STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

Basic Research, Analysis & Exploratory Investigations

KNIK ARM HIGHWAY CROSSING

ANCHORAGE, ALASKA

JANUARY 1972

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
HNTB

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
HNTB

1805 GRAND AVENUE, KANSAS CITY, MISSOURI 64108
TELEPHONE: CODE 816, 474-4900
CABLE: HOWARDNEED KANSASCITY

January 17, 1972

Mr. B. A. Campbell
Commissioner of Highways
State of Alaska
Department of Highways
Juneau, Alaska 99801

Re: Knik Arm Crossing Study
Anchorage, Alaska

Dear Mr. Campbell:

In accordance with the Contract executed on January 5, 1971, we are pleased to present herein the Report covering the results of the Knik Arm Crossing Studies. The Report summarizes the investigations of professionals specializing in port and coastal engineering, hydraulics, structures, geology, soil mechanics and ecology into the extraordinary and unprecedented problems associated with design and construction of a highway crossing of Knik Arm. It is concluded that:

1. The construction of a crossing is a feasible undertaking.

2. A bridge is the most advantageous type of structure.

3. The most favorable crossing site is located about 1-1/2 miles upstream from Cairn Point.

4. The estimated total project cost for the most favorable type and location of crossing is $140,000,000.

The studies indicate the urgent need that additional information regarding foundation soils (both on shore and under the water) be obtained prior to further project development. Likewise, extensive additional field investigations will be necessary to arrive at economical solutions of problems associated with ice and water currents.

Grateful acknowledgment is made of the assistance given by the Alaska Department of Highways in the performance of these investigations. Also, we are appreciative of the cooperation received from numerous other State and Federal agencies.

Respectfully submitted,

HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Paul L. Heineman

Paul L. Heineman






STATE OF ALASKA
DEPARTMENT OF HIGHWAYS

Basic Research, Analysis & Exploratory Investigations

KNIK ARM HIGHWAY CROSSING

ANCHORAGE, ALASKA

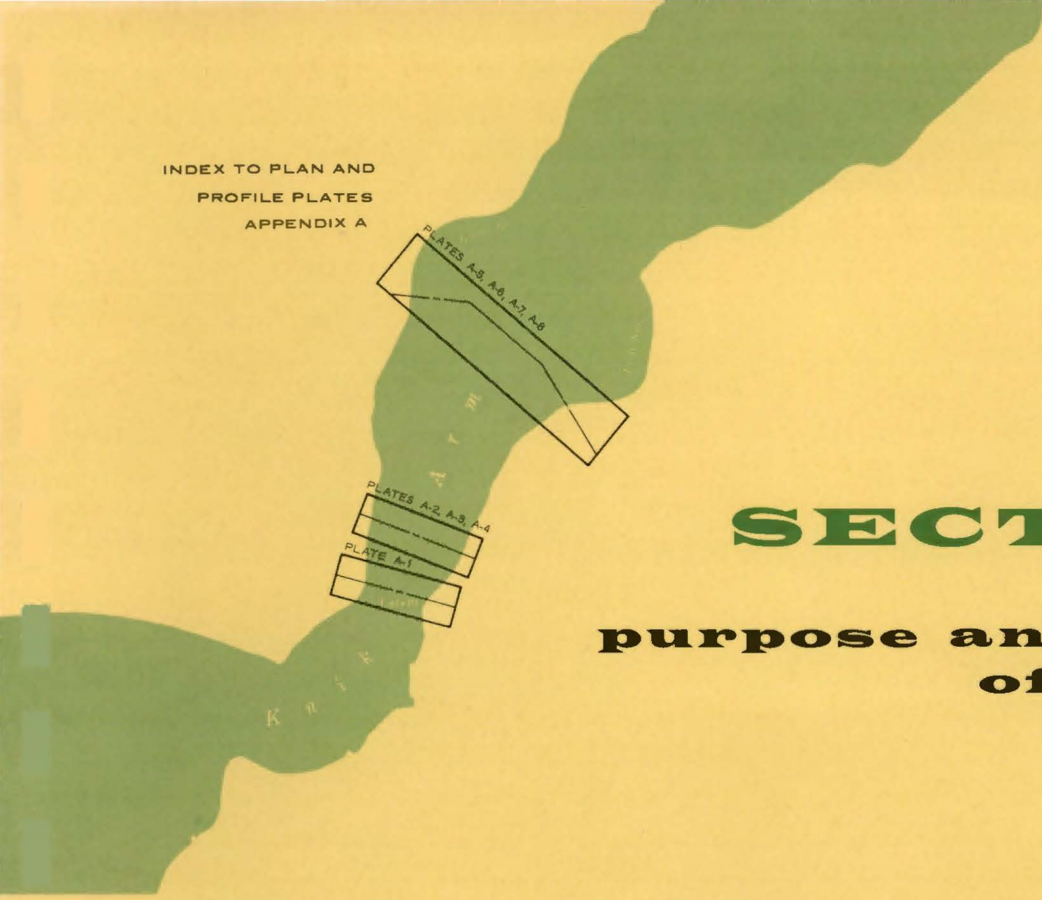
JANUARY 1972

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS | HNTB

table of contents

list of tables

list of figures



SECTION I

purpose and scope of report

Page

purpose and scope of report

INTRODUCTION

The history of Anchorage has been one of continual expansion. Purchase of Alaska by the United States in 1867 opened the area to economic development and fur trading and mining were actively pursued. The Cook Inlet area was always popular as a transportation center because of the availability of deep water. Several communities were established as shipping centers to supply the mining areas. Among these were Tyonek, Hope, Knik and the Ship Creek area. The choice of Ship Creek by the Alaska Engineering Commission in 1914 as a work and supply camp for construction of the Alaska Railroad established the townsite that was to become Anchorage.

The military became firmly established in the Anchorage area during World War II, and the Alaskan Command Headquarters are now at Elmendorf Air Force Base and Fort Richardson.

Anchorage enjoys the position of being the economic, industrial, transportation and military center of the State. While the Anchorage metropolitan area currently has a population of well over 100,000, predictions indicate about 200,000 people by 1990. This expansion could be jeopardized if proper transportation corridor construction is not soon implemented.

Knik Arm, Turnagain Arm, the Chugach Mountains and the extensive military land holdings constitute barriers to future expansion of the Anchorage metropolitan area. The discovery and exploitation of oil fields on the Kenai Peninsula provided the impetus needed to study the feasibility of crossing Turnagain Arm. Several comprehensive reports resulted from these studies; however, the project has not extended beyond this phase.

The proximity to Anchorage and favorable terrain makes the lands north and west of Knik Arm attractive for incorporation into the active Anchorage community. This has long been realized and in 1955 the Anchorage Chamber of Commerce commissioned Ivan Bloch and Associates to make an economic study for a crossing of Knik Arm at Cairn Point. This report stressed the defense benefits of such a crossing. However, Statehood and a change in U. S. Military posture have shifted the emphasis to civil benefits which are far greater than those envisioned in 1955.

Figure I-1 STUDY AREA LOCATION

A crossing of Knik Arm would shorten the Anchorage–Fairbanks highway by about 50 miles and would give a convenient start for a road to the Beluga area and the Alaska Peninsula with its mineral and recreation potential.

PURPOSE AND SCOPE OF STUDY

This study was initiated by the Alaska Department of Highways for the express purpose of exploring the technical problems associated with an engineering structure across Knik Arm. The limits of the study area were defined as Point Mackenzie on the west and Eagle River on the east.

Budgetary limitations have severely limited field investigations. Thus, existing data has been used as a basis for this study.

The study has encompassed various structure types (bridge, dam, tunnel); locations, in light of known and surmised soil conditions; and construction methods for the various proposals. Areas needing extensive laboratory and field investigations have been defined in preparation for subsequent design phases of the Knik Arm Crossing.

ACKNOWLEDGMENTS

Appreciation is expressed for the aid in searching out information given by the Anchorage offices of the U. S. Geological Survey and the U. S. Coast and Geodetic Survey. Cooperation of the various offices of the Alaska Department of Highways is gratefully acknowledged as is the general cooperation and interest displayed by all Anchorage people who were contacted during the performance of this study.

Acknowledgment is also made of the significant contributions of Dr. Per Bruun, Consulting Port and Coastal Engineer, Mr. George J. Eicher, Ecology Consultant, Jacobs Associates, Construction Consultants, Dr. R. B. Peck, Soils and Foundations Consultant and Shannon & Wilson, Inc., Soil Mechanics and Foundation Engineers.

INDEX TO PLAN AND
PROFILE PLATES
APPENDIX A

SECTION II

summary

Page

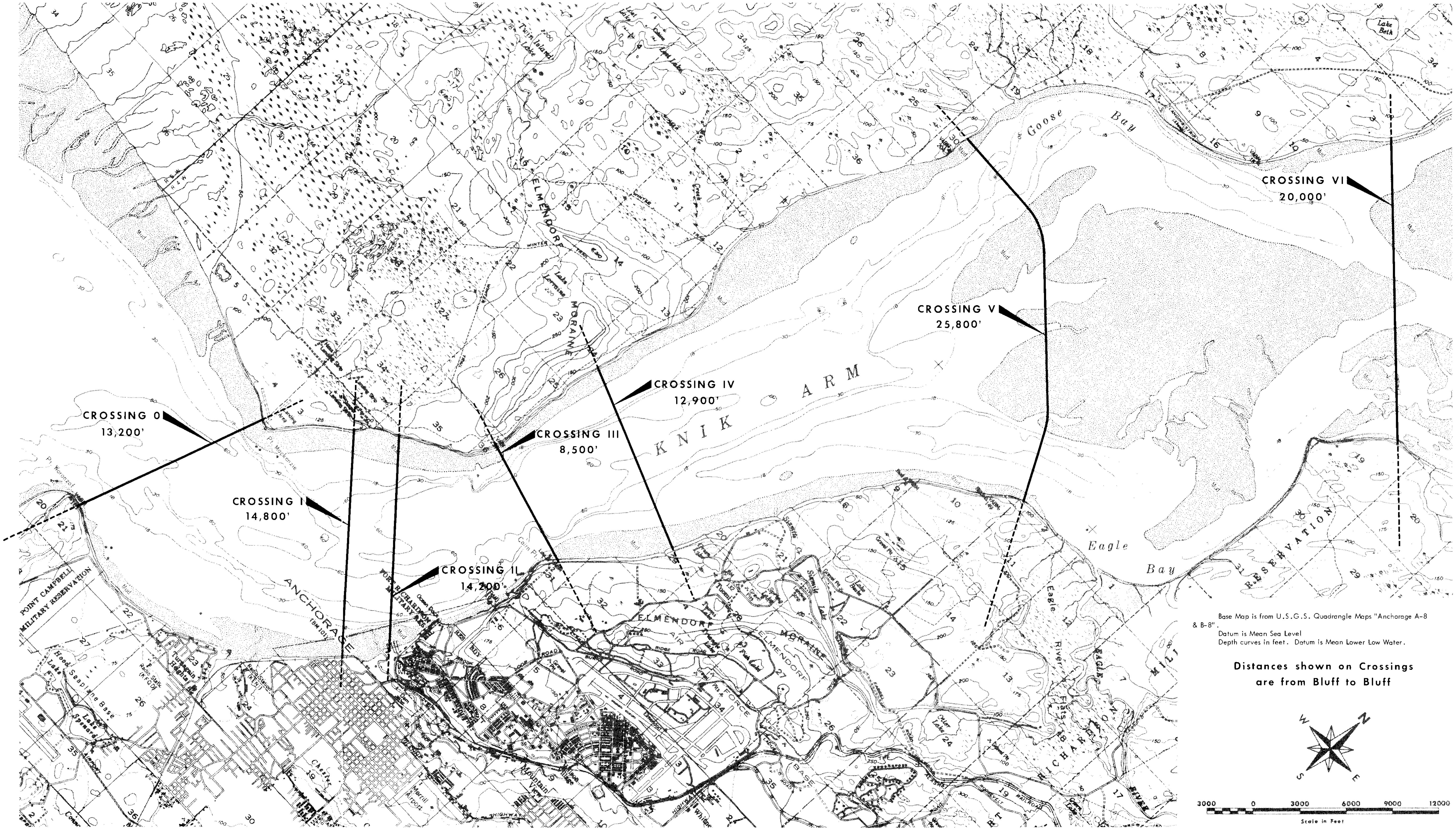


Figure II-1

ALTERNATIVE LOCATIONS OF CROSSING

summary

MOST FAVORABLE LOCATIONS

The findings presented in this report indicate a bridge crossing located about 8,000 ft. upstream of Cairn Point (Crossing IV) as the most favorable location and type of highway structure across Knik Arm. Crossings designated III, IV, and V were selected early in the studies for more detailed analysis as research indicated other crossings did not have significant advantages and in fact had many disadvantages. The various crossings considered are shown on Figure II-1. A detailed comparison of the various crossing locations is given in Section IV of this report.

All portions of the shoreline from Point Woronzof to Eagle Bay on the east, and Point Mackenzie to the Goose Bay area on the west, were included in the study area for this report.

The principal criterion for this report was to find a suitable highway crossing which can be built for the least amount of money. This structure must meet acceptable standards of safety and service and be readily accessible to the users.

COST OF ALTERNATE CROSSINGS

After preliminary design forces were ascertained, various structural systems were analyzed to determine the most economical systems which could successfully resist those forces. Once this was accomplished, quantities of material necessary to construct the various applicable structure types were calculated. These material quantities were multiplied by unit prices to determine estimated construction costs for each location and type of crossing.

Table II-1 indicates estimated bid price construction costs for six alternates. These alternates include a Suspension Bridge at Crossing III, a Truss Bridge at Crossing IV, a Causeway Dam at Crossing IV, a Truss Bridge at Crossing V, a Causeway Dam at Crossing V, and a Combination Bridge–Causeway at Crossing V. The estimated time for construction is also indicated. Both the cost estimate and the time schedule are predicated on an assumed start of construction date of January, 1972. These estimated contractor bid prices reflect probable wage and material price escalations occurring during the years this project would be under construction.

Table II-1 COMPARATIVE COSTS OF CONSTRUCTION

ALTERNATE	ESTIMATED BID PRICE (million $)	SCHEDULE (years)	DISTANCE (Bluff to Bluff-ft.)
Truss Bridge at Crossing IV	126.0	4	12,900
Causeway Dam at Crossing V	209.0	5	25,800
Truss Bridge at Crossing V	223.0	6	25,800
Bridge-Causeway at Crossing V	230.0	5.5	25,800
Suspension Bridge at Crossing III	249.0	7	8,500
Causeway Dam at Crossing IV	289.0	6	12,900

Note:

Crossing 0, I, II and VI cost estimates are not included since these crossings are not considered viable alternatives.

Distances shown do not include approaches.

Derivation of above estimates is given in Appendix "D".

In addition to the estimated bid prices for construction, other costs would be incurred for successful project development and completion. These costs include contractor contingencies, subsurface testing, surveys, test structures, model testing, engineering, and administration. Estimated project costs for the most viable solutions are shown on Table II-2.

Total estimated project cost for a Bridge at Crossing IV is $140,000,000
Total estimated project cost for a Causeway Dam at Crossing V is $231,000,000.

Data obtained during the soils reconnaissance indicate that substantial portions of the bluffs along Knik Arm have undergone varying degrees of sliding in the past. It should be pointed out that considerable cost could be added to the tabulated project cost of a Bridge Crossing at Line IV, if soils borings indicate the need for extensive slope protection.

BENEFITS

This generation and future generations would benefit greatly from a permanent highway crossing joining the two sides of Knik Arm near Anchorage. The Knik Arm Crossing connecting with the nearly completed Anchorage-Fairbanks Highway would reduce the distance by about 50 miles. Future

Table II-2 ESTIMATED TOTAL PROJECT COSTS

	BRIDGE CROSSING IV	CAUSEWAY DAM CROSSING V
Total for Crossing	$114,938,200	$189,590,600
Contingencies and Variations (10%)	11,061,800	19,409,400
*Estimated Construction Cost	$126,000,000	$209,000,000
**Borings and Soil Testing	410,000	837,000
Hydrographic and Land Surveys (0.75%)	945,000	1,567,500
Model Testing and/or Test Structure	520,000	225,000
Engineering and Administration		
Basic Design (4.0%)	5,040,000	8,360,000
Construction Supervision (4.0%)	5,040,000	8,360,000
Administration (1.5%)	1,890,000	3,135,000
Estimated Project Cost (nearest million)	$140,000,000	$231,000,000

*Based on 1971 start and 1975 finish
**Includes Geophysical Surveys

economic development of the west side of Knik Arm would certainly add to the potential of the metropolitan area of Anchorage.

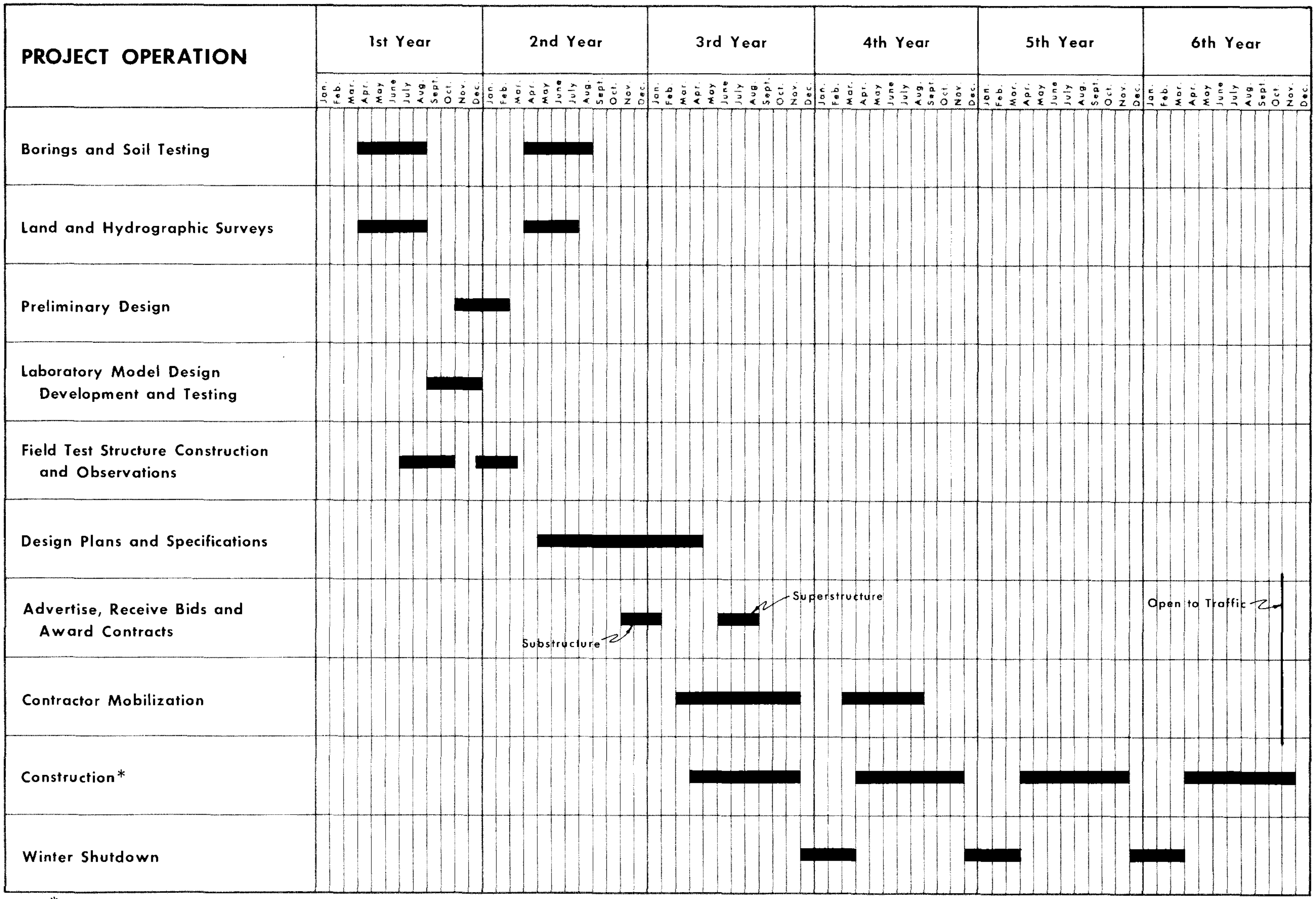
A highway link also would provide the necessary access for a new international airport which has been mentioned for location on the west side of the arm. Such a facility presents an interesting stimulus for the future economic development of the west side of Knik Arm.

Another advantage of providing means for development access of lands north of Knik Arm is the existing geographic position of Anchorage. The city is presently surrounded by water, mountains and military facilities. Also, the world-wide recognition which would accompany the construction of this unique and monumental project would certainly be valuable to the State of Alaska.

SCHEDULE FOR FUTURE STUDIES, DESIGN AND CONSTRUCTION

It is estimated that construction time for a bridge at Crossing IV or a causeway dam at Crossing V would be 4 and 5 years, respectively. The time for design, surveys, model studies, soils borings, etc., are estimated to be 2 years for the bridge and 3 to 4 years for the causeway dam. Therefore, it could be expected that the bridge can be completed 6 years after approval to proceed, and the causeway dam in 8 to 9 years. Figure II-2 shows a suggested schedule for subsequent project development. Bar graphs illustrating functions and estimated time schedules for the construction of other alternates are illustrated in Appendix D.

Further discussion concerning recommendations for future studies appears in Section IX, Conclusions and Recommendations, of this report.



*For details of construction sequence, see Appendix D - Construction Schedule for Truss Bridge at Crossing IV.

Figure II-2 SCHEDULE FOR SUBSEQUENT STUDIES, SURVEYS, BORINGS, DESIGN, CONTRACT AWARD AND CONSTRUCTION**

**Schedule assumes a bridge at Crossing IV.

SECTION III

research studies and design parameters

Page

Page

research studies and design parameters

INTRODUCTION

During the course of this study a broad range of field and office investigations were conducted with numerous experts contributing their individual professional skills. Aerial photographs were taken in March, 1971, to provide data regarding the extent of ice cover, the size of individual ice floes and the velocity of floe movements. Unique tidal variations necessitated field surveys of tides in Knik Arm. Data of these surveys, conducted in May and June, 1971, were combined with existing records from the Anchorage ocean dock tide recorder to establish a tidal prism.

Geological and soils reconnaissance were carried out in the field. Visual inspection of the soils and shoreline features was accomplished, and these findings were evaluated in combination with existing soils information. Preliminary designs were qualitatively evaluated for their susceptibility to seismic forces.

Preliminary designs were prepared for each feasible crossing location and type. These designs were revised and up-dated as the studies progressed.

Finally, construction costs and methods were investigated. The approach to this complex problem was to prepare contractor-type estimates insofar as the status of design and information allowed.

The results of simultaneous investigations in the disciplines of oceanography, hydraulics, soil mechanics and geology, foundations, structural design, construction methods, and construction costs are summarized in subsequent sections of this report.

CLIMATOLOGY

General

The Cook Inlet–Anchorage area is in the Transitional Climatic Zone of Alaska. The adjacent Continental and Maritime Zones plus the geographic

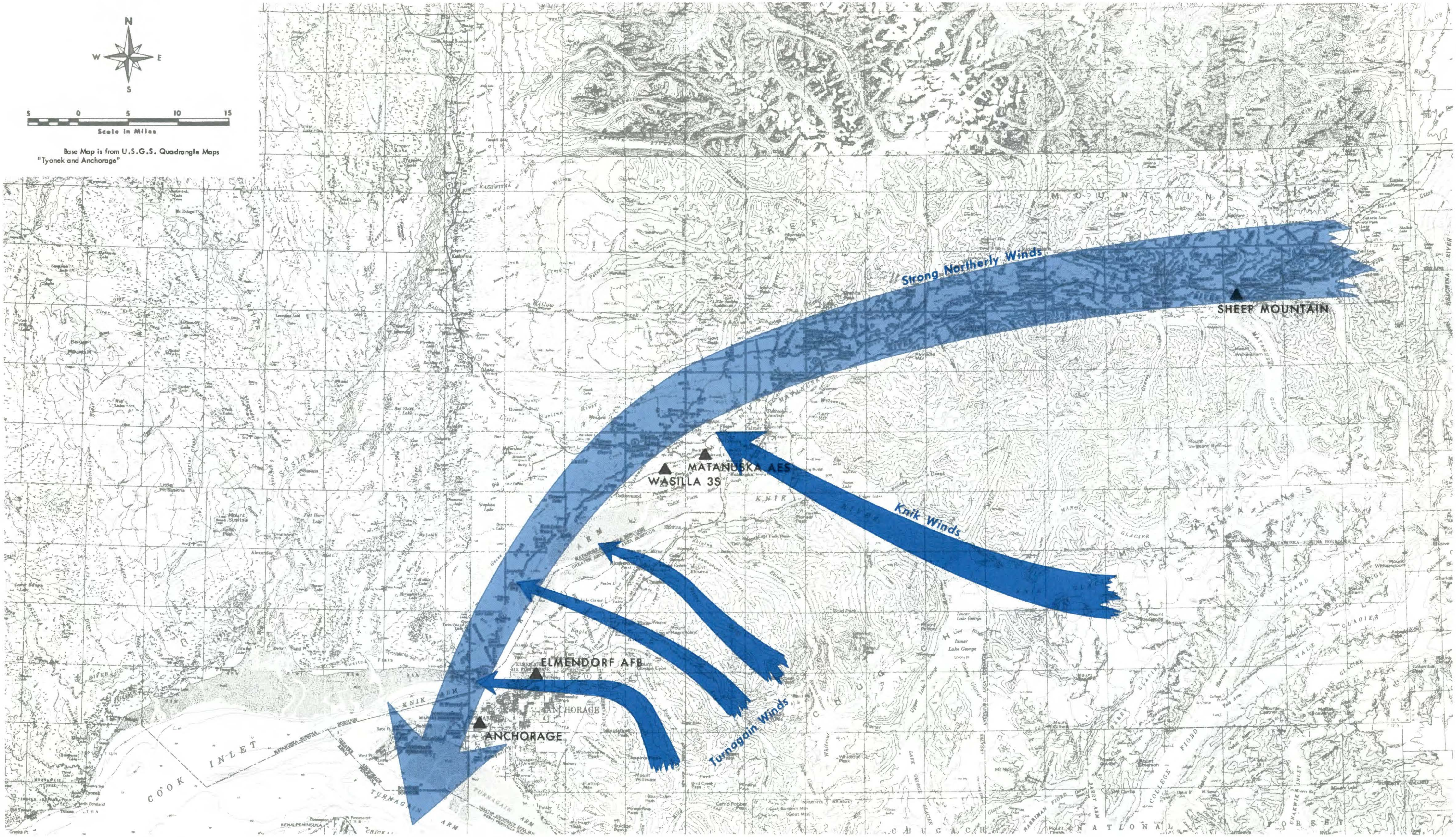


Figure III-1

WEATHER STATIONS AND BASIC WIND PATTERNS features of the area assure a variety of weather. The major geographic feature which modifies Knik Arm weather is the Chugach Mountain Range a few miles to the southeast. This range forms a barrier to the maritime environment of Prince William Sound which supplies the moisture to the ice fields in the upper reaches of the range. These ice fields, extending to 13,000 feet in elevation, supply cold, dense air that sometimes flows down the valleys in the form of strong, local winter winds. Major river valleys such as the Susitna, Copper, Matanuska, and Knik, all form channels which control low level air movements. Sea level pressure diagrams indicate a general low pressure area over the Aleutian Islands throughout the year with high pressure areas over the interior of Alaska or the Arctic Ocean (ESSA Technical Memorandum EDSTM 8, 1969). Storm tracks normally run to the northeast over Anchorage while the predominant most severe winds blow from the northeast.

Weather Station History

Figure III-1 shows the location of pertinent weather stations. The periods of record for weather data listed in this report are as follows:

Anchorage	1931-1969
Elmendorf AFB	1941-1959
Wasilla 3S	1953-1967
Matanuska Agriculture Exp. Sta.	1920-1952

The Anchorage Weather Bureau furnishes the major weather analysis for the area, while the Elmendorf Station is concerned with weather forecasting for military operations. The Wasilla and Matanuska Stations were established to collect data for use by the Matanuska Valley farming community. The Matanuska Station provides one of the two evaporation stations in the whole State, the other being at College, Alaska and outside the concern of this report.

Temperature

The highest recorded temperature in the Knik Arm area was 91 degrees, measured at the Matanuska Station in 1936. A low temperature of minus 50 degrees was recorded at Wasilla in 1947. The coldest period occurs in January and early February, while the warmest month is July, although the highest temperatures usually occur in June because of less cloud cover.

As the mean temperature rises, breakup, or ground thawing, begins in late March and early April and may continue through the month of May in some poorly drained areas. With rapidly decreasing sunshine, the mean temperature decreases to below the freeze line about October 25 each year. Temperature data listed in U.S. Department of Commerce Publication 60-49 ("Climates of the States–Alaska") converts to an annual average of 650 centigrade degree days below -7°C(20°F±).

Table III-1 shows the average date of the latest spring occurrence and earliest fall occurrence of 32 degrees F for a number of stations in the Knik Arm Watershed.

Table III-1 32°F. OCCURRENCES

Station	Dates of 32° F. Occurrences	
	Latest in Spring	Earliest in Fall
Anchorage	May 11	Sept. 18
Elmendorf AFB	April 30	Sept. 25
Eklutna	May 20	Sept. 7
Matanuska AES	May 25	Sept. 13
Sheep Mountain	June 5	Aug. 25

SHEEP MOUNTAIN IS ABOUT 85 AIR MILES N.E. OF ANCHORAGE, NEAR THE EASTERN LIMITS OF THE KNIK WATERSHED AT AN ELEVATION OF ABOUT 2,500 FT.

Figure III-2 shows a tabulation of temperature data for four Anchorage area weather stations and a graph of the mean monthly temperature at the Anchorage Station.

Precipitation

The heavy precipitation, for which the southern coast of Alaska is known, is blocked from the Anchorage area by the Chugach Mountains. While the precipitation has been accurately recorded at the low elevations in the Knik Arm area, the moisture must be estimated at the higher levels within the watershed. Mean annual precipitation within the Knik Arm watershed varies from about 80 inches in the upper Chugach Range to about 14 inches at Anchorage. Mean annual snowfall varies from about 200 inches in the mountains to about 75 inches at Anchorage. Approximately half of the area precipitation falls as rain during the summer months with the other half forming the winter snowfall. Major effects of precipitation would be to limit some

MONTH	ANCHORAGE					ELMENDORF AFB					WASILLA 3S					MATANUSKA AES				
	MEANS			EXTREMES		MEANS			EXTREMES		MEANS			EXTREMES		MEANS			EXTREMES	
	Daily Maximum	Daily Minimum	Monthly Mean	Record High	Record Low	Daily Moximum	Daily Minimum	Monthly Mean	Record High	Record Low	Daily Maximum	Daily Minimum	Monthly Mean	Record High	Record Low	Daily Moximum	Doily Minimum	Monthly Mean	Record High	Record Low
JAN.	19.9	4.3	12.1	44	-19	19.3	3.8	11.5	48	-37	22.3	4.0	13.1	52	-37	21.4	3.7	12.6	51	-40
FEB.	26.3	11.1	18.7	41	-21	26.6	9.1	17.9	59	-43	26.8	7.7	17.3	51	-22	27.3	9.3	18.3	56	-41
MAR.	32.5	16.6	24.9	49	-15	33.5	15.1	24.3	50	-25	36.6	13.1	23.4	55	-17	33.6	15.1	24.4	54	-22
APR.	44.5	29.3	36.9	62	10	44.8	27.5	36.1	63	-21	46.0	26.6	36.3	68	- 8	45.5	26.9	36.2	66	-16
MAY	55.6	39.1	47.4	75	17	55.0	37.7	46.4	81	0	58.9	35.7	47.3	75	19	57.8	35.7	46.8	83	8
JUN.	64.4	46.9	55.7	75	40	63.0	46.5	54.7	85	33	69.3	42.8	53.5	88	30	66.4	43.7	55.1	91	27
JUL.	66.0	50.3	58.2	78	38	65.6	50.6	58.0	83	34	68.8	47.8	58.3	85	28	67.7	47.4	57.6	85	31
AUG.	63.0	48.4	55.9	71	37	64.1	48.5	56.4	81	26	66.0	46.0	56.0	81	30	64.9	45.9	55.4	83	27
SEP.	55.5	40.4	48.0	70	29	55.3	40.8	48.1	75	20	58.0	39.0	48.5	76	29	56.7	38.5	47.5	75	16
OCT.	42.6	28.9	35.8	58	2	41.9	27.8	35.3	61	0	43.0	26.6	34.8	63	2	44.0	28.5	36.3	69	- 4
NOV.	28.9	15.5	22.2	53	- 8	27.8	14.3	21.1	57	-21	31.2	16.2	23.7	55	- 2	30.0	15.0	22.5	58	-22
DEC.	20.6	6.3	13.5	46	-30	19.1	5.3	12.2	53	-33	21.3	4.7	13.0	51	-31	21.5	5.5	13.5	55	-34
YR.	43.4	28.1	35.8	78	-30	43.6	27.9	35.6	85	-43	45.0	25.9	35.4	88	-37	44.7	26.3	35.5	91	-41

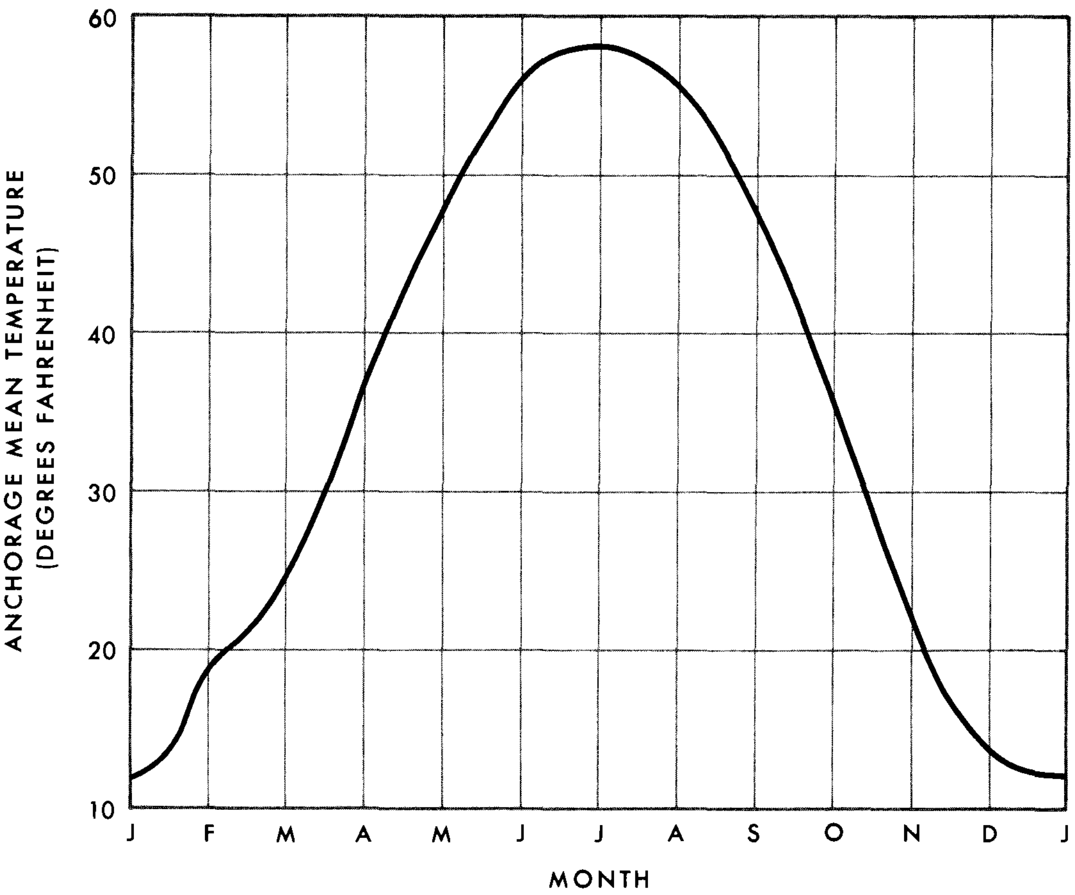


Figure III-2 TEMPERATURE DATA

work during the construction phase of a crossing and to cause snowdrifting and ice formation on a finished structure.

Figure III-3 shows mean and maximum precipitation values for three Anchorage area weather stations with a graph illustrating the mean monthly precipitation figures for the Anchorage Station.

MONTH	ANCHORAGE					WASILLA 3S					MATANUSKA AES				
	RAIN		SNOW			RAIN		SNOW			RAIN		SNOW		
	Mean Total	Maximum 24 Hours	Mean Total	Maximum 24 Hours	Max. Depth on Ground	Mean Total	Maximum 24 Hours	Mean Total	Maximum 24 Hours	Max. Depth on Ground	Mean Total	Maximum 24 Hours	Mean Total	Maximum 24 Hours	Max. Depth on Ground
JAN.	0.80	0.65	11.6	7.1	47	1.18	1.29	9.1	12.5	22	0.94	0.92	7.5	9.0	29
FEB.	0.71	0.48	12.8	6.8	39	1.10	0.81	11.3	8.1	5	0.69	0.84	7.0	8.5	46
MAR.	0.51	0.38	5.7	3.6	38	0.75	0.52	7.8	7.0	25	0.51	0.80	6.5	15.0	14
APR.	0.42	0.28	4.3	4.9	19	0.73	0.63	0.5	2.0	14	0.43	1.10	2.3	10.0	18
MAY	0.52	0.60	0.1	0.2	8	0.97	0.68	T	T	0	0.65	0.83	0.3	5.0	1
JUN.	0.98	0.65	0.0	0.0	T	1.56	0.82	0.0	0.0	0	1.31	1.61	0.0	0.0	0
JUL.	1.86	0.60	0.0	0.0	0	2.83	1.44	0.0	0.0	0	2.01	1.56	0.0	0.0	0
AUG.	2.57	0.82	0.0	0.0	0	3.34	1.67	0.0	0.0	0	2.84	1.20	0.0	0.0	0
SEP.	2.50	1.04	1.5	3.5	T	2.78	1.50	0.0	0.0	0	2.58	2.48	0.2	6.0	T
OCT.	1.87	0.73	7.8	7.4	13	1.87	1.87	4.5	9.0	10	1.72	1.32	3.5	11.0	11
NOV.	1.03	1.66	11.3	16.4	22	1.10	1.79	7.3	13.5	8	0.93	1.50	6.9	10.0	15
DEC.	0.94	0.62	21.3	10.6	47	1.27	1.04	10.7	17.0	23	0.98	1.30	9.2	14.0	18
YR.	14.7	1.66	76.4	16.4	47	19.4	1.87	51.2	17.0	25	15.6	2.48	45.4	15.0	46

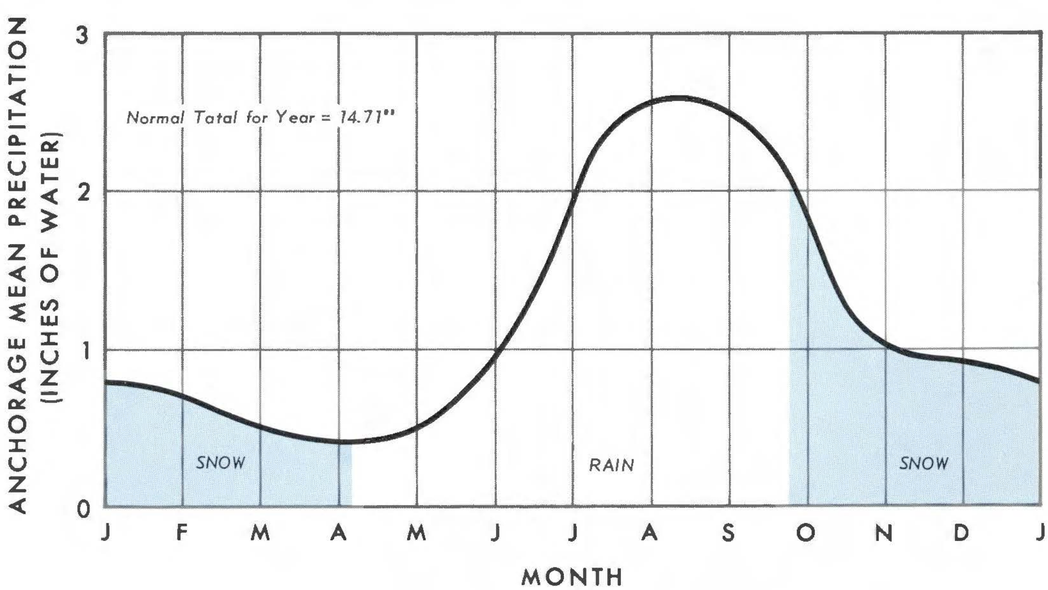


Figure III-3 PRECIPITATION DATA

Winds

Winds, while generally quite moderate in the Anchorage area, can reach serious proportions during the winter months. Strong winds are of two types (Weber, Anchorage Weather Bureau). Figure III-1, shows the general location and direction of these winds. The most severe is the strong, gusty north wind, sometimes called a Matanuska, which may persist for 2 or 3 days at speeds of 20 to 40 knots. During this time, wind speed may reach 35 to 50 knots for 6 hours or more, with short duration gusts of 70 or 80 knots.

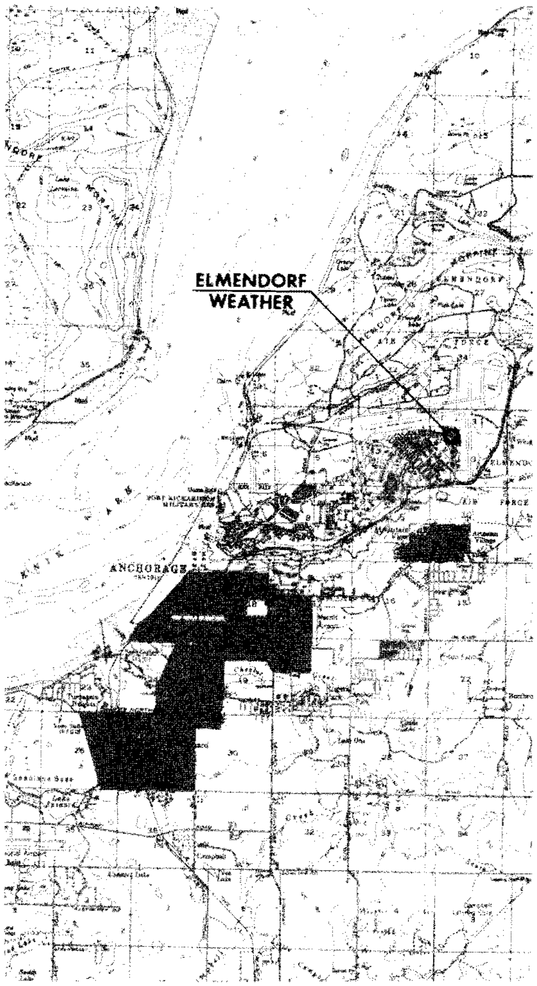
The strong northerly winds normally occur several times each winter and have their origin in a rapidly increasing pressure gradient between the Gulf of Alaska and the northern interior of the State. These winds start as a low level air movement from the Copper River Basin which moves westward through the Matanuska River Valley, turns southward near the Palmer area and flows down Cook Inlet. The wind sometimes moves far enough west to completely bypass Anchorage.

The second of the serious winds are the strong, gusty southeasterly winds which are formed by high pressure in the Gulf of Alaska and may occur at any time of the year, although most of the damaging winds at Anchorage are in the fall. The southeast winds flow over the Chugach Range and are called Knik Winds if they blow across Knik Glacier and Turnagain Winds if they blow across Turnagain Arm. The Turnagain Winds concentrate in valleys such as Eagle River, Peters Creek and Ship Creek. While velocities of 100 mph have been recorded at the west base of the Chugach Mountain valleys, these speeds diminish rapidly as the winds spread over the flat land and Knik Arm.

The tabulated data shown in Figure III-4 indicates a maximum measured speed of 51 mph during a 19 year period of record at the Elmendorf weather station. This data does not include gust speeds. Anchorage Weather Bureau experts indicate 70 to 80 knot (80 to 92 mph) gusts may be expected over Knik Arm due to lack of obstructions.

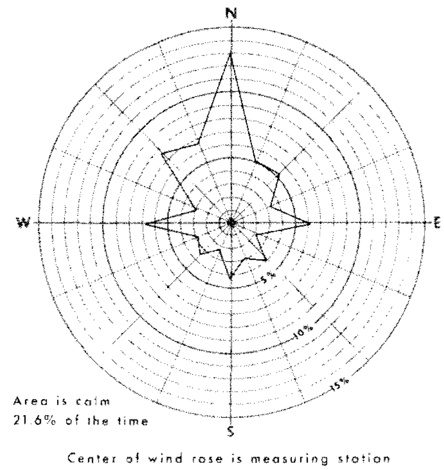
Figure III-4 also shows wind speed and percent of time in each quadrant. This data was plotted from Elmendorf AFB information which is closer to the area of interest than that collected by the Anchorage Station.

The previously mentioned high velocities of the northerly winds are not reflected in published data because surface roughness at the land stations prevents the speeds attainable over the smooth surface of Knik Arm and the Knik and Matanuska River beds.



Base Map is from U.S.G.S. Quadrangle Maps "Anchorage A-8 & B-8"

MEASURING STATION LOCATION



Center of wind rose is measuring station

PERCENTAGE OF TIME OF WIND ORIGIN

Month	Direction	Speed (knots)
Jan.	NNW	45
Feb.	NNW	41
Mar.	NW	35
Apr.	N	43
May	SSE	35
Jun.	SE	26
Jul.	E	30
Aug.	ESE	23
Sept.	SE	35
Oct.	SE	41
Nov.	NE	51
Dec.	SE	33

Record Period 1941 - 1959

ELMENDORF WIND VELOCITY EXTREMES

Figure III-4 WIND DIRECTION AND VELOCITY EXTREMES

Wind speed varies with height because of shear in the air mass. The law of wind change (Geiger, The Climate Near the Ground, pg. 118) in its simplest form is:

$$U = c \ln\left(\frac{Z}{Z_0}\right)$$

where U = wind speed in meters per second
Z = height above ground
Z_0 = surface roughness parameter
c = a constant which includes the Von Karman constant, temperature constant and unit constants.

The roughness parameter has a dimension of length and is listed by Geiger (pg. 275) as about 12 cm. for forests and about 0.02 cm. for sea, swampy plains and snow over short grass. The above formula shows that smoother surfaces (smaller Z_0) give higher speeds as do increased heights (larger Z).

The $\frac{1}{7}$ power law is often used to determine the variation of wind speed with height. The power law is stated by the following formula:

$$U_z = U_{30}\left(\frac{Z}{30}\right)^{\frac{1}{7}}$$

where U_z = wind speed at any height Z
U_{30} = wind speed at standard height of 30 feet above surface.

The gust factor is defined as that factor which must be used to multiply the sustained wind speed to obtain the gust speed (C.L. Bretschneider, The Handbook of Ocean and Underwater Engineering, 1969). This factor has been found to vary with elevation according to the formula:

$$F_z = F_{30}\left(\frac{Z}{30}\right)^{-\frac{5}{84}}$$

where F_z = gust factor at any height Z
F_{30} = gust factor at standard height of 30 feet above surface.

The U.S. Weather Bureau at Anchorage uses F_{30} equal to 1.3.

Figure III-5 has been plotted by using the Weather Bureau's maximum gust of 80 knots (92 mph) and reducing it by a gust factor of 1.3 to a steady wind speed of 70.8 mph at 30 feet above the surface. The $\frac{1}{7}$ power law has been used to form the steady speed curve between the heights of 3 feet and 300 feet and the gust speed curve has been formed using the gust formula shown above.

Effects besides property destruction may be attributed to the strong winds of the Anchorage Area. The Knik and Matanuska winds keep most of the snow blown off the flats above Knik Arm, providing much needed winter browse for the area's moose population. The Knik Wind also carries a great deal of silt from the Knik River bed and deposits it over Palmer and areas to the west. During the spring this silt settles on the snow and by decreasing the albedo (light reflectance) of the snow mass probably causes a slightly earlier snow melt than might otherwise be expected. Albedo is listed (Geiger, The Climate Near The Ground) as 75% to 95% for fresh snow cover, 40% to 70% for old snow cover, and 20% to 50% for dirty firn snow.

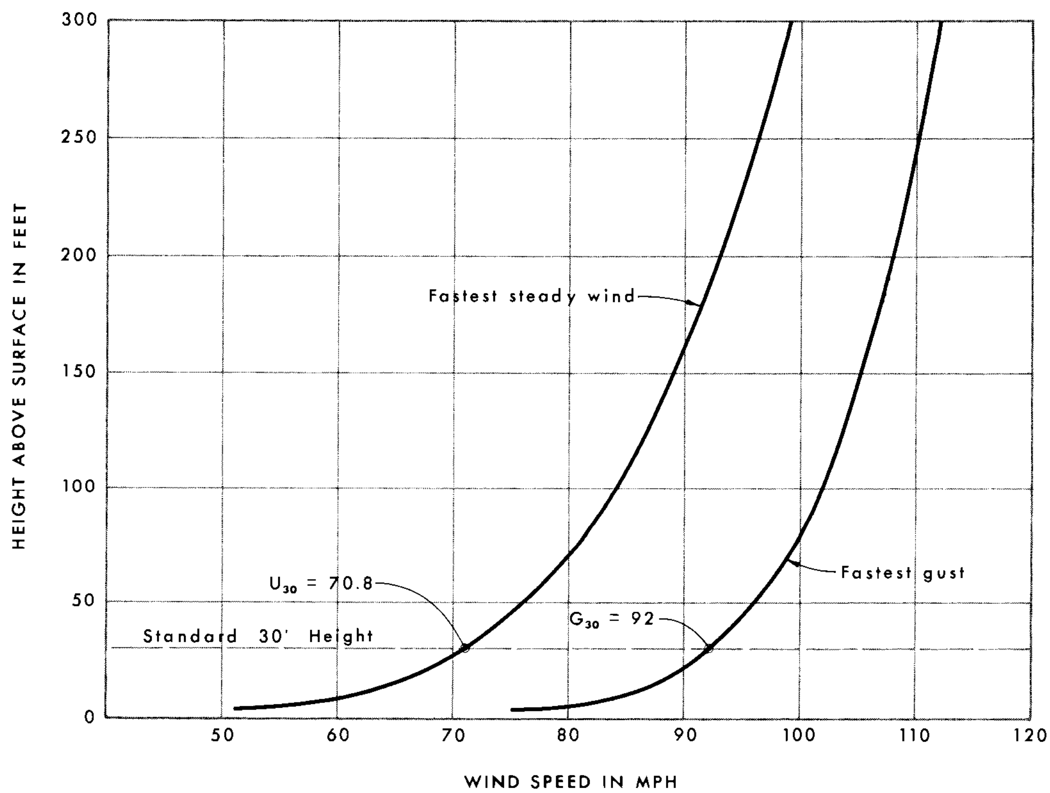


Figure III-5 WIND VELOCITY VS. HEIGHT

While extreme wind velocities will affect the design criteria, moderate winds will seriously decrease the efficiency of workmen during any winter construction in the Anchorage area. Wind chill is a factor which relates heat loss from human skin, caused by air movement, to an equivalent calm air temperature. Table III-2 shows the wind chill index as a function of temperature and wind velocity.

Table III-2 WIND CHILL INDEX

Wind Speed	Actual Air Temperature (Degrees Fahrenheit)												
(MPH)	25	20	15	10	5	0	-5	-10	-15	-20	-25	-30	-35
10	14	8	2	-4	-10	-15	-21	-27	-33	-39	-45	-50	-56
15	7	0	-6	-12	-18	-25	-31	-38	-44	-50	-57	-63	-69
20	2	-5	-12	-19	-25	-32	-39	-45	-52	-59	-66	-72	-79
25	-2	-9	-17	-24	-30	-37	-44	-51	-58	-65	-72	-78	-86
30	-5	-12	-20	-27	-33	-41	-48	-55	-63	-70	-77	-83	-91
35	-7	-14	-22	-29	-36	-44	-51	-58	-66	-73	-81	-87	-95
40	-9	-16	-24	-31	-38	-46	-53	-61	-69	-76	-84	-91	-98

Summary

While the Anchorage area has recorded weather extremes of serious proportion, the general weather does allow an extensive construction season. Steel erection has progressed on a high-rise hotel in downtown Anchorage throughout the winter season of 1970-1971 except for a few days interruption due to high winds. Earthmoving equipment can function as long as unfrozen soil is available, generally from May through October. Concrete freeze protection can be an item of considerable importance, since the months of June, July, and August are the only ones in which freezing temperatures need not be anticipated. Extensive daylight, as shown in Figure III-6 also aids the construction season.

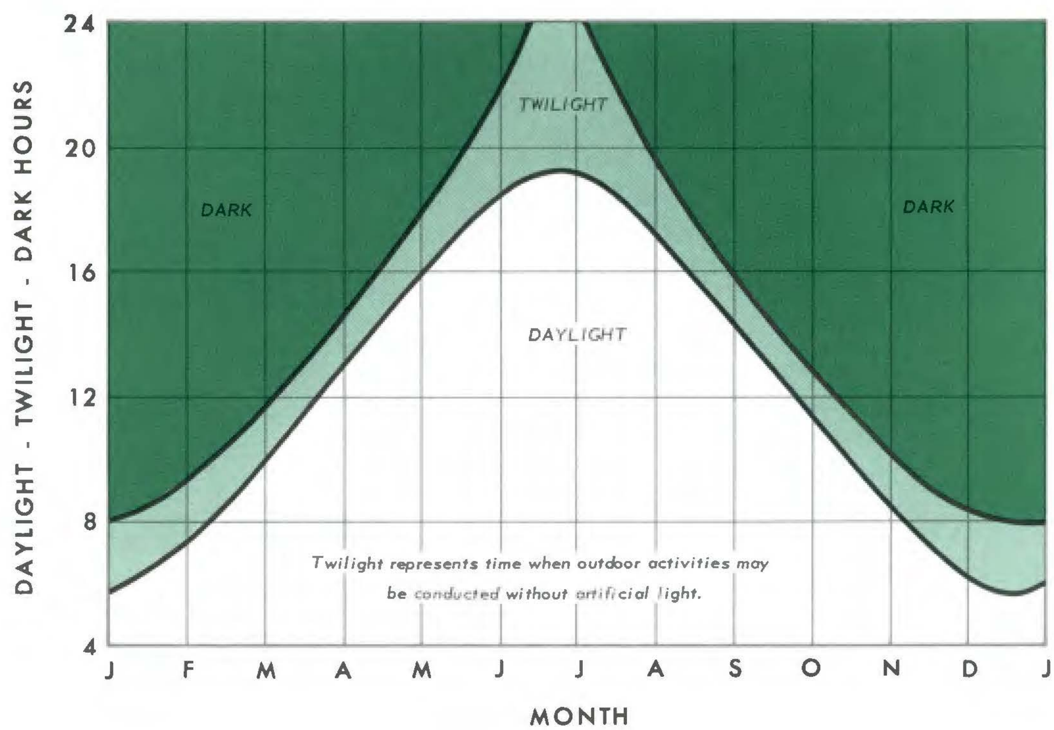


Figure III-6 DAYLIGHT HOURS

Design criteria should be based on the following extremes:

Wind velocity for structural design	100 MPH
Wind velocity for wave formation	60 MPH
Temperature limits for steel expansion	+120°F to -60°F
Median temperature	40°F

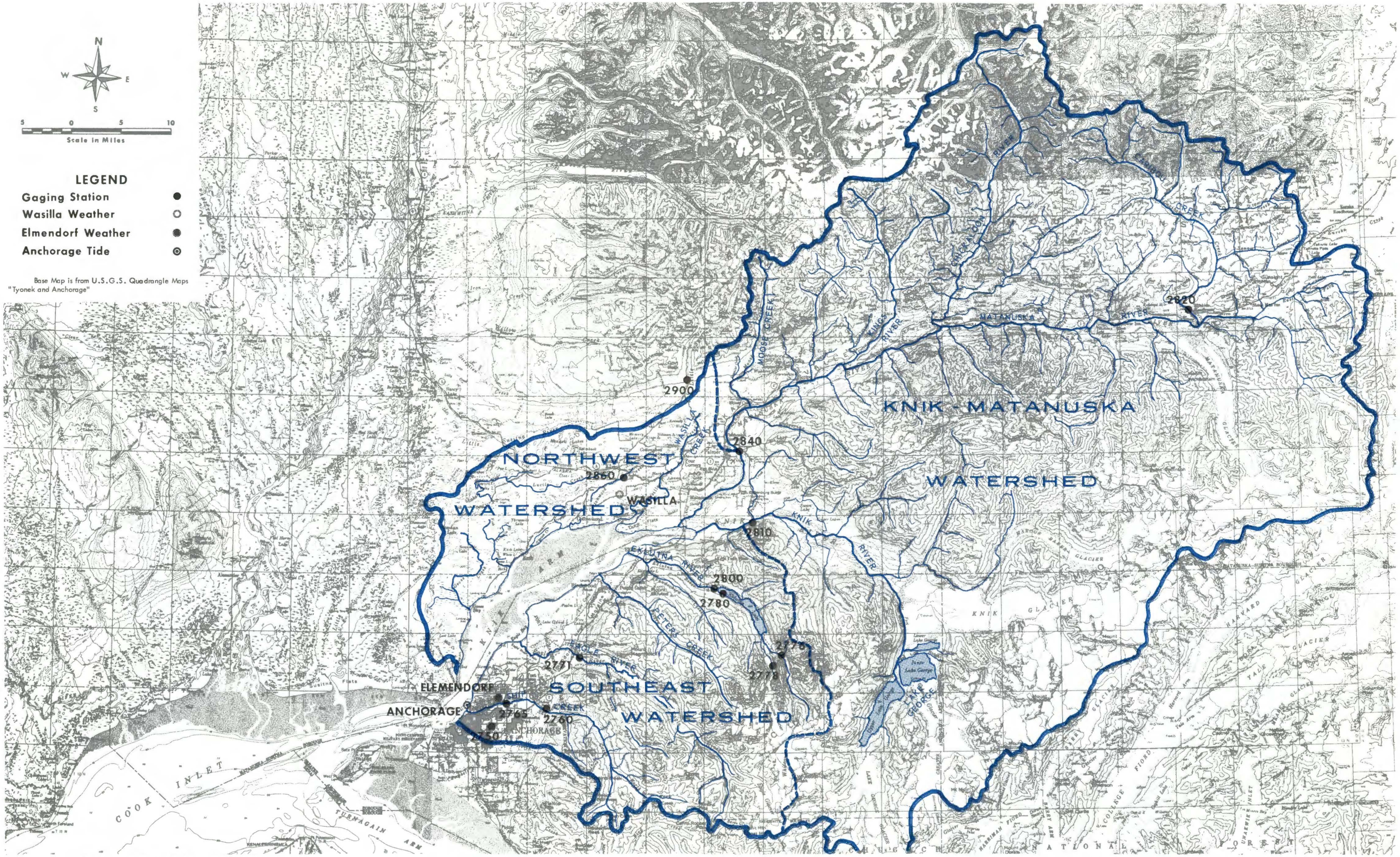


Figure III-7 KNIK ARM WATERSHED

FRESH WATER RUN-OFF

General

The purpose of this portion of the study is to review the extent of fresh water runoff from the Knik Arm watershed and the effect of this runoff upon the various crossing alternates. In general, the fresh water accumulation should have: (1) very little effect upon an open bridge type crossing, (2) a variable effect on a bridge-causeway type crossing, and (3) a major effect upon a causeway dam type of crossing.

Three sources of fresh water are available; runoff from normal precipitation (rainfall and snow), glacier melt water and ground water. The precipitation portion of the supply consists of the rainfall excess and snow melt over and above that required to fill soil interstices and surface depressions. The glacial melt portion of the supply is that furnished by normal cyclical warm periods which result in melting of glacier surfaces. The ground water portion consists of artesian springs and lakes.

Watershed Description

Knik Arm, north of Point Mackenzie, draws upon a total watershed of 4,570 square miles as shown in Figure III-7. The watershed consists of three main sub-watersheds which are identified herein as; the Knik-Matanuska, the Southeast, and the Northwest Watersheds. The Knik-Matanuska Watershed encompasses the major portion of the area (3,250 sq. mi.) and lies east of the upper end of Knik Arm along the Knik and Matanuska Rivers. The Southeast Watershed (725 sq. mi.) includes the west slope of the Chugach Mountains and the City of Anchorage with its environs and is drained by Ship and Peters Creeks, Eagle and Eklutna Rivers, and several small tributaries. The Northwest Watershed (385 sq. mi.) is drained by Goose, Fish, Cottonwood and Wasilla Creeks. The remainder of the area (210 sq. mi.) is occupied by Knik Arm proper and the large delta at the mouth of the Matanuska and Knik Rivers.

The Knik-Matanuska River Watershed and the Southeast Watershed have their origin in the Chugach and Talkeetna Mountains with a maximum altitude of approximately 13,000 feet. Above elevation 6,000, glaciers are common. Most of the 900 square miles of glacial area lies in the Knik River watershed rather than the Matanuska River watershed or the Southeast Watershed. The Knik-Matanuska Watershed also contains the Lake George impoundments and watershed. The Northwest Watershed is generally low, below elevation 400, and contains a high percentage of marshy areas and small lakes.

The orographic effect (the variation due to altitude) on precipitation is pronounced. The precipitation amounts at the higher altitudes, particularly in the Chugach Mountains, are considerably greater than those received at the lower elevations in the trough of the Knik Arm and the Knik–Matanuska Valley. The location of the Chugach Mountains, adjacent to the Prince Willaim Sound, also has a great effect upon precipitation amounts at the higher altitudes in that the moisture laden winds from the Gulf of Alaska are interrupted and most of the moisture is extracted prior to their arrival at the Knik Arm trough. Figure III-8 shows the extent of this variation as derived from data given in U.S. Weather Bureau Technical Papers 47 and 52.

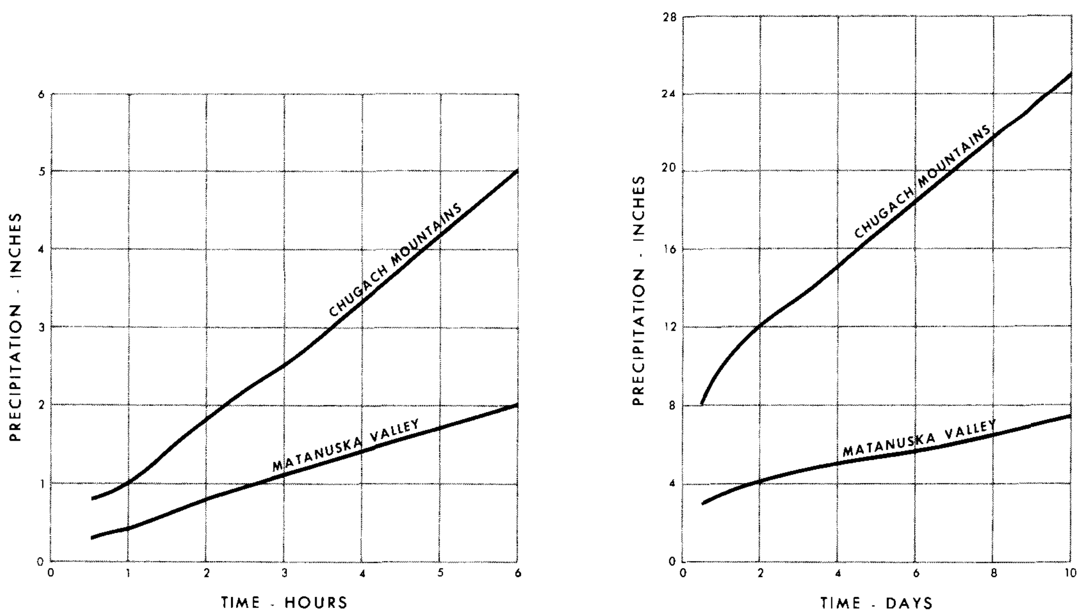


Figure III-8 100 YEAR PRECIPITATION DATA

Of the 4,570 square miles total watershed, the runoff from 3,724 square miles (81%) are recorded on stream gauges. Figure III-7 shows the location and Gauge No. for the watershed. These gauges are primarily U.S. Geological Survey gauges and the data is recorded in the Surface Water Records published annually by the U.S. Geological Survey. However, not all of this gauge data is considered usable. For instance, the dumping of Lake George into the Knik River makes all of the readings for the Knik River, and particularly peak discharge data, suspect in any analysis of stream produced peak discharges. On Eklutna Creek, the lake gauge data and the lower creek gauge data is of very little use due to diversion of water to the Knik

River from Eklutna Lake for power generation. The Ship Creek data is adversely affected by drawoff for the water supply to Fort Richardson. These three gauges represent 1,412 square miles of the 3,724 square miles of gauged area, leaving approximately 50% of the total watershed for the derivation of runoff data. This amount of data is considered sufficient for present purposes.

Stream Characteristics

Due to the terrain, the streams in the Matanuska–Knik River portion of the watershed and the southeast portion of the watershed all exhibit characteristics common to mountain streams. The flow rates are highly variable and some streams exhibit very little flow during the winter months. The small winter flows that do exist are apparently derived from ground water sources (USGS Professional Paper 544B, pg. B2, Waller, 1966). Since thunderstorms are not common to the area, 2 per year average, U.S. Weather Bureau, Technical Paper No. 47, there does not appear to be a great deal of storm generated flash type flooding on the streams.

The larger streams, particularly the Matanuska and Knik Rivers, show the heavy braided characteristics of glacial outwash streams. In these streams, the channels are continually shifting and it is to be expected that the major channel for one time period will not necessarily be the major channel for another time period. In the Northwest Watershed the streams exhibit the meandering characteristics common to flat coastal plain areas. Gauge data available for streams in this area indicates that flow in the streams is generally very low and more uniform than in the other portions of the Knik Arm Watershed.

Stream Discharge

Peak Discharge

Using the stream discharge records available, a study was made to determine the expected maximum rate of flow into Knik Arm from storms, snow melt, and other occurrences excluding major glacial lake breakouts such as those which occur at Lake George. The design of any spillway or bridge opening will need to be based upon a discharge value at least comparable to that represented by the summation of the record peaks on the individual streams. The effect of events such as the Lake George breakout will then need to be evaluated separately.

The gauge data used in this analysis includes both small and large gauged areas. The smallest gauged area (26 square miles) used was that of the West Fork of Eklutna Creek. The largest used was that of the Matanuska River, an area of 2,070 square miles. It should be noted that the Matanuska River data includes glacial melt water, but the data was used because it represents the best large gauged area available. The peak discharge rates vary from 1,470 cfs (cubic feet per second) at the West Fork Eklutna Creek gauge to 40,100 cfs at the Matanuska River Gauge. Figure III-9 shows a plot of peak discharge rates (reduced to a basis of cfs per square mile) vs. square miles of gauged area.

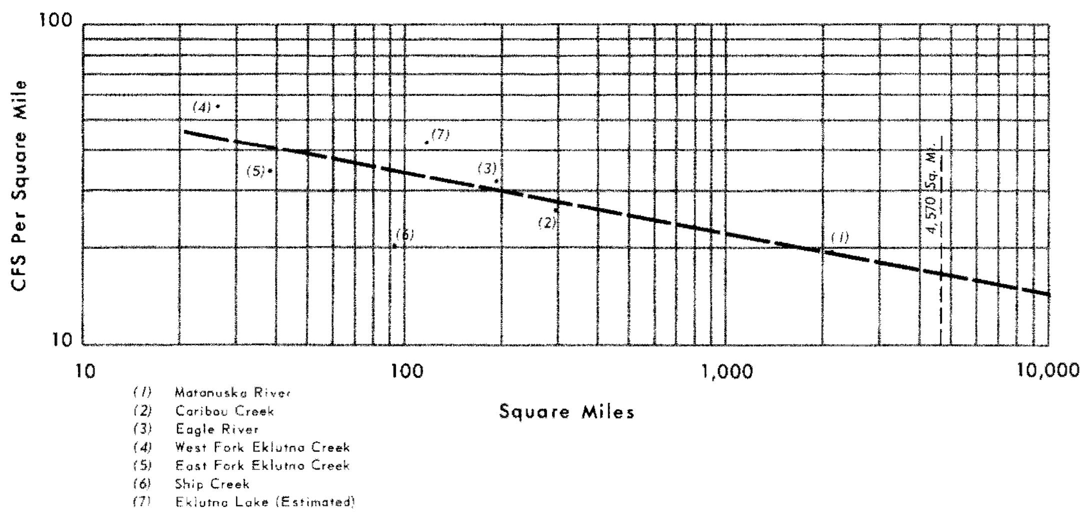


Figure III-9 PEAK STREAM DISCHARGES

Extrapolation on this plot indicates that for the total 4,570 square mile watershed a peak discharge rate of about 17 cfs per square mile can reasonably be expected. This translates to a peak discharge, based upon individual record peak discharges, of approximately 75,000 cfs into the whole of Knik Arm. This compares favorably with the value given for a 50-year flood in Figure III-6 of "Magnitude and Frequency of Floods in Alaska South of the Yukon River." As indicated earlier, this does not include the peak discharge produced by the breakout of Lake George.

The peak discharge developed in the Knik River by the breakout of Lake George appears to be independent of the peak discharges produced in the more normal streams of the area. The breakout of Lake George, which occurs when the water surface elevation in Lake George is high enough to force flow over Knik Glacier, is a function of the rate of movement of Knik Glacier and the rate of melting of Colony and Lake George Glaciers rather than of storm precipitation.

Maximum Probable Rainfall

For the Knik Arm watershed, the maximum probable rainfall for a 24-hour period varies from 10 inches in the Matanuska Valley to 19 inches in the Chugach Mountains. For a 6-hour period, it varies from 6.0 inches to 9.5 inches. (U. S. Weather Bureau, Technical Paper No. 47) Storms of this magnitude produce extremely large flood runoffs, but their use as a design parameter must be considered in order to protect downstream lives and properties.

Annual Discharge

In addition to the peak discharge which can be expected into Knik Arm, the total annual runoff into the Arm is also very important. This value could easily be the prime consideration in the feasibility of the construction of a causeway dam in that it could be the determining factor in the maintenance of adequate lake levels. The records of several of the gauging stations, notably the Knik, Matanuska and Eagle Rivers and Ship Creek, contain annual runoff data. A review of this data indicates that Knik Arm can reasonably expect to receive an average annual inflow of approximately 10 million acre feet.

Evaporation

From the fragmentary, Class A pan, evaporation data available (U.S. Weather Bureau, Technical Paper No. 13), an anticipated evaporation from lake surfaces of about 17 inches per year can be projected.

This evaporation will primarily occur in the summer months when the glacial melt rate is high and should be compensated by the glacial melt. For a 100 square mile lake, this rate of evaporation will require only 91,000 acre feet of replacement runoff. This amount is insignificant when compared to the 10 million acre feet annually available.

Lake George

Joseph M. Childers in "Flood Frequency in Alaska" reports: "Natural glacier-dammed lakes form in many places in Alaska, especially in the Pacific

Mountain System. These lakes occasionally break out causing spectacular floods that have very high peak-discharges." Lake George is such a lake and is created by Knik Glacier pressing against the base of Mount Palmer. It is located some 45 miles northeast of Anchorage and 25 miles southeast of Palmer.

The Lake George Breakout is erratic in that it does not occur every year with consistency and peak rates of flow vary considerably. Early reports, from Indians living in the area, indicated major floods occurred every 15 or 20 years. These floods were probably Lake George Breakouts. Modern records show yearly breakouts from 1948 to 1962, indicating an annual rate of occurrence. However, since 1962, only three breakouts have occurred (1964, 1965 and 1966).

Lake George is considered by some to be a scenic attraction suitable for designation as a national or state park (Progress Report, Lake George–Knik Glacier Investigation by R. E. Marsh, March 1961). An integral part of the Lake George attraction is the spectacular nature of the breakout.

These breakouts generally occur in mid-summer when the meltwater from Colony Glacier, Lake George Glacier and Knik Glacier is sufficient to overtop the damming effect of Knik Glacier. Draining of the lake generally requires about 5 days. Prediction of the time of breakout is not yet possible. The magnitude of the breakout, as illustrated by the rate of flow at the Glenn Highway Bridge (Gauge No. 2810) varies from 144,000 cfs (1966) to 359,000 cfs (1958) and 355,000 (1961) as indicated in Table III-3.

A two-year study of the Lake George phenomenon by the U.S. Geological Survey (Lake George–Knik Glacier Investigation, March, 1961) in 1959 and 1960 concluded that under certain conditions the approximate date of the Lake George breakout can be forecast two to three weeks in advance and that the flood peak will reach the railroad bridge some 8 to 10 hours after the peak occurs at the Lower Gorge. However, another report (Knik–Matanuska Rivers Near Palmer, Alaska, February, 1963) also concludes that magnitude of the peak at the railroad bridge cannot be accurately forecast, even though an estimate of 450,000 cfs is presented as a reasonable maximum design discharge at the railroad and highway bridges some seven miles downstream of Gauge No. 2810.

For the purpose of this study the 1961 breakout was utilized. This event was chosen because it presented the near maximum discharge (355,000 cfs) of record; the total volume of water was more than average but less

Table III-3

OBSERVATIONS OF PEAK FLOWS ON KNIK RIVER AT GLENN HIGHWAY BRIDGE*

Date	Water surface elevation above mean sea level (feet)	Discharge cfs
8-13-35	50.7	
8-20-36	50.5	
9-10-37	49.2	
9- 8-38	52.0	
8-17-39	50.5	
8- 6-40	49.7	
7-28-41	48.5	
8- 3-42	47.7	
7-24-43	48.5	
7-20-44	48.7	
8- 3-45	50.2	
7-28-46	48.9	
8- 1-47	50.0	
8- 5-48	50.06	198,000
8-22-49	49.88	193,000
8- 3-50	49.27	178,000
7-27-51	49.74	189,000
8- 8-52	50.17	200,000
7-23-53	52.20	262,000
7-26-54	52.33	260,000
8- 9-55	52.5	265,000
8- 8-56	52.35	260,000
7-18-57	54.7	333,000
7-18-58	55.5	359,000
7- 1-59	51.01	223,000
7-17-60	54.55	328,000
7-26-61	54.5	355,000
6-29-62	48.7	165,000
1963	No Breakout	
7- 1-64	50.2	216,000
7-11-65	51.6	236,000
6-24-66	47.8	144,000
1967-1971	No Breakout	

*1935-1961 Data from "Knik-Matanuska Rivers Near Palmer, Alaska" by U.S. Geological Survey, R.E. Marsh, District Engineer.

1962-1968 Data from "The Breakout of Alaska's Lake George" by U.S. Geological Survey

1969-1971 Data from local sources

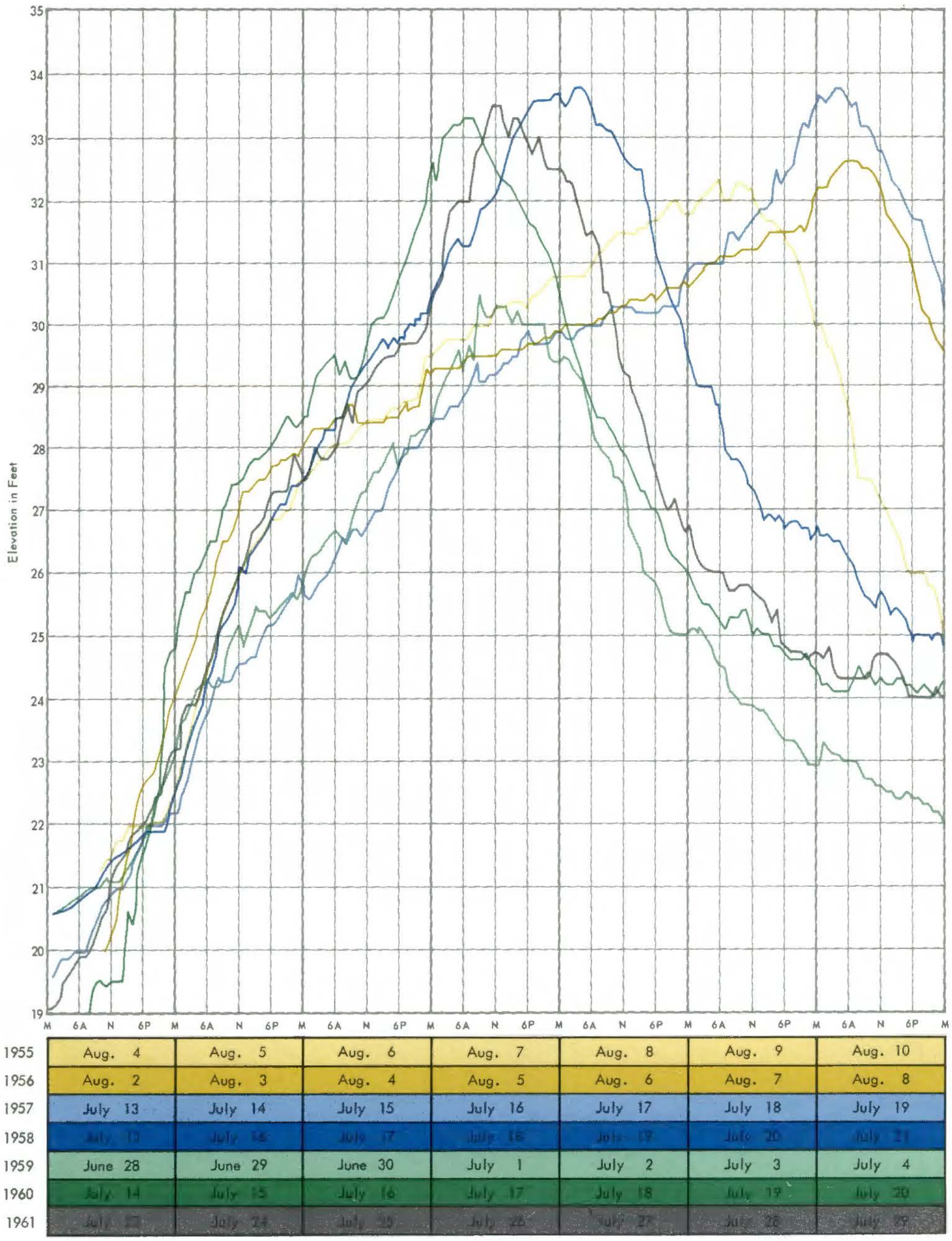
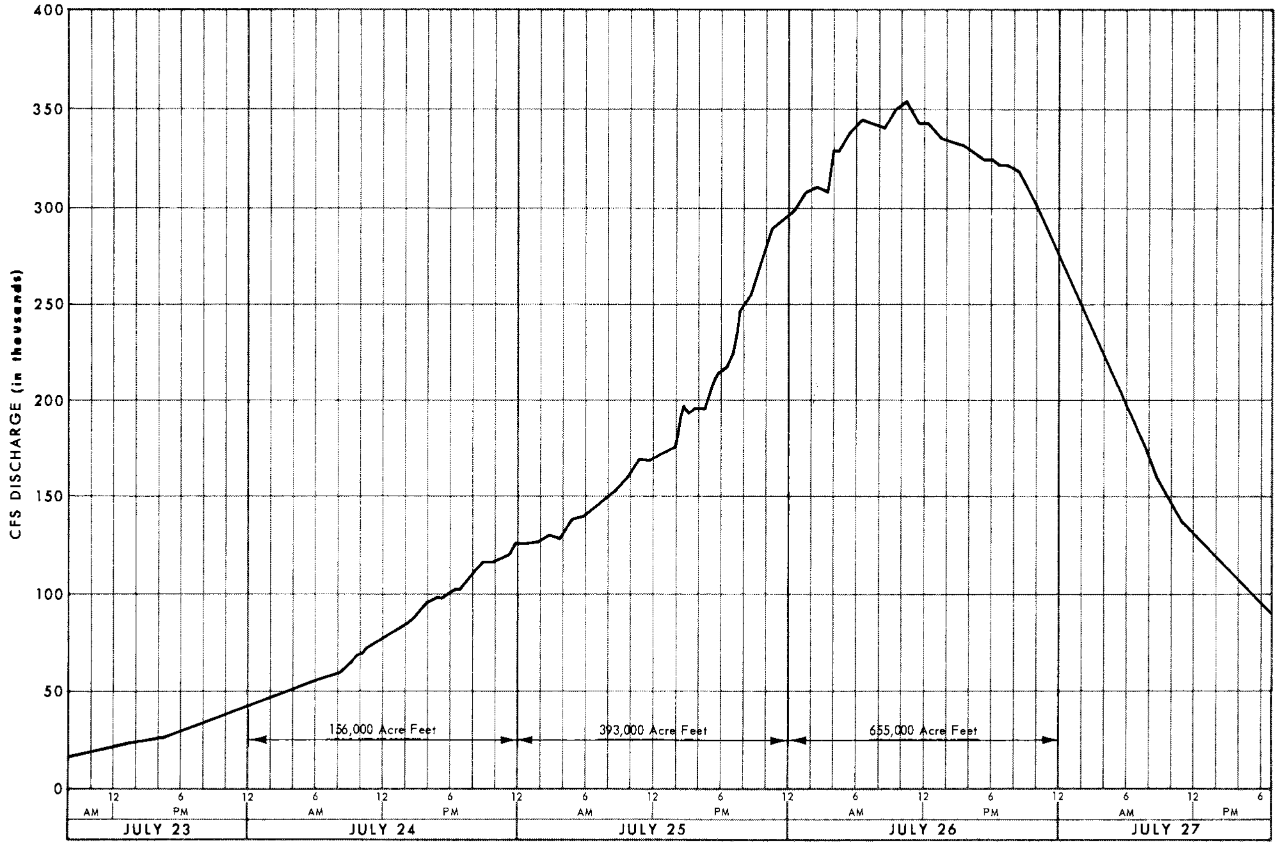


Figure III-10
LAKE GEORGE
BREAKOUTS

than maximum; hourly stage data was available, and gauge rating data was available. Also, the high discharges at Gauge No. 2810 from this breakout occurred over a period of 5 days instead of the more usual 10 to 15 days. The recorded discharges in the Knik River at the Gauge No. 2810 during the 1961 Lake George Breakout are shown on Figure III-10. In other studies



KNIK RIVER DISCHARGES 1961 LAKE GEORGE BREAKOUT

this data plot was used as the inflow hydrograph to Knik Arm. Also shown on Figure III-10 is a series of gauge reading plots based on data taken during Lake George Breakouts at the Eklutna Power Plant by personnel stationed at the plant.

