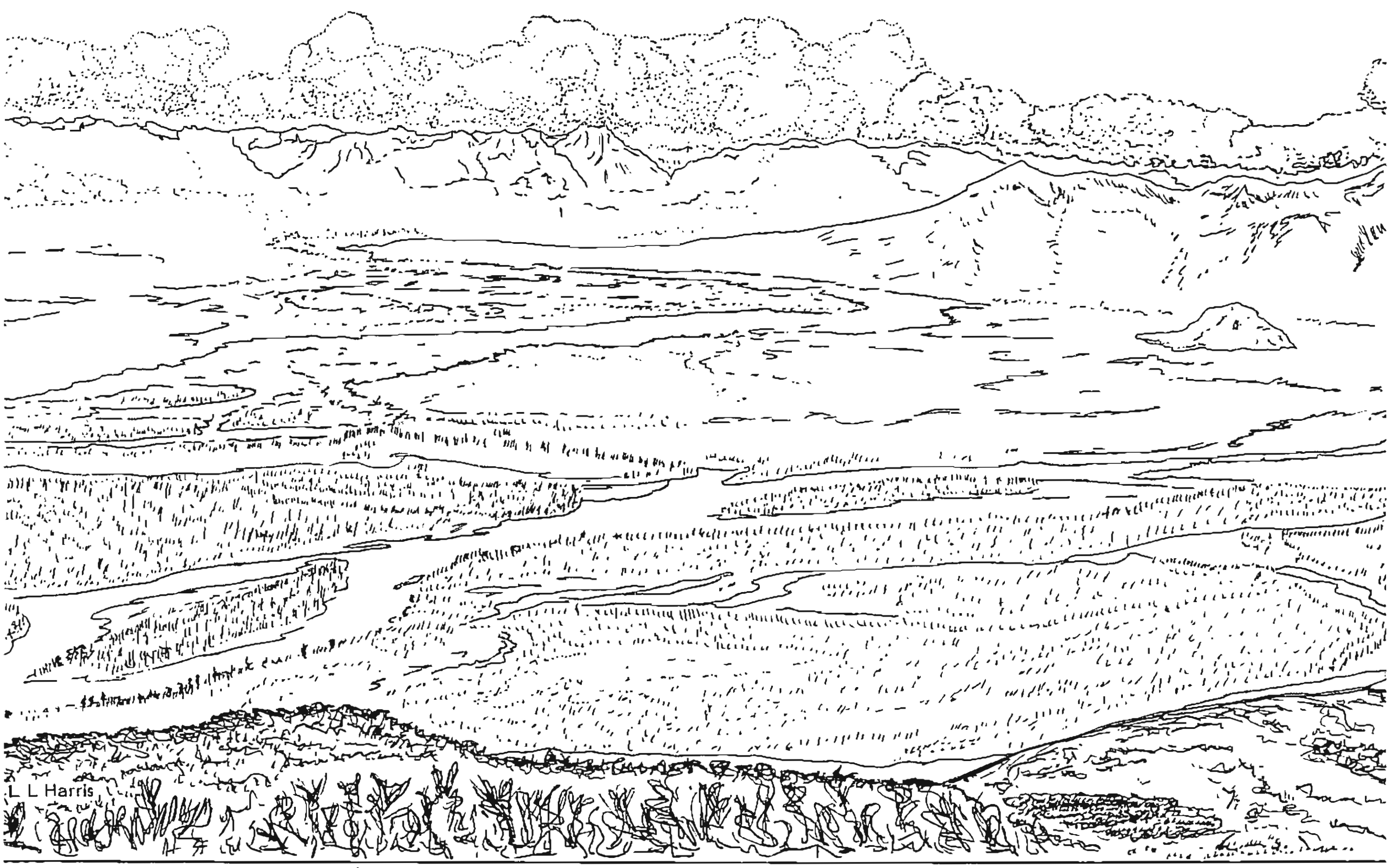
FLOW AND HYDRAULIC CHARACTERISTICS OF THE KNIK-MATANUSKA RIVER ESTUARY, COOK INLET, SOUTHCENTRAL ALASKA

U.S. GEOLOGICAL SURVEY
Water-Resources Investigations Report 89-4064



Prepared in cooperation with the
ALASKA DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES

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Anchorage, Alaska

1989

DEPARTMENT OF THE INTERIOR

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CONVERSION FACTORS

For the use of readers who prefer to use metric (International System) units rather than inch-pound terms used in this report, the following conversion factors may be used:

Multiply inch-pound unit	By	To obtain metric unit
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square foot (ft^2)	0.0929	square meter (m^2)
square mile (mi^2)	2.590	square kilometer (km^2)
acre-foot (acre-ft)	1,233	cubic meter (m^3)
foot per second (ft/s)	0.3048	meter per second (m/s)
mile per hour (mi/h)	1.609	kilometer per hour (km/h)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
degree Fahrenheit (°F)	°C = 5/9 x (°F-32)	degree Celsius (°C)

Other abbreviation in this report is:

μS/cm, microsiemens per centimeter at 25 degrees Celsius

Sea level:

In this report "sea level" refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929)--a geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called Sea Level Datum of 1929.

FLOW AND HYDRAULIC CHARACTERISTICS OF THE KNIK-MATANUSKA RIVER ESTUARY, COOK INLET, SOUTHCENTRAL ALASKA

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ABSTRACT

The lower reaches of the Knik and Matanuska Rivers in southcentral Alaska merge in a complex system of interconnected channels. This reach is subject to unsteady flow conditions that result from a semidiurnal tide wave propagated up the channels through Knik Arm of Cook Inlet. The tidal range in Cook Inlet is among the largest in the world, decreasing from 35 feet at Anchorage (30 miles from the study area) to approximately 10 feet at the study area.

The characterization of flows for the Knik-Matanuska River estuary is further complicated by the historic and potential formation of Lake George behind the Knik Glacier in the upper Knik River basin. Floods resulting from the breakout of this lake occurred annually until 1966 (except 1963), after which the ice dam ceased to form. Peak flows on the Knik River during breakout flood years typically were much greater than peak flows of nonbreakout flood years. The highest recorded peak for a breakout flood was 359,000 cubic feet per second, in July 1958. The highest recorded peak for a nonbreakout year was 60,200 cubic feet per second, in August 1979.

The U.S. Geological Survey's branch-network flow model was used to simulate physical features and flows within the study reach. Because limitations of the model precluded successful simulation of the complex physical and flow characteristics of the Matanuska River, it was eliminated from the analysis and efforts were concentrated on calibration of the model for the three channels of the Knik River. The data needed to calibrate and verify the model were obtained by making a series of detailed streamflow measurements, near the Glenn Highway at the downstream end of the study reach, during three separate tidal cycles. Verification of the model indicated that flows can be simulated with reasonable accuracy over some ranges of discharge, but further calibration will be needed to improve results for all discharges.

At the Glenn Highway crossing, the Knik River is divided into three channels, each conveying a part of the total flow through separate bridges. The model was configured to include a hypothetical single channel downstream from the three Glenn Highway channels, thus allowing use of a constant, total flow into and out of the modeled reach. This total flow was then routed (in the model) through various combinations of bridge-span reductions and (or) closings.

The model was run at flows of 40,000 and 50,000 cubic feet per second through six different bridge configurations. Model results (simulated distribution of flow and circulation patterns in the vicinity of the bridges) were used to assist the design of proposed new bridges. The model indicated that substantial changes in circulation and flow distribution, including some flow reversals, occur in channels upstream from the highway when either one or both of the smaller openings are closed.

INTRODUCTION

Rapid increases in population and development during the past 10 years have produced serious traffic problems along the Glenn Highway between Anchorage and the Palmer-Wasilla area (fig. 1) in Alaska. In an effort to reduce congestion, the Alaska Department of Transportation and Public Facilities (ADOT&PF) plans to widen the Glenn and Parks Highways between Eklutna and Wasilla. Included in this proposed plan is the construction of additional traffic lanes across the "flats" northeast of the village of Eklutna. This will require construction of additional bridges across the Knik and Matanuska Rivers at the present Glenn Highway location just upstream from Knik Arm. In 1984, the U.S. Geological Survey in cooperation with the ADOT&PF began a study of the flow characteristics of the rivers near the proposed bridge crossing sites.

Flow analysis of the two rivers is complicated by several physical factors. The lower reaches of the Knik and Matanuska Rivers merge in a system of interconnected channels that allow the flows to take a variety of routes to the mouth at tidewater. These reaches are subject to unsteady flow conditions which result from a semidiurnal tide wave propagated up the channels through the Knik Arm of Cook Inlet. The tidal range (difference between low and high tide elevations) in Cook Inlet is among the largest in the world, decreasing from 35 ft at Anchorage (30 mi from the study area) to approximately 10 ft at the Glenn Highway crossing. This tidal influence can be detected for several miles upstream from the mouth of these rivers. The analysis of design flows for the Knik River is further complicated by the historic formation of Lake George behind the Knik Glacier in the upper Knik River basin (fig. 1). Floods resulting from the breakout of this lake occurred annually until 1966 (except 1963), after which the ice dam ceased to form. Peak flows on the Knik River during breakout flood years were typically six to seven times higher than peak flows of nonbreakout flood years. The failure of the glacier-dammed lake to form in more than 20 years, does not preclude the possibility of its future formation (Post and Mayo, 1971). Therefore, there is uncertainty in deciding upon which flows should be considered in the design of highway structures in downstream reaches.

A U.S. Geological Survey branch-network flow model (BRANCH) (Schaffranek and others, 1981) was used to study the flow and hydraulic characteristics of the Knik-Matanuska River system. This one-dimensional, implicit, finite-difference model simulates unsteady flow in rivers composed of networks of interconnected channels. The implementation of this model requires the input of channel geometry data at critical locations throughout the reach as well as time series of stage and (or) discharge data (boundary values) at the upstream and downstream ends of the study reach. These data

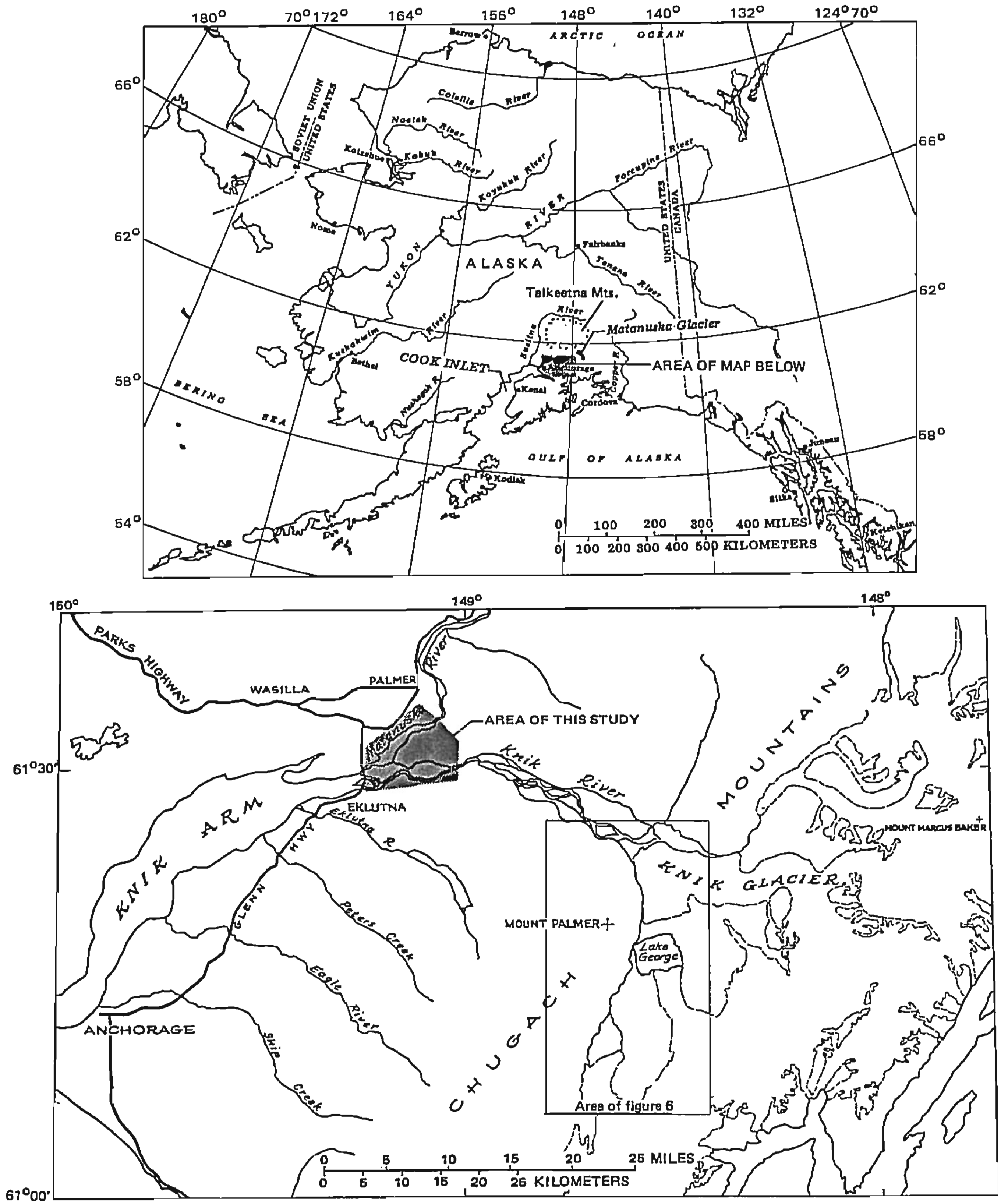

Figure 1.--Location of Knik and Matanuska Rivers study reach and upper Knik River basin.

were collected during the summers of 1984 and 1985, and subsequently were reduced to a format compatible with the modeling requirements. Model output consists of simulated discharges and water-surface elevations at the ends of the reach as well as at intermediate locations where channel geometry has been specified.

Purpose and Scope

This report describes the development and applicability of the U.S. Geological Survey's branch-network flow model for the lower, tide-affected reaches of the Knik and Matanuska Rivers. The model is used to compare simulated and observed discharge of the rivers from short-term records, and to investigate the effects of different channel configurations on flow distribution at the proposed new highway. The report also includes descriptions of the drainage basins and the flow and hydraulic characteristics of the two rivers.

Acknowledgments

The assistance of the Alaska Department of Transportation and Public Facilities for providing survey control throughout the study reach is appreciated. The assistance of Charles Savard of the U.S. Geological Survey for providing a system of programs that were invaluable in computing unsteady discharges resulting from the tides is also appreciated.

PHYSICAL SETTING

The Knik River originates about 40 mi northeast of Anchorage, in glaciers and icefields on the northern slopes of the Chugach Mountains (fig. 1). It flows northwesterly from its headwaters and empties into the Knik Arm of Cook Inlet near Eklutna. The Knik River basin is approximately 1,200 mi^2 in area and varies in topography from the rugged mountainous peaks at its headwaters to the broad, flat, glacially formed valley near tidewater. Altitudes range from 13,176-foot Mount Marcus Baker to sea level at Cook Inlet. Approximately 55 percent of the basin is covered by glaciers, which are responsible for the high concentrations of suspended sediments found in the river during the summer months. The 166-square-mile Knik Glacier in the upper part of the basin is hydrologically significant because of its potential to form an ice-dammed lake behind its terminus, where the glacier impinges against the eastern slopes of Mount Palmer.

The Matanuska River also originates on the northern slopes of the Chugach Mountains, at the terminus of the Matanuska Glacier. It flows westerly through the glacially widened Matanuska Valley and empties into Knik Arm about 1 mi north of the mouth of Knik River. The Matanuska River basin, approximately 2,100 mi^2 in area, is bounded on the north by the Talkeetna Mountains and on the south by the Chugach Mountains.

The Knik and Matanuska Rivers flow into an estuary at the upper end of the Knik Arm of Cook Inlet. The Glenn Highway crosses these rivers in an intertidal marsh at roughly the transition between the tidal-affected rivers and the estuary.

The Knik River study reach is 7.3 mi long; the water-surface elevation falls about 15 to 20 ft over that distance. Within the study reach, the river is characterized by a complex system of interconnected channels that meander across a 2-mile wide flood plain. Vegetation varies from sparse patches of willow and low shrubs on large sand bars near the active channels to dense stands of mature cottonwood and birch trees in more stable locations away from the river.

The Matanuska River reach is about 11 mi long with a fall in water surface elevation of more than 150 ft over that distance. The channel patterns of this reach are even more complex than those of the Knik River.

FLOW CHARACTERISTICS

Gaging Stations Records

The study reach extends downstream from U.S. Geological Survey stream-gaging stations on the Knik River (station No. 15281000) and Matanuska River (station No. 15284000) near Palmer, to where these rivers are crossed by the Glenn Highway (fig. 2). Continuous stage and discharge data have been collected since 1959 at the Knik River station. Data were collected from 1949 to 1973 at the Matanuska River gaging station, and in 1985 the station was reactivated. Because of the complexity and the steepness of the Matanuska River channels, the upper end of the reach was relocated for the purposes of this study to a point about 8 mi downstream from gaging station No. 15284000 (fig. 2).

Average Flows

The Knik River has an average flow of about 7,000 ft^3/s at the upstream end of the study reach (gaging station No. 15281000). This average is based on typical winter daily flows of 700 to 1,000 ft^3/s and typical summer daily flows of 10,000 to 25,000 ft^3/s (fig. 3). The plots in figure 3 are separated into the years when breakout of glacier-dammed Lake George occurred and years when no breakout occurred. The Matanuska River has a smaller flow, averaging about 4,000 ft^3/s at gaging station No. 15284000. Winter low flows of the Matanuska typically range from 400 to 900 ft^3/s while typical summer daily flows range from 6,000 to 12,000 ft^3/s (fig. 4). The annual mean discharges of the Knik and Matanuska Rivers are compared in figure 5.

Peak Flows

Analysis of peak flows for the Knik River is complicated by floods produced by the breakout of ice-dammed Lake George (Hulsing, 1981). The lake used to form in winter as the terminus of Knik Glacier advanced to impinge against the base of Mount Palmer, blocking the lake's natural outlet channel (fig. 6A). As temperatures rose in the spring, runoff from snow and glacier meltwater increased and began to fill the basin behind the ice dam. The level of the lake would typically rise 100 to 150 ft, until water slowly began to force its way through the ice-rock interface. Flow in an initially small channel would begin to erode the ice dam and eventually form a gorge (fig. 6B). Within a few days the gorge would be so enlarged that peak flows

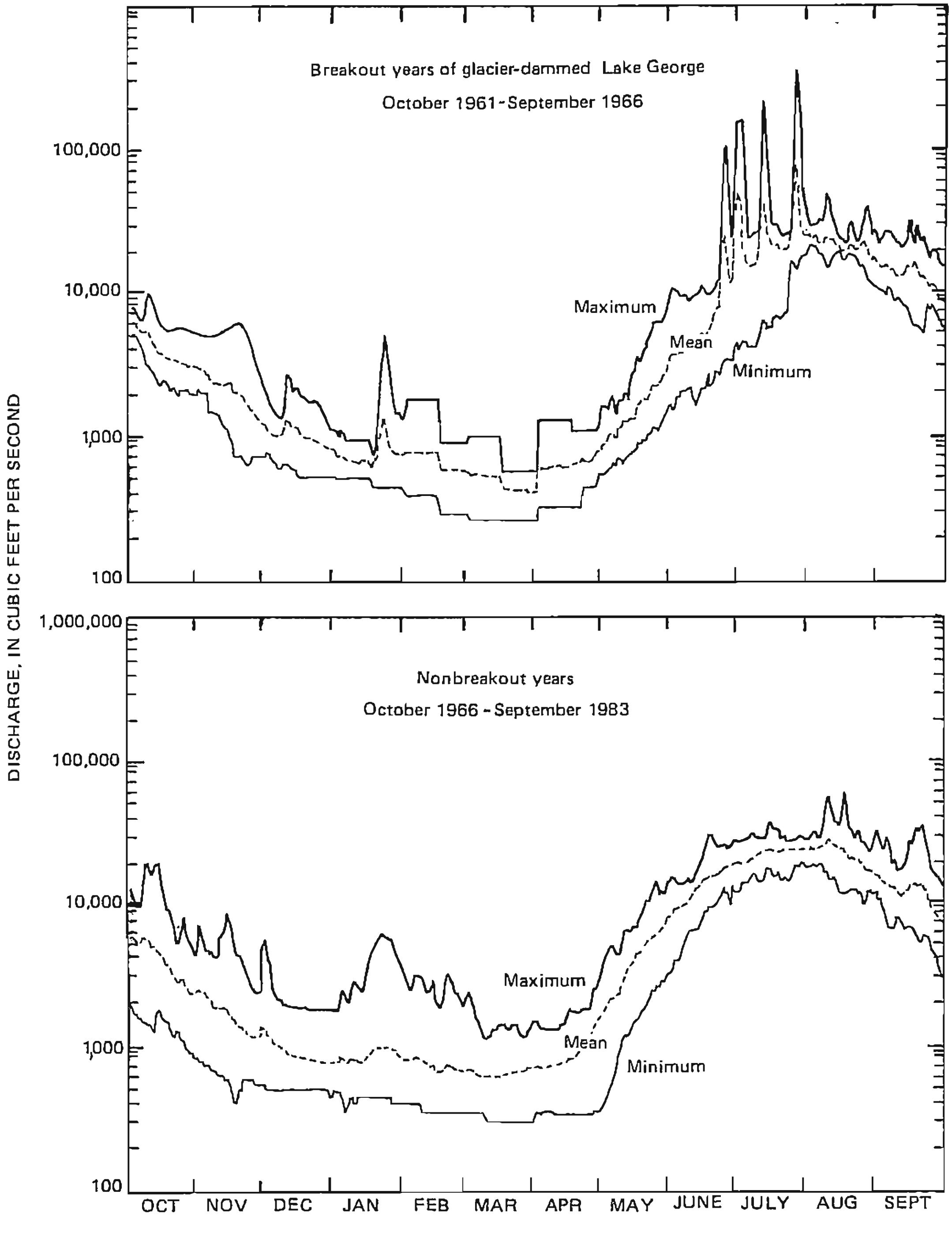

Figure 3.--Range in mean daily discharge at Knik River near Palmer (station No. 15281000) during Lake George breakout and nonbreakout years.

