3440

BEFORE THE
FEDERAL ENERGY REGULATORY COMMISSION

APPLICATION FOR LICENSE FOR MAJOR PROJECT

SUSITNA HYDROELECTRIC PROJECT

VOLUME 16

DRAFT

EXHIBIT F

HARZA-EBASCO
SUSITNA JOINT VENTURE

Alaska Power Authority

BEFORE THE
FEDERAL ENERGY REGULATORY COMMISSION

APPLICATION FOR LICENSE FOR MAJOR PROJECT

SUSITNA HYDROELECTRIC PROJECT
DRAFT LICENSE APPLICATION

VOLUME 16

EXHIBIT F
SUPPORTING DESIGN REPORT

November 1985

NOTICE

A NOTATIONAL SYSTEM HAS BEEN USED
TO DENOTE DIFFERENCES BETWEEN THIS AMENDED LICENSE APPLICATION
AND
THE LICENSE APPLICATION AS ACCEPTED FOR FILING BY FERC
ON JULY 29, 1983

This system consists of placing one of the following notations beside each text heading:

(o) No change was made in this section, it remains the same as was presented in the July 29, 1983 License Application

(*) Only minor changes, largely of an editorial nature, have been made

(**) Major changes have been made in this section

(***) This is an entirely new section which did not appear in the July 29, 1983 License Application

VOLUME COMPARISON

VOLUME NUMBER COMPARISON

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EXHIBIT F
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1 - PROJECT DATA (***)

This document provides the principal project data and design criteria for the Watana (Stages I and III) and Devil Canyon (Stage II) hydroelectric projects, and will form the basis of the detailed engineering design. It has been prepared to satisfy the requirements of Section 4.41 (g)(3) of the FERC Regulations which specify the submission of supporting information. The purpose of this information is to demonstrate that proposed structures are safe and will adequately fulfill their stated functions.

This exhibit has been prepared as a design criteria document containing a summary of project parameters, design data, and applicable codes and standards. The report and bibliography volumes include information in addition to that required in the FERC regulations. Pertinent project data is presented in Table F.1.1.1.

2 - PROJECT DESIGN DATA (**)

2.1 - Topographical Data (o)

The topography of the site is based on aerial survey mapping reduced to a scale of 1 inch: 200 feet. Contours are at 5-foot intervals.

2.2 - Hydrological Data (**)

Hydrological data are based on the records of the U.S. Geological Survey for gaging stations on the Susitna River at Gold Creek (No. 15292000) and near Cantwell (No. 152915000) (USGS 1950-1983). The periods of record for the gages are 1949 to the present for the Gold Creek gage and 1961-1972, 1980 to present for the gage near Cantwell. Additional data have been collected by the Applicant since 1981 on the Susitna River near the Watana damsite (R&M 1985). The manner of estimating project inflows is described in a report by the Applicant (APA 1985). The estimation of floods and flood frequency analyses for natural and with-project conditions are also described in two reports by the Applicant (HE 1984a, 1985). A more thorough description of flows and floods is contained in Exhibit E, Chapter 2, Sections 2.2 - Water Quantity, for natural conditions and Sections 4.1.3(a), 4.2.3(a) and 4.3.3(a) for with-project conditions. Natural flows at Gold Creek, Watana and Devil Canyon are given in Tables F.2.2.1, F.2.2.2 and F.2.2.3 respectively.

2.3 - Meteorological Data (*)

Historical records of precipitation, temperature, and other climatic parameters are collected by the National Weather Service (NWS) and the Federal Aviation Administration (FAA) at several stations in the study area. However, there were no stations located within the basin until the establishment of weather stations as part of this study. Consequently, no long-term weather records are available near the damsites. The closest stations with long-term records are at Talkeetna and Summit. Data from these stations are given in Tables F.2.3.1 to F.2.3.3.

2.4 - Reservoir Data (o)

Reservoir elevation, area and volume curves for Watana and Devil Canyon are given in Figures F.2.4.1 and F.2.4.2.

2.5 - Tailwater Elevations (o)

Tailwater elevations plotted against flows at Watana and Devil Canyon are given in Figures F.2.5.1 and F.2.5.2, respectively.

2.6 - Design Floods (**)

An analysis of major historical floods indicated that snowmelt contributes a major part of the floods. The Probable Maximum Flood (PMF) was therefore assumed to occur during the snowmelt season. Snowmelt was assumed to start on June 3 based on the adopted temperature sequence. The Probable Maximum Precipitation (PMP) of 8.7 inches above the Watana Dam site was used in the PMF analysis. The average PMP above Devil Canyon was 8.8 inches.

The PMF was derived through use of the Streamflow Synthesis and Reservoir Regulation (SSARR) watershed model. The PMF hydrograph was synthesized assuming an initial base flow of approximately 7,000 cfs and moist antecedent soil conditions. The analysis gave peak inflows of 326,000 cfs at Watana and 362,000 cfs at Devil Canyon. The PMF hydrograph is shown on Figure F.2.6.1.

The PMF was routed through the Watana - Stages I and III, and Devil Canyon Stage II Reservoirs and the peak outflows were 302,000 cfs, 284,000 cfs, and 302,000 cfs respectively. The flood routings reduced the peak inflows to Devil Canyon Reservoir to 358,000 cfs and 339,000 cfs in Stages II and III, respectively. The routed peak outflows from Devil Canyon were 351,000 cfs and 333,000 cfs for Stages II and III, respectively.

The 10,000-year flood peak inflows were estimated from the Gold Creek station record of 34 years to be 174,000 cfs and 184,000 cfs at Watana and Devil Canyon under natural conditions. In all three stages the spillways can pass the estimated 10,000-year flood with the water levels in the reservoirs at the 50-year flood storage pool level. Additionally, because the Gold Creek record is only 34 years, the 95 percent one-sided upper confidence limits were also estimated for the 10,000-year flood. These values are 248,000 cfs and 262,000 cfs at Watana and Devil Canyon, respectively. The combined spillway and outlet works capacities at both sites exceed these values with the water level at the 50-year flood storage pool level. Thus, it is estimated there is less than a 1 in 10,000 chance that water levels in Watana and Devil Canyon Reservoirs would exceed the 50-year flood storage pool levels in any one year.

The development of the PMF flood is presented in Appendix F3.

3 - CIVIL DESIGN DATA (*)

3.1 - Governing Codes and Standards (o)

Where specific standards and design criteria are not covered in these criteria, the following codes and standards shall apply:

3.1.1 - General (o)

- American National Standards Institute, ANSI A58.1;

- Uniform Building Code (UBC);

- Alaska State Building Construction Code; and

- Occupational Safety and Health Administration Standards (OSHA).

3.1.2 - Concrete (o)

- American Concrete Institute - ACI Standard 318 (for reinforced concrete)

- American Concrete Institute - ACI Standard 301

- American Concrete Institute - ACI Standard 207 (for mass concrete)

3.1.3 - Structural Steel (o)

- American Institute of Steel Construction, Steel Construction Manual.

3.2 - Design Loads (**)

3.2.1 - Dead Loads: (*)

Mass concrete	150	lbs/ft^3 (143 lbs/ft^3 when checking stability)
Reinforced concrete	152	lbs/ft^3
Steel	490	lbs/ft^3
Water	62.5	lbs/ft^3
Silt - vertical	120	lbs/ft^3
- horizontal	85	lbs/ft^3
Backfill (all Dams)		
- dry	115	lbs/ft^3)
- saturated	130	lbs/ft^3) - Provisional
- submerged	70	lbs/ft^3)

3.2.2 - Backfill Loads (o)

The lateral earth pressure against vertical faces of structures with horizontal backfill will be computed using the equivalent fluid pressures calculated from:

$$P = kwH$$

Where:

p = unit pressure
k = pressure coefficient
w = unit weight of fill
H = height of fill

For structures free to deflect or rotate about the base the pressure coefficient will be computed from Rankine's theory, which is:

$$k_A = \tan^2 (45-\emptyset/2)$$

Where $\emptyset$ = angle of internal friction (degrees).

For structures restrained from bending or rotation, the at-rest pressure coefficient will be used:

$$k_o = 1 - \sin \emptyset$$

Coulomb's theory will be used for computing lateral earth pressures on wall surfaces with slopes flatter than 10V:1H or with sloping backfill steeper than 1V:4H.

Where vehicular traffic can run adjacent to the face, a surcharge loading of 500 lbs/ft^2 should be applied.

3.2.3 - Snow and Ice Loads (o)

Special consideration shall be given to prevent accumulation of ice loading due to spray in the final design.

Snow Load .... 60 lbs/ft^2

3.2.4 - Powerhouse Floor Loads (o)

Generator Hall	-1000 lbs/ft^2
Machine Shop	- 500 lbs/ft^2
Switchgear Room	- 300 lbs/ft^2
Service Bay	- 1500 lbs/ft^2 or 90 kip concentrated load in designated areas
Control Room	- 200 lbs/ft^2
Transformer Gallery	- 300 lbs/ft^2
Offices and Stairs	- 100 lbs/ft^2

3.2.5 - Crane Loads (o)

The following percentages shall apply to the powerhouse crane and the power intake crane. The minimum deflection to span ratio of crane support beams shall be 1:1000.

Vertical impact	- 25 percent of static wheel load
	- 10 percent of crane capacity, trolley, hook, and lifting beam distributed equally between rails.
Longitudinal load	- 10 percent of static wheel loads.

3.2.6 - Spillway Deck Loads (o)

Area designated for service .... 500 lbs/ft^2
Other areas .... 200 lbs/ft^2

3.2.7 - Hydraulic Loads (o)

All structures shall be designed for full lateral water pressures where applicable, plus full hydrodynamic and uplift forces.

(a) Uplift (o)

Uplift pressures shall be taken as equivalent to the full head of water on a foundation or structure where no head differential exists across the structure. Safety factors in accordance with normal conditions will apply. Where a head differential exists across a structure, uplift forces shall be calculated as follows.

For water-retaining concrete structures provided with drainage galleries and drain holes deep into the foundations, uplift shall be considered across the complete rock/concrete interface varying linearly from H_1 at the upstream heel to $\frac{(H_1 - H_2)}{3} + H_2$ at the drains to H_2 at the toe.

Where H_1 = Static head upstream

H_2 = Static head downstream.

Safety factors in accordance with normal conditions will apply with drains operative.

Where there are no pressure relief drains, normal uplift shall be assumed to vary linearly from headwater at the upstream face to tailwater at the downstream face. Safety factors in accordance with normal conditions will apply. The latter uplift condition shall also apply for the extreme uplift where drains are to be provided but are assumed to be ineffective in reducing uplift. Safety factors in accordance with extreme conditions will then apply.

Allowable tensile strength at the rock-concrete interface shall be zero. If under earthquake loading conditions a crack is considered to develop at the upstream heel, the uplift pressure shall be taken as equal to the normal distribution as described above over 100 percent of the base area.

Under PMF conditions where cracking at the upstream heel develops, uplift shall be considered to be equal to full headwater within the full depth of the crack, reducing to the values at the line of drains and downstream toe as proportioned above.

Apron and chute slabs and slab walls against rock shall be designed against uplift resulting from sudden changes in water level.

Uplift from centrifugal forces shall be considered where contraction joints occur on the concave floor of chutes.

Toe curve pressures on the interior face of training walls at concave chute surfaces shall be calculated in accordance with Plate 21 of Hydraulic Design of Spillways EM 1110-2-1603 by U.S. Army Corps of Engineers (COE 1981).

Hydraulic loads due to earthquakes are given in the following section on seismic loads.

3.2.8 - Seismic Loads (o)

The largest mean peak horizontal ground acceleration that could affect the sites is 0.5g with a duration of 6 seconds (Acres 1982c).

(a) Watana (o)

Design of critical concrete structures will use an 80th percentile response spectrum from the "Safe Evaluation Earthquake" (SEE) with a 10 percent damping ratio scaled down by a factor of 80 percent.

(b) Arch Dam at Devil Canyon (o)

The arch dam is to be checked under seismic loading by dynamic analysis based on trial load method and the ADSAS program developed by the Department of the Interior.

The arch dam will be designed for a base ground acceleration of 0.8 x SEE = 0.57g.

Arch dam system damping ration - 0.10 of critical*.

Acceleration response spectrum - See Figure F.3.2.1.

For final design, a time-history finite element analysis will be carried out.

- Concrete Retaining Structures (other than arch dam)

  Mass concrete retaining structures will be designed for 0.8 x SEE using static analysis.

- Other Major Structures

  Non-reservoir retaining major structures will be designed for the 100/110-year return earthquake corresponding to 0.2g.

- Hydrodynamic Pressure

  The hydrodynamic pressure due to horizontal earthquake on water-retaining surfaces shall be computed using the theory of Westergaard for the dynamic change in pressure:

$$P = a.51.25\,(hy)^{1/2} \quad lbs/ft^2$$

Where h = total height of structure (ft)
y = depth below reservoir surface (ft)
a = ground acceleration/acceleration due to gravity

The distribution of pressure is parabolic; hence, the total force and moment at a section y feet below water level are given by:

$$F = 2/3.\ P.y$$
$$M = 0.4.\ F.y$$

*This damping ratio is similar to ones used at Swan Lake, El Cajon and Salinas Dams.

3.2.9 - Temperature and Thermal Loads (o)

Expansion and contraction resulting from temperature changes, moisture changes, creep in component materials, and movement resulting from differential settlement are combined with other forces and loadings for maximum unfavorable effects.

The maximum and minimum air temperatures are:

Maximum	90°F
Minimum	-60°F

3.2.10 - Horizontal Ice Loads (**)

Horizontal ice loads will be computed in accordance with the U.S. Army Corps of Engineers (COE) design manual, "Ice Engineering" (1982b).

Excessive ice buildup on trashracks, gates, gate guides, and support structures shall be prevented by heating such equipment.

3.3 - Stability (*)

3.3.1 - Loads and Forces (*)

The following loads and forces shall be used in stability analysis for concrete gravity structures in the loading cases given in Section 3.3.4:

- o Dead load or self weight;
- o Live load;
- o Hydrostatic uplift;
- o Earth pressure;
- o Water pressure; and
- o Earthquake loads.

3.3.2 - Computations (*)

The following values shall be computed at the foundation level and at selected intermediate levels within each structure or element of a structure to ensure adequate stability and economy of design within these design criteria:

- o Stress at upstream face (parallel to slope);
- o Stress at downstream face (parallel to slope);
- o Location of resultant force;
- o Sliding factor;
- o Shear friction factor;
- o Flotation factor of safety; and
- o Overturning factor.

(a) Sliding Stability Analysis (o)

The normal analysis of sliding has been used, relating the resistance to sliding along a horizontal or gently sloping plane to the driving force or horizontal load. The factor of safety, F, is the ratio of the resisting forces to the driving forces. The following "shear friction" formula shall be used (Acres 1982d, 1982a).

$$F = \frac{(V-U) \tan \phi + cA}{P_w}$$

Where, for a horizontal potential failure plane:

- V = total vertical force
- U = total vertical uplift force acting on the failure plane
- ϕ = angle of friction along plane
- c = unit cohesion along plane
- A = area of potential sliding plane
- P_w = total horizontal thrust

3.3.3 - Limiting Criteria, Safety Factors (**)

(a) Concrete Gravity Structures (*)

Load Conditions	Safety Factor: Sliding	Overturning[1]	Flotation	Compression
Normal	3.0 within concrete 4.0 within rock	Resultant within the center third[2]	1.5	3.0 on compressive strength of concrete 4.0 on compressive strength of rock
Unusual (including 1:100-yr earthquake load case)	2.5 within concrete 3.5 within rock	1.3	1.3	2.5 on compressive strength of concrete 3.5 on compressive strength
Unusual (inc. 100-year return earthquake & PMF load case)	2.0 within concrete 2.7 within rock	1.1	1.1	2.0 on compresive strength of concrete 2.7 on compressive strength of rock
Extreme (including 0.8 x safety evaluation earthquake) for arch dam and reservoir retaining structures only	1.0	1.0	1.0	1.0

1/ Opinions differ on the use of overturning safety factors. The criteria used herein is to retain this familiar concept, particularly in regard to unusual and extreme loadings where cracking may occur, in order to provide a measure of the relative safety of the structure.

2/ Safety factor implicitly greater than at least 1.5

(b) Summary of Results (**)

The results of the above load conditions for the reservoir retaining concrete gravity structures have been summarized on the Exhibit F Plates as follows:

Watana - Stage I spillway control structure, Plate F13
Devil Canyon - Stage II arch dam thrust blocks, Plate F46
Devil Canyon - Stage II main spillway control structure, Plate F55
Watana - Stage III spillway control structure, Plate F80

3.3.4 - Loading Cases (o)

Among loading combinations to be considered at the final design stage will be the following:

(a) Intake and Outlet Structures (o)

Case 1: Concrete in place, site dewatered
Case 2: Concrete in place, maximum water level outside structure, inside of structure dewatered

(b) Powerhouse Structure (surface structures, if applicable) (o)

Underground structures and individual elements of structures shall be analyzed for stability and stress considering all applicable loadings including water table in rock, grouting pressure, and rock support systems.

3.4 - Material Properties (o)

Reinforced Concrete in all structures except the Arch Dam shall have a compressive strength of 4,000 lb/in^2 at 28 days. The Arch Dam concrete shall have a compressive strength of 5,000 lb/in^2 at 365 days.

