1381

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SUSITNA HYDROELECTRIC PROJECT

EXTERNAL REVIEW BOARD

MEETING #4

INFORMATION PACKAGE

JANUARY 12 - 13, ~~1981~~ 1981

Prepared by:

ACRES

ALASKA POWER AUTHORITY

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ALASKA POWER AUTHORITY

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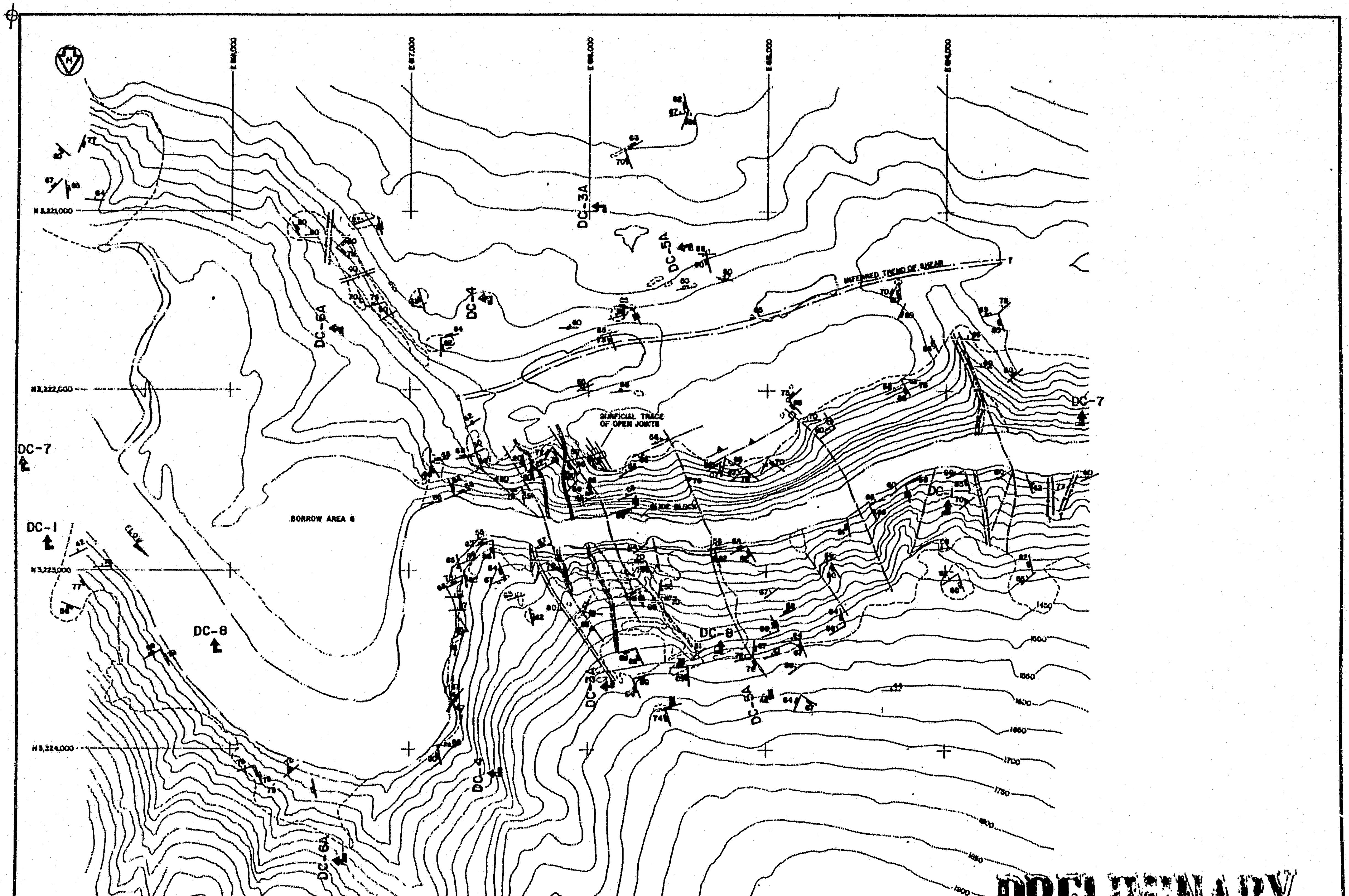
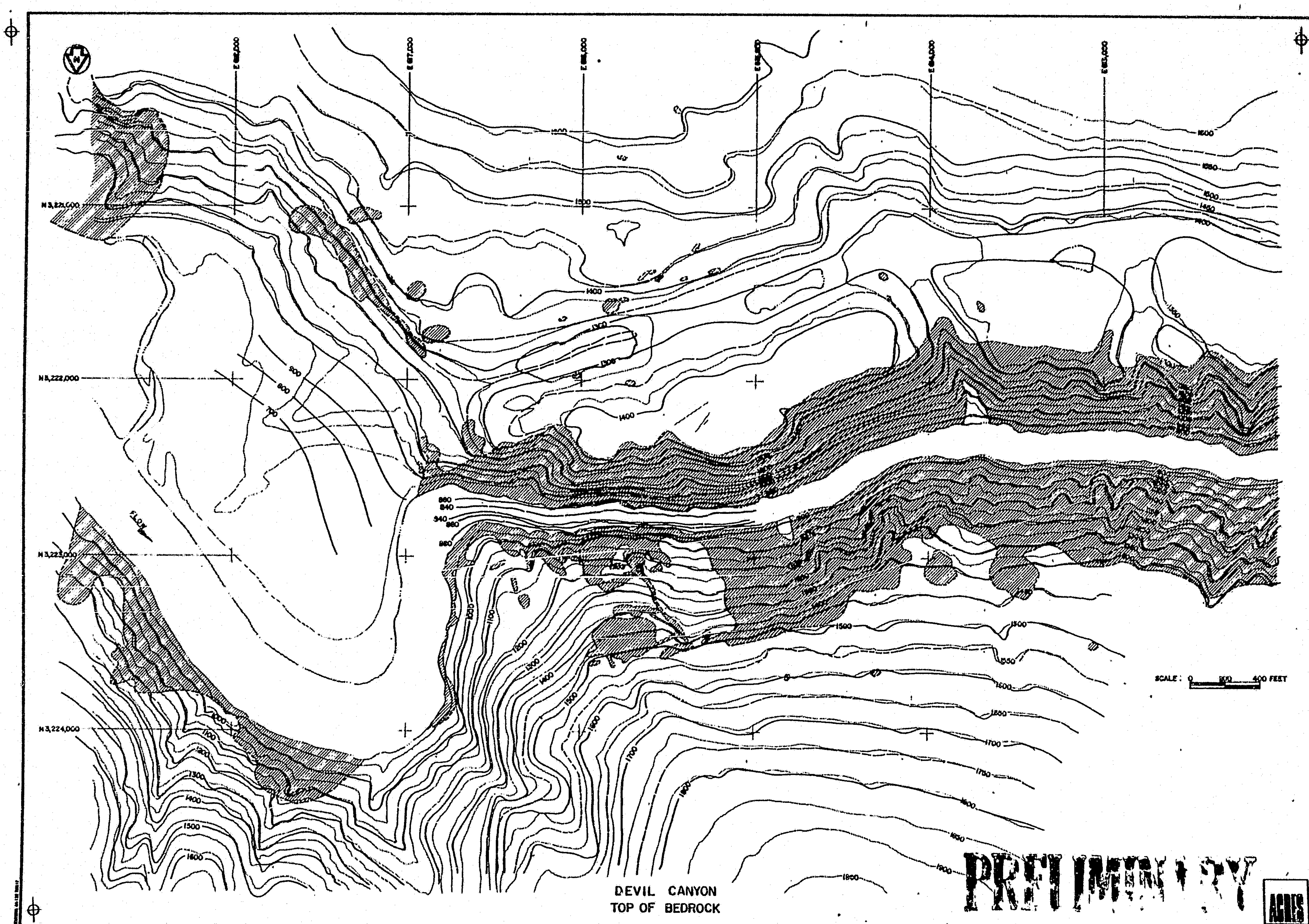
TASK 5

00138

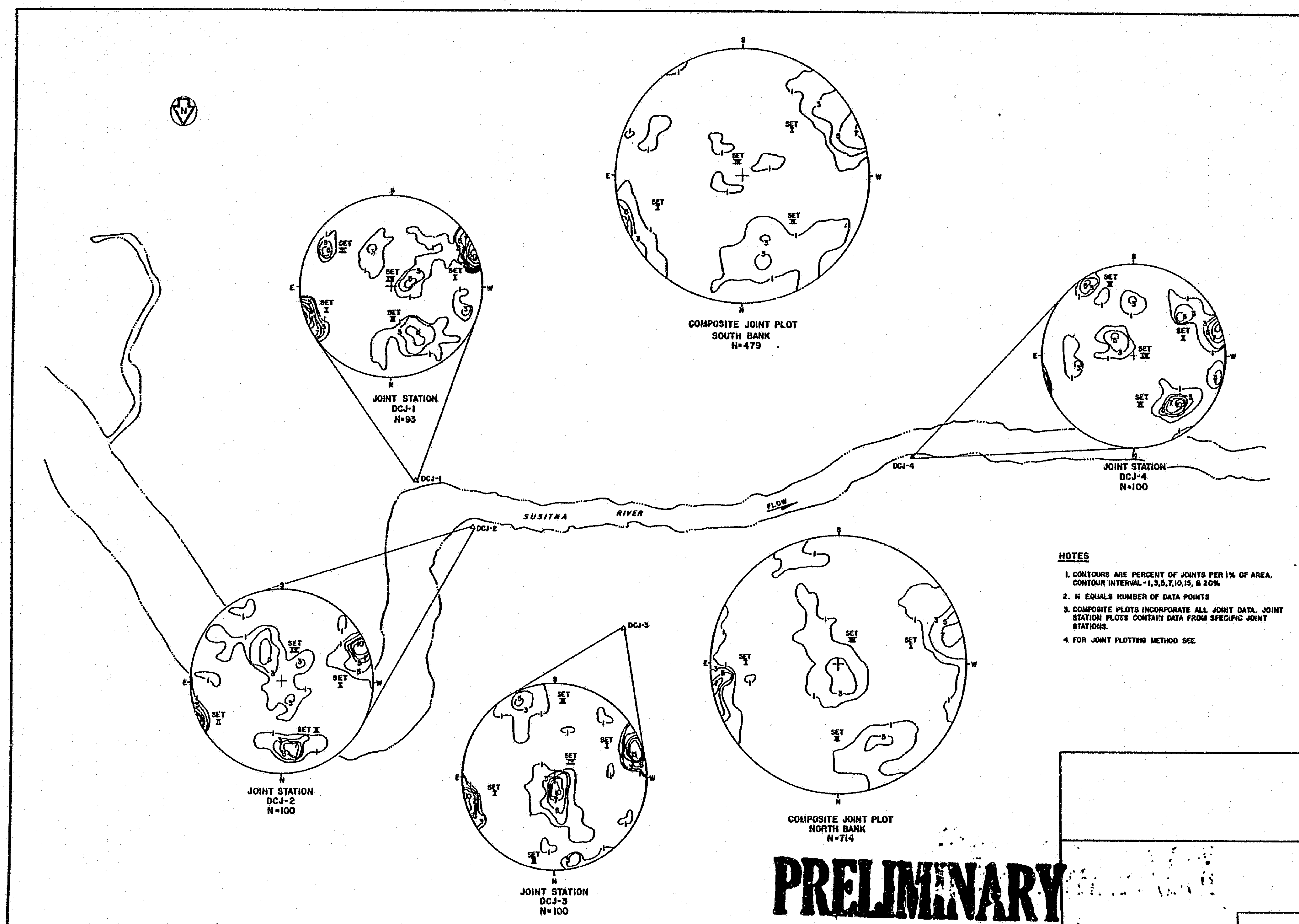


DEVIL CANYON
GEOLOGIC MAP

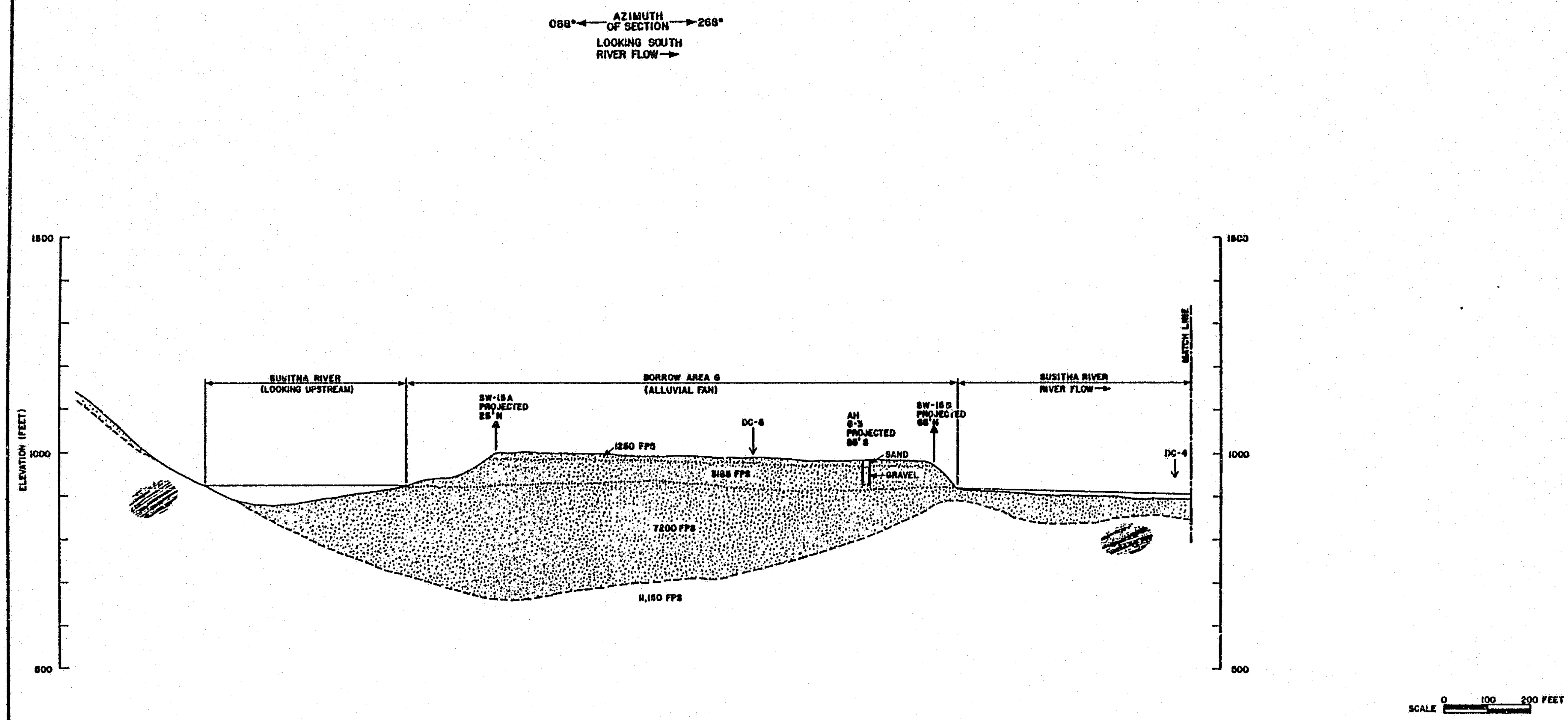
PRELIMINARY ACRES



DEVIL CANYON
TOP OF BEDROCK



COMPOSITE JOINT PLOT
SOUTH BANK
N=479
JOINT STATION
DCJ-1
N=93
JOINT STATION
DCJ-4
N=100
DCJ-1
DCJ-2
DCJ-3
DCJ-4
SUSITNA RIVER
FLOW
JOINT STATION
DCJ-2
N=100
JOINT STATION
DCJ-3
N=100
COMPOSITE JOINT PLOT
NORTH BANK
N=714
NOTES
1. CONTOURS ARE PERCENT OF JOINTS PER 1% OF AREA. CONTOUR INTERVAL - 1,3,5,7,10,15, & 20%
2. N EQUALS NUMBER OF DATA POINTS
3. COMPOSITE PLOTS INCORPORATE ALL JOINT DATA. JOINT STATION PLOTS CONTAIN DATA FROM SPECIFIC JOINT STATIONS.
4. FOR JOINT PLOTTING METHOD SEE
PRELIMINARY



DEVIL CANYON
GEOLOGIC SECTION DC-1
SHEET 1 OF 2

PRELIMINARY

ACRES

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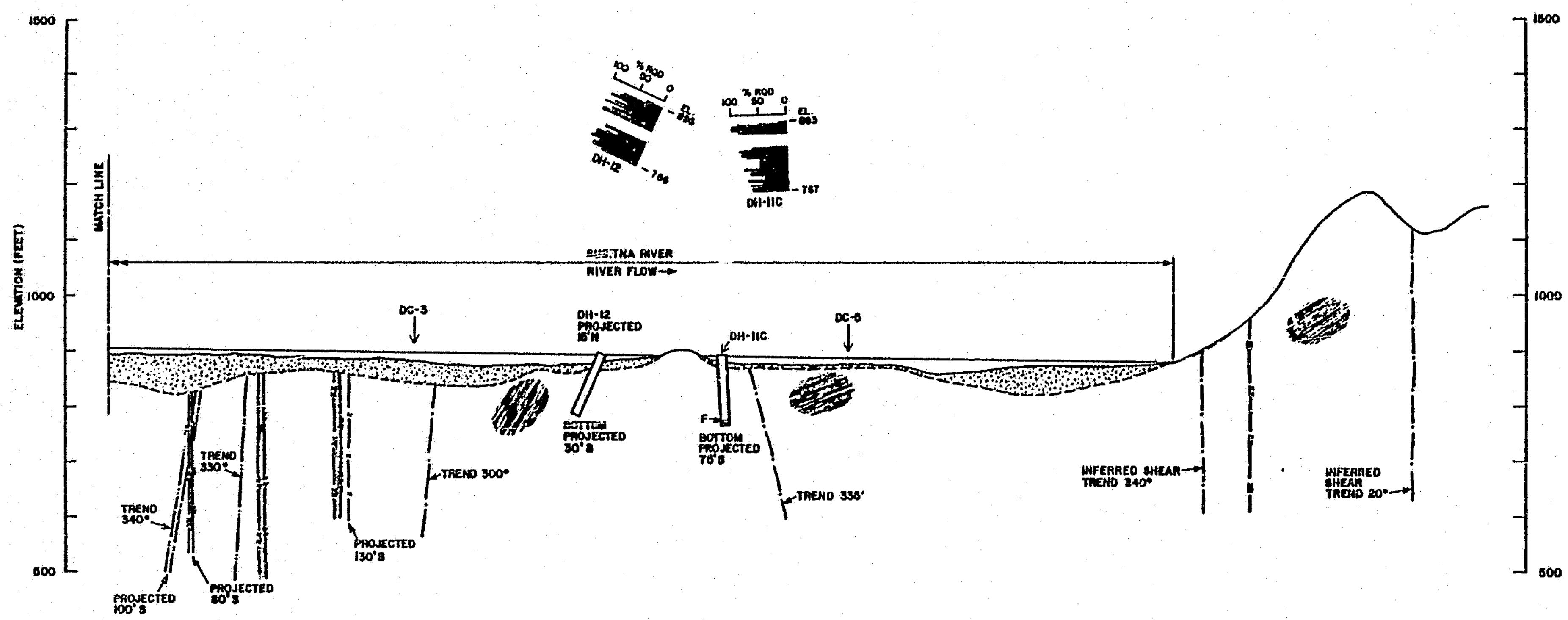
LOOKING SOUTH
RIVER FLOW →



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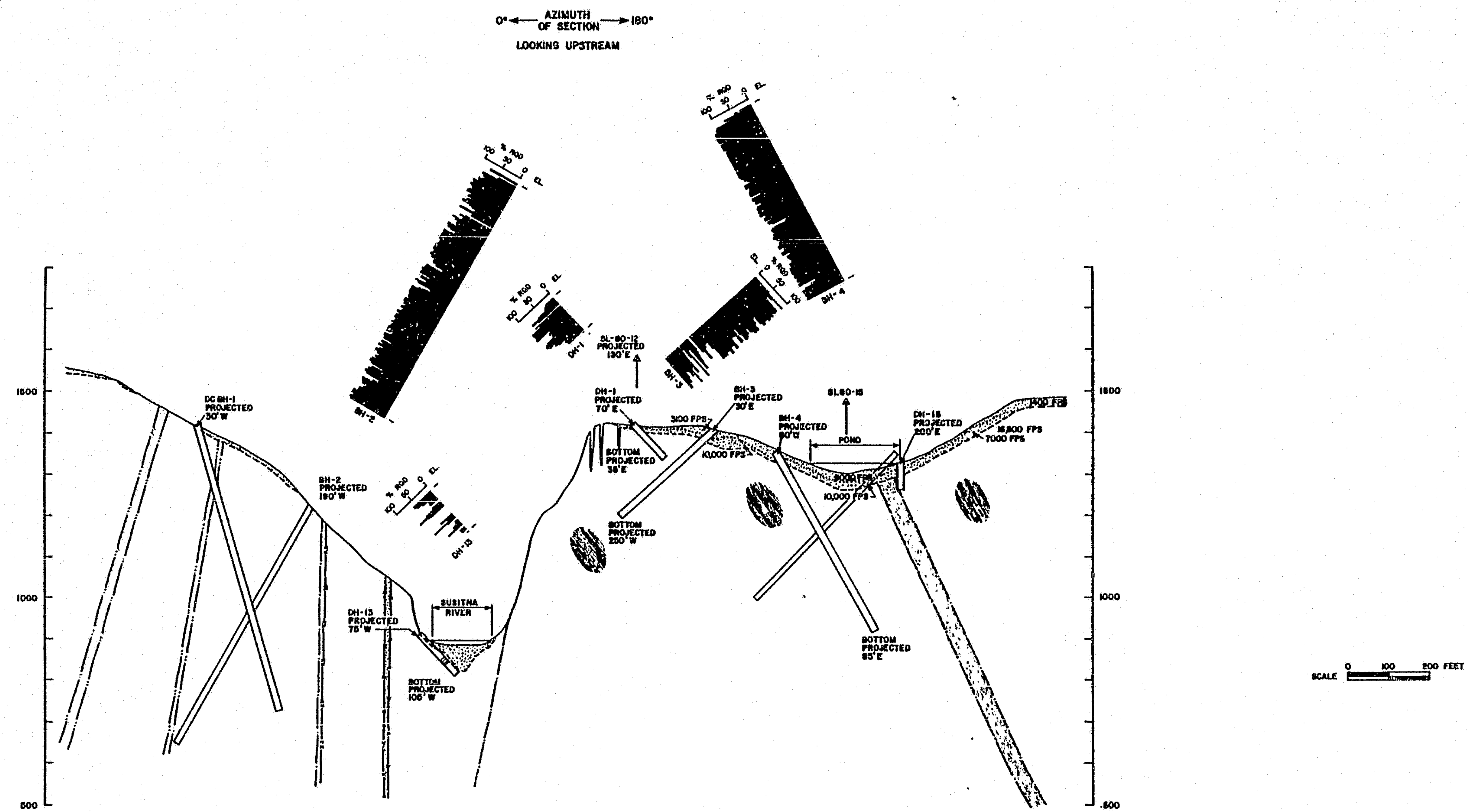
DEVIL CANYON
GEOLOGIC SECTION DC-1
SHEET 2 OF 2

PRELIMINARY



DEVIL CANYON

GEOLOGIC SECTION DC-3A

PRELIMINARY

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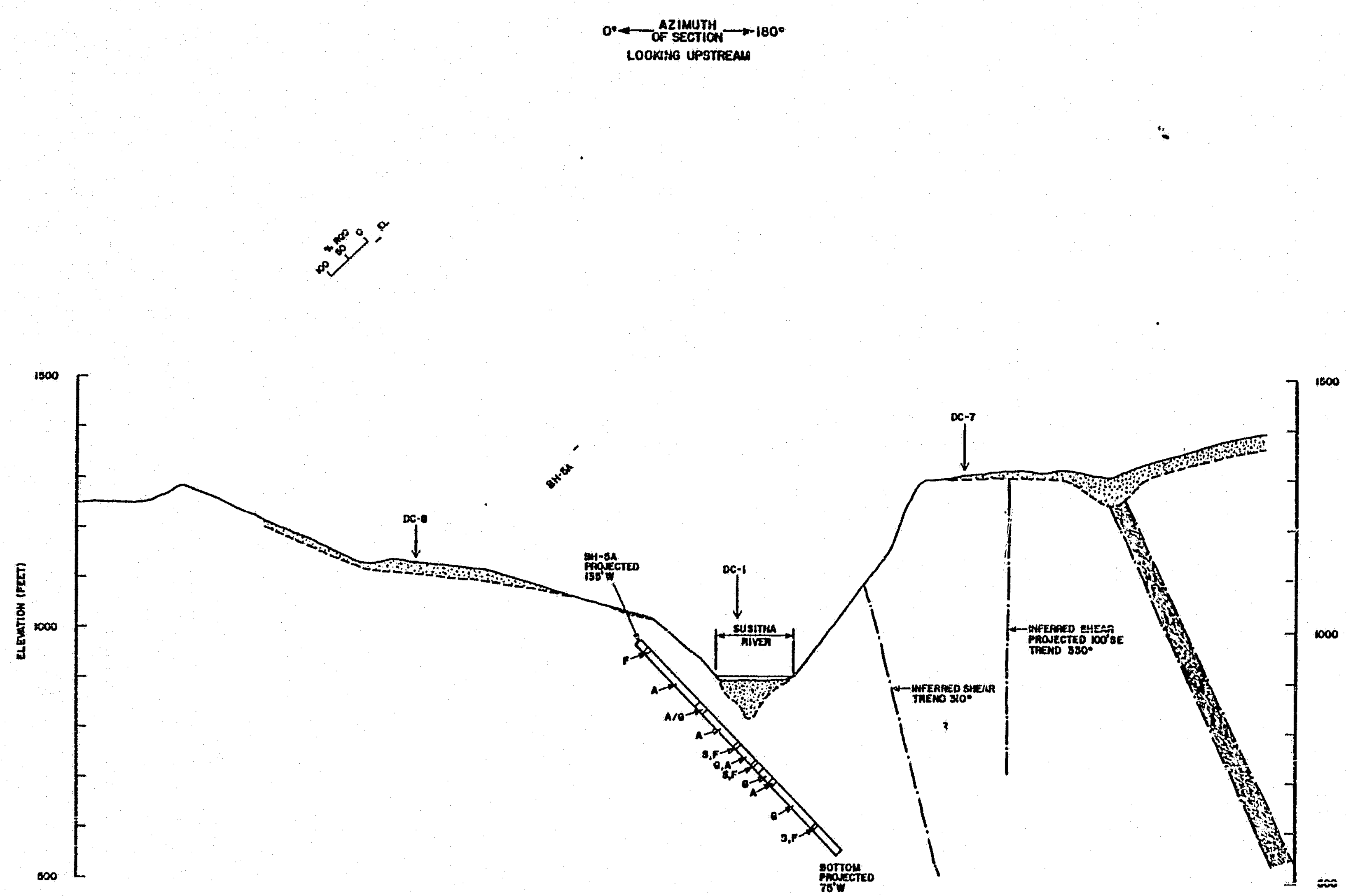
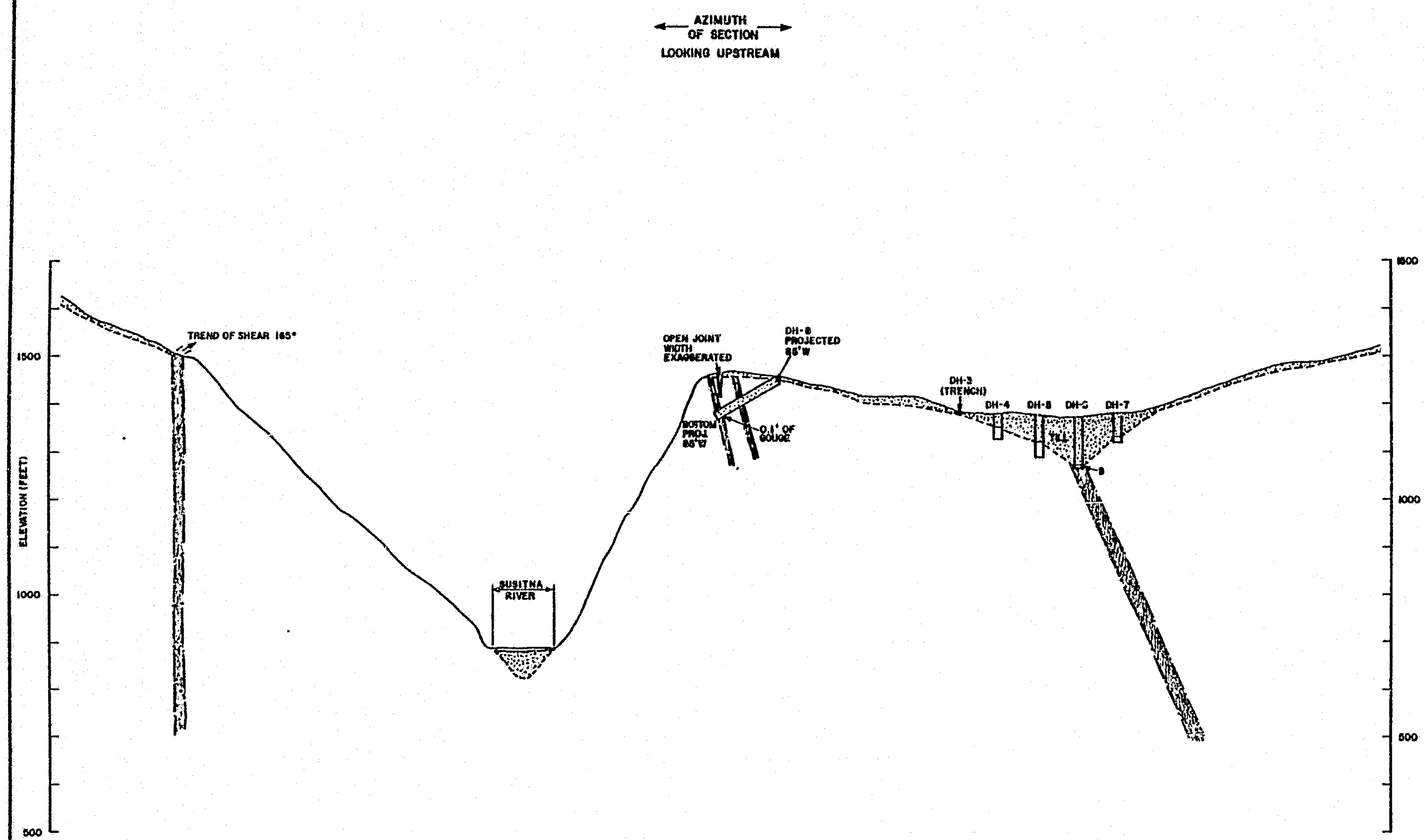


DEVIL CANYON

GEOLOGIC SECTION DC-4

PRELIMINARY



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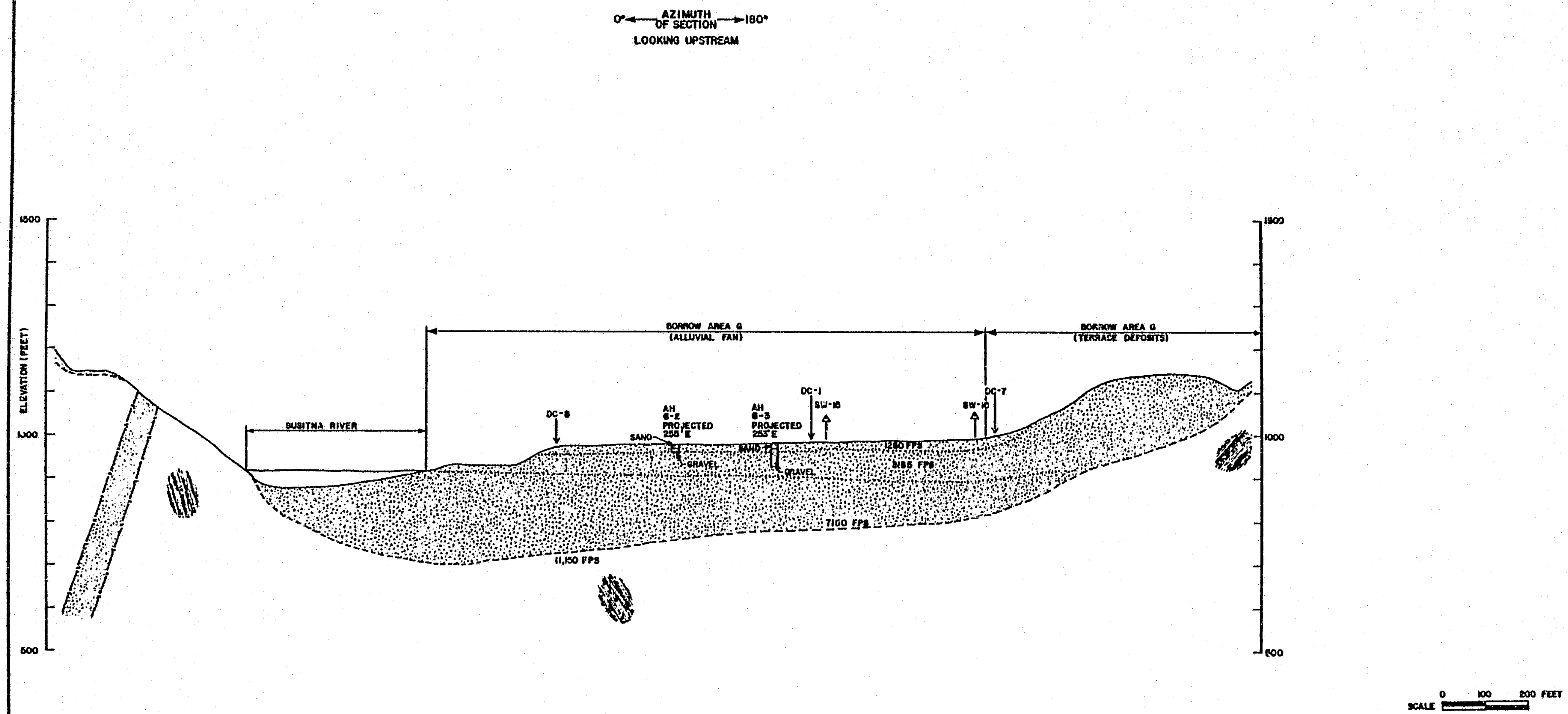
GEOLOGIC SECTION DC-5A
SHEET 1 OF 1