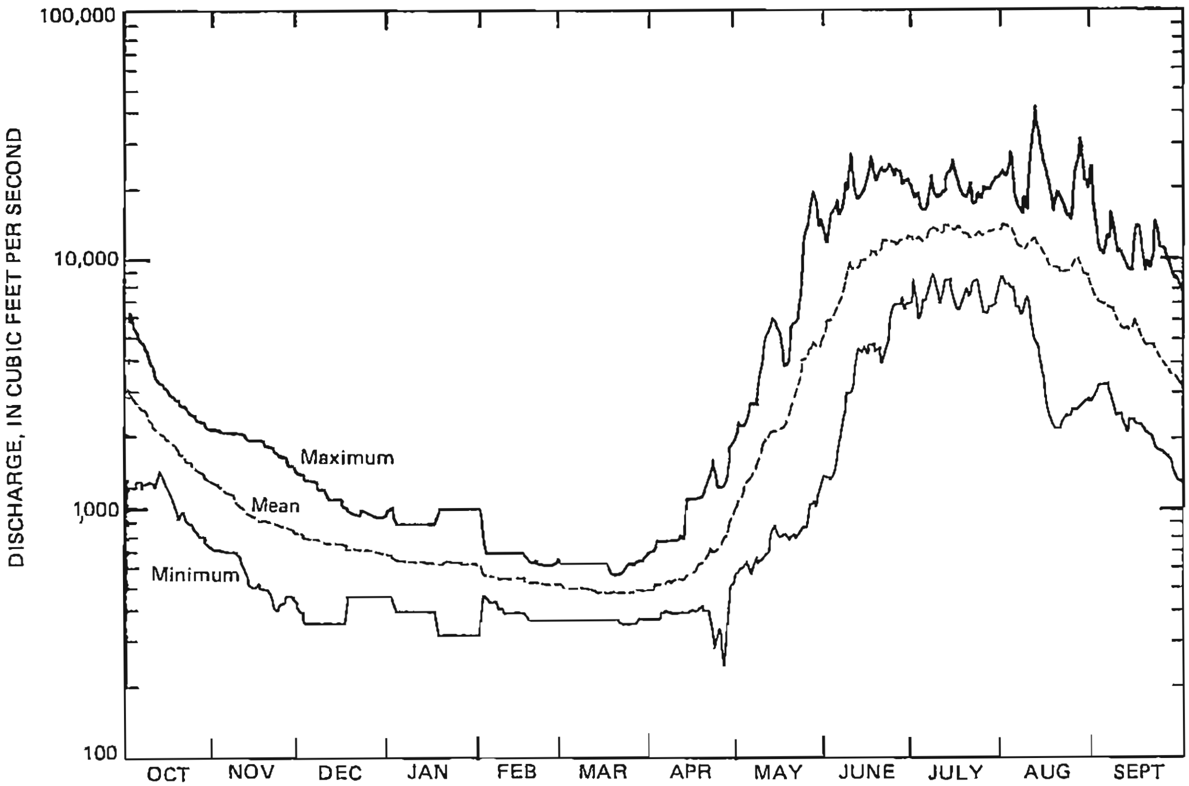

Figure 4.--Range in mean daily discharge at Matanuska River near Palmer (station No. 15284000) from October 1949 through September 1973.

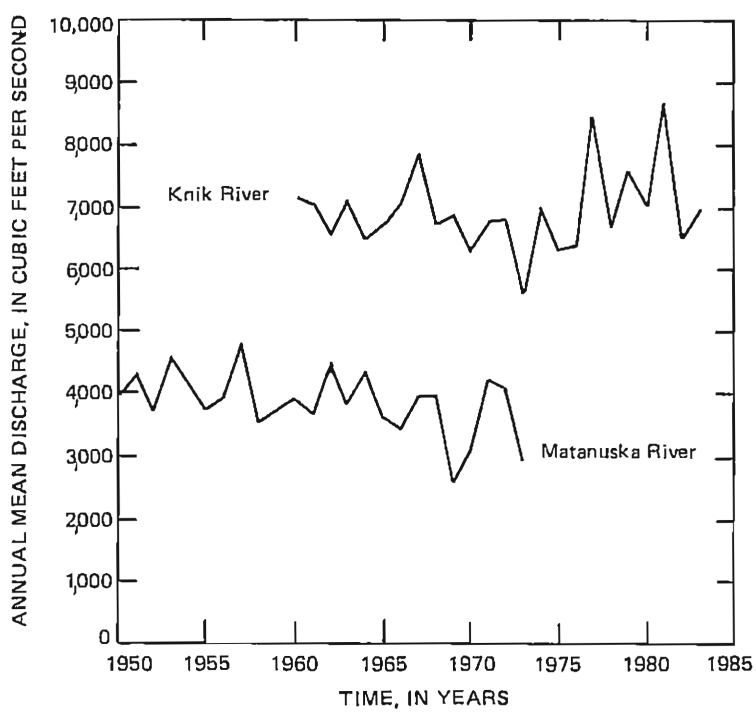

Figure 5.--Annual mean discharges of the Knik and Matanuska Rivers.

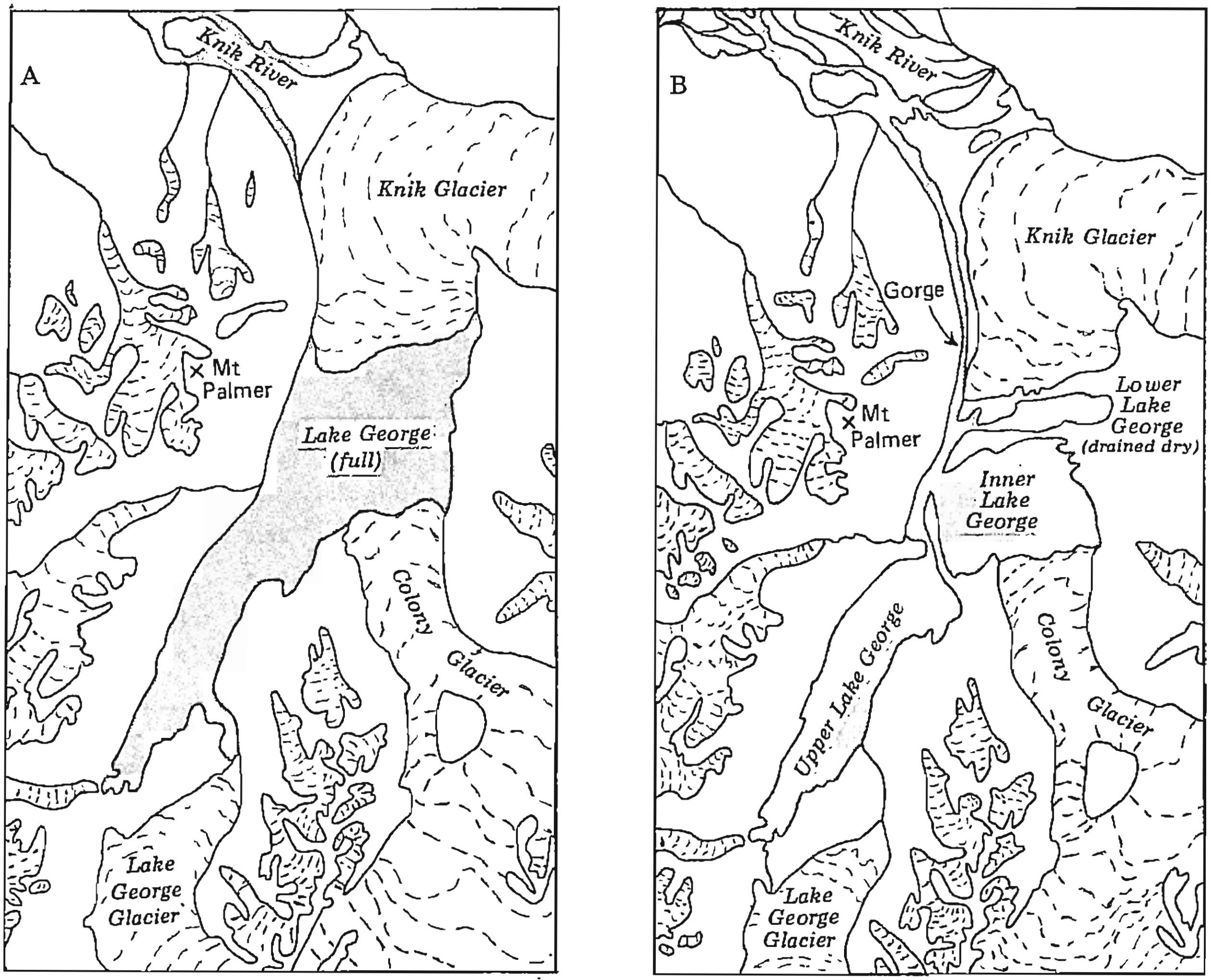

Figure 6.--Lake George with (A) ice formed by Knik Glacier encroaching against Mt. Palmer and (B) gorge that releases water into the Knik River. (See figure 1 for location.)

greater than 300,000 ft^3/s were not uncommon. From beginning to end the drainage of the lake took from 10 to 15 days, during which the broad alluvial valley that includes the study reach would become inundated.

Historic accounts indicate that the breakout floods occurred every 15 to 20 years until 1914, when they became almost annual events. In 1958, the U.S. Geological Survey, in cooperation with the Alaska Railroad Commission, began a study of the lake breakout and resulting floods (table 1). Discussions and data on cross-section geometry, water stage and velocity, suspended sediment, and scour at Knik River bridges during the 1965 and 1966 breakout floods are included in a report by Norman (1975, p. 39-78). The dam failed to form in 1963 and has not formed since 1967. An investigation by U.S. Geological Survey glaciologists L. R. Mayo and D. C. Trabant (written commun., 1984) suggests that formation of the ice-dam is episodic in nature and can be expected to re-form periodically in the future.

The peak flow data for Knik River are separated into breakout and nonbreakout annual peaks and the exceedance probabilities computed for each data set. The nonbreakout analysis is based on 20 years (1963, and 1967-85) of annual peaks and the breakout analysis on 18 years (1948-66) excluding 1963, when a breakout did not occur.

Table 1.--Summary of data for Lake George breakout and resulting floods

[ft^3/s, cubic feet per second]

Year	Date of breakout	Maximum lake level: Date measured	Maximum lake level: Feet above sea level	Fall in lake level (feet)	Water released (acre-feet)	Flood crest of Knik River near Palmer (station No. 15281000): Date	River stage (feet)	Discharge (ft^3/s)
1958	--	7/13	345.5	160	1,800,000	7/18	25.3	359,000
1959	6/26	6/26	300.2	115	900,000	7/12	20.8	223,000
1960	7/12	7/14	319.6	135	1,200,000	7/17	24.4	328,000
1961	7/20	7/23	326.7	142	1,400,000	7/26	24.3	355,000
1962	6/26	6/27	281.1	96	600,000	6/29	18.5	165,000
1963	Gorge remained open. No breakout--lake did not form							
1964	6/26	6/28	283.0	98	700,000	7/1	20.0	216,000
1965	7/8	7/9	290.0	105	900,000	7/11	21.4	236,000
1966	6/22	6/22	286.5	--	560,000	6/24	17.9	144,000
1967-88	Gorge has remained open. No breakout--lake has not formed							

Peak flows on the Knik River for breakout years are typically six to seven times higher than those of nonbreakout years. The maximum discharge observed since 1948 was 359,000 ft^3/s on July 18, 1958 (table 1), during a breakout flood prior to installation of the gaging station. The maximum discharge recorded during a non-breakout year was 60,200 ft^3/s, on August 17, 1979.

Examples of typical breakout and nonbreakout annual hydrographs for the Knik River are shown in figure 7. The 1961 hydrograph includes a breakout flood that occurred in late July. This particular flood, the largest recorded since the gaging station was installed in 1959, had a peak discharge of 355,000 ft^3/s. The 1984 hydrograph (fig. 7) illustrates a typical nonbreakout year and is characteristic of most glacier-fed streams in this area of Alaska. Between November and April the flow is usually less than 2,000 ft^3/s and the channels are ice covered. Warming weather in May produces steadily increasing flows due to runoff from melting snow and glacier ice. This process continues into July when rainstorms become more frequent and storm runoff combines with the discharge from meltwater runoff. The 1984 hydrograph illustrates a continual increase in discharge between May and early July. Relatively high flows continued through late August with variations that were dependent on precipitation and air temperature. The 1961 hydrograph shows a similar pattern from early May to mid-June. However, as the glacier advanced to form a dam at the outlet of Lake George, the runoff from the upper, glacier-covered basin was impounded, causing reduced discharges in the Knik River for about a month. When the ice dam failed and released the stored water over a period of a few days, the resulting flood inundated the entire flood plain downstream from Knik Glacier.

Plots of maximum daily discharge of Knik River for breakout and nonbreakout years have been combined in figure 8. This illustration shows the relation of the magnitude of the breakout flood to the date of the

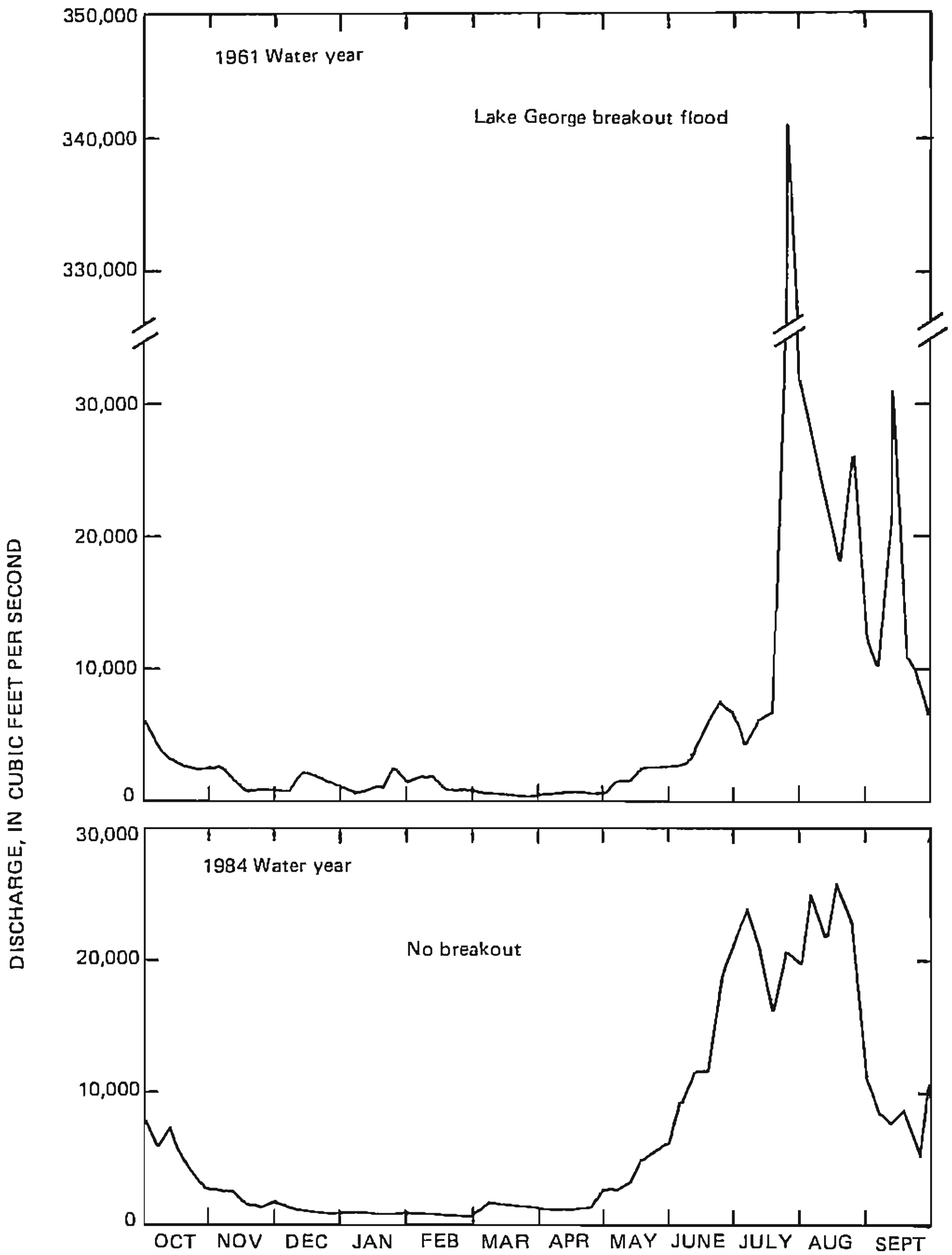

Figure 7.--Typical daily mean discharges of the Knik River near Palmer (station No. 15281000) during breakout (1961) and nonbreakout (1984) water years.

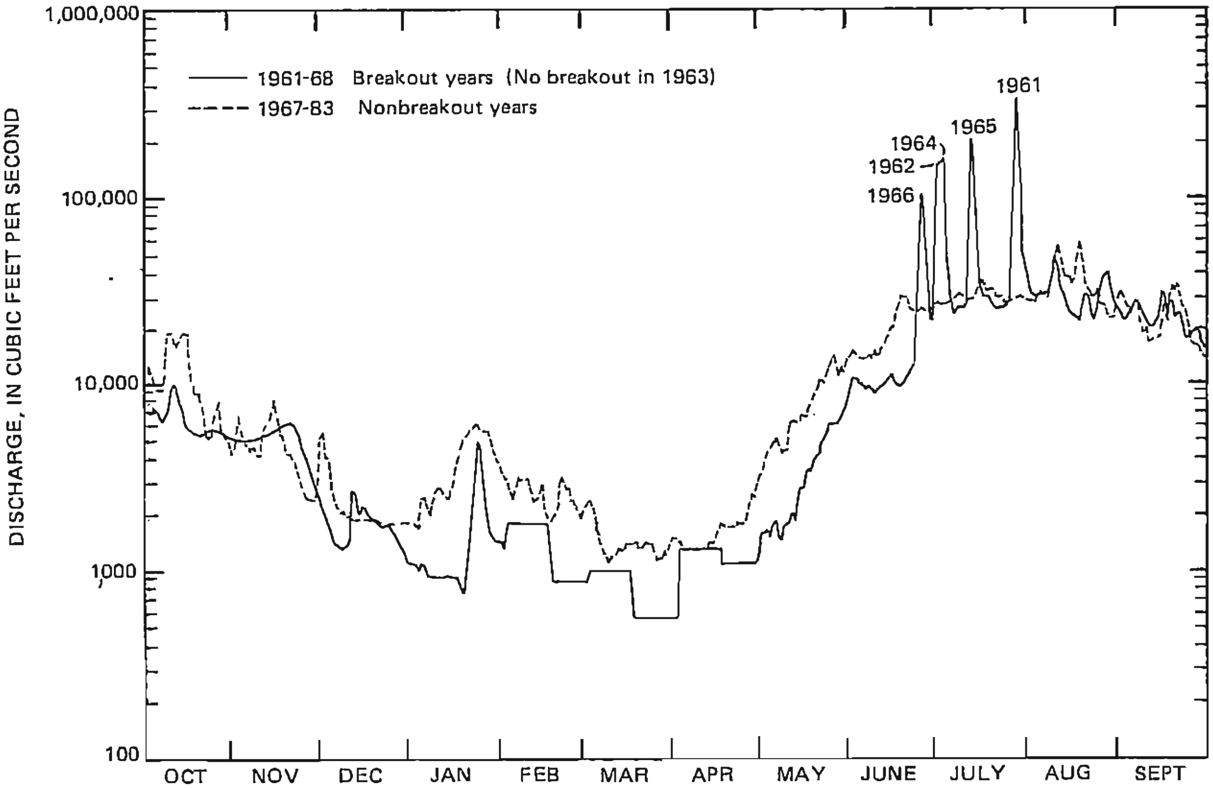

Figure 8.--Maximum daily mean discharge values at Knik River near Palmer (station No. 15281000) for breakout and nonbreakout years.

breakout. The magnitude of the flood is a function of the height of the ice dam, the capacity of the reservoir (lake) formed by the dam, and thus the volume of water impounded. If runoff rates to the lake are assumed to be approximately the same each year, then larger lakes take longer to fill than smaller lakes. In general, then, the greatest magnitude floods are those released from the highest ice dams, and occur later in the summer than smaller floods from lower ice dams.

