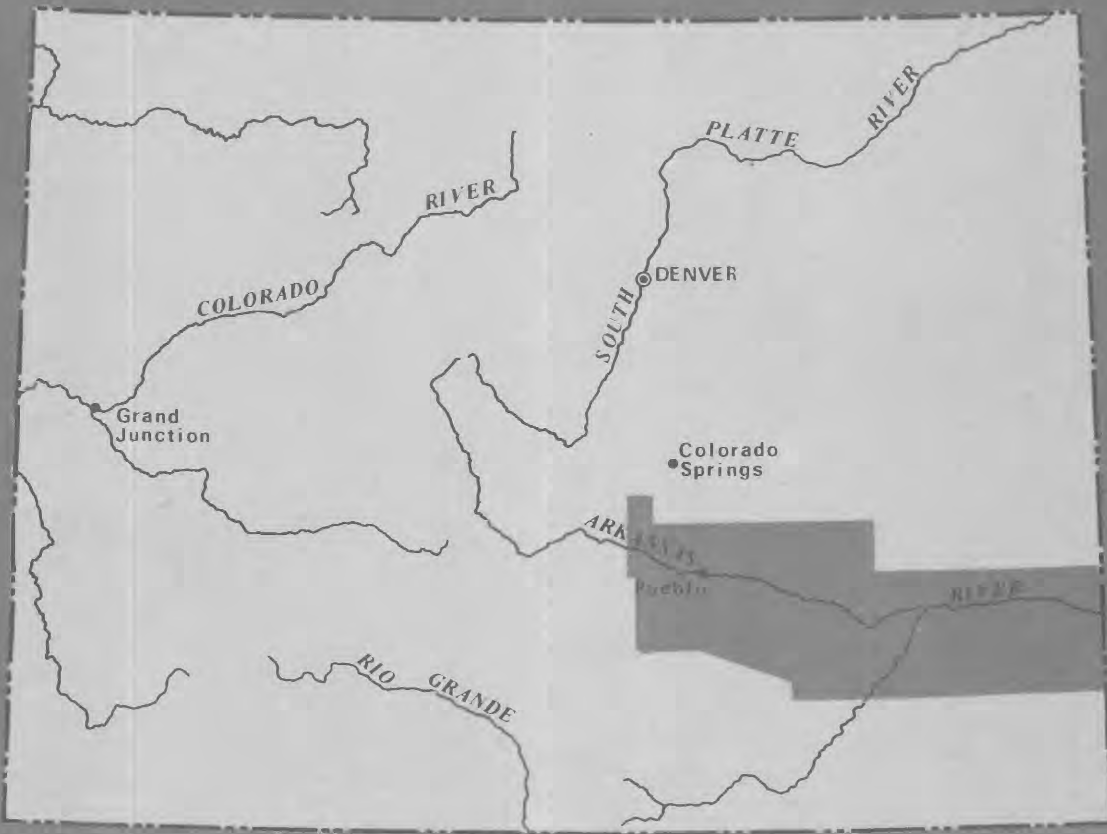
CHARACTERIZATION OF FLOODFLOWS ALONG THE ARKANSAS RIVER WITHOUT REGULATION BY PUEBLO RESERVOIR, PORTLAND TO JOHN MARTIN RESERVOIR, SOUTHEASTERN COLORADO

U. S. GEOLOGICAL SURVEY



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The need for a method for estimating flow characteristics of flood hydrographs between Portland, Colorado, and John Martin Reservoir has been prompted with the construction of the Pueblo Reservoir. To meet this need, the U.S. Geological Survey in cooperation with the Southeastern Colorado Water Conservancy District has developed a procedure for predicting floodflow peaks, traveltimes, and volumes at any point along the Arkansas River between Portland and John Martin Reservoir without considering the existing Pueblo Reservoir detention effects.

A streamflow-routing model was calibrated initially and then typical flood simulations were made for the 164.8-mile study reach. Simulations were completed for varying magnitudes of floods and antecedent streamflow conditions. Multiple-regression techniques were then used with simulation results as input to provide predictive relationships for flood peak, volume, and traveltime.

Management practices that may be used to benefit water users in the area include providing methods for the distribution and allotment of the flood waters upstream of Portland to different downstream water users according to Colorado water law and also under the Arkansas River Compact.

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SOUTHEASTERN COLORADO WATER CONSERVANCY DISTRICT

Lakewood, Colorado
1981

UNITED STATES DEPARTMENT OF THE INTERIOR

JAMES G. WATT, Secretary

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METRIC CONVERSION FACTORS

Inch-pound units used in this report may be converted to metric units by the following conversion factors:

Multiply inch-pound unit	*By*	*To obtain metric unit*
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
foot per mile (ft/mi)	0.1894	meter per kilometer (m/km)
square foot per day (ft^2/d)	0.09290	square meter per day (m^2/d)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
cubic foot per second per day [$(ft^3/s)/d$]	0.02832	cubic meter per second per day [$(m^3/s)/d$]
acre-foot (acre-ft)	0.001233	cubic hectometer (hm)
foot per second (ft/s)	0.3048	meter per second (m/s)
square foot per second (ft^2/s)	0.09290	square meter per second (m^2/s)
foot per foot (ft/ft)	0.3048	meter per meter (m/m)

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By John R. Little and Daniel P. Bauer

ABSTRACT

A method for predicting floodflow characteristics has been developed for the Arkansas River from Portland to John Martin Reservoir, Colo., that does not include the effects of regulation by Pueblo Reservoir. The legislation authorizing construction of the Pueblo Reservoir required that the river be managed as if the reservoir were not present. It was the intent of the authorization that ownership of flows of the Arkansas River would not be affected by the construction of the reservoir. Therefore it is necessary to determine the characteristics of the river in order that those charged with the responsibility of operating the river may authorize the delivery of water in accordance with applicable law in the light of conditions of the river as they existed prior to the reservoir. With the mechanisms described in this report, proper allocations can be determined; water may be delivered to those who, for example, would capture or benefit from floodflows.

The study reach was initially divided into seven subreaches between Portland and John Martin Reservoir, a distance of 164.8 river miles. A streamflow-routing model was then individually calibrated for the six most upstream subreaches, while values for the most downstream subreach were estimated on the basis of the nearest upstream subreach. Model simulations were made, using the calibration results. Simulations were computed based on various antecedent streamflow conditions and also different flood hydrographs for the starting location at Portland. Multiple-regression techniques were then used with the simulation results and subreach characteristics as input to provide predictive relationships for flood peak, flood volume, and flood-peak traveltime.

INTRODUCTION

Regulations contained in the Arkansas River Compact and the Prior Appropriation Doctrine (Radosevich and others, 1975) provide for the administration of streamflow during floods on the Arkansas River upstream from John Martin Dam. The Pueblo Reservoir was built with these legislative authorizations, which required the river be managed after the reservoir construction as if the reservoir were not present. It was the intent of the authorization that ownership of flows of the Arkansas River would not be affected by the construction of the reservoir.

Therefore it is necessary to determine the characteristics of the river in order that those charged with the responsibility of operating the river may authorize the delivery of water in accordance with applicable law in the light of conditions of the river as they existed prior to the reservoir. As a result, a technique is required to estimate the unregulated flood-hydrograph characteristics of such flows for proper allocation of the detained reservoir water.

Purpose and Scope

The purpose of this report is to develop a technique for estimating flood-hydrograph characteristics at locations along the Arkansas River between Portland and John Martin Reservoir without considering the detention effects of Pueblo Reservoir. The technique provides predictive equations for estimating flood peak, flood volume, and flood-peak traveltime.

The U.S. Army, Corps of Engineers (1977) has determined that potential flood damage begins when the Arkansas River reaches a discharge of 5,000 ft^3/s at the streamflow-gaging station near Avondale. They also have calculated that the "standard project flood" for the Arkansas River at Pueblo is 87,000 ft^3/s. Consequently, this study was limited primarily to flood discharges between 5,000 and 87,000 ft^3/s.

Acknowledgments

The authors gratefully acknowledge assistance of personnel from the Colorado Department of Natural Resources, Division of Water Resources, Office of the State Engineer, who provided some flow data used in the study.

Description of Study Reach

The study reach along the Arkansas River is approximately 165 river miles long between Portland and John Martin Reservoir (fig. 1). Throughout the study reach, the river traverses an alluvial aquifer composed of gravel, sand, silt, and clay. This aquifer is as much as 6 mi wide and 300 ft thick. The streambed slope ranges from about 12 ft/mi in the upper end of the study reach near Portland to about 6 ft/mi downstream from La Junta. Although the study reach is characterized by broad flood plains and mild streambed slopes, floodflows are typically less than 24 hours in duration with fairly rapid changes in discharge during both rising and receding portions of the hydrograph.

To facilitate modeling procedures, the study reach was divided into seven subreaches; the six most upstream subreaches are bounded by streamflow-gaging stations (fig. 1) and the most downstream subreach is bounded by the Las Animas streamflow-gaging station and John Martin Reservoir. The names and river-mile locations downstream from Portland of the seven streamflow-gaging stations and John Martin Reservoir are given in table 1.

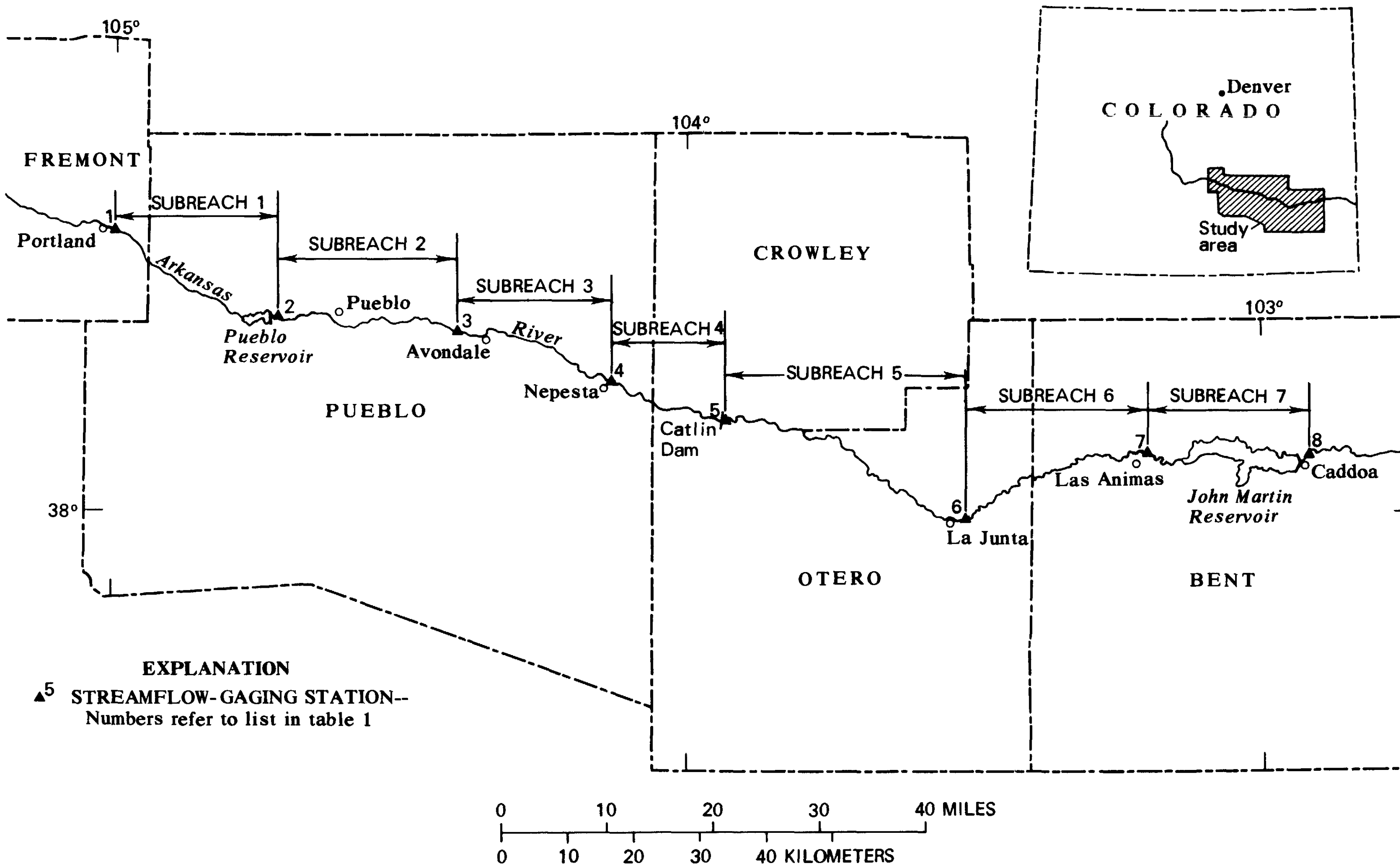

Figure 1.--Location of subreaches and selected streamflow-gaging stations.

