio of 0.32 multiplied by the ratio of channel length above station 1970 to its tributary

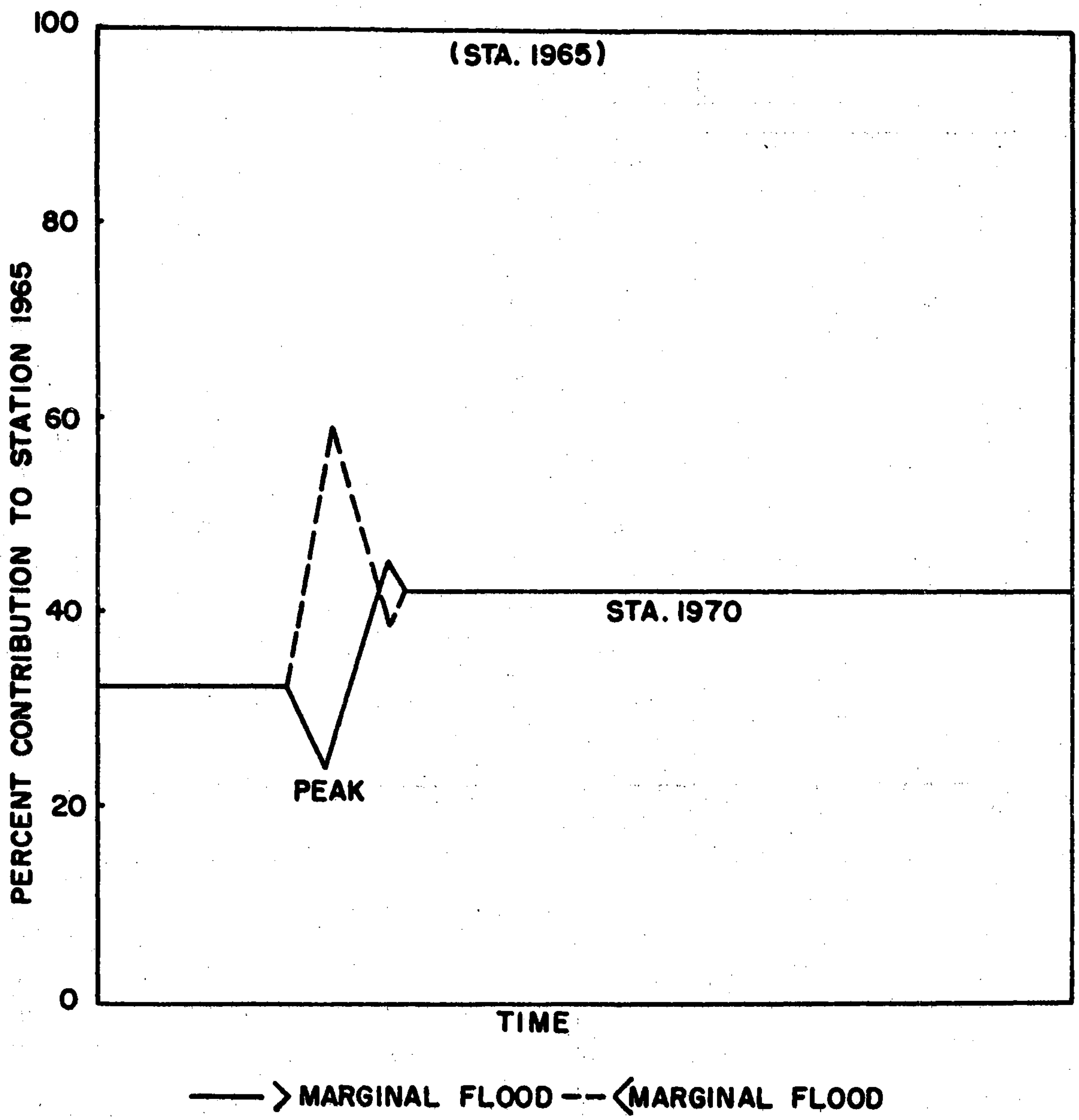

Fig. 11 - Ratio of Contribution during Floods between Station 1970 and Station 1965.

divided by the ratio of channel length above station 1965 to its tributary area. In other words, the average ratio of flow contribution from station 1970 after the peak is equal to

$$0.32 \times \frac{0.49}{0.41} = 0.40$$

However, the percent of flow from station 1970 was seen to vary from a value less than 30 per cent to a value higher than 55 per cent, depending on the magnitude of flow at station 1965. For flood peaks at station 1965 less than the marginal flood, it was noted that the ratio of flow from station 1970 was as high as 60 per cent. On the other hand, for floods at station 1965 higher than 12,700 cfs, the ratio of flow at station 1970 was as low as 20 per cent. Again, this was termed as further evidence of the effect of the basin morphology on high flows. In the case of floods less than the marginal flood, the small basin with higher channel length to drainage area ratio responds much faster and carries the flood peak without much attenuation, while the large basin flattens the peak so that the ratio of peaks becomes much greater. In addition, for floods of smaller magnitude, the channel capacity is not exceeded and no hindrance of flow is likely to occur at the confluence of the channels.

Under the conditions of floods of magnitudes greater than the marginal flood, the flows on the Illinois River at station 1965 are so great and have such high velocities that they will rapidly reach the confluence of the two streams and hinder the flows entering from station 1970 on Barren Fork. This will cause a retardation of tributary flows from station 1970 until the peak has passed, and the mainstream channel capacity can again handle the floods from the small basin.

B. Determination of Local Inflow

The previous chapter introduced the method used to calculate the inflow from the intervening area for floods prior to the construction of Tenkiller Ferry Reservoir. The floods considered ranged in magnitude from flows of approximately one-half the channel capacity at station 1965 up to, and including a flood of 90,400 cfs--the maximum flood of record. These floods were routed in two steps: first, from stations 1955 and 1960 to station 1965; and second, from stations 1965 and 1970 to station 1980. The approximate local inflows were determined for these floods.

1. Flood of April 26, 1947

This flood was observed to peak at station 1965 at 3,840 cfs, or about one-half the channel capacity at that gaging station. Inflows from stations 1955 and 1960 of 3,072 cfs and 230 cfs, respectively, on April 25 were routed to station 1965. A calculated peak of 3,932 cfs at station 1965 was within 2.14 per cent of the observed 3,840 cfs peak. Fig. 12 shows the inflow, outflow, local inflow, and routed outflow hydrographs of this flood.

This same flood was then routed from stations 1965 and 1970 to station 1980, where a peak of 5,100 cfs was

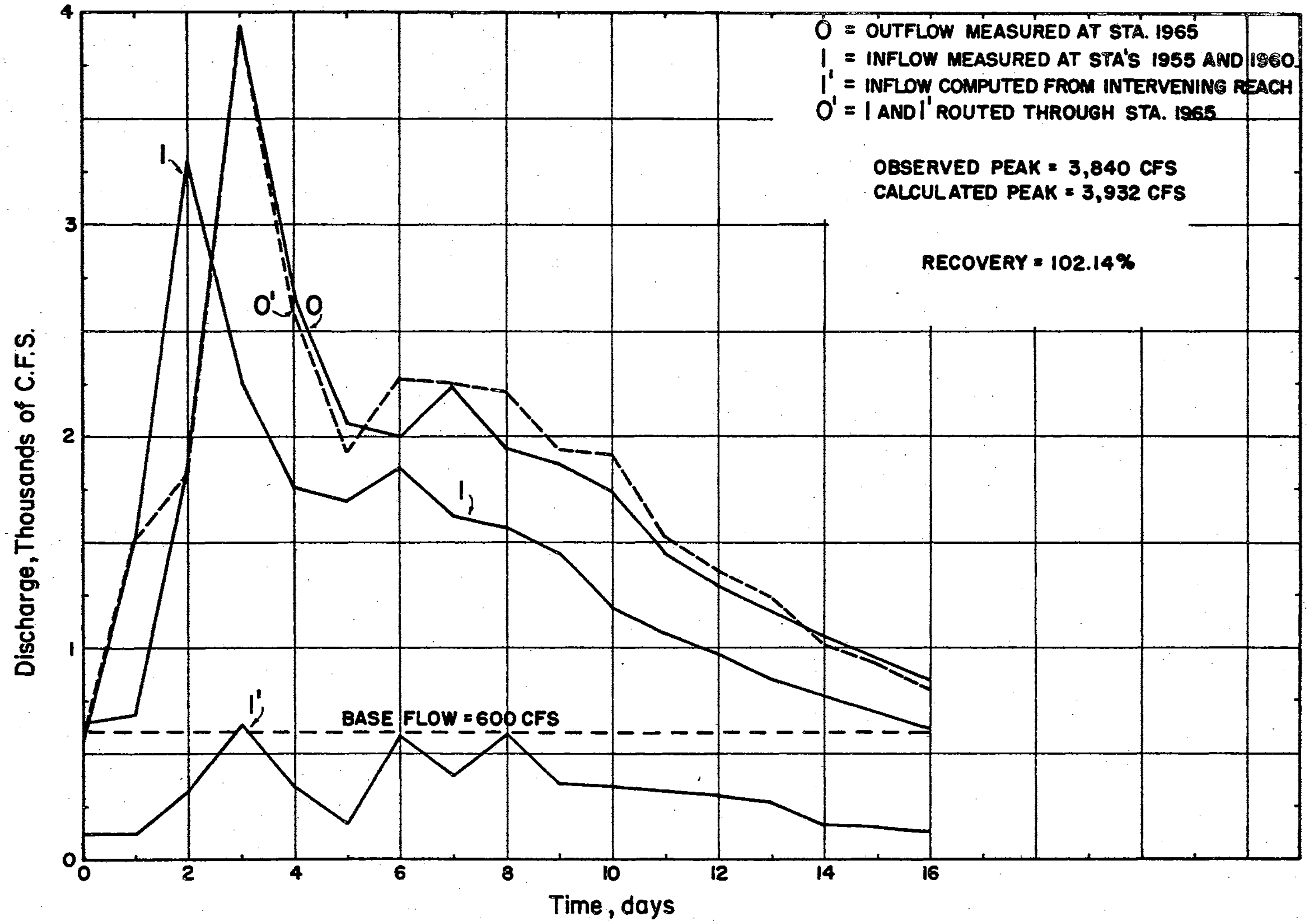

Fig. 12 - Flood of April 26, 1947, routed from Stations 1955 and 1960 to Station 1965.

observed. The inflows from station 1965 of 3,840 cfs and from station 1970 of 1,574 cfs yielded a routed peak of 5,087 cfs at station 1980. Hence, 99.75 per cent of the observed flow at station 1980 was predicted by this method. The applicable hydrographs are shown in Fig. 13.

2. Flood of February 14, 1950

A flood almost equal to the channel capacity is represented by this flood of 7,980 cfs at station 1965. Fig. 14 illustrates the hydrographs for the routing of this flood from stations 1955 and 1960 to station 1965, where a predicted peak of 8,021 cfs was calculated. This yields a recovery of 100.4 per cent.

This flood was then routed from stations 1965 and 1970, where respective inflows of 7,980 cfs and 4,788 cfs were recorded, to station 1980, where a flow of 14,500 cfs was registered. The routed peak was calculated to be 14,587 cfs, yielding a recovery of 100.6 per cent. The hydrographs for this reach are shown in Fig. 15.

3. Flood of February 16, 1949

This is the flood used as an example of the flood routing method in the previous chapter. A peak at station 1965 of 13,300 cfs due to inflows from station 1955 of 10,640 cfs and from station 1960 of 798 cfs was routed to be 13,135 cfs, or a recovery of 99.98 per cent.