The analysis of peak flows on the Matanuska River is based on 27 years of record (1949-73 and 1985-86). The maximum discharge recorded at the Matanuska River gaging station was 82,100 ft^3/s on August 10, 1971. However, this peak was excluded from the frequency analysis because it resulted from the breaching of a small landslide-dammed lake in the upper part of the basin. The discharge hydrograph for that day indicates that the peak flow excluding the lake breakout was about 47,500 ft^3/s; this value was used in the analysis.

Selected recurrence intervals, exceedance probabilities, and associated discharges for the Knik and Matanuska Rivers are listed in table 2 and shown in figure 9. The flood-frequency analyses are based on the log-Pearson Type III distribution recommended by the U.S. Water Resources Council (1981). On the basis of these analyses, the discharges with 100-year recurrence intervals for the Knik with breakout, Knik with nonbreakout, and the Matanuska Rivers are 446,000 ft^3/s, 68,200 ft^3/s, and 50,800 ft^3/s, respectively.

Table 2.--Peak flow frequency analysis summary for the Knik (breakout and nonbreakout years) and Matanuska Rivers

Recurrence interval (years)	Annual exceedance probability	Estimated discharge (cubic feet per second)		
		Knik River (nonbreakout)	Knik River (breakout)	Matanuska River
1.25	0.800	28,500	187,000	19,700
2.00	.500	34,100	233,000	24,500
5.00	.200	42,300	293,000	31,300
10.00	.100	48,100	331,000	35,800
25.00	.040	55,800	378,000	41,700
50.00	.020	61,900	412,000	46,200
100.00	.010	68,200	446,000	50,800

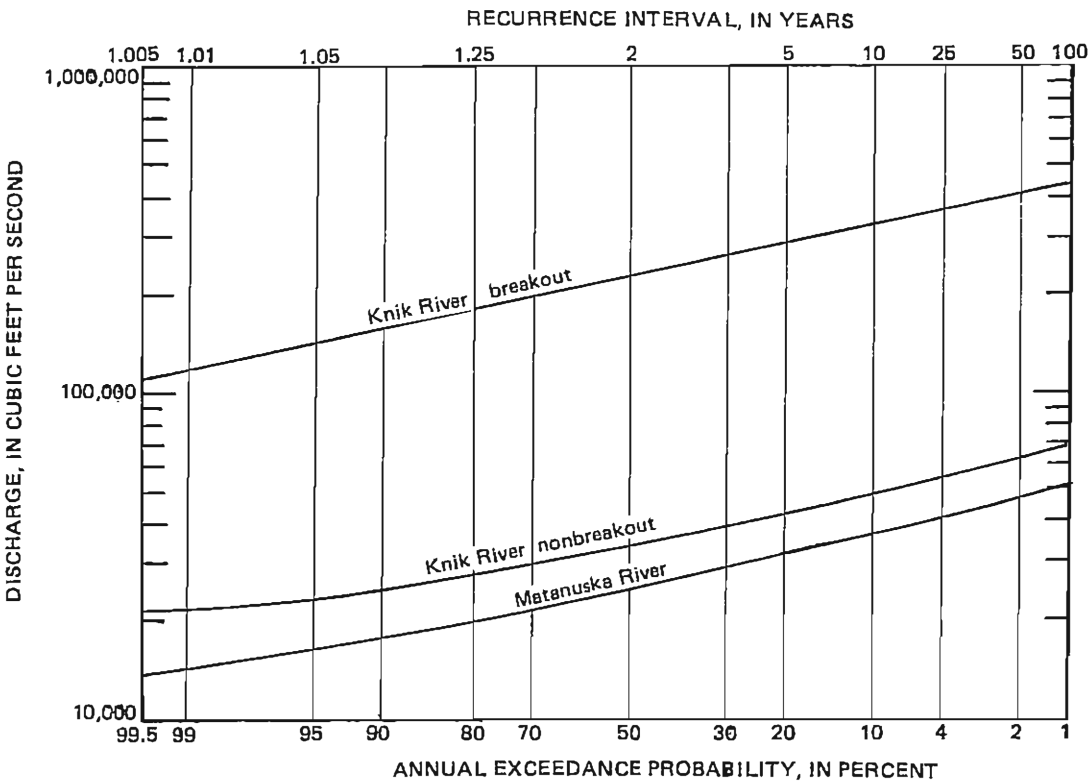

Figure 9.--Recurrence intervals and exceedance probabilities of peak discharge for the Knik and Matanuska Rivers.

TIDAL CHARACTERISTICS

The tide cycle in the Knik Arm of Cook Inlet has a period of about 12.5 hours. Tide elevations at Anchorage can fluctuate almost 40 ft between low and high tides. The relation of tide elevations at Anchorage to water stage at Knik River near Eklutna (station No. 15281110) is shown in figure 10. The large tide fluctuation at Anchorage is due primarily to the constricting shape of Cook Inlet and Knik Arm in that vicinity. The fluctuations in water-surface elevations in Knik Arm produce a tide wave that is propagated up the Knik and Matanuska Rivers. From figure 10 it is evident that the wave period (time between successive high tides) is unchanged between Anchorage and the gage at the Knik River. There is, however, a phase difference indicating a time of travel for the wave of about 2.25 hours. Because the distance between the two gages is approximately 30 mi, the mean wave celerity (velocity) is about 13.3 mi/h, or 19.6 ft/s.

As the tide wave moves up Knik Arm it is opposed by the current in the Knik and Matanuska Rivers. At Anchorage, the wave is unaffected by such currents and consequently is symmetrical. At the Knik River gage, however, the waveform has been modified by the opposing current in the river. As a result, the rising limb is much steeper than the falling limb. The rising limb, or flood tide, begins as a sharp rise in stage which reaches a peak after 1 to 2 hours and begins to fall during the ebb tide just as quickly. As the tide wave reaches its termination during the ebb stage, the slope of the hydrograph decreases gradually until a steady flow condition is again reached.

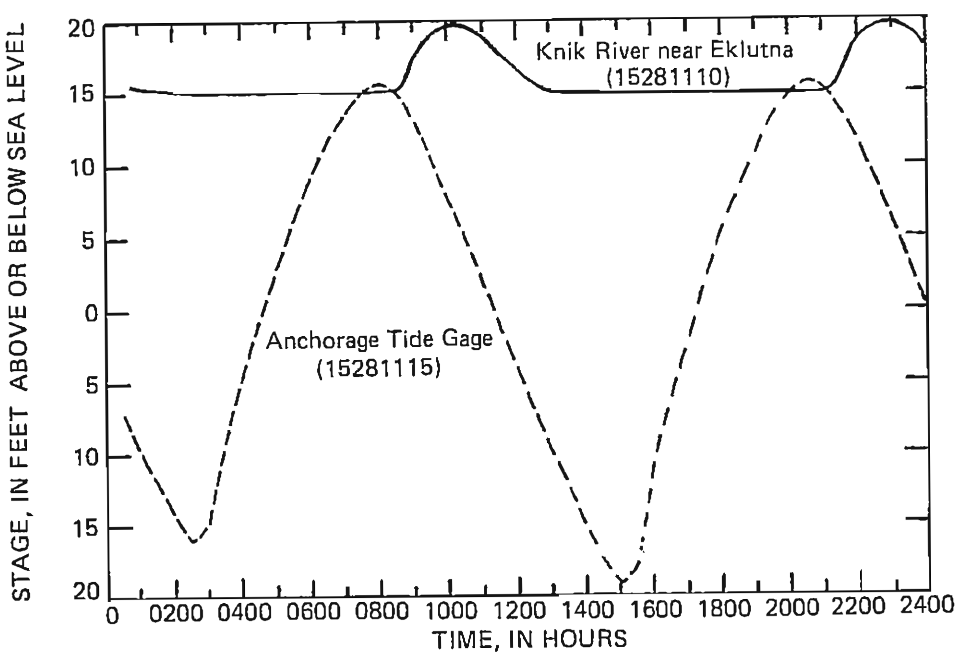

Figure 10.--Relation of river stage at Knik River near Eklutna to tide elevations at Anchorage on August 28, 1984.

Temporary stage gages, installed at two intermediate locations on the Knik River (station Nos. 15281003 and 15281005), were used to determine the distance that the tidal wave traveled up the channel. The stage hydrographs for these two stations and for the lower Knik River site (station No. 15281110) show that during the two high tides on September 15, 1985 the tide wave traveled as far as station No. 15281005 but was dissipated between that location and station No. 15281003 (fig. 11). This was due to the steepness of the channels between the two stations. Comparison of the waveforms for the two lower sites shows the attenuation of the wave as it progresses up the channel. This is especially apparent on the ebb cycle, or falling limb of the hydrograph, which is less steep at the upstream site than at the site closer to tidewater.

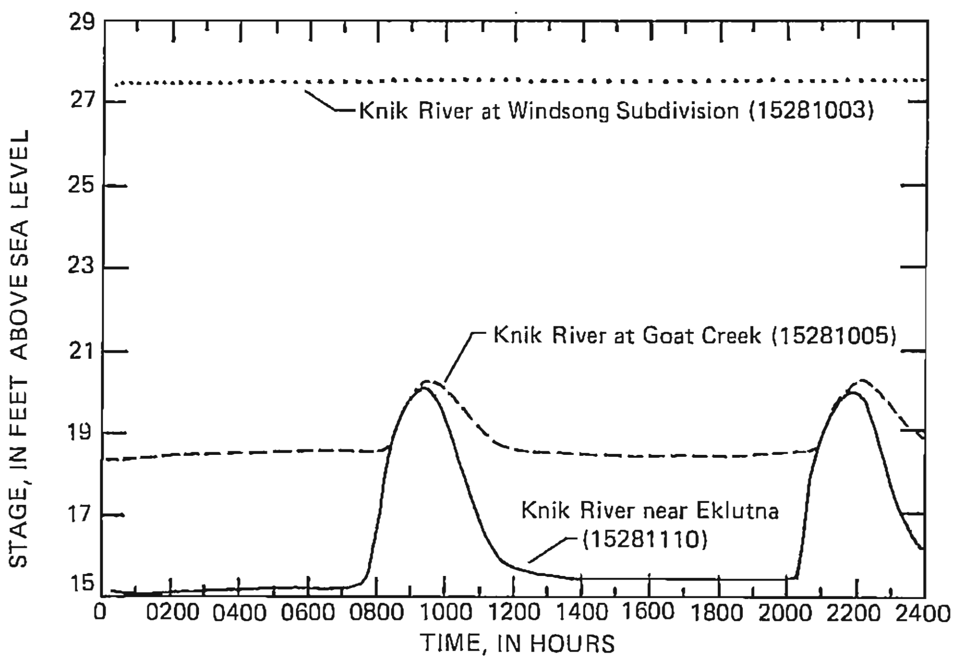

Figure 11.--River stages at three sites on the Knik River on September 15, 1985.

Another feature of the tide/river interaction is that of producing alternating periods of steady and unsteady flow. Figure 12 is a stage hydrograph of Knik River for August 24-28, 1984. The stage hydrograph is composed of relatively steady flow periods interposed with periods of tidal flood and ebb occurring every 12.5 hours and lasting from 4 to 5 hours, depending upon the magnitude of the tide.

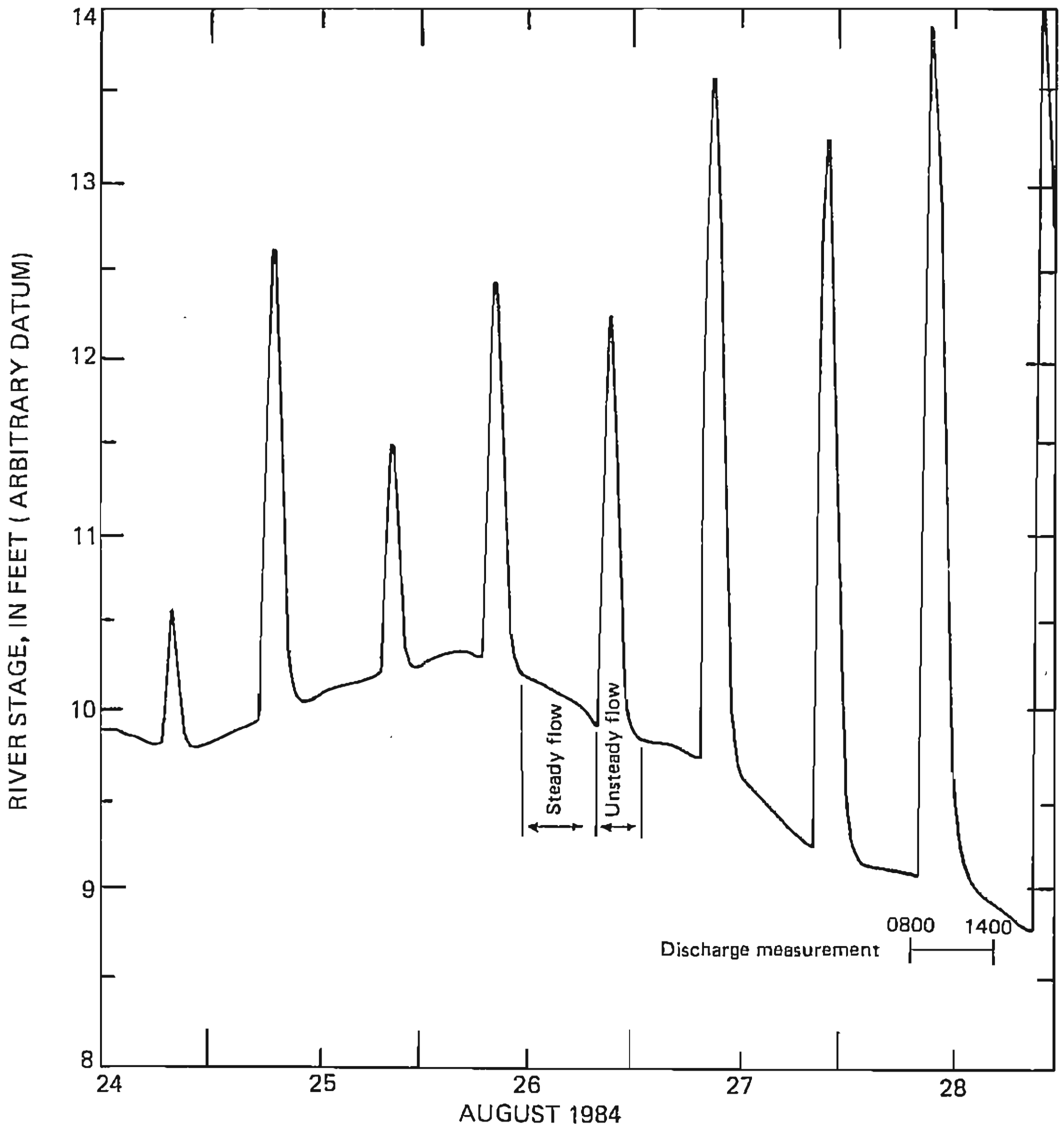

Figure 12.--River stage of Knik River near Eklutna (15281110) showing steady and unsteady flow periods due to tide influence, August 24-28, 1984.

DATA COLLECTION AND DISCUSSION

Instrumentation

In order to supplement data from the two long-term gaging stations, bubble-gage manometers and strip-chart recorders were installed in summer 1984 at the two primary channels of the Knik and Matanuska Rivers (station Nos. 15281110 and 15281140, respectively). In summer 1985, portable pressure transducers were installed on the two secondary channels of the Knik River (station Nos. 15281120 and 15281130), as well as at other miscellaneous sites within the study reach. Campbell Scientific CR21 microloggers[1] were installed in concert with the strip-chart recorders at the two primary channel sites to facilitate the conversion of the data to digital format. These water-level sensors and recorders provided stage data which can be used alone or in conjunction with discharge data as input to the branch-network model.

1 Use of trade names in this report is for identification purposes only and does not constitute endorsement by the U.S. Geological Survey.

Discharge Measurements and Computations

In addition to needing data for initial model implementation, sets of measured discharge data are needed for calibration purposes. These data consisted of time-series of discharge values obtained simultaneously at the four gage sites at the downstream end of the study reach (station Nos. 15281110, 15281120, 15281130, and 15281140). Measurements of discharge were made on three dates -- August 28, 1984, and June 20 and July 3 in 1985 -- when tides were maximum and discharge covered a wide range.

Standard procedures for making discharge measurements cannot be employed during tide cycles due to the unsteady flow conditions. (Figure 12 shows the rapidly changing stage during a discharge measurement on August 28, 1984.) In order to measure discharge during such periods of unsteady flow, the channels were subdivided into approximately ten subsections based upon assumed centroids of flow. Buoys were moored at the center of each subsection for stationing in the wide, primary channels, and taglines were used on the secondary channels. Velocity measurements were taken either at 0.2 and 0.8 depths or at 0.6 depth, using a standard Price current meter and a sounding weight suspended from a boat. Precise notation of the time was made during each velocity measurement.

The discharge measurement on August 28 was begun prior to the beginning of the tide cycle to obtain one complete set of velocity measurements at each subsection during steady flow conditions. Velocity and depth observations were then continued repeatedly from one side of the channel to the other during the entire tide cycle. This procedure was followed in four downstream channels simultaneously. One boat each was assigned to the primary Knik and Matanuska channels while observers in a third boat made measurements on both secondary channels of the Knik River. Due to low velocities, flow reversals occurred in the two secondary channels near the peak of the tide cycle. Flow direction was carefully noted at these times.

Depth-time and velocity-time plots, examples of which are shown in figures 13 and 14, were constructed for each subsection within each channel. The variation in the difference between stage and water depth shown on figure 13 is due to the difficulty of positioning the boat at the same point for successive depth readings. Areas were computed based on the depth-time plots and the subsection widths. Rapidly changing channel widths were accounted for when computing the areas of the left and right subsections. Using linear interpolation, the discharge for each subsection was computed at 1-minute intervals for the complete time period of the measurement (fig. 15). The discharges for all subsections within a particular channel were then summed to obtain the total discharge for the entire cross section at 1-minute intervals throughout the tide cycle (fig. 16). Discharge values at 15-minute intervals were required for subsequent use in a branch-network model. This was the initial time step to be used in the model.

Velocity Distribution

While making discharge measurements at the tide-influenced sections, it was noted that velocities at the 0.8 depth were sometimes equal to or greater than the corresponding velocities at the 0.2 depth. Because in a typical velocity distribution this condition is reversed, further

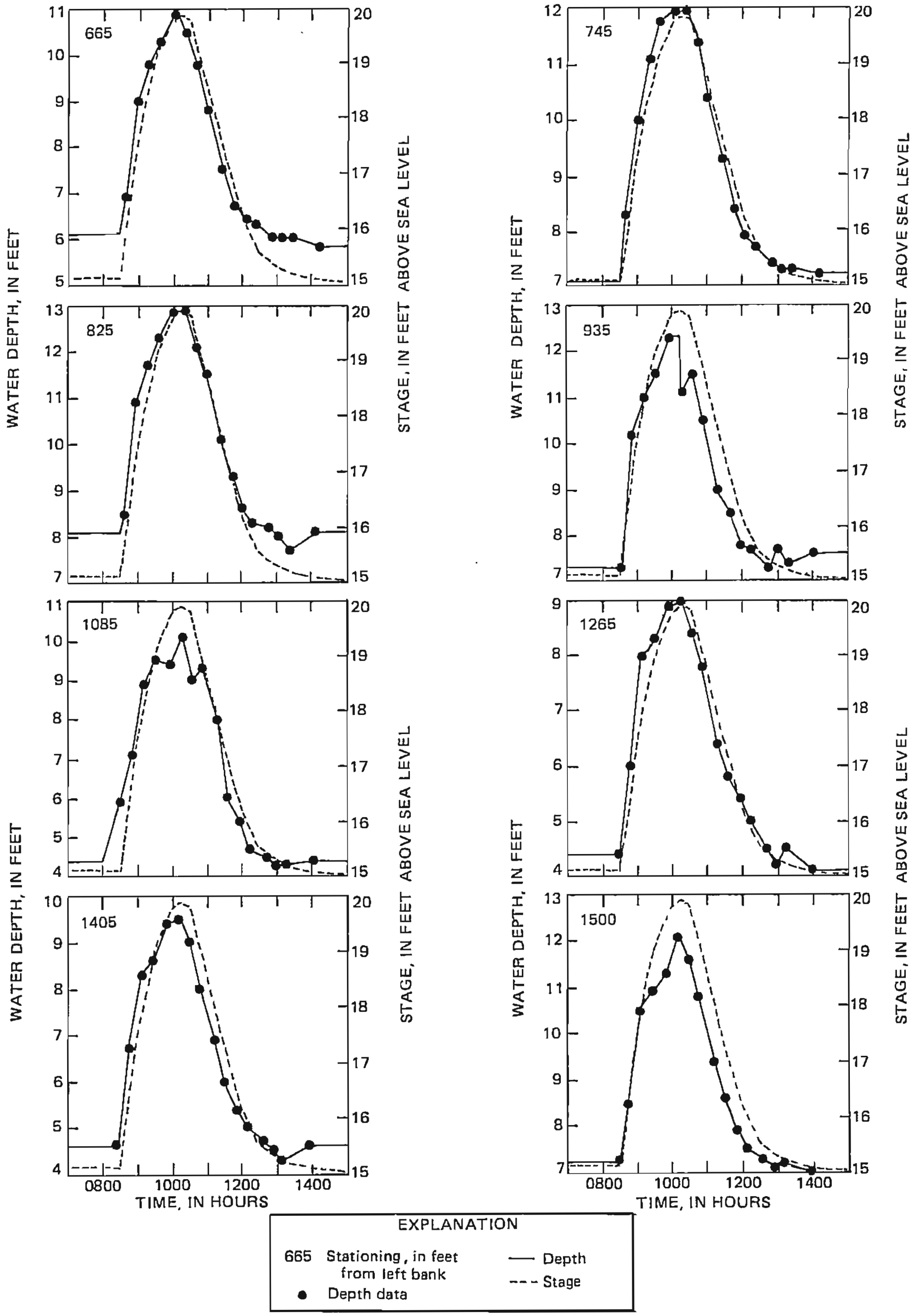

Figure 13.--Depth and stage at selected distances from left bank on cross section 32 (Knik River near Eklutna, station No. 15281110) during a tide cycle on August 28, 1984.

