ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

SUBTASK 6.08 - PRELIMINARY DEVIL CANYON ALTERNATIVES

MINUTES OF MEETING HELD
IN BUFFALO 9,10TH DECEMBER 1980

ACRES

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DISTRIBUTION:

M. Copen
D. MacDonald
H. Eichenbaum
J.D. Lawrence
J.W. Hayden
I.P.G. Hutchison
V. Singh
R. Ibbotson
L. Duncan
R. Curtis
O. Ugaz
D. Shandalov
D. Curtis
E. Skeba
D. Carlson (Buffalo)
P. Joselin (N. Falls)
G. Thompson (N. Falls)

MINUTES OF MEETING
held at the offices of
Acres American Incorporated
on Tuesday and Wednesday, December 9-10, 1980

December 19, 1980

PRESENT:

M. Copen
D. MacDonald
H. Eichenbaum
J.D. Lawrence
J.W. Hayden
I.P.G. Hutchison
V. Singh
R. Ibbotson
L. Duncan
R. Curtis
O. Ugaz
D. Shandalov
D. Curtis
E. Skeba

Attachments to these minutes:

1. Presentation on Geotechnical Considerations Made During Meeting.
2. Arch Dam Design Comments (A summary of comments made by M. Copen)
3. Arch Dam Design Criteria (Suggested by M. Copen)
4. Aspects of Construction Techniques
5. Pertinent Information
6. Presentation on Stress Analyses Conducted by Acres
7. Summary of Acres Stress Analyses
8. Dam Layouts at Devil Canyon and Watana.
9. Report of Visit to Acres American, by M.D. Copen

Minutes:

Follow Up Action By:

1. Overview of Acres Devil Canyon Arch Dam Design

Acres indicated:
- the concrete gravity dam proposed at the site by the COE is probably unstable;
- the thin arch dam proposed by the USBR is a very preliminary design.

Acres tabled their arch-gravity alternative and indicated they were still experiencing problems with tensile stresses on the downstream face. Preliminary cost estimates of dam and spillway made by Acres indicated a narrow spread between rockfill, arch gravity and thin arch alternative ($315 to $350 million). However more refinement of these estimates was required and is in progress — R. Ibbotson

Some of the dam layouts discussed are included in Attachment 8. Copen indicated a thin arch dam was more appropriate for the site and questioned the high cantilever tensile stresses on the downstream side obtained by Acres. He indicated the aim should be to design the arch for no tension under normal (hydrostatic, gravity and temperature or H, G&T) loading conditions. He also indicated that selection of dam type should be based firstly on a safety consideration and secondly on economic considerations. He did agree that comparison of dam safety for various dam types was difficult, particularly comparison of fill versus concrete dams.

During discussion it became apparent that a severe and very probable loading case was in the reservoir level drawn down (several hundred feet) and minimum temperature conditions. It was agreed that this loading case should not be taken into account in the early design stages but should be evaluated before the detailed design is finalized.

2. Results of Dam Stress Analyses

Acres briefly presented the stress analyses conducted on both the gravity arch and thin arch dams for static and dynamic loading conditions using the ADAP finite element program. Attachments 6 and 7 summarize the information presented by Acres. M. Copen questioned the results which indicated high tensile stresses (H,G&T loading) on the downstream face. He also indicated that the weight distribution used in ADAP, i.e. vertical and horizontal, was not appropriate. The weight should be distributed vertically as this was closer to accepted North American construction techniques in which grouting is commenced only after completion of all concrete placement. He indicated that a weakness of ADAP was poor representation of the abutment stresses due to the coarse mesh used and the lack of printout information on these stresses. Acres indicated that

Follow Up Action By:

the poor stress representation only occurred on the upstream and downstream faces of the dam and that they were planning to improve the printout of abutment stress information. E. Skeba

M. Copen indicated that ADSAS (a cantilever-arch type of model employing the trial load method) should be used as the main design tool.

3. Results of Geotechnical Analyses

Acres outlined the results of some very preliminary abutment stability analyses. These indicated that unless future exploration programs reveal unexpected conditions, there would be no problems with abutment stability. Attachment 1 contains some of the details discussed.

Acres briefly outlined a program for geotechnical exploration at the Devil Canyon site. This program, which is still subject to budgetary review, includes in priority order:

1. Drill, from both banks if possible, into the abutments.
2. Drill across and under the river bed.
3. Drill across E-W features located on the left abutment.

Additional geologic mapping would also be done this winter.

During discussion, it was indicated that at this stage of the study, it was acceptable to locate structures on the left hand abutment.

4. Discussion and Summary of Above Proceedings

Acres outlined briefly that the scope of the engineering studies at Devil Canyon was as follows:

Phase I - Feasibility study and FERC licensing	Schedule
Step 1 - Determine whether there is anything that could rule out an arch dam.	By spring 1981
Step 2 - Conduct a detailed feasibility design of an arch dam.	By spring 1982

Phase II - Preliminary Engineering and bid documents.

To follow Phase I, schedule is yet undetermined.

Acres emphasized that the purpose of this meeting was to discuss and finalize Step 1 studies. M. Copen indicated that he had the following suggestions/recommendations to make:

1. Acres should get the "ADSAS" program up and running as soon as possible. D. Curtis

Follow Up Action BY:

2. Acres should get the "HEATFLOW" program up and running as soon as possible (its methods are thoroughly documented and the USBR find it totally acceptable). This program would be used to evaluate the long term temperatures within the dam. — D. Curtis

3. Acres should consider only one concrete dam type at Devil Canyon, i.e. a thin double curvature arch dam. As a starting point the following dimensions would be appropriate: — R. Ibbotson

top width = 20 '
base width = 90'

Circular arches should be used and the line centers of the up-and downstream faces should be separated sufficiently to give more arch thickness towards the abutments. During the design process when/if dam sections are found to be in excessive tension, concrete should be removed and not added. The dam should be reorientated to improve symmetry, central angle at the crest should be between 100^o and 110^o and abutment shapes should be regular. This design should be based on the best interpretation of the sound rock profile that Acres has at this stage. A sound rock contour map is essential, the confidence limits of which should be goverened by the quality of the currently available data. — V. Singh

4. Design Approach

Dam foundations and abutments should extend 5' into sound rock. M. Copen outlined a design approach which involves basically designing a simple structure for static loading (H,G&T) and checking it for dynamic earthquake loading using a psuedo-dynamic type analysis. If the structure is well designed, it should withstand earthquake loading up to 0.4g without much problem. He indicated Acres should use the "HEATFLOW" and "ADSAS" programs for this and use the finite element program as a final check. Attachment 3 outlines this approach in more detail. — R. Ibbotson

5. Design Criteria

Outlined in Attchment 3, M. Copen indicated that Acres should review the latest USGS risk criteria w.r.t. earthquake design of major structures. Acres should also look at Karl Zwanger's improvement to the Westergaard assumption w.r.t. water mass in dynamic analyses. — R. Ibbotson

6. Watana Arch Dam Concept

Acres briefly tabled a preliminary arch dam layout for the Watana site (see Attachment 8). M. Copen's comments were as follows:

- Watana is a good site for a 3 centered arch;

Follow Up
Action By:

- in staging the dam, one should not consider adding concrete to the downstream face because of bonding difficulties which cause adverse temperature stress differentials. The height can be increased by merely adding concrete to the top of the dam;

- appropriate dam dimensions would be 100'-150' base width and 30' crest width.

7. General

- M. Copen indicated that to maximize his input to the project he should be continuously updated on analyses results and layouts; — R. Ibbotson

- If necessary, arrangements could be made for Acres to use the USBR ADSAS program in Denver (through APA);

- Alternatively B.C. Hydro have developed an IBM version of the program which would be compatible with the Acres VAX system;

- Good contact man at the USBR is Howard L. Boggs.

Reported by ____________________
I. Hutchison

IH:vb

ATTACHMENT 1

Presentation to Mr. M. Copen, December 9, 1980
Geologic & Geotechnical Consideration
Devil Canyon Arch Dam

1. Geologic Model of the Site

- Beginning to understand 2-dimensional model of the site.

- The information is still inadequate to develop a 3-dimension model.

- Geologic structure

In a broader sense, there are 3 major geologic structures:

- bedding plane subparallel to the river and steeply dipping to south;

- major joint set roughtly N-S and almost vertically dipping;

- minor joint set approximately E-W and almost vertical dip.

On the right abutment, bedding planes daylight in a drainage feature d/s.

Considering the course of the river, both present and past, there are some questions whether so called minor joint set is more developed near Devil Canyon site (to be further studied next year).

2. Sliding Stability - preliminary analyses

Two cases analyzed for the right abutment.

Case 1 - assume the dam is sitting on a hypothetical bedding plane daylighting d/s and could slide.

- to develop a S.F. = 4 for this block.

  - required $\emptyset \approx 55^{o}$ for $c = 0$

  - required $c \approx 85$ psi for $\emptyset = 0$

as compared to shear strength of rock

(10% of q_u) i.e. approximately 1700 psi

This assumes forces from the dam without consideration of potential hydrostatic pressures within cracks just u/s of dam.

Case 2 - Assuming relatively small black isolated from rest of the dam and failure plane 5 ft. below the foundation level (see figure) with additional hydrostatic pressure.

For a F.S. = 4

- required $\phi = 63^{o}$ for c = 0

- required c = 130 psi for $\phi = 0$

3. Planned Geotechnical Investigations for 1981

- Realizing the limitations on our budget we plan (in order of priority):

1 boring on L.A. and 1 boring on R.A., both close to river level and going into abutments.

1 boring (minimum) crossing the river.

1 boring crossing the suspected shear zone on the left abutment.

