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SUSITNA HYDROELECTRIC PROJECT

EXTERNAL REVIEW BOARD
MEETING #3

ALASKA POWER AUTHORITY

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OCTOBER 6-8, 1981

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October 8, 1981

SUSITNA HYDROELECTRIC PROJECT
EXTERNAL REVIEW PANEL
REPORT NO. 3

INTRODUCTION

The third meeting of the External Review Panel for the Susitna Hydroelectric Project was convened on October 6-8, 1981 at the Acres American office in Buffalo. In addition to Panel Members, representatives of the Alaska Power Authority and Acres American were present. Various members of the Acres American staff presented discussions regarding progress in geotechnical areas, seismicity, hydraulics, hydrology, and design. The discussions were well prepared and presented in such a manner as to give a maximum amount of information in a reasonable time.

Prior to the meeting Panel Members received a document entitled "Susitna Hydroelectric Project, External Review Board, Meeting #3, Information Package, October 6-8, 1981". During the meeting other printed information was presented to the Panel as required.

21.04
The Panel appreciates the efforts of the Acres American Staff in planning and preparing for this very informative and successful meeting.

SEISMICITY AND SEISMIC GEOLOGY

Excellent progress has been made during the summer months in resolving most of the uncertainties regarding the possible presence of active faults in the vicinity of the dam sites, in developing an adequate model of the seismic geology of the region, and in assessing the maximum levels of earthquake shaking which could result from events occurring along the major seismic sources. These studies have led to the following preliminary conclusions:

WATANA DAM SITE

Four major lineaments were originally identified as being possible faults in the vicinity of the dam:

- (1) The Talkeetna Thrust Fault
- (2) The Fins Feature
- (3) The Susitna Feature
- (4) The Watana River Feature

Field geologic studies during the past several months have developed evidence indicating that:

- (1) The Talkeetna Thrust Fault is not an active fault.
 - (2) The Watana River Feature is not a fault.
 - (3) The Susitna Feature is not a fault.
- and (4) The Fins Feature may well be a fault but it is relatively short in length and, since there are apparently no other active faults in the area, it is very unlikely that it could be active. In any case its length would preclude the possibility of it being the source of a significant earthquake.

In consequence, there are apparently no active faults crossing the site and the major sources of earthquake shaking at the site may be attributed to earthquakes occurring on the Benioff Zone underlying the site at depth, the Denali fault, the Castle Mountain Fault, and smaller local earthquakes occurring with no apparent surface expression in the crust of the Talkeetna terrain. Considerations of fault distances and possible earthquake magnitudes leads to the conclusion that the approximate maximum levels of shaking will be due to the following sources:

<u>Source</u>	<u>Closest Distance</u>	<u>Magnitude (Ms)</u>	<u>Peak Acc. (Mean)</u>
Benioff Zone	≈ 63 km	≈ 8½	≈ 0.35g
Benioff Zone	≈ 48 km	≈ 7½	≈ 0.32g
Denali Fault	≈ 70 km	≈ 8+	≈ 0.22g
Local Event	*	*	*

Seismic geology considerations have led Woodward-Clyde consultants to suggest that the maximum local earthquake which needs to be considered is a Magnitude 5½ to 6 event occurring at a distance of about 10 km from the site. Such an event would produce a peak acceleration (mean value) of about 0.35g and would therefore not be a controlling event. However, the Panel believes that in view of the past seismic history and other considerations it would probably be prudent to consider the possibility of a somewhat larger event at a slightly shorter distance. In which case the local earthquake would be responsible for the maximum accelerations likely to develop at the dam site. This does not mean however, that it will necessarily control the design.

For the Benioff Zone event, which seems to be controlling at this stage, the motions recommended by Woodward-Clyde Consultants for preliminary design evaluations appear to be entirely appropriate.

DEVIL CANYON SITE

At the end of 1980, nine lineaments were identified in the vicinity of the Devil Canyon site which could possibly be active faults. Field geologic studies during the past 6 months have led to the conclusion that only 3 of these features are faults, that the three features recognized as faults are inactive, and that in any case they are so short in length that they could not generate earthquakes which would be controlling events with regard to earthquake motions at the dam site. Thus since there are no active faults in the vicinity of the dam site, the design earthquake motions will be determined by similar considerations to those applicable for the Watana site. The Panel agrees with those conclusions.

* Information to be provided in Final WCC Report

Consideration of the most significant seismic sources of ground shaking leads to the following:

<u>Source</u>	<u>Closest Distance</u>	<u>Magnitude (Ms)</u>	<u>Peak Acc. (Mean)</u>
Benioff Zone	≈ 90 km	≈ 8½	≈ 0.3g
Benioff Zone	≈ 58 km	≈ 7½	≈ 0.3g
Denali Fault	≈ 64 km	≈ 8+	≈ 0.24g
Local Event	*	*	*

As for the Watana site, there is a need to establish very soon the significant characteristics of the local earthquake (in the crust of the Talkeetna Terrain) in order to finalize the seismic criteria to be used for project design.

In the light of the information presented at this meeting and on the basis of past experience, the Panel believes that through the use of appropriate design and construction procedures, dams with ample margins of seismic safety can be constructed at both sites. The Panel believes, however, that the question of seismic effects due to local crustal earthquakes should be resolved in the next few weeks so that more definitive design studies can be completed.

ROCK ENGINEERING CONSIDERATIONS

As a result of discussions during this meeting as well as observations made in the field by Panel member Merritt during the period of 23-25 September, we have the following comments regarding present designs.

WATANA

Every effort should be made to reduce the height of the cut slope at the inlet to the diversion tunnel. The structures can probably be moved closer to the river and perhaps shifted slightly in a downstream direction.

The surface excavation at the outlets of the tailrace tunnels and spillway structures is likewise very extensive. Further detailed examination is warranted to minimize possible slope stability problems.

* To be provided in final WCC Report

Recent borings in the proposed underground powerhouse site encountered a zone of soft hydrothermally altered diorite. This is not acceptable material to have in a major underground excavation. Some shifting of these openings is required. Considering all borings made in the right abutment, the general quality of the diorite is quite high and we foresee that acceptable rock can be found for the proposed structures.

DEVIL CANYON

The graywacke and argillite at this site appear to be of acceptable quality for the proposed underground structures. No major shear zones have been recognized in these areas. The underground openings have been oriented with respect to the major known joint systems and bedding planes. The present layout is acceptable and it is recognized that some slight shift could result based upon the results of future exploration.

The axis of the proposed surface spillway on the right abutment will nearly parallel the strike of the bedding of the rock. The required cuts will daylight the bedding which dips at about 50 degrees into the excavation. Potential major rock stability problems could result which might not be solved by simple rock bolting measures. This design likewise requires your review.

BURIED CHANNEL

The results of all geophysical surveys completed to date have defined a major channel beneath the plateau on the right abutment at the Watana Site. The channel is approximately 15,000 ft wide when measured with respect to that portion of the bedrock channel below the proposed reservoir pool level. The deepest portion of the channel lies about 450 ft below pool level; however, perhaps as much as 60-70% of the channel lies 100 ft or less below maximum pool level.

The borings completed during the Corps of Engineers study indicated that the channel is filled with glacial till, outwash, and perhaps lacustrine deposits. The boring logs show that boulders (some as large as 12 ft) can be expected in these heterogeneous deposits, either as individual units or as thick layers. Contour maps made of the bedrock surface suggest a

wide entrance channel or channels upstream of the damsite and a relatively narrow exit into Tsusena Creek downstream of the damsite.

The buried channel on the north slope of the reservoir at Watana Dam is much greater in extent than was anticipated a year ago and represents one of the greatest uncertainties associated with the Watana Dam project.

Major problems posed by the presence and extent of this channel are

- (1) The magnitude of possible seepage losses through the channel.
- (2) The possibility of piping within the channel resulting from seepage from the reservoir towards Tsusena Creek.
- (3) The possibility of seismic instability in the soils comprising the buried channel under strong earthquake shaking.

It appears that problems (1) and (2) above could be eliminated by construction of a cut-off wall and grout curtain through the soils filling the channel. However, the provision of such a cut-off would not solve any problems of seismic instability on the upstream side of the wall.

Since very little information is available concerning the nature of the soils forming the channel fill it is not possible to assess the magnitude of the seismic instability problem, if indeed it exists at all, or the need for an extensive cut-off wall, currently projected to be about 15,000 feet long and varying from a few feet to 450 feet in depth. However, it is clear that both the possibility of seismic instability and the cost of a cut-off would be dramatically reduced if the reservoir level were about 100 feet lower than currently planned. Such a lowering could reduce the length of the cut-off to about 4,000 feet, facilitate its construction and by lowering the water table in the soils, increase their seismic stability. In view of these advantages, together with the fact that economic advantages associated with the top 50 to 80 feet of Watana Dam do not appear to be very great, the Panel believes that careful consideration should be given to the potential benefits of reducing the height of Watana Dam by 50 to 100 feet. Such a reduced height might also facilitate layout problems for the dam.

The Panel cannot be sure that a reduction in dam height would be advantageous but believes that a careful study of the question is warranted in the next several months.

WATANA DAM EMBANKMENT

The Panel believes that the preliminary design section selected for Watana Dam is satisfactory and will produce a stable and economical structure. It is suggested however, that consideration be given to the following items:

- (1) If the shells are constructed of densely compacted gravel or rockfill and the core of a much more compressible sandy-silky-clay, there is a danger of deleterious stress redistribution due to differential settlements. Thus consideration should be given to minimizing this possibility by:
 - (a) inclining the core slightly upstream, providing this can be done without jeopardizing stability.
 - and/or (b) locating a relatively incompressible core material which is adequately impervious. Such a material appears to be available as a GC material in one of the borrow areas.
- (2) Deformations of the upstream shell of the dam due to strong earthquake shaking can be minimized either by densifying the shell material to such extent that high pore pressures cannot develop or by using highly pervious rock-fill which will dissipate any pore pressures resulting from earthquake shaking almost as rapidly as they develop. Consideration should be given to using gravel-fill and rock-fill in the upstream shell in such a way as to optimize their use from a seismic design point of view.
- (3) There is apparently ice in the rock joints in the abutments at Watana dam site and this will have to be thawed before grouting. It would be desirable to determine whether construction costs have allowed for this.
- (4) It appears that there may well be permafrost in the foundation soils for the saddle-dam. When this melts it could leave the soils in a very loose condition which may be adequate for static stability but inadequate for seismic stability. It would be desirable to explore this possibility further and examine the need for excavation of frozen foundations soils prior to saddle-dam or dike construction.

DEVIL CANYON DAM

Sufficient study has been completed to adequately support the present arch

dam design for feasibility purposes. However, the linear feature through the pond areas where the wing dam will be located should be further explored in the near future. Similar considerations to those discussed for the Watana Site should be given to the foundation soils under the Devil Canyon wing dam.

WATANA DAM DIVERSION TUNNELS

Two diversion tunnels are proposed for diverting up to a 1 in 50-year flood during construction of Watana Dam. One tunnel would be located at a low level so that it would flow full at all times. The second tunnel, located at a higher level, would have free flow. After diversion the lower tunnel would be plugged. Two plugs would be constructed in the upper tunnel with gated outlets through them to permit release of low flows until Devil Canyon is completed and serve to lower the reservoir in case of an emergency. The Panel concurs in the general concept of the diversion tunnels and modification of the high level tunnel for use as a low-flow and emergency release outlet, subject to refinements discussed by Acres.

WATANA DAM SPILLWAY

Spillway flows at Watana Dam would be handled by three separate flow release structures. Discharges corresponding up to a 1 in 100-year flood, would be released through a low-level tunnel controlled by three or more Howell-Bunger or similar valves located at the downstream end of the tunnel. Discharges corresponding to floods in excess of 1 in 100-years and up to 1 in 10,000-years would flow through an open chute spillway with a flip bucket. Discharges in excess of the 1 in 10,000-year flood up to the PMF would pass through a bypass channel controlled by a fuse plug.

The Panel concurs in the proposed concept of handling spillway flows. Release of floods up to 1 in 100-years by low level valves would maintain the nitrogen supersaturation level to an acceptable limit. The Panel suggests that fixed cone valves, as installed by the Corps of Engineers at New Melones Dam be used, since its greater rigidity makes it more suitable for high-head operation. The smaller spillway/chute flows reduce erosion in the downstream river channel. Hydraulic model tests will be required

to determine the extent of material that should be pre-excavated in the plunge pool area. In view of the infrequency and short duration of spillway operation and the relatively high quality of rock in the steep river banks, the Panel is of the opinion that excessive erosion would not occur due to service spillway operation. With respect to the emergency spillway bypass channel, the Panel is concerned over the 45-ft height of the fuse plug. This high plug would need to be designed as a small earth dam to retain the power pool at maximum levels and also be capable of failure as a fuse plug when it is overtopped. It is suggested that the entrance to the bypass channel be widened, thereby requiring a smaller height of fuse plug. This would also reduce the amount of reservoir lowering in the event of fuse plug failure.

DEVIL CANYON DIVERSION TUNNEL

One diversion tunnel is proposed for Devil Canyon Dam to divert flows up to a 1 in 50-year flood during dam construction. The tunnel would be plugged after it is no longer needed for diversion. The Panel suggests that this tunnel could be used for spillway flow releases in an alternative spillway design discussed hereinafter.

DEVIL CANYON SPILLWAYS

As for Watana Dam, spillway flows at Devil Canyon would be handled by three separate flow release structures. Flows up to the 1 in 100-year flood would be released by four or five outlets through the base of the concrete arch dam controlled by Howell-Bunger or other type high pressure valves. Discharges in excess of 1 in 100-years and up to 1 in 10,000-years would flow through an open chute spillway with a high level flip bucket. Discharges in excess of the 1 in 10,000-year flood up to the PMF would pass through a bypass channel controlled by a fuse plug.

The Panel concurs in the concept of handling the spillway flows subject to one question discussed below. Release of small flows through valves at the base of the dam will prevent excessive nitrogen supersaturation in the downstream river channel, as well as reduce discharges and flow frequency and duration in the chute/flip bucket spillway, thereby reducing plunge pool erosion. Based on a ground and air inspection of the river channel at the Devil Canyon Site by Panel member Douma and Acres repre-

sentatives on September 17, 1981, the Panel is of the opinion that the very high quality rock in the canyon walls should not experience excessive erosion due to spillway operation. In this case, pre-excavation of streambed material and weathered rock is probably not required. The Panel is concerned, however, over the deep sidehill rock cut required for construction of the spillway chute. It suggests that consideration be given to an alternate plan of providing spillway tunnels, as required, instead of the chute spillway. In this alternate plan, the diversion tunnel and probably only one additional tunnel would be required. With respect to the emergency bypass channel spillway, the Panel is concerned over the 57-foot high fuse plug for the reasons stated for the Watana fuse plug. Consideration should be given to increasing the length and reducing the height of this fuse plug as described for Watana.

DEVIL CANYON POWERHOUSE TAILRACE

The Panel concurs in extending the tailrace for the Devil Canyon powerhouse about 1 1/4 mile to take advantage of the additional approximately 30 feet of head.

CLOSING REMARKS

The Panel requests that the topics raised in this report be thoroughly discussed in the next External Review Board Meeting tentatively scheduled for the week of January 11, 1982 in Anchorage.

The Panel greatly appreciates the many courtesies extended to it by the staff of the Alaska Power Authority and the staff of Acres American, Inc.

Merlin D. Copen

Andrew H. Merritt

Jacob H. Douma

H. Bolton Seed

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- SUPPORT MATERIAL TO TASK 6 - DESIGN DEVELOPMENT
 - *WATANA LAYOUT STUDIES
 - *WATANA DAM DESIGN
 - *DEVIL CANYON LAYOUT STUDIES
 - *DEVIL CANYON ARCH DAM DESIGN
- MEETING AGENDA

PROJECT STATUS REPORT, SEPT. 1981

ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
PROJECT STATUS REPORT, OCTOBER 1981

The third meeting of the Power Authority's External Review Board for the Susitna Project will take place at the offices of Acres American Inc. in Buffalo October 6, 7 and 8, 1981. The objective of this meeting is to review the status of field work and engineering studies in hydrology, seismic and geotechnical aspects and in the development of layouts and designs for the main structures at the Watana and Devil Canyon sites.

This Project Status Report is intended to summarize the current status of all project activities, on a Task by Task basis.

Task 1 - Power Studies

This activity is complete, and the results were incorporated in the Project Development Selection Report issued in June 1981. Current studies of power alternatives by Battelle for the State of Alaska include an update of the load forecast for the Railbelt region. The results of these studies will be incorporated into the Acres feasibility assessment later this year. No major changes in the size or scale of the Susitna Project are anticipated at this time.

Task 2 - Surveys and Site Facilities

Operation of the Watana camp has continued in support of field activities. Surveys of dam and reservoir areas for each site have been completed and recent activity has been concentrated on completion of surveys of alternative access and transmission corridors. Work is currently being completed to provide a basis for selection of an access route before the end of the year.

Task 3 - Hydrology

Collection of stream gauge, snow course, climatological, bedload and water quality data at the stations set up for this purpose throughout the project area, has continued in support of the feasibility study and licensing requirements.

Engineering activities in support of project flood, energy yield, river morphology, ice and hydraulic design studies have continued to the point where the design requirements for hydraulic structures can be established.

Task 4 - Seismic Studies

The 1981 field program of geologic mapping and trenching has been completed. Evaluation of data is continuing and preliminary findings indicate no major concerns relative to project feasibility. The results of this work are discussed elsewhere in this document.

Task 5 - Geotechnical Investigations

The 1981 program of geologic mapping, seismic refraction surveys, drilling and testing at the Devil Canyon and Watana sites is continuing. Preliminary results of this work are presented elsewhere in this document. Sufficient data has been obtained to support with reasonable confidence the design requirements for major structures at each site such that the locations and scale of such structures can be established for feasibility study purposes.

The final detailed report on the 1980 field exploration activities is currently being distributed.

Task 6 - Design Development

The Project Development Selection Report issued in June 1981 recommended that the feasibility study be continued to refine and optimize the Susitna Project on the basis of the following components:

Watana: 880 feet high earth- and rockfill dam
400 MW 1993
400 MW 1996

Devil Canyon: 675 feet high thin arch dam
400 MW 2000

Under Task 6, engineering studies have continued at each site on the basis of the additional survey, hydrologic, geotechnical, seismic and environmental data which is becoming available. The studies have included at each site:

- establishment of design assumptions and/or criteria
- consideration of alternative types of dam
- optimization of reservoir levels and dam centerline
- selection of power plant installed capacity
- preliminary spillway design concepts
- consideration of alternative spillway types
- development of optimum diversion schemes
- development of reservoir release facilities
- consideration of alternative project layouts
- preliminary design of dams

The currently favored general arrangements at each site are illustrated in the attached Plates 8.1 and 8.4 (Watana) and 9.1 (Devil Canyon). Results of these studies to date are summarized elsewhere in this document.

Task 7 - Environmental

Wildlife, fisheries and all other environmental studies to establish current conditions in the project area are continuing at full pace. Water quality studies particularly as they relate to sediment transportation, the potential for nitrogen supersaturation and fisheries are also receiving considerable attention. These studies will form

the basis of assessments of the environmental impact of the project and the ultimate development of appropriate mitigation measures for the license application.

The socioeconomic impacts resulting from construction and operation of the project and its associated recreational potential are also being assessed.

Task 8 - Transmission

A report on transmission corridor selection is currently being distributed. Two 345 kv transmission lines from the Watana site will initially be installed with a third line to be added when Devil Canyon is constructed. The three lines will ultimately be used to transmit power to the Anchorage area and two lines to the Fairbanks area. Route selection studies within the proposed corridor are continuing on the basis of ongoing environmental and access studies. Studies are also continuing to optimize and advance the design of these lines and to examine the future development of the total transmission system for the Railbelt region with the Susitna Project incorporated.

Task 9 - Construction Cost Estimates and Schedules

Data gathering to provide labor, equipment and materials costs as a basis for cost estimates has been initiated. An "upper limit" magnitude cost estimate for the project was developed under Task 6 for purposes of ensuring the economic feasibility of the selected project. The currently preferred project layouts are being used as the basis of refined preliminary cost estimates which will be completed by November 30. These estimates will be further refined in parallel with continuing development of designs up to issue of the feasibility report which is scheduled for March 15, 1982.

Task 10 - Licensing

Activities under this Task have continued in terms of coordination with the various federal and state agencies involved and an assessment of the recent proposed changes in licensing requirements and format.

Task 11 - Marketing and Financing

Activities in this area have been limited pending clarification of recent State legislation concerning financing of the project.

Task 12 - Public Participation

Involvement of the public in the development of project concepts has been and continues to be actively encouraged through public meetings, workshops, the issue of newsletters and an action list program.

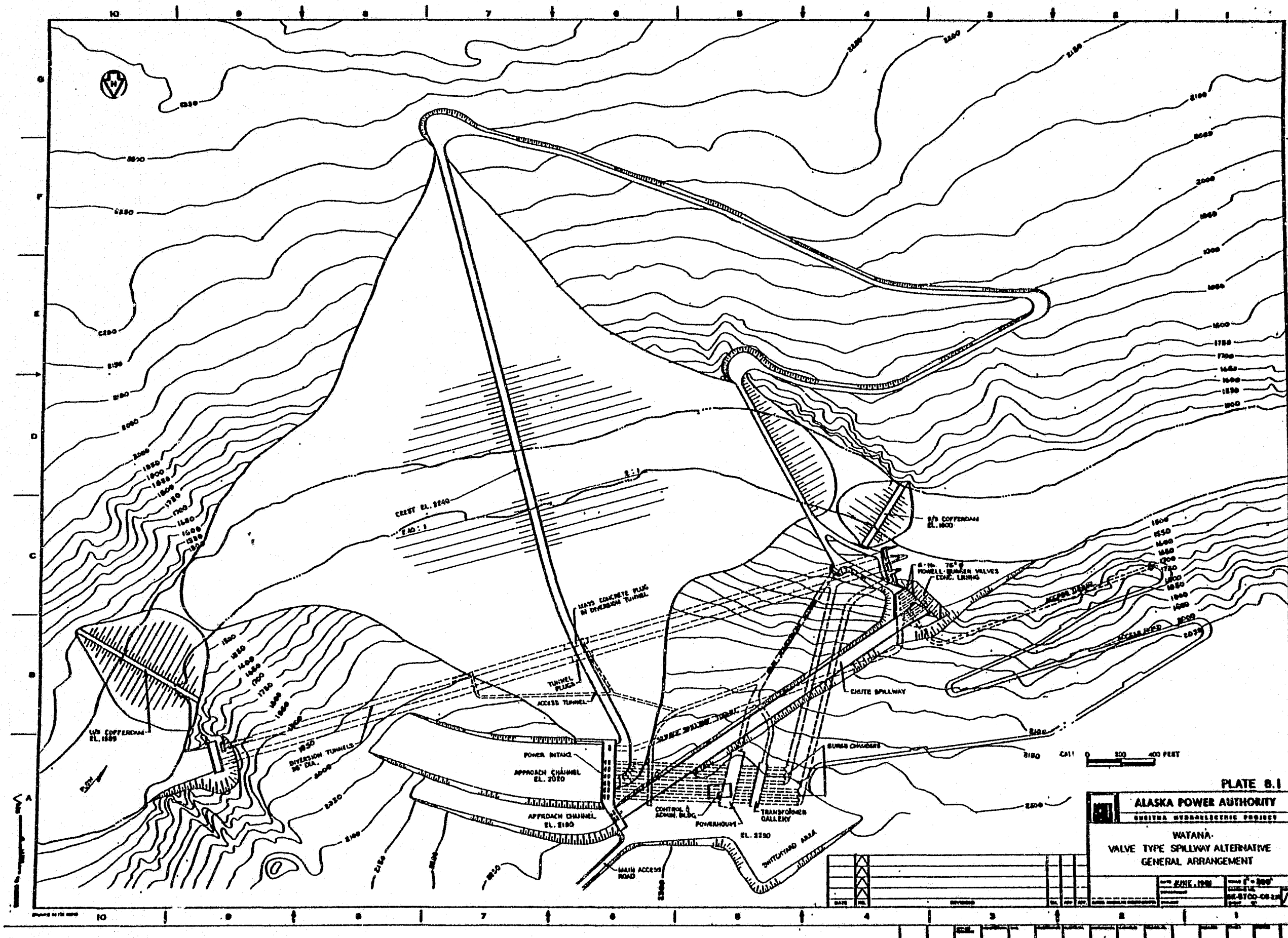


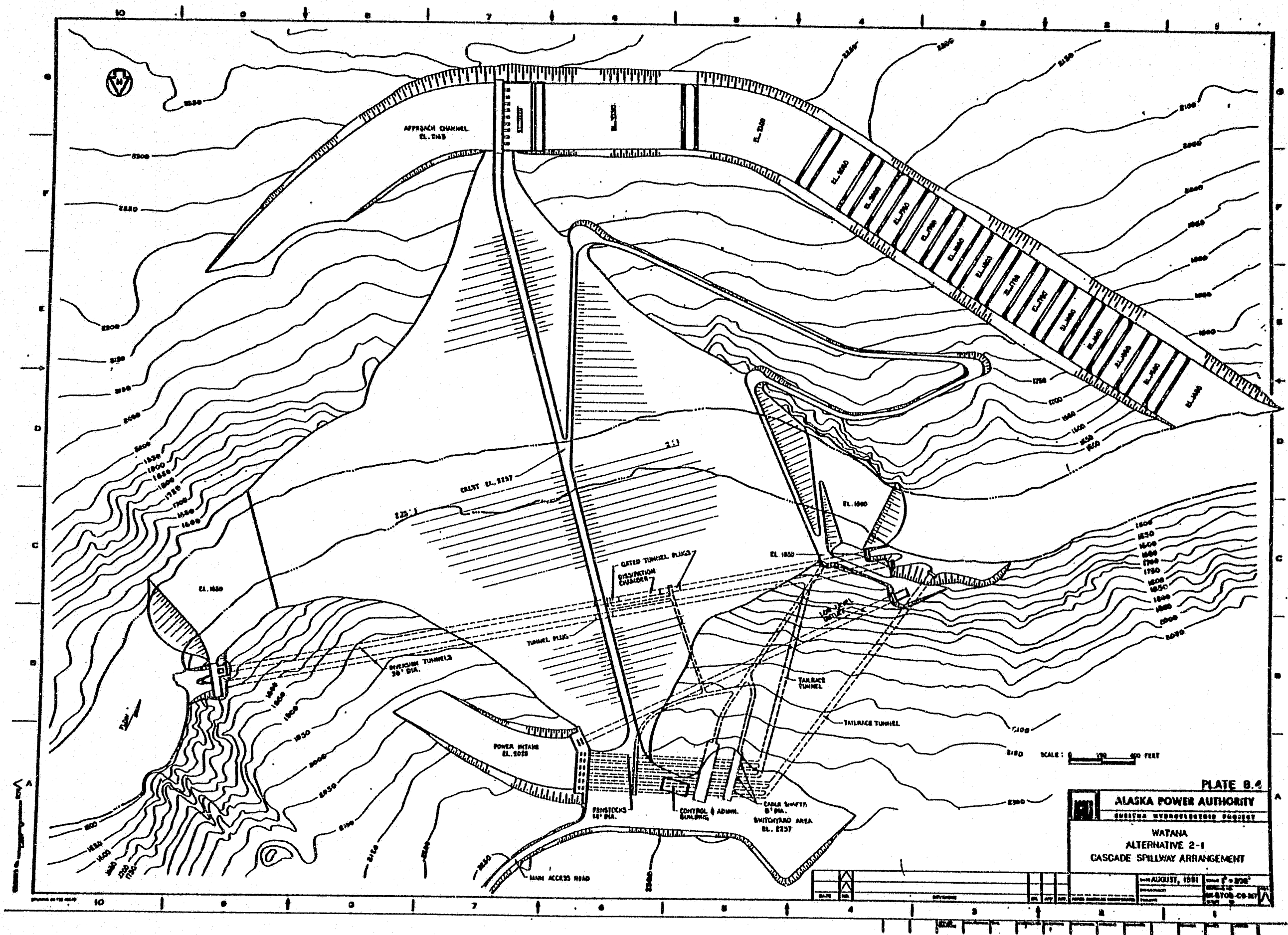
PLATE 8.1

ALASKA POWER AUTHORITY
CHITINA HYDROELECTRIC PROJECT

WATANÁ
VALVE TYPE SPILLWAY ALTERNATIVE
GENERAL ARRANGEMENT

DATE: JUNE 1958
SCALE: 1" = 500'
PROJECT: 66-8700-08.21

DATE	REV.	BY	CHKD.	APPROVED



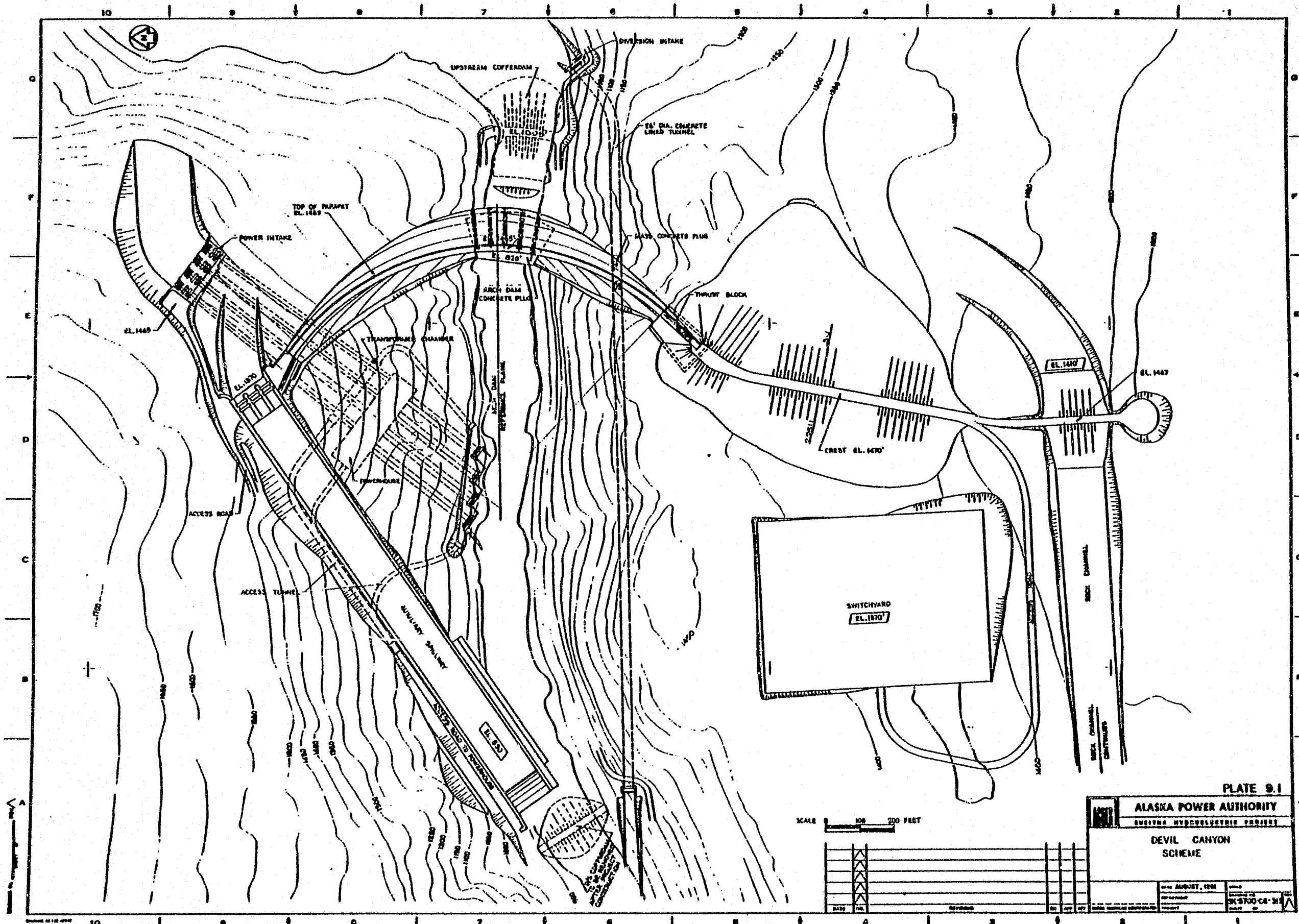


PLATE 9.1

ALASKA POWER AUTHORITY

SWITCHING HYDROELECTRIC PROJECT

DEVIL CANYON SCHEME

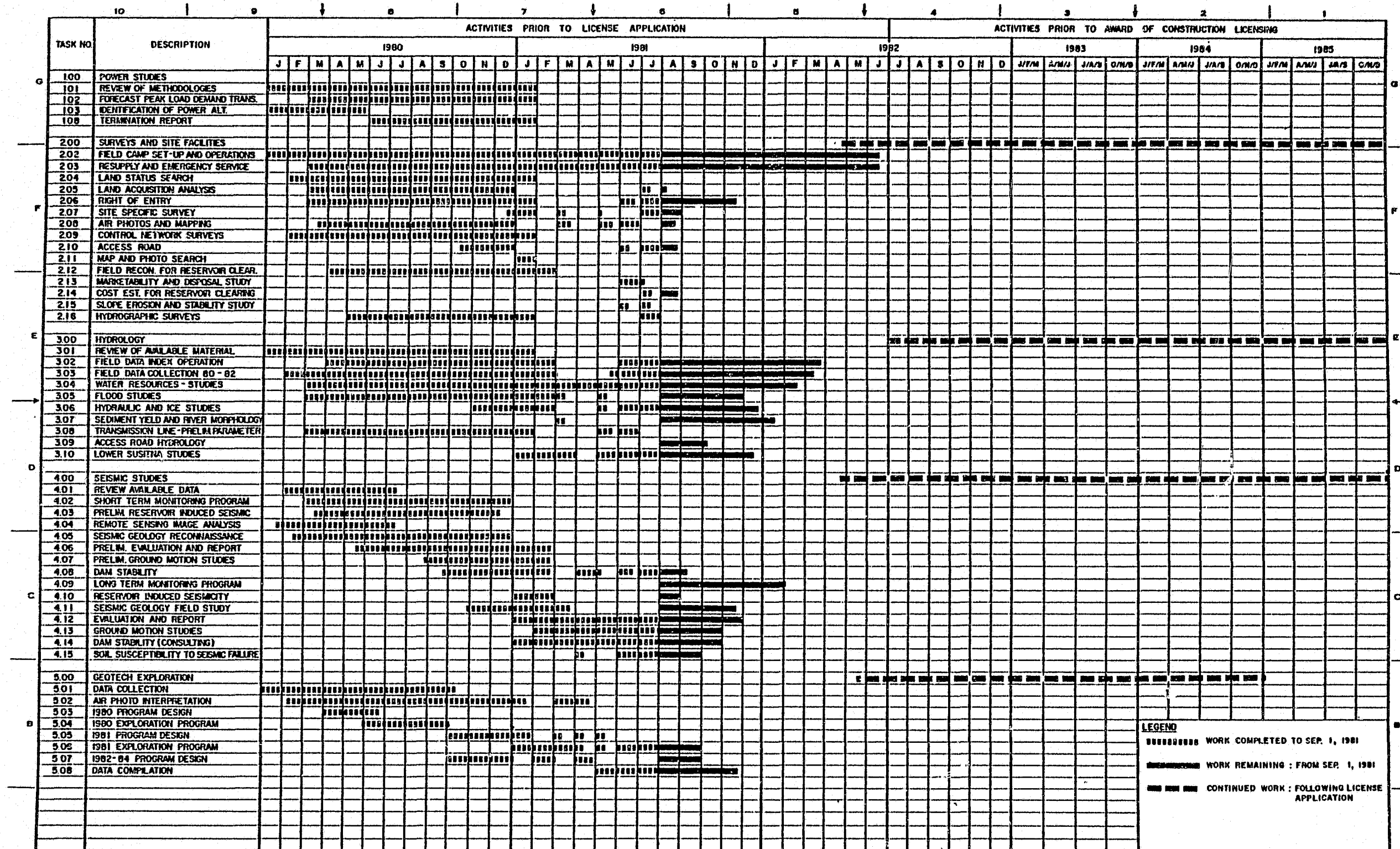
DATE: AUGUST, 1961

BY: [Signature]

CHK: [Signature]

- DEADLINES -

- FINAL DEVELOPMENT SELECTION REPORT - SEP.'81
- REVIEW OF WCC FIELD WORK - SEP.'81
- REVIEW OF ACRES/R&M FIELD WORK - SEP.'81
- APA EXTERNAL PANEL REVIEW - OCT.'81
- PRELIMINARY DEVIL CANYON ESTIMATE - OCT.'81
- PRELIMINARY WATANA ESTIMATE - NOV.'81
- FINAL GEOTECH. REPORT - FEB.'82
- FIRST DRAFT FEASIBILITY REPORT - FEB.'82
- FINAL DRAFT FEASIBILITY REPORT - MAR. 15, '82
- FERC LICENSE DOCUMENTS - MAY '82



SUPPORT MATERIAL TO TASK 3
HYDROLOGY

Water Resources of the Susitna River Basin

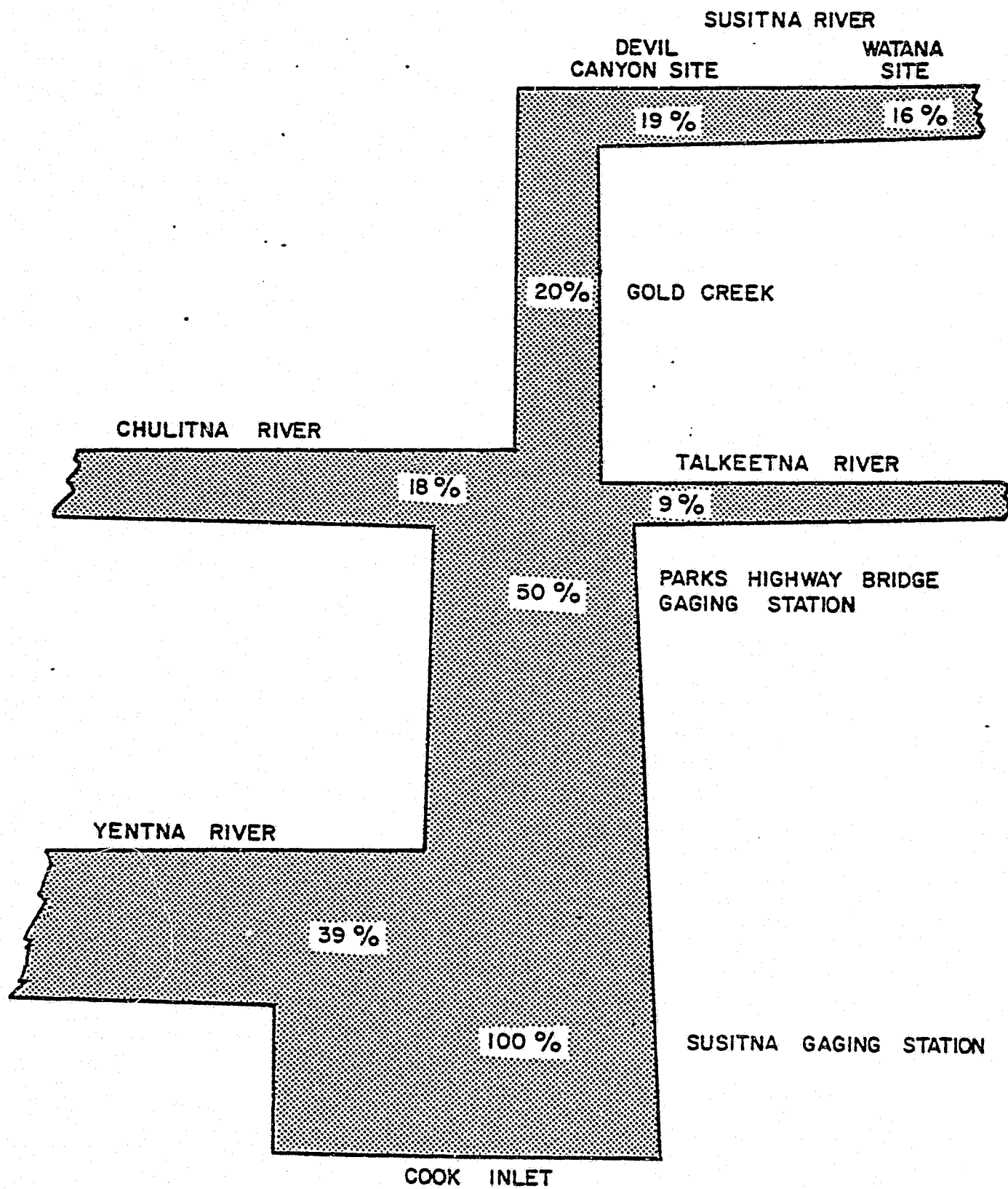
Water Resources of the Susitna River Basin

Streamflow data has been recorded by the USGS for a number of years at a total of 12 gaging stations on the Susitna River and its tributaries. The length of these records varies from 30 years at Gold Creek to about five years at the Susitna station. There are no historical records of streamflow at any of the proposed dam sites. For current study purposes, available streamflow records have been extended to cover the full 30 year period using a multisite correlation technique to fill the gaps in flow data at each of the stations. Flow sequences at the dam sites have subsequently been generated for the same 30 year period by extrapolation on the basis of drainage basin areas.

A gaging station was established at the Watana dam site in June 1980 and continuous river stage data is being collected. It is proposed to develop a rating curve at the station with streamflow measurements taken during the 1980 and 1981 seasons. River flows will be calculated and used to check the extrapolated streamflow data at the Watana site.

Seasonal variation of flows is extreme and ranges from very low values in winter (October to April) to high summer values (May to September). For the Susitna River at Gold Creek, the average winter and summer flows are 2100 and 20,250 cfs respectively, i.e., a 1 to 10 ratio. On average, approximately 88 percent of the streamflow recorded at Gold Creek station occurs during the summer months. At higher elevations in the basin, the distribution of flows is concentrated even more in the summer months. For the Maclaren River near Paxson (E1 4520 ft) the average winter and summer flows are 144 and 2100 cfs respectively, i.e., a 1 to 15 ratio.

The Susitna River above the confluence with the Chulitna River contributes only approximately 20 percent of the mean annual flow measured near Cook Inlet (at Susitna station). Attached figure shows how the mean annual flow of the Susitna increases towards the mouth of the river at Cook Inlet. Flow duration curves for the Watana and Devil Canyon dam sites for natural and post-project conditions are attached along with natural streamflow in the river at Gold Creek gaging station.



AVERAGE ANNUAL FLOW DISTRIBUTION
WITHIN THE SUSITNA RIVER BASIN

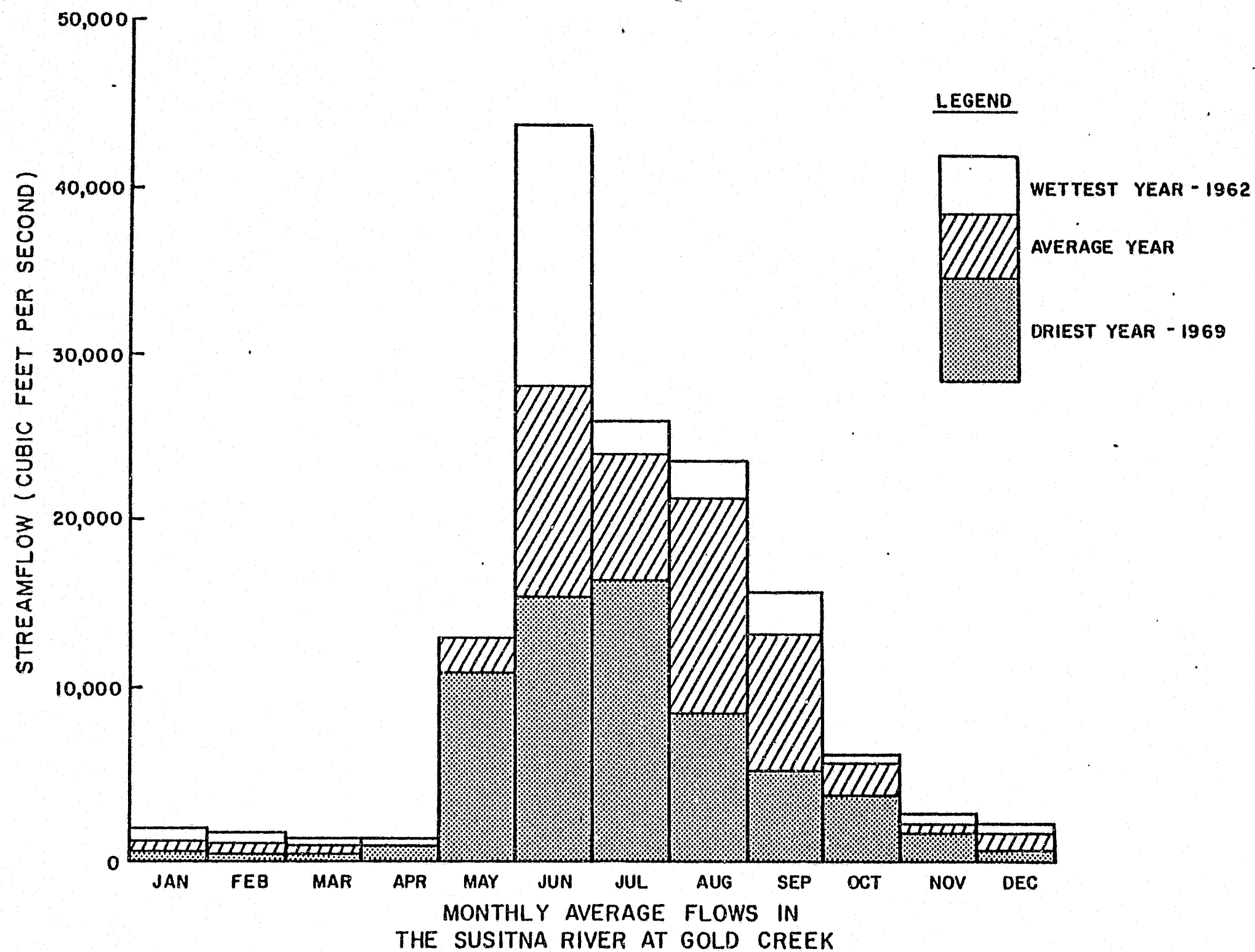
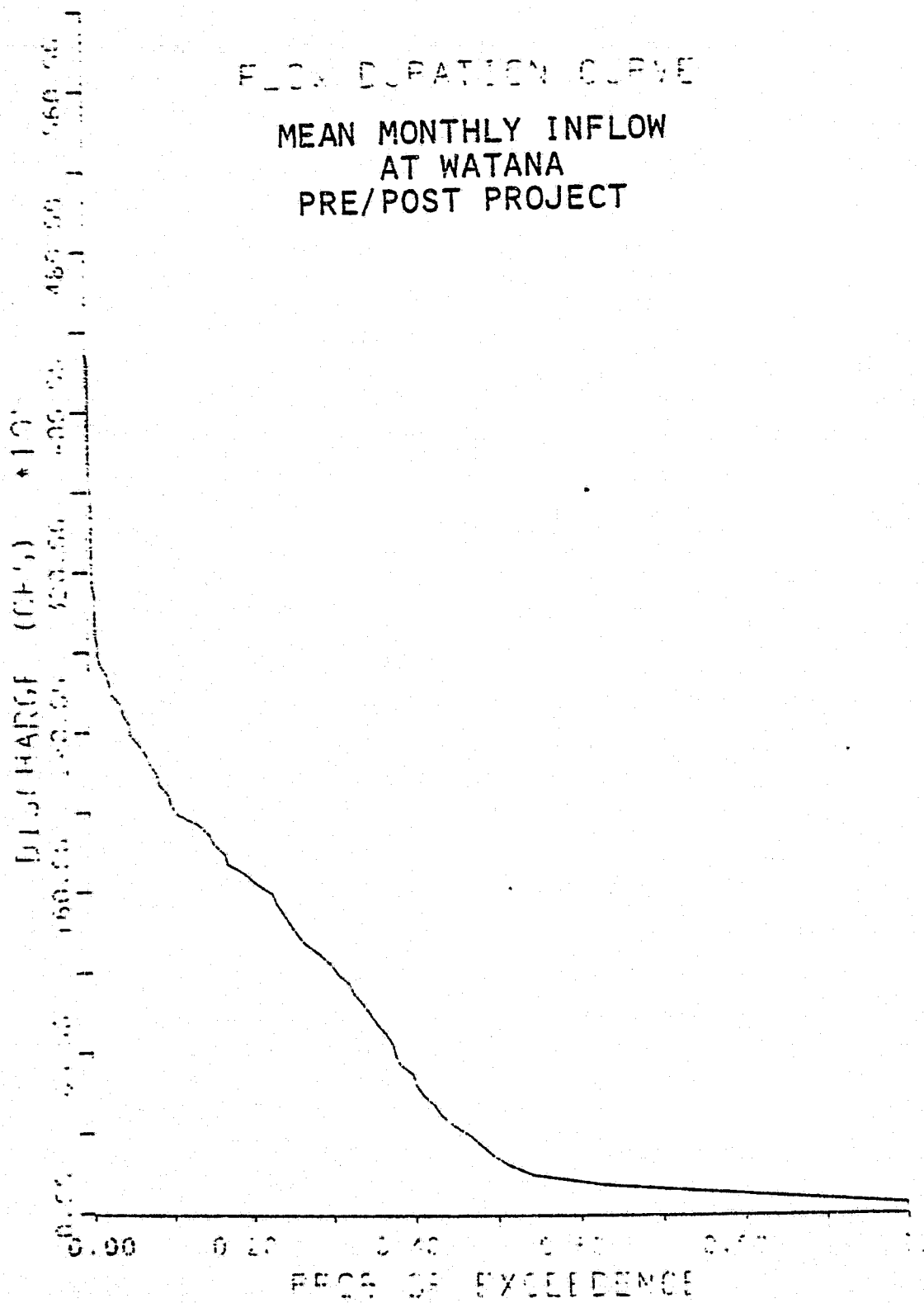
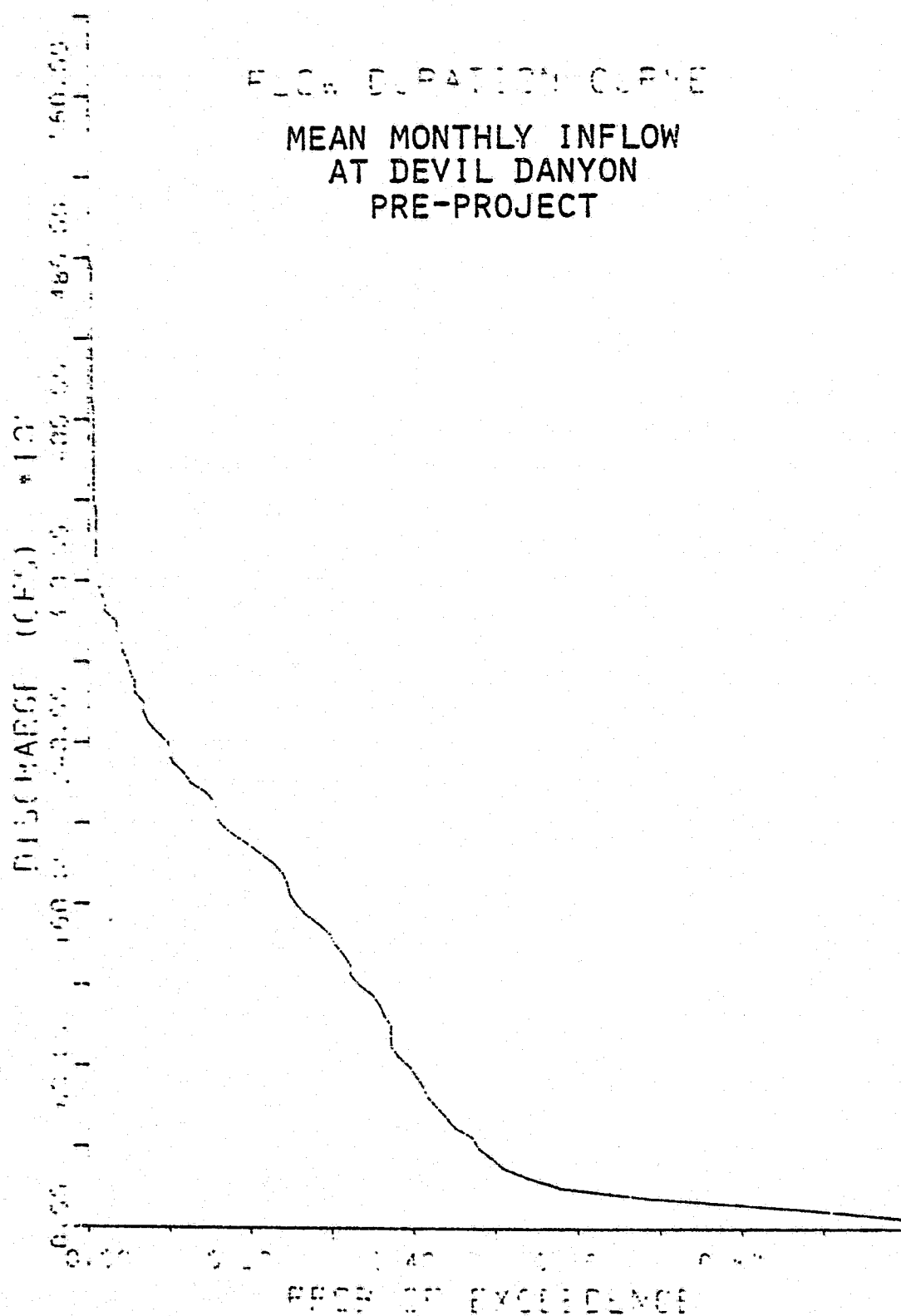


FIGURE 7.3

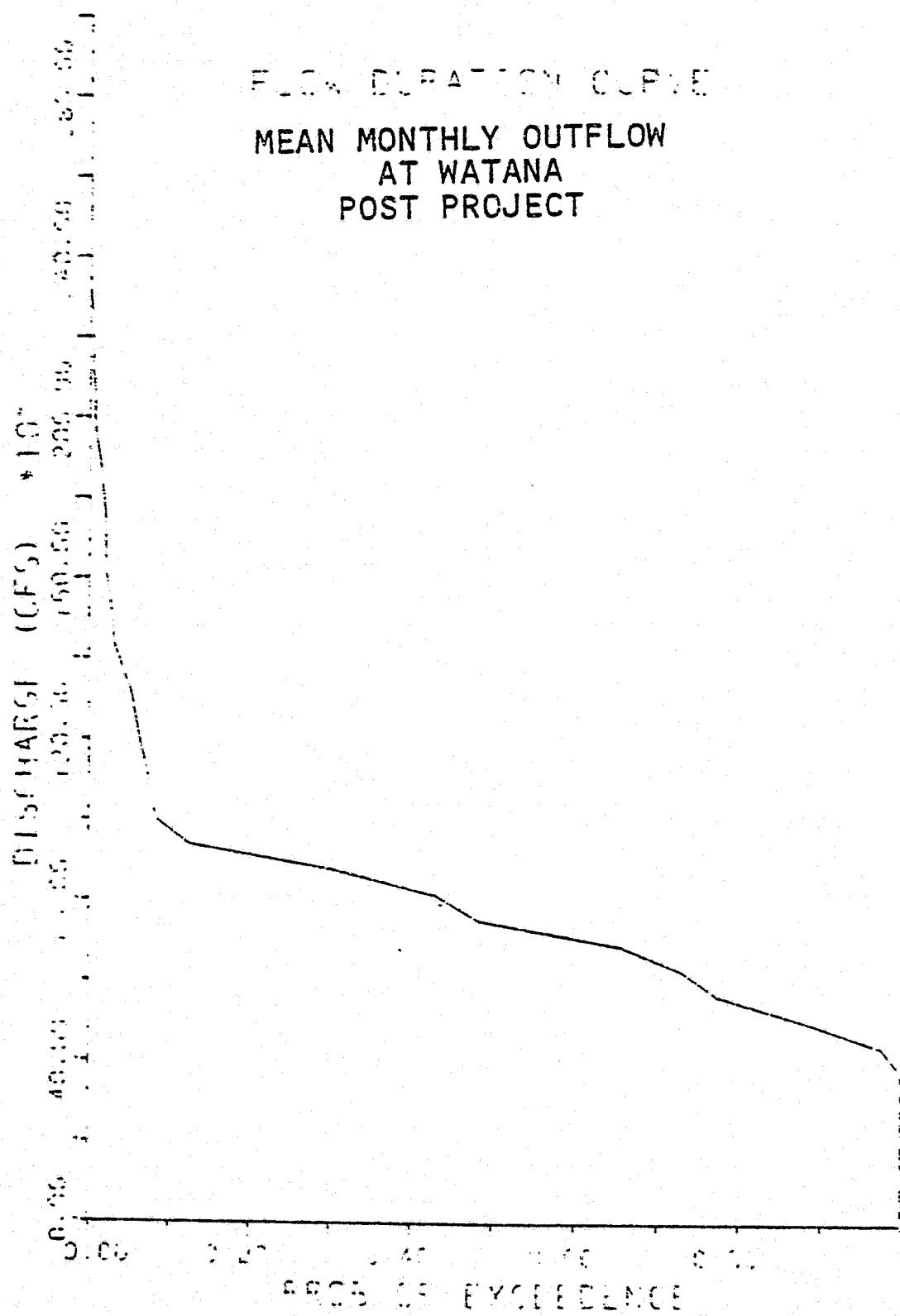
FLOW DURATION CURVE
 MEAN MONTHLY INFLOW
 AT WATANA
 PRE/POST PROJECT



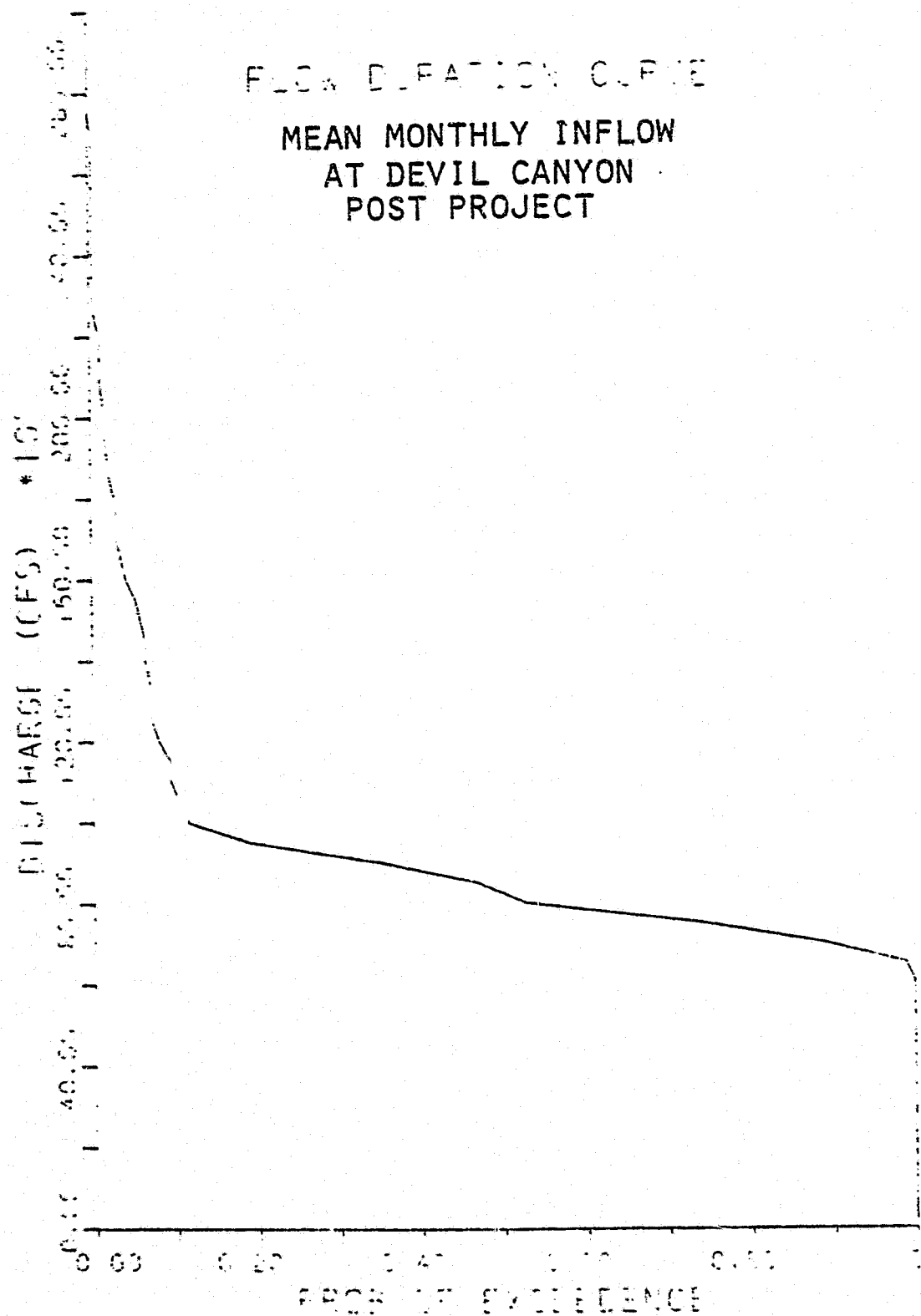
FLOW DURATION CURVE
MEAN MONTHLY INFLOW
AT DEVIL DANYON
PRE-PROJECT



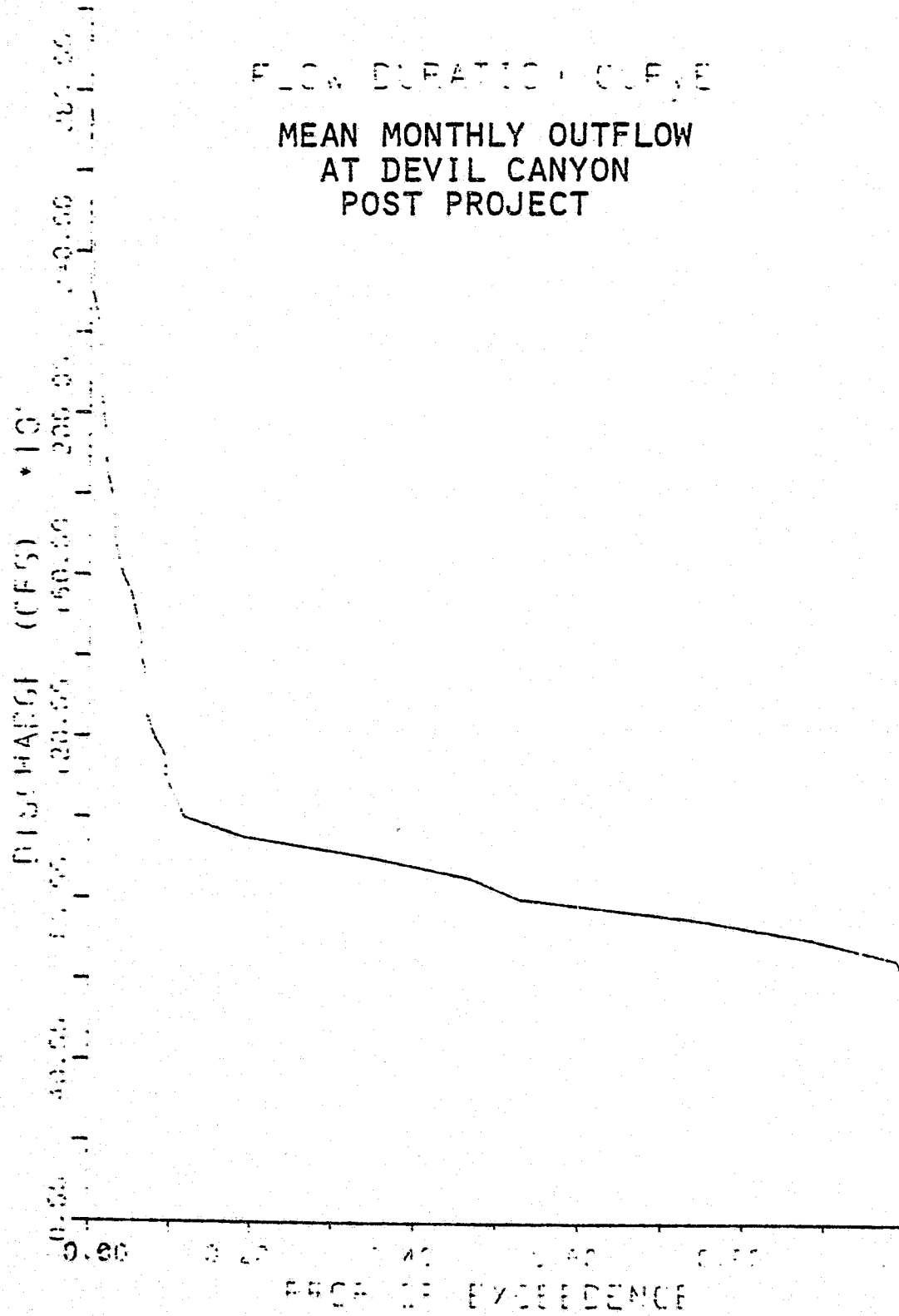
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FLOW DURATION CURVE
MEAN MONTHLY INFLOW
AT DEVIL CANYON
POST PROJECT



FLOW DURATION CURVE
 MEAN MONTHLY OUTFLOW
 AT DEVIL CANYON
 POST PROJECT



Regional Flood Frequency Analysis

Regional Flood Frequency Analysis

The objective of this study is to provide design flood peak information for the design of the project and for assessing pre- and post-project flood conditions in the Susitna River reaches located downstream and upstream from the proposed Watana and Devil Canyon dam sites. Within this context, two types of floods were studied: the largest annual floods and the largest annual floods during ice conditions (October-May). Procedures were developed to estimate the annual instantaneous peak and the October-May instantaneous peak for selected frequencies of occurrence on ungaged rivers within the upper Susitna River basin. Procedures were also developed to estimate the error associated with estimates made by the above mentioned procedures. Typical hydrographs were developed indicating flood shape, peak, and volume for selected frequencies of occurrence. Flood volume-duration frequency curves were also developed for the May-July period on the Susitna River at Gold Creek. Selected results are presented in the attached figures.

susi5/n1

TABLE 3.8
PHYSIOGRAPHIC AND CLIMATIC PARAMETERS¹

Station Name	Location	Drainage Area (sq. mi.)	Main Channel Slope (ft./mi.)	Stream Length (mi.)	Mean Basin Elevation (ft.)	Area of Lakes & Ponds (%)	Area of Forests (%)	Area of Glaciers (%)	Mean Annual Precipitation (in.)	Precipitation Intensity (in.)	Mean Annual Snowfall (in.)	Mean Minimum January Temperature (°F)
Susitna R.	at Gold Creek	6,160	10.2	189.0	3420.0	1.0	7.0	5.0	29.0	2.0	200.0	-4.0
Caribou Cr.	nr. Sutton	289	13.6	30.0	4190.0	0.0	10.0	0.0	28.0	1.5	80.0	2.0
Malanuska R.	at Palmer	2,070	79.7	77.0	4000.0	0.0	14.0	12.0	35.0	1.5	80.0	4.0
Susitna R.	nr. Denali	950	56.6	51.0	4510.0	1.0	1.0	25.0	60.0	2.0	400.0	-6.0
MacLaren R.	nr. Paxson	280	133.0	23.0	4520.0	1.0	0.0	19.0	55.0	1.5	400.0	-6.0
Susitna R.	nr. Cantwell	4,140	10.0	107.0	3560.0	2.0	5.0	7.0	32.0	1.5	200.0	-4.0
Chulitna R.	nr. Talkeetna	2,570	23.0	87.0	3760.0	1.0	22.0	27.0	55.0	1.6	250.0	-5.0
Talkeetna R.	nr. Talkeetna	2,006	35.0	90.3	3630.0	0.0	25.0	7.0	70.0	2.5	150.0	-2.0
Montana Cr.	nr. Montana	164	114.0	25.0	1930.0	3.0	54.0	0.0	40.0	2.2	90.0	0.0
Skwentna R.	nr. Skwentna	2,250	30.6	98.0	2810.0	5.0	34.0	16.0	43.0	2.0	140.0	-5.0
Tonsina R.	at Tonsina	420	71.0	46.0	3600.0	4.0	27.0	11.0	25.0	2.0	180.0	-2.0
Copper R.	nr. Chitina	20,600	14.4	178.0	3620.0	3.0	22.0	17.0	37.0	2.0	120.0	-4.0

¹ Values in this table are from the report entitled "Flood Characteristics of Alaska Streams" by Lanake (1979).