The importance of Lake George to the proposed project is directly related to the sudden surges of rates of flow which greatly exceed normal storm generated runoff. If any causeway dam alternate is chosen, the flow generated during a Lake George breakout must be considered in the design of the spillway and its attendant facilities. If a bridge alternate is chosen, the effect will be less but the increased rates of flow must be taken into consideration in the design of pier footings, etc. In either case, the possibility of a Lake George Breakout occurring in the middle of the construction season will need to be considered by the contractor.

TIDAL HEIGHTS AND FLOWS

The following definitions of terms related to the study of tides, tidal flow and water levels are included to assure that all agencies, both design and review, use the terms in the same manner. The definitions were obtained from ASCE Manual No. 43, "Nomenclature for Hydraulics."

level, sea, mean – The mean plane about which the tide oscillates; the average height of the sea for all stages of the tide. At any particular place it is derived by averaging the hourly tide heights over a 19-yr period.

level, water, tidal – The altitude reached by a tidal surface.

MLLW: Mean lower low water – The mean level of lower low water over a long period.

tide – The periodic rising and falling of the water that results from the gravitational attraction of the moon and sun acting upon the rotating earth.

tide, ebb – A term indiscriminately used for falling tide or seaward current.

tide, flood – A term indiscriminately used for rising tide or landward current.

tide, neap – High water which is lower than usual; usually when the moon is in the first and third quarters. At these times the tidal forces of the sun and moon are opposed.

tide, range of – The difference in height between a high water and a preceding or following low water.

tide, spring – High water, higher than usual, at the times of full moon and new moon resulting from tidal forces of the sun and moon acting in the same direction.

prism, tidal – The total amount of water that flows into a tidal basin or out again with movement of the tide, excluding any fresh water flow.

water, slack – In tidal waters, the state of a tidal current when its velocity becomes a minimum ... The term is also applied to the entire period of low velocity near the time of the turning of the current.

General

The tides in the Cook Inlet–Anchorage area are notable for the extreme tidal ranges which occur. The funnel shape of Cook Inlet with its exposure to the deep Pacific Ocean waters permits the amplification of tides causing tidal ranges in the vicinity of Anchorage to rank with the largest in the world. Up Knik Arm, above Anchorage, the limited data available indicates that the water surface elevation at the time of peak tide remains relatively level but that the tide range decreases from that noted at the Anchorage gauge. The photo below illustrates one effect of this large tidal range on a manmade structure.

POL Dock, Anchorage –
Note ice build-up on piling and slush build-up on ground behind dock (to left of photo).

Tidal studies for the design phase of a project such as the proposed highway crossing of Knik Arm should include the statistical analysis of a large number of actual tide cycles. Field data must be obtained by gauge or measurement at or near the project site. Where a large variation in tide heights and range is normal, as it is in Knik Arm, the use of data from the project site, as contrasted to interpolated data, takes on increased importance. For a feasibility study of this type, the use of available data is mandatory even though the data may not be as extensive or complete as desired. Considerable data is available for the Anchorage gauge but little data is available for the middle and upper portions of Knik Arm.

Therefore, due to the lack of certian key tide data, the limited amount of time available for this study, and the uncertainty of project location, data from several specific time periods was chosen as typical and used in the studies.

The time periods and data sources selected as representative of tide conditions are as follows:

1) "Tide Tables" – 1960 to 1971 – Published by the U.S. Coast and Geodetic Survey.

2) March & September, 1970 – Hourly data for the Anchorage gauge from U.S. Coast and Geodetic Survey.

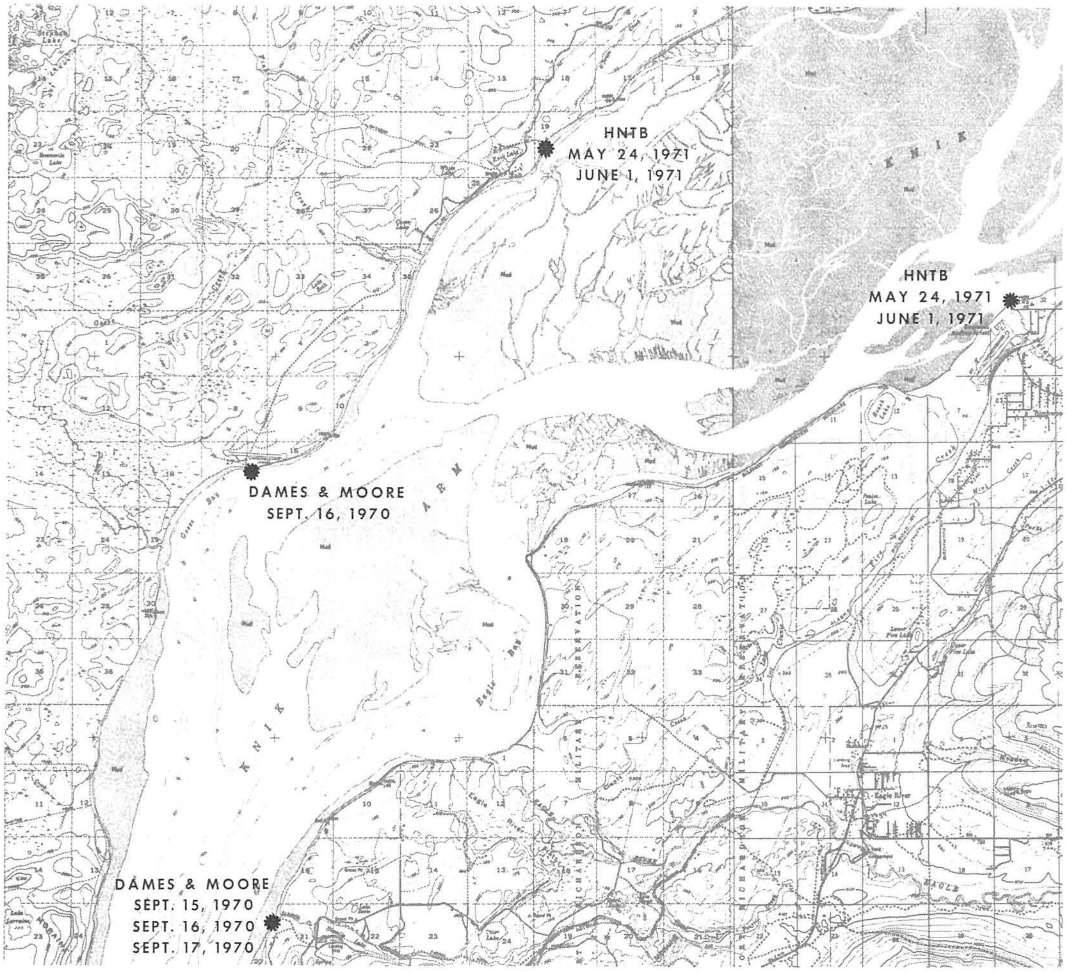
3) 1964 to 1970 – Highest and Lowest Tides as recorded on the Anchorage gauge by U.S. Coastal and Geodetic Survey.

4) September 15-17, 1970 – Dames and Moore high tide data.

5) May 24 & 25, June 1 & 2, 1971 – Field survey at two locations.

6) May 24 & 25, June 1 & 2, 1971 – Anchorage tide gauge data.

Items 1, 2 and 3 above are data regularly obtained by U. S. Coast and Geodetic Survey and are available on a continuing basis for the tide gauge at Anchorage. Item 4 data was obtained by Dames and Moore as a part of the September, 1970 boomer survey of Knik Arm. Item 5 data was obtained by joint HNTB–Alaska Department of Highways survey crews to extend the lateral range of data available and to allow a direct comparison for spring (May 24 and 25) and neap (June 1 and 2) tide elevations with Item 6, Anchorage tide gauge data. See Figure III-11.



Topographic data on this figure is from U.S.G.S. Quadrangle Maps.

Figure III-11 TIDE MEASUREMENT LOCATIONS

Elevation Datums

The determination of a fixed elevation datum in an area closely associated with tide and sea level fluctuations is a difficult matter in that local Mean Sea Levels are constantly fluctuating in relation to time. This fluctuation may be slight, but difficulties are encountered when attempts are made to establish datum elevations by different groups or interests at different times. In 1929 the United States Coast & Geodetic Survey established a sea level datum plane for the United States by averaging the mean sea levels at a number of points on the United States and Canadian coasts. This plane is referred to as the 1929 Sea Level Datum.

The 1929 Sea Level Datum is used in the Anchorage area in the establishment of horizontal controls. In 1964 and 1965 a Sea Level Datum based upon sea level readings at Seward, Whittier, Homer and Valdez was established and the net extended to the Anchorage area. The tidal bench marks in the Anchorage area are tied to Mean Lower Low Water at the Anchorage tidal gauge. On May 15, 1970, Mean Sea Level at Anchorage was calculated to be 16.43' above Mean Lower Low Water. (Wharton, USC&GS, Interoffice Memorandum, May 15, 1970). On October 28, 1971, Mean Sea Level at Anchorage was calculated to be 16.36' above Mean Lower Low Water. (Thurlouw, USC&GS, by phone, October 28, 1971). A recent (1968) tie between the SWHV net and a local tidal bench mark indicates that the SWHV net is 16.84' above Mean Lower Low Water (Henderson, ESSA, by phone, November 5, 1971) or 0.41' above the May, 1970 Anchorage Mean Sea Level and 0.48' above the October, 1971 Anchorage Mean Sea Level. In addition to the recent values listed above, a value of 15.84' for the relationship between Anchorage Mean Sea Level and Mean Lower Low Water was uncovered. The exact date and source of this value was not obtainable. Also, a value of 15.25' was given for the relationship between Mean Tide Level and Mean Lower Low Water at Anchorage for the time period August, 1964 to December, 1968 (USC&GS Tidal Bench Marks, April 7, 1970).

As is the normal case, the Anchorage tide gauge has been arbitrarily set so that the zero point on the gauge lies below any expected extreme low water in order that the extreme low condition can be measured. Consequently, Anchorage Mean Lower Low Water is not registered as 0.0 feet on the gauge. During the time period encompassed by the tide data used in this study, Anchorage Mean Lower Low Water was equivalent to a reading of +6.7' on the gauge.

Table III-4 summarizes the major elevation datum conversions used in this study:

Table III-4 ELEVATION CONVERSION FACTORS

	Tide Gauge	MLLW	MSL
Tide Gauge Zero	0	- 6.7'	-22.7'
Mean Lower Low Water	+ 6.7'	0	-16.0'
Mean Sea Level	+22.7'	+16.0'	0

In a study of this type, i.e., a study to determine the feasibility of some project, a minor difference between local datum and some theoretical general datum is relatively unimportant. The maps and sounding charts available for this study vary in accuracy and age to a large degree which causes this relationship to be of greatly reduced value at this time. At the time of further studies or construction, the relationships will vary from the current values.

Also, at the time that a specific location is agreed upon, it is recommended that permanent bench marks be established on either side of Knik Arm for use in establishing a local datum for use in all measurements and readings connected with the project. These bench marks can then be related to another elevation datum whenever it is deemed necessary or desirable.

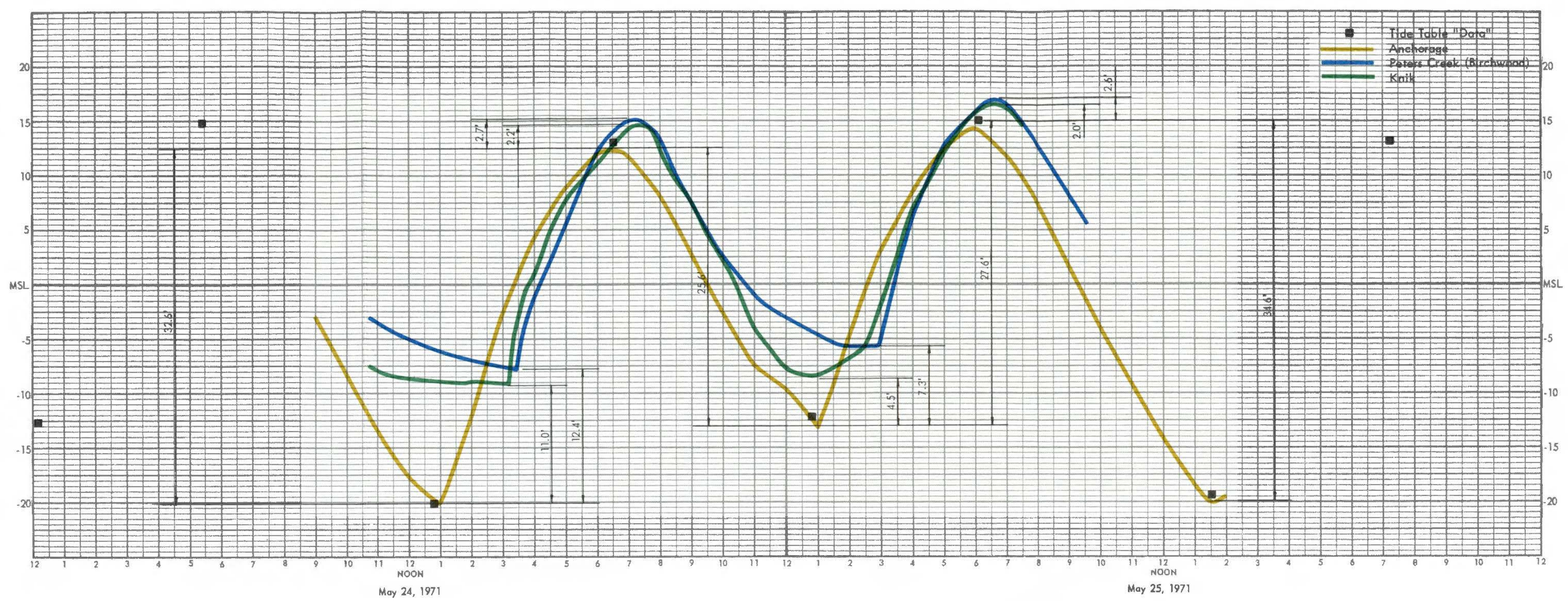
Tidal Ranges

Using 1970 as a "typical" year and "Tide Tables" data, an analysis was made of these predicted tidal ranges to determine the magnitude of water surface elevation fluctuations which could be anticipated at Anchorage and in Knik Arm. The number of occurrences in each selected range for 1970 are as follows in Table III-5.

Table III-5 TIDAL RANGES AT ANCHORAGE

Tide Range	Number Within Range			% of Total
	Flood Tide	Ebb Tide	Total	
< 5	0	0	0	0
5'-10'	3	0	3	0.2%
10'-15'	31	53	84	6.0%
15'-20'	132	113	245	17.5%
20'-25'	219	226	445	31.7%
25'-30'	229	186	415	29.6%
30'-35'	88	112	200	14.3%
35'-40'	3	15	18	1.3%
> 40'	0	0	0	0

From this it can be readily seen that at the Anchorage gauge, and presumably up Knik Arm, more than 60% of the tides will be in the 20 foot to 30 foot range. A review of the May 24 and 25, 1971 tidal range (See Figure III-12) indicates that toward the upper end of that 20' to 30' range (27.6' at Anchorage for the case in point) the range has been reduced by 4.7' on both the flood and ebb tides at Peters Creek, some 22 miles upstream of Anchorage. The June 1 and 2, 1971, data (Figure III-12) indicates less variation between Anchorage and Peters Creek for lower (20.6' and 20.8' at Anchorage) tidal ranges.



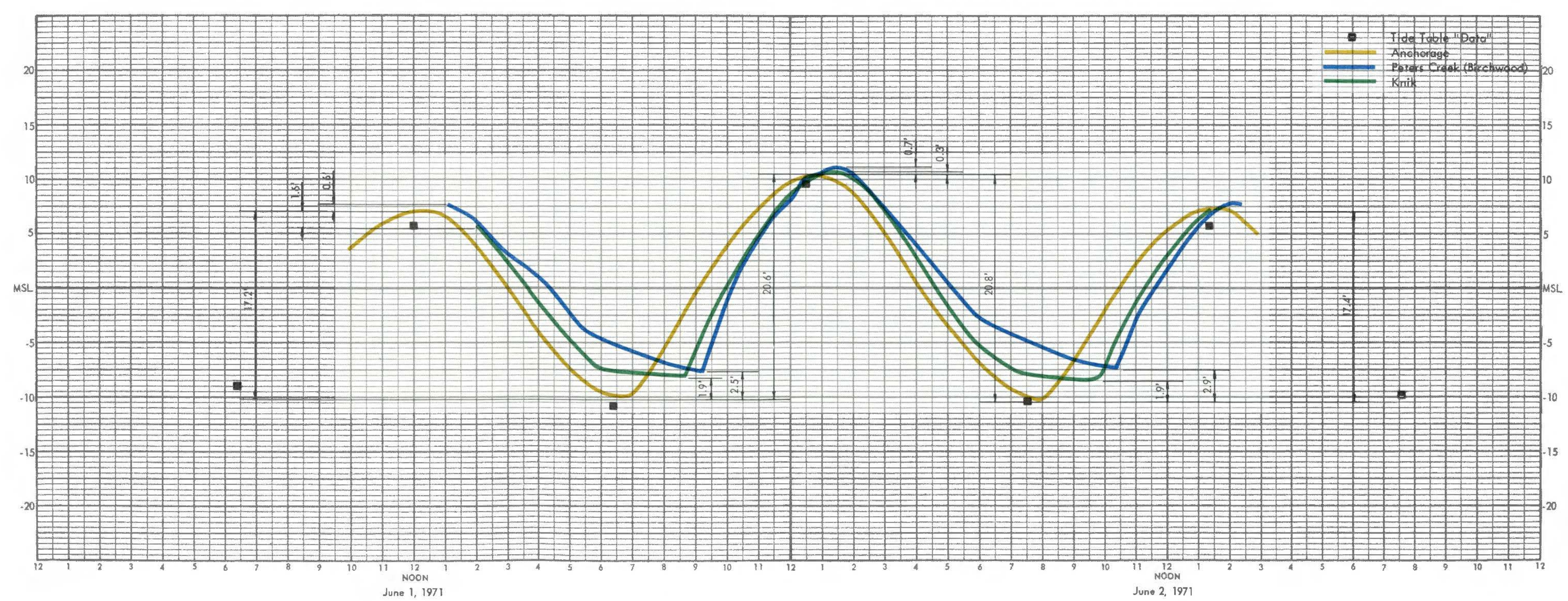


Figure III-12 COMPARISON OF TIDE DATA

For the purposes of this study, the maximum tidal range to be considered was derived from the 1964 to 1970 highest and lowest tides data supplied by the U.S. Coast and Geodetic Survey. April, 1967, had a tidal range extending from 16.7 MSL to −22.6 MSL, for a total range of 39.3 feet. It is not known whether these extremes were concurrent but since they occurred during the same month, they form a reasonable basis for a maximum tidal range.

Tidal Elevations

The months of March and September, 1970 were chosen as characteristic examples of tide variations, and the hourly gauge data was obtained for the Anchorage gauge. Table III-6 is a review of these readings, which do not consider wind or scour effect, which gives data relative to the percent of time that the water surface elevation at Anchorage was above a given elevation:

Table III-6 WATER SURFACE ELEVATION VS. TIME

	% of Time Above Elevation	
Elevation (MSL)	March, 1970	Sept., 1970
-20	100%	100%
-18	100%	99.9%
-16	99.0%	98.8%
-14	95.7%	96.0%
-12	90.5%	89.3%
-10	84.7%	83.3%
- 8	78.9%	77.3%
- 6	72.8%	70.8%
- 4	65.9%	64.1%
- 2	59.9%	58.0%
0	52.5%	52.0%
2	46.1%	45.7%
4	39.4%	38.9%
6	32.0%	30.8%
8	23.0%	21.0%
10	13.9%	13.1%
12	7.6%	6.3%
14	2.6%	1.5%
16	0.3%	0.1%
18	0%	0%

Because this analysis applies at the location of the Anchorage gauge, a question arises as to the applicability of the data, i.e., the elevation–time relationships, to the portion of Knik Arm above Anchorage. Consequently, a study was made of the elevation of the peak tide water surface at several upstream points relative to the peak tide water surface elevation at Anchorage. As indicated on Figure III-13, the upstream peak tide water surface elevations do not vary greatly from the Anchorage elevations within the limits of the study area. At Location IV and V a one-foot rise in peak tide water surface elevations is indicated. On a body of water of this size; with the tidal variations present, and the limited extent of the base data, this amount of variation is insignificant. Therefore, until more precise data is available, the time–elevation relationships for Anchorage have been assumed to hold for the entire study area.

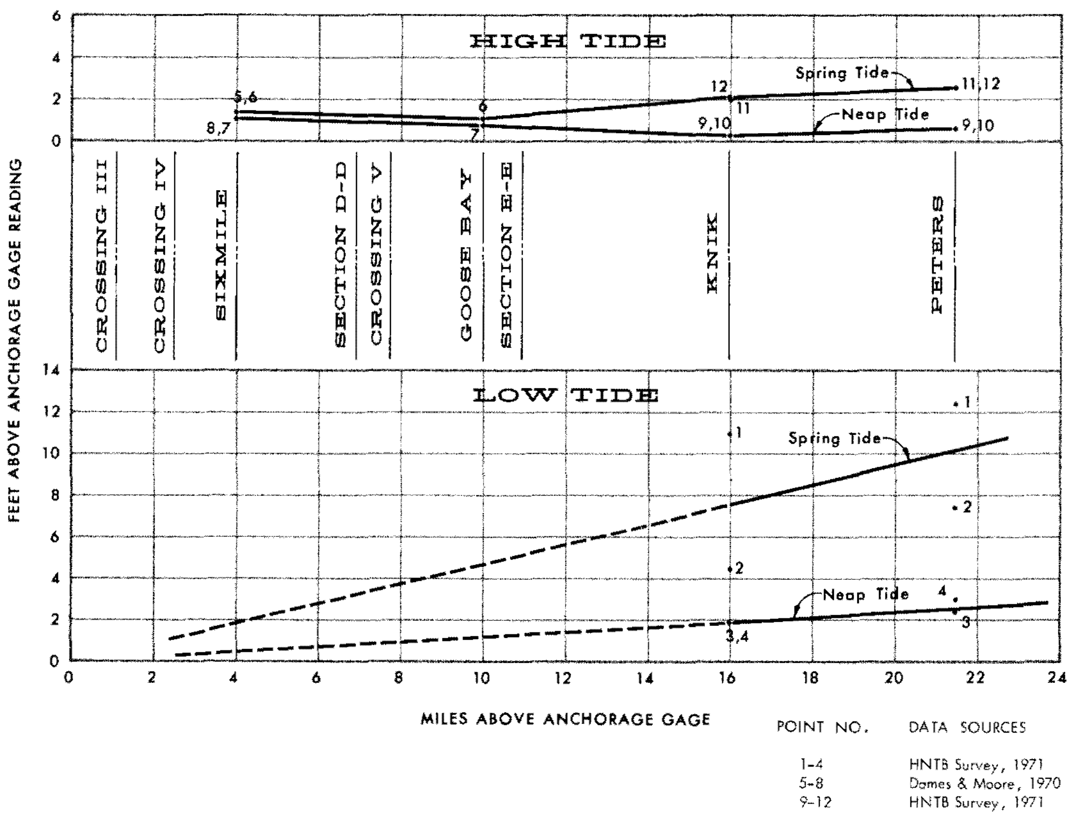


Figure III-13 TIDE ELEVATIONS IN RESPECT TO ANCHORAGE

The period of slack water is very important to construction operations. The movement and mooring of barges, the placement of marker buoys and other operations will probably be timed to coincide with slack water periods.

On Figure III-12 the slack water periods are indicated by the slope of the various lines. As the lines approach zero slope, slack water periods occur. From this it can be seen that peak tide slack water periods will be short, one hour or less, and that low tide slack periods may extend to about three hours at some locations.

Tidal Prisms

The tidal prism is another measure of the severity of the tidal cycle to be encountered. This is the total volume of water which must be considered, contained or controlled during and after the construction of any crossing of Knik Arm. Because the prism is a function of tide range, tide elevations and surface area upstream of the location being studied, the tidal prism varies considerably within the study area. Table III-7 shows the extent of this variation.

Table III-7 TIDAL PRISMS

(1×10^6 Acre Feet)

Crossing	Neap Tide (June 1&2, 1971)	Spring Tide (May 24&25, 1971)	Extreme Tide (April, 1967)
II	1.17	1.62	1.95
III	1.13	1.62	1.95
IV	1.10	1.55	1.88
V	0.90	1.18	1.43

In the derivation of the prisms shown, water surface elevations at high and low tide were adjusted to conform with surveyed data. Topographic data available was minimal but adequate for the purposes of this study.

Using similar procedures differential tidal prisms were calculated for each hour of the May 24, 1971, flood and subsequent ebb tide.

From a review of this data it can readily be noted that the maximum hourly volume of flow can be expected to occur during the second hour of the cycle (flood or ebb) and that approximately 50% of the total prism occurs in the second and third hour periods. A similar analysis of other tide cycles could give a variance in results but it was felt that the measured tide cycle (May 24 & 25, 1971) was an "average" spring tide and that therefore the results from this analysis would be applicable over a wide range of tides.

Table III-8 DIFFERENTIAL TIDAL PRISMS

May 24, 1971

Time (Hours)	CROSSING						
	II & III		IV		V		
	Diff. Ω (A.F.)	% of total	Diff. Ω (A.F.)	% of total	Diff. Ω (A.F.)	% of total	
1300							
	226,800	14.0	213,900	13.8	133,300	11.3	F
1400							L
	434,200	26.8	423,100	27.3	304,400	25.8	O
1500							O
	385,600	23.8	361,200	23.3	290,300	24.6	D
1600							
	312,700	19.3	294,500	19.0	241,900	20.5	T
1700							I
	234,900	14.5	227,800	14.7	186,400	15.8	D
1800							E
	25,900	1.6	29,500	1.9	23,600	2.0	
1815							
1855							
	31,300	2.1	29,500	2.2	24,400	2.3	
1900							
	270,200	18.6	253,700	18.7	209,700	19.5	E
2000							B
	366,300	25.2	337,000	24.9	278,600	25.9	B
2100							
	318,000	21.8	290,300	21.4	233,600	21.7	T
2200							I
	254,100	17.4	231,700	17.1	176,000	16.3	D
2300							E
	82,800	5.7	80,400	5.9	59,500	5.5	
2400							
	133,300	9.2	132,300	9.8	94,800	8.8	

As noted, the above results are based upon the study of only **one** measured tide cycle and that cycle was not measured at the location of a specific crossing. Also, this study did not consider the possible effects upon the tide cycle of seismic events, Lake George Breakouts, ice obstruction, spring floods, wind and scour effect, or final causeway construction. For more advanced studies, additional tide range and elevation data and more detailed mapping of the upper portion of Knik Arm will be required. Since the results of a study of this type will influence design, construction and possibly maintenance of the selected structure, data collection should begin as soon as possible after a decision to proceed is made. The extent of field measurements of tides and topographic features of the Knik Arm estuary will depend on the type of proposed structure.

TIDAL CURRENTS

The high tidal fluctuations of Cook Inlet give rise to high current velocities in certain localities of Knik Arm. Tidal bores are created at times in the Arm and were observed at Peters Creek and Goose Creek at the time of tide measurements by HNTB Personnel.

Due to the high current velocities good judgment and skill are required for safe navigation of the Arm. This fact is noted in most of the Hydrographic Survey Descriptive Reports.

Actual current velocity measurements were not undertaken for this study as several measurements and observations at various localities in Knik Arm have been made by others. The sites at which velocity measurements or observations have been made are shown on Figure III-14. Using the differential tidal prism data average, hourly velocities at Crossings II, III, IV and V were computed. See Figure III-16.

The maximum velocity recorded was 11.8 feet per second by the steamer Explorer during the 1914 Hydrographic Survey and occurred along the north shore just above Goose Creek.

The direction of the current is approximately parallel to the nearest shoreline as is evident in the study of ice floes.

Current velocities in the range of 7 to 9 feet per second have been recorded from Cairn Point to Goose Bay and Eagle Bay at varying water depths, near and far from shoreline, through total tide cycles, and on various dates. See Figure III-15. From a review of this data it is possible to conclude that the current velocity is not only a function of the tidal range and phase but is highly influenced by local shore configuration, bottom geometry and possibly wind effects in shallow areas. The effect upon the computed velocities

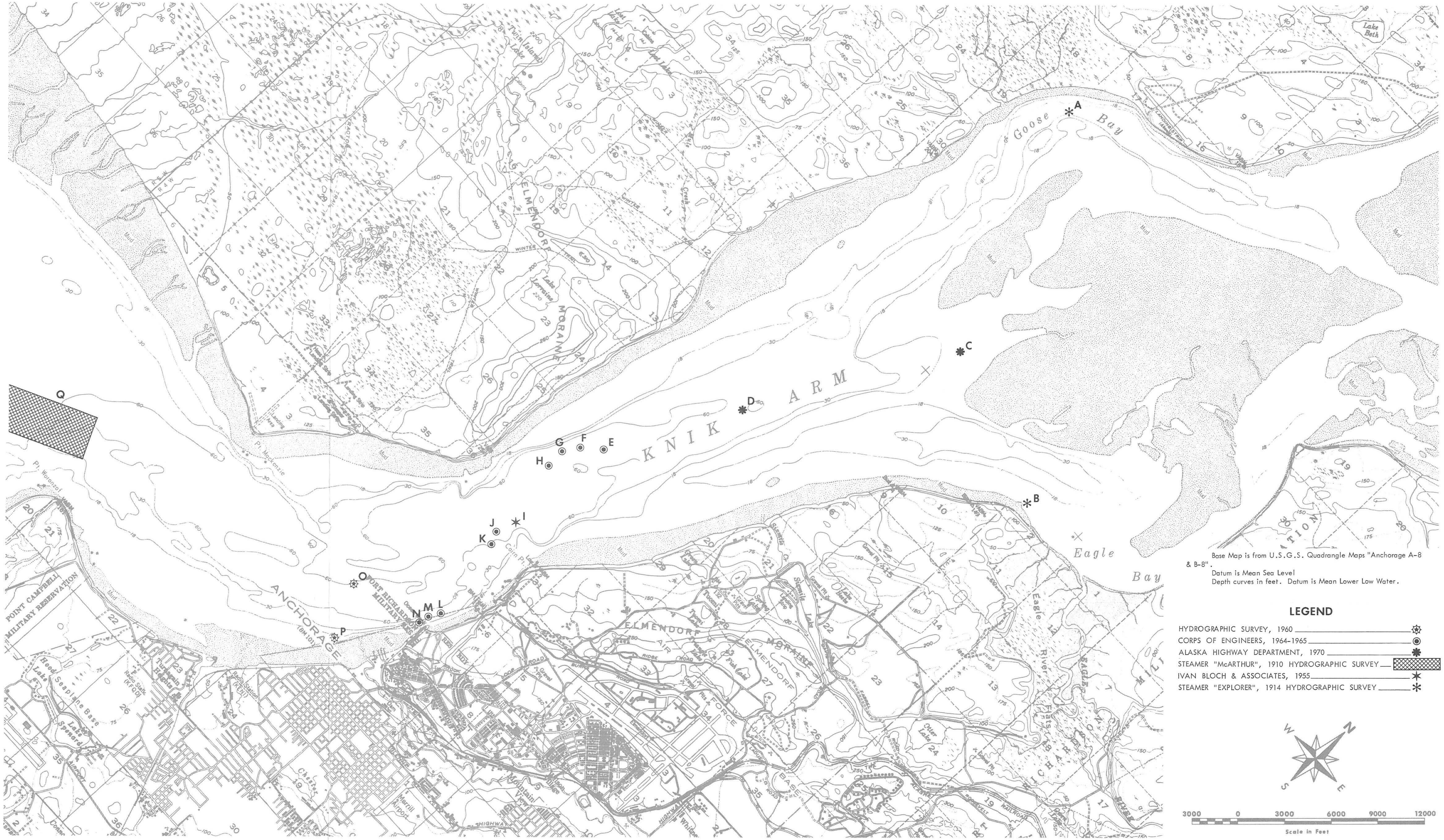


Figure III-14

CURRENT VELOCITY MEASUREMENT LOCATIONS

shown in Figure III-16 in Goose Bay and Eagle Bay by the construction of a combination Bridge–Causeway at Crossing V is shown on Figure III-17. This latter crossing alternate is discussed in Section VIII and Figure III-17 assumes that some scour will occur in the channels.

Bottom current velocities of 2-3 feet per second can be estimated from the formation of sand bottom waves in the mud flats. The estimated velocities compare favorably with the velocities of 2.2 to 2.6 feet per second measured near Crossing 0 for the Beluga Power Project (Route Selection Report – Submarine Cable, Retherford). Similar results were obtained by the Alaska Department of Highways (Dec., 1970 Report).

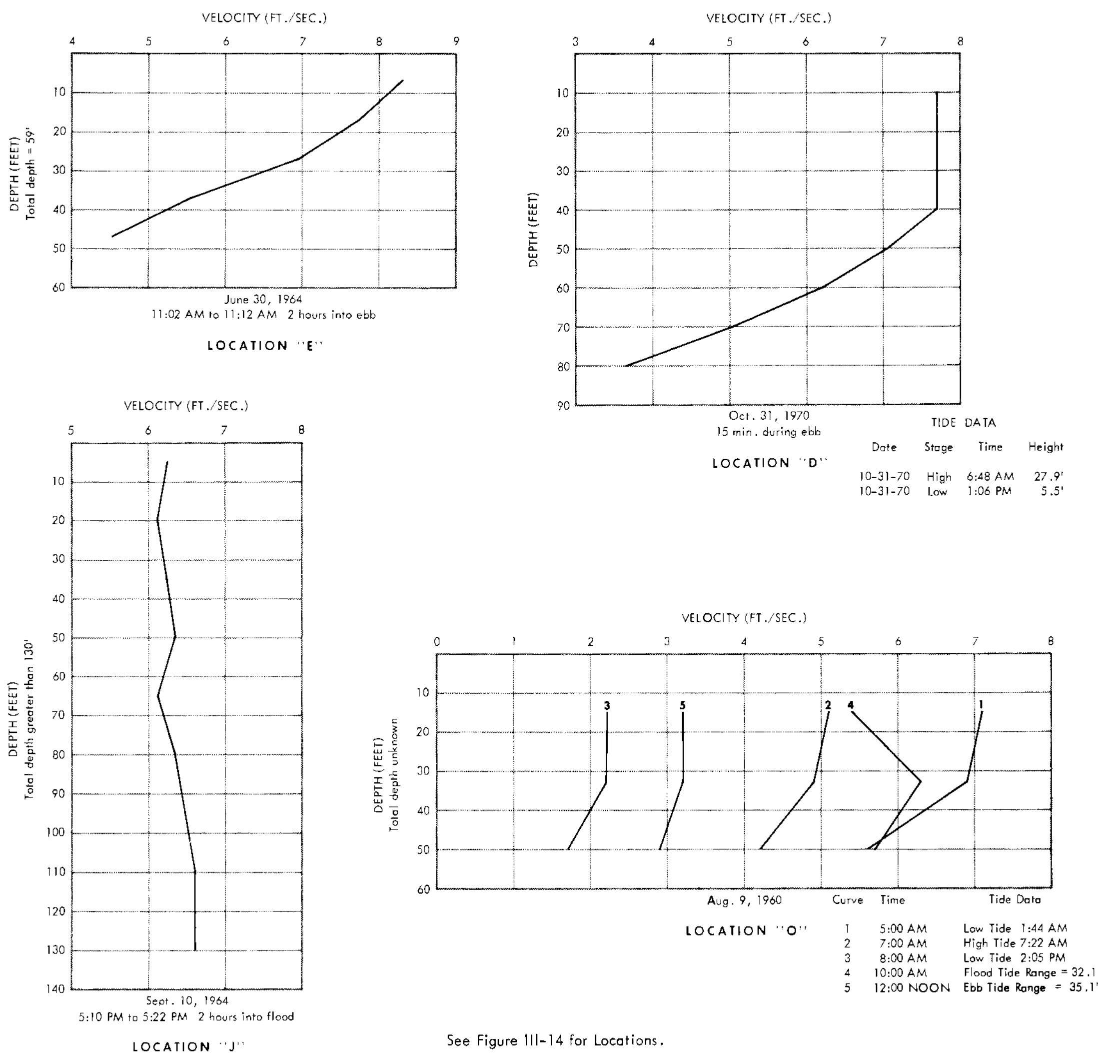


Figure III-15 CURRENT VELOCITY PLOTS

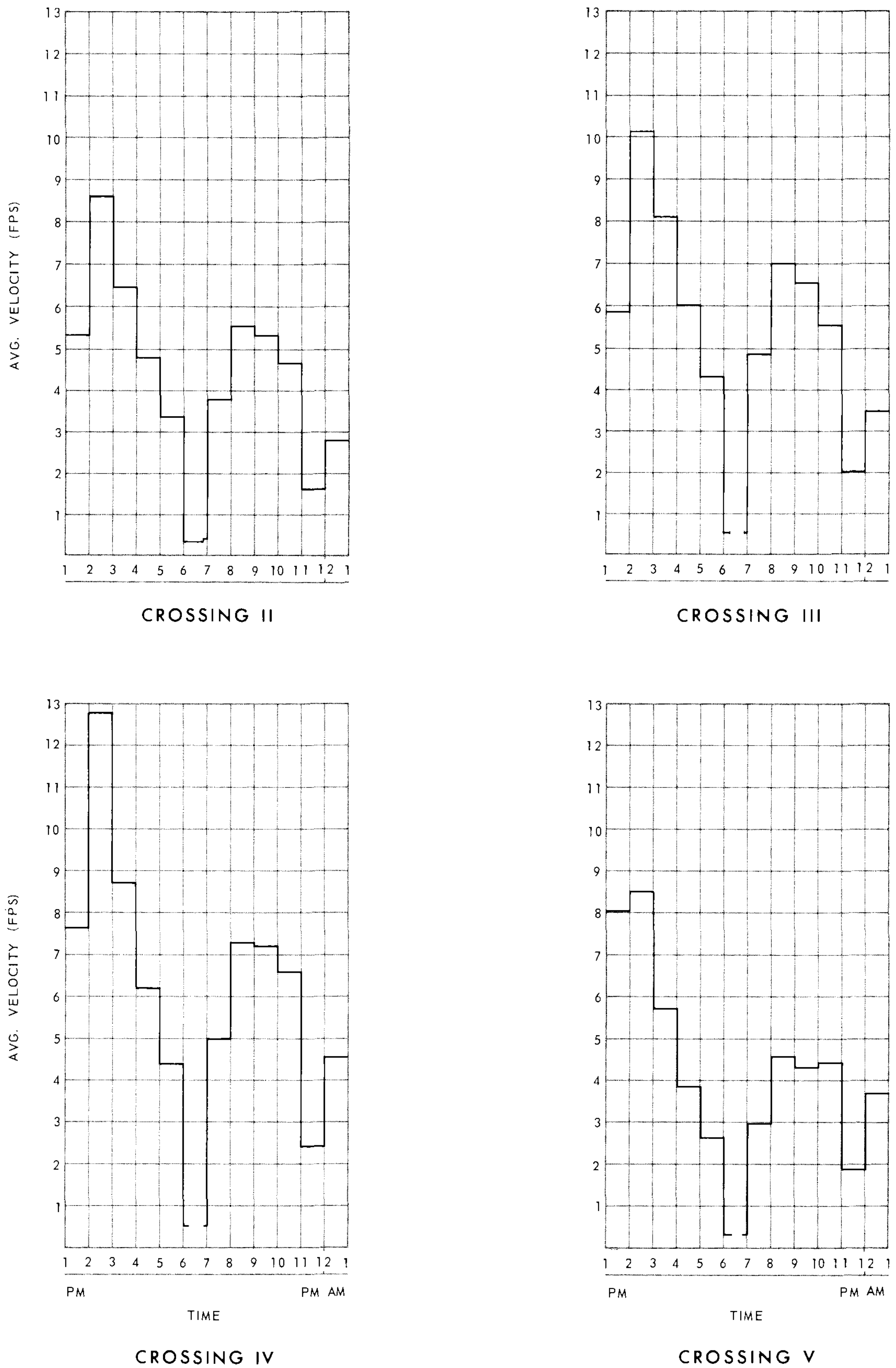


Figure III-16 CALCULATED AVERAGE VELOCITY AT CROSSINGS II, III, IV & V

(Based on May 24, 1971 calculated differential tidal prisms)

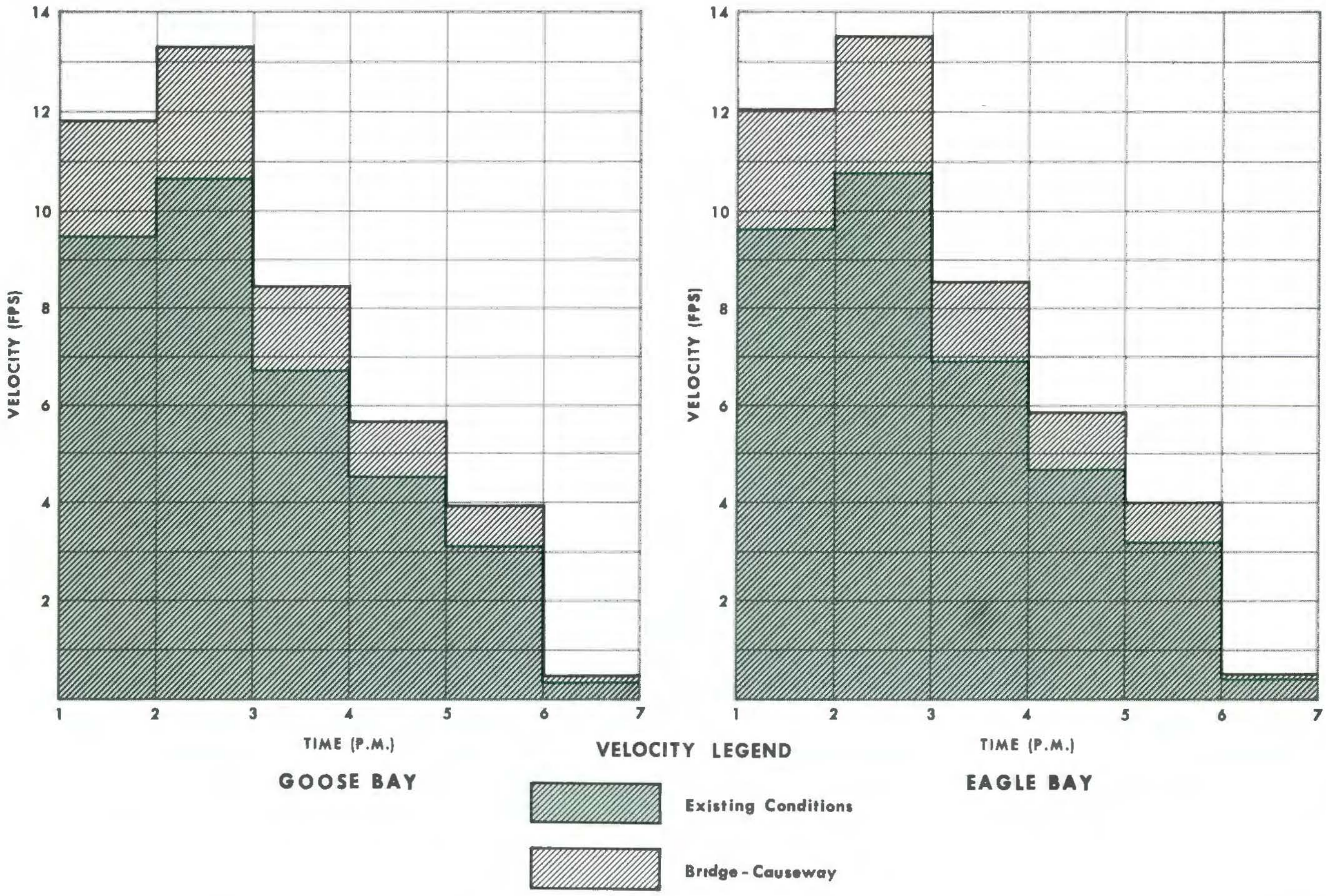


Figure III-17 CALCULATED AVERAGE VELOCITY AT CROSSING V EXISTING VS. BRIDGE- CAUSEWAY

(Based on May 24, 1971 spring tide data)

BOTTOM TOPOGRAPHY AND SEDIMENT TRANSPORT

Bottom Topography

The purpose of this portion of the study is to determine, if possible, whether or not major degradation or aggradation has occurred, or is occurring, in Knik Arm. If either of these phenomena have taken place, it will need to be considered in the design of the crossing regardless of the type of crossing chosen.

The data used in this review was as follows:

1. U. S. Coast and Geodetic Survey, Map No. 8557, dated 1969.

2. U. S. Geological Survey Quadrangle Maps.

3. Dames and Moore Geophysical Survey, 1970.

4. U. S. Coast and Geodetic Survey, Hydrographic Survey Smooth Sheets dated 1910-1964.

5. U. S. Coast and Geodetic Survey, Hydrographic Survey Descriptive Reports dated 1910-1964.

6. Aerial Photographs taken March 26, 1971

All of these data sources have limitations which restrict their effectiveness for this study. In order to better grasp the validity of the data, a brief commentary on each is included, followed by a further commentary on Crossings III, IV and V and a summary of the conditions in the entire Arm north of Cairn Point.

Map 8557

This map was prepared by the U. S. Coast and Geodetic Survey primarily for the use of navigators in the Arm. It is a compilation of data from a number of hydrographic surveys dating back to 1910. Apparently, as various hydrographic surveys were completed, portions of the map have been updated with December 20, 1969 being the most recent issuance of the map. The use of some Corps of Engineer's data is also indicated.

Of particular interest to the current project are the following limitations on the data:

1. Above the Goose Bay–Eagle Bay area the map apparently is based primarily on the 1914 hydrographic survey.

2. From the Goose Bay–Eagle Bay area to Cairn Point the map is apparently based upon data from the 1910 hydrographic survey. Some overlap of the surveys occurs in the Goose Bay–Eagle Bay area and the 1910 hydrographic survey overlaps later surveys south of Cairn Point.

3. South of Cairn Point the map has apparently been kept updated by a number of hydrographic surveys up to and including the 1964 survey.

U.S. Geological Survey Maps

The underwater data shown on these maps was obtained from selected hydrographic surveys. No reference is given on the maps as to the precise survey used on each map.

Dames and Moore Survey

In September 1970, Dames and Moore at the request of the Alaska Department of Highways ran several boomer survey lines to obtain as much information as possible concerning the foundation material in Knik Arm. An outgrowth of this survey was the production of bottom surface profiles along the lines surveyed. The survey lines are not coincidental with the crossing locations currently being considered. Dames and Moore Lines A-A and D-D are, however, in the vicinity of Crossings III and V respectively. On Line A-A some difficulty was encountered in matching the sounding points, probably due to the sharpness of Cairn Point and the difficulty of maintaining ship position in the area.

Smooth Sheets (USC&GS)

These sheets are hydrographic survey sheets showing the location and value of each sounding taken in the course of performing the survey. They differ from the "boat sheets" in that they have been reviewed in the office and obvious errors have been corrected. Some discrepancy on the early maps in horizontal locations was discovered and corrected. In checking this correction the National Oceanic and Atmospheric Administration was contacted and they concurred in the correction. However, if we assume that Map 8557 is accurate with respect to the plotting of latitude and longitude lines then the 1910 hydrographic survey map apparently still contains some horizontal error.

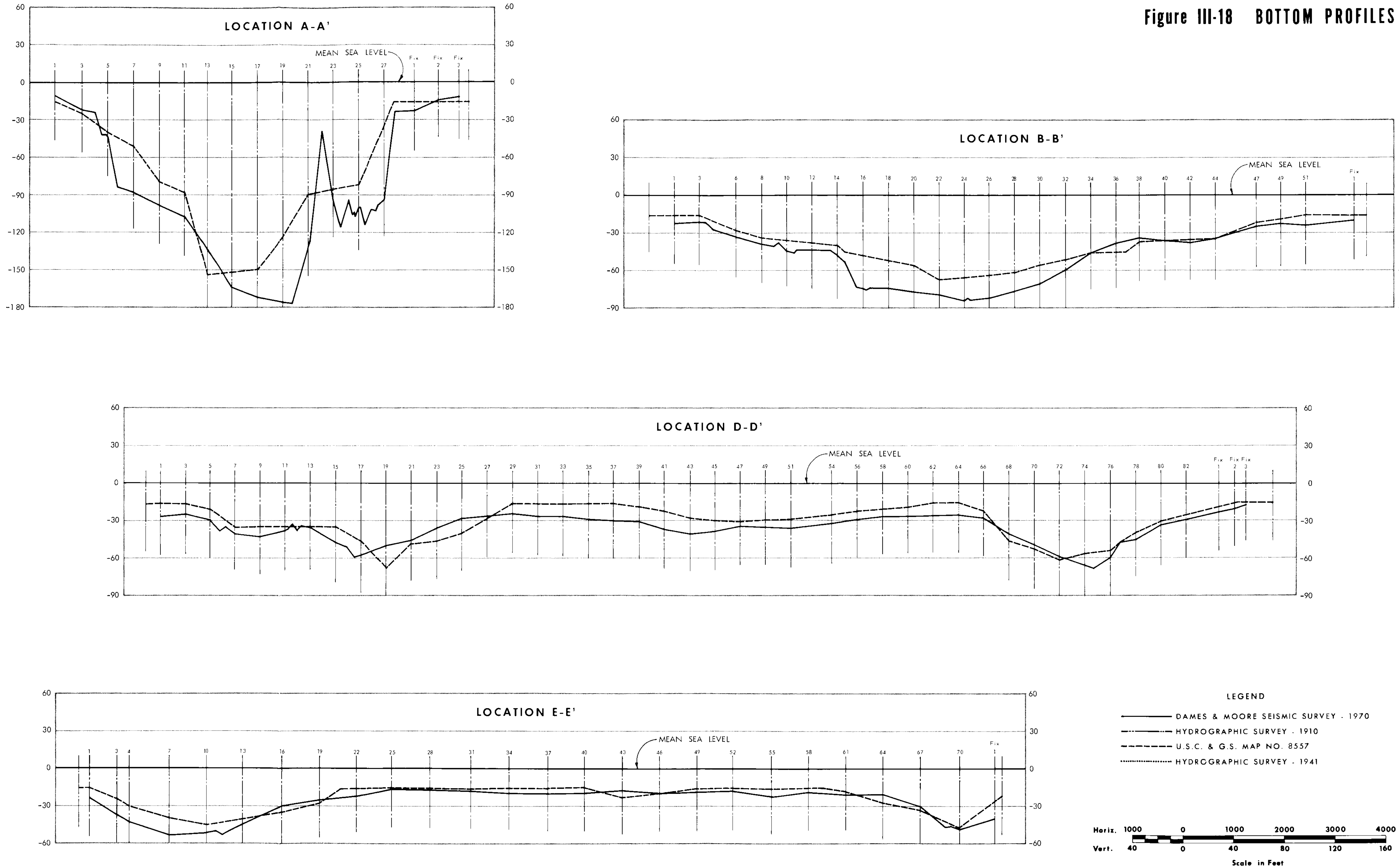
Descriptive Reports (USC&GS)

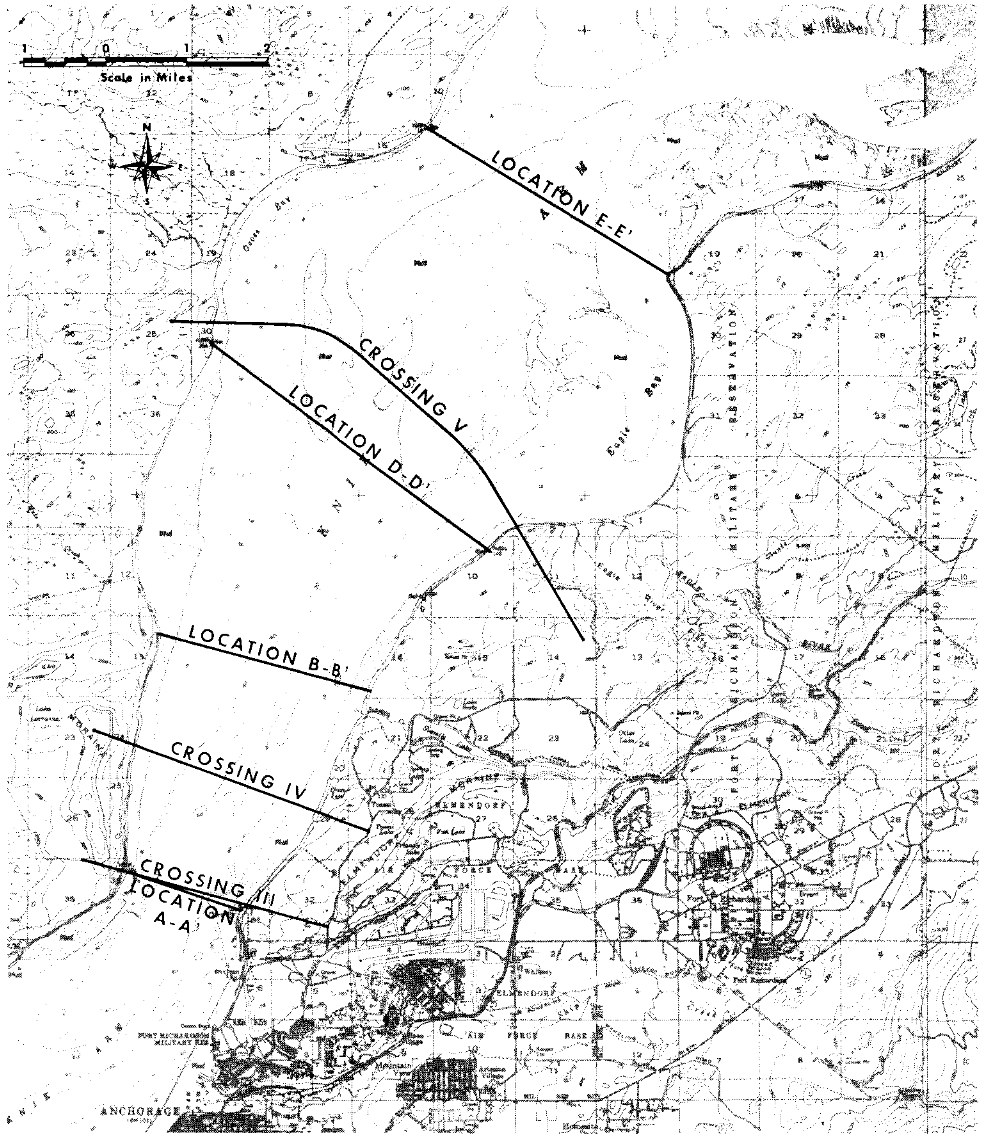
At the completion of each hydrographic survey, a report was prepared by the party chief outlining the actions taken, difficulties encountered, principal findings and other data of general interest uncovered in the course of the survey.

Aerial Photographs

These photographs were taken at the direction of HNTB to check ice cover and movement. Since the photographs were taken near low tide, they

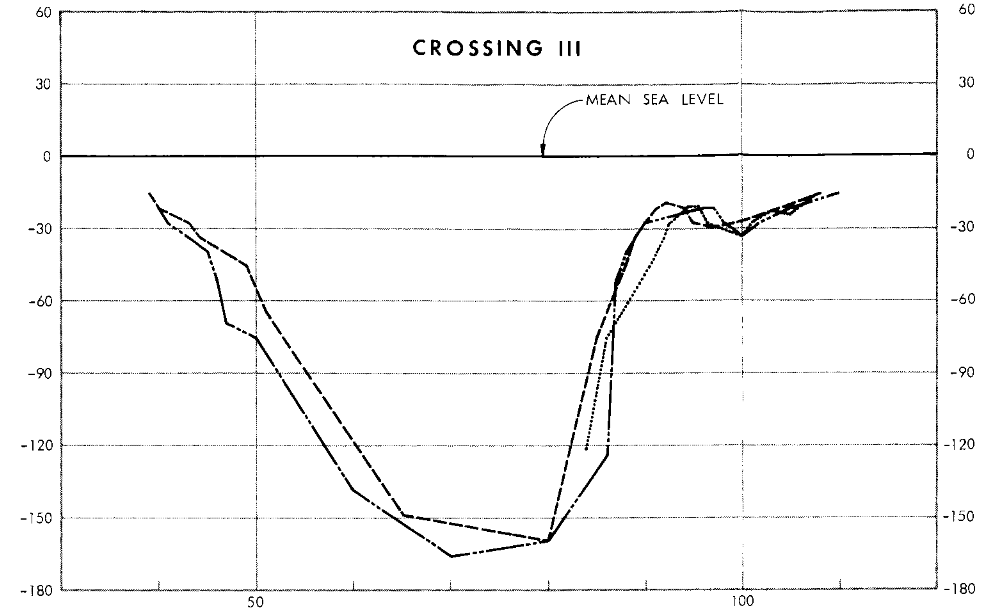
Figure III-18 BOTTOM PROFILES

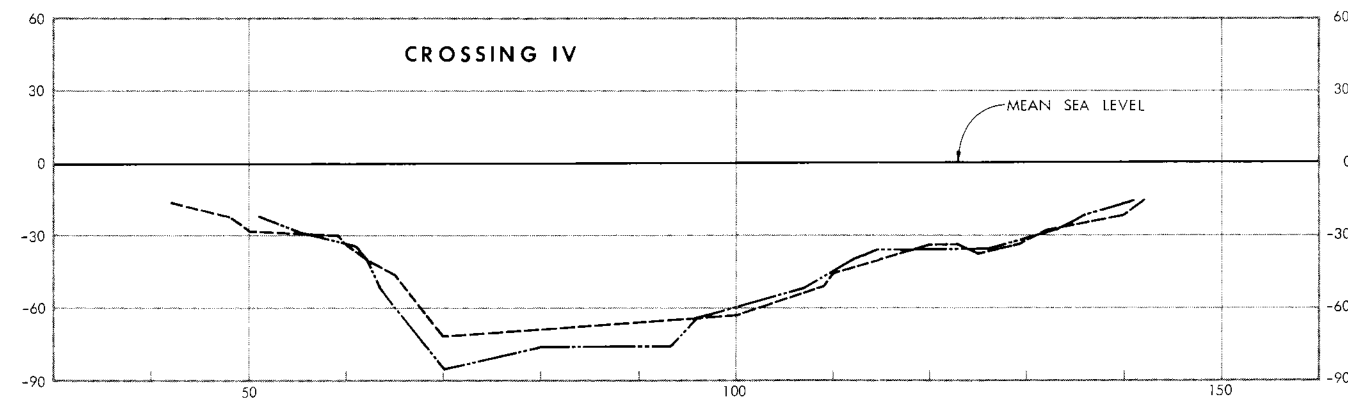


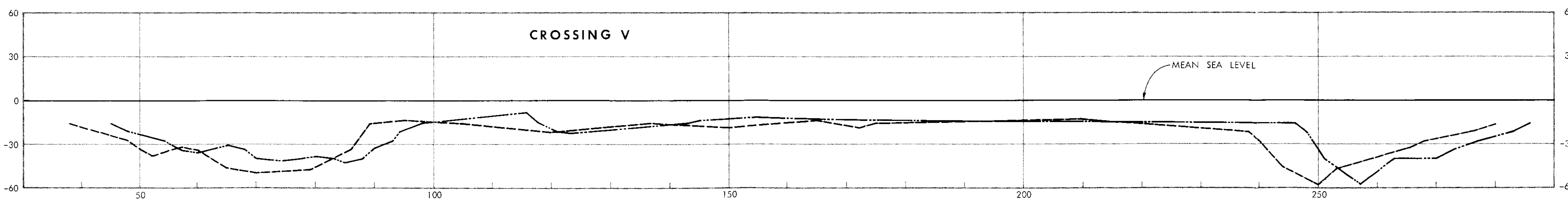


Topographic data on this figure is from U.S.G.S. Quadrangle Maps.

LOCATION MAP







are also useful in checking the extent of the mud flats areas near Goose Bay and Eagle Bay.

Data Review

A plot of the bottom profile as determined from the Dames and Moore Survey along Lines A-A, B-B, D-D and E-E is included on Figure III-18. Data from Map No. 8557 has been added for comparison purposes. Figure III-18 also shows bottom profiles for Crossings III, IV and V derived from appropriate hydrographic surveys and Map 8557.

On Line A-A, it can be noted that a major discrepancy occurs in the vicinity of Cairn Point. This probably is due to the difficulty of duplicating exactly the sounding position used in the Dames and Moore Survey. All other indications are that Cairn Point has remained relatively unchanged since the time of the 1910 hydrographic survey. On Line B-B degradation to a depth of 25′ has apparently occurred at Mark 16 near the west shore of the Arm. On Line D-D a general trend toward degradation is indicated as is some shifting of the main channels. On Line E-E no major change is indicated.

On Crossing III, the main channel from Sta. 40 to Sta. 92 has undergone considerable aggradation (58′ at Sta. 85) between the time of the 1910 survey and Map 8557. At Sta. 90 degradation of 10′ to 22′ is indicated in the time period between 1910 and 1941 and a similar amount of aggradation is indicated by comparing 1941 data and Map 8557. It does not seem plausible that aggradation would occur at this location, and therefore, some of the data is suspect. At Crossing IV the main channel between Sta. 62 and 96 has aggraded 6′ to 13′. Only minor changes have occurred outside the main channel. At Crossing V no major change in the bottom surface is indicated with the exception of a possible shift to the west of the main channels. However, this portion of Map 8557 is apparently based on the 1914 hydrographic survey giving only four years of record in the area. In addition, as indicated earlier, the horizontal position of the survey, indicated on the 1910 map, is suspect and a review of the profile indicates that if the 1910 profile were to be shifted 700′ to the west, an almost perfect match would be obtained and very little change would be indicated.

General Conclusions

An evaluation of the total data available indicates no major changes are occurring in the configuration or bottom elevations of Knik Arm within the study area. This conclusion is reached despite some evidence to the contrary which may be largely the result of varying horizontal controls. A review of the aerial photographs indicates essentially the same pattern of mud flats as is shown on Map 8557 (1914 survey?). Some rearrangement and shifting of the secondary channel is indicated, but this could easily be a temporary revision. The large amounts of material delivered to the Arm by the Knik and Matanuska Rivers have apparently been counterbalanced by the inflow and outflow of the tides, thereby preventing a major general change in the Arm bottom.

Sediment Transport

The Matanuska and Knik Rivers are glacier fed rivers which transport large quantities of suspended sediment into Knik Arm. The 1914 Hydrographic Survey makes mention of a breakout of Lake George which entirely silted in all channels in the Arm down to the town of Knik.

The United States Geological Survey started suspended sediment data collection from the Matanuska River in 1953, and the Knik River in 1962. The records for the first few years spanned only the months of April to September, the months of high sediment load. Later, in 1964, records were obtained for the entire year.

The average total suspended sediment for the Matanuska and Knik Rivers is approximately 16 million tons for the April to September period. The average for the October–March period is approximately 135,000 tons. The maximum daily suspended sediment load recorded for the Matanuska River is 1.3 million tons, August 25, 1959, and 2.0 million tons for the Knik River, July 10, 1965 (Lake George Breakout peaked at old highway bridge on July 11, 1965).

The suspended sediment load samples from the Matanuska and Knik River include material ranging in grain size from 0.002 mm to 1.0 mm. Bed load materials range up to 32 mm. Information on sediment load at or near Anchorage is not available so as to determine the amount of material deposited in Knik Arm.

From the studies initiated by Dr. Per Bruun the material equal to or less than 0.06 mm (material which would remain in suspension) was computed for all discharges together, as well as for discharges equal to or greater than 100,000 cfs. For all recorded discharges approximately 60% of the total material is equal to or less than 0.06 mm, and for discharges equal to or greater than 100,000 cfs approximately 6 to 10% of the total is equal to or less than

0.06 mm. Based on these figures, it is estimated that 60%, or approximately 10 million tons per year, of the material entering the Arm as suspended load leaves the Knik Arm in the highly turbulent waters and in the density currents moving along the bed. It is assumed that the other 40% of the suspended material is presently deposited in the Knik–Matanuska delta and not in the area considered in this report.

In the case of a bridge–causeway project the tendency to settling on the shoals north of the causeway may increase but no major change is expected. The situation is different if a causeway dam is built. Under normal conditions almost all material will settle behind the dam and under abnormal conditions, when discharges increase beyond 100,000 cfs, more of the material will escape. Of the yearly average 16 million tons, approximately 14 million tons per year will settle behind the dam. This would raise the lake bottom an average of 1.3 inches per year.

WAVES

The major considerations for wave effects on a highway structure crossing Knik Arm are: Wave magnitude and direction of winds, wave heights, wave runups, and minimum time necessary to develop maximum waves. These considerations are important factors in establishing final design criteria and also anticipating wave effects during all phases of construction.

Considerable design wave data and studies are available for Cook Inlet from oil platform construction studies and shipping sources. Due to lack of navigational movement and previous construction in Knik Arm, wave information for the study areas is very sketchy.

Figure III-19 is based on a design wind of 60 m.p.h. from the north and 45 m.p.h. from the south. The design winds were selected on the basis of information previously presented in another section of this report.

As indicated in Figure III-20, the probable maximum waves from the north exceed those from the south and have a very slight variation at Crossings III, IV and V. A maximum design wave of 15.0/2 = 7.5 ft. is, therefore, used for setting top of pier elevations in the bridge design (datum is calm water level with distance to top of trough = 7.5 ft. and distance to bottom of trough from datum = 7.5 ft.). Southerly waves, while less in probable maximum magnitude, are noteworthy, as they may occur during the construction season when work is being performed on the open water. The anticipated

			CROSSING		
			III	IV	V
WIND (mph)		North	60	60	60
		South	45	45	45
FETCH	LENGTH (miles)	North	27.0	25.5	20.0
		South	5.0	6.5	12.0
	WIDTH (miles)	North	4.5	4.8	5.0
		South	2.2	2.8	3.0
	Width/Length	North	.167	.19	.25
		South	.44	.43	.25
F_E/F		North	.37	.39	.45
		South	.63	.62	.45
EFFECTIVE FETCH (F_E) (miles)		North	10.0	9.9	9.0
		South	3.1	4.0	5.4
SIGNIFICANT WAVE HEIGHT (feet)		North	8.0	8.0	7.8
		South	5.0	5.8	6.3
MAXIMUM WAVE HEIGHT (feet)		North	15.0	15.0	14.6
		South	9.4	10.8	11.8
MINIMUM TIME TO DEVELOP MAXIMUM WAVE (hours)		North	1.50	1.50	1.36
		South	0.72	0.88	1.01

Note:

Wave calculations are based on data from "Shore Protection Planning and design", U.S. Army Corps of Engineers - Technical Report No. 4, 1966, ASCE Transactions, 1959, and other sources.

Some values indicated in tabulations are shown to one or two decimal places for continuity of calculations. For the purpose of this study such accuracy is not warranted and in fact the data should not be interpreted to represent more than a preliminary approximation.

Figure III-19 CALCULATED WAVE CHARACTERISTICS

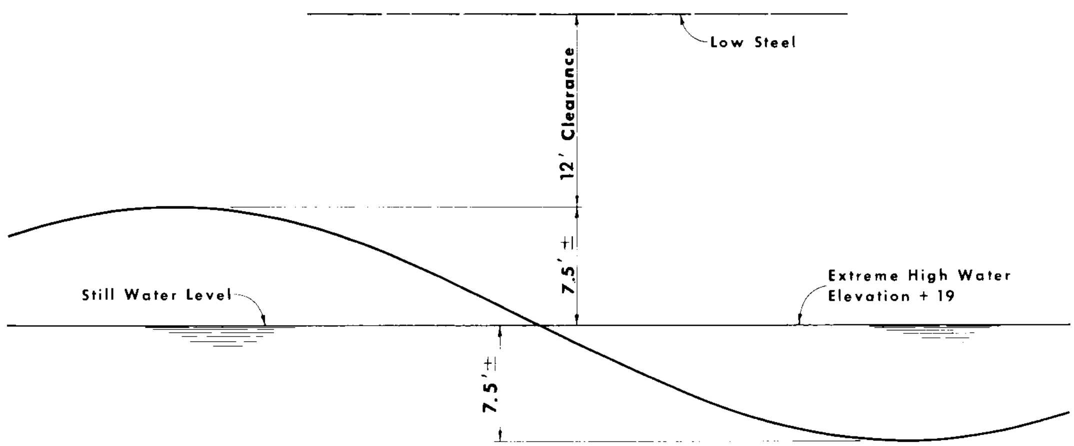


Figure III-20 CLEARANCE DIAGRAM FOR MAXIMUM WAVE

probable maximum waves from the south are 4.7 ft. at Crossing III, 5.4 ft. at Crossing IV and 5.9 ft. at Crossing VI.

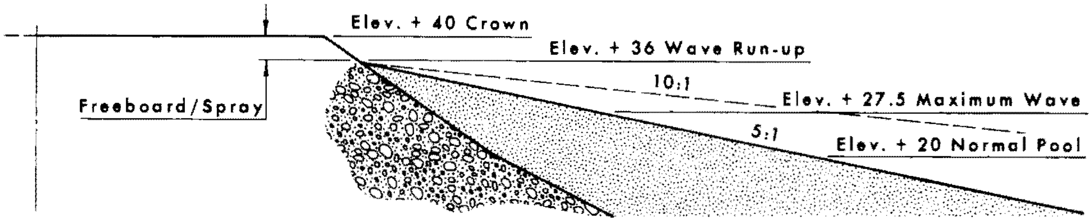
Anticipated minimum time necessary to develop maximum waves also would be important data for use during construction. Variations from 1.36 to 1.50 hours for northerly winds and 0.72 to 1.01 hours for southerly winds are listed in Figure III-19.

It should be noted that strong gusty winds blowing at extreme high tides may cause slightly larger waves than indicated in Figure III-19.

The following is recommended for applying the maximum probable wave criteria for determination of preliminary top of bridge pier elevations:

Extreme High Water	=	Elev.	19
Maximum Wave	=		7.5
Clearance (Spray & Safety)	=		12
Min. Low Structure	=		38.5
Allowance for Bearing Assembly			–2.5
Set Top of Pier Elevation			36.0 MSL

Another important consideration is determination of the crown elevation to be used for causeway dam studies. Design winds, maximum waves, tidal variation, pool elevations, slope of causeway face, wave runup, allowance



Note:

Assume sand blanket slope is steepened from 10:1 to 5:1 due to local scour.

For 45 M.P.H. design wind from the south consideration should be given to slopes of 3:1 or less from Extreme High Water Elevation +19 to Crown Elevation +40 to prevent spray and wave over-run. Further studies would be necessary for final design.

Figure III-21 CAUSEWAY DAM CROWN ELEVATION

for spray are all factors in determining a suitable crown elevation based on a design wind of 60 m.p.h. from the north. Sources of information include: "Shore Protection, Planning and Design" – U. S. Army Corps of Engineers – Technical Report No. 4, 1966 and various ASCE transactions and proceedings, including "ASCE, Journal of the Waterways, Harbors and Coastal Engineering Division," Vol. 97 No. WW1., February, 1971.

ICE

General

Introduction

Ice presents problems which must be dealt with in any engineering structure sited in the arctic or sub-arctic. This section will be limited to a discussion of ice as it is found in the sub-arctic, estuarian environment prevailing in the Anchorage area, and to a few problems specific to the alternates of the Knik Arm project.

Areas of concern generally fall in the following groups:

1. Static and dynamic loads on structures for either a full bridge crossing or a bridge–causeway combination.

2. Erosion of structural elements due to abrasion and freezing.

3. Ice pile-up by wind and currents on causeway fill slopes.

4. Rip-rap removal by shore ice (which will affect structure abutments and causeways).

5. Icing phenonomen (which will affect flow characteristics of a spill-way).

Mechanical Properties of Ice

Ice is formed by the crystalization of water and is affected by temperature, water turbulence and impurities. Physicists have done extensive studies on crystalization of fresh water because this material is readily available and exists in all three phases within a very small and convenient range of temperatures. Much of this ice research has been done on glaciers and is not directly related to sea ice.

One of the basic differences between pure water ice and sea ice is the impurities trapped in sea ice. Brine and salt crystals are found only in sea ice while silt particles and entrapped air may be found in all types of ice. Figure III-22 shows an idealized diagram of a sea ice crystal. The C-axis is the principal crystallographic axis, the B-axis is the direction of brine inclusions between platelets and the G-axis is the direction of crystal growth. The brine inclusions occur in natural ice as roughly circular, discontinuous tubes filled with brine.

The process of freezing begins when the temperature of the sea water is lowered to the freezing point of the saline solution. Pure ice platelets form with the excluded salts concentrated as brine in the brine cylinders. Freezing continues with crystal growth on the bottom surface of the ice layer where water is available. As the ice temperature continues to be lowered some salts in the brine inclusions reach their freezing point and form the salt-ice lining of the inclusions with a smaller volume of liquid brine in the center.

Ice crystals are strongest in compression in the G direction of the crystal. Ice is weaker in the other directions because of the brine inclusion discontinuities, slippage between platelets and between crystals. Ice sheets grow with the C-axis horizontal by natural crystal selection after the initial random formation of the surface freeze. The more turbulent the water, the deeper the random crystal layer extends. However, once frozen, the turbulence is decreased due to shelter from wind action. From this it may be seen that impact forces from drifting ice floes are generally directed along the weaker planes.

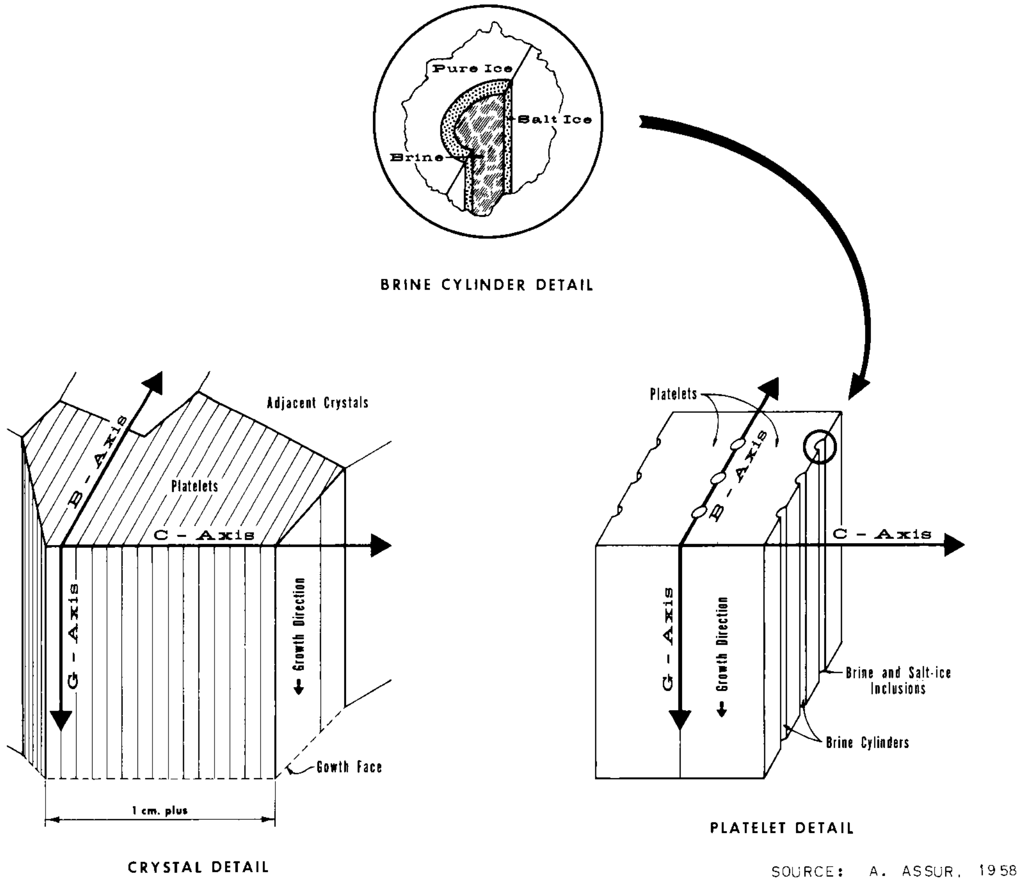


Figure III-22 SEA ICE CRYSTAL STRUCTURE

Rapid freezing tends to weaken ice structure by including more air bubbles in the matrix and by producing stress cracks within the ice mass. It has been stated (Weeks and Assur, CRREL Report 269) that quick frozen lake ice is 15% weaker than slow frozen lake ice.