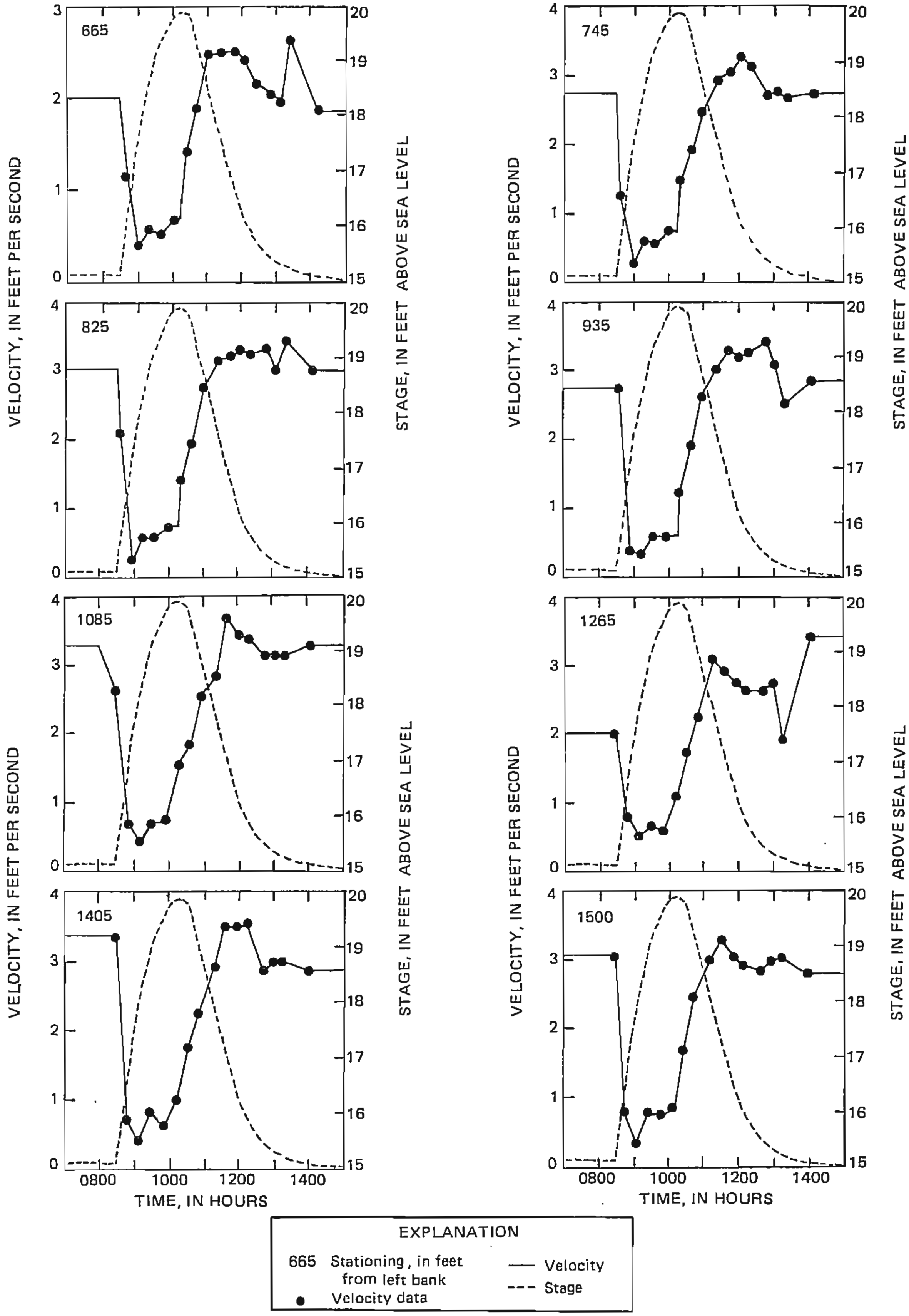

Figure 14.--Velocity and stage at selected distances from left bank on cross section 32 (Knik River near Eklutna, station No. 15281110) during a tide cycle on August 28, 1984.

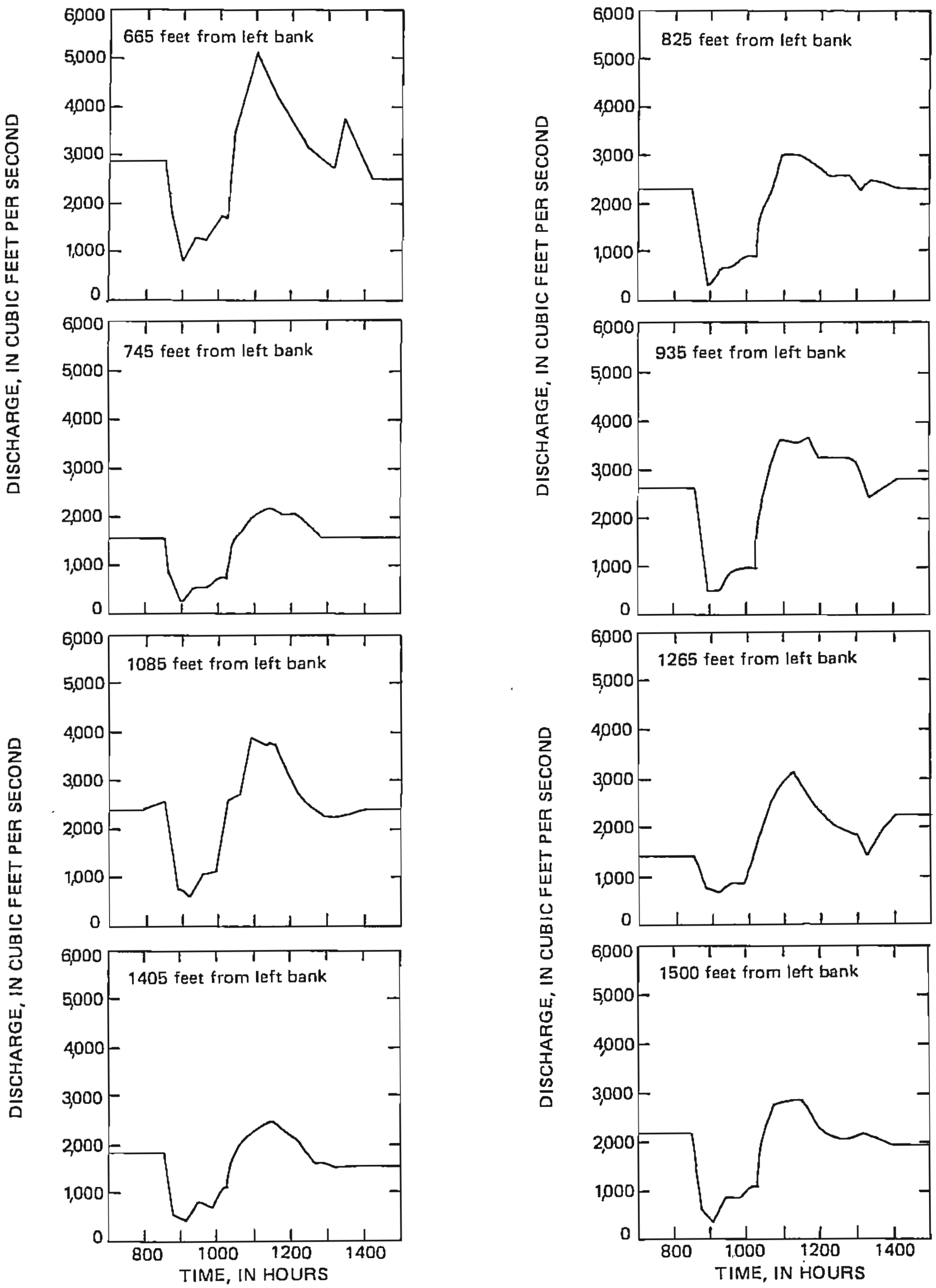

Figure 15.--Discharge at selected locations of the Knik River near Eklutna (station No. 15281110) during a tide cycle on August 28, 1984.

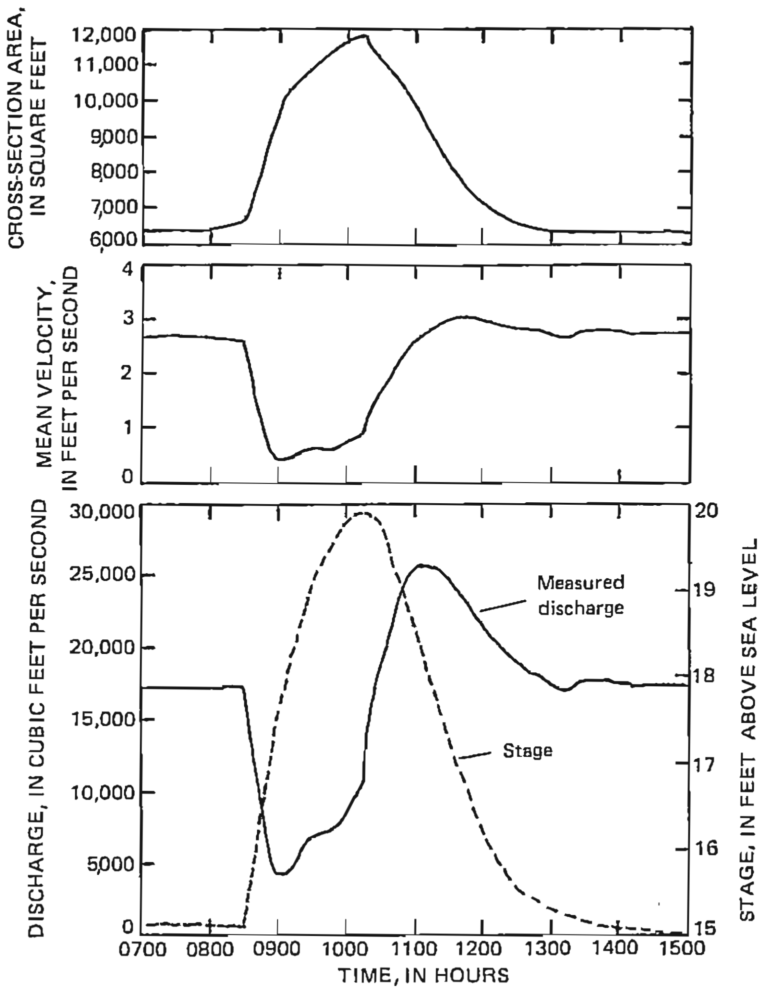

Figure 16.--Total area, mean velocity, stage, and discharge for the Knik River near Eklutna (station No. 15281110) during a tide cycle on August 28, 1984.

investigation was made. Flow reversals had been observed in the smaller secondary channels, and the possibility of a salt-water intrusion was considered. It is common in estuaries to have stratified flow layers that move independently of one another. This is due to saltwater being overridden by less dense freshwater.

To learn more about these conditions during a tidal cycle, velocity profiles were obtained using a Marsh-McBirney electromagnetic current meter with geomagnetic compass. This meter senses velocity vectors in a two-dimensional plane. The amount of suspended sediment transported in the Knik and Matanuska Rivers makes it impossible to see more than a few centimeters below the water surface, but the Marsh-McBirney meter enables the detection of flow direction at any depth. Flow directions and magnitudes were obtained on the main stem of the Knik River at the downstream end of the study reach. They were collected during a period of low discharge in conjunction with a relatively high tide, a condition most likely to produce saltwater intrusion.

Vertical velocity profiles were made at 1-foot increments from the water surface to depths of 9 or 10 ft over an entire tidal cycle at a location in the cross section where atypical 0.2- and 0.8-depth readings had been noted. Velocity readings were limited to a maximum depth of 10 ft and to points more than 2 ft above the streambed by the configuration of the current meter and sounding weight. The depth and velocity data are given in table 3 and selected plots of the data are shown in figure 17.

Specific Conductance

Specific conductance and temperature probes were attached to the flow meter to obtain data that would be useful in identifying saltwater intrusion. These specific conductance and temperature data are also given in table 3. The values under the "Bearing" heading in table 3 refer to the direction of flow based on magnetic north. These readings were made to determine the occurrence of flow reversals at depths beyond visual range.

Judging from the relatively consistent readings of specific conductance, water temperature, and bearing (flow direction) throughout the tide cycle, no stratification due to saltwater intrusion occurred in the Knik River at the measured location. Further investigations of other channels would be necessary to determine whether or not intrusions were occurring in them under similar conditions. Other than local perturbations, the velocity profile data indicate no significant departures from what would be considered a typical distribution.

Cross Sections

Channel cross-section data for the Knik and Matanuska Rivers were obtained by standard field surveying methods for portions of the cross sections above the water surface. In-channel depths were obtained using a recording fathometer in a moving boat. The distances from one edge of the water to another were also determined and the data used to determine the cross-section areas and widths.

The cross-section and stage data needed to develop and apply a branch-network model must be based on the same datum. As part of the study, the ADOT&PF provided vertical and horizontal control to key locations throughout the study reach. The vertical control was based on sea level datum. Benchmarks which served as the basis for the vertical control were resurveyed after the Alaska Earthquake of 1964. These surveys indicated that substantial subsidence (2 to 3 ft) occurred in the vicinity of the study reach. All surveys made in conjunction with this study were based on the most recent benchmark elevations.

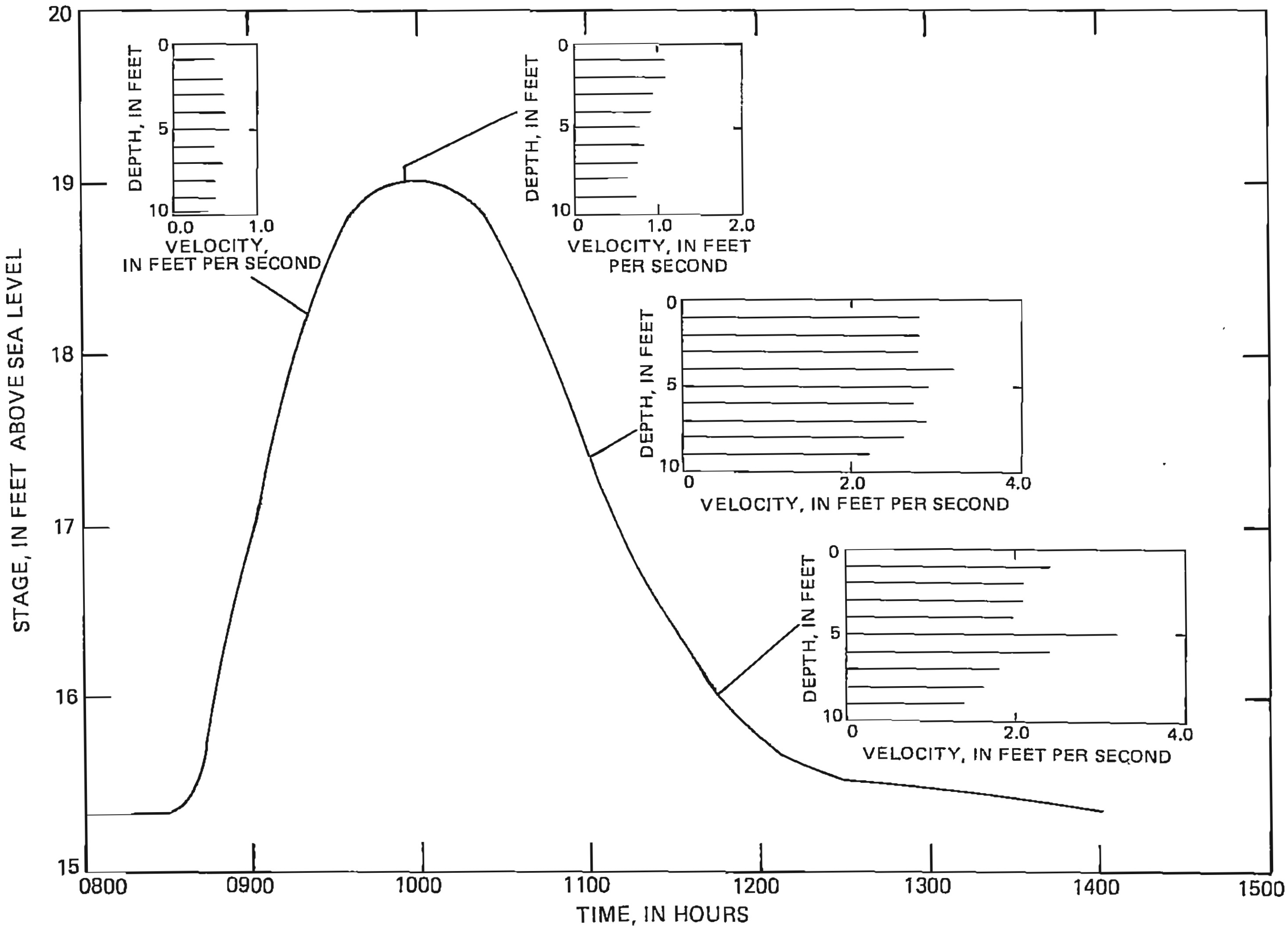

Figure 17.--River stage and selected velocity profiles at a distance 800 feet from left bank on cross section 32 (Knik River near Eklutna, station No. 15281110) during a tide cycle on September 16, 1985.

