SCOUR AT SELECTED BRIDGE SITES IN ALASKA

U. S. Geological Survey

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16. Abstracts
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By Vernon W. Norman

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UNITED STATES DEPARTMENT OF THE INTERIOR

GEOLOGICAL SURVEY

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CONTENTS

CONTENTS

Page

CONTENTS

ILLUSTRATIONS

ILLUSTRATIONS

ILLUSTRATIONS

ILLUSTRATIONS

TABLES

TABLES

Page

FACTORS FOR CONVERTING ENGLISH UNITS
TO INTERNATIONAL SYSTEM (SI) UNITS

The following factors may be used to convert the English units published herein to the International System of Units (SI).

Multiply English units	By	To obtain SI units
inches (in)	25.4	millimetres (mm)
feet (ft)	0.3048	metres (m)
cubic feet per second (ft^3/s)	.02832	cubic metres per second (m^3/s)
square miles (mi^2)	2.590	square kilometres (km^2)
miles	1.609	kilometres (km)
pounds (lb)	.4536	kilograms (kg)
feet per minute (ft/min)	.3048	metres per minute (m/min)
feet per second (ft/s)	.3048	metres per second (m/s)
ton (short)	.9072	tonne (t)
horsepower	745.7	watts

SCOUR AT SELECTED BRIDGE SITES IN ALASKA

By Vernon W. Norman

ABSTRACT

General scour at bridge crossings and local scour at bridge piers were measured at nine bridge sites in south-central and interior Alaska during the study period 1965-72. A detailed description of the physical setting, hydraulic characteristics, and channel geometry at low and high flows is given for each site to assist the reader in developing background knowledge on the scour phenomenon in various situations. Bridge openings ranged from 180 to 2,000 feet or 55 to 610 metres in width. Streambed material ranged from coarse sand to cobbles with most sites having predominantly gravel and cobbles. Periods of flood discharge, during which data were collected, had recurrence intervals which ranged from approximately 2 years at some sites to about 100 years on the Tanana River at Nenana.

Formulas to predict scour do not take into account all of the situations which can be encountered at a bridge site. Therefore, in order to obtain meaningful results judgment based on detailed studies of a site must be applied rather than the simple application of these formulas.

A comparison between measured and calculated mean depths of flow in contracted openings at three of the sites indicated that for streams whose beds are composed largely of gravel and cobbles the mean depth of flow in a contracted opening can be calculated from two established scour formulas to within 10 percent of the actual mean depth. At sites where no contraction occurred, there was virtually no general scour.

Measured local equilibrium scour depth and streambed-material sizes at bridge piers in this study and from four bridges described by other researchers were used to modify an existing pier-scour formula to better estimate maximum local equilibrium scour depth at round-or pointed-nosed piers alined with the flow.

Observations made during the study also showed:

1. In uniform or contracting reaches with fairly straight alinement the minimum streambed elevation tended to remain constant but its position migrated laterally with varying discharges. During floodflows, the depth of the minimum streambed elevation below the mean bed elevation was less than 60 percent of the mean depth.

2. Where dunes were present, the minimum streambed elevation of the scour hole at the nose of the pier fluctuated with a magnitude of about half that of the dune height, which corresponds to observations in model experiments.

The data suggest that local equilibrium scour depth at piers during a mean annual flood approaches that which might occur during floods of up to a 50-year recurrence interval. Further field study would be required to define more closely the effect of streambed-material size and pier-nose shape on the equilibrium depth of pier scour.

INTRODUCTION

Designing a bridge to cross waterways in alluvial channels requires some knowledge of the probability and magnitude of scour which might be expected. Aware of this fact, the design engineer in the past could often choose the location of a crossing, the length of span, and the alinement of the bridge to minimize the effects of scour.

Today, the alinement or other factors often govern where the crossing is to be located. This means the engineer needs to design for existing conditions. Also, the high cost of bridge construction forces him to consider the shortest spans possible which often results in constrictions of the approach channels to the bridge. Such constrictions cause an overall lowering of the streambed at the crossing. The design of the bridge needs to account for this general scour in addition to the local scour which may occur around the pier and abutments. The potential for scour during construction also needs to be considered.

To overdesign for scour can result in a more expensive structure than necessary. Yet to underdesign can result in the destruction of the structure, in loss of life, and in loss of revenue due to lack of transportation. As indicated by Laursen (1970), the risk of failure due to scour must be very small because the cost of failure is many times more than the cost of insuring against failure.

Considerable study has been made on the scour phenomenon. Karaki and Haynie (1963) have compiled an impressive bibliography of references from 1868 to 1963 on the subject. Since 1963 much additional work has been reported. Many of these studies have resulted in analytical or empirical formulas from laboratory experiments. In order to confirm the validity of the formulas for application to the design of actual structures, field measurements of scour are needed as indicated, for example, by Shen (1971), National Cooperative Highway Research Program (1970), Culbertson and others (1967), Neill (1970), and Sanden (1960).

In 1964 a study of scour at selected bridge sites in Alaska was begun. Its objective was to gather field-scour data to apply and if necessary to modify, existing scour formulas used by some bridge designers. At the time the project was started, the only known field measurement of scour at piers was that reported by Laursen and Toch (1956). Since then, other field measurements have been reported by Neill (1965, 1970), Breusers (1970), Shen and others (1969), and Larras (*in* Shen and others, 1969, p. 1935). Results of this study were used in conjunction with the recorded field measurements to evaluate and modify existing design formulas.

The purpose of this report is to describe the results of data collection at these bridge sites and compare the results with existing laboratory and field data and with those results predicted from selected scour formulas. The case histories of scour are illustrated in detail in order that the reader can compare them with similar situations with which he may be working.

All indications of scour were considered to be related to either channel contraction (general scour) or localized flow conditions at piers and abutments (local scour). General scour takes place in a contracted reach due to an increase in stream velocity in the contraction over that in the approach or uncontracted reach. The velocity is greater because the reduction in width decreases the area of flow and, in order to maintain continuity of discharge from the approach reach, the velocity in the contracted reach must increase to compensate for the reduced area. This higher velocity means the stream's capacity to move the bed material is also increased. Hence, in the contracted reach the stream not only can transport the bed material supplied from upstream, but also can remove bed material from the reach itself, which lowers the streambed. Generally the scour process continues until the area in the contracted reach is approximately that of the approach reach or until a highly resistant bottom is encountered.

Local scour occurs at a pier because the presence of the pier creates vortices with both vertical and transverse velocity components near the streambed. These localized velocity components, or systems, are sufficient to remove bed material near the pier and either force it downstream, where some of it is deposited behind the pier, or force it laterally onto the streambed where the undisturbed streamflow can move it out of the bridge opening. Shen and others (1969) and Neill (1970) can provide the reader with more detailed descriptions of the local scour process.

Degradation or aggradation of a streambed over a long reach is generally a slow process and is not considered in this study; however, the geomorphic aspects of channel behavior are described where applicable.

It must be remembered that a river is a dynamic feature of the landscape and what may appear to be a "stable" reach of river over a short period of time (say 10 to 20 years) can change over a longer period of time. Geomorphic changes can occur at a slow rate, such as general aggradation or degradation resulting from natural or manmade features located many miles from a bridge site, or they can occur rapidly. Two examples of rapid changes are the response of a river in the immediate vicinity, both upstream and downstream, of a newly built dam and the response of a river in the vicinity of a meander cutoff. Because of the potential for change, not only should the reach of the river be studied in detail at a bridge site, but also it must be considered as a part of the entire river system.

Scour at bridges can be divided into two more types: that which occurs when little or no bedload is delivered to the scour hole from upstream, and that which occurs when upstream bedload transport continuously delivers bedload to the scour hole. With the first type, scour proceeds so long as the local flow has the capacity to remove material from the scour hole. Relief bridges on flood plains might be expected to be subjected to scour under this condition. With the second type, scour continues until the local flow has only the capacity to remove the material from the scour hole that is supplied to it from upstream. For details on the theory and limitations of these two types of scour, the reader should refer to Laursen (1958, 1962) and Shen and others (1969).

With the exception of scour at the Moose Creek bridge site, all of the scour conditions described in this study probably occurred when there was significant bedload transport throughout the streams and Froude Numbers were less than 1. Bedload transport is assumed because scour at high flows was accompanied afterwards by fill, and the source of sediment supply, being the bed material upstream, seemed practically unlimited. Streambed materials were coarse (gravel and larger) with generally large ranges in sizes. The scour process reasonably would involve some selective transport of the finer particles in the streambed material as bedload, leaving the bed surface somewhat armored with the larger particles. Also, except for the Snow River site, all measured floods were within bankfull stage.

Detailed summaries of other researchers' work and the development of scour formulas are not included in this report. For interested readers, Shen (1971) and Neill (1970) present some of the latest reviews and evaluations of past work. A brief discussion of the development of scour formulas and their use in this report is presented in the treatment of the Susitna River site.

This project was a cooperative effort between the U.S. Geological Survey, Alaska Department of Highways, and Federal Highway Administration.

METHOD OF INVESTIGATION

Selected Sites

Field work began in 1965 with an intensive study of the bridge opening on the Knik River at the Old Glenn Highway crossing. This site was selected in the beginning because many annual floods of high discharge, resulting from the breakout of lake waters trapped annually behind the Knik Glacier, had occurred and were expected to continue. Post and Mayo (1971) describe the phenomenon. However, as of 1973, the last breakout occurred in 1966. Thus 2 years of peak flows were all that could be studied on the Knik River. A less intensive study than on the Knik River was also made in 1965 at the Susitna River at the Anchorage-Fairbanks Highway crossing. In 1966 a study was conducted at the Knik River at the New Glenn Highway crossing. From the experience gained at these sites, it was concluded that less intensive studies, using mobile equipment at more sites would better meet the objectives of the project. The highway bridge opening on the Tanana River at Nenana was added to the list of sites in 1967. During 1968 and 1969, seven more sites were investigated and included in the network, bringing the total number of sites to 11. The final network of sites is shown in figure 1.

Parameters evaluated in selecting a site for study included the following:

1. Size and shape of piers.
2. Direction of flow with respect to piers.
3. Contraction of bridge opening.
4. Size of bed material.
5. Flood potential.
6. Accessibility of site and practicality of collecting data.

Floods were not measured at either the Chatanika River bridge site or the Chena River bridge site at Fairbanks because of lack of significant flooding or available personnel.

Insufficient data were obtained at the Chena River near Two Rivers and the Moose Creek site. Discussion of these two sites and the available information is contained in the appendix.

Floodflow and Low-Flow Data

For purpose of this study information was obtained from floods (peak discharges) greater than or equal to the mean annual flood as determined from past discharge records or simply estimated. Floods greater than or equal to the assigned mean annual discharge were considered significant floods.

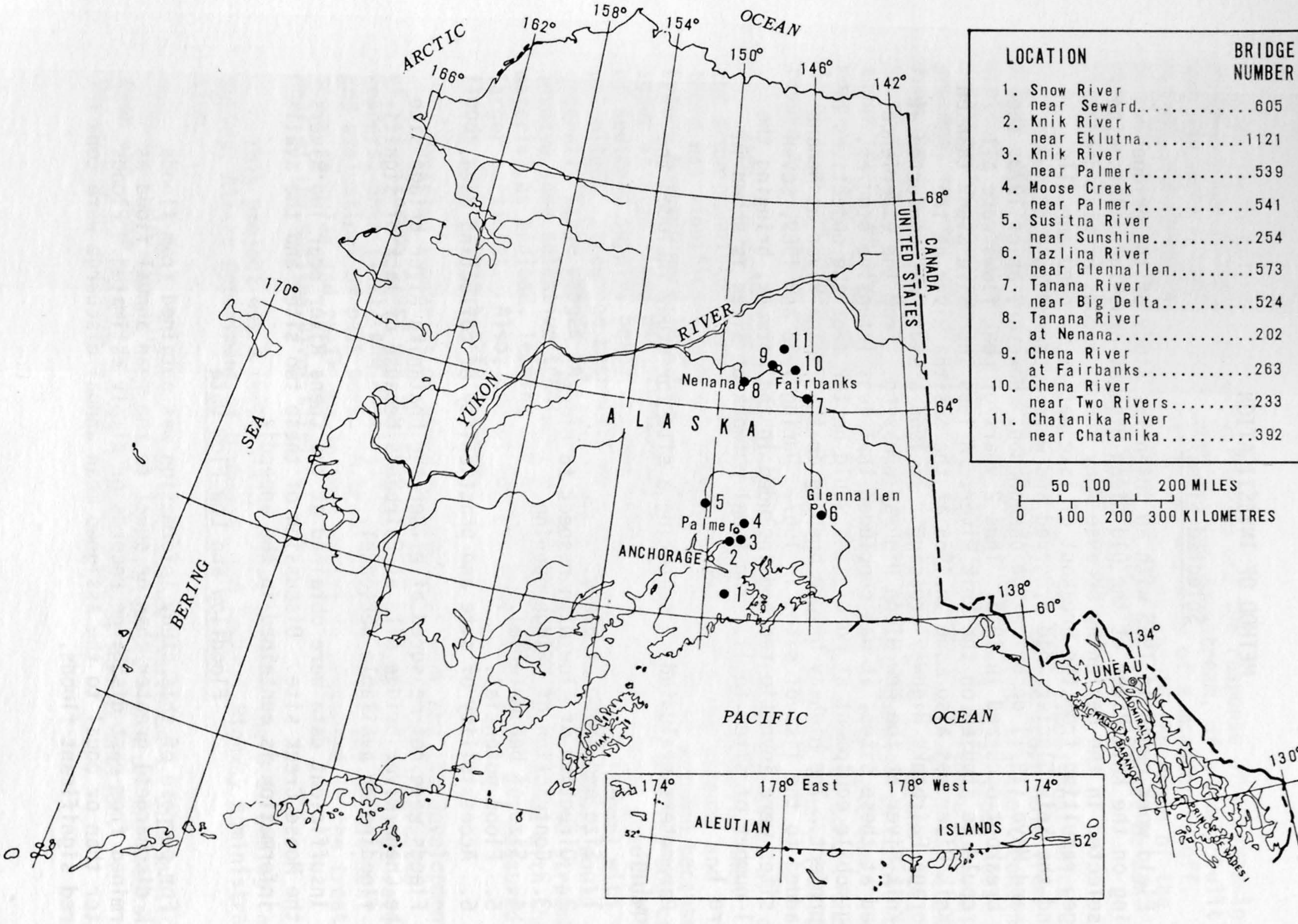


Figure 1.-- Scour surveillance sites.

Cross-sectional profiles were also obtained during low flows to determine the probable maximum height of streambeds and the depths of residual scour near piers. This low-flow information may also be useful to designers because often the only cross-sectional profiles available to them are obtained during low or medium flows. To define the scour process with time, data were collected also, when possible, prior to and (or) after the peak of a significant flood.

Instrumentation and Equipment

Soundings to the streambed to determine cross-sectional profiles, longitudinal profiles, and scour hole depths were generally obtained with a Raytheon Model DE119 D[1/] recording fathometer. Transducers used with the fathometer produced an 8° beam width. Transducers were mounted permanently at the nose, tail, and on both sides of pier 5 of bridge 539, on the Knik River, to monitor the change in depth of the scour hole around the pier. The only other fixed installation of a transducer was on the nose of pier 4 on bridge 1121, also on the Knik River. In addition, a transducer was held by hand along the side of a boat to measure cross-sectional and longitudinal profiles. Air bubbles created by the turbulence around the hand-held transducer caused poor readings on the recorder. More adequate and precise records of depth were obtained after a transducer was permanently mounted flush with the outside and in the center of the boat hull about 5 ft (1.5 m[2/]) in front of the transom.

The boat most often used was a 20-ft (6.1-m) doublehulled fiberglass boat with two 35-horsepower (26,100-watt) outboard engines (fig. 2). The bow of the boat was fitted with a boom and sheave arrangement. A "B" reel with conductor cable capable of handling 200-lb (91-kg) sounding weights was mounted to the boom.

Equipment used in taking soundings, in measuring velocities, and in making discharge measurements was standard U.S. Geological Survey equipment as described by Buchanan and Somers (1969). It consists basically of the "B" reel, Price Type AA current meters, and sounding weights ranging from 50 to 100 lb (22 to 45 kg). Stream widths and cross-sectional stationings were obtained by using a tagline (marked cable) or a sextant with a measured base line on shore.

Suspended-sediment samples were obtained with a model US-P-61 sampler, a 100-lb (45-kg) streamlined sampler which collects a sample of water-sediment mixture at ambient stream velocities. It is electrically operated so that samples can be obtained from single points at various depths and velocities. The sampler was described by Guy and Norman (1970) and by the U.S. Inter-Agency Committee on Water Resources (1963).

1/ Use of brand names in this report is for identification only and does not constitute endorsement by the U.S. Geological Survey.
2/ All measurements except sediment particle sizes were originally made in English units and converted to SI units.

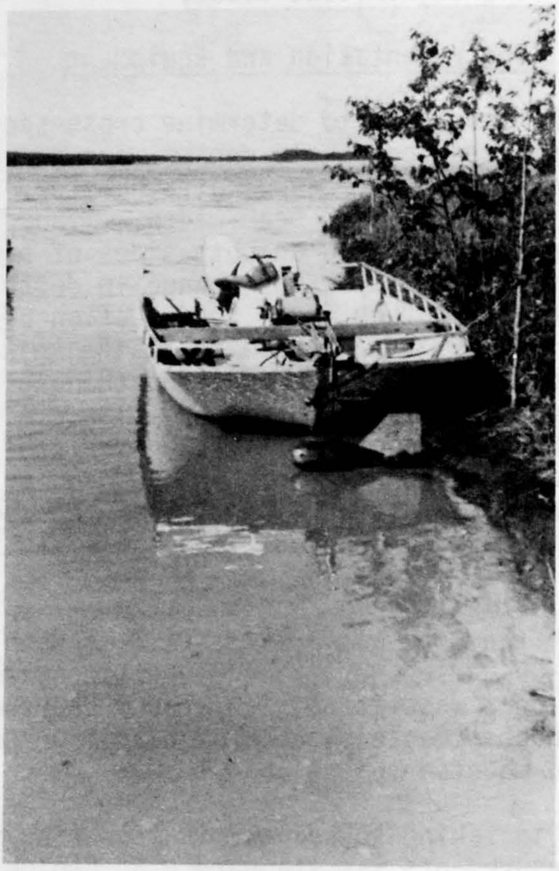


Figure 2.-- Project boat showing current meter, sounding weight and 'B' reel used in scour study.

Streambed material samples were collected using samplers appropriate to stream velocities. Streambeds of sand and small gravel were sampled with the US-BM 54, a 100-lb (45-kg) sampler which by spring action scoops up about 6.18×10^{-2} ft^3 (1.75×10^{-4} m^3) of bed material when it touches the streambed. This sampler too was described by Guy and Norman (1970) and by the U.S. Inter-Agency Committee on Water Resources (1963). A locally constructed drag sampler was used in conjunction with a 100-lb (45-kg) sounding weight to sample bed material from gravel and cobble bed streams (fig. 3). Photographs of exposed bed material in gravel and cobble bed streams during low-flow conditions were taken and analyzed by procedures described in Ritter and Helley (1969).

Water-surface elevations were measured using standard surveying instruments and techniques. Permanent reference marks were established at each of the sites. The end points of the wide cross sections on the Knik River at bridge 539 and the Susitna River at bridge 254 were also marked with large painted plywood markers for easy visibility.

River stages were recorded at some sites by automatic recorders. Stages at other locations were obtained by measuring down from a fixed point of known elevation on the bridge to the water surface with a wire weight gage or simply using a weighted measuring tape. Stage measuring equipment is described by Buchanan and Somers (1968).

Data Collection

Water Stage and Discharge

Water stage (elevation) was determined to establish a reference elevation from which streambed elevation could be determined from depth soundings. Recorded stage or periodic stage readings with time at a bridge defined the magnitude and shape of the flood hydrograph. When correlated with discharge measurements at various stages, a rating curve or table could be established. (See Buchanan and Somers, 1969, for methods of measuring discharge.) Using this relationship between stage and discharge, stage readings could be converted to discharge without having to make continuous discharge measurements.

Stage at the various cross sections at a site along with supplementary water-surface elevations between cross-sectional profiles were used to determine water-surface profiles and slope. Water-surface profiles were used to indicate the presence or absence of backwater effects due to channel constriction at the bridge crossing. Slope can be used in various hydraulic formulas to calculate other values such as roughness coefficients and sediment transport rates.

Flood frequency figures given for some of the sites were computed from daily discharge records using the Log Pearson Type III analysis described by Benson (1968).

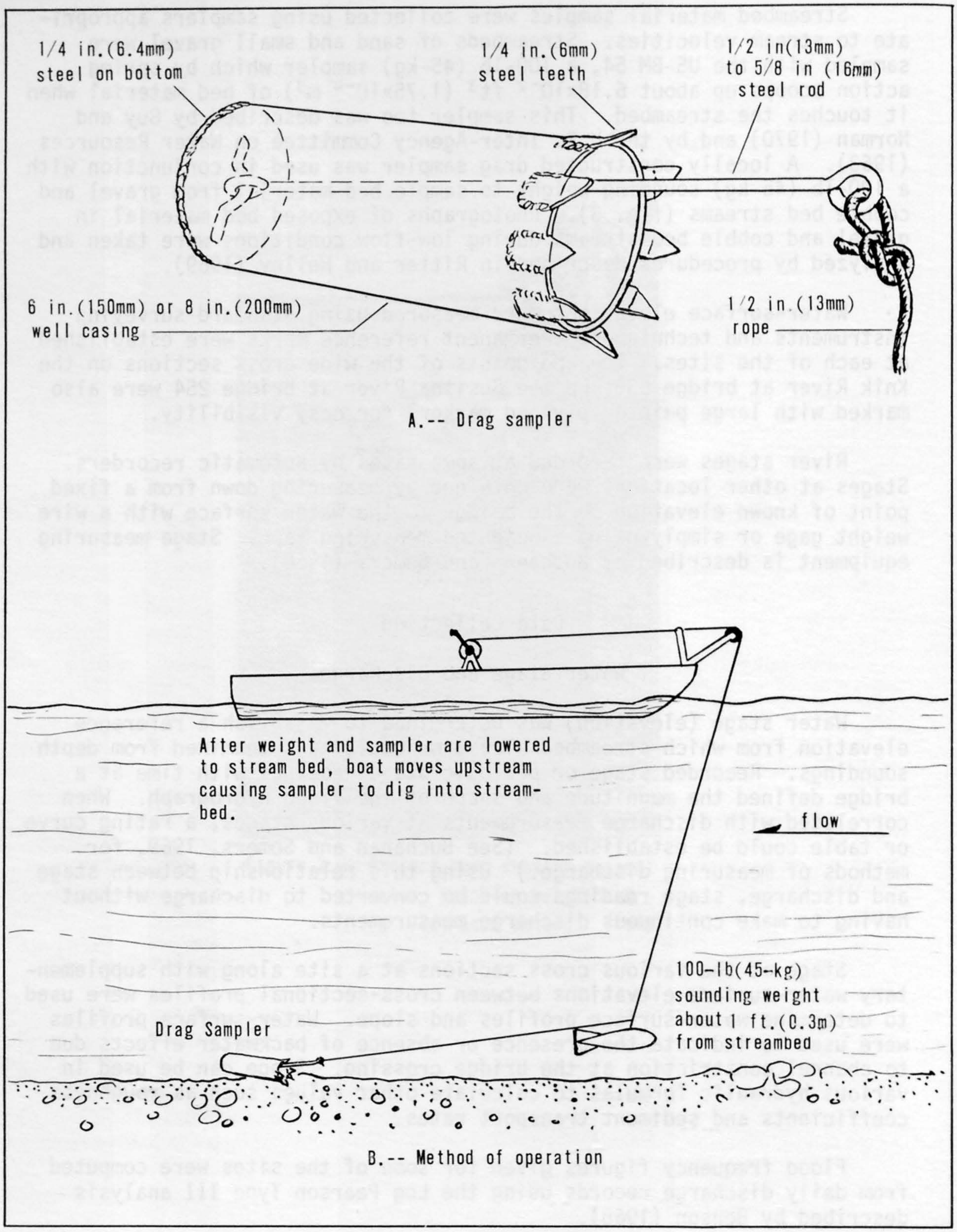


Figure 3.-- Drag sampler used to collect bed-material samples from streambeds of predominantly gravel and cobbles.

Stream Velocity

Velocities were measured at selected points across the streams to determine their lateral distribution and variation. These include those made as an integral part of discharge measurements. (See Buchanan and Somers, 1969, for methods of measuring velocity.) To describe the velocity profile with depth, velocity was measured also at several points in a vertical section. The angle of attack and magnitude of the approach velocity to the piers were measured; these are important parameters in the development of local scour at piers.

Widths and Depths

The widths of cross sections and the depths to the streambed at selected intervals throughout the cross-sectional profiles were measured and define the channel shapes. The difference in surface widths of two adjacent cross sections measured at the same time was considered a measure of contraction or expansion in the channel. The left and right convention used in this report is defined as that direction (left or right) as viewed when facing downstream. Depth is used to determine the bed elevation. Difference in bed elevation at a point or as a mean for the cross section was used as a measure of scour or fill.

Sediments

The size of streambed materials has been considered important in the scour phenomenon although its effect had not been clearly defined. Most investigators have diverse opinions relating to the effect of particle size and have worked only with sand-size particles in the past.

Particle size and concentration of suspended sediments were determined, using the methods described by Guy (1969), and are included in the report only in an attempt to describe further the sediments being transported. At the beginning of the study an attempt was made to collect suspended and bed material data in both the approach and bridge opening sections. The purpose was to compute total sediment discharges in order to describe sediment transport "budgets" of sediment being transported into and out of the bridge openings. Physical limitations in collecting precise and timely streambed material samples proved to be a deterrent to obtaining meaningful results.

Streambed Form

To describe the shape or form of the streambed, longitudinal profiles were obtained at some of the sites with the recording fathometer. The effect of dunes on the depth of scour at piers is to decrease and increase alternately the bed elevation as the dune waves move past the piers. A similar effect occurs when bars are present on the bed. Knowing the streambed form aided in interpreting the observed local scour at piers.

Channel Shapes

Photographs and surveys were used to aid in interpreting channel patterns, variations in channel cross-sectional shapes, and velocity distributions.

Bridge Construction

Records of test borings, cross-section soundings and as-built dimensions of the bridges were obtained from the Alaska Department of Highways. They were used in computation of scour and interpreting data.

DESCRIPTIONS, SUMMARY OF OBSERVATIONS, AND DISCUSSION OF RESULTS AT STUDY SITES

Susitna River near Sunshine - Bridge 254

Description

This study site is located on the Susitna River at mile 104 of the Anchorage-Fairbanks Highway, about 3.2 mi (5.2 km) west of Sunshine and 10.5 mi (16.9 km) downstream from the mouth of the Talkeetna River. An aerial photograph of the site is shown in figure 4. The bridge is 1,072 ft (327 m) long and supported by four piers spaced 250 ft (76 m) between centers. Figure 5A gives an elevation view of the bridge. Pier shapes and dimensions are shown in figure 5B. At high flows the piers are approximately alined with the flow. The location and general shape of the cross sections are shown in a plan view (fig. 6).

The Susitna River basin above the bridge site covers about 11,500 mi^2 (29,800 km^2). Less than 15 percent of the area is occupied by glaciers. In the vicinity of the bridge site the river channel is braided and consists of multiple bars and islands. Surface bed material consists of gravel and cobbles and some sand.

Floodflows on the Susitna result from snowmelt in the spring and from rainfall combined with glacial melt water in mid to late summer. Records of flood data have been collected about 50 mi (80 km) upstream on the Susitna River at Gold Creek since 1949 (U.S. Geological Survey, 1972). During these 23 years the peak flow occurred June 7, 1964. In 14 of these years, floods occurred in June.

A mean annual flood for the scour site was estimated to be about 80,000 ft^3/s (2,300 m^3/s). The highest discharge during which scour data was collected was 171,000 ft^3/s (4,840 m^3/s) on August 11, 1971, at a gage height of 60.8 ft (18.53 m). This was about 1.2 ft (0.37 m) lower than the peak of approximately 62.0 ft (18.90 m) which occurred the night of August 10. Using the flood frequency information from

Figure 4.-- Aerial view of Susitna River near Sunshine, bridge 254, May 21, 1968.

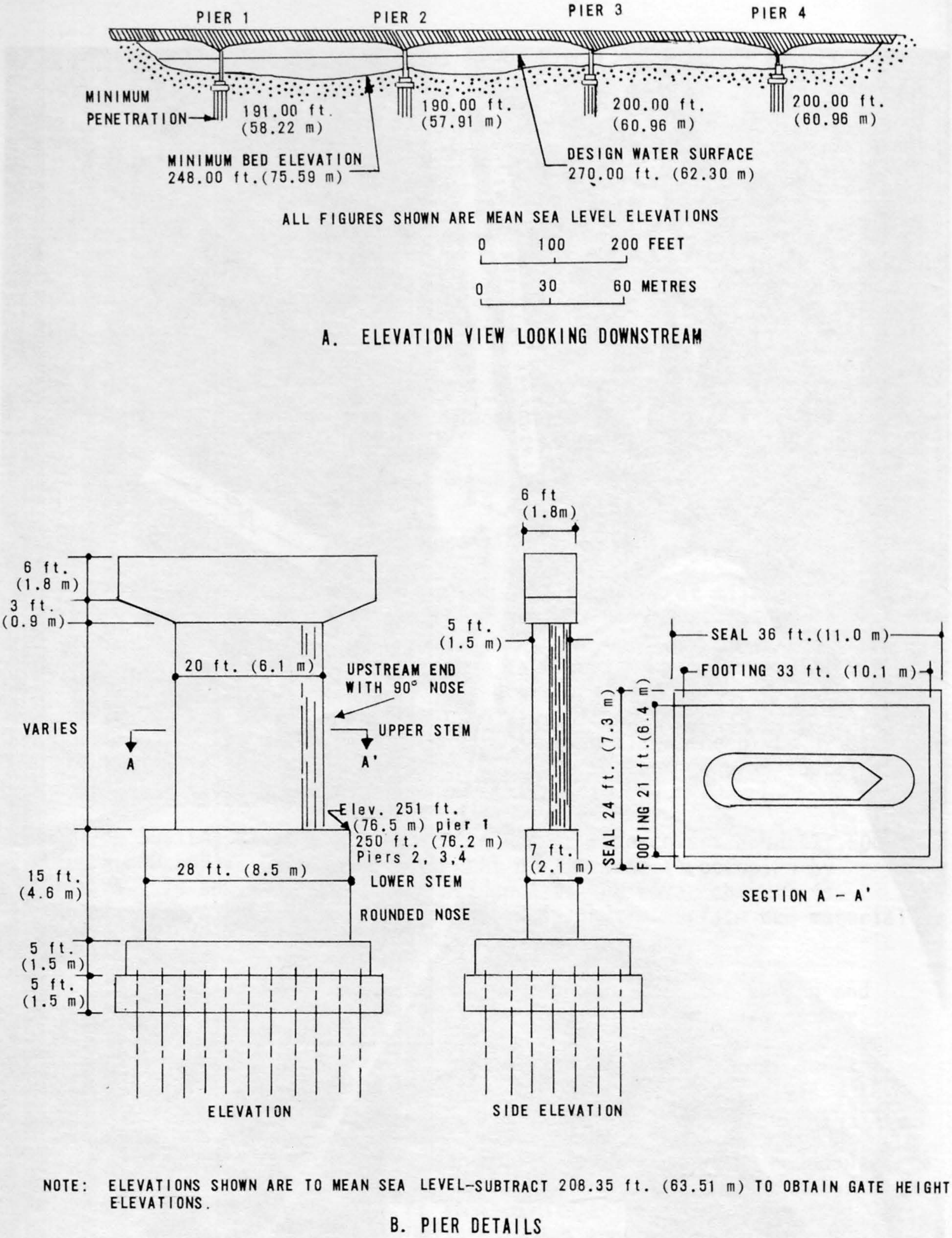


Figure 5.--Dimensions of Susitna River bridge 254.

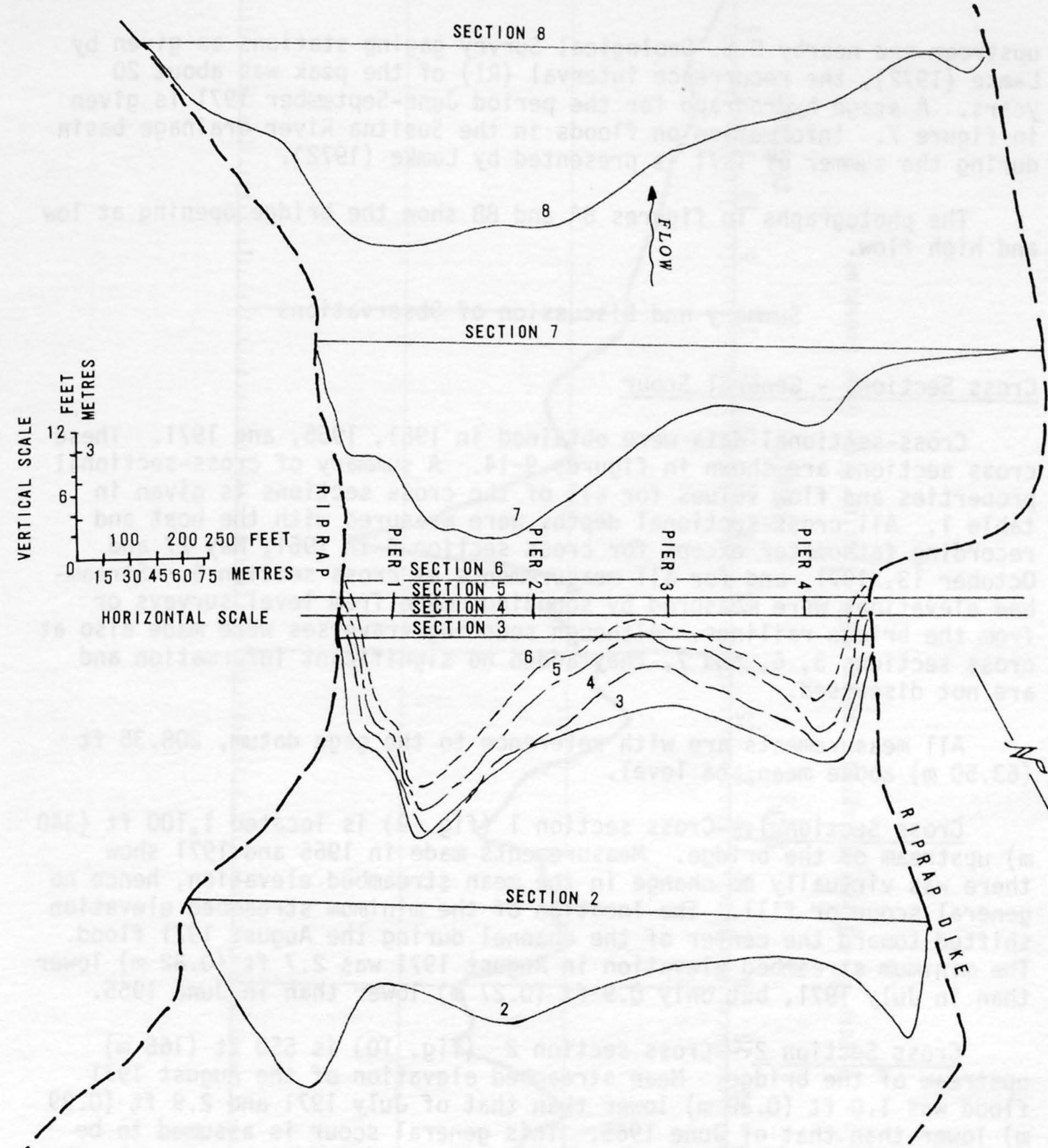


Figure 6.-- Location and general shape of cross sections, Susitna River near Sunshine.

upstream and nearby U.S. Geological Survey gaging stations as given by Lamke (1972), the recurrence interval (RI) of the peak was about 20 years. A stage hydrograph for the period June-September 1971 is given in figure 7. Information on floods in the Susitna River drainage basin during the summer of 1971 is presented by Lamke (1972).

The photographs in figures 8A and 8B show the bridge opening at low and high flow.

Summary and Discussion of Observations

Cross Sections - General Scour

Cross-sectional data were obtained in 1961, 1965, and 1971. These cross sections are shown in figures 9-14. A summary of cross-sectional properties and flow values for all of the cross sections is given in table 1. All cross-sectional depths were measured with the boat and recording fathometer except for cross section 4 in 1961, May 27 and October 13, 1971, and for all measurements at cross section 5. Streambed elevations were measured by soundings made from level surveys or from the bridge railings. Although sounding traverses were made also at cross sections 3, 6, and 7, they added no significant information and are not discussed.

All measurements are with reference to the gage datum, 208.35 ft (63.50 m) above mean sea level.

Cross Section 1.--Cross section 1 (fig. 9) is located 1,100 ft (340 m) upstream of the bridge. Measurements made in 1965 and 1971 show there was virtually no change in the mean streambed elevation, hence no general scour or fill. The location of the minimum streambed elevation shifted toward the center of the channel during the August 1971 flood. The minimum streambed elevation in August 1971 was 2.7 ft (0.82 m) lower than in July 1971, but only 0.9 ft (0.27 m) lower than in June 1965.

Cross Section 2.--Cross section 2 (fig. 10) is 550 ft (168 m) upstream of the bridge. Mean streambed elevation of the August 1971 flood was 1.0 ft (0.30 m) lower than that of July 1971 and 2.9 ft (0.99 m) lower than that of June 1965. This general scour is assumed to be the direct result of the width constriction imposed on the river by the riprap dike on the right bank (see fig. 6). As in cross section 1, the minimum streambed elevation in August 1971 shifted to the right toward the center of the channel.

Cross Section 4.--The measurement made in March 1961 along the center line of the roadway during low flow is compared in figure 11 with the low-flow measurement of May 1971, some 15 ft (4.6 m) upstream at cross section 4. The significant difference between the two is the scour along the right bank. The maximum difference in streambed elevation between the two measurements near the right bank is about 8 ft

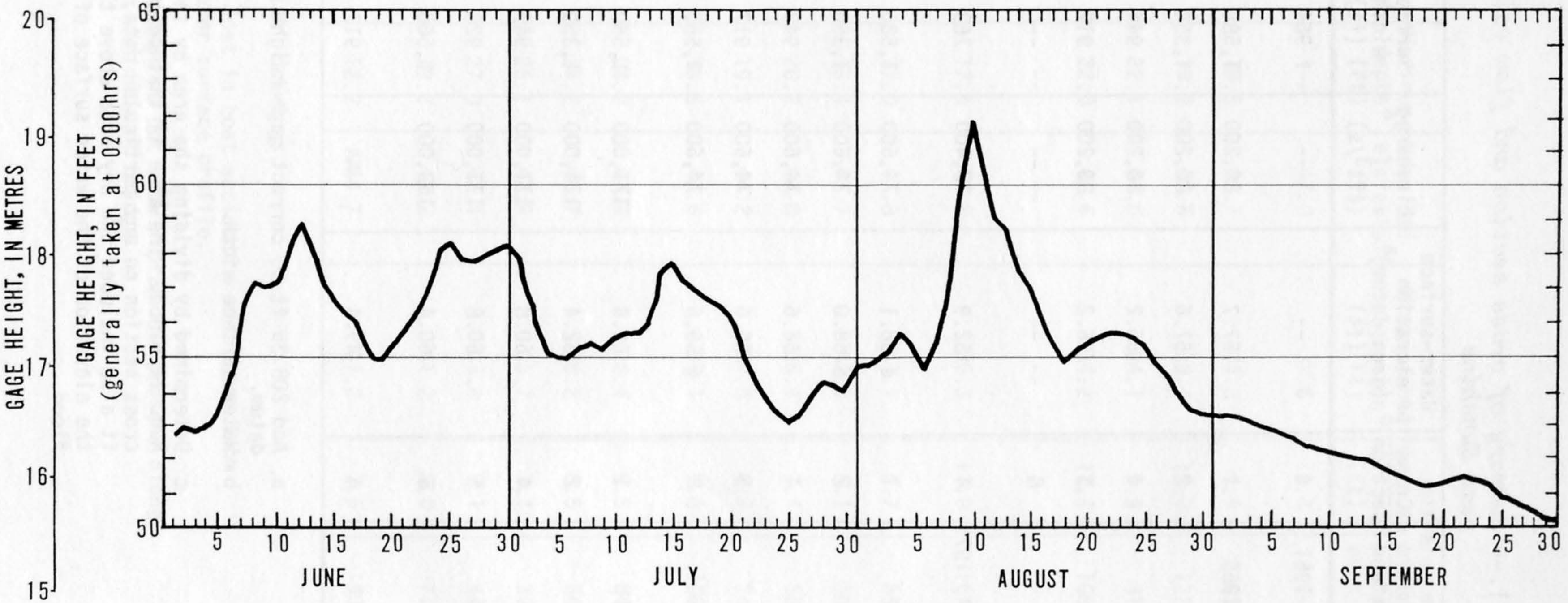


Figure 7.-- Stage hydrograph, summer 1971, Susitna River near Sunshine.

Table 1.-- *Summary of cross section and flow values, Susitna River near Sunshine*

Date	Cross section	Water-surface elevation (gage datum)[a] (ft)	Discharge (ft^3/s)	Surface width [b] (ft)	Wetted area (ft^2)	Mean velocity (ft/s)
March 1961	4	--	--	f 964	14,100	--
June 1965	1	57.7	80,200	1,580	15,100	5.3
	2	57.6	80,200	1,350	10,700	7.5
	4	56.2	80,200	944	11,600	6.9
	5	56.2	80,200	912	9,600	8.4
	8	--	--	--	--	--
May 27, 1971	4	52.9	37,400	763	6,240	6.0
July 2	1	e 58.1	74,600	1,580	16,300	4.6
	2	e 58.0	74,600	1,350	13,900	5.4
	4	56.6	74,600	949	12,100	6.2
	5	56.6	74,600	917	11,300	6.6
	8	e 55.6	74,600	1,560	12,500	6.0
Aug. 11	1	62.6	171,000	1,580	23,300	7.3
	2	62.4	171,000	1,350	21,200	8.1
	4	60.8	171,000	949	17,100	10.0
	5	60.8	171,000	928	17,100	10.0
	8	60.4	162,000	1,560	17,500	9.2
Oct. 13	4	49.4	NA	917	4,680	NA

a Add 208.35 ft to correct gage height to mean sea level datum.

b Water surface width.

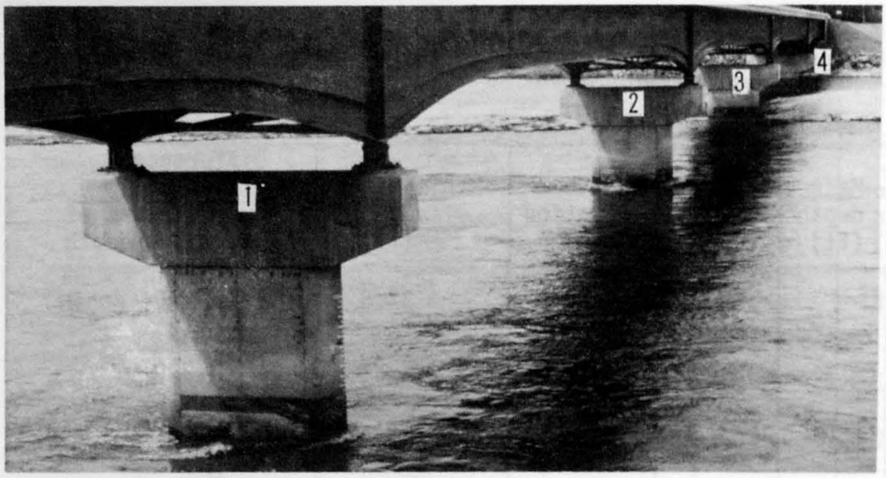
c Determined by dividing the area by the width obtained in Aug. 11 flood. The area is that described by the sounded cross section on any particular date, but for lower flows it also included the dry area above the water surface to the elevation of the water surface of the Aug. 11, 1971 flood.

Maximum mean velocity (ft/s)	Mean depth[c] (ft)	Maximum depth[a] (ft)	Mean bed elevation (ft)	Minimum bed elevation[a] (ft)	Difference mean to minimum (ft)	percent of mean depth
--	--	--	47.0	37.6	9.4	--
--	9.6	18.5	48.2	39.3	8.9	93
--	8.0	18.2	49.6	39.4	10.2	127
--	12.2	22.1	44.0	34.1	9.9	81
10.1	10.6	22.0	45.6	34.2	11.4	108
--	--	--	--	--	--	--
8.7	6.7	17.6	46.2	35.3	11.0	167
--	10.3	17.0	47.8	41.1	6.7	65
--	10.3	15.4	47.7	42.6	5.1	50
--	12.8	20.2	43.8	36.4	7.4	58
8.7	12.3	19.1	44.2	37.5	6.7	54
--	8.0	16.5	47.6	39.1	8.5	106
--	14.8	24.2	47.9	38.4	9.5	64
--	15.7	24.2	46.7	38.2	8.5	54
--	18.0	25.1	42.8	35.7	7.1	39
11.2	18.4	27.0	42.4	33.8	8.1	44
--	11.2	19.2	49.2	41.2	8.0	71
NA	4.7	12.2	44.7	37.2	7.5	160

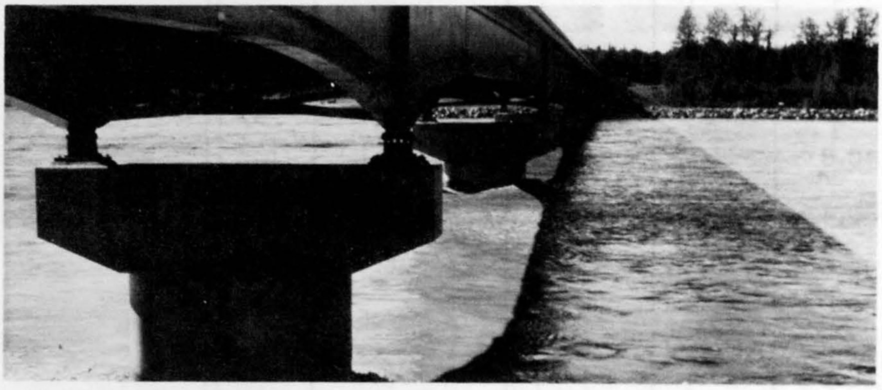
d From "as built" drawings.

e Level notes lost in boat accident - estimated from June 1965 water surface profile.