- Reinforcing Steel: ASTM A615 Grade 40 minimum
- Structural Steel: ASTM A36
- Penstock Steel Liner: ASTM A516 Grade 70
- Bolts, Nuts, and Washers: ASTM A325
- PVC water stops shall be provided in all water-retaining structures as follows:
  - In all expansion and contraction joints
  - In all vertical construction joints communicating with dry interior spaces; and
  - In all horizontal construction joints communicating with dry interior spaces where the concrete thickness is less than 10 feet.

4 - GEOTECHNICAL DESIGN DATA (**)

4.1 - Watana (**)

4.1.1 - General (**)

A detailed description of the geology and material properties for the Watana site are provided in the "1980-81 Geotechnical Report" (Acres 1982d) and the "1982 Supplemental Geotechnical Report" (Acres 1982a). Additional information is provided in the 1983 Geotechnical Program Report (HE 1983) and the 1984 Geotechnical Program Report (HE 1984b). Design parameters, quantities, and estimates have been based on a comprehensive evaluation of the site geotechnical conditions. Where significant data remains to be obtained, conservative assumptions have been made in the development of foundation preparation, and treatment, material properties, and costs. The following set forth the design consideration, parameters, and criteria for Watana Dam - Stages I and III.

4.1.2 - Dam Foundation Preparation and Treatment (**)

(a) General (*)

Bedrock foundations must meet the following criteria:

- The bedrock under the impervious core must be nonerodible under the seepage gradients;
- Impervious core material must be prevented from moving down into the foundation (e.g., into cracks or joints);
- The contact between the impervious fill and bedrock surface must have a permeability no higher than that of the impervious core; and
- Any seepage through the foundation must be controlled and discharged to avoid buildup of excessive seepage pressures under the structures.

(b) Excavation Under the Impervious Core, Filters, and Shells (**)

The impervious fill, filters, and shells of the dam (except at the toes of the slopes) will be founded on sound bedrock. All talus on valley slopes, and river alluvium and weathered rock in the valley bottom and on the abutments will be removed within the limits indicated on Plate F7. Dental excavation over and above normal excavation will be performed in intensely sheared and altered zones. Under the core and filters, dental concrete will be placed as appropriate to provide a regular surface for fill placement.

(c) Grouting (**)

Grouting will be performed as necessary to improve foundation and abutment rock conditions for load bearing, material piping, and seepage considerations.

(i) Consolidation Grouting (*)

The bedrock under the core and adjacent upstream and downstream filters will be consolidation grouted to provide a zone of relatively impermeable rock under the entire contact. Consolidation grouting will be performed on a 10 foot by 10 foot grid of approximately 30 foot deep holes. Consolidation grouting will be performed as required under the spillway and other appurtenant structures, as well as at the tunnel portals and in any fractured zones encountered underground which could be stabilized by this method.

(ii) Grout Curtain (**)

A grout curtain will be performed beneath the dam foundation to a maximum depth of 250 feet, as shown on Plate F-8. Grouting will be carried-out from a series of underground galleries which will also serve as a drainage collector for a system of drilled drain holes. A double-row grout curtain is proposed. Primary grout holes will be considered as exploratory holes and will be core drilled. Based on exploratory results, the depths and spacing of secondary holes will be decided.

All holes will be water-pressure tested. Grouting will be carried-out with the primary holes at 20-foot spacing. The secondary and tertiary holes will be located by split spacing, bringing the final hole spacing to 5 feet if required.

In areas of permafrost, additional boreholes may be required to induce thawing, or to be able to form an effective curtain. Further grouting may be required after the thawing effect of the reservoir has occurred.

Grout holes will be both vertical and inclined to intersect the main joint sets. Additional grouting will also be performed as required in sheared and altered zones and poor quality bedrock if it has been determined that they are potential avenues for seepage.

The dam grout curtain will also extend under the spillway intake structure to a minimum depth of 200 feet. The grout curtain will be stopped approximately 30 feet from the diversion tunnels. Radial grouting will be carried out from the diversion tunnels along the length of the concrete closure plugs to intersect with the grout holes from the surface and form a continuous cutoff of seepage from the reservoir or the diversion tunnel sections upstream of the grout curtain.

(iii) Contact Grouting (*)

Contact grouting will be performed behind all tunnel linings and tunnel plugs.

(d) Drainage (*)

Three-inch diameter drain holes will be drilled from the galleries beneath the dam foundation and abutments to intersect seepage water and to provide pressure relief.

A grid of drainage holes will be provided around the underground caverns to depths generally in excess of the deepest rock bolt. Seepage will be collected by pipes or channels and directed into the powerhouse drainage system.

All rock cuts will have surface drainage trenches at the crest to prevent small rocks and soil from being washed down the cut, and to reduce the amount of water to be channeled away at the base of the cut. Pressure relief holes will be drilled into the face and base of cuts as necessary to relieve areas of high ground water pressure.

(e) Intake Structure (o)

The intake structure will be founded on sound, unweathered rock. Although consolidation grouting is not expected to be necessary due to the excavation depth, it will be performed if required.

Under rapid drawdown conditions, water pressure could build up behind structures cast against rock. Therefore, drainage will be provided through all concrete/rock interfaces that could experience these conditions.

Rock excavation faces are anticipated to be stable at very steep slopes. Further data will be required in the area for detailed stability analysis and design of protective support systems.

(f) Spillway (*)

The spillway will be founded entirely on rock. The grouting and drainage curtains in the dam foundation and under the thrust block will extend under the spillway control structure to reduce seepage and uplift pressures.

A drainage grouting gallery will be formed in the concrete rollway of the control structure. This gallery will be similar in size to the rock tunnel beneath the dam and constructed as far upstream as possible to achieve a reduction in water pressure over the largest possible area of the foundation.

The spillway chute concrete/rock contact will be well drained to prevent uplift pressures. A longitudinal spillway drainage gallery would be constructed below the concrete spillway slab along the entire length of the spillway. A fan of drain holes drilled from the surface drains will drain into the drainage gallery. The gallery will be approximately 10 feet by 10 feet in section.

The foundation for the entire spillway will be consolidation grouted to a depth of 20 feet based on a grid of holes spaced 10 feet by 20 feet.

Rock anchors will be installed in the spillway chute walls to provide necessary support and fallout protection and in the slab foundation to prevent uplift.

(g) Relict Channel (**)

Studies indicate the existence of a buried channel running from the Susitna River gorge immediately upstream from the proposed damsite to Tsusena Creek, a distance of about 1.5 miles. Along the buried channel thalweg, the highest bedrock surface is approximately elevation 1,735, which is about 365 feet below reservoir level for Stage I and 450 feet below the reservoir level for Stage III. The maximum hydraulic gradient along the buried channel resulting from the normal maximum operating pool elevations 2,100 (Stage I) and 2,185 (Stage III) to Tsusena Creek will be approximately 2 percent and 6 percent for the respective pools.

Potential problems posed by the Relict Channel are:

- Subsurface Leakage - caused by permeable material that could result in the water loss and potential downstream piping.

- Permafrost - Increased thawing of permafrost in the relict channel over time resulting in increased seepage.

Remedial measures being presently considered for the relict channel are:

- Placement of a downstream toe drain to control the potential problem of piping (Stage I).

- Long-term monitoring to determine the hydraulic gradient and rate of thaw of permafrost (Stage I and III).

- Slurry trench seepage cutoff across the buried channel thalweg (Stage III).

Additional investigations are necessary to more closely define the actual need and/or type of treatment necessary.

4.1.3 - Rock Slopes (*)

(a) Design Methods (o)

Since jointing is the prominent geologic structure, planar, two-plane, and three-plane wedge failures were analyzed, providing the basis for excavation and support details.

(b) Factor of Safety (o)

Factors of safety employed in slope design for structures were:

Condition	F.S.
Construction-temporary	1.1
Permanent	1.5
Extreme loading	1.0

(c) Method of Analysis (o)

Plane failures and two-plane wedge failures were analyzed on an equal angle stereogram. No external loads were included in these analyses. Analyses included the four principal joint sets identified at the site.

Jointing is believed to be the controlling geologic structure. Planes and wedges created by these joints were analyzed. Design slopes were selected considering orientations and inferred continuity of each joint set. The following table summarizes recommended slopes for each generic orientation.

Strike	Dip	Cut Slopes
N-S	E	3.75V:1H
N-S	W	4.0:1H
E-W	N	3.75V:1H
NE-SW	NW	4V:1H
NE-SW	SE	4V:1H
NW-SE	NE	2.75V:1H
NW-SE	SW	3V:1H

4.1.4 - Water Tunnels (o)

Two orientations are favorable for tunnels at Watana, 345° to 025° and 070° to 090°. These two orientations cross the major discontinuities at high angles and subparallel minor ones. The least favorable orientations are 045° to 065° and 100° to 160°, since they parallel major joint sets and shear zones. Due to the site configuration, the tunnels predominately follow the 070°-090° favorable trend.

4.1.5 - Power Tunnels (*)

The power tunnels will be concrete lined over their entire lengths, with steel lined penstocks placed just upstream of the powerhouse. Penstocks will be spaced 3 times the diameter, center to center.

The length of steel liner and support required will be dependent on actual rock conditions.

4.1.6 - Caverns (*)

As discussed above, the most favorable orientation for underground structures are either 345° to 025° or 070° to 090°. The selected orientation is 023°.

Primary support in the powerhouse cavern will consist of rock bolts in the crown having a working load of approximately 80kips. The preferred rock bolt is a tensioned resin-anchored, resin-encapsulated rock bolt. Wall bolts will be similar to those described above.

Rock bolts for other caverns with spaces between 40 feet and 80 feet will use the same capacity, spacings, and percentages of bolts as the powerhouses with bolt lengths equal to 1/3 of the span for the crown and 1/10 of the wall height for walls. Shotcrete, concrete, and wire fabric will be used as required. Where shear zones intersect underground openings, more extensive support will be required.

Drainage will be provided for walls and crowns to prevent seepage pressures from affecting stability. Drain holes will be provided extending into the rock a distance greater than the greatest rock bolt length.

Caverns will be spaced a minimum of approximately 1.5 times the largest cavern span.

4.1.7 - Watana Dam Embankment (**)

(a) General (**)

The dam embankment Stages I and III will consist of an impervious core protected by fine and coarse filters both upstream and downstream. The outer shells will consist of rockfill. This feasibility level internal zoning design will be further refined and updated during the detailed design period based on detailed stability analyses and known shear strengths. The dam will be designed to provide a stable embankment under all loading conditions.

(b) Design Criteria (**)

To insure that the dam embankment consists of an earthquake resistant design, the following features will be incorporated into the dam embankment cross section:

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

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

- 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.

- The proposed width of the core will prevent arching by transfer of load from the core to the filter materials and shell.

- Compacted rockfill will be used to construct the upstream and downstream outer shells to provide free-draining stable slopes even under seismic conditions.

- Sufficient overburden foundation will be removed beneath the dam embankment to insure stability during expected seismic events.

(c) Freeboard and Embankment Settlement (**)

The governing crest elevation, excluding settlement, is elevation 2,025 for Stage I and elevation 2,205 for Stage III.

Possible settlement will be compensated for in the design height of the dam by including 2 feet of overbuild in Stage I and 5 feet in Stage III.

(d) Dam Embankment Internal Zoning (**)

The embankment typical cross section is shown on Plates F7 and F77. The upstream slope is 2.4H:1V and the downstream slope is 2H:1V. The upstream and downstream shells are composed of rockfill. The impervious core is inclined slightly upstream. The maximum hydraulic gradient through the core will be less than two. Although this is amply conservative, it will be verified based on future laboratory testing.

The impervious core is separated from the upstream and downstream rockfill by a fine filter and a coarse filter, both of variable, but ample, thickness. The upstream and downstream filters are provided as protection against possible leakage through transverse cracks in the core that could occur as a result of settlement or displacement during a seismic event. The wide filter zones provide sufficient material for healing of cracks in the core and the size of the downstream filter zones will ensure its capability to handle any abnormal leakage flows.

Detailed design of the embankment zoning, gradation, placement, and compaction will be finalized after borrow explorations and testing are complete.

To ensure rapid dissipation of excess hydrostatic pressures during a seismic event, the exterior shells of the embankment will consist of clean, compacted diorite rockfill.

Slope protection on the upstream slope within the zone of fluctuating pools will consist of a 40-foot wide rock raked zone at the upstream face consisting of oversized rock.

The crest width will be 35 feet for both Stage I and Stage III.

(e) Dam Embankment (***)

(i) Impervious Material (**)

The impervious core material will be obtained from Borrow Site D. Processing and blending will be necessary to provide the required moisture content and gradation, and to remove any oversize materials. Information to date indicates this can be accomplished by selection of near vertical-face method of excavation and on-fill processing.

Material will be placed in 9-inch uncompacted lifts at a maximum moisture content near optimum moisture content, and compacted to 98 percent of the maximum density obtained from the Standard Proctor Test.

(ii) Fine and Coarse Filters (**)

Fine and coarse filter material will be obtained from Borrow Site E. Borrow Site E is the preferred borrow source for all filter material in the dam. The material will require processing to provide the proper gradations for the fine and coarse filters.

(iii) Rockfill Material (***)

The rockfill material will be obtained from the spillway and approach channel required excavations during Stage I and from Quarry Area "A" during Stage III. The raked rock armour surfacing will be placed on the face of the upstream slope, and in certain areas of the downstream slope as protection against waves and erosion.

(f) Dam Embankment Stability Analysis (***)

The dam embankment as designed is considered to satisfy all present day safety criteria for seismic stability. Moreover, proven effective defense measures against seismic action have been employed, such as large freeboard, large filter thicknesses, along with the use of a free draining rockfill zone at the vulnerable upstream slope. Feasibility level stability analyses have been performed to establish the upstream and downstream slopes of the Watana Dam. The analyses indicate stable slopes under all conditions for a 2.4 horizontal to 1.0 vertical upstream slope, and a 2.0 horizontal to 1.0 vertical downstream slope. Therefore, these slopes have been adopted for preliminary design purposes. Although small portions of the sandy gravel and gravelly sand alluvium remain beneath the upstream and downstream toes, the dam will rest on bedrock over approximately 80 percent of its base.

4.2 - Devil Canyon (**)

4.2.1 - Foundation Preparation and Treatment (*)

(a) Main Dam (*)

The entire area under the dam will be excavated to sound, fresh bedrock. The overburden 100 feet upstream and downstream of the dam will be removed to enable foundation preparation. The overburden will be excavated to a safe slope, generally 2H:1V.

Dental excavation of shear zones and weathered rock will be performed. Such areas will be backfilled with concrete as necessary. Detached blocks of rocks will be removed or rock bolted and/or grouted. Rock overhangs will be trimmed and a regular surface formed.

(b) Grouting (**)

(i) Consolidation Grouting (**)

Consolidation grouting will be performed over the area of the dam foundation and will extend 100 feet upstream and downstream of the dam.

The grout holes will be at 10 feet spacing with depth approximating 30 feet. The orientation of the consolidation holes will be such that they intersect the majority of the discontinuities.

(ii) Grout Curtain (*)

The extent of the grout curtain is indicated in Plate F46. The depth of the holes will be a maximum of 0.7H (where H is the maximum head of water at that particular point of the foundation) up to a maximum of 300 feet. On the right bank, the grout curtain will extend under the thrust block and spillway gate structure and beyond the powerhouse. The curtain will be a minimum of 200 feet deep in this area to ensure minimal seepage into the powerhouse cavern area. The grout curtain will extend 100 feet below the excavated foundation of the intake structure.

Since the underground powerhouse is to be unlined and water pressures in the rock surrounding the powerhouse would considerably increase the rock support required, an extensive grouting program coupled with a comprehensive drainage system is proposed (Plate F46).

The grouting will be performed from tunnel galleries, the general arrangement shown on Plate F46. A maximum slope of 45° has been assumed for the inclined galleries.

The grout galleries will be 10 feet by 10 feet, based on the spacing of the grout and drain holes and the anticipated size of drilling equipment. Although there is no indication of permafrost at the site to date, if permafrost is encountered, thawing will be carried out by circulation of water in the grout holes before grouting.

(c) Drainage (**)

(i) Dam (**)

The underground galleries will be used for installation of the drain holes. The drain holes will be 3 inches in diameter and will follow a similar arrangement to the grout curtain.

The drain holes will be installed downstream from the grout curtain and generally extend 50 feet below the grout holes. The spacing will be selected to ensure that the maximum number of discontinuities are intersected. Extra holes may be required in shear zones and in possible joint planes. Where free draining of the lowest grouting/drainage gallery is not possible, pumps will be provided to keep the lowest galleries free of water. Access tunnels will be approximately 10 feet by 10 feet. Drain holes will be drilled upward from the gallery wherever possible to provide the most effective drainage system. Drain holes drilled from upper and lower galleries will overlap by at least 10 feet. Drain holes will be drilled from the gallery and inclined about 10 degrees downstream from the vertical.

(ii) Caverns (o)

Grouting in and around the powerhouse and transformer gallery may be required to reduce excess seepage. Drainage will be provided to relieve water pressure around the caverns.

Drainage will be provided all around the caverns to a depth generally in excess of the deepest rock bolt, and seepage will be collected by pipes or channels and directed into the powerhouse drainage system.

(iii) Rock Cuts (o)

All cuts will have a surface drainage trench at the top to prevent small rocks and soil from washing down the cut. Selective drilling of subhorizontal holes in the rock cuts may be performed to release build up of water pressure on the faces of the rock cuts.