Table 1.--*River-mile distances from the Portland streamflow gage downstream to selected sites along the study reach*

Site No.[1]	Gaging station name (U.S. Geological Survey station number)	River mile downstream from Portland[2]
1	Arkansas River at Portland (07097000)--------------	0.0
2	Arkansas River above Pueblo (07099400)-------------	22.9
3	Arkansas River near Avondale (07109500)------------	46.2
4	Arkansas River near Nepesta (07117000)-------------	64.8
5	Arkansas River at Catlin Dam (07119700)------------	86.0
6	Arkansas River at La Junta (07123000)--------------	117.0
7	Arkansas River at Las Animas (07124000)------------	144.0
8	John Martin Reservoir at Caddoa (07130000)---------	164.8

[1]Location shown in figure 1.

[2]River miles are for site locations as of January 1980. Determinations are from Portland for computational purposes and therefore have no relation to river-mile determinations of the U.S. Water and Power Resources Service or the U.S. Army, Corps of Engineers.

Approach

Only limited or observed flood information for the Arkansas River study reach prior to the closure of Pueblo Dam during 1974 is available for developing relationships for estimating the characteristics of unregulated floodflows. In order to overcome this deficiency, a streamflow-routing model (Land, 1977) was calibrated using the limited historical flood data. Then the model was used to simulate downstream flood hydrographs for a variety of flood conditions at Portland. Multiple-regression techniques were then used on these simulated data to develop relationships for predicting peak discharges, flood volumes, and traveltimes for downstream locations in the study reach.

STREAMFLOW-ROUTING MODEL

The streamflow-routing model used for this study mathematically simulates the response of the stream-aquifer system to the stress created by the movement of a flood wave through the study reach. As a flood wave moves downstream, water is lost both to channel and bank storage. After the flood peak passes, this water returns rapidly to the river from channel storage, but water from bank or aquifer storage returns much more slowly. The combined effect of this temporary storage is a reduction in peak discharge and an attenuation of the flood hydrograph over time.

Data Requirements

The streamflow-routing model requires input data as follows: (1) Flood hydrographs, (2) river-channel characteristics, (3) aquifer characteristics, and (4) streamflow characteristics. Tributary-inflow data and diversion data also can be input to the model but were not used in this study, as subsequently described.

Flood Hydrographs

Selection of flood hydrographs for model calibration involved screening all recorded floods in the study reach. Initially, all recorded annual and many secondary floods having peak discharges greater than 3,000 ft^3/s were considered. Flood hydrographs exhibiting significant effects of diversions or tributary inflow were initially eliminated. The downstream hydrograph shape was required to have a similar shape to the upstream hydrograph; otherwise the event also was eliminated. The scarcity of acceptable flood data for some subreaches dictated selection of flood hydrographs with peak discharges as small as 2,900 ft^3/s. Because the model was independently calibrated for each subreach, some flood hydrographs were selected for calibration in part of the subreaches but were eliminated from others. The floods selected for model calibration for subreaches 1 to 6 are listed in table 2. Because there is no downstream streamflow gage at this subreach-7 location and only the John Martin Reservoir storage-versus-time relationship data were available, no calibration was done for this subreach. Final results for this subreach were based, therefore, on the nearest upstream subreach-calibration results.

River-Channel Characteristics

The river-channel characteristics required for each subreach are length and average river-channel slope. Subreach lengths were obtained from Livingston (1973; 1978) while average river-channel slope was measured from U.S. Geological Survey topographic maps. The average river-channel slope was computed for this report on the basis of the subreach streambed-elevation change divided by the subreach length. These river-channel characteristics are listed in table 3.

Aquifer Characteristics

Aquifer characteristics required for model calibration are length, width, transmissivity, and storage coefficient. Values of aquifer length and width characteristics, listed by subreach in table 3, were obtained from Jenkins and Taylor (1972) and Livingston (1973; 1978). The corresponding transmissivity and storage-coefficient values shown in table 3 were obtained during the model calibration, as described on page 9.

Table 2.--*Floods used to calibrate the streamflow-routing model*

Subreach No.[1]	Abbreviated subreach name (Upstream and downstream U.S. Geological Survey station numbers)	Date of flood	Upstream peak discharge (cubic feet per second)
1	Portland to Pueblo (07097000 to 07099400)--------	July 8, 1947	13,500
		June 4, 1949	3,500
		June 5, 1949	21,100
		June 17, 1965	14,400
		June 18, 1965	7,400
2	Pueblo to Avondale (07099400 to 07109500)-------	June 6, 1949	12,800
		July 21, 1975	3,600
3	Avondale to Nepesta (07109500 to 07117000)-------	June 18, 1965	50,000
		Aug. 21, 1965	5,000
4	Nepesta to Catlin Dam (07117000 to 07119700)-------	June 18, 1965	43,100
		Aug. 19, 1965	15,500
		July 21, 1975	8,310
5	Catlin Dam to La Junta (07119700 to 07123000)-------	June 18, 1965	43,200
		July 21, 1975	6,050
		July 22, 1975	2,630
6	La Junta to Las Animas (07123000 to 07124000)-------	May 20, 1955	50,000
		June 17, 1965	29,000
		June 19, 1965	31,700
		Aug. 19, 1965	14,400
		Aug. 23, 1965	18,500
		May 26, 1967	15,100
		July 22, 1975	2,900
		Aug. 3, 1976	7,800

[1]Subreach locations shown in figure 1.

Table 3.--*Aquifer and channel characteristics used in the streamflow-routing model*

Subreach No.[1]	Abbreviated subreach name	Length of subreach (miles)	Subreach slope (feet per foot)	Length of aquifer (miles)	Average width of aquifer (feet)	Transmissivity of aquifer (square feet per day)	Storage coefficient of aquifer (dimensionless)	Losses to surface detention (acre-feet)
1	Portland to Pueblo------	22.9	0.0023	19	1,600	4,000	0.15	0
2	Pueblo to Avondale------	23.3	.0019	18	5,400	10,000	.10	0
3	Avondale to Nepesta-----	18.6	.0013	10	7,300	10,000	.10	0
4	Nepesta to Catlin Dam---	21.2	.0012	12	13,000	12,000	.10	600
5	Catlin Dam to La Junta--	31.0	.0012	19	19,000	11,000	.12	600
6	La Junta to Las Animas--	27.0	.0011	15	14,000	10,000	.15	3,400
7	Las Animas to John Martin Reservoir------	20.8	.0011	15	24,000	9,000	.15	3,400

[1]Subreach locations shown in figure 1.

Streamflow Characteristics

Streamflow characteristics required include the following discharge relations: stage, flood-wave speed, and flood-wave dispersion. The stage-discharge relations for each gaging station for the range of modeled streamflow are listed in table 4. The flood-wave speed versus discharge relations are listed in table 5 and flood-wave dispersion versus discharge relations are listed in table 6. The flood-wave speed and flood-wave dispersion values shown in tables 5 and 6 reflect final model-calibration values as described on page 9.

Table 4.--*Stage-discharge relationships used in the streamflow-routing model of the Arkansas River from Portland to John Martin Reservoir*

Site No.[1]	Upper number is stage, in feet; lower number is discharge, in cubic feet per second							
1	1 140	2 480	4 2,460	8 9,800	9 11,900	10 14,180	11 17,380	26 90,000
2	0 62	1 285	2 750	3 1,440	4 2,430	4.5 3,030	5 3,680	11 87,000
3	2.5 155	3 415	4 1,200	5 2,380	6 4,050	7 6,800	8 14,500	11 102,000
4	3.4 9	6 2,650	9 30,000	12 112,000	------ ------	------ ------	------ ------	------- -------
5	0 0	2 2,210	3 6,000	4 11,500	5 17,750	6 24,500	7 31,500	14 100,000
6	1.6 0	6 3,210	9 11,100	17 93,000	------ ------	------ ------	------ ------	------- -------
7	1.7 0	9 9,000	12 20,000	19 99,000	------ ------	------ ------	------ ------	------- -------
8	7 9	9 1,140	12 7,900	20 104,000	------ ------	------ ------	------ ------	------- -------

[1]Locations shown by site number in figure 1; site names given in table 1.

Table 5.--*Relations between flood-wave speed and flood-peak discharge as determined from the model calibration of the Arkansas River from Portland to John Martin Reservoir*

Subreach No.[1]	Upper number is flood-wave speed, in feet per second; lower number is flood-peak discharge, in cubic feet per second					
1	6.0 200	6.0 5,000	6.0 20,000	7.3 87,000	-------- --------	-------- --------
2	6.0 200	6.0 1,500	7.5 3,000	7.5 40,000	9.0 87,000	-------- --------
3	2.0 200	3.5 5,000	5.0 10,000	5.0 60,000	7.0 87,000	-------- --------
4	3.5 300	3.5 1,500	4.5 3,000	6.0 40,000	6.7 60,000	7.0 87,000
5	3.5 90	3.5 1,000	4.0 2,500	4.2 20,000	5.0 50,000	6.0 87,000
6	4.0 75	4.0 5,000	4.5 20,000	5.0 50,000	6.0 87,000	-------- --------
7	4.0 100	4.0 20,000	5.0 50,000	6.0 87,000	-------- --------	-------- --------

[1]Subreach locations shown in figure 1; subreach names given in table 3.

Model Calibration

The initial phase of the model calibration was accomplished by inputting values of flood hydrographs, aquifer characteristics, channel characteristics, and stage-discharge relations for each subreach and flood. All model calibrations were made for individual subreaches because all flood hydrographs selected for the entire 164.8-mi study reach from Portland to John Martin Reservoir indicated major tributary inflow at some point in the reach. Each respective flood hydrograph, therefore, was routed only to the next downstream streamflow-gage location and then compared with the observed hydrograph. An optimum fit between simulated and observed flood hydrographs at the downstream station was obtained by varying flood-wave speed, flood-wave dispersion, transmissivity, and channel storage. The transmissivity and channel-storage values (table 3), flood-wave speed (table 5), and flood-wave dispersion (table 6) reflect the final model-calibration results. For most of the study reach the final flood-wave speed values compare favorably to those computed by Livingston (1978).

Table 6.--*Relations between flood-wave dispersion and flood-peak discharge determined from the model calibration of the Arkansas River from Portland to John Martin Reservoir*

Subreach No.[1]	Upper number is flood-wave dispersion, in square feet per second; lower number is flood-peak discharge, in cubic feet per second				
1	30,000 200	30,000 87,000	------ ------	------ ------	------ ------
2	25,000 200	25,000 87,000	------ ------	------ ------	------ ------
3	5,000 200	10,000 10,000	10,000 87,000	------ ------	------ ------
4	7,500 200	7,500 87,000	------ ------	------ ------	------ ------
5	7,500 75	7,500 87,000	------ ------	------ ------	------ ------
6	15,000 75	15,000 3,000	10,000 15,000	5,000 30,000	5,000 87,000
7	15,000 100	15,000 2,000	10,000 15,000	5,000 30,000	5,000 87,000

[1]Subreach locations shown in figure 1; subreach names given in table 3.