Geologic mapping at the site and near Portage Creek.

4. The current state-of-knowledge does not disprove the feasibility of an arch dam.

V. Singh

ATTACHMENT 2

Arch Dam Design Comments

(Interesting Points Raised During Discussion)

- Generally, in a well designed arch dam the maximum deflection should occur several feet below the crest.

- Grout temperatures generally used are around 38°F.

- Note that statements made by K. Hansen (PCA) that thinner arches require thicker crests is incorrect.

ATTACHMENT 3

Arch Dam Design Approach and Criteria

(Suggested by M. Copen)

APPPOACH

Basic Approach - Acres should aim for a good and "tight" engineering design. This implies making maximum use of the concrete, i.e. under normal loading conditions (H,G&T) compressive stresses should all be near the maximum allowable and tensile stresses should be avoided or minimized. Should "political" pressures require a more conservative approach, this should be dealt with at a later stage.

Design Loads - The basi approach should involve designing the dam initially for static loads (H,G&T) and checking for dynamic loading. At this early stage, it would be appropriate to do a psuedo dynamic analysis (or a response spectrum analysis if data is available). Before the final design is completed one would have to do a time history dynamic analysis.

Temperature Effects - Placement temperature would probably be as low as 45^{O}F (compared to 50-55^{O}F in the lower 48). One would have to include refrigeration pipes to limit the maximum temperature to 80^{O}F. Cooling would be suspended until grouting commenced. At that stage the structure would be cooled to 39-40^{O}F. The "HEATFLOW" program should be used to calculate the long term steady state internal temperature (USBR have found that daily and weekly temperature fluctuations only penetrate up to a depth of 5 feet). We may have trouble because the long term mean temperature could be lower than the grouting temperature, and joints would not all be in compression. One should, thus, think of double or triple water stops. Effects of hydration of different types of cement on temperatures should be considered when selecting appropriate cement types.

The ADSAS approach, in which a linear temperature distribtuion through the structure is assumed, is adequate. In fact, at this early stage one could use a uniform temperature distribution.

Special Loads - These loading cases include earthquake during construction and severe temperature on a partially filled reservoir. The latter is important, but need not be considered at this early stage.

Design Guidelines - The basic objective should be to design the structure so that tension is eliminated (or minimized) under normal loading conditions (H,G&T). Generally, when tensions develop, concrete should be removed. We may have problems in the heel of the dam where it would be acceptable to allow tensile stresses of up to 150 psi.

Pseudo Dynamic Analysis - When using a pseudo dynamic analysis, maximum stresses generally occurred near the bottom of the dam. With response spectra and time history analyses, these maximum stresses move upward. This has to be considered when using these analyses techniques.

DESIGN CRITERIA

General:

Frost resistant concrete

Concrete strength (365 day) = 5000 psi

Density of concrete = 150 lb/ft^3

Static Loading: (H,G&T)

Factor of safety (in compression) = 4

Tensile strength (for purposes of estimating cracking only) = 250 psi (5% of strength)

Deformation modulus of concrete	3,000,000 psi
Deformation modulus of rock	2,000,000 psi (1 and 3,000,000 psi for sensitivity analyses)
Poissons ratio - concrete	0.2
rock	0.2 until better estimate available

Dynamic Loading: (earthquake)

Factor of safety	>1 (Max. credible earthquake)
	=2 (design earthquake, ± 100 year return period)
Tensile strength (250 + 2 x50% increase*)	approximately 500 psi (Acres to verify)
Elastic modulus of concrete	= 5,000,000 psi (Acres to verify)
Psuedo dynamic loading: Water:	50% acting horizontally
Concrete:	50% acting horizontally

*increase 50% for converting from tensile to flexural strength and further 50% to convert to allowable stresses under dynamic loading.

ATTACHMENT 4

Aspects of Construction Techniques
(Some of Which Must Be Taken Into Account
When Analyzing Stresses in Arch Dams)

General method of construction in the U.S. involves completing the vertical monoliths to dam crest level in 5' to 10' lifts and then grouting the vertical joints in roughly 60 ft. lifts (Glen Canyon dam was an exception to this rule. It was built in two stages). This practice is not ideal in terms of stress distribution within the dam, but simplifies construction procedures (and costs). It does mean that in analyzing stresses the dam the weight must be distributed vertically downwards only.

If the dam construction is staged (as in Glen Canyon), the second stage weight is distributed vertically down to the crest of stage one and then taken up by what can be considered a monolithic first stage wall (i.e. spread vertically and horizontally).

ADSAS can be used to model any of the above.

Rise in temperature of conceret due to hydration should be limited to 35^{o}F, otherwise cracking problems would be encountered.

ATTACHMENT 5

Pertinent Information

Auburn dam (thin arch version) maximum deflections were 0.5 ft (H,G&T loading) and 1.5 ft (H,G&T earthquake loading), respectively.

The USBR have looked at using fiberglass thermal insulation of the dam wall to improve temperature stresses. They concluded it was feasible, but have never implemented it.

ATTACHMENT 6

Presentation on Stress Analyses

Conducted By Acres (D. Curtis)

SUSITNA : DEVIL'S CANYON

RESULTS OF DAM STRESS ANALYSES

1./ LOAD CASES (STATIC)

2/ ADAP "Arch Dam Analysis Package"

3/ TEMPERATURE ANALYSES

4/ RESULTS OF STATIC ANALYSIS

5/ CRACKED ANALYSIS

6/ RESULTS OF CRACKED ANALYSIS ON THIN ARCH DAM

7/ DYNAMIC ANALYSIS

8/ RESULTS OF DYNAMIC ANALYSIS

9/ DISCUSSION OF RESULTS:

LOAD CASES (STATIC)

1/ Hydrostatic + GRAVITY. (H+G)

The hydrostatic load is applied at ELEV. 1450' – leaving 5' freeboard

The gravity load is applied as body loads through an unjointed structure. Hence the arches and cantilevers share the load-bearing function.

2/ Hydrostatic + Gravity + TEMPERATURE (H + G + T)

TEMPERATURES
- mean January air temp.
- reservoir temp 39°F
- grout temperature at a constant 37°F

- NO ICE LOADS have been included to date.

ADAP "ARCH DAM ANALYSIS PROGRAM"

The dam is modelled using 8-noded brick elements. Three elements through the thickness of the dam.

Foundation elements extend a distance equal to the height of the dam into the surrounding rock - in all three directions.

The program allows the user to specify :- gravity loads
- water loads (hydrostatic)
- temperature loads
- concentrated loads.

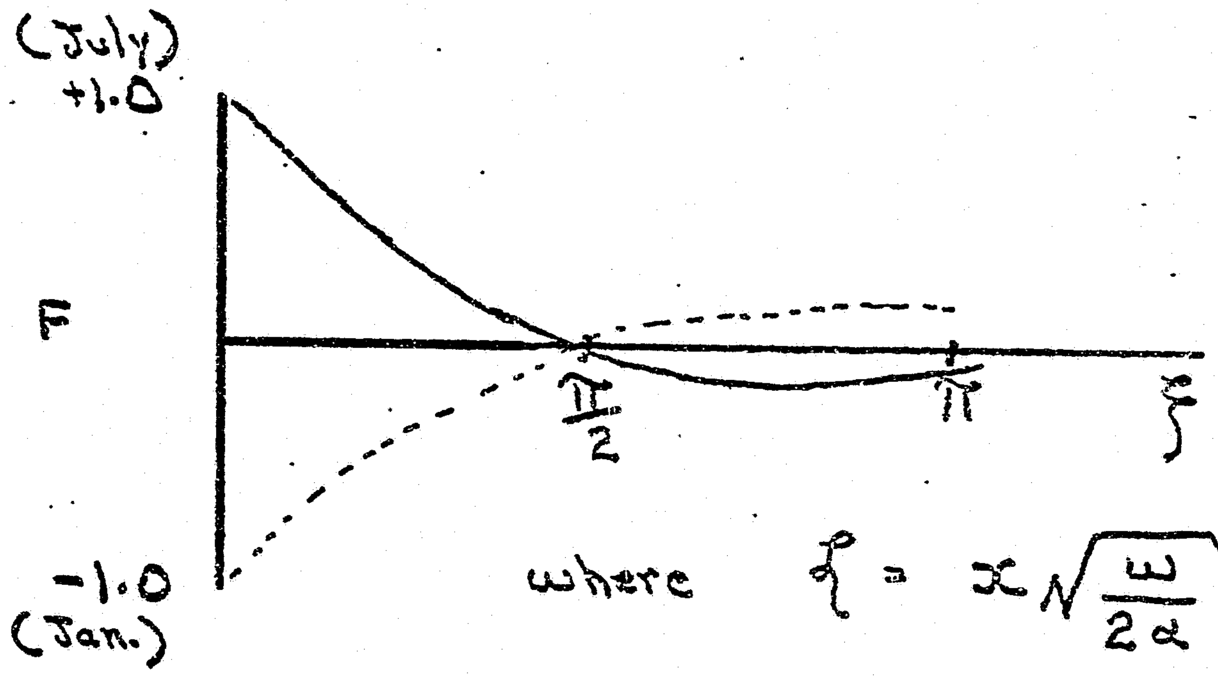
ADAP: SUSITNA MODELS

- 6 mesh elevations
- unsymmetric dams.
- 190 elements (90 - DAM
  100 - FOUNDATION)

TEMPERATURE DISTRIBUTION

- Assume the dam is a semi-infini[te] solid subjected to surface temperature which are harmonic functions of tim[e]

Variation of temperature with depth is as follows:



where $\xi = x\sqrt{\dfrac{\omega}{2\alpha}}$

x - depth of penetration
$\omega = 2\pi$ (one year cycle)
α = Diffusivity

Assuming a GRANITE Aggregate:

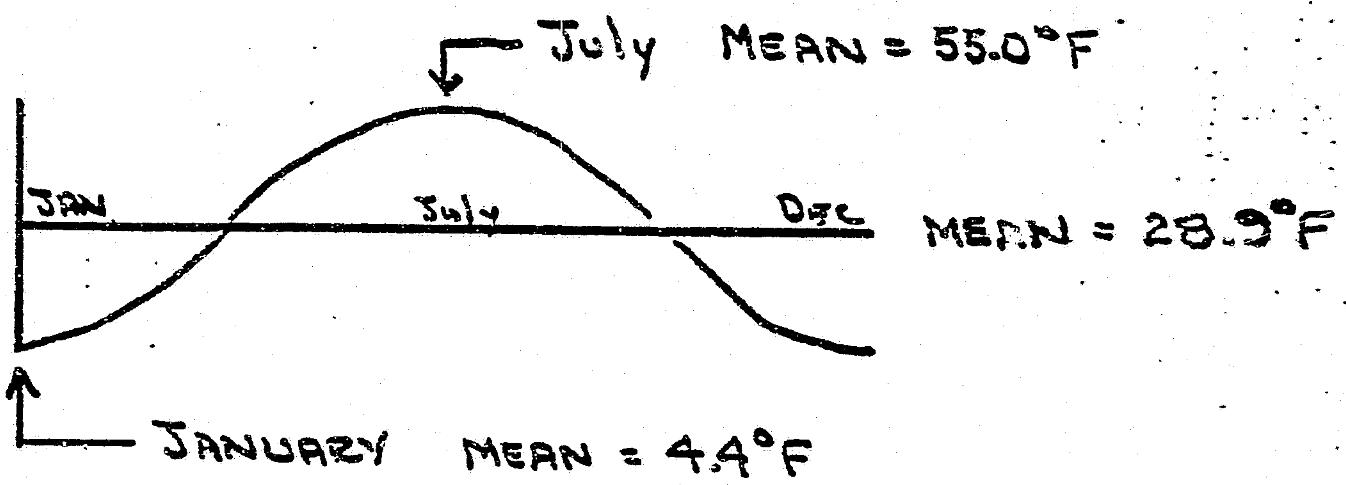
$\alpha = 0.045\ ft^2/hr$

$F = 0$ @ $\xi = \pi/2$

$\therefore\ x = 17.6$ feet (penetration)

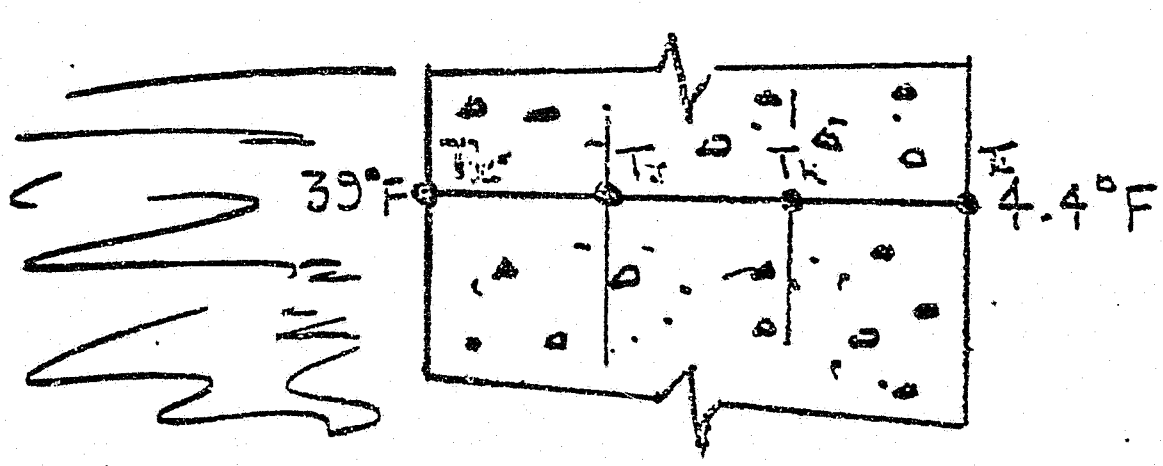
TEMPERATURE DISTRIBUTION (CONT'D)

Preliminary annual air temperature cycle is:



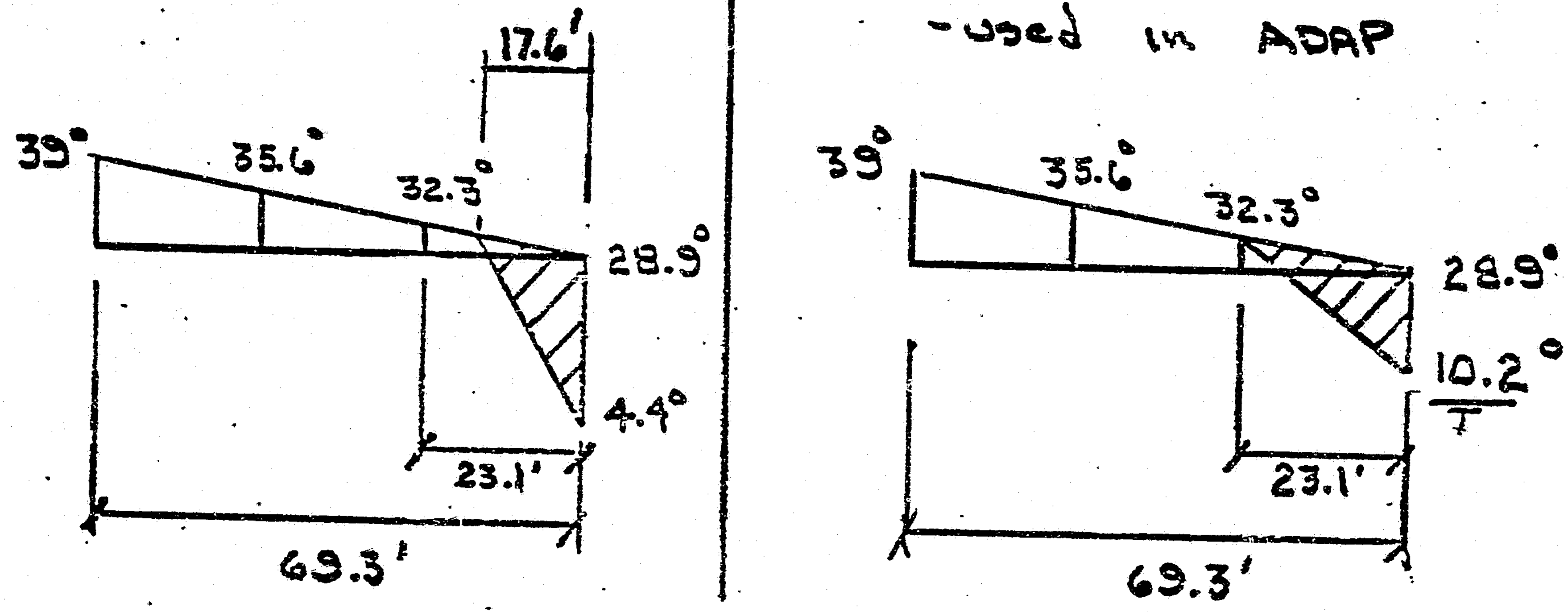
Below a depth of approx. 80 ft. the reservoir temperature will remain at 39°F year-round.

The program ADAP uses three BRICK elements through the thickness of the dam ∴ temperatures can be specified on the dam face & at two interior nodes.



TEMPERATURES

ACTUAL	IDEALIZED
	- used in ADAP



The "ACTUAL" & "IDEALIZED" will produce nearly the same GLOBAL thrusts and moments HOWEVER surface cracking must be computed by hand.