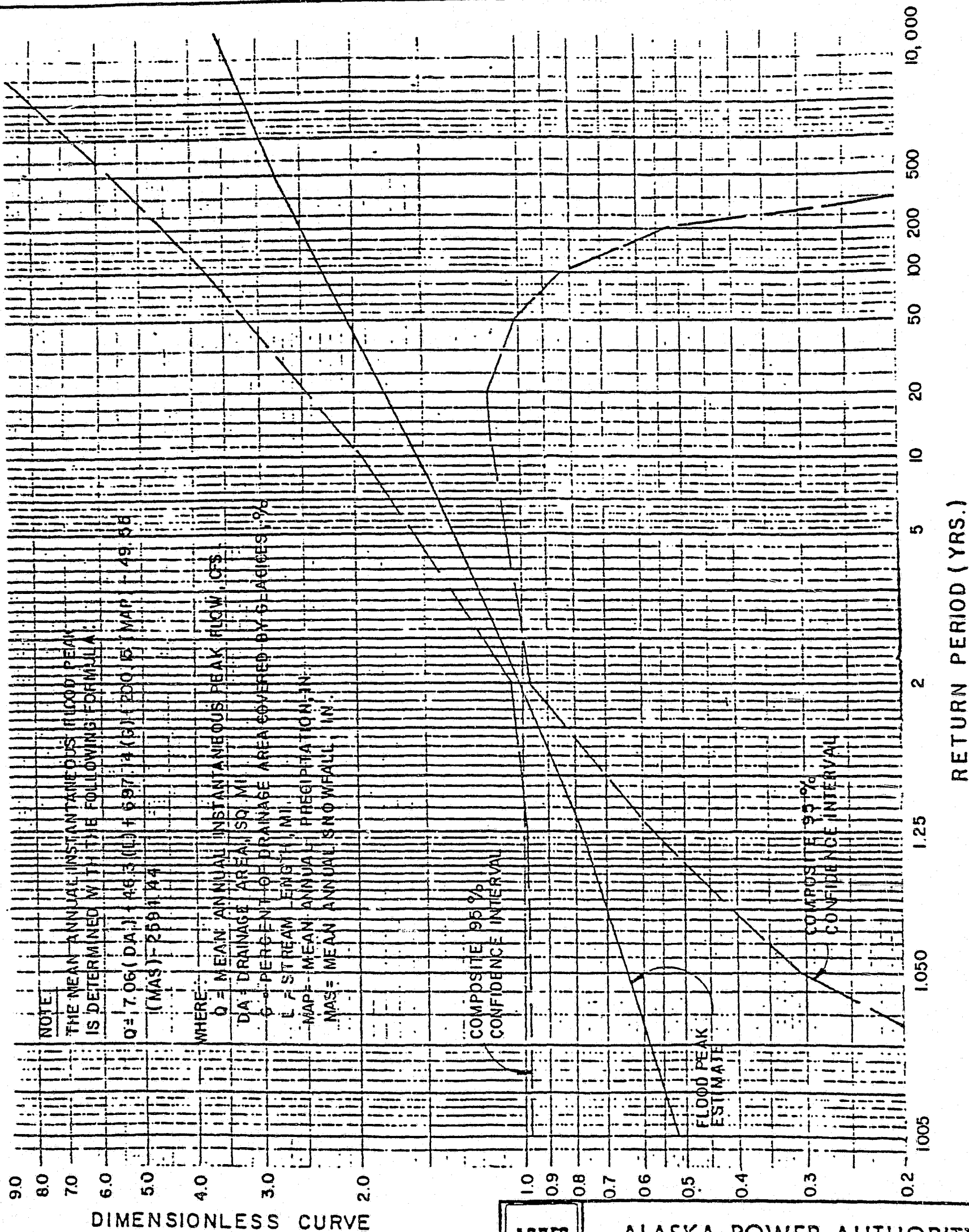


FIGURE 3.3

DESIGN DIMENSIONLESS
REGIONAL FREQUENCY CURVE

ACRES

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

FLOOD STUDIES

ANNUAL INSTANTANEOUS
FLOOD PEAKS

RSM

RSM CONSULTANTS, INC.

MARCH 1981

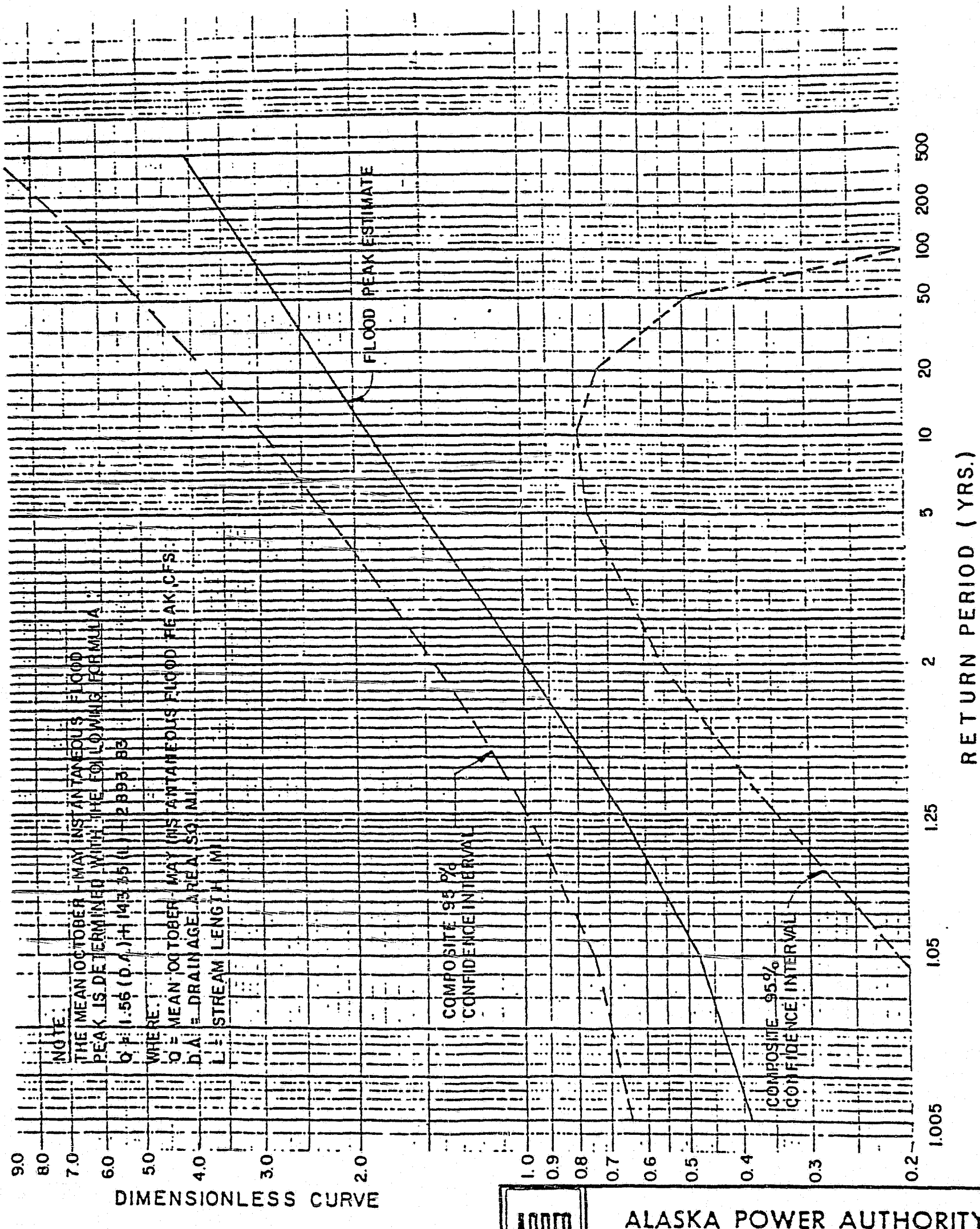
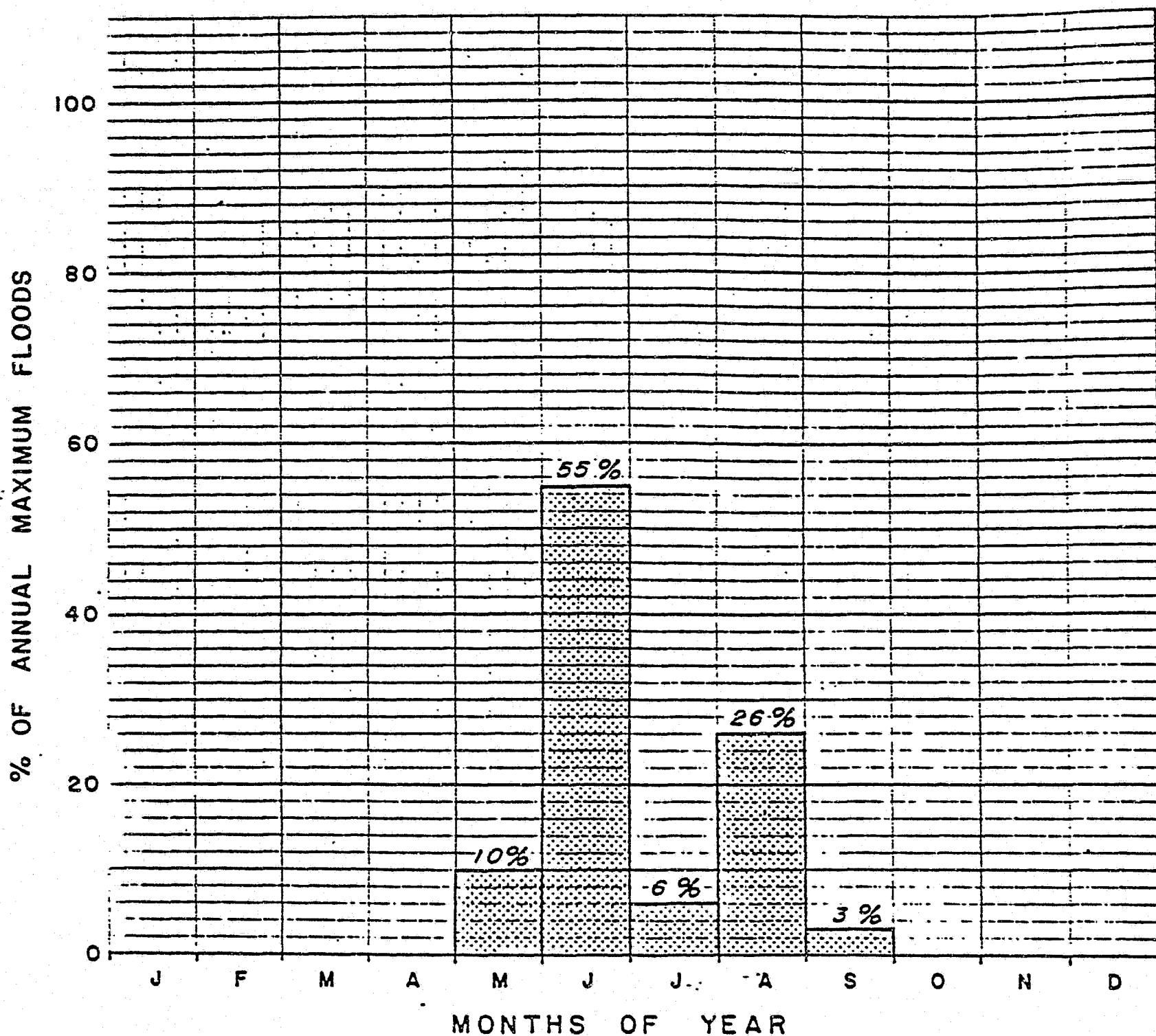


FIGURE 3.4
 DESIGN DIMENSIONLESS
 REGIONAL FREQUENCY CURVE

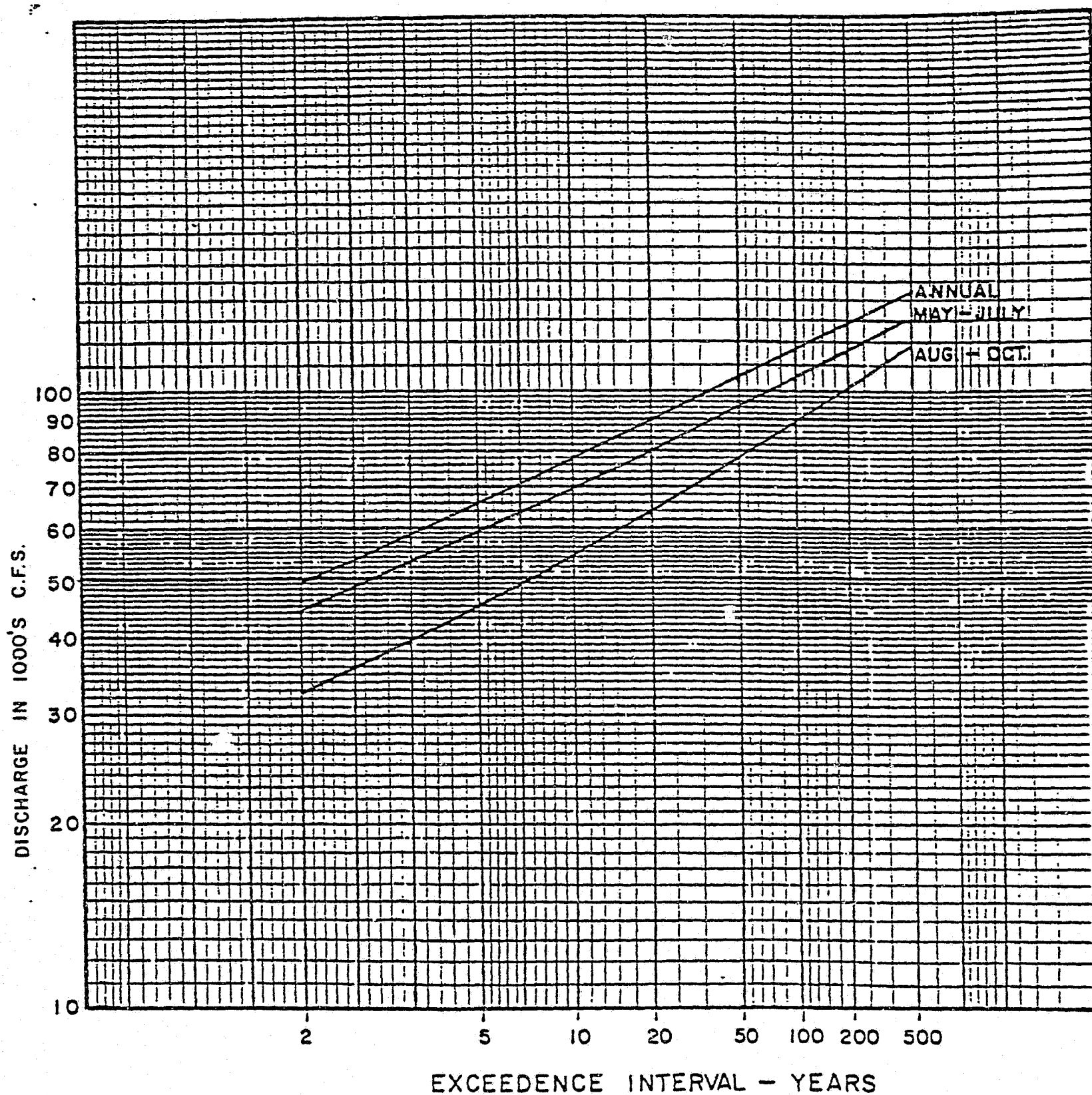
APAES	ALASKA POWER AUTHORITY	
	SUSITNA HYDROELECTRIC PROJECT	
FLOOD STUDIES		
OCTOBER-MAY INSTANTANEOUS FLOOD PEAKS		
R&M	R&M CONSULTANTS, INC.	MARCH 1981
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS		



SUSITNA RIVER AT GOLD CREEK
PERIOD OF RECORD - 1950 - 1980

FIGURE 5.1

	ALASKA POWER AUTHORITY	
	SUSITNA HYDROELECTRIC PROJECT	
FLOOD STUDIES PERCENT OF ANNUAL MAXIMUM FLOODS		
 R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	MARCH 1981	



SUSITNA RIVER AT GOLD CREEK

PERIOD OF RECORD - 1950-1980

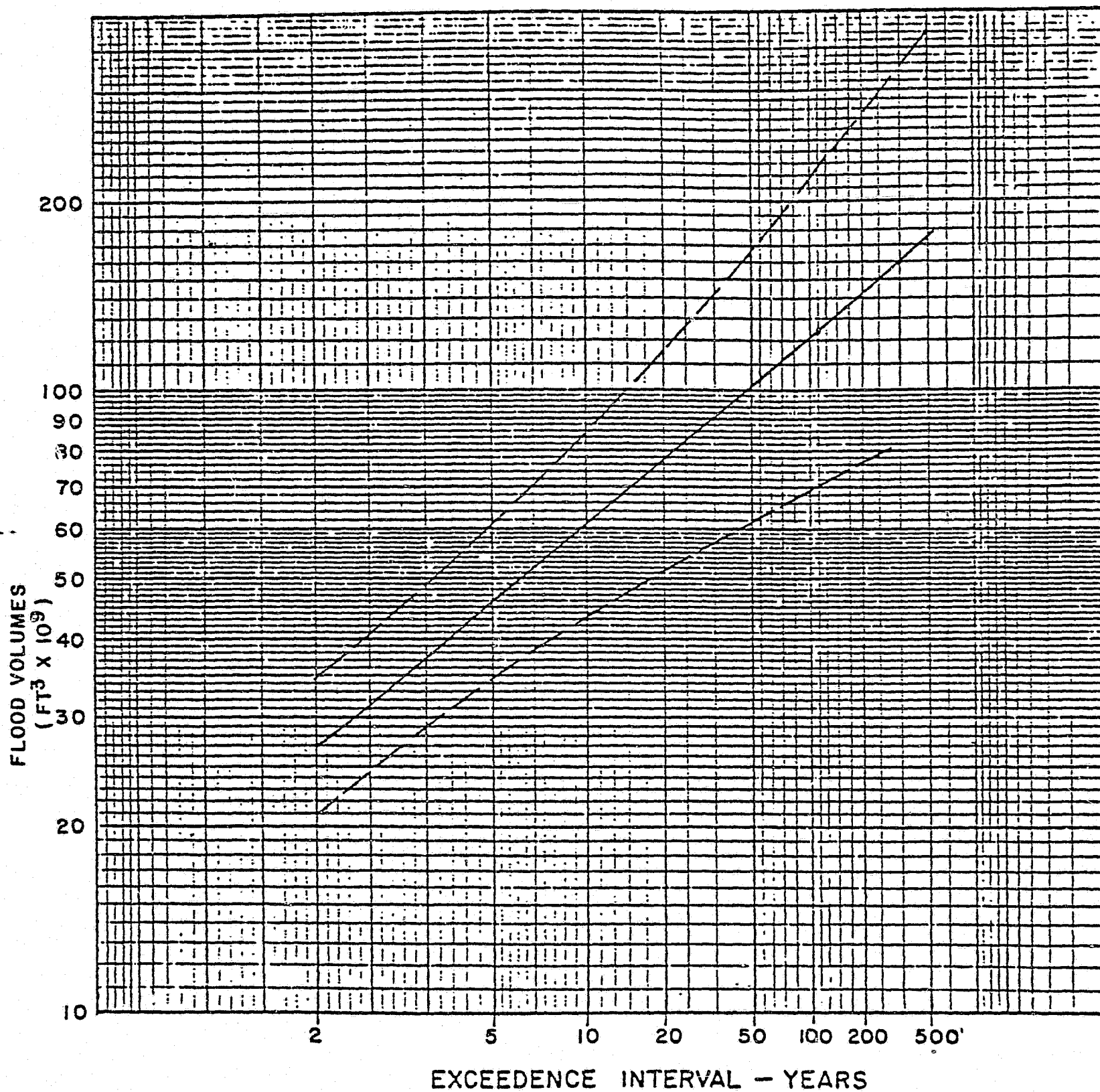
ANNUAL SKEW IS 0.6830

MAY-JULY SKEW IS 1.130

AUG-OCT SKEW IS 1.134

FIGURE 5.4

APRES	ALASKA POWER AUTHORITY	
	SUSITNA HYDROELECTRIC PROJECT	
FLOOD STUDIES		
SEASONAL DISCHARGE		
FREQUENCY CURVES		
R&M CONSULTANTS, INC.		MARCH 1981



SUSITNA RIVER AT GOLD CREEK

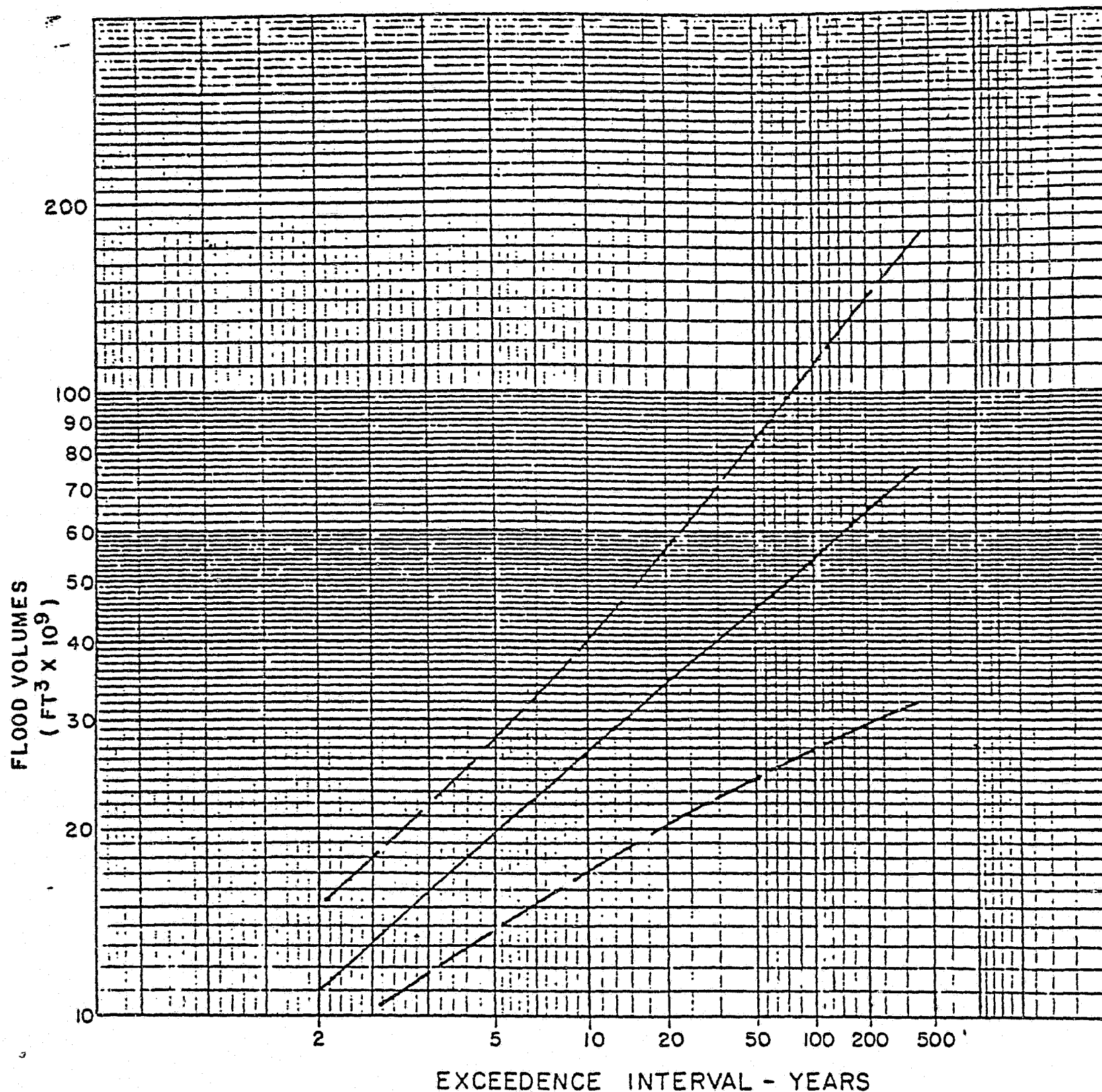
PERIOD OF RECORD - 29 yrs.

LEGEND

- 95 % CONFIDENCE LIMITS
- VOLUME FREQUENCY CURVE

FIGURE 5.5

	ALASKA POWER AUTHORITY	
	SUSITNA HYDROELECTRIC PROJECT	
FLOOD STUDIES		
FLOOD VOLUME FREQUENCY CURVE		
MAY - JULY		
		MARCH 1981
R&M CONSULTANTS, INC.		
ENGINEERS GEOLOGISTS PLANNERS SURVEYORS		



SUSITNA RIVER AT GOLD CREEK

PERIOD OF RECORD - 21 yrs.

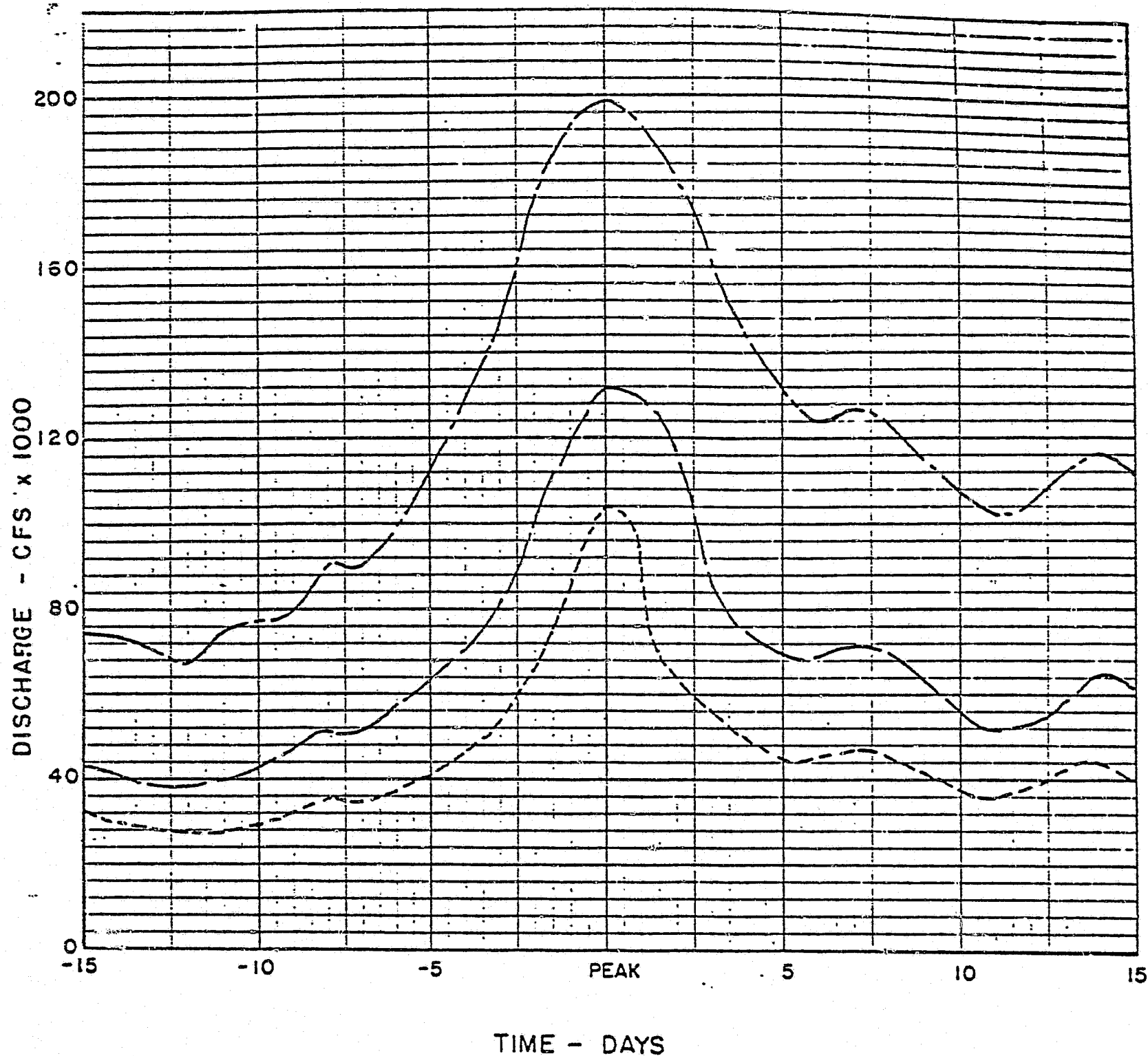
LEGEND

----- 95 % CONFIDENCE LIMITS

———— VOLUME FREQUENCY CURVE

FIGURE 5.6

	ALASKA POWER AUTHORITY	
	SUSITNA HYDROELECTRIC PROJECT	
FLOOD STUDIES		
FLOOD VOLUME FREQUENCY CURVE		
AUG - OCT		
 RSM CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	MARCH 1981	



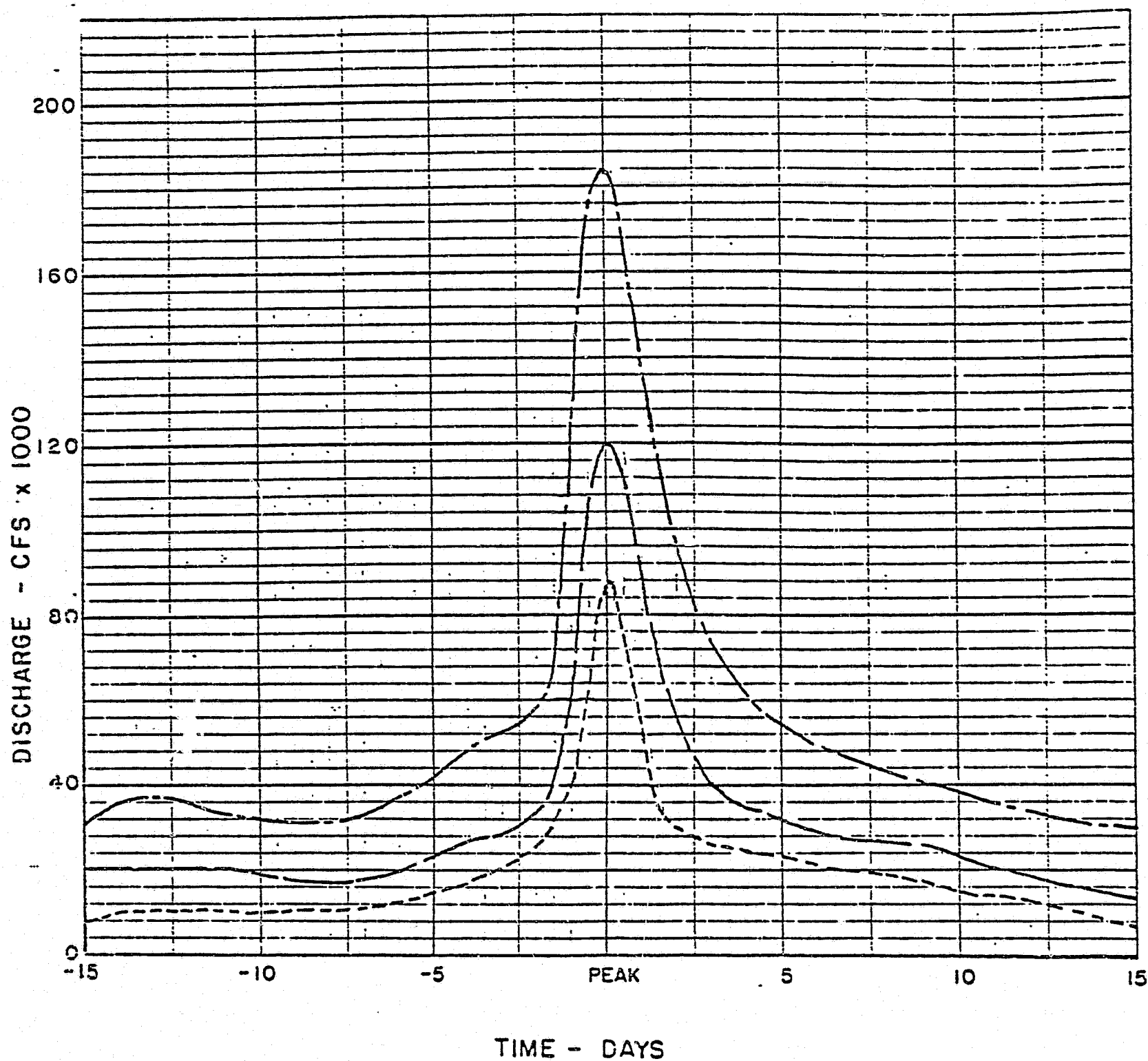
SUSITNA RIVER AT GOLD CREEK
100, 500, 10000 yr. FLOOD VOLUMES

FIGURE 5.9

LEGEND

	Flood Volume ft ³	Peak Discharge (cfs)
-----100 yr	122.3 X 10 ⁹	104,550
———500 yr	178.2 X 10 ⁹	131,870
-----10,000 y	310.0 X 10 ⁹	198,000

	ALASKA POWER AUTHORITY	
	SUSITNA HYDROELECTRIC PROJECT	
FLOOD STUDIES FLOOD HYDROGRAPHS MAY - JULY		
 R&M CONSULTANTS, INC. ENGINEERS GEOLOGISTS PLANNERS SURVEYORS	MARCH 1981	



SUSITNA RIVER AT GOLD CREEK

FIGURE 5.10

LEGEND

	Flood Volume ft ³	Peak Discharge (cfs)
----- 100 yr	53.8 X 10 ⁹	90,140
———— 500 yr	78.8 X 10 ⁹	119,430
----- 10,000 yr	140.0 X 10 ⁹	185,000

	ALASKA POWER AUTHORITY	
	SUSITNA HYDROELECTRIC PROJECT	
FLOOD STUDIES		
FLOOD HYDROGRAPHS		
AUG - OCT		
	RSM CONSULTANTS, INC.	MARCH 1981
ENGINEERS GEODETIC SURVEYORS		

Probable Maximum Flood Studies

Probable Maximum Flood Studies

The original scope of Acres work as described in the Plan of Study of February 1980, comprised a review of the PMF estimates made by the Corps of Engineers in their 1975 study. The assessment determined the sensitivity of the PMF estimates to changes in critical meteorological and basin parameters. The magnitude of these changes were considerably significant and the estimates could not be used in the design of facilities without more detailed study.

The meteorological data used in the COE estimates were developed by the National Weather Service (NWS) in a preliminary study which gave a general range of criteria within which it was believed values from a more comprehensive study would fall. In their conclusions to the study, the NWS noted . . . "Time hasn't allowed checks, evaluation, and comparison of the several types of data summarized here". The NWS naturally recommended further study. This is borne out by the increases to the PMF peak found in the sensitivity analysis.

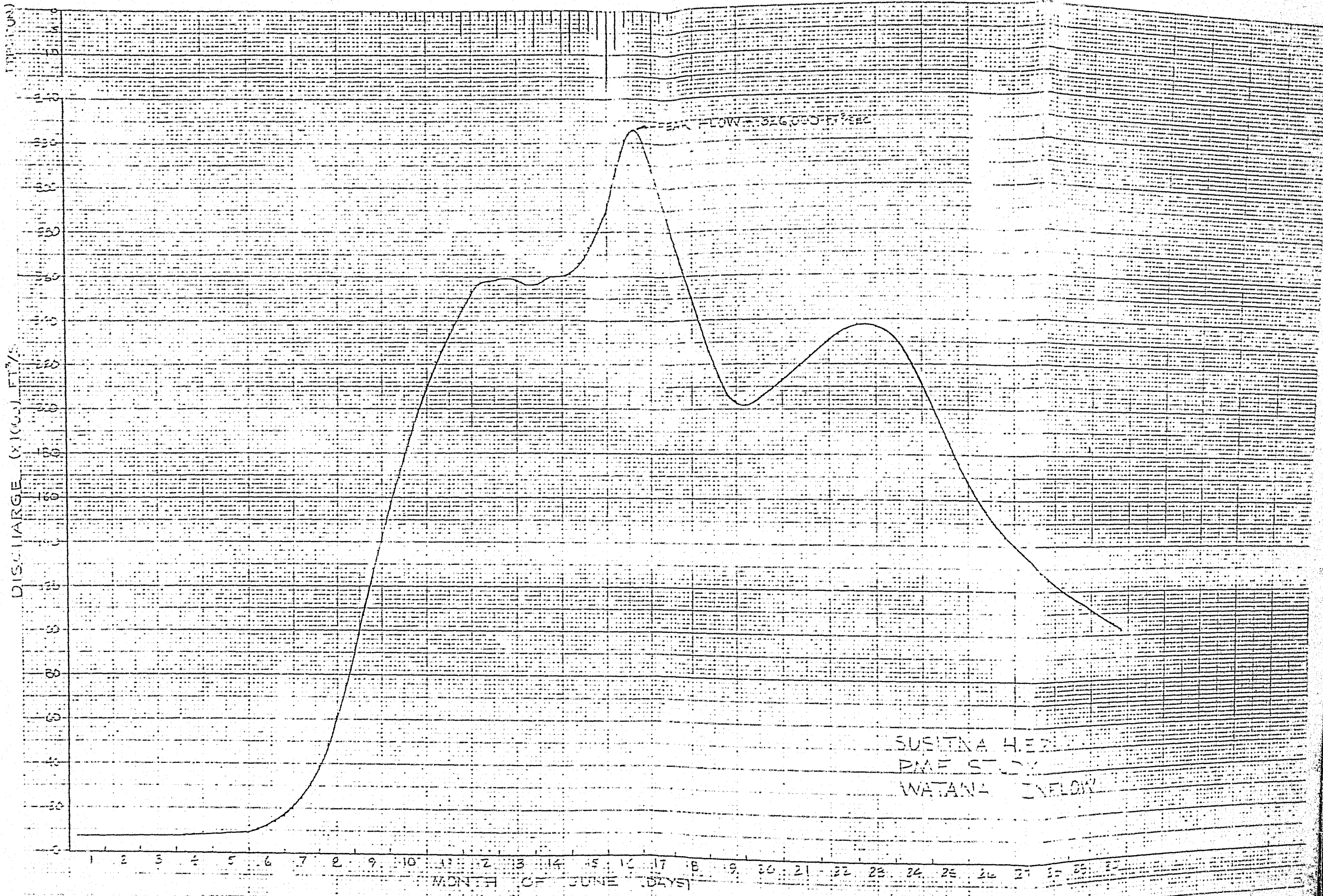
It was decided that a re-evaluation of the PMF is necessary to define flood discharge capacities of project facilities which presented an extension of the scope of Acres studies. The approach to the re-evaluation of the PMF is described below.

The approach entailed re-assessing precipitation maximums, temperature gradients and temperature maximums based on a thorough study of the meteorological characteristics of the Susitna River Basin. Applicable storm maximization techniques were used to develop a probable maximum storm for both spring and summer seasons.

Paralleling the climatological study, a further calibration of the SSARR model was undertaken to develop a reasonable watershed model based on procedures that follow generally accepted mathematical modeling techniques. The calibration started with assuming that the basin's meteorological and hydrological parameters used in the COE estimates of the PMF are the most representative. These parameters were adjusted as analysis progressed.

When the set of watershed parameters that gave the most reliable estimation of spring and summer floods were determined, a verification study was conducted using this data set. Several floods that were independent of the floods used in the calibration study were used in the verification process which determined the accuracy that can be reasonably expected from the SSARR model.

Estimates of the probable maximum flood at critical locations along the Susitna River for both spring and summer were determined using climatological data developed and the most reliable set of basin parameters. Synthesized PMF inflow hydrograph at Watana is attached.



Project Flood Flows

SUSITNA HYDROELECTRIC PROJECT

PROJECT FLOOD FLOWS

From the regional frequency analysis and the PMF study, a set of project flood flows were determined for design of discharge facilities. These are presented in the attached tables. Assumed reservoir levels at the start of flood routing and facilities available to discharge design floods are listed.

SITNA HYDROELECTRIC PROJECT

PROJECT FLOWS

TABLE 1 - RIVER FLOWS (FT³/S)

FLOW RETURN PERIOD YEARS	WATANA		DEVIL CANYON*	
	ANNUAL PEAK	SUMMER PEAK	ANNUAL PEAK	SUMMER PEAK
MEAN ANNUAL	36,000	-	41,000 ^{1/}	-
1 IN 50	84,000	-	53,000	-
1 IN 100	92,000	70,000	54,000	-
1 IN 10,000 (DESIGN)	156,000	-	140,000	-
PMF	326,000	-	320,000	-

WITH WATANA DEVELOPMENT UPSTREAM.

^{1/} NATURAL FLOW.

TABLE 2 - ROUTED FLOWS FOR DISCHARGE FACILITIES (FT³/S)

<u>RETURN PERIOD YRS.</u>	<u>WATANA</u>	<u>DEVIL CANYON</u>
1:50	68,000	53,200
1:100 ANNUAL	45,000	50,000
1:100 SUMMER	45,000	50,000
1:10,000	120,000	140,000
PMF	270,000	320,000

FIGURES TENTATIVE BASED ON SPECIFIC GATE SIZES AND OPERATION .
OPTIMIZATION STUDIES UNDERWAY.

[illegible]

Spillway Design Criteria

SPILLWAY DESIGN CRITERIA

A. Watana Development

1. Spillway facility comprises:

- (a) Service Spillway - tunnel outlet with H.B. valves
- (b) Auxiliary Spillway - chute with flip bucket
- (c) Emergency Spillway - fuse plug and rock channel

Discharge Capacity

- up to 1:100 yr flood
- 1:100 to 1:10,000 yr flood (project design flood)
- 1:10,000 yr to PMF

2. Spillway - Availability of Facilities

Flood Return Period

- (a) 1:100 yr (spring/summer)
- (b) 1:10,000 yr
- (c) PMF

Facilities Available

- service spillway and 75% powerhouse flow
- service spillway and auxiliary spillway-powerhouse not available
- service, auxiliary and emergency spillways - powerhouse not available

3. Starting Reservoir Levels for Flood Routing

1:100 yr spring flood	2172 ft
1:100 yr summer flood	2215 ft
1:10,000 yr design flood	2215 ft
PMF	2215 ft

- simulated maximum level at beginning of June
- normal maximum operating level
- normal maximum operating level
- normal maximum operating level

B. Devil Canyon Development

1. Spillway facility comprises:

- (a) Service Spillway - tunnel outlet with H.B. valves
- (b) Auxiliary Spillway - chute with stilling basin
- (c) Emergency Spillway - fuse plug and rock channel

Discharge Capacity

- up to 1:100 yr flood
- 1:100 to 1:10,000 yr flood (project design flood)
- 1:10,000 yr to PMF

2. Spillway - Availability of Facilities

Flood Return Period

- (a) 1:100 yr (spring/summer)
- (b) 1:10,000 yr
- (c) PMF

Facilities Available

- service spillway and 75% powerhouse flow
- service and auxiliary spillways - powerhouse not available
- service, auxiliary and emergency spillways - powerhouse not available

3. Starting Reservoir Levels For Routing

All Floods

1455 ft (normal maximum operating level)

SUPPORT MATERIAL TO TASK 4
SEISMIC STUDIES

TASK 4 - SEISMIC STUDIES

1 - Introduction

Seismic studies are being performed by Woodward-Clyde Consultants (WCC) under a subcontract to Acres. The studies were broken down in two stages: 1980 Activities and 1981 Activities. The results of 1980 Activities were presented in the "Interim Report on Seismic Studies for Susitna Hydroelectric Project, December 1980" by WCC and are summarized here for ready reference. 1981 activities are partially complete and preliminary conclusions drawn to date are included here along with the total Scope of Activities and the current status. Preliminary conclusions are considered to represent reasonably conservative seismic design parameters for the project studies and are not expected to change significantly as a result of ongoing studies.

2 - Summary of 1980 Activities

As a result of activities completed in 1980, it was concluded that the Talkeetna Terrain, in which the project is located, has the following boundaries: The Denali fault to the north and northeast; the Totschunda fault to the east; the Castle Mountain fault to the south; a broad zone of deformation and volcanoes to the west; and the Benioff zone at depth.

The Talkeetna Terrain was judged to be acting as a coherent tectonic unit within the present stress regime with major strain releases occurring along the boundary fault systems. Within the Terrain, strain release appears to be randomly occurring at depth within the crust. This strain release is possibly the result of crustal adjustments resulting from perturbation imposed by the Benioff zone and by stress (associated with plate motion) imposed along the Terrain margin through the Terrain.

Within the site region, 13 faults and lineaments were identified as requiring additional investigation to better define their potential effect on Project design considerations. These 13 faults and lineaments (designated significant features) were selected on the basis of their seismic source potential and potential for surface rupture through either site. Four of these features are in the vicinity of the Watana site (see Figure 4.1) and nine are in the vicinity of the Devil Canyon site (see Figures 4.2 and 4.3). Preliminary estimates of ground motions at the sites were made for the Denali and Castle Mountain faults and the Benioff zone. Of these sources, the Benioff zone was expected to govern the levels of peak horizontal ground acceleration, response spectra, and duration of strong shaking. The ground-motion estimates were preliminary in nature and did not constitute criteria for design of project facilities.

Further details of specific findings of the 1980 studies are presented in Section 6 of this summary. On the basis of these findings, a detailed scope of investigations for 1981 was developed and is currently in progress.

3 - Scope of 1981 Studies

The scope of 1981 activities was developed on the basis of the original Scope of Work included in the POS and results of 1980 Activities. A list of 18 objectives was formulated by WCC which formed the basis of discussions with Acres, Acres External Review Panel and APA Review Panel (Figure 4.6). The final Scope of Activities as agreed upon, is summarized as follows:

(a) Subtask 4.08 - Preliminary Dam Stability

Provide assistance in evaluation of dam stability during earthquake conditions.

(b) Subtask 4.09 - Seismology

Evaluate location and source(s) of moderate to large historical earthquakes within Talkeetna Terrain and north of Talkeetna Terrain to better define their source and impact on floating earthquake within the study areas. Evaluate available seismology data to determine stress regime within Talkeetna Terrain, refine MCE on Benioff zone and prepare long term seismologic network recommendations and operation manual.

(c) Subtask 4.10 - RIS

Evaluate RIS using results of 1980 and 1981 studies and its effect on the project.

(d) Subtask 4.11 - Seismic Geology

Evaluate 13 significant features identified during 1980 Activities (see Figures 4.1, 4.2 and 4.3) by performing remote sensing, Quarternary Geology, field mapping techniques, and determine if a feature is "Fault" and if it is a fault, with recent displacements, determine MCE along potential seismic sources.

(e) Subtask 4.12 - Report

Prepare final report to document these studies.

(f) Subtask 4.13 - Ground Motions

Refine attenuation relationship, determine earthquake design parameters for Watana and Devil Canyon sites, and perform seismic exposure analysis.

(g) Subtask 4.14 - Dam Stability

Provide assistance in evaluation of dam stability during earthquake.

(h) Subtask 4.15 - Transmission Line Seismicity

Evaluate ground motions along the transmission line, assess ground stability conditions along the transmission and access road corridors.

4 - Status of 1981 Activities

A schedule of the activities is shown on Figure 4.7. All activities are proceeding on schedule. Field geology studies were completed with the review of field data on September 2 and 3, 1981 at the site.

Completed activities are as follows:

- Aerial reconnaissance was performed for all 13 features.
- Field mapping was conducted at 300 locations.
- Thirteen ground magnetic surveys were conducted across the Talkeetna thrust fault, Susitna feature, and feature KD5-3 near the Devil Canyon site.
- One seismic refraction survey was conducted across the Talkeetna thrust fault.
- Low sun angle photography was flown (at a scale of 1:24,000) and interpreted.
- Existing color photography at a scale of 1:24,000 was interpreted.
- Approximately 2,500 ft of core in five borings which crossed the Watana River feature (KD3-7), the possible fault (KD-5-43) on the left abutment at Devil Canyon and the possible fault (DC-1) in Devil Canyon was reviewed.
- Three trenches (total length 450 ft), which were excavated within the zone of the Talkeetna thrust fault and the Susitna feature, were logged. The approximate location of these trenches is shown on Figure 4.8.
- Data for mines in the vicinity of the Talkeetna thrust fault (KC4-1), and features KD5-2 and KD5-3 in the vicinity of the Devil Canyon Dam site, were reviewed.
- Quaternary geology air photo and field mapping was conducted at more than 60 locations.
- Eleven samples were submitted for C^{14} dating.
- Two samples were submitted for K-Ar dating.
- Field review sessions were conducted with Bela Csejtey, George Plafker and Norm Ten Brink.
- Review of field activities and results was performed by WCC Internal Review Group on September 2 and 3, 1981 in the field.

5 - Preliminary Conclusions of 1981 Studies

The preliminary results presented below are subject to revision as analyses of the field studies and other data are conducted.

(a) There are three features of importance for seismic design of the Susitna Hydroelectric Project. These features are the Castle Mountain fault, Denali fault, and the Benioff zone.

(b) Of the thirteen features selected for study in 1981, the following table summarizes the conclusions:

<u>Feature Name</u>	<u>Feature No.</u>	<u>The Feature is a Fault</u>	<u>The Feature is an Active Fault</u>
WATANA SITE			
Talkeetna thrust fault	KC4-1	Yes	No
Susitna feature	KD3-3	No	No
Watana River Feature	KD3-7	No	No
Fins feature	KD4-27	Yes	No
DEVIL CANYON SITE			
-	KC5-5	Yes	No
-	KD5-2	Yes	No
-	KD5-3	No	No
-	KD5-9	No	No
-	KD5-12	No	No
-	KD5-42	No	No
-	KD5-43	Possible	No
-	KD5-44	No	No
-	KD5-45	No (see item d)	No

(c) A feature at Devil Canyon, designated as feature number DC-1, is being evaluated to determine if it is a fault. There are insufficient data available at the present time to evaluate whether or not the feature is a fault.

(d) Feature KD5-43 on the left abutment of the Devil Canyon site may be the result of an old fault or slope instability based on geophysical and borehole data. The feature does not exhibit geomorphic features typically associated with an active fault.

(e) The maximum credible earthquakes for the Denail and Castle Mountain faults were not re-evaluated as they will not govern the design. A maximum credible earthquake for the Benioff zone is estimated to be magnitude 7.5 (M_s) occurring 60 km from the Devil Canyon site and 50 km from the Watana site and a magnitude 8 plus (M_s) event occurring 90 km from the Devil Canyon site and 70 km from the Watana site.

- (f) Preliminary response spectra curves for the Watana site are represented in Figure 4.4 and for the Devil Canyon site in Figure 4.5.
- (g) A floating earthquake of magnitude 6 to 6.5 (M_s) may have to be considered at 10 km from either site in seismic exposure analysis. Its impact on local faults or design response spectra is being studied.
- (h) Both deterministic and probabilistic approaches will be used in seismic exposure analysis.

6 - Specific Findings of 1980 Studies

Denali and Castle Mountain Fault Systems

The only fault system within the site region (within 62 miles or 100 km of either Project site) which is known to have had displacement in Quaternary time (the last two million years) is the Denali fault. This fault is approximately 40 miles (64 km) north of the sites at its closest approach. The Castle Mountain fault system is immediately south of the site region. This fault system has had displacement in Quaternary time.

Susitna Feature

Reconnaissance level aerial and ground checking has produced no evidence of a fault in bedrock and no evidence of deformation in overlying surficial units for this feature.

Review of aerial gravity and magnetics data shows no evidence of a major tectonic dislocation. Earthquakes correlated with the southern portion of the feature by Gedney and Shapiro (1975) occurred at depths greater than 43 miles (70 km). These focal depths suggest that the earthquakes occurred on the Benioff zone well below the crust and well below the extent of the Susitna feature, if the latter is a fault. The feature may be the result of glaciation of stream drainages whose alignment reflects structural control such as joints or perhaps folding.

Talkeetna Thrust

The Talkeetna thrust fault is a northeast-southwest trending fault which may dip either to the northwest or the southeast. The fault may be connected with the Broxson Gulch thrust fault to the northeast which would result in a 167 mile (270 km) long fault that passes approximately 3.5 miles (5.4 km) upstream of the proposed Watana site. No evidence of displacement younger than Tertiary in age (approximately two to several tens of millions of years old) has been reported for either the Talkeetna or Broxson Gulch thrust faults. However, anomalous relationships in deposits of Tertiary age on the north side of the Susitna river were observed during this investigation and may be related to faulting.

Observed Seismic Events

Seismicity within the Talkeetna Terrain can be clearly delineated as crustal events occurring at depths to approximately 5 to 12 miles (8 to 20 km) and as Benioff zone events which occur at greater depths. The depth of the Benioff zone increases from approximately 25 miles (40 km) in the southeastern part of the site region to more than 50 miles (80 km) in the northwestern part of the microearthquake study area and more than 78 miles (125 km) in the northwestern site region.

The largest reported historical earthquake within the site region is the magnitude (M_S) 6-1/4 event of 1929 which occurred approximately 25 and 31 miles (40 and 50 km) south of the Devil Canyon and Watana sites, respectively. Four earthquakes greater than magnitude (M_S) 5 have occurred during the period 1904 through August 1980.

Earthquakes as large as magnitude (M_S) 5 to 5-1/2 may possibly occur in the site region without direct association with surface fault rupture. Such events would probably be constrained to rupture planes deeper than 6 miles (10 km).

The largest crustal event recorded within the microearthquake study area during 3 months of monitoring was magnitude (M_L) 2.8. It occurred 6.8 miles (11 km) northeast of the Watana site at a depth of 9.3 miles (15 km) on 2 July 1980.

Two clusters of microearthquake activity were observed within the microearthquake network during the three-month monitoring period. These two clusters occurred in the same general vicinity east of the southern portion of the Talkeetna Thrust fault. These clusters of seismicity occurred at depths of 6 to 12 miles (10 to 20 km). One of the clusters gives a composite focal plane mechanism of N230E, dipping 50°NW, consistent with local geologic trends. The sense of movement is reverse (toward the southeast) with a dextral component of slip.

The clusters of microearthquake activity described above appear to be related to a small subsurface rupture plane that does not extend to the surface. These clusters do not appear to be related to the Talkeetna thrust fault.

Seismicity in the vicinity of the site, including the clusters described above, appears to reflect relatively small-scale crustal adjustments at depth in the crust. These adjustments may be related to stresses imposed by the Benioff zone and/or by plate motion.

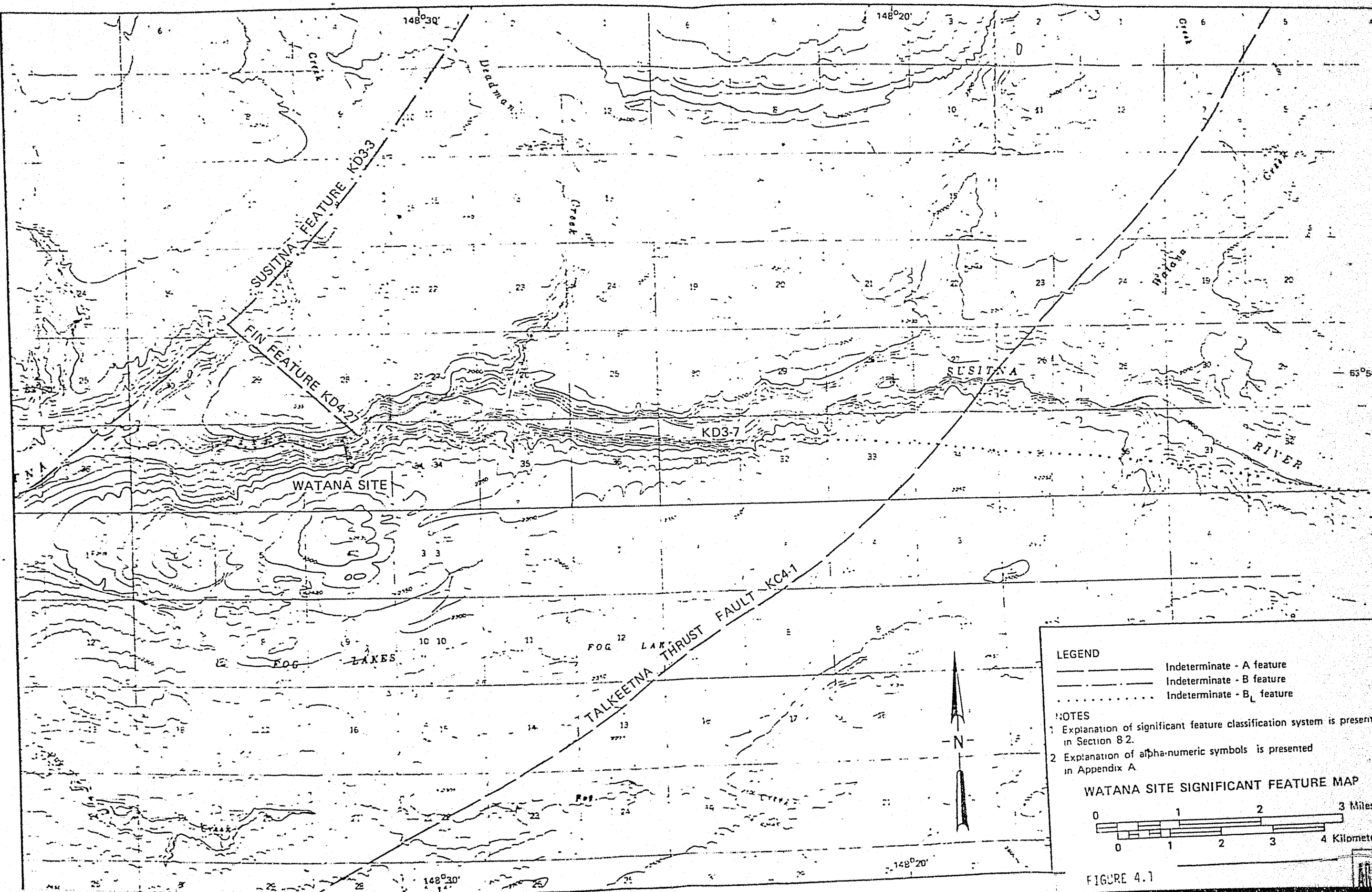
No association of microearthquake activity with candidate significant or significant features is apparent on the basis of information obtained to date.

Reservoir Induced Seismicity

The two reservoirs are considered as one reservoir hydrologically. This combined Watana-Devil Canyon reservoir would be among the deepest and largest in the world. It is concluded that the likelihood of a reservoir-induced earthquake of any size within the hydrologic regime of the proposed reservoir is high (0.9 on a scale of 0 to 1); this is primarily because water depth has a major apparent theoretical and empirical correlation with the occurrence of reservoir-induced seismicity.

Maximum Credible Earthquake Assessment

Preliminary maximum credible earthquakes (PMCEs) have been estimated for crustal faults with recent displacement in and adjacent to the site region and for the Benioff zone. The PMCE for the Denali fault is estimated to be a magnitude (M_s) 8.5 event occurring 40 miles (64 km) from the Devil Canyon site and 43 miles (70 km) from the Watana site. The PMCE for the Castle Mountain fault is estimated to be a magnitude (M_s) 7.4 event occurring 65 miles (105 km) from the Devil Canyon site and 71 miles (115 km) from the Watana site. The PMCE for the Benioff zone is estimated to be a magnitude (M_s) 8.5 event occurring 31 miles (50 km) beneath the Watana site and 37 miles (60 km) beneath the Devil Canyon site.



LEGEND

- Indeterminate - A feature
- - - Indeterminate - B feature
- Indeterminate - B₁ feature

NOTES

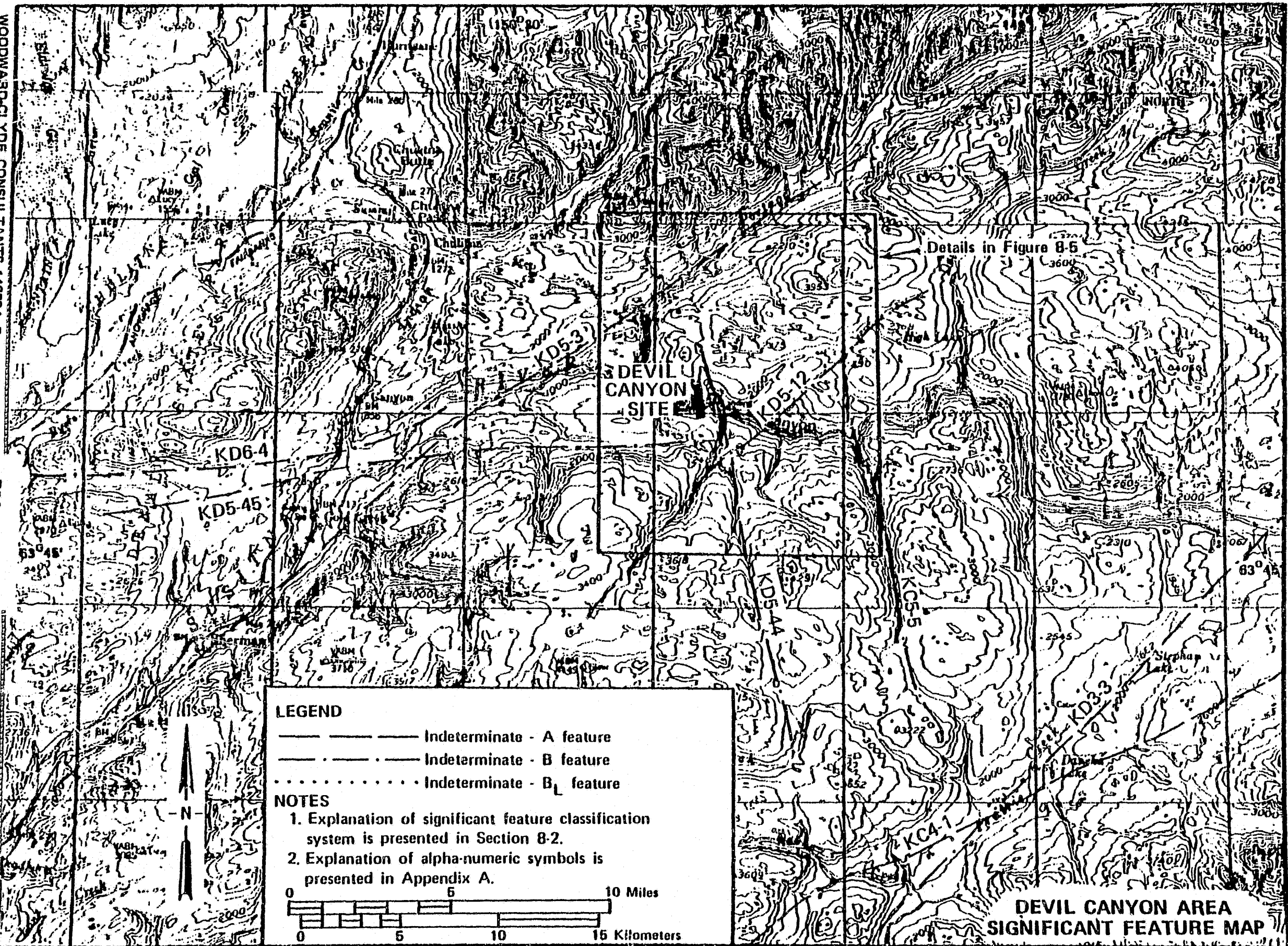
- 1 Explanation of significant feature classification system is presented in Section 8.2.
- 2 Explanation of alpha-numeric symbols is presented in Appendix A.