Comparisons of several typical measured and simulated flood hydrographs are shown in figures 2 to 5. These results indicate a fairly wide range of flood conditions, with peak discharges ranging from 11,000 to 46,000 ft^3/s. Accuracies of the simulated hydrographs, shown in figures 2 to 5, are considered good, with the computed peak-discharge values within 10 percent of the observed values. The accuracies of the majority of the other flood-hydrograph simulations (table 2) are considered only fair. Therefore the overall accuracy of the calibrated model is considered good to fair.

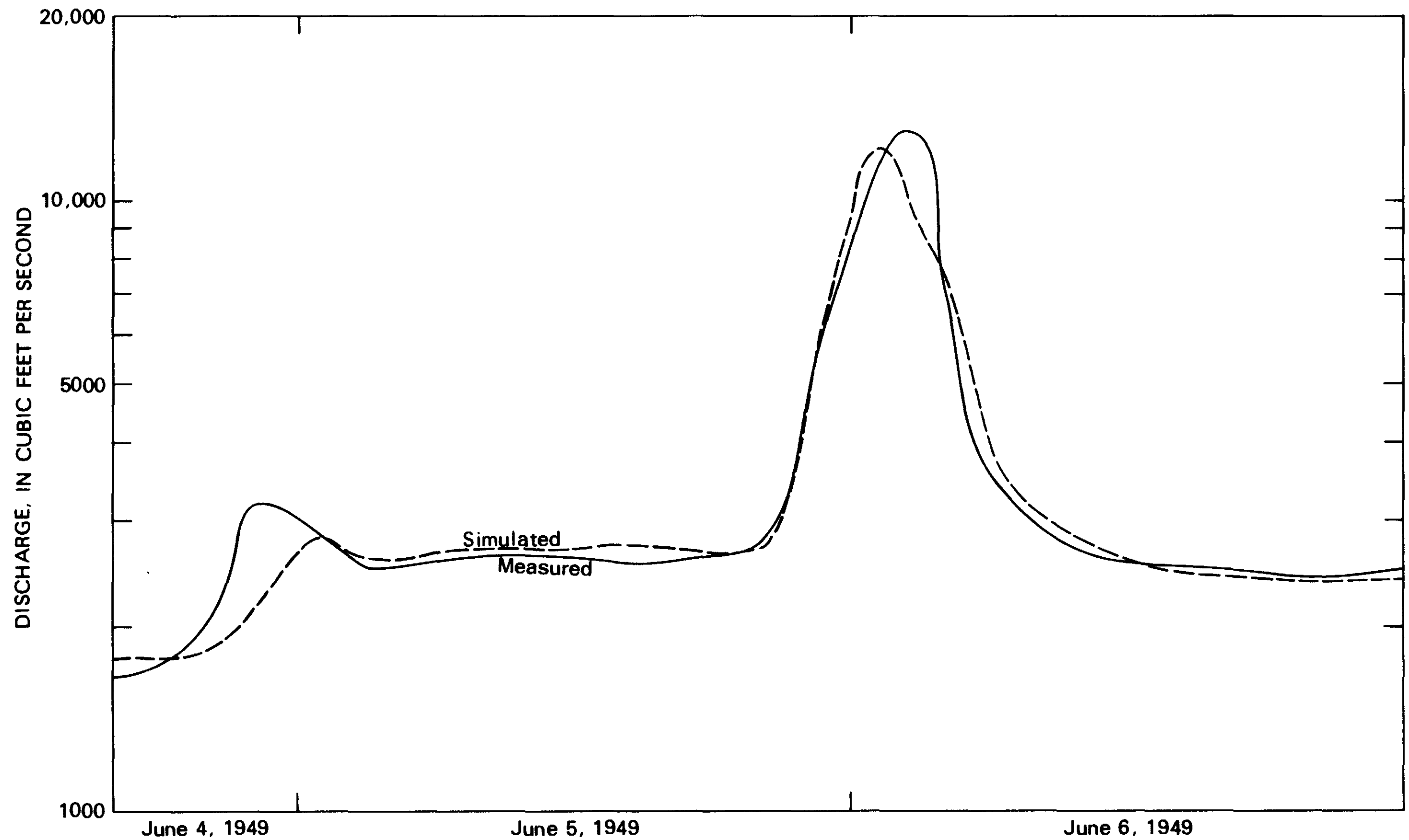

Figure 2.--Measured and simulated streamflow at the Arkansas River above Pueblo streamflow-gaging station, June 4-6, 1949.

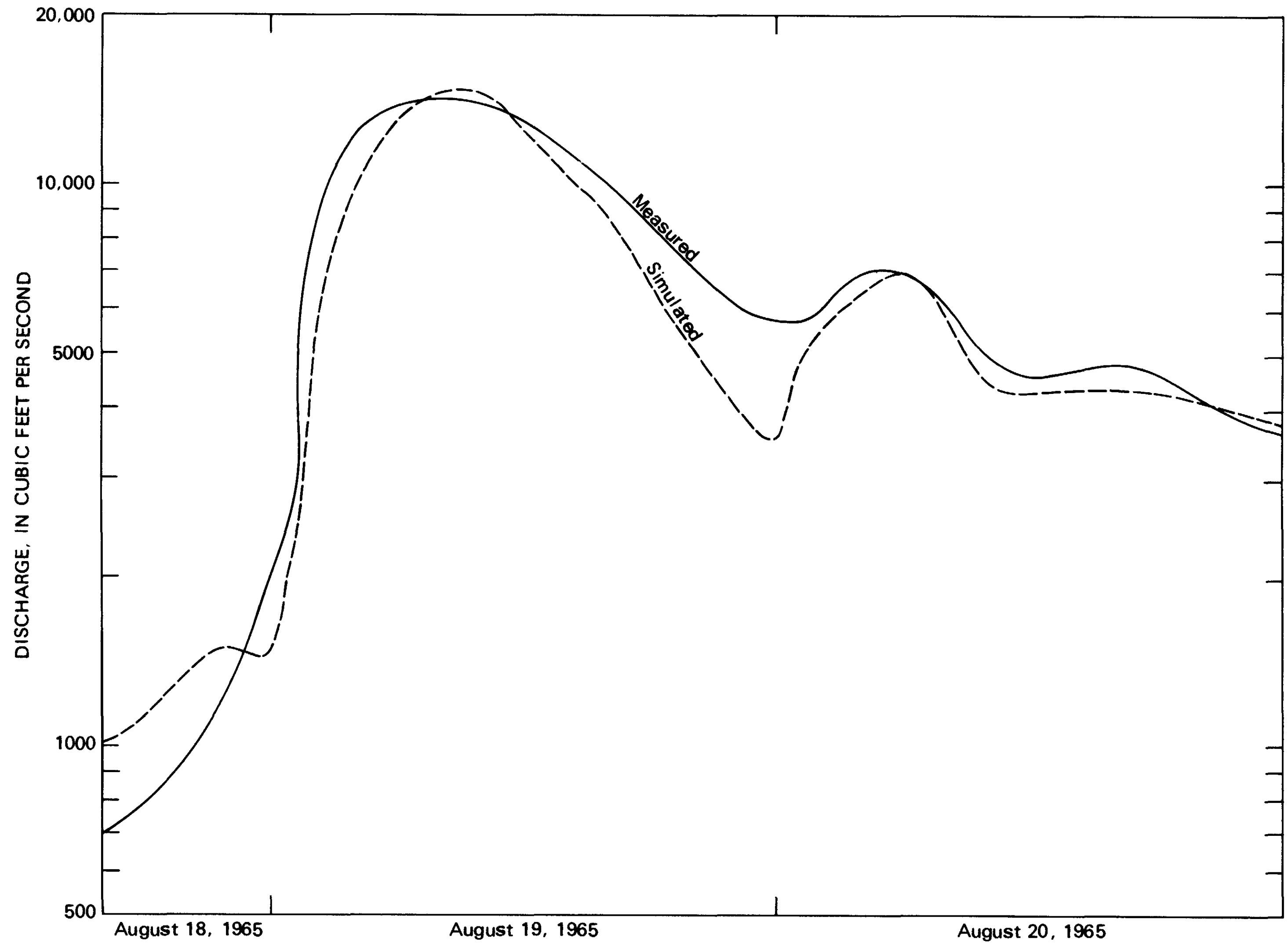

Figure 3.--Measured and simulated streamflow at the Arkansas River at Catlin Dam streamflow-gaging station, August 18-20, 1965.

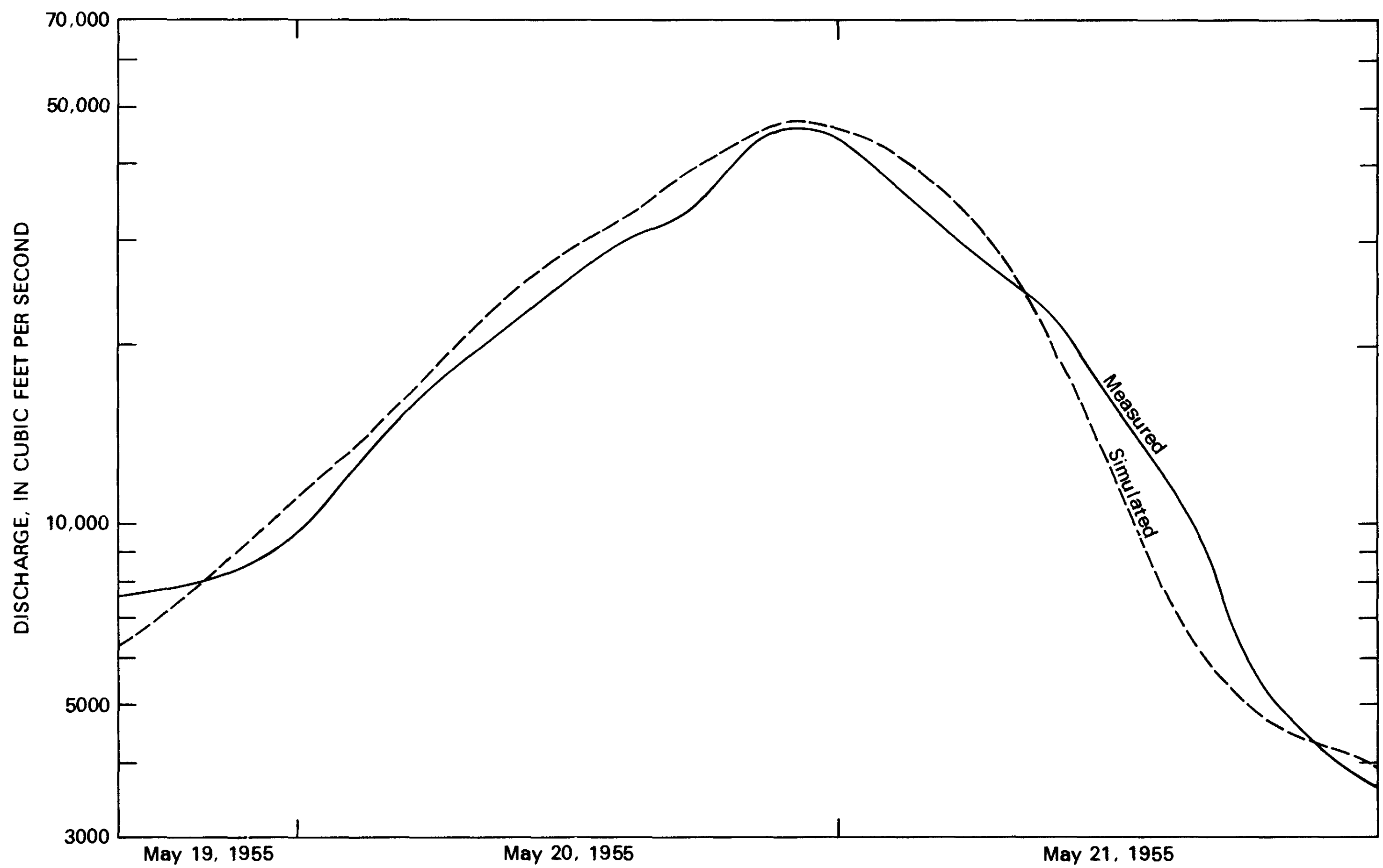

Figure 4.--Measured and simulated streamflow at the Arkansas River at Las Animas streamflow-gaging station, May 19-21, 1955.