Turbulence during freezing weakens ice by increasing the air bubble content and thickening the layer of random crystal orientation. This layer generally does not extend downward much more than 10 centimeters (4 inches).

A lowering of temperature strengthens sea ice by increasing the inter-crystalline bond and by forming more salt-ice in the brine channels which acts as reinforcement against failure in the weaker planes.

Tests by a number of experimenters have shown that testing of small scale samples cannot be expected to produce directly usable results for large scale design. Small samples (3 inch dia.) show breaking strengths much greater than that provided by large ice sheets. These test samples usually include fewer serious discontinuities and in extreme cases may be made from one single ice crystal. Cook Inlet experimenters have found that 55% of small sample strength compares with results obtained from test piles subjected to actual Cook Inlet ice forces. At most, small sample testing provides a means of classifying the ice structure.

Some full scale work has been done in Canada and the North Sea. This has consisted of instrumenting existing structures and measuring the force exerted by passing fresh water ice floes. The maximum force of 159 psi was measured in Canada on the Pembina River at Pembridge, Alberta (Neill, Studies of Ice Pressure on Bridge Piers in Alberta, Canada, IAHR Ice Symposium 1970). Furthermore, failure loads have been calculated for a number of Canadian bridges which have successfully withstood ice loads for years. One bridge constructed in 1906 would theoretically fail if subjected to 120 psi ice load and so far has suffered no damage. Peak pressures of 7.5 kp/cm^2 (108 psi) have been measured by Dr. Schwarz (The Pressure of Floating Ice Fields on Piles, IAHR Ice Symposium 1970) on an instrumented pier 60 cm (2.0 ft.) wide in the tidal estuary of the Eider River in the North Sea.

Ice Formation

Most ice blocks are formed as shore-fast ice or are built up on the periodically flooded mud flats in upper Knik and Turnagain Arms. One process of formation builds the shore ice by starting the freeze over a shoreline area exposed at lower tide levels and increasing block size by successive coatings of ice during each tide cycle. Horizontal layering is characteristic of this type of formation as shown in Photo 1. The depth of the ice formation is controlled by the thermal balance of heat transfer into the atmosphere during low tide exposure and heat flow into the soil from the warmer tidal water during high tide coverage. Any shoal area at an elevation higher than the heat-balance elevation is covered with solid ice, see Photos 2, 4 and 7. Higher current velocities tend to retard ice formation by changing the heat flow and erosion characteristics.

A second condition which aids in ice block formation is fresh water flow in stream areas. This flow spreads on top of the sea ice and thickens it well beyond the dimensions possible by tidal fluctuations. The twenty to thirty foot bergs reported in the Cook Inlet marine forecasts probably have their origins in fresh water seepage areas.

Photo 1 – Feb. 15, 1971
Near Cairn Point – Blocks in foreground are about 3 feet thick.

Photo 2 – Feb. 6, 1971
Ice is about 6 ft. to 8 ft. thick on west side of shoals, north of Goose Bay.

Once the ice begins to form, forces act to break it up, see Photos 3 and 4. Thermal stresses will cause cracking perpendicular to the shoreline and buoyancy and gravity forces will cause cracking parallel to the shoreline. After the blocks are broken, spring tides will float them free and move them into the main current channels where they become a menace to shipping and structures. Older blocks have a shape characterized by nearly equal length and width and sides that are rounded in toward the bottom thus giving them maximum dimensions at the water line, see Photo 5.

Floe or sheet ice is formed by the growth process discussed earlier in the mechanical properties part of this section. The thickness is governed by the degree-days of freezing available in the locality. Agitation by eddy currents and high velocities keeps large solid floes from forming in Knik Arm. Most large floes observed in Knik are composed of small floes held together by a matrix of thin, fresh ice as seen in Photo 6.

The Anchorage climate (650 centigrade degrees days below $-7^{\circ}C$) will allow, on the average, only about two and one half feet of floe ice to form with a 100 year maximum of about three and one half feet.

Photo 3 – Feb. 21, 1971
Near Cairn Point – shore ice break-up.

Photo 4 – Feb. 15, 1971
Shore ice near Cairn Point – ice face is 6 ft. to 8 ft. high with loose ice piled on top.

Photo 5 – Feb. 15, 1971
Ice block 8 ft. x 15 ft. x 15 ft. near Cairn Point.

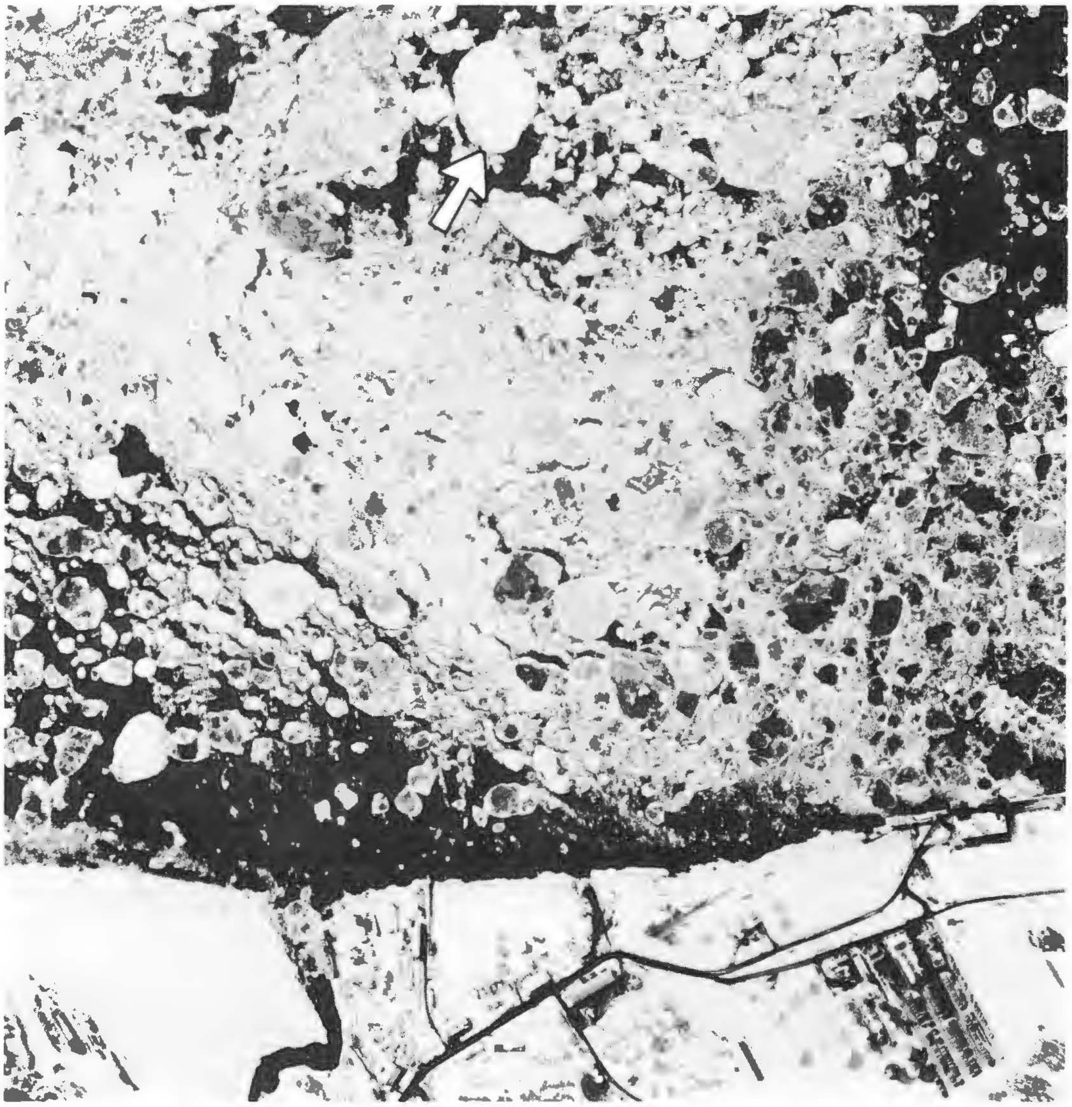


Photo 6 – Mar. 22, 1971
Note 400 ft. x 800 ft. floe near top of photo – Anchorage City Dock is at lower right.

Ice Strength

K. N. Korzhavin (Action of Ice on Engineering Structures, USSR–Novosibirsk, 1962) has developed relations for the compressive failure of Russian river ice. These formulas are based on the pier width and floe velocity. Some representative values are shown in Table III-9.

Table III-9 ICE STRENGTH

PIER WIDTH	FLOE VELOCITY	ICE STRENGTH
10 feet	2 m/sec (6.1 fps)	3.6 kg/cm^2 (51psi)
20 feet	2 m/sec (6.1 fps)	4.6 kg/cm^2 (65psi)
20 feet	2.5 m/sec (7.6 fps)	4.3 kg/cm^2 (61psi)
20 feet	3 m/sec (9.1 fps)	4.0 kg/cm^2 (57psi)

These values are based on indentation of a pier into ice sheets large enough so that they do not split.

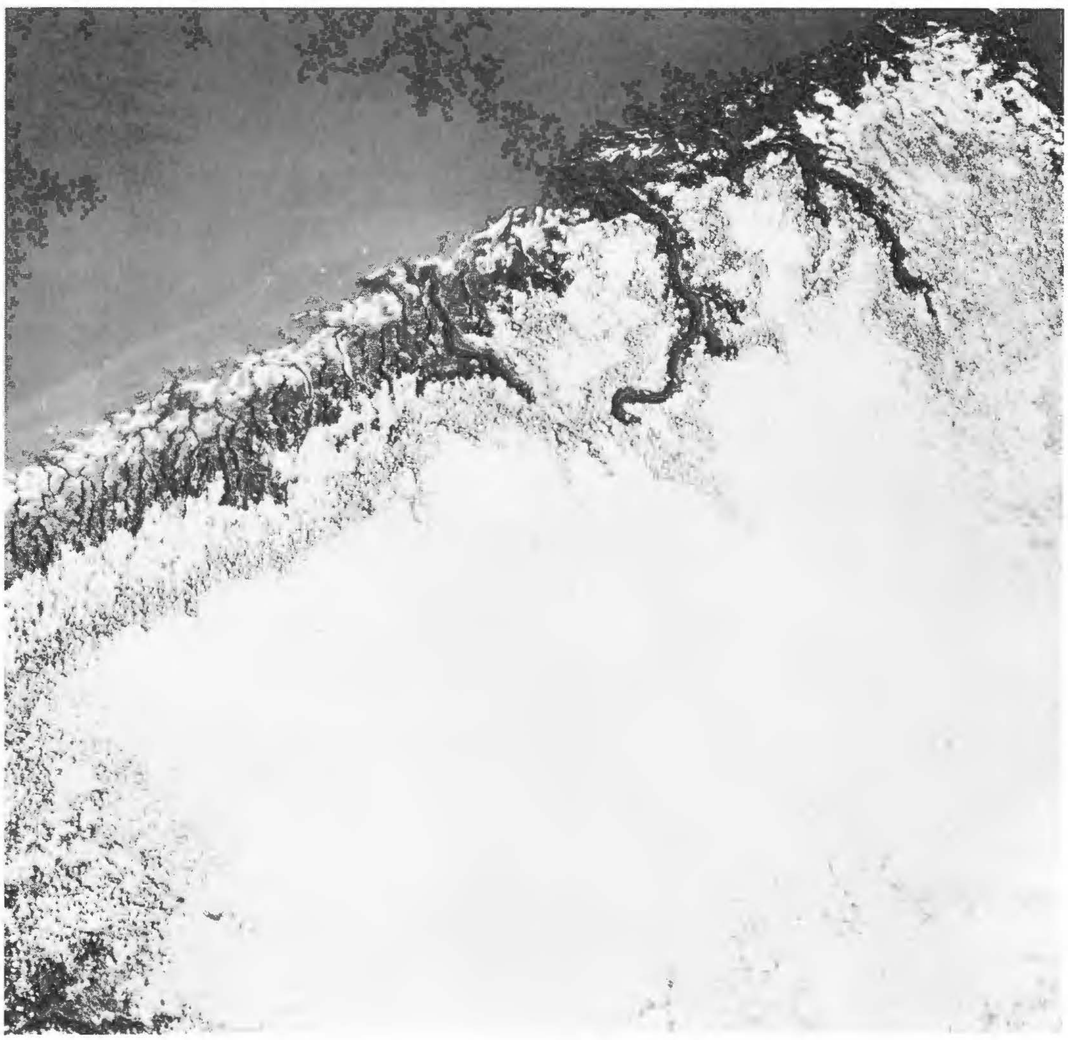


Photo 7 – Mar. 26, 1971
Shoals near Eagle Bay at low tide.

Boris Weinberg in his paper "Mechanical Properties of Ice" (page 532) gives a value of 9.6±3.3 kg/cm^2 (136±47psi) as the crushing strength of natural salt ice in northern Europe. Ice strengths from 4 to 9 kg/cm^2 (57 to 128psi) have been mentioned in the proceedings of the IAHR Symposium, 1970, at Reykjavik, Iceland. As previously mentioned in this section, Canadian researchers have measured forces ranging up to 159 psi (11.2 kg/cm^2) for fresh water ice.

Ice Structure Interaction

The failure of an ice floe passing around a pier has been shown to be vibratory in nature (Peyton, 1966) with the frequency being dependent on ice velocity and relatively independent of the natural frequency or deflection characteristics of the pier and superstructure. Peyton's experiments with loading rates on Cook Inlet ice indicate that strength depends on load rate with maximum strength developed at a rate of 2000 psi per minute. Small-sample strength was about 300 psi at this rate, degrading to about 150 psi at load rates approaching 10,000 psi per minute. The greatest ice loads are thus associated with velocities approaching zero and as such must be applied to the structure at mean high tide. The frequency of load application at these maximum loads appears to be about one cycle per second so structural frequency may need to be taken into account to prevent harmonic build-up and possible overstress of structural elements. Preliminary calculations for the natural frequency of the proposed structure indicate values of 1.5 to 2.1 cycles per second for the basic pier used in this study. When the pier weight and viscous damping by the water are added in, the frequencies approach the critical value of one cycle per second. So far, model tests and field experience by Peyton have not demonstrated any resonant load build-up. It has been stated by Matlock et. al. (A Model for Prediction of Ice-Structure Interaction, University of Texas, 1969) that their mechanical analog study indicated no resonant build-up when the ice impingement rate equaled the natural frequency of the structure. This conclusion was arrived at from computer studies only, and until it is demonstrated by full scale tests the worst conditions of resonant built-up cannot be discounted.

Another load condition is that caused by a sizeable moving block of ice striking a structure. A study of tidal velocities indicates peak speeds may be expected during the second hour after high or low tide. During ebb tide the velocities begin to build before there is an appreciable fall in water level. Figure III-23 is a generalized plot of this phenomena and shows that after a high tide, near maximum velocity may be expected while the water level has dropped only about five or six feet, thereby producing a near maximum impact moment arm on any pier structure.

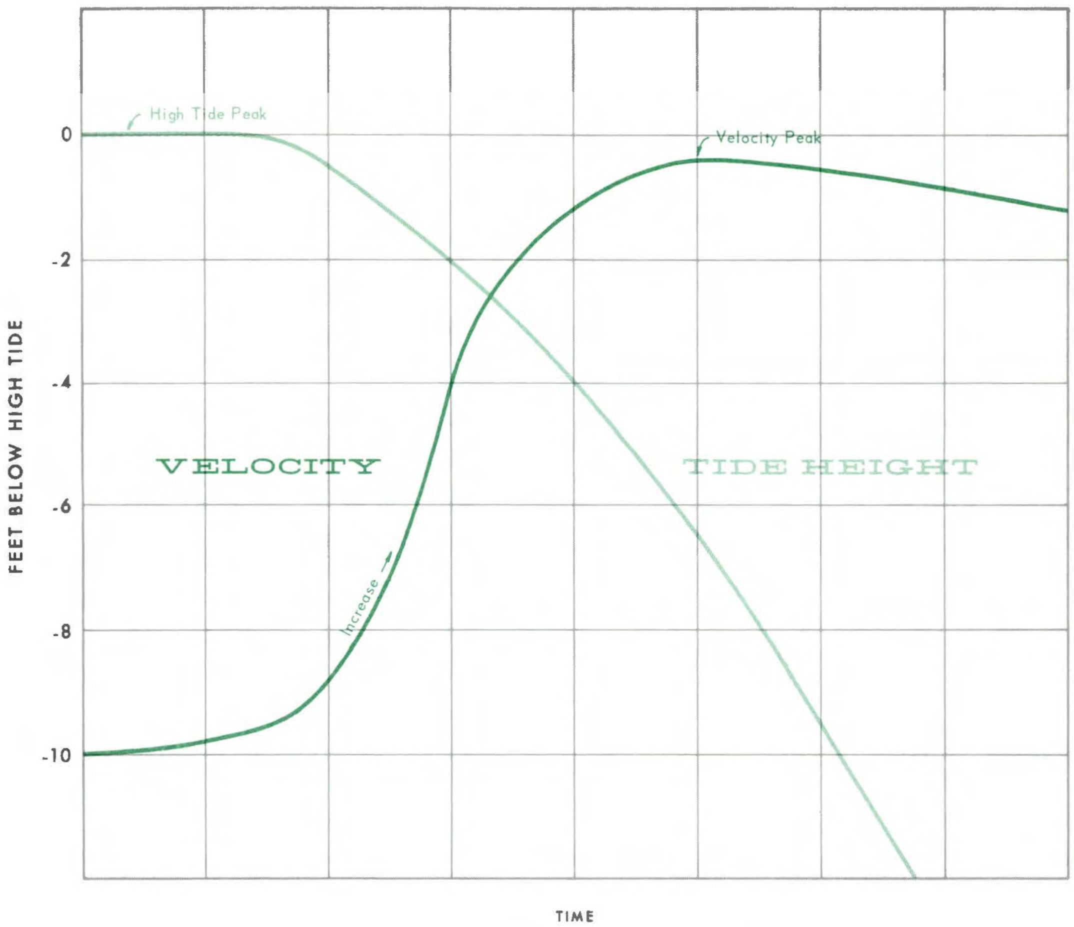


Figure III-23 TIDE FALL VS. VELOCITY

The collision energy of an ice block is modified by a number of factors. Ice blocks are asymmetrical in all three planes, therefore, the rotational and translational components serve to reduce the impact energy for most collisions. However, it must be assumed that at some time a major ice block would impact with the line of action coinciding with a line through the center of masses of the block and pier. If this happened the only source of shock attentuation would be the local crushing of the ice. With the present state of the art, a numerical value for this factor is difficult to ascertain.

Another factor which will increase impact loads is the hydrodynamic force caused by the moving water mass acting on the suddenly stopped ice block. F. Vasco Costa, Prof. of Harbour Works, Technical University, Lisbon stated in a paper (The Berthing Ship, Dock and Harbour Authority) that this force can be calculated as the kinetic energy of the vessel (ice block) times 1+2D/B where D is the draft and B is the width of the ship (ice block).

Specific Information Concerning Cook Inlet and Knik Arm

Ice Strength

Figure III-24 indicates direct compression failure stress versus load rate obtained from samples of Cook Inlet ice (Peyton 1966). Note that the strength peaks at about 300 psi at very low load rates.

The study of Turnagain Arm ice performed by the Alaska Department of Highways during 1968-1969 has developed ice strengths of 331 to 344 psi with a recommended design strength of 300 psi.

In light of the above mentioned work and cited strength values, it seems reasonable to reduce the AASHO ice load criterion from 400 psi to 300 psi and apply this value over the entire width of a pier and full thickness of the ice floe. Furthermore, this force should be applied at the mean high water elevation.

Ice Movement

The movement of ice blocks and floes in the Cook Inlet area is controlled by wind and tidal currents. The tide provides the major motive force but the presence, direction and duration of the wind can accelerate or impede the seaward migration of the floes. When a floe reaches several hundred feet in diameter, the water friction force may be considered infinite and the floe will fail in compression against any obstacle it meets. Current measurements during ice periods can be relied on to give quite accurate ice velocity values. Calculations made from the Knik Arm tidal prism have yielded velocities as high as 12 fps while actual current measurements made for the Corps of Engineers in 1964 have been as high as 8.8 fps.

Ice in the Cook Inlet area exhibits a net seaward migration due to prevailing winter winds which are down both Knik and Turnagain Arms. However, reports indicate that wind has on occasion held ice in Knik Arm or, conversely, caused the ice to vacate the Arm in 24 hours. The 1968-1969 Alaska Highway Department Study of Turnagain Arm gives an estimated seaward movement of two miles per day.

The large tidal currents and eddies caused by the configuration of Knik Arm not only prevent large floes from forming in the Arm but tend to

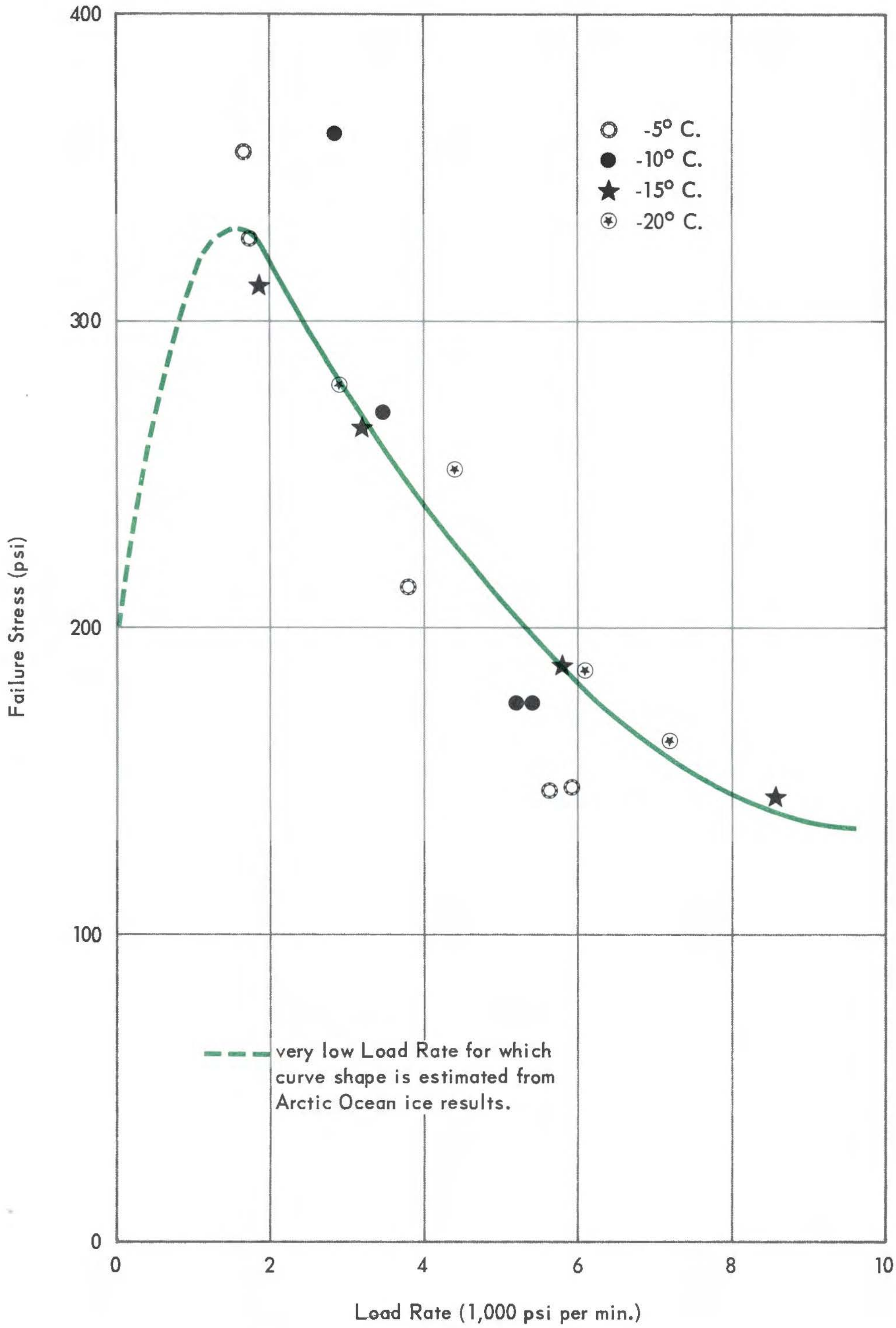


Figure III-24 ICE STRENGTH

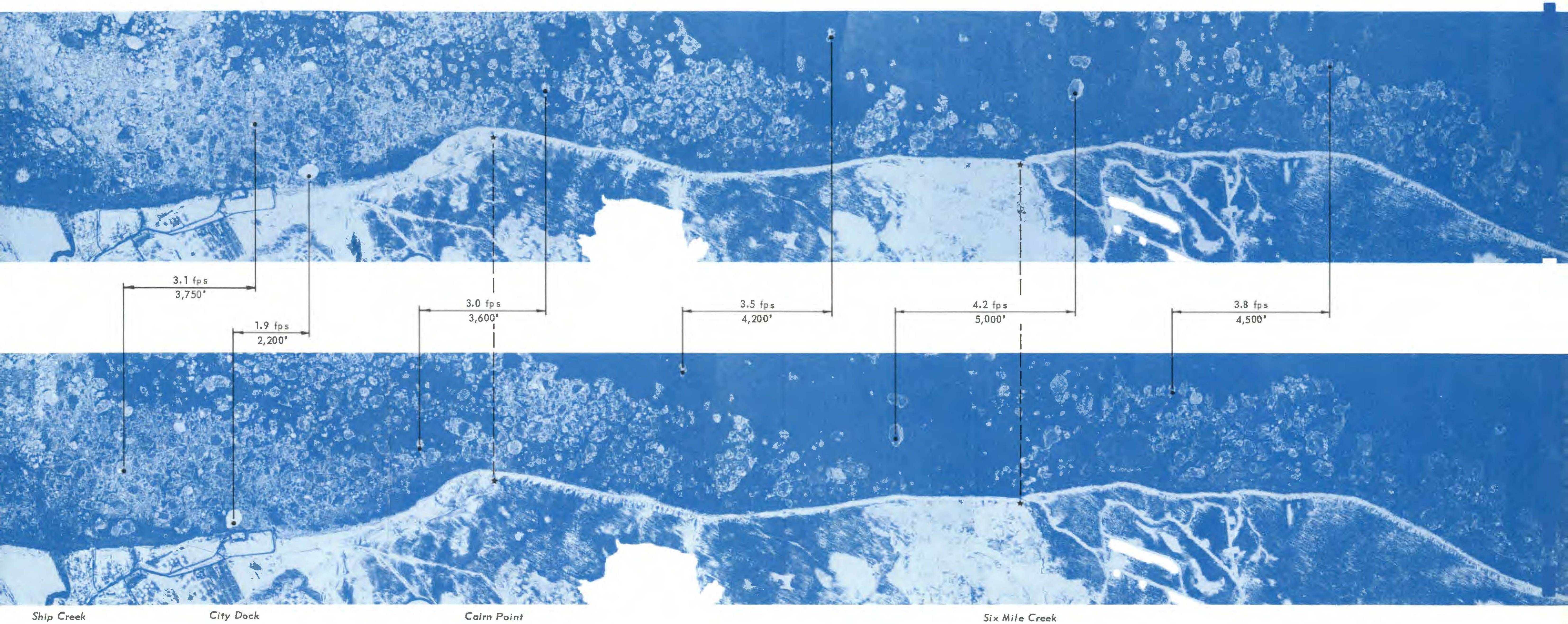
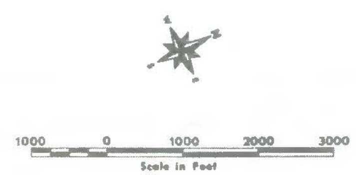


Photo Date: - March 22, 1971 (1 hour after High Tide)

Time Difference: - 20 minutes



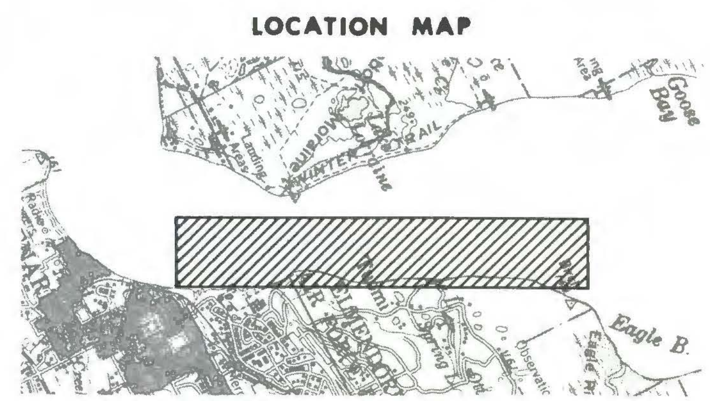


Figure III-25 ICE FLOE VELOCITIES

break up any that are carried into the area from Cook Inlet. Figure III-25 shows the results of two aerial flight lines photographed exactly 20 minutes apart and about one hour after high tide. This gives a good illustration of ice movement between Eagle Bay and Cairn Point. Photos 8 and 9 demonstrate the general current alignment of the ice. The angle of approach of ice may be predicted within quite a small angle, probably plus or minus ten degrees at Crossing IV.

Photo 8 – Feb. 21, 1971
Floe pattern during ebb tide. Looking north from vicinity of Pt. Mackenzie

Photo 9 – Feb. 21, 1971
Floe pattern during ebb tide – Cairn Point at extreme right.

Ice Size

The U.S.C.&G.S. publication, U.S. Coast Pilot 9, states that the average period of ice cover in Cook Inlet extends from 1 November to about 1 May each year. Coast Pilot 9 gives an account of block ice up to eight feet thick, covering upper Cook Inlet to a concentration of three to four tenths coverage during the month of April 1964. This ice originated in Knik and Turnagain Arms and melted on the flats north of Point Possession. The publication also states that the maximum theoretical thickness of sheet ice, based on freezing degree days is 2 to 3.5 feet at the head of the inlet.

Figure III-26 is reproduced from the Alaska Department of Highways Turnagain Arm Ice Study and shows predicted yearly ice growth in Cook Inlet with some actual values obtained from observations and measurements. It should be noted that the actual measured maximums have, to date, been less than 36 inches and less than prediction curves would indicate. Observers for the Turnagain study have measured several floes two feet thick by 2000 feet diameter and one block 10 feet x 20 feet x 30 feet (350 kips) during the 1968-1969 Study season.

Visual and photographic observations were made during February and March of 1971 in conjunction with the contract for this report. Photo 6 shows one of the more substantial floes which scales about 400 by 800 feet. This floe stands out in the photograph due to its snow cover. No thickness measurements were taken, but it is probably 18 inches thick and relatively old. Very few of the other floes in this photo appear to be firm enough to damage a major structure. Photo 5 shows a block about 8 by 15 by 15 feet (100 kips) which was observed on February 15, 1971, just south of Cairn Point. Most of the shoal and beach ice, as illustrated in Photos 2 and 4 reached a maximum thickness of about 8 feet. The high tides of February 28 floated much of this grounded ice free.

On March 26, 1971, a photo mission was flown to obtain information about ice coverage on the shoal between Goose Bay and Eagle Bay. About half of the area had been swept clear of ice by the February tides. A study of these photos shows a number of relatively large blocks. Conversion of sun angle and scaled shadow length reveals block thicknesses on the order of 9 to 12 feet. The largest block found is grounded near the south edge of Goose Bay and is about 14 by 30 by 40 feet (950 kips), and is probably foreign to the area.

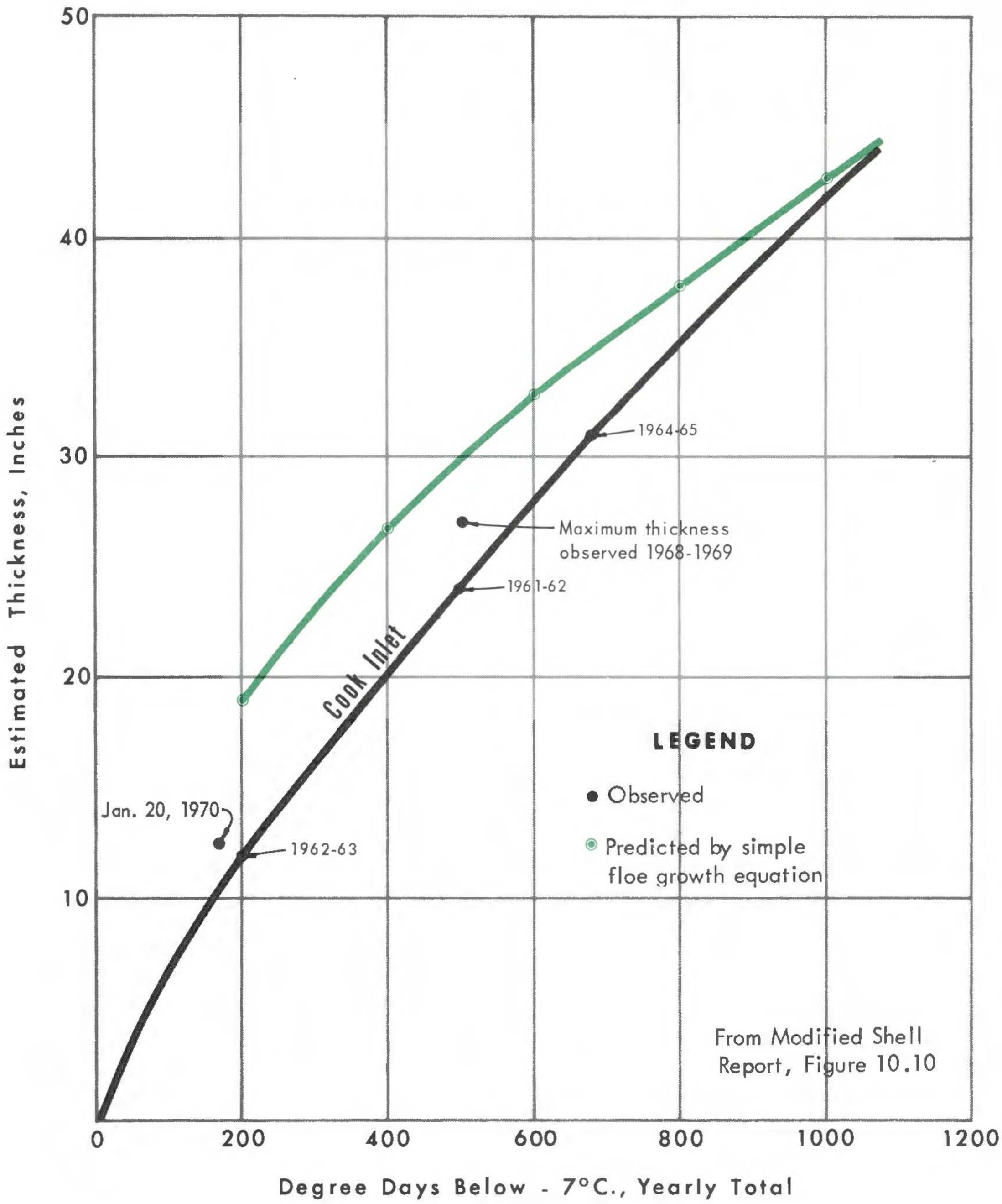


Figure III-26 SEA ICE GROWTH

The force of an impact is controlled by the deceleration time of the impacting object. Thus the time taken to locally crush the ice block controls the force of the impact. If 10 feet per second ice velocity and one half second deceleration time is assumed the impact force of the 1840 kip effective weight would be 1142 kips. Assuming 10 feet per second velocity and one second deceleration, the impact force would be 571 kips. Large scale testing is needed to more accurately determine the deceleration characteristics of ice blocks.

It is suggested that preliminary structural design be based on the following ice force parameters:

a. Floe failure of 300 psi times 30 inch ice depth times pier width applied at mean high water.

b. Ice block impact of 660 kips applied five feet below mean high water.

Structural Erosion

Consideration must be given in final design to erosion of structural elements from the "sandpaper" effect of ice-carried silt and gravel and to concrete spalling due to the marine environment. The Cook Inlet oil platforms have experienced several thousandths of an inch erosion on the steel legs within the tidal range. There is not enough experience in the Cook Inlet area yet to make a valued judgment of this problem. However, it appears desirable to use steel armor on concrete piles in the Knik Arm waters. Photo 10 illustrates the ice conditions at an oil platform in Cook Inlet.

Ice Piling

Ice piling is a problem caused by wind driven sheet ice being driven up a gently sloping shoreline. This phenomenon has not been a problem in the past in Knik Arm. However, if a causeway is built, backing up a fresh water lake, ice piling will probably occur.

An illustration of the force involved in the piling of ice is given in "Piling Up of Ice on Seashores and on Coastal Structures," by P. Bruun and A. O. Straumsnes of the Technical University of Norway. This paper states than an ice floe extending out from shore 1 km (3300 feet) and acted upon by a wind of 30 m/sec (67 mps) exerts a force on the shoreline of 250 kg/m (167 pounds per lineal foot). Bruun lists ice pile heights of 20 to 50 feet on Danish seashores, 20 to 35 feet in the Great Lakes region and 30 feet in the Alaskan arctic. These heights are much greater than expected in Knik Arm due to the smaller lake and the fact that some of the ice will be locked to the

PHOTO COURTESY OF SOLAR DIVISION OF INTERNATIONAL HARVESTER COMPANY.

Photo 10 –
Sheet ice failing against a Cook Inlet oil platform.

higher shoal areas. The 20-foot freeboard provided by the causeway design section should be more than adequate.

As regards a spillway, the 20-foot vertical face of that proposed will prevent any ice piling in this area, because deep water tends to make the ice pile form below the water level.

Rip Rap Movement by Ice

Removal of rip-rap by the "plucking" action of ice is a problem of major concern in the Anchorage area. The Alaska Railroad experiences an appreciable annual loss of rock from their roadbed along Turnagain Arm. The thick shore ice that forms on the rip-rap, and the high tides provide sufficient bouyancy to float and transport rocks weighing several tons. Many of these rocks can presently be seen scattered over the mud flats of the area.

The surest method to prevent plucking is to use massive rock which cannot be lifted by the available buoyancy. The required size is probably six tons or greater. The apparent difficulty in obtaining rock of this size in the Anchorage area may make it more economical to use smaller rock and replace it as it is lost.

The fill slopes which lose rip-rap on the Alaska Railroad appear to be quite steep, one and one-half to one or steeper. A flatter slope would require rocks to be raised higher before they can clear adjacent rocks and move laterally. This would allow smaller rock for protection against ice plucking. The proposed causeway slopes are much flatter than the existing railroad fill and might not undergo as severe a loss of rip-rap.

Icing

Icing is a phenomenon which should affect only a spillway design. The problem occurs when water flow is shallow, and slow enough to freeze completely to the bottom. Once this happens, hydrostatic pressure forces water to flow to the top of the ice where it continues to thicken until a substantial dam is built. While this ice dam probably would not cause any direct problems in winter, it could seriously raise the lake water level during the spring runoff before it melted out of the spillway crest.

Icing below the dam crest, due to the low flow and extreme spillway width, can be controlled by confining the water in a narrow channel and thus increasing the velocity.

Ice formation will be aided in the lower spillway by bank shadows from the low winter sun angle, shadows from any structures which cross the channel and ground water seepage in the cut areas.

Further Research

For the preparation of an economical crossing design, further ice study is needed. After a crossing site is selected, field observations to more accurately determine ice size and movement are required. Ice migration to Knik Arm from other areas must be determined. To refine floe failure forces, sampling and testing of native ice and construction of a test structure are indicated. One laboratory project which would yield worthwhile results would be the study of energy absorption by local crushing of ice. Laboratory testing could also be used to determine the magnitude of the hydrodynamic force associated with an ice block impact. It is recommended that further ice research be undertaken if this project extends beyond the feasibility study.

GEOLOGY

Subsurface investigations for this feasibility study were limited to a geophysical survey of Knik Arm, prepared by Dames & Moore for the Alaska State Highway Department, and one deep boring extending to elevation –135, and one very shallow boring to elevation –64, both drilled by Department forces.

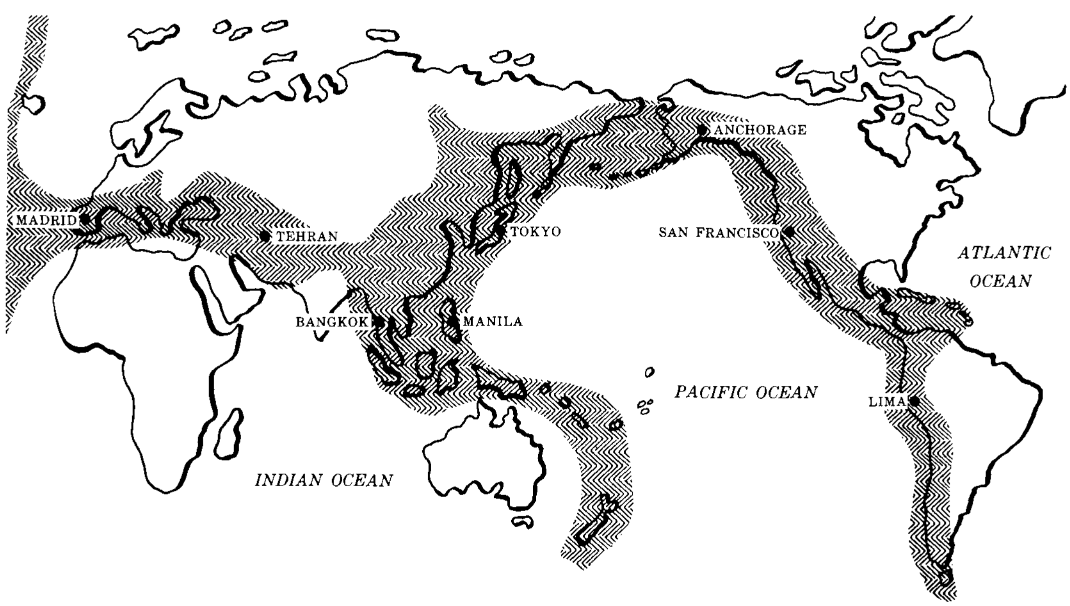
Aerial photographs flown before and after The Alaska Earthquake of March 27, 1964 were examined, as well as numberous Reports and Professional Papers written on the Earthquake. The well-drilling records and foundation explorations contained in these publications, and those furnished by governmental agencies, provide useful information for the Anchorage area in general. A four-day field inspection by a Geologist and several Soils and Foundations Engineers was conducted in early June, 1971. The results of that field inspection, and subsequent office studies, are contained in Appendix E, entitled, "Geologic and Engineering Reconnaissance, Proposed Knik Arm Crossing."

Appendix E provides an evaluation of geologic soils and foundation origins as they apply to the various types of crossing structure at the alternative locations.

EARTHQUAKE

Southern Alaska and the adjoining Aleutian Island chain together constitute one of the world's most active seismic zones. Extending from Fairbanks on the north to the Gulf of Alaska on the south, the Alaskan seismic zone is but a part of the vast, near-continuous seismically active belt that circumscribes the entire Pacific Ocean basin. See Figures III-27 and III-28. Between 1898 and May 1965, seven Alaska earthquakes have equalled or exceeded Richter magnitude 8, and more than 60 have equaled or exceeded magnitude 7. See Figure III-29. It has been estimated that about 7 percent of the seismic energy released annually on the globe originates in the Alaskan seismic zone.

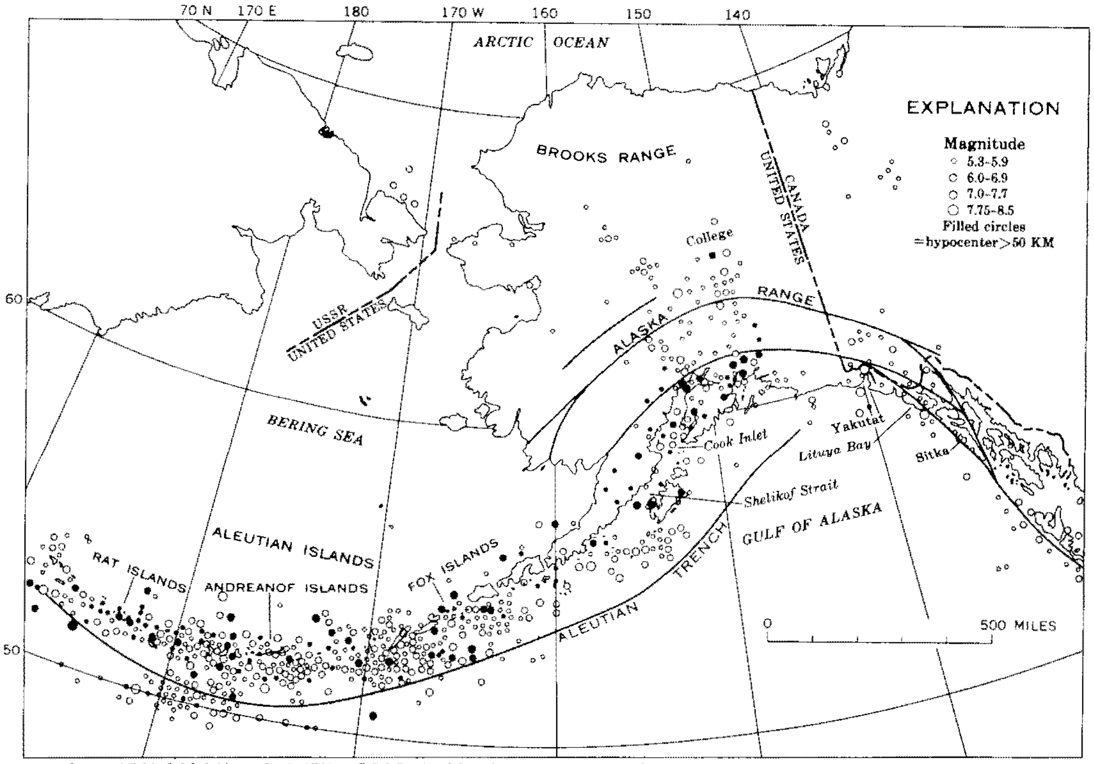
This highly active zone is circumferential to the Gulf of Alaska and parallel to the Aleutian Trench. It embraces the rugged mountainous region of southern Alaska, Kodiak, and the Aleutian Islands, the continental shelf, and continental slope of the Aleutian Trench. Most of the earthquakes originate at shallow to intermediate depths – mostly less than 50 kilometers – between the Alaskan Trench and the Aleutian Volcanic Arc. Foci are generally deeper away from the trench toward the arc.



U. S. GEOLOGICAL SURVEY PROFESSIONAL PAPER 541

These belts coincide with the earth's orogenic zones and contain most of the earth's active volcanos.

Figure III-27 EARTHQUAKE BELTS OF THE WORLD



U. S. GEOLOGICAL SURVEY PROFESSIONAL PAPER 541

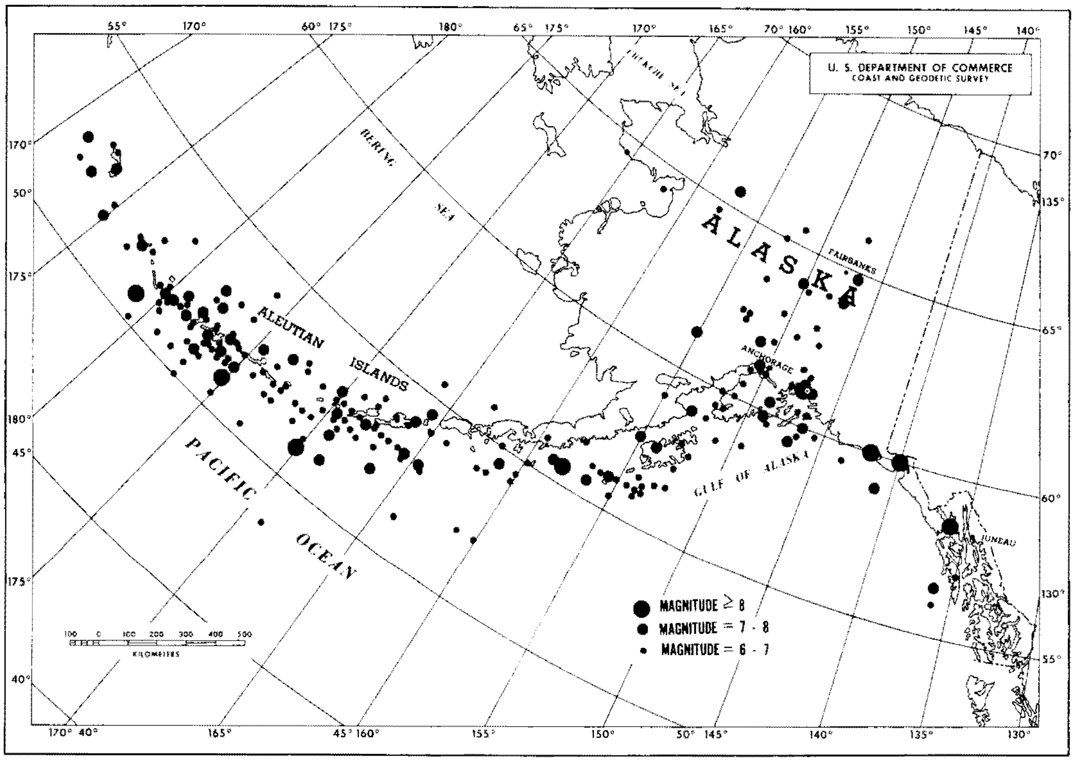
More recent earthquakes include the Alaska earthquake of 1964 (magnitude 8.3 - 8.4) and the Rat Islands earthquake of 1965 (magnitude 7.75).

Figure III-28 EPICENTERS OF MAJOR ALASKAN EARTHQUAKES, 1898-1961

The great majority of Alaskan earthquakes, as well as those throughout the world, are classified as tectonic – i.e., one which occurs when potential energy stored in crustal rocks during a long period of strain accumulation is released by faulting. Because faulting takes place along lines of weakness in the earth's crust, the geologic and tectonic setting of Alaska and the Aleutian Islands is basic to an understanding of the seismic history of Alaska, and more particularly the Knik Arm and Anchorage area.

Regional Geology

Knik Arm is part of the Cook Inlet Lowland physiographic province. The Cook Inlet Lowland is a structural trough underlain by rock of Mesozoic and Tertiary age, and mantled largely by Quaternary glacial deposits of variable thickness. In the Knik Arm area the lowlands are bordered to the southeast and east by the Chugach Mountains to the north by the Talkeetna Mountains and to the west by Mount Susitna. These mountains are underlain by Jurassic and younger metavolcanics and graywacke, locally intruded by granite, gabbro, and peridotite.



VOL. 1, U. S. DEPARTMENT OF COMMERCE PUBLICATION 10-3 (C. & G S.).
"THE PRINCE WILLIAM SOUND, ALASKA EARTHQUAKE OF 1964 AND AFTERSHOCKS"

Figure III-29 EARTHQUAKES WITH MAGNITUDE ≥ 6.0, 1899-1964

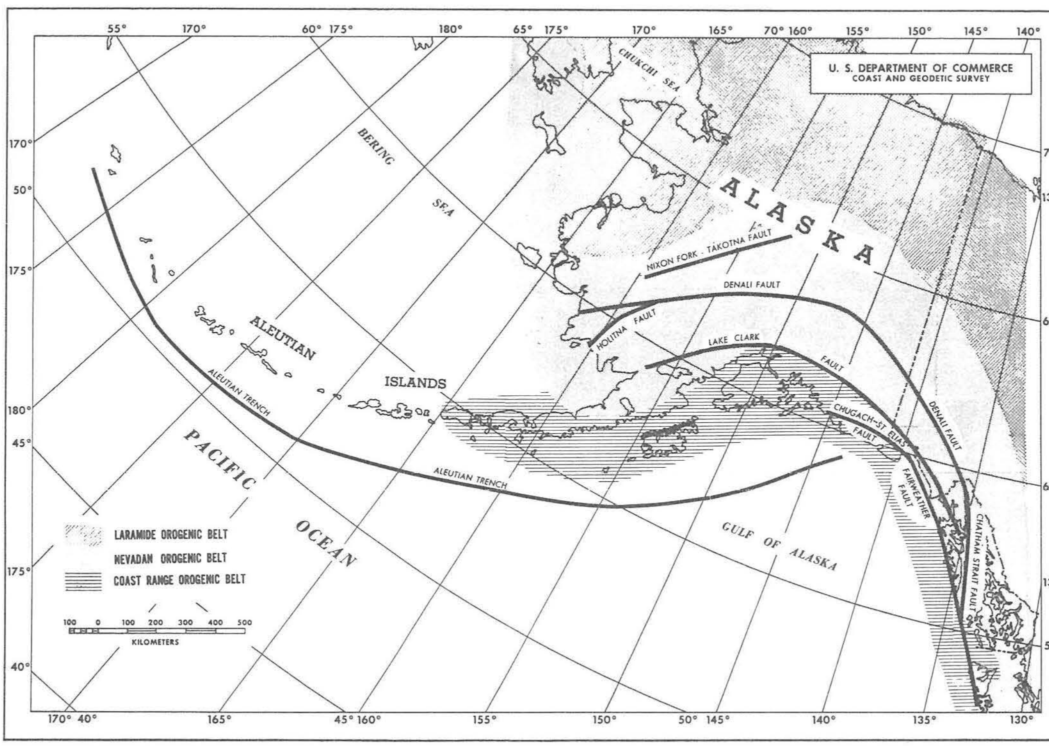
The topography of the lowland is primarily the result of five major Pleistocene glaciations and two post-Pleistocene glacial advances. Information from deep wells indicates the total thickness of glacial deposits in the Anchorage area is over 468 feet near Fort Richardson and over 778 feet at Elmendorf Air Force Base. Oil test wells indicate that glacial deposits of over 1,000 feet in thickness exist in some areas within the region.

Soil units of the Knik glaciation include the Bootlegger Cove Clay, and a lower till which outcrops beneath the Bootlegger formation south of the study area. Deposits of Naptowne glaciation overlie the Knik deposits and comprise the majority of the soil units exposed in the east and west bluffs of Knik Arm north of Anchorage.

Tectonic Setting

Since upper pre-Cambrian time, all orogenesis in Alaska has been in belts parallel or subparallel to the present margin of the continent. There are three orogenic belts recognized in Alaska: The Laramide, the Nevadan, and the Coast Range. See Figure III-30. The Coast Range orogenic belt is one link in the circum-Pacific belt of active orogeny. The belt is widest in the Prince William Sound and Cook Inlet region where it includes the Chugach and Kenai Mountains. Cenozoic structures within the Coast Range belt are gentle to well-developed folds in the Alaska Penninsula and Cook Inlet regions.

The major fault systems in Alaska are also shown in Figure III-30. The Lake Clark Fault is of particular interest to the Knik Arm region. The alignment of this fault is based on geomorphic evidence and is supported by field investigations for only a small segment of its total length. Study of the Lake Clark Fault over a portion of its length indicates right-lateral offset between equivalent rocks on the north and south sides of Lake Clark. The movement includes beds from Lower Paleozoic to Tertiary. Conclusion that this fault has been active in relatively recent time is supported by seismological evidence of movement along the fault during the August, 1948 earthquake.



VOL. 1, U. S. DEPARTMENT OF COMMERCE PUBLICATION 10-3 (C. & G. S.).
"THE PRINCE WILLIAM SOUND, ALASKA EARTHQUAKE OF 1964 AND AFTERSHOCKS"

Figure III-30 OROGENIC BELTS AND MAJOR FAULT SYSTEMS IN ALASKA

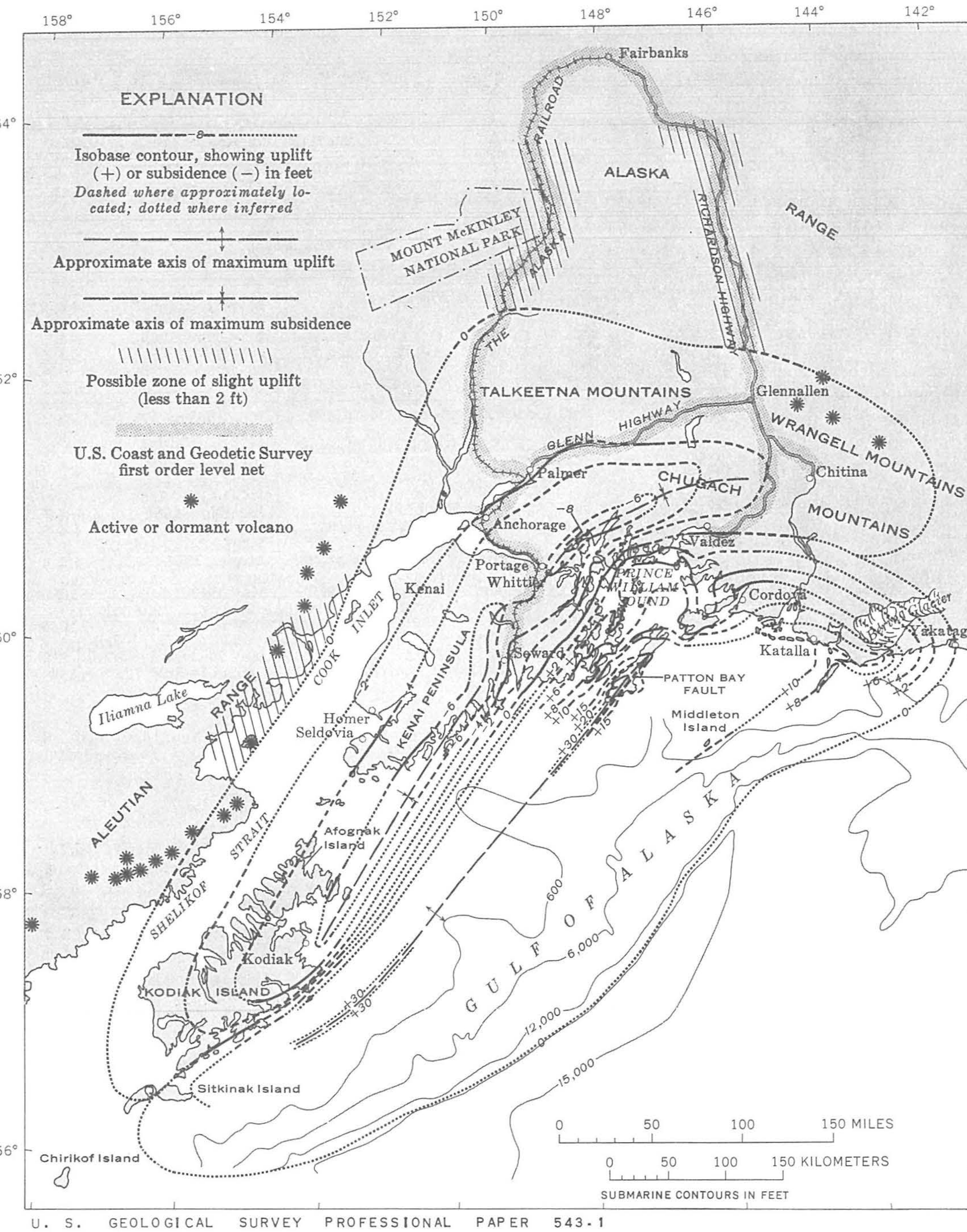
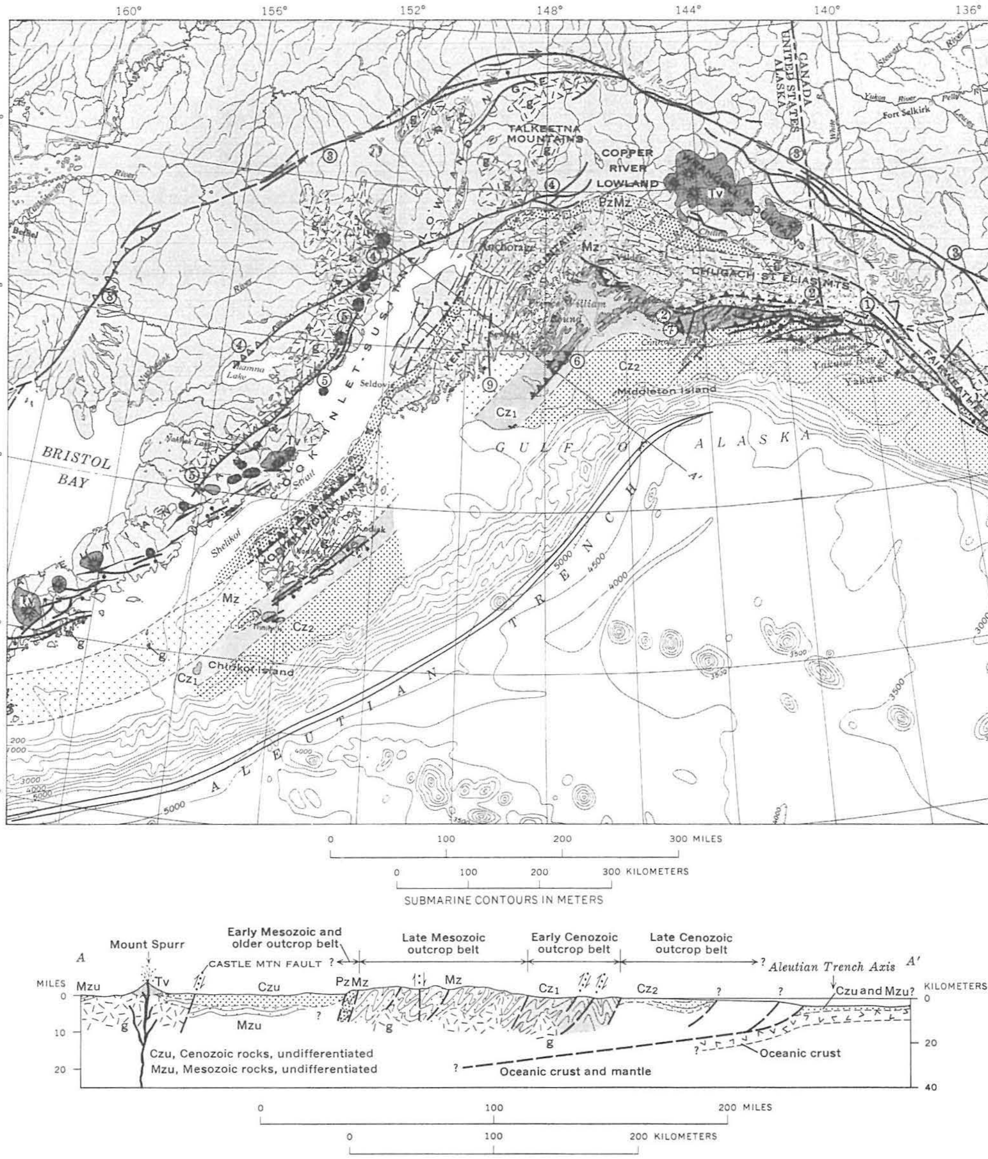


Figure III-31
DISTRIBUTION OF TECTONIC UPLIFT AND SUBSIDENCE IN SOUTH-CENTRAL ALASKA*

*Changes resulting from March, 1964 earthquake



EXPLANATION

Tv — Andesitic extrusive rocks of active or dormant volcanoes

Cz2 — Late Cenozoic bedded rocks
Lighter pattern where projected offshore

Cz1 — Early Cenozoic bedded rocks
Lighter pattern where projected offshore

Mz — Late Mesozoic bedded rocks
Lighter pattern where projected offshore

PzMz — Paleozoic and early Mesozoic bedded rocks
Lighter pattern where projected offshore

g — Granitic plutonic rocks

Undifferentiated rocks

Approximate contact
Includes possible fault contacts. Dashed where inferred or concealed

Thrust or reverse fault
Dashed where inferred. Sawteeth on upper plate. Open teeth indicate major fault

Steeply dipping fault
Dashed where inferred. Arrows indicate relative lateral displacement; bar and ball on relatively downthrown side

Trend lines showing strike of bedding, schistosity, and folds

Major faults and faults with known Holocene movement
Asterisk indicates known Holocene movement; double asterisk indicates historic movement

No	Fault	Data Source
1**	Fairweather	Tocher (1960); Tarr and Martin (1912); Plafker (1967)
2.	Chugach—St Elias (probable Holocene movement)	Miller and others (1959, p. 42); Plafker (1967)
3*	Denali	St. Amand (1957); Hamilton and Myers(1966); Grantz (1966)
4*	Castle Mtn-Lake Clark	Martin and Katz (1912, p. 72–75); Kelly (1963, p. 289); Grantz (1965, sheet 3)
5.	Bruin Bay	Burke (1966, p. 139); R. L. Detterman, (oral commun., 1967)
6**	Patton Bay and Hanning Bay	Plafker (1968)
7*	Ragged Mtn	Miller (1961)
8*	Holitna-Togiak	Hoare (1961, p. 608–610)
9.	Kenai lineament (possible 1964 movement)	This paper

Generalized tectonic map and idealized vertical section showing selected rock units and structural features of south-central Alaska. Indicated displacement direction on faults is the net late Cenozoic movement only. Geology modified from a manuscript tectonic map of Alaska by P. B. King and from unpublished U.S. Geological Survey data; the thickness of crustal layers and the structure shown in the section are largely hypothetical.

U. S. GEOLOGICAL SURVEY PROFESSIONAL PAPER 543-I

Figure III-32 GENERALIZED TECTONIC MAP AND IDEALIZED VERTICAL SECTION

Recent Seismic Activity

One of the greatest geotectonic events of our time occurred in southern Alaska March 27, 1964. The magnitude of this earthquake has been computed to be 8.3–8.4 on the Richter Scale. It was accompanied by crustal deformation – including warping, horizontal distortion, and faulting – over probably more than 110,000 square miles of land and sea bottom in south central Alaska. The distribution of tectonic movement is shown in Figures III-31 and III-32. In the Anchorage area, including portions of Knik Arm, damage was caused by direct seismic vibration, by ground cracks, and by landslides. Landslides caused the most devastating damage.

Triggering of landslides by the earthquake was related to the physical-engineering properties of the Bootlegger Cove Clay, a glacial lacustrine or estuarine-marine deposit that underlies much of the Anchorage area. The Bootlegger Cove Clay has generally low shear strength (ranging from 0.2 to 0.5 t.s.f.), high water content and high sensitivity (ranging from about 10 to 40). In addition, it contains thin layers and lenses of fine, loose sand. The extensive sliding is generally attributed both to liquefaction of these sand layers and to loss of strength under cyclic loading of the clay itself.

The geographic location of Knik Arm, in one of the world's most active seismic zones, and the presence of vibration sensitive glacial deposits in the Knik Arm region, combine to make this area one of considerable concern with respect to the design and construction of engineering structures to perform satisfactorily under earthquake conditions. Design studies for bridge structures as well as earth structures in this region will necessarily have to be executed with caution, with recognition of the current State of the Art for seismic designs. Following additional subsurface explorations, dynamic analyses will need to be conducted to develop and evaluate design criteria to insure the proper balance between cost and risk.

ENVIRONMENT

Description of Significant Elements of Knik Arm Ecosystem

Knik Arm is an estuarial bay of Cook Inlet under heavy tidal influence which is essentially drained twice daily under even moderately low tides. It is fed principally by the Knik and Matanuska Rivers, both of which carry moderate to heavy glacial silt loads. Because of these factors, the arm is floored with material which does not support bottom dwelling organisms, nor are resident fishes prevalent in the tidal area. The streams flowing into the estuary support minor populations of Dolly Varden "trout" and grayling. Only one major salmon stream exists. This is Fish Creek, draining Big Lake, and supporting a run of approximately 15,000 red salmon annually which the Alaska Department of Fish and Game believes to be about one-tenth of its potential. Minor numbers of pink, coho and king salmon run in the other streams.

Moderate numbers of waterfowl breed in the peripheral areas above the high tide mark. Some of the ducks appear to be local and are hunted for about a two-week period in September in the Palmer Flats area at the extreme upper end of the arm. The area around the perimeter of the arm is heavily utilized recreationally and is considered to be a quite important wildlife area. Large populations of moose abound, with over 800 being counted in one valley facing into the arm. A recreational fishery, principally in the Knik and Matanuska Rivers exists for Dolly Varden, grayling and nonresident planted rainbow trout. Occasionally, large marine mammals such as seals, Beluga whales and walrus, enter the arm at high tide.

There are no parks, recreation areas or refuges on Knik Arm and in the arm itself there is no recreational boating, fishing, swimming, water skiing, or shore-based recreation dependent upon water. This is largely due to the extreme tidal influence which causes water to be present at the shoreline for only a brief period each day. An old Indian cemetery at the village of Eklutna is one historic site near the Arm.

Evaluation of Project Impact on the Knik Arm Ecosystem

The selection of the causeway dam alternate would create a very large fresh water lake, and for this evaluation it is presumed that such a lake would be stable and subject to very little surface fluctuation. On this basis, the following observations are made:

1. The lake would be frozen from about mid-October to early June each year, which is longer than the present ice coverage period. This could have an effect on the fragile agricultural climate of the Matanuska farms.

2. Any effect on waterfowl populations would probably be salutary inasmuch as the flats at the upper end would still be available for breeding, nesting and hunting, and some permanent islands giving nesting protection would be formed.

3. A fish ladder would need to be constructed to pass anadromous fish over the dam.

4. The lake could become a nursery area for red or sockeye salmon, resulting in an expanded population of these fish spawning in the Knik and Matanuska Rivers, as well as minor streams of the area not presently supporting this species.

5. Resident populations of other desirable fish such as rainbow trout could be developed within the impoundment.

6. Terrestrial mammals would be affected to a certain extent. The proposed lake would permanently inundate about 20 square miles of flats which are covered with salt grass and provide winter browse for the moose population. This area is generally swept clear of snow by the winter winds.

7. From the standpoint of general recreation, the lake would supply an area close to Anchorage of outstanding value for boating and fishing. A large number of recreational areas would be available for shore development of much greater value than is presently possible.

The construction of a bridge across Knik Arm would have no direct effect on the natural environment of the Arm during or after construction. However, there would be considerable impact on the area west of Knik Arm associated with the opening of the area to residential and industrial development. The ramifications of this development are beyond the scope of this study.

SECTION IV

alternative crossing locations and types

alternative crossing locations and types

ALTERNATIVE CROSSING LOCATIONS

Within the preset limits of the study area, Point Mackenzie to Eagle River, six possible crossing locations, Crossings 0 through V, were chosen for study. One additional location, Crossing VI, just outside the study limits, was added later. The locations are shown on Figure IV-1 and the main features of each crossing are tabulated below in Table IV-1.

Table IV-1 COMPARISON OF CROSSING LOCATIONS

Crossing	Length Shore To Shore (Miles)	Maximum Water Depth* (ft.)	Distance From Ship Creek (Miles)	Possible Crossing Types	Most Favorable Crossing Type
0	2.4	123	5.0 SW	Bridge, Tunnel	Bridge
I	2.8	67	0.6 SW	Bridge, Tunnel	Tunnel
II	2.6	82	0.0	Bridge, Tunnel	Tunnel
III	1.5	133**	2.1 NE	Bridge	Bridge
IV	2.4	56	3.6 NE	Bridge, Tunnel, Causeway Dam Combination Bridge & Causeway	Bridge or Tunnel
V	4.9	40	7.7 NE	Bridge, Tunnel, Causeway Dam, Combination Bridge-Causeway	Causeway Dam
VI	3.6	40	12 NE	Causeway Dam	Causeway Dam

*Below Mean Lower Low Water - Source: U.S.C.&G.S. Map 8557
**Dames and Moore geophysical explorations show 160'±

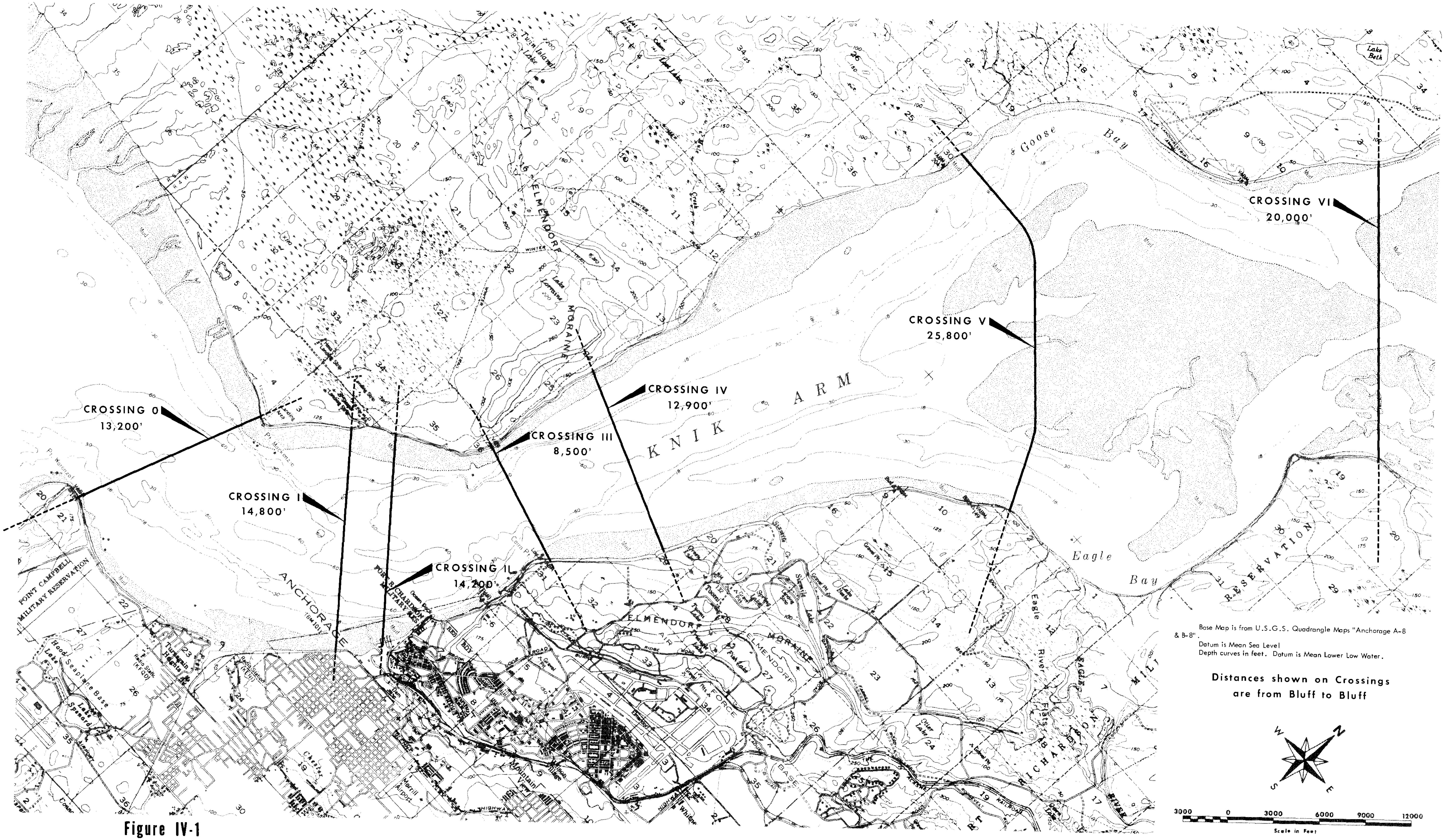


Figure IV-1

ALTERNATIVE LOCATIONS OF CROSSING

Crossing 0

A crossing from Point Woronzof to Point Mackenzie, the southernmost line considered, has a sufficient number of advantages to be considered among the apparently feasible crossing locations.

The most obvious advantage to this crossing is the connection of two outstanding points of land at the immediate upper end of Cook Inlet. The presence of a connection from the west side of Knik Arm to the vicinity of the International Airport could be considered an advantage as could the scenic value of a bridge forming a gateway to the Knik Arm–Anchorage area.

However, both Points Woronzof and Mackenzie have in the past shown indications of poor slope stability and the presence of a Bootlegger Cove type of clay near Point Woronzof has been established. This poor stability coupled with the known presence of relatively large boulders on the Arm bottom eliminate a tunnel from consideration. Navigation requirements and depth of water eliminate from consideration the use of a causeway dam.

A bridge at this location would also need to contend with the navigation clearance requirements and the ever present possibility that a ship will lose control in the tidal currents and collide with a pier. In this event, the pier would probably be destroyed and the bridge would be out of service for some time. In addition, the proposed north–south runway addition to International Airport and the necessary glide path clearances thereto must be considered. If the navigation span is located near the north end of the bridge, conflict with the runway glide path could probably be avoided. If the navigation span is located near the south end of the bridge, in the vicinity of the existing ship route (U.S. Coast Pilot 9, 7th Ed., 1964), a conflict is possible and Federal Aviation Administration regulations must be reviewed.

From the standpoint of general traffic service to the Anchorage area this crossing location is undesirable.

Comparing the apparent advantages and disadvantages it appears that a crossing at this location would not be in the best interest of all concerned, and therefore detailed consideration beyond that necessary to arrive at an approximate cost has not been given in this study.

Crossings I and II

A highway crossing of Knik Arm at either of these locations would afford direct access to the City of Anchorage from the west side of the Arm.

The locations shown have been selected for minimum water depth in this general area, as there is not a significant change in the length of the crossing at any point in this vicinity. Crossing I, though longer, traverses shallower water and for this reason is favored over Crossing II. Also, the eastern abutment of Crossing I might be slightly better than that for Crossing II.

A causeway at these locations would result in serious interference with shipping and therefore was eliminated from consideration. The use of a bridge at either of these locations was considered undesirable because of the navigation clearance requirements and the location of the crossings either in or adjacent to the turning basin for the various docks at Anchorage. The presence of high tidal currents in a maneuvering area increase the possibility of a ship losing control. As indicated for Crossing 0, the impact of a ship on a pier would probably take the bridge out of service for an extended period. For these reasons a tunnel, which would provide much less restriction to shipping, is favored for these crossings.

The eastern termini are located in areas known to be underlain by the Bootlegger Cove clay which in the event of an earthquake is suspect. Near the west termini recent slides have been noted at the Sleeper Landing Strip. It can be noted that the maximum depth of water at these locations is greater than that at Location IV which will cause greater difficulty of construction and greater cost per foot than the tunnel proposed for Crossing IV. In addition, these alternates, even though they terminate immediately at the City of Anchorage, can not satisfactorily be integrated into the highway and street system without undue congestion.

As in the case of Crossing 0, Crossings I and II were not given further, detailed consideration.

Crossing III

Crossing III, at Cairn Point, has historically been the crossing site favored by residents of Anchorage. The location would furnish good traffic service to both Anchorage and the west side of Knik Arm. Highway connections can be readily provided and the lack of navigation requirements at the site permit a lowering of the structure from the elevation required at Crossings 0, I and II.