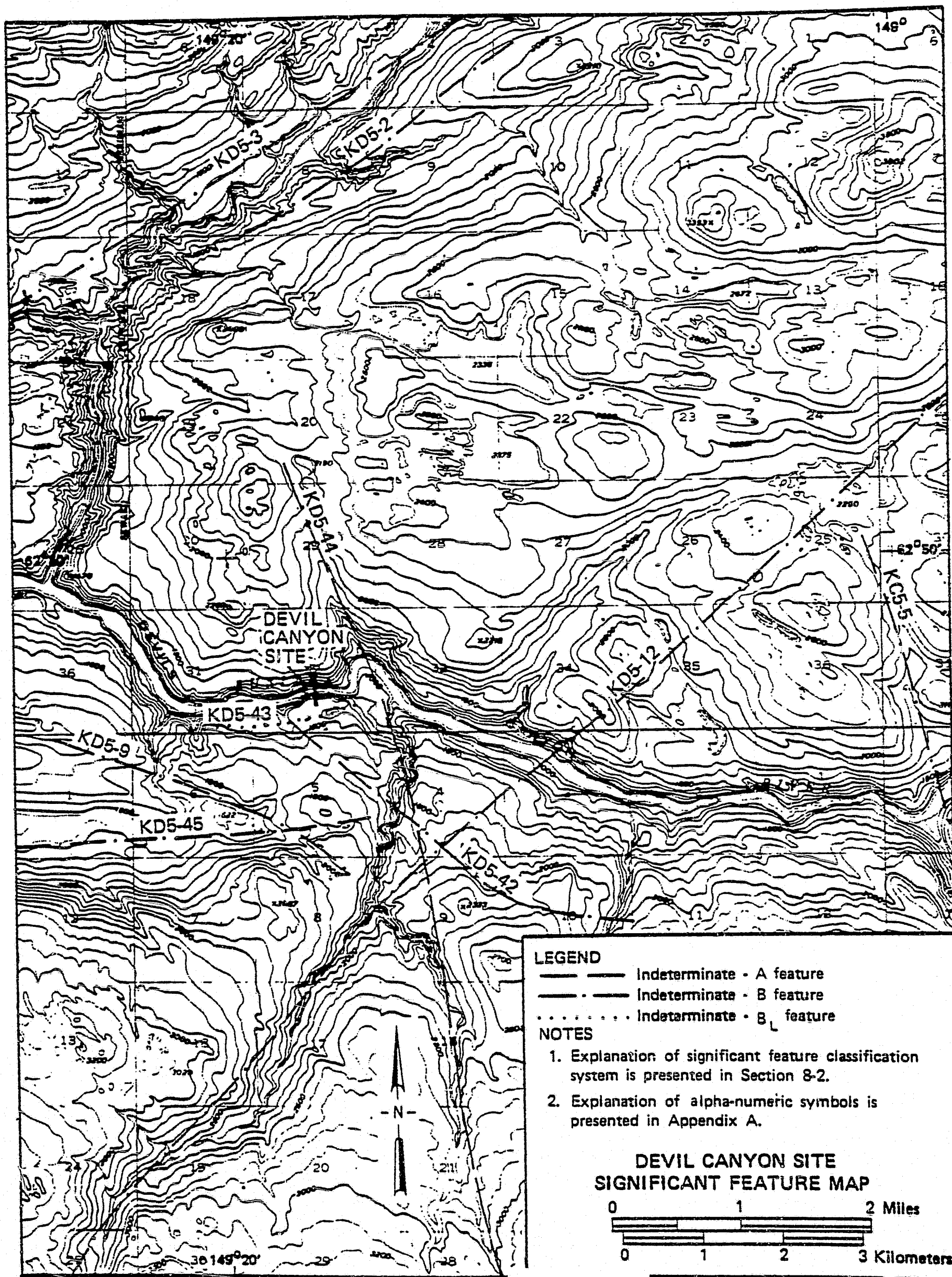
WATANA SITE SIGNIFICANT FEATURE MAP

0 1 2 3 Miles
0 1 2 3 4 Kilometers

FIGURE 4.1

FIGURE 4.2





WOODWARD-CLYDE CONSULTANTS 14658A December 1980

FIGURE 4.3

ACRES

Mean Surface Profile
During Rain to 0.5

Design Flood

Design Flood

Design Flood

PRELIMINARY

0.352

0.342

FIGURE 4.4

46 6013

RECEIVED
SEMPER PARITURIC 4 CYCLES X N DIVISION
MAILED 8 SEP 60 10 44 AM

ACRES

ATTACHMENT A

Memorandum

Woodward-Clyde Consultants

To: File
41610A
M44-83

From: J. R. Lovegreen RL

Chief: Orange:

Date: 30 April 1981

Subject: BUDGETS FOR THE 18 OBJECTIVES IN MEMO PM4-142

Objective No. 1	Subtask No. 2	Objective Description
1	4.11	Significant Features at Watana
2	4.11	Significant Features at Devil Canyon
3	4.09	Benioff Zone MCE
4	4.09	Talkeetna Terrain Earthquake Review
5	4.09	1904 & 1912 Earthquake Review
6	4.09	Talkeetna Terrain Stress Regime
7	4.11	MCE for Active Faults in Talkeetna Terrain
8	4.10	RIS Refinement
9	-	Denali MCE Refinement
10	-	1981 Seismic Network
11	4.13	Ground Motions at Watana
12	4.13	Ground Motions at Devil Canyon
13	4.08	Dam Stability
14	4.14	
15	4.15	Trans. Line and Road Stability
16	4.13	Benioff Zone Attenuation Revision
17	4.09	Revised Exposure Analysis
18	4.09	Network Manual
-	4.12	Data Analysis and Report
-	4.09	Budgets for Administration and Planning
-	4.11	Fixed Fee
TOTAL		

NOTES

- Objective Nos. and Descriptions are those in memorandum PM4-142 and letter WCAA4-40.
- Subtask Nos. are those in Acres American Inc. Agreement No. P57006.10.41 and letter WCAA4-55.
- Budgets are expressed in \$10³.
- Includes Quaternary geology, field mapping, trenching, remote sensing, limited geophysics, and review and travel. It excludes calibration studies on the Castle Mountain fault which have been deleted.
- Included in budget for Objectives 1 and 2.
- Included in budget for Objectives 11 and 12.
- This total (less the fixed fee of \$62,000) corresponds with that in letter WCAA4-55. It is \$23,000 less than the amount (\$574,000) in letter WCAA4-50. The \$23,000 has been set aside to defray, in part, the costs identified in letter WCAA4-48.

FIGURE 4.6

1981 AND 1982 SUSITNA TASK 4 SCHEDULE FOR SUBTASKS 4.08 THROUGH 4.15

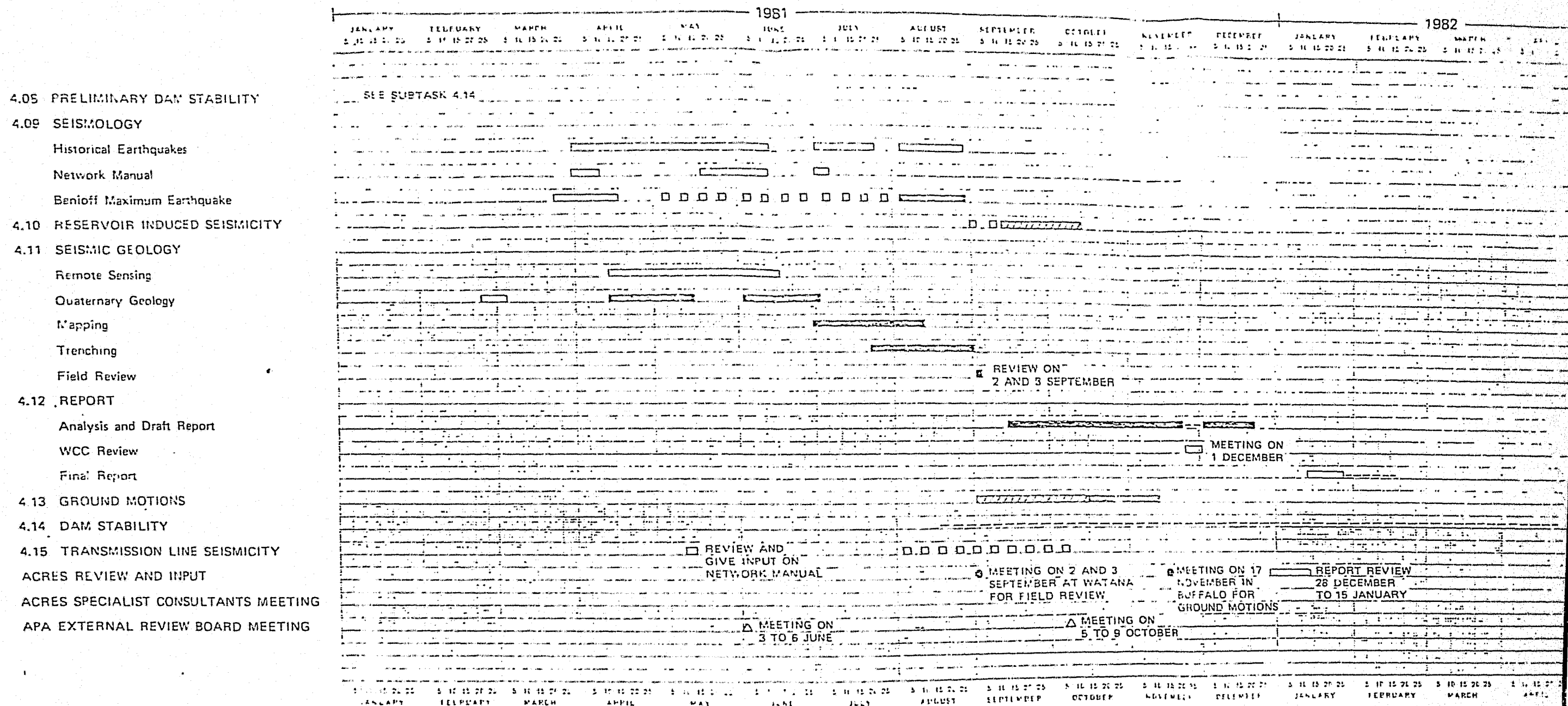
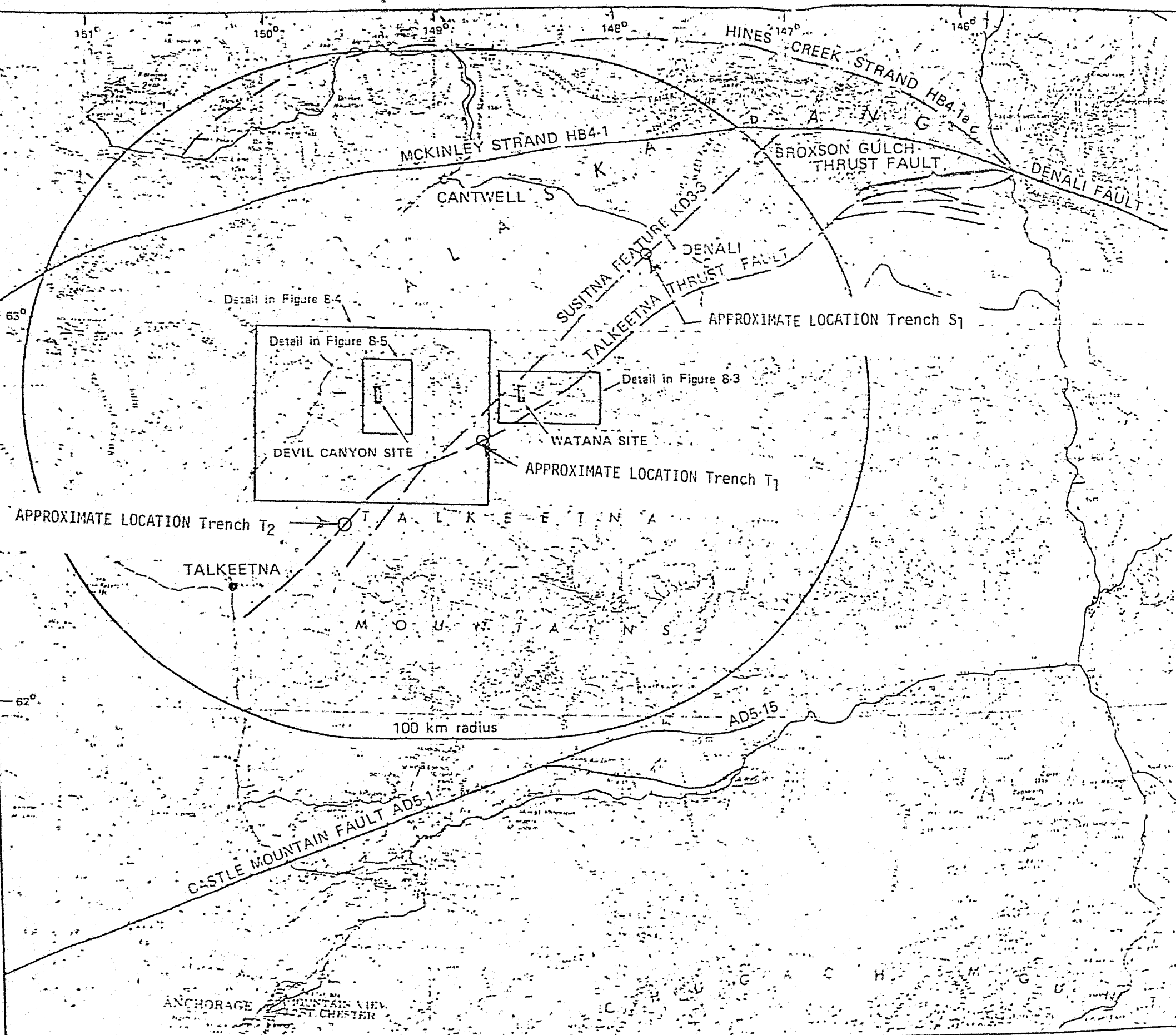


FIGURE 4.7



LEGEND

- BOUNDARY FAULTS**
- Faults with recent displacement
- SIGNIFICANT FEATURES**
- Indeterminate A feature
 - Indeterminate B feature

NOTES

1. Explanation of significant feature classification system is presented in Section 8-2.
2. Explanation of alpha-numeric symbols is presented in Appendix A.

BOUNDARY FAULT AND SIGNIFICANT FEATURE MAP FOR THE SITE REGION

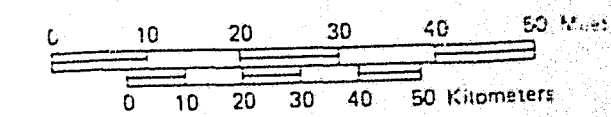


FIGURE 4.E

SUPPORT MATERIAL TO TASK 5
GEOTECHNICAL STUDIES

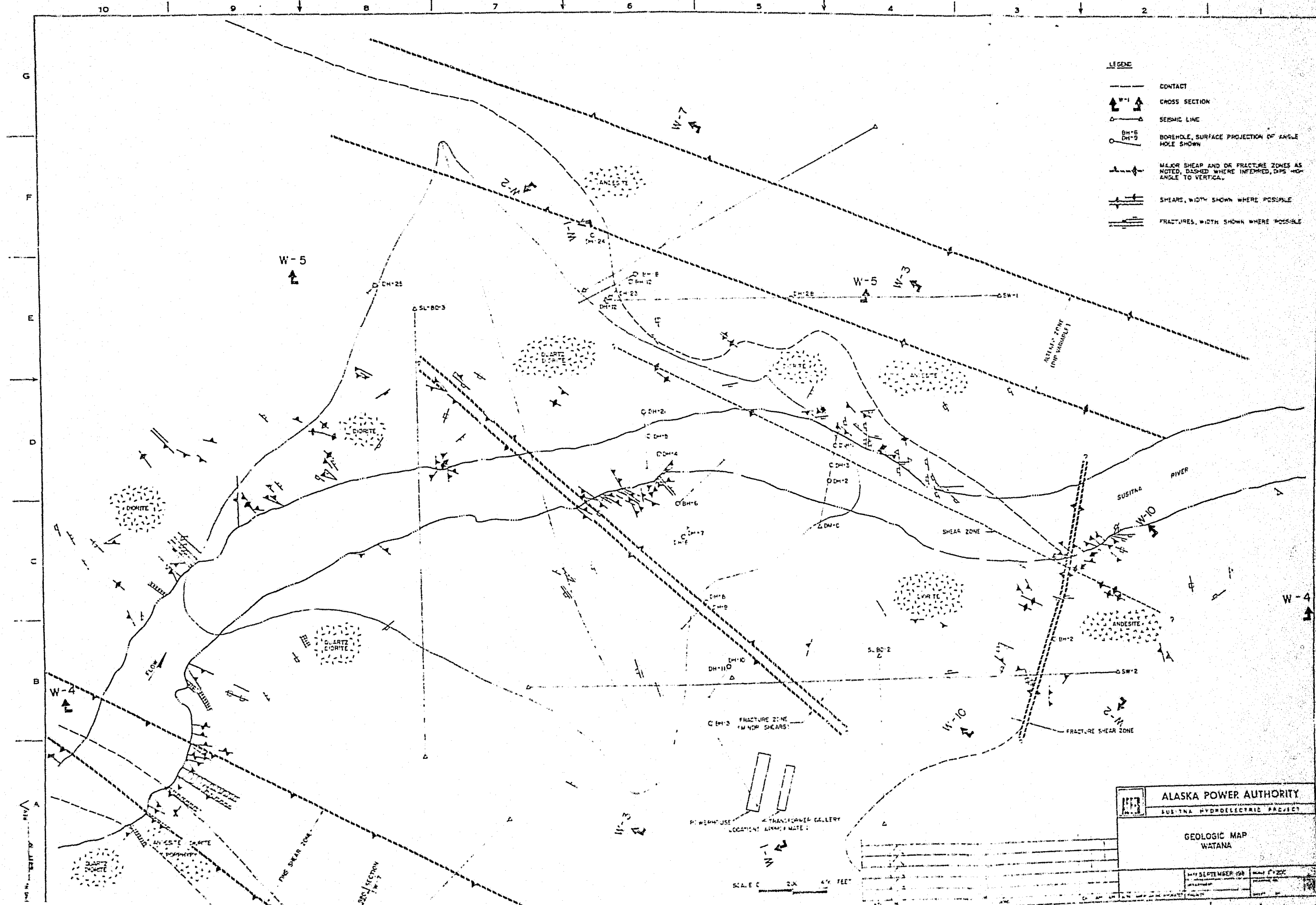
SUSITNA HYDROELECTRIC PROJECT

OCT 6-8 MEETING INFO PACKAGES

GEOTECHNICAL DATA INDEX

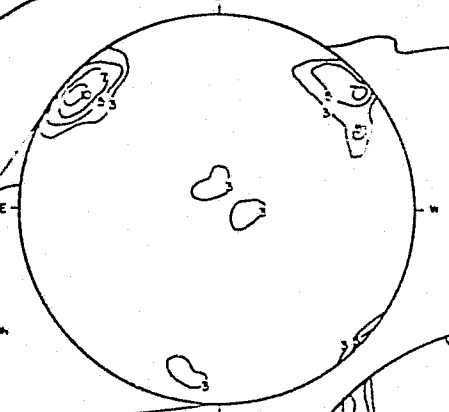
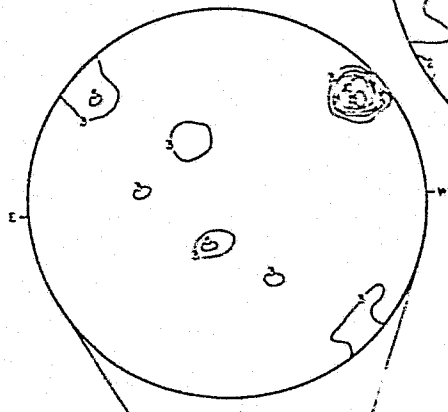
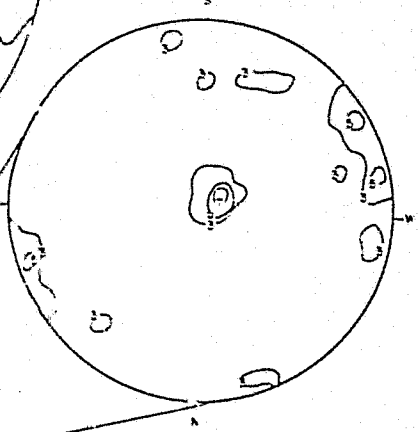
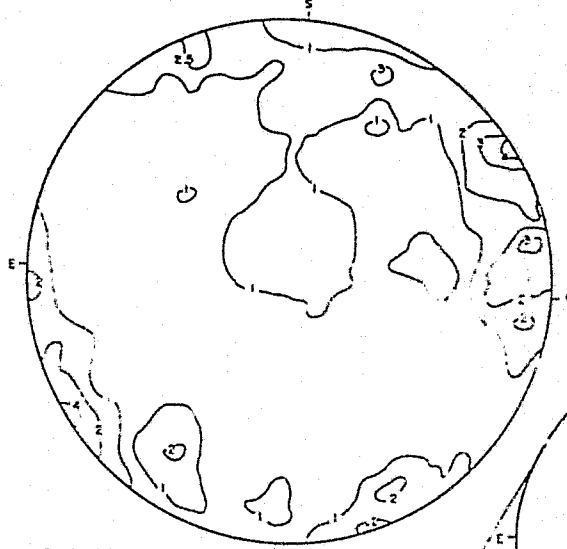
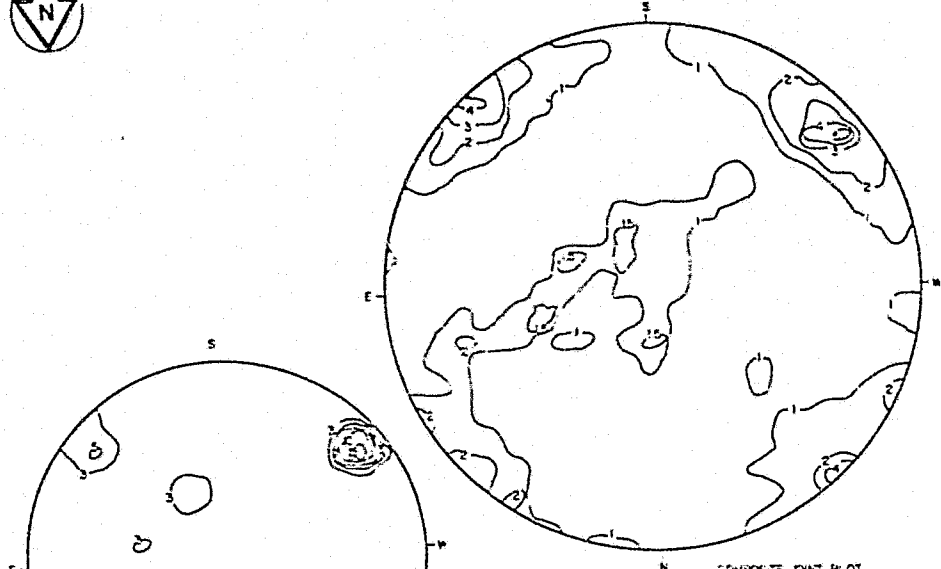
- 1) WATANA DAMSITE GEOLOGY MAPS & SECTIONS
- 2) DEVIL CANYON GEOLOGY MAPS & SECTIONS
- 3) WATANA DAMSITE EXPLORATIONS
- 4) DEVIL CANYON DAMSITE EXPLORATIONS
- 5) WATANA MATERIALS
- 6) DEVIL CANYON MATERIALS
- 7) WATANA RELICT CHANNEL
- 8) WATANA DAMSITE - FOUNDATIONS, EXCAVATION &
ROCK MECHANICS CRITERIA AND DATA
- 9) DEVIL CANYON DAMSITE - FOUNDATIONS, EXCAVATION &
ROCK MECHANICS CRITERIA AND DATA

1) WATANA DAMSITE GEOLOGY

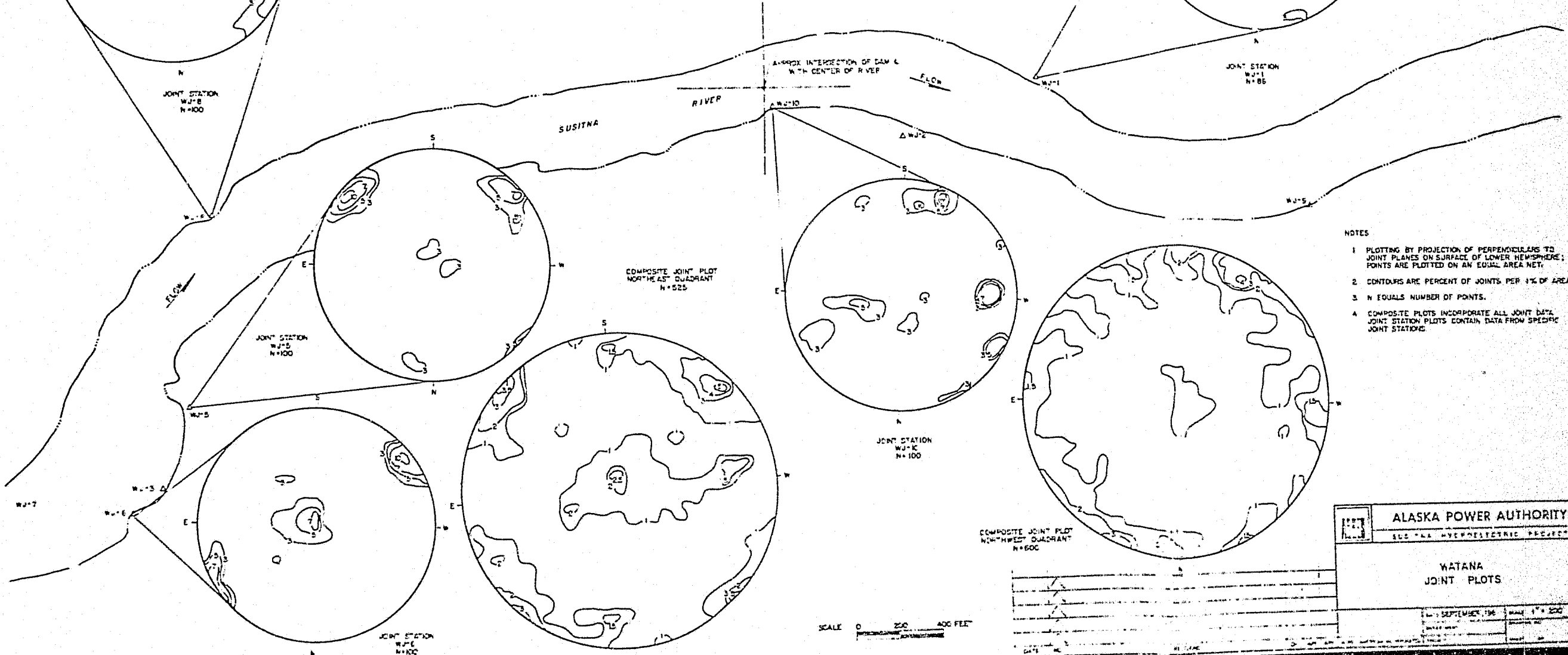
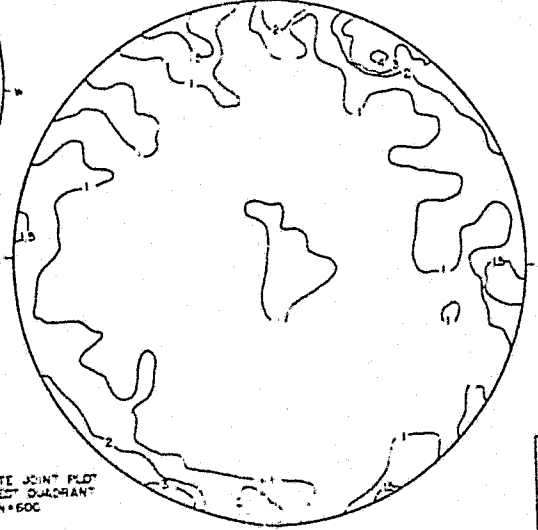
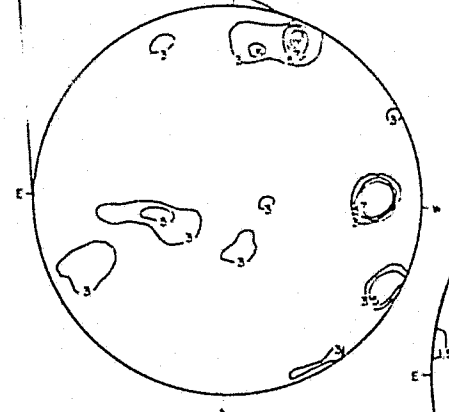


- LEGEND**
- CONTACT
 - CROSS SECTION
 - SEISMIC LINE
 - BOREHOLE, SURFACE PROJECTION OF ANGLE HOLE SHOWN
 - MAJOR SHEAR AND/OR FRACTURE ZONES AS NOTED, DASHED WHERE INFERRRED, DIPS 40° ANGLE TO VERTICAL
 - SHEARS, WIDTH SHOWN WHERE POSSIBLE
 - FRACTURES, WIDTH SHOWN WHERE POSSIBLE

ALASKA POWER AUTHORITY	
SUSITNA HYDROELECTRIC PROJECT	
GEOLOGIC MAP	
WATANA	
DATE: SEPTEMBER 1980	SCALE: 1" = 400'



COMPOSITE JOINT PLOT NORTHEAST QUADRANT N=525



- NOTES
1. PLOTTING BY PROJECTION OF PERPENDICULARS TO JOINT PLANES ON SURFACE OF LOWER HEMISPHERE; POINTS ARE PLOTTED ON AN EQUAL AREA NET.
 2. CONTOURS ARE PERCENT OF JOINTS PER 1% OF AREA.
 3. N EQUALS NUMBER OF POINTS.
 4. COMPOSITE PLOTS INCORPORATE ALL JOINT DATA. JOINT STATION PLOTS CONTAIN DATA FROM SPECIFIC JOINT STATIONS.

SCALE 0 200 400 FEET

ALASKA POWER AUTHORITY	
SUSITNA HYDROELECTRIC PROJECT	
WATANA JOINT PLOTS	
DATE: SEPTEMBER, 196	SCALE: 1" = 200'

10 9 8 7 6 5 4 3 2 1

G

F

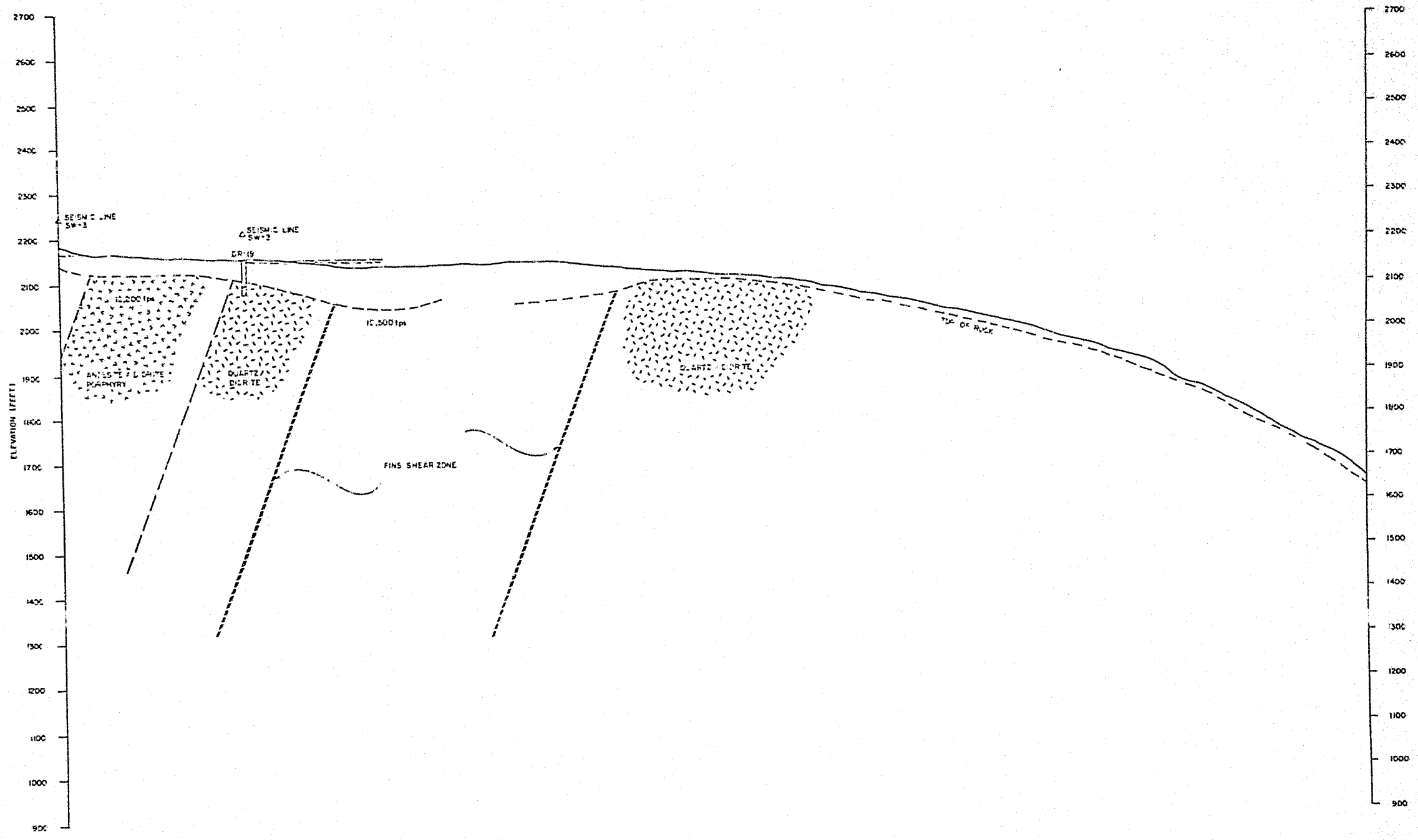
E

D

C

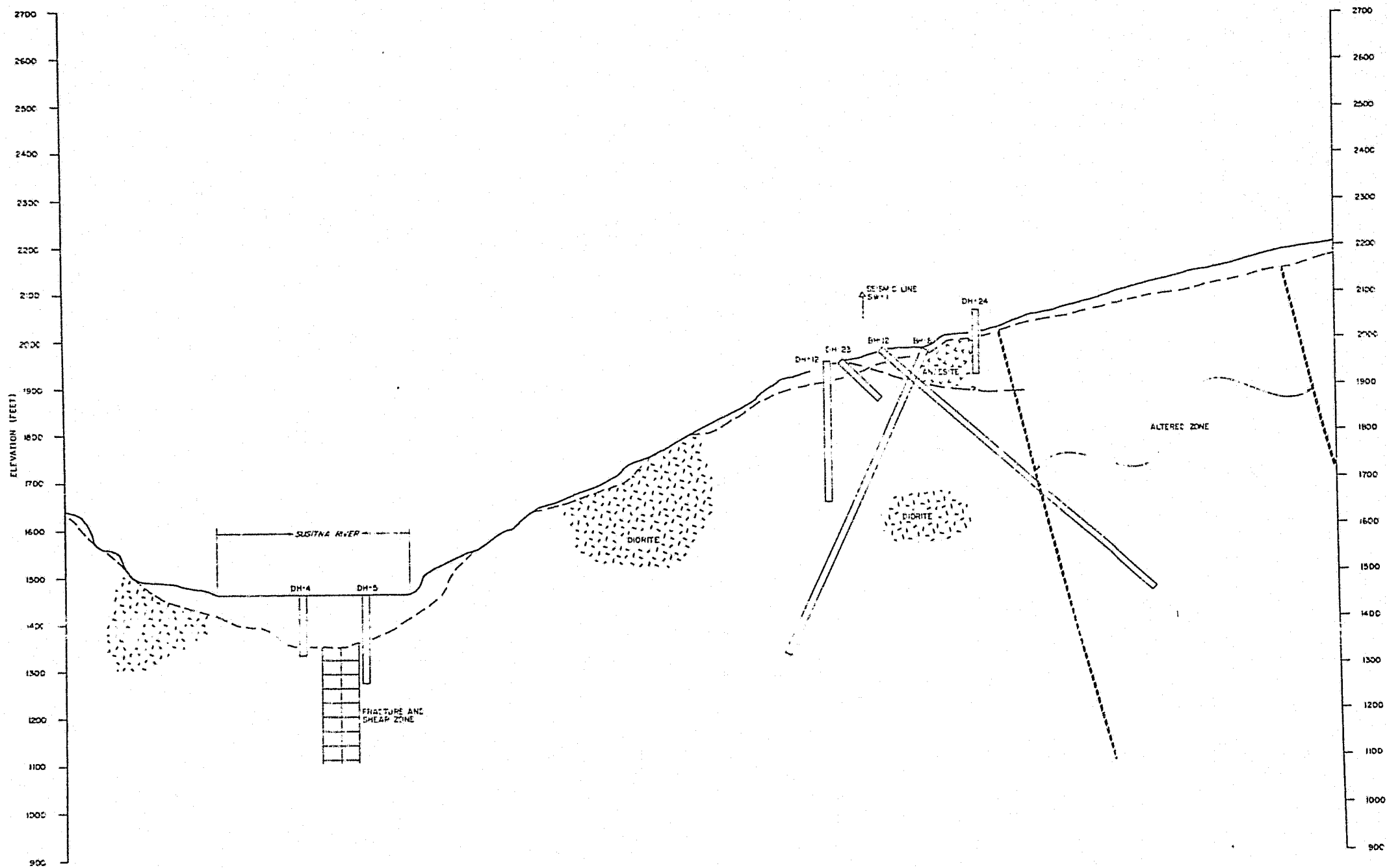
B

A



SCALE 1" = 100' HORIZ. 1" = 200' VERT.

ALASKA POWER AUTHORITY	
SUSITNA HYDROELECTRIC PROJECT	
WATANA	
GEOLOGIC SECTION W-7	
SHEET 1 OF 2	
DATE: SEPTEMBER 1960	BY: J. E. JOHNSON
CHECKED BY: J. E. JOHNSON	APPROVED BY: J. E. JOHNSON



SCALE 0 100 200 FEET

ALASKA POWER AUTHORITY SUSITNA HYDROELECTRIC PROJECT	
WATANA GEOLOGIC SECTION W-7 SHEET 2 OF 2	
DATE: SEPTEMBER 1984 DRAWN BY: [blank] CHECKED BY: [blank]	SCALE: 1" = 100' SHEET: 2 OF 2

G

F

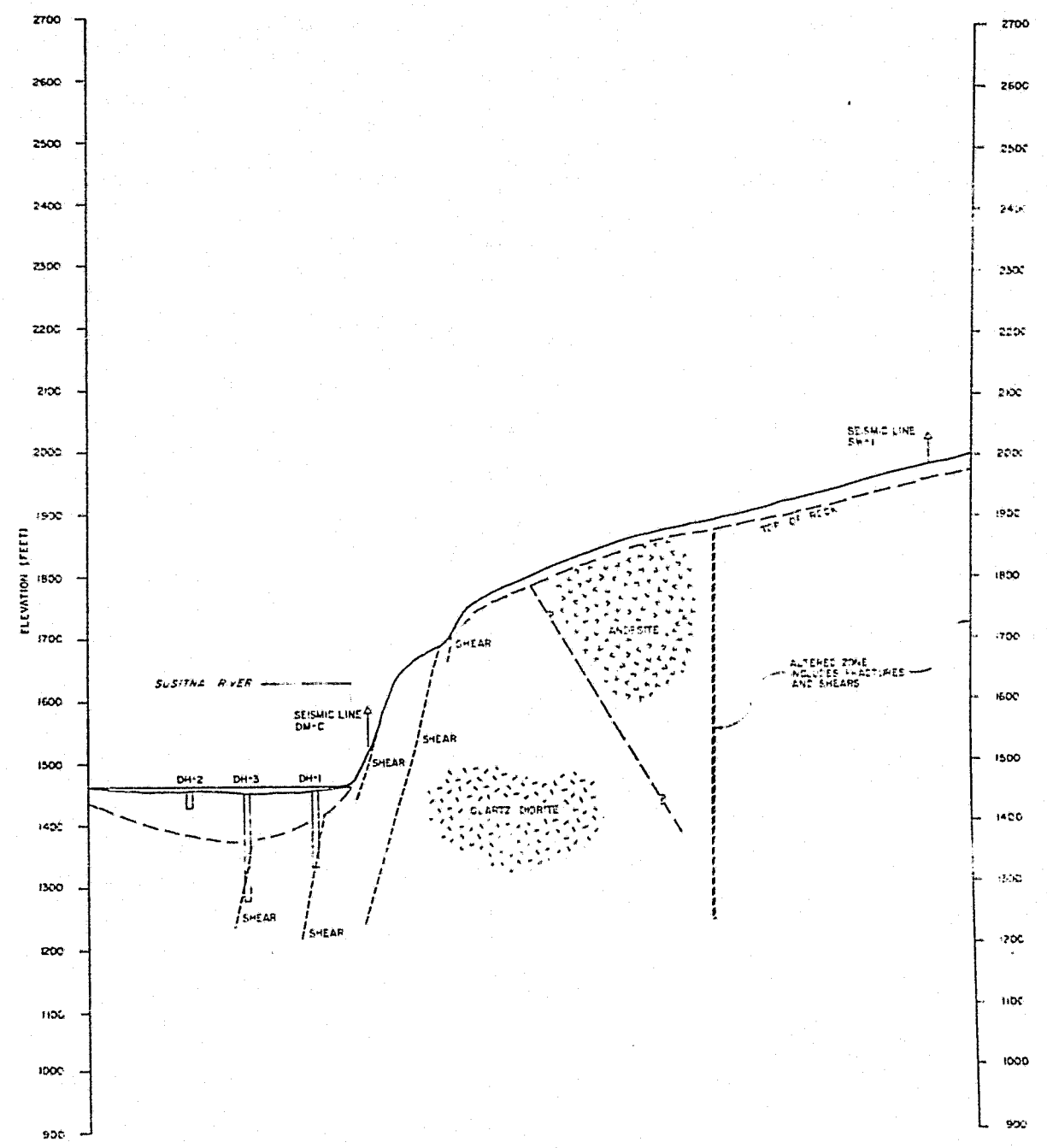
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D

C

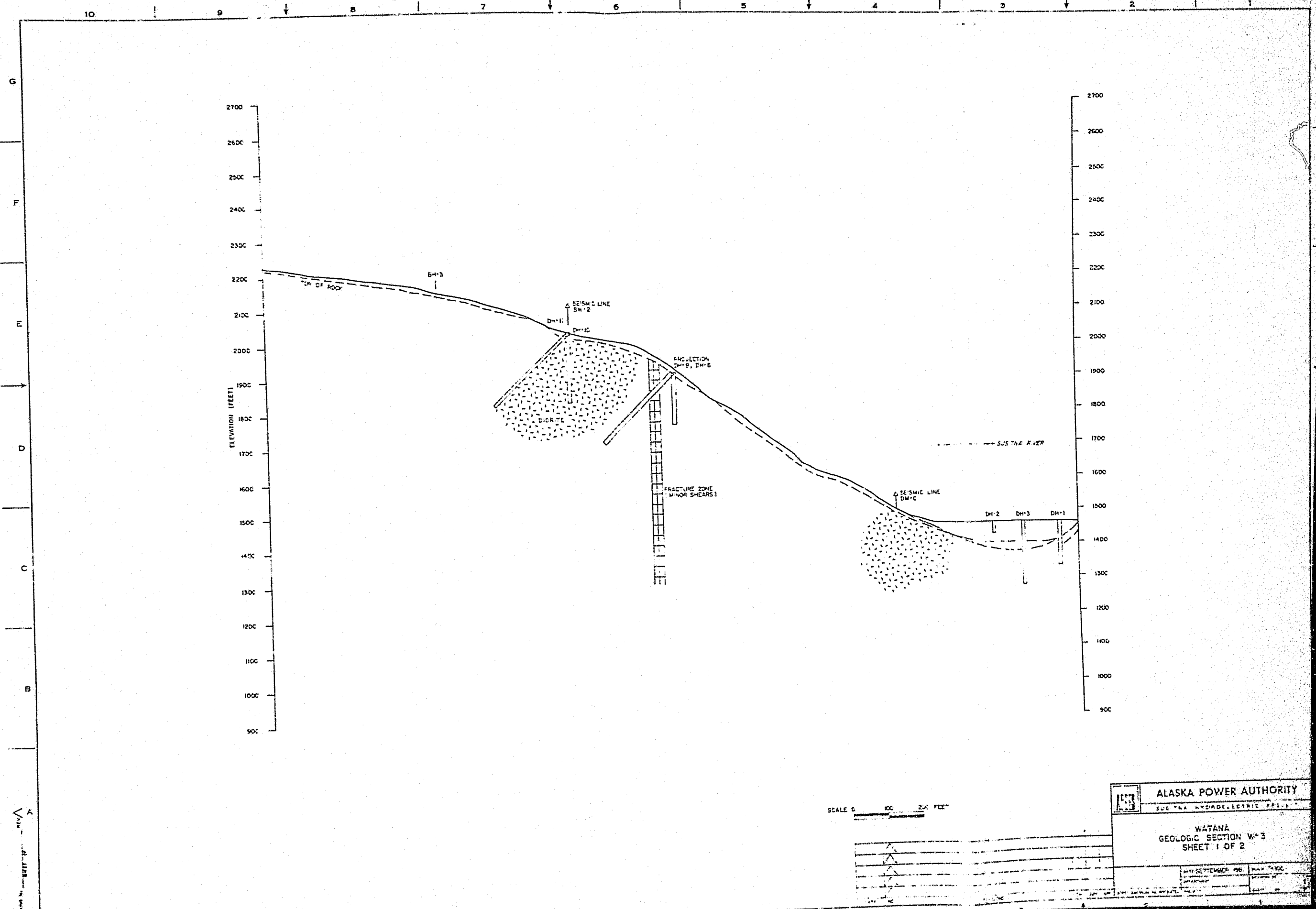
B

A



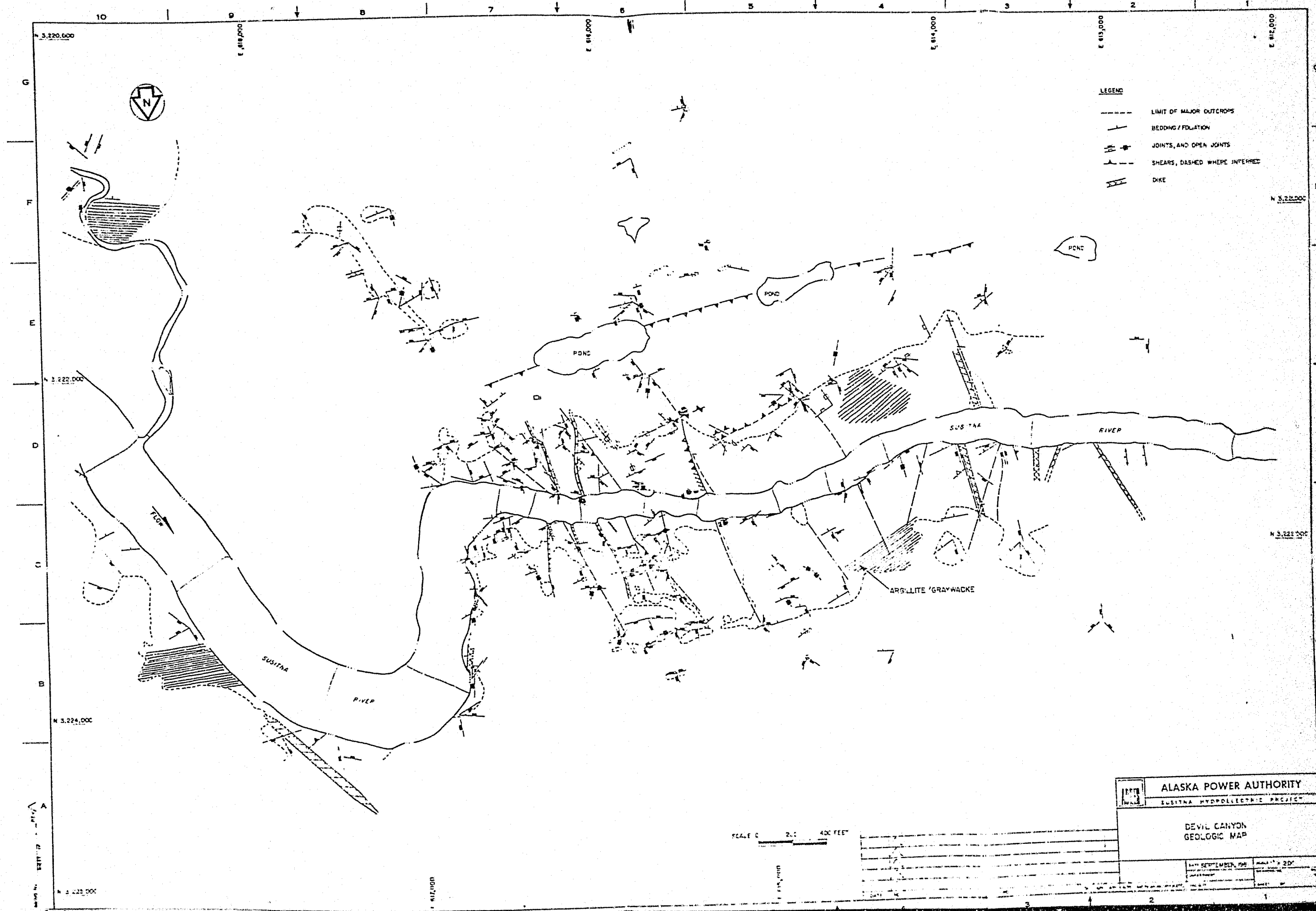
SCALE 0 100 200 FEET

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WATANA	
GEOLOGIC SECTION W-3	
SHEET 2 OF 2	
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BY: [Signature]	APP'D: [Signature]
CHECKED: [Signature]	REVIEWED: [Signature]



ALASKA POWER AUTHORITY	
SUS TAA HYDROELECTRIC PROJECT	
WATANA	
GEOLOGIC SECTION W-3	
SHEET 1 OF 2	
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APPROVED: [Signature]	REVIEWED: [Signature]

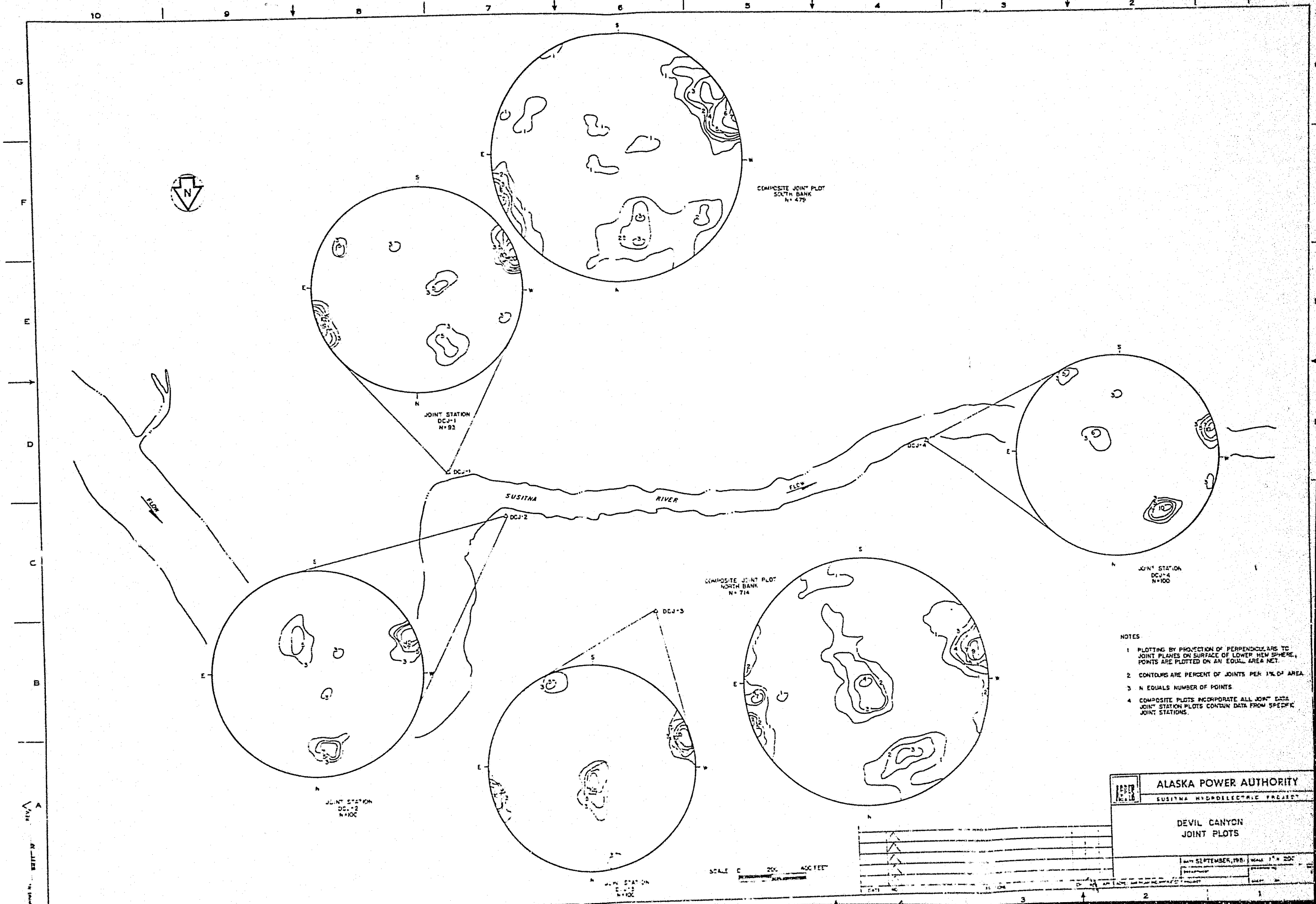
2) DEVIL CANYON DAMSITE GEOLOGY

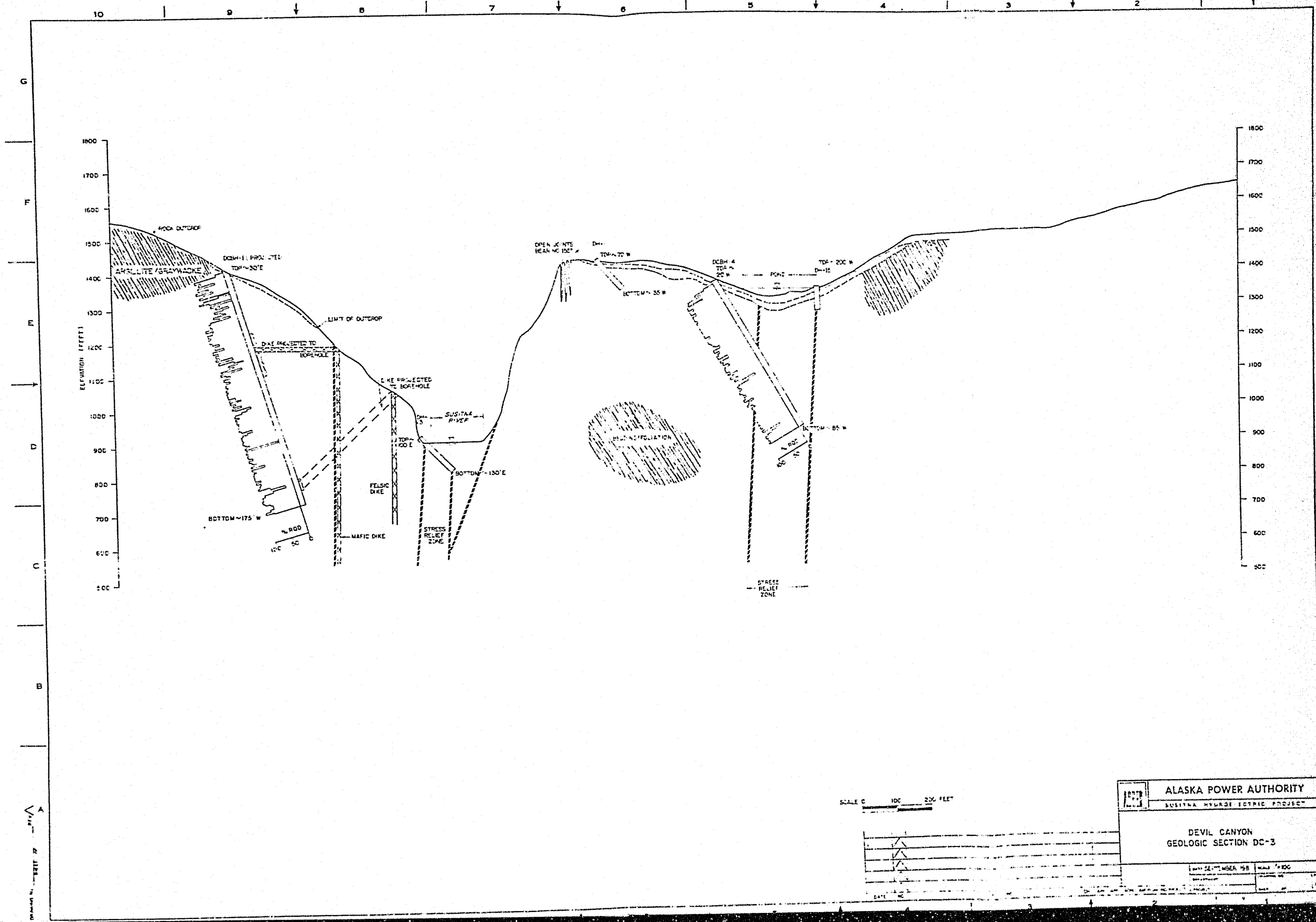


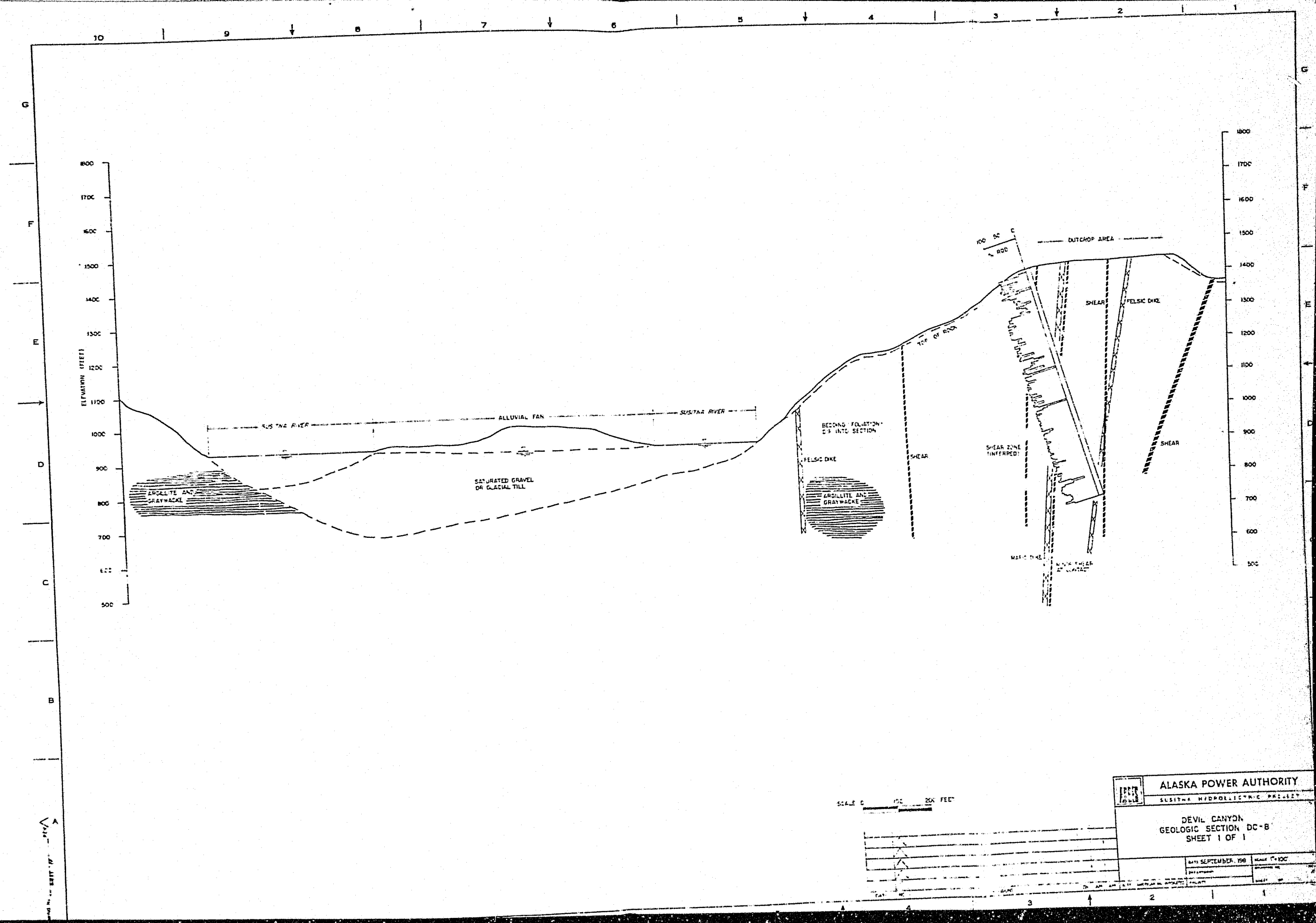
- LEGEND
- LIMIT OF MAJOR OUTCROPS
 - BEDDING / FOLIATION
 - JOINTS, AND OPEN JOINTS
 - SHEARS, DASHED WHERE INTERFERED
 - DIKE

SCALE 0 200 400 FEET

ALASKA POWER AUTHORITY	
SUSITNA HYDROELECTRIC PROJECT	
DEVIL CANYON	
GEOLOGIC MAP	
DATE: SEPTEMBER, 1964	SCALE: 1" = 200'
PROJECT: SUSITNA HYDROELECTRIC PROJECT	MAP NO. 1
BY: [illegible]	APP. BY: [illegible]









HORIZON LINE



ACRES

SUS-TNA HYDROELECTRIC PROJECT

DETAILED CAUTION: DIAGRAMMATIC REPRESENTATION OF THE GEOLGICAL FEATURES OF THE SUS-TNA RIVER AREA.

ACRES AMERICAN CORPORATION

3) WATANA DAMSITE EXPLORATIONS

WATANA

STRUCTURE: 910 ' HIGH ROCKFILL DAM

INVESTIGATIONS:

USBP. 1950-1953	RECONNAISSANCE		
COE 1975	RECONNAISSANCE	ACRES 1980	ACRES 1981
	22500 LF SEISMIC		
COE 1978	28 BOREHOLES	3 HOLES	4 HOLES *
	30-600 ' DEEP	376-752' DEEP	300-955' *
	18 AUGER HOLES	21 AUGER	18 AUGER
	27 TEST PITS		21 PITS *
	47665 LF SEISMIC	24800 LF	38200 LF
	10 PIEZOMETERS	1 PIEZO	4 PIEZOS*
	13 TEMP. PROBES	1 THERMISTER	2 THERMISTERS*

SIGNIFICANT FEATURES:

VALLEY 300' AT BASE, 6000' AT RIM, 30-60° SLOPES

10-40' OVERBURDEN

5-40' WEATHERED ROCK

ALLUVIUM - SANDS AND GRAVELS, SOME TALUS, POSSIBLY PARTLY FROZEN

RELICT CHANNEL UPSTREAM OF DAM, GLACIAL AND ALLUVIAL MATERIALS

UP TO 454' IN DEPTH.

MAJOR SHEAR ZONES ("THE FINS", "FINGERBUSTER") BORDER DAMSITE.

LOCALIZED NARROW ZONES OF SHEARING AND INTENSE THERMAL ALTERATION.

3 MAJOR JOINT SETS (PROMINENT N30-60°W)

DAMSITE IN PLUTON, BOUNDED BY VOLCANICS AND SEDIMENTS.

*NOTE: IN PROGRESS, NUMBER MAY INCREASE.

9/16/81

MATERIALS SOURCES:

CONCRETE AGGREGATE & FILTER MATERIALS AVAILABLE 2-4 MILES DOWNSTREAM OF SITE.

ROCKFILL AVAILABLE FROM EXCAVATIONS AND IMMEDIATELY ADJACENT TO LEFT END OF DAM CREST.

GRAVEL SHELL MATERIAL AVAILABLE IN RIVER WITHIN 6-8 MILES UPSTREAM AND DOWNSTREAM - STILL BEING EVALUATED FOR SUITABILITY.

IMPERVIOUS/SEMI-IMPERVIOUS MATERIAL FOR CORE AVAILABLE 1-3 MILES UPSTREAM OF AXIS.

AREAS TO ADDRESS IN DESIGN:

MAJOR SHEAR ZONES LIMIT UPSTREAM AND DOWNSTREAM LAYOUT FLEXIBILITY.

LEFT ABUTMENT SHEAR ZONE AND ALTERED ZONE LIMIT STRUCTURES, WILL REQUIRE ADDITIONAL GROUTING AND DRAINAGE PROVISION.

DEEP ALLUVIUM IN CHANNEL, OF UNKNOWN DYNAMIC (SEISMIC RESPONSE) PROPERTIES MANDATES REMOVAL UNTIL PROVEN SUITABLE AS FOUNDATION.

LOCAL SHEAR ZONE INFLUENCE UNDERGROUND STRUCTURE ORIENTATION.

DEEP RELICT CHANNEL ALLUVIUM REQUIRES INVESTIGATION FOR WATER-TIGHTNESS AND SENSITIVITY TO HYDROSTATIC HEAD FROM RESERVOIR.

SPORADIC PERMAFROST WILL REQUIRE CONSIDERATION IN CONSTRUCTION.

UNDERGROUND STRUCTURES:

ROCK QUALITY GOOD BELOW 250' IN DEPTH, PERMEABILITIES LOW.

STRUCTURES REQUIRE ORIENTATION TO AVOID LOCAL JOINTING, SHEARS.

9/16/81

MATERIALS SOURCES:

CONCRETE AGGREGATE & FILTER MATERIALS AVAILABLE 2-4 MILES DOWNSTREAM OF SITE.