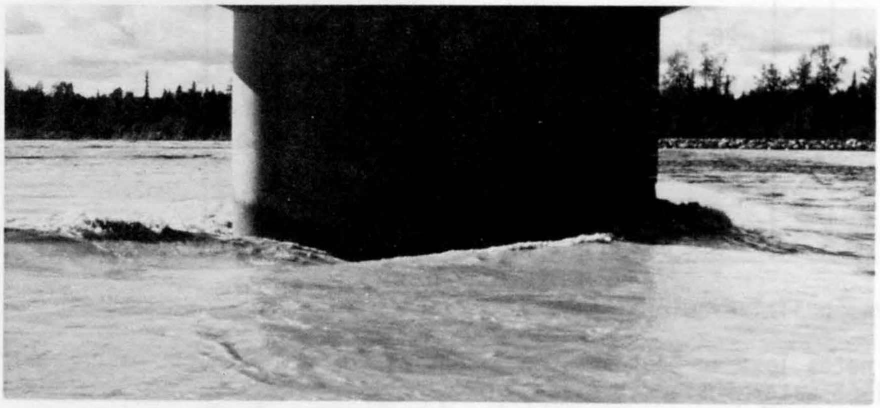
f As-built drawings do not show riprap now present along left bank in vicinity of bridge.



A. May 27,1971, Q=37,400 ft.^{3}/s (1,060 m^3/s)
Note bar at pier 3.



B. August 11,1971, Q=171,000 ft.^{3}/s (4,840 m^3/s)



C. August 11,1971, Q=171,000 ft.^{3}/s (4,840 m^3/s)
Close-up of water surface profile at pier 1.

Figure 8.-- Susitna River near Sunshine.

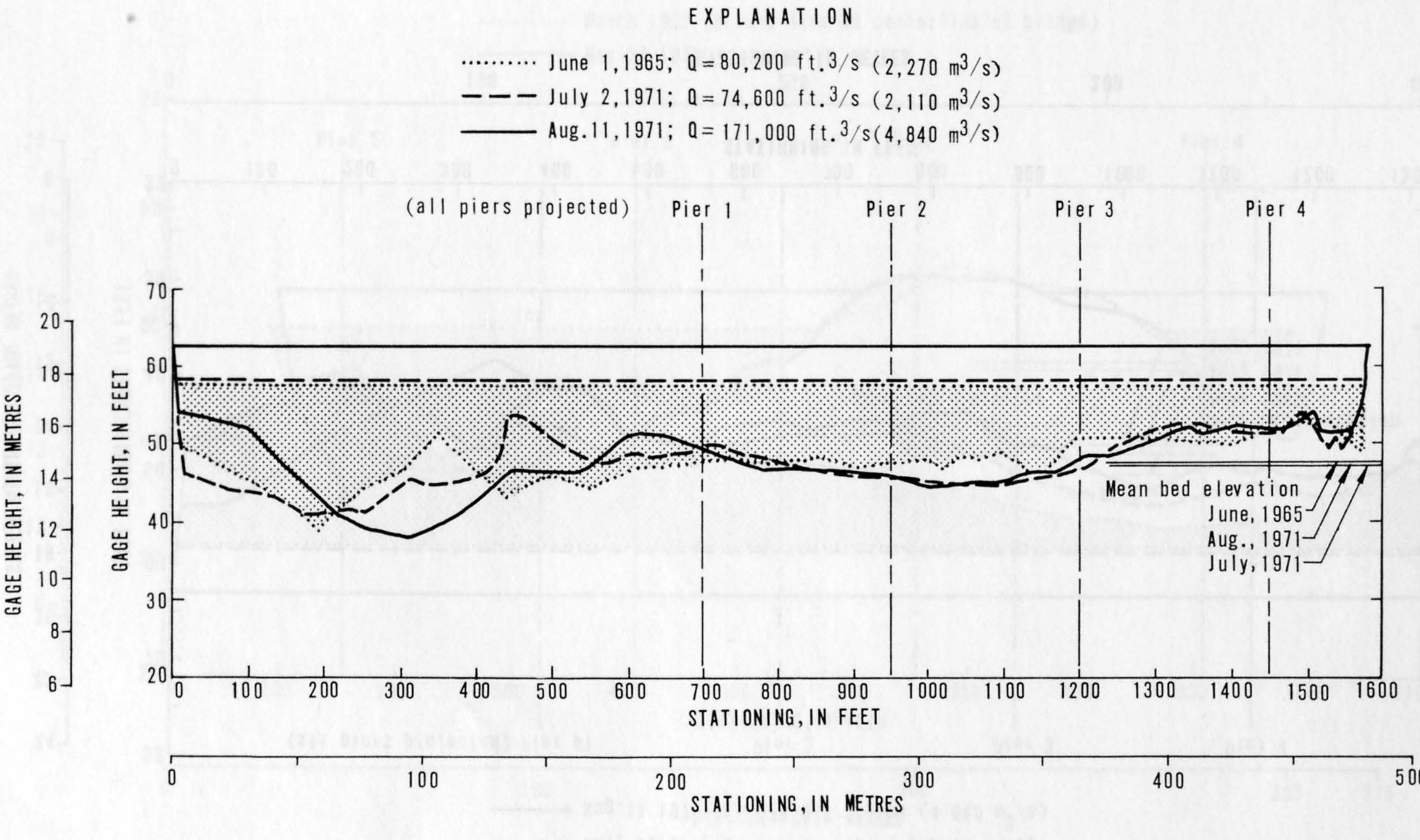


Figure 9.-- Cross section 1, Susitna River near Sunshine.

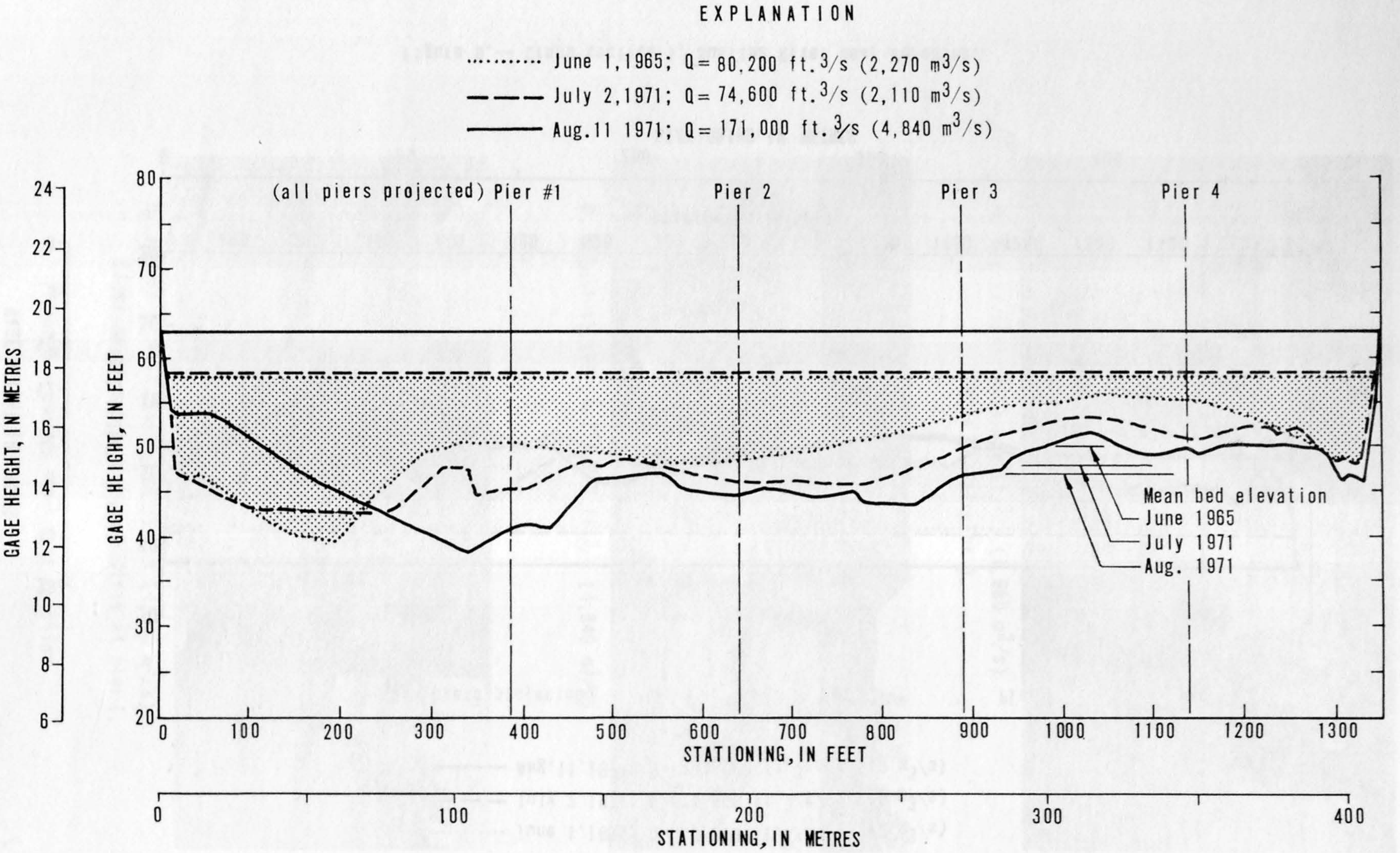


Figure 10.-- Cross section 2, Susitna River near Sunshine.

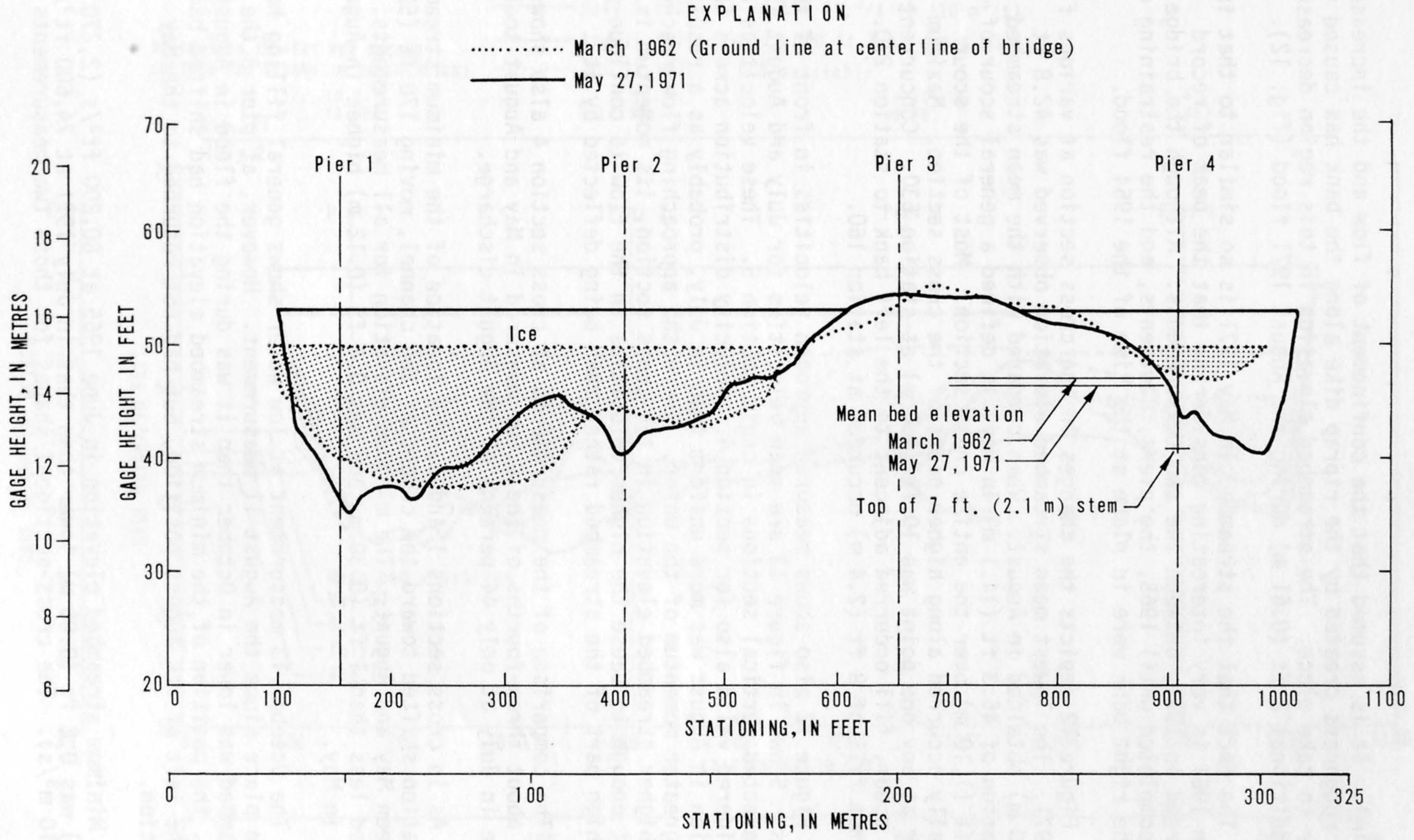


Figure 11.-- Cross section 4 at low water, Susitna River near Sunshine.

(2.4 m). It is assumed that the confinement of flow and the increased bank roughness created by the riprap dike along the bank has caused this scour to take place. The streambed elevation in this region decreased an additional 2 ft (0.61 m) during the August 1971 flood (fig. 12).

The fact that the streambed in May 1971 is so similar to that in March 1961 is very interesting considering that the peak of record occurred in 1964, between the two measurements. Although the bridge was not completed until 1965, the piers, abutments, and the restraining dike on the right bank were in place at the time of the 1964 flood.

Figure 12 depicts the changes in the cross section at various flows in 1971. The lowest mean streambed elevation observed was 42.8 ft (13.0 m) obtained in August. When compared with the mean streambed elevation of 46.3 ft (14.1 m) in May, it defined a general scour of 3.5 ft (1.10 m) over the entire cross section. Most of the scour actually occurred along higher parts of the cross section. Maximum scour at any one point was 10 ft (3.0 m) at station 630. Concurrent with scour, fill occurred adjacent to the left bank to station 270. A maximum fill of 8 ft (2.4 m) occurred at station 160.

Figure 12 also shows measured approach velocities in front of the piers. Shown in figure 13 are mean velocities for July and August 1971 at selected vertical sections in cross section 5. These velocities are considered valid also for section 4. Velocity distribution across the section in August was more uniform than in July, probably as a result of the greater momentum of the water. Where the approaching flow reaches the higher streambed elevation in the cross section, its momentum is great enough to cause the higher velocities in the flow to continue over the high part of the streambed rather than being deflected by it.

The comparison of the measurements at cross section 4 also shows that about three-fourths of the scour measured in May and August took place in July at only 44 percent of the August discharge.

As in cross sections 1 and 2, the location of the minimum streambed elevation shifted toward the center of the channel, moving 170 ft (52 m) between May and August. The minimum elevation for all measurements varied less than 1 ft (0.30 m) and was 0.4 ft (0.12 m) higher in August than in May.

The October 13 measurement at low water shows general filling had taken place since the August 11 measurement. However, at pier 3 the streambed was lower in October than it was during the flood in August. Also, the position of the minimum streambed elevation had shifted back to the left of its August position, but had not returned to the May position.

Minimum streambed elevation in June 1965 at 80,200 ft^3/s (2,270 m^3/s) was 0.2 ft (0.06 m) lower than that in July 1971 at 74,600 ft^3/s (2,110 m^3/s). The cross-sectional shapes for those two measurements

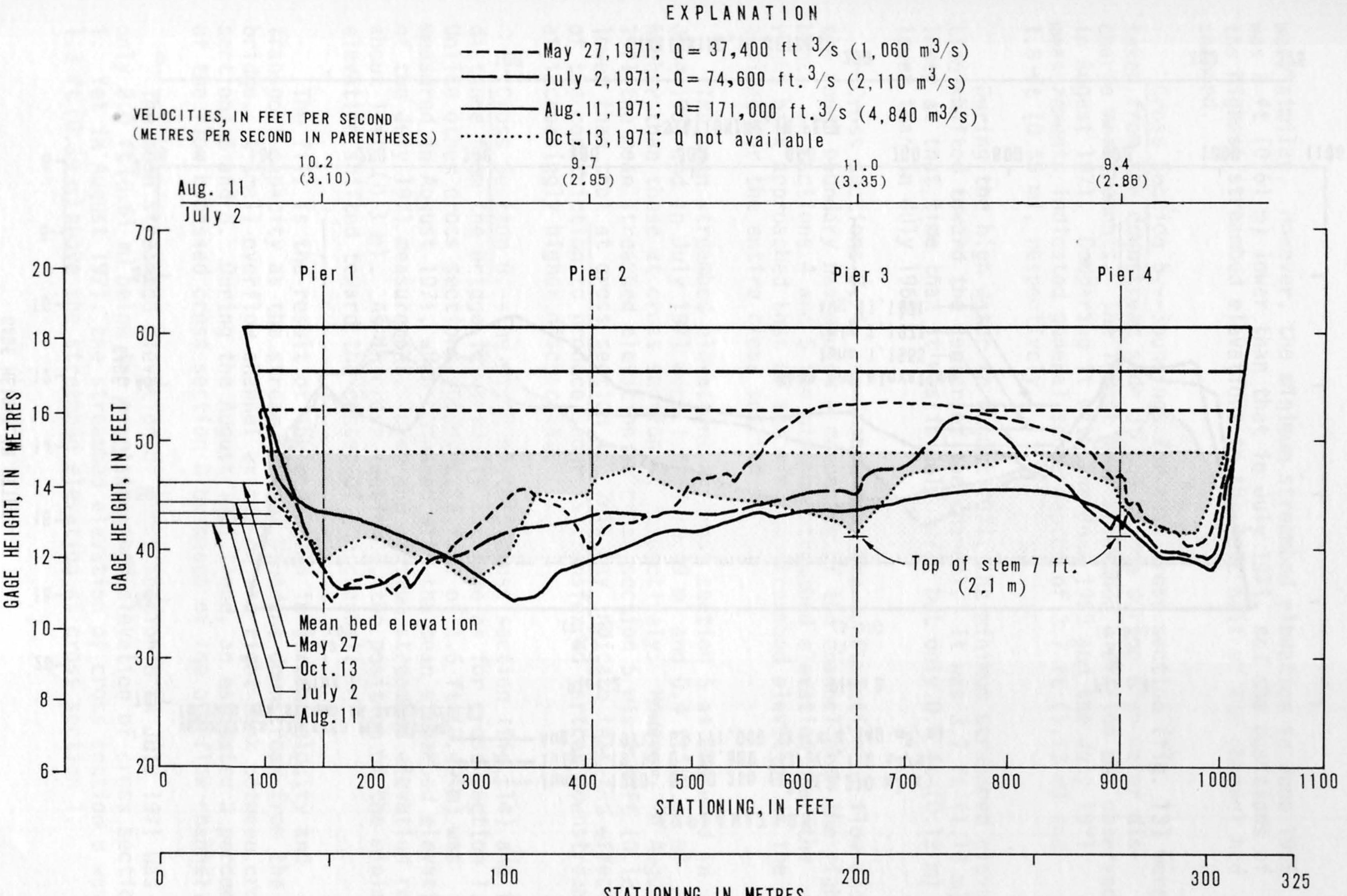


Figure 12.-- Cross section 4, Susitna River near Sunshine.

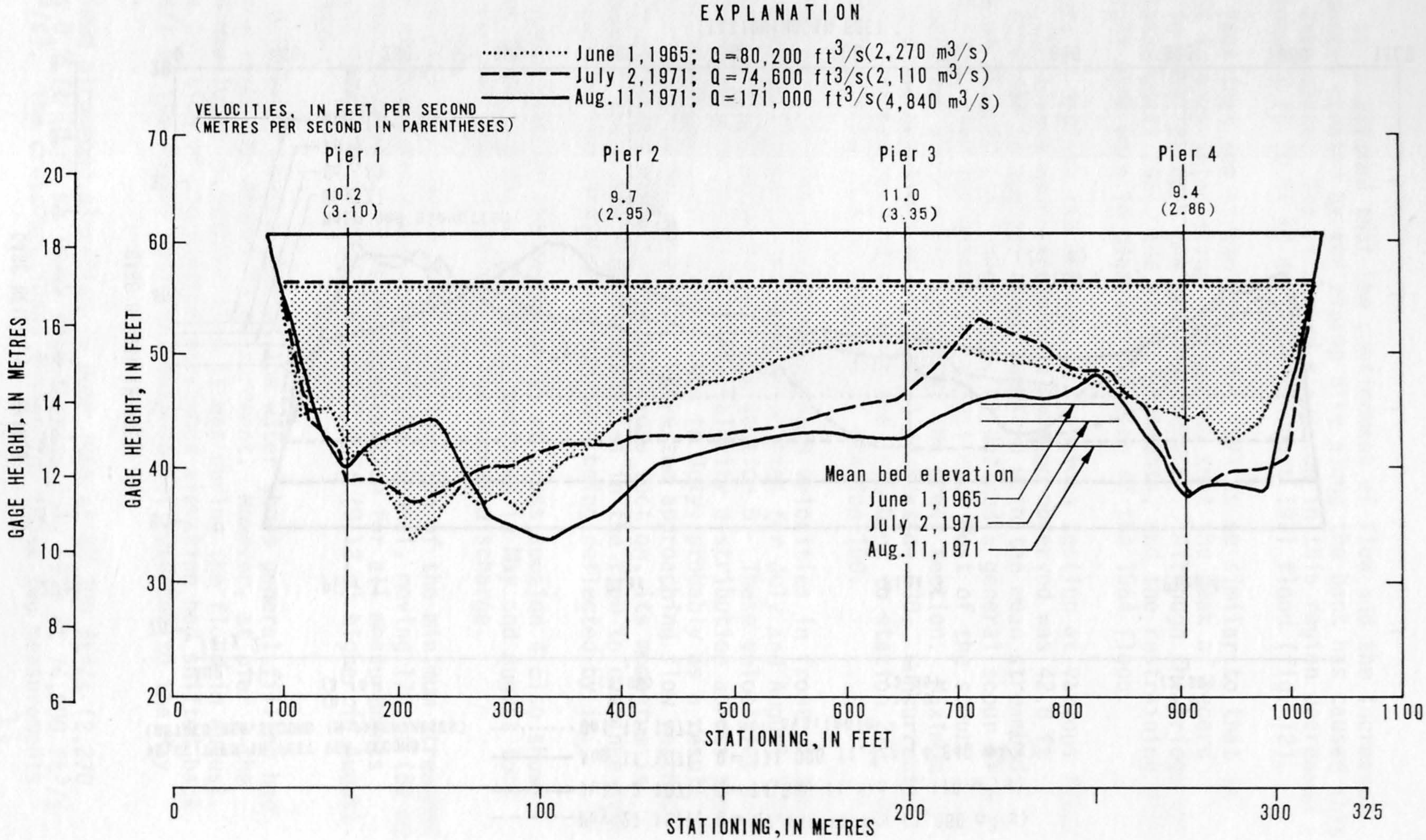


Figure 13.-- Cross section 5, Susitna River near Sunshine.

were similar. However, the minimum streambed elevation in June 1965 was 2 ft (0.61 m) lower than that in July 1971, and the positions of the highest streambed elevations in the left half of the channel had changed.

Cross Section 5.--Soundings for this cross section (fig. 13) were taken from the downstream side of the bridge during high-water discharge measurements. The lowest mean streambed elevation was observed in August 1971. Comparing it with the June 1965 and the July 1971 measurements indicated general scour depths of 3.7 ft (1.13 m) and 1.8 ft (0.55 m), respectively.

During the high water in August 1971, the minimum streambed elevation shifted toward the center of the channel. It was 3.7 ft (1.13 m) lower at that time than it was in July 1971, but only 0.4 ft (0.12 m) lower than in July 1965.

Cross sections 2, 4, and 5 show that the constriction of flow and the rough boundary has caused a deepening of the channel near the right bank. In sections 4 and 5 the minimum streambed elevation near the right bank approached that of the minimum streambed elevation at the thalweg for the entire cross section.

The mean streambed elevations at cross section 5 as measured in June 1965 and in July 1971 were 1.6 ft (0.49 m) and 0.4 ft (0.12 m) higher than those at cross section 4, respectively. However, in August 1971 the mean streambed elevation at cross section 5 was 0.4 ft (0.12 m) lower than that at cross section 4. This may indicate that the effect of the constriction to produce scour is transferred farther downstream at increasingly higher discharges.

Cross Section 8.--The width of this cross section (fig. 14) and its distance from the bridge is virtually the same as for cross section 1. Unlike other cross sections, however, a fill of 1.6 ft (0.49 m) was measured in August 1971, when compared with the mean streambed elevation of the July 1971 measurement. Even the minimum streambed elevation rose about 1 ft (0.3 m). As in cross section 5, the position of the minimum elevation shifted toward the center of the channel.

The fill is the result of the reduction in stream velocity and transport capacity as the stream channel broadens downstream from the bridge. A small overflow channel exists on the right bank between cross sections 5 and 8. During the August 1971 flood, an estimated 5 percent of the flow bypassed cross section 8 by means of the overflow channel.

The mean streambed elevation of cross section 8 in July 1971 was only 2.0 ft (0.61 m) below the mean streambed elevation of cross section 1. Yet in August 1971, the streambed elevation of cross section 8 was 1.3 ft (0.39 m) above the streambed elevation at cross section 1.

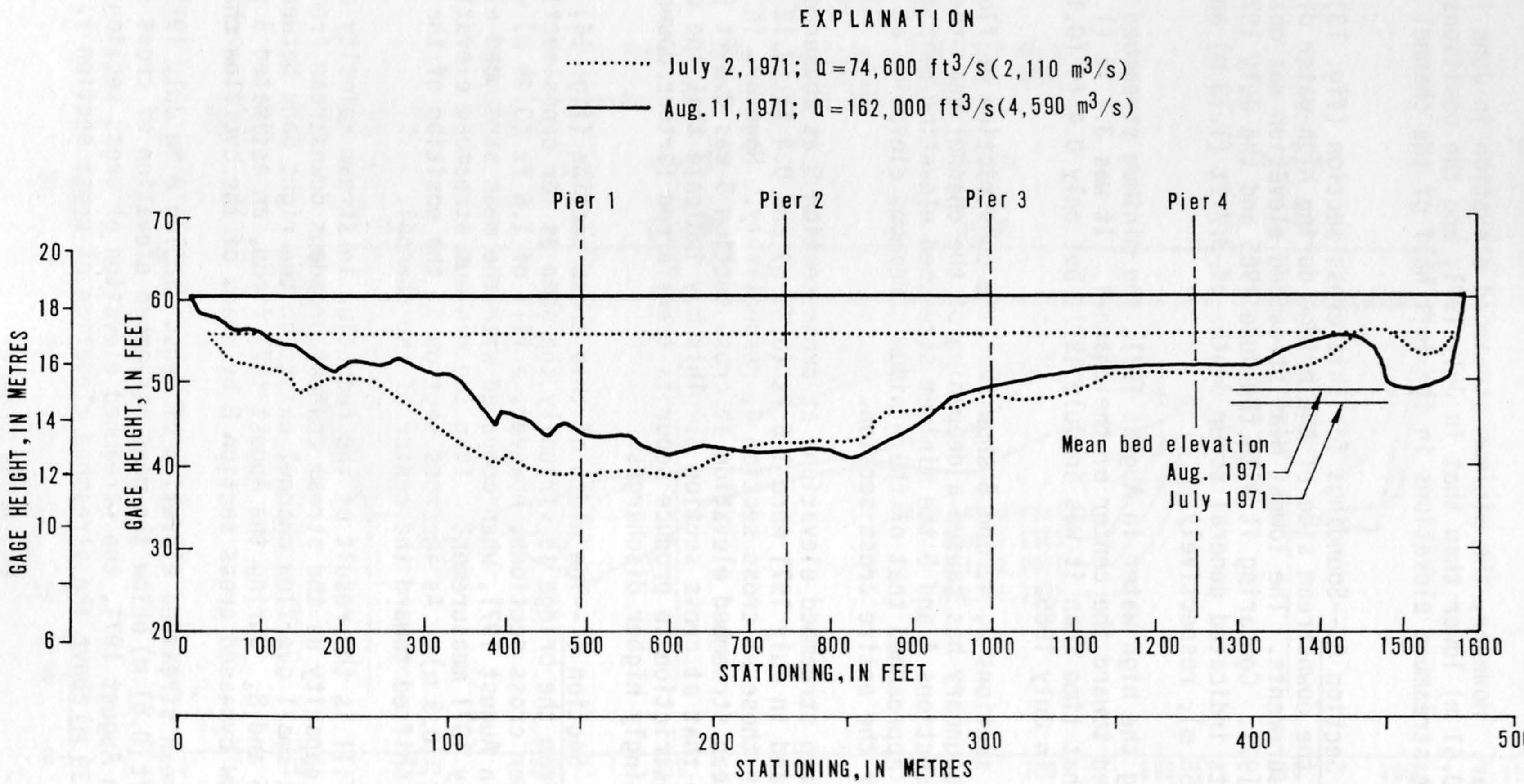


Figure 14.-- Cross section 8, Susitna River near Sunshine.

Water-surface Slopes and Streambed Profiles

The water-surface slopes for June 1, 1965, and August 11, 1971, are shown in table 2. The slope above the bridge was slightly higher in August 1971, at about twice the discharge, than that in June 1965. Below the bridge, the slope in August 1971 was about one-half of the slope in June 1965. These differences in slope reflect the scouring in the bridge opening and the filling downstream during the August 1971 flood.

Longitudinal streambed profiles were fairly smooth, the major variations in elevations result from changes in cross-sectional shapes. In June 1965 and July 1971 parts of the streambed for several hundred feet exhibited a dune configuration having amplitudes of 0.5-1.0 ft (0.15-0.30 m) in height and wave lengths of 20-25 ft (6-8 m). In August 1971 some longitudinal profiles were obtained and no dune-like bed forms were detected on the smooth bed.

Velocity Distribution

Detailed velocity profiles were obtained at selected vertical sections in cross section 3 in June 1965 (fig. 15). Also shown are velocities based on measurements at 0.2 and 0.8 of the depth in August 1971.

At the higher flows the angle of attack on the piers observed on the water surface was less than about 5°. However, angles of attack up to 20° were measured in June 1965 at cross section 3 using a subsurface directional indicator mounted in a sounding weight. At the same time, angles of attack measured on the water surface at cross section 5 were less than about 5°. Cross section and longitudinal profile data do not indicate any reason why the subsurface flow should be going in a different direction than surface flow, consequently the accuracy of the subsurface device is questioned.

Sediment Analyses

Streambed material and suspended-sediment samples were obtained in 1971 and 1972. The results of analyses are given in table 3. Analytical procedures used were described by Guy (1969) and by Ritter and Helley (1969).

Photographs of surface streambed material were taken June 3, 1972, on an exposed bar at cross section 8. This material is imbricated (shingled) and is considered to be the same material which was removed by the flow from the constricted opening at the bridge during the August 1971 flood. This conclusion was reached from analysis of the cross sections at the bridge and at cross section 8 which showed that while scour was occurring at the bridge, deposition was taking place at cross section 8. As shown in figure 6, cross section 8 is in an expanding reach directly downstream from the bridge and deposition of streambed

Table 2.--*Measured discharge and slope, Susitna River near Sunshine*

Date	Discharge (ft^3/s)	Slope ft/ft			
		Sec. 1-2	Sec. 2-4	Sec. 1-4	Sec. 5-8
June 1, 1965	80,200	0.0002	0.0025	0.0014	0.0010
Aug. 11, 1971	171,000	.0004	.0029	.0017	.0004

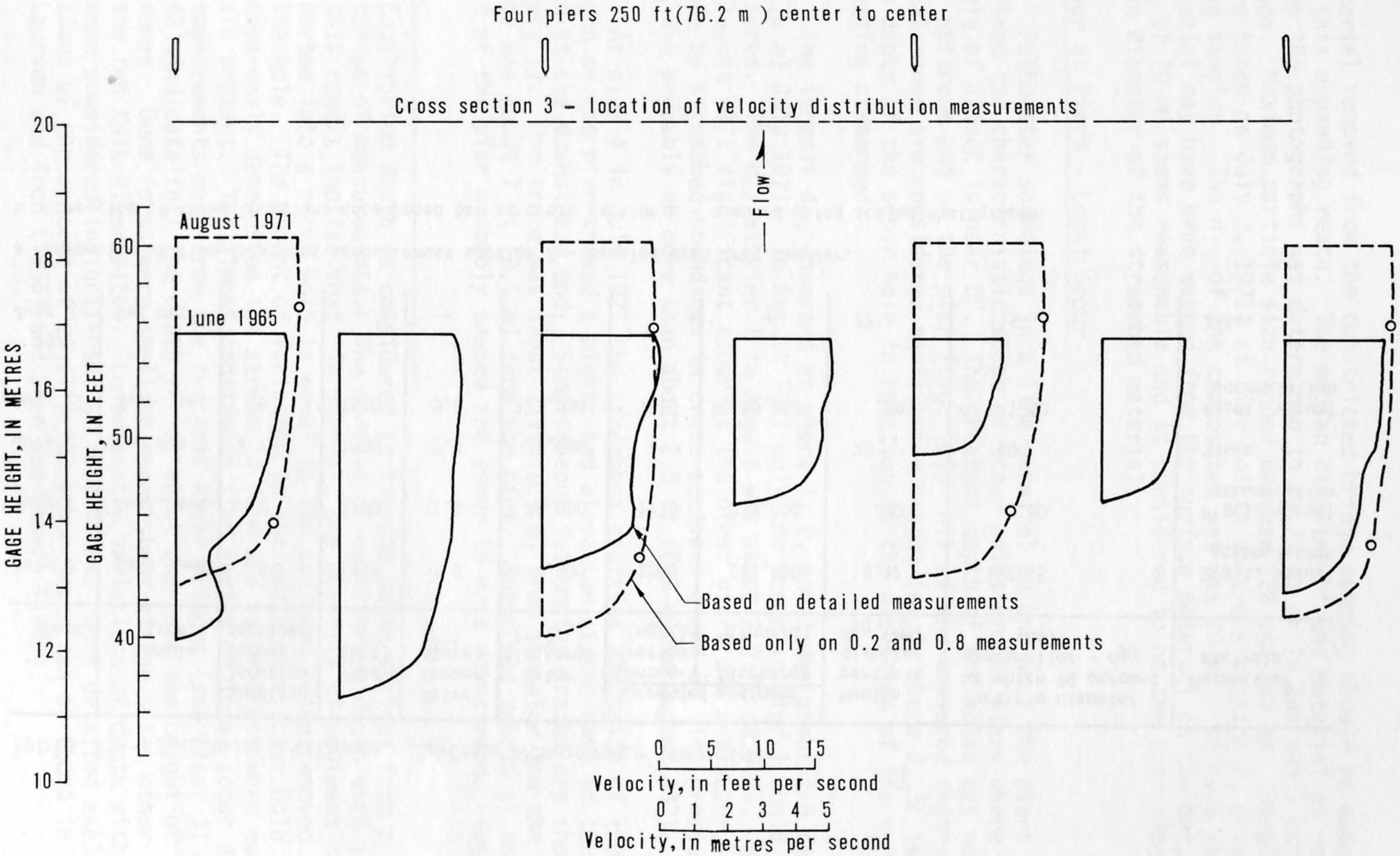


Figure 15.-- Velocity distributions, Susitna River near Sunshine.

Table 3.-- *Sediment analyses, Susitna River near Sunshine*

Date	Sample type	Sampling location (cross section)	Time (hrs)	Water temper-ature (°C)	Water discharge (ft^3/s)	Suspended sediment: Concen-tration (mg/l)	Suspended sediment: Discharge (tons/d)	Median particle diameter d_{50} (mm)	Particle diameter at which 90 percent are smaller - d_{90} (mm)	Methods of analysis
1971										
July 2	Susp. sed.	5	1430	7.5	74,600	1050	211,000	0.37	2.65	Pipet, visual accumulation
July 2	Susp. sed.	2	1700	7.5	71,000	1170	224,000	.30	2.60	Pipet, visual accumulation
July 2	Bed mat'l	a 2	1800	7.5	71,000	--	--	70	90	Sieve
Aug. 11	Susp. sed.	5	1640	9.0	171,000	3510	1,620,000	.20	1.50	Pipet, visual accumulation
1972										
June 3	Bed mat'l	b 8	--	--	--		--	77	115	Zeiss

a Composite of five locations across cross section 2 - sampled with drag sampler.

b Composite of three locations on exposed bar at cross section 8 - sampled using scaled photographs.

material removed from the constricted bridge opening should be expected in this expanding reach. The median diameter of the material as derived from the photographs was determined to be 77 mm or in the small cobble range. Maximum particle size ranged between 128 and 167 mm. Samples were taken on July 2, 1971, at five points on cross section 2, using the drag sampler. The d_{50} of the composited set was 70 mm. Because fine material may have been washed from the surface material on the bar, the d_{50} of 70 mm seems reasonable and is considered to be the approximate mean diameter of the streambed material.

Scour at Piers - Local Scour

Fathometer soundings in a line parallel to and near the piers defined the characteristic shape of scour holes around piers where the angle of attack is near 0°. The greatest depth of the holes was near the upstream end of the pier (fig. 16). All of the measured depths of local scour are considered equilibrium depths (d_{se}), where d_{se}[3/] is the depth of the scour hole in relation to the elevation of the surrounding streambed.

The largest d_{se} detected at piers 1, 2, and 3 was about 2.5 ft (0.76 m) July 1971. In August 1971 only the scour at pier 1 could be measured. The d_{se} at pier 1 in August was about 2.0 ft (0.6 m). However, in August 1971 significant scour at the other piers did not occur, as shown by streambed soundings in cross sections 4 and 5. The greatest d_{se} was probably no more than about 2.5 ft (0.7 m) in August 1971.

At pier 4 in July 1971 the streambed profiles alongside of the length of the pier showed a higher bed elevation at the nose of the pier than at the downstream end. Simultaneous fathometer traces are shown in figure 17. One trace was about 3 ft (0.9 m) from the pier and the other trace was about 7 ft (2.1 m) from the pier. Submerged debris at the nose of the pier probably caused the scour to move downstream. The d_{se}

[3/] Equilibrium depth is considered an average depth derived from a time series of measurements. The concept of equilibrium depth used in this report implies that, in general, the quantity of sediment transported into a scour hole is equal to the sediment being removed from the hole. The amount of variation in the depth of a scour hole apparently depends on the streambed form and is greatest when dunes are present. The almost continuous time series of local scour depth measurements made from the two Knik River bridges (see figs. 31 and 42) indicate that the depth varies about one-half the height of dunes. Dune forms were small or nonexistent at the sites other than the two Knik River sites. Consequently, variations of depth with time were considered insignificant at most sites. More detailed explanations of the concept of equilibrium scour depth can be found in Laursen and Toch (1956) and Shen and others (1969).

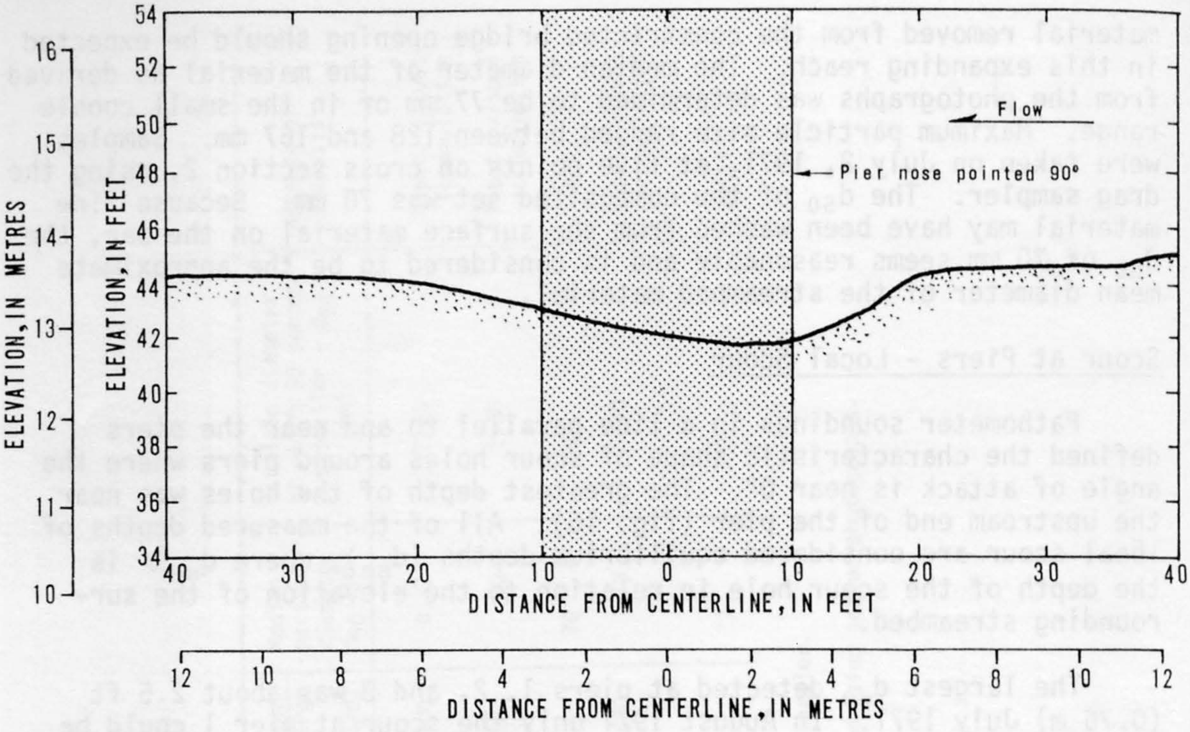


Figure 16.-- Typical profile of streambed along piers 1,2 and 3, Susitna River near Sunshine.

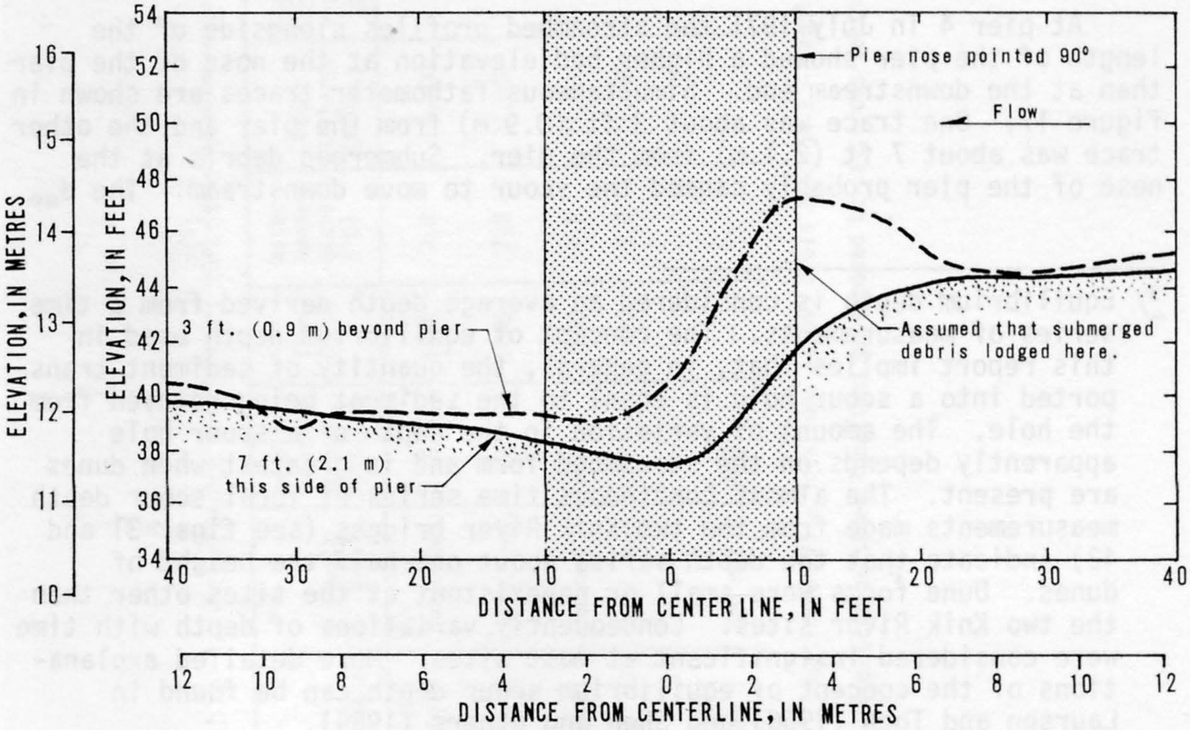


Figure 17.-- Profile of streambed along pier 4, July 2, 1971, Susitna River near Sunshine.

at the pier was about 5 ft (1.5 m), or twice that expected if the debris had not been present. If the debris had been buoyed up by the water, or had there been debris along the entire pier nose, d_{se} would probably have been near the front of the debris. Turbulence and the small amount of debris at the surface at pier 4 did not appear very different than that at the other piers.

Figure 8C shows the water-surface profile parallel to pier 1 at the high flow on August 11. Table 4 summarizes the scour depths and hydraulic parameters measured near the piers in July and August 1971.