Table 3.--Velocity, specific conductance, and temperature profiles for cross section 32 (800 feet from left bank) Knik River near Eklutna (channel 1), during a tide cycle on September 16, 1985

[ft, feet; ft/s, feet per second; µS/cm, microsiemens per centimeter at 25 °C; °C, degrees Celsius]

Time	Depth (ft)	Velocity (ft/s)	Spec. cond. (µS/cm)	Temp (°C)	Bearing (degrees)
0917	1.0	0.50	81	2.1	270
0919	2.0	0.60	81	1.8	255
0921	3.0	0.60	82	1.9	265
0922	4.0	0.60	82	1.9	270
0923	5.0	0.65	82	1.9	265
0924	6.0	0.50	82	1.9	260
0924	7.0	0.60	81	1.9	245
0925	8.0	0.55	82	1.8	245
0926	9.0	0.50	82	1.8	270
0927	10.0	0.40	82	1.8	270
0930	1.0	0.70	51	1.8	220
0930	2.0	0.65	57	1.8	230
0931	3.0	0.80	81	1.8	230
0932	4.0	0.58	82	1.8	230
0932	5.0	0.65	82	1.8	230
0933	6.0	0.60	82	1.8	230
0933	7.0	0.60	82	1.8	230
0934	8.0	0.55	82	1.8	230
0934	9.0	0.65	82	1.8	245
0935	10.0	0.30	82	1.8	270
0936	1.0	0.65	56	1.8	210
0936	2.0	0.75	72	1.8	220
0937	3.0	0.85	81	1.8	220
0938	4.0	0.60	81	1.8	220
0938	5.0	0.75	82	1.8	220
0939	6.0	0.75	82	1.8	220
0939	7.0	0.75	82	1.8	220
0940	8.0	0.80	82	1.8	220
0940	9.0	0.65	82	1.7	220
0941	10.0	0.55	82	1.7	265
0942	1.0	0.80	82	1.7	220
0943	2.0	0.85	82	1.7	220
0943	3.0	0.90	82	1.8	220
0944	4.0	0.80	82	1.7	220
0944	5.0	0.90	83	1.7	220
0945	6.0	0.90	83	1.7	220
0946	7.0	0.90	82	1.7	220
0946	8.0	0.85	82	1.7	220
0946	9.0	0.70	82	1.7	220
0947	10.0	0.75	83	1.8	260
0948	1.0	1.10	87	1.8	220
0948	2.0	1.10	84	1.7	230
0949	3.0	0.90	84	1.8	235
0950	4.0	1.00	84	1.8	235
0951	5.0	0.75	84	1.8	230
0951	6.0	0.65	84	1.8	230
0952	7.0	0.90	84	1.8	230
0952	8.0	0.65	84	1.8	230
0953	9.0	0.60	85	1.8	230
0953	10.0	0.60	85	1.8	230

Time	Depth (ft)	Velocity (ft/s)	Spec. cond. (µS/cm)	Temp (°C)	Bearing (degrees)
0956	1.0	1.10	87	1.8	220
0957	2.0	1.10	87	1.8	240
0958	3.0	0.95	90	1.9	240
0958	4.0	0.95	90	1.9	240
0959	5.0	0.80	90	1.9	240
0959	6.0	0.85	90	1.9	240
1000	7.0	0.75	90	1.9	240
1000	8.0	0.65	91	1.9	240
1001	9.0	0.75	91	1.9	245
1009	1.0	1.50	91	1.9	240
1009	2.0	1.40	92	1.9	245
1010	3.0	1.35	90	1.9	245
1011	4.0	1.25	91	1.9	245
1011	5.0	1.30	91	1.9	245
1012	6.0	1.00	91	1.9	240
1012	7.0	1.00	91	1.9	240
1012	8.0	1.10	91	1.9	240
1013	9.0	1.10	91	1.9	240
1014	1.0	1.70	91	1.9	240
1015	2.0	1.60	91	1.8	245
1015	3.0	1.50	91	1.8	245
1015	4.0	1.50	90	1.8	245
1016	5.0	1.55	90	1.8	245
1016	6.0	1.30	91	1.8	245
1017	7.0	1.40	90	1.8	245
1017	8.0	1.30	90	1.8	245
1018	9.0	1.30	90	1.8	245
1019	1.0	1.90	90	1.8	230
1020	2.0	1.80	91	1.8	245
1020	3.0	1.70	92	1.8	240
1021	4.0	1.70	91	1.8	240
1021	5.0	1.40	90	1.8	240
1022	6.0	1.55	90	1.8	240
1022	7.0	1.55	90	1.8	240
1023	8.0	1.20	90	1.8	240
1024	9.0	1.30	89	1.8	240
1024	1.0	2.00	89	1.8	240
1025	2.0	1.85	89	1.7	240
1026	3.0	1.80	89	1.7	240
1026	4.0	2.10	88	1.7	240
1027	5.0	2.20	89	1.8	240
1028	6.0	1.80	89	1.8	240
1028	7.0	1.70	89	1.8	240
1028	8.0	1.70	89	1.8	240
1029	9.0	1.80	88	1.7	240

Table 3.--Velocity, specific conductance, and temperature profiles for cross section 32 (800 feet from left bank) Knik River near Eklutna (channel 1), during a tide cycle on September 16, 1985--Continued

Time	Depth (ft)	Velocity (ft/s)	Spec. cond. (µS/cm)	Temp (°C)	Bearing (degrees)
1030	1.0	2.20	88	1.8	245
1031	2.0	2.00	89	1.8	245
1032	3.0	2.00	89	1.8	245
1032	4.0	1.90	89	1.8	245
1033	5.0	1.80	89	1.8	245
1033	6.0	1.60	89	1.8	245
1033	7.0	1.20	89	1.8	245
1034	8.0	1.50	89	1.8	245
1034	9.0	1.40	88	1.7	245
1035	1.0	2.40	88	1.7	245
1035	2.0	2.20	88	1.7	245
1036	3.0	1.90	88	1.7	245
1036	4.0	1.90	88	1.7	245
1037	5.0	1.90	87	1.7	245
1037	6.0	2.00	87	1.7	245
1038	7.0	2.20	87	1.7	245
1038	8.0	1.80	87	1.7	245
1038	9.0	1.90	87	1.7	240
1040	1.0	2.40	87	1.7	245
1041	2.0	2.60	87	1.7	245
1041	3.0	2.40	87	1.7	245
1042	4.0	2.30	87	1.7	245
1042	5.0	2.30	87	1.7	245
1042	6.0	2.10	87	1.7	245
1042	7.0	2.20	87	1.7	245
1043	8.0	1.80	87	1.7	245
1043	9.0	1.50	87	1.7	245
1044	1.0	2.70	87	1.7	245
1044	2.0	2.50	87	1.7	245
1045	3.0	2.50	87	1.7	245
1045	4.0	2.40	87	1.7	245
1045	5.0	2.10	87	1.7	245
1046	6.0	2.10	87	1.7	245
1046	7.0	2.40	87	1.7	245
1046	8.0	2.00	87	1.7	245
1047	9.0	1.40	87	1.7	245
1049	1.0	3.00	86	1.7	245
1050	2.0	2.60	86	1.7	245
1050	3.0	2.40	86	1.7	245
1051	4.0	2.80	86	1.7	245
1051	5.0	2.60	86	1.7	245
1051	6.0	2.40	86	1.7	245
1051	7.0	2.10	86	1.7	245
1052	8.0	2.00	86	1.7	245
1052	9.0	2.00	86	1.7	245
1053	1.0	2.60	86	1.7	245
1053	2.0	2.30	86	1.7	245
1054	3.0	2.60	86	1.7	245
1054	4.0	2.40	86	1.7	245
1054	5.0	2.40	86	1.7	245
1055	6.0	2.50	86	1.7	245
1055	7.0	2.70	86	1.7	245
1055	8.0	2.20	86	1.7	245
1057	1.0	2.80	86	1.6	245
1058	2.0	2.80	86	1.6	245
1058	3.0	2.80	86	1.6	245
1058	4.0	3.20	86	1.6	245
1059	5.0	2.90	86	1.6	245
1059	6.0	2.70	86	1.6	245
1059	7.0	2.90	86	1.6	245
1059	8.0	2.60	86	1.6	245
1100	9.0	2.20	86	1.6	245
1112	1.0	2.50	86	1.6	245
1113	2.0	2.80	86	1.7	245
1113	3.0	2.80	86	1.7	245
1114	4.0	2.60	86	1.7	245
1114	5.0	3.20	86	1.7	245
1114	6.0	2.50	86	1.7	245
1115	7.0	2.20	86	1.7	245
1115	8.0	2.20	86	1.7	245
1115	9.0	1.60	86	1.7	235
1122	1.0	2.30	86	1.7	260
1123	2.0	2.50	86	1.7	260
1123	3.0	2.80	86	1.7	260
1123	4.0	3.20	86	1.7	255
1124	5.0	2.60	86	1.7	255
1124	6.0	2.80	86	1.7	255
1125	7.0	2.80	86	1.7	255
1125	8.0	2.30	86	1.7	255
1125	9.0	2.10	85	1.7	230
1132	1.0	3.00	83	1.7	260
1133	2.0	3.50	83	1.7	260
1133	3.0	3.30	83	1.7	260
1134	4.0	3.40	83	1.7	260
1134	5.0	3.00	83	1.7	255
1134	6.0	3.00	83	1.7	255
1135	7.0	3.20	83	1.7	245
1135	8.0	2.50	83	1.7	245
1135	9.0	2.00	83	1.7	245
1143	1.0	2.40	83	1.7	235
1143	2.0	2.10	83	1.7	235
1144	3.0	2.10	83	1.7	235
1144	4.0	2.00	83	1.7	250
1144	5.0	3.20	83	1.7	250
1144	6.0	2.40	83	1.7	250
1145	7.0	1.80	83	1.7	250
1145	8.0	1.60	83	1.7	250
1145	9.0	1.40	84	1.7	240

The ADOT&PF also provided additional ground surface elevations along transects within the study reach which generally cross the flood plain from north to south (fig. 2). In some cases these elevations were used as a basis for cross-section data where they crossed the river at flow junctions. Elevations along the transects were obtained using photogrammetric methods and were based upon the vertical control points provided by the ADOT&PF.

GENERAL DESCRIPTION OF BRANCH-NETWORK MODEL AND REQUIREMENTS

The U.S. Geological Survey's branch-network flow model, referred to as BRANCH (Schaffranek and others, 1981), was used to simulate flow characteristics in the study reach. The model is based on the one-dimensional partial-differential equations of continuity and momentum which govern unsteady flow. For computational purposes, these equations are replaced by implicit finite-difference equations which approximate the actual solution. A detailed discussion of the equations is beyond the scope of this report; however, a list of conditions considered to be valid is given:

1. The channel slope is mild and constant over the reach length so that flow remains subcritical.

2. Lateral inflow or outflow between channel junctions is negligible.

3. Manning's roughness coefficient ("n" value) is representative of frictional resistance in unsteady and steady flows.

4. The density of the flow is substantially homogeneous.

5. Hydrostatic pressure exists throughout the channel.

6. A moderately uniform velocity distribution is present within any cross section.

7. Channel beds are stable and are not subject to significant scour or fill.

8. No channel is allowed to go dry.

This model was selected for use on the Knik and Matanuska Rivers because it can accommodate river reaches consisting of interconnected channels. It is also capable of accounting for point source inflows and outflows within the modeled reach. Furthermore, it can compensate for the effects of wind shear on the water surface, and for some degree of nonuniformity in the velocity distribution throughout a cross section.

Three requirements must be met when developing a model of the river to be studied using the branch-network model. These requirements include:

1. The reach to be modeled must be properly represented or schematized to accurately depict the controlling features of the river.

2. Channel cross-section data (areas and top widths) must be provided at critical locations that are determined when schematizing the reach.

3. Sets of time-series data, consisting of synchronous stage and (or) discharge values, must be provided at all external ends of study reach.

The accuracy of the data used to fulfill each of these requirements is critical to the accuracy of the model simulations. In addition to these requirements, several computational control parameters must be adjusted to facilitate calibration of the model.

Schematization of the Study Reach

The purpose of the schematization process is to identify the important controlling features of the study reach and include them in such a way as to represent their effect on the flow system. These features include external channel junctions which delimit the ends of the study reach, internal junctions where two or more channels either diverge or converge, branches which are reaches between junctions, and the branch lengths. Other features that require definition are constricting or expanding reaches and locations where simulated data is sought once the model has been calibrated.

Owing to the complexity of the channel patterns within the Knik-Matanuska study reach, it was impractical to represent exactly every channel within the system. It was decided to initially schematize the river as simply as possible, and to add more detail after successful simulations were achieved. Thus the model was first schematized using only the main stem of the Knik River (fig. 18). This approach was taken early in the study to gain experience in using the model and its associated support programs. Once this was accomplished, cutoff and overflow channels were added to complete the necessary detail of the Knik River, and a schematization of the Matanuska River was included (fig. 18).

The complete schematization of both the Knik and Matanuska Rivers included 6 external junctions (ends of reaches), 19 internal channel junctions, and 33 branches (subreaches). The branches are further subdivided into segments in locations where greater detail was desired. Some areas are represented in greater detail than others, depending upon the need for information at those locations. Branch and segment lengths (fig. 18) were determined from topographic maps by scaling the distances along the channel thalweg of interest.

Channel Area and Width Data

Certain cross-section properties must be determined for all cross sections at external and internal junctions as well as at the termini of all segments. These properties, in the form of area and top width as a function of stage, are used by the model during computation. The area and width tables must cover the entire range of stages to be modeled.

Reduction of the field cross-section data was aided by a computer program that provided the stage, area, and top-width tables in a format acceptable to the BRANCH model. The Channel Geometry Analysis Program (CGAP) by Regan and Schaffranek (1985) is designed as a modular addition to the branch-network flow model and provides a simple means for the preparation of cross-section data (fig. 19) for use by the BRANCH model.

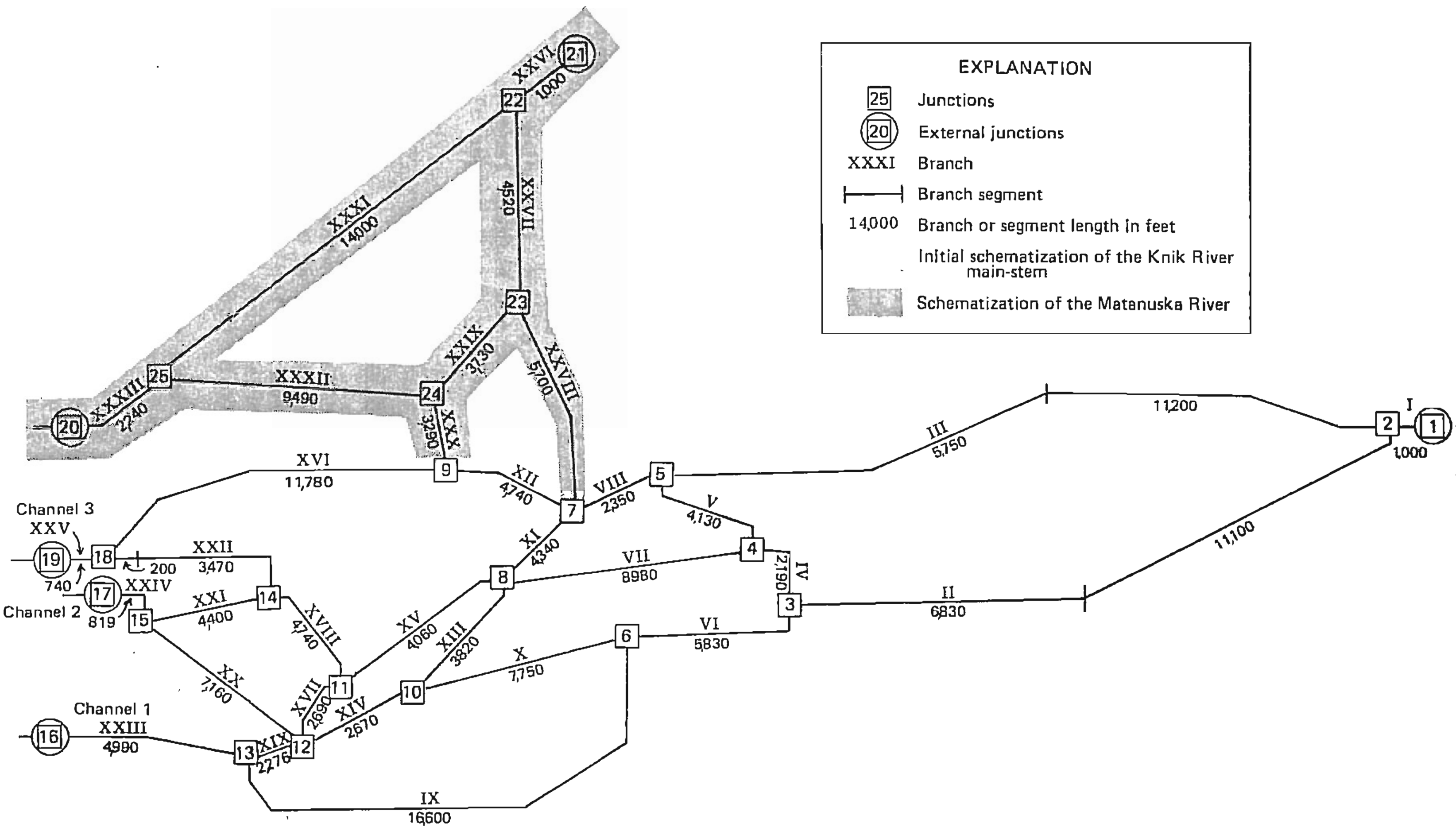

Figure 18.--Schematization and branch lengths of the Knik and Matanuska Rivers for the branch-network flow model.

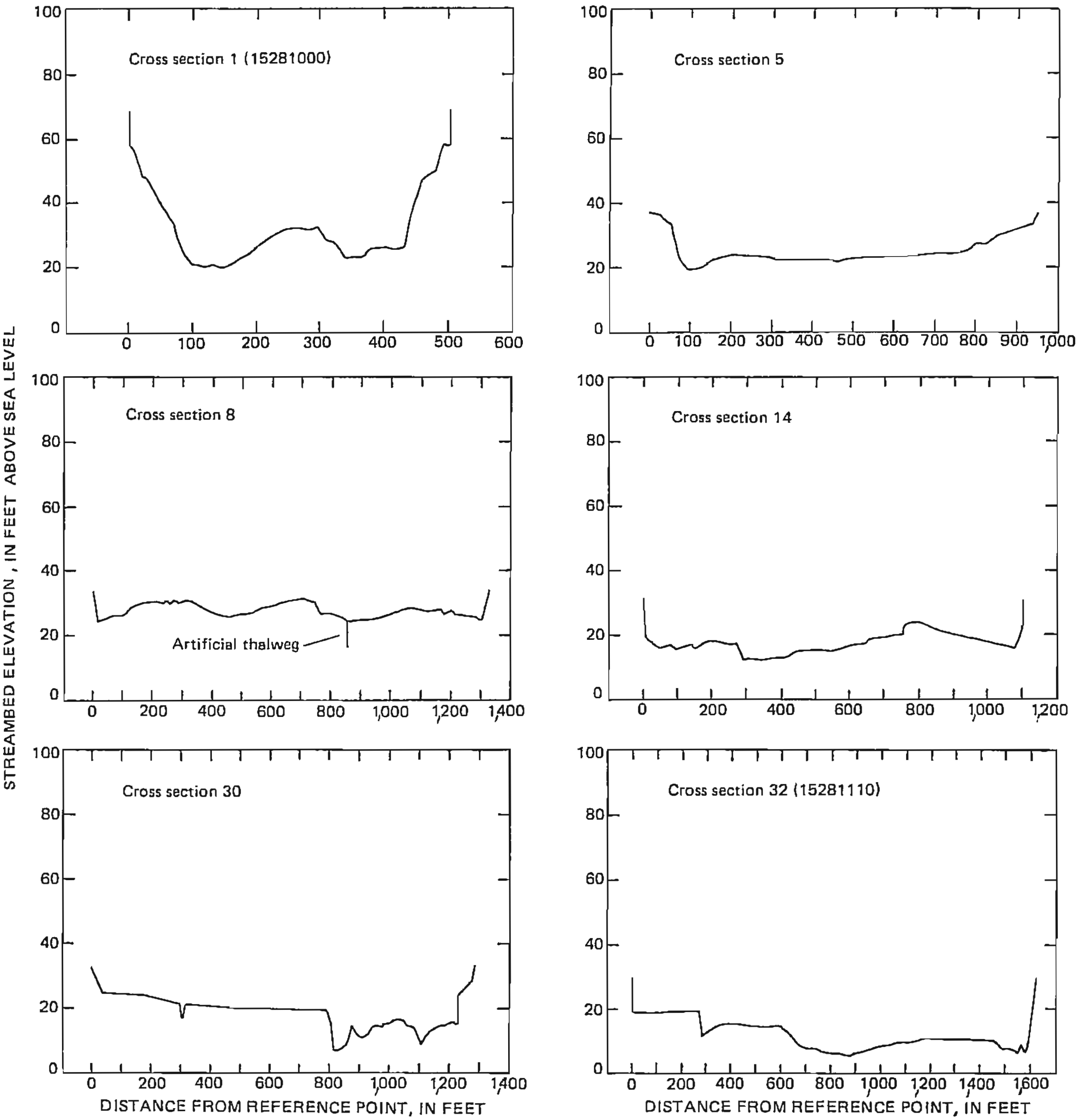

Figure 19.--Selected cross sections of the Knik and Matanuska Rivers. (See figure 2 for cross-section locations.)