This location provides the shortest overwater crossing available within the limits of the study, but, as could be expected, the deepest water also occurs at this location. Tunnel or causeway dam construction at this location would be extremely difficult due to the depth of water and the flow conditions. Also, a causeway dam would be a potential source of problems in the vicinity of the Anchorage port facilities due to changes in the tidal flow patterns which in turn would cause changes in siltation rates, ice movements, etc.

The water depths, up to 175 feet, fast currents and questionable foundation material at this location make substructure construction very difficult and expensive, indicating the need for a structure with the minimum number of piers. It was concluded that the only type of structure applicable to the conditions at this location is a long span suspension bridge.

However, the east bridge tower, at Cairn Point, while not directly in the glide path of the main east–west runway of Elmendorf Air Force Base, will protrude into the air space in such a manner as to require a variance from the Federal Aviation Administration prior to further studies.

As a part of this study, detailed consideration was given to a suspension bridge at this location in order that the cost and feasibility of such a structure might be determined.

Crossing IV

Crossing IV, located 1-1/2 miles north of Cairn Point, is situated at the southernmost site which combines reasonable water depth, relatively uniform flow conditions, relatively short overwater length and acceptable traffic service to both sides of the Arm. There are no navigation requirements and the resulting low bridge profile avoids conflict with the Elmendorf Air Force Base air space.

Bridge, tunnel, causeway dam, or combination bridge–causeway construction is considered possible at Crossing IV. However, a combination bridge –causeway appears inadvisable due to the high current velocities that would be induced through the bridge opening as a result of the normal tidal fluctuation. Consequently, the bridge, tunnel, and causeway dam alternates were reviewed in detail as a part of this study.

Crossing V

This location, immediately south of Eagle Bay and Goose Bay, is very near the preset northern limit of the study area. The alignment at this location was adjusted to cross the Eagle Bay and Goose Bay deep water channels at a minimum skew while utilizing the shoals area to the greatest possible degree. Traffic service to the Anchorage area is marginal, but highway connections can be made with relative ease.

The four structure types listed for Crossing IV are also applicable at this location. The tunnel was excluded from further consideration by reason of the great length of the crossing and the obvious prohibitive cost of a tunnel of that length.

Crossing VI

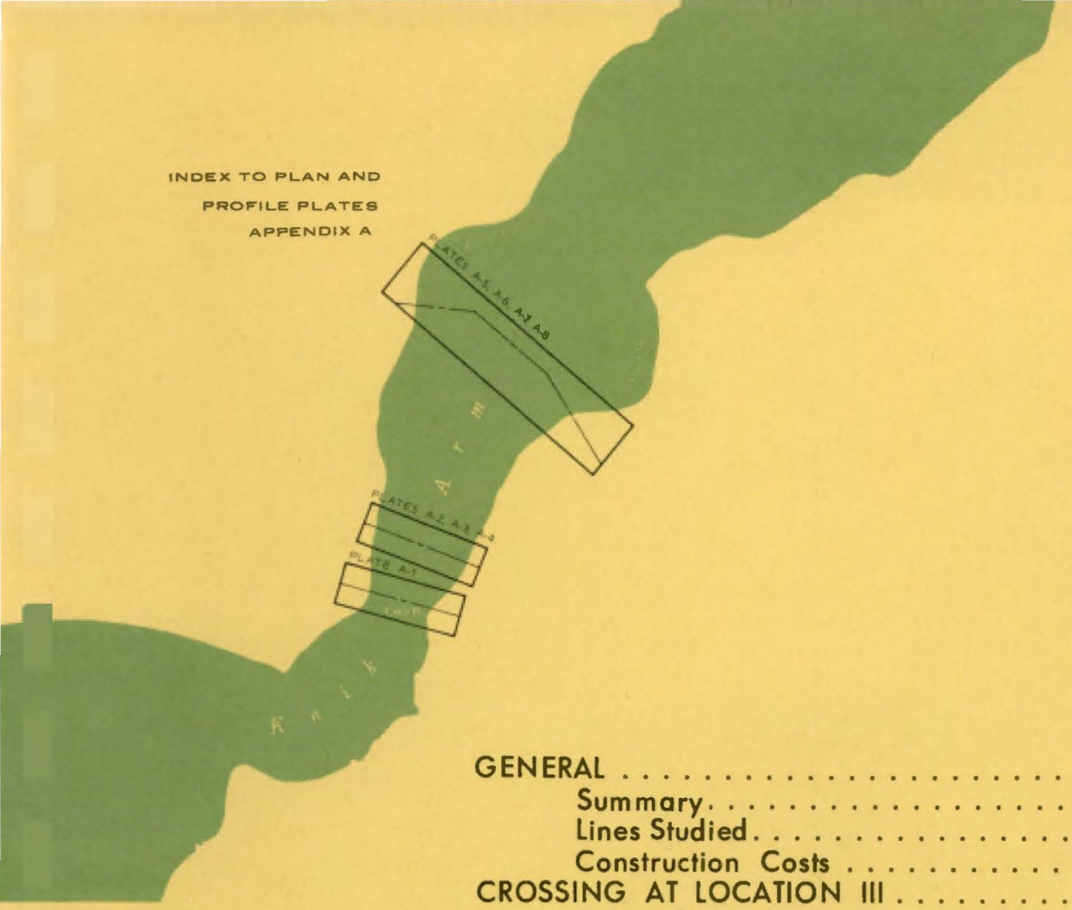
This location lies north of the preset study limits but since it is regarded as the most favorable location for a causeway dam type of crossing, a brief study of a causeway dam at this location has been included. The crossing is relatively short, encounters shallow water and the terrain is favorable for spillway construction. A major drawback of the crossing site is that it does not provide access to the area south of Goose Creek; it would serve an area which is already accessible by road, and the eastern terminal is too far from Anchorage to provide reasonable traffic service.

ALTERNATIVE CROSSING TYPES

Structure types given consideration during the course of this study were tunnel, both buried and floating, floating bridge, conventional bridge, full closure causeway dam and combination bridge–causeway.

The floating bridge concept was discarded due to the ice conditions and the extreme tidal fluctuations. Likewise, a floating tunnel was considered to have no construction or cost advantages over a buried tunnel.

Detailed study has been given to bridge and causeway dam alternates on Lines IV and V with special attention to solutions for the novel problems associated with environment, earthquake and extreme tide and ice conditions. The following sections of this report detail the economics and structural aspects of the various crossing structures.



INDEX TO PLAN AND
PROFILE PLATES
APPENDIX A

Page

SECTION V

bridge crossings

bridge crossings

GENERAL

Summary

Based upon the review of site conditions, preliminary designs and cost estimates presented in this report, Location IV is the most favorable site for a bridge crossing of Knik Arm.

The main structure would be a double-deck, orthotropic Warren truss with spans of 407.ft.

An adjustment in this 407-ft. span length might be required in subsequent stages of design, but the overall cost of construction should remain the same.

The estimated construction cost of the bridge, including 10% for contingencies and variations, is $126,000,000.

Lines Studied

Lines I and II were not favored as bridge sites due to cost, expected unstable soils and poor traffic service. Cost estimates for bridge crossings at these locations are not given in this report.

Lines III, IV and V were selected as suitable locations for a bridge crossing. Preliminary designs were made for these sites and a detailed quantity and cost summary is given for each site.

Line 0 was studied as a possible bridge site but it is not favored due to cost, expected unstable soils, navigational clearance requirements, length over deep water, more severe exposure to climatic conditions and probable interference with air traffic from the International Airport. An approximate cost of construction is presented herein.

Construction Costs

Quantities of materials were obtained from preliminary designs for each site studied. These reflect bridge type, span length, pier type and channel depth.

Insofar as allowed by the preliminary status of the designs, contractor-type estimates were made for the bridge structures (see Appendix D). Where relevant recent Alaska bid prices were available, these were used for comparative purposes.

A mobilization cost has also been estimated for each site. This item covers the anticipated cost of pier access dredging, caisson rings, anchor lines, dock and yard facilities and other incidental items required to begin construction of structures of this magnitude.

The cost evaluations presented herein are considered adequate for comparisons.

CROSSING AT LOCATION III

General

From the standpoints of convenience, travel distance and length of crossing, a highway crossing of Knik Arm at its narrowest width is quite attractive.

There are some difficult features of this location, however, which also must be considered. One is the extreme depth of water (maximum depth is 175 ft. below mean sea level) which eliminates short and medium span bridges from consideration for practical and economic reasons and imposes the requirement of a long span suspension bridge. Another is the existence of a landslide area near the west abutment. This latter feature, which in other literature is known as the Sleeper Landing Slide, can be noted on Photo 8, Page III-61, on the unnamed point to the left center of the photo.

While there is a question concerning the stability of some of the deposits at this site, as evidenced by the landslide area, from geological data available it can be inferred that the narrows at Cairn Point are at the southern edge of the Naptowne glacial deposits. The very existence of the constriction infers that the deeper underlying subsurface materials are compact and relatively stable. Whether they are adequate to support the piers and anchorages for a monumental suspension bridge can only be verified by subsurface explorations.

The cable anchorages assumed for the suspension bridge are of the gravity type. A factor of safety against sliding for the anchorages cannot be determined at this phase of the study. Extensive subsurface investigation at the anchorages would be required to determine such critical properties as shear strength, the possibility of creep under the sustained anchorage loads and the consequence of seismic loads upon the bridge anchorages. The preliminary design which is the basis for the cost estimate in this report, assumes an effective coefficient of friction for the anchorage and underlying soil of 0.3. This value is consistent with values used for other major suspension bridges utilizing gravity anchorages and is considered adequate for estimating purposes. Should borings, however, disclose the presence of the Bootletter Cove clay extending to an appreciable depth below sea level, major modifications in the preliminary design would be required.

The topography of the site dictates that a 4,500′ main span is necessary.

This main span length surpasses all existing suspension bridges and would place the Knik Arm Suspension Bridge in a class with such great structures as the Golden Gate Bridge and Verrazano Narrows Bridge.

The Golden Gate Bridge has a main span of 4,200′ and was opened to traffic in 1937. The Verrazano Narrows Bridge has a main span of 4,260′ and was opened to traffic in 1964. Presently, design is being completed on the Humber River Bridge in England with construction to commence in 1972. This suspension bridge will have a 4,580′ main span.

Such a crossing at this site is considered a feasible alternative at this time but is not the most favorable solution due to its relatively high cost of construction.

Layout

The layout of the structure, as shown on Plate A-1, Appendix A, is determined by the location of submerged shoals extending from the east shore and the least distance to a reasonable depth of water on the west side. These considerations fix centerline direction and the location and length of main span. Side and approach spans are dimensioned to reach the bluffs at each shore.

Features of Superstructure

Suspended Spans :	1,500, 4,500, 1,500	= 7,500′
West Approach :		= 330′
East Approach :		= 1,060′
Total Length of Open Structure		8,890′

Cable Sag – Main Span :	420′
Distance Ctr. to Ctr. of Cables:	70′-0″
Roadway :	2 @ 30′-0″
Safety Curbs :	None
Roadway Deck :	Orthotropic steel with asphalt wearing surface
Stiffening Structure: :	54″ deep Warren truss with 50′ panel length
Lateral Bracing :	K-type, top and bottom of stiffening truss
Main Towers :	Cellular box legs with trussed box member bracing; 70′ c. to c. at top, 83′ c. to c. at base; tower legs 27′ x 10.5′ at top, 35′ x 10.5′ at base; tower height 551′

Principal Materials :

Main Cables – High-strength wire

Suspenders – Galvanized bridge rope

Cable Bands – Galvanized steel castings

Roadway Deck, Stringers, Railings,
Stiffening Trusses, Towers, and Girder
Approaches – ASTM A588 Weathering Steel

Features of Substructure

Main Piers :	Cellular, open dredged caissons of reinforced concrete, surmounted by oblong reinforced concrete pedestals
Anchors :	Gravity anchors of concrete filled with sand and located in the bluffs

Approach Piers	:	Reinforced concrete shafts with footings supported on circular pipe piles
Abutments	:	Stub type, supported on steel H piles

Construction Methods

The construction of the superstructure is anticipated to be by the conventional methods employed for the erection of large suspension bridges. Due to the winter extremes, erection will need careful timing to insure that the structure is not left in a vulnerable state during work stoppage.

The type of substructure proposed is necessarily based on sparse subsurface data. Subsequent explorations could drastically alter these proposals.

Construction of the anchors on land should not offer any formidable obstacles and work could proceed year-round. The use of the Benoto trench method of wall construction could be used to greatly reduce the amount of sheeting and shoring required for excavation.

The west main pier presents, perhaps, the greatest construction challenge. Due to the water depth and swift currents, the usual sand island method of construction is probably not feasible. At present, the two possible methods are believed to be: (1) The use of a sand island with walls comprised of large cellular cofferdams, or (2) A floating steel cofferdam enclosed in a braced stall. For either of these methods it may be desirable to place fill on the bottom to decrease the depth of water.

The east main pier should be less of a problem, though borings may show extensive cobbles and boulders which could make caisson sinking difficult.

The approach piers on the east side have been estimated as constructed in braced steel cofferdams with tremie placed seals. Here, also, surface observations at low tide indicate extensive boulders and cobbles and the driving of sheeting and piles may not be possible.

Estimated Construction Cost

The estimated construction cost for a Cairn Point Bridge Crossing including 10% for contingencies and variations, is $249,000,000. This estimate includes slope stabilization costs of about $26,000,000 which probably will be required. The detailed estimate follows:

Table V-1

ESTIMATE OF COST SUSPENSION BRIDGE AT CROSSING III

Spans : 2 @ 165', 1,500', 4,500', 1,500', 4 @ 265' = 8,890'
Roadway: 2 @ 30'-0"

MAIN BRIDGE						
Superstructure						
Structural Steel (A588)	76,301,000	Lbs.	@	$ 0.70	=	$ 53,410,700
Main Cables	41,980,000	Lbs.	@	0.90	=	37,782,000
Cable Wrapping	761,000	Lbs.	@	0.90	=	684,900
Cable Suspenders	1,442,000	Lbs.	@	2.30	=	3,316,600
Cable Bands and Fittings	1,384,000	Lbs.	@	1.70	=	2,352,800
Hand Ropes	17,800	L.F.	@	4.50	=	80,100
Roadway Protective Coating	53,000	S.Y.	@	2.30	=	121,900
Roadway Wearing Surface	53,000	S.Y.	@	18.40	=	975,200
Handrail	15,000	L.F.	@	23.00	=	345,000
Guardrail	7,500	L.F.	@	23.00	=	172,500
Sub-Total Superstructure						$ 99,241,700
Anchorages						
Concrete	124,400	C.Y.	@	$180.00	=	$ 22,392,000
Sand Fill	60,600	C.Y.	@	4.50	=	272,700
Reinforcing Steel	10,000,000	Lbs.	@	0.37	=	3,700,000
Eye Bars	3,120,000	Lbs.	@	0.70	=	2,184,000
Structural A36 Steel	1,530,000	Lbs.	@	0.70	=	1,071,000
Cast Steel	332,000	Lbs.	@	1.70	=	564,400
Forged Steel	286,000	Lbs.	@	1.70	=	486,200
Excavation Above Elev. 100	1,775,000	C.Y.	@	0.90	=	1,597,500
Excavation Below Elev. 100	660,000	C.Y.	@	2.30	=	1,518,000
Granular Backfill	440,000	C.Y.	@	5.50	=	2,420,000
Sub-Total Anchorages						$ 36,205,800
Main Piers						
Concrete in Shafts	25,800	C.Y.	@	$180.00	=	$ 4,644,000
Concrete in Caissons	62,500	C.Y.	@	275.00	=	17,187,500
Concrete in Seals	24,100	C.Y.	@	180.00	=	4,338,000
Reinforcing Steel	7,100,000	Lbs.	@	0.37	=	2,627,000
Excavation	128,700	C.Y.	@	37.00	=	4,761,900
Sub-Total Main Piers						$ 33,558,400
End Piers and Approach Spans						
Handrail	2,800	L.F.	@	$ 23.00	=	$ 64,400
Guardrail	1,400	C.Y.	@	23.00	=	32,200
Deck Concrete	2,060	C.Y.	@	460.00	=	947,600
Reinforcing Steel	2,300,000	Lbs.	@	0.37	=	851,000
Structural Steel (A588)	5,872,000	Lbs.	@	0.70	=	4,110,400
Abutment Concrete	250	C.Y.	@	230.00	=	57,500
Pier Concrete	7,300	C.Y.	@	275.00	=	2,007,500
Seal Concrete	7,940	C.Y.	@	180.00	=	1,429,200
Excavation	7,200	C.Y.	@	55.00	=	396,000
14HP73 (60' Long)	6,000	L.F.	@	35.00	=	210,000
18" Ø Pipe Piles (90' Long)	34,800	L.F.	@	40.00	=	1,392,000
Sub-Total End Piers and Approach Spans						$ 11,497,800
Slope Protection						
Excavation Above Elev. 100	500,000	C.Y.	@	$ 0.90	=	$ 450,000
Excavation Below Elev. 100	700,000	C.Y.	@	1.65	=	1,155,000
Sand and Gravel Bedding	50,000	C.Y.	@	18.50	=	925,000
Rock Spalls Bedding	50,000	C.Y.	@	27.50	=	1,375,000
1'-3' Rock Bedding	75,000	C.Y.	@	37.00	=	2,775,000
Armor Stone (Tetrapods)	250,000	C.Y.	@	73.50	=	18,375,000
Toe Drain	2,000	L.F.	@	320.00	=	640,000
Sub-Total Slope Protection						$ 25,695,000
Total for Structure						$206,198,700
Mobilization						20,000,000
Total for Crossing						$226,198,700
Contingencies and Variations (±10%)						22,801,300
TOTAL ESTIMATED CONSTRUCTION COST						$249,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

CROSSING AT LOCATION IV

General

Location IV was selected as being the southernmost and, thus, the shortest, crossing for which flow conditions in Knik Arm are relatively uniform. The maximum depth of water is about 70 ft. below mean sea level.

A crossing located northward at, or beyond, Sixmile Creek would offer no significant decrease in water depth and would be considerably longer, while a southward shift would decrease the crossing length but would place it over substantially deeper water.

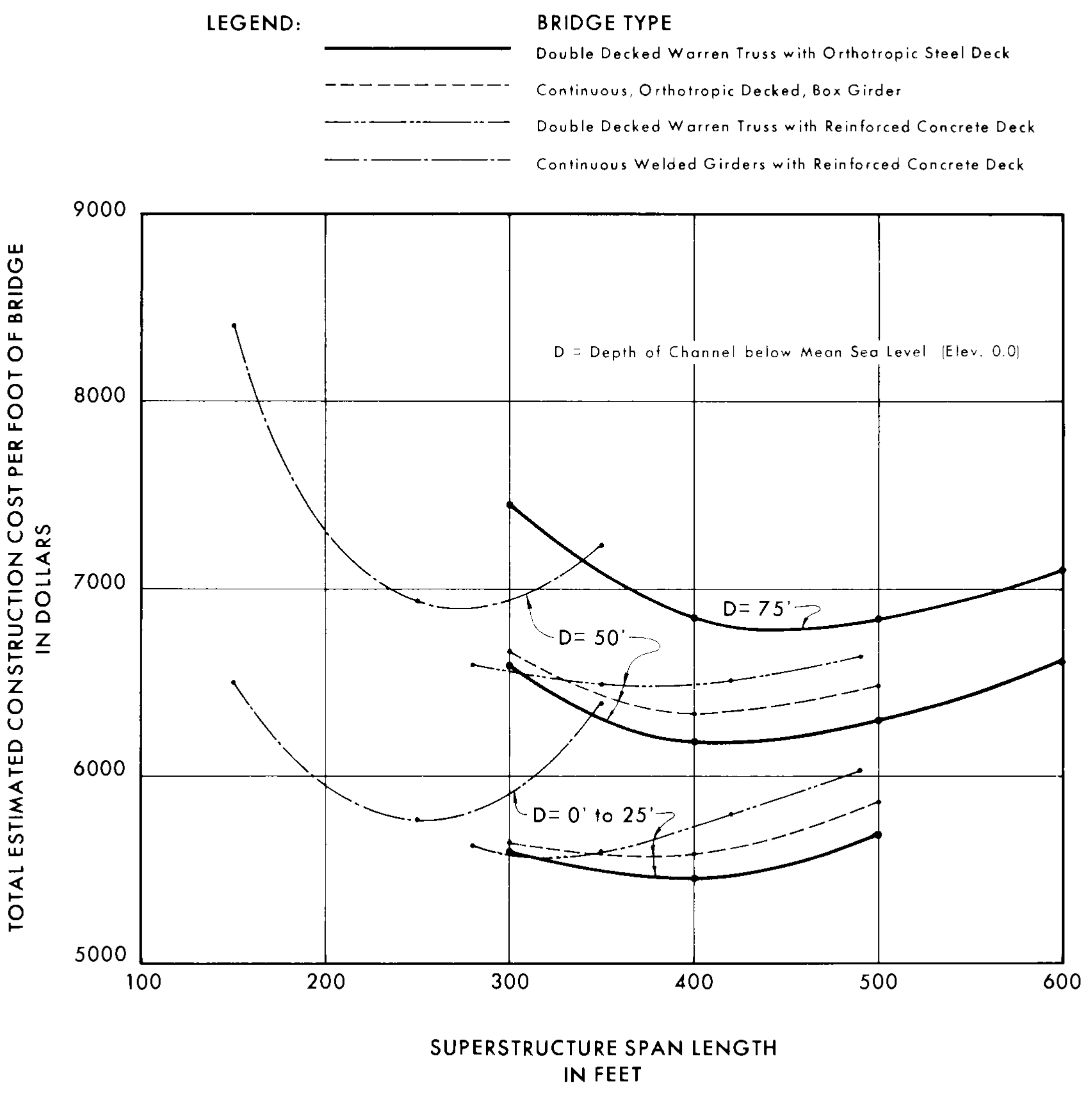


Figure V-1 GRAPHIC COMPARISON OF BRIDGE COST

Layout

There are no controls over the layout of spans other than economy of construction. Four different superstructure types were studied to determine the most economical type and corresponding span length for the site. The results of these studies are described in the following section and are illustrated on Figure V-1. A constant 407-ft. span length was selected for the main bridge at the site.

The main bridge, which is 13,431'-0" in length, extends from bluff line to bluff line. Approach structures, which extend beyond the bluff lines, are required to separate the profile grades of the eastbound and westbound roadways.

Bridge Type Studies

Four different superstructure types, which are illustrated in Figure V-2, were studied for this crossing location. They are:

1. Continuous welded steel girders with a reinforced concrete deck slab supported by steel stringers and floorbeams.

2. Continuous double-deck, Warren truss with reinforced concrete deck slabs.

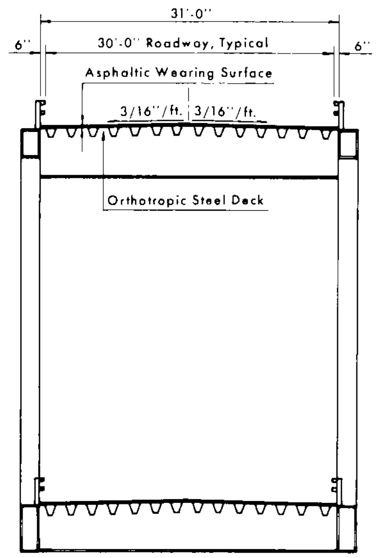
3. Continuous orthotropic steel box girders.

4. Continuous, double-deck, orthotropic Warren truss.

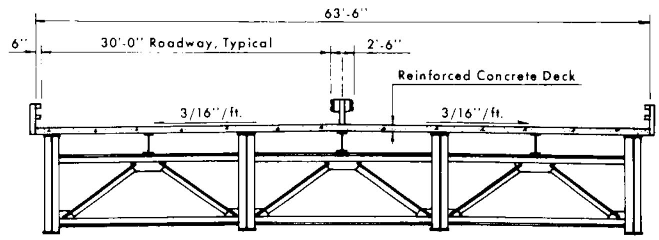
A preliminary design was made for each superstructure type for a series of span lengths. Quantities were calculated for each design.

A preliminary substructure design was then made for each of the superstructure designs for a series of channel depths. Substructure quantities were calculated.

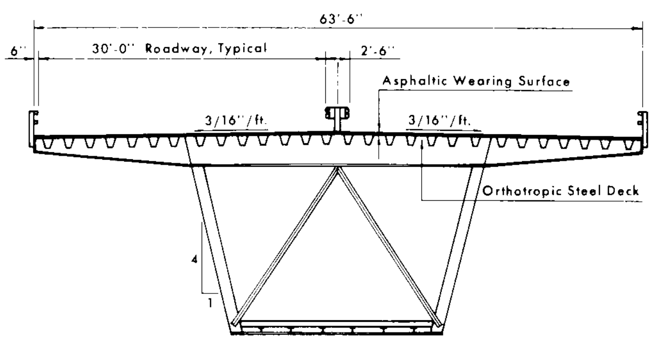
Unit materials prices were then applied to the superstructure and substructure quantities. The resulting bridge costs (per foot) were then plotted for each superstructure type and channel depth. The resulting graph, Figure V-1, indicates that Structure Type 4, the continuous, double-deck, orthotropic Warren truss in the 400-ft. span range is the most economical bridge for this site. This layout is illustrated by Plate A-2, Appendix A.



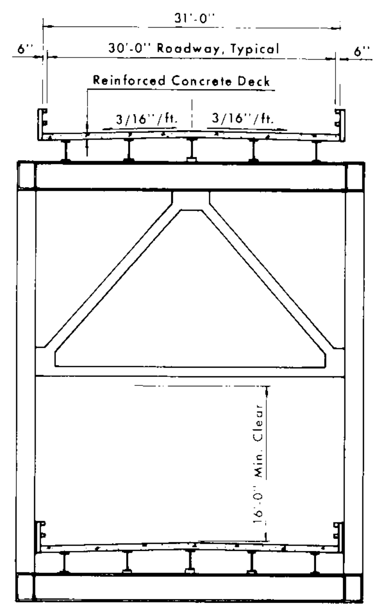
DOUBLE DECKED WARREN TRUSS
WITH ORTHOTROPIC DECKS



CONTINUOUS WELDED GIRDERS
WITH REINFORCED CONCRETE DECK



CONTINUOUS, ORTHOTROPIC DECKED,
BOX GIRDER



DOUBLE DECKED WARREN TRUSS
WITH REINFORCED CONCRETE DECKS

Figure V-2 BRIDGE TYPES STUDIED FOR CROSSINGS IV & V

Features of Superstructure

Main Spans	:	33 spans @ 407'	= 13,431'
West Approach	:		= 870'
East Approach	:		= 780'
Total Length of Structure			15,081'

Roadway	:	2 @ 30'-0" with no curbs or parapets.
Main Spans	:	
Roadway Deck	:	Orthotropic steel deck with asphaltic concrete wearing surface supported by transversefloor-beams
Trusses	:	Continuous, double-deck trusses, Warren system, 33'-6" c. to c. trusses, constant depth utilizing the orthotropic deck system as partial chord material
Lateral System	:	The orthotropic deck system will be designed to resist lateral forces.
Approach Spans	:	
Roadway Deck	:	Reinforced concrete slab supported on multiple girders
Girders	:	Continuous multiple welded girder spans
Structural Steel	:	ASTM A588 Weathering Steel

Features of Substructure

Main Piers	:	Large diameter (36" Ø steel) piles supporting large, precast concrete shells, sealed with tremie concrete and then filled with cast-in-place reinforced concrete. Piers are topped with hammer head caps.
Approach Piers	:	Conventional, cast-in-place circular concrete columns supported on pile footings.
Abutments	:	Conventional, pile supported concrete semi-deep abutments.

Construction Methods

The truss structure lends itself readily to cantilever construction or for the relatively short truss spans envisioned it is feasible to fabricate large sections on shore which are than floated to the site and set in place.

The use of the orthotropic steel deck, in lieu of a concrete slab, reduces the volume of material to be handled and the deck can be placed in weather unsuitable for concrete work, thus extending the construction season.

The main piers utilize a precast concrete base section through which the large diameter piles are driven (through precast holes). Cylindrical, precast concrete shells are then placed upon the base to form the exterior shape of the pier. These shells extend to an elevation above anticipated extreme high water. A tremie seal is placed in the base shell. The shell is pumped dry, footing and column reinforcement is placed and the shell is filled with concrete. The hammer head pier caps are then formed and placed to complete the pier.

A circular shape was assumed for the footings and shafts of the main piers. Future study, in a final design phase, might indicate an elliptical or similar shape as being more efficient to resist transverse horizontal forces, principally from ice. The circular shape assumed, however, is structurally feasible, has considerable merit from the constructional standpoint and its use allows the computation of substructure quantities with sufficient accuracy for the preliminary estimates.

Utilization of this type of construction precludes the necessity for elaborate cofferdams which, for the conditions at this site, would be a sizeable construction item. This approach has been used on a number of other major structures with satisfactory results.

The approaches over land utilize multiple welded girder spans to bring the double-deck directional roadways to the same elevation. Conventional land construction methods may be used to build the piers and erect the superstructure.

Extensive grading of the bluffs at the ends of the main structure has been considered in the estimate for the approach structures. The use of rock slope protection for stabilization of the slopes is not considered necessary based upon existing knowledge of the site.

Estimated Construction Cost

The construction cost for a Line IV Bridge, including 10% for contingencies and variations, is estimated as $126,000,000.

An estimated $8,000,000 may be saved in the initial construction cost by construction of only two of the four roadway lanes. The remaining two could be constructed at some future date for an estimated cost of $12,000,000, assuming no allowance for escalation in the unit prices. This would, however, increase the effective cost of the total bridge by $4,000,000.

The detailed estimate of the four-lane bridge follows:

Table V-2

ESTIMATE OF COST TRUSS BRIDGE AT CROSSING IV

2 Approach Structures at 870' and 780' = 1,650'
Main Bridge 33 Spans at 407' = 13,431'
Total Bridge Length = 15,081'
Roadway: 2 at 30'-0"

APPROACH STRUCTURES						
Superstructure						
Structural Steel (A588)	1,792,000	Lbs.	@	$ 0.65	=	$ 1,164,800
Cast Steel	36,000	Lbs.	@	1.45	=	52,200
Concrete in Deck	1,320	C.Y.	@	200.00	=	264,000
Reinforcing Steel	360,000	Lbs.	@	0.28	=	100,000
Handrail	3,300	L.F.	@	20.00	=	66,000
Sub-Total Superstructure						$ 1,647,800
Substructure						
Concrete in Columns and Cap Beams	480	C.Y.	@	$200.00	=	$ 96,000
Concrete in Footings and Abutments	520	C.Y.	@	170.00	=	88,400
Reinforcing Steel	190,000	Lbs.	@	0.28	=	53,200
14HP73 (45' Long)	12,660	L.F.	@	30.00	=	379,800
Excavation (Above Grade)	460,000	C.Y.	@	2.40	=	1,104,000
Excavation (Below Grade)	1,600	C.Y.	@	10.00	=	16,000
Sub-Total Substructure						$ 1,737,400
MAIN BRIDGE:						
Superstructure						
Structural A588 Steel	57,465,000	Lbs.	@	$ 0.64	=	$ 36,777,600
Cast Steel	1,140,000	Lbs.	@	1.44	=	1,641,600
Asphaltic Wearing Surface	92,650	S.Y.	@	16.00	=	1,482,400
Deck Protection	92,650	S.Y.	@	2.00	=	185,300
Handrail	53,700	L.F.	@	20.00	=	1,074,000
Sub-Total Superstructure						$ 41,160,900
Substructure						
Pier Excavation	60,000	C.Y.	@	$ 31.20	=	$ 1,872,000
36" Dia. Pipe Piles	60,800	L.F.	@	200.00	=	12,160,000
Precast Concrete	12,500	C.Y.	@	1,280.00	=	16,000,000
Tremie Concrete	22,800	C.Y.	@	165.00	=	3,762,000
Shell Filler Concrete	21,800	C.Y.	@	165.00	=	3,597,000
Pier Cap Concrete	5,500	C.Y.	@	468.00	=	2,574,000
Reinforcing Steel	7,302,000	Lbs.	@	0.50	=	3,651,000
Shaft Armoring (Steel)	980,000	Lbs.	@	1.20	=	1,176,000
Foundation Armor Rock	65,000	C.Y.	@	40.00	=	2,600,000
Sub-Total Substructure						$ 47,392,200
Total for Structure						$ 91,938,200
Mobilization						$ 23,000,000
Total for Crossing						$114,938,200
Contingencies and Variations (±10%)						$ 11,061,800
TOTAL ESTIMATED CONSTRUCTION COST						$126,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

CROSSING AT LOCATION V

General

Location V was selected for its relatively shallow water depths. The maximum depth is about 50 ft. below mean sea level in each of two distinct channels which are separated by a wide expanse of low lying land which is submerged during high tide. This line is the longest of those studied as feasible bridge sites.

Layout

As for the bridge at Location IV, there are no controls over the layout of spans other than economy. The major length of structure is over the shoals. This arrangement takes advantage of the economy afforded by reduced pier heights.

The main bridge extends from bluff line to bluff line. Approach structures, which extend beyond the bluff lines, are required to separate the profiles of the eastbound and westbound roadways.

Bridge Type Studies

The bridge type studies made for Line IV also apply to Line V.

The same structure type, that is, the continuous double-deck, orthotropic Warren truss with a 400-ft. span was chosen as the most economical for Line V. This layout is illustrated by Plate A-5, Appendix A.

Features of Superstructure

Main Spans	:	65 spans @ 400′	= 26,000′
West Approach	:		= 747′
East Approach	:		= 785′
Total Length of Structure			27,532′

The roadway dimensions, roadway deck trusses, lateral system and approach spans are similar to those considered for Line IV.

Features of Substructure

The Features of Substructure for Line V are similar to those for Line IV.

Construction Methods

Same as those for Crossing IV.

Estimated Construction Cost

The construction cost, including 10% for contingencies and variations, is estimated as $223,000,000. The detailed estimate follows:

Table V-3

ESTIMATE OF COST TRUSS BRIDGE AT CROSSING V

2 Approach Structures at 747' and 785' = 1,532'-0"
Main Bridge 65 Spans at 400'-0" = 26,000'-0"
Total Bridge Length = 27,532'-0"
Roadway: 2 @ 30'-0"

APPROACH STRUCTURES:						
Superstructure						
Structural Steel (A588)	1,670,000	Lbs.	@	$ 0.64	=	$ 1,068,800
Cast Steel	34,000	Lbs.	@	1.45	=	49,300
Concrete in Deck	1,230	C.Y.	@	200.00	=	246,000
Reinforcing Steel	335,000	Lbs.	@	0.28	=	93,800
Handrail	3,055	L.F.	@	20.00	=	61,100
Sub-Total Superstructure						$ 1,519,000
Substructure						
Concrete in Columns and Cap Beams	480	C.Y.	@	$ 200.00	=	$ 96,000
Concrete in Footings and Abutments	520	C.Y.	@	170.00	=	88,400
Reinforcing Steel	190,000	Lbs.	@	0.28	=	53,200
14HP73 (45' Long)	12,660	L.F.	@	30.00	=	379,800
Excavation (Above Grade)	130,000	C.Y.	@	2.40	=	312,000
Excavation (Below Grade)	1,600	C.Y.	@	10.00		16,000
Sub-Total Substructure						$ 945,400
MAIN BRIDGE:						
Superstructure						
Structural Steel (A588)	111,280,000	Lbs.	@	$ 0.64	=	$ 71,219,200
Cast Steel	2,210,000	Lbs.	@	1.44	=	3,182,400
Asphaltic Wearing Surface	179,400	S.Y.	@	16.00	=	2,870,400
Deck Protection	179,400	S.Y.	@	2.00	=	358,800
Handrail	104,000	L.F.	@	20.00	=	2,080,000
Sub-Total Superstructure						$ 79,710,800
Substructure						
Pier Excavation	82,400	C.Y.	@	$ 31.50	=	$ 2,595,600
36" Dia. Pipe Piles	109,000	L.F.	@	200.00	=	21,800,000
Pre-Cast Concrete	20,200	C.Y.	@	1,440.00	=	29,088,000
Tremie Concrete	33,000	C.Y.	@	165.00	=	5,445,000
Shell Filler Concrete	30,100	C.Y.	@	165.00	=	4,966,500
Pier Cap Concrete	11,000	C.Y.	@	448.00	=	4,928,000
Reinforcing Steel	10,900,000	Lbs.	@	0.52	=	5,668,000
Shaft Armoring (Steel)	2,850,000	Lbs.	@	1.20	=	3,420,000
Foundation Armor Rock	106,000	C.Y.	@	40.00	=	4,240,000
Sub-Total Substructure						$ 82,151,100
Total for Structure						$164,326,300
Mobilization						$ 38,000,000
Total for Crossing						$202,326,300
Contingencies and Variations ±10%						$ 20,673,700
TOTAL ESTIMATED CONSTRUCTION COST						$223,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

OTHER LOCATIONS

Several other locations for a bridge crossing were studied but were not considered as desirable as those chosen for detailed study (III, IV and V).

Lines I and II

Lines I and II were investigated as possible bridge sites in Phase I of these studies but were eliminated from further consideration because of expected high cost, expected unstable soils at the approaches and poor traffic service on the Anchorage side.

No construction costs for these sites are included in this report.

Line 0

Line 0 was studied only to obtain an "order of magnitude" cost figure for a bridge at the site.

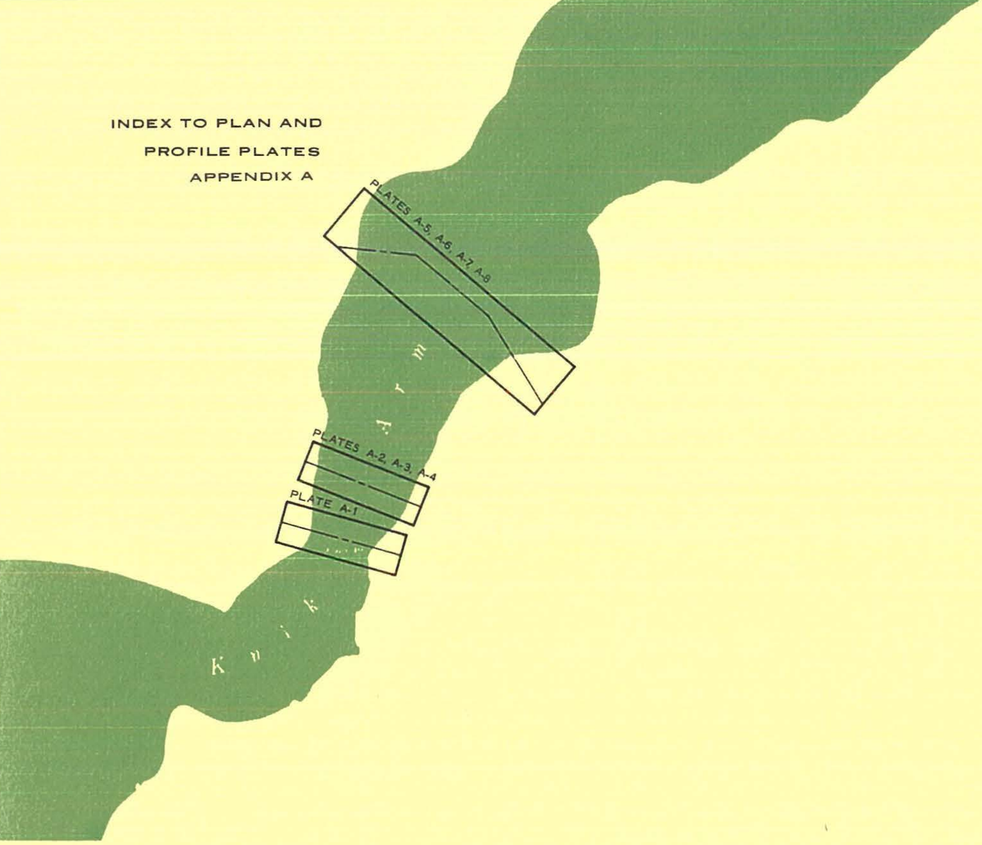
This location is not considered as a desirable bridge site because of the navigational clearance requairements, the length over deep water and the exposure to wind and ice forces which are envisioned as larger than the forces at Lines III, IV and V and possible interference with the glide path for the proposed north–south runway of the International Airport.

A bridge at this location would also be subject to damage by ships moving in and out of the port of Anchorage. This would impose a requirement for expensive protection for piers in the navigable depth waters of the channel. The cost of pier protection for all piers in the navigable waters would be prohibitive. For this estimate, only the two piers at the navigation span were provided with protection in the form of dumped rock islands concentric about the pier shafts. Other piers are unprotected and could easily be damaged by a ship maneuvering in the strong tidal currents.

The quantities for a structure at this site were obtained from preliminary layouts and designs. Unit costs were applied to these quantities to obtain the bridge construction cost.

The estimated construction cost for a bridge at Location 0, including 10% for contingencies and variations, is $193,000,000. The length of the structure is 15,240'.

INDEX TO PLAN AND
PROFILE PLATES
APPENDIX A



SECTION VI

causeway dam crossings

causeway dam crossings

GENERAL

The possible use of a dam across Knik Arm to provide the supporting structure for a roadway crossing of Knik Arm has been considered to compliment the possible use of other means of effecting the crossing. A dam of this magnitude requires the solution to many problems which could take years to resolve and which will need extensive data collection prior to the start of design or construction. A number of the solutions to the intermediate problems are beyond the scope of data available and indeed beyond the purpose of this portion of the study, which is to determine the feasibility of constructing a dam across Knik Arm.

The construction of dams across tidal estuaries has been accomplished at a number of locations in Holland and on portions of the Passamaquoddy tidal-project (Second Congress on Large Dams, Casey, 1936) in the vicinity of Eastport, Maine. The use of a dam to effect a crossing of the Turnagain Arm has been reported on favorably by Armstrong Associates. The construction of dams by dumping rock into water has been used widely, one of the most recent notable examples being the Aswan Dam base. Therefore, it can be preliminarily concluded that such construction in Knik Arm is possible.

In the initial phases of the study, the possibility of this type of construction was considered at all crossing locations. At Crossings 0, I and II, the history of unstable soils and the interference with shipping make the construction of a dam undesirable. At Crossing III the extreme depth of water makes the site unattractive. At Crossings IV and V the possibility of constructing a dam is better. Crossing VI was considered to the point of obtaining an "order of magnitude" cost but was not favored because of the excess distance from the City of Anchorage.

Crossing V was selected for more detailed study over Crossing IV, Plate A-7, Appendix A, even though it is longer, principally to take advantage of the shallower water, the wide expanse of exposed tidal flats, the decreased tidal prism and the presence of two distinct channels for tidal flow. A less extensive study at Crossing IV, Plate A-3, Appendix A, was performed and the data from both locations used to obtain the "order of magnitude" cost for a dam at Crossing VI.

For the dam proper three primary items are in need of study to determine the feasibility of construction. They are: 1) Foundation and Geology; 2) Typical Section; and 3) Construction Methods.

FOUNDATION AND GEOLOGY

Geologic and engineering reconnaissance indicated that at one or both of the terminals of Crossings III, IV, and V, the Bootlegger Cove Clay, or a clay similar to it, may exist at or below beach level, and may extend for some distance out beneath Knik Arm. It is therefore quite possible that while the Preliminary Causeway Dam Section (Section 8, Plate A-6, Appendix A) may be applicable for large portions of the crossing, modification of the embankment section might be required in the vicinity of the landward portions, in order to attain the same relative stability for seismic conditions.

TYPICAL SECTION

In the course of these studies a number of typical sections have been proposed. Plate A-6, Appendix A, shows some of the sections considered. The availability of materials, the stability of the interim and completed embankment, the effect of seismic activity, regardless of the crossing location, and the possible methods of construction all were considered in selecting Section 8 as the most reasonable section to study further at this time. This section is a slight modification of the section suggested by Shannon and Wilson as suitable when considering seismic conditions.

Relatively large quantities of sand and gravel are present in the Pleistocene glacial units in the uplands adjacent to the bluffs along Knik Arm, and in the flood plains of the Knik and Matanuska Rivers. Rock for embankment construction and rip-rap may be obtained by quarrying in Chugach Range. Based on the jointing of the metavolcanic rocks observed in the Alaska Railroad quarries near Eklutna, and near Rainbow on the Turnagain Arm, large stone, of the 15-to-20 ton size blocks will be difficult to obtain. However, there are indications that the required stone size will not exceed 3 to 6 tons which increases the availability of the rock. Smaller size rock are available in good quantity.

Theoretical analyses were made of the stability of a number of embankment cross-sections, by both circular arc and wedge analyses, including seismic factors. The assumed angles of internal friction and unit weights of the sand and gravel, and rock embankment materials (ϕ varied from 30° to 40°, and unit weights varied from 125 to 140 pcf), were considered to be realistic values. The absence of any valid strength parameters for the soils beneath Knik Arm, prevented determination of the most desirable embankment-foundation section for the causeway dam. Without the benefit of actual strength parameters for the embankment and foundation materials, refinements in embankment design are not possible.

A rock fill is recommended over a sand and gravel fill primarily to achieve acceptable seismic stability. The rock fill would be less likely to liquefy in the event of a major earthquake, even though some subsidence due to consolidation could be expected in case of a major earthquake.

The recommended Preliminary Causeway Dam Section provides for no positive cut off within the embankment or the underlying foundation. It is expected that sedimentation of fines on the upstream face will rapidly reduce the seepage through the structure.

A very comprehensive program of surface and submarine profiles, borings, laboratory testing, geophysical explorations, and model testing will be required for further refinement in the causeway dam design. Special note should be taken of the comments regarding the Typical Section offered by Shannon and Wilson and Dr. R. B. Peck, Appendix E and by Dr. Per Bruun, Appendix C.

CONSTRUCTION METHODS

A portion of the requirements of a study of a Causeway Dam type of crossing under the conditions presented in Knik Arm is a review of possible construction methods to determine the feasibility of any construction. The basic problems which arise in the construction of a permanent dam at this location are: 1) the large tidal fluctuations and the consequent large volumes of water moving across the site; 2) the availability of sufficient quantities of suitable material, labor and equipment; and 3) the development of a method of construction suitable for the project.

A number of methods utilizing both natural and man-made components were considered and rejected during the course of these studies. The most abundant natural material available in quantities sufficient for a dam of the magnitude being considered are the sand and gravel in Knik Arm. Sand has been successfully used in the construction of dams across tidal estuaries in Holland. To contain the sand during construction, precast concrete elements, asphaltic pavements, willow mattresses, etc., have been used on various projects in Holland. However, the tidal range and the climatic variations are not as extreme as on Knik Arm.

Noting the success which has been realized in Holland in the use of sand embankments and the large quantity of sand and gravel available in Knik Arm an early attempt was made to devise a means of utilizing sand and gravel as the basic embankment material under the conditions present on this project.

One possible method of containing the sand during construction is the use of ice dams as cofferdams or temporary dams to eliminate tidal currents across the site. Under this method natural ice jams would be enlarged by encouraging additional jamming; thickened by spraying during freezing weather, and grounded along the line of the proposed dam. These ice dams would then form a bulwark against which sand could be placed. The required volumes of ice and sand; the difficulty of maintaining the ice dams during the summer months, and the problems associated with spraying and dredging in extremely cold weather all combine to indicate a doubt that the method will be practical. At best an extensive investigation, including prototype construction, would be required.

Other methods of providing temporary dams or velocity attenuation to permit the use of dredged sand as the basic construction material included: 1) sunken Liberty Ships; 2) sunken barges; 3) concrete caissons; 4) concrete blocks; 5) precast concrete shapes such as tetrapods, and others. All of these were judged to be too expensive or too difficult to accomplish under the conditions in Knik Arm. The use of quarried rock as a base for a sand or rock embankment was judged to be feasible and further studies and cost estimates proceeded on that basis.

To the problem of material type must be added the problem of the large tidal ranges and flows. For this study, the peak flow generated during the second hour of the May 24, 1971, tide cycle was utilized. This particular tide was considered to be an average spring tide and the results from the use of this data would be close to the range required for greater, rarer, tide ranges. Additional tide cycles must be measured and investigated prior to completion of more advanced designs. Also, better underwater contours will be needed to permit a more accurate determination of the tidal prisms and material quantities.

At Crossing V the peak hour discharge for the May 24, 1971, tide cycle was 3,690,000 cubic feet per second (cfs) at an average water surface elevation = -7 MSL. The Crossing V Section contains three distinct hydraulic zones or sub-sections; i.e. Eagle Bay Channel, Center or flats area and the Goose Bay Channel. The design flow was distributed among these three sub-sections, in accordance with the hydraulic conveyance factor for each section, as follows:

Eagle Bay Channel	1,340,000 cfs
Center	730,000 cfs
Goose Bay Channel	1,620,000 cfs

Using these basic flows, velocities were determined for each section for a sequence of construction stages. From these velocities the required minimum rock size for each section at each stage was established.

The methods and formulae used to determine allowable velocity/rock size relationships were developed by S. V. Isbash and presented at the Second Congress on Large Dams in his paper entitled "Construction of Dams by Depositing Rock in Running Water." These same formulae and graphical solutions are included in September, 1970, revision to Section 712-1 of the U.S. Corps of Engineers publication "Hydraulic Design Criteria." Velocity and discharge checks were made as the construction stages advanced and discharges were reduced during the later stages as necessary to effect a hydraulic balance across the dam section.

It can be noted from Figure VI-1 that construction proceeding from east to west along the embankment is in general anticipated. This causes the final closure to occur in the vicinity of the Goose Bay Channel. The primary reason for choosing the closure in this location is to place the bulk of the embankment which must be placed prior to closure as near as possible to the source of the rock in the Chugach Range.

It is anticipated that barges can be used for hauling and dumping up to a top of embankment elevation of –10 MSL. Operation of barges for higher embankment elevations up to MSL, may be possible providing barge control can be maintained. The use of a cableway to place some of the large material in the Goose Bay Channel area is anticipated. At MSL the top of the embankment will be above the tide generated water surface approximately 50%, about six hours, of the time. During this time interval, the embankment can be utilized as a haul road and with proper scheduling considerable fill material can be delivered to the operating face.

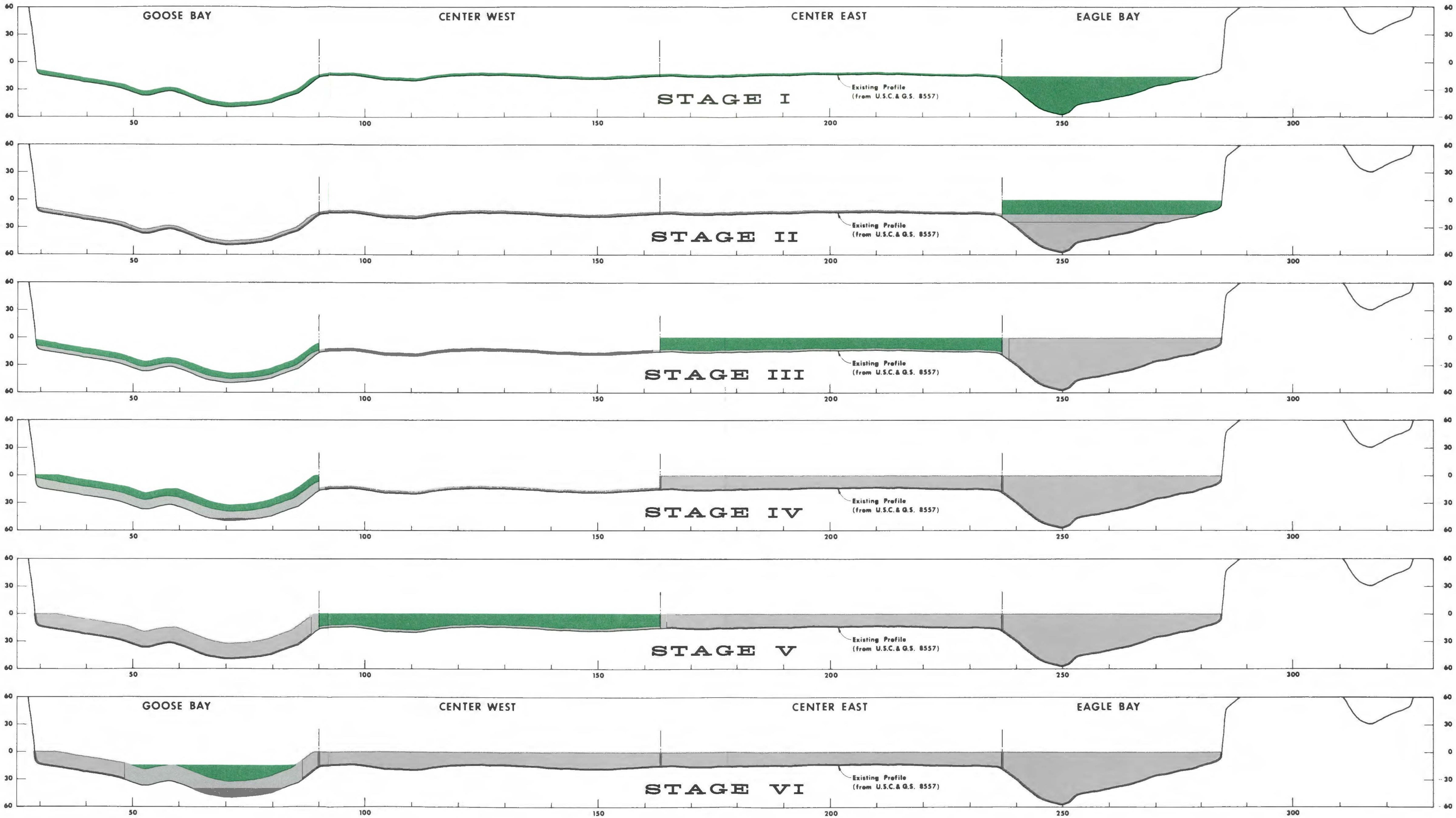
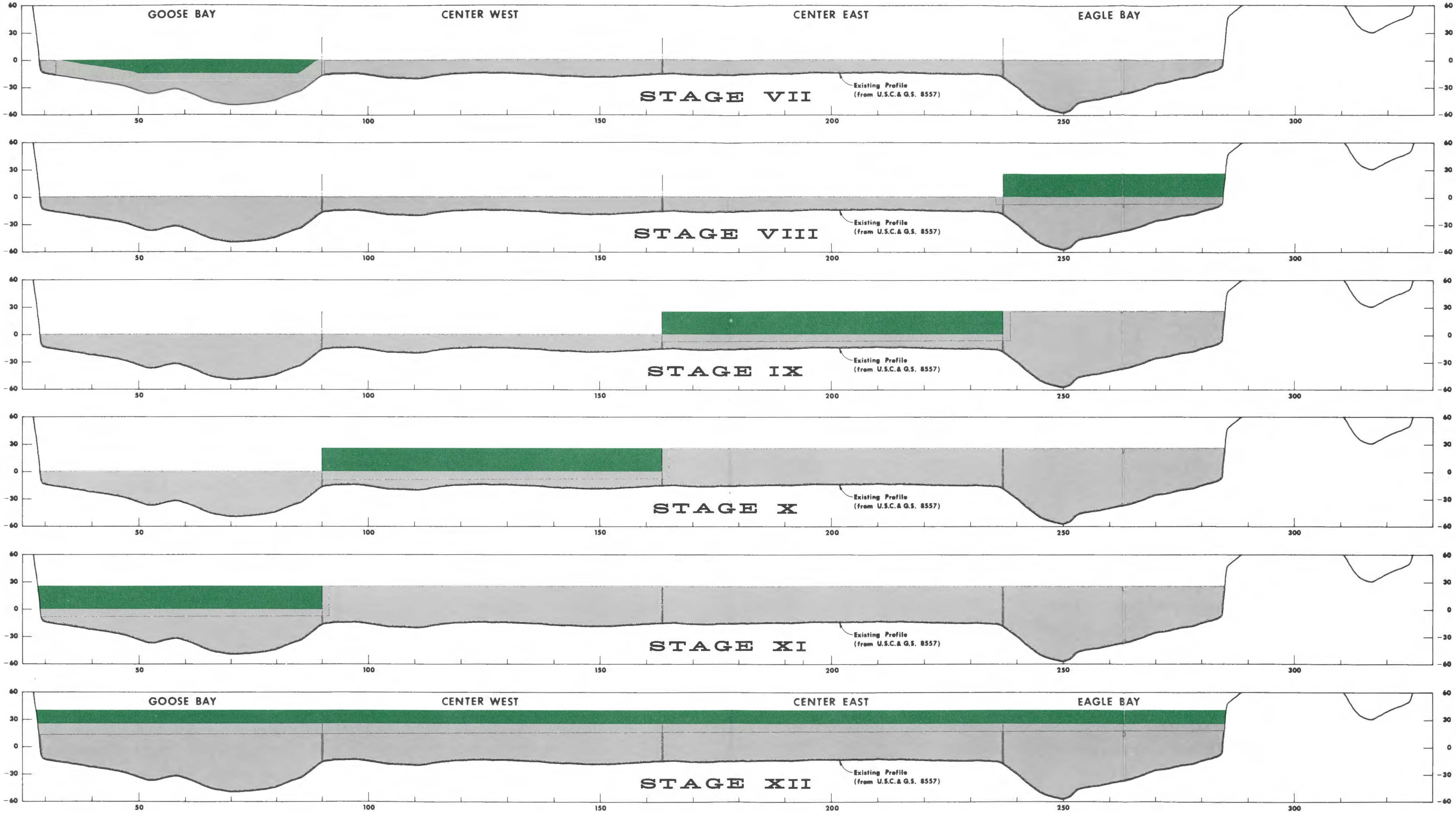


Figure VI-1

CAUSEWAY DAM CONSTRUCTION SEQUENCE

CROSSING V


GOOSE BAY
CENTER WEST
CENTER EAST
EAGLE BAY
STAGE VII
STAGE VIII
STAGE IX
STAGE X
STAGE XI
STAGE XII
Existing Profile
(from U.S.C.& G.S. 8557)
60
30
0
-30
-60
50
100
150
200
250
300

The order of construction of the various segments of the embankment can be varied to suit a large number of conditions or assumptions. Any order or procedure chosen must be checked against the tidal and weather conditions anticipated during construction. To check the feasibility of the proposed construction, one sequence of construction was reviewed in depth and the results shown on Figure VI-1. Other sequences are possible, are as feasible as the one reviewed, and are comparable as to practicality and cost.

The sequence chosen for review consisted of twelve (12) construction stages. Final closure was in the Goose Bay Channel vicinity and the peak discharge (3.9×10^6 cfs) indicated by the May 24-25, 1971, tide cycle was used to determine velocities and rock sizes.

Each stage consists of certain actions and requirements as follows:

Stage 1:

Under this initial phase of construction the Eagle Bay Channel is filled to the level of the flats.

To compensate for the loss of waterway area; the Goose Bay Channel, the Center–West and the Center–East Sections must be armored to protect against increased velocities. The Goose Bay Channel will require two foot, median diameter, stone while the Center Sections will require one foot stone. Anticipated resulting velocities are: Goose Bay – 15± fps, Center Sections – 8± fps, Eagle Bay – 8± fps. No reduction in discharge is anticipated.

Stage 2:

This stage continues the raising of the embankment in the Eagle Bay area to elevation 0 MSL. Stone size requirements are 0.75′ median diameter. The armoring provided in Stage 1 is adequate and resulting velocities are: Goose Bay – 17± fps, Center Sections – 9± fps, Eagle Bay – 8± fps. No discharge reduction is anticipated.

Stage 3:

This phase extends the embankment across the Center–East Section to about the middle of the flats area to elevation 0 MSL with 0.75′ median diameter stone. The stone size requirements should be modified to include a double layer of 2′ diameter stone at the top of the embankment to accommodate future stage requirements. Concurrent with or prior to this embankment construction, additional protection for the Goose Bay Channel in the form of 3' diameter stone armoring must be provided. Prior armoring in the Center–West Section will be adequate. Resulting velocities are: Goose Bay – 20± fps, Center–West – 10± fps, Center–East and Eagle Bay – 8± fps. A reduction in discharge may be beginning to occur, but none was used for this stage.

Stage 4:

Because of the relatively large amount of large stone (4 ft. diameter) required for the armoring of the Goose Bay Channel when the Center–West Section is filled, the armoring operation has been designated a separate stage of construction. The placing of this stone will be difficult in the now-swiftly moving waters of Goose Bay Channel. For the design discharge (peak hour, average spring tide, no discharge reduction) velocities in the 20 fps range will be encountered. Barging may be possible for part of the material during slack tide periods, but a cableway may be required for the bulk of the material. Resulting velocities – not calculated.

Stage 5:

The closing of the Center–West Sections will complete the crossing of the flats area to elevation 0 MSL. Indicated stone size requirements are 1.5' median diameter but as is the case in Stage 3, the upper portion shougd include 2' stone. A discharge reduction at this stage is probable but for conservative results, none was considered. Resulting velocities include 24± fps in the Goose Bay Channel.

Stage 6:

In this stage the Goose Bay Channel is filled to elevation -16 MSL. At the beginning of this stage the high velocities resulting from Stage 5 will require the use of stone size comparable to that used in Stage 4 (4' diameter). As filling progresses a discharge reduction (30%± for the peak hour discharge) will occur which will further attenuate the velocities so that stone size can be reduced to 3' diameter as the top of this stage is approached. Calculated velocity for the reduced discharge through the Goose Bay Channel is 14± fps.

Stage 7:

This stage continues the filling of the Goose Bay Section to elevation 0 MSL. As the filling progresses and the discharge reduction continues, velocities and stone size requirements decrease accordingly. It is, however, suggested that stone size 2' or greater be utilized. Resulting velocity is 10±fps over the entire embankment.

Stage 8:

At the start of this stage the entire width of Knik Arm is filled to elevation 0 MSL. Top width of the embankment is 160'± and the surface is protected with 2' stone. Tide data analysis indicates that the top of the embankment will be above tide elevation approximately 50% of the time and that the top of the embankment will not be flooded until about the third hour of the flood tide cycle. By choking the center portion of the embankment, 50'± with 0.5' to 1.0' stone, the embankment can be used as a haul road. Also, the contractor has, at this stage, the option of side dumping or end dumping the material. This study assumes that end dumping, i.e. uniform raising of the full top width, will be employed.

In Stage 8 the Eagle Bay Section will be raised to elevation 25 MSL with 0.75' stone. Armoring provided in Stages 3, 5 and 7 should resist the resulting 13±fps velocity.

Stage 9 and 10:

These stages continue the raising of the embankment to elevation 25 MSL across the Center Sections. As the waterway area for the higher portion of the tide cycle is reduced, velocities in the Goose Bay Channel will increase. This increase in velocity will in part be offset by a reduction in the discharge. It is expected that velocities will reach 17± fps over the Stage 7 armoring.

Stage 11:

This stage completes the rock core of the embankment to elevation 25 MSL which is above high tide levels.

Stage 12:

In this stage the remainder of the embankment from elevation 25 MSL to 40 MSL is to be completed. If no settlement period is required, the roadway section can be completed.

General

After the completion of Stage 8, the raising of the embankment to final elevation 40 MSL can proceed concurrently with Stages 9, 10 and 11. Concrete work on the spillway can be effected at any stage of the operation providing the spillway and channel are usable at the time of the completion of Stage 11. If the material from the spillway channel is to be used in the causeway dam the start of the excavation will be delayed until Stage 8 is well advanced. If the spillway channel material is to be wasted, the excavation can proceed at the contractor's option.

One of the factors which the contractor will have to consider is the possibility that a Lake George breakout will occur during construction. If this occurs, the contractor can expect to encounter unusual variations in the tidal fluctuations, greater velocities, greater variations in upstream vs. downstream velocities and possible fresh water currents on top of salt water currents. Since the discharge of record for Lake George is about 10% of the discharge used as a base for the construction procedure outlined and will be a greater percentage of lesser tide ranges, its effect must be considered. It is possible that the contractor will elect to try to prevent a Lake George breakout rather than have the problem at the project site.

It should be re-emphasized that the analysis outlined above is a theoretical review of one method of construction. Any method or procedure will need to be subjected to more extensive analysis than contained herein and model testing to verify stone size requirements. Studies of possible quarry sites will need to be initiated as will studies of material characteristics and handling.

SPILLWAY

One of the key elements in the causeway dam alternates, irrespective of the location chosen, is the spillway required to safely pass excess flow from the lake formed by the dam. The failure or inadequacy of the spillway could result in major damage to the project.

The final configuration of the spillway and spillway channel will undoubtedly be the result of extensive studies and model testing. For the purpose of arriving at approximate costs and determining general feasibility, a standard concrete ogee section was chosen for study at this time. Standard flood routing procedures, U.S. Corps of Engineers spillway discharge coefficients, and U.S. Geological Survey topographic data were used in this study.

The spillway location selected for study considered such factors as accessibility for construction and maintenance, general conformity with good hydraulic placement, cost and ease or difficulty of construction. The use of a concrete box culvert or ogee type of spillway located in the center flats area was considered and rejected because of foundation and construction difficulties. The use of an ogee type spillway in the Goose Bay area was considered and rejected because of greatly increased construction cost and the difficulty of transporting labor and materials to the site. The use of a long elaborate spillway channel (hydraulically more appealing), low level channels, high and low control gates, curved entrance walls and other items were considered for inclusion but deemed either too expensive or inappropriate for study at this stage of the project.

The inclusion of low level control gates to assist in the control of tidal flows and velocities during construction is one particularly intriguing possibility which was considered. If this system could be made applicable by providing sufficient waterway area to significantly reduce tidal velocities over the partially completed embankment, some savings in large rock quantities could be made. However, a preliminary investigation revealed that the expenditure for the gates, mechanisms and increased concrete form work would offset other possible savings. In addition, the system would require lowering of the spillway channel thereby further increasing costs. Finally, the waterway area obtainable was insufficient.

The spillway crest elevation was set at 20 MSL to place it above the extreme tide elevation (19 MSL) thereby excluding salt water from the lake and insuring the eventual existence of a fresh water lake in Knik Arm. There apparently is sufficient fresh water available to maintain such a lake. However, future studies may show that it is desirable to lower the spillway crest elevation to better control major flows or to allow the incursion of some salt water to retard the freshening process. Conversely, the studies could show the need for low level gates to hasten the freshening process.

The criteria adopted for a spillway determines to a large extent the degree of protection afforded the project and the general area as a whole. On this project the danger to downstream life and facilities is somewhat attenuated by the relatively wide expanse of Knik Arm (8,000'±@ Cairn Point) which could pass the flow from a major dam failure without excessive damage.

(For instance, no recollection was found to indicate that the 1961 Lake George Breakout had any effect at Anchorage.) Upstream, the Glenn Highway and the Alaska Railroad are important transportation arteries which warrant some degree of protection. The tailrace of the Eklutna Power Plant is another item to be considered. The communities of Birchwood, Eklutna, Palmer, Matanuska and Wasilla appear to be safe from high water generated at the spillway.

For the design of the spillway, a 2,000' length was chosen as the maximum practical size for this type of spillway. The 1961 Lake George Breakout, as recorded at the Glenn Highway Bridge, (Figure III-10) was chosen as the design storm.

The results of the study indicate that a head of 9 feet will be required to pass a repeat of the 1961 Lake George Breakout. The peak rate of flow over the spillway will approximate 210,000 cfs. For a flood generated by a storm equal to 40% of the Maximum Probable Precipitation, approximately 7 feet of head will be required for a maximum outlfow of about 140,000 cfs. From this a maximum pool elevation of 29 MSL was derived.

BRIDGE OVER SPILLWAY

The bridge over the spillway is a continuous welded girder structure with a conventional reinforced concrete deck slab. The piers are of the single shaft, tee configuration supported by pile footings. The single shaft is proposed to minimize the turbulence in the spillway which will be caused by the piers and to present minimum exposure to potential ice forces.

Conventional land construction methods may be used to build the piers and erect the superstructure.

EFFECT OF DAM CONSTRUCTION

At normal pool (Elevation 20 MSL) a fresh water lake with a surface area greater than 100 square miles will be formed. Figure VI-2 shows the extent of lake coverage with the dam at Crossing V. The lake formed by dam at Crossing IV would have similar coverage.

At the maximum pool elevation, which can be expected to remain effective for several days at a time, the indications are: That the Eklutna power

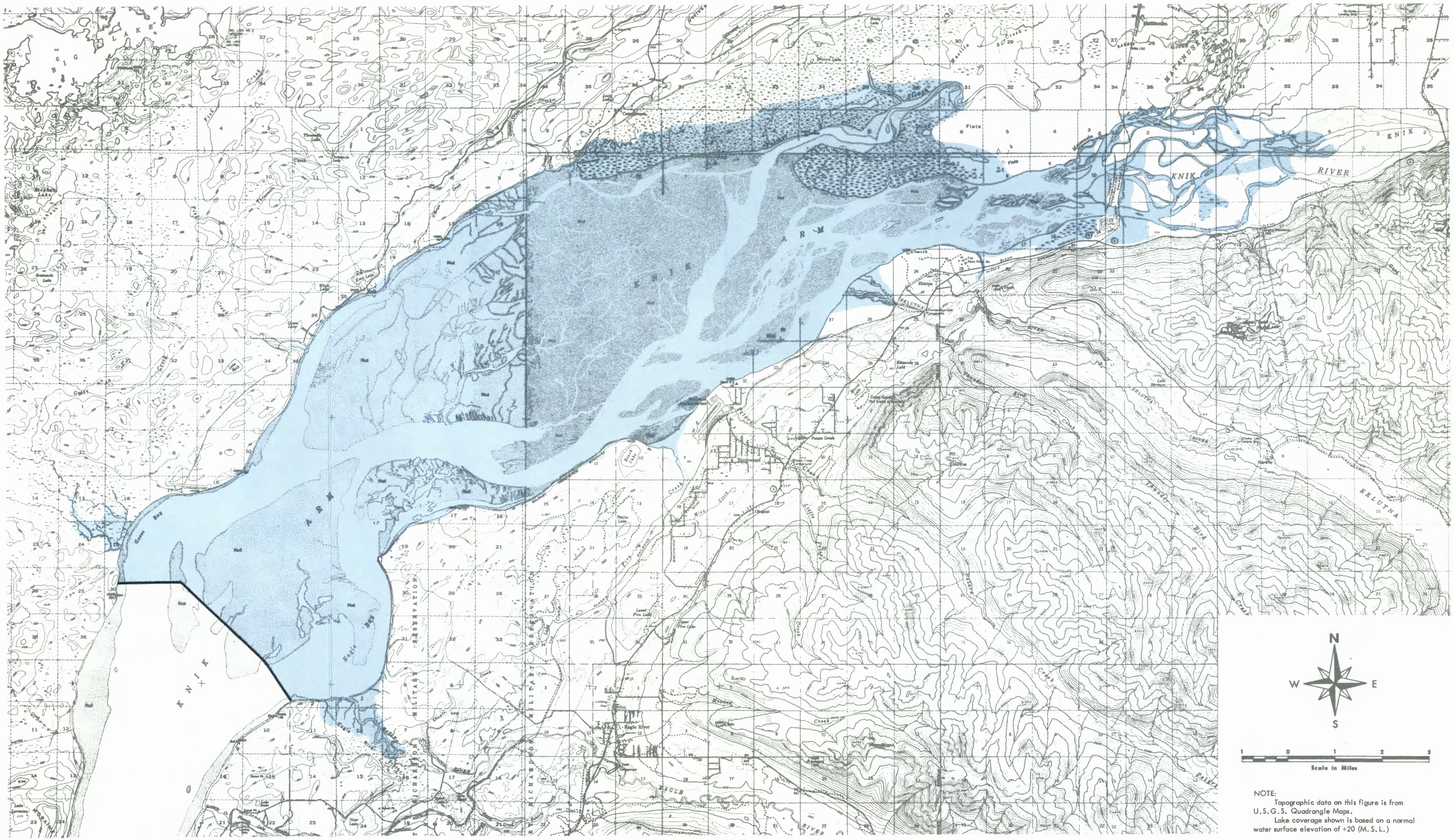


Figure VI-2

CAUSEWAY DAM LAKE COVERAGE – CROSSING V

plant will not be affected; that the Alaska railroad facilities will not be affected; that the towns and villages will not be affected, but that the Glenn Highway in the vicinity of Eklutna may be adversely affected, and the design high water at the Knik–Matanuska Rivers will be affected. It is reasonable to assume that during major storms, some rearrangement of the sand and gravel deposits near the outlet of the spillway channel will occur. This and the possibility of unusual currents below the dam will need to be investigated by means of model testing.

In addition, a study performed by Dr. Per Bruun, see Appendix C, indicates that there will be some amplification of tide elevations at Anchorage. This amplification could in the case of a dam at Crossing V increase the high tide at Anchorage by 0.8′ to 1′. This is approximately the same as that caused by the buildup of ice on the shoals in winter.

COST ESTIMATES

The construction of a dam at Crossing V is estimated to cost $209,000,000 and require 5 years for construction. The construction of a dam at Crossing IV is estimated to cost $289,000,000 and requires a 6-year construction period. The "order of magnitude" estimated construction cost for a dam at Crossing VI is $177,000,000. All cost estimates included 10% for contingencies and variations. For detailed cost estimates for Crossings IV and V see the following tabulations and Appendix D.

Table VI-1

ESTIMATE OF COST CAUSEWAY DAM AT CROSSING IV

Spillway Bridge at 2,640'
Spillway Ogee Section at 2,000'
Total Crossing Length = 15,950'
Roadway: 2 @ 30'-0"

SPILLWAY BRIDGE:						
Superstructure						
Structural Steel (A588)	3,855,000	Lbs.	@	$ 0.72	=	$ 2,775,600
Cast Steel	80,000	Lbs.	@	1.64	=	131,200
Concrete in Deck	4,180	C.Y.	@	230.00	=	961,400
Reinforcing Steel	1,150,000	Lbs.	@	0.32	=	368,000
Handrail	5,280	L.F.	@	22.50	=	118,800
Guardrail	2,640	L.F.	@	22.50	=	59,400
Sub-Total Superstructure						$ 4,414,400
Substructure						
Concrete in Piers	2,620	C.Y.	@	$230.00	=	$ 602,600
Concrete in Footings	1,560	C.Y.	@	180.00	=	280,800
Reinforcing Steel	687,500	Lbs.	@	0.32	=	220,000
14HP73 (45' Long)	29,700	L.F.	@	36.00	=	1,069,200
Excavation	4,800	C.Y.	@	11.00	=	52,800
Sub-Total Substructure						$ 2,225,400
SPILLWAY AND CHANNEL:						
Spillway Excavation						
Excavate Above Elev. 20	21,500,000	C.Y.	@	$ 3.40	=	$ 75,250,000
Excavate Below Elev. 20	2,600,000	C.Y.	@	4.50	=	11,700,000
Spillway Concrete						
Ogee Section Concrete	37,500	C.Y.	@	$180.00	=	$ 6,750,000
Reinforcing Steel	3,500,000	Lbs.	@	0.33	=	1,155,000
DAM:						
Rock-End Dumped						
Zone 1 0.5'-1.0'	2,120,000	C.Y.	@	$ 6.70	=	$ 14,204,000
Zone 2 1.0'-2.3'	500,000	C.Y.	@	7.10	=	3,550,000
Zone R 36" Riprap	520,000	C.Y.	@	9.10	=	4,732,000
Rock-Barge Dumped						
Zone 2 1.0'-2.3'	1,600,000	C.Y.	@	$ 12.50	=	$ 20,000,000
Zone R 36" Riprap	280,000	C.Y.	@	13.70	=	3,836,000
Rock-Barged and Rehandled						
Zone 1 0.5'-1.0'	480,000	C.Y.	@	$ 15.00	=	$ 7,200,000
Zone 2 1.0'-2.3'	150,000	C.Y.	@	15.40	=	2,310,000
Rock-Cableway Placed						
Zone 2 1.2'-2.3'	850,000	C.Y.	@	$ 13.60	=	$ 11,560,000
Zone 3 2.6'-3.0'	1,300,000	C.Y.	@	14.30	=	18,590,000
Zone 4 3.8'-4.0'	1,400,000	C.Y.	@	15.20	=	21,280,000
Zone 5 4.0'-5.0'	500,000	C.Y.	@	16.80	=	8,400,000
Causeway Sand Fill						
Dredged Sand Fill	6,200,000	C.Y.	@	$ 2.65	=	$ 16,430,000
Gravel Filter	350,000	C.Y.	@	10.50	=	3,675,000
Roadway Paving						
Complete w/Fencing	13,000	L.F.	@	$ 65.00	=	$ 845,000
Total for Structure						$238,106,800
Mobilization						$ 25,000,000
Total for Crossing						$263,106,800
Contintencies and Variations (±10%)						$ 25,893,200
TOTAL ESTIMATED CONSTRUCTION COST						$289,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

Table VI-2

ESTIMATE OF COST CAUSEWAY DAM AT CROSSING V

Spillway Bridge at 2,640'
Spillway Ogee Section at 2,000'
Total Crossing Length = 28,050'
Roadway: 2 @ 30'-0"

SPILLWAY BRIDGE:					
Superstructure					
Structural Steel (A588)	3,855,000 Lbs.	@	$ 0.70	=	$ 2,698,500
Cast Steel	80,000 Lbs.	@	1.56	=	124,800
Concrete in Deck	4,180 C.Y.	@	220.00	=	919,600
Reinforcing Steel	1,150,000 Lbs.	@	0.30	=	345,000
Handrail	5,280 L.F.	@	22.50	=	118,800
Guardrail	2,640 L.F.	@	22.50	=	59,400
Sub-Total Superstructure					$ 4,266,100
Substructure					
Concrete in Piers	2,620 C.Y.	@	$220.00	=	$ 576,400
Concrete in Footings	1,560 C.Y.	@	175.00	=	273,000
Reinforcing Steel	687,000 Lbs.	@	0.30	=	206,100
14HP73 (45' Long)	29,700 L.F.	@	35.00	=	1,039,500
Excavation	4,800 C.Y.	@	10.50	=	50,400
Sub-Total Substructure					$ 2,145,400
SPILLWAY AND CHANNEL:					
Spillway Excavation					
Excavate Above Elev. 20	17,500,000 C.Y.	@	$ 1.75	=	$ 30,625,000
Excavate Below Elev. 20	2,900,000 C.Y.	@	2.65	=	7,685,000
Spillway Concrete					
Ogee Section Concrete	37,500 C.Y.	@	$175.00	=	$ 6,562,500
Reinforcing Steel	3,500,000 Lbs.	@	0.30	=	1,050,000
DAM:					
Rock-End Dumped					
Zone 1 0.5'-1.0'	4,050,000 C.Y.	@	$ 6.10	=	$ 24,705,000
Zone 2 1.0'-2.3'	2,500,000 C.Y.	@	6.35	=	15,875,000
Zone R 36" Riprap	1,000,000 C.Y.	@	8.25	=	8,250,000
Rock-Barge Dumped					
Zone 2 1.0'-2.3'	500,000 C.Y.	@	$ 12.50	=	$ 6,250,000
Zone 3 2.6'-3.0'	750,000 C.Y.	@	13.00	=	9,750,000
Zone R 36" Riprap	200,000 C.Y.	@	14.30	=	2,860,000
Rock-Cableway Placed					
Zone 2 1.2'-2.3'	200,000 C.Y.	@	$ 16.00	=	$ 3,200,000
Zone 3 2.6'-3.0'	200,000 C.Y.	@	16.80	=	3,360,000
Zone 4 3.8'-4.0'	900,000 C.Y.	@	18.00	=	16,200,000
Causeway Sand Fill					
Dredged Sand Fill	8,500,000 C.Y.	@	$ 1.82	=	$ 15,470,000
Gravel Filter	500,000 C.Y.	@	9.55	=	4,775,000
Causeway Crest Road					
Crest Road - Complete	25,600 L.F.	@	$ 61.00	=	$ 1,561,600
Total for Structure					$164,590,600
Mobilization					$ 25,000,000
Total for Crossing					$189,590,600
Contingencies and Variations ($\pm$10%)					$ 19,409,400
TOTAL ESTIMATED CONSTRUCTION COST					$209,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

INDEX TO PLAN AND
PROFILE PLATES
APPENDIX A

SECTION VII

tunnel crossings

tunnel crossings

GENERAL

The apparent advantages of the use of a vehicular tunnel under Knik Arm to effect the desired crossing are sufficient to warrant the study of a tunnel alternate even though the apparent difficulties would seem to eliminate a tunnel from consideration.