(d) Intake Structure (*)

The foundation for the intake structure will be on sound, unweathered bedrock. Drainage will be provided through the concrete structure from the concrete/rock interface.

Rock excavation faces, against which the structural concrete is to be placed, should be approximately vertical.

Further stability analysis will be required when more information is available on joint shear strength, orientation, and structure location, but no stability or mass structure shear weaknesses are expected to be found.

(e) Spillway (*)

The spillway will be founded entirely on bedrock. The grout curtains and drainage systems in the dam foundation and under the thrust block will extend under the spillway control structure to reduce seepage under the structure and reduce uplift pressures.

The drainage/grouting gallery will be formed in the concrete control structure. This gallery will be of similar size to the rock tunnel beneath the dam and constructed as far upstream as possible to achieve a reduction in water pressure over the largest area of the foundation. To minimize build up of ground water pressure beneath the spillway chute foundation, the concrete/rock contact will be well drained. Steel anchor bars will also be provided for increased stability. Preliminary calculations indicate that these bars should be at 5-foot centers over the foundation area.

A longitudinal drainage gallery will be constructed below the concrete spillway slab, along the entire length of the spillway. A fan of drain holes drilled from the surface drains will drain into the drainage gallery. The gallery will be approximately 10 feet by 10 feet in size. The foundation for the entire spillway will be consolidation grouted as required. The grouting will be to a depth of 20 feet based on a grid of holes spaced 10 feet by 10 feet. Anchors will also be provided from the spillway chute walls into rock.

(f) <u>Saddle Embankment Dam</u> (*)

(i) <u>Foundation Excavation Preparation</u> (*)

All overburden beneath the proposed saddle dam will be removed. The foundation area for the impervious core and filters will be excavated to sound bedrock, while the rockfill shells will be excavated to the top of bedrock. The final excavated foundation slopes will be no steeper than 1H:1V. Local steep slopes and overhangs will be treated with concrete or appropriately trimmed.

Dental excavation over and above normal excavation will be performed in zones of intense shearing or alteration.

(ii) <u>Grouting</u> (*)

- <u>Consolidation Grouting</u> (*)

The rock under the impervious core, upstream filter, and downstream filter will be consolidation grouted to provide a zone of relatively impermeable rock under the entire contact. The consolidation grout holes will be drilled on a 10 foot by 10 foot grid approximately 30 feet deep.

- Grout Curtain (*)

The depth of grout holes beneath the dam will be 0.7 x H, where H is the maximum head of water at that particular point on the foundation. The grout curtain will have a minimum depth of 50 feet.

On the right side of the saddle dam, the curtain will extend beneath the thrust block of the arch dam to meet the main dam grout curtain.

The grouting will be carried out using the split spacing method, with primary holes at 20-foot spacing. Using secondary and tertiary holes, it will bring the spacing to 5 feet if required. A two-row curtain will be required. The spacing between rows will be 5 feet, with the holes in a staggered pattern.

Grouting will be performed from a gallery running under the dam along the center of the core. The gallery will be a minimum of 50-foot depth into rock. Access will be from the left side of the dam; on the right side, access will be from the main dam abutment drainage gallery. On the right side, the gallery under the dam will slope at two percent to connect with the abutment drainage gallery. This arrangement allows free drainage of the gallery into the main dam drainage system. The galleries will be 10 feet by 10 feet in cross section.

Permafrost is not expected at the Devil Canyon site, but isolated frozen lenses may occur, in which case thawing will be carried out prior to grouting.

(iii) Drainage (*)

The grout gallery will also be used for drainage. The drain holes will be 3 inches in diameter and will follow a similar arrangement to the grout curtain. The drain holes will be inclined downstream by 10 degrees from the vertical.

The drain holes will be downstream from the grout curtain and generally extend 50 feet deeper than the grout holes. The spacing will be selected to ensure that the maximum number of discontinuities are intersected and is expected to be approximately 10 feet. Extra holes may be required in the shear and fault zones.

4.2.2 - Rock Slopes and Foundation Design (*)

(a) General Rock Slopes (*)

Jointing was assumed as the controlling geologic structure for rock slopes. Design slopes were selected considering orientations and continuity of the joint set or sets involved. Sets I and II were assumed to control while Sets III and IV are localized, thus presenting minor problems. Therefore, Sets I and II will be cut back to provide intrinsically stable slopes. Where Set III is present, flatter slopes or heavy support may be required. Set IV joints with 060°/40° NW orientation may present localized stability problems. Other members of this set have shallow dips and should not create problems.

Except as noted below for the spillway and intake structure foundations, the following table summarizes the slopes for each cut orientation.

Strike	Dip	Cut Slopes
N-S	E	4V:1H
N-S	W	4V:1H
NE-SW	SE	2V:1H
NE-SW	NW	4V:1H
E-W	N	4V:1H
E-W	S	2V:1H
NW-SE	NE	4V:1H
NW-SE	SW	2.75V:1H

(b) Arch Dam (o)

It is expected that the treated rock mass foundation modulus is in the range of $1\text{x}10^6$ psi to $3\text{x}10^6$ psi. However, if the abutments do prove to be compressible, they may be treated with pre-tensioned cable anchors, thrust blocks may be used to distribute loads, or short adits can be driven back to sound rock and backfilled with concrete to apply loads deeper in the abutment.

(c) Spillway and Intake Structure Foundations (o)

The orientation of subhorizontal joints (Set IV) will control sliding stability of these structures. A better value for shear strengths of these subhorizontal joints is required before anchoring requirements can be determined. Anchors may not be required if these joints are rough and irregular. No subhorizontal shear zones have been detected so a friction angle of 35° was used in the sliding stability analysis.

Design foundation bearing loads should be less than 5 ksf, and the allowable bearing load on "sound" rock will be greater than 10 ksf, so foundation loads will not create excessive differential deformations. These structures will be founded on sound rock.

4.2.3 - Tunnels and Penstocks (o)

Orientations creating the least problems for the Devil Canyon site tunnels are between 95° and 110° with an acceptable range of 90° to 120°. These tunnel orientations cross major shear zones at high angles. Analysis of the jointing indicates that another favorable orientation may be 175° to 185°. The primary tunnel orientations follow a direction of 70° to 100°.

The penstock tunnels will be concrete-lined over their entire lengths and steel linings will be included just upstream from the powerhouse. These steel linings will be designed to withstand full static and dynamic heads. Their lengths will be determined when actual rock conditions are known. Contact grouting is required to insure good contact between the rock, concrete, and steel.

Four penstocks of 20-foot diameter are proposed. Penstock spacing will be 3 times the diameter, center to center. If further investigations prove excellent rock conditions in the penstock area, spacings may be reduced to twice the diameter.

4.2.4 - Caverns (*)

(a) Support (o)

Because of powerhouse sizes and shear zone spacings, several minor shear zones (less than 5 feet thick) may intersect the powerhouse. These zones will require more than nominal support.

The intersection of nearly vertical and horizontal joints will create blocks in the crown requiring support. This support will be provided by pattern bolting. A detailed analysis will be performed when more specific geologic data are available.

The crown rock bolts recommended for preliminary design have a working load of approximately 80 kips. The preferred type is a tensioned, resin-anchoraged, and resin-encapsulated rock bolt. Wall bolts will be similar to those described in the tunneling section.

Rock bolts for other caverns in the powerhouse complex with spans between 40 feet and 80 feet will use the same capacity, spacings, and percentages of bolts as the powerhouse, with bolt lengths equal to 1/3 of the span for the crown and 1/10 of the wall height for walls. Shotcrete, sets, concrete and wire fabric will be used as required. Caverns with spans less than 40 feet will be supported using the tunnel criteria.

Where shear zones intersect underground openings, more extensive support may be required. Longer, higher-capacity bolts, more closely spaced may be necessary.

(b) Drainage (*)

Drainage will be provided for the walls and crown to prevent seepage from affecting stability. Drain holes will be provided, extending into the rock a distance equal to the greatest rock bolt length or greater. Detailed geology of the powerhouse area is required before the drainage system can be fully designed. By selecting a good to excellent rock body, grouting may be minimized.

(c) Spacing (o)

The rib spacing between the sides of caverns will be kept to 1.5 times the largest cavern span.

(d) Orientation (o)

The most desirable orientation for caverns is either 090° to 120° or 175° to 185°. However, for definite orientation and location, additional investigations and testing, using borings and exploratory adits, are required. The selected cavern orientation represents a compromise of rock support and civil arrangement requirements.

4.2.5 - Arch Dam (*)

(a) Material and Thermal Properties (*)

The material and thermal properties for the Devil Canyon arch dam are as follow.

(i) Static Properties (*)

Concrete

- unit weight .... 150 lb/ft^3
- ultimate uniaxial compressive strength at 365 days .... 5000 psi
- allowable compressive stress .... 1250 psi
- sustained modulus of elasticity .... 3x10^6 psi
- allowable tensile stress .... 325 psi
- Poisson's ratio 0.2

Bedrock

- ultimate compressive strength ... 20,000 psi (unconfined)
- allowable compressive stress..... 5,000 psi
- static modulus of elasticity..... 2 x 10^6 psi
- Poisson's ratio 0.2

(ii) Dynamic Properties (*)

Concrete

- uniaxial dynamic comrpessive strength .... 6,000 psi
- instantaneous modulus of elasticity .... 5 x 10^6 psi
- allowable linear rapid loading tensile strength .... 750 psi
- Poisson's ratio 0.2

Bedrock

- properties assumed as for static conditions

(iii) Thermal Properties (o)

Concrete

- conductivity of concrete ...... 1.52 Btu/ft/hr/°F
- specific heat .... 0.22 Btu/lb/°F

- coefficient of thermal expansion .... 5.6×10^{-6}/ft/°F
- diffusivity .... $-0.0646 ft^2/hr$

(b) General Parameters (o)

The geometry of the dam is shown on Plates F42 and F43. General criteria are as follows:

- o normal maximum reservoir operating level .... Elevation 1,455
- o minimum reservoir operation level. Elevation 1,405
- o dam crest elevation .... Elevation 1,463
- o minimum foundation level .... Elevation 820

(c) Stability Analysis (*)

The arch dam has been analyzed for static loadings and seismically-induced ground motion using the computer program, (ADAS), developed by the USBR based on the trial load method for three-dimensional structures and (SAPIV) for the two-dimension crown cantilever (Acres 1982b; Vol. 5, Appendix B).

The loads and conditions analyzed follow.

(i) Static Loads (o)

- weight of the dam
- hydrostatic pressure from the reservoir
- temperature changes
- ice load

(ii) Dynamic Loads Caused by Seismic Events (o)

- (0.57g) seismic shaking of the dam
- hydrodynamic loads from the reservoir

(iii) Loading Combination (o)

- Usual Load Combination (o)

This consists of groups of sustained loadings which can occur simultaneously over the design life of the dam.

UL1 - Dam self weight + hydrostatic load with reservoir at el. 1,455;
UL2 - Dam self weight + hydrostatic load with reservoir at el. 1,405;

UL3 - As UL1 plus extreme winter temperature effects; and,
UL4 - As UL2 plus extreme winter temperature effects.

- Extreme Load Combination (o)

  This consists of the combination of sustained loads together with short-duration loads caused by seismic motion.

  EL1 - UL1 + extreme earthquake loading.

(iv) Results (o)

The results of the above loading combinations are represented on Plates F45A and F45B. The ice load condition which is not shown in the above mentioned Plates when applied to the UL1 combination produced a maximum stress increase of 12 psi in the arch stresses and 11 psi in the cantilever stresses.

4.2.6 - Saddle Dam Embankment (*)

(a) General (*)

The design philosophy for the saddle dam is essentially the same as that for the dam at Watana.

(b) Embankment Internal Zoning (**)

The impervious core will be protected by fine and coarse filters on both upstream and downstream slopes and supported by rockfill shells. The core will have a crest width of 18 feet and side slopes of 0.7H:1V upstream and 0.2H:1V downstream to provide a core thickness to dam height ratio slightly in excess of 0.5.

The wide filter zones will provide sufficient material to seal any cracks which might occur in the core due to settlement or as the result of seismic displacement.

The saturated sections of both shells will be constructed of compacted clean rockfill, in order to minimize pore pressure generation and ensure rapid dissipation during and after a seismic event.

Protection on the upstream slope will consist of larger stone provided by a 40-foot rock raked zone.

(c) Dam Material (*)

No source of material suitable for impervious has been positiviely identified closer than the borrow areas at Watana (Sites D and H). The current proposal is to use Site D for impervious material for the saddle dam.

The filter material will be obtained from the river deposits (Site G) immediately upstream of the main arch dam. This area will also be exploited for concrete aggregates. The total volume available in Site G is estimated to be 6 million cubic yards, while the concrete aggregate demand is some 2.7 million cubic yards. The estimated volumes required for the dam are 228,000 and 181,000 cubic yards for the fine and coarse filters, respectively. The shell material will be rockfill obtained primarily from the required excavations. The total rockfill required will be approximately 1.2 million cubic yards. The proportion of sound rock suitable for use in the dam, which can be obtained from the excavations, cannot be accurately assessed at this stage. If, the required excavations yield insufficient quantities of rock for construction materials, Quarry Site K will be utilized as a primary rock source.

(d) Stability Analysis (*)

Special precautions have been taken to ensure stability under earthquake loading by the use of free draining rockfill in the saturated zones of the dam, the incorporation of very wide filter zones, and the removal of all unconsolidated natural material from beneath the dam.

Stability analyses of the upstream slopes of the Watana dam, have confirmed stable slopes under all conditions for a 2.4H:1V upstream slope and a 2H:1V downstream slope (see Section 4.1.7 (f)). However, further analyses will be made during design for the Devil Canyon saddle dam.

5 - HYDRAULIC DESIGN DATA

5.1 - River Flows (**)

	Watana	Devil Canyon
Average annual flow	8,050 cfs	9,160 cfs
Maximum average monthly flow (June)	42,800 cfs	47,800 cfs
Minimum average monthly flow (March)	570 cfs	660 cfs

5.2 - Design Flows (**)

Probable Maximum Flood (Routed outflow): Derived from SSARR watershed model. Reservoirs assumed at normal maximum operating level.	302,000 cfs (Stages I&II) 284,000 cfs (Stage III)	351,000 cfs (Stage II) 333,000 cfs (routed through Watana Reservoir) (Stage III)
10,000 year flood inflow, derived from Gold Creek Station record routing through Watana not included in Devil Canyon estimate		
o Mean estimate	174,000 cfs	184,000 cfs
o 95% one-sided upper confidence limit	248,000 cfs	262,000 cfs
Environmental Criteria: 1:50-year flood (routed). Derived from annual and summer flood series frequency analysis with normal generation assumed. Summer flood series controls design with full reservoir conditions in August and September.	34,000 cfs	42,000 cfs
Diversion Design: 1:50-year flood inflow peak at Watana 1:25-year flood at Devil Canyon. Devil Canyon diversion assumes normal power operation and storage at Watana.	89,500 cfs	43,300 cfs

5.3 - Reservoir Levels (**)

Normal Maximum operating level	2,000 ft MSL (Stages I&II) 2,185 ft MSL (Stage III)	1,455 ft MSL
Normal Minimum Operating Level	1,850 ft MSL (Stages I&II) 2,065 ft MSL (Stage III)	1,405 ft MSL

Maximum Reservoir Level	Watana	Devil Canyon
Routed PMF	2,017.1 ft MSL (Stages I&II) 2,199.3 ft MSL (Stage III)	1,465.6 MSL (Stage II) 1,463.1 ft MSL (Stage III)
Environmental Surcharge Level	2,014.0 ft MSL (Stages I&II) 2,193.0 ft MSL (Stage III)	1,456.0 ft MSL
Water Level for passing 10,000 year flood	2,014.3 ft MSL (Stages I&II) 2,193.3 ft MSL (Stage III)	1,456.0 ft MSL

5.4 - Reservoir Operating Rule (**)

	Watana	Devil Canyon
1:50-year surcharge level (1:50-year surcharge due to operating rule for restricted discharges and reduced nitrogen supersaturation.) Reservoirs allowed to surcharge before main spillway operation.	2,014 ft MSL (Stages I&II) 2,193 MSL (Stage III)	1,456 ft MSL

5.5 - Reservoir Data (**)

Reservoir area at normal maximum operating level	20,000 acres (Stages I&II) 38,000 acres (Stage III)	7,800 acres
Reservoir Live Storage: (storage between normal maximum and minimum reservoir levels)	2,400,000 acre-ft (Stages I&III) 3,740,000 acre-ft (Stage III)	350,000 acre-ft
Reservoir Total Storage: (at normal maximum operating level)	4,400,000 acre-ft (Stages I&II) 9,470,000 acre-ft (Stage III)	1,100,000 acre-ft

5.6 - Wind Effect (**)

Wind Effect	Watana	Devil Canyon
Significant Wave Run-up (feet)	3.4	2.2
Wind Set-up (feet)	0.1	0.1

5.7 - Criteria (***)

5.7.1 - Spillways (***)

To be designed in accordance with standard practice as given in U.S. Army Corps of Engineers (COE 1981) and U.S. Bureau of Reclamation guidelines (USBR 1973, Falvey 1980, Peterka 1978) Physical hydraulic model tests will be undertaken to check the design.

(a) Capacity (**)

Pass PMF while maintaining the integrity of the main water retaining structures. An outlet facility for general operation with a main spillway is acceptable. Limited damage to water passages is allowable.