CRACKING ANALYSIS

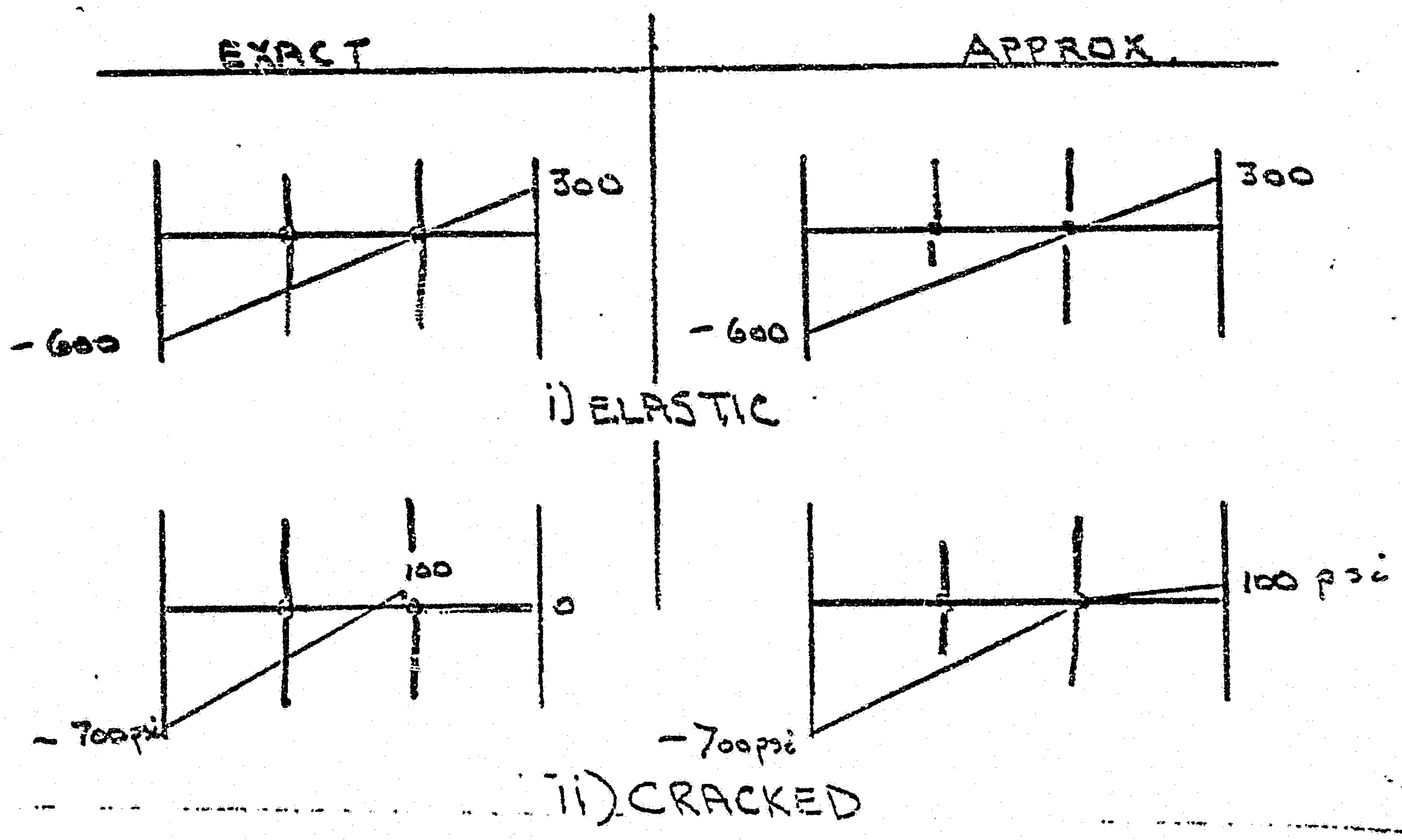
EXACT METHOD:

Tensions exceeding specified allowable values are relieved such that at the crack tip we have allowable tensions. and along the crack no tensions exist.

Approximate Method:

Scale stresses in the element so that element stresses are less than a specified maximum.

HYPOTHETICAL EXAMPLE



DYNAMIC ANALYSIS:

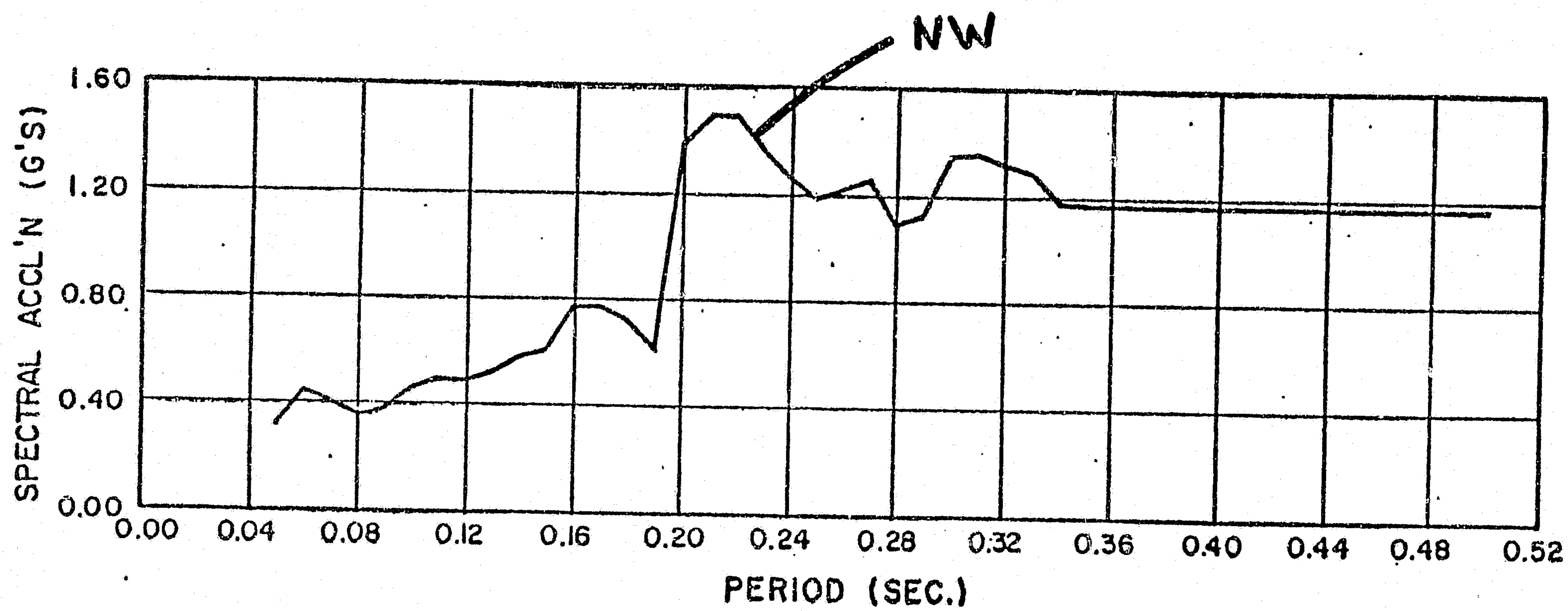
- Response Spectrum technique
- 1952 Taft earthquake
  - largest component (N-W) applied in upstream - dnstream direction
  - all 3 directions are considered.
- A rigid foundation was used.
- the dam is analyzed as unjointed (as in STATICS) hence tensile arch stresses are possible in the analysis.

- Stress results are RMS hence they may be compressive or tensile.

- The original version of ADAP does not include hydro dynamic effects but the program has been modified to account for "added mass".

RESPONSE SPECTRA -TAFT- 1952 NORTHWEST

SCALED TO 0.5G PEAK HORIZONTAL

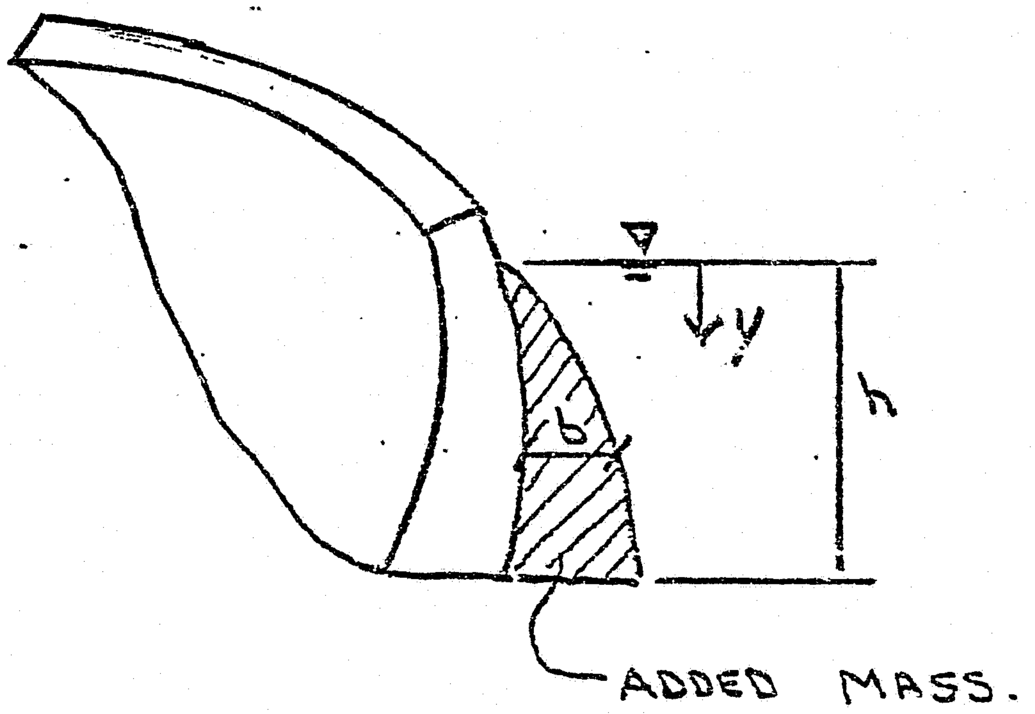


To 0.4 g @ 2 sec

DYNAMIC ANALYSIS

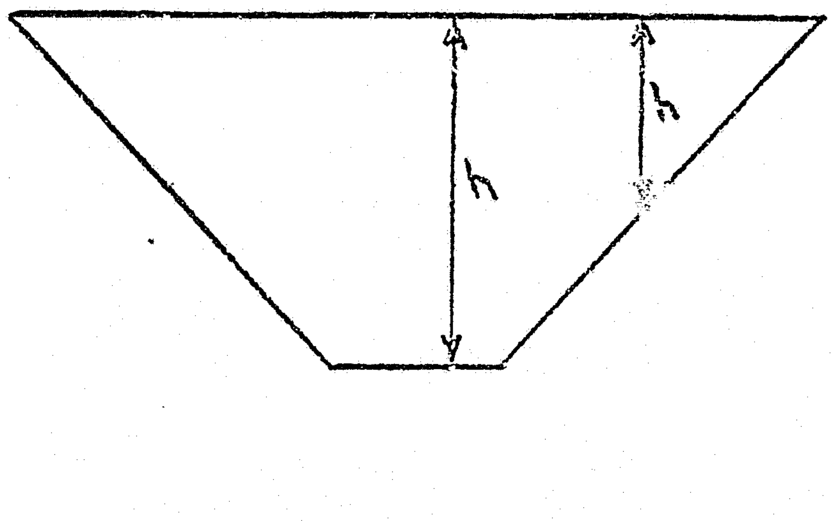
"Added Mass"

ADAP has been modified to use WESTERGAARD'S "added mass" approach.