ROCKFILL AVAILABLE FROM EXCAVATIONS AND IMMEDIATELY ADJACENT TO LEFT END OF DAM CREST.

GRAVEL SHELL MATERIAL AVAILABLE IN RIVER WITHIN 6-8 MILES UPSTREAM AND DOWNSTREAM - STILL BEING EVALUATED FOR SUITABILITY.

IMPERVIOUS/SEMI-IMPERVIOUS MATERIAL FOR CORE AVAILABLE 1-3 MILES UPSTREAM OF AXIS.

AREAS TO ADDRESS IN DESIGN:

MAJOR SHEAR ZONES LIMIT UPSTREAM AND DOWNSTREAM LAYOUT FLEXIBILITY.

LEFT ABUTMENT SHEAR ZONE AND ALTERED ZONE LIMIT STRUCTURES, WILL REQUIRE ADDITIONAL GROUTING AND DRAINAGE PROVISION.

DEEP ALLUVIUM IN CHANNEL, OF UNKNOWN DYNAMIC (SEISMIC RESPONSE) PROPERTIES MANDATES REMOVAL UNTIL PROVEN SUITABLE AS FOUNDATION.

LOCAL SHEAR ZONE INFLUENCE UNDERGROUND STRUCTURE ORIENTATION.

DEEP RELICT CHANNEL ALLUVIUM REQUIRES INVESTIGATION FOR WATER-TIGHTNESS AND SENSITIVITY TO HYDROSTATIC HEAD FROM RESERVOIR.

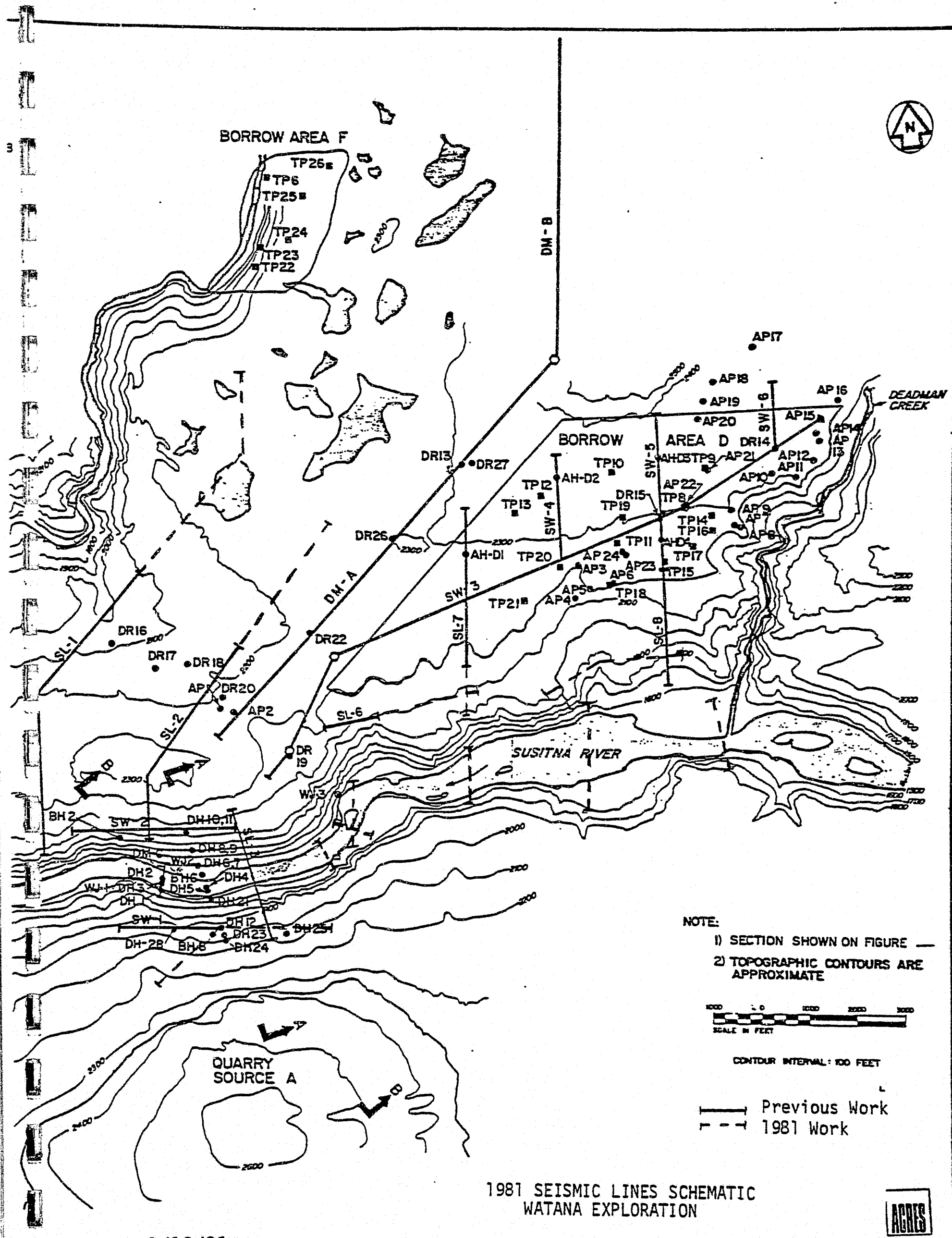
SPORADIC PERMAFROST WILL REQUIRE CONSIDERATION IN CONSTRUCTION.

UNDERGROUND STRUCTURES:

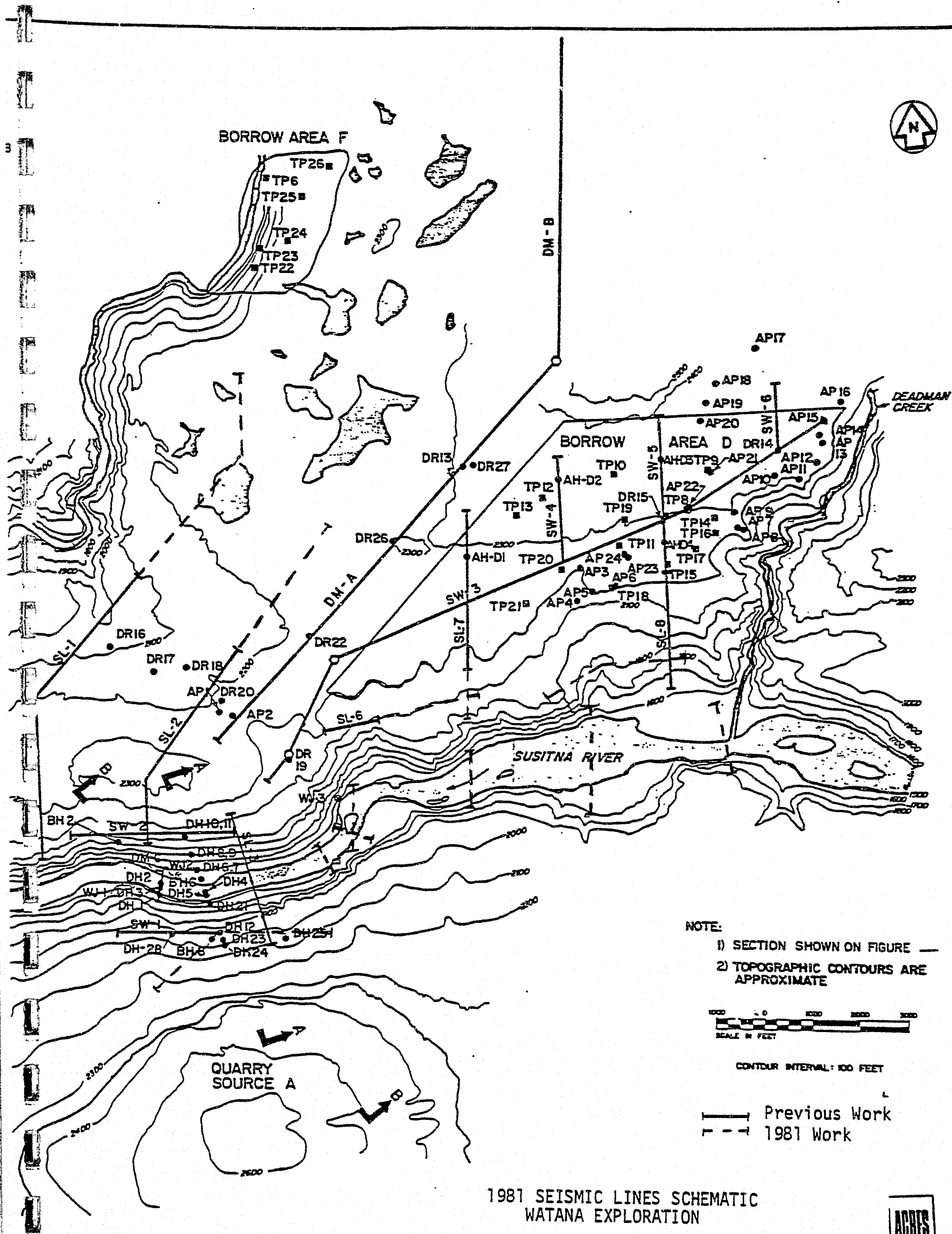
ROCK QUALITY GOOD BELOW 250' IN DEPTH, PERMEABILITIES LOW.

STRUCTURES REQUIRE ORIENTATION TO AVOID LOCAL JOINTING, SHEARS.

00138



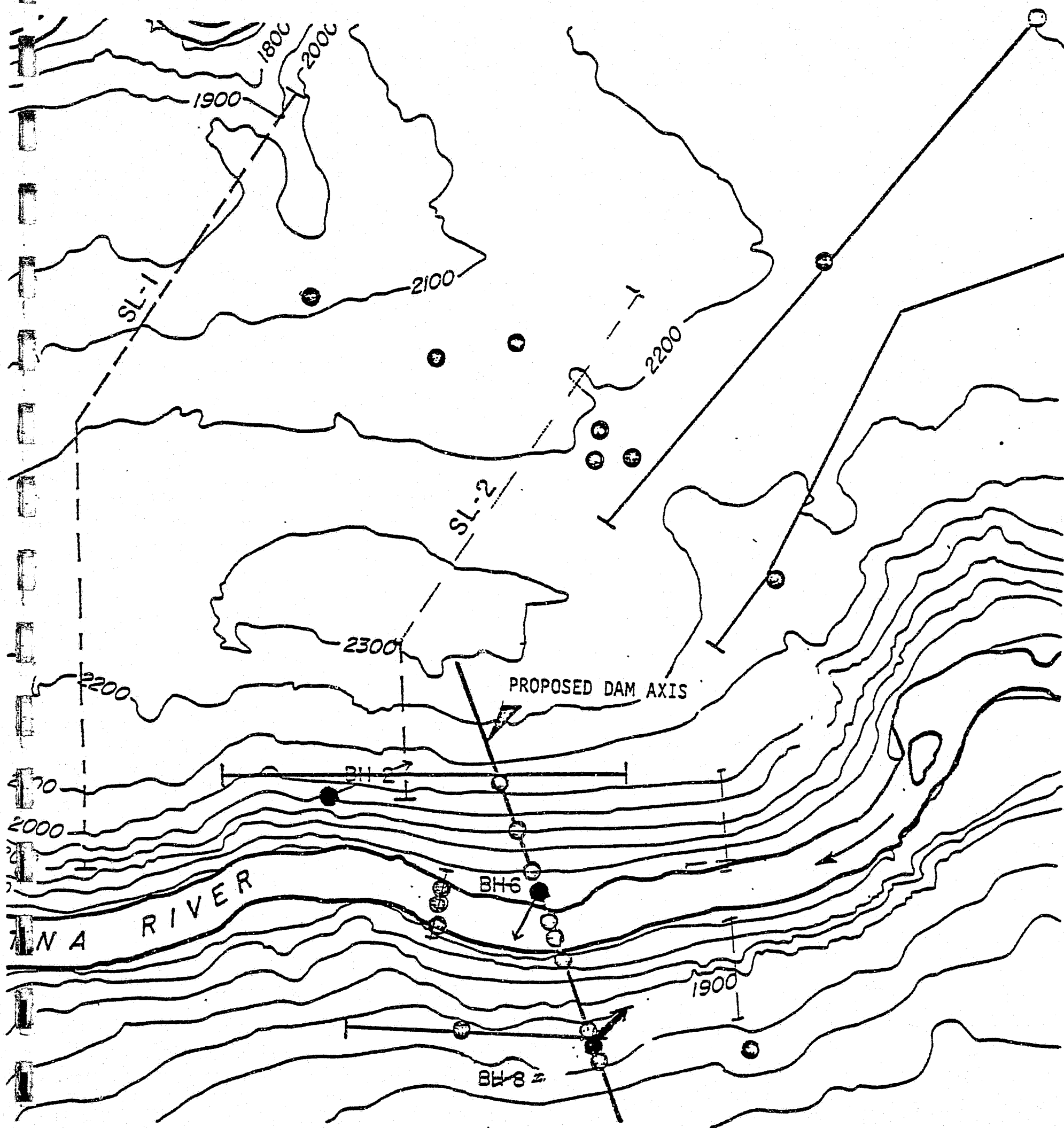
0/16/91



0/16/81



8
3
0
0
1
3



WATANA SITE

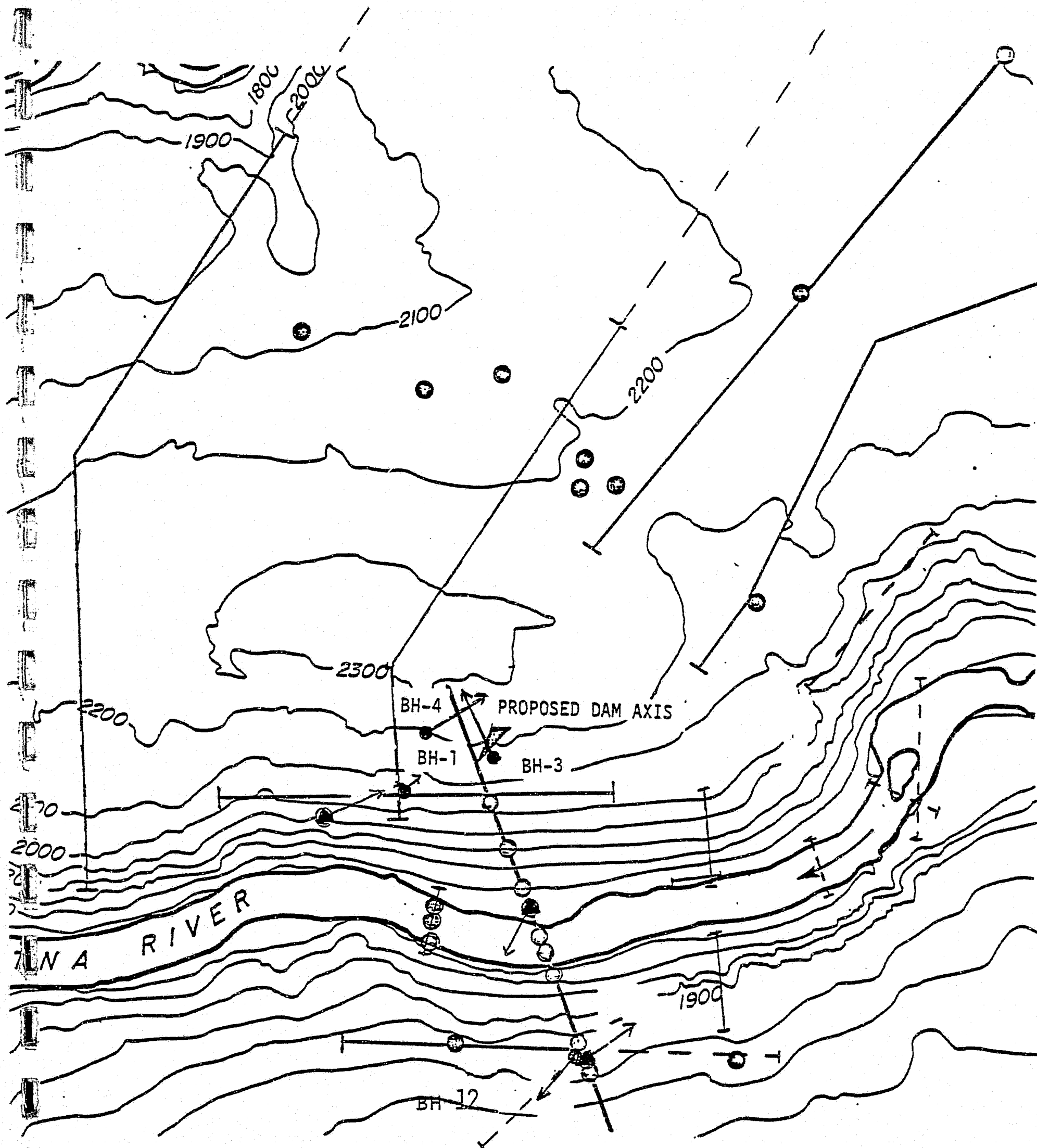
1980 GEOTECHNICAL INVESTIGATION

- PREVIOUS WORK
- 1980 PROGRAM
- DIAMOND CORE HOLES
- SEISMIC LINES



0/16/81

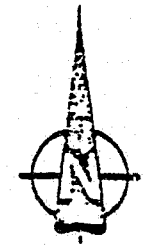
8
3
1
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WATANA SITE
1981 GEOTECHNICAL INVESTIGATION

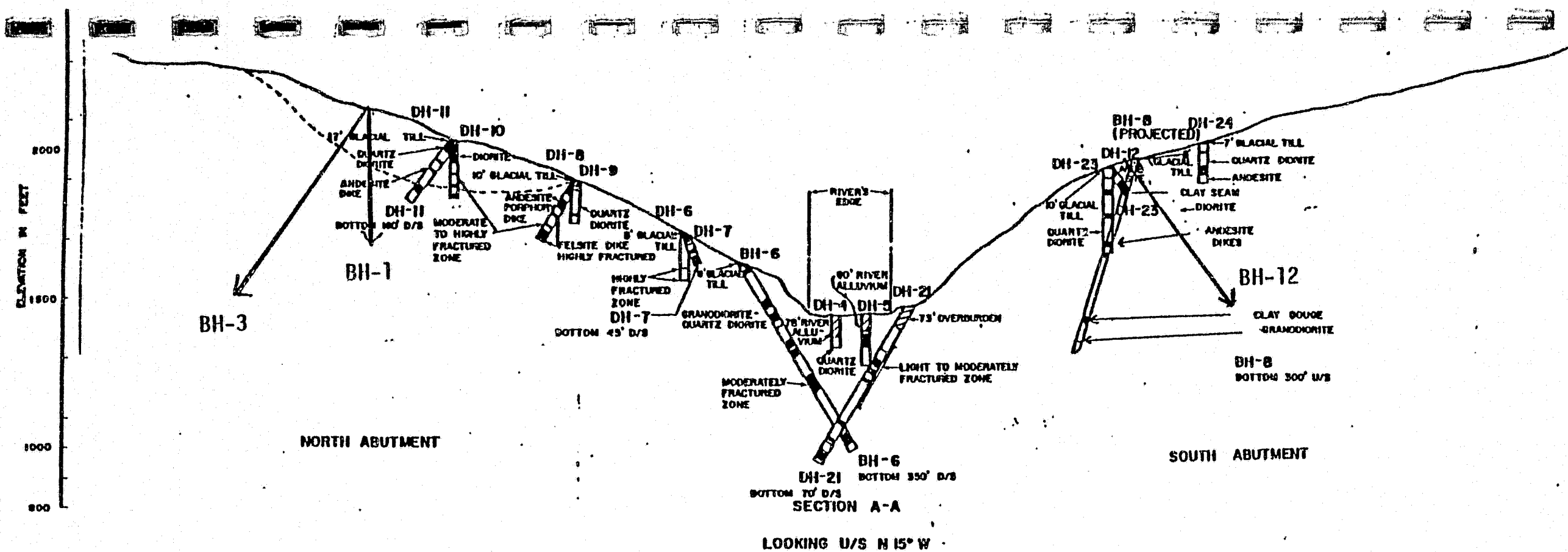
- | | | |
|---|---------------|----------------------|
| ● | PREVIOUS WORK | BOREHOLES |
| ● | 1980 PROGRAM | - DIAMOND CORE HOLES |
| ● | 1981 PROGRAM | |

0/10/01



- SEISMIC LINES**
- | | |
|-----|------|
| --- | COE |
| --- | 1980 |
| --- | 1981 |

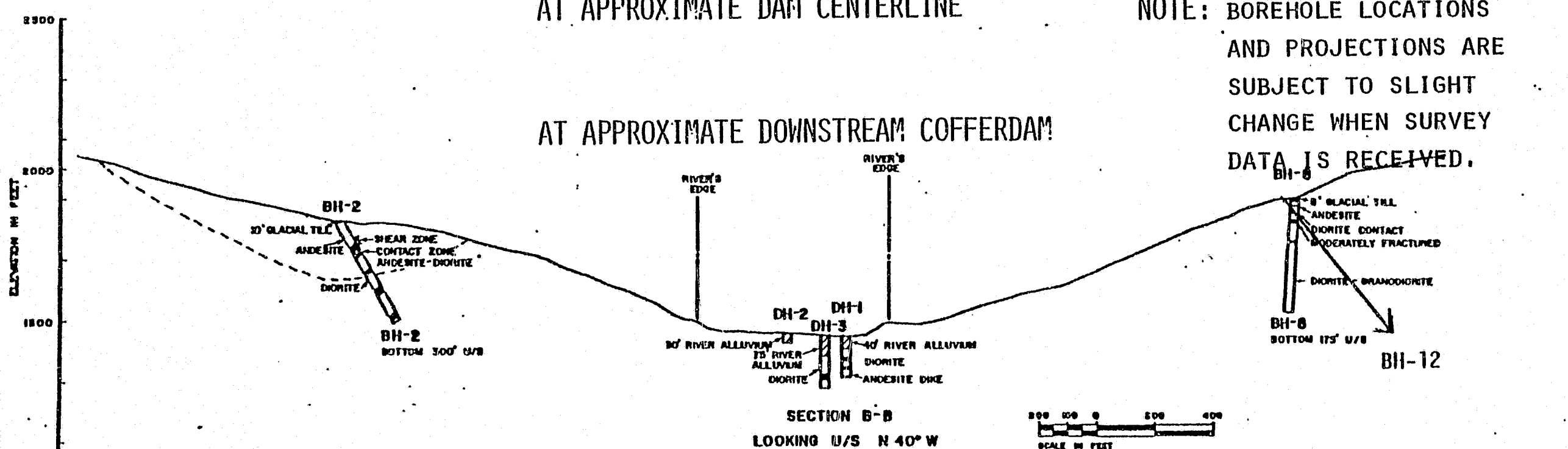




AT APPROXIMATE DAM CENTERLINE

AT APPROXIMATE DOWNSTREAM COFFERDAM

NOTE: BOREHOLE LOCATIONS AND PROJECTIONS ARE SUBJECT TO SLIGHT CHANGE WHEN SURVEY DATA IS RECEIVED.



[] PREVIOUS
 ↓ 1981

GEOLOGIC CROSS-SECTION WATANA



4) DEVIL CANYON DAMSITE EXPLORATIONS

DEVIL CANYON

STRUCTURE: 635' HIGH CONCRETE GRAVITY-ARCH DAM

INVESTIGATIONS:

USBR 1957-1958; COE 1978
22 BOREHOLES 20-150' DEEP
19 TRENCHES AND TEST PITS

1300 LF SEISMIC LINES

LAB TESTS:
PETROGRAPHIC, ROCK ELASTIC
PROPERTIES, STRENGTHS

ACRES-1980
3 HOLES, 501-750'
2 AUGER HOLES

GRADATIONS

ACRES-1981
4 HOLES, 150-600'
6 AUGER HOLES
6 TEST PITS
1600 LF SEISMIC

GRADATIONS *
AGGREGATE TESTS *

SIGNIFICANT FEATURES:

150-900 FOOT CANYON WIDTH (BASE TO RIM), 45-80° SLOPES
75± FEET OF ALLUVIUM UNDER SADDLE DAM AXIS
VIRTUALLY ZERO ALLUVIUM ON GORGE WALLS
SHEAR ZONES AND FAULTS IN BOTH ABUTMENTS
LARGE OPEN JOINTS AND DETACHED BLOCKS ON LEFT ABUTMENT
3 JOINT SETS (PROMINENT N25W)
BEDDING DIPS SOUTH, 50-70°
35-50 FEET OF WEATHERED ROCK

MATERIALS SOURCES:

CONCRETE AGGREGATE AND FILTER MATERIALS READILY AVAILABLE WITHIN
1000' UPSTREAM OF AXIS

ROCKFILL AVAILABLE FROM EXCAVATIONS (BEING EVALUATED FOR SUITABILITY)
OP. QUARRY SITES WITHIN 2 MILES OF AXIS, ON SOUTH SIDE.

9/16/81

*NOTE: IN PROGRESS

00138

POTENTIAL FREEZE-THAW RESISTANCE PROBLEM IN ROCK AND/OR AGGREGATE -
TESTING PROGRAM WILL EVALUATE MAGNITUDE OF PROBLEM.

IMPERVIOUS FOR SADDLE DAM - NO READILY AVAILABLE SOURCE IDENTIFIED:
WILL REQUIRE PROCESSED ROCK OR LONGER HAUL FROM GLACIAL TILL SOURCE.

AREAS TO ADDRESS IN DESIGN:

LEFT ABUTMENT:

SOUTHERLY DIPPING BEDS OVERHANG, WILL REQUIRE EXCESS EXCAVATION
AND EXTENSIVE DENTAL TREATMENT.

SIGNIFICANT ROCK SUPPORT MAY BE REQUIRED LOCALLY.

THRUST BLOCK WILL REQUIRE DEEP ANCHORING.

RIGHT ABUTMENT:

BEDDING DIPS TOWARDS RIVER, WILL REQUIRE LOCAL SUPPORT

SADDLE DAM:

EAST-WEST TRENDING SHEAR ZONE AND BURIED CHANNEL.

BEDROCK UP TO 90 FEET BELOW LAKE LEVEL.

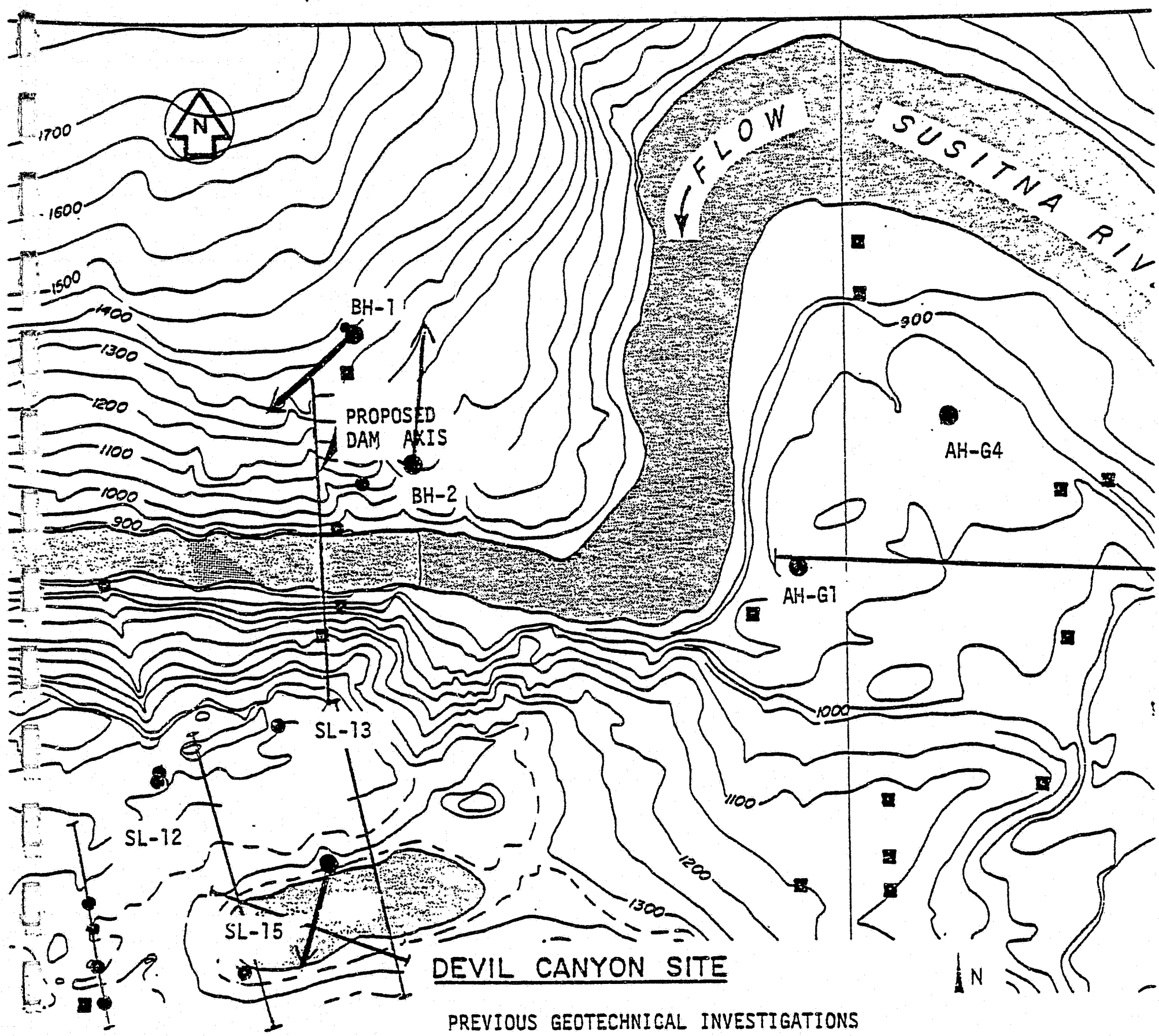
SPORADIC TO POSSIBLE DEEP PERMAFROST.

UNDERGROUND STRUCTURES:

ROCK IMPROVES WITH DEPTH, TO UNIFORM GOOD QUALITY BELOW 150 FEET.

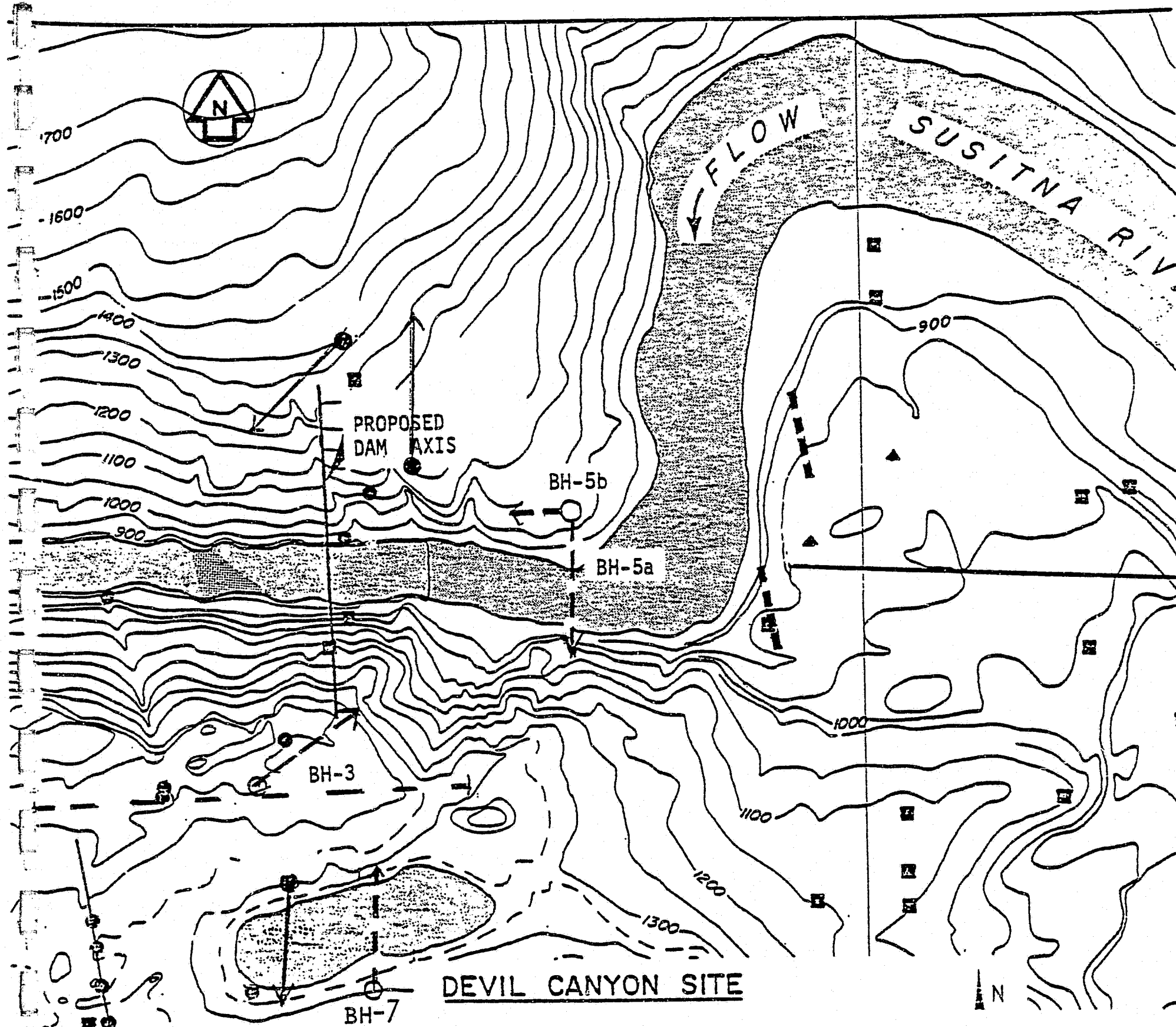
PERMEABILITIES LOW AT DEPTH.

9/16/81



● BOREHOLES BH
■ TEST PITS SL
● AUGER AH





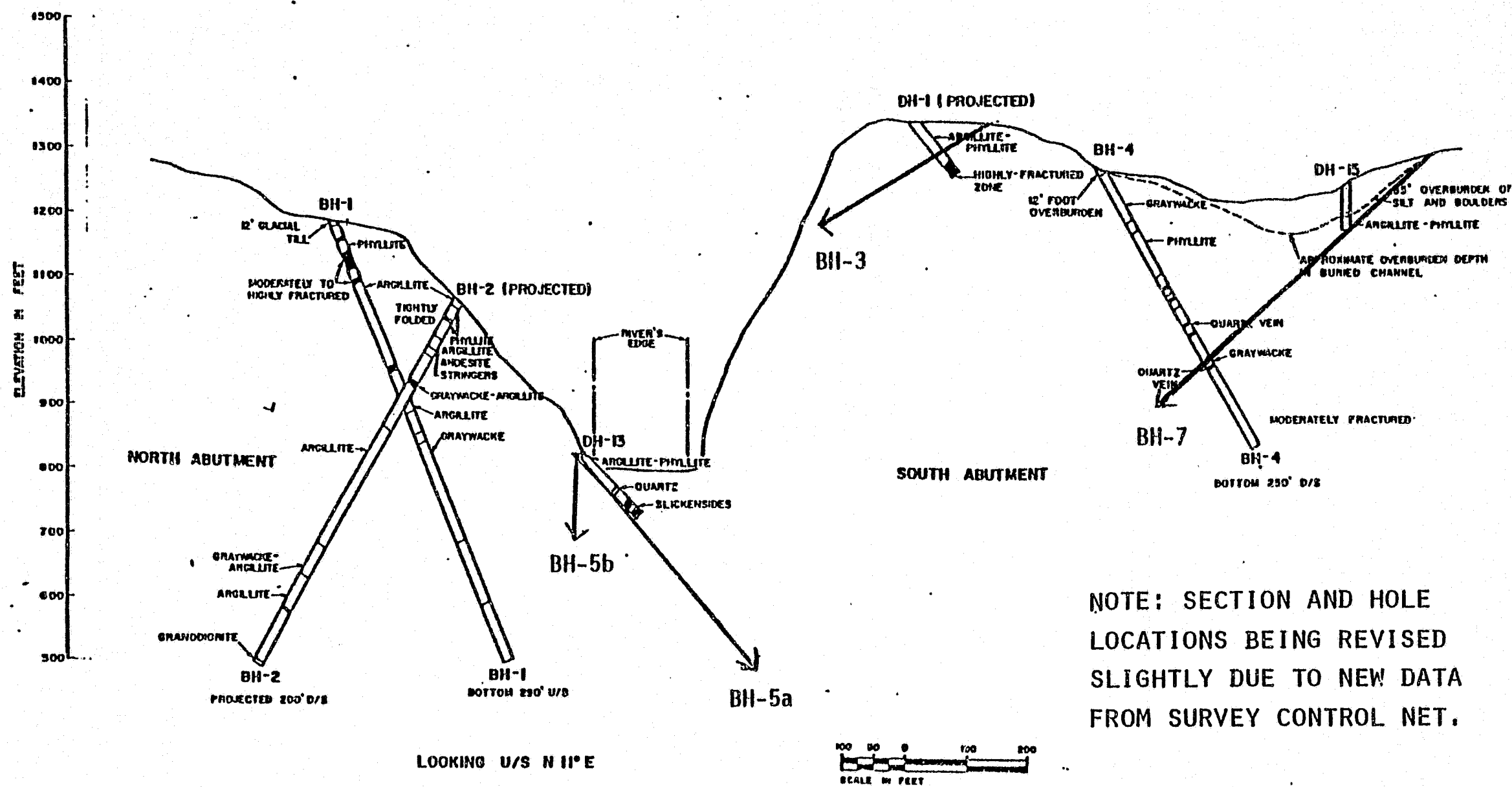
DEVIL CANYON SITE

1981 GEOTECHNICAL INVESTIGATIONS

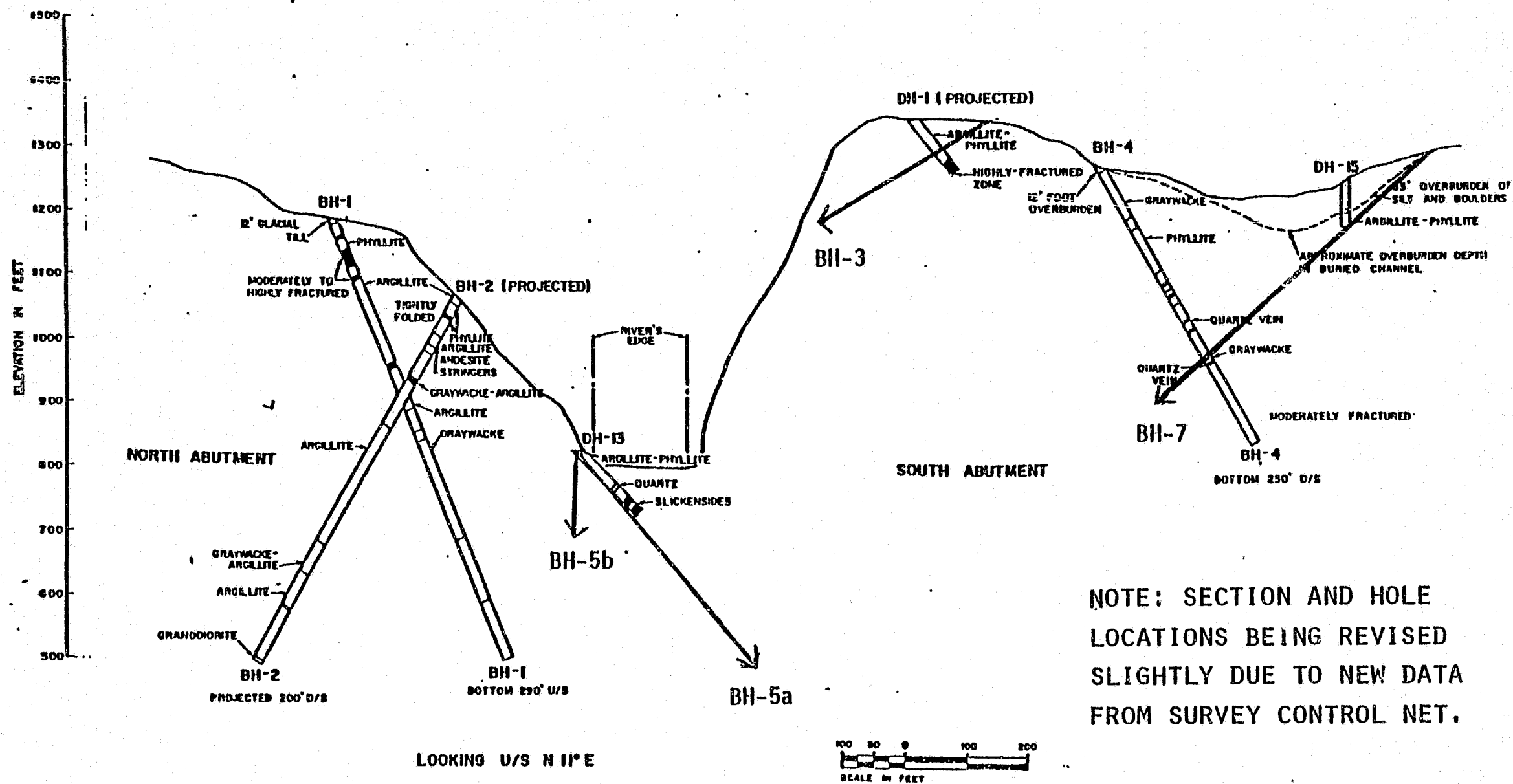
PREVIOUS	1981
● BOREHOLES ■ TEST PITS	○ BOREHOLES △ TEST PITS
— SEISMIC LINES	- - - SEISMIC LINES

9/16/81





GEOLOGIC CROSS-SECTION DEVIL CANYON



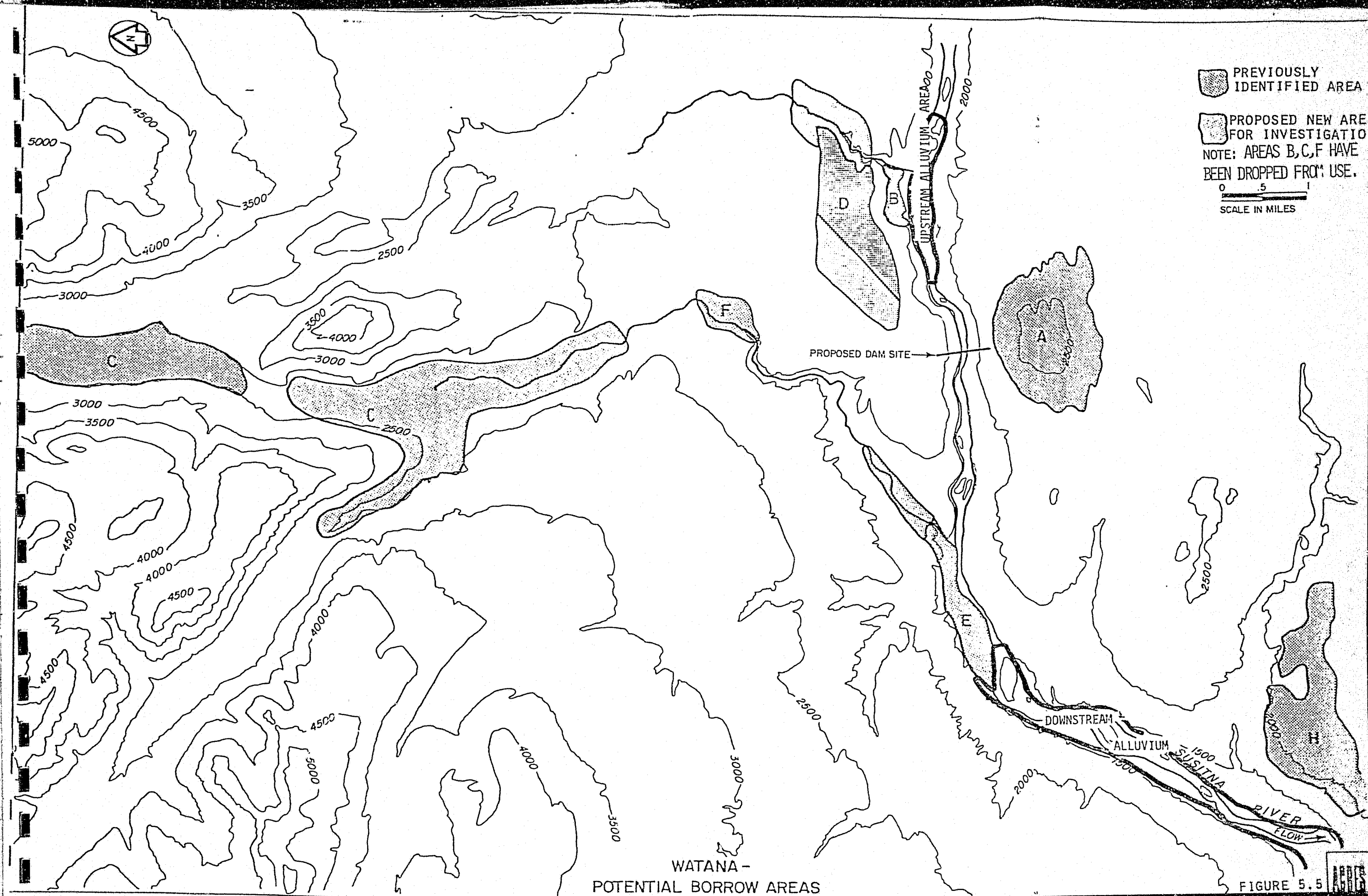
NOTE: SECTION AND HOLE LOCATIONS BEING REVISED SLIGHTLY DUE TO NEW DATA FROM SURVEY CONTROL NET.

REFERENCES: 1 BUREAU OF RECLAMATION, 1980
2 CORPS OF ENGINEERS, 1978
3 SUMMER 1980 INVESTIGATION

GEOLOGIC CROSS-SECTION DEVIL CANYON

ACRES

5) WATANA DAMSITE MATERIALS



WATANA BORROW AREA EXPLORATIONS

(ALL INVESTIGATORS TOTAL)

<u>AREA</u>	<u>TEST PITS</u>	<u>SEISMIC LINES</u>	<u>AUGER HOLES</u>	<u>APPROX AREA</u>	<u>EST. VOL.</u>
A (QUARRY)	NOT NECESSARY - LARGE AREA OF EXPOSED OUTCROPS.			700+ ACRES	100+ MCY (1.13MCY/FT OF DEPTH)
B (QUARRY)	COE SITE - SEISMIC LINES SHOWED EXCESSIVE DEPTH OF OVERBURDEN - ABANDONED AS POSSIBLE AREA.				
C (GRAVELS)	1	3	0	1000-1500	200-400 MCY COE FILTER & AGGREGATE SOURCE - 13 MILE HAUL NOT CONSIDERED AT THIS TIME - AREA E MORE USEFUL.
D (CORE MTL)					
COE	14	6	24	630	20+ MCY
ACRES	2(GRAB)	6	14	650+	73+ MCY (1MCY/FT)
E (FILTER & AGG)					
COE	6	5	0	550±	NOT CALC
ACRES	21	3	9	600±	28.5 MCY (.95MCY/FT)
F (FILTER&AGG)	6	0	0	130	NOT KNOWN
COE AREA, 10 MILE HAUL, UNKNOWN DEPTH AND AREA. NOT CONSIDERED FOR USE AT THIS TIME.					
H (CORE MTL)ACRES	2(GRAB)	0	9	350-1000	14-40 MCY (20-50' DEEP)
RIVER ALLUVIUM (SHELL GRAVELS)	IN PROGRESS	12	0	210 UPSTREAM 1770 DOWNSTREAM	19+ MCY 145+MCY (AVG 50')

9/16/81

WATANA DAM
EMBANKMENT QUANTITIES
(x10⁶ CY)

(DEPENDENT ON FINAL LAYOUT REFINEMENTS)

TYPE OF MATERIAL	REQUIRED	EST. AVAIL.	SOURCE
IMPERVIOUS	10-15	50-75+ 15-40	AREA D AREA H
FINE FILTERS	5-6	12.5	AREA E
COARSE FILTERS	1-2	16 40+	AREA E RIVER ALLUVIUM
ROCKFILL GRAVEL FILL	55-60	100+ 47-104 164+	QUARRY A WITHIN 6 MILES OF AXIS, IN RIVER. WITHIN 11 MILES.
CONCRETE AGG.	<1 MCY	10+	AREA E (SAME AS FILTER SOURCE)
ROUGH TOTAL	71-83 MCY	2-5 TIMES REQUIREMENTS, BASED ON PRELIMINARY TAKEOFFS.	

9/16/81

WATANA DAMSITE - AREA D

GENERAL MATERIAL PROPERTIES

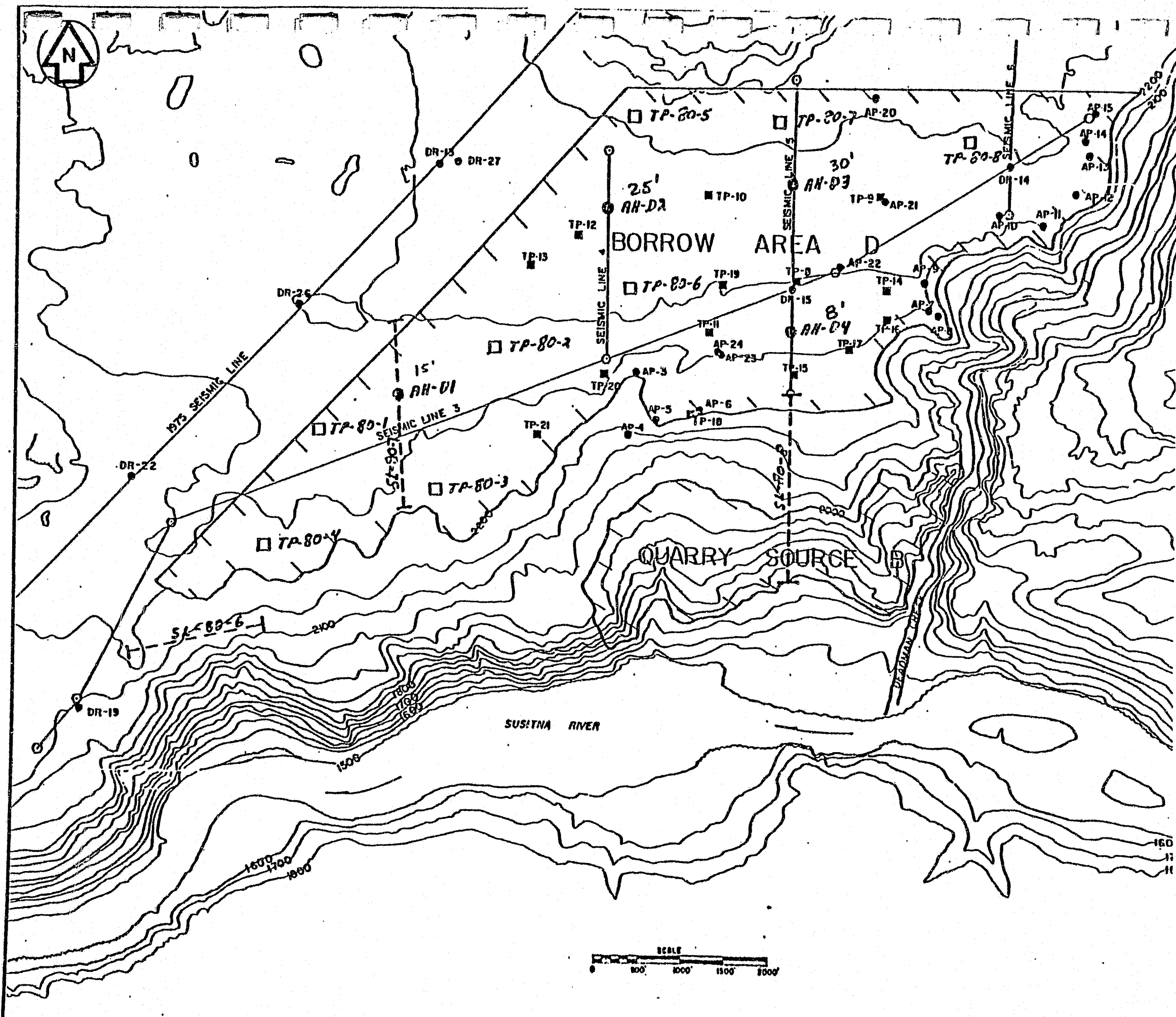
(CORE MATERIAL SOURCE)

FROM THE DATA TO DATE (MOST OF THE 1981 SAMPLES ARE BEING TESTED AT THIS TIME) IT APPEARS THAT THE MOISTURE WILL BE THE MOST CRITICAL FACTOR IN USING THIS MATERIAL. THE TESTS INDICATE A DIFFERENCE IN PROPERTIES BETWEEN THE SHALLOW OUTWASH MATERIAL, WHICH IS LESS FINE GRAINED AND WETTER, AND THE MATERIAL BELOW ABOUT 20-30 FEET IN DEPTH. THE MOISTURE CONTENT ALSO APPEARS TO BE RELATED TO EITHER FROZEN MATERIAL OR OVERCONSOLIDATED MATERIAL, OR BOTH. FURTHER DATA IS NEEDED TO CONFIRM IF THESE ARE TRENDS.

MOISTURE CONTENTS -	0-20' 7-25%	20-75' 6-12%	
LIQUID LIMITS- EXTREME	14-56	13-23	
NORMAL	14-23	13-17	
PLASTIC LIMIT -EXTREME	11-22	13-15	
NORMAL	11-22	13-15	SIGNIFICANT NUMBER OF :
PLASTICITY INDEX-EXTREME	0-33	2-7	SAMPLES THAT
NORMAL	2-6	2-4	DID NOT SHOW ANY PLASTICITY.

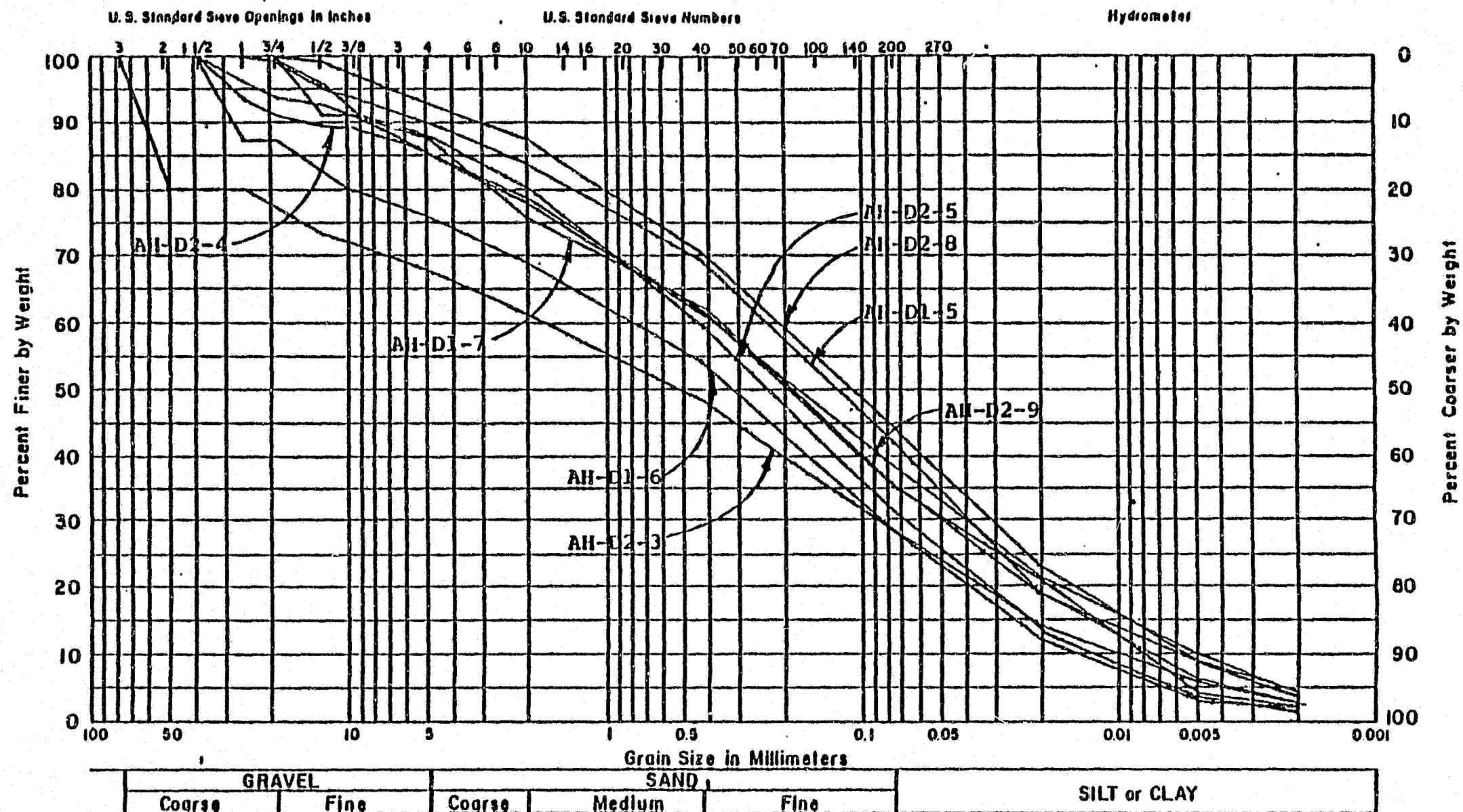
GRADATION CURVES SHOWING TYPICAL AND A FULL ENVELOPE OF SAMPLES TO DATE (COE AND ACRES) FOLLOW. DATA OBTAINED BY THE COE IS PRESENTED BELOW ON CORE PROPERTIES FOR DESIGN - ACRES TESTING IS IN PROGRESS.

WATANA DAM SITE
BORROW AREA D
15-1981 AUGER HOLES NOT SHOWN

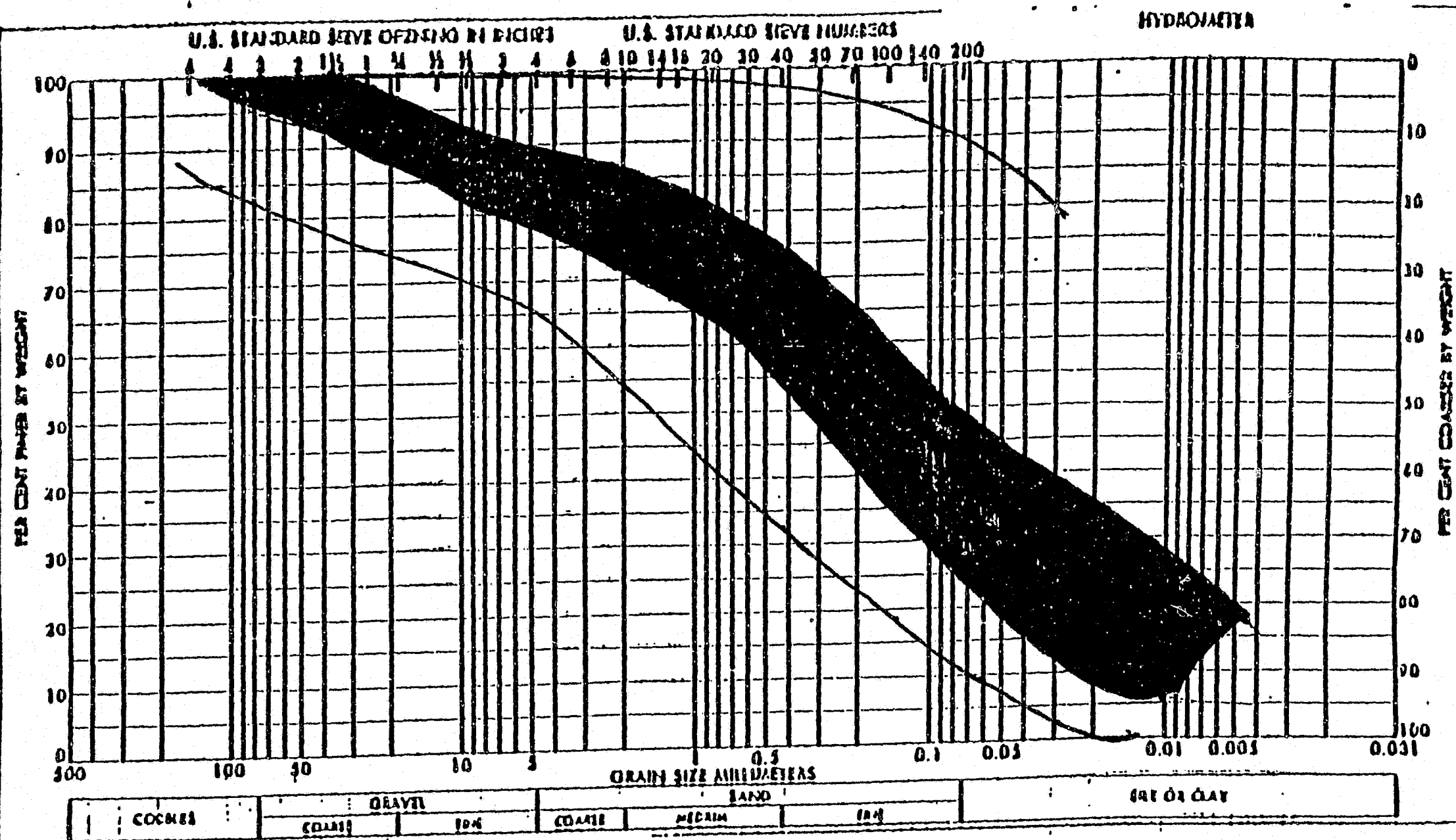


SCALE
0 500 1000 1500 2000

AFR



SAMPLE NO.	MOISTURE CONTENT	DRY DENSITY	LL	PI	CLASSIFICATION & DESCRIPTION	
AII-D1-5					SM	TYPICAL GRADATIONS - APPARENT VARIATION WITH DEPTH LIMITED TO MOISTURE CONTENT - AND ATTERBERG LIMITS. GRADATIONS FOLLOW FAIRLY NARROW GRADATION.
AII-D1-6					SM	
AII-D1-7					SM	
AII-D2-3					SM	
AII-D2-4					SM	
AII-D2-5					SM	
AII-D2-8					SM	
AII-D2-9					SM	
R & M CONSULTANTS, INC.					BORROW AREA D	
					SUMMARY OF GRAIN SIZE DISTRIBUTIONS	
					DRAWN BY DL	
					APPROVED BY	
					DATE DEC. 1980	
					PROJECT NO. 052504	



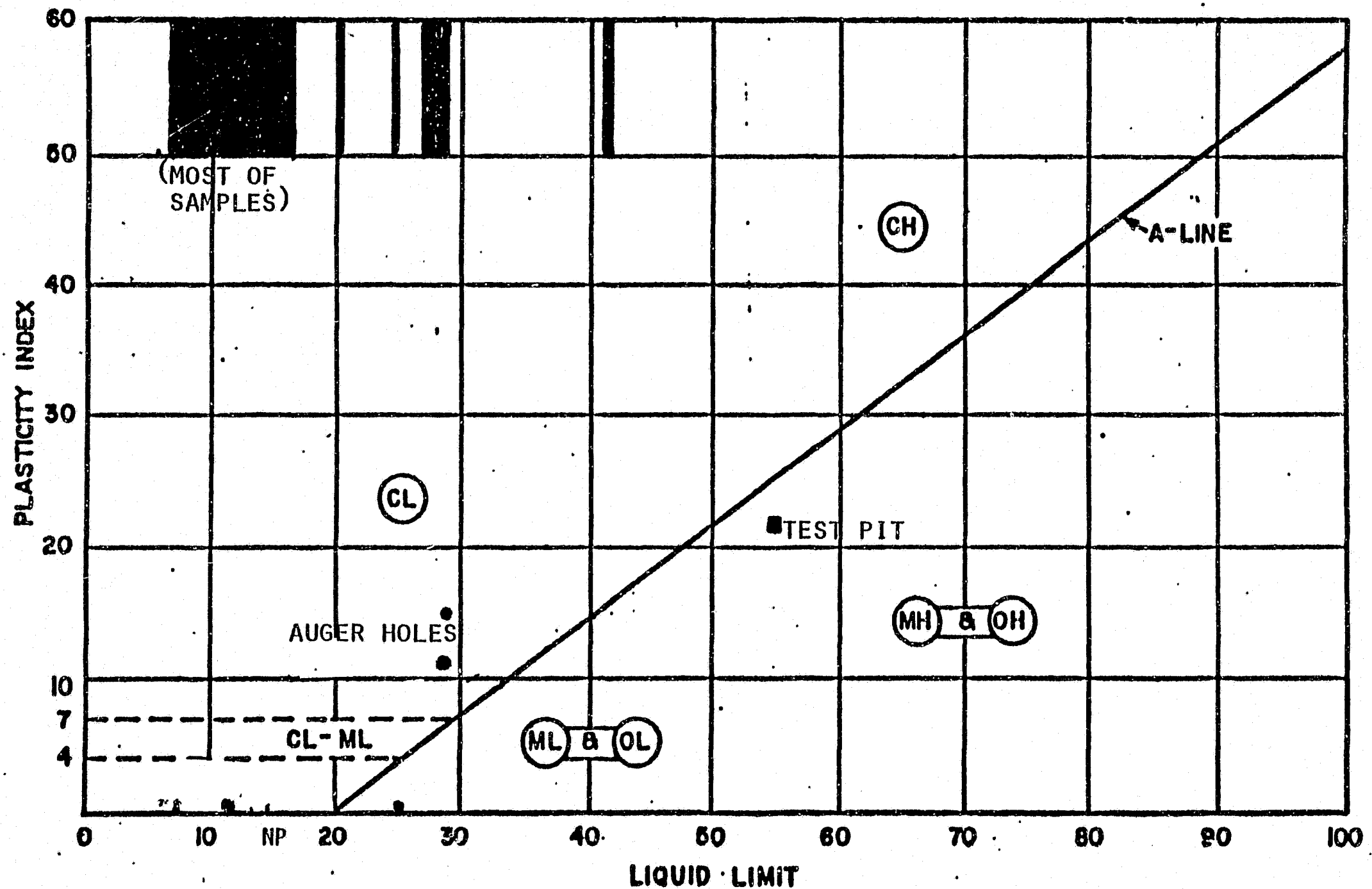
ENVELOPE OF GRADATIONS - WATANA AREA D. SOLID ZONE IS COE SAMPLES, OUTER LINES ARE EXTREME RANGE OF ACRES INVESTIGATIONS. MAJORITY OF SAMPLES FOLLOW SOLID BAND.