PRELIMINARY

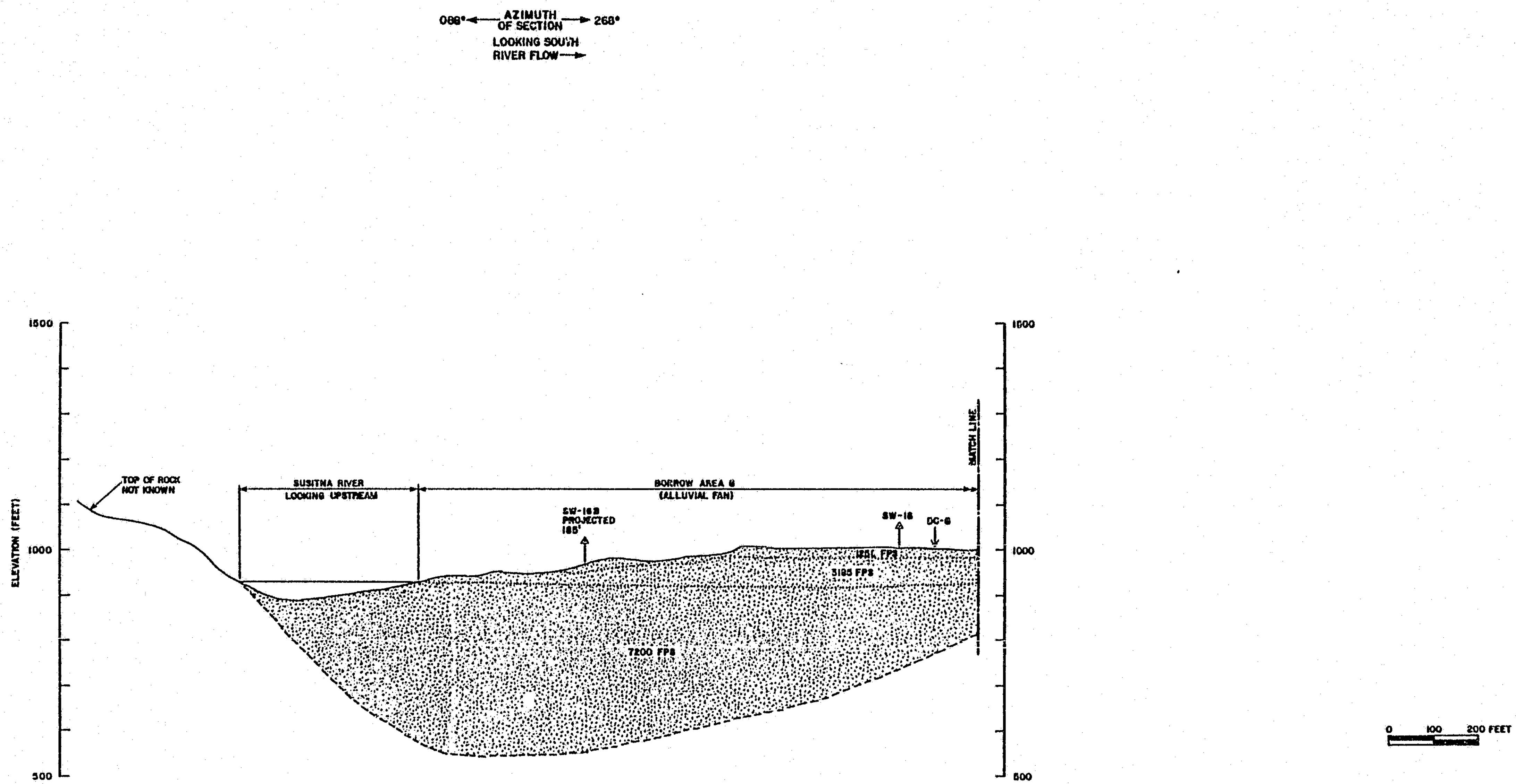
DC-5A
1 OF 1



DEVIL CANYON
GEOLOGIC SECTION DC-6A

PRELIMINARY

ACRES



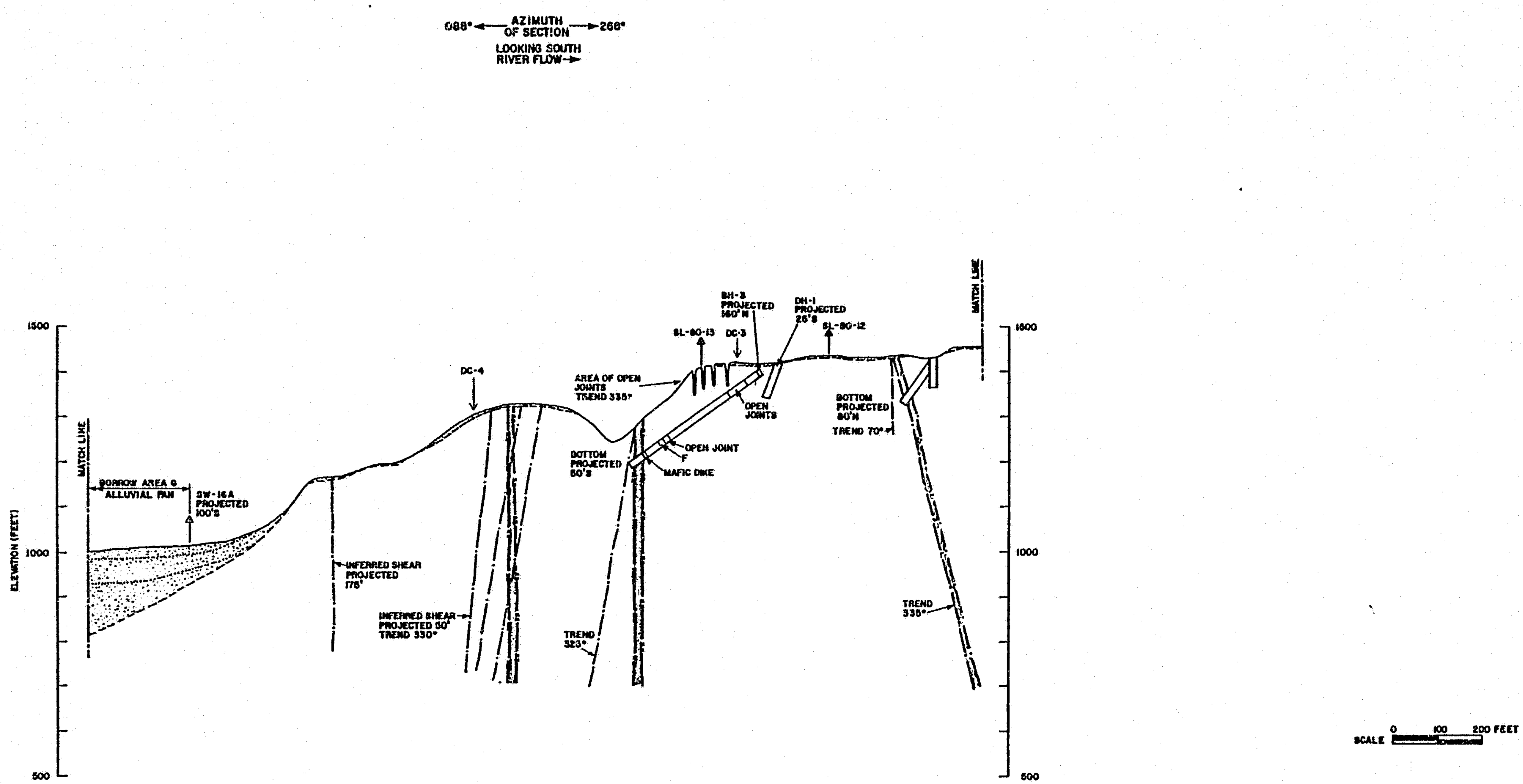
DEVIL CANYON

GEOLOGIC SECTION DC-7

SHEET 1 OF 3

PRELIMINARY

ACRES



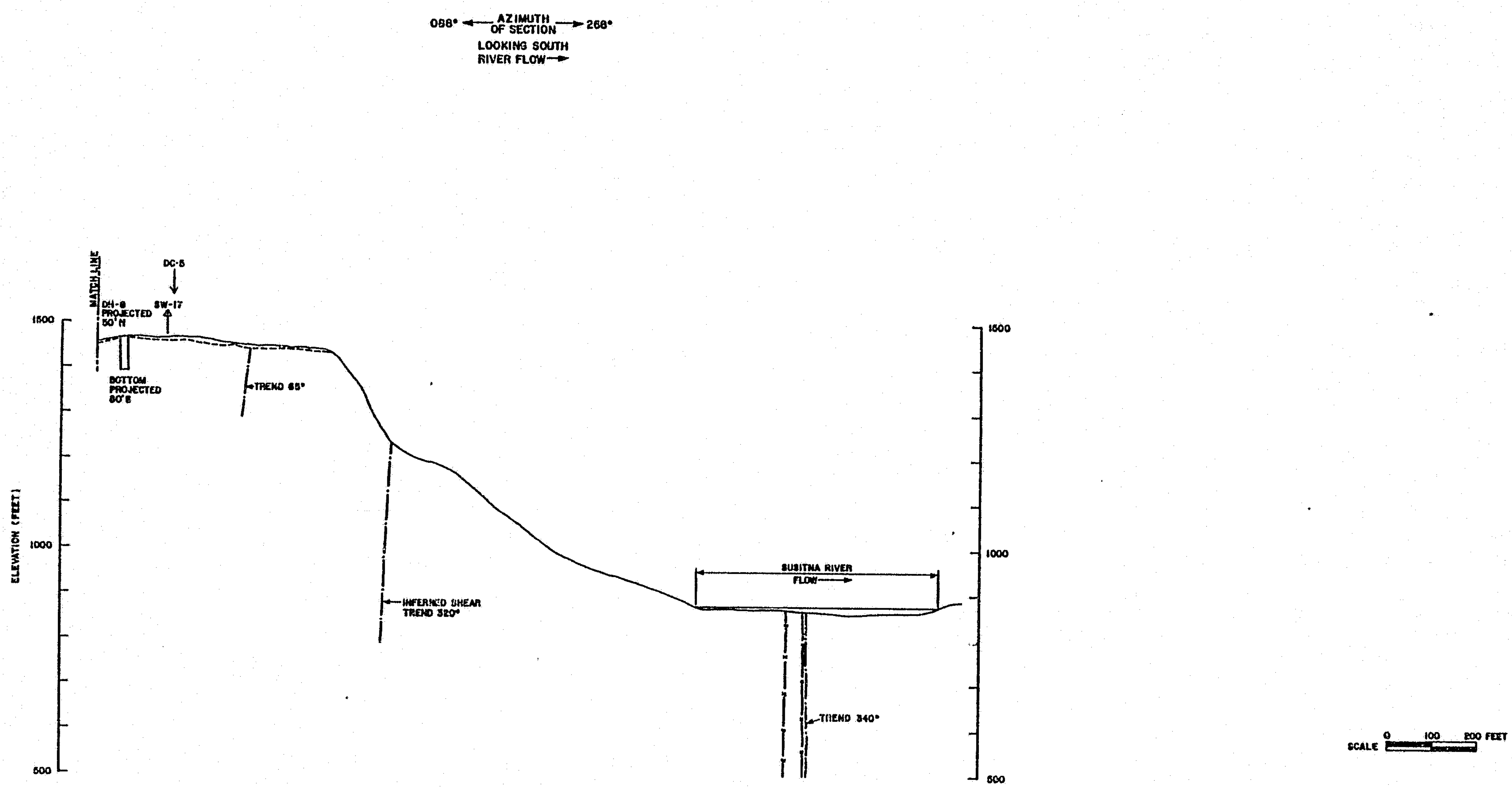
DEVIL CANYON

GEOLOGIC SECTION DC-7

SHEET 2 OF 3

PRELIMINARY

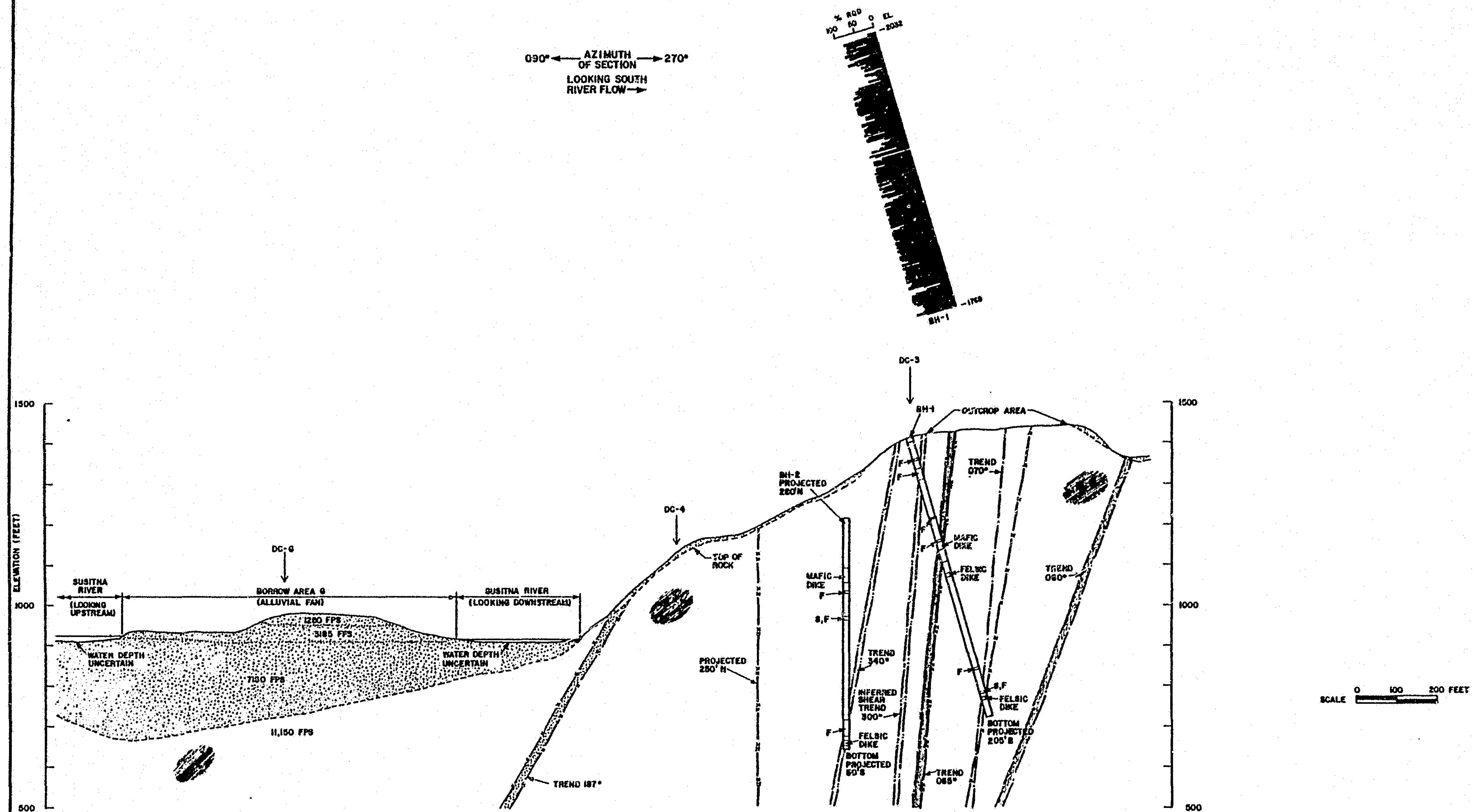
ACRES



DEVIL CANYON
GEOLOGIC SECTION DC-7
SHEET 3 OF 3

PRELIMINARY

ACRES

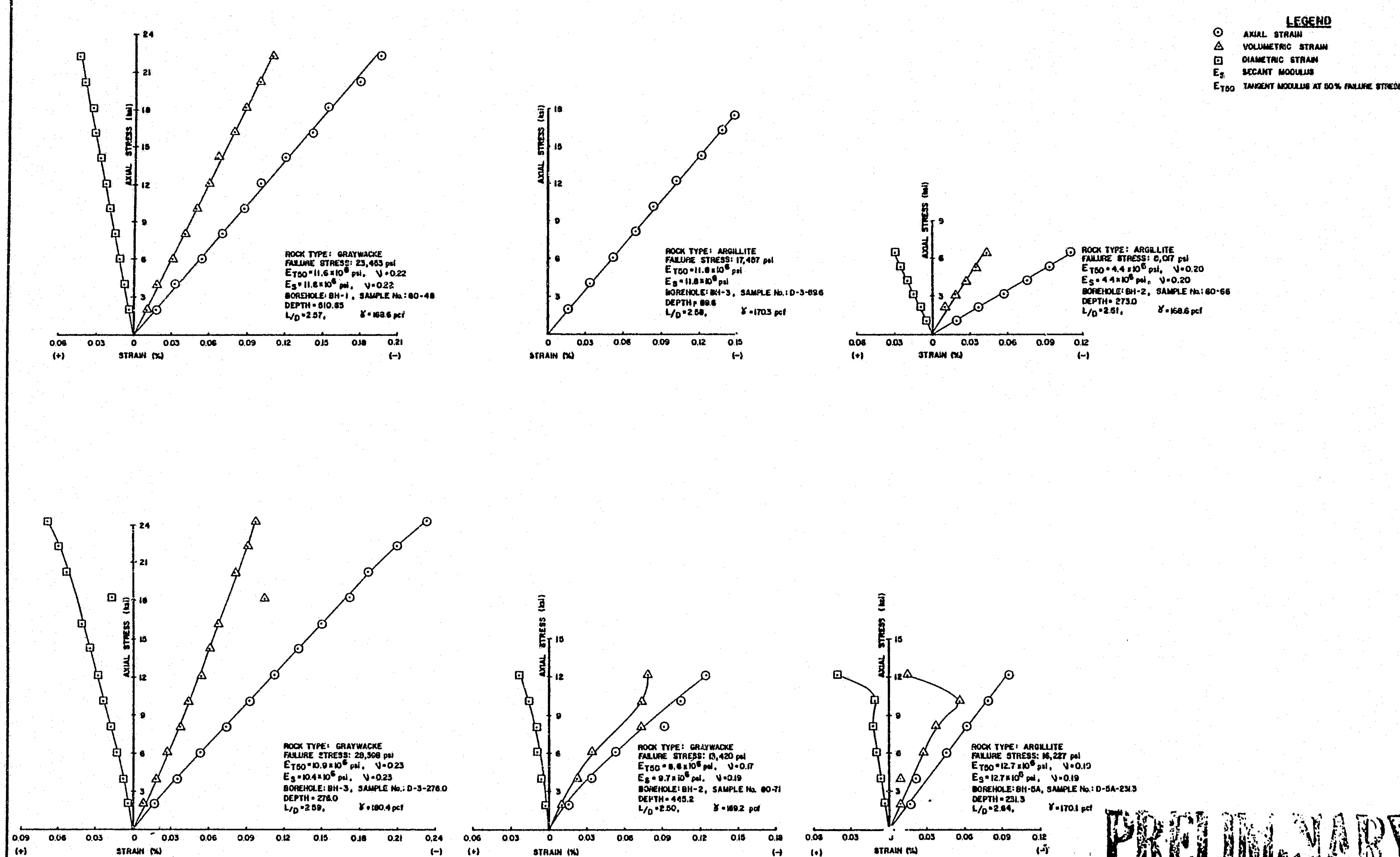


DEVIL CANYON
GEOLOGIC SECTION DC-6

PRELIMINARY

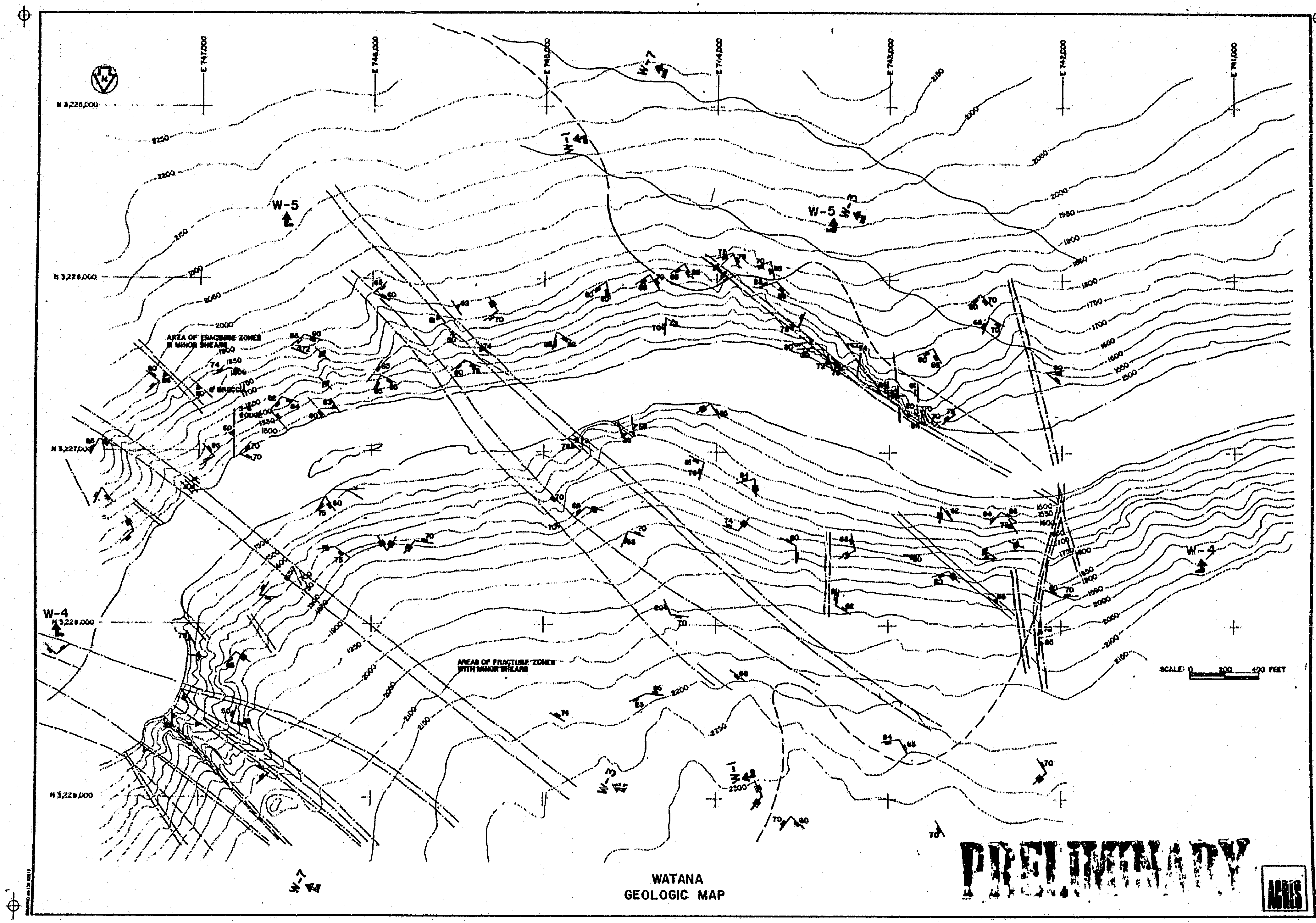
ACRES

00138



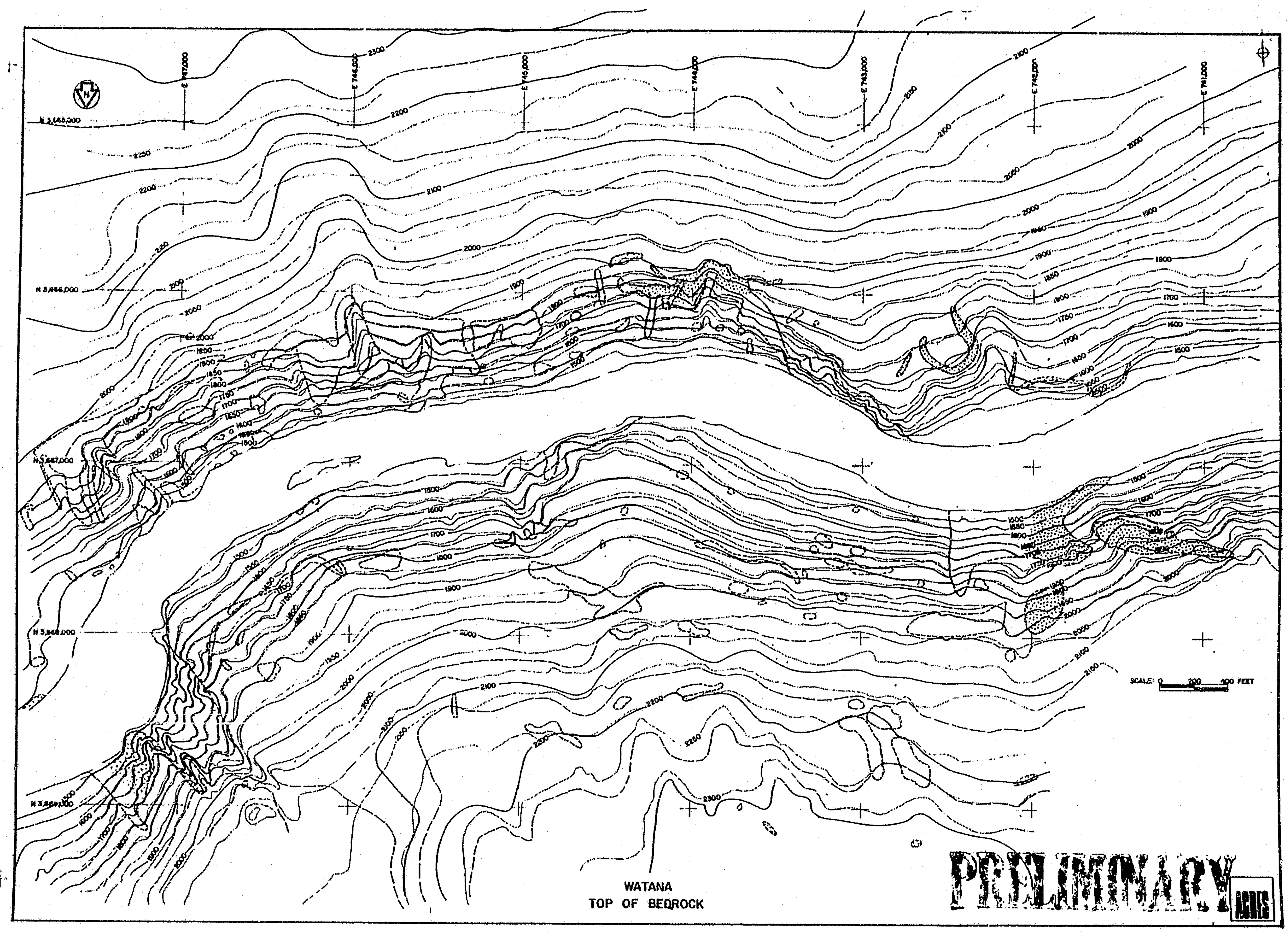
PRELIMINARY

DEVIL CANYON
STRESS – STRAIN CURVES



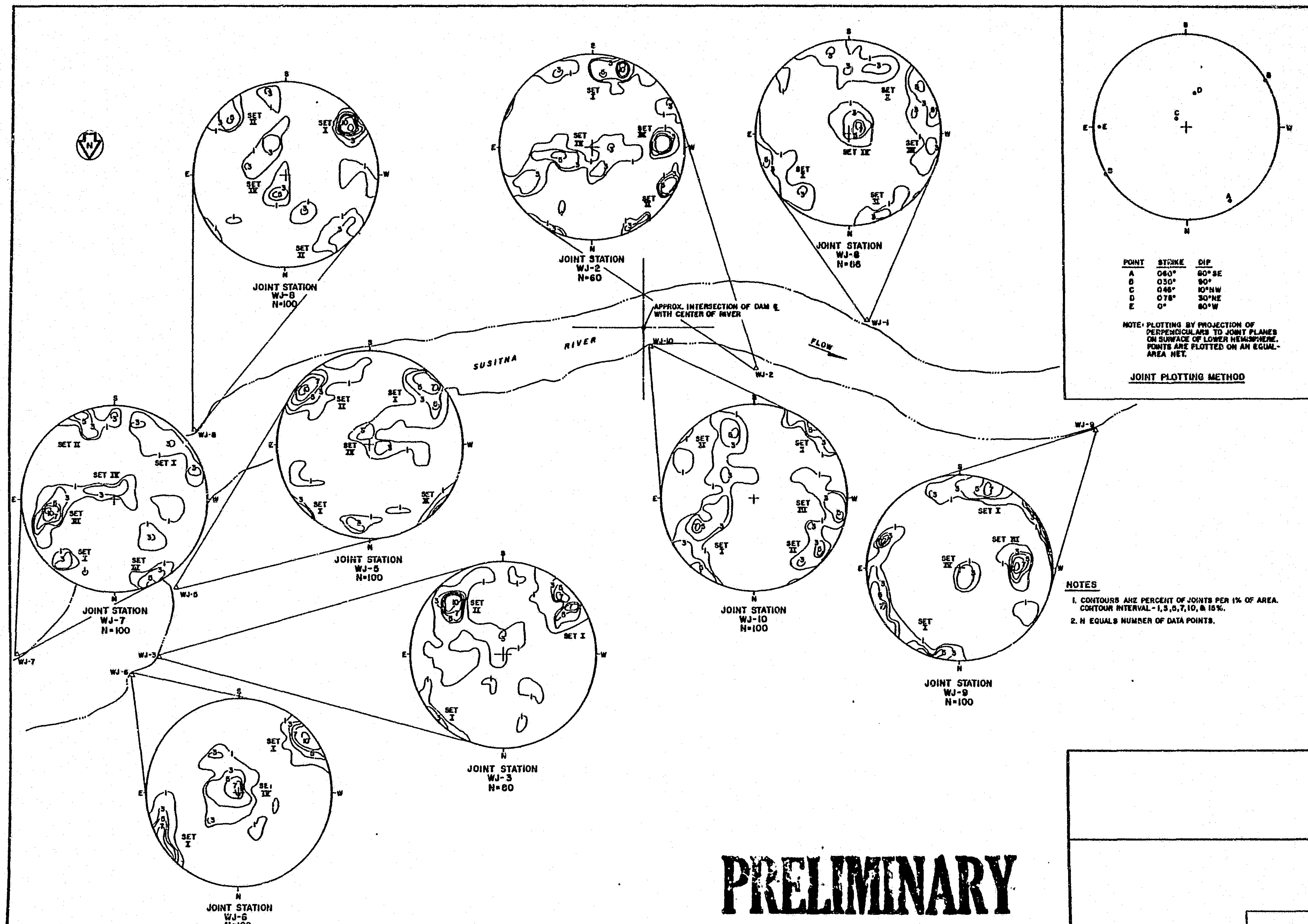
WATANA
GEOLOGIC MAP

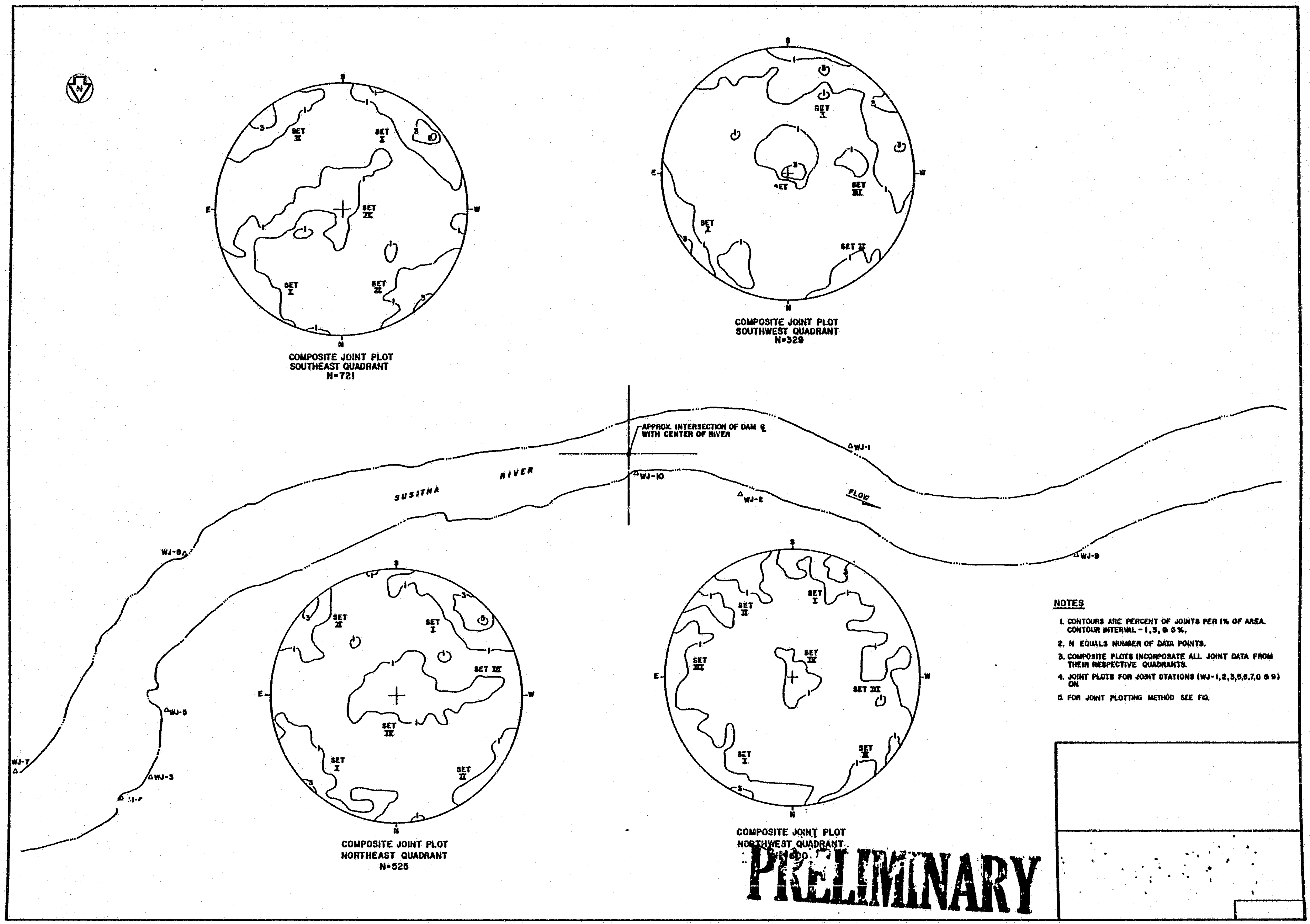
00138



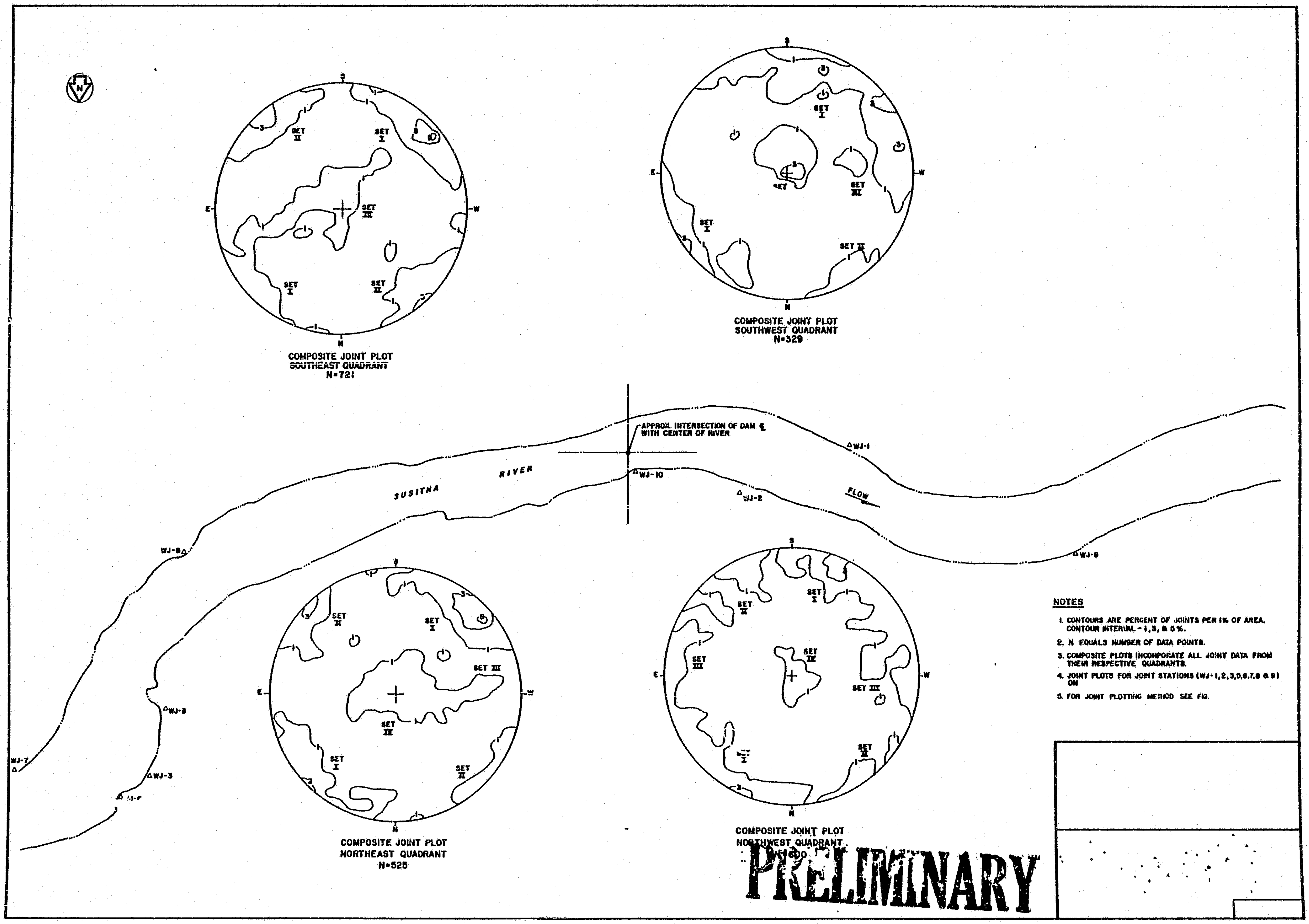
WATANA
TOP OF BEDROCK

PRELIMINARY


JOINT STATION
WJ-8
N=100
JOINT STATION
WJ-2
N=60
JOINT STATION
WJ-8
N=66
APPROX. INTERSECTION OF DAM ℄ WITH CENTER OF RIVER
SUSITNA RIVER
FLOW
WJ-1
WJ-2
WJ-10
WJ-9
WJ-8
WJ-5
WJ-7
WJ-3
WJ-6
JOINT STATION
WJ-7
N=100
JOINT STATION
WJ-5
N=100
JOINT STATION
WJ-10
N=100
JOINT STATION
WJ-9
N=100
JOINT STATION
WJ-3
N=60
JOINT STATION
WJ-6
N=100
POINT STRIKE DIP
A 060° 80° SE
B 030° 90°
C 045° 10°NW
D 078° 30°NE
E 0° 60°W
NOTE: PLOTTING BY PROJECTION OF PERPENDICULARS TO JOINT PLANES ON SURFACE OF LOWER HEMISPHERE. POINTS ARE PLOTTED ON AN EQUAL-AREA NET.
JOINT PLOTTING METHOD
NOTES
1. CONTOURS ARE PERCENT OF JOINTS PER 1% OF AREA. CONTOUR INTERVAL - 1, 3, 5, 7, 10, & 15%.
2. N EQUALS NUMBER OF DATA POINTS.
PRELIMINARY


COMPOSITE JOINT PLOT
SOUTHEAST QUADRANT
N=721
COMPOSITE JOINT PLOT
SOUTHWEST QUADRANT
N=329
APPROX. INTERSECTION OF DAM ℄ WITH CENTER OF RIVER
SUSITNA RIVER
FLOW
WJ-1
WJ-2
WJ-3
WJ-5
WJ-6
WJ-7
WJ-9
WJ-10
COMPOSITE JOINT PLOT
NORTHEAST QUADRANT
N=525
COMPOSITE JOINT PLOT
NORTHWEST QUADRANT
NOTES
1. CONTOURS ARE PERCENT OF JOINTS PER 1% OF AREA. CONTOUR INTERVAL - 1, 3, & 5%.
2. N EQUALS NUMBER OF DATA POINTS.
3. COMPOSITE PLOTS INCORPORATE ALL JOINT DATA FROM THEIR RESPECTIVE QUADRANTS.
4. JOINT PLOTS FOR JOINT STATIONS (WJ-1,2,3,5,6,7,8 & 9) ON
5. FOR JOINT PLOTTING METHOD SEE FIG.
PRELIMINARY

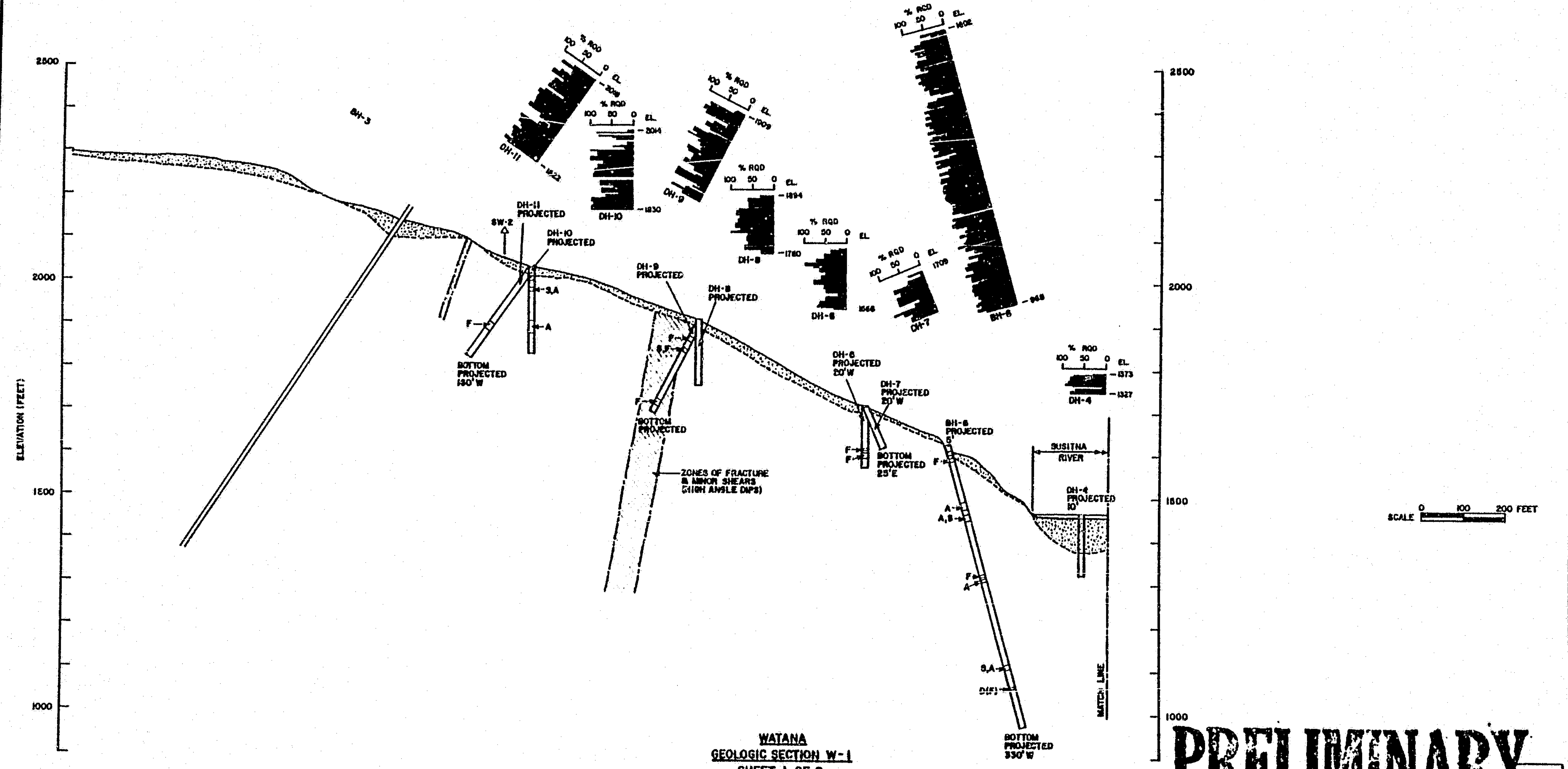
00138



NOTES

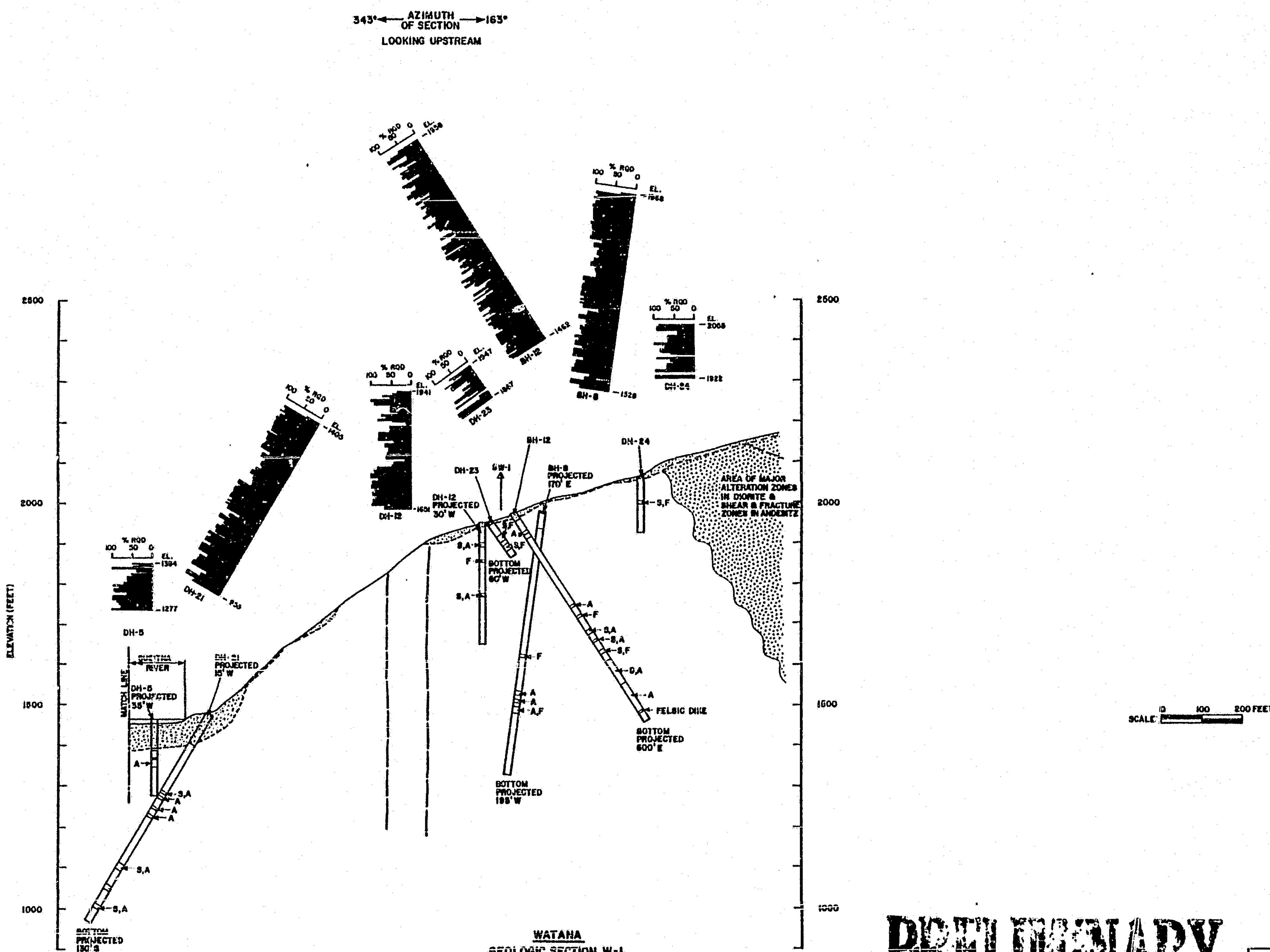
1. CONTOURS ARE PERCENT OF JOINTS PER 1% OF AREA. CONTOUR INTERVAL - 1, 3, & 5%.
2. N EQUALS NUMBER OF DATA POINTS.
3. COMPOSITE PLOTS INCORPORATE ALL JOINT DATA FROM THEIR RESPECTIVE QUADRANTS.
4. JOINT PLOTS FOR JOINT STATIONS (WJ-1,2,3,5,6,7,8 & 9) ON
5. FOR JOINT PLOTTING METHOD SEE FIG.