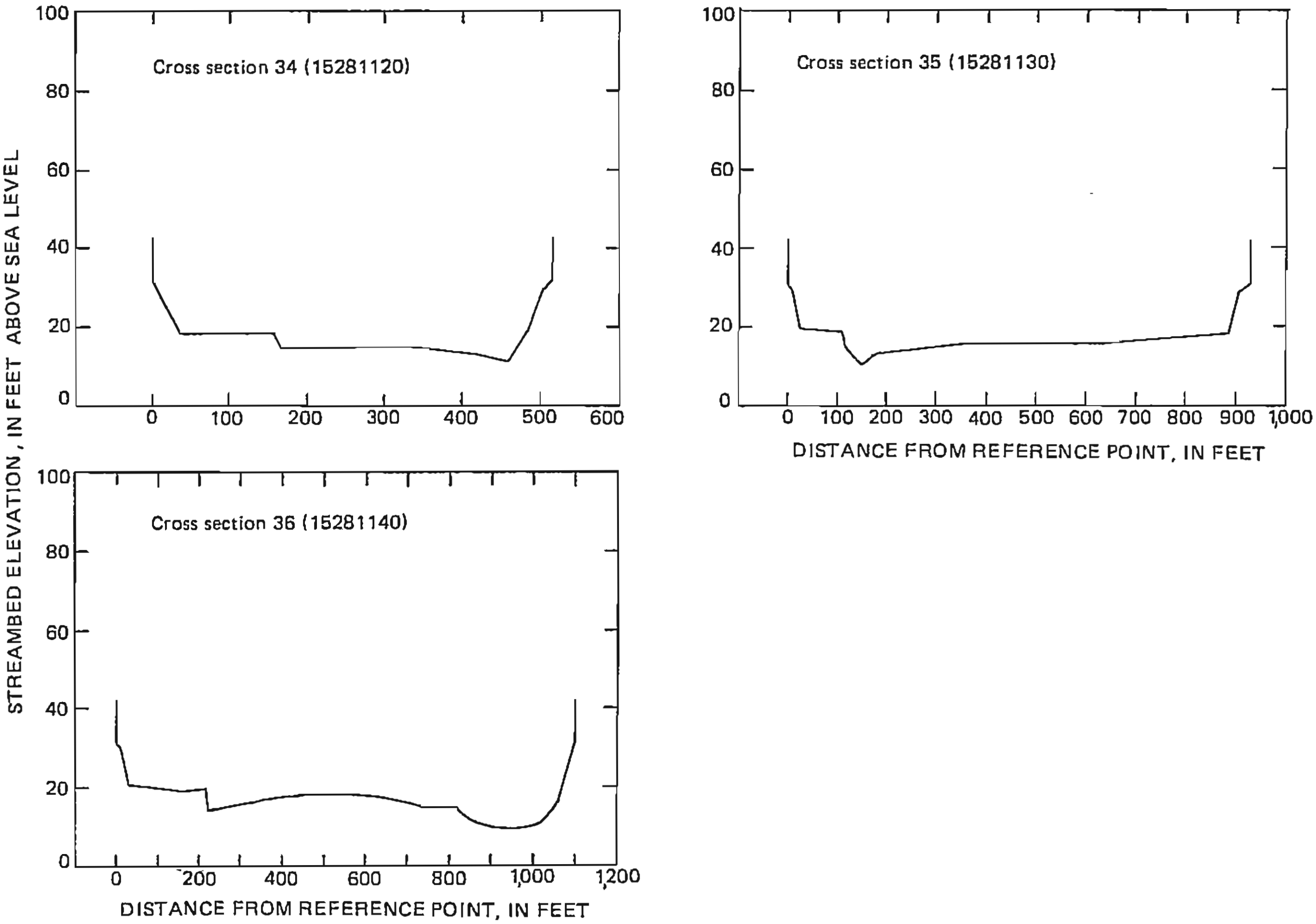

Figure 19.--Continued.

The CGAP is capable of presenting the data in a variety of ways so that it can be edited, thereby insuring its accuracy.

A constraint that all channels schematized in the modeled reach are always actively conveying water dictates that channels cannot be allowed to go dry during the computational process. The study reach has overflow channels at several locations that in fact convey flow only at higher stages. In order to accurately represent the modeled channel without causing the model to fail, it was necessary to modify the channel cross-sectional shape to include an artificially deep thalweg or "spike." The cross-sectional area of this spike is negligible and a channel roughness value n = 0.90 was assigned, thereby retaining the integrity of the channel conveyance properties while still satisfying the constraints of the model (fig. 19, cross section 8). Data used to define channel geometry for the modeled reach were compiled by Lipscomb (1985).

Stage and Discharge Data

The boundary values required for the branch-network flow model consist of water stages and (or) discharges at all external channel junctions of the modeled river. These data must be precisely timed so that values provided to the model for use at the ends of the reach are synchronous. Stage data are generally used because of their relative ease of measurement. The stage data are timed using various types of solid-state clocks depending on the type of gage used at a measuring location. The times recorded while collecting the stage data were verified against the solid-state clocks before and after collecting each set of data. Levels were surveyed to the water surface before and after collection of all stage data to insure accuracy of the elevations.

During the course of the study, water stage was recorded continuously during the open-water (ice-free) period at station Nos. 15281000 and 15284000 (Knik and Matanuska Rivers near Palmer) and at station Nos. 15281110 and 15281140 (Knik and Matanuska Rivers near Eklutna). For several days preceding scheduled field trips, stage data were also obtained at the two downstream secondary channels (station Nos. 15281120 and 15281130) and at two other selected sites (station Nos. 15281003 and 15281005) within the study reach. Stage data collected at the latter two sites were not required as model input but were used to aid in calibrating the model and in determining the distance the tide wave travels up the channel.

In order to obtain discharge data at the downstream ends of the study reach during periods of unsteady flow, measurements were made during maximum tides in the summers of 1984 and 1985. These data were used to calibrate and verify the model. Stage and discharge data used in the BRANCH model, as well as stage data collected at the miscellaneous sites (station Nos. 15281003 and 15281005), can be found in the compilation by Lipscomb (1985).

A system of computer programs is available to reduce raw field data to a format that can be used in the BRANCH model (J.A. Lorens, U.S. Geological Survey, written commun., 1985). The Time Dependent Data System (TDDS), a modular addition to the branch-network flow model, has proven to be a useful tool in this study. This system includes programs for converting digitally recorded stage data into model input format as well as providing a means to edit and plot the data to insure its accuracy. Figure 20, as well as figures 10 and 11, illustrates the TDDS output option for producing digital plots of stage hydrographs. The plots show stage data for June 18-20, 1985, obtained at the Knik and Matanuska Rivers and at both secondary channels at the lower end of the modeled reach (station Nos. 15281110, 15281130, 15281140, and 15281120, respectively).

USE OF THE BRANCH-NETWORK MODEL TO SIMULATE FLOW AND HYDRAULIC CONDITIONS

Calibration and Verification

Once the data required to schematize the study reach in the branch-network model were available, model runs were made to simulate stage and discharge at all locations where channel area and width were defined. Initial calibration consisted of comparing measured discharges with those computed by the model. The goal of the calibration process is to adjust the parameters and coefficients that affect the model computation until the computed output agrees with the measured input as closely as possible.

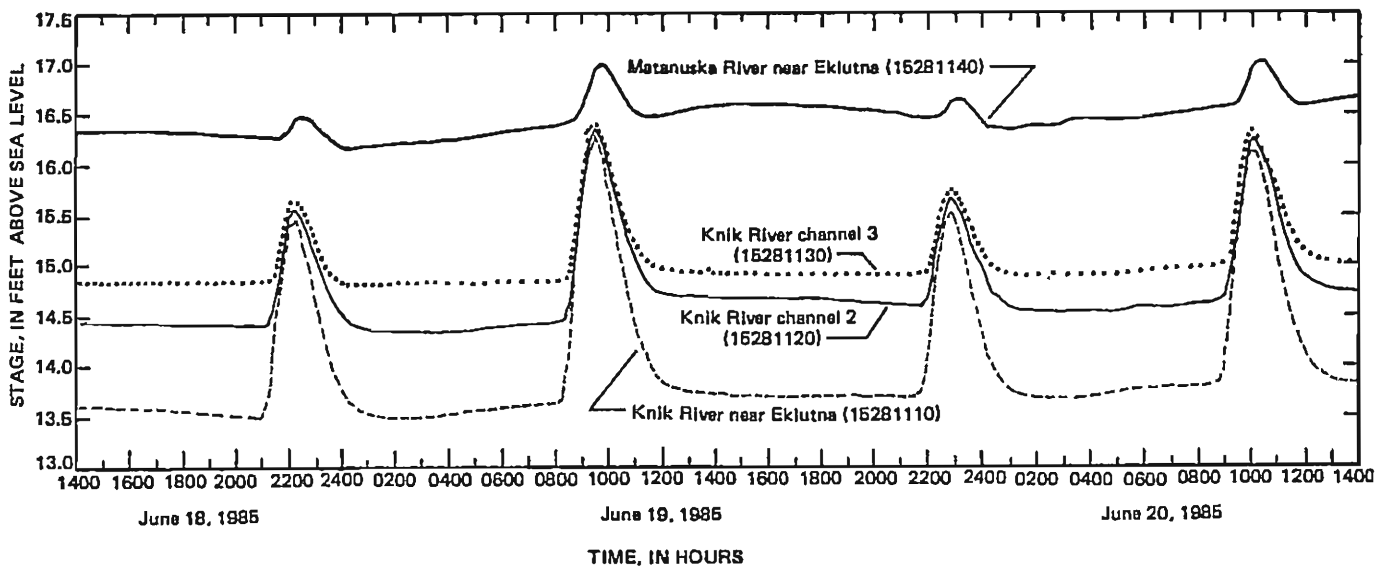

Figure 20.--Example of a Time Dependent Data System digital plot of the boundary-value stage data for four stations.

These parameters and coefficients include such variables as the channel roughness, η, and a coefficient of momentum, or β value, which accounts for non-uniform velocity distributions at a given cross section. The calibration may also require the adjustment of the water-surface slope, cross-sectional area, and top widths. These adjustments are usually necessary when datum errors are present either in the cross-section data or the stage data.

Calibration parameters are initially set on the basis of reasonable estimates, and then are adjusted systematically to improve the model results. The roughness coefficient η is set individually for each branch. During initial runs, η was set to 0.028 for all branches; final calibration yielded values ranging from 0.036 to 0.040. The weighting factors θ and χ, which control the finite-difference approximations of the differential equations (Schaffranek and others, 1981), were set to 1.0 and 0.5, respectively, because these values produced the most stable solution. The model was more sensitive, however, to the θ than to the χ value. The momentum coefficient β was initially set to 1.06, which is generally thought to be typical of turbulent flows in a natural channel. This value was varied from 1.0 to 1.1 but little change was observed in the results. The time step (Δt) was initially set to 15 minutes, but was changed to 5 minutes when it was found that this change improved model stability.

In the initial runs, which included only the Knik River sites and the two secondary lower downstream sites (Nos. 15281000, 15281110, 15281120, and 15281130), satisfactory results were obtained using a 5-minute time interval. After obtaining reasonable results from this setup, the Matanuska River was included in the schematization and additional runs were made. When a 5-minute time interval was used for the Matanuska River simulation, the model became unstable. The interval was then reduced to 2 minutes, which improved the stability slightly but required considerably more computation time. The additional computational instability encountered as the calibration progressed was attributed to factors such as oversimplified schematization, inclusion of overflow channels with associated artificial thalwegs, steep gradients, and short reach lengths.

Near the lower end of the study reach, the Matanuska River is a complicated network of interconnected channels (fig. 2). Consequently, the schematization of this reach had to be simplified considerably for use in the model. This was accomplished by incorporating into the schematization only those channels that were determined to be of most significance.

At times during model runs, the artificial deep channels, which had been simulated to prevent the occurrence of zero discharges in the branches, were shown to be conveying considerable flows, resulting in unrealistic circulation patterns and fluctuations of stage within the network. This problem was most pronounced at low stages and made calibration of the model in this range difficult. Because one objective of using the model was to provide a tool for the analysis of flood flows, it was determined that calibration of the model in the higher ranges of stage was most critical. Therefore, the model was calibrated using measured data for higher stages only. Three complete sets of measured data were available for calibration and verification of the model. Of these three data sets, however, only two proved to be useful. The third set was obtained during a period of relatively low discharge, which resulted in the problem of zero discharge occurring in a branch.

The BRANCH model expects branch lengths on the order of 5,000 to 25,000 ft as optimum for simulation purposes. Branches shorter than this require correspondingly smaller simulation time increments. This is based on the Courant restriction (Schaffranek and others, 1981), which states that the simulation time increment (Δt) must be less than the ratio of the branch length (Δx) to the wave celerity ($\sqrt{gH}$) and the flow velocity (U). This relation is expressed in equation form as

$$\Delta t \leq \frac{\Delta x}{|U \pm \sqrt{gH}|}$$

where g is acceleration due to gravity, and
H is flow depth.

Although this restriction need not be rigidly adhered to in an implicit solution, as is used in the BRANCH model, it is still a valid index that should not be exceeded by more than a factor of five (Schaffranek and others, 1981). In the schematization of the Knik and Matanuska Rivers, most of the branches are less than 5,000 ft long and some are as short as 700 to 800 ft. As a result, the simulation time increment had to be reduced from 15 minutes as originally planned, to 5 minutes, and finally to 2 minutes.

Due to persistent problems of model instability when the Matanuska River was included, it was decided to eliminate the Matanuska River from the analysis. This resulted in the loss of some inflow into the Knik from the Matanuska, but was necessary in light of the difficulties experienced. Efforts were then concentrated on calibrating the model for the main channel and the two secondary channels of Knik River using the measured data for the three remaining downstream sites (station Nos. 15281110, 15281120, and 15281130).

The July 3, 1985 data were initially used for calibration because they were collected at the highest steady discharge and produced the most stable model results. After a lengthy trial and error process, the calibration coefficients that gave the best overall results were obtained. It was established that the calibration was satisfactory when the modeled discharge throughout the period of a tide cycle was within 10 percent of the measured values. This criterion was based on the many uncertainties involved in obtaining the measured data. Due to the complexity of collecting and reducing raw field data into a suitable set of data for calibration purposes, it is not unlikely that measured discharges differ from actual discharges by as much as 20 percent.

Comparisons of measured and simulated discharge on July 3, 1985 for the three downstream sites are shown in figure 21. In most cases, the computed values are within 10 percent of the measured values and are usually within 5 percent. Once the model was adjusted to obtain these results, it was run again using the data for August 28, 1984 to verify the calibration. The results from this run are shown in figure 22. The comparison of measured and computed values are in good agreement over some ranges of discharge for channels 1 and 2 (station Nos. 15281110 and 15281120); for channel 3 (station No. 15281130), however, the results are poor.

Further calibration of the model would require that additional data be collected, preferably at higher flows. With this additional data, it would be possible to calibrate the model over a wider range of flow conditions. The model probably could be improved by using a functional relation (involving stage or discharge) to define the roughness coefficient η rather than using a constant value as was done in this analysis. This would account for changing channel roughness as stage and discharge changed. Since the downstream sites are subject to a wide fluctuation in stage due the tides, the actual channel roughness is probably variable and could best be described by the functional relation. However, to define this relationship would require several additional sets of measured data obtained over a wide range of flow and tidal conditions, and could not be obtained within the time constraints and scope of this study.

Model Usage and Limitations

The intended use of the BRANCH model was to route design floods through the lower, tide-affected reaches of the Knik and Matanuska Rivers and observe their effect at locations of interest. Sites of particular interest were the highway and railroad crossings near the lower end of the reach.

Model limitations are inherent when attempting to use the model for complicated river conditions. The computations made by the model are based on the data used to describe the modeled river reach. Specifically, the cross-section data define the limits of stage and, therefore, of discharge that the model is capable of simulating. Hydrologic conditions in the Knik River basin present two distinctly different potential flood scenarios -- with and without a breakout of glacier-dammed Lake George. Past observations have indicated that during a breakout flood, the channels

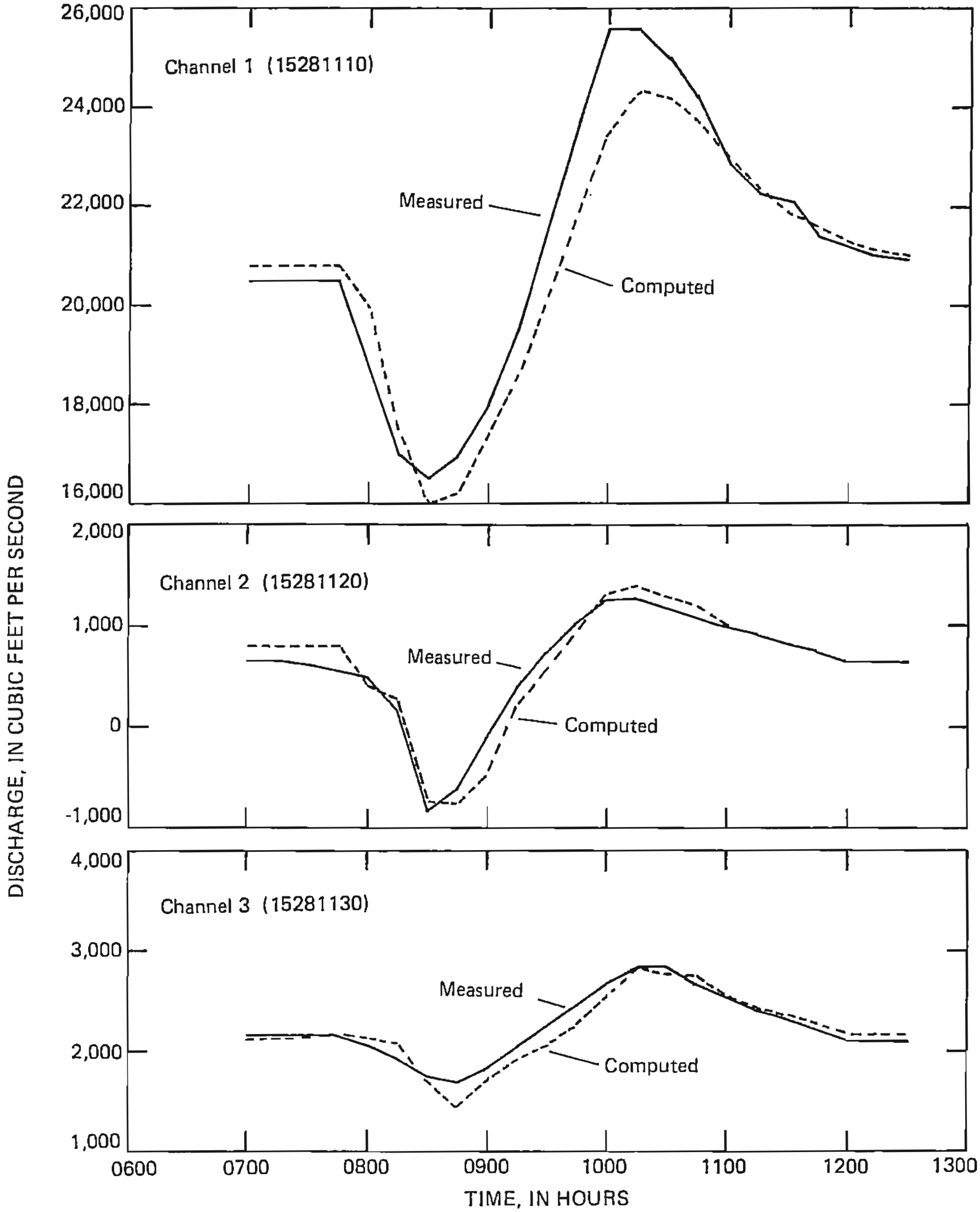

Figure 21.--Comparison of measured and computed discharge for the Knik River near Eklutna on July 3, 1985.

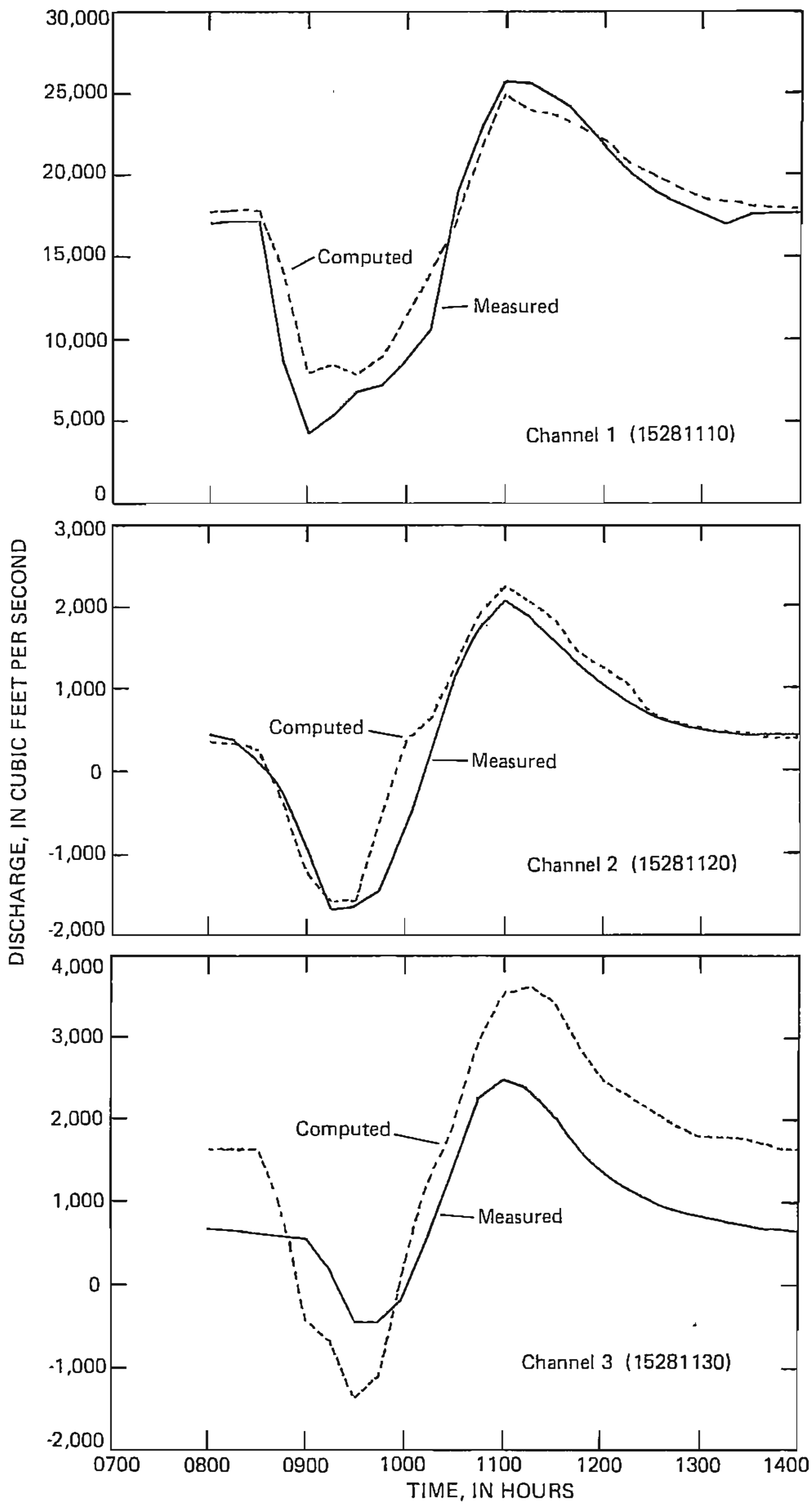

Figure 22.--Comparison of measured and computed discharge for the Knik River near Eklutna on August 28, 1984.

within the study reach were overtopped and the entire flood plain was inundated. When bankfull stage is reached in any of the channels, model simulation terminates. Consequently, the model was developed with the understanding that it could be applied using input hydrographs only for nonbreakout floods and then only for those runoff events that would not produce significant overbank flow.