BORROW AREA D

ATTERBERG LIMITS FOR MATERIAL 0-10 FEET

Natural Water Contents

#200 SIEVE

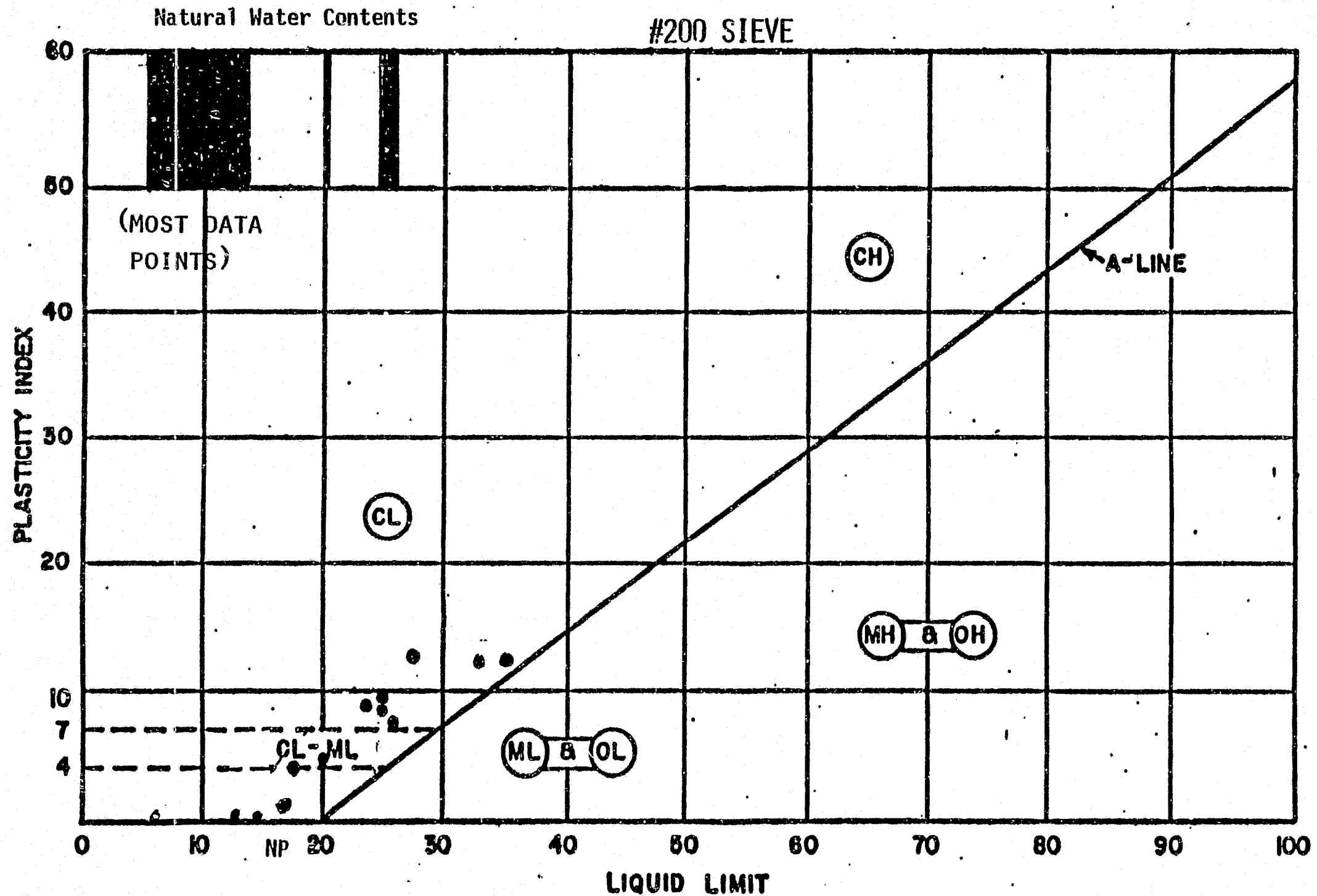


AH = Auger Hole Results
TP = Test Pits
NP = Non Plastic Samples

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BORROW AREA D

ATTERBERG LIMITS FOR MATERIAL 10-20 FEET



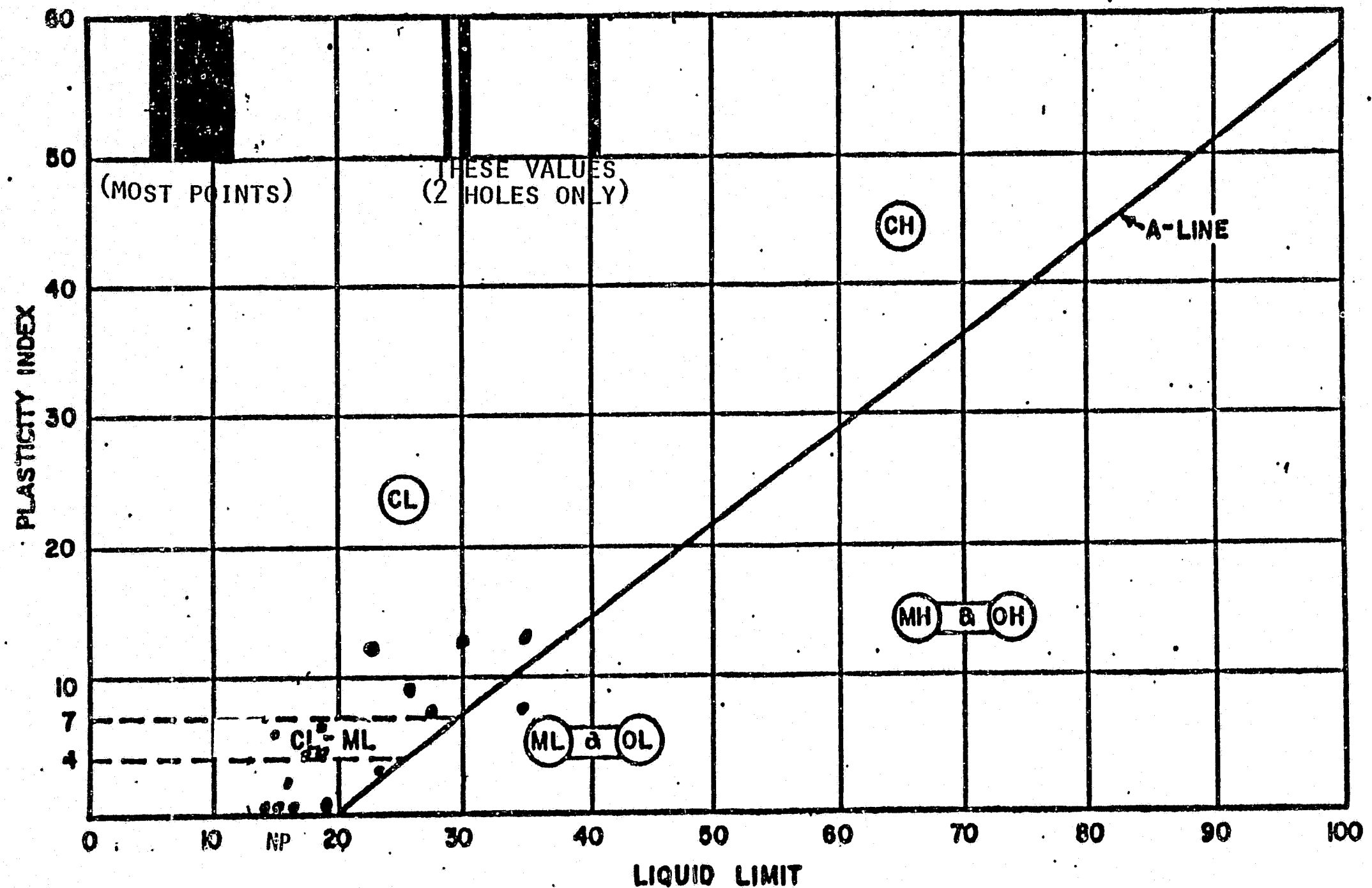
9/16/81

BORROW AREA D

ATTERBERG LIMITS FOR MATERIAL >20 FEET

Natural Water Contents

#200 SIEVE



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PRELIMINARY DESIGN VALUES
WATANA - AREA "D" SAMPLES

OPTIMUM MOISTURE CONTENT (95% STD PROCTOR)	6 % (4" MOLD) 7.5% (6" MOLD)
OPTIMUM DENSITY	129-133 PCF
SPECIFIC GRAVITY (BULK SSD)	2.671
PERMEABILITY (MINUS 1 INCH) AT 126.6 PCF	1.0 x 10 ⁻⁵ CM/SEC

TRIAxIAL TEST DATA: UNSATURATED, UNCONSOL, UNDRAINED

ANGLE OF INTERNAL FRICTION. PHI AND COHESION, CONTROLLED STRAIN TEST PROCEDURE SHOWING RANGE OF TEST VALUES FOR VARIOUS MOISTURE ABOVE, AT, AND BELOW OPTIMUM.	35.5° , 0.14 TSF (OPTIMUM WATER-4%)
	33.5° , 0.66 TSF (OPTIMUM WATER)

: BACK PRESSURE METHOD
(CONSOLIDATED, UNDRAINED TEST)

1.75° , 0.44 TSF
(OPTIMUM + 4%)

12.3° , 1.07 TSF
(OPTIMUM WATER)

12.8° , 0.52 TSF
(OPTIMUM MINUS 4%)

CONSOLIDATION - PERCENT AT	1 TSF	0.85%	0.76%	2.12%
	10 TSF	2.38	2.46	4.49
	32 TSF	4.82	4.83	6.86
		(OPT-4%)	(OPT.W _C)	(OPT+4%)

R&M Consultant Inc.

LABORATORY COMPACTION CONTROL REPORT

Job Name and Location Susitna (Matana Dam Site)

Architect or Engineer Acres American Inc.

Contractor _____

A. Description of Soil: Well Graded 'Till' - GRAVELLY, SILTY SAND W/TRACE CLAY

Material Mark B Unified Classification SM AASHO Classification _____

Source of Material Deadman Creek Sample No. W-80-300 (Area D)

Natural Water Content 6.6% Natural Dry Density _____ PCF Specific Gravity _____

Liquid Limit Non Viscous % Plastic Limit _____ % Plasticity Index Non Plastic

B. Test Procedure Used T-180 "B" - AASHTO

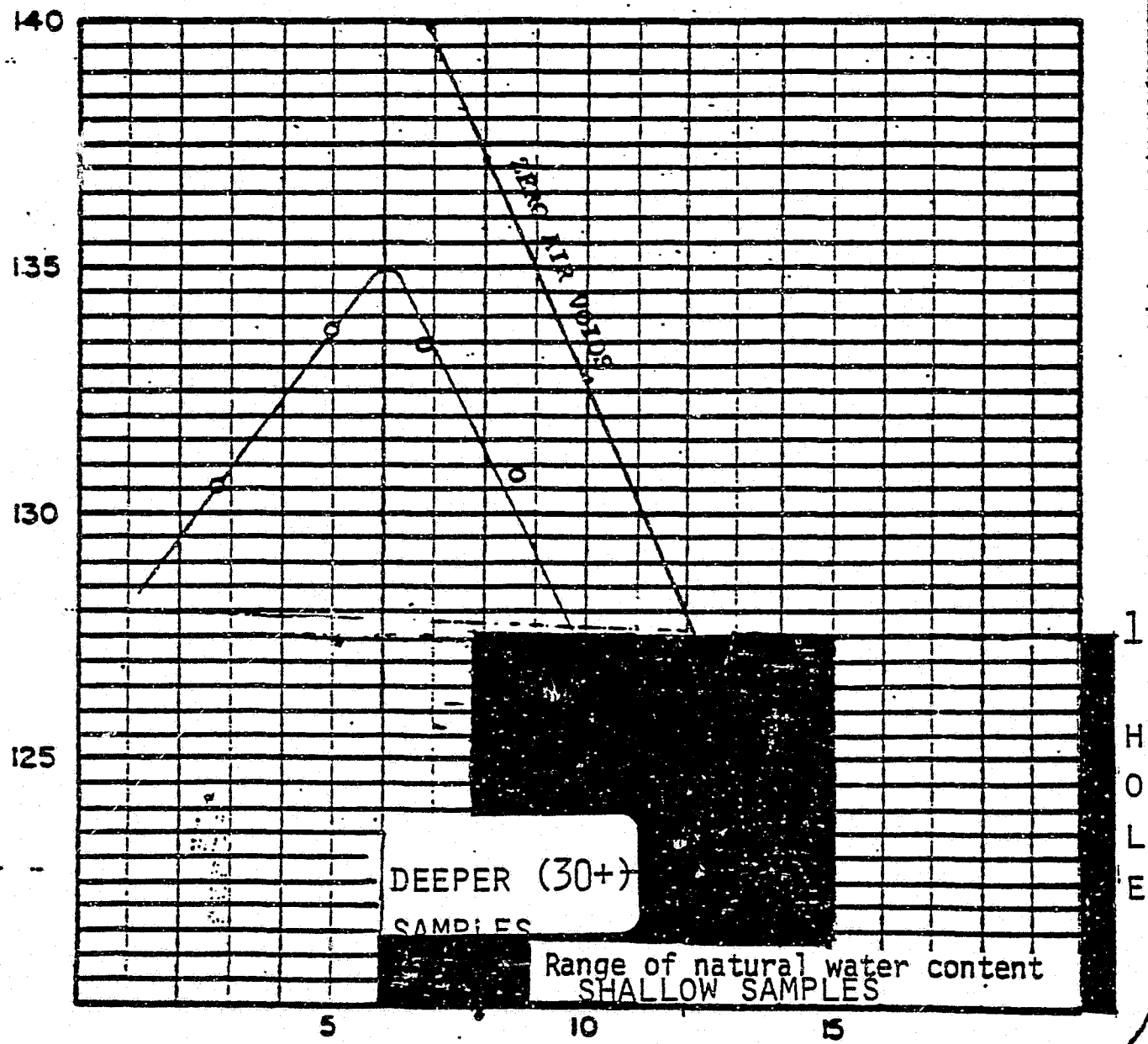
C. Test Results: Maximum Dry Density 135.0 PCF Optimum Water Content 6.0

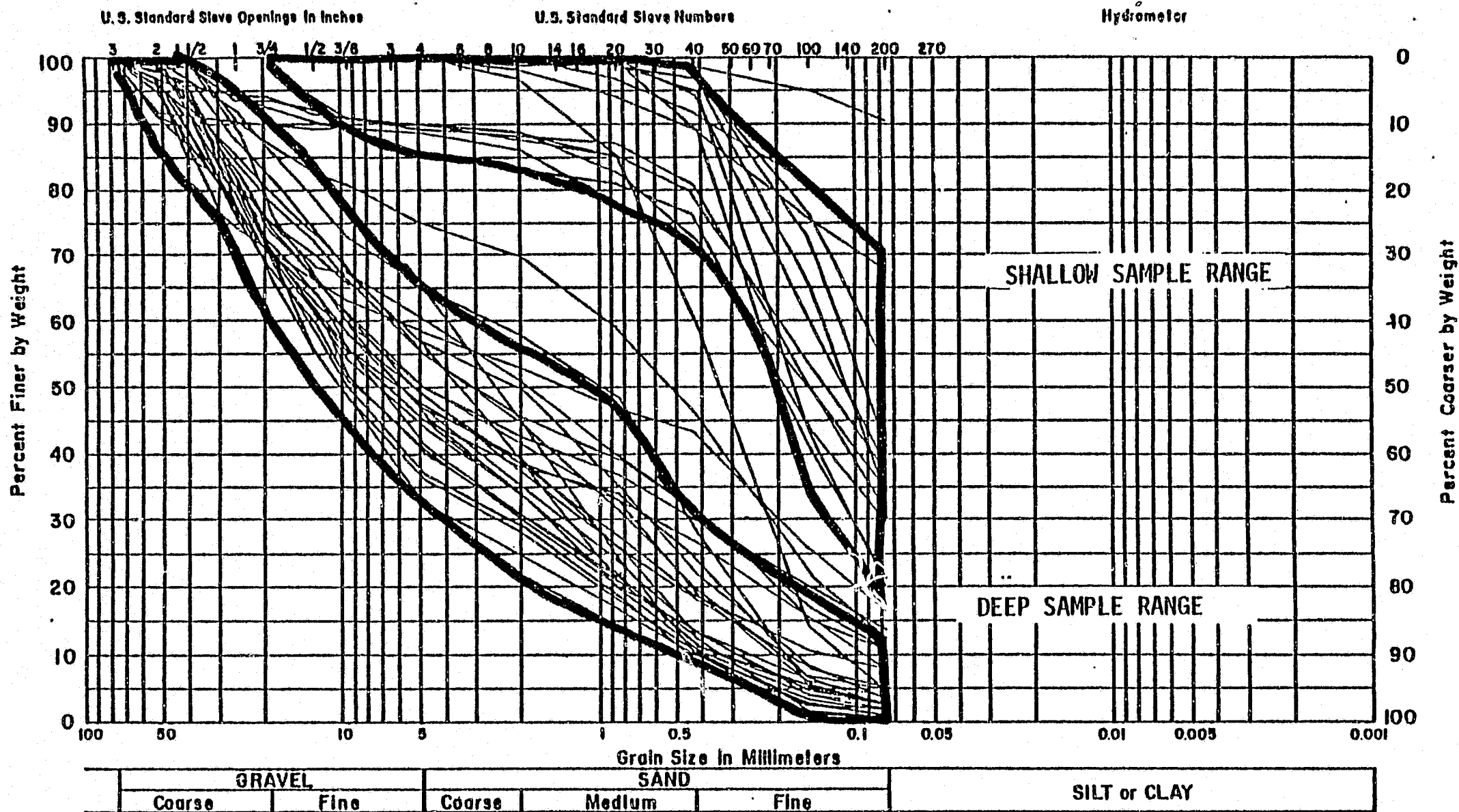
Sieve Analysis

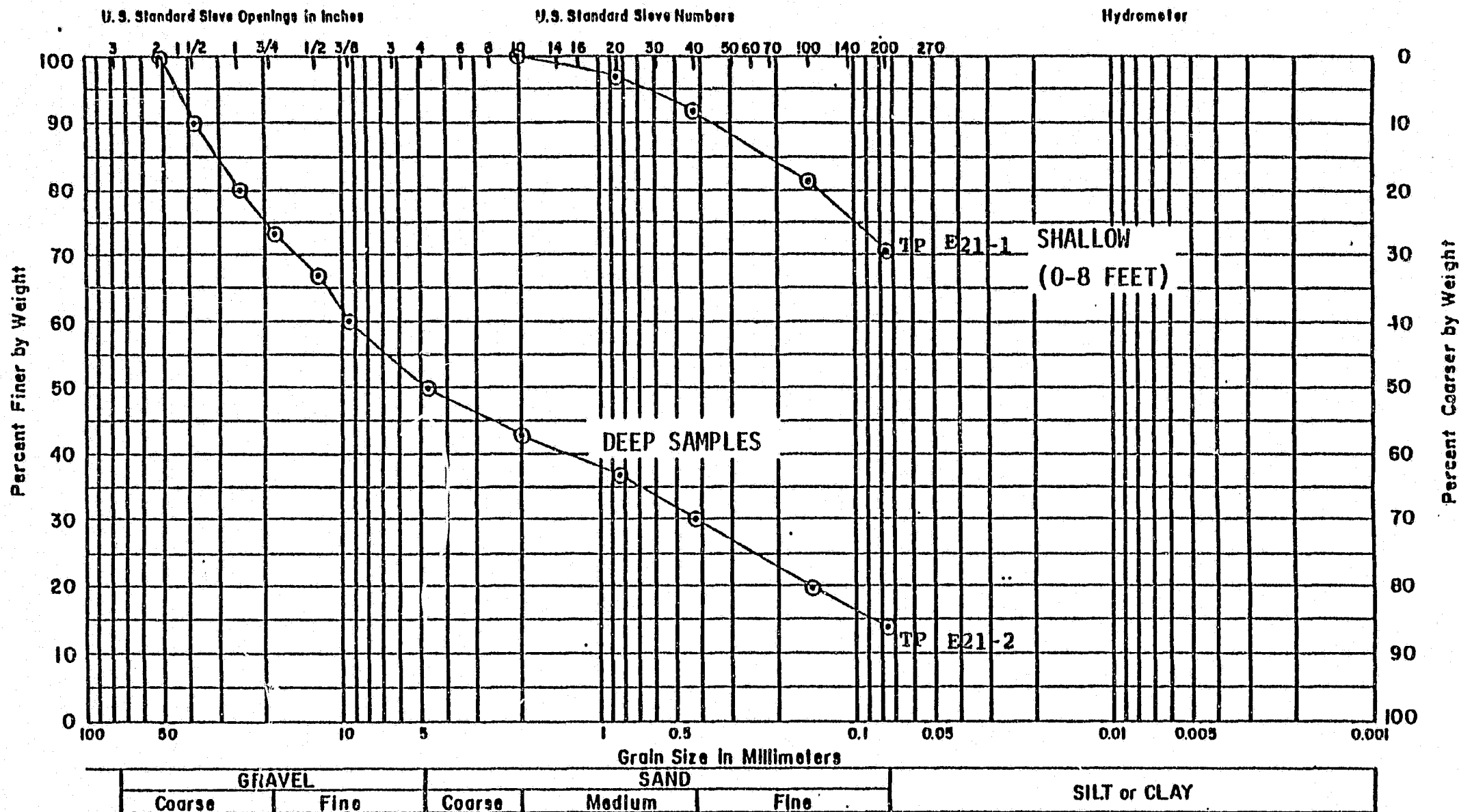
Size % Passing

2"	100
1 1/2"	95
1"	93
3/4"	89
1/2"	87
3/8"	86
# 4	80
# 10	76
# 40	58
# 200	26.9
.02mm	9.2
.005	3.0
.002	1.3

DRY DENSITY - LBS PER CUBIC FOOT

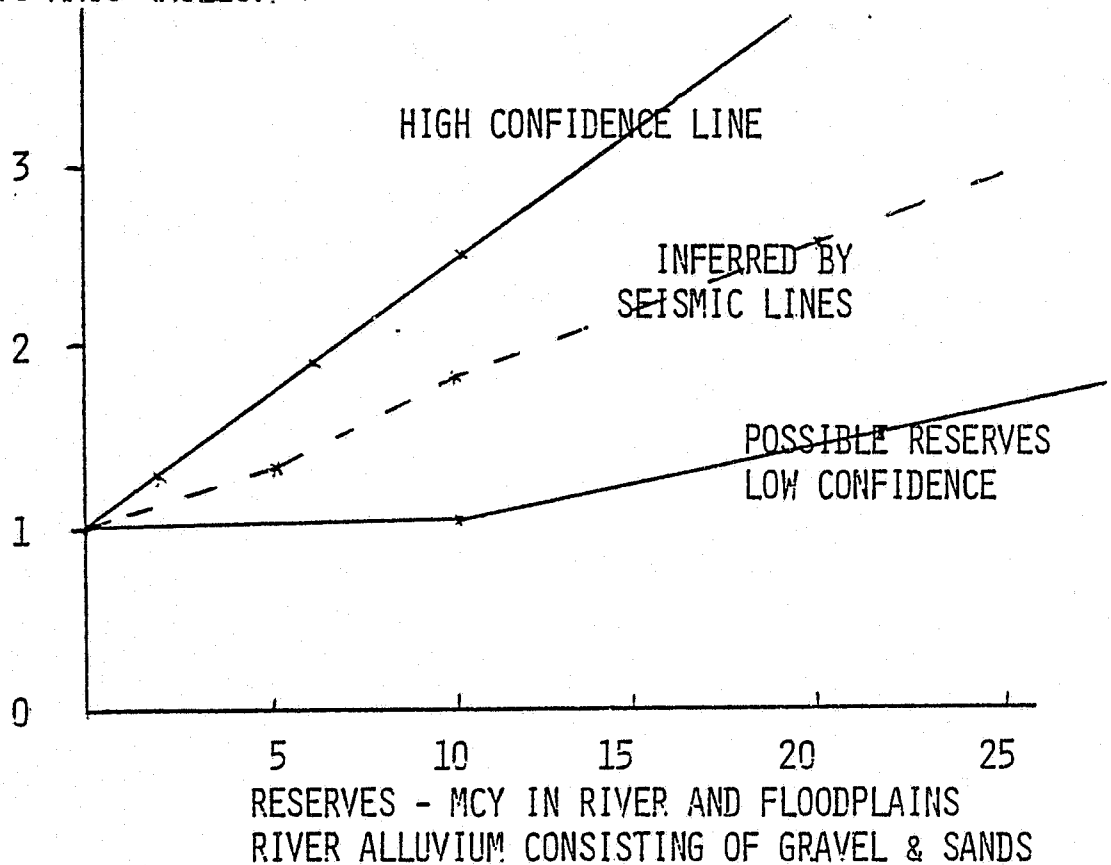






WATANA DAMSITE
UPSTREAM RIVER ALLUVIUM
QUANTITY -DISTANCE CHART

MAXIMUM HAUL DISTANCE
TO AXIS (MILES).



NOTE: FOR A SKETCH OF THE AREA UNDER CONSIDERATION, SEE
"WATANA RIVER ALLUVIUM" 11x17 FIGURE. ENTIRE AREA
INVESTIGATED IS IN THE SKETCH.



DEADMAN CREEK

SUSITNA RIVER

SL81-2

1500

1400

1300

8000

7000

5000

1500

1400

1300

TERRACE

WATANA - UPSTREAM RIVER GRAVEL SOURCE

INFERRED MATERIALS

5000 FPS

RIVER ALLUVIUM, SANDY GRAVEL AS OBSERVED
IN GRAVEL BARS.

7000

SATURATED, COMPACTED GRAVELS OR TILL

8000

GRAVEL OR TILL. BOTH ZONES TAKEN AS TILL
UNTIL PROVEN OTHERWISE.

LS81-3

1500

1400

5000

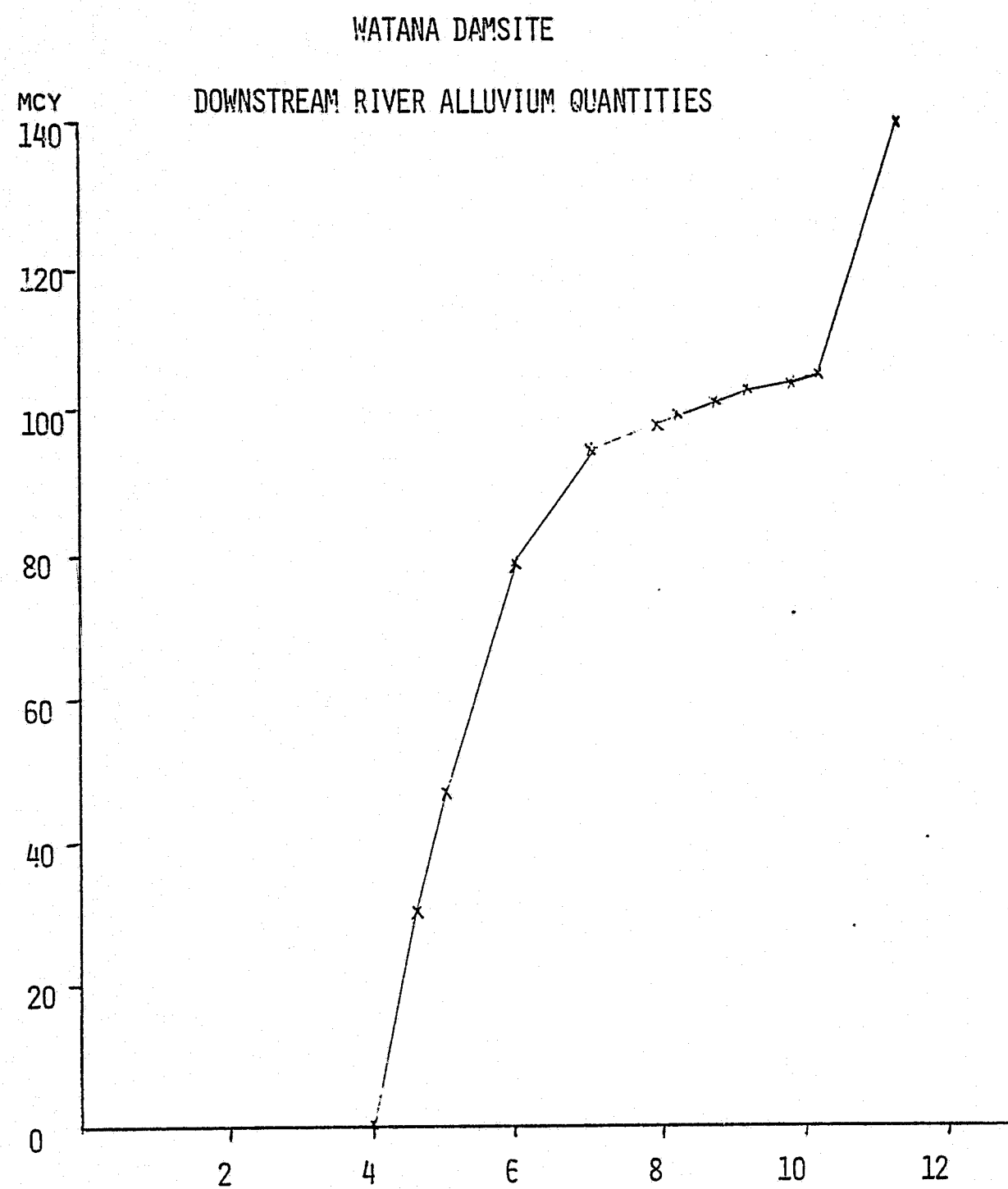
FLOW

THE LENS

WATANA - RIVER ALLUVIUM

UPSTREAM CORRIDOR

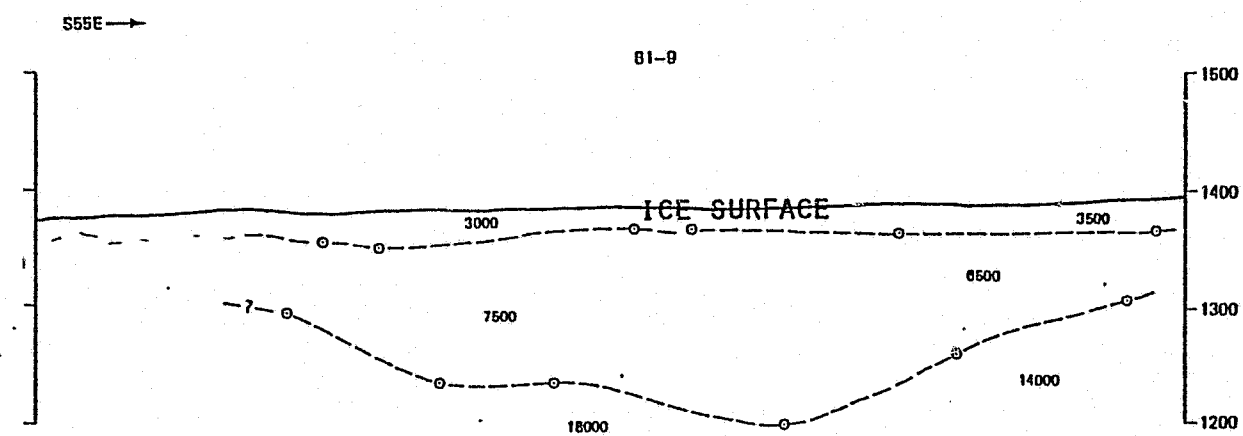
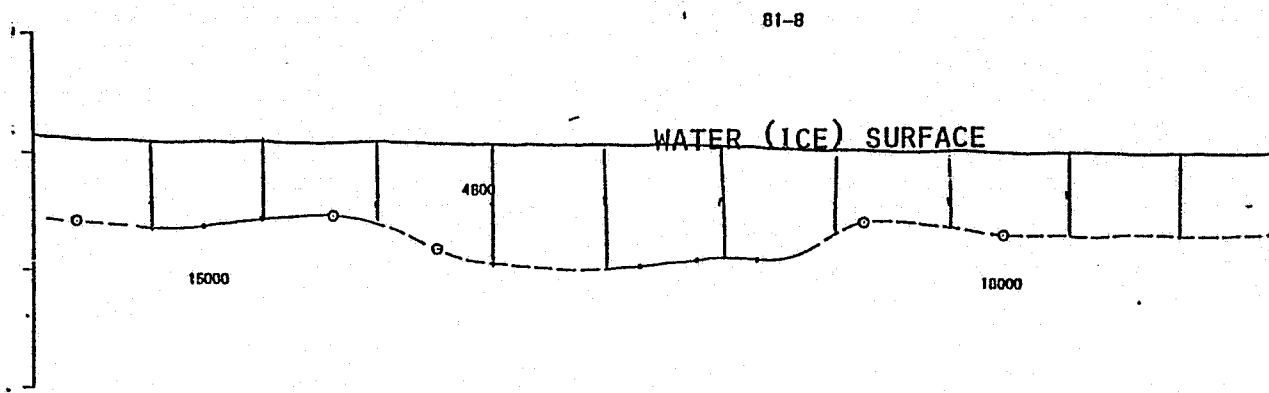
ACRES



MAXIMUM HAUL DISTANCE TO DAM AXIS

NOTE: FOR A SKETCH OF AREA APPEARANCE, SEE "WATANA RIVER ALLUVIUM" BORROW AREA E PLATE. ALLUVIUM BORROW AREA STARTS AT ISLAND SHOWN AND CONTINUES APPROXIMATELY 8 MILES DOWNSTREAM.

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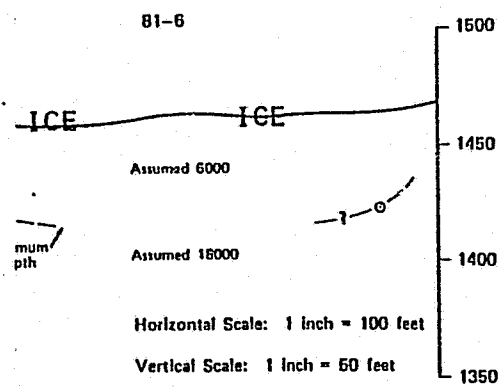
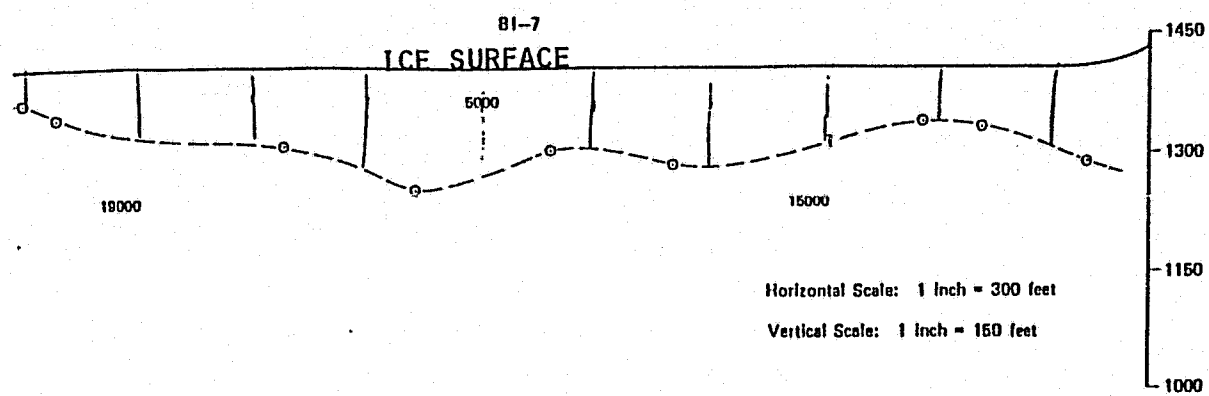


VELOCITIES IN FEET PER SECOND, SECTIONS LOCKING UPSTREAM

TYPICAL SEISMIC LINES - DOWNSTREAM ALLUVIUM

9/16/81

001380



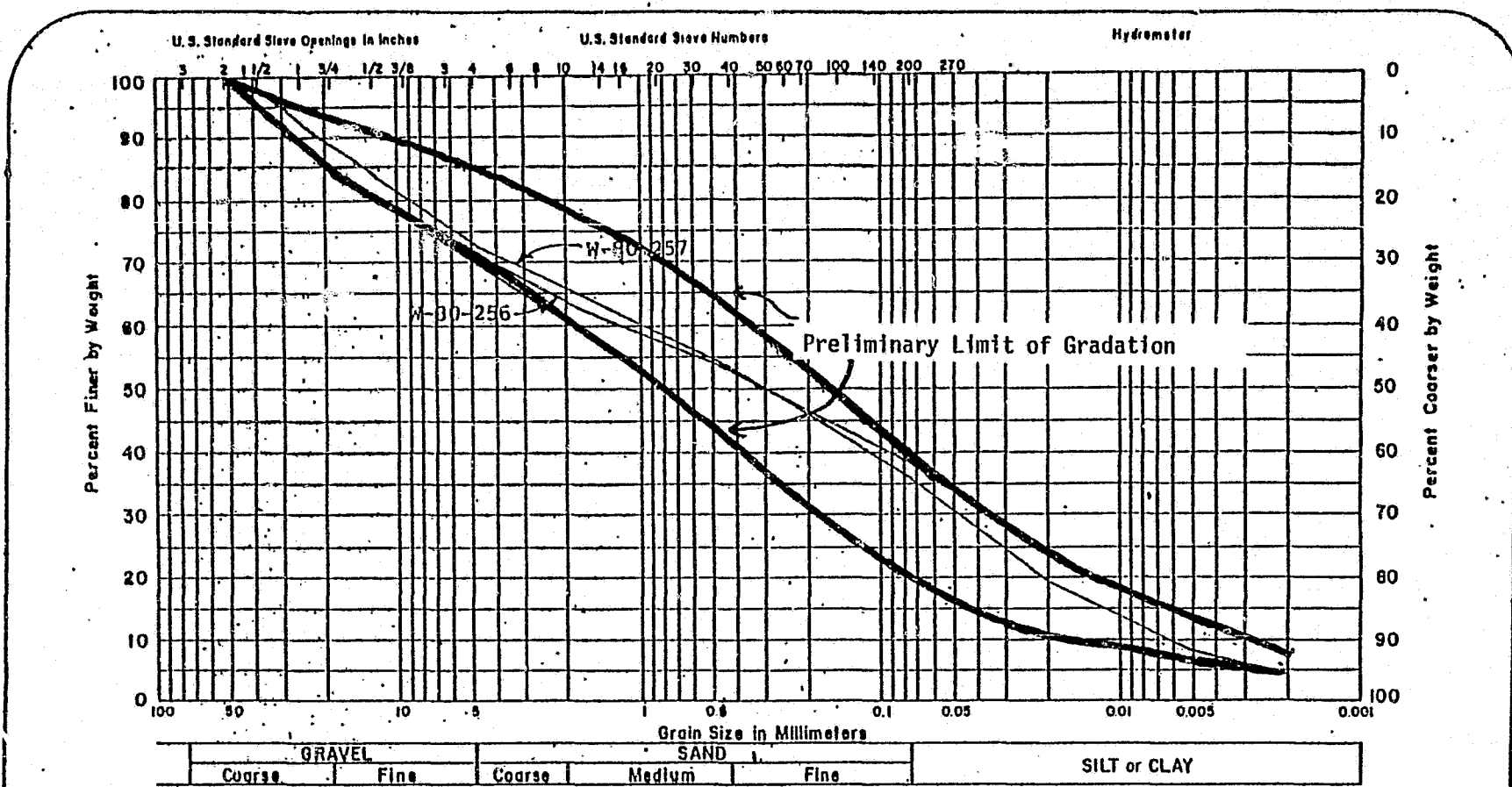
COMPRESSIONAL WAVE VELOCITIES IN FT/SEC

SECTIONS LOOKING UPSTREAM

TYPICAL SEISMIC LINE - DOWNSTREAM ALLUVIUM

9/16/81

001380



R&M Consultant Inc.
LABORATORY COMPACTION CONTROL REPORT

Job Name and Location Susitna (Watana Dam Site)

Architect or Engineer Acres American Inc.

Contractor _____

A. Description of Soil: Poorly Graded 'Till', SILTY GRAVEL AND SAND W/TRACE CLAY

Material Mark C Unified Classification GC-SC AASHO Classification _____

Source of Material Borrow Area B Sample No. W-80-256

Natural Water Content 10.9 % Natural Dry Density _____ PCF Specific Gravity _____

Liquid Limit 21.7 % Plastic Limit _____ % Plasticity Index 9.2

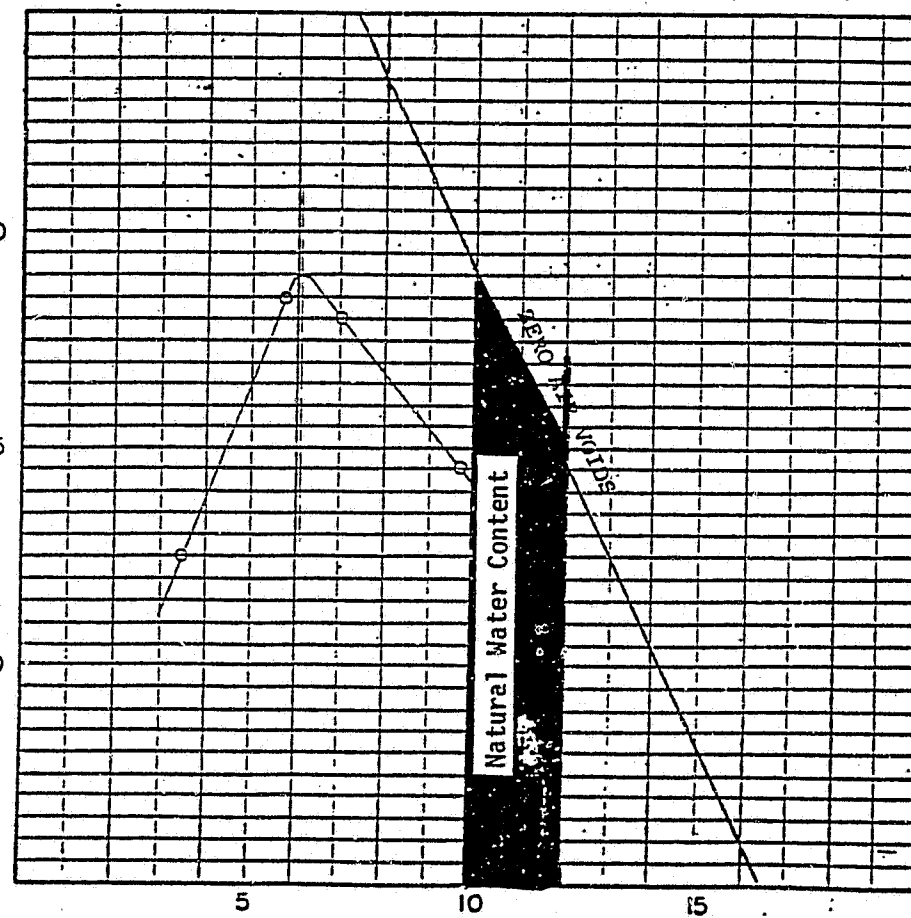
B. Test Procedure Used T-180 Method "p" - AASHTO

C. Test Results: Maximum Dry Density 139.0 PCF Optimum Water Content 6.2

Sieve Analysis

Size	% Passing
"	100
1/2"	95
3/8"	88
4"	84
2"	81
8"	78
4	71
10	64
40	53
00	38.2
.02mm	24.3
.005	13.6
.002	8.6

DRY DENSITY -- LBS. PER CUBIC FOOT



WATER CONTENT -- PERCENT OF DRY WEIGHT

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6) DEVIL CANYON DAMSITE MATERIALS



DEVIL CANYON DAMSITE BORROW AREAS

EXPLORATIONS

<u>AREA</u>	<u>TEST PITS</u>	<u>SEISMIC LINES</u>	<u>AUGER HOLES</u>	<u>APPROX AREAS</u>	<u>EST VOL</u>
G - ALLUVIAL FAN					
USBR	13	0	0	38 ACRES	NOT CALC
COE	RESAMPLED	2	0	38	6 MCY
ACRES	2 TRENCHES	0	4	38 TESTED	*
				34 ADDL INFERRED	
QUARRY	0	0	0	BEING MAPPED BY	
	EXPOSED BEDROCK			GEOLOGISTS AT	
				THIS TIME.	
CORE MATERIAL FOR SADDLE DAM					
ACRES	4	0	4	BEING EXPLORED	
				AT THIS TIME.	

* NOTE: SEE VOLUME - ELEVATION TABLE AND GRAPH FOLLOWING.

DEVIL CANYON DAM SITE
 CONCRETE AGGREGATE AND FILTER MATERIALS
 (x 10⁶ MCY)

DIVERSION POOL ELEV.	QUANTITY SAMPLED	QUANTITY INFERRED (INCLUDES SAMPLED)
920 FEET	2.8	5.0
935	2.0	4.0
950	1.4	2.9
980	0.5	1.5
1000	0.0	0.5

TOTAL CONCRETE AND FILTER REQUIREMENTS ESTIMATED FOR SITE:

CONCRETE	1.94 MCY
FILTERS	0.08 MCY
	2.02 MCY

9/16/81

DEVIL CANYON DAMSITE

AGGREGATE QUANTITIES

QUANTITY -ELEVATION CHART

DIVERSION HEADWATER
ELEVATION (FEET)

1000

980

960

940

920

(EXISTING)

1

2

3

4

5

6

RESERVES - MCY OF CONCRETE AGGREGATE AND EARTH DAM
FILTER MATERIAL

NOTE: ELEVATION OF DIVERSION
POOL MAY BE SET PARTLY
BY ECONOMICS OF BORROW
EXPLOITATION COSTS.

SAMPLED

INFERRED

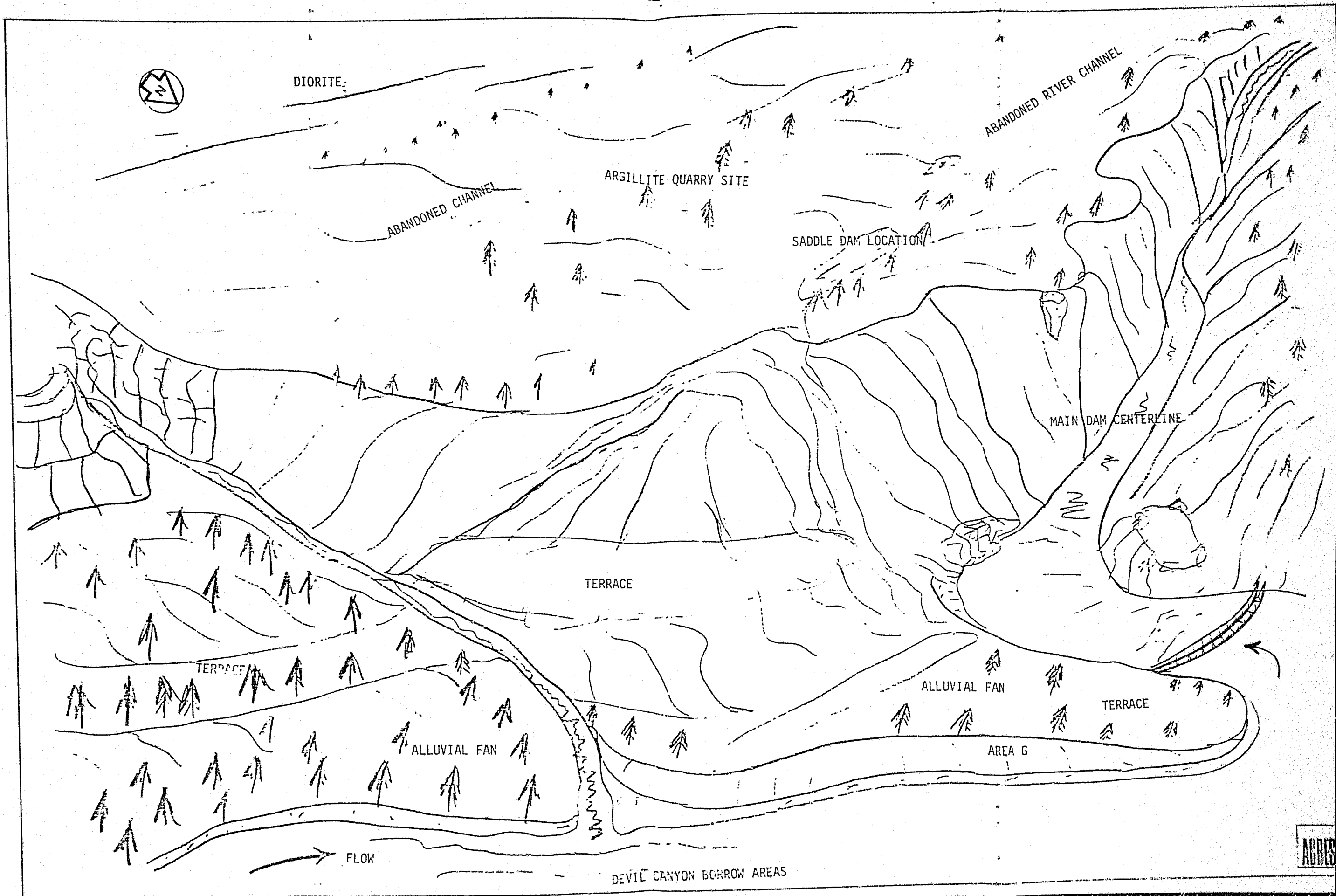
TOTAL REQUIREMENTS - ESTIMATED

CONCRETE 1.94 MCY
FILTERS 0.08 MCY

2.02 MCY

SOURCES: SAMPLED RESERVES ARE IN CHEECHAKO CREEK ALLUVIAL FAN
INFERRED RESERVES ARE IN ADJACENT TERRACES, FAN DEPOSITS.

9/16/81



ACRES

7) WATANA DAMSITE RELICT CHANNEL

WATANA DAMSITE
RELICT CHANNEL CUTOFF SCHEME

DETAILS OF GENERAL SLURRY WALL & GROUT CURTAIN CUTOFF PROPOSED
IF FURTHER INVESTIGATIONS (IN DESIGN STAGE) SHOW THE NEED.

TOTAL LENGTH OF CUTOFF	14275 LF	ALL NUMBERS
SLURRY TRENCH LENGTH	14275 LF	SUBJECT TO
GROUT CURTAIN LENGTH	6000 LF	REFINEMENT
SADDLE DAM LENGTH	2300 LF	PENDING FINAL
(MAX 23', AVERAGE 20' HIGH)		INTERPRETATION
SHORE RIPRAP PROTECTION	2800 LF	OF MAPPING IN
		PROGRESS, AND
		RECEIPT OF 1981
		SEISMIC LINE
		REPORT.

RELICT CHANNEL SPECIFICS (ASSUMING 2215 NORMAL OPERATING POOL,
2235' MAX SURCHARGE ELEVATION.)

MAXIMUM OBSERVED DEPTH	454 FEET FROM SURFACE
DEPTH UNDER SADDLE DAM	450 FEET FROM POOL ELEV.
	0-245' POSSIBLE
	145 AVERAGE BASED ON
	INTERPOLATION.
SHORTEST PATH FROM POOL TO OUTLET (TSUSENA CREEK)	6200 LF
ESTIMATED FLOW DISTANCE	7700 LF AT DEEPEST POINT
ESTIMATED WIDTH (POOL ELEV)	14275 LF
HEAD LOSS - MAX OPER POOL	590 FEET
- MAX FLOOD POOL	610 FEET

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LEGEND

DATA POINTS

- DRILL HOLE
- △ SEISMIC LINE STATION

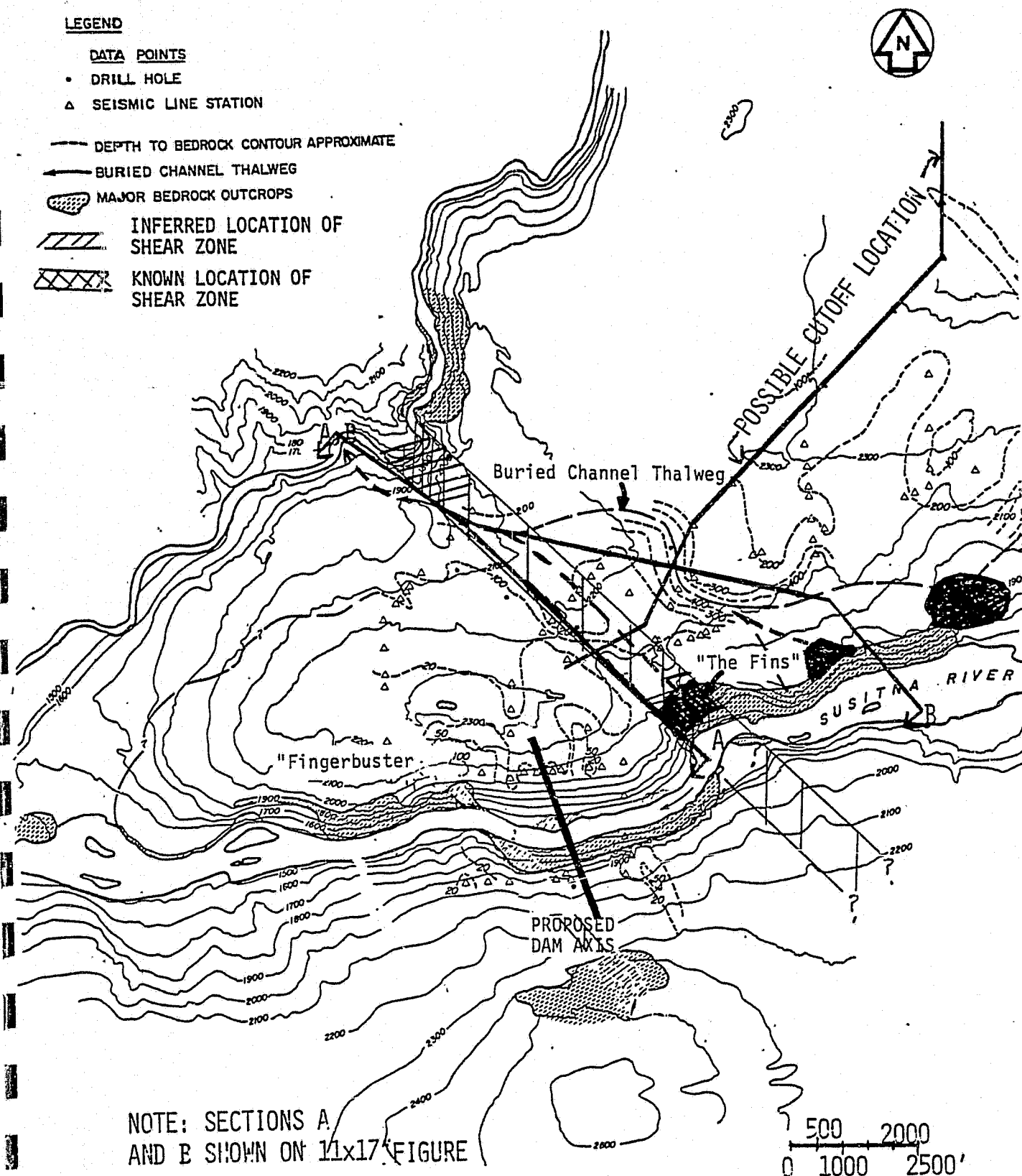
— DEPTH TO BEDROCK CONTOUR APPROXIMATE

— BURIED CHANNEL THALWEG

MAJOR BEDROCK OUTCROPS

INFERRED LOCATION OF SHEAR ZONE

KNOWN LOCATION OF SHEAR ZONE



NOTE: SECTIONS A
AND B SHOWN ON 11x17 FIGURE

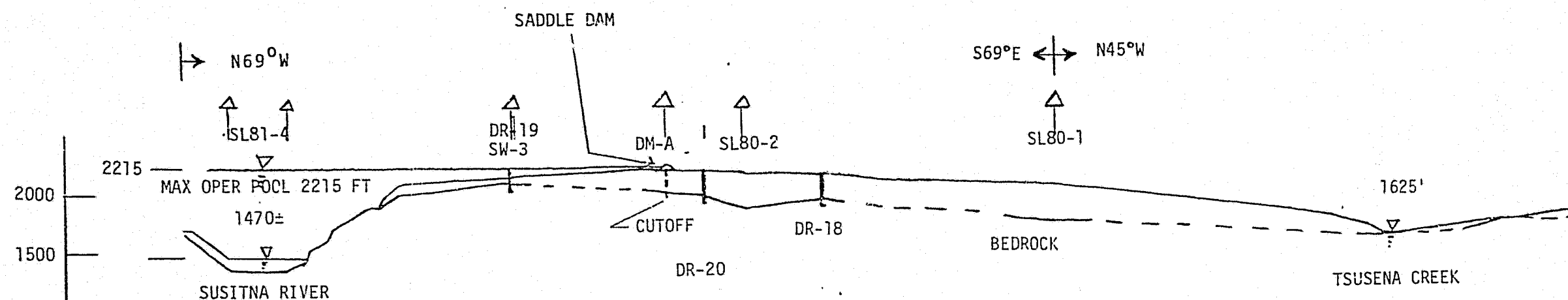
PROBABLE
RELICT CHANNEL
ENTRANCES

WATANA --- SPECIAL FEATURES

ACRIS

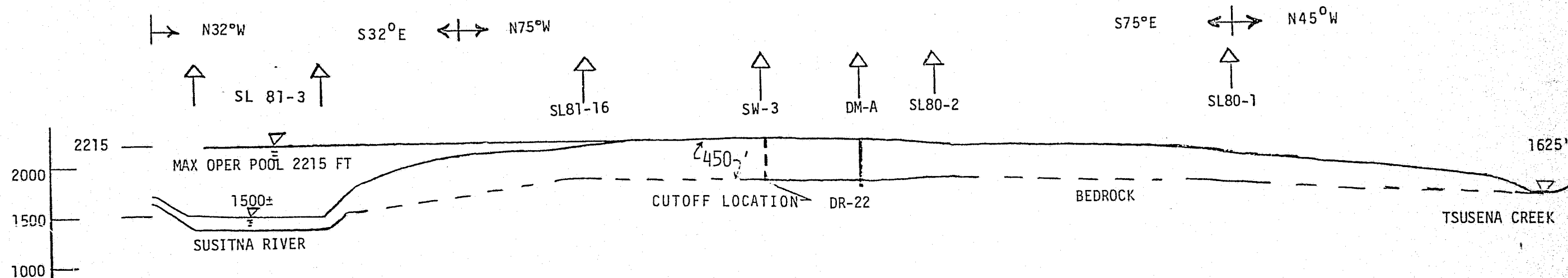
0/16/81

WATANA - RELICT CHANNEL PROFILE



EXISTING EDGE OF RIVER TO TSUSENA CREEK = 9300'
OPERATING POOL = 6200'

SECTION A - A
"THE FINS" to TSUSENA CREEK
DIRECT LINE, SHORTEST DISTANCE ON UPSTREAM
FLANK OF DAM SITE PLUTON



EXISTING EDGE OF RIVER TO TSUSENA CREEK = 12,000'
OPERATING POOL TO TSUSENA CREEK = 7,700'

SECTION B - B
SECTION ON APPROXIMATE THALWEG, LOCATION ABOUT
4000' UPSTREAM OF UPSTREAM COFFERDAM

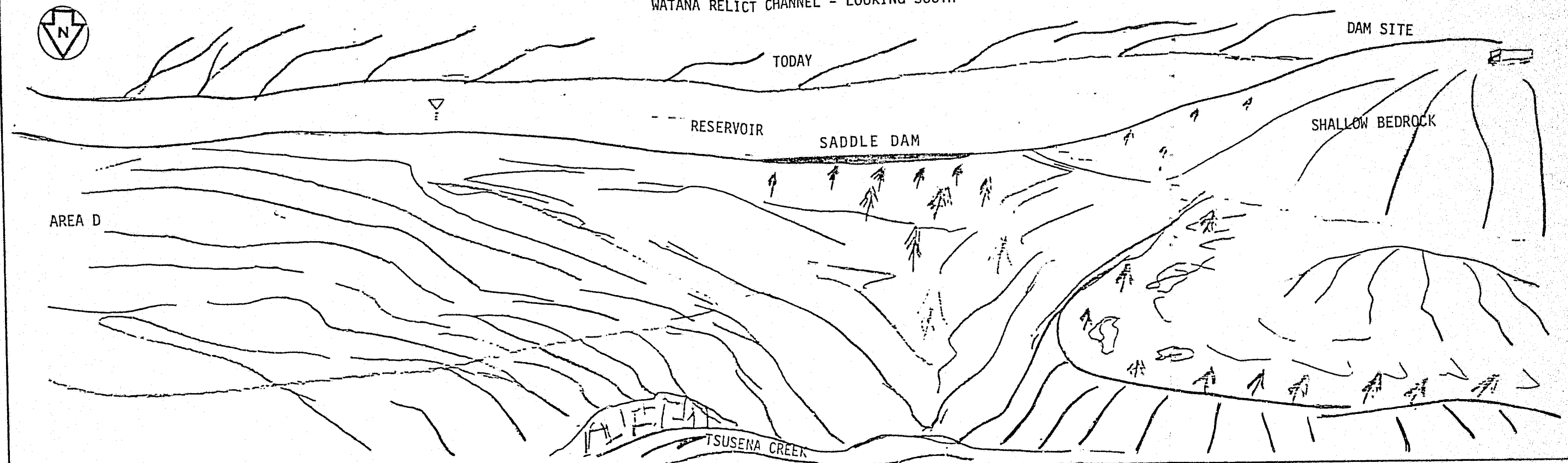
1"=1000'

LOOKING DOWNSTREAM

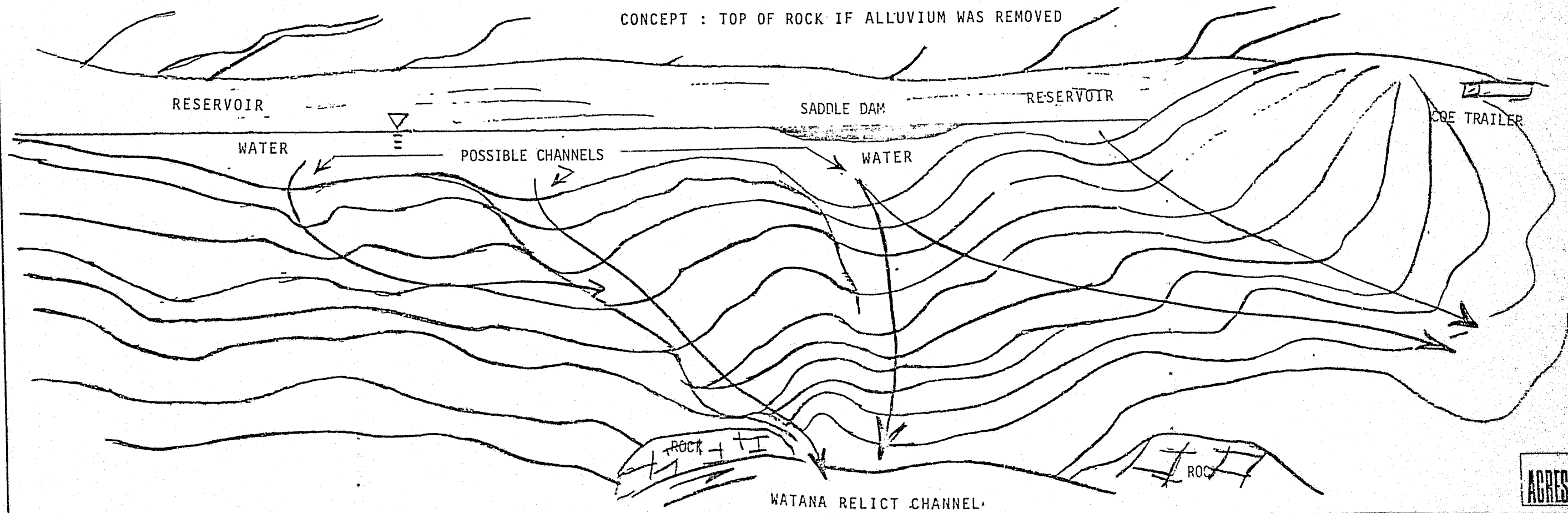
RELICT CHANNEL PROFILES

ACRES

WATANA RELICT CHANNEL - LOOKING SOUTH



CONCEPT : TOP OF ROCK IF ALLUVIUM WAS REMOVED



ACRES

8) WATANA DAMSITE
FOUNDATIONS, EXCAVATION & ROCK MECHANICS
GENERAL FEASIBILITY LEVEL DESIGN CRITERIA

SUSITNA HYDROELECTRIC PROJECT
WATANA DAM

MAIN DAM

EXCAVATION

OVERBURDEN - AVERAGE 20 FEET DEPTH OVER ALL FOUNDATION AREA.

WEATHERED ROCK UNDER CORE AND FILTERS - 40' DEPTH.

WEATHERED ROCK UNDER SHELLS - 10' DEPTH.

MAXIMUM SLOPES - 1H:2V BELOW 1800' ELEVATION
1H:1V ABOVE 1800' ELEVATION

CONSOLIDATION GROUTING

10' X 10' GRID OF HOLES 30' DEEP OVER AREA OF CORE AND FILTERS.

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SUSITNA HYDROELECTRIC PROJECT
WATANA DAM

CURTAIN GROUTING

DOUBLE ROW CURTAIN - VERTICAL.

350' MAXIMUM DEPTH (AT MAXIMUM HEAD).

50' MINIMUM DEPTH IN ABUTMENTS.

HOLE SPACING PRIMARY 40'.

SECONDARY

TERTIARY

QUATERNARY

SPLIT SPACING TO GIVE
FINAL SPACING 5'.

GALLERIES FULL LENGTH OF DAM, APPROXIMATE SIZE 10' X 10'.