The principal advantage of the tunnel mode of crossing would accrue to the driver. While in the tunnel the driver would avoid the weather conditions, particularly ice-covered roadways and high winds, present with any surface method of crossing. A tunnel, once in place, would cause no difficulty with navigation with the possible exception that the tunnel would need to be deep enough to avoid damage from dragging anchors and allowance would need to be made for any future port dredging operations. It has also been suggested that the tunnel might be more stable during seismic occurrences than other means of crossing. Depending upon the method of construction chosen, it is possible that a major portion of the construction operations could be based in a warmer climate where year-round operations are more feasible. Because the structure would, except for ventilation towers, be entirely below water level, no obstruction with aircraft traffic requirements would be present.

However, the use of a tunnel has some inherent disadvantages in addition to the higher cost which is normally attributable to this type of construction. The ventilation of a tunnel of the magnitude required for this project is a serious problem which in the event of a failure of even a minor piece of equipment could cause a complete stoppage of traffic through the tunnel. The use of the tunnel possibly would have to exclude trucks carrying inflammable liquids or explosives or restrict the use by these vehicles to certain periods of extremely light traffic. Even a ruptured automobile gasoline tank resulting from an accident or an engine fire is a major concern in a tunnel. The depth of water, the high tide fluctuations, the foundation material and the joining of the underwater portions to the land portions, all present serious design construction and maintenance problems. In the event major structural damage to the tunnel should occur, repair would be very difficult.

Two methods of constructing a tunnel were reviewed for applicability. Because of the higher cost of a four-lane tunnel, the studies were based upon a two-lane tunnel which obviously does not give traffic service comparable with the other types of crossing which have four-lane facilities.

TUNNEL LOCATIONS

Initially, tunnel crossings were considered at Lines I, II, III and IV.

At Crossings I and II the extended length of the tunnel and the unstable soils at either terminus coupled with the anticipated poor traffic service at the Anchorage terminal resulted in the exclusion of these crossings from consideration as possible tunnel sites. In addition, the preliminary studies indicated a construction cost of about $300,000,000 for a two-lane tunnel at either of these crossing locations.

At Crossing III the depth of water for all practical purposes eliminated consideration of a normal type of tunnel. The depth of water did, however, make the construction of a floating tunnel appear possible. With modern construction technology a tunnel of this type could be constructed. However, since a preliminary estimate indicated a cost of about $280,000,000, the solution did not appear to be economically feasible.

Crossing IV, presenting the best compromise among traffic service requirement, depth of water and total length was therefore studied as a possible two-lane tunnel site. In order to further reduce the length of tunnel required at this crossing a combination causeway–tunnel type of installation was used. A layout of a tunnel at Crossing IV is shown on Plate A-4, Appendix A.

As was the case for other crossing alternates, approximate quantities were obtained and unit prices applied thereto to arrive at a preliminary estimate of construction cost.

CONSTRUCTION METHODS

There are two acceptable methods of tunnel construction which might be applicable to this location. They are: A cast-in-place bored or driven tunnel and a trench-type tunnel constructed of large precast elements.

Bored Tunnel

The normal procedure in driving a tunnel of this type is to provide sufficient air pressure at the tunnel heading to support the overlying material and water. In this case the air pressure required would be about 70 psi which is beyond the limit of human endurance. Also, the rapidly fluctuating water levels, and consequently, air pressure requirements, brought about by the tide fluctuation would require an ever-changing set of working conditions at the heading. It has been suggested that these compressed air requirements could be somewhat reduced by the placement of a heavy blanket of fill on the existing bottom in the tunneling area. It is very questionable that this procedure would be successful in that upon saturation, it would actually increase the loads at the heading and it is doubtful that the tidal currents would leave the material in place as planned. Perhaps the use of a procedure which did not require men to work under compressed air conditions could be made to work satisfactorily.

After reviewing the above conditions it was decided that a bored or driven tunnel would be too impracticable and hazardous to consider further.

Trench-Type Tunnel

Under this method of tunnel construction, normal procedure is to excavate a trench in the bottom of the river or bay to be crossed, place precast tunnel elements in the trench and place backfill over the tunnel to protect it and to prevent movement.

In this case the excavation and maintenance of the trench will present substantial problems. At some locations the trench depth below water surface will be near or beyond the limit of capability of modern dredging equipment. Tidal currents as well as water depth fluctuations occasioned by the tides will cause the positioning and anchoring of dredges to be very difficult. Also, it is expected that a fairly large number of boulders which cannot be handled by a suction dredge will be encountered. As the trench is excavated and the regimen of the Knik Arm bottom is distributed, the natural tendency of the Arm to re-establish natural regimen will take place. If the trench is to be maintained, some means, such as windrowing the excavated material either side of the trench, will have to be devised and tested.

The tunnel elements, usually about 300' in length, can be precast in dry dock either in the general locale or at some location which will permit year-round casting. This decision should be left to the contractor as a matter of economics.

The placing of the tunnel elements will be at best difficult due to the rapid water surface elevation fluctuations and the attendant high water velocities. Elements will need to be placed during slack water periods. As indicated earlier in this report, slack water periods are of short duration and, therefore, a placement of the elements must be accomplished rapidly – not an easy task considering the size of the elements and complexity of the installation. Also, work by divers required in making the final connection to the previously laid elements will be extremely hazardous, due to the lack of clarity of the water and the swift currents.

Causeway Approaches

By constructing relatively short causeway approaches the portal to portal length of the tunnel can be reduced to about 9,600' from the 13,000' bluff to bluff distance. These approaches will require special design and protection. In addition, the reduction of the waterway area across this section of the Arm will require the use of protective material over the tunnel to prevent excessive scour and uncovering of the tunnel.

ESTIMATED CONSTRUCTION COST

The following tabulation itemizes the estimated construction cost of a two-lane tunnel at Crossing IV constructed by the trench method. Estimated construction cost for this alternate is $268,000,000.

Table VII-1

ESTIMATE OF COST TWO-LANE TUNNEL AT CROSSING IV

Total Crossing Length - 12,800'
Roadway: 2 @ 16'-0"

Cost Estimate						
Dredging	9,000,000	C.Y.	@	$ 2.70	=	$ 24,300,000
Tunnel backfill	5,300,000	C.Y.	@	2.25	=	11,925,000
Approach embankments	2,600,000	C.Y.	@	2.70	=	7,020,000
Rip-rap protection	1,700,000	Tons	@	22.50	=	38,350,000
Structure concrete, precast tunnel sections	125,000	C.Y.	@	225.00	=	28,125,000
Structural concrete tunnel cast in place, ventilation buildings, and approaches	55,000	C.Y.	@	225.00	=	12,375,000
Reinforcing steel	29,000,000	Lbs.	@	0.32	=	9,250,000
Tremie and ballast concrete	165,000	C.Y.	@	180.00	=	29,700,000
Steel membrane	1,300,000	S.F.	@	9.00	=	11,700,000
Launch, outfit, sink, join and underpack precast elements	30	EA.	@	1,080,000.00	=	32,400,000
Ventilation, illumination and other mech.-elect. work		L.S.	@			4,500,000
Miscellaneous ($\pm$5%)						10,427,000
Total for Structure						$218,972,000
Mobilization						$ 25,000,000
Total for Crossing						$243,972,000
Contingencies and Variations ($\pm$10%)						$ 24,028,000
TOTAL ESTIMATED CONSTRUCTION COST						$268,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

Page

SECTION VIII

combination bridge causeway crossing

combination bridge - causeway crossing

GENERAL

Crossing V is the only location for which it was deemed that a combination bridge–causeway type structure had any merit. Consequently, the combination was studied for only this location.

BRIDGE LAYOUT

Economy of construction is the only control over bridge type and span length for the bridges over the Goose Bay and Eagle Bay channels. As in the case of the total bridge alternate for this location, the 400-foot span, double-decked, orthotropic Warren Truss is the most desirable bridge type. See Figure V-1.

For the Goose Bay and Eagle Bay channels, the main bridge was assumed to extend from the bluff line to the mud line of the center shoal area at mean sea level. Approach structures, which extend beyond these limits, are required to separate the profile grades of the eastbound and westbound roadways. This yields a structure length of 7,967'-0" for the Goose Bay channel and 6,805'-0" for the Eagle Bay channel for an overall bridge length of 14,772'-0". This layout is illustrated by Plate A-8, Appendix A.

FEATURES OF SUPERSTRUCTURE

Goose Bay Channel –	Main Bridge :	16 Spans at 400'	=	6,400'
	West Approach:		=	747'
	East Approach :		=	820'
		Bridge Length		7,967'
Eagle Bay Channel –	Main Bridge :	13 Spans at 400'	=	5,200'
	West Approach:		=	820'
	East Approach :		=	785'
		Bridge Length		6,805'
		Total Bridge Length	=	14,772'

The roadway dimensions, roadway deck trusses and approach spans are similar to those considered for the bridge at Line IV.

FEATURES OF SUBSTRUCTURE

The Features of Substructure are similar to those for the bridge at Line IV.

BRIDGE CONSTRUCTION METHODS

In general, the discussion of construction methods for the bridge at Line IV, applies to the bridges required for this alternate. An added consideration, however, is the scour which will occur in the Goose Bay and Eagle Bay channels. This scour will be the result of the increase in velocity of flow through the channels caused by the causeway which forms a constriction to normal flow.

Two approaches to handling this problem were considered. One was to armor the channel to retard the scour and the other was to allow the channel to scour and design the piers accordingly. The latter approach was chosen for this estimate.

This approach, that is to allow the channel to scour to a stable configuration and then construct the piers, has the disadvantage of requiring the causeway to be constructed first. Also, there would need to be a time lag between the construction of the causeway and the construction of the bridge to allow the channels to stabilize.

To construct the piers at the same time as the causeway to some estimated bottom configuration, after scour, is considered very speculative and is not recommended. A bottom of channel configuration which accounts for the scour can be assumed for estimating purposes but cannot be considered reliable for final design.

The other approach to the problem, that is to design the piers for the existing bottom of channel profile and then armor the bottom against the scour was temporarily discarded because of the considerable expense of the armoring and its uncertain reliability.

The speculative nature of the method and timing of pier construction could be alleviated to a degree by the advance performance of hydraulic model tests to ascertain approximate depths of scour, approximate resulting velocities, methods of controlling channel locations and depths, and time for stabilization. Considerable bottom topography and tide data must be obtained prior to model construction.

CAUSEWAY EMBANKMENT

The embankment for the causeway portion of this alternate will be very similar to the embankment used for the causeway dam alternate. Construction material is assumed to be rock, for seismic stability, and construction methods will parallel those used for the causeway dam alternate. It will be necessary to construct as much of the embankment as possible with barge transported material and considerable rehandling of material is anticipated in the later stages of construction. There is no need to seal the embankment, and therefore, the sand cover used in the Causeway Dam Typical Section has been eliminated. The one on five slope has been included on the upstream side to make allowances for greater wave heights generated by the higher wind velocities anticipated from the north.

ESTIMATED CONSTRUCTION COST

The construction cost for the combination bridge–causeway crossing at Line V is estimated to be $230,000,000 (including 10% for contingencies and variations). For detailed cost estimates see the following tabulation and Appendix D.

Table VIII-1

ESTIMATE OF COST BRIDGE-CAUSEWAY COMBINATION AT CROSSING V

Goose Bay:	2 Approach Structures @ 747' and 820'	= 1,567'-0"
	Main Bridge 16 Spans @ 400'-0"	= 6,400'-0"
		7,967'-0"
Eagle Bay:	2 Approach Structures @ 820' and 785'	= 1,605'-0"
	Main Bridge 13 Spans @ 400'	= 5,200'-0"
		6,805'-0"

Total Bridge Length = 14,772'-0"

Roadway: 2 @ 30'-0"

APPROACH STRUCTURES:						
Superstructure						
Structural Steel (A588)	3,475,000	Lbs.	@	$ 0.64	=	$ 2,224,000
Cast Steel	70,000	Lbs.	@	1.45	=	101,500
Concrete in Deck	2,560	C.Y.	@	200.00	=	512,000
Reinforcing Steel	695,000	Lbs.	@	0.28	=	194,600
Handrail	6,335	L.F.	@	20.00	=	126,700
Sub-Total Superstructure						$ 3,158,800
Substructure						
Concrete in Columns and Caps	960	C.Y.	@	$ 200.00	=	$ 192,000
Concrete in Footings and Abutments	1,040	C.Y.	@	170.00	=	176,800
Concrete in Retaining Walls	4,480	C.Y.	@	170.00	=	761,600
Reinforcing Steel	690,000	Lbs.	@	0.28	=	193,200
14HP73 (45' Long)	51,720	L.F.	@	30.00	=	1,551,600
Excavation (Above Grade)	130,000	C.Y.	@	2.40	=	312,000
Excavation (Below Grade)	8,800	C.Y.	@	10.00	=	88,000
Sub-Total Substructure						$ 3,275,200
MAIN BRIDGE:						
Superstructure						
Structural Steel (A588)	49,650,000	Lbs.	@	$ 0.64	=	$ 31,776,000
Cast Steel	970,000	Lbs.	@	1.44	=	1,396,800
Asphaltic Wearing Surface	80,050	S.Y.	@	16.00	=	1,280,800
Deck Protection	80,050	S.Y.	@	2.00	=	160,100
Handrail	46,400	L.F.	@	20.00	=	928,000
Sub-Total Superstructure						$ 35,541,700
Substructure						
Pier Excavation	43,000	C.Y.	@	$ 31.60	=	$ 1,358,800
36" Dia. Pipe Piles	54,000	L.F.	@	206.00	=	11,124,000
Pre-Cast Concrete	10,500	C.Y.	@	1,310.00	=	13,755,000
Tremie Concrete	18,800	C.Y.	@	165.00	=	3,102,000
Shell Filler Concrete	18,100	C.Y.	@	165.00	=	2,986,500
Pier Cap Concrete	4,600	C.Y.	@	497.00	=	2,286,200
Reinforcing Steel	6,150,000	Lbs.	@	0.51	=	3,136,500
Shaft Armoring (Steel)	980,000	Lbs.	@	1.20	=	1,176,000
Foundation Armor Rock	58,000	C.Y.	@	24.25	=	1,406,500
Sub-Total Substructure						$ 40,331,500
Sub-Total Approach Structures and Main Bridge						$ 82,307,200
Causeway Fill						
Zone 1 Rockfill 0.5'-1.0'	4,836,000	C.Y.	@	$ 10.40	=	$ 50,294,400
Zone 2 Rockfill 1.0'-2.0'	1,196,000	C.Y.	@	11.40	=	13,634,400
Riprap - 36" Dia.	1,397,500	C.Y.	@	14.40	=	20,124,000
Roadway Paving	14,700	L.F.	@	56.00	=	823,200
Sub-Total Causeway						$ 84,876,000
Total for Structure						$167,183,200
Mobilization						$ 42,000,000
Total for Crossing						$209,183,200
Contingencies and Variations ±10%						$ 20,816,800
TOTAL ESTIMATED CONSTRUCTION COST						$230,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

conclusions and recommendations

SECTION IX

conclusions and recommendations

GENERAL

From the results of the research, analyses and investigations conducted during the preparation of this report, it was found that a bridge at Crossing IV is the most favorable alternative for a highway structure across Knik Arm. The next most favorable alternative is a causeway dam at Crossing V. Other alternatives are deemed to be not feasible or to have higher estimated cost.

Basically, the most certain and least costly method of successfully resisting the forces and conditions of tides, ice, winds, and waves is to present the least possible physical hindrance to these forces and conditions. The causeway dam presents a tremendous obstacle to the forces of nature, as tides and ice attempt to move up and down Knik Arm. Soils reports indicate, "...a bridge crossing presently appears to be a more positive and dependable alternative." It is noted that further soils exploration might indicate the need for rock fill causeway dam side slopes of 5:1 (for seismic safety). This would greatly increase the construction quantities and cost to construct a causeway dam.

A bridge at Crossing IV also presents obstacles to the forces and conditions of nature, but the 32 piers are separated by 400 ft. of open water, thus permitting tidal and ice flow to pass with a minimal amount of obstruction.

RECOMMENDATIONS

Bridge

If it is decided to proceed with a bridge project, the following steps are recommended:

1. Concurrence of federal authorities should be obtained in the location of the eastern terminus and approach on military property.

2. Field surveys should be performed to determine topographic features above and below water level in sufficient detail for a refined preliminary design and to establish local bench marks.

3. A program of preliminary subsurface explorations should be undertaken in the vicinity of the crossing, on both land and water, to determine the best location for the bridge.

4. The preliminary design should be re-examined in the light of the results of the preliminary subsurface explorations to refine the determinations of feasibility and costs.

5. A test structure should be constructed in Knik Arm for the purpose of determining the magnitude of ice loadings on bridge piers. This test structure should, ideally, be constructed by procedures similar to those contemplated for actual pier construction. A test structure would also provide valuable information regarding the extent of scour around bridge piers.

6. Laboratory studies of bridge piers should be performed to evaluate bottom scour conditions, pier configurations, scour protection measures and channel or bottom stabilization. Prior to this model testing, additional tide data and dye studies of existing currents would be necessary.

Following the completion and evaluation of the above steps, the detailed field surveys, design subsurface explorations and final design could be made. A suggested schedule for the subsequent development and construction of a bridge at Crossing IV is shown in Section II, Figure II-2.

Causeway Dam

If it is decided to proceed with further development of a causeway dam, the following steps are recommended:

1. Concurrence of federal authorities should be obtained in the location of the eastern terminus and approach on military property.

2. Field surveys should be performed to determine topographic features above and below water level in sufficient detail for a refined preliminary design and to establish local bench marks.

3. A program of preliminary subsurface explorations should be undertaken in the vicinity of the crossing, on both land and water, to determine the best location for the causeway dam and spillway.

4. A program of tidal range, elevation and current data collection in the vicinity of the crossing should be carried out.

5. The preliminary design should be re-examined to refine the feasibility and costs in light of data acquired through the preliminary subsurface exploration and testing, and tide data collection programs.

6. Model studies should be performed to further delineate material requirements, type and location of spillway, spillway configuration, typical section and construction procedures.

7. The source of construction material should be the subject of an extensive field exploration program.

8. An ecological inventory of the upper Arm should be conducted.

CONCLUSIONS

From the studies it is concluded:

1. A highway crossing Knik Arm is feasible.

2. A bridge crossing is substantially more economical to construct than a causeway dam.

3. A bridge crossing would have no direct effect on the natural environment of the Knik Arm area.

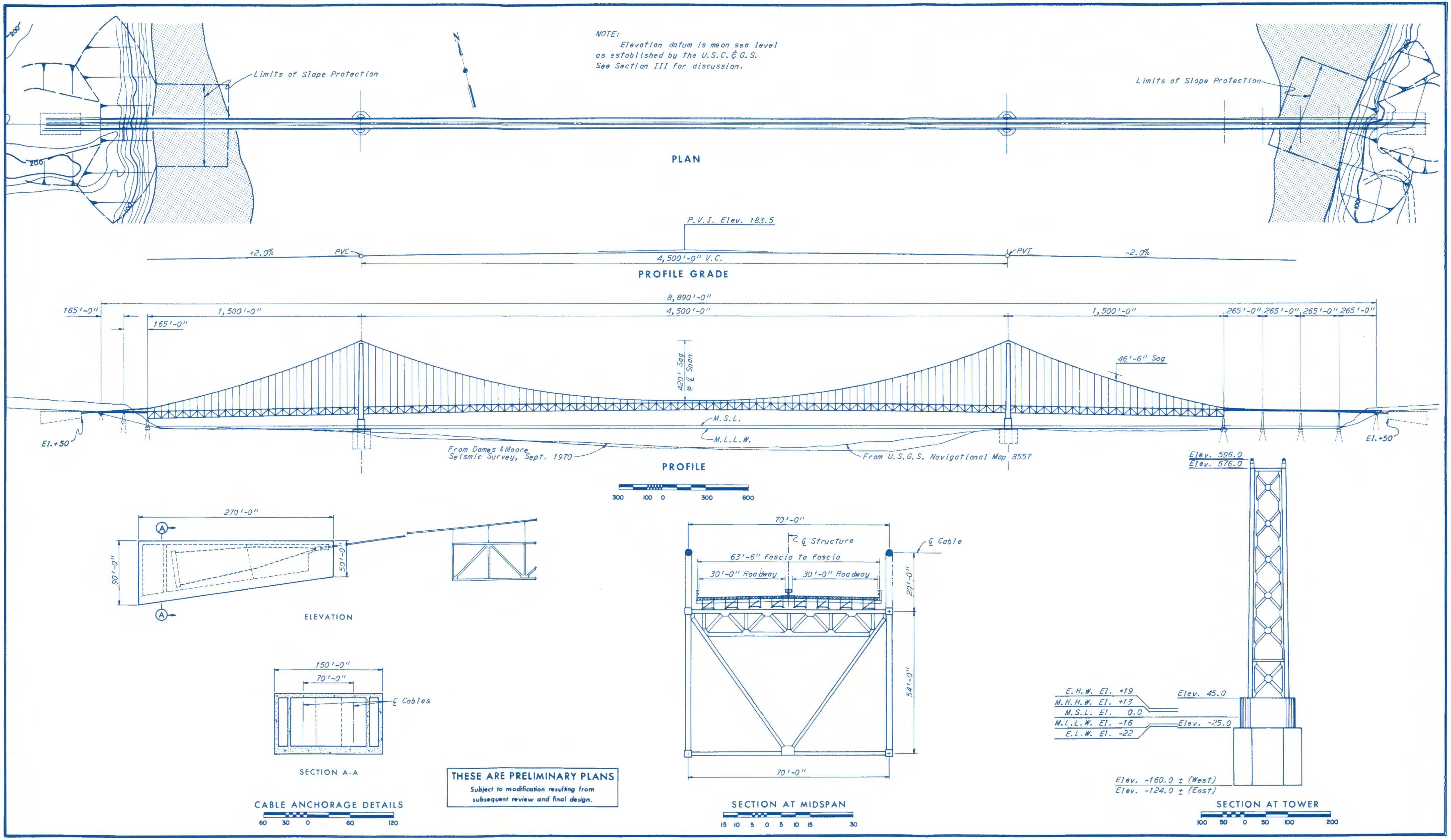
4. The most favorable structure is a bridge located at, or near, Crossing IV.

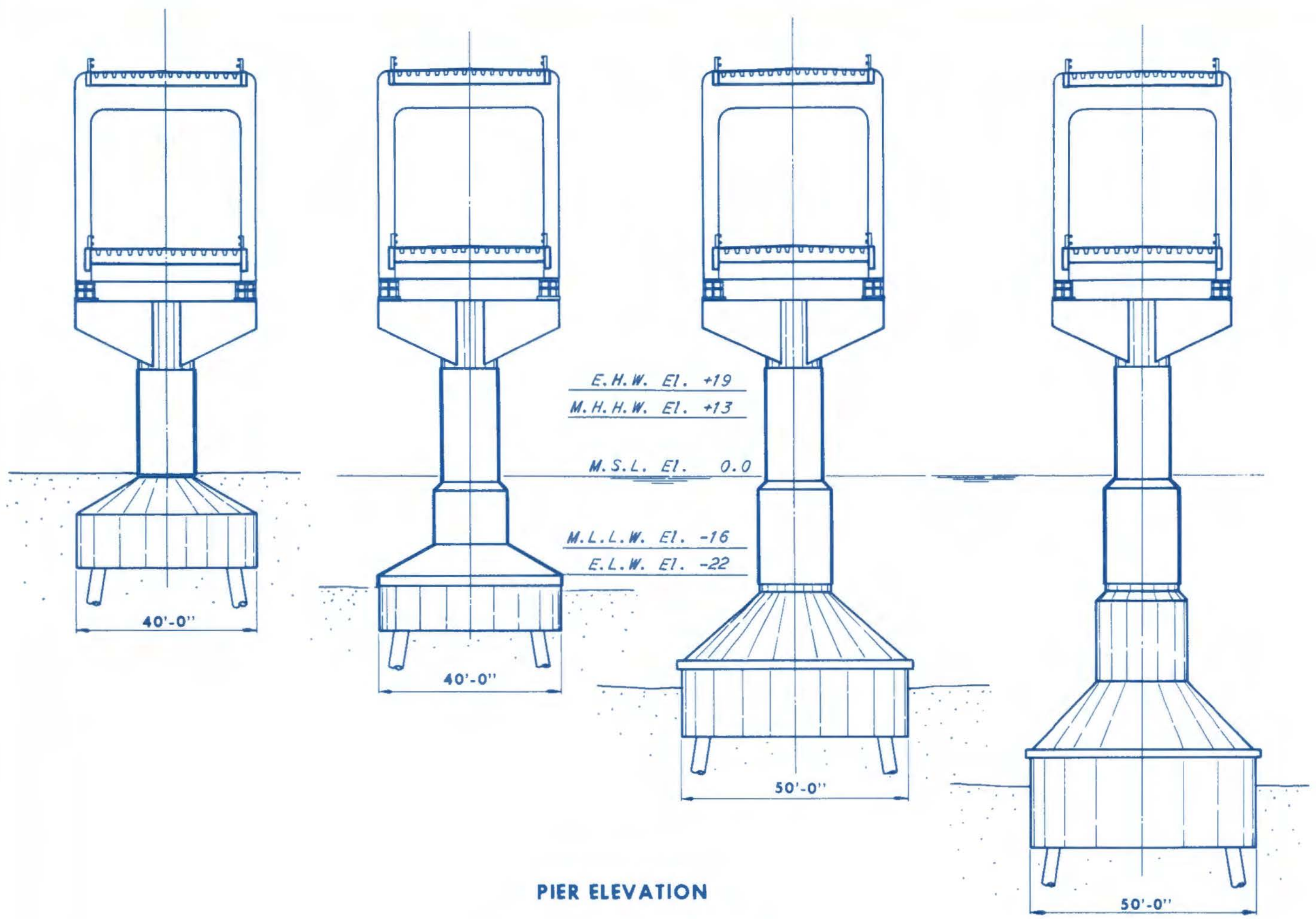
5. Additional engineering design studies should only be initiated after additional subsurface and tide information is available.

INDEX TO PLAN AND PROFILE PLATES APPENDIX A

preliminary plans

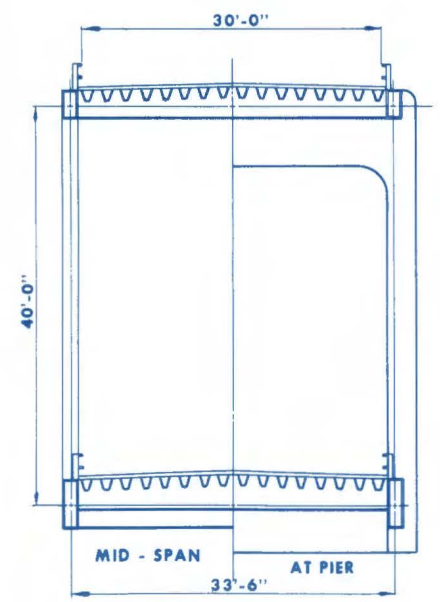
APPENDIX A


NOTE:
Elevation datum is mean sea level as established by the U.S.C.& G.S.
See Section III for discussion.
Limits of Slope Protection
Limits of Slope Protection
N
PLAN
P.V.I. Elev. 183.5
+2.0%
PVC
4,500'-0" V.C.
PVT
-2.0%
PROFILE GRADE
8,890'-0"
165'-0"
1,500'-0"
4,500'-0"
1,500'-0"
165'-0"
265'-0" 265'-0" 265'-0" 265'-0"
420' Sag @ ℄ Span
46'-6" Sag
M.S.L.
M.L.L.W.
El.+50
El.+50
From Dames & Moore Seismic Survey, Sept. 1970
From U.S.G.S. Navigational Map 8557
PROFILE
300 100 0 300 600
270'-0"
90'-0"
50'-0"
A
A
ELEVATION
150'-0"
70'-0"
℄ Cables
SECTION A-A
CABLE ANCHORAGE DETAILS
60 30 0 60 120
THESE ARE PRELIMINARY PLANS
Subject to modification resulting from subsequent review and final design.
70'-0"
℄ Structure
℄ Cable
63'-6" fascia to fascia
30'-0" Roadway
30'-0" Roadway
20'-0"
54'-0"
70'-0"
SECTION AT MIDSPAN
15 10 5 0 5 10 15 30
Elev. 596.0
Elev. 576.0
E.H.W. El. +19
M.H.H.W. El. +13
M.S.L. El. 0.0
M.L.L.W. El. -16
E.L.W. El. -22
Elev. 45.0
Elev. -25.0
Elev. -160.0 ± (West)
Elev. -124.0 ± (East)
SECTION AT TOWER
100 50 0 50 100 200



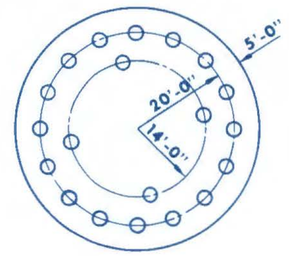
PIER ELEVATION

20 10 0 10 20 30 40

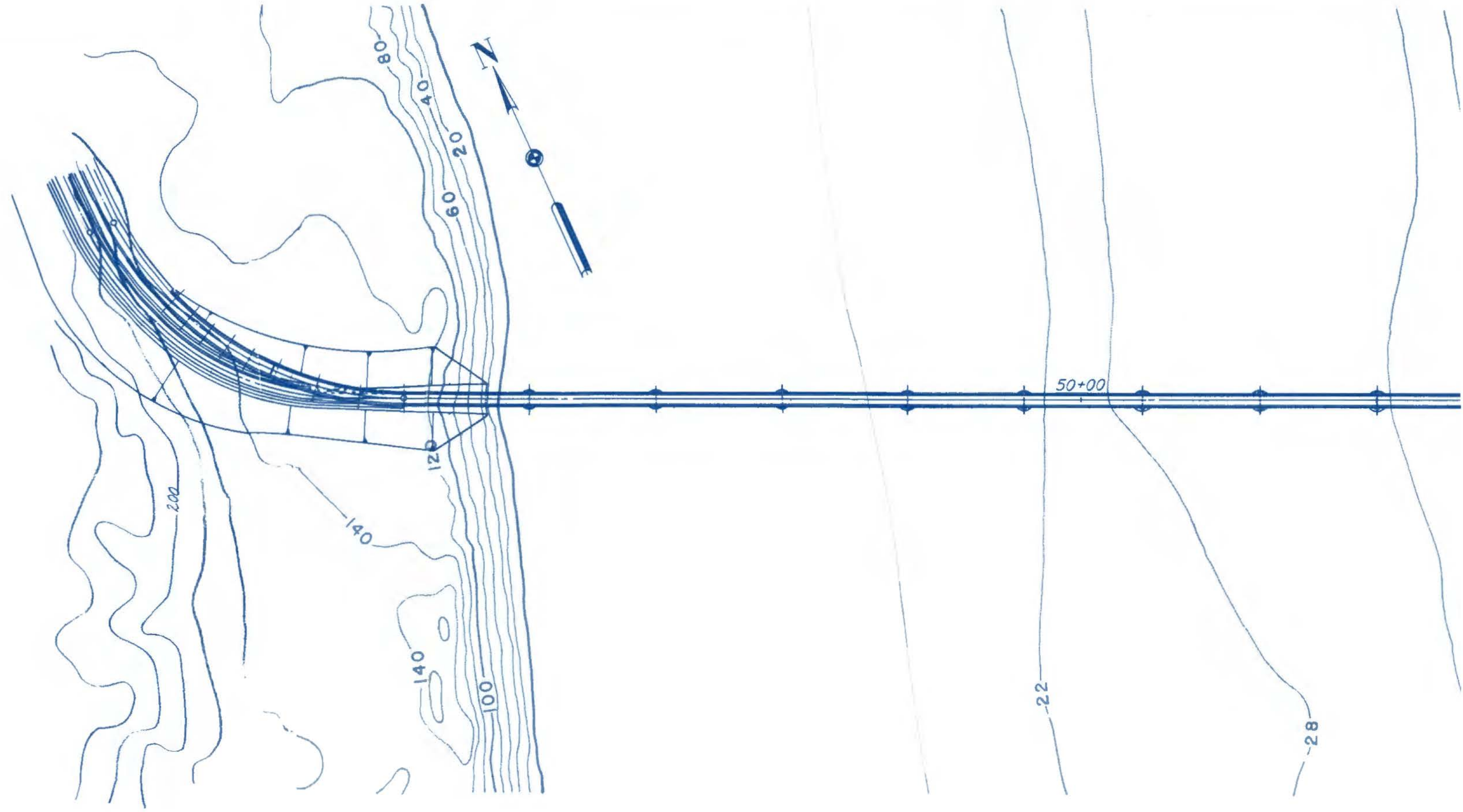


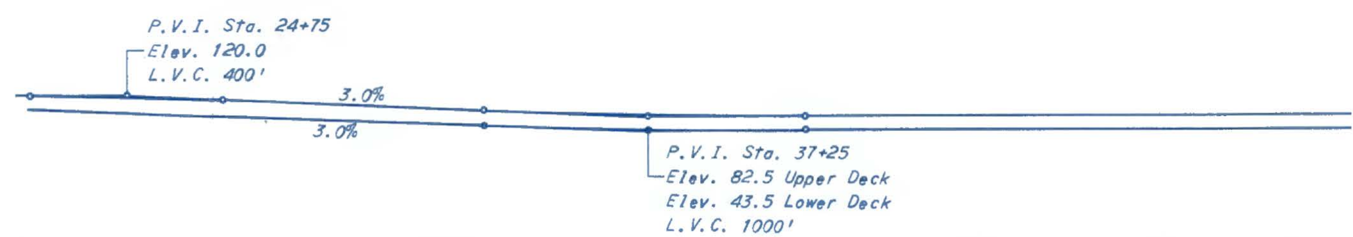
TRUSS SECTION

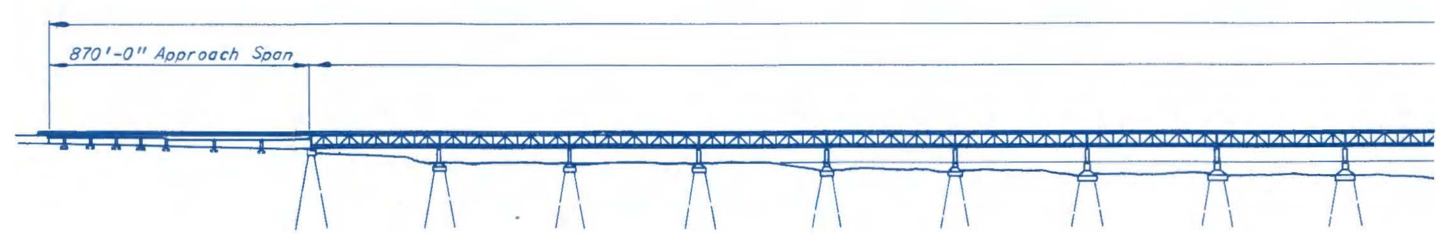
10 5 0 5 10 20



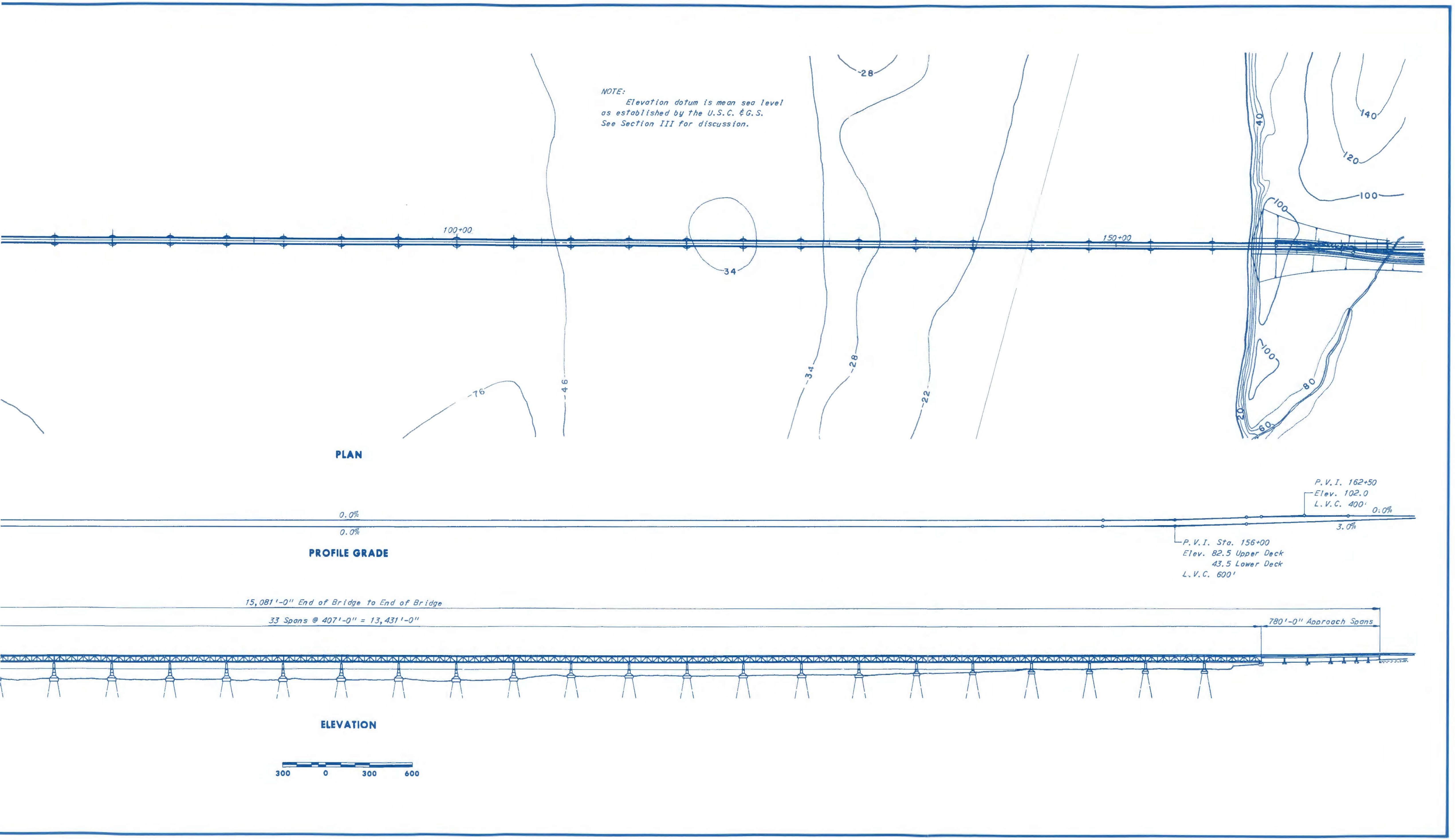
PILE PLAN

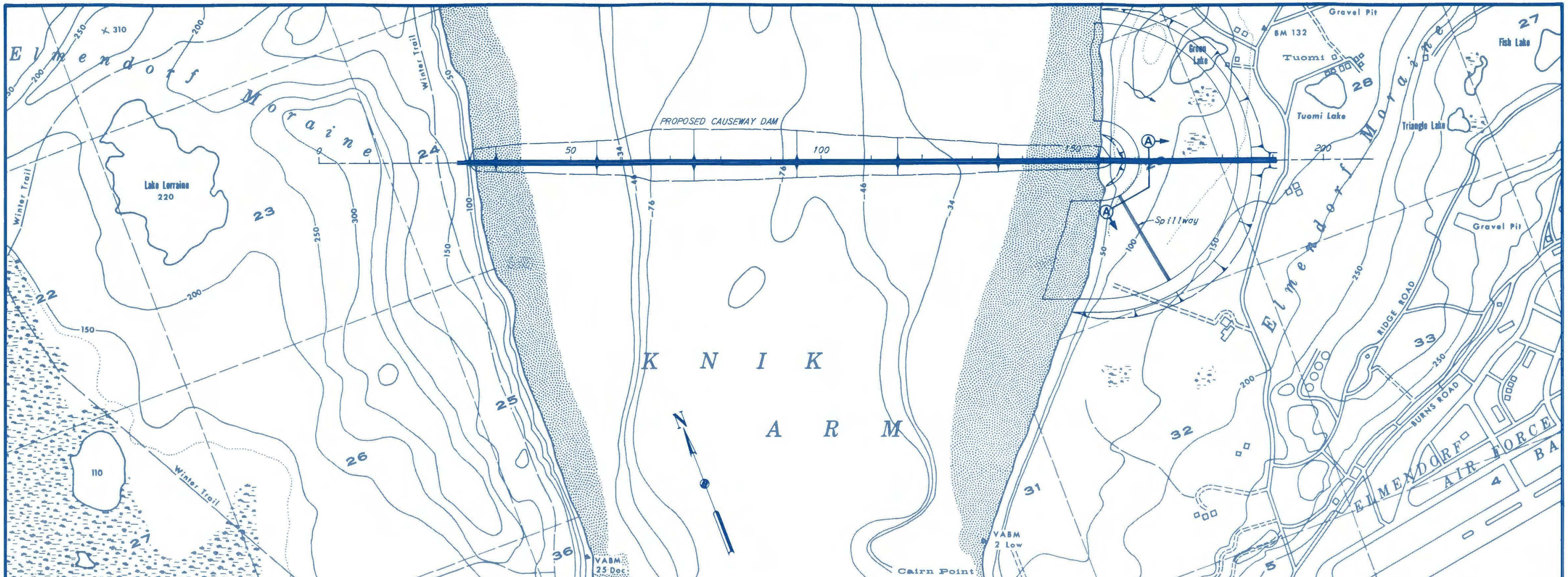






THESE ARE PRELIMINARY PLANS
Subject to modification resulting from subsequent review and final design.


NOTE:
Elevation datum is mean sea level as established by the U.S.C. & G.S. See Section III for discussion.
-28
-34
-46
-76
-28
-34
-22
40
140
120
100
100
100
80
60
20
100+00
150+00
PLAN
P.V.I. 162+50
Elev. 102.0
L.V.C. 400'
0.0%
0.0%
0.0%
3.0%
P.V.I. Sta. 156+00
Elev. 82.5 Upper Deck
43.5 Lower Deck
L.V.C. 600'
PROFILE GRADE
15,081'-0" End of Bridge to End of Bridge
33 Spans @ 407'-0" = 13,431'-0"
780'-0" Approach Spans
ELEVATION
300 0 300 600

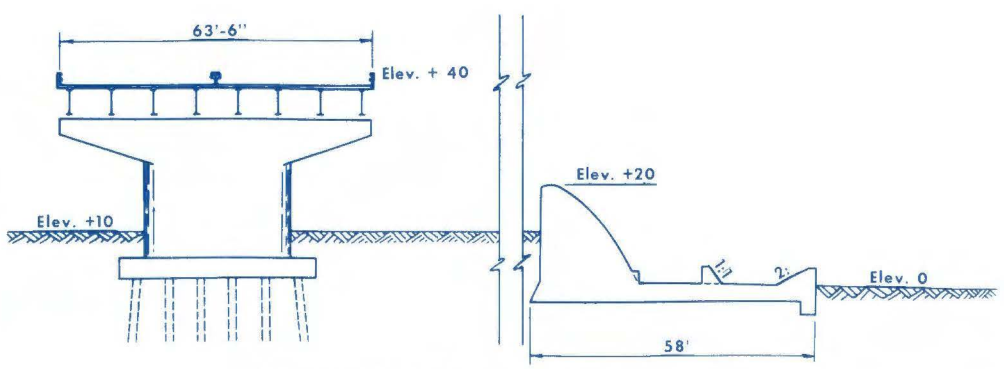


PLAN

THESE ARE PRELIMINARY PLANS
Subject to modification resulting from subsequent review and final design.

NOTE:
Elevation datum is mean sea level as established by the U.S.C.& G.S. See Section III for discussion.

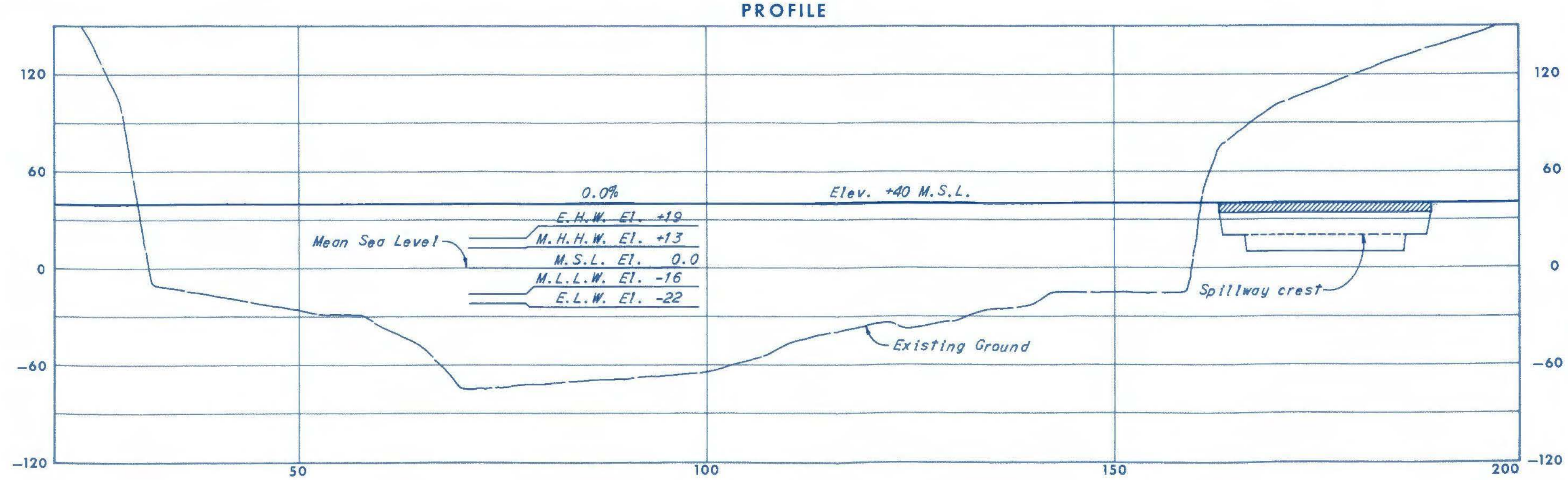
Horiz. 1000 0 1000 2000 3000 4000
Vert. 50 0 50 100 150 200
Scale in Feet



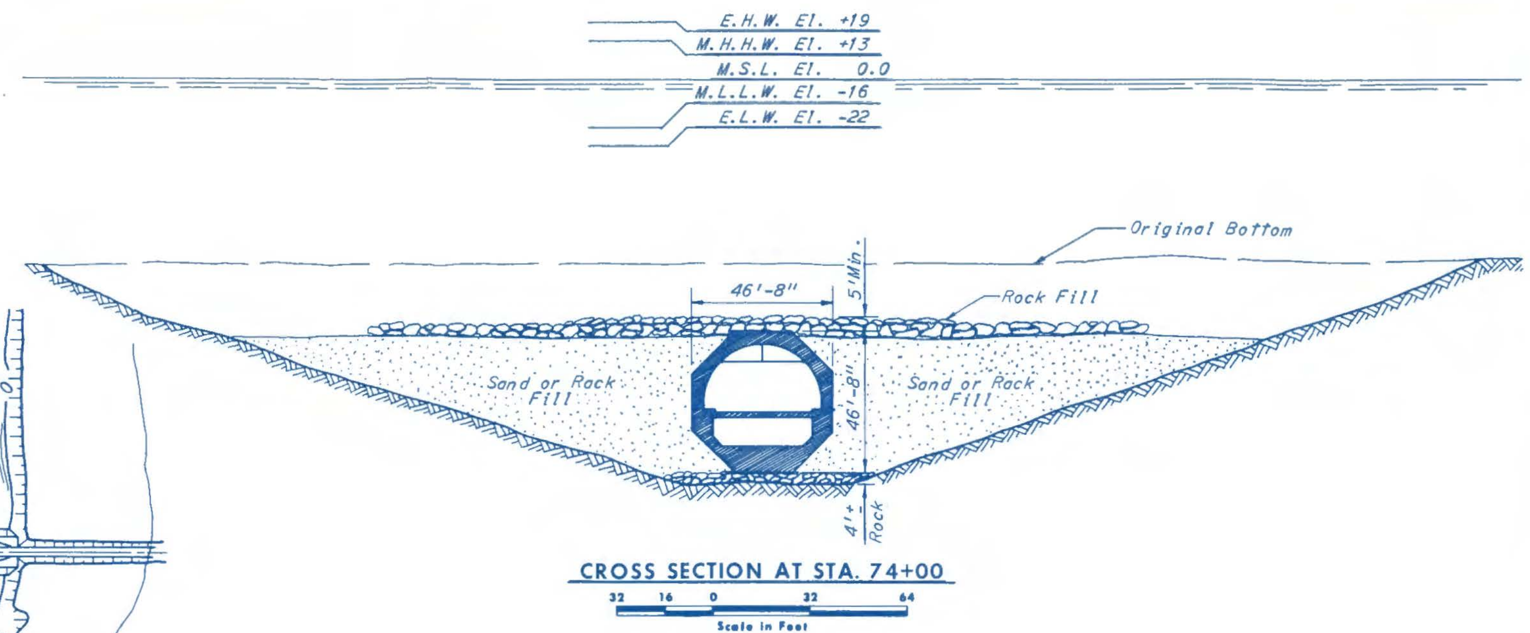
TYPICAL BRIDGE - SPILLWAY SECTION

SECTION "A-A"

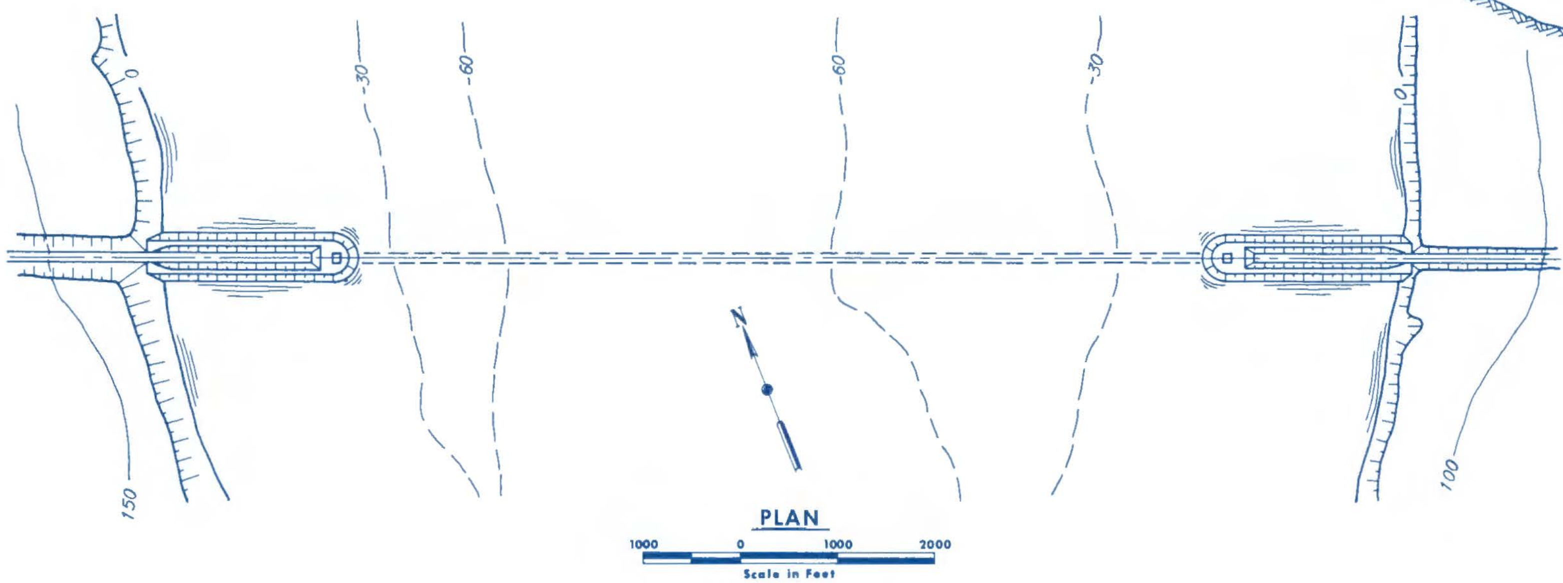
PROFILE



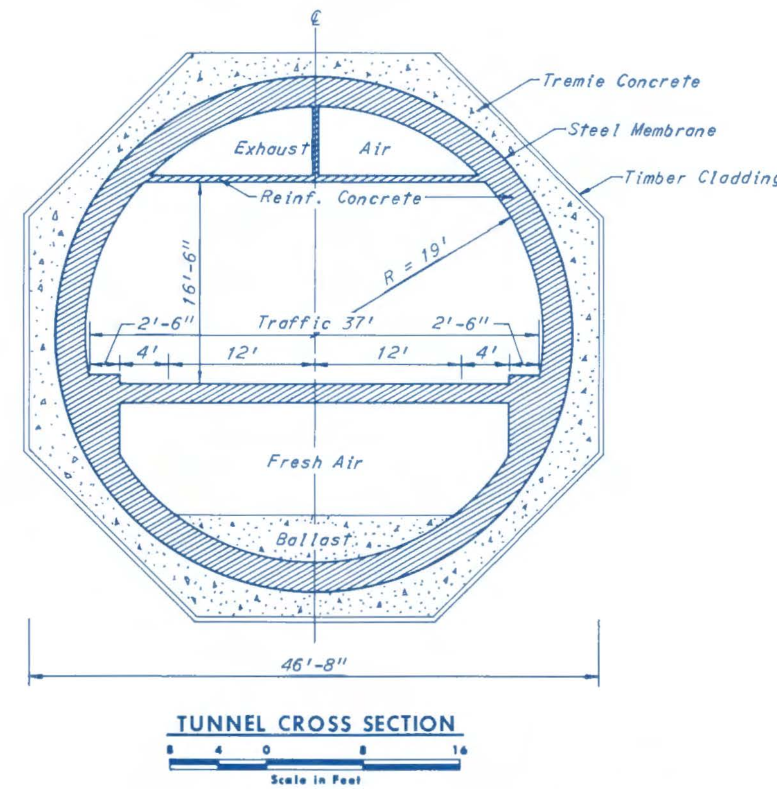
NOTE:
Elevation datum is mean sea level as established by the U.S.C.&G.S. See Section III for discussion.



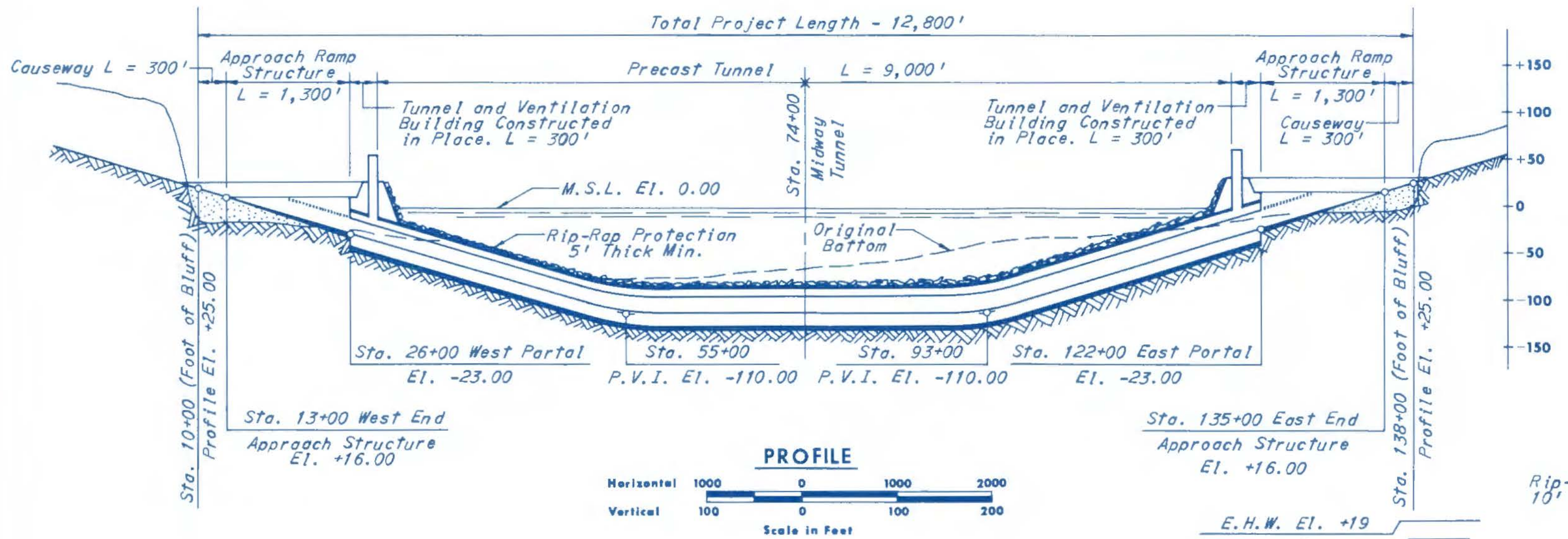
CROSS SECTION AT STA. 74+00



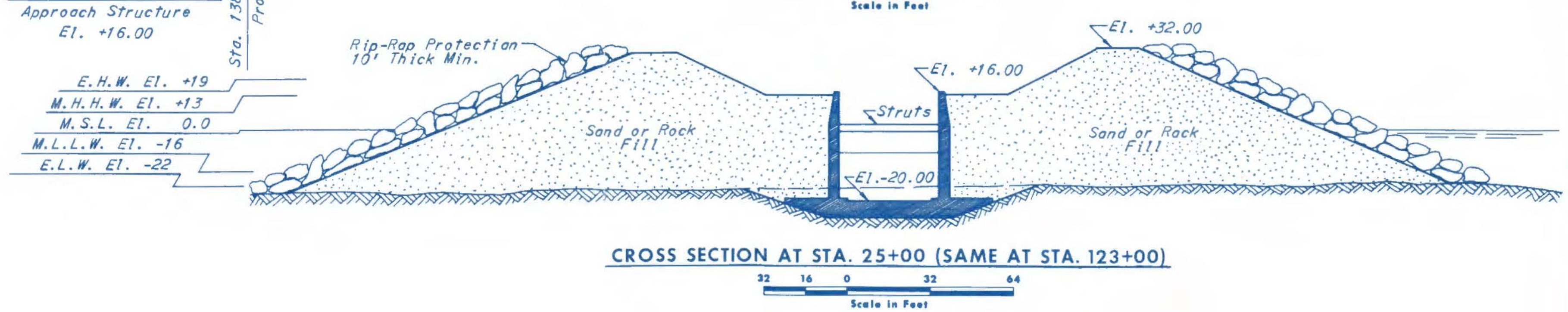
PLAN



TUNNEL CROSS SECTION

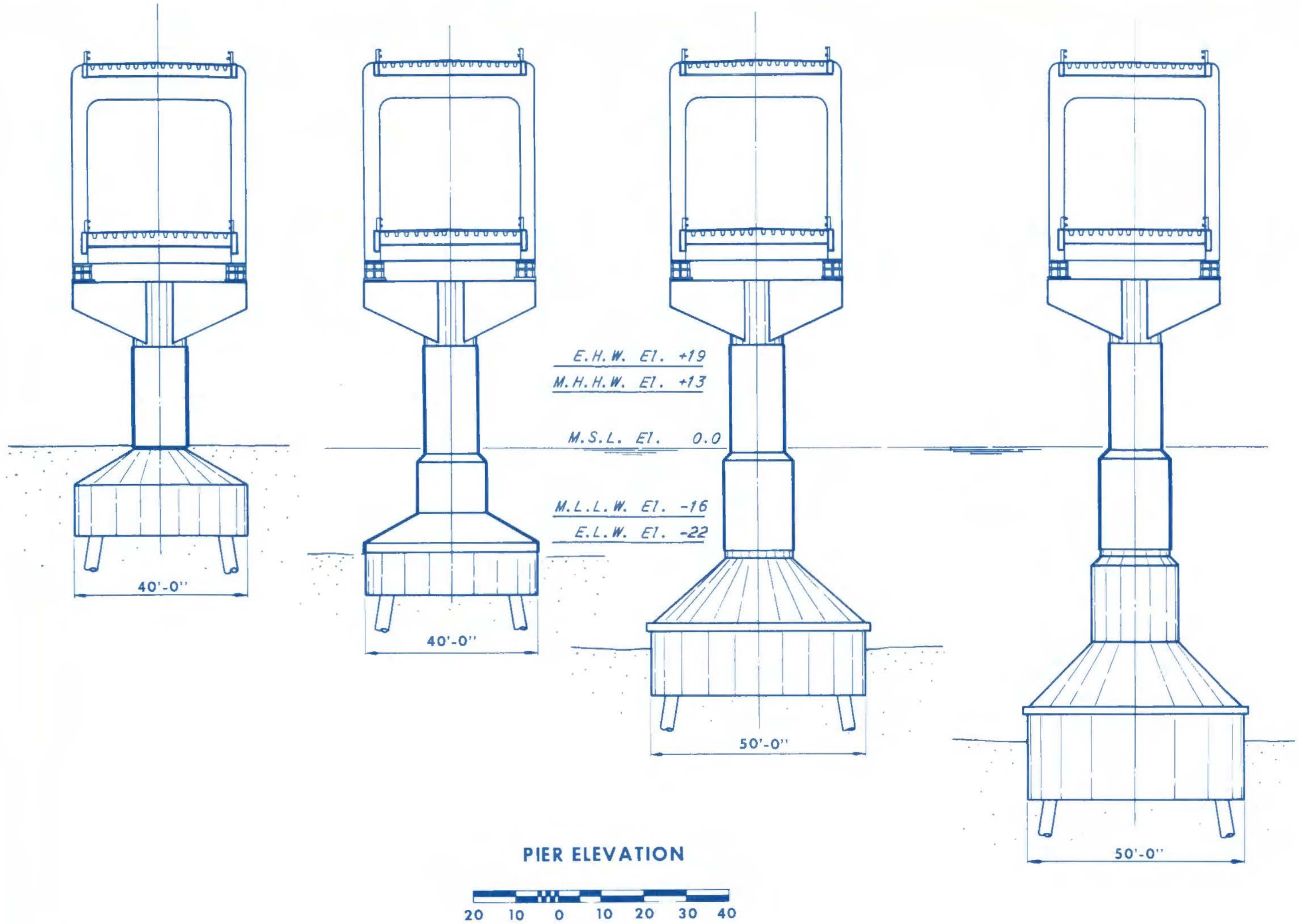


PROFILE

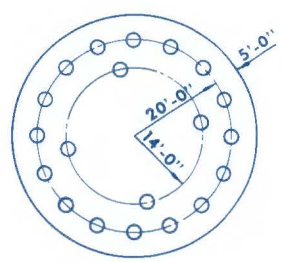


CROSS SECTION AT STA. 25+00 (SAME AT STA. 123+00)

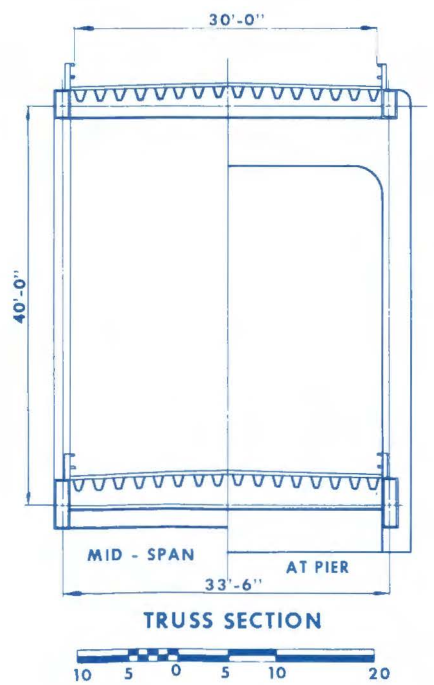
THESE ARE PRELIMINARY PLANS
Subject to modification resulting from subsequent review and final design.



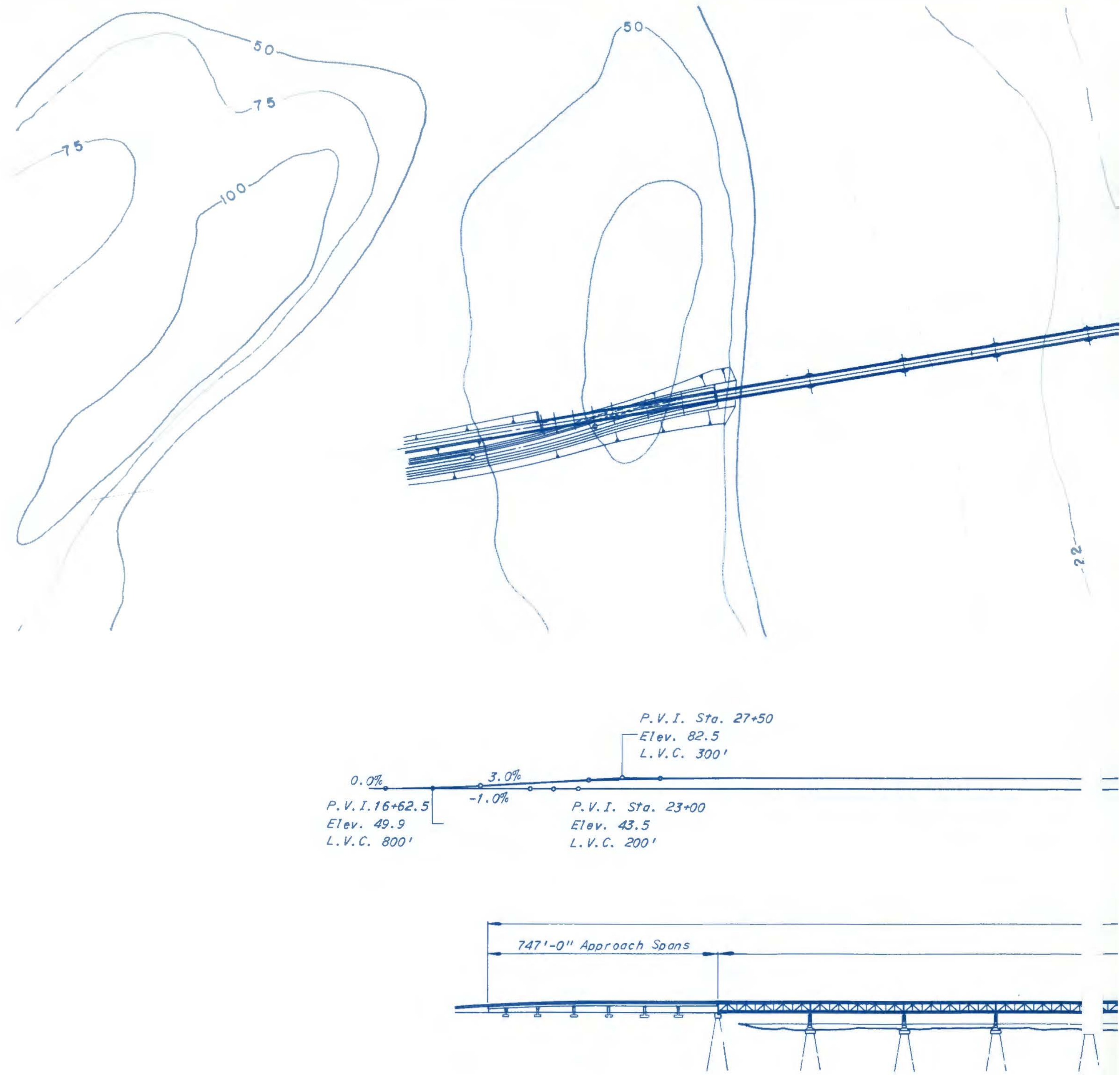
PIER ELEVATION



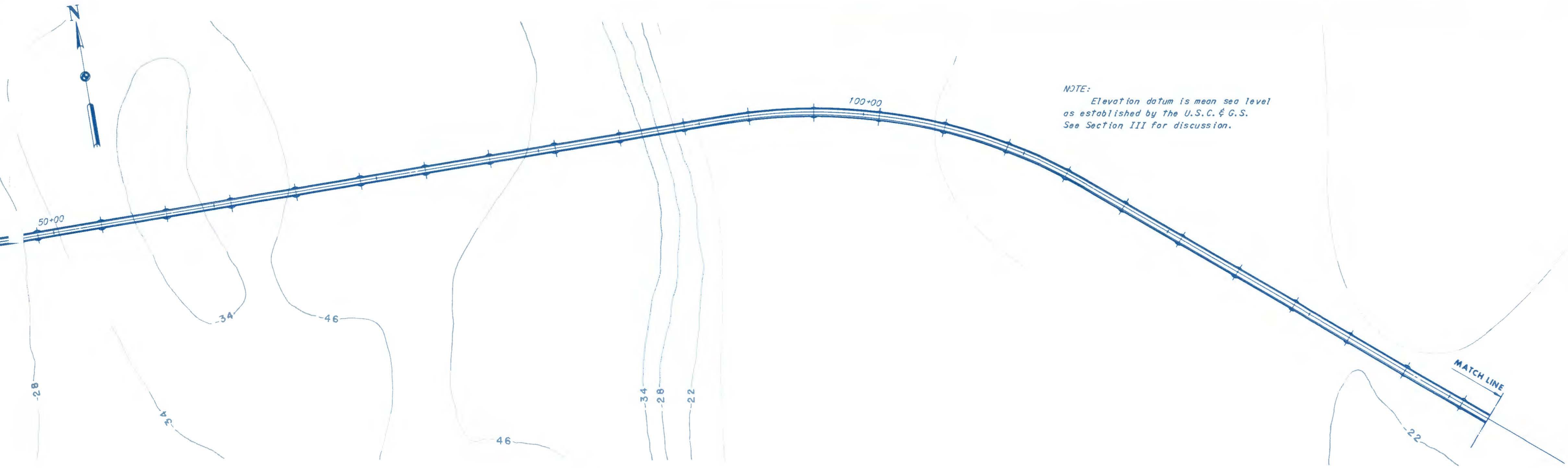
PILE PLAN



TRUSS SECTION

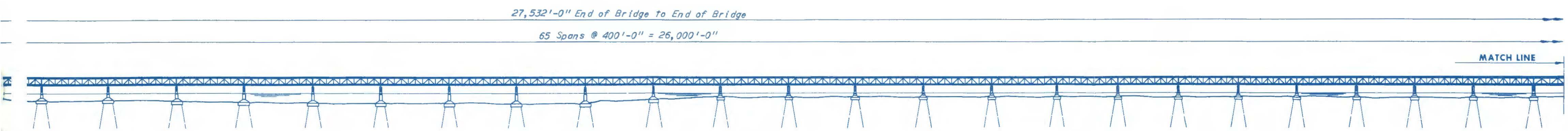


THESE ARE PRELIMINARY PLANS
Subject to modification resulting from subsequent review and final design.

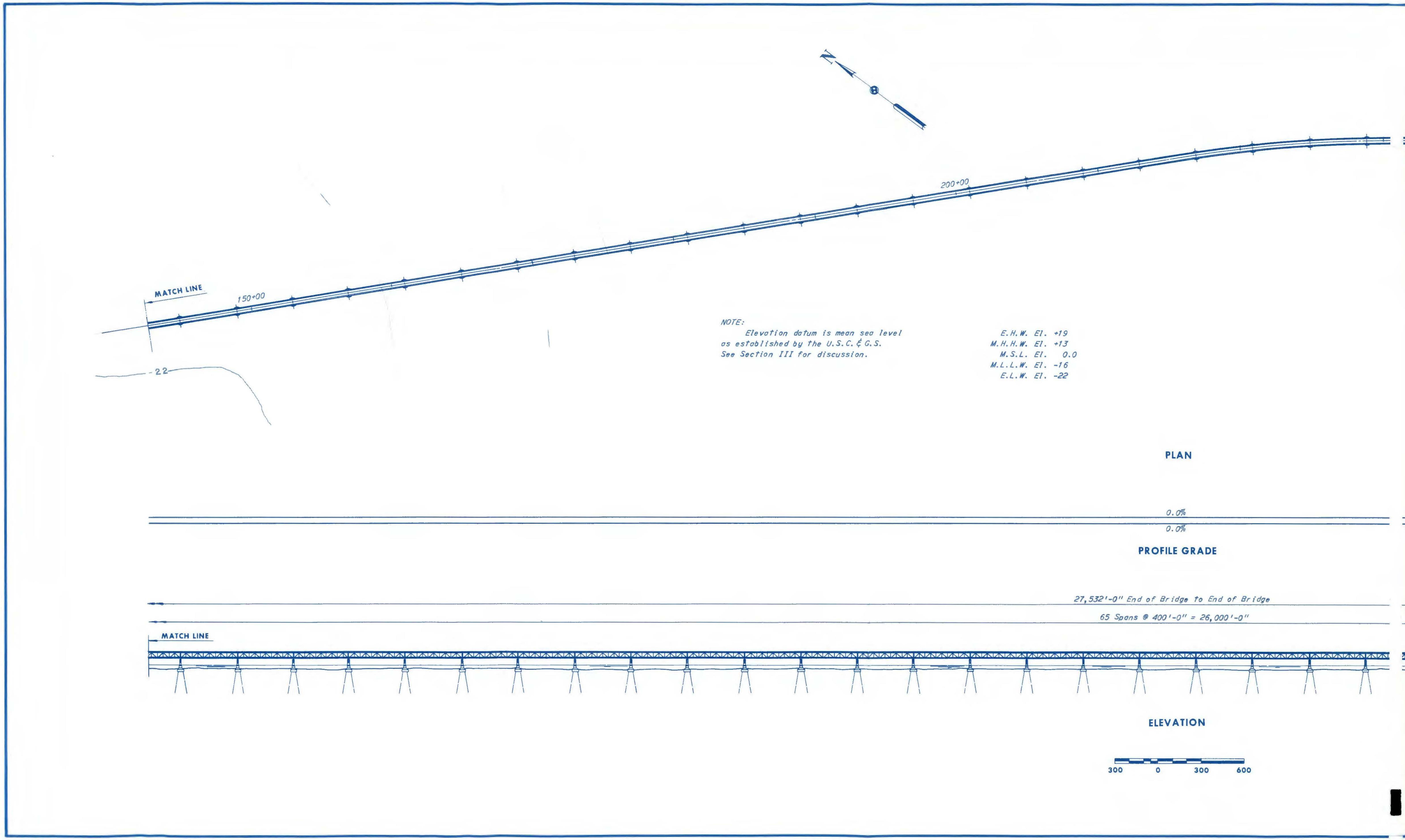


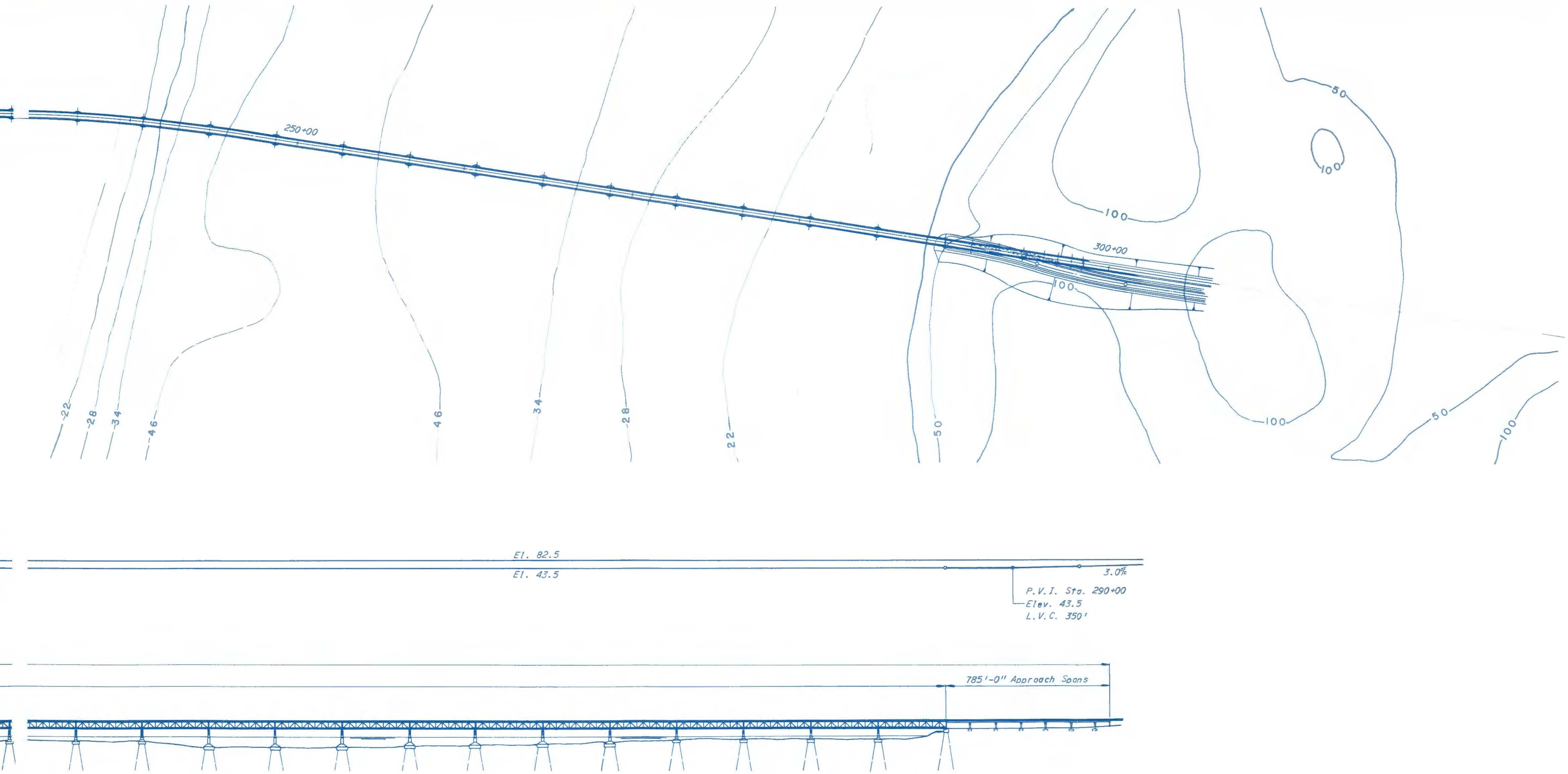
PLAN

PROFILE GRADE



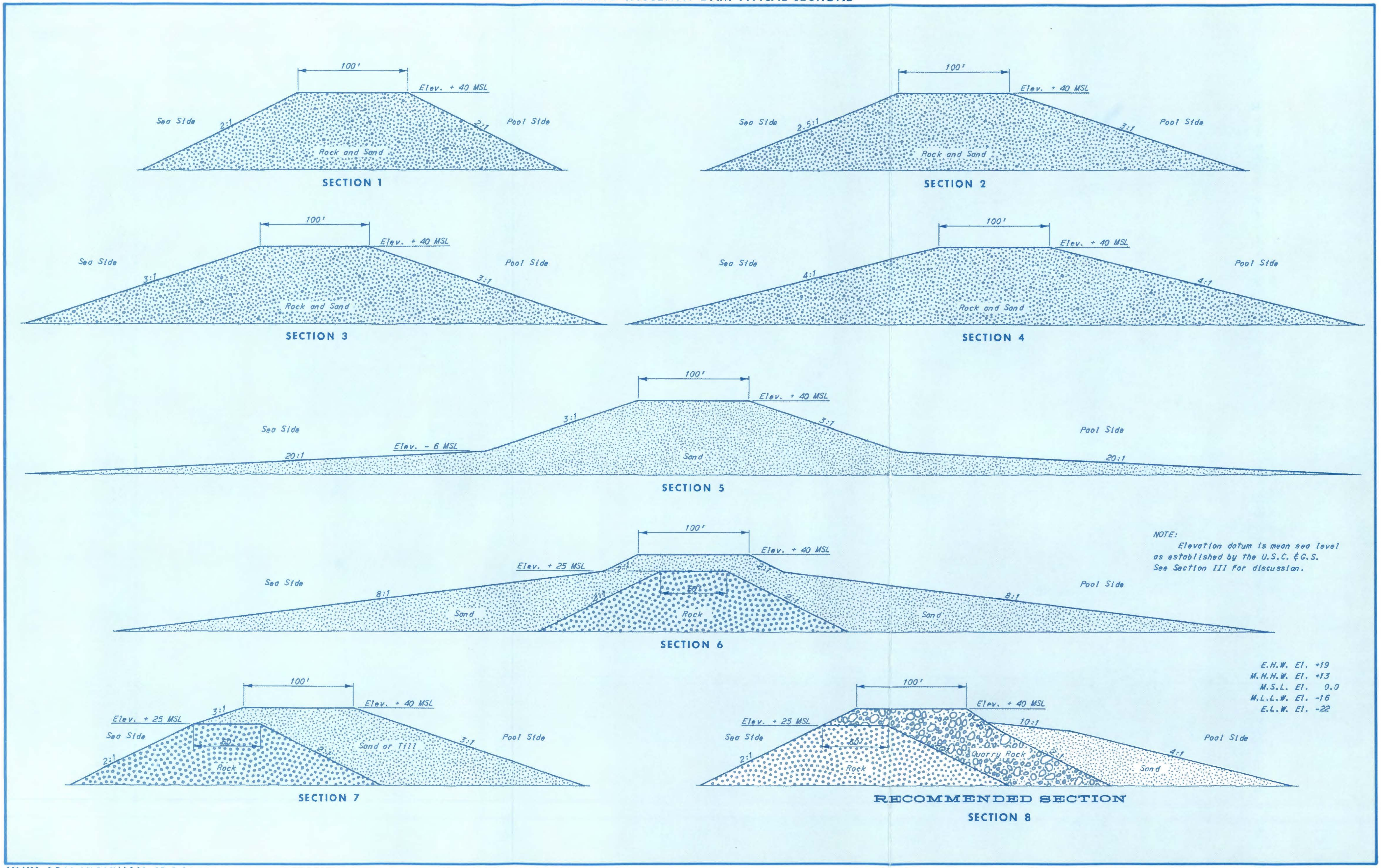
ELEVATION

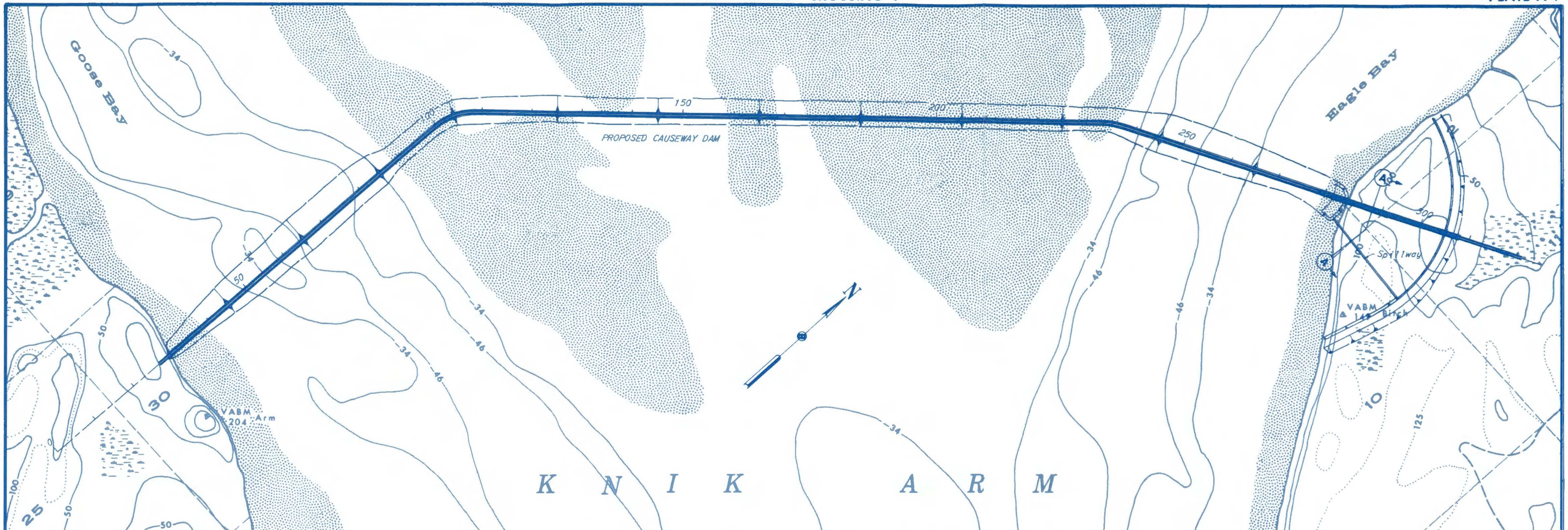

N
MATCH LINE
150+00
200+00
-22
NOTE:
Elevation datum is mean sea level
as established by the U.S.C.& G.S.
See Section III for discussion.
E.H.W. El. +19
M.H.H.W. El. +13
M.S.L. El. 0.0
M.L.L.W. El. -16
E.L.W. El. -22
PLAN
0.0%
0.0%
PROFILE GRADE
27,532'-0" End of Bridge to End of Bridge
65 Spans @ 400'-0" = 26,000'-0"
MATCH LINE
ELEVATION
300
0
300
600



THESE ARE PRELIMINARY PLANS
Subject to modification resulting from subsequent review and final design.