$b = 7/8 \sqrt{hy}$

b is width of water accelerated with the dam



DYNAMIC ANALYSIS

As an approximation the "added mass" is the same for all modes. This is reasonable since the first mode contributes the major portion of dynamic stresses.

The next step would be: to modify ADAP to calculate added mass as a function of the mode shape. This has been done for 2-D structures (GRAVITY DAMS — EADHI) but I am unaware of a 3-D model.

DYNAMIC ANALYSIS RESULTS:

- All dynamic analyses were carried out assuming a full reservoir.

i) Natural Frequencies (cps) ω

MODE	ARCH GRAVITY	THIN ARCH
1	1.60	1.42
2	1.93	1.55
3	2.81	2.39

The lower natural frequency of the Thin Arch and associated water is due to the lower stiffness to mass ratio of the Thin Arch (ie. $\omega \doteq \sqrt{\frac{k}{m_D + m_w}}$)

ATTACHMENT 7

Summary of Acres Stress Analyses,

Indications of Where Dam Design Improvements

Can be Made and Other Material

Presented By D. Shandalov

LOADS	US	DS	US	DS
	← GRAVITY ARCH →		← THIN ARCH →	
<u>HYDR. + GR.</u>				
PRINCIPAL COMPR. ST	520	670	993	1040
" TENSILE "	8	153	60	259
<u>HYDR. + GR + TEMP.</u>				
PRINCIPAL COMPR. ST	624	479	1212	821
" TENSILE "	30	512	34	762

<u>RATIO OF STRESSES</u> : $\frac{\text{ARCH DAM}}{\text{ARCH GR. DAM}}$

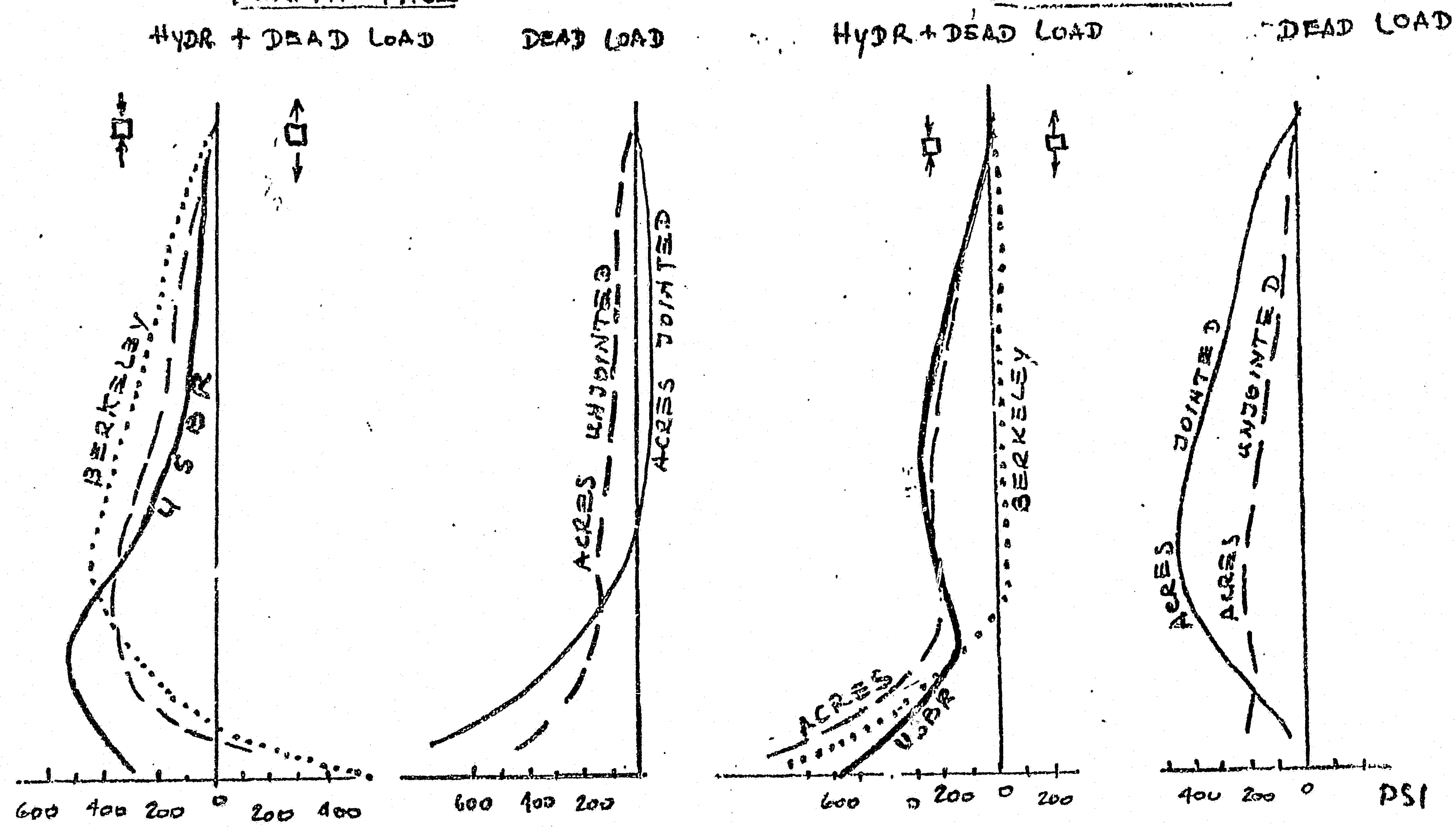
HYDR. + GR.	COMPR.	U.S. CROWN	$\frac{993}{520} = 1.9$
		D.S. ABUTMENT	$\frac{1040}{670} = 1.55$
	TENSION	D.S. CROWN	$\frac{259}{153} = 1.7$
HYDR + GR. + TEMP		U.S. CROWN	1212 / [illegible] = 1.95

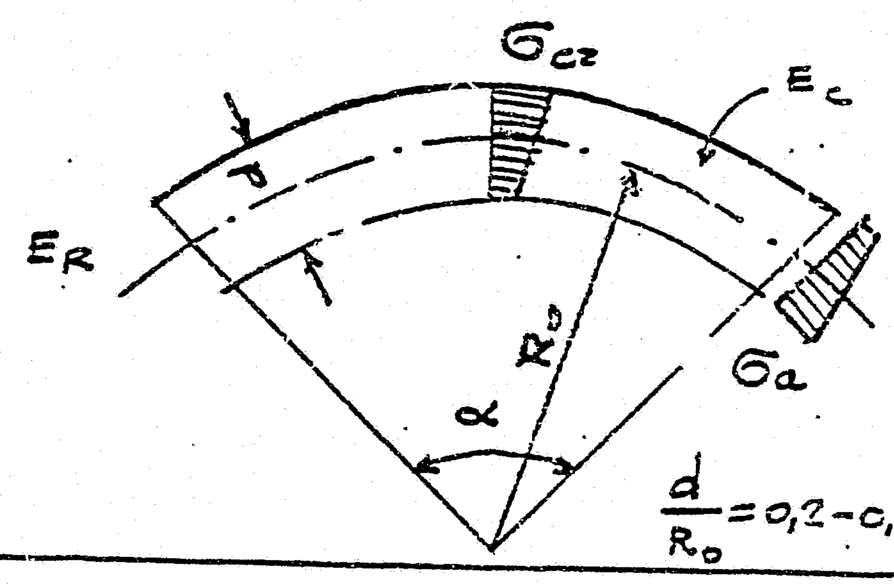
TENSION IS +ve

AIM	MEASURES	A	AG
1. REDUCTION OF COMPRESSIVE STRESSES AND TORSION	- IMPROOVING OF SIMMETRY:		
	- RELOCATION OF CENTERINE	+	—
	- THRUST BLOCK AT RIGHT ABUT.	+	—
	- ABUTMENT PADS	+	—
	THREE CENTERED ARCHES	+	+
2. REDUCTION OF TENSILE STRESS IN CANTILEVER			
(a) DOWNSTREAM FACE UPPER PART	- INCREASE OF CREST OVERHANG	+	—
	- IMPOUNDING WITH TEMPORARY OPEN JOINTS AT CREST	+	+
(b) UPSTREAM HEEL	- THICKENING OF CROWN CANTILEVER BASE	+	
	- PARTIAL HORIZONTAL JOINTS AT UPSTREAM BASE	+	+
	- Hollow AT BASE	—	+
3. REDUCTION OF TENSILE STRESSES IN ARCHES	- THREE CENTERED ARCHES	+	+
	- SECONDARY GROUTING AFTER IMPOUNDING	+	+
	- WIDE TRANSVERSE JOINTS (SLOTS) CONCRETED WHEN THE BASIC CONCRETE IS FROZEN	+	+
4. [illegible]DUCTION OF TENSILE T[illegible] STRESSES	THERMAL INSOLATION OF DOWN[illegible]EAM FACE	+	+