DRAINAGE

50' DEEPER THAN GROUT CURTAIN.

HOLE SPACING 10'.

DRILLED FROM GROUT GALLERIES.

FULL LENGTH OF DAM, EXTENDING 600' INTO LEFT ABUTMENT.

CONNECTING TO INTAKE AND SPILLWAY STRUCTURES RIGHT ABUTMENT.

HOLES INCLINED DOWNSTREAM 15° FROM VERTICAL.

9/16/81

SUSITNA HYDROELECTRIC PROJECT
WATANA DAM

SURFACE STRUCTURES

INTAKE AND SPILLWAY

EXCAVATION - OVERBURDEN - AVE 10'

WEATHERED ROCK - AVE 40' (MINIMUM).

CONSOLIDATION GROUTING - 10' X 10" GRID OF HOLES 30' DEEP.

CURTAIN GROUTING - CONTINUATION OF DAM CURTAIN. 50' MINIMUM
DEPTH. SPLIT SPACING AS MAIN DAM.

DRAINAGE - FROM GALLERIES IN BASE OF STRUCTURES.

TUNNEL PORTALS (TUNNEL SPAN = D)

COVER TO TOP SOUND ROCK 1.5D.

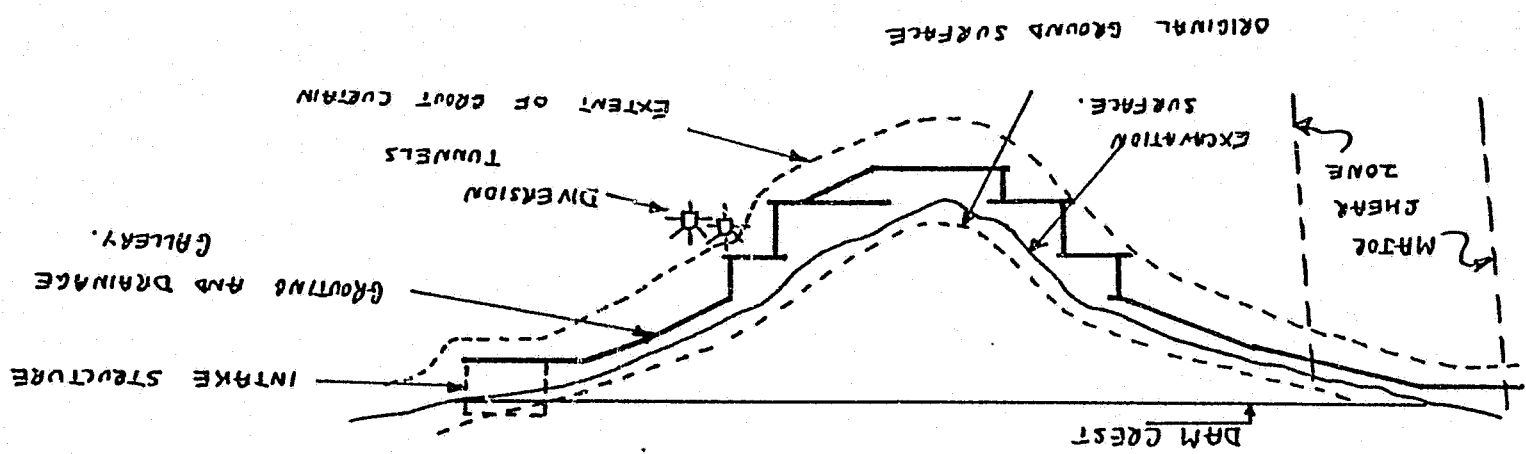
SPACING CENTER TO CENTER 2.5D. WHERE TUNNEL IS STRUCTURALLY
REINFORCED, SPACING MAY BE REDUCED TO 2.0D.

ALL TUNNEL LININGS CONTACT GROUTED.

DRAINAGE PROVIDED IN LININGS OF DIVERSION TUNNELS.

9/16/81

SUSITNA HYDRO ELECTRIC PROJECT
WATANA DAM
GROUTING ARRANGEMENT



(LOOKING DOWN STREAM)

SECTION 1981

WATANA RQD SUMMARY

Borehole Ground Surface El. Top Of Rock El. Borehole Dip		DH-1 1459 1415	DH-4 1462 1384	DH-5 1462 1402	DH-6 1716 1713	DH-7 1716 1708 59%	DH-8 1910 1894	DH-9 1913 1909 45%	DH-10 2033 2020	DH-11 2034 2018 45%	DH-12 1951 1942	DH-21 1480 1407 60%	DH-23 1952 1947 45%	DH-24 2061 2054	DH-28 1971 1958	BH-2 1835 1826	BH-6 1605 1598	BH-8 1976 1964	Depth Average
Vertical Depth	RQD																		
0'-50'	Drilled (ft) RQD length (ft) RQD%	51.4 14.25 28%	45.2 29.95 66%	47.9 22.15 46%	48.9 24.35 50%	49.1 23.6 48%	54.5 32.3 59%	67.0 39.6 59%	48.6 10.2 21%	70.3 49.6 71%	51.6 30.8 60%	59.7 50.95 85%	70.5 31.8 45%	50.9 35.5 70%	29.5 10.6 36%	61.2 33.55 55%	58.5 29.35 50%	59.2 25.75 43%	924.0 494.3 52%
50'-150'	"	27.6 12.55 45%		69.2 57.3 83%	97.2 49.25 51%	64.6 38.95 60%	79.3 52.8 67%	139.5 86.4 62%	100.5 68.5 68%	144.1 111.65 77%	99.9 53.35 53%	110.0 93.55 85%	41.7 20.5 49%	82.1 57.4 70%	62.4 6.3 10%	119.8 28.1 23%	118.4 75.5 64%	116.2 86.85 75%	1472.5 898.95 61%
150'-250'	"							72.1 31.8 44%	34.8 31.1 89%	65.0 55.3 85%	105.1 80.5 77%	121.7 101.4 83%				124.4 67.2 54%	115.8 99.15 86%	114.5 94.45 82%	753.4 560.9 74%
250'-350'	"										33.9 28.8 85%	118.3 96.8 82%				85.6 64.55 75%	115.3 93.65 81%	114.6 91.1 79%	467.7 374.9 80%
350'-450'	"											109.5 91.0 83%					114.4 93.65 82%	114.3 95.85 84%	338.2 280.5 83%
450'-550'	"																115.2 81.35 71%	117.8 95.3 81%	233.0 176.65 76%
550'-650'	"																94.8 74.7 79%	102.0 89.3 88%	196.8 164.0 83%
Hole Average	"	79.0 26.8 34%	45.2 29.95 66%	117.1 79.45 68%	146.1 73.6 50%	113.7 62.55 55%	133.8 85.1 64%	278.6 157.8 57%	183.9 109.8 60%	279.4 216.55 78%	290.5 193.45 67%	519.2 433.7 84%	112.2 52.3 47%	133.0 92.9 70%	91.9 16.9 18%	391.0 193.4 49%	732.4 547.35 75%	738.6 578.6 78%	4485.6 2950.2 67%

9) DEVIL CANYON DAMSITE
FOUNDATIONS, EXCAVATION & ROCK MECHANICS
GENERAL FEASIBILITY LEVEL DESIGN CRITERIA

SUSITNA HYDROELECTRIC PROJECT
DEVIL CANYON DAM

MAIN DAM

EXCAVATION

- OVERBURDEN - RIVER BED 80' MAX, 20' MIN.
- CANYON WALLS 15-20' MAX, 1-2' MIN.
- ABOVE EL 1300' 5'-35' AVERAGE 20'

WEATHERED ROCK - VARIES FROM 10' DEPTH TO 80' OR MORE WHERE
JOINTING IS OPEN

ADDITIONAL EXCAVATION WILL BE REQUIRED TO SHAPE CANYON WALLS

CONSOLIDATION GROUTING

10' X 10' GRID OF HOLES 30' TO 70' DEEP OVER FOUNDATION AREA
(100' UPSTREAM AND DOWNSTREAM) AND UNDER THRUST BLOCKS.

9/16/81

SUSITNA HYDROELECTRIC PROJECT
DEVIL CANYON DAM

MAIN DAM
(CONT'D)

CURTAIN GROUTING

DOUBLE ROW CURTAIN VERTICAL
300' MAXIMUM DEPTH (AT MAXIMUM HEAD)
50' MINIMUM DEPTH IN ABUTMENTS
HOLE SPACING PRIMARY 40'

SECONDARY
TERTIARY
QUATERNARY

SPLIT SPACING TO GIVE
FINAL SPACING 5'

GALLERIES FULL LENGTH OF DAM, APPROXIMATE SIZE 10' x 10' EXTENDING
UNDER THRUST BLOCKS.

DRAINAGE

50' DEEPER THAN GROUT CURTAIN
HOLE SPACING 10'
DRILLED FROM GROUT GALLERIES

HOLES INCLINED DOWNSTREAM 15°

ADDITIONAL DRAINAGE HOLES DRILLED FROM SURFACE MAY BE REQUIRED
FOR SLOPE STABILITY OF CANYON WALLS BOTH UPSTREAM AND DOWNSTREAM
OF DAM

9/16/81

SUSITNA HYDROELECTRIC PROJECT
DEVIL CANYON DAM

SURFACE STRUCTURES

INTAKE AND SPILLWAY

EXCAVATION - OVERBURDEN - 20' APPROXIMATELY
- WEATHERED ROCK - 20' APPROXIMATELY

CONSOLIDATED GROUTING - 10' x 10' GRID OF HOLES 30' DEEP

CURTAIN GROUTING (SPILLWAY ONLY) - CONTINUATION OF DAM CURTAIN FROM GALLERY
UNDER STRUCTURE
- 50' MINIMUM DEPTH
- SPLIT SPACING AS MAIN DAM

DRAINAGE (SPILLWAY ONLY) - FROM GALLERIES IN BASE OF SPILLWAY
STRUCTURE

TUNNEL PORTALS (TUNNEL SPAN = D)

COVER TO TOP SOUND ROCK 1.5D
SPACING CENTER TO CENTER 2.5 D. WHERE TUNNEL IS STRUCTURALLY
REINFORCED SPACING MAY BE REDUCED TO 2.0 D

ALL TUNNEL LININGS CONTACT GROUTED

DRAINAGE PROVIDED IN LININGS OF DIVERSION TUNNELS

9/16/81

SUSITNA HYDROELECTRIC PROJECT
DEVIL CANYON DAM

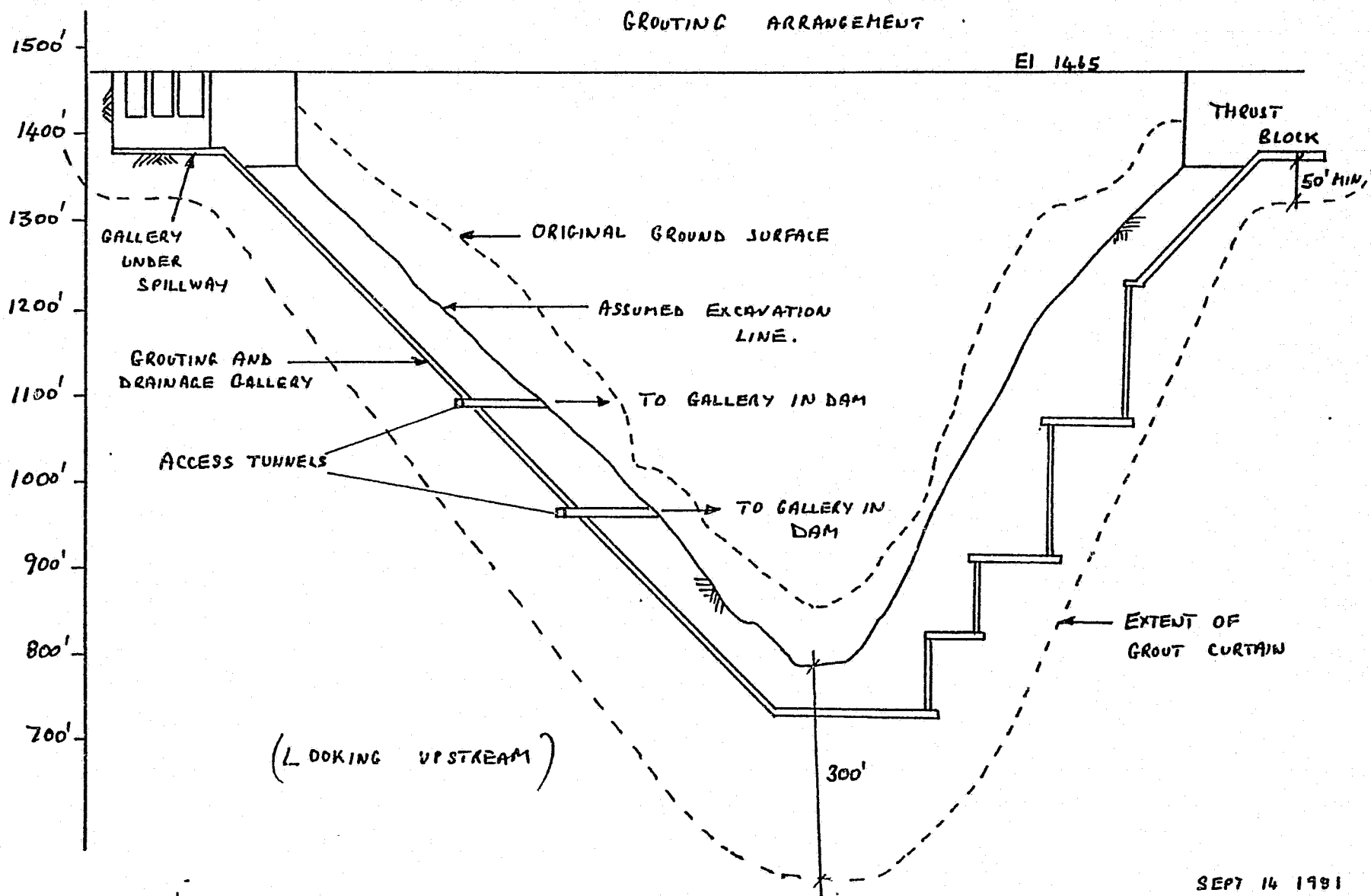
UNDERGROUND STRUCTURES - GROUTING

PENSTOCKS - RADIAL CURTAIN GROUTING MAY BE REQUIRED IN PLANE OF
DAM GROUT CURTAIN

DRAINAGE - GALLERY IN AREA BETWEEN TRANSFORMER GALLERY, PENSTOCKS
AND POWERHOUSE

9/16/81

SUSITNA HYDROELECTRIC PROJECT
DEVIL CANYON DAM
GROUTING ARRANGEMENT



DEVIL CANYON RQD SUMMARY

Borehole		BH-1	BH-2	BH-4	BH-5a	BH-5b	BH-7	BH-3	Depth Average
Ground Surface El.		1415	1214	1353	980	980	1335	1405	
Top of Rock El.		1404	1212	1346	--	--	--	--	
Borehole Dip		67°	60°	60°	45°	45°	50°	32°	
Vertical Depth	RQD								
0'-50'	Drilled (ft)	52.7	60.2	58.2	72.5	71.8	67.9	94.3	477.6
	RQD Length- RQD% (ft)	34.7 66%	47.6 79%	50.6 87%	46.35 64%	45.55 63%	39.6 58%	77.2 82%	341.6 72%
50'-150'	"	107.0 69.8 65%	113.4 82.15 72%	113.8 111.0 98%	138.7 110.2 79%	128.5 105.5 82%	134.8 85.25 63%	188.70 139.0 74%	924.9 702.9 76%
150'-250'	"	108.8 94.2 87%	116.4 104.4 90%	117.9 105.4 89%	143.2 117.9 82%		141.2 121.9 86%	100.2 45.4 45%	727.7 589.2 81%
250'-350'	"	111.1 95.6 86%	115.1 100.4 87%	114.3 103.2 90%	142.2 125.55 88%		143.4 117.95 82%		626.1 542.7 87%
350'-450'	"	109.8 98.7 90%	117.2 109.6 94%	84.5 80.65 95%	101.3 84.8 84%				412.8 373.75 91%
450'-550'	"	107.7 103.1 96%	116.2 98.25 85%						223.9 201.35 90%
550'-650'	"	113.8 95.1 84%	15.0 14.0 96%						128.8 109.5 85%
650'-750'	"	34.5 29.3 85%							34.5 29.3 85%
Hole Average	"	745.4 620.5 83%	653.5 556.8 85%	488.7 450.85 92%	597.9 484.8 81%	200.3 151.05 75%	487.3 364.7 75%	383.2 261.6 68%	3556.3 2890.3 81%

SUPPORT MATERIAL TO TASK 6
DESIGN DEVELOPMENT

WATANA LAYOUT STUDIES

1 - INTRODUCTION

As discussed in the DSR, the recommended Watana development (Plates 12 and 13) consisted of the following:

- Dam: Earth and rockfill
 - . Crest: 2,225 feet
 - . Height: 880 feet
 - . Volume: 63 million cu. yds.
 - . Slopes: 2.75:1V upstream
2H:1V downstream
 - . Cofferdams: Integral
- Spillway: Three-gated ogee section
 - . Capacity: 235,000 cfs (PMF)
 - . Type: Chute and flip bucket
 - . Location: Right bank
- Power Plant: 800 MW (ultimate), left bank

Studies have continued to develop an optimized general arrangement for the Watana site.

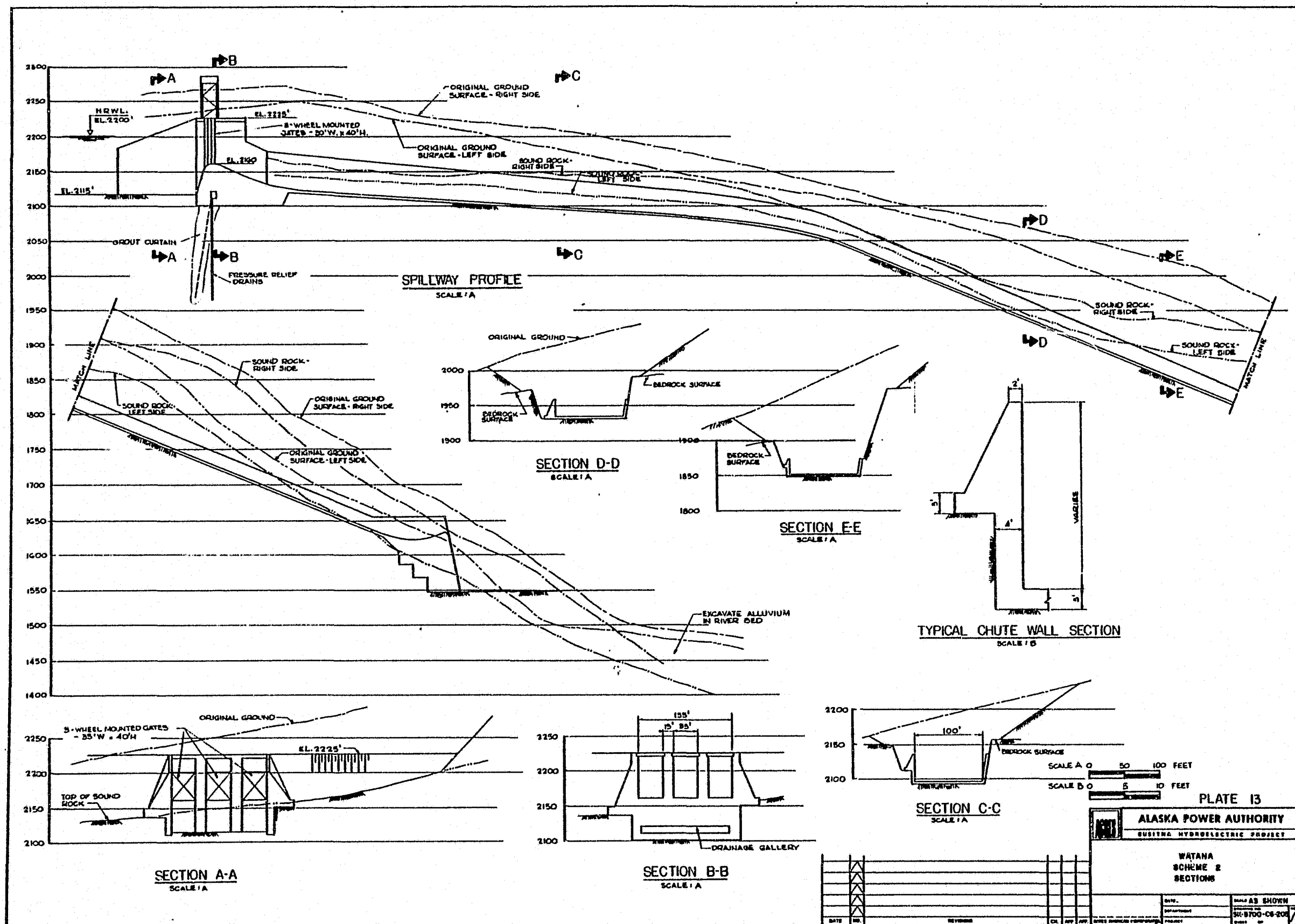


Table 6.1. Description of Alternative Layouts

Layout Alter.	Main Dam		Service Spillway		Auxiliary Spillway		Power Facilities	
	Slopes	Configuration	Type	Location	Type	Location	Powerhouse Type	Location
1	u/s 2.5:1 d/s 2:1	right side curved upstream	flip bucket	right bank	--	--	underground	left bank
2	u/s 2.25:1 d/s 2:1	right and left sides curved upstream	cascade, single control structure	left bank	--	--	surface	right bank
2A	u/s 2.25:1 d/s 2:1	right and left sides curved upstream	cascade, dual control structures	left bank	--	--	surface	right bank
2B	u/s 2.25:1 d/s 2:1	right and left sides	double stilling basin	left bank	--	--	surface	right bank
2C	u/s 2.25:1 d/s 2:1	right and left sides curved upstream	double stilling basin	left bank	inclined unlined channel	left bank	surface	right bank
2D	u/s 2.25:1 d/s 2:1	right and left sides curved upstream	flip bucket	left bank	--	--	underground	right bank
3	u/s 2.5:1 d/s 2:1	right and left sides curved upstream	cascade	left bank	flip bucket	right bank	underground	left bank
4	u/s 2.5:1 d/s 2:1	left bank skewed upstream	flip bucket	left bank	--	--	underground	left bank

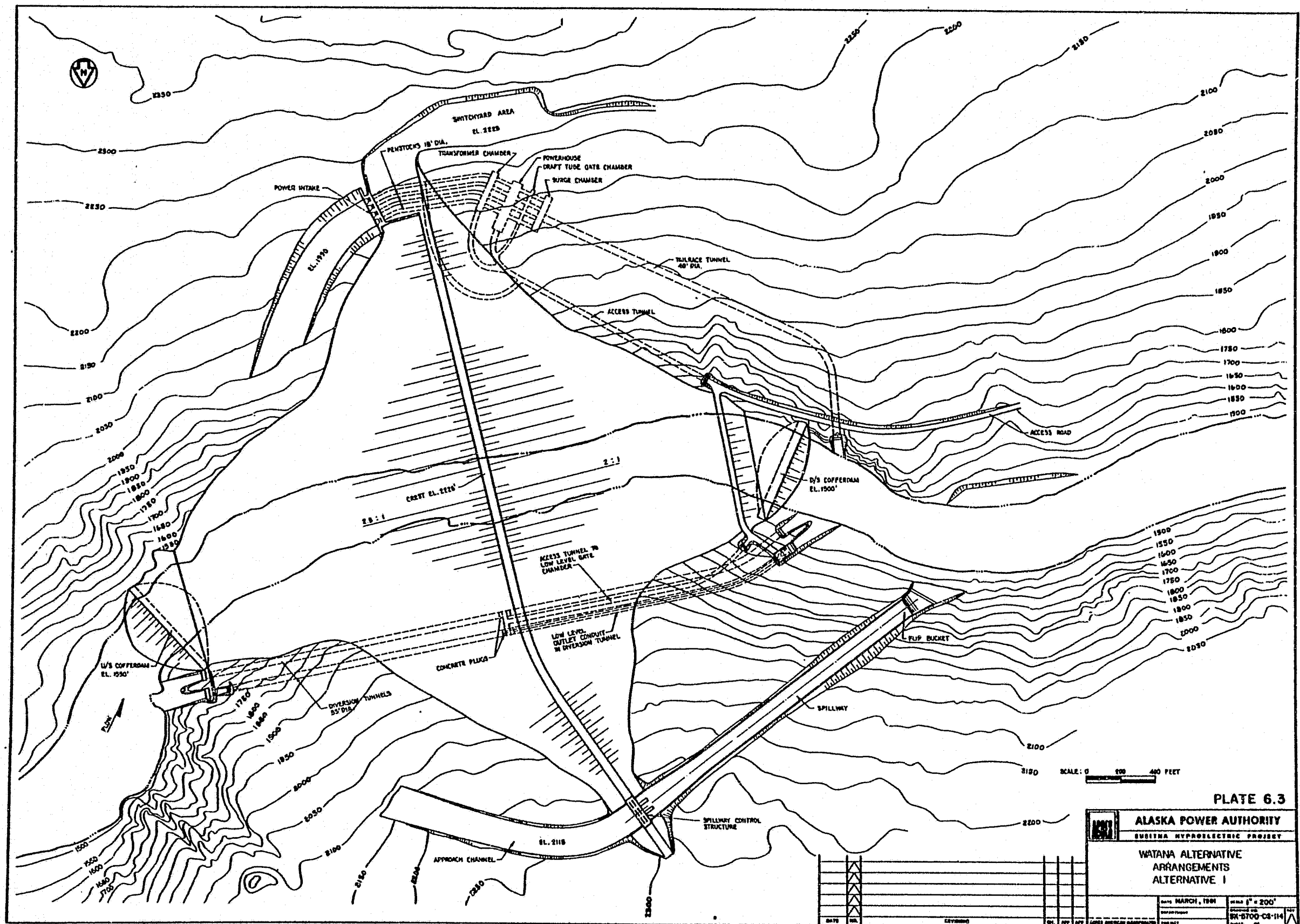


PLATE 6.3

ALASKA POWER AUTHORITY EUSITNA HYDROELECTRIC PROJECT	
WATANA ALTERNATIVE ARRANGEMENTS ALTERNATIVE 1	
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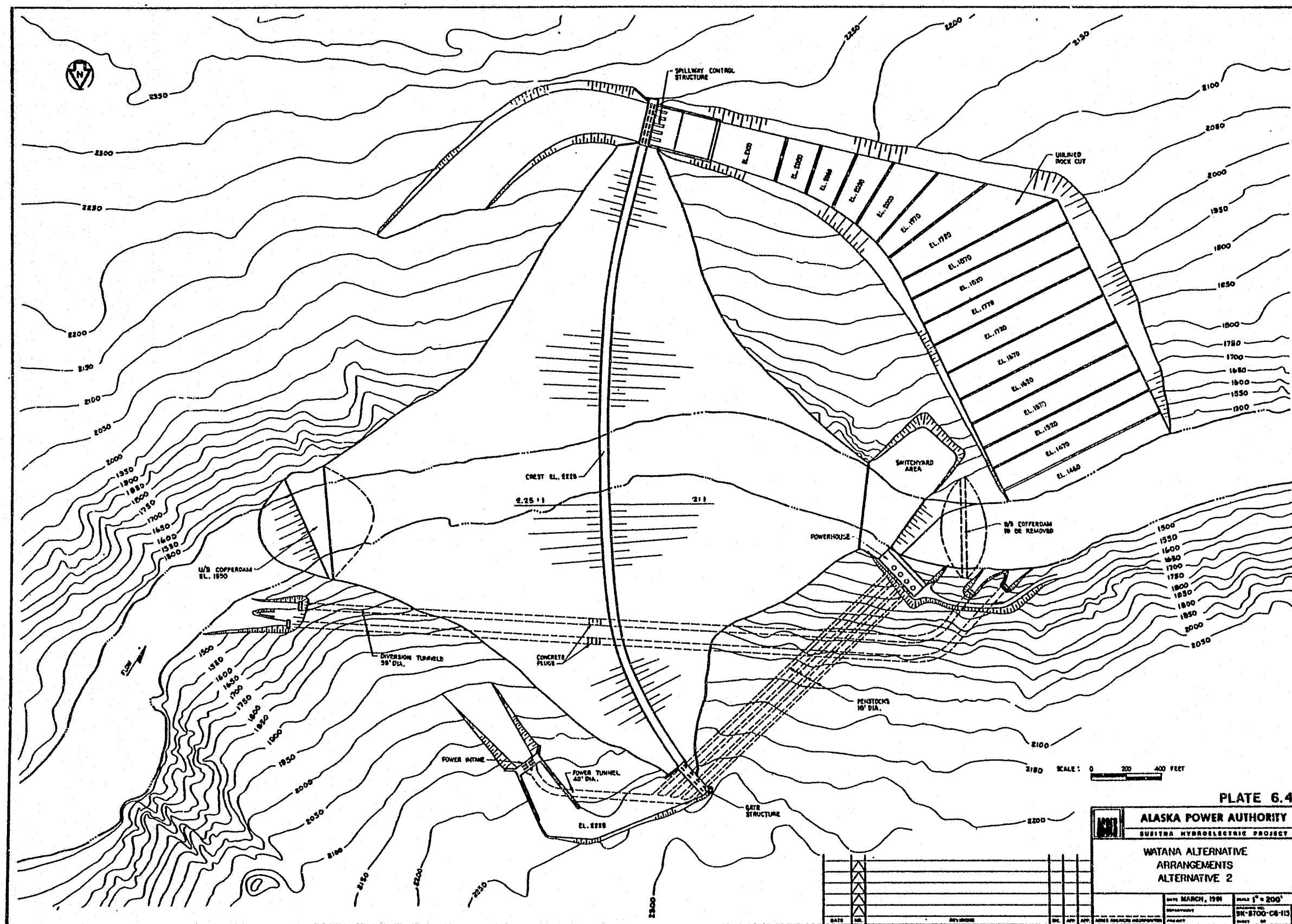
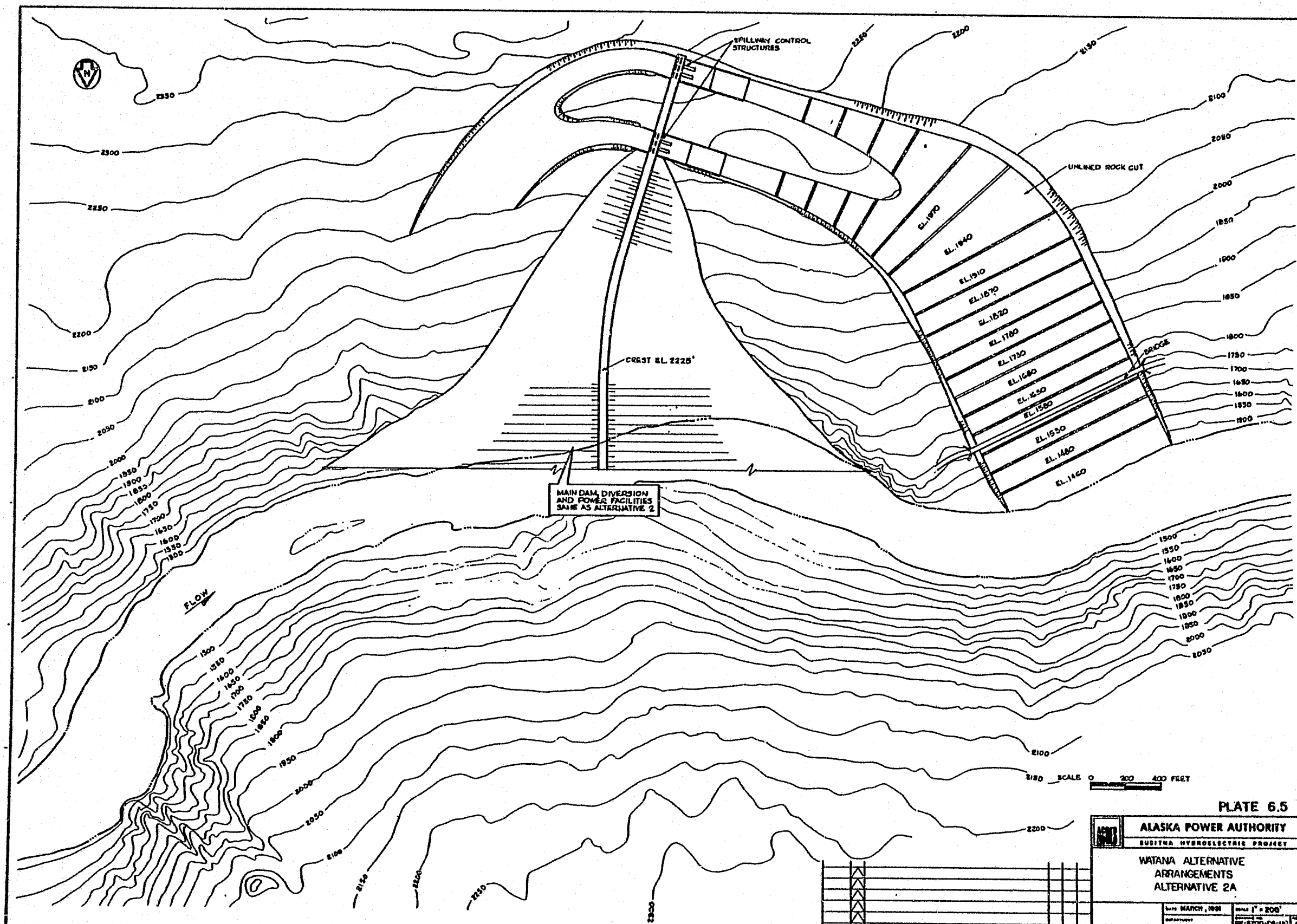


PLATE 6.4

ALASKA POWER AUTHORITY	
SUSTINA HYDROELECTRIC PROJECT	
WATANA ALTERNATIVE ARRANGEMENTS ALTERNATIVE 2	
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DESIGNED BY: [blank]	CHECKED BY: [blank]
APPROVED BY: [blank]	DATE: [blank]
PROJECT NO. SK-8700-C6-113	





ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

WATANA ALTERNATIVE
ARRANGEMENTS
ALTERNATIVE 2C

DATE MARCH, 1901

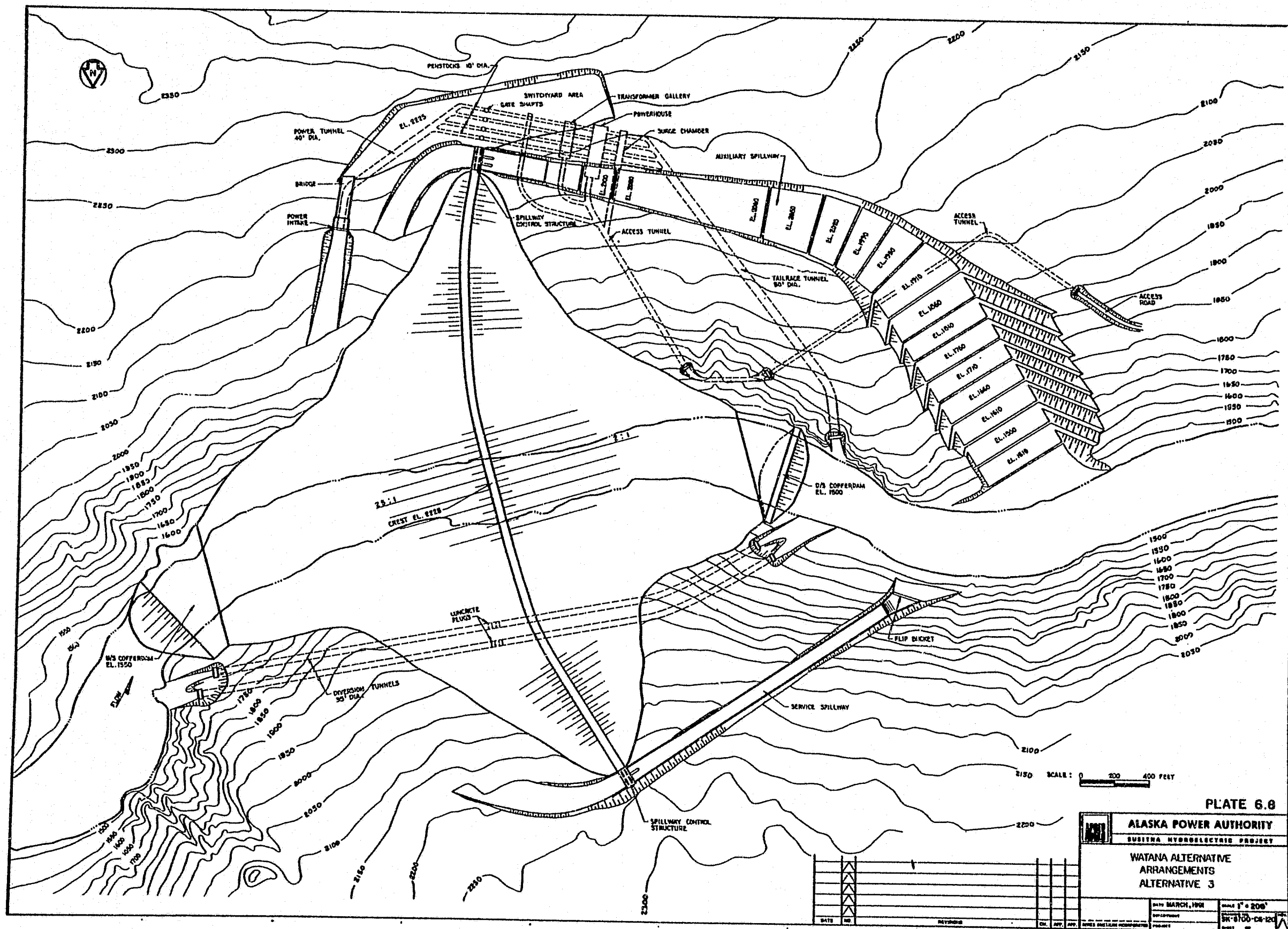
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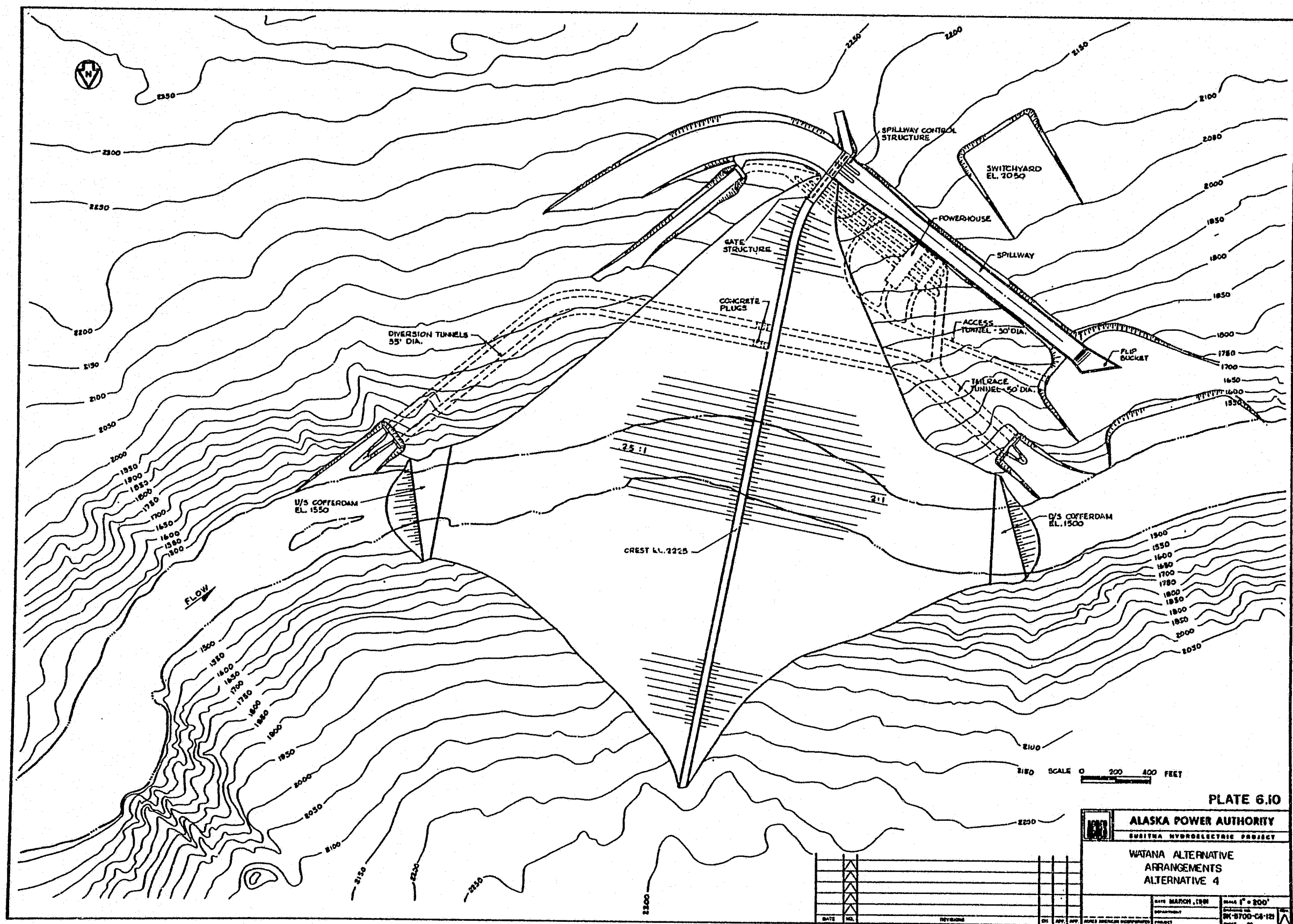
Page 2 of 2



SCALE: 0 200 400 FEET

PLATE 6.8

ALASKA POWER AUTHORITY	
SUSITNA HYDROELECTRIC PROJECT	
WATANAH ALTERNATIVE ARRANGEMENTS	
ALTERNATIVE 3	
DATE: MARCH, 1961	SCALE: 1" = 200'
BY: [Signature]	SH: 6100-CH-120
CH. []	APP. []
REVISIONS	NO. []



3 - COMPARISONS OF FOUR PREFERRED LAYOUTS

The four layouts described in Section 2 were developed in greater detail (see Plates 7.1 and 7.3 to 7.7), taking into account any new data that had become available and based on expanded and updated design criteria. Capital cost estimates of the layouts were prepared and the schemes were evaluated to determine the two arrangements that were the most favorable.

- Design Criteria

The principal project parameters and design criteria on which the layouts were based are shown in Table 3.1. Parts of this criteria will be superseded as more information becomes available. Where assumptions were made, they were based on the best information available at that time.

- Evaluation Criteria

The review of layouts was carried out and assessments of the different schemes made on the basis of the following evaluation criteria:

- (a) Technical feasibility of the scheme;
- (b) Overall cost of the scheme;
- (c) Ease of construction of the project. This will partly be reflected in the cost of the scheme and be evaluated under (b);
- (d) Impact on construction schedule;
- (e) Environmental considerations; and
- (f) Operating characteristics.

- Selected Schemes

The selected schemes for further development are:

- (a) Right bank diversion, mid-level release acting as a first-stage service spillway, right bank chute and flip bucket as a backup spillway, rock channel as an emergency spillway, and a right bank underground powerhouse;
- (b) Right bank diversion, left bank rock cascade service spillway, right bank rock channel emergency spillway, and a right bank underground powerhouse.

These schemes are shown on Plates 8.1 through 8.4.

- Cost Comparisons

Table 1 summarizes cost comparisons for the four selected developments.

TABLE 3.1 - DESIGN CRITERIA

River Flows

Average flow (over 30 years of record):	7,860 cfs
Probable maximum flood (routed):	235,000 cfs
Maximum inflow with return period of 1:10,000 years:	155,000 cfs
Maximum flood with return period of 1:500 years:	116,000 cfs
Maximum flood with return period of 1:50 years:	87,000 cfs
Reservoir normal maximum operating level:	2,200 ft MSL
Reservoir minimum operating level:	2,050 ft MSL
Area of reservoir at maximum operating level:	40,000 acres
Reservoir live storage:	4.6×10^6 acre ft
Reservoir full storage:	10.0×10^6 acre ft

Dam

Type:	Rockfill
Crest elevation at center:	2,225 ft MSL
Height:	890 ft above foundation
Cutoff and foundation treatment:	Core founded on rock grout curtain and downstream drains
Upstream slope:	1V:2.75H
Downstream slope:	1V:2.0H
Crest width:	80 ft

Diversion

Cofferdam types:	Rockfill
Discharge capacity:	1:50 yrs. routed flow
Cutoff and foundation:	Slurry trench to bedrock
Upstream cofferdam crest elevation:	1,560 ft MSL
Downstream cofferdam crest elevation:	1,500 ft MSL
Maximum pool level during construction:	1,555 ft MSL
Water passages	Concrete lined
Outlet structures:	Low level structure with high head slide gates to operate under low heads
Final closure of diversion tunnels:	Mass concrete plugs in line with dam grout curtain
Releases during impounding:	2,000 cfs min. via bypass to outlet structure

Spillway

Design floods:	Passes pmf, preserving integrity of dam Passes routed 1:10,000- year flood with no damage to structures
----------------	---

TABLE 3.1 (Continued)

Spillway (continued)

Main spillway - Capacity:

Routed 1:10,000-year
flood with 5 ft
surcharge

- Control structure:
Emergency spillway - Capacity:

Gated ogee crests
Pmf minus 1:10,000-yr
flood

- Type:

Fuse plug

Power Intake

Type:

Massive concrete
structure embedded in
rock

Number of intakes:
Draw-off requirements:

4
Multi-level corresponding
to temperature strata
150 feet

Drawdown:

Penstocks

Type:

Concrete-lined tunnels
with downstream steel
liners

Number of penstocks:

4

Powerhouse

Type:

Underground
Separate gallery
Surface
Rock tunnel
Elevator from surface

Transformer area:
Control room and administration:
Access - Vehicle:
- Personnel:

Power Plant

Type of turbines:
Number and rating:
Rated net head:
Design flow:
Normal maximum gross head:
Type of generator:
Rated output:
Power factor:
Frequency:
Transformers:

Francis
4 x 200 MW
690 ft
5,300 cfs per unit
745 ft
Vertical synchronous
222 MVA
0.9
60 HZ
222 MVA - 13.8-345 kV,
3-phase

TABLE 3.1 (Continued)

Tailrace

Water passages:
Elevation of water passages:
Surge:
Average tailwater elevation:

2 concrete-lined tunnels
Below minimum tailwater
Separate surge chambers
1,475 ft MSL

Main Access:

Assumed from the north
side

Transmission:

Assumed from the north
side

TABLE 1 - WATANA
COMPARATIVE COST ESTIMATE - NON-COMMON ITEMS
JULY 1981 (JANUARY 1982 DOLLARS)

Page 1 of 5

Description	Unit	Unit Price	Scheme 2A		Scheme 3A1		Scheme 3A2		Scheme 4A	
			Quantity	\$000	Quantity	\$000	Quantity	\$000	Quantity	\$000
<u>Diversion/Cofferdams</u>										
Excavate Rock Portal	cy	15	290,000	\$ 4,350	290,000	\$ 4,350	290,000	\$ 4,350	290,000	\$ 4,350
Excavate Tunnels	cy	55	435,000	23,925	536,000	29,480	435,000	23,925	450,000	24,750
Concrete Liner at Portals	cy	260	4,200	1,092	4,200	1,092	4,200	1,092	4,200	1,092
Concrete Liner Tunnel	cy	250	67,600	16,900	83,600	20,900	67,600	16,900	69,900	17,475
Concrete Inlet Portal Headwall	cy	260	17,200	4,472	17,200	4,472	17,200	4,472	17,200	4,472
Concrete Inlet Portal Pier	cy	295	600	177	600	177	600	177	600	177
Concrete Outlet Headwall	cy	260	7,500	1,950	7,500	1,950	7,500	1,950	7,500	1,950
Concrete Plugs	cy	500	20,000	10,000	20,000	10,000	20,000	10,000	20,000	10,000
Upstream Cofferdam	cy	4.20	3,140,000	13,188	3,140,000	13,188	3,140,000	13,188	3,140,000	13,188
Downstream Cofferdam	cy	7.35	125,000	919	125,000	919	125,000	919	125,000	919
Dewatering	LS	3,675,000		3,675		3,675		3,675		3,675
Cutoff	LS	6,825,000		6,825		6,825		6,825		6,825
Reinforcing Steel	Ton	2,100	1,226	2,575	1,386	2,911	1,226	2,575	1,249	2,623
Rockbolts Tunnels	Ton	3,500	475	1,663	575	2,013	475	1,663	500	1,750
Rockbolts Portals	Ton	3,500	125	438	125	438	125	438	125	438
Rock Surface Treatment	sy	15.80	2,700	43	2,700	43	2,700	43	2,700	43
Support Steel	Ton	3,675	1,250	4,594	1,525	5,604	1,250	4,594	1,300	4,777
Gates, Etc.	LS	4,600,000		4,600		4,600		4,600		4,600
SUBTOTAL DIVERSION/ COFFERDAMS				\$101,386		\$112,637		\$101,386		\$103,104

WATANA
COMPARATIVE COST ESTIMATE - NON-COMMON ITEMS (Cont'd)

Page 2 of 5

Description	Unit	Unit Price	Scheme 2A		Scheme 3A1		Scheme 3A2		Scheme 4A	
			Quantity	\$000	Quantity	\$000	Quantity	\$000	Quantity	\$000
<u>Service Spillway</u>										
Excavate Rock	cy	17	1,567,200	26,642	3,114,000	52,938	1,456,000	24,752	10,074,000	171,250
Excavate Overburden	cy	6.30	1,301,000	8,196	969,000	6,105	1,224,000	7,711	2,150,000	13,545
Concrete Walls Formed One Face	cy	260	32,600	8,476	59,000	15,340	30,700	7,982	950	247
Concrete Walls Formed Both Faces	cy	280	19,100	5,348	75,300	21,084	19,500	5,460	19,700	5,516
Concrete Slab Approach	cy	190	2,900	551	2,600	494	2,600	494	3,900	741
Concrete Slab No Forms	cy	170	79,600	13,532	284,500	48,365	70,400	11,968	13,000	2,210
Concrete Slab Formed One Face	cy	190	2,100	399	8,100	1,539	9,100	1,729	20,000	3,800
Concrete Slab Formed Both Faces	cy	200	15,500	3,100	NA	NA	13,800	2,760	NA	NA
Concrete Gate/Chute Formed One Side	cy	295	2,500	738	2,200	649	2,200	649	3,100	914
Concrete Gate/Chute Formed Both Sides	cy	315	5,100	1,607	4,700	1,481	4,700	1,481	11,200	3,528
Concrete Structure No Forms	cy	210	2,100	441	3,400	714	3,400	714	11,300	2,373
Concrete Structure Formed	cy	250	11,400	2,850	10,100	2,525	10,100	2,525	17,400	4,350
Concrete Bridge	cy	600	2,150	1,290	2,000	1,200	2,000	1,200	3,000	1,800
Concrete Lower Piers Bridge	cy	295	NA	NA	NA	NA	NA	NA	9,700	2,861
Concrete Lower Bridge Deck	cy	500	NA	NA	NA	NA	NA	NA	3,900	1,950
Reinforcing Steel	Ton	2,100	5,896	12,382	15,735	33,043	6,081	12,770	5,577	11,712
Rockbolts	Ton	3,500	1,000	3,500	186	651	920	3,220	8,460	29,610

WATANA
COMPARATIVE COST ESTIMATE - NON-COMMON ITEMS (Cont'd)

Page 3 of 5

Description	Unit	Unit Price	Scheme 2A		Scheme 3A1		Scheme 3A2		Scheme 4A	
			Quantity	\$000	Quantity	\$000	Quantity	\$000	Quantity	\$000
Gates	LS			4,500		2,900		2,900		4,000
Dental Concrete	LS			4,800		4,800		4,800		NA
Grouting	LS			2,100		2,100		2,100		500
Winterization	LS			7,000		11,000		6,500		6,000
Alluvium Removal Above Water Level	cy	7	760,000	5,320		NA	760,000	5,320		NA
Alluvium Removal Below Water Level	cy	17	860,000	14,620		NA	860,000	14,620		NA
Additional Haul Roads	Mile	40,000	2	80		NA	2	80		NA
Surface Treatment Vertical	sy	15.80	17,000	274	50,000	791	19,100	302	9,000	142
Surface Treatment Horizontal	sy	10.50	38,400	403	56,100	589	33,100	348	11,500	121
SUBTOTAL SERVICE SPILLWAY				128,149		208,308		122,385		267,178
<u>Emergency Spillway</u>										
Excavate Rock	cy	17		NA	1,383,000	23,511	1,383,000	23,511		NA
Excavate Overburden	cy	6.50		NA	3,551,000	23,082	3,551,000	23,082		NA
Place Fuse Plug	cy	6		NA	54,200	325	54,200	325		NA
SUBTOTAL EMERGENCY SPILLWAY						46,918		46,918		

WATANA
COMPARATIVE COST ESTIMATE - NON-COMMON ITEMS (Cont'd)

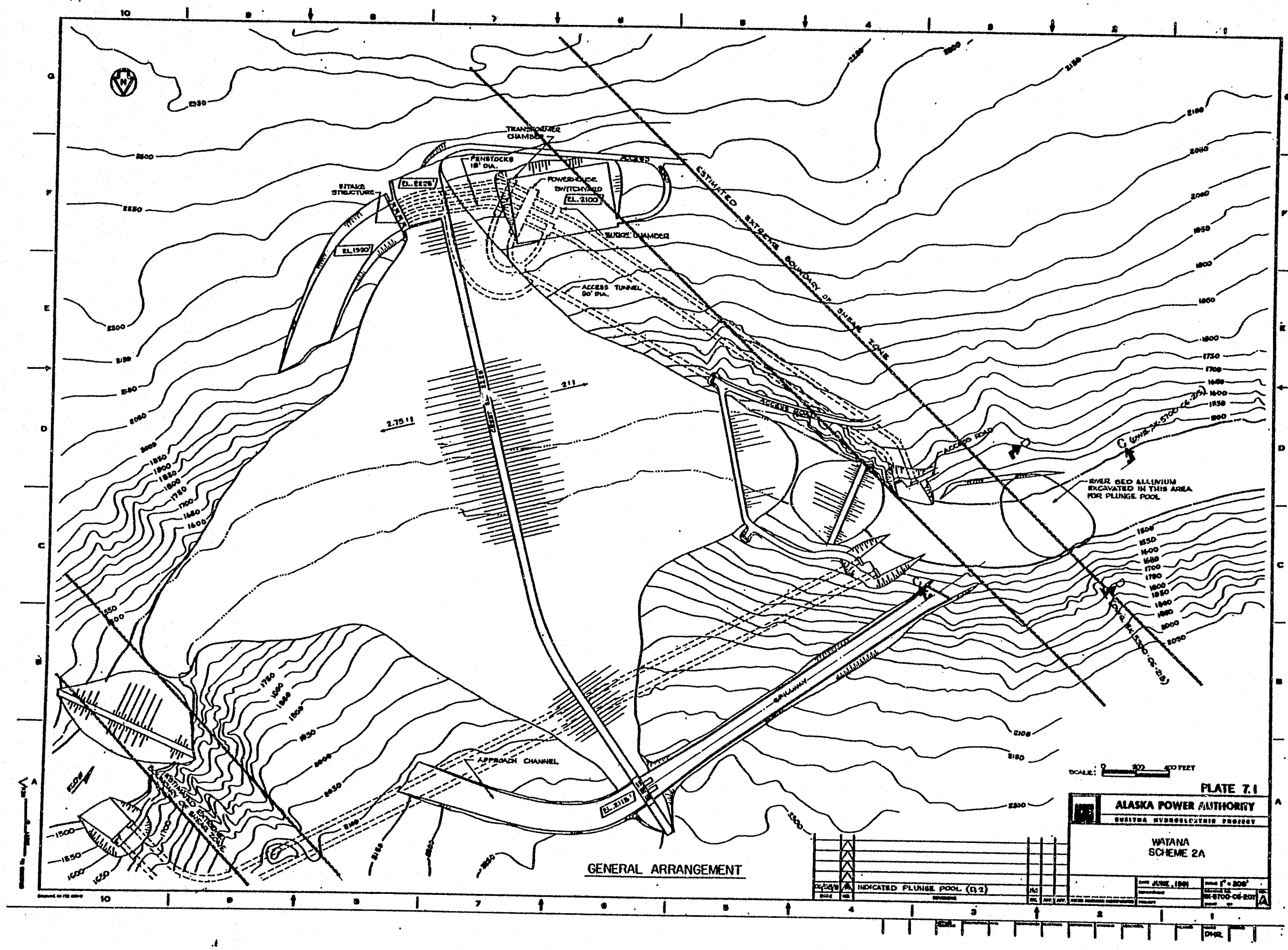
Page 4 of 5

Description	Unit	Unit Price	Scheme 2A		Scheme 3A1		Scheme 3A2		Scheme 4A	
			Quantity	\$000	Quantity	\$000	Quantity	\$000	Quantity	\$000
<u>Tailrace Tunnel</u>										
Excavate Tunnels	cy	50	97,200	4,860	97,200	4,860	97,200	4,860	59,000	2,950
Concrete Line Tunnels	cy	275	14,850	4,084	14,850	4,084	14,850	4,084	9,000	2,475
Reinforcing Steel	Ton	2,100	150	315	150	315	150	315	90	189
Rockbolts	Ton	3,500	480	1,680	480	1,680	480	1,680	315	1,102
Support Steel	Ton	3,675	590	<u>2,168</u>	590	<u>2,168</u>	590	<u>2,168</u>	360	<u>1,323</u>
SUBTOTAL TAILRACE TUNNEL				13,107		13,107		13,107		8,039
Credit for Use of Rock in Dam	cy	8	1,466,000	<11,728>	3,900,000	<31,200>	2,344,000	<18,752>	9,044,000	<72,352>
TOTAL NON-COMMON ITEMS				230,914		349,770		265,044		305,969

ESTIMATE SUMMARY - WATANA
COMPARATIVE COST ESTIMATE - JULY 1981
(\$000,000 JANUARY 1982 DOLLARS)

Page 5 of 5

Description	Scheme 2A	Scheme 3A1	Scheme 3A2	Scheme 4A
NON-COMMON ITEMS	231	350	265	306
COMMON ITEMS	<u>1,643</u>	<u>1,643</u>	<u>1,643</u>	<u>1,643</u>
SUBTOTAL	1,874	1,992	1,908	1,949
Camp & Support Costs (16%)	<u>300</u>	<u>319</u>	<u>305</u>	<u>311</u>
SUBTOTAL	2,174	2,312	2,213	2,260
Contingency 20%	<u>435</u>	<u>462</u>	<u>442</u>	<u>452</u>
SUBTOTAL	2,609	2,774	2,655	2,712
Engr/Admin 12.5%	<u>326</u>	<u>347</u>	<u>332</u>	<u>339</u>
TOTAL	2,935	3,121	2,987	3,051



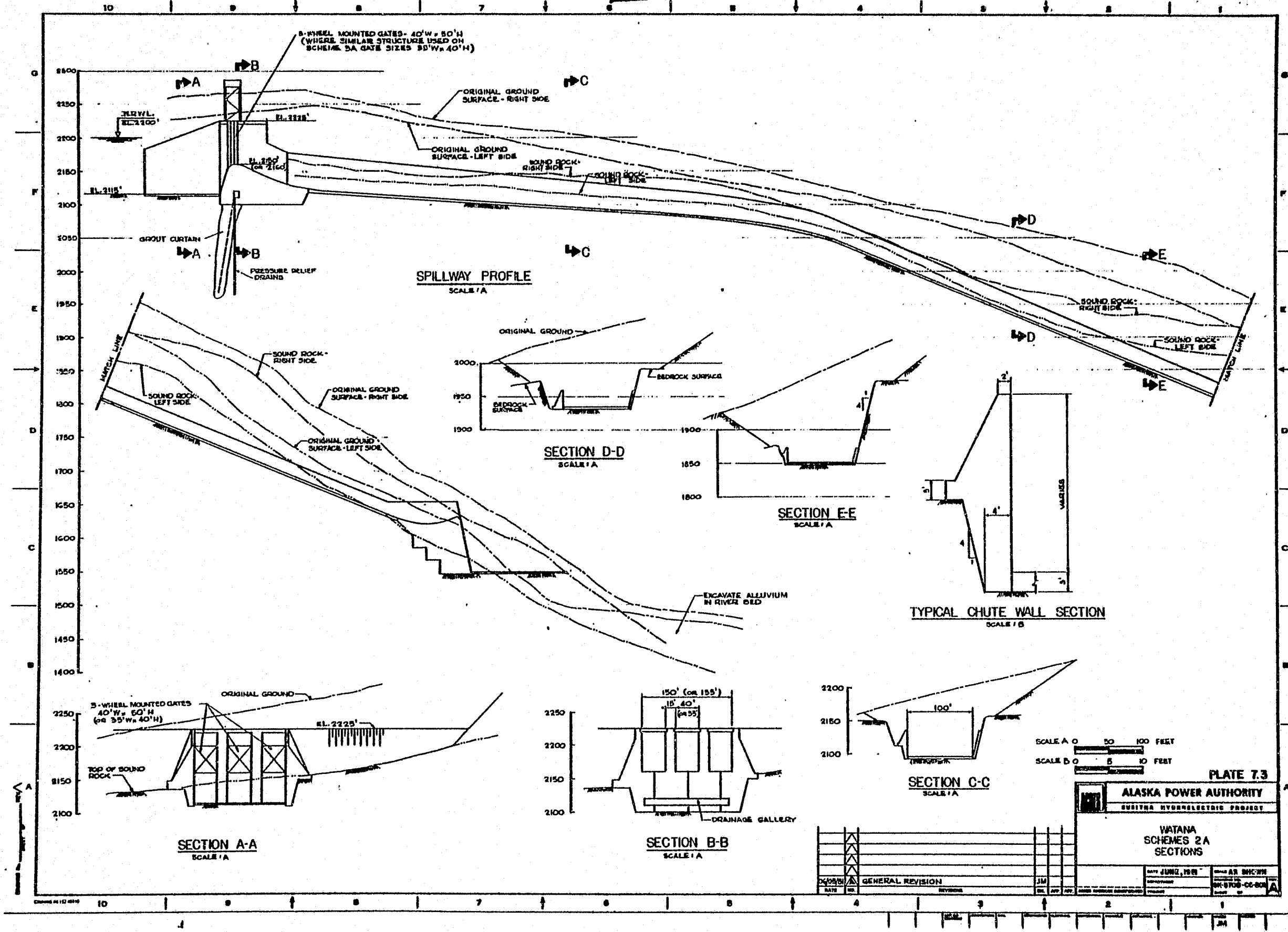


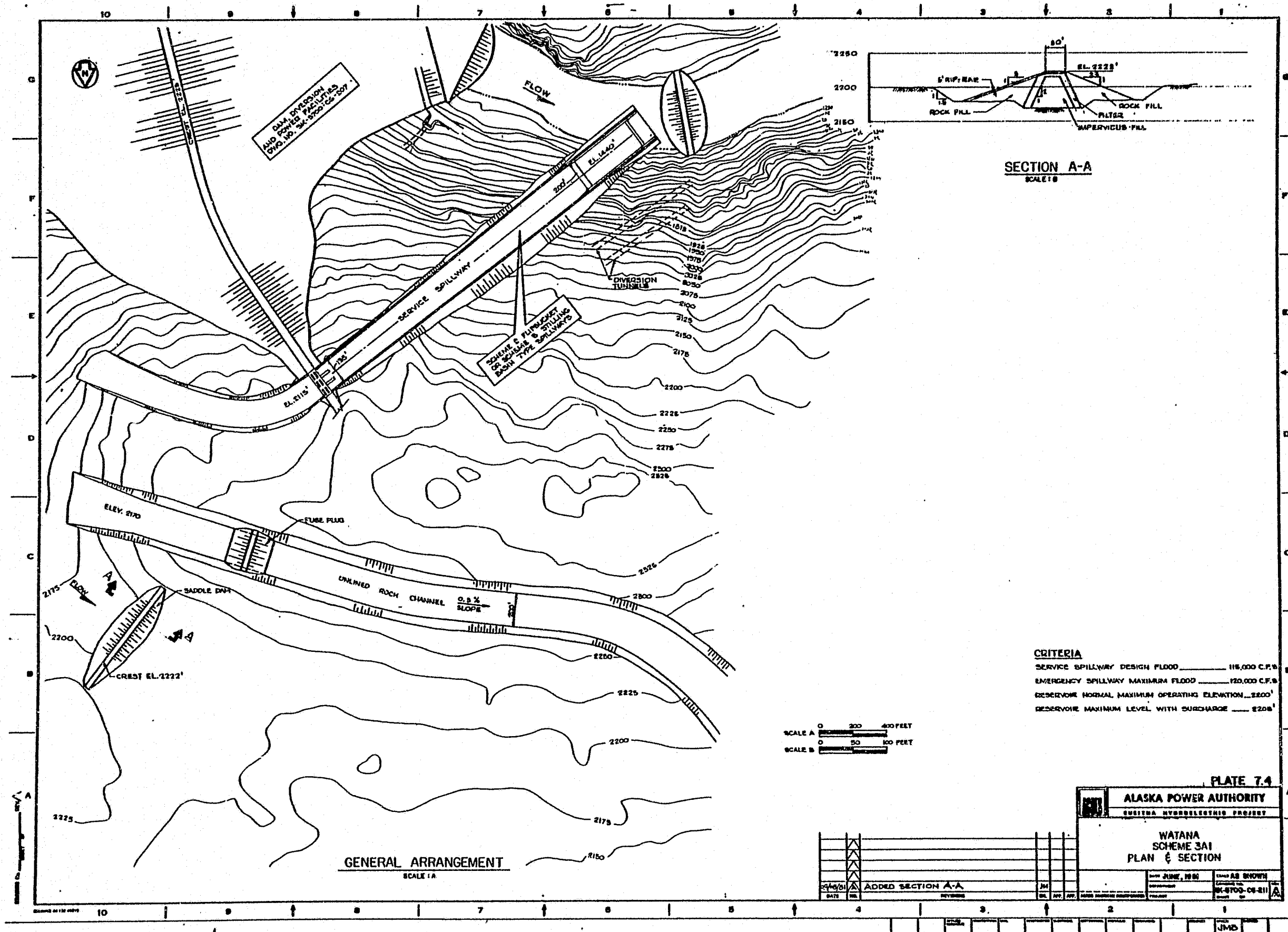
PLATE 7.3

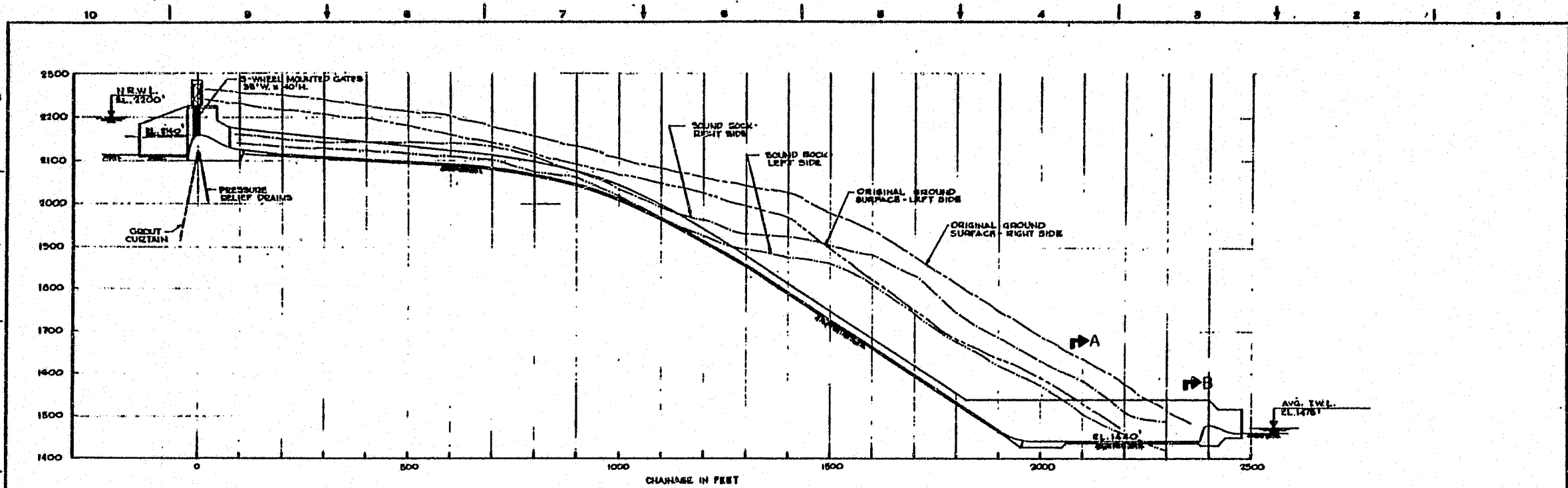
ALASKA POWER AUTHORITY
GUSTINA HYDROELECTRIC PROJECT

WATANA
SCHEMES 2A
SECTIONS

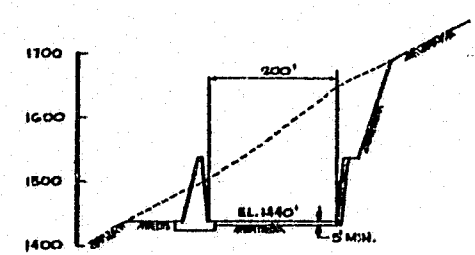
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PROJECT: GUSTINA HYDROELECTRIC PROJECT