Pass routed 1:10,000-year flood with no damage. An outlet facility for general operation with a main spillway is acceptable. Maintain freeboard for wave height and run-up during event.

Pass routed 1:50-year flood while minimizing gas supersaturation downstream of Project.

(b) Ogees (***)

Ogee capacity, geometry and pressures will be determined from Corps of Engineers and Bureau of Reclamation guidelines cited previously. Pressures resulting from minimum gate openings will be checked.

(c) Piers (***)

Pier geometry will be determined from COE 1981, USBR 1973, Falvey 1980, Peterka 1978.

(d) Walls (***)

Wall heights will be designed using previously cited references to contain flows from the PMF and the 10,000-year flood with appropriate freeboard. Effects of offsets in walls on flow depths will be considered.

(e) Chutes (***)

Chute width, curvature and pressures will be determined in accordance with previously cited references. Aeration devices will be provided as necessary to minimize the potential for cavitation-induced damage.

(f) Energy dissipation (***)

A flip bucket will be provided to disperse the flow and to cause energy dissipation and ensure the safety of the dam during passage of large floods. Geometry and pressures in this structure will be determined from previously cited references and model studies.

(g) Approach channel (***)

The effect of the approach channel geometry on spillway capacity will be determined from model studies and the previously cited references.

5.7.2 - Intakes to Powerhouse and Outlet Works (***)

(a) Powerhouse Multi-Level Intakes (***)

The powerhouse intake works will be designed in accordance with standard practice as given in guidelines of the U.S. Army Corps of Engineers (COE 1960, COE 1980, COE 1982a) and the Bureau of Reclamation (USBR 1973, Hayes 1974). Intake ports will be provided at five levels in Watana Stage I, four levels in Watana Stage III and two levels in Devil Canyon so that water can be withdrawn over the full operating range and temperature impacts downstream, minimized. Trashrack and port geometry design will be based on each port having the capacity to pass the maxmium expected capacity of one turbine. Minimum submergence requirements will be set to prevent the entrainment of air in the flow to the turbines. Surfaces exposed to flow will be shaped to minimize separation of the flow from the surface and to minimize head losses, as far as possible.

While each port will be designed to be capable of supplying flow to a single turbine, temperature control can be enhanced by using multiple ports at a single level to supply each turbine. Therefore, support walls separating each column of ports will contain holes. These holes will also prevent the possibility of unequal hydrostatic loads on either side of the walls.

It is not currently believed that hydraulic model studies will be required to design the multiple level intakes.

(b) Watana Outlet Works (***)

The outlet works intake will be designed in accordance with the guidelines of the U.S. Army Corps of Engineers and Bureau of Reclamation noted for the multi-level intake towers.

The purpose of the outlet works is to pass excess flows to maintain dam safety, pass flows in excess of powerhouse flows necessary to meet environmental flow requirements and, in Stage III, to provide a means for evacuating the upper portion of the reservoir should that become necessary. The intake elevation in Stage I is set to provide the minimum submergence on the intake required to prevent entrainment of air in the flow for a normal maximum water level of el. 2,000. Simulations of reservoir operation showed for Stages I and II showed that outlet works releases were not required when the water level was below el. 2,000. Therefore, in order to provide for the warmest possible release in Stages I, II and III the intake was placed as high as possible subject to the minimum submergence constraint.

Intake approach surfaces and walls will be shaped to minimize the potential for separation of flow from the surfaces. It is not believed that physical model studies will be required to design the outlet works intake.

(c) Devil Canyon Outlet Works (***)

The intakes to the Devil Canyon outlet works will be designed in accordance with standard practice using the guidelines of the U.S. Army Corps of Engineers and the Bureau of Reclamation described before.

The purposes of the Devil Canyon outlet works are to:

- pass flows in excess of powerhouse flows required to meet environmental flow requirements and to provide freeboard on the dam,
- provide a means for evacuating Devil Canyon reservoir, if necessary, and
- provide a means for diverting flow during construction of the dam.

The intake elevation is set based on the requirement to divert flow during construction and so must be relatively low in the dam. Intake surfaces will be designed to minimize separation of flow from the surfaces and to provide efficient operation of the cone valves. A separate intake is provided for each valve.

(d) Diversion Tunnel Intakes (***)

Diversion tunnel intakes will be designed in accordance with standard practice using guidelines of the U.S. Army Corps of Engineers and Bureau of Reclamation cited before. The purpose of the intake works is to guide the river flow into the diversion tunnels without creating excessive head loss or premature pressurization of the tunnel, thus minimizing the combined cost of the diversion cofferdams, and tunnels. Additionally, the diversion tunnel intake will be designed to allow the passage of floating ice through the tunnel without creating a pond upstream of the diversion tunnel. The level of the lowest diversion tunnel intake is set to minimize the water level during the closure of the upstream cotterdam.

Since the diversion tunnels will be required to pass floating ice during the spring breakup period the location, orientation, and geometry of the intake may require physical model studies.

5.7.3 - Water Conductors (***)

(a) Power Tunnels (***)

Power tunnels will be sized to minimize the total costs resulting from construction and energy losses due to friction and form losses. Friction and minor losses will be computed according to standard practice using guidelines of the U.S. Army Corps of Engineers (COE 1960) and the U.S. Bureau of Reclamation (USBR 1977). Internal pressures resulting from transient operations of the powerhouse including load rejection and acceptance will be computed and used in the design of the tunnel linings.

Tunnel sizing will include consideration of the effect on generator speed regulation. Computations of this effect will be based on standard practice and guidelines given in the Handbook of Applied Hydraulics (Rich 1969) and other standard texts and manuals.

(b) Watana Outlet Works Tunnels (***)

Outlet works tunnels will be designed to provide the minimum cost necessary to pass the required flows of 24,000 cfs at the headwater level of el. 2,000. Friction and minor losses will be computed as for the power tunnels. The manifold to distribute flow from the outlet works tunnel to the cone valves will be designed to evenly distribute the flow through all valves and to provide for efficient operation of the valves in dispersing the flow. Physical model studies may be required in the design of the manifold.

(c) Diversion Tunnels (***)

The diversion tunnels and cofferdams for both Watana and Devil Canyon are designed to provide approximately a risk of 1 in 10 that the cofferdams would be overtopped and construction work interrupted during the entire period of construction. A detailed risk analysis will be made to determine the optimum level of protection during construction.

(i) Watana (***)

Watana diversion tunnels will be designed to minimize the total cost of the diversion cofferdam and tunnel system. The criteria is that the tunnels must pass the routed 50-year flood. Additionally, the tunnels must pass ice during freezeup and breakup. The tunnels will be designed to flow open channel under most conditions but will pressurize for large floods. The tunnels will be designed to pressurize from the upstream end first to avoid hydraulic jumps in the tunnels. The lower tunnel will serve as the diversion tunnel during closure and will be set low enough that diversion of the river can be completed without difficulty. The tunnels will be set high enough that bed load sediment carried into the tunnels during high flows will not be trapped in the tunnels and reduce their capacity.

Since the upper diversion tunnel is planned to serve as the emergency release facility during project operation, a section of the tunnel near the center may be wider than the rest of the tunnel to accommodate the gates. A gradual transition will be provided at both upstream and downstream ends of this section to ensure that hydraulic jumps resulting in tunnel pressurization do not occur during operation of this tunnel during diversion. A portion of the lower diversion tunnel will serve as a tailrace tunnel for the powerhouse. This will serve as a constraining factor in the diversion tunnel design as consideration must be given to energy losses when the tunnel is being used as a tailrace tunnel.

Physical hydraulic model studies of the Watana diversion works may be carried out to ensure satisfactory performance in passing the required flows and in the manner of pressurization.

(d) Devil Canyon (***)

The Devil Canyon diversion tunnel is designed to pass the 25-year flood rather than the 50-year flood because of its shorter period of use and the resulting reduction in risk. The Devil Canyon diversion tunnel will be designed in the same manner as Watana. The same general considerations regarding ease of closure operations and minimization of cost of diversion works will apply here as well.

(e) Watana Emergency Release Facilities (***)

Prior to filling of the reservoir and after the dam has been raised to a sufficient level, the upper diversion tunnel will be closed and the emergency release facilities installed. These are more fully described in Exhibit A, Section 1. The capacity of the facility (30,000 cfs) is such that the reservoir can be drawn down, if necessary, in from full pool in 14 months.

The design of this facility will be based on recent world experience with similar structures as at Mica Dam (Meidal and Webster 1973).

(f) Tailrace Tunnels (***)

Tailrace tunnels will be designed in the same manner as power tunnels, to minimize the total cost of energy losses due to friction and minor losses and the construction cost.

Tailrace tunnel and surge facility design will be coordinated to ensure the optimum design of all the downstream facilities between the powerhouse and the river. Tailrace and surge facilities will be designed to meet the turbine manufacturers' specifications with regard to submergence of the units.

A portion of the lower Watana diversion tunnel will serve as part of one of the tailrace tunnels during project operation. A smooth transition will be provided between the tailrace tunnel and the diversion tunnel to ensure hydraulic efficiency.

5.7.4 - Surge Facilities (***)

Surge chambers will be provided downstream of the Watana and Devil Canyon powerhouses to control pressure fluctuations resulting from normal and transient operations of the powerhouses. As indicated for tailrace tunnels, the surge chambers and tailrace tunnels will be designed to minimize the total cost of these facilities, including energy losses and construction costs. All the units from each powerhouse will discharge to a common surge chamber to equalize pressures on the units.

5.7.5 - Outlet Facilities (***)

(a) Diversion Tunnels Tailrace Channels (***)

(i) Watana (***)

The Watana diversion tunnels will discharge into the river through unlined tailrace channels excavated in the rock.

The downstream invert of the lower or first tunnel will be below the bed elevation to ensure supercritical flow in the tunnel and pressurization of the tunnel from the upstream end. The invert will be set high enough to ensure that material passing through the tunnel will not accumulate in the tunnel or tailrace channel. The channel will slope upwards from the tunnel to the existing streambed. An hydraulic jump is expected to occur in the downstream end of the tunnel for low flows. This jump will not reach the crown of, nor pressurize the tunnel. For average summer flows of 20,000 cfs, the jump will occur in the channel downstream of the tunnel.

The downstream invert of the upper tunnel will be near the elevation of the streambed, and the tunnel slope will ensure supercritical flow in that tunnel. An hydraulic jump is expected to form in the tailrace channel at low flows and in the river downstream at higher flows.

Hydraulic jumps can entrain air and cause elevated gas concentrations. Therefore the diversion tunnel outlet facilities will be designed to minimize the potential for gas supersaturation. Guidelines of the U.S. Bureau of Reclamation (Johnson 1975) will be used in designing the structure. Physical model studies may be necessary to ensure performance of these facilities.

(ii) Devil Canyon (***)

The Devil Canyon diversion tunnel outlet structure is at approximately the same elevation as the streambed at its downstream portal. An unlined channel will be excavated in the rock to convey the flow to the river. Supercritical flow will be assured in the tunnel and an hydraulic jump is expected to form in the tailrace channel or the river downstream. The tailrace channel will be designed to minimize the potential for gas supersaturation to occur. U.S. Bureau of Reclamation guidelines (Johnson 1975) will be used and physical model studies may be required. Experience gained in the design and operation of the Watana diversion facilities may negate the need for physical model studies of the Devil Canyon diversion tunnel outlet works.

(b) Watana Emergency Release Facilities (***)

The upper diversion tunnel tailrace channel will be modified when the emergency release facilities are placed in that tunnel. A flip bucket will be constructed which will disperse flows released through that tunnel to achieve energy dissipation without an hydraulic jump. The flip bucket lip will be above the tailwater elevation.

(c) Power Tunnel Tailrace Channels (***)

(i) Watana (***)

The tailrace outlet channels will be similar to the diversion tunnel outlet channels. In fact, the lower diversion tunnel is designed to function as a tailrace tunnel and its outlet channel will be unmodified from the diversion to the operating phase. In the design of the tailrace channels costs resulting energy losses in the tailrace channels will be added to construction costs to ensure that the optimum design is obtained.

(d) Outlet Works Facilities (***)

Fixed cone valves will be provided at Watana and Devil Canyon Dam outlet works to control releases required:

- to meet environmental flow requirements in excess of powerhouse flows.

- o for floods with return periods of less than 50 years.

Unhooded fixed cone valves have been selected as the means for dissipating energy from floods up to the 50-year event in order to minimize the potential for gas concentrations downstream of the project to exceed naturally occurring levels. Six valves are provided at Watana and seven at Devil Canyon. The required capacities are 24,000 cfs total at Watana during all three stages and 42,000 cfs at Devil Canyon in Stages II and III. These capacities would be provided with the valves operating at 80 percent of full gate stroke to minimize vibration.

(e) Flip Buckets (***)

Both Watana and Devil Canyon spillways terminate in flip buckets. These structures are designed to direct spillway flow into the river clear of the concrete structures and well downstream in the river, so that the potential erosion of material induced by the flow does not endanger project structures.

This may be achieved by designing the flip bucket surface and chute geometry, and exit angle and shape, to disperse the flow over as large an area within the streambanks as possible to minimize the ability of the flow to erode the streambed. This would have the additional benefit of dispersing low spillway flows, and minimizing the potential for gas supersaturation as a result of spillway operation. Flip bucket design normally requires physical model studies to determine dynamic pressures, flow dispersion and erosion patterns.

6 - EQUIPMENT DESIGN CODES AND STANDARDS (**)

6.1 - Design Codes and Standards (*)

6.1.1 - Turbines (o)

- ASME Boiler and Pressure Vessel Code, Section VIII, Pressure Vessels;
- ANSI Standard B49.1;
- ANSI Standare B31.1 - Power Piping;
- AWS Standard D1.1 - Structural Welding Code;
- IEC Publication 193 "International Code for Model Acceptance Tests of Hydraulic Turbines"; and
- IEC Publication 41 "International Code for Field Acceptance Tests of Hydraulic Turbines, Storage Pumps and Pump-Turbines.

6.1.2 - Gate Equipment (o)

- AISC Specification for Design, Fabrication and Erection of Structural Steel for Buildings;
- AWS D1.1 - Structural Welding Code;
- ACI 318 - Building Code Requirements for Reinforced Concrete;
- ASME Boiler and Pressure Vessel Code, Section VIII, Pressure Vessels;[1/] and
- ANSI Standard B31.1[1/]

6.1.3 - Guard Valve Equipment (**)

- Applicable parts of ANSI, AWWA and API Standards;
- ASME Boiler and Pressure Vessel Code, Section VIII, Pressure Vessels.

6.1.4 - Crane Equipment (o)

- CMAA Specification No. 70 - Specifications for Electric Overhead Traveling Cranes;
- CMAA Specification No. 74 - Specifications for Single Girder Overhead Traveling Cranes; and
- OSHA Standards.

6.1.5 - Elevators (o)

- ANSI Standard A17.1; and
- Stage Building Codes.

1/ Hydraulic hoist design

6.1.6 - Mechanical Systems (o)

	Applicable Systems[2/]
- ANSI Standard B31.1	1, 2, 3, 4, 6, 7
- AWS Standard D1.1	1, 2, 3, 4, 5, 6, 7
- NFPA Standards	3, 5, 8
- ASME Boiler and Pressure Vessel Code, Sections 2, 8 and 9	3, 4
- API Standard 650, Welded Steel Tanks for Oil Storage	5
- ANSI Standard D31.3, Petroleum Refining Piping	5
- AWWA Standards	6
- Environmental Protection Agency	6
- ASHRAE Guide	8
- State Building Codes	6, 2

6.2 - General Criteria (*)

6.2.1 - Turbines (*)

(a) Operation (o)

The turbines will be capable of continuous operation at speed-no-load and at any gate opening between 100 percent and 50 percent full gate output without excessive surges in power, detrimental vibrations or excessive noises. The turbines will be designed for continuous operation at maximum runaway speed.

(b) Stresses (o)

Stresses in turbine components under normal operating conditions, including pressure rise on full load rejection, will not exceed 1/3 of the yield strength for materials of steel construction. For miscellaneous materials, stress levels will not exceed the following:

2/ 1. Service Water Systems
2. Domestic Water Systems
3. Fire Protection Systems
4. Compressed Air Systems
5. Oil Storage and Handling Systems
6. Drainage Systems
7. Dewatering Systems
8. Heating and Ventilating Systems

- Cast iron	2,000 lb/in^2 tension 10,000 lb/in^2 compression
- Bronze bearings	3,000 lb/in^2
- Babbitt bearings	5,000 lb/in^2

On extreme loading conditions such as operation at runaway speed, stress levels may be increased provided they do not exceed 2/3 of the yield strength of the material.

- Critical Speed (o)

The first critical speed in shaft bending for the combined turbine and generator will be at least 125 percent of the maximum runaway speed of the turbine.

- Cavitation (*)

The maximum metal loss (in lb) due to cavitation pitting during any 8,000-hour operating period will not exceed 0.2 times the square of the discharge diameter of the runner (in ft).

6.2.2 - Gate Equipment (o)

(a) Gates and Guides (o)

For normal loading conditions including hydrostatic and applicable hydrodynamic and lifting loads, stress levels on structural components will not exceed those permitted in the AISC Specification for Design, fabrication and Erection of Structural Steel for Buildings. Stresses in welded and bolted connections will not exceed 90 percent of the values permitted by the AISC Specification. For gates subjected to dynamic loading, stresses in structural components and in connections will be reduced a further 20 percent. For crowned gate wheels or rollers on a flat track, the load (in lb) per inch width of roller contact will not exceed 1,600 times the roller diameter.