MORROW POINT DAM

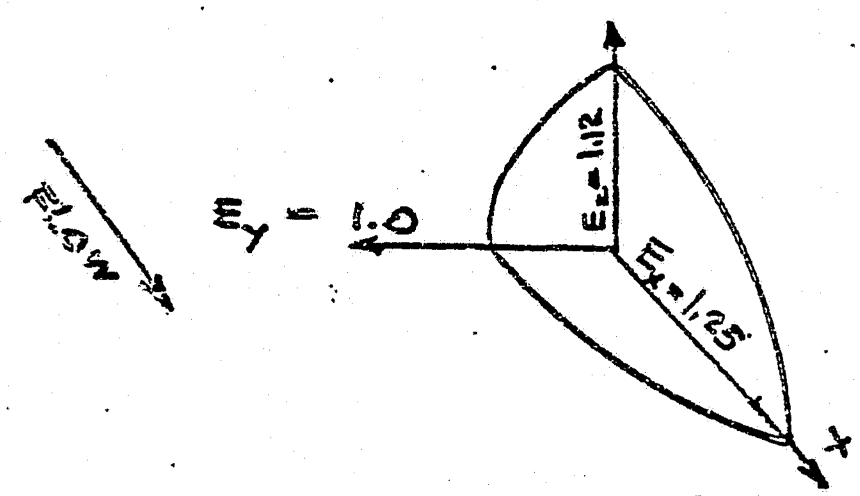
CROWN CANTILEVER STRESSES (PSI)

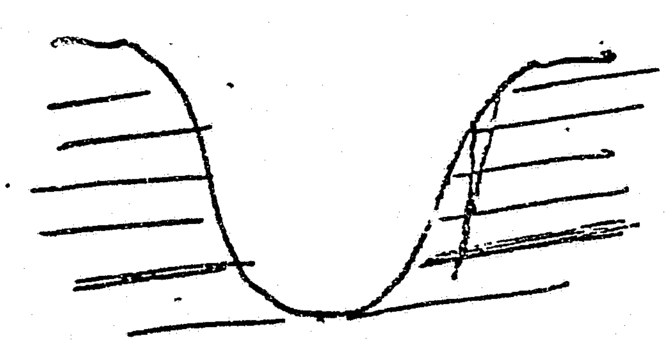




$\frac{d}{R_0} = 0{,}2 - 0{,}3$; $\alpha = 100°$

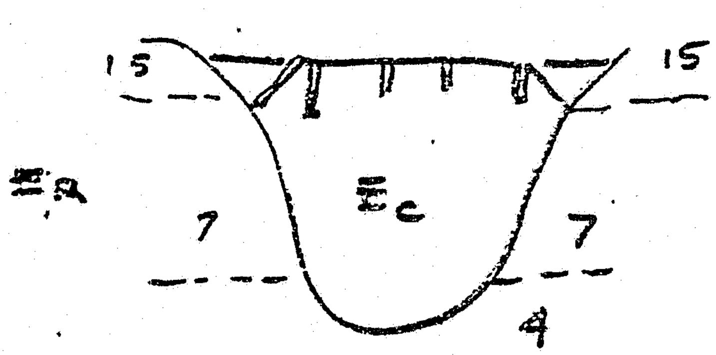
E_c/E_R	σ_{cr} %	σ_a %
1	100	100
2	105	95
3	115	90
4	120	87
5	125	85
6	130	82





CHIRKEY

	E R (T/cm²)	(PSI) ×10⁶
WEEK	30-70	0,4 - 1
NORMAL	70 - 150	1 - 2,1
RIDGID	150 - 300	2,1 - 4,2
VERY RIDGID	> 300	> 4.2



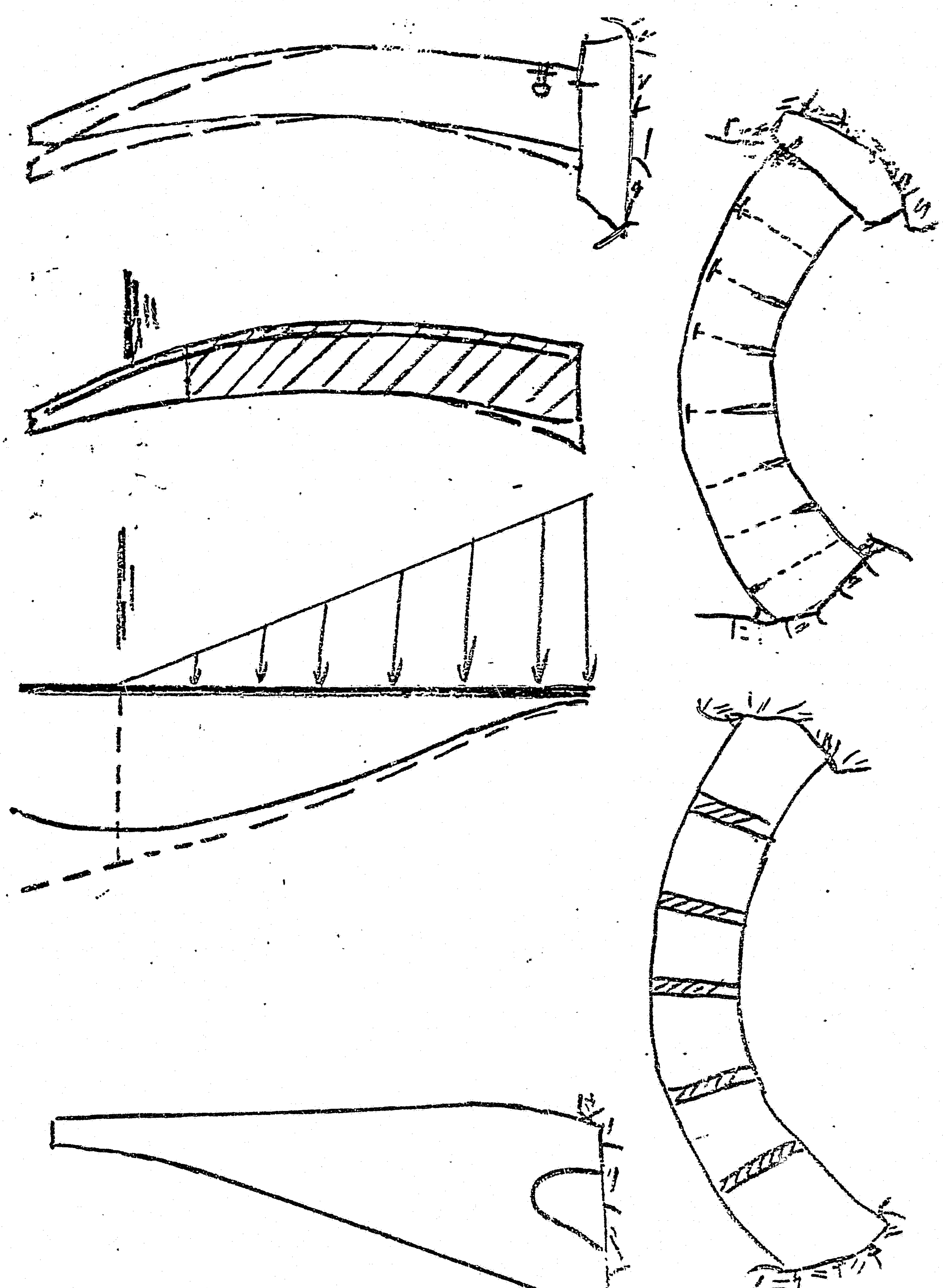
KUROBE - 4 (186 M)

$\frac{E_c}{E_R} = 4; 7; 15$

E_R = 15-25 T/cm²

(200 - 350) ×10³ PSI

ROSSENS (83 M)



ATTACHMENT 8

Dam Layouts at Devil Canyon and Watana

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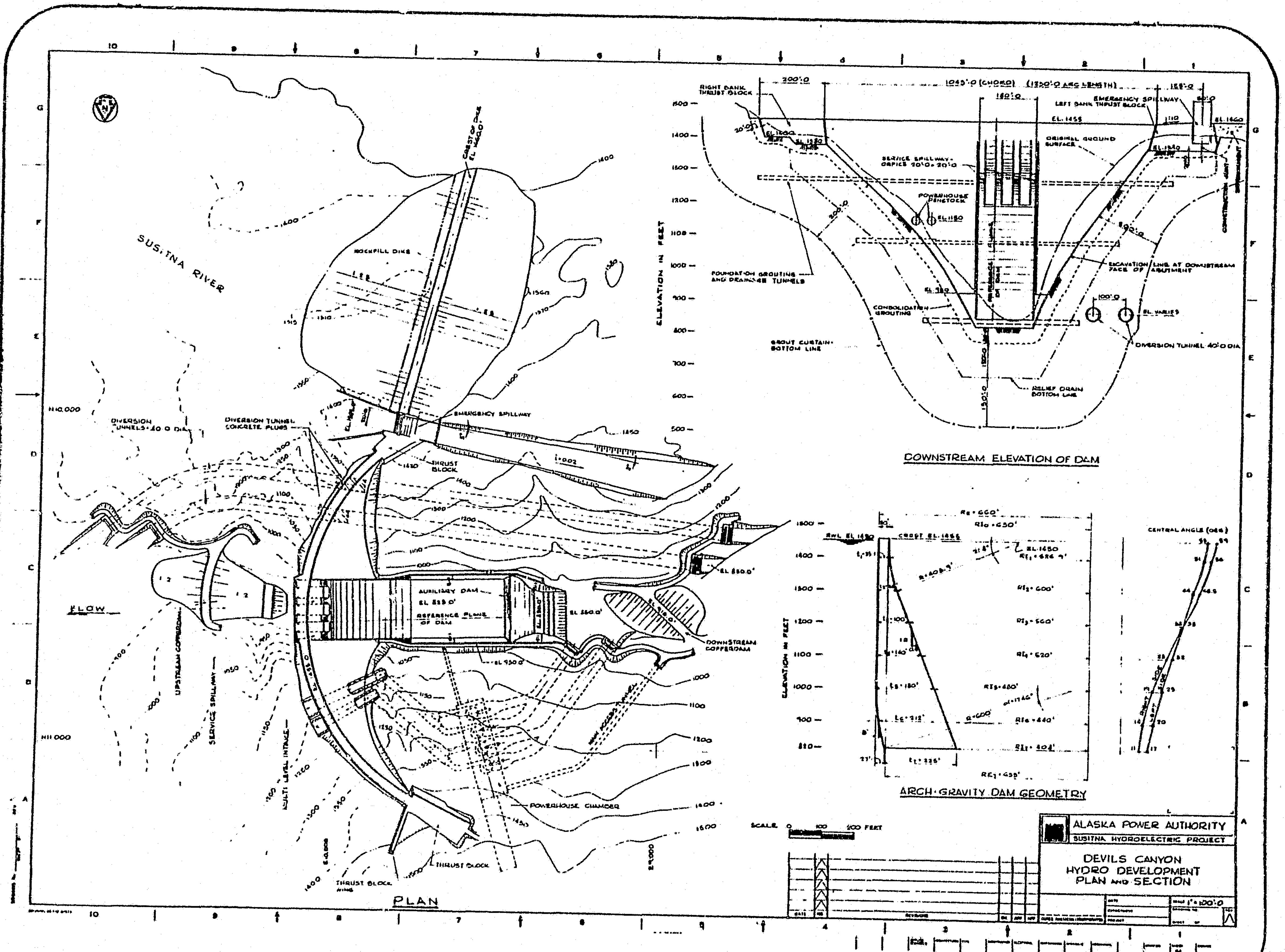