Comparison of Measured Scour with Scour Calculated from Formulas

General Scour

In the equations used to estimate general scour (of the sediment transport type) in a contraction, the width constriction is considered gradual. Laursen (1958) first used the term "long contraction" to describe this situation, and subsequent researchers have continued its use. Most of the equations are based on the principle of continuity (see Culbertson and others, 1967), but some of them include refinements which try to account for sediment transport. Three of these equations, expressed in terms of y_2/y_1 and B_1/B_2, are:

(1) $$\frac{y_2}{y_1} = \left(\frac{B_1}{B_2}\right)^{0.64}$$ Griffith (1939) and Straub (1940) (*in* Culbertson and others, 1967)

(2) $$\frac{y_2}{y_1} = \left(\frac{B_1}{B_2}\right)^{p}$$ Laursen (1962) (Where p varies from 0.59 to 0.69 depending on the ratio of shear velocity to the fall velocity of the streambed sediments)

(3) $$\frac{y_2}{y_1} = \left(\frac{\tau_0'}{\tau_2'}\right)^{2/7} \left(\frac{B_1}{B_2}\right)^{6/7}$$ Komura (1966) (Where $\frac{\tau_0'}{\tau_2'}$ is the ratio of shear stresses at the uncontracted and contracted sections, respectively)

Table 4.-- *Summary of measured local equilibrium depth of scour, Susitna River near Sunshine*

Date	Discharge Q (ft^3/s)	Pier	Equilibrium depth of scour d_{se} (ft)	Mean approach velocity v_a (ft/s)	Approach depth y_a (ft)	Water temp. (°C)
July 2, 1971	74,600	1	2.5	6.5	19.0	7.5
		2	2.5	8.5	13.5	
		3	2.0	7.0	11.0	
		4	a 5.0	5.0	13.5	
Aug. 11	171,000	1	2.0	10.0	17.5	9.0
		2	b 2.0	9.5	21.5	
		3	c 2.0	11.5	17.0	
		4	c 5.0	9.5	17.5	

a There was apparently submerged debris on the nose of the pier, and d_{se} was near the downstream end of the pier.

b Estimated from partial fathometer trace.

c Estimated from soundings on cross sections 4 and 5 and results obtained in July 1971.

In these equations, y_1 and B_1 are the mean depth and the surface width, respectively, at the approach or uncontracted section; y_2 and B_2 are the mean depth and the surface width, respectively, at the contracted section. If the depth of scour at the contracted section (d_{sc}) is desired, then y_2/y_1 can be expressed as $d_{sc} + 1$. The coefficient p in equation 2 was 0.59 for all of the sites in this study.

Although not ideal, the bridge site on the Susitna River may be considered a fair approximation of the long contraction. Mean depths determined in the field during the August 1971 flood are given in table 5 along with those calculated using equations 1-3. Note that equations 1 and 2 overestimated the measured mean depth by less than 10 percent.

The water-surface profiles indicated that some backwater effect occurred during the August 1971 flood. This could mean that equilibrium had not been reached, that scour was still occurring, and that the mean depths would have approached those calculated by equations 1 and 2 if the flood had continued. It is more probable that the coarse bed material had armored the streambed somewhat and the sediment transport through the reach was near equilibrium. However, if the peak flow had been sustained, fill might have occurred at cross section 1 and scour might have occurred at cross section 8 and the mean depth through the bridge opening might have approached closely that predicted by equations 1 and 2. Using the depths as measured, the equation which best describes the Susitna River data in August 1971 is:

$$(4)\quad \frac{y_2}{y_1} = \left(\frac{B_1}{B_2}\right)^{0.4}$$

However, if the backwater effect (about 1.5 ft or 0.46 m) is substracted from the measured approach depth in cross section 1, then equation 2 yields depths quite comparable to those measured at cross sections 2, 4, and 5.

Pier Scour

Unlike the theoretical approach to the general scour problem, solutions for determining the depth of local scour at piers have come primarily from model studies using sand-size sediment. Studies by Laursen and Toch (1956) and further reports by Laursen (1958, 1962) gave design charts and formulas for determining the equilibrium depth of scour at piers where a continuous supply of bedload material is available to the scour hole. The National Highway Research Program (1970) reports that Laursen and Toch's (1956) report was used more than any other reference (except engineering judgment) to predict scour. Neill (1970) transcribed Laursen's basic design curve for a square-nosed pier alined with the flow into the formula:

$$(5)\quad d_{se} = 1.5b^{0.7}\, y_a^{\,0.3}$$

Table 5.--*Comparison of measured mean depths to calculated depths, August 11, 1971, Susitna River near Sunshine*

Cross section	Measured (ft)	Calculated		
		Equation 1 (ft)	Equation 2 (ft)	Equation 3 (ft)
1	14.8	--	--	
2	15.7	16.4	16.2	16.7
4	18.0	20.5	20.0	22.6
5	18.4	20.8	20.2	23.1

Where d_{se} is the equilibrium depth of local scour below the surrounding streambed, b is the pier width, and y_a is the depth of the approaching flow. For other than square-nosed shapes Laursen and Toch (1956) gave the multiplying coefficients shown in table 6. Where the pier was at some angle to the flow, they gave another coefficient to be applied. However, the coefficients for angle of attack are only for square-nosed piers; both coefficients cannot be applied at the same time. In fact, data from others (Chabert and Engeldinger, and Varzeliotis *in* Neill, 1970, p. 22) show that the angle correction curves given by Laursen should be concave upward instead of concave downward, where the curves are plotted with increasing angle of attack on the abscissa and increasing correction coefficients on the ordinate.

Shen and others (1969) concluded from their study of all available data that for scour with continuous sediment motion and for piers alined with the flow, either the equation given by Breusers (1965), which is

(6) $d^*_{se} = 1.4b$ (for cylindrical piers)

or the one proposed by Larras (*in* Shen and others, 1969, p. 1935) where

(7) $d^*_{se} = 1.42kb^{0.75}$

in which d^*_{se} is the maximum depth of scour as shown in figure 18, and k is a coefficient depending on pier shape, be used to estimate pier scour.

The predictive d_{se} and d^*_{se} from these equations are compared in table 7 with the measured values of scour at the piers for the Susitna River site. The measured values of scour at the piers were considered equilibrium depths of local scour (d_{se}) with continuous sediment motion. To compare these measured values with those from equations 6 and 7 a "measured" d^*_{se} was obtained by dividing the measured d_{se} by 0.90 as suggested by Shen's (1971) graph (see fig. 18 this report) describing the process of pier scour development.

A summary and discussion comparing field measurements at all sites with scour calculated from selected scour formulas is given beginning on page 135.

Knik River near Palmer - Bridge 539

Description

This study site is located at bridge 539 where it crosses the Knik River at mile 39 on the original Glenn Highway about 7 mi (11.3 km) south of Palmer. An aerial photograph of the site is shown in figure 19. The bridge opening is 2,000 ft (610 m) long. The principal structure is 1,500 ft (457 m) long, supported by six 6-ft (1.8-m)-wide piers with pointed noses and spaced 250 ft (76.2 m) apart. A 500-ft (152-m) approach on wooden pilings extends from the right bank to the bridge. All piers are approximately alined with the flow.

Table 6.--*Shape coefficients K_s for nose forms (after Laursen and Toch, 1956)*

[To be used *only* for piers alined with flow]

Nose form	Length-width ratio	K_s
Rectangular		1.00
Semicircular		0.90
Elliptic	2:1	0.80
	3:1	0.75
Lenticular	2:1	0.80
	3:1	0.70

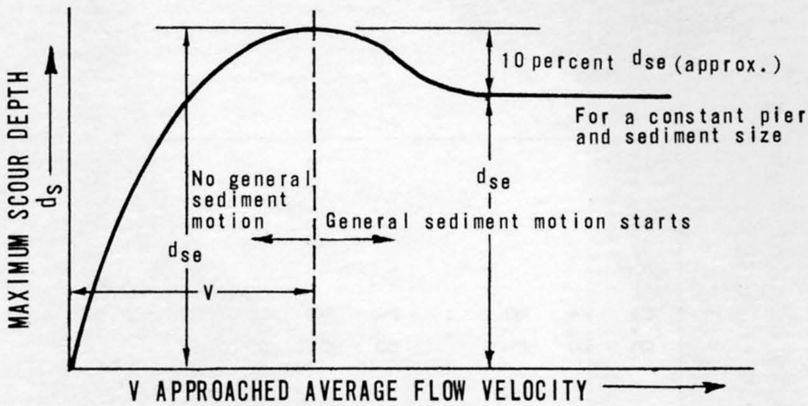


Figure 18.-- Typical results by Chabert and Engeldinger (in Shen and others, 1969). Variation of scour depth with velocity. After Shen and others (1971).

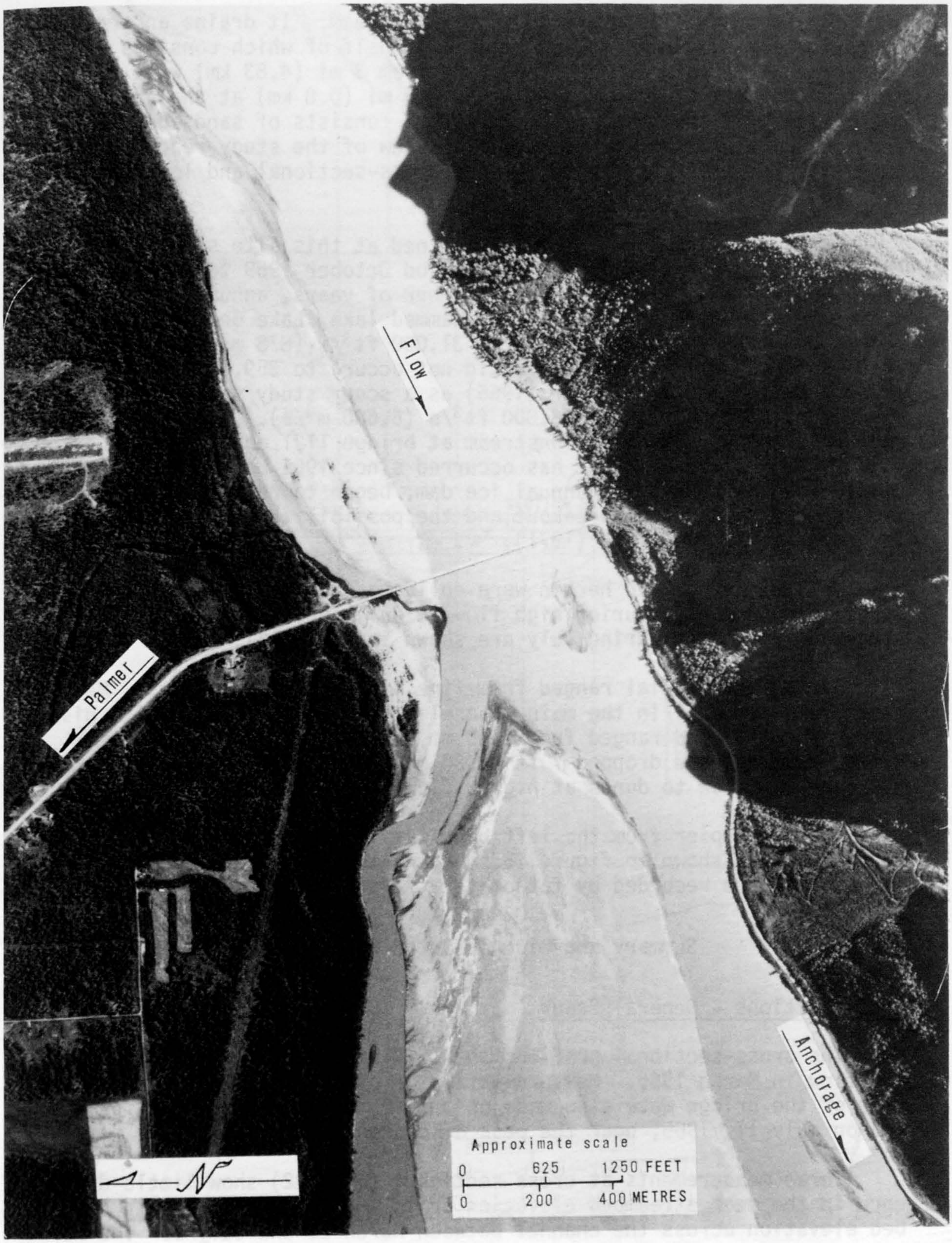
Table 7.-- *Comparison of measured and calculated local depths of scour with continuous sediment movement, Susitna River near Sunshine*

Date	Pier	Measured		Calculated		
		d_{se} (ft)	d^{*}_{se} $d_{se}/0.90$ (ft)	Equation 5 d_{se} x 0.80 to account for pointed nose (ft)	Equation 6 d^{*}_{se} x 0.90 as implied by table 6 (ft)	Equation 7 d^{*}_{se} x 0.90 as implied by table 6 (ft)
July 2, 1971	1	2.5	2.8	9.0	6.3	4.2
[Q=74,600 ft^3/s RI 2 yrs]	2	2.5	2.8	8.1	6.3	4.2
	3	2.0	2.2	7.6	6.3	4.2
	4	a 5.0	5.6	--	--	--
Aug. 11	1	2.0	2.2	8.7	6.3	4.2
[Q=171,000 ft^3/s RI 15-20 yrs]	2	b 2.0	2.2	9.3	6.3	4.2
	3	c 2.0	2.2	8.6	6.3	4.2
	4	c 5.0	5.6	--	--	--

a There was apparently submerged debris on the nose of the pier at the bed - total width approximately 11 ft; however, d_{se} was located near the downstream end of the pier. All other d_{se} were at the nose of the pier.

b Estimated from partial fathometer trace.

c Estimated from soundings on cross sections 4 and 5 and results obtained in July 1971.



STATE OF ALASKA DEPARTMENT OF HIGHWAYS

Figure 19.--Aerial view of Knik River near Palmer at bridge 539, August 26,1970.

The Knik River is a braided glacial stream. It drains an area of approximately 1,200 mi^2 (3,100 km^2), over half of which consists of glaciers. The braided channel narrows from 3 mi (4.83 km) wide at the terminus of Knik Glacier to less than 0.5 mi (0.8 km) at the bridge. In the vicinity of the bridge, the streambed consists of sand and gravel and some cobbles. Figure 20 is a plan view of the study reach which shows the locations and shapes of the cross-sectional and longitudinal profiles.

Daily discharges have been determined at this site since October 1959. The average flow during the period October 1959 to October 1965 was 6,960 ft^3/s (197 m^3/s). For a number of years, annual peaks were caused by the breakout of a glacier-dammed lake, Lake George. Recorded peaks from 1959 to 1965 ranged from 31,000 ft^3/s (878 m^3/s) (in 1962 when the breakout of Lake George did not occur) to 359,000 ft^3/s (10,200 m^3/s). During the first year (1965) as a scour study site, the peak discharge was measured as 236,000 ft^3/s (6,680 m^3/s). Scour during the 1966 breakout was studied downstream at bridge 1121 and is described on pages 61 to 78. No breakout has occurred since 1966 because the Knik Glacier, which caused the annual ice dam, began to retreat. A description of the Lake George breakout and the possibility of its recurrence is given by Post and Mayo (1971).

The data described herein were collected during low flow in March, April, and June, and during high flow in July 1965. Hydrographs of stage and discharge during July are shown in figure 21.

Streambed material ranged from fine sand to small cobbles. Median particle size (d_{50}) in the main channel at the bridge varied with discharge and time and ranged from 2.45 mm at the peak flow to 0.93 mm after the stage had dropped 7 ft (2.38 m). Streambed form changed from plane at low flow to dunes at higher flows.

The fifth pier from the left bank was instrumented with four fixed transducers as shown in figure 28. Depths to the streambed below each transducer were recorded by fathometer.

Summary and Discussion of Observations

Cross Sections - General Scour

The cross-sectional profiles shown in figure 20 were obtained at low flow in March 1965. Measurements at cross section 4 on the upstream side of the bridge were also made on July 9, prior to the flood peak, and on July 11, 1965, near the peak discharge.

Three measurements at cross section 4 (fig. 22) show little difference in the mean streambed elevation or in the irregularities in streambed elevation across the channel between March 23 and July 9. The mean

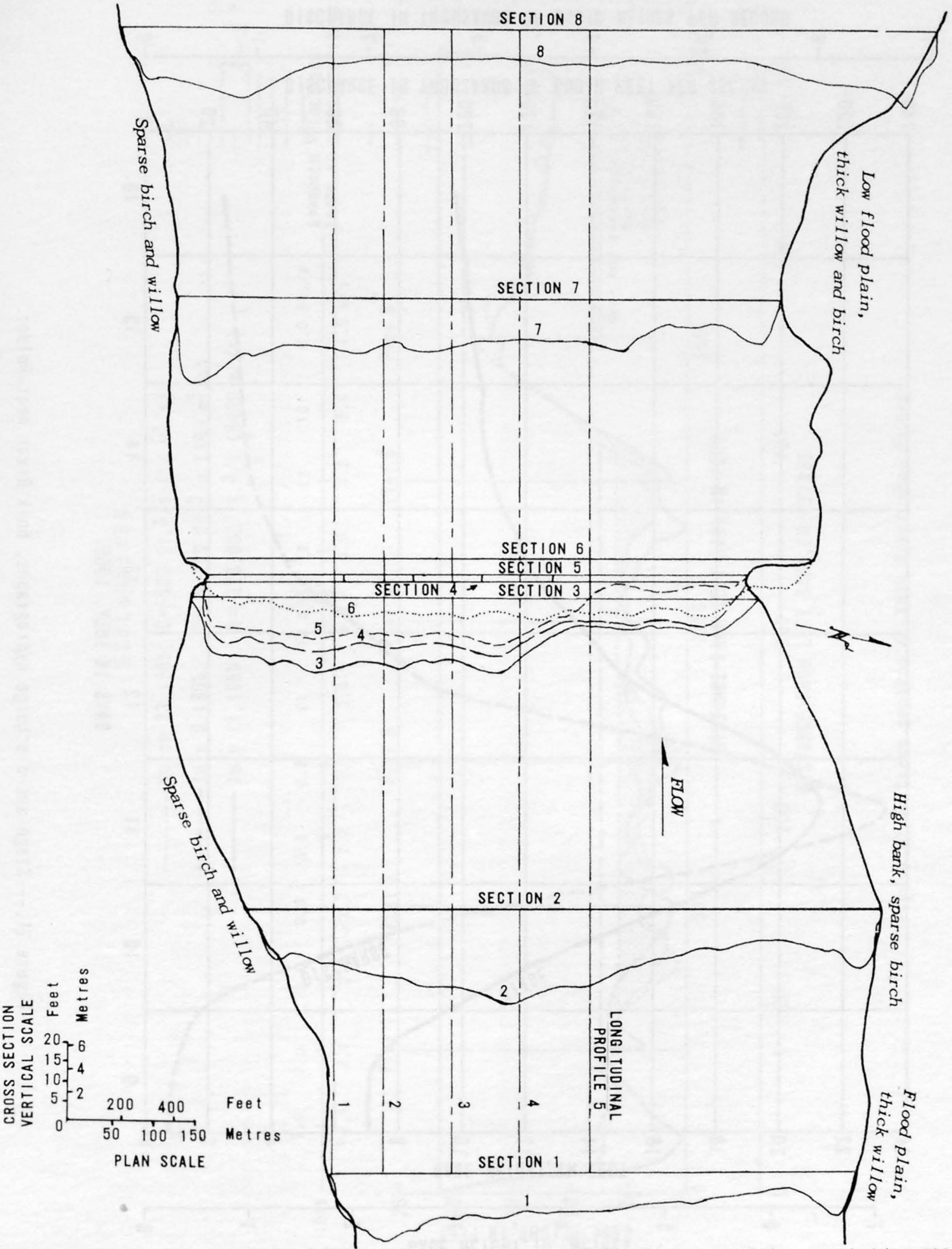


Figure 20.--Location and general shape of cross sections, Knik River near Palmer at bridge 539.

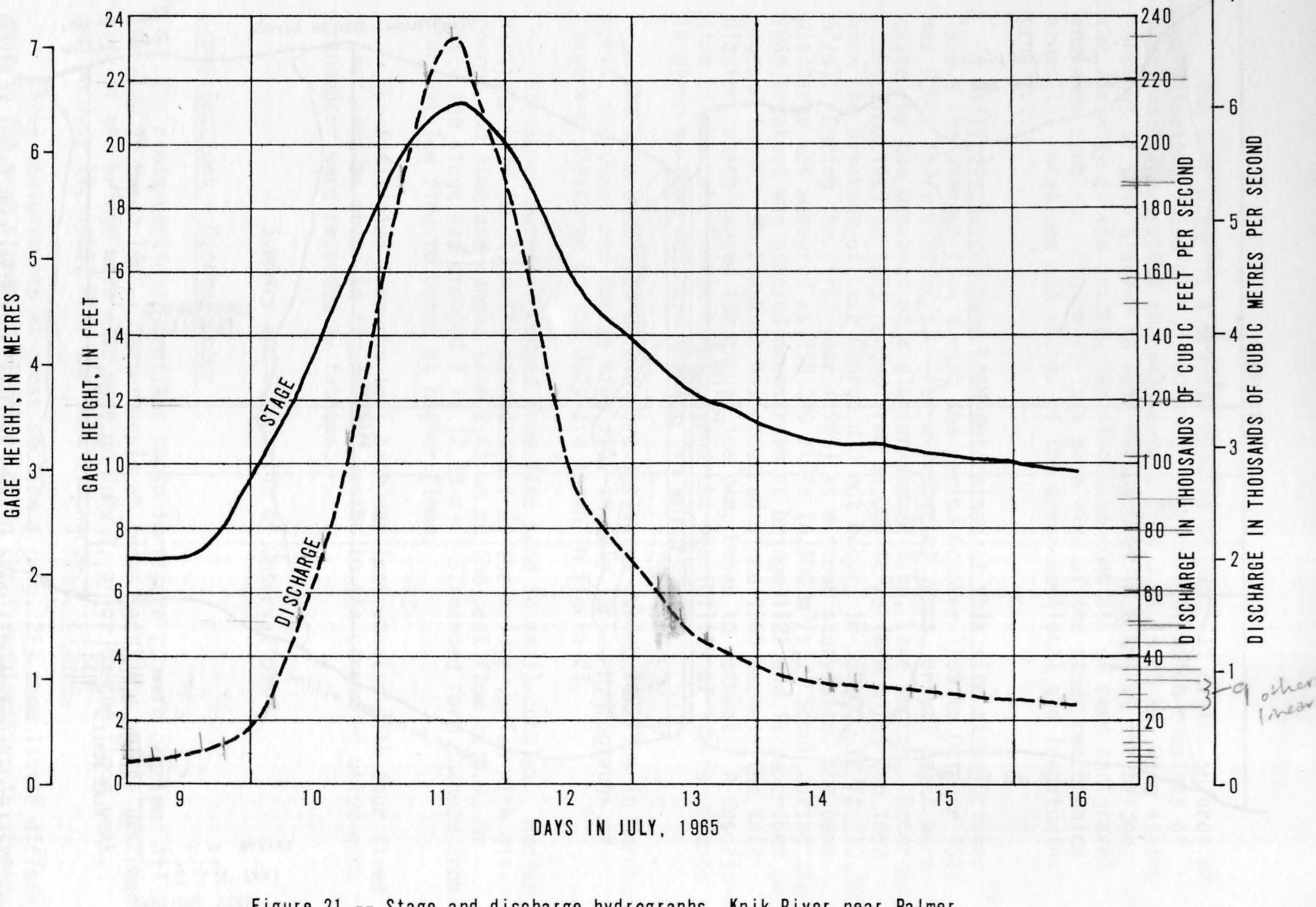


Figure 21.-- Stage and discharge hydrographs, Knik River near Palmer.

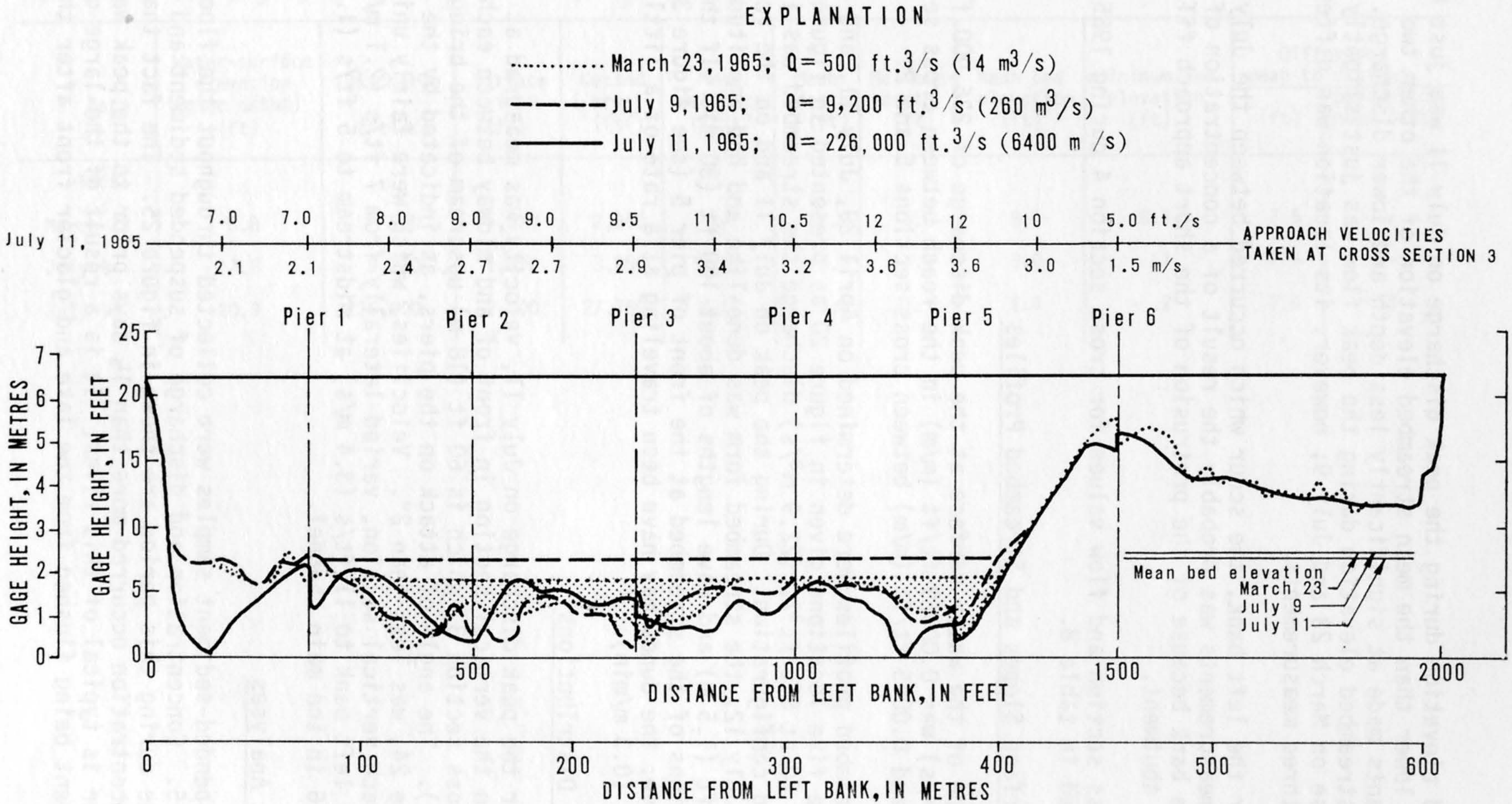


Figure 22.-- Cross section 4, Knik River near Palmer.

streambed elevation during the peak discharge on July 11 was just 0.6 ft (0.18 m) lower than the mean streambed elevation of the other two measurements made at significantly less depth and lower discharge. The minimum streambed elevation during the peak flow was just slightly lower than those on March 23 and July 9; however, its location was different for all three measurements.

Near the left bank, the scour which occurred between the July 9 and July 11 measurements was probably the result of a concentration of flow along the bank because of the protrusion of the short approach fill at the left abutment.

Cross section and flow values for cross section 4 during 1965 are summarized in table 8.

Water-surface Slopes and Streambed Profiles

Slope of the water surface at the peak discharge of 236,000 ft^3/s (6,680 m^3/s) was 0.00069 ft/ft (m/m) in the reach between cross sections 1 and 4 and 0.0045 ft/ft (m/m) between cross sections 5 and 8.

Streambed profiles were determined on April 20, July 11, and July 12 at the five locations given in figure 20 as presented in figure 23. On April 20 at 810 ft^3/s (22.9 m^3/s) discharge the streambed was in a plane bed configuration. During the peak on July 11 and on the recession of July 12, the streambed form was dune-like and had amplitudes of about 5 ft (1.5 m) and wave lengths of about 100 ft (30 m). If the oscillations of the streambed at the front of pier 5 (see figure 31) are indicative, the dunes may have been traveling at a rate of a little over 1 ft/min (0.3 m/min).

Velocity Distributions

Near the peak discharge on July 11, velocity was measured at six points in the vertical section in front of and midway between each pier along cross section 3, which is 60 ft (18 m) upstream of the bridge (fig. 24). The angle of attack on the piers, as indicated by the arrows in figure 24, was less than 2°. Velocities, which were fairly uniform within each vertical section, varied laterally from 7 ft/s (2.1 m/s) near the left bank to 11 ft/s (3.4 m/s) at midstream to 5 ft/s (1.5 m/s) at pier 6 in the main channel.

Sediment Analyses

Suspended-sediment samples were collected throughout the flood of July 9-15. Concentration and discharge of suspended sediment and water discharge during this period are shown in figure 25. The fact that the peak concentration occurred more than 1½ days prior to the peak water discharge is typical of this river. It is a result of the large quantity of sediment being flushed from the lake and glacier front after the lake

Table 8.-- *Summary of cross section and flow values, cross section 4, Knik River near Palmer*

Date	Water-surface elevation (gage datum) (ft)	Discharge (ft^3/s)	Surface width (ft)	Wetted area (ft^2)	Mean velocity (ft/s)	Mean depth (ft)	Maximum depth (ft)	Mean bed elevation (ft)	Minimum bed elevation (ft)	Difference mean to minimum (ft)	
										(ft)	percent of mean depth
1965											
March 23	5.8	500	1,020	2,860	--	-2.2	5.3	8.0	0.5	7.5	--
July 9	7.2	9,220	1,070	3,050	3.02	-0.8	7.1	8.0	.1	7.9	--
July 11	20.9	226,000	2,000	27,100	8.50	13.5	21.1	7.4	- .2	7.6	56

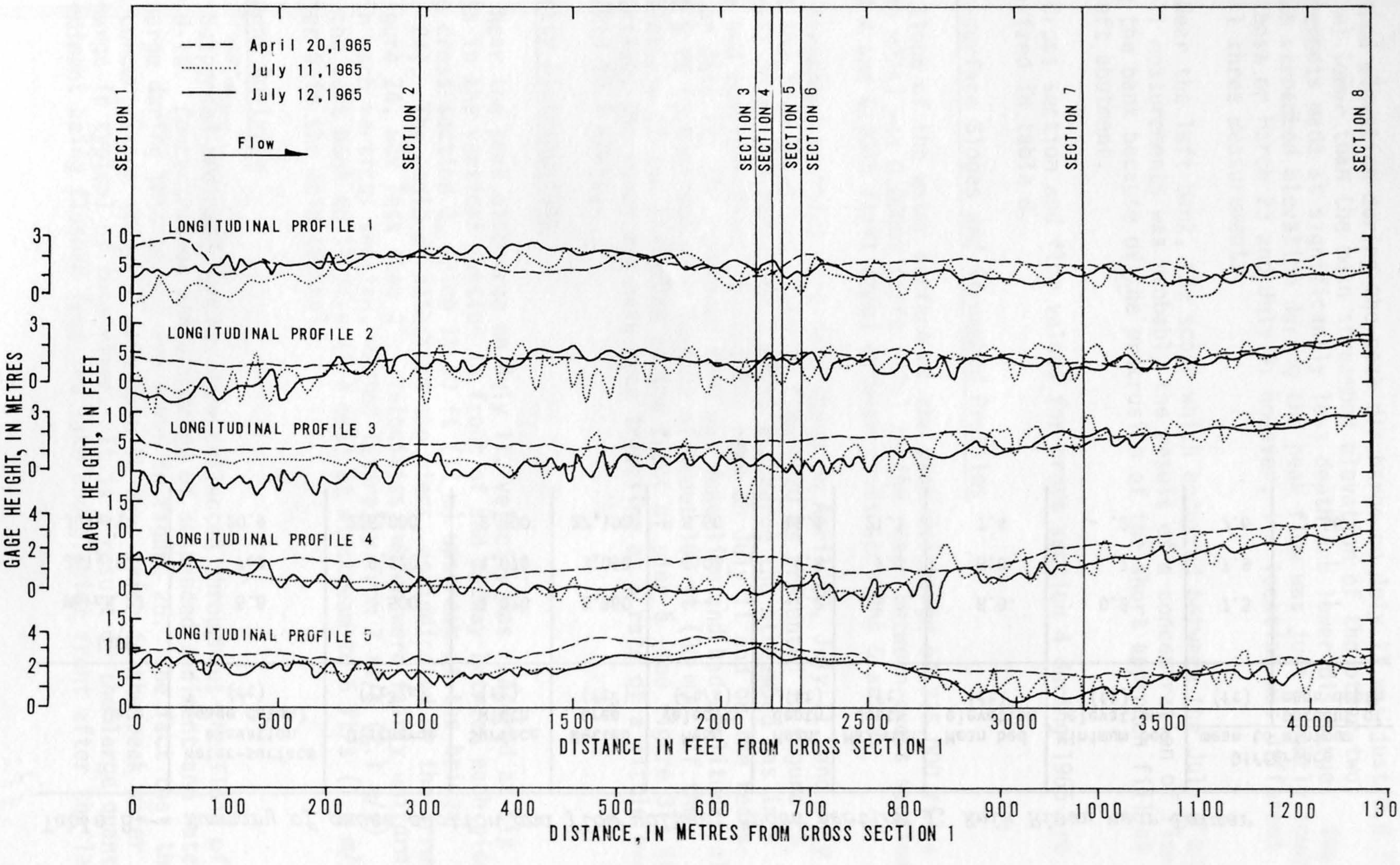


Figure 23.-- Longitudinal streambed profiles at the Knik River crossing.

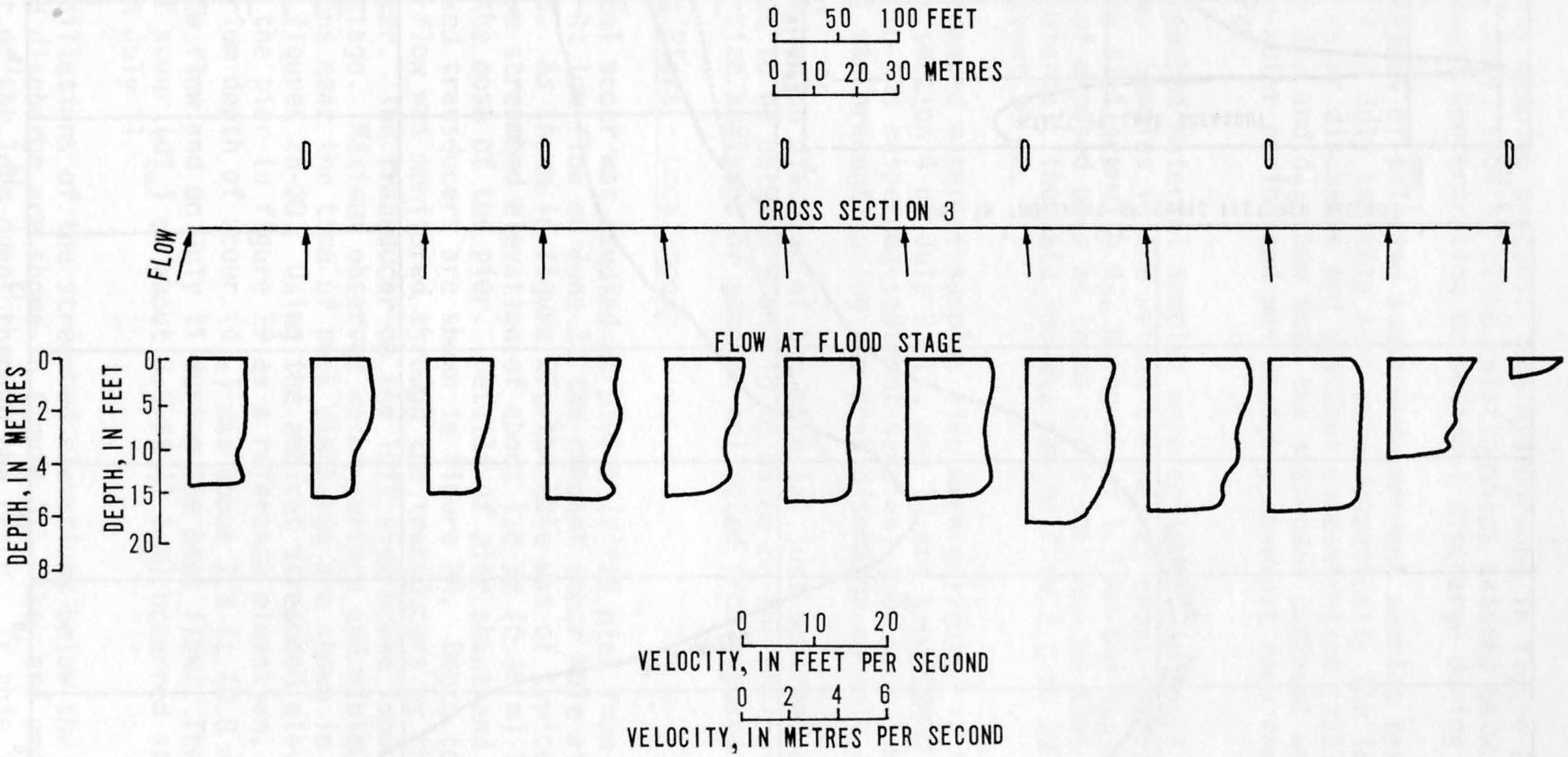


Figure 24.-- Velocity distribution and direction of flow in cross section 3 at Knik River bridge near Palmer, July 11, 1965

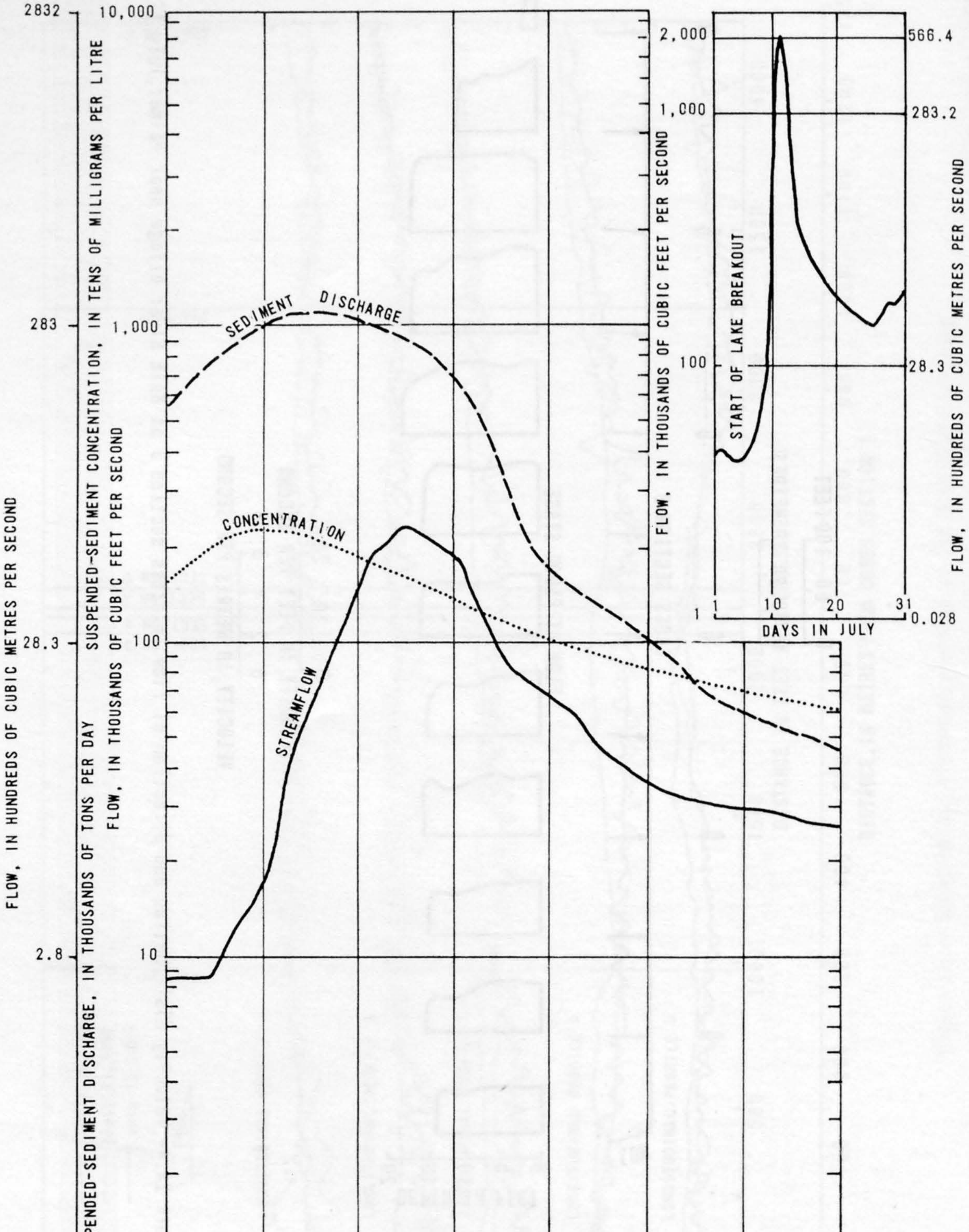


Figure 25.-- Sediment and discharge hydrographs, Knik River near Palmer.

begins its dumping process. The hydrographs in figure 25, for the entire month of July, show no close relationships existed between water discharge and sediment concentration or sediment discharge during this event.

Analyses of selected suspended-sediment samples taken at cross section 4 in July indicate even more dramatically the lack of comparison between water discharge and sediment concentration (table 9). The data for d_{50} and d_{90} show that the suspended sediment was coarser during the high-water period and was probably coarsest near the peak discharge on July 11.

Streambed material samples were collected on April 12 during very low flow. Samples in the wetted channel at cross sections 1, 3, and 8 were very similar, with d_{50} being about 1.3 mm and d_{90} about 9.0 mm. Samples of exposed bars at those cross sections on that day contained larger material; the data for d_{50} ranged from 2.2 to 10 mm and d_{90} from 9.3 to about 30 mm.

Streambed material samples also were collected in the main channel at cross section 4 on July 9, 11, and 12 and are listed in table 9. Similar to the suspended-sediment samples, they show that the coarsest material was present during the peak discharge on July 11.

The median diameter of the material just upstream of pier 5 is estimated to be between 5 and 10 mm based on the photograph in figure 26 and the size analyses of samples collected from exposed bars in April.

Scour at Piers - Local Scour

Local scour was studied at pier 5 (fifth pier from the left abutment). At low flow on June 23 the remnant scour hole at the pier was surveyed. As shown in figure 27, the hole was of typical shape and had a minimum streambed elevation of about 1.2 ft (0.36 m) located at and around the nose of the pier. Details of pier shape and the placement of four fixed transducers are shown in figure 28. Depth to the streambed at high flow was monitored through the transducers by the recording fathometer. The transducer on the left side broke loose during the rising stage. Maximum observed water-surface and minimum streambed elevations near the time of peak discharge are shown in relation to the pier in figures 28-30. Using the ambient streambed elevation to the left of the pier in figure 29 as a reference elevation, the local equilibrium depth of scour (d_{se}) was about 2.5 ft (0.8 m) both on June 23 at low flow and on July 11 during the peak flow. The maximum local depth of scour (d_{se}^*) of about 3.5 ft (1.1 m) occurred at 1530 and 1700 hours on July 11.

Oscillations of the streambed elevations below the transducers with time and discharge are shown in figure 31. They are probably caused by movement of the long dunes through the reach. If this is true, the 2 ft (0.6 m) amplitude of the oscillations are less than half that of

Table 9.-- *Sediment analyses, cross section 4, Knik River near Palmer*

Date	Sample type	Time (hrs)	Water temperature (°C)	Water discharge (ft^3/s)	Suspended sediment: Concentration (mg/l)	Suspended sediment: Discharge (tons/d)	Median particle diameter d_{50} (mm)	Particle diameter at which 90 percent are smaller - d_{90} (mm)	Methods of analysis
1965									
July 1	Susp. sed	1515	10.5	4,760	960	12,300	0.006	0.056	Pipet, visual accumulation
July 9	Susp. sed	1445	8.5	9,220	2,200	54,800	.021	.125	Pipet, visual accumulation
July 12	Susp. sed	0600	4.0	164,000	1,300	576,000	.020	.360	Pipet, visual accumulation
July 12	Susp. sed	1930	5.0	80,900	1,000	218,000	.008	.272	Pipet, visual accumulation
July 14	Susp. sed	1715	6.0	33,000	920	82,000	.008	.142	Pipet, visual accumulation
July 26	Susp. sed	1455	8.0	15,100	390	15,900	.003	.130	Pipet, visual accumulation
1965									
a July 9	Bed mat'l	1500	10.5	4,760	960	12,300	1.0	13.0	Sieve
a July 11	Bed mat'l	1415	4.0	234,000	1,400	--	2.5	25.0	Sieve
a July 12	Bed mat'l	0600	4.0	164,000	1,300	576,000	1.5	6.5	Sieve
a July 12	Bed mat'l	1930	5.0	80,900	1,000	197,900	1.0	9.0	Sieve

a Samples collected with U.S. BM-54 at 5 to 7 points in the main channel from the left bank to pier 6.