GENERAL REVISION

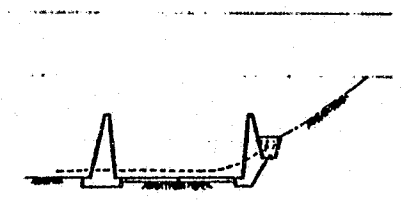




SPILLWAY PROFILE



SECTION A-A

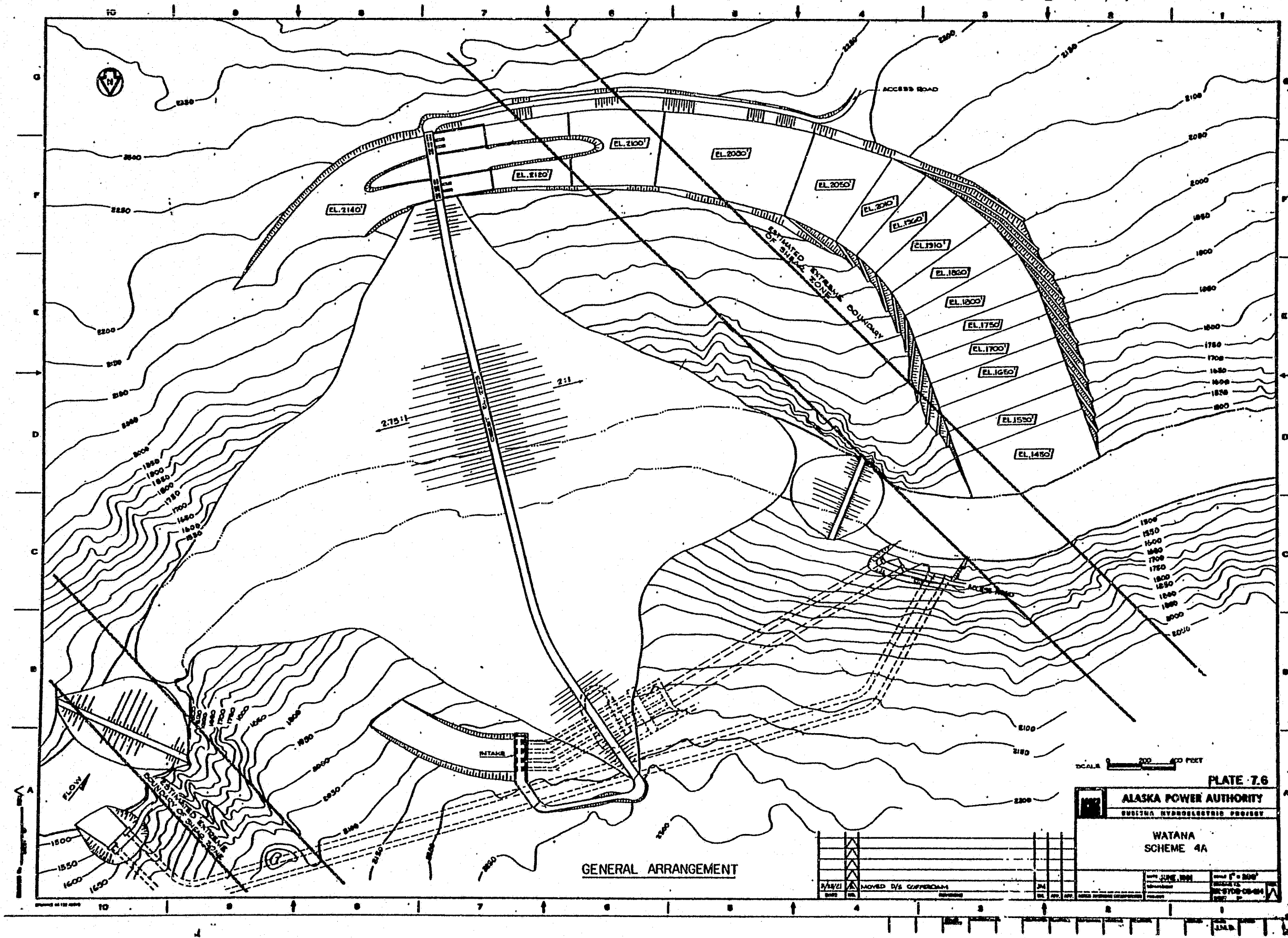


SECTION B-B

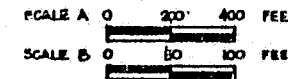
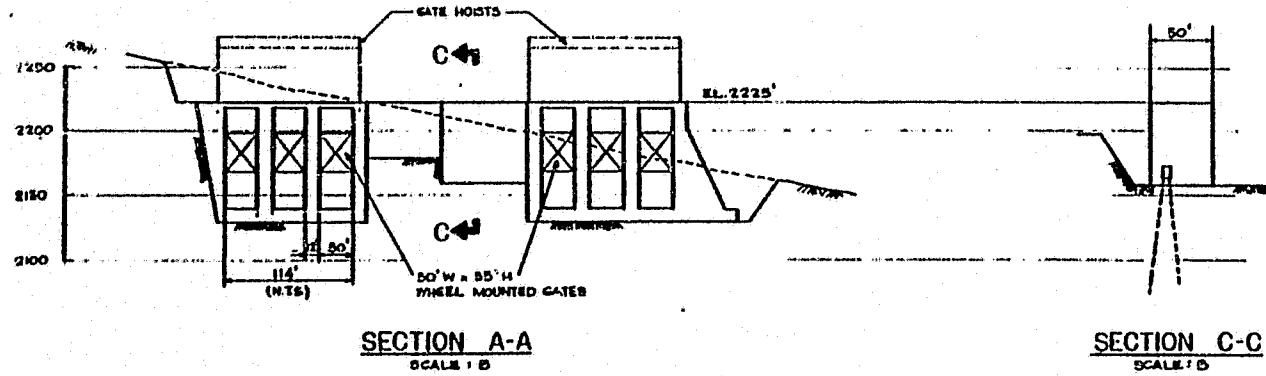
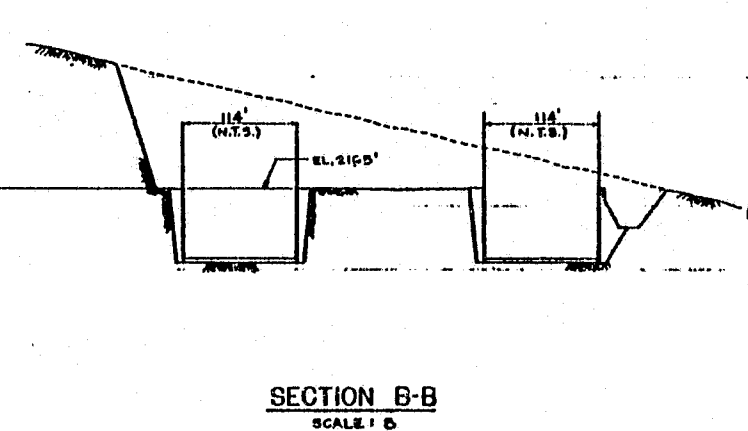
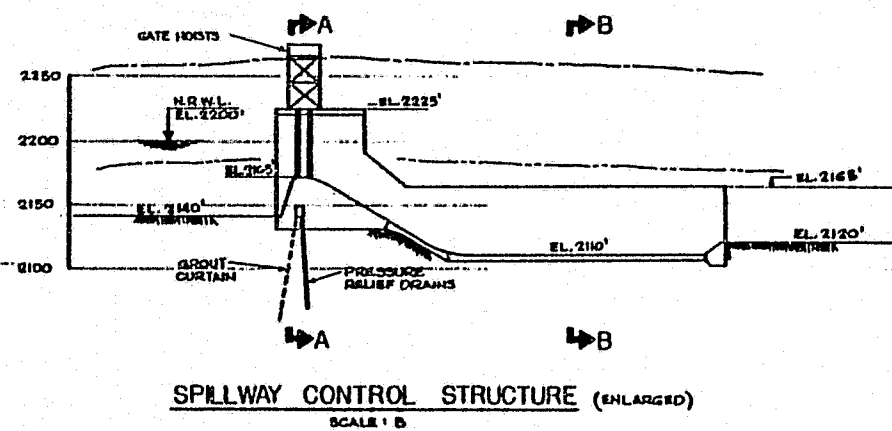
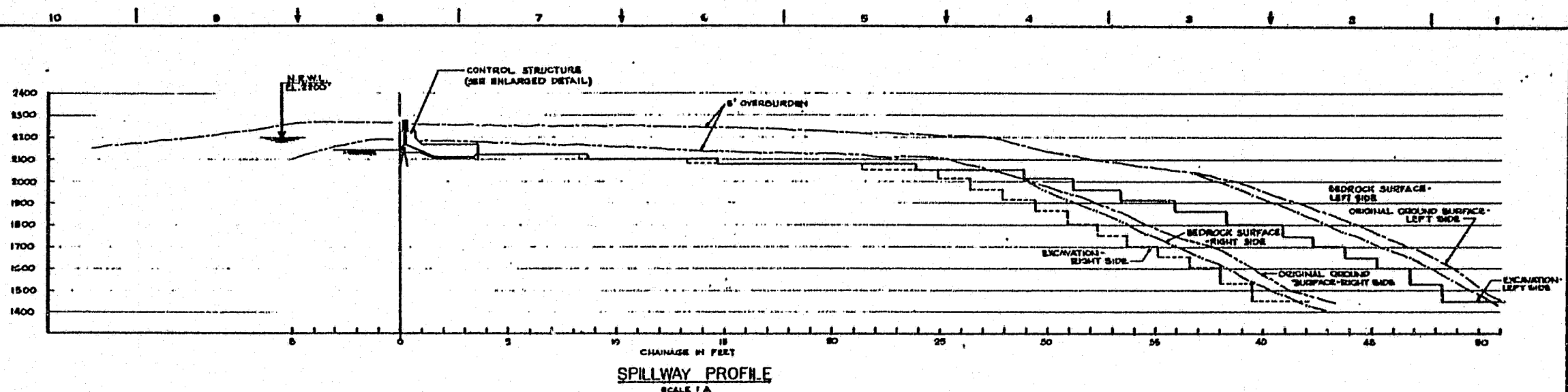
SCALE 0 100 200 FEET

DATE	NO.	REVISION	BY	CHK.	APP.	APPROVED
	4					

ALASKA POWER AUTHORITY	
SUBMITTA HYDROELECTRIC PROJECT	
WATANA	
SCHEME 3A1	
SPILLWAY	
DATE: JUNE, 1968	SCALE: 1" = 100'
PROJECT NO.: SK-5700-08-210	BY: JH



00158



DATE	BY	REVISION
		GENERAL REVISION

PLATE 7.7

ALASKA POWER AUTHORITY

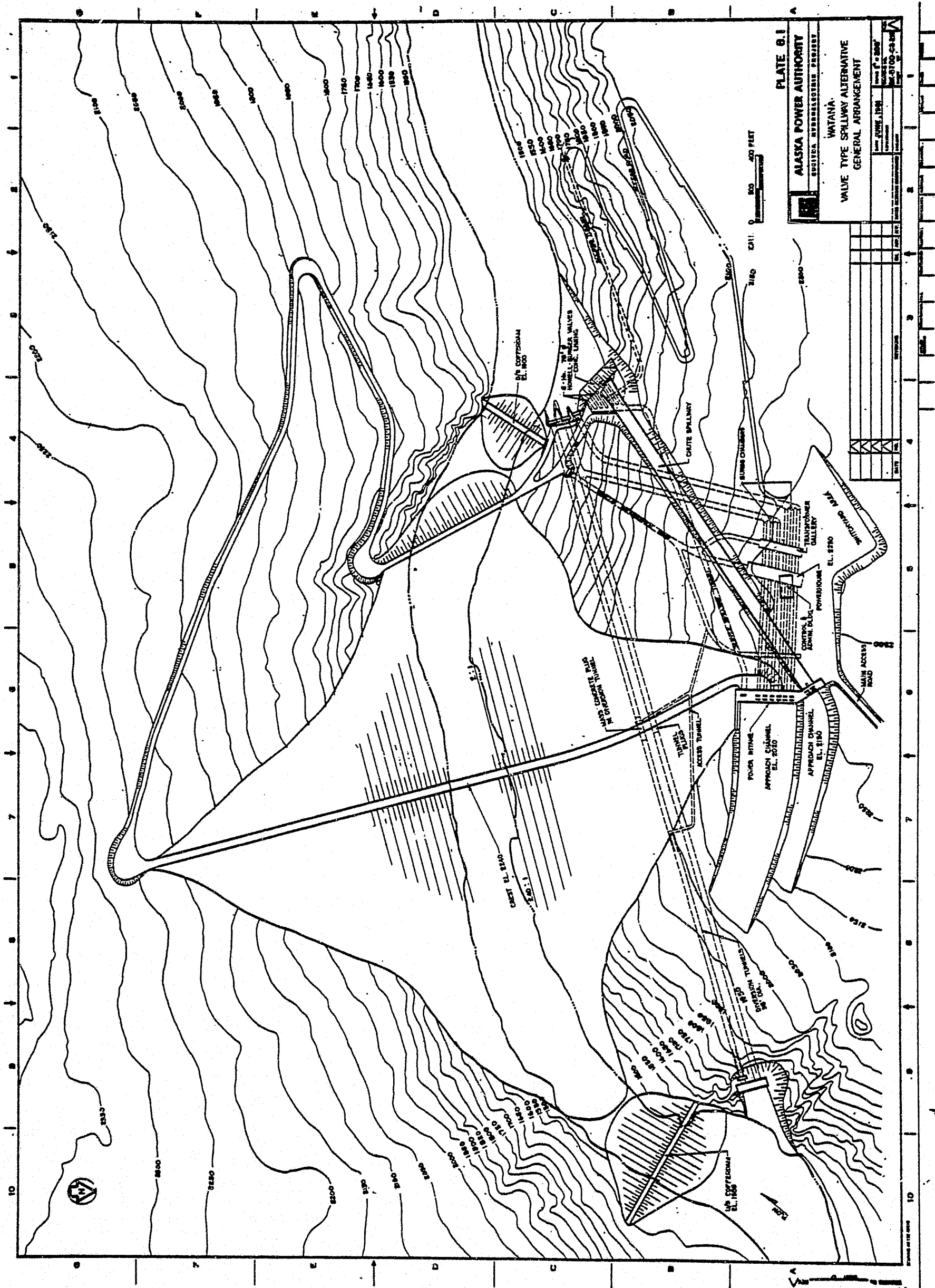
SMITHA HYDROELECTRIC PROJECT

WATANA
SCHEME 4A
SPILLWAY

DATE: JUNE, 1974

SCALE: AS SHOWN

5700-2-210



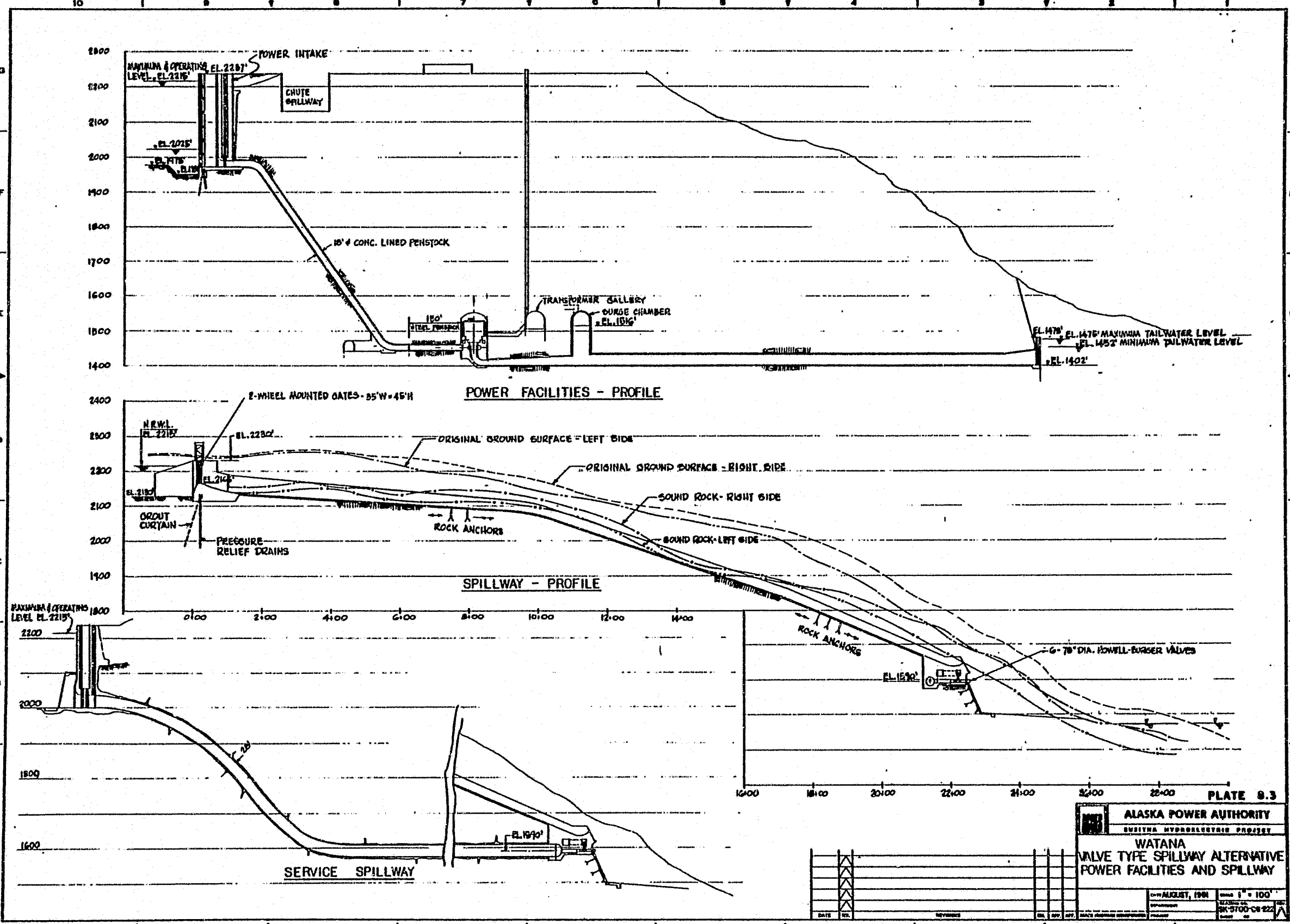


PLATE 8.3

ALASKA POWER AUTHORITY
 SUSITNA HYDROELECTRIC PROJECT

WATANA
 VALVE TYPE SPILLWAY ALTERNATIVE
 POWER FACILITIES AND SPILLWAY

DATE: AUGUST, 1981 SCALE: 1" = 100'

NO.	DATE	BY	CHKD.	REV.	DESCRIPTION
1					
2					
3					
4					

3K-5700-08-222

0001380

—
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—
—



WATANA DAM DESIGN

—
—
—
—



001380

WATANA DAM DESIGN

CONTENTS

1 - GENERAL

2 - TYPICAL CROSS SECTIONS

2.1 - GENERAL

2.2 - CREST DETAIL

3 - INSTRUMENTATION

00138

WATANA DAM

1 - GENERAL

THE WATANA DAM WILL CONSIST OF A CENTRAL VERTICAL IMPERVIOUS CORE PROTECTED BY FINE AND COARSE FILTERS ON THE UPSTREAM AND DOWNSTREAM SLOPES. THE OUTER SHELLS OF THE DAM WILL CONSIST OF RIVER ALLUVIUM GRAVELS AND ROCKFILL MATERIAL. THE DESIGN SHOULD PROVIDE A STABLE EMBANKMENT UNDER ALL LOADING CONDITIONS. DYNAMIC AND STATIC STABILITY ANALYSIS ARE IN PROGRESS.

2 - TYPICAL CROSS SECTION

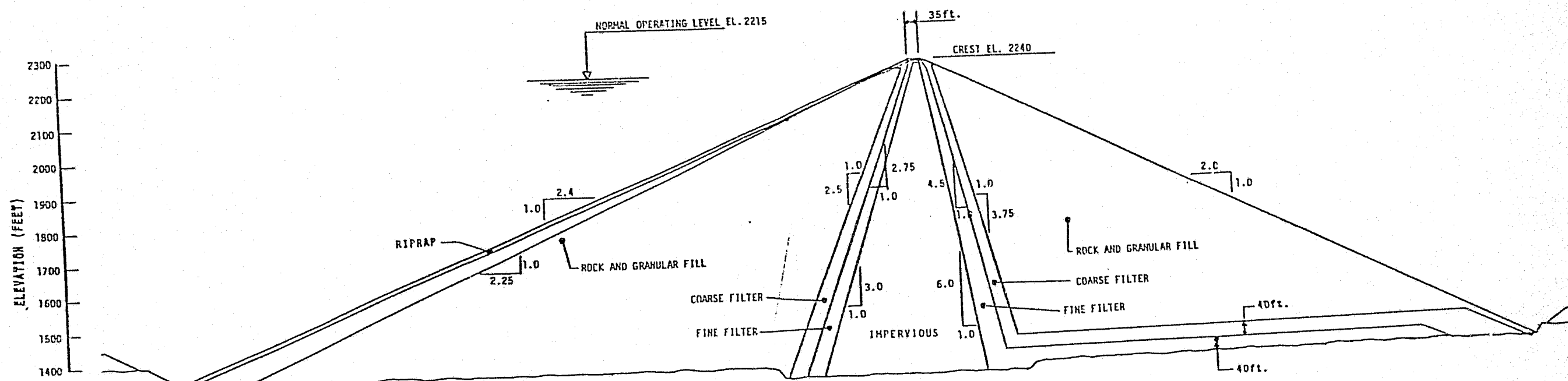
2.1 - GENERAL

THE TYPICAL CROSS SECTION OF THE DAM AT THE MAXIMUM HEIGHT IS SHOWN ON FIGURE NO. 1. THE SLOPES AND THICKNESS OF THE VARIOUS ZONES ARE BASED ON A REVIEW OF PREVIOUS DESIGNS AND THE PROPERTIES OF THE CONSTRUCTION MATERIALS. MINIMUM CORE-FOUNDATION CONTACT WILL BE 100 FEET REQUIRING FLARING OF THE CROSS SECTION AT EACH END OF THE EMBANKMENT. THE UPSTREAM AND DOWNSTREAM FILTERS ARE SIZED TO PROVIDE PROTECTION AGAINST POSSIBLE LEAKAGE THROUGH TRANSVERSE CRACKS IN THE CORE THAT COULD OCCUR DUE TO SETTLEMENT OR RESULTING FROM DISPLACEMENT DURING A SEISMIC EVENT. THE WIDE FILTER ZONES PROVIDE SUFFICIENT MATERIAL FOR HEALING OF ANY CRACKS IN THE CORE AND THE INCREASED SIZE OF THE DOWNSTREAM COARSE FILTER WILL ENSURE ITS CAPABILITY TO HANDLE ANY ABNORMAL LEAKAGE FLOWS.

THE SHELLS OF THE DAM WILL CONSIST OF ZONES OF ROCKFILL AND RIVER ALLUVIUM GRAVELS. SLOPE PROTECTION ON THE UPSTREAM SLOPE WILL CONSIST OF A 10 FOOT ZONE OF OVERSIZED MATERIAL UP TO 6 FEET IN DIAMETER.

2.2 - CREST DETAIL

THE PROPOSED CREST DETAIL IS SHOWN ON FIGURE NO. 2. THE SLOPES AND THICKNESS OF THE VARIOUS ZONES ARE CONTROLLED BY THE 35 FOOT WIDE CREST.



MAIN DAM SECTION AT MAXIMUM HEIGHT
FIGURE NO. 1

3 - INSTRUMENTATION

INSTRUMENTATION WILL BE INSTALLED WITHIN ALL PARTS OF THE DAM TO PROVIDE MONITORING DURING CONSTRUCTION AS WELL AS DURING OPERATIONS. INSTRUMENTS FOR MEASURING INTERNAL VERTICAL AND HORIZONTAL DISPLACEMENTS, STRESSES AND STRAINS, AND TOTAL AND FLUID PRESSURES AS WELL AS SURFACE MONUMENTS AND MARKERS WILL BE INSTALLED. THE QUANTITY AND LOCATION WILL BE DECIDED DURING FINAL DESIGN. TYPICAL INSTRUMENTATION IS AS FOLLOWS:

- A) PIEZOMETERS TO MEASURE THE STATIC PRESSURE OF THE FLUID IN THE PORE SPACES OF THE SOIL AND ROCKFILL.
- B) DEVICES FOR MEASURING INTERNAL VERTICAL MOVEMENT.
 - 1) CROSS-ARM SETTLEMENT DEVICES AS DEVELOPED BY THE USBR.
 - 2) VARIOUS VERSIONS OF TAUT-WIRE DEVICES HAVE BEEN DEVELOPED TO MEASURE INTERNAL SETTLEMENTS.
 - 3) HYDRAULIC SETTLEMENT DEVICES OF VARIOUS KINDS.
- C) INTERNAL HORIZONTAL MOVEMENT DEVICES
 - 1) TAUT-WIRE ARRANGEMENTS.
 - 2) CROSS-ARM DEVICES.
 - 3) INCLINOMETERS.
 - 4) STRAIN METERS.

D) OTHER MEASURING DEVICES

- 1) STRESS METERS.
- 2) SURFACE MONUMENTS AND ALIGNMENT MARKERS.
- 3) SEISMOGRAPHIC RECORDERS AND SEISMOSCOPES.

STATIC AND DYNAMIC
STABILITY ANALYSES
FOR THE
WATANA DAM

TABLE OF CONTENTS

- 1 - INTRODUCTION
- 2 - METHODOLOGY
- 3 - STATIC ANALYSES
 - 3.1 - General
 - 3.2 - Soil Properties
 - 3.3 - Loading Conditions and Factors of Safety
 - 3.4 - SAP IV Analyses
- 4 - DYNAMIC ANALYSES
 - 4.1 - General
 - 4.2 - Soil Properties
 - 4.3 - Design Earthquake
 - 4.4 - Dynamic Analyses

NOTE: Only Figure 1 Dam Cross Section, Figure 3 Static Stability Analysis, Normal Operating Case and Figure 12 Design Earthquake Time History are included with this submittal.

1 - INTRODUCTION

Static and dynamic stability analyses are being performed to establish the upstream and downstream slopes of the Watana dam. Static and slope stability analyses indicates stable slopes under all conditions for a 2.25 horizontal to 1.0 vertical upstream slope and a 2.0 horizontal to 1.0 vertical downstream slope. Typical cross section and details are shown in Figure No. 1.

The static analyses have been done using the STABL computer program developed to handle general slope stability problems by adaptation of the Modified Bishop method and a linear elastic finite element program (SAPIV) to determine the initial stresses in the dam during normal operating conditions. The results and conclusions are presented in the Static Analyses Section.

The dynamic analyses have been done using the QUAD 4 finite element program which incorporates strain dependent shear modulus and damping parameters. The design earthquake for the dynamic analyses was developed by Woodward-Clyde Consultants for a Benioff zone event. The results available at this time are presented in the Dynamic Analyses Section.

2 - METHODOLOGY

An assessment of the static and seismic response of the Watana dam for the static and postulated seismic loading involves the following:

Static Analyses

- STABL program to determine general slope stability.
- SAPIV (linear elastic finite element program) to determine the initial stresses in the dam prior to seismic event.

Dynamic Analysis

- QUAD 4 program to determine the dynamic shear stresses due to the postulated earthquake.
- GADFLEA program to determine the pore water pressure generation and dissipation.

The data available on-site specific materials are limited, and therefore, the static and dynamic properties were assigned using the material characteristics and published information.

3 - STATIC ANALYSES

3.1 - General

The slope stability analyses were done using the STABL computer program for the general solution of slope stability problems by a two-dimensional limiting equilibrium method. The calculation of the factor of safety against instability of a slope is performed by an adaptation of the Modified Bishop method of slices which allows the analysis of trial failure surfaces other than those of circular shape.

3.2 - Soil Properties

The following soil properties were used in the analyses:

	<u>γ Wet (lb/ft³)</u>	<u>γ Sat (lb/ft³)</u>	<u>ϕ°</u>	<u>C (lb/ft²)</u>
Core Material	146	147	30	500
Filter Material	146	150	35	0
Rockfill Material	129	140	40	0

3.3 - Loading Conditions and Factors of Safety

The following conditions were analyzed:

<u>Condition</u>	<u>Minimum Factor of Safety</u>	<u>Calculated Factor of Safety</u>	
		<u>U/S Slope</u>	<u>D/S Slope</u>
Construction	1.3	2.0-2.2	
Normal Operating	1.5	2.0	
Rapid Drawdown	1.0	1.8-2.0	
Normal Operating With Maximum Pool	1.3	2.0-2.1	

The calculated factors of safety as shown in the above table and on Figures No. 2 through 5 indicate no general slope stability problems.

3.4 - SAP IV Analysis

The static analyses using the linear elastic finite element program (SAPIV) were done to determine the initial stresses in the dam during normal operating conditions. Young's Modulus was determined from the following relationship:

$$E = K p_a \left(\frac{\sigma_3}{p_a} \right)^n$$

where: E == Young's Modulus

p_a == Atmospheric Pressure

σ₃ = Confining Pressure

K, n = Constants

The initial values of the parameters K and n and the Poisson's Ratio (ν) for the various dam materials used in the program are as follows:

	<u>-K</u>	<u>ν</u>	<u>n</u>
Core Material	300	0.333	0.2
Filter Material	2000	0.299	0.2
Rockfill Material	2000	0.263	0.3

A parametric study utilizing a reasonable range for the K values will be performed at a later date.

The resulting initial stresses are used to determine material properties for QUAD 4 and for assessment of dynamic undrained strength.

The following plots were developed from the linear elastic finite element program. (SAPIV):

Figure No.

Subject

6

Finite Element Mesh

7

Two-Dimensional Confining Pressure

8

Three-Dimensional Confining Pressure

9

Vertical Effective Stress

10

Shear Stress

11

Maximum Shear Stress

4.4 - Dynamic Analyses

Two one-dimensional models were analyzed to determine the sensitivity to the number of elements. One model consisted of 20 elements of equal size while the other model consisted of 8 elements. Both were the same height as the full dam model. Convergence was reached in three iterations. A comparison of the results indicate that the full dam model as proposed would provide satisfactory results.

The two-dimensional finite element model of the dam Figure No. 6 was analyzed and the following plots were developed:

<u>Figure No.</u>	<u>Subject</u>
13	Maximum Dynamic Shear Stresses
14	Maximum Dynamic Shear Stresses Divided by the Vertical Effective Stress
15	Vertical Slice Acceleration at Center of Dam
16	Stress History for Elements 20 and 30
17	Stress History for Elements 33 and 35
18	Stress History for Elements 54 and 57
19	Stress History for Elements 59 and 78
20	Stress History for Elements 81 and 83
21	Stress History for Elements 94 and 96
22	Stress History for Element 105

A copy of the computer output showing the accelerations, material properties, and maximum stresses from the three iterations is included in Appendix A.

No laboratory data is available for the cyclic shear strength of rockfill. The procedure adopted by Sidigh et. al. (1978) will be followed, i.e. cyclic strength values for sand at the same relative density as the rockfill will be increased by 70%. Strength will be defined as 5% strain in 25 cycles.

A comparison of the results from the dynamic analyses and the calculated available shear strength of the rockfill material is in progress at this time.

4 - DYNAMIC ANALYSES

4.1 - General

The dynamic analyses were done using the QUAD-4 computer program. Complete details of the program including theoretical basis, data preparation, and output are given in the manual (Idriss, Lysmer, Wang, and Seed, 1973). The dynamic analysis model is shown on Figure No. 6.

4.2 - Soil Properties

The initial values of shear modulus and damping ratio used in the analyses were derived from typical values available in the literature and are as follows:

	<u>G/Su</u>	<u>K₂</u>	<u>Damping/Shear Type Curve</u>
Core Material	2500	--	Clay
Filter Material	-	100	Sand
Rock Material	-	180	Sand

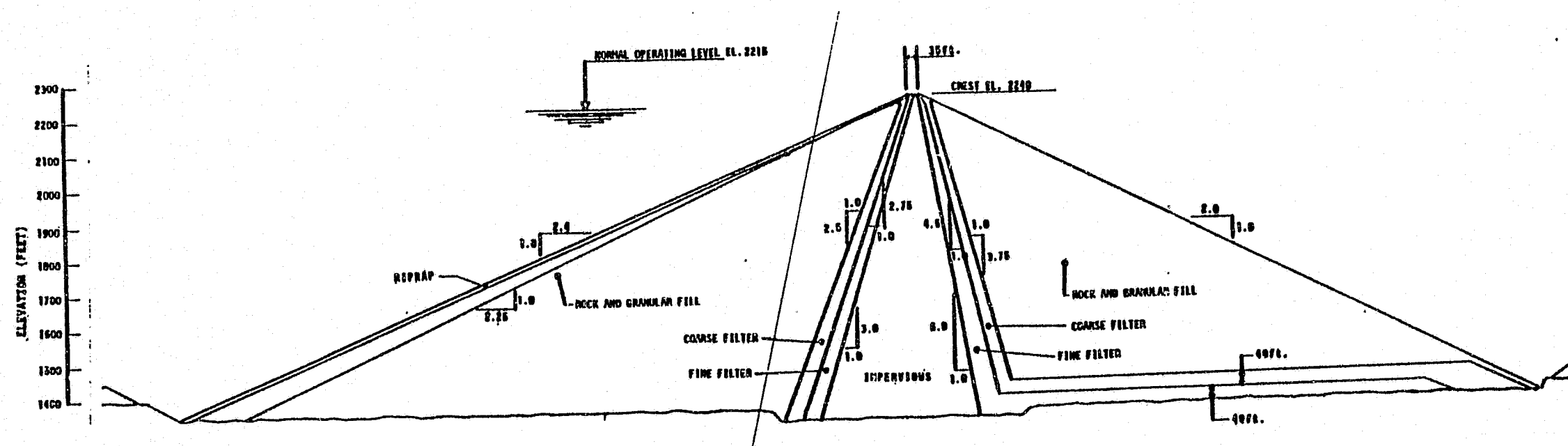
A parametric study utilizing a reasonable range of K₂ values will be performed at a later date.

4.3 - Design Earthquake

The required earthquake time history was developed by Woodward-Clyde Consultants and is shown on Figure No. 12. The significant features of this earthquake time history are as follows:

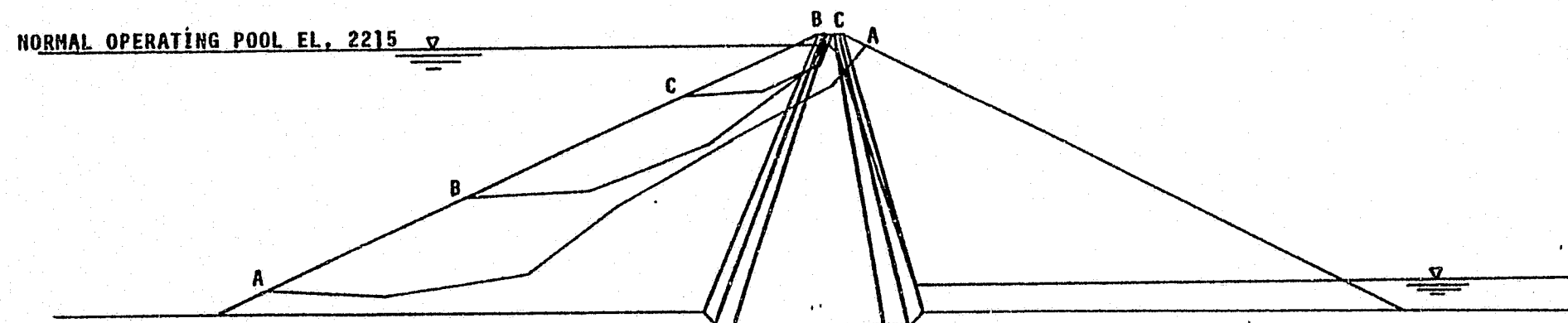
- Magnitude 8.5 Richter.
- Location 40 kilometers below site (Benioff Zone).
- Maximum acceleration of .436g.
- Duration of strong motion - 45 sec.
- Significant number of cycles - 25.

The permeability of the rockfill material is estimated to be greater than 100 cm/sec indicating that possibly the rate of pore pressure dissipation will be equal to or greater than the pore pressure generation. A parametric study will be done using the GADFLCA program to establish the required rockfill properties for equal generation and dissipation.



MAIN DAM SECTION AT MAXIMUM HEIGHT
FIGURE NO. 1





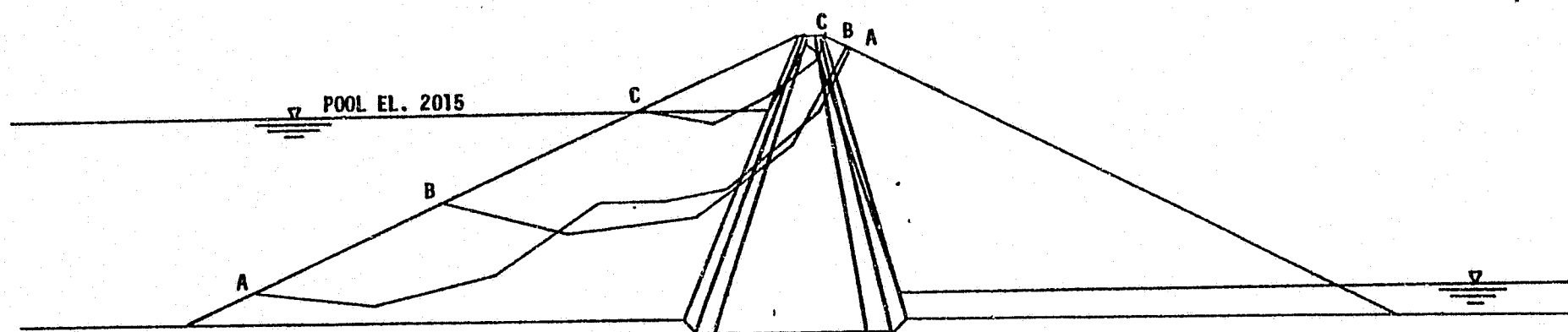
FAILURE SURFACE. SAFETY FACTOR

A-A	2.0
B-B	2.0
C-C	2.0

NORMAL OPERATING CASE

FIGURE NO. 3



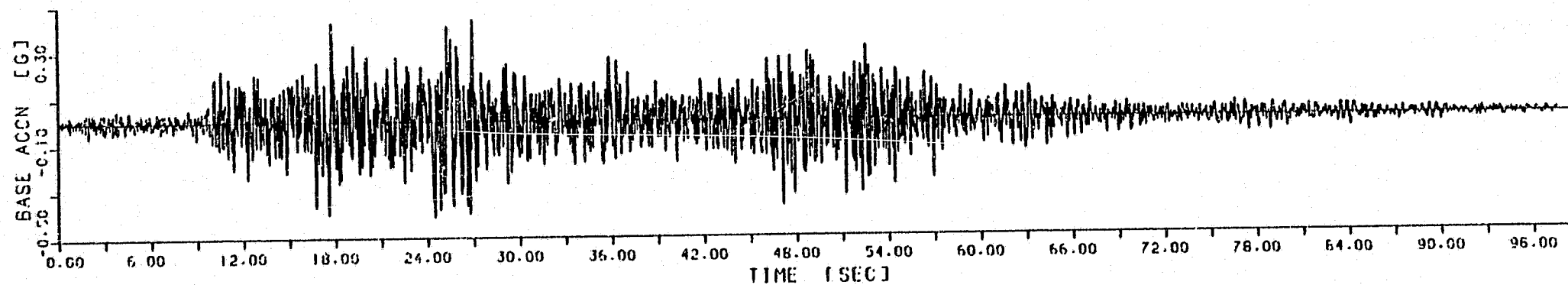


FAILURE SURFACE	SAFETY FACTOR
A-A	1.8
B-B	1.8
C-C	2.0

RAPID DRAWDOWN CASE

FIGURE NO. 4





DESIGN EARTHQUAKE
TIME HISTORY

FIGURE NO. 12



REFERENCES

1. Idriss, I.M., Lysmer, J., Hwang, R., and Seed, H. Bolton (1973) "QUAD-4, A Computer Program for Evaluating the Seismic Response of Soil Structures by Variable Damping Finite Elements," Earthquake Engineering Research Center, Report No. EERC73-16, University of California, Berkeley, August.
- X 2. Sadigh, K., Idriss, I.M., and Youngs, R.R. (1978) "Drainage Effects on Seismic Stability of Rockfill Dams," Proceedings ASCE Geo. Engin. Div., Specialty Conference on Earthquake Engin. and Soil Dynamics, Vol. III, June 19-21, Pasadena, California.

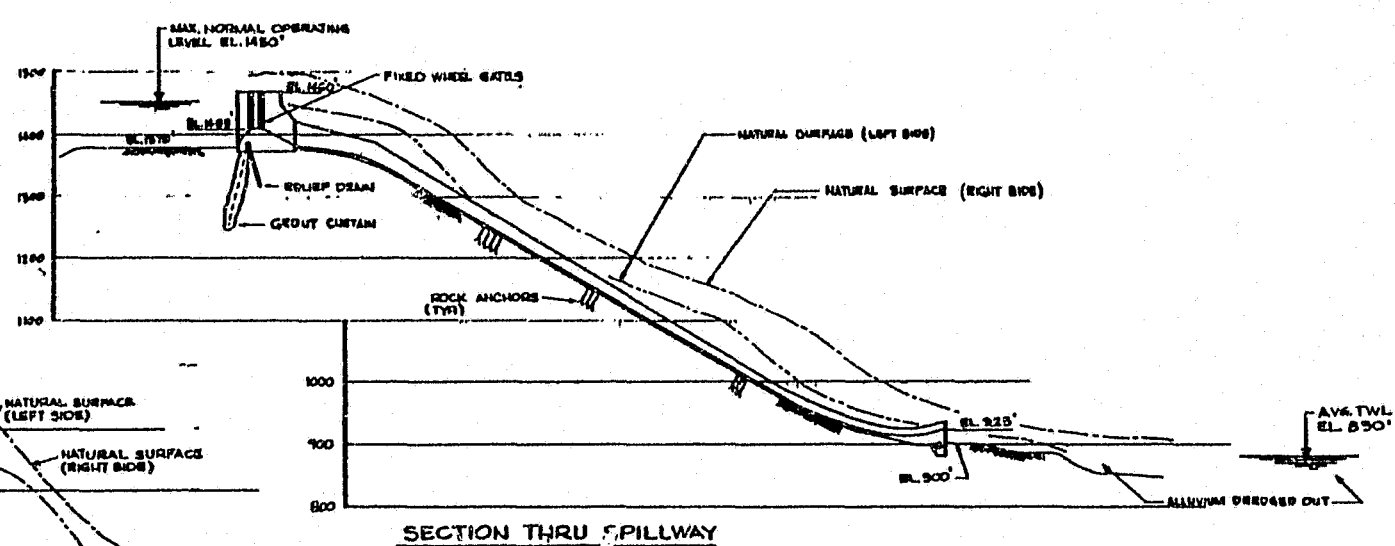
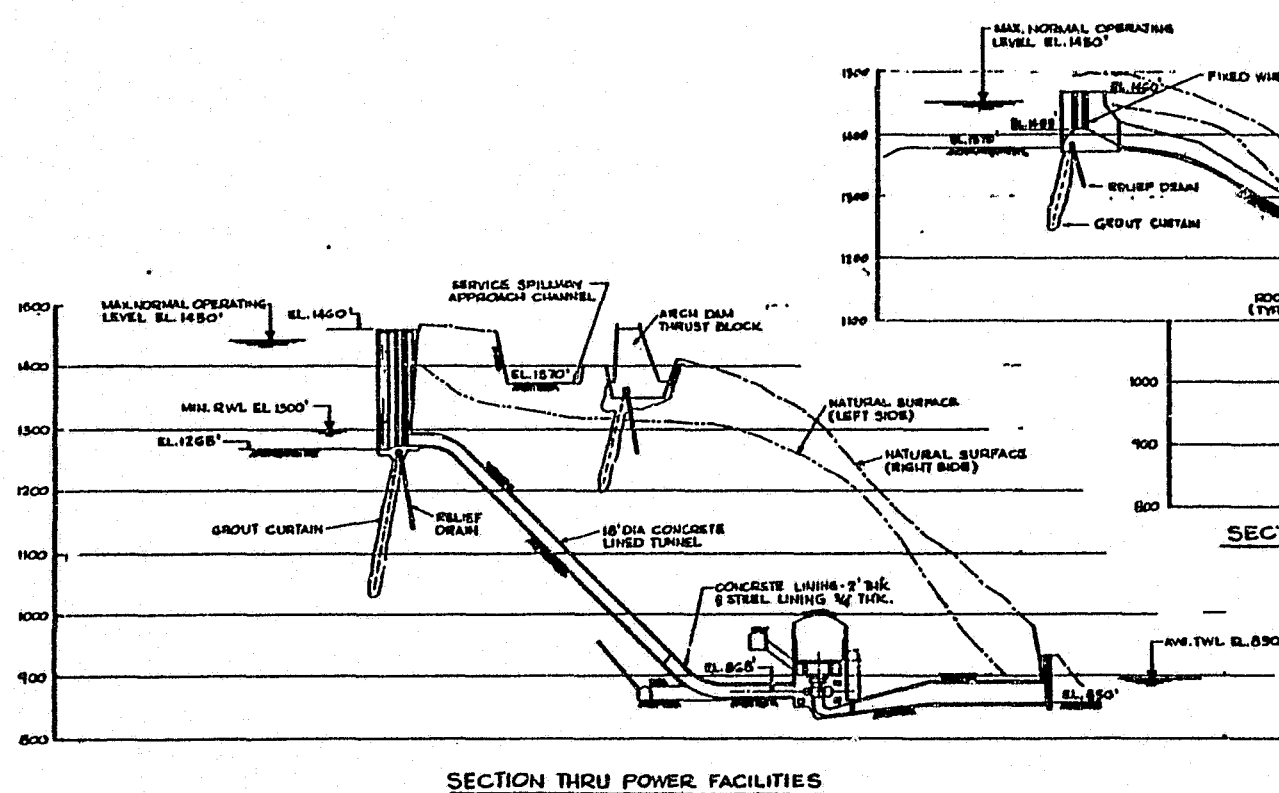
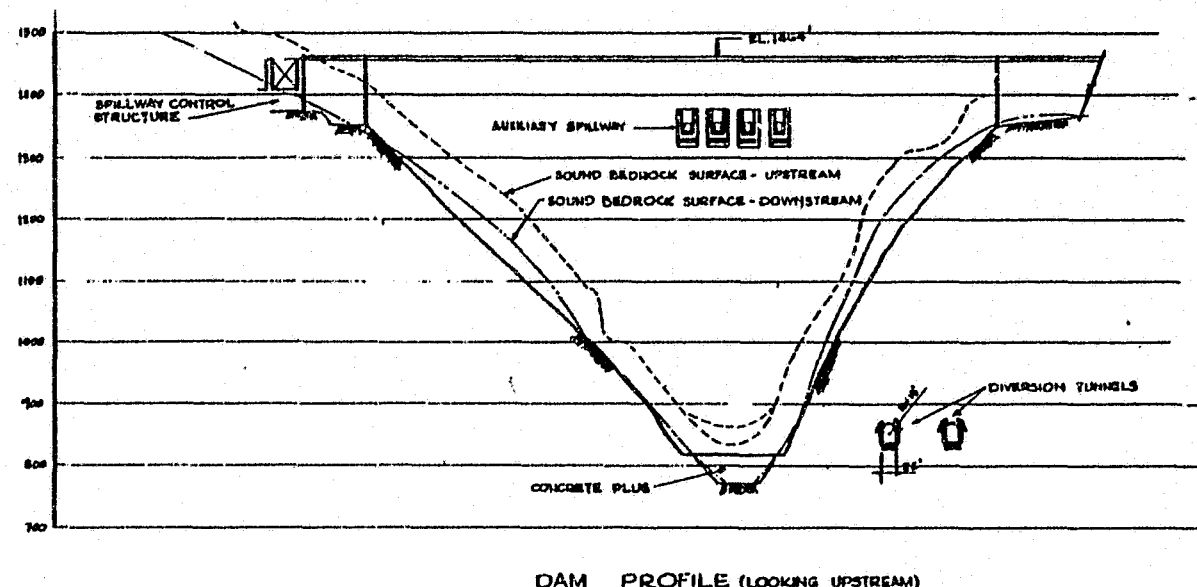
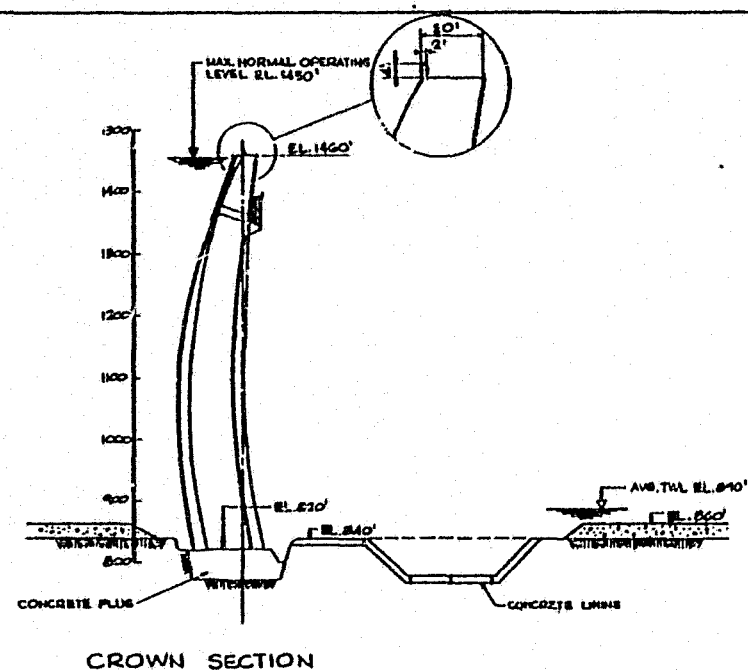
DEVIL CANYON LAYOUT STUDIES

1 - INTRODUCTION

The layout for the Devil Canyon Project presented in the DSR (Plates 10 and 11) were the basis of further refinement and optimization studies.

Principal features of the DSR layout are as follows:

- Dam: Thin arch concrete with earthfill saddle dam (left abutment)
 - . Crest Elevation: 1,460 feet
 - . Height: 650 feet
- Diversion: Twin concrete-lined tunnels, 26 feet diameter
- Main Spillway:
 - . Type: Three-gated ogee section, chute and flip
 - . Capacity: 90,000 cfs
 - . Location: Right abutment
- Auxiliary Spillway:
 - . Type: Three-gated orifice, 15 feet x 15 feet
 - . Capacity: 40,000 cfs
 - . Location: In arch dam
- Emergency Spillway:
 - . Type: Fuse plug
 - . Capacity: 100,000 cfs maximum
 - . Location: Left bank
- Power Plant:
 - . Location: Underground, right bank
 - . Capacity: 400 MW, 4 units



SCALE 0 100 200 FEET

PLATE II

ALASKA POWER AUTHORITY

SUSTINA HYDROELECTRIC PROJECT

DEVIL CANYON

SCHEME 1

SECTIONS

DATE	BY	CHECKED	APPROVED

2 - ALTERNATIVE LAYOUTS CONSIDERED

The DSR layout formed the basis of the continuation of Devil Canyon layout studies from which three basic components were developed for comparison (Plates 7.1 through 7.4).

These layouts were based on the criteria listed in Table 2.1.

Cost comparisons are listed in Table 1.

TABLE 2.1 - PROJECT PARAMETERS AND DESIGN CRITERIA

River Flows

Average flow (over 30 years of record):	8,960 cfs
Probable maximum flood:	270,000 cfs
Maximum flood with return period of 1:10,000 years:	135,000 cfs (after routing through Watana)
Maximum flood with return period of 1:50 years:	42,000 cfs (after routing through Watana)
Reservoir normal maximum operating level:	1,455 feet MSL
Reservoir minimum operating level:	1,400 feet MSL
Area of reservoir at maximum operating level:	21,000 acres
Reservoir live storage:	0.75×10^6 acre feet
Reservoir full storage:	1.1×10^6 acre feet

Dam

Type:	Concrete arch
Crest elevation:	1,455 feet MSL
Height:	635 feet above foundation
Cut-off and foundation treatment:	Founded on rock. Grout curtain and downstream drains

Diversion

Discharge capacity:	42,000 cfs
Cofferdam types:	Rockfill
Cut-off and foundation:	Founded on alluvium with slurry trench to rock
Upstream cofferdam crest elevation:	960 feet MSL
Downstream cofferdam crest elevation:	900 feet MSL
Maximum pool level during construction:	955 feet MSL
Water passages:	Concrete lined
Outlet structures:	Low level structure with slide closure gate
Final closure:	Mass concrete plugs in line with dam grout curtain
Releases during impounding:	2,000 cfs minimum

Spillway

Design floods:	Passes P.M.F., preserving integrity of dam
	Passes routed 1:10,000 year flood with no damage to structures

TABLE 2.1 (Continued)

Power Intake

Type:

Massive concrete
structure embedded in
rock

Number of intakes:

4

Penstocks

Type:

Concrete-lined rock
tunnels with downstream
steel liner

Number of penstocks:

4

Powerhouse

Type:

Underground

Transformer area:

Separate gallery

Control room and administration:

Separate gallery

Access:

Rock tunnel

TABLE 1 - DEVIL CANYON
COMPARATIVE COST ESTIMATE - NON-COMMON ITEMS
JULY 1981 (JANUARY 1982 DOLLARS)

Page 1 of 4

Description	Unit	Unit Price	Scheme 1		Scheme 2		Scheme 3	
			Quantity	\$000	Quantity	\$000	Quantity	\$000
<u>Diversion/Cofferdams</u>								
Excavate Rock Portal	cy	15	114,500	\$ 1,718	114,500	\$ 1,718	114,500	\$ 1,718
Excavate Overburden Portal	cy	5.50	146,000	803	146,000	803	146,000	803
Excavate Tunnels	cy	55	107,550	5,915	107,550	5,915	130,050	7,153
Concrete Liner at Portals	cy	260	3,250	845	3,250	845	3,250	845
Concrete Linor Tunnel	cy	250	19,500	4,875	19,500	4,875	24,000	6,000
Concrete Inlet Portal Headwall	cy	260	5,200	1,352	5,200	1,352	5,200	1,352
Concrete Inlet Portal Pier	cy	295	500	148	500	148	500	148
Concrete Outlet Headwall	cy	260	9,730	2,530	9,730	2,530	9,730	2,530
Concrete Plugs	cy	500	5,600	2,800	5,600	2,800	5,600	2,800
Upstream Cofferdam	cy	7.50	92,000	690	92,000	690	92,000	690
Reinforcing Steel	Ton	2,100	550	1,155	550	1,155	600	1,260
Rockbolts Tunnels	Ton	3,500	135	473	135	473	165	578
Rockbolts Portals	Ton	3,500	350	1,225	350	1,225	350	1,225
Rock Surface Treatment	sy	15.80	1,600	25	1,600	25	1,600	25
Support Steel	Ton	3,675	380	1,397	380	1,397	460	1,691
Gates, Etc.	LS	3,100,000		3,100		3,100		3,100
SUBTOTAL DIVERSION/ COFFERDAMS				\$ 32,051		\$ 32,051		\$ 34,918

DEVIL CANYON
COMPARATIVE COST ESTIMATE - NON-COMMON ITEMS (Cont'd)

Page 2 of 4

Description	Unit	Unit Price	Scheme 1		Scheme 2		Scheme 3	
			Quantity	\$000	Quantity	\$000	Quantity	\$000
Service Spillway								
Excavate Rock	cy	17	625,000	\$10,625	559,000	\$ 9,503	1,176,000	\$19,992
Concrete Walls Formed One Face	cy	260	20,200	5,252	33,500	8,710	57,700	15,002
Concrete Walls Formed Both Faces	cy	280	6,100	1,708	8,900	2,492	26,700	7,476
Concrete Slab No Forms	cy	170	29,100	4,947	30,500	5,185	44,800	7,616
Concrete Slab Formed One Side	cy	190	16,800	3,192	22,600	4,294	16,800	3,192
Concrete Slab Formed Two Sides	cy	200		NA		NA	17,400	3,480
Concrete Piers	cy	295	6,700	1,977	6,700	1,977	6,700	1,977
Concrete Road Deck	cy	170		NA		NA	6,700	1,139
Reinforcing Steel	ton	2,100	2,664	5,595	3,274	6,875	5,418	11,387
Rockbolts	ton	3,500	500	1,750	580	2,030	750	2,625
Gates	LS	2,900,000		2,900		2,900		2,900
Dental Concrete	LS	2,000,000		2,000		2,000		2,000
Grouting	LS	1,000,000		1,000		1,000		1,000
Winterization	LS			1,800		2,300		4,000
Auxiliary Cofferdam	cy	7	36,000	252	36,000	252	36,000	252
Alluvium Removal	cy	12	23,000	276	23,000	276	54,000	648
Plunge Pool Excavation	cy	7	398,000	2,786	398,000	2,786		NA
Additional Haul Roads	mile	40,000	1	40	1	40		NA
Dewatering	LS			300		300		NA
Surface Treatment Vertical	sy	15.80	10,250	162	11,300	179	18,900	299
Surface Treatment Horizontal	sy	10.50	18,400	193	21,300	224	23,100	242
SUBTOTAL SERVICE SPILLWAY				\$ 46,755		\$ 53,323		\$ 85,227

DEVIL CANYON
COMPARATIVE COST ESTIMATE - NON-COMMON ITEMS (Cont'd)

Page 2 of 4

Description	Unit	Unit Price	Scheme 1 Quantity	\$000	Scheme 2 Quantity	\$000	Scheme 3 Quantity	\$000
<u>Service Spillway</u>								
Excavate Rock	cy	17	625,000	\$10,625	559,000	\$ 9,503	1,176,000	\$19,992
Concrete Walls Formed One Face	cy	260	20,200	5,252	33,500	8,710	57,700	15,002
Concrete Walls Formed Both Faces	cy	280	6,100	1,708	8,900	2,492	26,700	7,476
Concrete Slab No Forms	cy	170	29,100	4,947	30,500	5,185	44,800	7,616
Concrete Slab Formed One Side	cy	190	16,800	3,192	22,600	4,294	16,800	3,192
Concrete Slab Formed Two Sides	cy	200		NA		NA	17,400	3,480
Concrete Piers	cy	295	6,700	1,977	6,700	1,977	6,700	1,977
Concrete Road Deck	cy	170		NA		NA	6,700	1,139
Reinforcing Steel	ton	2,100	2,664	5,595	3,274	6,875	5,418	11,387
Rockbolts	ton	3,500	500	1,750	580	2,030	750	2,625
Gates	LS	2,900,000		2,900		2,900		2,900
Dental Concrete	LS	2,000,000		2,000		2,000		2,000
Grouting	LS	1,000,000		1,000		1,000		1,000
Winterization	LS			1,800		2,300		4,000
Auxiliary Cofferdam	cy	7	36,000	252	36,000	252	36,000	252
Alluvium Removal	cy	12	23,000	276	23,000	276	54,000	648
Plunge Pool Excavation	cy	7	398,000	2,786	398,000	2,786		NA
Additional Haul Roads	mile	40,000	1	40	1	40		NA
Dewatering	LS			300		300		NA
Surface Treatment Vertical	sy	15.80	10,250	162	11,300	179	18,900	299
Surface Treatment Horizontal	sy	10.50	18,400	193	21,300	224	23,100	242
SUBTOTAL SERVICE SPILLWAY				\$ 46,755		\$ 53,323		\$ 85,227

DEVIL CANYON
COMPARATIVE COST ESTIMATE - NON-COMMON ITEMS (Cont'd)