The bridges on the present-day Glenn Highway were designed to accommodate the large floods that would result from a breakout of Lake George. The threat of a flood of that magnitude has been temporarily removed because the ice dam has failed to form since the late 1960's. In an attempt to reduce costs of the new highway, ADOT&PF planners have considered the possibility of eliminating one or more of the four existing bridges. The branch-network model was used to experiment with several bridge configurations--varying the number and (or) width of the openings-- thereby helping to optimize the design of the new roadway.

Flow Simulation Alternatives and Results

The BRANCH model uses time series of water level (stage) or discharge data at both upstream and downstream ends of the modeled reach. This allows the model to account for both flood waves and tide waves, and thereby permits simulation of unsteady flows. Simulation of a design flood would therefore require the input of time-series stage or discharge data that define the rising and falling limbs of the floodwave. These input time-series data are required at both the upstream as well as the downstream ends of the modeled reach where they must reflect wave attenuation and travel time effects resulting from the passage of the flood wave through the various channels. Further, they must reflect the combined effects of the floodwave moving downstream and the tide-wave propagating upstream. Because there is no feasible way to synthesize these time-series data with any confidence, two alternatives were devised to estimate the routing of a design flood through the study reach.

The first alternative involved using a synthesized input time-series at the upstream end of the reach and a "self-setting stage" option at the downstream ends. This option allows the model to compute the stage at the lower end of the modeled reach on the basis of the computed stage for the previous time-step at the nearest cross section (upstream in this case). To use this option, the tide wave and associated unsteady flows are assumed either to be absent or to have inconsequential effects in the lower reaches of the river. Study of figure 12 shows that this assumption cannot be made for the case involving a medium river discharge and maximum tide event. It can be shown, however, that discharges of progressively greater magnitude eventually override the effects of the tide until those effects are completely nullified. A design flood of 40,000 to 60,000 ft^3/s probably will cause the tidal influence to be insignificant.

After several model runs (simulations) were made using the self-setting stage option, an inspection of the results indicated that simulated stages were significantly higher than expected for the given discharge. In addition, the simulated mean velocities were substantially lower than measured velocities. This was a result of the larger cross-sectional areas associated with higher stages. These results led to the determination that the self-setting stage option was not performing as intended. The cause of the error was not conclusively determined, but it is possibly the result of having multiple lower flow channels specified. An additional disadvantage of using the self-setting stage option in this study is that the downstream ends of the reach, where the option was invoked, is coincidental with the locations of most interest -- that is, the bridge sites. Any inaccuracies in the stage or discharge data used for the bridge sites in the model will consequently have a direct impact on the results at those locations.

The second alternative addressed the problem introduced by the self-setting stage option by extending the model downstream to a new single channel (fig. 23, branches XXVI-XXIX). This approach offered two advantages: first, it enabled the separation of the troublesome lower channels of the model from the locations to be evaluated; and secondly, it reduced the number of modeled channels from three, at the bridge sites, to one farther downstream. By making this change, it was possible to use a constant discharge at the upstream and downstream ends of the reach. It would also be possible to observe the circulation patterns and determine the percentages of flow in the various channels throughout the modeled reach.

To implement the second alternative of extending the model reach, it was necessary to first construct a downstream cross section that had appropriate dimensions and conveyance properties. A surveyed cross section could not be obtained easily at this location because the actual separate channels converge not into a singular channel, but into a complex braided reach. Therefore, a stage versus top-width and area table for the hypothetical single channel was developed from the area-width tables for the three original channels.

Several runs (simulations) were made to test the sensitivity of the model to variations in channel cross-section data at the new downstream location. The changes to the cross-section data tables were accomplished by adding datum adjustments to one or both of the cross sections in branch XXIX (fig. 23). The adjustments varied from 4 to 5 ft in both sections, which in effect produced a wide range of channel areas and widths. The sensitivity analysis for these latest runs was based on a comparison of computed stages, velocities, and discharges for the three original upstream channels at the highway bridges. The results of these runs showed that major variations in the areas and widths of the synthesized cross section have little or no influence on model-computed stage and discharge values at the bridges (table 4). Thus extending the lower end of the study reach and representing (in the model) the river as a single channel seemed to be a reasonable approach.

The assumption, discussed above, that flows greater than 40,000 ft^3/s would override any tidal influence enabled using a constant flow throughout the study reach without having to consider unsteady flows due to the tide. Initial runs gave satisfactory results using both 40,000 and 50,000 ft^3/s as constant flows. However, when the discharge was increased to 60,000 ft^3/s, the model terminated because the stage went overbank at the downstream cross section of branch XIX.

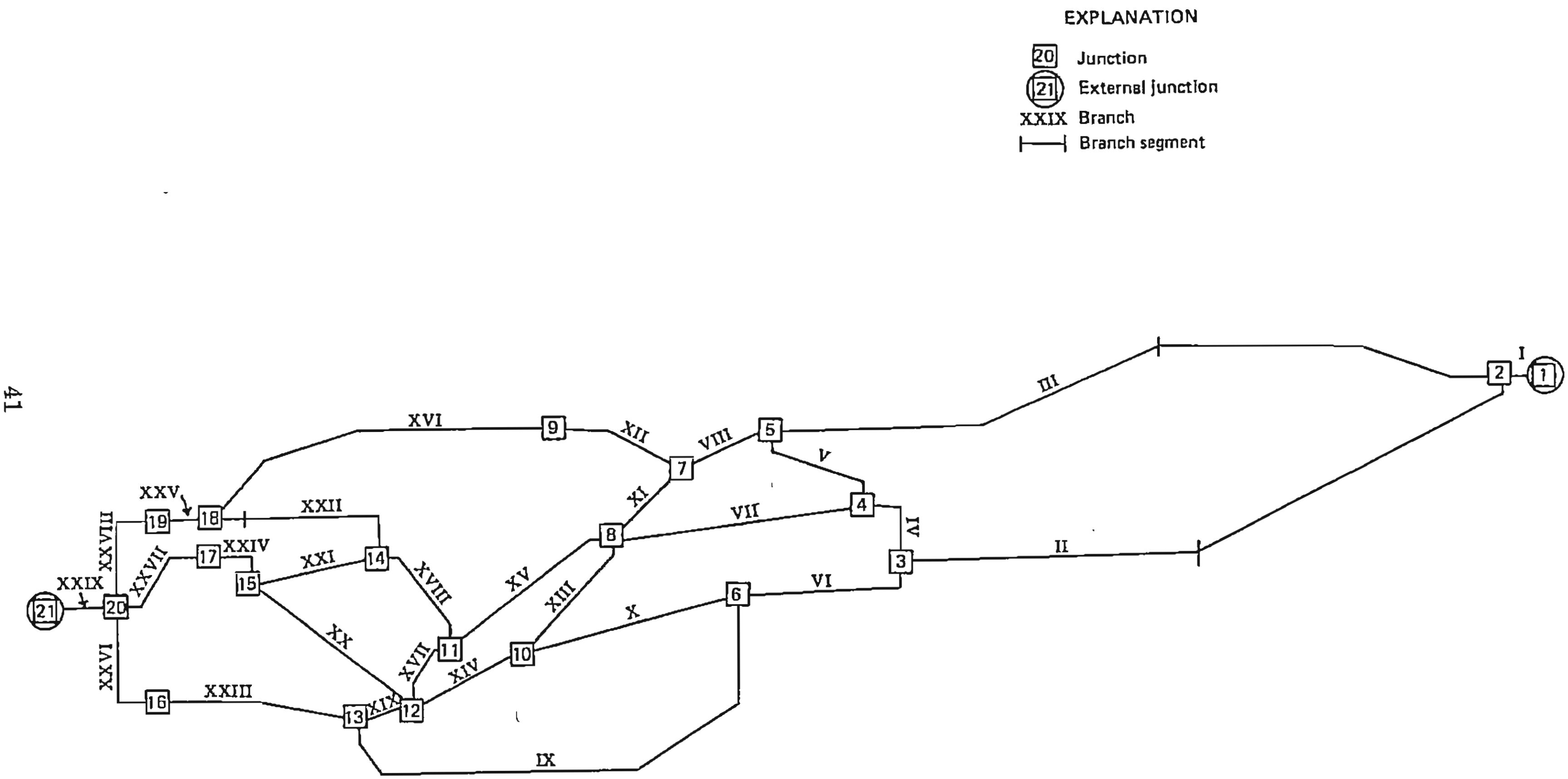

Figure 23.--Schematization of Knik River reach showing addition of branches XXVI - XXIX and single downstream boundary.

Table 4.--Results of simulations indicating sensitivity of the model to changes in geometry of upstream and downstream cross sections of branch XXIX

[ft, feet; ft/s, feet per second; ft^3/s, cubic feet per second]

Branch XXIX datum adjustment (ft) Section*		Computed hydraulic values								
		Branch XXIII			Branch XXIV			Branch XXV		
		Stage	Velocity	Discharge	Stage	Velocity	Discharge	Stage	Velocity	Discharge
1	2	(ft)	(ft/s)	(ft^3/s)	(ft)	(ft/s)	(ft^3/s)	(ft)	(ft/s)	(ft^3/s)
0.0	-0.3	20.27	2.49	33,105	19.81	1.18	2,404	19.75	1.27	4,480
0.0	-1.0	20.27	2.49	33,105	19.81	1.18	2,403	19.74	1.27	4,480
0.0	1.0	20.27	2.49	33,118	19.81	1.18	2,404	19.75	1.27	4,485
-0.5	-1.0	20.26	2.49	33,114	19.80	1.18	2,403	19.74	1.27	4,481
0.5	0.0	20.27	2.49	33,105	19.81	1.18	2,404	19.75	1.27	4,480
-1.5	-2.0	20.26	2.49	33,114	19.80	1.18	2,403	19.74	1.27	4,480
1.5	1.0	20.27	2.49	33,113	19.81	1.18	2,403	19.74	1.27	4,481
-2.5	-3.0	20.27	2.49	33,105	19.81	1.18	2,403	19.74	1.27	4,480

*Section 1, upstream; section 2, downstream

Alternative Channel Configurations and Results

Discharge values of 40,000 and 50,000 ft^3/s were used in model runs made for various configurations at the lower bridge sites (station Nos. 15281110, 15281120, and 15281130). These runs allowed for an evaluation of conditions under the possible potential construction scenarios. The model "setups" and configurations analyzed, with channels 1, 2, and 3 referring to stations Nos. 15281110, 15281120, and 15281130 respectively, include:

Setup	Configuration
1	Channels 1, 2, and 3 unaltered (existing bridge openings)
2	Channel 2 closed
3	Channel 3 closed
4	Channel 3 width reduced from 928 ft to 250 ft
5	Channels 2 and 3 closed
6	Channel 2 closed, channel 3 width reduced from 928 ft to 250 ft

Individual channels were "closed" in the model by setting the associated discharge to zero throughout the simulation. The reduction in width of channel 3 in setups 4 and 6 was accomplished by altering the channel cross-section data accordingly.

The results of the simulations -- distribution of flow among the three channels at the bridges (represented in the model by branches XXIII, XXIV, and XXV, figs. 18 and 23) -- are given in tables 5 and 6. Figures 24 through 29 show the distribution and direction of flow through the study reach for each model setup with a constant discharge of 50,000 ft^3/s. Simulated stage and velocity results at the bridge sites are of limited value in determining changes in backwater conditions; therefore they are not included. These data are of limited value because the BRANCH model does not address head losses through a constricted opening in the same way that various water-surface profile models specifically designed for analyzing bridge sections would. The results from the model, however, are useful in determining changes in the distribution of flow and circulation patterns using the various bridge configurations studied. These results can then be used as input to a water-surface profile model such as WSPRO (Shearman and others, 1986) to obtain detailed flow data for the bridge sections.

The circulation patterns that emerge after the closure of either or both of the secondary bridge openings are interesting. Significant changes in the flow distribution occur, particularly through branches XVIII, XX, XXI, and XXII (figs. 25, 26, 28, and 29). Flow reversals in these branches also develop as water entering these channels from an upstream junction are forced back into the main stem of the Knik River in order to exit from the system.

SUMMARY AND DISCUSSION

The Knik and Matanuska Rivers merge in a combination riverine-estuarine reach at the head of the Knik Arm of Cook Inlet in southcentral Alaska. The flow characteristics of this reach are complicated by a number of factors: unsteady flows produced by semidiurnal tides; a network of interconnected channels, some of which convey flow only at high stages and are otherwise dry; relatively steep channel gradients; and the historic formation and subsequent breakout of a glacier-dammed lake in the upper reaches of the Knik basin, which produced floods six to seven times greater than those of nonbreakout years.

A branch-network flow model developed by the U.S. Geological Survey was used to simulate flow conditions in the lower, tide-affected reaches of the two rivers. The one-dimensional flow model was used to simulate flows at bankfull stage, but larger overbank flows, such as those likely to be produced by breakout of a glacier-dammed lake, could not be simulated.

The use of the branch-network model has three prerequisites. First, proper layout or schematization of the modeled reach into discrete channels is required. This means establishing appropriate boundaries of the modeled reach and identifying all flow-controlling features of the channels. Second, channel cross-section data must be provided at critical locations within the reach, to define channel conveyance properties. Finally, the model requires that synchronous stage and (or) discharge data be available at the ends of the modeled reach.

Initial calibration attempts were unsuccessful, resulting either in a failure of the model to complete the specified simulation, or when

Table 5.--Results of BRANCH model simulations using 40,000 cubic feet per second throughout the reach

[Discharge in cubic feet per second]

	Distribution of flow					
	Branch XXIII		Branch XXIV		Branch XXV	
Setup	Discharge	Percent of discharge	Discharge	Percent of discharge	Discharge	Percent of discharge
1	33,100	83	2,400	6	4,480	11
2	33,800	84	0	0	6,200	16
3	34,400	86	5,570	14	0	0
4	33,100	83	2,730	7	4,170	10
5	40,000	100	0	0	0	0
6	34,000	85	0	0	5,960	15

Table 6.--Results of BRANCH model simulations using 50,000 cubic feet per second throughout the reach

[Discharge in cubic feet per second]

	Distribution of flow					
	Branch XXIII		Branch XXIV		Branch XXV	
Setup	Discharge	Percent of discharge	Discharge	Percent of discharge	Discharge	Percent of discharge
1	40,000	80	3,680	7	6,310	13
2	41,500	83	0	0	8,540	17
3	42,300	85	7,690	15	0	0
4	40,000	80	4,140	8	5,840	12
5	50,000	100	0	0	0	0
6	41,800	84	0	0	8,170	16

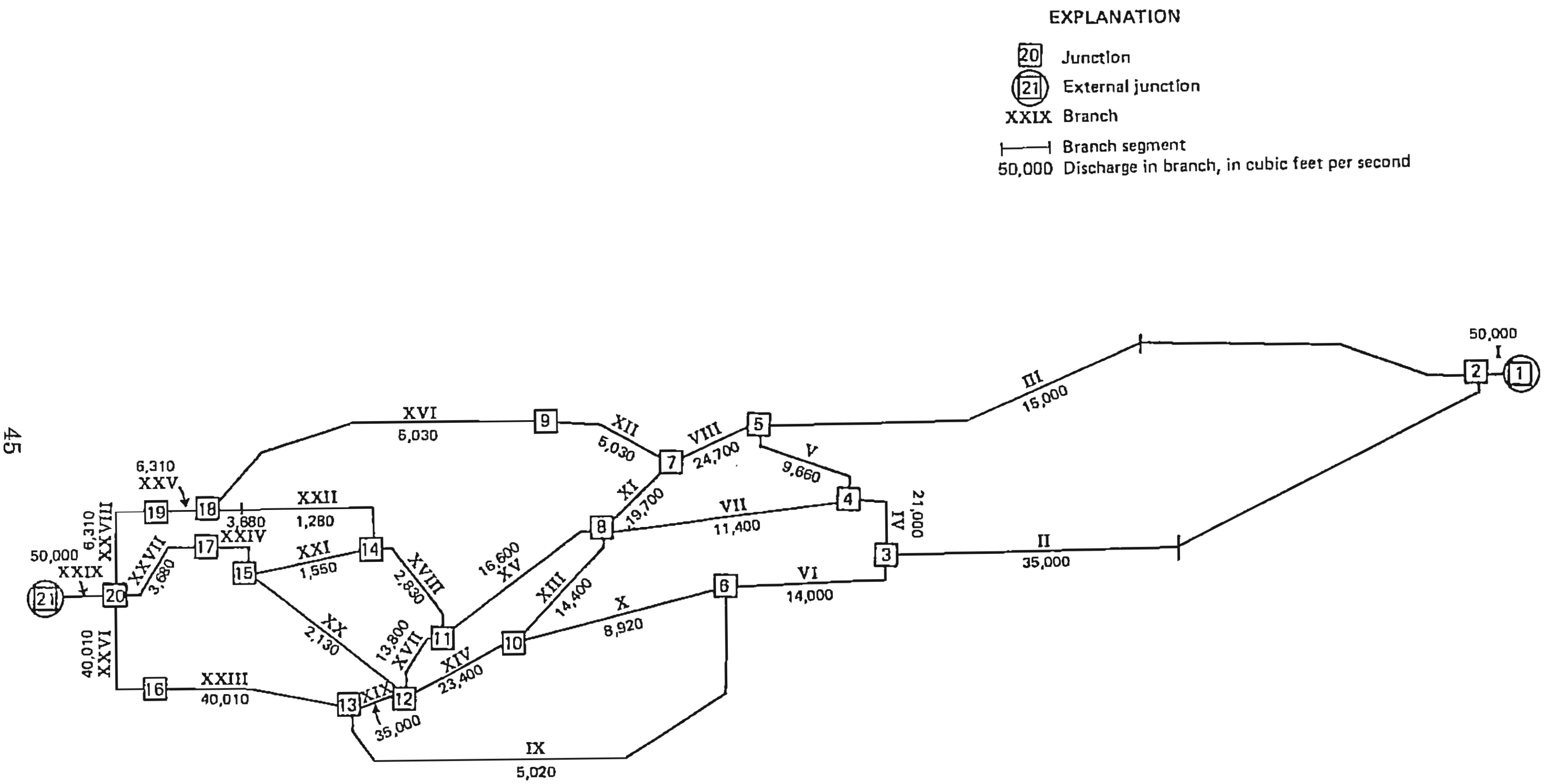

Figure 24.--Simulated distribution of flow through study reach with existing channel configuration (setup 1).