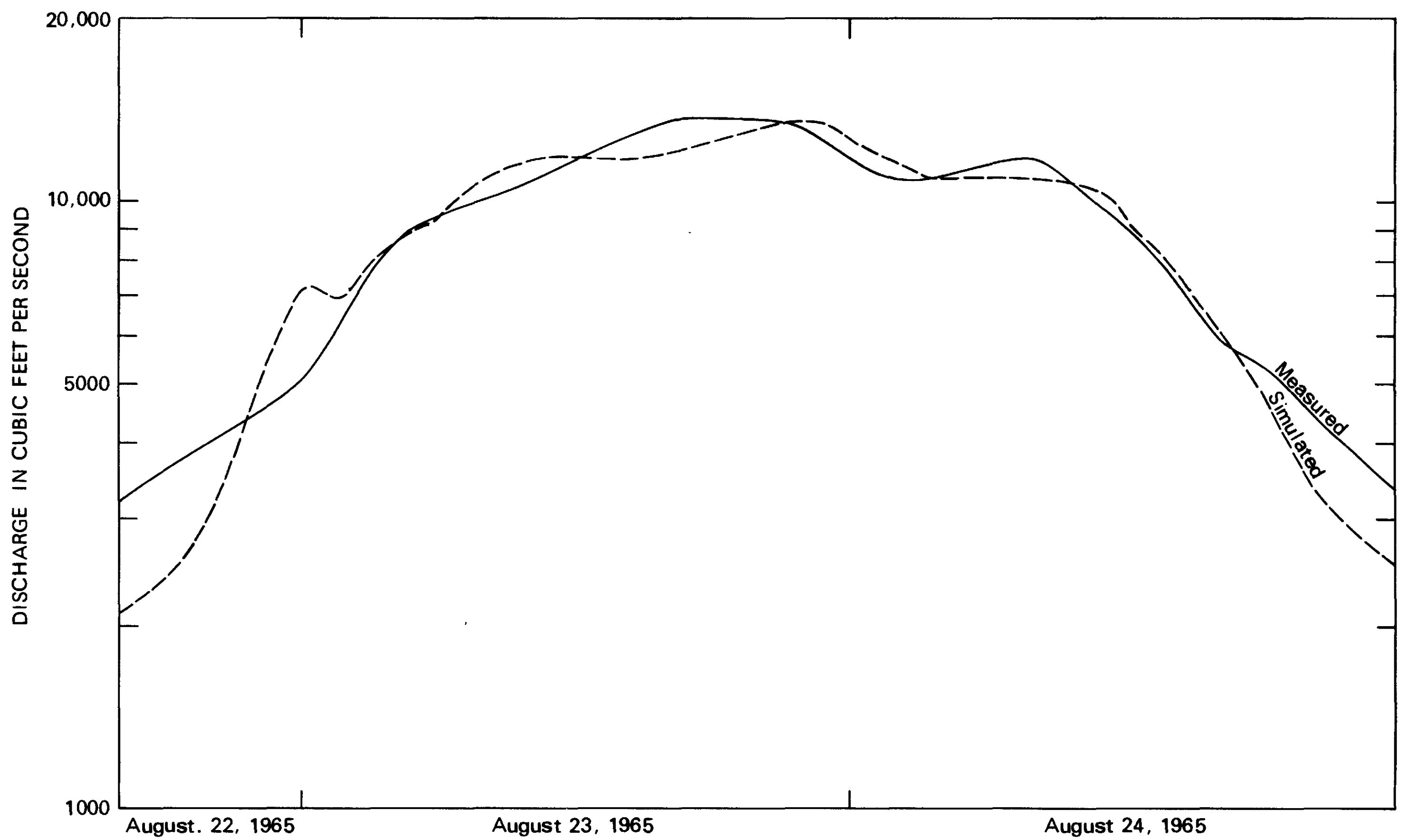

Figure 5.--Measured and simulated streamflow at the Arkansas River at Las Animas streamflow-gaging station, August 22-24, 1965.

Model Accuracy and Sensitivity

In most studies of this type, one sample of data is used for model calibration and another sample of data is used for model verification. In this study, the limited number of acceptable flood events permitted only completion of model calibration. Without data for model verification, other methods were used as indicators of the reliability of simulations that would be obtained from the calibrated model.

To give some indication of the accuracy of the model to predict specifically flood peaks, magnitude and traveltime, and the hydrograph volume, a larger, less restrictive number of floods than previously described for the model calibration was used. The observed floods that were used in this analysis included all measured annual peaks and most secondary peaks greater than 2,000 ft^3/s. The only screening criterion was that the peak discharge at the upstream station had to be greater than the peak discharge at the downstream station. This allowed more floods to be used, but tributary inflows will have adverse effects, especially on the observed-volume values and the peak and traveltime values in some instances. Values for flood volumes and traveltimes were not available for some floods and could not be considered in this analysis. Several multiple-peak events not considered in the calibration phase were, however, included as part of this analysis. Comparisons between predicted and observed flood-peak discharge, flood-peak traveltime, and flood-hydrograph volumes are shown in tables 7 and 8. A total of 53 floods was used for the flood-peak discharge, 16 events for the flood-peak traveltime, and 14 events for the flood-hydrograph volume. In each instance the tabulations list the observed, predicted, and percentage difference. The percentage difference was computed as:

$$\text{Percentage difference}=[(\text{predicted}-\text{observed})/\text{observed}]100. \tag{1}$$

Results of the peak-discharge predictions indicated a mean percentage difference of +31.8 percent for the 53 events. Prediction accuracies ranged from +134 to -26 percent with three outliers greater than +100 percent difference for the predicted and observed values. The mean percentage difference for the flood-peak traveltime was 1.4 percent with a range of values from 100 to -50 percent. All values were equal to or less than ±50 percent difference except one outlier. The mean percentage difference for flood-hydrograph volume was +5.5 percent. The percentage differences ranged from +78 to -49 percent with all values less than ±50 percent except two outliers.

Results of these accuracy tests are considered favorable, considering that these data basically were not restricted in the flood-selection process described earlier, and therefore large differences between the observed and predicted values could be expected. In all three comparisons the majority of these differences was less than about 50 percent. Predictions using multiple-peak events exhibited the same approximate percentage differences as the single-peak events.

Table 7.--*List of observed and predicted flood-peak discharges and traveltimes for selected floods*

Subreach No.[1]	Date of flood	Flood-peak discharges (cubic feet per second)		Percent difference	Flood-peak traveltimes (hours)		Percent difference
		Observed	Predicted		Observed	Predicted	
1	July 1944	5,980	7,200	20	--	--	---
	July 1947	7,280	9,900	36	8	4	-50
	June 1949	12,800	15,400	20	8	4	-50
	June 1965	4,900	5,600	14	--	--	---
	June 1965	10,400	10,500	1	--	--	---
	June 1965	6,800	5,700	-16	--	--	---
	August 1965	23,500	17,500	-26	--	--	---
	July 1967	4,780	4,150	-13	--	--	---
2	July 1941	6,850	6,000	-12	--	--	---
	August 1945	8,050	7,500	-7	--	--	---
	August 1946	6,720	5,700	-15	--	--	---
	June 1949	9,770	10,600	8	4	5	25
	July 1949	5,400	4,800	-11	--	--	---
	August 1966	8,550	9,100	6	6	5	-17
3	April 1942	12,000	12,700	6	--	--	---
	June 1948	12,200	12,100	-1	--	--	---
	June 1965	43,100	45,500	6	10	6	-40
	August 1965	4,500	4,400	-2	6	7	17
	August 1968	4,260	5,100	20	--	--	---
	July 1969	5,900	12,300	108	--	--	---
	August 1972	11,900	13,900	17	--	--	---
4	August 1965	14,000	13,500	-4	2	4	100
	September 1966	13,200	17,000	29	--	--	---
	August 1972	10,000	10,100	1	--	--	---
	July 1973	4,850	5,300	9	--	--	---
	July 1975	6,050	6,800	12	8	4	-50
	August 1976	7,400	8,000	8	6	4	-33

Table 7.--*List of observed and predicted flood-peak discharges and traveltimes for selected floods*--Continued

Subreach No.[1]	Date of flood	Flood-peak discharges (cubic feet per second)		Percent difference	Flood-peak traveltimes (hours)		Percent difference
		Observed	Predicted		Observed	Predicted	
5	June 1965	31,700	38,000	20	14	8	-43
	September 1966	11,700	10,500	-10	--	--	---
	August 1968	6,820	6,400	-6	--	--	---
	August 1970	2,090	4,900	134	--	--	---
	August 1972	9,110	7,400	-19	--	--	---
	July 1975	2,900	3,900	34	10	9	-10
6	April 1942	23,600	27,500	17	--	--	---
	August 1945	8,840	7,800	-12	--	--	---
	June 1947	9,580	7,800	-19	--	--	---
	June 1948	8,780	10,300	17	--	--	---
	June 1950	8,170	13,500	65	--	--	---
	July 1953	6,260	11,800	88	--	--	---
	August 1954	5,880	5,400	-8	--	--	---
	May 1955	44,000	41,000	-7	9	10	11
	July 1958	6,280	14,200	126	--	--	---
	July 1960	4,950	5,600	13	--	--	---
	August 1961	4,340	3,600	-17	--	--	---
	July 1962	3,820	4,200	10	--	--	---
	June 1965	21,000	22,100	5	16	10	-38
	June 1965	22,100	24,600	11	16	10	-38
	August 1965	10,000	9,100	-9	--	--	---
	August 1965	13,400	12,600	-6	--	--	---
	August 1968	2,660	3,500	32	6	10	67
	August 1972	4,130	5,100	23	--	--	---
	August 1976	3,900	4,200	8	9	10	11
	August 1977	4,300	4,500	5	--	--	---

[1]Subreach locations shown in figure 1.

Table 8.--*List of observed and predicted flood volumes for selected floods*

Subreach No.[1]	Date of flood	Flood volumes (acre-feet)		Percent difference
		Observed	Predicted	
1	July 1947	2,700	2,900	7
	June 1949	4,800	5,100	6
2	June 1949	4,100	5,500	34
3	June 1965	25,200	37,000	47
	August 1965	1,200	1,700	42
4	August 1965	18,000	9,100	-49
	July 1975	3,000	3,300	10
	August 1976	6,400	3,900	-39
5	June 1965	31,700	33,000	4
	July 1975	1,070	1,900	78
6	May 1955	59,800	41,000	-31
	June 1965	22,500	17,000	-24
	June 1965	15,600	26,000	67
	August 1968	1,400	1,300	-7

[1]Subreach locations shown in figure 1.

A partial sensitivity analysis also was performed on some of the model parameters, namely the aquifer transmissivity and storage coefficient; all other model parameters were kept constant during this particular test. Results of this analysis are given in table 9. The sensitivity analysis was made only for one flood in the La Junta to Las Animas subreach. The flood occurred on May 20, 1955 (table 2), and had an upstream peak discharge of 50,000 ft^3/s. During the sensitivity analysis, the transmissivity was allowed to vary from 1,000 to 100,000 ft^2/d, and the storage coefficient was allowed to vary from 0.05 to 1.0. For purpose of comparison, the downstream peak discharge and volume computed during the sensitivity analysis were compared with values computed for the calibrated model (table 9). The percentage changes in peak discharge ranged from +1.1 to -2.9 percent with the largest change occurring with the model transmissivity set at the maximum reasonable value of 100,000 ft^2/d (R. T. Hurr, U.S. Geological Survey, oral commun., 1979). The percentage changes in volume, similarly, ranged from +0.80 to -2.7 percent with the largest change again occurring with the transmissivity set at 100,000 ft^2/d. These results indicate the model is relatively insensitive to changes in aquifer characteristics.