SUSITNA RIVER
ROCKFILL DIKE
CREST OF DIKE EL 1460.0'
EMERGENCY SPILLWAY
DIVERSION TUNNELS 40.0 DIA.
DIVERSION TUNNEL CONCRETE PLUGS
THRUST BLOCK
FLOW
UPSTREAM COFFERDAM
SERVICE SPILLWAY
MULTI LEVEL INTAKE
AUXILIARY DAM
REFERENCE PLANE OF DAM
DOWNSTREAM COFFERDAM
POWERHOUSE CHAMBER
THRUST BLOCK WING
PLAN
ELEVATION IN FEET
RIGHT BANK THRUST BLOCK
LEFT BANK THRUST BLOCK
EMERGENCY SPILLWAY
ORIGINAL GROUND SURFACE
SERVICE SPILLWAY ORIFICE 20'0 x 20'0
POWERHOUSE PENSTOCK
FOUNDATION GROUTING AND DRAINAGE TUNNELS
CONSOLIDATION GROUTING
GROUT CURTAIN BOTTOM LINE
RELIEF DRAIN BOTTOM LINE
EXCAVATION LINE AT DOWNSTREAM FACE OF ABUTMENT
DIVERSION TUNNEL 40'0 DIA
CONSTRUCTION JOINT
DOWNSTREAM ELEVATION OF DAM
CENTRAL ANGLE (DEG)
ARCH-GRAVITY DAM GEOMETRY
SCALE 0 100 200 FEET
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
DEVILS CANYON
HYDRO DEVELOPMENT
PLAN AND SECTION

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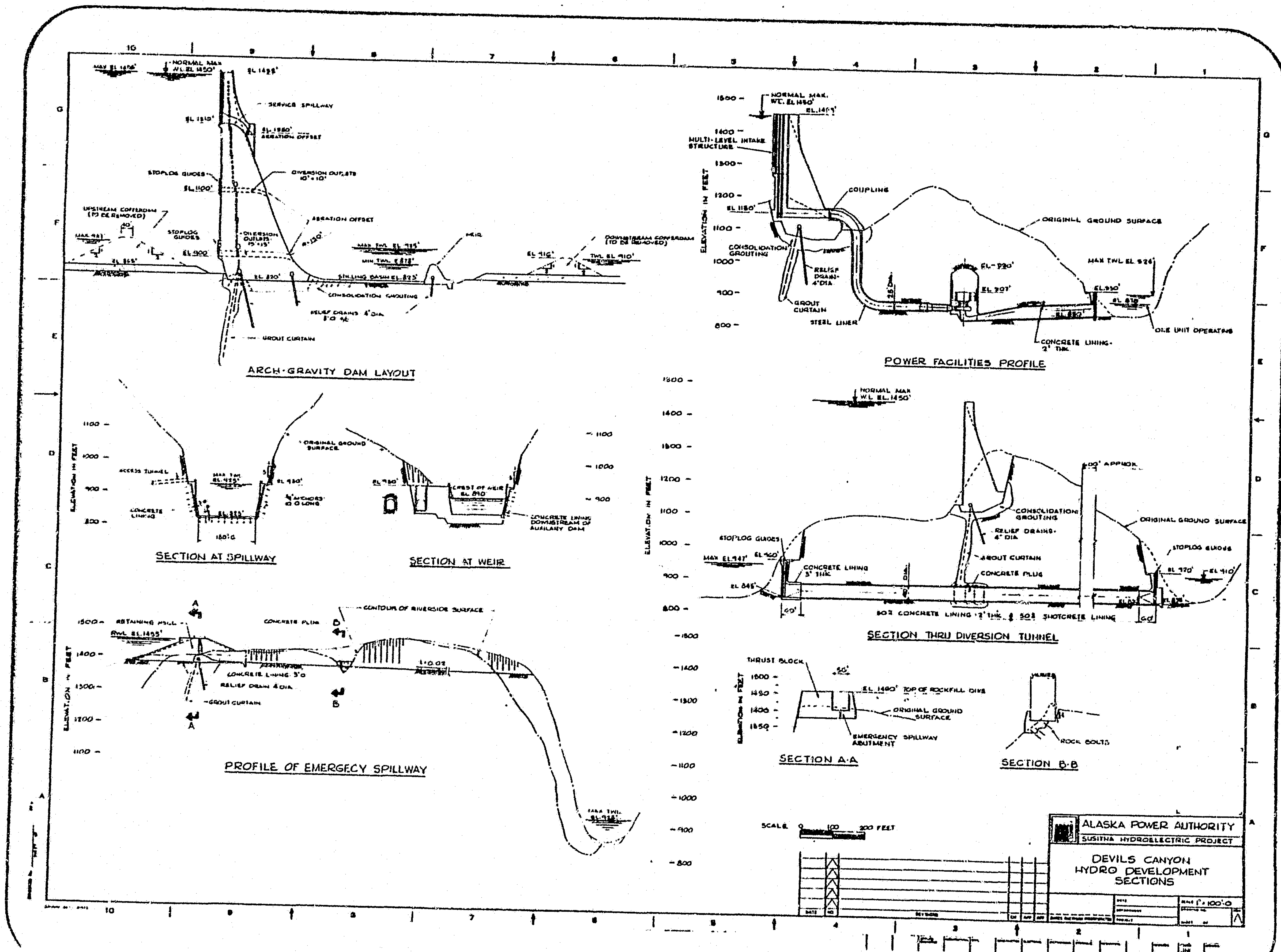
ARCH-GRAVITY DAM LAYOUT
POWER FACILITIES PROFILE
SECTION AT SPILLWAY
SECTION AT WEIR
SECTION THRU DIVERSION TUNNEL
PROFILE OF EMERGENCY SPILLWAY
SECTION A-A
SECTION B-B
ORIGINAL GROUND SURFACE
COUPLING
CONCRETE LINING
STOPLOG GUIDES
GROUT CURTAIN
CONSOLIDATION GROUTING
THRUST BLOCK
ROCK BOLTS
SCALE 0 100 200 FEET
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
DEVILS CANYON
HYDRO DEVELOPMENT
SECTIONS

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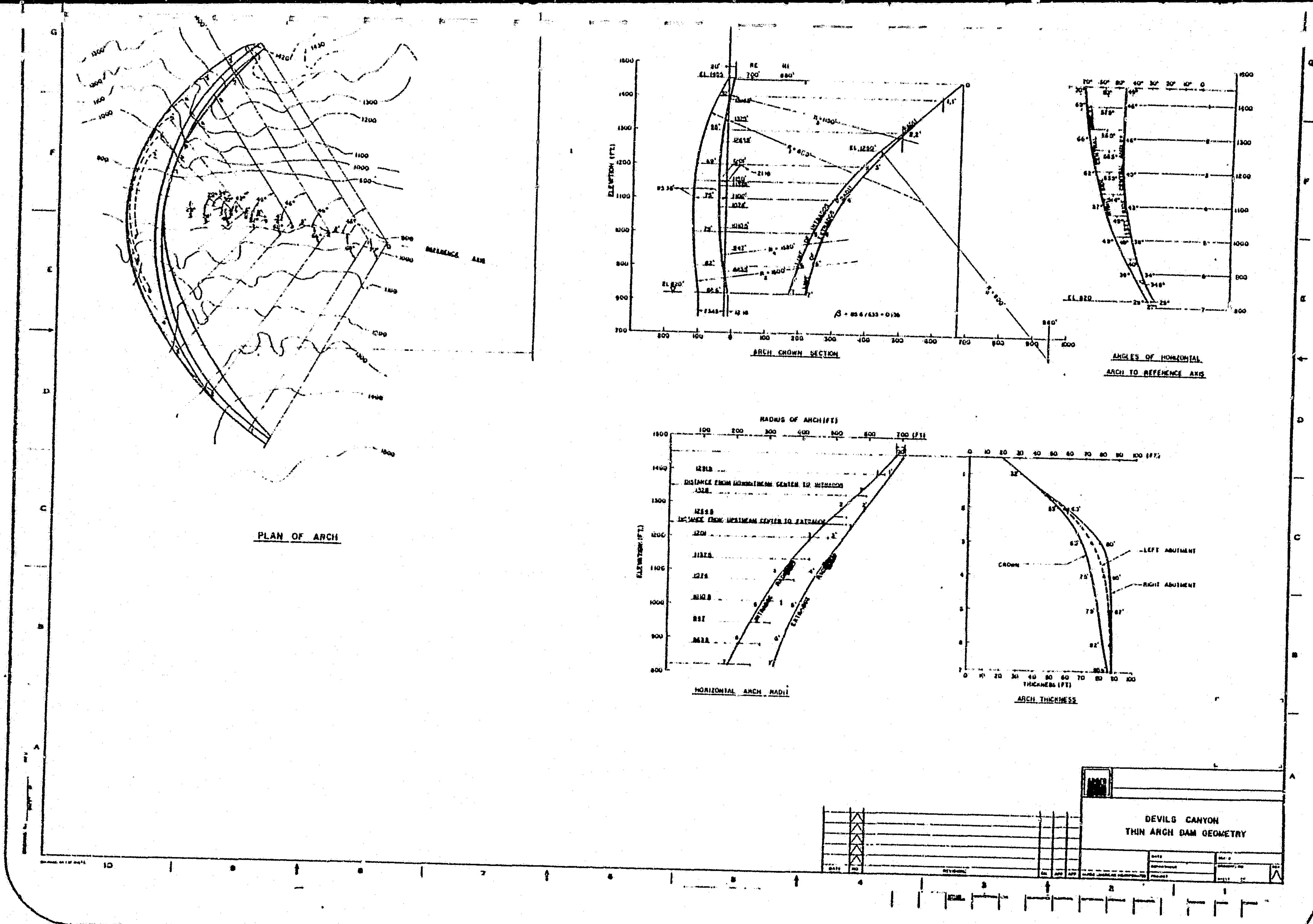