The same flood was then routed from stations 1965 and 1970 to station 1980. Observed inflows of 13,300 cfs from station 1965, and 3,325 cfs from station 1970 produced an

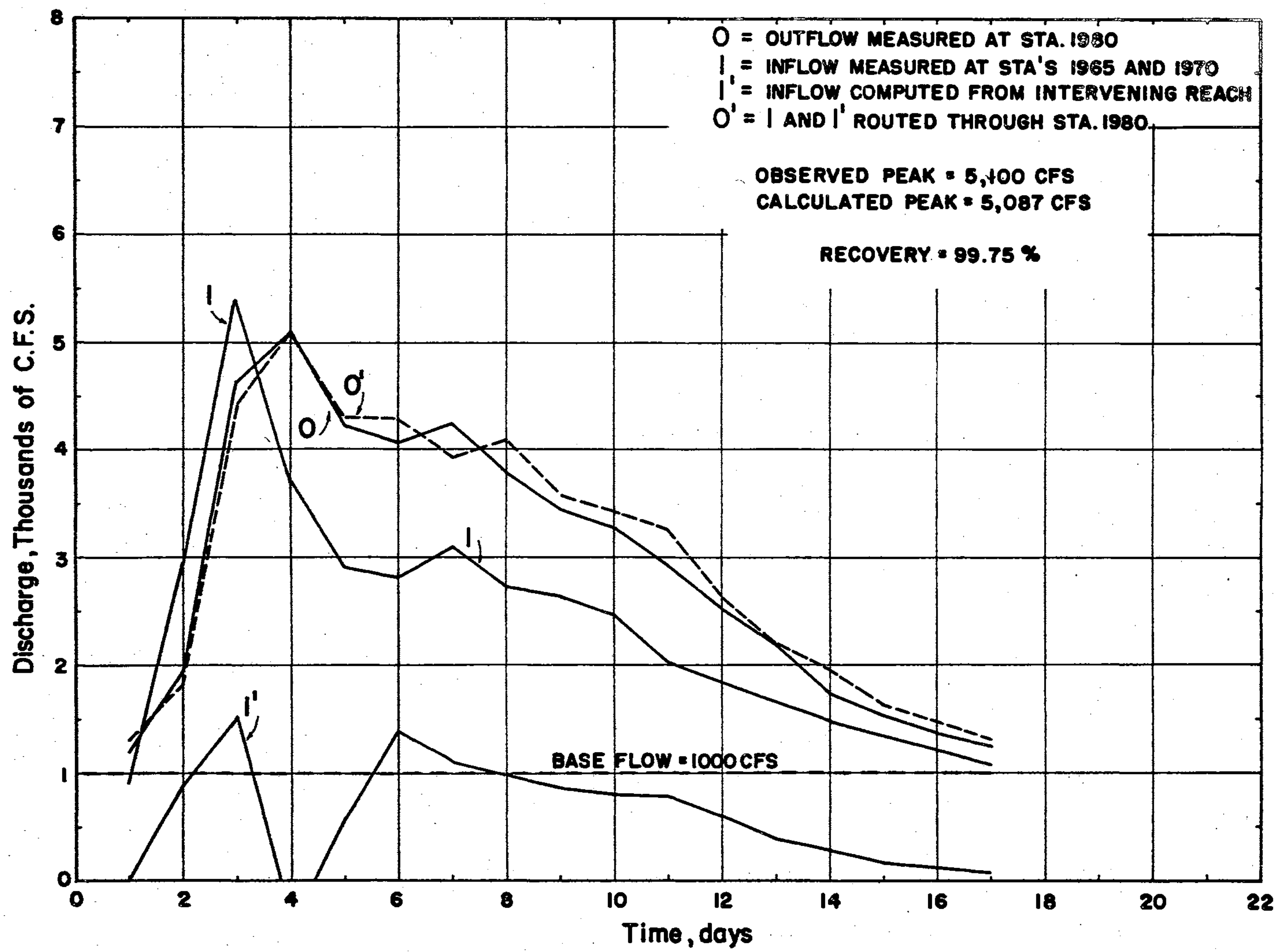

Fig. 13 - Flood of April 26, 1947, routed from Stations 1965 and 1970 to Station 1980.

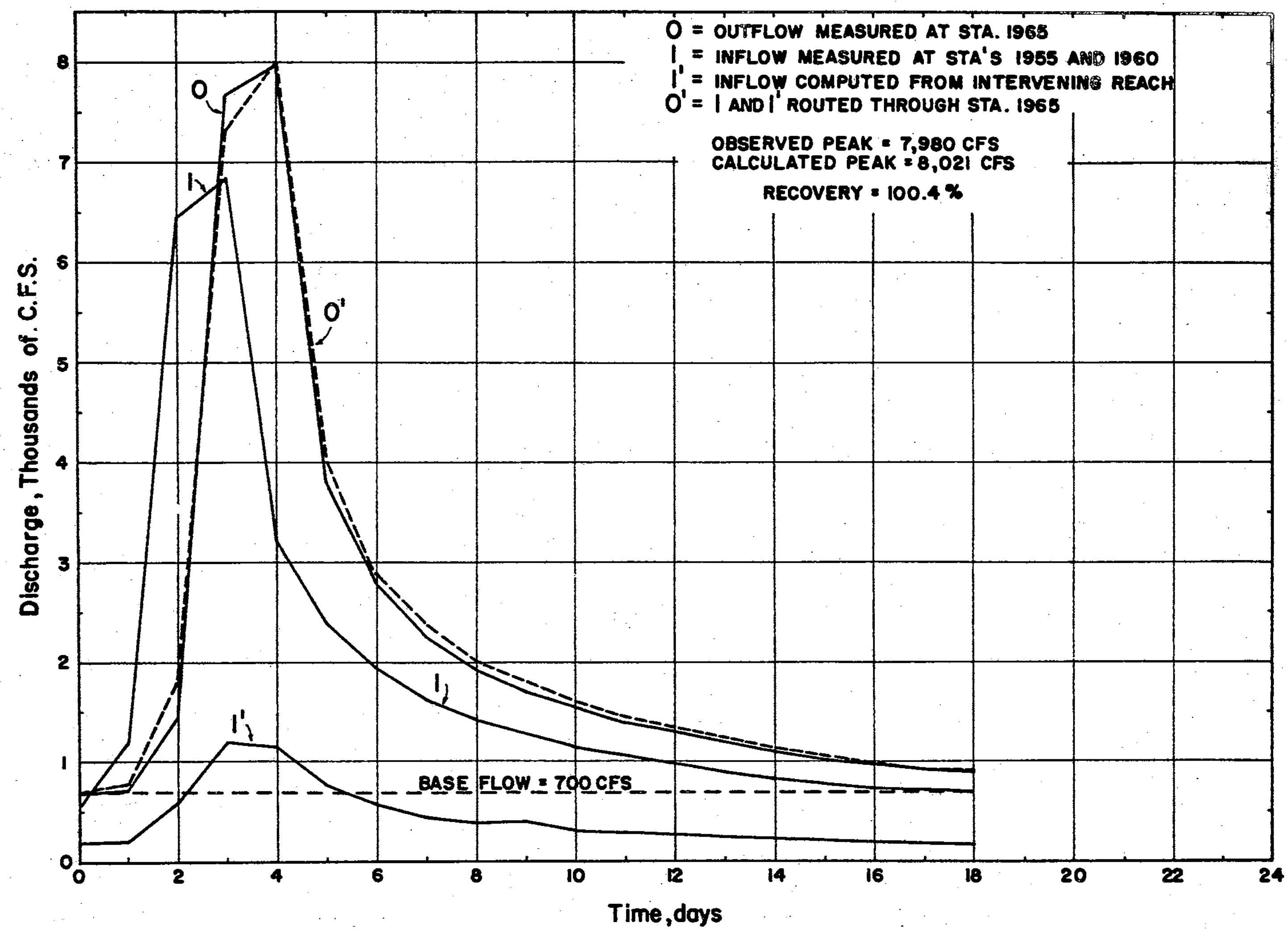

Fig. 14- Flood of February 14, 1950, routed from Stations 1955 and 1960 to Station 1965.

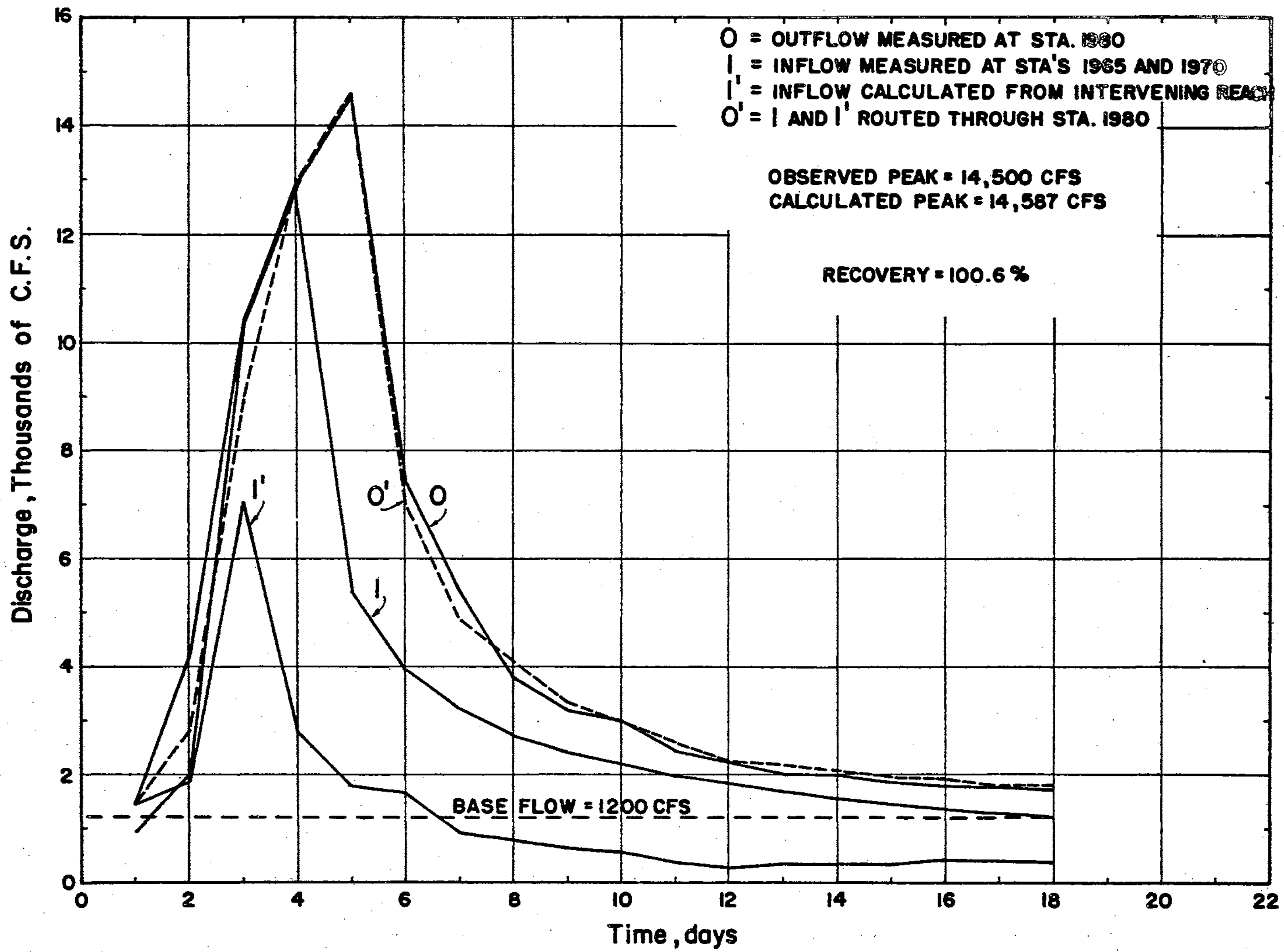

Fig. 15 - Flood of February 14, 1950, routed from Stations 1965 and 1970 to Station 1980.

outflow of 16,200 cfs at station 1980. The routed peak at station 1980 was found to be 16,258 cfs, yielding a recovery of 100.4 per cent.

The two steps employed in the routing of this flood are shown in Fig. 16 and Fig. 17.

4. Flood of November 1, 1941

This flood produced a peak of 25,000 cfs at station 1965, or approximately three times the channel capacity at that point. Using inflows recorded at station 1955 of 20,000 cfs and at station 1960 of 1,500 cfs, the calculated peak was found to be 23,300 cfs. This produced a recovery factor of 93.33 per cent.

This flood was then routed to station 1980, where an observed peak of 36,400 cfs was compared with a predicted peak of 36,795 cfs to yield a recovery of 100.8 per cent.

The hydrographs of the two steps involved in the routing of this flood are shown in Figures 18 and 19.

5. Flood of May 11, 1950

To show the applicability of this flood routing method to various flow magnitudes, the maximum flood of record at station 1965 was also routed. This flood, with a crest of more than eleven times the channel capacity at station 1965, recorded a peak of 90,400 cfs. Using the selected flood routing method, a peak of 90,065 cfs was calculated, yielding a 99.63 per cent recovery.