Table 9.--*Results of model transmissivity and storage-coefficient sensitivity analysis using the May 1955 flood in the La Junta to Las Animas subreach as reference*

Transmissivity (square feet per day)	Storage coefficient (dimensionless)	Downstream peak discharge		Downstream volume	
		Computed (cubic feet per second)	Difference from calibration results (percent)	Computed (acre-feet)	Difference from calibration results (percent)
10,000	0.15	[1]45,850	----	[1]113,000	----
1,000	.15	46,350	1.1	113,900	0.8
5,000	.15	46,150	.7	113,400	.4
8,000	.15	46,000	.3	113,100	.1
20,000	.15	45,550	-.7	112,400	-.5
40,000	.15	45,250	-1.3	111,700	-1.2
[2]100,000	.15	44,500	-2.9	110,000	-2.7
10,000	.05	46,000	.3	113,500	.4
10,000	.10	45,900	.1	113,200	.2
10,000	.20	45,700	-.3	112,800	-.2
10,000	.30	45,500	-.8	112,400	-.5
10,000	.40	45,400	-1.0	112,200	-.7
10,000	.50	45,250	-1.3	111,900	-1.0
10,000	[3]1.00	45,000	-1.9	110,900	-1.9

[1]Model calibration results for peak discharge and volume.
[2]Maximum reasonable transmissivity value (R. T. Hurr, U.S. Geological Survey, oral commun., 1979).
[3]Body of open water has storage coefficient of 1.0.

Model Simulations

Model simulations were made for the entire study reach from Portland to John Martin Reservoir. Sixteen simulations were completed for a wide range of peak discharges and antecedent streamflow conditions. Four typical hydrographs representing peak discharges of 87,000, 40,000, 20,000, and 5,000 ft^3/s were used as the initial flood conditions in the simulation procedure. These typical flood hydrographs, shown in figure 6, are for an antecedent streamflow of 400 ft^3/s. The plotted hydrographs were derived from observed flood hydrographs at the Portland streamflow-gaging station.

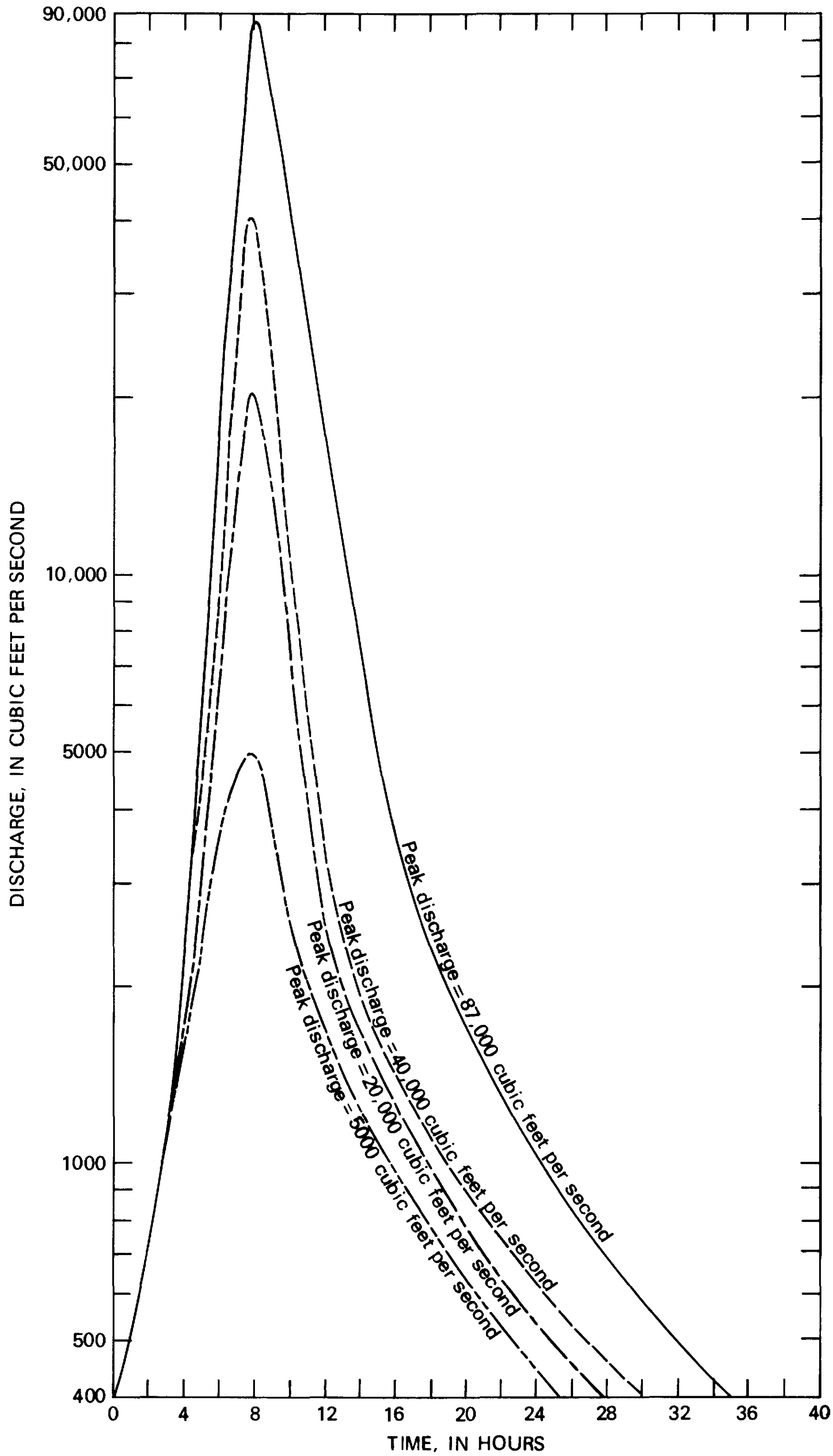

Figure 6.--Model simulation, typical flood hydrographs routed downstream from Portland.

Four different antecedent streamflow conditions also were used for each of the four typical flood hydrographs. Average subreach values were obtained from a correlation study of seasonal monthly streamflow along the Arkansas River from Pueblo to Las Animas (R. K. Livingston, U.S. Geological Survey, written commun., 1979). For this study the antecedent streamflow conditions at Portland were assumed to equal the antecedent conditions at Pueblo. Antecedent streamflow initial values at Portland derived from this analysis were 2,000, 1,350, 600, and 400 ft^3/s and were assumed to represent arbitrary conditions of very high, high, medium, and low antecedent streamflow. The average subreach antecedent streamflow conditions used in this analysis are listed in table 10.

Results of the flood-peak simulations shown as discharge profiles are presented in figures 7 to 10. The approximate shapes of the curves are similar for the same initial peak-discharge values (figs. 7-10). The corresponding downstream flood-peak-discharge values at the John Martin Reservoir did not vary appreciably with changes in antecedent streamflow conditions. For example, using the 40,000-ft^3/s initial peak discharge, most downstream values ranged from only 8,600 to 8,150 ft^3/s.

The curves for the larger initial peak discharges, 20,000, 40,000, and 87,000 ft^3/s, had an approximate straight line slope on the semi-logarithmic graph paper (figs. 7-10) for the Portland to Las Animas subreaches and then a flatter slope for the Las Animas to John Martin Reservoir subreach. The curve for the 5,000-ft^3/s initial peak discharge exhibited a more inconsistent curve slope (figs. 7-10). Greater stream interaction during smaller flows can contribute to the inconsistency in the discharge-profile slope (figs. 7-10).

Table 10.--*Average subreach antecedent streamflow conditions used for model simulations*

[Values in cubic feet per second obtained from R. K. Livingston, U.S. Geological Survey, written commun., 1979]

Antecedent streamflow condition	Subreach no.[1]						
	1	2	3	4	5	6	7
Very high-------	2,000	2,100	1,700	1,550	600	500	700
High------------	1,350	1,450	1,100	1,000	375	300	450
Medium----------	600	800	500	450	150	100	200
Low-------------	400	600	400	350	90	80	130

[1]Subreach locations shown in figure 1.

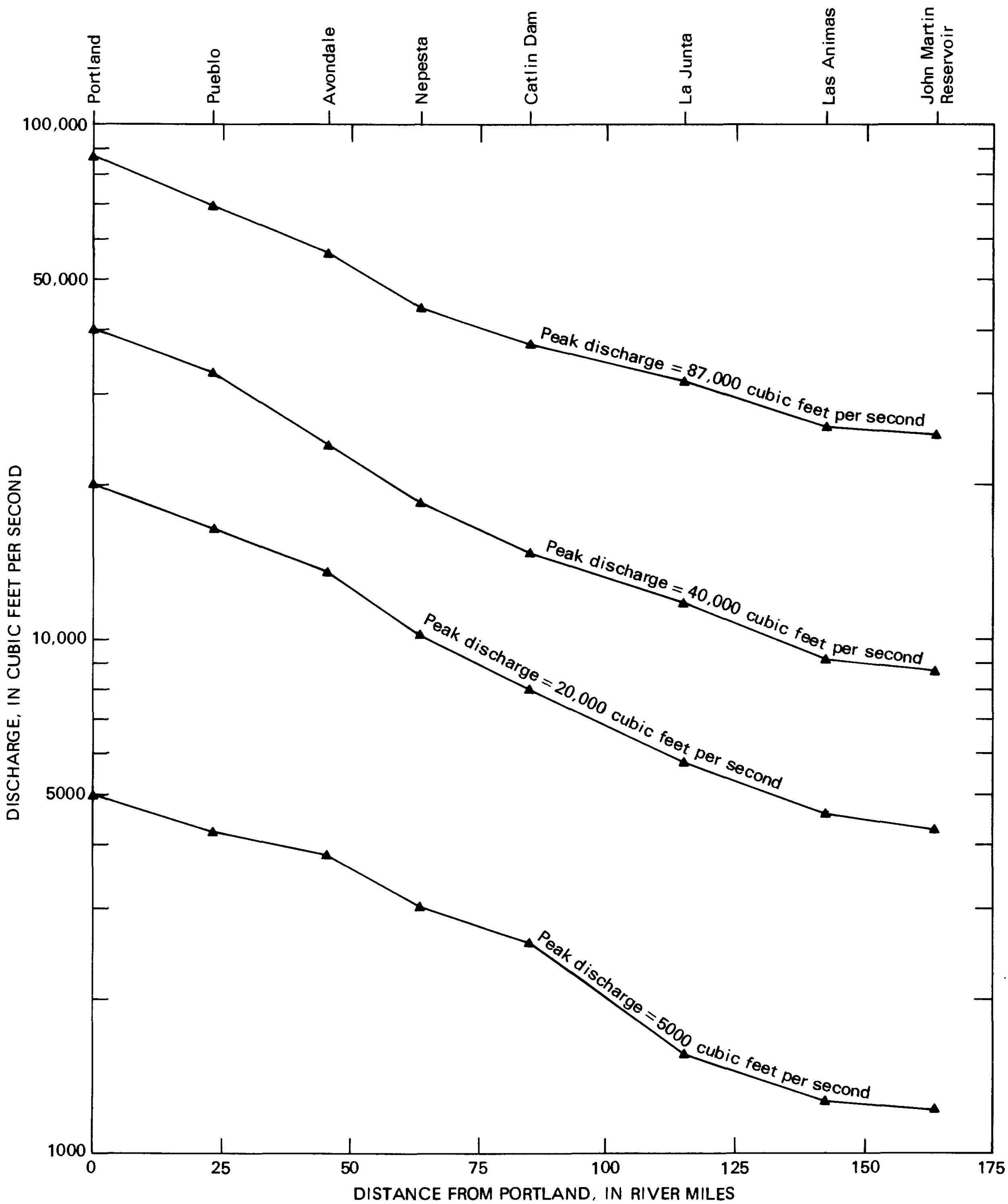

Figure 7.--Simulated peak-discharge values using an antecedent streamflow of 2,000 cubic feet per second at Portland and varying initial discharge values.