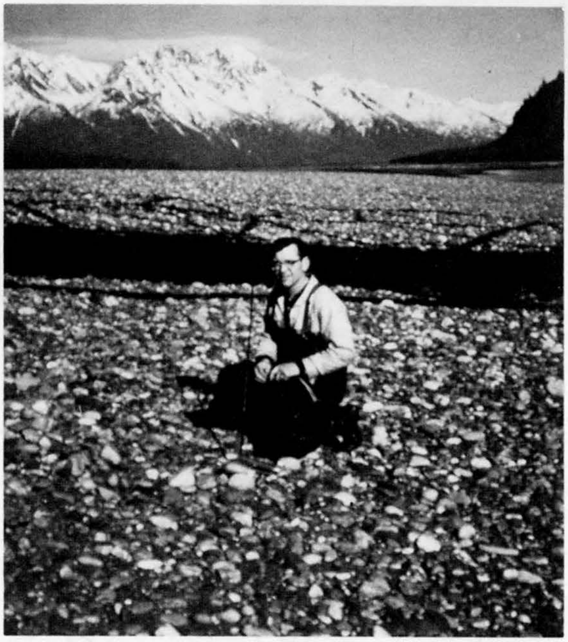
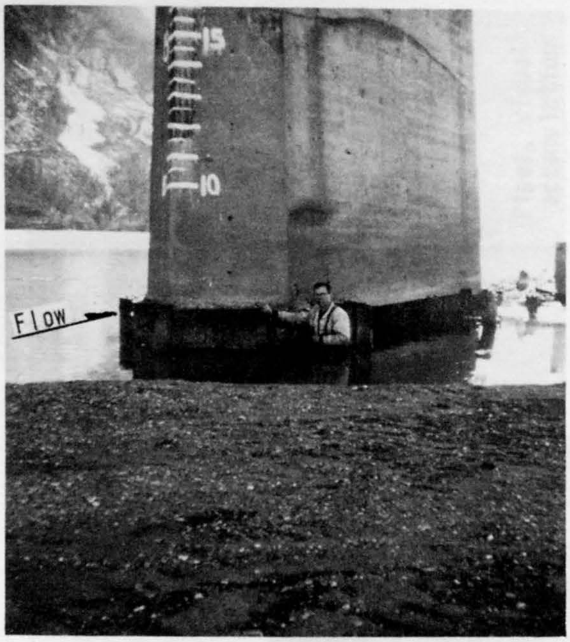


Figure 26.-- View upstream of bed material in vicinity of pier 5, bridge 539, on April 13, 1965.



View of pier at time of contour survey.

Contour interval 0.2 ft. (0.06m)

Gage height of 5.78 ft. (1.76 m)

Datum is gage datum 30.2 ft. (9.20 m) above mean sea level

0 1 2 3 4 FEET

0 1 METRE

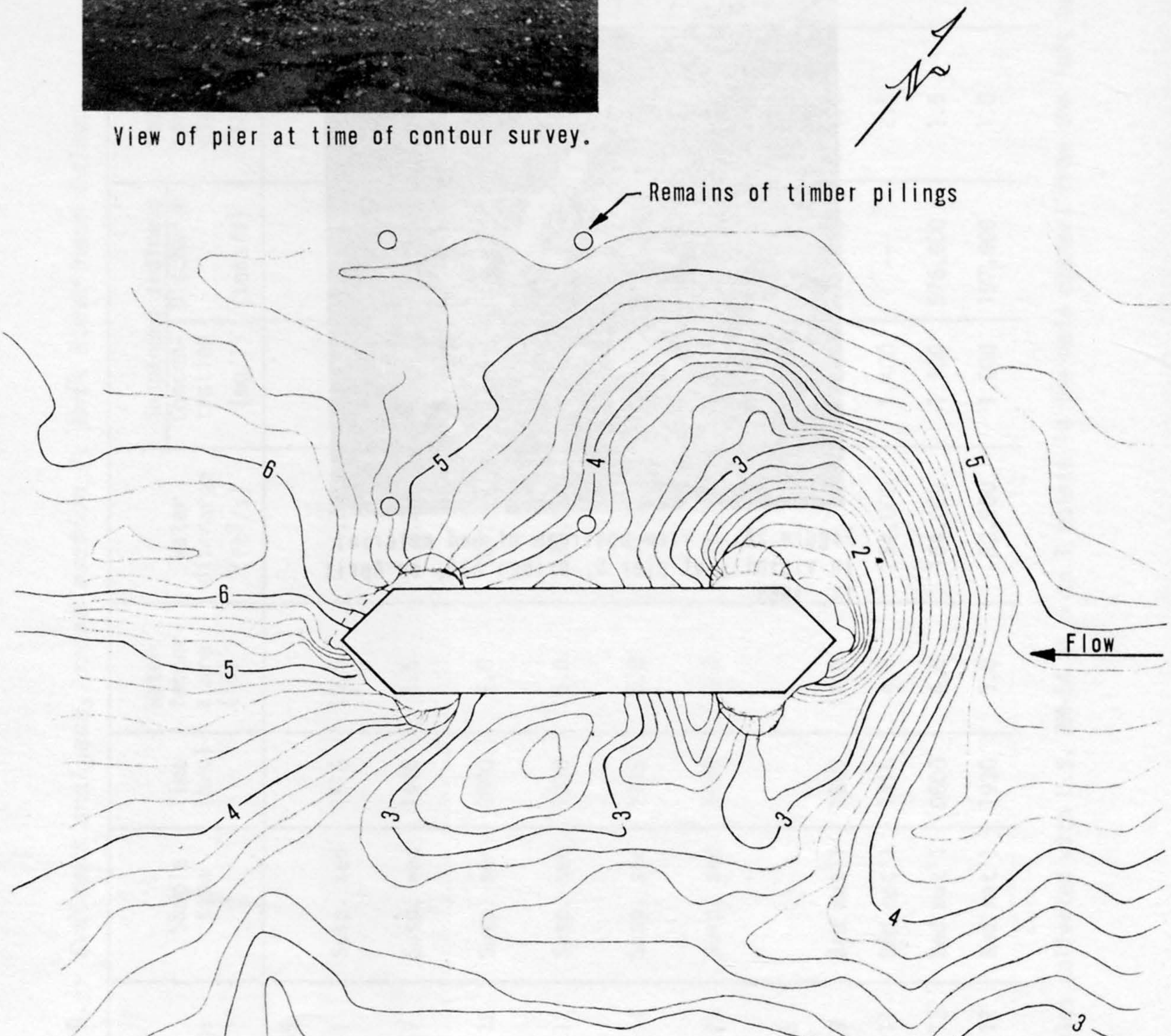


Figure 27.-- Contour map of streambed at pier 5, June 13, 1965, at Knik River near Palmer.

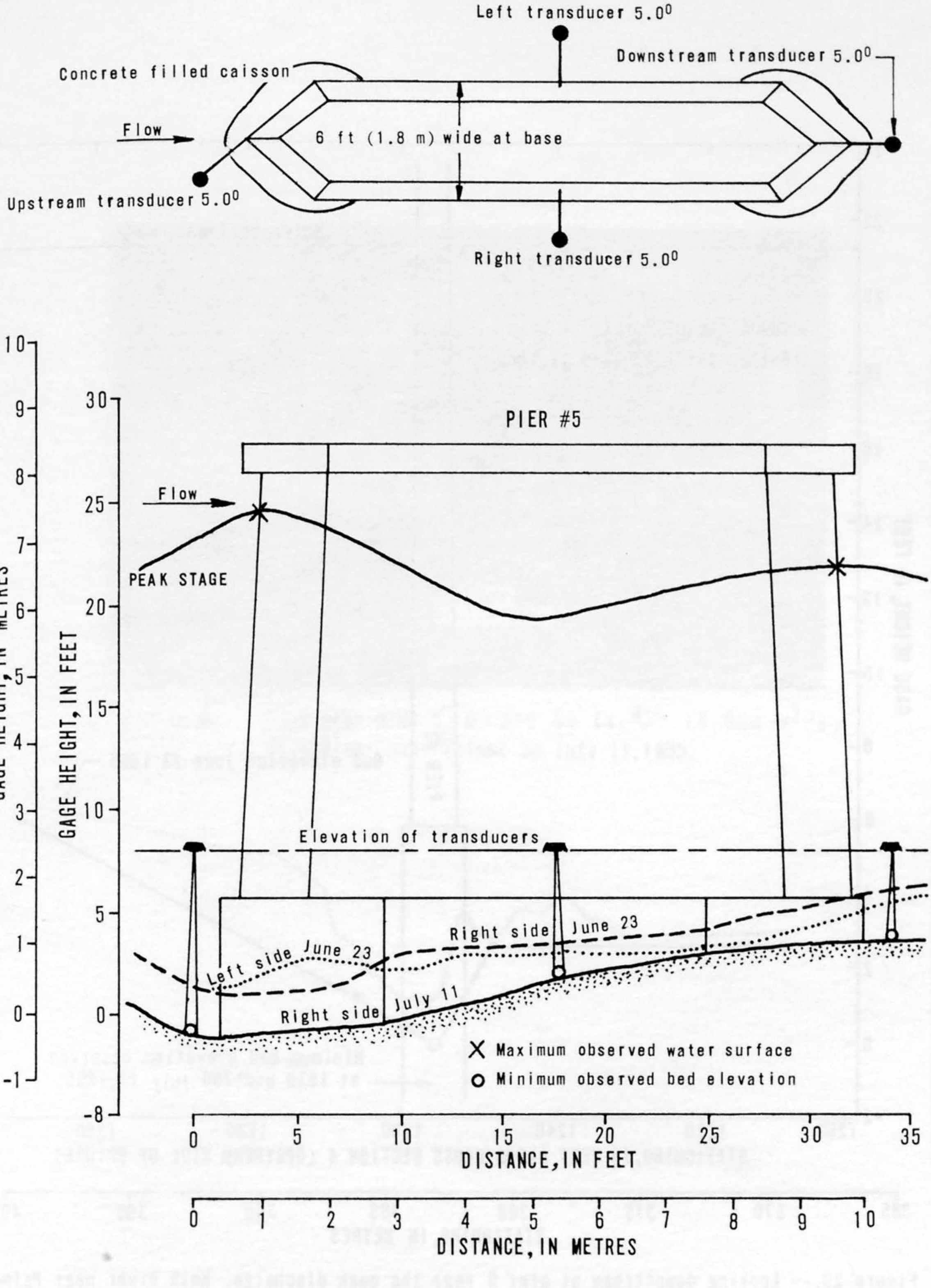


Figure 28.-- Pier 5 near the peak discharge, Knik River near Palmer on July 11, 1965.

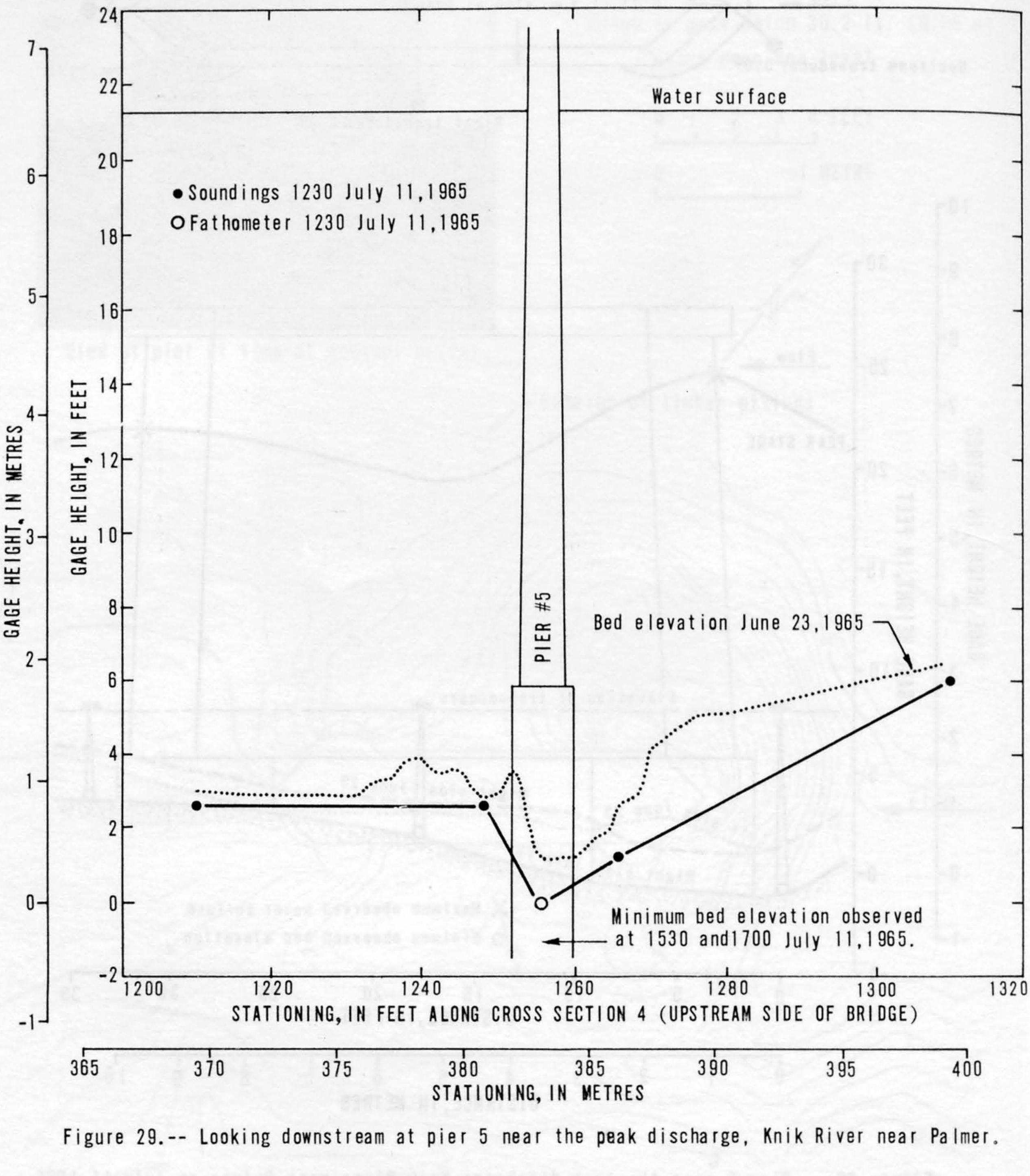


Figure 29.-- Looking downstream at pier 5 near the peak discharge, Knik River near Palmer.

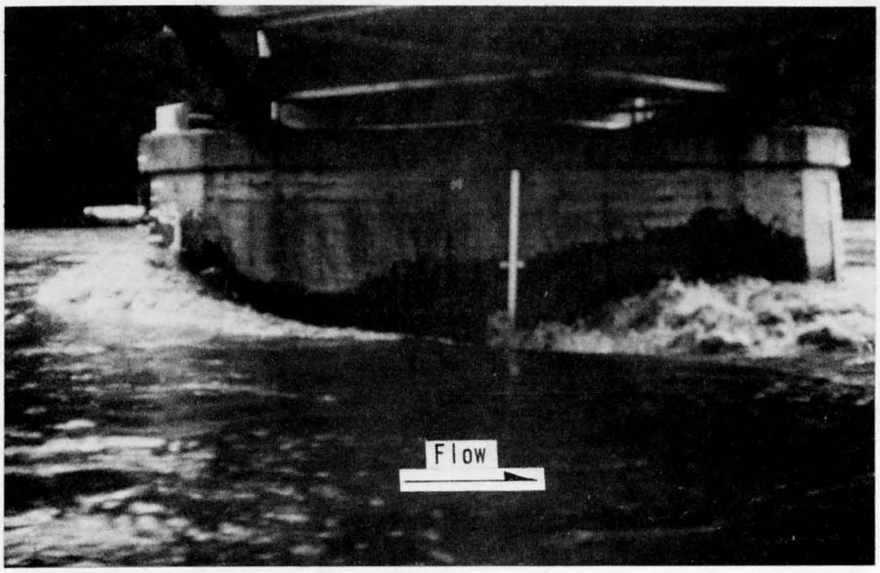


Figure 30.-- Flow past pier 5, Q = 236,00 ft.^{3}/s (6,684 m^3/s) Knik River near Palmer on July 11,1965.

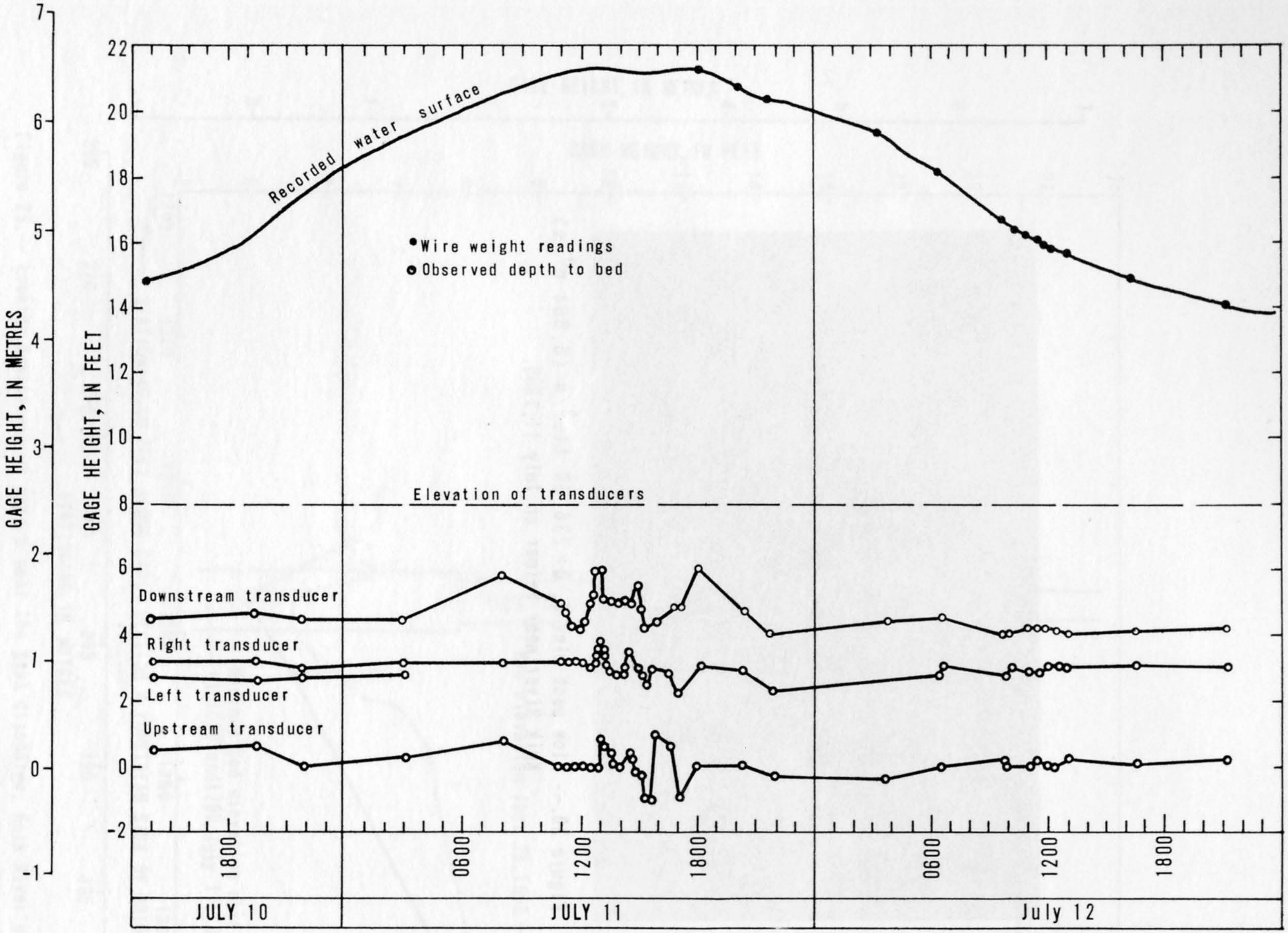


Figure 31.-- Indicated bed elevations at pier 5, Knik River near Palmer on July 10 - 12, 1965.

the 5 ft (1.5 m) amplitudes of the dunes. This agrees with Shen and others' (1969) observations of model studies where dunes were present.

At low flow riprap was visible at piers 1-4 but not at pier 5. However, during the survey of the scour hole (fig. 27), a "rock" was detected near the left side of the upstream caisson. Following the flood, riprap material was found at about the 1 ft (0.3 m) elevation.

Comparison of Measured Scour with Scour Calculated from Formulas

General Scour

Because there was no width constriction, there was no significant general scour. Of importance was the fact that the minimum streambed elevation during the peak flow was less than 1 ft (0.3 m) below that during low-flow conditions which is similar to the findings on the Susitna River near Sunshine at bridge 254.

Pier Scour

Using the projected width (8 ft or 2.4 m) of the exposed concrete-filled caisson as the pier width and an approach depth of 18 ft (5.8 m) the measured d_{se} and d^*_{se} are compared with computed depths of scour obtained from equations 5, 6, and 7.

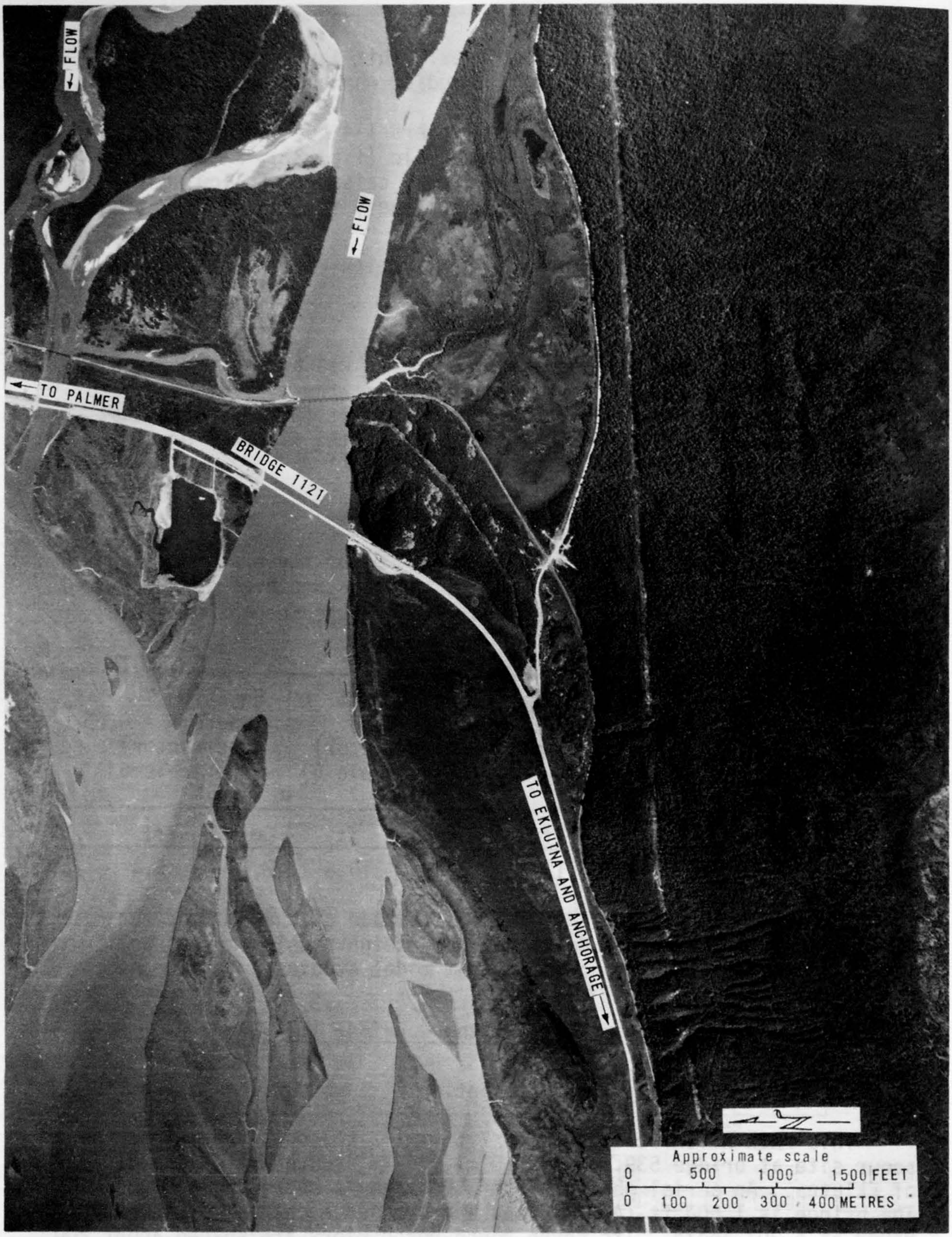
Measured				Equation 5 (x 0.85 nose factor)		Equation 6 (x 0.95 nose factor) (see table 6)		Equation 7 (where K = 0.95 nose factor) (see table 6)	
d_{se}		d^*_{se}							
m	ft	m	ft	m	ft	m	ft	m	ft
0.82	2.5	1.15	3.5	14.26	13.0	3.48	10.6	2.10	5.4

The measured oscillations of the streambed at the nose of the pier were about half the magnitude of the dune heights which agrees well with Shen and others' (1969) conclusion from model studies.

Knik River Near Eklutna - Bridge 1121

Description

This Knik River study site is 7.5 mi (12 km) downstream from the scour site at bridge 539. It is 10 mi (16 km) southwest of the village of Eklutna. An aerial photograph of the site is shown in figure 32. The bridge is 1,500 ft (457 m) long and crosses a channel of the Knik River at a 20° angle. Its seven round-nosed piers are spaced about 200 ft (61 m) apart and alined with the flow.



NORTH PACIFIC AERIAL SURVEY

Figure 32.-- Aerial view of Knik River crossing near Eklutna at bridge 1121 on July 10, 1972.

The Knik River at this location has a braided channel; the islands are inundated at flood stage. The channel streambed consisted of sand and gravel and was in a dune regime at the time of the peak discharge. In the study channel the river begins to widen as it flows beneath the Alaska Railroad bridge, about 2,000 ft (610 m) upstream from the highway bridge, and continues to widen for about 0.5 mi (0.8 km) where it merges with the Matanuska River to form the upper end of Knik Arm. Tides reach the highway bridge, but even at flood stage their effect probably is insignificant.

For a description of the cause and magnitude of flood discharges on the Knik River, the reader is referred to the description of the Knik River near Palmer at bridge 539.

Figure 33 is a plan view of the study site and shows the location and shapes of the channel at four cross sections.

The data described herein were collected at low and high flows during the period June 17 to 28, 1966. The high-water data is the latest flood breakout data for Lake George. A stage hydrograph at this site during the period June 15 to 28 and a discharge hydrograph at bridge 539 for the month of June are shown in figure 34.

The fourth pier from the left bank was instrumented with a single transducer at the nose of the pier. Depth to the streambed below the transducer was recorded by fathometer. Figures 35 and 36 show the bridge and the instrumented pier.

Summary and Discussion of Observations

Cross Sections - General Scour

Measurements from the upstream side of the bridge at cross section 3 were made at low flow on June 17, near the peak flow on June 24, and at medium flow on June 28. Figure 37 shows the three measurements superimposed on each other.

Because the bridge did not form a contraction, little general scour occurred in the cross section. Scour at some points was offset by fill at other points. Cross section and flow values are summarized in table 10.

Traverses were also made at cross sections 1, 2, and 4 during low flow and near the peak flow. No significant changes in streambed elevations occurred.

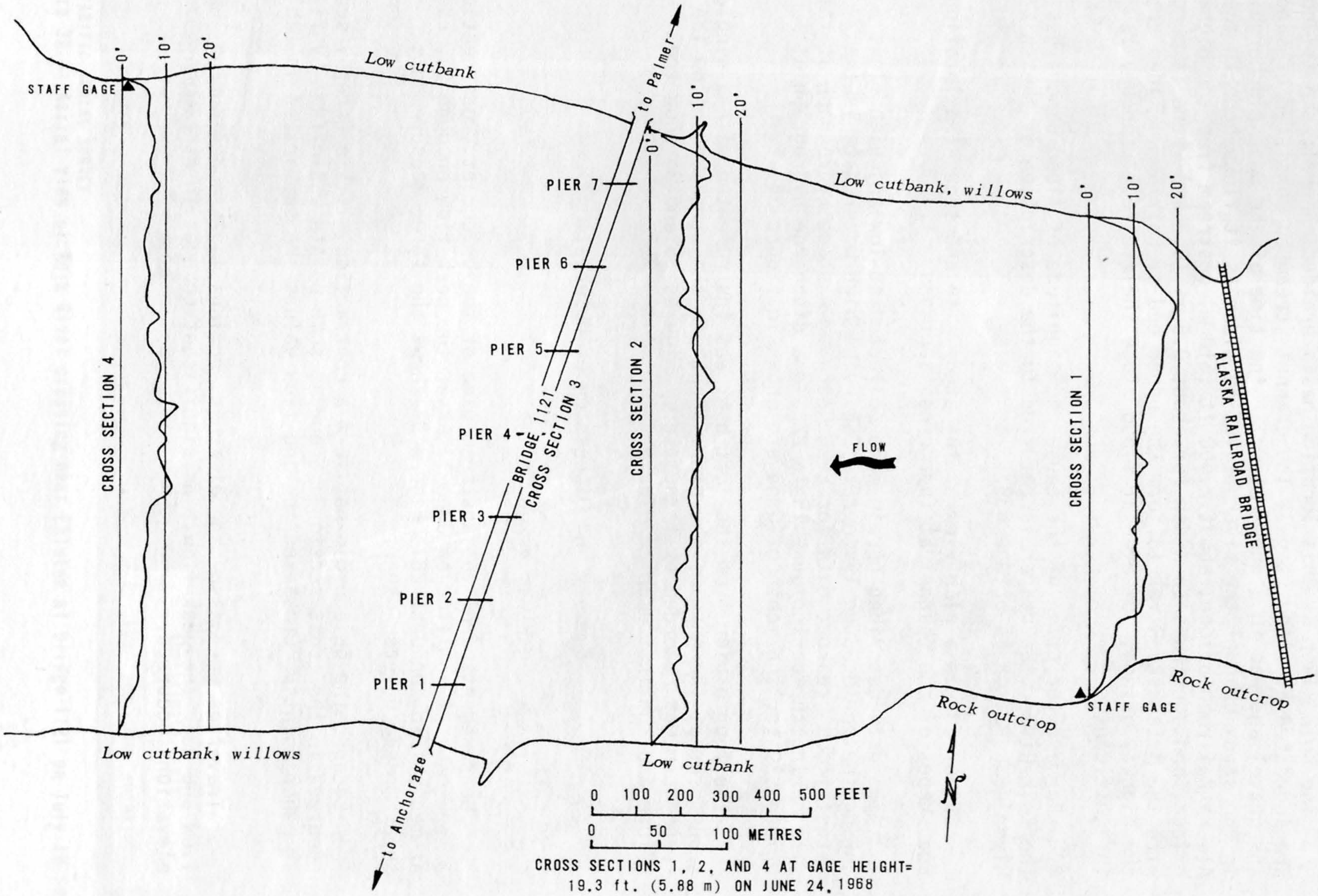
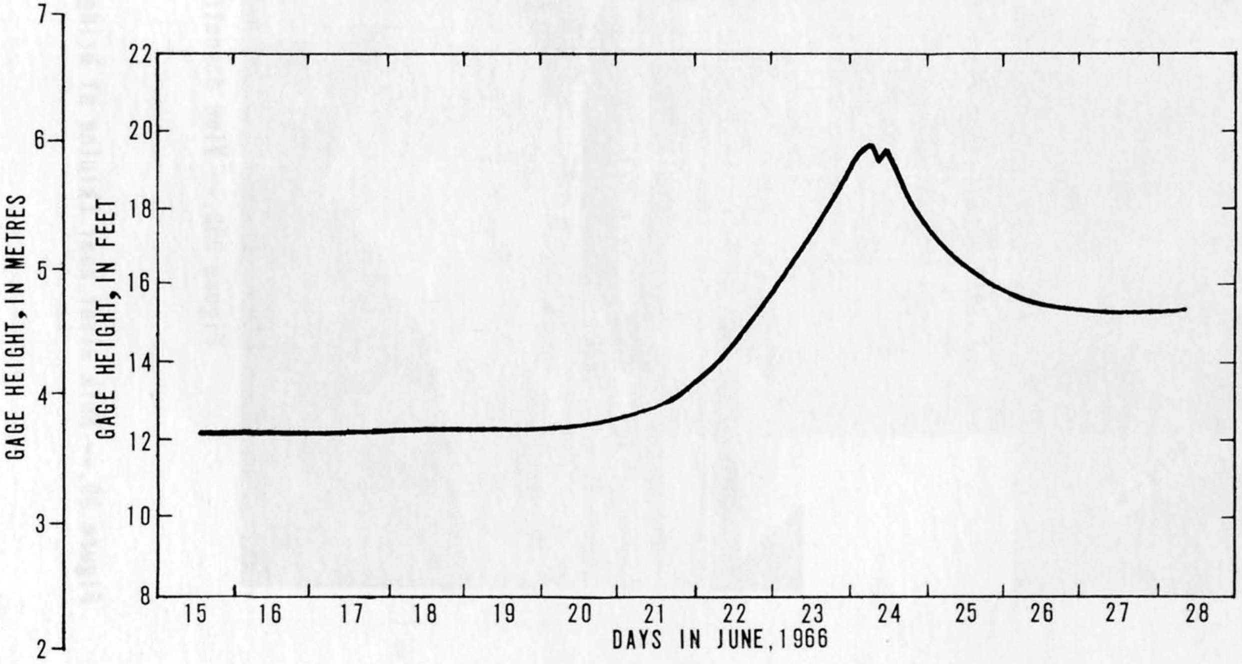


Figure 33.-- Plan view of Knik River near Eklutna at bridge 1121.



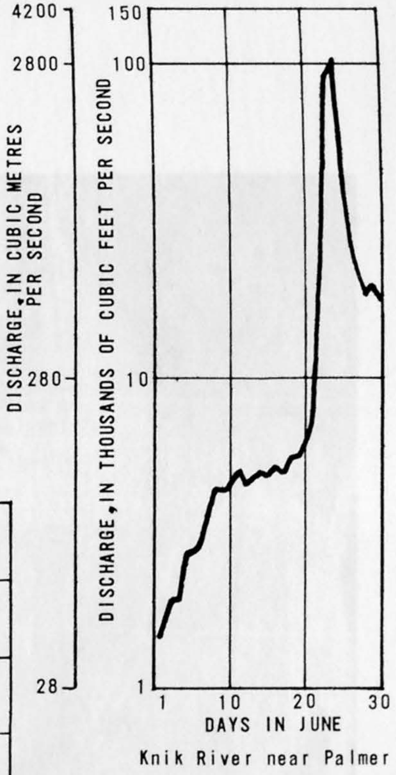


Figure 34.-- Stage hydrograph at Knik River near Eklutna.

Figure 35a.-- View upstream.

Figure 35b.-- View downstream.

Figure 35.-- Knik River near Eklutna at bridge 1121 on July 10,1965.

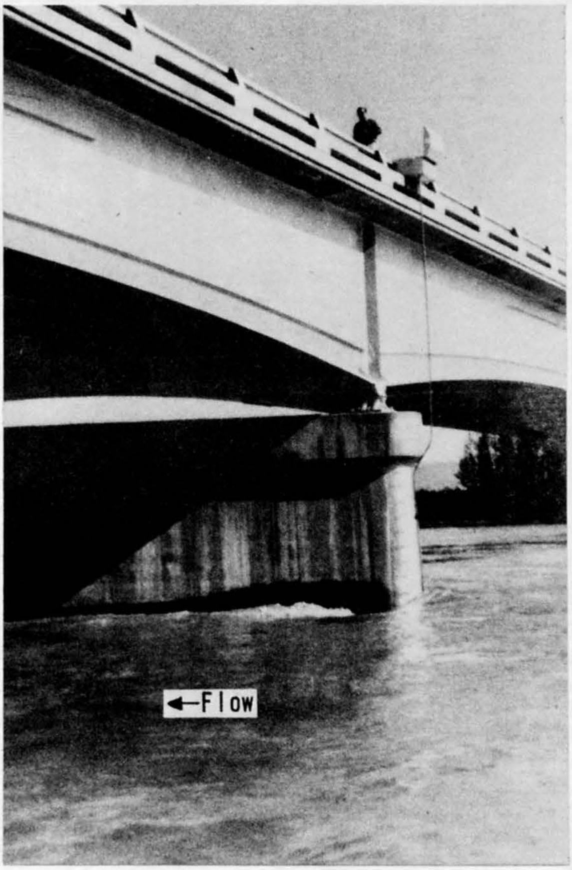


Figure 36.-- Instrumented pier 4, Knik River near Eklutna in June 1966.

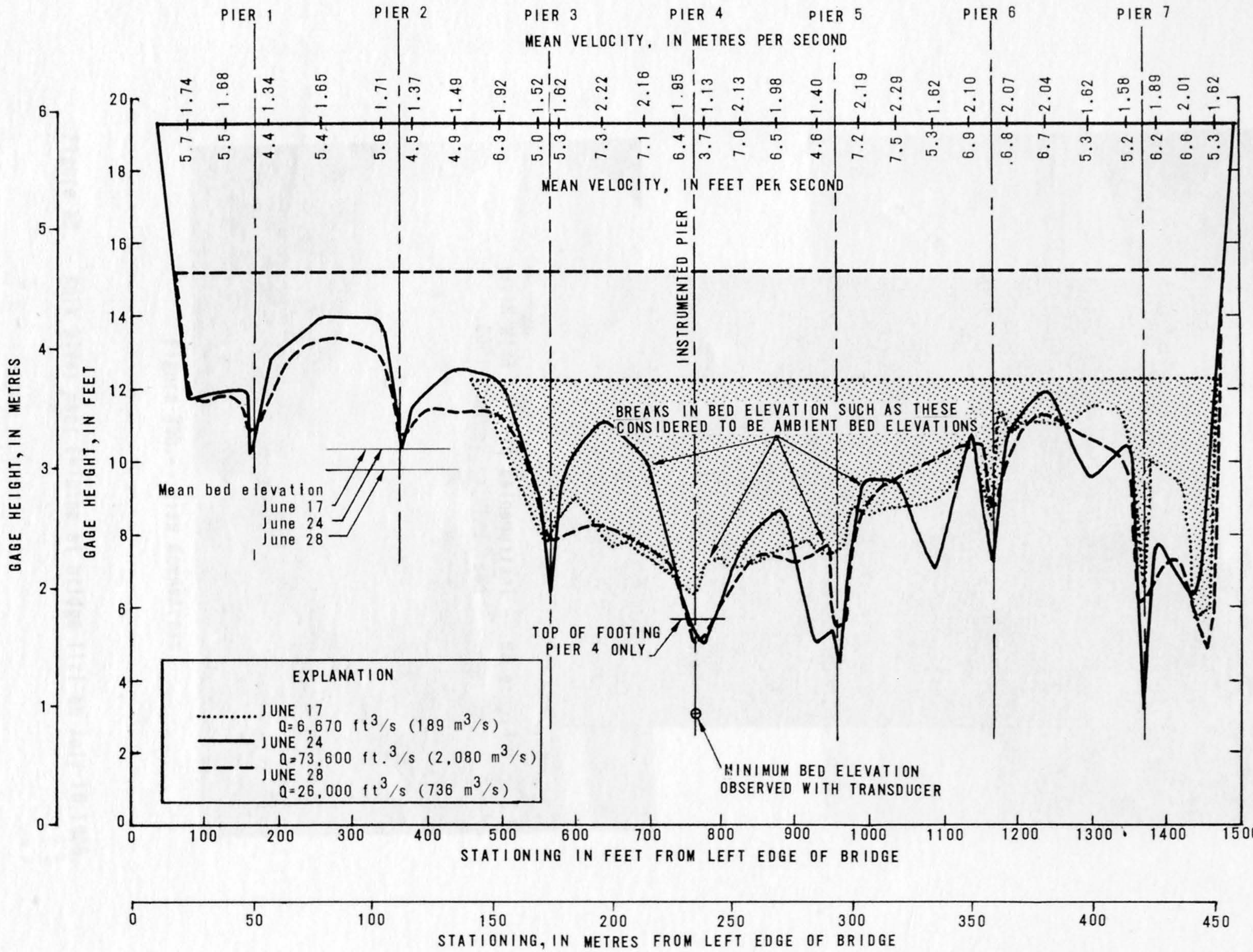


Figure 37.-- Comparison of cross-sectional traverses at cross section 3, Knik River near Eklutna, 1966.

Water-surface Slopes and Streambed Profiles

At the peak discharge of 73,600 ft^3/s (2,080 m^3/s) the water-surface slope from cross section 1 to the midpoint of cross section 3 was 0.0005 ft/ft (m/m) and from cross sections 3 to 8 was 0.0010 ft/ft (m/m).

A longitudinal bed profile was run between piers 4 and 5 during the peak flow. The recorded streambed form was dunes with amplitudes of about 4 ft (1.2 m) and wave lengths of 40 ft (12 m). Assuming the oscillations of the bed below the transducer at pier 4 (fig. 42) are indications of the rate of dune movement, the dunes on June 24 were moving between 0.1 ft/min (0.03 m/min) and 0.2 ft/min (0.06 m/min).

Velocity Distributions

Near the peak discharge on June 24, velocity was measured at six points in the vertical section upstream and between the piers along cross section 2. Figure 38 shows the velocity distribution in several vertical sections was irregular. The velocities were more erratic than those measured in 1965 at bridge 539 on the Knik River because the water depths measured at bridge 539 were almost twice those measured at bridge 1121 while the dune amplitudes at each site were within 20 percent of each other.

The direction of flow through the bridge opening was within 5 degrees of being parallel with the long dimension of the piers.

Mean velocity within the measured vertical sections across cross section 2 ranged from 4.5 to 6.5 ft/s (1.37 to 1.98 m/s). At cross section 3 the mean velocity ranged from 4.5 to 7.5 ft/s (1.37 to 2.28 m/s). The mean velocity upstream of instrumented pier 4 was 5.8 ft/s (1.77 m/s).

Sediment Analyses

At cross section 2, suspended-sediment samples were collected at four points in each vertical section upstream and between each pier. The distribution of sediment defined by these samples is shown in figure 39. Concentrations near the water surface were about 1,000 mg/l throughout the cross section. At 0.5 ft (0.15 m) above the streambed the concentrations ranged from 2,000 mg/l from the sides of the channel to 6,000 mg/l near the center. The mean concentration and discharge of suspended sediment for the entire cross section 2 at the peak was 1,910 mg/l and 380,000 tons/d (345,000 tonnes/d), respectively.

Additional point samples for particle-size analyses were collected at cross section 2 upstream of piers 3 and 4 and midway between piers 4 and 5, 6 and 7, and 7 and 8. The distribution of clay, silt, and sand in these vertical sections is illustrated in figure 40. The clay and

Table 10.-- *Summary of cross section and flow values, cross section 3, Knik River near Eklutna*

Date	Water-surface elevation (gage datum) (ft)	Discharge (ft^3/s)	Surface width (ft)	Wetted area[a] (ft^2)	Mean velocity (ft/s)	Maximum mean velocity (ft/s)
1966						
June 17	12.2	6,670	993	3,050	2.2	3.1
June 24	19.4	73,600	1,455	12,500	5.9	7.3
June 28	15.4	26,000	1,417	7,600	3.4	4.3

a Area corrected for angle which section makes with direction of flow.

b At locations other than at the piers.

Mean depth (ft)	Maximum depth[b] (ft)	Mean bed elevation (ft)	Minimum bed elevation (ft)	Difference mean to minimum (ft)	
					percent of mean depth
1.9	5.4	10.3	6.8	3.5	180
9.1	14.0	10.3	5.4	4.9	54
5.7	9.0	9.7	6.4	3.3	58

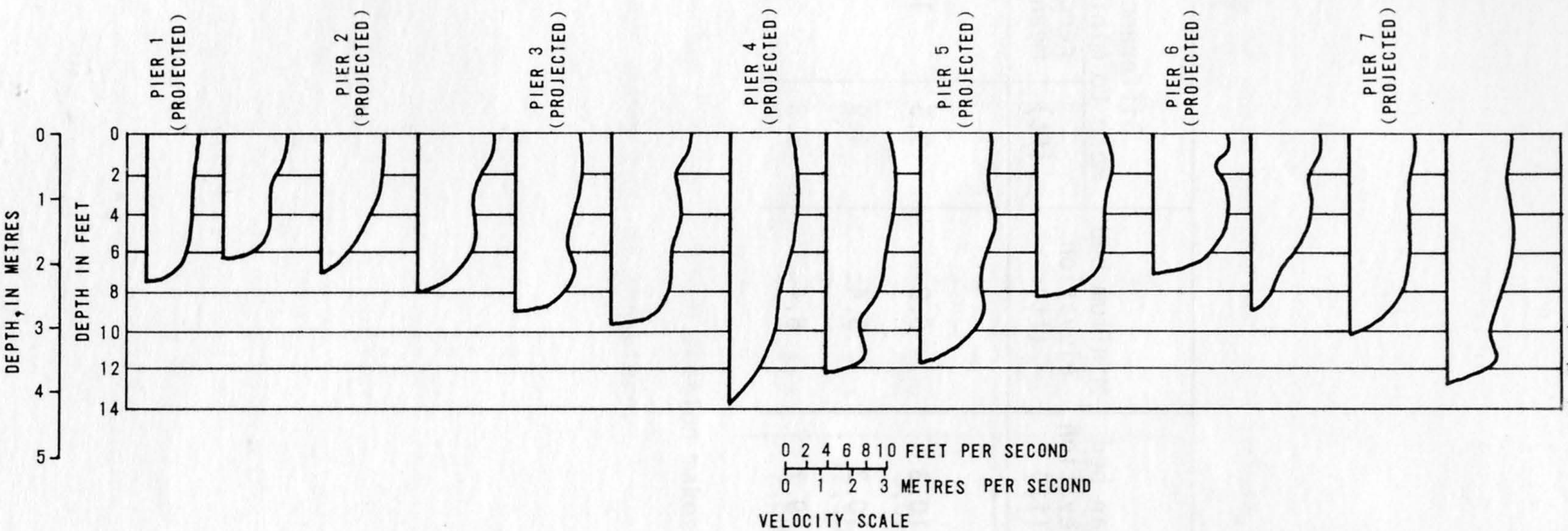


Figure 38--Velocity distribution along cross section 2, Knik River near Eklutna on June 24, 1966.