PRELIMINARY



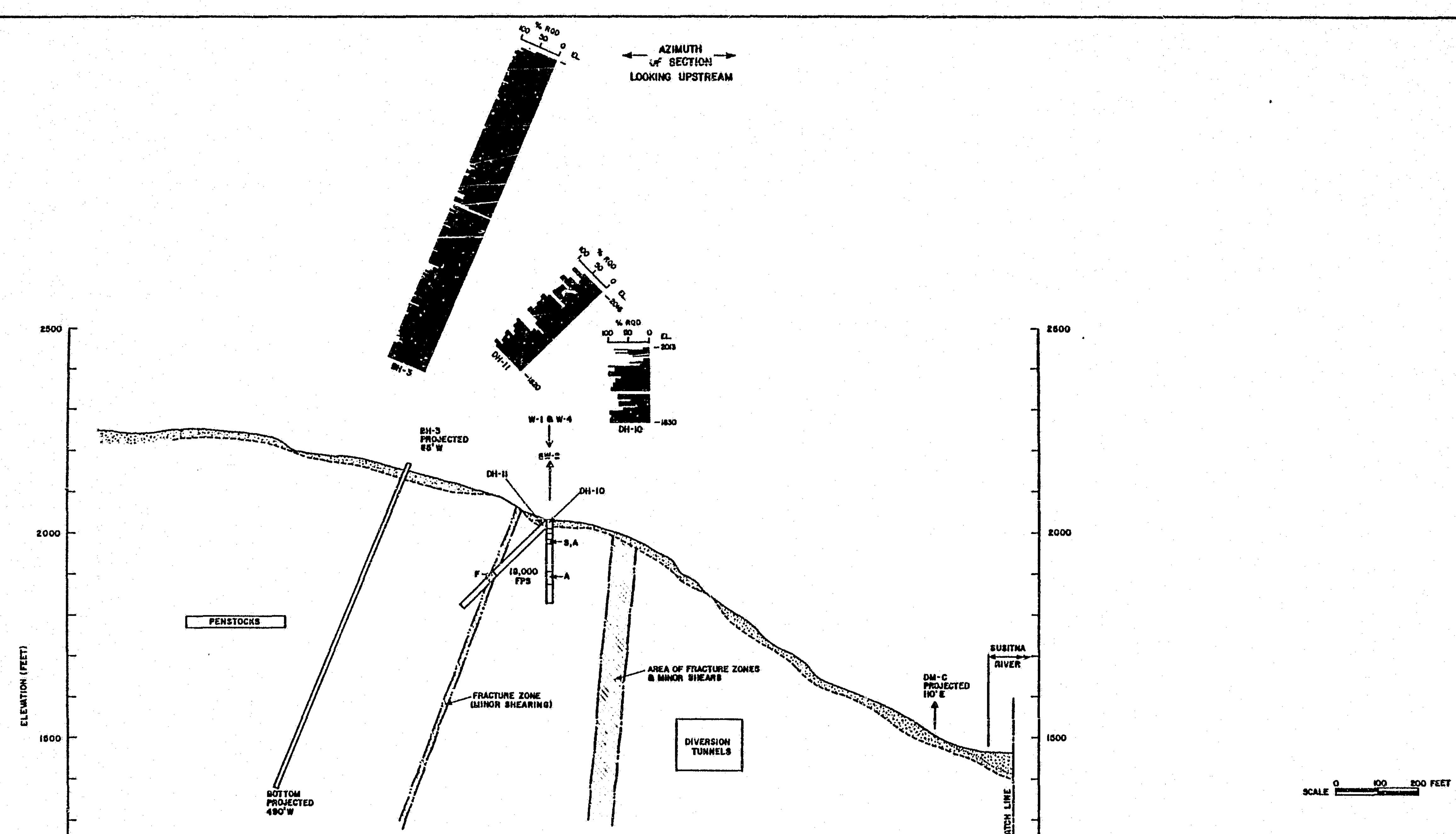
WATANA
GEOLOGIC SECTION W-1
SHEET 1 OF 2

PRELIMINARY



WATANA
GEOLOGIC SECTION W-1
SHEET 2 OF 2

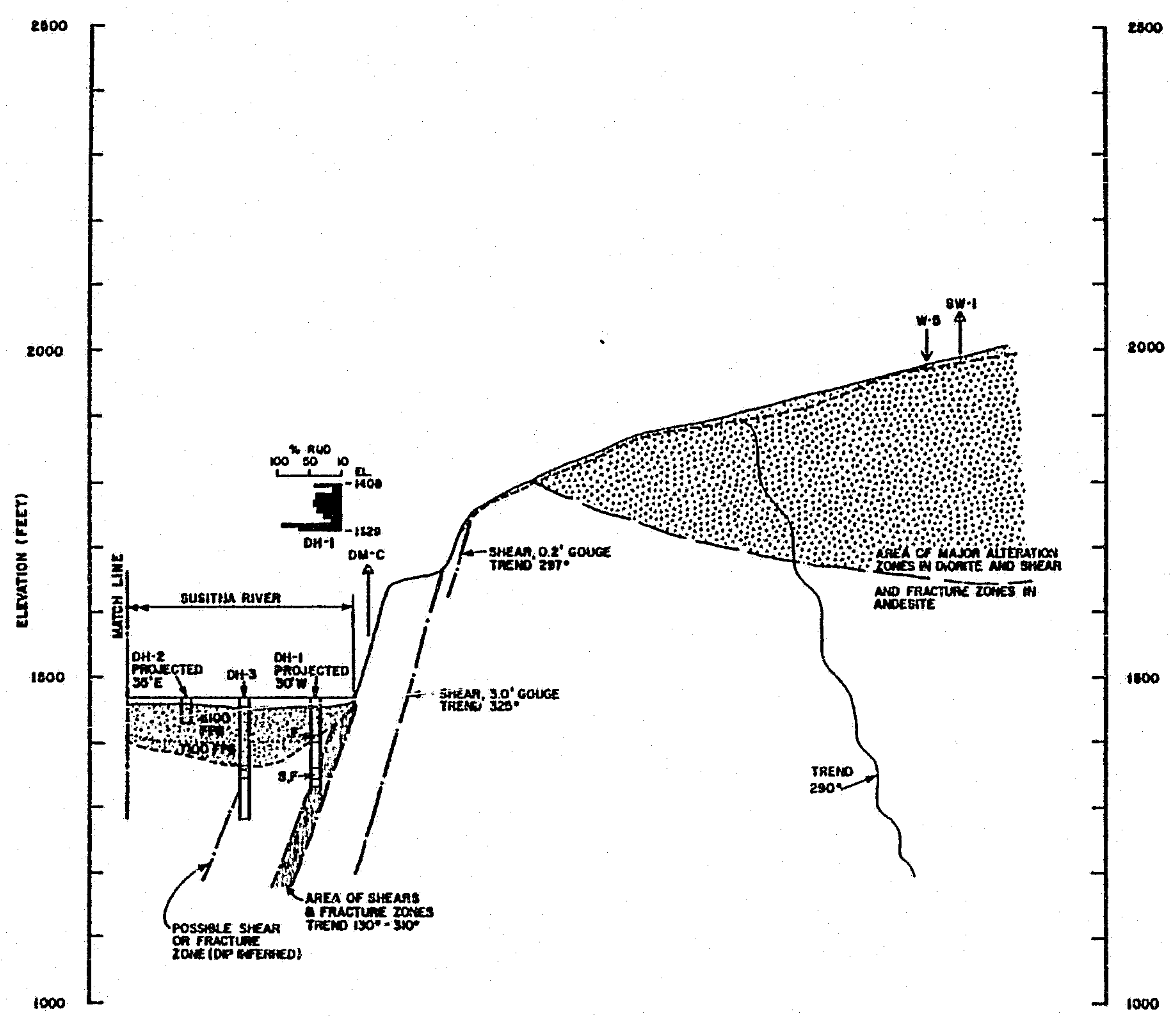
PRELIMINARY



WATANA
GEOLOGIC SECTION W-3
SHEET 1 OF 2

PRELIMINARY ACRES

028° ← AZIMUTH OF SECTION → 208°
LOOKING UPSTREAM
RIVER FLOW →



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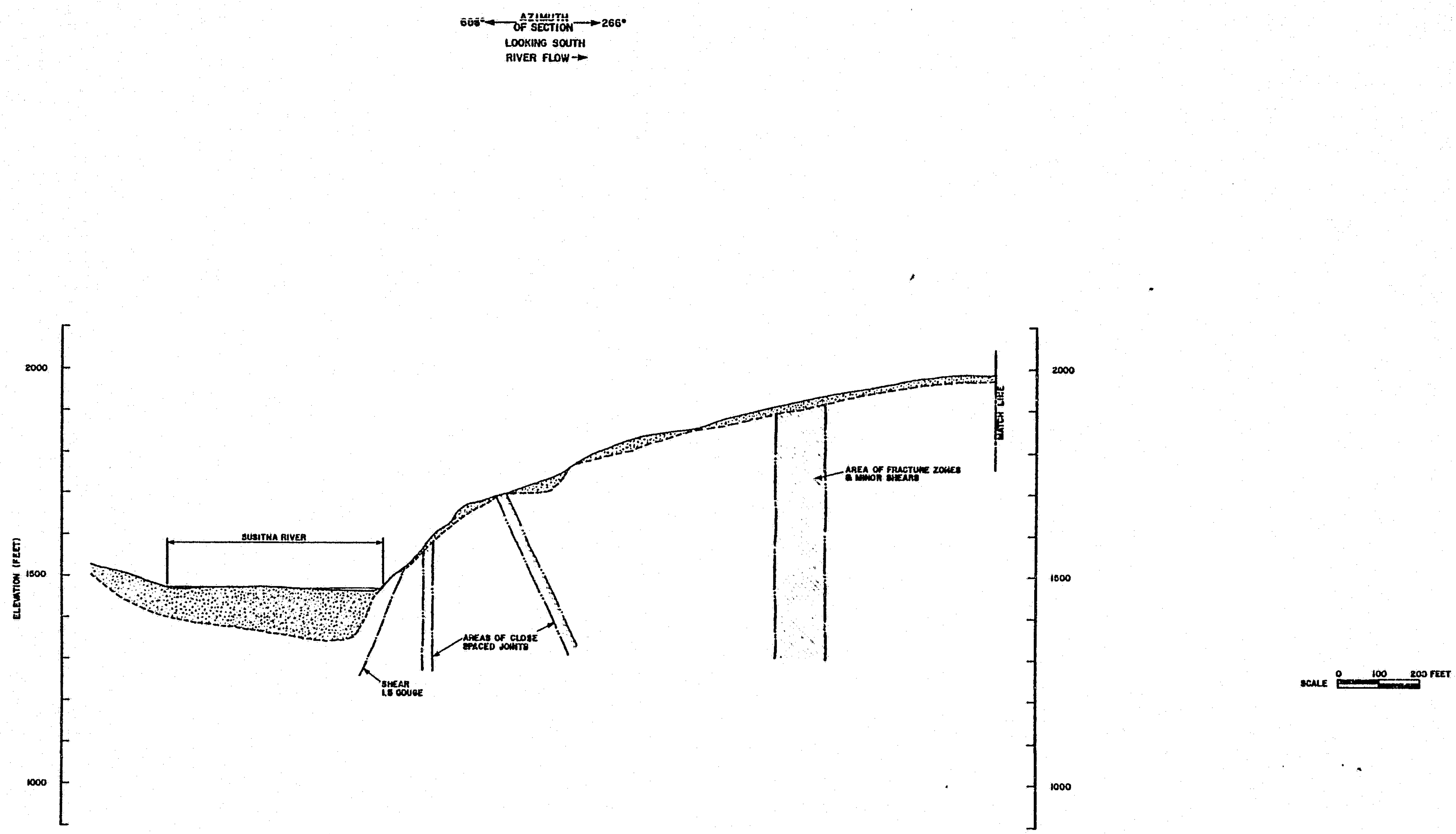
WATANA
GEOLOGIC SECTION W - 3
SHEET 2 OF 2

PRELIMINARY

ACRES

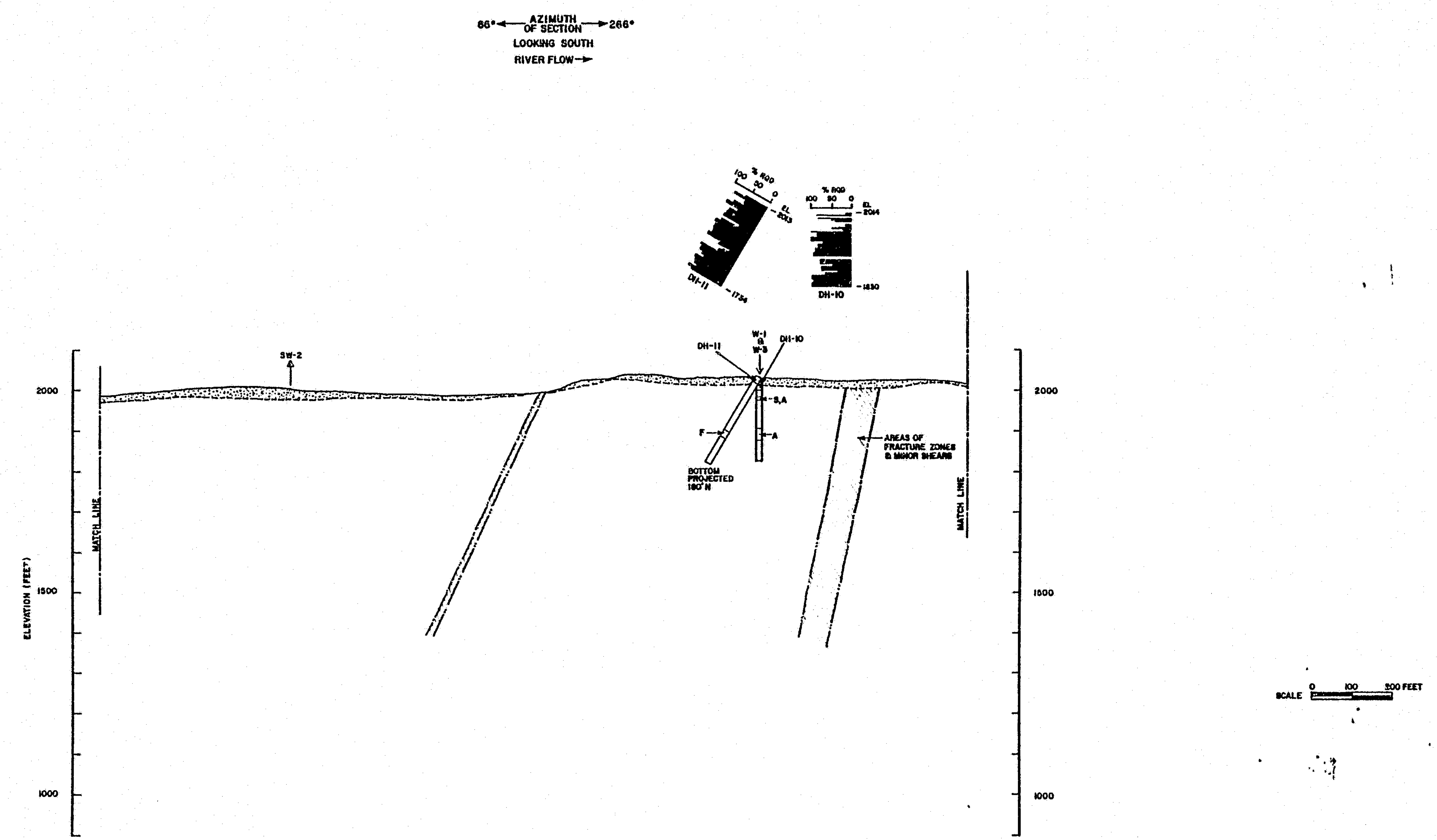


WATANA
GEOLOGIC SECTION W-4
SHEET 1 OF 3

PRELIMINARY

W-4

AGRS



WATANA
GEOLOGIC SECTION W-4
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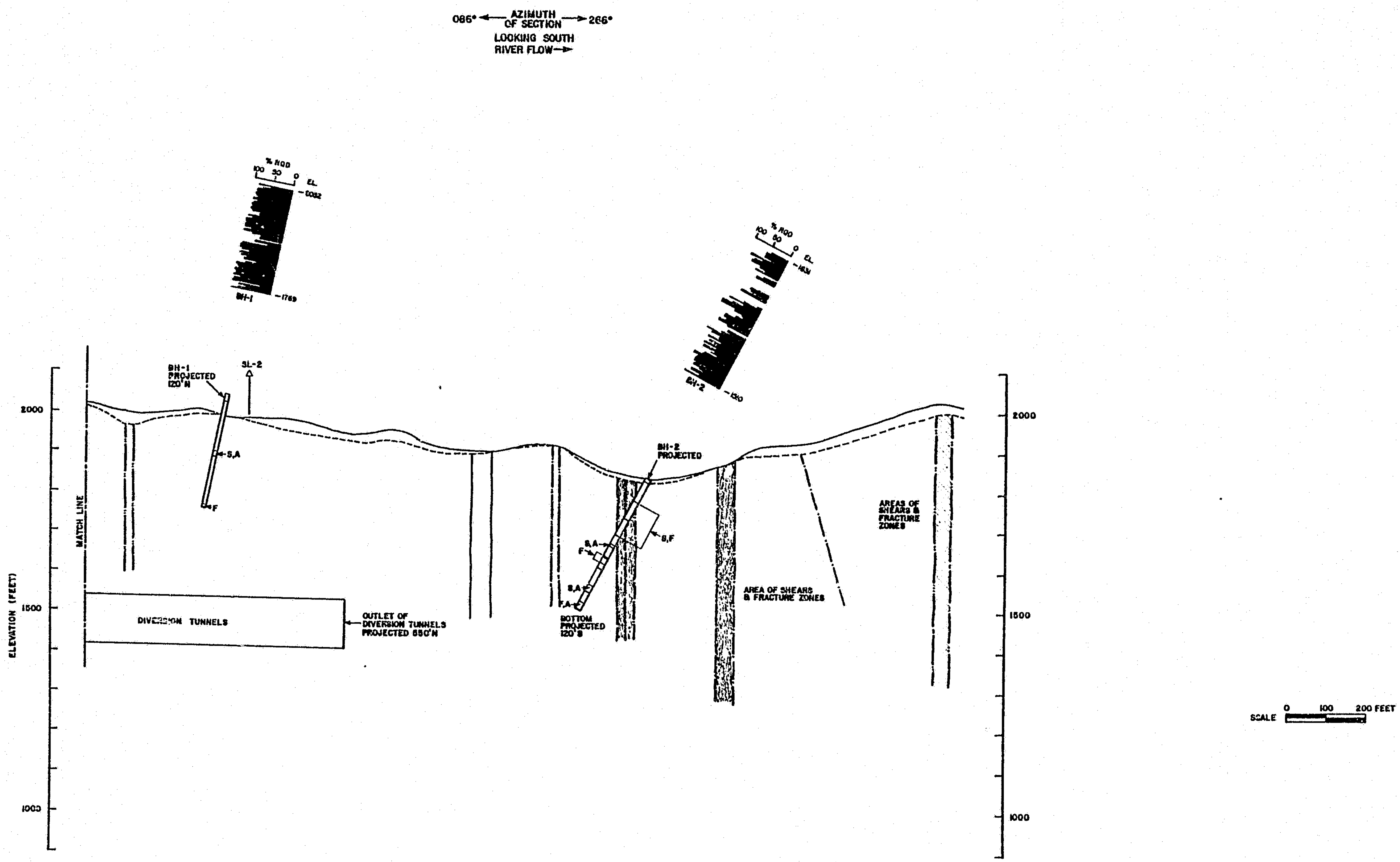
PRELIMINARY

ACRES



WATANA
GEOLOGIC SECTION W-4
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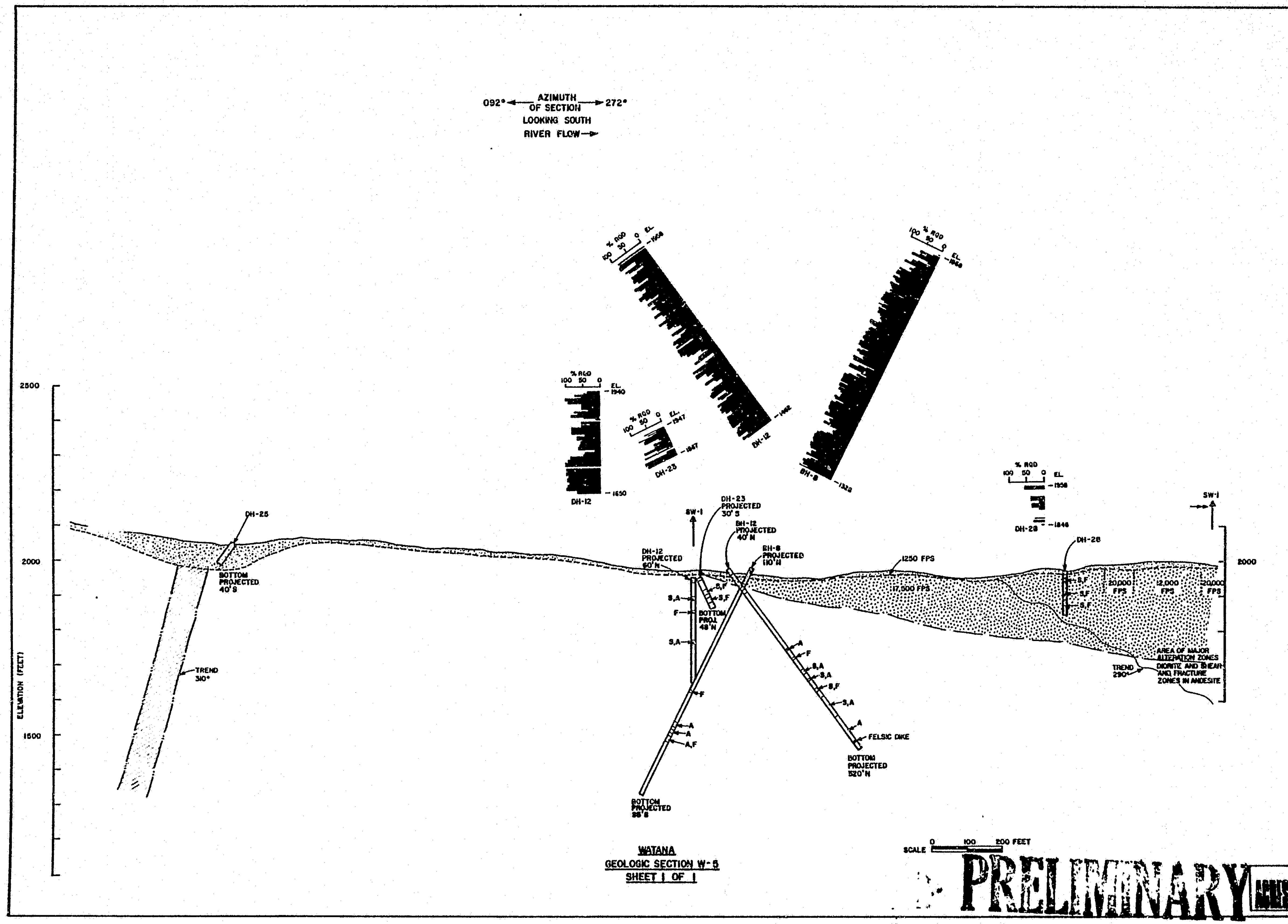
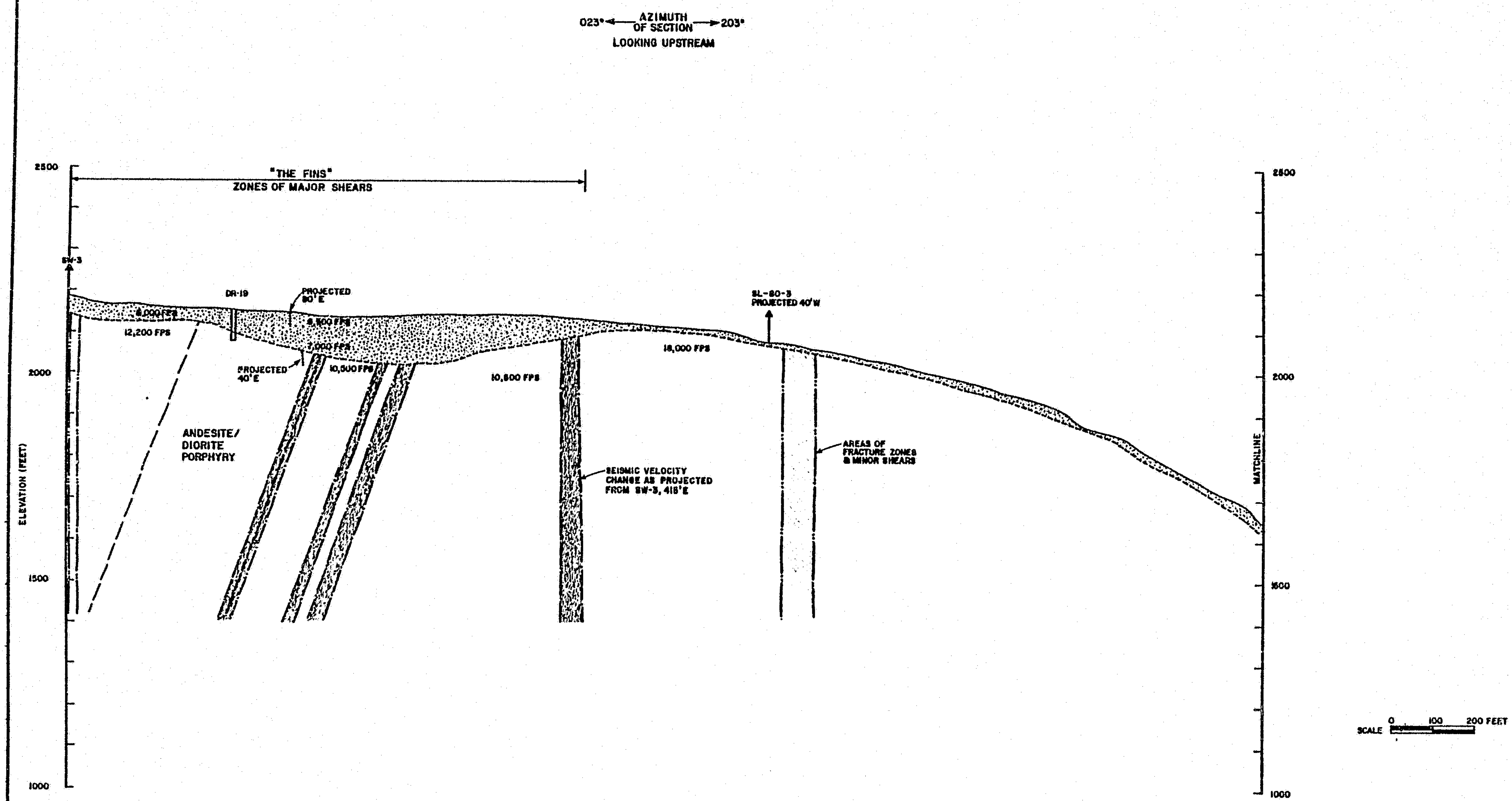
PRELIMINARY

ACRES



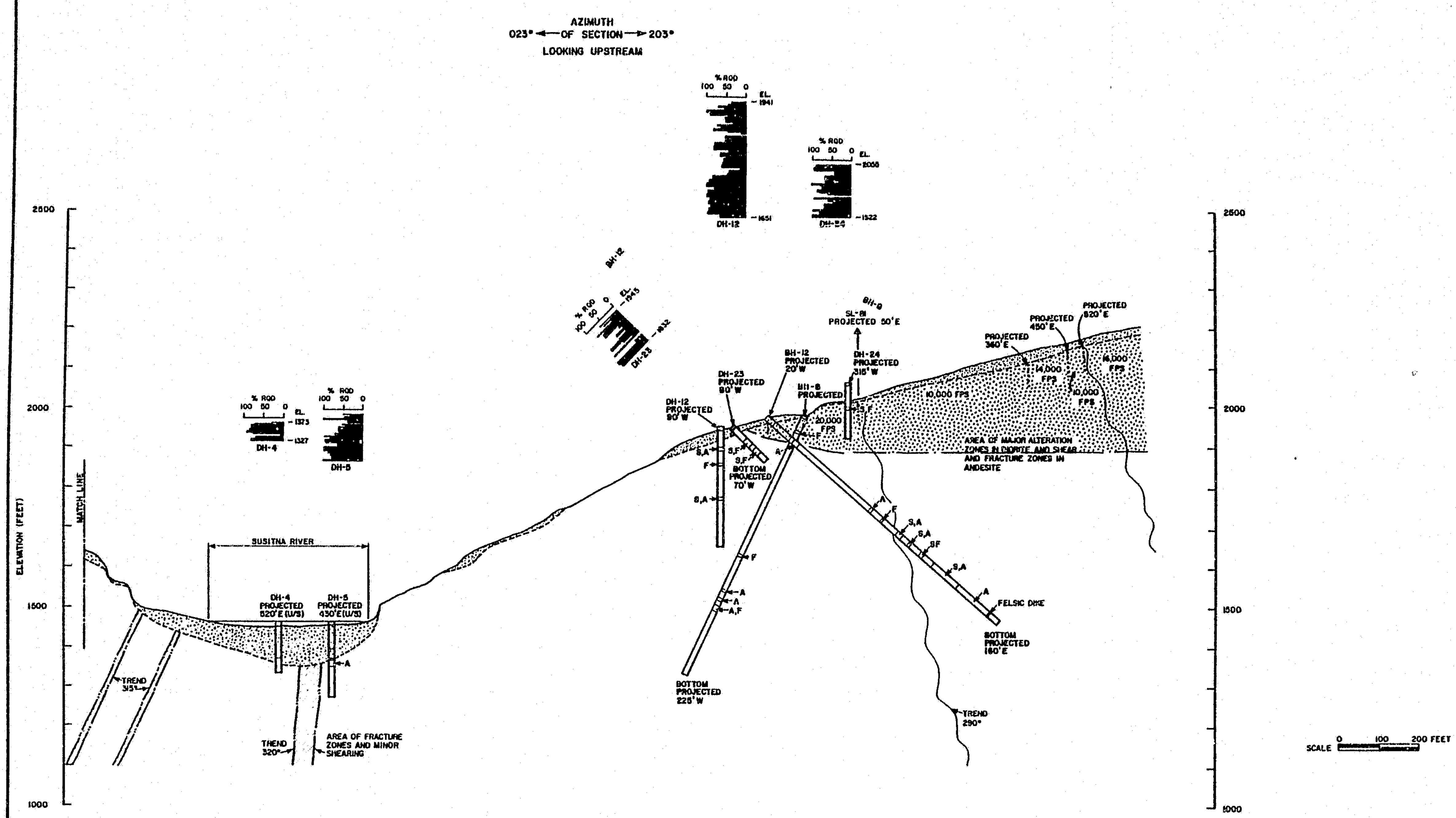
WATANA
GEOLOGIC SECTION W-5
SHEET 1 OF 1



WATANA
GEOLOGIC SECTION W-7
SHEET 1 OF 2

PRELIMINARY

ACRES

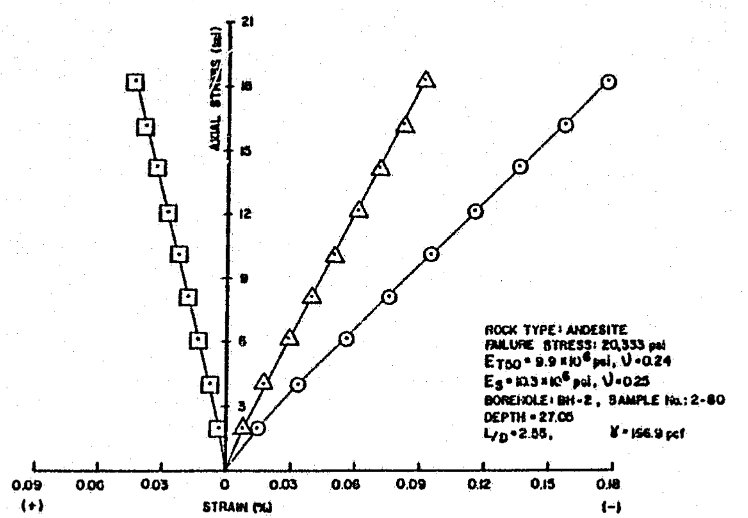


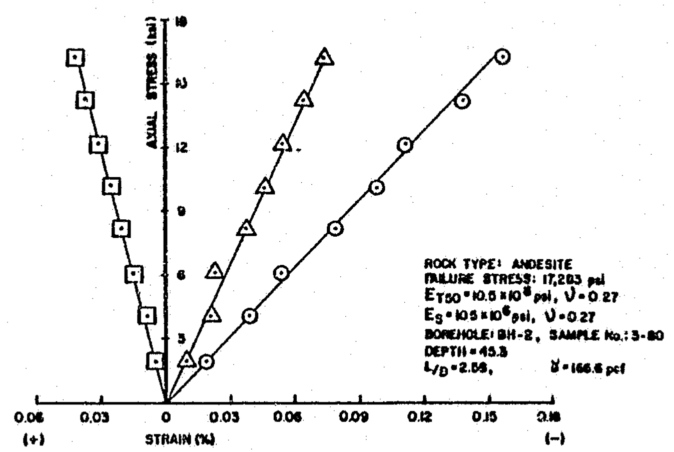
WATANA
GEOLOGIC SECTION W-7
SHEET 2 OF 2

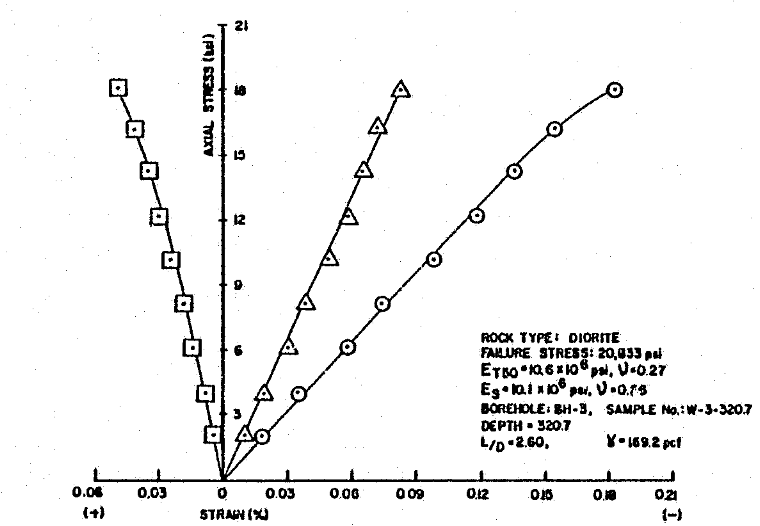
PRELIMINARY

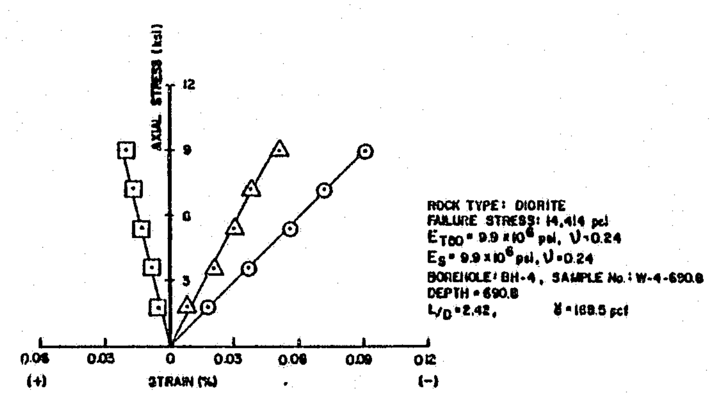
LEGEND

- ⊙ AXIAL STRAIN
- △ VOLUMETRIC STRAIN
- ⊡ DIAMETRIC STRAIN
- E_S SECANT MODULUS
- E_{T50} TANGENT MODULUS AT 50% FAILURE STRESS









WATANA
STRESS-STRAIN CURVES

PRELIMINARY

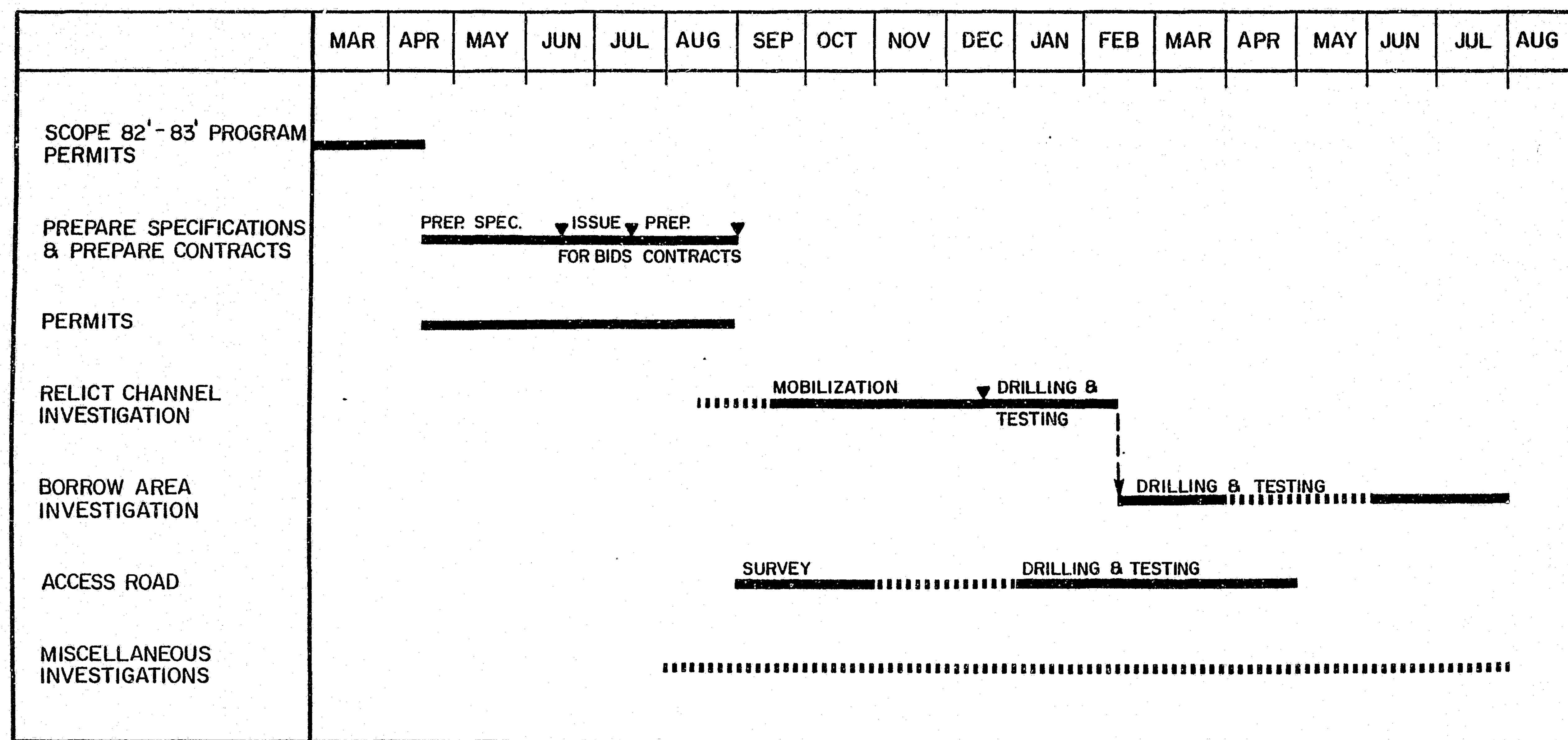
ACRES

EXPLORATION PROGRAM

1982-83

PROPOSED 1982-1983
GEOTECHNICAL ACTIVITIES

- O BORROW AREA INVESTIGATION
  - D
  - E
  - H
  - I
  - J
- O RELICT CHANNEL
- O ACCESS ROAD
- O MISCELLANEOUS STUDIES IN SUPPORT OF FERC LICENSE



PROPOSED 1982 - 1983
GEOTECHNICAL PROGRAM

TASK 6

SUSITNA HYDROELECTRIC PROJECT

UPDATE OF DESIGNS FOR
WATANA DAM AND RELICT CHANNEL TREATMENT

1 - INTRODUCTION

The approach to the Watana Dam design and treatment of the relict channel immediately downstream of the Watana Dam have been revised to take account of comments resulting from the consultant review meetings held in Buffalo in October (APA Panel) and November (Acres Panel). The major changes from the presentation made in October are as follows.