Therefore, the method devised for routing floods on the Illinois River basin produces accurate results when

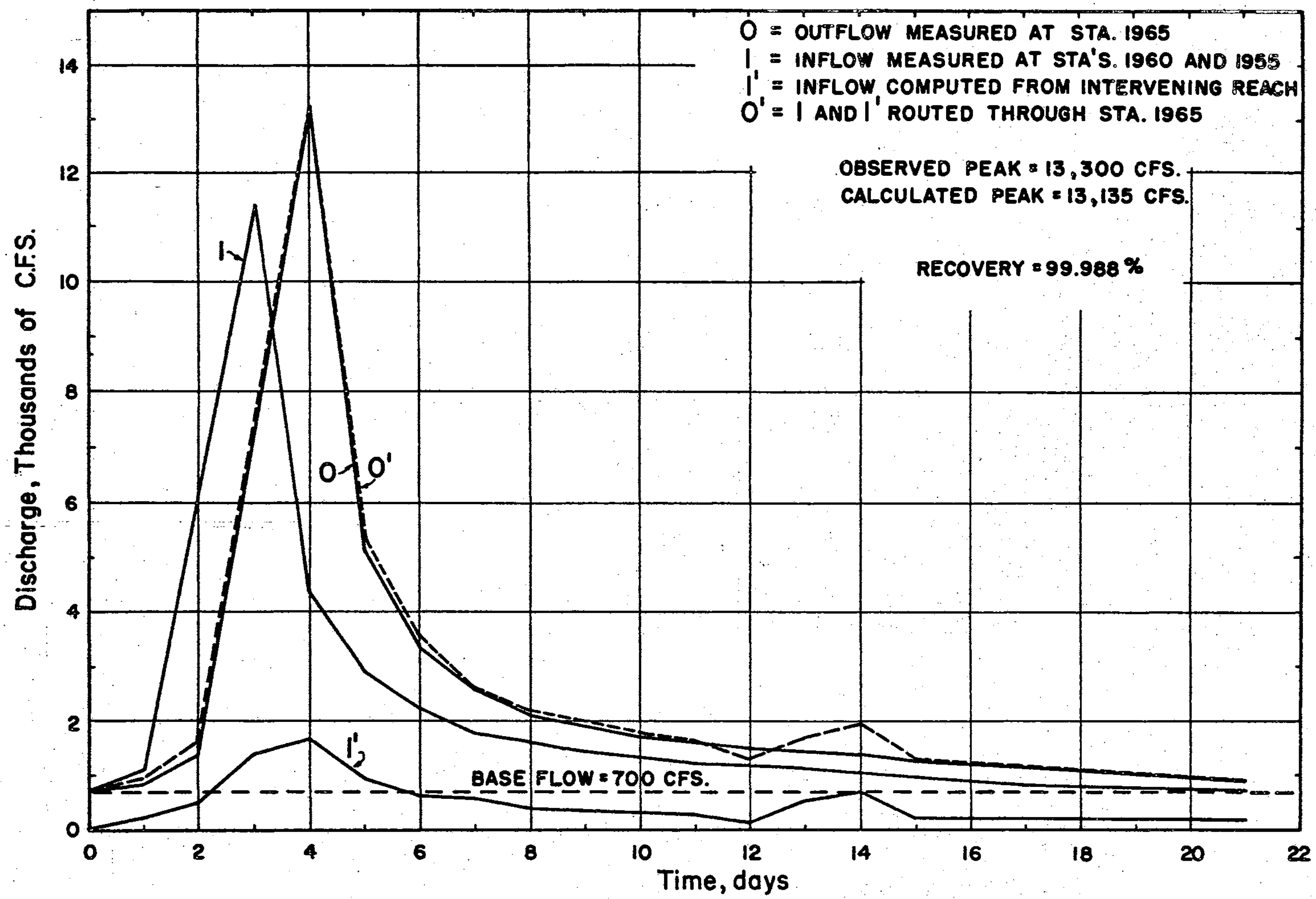

Fig. 16 - Flood of February 16, 1949, routed from Stations 1955 and 1960 to Station 1965.

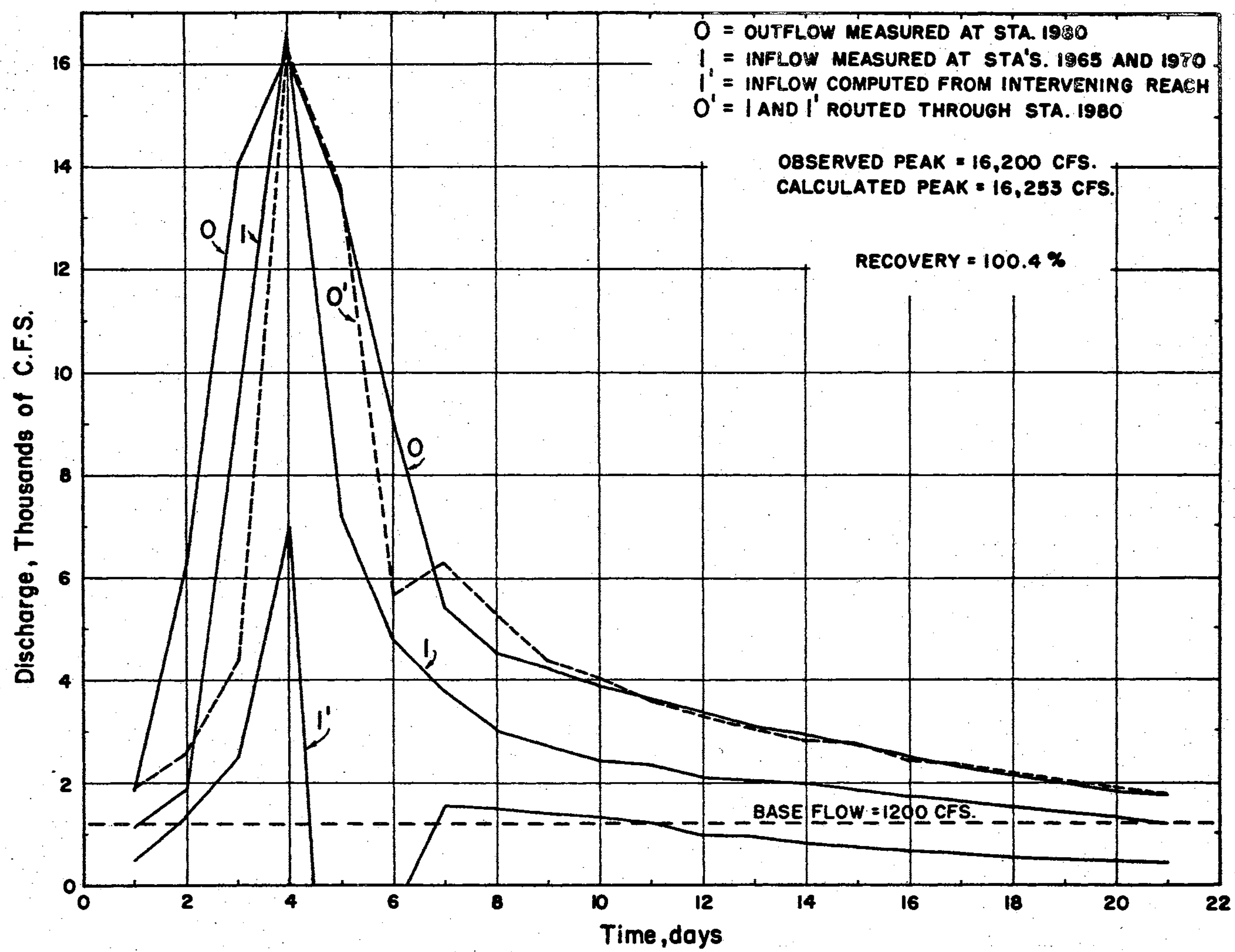

Fig. 17 - Flood of February 16, 1949, routed from Stations 1965 and 1970 to Station 1980.

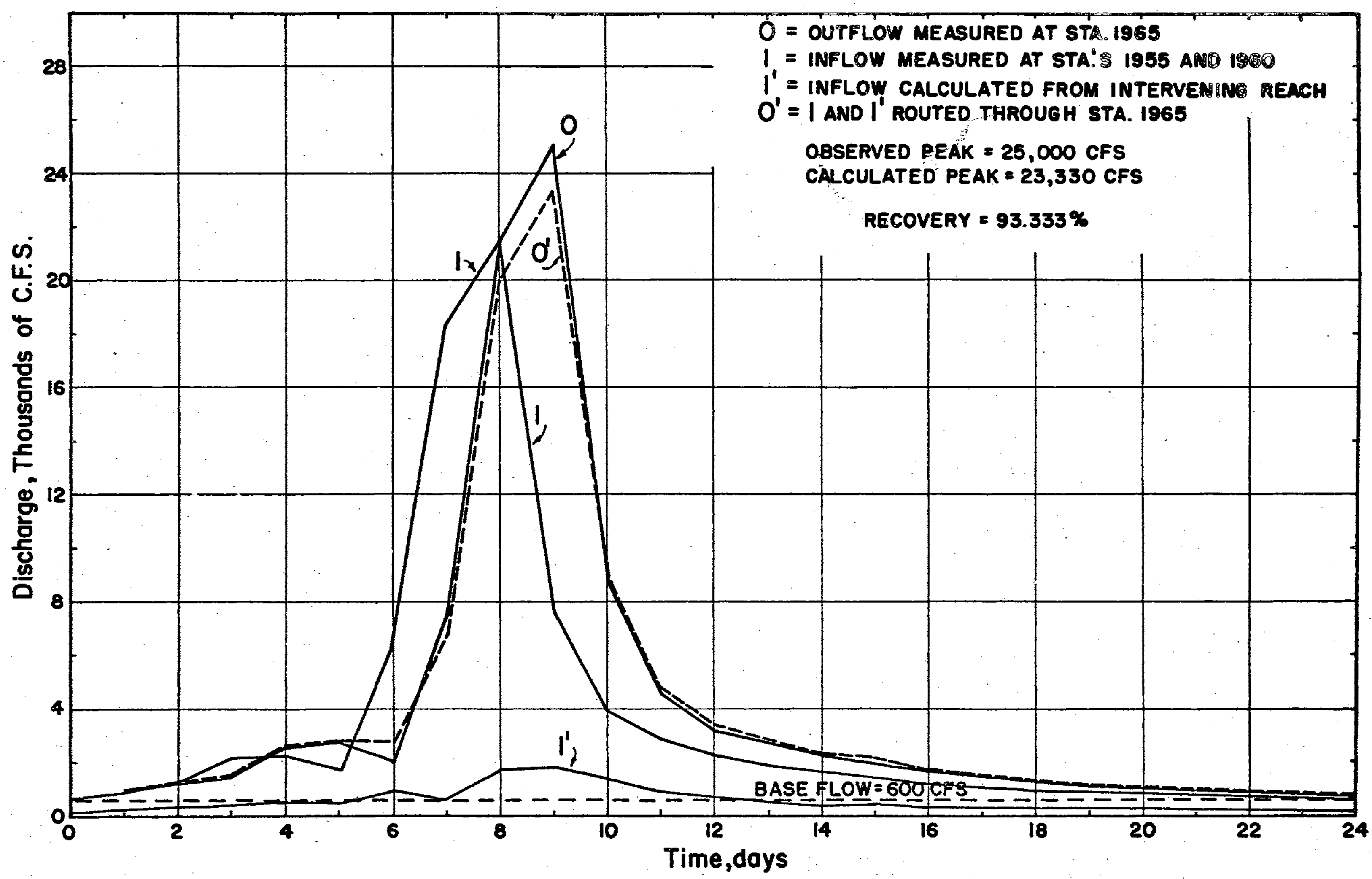

Fig. 18 - Flood of November 1, 1941, routed from Stations 1955 and 1960 to Station 1965.