On extreme loading conditions with the gate becoming jammed on raising, stress levels may be increased by 33 percent.

A corrosion allowance of 1/16-inch will be allowed on all gate components in contact with water.

Unless provision is made for forcing at gate down, the preponderance for all gates will be at least 15 percent assuming static friction coefficient.

(b) Hoists (o)

Hydraulic hoists will be designed in accordance with the ASME Boiler and Pressure Vessel Code, with a rated capacity of at least 140 percent of the calculated lifting loads and a maximum working pressure of 2,000 lb/in^2. The cylinder will also conform to the criteria recommended by the National Fluid Power Association.

For wire rope hoists, stresses will not exceed 1/3 of the yield strength of the material for normal loading including an allowance for impact. The load on wire rope will not exceed 1/5 of the minimum breaking strength. For extreme loading conditions with a gate becoming jammed, stress levels may be increased provided they do not exceed 67 percent of the yield strength of the material. For extreme loading conditions, the load on wire rope will not exceed 80 percent of the minimum breaking strength.

6.2.3 - Valves (o)

Valves will be designed in accordance with the ASME Boiler and Pressure Vessel Code. For fixed cone valves, special attention will be given to the prevention of vibration and cavitation.

6.2.4 - Trashracks(o)

Trashracks will be designed with the allowable stresses permitted in (b) above for gate equipment. Rack vibration will also be considered in the design.

6.2.5 - Cranes (*)

Cranes will be designed in accordance with the applicable CMAA Specification. For cranes which handle gates, a jammed gate condition will be considered where stress levels and wire rope loads will not exceed those permitted in 6.2.2 above for extreme loading on wire rope hoists.

6.2.6 - Mechanical Systems (o)

Full redundancy will be provided for pumps, strainers, and similar equipment which are critical for generating unit operation.

6.3 - Diversion Structures and Emergency Release Facilities (*)

6.3.1 - Diversion Control Gates (*)

Fixed wheel vertical lift gates will be provided at the intakes to the diversion tunnels. The gates will be used for closure of the diversion tunnels to permit plugging operations. The gates will also be used to control flows as necessary when the water level is below the gate opening lintel to prevent passage of ice through the diversion tunnels.

The gates will have downstream skinplates and seals. Provision will be made for gate and guide heating if the gates are used for control during cold weather. The gates will be operated by fixed hoists mounted in a tower and bridge structure.

The gates in the upper diversion tunnel at Watana will be removed once tunnel plugging is complete. The gates for the other tunnels will have retractable wheels for transfer of hydrostatic loads to the guides after diversion closure when the head increases as the reservoir is impounded.

6.3.2 - High Pressure Slide Gates (Watana) (*)

High pressure slide gates will be installed in the tunnel plugs in the upper diversion tunnel at Watana. The slide gates will be used for:

- o Passing required releases during reservoir impoundment; and
- o Emergency draining of the reservoirs throughout the life of the plant.

The gates will be installed after initial closure of the diversion tunnel. The arrangement will consist of three sets of three gates in series. Each set will consist of two gates in an upstream plug (one emergency and one operating gate) and one operating gate in a downstream plug. The area between the plugs will act as an expansion chamber to assist in emergency dissipation. The gates will be designed to operate at full or partial opening for heads up to 500 feet. This will allow the drawing down of the reservoir from the Stage I 50-year flood pool level of el. 2,014 in an emergency. When closed, the gates will withstand full reservoir level. The two operating gates in series will be operated at equal openings at all times to effectively balance the head across the gates.

6.3.3 - Trash Beams (Watana) (*)

Trash beams will be installed at the Watana upper diversion tunnel at the same time the high pressure slide gates are installed. Provision for rack removal is not considered necessary.

The criteria for the trash beams are as follows:

Maximum beam spacing	2/3 of the high pressure slide gate width
Maximum velocity through beams (net)	12 ft/s
Design differential level	40 ft (approximate)

6.3.4 - Diversion Tunnel Stoplogs (o)

Stoplog guides and stoplogs will be provided at the downstream end of the diversion tunnels to permit tunnel dewatering after diversion closure for plugging operations. The stoplogs will be handled by a mobile crane with a follower.

6.4 - Spillway (**)

6.4.1 - Spillway Gates (**)

The spillway gates will be radial gates operated by hydraulic hoist mounted on the piers and abutments' guide walls. The hoist and gate trunnions and hoist connection to gate will be equipped with self-lubricating bearings suitable for temperatures down to -100°F. The hydraulic fluid for the hoist hydraulic system will be according to MIL-H-5606A which is expected to give trouble-free operation (without freezing) down to -40°F.

Provision will be made for installation of heaters in the gate side seal plates.

6.4.2 - Stoplogs (**)

A set of stoplog guides will be provided upstream from each spillway gate to permit installation of stoplogs and inspection of the spillway gate guides or raising the spillway gate for maintenance without passing water over the spillway.

One set of stoplogs will be provided to be handled by a mobile crane and follower.

At Watana, to allow for installation of stoplogs for raising to the Stage III spillway crest level stoplog slots will be provided in each pier. The bearing surface will lie in the vertical plane of the upstream spillway pier noses.

6.5 - Outlet Facilities (*)

6.5.1 - Fixed Cone Valves (*)

Fixed cone valves will be used to pass normal discharges, other than the flows through the powerhouse. The valves will also assist in passing required release during reservoir impoundment. At Watana, a single tunnel with a manifold and six valves will be provided. Devil Canyon will have seven valves and individual conduits for each valve.

The valves will be selected within current experience with respect to valve size and design head. The thickness of the internal vanes of the valves will not be less than 3 percent of the valve nominal diameter to provide a margin of safety against destructive vane vibration. In sizing the valves, the cylindrical gate opening will be assumed to be restricted to about 80 percent of its theoretical maximum to prevent possible vibration. The valves at Watana will be designed for operation conditions occurring during Stage III.

Each valve body will be heated for winter operation. A heated valve gallery will be provided with crane equipment for servicing and maintaining the valves.

6.5.2 - Ring Follower Gates (o)

One ring follower gate will be provided immediately upstream from each fixed cone valve to:

- o Relieve the hydrostatic load on the valve when it is not in operations;
- o Permit inspection and maintenance of the valve; and
- o Close under full flow conditions in the event of malfunction of the valve.

The ring follower gate will be located within a heated enclosure with suitable provision for servicing the equipment.

6.5.3 - Upstream Maintenance Gate (*)

Provision will be made for installation of a gate at the upstream entrance to the outlet tunnel. At Watana, fixed wheel gates will be provided which can close under flowing water conditions. Bulkhead type gates will be provided at Devil Canyon because of the extremely high head.

At Watana, because of a single tunnel, the gates will be operated by hydraulic hoists located in the gate walls. A gantry crane will be used to handle the gates at Devil Canyon.

6.5.4 - Trashracks (*)

Trashracks will be provided at the upstream end of the outlets. Because the valves serve as the primary discharge facilities, consideration will be given to making provision for rack removal.

The criteria for the trashracks will be as follows:

Maximum velocity (net)	4.5 ft/s
Spacing	0.1 x valve size (approximate)
Design differential head	40 ft (approximate)

6.6 - Power Intake (*)

6.6.1 - Trashracks (*)

Trashracks will be installed upstream from each intake opening. Provision will be made for rack removal.

The criteria for the trashracks will be as follows:

Maximum velocity (based on gross area	4.5 ft/s (approx.)
Beam spacing)	maximum spacing not to exceed minimum distance between runner blades
Design differential head	20 ft (approx.)

6.6.2 - Intake Gates (*)

Fixed wheel type vertical lift gates will be installed at the entrance to each power tunnel. The gates will be used to permit dewatering of the power tunnel, penstocks, and turbine water passages for turbine inspection and maintenance and for closure in an emergency in the event of loss of control of the turbine.

The gates will be operated by individual hydraulic hoists mounted in the gate wells.

6.6.3 - Intake Bulkhead Gates (o)

Intake bulkhead gates will be provided for installation upstream from the intake gates. The gates will be handled by a gantry crane or overhead traveling crane. Sufficient gates for one intake opening at each project will be provided.

6.6.4 - Water Level Shutters (o)

Removable shutters will be installed in the intake at Watana and Devil Canyon to permit drawing off water at selected elevations. One set of shutters will be provided at each intake opening. The shutters will be designed for approximately 15 feet of differential head. The arrangement will be such that a higher differential head will not occur.

6.7 - Powerhouse (**)

6.7.1 - Turbines (**)

The turbines will be vertical shaft Francis type directly connected to synchronous generators. The turbines will have steel spiral cases and steel-lined concrete elbow draft tubes. The four Stage I turbines will be designed to operate under the conditions of Stages I and III without modification.

The ultimate installed capacity and the ultimate number of generating units to be installed at Watana were established in the Susitna Hydroelectric Project Feasibility Report, Volume 1, Section 9 (Acres 1982b), to meet system needs for flexibility of operation, reliability, and reserve capacity and to match the hydraulic capacity of the river with a suitable load factor. The feasibility studies resulted in an ultimate installation of six units each with a rated unit output of 170 megawatts with the minimum head expected to prevail in the peak load month of December with the selected Case "C" flow regime for the project reservoir corresponding to Stage III. In the data tabulated below, the rated head is defined as the head which prevails at the rated unit output.

The units in this document are identical in their ratings to those selected in that Feasibility Report with the minor revision that the minimum head criterion used is based upon the average of the minimum reservoir prevailing in December and January in Stage III and using the current flow regime Case E-VI. Further analysis since the original application has shown the critical load months to be both December and January. The data in the table presented below indicate the turbine capacity for the Watana turbines in Stage I as well as Stage III.

The usual maximum range of operation of a Francis turbine is taken as from approximately 65 percent of its design head (the design head is the head at which optimum efficiency is obtained) to approximately 125 percent of its design head. Using these criteria, the design head for the Stage I turbines is established at 590 feet in order to permit these units to operate with suitable efficiencies with the reservoir raised in Stage III. The two turbines which are installed in Stage III will have their design head at 680 feet to have their peak efficiency within the narrower range of heads which will prevail in Stage III.

The Devil Canyon units are sized to have an output of 150 MW at the minimum normal water surface elevations. These are the same units that were described in the original license application (APA 1983).

The preliminary turbine data have been established as follows:

	Stage I Watana	Stage II Devil Canyon	Stage III Watana
Number of units	4	4	4/2*
- Head (net) (ft.)			
- rated	N/A	545	645/645
- design	590	590	590/680
- maximum operating	537	600	719/719
- average operating	490	597	680/680
- minimum operating	384	545	600/600
Power (MW)			
- at rated head	N/A	153.1	173.5/173.5
- at maximum operating head	128.5	177.0	204.3/204.3
- at average operating head	108.9	175.7	188.1/188.1
- at minimum operating head	64.7	150.1	154.7
Synchronous Speed (rpm)	257.1	225	257.1
Specific Speed	39.8	35	39.8/37.1

* Stage I Units/Stage III Units

6.7.2 - Turbine Guard Valves (***)

At Watana 12.5 foot diameter butterfly valves will be provided upstream of each turbine. These guard valves will be designed to operate under the maximum head conditions occurring in Stage III. Valve closure will allow the inspection of turbine passages without dewatering of the power tunnel and penstocks.

6.7.3 - Powerhouse Cranes (*)

The powerhouse cranes will be of the electric overhead traveling type with main and auxiliary hoists. The cranes will be used for:

- o Installation of the turbines, guard valves (at Watana), generators, and other equipment; and
- o Dismantling and reinstallation of equipment during maintenance overhaul once the station is in operation.

Each station will have two cranes. The combined main hoist capacity of the two cranes will be at least equal to the weight of the generator rotor plus lifting beams.

6.7.4 - Draft Tube Gates (o)

Draft tube gate guides will be provided at the end of each draft tube to permit installation of draft tube gates and dewatering of the turbine water passages for inspection and maintenance of the turbines.

The draft tube gates will be of the bulkhead type handled by a traveling gate crane.

6.7.5 - Miscellaneous Mechanical Equipment (**)

Miscellaneous mechanical euqipment will include:

- o A passenger elevator in the powerhouse;
- o An access elevator from the surface to the powerhouse;
- o "Alimak" type inspection hoists in the SF_6 bus shaft; and
- o Small motorized or hand-operated monorail hoists or A-frames provided where necessary for servicing miscellaneous equipment.

6.7.6 - Mechanical Services (o)

The mechanical services within the powerhouse will include:

- o Station service water systems
  - . water supply
  - . cooling water
  - . domestic water
- o Fire protection
  - . fire protection water system
  - . sprinkler system
  - . portable fire protection system

- o Compressed air system
  - . service area system
  - . tailwater depression air
  - . governor air
  - . circuit breaker air
- o Oil storage and handling
  - . transformer oil system
  - . governor and lubricating oil system
- o Drainage and dewatering system
  - . clearwater drainage
  - . unit dewatering and filling system
  - . sanitary drainage system
- o Heating and ventilating system

6.8 - Tailrace Tunnels (**)

6.8.1 - Stoplogs (**)

Stoplog guides and stoplogs will be provided at the downstream end of the tailrace tunnels to permit dewatering of the tunnels for inspection and maintenance. The stoplogs will be handled by a mobile crane with a follower.

At Watana during Stage I there will only be one tailrace tunnel, and plant shut-down will be required when dewatering for inspection is required. During Stage III there will be two tailrace tunnels. Stoplog guides and stoplogs will be provided for the tunnel inlet (in the surge chamber) to allow dewatering of one tailrace tunnel while still permitting plant operations using the other tunnel. The stoplogs will be handled by a traveling gate crane.

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TABLES

TABLE F.1.1.1: PERTINENT PROJECT DATA

Item	Stage I Watana	Stage II[1/] Devil Canyon	Stage III[1/] Watana	Stage III[1/] Devil Canyon
Hydrology				
- Average River Flow (cfs)	8,050	9,160	----	----
- Peak Flood Inflows (cfs)				
. PMF	326,000	358,000 with Watana 362,000 without Watana	----	339,000 with Watana 362,000 without Watana
. 10,000-year	174,000	184,000 without Watana	----	----
. 50-year	89,500	46,000 with Watana 94,800 without Watana	----	44,600 with Watana (Yr 2008) 35,300 with Watana (Yr 2020) 94,800 without Watana
. 25-year	79,800	44,600 with Watana 84,500 without Watana	----	43,200 with Watana (Yr 2008) 31,600 with Watana (Yr 2020) 84,500 without Watana
- Peak Flood Flows through the Dam (cfs)				
. PMF	302,300	351,000 with Watana	283,600	333,000 with Watana
. 50-year	34,000	42,000 with Watana	33,900	42,000 with Watana
Reservoir Characteristics				
- Normal Maximum Operating Level [3/]	2,000	1,455	2,185	----
- Maximum Level, PMF [3/]	2,017.1	1465.6	2,199.3	1,463.1
- Minimum Operating Level [3/]	1,850	1,405	2,065	----
- Area at NMOL (acres)	19,900	7,800	38,000	----
- Length at NMOL (miles)	39	26	48	----
- Total Storage (acre-feet)	4.3×10^6	1.1×10^6	9.5×10^6	----
- Live Storage (acre-feet)	2.4×10^6	0.35×10^6	3.7×10^6	----
- Maximum Allowable Surcharge Level for 50-year Flood [3/]	2,014	1,456	2,193	----
- Average Tailwater [3/]	1,455	850	1,455	----
Project Outputs				
- Dependable Plant Capability (December-January) (MW)	360	600	1,020	----
- Nominal Plant Capability (MW) [4/]	440	680	1,110	----
- Annual Generation (GWh)				
. Firm	1,950	4,490[2/]	5,720[2/]	
. Average	2,400	4,750[2/]	6,900[2/]	

N.A. - Not Applicable

1/ Watana Stage I data as shown applies both before and after construction of Devil Canyon, except where indicated for Stage III. Devil Canyon Stage II data, as shown, applies both before and after construction of Watana Stage III except where indicated otherwise for Stage III.

2/ Total generation from Watana and Devil Canyon.

3/ Contour elevation (feet above mean sea level)

4/ At average operating head

TABLE F.1.1.1 (Page 2 of 4)

Item	Stage I Watana	Stage II 1/ Devil Canyon	Stage III 1/ Watana	Stage III 1/ Devil Canyon
Dams				
- Type	Earth/Rockfill, Inclined Core	Concrete Arch (Earth/Rockfill Saddle)	Earth/Rockfill Central Core	----
- Crest Elevation 3/	2,027	1,463 (1472)	2,210	----
- Crest Length (ft)	2,700	1,650 (950)	4,100	----
- Height Above Foundation 3/	702	646 (245)	885	----
- Crest Width (ft)	35	20 (35)	35	----
- Upstream Slope (H:V)	2.4:1	N.A. (2.4:1)	2.4:1	----
- Downstream Slope (H:V)	2:1	N.A. (2:1)	2:1	----
- Allowance for Settlement (ft)	2	0 (2)	5	----
- Top of Parapet 3/	N.A.	1,466.0	N.A.	----
Diversion				
- Recurrence Interval of Design Flood (yrs)	50	25	N.A.	N.A.
- Cofferdams				
. Type	Earth & Rockfill	Earth & Rockfill	----	N.A.
. Upstream Crest Elevation 3/	1,550	947	N.A.	N.A.
. Downstream Crest Elevation 3/	1,495	898	1,495	N.A.
. Maximum U/S Water Level for Design Flood 3/	1,532	944	----	----
- Tunnels				
. Number/Type	2 - Circular, Concrete-Lined	1 - Horseshoe, Concrete-Lined	N.A.	N.A.
. Diameter (ft)	36	35.5	N.A.	N.A.
. Capacity for Design Flood (cfs)	77,000	43,300	N.A.	N.A.
Outlet Facilities				
- Control Structures	6-Fixed Cone Valves	7-Fixed Cone Valves	6-Fixed Cone Valves	----
- Diameter (in)	78	4-102, 3-90	78	----
- Water Passage Diameter (ft)	28	8.5/7.5	28	----
- Capacity (cfs)	24,000	42,000	30,000	----

N.A. - Not Applicable

1/ See first page of this Table.
3/ See first page of this Table.