2 - WATANA DAM

2.1 - Upstream Shell

The Acres panel were strongly of the opinion that quarried rock fill would be subject to more settlement due to crushing of point contact between individual rock pieces during reservoir filling than a compacted river gravel. Such settlement tends to produce longitudinal cracks along the core. In addition, the compaction of rock fill would tend to produce fines which would result in lower permeabilities than would be expected from the initial gradation of the material. It is now proposed that the upstream shell be constructed of compacted processed river alluvium extracted from borrow areas along the Susitna river. The river gravels will be processed to remove all material finer than 1/2" and all pieces larger than 18". It is proposed that this material be compacted in 3-foot lifts to provide a dense, free draining upstream shell. The objective is to provide a strong relatively incompressible shell of high permeability such that there will be no build up of pore pressures in the saturated upstream zone of the dam during seismic shaking and relatively minor seismic slumping.

2.2 - Downstream Shell

The downstream shell will consist of compacted unprocessed river gravels except that material larger than 18" will be removed. Since this zone will be unsaturated, there is no possibility of pore pressures developing during seismic loading and hence low permeability is not a requirement. However the compressibility characteristics of the downstream shell will be comparable to those for the upstream shell.

The saturated toe of the downstream shell below maximum tail water level will be constructed of processed gravel, similar to the upstream shell, to eliminate any risk of instability in this area underseismic loading.

2.3 - Core Material

Questions were raised at the October meeting regarding the suitability of material from Borrow Area D for the core. Although this material is marginallaly SM by classification, it is relatively well graded with typically 75 to 85% passing No. 4 sieve and 35 to 45% passing No. 200 sieve. The grading is very similar to tills utilized in a number of large dams in Canada, for example Mica, and the James Bay dams. Additional testing has been carried out on composite samples from Areas D & H, particularly Standard Proctor compaction on material passing No. 4 sieve, which as expected, gave a higher optimum moisture content.

2.4 - Filter Zones

Granular material from borrow area E will be used for both fine and coarse filters. The Acres panel emphasized the need for careful design of filters in accordance with current practice to ensure that core material could not pass into the filters.

2.5 - Core Geometry

The essentially central core has been maintained to ensure higher vertical stresses in the core as a result of downdrag from the shells, which will tend to shake down more than the core.

2.6 - Dam Design

The design criteria are discussed in the attached draft entitled "Watana Dam - Design Criteria" while the methodology proposed for dynamic analysis of the dam is discussed in attachment entitled "Outline of Methodology for Stability Analysis - Watana Dam". Both of the above are dated December 1981.

3 - RELICT CHANNEL

3.1 - Introduction

At the October meeting it was proposed that the relict channel treatment should include an extensive grouted slurrywall cut-off and saddle dam at the critical section. Available soils data is limited but in view of high permeabilities in some boreholes it was considered necessary to provide a positive cut-off to limit leakage. Other problem areas identified in the relict channel include zones of permafrost and soils which may be subject to liquefaction under earthquake loading. As the permafrost thaws due to heat flow from the reservoir, the effective permeability may increase, there could be settlement due to drainage following thawing and the risk of liquefaction would increase.

However it is not possible now and in any event would require extensive site investigation to completely define the nature of the problems outlined above. It was proposed to make allowance for settlement by over-building the saddle dam, and on the basis of the results of further investigations, locate the dam clear of any foundation materials with high potential for liquefaction. Even so, any settlement due to thawing permafrost or liquefaction would not be uniform along the saddle dam and there would be a risk of transverse cracking of the dam.

The only method of completely eliminating the potential problems associated with the saddle dam is to eliminate the need for the dam by lowering the top water level in the reservoir. The limiting level is then that level which will provide sufficient width of natural ground above water tnat local settlement in that area due to thawing and drainage will not produce a channel which would allow leakage. This width is somewhat arbitrary but grades are very flat in the natural saddle.

It is proposed that the maximum reservoir level under PMF conditions should not exceed lowest ground level at the natural saddle. Under normal conditions, this will provide at least 1000 feet of "dry" ground at the saddle.

With such an arrangement; surface flows are controlled but subsurface leakage must also be controlled. Of the three alternatives considered - upstream blanket, grouted/slurry cut-off, downstream filter, - the upstream blanket is costly and probably impractical, the cut-off wall as previously proposed is costly and its effectiveness cannot be demonstrated until permafrost has thawed throughout the channel area. However major initial costs are incurred during initial construction.

The downstream filter concept was originally rejected on grounds of cost, assuming that a very large area would have to be protected. However, the Acres Review Panel considered that since the estimated water loss is not economically significant, control of downstream seepage is the most positive control measure. It would not be necessary to treat the whole area, but only those zones of emergence if piezometric observations indicate potential piping. The observational approach has the advantage of only treating critical areas and hence minimum cost. However it may be many years before equilibrium with respect to permafrost is established in the relict channel area following reservoir filling. Monitoring throughout this period by suitably qualified personnel will be essential, and there must be a continuing ability to treat any areas. This would involve the placement of granular material excavated from borrow area E, downstream of the main dam.

In summary, the treatment of the relict channel area presently proposed will involve:

1. Monitoring of piezometric levels and temperature throughout the relict channel.
2. Inspection of the potential area of seepage emergence to identify critical zones and any zones now showing signs of seepage and piping.
3. Development of access roads from borrow area E to the area.
4. Treatment of active zones of seepage discharge.
5. Establishment of stockpiles of filter material for emergency use.
6. Monitoring of ground elevation in critical areas of the channel and making good any "lost" ground following thawing of permafrost.

WATANA DAM DESIGN CRITERIA

DRAFT

DECEMBER 1981

WATANA DAM DESIGN CRITERIA

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1 - MAIN DAM EMBANKMENT

1.1 - General

The main dam will consist of a compacted core protected by fine and coarse filters on the upstream and downstream slopes. The downstream outer shell will consist of alluvium gravel; and the upstream outer shell of clean alluvium gravel. The dam is designed to provide a stable embankment under all conditions.

1.2 - Design Philosophy

1.2.1 - General

The design of the embankment is dependent on the type of core chosen, either a vertical core or an inclined core, and its location, upstream or central in the embankment.

1.2.2 - Impervious Core Design

The advantages to both types of cores are as follows:

(a) Vertical Core

- Provides better contact with the foundation;

- Provides slightly more thickness of the core for the same quantity of the core material; and

- Settlement of the core will be independent of the post-construction or seismic displacement settlement of the downstream shell.

(b) Inclined Core

- Can place bulk quantity of downstream shell before placing core material;

- Can carry out foundation treatment during placement of shell material; and

- Pore pressure resulting from rapid drawdown are less.

The major disadvantages for each type of core are as follows:

(a) Vertical Core

- Placement of core material controls placement of filters and shell materials; and

- Possible arching of a thin core by transferring weight to adjacent filters and shell materials during settlement or seismic displacements.

(b) Inclined Core

- Excessive post-construction settlement or seismic displacement of downstream shell may cause rupture of core.

- Location of core may effect upstream slope by making it flatter for stability reasons.

A central vertical core was chosen for the embankment based on a review of previous designs and the nature of the proposed impervious material. The majority of rockfill dams in high seismic areas have a central vertical core with slopes ranging from .25 to .7 horizontal to 1 vertical for the upstream slope, and .1 to .75 horizontal to 1 vertical for the downstream slope. For preliminary design, slopes of 1.0 horizontal to 3.0 vertical for the upstream slope and 1.0 horizontal to 6.0 vertical for the downstream slope were chosen (see Figure No A1).

The impervious material is a glacial moraine material with a wide grain size distribution. This material is nonplastic and would tend to crack rather than deform under tensile stress and may be very susceptible to erosion. For a sloping core the possibility exists of cracks developing in the core for a nonplastic material due to lateral settlement or displacement during a seismic event. It also becomes difficult to avoid high tensile and shearing stresses in an inclined core. Settlement data indicates that the magnitude of water load settlements in rockfill dams may increase at a rate greater than direct proportion to the height of the dam. For these reasons a central vertical core will be used in the Watana Dam cross section.

1.2.3 - Earthquake Resistance Design Features

Due to the apparent low plasticity of the material to be used in the impervious core and the requirement for an earthquake resistance design, the following design features will be incorporated into the main dam cross sections:

(a) The core-foundation contact will be widened near the ends of the embankment to ensure seepage control during normal operating conditions and any seismic event.

(b) Thick filter zones will be placed upstream and downstream of the impervious core to prevent breaching of the core from either post-construction settlement and cracking or from any cracking resulting from a seismic event.

(c) The filters will be designed to be self-healing in case of transverse cracks in the core resulting from either post-construction settlement or a seismic event.

(d) The downstream filters will be designed to be capable of handling any abnormal flows which could result from transverse cracking at the core from post-construction settlement or a seismic event.

(e) The proposed width of the core will prevent arching of the core caused by transfer of load to the shell or filter materials.

(f) Compacted river alluvium gravel will be used to construct the downstream outershell and compacted clean river alluvium gravel will be used to construct the upstream outershell to minimize settlement and displacement that could be caused by a seismic event.

1.2.4 - Freeboard and Static Settlement

The maximum required crest elevation of the Watana Dam, not including static and seismic settlement, was determined for each of the following conditions.

Condition (1) - top of core settles under dynamic loading to reservoir 10,000 year flood level.

Condition (2) - minimum crest elevation based on normal operation conditions.

Condition (3) - minimum crest elevation based on 1:10,000 year reservoir level.

Condition (4) - fuse plug crest based on 50 year flood and overtopping and additional freeboard on main dam.

Condition (5) - fuse plug based on 10,000 year flood and overtopping and additional freeboard on main dam.

Table I shows the resulting crest elevations for each condition.

The required crest elevations shown in Table I are at the maximum section of the dam and are based on a normal operating water level of 2185. The governing crest elevation excluding static and seismic settlement is 2205 at the maximum section and 2201 at the abutments.

TABLE I

REQUIREMENT	CONDITION (1)	(2)	(3)	(4)	(5)
Normal Operating Level	2,185	2,185	2,185	2,185	2,185
1:50 year reservoir elevation (6 ft. surcharge)	--	6	--	6	--
1:10,000 year reservoir elevation (8 ft. surcharge)	8	--	8	--	8
Wave run-up	--	6 ft	6 ft	6 ft	6 ft fuse plug
Dry freeboard	--	3 ft	--	3 ft fuse plug	--
Water depth over use	--	--	--	2 ft	2 ft
Road base	3 ft	3 ft	3 ft	3 ft	3 ft
Crest Elevation	2,196	2,203	2,202	2,205	2,204

The expected static settlement of 1 percent of the height of the dam (and seismic settlement of 0.5 percent of the height of the dam) will be incorporated in the design by locally steepening the slopes of the top of the dam to obtain 15 feet additional freeboard at the maximum section and 3 feet additional freeboard at the abutments. The expected static settlement will result from post-construction settlement and does not include any construction settlement. The expected seismic settlement will result from the postulated earthquake event.

1.3 - Typical Cross Section

The typical cross section would be as shown in FigureA1. The impervious core slopes would be, on the reservoir side, 1.0 horizontal to 3.0 vertical sloped upstream, and on the tailwater side, 1.0 horizontal to 6.0 vertical sloped downstream with a crest width of 15 feet. Minimum core-foundation contact would be 50 feet requiring flaring of the cross section at left end of the embankment.

The upstream filters will have the following slopes:

(a) Fine filter zone will be 1.0 horizontal to 2.7 vertical sloped upstream on the reservoir side, and on the core side 1.0 horizontal to 3.0 vertical sloped upstream.

(b) Coarse filter zone will be 1.0 horizontal to 2.35 vertical sloped upstream on the reservoir side, and on the fine side 1.0 horizontal to 2.7 vertical sloped upstream.

The downstream filter zones will have the following slopes:

(a) Fine filter zone will be 1.0 horizontal to 4.5 vertical sloped downstream on the tailwater side, and on the core side 1.0 horizontal to 6.0 vertical sloped downstream.

(b) Coarse filter zone will be 1.0 horizontal to 3.75 vertical sloped downstream on the tailwater side, and on the fine filter side 1.0 horizontal to 4.5 vertical sloped downstream.

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 size of the downstream filter zones will ensure its capability to handle any abnormal leakage flows.

The shells of the dam will consist of compacted river alluvium gravels. To minimize pore pressure generation and ensure rapid dissipation during a seismic event, the saturated upstream shell will consist of compacted clean river alluvium gravels. This material will be processed to remove all fines less than 1/2" size. The downstream shell will consist of compacted unprocessed alluvium gravels since it will not be effected by pore pressure generation during a seismic event.

Slope protection on the upstream slope will consist of a 10 foot zone of oversize aterial up to 6 feet in diameter placed and compacted by suitable equipment.

The typical crest detail is shown in FigureA2. Because of the narrowing of the crest dam, the filter zones are reduced in width and the upstream and downstream coarse filter is eliminated above elevation 2230. A layer of filter fabric is incorporated to protect the core material from damage from frost penetration and dessication, and to act as a coarse filter where required.

1.4 - Core Material

1.4.1 - General

The core material will be obtained from Borrow Area "D" which consists of a series of glacial tills separated by alluvial and lacustrine materials. Processing and blending will be necessary to provide the required moisture content and gradation and to remove any oversize material. Obtaining the required moisture content may prove difficult because of sensitivity of material to changes in moisture content. Frozen material will either be hauled to a waste area or left in place.

1.4.2 - Basic Data

1.4.2.1 - Gradation

The proposed gradation will be as shown in Figure 1. Maximum size particle will be 6 inches in greatest dimension. Composite gradation curves are shown in Figures 2 and 3.

1.4.2.2 - Atterberg Limits

The Atterberg Limits will be within the following limits:

a) Plasticity Index - 0-20
b) Liquid Limit - 10-45
c) Above the "A" line

1.4.2.3 - Permeability

Initial permeability tests indicate a permeability of 10^{-5}cm/sec. Comparison of the proposed core material to other glacial tills used in various dams indicate an inplace permeability range of 10^{-7} to 10^{-9}cm/sec.

1.4.2.4 - Compaction Properties

Modified Proctor Compaction tests on material passing 3/4" sieve indicate the optimum moisture content was 7.5 percent with a maximum dry density of 135.5. pcf. Standard Proctor Compaction test on material passing No. 4 sieve indicate the optimum moisture content was 10.4 percent with a maximum dry density of 127.6 pcf. See Figures 4 and 5.

1.4.2.5 - Consolidation Parameters

Consolidation tests indicate a compression index (C_c) of *.

* testing in progress.

1.4.2.6 - Specific Gravity

Specific gravity tests indicate a specific gravity of 2.71.

1.4.2.7 - Natural Water Content

The natural water contents of samples tested ranged from 7 to 26 percent. The optimum moisture content is 7.5 percent, based on Modified Proctor Compaction tests. The material placed in the core zone will be allowed to be a maximum of 3 percent above the optimum (10.5 percent). It is anticipated that the blending and processing of the core material will enable it to meet this requirement.

1.4.2.8 - Shear Strength

Consolidated undrained test results, see Figure 6, at 95 percent Modified Proctor Density at 2 percent above optimum moisture content, indicate the following strength parameters.

Unit Weight (pcf)	C'	Ø°'
126	0	37

1.4.2.9 - Dispersion Potential

Because of the nature of the core material, a glacial till, it is not expected to be dispersive.

1.4.3 - Placement and Compaction

1.4.3.1 - Excavation

The borrow area is approximately 40 to 60 feet in depth and will be developed in three branches ranging from 13 to 20 feet. Processing and blending of the material will be done during excavation. Oversized material (greater than 6 inches) will have to be removed by grizzlies. Frozen material will have to be left in place or loosened by blasting and ripping for haulage to waste area. Moisture conditioning should be done in the borrow area and will have an effect on the placement and compaction operation.

1.4.3.2 - Placement and Compaction

Material will be placed in 8-inch lifts at a maximum moisture content of 3 percent above optimum moisture content, and compacted to 95 percent of the maximum density obtained from the Modified Proctor Test (ASTM D698). Type of roller, number of passes, thickness of lift and moisture content can be adjusted based on field tests and equipment to be used.

1.5 - Fine and Coarse Filters

1.5.1 - General

Fine and coarse filter material will be obtained from Borrow Areas E, I, and J. The material will require processing to provide the proper gradations for the fine and coarse filters as shown in Figure 1. No frozen material is expected to be found within the borrow areas.

Design of the fine and coarse filters are based on the following criterias and the average gradation curve for the core material.

Criterion 1: The 15 percent size (D15) of a filter material must be not more than four or five times the 85 percent size (D85) of a protected soil.

Criterion 2: The 15 percent size (D15) of a filter material should be at least four or five times the 15 percent size (D15) of a protected soil.

Criterion 3: The 50 percent size (D50) of a filter material must be not more than twenty-five times the 50 percent size (D50) of a protected soil.

1.5.2 - Basic Data

1.5.2.1 - Gradation

The required gradation of the fine and coarse filter material are shown in Figure 1. All filter material is to be well graded. Composite gradations for Borrow Areas I, J, and E are shown in Figures 7, 8, and 9, respectively.

1.5.2.2 - Permeability

Permeability of the fine filter and coarse filter will be greater than 1 cm/sec and 100 cm/sec, respectively. Permeability will be verified by large scale field or laboratory tests.

1.5.2.3 - Shear Strength

Assumed properties for the fine and coarse filter material are based on available data:

Unit Weight	C	Ø
145 pcf	0	35°

Actual properties to be determined from large scale triaxial tests and/or by modeling the gradation for standard triaxial tests for final design.

1.5.3 - Excavation, Placement, and Compaction

1.5.3.1 - Excavation

The borrow areas will be developed by a method which will supply the required amounts of fine and coarse filter material for construction. Material will be processed by screening and blending using wet screening methods. Oversized material will have to be removed and either used as an aggregate source or possibly used in the outershell of the dam.

1.5.3.2 - Placement and Compaction

The method of placement and compaction will depend on the results of full scale test fills to be done prior to construction using the proposed equipment and materials. It is assumed that 12-inch lifts with four passes of a large vibratory roller will provide the required compaction.

1.6 - Alluvium Fill Material

1.6.1 - General

The alluvium fill will be obtained from Borrow Areas I and J. The upstream shell of the dam will be constructed using processed river alluvium gravel with the material less than 1/2-inch removed. The downstream shell will be constructed using unprocessed alluvium fill material. Any oversized material (greater than 24 inches) will be stockpiled and used in the rip-rap zones.

1.6.2 - Basic Data

1.6.2.1 - Gradation

The gradation of the alluvium fill will be as shown in Figures 7 and 8. Maximum size of river gravel will be 18 inches in the greatest dimension.

1.6.2.2 - Permeability

Permeability of the processed alluvium fill will be greater than 100 cm/sec.

1.6.2.3 - Shear Strength

Assumed properties for the alluvium fill material are based on available data:

Unit Weight	C	Ø°
145 pcf	0	35°

Actual properties to be determined from large scale triaxial tests and/or by modeling the gradation for standard triaxial tests for final design.

1.6.3 - Excavation, Placement, and Compaction

1.6.3.1 - Excavation

The alluvium fill material will be obtained from the main dam foundation excavation and from the river bed upstream and downstream of the main dam (Borrow Areas I and J). Method of excavation would either be by hydraulic mining or dragline operation. The material would have to be processed to remove the undersized and oversized material.

1.6.3.2 - Placement and Compaction

The method of placement and compaction will depend on the results of full scale test fills to be done prior to construction using the proposed equipment and material. It is assumed that 24-inch lifts for the alluvium fill material with four passes of a large vibratory roller will provide the required compaction.

1.7 - Rip-Rap Material

1.7.1 - General

The rip-rap material (slope protection) will be obtained from the oversize material from the various borrow areas, Quarry A and any other rock excavations. The rip-rap material will be placed on the upstream slopes and in certain areas on the downstream slope of the dam.

1.7.2 - Basic Data

1.7.2.1 - Gradation

The gradation of the rip-rap material, Figure 1, is based on a 6-foot wave height using a nomograph, Figure 5-6, from EM1110-2-2300. The maximum size of rip-rap material will be 24 inches in the greatest dimension.

1.7.3 - Excavation, Placement, and Compaction

1.7.3.1 - Excavation

The rip-rap material would be obtained from Quarry Area A, oversize from various borrow areas, and various rock excavations by blasting or ripping. The material would have to be processed to remove any undersized and oversized material.

1.7.3.2 - Placement and Compaction

The method of placement and compaction will depend on the results of full scale test fills to be done prior to construction using the proposed equipment and material. It is assumed that 36-inch lifts for the rip-rap zone with four passes of a large vibratory roller will provide the required compaction.

1.8 - Stability Analysis

1.8.1 - Introduction

Static and dynamic stability analyses have been performed to establish the upstream and downstream slopes of the Watana dam. The 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 maximum cross section is shown in Figure 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 finite element program for static analysis of earth and rockfill dams (FEADAM) 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 and conclusions are presented in the Dynamic Analyses Section.

1.8.2 - 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.

*These analyses have not yet been completed
See attachment for outline of proposed methodology.

- FEADAM - Finite Element Analysis of Dams to determine the initial static stresses in the dam by stage construction.

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

1.8.3 - Static Analyses

1.8.3.1 - 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 a circular slope.

1.8.3.2 - Soil Properties

The following soil properties were used in the analyses:

	Unit Weight (lb/ft^3)	Ø°	C (lb/ft^2)
Core Material	140	35	0
Transition Material	145	35	0
Shell Material	145	35	0

1.8.3.3 - Loading Conditions and Factors of Safety

The following conditions were analyzed:

Condition	Minimum Factor of Safety	Calculated Factor of Safety U/S Slope	Calculated Factor of Safety D/S Slope
Construction	1.3	2.2 - 2.2	1.7
Normal Operating	1.5	2.0	1.7
Rapid Drawdown	1.0	1.8 - 2.0	1.7
Normal Operating with Maximum Pool	1.3	2.0 - 2.1	1.7

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

1.8.3.4 - FEADAM Analyses

(To follow when analyses completed.)

1.9 - Construction

1.9.1 - Climatic Effects

Construction of the embankment will be controlled by the climatic conditions at the site. The placement of river alluvium gravel all year round appears to be possible and will control the construction schedule.

1.9.2 - Impervious Core Material and Fine Filter Placement

The majority of the impervious core material and fine material will be placed during the short summer periods. Sufficient amounts of this material will be placed each summer to allow placement of large quantities of coarse filter and exterior shells during the winter months without having to stop work for extensive periods of time.

Placement of core material and fine filter material during the winter months may be possible if sufficient quantities are stockpiled during the summer months and prevented from freezing during the winter. The stockpiled soil can be kept from freezing by various means suchas heating under a tarpaulin cover. The placement of this material requires additional steps during the winter months and would reduce the quantity and area that can be placed and worked.

1.9.3 - Coarse Filter Alluvium Fill Placement

The coarse filter and alluvium fill will be stockpiled during the summer months and placed in the embankment during the summer and winter months. The summer months will allow these materials to drain so that freezing of the material during the winter months will not result. Once these materials drain it will not be necessary to prevent exposure of these materials to freezing temperatures.

1.9.4 - Schedule

The schedule for placement of material will have to be established so that the available equipment and personnel will not vary extensively from the summer period to the winter period. Extra equipment and personnel may be used during the summer period to provide the required stockpiled material. The schedule will be controlled by the amount of placement of the coarse filter and shell materials during the winter period. Placement of core and fine filter material will be done during the summer period with sufficient stockpiles to allow placement during the winter period if required.

1.9.5 - Longterm Exposure

The effects of longterm exposure on the embankment will vary between the upstream and downstream slopes. The exposed downstream shell and crest will be susceptible to forming a permafrost regime and possible related ice lenses will develop due to rainfall and snowmelt. Any disturbances of the downstream slopes due to ice lens build up can be corrected during normal maintenance. Extensive disturbance of the shell material is not expected. The upstream slopes depending on the reservoir level will be unfrozen due to the reservoir water and will not be subject to ice lens build up. If large variations in the reservoir level occur during the winter period, possible ice and ice lens damage could result in the upstream slopes. The use of oversize material for slope protection should minimize any damage. Minor corrective measures will have to be taken during normal maintenance. The expected seepage will be minimal, however, it will be of sufficient volume to prevent freezing of the drainage layers and possible hydrostatic pressure build up within the downstream embankment shell.

1.10 - 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 of 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:

1.10.1 - Piezometers

Piezometers are used to measure static pressure of fluid in the pore spaces of soil and rockfill.

1.10.2 - Internal Vertical Movement Devices

(a) Cross-arm settlement devices as developed by the USBR.

(b) Various versions of the taut-wire devices have been developed to measure internal settlement.

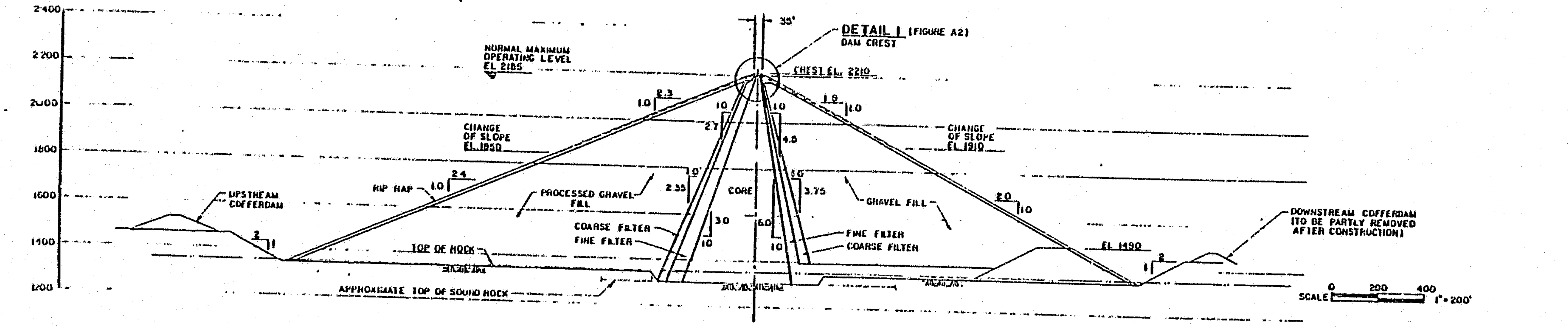
(c) Hydraulic settlement devices of various kinds.

1.10.3 - Internal Horizontal Movement Devices

(a) Taut-wire arrangements.
(b) Cross-arm devices.
(c) Inclinometers.
(d) Strain meters.

1.10.4 - Other Measuring Devices

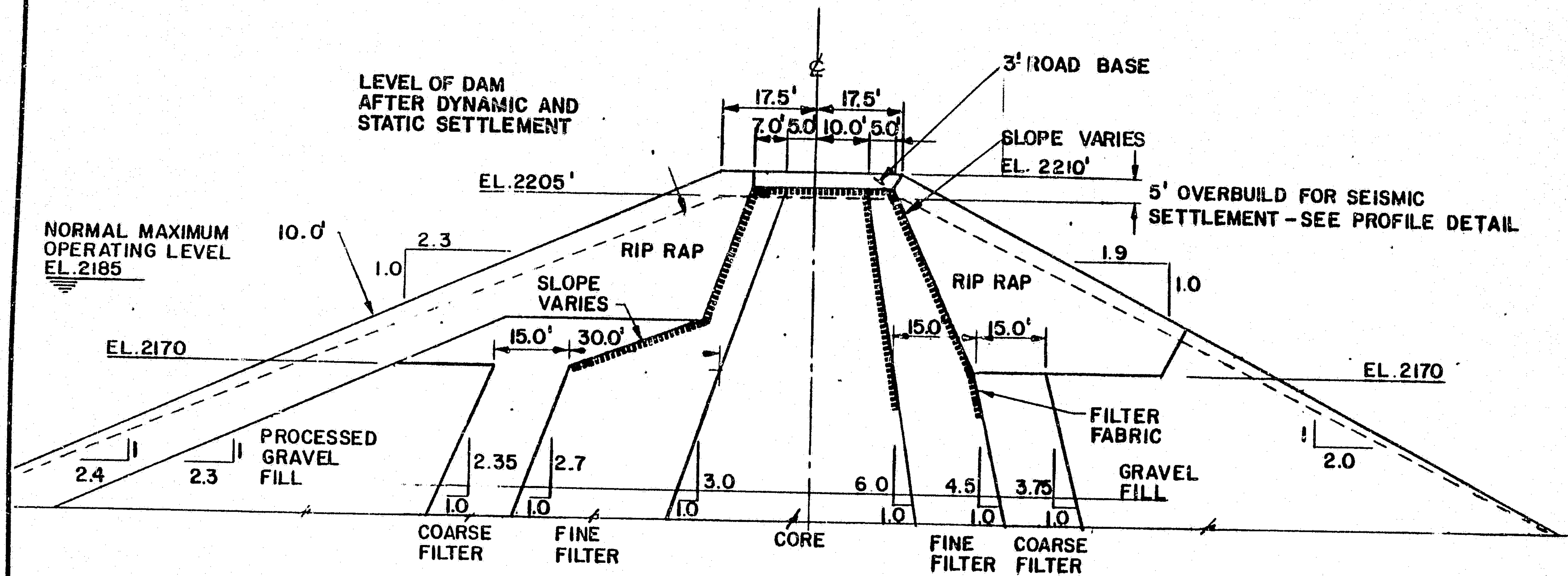
(a) Stress meters.
(b) Surface monuments and alignment markers.
(c) Seismographic recorders and seismoscopes.