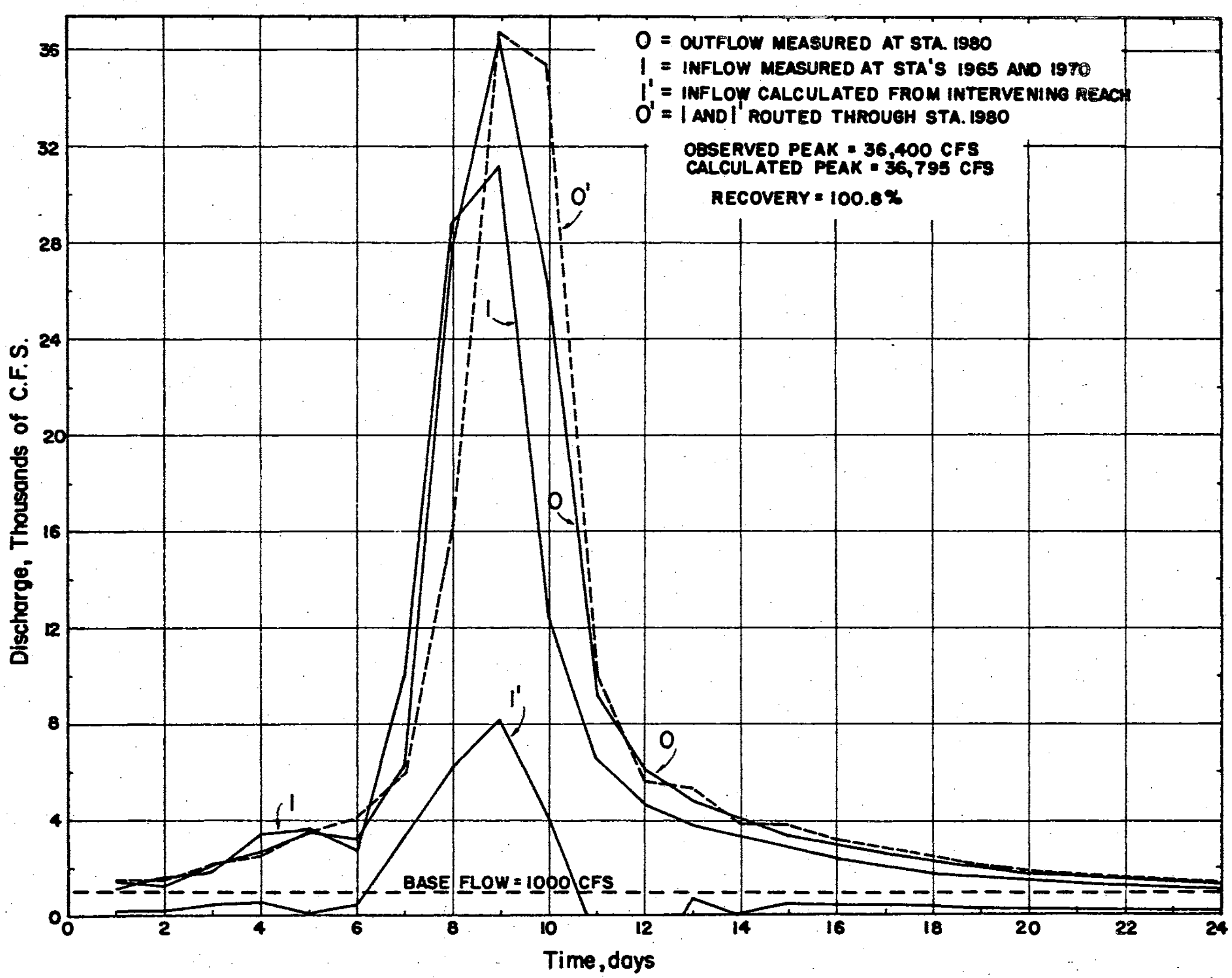

Fig. 19 - Flood of November 1, 1941, routed from Stations 1965 and 1970 to Station 1980.

applied to a wide range of flood magnitudes from high flows to the maximum flood of record.

When this flood was routed through station 1980, a recovery of 100.3 per cent of the observed 147,000 cfs peak by the calculated peak of 147,423 cfs confirms the applicability of this technique to a variety of flood magnitudes.

The hydrographs shown in Figures 20 and 21 depict the two-step routing procedure graphically.

C. Application of Local Inflow Hydrographs to Floods after 1952

Since the hydrographs of local inflow for the floods of different magnitudes have been determined, they may then be used to estimate the peaks that could have occurred at station 1980 had the reservoir not been built. By routing floods of similar magnitude as those described above, these respective local inflow hydrographs may be added to the observed channel inflow hydrographs in order to predict the outflow hydrograph at station 1980. Therefore, by comparing the routed peaks with the actual peaks registered at station 1980, the reservoir effect of these floods may be determined.

1. Flood of June 15, 1961

The peak of this flood at station 1965 was observed to be 3,820 cfs. The values of the local inflow hydrograph from the flood of April 26, 1947, can therefore be used to route this flood since the floods are of similar magnitude.

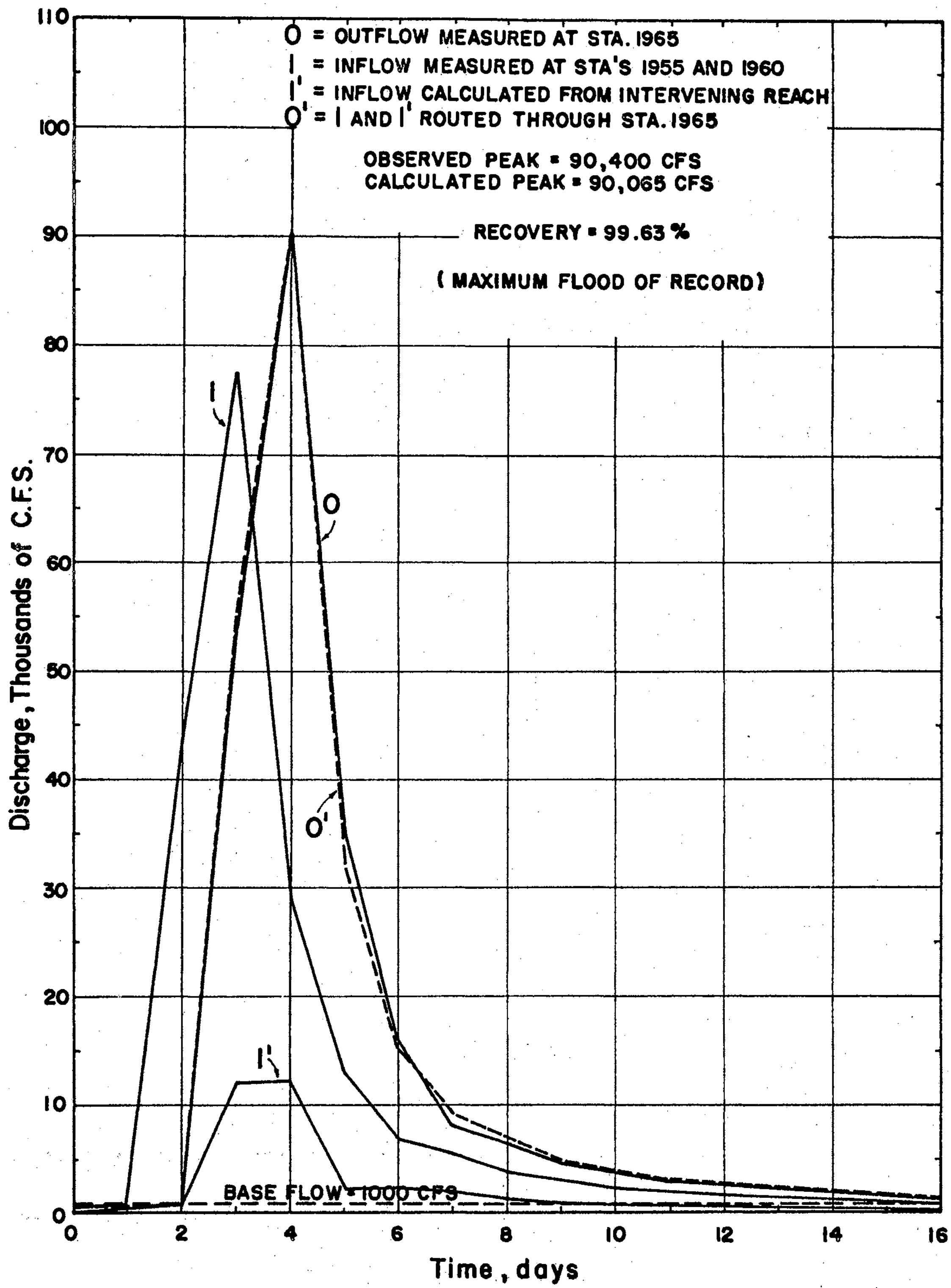

Fig. 20 - Flood of May 11, 1950, routed from Stations 1955 and 1960 to Station 1965.

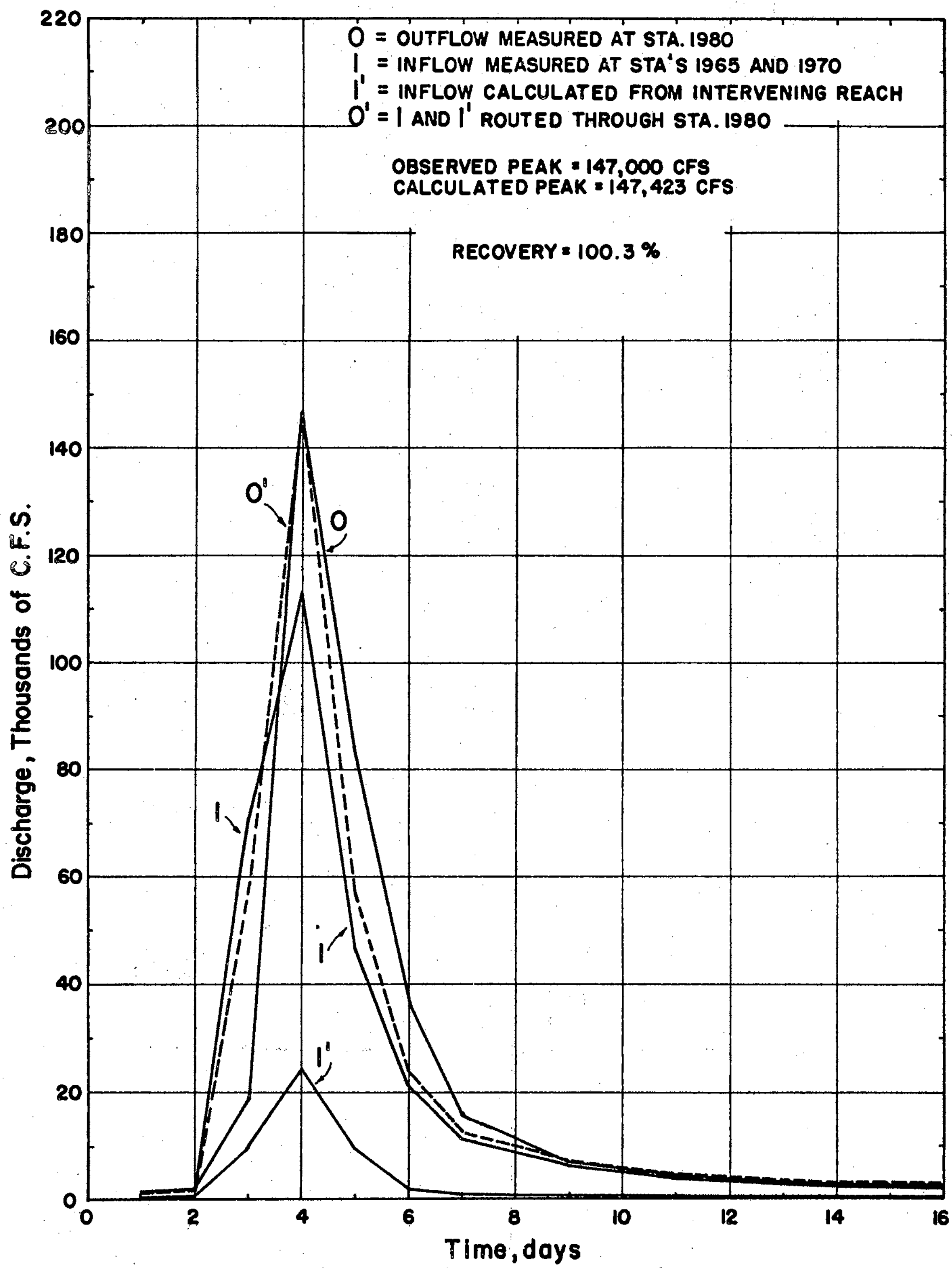

Fig. 21 - Flood of May 11, 1950, routed from Stations 1965 and 1970 to Station 1980.

Using the combined flows from stations 1965 and 1970 as the inflow, Table XI and Fig. 22 show how the local inflow is added to the channel inflow to predict the outflow hydrograph at station 1980. The peak of this flood was estimated as 3,699 cfs.

The actual flow at station 1980 on that day was recorded as 3,540 cfs. This means that because the reservoir was in operation at that time the flood was dampened by 159 cfs, or 4.30 per cent.