ALTERNATIVE CAUSEWAY DAM TYPICAL SECTIONS



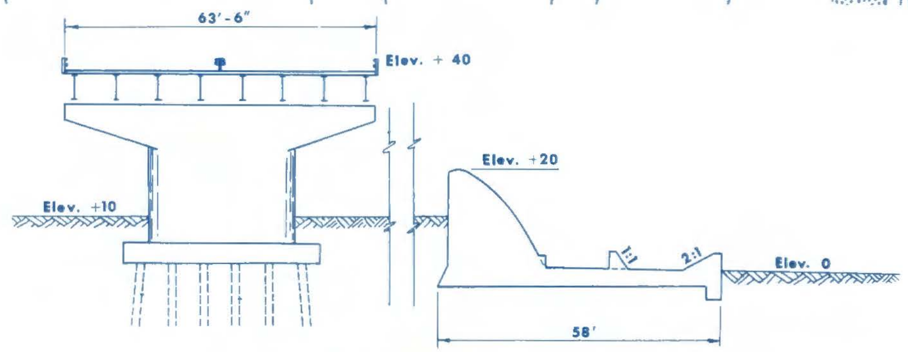


PLAN

NOTE:
Elevation datum is mean sea level as established by the U.S.C. & G.S. See Section III for discussion.

THESE ARE PRELIMINARY PLANS
Subject to modification resulting from subsequent review and final design.

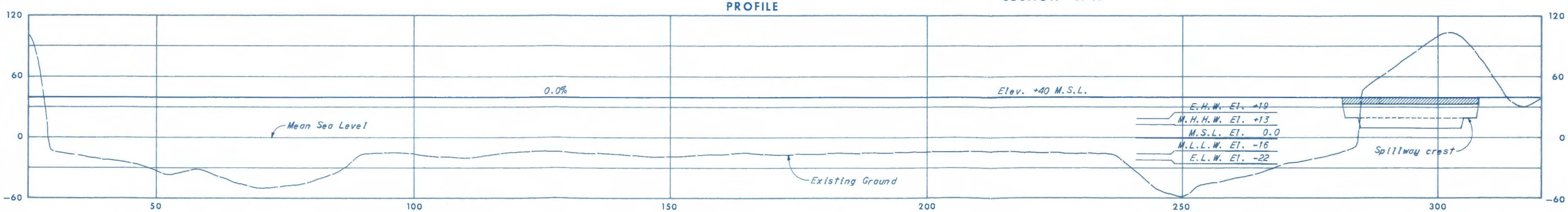
Horiz. 1000 0 1000 2000 3000 4000
Vert. 50 0 50 100 150 200
Scale in Feet

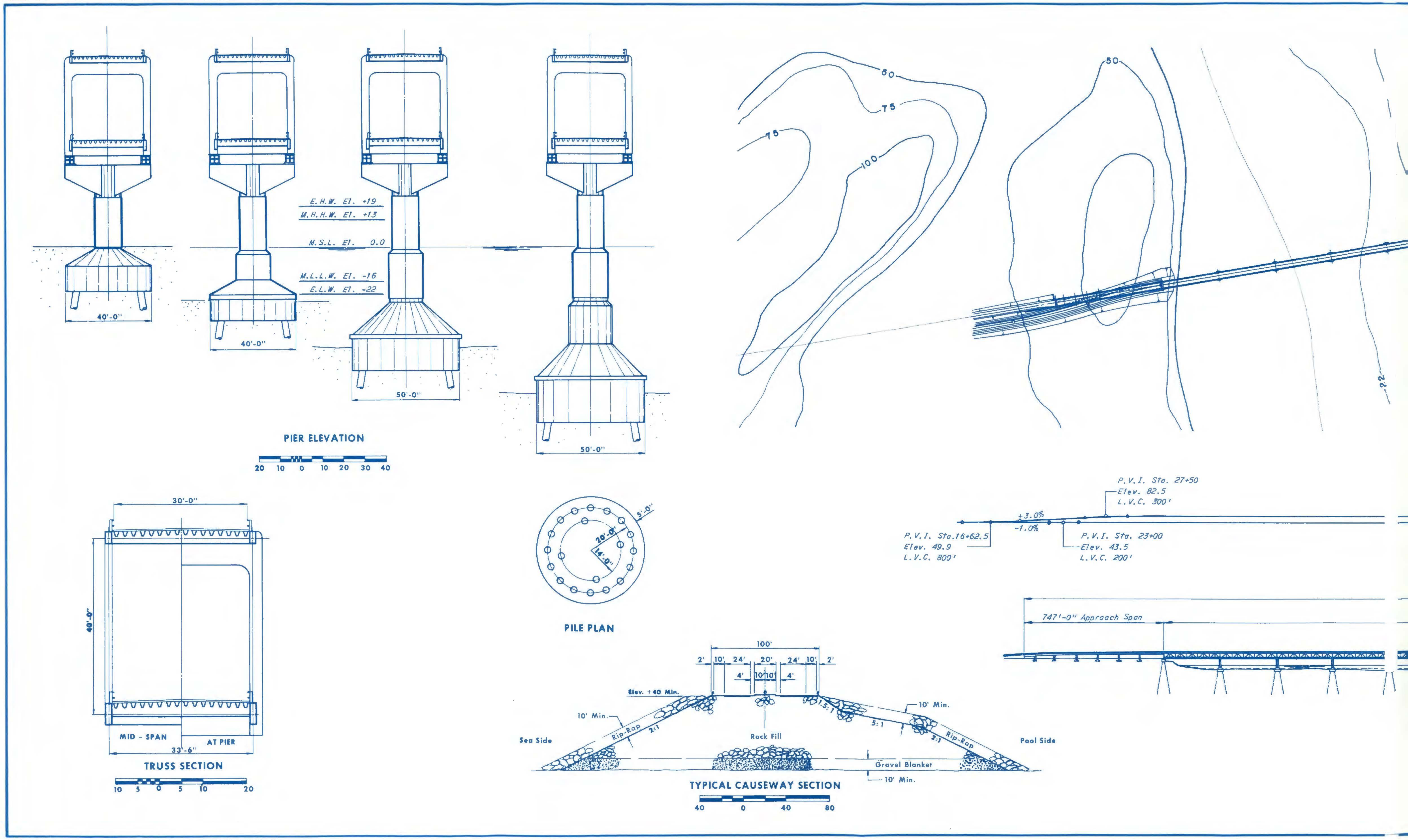


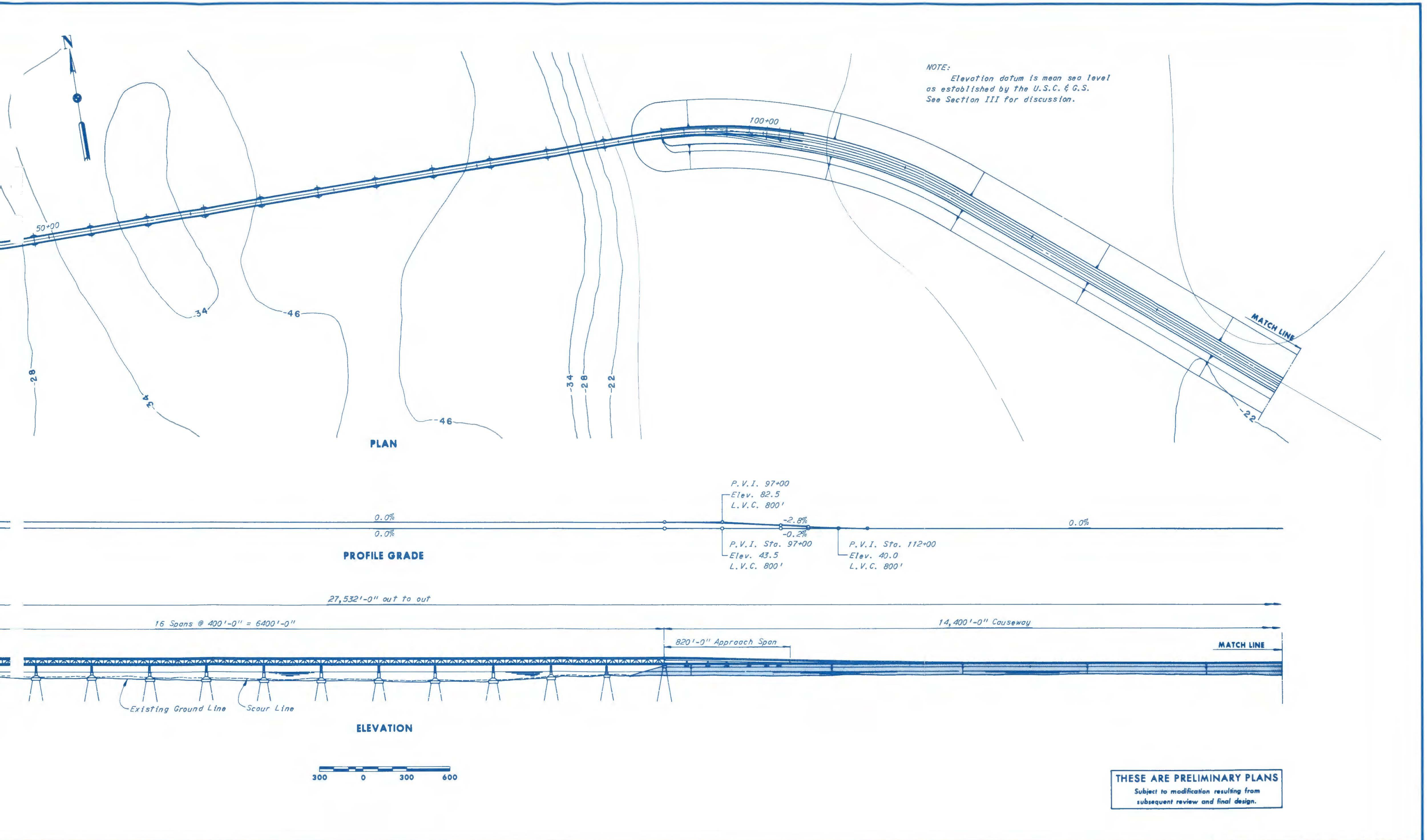
TYPICAL BRIDGE - SPILLWAY SECTION

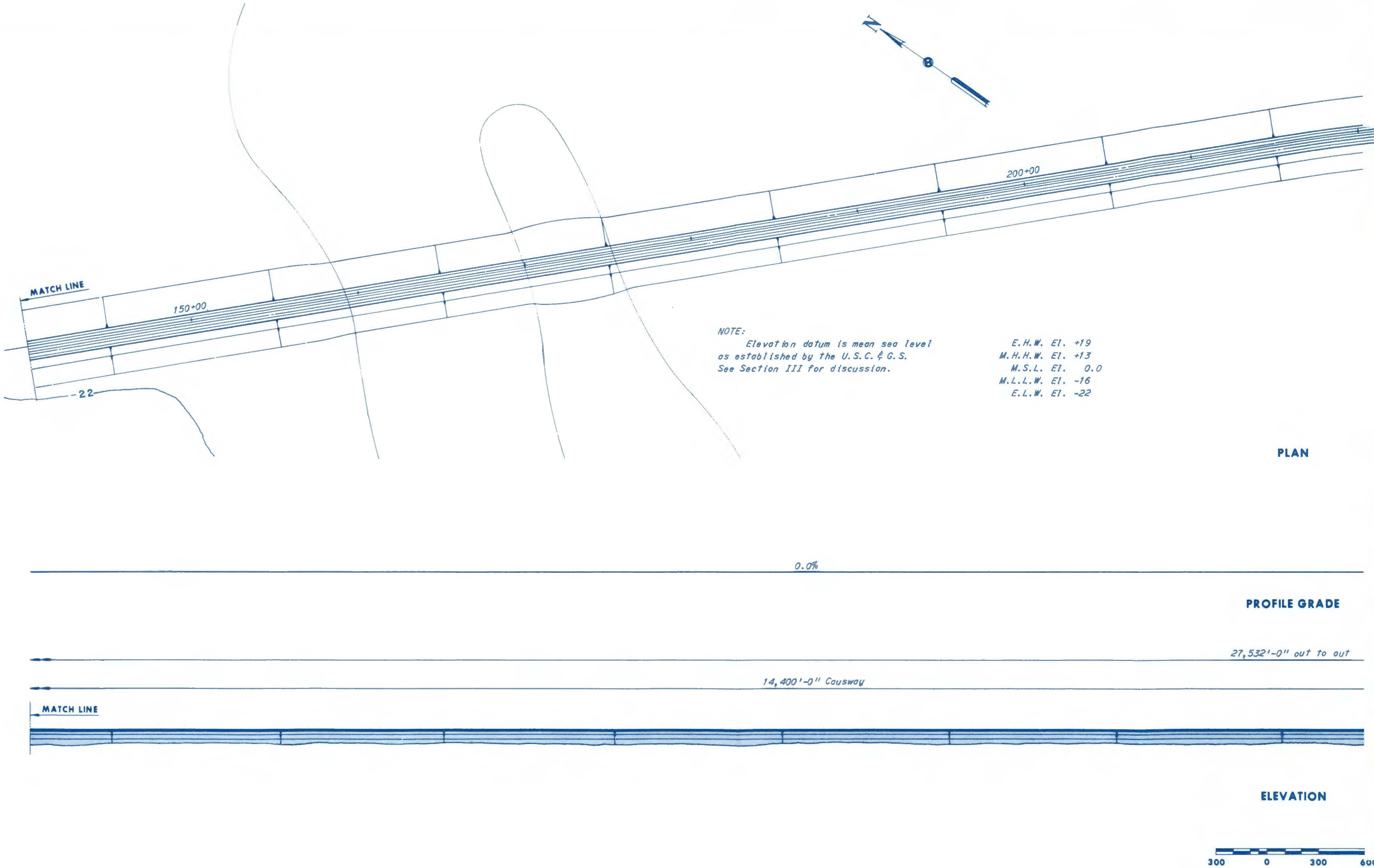
SECTION "A-A"

PROFILE

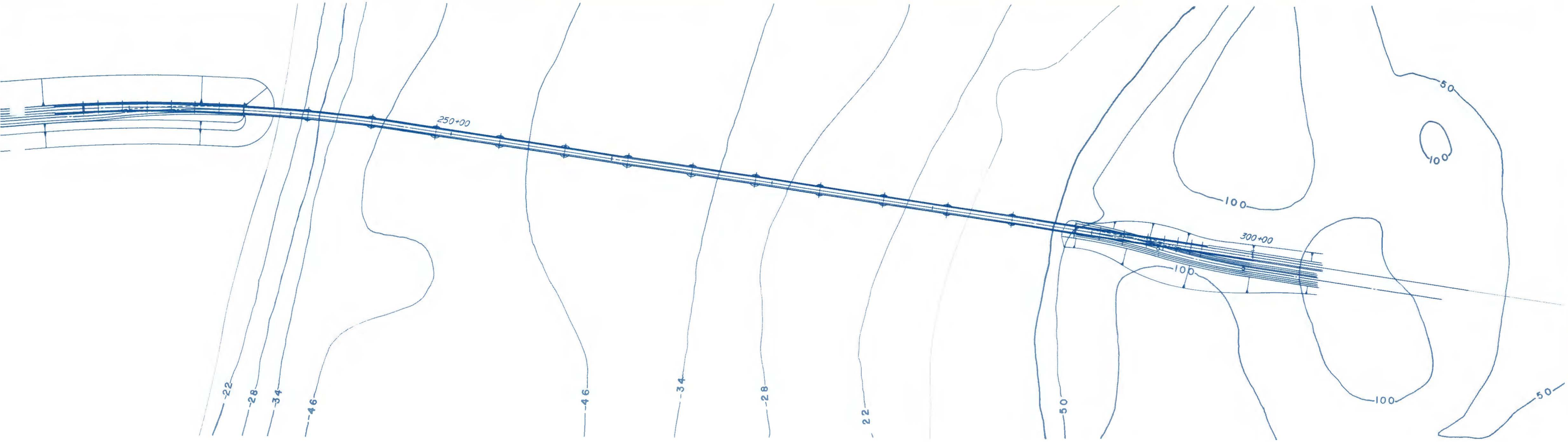


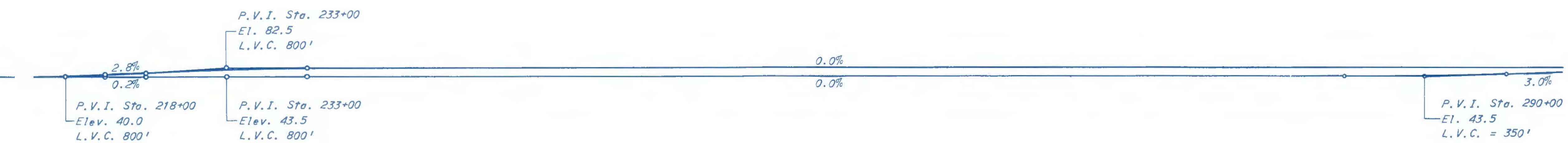

E.H.W. El. +19
M.H.H.W. El. +13
M.S.L. El. 0.0
M.L.L.W. El. -16
E.L.W. El. -22
40'-0"
40'-0"
50'-0"
50'-0"
PIER ELEVATION
20 10 0 10 20 30 40
30'-0"
40'-0"
MID - SPAN
AT PIER
33'-6"
TRUSS SECTION
10 5 0 5 10 20
5'-0"
20'-0"
14'-0"
PILE PLAN
100'
2' 10' 24' 20' 24' 10' 2'
4' 10' 10' 4'
Elev. +40 Min.
10' Min.
Rip-Rap
2:1
1.5:1
Rock Fill
5:1
10' Min.
Rip-Rap
2:1
Sea Side
Pool Side
Gravel Blanket
10' Min.
TYPICAL CAUSEWAY SECTION
40 0 40 80
50
75
75
100
50
22
P.V.I. Sta. 27+50
Elev. 82.5
L.V.C. 300'
+3.0%
-1.0%
P.V.I. Sta.16+62.5
Elev. 49.9
L.V.C. 800'
P.V.I. Sta. 23+00
Elev. 43.5
L.V.C. 200'
747'-0" Approach Span

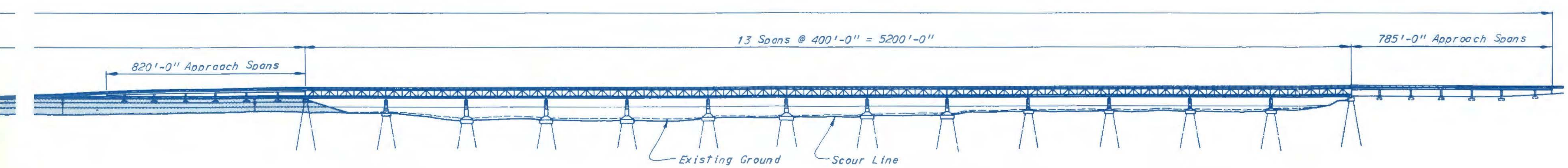

N
NOTE:
Elevation datum is mean sea level
as established by the U.S.C. & G.S.
See Section III for discussion.
50+00
100+00
-34
-46
-28
-34
-28
-22
-46
-22
MATCH LINE
PLAN
P.V.I. 97+00
Elev. 82.5
L.V.C. 800'
0.0%
0.0%
-2.8%
-0.2%
0.0%
P.V.I. Sta. 97+00
Elev. 43.5
L.V.C. 800'
P.V.I. Sta. 112+00
Elev. 40.0
L.V.C. 800'
PROFILE GRADE
27,532'-0" out to out
16 Spans @ 400'-0" = 6400'-0"
14,400'-0" Causeway
820'-0" Approach Span
MATCH LINE
Existing Ground Line
Scour Line
ELEVATION
300 0 300 600
THESE ARE PRELIMINARY PLANS
Subject to modification resulting from
subsequent review and final design.



THESE ARE PRELIMINARY PLANS
Subject to modification resulting from
subsequent review and final design.


250+00
300+00
-22
-28
-34
-46
-46
-34
-28
22
50
100
100
100
50
100
50


P.V.I. Sta. 233+00
El. 82.5
L.V.C. 800'
2.8%
0.2%
0.0%
0.0%
3.0%
P.V.I. Sta. 218+00
Elev. 40.0
L.V.C. 800'
P.V.I. Sta. 233+00
Elev. 43.5
L.V.C. 800'
P.V.I. Sta. 290+00
El. 43.5
L.V.C. = 350'


13 Spans @ 400'-0" = 5200'-0"
785'-0" Approach Spans
820'-0" Approach Spans
Existing Ground
Scour Line

INDEX TO PLAN AND
PROFILE PLATES
APPENDIX A

bridge design criteria

APPENDIX B

bridge design criteria

GENERAL

Purpose

The purpose of this design memorandum is to establish the basic data required for the comparative bridge designs presented in this report.

Project Location and Length

Four lines across the Knik Arm of Cook Inlet near Anchorage, Alaska are considered for bridge crossings. Structure lengths, given below, are from abutment to abutment.

1. Line 0: 13,600'-0" – approaches at 1,640'-0" – 15,240'-0"
   A crossing from Point Woronzof to Point Mackenzie.

2. Line III: 8,890'-0"
   A Cairn Point Crossing.

3. Line IV: 13,431'-0" – approaches at 1,650'-0" – 15,081'-0"
   A crossing about 8,100' upstream from Cairn Point.

4. Line V: 26,000'-0" – approaches 1,532'-0" – 27,532'-0"
   A crossing over the shoals just southwest of a line between Goose Creek and Eagle River.

Datum

All elevations refer to Mean Sea Level (M.S.L.) as determined by the United States Coast and Geodetic Survey. An approximate difference between Mean Sea Level and Mean Lower Low Water (M.L.L.W.) at Anchorage of 16 feet was used for the bridge studies in this report. See Section III for datum discussion.

Clearances

1. Navigation clearances are necessary for lines south of Cairn Point.

A vertical clearance of 140′ above mean higher high water (m.h.h.w.) has been assumed for the navigation channel (for estimating purposes).

A horizontal clearance of about 1,000′ has been assumed for the navigation channel.

2. Spans other than the navigation span south of Cairn Point and all spans on lines north of Cairn Point will be set to provide maximum economy with no consideration for navigational clearance. Grades for these spans will be set so that the bottom of the superstructure will be a minimum of 12′ above the highest water surface at extreme high tide with the superimposed maximum anticipated wave.

DESIGN SPECIFICATIONS

General

1. Specifications: The Standard Specifications for Highway Bridges, 1969, and Interim Specifications, 1970 and 1971, as adopted by the American Association of State Highway Officials (AASHO), will be used with modifications as set forth in these criteria.

Design Loads

1. Live Load: AASHO loading Class HS20 will be used.

2. Wind Load: Maximum wind load will be based on AASHO Specifications.

3. Ice Load: Two separate ice loadings will be considered for final design but will not apply simultaneously.

   a. Sheet ice 30 inches thick applying a continuous force equal to 300 psi at the point of mean high water.

   b. Massive ice blocks traveling with the current applying an impact load to the substructure. The magnitude of this impact force can only be estimated with the information at hand. It will be more precisely determined in subsequent stages of the project.

4. Stream Flow: Currents due to tides will be assumed to have a maximum velocity of 12 f.p.s.

5. Wave Forces: Maximum forces due to wave action will be based on waves generated by a 60 m.p.h. sustained wind.

6. Earthquake: For preliminary design, lateral forces due to earthquake will be computed as .10 times the dead load of the structure applied horizontally in any direction at the center of gravity of the weight of the structure.

7. Other Loads: Loads not specified above will be applied in accordance with AASHO Specifications.

Materials

1. Structural Carbon Steel

   a. This steel shall be ASTM A 36.

   b. It will be used for piles and other parts, as required.

2. High Strength Low Alloy Structural Steel

   a. This steel shall be ASTM A 588.

   b. It will be used for deck plates and stiffeners, floorbeams and truss members in the superstructure and for the protective armoring for the substructure.

3. Cast Steel

   a. This steel shall be ASTM A 296.

   b. It will be used for shoes.

4. Special Steels

   a. ASTM designation shall be used.

   b. Special steels may be used for parts, as required.

5. Reinforcing Steel
   Billet steel, Grade 40 (ASTM A 615) shall be used.

6. Concrete
   The concrete in pier shafts, bases and deck slabs shall be f'c = 4,000 psi.

7. Asphaltic Wearing Surface
   The wearing surface shall be asphaltic concrete with a minimum thickness of 2-1/2" and shall be applied in two layers.

8. Roadway Protective Coating
   The top of the steel deck plate is to be grit blasted and coated with epoxy resin and stone chips prior to placing the wearing surface.

FEATURES OF SUPERSTRUCTURE

Roadway Section

The roadway section will consist of two 30' wide roadways (face to face of railing) with no curbs or parapets.

Exterior Bridge Rail

Exterior bridge rail shall be galvanized steel with two continuous 6" x 3" structural steel tubes supported by 6W 20 posts spaced at nominal 9' centers. The top rail shall be 2'-6 1/4" above the roadway surface.

Median Rail

The median rail, if required, shall be double-faced, galvanized steel, guardrail on centerline supported by 6W 15.5 posts spaced at nominal 6' centers. The rail shall be blocked out from the posts by 8" x 8" wooden blocks. The top of rail shall be 2'-3" above the roadway surface.

Roadway Cross Slope

The roadway cross slope on tangent sections shall be 3/16" per foot each way from the centerline of the roadway cross section.

Grades

The maximum grade shall be 3 percent

Superstructure Type

The superstructure type shall be determined by economic type studies for each line, consistent with navigation clearances and channel depth.

FEATURES OF SUBSTRUCTURE

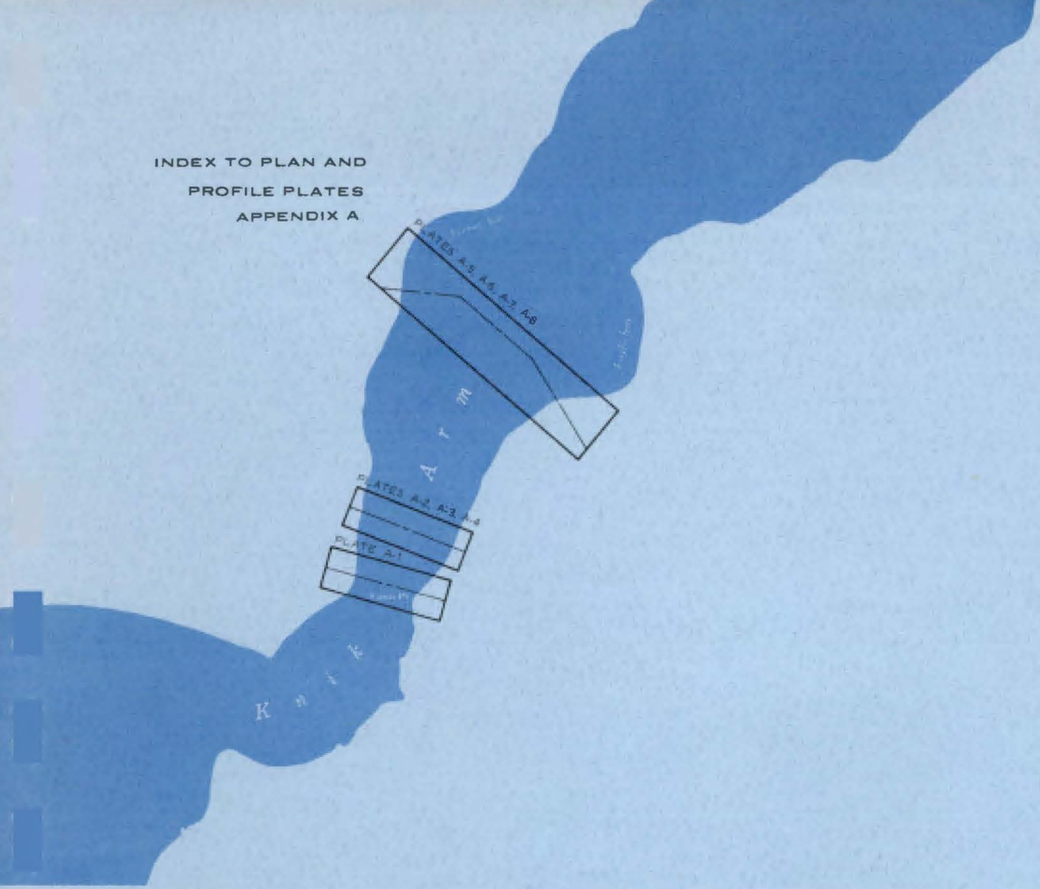
Piers

1. Piers are to be single, solid concrete shafts.

2. Pier bases will be supported by steel friction piles.

3. Pile length will be determined by the required pile capacity for the piles at individual piers.

4. Pier construction will be consistent with requirements imposed by water depth, currents and tidal variations.



causeway dam design criteria and tide amplification study

APPENDIX C

causeway dam
design criteria and
tide amplification study

The design of a dam to be constructed under the conditions which exist in Knik Arm will require the review and consideration of a large number of items. The following is a list of items which will need to be considered in this design:

1. Purpose of Causeway

    A. Purpose
    B. Ease of Construction
    C. Side Benefits

2. Crown Width

    A. Number of traffic lanes
    B. Railroad
    C. Hydraulic controls
    D. Structural and soils control
    E. Type of pavement

3. Crown Elevation

    A. Sea side

        a. Extreme high water
        b. Tide run up
        c. Wave height
        d. Wave run up
        e. Spray
        f. Freeboard

    B. Pool side

        a. Extreme high water

b. Wave height
c. Wave run up
d. Spray
e. Freeboard
f. Lake George Breakout

4. Lake Elevation

A. Permanent Pool
B. Design High Water
C. Effect on Eklutna Power Plant
D. Effect on Glenn Highway
E. Effect on Railroad
F. Effect on Adjacent Villages, Farmland and Residences
G. Lake George Breakout

5. Spillway Design

A. Principal Spillway

a. Elevation
b. Location
c. Type
d. Design Storm
e. Stilling Basin
f. Ice passage
g. Sluice Gate
h. Control
i. Operation
j. Lake George Breakout

B. Auxilliary Spillway

a. Elevation
b. Location
c. Type
d. Design Storm or Discharge

6. Causeway Section

A. Type

a. Homogeneous
b. Impervious Cove
c. Impervious Surface Membrane

B. Dimensions

C. Armoring

a. Sea side
b. Pool side

D. Material Availability

E. Material Quality

F. Seismic Stability

7. Foundation

A. Geology
B. Surface Conditions
C. Seepage

8. Construction Methods

A. Embankment Construction
B. Spillway
C. Diversion
D. Closure

9. Ecological Effects

A. Wildlife
B. Plant life
C. Fish Ladder
D. Weather

10. Maintenance

A. Frequency

   a. Normal
   b. Emergency

B. Lake Bed

C. Lake Shore

11. Costs

A. Dam
B. Spillway

PORT COASTAL OCEAN RIVER

Tidemandsgate 30
****
TRONDHEIM, NORWAY
TELEPHONE **** 36597

PER BRUUN
DR. TECH. SC.
CONSULTING ENGINEER

******
****
RESEARCH MANAGEMENT CORP.
WASHINGTON, D. C. 20009
TELEPHONE 202 234-5033

3520 Prospect Street
Wash., D.C. 20007

Howard, Needless, Tammen & Bergendoff
1805 Grand Ave.
KANSAS CITY, Mo. 64108

July , 1971

Gentlemen:

This letter was mailed to you from Copenhagen Airport just before my departure for India. I will be back app. July 31st.

I am enclosing a number of separate reports each dealing with particular subjects discussed in KANSAS CITY.

Encl. 1 Preliminary report on the influence of projects IV and V on tides at Anchorage.

Encl. 2* Report on scour including report on model study in Florida.

Encl. 3* Compressive strengths of ice as listed by various authors.

Encl. 4* Forces by Icebergs on piers.

Encl. 5* Construction of double training wall of ice for sand fill.

Encl. 6* Revetments for protection of sand fills.

Encl. 7* Model experiments for the Anchorage Crossing projects.

When reply has been received by Mr. Schwab on his letter of June 23 on sediments I shall comment on the sediment transport problem. Also you will receive abstracts of papers to be presented at the POAC conference in August with my comments in relation to this project.

Yours sincerely,

Per Bruun

* Enclosures 2 thru 7 are not directly applicable and are not included herein.

ENCL 1

AMPLIFICATION OF TIDE HEIGHT ELEVATIONS IN KNIK ARM
Dams at Sites III, IV and V
by Per Bruun

On Fig. 1 and 2 the distribution of peak elevations of the water table in Knik Arm and Turnagain Arm are sketched. Fig. 1 is based on observations by Messrs. Schwab and Wangelin. The June and September observations are all adjusted to May spring amplitudes assuming linear relations. The June-records show lower watertables than the May-records indicating nonlinearity due to friction for lower tidal ranges. The data on Fig. 2 are based on ref. (1).

By use of Dorresteins theories (2) for amplification of long waves penetrating in bay or channels one may evaluate whether a correlation between his results and the measurements shown in Figs. 1 and 2 exists. Besides observations of extreme high tides at Anchorage during the sixties show that the average high tides as well as tidal ranges are approximately 0,7 ft higher in January, February and March than in May, June and July (table 1). This may be caused by the freezing up of the Knik Arm shoals.

Assuming linear relationship as in Fig. 1, the rise at the closed end of Knik Arm is 3.5' above Anchorage elevations. The slope of water table between Fire Island and Anchorage is reduced to approximately 2/3 of the slope in Knik Arm due to the expansion of the fiord (comparing Fig. 2 for Turnagain Arm) resulting in a drop of 0,8 ft in tide elevation compared to the Anchorage tide (+17' above MSL). The amplification for Knik Arm therefore is $R = (17+4,3)/17 = 1,25$. According to (1) the elevation of high water measured near the mouth of the Placer River in Turnagain Arm is 26 feet above mean sea level and 19 feet above MSL at the open end (Gull Rock-Isle). At the closed end the Turnagain Arm is very narrow. The steep water table at the narrow end of the bay might therefore be due partly to freshwater flow by the rivers. The rise due to amplification is therefore assumed to increase lineally from Bird PT (dotted line on Fig. 2). Thus the amplification for Turnagain Arm is R = (19+4,2)/19 = 1,22.

An attempt is now made to correlate these data from nature to the mathematical model (2). Table 2 shows simplified data used for the comparison, where the most important (and not accurate) assumption is that of "uniform depth." The average velocity during one spring tidal cycle in Knik Arm is due to changes in tidal prism calculated to approximately 1,5 m/sec. This velocity is assumed to be approximately 1,0 m/sec in case of a dam (ice barrier) at Eagle River. The average velocity for Turnagain Arm was assumed to be 1,5 m/sec (see table 8 in (1)). Considering distortion the coefficient of friction is reduced to 50% (f*) as shown in colomn 5 of table 2. To find the amplification for the entire bay, "case D" (Fig. 3, sloping bottom, covering banks) should be used. Unfortunately Dorrestein has here not included the influence of friction. Case A does however not deviate too much from this conditions and will therefore be used for preliminary calculation for the entire bay areas. As shown in table 2 this results in an amplification of 1,18 and 1,17 for Knik Arm and Turnagain Arm respectively or 5,6% below the observed values. To find the amplification in case of an ice dam near Eagle River conditions in between "Case C" and "Case A" in Fig. 3 are probably indicative. For R- ratios less than 0,05% the effect from friction is negligible. Choosing an R-value slightly above the "A" curve would result in an amplification of approximately R = 1,07 for an ice dam at Eagle River (V) and R = 1,04 for dam at Cairn PT (III).

Adjusting the amplification ratio in case of ice dam at Eagle River according to the elevation observed one gets $R = 1{,}075 \cdot \frac{1{,}25}{1{,}18} = 1{,}14$. This results in peak tide-elevation curves as shown in Fig. 6.

The rise in water elevation at Anchorage is approximately 0,8 ft according to the adjusted water table curve or in fairly good accordance with the previously mentioned 0,7 ft increase in height of the water table at Anchorage due to freezing up of the Knik Arm shoals (table 1).

CONCLUSIONS

In case of a dam at sites III or IV high tide at Anchorage will increase approximately 1 ft.

In case of a dam at site V high tide at Anchorage will increase 0,8 to 1 ft. Combined bridge - causeway projects will have little influence on high tides at Anchorage although tendency will be a slight increase.

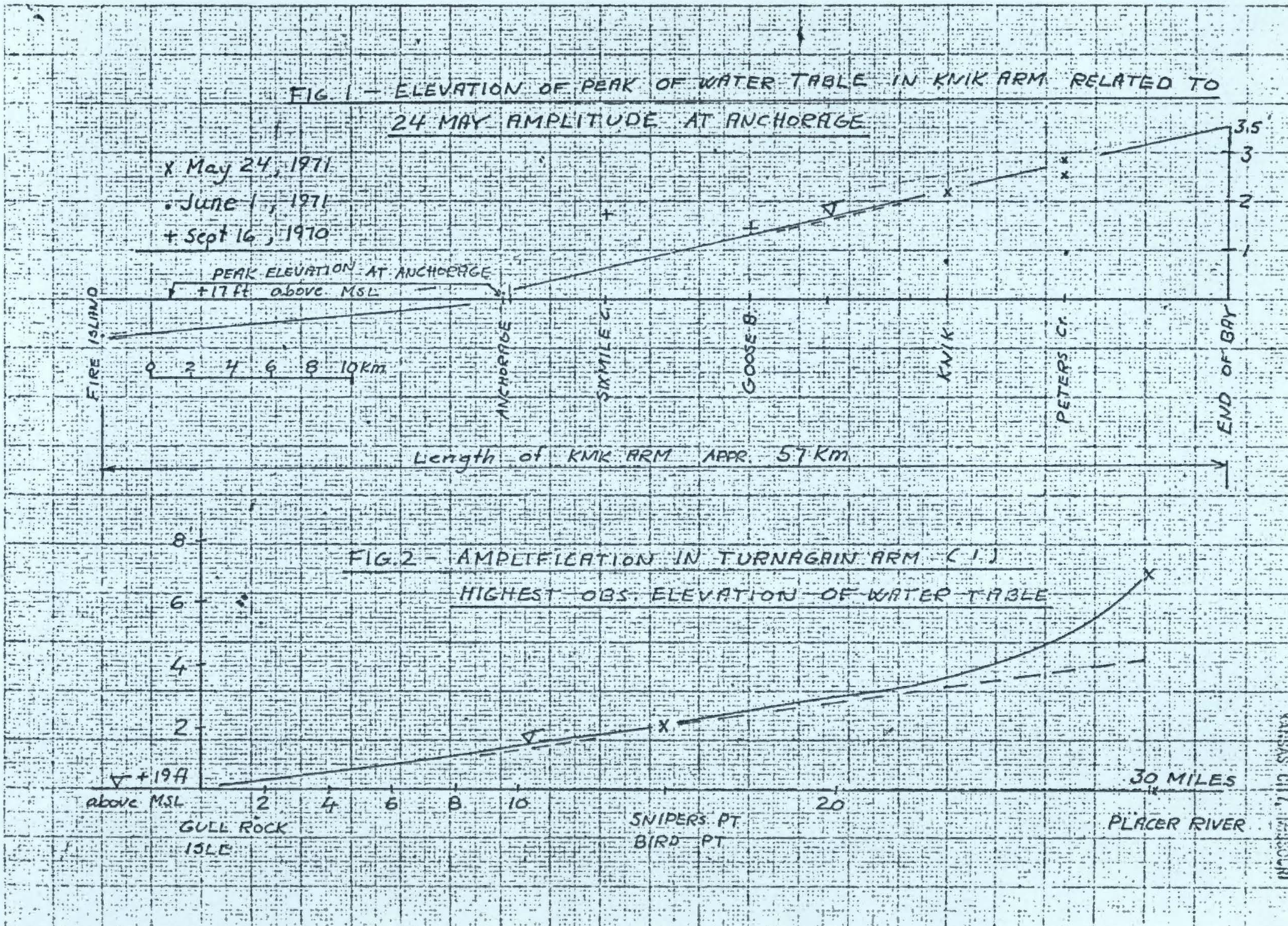
References

(1) "Feasibility Study Turnagain Arm Crossing" - State of Alaska, Department of Highways, March 8, 1963, by Porter, O'Brien and Armstrong and Trych, Nyman and Associates.

(2) "Amplification of Long Waves in Bays." Engineering Progress at the University of Florida, December 1961 by Richard Dorrestein.

FIG 1 – ELEVATION OF PEAK OF WATER TABLE IN KNIK ARM RELATED TO 24 MAY AMPLITUDE AT ANCHORAGE

FIG. 2 – AMPLIFICATION IN TURNAGAIN ARM (1.) HIGHEST OBS. ELEVATION OF WATER TABLE



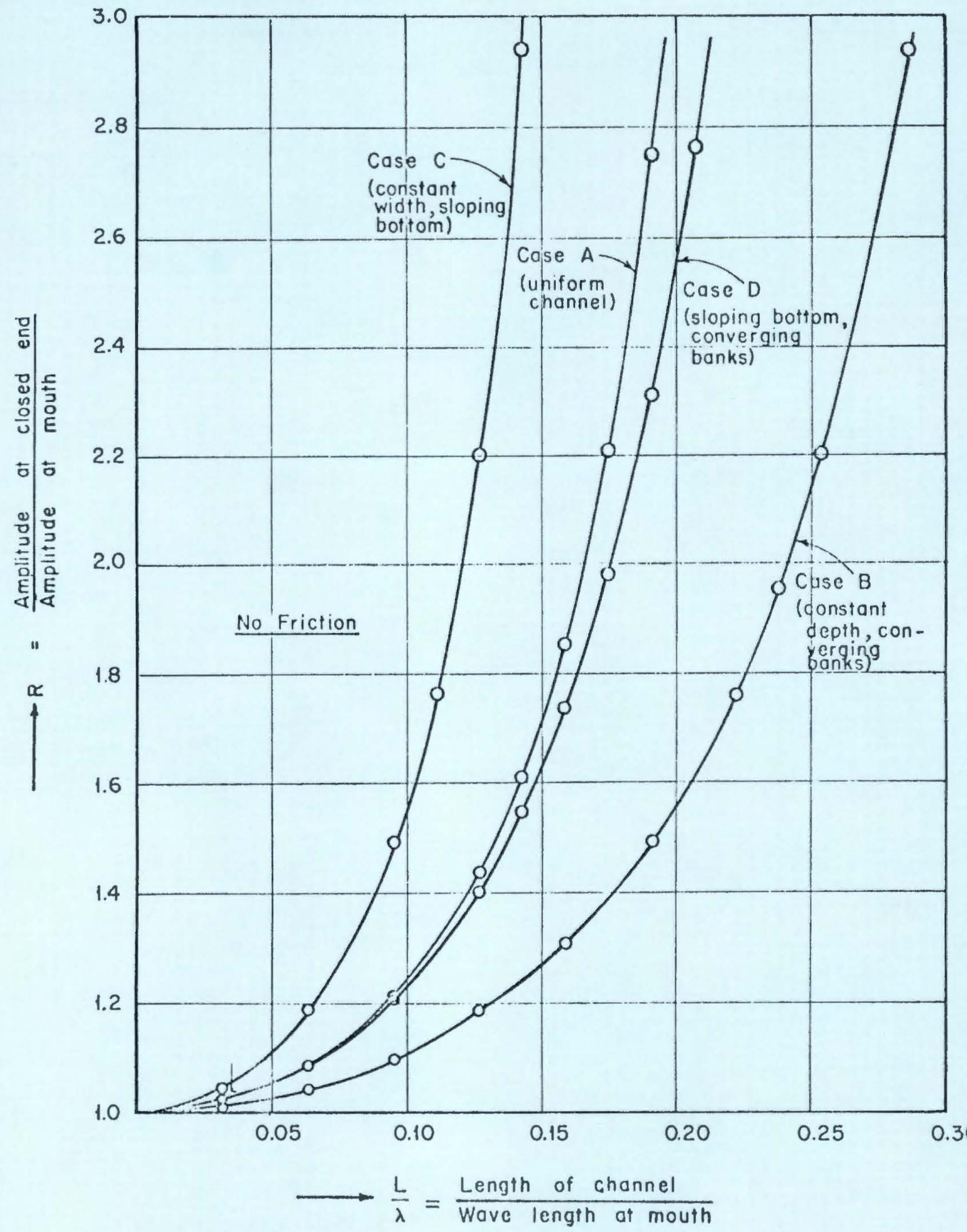
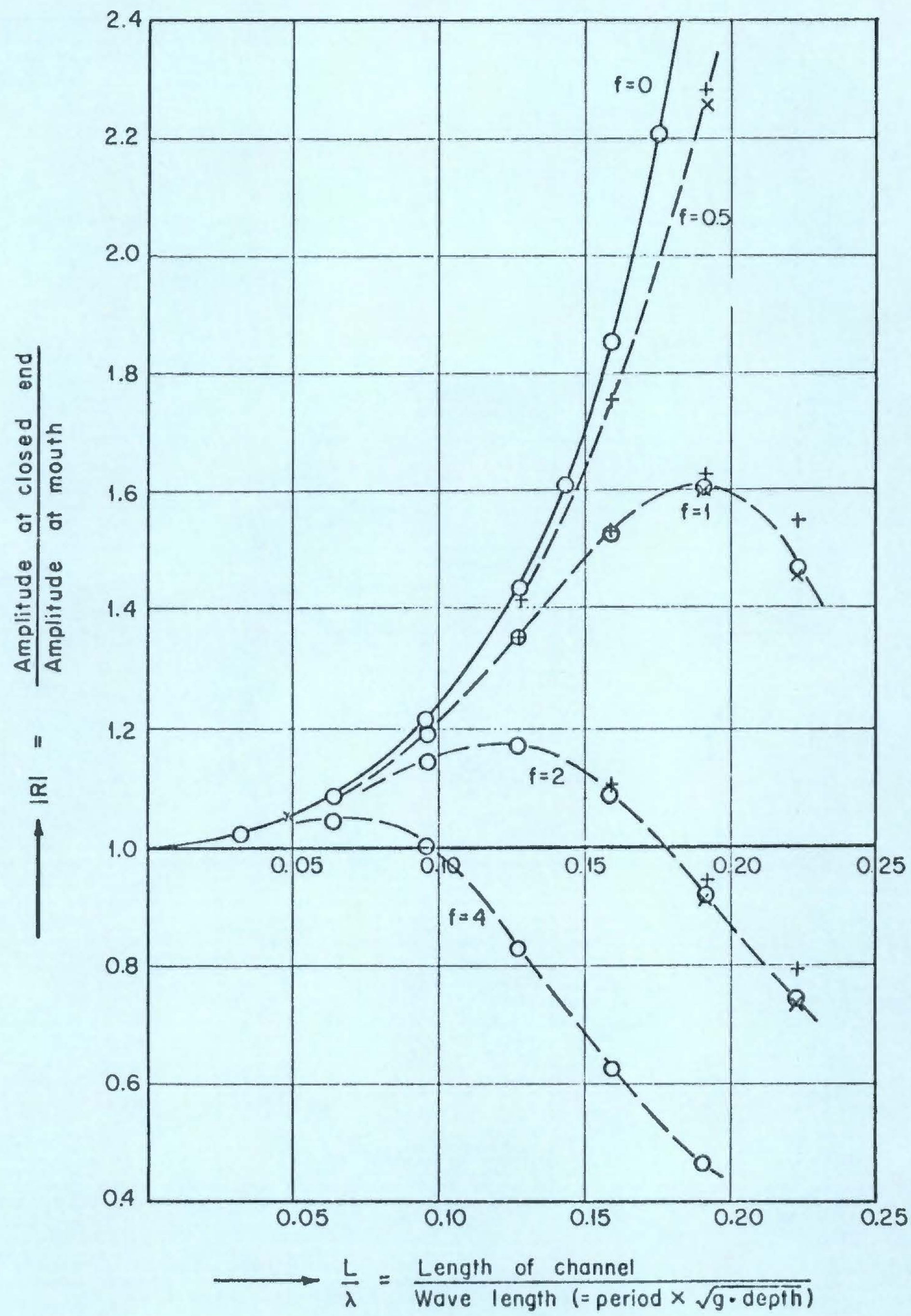
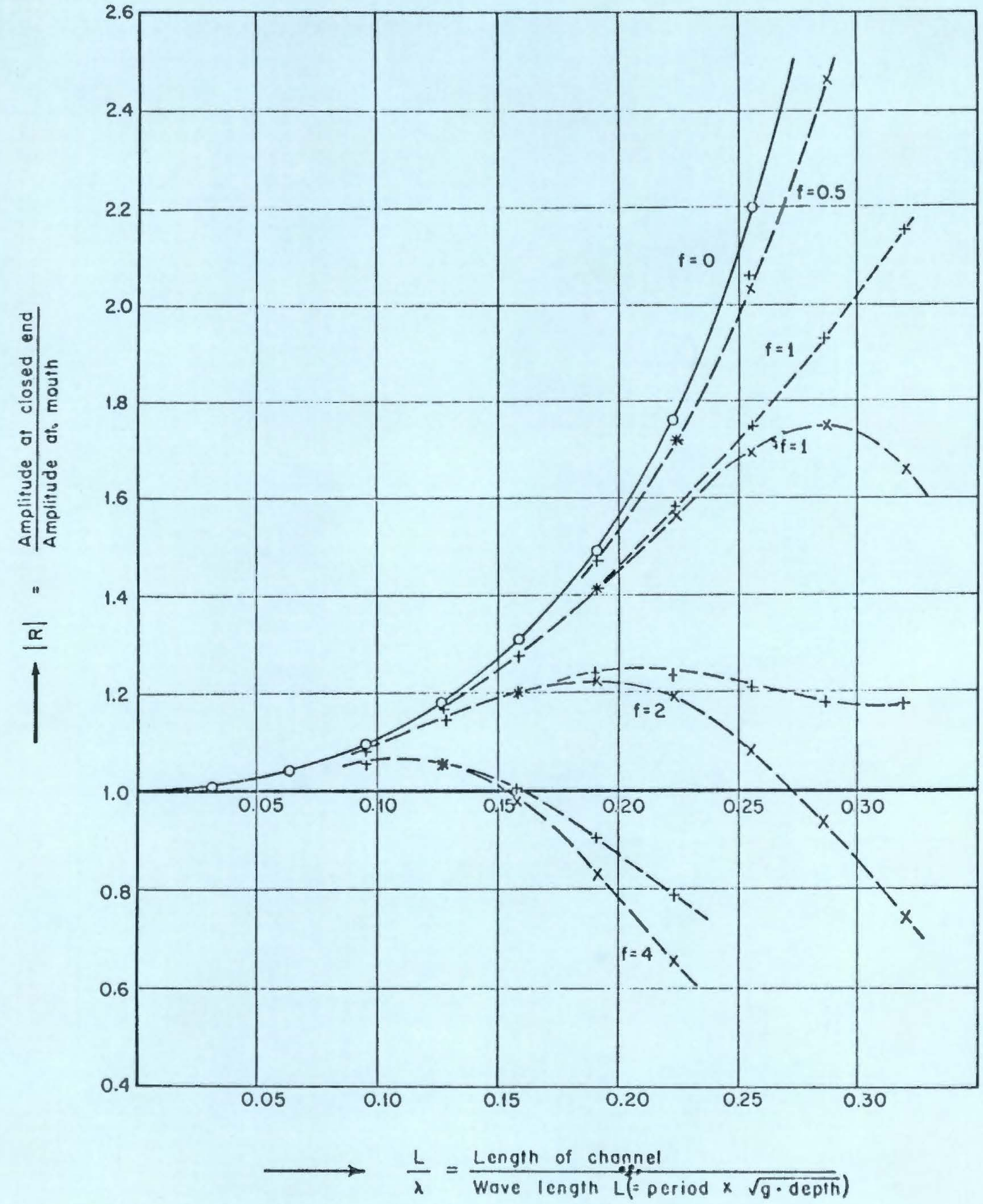

3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.05
0.10
0.15
0.20
0.25
0.30
Case C
(constant width, sloping bottom)
Case A
(uniform channel)
Case D
(sloping bottom, converging banks)
Case B
(constant depth, converging banks)
No Friction
R = Amplitude at closed end / Amplitude at mouth
L/λ = Length of channel / Wave length at mouth

Fig 3 (2)



Case A: Uniform Channel

Fig 4



2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.05
0.10
0.15
0.20
0.25
0.30
f=0
f=0.5
f=1
f=1
f=2
f=4
|R| = Amplitude at closed end / Amplitude at mouth
L/λ = Length of channel / Wave length L (= period × √g·depth)

Fig 5

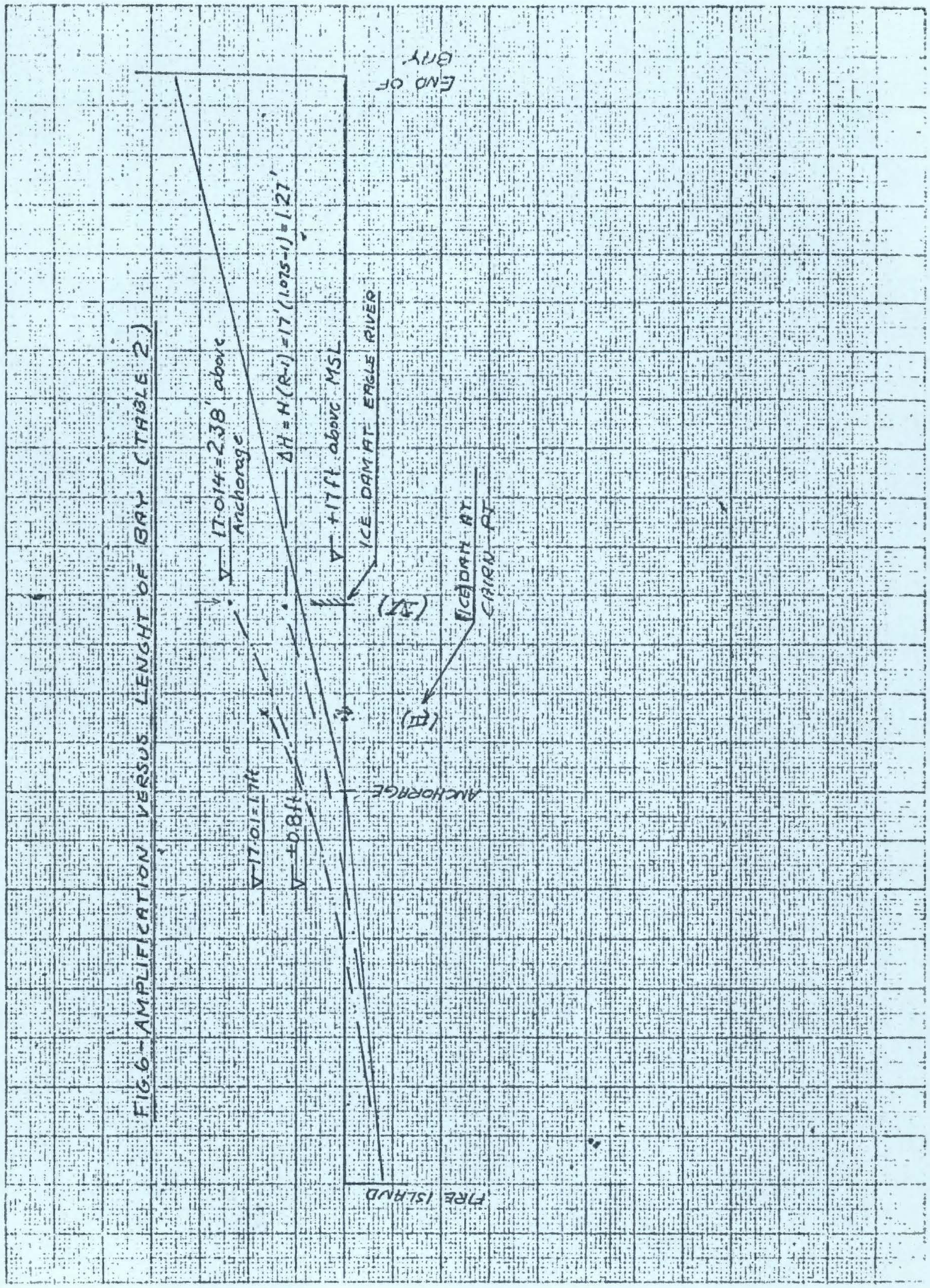


FIG. 6 - AMPLIFICATION VERSUS LENGHT OF BAY (TABLE 2.)

KNIK ARM CROSSING, ALASKA

FIGURE 7 ALTERNATE LOCATIONS

FORM C&GS-472e
(2-62)
USCOMM-DC 18420-P64

TABLE 1

U.S. DEPARTMENT OF COMMERCE
COAST AND GEODETIC SURVEY

TIDES: MONTHLY

Highest Tides

Station: Anchorage, Alaska

Latitude .... *Longitude* ....

Observations begin ....

Observations end ....

Datum is 6.7 ft. below MLLW

which is .... *feet below B. M.* ....

Linear quantities in feet ....

Time in hours ....

	Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	For Year Sum	For Year Mean	Total Sum	Total ~~Mean~~	
(1)	1964					39.0	38.7	39.0	38.7	37.4	39.0	39.8	38.7				39.8	11/64
(2)	1965	39.9	38.5	37.9	38.0	38.1	38.1	38.3	39.2	39.0	39.6	38.2	39.2				39.9	1/65
(3)	1966				39.7	39.0	39.1	37.8	39.0	40.3	40.5	39.2	39.0				40.5	10/66
(4)	1967	38.9	40.3	40.8	39.3	39.1	37.9	37.5	38.1	40.2	40.1	40.2	39.5				40.8	3/67
(5)	1968	37.2	37.9			38.8	39.0	38.3	37.6	38.9	40.1	39.9	38.0				40.1	10/68
(6)	1969	38.6	40.2	38.4	38.8			38.4	38.6	38.9	38.4	38.0	39.3				40.2	2/69
(7)	1970	38.6		39.8	38.7	37.7	37.3	38.1	39.3	39.3	38.8	38.5	38.4				39.8	3/9
(8)																		
(9)																		

TABLE 2 - AMPLIFICATION AT THE CLOSED END OF BAY

	L (km)	$\bar{D}$ (m)	λ (km)	f	f*	L/λ	AMPLIFICATION - R	
							MODEL	OBSERVED
KNIK ARM	57	14	520	3.6	1.8	0.11	1.18	1.25
KNIK ARM / DAM AT EAGLE RIVER	30	15.5	550	2.4	(1.2)	0.055	1.075	(1.14**)
K. A. / DAM AT CAIRN PT (III)	20	18	590	1.9	(1)	0.034	1.04	(1.10**)
TURNAGAIN ARM	48	15	540	3.4	1.7	0.089	1.17	1.22

* reduction considering distorsion

** adjusted according to observed values

L = length of fiord

D = assumed average depth of the fiord

λ = tidal wave length = T· GD

T = tidal period = 44.700 sec

ω== angular velocity = T/2π = 7100

G = 9,81 m/sec^2

f = coefficient of friction $= \frac{G}{c^2\omega} \cdot \frac{v}{d} = 34.3 \cdot \frac{v\,(m/sec)}{d\,(cm)}$

R = amplitude at closed end/amp. at mouth

Comments on Typical Dam Section
by
Dr. Per Bruun
Nov. 1, 1971

An S-formed profile like the one shown in Fig. A below is hydraulically (wave mechanically) better. The stability against "pressure blowouts" in the most exposed zone, B C, is increased due to gravity forces. This profile also reduces runup, spray and separation (see POAC abstract papers by Bruun, Straumsnes and Johanessen plus Bernie Rottinghaus's paper). The probability for ice formation on the road is also decreased hereby.

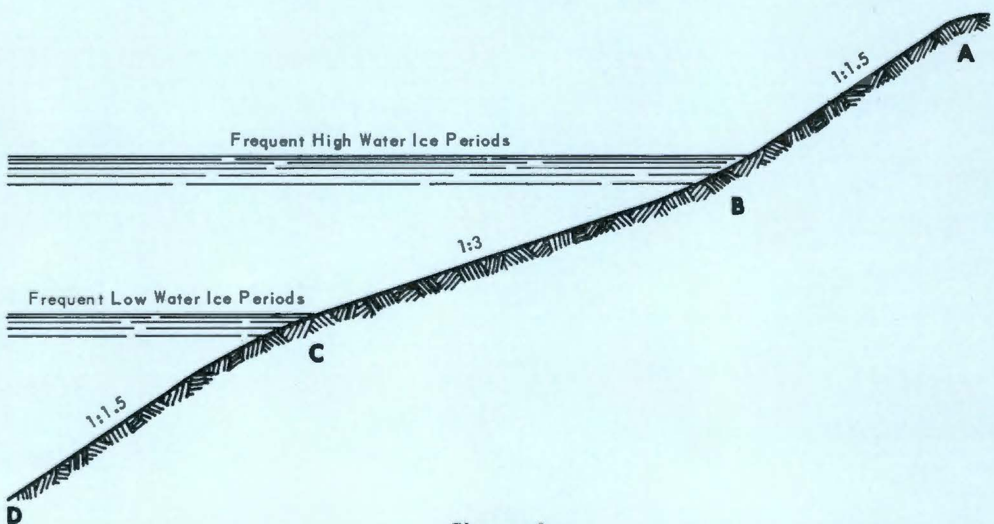


Figure A

Comment on "typical cross section"

In zone A Fig. B (fluctuating water table) beach is exposed to wave action) "sediment of silt and fine sand" on this part of the slope will therefore hardly occur. More likely wave action will partly wash out sand from the "sand and gravel filter" as the slope probably is too steep. Sand from the filter might easily wash into the rock fill with the water flow through the structure.

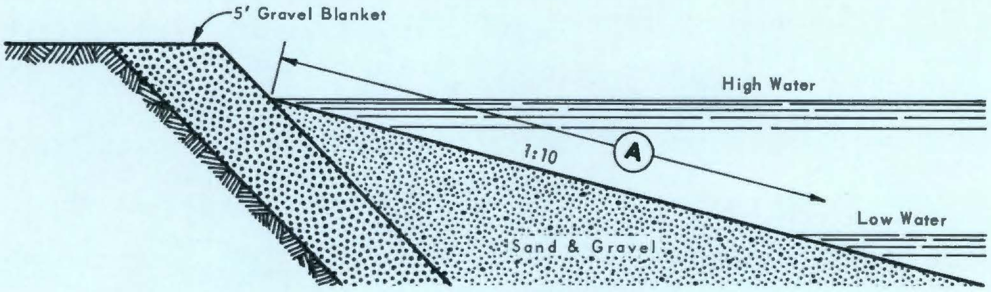


Figure B

Comment on "Alternate Lakeside Treatment

Riprap placed in the zone of ice and wave action is necessary but it is not likely that the sand and gravel layer between the riprap cover and the rock fill will be stable. A graded filter similar to detail B in Fig. 8 (S & W Report) may be necessary if seepage through this zone shall be controlled, but - as mentioned earlier - such filter is difficult to place and it gives in fact a false feeling of safety.

Figure C below is probably a better solution.

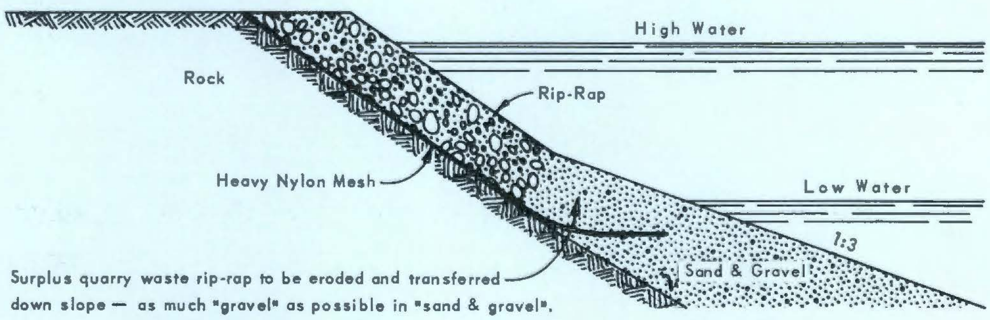
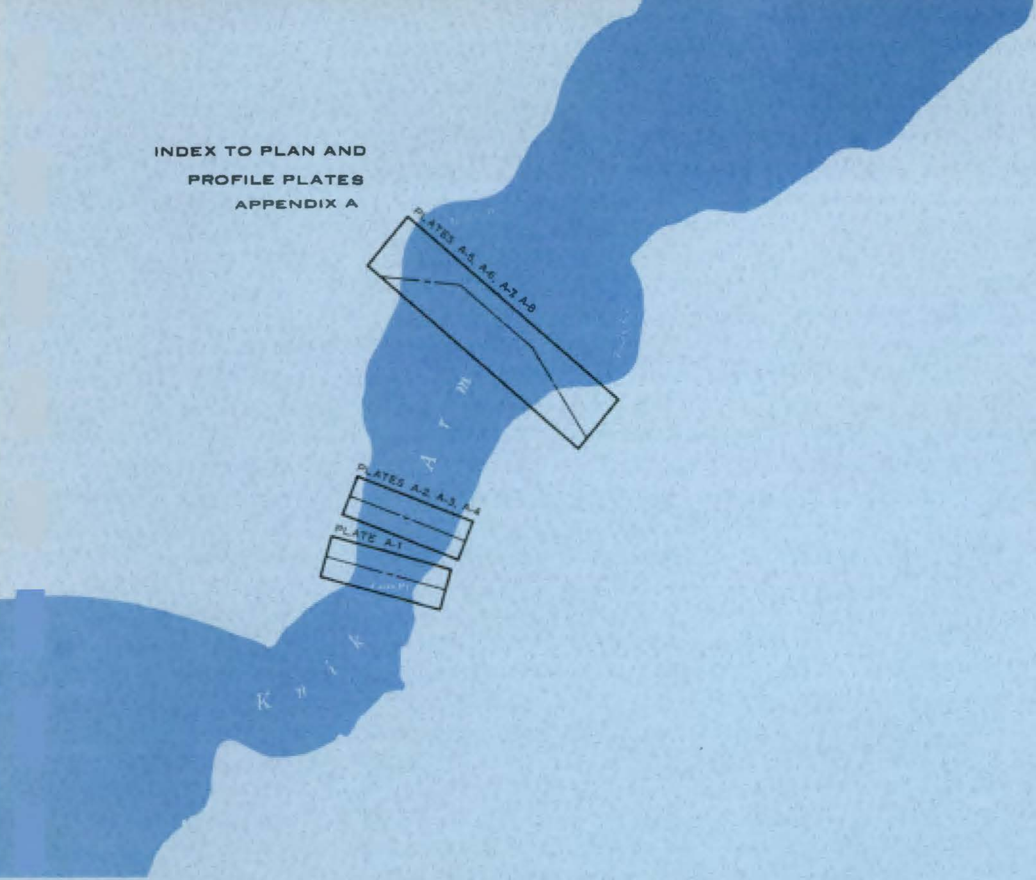


Figure C



construction methods and costs

APPENDIX D

COST AND METHODS STUDIES

PROPOSED KNIK ARM CROSSING

ANCHORAGE, ALASKA

FOR

HOWARD, NEEDLES, TAMMEN & BERGENDOFF

KANSAS CITY, MISSOURI

SEPTEMBER 10, 1971

JACOBS ASSOCIATES
500 SANSOME STREET
SAN FRANCISCO, CA 94111

JACOBS ASSOCIATES ENGINEERS AND CONSULTANTS CONSTRUCTION PERFORMANCE SERVICES

500 SANSOME · SAN FRANCISCO 94111 · TELEPHONE (415) 434-1822 · CABLE: DONJAY

September 10, 1971

Howard, Needles, Tammen & Bergendoff
1805 Grand Avenue
Kansas City, Missouri 64108

Gentlemen:

We are sending you herewith ten copies of our report "COST AND METHODS STUDIES, PROPOSED KNIK ARM CROSSING, ANCHORAGE, ALASKA." This report has been prepared by us from two viewpoints. First, that of an experienced heavy construction contractor preparing bids for the alternate designs. Secondly, that of a construction engineering consultant offering advice and opinions on the construction problems inherent to work of the nature proposed.

This work was performed in accordance with our proposal letter of July 14, 1971 that was accepted by you on July 19, 1971.

It is hoped that as a result of this report, all interested parties may better understand the major construction problems and high construction costs of the various alternatives proposed for constructing a crossing of the Knik Arm.

Very truly yours,

A M Petrofsky

A. M. Petrofsky

AMP:lb

Enclosures

TABLE OF CONTENTS

SUMMARY

We have made contractor-type feasibility level cost estimates for six alternate designs prepared by Howard-Needles-Tammen & Bergendoff as possible crossings of the Knik Arm of the Cook Inlet near Anchorage, Alaska. These estimates represent the prices that we think would be bid in the fall of 1971 for work to be started in January, 1972. We have also prepared time schedules for completing the work on a normal, efficient basis. The bid price and construction period for these six alternates are as follows:

Alternate	Bid Price* (Million $)	Schedule (Years)
Truss Bridge at Line IV	114.9	4
Causeway Dam at Line V	189.6	5
Truss Bridge at Line V	202.3	6
Bridge-Causeway at Line V	209.2	5.5
Suspension Bridge at Line III	226.2	7
Causeway Dam at Line IV	263.1	6

* The amounts do not include any allowance for contingencies and variations.

All of the alternates considered present major construction problems. The bid prices reflect our evaluation of the degree of difficulty of overcoming the known problems, plus a relatively small contingency factor in the markup to cover the unanticipated problems. We would suggest that it would be reasonable to add an overall contingency factor of about 10% to these prices to reflect possible changes in design that will likely occur as more detailed investigations and designs are made.

Bid item schedules for each of the six alternates are presented on the following pages. It is noted that the same unit price is not applied to seemingly identical work items in different alternates. This is due to quantity variations and the relatively high indirect costs for the bridge substructure work that must be expended for access, docks, yards, and major equipment. There is further discussion in this report on the manner of establishing these completely balanced bid prices.