TABLE F.1.1.1 (Page 3 of 4)

Item	Stage I Watana	Stage II[1] Devil Canyon	Stage III[1] Watana	Stage III[1] Devil Canyon
Spillway				
- Capacity at 50-yr flood surcharge (cfs)	258,000	240,000	220,000	----
- Capacity at PMF surcharge (cfs)	278,400	309,000	259,600	291,000
- Control Structure				
. Type	Gated Ogee	Gated Ogee	Gated Ogee	----
. Crest Elevation [3]	1,950	1,398	2,135	----
. Gates				
Number	3	3	3	----
Dimensions (HxW, ft)	64 x 44	58 x 48	64 x 44	----
Top of Gate Level [3]	2,014	1,456	2,199	----
- Chute Width (ft)	164 x 120	176 x 150	164 x 120	----
- Energy Dissipation	Flip bucket	Flip bucket	Flip bucket	----
Power Intakes				
- Intake Structures	Multi-level, Gated	Multi-level, Gated	Multi-level, Gated	----
. Number of Levels	5	2	4	----
. Number of Shutters per Level	4	4	6	----
. Dimensions of Shutters (HxW, ft)	25 x 24	20 x 34±	25 x 24	----
- Control Gates				
. Number	4	+ 4	6	
. Dimensions (HxW, ft)	24 x 12	24 x 20	24 x 12	
- Invert Elevation [3]	1,800	1,365	1,800 & 2012	----
Power Tunnels				
- Number	2	See Penstocks	3	----
- Type	Inclined/Horizontal	See Penstocks	Inclined/Horizonal	----
- Concrete-Lined Diameter (ft)	24	See Penstocks	24	----
Penstocks				
- Number	4	4	6	----
- Type	Horizontal	Inclined/Horizontal	Horizontal	
- Diameter (ft)				----
. Concrete-lined	18	20	18	----
. Steel-lined	15	15	15	----

1/ See first page of this Table.
3/ See first page of this Table.

Item	Stage I Watana	Stage II[1] Devil Canyon	Stage III[1] Watana	Stage III[1] Devil Canyon
Powerhouses				
- Type	Underground	Underground	Underground	----
- Cavern Size (L x W x H, ft)	365 x 78 x 136	360 x 74 x 126	520 x 78 x 136	
- Turbine (No. and Type)	4 Vertical Francis	4 Vertical Francis	6 Vertical Francis	----
- Speed (rpm)	257	225	257	----
- Nominal Unit Capability at Average Operating Head (MW)	110	170	185	----
- Maximum Unit Capability				
. Net Head (ft)	537	600	719	----
. Flow (cfs)	3,080	3,790	3,800	----
. Output (MW)	125	173	200	----
- Minimum Unit Capability				
. Net Head (ft)	384	545	600	----
. Flow (cfs)	2,310	3,615	3,310	----
. Output (MW)	65	150	150	----
- Generators				
. Type	Vertical Synchronous	Vertical Synchronous	Vertical Synchronous	----
. Rated Capacity (MVA)	223 Air Cooled	192 Air Cooled	223 Air Cooled	----
. Power Factor	0.9	0.9	0.9	----
. Voltage (kV)	15	15	15	----
. Frequency (Hz)	60	60	60	----
. Speed, rpm	257	225	257	----
- Transformers				
. Location	Upstream Gallery	Upstream Gallery	Upstream Gallery	----
. Cavern Size (L x W x H, ft)	308 x 45 x 40	446 x 43 x 40	414 x 45 x 40	
. Number	6	12	9	----
. Rating (MVA)	150	70	150	----
. Voltage (kV)	15-345/1.73 Single Phase	15-345/1.73 Single Phase	15-345/1.73 Single Phase	----
Tailrace Tunnels				
- Number/Type	1 - Horseshoe, Concrete-Lined	1 - Horseshoe Concrete-Lined	2 - Horseshoe Concrete-Lined	----
- Diameter (ft)	34	38	34	----
- Surge Chamber Size (L x W x H, ft)	250 x 50 x 150	240 x 75 x 190	445 x 50 x 150	----

N.A. - Not Applicable

1/ See first page of this Table.

Table F.2.2.1 WATANA NATURAL MONTHLY FLOWS (CFS)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
1951	3299	1107	906	808	673	620	1302	11650	18518	19787	16478	17206	7734
1952	4593	2170	1501	1275	841	735	804	4217	25773	22111	17356	11571	7777
1953	6286	2757	1281	819	612	671	1382	15037	21470	17355	16682	11514	8035
1954	4219	1600	1184	1088	803	638	943	11697	19477	16984	20421	9166	7401
1955	3859	2051	1550	1388	1051	886	941	6718	24881	23788	23537	13448	8719
1956	4102	1588	1039	817	755	694	718	12953	27172	25831	19153	13194	9051
1957	4208	2277	1707	1373	1189	935	945	10176	25275	19949	17318	14841	8381
1958	6035	2936	2259	1481	1042	974	1265	9958	22098	19753	18843	5979	7770
1959	3668	1730	1115	1081	949	694	886	10141	18330	20493	23940	12467	8011
1960	5166	2214	1672	1400	1139	961	1070	13044	13233	19506	19323	16086	7954
1961	6049	2328	1973	1780	1305	1331	1965	13638	22784	19840	19480	10146	8603
1962	4638	2263	1760	1609	1257	1177	1457	11334	36017	23444	19887	12746	9833
1963	5560	2509	1709	1309	1185	884	777	15299	20663	28767	21011	10800	9278
1964	5187	1789	1195	852	782	575	609	3579	42842	20083	14048	7524	8263
1965	4759	2368	1070	863	773	807	1232	10966	21213	23236	17394	16226	8451
1966	5221	1565	1204	1060	985	985	1338	7094	25940	16154	17391	9214	7374
1967	3270	1202	1122	1102	1031	890	850	12556	24712	21987	26105	13673	9096
1968	4019	1934	1704	1618	1560	1560	1577	12827	25704	22083	14148	7164	8032
1969	3135	1355	754	619	608	686	1262	9314	13962	14844	7772	4260	4912
1970	2403	1021	709	636	602	624	986	9536	14399	18410	16264	7224	6115
1971	3768	2496	1687	1097	777	717	814	2857	27613	21126	27447	12189	8589
1972	4979	2587	1957	1671	1491	1366	1305	15973	27429	19820	17510	10956	8963
1973	4301	1978	1247	1032	1000	874	914	7287	23859	16351	18017	8100	7112
1974	3057	1355	932	786	690	627	872	12889	14781	15972	13524	9786	6314
1975	3089	1474	1277	1216	1110	1041	1211	11672	26689	23430	15127	13075	8403
1976	5679	1601	876	758	743	691	1060	8939	19994	17015	18394	5712	6835
1977	2974	1927	1688	1349	1203	1111	1203	8569	31353	19707	16807	10613	8233
1978	5794	2645	1980	1578	1268	1257	1408	11232	17277	18385	13412	7133	6992
1979	3774	1945	1313	1137	1055	1101	1318	12369	22905	24912	16671	9097	8184
1980	6150	3525	2032	1470	1233	1177	1404	10140	23400	26740	18000	11000	8908
1981	6632	3044	1790	1858	1592	1262	1641	14416	16739	27601	30542	11669	9985
1982	5700	2650	1863	1700	1234	898	1196	10879	21444	20445	13206	13890	7968
1983	5154	2132	1893	1797	1610	1427	1565	11672	20401	18761	20862	11192	8253
MAX	6632	3525	2259	1858	1610	1560	1965	15973	42842	28767	30542	17206	9985
MIN	2403	1021	709	619	602	575	609	2857	13233	14843	7772	4260	4912
MEAN	4567	2064	1453	1225	1035	936	1158	10625	22980	20747	18366	10875	8046

Table F.2.2.2 DEVIL CANYON NATURAL MONTHLY FLOWS (CFS)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
1951	3652	1231	1031	906	768	697	1505	13219	19979	21576	18530	19799	8616
1952	5222	2539	1758	1484	943	828	879	4990	30014	24862	19647	13441	8918
1953	7518	3233	1550	1000	746	767	1532	17758	25231	19184	19207	13928	9356
1954	5109	1921	1387	1224	930	729	1131	15286	23188	19154	24072	11579	8867
1955	4830	2507	1868	1649	1275	1024	1107	8390	28082	26213	24960	13989	9707
1956	4648	1789	1207	922	893	852	867	15979	31137	29212	22610	16496	10608
1957	5235	2774	1987	1583	1389	1105	1109	12474	28415	22110	19389	18029	9669
1958	7435	3590	2905	1792	1212	1086	1437	11849	24414	21763	21220	6989	8867
1959	4403	2000	1371	1317	1179	878	1120	13901	21538	23390	28594	15330	9650
1960	6061	2623	2012	1686	1340	1113	1218	14803	14710	21739	22066	18930	9084
1961	7171	2760	2437	2212	1594	1639	2405	16031	27069	22881	21164	12219	10021
1962	5459	2544	1979	1796	1413	1320	1613	12141	40680	24991	22242	14767	10947
1963	6308	2696	1896	1496	1387	958	811	17698	24094	32388	22721	11777	10432
1964	5998	2085	1387	978	900	664	697	4047	47816	21926	15586	8840	9251
1965	5744	2645	1161	925	829	867	1314	12267	24110	26196	19789	18234	9556
1966	6497	1908	1478	1279	1187	1187	1619	8734	30446	18536	20245	10844	8697
1967	3844	1458	1365	1358	1268	1089	1054	14436	27796	25081	30293	15728	10460
1968	4585	2204	1930	1851	1779	1779	1791	14982	29462	24871	16091	8226	9176
1969	3577	1532	836	687	682	770	1421	10430	14951	15651	8484	4796	5352
1970	2867	1146	810	757	709	722	1047	10722	17119	21142	18653	8444	7064
1971	4745	3082	2075	1319	944	867	986	3428	31031	22942	30316	13636	9657
1972	5537	2912	2313	2036	1836	1660	1566	19777	31930	21717	18654	11884	10199
1973	4639	2155	1387	1140	1129	955	987	7896	26393	17572	19478	8726	7739
1974	3491	1463	997	843	746	690	949	15005	16767	17790	15257	11370	7161
1975	3507	1619	1487	1409	1342	1272	1457	14037	30303	26188	17032	15155	9607
1976	7003	1853	1008	897	876	825	1261	11305	22814	18253	19298	6463	7706
1977	3552	2392	2148	1657	1470	1361	1510	11212	35607	21741	18371	11916	9439
1978	6936	3211	2371	1868	1525	1481	1597	11693	18417	20079	15327	8080	7765
1979	4502	2324	1549	1304	1204	1165	1403	13334	24052	27463	19107	10172	9023
1980	6900	3955	2279	1649	1383	1321	1575	11377	26255	30002	20196	12342	9994
1981	7335	3382	1841	1958	1839	1470	1898	15789	18387	31679	35256	13033	11254
1982	6384	3270	2207	2086	1559	1094	1574	12490	24439	22877	14536	16427	9115
1983	6272	2454	2192	2098	1858	1596	1781	13777	22789	20295	23203	12731	9307
MAX	7518	3955	2905	2212	1858	1779	2405	19777	47816	32388	35256	19799	11254
MIN	2867	1146	810	687	682	664	697	3428	14710	15651	8484	4796	5352
MEAN	5363	2402	1703	1429	1216	1086	1340	12462	26043	23075	20654	12555	9159

Table F.2.2.3 GOLD CREEK NATURAL MONTHLY FLOWS (CFS) 1/

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
1951	3848	1300	1100	960	820	740	1617	14090	20790	22570	19670	21240	9106
1952	5571	2744	1900	1600	1000	880	920	5419	32370	26390	20920	14480	9552
1953	8202	3497	1700	1100	820	820	1615	19270	27320	20200	20610	15270	10090
1954	5604	2100	1500	1300	1000	780	1235	17280	25250	20360	26100	12920	9682
1955	5370	2760	2045	1794	1400	1100	1200	9319	29860	27560	25750	14290	10256
1956	4951	1900	1300	980	970	940	950	17660	33340	31090	24530	18330	11473
1957	5806	3050	2142	1700	1500	1200	1200	13750	30160	23310	20540	19800	10384
1958	8212	3954	3264	1965	1307	1148	1533	12900	25700	22880	22540	7550	9476
1959	4811	2150	1513	1448	1307	980	1250	15990	23320	25000	31180	16920	10560
1960	6558	2850	2200	1845	1452	1197	1300	15780	15530	22980	23590	20510	9712
1961	7794	3000	2694	2452	1754	1810	2650	17360	29450	24570	22100	13370	10809
1962	5916	2700	2100	1900	1500	1400	1700	12590	43270	25850	23550	15890	11565
1963	6723	2800	2000	1600	1500	1000	830	19030	26000	34400	23670	12320	11073
1964	6449	2250	1494	1048	966	713	745	4307	50580	22950	16440	9571	9800
1965	6291	2799	1211	960	860	900	1360	12990	25720	27840	21120	19350	10169
1966	7205	2098	1631	1400	1300	1300	1775	9645	32950	19860	21830	11750	9432
1967	4163	1600	1500	1500	1400	1200	1167	15480	29510	26800	32620	16870	11219
1968	4900	2353	2055	1981	1900	1900	1910	16180	31550	26420	17170	8816	9811
1969	3822	1630	882	724	723	816	1510	11050	15500	16100	8879	5093	5596
1970	3124	1215	866	824	768	776	1080	11380	18630	22660	19980	9121	7591
1971	5288	3407	2290	1442	1036	950	1082	3745	32930	23950	31910	14440	10251
1972	5847	3093	2510	2239	2028	1823	1710	21890	34430	22770	19290	12400	10886
1973	4826	2253	1465	1200	1200	1000	1027	8235	27800	18250	20290	9074	8086
1974	3733	1523	1034	874	777	724	992	16180	17870	18800	16220	12250	7631
1975	3739	1700	1603	1516	1471	1400	1593	15350	32310	27720	18090	16310	10275
1976	7739	1993	1081	974	950	900	1373	12620	24380	18940	19800	6881	8189
1977	3874	2650	2403	1829	1618	1500	1680	12680	37970	22870	19240	12640	10109
1978	7571	3525	2589	2029	1668	1605	1702	11950	19050	21020	16390	8607	8195
1979	4907	2535	1681	1397	1286	1200	1450	13870	24690	28880	20460	10770	9489
1980	7311	4192	2416	1748	1466	1400	1670	12060	29080	32660	20960	13280	10748
1981	7725	3569	1915	2013	1975	1585	2040	16550	19300	33940	37870	13790	11961
1982 2/	7463	3613	2397	2300	1739	1203	1783	13380	26100	24120	15270	17780	9800
1983 2/	6892	2633	2358	2265	1996	1690	1900	14950	24510	21150	24500	13590	9926
MAX	8212	4192	3264	2452	2028	1900	2650	21890	50580	34400	37870	21240	11961
MIN	3124	1215	866	724	723	713	745	3745	15500	16100	8879	5093	5596
MEAN	5825	2589	1844	1543	1317	1169	1441	13483	27795	24390	21911	13493	9785

1/ The USGS gage at Gold Creek has been operational since October 1949. Reservoir operation simulations were made for the 34 calendar years beginning January 1950, and ending December 1983. Hydrologic statistics used in comparing natural and with-project conditions are based on the 33 complete water years (standard statistical year) beginning October 1, 1950 and ending September 30, 1983.

2/ Provisional data were used for water years 1982 and 1983. Final date published for these years are identical to provisional data except in the November 1981 to March 1982 period. The final published values (USGS 1983) for these months are: November, 3,260; December, 1,877; January, 1,681; February, 1,486; March, 1,347. These differences amount to approximately 1 percent of the yearly flow and do not affect the validity of the results.