2. Flood of May 4, 1958

Since the peak at station 1965 of 7,980 cfs is equal to that of the flood of February 14, 1950, the local inflow hydrograph from the latter can be used in the routing of this flood.

Due to inflows at stations 1965 and 1970 of 7,980 cfs and 2,880 cfs on May 3, 1958, the routed outflow hydrograph was found to have a peak on the succeeding day of 12,679 cfs at station 1980. When this peak is compared with the actual flow at station 1980 of 12,400 cfs, it can be seen that the reservoir dampened the flood by 2.25 per cent.

Table XII and Fig. 23 show the data and the hydrographs used in routing this flood.

3. Flood of May 3, 1954

This flood, with an observed peak of 13,000 cfs at station 1965, is similar to the flood of February 16, 1949. Therefore, the values of local inflow from the 1949 flood can be used in the prediction of the outflow at station 1980.

TABLE XI

FLOOD OF JUNE 15, 1961

Day	I_{1965}	I_{1970}	I_{Total}	I'	O'	O
1	739	154	893	-	-	1,040
2	1,130	166	1,296	893	1,786	1,620
3	3,820	206	4,026	1,521	2,817	3,540
4	2,860	192	3,052	-327	3,699	3,540
5	1,980	166	2,146	575	3,627	3,540
6	1,560	154	1,714	1,391	3,537	2,550
7	1,290	141	1,431	1,101	2,815	2,950
8	1,060	133	1,393	982	2,413	1,280
9	960	128	1,088	856	2,249	1,500
10	862	122	984	805	1,893	1,300
11	765	115	880	792	1,776	79
12	687	110	797	606	1,486	58
13	637	100	737	383	1,180	1,190
14	582	98	680	300	1,037	1,260
15	546	96	642	161	841	742
16	535	98	633	135	777	217
17	558	100	658	82	715	332

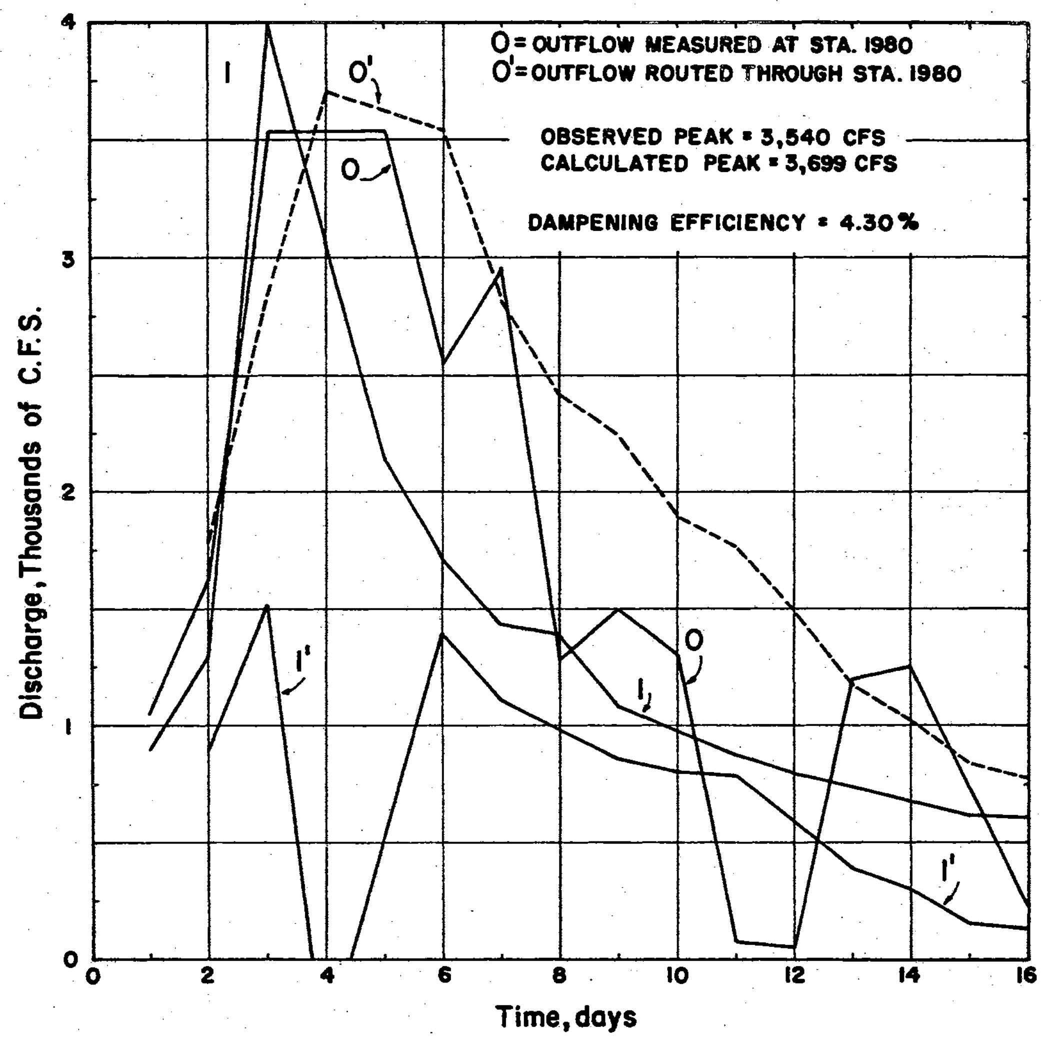

Fig. 22 - Flood of June 15, 1961, showing Difference between Actual and Calculated Outflow at Station 1980.

TABLE XII

FLOOD OF MAY 4, 1958

Day	I 1965	I 1970	I Total	I'	O'	O
1	1,360	455	1,815	1,500	1,500	2,130
2	1,320	703	2,023	1,913	3,728	2,890
3	2,680	3,480	6,160	7,048	9,071	4,320
4	7,980	2,880	10,860	2,814	8,974	10,200
5	5,580	2,490	8,070	1,819	12,679	12,400
6	5,690	1,560	7,250	1,657	9,727	11,400
7	3,590	1,160	4,750	942	8,192	10,100
8	2,800	894	3,694	809	5,559	6,510
9	2,400	1,330	3,730	661	4,355	5,820
10	2,790	1,260	4,050	589	4,319	6,840
11	2,650	863	3,513	397	4,447	4,960
12	2,100	667	2,767	281	3,794	2,340
13	1,770	540	2,310	345	3,112	2,760
14	1,540	460	2,000	392	2,702	2,730
15	1,360	408	1,768	385	2,385	2,290
16	1,230	364	1,594	445	2,213	1,140
17	1,100	332	1,432	420	2,014	180
18	1,010	302	1,312	400	1,832	100

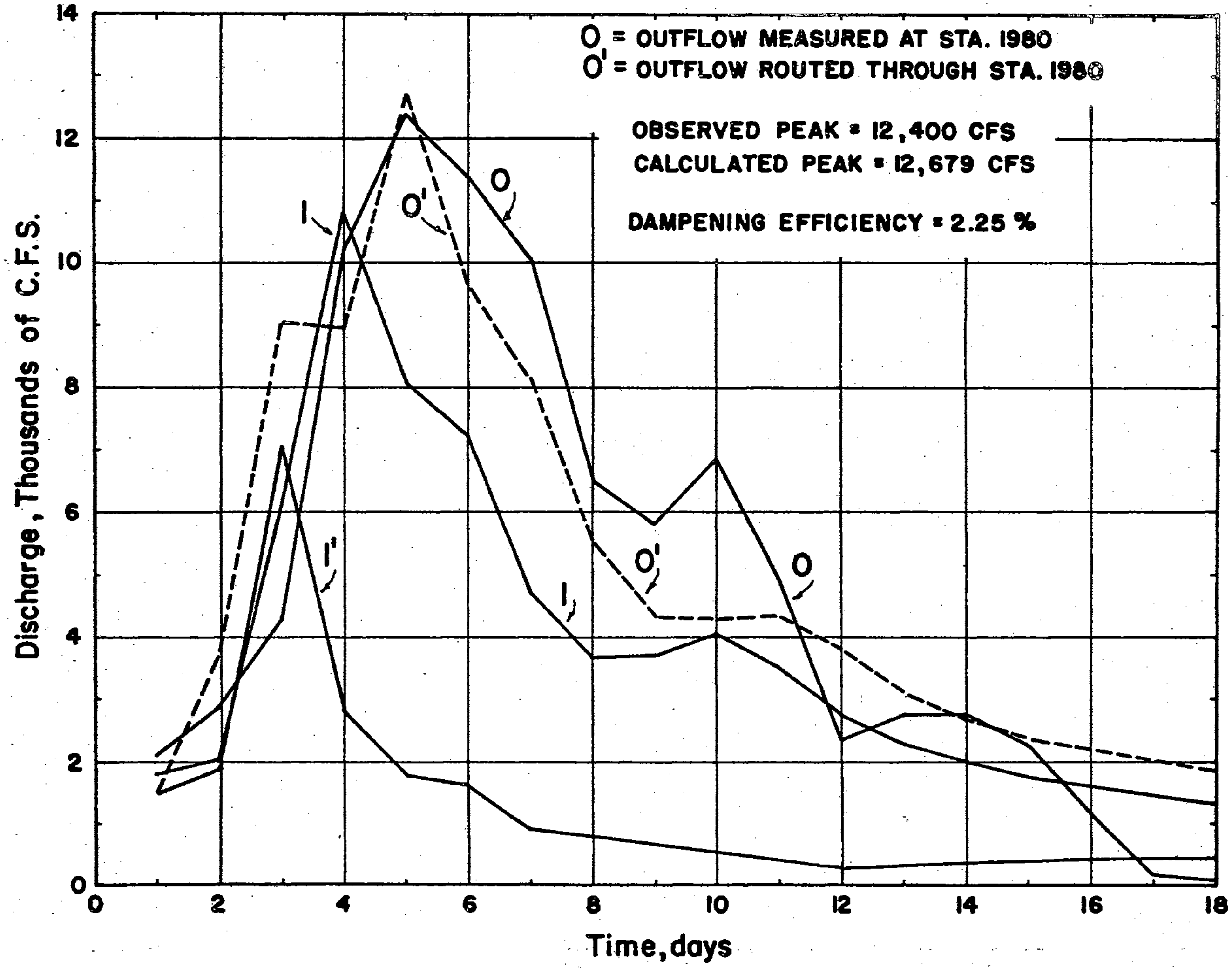

Fig. 77 - Flood of May 4, 1958, showing Difference between Actual and Calculated Outflow at Station 1980.

With inflows of 13,000 cfs and 3,250 cfs from stations 1965 and 1970, respectively, the routed peak was calculated to be 22,158 cfs at station 1980. Table XIII and Fig. 24 illustrate the calculations and hydrographs that were applied to this flood.

A dampening efficiency of 99.1 per cent was determined from comparing the routed 22,158 cfs peak with the flow of 270 cfs on that day.

4. Flood of July 26, 1960

The peak of this flood at station 1965 of 23,200 cfs compares favorably with that of 25,000 cfs at station 1965 during the flood of November 1, 1941. Since these floods are of similar magnitude, the local inflow calculations from the 1941 flood can be used in routing this flood.

During this flood, stations 1965 and 1970 were observed to contribute inflows of 23,200 cfs and 1,930 cfs, respectively, on the 25th of July. When routed, these inflows produced an outflow at station 1980 of 33,330 cfs on the 26th of July.

Since the actual outflow at station 1980 on the 26th of July was only 9,240 cfs, it can be said that the reservoir dampened the flood by 72.3 per cent. The data and hydrographs for this flood are shown in Table XIV and Fig. 25.