ESTIMATE OF COST

TRUSS BRIDGE AT CROSSING IV

2 Approach Structures at 870' and 780' = 1,650'
Main Bridge 33 Spans at 407' = 13,431'
Total Bridge Length = 15,081'
Roadway: 2 at 30'-0"

APPROACH STRUCTURES						
Superstructure						
Structural Steel (A588)	1,792,000	Lbs.	@	$ 0.65	=	$ 1,164,800
Cast Steel	36,000	Lbs.	@	1.45	=	52,200
Concrete in Deck	1,320	C.Y.	@	200.00	=	264,000
Reinforcing Steel	360,000	Lbs.	@	0.28	=	100,000
Handrail	3,300	L.F.	@	20.00	=	66,000
Sub-Total Superstructure						$ 1,647,800
Substructure						
Concrete in Columns and Cap Beams	480	C.Y.	@	$200.00	=	$ 96,000
Concrete in Footings and Abutments	520	C.Y.	@	170.00	=	88,400
Reinforcing Steel	190,000	Lbs.	@	0.28	=	53,200
14HP73 (45' Long)	12,660	L.F.	@	30.00	=	379,800
Excavation (Above Grade)	460,000	C.Y.	@	2.40	=	1,104,000
Excavation (Below Grade)	1,600	C.Y.	@	10.00	=	16,000
Sub-Total Substructure						$ 1,737,400
MAIN BRIDGE:						
Superstructure						
Structural A588 Steel	57,465,000	Lbs.	@	$ 0.64	=	$ 36,777,600
Cast Steel	1,140,000	Lbs.	@	1.44	=	1,641,600
Asphaltic Wearing Surface	92,650	S.Y.	@	16.00	=	1,482,400
Deck Protection	92,650	S.Y.	@	2.00	=	185,300
Handrail	53,700	L.F.	@	20.00	=	1,074,000
Sub-Total Superstructure						$ 41,160,900
Substructure						
Pier Excavation	60,000	C.Y.	@	$ 31.20	=	$ 1,872,000
36" Dia. Pipe Piles	60,800	L.F.	@	200.00	=	12,160,000
Precast Concrete	12,500	C.Y.	@	1,280.00	=	16,000,000
Tremie Concrete	22,800	C.Y.	@	165.00	=	3,762,000
Shell Filler Concrete	21,800	C.Y.	@	165.00	=	3,597,000
Pier Cap Concrete	5,500	C.Y.	@	468.00	=	2,574,000
Reinforcing Steel	7,302,000	Lbs.	@	0.50	=	3,651,000
Shaft Armoring (Steel)	980,000	Lbs.	@	1.20	=	1,176,000
Foundation Armor Rock	65,000	C.Y.	@	40.00	=	2,600,000
Sub-Total Substructure						$ 47,392,200
Total for Structure						$ 91,938,200
Mobilization						$ 23,000,000
Total for Crossing						$114,938,200
Contingencies and Variations (±10%)						$ 11,061,800
TOTAL ESTIMATED CONSTRUCTION COST						$126,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

ESTIMATE OF COST

CAUSEWAY DAM AT CROSSING V

Spillway Bridge at 2,640'
Spillway Ogee Section at 2,000'
Total Crossing Length = 28,050'
Roadway: 2 @ 30'-0"

SPILLWAY BRIDGE:						
Superstructure						
Structural Steel (A588)	3,855,000	Lbs.	@	$ 0.70	=	$ 2,698,500
Cast Steel	80,000	Lbs.	@	1.56	=	124,800
Concrete in Deck	4,180	C.Y.	@	220.00	=	919,600
Reinforcing Steel	1,150,000	Lbs.	@	0.30	=	345,000
Handrail	5,280	L.F.	@	22.50	=	118,800
Guardrail	2,640	L.F.	@	22.50	=	59,400
Sub-Total Superstructure						$ 4,266,100
Substructure						
Concrete in Piers	2,620	C.Y.	@	$220.00	=	$ 576,400
Concrete in Footings	1,560	C.Y.	@	175.00	=	273,000
Reinforcing Steel	687,000	Lbs.	@	0.30	=	206,100
14HP73 (45' Long)	29,700	L.F.	@	35.00	=	1,039,500
Excavation	4,800	C.Y.	@	10.50	=	50,400
Sub-Total Substructure						$ 2,145,400
SPILLWAY AND CHANNEL:						
Spillway Excavation						
Excavate Above Elev. 20	17,500,000	C.Y.	@	$ 1.75	=	$ 30,625,000
Excavate Below Elev. 20	2,900,000	C.Y.	@	2.65	=	7,685,000
Spillway Concrete						
Ogee Section Concrete	37,500	C.Y.	@	$175.00	=	$ 6,562,500
Reinforcing Steel	3,500,000	Lbs.	@	0.30	=	1,050,000
DAM:						
Rock-End Dumped						
Zone 1 0.5'-1.0'	4,050,000	C.Y.	@	$ 6.10	=	$ 24,705,000
Zone 2 1.0'-2.3'	2,500,000	C.Y.	@	6.35	=	15,875,000
Zone R 36" Riprap	1,000,000	C.Y.	@	8.25	=	8,250,000
Rock-Barge Dumped						
Zone 2 1.0'-2.3'	500,000	C.Y.	@	$ 12.50	=	$ 6,250,000
Zone 3 2.6'-3.0'	750,000	C.Y.	@	13.00	=	9,750,000
Zone R 36" Riprap	200,000	C.Y.	@	14.30	=	2,860,000
Rock-Cableway Placed						
Zone 2 1.2'-2.3'	200,000	C.Y.	@	$ 16.00	=	$ 3,200,000
Zone 3 2.6'-3.0'	200,000	C.Y.	@	16.80	=	3,360,000
Zone 4 3.8'-4.0'	900,000	C.Y.	@	18.00	=	16,200,000
Causeway Sand Fill						
Dredged Sand Fill	8,500,000	C.Y.	@	$ 1.82	=	$ 15,470,000
Gravel Filter	500,000	C.Y.	@	9.55	=	4,775,000
Causeway Crest Road						
Crest Road - Complete	25,600	L.F.	@	$ 61.00	=	$ 1,561,600
Total for Structure						$164,590,600
Mobilization						$ 25,000,000
Total for Crossing						$189,590,600
Contingencies and Variations (±10%)						$ 19,409,400
TOTAL ESTIMATED CONSTRUCTION COST						$209,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

ESTIMATE OF COST

TRUSS BRIDGE AT CROSSING V

2 Approach Structures at 747' and 785' = 1,532'-0"
Main Bridge 65 Spans at 400'-0" = 26,000'-0"
Total Bridge Length = 27,532'-0"
Roadway: 2 @ 30'-0"

APPROACH STRUCTURES:						
Superstructure						
Structural Steel (A588)	1,670,000	Lbs.	@	$ 0.64	=	$ 1,068,800
Cast Steel	34,000	Lbs.	@	1.45	=	49,300
Concrete in Deck	1,230	C.Y.	@	200.00	=	246,000
Reinforcing Steel	335,000	Lbs.	@	0.28	=	93,800
Handrail	3,055	L.F.	@	20.00	=	61,100
Sub-Total Superstructure						$ 1,519,000
Substructure						
Concrete in Columns and Cap Beams	480	C.Y.	@	$ 200.00	=	$ 96,000
Concrete in Footings and Abutments	520	C.Y.	@	170.00	=	88,400
Reinforcing Steel	190,000	Lbs.	@	0.28	=	53,200
14HP73 (45' Long)	12,660	L.F.	@	30.00	=	379,800
Excavation (Above Grade)	130,000	C.Y.	@	2.40	=	312,000
Excavation (Below Grade)	1,600	C.Y.	@	10.00		16,000
Sub-Total Substructure						$ 945,400
MAIN BRIDGE:						
Superstructure						
Structural Steel (A588)	111,280,000	Lbs.	@	$ 0.64	=	$ 71,219,200
Cast Steel	2,210,000	Lbs.	@	1.44	=	3,182,400
Asphaltic Wearing Surface	179,400	S.Y.	@	16.00	=	2,870,400
Deck Protection	179,400	S.Y.	@	2.00	=	358,800
Handrail	104,000	L.F.	@	20.00	=	2,080,000
Sub-Total Superstructure						$ 79,710,800
Substructure						
Pier Excavation	82,400	C.Y.	@	$ 31.50	=	$ 2,595,600
36" Dia. Pipe Piles	109,000	L.F.	@	200.00	=	21,800,000
Pre-Cast Concrete	20,200	C.Y.	@	1,440.00	=	29,088,000
Tremie Concrete	33,000	C.Y.	@	165.00	=	5,445,000
Shell Filler Concrete	30,100	C.Y.	@	165.00	=	4,966,500
Pier Cap Concrete	11,000	C.Y.	@	448.00	=	4,928,000
Reinforcing Steel	10,900,000	Lbs.	@	0.52	=	5,668,000
Shaft Armoring (Steel)	2,850,000	Lbs.	@	1.20	=	3,420,000
Foundation Armor Rock	106,000	C.Y.	@	40.00	=	4,240,000
Sub-Total Substructure						$ 82,151,100
Total for Structure						$164,326,300
Mobilization						$ 38,000,000
Total for Crossing						$202,326,300
Contingencies and Variations ±10%						$ 20,673,700
TOTAL ESTIMATED CONSTRUCTION COST						$223,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

ESTIMATE OF COST

BRIDGE-CAUSEWAY AT CROSSING V

Goose Bay:	2 Approach Structures @ 747' and 820'	=	1,567'-0"
	Main Bridge 16 Spans @ 400'-0"	=	6,400'-0"
			7,967'-0"
Eagle Bay:	2 Approach Structures @ 820' and 785'	=	1,605'-0"
	Main Bridge 13 Spans @ 400'	=	5,200'-0"
			6,805'-0"

Total Bridge Length = 14,772'-0"

Roadway: 2 @ 30'-0"

APPROACH STRUCTURES:							
Superstructure							
Structural Steel (A588)	3,475,000	Lbs.	@	$ 0.64	=	$ 2,224,000
Cast Steel	70,000	Lbs.	@	1.45	=	101,500
Concrete in Deck	2,560	C.Y.	@	200.00	=	512,000
Reinforcing Steel	695,000	Lbs.	@	0.28	=	194,600
Handrail	6,335	L.F.	@	20.00	=	126,700
Sub-Total Superstructure						$ 3,158,800
Substructure						
Concrete in Columns and Caps	960	C.Y.	@	$ 200.00	=	$ 192,000
Concrete in Footings and Abutments	1,040	C.Y.	@	170.00	=	176,800
Concrete in Retaining Walls	4,480	C.Y.	@	170.00	=	761,600
Reinforcing Steel	690,000	Lbs.	@	0.28	=	193,200
14HP73 (45' Long)	51,720	L.F.	@	30.00	=	1,551,600
Excavation (Above Grade)	130,000	C.Y.	@	2.40	=	312,000
Excavation (Below Grade)	8,800	C.Y.	@	10.00	=	88,000
Sub-Total Substructure						$ 3,275,200
MAIN BRIDGE:						
Superstructure						
Structural Steel (A588)	49,650,000	Lbs.	@	$ 0.64	=	$ 31,776,000
Cast Steel	970,000	Lbs.	@	1.44	=	1,396,800
Asphaltic Wearing Surface	80,050	S.Y.	@	16.00	=	1,280,800
Deck Protection	80,050	S.Y.	@	2.00	=	160,100
Handrail	46,400	L.F.	@	20.00	=	928,000
Sub-Total Superstructure						$ 35,541,700
Substructure						
Pier Excavation	43,000	C.Y.	@	$ 31.60	=	$ 1,358,800
36" Dia. Pipe Piles	54,000	L.F.	@	206.00	=	11,124,000
Pre-Cast Concrete	10,500	C.Y.	@	1,310.00	=	13,755,000
Tremie Concrete	18,800	C.Y.	@	165.00	=	3,102,000
Shell Filler Concrete	18,100	C.Y.	@	165.00	=	2,986,500
Pier Cap Concrete	4,600	C.Y.	@	497.00	=	2,286,200
Reinforcing Steel	6,150,000	Lbs.	@	0.51	=	3,136,500
Shaft Armoring (Steel)	980,000	Lbs.	@	1.20	=	1,176,000
Foundation Armor Rock	58,000	C.Y.	@	24.25	=	1,406,500
Sub-Total Substructure						$ 40,331,500
Sub-Total Approach Structures and Main Bridge						$ 82,307,200
Causeway Fill						
Zone 1 Rockfill 0.5'-1.0'	4,836,000	C.Y.	@	$ 10.40	=	$ 50,294,400
Zone 2 Rockfill 1.0'-2.0'	1,196,000	C.Y.	@	11.40	=	13,634,400
Riprap - 36" Dia.	1,397,500	C.Y.	@	14.40	=	20,124,000
Roadway Paving	14,700	L.F.	@	56.00	=	823,200
Sub-Total Causeway						$ 84,876,000
Total for Structure						$167,183,200
Mobilization						$ 42,000,000
Total for Crossing						$209,183,200
Contingencies and Variations ±10%						$ 20,816,800
TOTAL ESTIMATED CONSTRUCTION COST						$230,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

ESTIMATE OF COST

SUSPENSION BRIDGE AT CROSSING III

Spans : 2 @ 165', 1,500', 4,500', 1,500', 4 @ 265' = 8,890'
Roadway: 2 @ 30'-0"

MAIN BRIDGE						
Superstructure						
Structural Steel (A588)	76,301,000	Lbs.	@	$ 0.70	=	$ 53,410,700
Main Cables	41,980,000	Lbs.	@	0.90	=	37,782,000
Cable Wrapping	761,000	Lbs.	@	0.90	=	684,900
Cable Suspenders	1,442,000	Lbs.	@	2.30	=	3,316,600
Cable Bands and Fittings	1,384,000	Lbs.	@	1.70	=	2,352,800
Hand Ropes	17,800	L.F.	@	4.50	=	80,100
Roadway Protective Coating	53,000	S.Y.	@	2.30	=	121,900
Roadway Wearing Surface	53,000	S.Y.	@	18.40	=	975,200
Handrail	15,000	L.F.	@	23.00	=	345,000
Guardrail	7,500	L.F.	@	23.00	=	172,500
Sub-Total Superstructure						$ 99,241,700
Anchorages						
Concrete	124,400	C.Y.	@	$180.00	=	$ 22,392,000
Sand Fill	60,600	C.Y.	@	4.50	=	272,700
Reinforcing Steel	10,000,000	Lbs.	@	0.37	=	3,700,000
Eye Bars	3,120,000	Lbs.	@	0.70	=	2,184,000
Structural A36 Steel	1,530,000	Lbs.	@	0.70	=	1,071,000
Cast Steel	332,000	Lbs.	@	1.70	=	564,400
Forged Steel	286,000	Lbs.	@	1.70	=	486,200
Excavation Above Elev. 100	1,775,000	C.Y.	@	0.90	=	1,597,500
Excavation Below Elev. 100	660,000	C.Y.	@	2.30	=	1,518,000
Granular Backfill	440,000	C.Y.	@	5.50	=	2,420,000
Sub-Total Anchorages						$ 36,205,800
Main Piers						
Concrete in Shafts	25,800	C.Y.	@	$180.00	=	$ 4,644,000
Concrete in Caissons	62,500	C.Y.	@	275.00	=	17,187,500
Concrete in Seals	24,100	C.Y.	@	180.00	=	4,338,000
Reinforcing Steel	7,100,000	Lbs.	@	0.37	=	2,627,000
Excavation	128,700	C.Y.	@	37.00	=	4,761,900
Sub-Total Main Piers						$ 33,558,400
End Piers and Approach Spans						
Handrail	2,800	L.F.	@	$ 23.00	=	$ 64,400
Guardrail	1,400	C.Y.	@	23.00	=	32,200
Deck Concrete	2,060	C.Y.	@	460.00	=	947,600
Reinforcing Steel	2,300,000	Lbs.	@	0.37	=	851,000
Structural Steel (A588)	5,872,000	Lbs.	@	0.70	=	4,110,400
Abutment Concrete	250	C.Y.	@	230.00	=	57,500
Pier Concrete	7,300	C.Y.	@	275.00	=	2,007,500
Seal Concrete	7,940	C.Y.	@	180.00	=	1,429,200
Excavation	7,200	C.Y.	@	55.00	=	396,000
14HP73 (60' Long)	6,000	L.F.	@	35.00	=	210,000
18" Ø Pipe Piles (90' Long)	34,800	L.F.	@	40.00	=	1,392,000
Sub-Total End Piers and Approach Spans						$ 11,497,800
Slope Protection						
Excavation Above Elev. 100	500,000	C.Y.	@	$ 0.90	=	$ 450,000
Excavation Below Elev. 100	700,000	C.Y.	@	1.65	=	1,155,000
Sand and Gravel Bedding	50,000	C.Y.	@	18.50	=	925,000
Rock Spalls Bedding	50,000	C.Y.	@	27.50	=	1,375,000
1'-3' Rock Bedding	75,000	C.Y.	@	37.00	=	2,775,000
Armor Stone (Tetrapods)	250,000	C.Y.	@	73.50	=	18,375,000
Toe Drain	2,000	L.F.	@	320.00	=	640,000
Sub-Total Slope Protection						$ 25,695,000
Total for Structure						$206,198,700
Mobilization						20,000,000
Total for Crossing						$226,198,700
Contingencies and Variations (±10%)						22,801,300
TOTAL ESTIMATED CONSTRUCTION COST						$249,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

ESTIMATE OF COST

CAUSEWAY DAM AT CROSSING IV

Spillway Bridge at 2,640'
Spillway Ogee Section at 2,000'
Total Crossing Length = 15,950'
Roadway: 2 @ 30'-0"

SPILLWAY BRIDGE:						
Superstructure						
Structural Steel (A588)	3,855,000	Lbs.	@	$ 0.72	=	$ 2,775,600
Cast Steel	80,000	Lbs.	@	1.64	=	131,200
Concrete in Deck	4,180	C.Y.	@	230.00	=	961,400
Reinforcing Steel	1,150,000	Lbs.	@	0.32	=	368,000
Handrail	5,280	L.F.	@	22.50	=	118,800
Guardrail	2,640	L.F.	@	22.50	=	59,400
Sub-Total Superstructure						$ 4,414,400
Substructure						
Concrete in Piers	2,620	C.Y.	@	$230.00	=	$ 602,600
Concrete in Footings	1,560	C.Y.	@	180.00	=	280,800
Reinforcing Steel	687,500	Lbs.	@	0.32	=	220,000
14HP73 (45' Long)	29,700	L.F.	@	36.00	=	1,069,200
Excavation	4,800	C.Y.	@	11.00	=	52,800
Sub-Total Substructure						$ 2,225,400
SPILLWAY AND CHANNEL:						
Spillway Excavation						
Excavate Above Elev. 20	21,500,000	C.Y.	@	$ 3.40	=	$ 75,250,000
Excavate Below Elev. 20	2,600,000	C.Y.	@	4.50	=	11,700,000
Spillway Concrete						
Ogee Section Concrete	37,500	C.Y.	@	$180.00	=	$ 6,750,000
Reinforcing Steel	3,500,000	Lbs.	@	0.33	=	1,155,000
DAM:						
Rock-End Dumped						
Zone 1 0.5'-1.0'	2,120,000	C.Y.	@	$ 6.70	=	$ 14,204,000
Zone 2 1.0'-2.3'	500,000	C.Y.	@	7.10	=	3,550,000
Zone R 36" Riprap	520,000	C.Y.	@	9.10	=	4,732,000
Rock-Barge Dumped						
Zone 2 1.0'-2.3'	1,600,000	C.Y.	@	$ 12.50	=	$ 20,000,000
Zone R 36" Riprap	280,000	C.Y.	@	13.70	=	3,836,000
Rock-Barged and Rehandled						
Zone 1 0.5'-1.0'	480,000	C.Y.	@	$ 15.00	=	$ 7,200,000
Zone 2 1.0'-2.3'	150,000	C.Y.	@	15.40	=	2,310,000
Rock-Cableway Placed						
Zone 2 1.2'-2.3'	850,000	C.Y.	@	$ 13.60	=	$ 11,560,000
Zone 3 2.6'-3.0'	1,300,000	C.Y.	@	14.30	=	18,590,000
Zone 4 3.8'-4.0'	1,400,000	C.Y.	@	15.20	=	21,280,000
Zone 5 4.0'-5.0'	500,000	C.Y.	@	16.80	=	8,400,000
Causeway Sand Fill						
Dredged Sand Fill	6,200,000	C.Y.	@	$ 2.65	=	$ 16,430,000
Gravel Filter	350,000	C.Y.	@	10.50	=	3,675,000
Roadway Paving						
Complete w/Fencing	13,000	L.F.	@	$ 65.00	=	$ 845,000
Total for Structure						$238,106,800
Mobilization						$ 25,000,000
Total for Crossing						$263,106,800
Contintencies and Variations (±10%)						$ 25,893,200
TOTAL ESTIMATED CONSTRUCTION COST						$289,000,000

Estimate is based on fall 1971 bidding date with construction start in January, 1972.

SCOPE

The assignment undertaken was to prepare a report considering a number of alternate crossing designs and locations for the proposed Knik Arm crossing near Anchorage, Alaska. Early studies resulted in elimination of a number of alternates and ultimately six estimates were made for different designs at three locations. The report was to be concerned only with the section of the crossing between the bluffs, including the spillway area in the case of the two Causeway Dam designs. No consideration was to be given to the approaches and access roads.

The primary objective of the report was to present feasibility level cost estimates prepared fromthe viewpoint of the contractor. Due to the lack of detailed design, it is an impossible task to prepare a thorough contractor-type. cost estimate. These estimates were prepared generally in approximately the manner used by contractors with separate consideration of direct costs, overhead, escalation, contingencies and profit.

In addition to the costs the report was to present our opinion on significant aspects of the construction problems. No matter which of the alternate crossing designs is selected, the construction problems are virtually unprecedented. The most significant problems to be overcome are obviously the large tide differentials, the swift water currents, the large volume of work to be accomplished and the weather.

The report was not to make a recommendation as to which of the alternates is the best answer to the problem. It was to provide the hard facts on construction costs and other problems that can be expected, so that these could be weighed against the other factors of desirable location, ability to carry railroad, the possible large fresh water lake, and other economic and environmental factors that enter into a final recommendation.

STATUS OF DESIGN

In any feasibility level study, the preliminary status of the design leaves considerable of the detail of the work to the judgment and imagination of the estimator. The design drawings, preliminary reports and HNT&B quantity takeoffs furnished to us were adequate for feasibility estimates. Nevertheless, we have found that preliminary designs nearly alway fail to show details that always tend to add to the costs, and in summation add substantially to the costs. Our practice in feasibility level estimating is to compensate for the lack of details by using conservative crews and production rates throughout the estimate.

Another problem with preliminary designs is that they tend to change. The receipt of the Shannon & Wilson Soils Report made very drastic changes in the causeway dam fill section and in the line III and line IV bridge abutment slope protection requirements. As the major portion of our estimate details had been based on the prior designs, some rapid adjustments of quantities and operations costs were required to fit the new designs.

The estimates now reflect the work required by Figure 8 and Figure 9 of the Shannon & Wilson report, with only slight modification. We have assumed that the 10' gravel blanket in Figure 9 will be zone 1(0.5'-1.0') rock. This assumption is largely based on the HNT&B hydraulic study that determined the sizes of rock needed for stability in the current at the various stages of construction. It is also necessary to assume substantial rock protection for the ends and upstream face of the island formed by the causeway-bridge at Line V.

AVAILABILITY OF LABOR

The availability of labor possessing the required skills can be a major problem for large scale construction work in Alaska. Historically there have been "booms and busts" that resulted in major swings in the labor market. While currently the labor supply is reasonably good, the possibility of the oil pipeline to the North Slope and a number of other major construction projects getting started next year would drastically alter the situation and lead to the necessity of paying sizeable bonuses to attract a sufficient supply of skilled workers.

Our estimates were based on the assumption that the nearness to Anchorage would make the job somewhat more attractive to labor than remote sites, primarily because family status accommodations could be found in town or provided in trailer camps. We have estimated that a construction camp or subsistence payments will not be required. Still, some inducement for labor will be required, and we estimate that the 28.6% premium for the scheduled seven day work week (i.e., 56 hours work per week with double overtime for all hours over 40. 72/56=1.286) will be sufficient to attract the necessary work force. It is noted that this will give the skilled craftsmen an average of about $14.00 per hour worked per week in base plus overtime pay plus $1.85 to $2.45 per hour in fringes for the average of the projected 1973-74 wage rates. For the estimated eight month season worked, the skilled craftsman will receive about $27,000 plus nearly $4,000 in fringe benefits.

WORKING SEASON

The basic working season is considered to be an eight month period, with a four month winter shutdown. For the excavation work, rainfall, fog, frost breakup, and snowfall will hinder production at various times in the eight month period to the extent that our scheduling is based on the equivalent of 170 days per season of full production in good weather. For work off floating equipment the same four month winter shutdown is required to avoid the icing problem. Rainfall will not have as serious an effect on floating equipment operations, and an equivalent of 200 days per season of full production is allowed in scheduling. Structural steel work might well be accomplished during the winter as is reported to have been done on Anchorage building work. However, the bridge steel has to be barged out to the bridge for erection, and barge movement is not considered practical in the four month winter period.

DIRECT COSTS

Direct costs are considered to be those incurred by field crews in performing work operations directly related to, or required by, the pay items of work. Our estimate of the direct costs of the various items of work is for the most part based on a work operations analysis. This consists of breaking the bid items into the many different work operations required for a complete bid item. Then each operation is estimated by calculating the crew, equipment, and materials needed for the operation, based on the anticipated production. The factors of crew sizes, equipment requirements, and productivity are basically determined by the experience and judgment of the estimator. In a few cases we have used some historical cost data, adjusted to fit the job conditions.

Detailed unit direct costs were calculated for nearly all of the operations in the two lowest priced estimates, the Bridge at Line IV and the Dam at Line V. From these detailed operations costs, adjustments were made to fit similar type operations on the other alternates. In some cases it was necessary to work out new operation details for some items in the other four estimates.

OVERHEAD AND GENERAL ADMINISTRATION

While it is normal for a contractor to make a detailed estimate of his overhead and general administrative cost,this was not done for the estimates in this report. On the estimates that involve causeway dam construction, we have simply used an OH & GA factor of 20%. This allowance for OH & GA is based on our experience and judgment with regard to large construction projects, spanning several seasons, in Alaska.

For bridge substructure work we have estimated a number of indirect items of work that are large in dollar cost and can only be arbitrarily allocated to a bid item breakdown. As these items include the contractors' yard and shop facilities that were included in the OH & GA costs for the causeway work, we have reduced the OH & GA factor for bridge substructure work to 16% of the total of the direct cost and other indirect costs. Again, this factor is based on experience and judgment.

ESCALATION

The estimates in this report are based on the contractor bid prices that could be expected if bids were taken in the fall of 1971, with construction to start in January 1972. No attempt has been made to forecast the escalation that normally would occur during the delay that will elapse before a single alternate is selected, detailed designs prepared, and bids taken.

However, the estimates do take into consideration the likely escalation in costs that could be expected in the four to six-year construction period. The forecast of labor cost escalation is based on annual increases that decrease from 10% for 1971 - 1972 to 6% between 1975 and 1976. This slowdown in the rate of increases in construction wages is based on the anticipated effect of the Administration's Construction Industry Stabilization Committee organized in the spring of 1971. This represents our forecast of the effectiveness this committee will have on controlling construction wage rates. It is impossible to forecast the effect of the more recent wage freeze and any successor economic controls.

The equipment operating costs are based on 1971 costs with an escalation factor of 20% added. This is the approximate equivalent of 6% escalation per year for three years. This would cover the year 1973, which is likely to be the peak year of use of construction equipment and be close enough to the midpoint of the schedules to use an an average cost.

For materials costs, a factor of 15% over 1971 was used. The materials are not a large part of the construction costs except for bridge superstructures, and the superstructure estimate was made by HNT&B.

MARKUP

The items that a contractor considers in markup are normally escalation, contingency and profit. In these estimates, escalation during the life of the job is already included in the direct and indirect costs and no escalation for delay in starting is to be included. The normal contractor's profit on a job of this magnitude, but with relatively slight risk of unforseeable difficulties, would likely be in the range of 8% to 10%. These estimates have included in the direct cost all the known difficulties in the work, as well as we can forsee them. There is still a sizeable risk in any of these jobs that the foreseen difficulties will be worse than anticipated. Further, in any of these alternate designs, the required construction techniques will stretch the state-of-the-art to the limit, and the partially completed structures and contractors' plant will be exposed to possible severe winter storms for one or more seasons. Inasmuch as a contingency is an allowance for unforeseen difficulties, it is impossible to estimate a contingency. In view of these factors we have used a total markup (profit and contingency combined) of 15% of total costs for all the estimates except the causeway dam at line IV. This latter alternate is considered to be the most hazardous undertaking for a contractor and consequently we have used a total markup of 20% of total cost of this estimate. The difference between the total markup and normal profit represents an assessment of the contingency allowance a contractor would make.

BID PRICES

The determination of the bid prices used in these estimates has been based on a completely balanced bid. A factor is determined by dividing the total of direct costs, indirect costs and markup by the direct costs. Then this factor is uniformly applied to all unit direct costs and the result is slightly rounded off to give reasonably round numbers for bid unit prices. After extension of the rounded bid unit price times the quantity of work, the resulting amounts are totaled and checked back to make sure that the rounding-off procedure did not result in a significant increase or decrease in the total bid.

In actual bidding, contractors seldom make a completely balanced bid. For work spanning several seasons they will commonly bid higher on the items of work to be performed early in the job in order to gain a financing advantage. There are many other philosophies and systems used in getting from direct costs to bid prices for work items, but such a discussion is outside the scope of this assignment. It should be noted that sizeable mobilization items have been included in the cost estimates. Very little pay work items will be accomplished in the first season of work, but the contractor will have invested at least these amounts in the job. As the State can borrow money at interest rates likely to be in the range of 4% less than most contractors pay, the inclusion of a mobilization item in the bid tabulation should result in a net cost savings to the State.

BRIDGE CONSTRUCTION

We have made a detailed study of the proposed bridge pier construction sequence suggested by HNT&B. We are in basic agreement that the use of precast concrete shells supported on 36" diameter pipe piles is a good workable solution to the problem. Further study might show that fabricated steel shells might be competitive with the precast concrete shells. The steel shells might weigh much less and likely could be fabricated in the Washington-Oregon area and barged to site.

We have generally concurred with the various steps in the sequence of construction proposed by HNT&B. One modification that we believe is necessary is to utilize a ring caisson with cutting edge to support the sides of the bridge pier excavation to prevent current damage or silting of the excavation and help break the force of the current after setting the base shell in a precise location. An extensive anchoring system will be required to hold the caissons, precast pier sections, and the floating equipment in place against the force of the tidal current.

While we would plan to use jackup-type dredges, derricks and pile drivers for this work, it would still be necessary to provide access dredging to the piers located in the shallower water on each side of the river. This would be required because movement of tugs and barges would be necessary at all stages of tide levels, and cannot be restricted to periods of high tides only. Even the jackup-type equipment will require a fairly large draft when being moved from pier to pier. We have assumed that access dredging will be required to elevation minus 35 MSL and to a width of 150' with 4:1 side slopes for the necessary flotation and maneuvering space.

Another major consideration in bridge construction will be the construction of a major dock and yard area. This would be located as close as possible to one abutment of the bridge, on the Anchorage side. The dock will not only be required for day to day usage in shipping materials to the piers and handling tugs and workboats, but also will be required for docking the large floating equipment during winter layup and major storms.

A large yard area will be required immediately adjacent to the dock facilities for precasting of the major bridge pier shell sections and storage of superstructure steel. It may also be used as the site of considerable fabrication of superstructure steel, in particular the orthotropic deck sections. The most likely method of superstructure erection will be assembly of complete 400' truss spans on barges at the jobsite dock facilities for placement by the barge sinking method, except that the outgoing tide action will eliminate the need to sink the barge. A system for handling the major lifts of precast concrete segments will be required in the yard and at the dock side for loading these segments onto barges for transfer to the pier sites.

All of the facilities noted above will be required for any of the various bridge alternates. The detailed layout and estimate of the required facilities has not been made at this time. We have made what we believe is a reasonable allowance for these facilities in our calculation of indirect costs of the bridges.

We believe that the work on the bridges will basically proceed on a single shift basis seven days per week. This is primarily due to the need to have the best possible visibility for this type of work. We have assumed that it will be necessary to do some juggling of the starting and finishing times each day and to pay some overtime during most critical operations in order to make best use of tidal fluctuations. Productivity will be low due to the adverse tide situation. We have planned to use helicopters rather extensively to facilitate communications between floating plant and shore. We have even planned to use helicopters for changing crews, as we believe too much time would be lost in attempting to use more standard type crewboats on these long crossings in swift tidal currents.

PRE-CAST CONCRETE PIER SHELL

The largest item in bridge substructure costs is the pre-cast concrete pier shells. About 40% of the direct cost of this item is in the fabrication on shore of the pre-cast segments. The high cost of this work is due to the very high form ratio (SF/CY) of the shell sections, together with the need for precision work in order to get watertight seals when the sections are joined.

About 50% of the cost of this item is in setting the pre-cast shell sections. We have estimated that it will take the large derrick barge one day to set the erection guide towers, one day to set each light (25-80 ton) lift, and three days to set each heavy (200-400 ton) lift. For the bridge at Line IV, a total of 300 days will be required to set the precast segments. The production on this item will be the control on overall progress of the bridge pier construction.

OTHER BRIDGE CONCRETE

The remaining bridge concrete was seperated into items for tremie concrete, shell-filler concrete, and pier cap concrete. These items of work are relatively straight-forward operations with no particular difficulty foreseen beyond the general job conditions. The large range in unit prices is primarily a function of the formwork ratio and finishing work required by the respective items.

36" PIPE PILES

A large portion of the substructure cost is in the 36" pipe piles. We have used the HNT&B supplied takeoff for the quantity of piles to be driven. We have assumed a 1/2" wall thickness for the steel piles. When the required concrete fill is added to the pipe cost we have a material cost of $48.00 per LF that is about half of the direct costs of $95 to $102 per LF for this item. We also note that about 5,000 LF extra pipe will have to be brought in order that the piles will stick out above high tide until separate crews can come in to cleanout the piles and place the concrete fill. We have estimated that pile driving production of 4 piles (400 LF) per day is all that can be obtained in these tidal water and dense sand conditions. Cleaning out the piles will move faster, at the rate of 6 piles per day. The cutoff and concrete fill operation should proceed at a pace of 10 piles per day, or two days per pier.

It is noted that extra heavy special design pile leads will be required to handle these large diameter long piles in swift currents. The peak tidal current of about 13 FPS is many times the normal current condition for this type of pile driving, and will cause large bending loads on the pile leads. Divers will have much trouble in these waters, as they commonly have difficulty working in 2FPS currents.

BRIDGE SUPERSTRUCTURE

The estimates for the bridge superstructures are basically those supplied by HNT&B. These cover a 4,500' main span suspension bridge at Line III and a series of 400' span orthotropic deck truss spans at Line IV and Line V. We have not attempted to make a detailed estimate of the structural steel superstructure work as this was not in the scope of the assignment. However, we have reviewed the plans and estimates with a major bridge contractor and he concurs that your estimated bid prices for superstructure steel are reasonable. This concurrence assumes that the specifications will require use of American steel and an American contractor. If foreign competition for the supply of the steel and construction is permitted, the bids received may be in the range of 10% less than those shown. The major competition would come from the Japanese: The effect of the recent 10% import duty on foreign goods would narrow but probably not eliminate the differential. The truth of the situation is that the Japanese know the markets well enough so that they never bid more than 10% under American competition, but we do not know how much more they could afford to cut prices if they were forced to do so.

The HNT&B superstructure estimates were included herein to give a valid comparison of the total bid prices for a complete crossing, and not just on the portions of the work that we independently priced.

SOURCE OF ROCKFILL:

Shannon & Wilson have reported that rock is to be found at both the Eklutna Quarry and Rainbow Quarry. It may be possible to find the rock a little closer to the crossing sites (in the Eagle River Valley), but the existence of the Alaska Railroad adjacent to these two quarry sites would cause one of them to be used. Further investigation at Eklutna would be necessary to determine if adequate large rock could be found there, but it apparently would be found at Rainbow.

It happens that both sites are just about exactly equally distant from the causeway abutments after constructing access rail lines to the abutment areas. Therefore the choice of Eklutua over Rainbow is largely to avoid running trains through the city on a 24-hour basis. Rainbow is partially developed, but also is tightly packed next to the mainline and between the water and mountain.

For the 26-mile haul from the quarry to the causeway the logical choice for quantities in excess of 1,000,000 CY is rail haulage. The savings over truck haul are estimated to be $1.50 per CY. About 9 miles of haulage track construction and 2 miles of sidings and yards would have to be constructed to gain access to the causeways.

The quarrying operation would be relatively standard type work. Substantial clearing and stripping work would be required to develop sufficient quantity of rock in total, and open up enough face for volume production. The prime loading units would be rubber-tired front end loaders of 10 CY capacity, giving both productivity and flexibility. Much selection and temporary stockpiling of rock will be required to save the larger rock for closure and rip rap work. Considerable undersized waste will be generated in trying to get large rock. We have estimated as much as 60% of the quarry may have to be wasted in producing 4.0' rock, with an overall wastage of 22% of the rock quarried for all classes of rock required for the causeway dam at Line V.

DUMPED ROCKFILL

All of the rock that can be directly dumped into place will be so placed as this is the cheapest placement operation. It is not feasible to attempt to place the rock by dumping directly from the railroad cars used for haulage from the quarry. The railroad cars would be side-dumped from a trestle, or retained fill, at a convenient location on the shoreline. The typical 10 CY front-end loader will be used at the rail trestle to pick up the rock and load 60 ton rocker dump trucks. These trucks were picked for tight turning radius, steep dump angle, and drivers on firm ground when dumping at edge of fill. However, others might select straight rear-dump trucks and possibly gain a slight cost advantage.

No compaction other than traffic and the washing action of the tidal water flow was considered necessary for compacting rockfill. This assumption should be checked with the fill designers, but we see no way of compacting rock below water.

BARGED ROCKFILL:

These estimates are based on using Manitowac hydro-dump barges of 2,000 ton capacity and 14-foot draft. A 600' long barge loading pier would have to be constructed. We have planned to use filled sheet pile cells for the barge pier. An access channel would have to be dredged to the pier for flotation at all tide levels. The rock would have to be lowered into the barge by large clamshell cranes to minimize the damage caused by the free dropping of large rock. For the Causeway Dam at Line V, three barges and two tugs would have to average three trips total per day. This would be a 75% efficiency factor on the four slack tides per day. Only one trip per tide is seen possible for each tug in these current conditions.

BARGED AND REHANDLED ROCKFILL:

In several instances it will be necessary to barge rockfill, and then pick it up again to place it higher on the fill. We have planned to barge dump into place only that rockfill below Elev. - 10 MSL. This is the level at which you would have sufficient freeboard under a loaded barge over 30% of the time in the total tide range. We would not estimate that we could plan to maintain barge production over any tighter range of available tides.

Rehandling would be done by reclaiming with a dragline handling a 6 CY bucket. In some cases the dragline could rehandle the material directly into place on the fill by swinging 90° - 180°. In other cases it was necessary to load trucks for hauling reclaimed material to the final location:

CABLEWAY PLACED ROCKFILL:

While the lower portions of the 6,000' closure sections can be placed by direct barge dumping, the only feasible way to place the critical middle stages of the closure is by cableway. Considerable research of the literature and discussions with manufacturers led to the adopted cableway design. This would be Tramway-type system with 4" suspension cables and 1-1/2" haulage cable. Carriages for 20 ton rock skips would be spaced at 700' centers on the haul cable. The cableway would be supported in 1400' spans on intermediate towers erected on sunken barges specially designed and fabricated for this purpose. No single 4" cable can be made to span the full 6,600' overall length, but special fittings can be made to permit use of multiple lengths of cables dead-ended at the tower and still have the Tramway carriages pass over the splice locations.

The Tramway haul cable would travel at a line speed of 400 FPM, and at each end the Tramway would have a turnaround track to reverse the direction of the carriages and haul cable. The skips would be automatically dumped by a timing device at the proper location for maintaining the desired level weir crest. The estimated average production for this cableway would be 400 CY/HR worked, just sufficient to make the Closure at Line V in one season of around the clock operation. For the Closure at Line IV, three seasons of cableway placement would be required. It is somewhat doubtful that a prudent man would undertake the latter closure, but as this is the highest priced alternate it is unlikely that anyone will have to try it.

DREDGED SAND FILL

The present design section of the causeway fill (Shannon & Wilson Fig. 9) calls for a large volume of dredged sand fill to be placed on the upstream face. We have assumed that this will be placed in the last season after the rockfill closure has been made for the two dam alternates. While the rockfill will be quite pervious, the tidal range will be drastically reduced to a fluctuation that would permit use of standard suction dredges. However, because the channel will be blocked, it will not be possible to use the more economical large size dredges warranted by the volume of fill required. Therefore, three to six smaller portable type dredge will have to be hauled overland and assembled on the newly formed lake. Thus the costs for dredging will be higher than might have been expected.

For the dam at Line V, a large volume of material lies close at hand in the flats between the two main channels. But for the dam at Line IV it would be necessary to go several miles upstream to find sufficient material at shallow depths along the shoreline taking care not to undermine the existing bluffs. Thus the dredging costs at Line IV will be higher than those at Line V.

For the causeway island between bridge sections at line V, we would propose creating artificial lakes for the portable suction dredges in the swampy flat areas near each bridge abutment. The dredge discharge pipeline would be run across the completed bridge decks, using booster pumps as necessary.

SPILLWAY EXCAVATION:

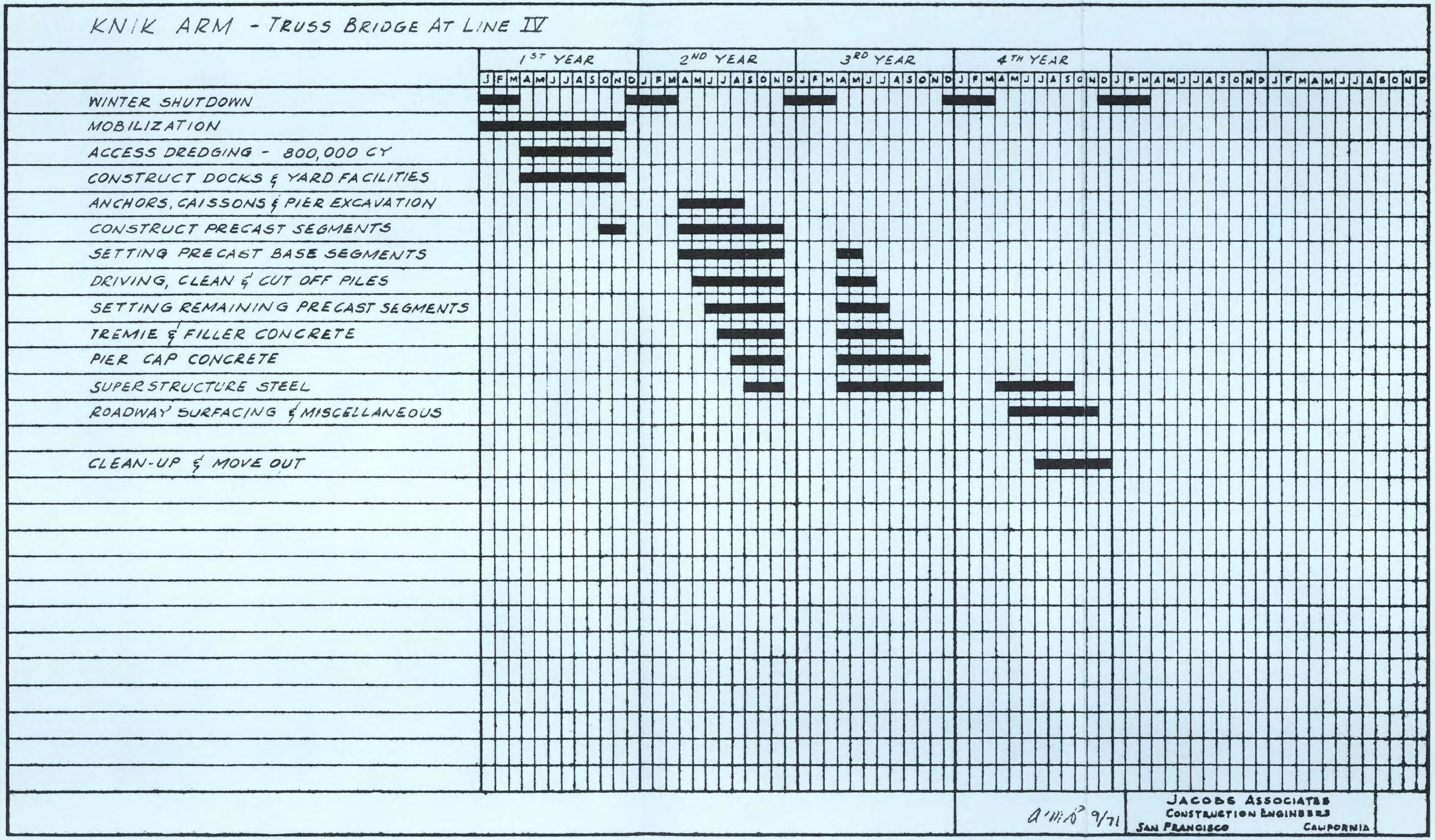
It would be desirable to utilize the spillway excavation in the causeway dam fill if possible, and further study should be made to see if this can not be done. Our estimates are based on wasting the required excavation due to it being unsatisfactory for fill in the opinion of Shannon & Wilson. The tremendous volume of required excavation will present a major disposal problem. We have assumed that it cannot be wasted into the Knik Arm because it would silt up the harbor and cause ecological damage to the fish industry. Our choice of disposal location is adjacent to the bluffs forming the southwest bank of the Eagle River Flats. This is relatively close to Line V, but a four-mile haul from Line IV. It is also a haul up out of a hole and over the bluffs from Line IV, greatly adding to the costs of that alternate.

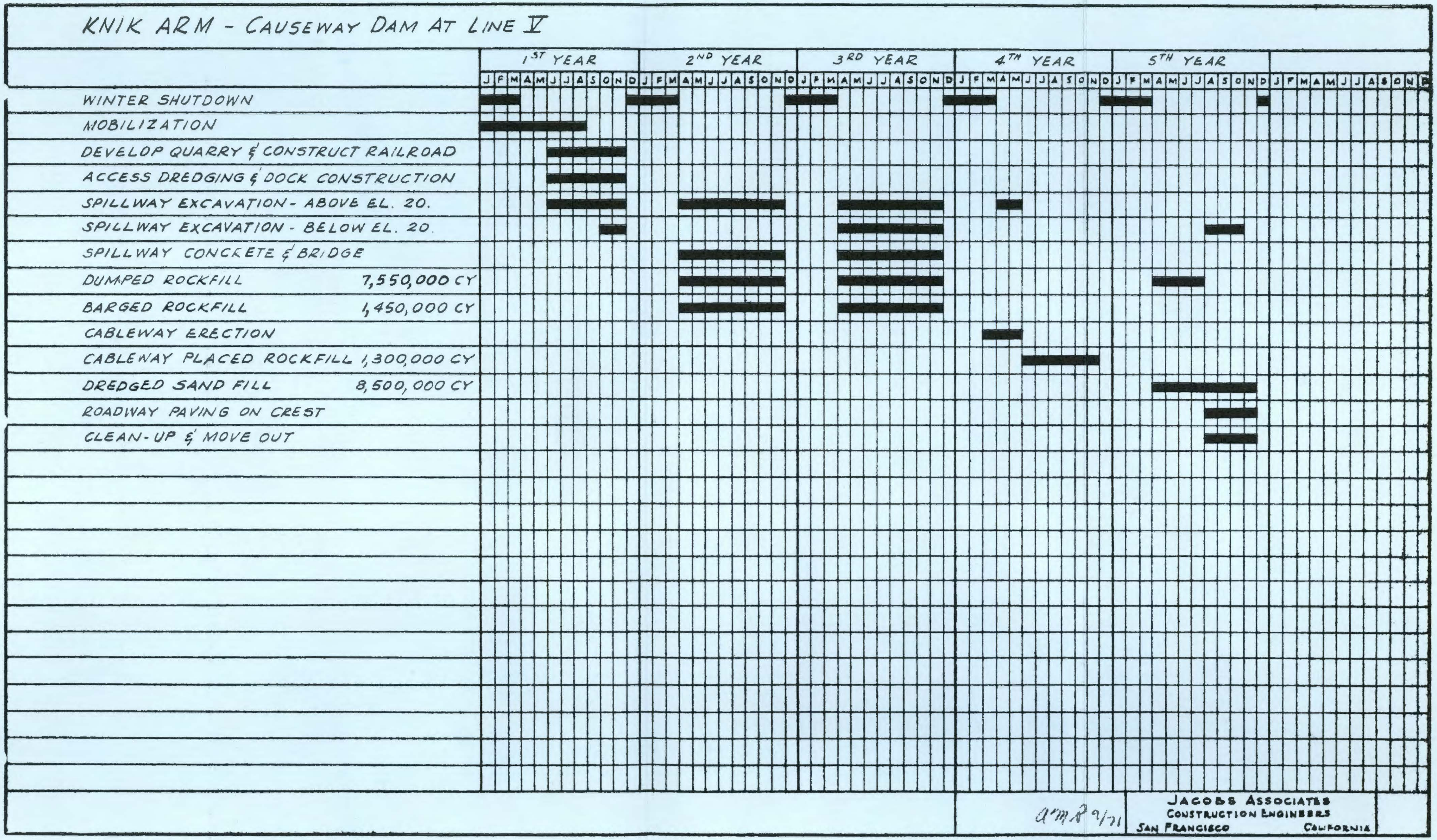
The excavation is estimated to be performed in the relative dry down to elevation 20 MSL with judicious sumping and planning of excavation procedure. This dry excavation will be performed by the typical 10 CY front-end loaders and the hauling by the typical 60-ton wagon dumps or rear dumps.

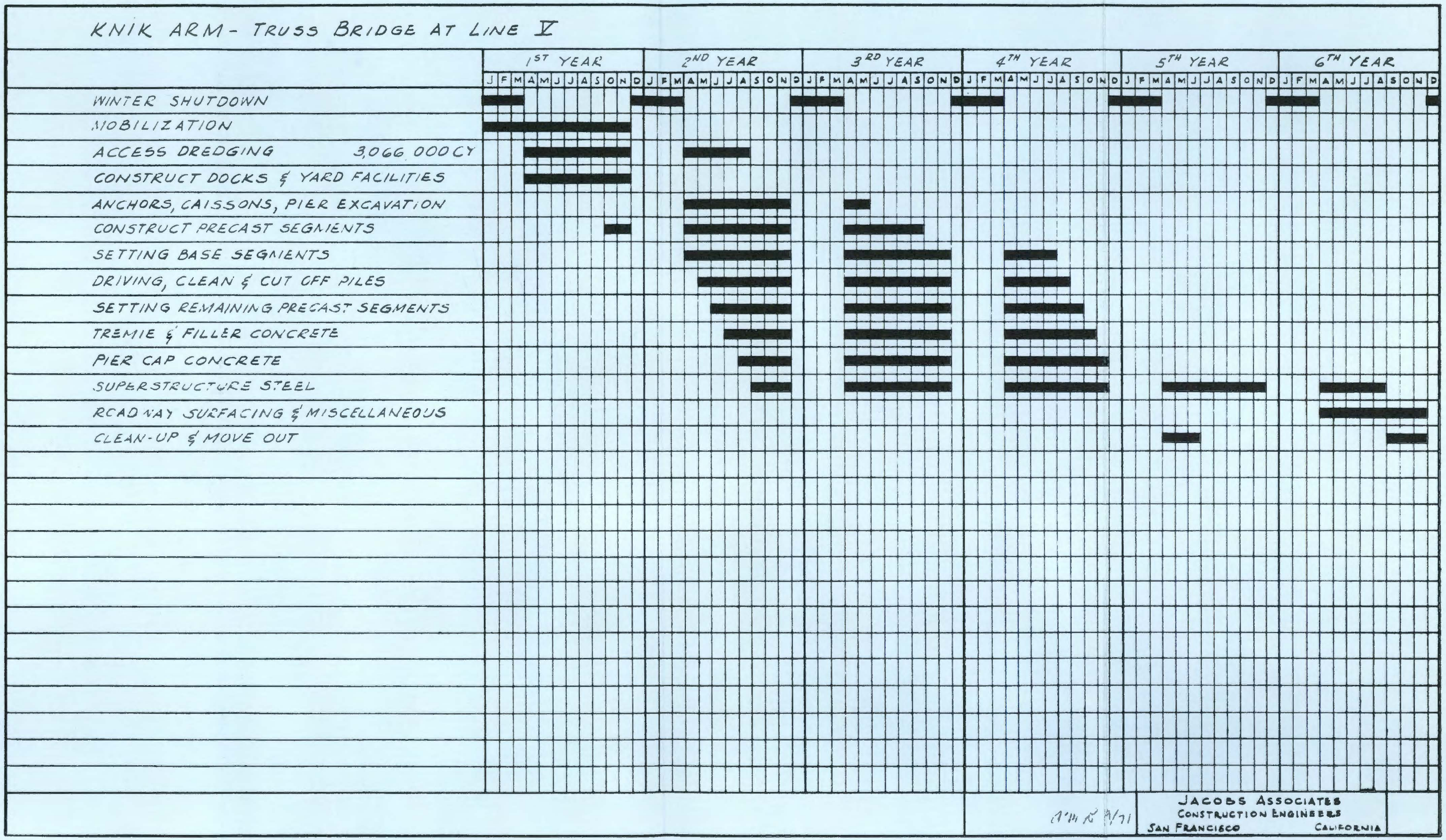
Below Elev. 20 the excavation will be relatively wet. It is assumed to be handled by 6 CY draglines operating off Elev. 20 and digging down to grade. The haul units will be the same as above.

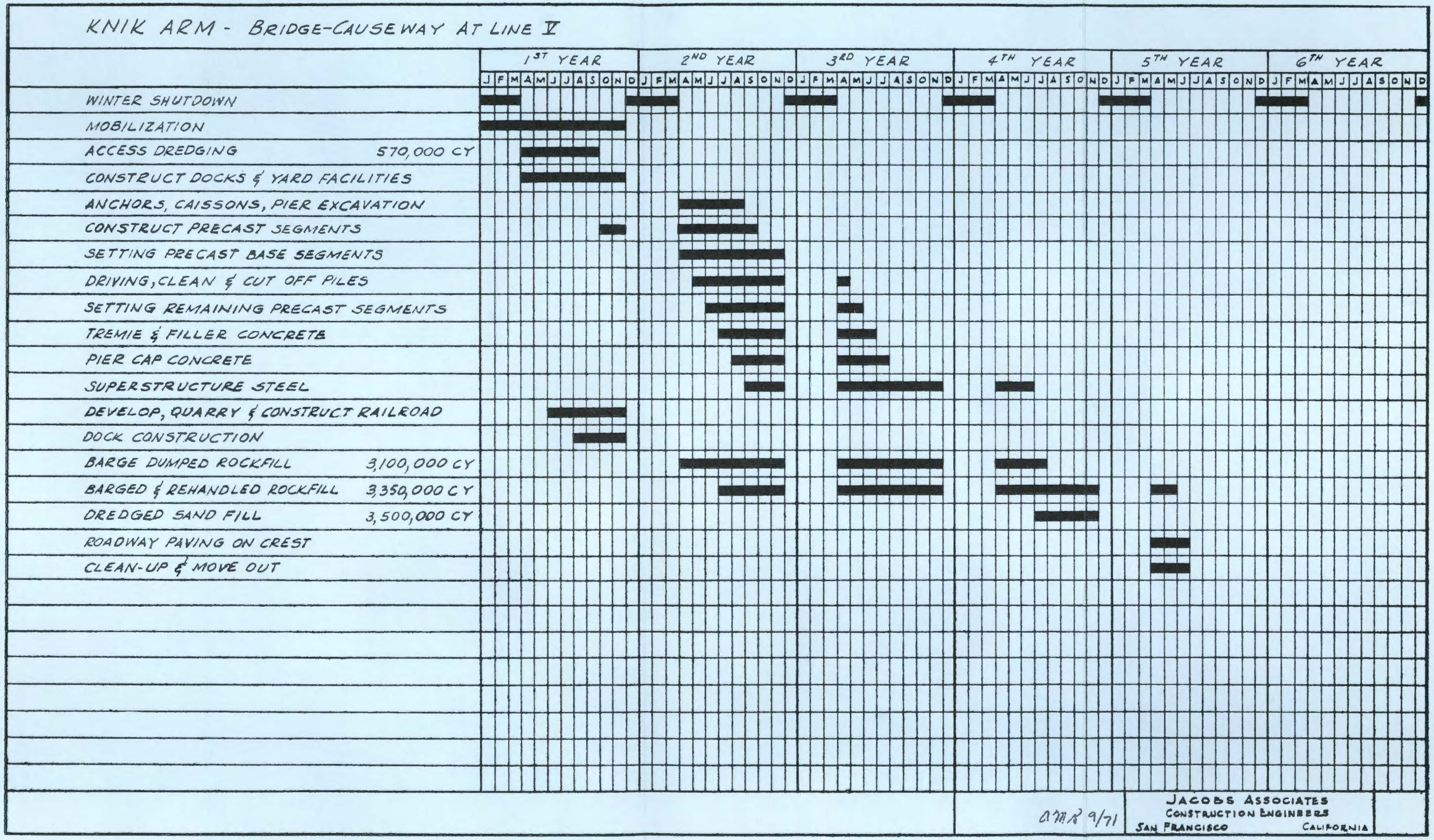
SPILLWAY CONCRETE

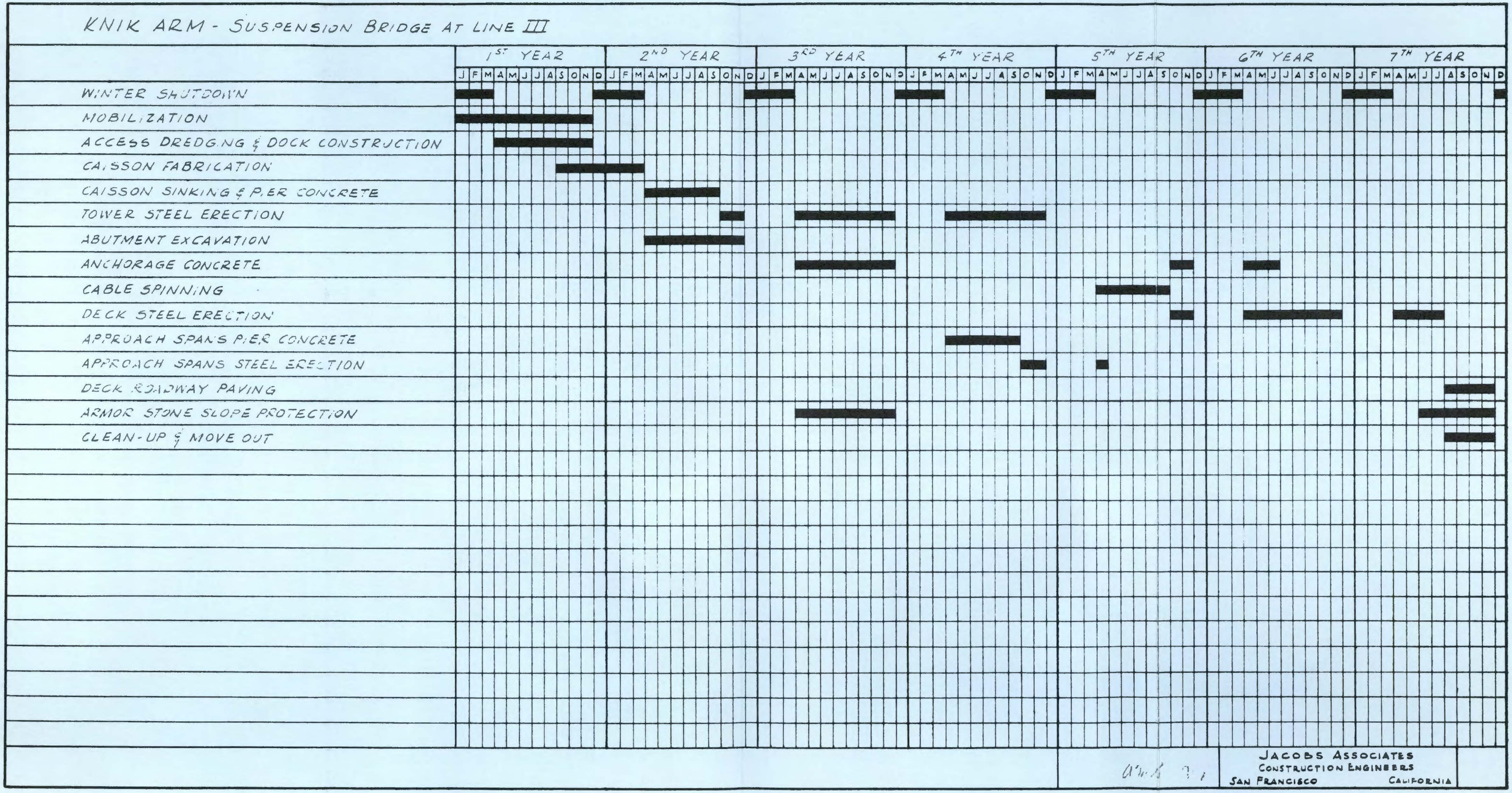
The spillway concrete work is relatively simple compared to other work estimated. As long as the design remains simply an ungated overflow structure with clean lines, the cost will be quite reasonalbe. Addition of any types of gates would add substantially to the costs. While we were not furnished a design of a bridge over the spillway, one will be required and we have made assumptions that we think fit a series of short steel girder spans on simple concrete piers.

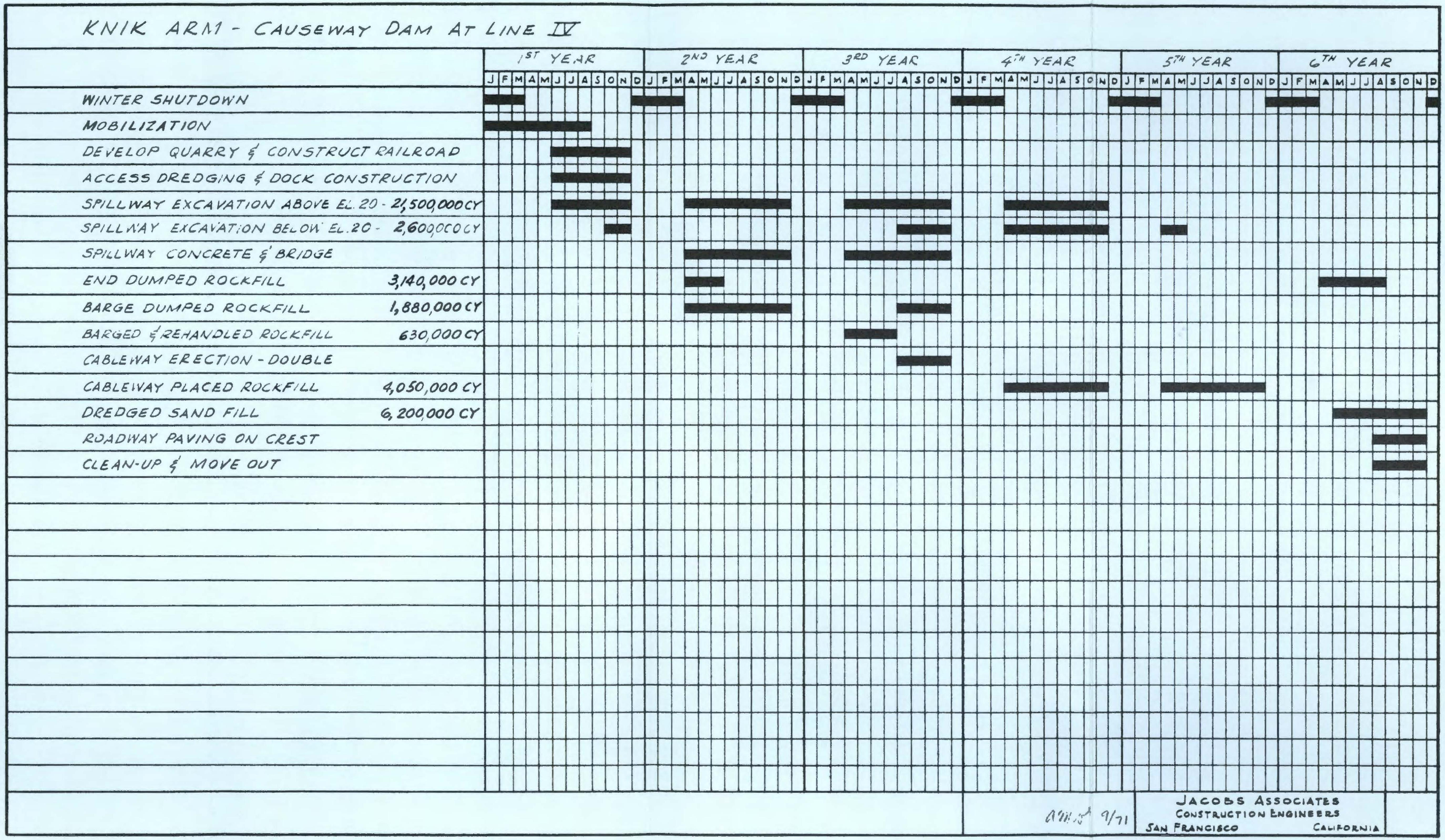

KNIK ARM - TRUSS BRIDGE AT LINE IV
1ST YEAR
2ND YEAR
3RD YEAR
4TH YEAR
J F M A M J J A S O N D
WINTER SHUTDOWN
MOBILIZATION
ACCESS DREDGING - 800,000 CY
CONSTRUCT DOCKS & YARD FACILITIES
ANCHORS, CAISSONS & PIER EXCAVATION
CONSTRUCT PRECAST SEGMENTS
SETTING PRECAST BASE SEGMENTS
DRIVING, CLEAN & CUT OFF PILES
SETTING REMAINING PRECAST SEGMENTS
TREMIE & FILLER CONCRETE
PIER CAP CONCRETE
SUPERSTRUCTURE STEEL
ROADWAY SURFACING & MISCELLANEOUS
CLEAN-UP & MOVE OUT
9/71
JACOBS ASSOCIATES
CONSTRUCTION ENGINEERS
SAN FRANCISCO CALIFORNIA


KNIK ARM - CAUSEWAY DAM AT LINE V
1ST YEAR
2ND YEAR
3RD YEAR
4TH YEAR
5TH YEAR
J F M A M J J A S O N D
WINTER SHUTDOWN
MOBILIZATION
DEVELOP QUARRY & CONSTRUCT RAILROAD
ACCESS DREDGING & DOCK CONSTRUCTION
SPILLWAY EXCAVATION - ABOVE EL. 20.
SPILLWAY EXCAVATION - BELOW EL. 20.
SPILLWAY CONCRETE & BRIDGE
DUMPED ROCKFILL 7,550,000 CY
BARGED ROCKFILL 1,450,000 CY
CABLEWAY ERECTION
CABLEWAY PLACED ROCKFILL 1,300,000 CY
DREDGED SAND FILL 8,500,000 CY
ROADWAY PAVING ON CREST
CLEAN-UP & MOVE OUT
JACOBS ASSOCIATES
CONSTRUCTION ENGINEERS
SAN FRANCISCO CALIFORNIA


KNIK ARM - TRUSS BRIDGE AT LINE V
1ST YEAR
2ND YEAR
3RD YEAR
4TH YEAR
5TH YEAR
6TH YEAR
J F M A M J J A S O N D
WINTER SHUTDOWN
MOBILIZATION
ACCESS DREDGING 3,066,000 CY
CONSTRUCT DOCKS & YARD FACILITIES
ANCHORS, CAISSONS, PIER EXCAVATION
CONSTRUCT PRECAST SEGMENTS
SETTING BASE SEGMENTS
DRIVING, CLEAN & CUT OFF PILES
SETTING REMAINING PRECAST SEGMENTS
TREMIE & FILLER CONCRETE
PIER CAP CONCRETE
SUPERSTRUCTURE STEEL
ROADWAY SURFACING & MISCELLANEOUS
CLEAN-UP & MOVE OUT
JACOBS ASSOCIATES
CONSTRUCTION ENGINEERS
SAN FRANCISCO CALIFORNIA


KNIK ARM - BRIDGE-CAUSEWAY AT LINE V
1ST YEAR
2ND YEAR
3RD YEAR
4TH YEAR
5TH YEAR
6TH YEAR
J F M A M J J A S O N D
WINTER SHUTDOWN
MOBILIZATION
ACCESS DREDGING 570,000 CY
CONSTRUCT DOCKS & YARD FACILITIES
ANCHORS, CAISSONS, PIER EXCAVATION
CONSTRUCT PRECAST SEGMENTS
SETTING PRECAST BASE SEGMENTS
DRIVING, CLEAN & CUT OFF PILES
SETTING REMAINING PRECAST SEGMENTS
TREMIE & FILLER CONCRETE
PIER CAP CONCRETE
SUPERSTRUCTURE STEEL
DEVELOP, QUARRY & CONSTRUCT RAILROAD
DOCK CONSTRUCTION
BARGE DUMPED ROCKFILL 3,100,000 CY
BARGED & REHANDLED ROCKFILL 3,350,000 CY
DREDGED SAND FILL 3,500,000 CY
ROADWAY PAVING ON CREST
CLEAN-UP & MOVE OUT
9/71
JACOBS ASSOCIATES
CONSTRUCTION ENGINEERS
SAN FRANCISCO CALIFORNIA


KNIK ARM - SUSPENSION BRIDGE AT LINE III
1ST YEAR
2ND YEAR
3RD YEAR
4TH YEAR
5TH YEAR
6TH YEAR
7TH YEAR
J F M A M J J A S O N D
WINTER SHUTDOWN
MOBILIZATION
ACCESS DREDGING & DOCK CONSTRUCTION
CAISSON FABRICATION
CAISSON SINKING & PIER CONCRETE
TOWER STEEL ERECTION
ABUTMENT EXCAVATION
ANCHORAGE CONCRETE
CABLE SPINNING
DECK STEEL ERECTION
APPROACH SPANS PIER CONCRETE
APPROACH SPANS STEEL ERECTION
DECK ROADWAY PAVING
ARMOR STONE SLOPE PROTECTION
CLEAN-UP & MOVE OUT
JACOBS ASSOCIATES
CONSTRUCTION ENGINEERS
SAN FRANCISCO
CALIFORNIA


KNIK ARM - CAUSEWAY DAM AT LINE IV
1ST YEAR
2ND YEAR
3RD YEAR
4TH YEAR
5TH YEAR
6TH YEAR
J F M A M J J A S O N D
WINTER SHUTDOWN
MOBILIZATION
DEVELOP QUARRY & CONSTRUCT RAILROAD
ACCESS DREDGING & DOCK CONSTRUCTION
SPILLWAY EXCAVATION ABOVE EL. 20 - 21,500,000 CY
SPILLWAY EXCAVATION BELOW EL. 20 - 2,600,000 CY
SPILLWAY CONCRETE & BRIDGE
END DUMPED ROCKFILL 3,140,000 CY
BARGE DUMPED ROCKFILL 1,880,000 CY
BARGED & REHANDLED ROCKFILL 630,000 CY
CABLEWAY ERECTION - DOUBLE
CABLEWAY PLACED ROCKFILL 4,050,000 CY
DREDGED SAND FILL 6,200,000 CY
ROADWAY PAVING ON CREST
CLEAN-UP & MOVE OUT
AMS 9/71
JACOBS ASSOCIATES
CONSTRUCTION ENGINEERS
SAN FRANCISCO CALIFORNIA

geologic and engineering reconnaissance

APPENDIX E

GEOLOGIC & ENGINEERING RECONNAISSANCE
PROPOSED KNIK ARM CROSSING
ANCHORAGE, ALASKA

FOR

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
KANSAS CITY AND SEATTLE

AUGUST 18, 1971

SHANNON & WILSON, INC.
SOIL MECHANICS AND FOUNDATION ENGINEERS
1105 NORTH 38TH STREET
SEATTLE, WASHINGTON 98103

TABLE OF CONTENTS

LIST OF FIGURES

GEOLOGIC AND ENGINEERING RECONNAISSANCE VICINITY OF THE PROPOSED KNIK ARM CROSSING ANCHORAGE, ALASKA

1. Introduction

This report presents a description of the investigations conducted and conclusions developed during a four-day geologic and engineering reconnaissance of the Knik Arm area conducted in early June 1971. The field inspection was accomplished as a joint effort by Mr. Gay D. Jones of HNTB and Messrs. R. J. Deacon, E. D. Schwantes and W. L. Shannon of Shannon & Wilson, Inc. Because of a general lack of roads into the study area and the severe tidal currents in the Arm, access into the area for this reconnaissance was via a chartered helicopter.

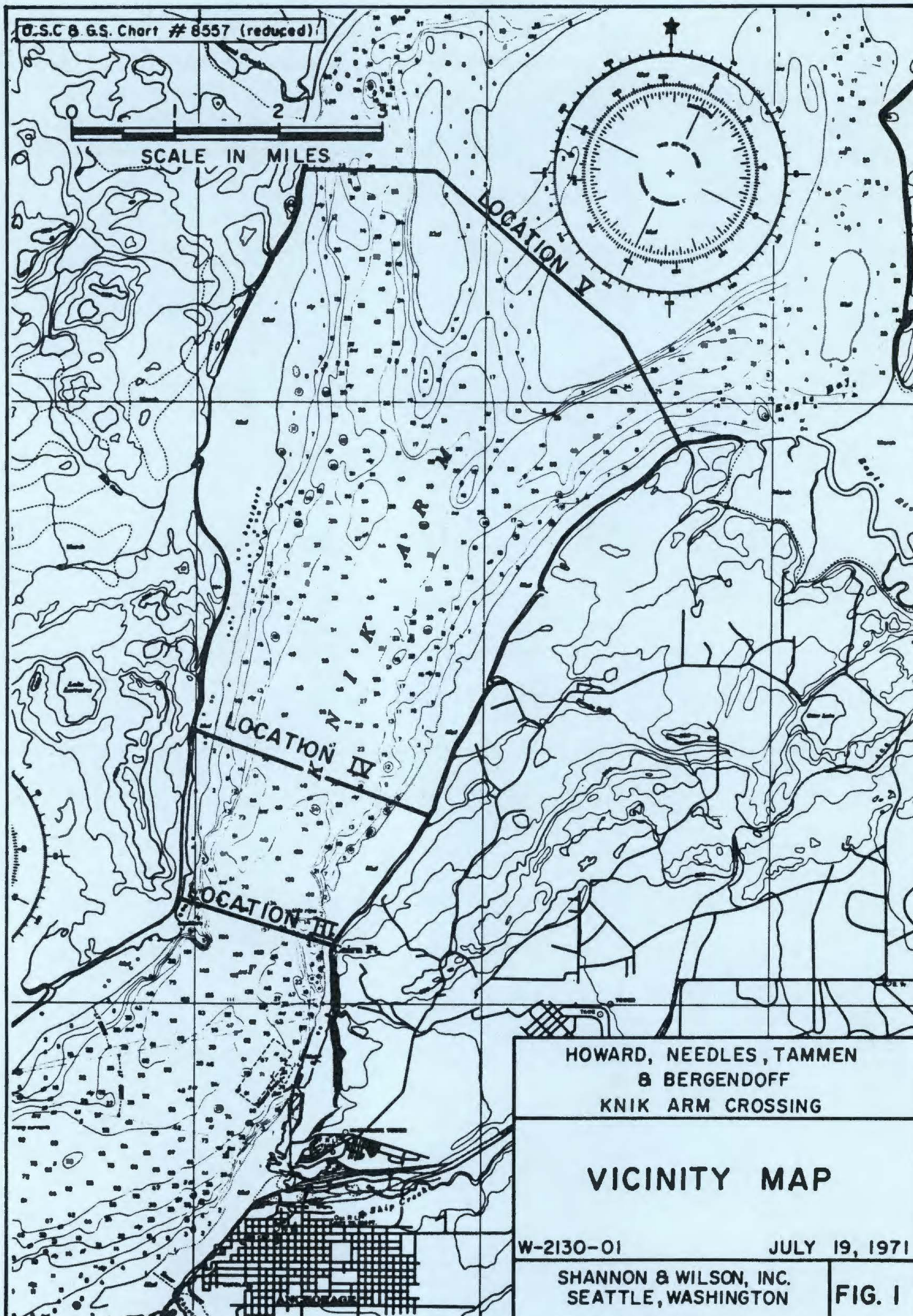
1.1 Purpose. The Knik Arm reconnaissance was conducted to determine the geologic and engineering properties of the soils and shoreline features by means of visual inspection. Also included is a review of published and unpublished data in the vicinity of three alternate routes for the proposed Knik Arm crossing. In addition, a preliminary evaluation of potential sources for construction materials (such as gravel and rip rap) is included.

1.2 Scope. This investigation consisted of a visual reconnaissance of the site, review of published and unpublished data on the geology of the region, and study of available borings and sub-bottom geophysical soundings. The reconnaissance included both a general review of the entire Knik Arm as well as a detailed inspection of each proposed abutment site. Where possible, the detailed inspection included scaling the bluff to permit more complete observation and mapping of the exposed soils.

Because of this project's close proximity to Anchorage, much of the published geologic and foundation engineering data can be projected over the Knik Arm area. To supplement this information, meetings with personnel from the U.S. Geologic Survey (U.S.G.S.), the Corps of Engineers and the Bureau of Land Management (B.L.M.) were held to discuss various aspects of the proposed crossings. A list of the various references used is presented at the end of this report.

Because the proposed alignments are not staked and existing maps are to a relatively small scale (limited detail), elevations and locations described herein should be considered as approximate.

1.3 Project Description. Three alternate routes for the Knik Arm crossing (designated: III, IV and V) are presently being considered as shown in the Vicinity Map, Fig. 1. Because of widely varying conditions (length, depth of water, current velocity, etc.) a different type of crossing



SHANNON AND WILSON, INC.

structure is being considered at each of the three sites. It is our understanding that the type of structure has been narrowed to either a bridge or causeway-dam and that three other sites (designated: I, II and VI) and a tunnel scheme were evaluated previously and discarded as not being feasible. As presently laid out, an 8,585-foot suspension bridge would be used at Location III, a 13,200-foot, double deck, orthotropic truss bridge would be used at Location IV and an 25,500-foot causeway-dam at Location V.

2. Geology

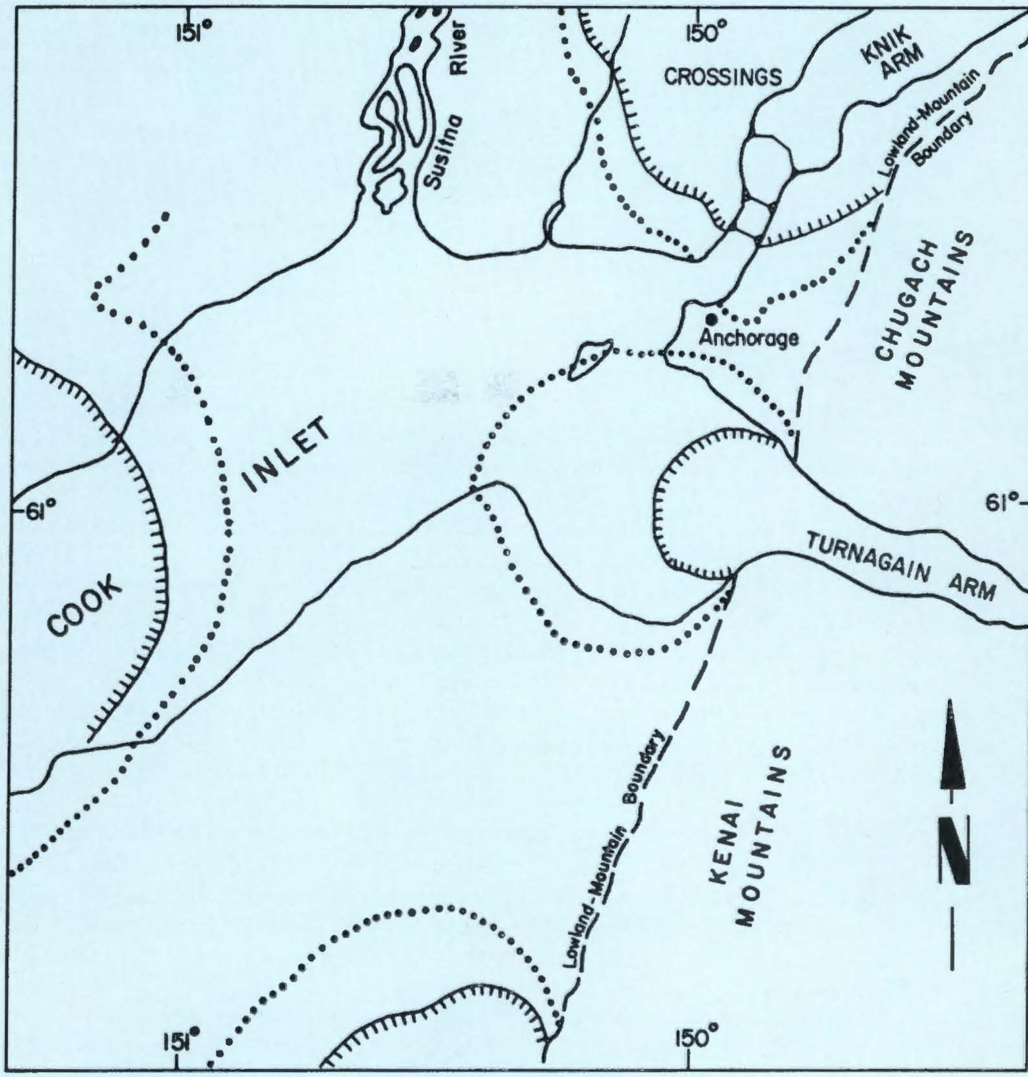
2.1 Regional Geology. The Knik Arm is part of the Cook Inlet Lowland physiographic province, which is a major marine re-entrant along the Pacific Ocean coastline in southcentral Alaska. The Cook Inlet Lowland is a structural trough underlain by rocks of Mesozoic and Tertiary age and mantled largely by Quaternary glacial deposits of variable thickness. In the Knik Arm area the lowlands are bordered to the southeast and east by the Chugach Mountains, to the north by the Talkeetna Mountains and to the west by Mount Susitna. These mountains are underlain by Jurassic and younger metavolcanics and graywacke, which locally are intruded by large masses of granite and by small areas of gabbro and peridotite.