Thin Arch Geometry

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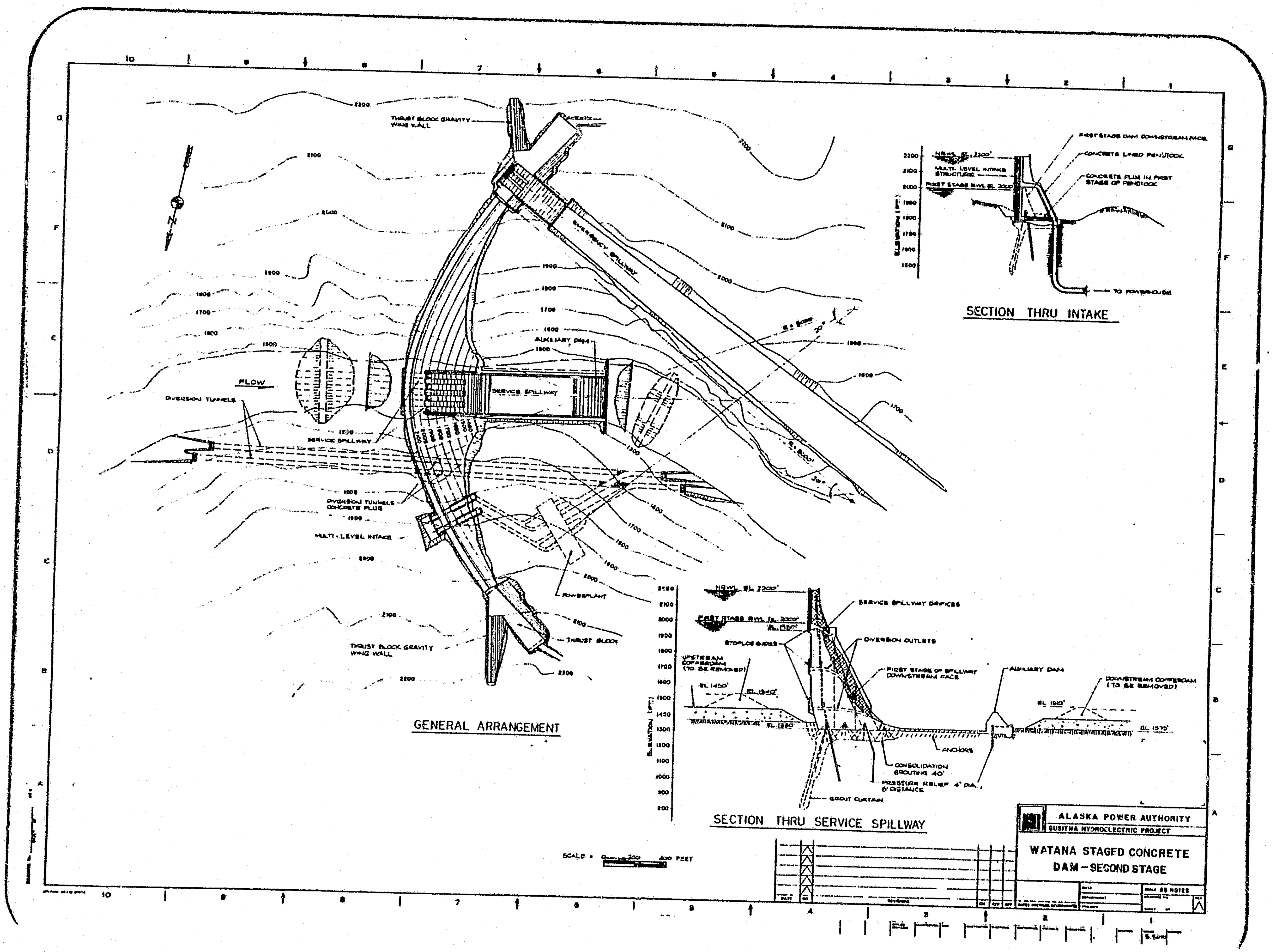
THRUST BLOCK GRAVITY WING WALL
EMERGENCY SPILLWAY
FLOW
DIVERSION TUNNELS
SERVICE SPILLWAY
AUXILIARY DAM
DIVERSION TUNNELS CONCRETE PLUG
MULTI-LEVEL INTAKE
POWERPLANT
THRUST BLOCK
GENERAL ARRANGEMENT
SECTION THRU INTAKE
FIRST STAGE DAM DOWNSTREAM FACE
CONCRETE LINED PENSTOCK
CONCRETE PLUG IN FIRST STAGE OF PENSTOCK
MULTI-LEVEL INTAKE STRUCTURE
TO POWERHOUSE
ELEVATION (FT.)
SECTION THRU SERVICE SPILLWAY
NGWL EL 2200'
SERVICE SPILLWAY ORIFICES
DIVERSION OUTLETS
STOPLOG GUIDES
UPSTREAM COFFERDAM (TO BE REMOVED)
FIRST STAGE OF SPILLWAY DOWNSTREAM FACE
AUXILIARY DAM
DOWNSTREAM COFFERDAM (TO BE REMOVED)
ANCHORS
CONSOLIDATION GROUTING 40'
PRESSURE RELIEF 4' DIA.
GROUT CURTAIN
SCALE 0 200 400 FEET
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
WATANA STAGED CONCRETE DAM - SECOND STAGE

ATTACHMENT 9

REPORT OF VISIT TO
ACRES AMERICAN INCORPORATED, BUFFALO, N.Y.

SUSITNA PROJECT, ALASKA

December 8-11, 1980

Merlin D. Copen, P.E.

During the morning of December 9, I presented a discussion of arch dam analysis and design to members of Acres American staff. Slides and sketches were used to illustrate various types of arch dams and analyses used in the design of arch dams. I particularly emphasized the use of the Trial Load Method of Arch Dam Analysis and evidence to support the reliability of this method.

In the afternoon of December 9, members of Acres staff reviewed the work accomplished at the Devil Canyon site for an arch dam design. Two designs, a thick arch and a thin arch, had been analyzed using finite element procedures based on the computer program ADAP. Details of loading and the results of stress studies were discussed. The results of these studies indicated stresses which were not consistent with those normally found in such structures. I discussed my concerns with Messrs Lawrence and Hayden and suggested that I study the results presented during the evening and then suggest an approach for continued arch dam design.

On Wednesday morning, December 10, I suggested the following procedures:

1. Proceed immediately to get ADSAS (the computerized version of the Trial Load Method) in operation.

2. Become familiar with the method required to estimate temperatures of the concrete in the dam. The

computer program "Heat Flow" should be used to obtain the necessary temperature loads for the ADSAS computer program.

3. To expedite the design for Devil Canyon Dam, concentrate on only one design. This could have a top thickness of 20 feet and a base thickness at the crown cantilever of about 90 feet. The overhang and undercut should be similar to that used in the present thin arch design. The crown section should have approximately the same thickness from its midpoint to its base. The central angle for the arch at its axis should be between 100° and 110°. Arches should be circular with some thickening toward the abutments.

4. Concrete compressive strength should be 5,000 psi at 365 days. A factor of safety of 4.0 based on compressive stress is suggested. An estimated concrete tensile strength of 250 psi for static loads and 600 psi for dynamic loads is suggested. A sustained modulus for concrete of 3,000,000 psi and a deformation modulus of 2,000,000 psi for abutment rock are suggested.

5. Stress analyses and design changes should be based on normal full reservoir, minimum usual temperature loads, ice and silt loads as appropriate. Temperature loads should be assumed to be uniform throughout each arch.

6. No attempt should be made to apply dynamic loads until a satisfactory design has been obtained for the above criteria. This is true also for other loading conditions which should eventually be applied to assess the adequacy of the dam.

During Wednesday afternoon the possibility of an arch dam design at the Watana site was discussed. I suggested consideration of a three-centered arch in the top portion of the site, blending into a single centered arch in the lower portion. Stage construction, grouting procedures, spillways and plunge pools were discussed in a general way.

Merlin D. Copen