SECTION THROUGH MAIN DAM
AT MAXIMUM HEIGHT
SCALE A

WATANA DAM
FIGURE A1

DEC. 1981
AS NOTED



DETAIL 1

CREST AT MAXIMUM HEIGHT SECTION

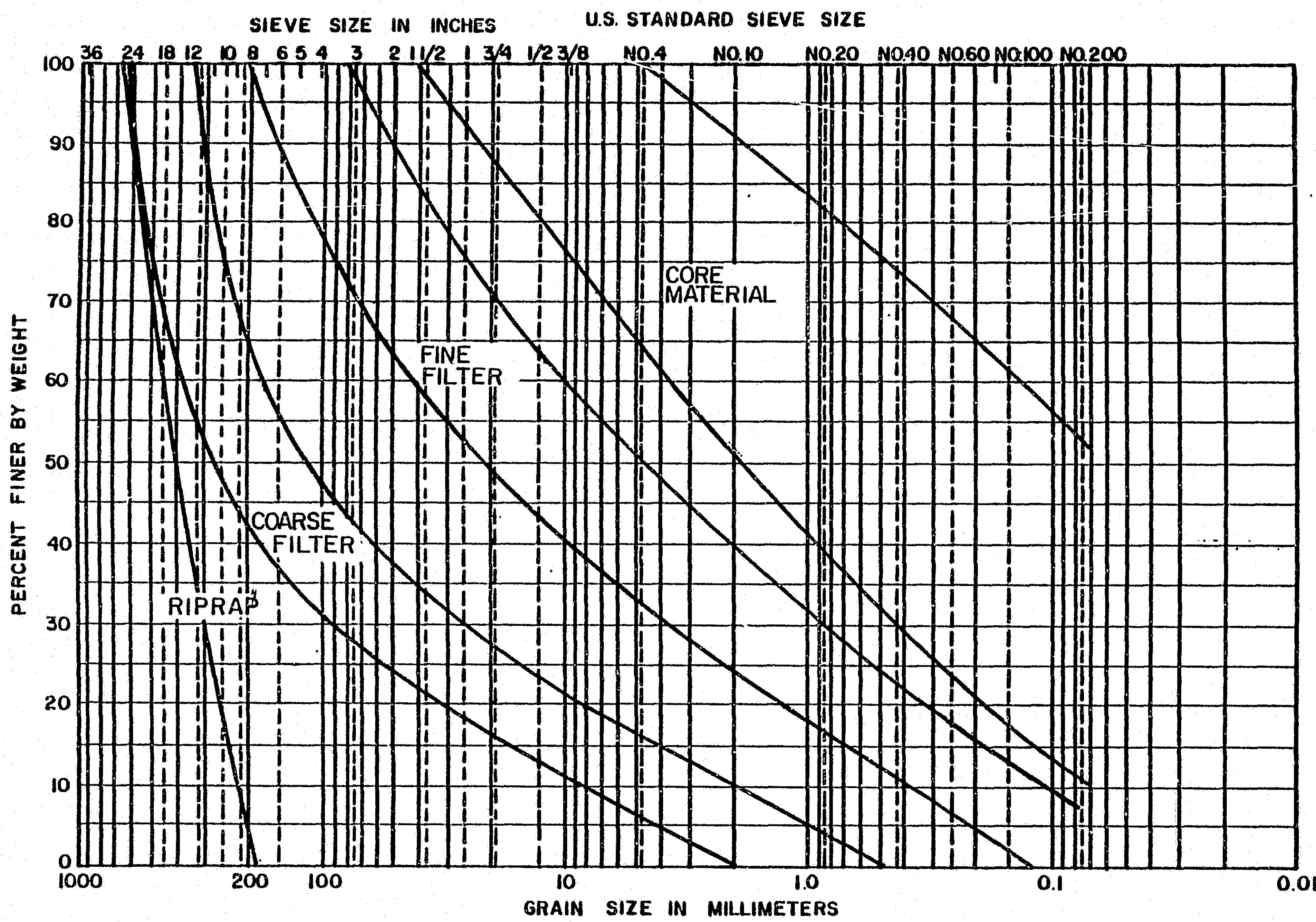
SCALE B

ACRES

WATANA DAM

FIGURE A2

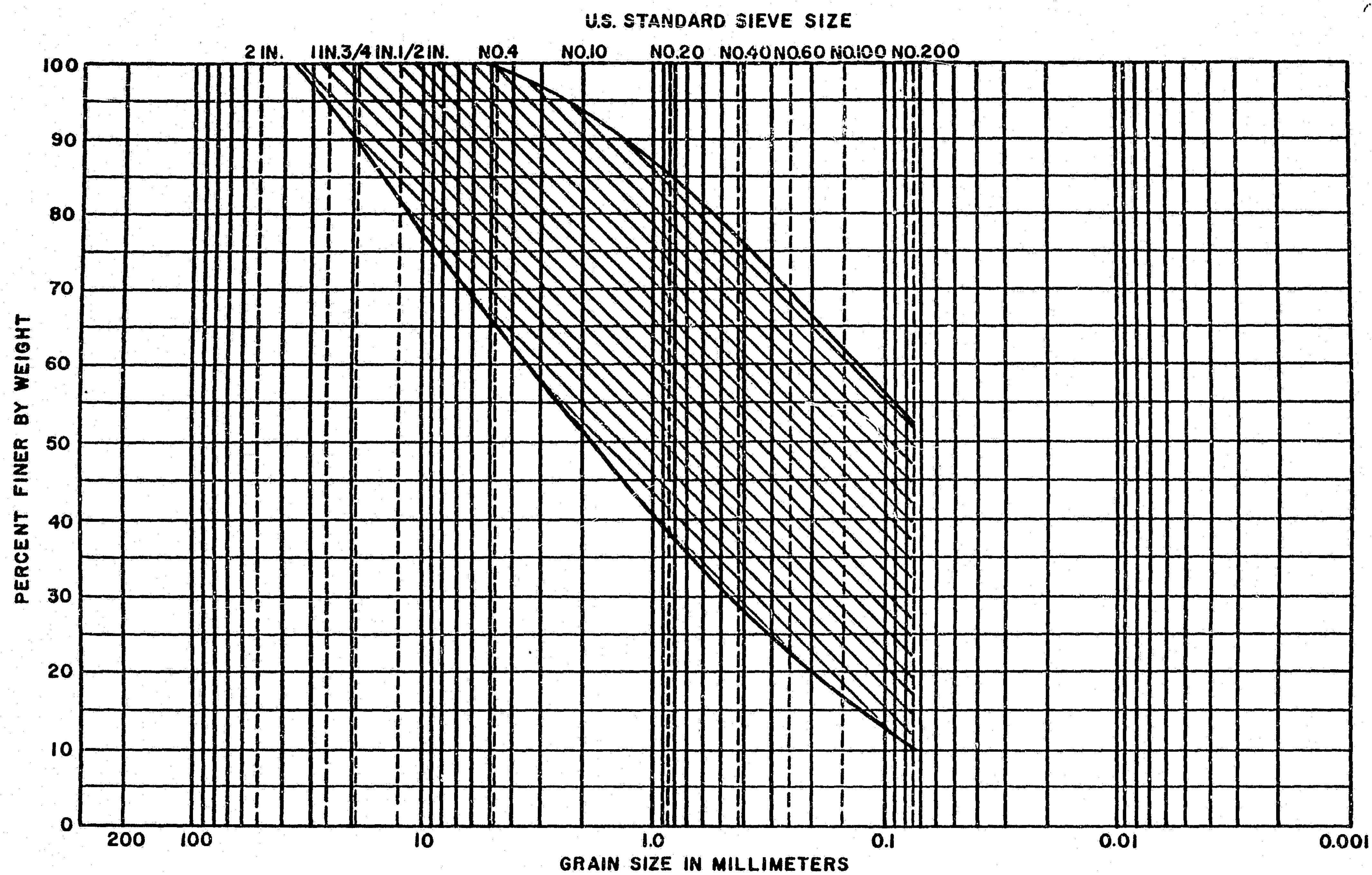
ACRES AMERICAN INCORPORATED	DATE DEC., 1981	SCALE AS NOTED
	DEPARTMENT	DRAWING NO. REV
	PROJECT	SHEET OF



REQUIRED GRADATIONS
WATANA DAM

FIGURE 1

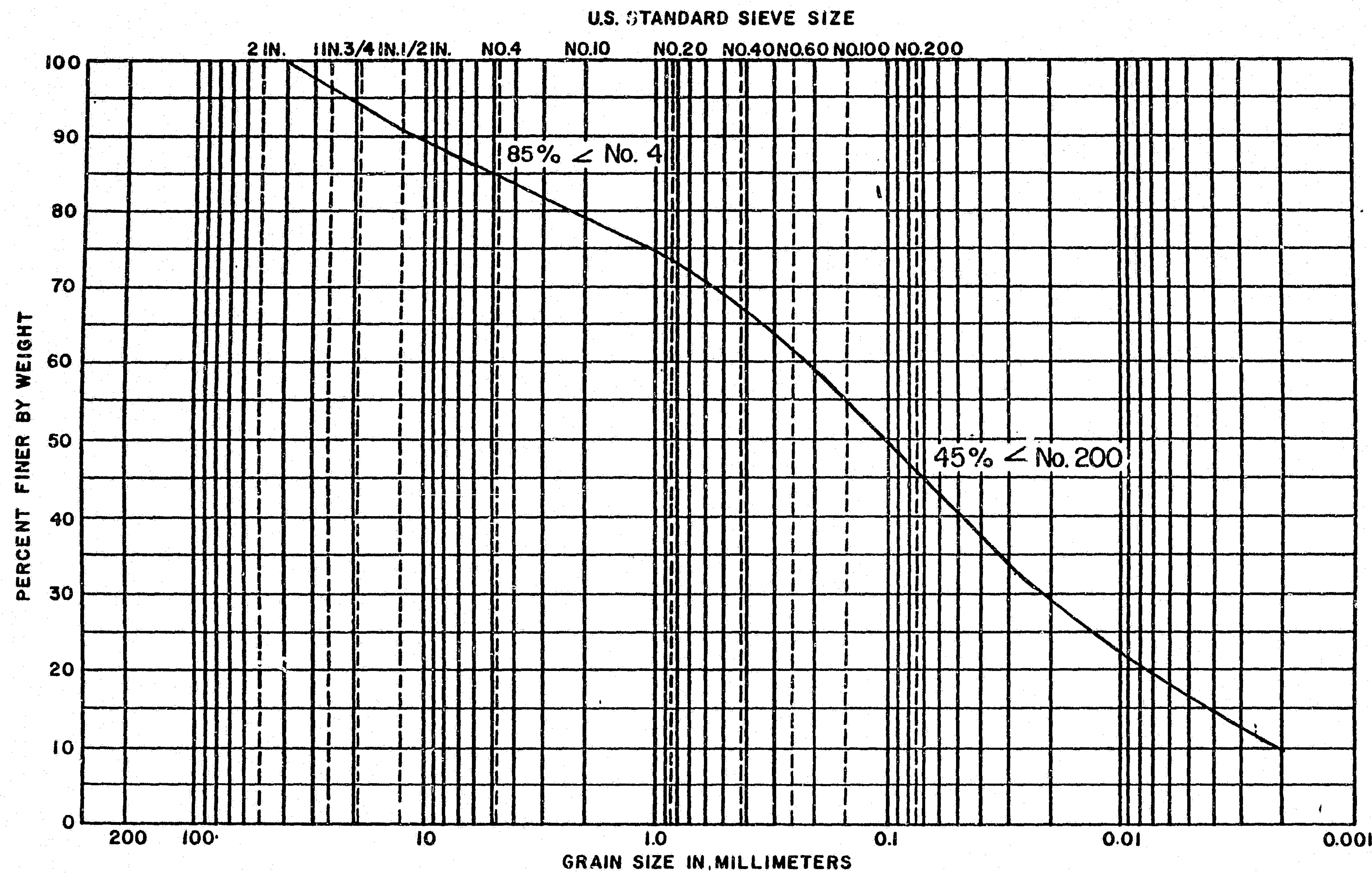
ACRES



NOTE:
NATURAL MOISTURE CONTENT RANGED FROM 6 TO 16% - AVERAGE OF 11%

COMPOSITE CURVES FOR D-5 THRU D-9

FIGURE 2

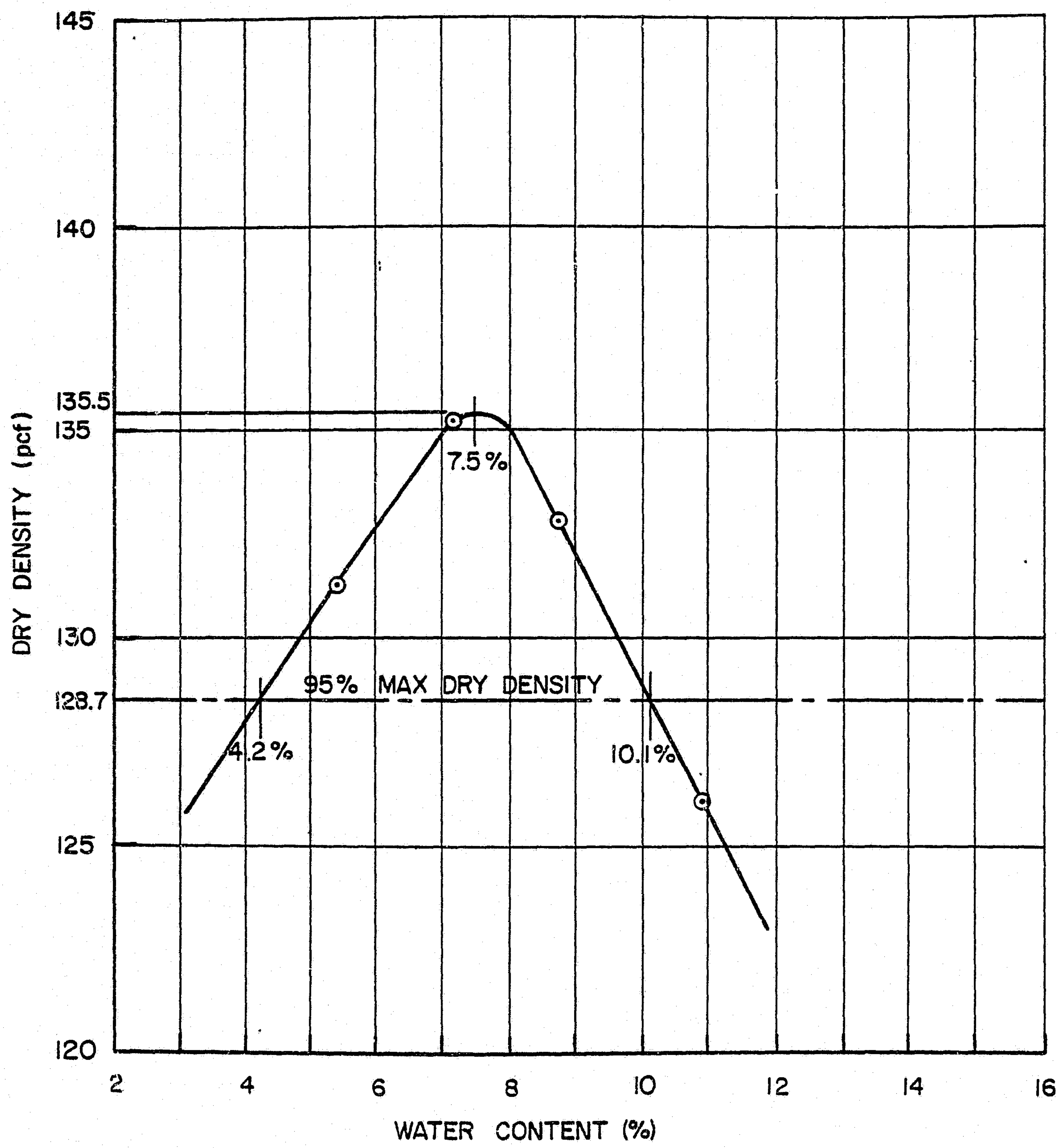


NOTE:
G=2.71

COMPOSITE CURVES FOR AREA D

ACRES

FIGURE 3

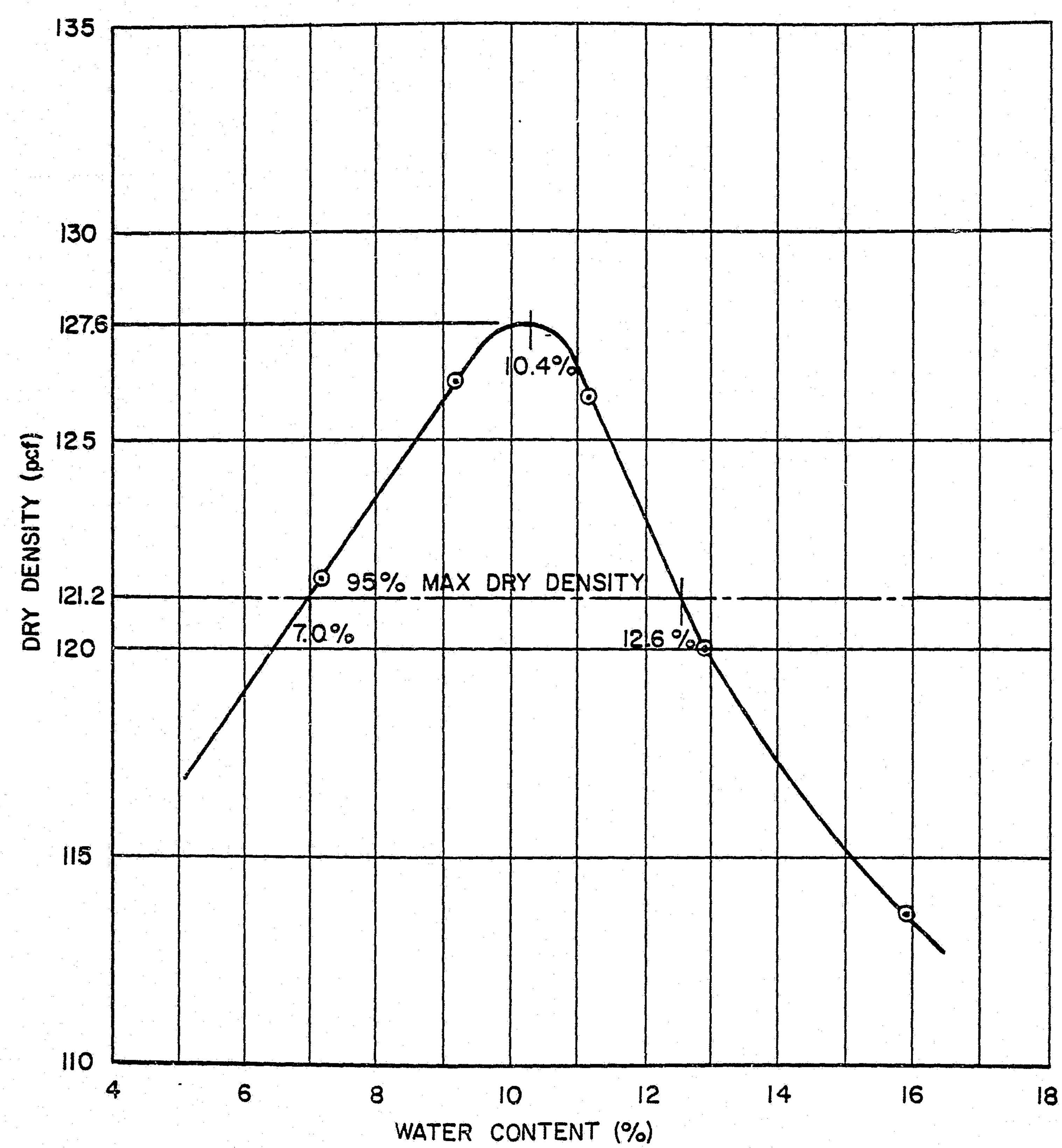


NOTE:
MATERIAL PASSING 3/4" SIEVE

MODIFIED PROCTOR COMPACTION
COMPOSITE SAMPLE
BORROW AREA D

ACRES

FIGURE 4

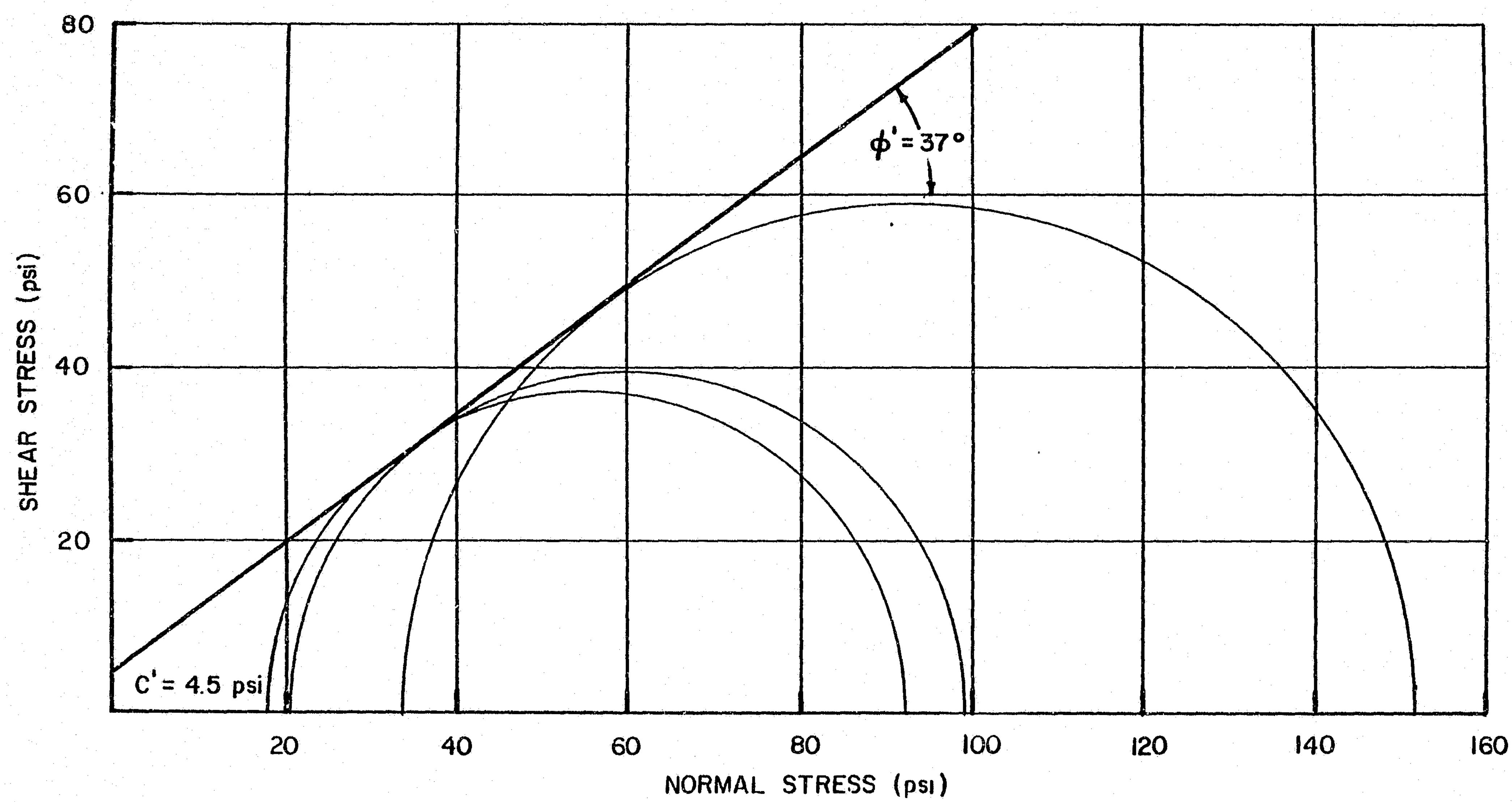


NOTE:
MATERIAL PASSING No. 4 SIEVE

STANDARD PROCTOR COMPACTION
COMPOSITE SAMPLE
BORROW AREA D

ACRES

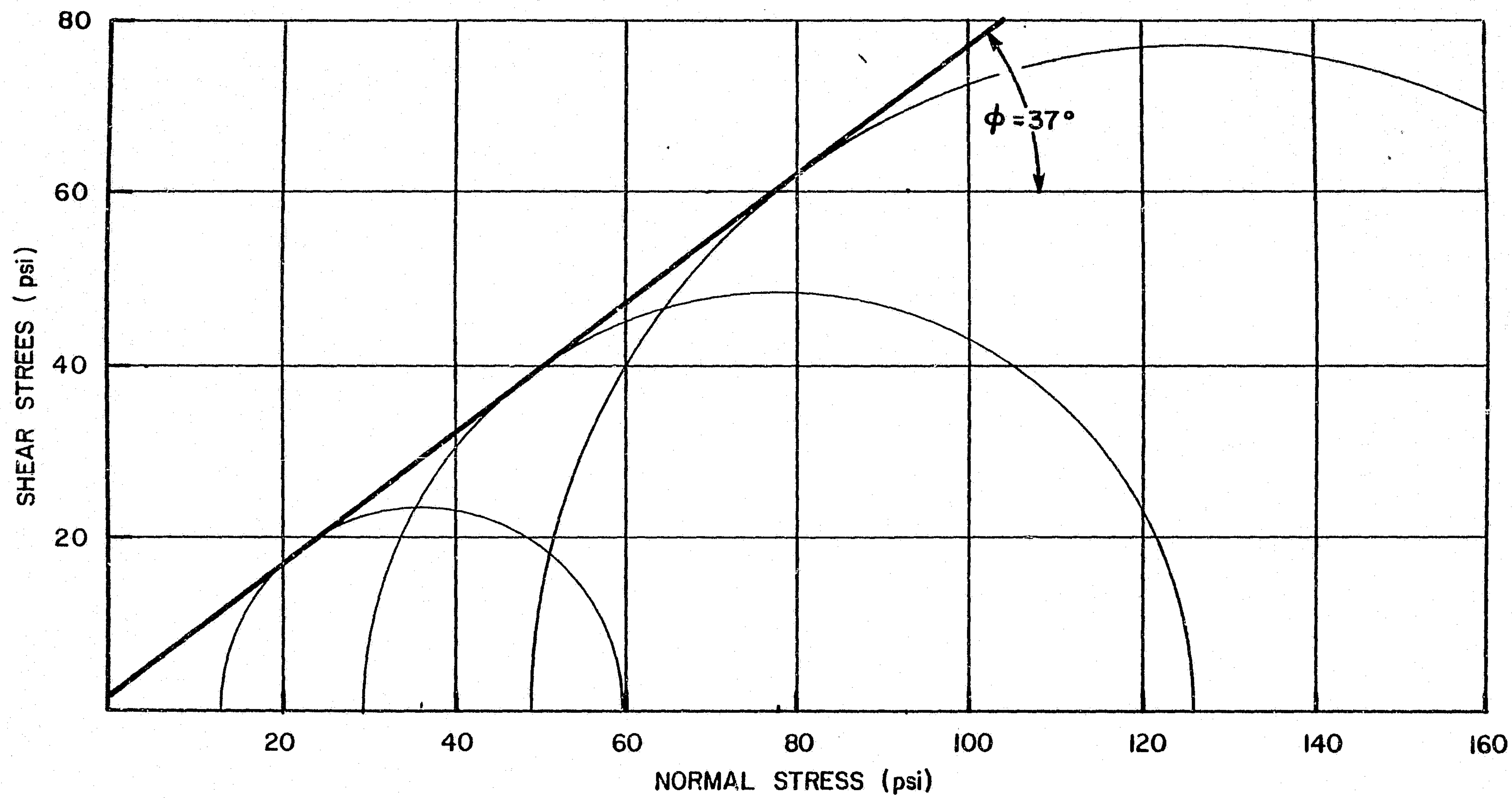
FIGURE 5



NOTE:
(EFFECTIVE STRESSES - CONSOLIDATED UNDRAINED SHEAR)

COMPOSITE SAMPLE
BORROW AREA H

FIGURE 6

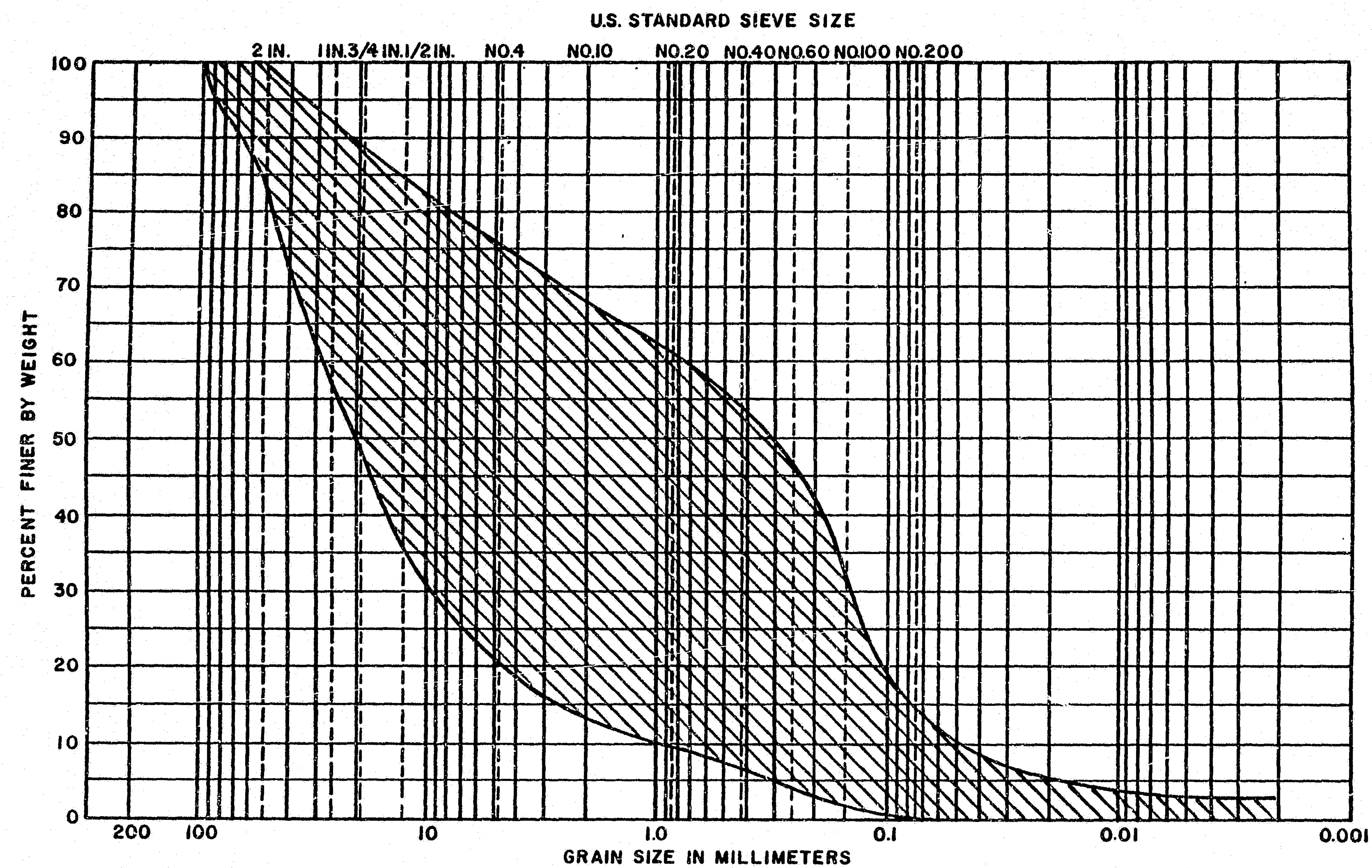


NOTE:
(EFFECTIVE STRESSES - CONSOLIDATED UNDRAINED SHEAR)

COMPOSITE SAMPLE
BORROW AREA D

FIGURE 6A

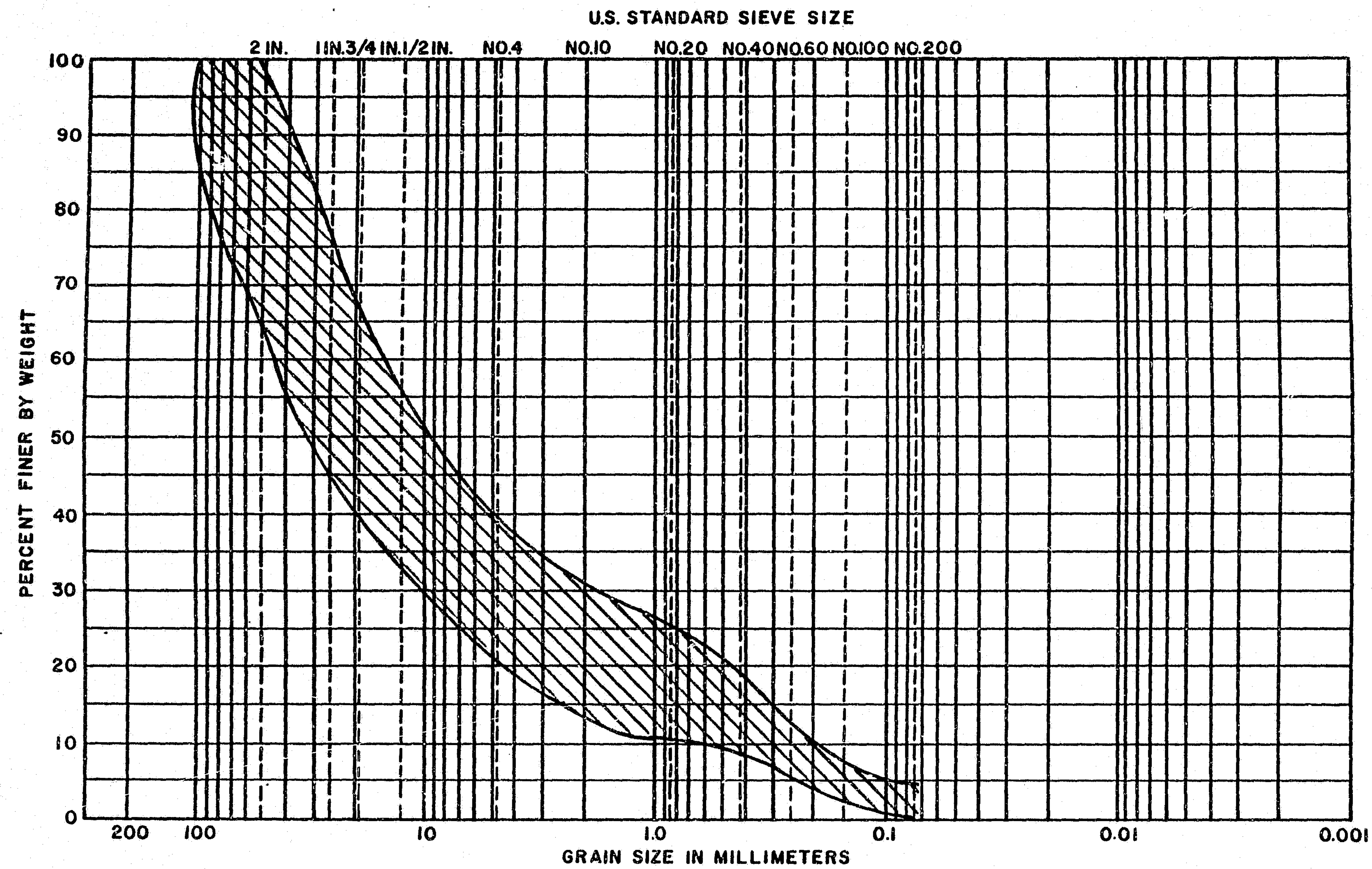
ACRES



COMPOSITE CURVES FOR AREA I

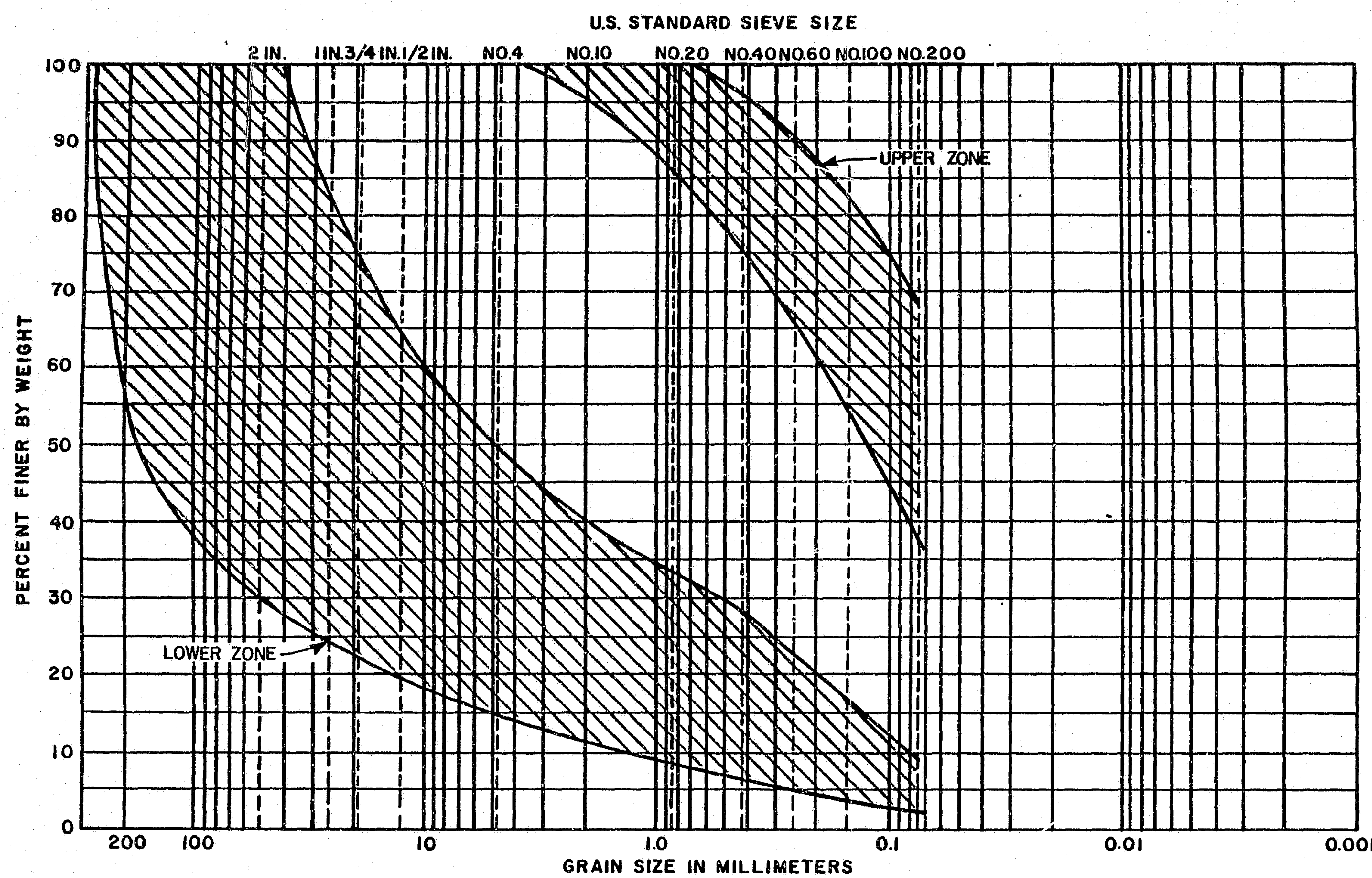
FIGURE 7

ACRES



COMPOSITE CURVES FOR AREA J

FIGURE 8



COMPOSITE CURVES FOR TP-E1 THRU TP-E21

FIGURE 9

ACRES

SUSITNA HYDROELECTRIC PROJECT

OUTLINE OF METHODOLOGY
FOR STABILITY ANALYSIS
WATANA DAM

1 - INTRODUCTION

A static and dynamic stability analyses of the proposed Watana Dam will be performed. The static analysis will be done using the FEADAM computer program (Finite Element Analysis of Dams) to determine the initial stresses, strains, and displacements in the dam during normal operating conditions. The dynamic analysis will be done using the QUAD 4 finite element program which incorporates the strain dependent shear modulus and damping parameters with the results from the static analyses.

2 - FINITE ELEMENT MODEL

The finite element model will consist of 20 layers of elements with approximately 550 nodes and 520 elements. Different soil parameters as described in the following sections have been chosen for the core, transition material and the shell material. The transition material will consist of the fine and coarse filter zones. A detailed finite element mesh has not been developed at this time.

3 - STATIC ANALYSIS

The static analysis using the finite element program for static analysis of earth and rockfill dams (FEADAM) will be done to determine the initial stresses in the dam during normal operating conditions. The program calculates the stresses, strains and displacements in the dam simulating the actual sequence of construction operations. Appropriate nonlinear and stress-dependent stress-strain properties for the soils were taken from information compiled in Table 5 in Duncan et al (1980). Table 1 presents the values which will be used in the analysis. Two analyses will be performed to show the effects of relatively soft versus stiff core material.

4 - DYNAMIC ANALYSIS

The dynamic analysis will be done using the QUAD 4 computer program. The initial values of shear modulus and damping ratio to be used in the analyses were derived from typical values available in Banerjee et al (1979) and are as follows:

ZONE	K_2	Damping Shear Type Curve
Core Material		
Soft	90	sand
Stiff	120	sand
Transition Material	150	sand
Shell Material	180	sand

The design earthquake time history was developed by Woodward Clyde Consultants and is shown on Figure No. 1. The significant features are as follows:

(a) magnitude 8.5 Richter;
(b) location 40 kilometers below site (Benioff Zone);
(c) maximum acceleration of 0.35g;
(d) duration of strong motion - 45 sec; and
(e) significant number of cycles - 25.

The preliminary dynamic analysis had peak output values occurring about 24 seconds into the earthquake acceleration time history. Based on these results, the three iterations for the proposed dynamic analysis will be performed using the following sections of the earthquake time history (see Figure No. 1):

Iteration No. 1 - from 10 to 30 seconds
Iteration No. 2 - from 10 to 30 seconds
Iteration No. 3 - from 10 to 70 seconds

It is expected that this will minimize cost and provide the required output for the preliminary dynamic analyses.

TABLE 1

γ	-	Unit Weight, pcf
K	-	Modulus Number, ksf
Kur	-	Elastic Unloading Modulus Number, ksf
n	-	Modulus Exponent
Rf	-	Failure Ratio
K	-	Bulk Modulus Number, ksf
m	-	Bulk Modulus Exponent
C	-	Cohesion, psf
ϕ	-	Friction Angle, degrees
$\Delta\phi$	-	Decrease in Friction Angle Perlog Cycle Increase in σ_3, degrees
Ko	-	Earth Pressure Coefficient

	γ	K	Kur	n	Rf	Kb	m	C	ϕ	$\Delta\phi$	Ko
CORE MATERIAL											
Soft	140	200	300	.8	.6	60	.8	0	35	0	.43
Stiff	140	700	800	.35	.8	280	.2	0	35	0	.43
Transition Material	145	1300	1500	.4	.72	900	.22	0	35	6	.43
Shell Material	145	1800	2000	.4	.67	1300	.16	0	35	6	.43

REFERENCES

1. Duncan, J.M., Byrne, P., Wong, K.S. and Mabry, P. (1980) "Strength, Stress-Strain and Bulk Modulus Parameters for Finite Element Analyses of Stresses and Movements in Soil Masses," Geotechnical Engineering Research Report No. UCB/GT/80-01, Department of Civil Engineering, University of California, Berkeley, August 1980.

2. Banerjee, N.G., Seed, H.B. and Chan, C.K. (1979) "Cyclic Behavior of Dense Coarse-Grained Material in Relation to The Seismic Stability of Dams", Earthquake Engineering Research Center Report No. UCB/EERC-79/13, College of Engineering, University of California, Berkeley, June 1979.

SUSITNA HYDROELECTRIC PROJECT

TASK 6 - DESIGN DEVELOPMENT

SUBTASK 6.25 - CLOSEOUT REPORT
OPTIMIZATION OF
DAM HEIGHTS

DECEMBER 1981

FIRST DRAFT

WATANA DAM HEIGHT OPTIMIZATION

The level of Watana reservoir and hence the crest elevation of the rockfill dam is determined on the basis of the relative costs of Susitna energy and the energy produced from the alternative cheapest but acceptable source; a mix of thermal installations and small hydroelectric developments. Average annual energies and construction and operating costs are derived for a Susitna Development with different Watana Dam heights and their impact is assessed on the overall cost of generation within the railbelt over a 50 year period.

Firm and average annual energies produced by the Susitna development are based on 30 years of hydrological records and are determined from computer simulations of both the Watana and Devil Canyon Reservoirs.

Operation of the Susitna development within the system is matched with existing and new generating sources, constructed as required, to meet the load demand on the system.

Different reservoir drawdowns have been examined at both developments and drawdowns producing the maximum firm energy consistent with economic costs of the intake structures have been selected. (See Section ___).

Certain minimum flows have been imposed at both project sites based on mechanical plant and fisheries related restrictions. (See Section ___).

In order to match system demand Susitna development is staged, with 900 MW of capacity coming on line at Watana in 1993 and 600 MW at Devil Canyon in the year 2000.