D. Determination of Reservoir Efficiency for Flood Dampening

In the previous section of this chapter, floods of four

TABLE XIII

FLOOD OF MAY 3, 1954

Day	I 1965	I 1970	I Total	I'	O'	O
1	179	1,190	1,369	500		904
2	1,810	1,740	3,550	1,380	2,749	305
3	5,440	9,730	15,170	2,521	6,071	1,770
4	13,000	3,230	16,230	6,988	22,158	270
5	5,470	1,600	7,070	-2,998	13,232	731
6	2,600	984	3,584	-1,597	5,473	710
7	1,870	686	2,556	1,515	5,099	776
8	1,520	766	2,286	1,500	4,056	785
9	1,240	505	1,745	1,377	3,663	515
10	1,050	382	1,432	1,300	3,045	132
11	859	320	1,179	1,172	2,604	740
12	746	276	1,026	973	2,152	751
13	650	245	895	929	1,954	248
14	554	223	777	792	1,687	264
15	481	195	676	729	1,506	380
16	432	175	607	654	1,330	236
17	404	155	559	627	1,234	67
18	376	153	529	534	1,093	380

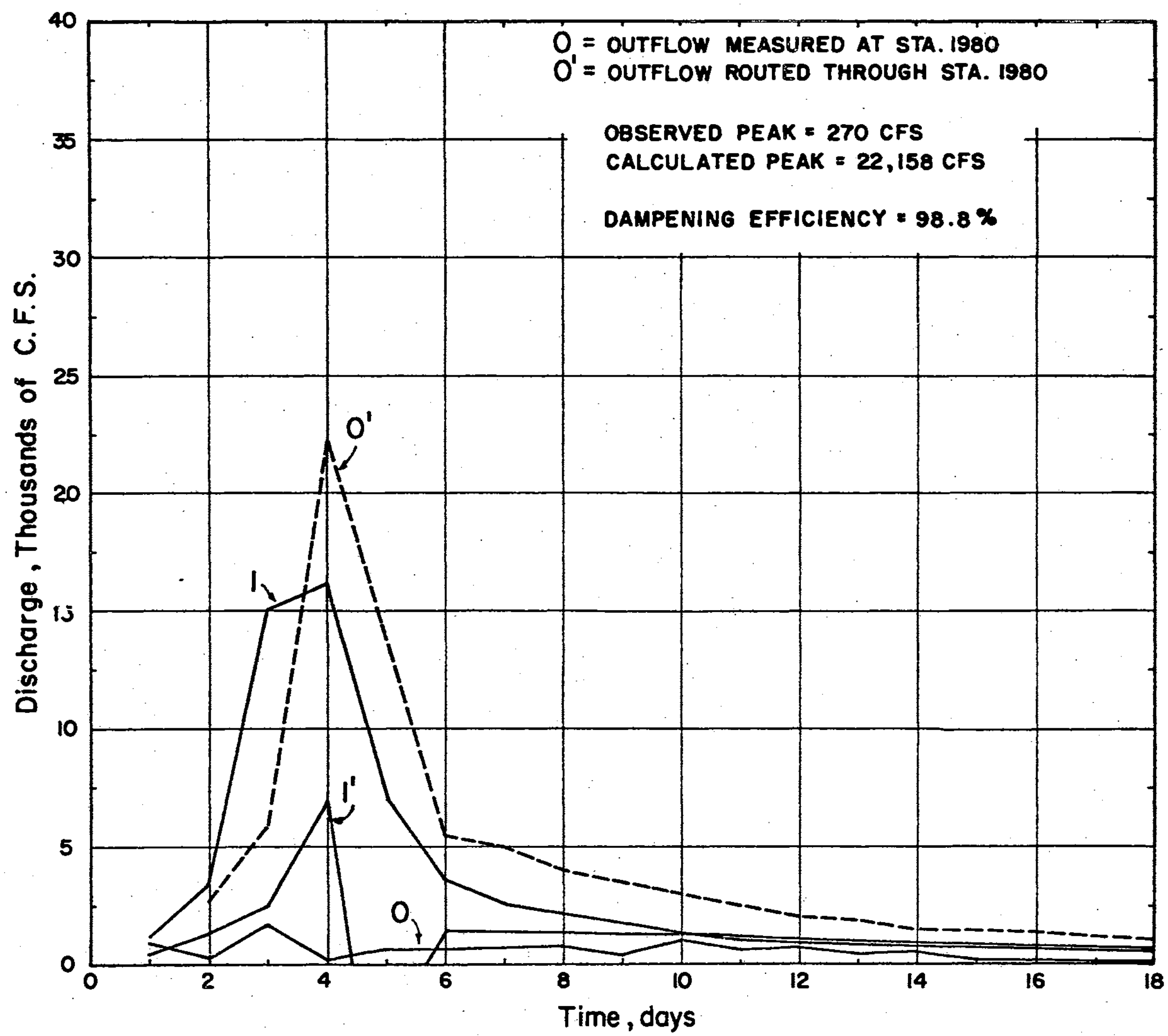

Fig. 24 - Flood of May 3, 1954, showing Difference between Actual and Calculated Outflow at Station 1980.

TABLE XIV

FLOOD OF JULY 26, 1960

Day	I 1965	I 1970	I Total	I'	O'	O
1	195	65	260	250	-	1,460
2	199	65	264	484	744	1,580
3	221	81	320	566	830	778
4	382	881	1,263	100	402	877
5	1,062	1,770	2,832	425	1,688	676
6	3,830	5,470	9,300	3,415	6,247	3,230
7	23,200	1,930	25,130	5,300	15,600	4,510
8	12,400	951	13,351	8,200	33,330	9,240
9	2,810	658	3,468	4,131	17,482	11,600
10	1,950	479	2,429	-1,372	2,096	8,910
11	1,490	380	1,870	- 924	1,505	4,950
12	1,210	310	1,520	725	2,595	3,600
13	995	270	1,265	102	1,522	2,870
14	862	235	1,097	566	1,831	2,820
15	732	212	944	421	1,518	2,570
16	650	184	834	521	1,465	1,470
17	582	166	748	484	1,318	1,260

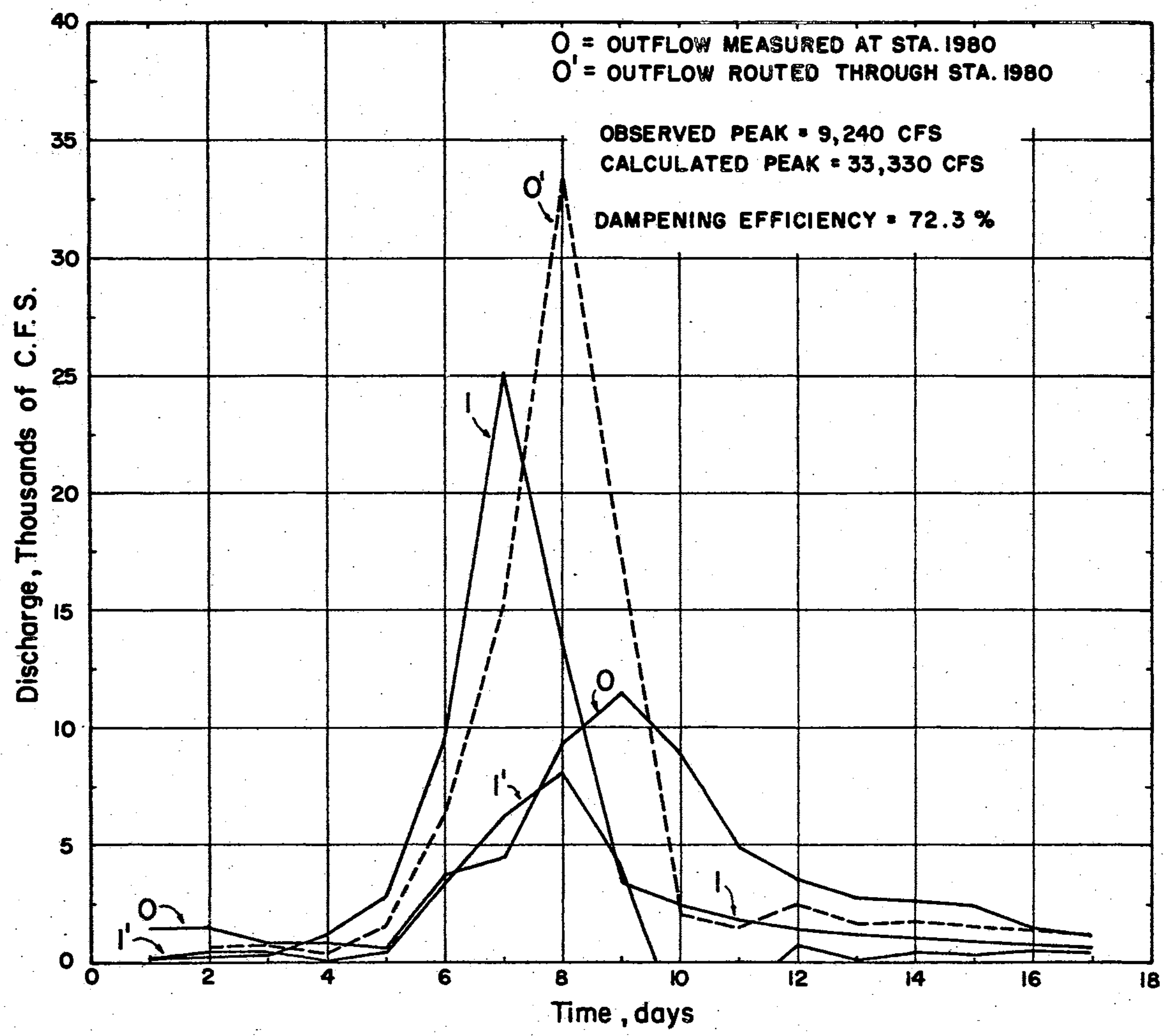

Fig. 25 - Flood of July 26, 1960, showing Difference between Actual and Calculated Outflow at Station 1980.

different magnitudes were routed to station 1980. These floods represented a wide range of peaks ranging from a high flow equal to about one-half the channel capacity at station 1965 to a flood approximately three times the channel capacity at that station. When these floods were routed to station 1980, the routed peaks were compared with the flows registered at 1980 to give an indication of the efficiency of the reservoir in controlling the floods.

Table XV is presented to show the floods, the routed and observed peaks, and the dampening efficiency of the reservoir for these floods. The efficiency was determined by comparing the routed and actual flows at station 1980. It may be pointed out that the actual flows shown in Table XV are those recorded at station 1980 on the suceeding day. Hence, these flows are compared with flows that would have occurred on that day if the reservoir had not been in operation.

The actual reservoir releases, measured at station 1980, are in accordance with the downstream requirements and do not, at any time, exceed the 16,000 cfs channel capacity at that station.

E. Comparative Peaks

From the analysis of several floods on the Illinois River basin, a relationship was observed between peaks at station 1955 and 1960 and station 1965. In other words, for peaks recorded at the upstream stations, the flood peak can be predicted at station 1965. This is shown in

TABLE XV

DAMPENING EFFECT OF TENKILLER FERRY RESERVOIR ON SELECTED FLOODS

Flood	Routed Peak at Station 1980	Actual Flow at Station 1980	Dampening Efficiency
June 15, 1961	3,699 cfs	3,540 cfs	4.30%
May 4, 1958	12,679 cfs	12,400 cfs	2.25%
May 3, 1954	22,158 cfs	270 cfs	99.1%
July 26, 1960	33,330 cfs	9,240 cfs	72.3%

Fig. 26. As an example, if the total inflow from stations 1955 and 1960 is known to be 30,000 cfs on a given day, the peak of this flood at station 1965 on the following day can be estimated at 35,000 cfs. This relationship was extended to include the maximum flood of record at station 1965 and, although the point on Fig. 26 is not shown, the peak was accurately predicted at station 1965.

A similar, although not as precise, relationship was found to exist in the reach between stations 1965 and 1970 and station 1980. Using the 35,000 cfs peak at station 1965 again as an example and assuming a flow from station 1970 of 8,400 cfs, the combined inflow of 43,400 cfs can be estimated to produce a peak of 47,300 cfs at station 1980, as shown in Fig. 27.