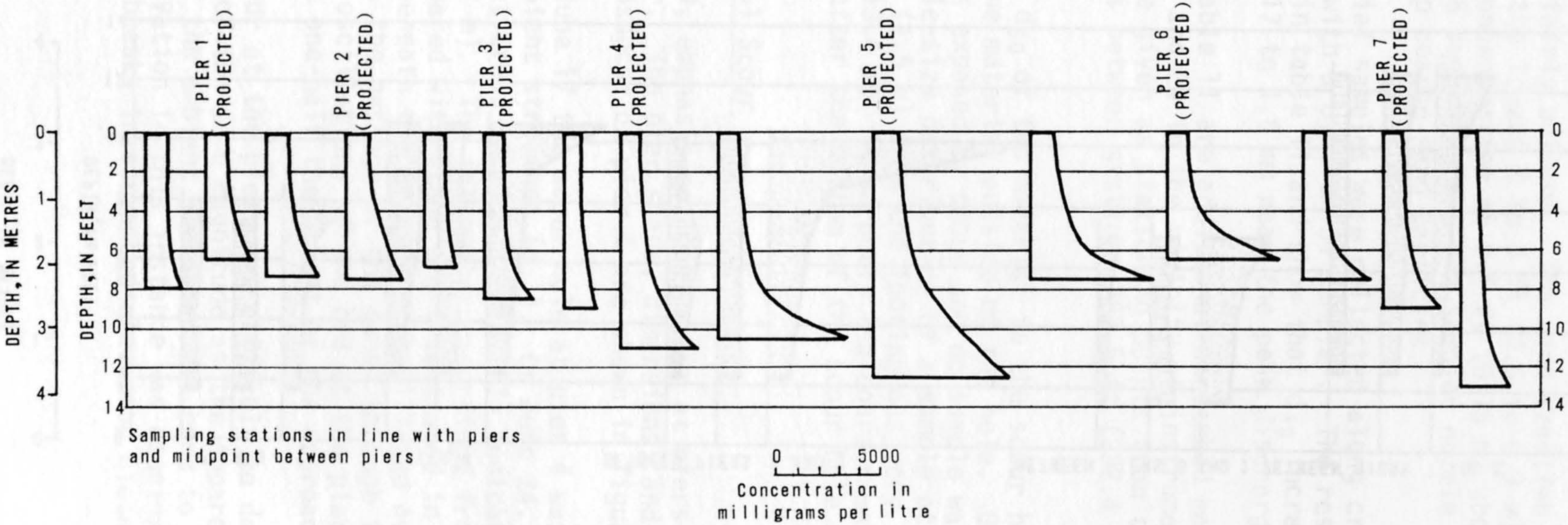


Figure 39--Suspended-sediment distribution along cross section 2, Knik River near Eklutna on June 24, 1966.

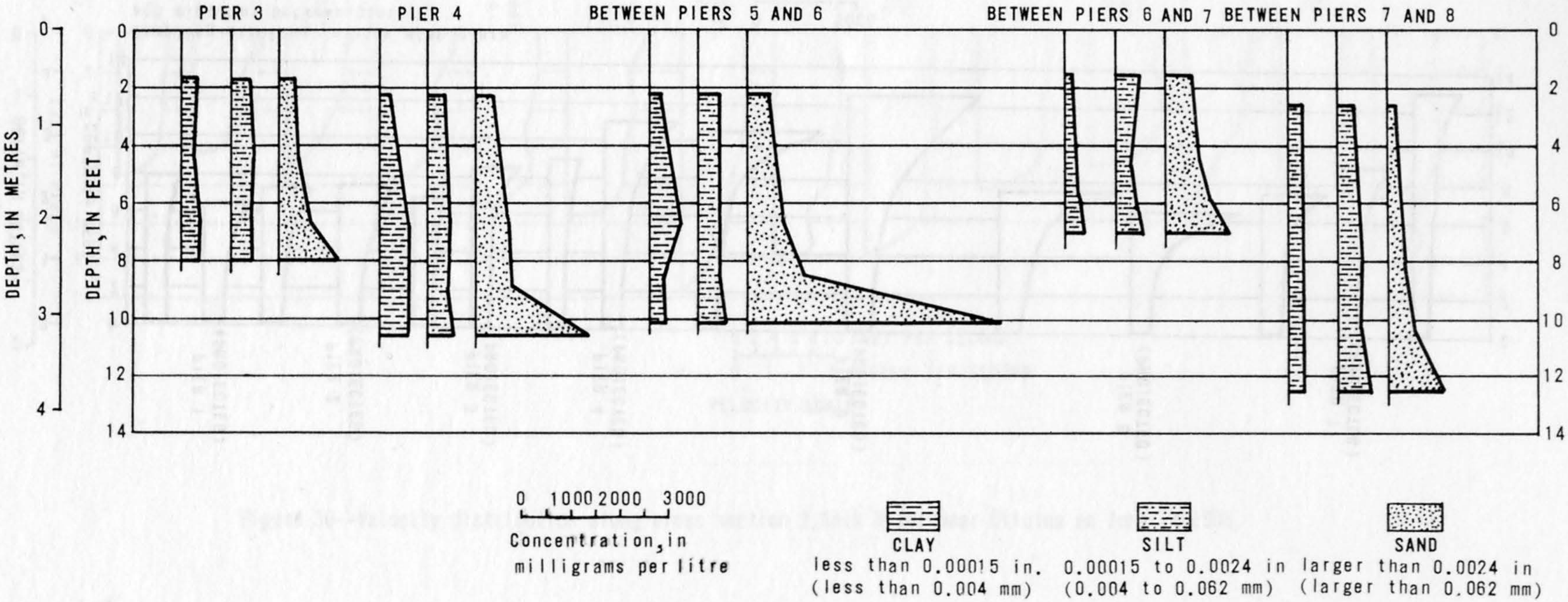


Figure 40--Distribution of suspended-sediment sizes along cross section 2, Knik River near Eklutna on June 24, 1966

silt fractions were fairly uniform. The sand fraction increased with depth especially in the lowest 2 to 3 ft (0.6 to 0.9 m). The extremely high value of sand concentration at 0.5 ft (0.15 m) above the bottom between piers 5 and 6 suggests that the sampler nozzle may have dug into the bed and picked up coarse sand particles.

Streambed material samples were collected along cross section 3 on June 17, 23, and 24 with a U.S. BM-54 sampler. The results of the analyses are listed in table 11 and show that d_{50} increased from 0.58 mm at low flow on June 17 to 1.8 mm near the peak discharge on June 24.

Also given in table 11 are additional streambed material samples collected on June 17 and 24 in the vicinity of instrumented pier 4. The collection points are given as stationing points from the left bank and the base of pier 4 is between stations 762.5 ft (232.4 m) and 767.5 ft (233.9 m).

On June 17, the d_{50} of the material in the scour hole was about four times that of the material outside of the hole. On June 24, the footing of pier 4 was exposed by scour and no sample was obtained next to the pier. Particle-size distribution of a sample at station 778 ft (237.1 m) about 5 ft (1.5 m) from the footing was very similar to that at station 700 ft (213.4 m). Apparently the coarser material in the hole was swept away after the bottom of the scour hole reached the footing.

Scour at Piers - Local Scour

As at bridge 539, emphasis on local scour at piers was placed on the instrumented pier. The pier shape, dimensions, and the location of the single fixed transducer on pier 4 are shown in figure 41.

At low flow on June 17 the scour hole at pier 4 was about 1.0 ft (0.3 m) below the ambient streambed (d_{se}). On June 24, during the peak flow, the streambed at cross section 3 (fig. 37) indicated d_{se} at pier 4 was about 4.0 ft (1.2 m). The fathometer recordings from the transducer at the pier nose compared with stage are illustrated in figure 42. The streambed elevation beneath the transducer oscillated over a 2 ft (0.6 m) range probably due to the passage of the dunes through the scour hole. The amplitude of the oscillations of the bed at the pier nose, as on bridge 539, was about one-half the height of the approaching dunes.

The depth of scour at the piers was difficult to determine because the dune heights are of the same magnitude as the apparent depth of local scour holes and the ambient streambed was hard to describe. The ambient streambed elevation in this instance was considered to be the average of the sharp breaks in streambed elevation closest to the piers (see fig. 37).

Table 11.-- *Sediment analyses, cross section 3, Knik River near Eklutna*

Date	Sample type	Sampling location	Time (hrs)	Water temperature (°C)	Water discharge (ft^3/s)	Suspended sediment Concentration (mg/l)	Suspended sediment Discharage (tons/d)	Median particle diameter d_{50} (mm)	Particular diameter at which 90 percent are smaller - d_{90} (mm)	Method of analyses
1966										
June 17	Bed mat'l	--	1800	9.5	6,670	--	--	0.58	7.0	Sieve
June 23	Bed mat'l	--	1715	4.5	50,000	--	--	1.1	8.0	Sieve
June 24	Bed mat'l	--	0800	4.0	73,600	1,910	380,000	1.8	13.0	Sieve
June 17	Bed mat'l	Sta. 700	1800	9.5	6,670	--	--	.53	1.6	Sieve
June 17	Bed mat'l	Sta. 760	1800	9.5	6,670	--	--	4.0	16.0	Sieve
June 17	Bed mat'l	Sta. 800	1800	9.5	6,670	--	--	.42	2.3	Sieve
June 24	Bed mat'l	Sta. 700	0800	4.0	73,600	--	--	1.4	4.8	Sieve
June 24	Bed mat'l	Sta. 778	0800	4.0	73,600	--	--	1.6	5.4	Sieve

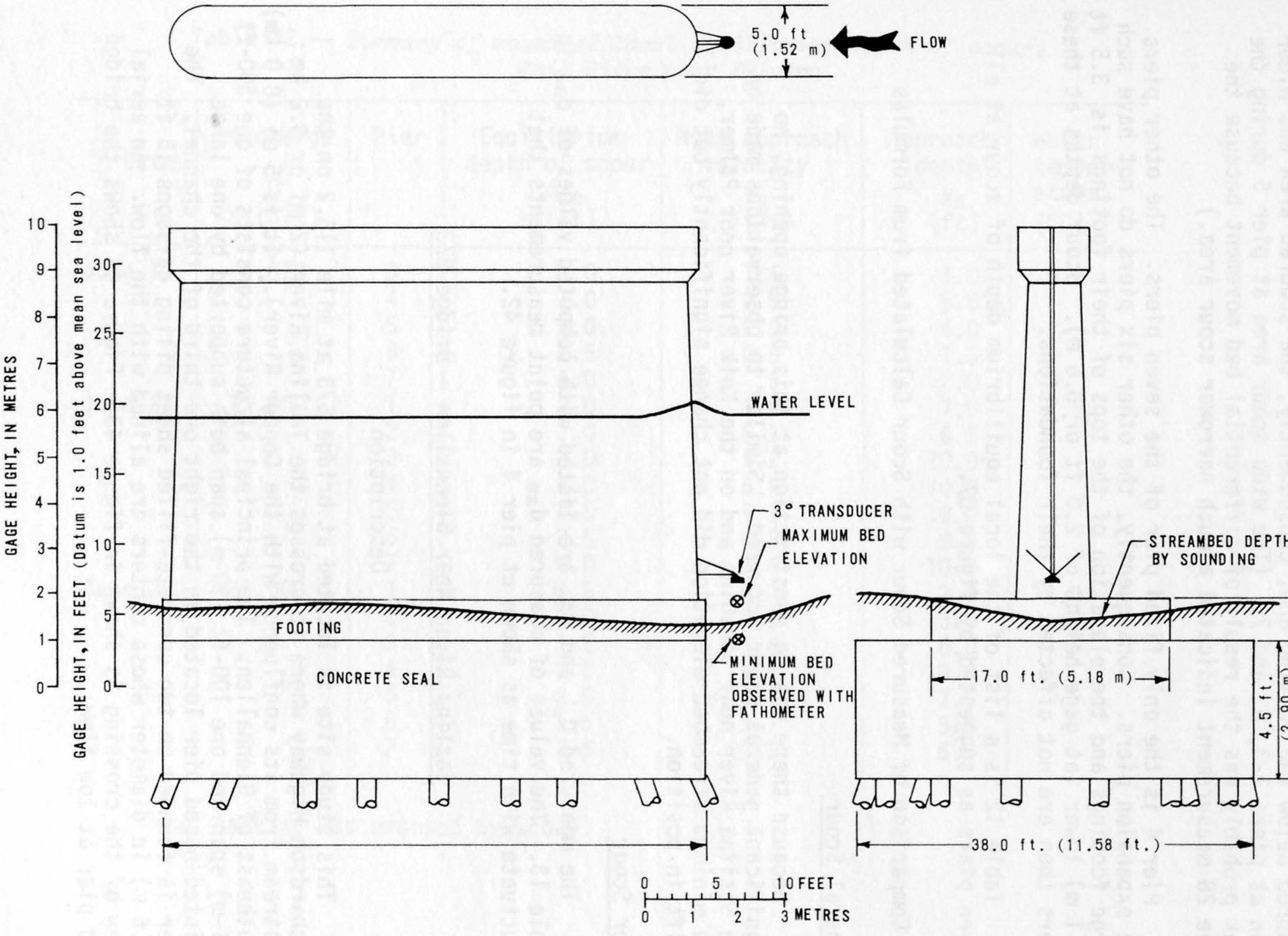


Figure 41.--Pier 4 and streambed near peak discharge, Knik River near Eklutna, on June 24, 1966.

Maximum scour, due solely to the pier, at pier 4 during the peak probably was not observed. As figure 41 shows, the streambed was scoured below the footing and, consequently, the scoured area was wider than at piers 1-3, 6, and 7. (The wide scour area at pier 5 during the peak probably was the result of differential bed movement because the June 28 measurement indicated a much narrower scour area.)

Pier 4 is the only fixed pier of the seven piers. The other piers are expansion piers. Consequently, the other six piers do not have such large footings and the elevation of the tops of their footings is 3.5 ft (1.1 m) lower (at gage height of 2.0 ft or 0.6 m). Scour depths at these piers then are not affected by their foundations.

Table 12 is a list of the local equilibrium depth of scour at all seven piers as suggested by figure 37.

Comparison of Measured Scour with Scour Calculated from Formulas

General Scour

Because there was no constriction at this bridge opening, no significant general scour occurred. Similar to observations made on the Susitna River near Sunshine and on the Knik River near Palmer, the minimum streambed elevation did not change significantly but did shift in position.

Pier Scour

The measured d_{se} and d_{se}^{*} are listed with computed values of d_{se} in table 13. The values of measured d_{se} are point measurements that fluctuate with time as shown at pier 4 in figure 42.

Tazlina River Near Glennallen - Bridge 573

Description

This study site is located at bridge 573 at mile 116.2 on the Richardson Highway where it crosses the Tazlina River (2 mi or 3.2 km upstream from its confluence with the Copper River). It is 5 mi (8.0 km) southeast of Glennallen. The principal structure consists of one 300-ft (91-m) span and one 100-ft (30-m) span both supported by one large pointed-nosed pier located in the right one-third of the channel. The pier is founded on two concrete-filled sheet piling caissons 15 ft (4.6 m) in diameter whose centers are alined with the flow. An aerial view of the crossing is shown in figure 43. Figure 44 shows the bridge and pier at low flow.

Table 12.-- *Summary of measured local equilibrium depth of scour, Knik River near Eklutna*

Date	Discharge Q (ft^3/s)	Pier	Equilibrium depth of scour d_{se} (ft)	Mean approach velocity v_a (ft/s)	Approach depth y_a (ft)	Water temperature (°C)
June 17	6,670	3	1.0	1.6	4.0	9.5
		4	1.0	2.5	5.0	
		5	1.0	2.9	4.0	
		6	2.5	0.9	1.5	
		7	4.0	.5	2.0	
June 24	73,600	1	2.0	5.0	7.0	4.5
		2	2.0	5.1	6.5	
		3	3.0	5.2	10.0	
		4	a 4.0	6.5	10.5	
		5	4.5	5.9	10.0	
		6	3.5	6.8	8.5	
		7	6.0	6.0	10.0	
June 28	26,000	1	1.5	3.1	3.0	4.0
		2	2.0	3.2	3.0	
		3	1.5	3.6	6.0	
		4	a 2.0	3.8	8.0	
		5	2.5	3.7	7.5	
		6	1.5	3.7	5.0	
		7	2.5	3.2	6.5	

a Footing was exposed by scour.

Table 13.-- *Comparison of measured and calculated local depths of scour with continuous sediment motion, Knik River near Eklutna*

Pier	Date	Observed		Calculated		
		d_{se} (ft)	d^{*}_{se} (ft)	Equation 5 x 0.90 nose factor (ft)	Equation 6 (ft)	Equation 7 (ft)
1	June 24	2.0		8.3	7.0	4.8
	28	1.5		6.4		
2	24	2.0		8.1		
	28	2.0		6.4		
3	June 17	1.0		7.0		
	24	3.0		9.2		
	28	1.5		7.9		
4	17	1.0		7.5		
	a 24	4.0	6.0	9.5		
	a 28	2.0		8.6		
5	17	1.0		7.0		
	24	4.5		9.2		
	28	2.5		8.4		
6	17	2.5		5.2		
	24	3.5		8.8		
	28	1.5		7.5		
7	17	4.0		5.7		
	24	6.0		9.2		
	28	2.5		8.1		

a Footing was exposed by scour.

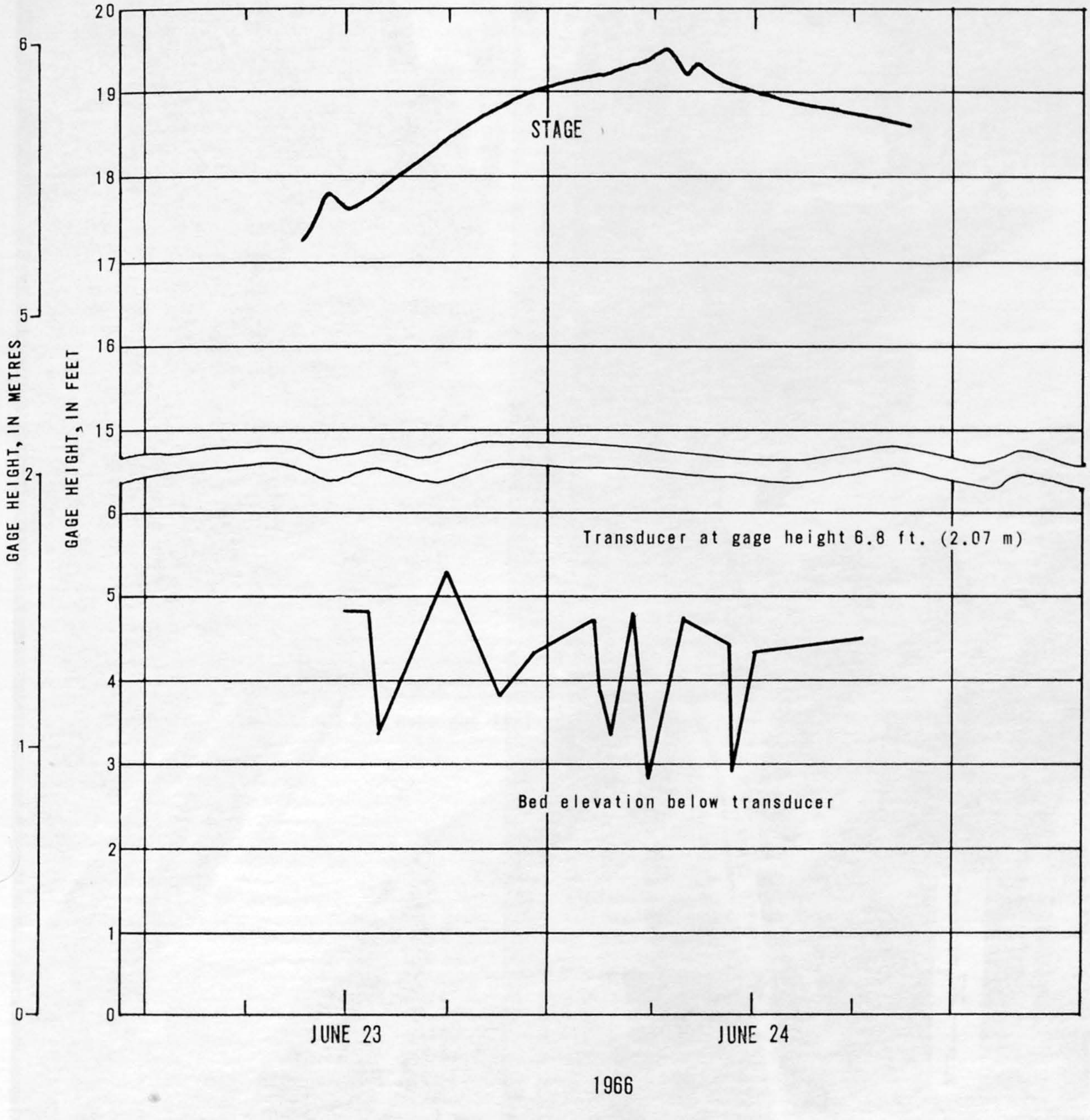
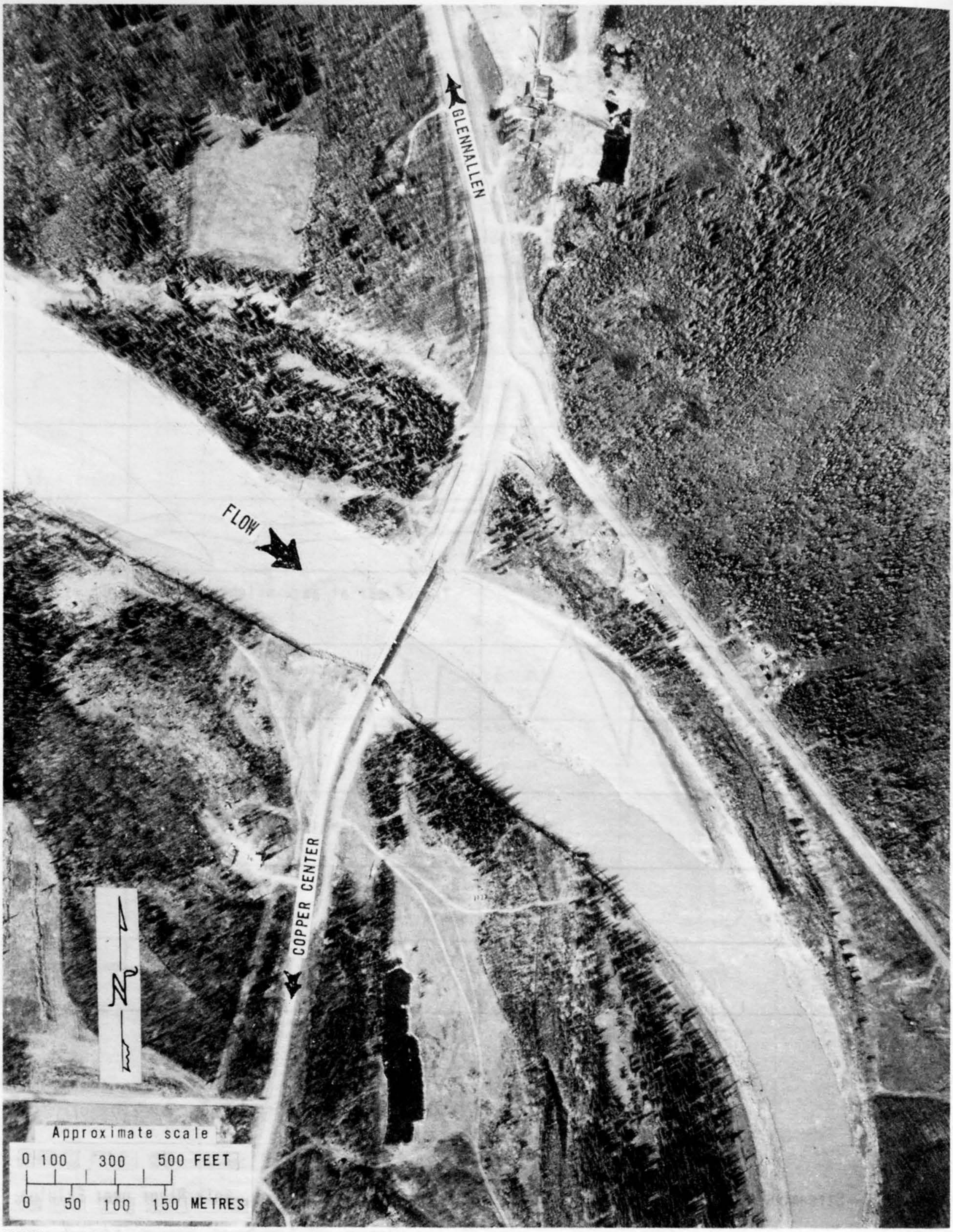


Figure 42--Streambed elevation and stage at nose of pier 4 during peak flow, Knik River near Eklutna.



NORTH PACIFIC AERIAL SURVEY

Figure 43--Aerial view of Tazlina River near Glennallen, bridge 573, May 25, 1972.

Figure 44--Bridge 573 over the Tazlina River at low flow, April 22,1969

The Tazlina River flows from a large glacier-fed lake about 26 mi (42 km) west of the study site. The variations in discharge in the Tazlina River are subdued by the lake. Almost annually one of several glacier-dammed lakes above the lake breaks out to produce floodflows in the river. Post and Mayo (1971) discuss these lake breakouts in their report on glacier-dammed lakes. Stream-gaging records have been maintained at the bridge since 1951. Recorded annual peak flows range from a low of 15,300 ft^3/s (433.3 m^3/s) in 1956 to a high of 60,700 ft^3/s (1,719 m^3/s) in 1962. The mean annual and 50-year recurrence-interval floods are about 25,000 and 78,000 ft^3/s (708 and 2,210 m^3/s), respectively. The drainage area is approximately 2,670 mi^2 (6,910 km^2).

Brice (1971) suggests that the recent history of the Tazlina River has been one of slow degradation. He also discusses a large meander in the river about 4,000 ft (1,220 m) upstream from the bridge which may eventually be cut off by erosion.

As shown in figure 43, alternate bars composed largely of gravel and cobbles but containing occasional boulders are located in the study area. Some of the larger boulders are visible in the photograph. Heavy riprap protection is provided on both banks at the bridge opening and on the right bank for a distance of 200 ft (61 m) above the bridge.

The largest flood during the study, a peak discharge of 39,700 ft^3/s (1,124 m^3/s), occurred in September 1971. The data collected during this flood, whose recurrence interval is about 6 years, are described below. Hydrographs of river stage and discharge during the flood are shown in figure 45.

Summary and Discussion of Observations

Cross Sections - General Scour

Measurements for scour have been made at an approach section 400 ft (120 m) above the bridge and on the upstream side of the bridge. Soundings at high flow in the approach section were made using a boat and fathometer. At the bridge soundings were made using a 150-lb (68-kg) sounding weight. Waves on the water surface at high flows made sounding difficult and the accuracy is probably on the order of ± 0.3-0.5 ft (± 0.09-0.15 m).

A low-flow measurement was made at the approach section in April 1969 as were two high-flow measurements during the flood in September 1971. Figure 46 shows the three measurements superimposed on each other. The scour occurring between the low and high flows was insignificant. Minimum streambed elevation varied less than 0.5 ft (0.15 m), but its position shifted laterally.

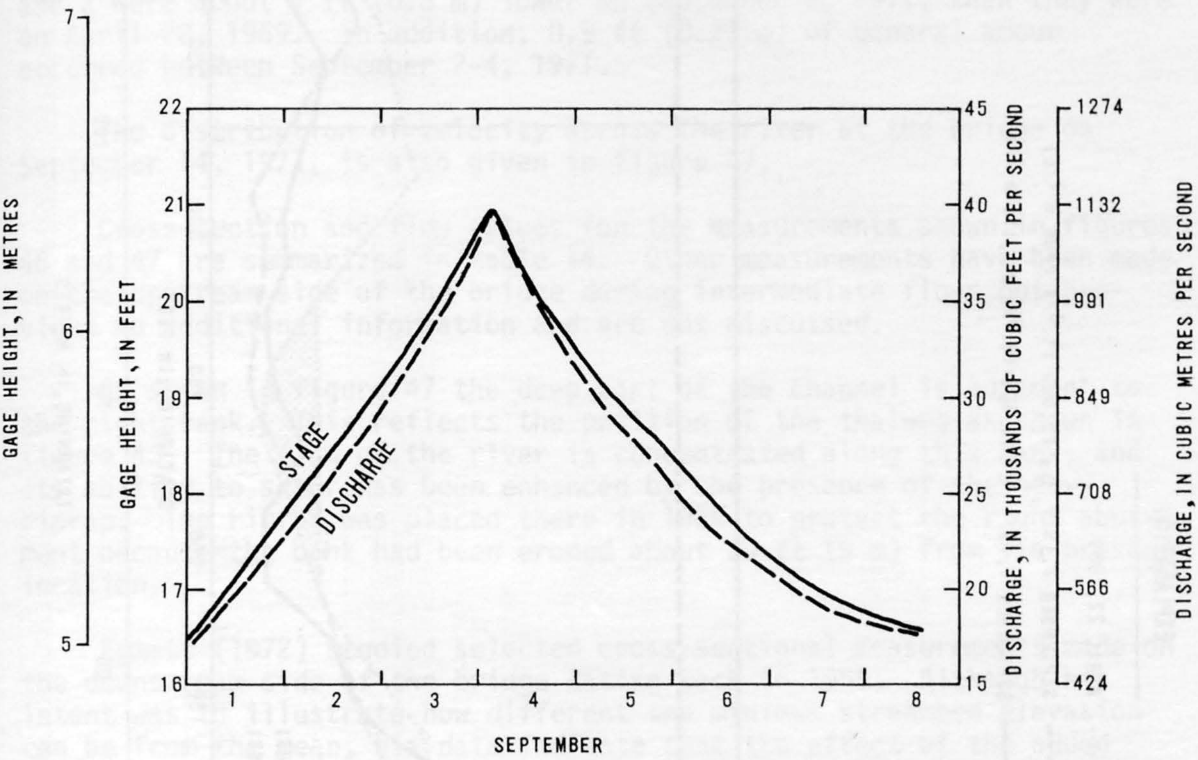


Figure 45--Hydrographs of Tazlina River near Glennallen, September 1971.

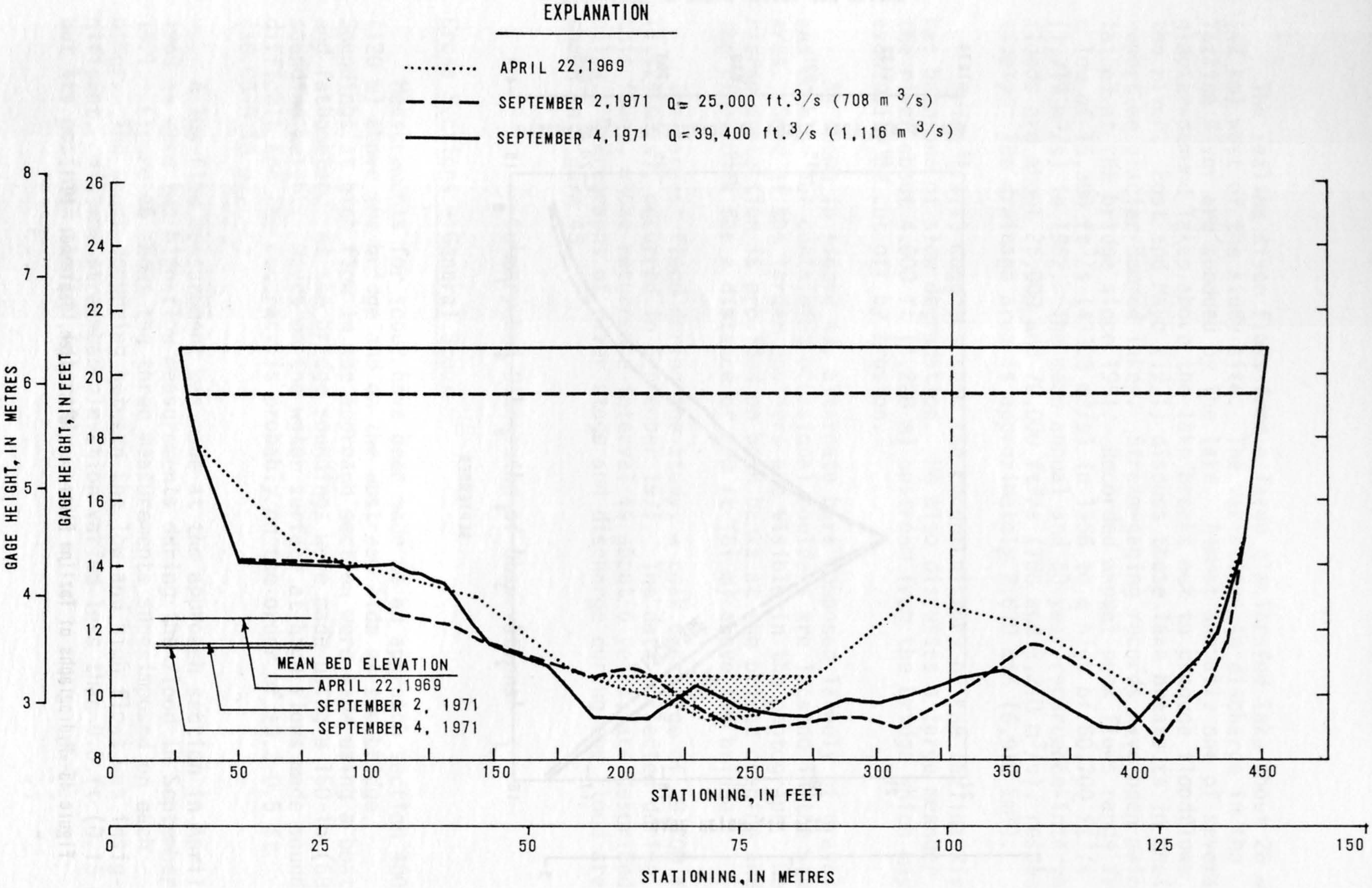


Figure 46--Approach cross section, 400 ft. (120 m) upstream from bridge 573, Tazlina River near Glennallen.

Figure 47 shows measurements made during low, medium, and high flows in 1971 and during low flow in April 1969 at the upstream side of the bridge. It illustrates the general scour which took place between low and high flows in the right half of the channel. Maximum general scour as shown by the difference in the mean streambed elevation of April 1969 and that of September 4, 1971, amounts to 1.8 ft (0.55 m). About 1 ft (0.3 m) of this amount may be considered to be general degradation because the mean streambed elevations at both cross sections 1 and 2 were about 1 ft (0.3 m) lower on September 2, 1971, than they were on April 22, 1969. In addition, 0.9 ft (0.27 m) of general scour occurred between September 2-4, 1971.

The distribution of velocity across the river at the bridge on September 14, 1971, is also given in figure 47.

Cross-section and flow values for the measurements shown in figures 46 and 47 are summarized in table 14. Other measurements have been made on the upstream side of the bridge during intermediate flows but provided no additional information and are not discussed.

As shown in figure 47 the deep part of the channel is adjacent to the right bank. This reflects the position of the thalweg as shown in figure 43. The flow of the river is concentrated along this bank, and its ability to scour has been enhanced by the presence of the heavy riprap. The riprap was placed there in 1964 to protect the right abutment because the bank had been eroded about 30 ft (9 m) from its present location.

Emmett (1972) studied selected cross-sectional measurements made on the downstream side of the bridge dating back to 1955. Although his intent was to illustrate how different the minimum streambed elevation can be from the mean, his data indicate that the effect of the added riprap may have been to decrease the bed elevation along the right bank by about 4 or 5 ft (1.2 or 1.5 m).

Water-surface Slopes and Streambed Profiles

During the peak on September 4, 1971, the water-surface slope was about 0.0021 ft/ft (m/m) which was a slight increase over the slope of 0.0018 ft/ft (m/m) measured on September 2, 1971.

The water surface was rough and at times exhibited standing waves which were visible above the approach section and several hundred feet below the bridge.

Streambed profiles obtained on September 2 indicated a wave form between the approach section and the bridge in the left half of the channel. The wave amplitude was 1 ft (0.3 m) and wave length was 40 ft (12 m). In the right one-quarter of the channel, wave forms with amplitudes of 0.5 ft (0.15 m) and wave lengths of 20 ft (6 m) were observed.

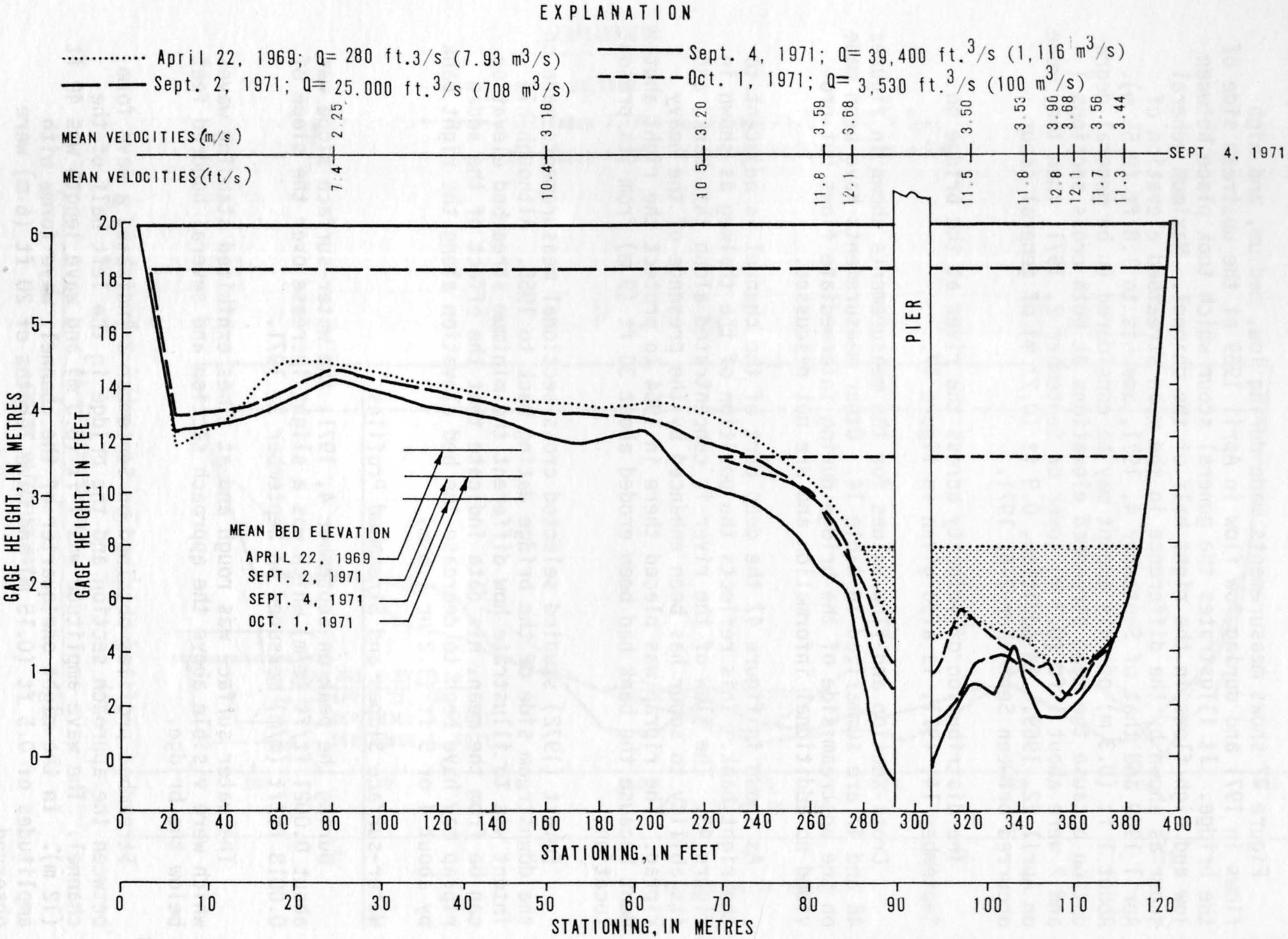


Figure 47.— Cross section 2 on upstream side of bridge 573, Tazlina River near Glennallen

On September 4, about 6 hours after the peak discharge, no wave form was observed on the streambed.

Sediment Analyses

Suspended-sediment samples taken at the bridge near the peak on September 4, 1971, showed a concentration of 2,800 mg/l which yielded a sediment discharge of 290,000 tons/day (263,000 tonnes/day). The median diameter (d_{50}) of the sediment was 0.014 mm and the d_{90} was 0.25 mm.

Photographs of the exposed streambed material at cross section 1 on April 22, 1969, were analyzed by the Zeiss method (Ritter and Helley, 1969). The d_{50} from the analyses was 90 mm (small cobbles) and d_{90} was 130 mm. Photographs of some of the larger material near the right bank in cross section 1 showed large cobbles of 200 to 250 mm in diameter and a few boulders.

Scour at Piers - Local Scour

The photographs in figure 48 illustrate the turbulence around the pier during high water. This turbulence was too severe to allow fathometer soundings near the pier to define the longitudinal shape of the scour hole. However, soundings, using a sounding weight, made from the bridge on both the upstream and downstream sides indicate that the minimum bed elevation probably was located near the nose of the pier.

The minimum observed depth of local scour at the pier was about 2 ft (0.61 m) during low flow on April 22, 1969. The maximum observed pier scour was about 5.5 ft (1.68 m) near the peak on September 4, 1971. These scour depths and those obtained on September 2 and October 1, 1971, are given in table 15. The depths of pier scour given were obtained from figure 47, using the average elevation of the first sharp breaks in streambed elevation away from the pier as the ambient streambed elevation.

An interesting observation of the performance of this pier is the way in which the bow or nose wave it creates at high flow sheds almost all of the debris which the current directed toward it. Trees with diameters of 2 ft (0.6 m) or more were diverted away from the pier before they could even come in contact with it. Only occasionally would a tree be so oriented with regard to the pier that it pierced the nose wave. Figure 49 shows the extent of the nose wave during the peak flow.

Comparison of Measured Scour with Scour Calculated from Formulas

General Scour

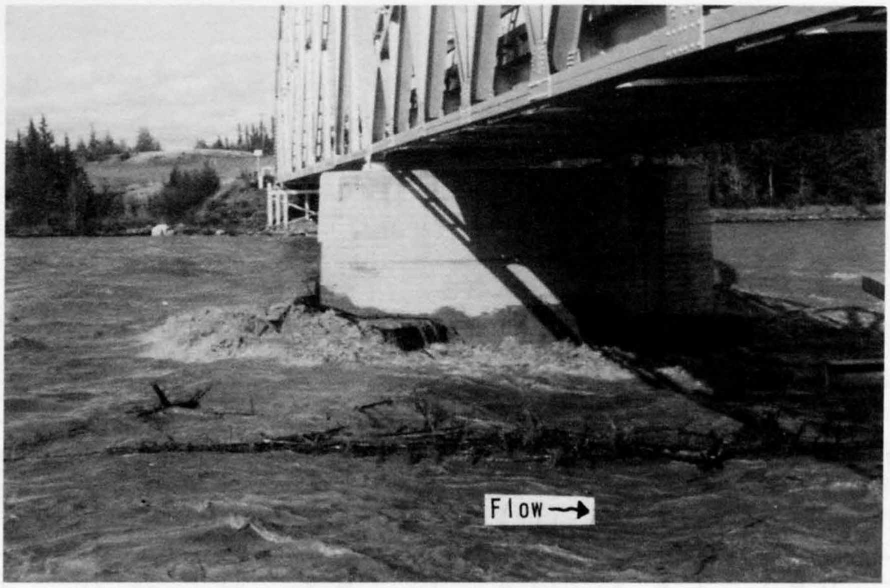
A 10 percent reduction in surface width occurs between the approach section and the bridge section at high flows which suggested general

Table 14.-- *Summary of cross section and flow values, Tazlina River near Glennallen*

Date	Cross section	Water-surface elevation (gage datum) (ft)	Discharge (ft^3/s)	Surface width (ft)	Wetted area (ft^2)	Mean velocity (ft/s)
April 22, 1969	1	10.6	280	85	64	4.4
Sept. 2, 1971	1	19.4	25,000	415	3,340	7.5
Sept. 4	1	20.8	39,400	420	3,850	10.2
April 22, 1969	2	7.8	280	87	271	1.0
Sept. 2, 1971	2	18.4	25,000	369	2,880	8.7
Sept. 4	2	20.0	39,400	379	3,900	10.1
Oct. 1	2	11.2	3,530	150	717	4.9

a Maximum depth at a point away from the scour hole at the pier.

Maximum mean velocity (ft/s)	Mean depth (ft)	Maximum depth (ft)	Mean bed elevation (ft)	Minimum bed elevation (ft)	Difference mean to minimum (ft)	
						percent of mean depth
--	0.8	1.3	12.4	9.3	--	--
--	8.0	10.4	11.4	9.0	2.4	30
--	9.2	11.7	11.6	9.1	2.5	27
--	2.0	a 4.2	11.5	3.6	7.9	--
10.8	7.8	a 16.4	10.6	2.0	8.6	81
12.8	10.3	a 18.6	9.7	1.4	8.3	85
8.0	4.3	a 8.9	10.5	2.3	--	--



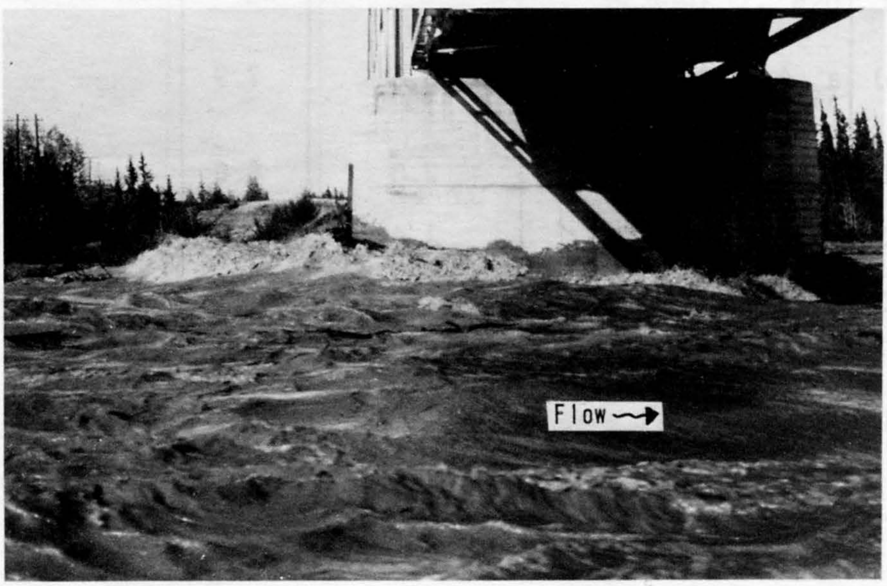


Figure 48.--Views of the surface effects of turbulence created by the pier during high flow, on September 4, 1971, Tazlina River near Glennallen.