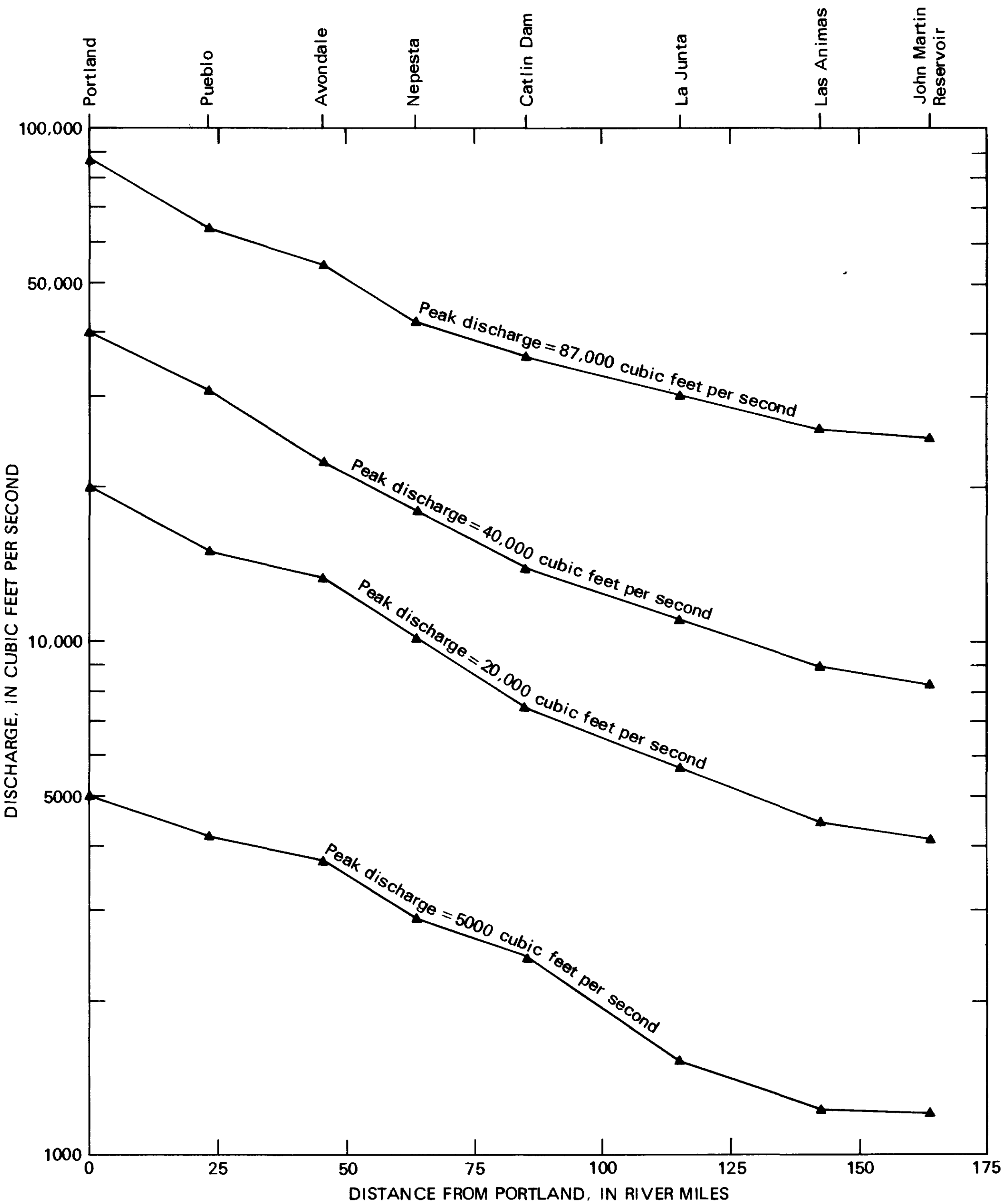

Figure 8.--Simulated peak-discharge values using an antecedent streamflow of 1,350 cubic feet per second at Portland and varying initial discharge values.

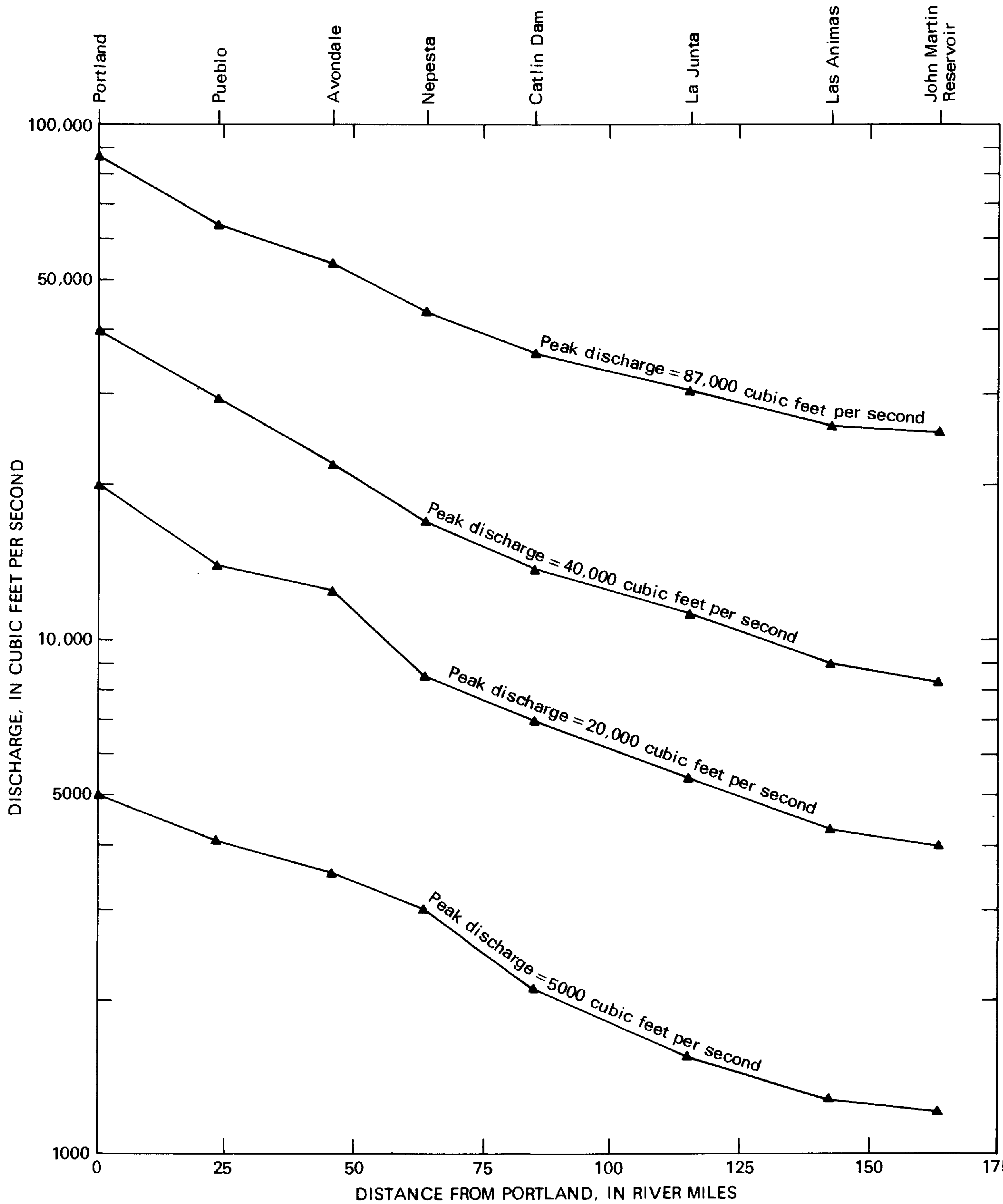

Figure 9.--Simulated peak-discharge values using an antecedent streamflow of 600 cubic feet per second at Portland and varying initial discharge values.

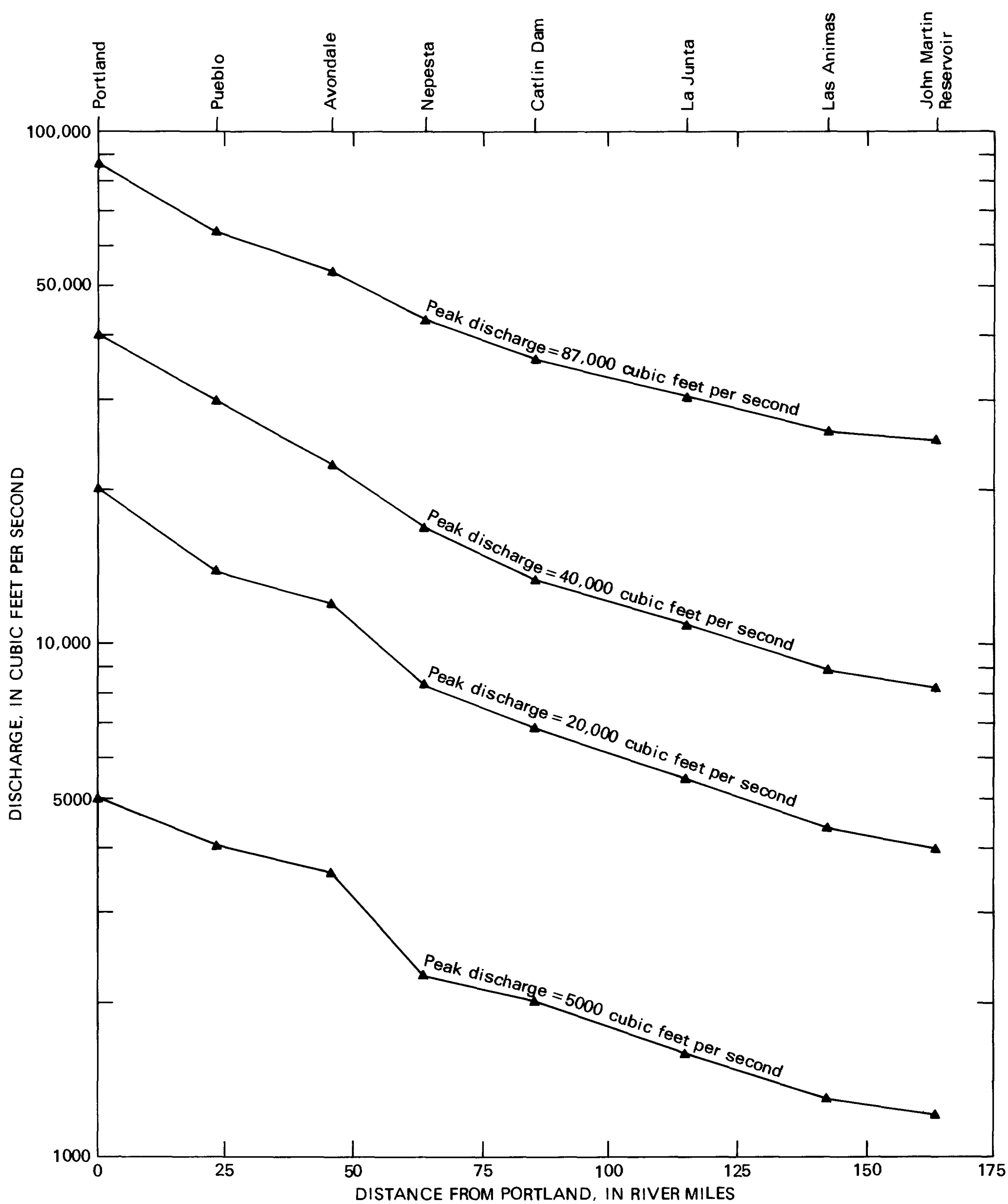

Figure 10.--Simulated peak-discharge values using an antecedent streamflow of 400 cubic feet per second at Portland and varying initial discharge values.