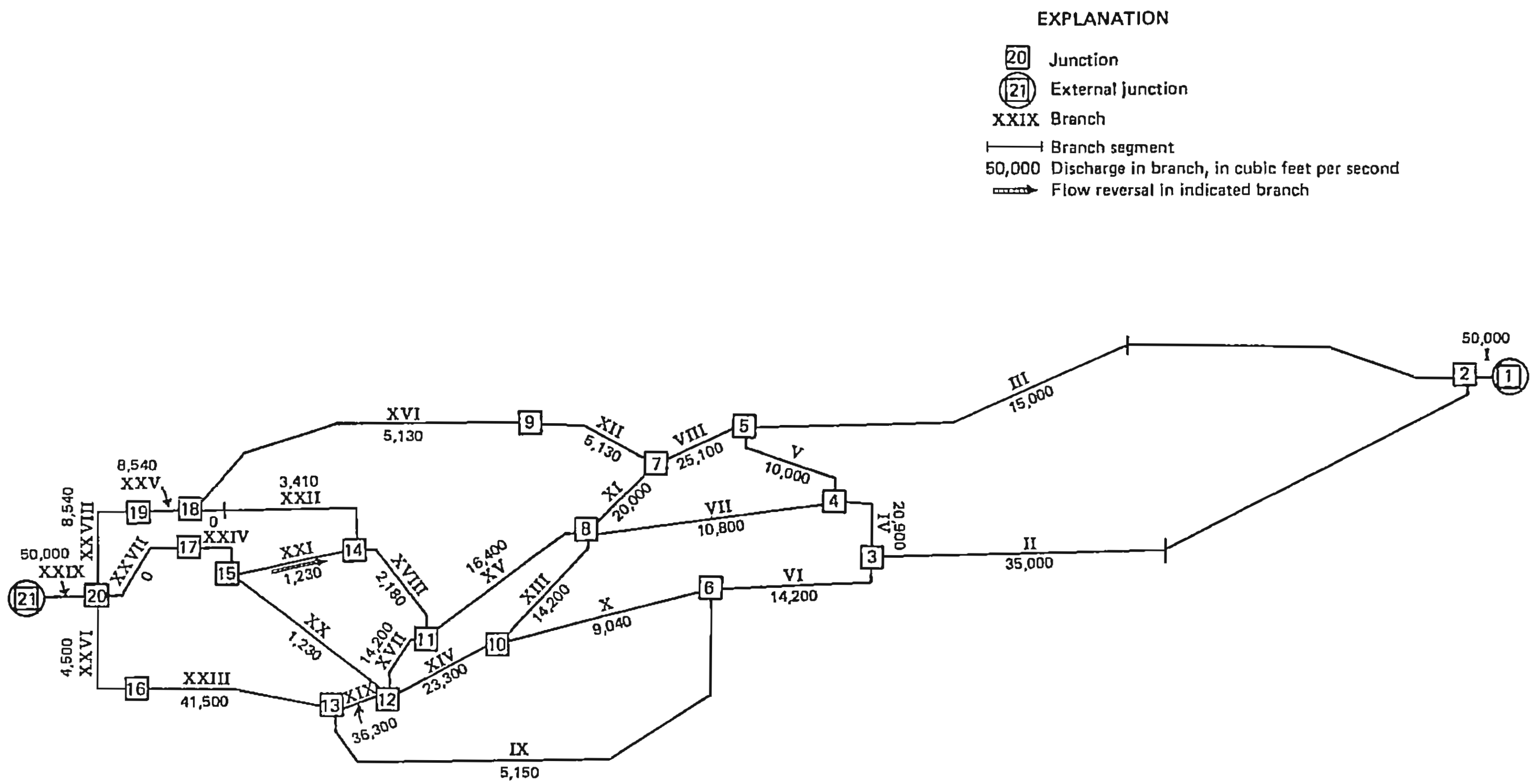

Figure 25.--Simulated distribution of flow through study reach with channel 2 closed (setup 2).

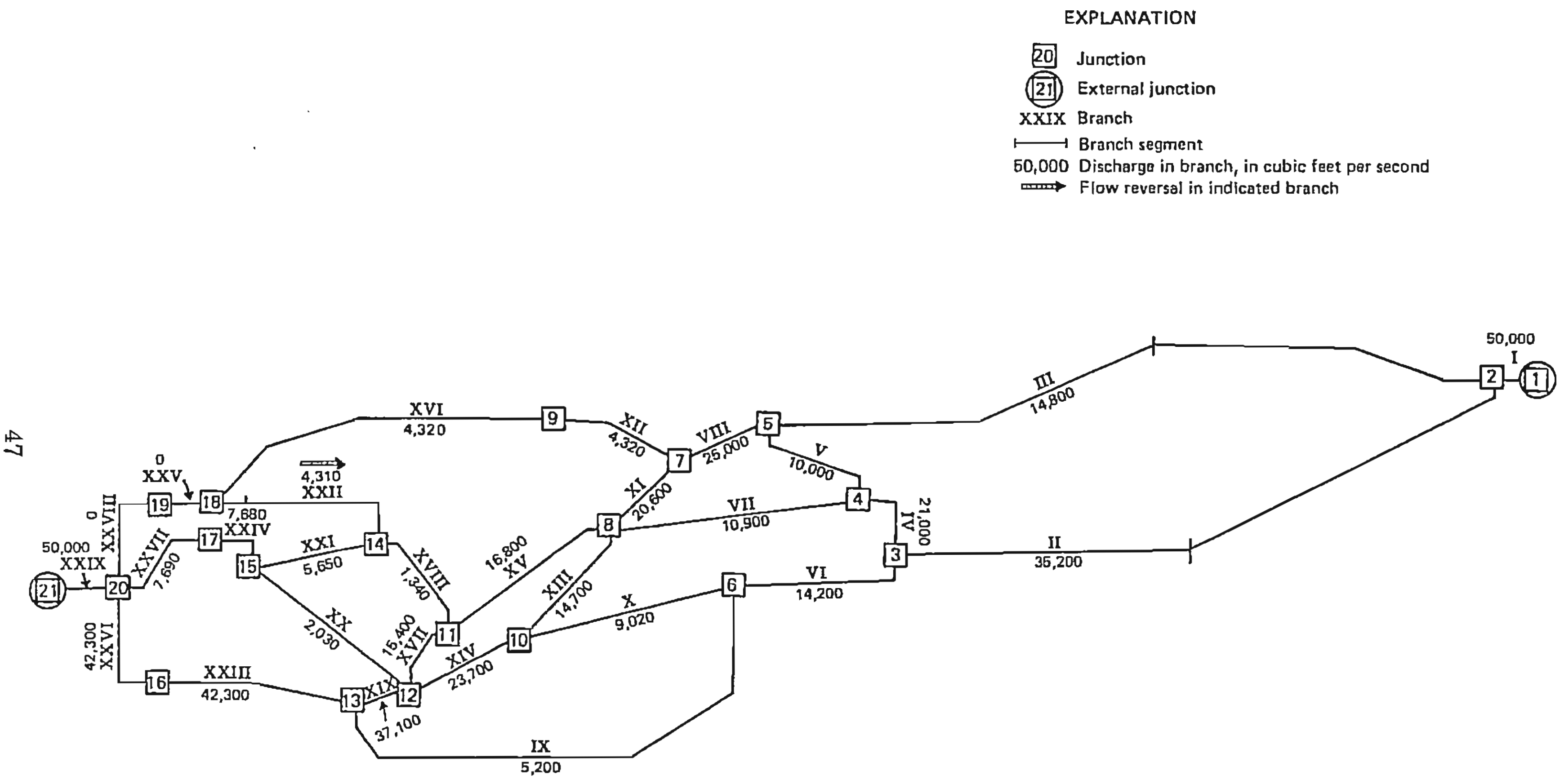

Figure 26.--Simulated distribution of flow through study reach with channel 3 closed (setup 3).

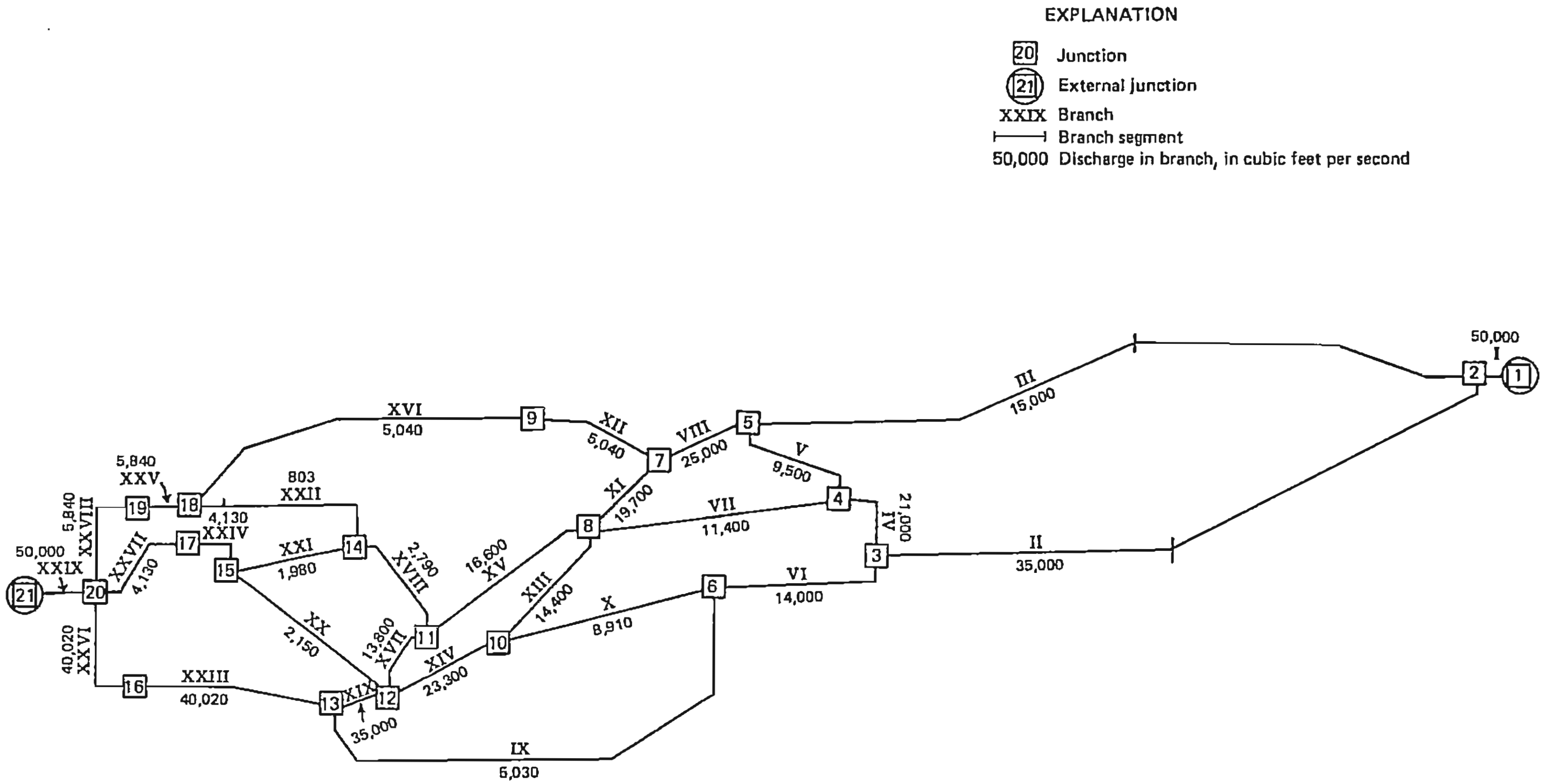

Figure 27.--Simulated distribution of flow through study reach with width of channel 3 reduced (setup 4).

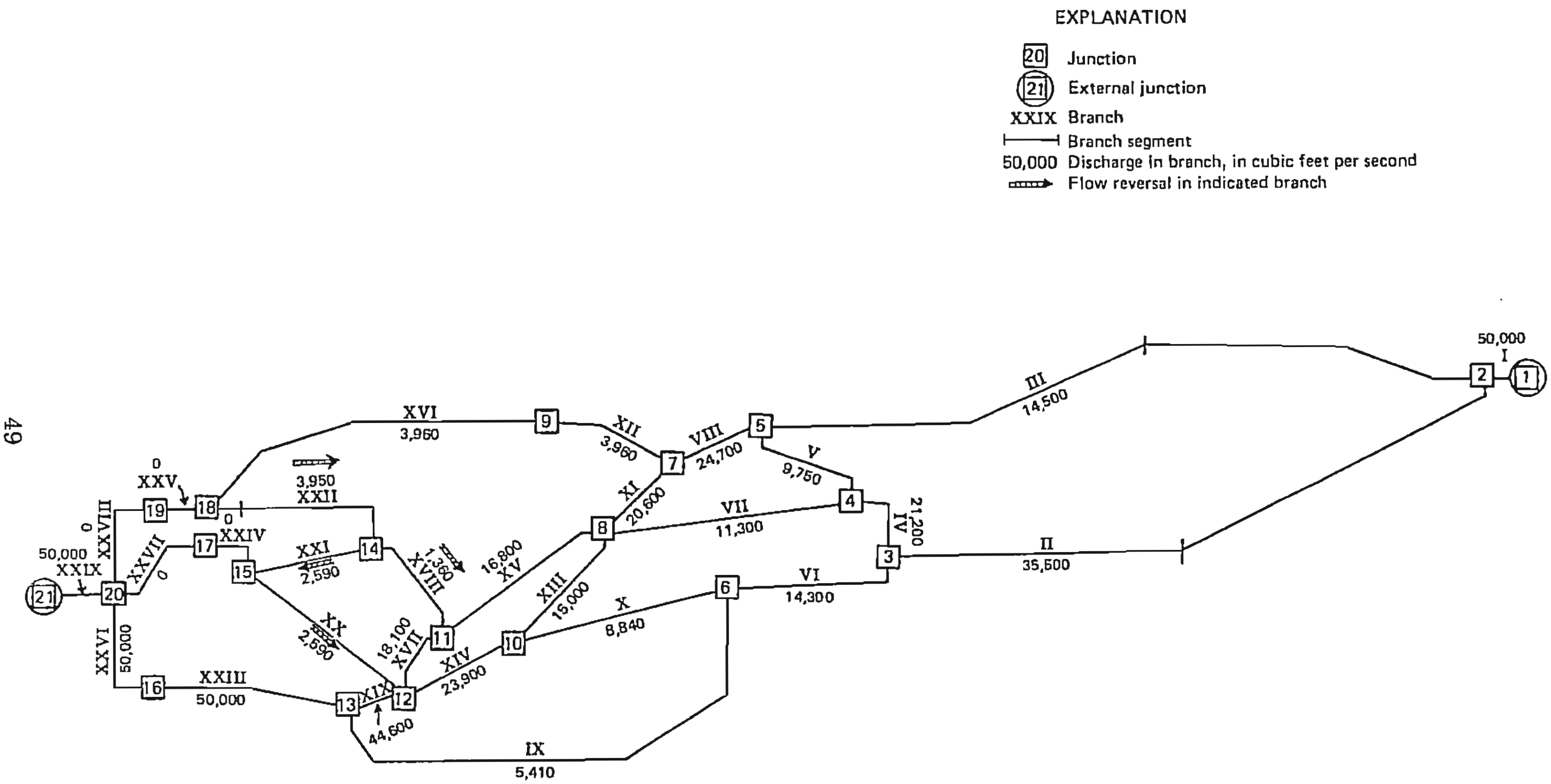

Figure 28.--Simulated distribution of flow through study reach with channels 2 and 3 closed (setup 5).

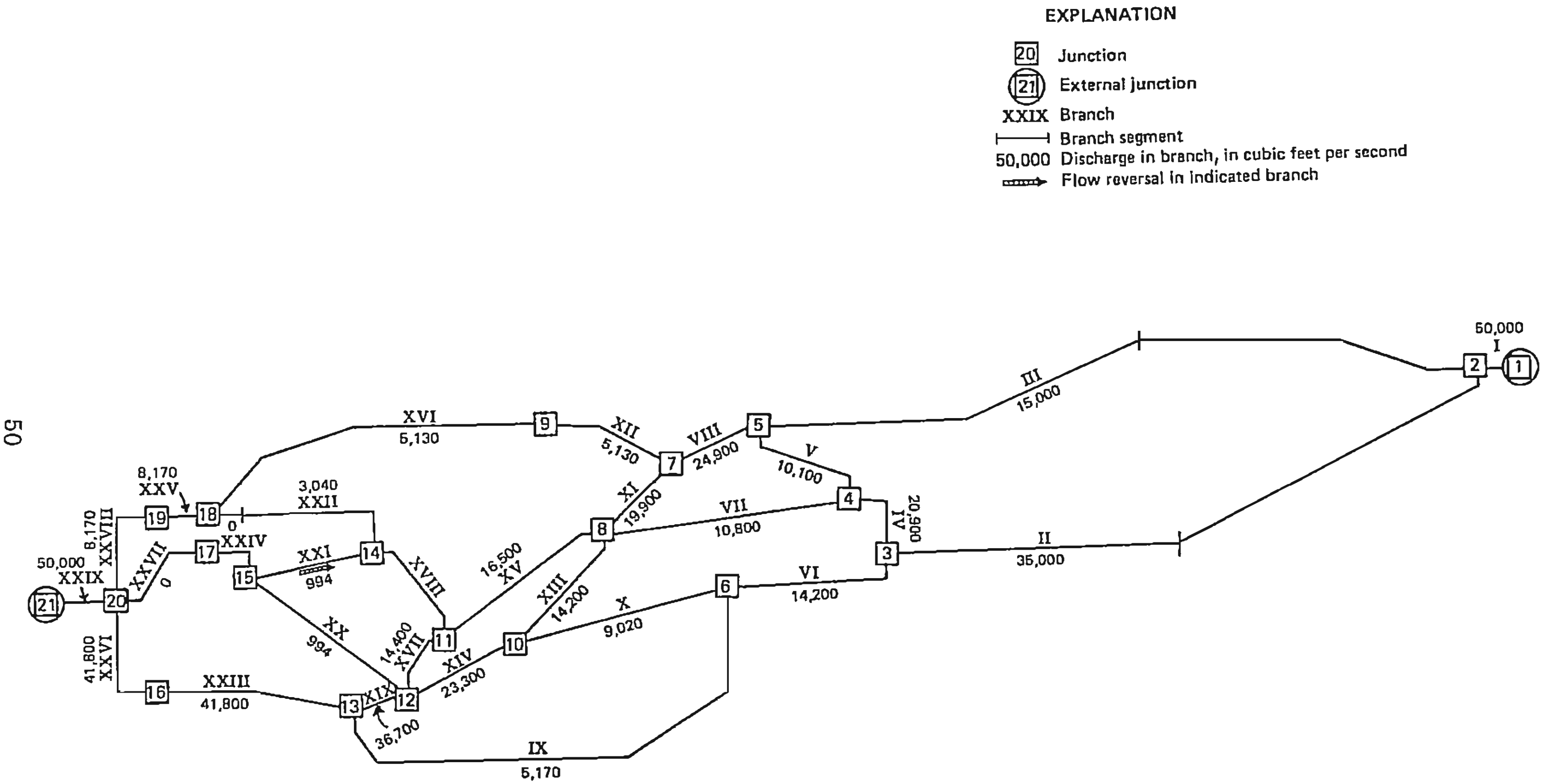

Figure 29.--Simulated distribution of flow through study reach with channel 2 closed and width of channel 3 reduced (setup 6).

completed, a failure to converge on a solution within the allowed number of iterations. These difficulties most likely resulted from the steep gradients and short segment lengths of the Matanuska River and the simplified schematization used to describe its complicated system of interconnected channels. After considerable effort to resolve these difficulties, the Matanuska River part of the model was deleted and calibration was successfully accomplished for the Knik River alone. At the lower end of the modeled reach, the Knik River is divided into three channels, each conveying a percentage of the total discharge.

Three sets of discharge data were available to calibrate the model. Only two sets were usable: one was used for calibration and the other for verification. Calibration parameters were adjusted until the simulated discharge values were within 10 percent of the measured values. Verification using the second data set produced results that compared closely with the measured data.

However, the model results could be improved with additional data and further refinement of calibration coefficients. Specific needs are as follows:

1. Additional stage and discharge measurements that cover a wider range of flow conditions for use in calibration and verification.

2. Utilization of a functional relation to define the channel roughness coefficient η rather than using a constant value.

3. Additional channel cross-section data at critical locations, such as constricting or expanding reaches, to improve the schematization of the modeled reach.

Two modeling alternatives were used. The first was based on a self-setting stage option provided by the branch-network model. By using this option, computed stages in the channels at the bridges (lower ends of the study reach) were substantially higher than measured stages. The second alternative involved extending the end of the study reach downstream from the bridges to a location where the river channels converged. This approach effectively overcame modeling problems experienced at the highway bridge locations and led to satisfactory results.

Experimentation with the model indicated that river flows of 40,000 ft^3/s or greater would override the effects of backwater from tides, but that the simulation terminated at a flow of 60,000 ft^3/s because of excessive overbank flow. Thus the model was run at flows of 40,000 and 50,000 ft^3/s using six different bridge opening configurations. These configurations included the existing three-bridge condition as well as constricted widths and (or) closure of one or both of the smaller openings. Model results indicated that significant changes in flow distribution and circulation, including some flow reversals, would occur in channels upstream from the highway when either one or both of the smaller openings (bridges) was closed.

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