The topography of the lowland is primarily the result of five major Pleistocene glaciations and two post-Pleistocene glacial advances, Karlstrom (1964). The Pleistocene glaciations are from oldest to youngest: Mount Susitna, Caribou Hills (both Kenai Lowland glaciations), Eklutna, Knik and Naptowne. The latter three are the principal glaciations in the Knik Arm area. The approximate limits of ice advance during the Knik and Naptowne glaciations are presented in Fig. 2.

Information from deep water wells indicates that the total thickness of glacial deposits in the Anchorage area is over 468 feet near Fort Richardson and over 778 feet at Elmendorf Air Force Base. Oil test wells indicate that glacial deposits of over 1,000 feet in thickness exist in some areas within the region.

Soil units of the Knik glaciation include the Bootlegger Cove Clay as described by Miller and Dobrovolny (1959) and a lower till which outcrops beneath the Bootlegger formation south of the study area.

Deposits of Naptowne glaciation overlie the Knik deposits and comprise the majority of the soil units exposed in the east and west bluffs of Knik Arm north of Anchorage. In the vicinity of Location III these include widely varying end-moraine deposits consisting of waterlain till mixed with outwash gravels. Farther to the north there are younger ground moraine and recessional outwash deposits.



GLACIATIONS

Naptowne

Knik

0 10 20

APPROXIMATE SCALE IN MILES

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
KNIK ARM CROSSING

SKETCH MAP OF GLACIATIONS

W-2130-01 JULY 19, 1971

SHANNON & WILSON, INC.
SEATTLE, WASHINGTON

FIG. 2

SHANNON AND WILSON, INC.

On the bottom of Knik Arm the glacial deposits locally are overlain by recent deposits of soft silt and fine sand. Extensive sand-gravel deposits overlie the glacial soils at the mouth of the Knik and Matanuska Rivers at the head of the Arm.

In general, the shoreline of Knik Arm north of Anchorage has relatively steep, gravel paved beaches with 100 to 150-foot high bluffs rising above the high water line. In a few areas north of Location V there are wide breaks in the bluffs. At these points, such as Eagle River Flats, the terrain is that of flat, low-lying marsh land. In general, the upland areas immediately inshore from the bluffs exhibit a hummocky, knob and kettle topography. A number of the kettles have filled with water, forming small lakes or ponds.

Though there are no mapped faults within the immediate vicinity of Knik Arm, this is a seismically active area. In addition to the well publicized landslides, the March 27, 1964 earthquake caused widespread tectonic changes in elevation throughout most of southcentral Alaska. In general, post-earthquake surveys indicate that the Knik Arm area subsided slightly over 2 feet.

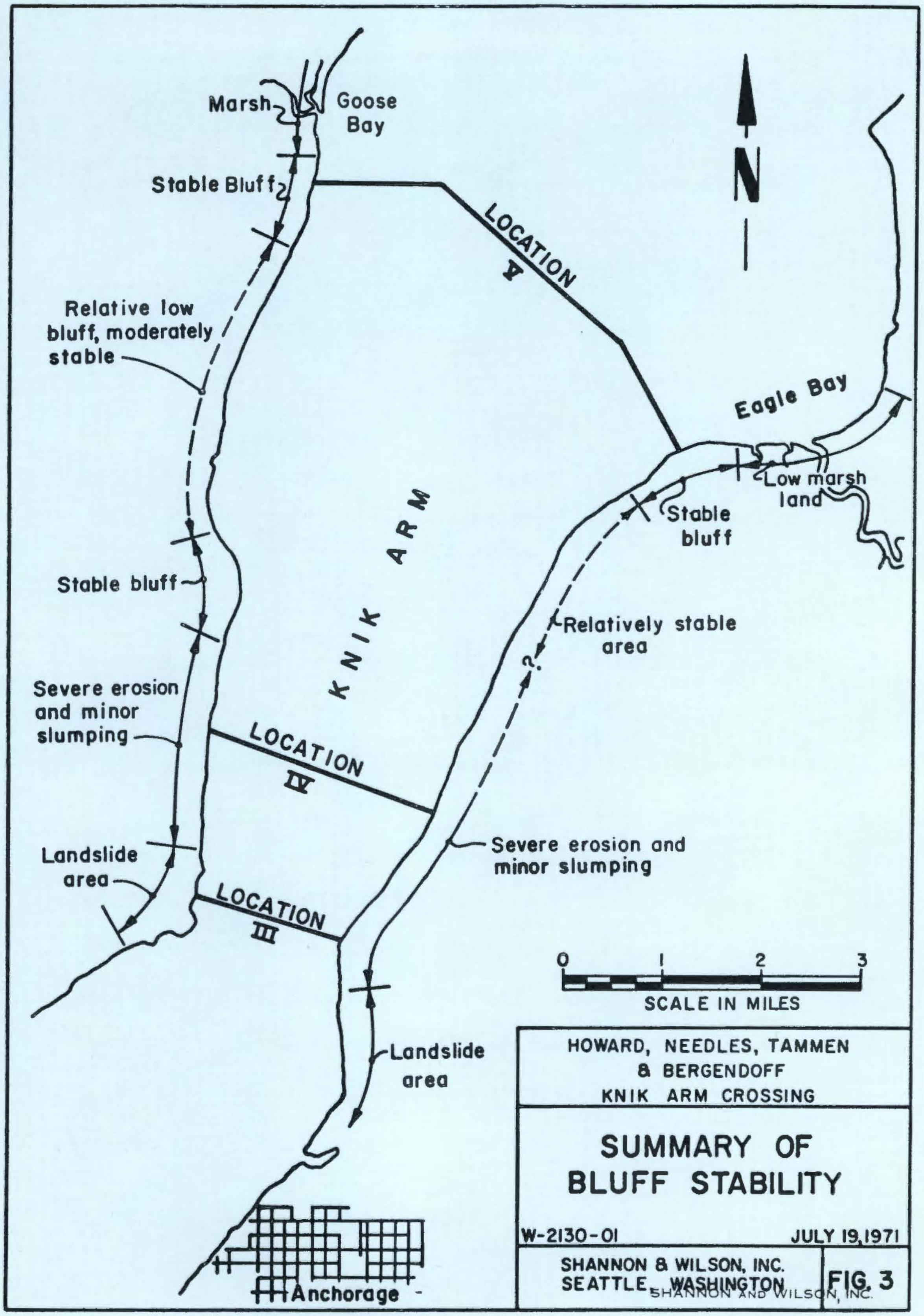
2.2 Site Studies. The following paragraphs summarize the results of our studies of the three selected crossings, designated III, IV and V. Included is our interpretation of the sub-bottom conditions based upon the geophysical soundings made by Dames & Moore for the Alaska State Highway Department and the two borings, one deep and one very shallow, made by the Alaska State Highway Department.

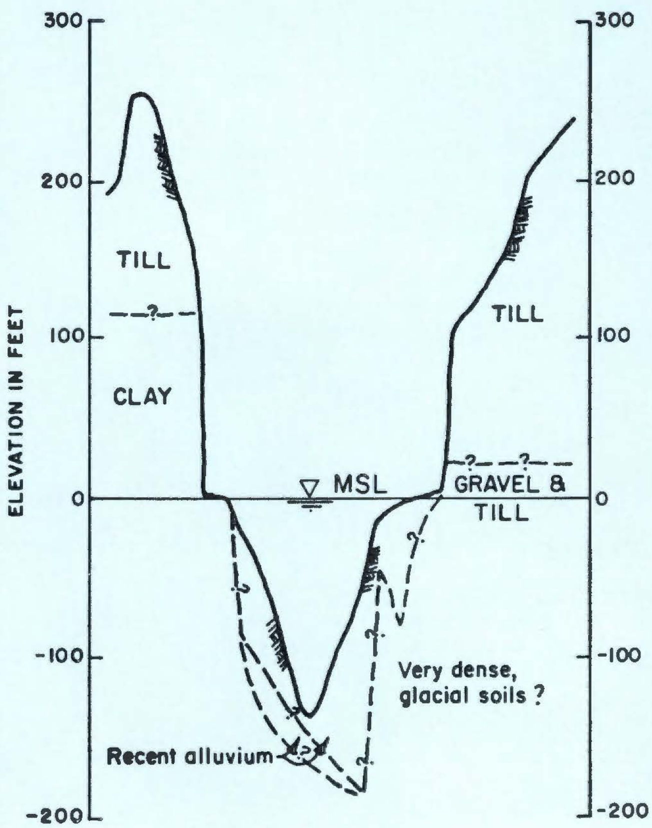
The three proposed crossings are shown in Fig. 3. Included on this figure is a general summary of our observations and evaluation of the stability of the bluffs at each end of each crossing.

2.2.1 Crossing Location III. Location III commences on the east side of Knik Arm about two miles north of Anchorage at Cairn Point and crosses the Arm on a bearing of about N70W reaching the west side of the Arm about two miles north of Point McKenzie.

A profile at Location III is shown in Fig. 4, on which is shown our interpretation of the subsurface conditions. Beneath Knik Arm two separate formations are believed to be present. The upper formation is described as a medium dense sand and is believed to be of recent origin and consists of sediments brought into the Arm by the Knik and Matanuska Rivers. The underlying formation is believed to be of the same origin as the bluffs on either side and consisting of dense, unidentified soils.

East Abutment Area - The bluff on the east side is in excess of 100 feet in height. It is nearly vertical and contains extensive gullying in the upper part with numerous shallow slump "pop outs" on the sides of the gullies. The lower portion of the slope is also scarred by shallow slump features. The soils exposed in the bluff can be divided into two major units: (1) the upper unit consists of over 100 feet of compact, gray-brown, massive pebble and cobble TILL with a matrix of silty and clayey fine sand with lenses and beds of clean fine sand; (2) the lower


Marsh
Goose Bay
Stable Bluff
LOCATION V
N
Relative low bluff, moderately stable
Eagle Bay
Low marsh land
Stable bluff
KNIK ARM
Stable bluff
Relatively stable area
Severe erosion and minor slumping
LOCATION IV
Severe erosion and minor slumping
Landslide area
LOCATION III
0 1 2 3
SCALE IN MILES
Landslide area
Anchorage
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
KNIK ARM CROSSING
SUMMARY OF BLUFF STABILITY
W-2130-01
JULY 19,1971
SHANNON & WILSON, INC.
SEATTLE, WASHINGTON
SHANNON AND WILSON, INC.
FIG. 3



LOCATION III
(Looking North)

NOTES

1. ELEVATIONS SHOWN ARE APPROXIMATE.
2. SUBMARINE PROFILE BASED ON 1970 GEOPHYSICAL SURVEY BY DAMES & MOORE.

0 1 2
HORIZONTAL SCALE IN MILES

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
KNIK ARM CROSSING

TYPICAL SUBSURFACE PROFILE

W-2130-01 JULY 19, 1971

SHANNON & WILSON, INC.
SEATTLE, WASHINGTON

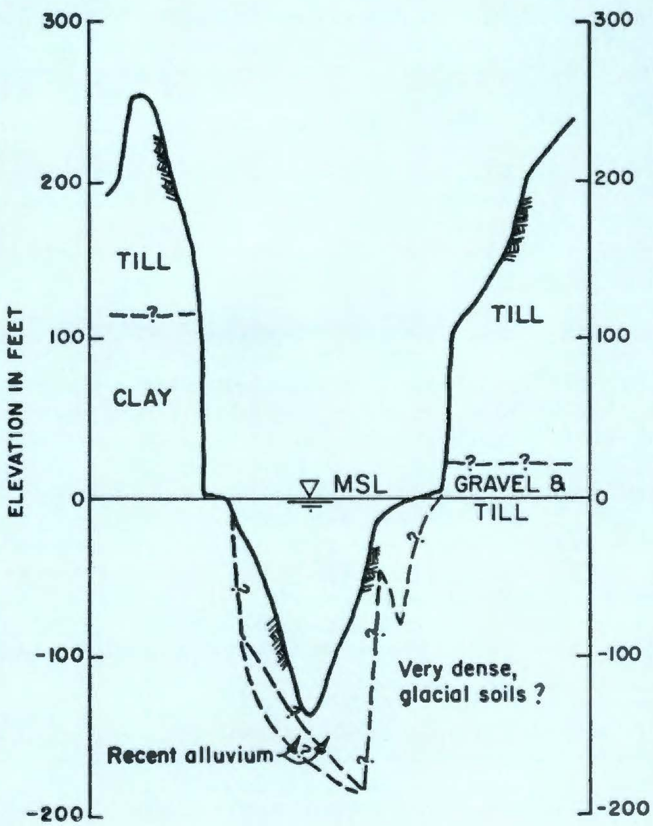
FIG. 4

SHANNON AND WILSON, INC.

unit consists of interbedded layers of weakly cemented gravel and brown to gray TILL. Geologic mapping by Miller and Dobrovolny (1959) indicates the presence of the Bootlegger Cove Clay along the toe of the bluff at Cairn Point; however, no outcrop of Bootlegger was observed at this location during our inspection.

West Abutment Area - The bluff on the west side of the Arm at Location III ranges in height from about 150 to 175 feet. As presently designed, the west approach is located immediately north of an extensive landslide which occurred during the 1964 earthquake. All surface indications are that this massive slide is similar in nature to those which occurred in the City of Anchorage at the same time. Extensive, post-earthquake studies have shown that the Anchorage slides developed as a result of liquefaction of thin sand seams within the Bootlegger Cove Clay. Bordering this recent slide on the north and extending northerly along the bluff for two or three hundred feet is an older slide with several steplike displacement blocks.

The soils in the bluff consist of two principal units: (1) the upper unit, about 35 to 40 feet thick, consists of gray, massive, pebble and cobble TILL with scattered boulders and a fine, silty sand matrix. At the south edge of the bluff, above the recent (1964) landslide, the TILL grades upward and laterally to a brown-gray, cross-bedded and penecontemporaneous slump contorted, pebble gravel with local iron staining. (2) The lower unit exposed at this location is Bootlegger Cove Clay. It consists of dark gray,



LOCATION III
(Looking North)

NOTES

1. ELEVATIONS SHOWN ARE APPROXIMATE.
2. SUBMARINE PROFILE BASED ON 1970 GEOPHYSICAL SURVEY BY DAMES & MOORE.

0 1 2

HORIZONTAL SCALE IN MILES

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
KNIK ARM CROSSING

TYPICAL SUBSURFACE PROFILE

W-2130-01 JULY 19, 1971

SHANNON & WILSON, INC.
SEATTLE, WASHINGTON

FIG. 4

SHANNON AND WILSON, INC.

unit consists of interbedded layers of weakly cemented gravel and brown to gray TILL. Geologic mapping by Miller and Dobrovolny (1959) indicates the presence of the Bootlegger Cove Clay along the toe of the bluff at Cairn Point; however, no outcrop of Bootlegger was observed at this location during our inspection.

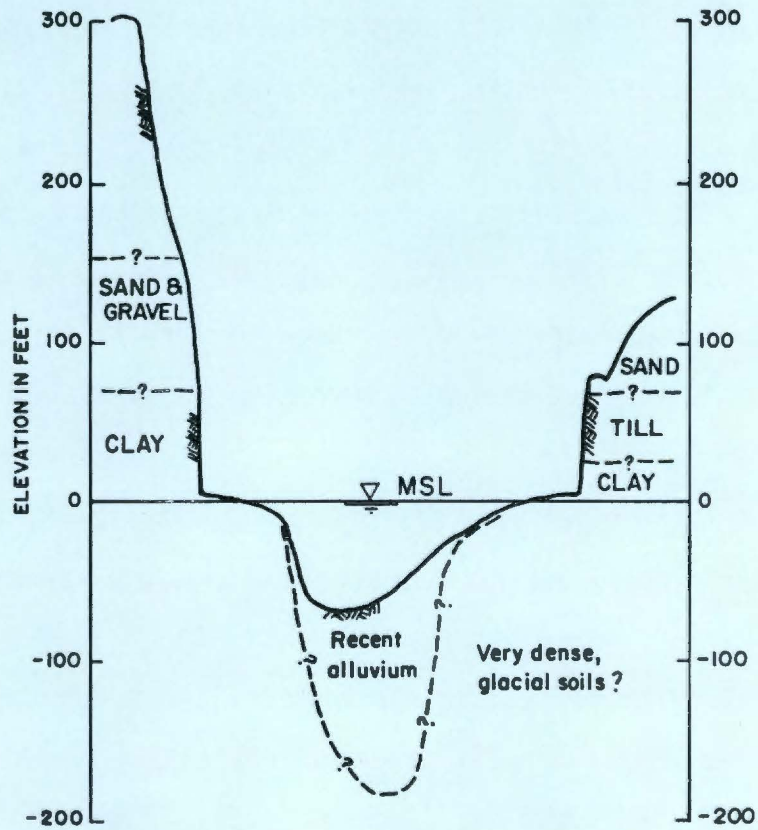
West Abutment Area - The bluff on the west side of the Arm at Location III ranges in height from about 150 to 175 feet. As presently designed, the west approach is located immediately north of an extensive landslide which occurred during the 1964 earthquake. All surface indications are that this massive slide is similar in nature to those which occurred in the City of Anchorage at the same time. Extensive, post-earthquake studies have shown that the Anchorage slides developed as a result of liquefaction of thin sand seams within the Bootlegger Cove Clay. Bordering this recent slide on the north and extending northerly along the bluff for two or three hundred feet is an older slide with several steplike displacement blocks.

The soils in the bluff consist of two principal units: (1) the upper unit, about 35 to 40 feet thick, consists of gray, massive, pebble and cobble TILL with scattered boulders and a fine, silty sand matrix. At the south edge of the bluff, above the recent (1964) landslide, the TILL grades upward and laterally to a brown-gray, cross-bedded and penecontemporaneous slump contorted, pebble gravel with local iron staining. (2) The lower unit exposed at this location is Bootlegger Cove Clay. It consists of dark gray, massive, very stiff to hard CLAY with occasional interbedded thin sheets (2 to 3 feet thick) of pebble till. This lower unit occupies about one-half to two-thirds of the bluff and is estimated to be about 115 feet thick. The apparent hardness of this clay unit as exposed in the face of the bluff is probably the result of desiccation and not representative of the unit as a whole.

Lateral variability of the soil section is indicated in the bluff about 300 feet north of the section discussed above. Here the bluff consists of: (1) a 25-foot deposit of brown, cross-bedded to massive SAND at the top, underlain by, (2) a 35-foot layer of brown-gray, cross-bedded SAND and GRAVEL with water flowing at the base of the unit, and (3) a lower, 75-foot thick unit of Bootlegger Cove Clay. Shallow mudflows are common in the Bootlegger Cove Clay as a result of water seeping from the base of the sand and gravel unit.

2.2.2 Crossing Location IV. Location IV commences on the east side of the Arm near Green Lake about two miles north of Location III. It extends across the Arm on a bearing of about N70W and reaches the bluff on the west side about a mile east of Lake Lorraine.

As at Location III, the subsurface conditions beneath the Arm, as presented in Fig. 5, are based on our interpretation of the 1970 geophysical investigation by Dames & Moore. In general, this indicates



LOCATION IV
(Looking North)

NOTES

1. ELEVATIONS SHOWN ARE APPROXIMATE.
2. SUBMARINE PROFILE BASED ON 1970 GEOPHYSICAL SURVEY BY DAMES & MOORE.

0 1 2
HORIZONTAL SCALE IN MILES

HOWARD, NEEDLES, TAMMEN
& BERGENDOFF
KNIK ARM CROSSING

TYPICAL SUBSURFACE PROFILE

W-2130-01 JULY 19, 1971

SHANNON & WILSON, INC.
SEATTLE, WASHINGTON

FIG. 5

SHANNON AND WILSON, INC.

that medium to dense alluvial deposits are underlain by dense glacial soils. Boring BC-1, drilled close to the center of the Location IV crossing, was too shallow (9') to be of any significant value.

East Abutment Area - Topographic information indicates that the bluff on the east side at Location IV ranges from 60 - 75 feet in height. Soils in the bluff vary rapidly over short horizontal and vertical distances. The section consists of three main units: (1) an upper unit of brown SAND with thin lenses of gravel which overlies both of the older units. This upper unit ranges in thickness from 5 to 10 feet along the top of the bluff. (2) The middle unit consists of very compact, light brown pebble and cobble TILL, which phases laterally to the north into a vertically cross-bedded sand and gravel. This unit ranges in thickness from 40 to 60 feet. (3) The lower unit is a stiff, massive, dark gray, CLAY similar to the Bootlegger Cove Clay to the south. However, it is our understanding (personal communication with W. W. Barnwell) that the U.S.G.S. does not map this unit as Bootlegger, even though it appears to be almost identical.

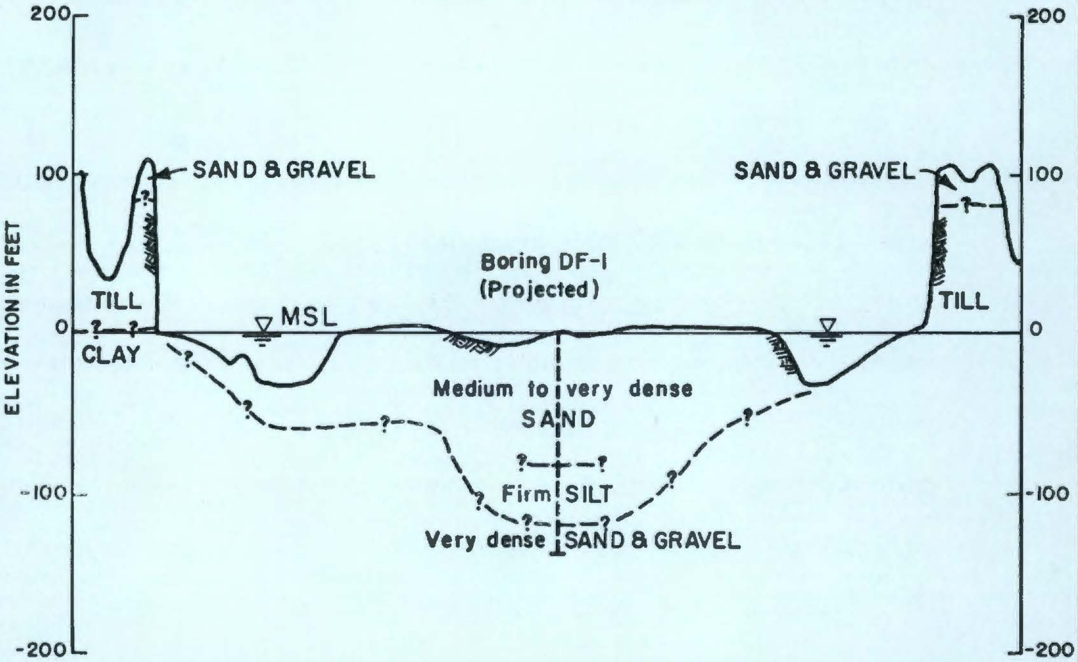
Gullying and sloughing is common in the upper half of the slope at this location. One relatively recent, 50-foot wide, slump block rests on the beach at the toe of the bluff adjacent to the proposed abutment.

West Abutment Area - The bluff on the west side at Location IV ranges from about 125 to 150 feet in height. The bluff section is characterized by numerous shallow slump areas and mudflows. The bluff contains two principal soil units: (1) An upper unit 60 to 70 feet thick consisting of relatively loose, brown SAND and GRAVEL with several 10 to 20-foot thick layers of dense, brown TILL. Water was observed seeping from the base of this unit at numerous localities. (2) The lower unit, comprising about one-half the height of the bluff (50 to 70 feet), consists of very stiff, dark blue-gray CLAY (Bootlegger type) with abundant rounded pebbles. Mudflows are numerous in the lower clay unit as a result of water flowing over the clay from the base of the overlying sand and gravel.

About a mile north of this location, the upper unit in the bluff section grades into a compact, massive, pebble and cobble TILL. Though the Bootlegger type clay is present in the lower third of the bluff, the slope stands nearly vertical with little or no sloughing or mudflows.

2.2.3 Crossing Location V. Location V commences at the bluff just west of Eagle Bay and extends northwesterly across the tideflats in the center of the inlet to a bluff on the west side at the south edge of Goose Bay. The approach on the east side is about 7 miles north of Anchorage.

Our interpretation of the subsurface conditions beneath Knik Arm at Location V are presented in Fig. 6. In addition to the recent Dames & Moore geophysical study, which roughly parallels the proposed crossing,



LOCATION V
(Looking North)

NOTES

1. ELEVATIONS SHOWN ARE APPROXIMATE.
2. SUBMARINE PROFILE BASED ON 1970 GEOPHYSICAL SURVEY BY DAMES & MOORE AND BORING DF-1.

0 1 2
HORIZONTAL SCALE IN MILES

HOWARD, NEEDLES, TAMMEN
& BERGENDOFF
KNIK ARM CROSSING

TYPICAL
SUBSURFACE PROFILE

W-2130-01 JULY 19, 1971

SHANNON & WILSON, INC.
SEATTLE, WASHINGTON

FIG. 6

SHANNON AND WILSON, INC.

the profile is based on a single, 120-foot boring (DF-1) drilled by the Alaska Department of Highways. The layered system of silt and fine sand encountered in the boring between the 62 to 103-foot depth apparently did not appear as a distinctive, separate unit in the geophysical study. As at Locations III and IV, the more recent sediments are underlain by very dense soils, probably of glacial origin.

East Abutment Area - The bluff on the east side of the Arm at Location V ranges from about 75 to 100 feet in height. It is nearly vertical with no significant slumping or sloughing. The soils comprising the bluff consist of two units: (1) an upper unit of brown, cross-bedded SAND and GRAVEL which varies in thickness from 10 to 20 feet, and (2) a lower unit consisting of very compact light brown to tan, pebble TILL.

West Abutment Area - The bluff at the west approach is about 1/2 mile long and rises to heights of 100 to 150 feet; slopes are near vertical and stable. However, it is isolated on the north and south by relatively low marshlands. Soil units exposed in the bluff from the top downward are: (1) an upper 5 to 30-foot thick unit, which is exposed only at the south end of the bluff, consisting of brown, cross-bedded SAND and GRAVEL. Minor sloughing occurs in this unit. (2) A massive pebble and cobble TILL with scattered boulders underlies the upper unit and extends to the base of the bluff. The till is very compact and varies in color from gray at the top to brown at the bottom. Although not exposed along most of this section of the bluff, about 500 feet to the north, exposures of stiff, gray, Bootlegger type clay were observed, which indicates that the massive TILL is probably underlain by clay immediately beneath the beach surface.

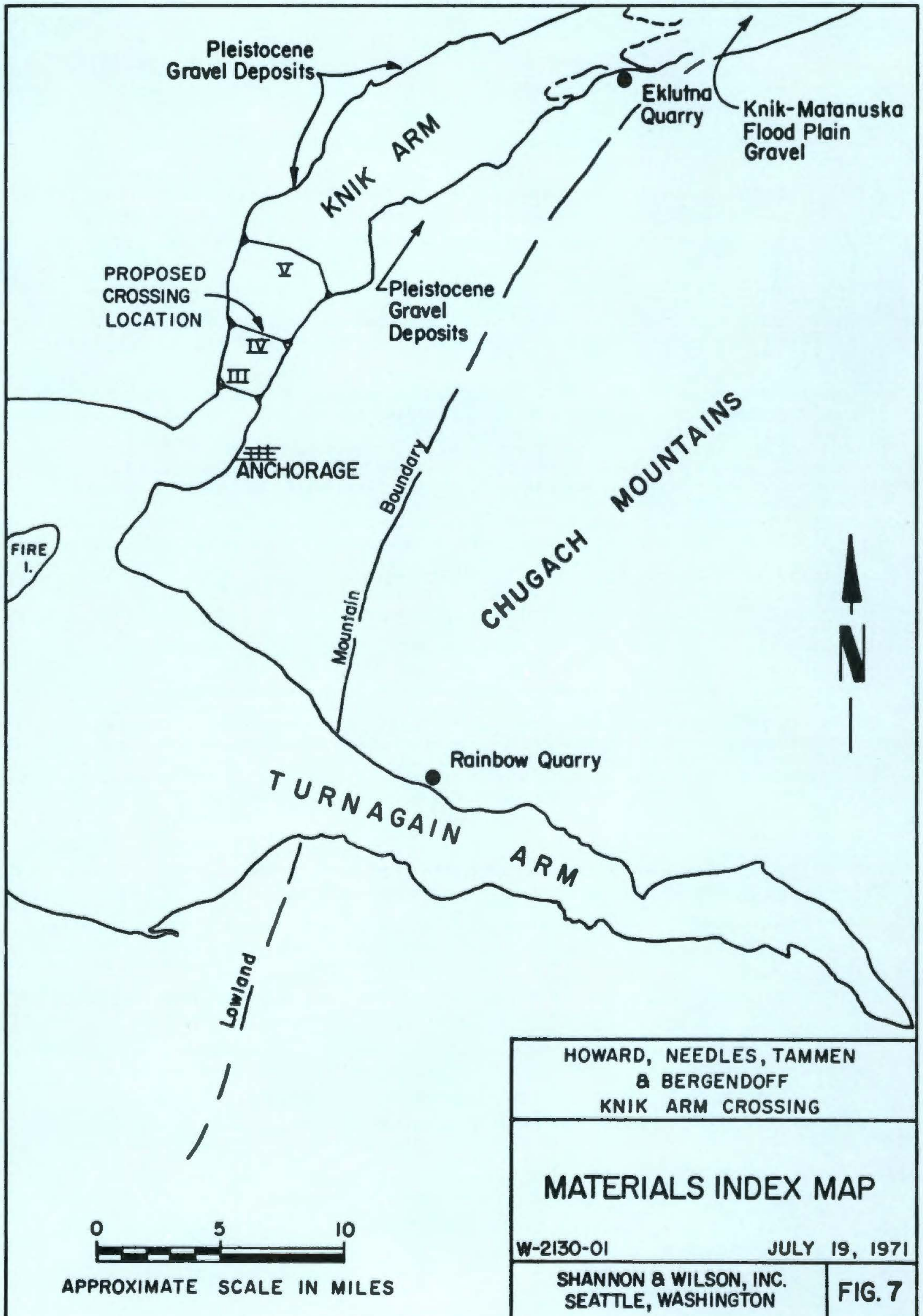
2.3 Present Geologic Activity. As a result of natural geologic processes, the bluffs along Knik Arm are retreating at a significant rate. At each of the abutment sites the bluffs are being eroded by the combined action of waves, ice, tidal variation, flood, wind, frost, gravity, surface water runoff and subsurface seepage. It has not been possible to determine the rate of retreat; however, based on visual observations we are of the opinion that this rate may average roughly 1/2 foot per year and range from place to place within limits of nearly zero to more than one foot per year. The 1964 tectonic subsidence of approximately two feet of the region would be expected to cause an acceleration of bluff erosion and an increase in sedimentation. In addition, immediately south of Location III earthquake related landslides have disturbed extensive areas along both the east and west shores of the Arm.

In recent years the spectacular breakup of Lake George and resulting Knik River flooding has not occurred. However, in the past floods of five or six days duration resulting from this breakup have resulted in maximum peak discharges of 355,000 cfs. Though this probably would have little effect on the proposed bridge structures, it would be a factor in the design of the spillway for a causeway-dam.

3. Material Resources

Preliminary field investigations and office studies were conducted to determine the availability of natural materials suitable for use in embankments and as armor stone for rip-rap. In the field, traverses were made by helicopter of the bluffs bordering both sides of Knik Arm and at the mouth of the Knik and Matanuska Rivers to inspect exposed sand and gravel deposits. The field reconnaissance also included the inspection of several existing stone quarries developed by the Alaska Railroad. The quarries inspected were located near Eklutna northeast of the crossings and near Rainbow on Turnagain Arm southeast of Anchorage as shown in Fig. 7. Published information, notably Miller and Dobrovolny (1959) and Cederstrom, Trainer and Waller (1964) has provided an important source of preliminary information on the distribution of gravel deposits and on rock sources. Sandra Clark, U.S. Geological Survey, Menlo Park, California (presently in the field at Anchorage) has also provided specific information (written communication, June 19, 1971) on the Eklutna Quarry. The 1969 report on the proposed Turnagain Arm Crossing by Armstrong Associates prepared for the State of Alaska Department of Highways provides information on the Alaska Railroad's quarry near Rainbow.

Relatively large sand and gravel deposits are present in the Pleistocene glacial units in the uplands adjacent to the bluffs along Knik Arm and in the modern flood plain of the Knik and Matanuska River. The presence of small sand and gravel pits within the Military Reservation north



SHANNON AND WILSON, INC.

of Elmendorf Air Force Base indicates that sand and gravel underlies an extensive area. These generally are recessional outwash deposits as indicated by Cederstrom, Trainer and Waller (1964). While published information covering the west side of Knik Arm is not yet available, preliminary information indicates that glacial deposits should be similar to those on the east side of the Arm. Helicopter reconnaissance of the west side bluff north of Goose Bay revealed extensive sand and gravel deposits. In the bluff just south of the U.S. Air Force Radar Station (Sec. 16 & 17, T15N, R3W) the deposits consisted of brown to gray, cross-bedded sand and gravel. The gravel portion, approximately 50-percent, consists of about 85-percent pebbles from 1 to 2.5 inches, about 14-percent cobbles ranging from 3 to 8 inches and about 1 percent boulders over 12 inches. A 10 to 20-foot thick till layer caps the bluff at this location.

Modern flood plain gravels are found near the mouth of the Knik and Matanuska Rivers at the head of the Arm. These deposits appear in braided flood plains which extend upstream several miles above the Glenn Highway crossing and downstream in isolated bars for about 5 miles, as indicated in Fig. 7. Inspection of these flood plain gravels indicates that they are predominantly sandy pebble gravels. The gravels range in size from 1 to 3 inches with a small percentage of cobbles to 5 inches.

Preliminary information indicates that most of the gravels would be suitable for use in embankment construction or for concrete with proper processing.

Two sources of rock for embankment construction and rip-rap were inspected during our field investigation. These were the Alaska Railroad quarries near Eklutna on Knik Arm and near Rainbow on Turnagain Arm.

A brief inspection of the southeast face of the quarry at Eklutna, 12 miles northeast of the Location V crossing, indicated that the rock is a hard, greenish-gray metavolcanic. Joints and fractures are close, ranging from a few inches to about 1.5 to 2 feet. The rock appears to be of suitable quality for small size stone for use in a rock embankment, but not sufficiently massive to produce armor stone. A recent letter from Sandra Clark, U.S.G.S. geologist, indicates that massive quartz diorite is also present at this locality and that it might produce suitable large stone. The diorite was not observed during our inspection. It is considered possible that a new quarry in the Chugach Range east of Eklutna could be developed for rock of all sizes.

The Alaska Railroad quarry near Rainbow is about 50 barge miles south down Knik Arm and easterly up Turnagain Arm from the Location V crossing. The rock generally consists of hard, massive to close jointed, dark green to gray metavolcanics. Joints and fractures vary from a few inches to several feet. U.S. Geological Survey information indicates that the specific gravity of the rock ranges from 2.76 to 2.80, which results in a weight of 172 to 175 pcf. Armstrong Associates (1969) have presented information on the development and operation of this quarry. The railroad's requirement for a well-graded rock mixture for use in embankments specified that 70 percent of the material was to be between 2-1/2 cubic feet and 1 cubic yard in size with not more than 10 percent less than 6 inches in diameter. The quarry reportedly produced about 400,000 cubic yards of rock in these size ranges with about 20 percent fine materials. No information is available on the percentage of large stone that was produced. Judging from the jointing exposed in the quarry, it is estimated that 10 percent of the stone could be produced in 15 to 20 ton blocks.

From our inspections and studies we conclude that large quantities of sand and gravel for embankment construction or for concrete aggregate are available within 10 miles from the crossings. Rock is also available for embankment construction from nearby quarries, though an established source for large quantities of armor stone is presently not available. However, the Alaska Railroad's quarry near Rainbow may prove to be the best source even though it apparently contains a relatively small percentage of large stone. For estimating purposes we are of the opinion that suitable rock in adequate quantity for a rock embankment can be located within 50 miles barge haul or within 15 to 35 miles rail or truck haul from Location V.

4. Engineering Considerations

4.1 General

The geologic history and present activity of the Knik Arm region combine to pose significant problems that must be considered in the design of structures under such circumstances. Earthquakes, waves, ice, tidal variation, wind, frost, gravity and both surface and subsurface water will influence the design of foundations, cut slopes, embankments and structures along the Arm.

Each of the three proposed alignments will terminate in bluffs composed of glacial soils varying from compact till to stiff clay to sand and gravel. It is unlikely that bedrock or permafrost will be encountered at any of the sites. The following paragraphs more specifically relate the engineering problems of each site with the structure being considered.

4.2 Site Evaluation

4.2.1 Crossing Location III. Because of its close proximity to Anchorage and the relatively short span (8,500 feet), Location III is a logical choice for some type of crossing scheme. However, deep water, strong currents, sea ice conditions and other considerations have apparently ruled out all structural schemes, except the suspension bridge presently being evaluated.

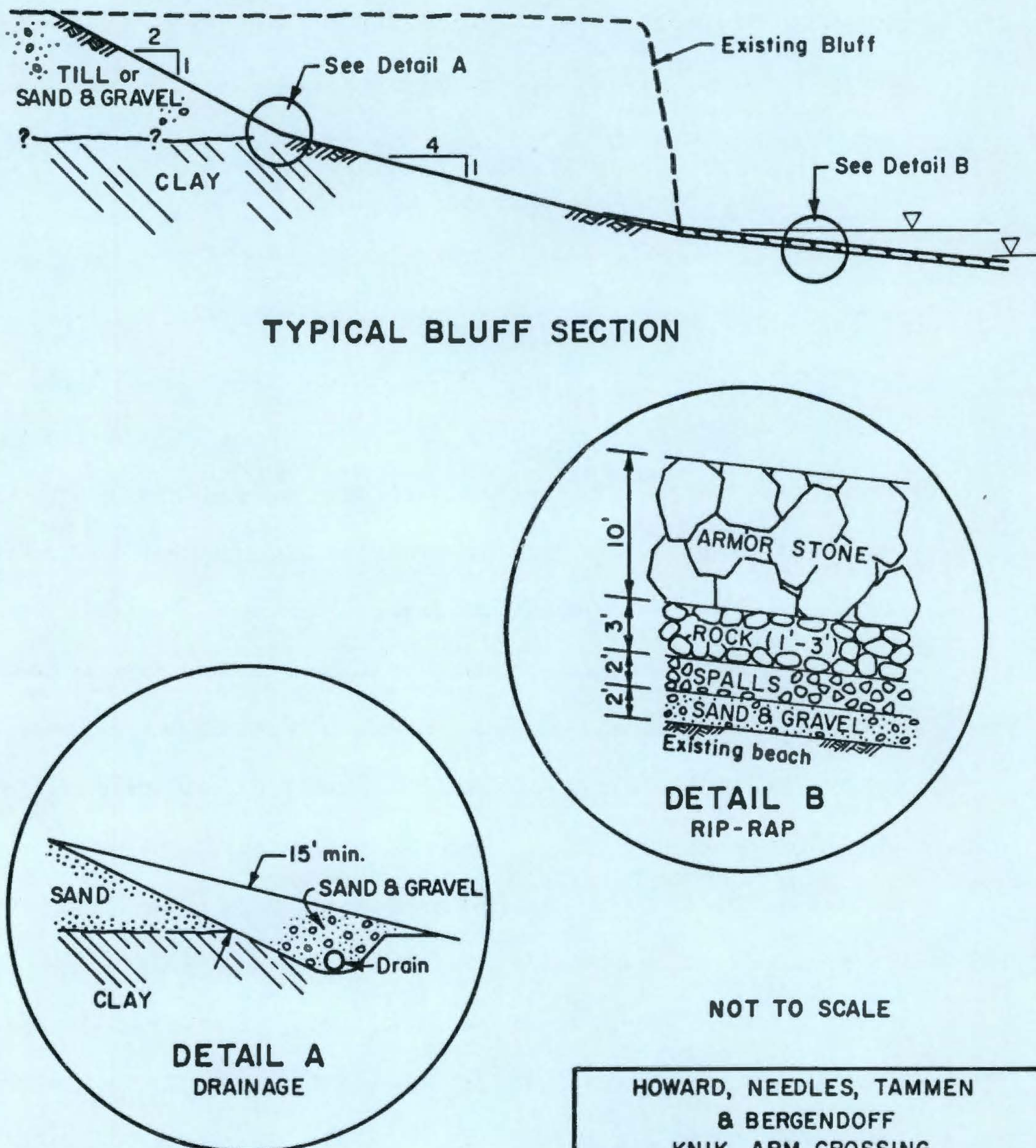
The west abutment area at Location III is characterized by landslide topography. As noted previously, this represents both recent (1964) earthquake induced slides and older slumps, which resulted either from erosion at the toe of the bluff or by an earlier earthquake.

It is recommended that the west abutment cable anchorage be founded in an open excavation below the bottom of the Bootlegger Cove Clay. It is probable that water bearing granular soils underlie this clay, hence dewatering should be included. For estimating purposes and lacking more definitive data the bottom of the clay may be assumed as not higher than elevation -10 MSL and possibly much deeper. In our opinion, anchorages are probably feasible only if the Bootlegger Cove Clay bottoms at -10 or above. Excavation slopes of 1V on 4H are recommended with granular free draining backfill around the anchorage. With this configuration, the cable anchorage should be designed to not only retain the cable tension but also act as a buttress for the slope to the west in the event of an earthquake similar to that which occurred in 1964. It is anticipated that the combined dynamic forces imposed by the resloped bluff and bridge cables could be very sizeable.

Though there was no significant lateral earth movement during the 1964 earthquake at Cairn Point (east abutment Location III) and Bootlegger Cove Clay was not observed during our inspection of the site, we believe it reasonable to assume that the anchorage conditions may be only slightly better than those at the west abutment. The severe gullying and minor sloughing indicate that surface runoff and subsurface seepage will combine to create maintenance problems in cut slopes. However, this potential problem could be effectively controlled during construction by slope grading and installing surface drainage facilities combined with subsurface drains and graded filters as shown in Fig. 8. The cable anchorage for the east abutment should be similar to that for the west abutment.

To resist erosion at the toe of both the east and west bluffs, shore protection works will be required. These should extend for several hundred feet both north and south from the bridge and from above the maximum high water line to at least 20 feet below the low water line. Since this shore protection will be subjected to the action of ice as well as waves and tidal currents, consideration should be given to using massive (15 ton $\pm$) units of armor stone or precast concrete shapes as shown in Fig. 8.

It is recommended that the two main piers for the suspension towers be founded upon either cellular box caissons sunk by excavating in the wet from within the cells or upon deep piers constructed within steel sheet pile cofferdams. The use of sand islands probably will not be feasible because of the high tidal currents which probably would cause severe erosion and maintenance problems. The cellular box caissons should be sunk to a depth of at least 70 feet below channel bottom. The deep piers may be founded at 30 foot depth on steel H piles driven and jetted to a depth of approximately 100 feet below channel bottom. Dewatering of the cofferdammed excavation should follow the pouring of a



HOWARD, NEEDLES, TAMMEN
& BERGENDOFF
KNIK ARM CROSSING

ABUTMENT TREATMENT
LOCATIONS III & IV

W-2130-01 JULY 19, 1971

SHANNON & WILSON, INC.
SEATTLE, WASHINGTON

FIG. 8

SHANNON AND WILSON, INC.

thick tremie seal. An allowable bearing not in excess of 15 tons per sq. ft. may be assumed for the cellular caissons and deep piers, and 100 to 200 tons per steel H pile depending upon pile dimensions, or in excess of 500 tons with specially designed pipe sections and driving techniques.

4.2.2 Crossing Location IV. The east and west abutment areas are both characterized by severe gullying and minor slumping caused primarily by wave and ice erosion at the toe of the bluff, subsurface seepage from the base of the sandy units exposed in the bluff and surface runoff. Though the clay exposed along the toe of the bluff appears to be Bootlegger type, there was no visual evidence of major, earthquake related landslide activity, such as found at Location III and within the City of Anchorage. It is possible that the apparently stronger clay at Location IV is the result of glacial loading during the last ice advance, which did not extend over the City. This glacial preloading presumably consolidated the clays and somewhat densified any interbedded sand seams, such that the shear strength of these soils is increased and the susceptibility to liquefaction decreased.

Based on the present bridge configuration, we believe there will be no significant problems in designing either footing or pile type foundations for the east and west approaches and abutments. However, a considerable grading, drainage, and shore protection will be necessary to dress up and locally stabilize the degrading bluffs.

The 25 offshore bridge piers will be difficult to construct because of the persistent strong tidal currents, and during the winter, cold temperatures and sea ice. Possible boulders in the subsoil could make the installation of sheet piling for cofferdams impractical and difficult to maintain particularly under winter sea ice conditions. As at Location III, the use of sand islands may not be practical. Strong currents, water temperature and poor visibility also will greatly limit the use of divers to accomplish any underwater work.

It is recommended the foundations for the piers consist of one of the following:

a) Cellular box caissons founded at a depth of approximately 50 to 100 feet or more below the bottom in the medium dense granular soils.

b) Piers founded within the same depth inside cofferdammed excavations using a thick tremie seal followed by dewatering.

c) Piers founded at a depth of 30 feet, within cofferdammed excavation, on steel H piles driven to refusal at a penetration of about 50 feet or more. Tremie seal cast then cofferdam dewatered.

d) Piers founded in dredged excavation on piles as above except that all construction carried out without dewatering utilizing techniques developed for offshore platforms by the oil industry.

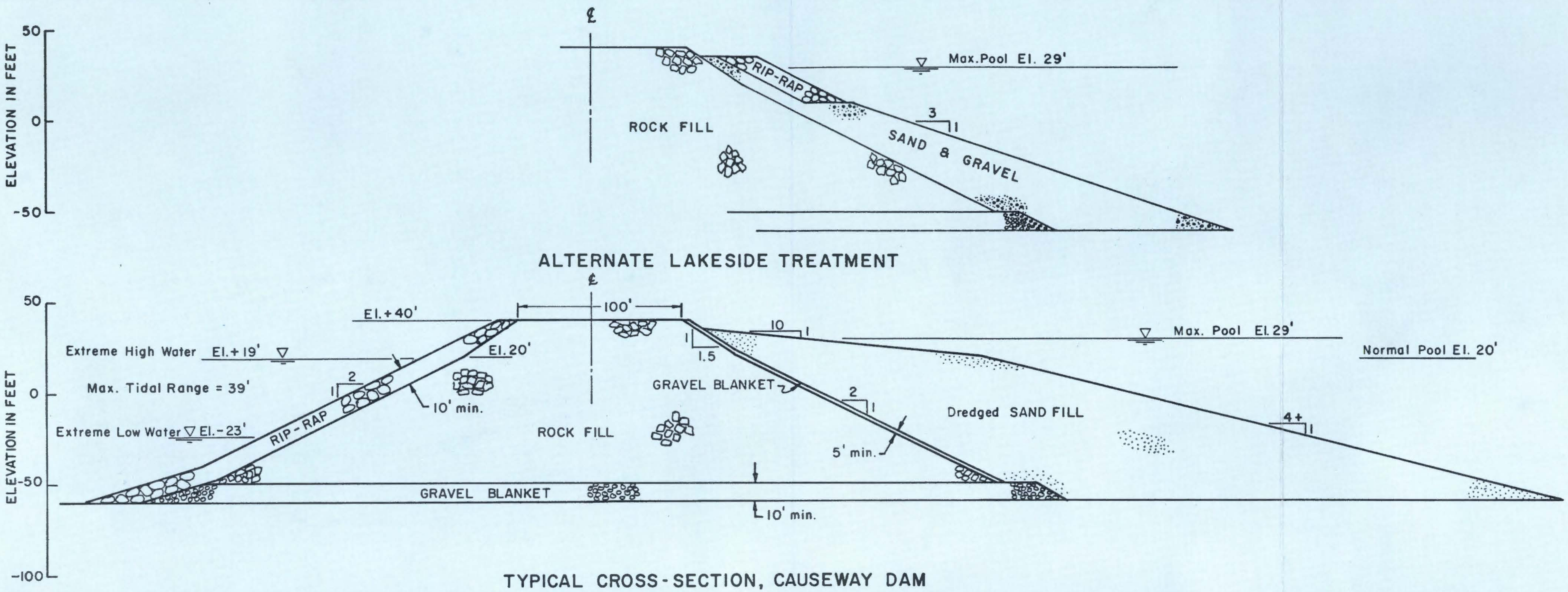
For cost estimating it is recommended that an allowable bearing pressure not in excess of 15 tons per sq. ft. be used for the cellular box caissons and the piers. Steel H piles may be designed for 100 to 200 tons per pile depending upon pile size, or in excess of 500 tons with specially designed pipe sections and driving techniques.

4.2.3 Crossing Location V. As presently designed, the Location V crossing would be made using a 25,500-foot long causeway-dam. Such a structure would create a large, constant level, fresh water lake in the upper end of the Arm. The lake level would be controlled by a spillway to be located in the east abutment area.

With respect to stability, the existing bluffs at the east and west abutments are as good as any in Knik Arm. Since these bluffs consist mainly of till, cut slopes could be relatively steep while abutment problems for the dam should be minimal. The till should prove to be an excellent foundation material for the spillway.

Fig. 9 presents our preliminary recommendation for the causeway-dam cross section. This configuration, in our opinion, should have adequate stability against failure assuming the foundation conditions prove to be reasonably good with the exception that some subsidence due to consolidation should be anticipated in a major earthquake. The essential features and the rationale for the section are described in the following paragraphs.

A rock fill is selected over a sand and gravel fill primarily to achieve acceptable seismic stability. Also important in our opinion is the probable necessity to quarry large volumes of rock to obtain the needed rip-rap. And finally, it will be necessary to close two channels with water velocities which probably will be much greater than those presently occurring. These closures may be possible only with a rock fill. It is noted that the closure plan will probably require a hydraulic model study to determine sequence and particle size required for closure. The rock fill would be placed on the specially prepared gravel blanket, which acts as a filter, by bottom dump barges up to approximately elevation 0 MSL or possibly by end dumping from trucks. Above that elevation the rock would be placed in layers and compacted by the hauling and spreading equipment. It is possible that a substantial part of this upper portion of the fill could consist of clean, well graded, compacted sand and gravel. Upon completion of the rock fill, rip-rap would be placed from barges on the seaward face and a gravel blanket by skips on the lakeside face. Against the gravel blanket, fine sand from bars north of the dam would be placed by dredged fill method. Sufficient sand would be placed to develop a flat stable beach within the range of lake fluctuation and wave action. Though a major earthquake could cause subsidence and failure of the upstream sand fill, such a failure would not be



TYPICAL CROSS-SECTION, CAUSEWAY DAM
LOCATION V

NOTES:

1. WATER LEVELS AND OTHER CONTROLLING ELEVATIONS FURNISHED BY HNTB.

2. CONFIGURATION OF DOWNSTREAM TOE WILL VARY DEPENDING ON SEEPAGE CONDITIONS, BOTTOM ELEVATION AND FUTURE STABILITY STUDIES.

Revised Sept. 3, 1971

0 50 100 150
SCALE IN FEET

HOWARD, NEEDLES, TAMMEN & BERGENDOFF KNIK ARM CROSSING	
PRELIMINARY DAM SECTION	
W-2130-01	JULY 19, 1971
SHANNON & WILSON, INC. SEATTLE, WASHINGTON	FIG. 9

expected to result in a failure of the dam or roadway. An alternate lakeside treatment is shown in the insert on Fig. 9. It is expected that sedimentation of silt and fine sand on the upstream face would rapidly reduce the seepage through the structure to a small amount.

4.3 Seismic Design Considerations

Knik Arm is in a geographic area of high and frequent seismic activity with the 1964 Alaska Earthquake (Richter magnitude 8.4 to 8.6) being the most recent major earthquake. This earthquake caused not only extensive damage to structures but also large landslides, including submarine slides, and land subsidence.

Landslides occurred at Anchorage related to the Bootlegger Cove clay formation. This clay is recognized as being present at Crossing Location III and recommendations presented in Section 4.2.1 for stabilizing the slopes in the vicinity of the structure utilizing buttressing concepts developed as a result of studies following the 1964 earthquake. Slope flattening of other slopes is considered necessary for both static stability and seismic considerations but the latter is not expected to be significant. "Bootlegger-like" clays were also mapped at several spots north of Location III.

Since bottom topography is gentle and bottom sediments, based on the single deep boring are believed to be dense or hard, the probabilities of submarine landslides are considered to be remote, with the exception of any loosely placed fills or embankments. Similarly, land subsidence as a result of densification during an earthquake of bottom sediments is considered to be minor.

Since tectonic displacements of large magnitude have occurred in this locale, the possibility of future tectonic displacements should be considered as well as the effects of the tectonic displacement which occurred in the 1964 earthquake.

The design of piles and caissons should include seismic considerations. In dense or hard soils, such as are believed to exist at all locations, foundations can be expected to perform satisfactorily if designed by accepted practices for design loads which include a seismic component. For preliminary design and cost estimating, reasonable assumptions can be made based on generally accepted Zone III criteria, though for final design the seismic design criteria should be based on the results of a detailed seismic response study.

E. DOUGLAS SCHWANTES, JR.
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

SHANNON & WILSON, INC.

by E. D. Schwantes Jr.
E. D. Schwantes, P. E.

WILLIAM L. SHANNON
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

by W. L. Shannon
W. L. Shannon, P. E.

PARTIAL LIST OF REFERENCES

Alaska Dept. of Hwys, Dec. 1970, Engineering Geology and Foundation Report (Knik Arm), Engineering Geology Sect., Materials Div.

Armstrong and Associates Engineers and Consultants, Jan. 1969, "Causeway Studies, Turnagain Arm Crossing Alaska" for State of Alaska, Dept. of Highways

Cederstrom, Trainer & Waller, 1964, Geology & Ground Water Resources of the Anchorage Area, Alaska, USGS Water Supply Paper 1733.

Dames & Moore, Dec. 1970, Report of Seismic Reflection Survey, Proposed Knik Arm Highway Crossing for State of Alaska, Dept. of Highways.

Karlstrom, 1964, Quaternary Geology of the Kenai Lowland and Glacial History of Cook Inlet Region, Alaska, USGS Professional Paper 443.

Miller & Dobrovolny, 1959, Surficial Geol. of Anchorage & Vicinity, Alaska, USGS Survey Bull. 1093.

Plafker, 1969, Tectonics of the March 27, 1964 Alaska Earthquake, USGS Prof. Paper 543-1.

Porter, O'Brien & Armstrong and Tryck, Nyman & Associates, Jan. 1964, Feasibility Study, Turnagain Arm Crossing for State of Alaska, Dept. of Highways.

RBP

RALPH B. PECK CIVIL ENGINEER : GEOTECHNICS

September 7, 1971
J867

Howard, Needles, Tammen & Bergendoff
Consulting Engineers
1805 Grand Avenue
Kansas City, Missouri 64108

SUBJECT: Knik Arm Crossing Studies

Gentlemen:

This letter contains comments concerning the influence of soil conditions on the proposed crossings of the Knik Arm at Lines III, IV, and V. Information available from various sources up to 1 September, 1971 has been given consideration.

Line III. It is my understanding that preliminary studies have indicated the cost of a single-span suspension bridge at this location to be considerably in excess of alternatives at Lines IV and V, and that such a bridge is no longer considered a likely alternative.

Even if the economic comparison were more favorable, technical considerations with respect to the stability of the abutments might increase the uncertainty of performance of the structures to an undesirable extent. A long-span suspension bridge should not be considered suitable for this site until the elevation of the bottom of the Bootlegger Cove Clay is definitely established at both anchorages. The anchorages, as pointed out by Shannon and Wilson, should be established below this formation. Otherwise, they would be located above a horizontally layered formation of clay containing at least some seams or zones weaker than the average. Under the steady horizontal force associated with the cable tension, the anchorage would have a tendency to creep along one or more of the weak zones unless the stresses were restricted to a very low magnitude. Extensive tests would be required to determine the shearing stresses that could safely be withstood without creep; it is quite possible that the size of anchorage required to keep the stresses within appropriate limits would be excessive and would substantially increase the cost of the structure.

1911 G.H. BAKER DRIVE URBANA, ILLINOIS 61801 OFFICE 217-333-2542 HOME 217-367-4791

The establishment of anchorages for long-span suspension bridges on materials subject to creep has rarely been attempted and only after exhaustive investigation. Hence, before such an alternative could be considered, the subsurface conditions at the proposed anchorages would need to be defined by boring, sampling, and testing. Only if the results of such investigations proved favorable should the alternative be entertained.

Line IV. The proposed multi-span bridge crossing at Line IV would involve the construction of piers in water as much as 50 to perhaps 75 feet deep. A novel construction procedure has been studied by your organization; it includes means for establishing a base shell through which 36-inch pipe piles can be driven and on which can be erected segments forming an enclosure within which the pier shafts can be concreted in the dry except for a seal at the bottom. Although unusual, the techniques include procedures adapted from the construction of off-shore drilling rigs and appear to be feasible.

The design criteria for the bridge indicate that earthquakes will be taken into account by computing lateral forces equal to 0.1 times the dead load applied horizontally in any direction at the center of gravity. This procedure may be satisfactory for an initial estimate of the cost of the bridge based on preliminary designs. However, because of the great depth of the sediments at the bridge crossing, it is likely that earthquake motions transmitted to the soil through the underlying rock will be considerably modified in the upper part of the alluvial materials. It is likely that the accelerations may be relatively low but the periods of vibration may be long and the displacements large. Hence, final design should be based on a dynamic analysis in which the relative movements of the tops of the piers are investigated to provide a basis for assuring that the spans remain in position on the piers. In the earthquakes considered in the dynamic analysis, vertical as well as horizontal acceleration should be taken into account.

The nature of the alluvium in the buried valley cut into the dense glacial soils beneath the Knik Arm has not yet been determined reliably. Hence, estimates of pile lengths or sizes to achieve postulated capacities cannot be made with confidence and conservative values are justified. The susceptibility of the alluvial materials to loss of strength during earthquakes also requires investigation by detailed sampling and testing before foundation types can be compared in a meaningful way.

Line V. Comments with respect to a proposed bridge crossing at Line V are similar to those for Line IV, except that conditions are considerably less severe with respect to depth of water. Foundation conditions in the upper materials are as yet, however, virtually unknown.

A seriously considered alternative at this location is a causeway. The principal factor to be considered in this alternative is the possibility of liquefaction during earthquakes. In my judgment, no sand placed under water without extensive artificial densification can be relied upon to remain in position during a major earthquake such as that of 1964. The portions of a causeway beneath water level should, in my judgment, consist of rock.

Fairly steep slopes, on the order of 2 (horizontal): 1 (vertical), have been proposed for rock fill in the causeways. These slopes represent, in my judgment, the steepest likely to be successful and require that the foundation material have at least the strength of a dense sand. Present knowledge concerning subsurface materials along Line V is based on a single boring and the results of reflection soundings. The echo profile obtained from the soundings is complex and indicates several buried valleys. It does not appear that the presense of loose silty materials, loose sandy materials, or even soft clay can be ruled out on the basis of present information. Such materials would require much flatter embankment slopes irrespective of the strength of the embankment materials. Furthermore, if materials subject to liquefaction should be present, reduction of the danger of liquefaction by flattening slopes may be desirable, inasmuch as reduced slopes are associated with smaller shearing stresses in the foundation material.

On account of the foregoing possibilities, it would seem advisable to furnish two cost estimates for the causeway alternative. One, the more optimistic, might be based on the 2:1 slopes for which analyses have been performed on the assumption that satisfactory foundation conditions exist. The other, which might be considered reasonable if foundation conditions are as poor as is compatible with our present knowledge about the geology, would represent a cost that is not likely to be exceeded. For such an estimate, I would judge that the average rock slopes might be taken as flat as about 5:1.

There is little merit in making additional stability analyses on the basis of assumed weak subsurface conditions; it is preferable to assume the overall slopes. Until factual knowledge of the character of the subsurface materials is obtained by a series of borings with samples and appropriate tests, the required cross-section of the causeway is considered a matter for speculation. The single boring at our disposal indicates very favorable conditions; furthermore, it corresponds reasonably well with conditions encountered in the Turnagain Arm. Nevertheless, in my judgment, it would at present be unwarranted optimism to assume that the foundations for the entire causeway, or even a substantial part of it, would be so favorable as to justify an estimate based on 2:1 slopes throughout. At the present state of knowledge, it would be more conservative to assume that bridge foundations could be designed and constructed as to remain stable during an earthquake than to assume that liquefaction would not occur at at least some locations beneath a causeway. Hence, a bridge crossing presently appears to be a more positive and dependable alternative. Nevertheless, no rational assessment of the relative dependability of the two types of construction can be made until adequate subsurface exploration has been carried out.

Conclusion. The discussion in connection with Lines IV and V indicates that a substantial margin of uncertainty is inevitable in the engineering evaluation and cost estimates that can be made at the present time. This situation cannot be improved until adequate technical data are obtained concerning subsurface condition along the proposed lines. In my judgment, expenditures to this end would be more rewarding than further detailed analyses which must inevitably be speculative.

Yours very sincerely,

Ralph B Peck

Ralph B. Peck

RBP/gmc

INDEX TO PLAN AND PROFILE PLATES APPENDIX A

bibliography

APPENDIX F

bibliography

Climate

Bretschneider, C. L. — *The Handbook of Ocean and Underwater Engineering.* McGraw-Hill, 1969

Geiger, Rudolf — *The Climate Near the Ground.* Cambridge-Harvard University Press, 1965

Myers, J. J.
Holm, C. H.
McAllister, R. F. — *Handbook of Ocean and Underwater Engineering.* North American Rockwell Corporation. McGraw-Hill, 1969

U. S. Dept. of Commerce — *Annual Surface Winds Summary and Extreme Values of Surface Winds Summary,* from Revised Uniform Summary of Surface Weather Observations. Elmendorf AFB, Anchorage, Alaska. National Oceanic and Atmospheric Administration. Environmental Data Service

U. S. Dept. of Commerce — *The Climate of the Matanuska Valley.* Technical Paper No. 27, 1956

U. S. Dept. of Commerce — *Climatography of the United States No. 82-49, Alaska.* 1963

U. S. Dept. of Commerce — *Climatography of the United States No. 60-49, Climates of the States – Alaska.* Environmental Science Services Administration. Environmental Data Service. May, 1968

U. S. Dept. of Commerce — *Local Climatological Data – 1969, Anchorage, Alaska.* Environmental Science Services Administration. Environmental Data Service

U. S. Dept. of Commerce — *Coastal Weather and Marine Data Summary for Gulf of Alaska, Cape Spencer Westward to Kodiak Island.* Environmental Science Services Administration. Technical Memorandum EDSTM 8. May, 1969

U. S. Dept. of Commerce — *Probable Maximum Precipitation and Rainfall-Frequency Data for Alaska.* Weather Bureau. Technical Paper No. 47.

U. S. Dept. of Commerce — *Storm Data.* National Oceanic and Atmospheric Administration. Environmental Data Service

U. S. Dept. of Commerce — *Summary of Hourly Observations 1956 – 1960, Anchorage, Alaska International Airport.* Weather Bureau

U. S. Dept. of Commerce — *Two-to Ten-Day Precipitation For Return Periods of 2 to 100 Years in Alaska.* Weather Bureau. Technical Paper No. 52

U. S. Dept. of the Navy, Naval Facilities Command — *Cold Regions Engineering NAVFAC DM-9.* December, 1967

U. S. Weather Bureau Anchorage — *Southeasterly Winds in the Anchorage Area Since 1943.* November 12, 1969

U. S. Weather Bureau Anchorage — Weber, G. Philip. *Strong, Gusty North Winds at Anchorage, Alaska*

U. S. Weather Bureau — Records from the Matanuska Agricultural Experiment Station and the Wasilla 3S Station.

Dams

Casey, H. J. — *Construction of Rock Filled Dams in Flowing Water on the Passamaquoddy Tidal Power Project.* Communication No. 3, Second Congress on Large Dams, Washington, D. C. 1936.

Isbash, S. V. — *Construction of Dams by Depositing Rock in Running Water,* Communication No. 3, Second Congress on Large Dams, Washington, D. C. 1936

United States Committee on Large Dams — *Supervision of Dams by State Authorities,* July, 1966.

U. S. Army Corps of Engineers — *Hydraulic Design Criteria.* Sept., 1970 revision.

U. S. Dept. of the Interior	*Design of Small Dams.* Bureau or Reclamation, First Edition, 1960

Earthquake

Shannon & Wilson Inc.	*Anchorage Area Soil Studies, Alaska.* Report prepared for U. S. Army Engineer District, Anchorage, Alaska as authorized by the Office of Emergency Planning. August 28, 1964
Shannon & Wilson Inc.	*Geologic & Engineering Reconnaissance – Proposed Knik Arm Crossing.* Anchorage, Alaska, 1971
U. S. Dept. of Commerce	Wood, F. J. *The Prince William Sound, Alaska, Earthquake of 1964 and Aftershocks.* Environmental Science Services Administration. Coast and Geodetic Survey. 1966. Vol. 1
U. S. Dept. of the Interior	Hansen, Wallace R. *The Alaska Earthquake March 27, 1964. Effects on Communities – Effects of the Earthquake of March 27, 1964 at Anchorage, Alaska.* Geological Survey Professional Paper 542-A. 1965.
U. S. Dept. of the Interior	Hansen, Wallace R., Edwin B. Eckel, William E. Schaem, Robert E. Lyle, Warren George and Genie Chance. *The Alaska Earthquake March 27, 1964. Field Investigations and Reconstruction Effort.* Geological Survey Professional Paper 541. 1966
U. S. Dept. of the Interior	Miller, R. D. and Dobrovolny, E. *Surficial Geology of Anchorage and Vicinity, Alaska.* Geological Survey Bulletin 1093. 1959
U. S. Dept. of the Interior	Plafker, George. *The Alaska Earthquake March 27, 1964. Regional Effects – Tectonics of the March 27, 1964 Alaska Earthquake.* Geological Survey Professional Paper 543-I. 1969.

Environmental

University of Alaska	Johnson & Hartman, *Environmental Atlas of Alaska.* 1969

Geology

Alaska Department of Highways, Engineering Geology Section, Materials Division	*Engineering Geology and Foundation Report of Preliminary Field and Laboratory Soils Data – Knik Arm Project No. X12750.* December, 1970
Alaska Department of Highways, Engineering Geology Section, Materials Division	*Engineering Geology and Foundation Report of Field and Laboratory Soils Data – Turnagain Arm Crossing Project No. F-021-2(10) Anchorage District.* May, 1969
Alaska Geological Society	*Guidebook.* Anchorage, Alaska. October, 1964
U. S. Dept. of the Interior	Dobrovolny, Ernest and Schmell, Henry R. *Map of Geological Materials, Anchorage and Vicinity, Alaska.* Geological Survey Open-File Report. 1966.
U. S. Dept. of the Interior	*Physiographic Divisions of Alaska 1964.* Geological Survey Professional Paper 482
U. S. Dept. of the Interior	*Quaternary Geology of the Kenai Lowland and Glacial History of the Cook Inlet Region, Alaska.* Geological Survey Professional Paper 443. 1964.
U. S. Dept. of the Interior	*Stability of the West Slope of Government Hill Port Area of Anchorage, Alaska.* Geological Survey Bulletin 1258-D. 1969.
U. S. Dept. of the Interior	*Surficial Geology of Anchorage and Vicinity Alaska.* Geological Survey Bulletin 1093. 1959.
U. S. Dept. of the Interior	Wahrhaftig, Clyde. *Physiographic Divisions of Alaska.* Geological Survey Professional Paper 482. 1965

Hydrology

American Society of Civil Engineers	*Nomenclature for Hydraulics.* Manuals and Reports on Engineering Practice – No. 43
Billington, David	*Holland – Between River and Sea.* Civil Engineering, May, 1967.
Chow, Ven Te	*Open-Channel Hydraulics.* University of Illinois. 1959
Sharma, G. D. and Burrell, D. C.	*Sedimentary Environment and Sediments of Cook Inlet, Alaska.* 1969
U. S. Dept. of Commerce	*Descriptive Reports.* Hydrographic Surveys. Coast and Geodetic Survey.
U. S. Dept. of Commerce	Environmental Science Services Administration Coast & Geodetic Survey. *Anchorage – Prince William Sound Area, Alaska, Primary Network and Intersection Points*
U. S. Dept. of Commerce	*Hydrographic Surveys Cook Inlet, Alaska. H-3200 (1910), H-3674(1914), H-5104(1930), H-6657(1941), H-8203(1955), H-8527(1960), H-8786(1964), H-8787 (1964).* Environmental Science Services Administration. Coast and Geodetic Survey
U. S. Dept. of Commerce	*Mean Monthly and Annual Evaporation Data From Free Water Surface for the United States, Alaska, Hawaii and the West Indies.* Weather Bureau, Technical Paper No. 13.
U. S. Dept. of Commerce	*Probable Maximum Precipitation and Rainfall – Frequency Data for Alaska.* Weather Bureau Technical Paper No. 47.
U. S. Dept. of Commerce	*Two-to Ten-Day Precipitation for Return Periods of 2 to 100 Years in Alaska.* Weather Bureau, Technical Paper No. 52

U. S. Dept. of the Interior	Waller, R. M., *The Alaska Earthquake, March 27, 1964, Effects on Hydrologic Regimen, Anchorage Area.* Geological Survey, Professional Paper 544-B. 1966.
U. S. Dept. of the Interior	Bureau of Reclamation. Bertle, F. A. Report – *Inflow Design Flood Study – Eklutna Dam Site – Eklutna Project, Alaska*
U. S. Dept. of the Interior	Compilation of Records of Surface Waters of Alaska, October, 1950 to September, 1960 *Geological Survey Water – Supply Paper 1740*
U. S. Dept. of the Interior	Childers, Joseph M. *Flood Frequency in Alaska,* Geological Survey – Water Resources Division, Open-File Report. 1970.
U. S. Dept. of the Interior	Geological Survey. Water Resources Data for Alaska. *Part 1. Surface Water Records.* 1961 to 1969.
U. S. Dept. of the Interior	Geological Survey Pamphlet – *The Breakout of Alaska's Lake George.* 1958-1968
U. S. Dept. of the Interior	Marsh, R. E. *Knik–Matanuska Rivers Near Palmer, Alaska.* Geological Survey, February, 1963.
U. S. Dept. of the Interior	Marsh, R. E. *Lake George–Knik Glacier Investigation.* Geological Survey, March 1961
U. S. Dept. of the Interior	*Magnitude and Frequency of Floods in Alaska South of the Yukon River.* Geological Survey Circular 493. 1964.
U. S. Dept. of the Interior	*Progress Report – Lake George – Knik Glacier Investigation,* by R. E. Marsh, District Engineer. Geological Survey. March, 1961.

U. S. Dept. of the Interior — *Quantity and Quality of Surface Waters of Alaska, 1950 to 1953, 1953 to 1956, 1957, 1958, 1959, 1960.* Geological Survey, Water-Supply Paper Nos. 1466, 1486, 1500, 1570, 1640 and 1720.

U. S. Dept. of the Interior — *Compilation of Records of Surface Waters of Alaska, October 1950 to September 1960.* Geological Survey, Water-Supply Paper 1740.

U. S. Dept. of the Interior — *Quality of Surface Waters of Alaska, 1961-63.* Geological Survey Water-Supply Paper No. 1953.

U. S. Dept. of the Interior — *Tailwater Elevations At Eklutna Plant During Lake George Breakup, 1955-1961.* Bureau of Reclamation

Ice

Assur, A. — *Composition of Sea Ice and It's Tensile Strength,* Artic Sea Ice, NAS-NRC Publication No. 598, 1958

Bruun, P. & Straumsnes, A. D. — *Piling Up of Ice on Seashores and on Coastal Structures.* Department of Port and Ocean Engineering. The Technical University of Norway. Trondheim, Norway

Costa, F. Vasco — *The Berthing Ship,* The Dock and Harbour Authority.

Dykins, J. E. — *Review of Sea Ice Sheet Loading.* U. S. Naval Civil Engineering Laboratory. Port Hueneme, California

Int'l. Assoc. of Hydraulic Research — *Ice Symposium Reykjavik, Iceland.* September 8-10, 1970

Korzhavin, K. N. — *Action of Ice on Engineering Structures.* Academy of Sciences of USSR, Siberian Section. 1962

Lewis, E. L. — *Introduction to Arctic Sea Ice.* Paper Presented at The Western Congress of Engineering, Jasper Alberta. May, 1970

Peyton, Harold R. — *Ice and Marine Structures.* Ocean Industry. Parts 1, 2, & 3. March, September, December, 1968

Peyton, Harold R. — *Sea Ice Strength.* Report prepared for Department of Navy, Office of Naval Research - University of Alaska, Geophysical Institute. December, 1966

Matlock, Hudson, etal — *A Model for Prediction of Ice Structure Interaction,* University of Texas, 1969

National Academy of Science — National Research Council *Physics and Mechanics of Sea Ice & Sea Ice Formation, Growth and Disintegration.* Arctic Sea Ice. Publication No. 598, Sections 3 & 4. 1958

U. S. Dept. of Commerce — *Ice Conditions in Cook Inlet and Anchorage Approaches.* U. S. Coast Pilot No. 9, 1964. Environmental Science Services Administration, Coast and Geodetic Survey

Weeks, W. F. & Assur, A. — *Fracture of Lake and Sea Ice.* U. S. Corps of Engineers. Cold Regions Research and Engineering Laboratory. Research Report 269. September, 1969.

Weeks, W. F. & Assur, A. — *The Mechanical Properties of Sea Ice.* U. S. Army Material Command. Cold Regions Research and Engineering Laboratory. DA Project 1V025-001A130

Wilson, H. H. — *Ice Study – Turnagain Arm, 1968-1969 Season.* Conducted by Alaska Department of Highways in cooperation with Peyton, H. R. Project F-021-2(10)

Reeh, Niels — *Natural Frequency of the System of a Heavy Elastic Plate Covering Shallow Water.* Reprinted from Bygningsstatiske Meddelelser Vol. 41, No. 3, 1970.

Maps

U. S. Dept. of Commerce — *Navigation Maps 8557 and 8553,* U. S. Coast and Geodetic Survey.

U. S. Dept. of Interior — U. S. Geological Survey. *Quadrangle Maps*

Previous Reports

Armstrong Associates — *Causeway Studies, Turnagain Arm Crossing, Alaska.* January, 1969.

Bloch, Ivan & Associates — *Preliminary Appraisal of Proposed Knik Arm Causeway, Anchorage, Alaska.* 1955

Dames & Moore — *Report of Seismic Reflection Survey Proposed Knik Arm Highway Crossing.* Prepared for State of Alaska, Department of Highways. December 1, 1970

Dames & Moore — *Report of Seismic Reflection Survey Proposed Monorail Crossing Seward's Success, Alaska.* Prepared for the Great Northern Corporation. November 20, 1970

Porter, O'Brien & Armstrong & Tryck, Nyman & Associates — *Preliminary Report – Feasibility Study Turnagain Arm Crossing – Phase I – Research of Existing Data.* March 8, 1963

Porter, O'Brien & Armstrong & Tryck, Nyman & Associates — *Preliminary Report – Feasibility Study Turnagain Arm Crossing – Phase II – Alternate Crossing Studies.* January 6, 1964

Retherford, Robert W. Assoc. & Alaska Geological Consultants — *Route Selection Report – Submarine Cable Crossing of Knik Arm – 138 KV Transmission Line – Beluga Project.* Prepared for Chugach Electric Association, Inc.

Structural

Cloyd, Marshal P. — *Monopod. – Cook Inlet, Alaska.* Civil Engineering. March, 1968

Gaither, William S. & Dalton, Robert E. — *All-Weather Tanker Terminal for Cook Inlet, Alaska* ASCE Proc. 95

Tides & Currents

American Society of Civil Engineers — *Definitions of Surveying, Mapping, and Related Items* Manuals of Engineering Practice - No. 34, 1954

American Society of Civil Engineers — *Nomenclature for Hydraulics.* Manuals and Reports on Engineering Practice – No. 43

Bretschneider, Charles L. — *On the Generation of Wind-Driven Currents Over The Continental Shelf*

Bretschneider, Charles L. — *Steady-State Wind Tides Over Sloping Continental Shelf Caused by Winds Blowing At An Angle to The Coastline.* Shore and Beach April, 1966

Bruun, Per — *Tidal Inlets and Littoral Drift.* 1966

Bruun, Per — *Tidal Inlets Housekeeping.* ASCE Proc. 93 (HY5 No. 5435)

Bruun, P. & Gerritsen — *Stability of Coastal Inlets.* 1955

Carlson, Robert F. — *The Nature of Tidal Hydraulics in Cook Inlet.* The Northern Engineer. Vol. 2, No. 4.

City of Anchorage, Alaska — U. S. Coast & Geodetic Survey. *Tidal Recordings at City Dock for May 25, 26 & June 1, 2, 1971*

Dronkers, J. J. — *Tidal Computations In Rivers and Coastal Waters.* 1964

Hicks, Steacy D. and Shofnos, William — *Yearly Sea Level Variations For The United States.* ASCE Proc. 91 (HY5 No. 4468)

Hunt, Ira A. & Volker, A. — *Dutch Close Tidal Breach at Ouwerkerk.* Civil Engineering, October 1954

National Bureau of Standards — *Third Progress Report on Tidal Flow in Entrances Water-Level Fluctuations of Basins in Communications with Seas.* By Garbis H. Keulegan for Beach Erosion Board Corps of Engineers Department of the Army. September 10, 1951

U. S. Dept. of Commerce — *Ice Conditions in Cook Inlet and Anchorage Approaches.* U. S. Coast Pilot No. 9, 1964. Environmental Science Services Administration, Coast and Geodetic Survey

U. S. Dept. of Commerce — *Tide Tables – High and Low Water Predictions – West Coast of North and South America 1960-1971.* Environmental Science Services Administration, Coast and Geodetic Survey.

U. S. Dept. of Commerce — *Tidal Bench Marks Southwest Alaska – 53. Anchorage, Knik Arm, Cook Inlet.* Environmental Science Services Administration, Coast and Geodetic Survey. April 7, 1970

Waves

American Society of Civil Engineers — Bretschneider, C. L., *Hurricane Design – Wave Practices.* Paper No. 2965, Transactions Vol. 124, 1959

Battjes, Jurjen A. — *Run-Up Distributions of Waves Breaking on Slopes.* ASCE Proc. 97 (WW1 No. 7923)

U. S. Army Corps of Engineers — *Shore Protection, Planning and Design, Coastal Engineering Research Center,* Technical Report No. 4, 1966