The variation of simulated flood volumes for the four typical floods and with varying antecedent streamflow conditions is given in table 11 for each site number. Flood-hydrograph volumes at the downstream locations were computed using the procedure described by Livingston (1978). With this procedure, the volume is computed based on the specific time for the hydrographs to decrease to 5 percent of net maximum peak value. The net maximum peak value is defined as the net peak value greater than a given antecedent streamflow base. For example, with a total peak discharge of 1,000 ft^3/s and an antecedent streamflow of 100 ft^3/s the volume would be based on 5 percent of 900 ft^3/s or a net value of 145 ft^3/s (100+45 ft^3/s =145 ft^3/s). The computed flood hydrograph volume in this instance would be based on all streamflow values greater than 100 ft^3/s until the hydrograph recession value is equal to 145 ft^3/s. An illustrative example of this computation is shown on figure 11. The flood-hydrograph volumes given in table 11 were computed by the following procedure:

1. Multiply the predicted 2-hour time interval discharge values by the constant 0.08333 (2 hours/24 hours).

2. Sum the values in step 1 for all values greater than 5 percent of the net maximum peak values.

3. Compute the antecedent parts of the hydrograph by multiplying given antecedent streamflow by the same time base used for step 1.

4. Subtract item 3 from item 2, which will yield the net hydrograph volume, in cubic feet per second per day.

5. Convert item 4 to acre-feet by multiplying by the constant 1.9835.

Additional discussion of the flood-hydrograph volume computation will be given in a subsequent section of this report. The flood-hydrograph volumes listed in table 11 are given in a matrix format covering the different initial peak discharge, antecedent streamflow, and site locations. The listed values indicate good correlation with both the antecedent streamflow and the initial peak discharge. Flood volumes, for example, for an initial peak discharge of 5,000 ft^3/s have values ranging from 760 to 2,080 acre-ft (table 11) for the downstream John Martin Reservoir site location. Percentage volume losses per mile of study reach ranged from 0.06 percent per mile for the higher antecedent flows to 0.11 percent per mile for the lower antecedent flows. These percentage volume losses per mile of reach compare favorably to those computed by Livingston (1978).

The traveltimes of peak discharge for varying peak discharges at Portland and antecedent streamflow conditions are given in table 12. These results show little change in traveltime with either different initial peak-discharge values or antecedent streamflow conditions. These small changes in traveltime are partly due to the small differences in flood-wave speed with changes in peak discharge (table 5).

Table 11.--*Initial and computed discharge hydrograph volumes for varying initial and antecedent conditions*

Initial peak discharge (cubic feet per second)	Site No.[1]							
	[2]1	2	3	4	5	6	7	8
	Very high antecedent streamflow condition (acre-feet)[3]							
87,000	27,800	27,300	27,100	26,700	26,600	25,900	25,500	25,200
40,000	9,720	9,450	9,340	9,180	9,110	8,820	8,600	8,410
20,000	4,920	4,800	4,650	4,580	4,540	4,370	4,270	4,180
5,000	930	890	840	830	810	780	760	760
	High antecedent streamflow condition (acre-feet)[3]							
87,000	28,600	28,100	27,800	27,500	27,300	26,600	26,200	25,800
40,000	10,200	10,000	9,840	9,680	9,570	9,280	9,070	8,860
20,000	5,490	5,310	5,170	5,070	5,020	4,850	4,730	4,630
5,000	1,390	1,320	1,260	1,250	1,220	1,170	1,150	1,140
	Medium antecedent streamflow condition (acre-feet)[3]							
87,000	30,000	29,000	28,600	28,300	28,200	27,700	27,200	26,800
40,000	11,300	10,800	10,600	10,500	10,400	10,100	9,910	9,700
20,000	6,350	6,130	5,940	5,860	5,800	5,610	5,490	5,370
5,000	2,140	2,020	1,910	1,890	1,860	1,800	1,780	1,740
	Low antecedent streamflow condition (acre-feet)[3]							
87,000	30,300	29,300	28,900	28,600	28,500	28,000	27,600	27,200
40,000	11,700	11,100	10,900	10,800	10,800	10,500	10,300	10,000
20,000	6,740	6,430	6,240	6,190	6,120	5,930	5,810	5,690
5,000	2,540	2,400	2,280	2,260	2,220	2,160	2,130	2,080

[1]See table 1 and figure 1 for respective site numbers, names, and locations.
[2]Initial discharge hydrograph volume at Portland.
[3]See table 10 for antecedent streamflow values.

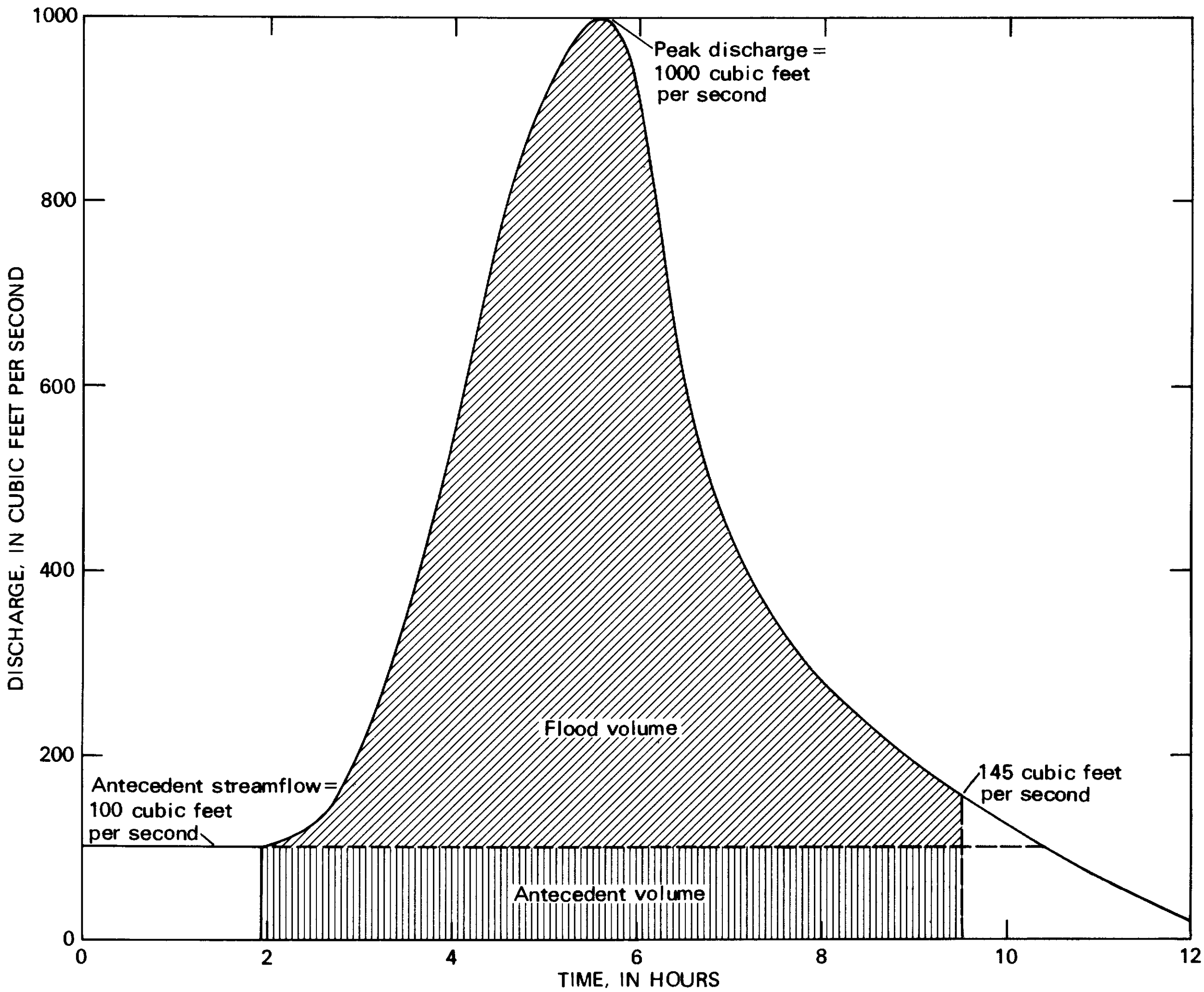

Figure 11.-- Flood hydrograph-volume computations.

Table 12.--*Peak-discharge traveltimes for varying initial and antecedent Portland peak-discharge values*

Initial peak discharge (cubic feet per second)[1]	Site No.[2]						
	2	3	4	5	6	7	8
	Very high antecedent streamflow (hours)[3]						
87,000	5	9	16	25	36	46	52
40,000	5	9	16	25	36	46	52
20,000	5	9	16	25	36	47	53
5,000	5	9	16	26	39	50	55
	High antecedent streamflow (hours)[3]						
87,000	5	9	16	26	36	46	52
40,000	5	9	16	26	36	46	52
20,000	5	9	16	26	37	47	53
5,000	5	9	16	27	40	51	56
	Medium antecedent streamflow (hours)[3]						
87,000	5	9	16	26	36	46	52
40,000	5	9	16	26	36	46	52
20,000	5	9	16	26	37	48	53
5,000	5	9	16	28	41	52	57
	Low antecedent streamflow (hours)[3]						
87,000	5	9	16	26	36	46	52
40,000	5	9	16	26	36	46	53
20,000	5	9	17	28	38	49	54
5,000	5	9	18	28	41	52	58

[1]Initial peak-discharge values at Portland.
[2]See table 1 and figure 1 for respective site numbers, names, and locations.
[3]See table 10 for subreach antecedent streamflow conditions.

MULTIPLE-REGRESSION ANALYSIS

One of the most effective ways known for defining streamflow characteristics on a regional basis is by applying multiple-regression techniques to a given data base. A similar approach for a stream reach, for example, as described by Boning (1974), also may be applied using the same multiple-regression techniques. Therefore for this study, multiple-regression equations were derived for the prediction of streamflow characteristics for the Arkansas River downstream from Portland.

The multiple-regression analysis equation is given in two forms:

$$Y=aX_1^{b1}.X_2^{b2}.X_3^{b3} \; . \; . \; . \; . \; X_i^{bi}, \text{ or} \tag{2}$$

$$logY=loga+b_1logx_1+b_2logX_2 \; . \; . \; . \; . \; b_ilogX_i, \text{ and}$$

$$Y=a+b_1X_1+b_2X_2 \; . \; . \; . \; . \; b_iX_i, \tag{3}$$

where Y=dependent variable (streamflow characteristic),

a=regression constant,

$X_1,X_2,X_3 \; . \; . \; . \; . \; X_i$=independent variables (channel or aquifer characteristics, initial conditions, or antecedent-streamflow conditions), and

$b_1,b_2,b_3, . \; . \; . \; . \; b_i$=regression coefficients.

Dependent and Independent Variables

The dependent variables used for the regression analysis were peak discharge (*PK*), flood-peak traveltime (*TT*), and flood hydrograph volume (*VOL*). Values used for these three variables are given in figures 7 to 10 and in tables 11 and 12.

Independent variables used in the analysis included distance downstream from Portland (*DIST*), subreach slope (*SLOP*), transmissivity (*TR*), storage coefficient (*STOR*), aquifer length (*AQLEN*), aquifer width (*AQWID*), average flood-wave speed (*AVGWVSP*), antecedent streamflow (*ANTFLOW*), initial peak discharge (*INTPK*), and initial flood volume (*INTVOL*). All independent variables listed in table 13, except distance, which was the weighting mechanism, and aquifer length, which was accumulated from Portland, were computed as weighted averages on the basis of stream distance downstream from Portland. In effect, this weighting procedure has most of the independent variables representing a subreach that originates at Portland. The weighting equation has the following form:

$$\text{weighted value}=(VAR_1DIST_1+VAR_2DIST_2 \; . \; . \; . \; . \; VAR_nDIST_n)/\sum_{i=1}^{n} DIST_i \tag{4}$$

where $VAR_1,VAR_2,VAR_3 \; . \; . \; . \; . \; VAR_n$=variable values for given subreach locations, and

$DIST_1,DIST_2,DIST_3 \; . \; . \; . \; .DIST_n$=individual subreach lengths.