Table 15.--*Summary of measured local equilibrium depth of scour, Tazlina River near Glennallen*

Date	Discharge Q (ft^3/s)	Equilibrium depth of scour d_{se} (ft)	Mean approach velocity v_a (ft/s)	Approach depth y_a (ft)	Water temperature (°C)
April 22, 1969	280	2	--	2	1
Sept. 2, 1971	25,000	5.0	9.5	12	8.5
Sept. 4	39,400	5.5	11.5	15	8.5
Oct. 1	3,530	3	2.0	5	6.0

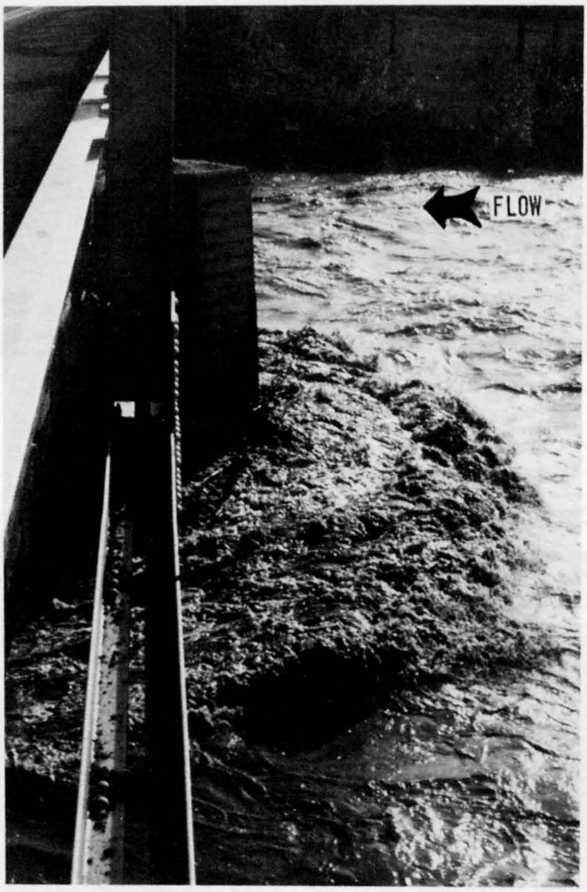


Figure 49.-- View from bridge showing the extent of the bow or nose wave created by the pier near peak flow of 39,700 ft^3/s (1124 m^3/s) on September 4, 1971, Tazlina River near Glennallen.

scour could occur. This was verified by general scour of 0.9 ft (0.27 m) at the upstream side of the bridge between September 2 and 4, 1971. However, virtually no scour occurred at the approach section.

Using the measured mean depth and width at the approach section, the measured width at the bridge during September 4, and the scour formulas, the predicted depths at the bridge are listed below.

	Mean depth		General scour	
	m	ft	m	ft
Measured	3.14	10.3	0.27	0.9
Equation 1	2.99	9.8	.18	.6
Equation 2	2.99	9.8	.18	.6
Equation 3	3.60	11.8	.79	2.6

Pier Scour

The maximum depth of local scour measured at the pier on September 4, 1971, is compared below with the computed depths obtained from equations 5, 6, and 7.

Measured		Equation 5 (x 0.90 nose factor)		Equation 6		Equation 7	
m	ft	m	ft	m	ft	m	ft
1.68	5.5	6.10	20.0	6.40	21.0	3.32	10.9

Tanana River at Big Delta - Bridge 524

Description

This study site is located at bridge 524 which spans the Tanana River at mile 281 on the Richardson Highway and is 0.5 mi (0.8 km) northwest of Big Delta. An aerial view of the site is shown in figure 50.

The bridge is 784 ft (239 m) long and consists of one overhead truss span 399 ft (122 m) long and four girder spans each about 95 ft (29 m) long, supported by four round-nosed concrete piers. The widths of piers 1, 2, and 3 taper from 2.5 ft (0.76 m) wide at the cap to 5 ft (1.52 m) wide at the footing. Pier 4 is longer and wider than the other piers, ranging from 3.0 ft (0.9 m) wide at the cap to 5 ft (1.52 m) at the footing. The piers are perpendicular to the roadway but are skewed to the flow. At high stages the angle of attack varies between 35 and 40°. Figure 51 is an oblique view of the bridge at low flow as seen from the left bank.

NORTH PACIFIC AERIAL SURVEYS

Figure 50.-- Aerial view of Tanana River at Big Delta bridge 524 on May 26, 1971.

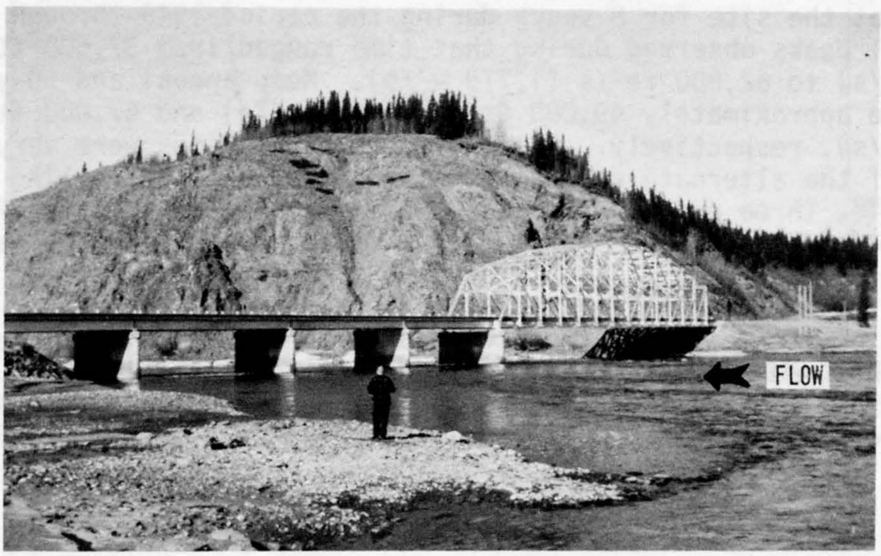


Figure 51.-- Bridge 524 at low flow on April 3, 1968, Tanana River at Big Delta.

Upstream from the bridge, the Tanana River channel is braided and contains gravel bars and islands. Immediately downstream from the bridge the Delta River empties into the Tanana River and forms a delta. This forces the Tanana River into a narrow channel against a bluff on the opposite bank which creates turbulence and has caused the bed of the stream to scour a large hole to a depth of about 40 ft (12 m). The streambed in the vicinity of the bridge is composed of sand, gravel, and some cobbles.

The drainage area above the bridge is about 13,500 mi^2 (35,000 km^2), a small part of which is covered by glaciers. Daily discharge was recorded at the site for 8 years during the period 1949 through 1957. The annual peaks observed during that time ranged from 37,600 ft^3/s (1,064 m^3/s) to 62,800 ft^3/s (1,778 m^3/s). Mean-annual and 50-year peak flows were approximately 49,000 ft^3/s (1,400 m^3/s) and 67,000 ft^3/s (1,900 m^3/s), respectively. Stage-discharge relations were very poor because of the alternate advance and retreat of the controlling delta. For example, three discharge measurements in 1971 ranged from 27,700 ft^3/s (784 m^3/s) to 52,600 ft^3/s (1,490 m^3/s) while the stage varied less than 1 ft (0.3 m).

Most of the measurements discussed in the following were made in 1971. Other measurements have been made but added little or no additional information and are not discussed. Maximum discharge observed during the study was 52,600 ft^3/s (1,490 m^3/s) on August 13, 1971.

Summary and Discussion of Observations

Cross Sections - General Scour

Measurements have been made on the four cross sections shown on the photograph in figure 50.

Cross section 1 is considered to be the approach section. Figure 52 presents the three measurements made at this cross section in 1971. As indicated, discharges ranged from a low of 27,700 ft^3/s (784 m^3/s) in June to a high of 52,600 ft^3/s (1,490 m^3/s) in August. The figure shows the streambed scoured primarily at its higher elevations, whereas the minimum streambed elevations remained nearly the same. The August measurement indicates that the mean streambed elevation at that time was about 2 ft (0.6 m) lower than in June and July. However, the streambed elevations in August are not considered as accurate as those in June and July because the distances from the edge of water had to be estimated. The measurements indicate, however, that some general lowering of the bed probably occurred between July and August.

Measurements made in May, July, and August at cross section 2 on the upstream side of the bridge are shown in figure 53. Major changes in the cross section occurred between pier 4 and the right bank probably

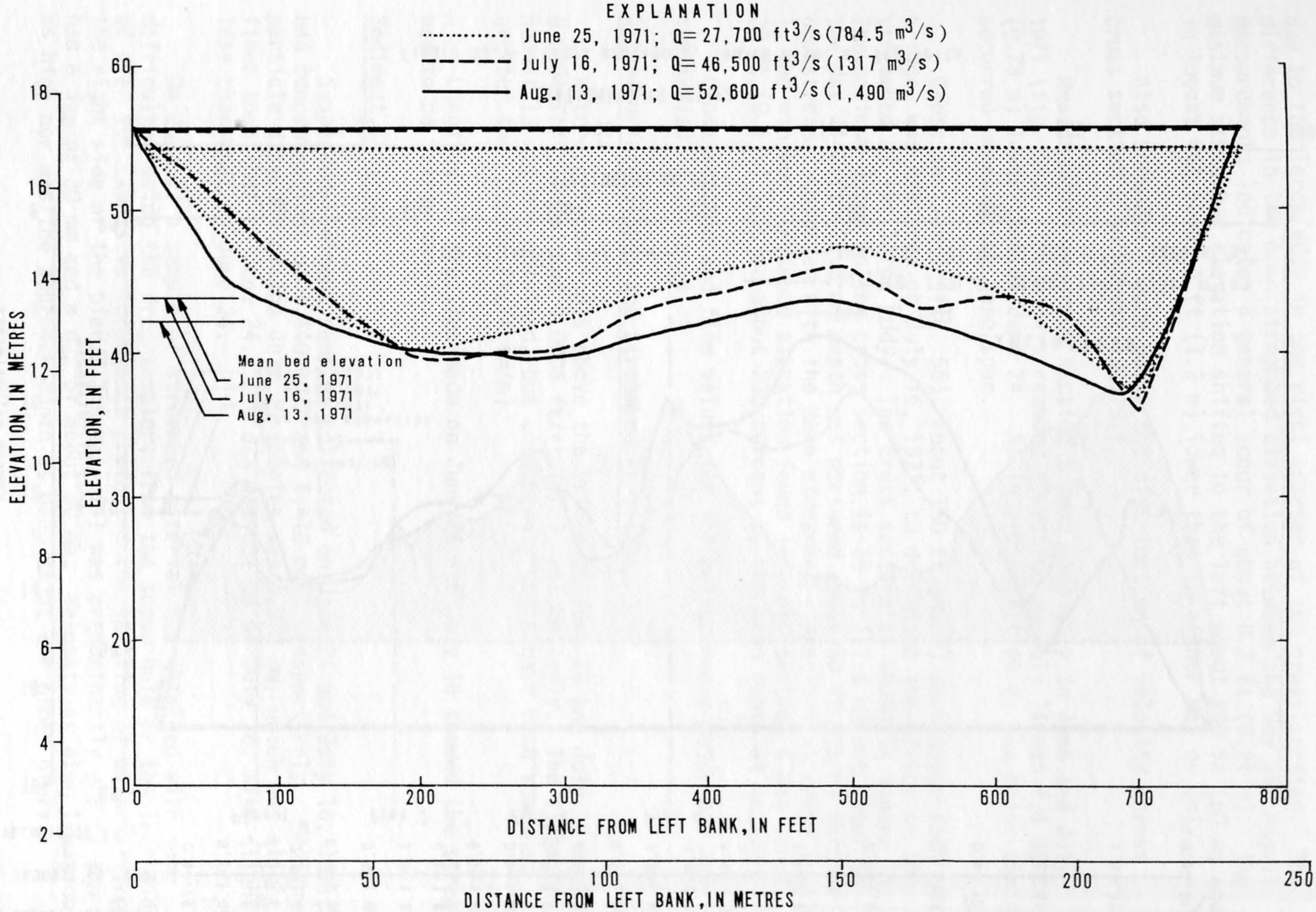


Figure 52.-- Cross section 1, Tanana River at Big Delta.

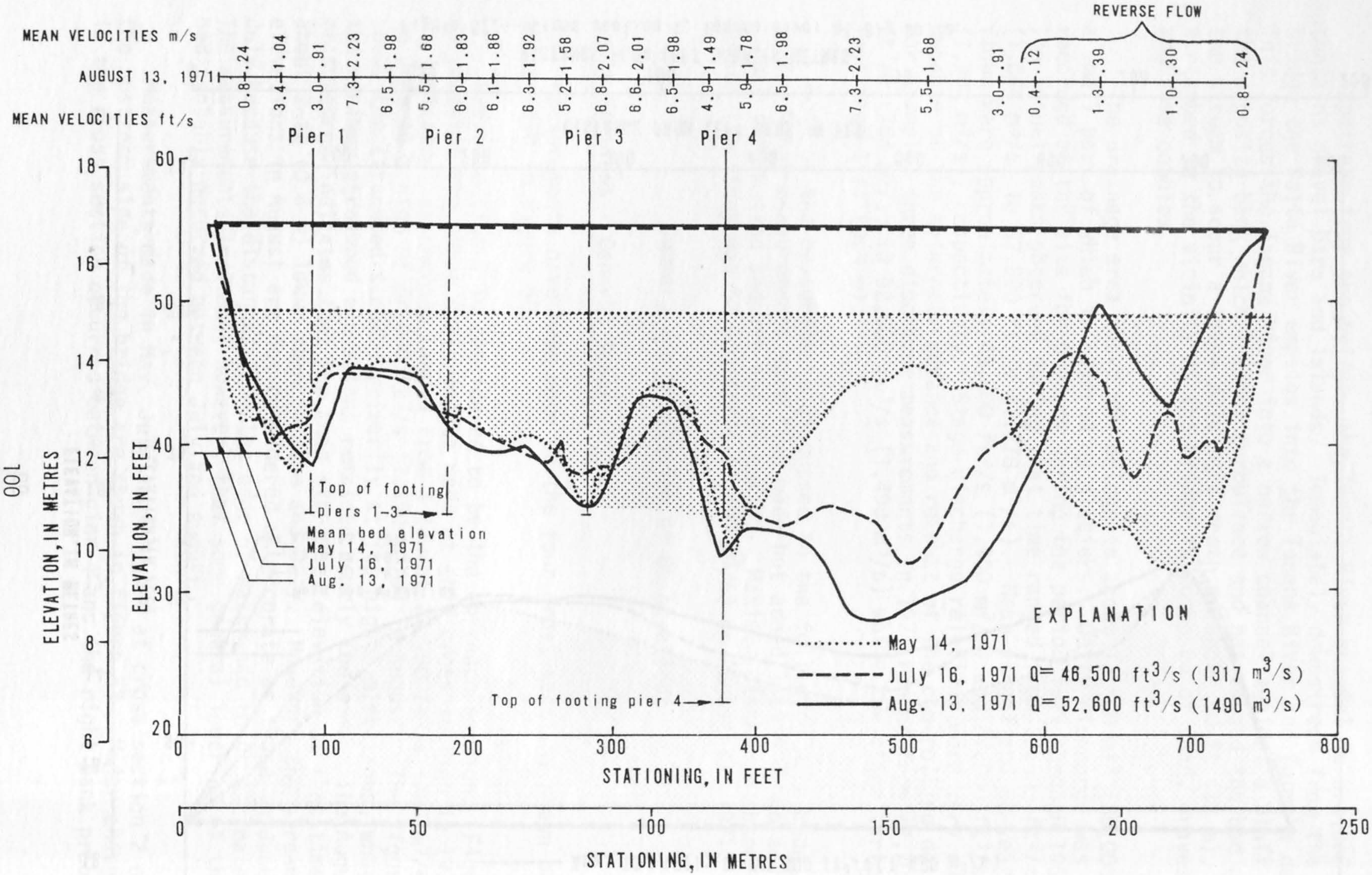


Figure 53.-- Cross section 2, Tanana River at Big Delta

due to the large eddy in the right quarter of the cross section. The difference in the mean streambed elevation between the May and August measurements indicates a general scour of about 0.7 ft (0.21 m). The minimum streambed elevation shifted to the left about 200 ft (60 m) and in August was about 4 ft (1.2 m) lower than in either May or July.

Also shown in figure 53 is the distribution of velocity across cross section 2 on August 13, 1971.

Measurements on cross section 3 were made only in June and July 1971 (fig. 54). The mean streambed elevation on July 16 was 2.4 ft (0.73 m) lower than on June 24. As in cross section 2, the major change occurred beneath the long span.

Cross section 4 (fig. 55), about 150 ft (46 m) downstream from the bridge, was measured on July 16, 1971, to determine the effect on the streambed of the large eddy. The cross section is shown in figure 55. The maximum depth in the cross section is 38 ft (11.6 m) but later in the day a 42-ft (12.8-m) depth was measured about 50 ft (15 m) farther downstream. Compared with the mean streambed elevations at cross section 1, the minimum streambed elevation found in cross section 4 was about 30 ft (9.1 m) less or about 230 percent of the mean depth at section 1.

Cross section and flow values for the measurements described are summarized in table 16.

Water-surface Slopes and Streambed Profiles

Water-surface slopes above the bridge on June 24 and July 16 were 0.0004 ft/ft (m/m) and 0.0006 ft/ft (m/m), respectively. The slope on August 13 was not obtained, but undoubtedly would have been somewhat greater than 0.0006 ft/ft (m/m).

Longitudinal profiles made on June 24 and July 16 showed the streambed to be fairly smooth.

Sediment Analyses

Suspended-sediment samples collected on June 24 and July 16, 1971, had concentrations of 1,440 mg/l and 4,410 mg/l, respectively. Median-particle diameter of the July 16 samples was 0.012 mm, which was slightly finer than that on June 24. The d_{90} was 0.1 mm on July 16, also slightly less than that on June 24.

On June 24 a sample of streambed material was obtained with some difficulty, using the drag sampler, from the scour hole on the left side of pier 1. Excessive turbulence prevented the sampling of material from the right side of the pier. The material was predominantly gravel and had a d_{50} of 30 mm and a d_{90} of about 50 mm. Velocity at pier 1 was not as high nor was the approach depth as deep as at the other piers.

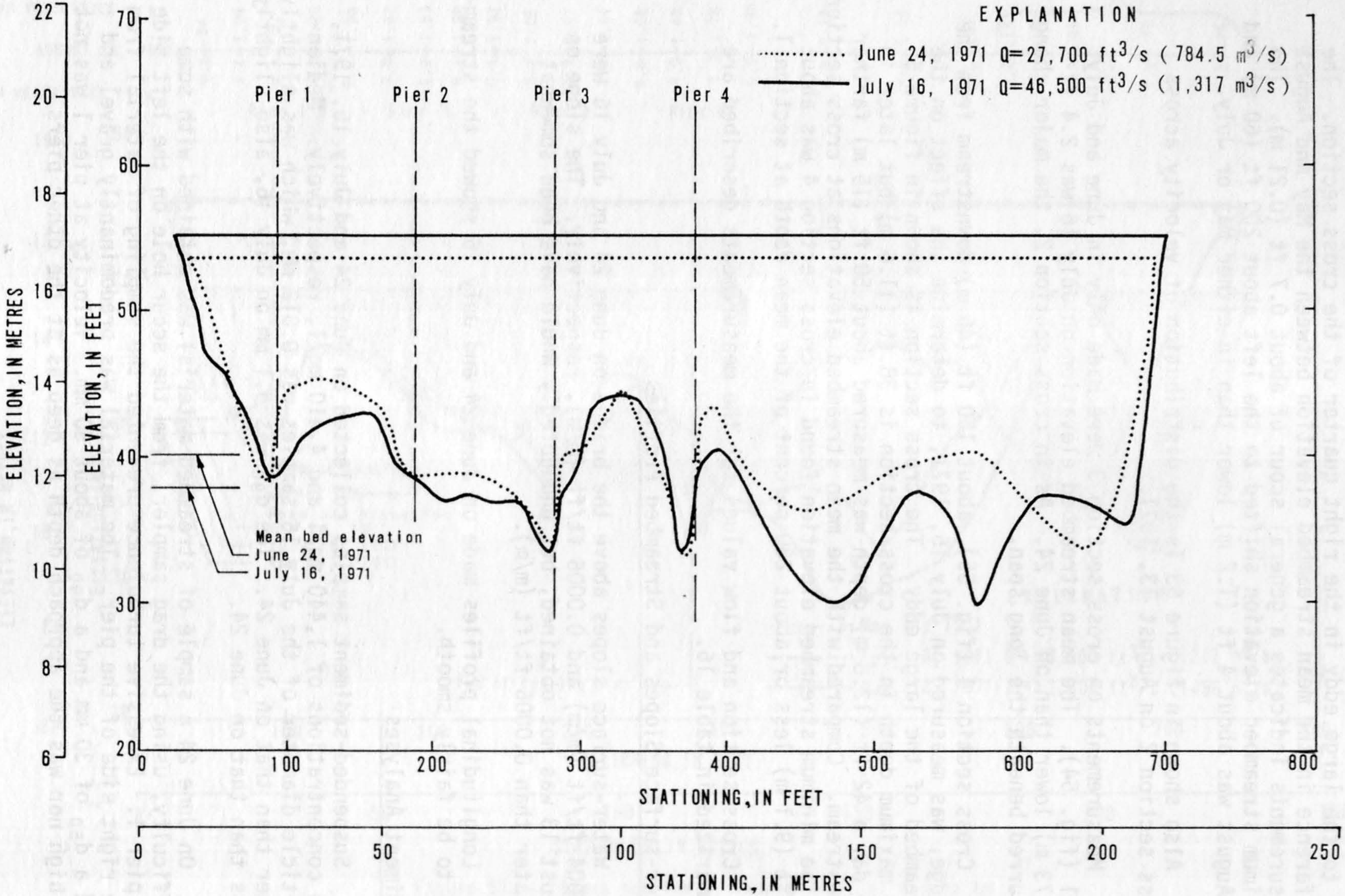


Figure 54.-- Cross section 3, Tanana River at Big Delta

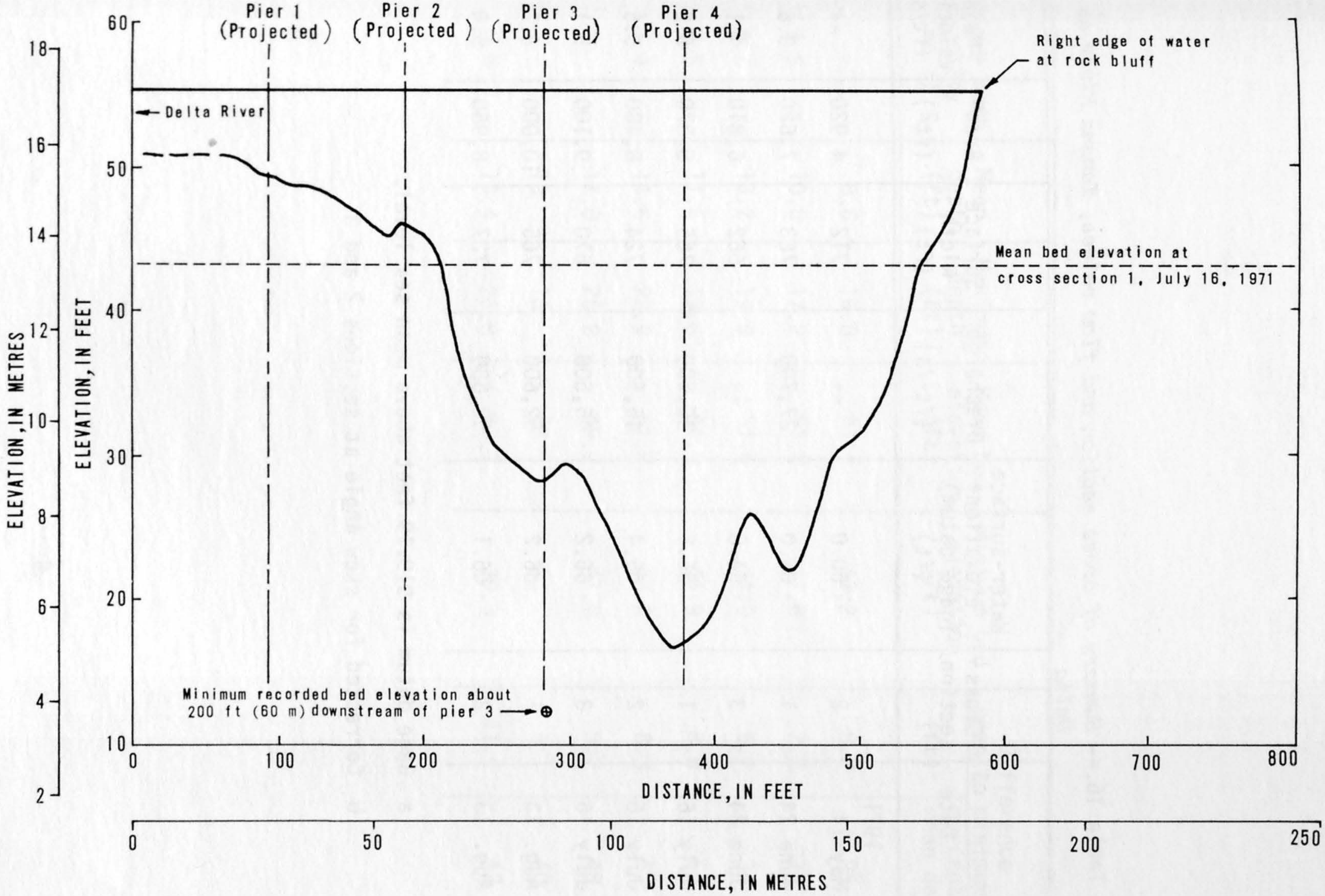


Figure 55.-- Cross section 4, 150 ft (45.7 m) downstream from bridge 524, July 16, 1971, Tanana River at Big Delta.

Table 16.-- *Summary of cross section and flow values, Tanana River at Big Delta*

Date	Cross section	Water-surface elevation[a] (gage datum) (ft)	Discharge (ft^3/s)	Surface width (ft)	Area[b] (ft^2)	Mean velocity (ft/s)
1971						
May 14	2	60.0	--	712	4,920	--
June 24	1	65.0	27,700	763	7,620	3.6
June 24	3	64.8	--	662	6,810	4.1
July 16	1	66.5	46,500	765	8,870	5.2
July 16	2	66.4	46,500	724	8,480	5.5
July 16	3	66.2	46,500	670	9,100	5.1
Aug. 13	1	66.2	52,600	765	10,000	5.2
Aug. 13	2	66.1	52,600	727	8,960	5.0

a Gage datum is 919.19 feet above mean sea level.

b Corrected for skew angle at sections 2 and 3.

Maximum mean velocity (ft/s)	Mean depth (ft)	Maximum depth (ft)	Mean bed elevation (ft)	Minimum bed elevation (ft)	Difference mean to minimum (ft)	Difference mean to minimum percent of mean depth
--	8.6	16.8	51.4	43.2	8.2	95
5.5	10.0	17.2	55.0	47.8	7.2	72
6.3	13.2	19.9	51.6	44.9	6.7	34
6.5	11.6	19.6	54.9	46.9	8.0	69
7.4	14.6	22.9	51.8	43.5	8.3	57
--	17.0	25.2	49.2	41.0	8.2	48
--	13	18	53	48	5	40
8.9	15.4	26.8	50.7	39.3	11.4	42

Consequently streambed material in the scour holes at the other piers probably was coarser than that at pier 1. Also, the turbulence on the left side of the piers was minor compared to the turbulence on the right side, and streambed material on the right side probably is coarser than that on the left side.

Streambed material samples were collected at cross section 1 on July 16. The median particle diameter was 14 mm, and d_{90} was 58 mm. The streambed material on August 13 probably was coarser than on July 16 because of the slightly higher velocities and depths on the later date.

Scour at Piers - Local Scour

The piers, being skewed to the flow, affected the flow pattern at the bridge as illustrated in figure 56. Water on the left side of the piers had a placid appearance, whereas the water surface on the right side was extremely turbulent. Turbulence was also created by debris which had collected on the piers (fig. 57). At an average angle of attack of 37° the projected widths of piers 1-3 and of pier 4 are about 22 ft (6.7 m) and 28 ft (8.5 m), respectively.

Soundings of the streambed adjacent to the piers were made on June 24 and July 16, 1971. The best profiles were obtained July 16 (fig. 58). The only characteristic of the streambed profiles common to all four piers is the minimum streambed elevation which apparently was located at the downstream end of the piers. The same approximate minimum streambed elevations also occurred beside the piers at several locations. The particular side of the pier on which the streambed was predominantly lower was not consistent from pier to pier.

Adjacent to pier 3 the bed scoured down to the top of the footing for a length of at least 10 ft (3 m) at the downstream end of the pier. The minimum streambed elevation occurred slightly downstream from the pier. The depth of scour to be expected in the absence of the footing is not known, but probably would not have been significant.

The maximum measured depth of local scour below the estimated approach streambed elevation at each pier is given in table 17. The maximum local depth of scour was that scour in the vicinity of the pier regardless of its location.

Comparison of Measured Scour with Scour Calculated from Formulas

General Scour

This bridge site is not of an ideal configuration for computing general scour because the contraction is downstream from the bridge and almost entirely on the left bank. Computation is further complicated by

A

B

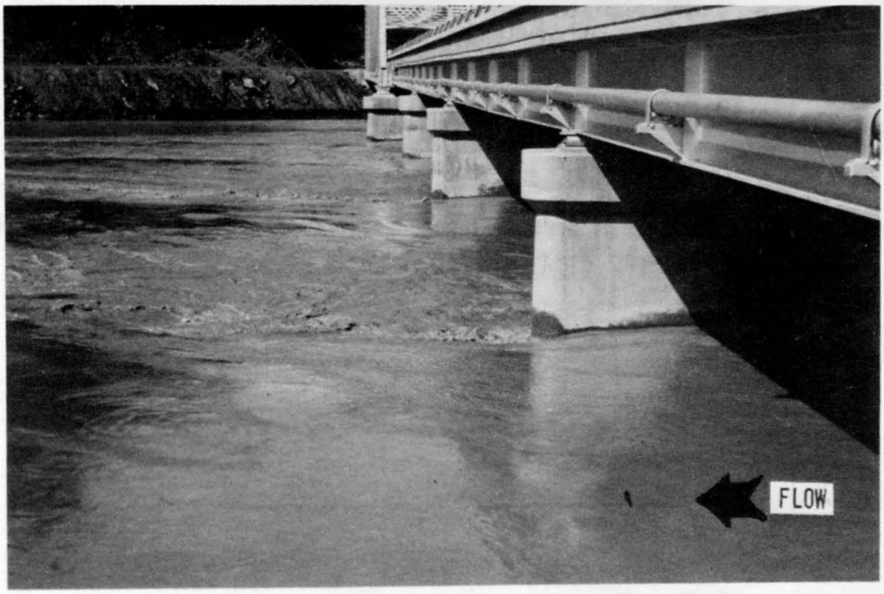


Figure 56.-- Water surface at Tanana River at Big Delta on July 16, 1971.
A. Upstream side of bridge looking toward left bank.
B. Downstream side of bridge looking toward right bank.

Figure 57.-- Debris at nose of pier 4, Tanana River at Big Delta on July 16, 1971. Q=46,500 ft^3/s (1,417 m^3/s)

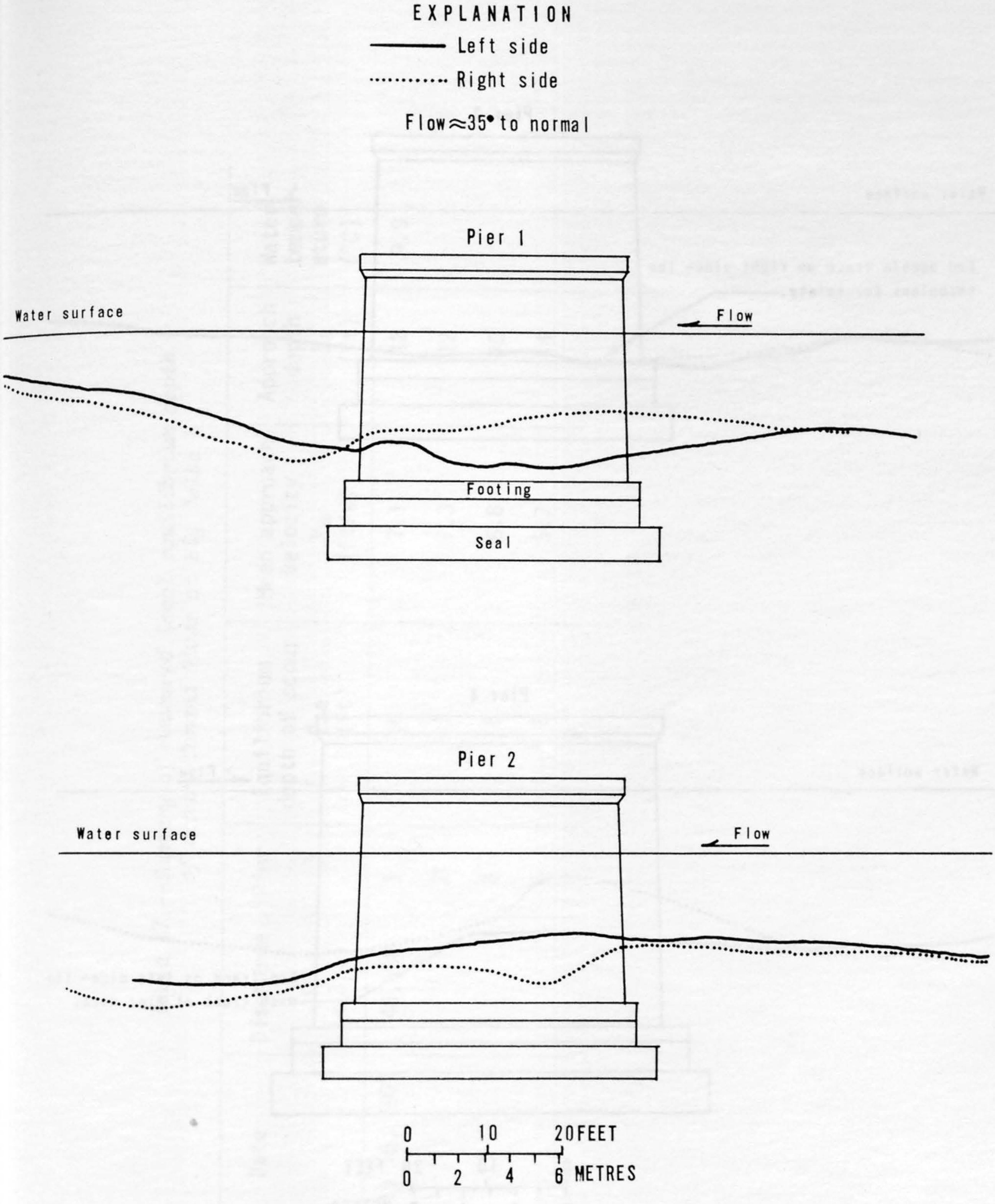


Figure 58.-- Streambed profiles parallel to and 2 — 5 ft (0.6-1.5 m) from the sides of piers 1-4, July 16, 1971, Tanana River at Big Delta

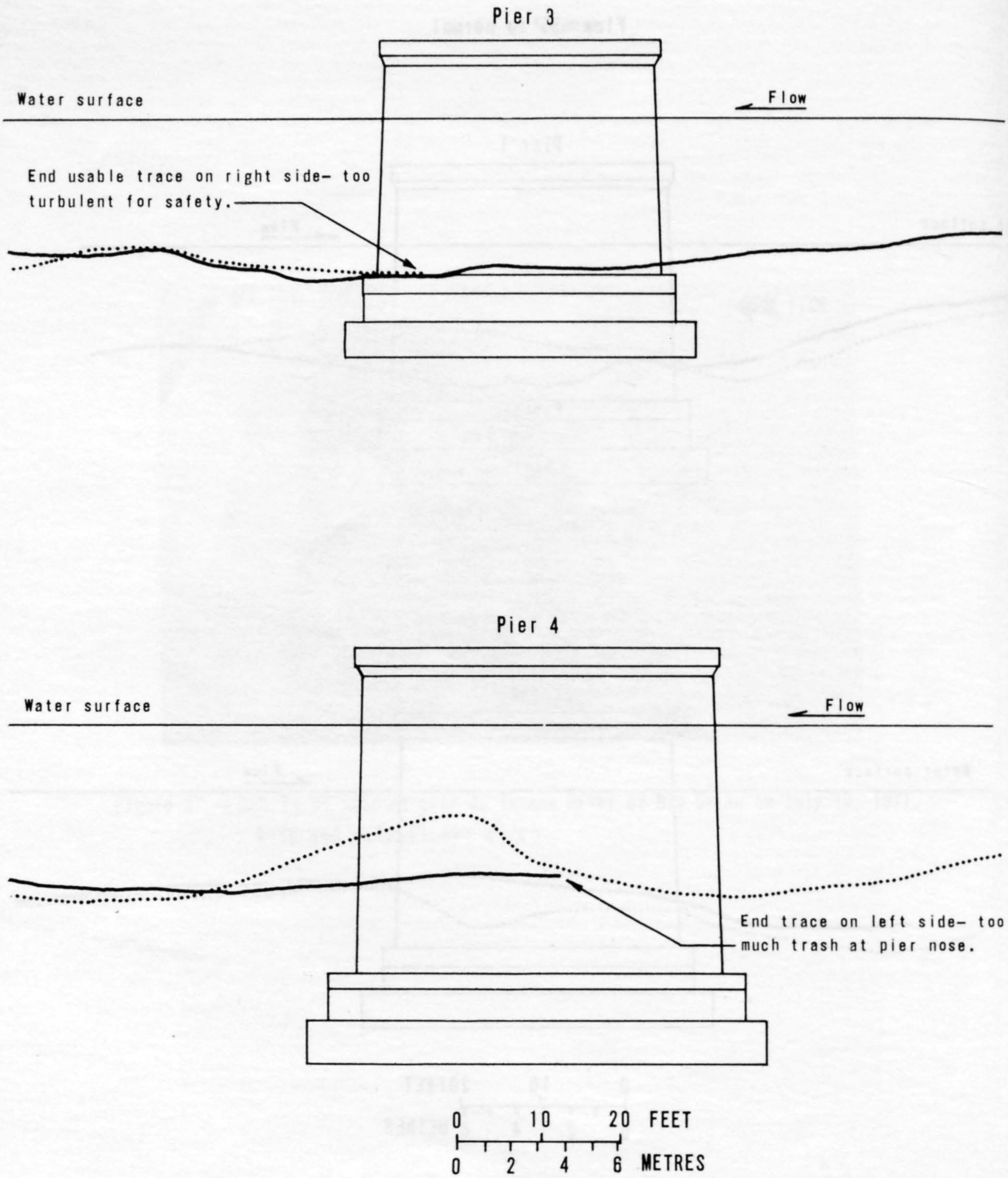


Figure 58.-- Continued

Table 17.--*Summary of measured local equilibrium depth of scour, Tanana River at Big Delta*

Date	Discharge Q (ft^3/s)	Pier	Equilibrium depth of scour d_{se} (ft)	Mean approach velocity v_a (ft/s)	Approach depth y_a (ft)	Water temper-ature (°C)
July 16, 1971	46,500	1	6	7.1	12	9.0
		2	7	7.3	12	
		3	6	6.8	15	
		4	8	5.7	14	

the presence of an eddy and by turbulence beneath the long span and by the angle with the flow made by cross sections 2 and 3. Because of contraction, however, it was desirous to compare the measured general scour with predictive formulas.

Assuming that the width and depth at cross section 1 was comparable to the approach conditions, computed values of mean depth at sections 2 and 3 for the measured high flows of July 16 and August 13, 1971, are given in table 18. The widths used at sections 2 and 3 were corrected for skew.

The maximum difference between the measured and computed values of scour was that at cross section 2 on August 13. The calculated mean depth exceeded the measured mean depth by slightly more than 8 percent. However, the calculated mean depth on July 16 at cross section 3 was 8 percent less than the measured mean depth. Although comparison between the measured and calculated mean depth is close, equations 1 and 2 should not be used without consideration of the hydraulic and channel features at a site.

Pier Scour

Although the exact effect of skewness on the depth of scour at piers is somewhat controversial, most researchers agree that skewness does increase pier scour. From Laursen and Toch's report (1956), a multiplication correction factor of about 3.5 to 4.0 would be applied for skew of the piers at this site. Similarly, from Neill's report (1970), a correction factor of about 3 would be applied.

The measured pier scour of July 16, 1971, is compared in table 19 with that obtained using the predictive equations 5, 6, and 7 and applying a compromising correction factor of 3.5. For the purpose of comparison only, the maximum scour of 7 ft (2.1 m) for piers 1, 2, and 3 and a pier width of 5.0 ft (1.52 m) was used. This indicates that much less scour occurred than would be predicted. If the correction factor is not applied, d_{se} from equation 5 would be slightly greater than measured, and d^{*}_{se} from equations 6 and 7 would be slightly less than the d^{*}_{se} "measured"

Tanana River at Nenana - Bridge 202

Description

The study site is located at bridge 202 which crosses the Tanana River at the town of Nenana on the Anchorage-Fairbanks Highway. An aerial view of the site is shown in figure 59.

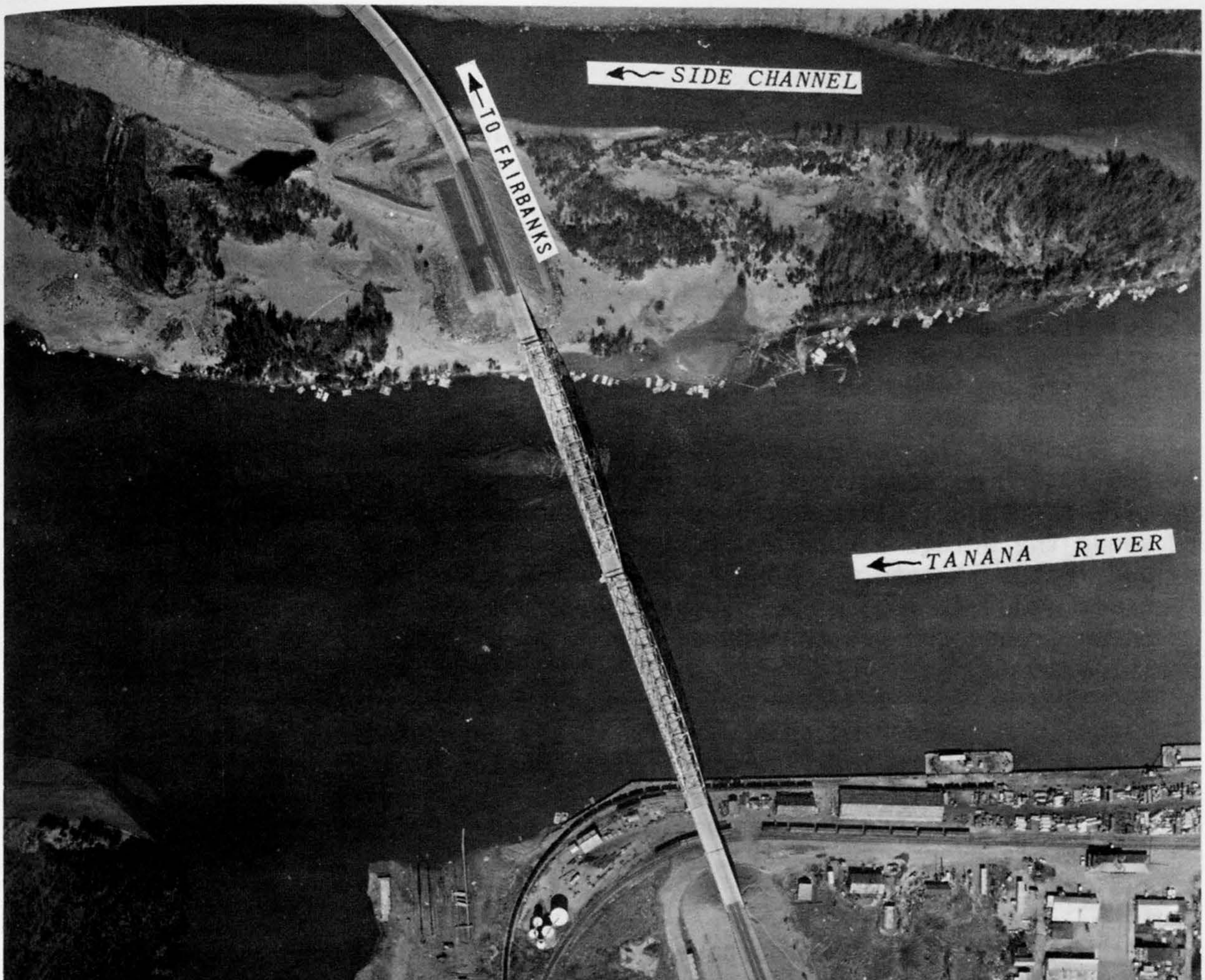
The principal bridge consists of two 500-ft (152-m) overhead truss spans supported in the center by a single pier. The pier constriction

Table 18.--*Comparison of measured mean depths to calculated depths, Tanana River at Big Delta*

Date	Cross section	Measured (ft)	Calculated	
			Equation 1 (ft)	Equation 2 (ft)
July 16, 1971	1	11.6	--	--
Aug. 13		13.0	--	--
July 16	2	14.6	15.0	15.0
Aug. 13		15.4	16.7	16.7
July 16	3	17.0	15.7	15.6

Table 19.--*Comparison of measured and calculated local depth of scour with continuous sediment motion, Tanana River at Big Delta, July 16, 1971*

Pier	Observed		Calculated (includes 3.5 skew factor)		
	d_{se}	$d^*_{se} = d_{se}/0.90$	Equation 5 (x 0.9 nose factor)	Equation 6	Equation 7
	(ft)	(ft)	(ft)	(ft)	(ft)
1-3	7	7.8	31	24	17
4	8	8.9	32	24	17



NORTH PACIFIC AERIAL SURVEY

Figure 59.-- Aerial view of the Tanana River at Nenana and bridge 202 on June 6, 1971.

varies from a 14-ft (4.3-m)-wide round-nosed stem at the foundation to two 6-ft (1.8-m)-diameter columns supporting the pier cap. These are separated by a pointed 10-ft (3.0-m)-wide stem with an extreme positive rake designed to resist ice forces. At high flows the pier is approximately parallel with the direction of the current at the water surface, but at low flows the pier is skewed at an angle of 10° to the flow. Not all of the flow of the Tanana River passes beneath this main bridge because a bridged side channel contains water at high flows.