System demand and monthly and daily load patterns within the railbelt over a 50 year period are based on forecasts developed by "ISER" and "Woodward Clyde Consultants".

System present worth costs with different levels of Watana development within the system have been assessed by means of the OGP V computer programme. (See Section ___).

The overall development pattern of generation is determined for the railbelt based on the load and load pattern forecasts and the introduction within the system at required intervals of time of the most economic acceptable energy source.

The costs for the Watana and Devil Canyon projects have been based on the final conceptual layouts and latest construction rates and methods. Costs for construction and operation of alternative sources have been provided by 'Batelle'.

Optimization of the dam height was initially based on three dam heights with crest elevations at the center part of the dam 2,240', 2,190', and 2,140'. These crest elevations correspond to a maximum operating level of the reservoir of 2,215', 2,165', and 2,115', respectively. Firm and average energies were determined from the simulation model and are given below together with corresponding Susitna development costs.

The data from Table 1 was used as input to the OGP V computer programme to determine pattern of development of the system over 50 years; thence, to derive the present worth costs of constructing and operating the different system incorporating the corresponding levels of Watana development.

Present worths of the system are shown in Table 2.

The system present worths are plotted against dam height in Figure 1. The lowest system present worth (cost of producing a specific energy demand) appears to occur at crest elevation 2,190 ft (reservoir elevation 2,165 ft) and this represents the optimum elevation of the dam crest. At this point the cost benefit ratio, relative to alternative energy sources, of additional energy produced at Watana by raising the dam height is 1.0. As the dam height is raised above this point the cost benefit ratio of additional energy gradually increases making it uneconomic. As the dam height is lowered the cost benefit ratio of the incremental decrease in energy falls below 1.0, indicating that economic energy is not being developed.

REVIEW OF OPTIMIZATION

The three values of present worth from the OGP V computer runs poorly define the relationship between system present worths and Watana dam height. In order to examine further the cost of energy increments with variation of dam height the costs of the Susitna Development and the associated energies are examined separately from and compared graphically to the cost of usable alternative energy over the period considered.

The costs of different levels of project development are determined and average annual energies are determined from the simulation programme.

The costs of alternative energy were determined from the present worths already obtained from the OGP V programme as shown in Table 3.

It was anticipated that there would be little variation in the value of increments of alternative energy over a restricted range of dam heights as this energy would generally continue to be provided by a similar method of generation. A possible difference in the present worth would arise from timing of installation of generating capacity e.g. 200 GWH of coal fired thermal generation if introduced in 2005 would have a smaller present worth when attendant operating costs were considered than the same installation introduced in year 2000. Such a difference would be very minor however in the context of the 50 year period considered and over the small range of energies examined. This is born out by the values of $\$1.05 \times 10^6$ and $\$1.00 \times 10^6$ per GWH for incremental energy (the difference is well within the accuracy of OGP V). Hence over, the range of energies considered the value of equal increments of energy is assumed constant.

Average energies and costs for the Susitna Project corresponding to different Watana dam heights are shown in Table 4.

Present worth of alternative energy = $1.05 X 10^6/GWH.

The costs of the various levels of the Susitna Project are plotted against corresponding average annual energy in Figure 2. The value of alternative increments of energy is also plotted as a straight line together with lower and higher present worths to show dam height sensitivity. The gradient of these graphs indicates the present worth/GWH of incremental energy at a particular level of Susitna (or Watana) development.

The point at which the gradient of the curve matches the gradient of the alternative energy line is the point of optimum development (optimum dam height) at which the cost benefit ratio will be 1.

Results of the graphical solution of Figure 2 based on a replacement cost of $1.05 X 10^6/GWH for incremental energy show an optimum dam crest elevation of 2,193 ft.

As a measure of sensitivity, variations of approximately 30 percent in the cost of alternative energy were examined. A replacement enegy cost of $1.40 X 10^6 per GWH would give a crest elevation of 2,224 ft. An energy cost of $0.70 X 10^6 per GWH would give an elevation of 2,137 ft. Similar crest elevating would arise from approximately 30 percent decrease or increase, respectively in the cost of materials for constructing the additional dam height.

Conclusions: From the above the optimum level would be 2,193 ft.

On the right side of the reservoir there is a low saddle which dips to an elevation of 2,000 ft. If water levels rise above this a saddle dam would be required. Although the cost of this saddle dam has been included in the project estimates for higher elevations, it would involve construction in an area of the site otherwise unworked and the point at which the dam becomes necessary, would seem to provide a convenient physical limitation on reservoir and main dam height (providing the cost benefit ratio of the final increment of energy from raising the dam is close to 1.0). Hence, the elevation of the center of the dam has been set at elevation 2,210 ft (corresponding to 2,207 ft at the abutments). At this elevation reservoir levels will not reach the low spot on the saddle for floods of up to 1:10,000 year frequency.

Raising the dam to elevation 2,210 ft the cost of the last increment of energy gained is $1.20 X 10^6/GWH. The cost/benefit ratio of this energy is 1.05/1.20 = 0.875.

TABLE 1

Combined Watana and Devil Canyon Operation

Watana Dam Crest Elevation (ft MSL)	Watana*+ Cost ($ X 10^6)	Devil Canyon** Cost ($ X 10^6)	Total Cost ($ X 10^6)	Average Annual Energy (GWH)	Firm Annual Energy (GWH)
2,240 (2,215 ft reservoir elevation)	4,076	1,711	5,787	6,809	5,809
2,190 (2,165 ft reservoir elevation)	3,785	1,711	5,496	6,586	5,401
2,140 (2,115 ft reservoir elevation)	3,516	1,711	5,227	6,264	

Watana Project alone (prior to year 2000)

Crest Elevation (ft MSL)	Average Annual Energy (GWH)	Firm Annual Energy (GWH)
2,240	3,542	3,179
2,190	3,322	2,864
2,140	3,071	2,534

*Cost in January 1982 dollars.
+Original costs adjusted to exclude Watana relict channel cut-off but include drainage blanket.

TABLE 2

Watana Dam Crest Elevation (ft MSL)	System Present*+ Worth ($ X 10^6)
2,240 (reservoir elevation 2,215 ft)	7,110
2,190 (reservoir elevation 2,165 ft)	7,053
2,140 (reservoir elevation 2,115 ft)	7,099

*January 1982 dollars.
+Original costs adjusted to exclude Watana relict channel cut-off but include drainage blanket.

TABLE 3

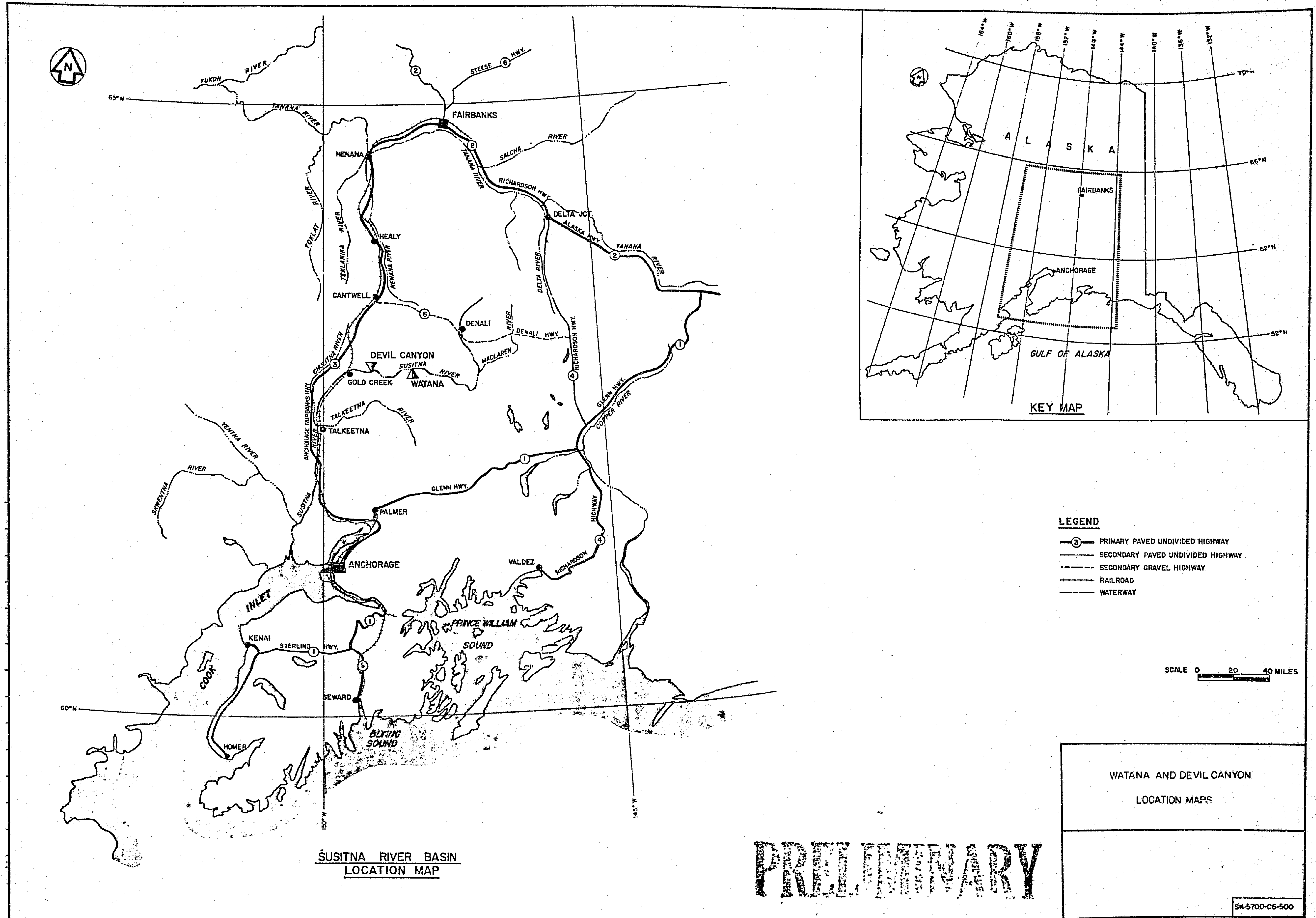
Watana Dam Crest Elevation (ft MSL)	System Present Worth ($ X 10^6)	(a) Present Worth ($ X 10^6)	Average Susitna Engery (GWH)	(b) Susitna Energy (GWH)	Susitna Capital Cost ($ X 10^6)	(c) Watana Cost ($ X 10^6)	$\frac{c-a}{b}$* ($ X 10^6/GWH
2,240	7,110	57.0	6,809	223	5,787	291	1.05
2,190	7,053	-54.0	6,586	322	5,496	269	1.00
2,140	7,099		6,264		5,227		

*$\frac{c-a}{b}$ = Cost/GWH of incremental alternative energy.

TABLE 4

Watana Dam Crest Elevation (ft MSL)	Susitna Capital Cost ($ X 10^6)	Average Annual Energy (GWH)
2,240	5,787	6,809
2,215	5,635	6,697
2,190	5,496	6,586
2,165	5,357	6,423
2,140	5,227	6,264

6790
6264
526



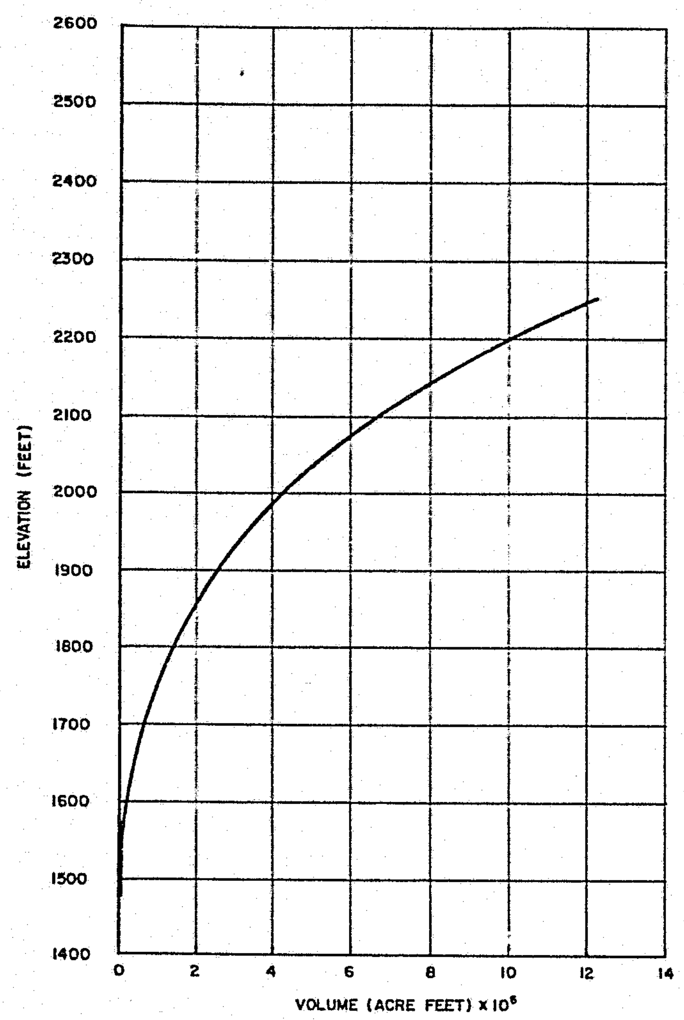
PRELIMINARY

WATANA AND DEVIL CANYON

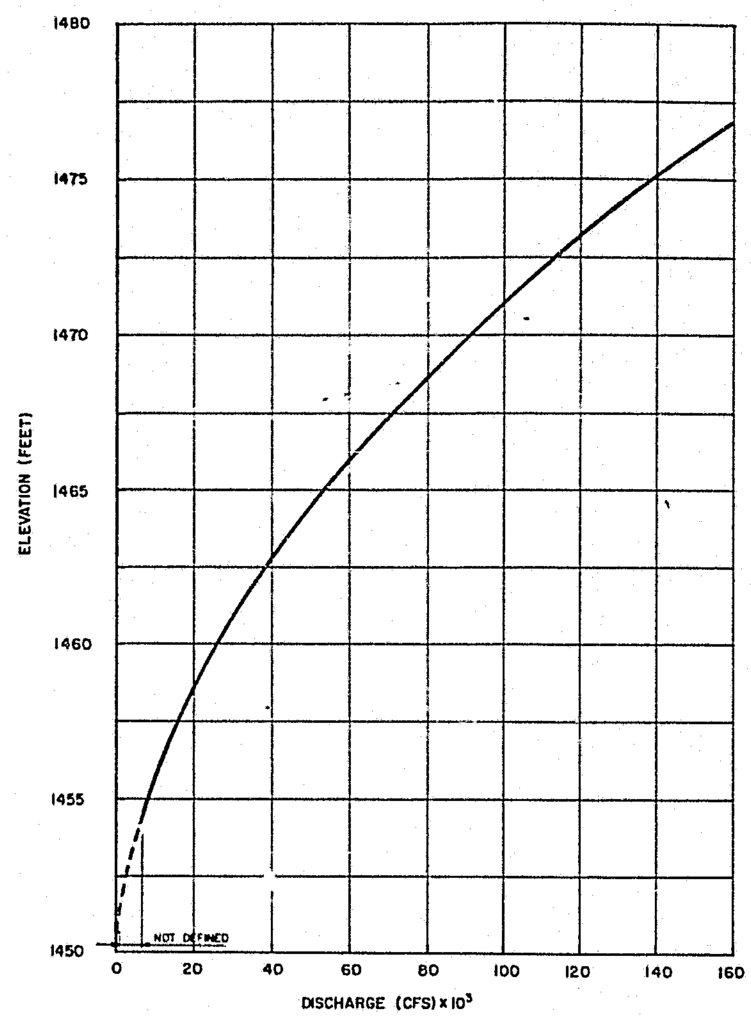
LOCATION MAPS

SK-5700-C6-500

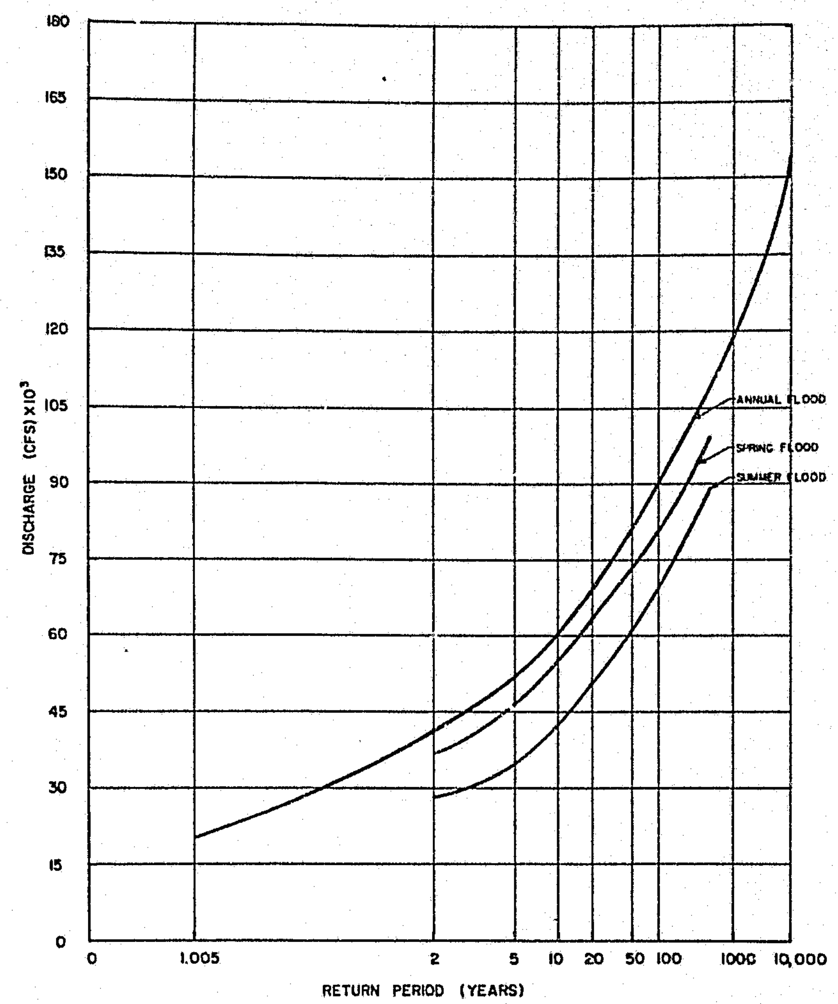
0 0 1 3 8 1



RESERVOIR VOLUME



TAILWATER RATING

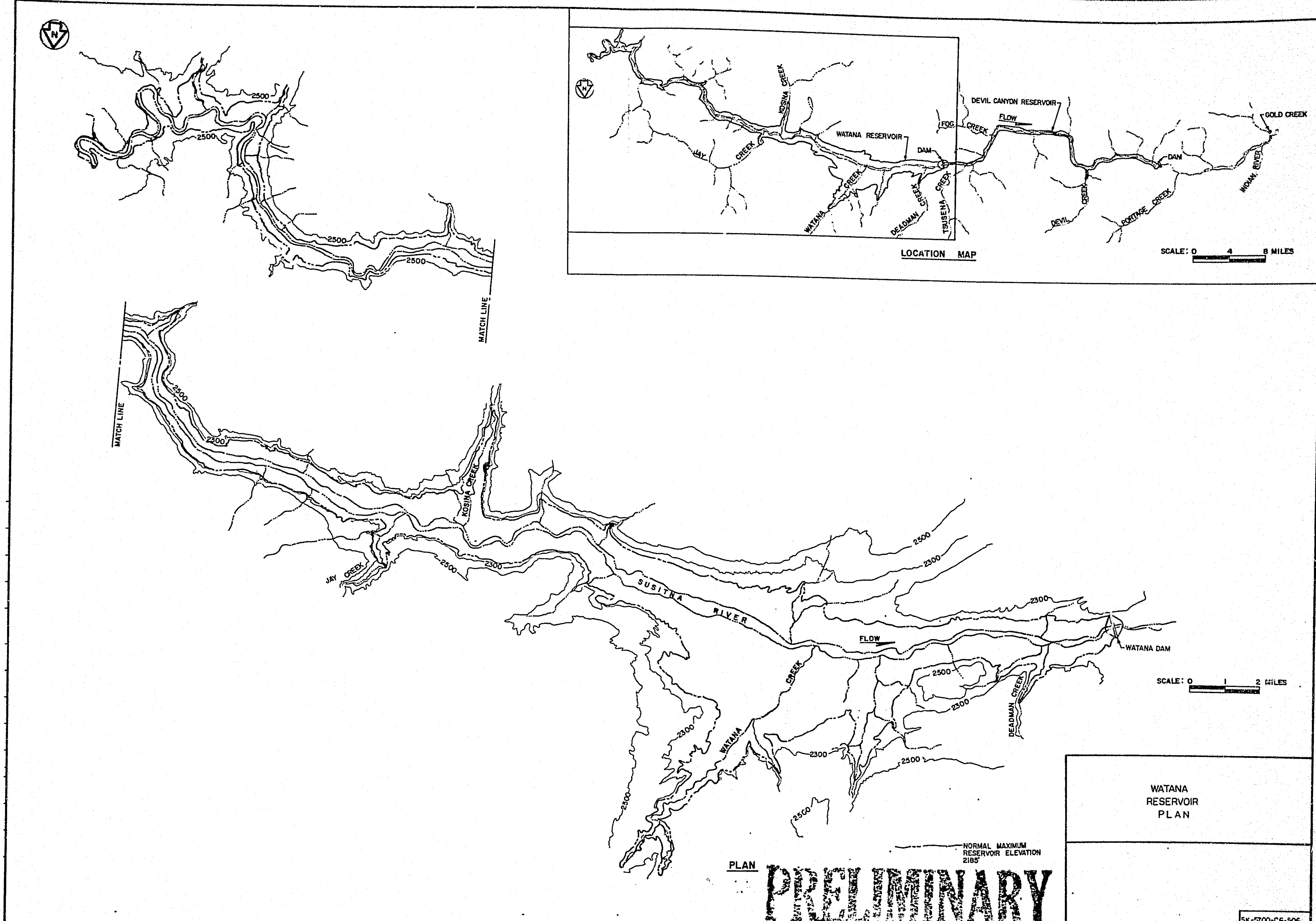


FLOOD FREQUENCY CURVE

WATANA
HYDROLOGY
SHEET 1 OF 4

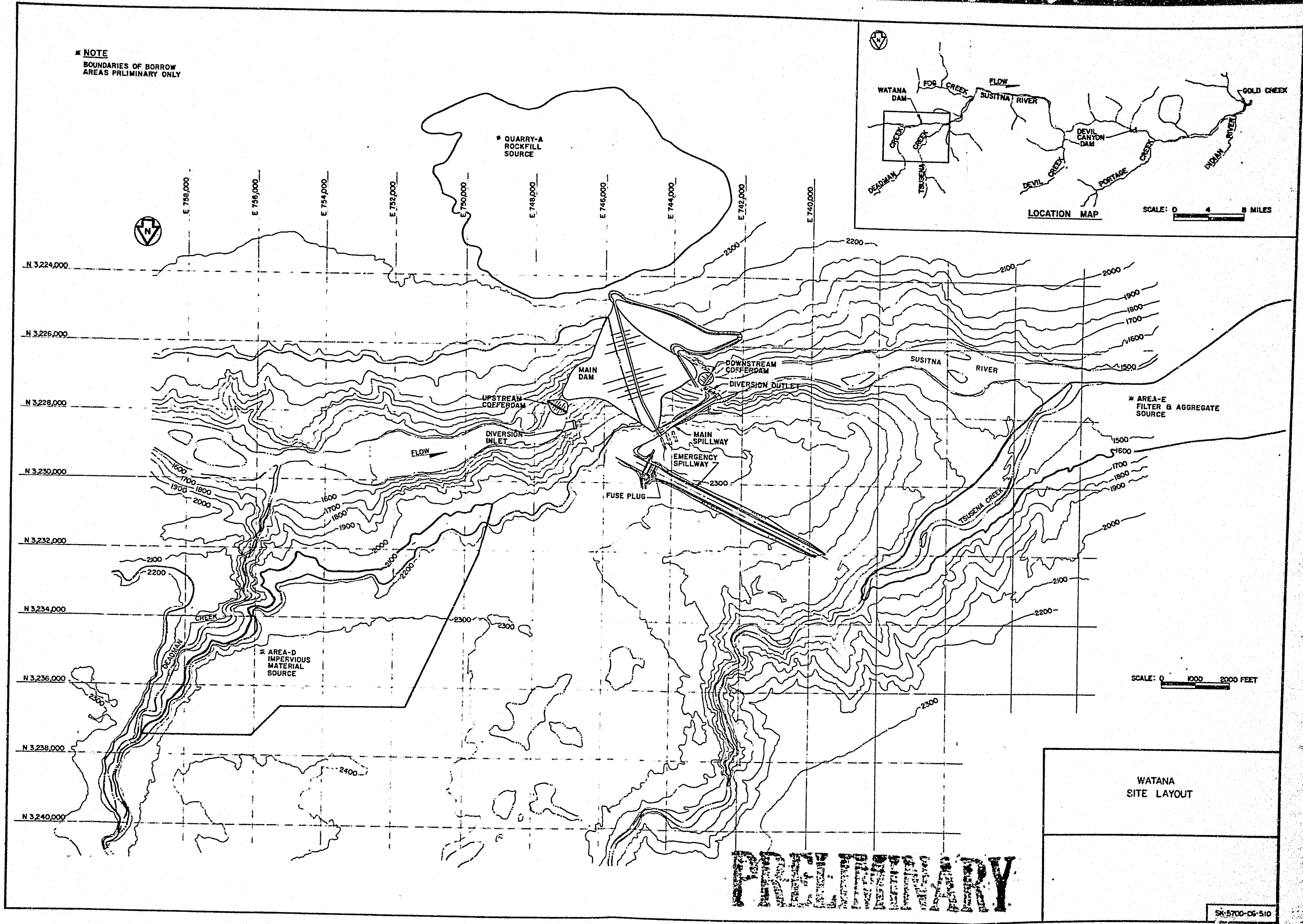
PRELIMINARY

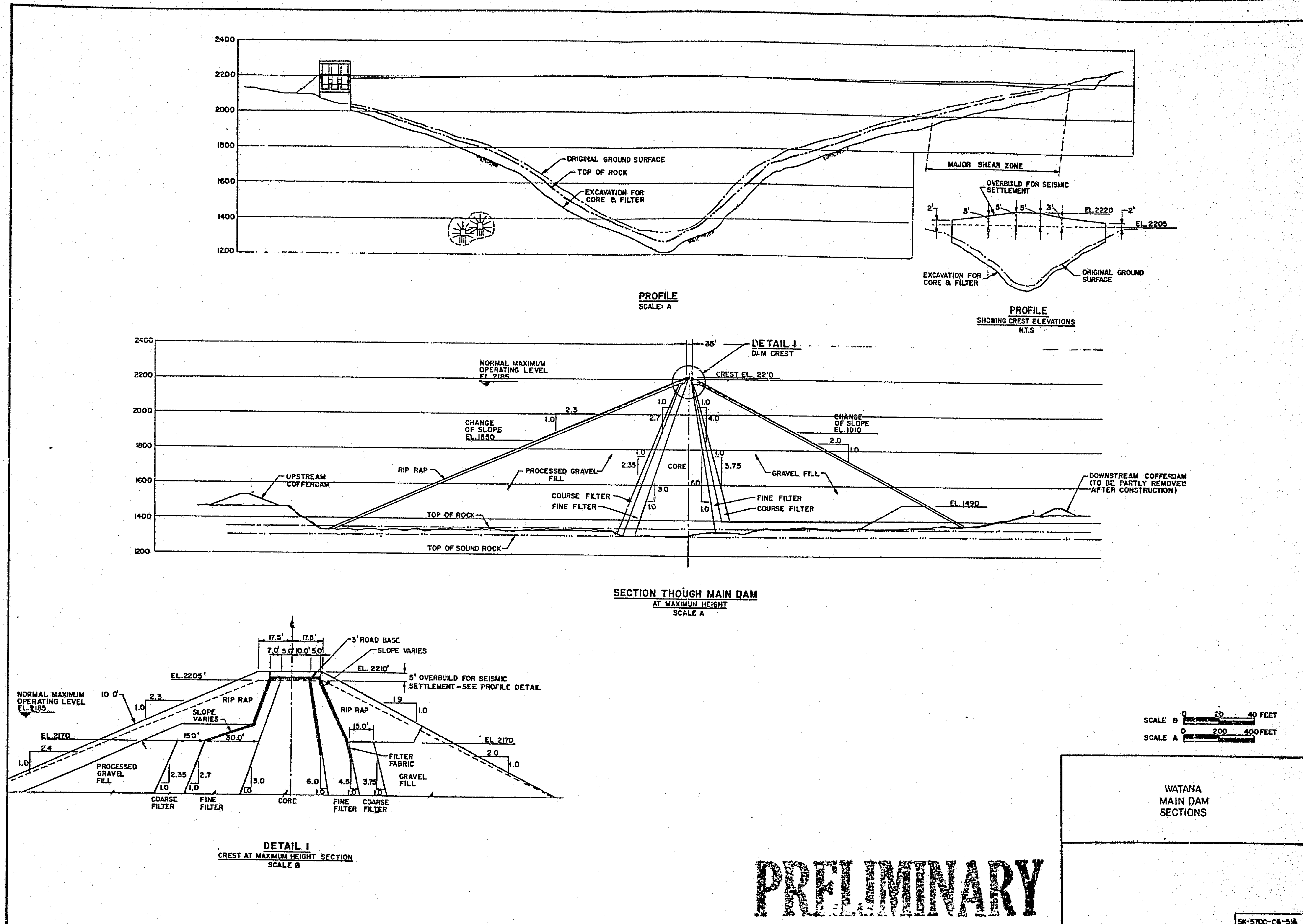
SK-5700-C6-501

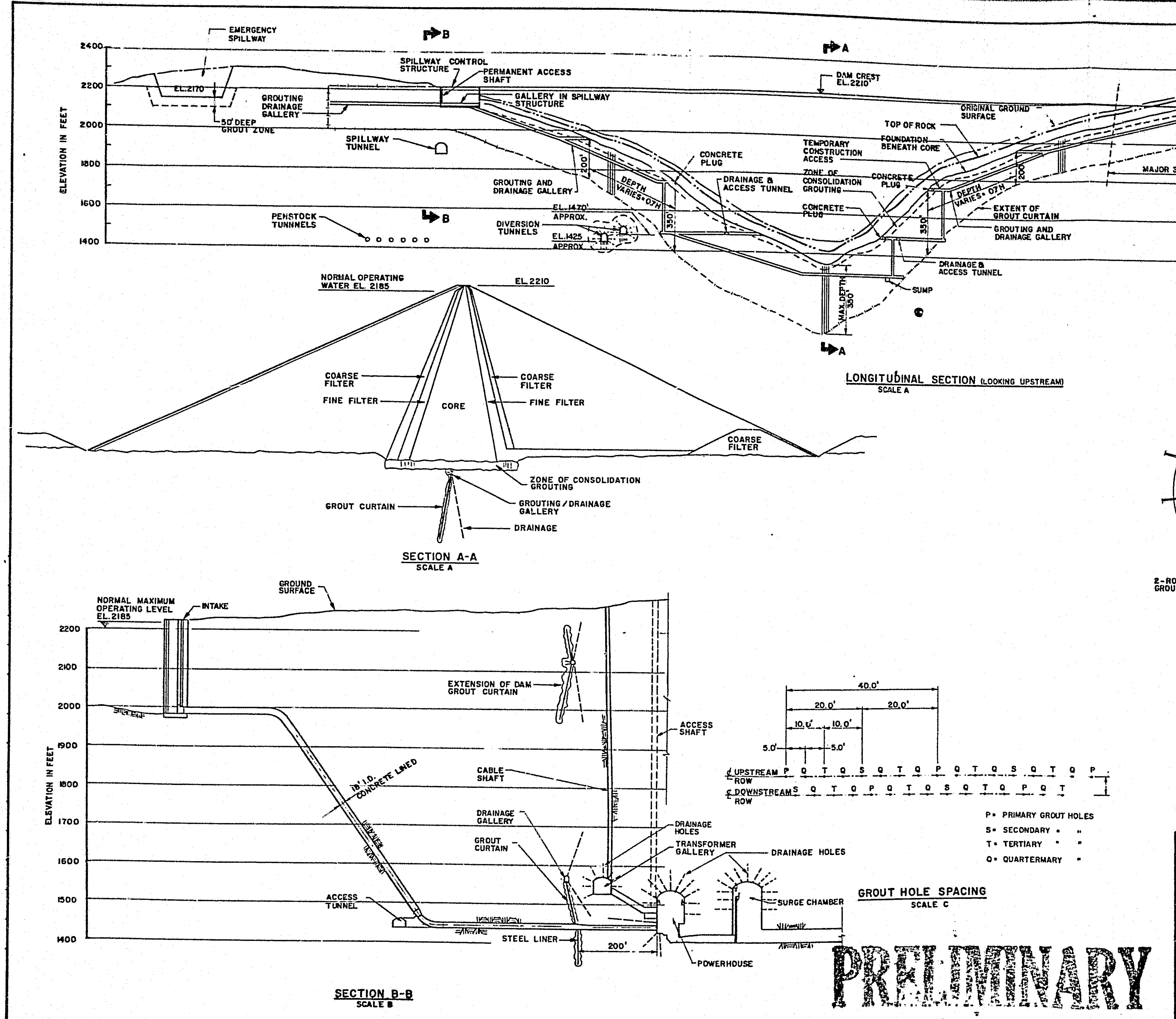


WATANA
RESERVOIR
PLAN

5K-5700-C6-506


* NOTE
BOUNDARIES OF BORROW AREAS PRLIMINARY ONLY
* QUARRY-A ROCKFILL SOURCE
E 758,000
E 756,000
E 754,000
E 752,000
E 750,000
E 748,000
E 746,000
E 744,000
E 742,000
E 740,000
N 3,224,000
N 3,226,000
N 3,228,000
N 3,230,000
N 3,232,000
N 3,234,000
N 3,236,000
N 3,238,000
N 3,240,000
MAIN DAM
UPSTREAM COFFERDAM
DOWNSTREAM COFFERDAM
DIVERSION OUTLET
DIVERSION INLET
FLOW
MAIN SPILLWAY
EMERGENCY SPILLWAY
FUSE PLUG
SUSITNA
RIVER
* AREA-E FILTER & AGGREGATE SOURCE
TSUSENA CREEK
DEADMAN
CREEK
* AREA-D IMPERVIOUS MATERIAL SOURCE
WATANA DAM
FOG CREEK
SUSITNA RIVER
GOLD CREEK
DEVIL CANYON DAM
DEVIL CREEK
PORTAGE CREEK
INDIAN RIVER
DEADMAN CREEK
TSUSENA CREEK
LOCATION MAP
SCALE: 0 4 8 MILES
SCALE: 0 1000 2000 FEET
WATANA
SITE LAYOUT
PRELIMINARY
SK-5700-C6-510

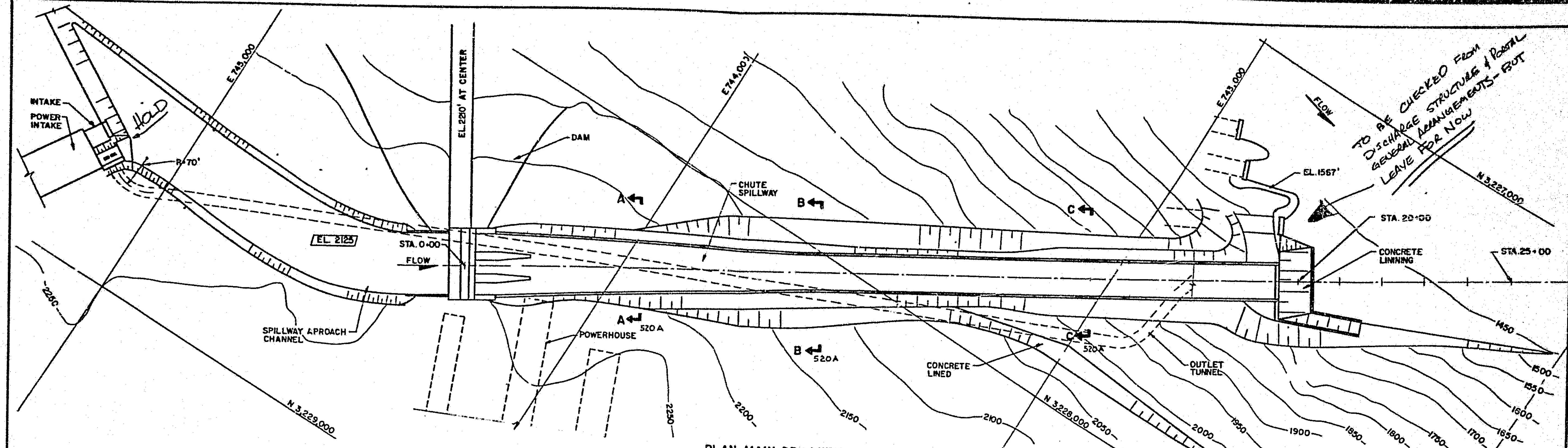

ORIGINAL GROUND SURFACE
TOP OF ROCK
EXCAVATION FOR CORE & FILTER
MAJOR SHEAR ZONE
OVERBUILD FOR SEISMIC SETTLEMENT
EL.2220
EL.2205
EXCAVATION FOR CORE & FILTER
ORIGINAL GROUND SURFACE
PROFILE
SCALE: A
PROFILE
SHOWING CREST ELEVATIONS
N.T.S
DETAIL I
DAM CREST
NORMAL MAXIMUM OPERATING LEVEL EL.2185
CREST EL. 2210
CHANGE OF SLOPE EL.1850
CHANGE OF SLOPE EL.1910
RIP RAP
PROCESSED GRAVEL FILL
CORE
GRAVEL FILL
UPSTREAM COFFERDAM
DOWNSTREAM COFFERDAM (TO BE PARTLY REMOVED AFTER CONSTRUCTION)
COURSE FILTER
FINE FILTER
FINE FILTER
COURSE FILTER
EL.1490
TOP OF ROCK
TOP OF SOUND ROCK
SECTION THOUGH MAIN DAM
AT MAXIMUM HEIGHT
SCALE A
3' ROAD BASE
SLOPE VARIES
EL.2205'
EL.2210'
5' OVERBUILD FOR SEISMIC SETTLEMENT - SEE PROFILE DETAIL
NORMAL MAXIMUM OPERATING LEVEL EL.2185
RIP RAP
SLOPE VARIES
RIP RAP
EL.2170
EL.2170
FILTER FABRIC
PROCESSED GRAVEL FILL
GRAVEL FILL
COARSE FILTER
FINE FILTER
CORE
FINE FILTER
COARSE FILTER
DETAIL I
CREST AT MAXIMUM HEIGHT SECTION
SCALE B
SCALE B 0 20 40 FEET
SCALE A 0 200 400 FEET
WATANA
MAIN DAM
SECTIONS
PRELIMINARY
SK-5700-C6-516