Table 13.--*Selected aquifer, streamflow, and subreach characteristics used in the multiple-regression analysis*

Subreaches originating from Portland[1]						
Pueblo	Avondale	Nepesta	Catlin Dam	La Junta	Las Animas	John Martin Reservoir
Distance downstream from Portland (river miles)						
22.9	46.2	64.8	86.0	117.0	144.0	164.8
Subreach slope (feet per foot)[2]						
0.0023	0.0021	0.0019	0.0017	0.0015	0.0015	0.0014
Transmissivity (square feet per day)[2]						
4,000	7,000	8,000	9,000	9,500	9,500	9,500
Storage coefficient (cubic feet per cubic foot)[2]						
0.15	0.12	0.12	0.11	0.12	0.12	0.13
Aquifer width (feet)[2]						
1,600	3,500	4,600	6,700	9,900	10,700	12,400
Aquifer length (miles)[3]						
19	37	47	59	78	93	108
Flood-wave speed averaged for flood-peak range (feet per second)[2,4]						
6.6	7.1	6.6	6.4	6.0	5.8	5.7
Very high antecedent streamflow (cubic feet per second)[2]						
2,000	2,030	1,990	1,900	1,680	1,470	1,360
High antecedent streamflow (cubic feet per second)[2]						
1,350	1,380	1,350	1,270	1,120	970	900
Medium antecedent streamflow (cubic feet per second)[2]						
600	650	650	610	530	450	410
Low antecedent streamflow (cubic feet per second)[2]						
400	400	450	460	440	380	330

[1]Subreaches noted are referenced from Portland; see figure 1 for locations.
[2]Table values are distance-weighted average values from Portland.
[3]Cumulative total of aquifer length downstream from Portland.
[4]Values computed by averaging subreach values for the flood peaks of 87,000, 40,000, 20,000, and 5,000 cubic feet per second.

Individual subreach values for subreach length, channel slope, aquifer width, aquifer length, transmissivity, and storage coefficient are listed in table 3, individual subreach antecedent streamflow conditions are listed in table 10, and the flood-wave speed data used to compute the average speed over the 5,000 to 87,000-ft^3/s peak discharge range are given in table 5. An example computation of subreach slope, using equation 4, for a reach from Portland to Catlin Dam is as follows:

weighted value=[0.0023(22.9)+0.0019(23.3)+0.0013(18.6)+0.0012(21.2)]/86;

weighted value=0.0017 feet per foot.

All data for this weighting analysis were based on values from tables 3, 5, and 10, and equation 4. The weighting technique presented was used in this report so that any predictive equation for *PK*, *VOL*, and *TT* would be referenced to the Portland starting location.

Discussion of Results

Initially, the multiple-regression statistical models formulated considered all independent variables; namely, aquifer and channel characteristics, initial conditions, and antecedent streamflow conditions. The analysis was completed using the forward-selection method for all variables. This method operates by adding variables one at a time to the model. An F (variance-ratio test) and R (correlation coefficient)-squared statistic is computed after the addition of each variable. Variables are thus added one by one to the model until no remaining variable produces a significant improvement in the F and R-squared statistic. The final analysis, therefore, included only those independent variables that are significant at the F-statistic, 5-percent level and also are not highly correlated with another variable. Some personal judgment considerations on ease of user application also were made on the final model equations with some further simplifications being considered for the final predictive equations.

The final model equations for dependent variables, flood peak (*PK*), and hydrograph volume (*VOL*) were computed on the basis of the *log-transformed version* of equation 2. The flood-peak traveltime (*TT*) is given in the form of the linear equation version (equation 3). These final equations were selected on the basis of the minimum average standard errors of estimate (*SE*), greater correlation coefficients (*R*), significant at the 5-percent level, and ease of the equation application. The following are the final selected regression equations:

$$PK=7.51(INTPK)^{0.989}(DIST)^{-0.649} \qquad (SE=14 \text{ percent}, R=0.99); \qquad (5)$$

$$VOL=0.802(INTVOL)^{1.027}(DIST)^{-0.0548}(ANTFLOW)^{0.0178} \qquad (SE=2 \text{ percent}, R=0.99); \qquad (6)$$

and

$$TT=41.14+(0.3050)(DIST)-(6.560)(AVGWVSP) \qquad (SE=1.55 \text{ hours}, R=0.99). \qquad (7)$$

Equations 5, 6, and 7 are based on data generated only from the various simulations from the streamflow-routing model. Therefore the result accuracies shown (*SE* and *R*) do not represent a measure of fit to the actual observed data.

APPLICATIONS

The flood-peak discharge (*PK*), flood-hydrograph volume (*VOL*), and flood-peak traveltime (*TT*) relationships (equations 5, 6, and 7) provide a convenient means of predicting these variables at any stream location between Portland and John Martin Reservoir.

As an example, assume we would like to know the *PK*, *VOL*, and *TT* at Las Animas streamflow-gaging station for a flood with an initial peak (*INTPK*) of 50,000 ft^3/s and an initial volume (*INTVOL*) of 10,000 acre-ft at the Portland streamflow gage. Antecedent streamflow prior to the flood is approximately 400 ft^3/s at the Portland and Pueblo streamflow gages which matches low streamflow conditions given in tables 10 and 13. The *PK* is computed using equation 5, as follows:

$$PK = 7.51\,(INTPK)^{0.989}(DIST)^{-0.649},$$

$$PK = 7.51\,(50{,}000)^{0.989}(144.0)^{-0.649},$$

$$PK = 13{,}200 \text{ cubic feet per second; and}$$

The *VOL* is computed using equation 6, as follows:

$$VOL = 0.802\,(INTVOL)^{1.027}(DIST)^{-0.0548}(ANTFLOW)^{0.0178},$$

$$VOL = 0.802\,(10{,}000)^{1.027}(144.0)^{-0.0548}(380)^{0.0178},$$

$$VOL = 8{,}710 \text{ acre-feet.}$$

The reach *TT* can be computed directly from equation 7 as follows:

$$TT = 41.14 + (0.3050)(DIST) - 6.560(AVGWVSP),$$

$$TT = 41.14 + (0.3050)(144.0) - 6.560(5.8),$$

$$TT = 47 \text{ hours.}$$

For the *VOL* computations the antecedent streamflow average value of 380 ft^3/s is used for the Portland to Las Animas streamflow gage. The value of the initial volume (*INTVOL*) is given, but it could have been computed using the procedure described on page 26. The example computation on page 26 uses a 2-hour time-interval discharge value but another time interval can be used, if desired. Greater accuracy in the *VOL* computation will result if a smaller time interval is used in the computation.

As a second example, assume we would like to know the *PK*, *VOL*, and *TT* at the Fort Lyon Canal headgate, which is located 93.6 miles downstream from Portland (Livingston, 1978) between the Catlin Dam and La Junta streamflow-gaging stations. The initial flood peak (*INTPK*) at Portland is 15,000 ft^3/s, the initial flood volume (*INTVOL*) at Portland is 5,000 acre-ft, and the antecedent streamflow prior to the flood at the Portland and Pueblo streamflow-gaging stations is approximately 1,000 ft^3/s.

Where an estimate of *PK*, *VOL*, and *TT* is desired at one point between streamflow gages, it is easier to compute values for the two streamflow-gage locations and then interpolate to the desired location. The *PK* is computed at Catlin Dam and La Junta as follows:

Catlin Dam

$$PK=7.51(15{,}000)^{0.989}(86.0)^{-0.649},$$

$$PK=5{,}630 \text{ cubic feet per second.}$$

La Junta

$$PK=7.51(15{,}000)^{0.989}(117.0)^{-0.649},$$

$$PK=4{,}610 \text{ cubic feet per second.}$$

The Fort Lyon headgate peak discharge is then computed as follows:

$$PK=5{,}630-[(93.6-86.0)/(117-86)](5{,}630-4{,}610),$$

$$PK=5{,}380 \text{ cubic feet per second.}$$

The *VOL* is computed using equation 6, but initially we must interpolate for the antecedent streamflow (*ANTFLOW*) conditions at the Catlin Dam and La Junta streamflow-gage locations. The *ANTFLOW* amount of 1,000 ft^3/s at the Portland and Pueblo streamflow gages is between the medium and high antecedent streamflow listed in table 13. The *ANTFLOW* values are computed as follows:

Catlin Dam

$$ANTFLOW=610+[(1{,}000-600)/(1{,}350-600)](1{,}270-610),$$

$$ANTFLOW=960 \text{ cubic feet per second.}$$

La Junta

$$ANTFLOW=530+[(1{,}000-600)/(1{,}350-600)](1{,}120-530),$$

$$ANTFLOW=840 \text{ cubic feet per second.}$$

The *VOL* is computed at Catlin Dam and La Junta as follows:

Catlin Dam

$$VOL=0.802(5{,}000)^{1.027}(86.0)^{-0.0548}(960)^{0.0178},$$

$$VOL=4{,}470 \text{ acre-feet.}$$

La Junta

$$VOL=0.802(5{,}000)^{1.027}(117)^{-0.0548}(840)^{0.0178},$$

$$VOL=4{,}380 \text{ acre-feet.}$$

The Fort Lyons head gage *VOL* can be computed as follows:

$$VOL=4{,}470-[(93.6-86.0)/(117-86)](4{,}470-4{,}380)$$

$$VOL=4{,}450 \text{ acre-feet.}$$

The *TT* is computed initially for the Catlin Dam and La Junta streamflow gage locations, as follows:

Catlin Dam

$$TT=41.14+(0.3050)(86.0)-(6.56)(6.4)$$

$$TT=25.4 \text{ hours.}$$

La Junta

$$TT=41.14+(0.3050)(117.0)-(6.56)(6.0)$$

$$TT=37.5 \text{ hours.}$$

The Fort Lyon headgate traveltime can then be estimated as follows:

$$TT=25.4+[(93.6-86.0)/(117-86)](37.5-25.4)$$

$$TT=28.4 \text{ hours.}$$

SUMMARY

Regulation of the Arkansas River by Pueblo Reservoir has altered significantly the character of historical floods downstream from Pueblo. Water-management decisions during and after periods of floodflow detention in Pueblo Reservoir require a knowledge of unregulated downstream flood characteristics. This report provides a technique for estimating the characteristics of a flood along the Arkansas River between Portland and John Martin Reservoir that eliminates the detention effects of Pueblo Reservoir.

Due to a lack of adequate historical flood data, the study consisted of calibrating a streamflow-routing model for six subreaches and routing typical floods from Portland to seven downstream locations. The typical floods at Portland had peak discharges ranging from 5,000 to 87,000 ft^3/s and represented antecedent streamflow conditions ranging from low-flow to very high-flow conditions. Multiple-regression analysis of data from these synthetic hydrographs was then used to provide a technique by which the peak discharge, volume, and traveltime of floods can be predicted at any location in the reach.

REFERENCES

Boning, C. W., 1974, Generalization of stream travel rates and dispersion characteristics from time-of-travel measurements: U.S. Geological Survey Journal of Research, v. 2, no. 4, p. 495-499.

Jenkins, C. T., and Taylor, O. J., 1972, Stream depletion factors, Arkansas River Valley, southeastern Colorado: U.S. Geological Survey open-file report, 15 p.

Land, L. F., 1977, Streamflow routing with losses to bank storage or wells: Bay St. Louis, Miss., U.S. Geological Survey Computer Contribution; available only from U.S. Department of Commerce, National Technical Information Service, Springfield, VA 22151, as Report PB-271 535, 117 p.

Livingston, R. K., 1973, Transit losses and traveltimes for reservoir releases, upper Arkansas basin, Colorado: Colorado Water Resources Circular 20, 39 p.

_____1978, Transit losses and traveltimes of reservoir releases along the Arkansas River from Pueblo Reservoir to John Martin Reservoir, southeastern Colorado: U.S. Geological Survey, Water-Resources Investigations 78-75, 30 p.

Radosevich, G. E., Hamberg, D. H., and Swick, L. L., eds., 1975, Colorado water laws--A compilation of regulations, compacts, and selected cases: Fort Collins, Colorado State University, Center for Economic Education and Environmental Resources Center, Information Series 17, 2 vols.

U.S. Army, Corps of Engineers, 1977, Arkansas River basin master water control manual, Pueblo Dam and Reservoir: Albuquerque, New Mexico District, 79 p.

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