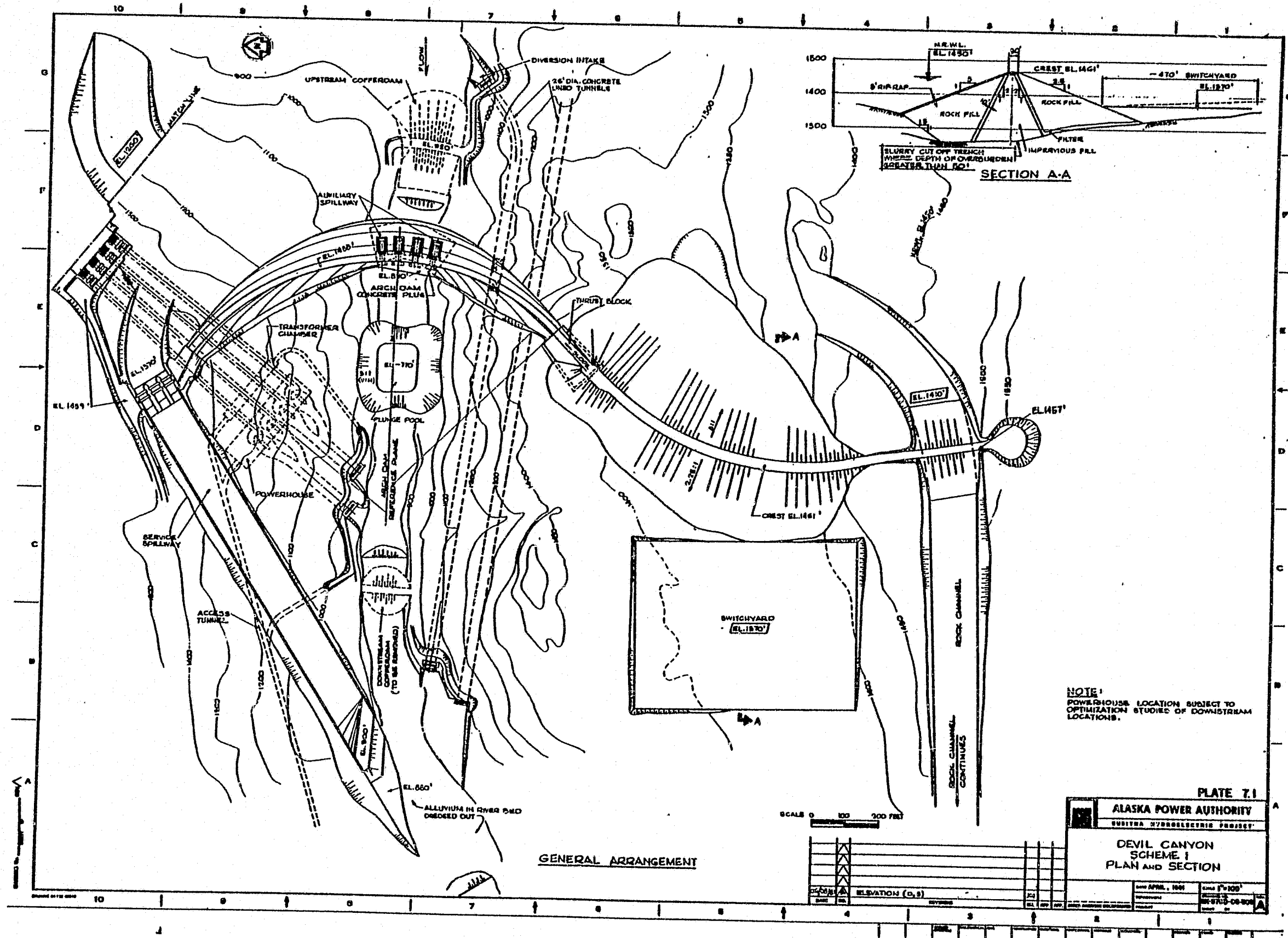
Page 3 of 4

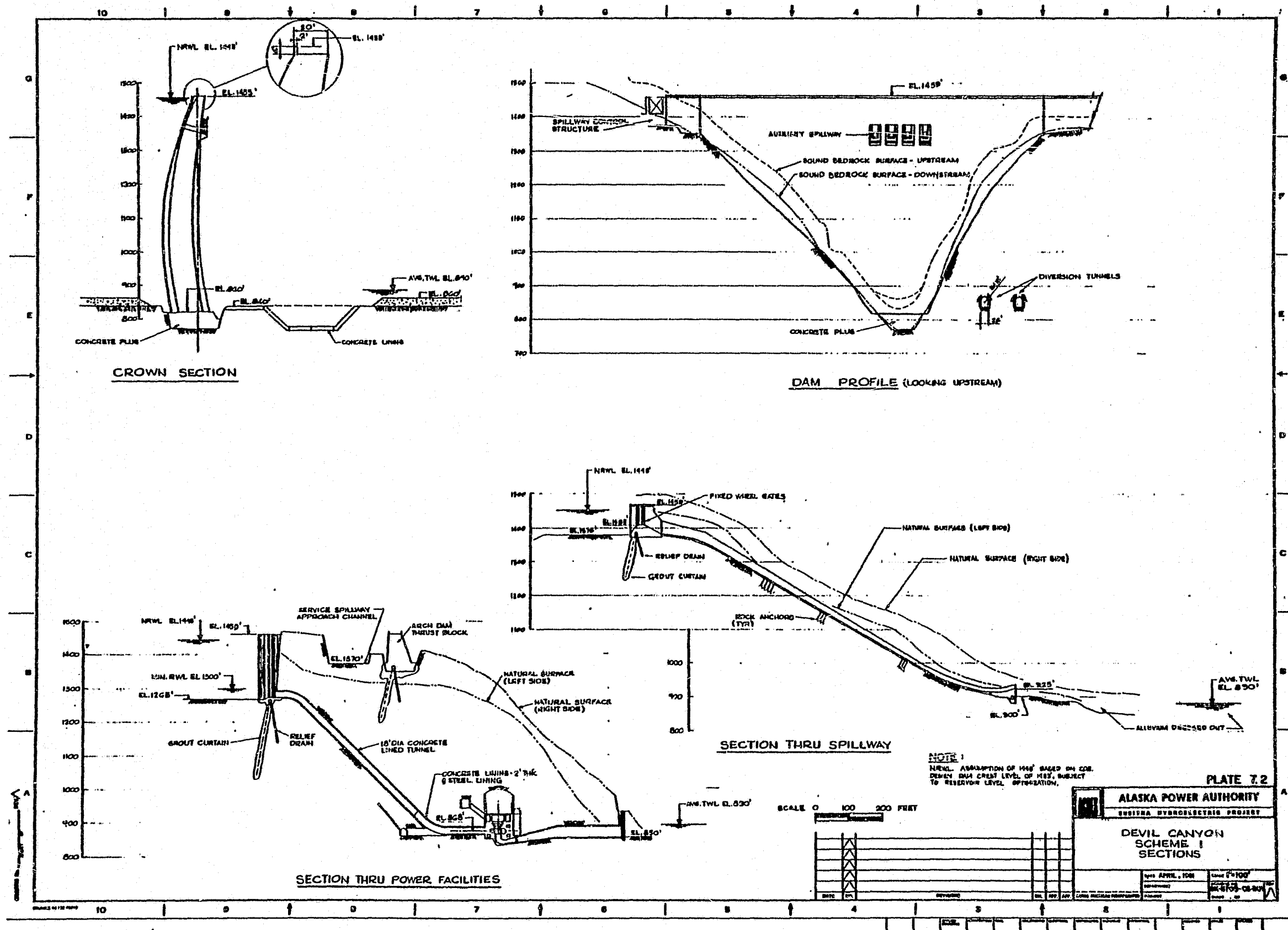
Description	Unit	Unit Price	Scheme 1 Quantity	\$000	Scheme 2 Quantity	\$000	Scheme 3 Quantity	\$000
<u>Emergency Spillway</u>								
Excavate Rock	cy	17	1,468,000	\$24,956	1,468,000	\$ 24,956	1,468,000	\$24,956
Place Fuse Plug	cy	6	43,000	258	43,000	258	43,000	258
SUBTOTAL EMERGENCY SPILLWAY				25,214		25,214		25,214
<u>Saddle Dam</u>								
Excavate Overburden	cy	6.30	240,000	1,512	251,000	1,581	240,000	1,512
Earth Fill	cy	12.50	700,000	8,750	496,000	6,200	700,000	8,750
Core	cy	10.50	285,000	2,993	347,000	3,644	285,300	2,993
Transition	cy	16.00	76,000	1,216	91,000	1,456	76,000	1,216
Rip Rap	cy	12.50	34,000	425	23,500	294	34,000	425
Slurry Trench	sy	615	6,800	4,182	7,400	4,551	6,800	4,182
Rock Surface Treatment	sy	15.80	54,300	858	52,300	826	54,300	858
SUBTOTAL SADDLE DAM				19,936		18,552		19,936
TOTAL NON-COMMON ITEMS				123,956		129,139		165,295

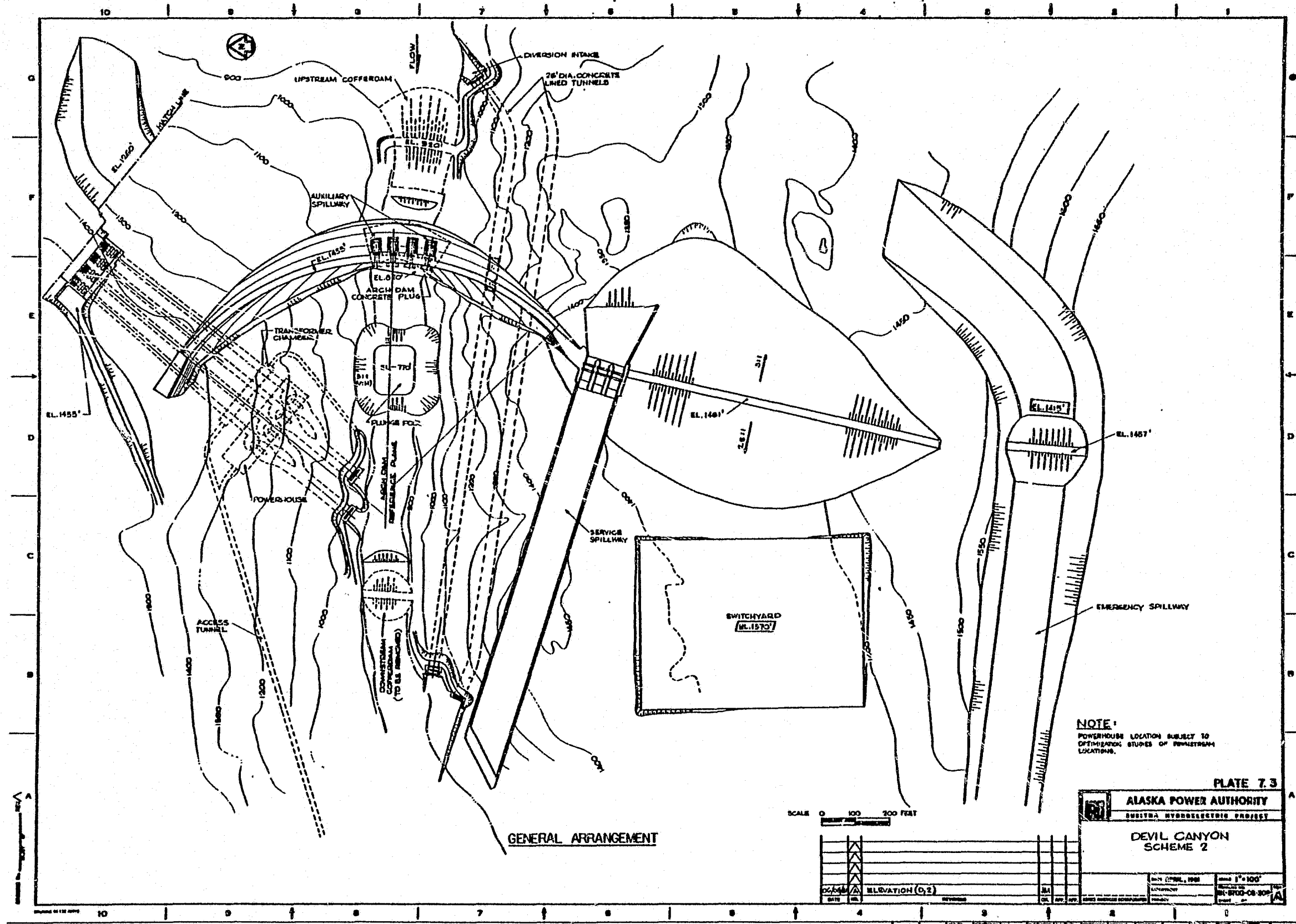
ESTIMATE SUMMARY - DEVIL CANYON
COMPARATIVE COST ESTIMATE - JULY 1981
(\$000,000 JANUARY 1982 DOLLARS)

Page 4 of 4

Description	Scheme 1	Scheme 2	Scheme 3
NON-COMMON ITEMS	124	129	165
COMMON ITEMS	<u>758</u>	<u>758</u>	<u>758</u>
SUBTOTAL	882	887	923
Camp & Support Costs (16%)	<u>141</u>	<u>142</u>	<u>148</u>
SUBTOTAL	1,023	1,029	1,071
Contingency 20%	<u>204</u>	<u>206</u>	<u>214</u>
SUBTOTAL	1,227	1,235	1,285
Engr/Admin 12.5%	<u>153</u>	<u>154</u>	<u>161</u>
TOTAL	<u>1,380</u>	<u>1,389</u>	<u>1,446</u>







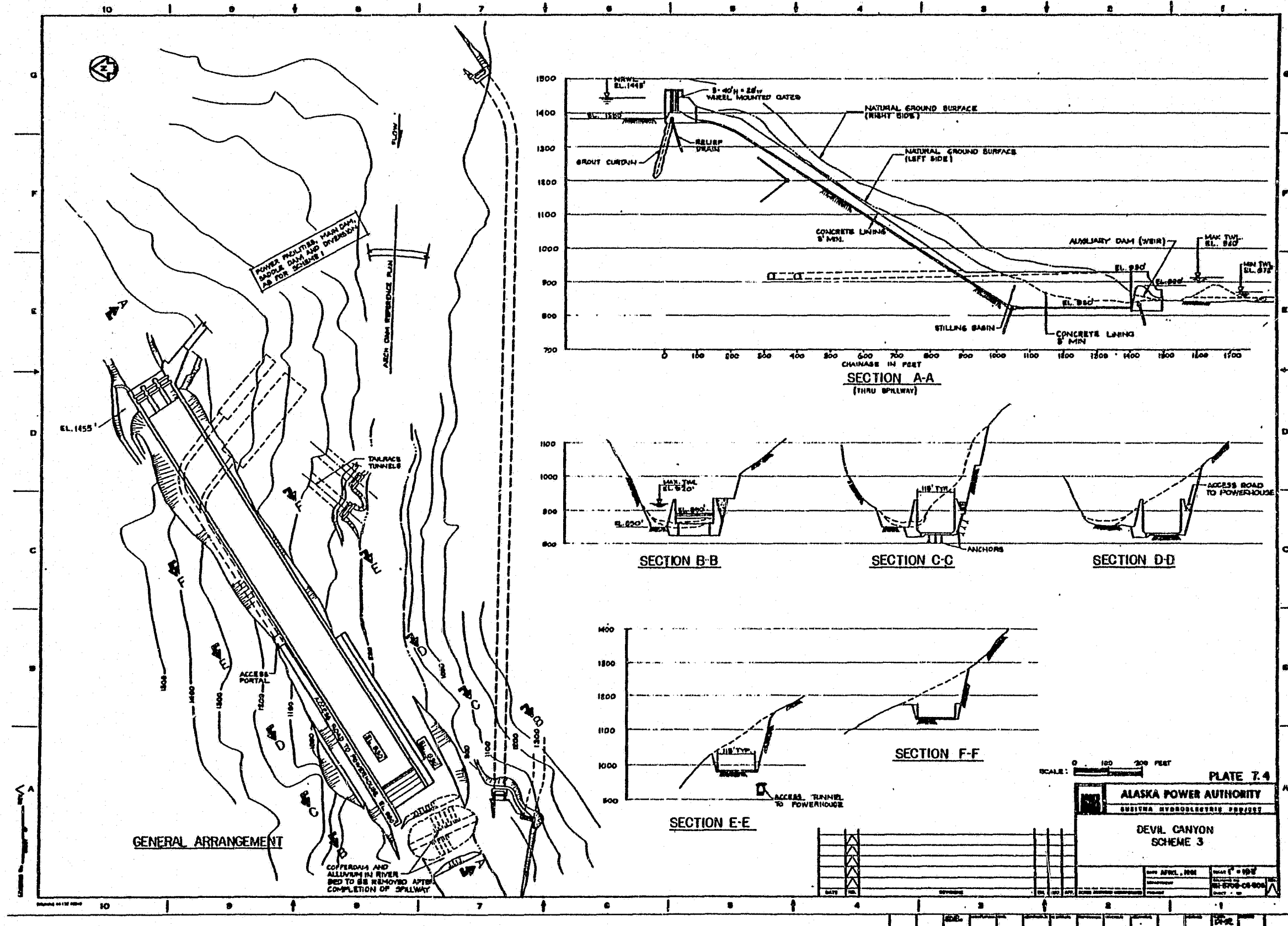
NOTE:
POWERHOUSE LOCATION SUBJECT TO
OPTIMIZATION STUDIES OF PENSTOCK
LOCATIONS.

PLATE 7.3

ALASKA POWER AUTHORITY	
SUBSIDIA HYDROELECTRIC PROJECT	
DEVIL CANYON SCHEME 2	
DATE: 12/1/58	SCALE: 1"=100'
BY: J. H. BROWN	CHECKED: J. H. BROWN
APPROVED: J. H. BROWN	REVISION: 1

SCALE 0 100 200 FEET

DATE	BY	REVISION	ELEVATION (D. 2)
12/1/58	J. H. BROWN	1	



3 - COMPARISON OF LAYOUTS

As the arch dam, saddle dam, power facilities and diversion vary only in a minor degree between the alternatives, a comparison of schemes rests solely with a comparison of the spillway facilities.

As can be seen from a comparison of costs in Table 1, the flip-bucket spillways are substantially cheaper to construct than the stilling-basin type of Scheme 3. The left side spillway of Scheme 2 runs at a sharp angle to the river eject the discharge jet from high on the canyon face towards the opposite side of the canyon. Over a long period of operation, scour of the heavily jointed rock could be a considerable problem causing undermining of the canyon sides and their consequent instability together with the possibility of a build-up of material downstream with a corresponding elevation of the tailrace.

Construction of a spillway on the steep left side of the river could be more difficult than on the right side because of the presence of deep fissures and large unstable blocks of rock close to the top of the canyon.

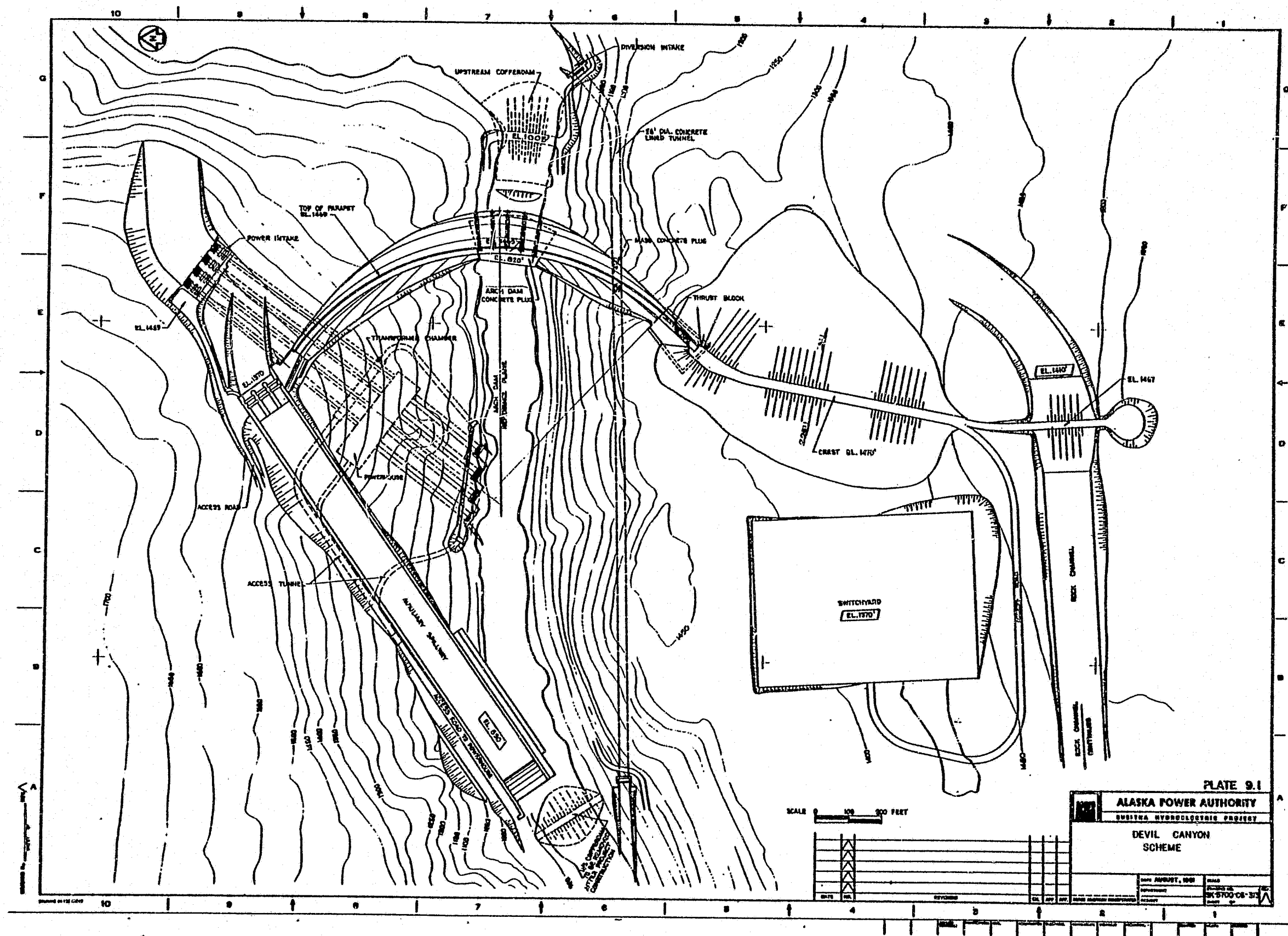
The right side flip-bucket spillway takes advantage of a downstream bend in the river to eject discharges parallel to the course of the river. This will reduce the effects of erosion, but it could still be a serious problem.

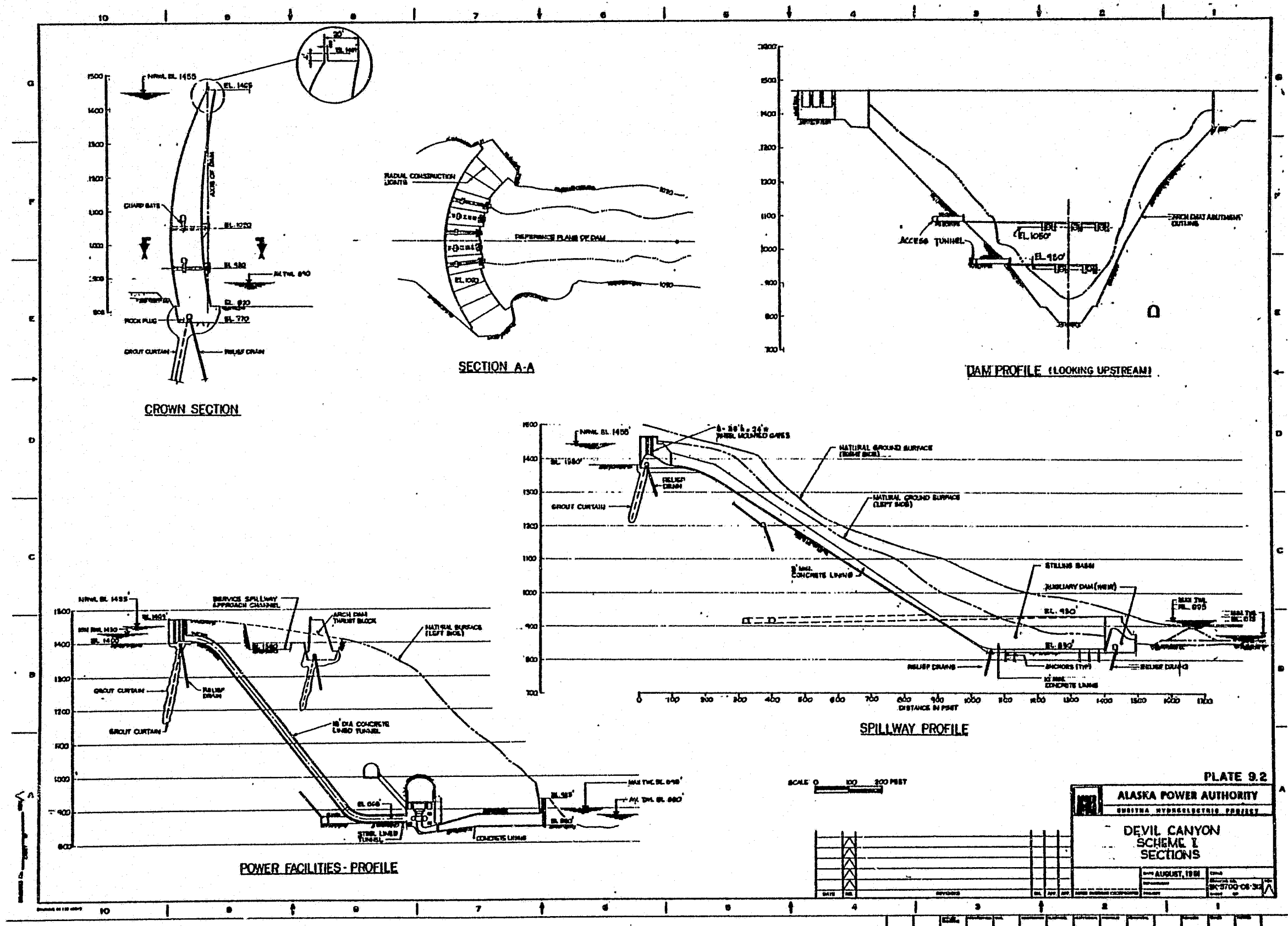
The safest spillway would be the stilling-basin type of Scheme 3 which would greatly reduce any erosion problems within the canyon. Cavitation could be a problem under the high flow velocities experienced at the base of the chute but this would be alleviated by aeration of the flows.

4 - SELECTED LAYOUT

The chute and flip-bucket spillways of Schemes 1 and 2 pose large erosion problems which will entail considerable maintenance costs and reduced efficiency in operation of the project at a future date. The anticipated maximum scour will also be unacceptable environmentally. The additional cost of a stilling basin over the right side flip-bucket spillway is \$62 million including contingencies, engineering, and administration. This appears a relatively small cost for the additional security of this type of spillway, and Scheme 3 was therefore selected as the most favorable layout for additional study.

Further development of the Scheme 3 layout has resulted in the currently preferred arrangement shown in Plates 9.1 and 9.2.





DEVIL CANYON ARCH DAM DESIGN

ABSTRACT FROM

DEVIL CANYON ARCH DAM DESIGN REPORT

4 - DESIGN CRITERIA

4.1 - Material Properties

a) Concrete

Frost Resistance Concrete Strength (365 day)	5,000 psi
Unit Weight	150 lb/ft ³
Static Modulus of Elasticity (sustained)	3×10^6 psi
Dynamic Modulus of Elasticity (instantaneous)	5×10^6 psi
Poissons Ratio	0.2
Tensile Strength:	
Static (for estimating cracking only) 5% of strength	250 psi
Dynamic Flexural 15% of strength	750 psi
Thermal Properties:	
Conductivity	1.52 BTU/ft/hr/°F
Specific Heat	0.22 BTU/lb/°F
Coefficient of Thermal Expansion	5.6×10^{-6} ft/ft/°F
Diffusivity	0.046 ft ² /hr

b) Foundation Rock

Deformation Modulus (sustained)	2×10^6 psi
Poissons Ratio	0.2

4.2 - Temperatures (°F)

Air Temperature:

Mean Annual	28.9
High Mean Monthly	55.0
Low Mean Monthly	4.4
Highest Mean Monthly Maximum	63.8
Lowest Mean Monthly Minimum	-3.6

Highest Maximum	91.0
Lowest Minimum	-48.0
Lowest Difference Between Any Mean Monthly Maximum and the Corresponding Mean Monthly Minimum	-14.5

RESERVOIR WATER TEMPERATURE

Depth Below Surface (ft)	M O N T H											
	4	5	6	7	8	9	10	11	12	1	2	3
0 - 50	32	32	46	57	53	45	39	32	32	32	32	32
70 to Reser- voir Bottom	39	39	39	39	39	39	39	39	39	39	39	39

The effect of solar radiation has been at this stage neglected.

Grouting temperature of vertical construction joints: 39°F

4.3 - Earthquake

For maximum credible earthquake conditions two versions of the mean response spectra for the Penioff zone, developed by Woodward Clyde Consultants have been used.

Peak Ground Acceleration	Damping Factor
0.5 g	5%
0.4 g	10%

4.4 - Hydraulic Data

Reservoir Water Levels:

Normal Maximum	1,455 ft
*Normal Minimum	1,430 ft
1:10,000 Yr Flood Level	1,460 ft
Probable Maximum Flood	1,465 ft

Effect of tailwater, silt deposits, ice load, and uplift loads (internal pressure within the dam) have been neglected.

*This was assumed as 1,295 ft for stress calculations. However, minimum operating level has now been maintained at 1,430 ft from standpoint of firm energy considerations. Hence, this condition will be far less extreme.

4.5 - Loading Combinations:

- a) Usual Loading Combination - Combination of basic loads that can simultaneously occur during time design life of the dam (self-weight, temperature and hydrostatic load condition.)
- b) Unusual Loading Combination - Combination of loads that are possible, but which are unlikely to occur during the design life of the dam (probable maximum flood conditions.)
- c) Extreme Loading Combination - Are related to earthquakes.

The loading combinations cases are given in Table 4.1.

4.6 - Factors of Safety:

a) Usual Loading Case UL-1, UL-2

- Compressive stresses - F.O.S. ≥ 4
- Tension stresses - not allowable.*

b) Usual Loading Case UL-3, UL-4, and Unusual Case

- Compressive stresses - F.O.S. ≥ 3
- Tension stresses not to exceed 250 psi.
- Tensile stresses above 250 psi are to be redistributed to other resistance mechanisms by local joint openings.

*These factors of safety correspond to the trial load method and are in line with the previous practice. They do not necessarily apply to other methods of analysis.

c) Extreme Loading Case EL-1, EL-2

- Compressive stresses - F.O.S. ≥ 1 .
- Tension stresses exceeding the tensile strength of 750 psi are to be redistributed to other resistance mechanisms.

In case of horizontal tensile stresses across the arches the dam should be considered as a set of unrestrained cantilevers 50 percent of full height, because of opening vertical construction joints.

TABLE 4.1

Load Combination			U S U A L				Unusual	Extreme	
Combination Class			UL-1	UL-2	UL-3	UL-4	UNL-1	EL-1	EL-2
Combination Number									
B A S I C L O A D C A S E S	S T A T I C L O A D S	D E A D L O A D		X	X	X	X	X	X
		Air and Reservoir Water Temperatures	February		X				
			April			X			
		Reservoir Water Levels	1,455	X	X			X	X
			1,465				X		
			1,295		X				
			1,430		X	X			
	D Y N A M I C L O A D S	Maximum Credible Earthquake	0.5 G 5% Damp.					X	
			0.4G 10 Damp.						X

5 - METHOD OF ANALYSIS

5.1 - General

The Arch Dam Stress Analysis System (ADSAS) program which is a computerized version of the trial load method, is used for static and earthquake dynamic analysis. In the analysis the arch dam is assumed to be a continuous structure. The dead load is applied in the cantilever direction (construction joints grouted at full height of sections).

The computer program SAP IV is used for the unrestrained crown cantilever analysis in cases where the dam is subjected to strong earthquake motions, causing opening of the upper part of vertical construction joints.

5.2 - Method of Definition of Loads

a) Temperature Load

The two-dimensional heat transfer program (heatflow) is used for determination of temperature distribution in the dam body.

The USBR Engineering Monography N34 is used for computation of the amplitude of the sinusoidal cycles,

(Annual, 15-Day and Daily). The temperature loads input into ADSAS are presented in Appendix A.

b) Hydrodynamic Load

The hydrodynamic pressure due to horizontal earthquake on the dam upstream face ("added mass") is defined by using Westergaards Formula

$$P_z = 7/8 \ a_n \ \gamma \ \sqrt{H_{1z}}$$

and is reduced to 60%, due to the effect of narrowness of the gorge, inclination of the dam face and water compressibility (see Appendix C.)

6 - ARCH DAM GEOMETRY

The arch dam abutments are founded on the sound bedrock of the canyon. The sound unweathered rock is determined as generally 40 feet below the bedrock surface and the foundation is trimmed so as not to cause an abrupt change in the dam profile and hence a concentration of stresses.

At the bottom of the valley, the dam sits on a massive concrete plug which can adjust to any disconformities of the bedrock at the valley floor without changing the geometry of the dam. Sound bedrock does not continue above approximately elevation 1350 feet on the left bank and a massive thrust block is constructed to take the thrust of the upper 100 feet of arches. A similar block is founded deep in the rock on the right side in order to preserve the symmetry of the dam profile.

The dam geometry is shown on Plate 6.2. It is a double curvature structure with the cupola shape of the crown cantilever defined by vertical curves of approximately 1352 feet and 869 foot radius. The horizontal arches are prescribed by varying radii moving along two pairs of center lines. The shorter radii of the intrados face cause a broadening of the arches at the abutment reducing the contact stresses. The dam reference plane is approximately central to the bottom of the valley and the two center configuration assign longer radii to the arches on the wider side of the valley thus providing comparable contact areas on both sides of the arches at the concrete/rock interface. The longer radii will also allow the thrust from the arches to be directed more into the abutment rather than parallel to the river. The net effect of this two center layout will be to improve the symmetry of the stresses right across the dam.

The crown cantilever is 635 feet high. It is 20 feet thick at the crest and 90 feet thick at the base. The bottom mass concrete plug is 50 feet high. The slenderness coefficient of the arch is equal to $90/635 = 0,142$ and the radii of the dam axis at crest level are 710 feet and 780 feet for the left and right angles of the dam, respectively. The central angles vary between 51.5 DEG at El. 1300 and 25 DEG at the base for the left side of the arch dam and 58 DEG to 30 DEG for the right side. The ratio of crest length to height for the dam is $1260:635 = 1.98:1$ (thrust blocks not included).

The left bank thrust block is 105 feet high and 170 feet long at the base. The right bank thrust block has a maximum height of 100 feet and a length of 155 feet and is adjacent to the spillway control structure, which will behave as a continuation of the thrust block, transferring the thrust directly into the rock.

7 - STATIC LOAD CONDITIONS

7.1 - Dead Load

In all analyses, the vertical construction joints within the dam are assumed to be ungrouted and hence the weight of the dam is considered as confined within the cantilevers, with no distribution through the arches, and directed vertically downwards into the foundation.

7.2 - Hydrostatic

Hydrostatic loadings induced by the reservoir at specified levels were considered in all load combinations. The effect of tailwater and uplift pressures will have little effect on the overall stresses and are not considered at this time.

7.3 - Temperature

(a) Solar Radiation

The dam orientation, running north-south, and the narrow valley will cause solar radiation to have only minor effects on concrete temperatures and hence stresses from radiation will be neglected at this time.

(b) Air Temperatures

Because of absence of temperature records, temperatures at the Devil Canyon site have been interpolated from records taken at two stations: Summit (El. 2405 feet) and Talkeetna (El. 345 feet). The stations are equidistant from Watana and their average altitude is similar to river level at Watana. The temperatures from the two stations were averaged to obtain the following temperatures at the dam site:

AMBIENT AIR TEMPERATURE (°F)

Mean Annual	28.9
High Mean Monthly	55.0
Low Mean Monthly	4.4
Highest Mean Monthly Maximum	63.8
Lowest Mean Monthly Minimum	-3.6
Highest Maximum	91.0
Lowest Minimum	-48.0
Lowest Difference between any Mean Monthly Maximum and the Corresponding Mean Monthly Minimum	14.5

Three sinusoidal temperature cycles - annual, 15-day and daily are developed based on USBR ENG MONOGRAPH No. 34.

The temperatures obtained are as follows:

	EXTREME CONDITIONS		USUAL CONDITIONS	
	Above Mean (DEGF)	Below Mean (DEGF)	Above Mean (DEGF)	Below Mean (DEGF)
Annual	26.1	24.5	26.1	24.5
15-day	28.8	42.15	15.15	22.95
Daily	7.25	7.25	7.25	7.25

(c) Reservoir Water Temperature

Average monthly reservoir temperatures are given below. Temperatures throughout the top 50 feet are as shown and below 50 feet they vary linearly to 39°F at a depth of 70 feet.

<u>Month</u>	<u>Top 50 ft (°F)</u>	<u>Below 70 ft From Surface (°F)</u>
April	32	39
May	32	
June	46	
July	57	
August	53	
September	45	
October	39	
November	32	
December	32	
January	32	
February	32	
March	32	39

(d) Grouting Temperature

On account of the cold climate and the possibility of freezing, grouting temperature was selected at 39°F, as low as considered practicable, in order to reduce tension in the dam induced by shrinkage at lower temperatures.

(e) Temperature Distribution

The temperature distribution in the dam body was determined using the two dimensional heat transfer program "HEATFLOW" obtained from the U.S. Department of the Interior (formerly USBR) and was input as a uniform temperature combined with a linear distribution as described in Appendix A.

7.4 - Load Combinations

Static analyses were performed for the following normal loading combinations:

UL-1 - Hydrostatic and dead loads at normal reservoir level 1445 feet

UL-2 - Hydrostatic and dead loads at maximum drawdown reservoir level 1295 feet

UL-3 - The same as UL-1 plus temperature (February)

UL-4 - The same as UL-2 plus temperature (April)

UL-1 and UL-2 Conditions

The cantilever and arch stresses along the face of the dam are shown in Figures B7-1 to B7-4 in Appendix B. In both the arch and cantilever directions, the entire structure is in compression and below the allowable stress of 1250 psi, except for a few isolated areas where small tensile stresses occur. Maximum (compression) and minimum (tension) stress for conditions UL-1 and UL-2 are shown in Table 7.1.

The arch and cantilever stresses for loading combinations UL-3 and UL-4 are shown in Figures 7.5 to 7.12.

The maximum and minimum stresses along the rock/concrete interface and in the dam above the foundation are given in Table 7.2.

7.5 - Conclusion

- (1) Under hydrostatic loading, minor isolated tensile stresses occur up to a maximum of 97 psi.
- (2) In both cases with temperature loadings UL-3 and UL-4, the compressive stresses are below the allowable limit.

- (3) In UL-3 case, tensile stresses are acting in isolated areas. The tensile stresses is possible to eliminate by refining the shape of the arch.
- (4) In UL-4 case, the crest of the dam is in the arch direction subjected to almost axial tension. Tensile stresses up to 200 psi are found at the whole height of the crown downstream face.

Prevention of these tensile stresses is possible only by application of special measures such as:

- Low closure temperatures at the upper part of the arch which may be obtained by using closure slots between adjacent blocks filled up with concrete in spring time when the blocks are at minimum temperature.
- Thermal insulation of the downstream face.
- Prestressing the upper part of the dam by means of flat jacks.

TABLE 7.1
EXTREME MAGNITUDES OF STRESSES
AT ROCK/CONCRETE INTERFACE

Loading Combination (stresses in psi)

	UL-1	UL-2
Arch		
Max	792 (D. E1 1100)	432 (U E1 900)
Min	23 (U. E1 1000)	3 (U E1 1000)
Cantilever		
Max	722 (D. E1 820)	760 (U E1 900)
Min	-27 (D E1 1370)	-97 (D E1 1200)
Principal		
Max	1049 (D. 1000)	
Min	-140 (D. 900)	

- indicates tension
D indicates downstream face
U indicates upstream face

MAXIMUM STRESSES IN DAM ABOVE FOUNDATION

	UL-1	UL-2
Arch		
Max	958 (U E1 1100)	548 (U E1 1000)
Min	182 (D E1 1000)	-36 (D E1 1370)
Cantilever		
Max	575 (D E1 1000)	542 (U E1 1000)
Min	0 (D E1 1370)	-44 (U E1 1295)

TABLE 7.2

EXTREME MAGNITUDES OF STRESS

ALONG ROCK/CONCRETE INTERFACE

Loading Combination

	UL-3 (point)	UL-4 (point)
Arch		
Max	747 (U E1 900)	381 (D 1100)
Min	-182 (U E1 1455)	-157 (D 900)
Cantilever		
Max	689 (D E1 820)	804 (U E1 900)
Min	-393 (D E1 1370)	-281 (D E1 1455)

EXTREME MAGNITUDES OF STRESSES

IN DAM ABOVE FOUNDATION

Loading Combination

	UL-3 (point)	UL-4 (point)
Arch		
Max	1180 (U E1 1200)	717 (U E1 1100)
Min	-134 (D E1 1000) (U E1 1455)	-268 (U E1 1455)
Cantilever		
Max	515 (U E1 900)	608 (U E1 1000)
Min	-75 (D E1 1370)	-62 (U E1 1295)

8 - DYNAMIC ANALYSES

Preliminary assumptions for purposes of analysis are as follows:

The assumed response spectra input to ADSAS is from Figure 3-4 of the Woodward Clyde Draft Report "Preliminary Earthquake Ground Motion Studies for the Proposed Susitna Hydroelectric Project". The mean response spectra for the Benioff zone is scaled up to 0.5g peak from 0.37g. The damping ratio is five percent. The response spectrum is shown in Figure 8.1. The response spectrum analysis was initially attempted for 1 to 20 modes. A larger displacement mode was encountered on mode 19. The high displacement induced unreasonable stresses in the dam and therefore made the results useless. The problem was re-analyzed using 14 modes of vibration.

The response spectrum analysis assumed an instantaneous concrete modulus of 5,000,000 psi.

The results of positive and negative earthquake are presented in the following tables. The load combinations are hydrostatic and gravity $\pm$ earthquake and hydrostatic + gravity + uniform and linear temperature $\pm$ earthquake:

Table 8.1 - Response Spectrum Analysis - Arch Stresses

Table 8.2 - Crown Cantilever Stresses

The resultant tensile stresses of 2580 psi and 729 psi in the arch and cantilevers, respectively, are greatly in excess of the allowable tensile stress of 500 psi.

The results of a dynamic analysis of Devil Canyon Arch Dam based on a 0.4g peak ground acceleration, 10% damping, the Woodward Clyde Consultants response spectrum (see Figures 8.2) and using the ADSAS program are shown on Figures B.15 and B.16. For comparison, the results of dynamic analyses for a peak ground acceleration 0.5g and 5% damping are presented on Figures B.13 and B.14

The change of earthquake parameters to 0.4g and 10% damping has reduced the compressive, tensile and shear stresses at all points on the dam faces by a factor of 1.58 compared to the 0.5g acceleration and 5% damping case.

The case of upstream ground movement (hydrostatic, gravity and earthquake loads), the maximum cantilever tensile stress at the upstream face dropped from 729 psi to 427 psi (at elevation 1285 feet on the crown cantilever). The maximum compressive arch stresses at the upstream face (crown El. 1370) dropped from 3657 psi to 2551 psi. Stresses on the downstream face are much lower than on the upstream.

Downstream ground movement (hydrostatic and gravity minus earthquake load) shows extremely high tensile stresses across the arches (see Table 8.3). The stresses computed are not realistic. As discovered by field observations and model tests on other projects, earthquake induced ground movement in the downstream direction causes the radial construction joints at the upper part of an arch dam to open. The tension induced in the upper part of these arches is relaxed and the dam evolves into a set of independent, unrestrained cantilevers, deflecting freely in the upstream direction.

In order to accord more closely with the actual behavior of the Devil Canyon Arch Dam, when subjected to strong earthquake motions, dynamic analyses on the unrestrained crown cantilever were performed using the computer program SAPIV.

Model test on other arch dams with simulated radial construction joints, performed by "ISMES" have shown that opening of the joints take place over the top 1/3 to 1/2 (depending on the narrowness at the gorge) of the dam, while the lower part remained intact.

The analyses are based on:

- (1) The Woodward Clyde Consultants response spectrum curves for the Benioff zone with peak ground accelerations of 0.5g and 0.4g and damping rates of 5% and 10%.

The change of earthquake parameters to 0.4g and 10% damping has reduced the compressive, tensile and shear stresses at all points on the dam faces by a factor of 1.58 compared to the 0.5g acceleration and 5% damping case.

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Model test on other arch dams with simulated radial construction joints, performed by "ISMES" have shown that opening of the joints take place over the top 1/3 to 1/2 (depending on the narrowness at the gorge) of the dam, while the lower part remained intact.

The analyses are based on:

- (1) The Woodward Clyde Consultants response spectrum curves for the Benioff zone with peak ground accelerations of 0.5g and 0.4g and damping rates of 5% and 10%.

- (2) The hydrodynamic stress distribution as proposed by Westerguard approach and reduced to 60% due to the effect of narrowness of the gorge, inclination of the dam upstream face and water compressibility (see Appendix C).
- (3) Full reservoir water level 1445 feet computer program for dynamic analysis has been used.

The following combinations of earthquake parameters have been examined:

<u>Peak Ground Acceleration "G"</u>	<u>Damping Ratio (Percent)</u>	<u>Added Mass (Percent)</u>
0.5	5	100
		60
	10	100
		60
0.4	5	100
		60
	10	100
		60

The results of the cantilever dynamic analysis are as follows:

- (1) The natural period of vibration "T" is 0.62 sec, 0.15 sec and 0.09 sec. (Various magnitudes of acceleration and added mass have little effect). For comparison, a full height cantilever, which is slender, has been computed. The periods were found 2.42 sec, 0.49 sec and 0.20 sec. The stresses in the upper part of the arch in this case were smaller than in the short cantilever.

- (2) The stresses due to hydrostatic and gravity and dynamic loads are presented separately and in combinations. In Tables 8.4 and 8.5 and in Figure 4, maximum tensile stresses of 880 psi at the downstream face were obtained in the case of 0.5g, 5% damping and full Westergaard's added mass at 170 feet below crest level. Compressive stresses at the upstream face at that level are 1100 psi. The maximum tensile stresses in case of 0.5g, 10% damping and 60% of Westergaard's added mass are equal to 451 psi.

The change of damping from 5% to 10% decreases the maximum tensile stresses approximately 1.6 times. The application of 60% added mass instead of full Westergaard's provides a reduction of the maximum tensile stresses of about 25%.

In all combinations of dynamic loads considered, the tensile stresses at the base of cantilever have changed to compressive (except of case 0.5g, 5% damping and full added mass, where tension is reduced to 55 psi) (Figure 8.4).

In the case of 0.4g, ground acceleration, the maximum tensile stresses at the downstream face of cantilever dropped to 509 psi 120 feet below the crest with 5% damping and full added mass, and to 272 psi with 10% damping and 60% added mass.

The effects of the change in damping and added mass are approximately the same as in the case of 0.5g acceleration.

TABLE 8.1

RESPONSE SPECTRUM ANALYSIS

i) Arch at Elv. 1455'

STATION	FACE	HYDRO + GRAVITY	HYDRO + GRAVITY + EQ	HYDRO + GRAVITY-EQ	HYDRO + GRV + TEMP + EQ	HYDRO + GRV + TEMP - EQ
1000	E	467	3404	-2470	3294	-2580
	I	313	1943	-1630	1784	-1476
1143	E	516	3229	-2197	3122	-2304
	I	307	2304	-1690	2146	-1848
1259	E	484	2948	-1980	2843	-2085
	I	366	2749	-2017	2611	-2155
1393	E	406	2498	-1686	2383	-1801
	I	438	3019	-2143	2896	-2266
1526	E	324	2033	-1385	1877	-1541
	I	417	2566	-1732	2376	-1922
1638	E	303	1591	- 985	1356	-1220
	I	342	2232	-1548	1962	-1818
1711	E	274	2513	-1965	2105	-2373
	I	576	2409	-1257	2465	-1201
1714	E	267	2574	-2040	2125	-2409
	I	607	2478	-1267	2596	-1146

TABLE 8.1
RESPONSE SPECTRUM ANALYSIS

ii) Arch at Elev. 1370'

STATION	FACE	HYDRO + GRAVITY	HYDRO + GRV + EQ	HYDRO + GRV - EQ	HYDRO + GRV + TEMP + EQ	HYDRO + GRV + TEMP - EQ
1000	E	642	3657	-2373	4119	-1911
	I	255	949	-439	697	-691
1143	E	707	3222	-1808	3677	-1353
	I	258	1597	-1081	1355	-1323
1259	E	593	2461	-1275	2884	-852
	I	396	2247	-1455	2021	-1681
1393	E	416	1634	-802	2006	-430
	I	518	2558	-1522	2323	-1757
1526	E	295	1188	-598	1511	-275
	I	498	2383	-1387	2111	-1659
1638	E	206	1071	-659	1297	-433
	I	413	1979	-1153	1733	-1399
1711	E	110	1220	-1000	1266	-954
	I	374	1449	-701	1345	-805

TABLE 8.2

RESPONSE SPECTRUM ANALYSIS

CROWN CANTILEVER

ELEV.	FACE	HYDRO + GRAVITY	HYDRO + GRAVITY + EQ	HYDRO + GRAVITY-EQ	HYDRO + GRV + TEMP + EQ	HYDRO + GRV + TEMP - EQ
1455	U	0	0	0	0	0
	D	0	0	0	0	0
1370	U	109	-581	799	-564	816
	D	56	653	-561	658	-576
1285	U	98	-729	925	-655	999
	D	222	1021	-577	950	-648
1200	U	71	-629	771	-508	892
	D	402	1111	-307	988	-430
1100	U	102	-435	639	-282	792
	D	544	1110	-22	948	-184
1000	U	223	-142	638	-31	799
	D	575	1026	124	851	-51
900	U	383	-19	785	113	917
	D	539	988	90	842	-56
820	U	305	-402	1012	-373	1041
	D	722	1541	-97	1508	-130

TABLE 8.3

EARTHQUAKE	ELEVATION OF ARCH (FT.)	FACE OF DAM	CROWN 0	DISTANCE ALONG THE ARCH (FT)			ABUTMENT
				143	394	638	714
0.5G	1455	U	-2470	-2197	-1686	-985	-2040
		D	-1630	-1690	-2143	-1548	-1267
5% Damp.	1370	U	-2373	-1808	-803	-659	-1000
		D	-439	-1081	-1522	-1153	-701
0.4G	1455	U	-1392	-1203	-919	-512	-1149
		D	-720	-957	-1196	-855	-757
10% Damp.	1370	U	-1267	-887	-355	-341	-592
		D	-185	-589	-774	-578	-306

Table 8.4

DEVIL CANYON ARCH DAM RESULTS OF SINGLE CANTILEVER DYNAMIC
ANALYSIS FOR 0.5 G PEAK GROUND ACCELERATION

Node	Elevation (ft)	Stresses Due to Static Loads (Hydrostatic & Gravity) (psi)		Stresses Due to Earthquake Loads (Concrete Inertia & Added Mass of Water - Wester Gaard)				Resultant Stresses at Downstream Face (psi)				Resultant Stresses at Upstream Face (psi)			
		Upstream Face	Downstream Face	100% Added Mass		60% Added Mass		100% Added Mass		60% Added Mass		100% Added Mass		60% Added Mass	
				5% Damp	10% Damp	5% Damp	10% Damp	5% Damp	10% Damp	5% Damp	10% Damp	5% Damp	10% Damp	5% Damp	10% Damp
2	1428	-5	29	+ 180	+ 144	+ 177	+ 141	-151	-114	-148	-112	175	139	172	136
3	1375	-72	148	+ 760	+ 604	+ 685	+ 546	-612	-456	-537	-398	608	532	613	474
4	1322	-265	405	+1210	+ 960	+1075	+ 856	-805	-555*	-670	-451*	945	695°	810	591°
5	1269	-530	750	+1630	+1300	+1450	+1150	-880*	-550	-700*	-400	1100°	770	920°	620
6	1216	-930	1230	+2060	+1635	+1823	+1440	-830	-405	-593	-210	1135	705	893	510
7	1163	-1495	1885	+2620	+2081	+2300	+1825	-735	-196	-415	60	1125	536	805	330
8	1110	-2295	2785	+2840	+2255	+2490	+1570	-55	530	205	315	545	-70	195	-325

- Notes: 1. Resultant stresses are computed for dynamic loads applied upstream
2. "*" indicates maximum tensile stresses; "°" indicates corresponding compressive stresses at the opposite side of the same level.
3. (-) indicates tension.

Table 8.5

DEVIL CANYON ARCH DAM RESULTS OF SINGLE CANTILEVER DYNAMIC
ANALYSIS FOR 0.4 G PEAK GROUND ACCELERATION

Node	Elevation (ft)	Stresses Due to Static Loads (Hydrostatic & Gravity) (psi)		Stresses Due to Earthquake Loads (Concrete Inertia & Added Mass of Water - Wester Gaard)				Resultant Stresses at Downstream Face (psi)				Resultant Stresses at Upstream Face (psi)			
				100% Added Mass		60% Added Mass		100% Added Mass		60% Added Mass		100% Added Mass		60% Added Mass	
		Upstream Face	Downstream Face	5% Damp	10% Damp	5% Damp	10% Damp	5% Damp	10% Damp	5% Damp	10% Damp	5% Damp	10% Damp	5% Damp	10% Damp
2	1425	-5	29	+ 143	+ 114	+ 140	+ 112	-114	-85	-111	-83	138	109	135	107
3	1375	-72	148	+ 577	+ 459	+ 528	+ 420	-429	-311	-380	-272*	505	382	456	348°
4	1322	-265	405	+ 914	+ 727	+ 826	+ 656	-509*	-322*	-421*	-251	649°	462°	561°	391
5	1269	-530	750	+1233	+ 977	+1110	+ 885	-483	-227	-360	-135	703	447	580	355
6	1216	-930	1230	+1547	+1230	+1394	+1105	-317	0	-164	125	617	300	464	175
7	1163	-1495	1885	+1971	+1560	+1750	+1390	-86	325	135	495	476	65	255	-105
8	1110	-2295	2795	+2130	+1692	+1897	+1504	655	1093	888	1381	-165	-603	-398	-791

- Notes: 1. Resultant stresses are computed for dynamic loads applied upstream
2. "*" indicates maximum tensile stresses; "°" indicates corresponding compressive stresses at the opposite side of the same level.
3. (-) indicates tension.

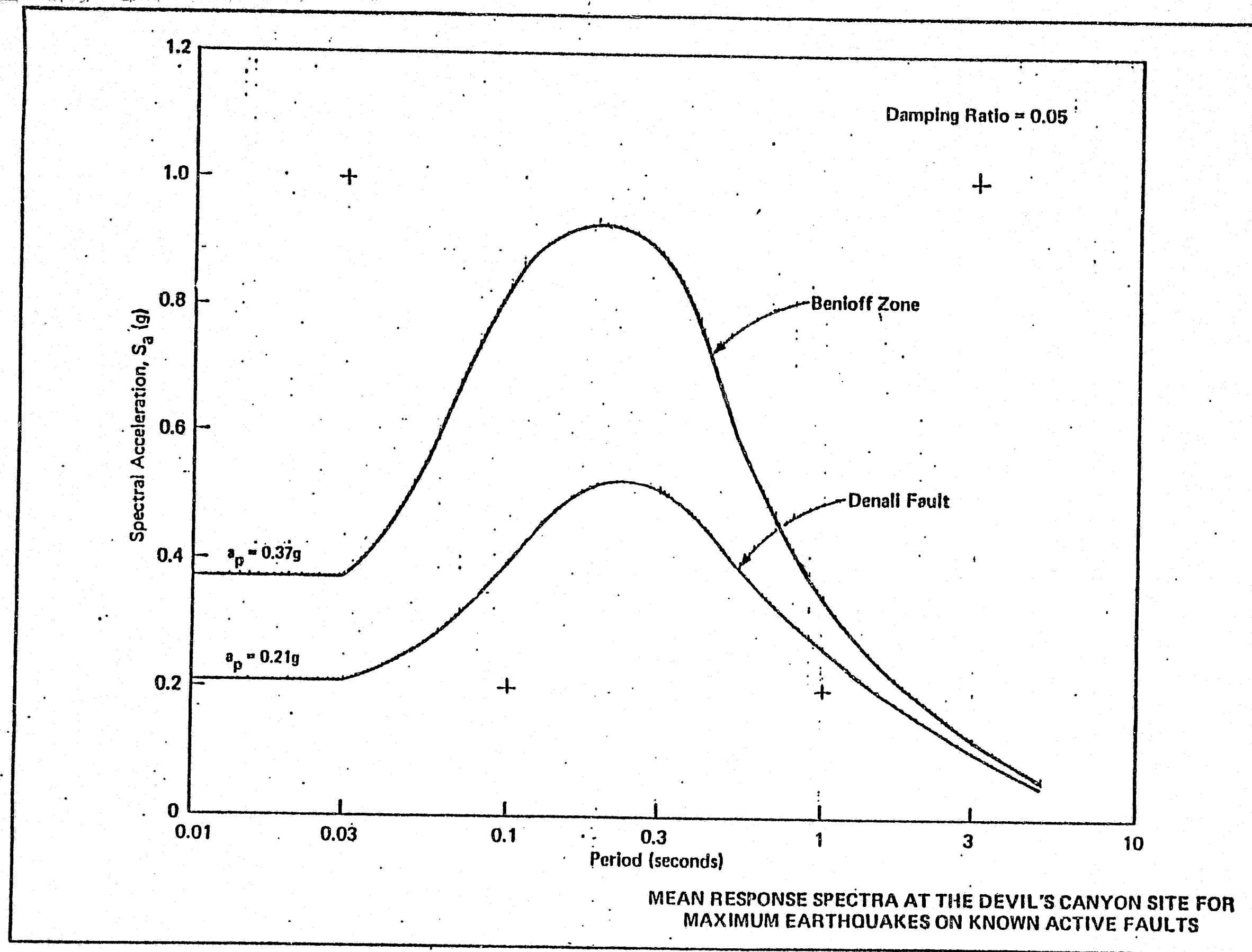
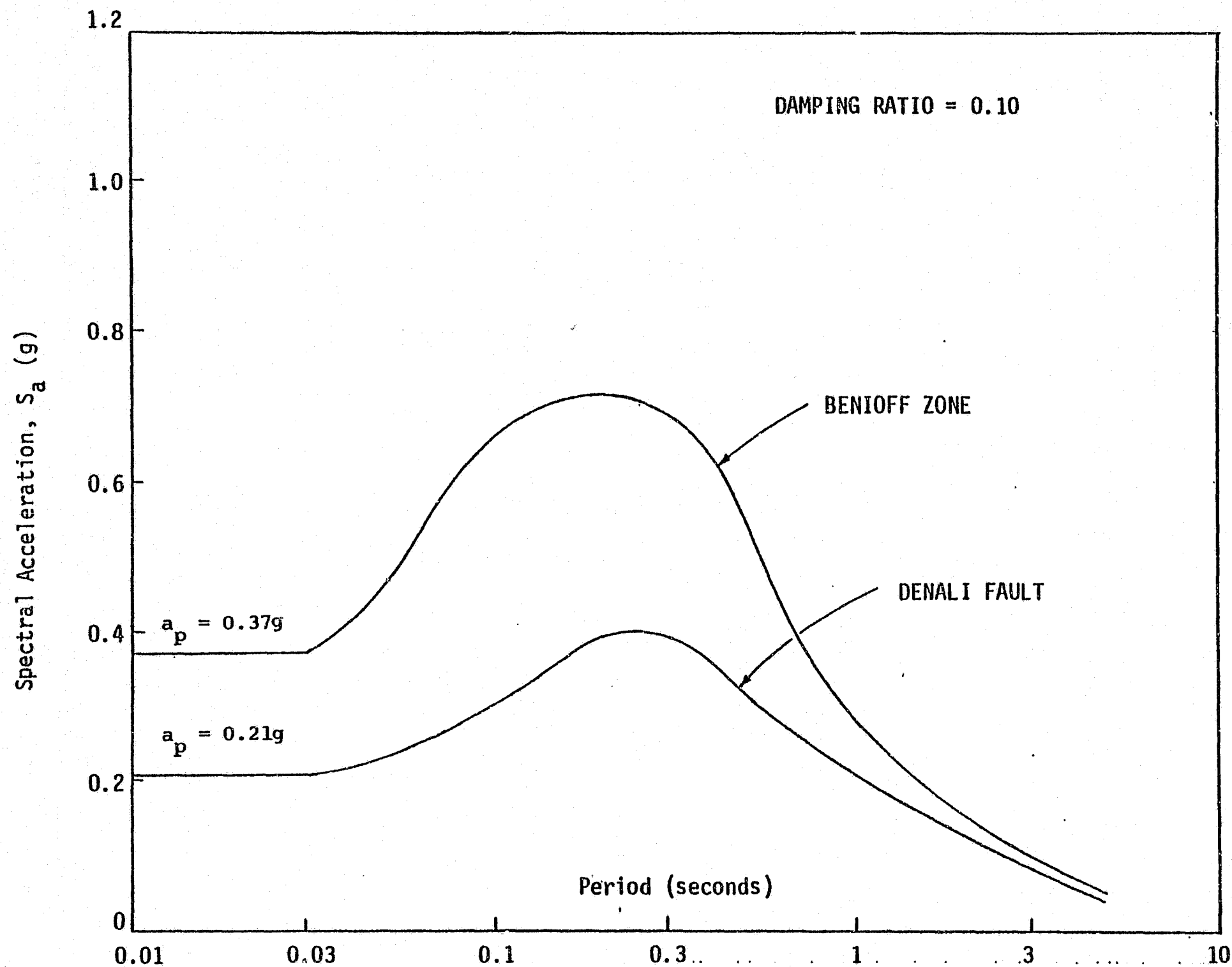


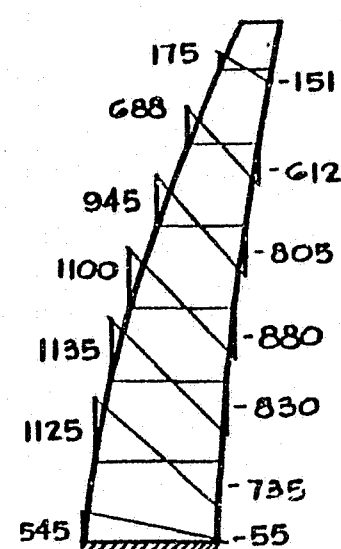
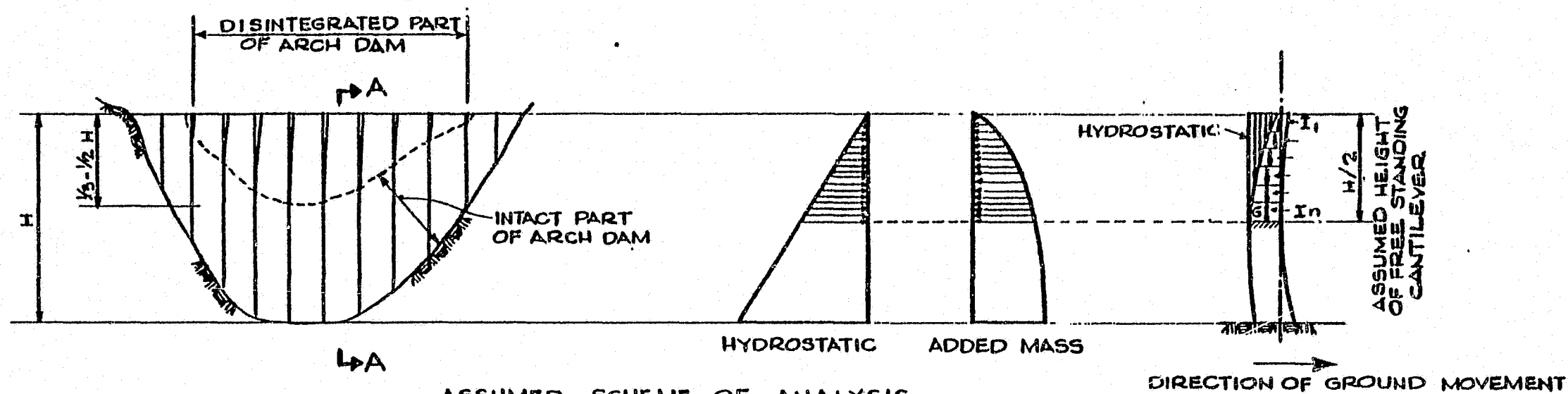
Figure 8.1



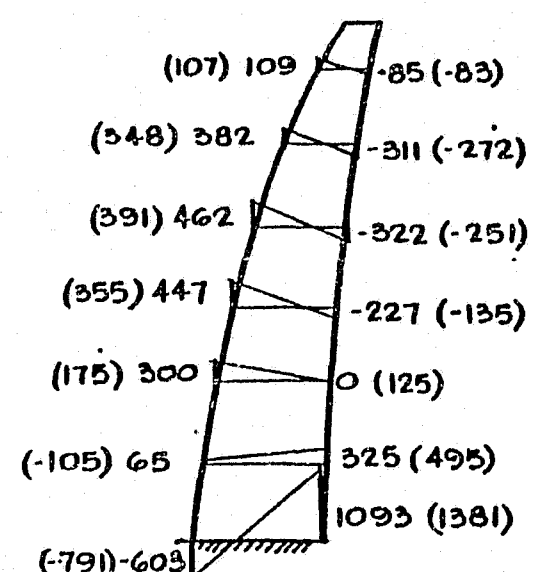
Prepared by Acres, 6/4/81 from data
provided by M. Powers, WCC

MEAN RESPONSE SPECTRA AT THE DEVIL CANYON SITE
FOR MAXIMUM EARTHQUAKES ON KNOWN ACTIVE FAULTS

Figure 8.2



0.5 G & 5% DAMPING
100% ADDED MASS



0.4 G & 10% DAMPING
100% ADDED MASS (& 50% IN BRACKETS)

CANTILEVER STRESSES (PSI)

SECTION A-A

NOTE:
⊖ (MINUS) INDICATES TENSILE STRESS.
⊕ (PLUS) INDICATES COMPRESSIVE STRESS.

DEVIL CANYON ARCH DAM
EARTHQUAKE DYNAMIC ANALYSIS

0001380

MEETING AGENDA

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

EXTERNAL REVIEW BOARD MEETING #3
OCTOBER 6-8, 1981, BUFFALO, N.Y.

AGENDA

OCTOBER 6.

Moderator: D. Wozniak

- 0830 Introductory remarks - E. Yould
- 0845 Meeting objectives and study status - J. Lawrence
- 0915 Report on hydrologic field program - J. Hayden
- 1015 Coffee
- 1030 Report on seismic studies - J. Lovegreen
- 1130 Discussion
- 1200 Lunch (brought in)
- 1300 Report on geotechnical field program - J. Gill
- 1400 Geotechnical interpretation: Watana - S. Thompson
(Geology, construction materials, bedrock conditions,
underground structures, relict channel)
- 1500 Coffee
- 1515 Discussion
- 1545 Geotechnical interpretation: Devil Canyon - S. Thompson
(Geology, construction materials, bedrock conditions,
underground structures)-
- 1645 Discussion
- 1715 Adjourn
- 1830 Dinner - courtesy of Acres (M&T Plaza Suite)

OCTOBER 7.

Moderator: J. Gill

- 0830 Introductory remarks - J. Lawrence
- 0845 Report on hydraulic studies - J. Hayden
(power/energy estimates, flood estimates, reservoir
level optimization, sedimentation studies)
- 1000 Coffee
- 1015 Discussion

1045 Watana Dam Design - D. W. Lamb/A. S. Burgess
1200 Lunch (brought in)
1300 Watana Spillway Studies - J. Hayden
1345 Watana Layout Studies - J. Lawrence
1430 Discussion
1500 Coffee
1515 Watana/Devil Canyon Low Level Outlets - R. Ibbotson
1545 Watana/Devil Canyon Power Developments - J. Hayden
1615 Discussion
1715 Adjourn

OCTOBER 8.

Moderator: D. Wozniak

0830 Introductory remarks - J. Lawrence
0845 Devil Canyon Dam Design - R. Ibbotson
0930 Discussion
1000 Coffee
1015 Devil Canyon Spillway Studies - J. Hayden
1100 Devil Canyon Layout Studies - J. Lawrence
1200 Lunch (as required)
1300 Discussion
1400 Adjourn (Panel to prepare report)
1630 Closing Statements: E. Yould/panel.