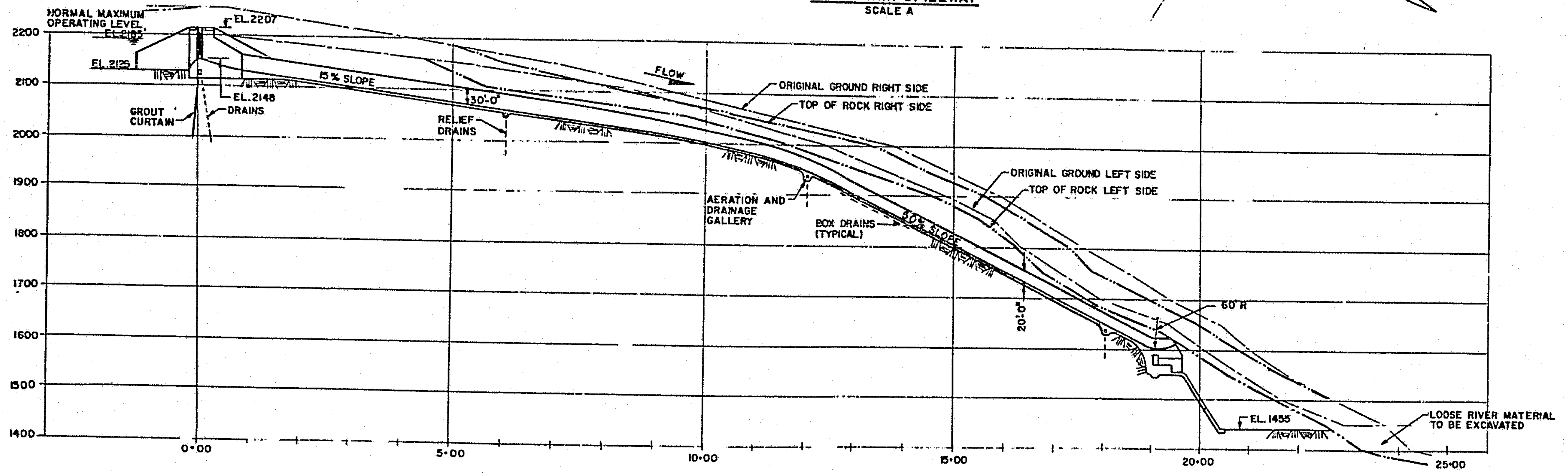
WATANA
MAIN DAM
GROUTING & DRAINAGE

PRELIMINARY

SK-5700-C6-518



PLAN-MAIN SPILLWAY
SCALE A



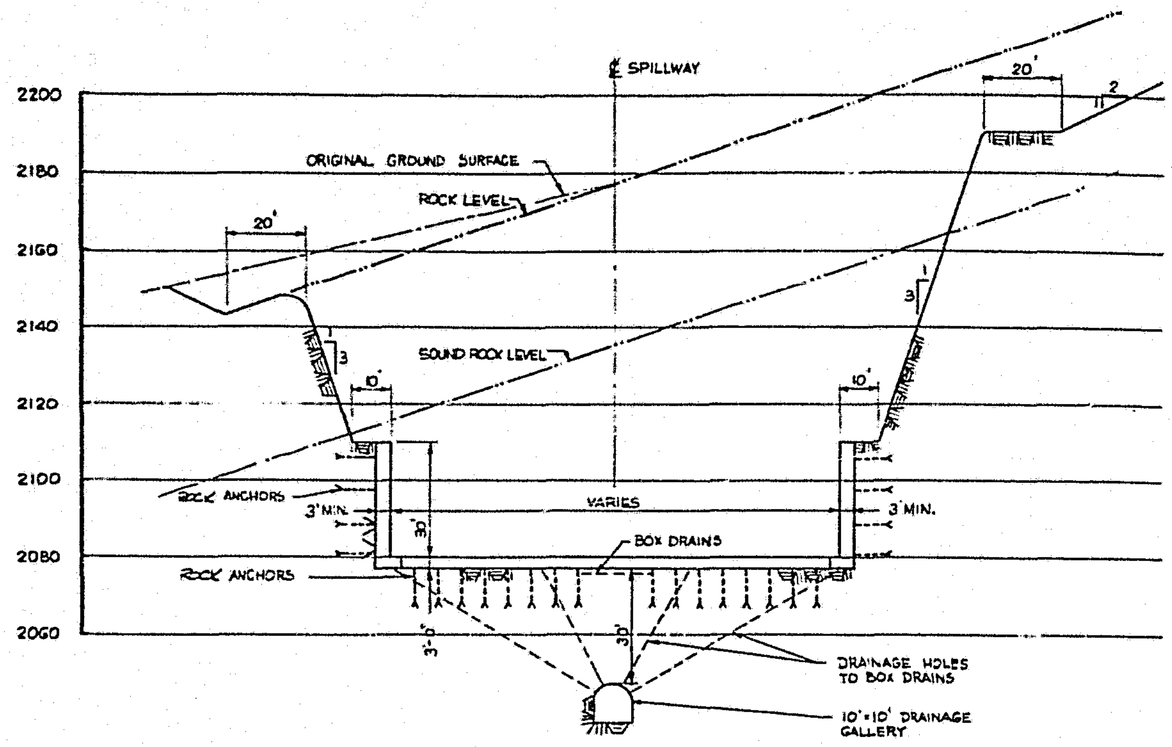
PROFILE MAIN SPILLWAY
SCALE A

SCALE A 0 100 200

WATANA
MAIN SPILLWAY
GENERAL ARRANGEMENT
PLAN & PROFILE

PRELIMINARY

SK-5700-C8-B19



SECTION A-A
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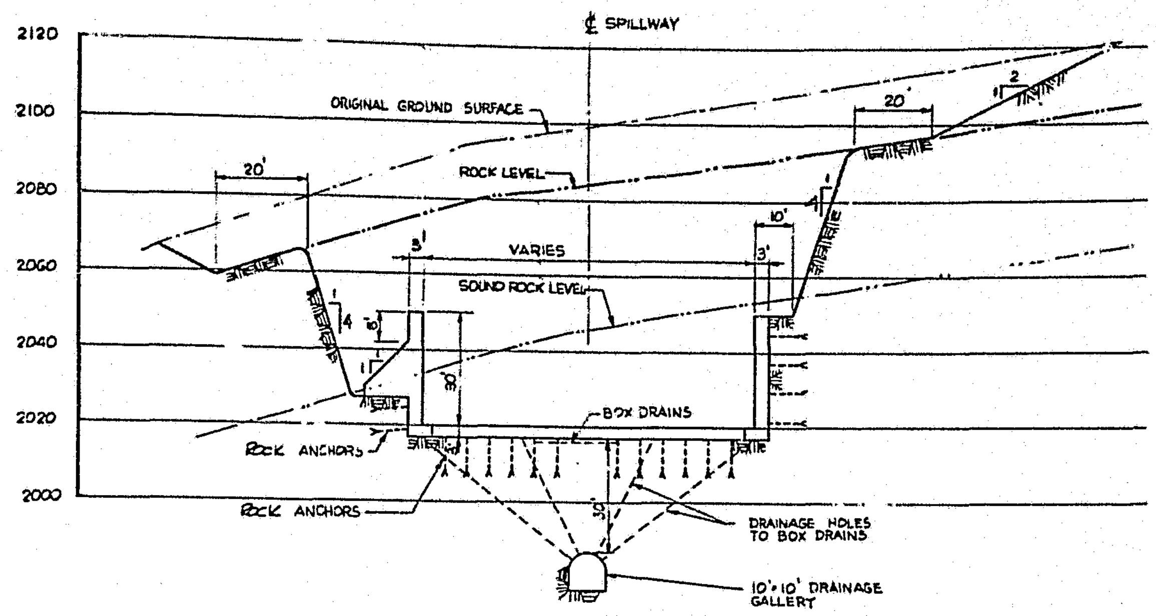
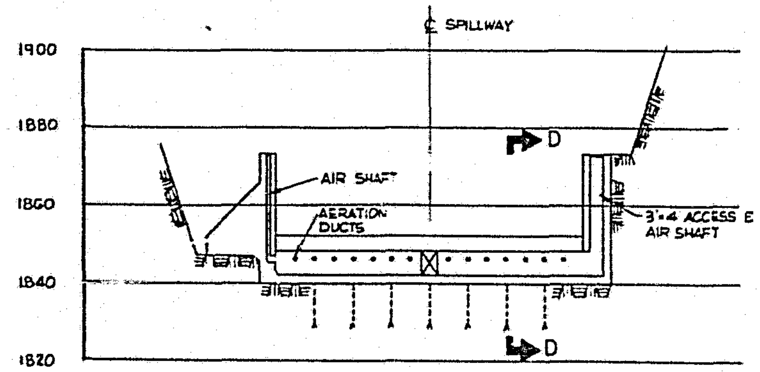
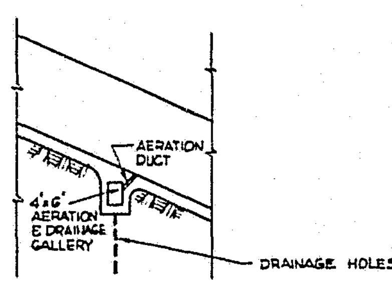


SECTION B-B
SCALE A



SECTION C-C
SCALE A



SECTION D-D
SCALE

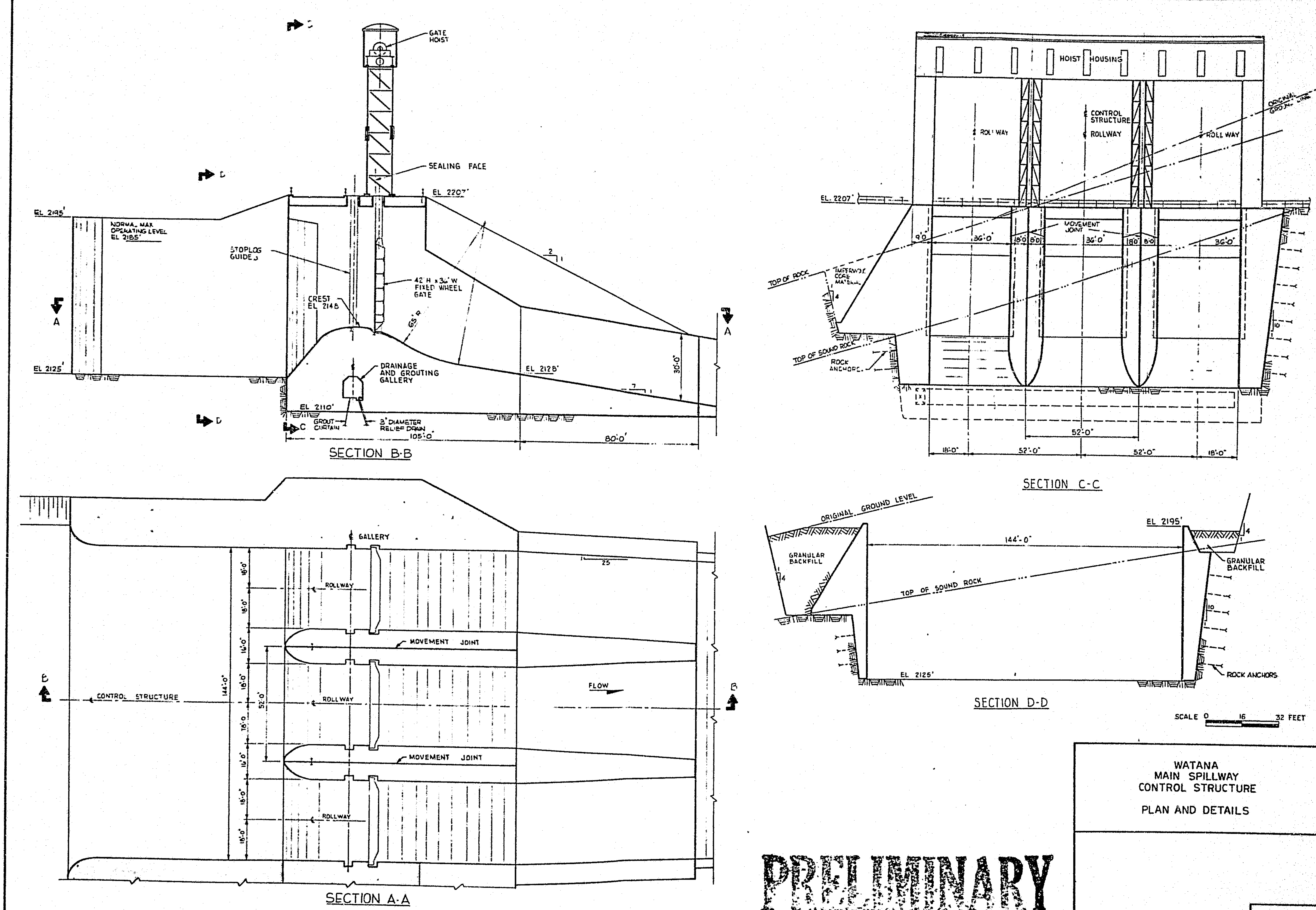
AERATION GALLERY

SCALE A 0 20 40

WATANA
MAIN SPILLWAY
SECTIONS

PRELIMINARY

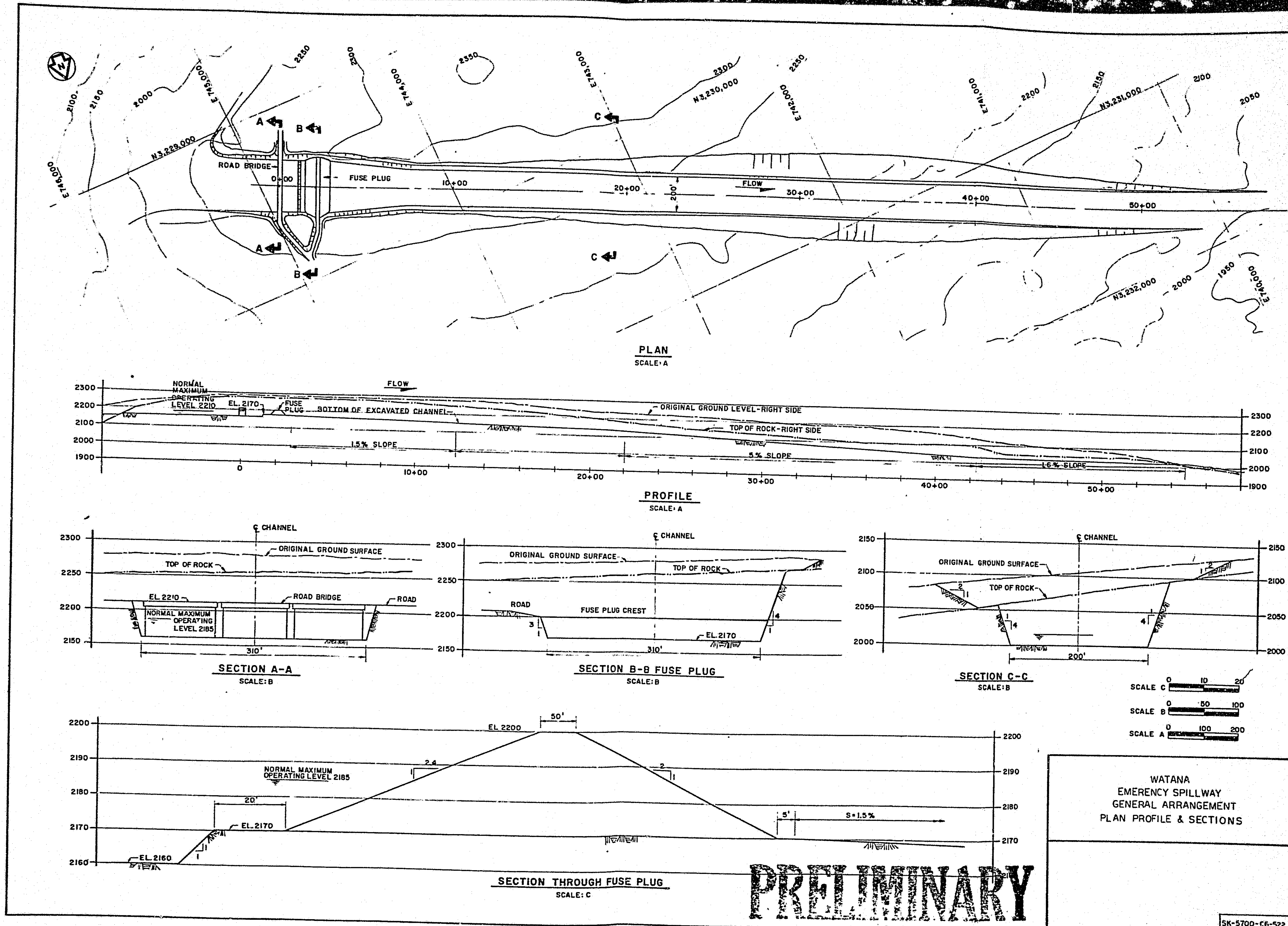
SK-5700-C6-520A



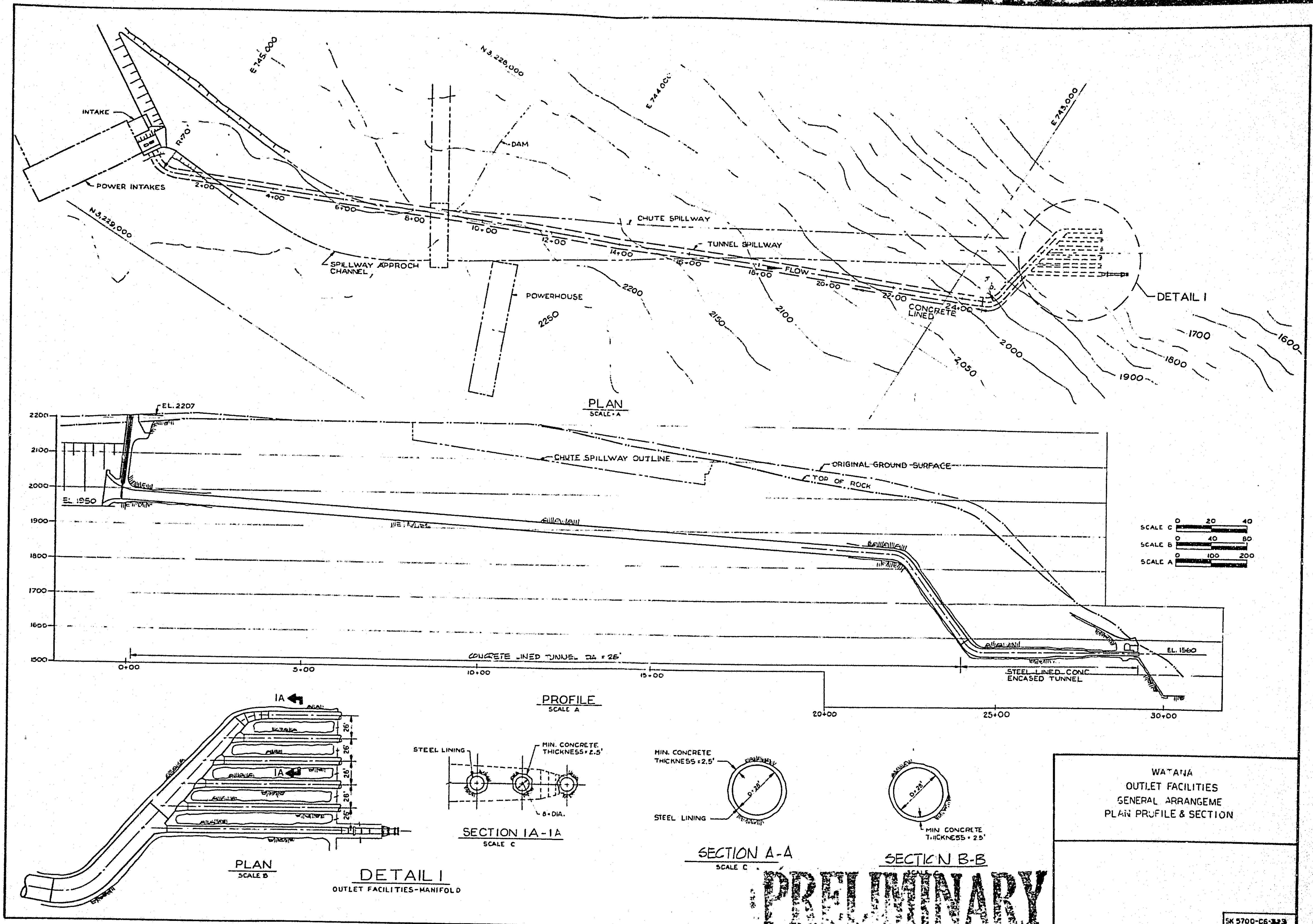
WATANA
MAIN SPILLWAY
CONTROL STRUCTURE

PLAN AND DETAILS

SK-5700-C6-521

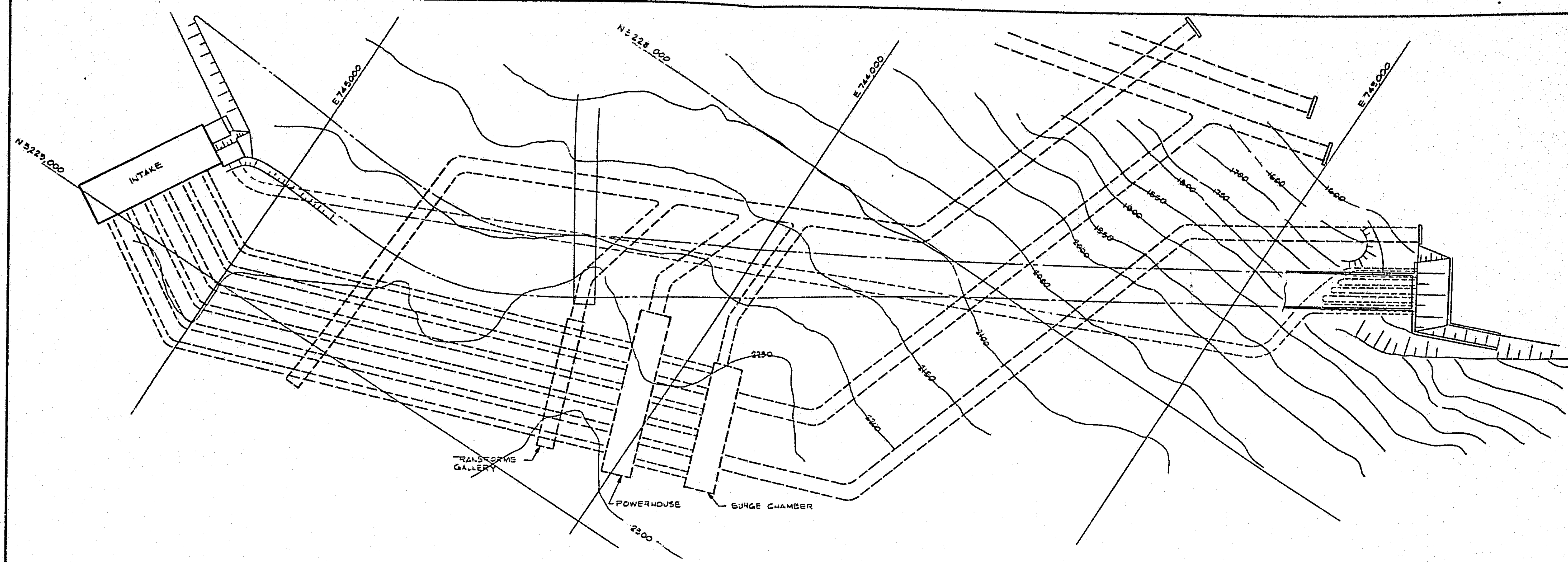


N3,229,000
N3,230,000
N3,231,000
N3,232,000
E746,000
E745,000
E744,000
E743,000
E742,000
E741,000
E740,000
ROAD BRIDGE
FUSE PLUG
FLOW
0+00
10+00
20+00
30+00
40+00
50+00
200'
PLAN
SCALE: A
NORMAL MAXIMUM OPERATING LEVEL 2210
EL. 2170
FUSE PLUG
BOTTOM OF EXCAVATED CHANNEL
ORIGINAL GROUND LEVEL-RIGHT SIDE
TOP OF ROCK-RIGHT SIDE
1.5% SLOPE
5% SLOPE
1.6% SLOPE
PROFILE
SCALE: A
℄ CHANNEL
ORIGINAL GROUND SURFACE
TOP OF ROCK
EL 2210
ROAD BRIDGE
ROAD
NORMAL MAXIMUM OPERATING LEVEL 2185
310'
SECTION A-A
SCALE: B
FUSE PLUG CREST
EL.2170
SECTION B-B FUSE PLUG
SCALE: B
200'
SECTION C-C
SCALE: B
SCALE C 0 10 20
SCALE B 0 50 100
SCALE A 0 100 200
EL 2200
50'
NORMAL MAXIMUM OPERATING LEVEL 2185
20'
EL.2170
EL.2160
5'
S=1.5%
SECTION THROUGH FUSE PLUG
SCALE: C
PRELIMINARY
WATANA
EMERENCY SPILLWAY
GENERAL ARRANGEMENT
PLAN PROFILE & SECTIONS
SK-5700-C6-522

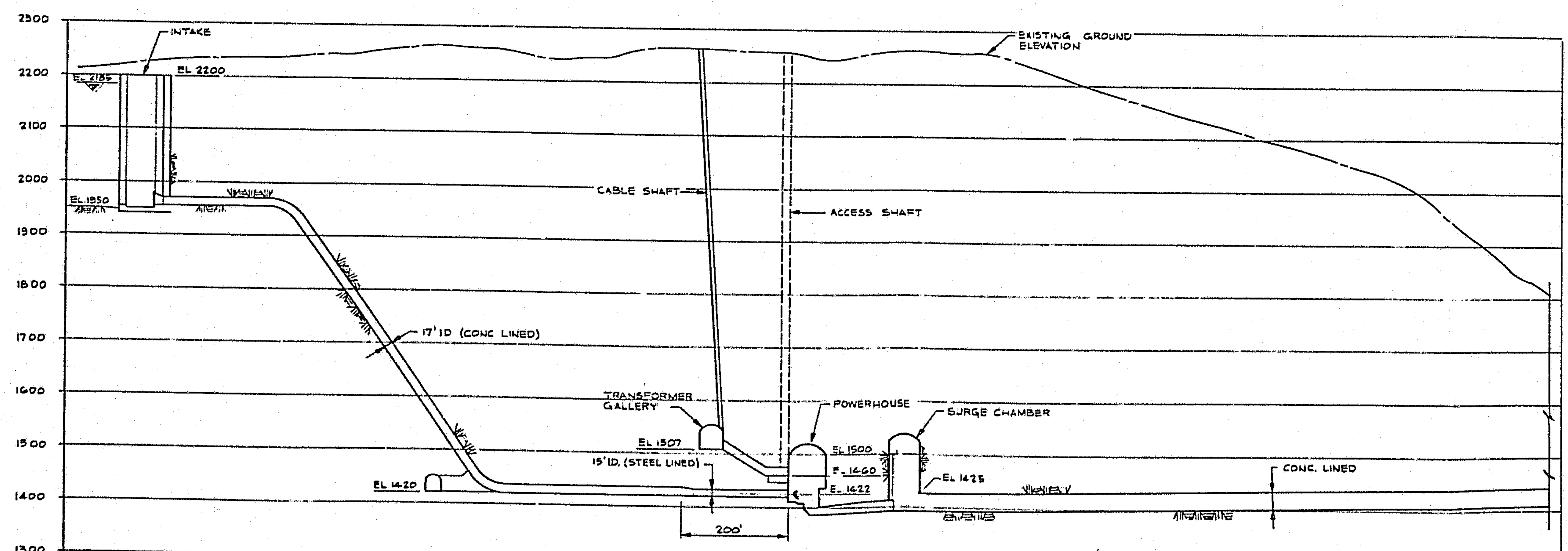


INTAKE
POWER INTAKES
DAM
CHUTE SPILLWAY
TUNNEL SPILLWAY
SPILLWAY APPROCH CHANNEL
POWERHOUSE
FLOW
CONCRETE LINED
DETAIL I
PLAN
SCALE A
EL. 2207
CHUTE SPILLWAY OUTLINE
ORIGINAL GROUND SURFACE
TOP OF ROCK
EL. 1950
EL. 1560
CONCRETE LINED TUNNEL DIA = 26'
STEEL LINED CONC ENCASED TUNNEL
PROFILE
SCALE A
SCALE C
SCALE B
SCALE A
PLAN
SCALE B
DETAIL I
OUTLET FACILITIES-MANIFOLD
STEEL LINING
MIN. CONCRETE THICKNESS = 2.5'
8-DIA.
SECTION IA-IA
SCALE C
MIN. CONCRETE THICKNESS = 2.5'
STEEL LINING
SECTION A-A
SCALE C
MIN CONCRETE THICKNESS = 2.5'
SECTION B-B
WATANA
OUTLET FACILITIES
GENERAL ARRANGEME
PLAN PROFILE & SECTION
PRELIMINARY
SK 5700-C6-323

001381



PLAN

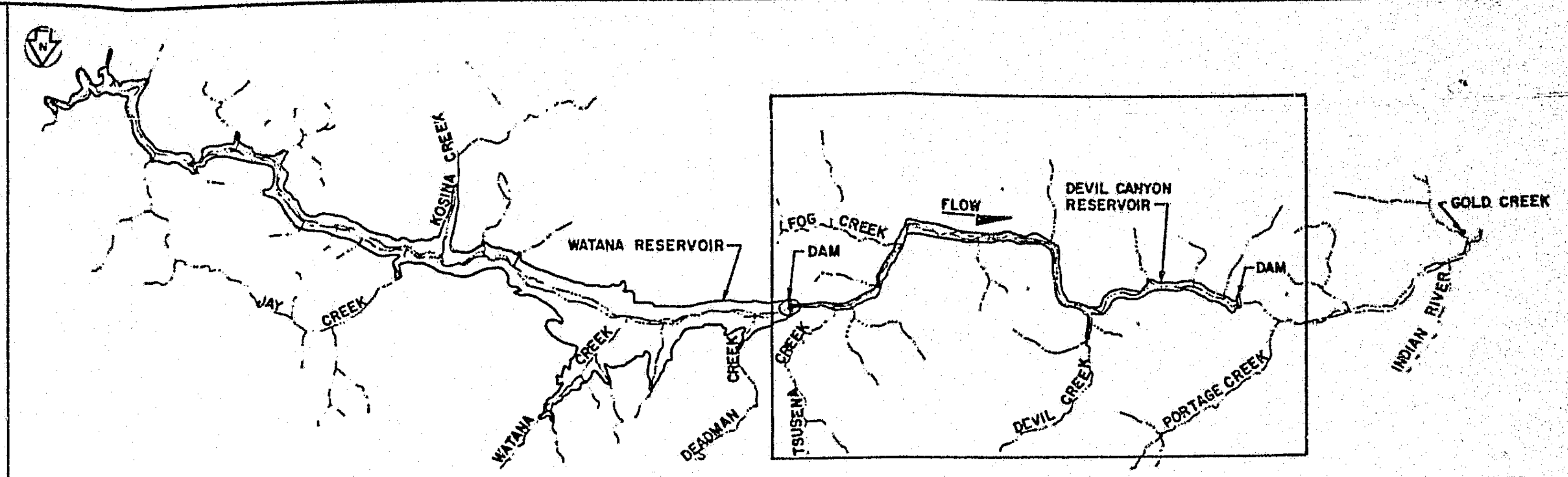


PROFILE

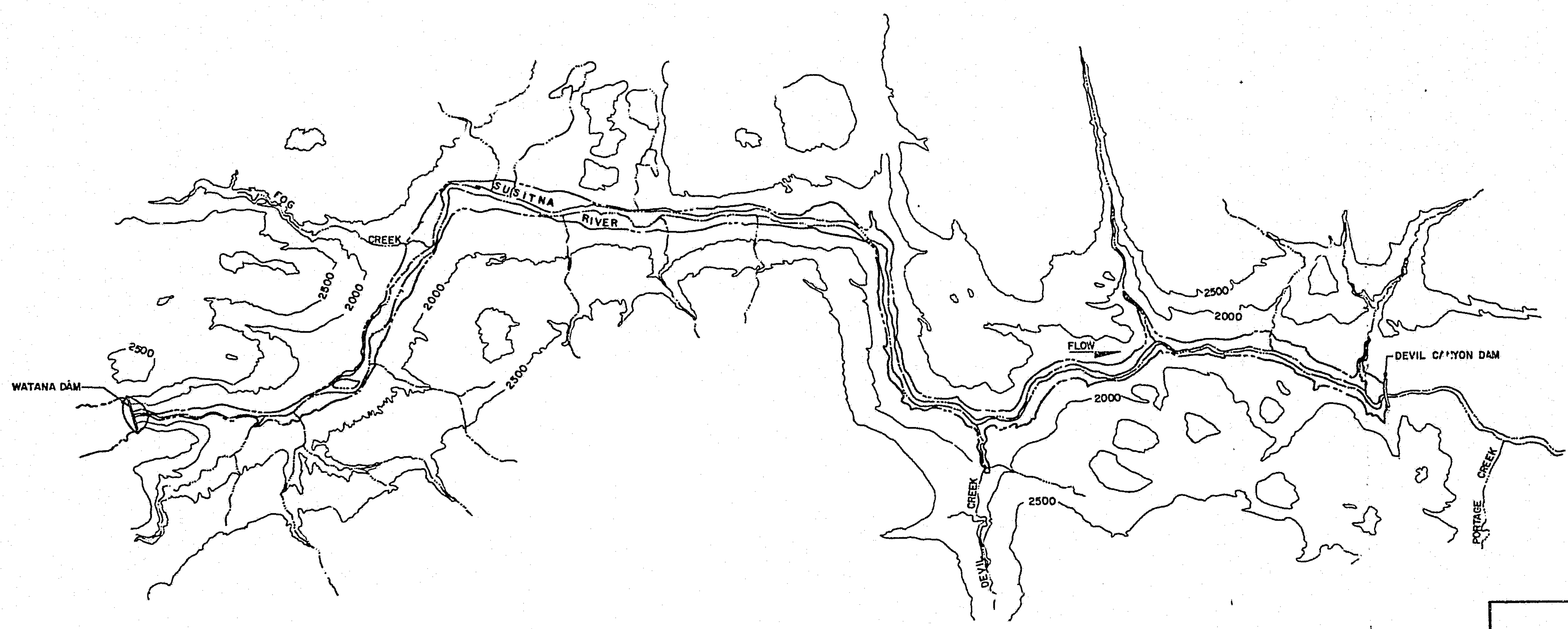
PRELIMINARY

WANTANA POWER FACILITIES
GENERAL ARRANGEMENT
PLAN & PROFILE

SK-5700-C6-526



LOCATION MAP



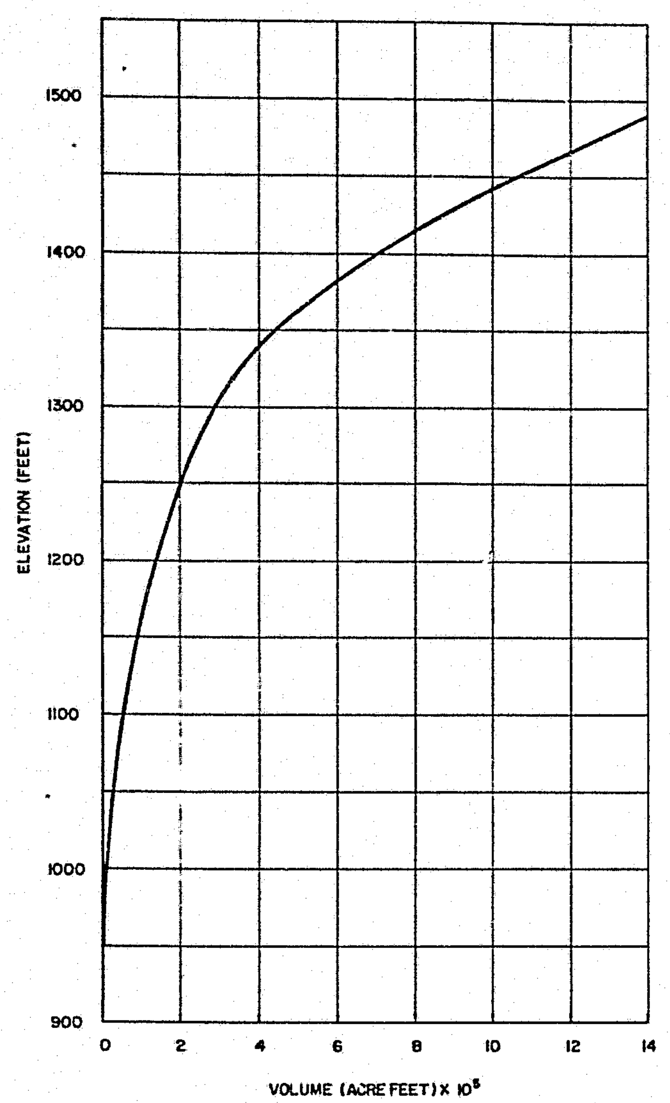
PLAN

NORMAL MAXIMUM RESERVOIR ELEVATION 1455'