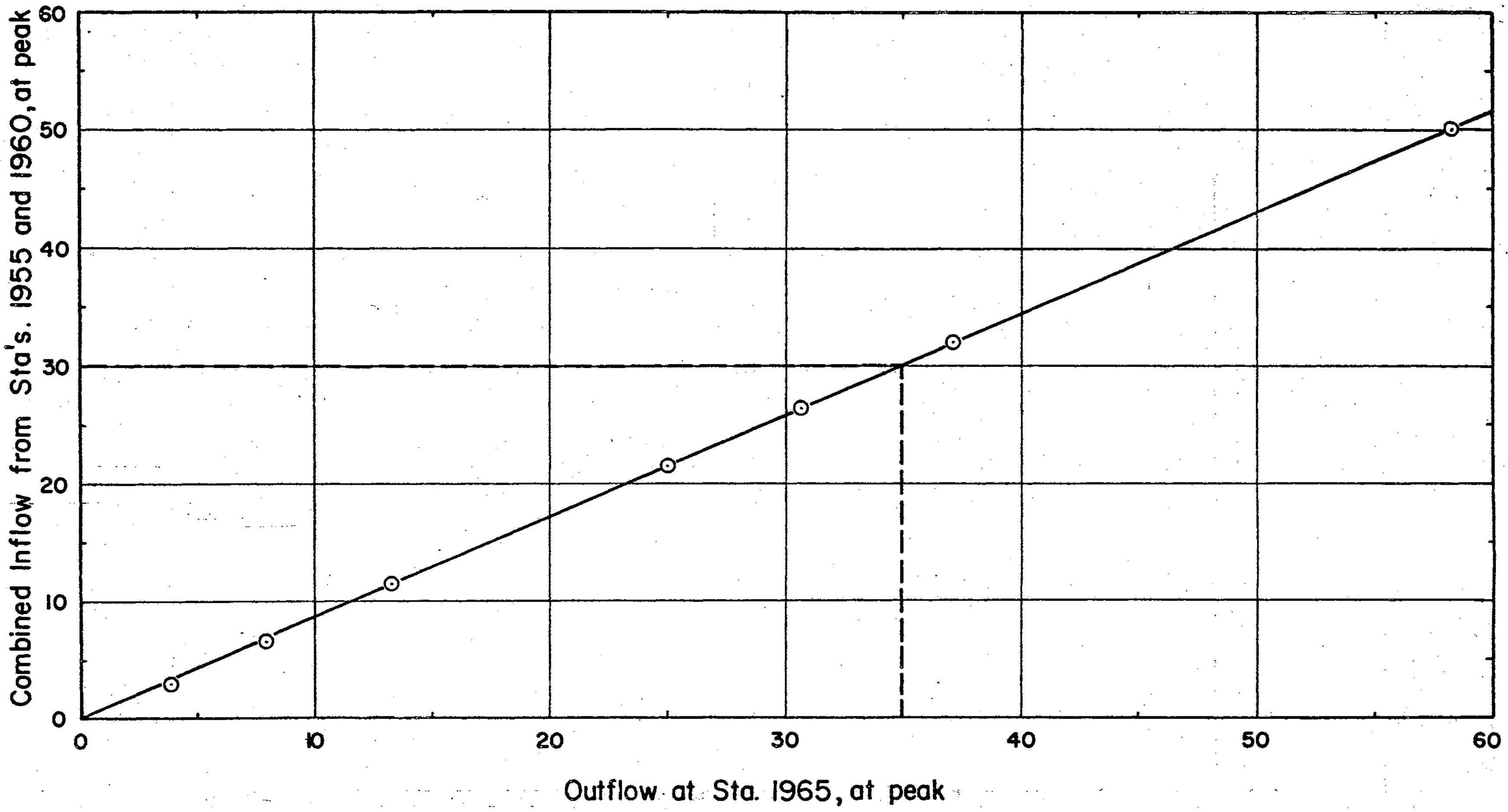

Fig. 26 - Curve of Relation between Inflow Peaks from Stations 1955 and 1960 and Outflow Peaks at Station 1965.

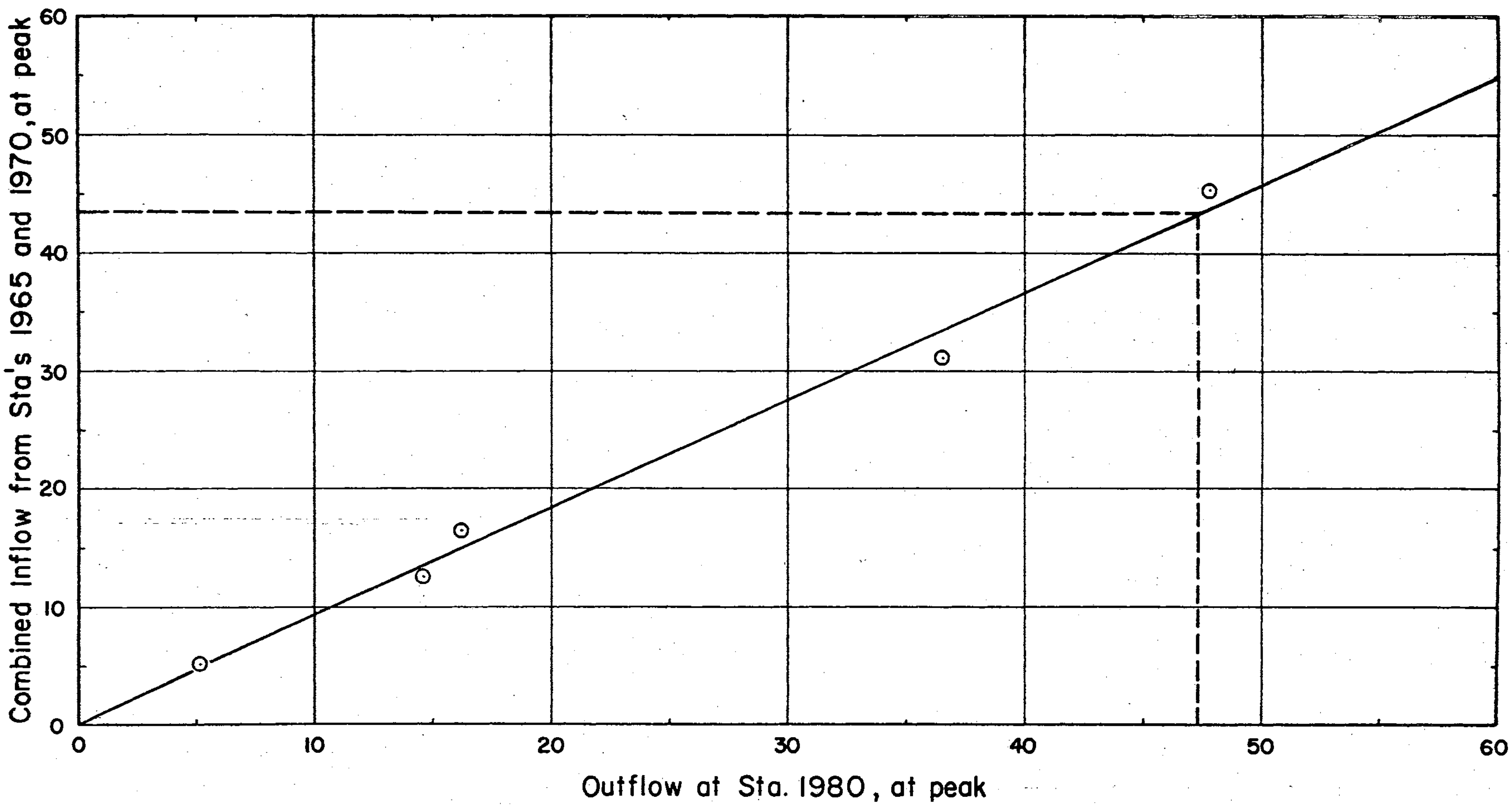

Fig. 27 - Curve of Relation between Inflow Peaks from Stations 1965 and 1970 and Outflow Peaks at Station 1980.

Therefore, using these figures, the peak at station 1980 can be predicted if the combined inflow from stations 1955 and 1960 are known. However, for evaluating the flow characteristics in a river as a function of time, a complete hydrograph is needed. For this reason, although the peaks of floods are of primary concern in flood control, the rising and falling limbs of a hydrograph are also required for successful reservoir operation.

CHAPTER V

DISCUSSION

A. Difficulties Encountered in Flood Routing

Four difficulties encountered in flood routing will now be discussed. These problems may be broken down into the determination of (1) storage; (2) inflow from the intervening reach; (3) storage-discharge relationships; and (4) variable stage-discharge relationships. In light of the method of flood routing presented, it was found that these parameters must be accurately assessed in order to evaluate the hydraulic characteristics of a stream.

1. Determination of Storage

The storage capacity of the river at various stages must be known to provide the relationship between discharge and storage. Accurate topographic data of the type often used to determine reservoir capacity are frequently not available for long river reaches. Thus, the storage is usually determined by any one of two other methods. If the hydrographs of a flood are available at the upper and lower ends of the reach, the method introduced by Wisler and Brater (27) may be used. In this method, the fact that the lower portion of the recession side of the hydrograph represents outflow from storage is utilized. This concept has also been presented by Horton (8).

A second method requires, in addition to hydrographs at the upper and lower ends of the reach, a hydrograph of inflow from the intervening area; i.e., the drainage area contributing flow to the river between the upper and lower ends of the reach. With this information available, all quantities of the continuity equation, except the change in storage, become known, and increments of storage may be determined for all intervals during the flood period. If some assumption is made as to the value of storage at the beginning of the flood, the increments may be added cumulatively to determine actual values of storage.

2. Inflow from the Intervening Area

Usually in small reservoirs, the increase in drainage area between the upper and lower ends of the reservoir is so small that the inflow from this area may be neglected. However, in the case of Tenkiller Ferry Reservoir, where the surrounding drainage area comprises almost the entire reach between gaging stations 1965, 1970, and 1980, the inflow from this area cannot be omitted from the routing of floods through this reach.

For river reaches, this area may be of considerable magnitude, and the local inflow cannot be neglected. If hydrographs of a flood at the upper and lower ends of a reach are available, and if the relationship between discharge and storage is determined, the inflow from the intervening reach can be found by successive applications of the continuity equation.

If either the upper or lower hydrograph or the storage is unknown, the inflow from the intervening reach must be estimated from rainfall, utilizing an assumed distribution graph to synthesize the hydrograph. This procedure is likely to give uncertain results unless hydrographs from a very similar watershed are available as a guide in estimating the shape of the distribution graph.

3. <u>Storage-Discharge Relationship</u>

In very large reservoirs, the water surface elevation is nearly level at all times, so that a change in storage must be accompanied by a corresponding change in water-surface elevation at all points within the reservoir. Therefore, since the rate of outflow is directly related to the water-surface elevation, it may also be directly related to the storage.

In long river reaches, however, the storage begins to increase as soon as the flood wave arrives at the upper end of the reach. It continues to increase as the wave front reaches the lower station, which may be several hours later. During this period of increasing storage the outflow may have been constant. It follows, therefore, that outflow is not directly related to storage. When assuming that storage is related to the average water-surface elevation at the two ends of a reach and therefore to the average discharges at both ends, the storage may be plotted against

(I+O) with satisfactory results.

4. Variable Stage-Discharge Relationships

Unlike outflow from a reservoir, discharge at a river gaging station may vary with the slope of the hypothetical energy gradient as well as with the stage. At such stations the relationship between gage height and discharge are somewhat different for rising and falling stages. It follows that a relation between channel storage and discharge would also depend to some extent upon whether the discharge is increasing or decreasing. Several methods of correcting for such effects have been described. Each of these is developed for a particular situation and is not sufficiently general to warrant a detailed description here.

In the case of the Illinois River, it was believed that the Wisler and Brater (27) method, utilizing a curve of relation between inflow plus outflow versus storage, eliminates the need for a variable stage-discharge or a variable stage-storage relationship.

B. Evaluation of the Flood Routing Method Used

1. Assumptions and Required Information

A number of simplifying assumptions are usually made when a flood is to be routed. The first assumption is that the channel is divided into a number of reaches. Each reach is relatively short and has practically constant physical characteristics. The flood is then routed successively from reach to reach. In general, the shortest practical reach is the section between the two nearest gaging stations.

A second assumption is that the discharge data are given at equal time intervals or routing periods. Within this period the increase or decrease of inflow or outflow is assumed to vary linearly. Therefore, in the Illinois River basin, a routing period of one day was found to be most suitable since the records are gaged daily and no records are available for a shorter period of time.

Another basic assumption is that the inflow and outflow are both taken as a measure of the storage within the reach. This assumption is also true if a flood is being routed through a level-pool reservoir where the variation in storage between the falling and rising stages of the flood wave is not appreciable. In the case of a stream, the length of a reach must not be too long or these variations will be exaggerated. Theoretically, the length of a reach should not exceed the product of the routing period and the average velocity of the flow in the reach.

Finally, the fourth assumption is that the inflow in the reach, local accretions from ungaged tributary flows, ground water, rainfall, or any other form of precipitation, and local decrements due to evaporation or seepage are ignored if the amounts are small. If the amounts are large they are either added to or deducted from the inflow, as the case may be. For instance, in the case of the Illinois River basin, the local inflow was added to the channel inflow, because of its magnitude and inference on the channel outflow.

To meet these assumptions, a large amount of data was required to route the floods in the Illinois River basin. The daily mean flows registered at the gaging stations within this basin were used from the records of the U. S. Geological Survey. These records, published annually as the U.S.G.S. "Water Supply Papers," covered a twenty-eight year period of flows in this basin spanning the water years 1938 through 1965. Since some of these gaging stations did not cover this period, the missing data was synthesized based on correlating flows in the overlapping years of record between these stations. The synthesis technique has been presented in Chapter III.