TABLE F.2.3.1: TYPICAL NOAA CLIMATE DATA RECORD

Meteorological Data For 1976

Station SUMMIT, ALASKA #26414 SUMMIT AIRPORT Standard time used: ALASKAN Latitude: 63º 20' N Longitude: 149º 08' W Elevation (ground): 2397 feet Year: 1976

Month	Temperature ºF – Averages – Daily maximum	Temperature ºF – Averages – Daily minimum	Temperature ºF – Averages – Monthly	Temperature ºF – Extremes – Highest	Temperature ºF – Extremes – Date	Temperature ºF – Extremes – Lowest	Temperature ºF – Extremes – Date	Degree Days Base 65ºF – Heating	Degree Days Base 65ºF – Cooling	Precipitation in inches – Water equivalent – Total	Precipitation in inches – Water equivalent – Greatest in 24 hrs.	Precipitation in inches – Water equivalent – Date	Precipitation in inches – Snow, Ice pellets – Total	Precipitation in inches – Snow, Ice pellets – Greatest in 24 hrs.	Precipitation in inches – Snow, Ice pellets – Date	Relative humidity pct. – Hour 02 (local time)	Relative humidity pct. – Hour 08 (local time)	Relative humidity pct. – Hour 14 (local time)	Relative humidity pct. – Hour 20 (local time)
JAN	9.0	-3.8	2.6	34	30	-26	9	1931	0	2.17	1.15	18-19	49.7	21.5	18-19	67	70	73	71
FEB	4.2	-10.4	-3.1	33	5	-28	11	1975	0	1.11	0.50	4	19.6	8.7	5-6		65	65	68
MAR	18.2	2.2	10.2	30	6	-14	15	1696	0	1.65	0.45	3-4	41.1	8.7	3		75	67	
APR	36.3	14.3	23.4	51	30	-3	13	1180	0	0.14	0.08	26	5.8	3.1	26		68		
MAY	43.6	29.4	36.5	54	2	17	7	878	0	2.90	1.90	8	8.7	2.6	8		69		
JUN	60.6	40.9	30.8	74	27	34	8	420	0	0.51	0.30	30	0.0	0.0			69		
JUL	62.1	43.6	52.9	76	23	33	6	368	0	1.05	0.33	23	0.0	0.0			81		
AUG	62.8	41.8	52.3	78	2	31	29	383	0	0.96	0.20	7	0.0	0.0			80		
SEP	49.8	31.7	40.8	59	14	16	30	718	0	1.59	0.48	9	0.4	0.3	20		76		
OCT																			
YEAR																			

Month	Wind – Resultant – Direction	Wind – Resultant – Speed mph	Wind – Average speed mph	Wind – Fastest mile – Speed mph	Wind – Fastest mile – Direction	Wind – Fastest mile – Date	Percent of possible sunshine	Average sky cover, tenths sunrise to sunset	Number of Days – Sunrise to Sunset – Clear	Number of Days – Sunrise to Sunset – Partly Clear	Number of Days – Sunrise to Sunset – Cloudy	Number of Days – Precipitation .01 inch or more	Number of Days – Snow, Ice pellets 1.0 inch or more	Number of Days – Thunderstorms	Number of Days – Heavy fog visibility 1/4 mile or loss	Number of Days – Temperature ºF – Maximum – 90º and above (b)	Number of Days – Temperature ºF – Maximum – 32º and below	Number of Days – Temperature ºF – Minimum – 32º and below	Number of Days – Temperature ºF – Minimum – 0º and below	Average station pressure mb – Elev. 2405 feet m.s.l.
JAN				28	23	30		6.0	11	4	16	12	7	0	2	0	29	31	20	
FEB				31	07	23		3.9	17	4	8	7	6	0	0	0	27	29	24	
MAR				35	07	17		8.0	4	4	23	11	8	0	0	0	31	31	15	
APR				20	08	14		6.2	8	8	14	3	2	0	0	0	8	30	2	
MAY				17	24	18		7.5	5	6	20	7	4	0	0	0	0	27	0	
JUN				18	22	17		6.9	6	8	16	4	0	0	0	3	0	0	0	
JUL				29	23	27		8.1	3	7	21	14	0	0	1	4	0	0	0	
AUG				20	26	7						13	0			5	0	1	0	
SEP				25	25	19		7.0	3	9	18	13	0	0	2	0	0	17	0	
OCT				20	08	12														
YEAR																				

Table F.2.3.1 (Page 2 of 2)

Normals, Means, And Extremes - through 1975#

Month	Temperature °F: Averages: Daily maximum	Averages: Daily minimum	Averages: Monthly	Extremes: Record Highest	Year	Extremes: Record Lowest	Year	Normal Degree Days Base 65°F: Heating	Cooling	Precipitation in inches: Water equivalent: Normal	Maximum monthly	Year	Minimum monthly	Year	$ Maximum in 24 hrs.	Year	Snow, Ice pallets: Maximum monthly	Year	Maximum in 24 hrs.	Year	Relative humidity pct.: Hour 02 (local time)	Hour 08	Hour 14	Hour 20	Wind: $ Mean speed mph @	Prevailing direction	Fastest mile: Speed mph	Direction	Year	Pct. of possible sunshine	Mean sky cover, tenths sunrise to sunset	Mean Number of Days: Sunrise to Sunset: Clear	Partly Cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog visibility 1/4 mile or less	Temperatures °F: Maximum: 90° (b) and above	Maximum: 32° and below	Minimum: 32° and below	Minimum: 0° and below	Average station pressure mb (Elev. 2405 feet m.s.l.)
(a)				35		35					35		35		35		34		35		5	7	7	6	8	5	7	7			7	7	7	7	20	8	8	8	34	34	34	34	2
JAN	7.9	-4.8	1.6	44	1945	-45	1971	1965	0	0.91	3.38	1948	0.09	1945	0.80	1948	64.8	1948	16.3	1973	68	68	69	68	15.1	NE	44	05	1968		5.2	13	5	13	9	4	0	*	0	30	31	20	921.4
FEB	13.5	-.4	6.6	45	1942	-45	1947	1635	0	1.23	4.31	1951	T	1950	2.79	1951	44.5	1951	28.0	1964	76	75	75	76	11.9	NE	46	07	1974		7.0	6	5	17	10	5	0	1	0	26	28	15	918.8
MAR	19.4	3.0	11.2	49	1961	-35	1971	1668	0	1.04	4.53	1946	0.07	1961	1.67	1946	59.1	1946	18.1	1946	76	76	70	73	11.1	NE	48	10	1971		6.2	9	6	16	10	5	0	1	0	27	31	14	917.2
APR	32.9	14.1	23.5	57	1956	-30	1944	1245	0	0.67	4.45	1966	0.06	1944	0.97	1963	28.7	1970	9.7	1963	80	75	65	75	7.6	NE	33	08	1971		7.2	5	7	18	7	4	0	1	0	13	30	3	922.9
MAY	45.7	29.1	37.4	76	1960	-14	1945	856	0	0.77	2.66	1966	0.04	1949	0.96	1946	17.4	1958	7.5	1946	83	70	58	67	7.7	W	28	07	1969		7.5	3	9	19	7	2	*	1	*	1	22	*	923.1
JUN	58.0	39.9	49.0	89	1961	25	1947	480	0	2.19	4.45	1949	0.41	1942	2.22	1967	9.4	1974	8.7	1974	84	73	57	65	8.3	SW	29	22	1970		8.2	2	6	22	12	1	2	1	3	0	2	0	924.7
JUL	60.2	43.8	52.0	81	1961	32	1970	403	0	3.09	5.56	1959	1.17	1955	1.95	1948	9.7	1970	9.7	1970	89	78	62	72	7.8	SW	30	23	1974		8.2	2	7	22	16	*	2	1	5	0	*	0	929.1
AUG	56.0	41.1	48.6	81	1968	20	1955	508	0	3.30	6.33	1955	0.70	1941	2.10	1944	9.0	1955	6.0	1955	88	81	62	76	7.4	SW	31	22	1975		8.3	2	6	23	18	0	*	1	1	0	2	0	930.3
SEP	47.1	32.6	39.9	75	1957	6	1956	753	0	2.81	6.13	1965	0.29	1969	2.07	1944	21.5	1958	14.0	1955	85	81	59	75	7.5	NE	32	23	1972		7.4	5	5	20	16	2	*	1	*	1	14	0	926.1
OCT	30.4	17.5	24.0	59	1969	-15	1975	1271	0	1.62	3.79	1952	0.12	1967	1.24	1963	54.8	1970	12.6	1970	83	85	76	81	8.0	NE	35	23	1970		7.6	3	5	21	13	7	0	2	0	18	30	2	916.7
NOV	15.7	3.7	9.7	44	1962	-29	1948	1659	0	1.23	4.85	1952	0.06	1963	1.30	1964	75.1	1967	21.9	1970	79	79	78	79	11.7	NE	39	25	1970		7.1	7	4	19	9	5	0	1	0	27	30	13	921.3
DEC	9.2	-3.6	2.9	42	1969	-43	1961	1925	0	1.20	4.63	1951	0.24	1945	1.09	1967	50.7	1970	27.4	1970	76	78	76	77	11.3	NE	44	11	1970		6.5	9	5	17	11	6	0	1	0	30	31	19	914.7
YEAR	33.0	18.0	25.5	89	JUN 1961	-45	JAN 1971	14368	0	20.06	6.74	AUG 1944	T	FEB 1950	2.79	FEB 1951	75.1	NOV 1967	28.0	FEB 1964	81	76	67	74	9.7	NE	48	10	MAR 1971		7.2	68	70	227	138	41	5	12	9	173	251	86	922.0

NOTE: Due to less than full time operation on a variable schedule, manually recorded elements are from broken sequences in incomplete records. Daily temperature extremes and precipitation totals for portions of the record may be for other than a calendar day. The period of record for some elements is for other than consecutive years.

(a) Length of record, years, through the current year unless otherwise noted, based on January data.
(b) 70° and above at Alaskan stations.
* Less than one half.
T Trace.

NORMALS - Based on record for the 1941-1970 period.
DATE OF AN EXTREME - The most recent in cases of multiple occurrence.
PREVAILING WIND DIRECTION - Record through 1963.
WIND DIRECTION - Numerals indicate tens of degrees clockwise from true north. 00 indicates calm.
FASTEST MILE WIND - Speed is fastest observed 1-minute value when the direction is in tens of degrees

$ For calendar day prior to 1968.
@ For the period 1950-1954 and January 1968 to date when available for full year.
| For the period 1942-1953 and January 1968 to date when available for full year

Data for this station not available for archiving nor publication of summary effective October 1976.

TABLE F.2.3.2: SUMMARY OF CLIMATOLOGICAL DATA

STATION	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
	Mean Monthly Precipitation (Inches)												
Anchorage	0.84	0.56	0.56	0.56	0.59	1.07	2.07	2.32	2.37	1.43	1.02	1.07	
Big Delta	0.36	0.27	0.33	0.31	0.94	2.20	2.49	1.92	1.23	0.56	0.41	0.42	11.44
Fairbanks	0.60	0.53	0.48	0.33	0.65	1.42	1.90	2.19	1.08	0.73	0.66	0.65	11.22
Gulkana	0.58	0.47	0.34	0.22	0.63	1.34	1.84	1.58	1.72	0.88	0.75	0.76	11.11
Matanuska Agr. Exp. Station	0.79	0.63	0.52	0.62	0.75	1.61	2.40	2.62	2.31	1.39	0.93	0.93	15.49
McKinley Park	0.68	0.61	0.60	0.38	0.82	2.51	3.25	2.48	1.43	0.42	0.90	0.96	15.54
Summit WSO	0.89	1.19	0.86	0.72	0.60	2.18	2.97	3.09	2.56	1.57	1.29	1.11	19.3
Talkeetna	1.63	1.79	1.54	1.12	1.46	2.17	3.48	4.89	4.52	2.54	1.79	1.71	28.64

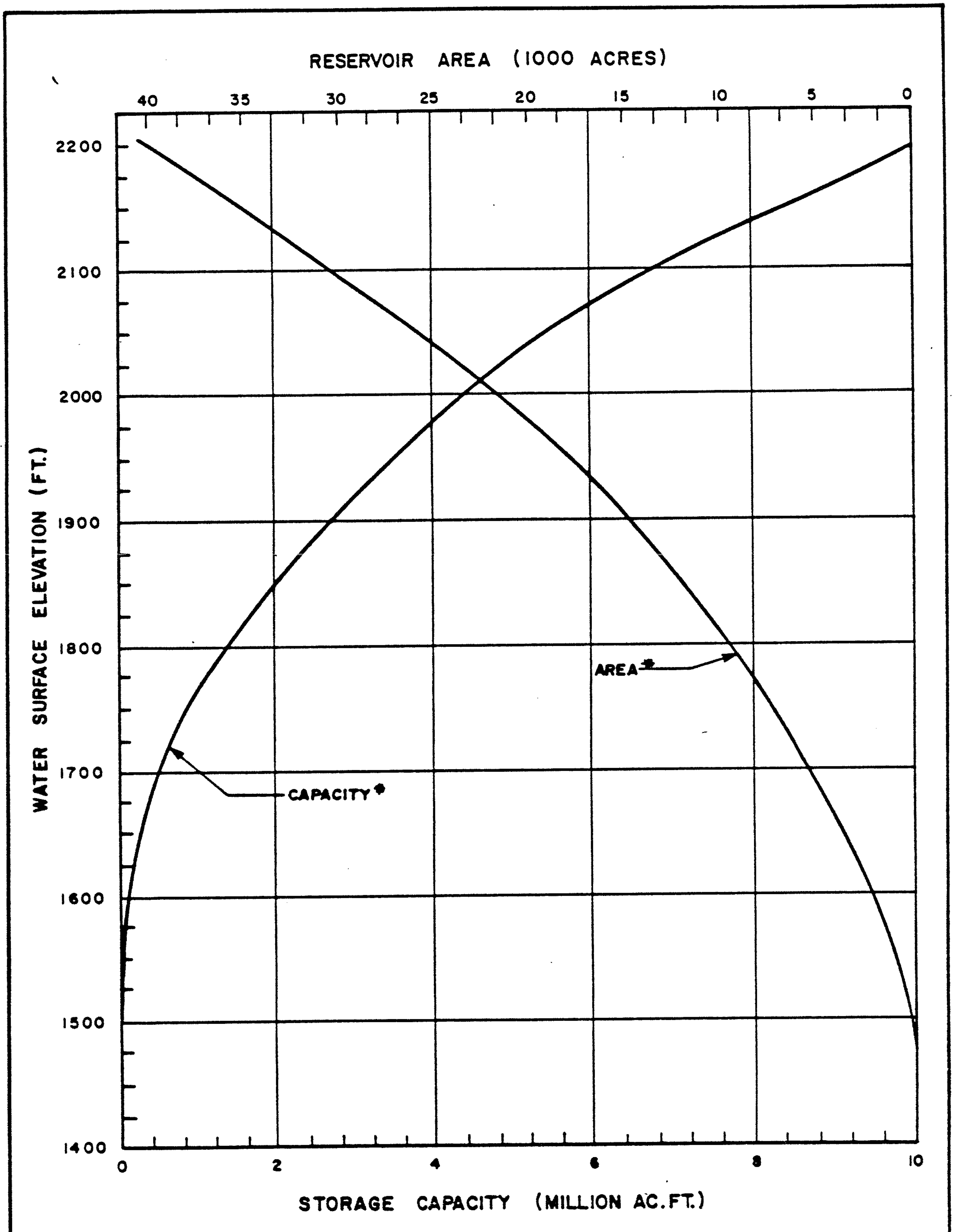
MEAN MONTHLY TEMPERATURES (°F)

STATION	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
Anchorage	11.8	17.8	23.7	35.3	46.2	54.6	47.9	55.9	48.1	34.8	21.1	13.0	
Big Delta	- 4.9	4.3	12.3	29.4	46.3	57.1	59.4	54.8	43.6	25.2	6.9	- 4.2	27.5
Fairbanks	11.9	- 2.5	9.5	28.9	47.3	59.0	60.7	55.4	44.4	25.2	2.8	-10.4	25.7
Gulkana	- 7.3	3.9	14.5	30.2	43.8	54.2	56.9	53.2	43.6	26.8	6.1	- 5.1	26.8
Matanuska Agr. Exp. Station	9.9	17.8	23.6	36.2	46.8	54.8	57.8	55.3	47.6	33.8	20.3	12.5	34.7
McKinley	- 2.7	4.8	11.5	26.4	40.8	51.5	54.2	50.2	40.8	23.0	8.9	- 0.10	25.8
Summit WSO	- 0.6	5.5	9.7	23.5	37.5	48.7	52.1	48.7	39.6	23.0	9.8	3.0	25.0
Talkeetna	9.4	15.3	20.0	32.6	44.7	55.0	57.9	54.6	46.1	32.1	17.5	9.0	32.8

TABLE F.2.3.3: RECORDED AIR TEMPERATURES AT TALKEETNA AND SUMMIT IN °F

	TALKEETNA			SUMMIT		
Month	Daily Max.	Daily Min.	Monthly Average	Daily Max.	Daily Min.	Monthly Average
Jan	19.1	- 0.4	9.4	5.7	- 6.8	- 0.6
Feb	25.8	4.7	15.3	12.5	- 1.4	5.5
Mar	32.8	7.1	20.0	18.0	1.3	9.7
Apr	44.0	21.2	32.6	32.5	14.4	23.5
May	56.1	33.2	44.7	45.6	29.3	37.5
June	65.7	44.3	55.0	52.4	39.8	48.7
Jul	67.5	48.2	57.9	60.2	43.4	52.1
Aug	64.1	45.0	54.6	56.0	41.2	48.7
Sept	55.6	36.6	46.1	46.9	32.2	39.6
Oct	40.6	23.6	32.1	29.4	16.5	23.0
Nov	26.1	8.8	17.5	15.6	4.0	9.8
Dec	18.0	- 0.1	9.0	9.2	- 3.3	3.0
Annual Average			32.8			25.0