The drainage area of the Tanana River above Nenana is approximately 25,600 mi^2 (66,300 km^2), a small part of which is covered by glaciers. As shown in figure 59, the Nenana River enters the Tanana River about 200 ft (60 m) downstream from the bridge. Above and below the study site the Tanana River exhibits a meandering pattern. About 1 mi (1.6 km) above the bridge a rock bluff resists a northward migration of the river and forces it into a channel extending in a west-southwestward direction. About 2,000 ft (600 m) below the bridge the river again turns to the north around the western end of low-lying hills.

The bridge is located at a crossover of the river channel. Crossovers on a meandering channel usually are characterized by scouring during low and medium flow and filling during high flows. At the bridge, the left or south bank has been stabilized by vertical bulkheads which form a loading dock for commercial rail and barge operations. Particle-size analyses of streambed material show that it is comprised of sand and gravel.

Stream-gaging records have been maintained at Nenana since 1962. From 1962 to 1971 annual peaks have ranged from 73,900 ft^3/s (2,093 m^3/s) in 1968 to 186,000 ft^3/s (5,267 m^3/s) in 1967. The mean-annual and 50-year floods are approximately 84,000 ft^3/s (2,380 m^3/s) and 168,000 ft^3/s (4,757 m^3/s), respectively. The recurrence interval of the 1967 peak is estimated to be about 100 years (R.D. Lamke, oral commun., February 1973). A description of the 1967 flood at Nenana is included in a flood report by Childers, Meckel, and Anderson (1972).

The measurements described herein were made in 1967. The maximum measured discharge was 174,000 ft^3/s (4,928 m^3/s). Stage and discharge hydrographs of July-September 1967 are presented in figure 60 to illustrate the variations of flow which occurred during the study period.

Summary and Discussion of Observations

Cross Sections - General Scour

Several cross-sectional measurements were made on the downstream side of the bridge during and after the high-water periods in 1967. Four of these measurements, which were made at discharges through the bridge opening, ranging from 19,300 ft^3/s (546.6 m^3/s) to 156,000 ft^3/s

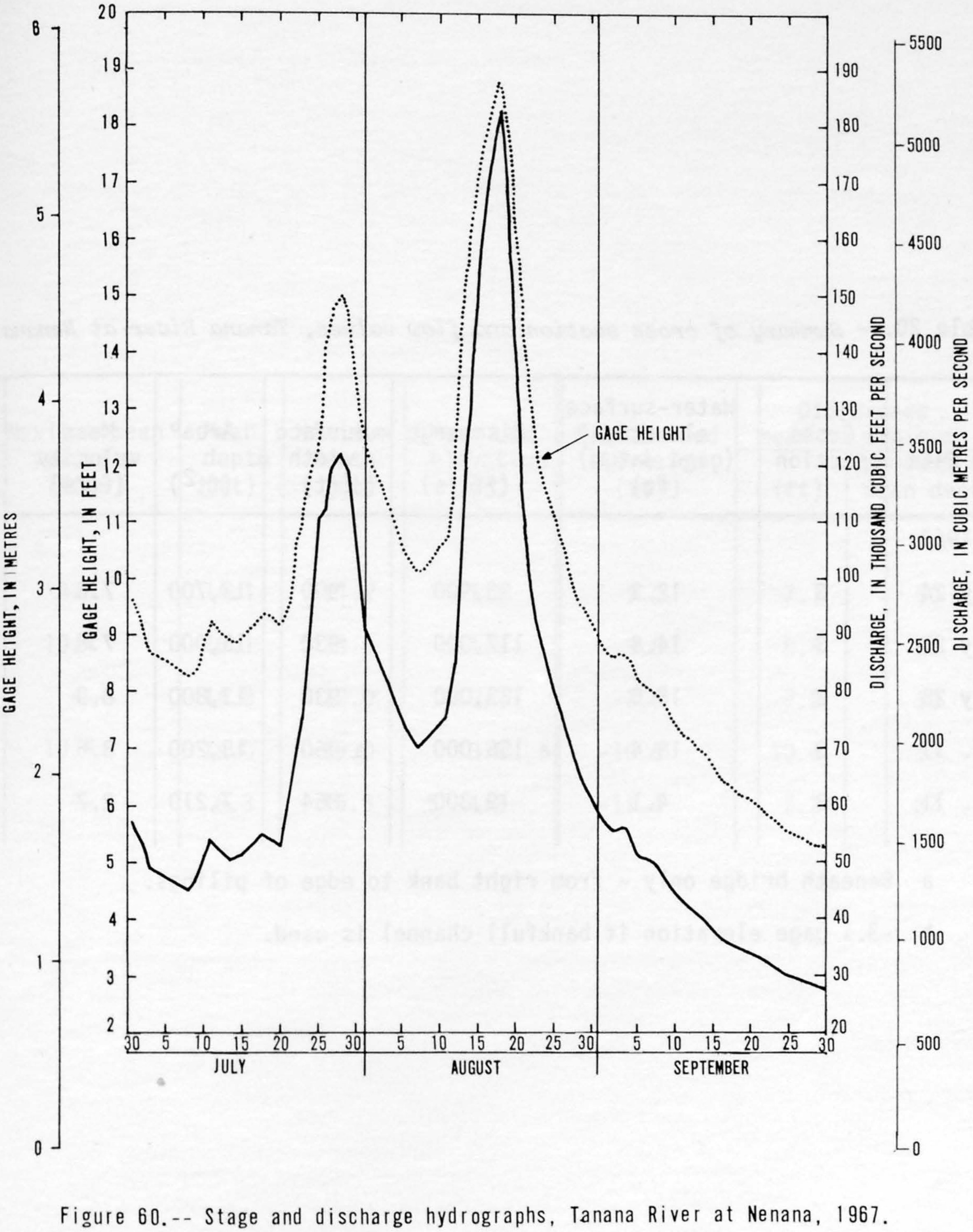


Figure 60.-- Stage and discharge hydrographs, Tanana River at Nenana, 1967.

Table 20.-- *Summary of cross section and flow values, Tanana River at Nenana*

Date	Cross section	Water-surface elevation[a] (gage datum) (ft)	Discharge (ft^3/s)	Surface width (ft)	Area[b] (ft^2)	Mean velocity (ft/s)
1967						
July 24	3	12.2	93,900	900	12,700	7.6
July 27	3	14.8	117,000	930	15,000	7.8
July 28	2	15.0	123,000	930	13,800	8.9
Aug. 17	3	18.4	a 156,000	a 960	18,200	8.6
Oct. 11	3	4.1	19,300	864	7,210	2.7

a Beneath bridge only - from right bank to edge of pilings.

b -3.1 gage elevation if bankfull channel is used.

Maximum mean velocity (ft/s)	Mean depth (ft)	Maximum depth (ft)	Mean bed elevation (ft)	Minimum bed elevation (ft)	Difference mean to minimum (ft)	
						percent of mean depth
9.6	14.1	21.2	-1.9	-9.0	7.1	50
10.4	16.1	24.6	-1.3	-9.8	8.5	53
--	14.8	22.0	0.2	-7.0	7.8	53
10.6	19.0	29.0	-0.6	-10.6	10.0	53
--	8.3	15.5	b -4.2	-11.4	7.2	87

(4,418 m^3/s), are compared and show that no significant general scour took place (fig. 61). This confirms the findings of Lane and Borland (1954) and Neill (1964) which indicate that at crossovers in meandering streams, the streambed tends to fill during high flows and scour during medium and low flows. The mean streambed elevation on August 17, 1967, was 3.6 ft (1.10 m) ft) higher than the wetted streambed and 2.5 ft (0.76 m) higher than the streambed across the entire bridge opening during the low flow of October 11.

On July 28, a cross-sectional measurement on the upstream side of the bridge was made using the boat and fathometer. The discharge at the time was 123,000 ft^3/s (3,483 m^3/s) which was 5,000 ft^3/s (142 m^3/s) more than that measured the previous day on the downstream side of the bridge. A comparison of these two cross-sectional profiles shows an insignificant change in streambed elevation in the right half of the cross sections. In the left half, the streambed was 3-4 ft (0.9-1.2 m) higher on July 28 than on July 27.

Cross section and flow values of the measurements described above are summarized in table 20. The distribution of velocity across the cross section on the downstream side of the bridge during the peak flow is shown in figure 61. A uniform change from left to right from about 10 ft/s (3.0 m/s) to 7 ft/s (2.1 m/s) was measured.

Water-surface Slopes and Streambed Profiles

Water-surface slopes at high stage were comparatively small. In 1963, prior to bridge construction, the slope was measured at 0.00008 ft/ft (m/m) (V.K. Berwick and J.P. Meckel, written commun., November 1963). Measurements of high-water marks after the peak flood of record on August 18, 1967, indicated that the slope was 0.00015 ft/ft (m/m) (R.D. Lamke, oral commun., February 1973).

A longitudinal profile along the center of the channel was obtained with the fathometer on July 29 at a discharge of 120,000 ft^3/s (3,398 m^3/s). At a point about 1,000 ft (300 m) upstream from the bridge dunes with amplitudes of about 2 ft (0.6 m) and wave lengths of about 40 ft (13.1 m) were present. Within 200 ft (60 m) downstream of this point, the amplitude and wave length of the dunes decreased until the streambed form was more like ripples at about 400 ft (120 m) upstream of the bridge. Two dune-like forms were superimposed on the streambed about 200 ft (60 m) upstream of the bridge.

Sediment Analyses

Samples collected on July 27, 1967, contained a concentration of 2,680 mg/l of suspended sediment. The water discharge at the time was 117,000 ft^3/s (3,313 m^3/s), and therefore the suspended-sediment discharge was 850,000 tons/d (771,000 tonnes/d). Size distribution of the samples showed d_{50} and d_{90} to be 0.04 mm and 0.15 mm, respectively.

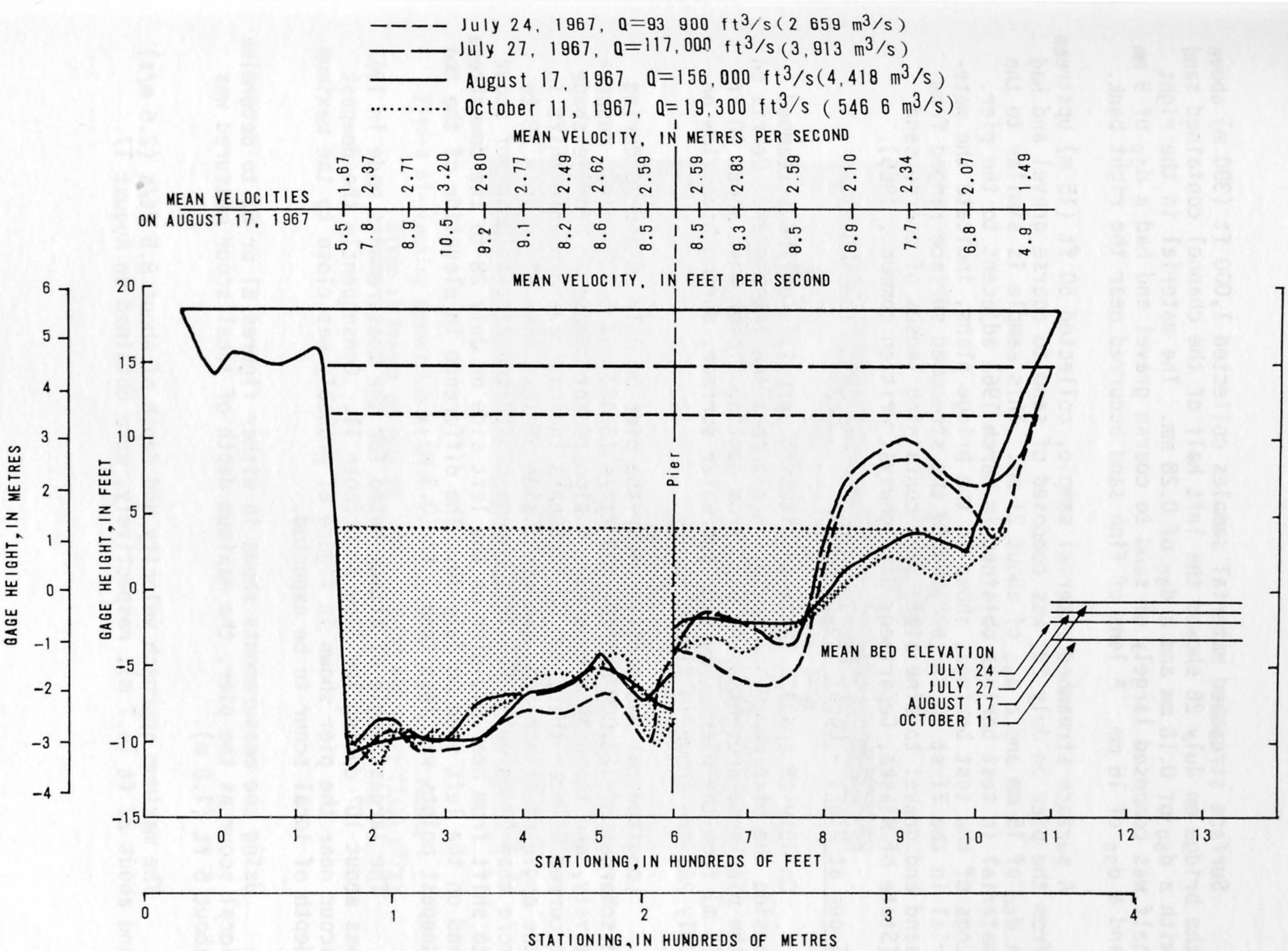


Figure 61.-- Cross section on the downstream side of the bridge, Tanana River at Nenana.

Surface streambed material samples collected 1,000 ft (300 m) above the bridge on July 28 showed the left half of the channel contained sand with a d_{50} of 0.18 mm and a d_{90} of 0.28 mm. The material in the right half was composed largely of sand to coarse gravel and had a d_{50} of 9 mm and a d_{90} of 16 mm. A lens of fine sand occurred near the right bank.

A surface streambed material sample, collected 50 ft (15 m) upstream from the pier on July 28, was composed of sand to coarse gravel and had a d_{50} of 15 mm and a d_{90} of about 21 mm. This sample is similar to the material in test borings obtained in March 1963 adjacent to the pier. Logs of the test borings, shown in the bridge plans, indicate the material in the first 20 ft (6 m) below the streambed surface ranged from sand and gravel to fine silty sand containing lenses of coarse sand (State of Alaska, Department of Highways, written commun., 1965).

Scour at Piers - Local Scour

On July 28 and 30, 1967, longitudinal profiles of the streambed beside the pier were obtained using the boat and fathometer. Debris on the pier nose prevented the boat from getting closer than about 10 ft (3 m) from the pier. The pier and water surface, during high flow on July 28, are shown in figure 62.

Longitudinal bed profiles near the pier on July 28 and 30 during discharges of 123,000 and 107,000 ft^3/s (3,483 and 3,030 m^3/s), respectively, are shown in figure 63. A significant change in profile shape occurred between these two measurements indicating a probable shift in the angle of attack from the right side of less than 5° on July 28 to more than 5° on July 30. This change caused the deepest point of scour to shift from near mid-pier on the left side on July 28 to the downstream end on the left side on July 30. The difference in elevation of the two deepest points was less than 1 ft (0.3 m).

The largest angle of attack noted for the measurements made in 1967 was about 10° during low flow on October 11. Consequently the deepest scour near the pier shown in figure 61 probably was close to the maximum depth of local scour to be expected.

Using the measurements shown in either figures 61 or 63 to determine local scour at the pier, the maximum depth of local scour measured was about 6 ft (1.8 m).

The maximum approach velocity and depth of about 8.5 ft/s (2.6 m/s) and about 22 ft (6.7 m), respectively, were obtained on August 17.

Figure 62--Looking downstream at pier with debris on pier nose, Tanana River at Nenana on July 28, 1967. Q=123,000 ft^3/s (3,483 m^3/s)

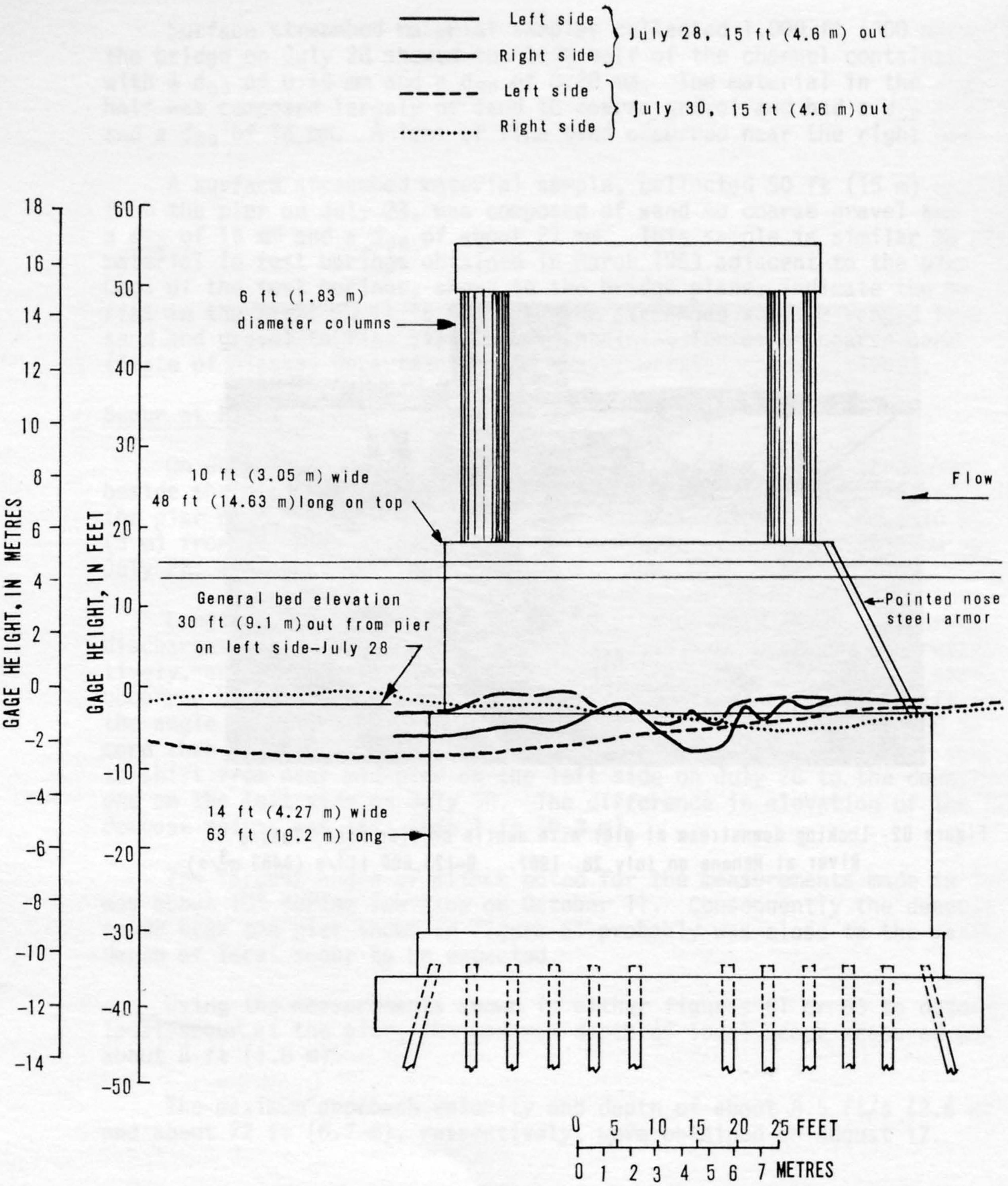


Figure 63. -- Bed profiles past pier July 28 and 30, 1967, Tanana River at Nenana

Comparison of Measured Scour with Scour Calculated from Formulas

General Scour

Because little or no contraction of river channel occurred at the bridge opening, general scour would not be expected. The measured difference in the mean streambed elevations beneath the bridge was a maximum of 2.5 ft (0.76 m); the streambed was at its highest elevation during the peak flood of record in August. The minimum mean streambed elevation was measured at low flow in October. This relationship between discharge and mean streambed elevation is that which would be expected at crossover locations in a meandering river (Lane and Borland, 1954; Neill, 1964).

Pier Scour

The measured local scour at the pier (d_{se}) for all flow conditions was about 6 ft (1.8 m). Using equation 5 and a 0.7 nose factor as given in table 6 gives a d_{se} of 13.3 ft (4.05 m). The d_{se}^* obtained using equations 6 and 7 and a 0.8 nose factor was 11.2 ft (3.4 m) and 6.4 ft (1.95 m), respectively. Thus, the predicted values are larger than those measured.

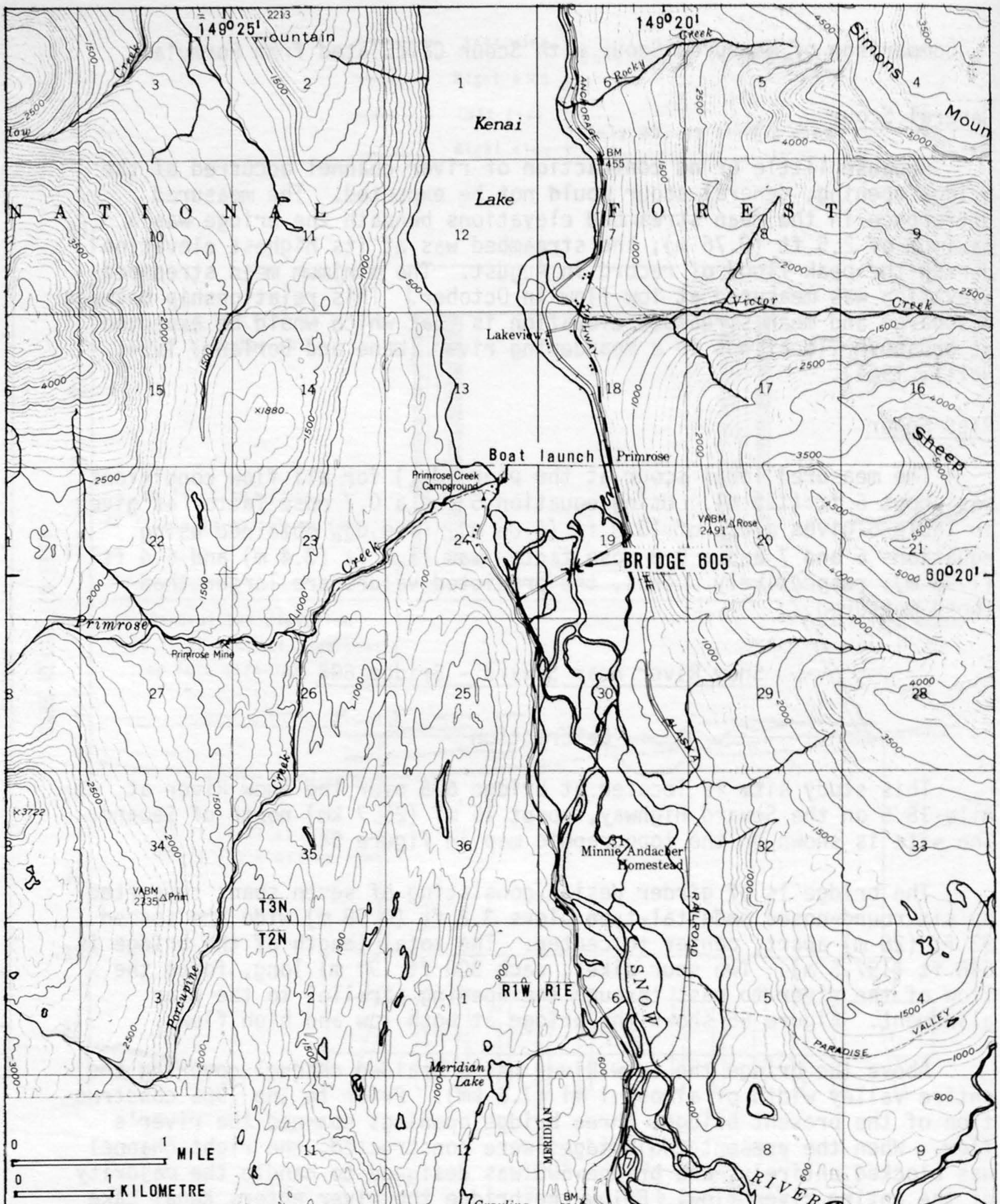
Snow River near Seward - Bridge 605

Description

This study site is located at bridge 605 over the Snow River at mile 18.5 on the Seward Highway, about 16 mi (25.7 km) north of Seward. The site is shown on the topographic map in figure 64.

The bridge is of girder design consisting of seven spans supported by six round-nosed pedestal-type piers 3.2 ft (0.98 m) wide and spaced 92 ft (28 m) apart, center to center. The total length of the bridge is 648 ft (197.5 m). Two spur dikes, each 300 ft (91 m) long, force the flow of the river to pass through the opening parallel to the pier alinement. Figure 65 shows the bridge at both low and high flows.

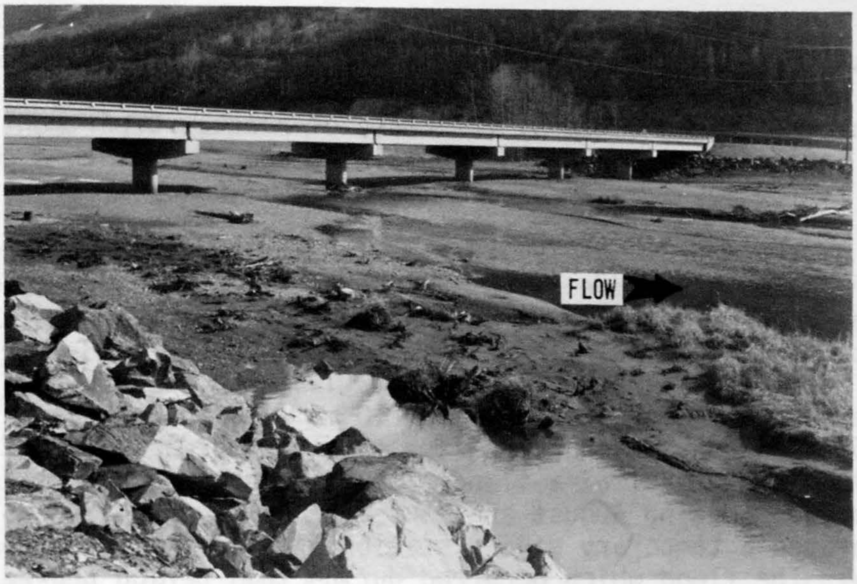
Above the bridge the Snow River has a braided channel covering the entire valley width of almost 1 mi (1.6 km). Prior to the 1966 construction of the present bridge, three bridge openings spanned the river's flow. When the present two bridges were constructed, the right channel was blocked entirely, and bridge 605 was designed to handle the majority of the entire river flow. Below the bridge the river enters Kenai Lake which may cause some backwater effect on the flow beneath the bridge when the lake level is high.



Base from U.S.Geological Survey Seward (B-7) Alaska 1:63 360, 1951

Figure 64 -- Topographic map of Snow River study site at bridge 605

A



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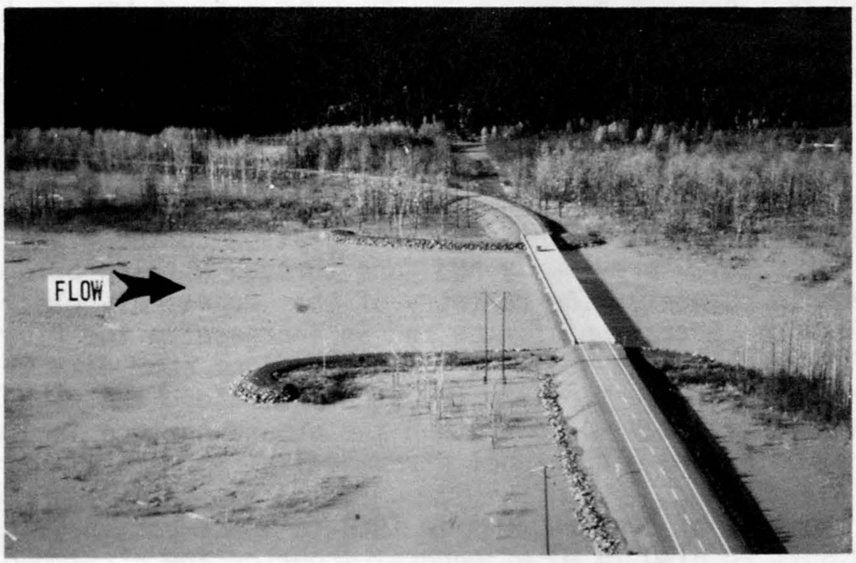

Figure 65.-- Bridge 605 over Snow River
A. View upstream from right bank during low flow on May 13, 1969.
B. Aerial view toward Seward, September 22, 1970. Q= 15,000 ft^3/s (425 m^3/s)

The surface streambed material in the vicinity of the bridge ranges from fine sand to coarse gravel. The foundation study along the centerline of the bridge conducted by the State of Alaska, Department of Highways, indicates silt and sand containing some gravel to a depth of about 100 ft (30 m).

Much of the approximately 150 mi^2 (388 km^2) drainage basin of the Snow River above the bridge is covered by glaciers. One, known locally as the Snow River Glacier, dams an unnamed lake from which water is released,causing flooding. The breakout occurs at 2- to 3-year intervals (Post and Mayo, 1971). During the period 1961-65, the U.S. Geological Survey operated a gaging station about 10 mi (16 km) upstream from bridge 605. The peak discharge during that period was 25,000 ft^3/s (708 m^3/s). In August 1967 a flood of an estimated 55,000 ft^3/s (1,560 m^3/s) occurred. During August and September 1970, the U.S. Geological Survey operated a temporary gaging station 3 mi (4.8 km) upstream from bridge 605. The flood which occurred during this period peaked on September 22 at a discharge of 17,800 ft^3/s (504.1 m^3/s). The majority of data presented herein was collected during this flood.

Summary and Discussion of Observations

Cross Sections - General Scour

After the site was selected for study, a cross-section measurement was made during low flow on May 13, 1969. This measurement is compared in figure 66 with two of the measurements made during the flood in September 1970 and with the ground-surface profile from the "as built" plans dated 1966. Figure 66 shows that the lowest point in the cross section has shifted from the old channel location to the right side where the water which previously had gone through a third bridge opening joined the main channel. The re-entry of this relatively clear water resulted in a concentration of flow and an increase in the river's capacity for transporting sediment along the right spur dike which caused the bed to scour. In 1970 the streambed along the right spur dike was 8-9 ft (2.4-2.7 m) lower than the ground surface in 1966 and 5 ft (1.5 m) below the old channel streambed.

From the time of construction in 1966 to September 1970, approximately 2-2.5 ft (0.61-0.76 m) of general scour of the streambed had occurred in the bridge opening. The three measurements made during this study suggest very little general scour (fig. 66). Additional measurements made during the flood in September 1970 indicated no significant differences.

Cross section and flow values for the three measurements are summarized in table 21.

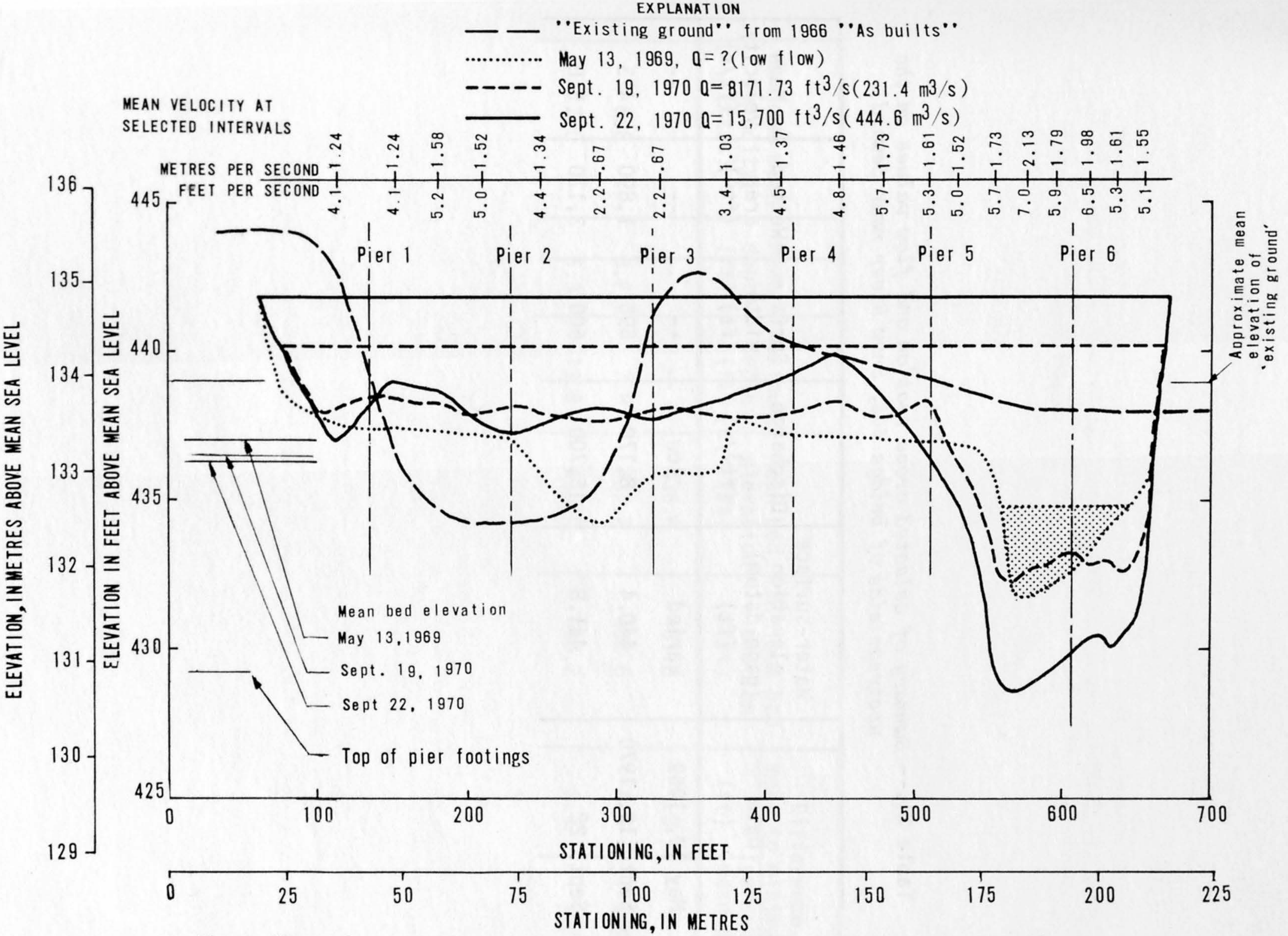

Figure 66. -- Cross section on upstream side of bridge 605, Snow River near Seward

Table 21.-- *Summary of selected cross section and flow values on the upstream side of bridge 605, Snow River near Seward*

Date	Water-surface elevation (gage datum) (ft)	Discharge (ft^3/s)	Surface width (ft)	Wetted area (ft^2)	Mean velocity (ft/s)
May 13, 1969	varied	low	--	--	--
Sept. 19, 1970	440.4	8,170	595	1,890	4.3
Sept. 22	441.8	15,700	600	3,170	5.0

Maximum mean velocity (ft/s)	Mean depth (ft)	Maximum depth (ft)	Mean bed elevation (ft)	Minimum bed elevation (ft)	Difference mean to minimum (ft)	Difference mean to minimum percent of mean depth
--	--	--	436.6	431.8	4.8	--
6.6	3.2	7.8	437.2	432.6	4.6	144
7.0	5.3	13.6	436.5	428.2	8.3	157

Streambed Profile and Sediment Analyses

A longitudinal streambed profile obtained on September 23, 1970, in the deep channel along the right spur dike indicated a maximum depth near the nose of the dike of about 23 ft (7.0 m). From this point the depth decreased uniformly in the downstream direction to about 8 ft (2.4 m) about 50 ft (15 m) upstream from the bridge. The stage at the time was 0.7 ft (0.21 m) lower than during the peak flow on the previous day.

For most of the channel width the depth was too shallow to obtain a useable fathometer record of the streambed. As shown in figure 65B, the streambed form in the shallow areas probably consisted of standing waves.

Streambed material particle size in samples collected along the cross section on the upstream side of the bridge varied widely. The analyses of surface samples collected upstream and between the piers showed that the d_{50} ranged between 0.1 mm and 9.8 mm. No particular pattern in the distribution was observed, and a computed composite of all the samples collected showed the average d_{50} to be about 3 mm. The samples of fine material probably represented the material being transported in the standing waves. The material which controlled the scour at the piers is that obtained in the samples containing the coarser materials. The average d_{50} of these coarse material samples was 7.6 mm and the average d_{90} was 23 mm.

Scour at Piers - Local Scour

The scour holes at this study site were typically shaped for piers where the flow is parallel to the pier. Figure 67 illustrates this as seen at pier 1 during low flow on May 13, 1969. In relation to the surrounding streambed elevation, the remnant scour hole was about 1.5 ft (0.46 m) deep. Some of the coarse streambed material described above can be seen in the shadow of the bridge.

Depths of remnant scour holes at the other piers during low flow in May 1969 are: pier 2, 1.3 ft (0.40 m); pier 3, 1.5 ft (0.46 m); pier 4, 0.7 ft (0.21 m); pier 5, 0.9 ft (0.27 m); and pier 6, 0.7 ft (0.21 m).

Fathometer records of the streambed alongside piers 5 and 6 were obtained on September 23, 1970, the day after the peak flow and discharge had dropped to 6,800 ft^3/s (192 m^3/s). No evidence of a scour hole on either side of pier 6 was observed, but at pier 5 the scour hole was 2.5-3 ft (0.76-0.91 m) deep. Although shallow depths prevented the accurate recording of scour at the other piers, the fathometer traces show it probably did not exceed that measured at pier 5.

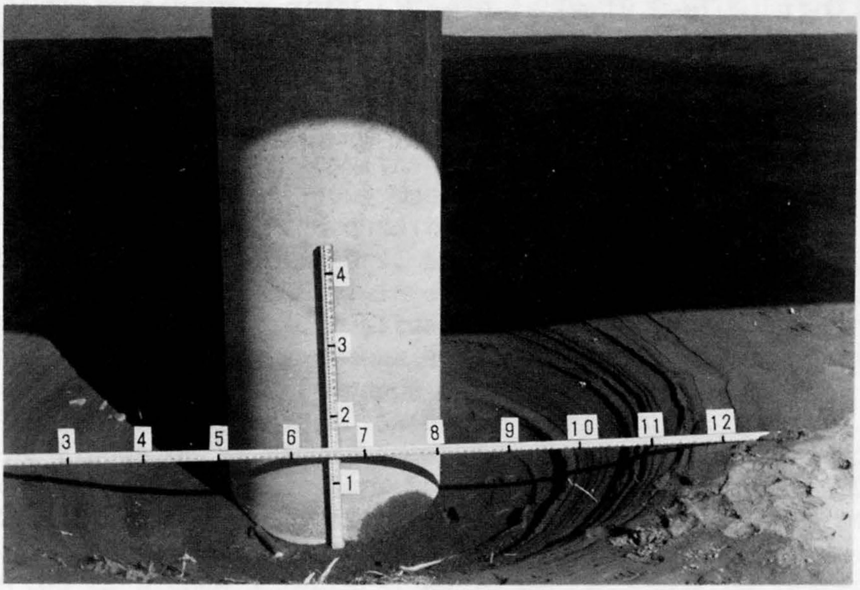

Figure 67.-- Scour hole at nose of pier 1 during low flow of May 13, 1969, Snow River near Seward. (Scales are level rods graduated in feet and tenths of feet)

Comparison of Measured Scour with Scour Calculated from Formulas

General Scour

No scour formula describes the conditions present at bridge 605. However, the cross-sectional data described above (p. 128) gives the reader an idea of what might take place in a similar situation.

Pier Scour

The maximum local equilibrium depth of scour (d_{se}) was measured as 2.5-3.0 ft (0.76-0.91 m) at pier 5. This compared closely with d_{se} = 5.7 ft (1.74 m) from equation 5 and 4.5 and 3.4 ft (1.37 and 1.04 m) from equations 6 and 7, respectively. Thus the three equations would have safely predicted the amount of pier scour; equation 7 predicting the closest value to the measured d_{se} or an adjusted $d^*_{se} = d_{se}/^{0.90}$.

SUMMARY OF COMPARISON OF FIELD MEASUREMENTS AT ALL SITES WITH SCOUR CALCULATED FROM SELECTED SCOUR FORMULAS

General Scour

At the three sites where contraction was present, the mean depths were calculated by the general scour formulas of Griffith (*in* Culbertson and others, 1967, p. 30), Straub (*in* Culbertson and others, 1967, p. 29), Laursen (1958), and Komura (1966). The results are summarized in table 22. Equations 1 and 2 predicted mean depths within 10 percent of the measured depths whereas depths predicted by equation 3 were as much as 36 percent less and 26 percent greater than the measured depths.

Although not computed by formula, the minimum streambed elevation in a cross section is always less than the mean streambed elevation for that cross section. Blench (1969) suggested the application of adjustment factors for various stream conditions to estimate the minimum streambed elevation. These factors, applied to the mean-flow depth, range from 1.50 to 2.75. He explained that the factors are based on practical conditions and include situations involving extreme attack but do not preclude engineering judgment and experience. For comparison with Blench's factors, the difference between the mean streambed elevation and minimum streambed elevation observed at bridge openings in this study (during floods only) ranged from 50 percent of the mean depth at the Susitna River site in a uniform but contracting reach, to 160 percent of the mean depth at the Snow River site where incoming flow from a side channel was concentrated along a rough riprap guide bank. These percentages, converted to Blench's factors, would be 1.5 and 2.6, respectively.

An important observation from all of the sites was that the minimum streambed elevations in the cross sections remained significantly constant even though their locations changed as the discharges changed from low flows to flood flows.

Pier Scour

Measured scour at piers was compared with equations 5, 6, and 7. At the sites occupied by bridges supported by multiple piers but where the cross-sectional depths varied, no consistent variation of pier scour with depth occurred. The computed values of pier scour using equation 5 (Laursen and Toch, 1956) were generally much higher than the measured pier scour.

Pier scour computed by equations 6 and 7 was closer to the measured values than those from equation 5. Equation 7 by Larras (*in* Shen and others, 1969, p. 1935; *in* Shen, 1971, p. 10) consistently yielded the value closest to the scour measured in this study.

Table 22.--*Comparison of measured mean depths to calculated depths at bridges where contraction was present during floodflows*

Location	Cross section	Measured	Mean depth		
			Griffith[1/] Straub[2/]	Laursen(1958)	Komura(1966)
			Equation 1	Equation 2	Equation 3
		(ft)	(ft)	(ft)	(ft)
Susitna River - bridge 254 (August 11, 1971)	1	14.8	--	--	--
	2	15.7	16.2	16.3	16.7
	4	18.0	20.0	20.5	22.6
	5	18.4	20.2	20.8	23.1
Tazlina River - bridge 573 (September 4, 1971)	1	9.2	--	--	--
	2	10.3	9.8	9.8	11.8
Tanana River - bridge 524[3/] (August 13, 1971)	1	13.0	--	--	--
	2	15.4	16.7	16.7	9.8

1/ In Culbertson and others, 1967, p. 30.
2/ In Culbertson and others, 1967, p. 29.
3/ The channel and hydraulics at this site are very complex; whether or not the close agreement between the measured depth and the depths predicted by equations 1 and 2 would be found in other complex situations is questionable.

Figure 68 was modified from Shen and others (1969) and compares the curves of equations 6 and 7 with field data points from this study and from other sources. The data from Shen and others (1966) and from Chabert and Engeldinger (*in* Shen and others, 1969, p. 1933) were laboratory data. Larras' data points (*in* Shen and others, 1969, p. 1935) were measurements of scour depth in the field after floods had passed and the streambed was not moving. Bata's field data (*in* Shen and others, 1969, p. 1933) was obtained where there was medium-sized riprap near the pier; the conditions under which it was placed are not known.