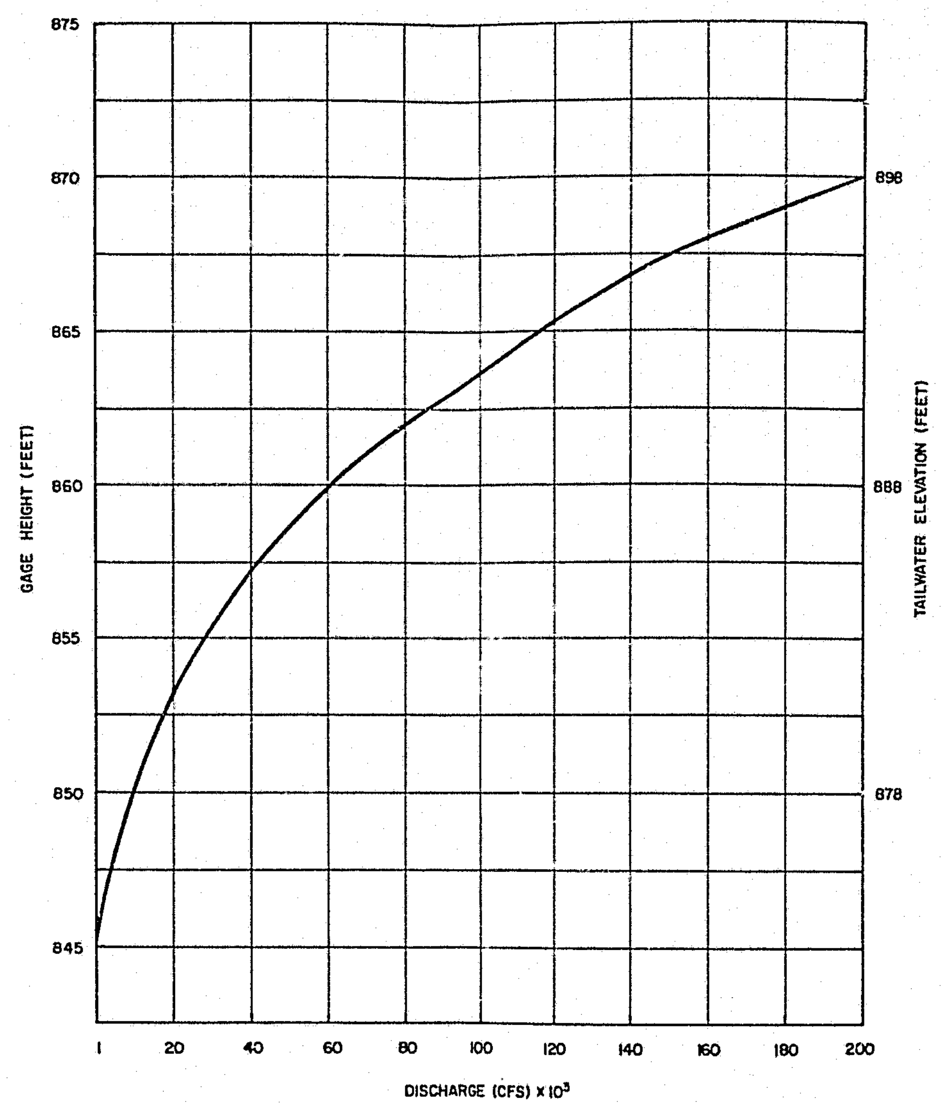
PRELIMINARY

DEVIL CANYON
RESERVOIR
PLAN

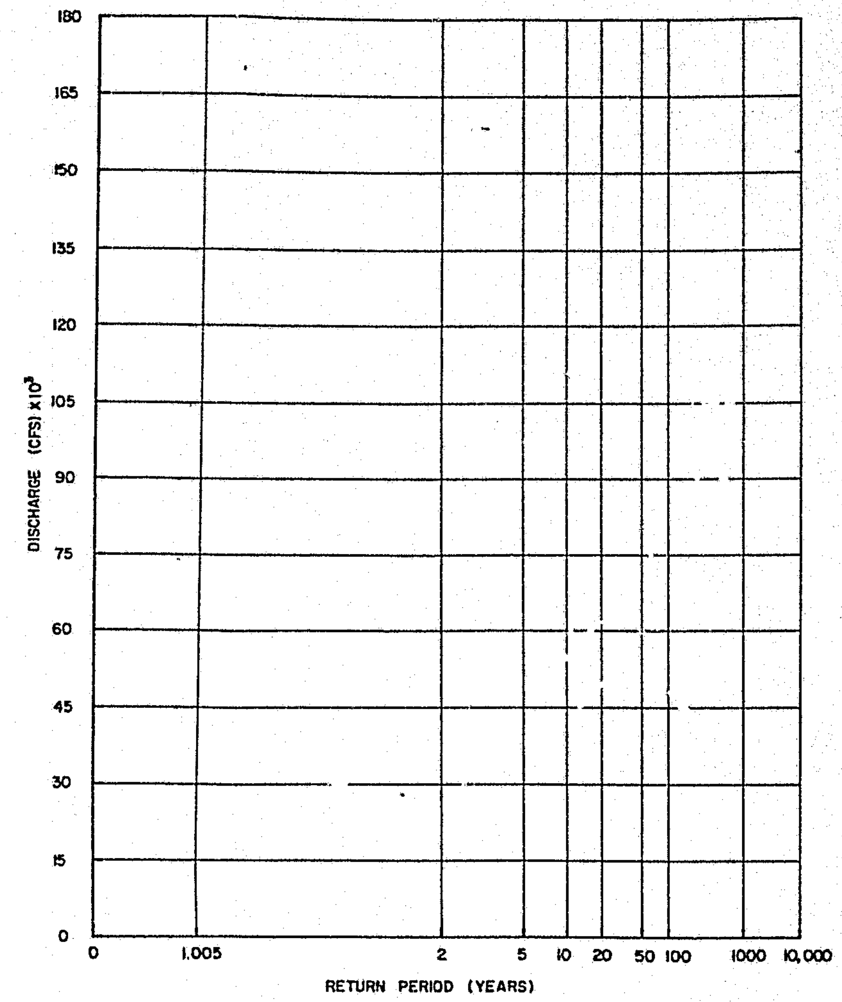
SK 5700 CG 601



RESERVOIR VOLUME



TAILWATER RATING CURVE

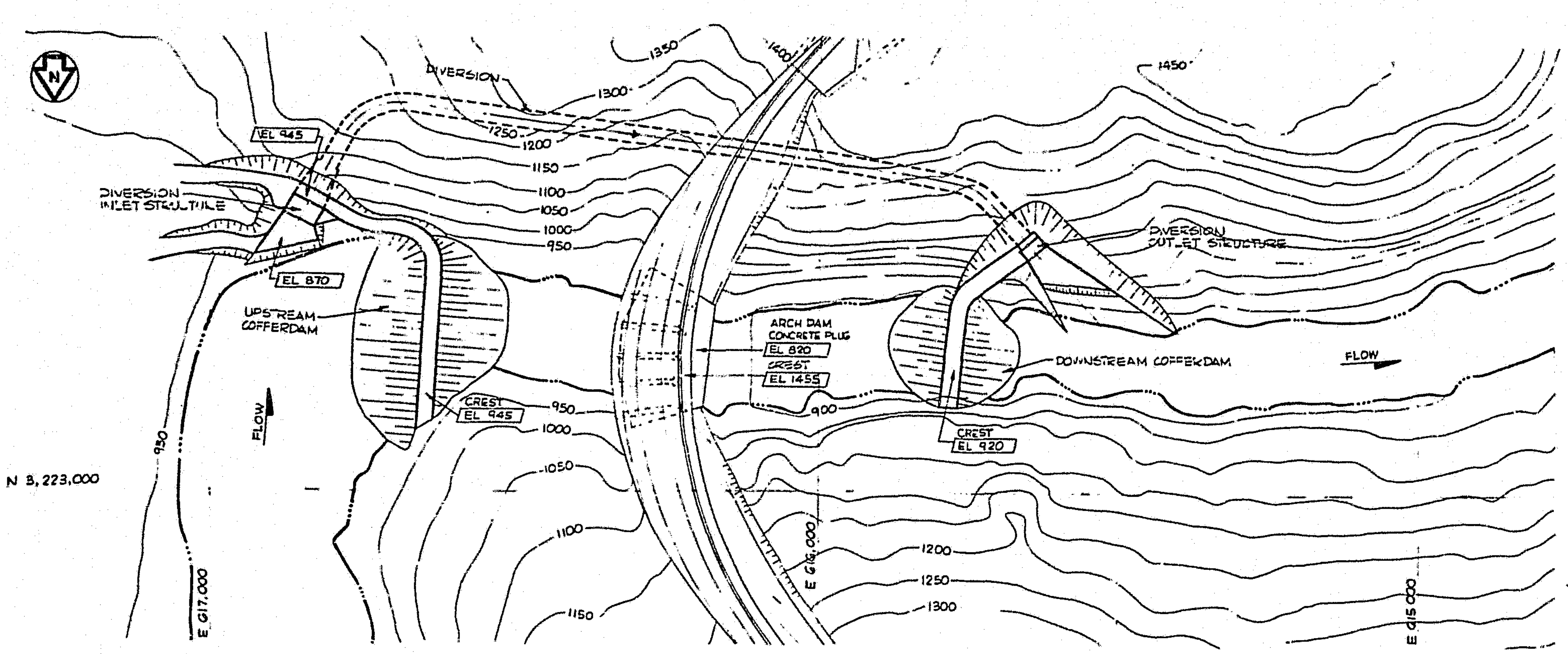


FLOOD FREQUENCY CURVE

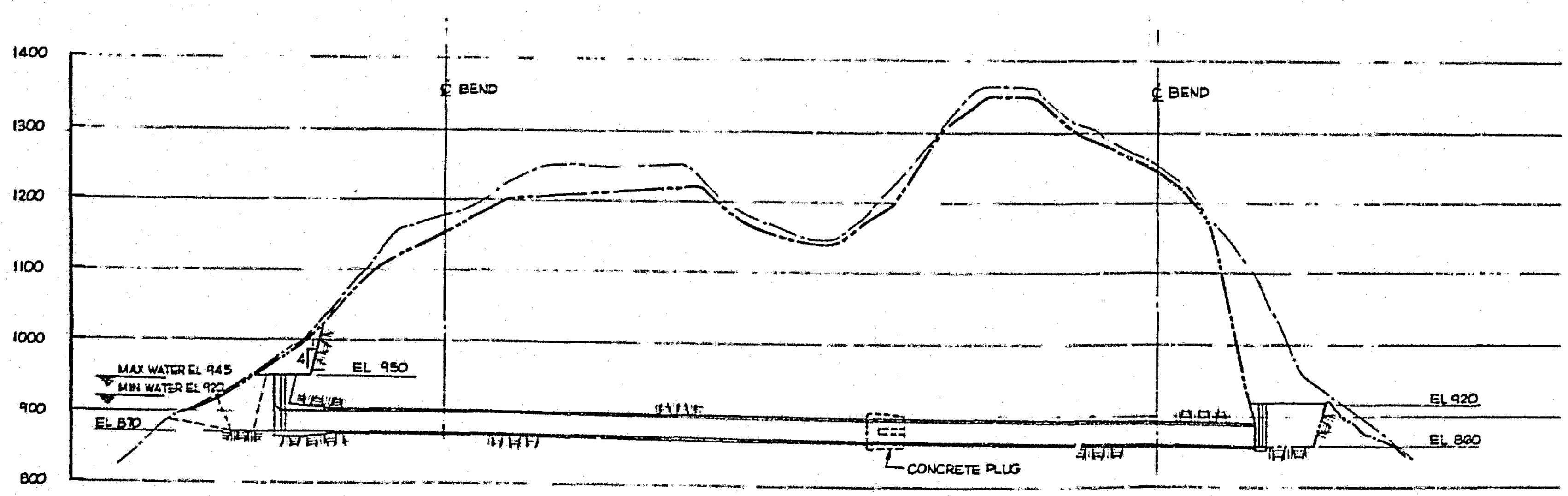
PRELIMINARY

DEVIL CANYON
HYDRAULICS
SHEET 1 OF 3

SK-5700-C6-602



PLAN



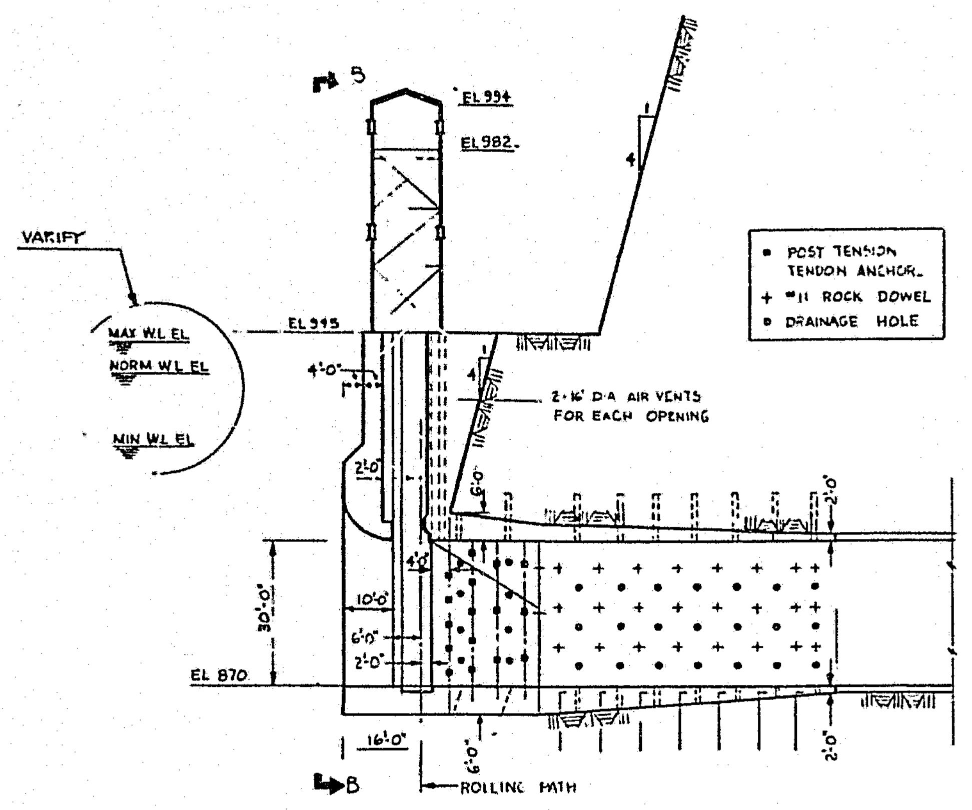
PROFILE

SCALE 0 100 200 FEET
1" = 100'

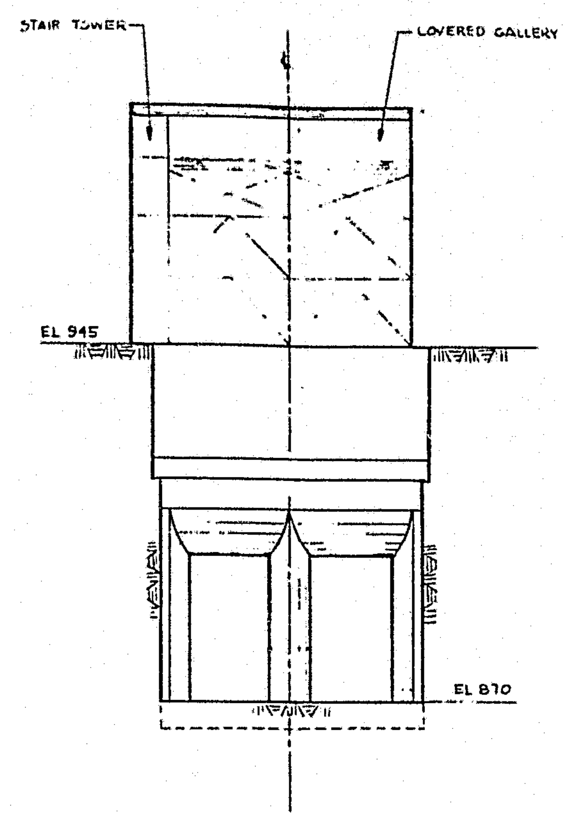
DEVIL CANYON
DIVERSION
GENERAL ARRANGEMENT
PLAN AND PROFILE

PRELIMINARY

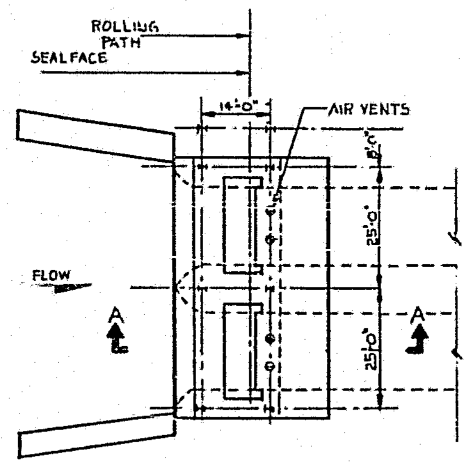
SK-5700-C6-611



SECTION A-A

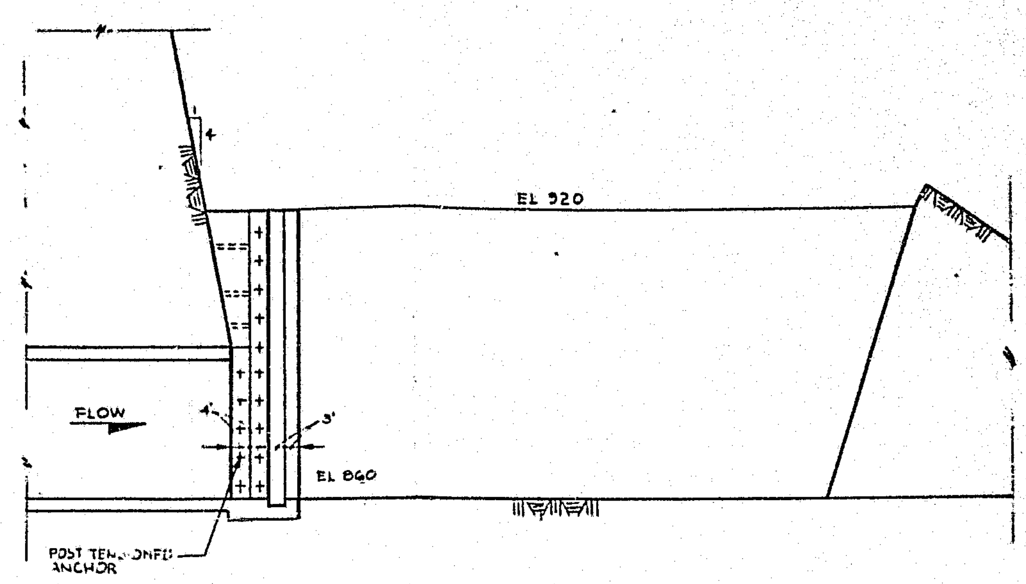


ELEVATION B-B

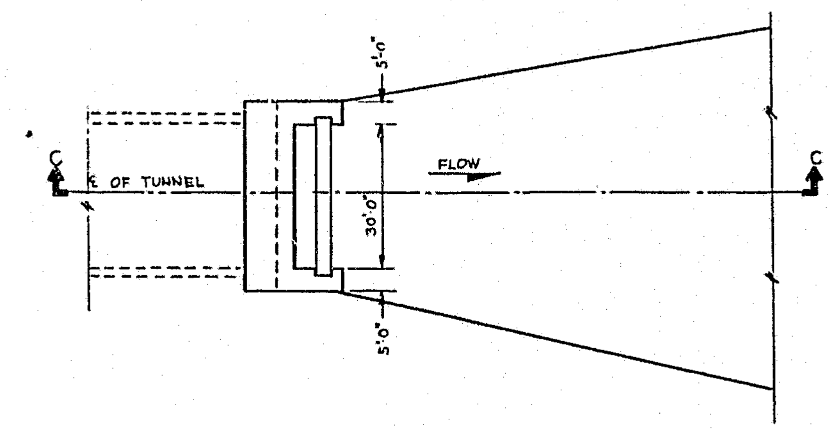


PLAN @ ELEVATION 945

DIVERSION - INLET STRUCTURE



SECTION C-C



PLAN @ ELEVATION 920.0

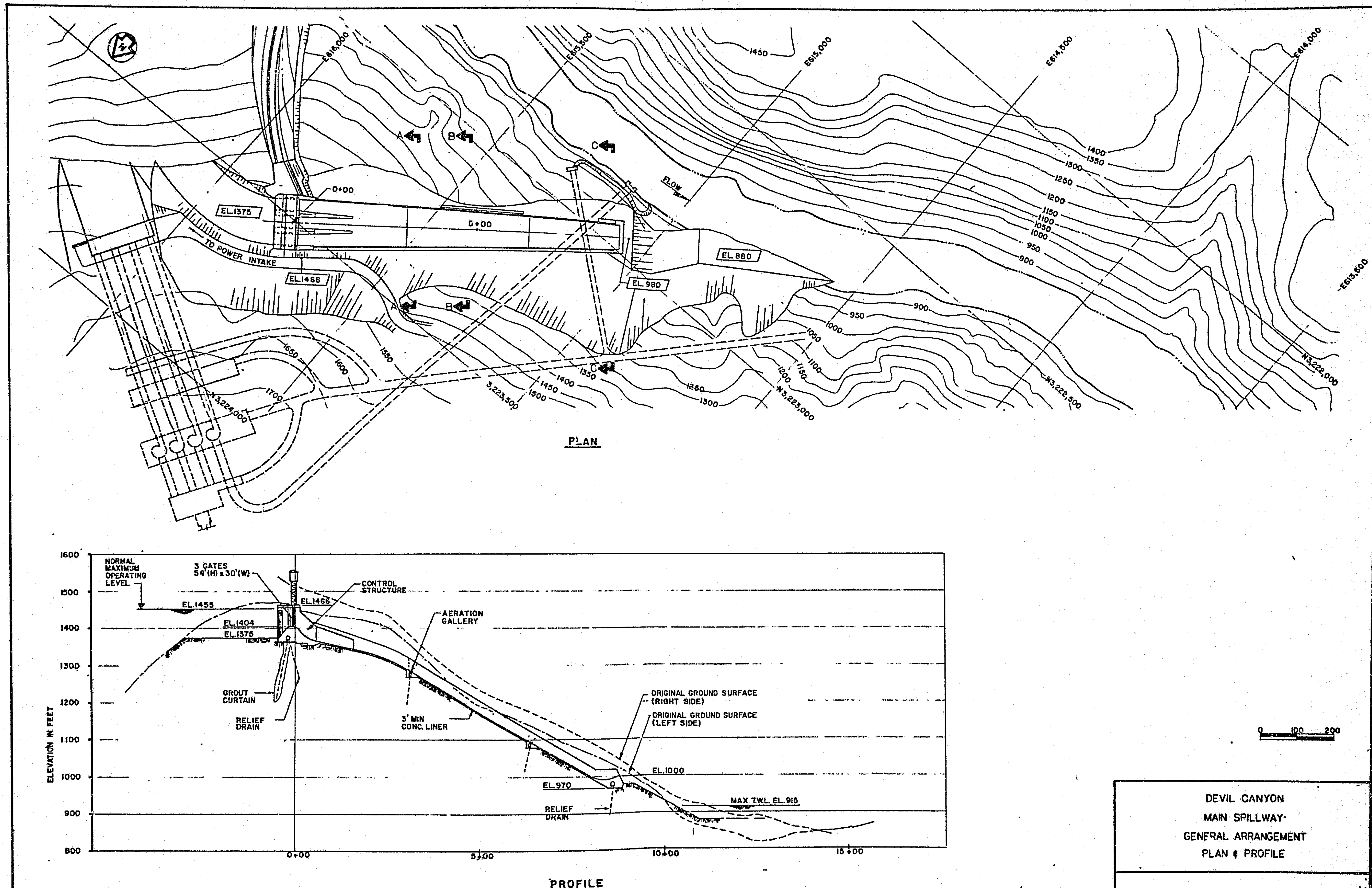
DIVERSION OUTLET STRUCTURE

SCALE 0 16 32

DEVIL CANYON
DIVERSION
INLET AND OUTLET STRUCTURES
PLANS AND SECTIONS

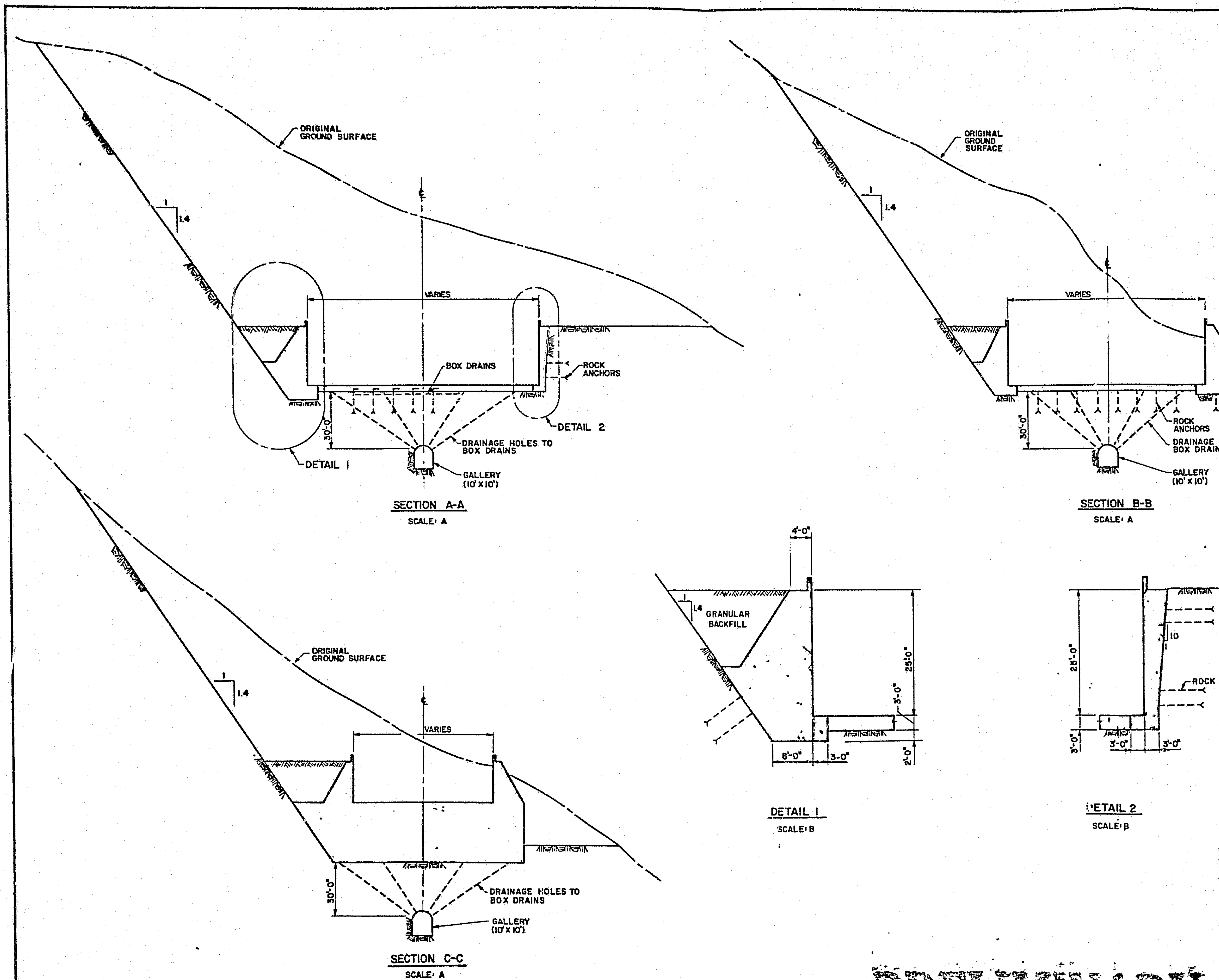
PRELIMINARY

SR-5700-C6-612



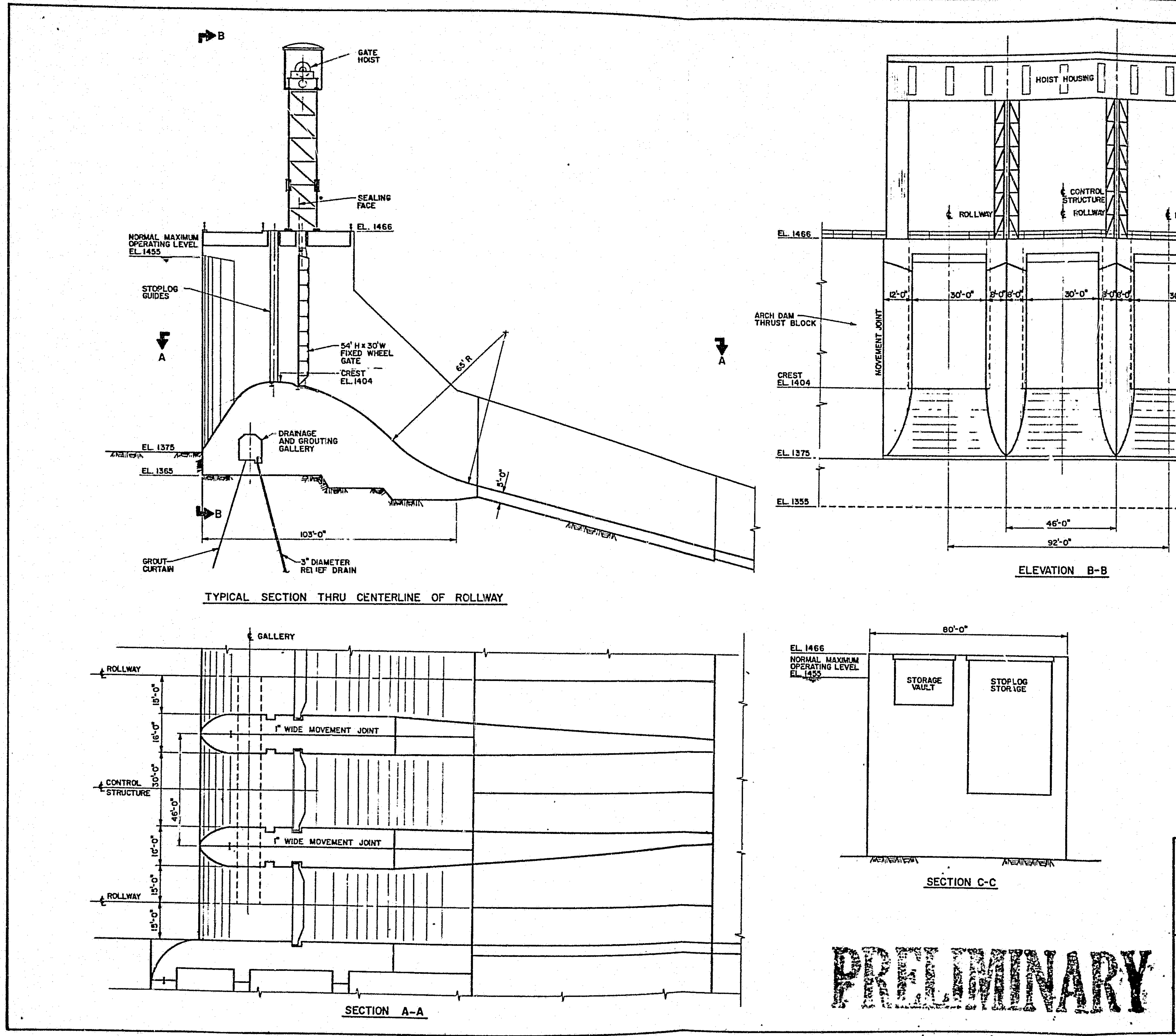
DEVIL CANYON
MAIN SPILLWAY
GENERAL ARRANGEMENT
PLAN & PROFILE

PRELIMINARY



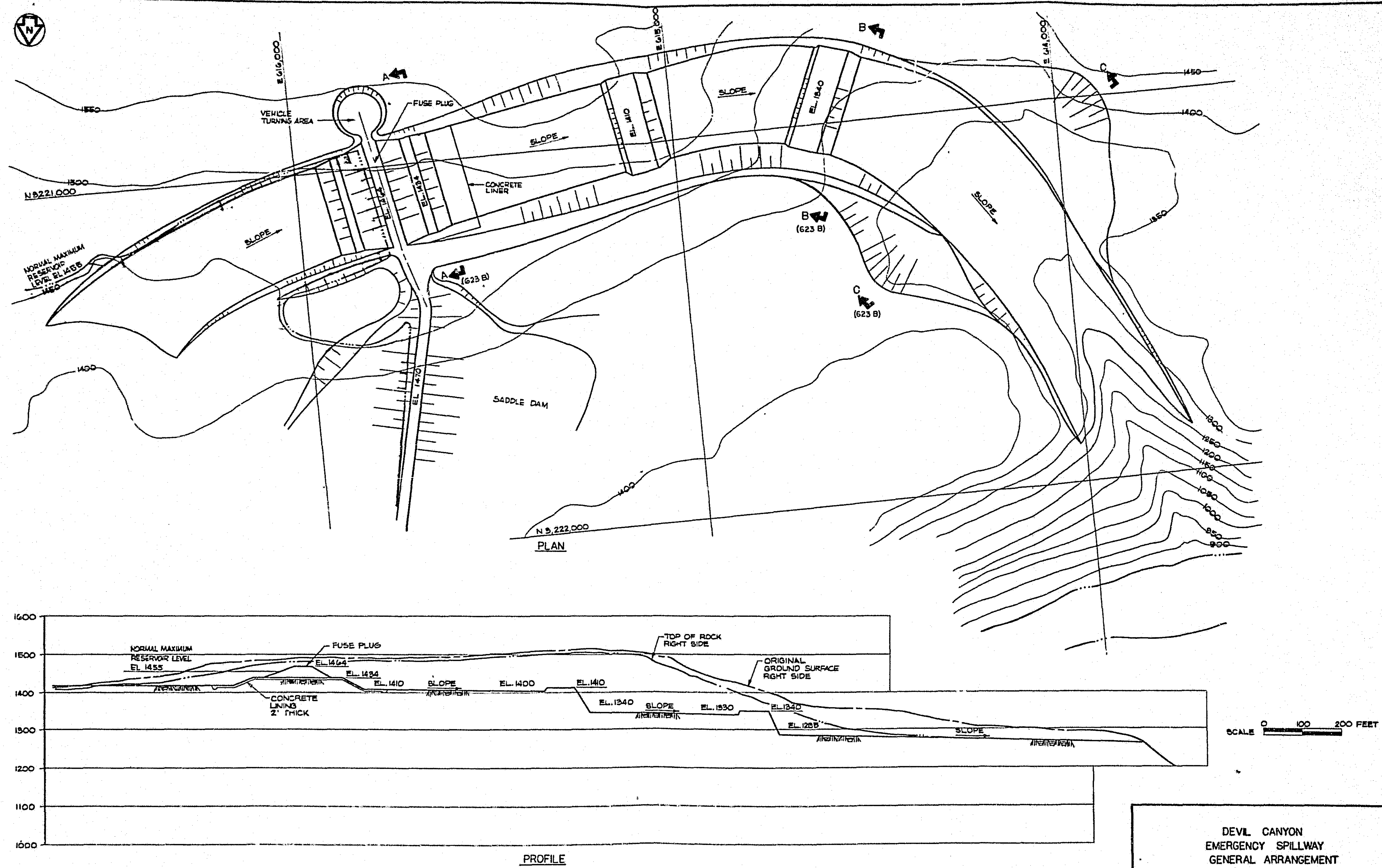
DEVIL CANYON
MAIN SPILLWAY
CHUTE SECTIONS & DETAILS

SK-5700-C6-622A



DEVIL CANYON
MAIN SPILLWAY
CONTROL STRUCTURE
PLAN AND SECTIONS

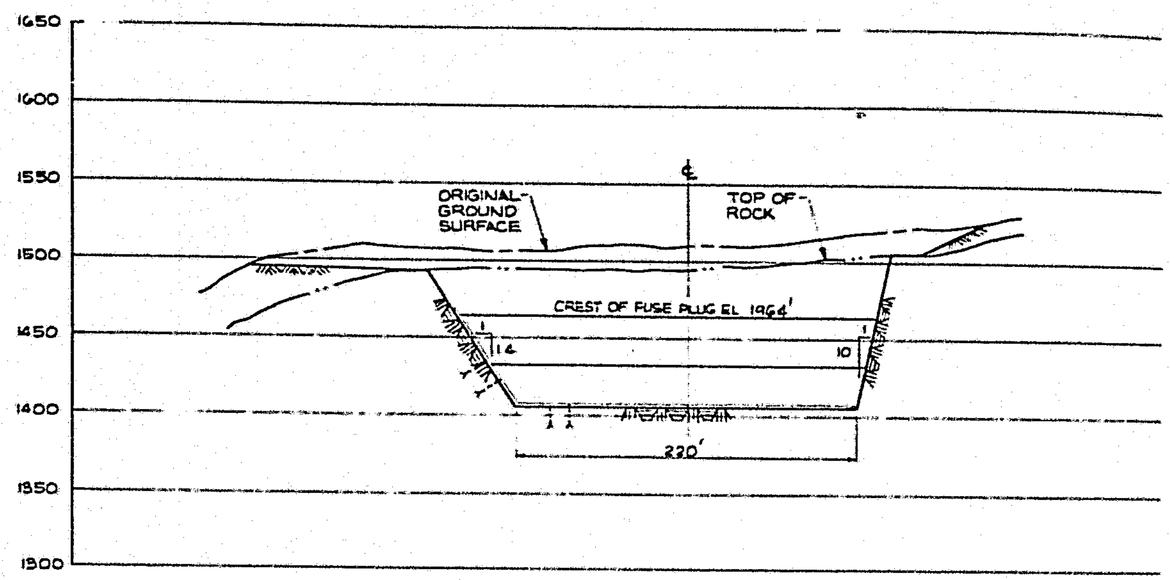
SK-5700-CE-622 B



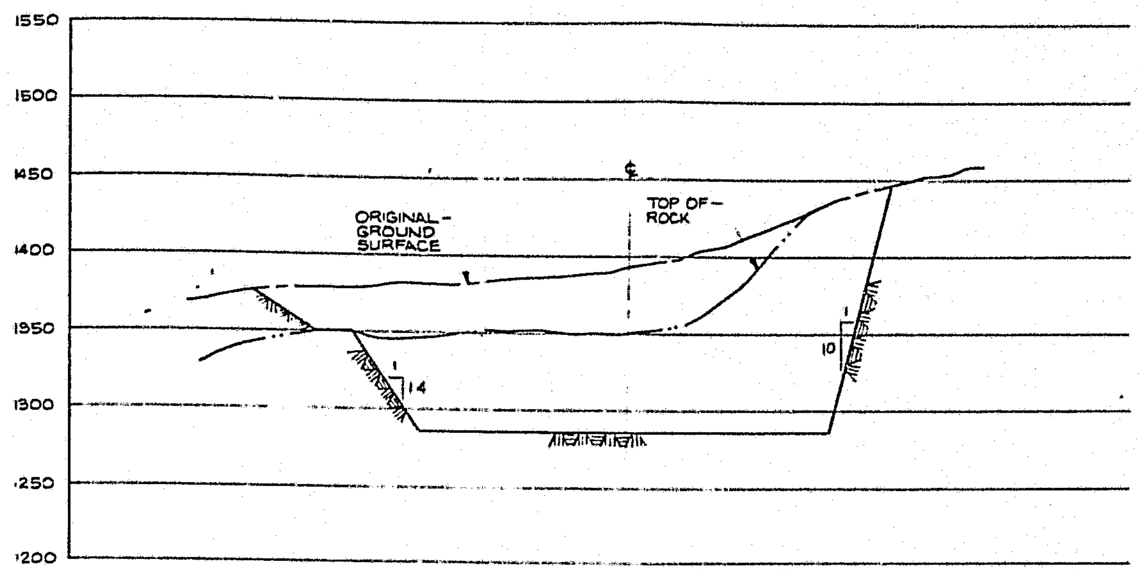
DEVIL CANYON
EMERGENCY SPILLWAY
GENERAL ARRANGEMENT

PRELIMINARY

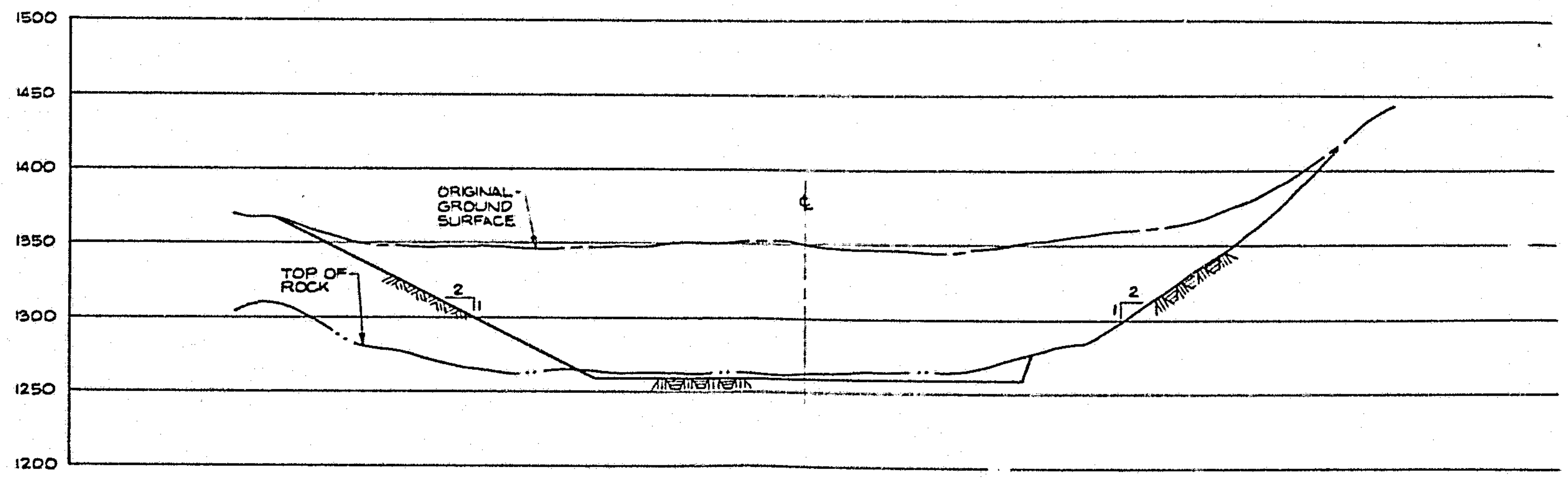
SK-5700-C6-623A



SECTION A-A (623 A)



SECTION B-B' (623 A)



SECTION C-C (623A)

SCALE 0 50 100 FEET

DEVIL CANYON
EMERGENCY SPILLWAY
SECTIONS

PRELIMINARY

SK-5700-C6-623B