2. <u>Data Computed</u>

To fulfill the requirements of the flood routing method employed, it was necessary to compute the inflow from the intervening reach for each flood routed. This procedure was also outlined in Chapter III.

In order to determine the local inflow, a storage-discharge relationship was established for each flood. From this relationship it was possible to compute the storage within the reach for any channel inflow and outflow values.

3. <u>Application of Method to Unknown Floods</u>

After a relationship between channel inflow and outflow versus storage and a local inflow hydrograph was determined for a variety of floods prior to 1952 (the year the Tenkiller reservoir was put in operation) they were applied to floods of similar magnitudes and durations after 1952. For example, the local inflow hydrograph for the flood of February 16, 1949, at station 1965 (peak = 13,300 cfs) for the reach between stations 1965, 1970, and 1980 was used in routing of the flood of May 3, 1954, through the same reach where the peak at station 1965 was observed to be 13,000 cfs.

4. Use of the Method in Determining Reservoir Flood Control Efficiency

Again using the flood mentioned above as an example, the predicted peak at station 1980 was computed to be 22,158 cfs. When this is compared with the observed flow of 270 cfs on that day at station 1980, it can be seen that the reservoir reduced the peak by 99.1 per cent.

The percent reduction due to the reservoir may be somewhat misleading, however, since the reservoir storage is often low during the summer months and, although a sizeable flood entered the reservoir, it may not have produced a sufficient rise in water-surface elevation to warrant large reservoir releases. In other words, the reservoir was so low that its releases were not effected by this flood. On the other hand, when the reservoir is full prior to a flood, it must release enough water so that the incoming flow does not affect the reservoir storage in such a way that the reservoir overflows. It is unfortunate that the majority of the floods observed after 1952 occurred during the summer months when the reservoir was low, because the true reservoir dampening efficiency was not found for conditions of high reservoir water-surface elevation.

However, it may be pointed out that if the reservoir had not been in operation at these times, the flows routed to station 1980 would be an estimate of the flows that would have occurred. Hence, by having the reservoir in operation, the increase in available storage in the reach was the significant factor in controlling floods on the Illinois River. Therefore, the dampening efficiencies computed do reflect the capacity of Tenkiller Ferry Reservoir for controlling flood peaks.

5. Errors introduced by this Method

Surprisingly few errors were introduced by this flood routing method in comparison with other methods described in the literature. One effect that this method does not include, nor is it included in other methods, is that of the backwater effect of tributary flows into the main channel. However, it was felt that, although the tributary inflow is controlled as the flood peak in the main channel passes the tributary, the tributary inflow is not limited provided the peak is of relatively short duration. Since none of the floods routed contained peaks of excessive duration, the backwater effect may be excluded from the method.

An inherent error in comparing routed and observed flows is that of the actual flow recorded at the gaging stations. Since most flows are determined on the basis of stage-discharge curves, periods of high flows and floods may often be measured solely by extending this curve to include the observed stage. Also, the fact that the stage will vary depending on whether the discharge is increasing or decreasing introduces an error which affects the inflow and therefore the routed outflow recording accuracy.

However, the effects due to backwater and poor gaging station measurements are not accounted for in any other method discussed in the literature. Hence, this method introduces no new errors other than that of applying a local inflow hydrograph from one flood to another. This procedure, however, tends to increase the routing accuracy if the two floods are of similar magnitude. It is felt that, as long as the two floods are similar, the local inflow hydrograph will be approximately equal in both cases.

6. Application of this Method to Other Rivers

This method was applied only to floods on the Illinois River, but it may be modified to route floods on any stream provided the basic assumptions and data requirements are met.

This method has been used here to predict flood peaks and to measure the effectiveness of Tenkiller Ferry Reservoir on flood peak dampening. There is no reason why this method cannot be used on other river basins for similar purposes. It may also be used to advantage in determining the operating schedule for a system of flood-control or navigation dams.

CHAPTER VI

CONCLUSIONS

Based upon the results reported in this dissertation, the following conclusions may be drawn:

1. The volume of storage within the Illinois River basin may be accurately computed based on a relationship between inflow plus outflow and storage above the base flow. It was shown that, given sufficient inflow and outflow data, the storage within any given reach could be determined for floods of any size.

2. The inflow from the intervening reach between the upper and lower gaging stations can be obtained by applying the inflow, outflow, and storage parameters in the continuity equation as follows:

$$I' = \frac{(2S_2+O_1+O_2) - (2S_1+I_1+I_2)}{2}$$

3. The known inflow hydrograph can be added to the calculated local inflow hydrograph according to the method proposed by Wisler and Brater (27). Hence, it was observed that the resulting outflow hydrograph is a good approximation of the actual outflow hydrograph from that reach.

4. The inflow hydrograph for a flood of record of known magnitude can be applied to the routing of an unknown flood of similar magnitude through the same reach.

5. The predicted outflow hydrograph can be used in comparison with an observed reservoir outflow hydrograph to reflect the efficiency of the reservoir for controlling flood peaks. As an illustration, Tenkiller Ferry Reservoir on the Illinois River was studied. Floods of various magnitudes registered at the gaging stations above the reservoir were routed through the reservoir and compared with flows recorded at the gaging station below the reservoir to show the capacity of the reservoir for dampening flood crests.

6. The flood routing technique applied to this basin contained relatively few errors. Effects due to backwater and poor gaging station records are obviated by this method and by most of the methods reported in the literature.

7. The application of the local inflow hydrograph from one flood to another within the same reach does not affect the accuracy of this method provided the two floods are of similar magnitude. For floods of different magnitudes, the local inflow hydrograph from the flood of record should not be applied to the unknown flood unless the local inflow hydrograph is corrected.

8. The flood routing technique devised was applied only to floods on the Illinois River but it could easily be modified to route floods on any stream provided the

basic assumptions and data requirements are met.

9. This method has been used herein to predict flood peaks and measure the effectiveness of the reservoir on flood peak dampening. It may also be used to determine the design and operation scheme for a reservoir or a series of flood-control or navigation dams.

CHAPTER VII

SUGGESTIONS FOR FUTURE WORK

Based on the results of this investigation, the following suggestions are made for possible future research in the area of flood routing:

1. A study on the applicability of this technique for estimating reservoir flood dampening efficiency to other river basins should be undertaken to determine the suitability of this approach to a variety of river basin morphologies.

2. A study of the effectiveness of this approach for a higher range of flood magnitudes is needed to evaluate the accuracy of this concept in predicting floods on large river basins.

3. A study of the use of this method for streams in which a series of flood-control reservoirs has been constructed is needed to gain insight into the design of effective flood-control systems.

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APPENDIX

APPENDIX

PROGRAM FOR SYNTHESIS OF MISSING DATA

```
C       SYNTHESIS MISSING DATA
        DIMENSION P(12),X(6),Y(12),NY(12)
        READ 500,(P(I),I=I,12)
10      READ 501,X(1),X(2),X(3),X(4),X(5),X(6)
        DO 20 J=1,6
20      Y(J) = X(J)*P(J)
        READ 501,X(1),X(2),X(3),X(4),X(5),X(6)
        DO 30 J = 7,12
30      Y(J) = X(J-6)*P(J)
        DO 40 J=1,12
40      NY(J) = Y(J)
        PUNCH 502,NY(1),NY(2),NY(3),NY(4),NY(5),NY(6)
        PUNCH 502,NY(7),NY(8),NY(9),NY(10),NY(11),NY(12)
        GO TO 10
500     FORMAT(12F5.0)
501     FORMAT(10X,6F10.0)
502     FORMAT(10X,6I10)
        END
```

```
      PROGRAM FOR PLOTTING MEAN MONTHLY FLOWS
C     RVRPLT
      DIMENSION Y(6)
1     READ 100,N,YMAX
      NCOUNT = 0
5     XMIN = 0.0
      XN = N
      XMAX = XN*0.6
      XL = XMAX
      XD = XMAX
      YMIN = 0.0
      YL = 10.
      YD = YMAX
      CALL PLOT(201,XMIN,XMAX,XL,XD,YMIN,YMAX,YL,YD)
      X = XMIN
      CALL PLOT(90,XMIN,YMIN)
      CALL PLOT(90,XMIN,YMAX)
      CALL PLOT(90,XMIN,YMIN)
      CALL PLOT(90,XMAX,YMIN)
      CALL PLOT(90,XMIN,YMIN)
10    NCOUNT = NCOUNT+1
      READ 101,Y(1),Y(2),Y(3),Y(4),Y(5),Y(6)
      DO 20 J=1,6
      CALL PLOT(90,X,Y(J))
20    X = X+0.1
      IF(N-NCOUNT)30,30,10
30    CALL PLOT(99)
      CALL PLOT(7)
      PRINT 102
      GO TO 1
100   FORMAT(I10,F10.0)
101   FORMAT(10X,6F10.0)
102   FORMAT(15HLOAD NEXT DATA./30HADJUST PLOTTER FOR
      NEXT GRAPH./18HPUSH READER START.)
      END
```

```
        PROGRAM FOR SYNTHESIS OF FLOOD FLOWS
        STA. 1955 AND 1960 FROM STA. 1965
C       FLOOD SIMULATION
 1      PUNCH 400
 5      READ 500,AMAX,AMIN,BMAX,BMIN,DAYS1,DAYS2
        L=1
        NN=DAYS1
        NNN=DAYS2
        A1=(AMAX-AMIN)/DAYS1
        B1=(BMAX-BMIN)/DAYS1
        A2=(AMAX-AMIN)/DAYS2
        B2=(BMAX-BMIN)/DAYS2
        X=0.
        Y=0.
        N=0
 10     READ 505,CFLOW
        AFLOW=CFLOW*(AMIN+X)
        BFLOW+CFLOW*(BMAX-Y)
        GO TO (80,20),L
 20     PUNCH 510,N,CLAST,AFLOW,BFLOW
 30     IF(N-NN)70,35,35
 35     N=N+1
        CLAST=CFLOW
        X=A2
        Y-B2
 40     READ 505,CFLOW
        AFLOW=CFLOW*(AMAX-X)
        BFLOW=CFLOW*(BMIN+Y)
        PUNCH 510,N,CLAST,AFLOW,BFLOW
        IF(N-NN-NNN)60,50,50
 50     N=N+1
        PUNCH 515,N,CFLOW
        PAUSE
        GO TO 1
 60     N=N+1
        X=X+A2
        Y=Y+B2
        CLAST=CFLOW
        GO TO 40
 70     N=N+1
        X=X+A1
        Y=Y+B1
        CLAST=CFLOW
        GO TO 10
 80     PUNCH 520,N,AFLOW,BFLOW
        L=2
        GO TO 30
 400    FORMAT(8H DAY NO.,6X,6HC-FLOW,9X,6HA-FLOW,9X,
        6HB-FLOW/ )
 500    FORMAT(6F10.0)
 505    FORMAT(F20.0)
 510    FORMAT(I5,3F15.0)
 515    FORMAT(I5,F15.0)
 520    FORMAT(I5,F30.0,F15.0)
```

```
        PROGRAM FOR SYNETHESIS OF FLOOD FLOWS
        STA. 1970 FROM STA. 1965
C       SYNTHESIS OF FLOOD FLOWS
        DIMENSION BFLOW(50),AFLOW(50)
    1   READ 100,NB,NA,PB,P1,P2,PA,PCON
        N=0
        PUNCH 200
        DO 10 I-1,NB
        READ 110,BFLOW(I)
        N=N+1
        BNFLOW=BFLOW(I)*PB
        PUNCH 120,N,BFLOW(I),BNFLOW
   10   CONTINUE
        READ 110,PFLOW
        N=N+1
        IF (PFLOW-PCON)20,20,30
   20   PKFLOW=PFLOW*P1
        PUNCH 120,N,PFLOW,PKFLOW
        GO TO 40
   30   PKFLOW=PFLOW*P2
        PUNCH 120,N,PFLOW,PKFLOW
   40   CONTINUE
        DO 50 J=1,NA
        READ 110,AFLOW(J)
        N=N+1
        ANFLOW=AFLOW(J)*PA
        PUNCH 120,N,AFLOW(J),ANFLOW
   50   CONTINUE
        GO TO 1
  100   FORMAT(2I4,4F10.6,F10.1)
  110   FORMAT(F20.0)
  120   FORMAT(I6,2F12.0)
  200   FORMAT(12X,5HCFLOW,7X,5HDFLOW)
        END
```

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