The numbered field-data points are explained in table 23. Data point 11, attributed to Laursen and Toch (1956), was shown on Shen and others' (1969) graph as having a pier width of 11 ft (3.4 m). Laursen (written commun., March 1973) suggested that the pier probably is 4 ft (1.2 m) wide and was plotted as such for this analysis. Except for points 2, 3, and 11, all field-data points are plotted using an adjusted d^*_{se} obtained by dividing the measured d_{se} by 0.90 as implied by figure 18. This seems reasonable because the measurements probably are only accurate to within ±0.5 ft (±0.15 m) for scour depths less than about 5 ft (1.5 m).

By taking into account the conditions at the time of measurement for each of the sites, the field data could be divided into three classes according to the particle size of streambed material upstream from the piers. The slope suggested by Larras' equation (*in* Shen and others, 1969, p. 1935; *in* Shen, 1971, p. 10) seemed to divide the data into these classes reasonably well. For practical purposes, however, a slope of 0.8 was chosen because the accuracy of the measurements and the variability of the flow and streambed conditions were not sufficiently defined to warrant the use of a more precise slope value.

The empirical equations suggested by this study for maximum depth of scour at cylindrical piers and piers with round or pointed noses alined with the flow are as follows:

(8) $d^*_{se} = 3b^{0.8}$ for bed material of medium silts to fine sands (d_{50} = 0.03-0.25 mm)

(9) $d^*_{se} = 2b^{0.8}$ for bed material of medium sands to fine gravels (d_{50} = 0-25-8 mm)

(10) $d^*_{se} = 1.2b^{0.8}$ for bed material of medium gravel to coarser (d_{50} > 8 mm)

These equations are graphically presented in figure 68. Each equation represents an upper limit for the maximum amount of scour to be expected at a pier situated in material of a particular particle-size classification. Where pier nose shapes are other than round, a coefficient, as suggested by table 6, may be applied to the calculated scour depth. If debris can be expected to lodge on the pier near the streambed, additional scour should be anticipated.

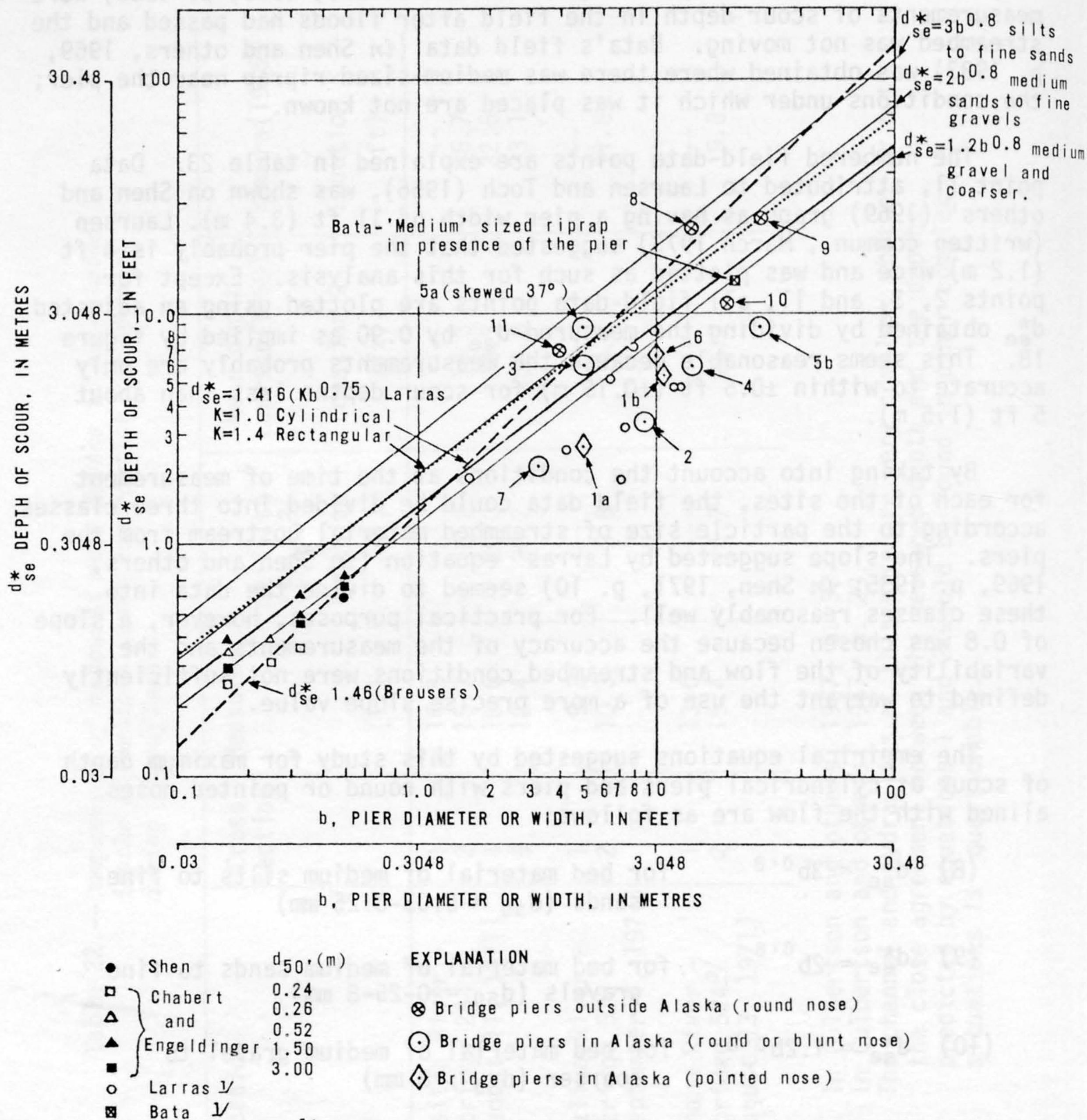

Figure 68.-- Maximum equilibrium scour depth as a function of pier width with streambed material as a third variable. Except where noted, all piers are assumed to be alined with the flow. (Modified from Shen and others, 1969)

Table 23.-- *Explanation of numbered field points for figure 68*

No.	Location	Maximum d_{se} (ft)	Maximum d^*_{se} (ft)	Estimated d^*_{se} (ft)	Pier width (ft)	Nose shape	Bed material d_{50} mm	Remarks
1a	Susitna R. nr Sunshine, bridge 254	2.5	--	2.8	5.0	pointed	70	--
1b	Susitna R. nr Sunshine, bridge 254	5	--	5.5	11.0	(trash)	70	Submerged debris on pier.
2	Knik R. nr Palmer, bridge 539	2.5	3.5	--	8.0	pointed blunt	8	Instrumented pier - 5-ft dunes present, rip-rap near pier.
3	Knik R. nr Eklutna, bridge 1121	4	6	--	5.0	round	1.5	Instrumented pier - 4-ft dunes present.
4	Tazlina R. nr Glennallen, bridge 573	5.5	--	6.0	15.0	round	90	--
5a	Tanana R. at Big Delta, bridge 524	8	--	8.9	5.0	round	14	Skewed 37°
5b	Tanana R. at Big Delta, bridge 524	8	--	8.9	28.0	skewed	14	Same as 5 but with a projected width of 28 ft.
6	Tanana R. at Nenana, bridge 202	6	--	6.6	10.0	pointed	15	Extreme positive rake, debris on nose.
7	Snow R. nr Seward, bridge 605	2	--	2.2	3.2	round	8	--
8	Oosterschelde Bridge, Netherlands (Breusers, 1970)	--	23	--	13.9	round	0.2	Tidal reach
9	Niger Bridge nr Onitsha, Netherlands (Breusers, 1970)	--	26	--	27.9	round	0.67	--
10	Suspension bridge, Alberta, Canada (Neill, 1970)	11	--	12	20.0	round	gravel	--
11	Skunk R. nr Ames, Iowa, (Laursen and Toch, 1956; Laursen, written commun., March 1973)	--	7	--	4.0	round	sand	General scour apparently not accounted for.

These equations should not be considered a replacement for engineering judgment because they do not account for all of the situations at a bridge site. The judgment needed to "temper" these equations must be based on detailed studies of the bridge sites and scour as described in this report.

Additional field data should be collected to verify, refine, or modify the equations presented.

EVALUATION OF THE STUDY

As stated in the introduction, the objectives of the study were to gather scour-related data under field conditions which would aid in evaluating and perhaps modifying predictive scour formulas derived principally from model studies. These objectives were met because at seven of the nine bridge sites scour data were collected of sufficient quality to compare, verify, and modify some predictive scour formulas.

At many of the sites during the study period, only one significant flood of mean annual or greater magnitude occurred or was measured. It would have been preferable to have had a range in flood conditions at each site. However, establishment of this network of sites did allow measurements of floods ranging in magnitude from mean annual to 20- to 30-year floods, and some data on a possible 100-year flood were collected. Although two of the 11 sites were not measured during flooding conditions and considerable effort was used in surveying these sites at low flow, the concept of conducting this type of research by means of a network is valid.

The 20-ft (6.1-m) double-hulled fiberglass boat worked reasonably well for the larger rivers, but smaller boats were necessary on rivers such as the Chena River. For the majority of the sounding work, the hull-mounted transducer on the fiberglass boat had a definite advantage over one held alongside the boat. A disadvantage, however, was that the boat was difficult to maneuver close enough to the piers to obtain the maximum depth in the local scour holes because the surface effects of turbulence around the piers often caused the boat operator to refrain from getting too close, and the depth was measured about 5-10 ft (1.5-3 m) away from the pier. In some instances two fathometers were used simultaneously, one using the transducer in the hull, and the other using a portable transducer held alongside the boat on the side nearest the pier. However, working close to a pier was definitely dangerous; in fact, the fiberglass boat being used was swamped during an attempt to obtain fathometer records of the streambed near a pier on the Susitna River. Fortunately the floatation chamber between the two hulls prevented the boat from sinking completely. Debris also prevented the boat from getting sufficiently close to piers. For future work, it is suggested that the portable transducer be mounted on a lightweight rod by which an operator in the boat (at a safe distance from the pier) can position the transducer near the pier or around debris.

The mobility and maneuverability of the boat were desirable assets for determining bed forms, measuring cross sections, and collecting data adjacent to the piers. However, because it is dangerous near the piers at some bridges, it is preferable to use sounding weights for determining streambed elevations along the upstream and downstream sides of the bridge. Where measuring velocities and depths from the upstream side of a bridge was found to be difficult, soundings alone could be made easily without the current meter in place above the sounding weight. Velocities and depths could then be measured with reasonable ease from the downstream side of the bridge.

The U.S. BM-54 was designed to be used for sampling streambeds composed of sand to small gravel-size material; this was verified by the study. Some success in sampling streambeds of gravel and cobbles during floods was obtained using the drag sampler from a boat. In two instances a close comparison of the particle-size distribution of material in the same stream was obtained using both the drag sampler and the analysis from photographs of material taken in the same location. Although not definitive, the comparison of particle-size distribution of surface streambed material by these two methods indicates that analysis of coarse streambed material by photographic methods at low flow may be sufficient to describe the size characteristics. Although some fines may be lost, the drag sampler, as shown in figure 3, should have holes in the bottom and sides to lessen its resistance to the flow along the streambed.

The recording fathometer was considered a practical means to measure and record depths even though mechanical problems arose. The new models, however, have been improved.

The use of mobile equipment to service the large network of sites was found to be practicable and efficient. However, it is doubtful that definition of the oscillations of the streambed at the bottom of the scour hole would have been obtained without the transducers permanently mounted on the two Knik River bridges. The permanent installation should be seriously considered for streams which develop large streambed forms. The two major problems with fixed installations are destruction by vandals at low flow and by debris at flood flows. Another consideration is the possibility that for many years the floods at a particular site may be minor and that a permanently-mounted transducer would serve no worthwhile purpose. The latter problem was evident in this study because no significant floods occurred at any of the sites in 1968, 1970, and 1972.

For smaller streams, where bridge openings were less than 100 ft (30 m) wide, debris collected on the piers and across the bridge openings presented measuring problems which were not overcome during this study. At bridge 233 on the Chena River near Two Rivers, reliable measurements could have been made if a crane had been available to remove the debris from the pier nose and from the left abutment.

However, this was impossible because of road washouts, and the only access to the site during the period of high water was by boat. At bridge 541 on Moose Creek near Palmer, the velocities were so high and the quantity of debris so large that making soundings with a 100-lb (45-kg) sounding weight was very difficult and hazardous. Sites such as Moose Creek near Palmer should not be considered high priority network sites until studies from safer sites have been completed.

Additional field measurements are needed to better define the relationships of factors involved in predicting scour. In connection with future studies the following measurements should be made at all sites:

1. Streambed configuration (cross-sectional shape) in the approach reach preferably at a location one bridge width upstream from the bridge during at least one low-flow period and again during floods.
2. Cross-sectional shapes on both the upstream and downstream sides of the bridge during both low flow and floods.
3. Flow depth, velocity, and direction of flow approaching piers during high flows.
4. Streambed configuration around bridge piers or abutments during low flow and floods.
5. Streambed form during floods.
6. Particle-size distribution of the streambed material in the approach to piers and abutments or at least the determination of the size of material that would likely be present during flood conditions.
7. Sizes of streambed material in the scour holes during floods. (In some instances streambed material at the maximum depth of scour may be obtainable by excavating during low water after a flood has receded.)
8. Water stage at bridge and approach sections and intermediate determinations of stage if possible.
9. Bridge and channel geometry.

To interpret the measurements it would be desirable to have the following:

1. Records of bridge construction and maintenance.
2. Photographs during low and high flows and aerial photographs and topographic maps of the bridge site including the channel upstream and downstream.
3. Records of flood histories.

A study which apparently has not yet been done and which would seem desirable would be one that began before a bridge was constructed and that carefully followed the bridge and channel changes during and after construction. Currently, the approach cross section is being used as a reference point for general scour based on the assumption that the bridge construction does not alter the approach cross section. This assumption may or may not be valid and a further study would better define the changes which actually take place in the bridge opening before and after construction.

CONCLUSIONS

This study has resulted in the following conclusions:

1. The general-scour formulas for long contractions, $\frac{y_2}{y_1} = \left(\frac{B_1}{B_2}\right)^{0.64}$ (Griffith and Straub *in* Culbertson and others, 1967, p. 30 and 29, respectively) and $\frac{y_2}{y_1} = \left(\frac{B_1}{B_2}\right)^{0.59}$ (Laursen, 1958) will predict the mean depth of flow in a contracted opening within 10 percent of the actual depth for streams with gravel and cobble beds.

2. In a fairly uniform or contracting reach, minimum streambed elevation tends to remain constant, but its position can migrate laterally. The depth of the minimum streambed elevation below the mean streambed elevation during floodflows was generally less than 60 percent of the mean depth. Exceptions occurred where flows were concentrated along riprap banks.

3. Streambed particle size and pier width were the dominant parameters in describing local scour for round or pointed-nosed piers alined with the flow. Proposed design equations for such piers are as follows:

   $d^*_{se} = 3b^{0.8}$ where bed material d_{50} is 0.03-0.25 mm

   $d^*_{se} = 2b^{0.8}$ where bed material d_{50} is 0.25-8mm

   $d^*_{se} = 1.2b^{0.8}$ where bed material d_{50} is > 8 mm

   Each equation represents an upper limit for the maximum amount of scour at a pier in material of a particular particle-size class. Additional instances of scour, which may result from alinement of the pier with the flow, must be considered separately.

4. Where dunes are present, the minimum streambed elevation of the scour hole at the nose of a pier (with zero angle of attack) fluctuates with a magnitude about half that of the dune height.

5. Skewness of a long pier increases the depth of local scour and moves the point of maximum scour downstream from the pier nose.

6. More field studies would be required to better define the effects of streambed particle size and pier-nose shapes on local pier scour.

7. Pier scour during the mean annual flood (RI about 2 years) probably approaches that which might occur during design floods, and important scour data, for the sediment transport situation, can be obtained at that time.

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LIST OF SYMBOLS

B_1 = Width of water surface in uncontracted or approach section

B_2 = Width of water surface in contracted section

b = Width of pier

d = Representative sediment size

d_{50} = Median diameter, sediment size at which 50 percent of material is finer

d_{90} = Sediment size at which 90 percent of material is finer

d_{sc} = Depth of mean scour in contracted section

d_{se} = Equilibrium scour depth, measured from the mean or ambient bed elevation around the scour hole

d^*_{se} = Maximum equilibrium scour depth for a given pier and sediment size, measured from the mean or ambient bed elevation around the scour hole

k_s = Shape coefficient for various pier nose forms

Q = Discharge of water

RI = Recurrence interval for a flood of a given magnitude

y_1 = Mean depth of flow in uncontracted or approach section

y_2 = Mean depth of flow in contracted section

y_a = Mean depth of undisturbed flow approaching a pier

APPENDIX

Chena River Near Two Rivers - Bridge 233

Description

This study site is located at bridge 233 which spans the Chena River 41 mi (66 km) east of Fairbanks at mile 42 on the Chena Hot Springs Road. An aerial view of the site is shown in figure 69.

The bridge is 160 ft (48.8 m) long and is supported at midspan by a 1-ft (0.30-m)-wide pointed-nosed pier with a slight positive rake. A large pile of debris had collected on the pier nose at the time of study and is visible in the downstream view of the bridge at low flow in figure 70 and during high flow in figure 71.

The Chena River in the vicinity of the study site is a meandering nonglacial stream and the bridge crosses a straight reach of the channel (fig. 69). Streambed particle size ranges from gravel to cobbles.

Daily streamflow records have been collected since 1967 at a site 1.6 mi (2.6 km) downstream from bridge 233. The drainage area at that site is 941 mi^2 (2,436 km^2), and annual peak flows from 5 years of record have ranged from a low of 3,500 ft^3/s (99.1 m^3/s) in 1970 to a high of 10,800 ft^3/s (305.8 m^3/s) in August 1969. A flood in August 1967, of unknown discharge, was 6.6 ft (2.01 m) higher than the stage of the peak in August 1969.

A plan view of the study site is shown in figure 72. Gage datum used at the bridge is 704.15 ft (214.63 m) above mean sea level.

Little was known of the streamflow when the site was established in 1969; a significant flood for the study was estimated to be about 9,000 ft^3/s (254 m^3/s). Data were obtained during such a flood on August 6, 1969. However, the data were incomplete and definite conclusions could not be established for this particular flood situation. Some data were obtained at other times, however, and are considered of sufficient interest to be included as part of the appendix.

Summary and Discussion of Observations

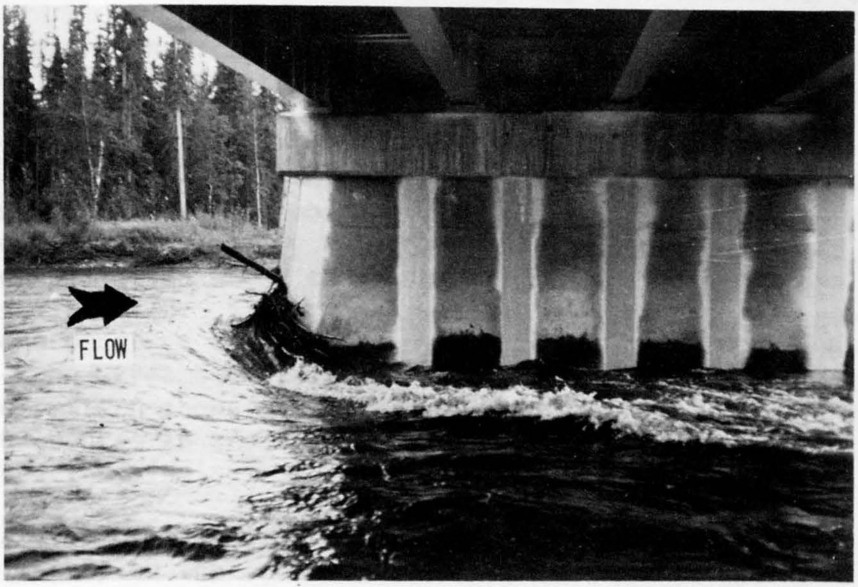
Cross-sectional measurements were obtained at the approach section during low flow on June 5, 1969, at a discharge of about 9,000 ft^3/s (254 m^3/s) on August 6, 1969, and at a discharge of 7,500 ft^3/s (212 m^3/s) on May 20, 1971.

Unfortunately, on August 6, 1969, water-surface elevations were not obtained. An estimate of water-surface elevation at the bridge was made from the photograph in figure 71 from which the water surface at the

NORTH PACIFIC AERIAL SURVEY

Figure 69. - - Aerial view of Chena River near Two Rivers at bridge 233 on May 23, 1972.

Figure 70.-- Downstream view of bridge 233 during low flow on June 5,1969, at Chena River near Two Rivers.



A

B

Figure 71.-- Bridge 233 during high flow, August 6, 1969.
A. Water surface at pier, Q=9000 ft ^{3}/s (254 m ^{3}/s)
B. Upstream view after peak of 9000 ft ^{3}/s (254 m ^{3}/s)

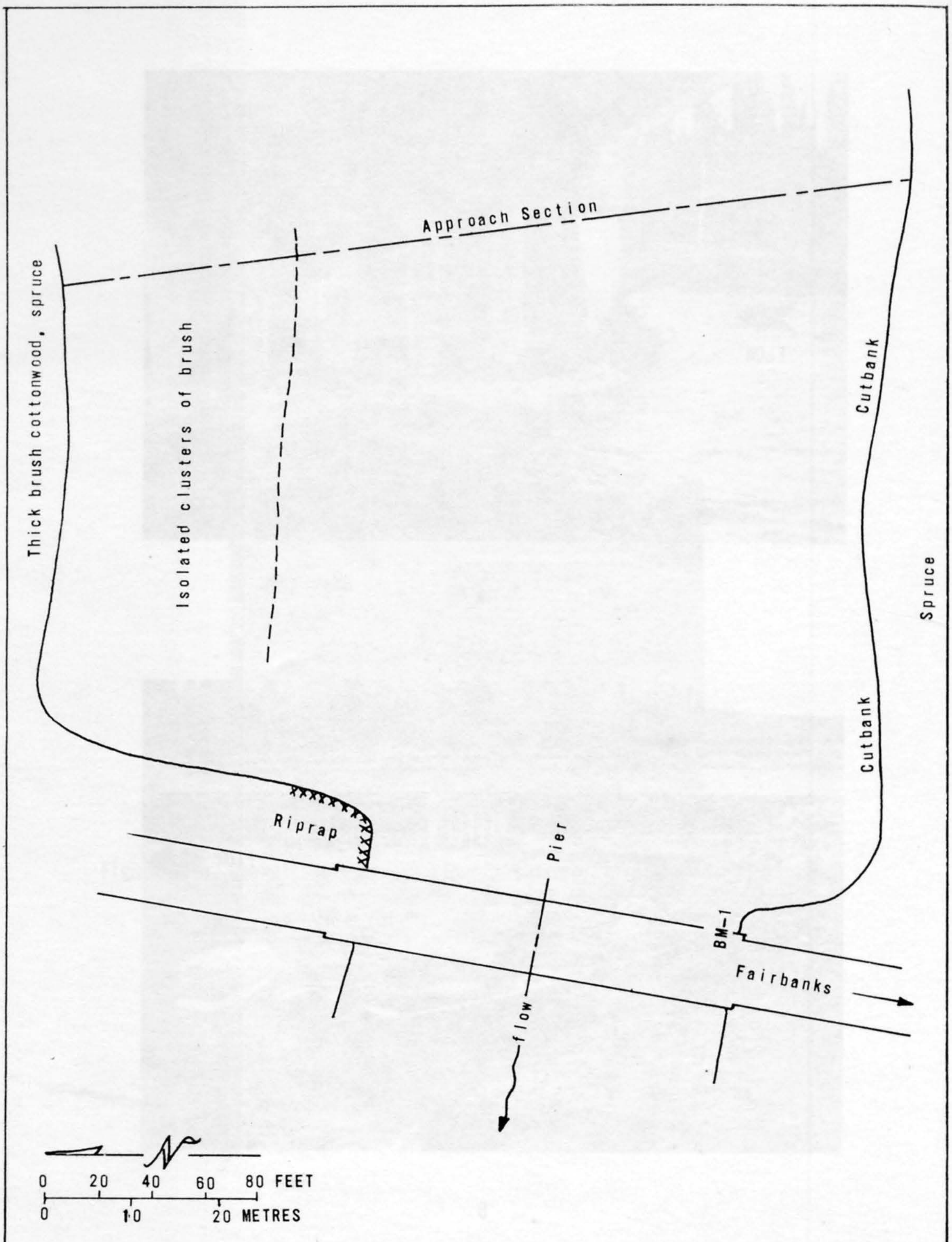


Figure 72. - - Plan view of Chena River near Two Rivers at bridge 233.

approach section was then estimated using a slightly greater water-surface slope than that measured on May 20, 1971 (0.002 ft/ft or m/m).

A comparison of the three measurements at the approach section indicated no significant differences in the streambed elevation (assuming the estimated water-surface elevation of August 6, 1969, is correct).

At the upstream side of the bridge, five partial or complete measurements of the streambed elevation show that the mean streambed elevation is approximately 4-6 ft (1.2-1.8 m) lower than the original streambed elevation prior to construction. These five measurements are shown in figure 73. Since construction the lowest mean streambed elevations occurred during the low and intermediate flows, and the highest mean streambed elevations occurred at the higher flows. This is the same pattern shown on the Tanana River at Nenana at bridge 202. Both are located in crossover cross sections which, as noted earlier, reinforces the findings of Lane and Borland (1954) and Neill (1964).

Analysis of the data is complicated by the effect of the debris which piled up on the left abutment and the pier (figs. 70-71). However, the measurements illustrate that the effects of debris and rough riprap banks must be considered when the designer estimates the potential for scour.

Although attempts were made to sound near the pier, the debris and turbulence prevented reliable measurements. Measurements about 10 ft (3 m) upstream from the pier and debris show a streambed elevation about 2 ft (0.6 m) higher than the surrounding streambed during high flow. It is not known whether the hump is of alluvium or debris.

Analyses of streambed material at low and high flow in the approach section indicate the d_{50} and the d_{90} to be about 64 and 100 mm, respectively.

Moose Creek Near Palmer - Bridge 541

Description

The study site is located where bridge 541 spans Moose Creek, 6 mi (6.4 km) northeast of Palmer at mile 54.9 on the Glenn Highway. The girder bridge is 180 ft (54.9 m) long and is supported by two 1.75-ft (0.53-m)-wide pointed-nosed piers.

Moose Creek is a steep nonglacial stream draining about 60 mi^2 (155 km^2) of mountainous terrain above the bridge. At low flow the water flows parallel with the piers beneath the bridge. During high flows, a sharp bend of the channel just above the bridge causes the flow to approach at angles up to 40°. Streambed material is composed of gravel, cobbles, and boulders as shown in the photograph in figure 74 showing the channel above the bridge at low flow. Figure 75 is a plan view of the site.

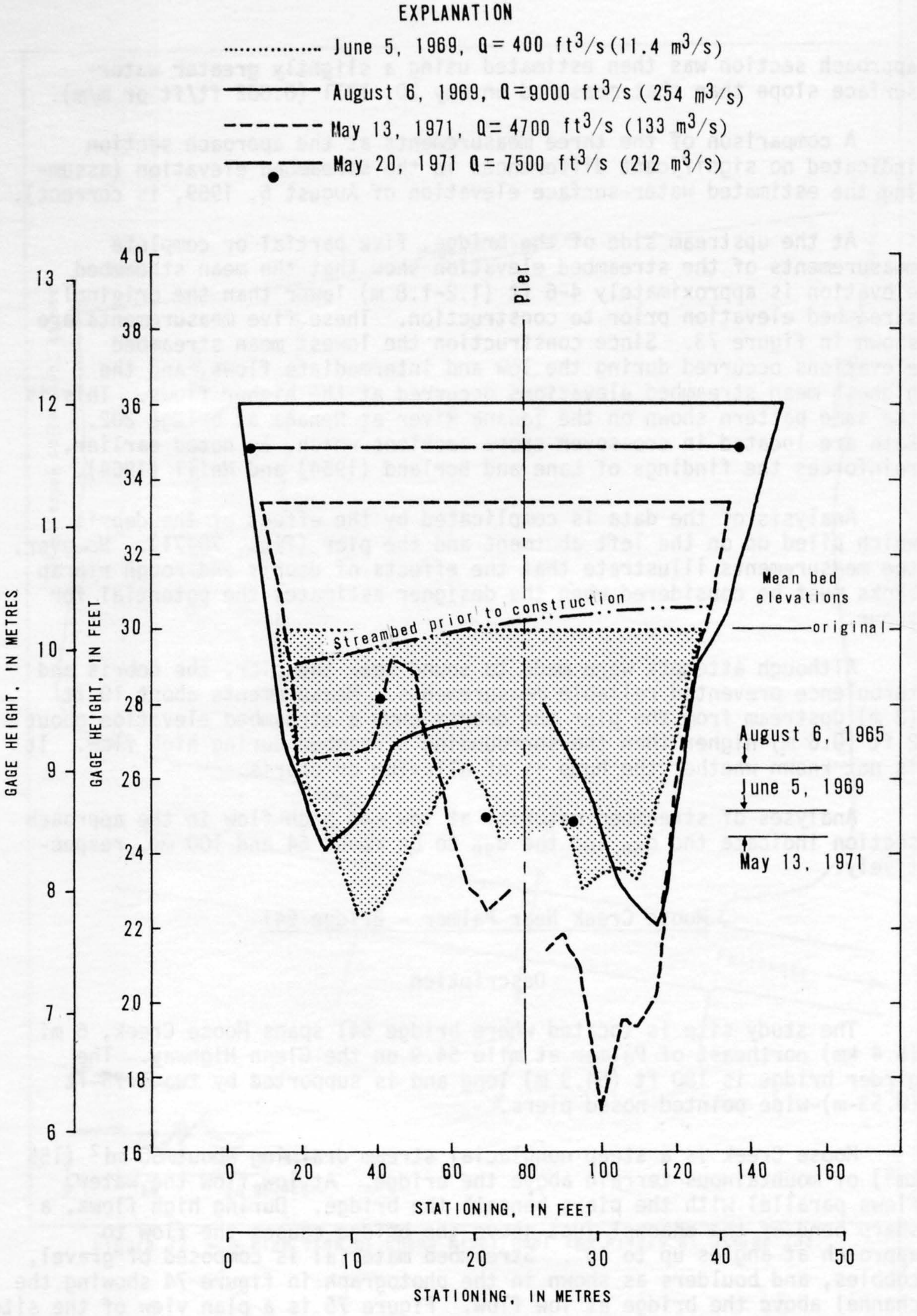


Figure 73. -- Cross section on upstream side of bridge 233, Chena River near Two Rivers.

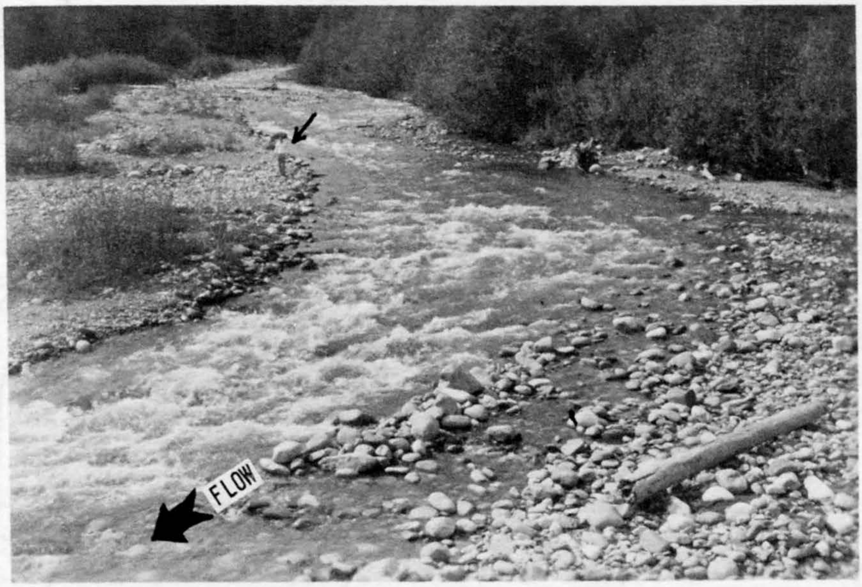


Figure 74.-- View of low flow channel upstream from bridge 541, Moose Creek near Palmer, May 27, 1969. (Man, above and to the left of center, suggests scale.)

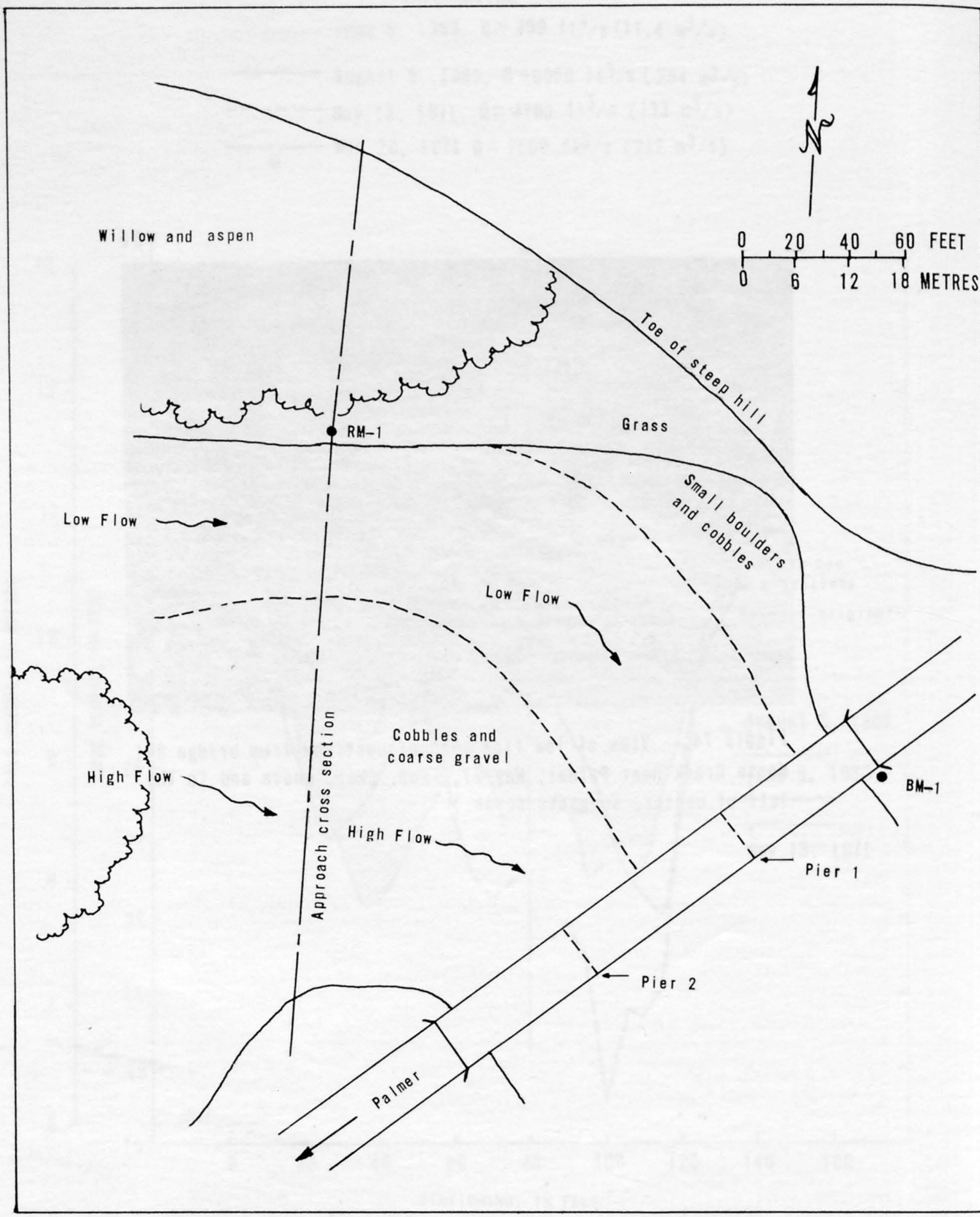


Figure 75.-- Plan view of Moose Creek near Palmer, bridge 541.

Significant floods at this site are the result of rainfall. During floods, velocities beneath the bridge exceed 25 ft/s (7.62 m/s) and the flow is super critical.

In August 1971 heavy rainfall caused flooding conditions on Moose Creek and other streams in south-central Alaska (Lamke, 1972). An attempt was made during that flood to measure the changes in the cross-sectional profile at the bridge, but the data were unobtainable because of the extreme velocities and quantities of debris at the site. The right-bank approach fill washed out on the morning of August 10. However, some data were obtained on the evening of August 9.

Summary and Discussion of Observations

On the upstream side of the bridge, cross-sectional measurements were made during low flow on May 27, 1969, and on August 14, 1971, after a peak of about 18,000 ft^3/s (510 m^3/s) (Lamke, 1972) occurred during the morning of August 10, 1971. Two high-water measurements were made in the evening of August 9, 1971, during a discharge of about 5,000 ft^3/s (140 m^3/s). These latter measurements were not complete or accurate because of extreme high velocities and effects of debris choking parts of the bridge opening. A composite of the measurements is shown in figure 76.

Figure 77 shows the appearance of the water surface and debris on the upstream side of the bridge during the measurements of August 9. Velocities of over 25 ft/s (7.6 m/s) through the center span were estimated by timing floating trees (fig. 77). By late evening on August 9, debris had essentially closed both end spans of the bridge and all of the flow was going through the center opening.

Additional precipitation during the night of August 9 and early August 10 produced the peak of about 18,000 ft^3/s (510 m^3/s) during which the right approach fill was washed out. Undoubtedly the plugging of the two outside spans was one of the principal causes that forced the high water to attack the approach fill severely and caused it to fail. However, if the approach had not failed, the piers could have been undermined by scour. Figure 78 shows two views of the site on the morning of August 10 in which the debris is still trapped in the bridge opening and most of the flow is bypassing the bridge.

The dynamics of the water surface and the streambed on August 9 are illustrated in figure 76. The water-surface elevation varied as much as 4 ft (1.2 m) across the opening and in little more than an hour the streambed had been scoured more than 1 ft (0.3 m) in the center opening and had been filled more than 1.5 ft (0.46 m) in the left opening. The August 14 measurement at low flow shows that the streambed was slightly higher than the original low-flow elevation.

From the experiences at Moose Creek near Palmer and Chena River near Two Rivers, it should be obvious that the effects of debris cannot be ignored in designing bridge openings for smaller streams.

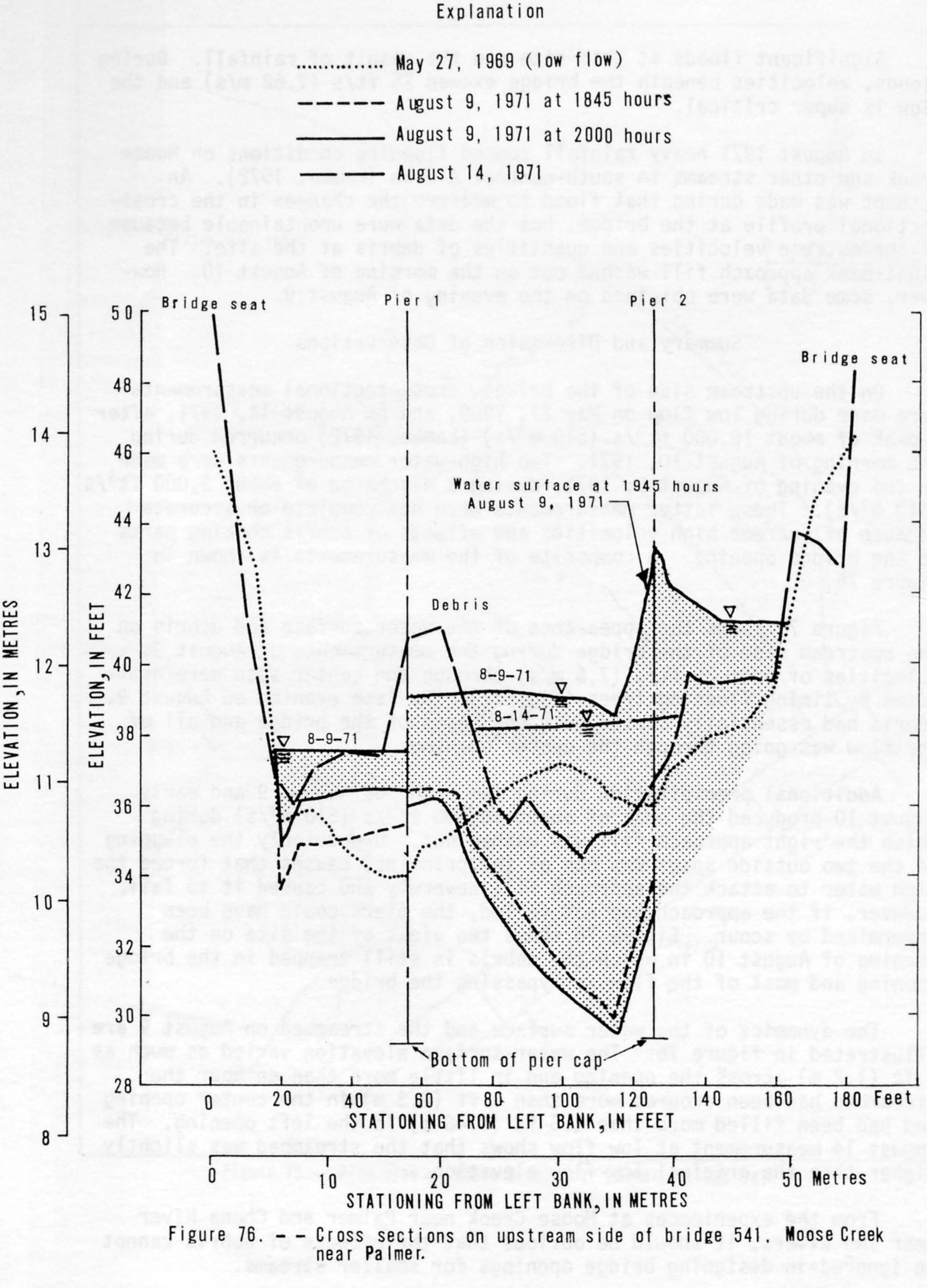
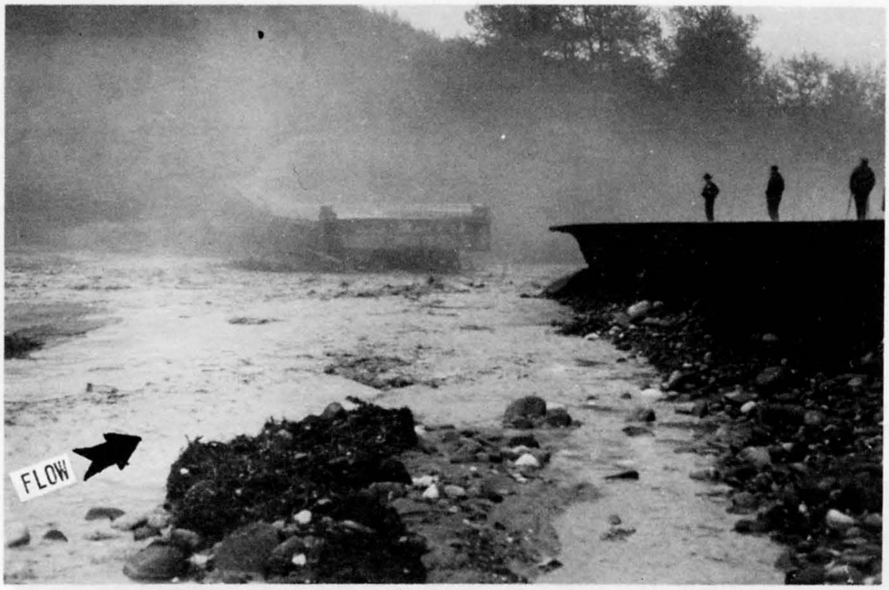


Figure 76. - - Cross sections on upstream side of bridge 541, Moose Creek near Palmer.

Figure 77.-- Bridge opening on August 9, 1971, at Moose Creek near Palmer. $Q = 5{,}000\ ft^3/s$ ($140\ m^3/s$)



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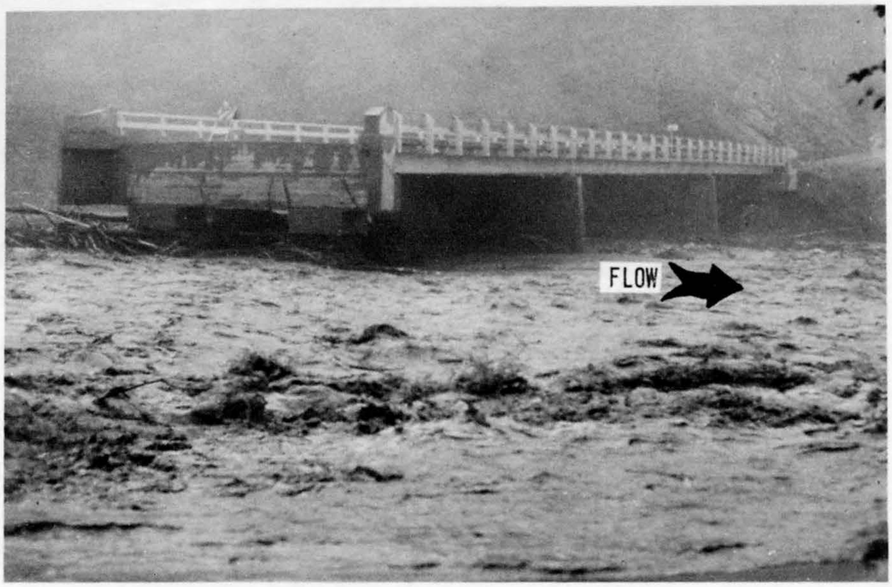

B

Figure 78.-- Approach failure as seen from the right bank at Moose Creek near Palmer on August 10, 1971.

A. Upstream side of bridge.

B. Downstream side of bridge.

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