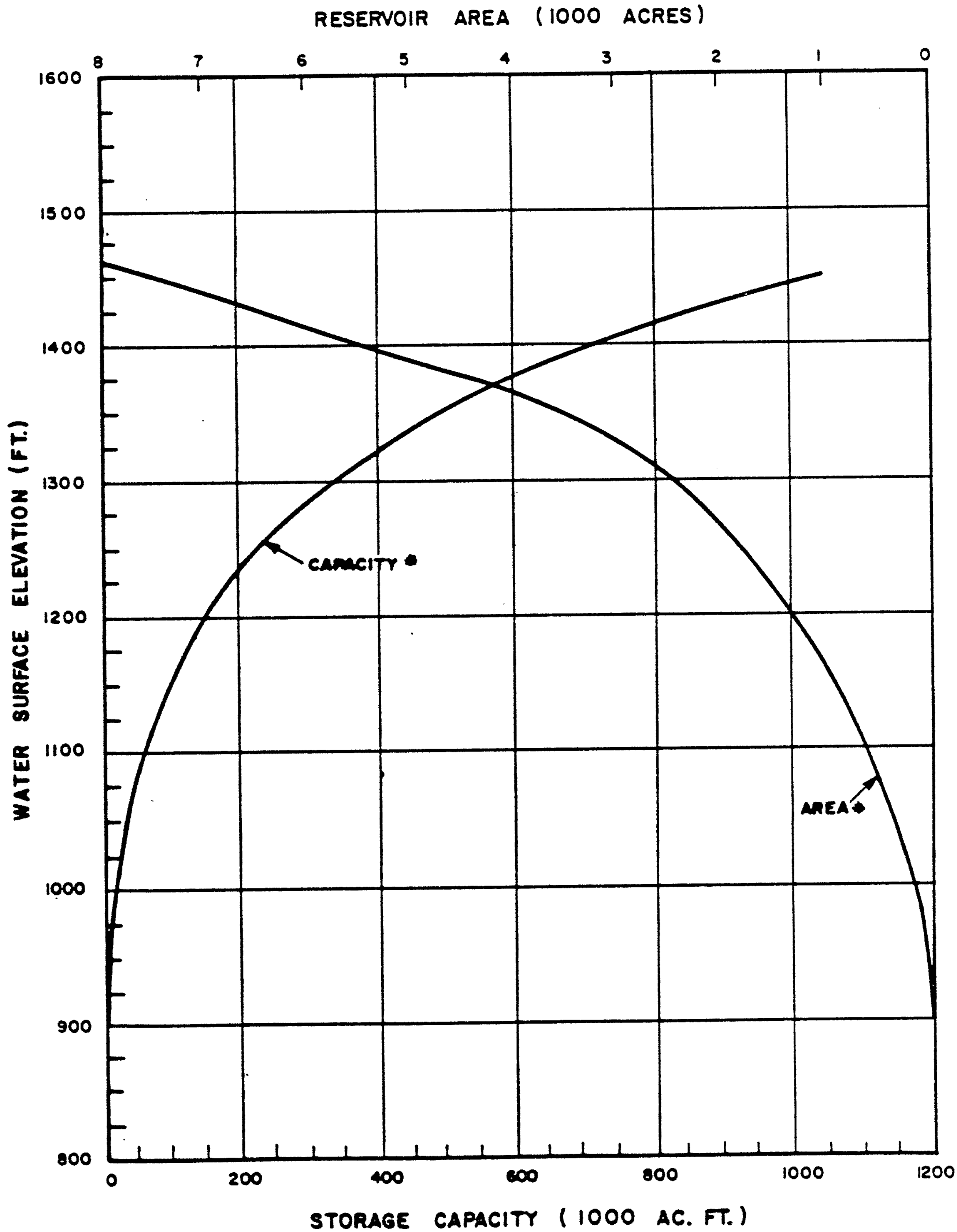
FIGURES



AREA AND CAPACITY CURVES
WATANA RESERVOIR

*WITHOUT RESERVOIR SILTATION

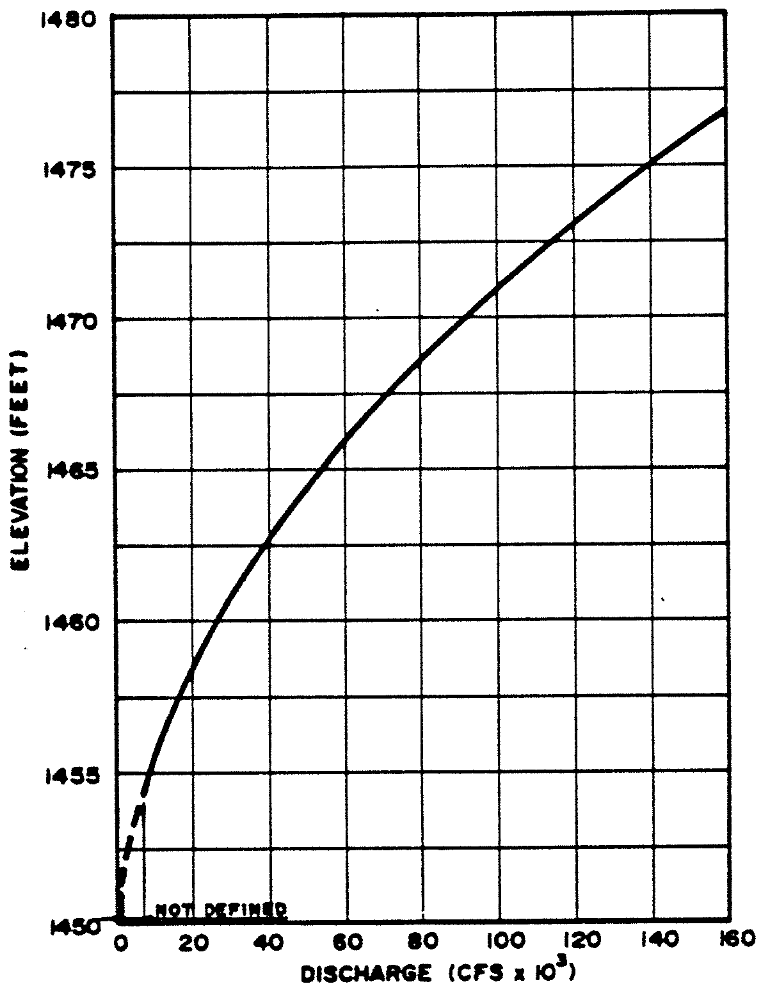
FIGURE F.2.4.1



AREA AND CAPACITY CURVES
DEVIL CANYON RESERVOIR

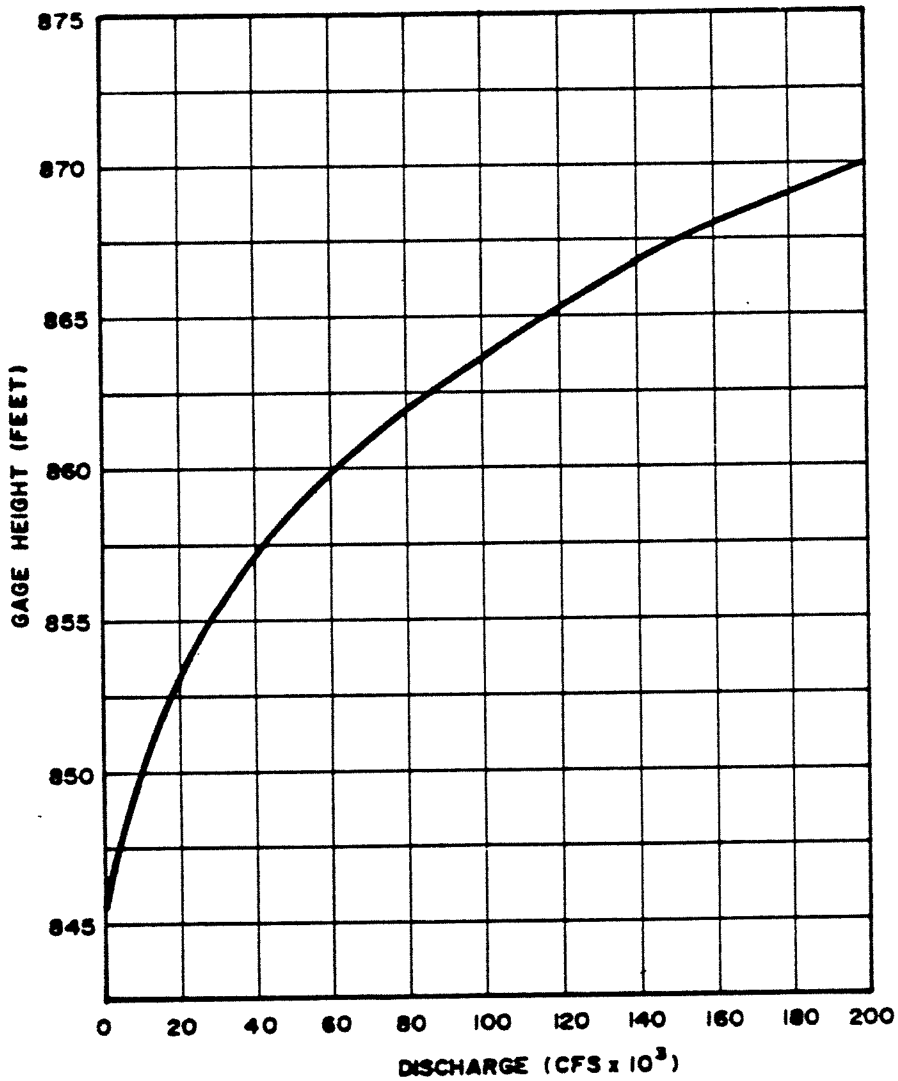
*WITHOUT RESERVOIR SILTATION

FIGURE F.2.4.2



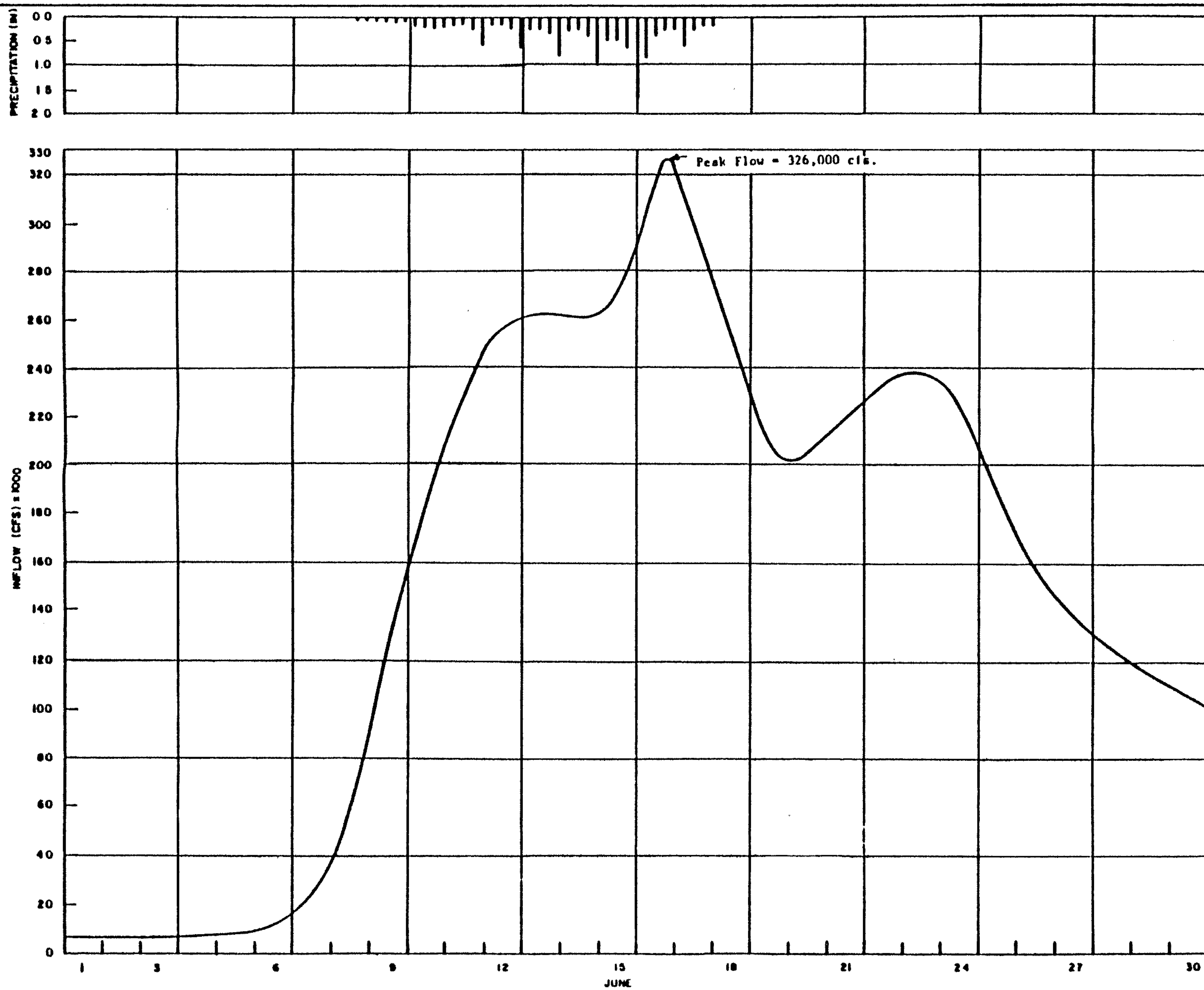
WATANA TAILWATER RATING

FIGURE F.2.5.1



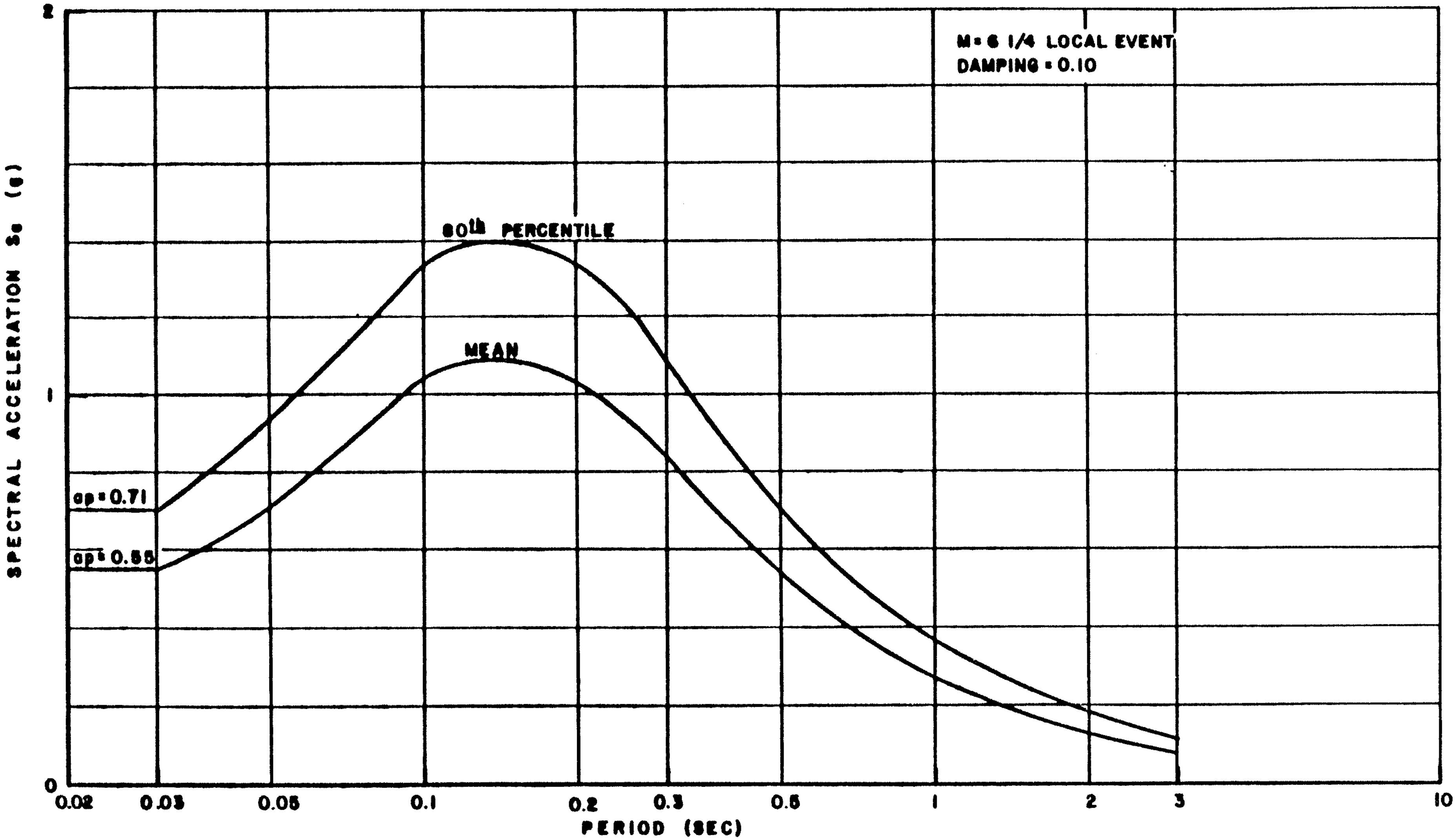
DEVIL CANYON
TAILWATER RATING
(TAILRACE TO PORTAGE CREEK)

FIGURE F.2.5.2



WATANA PMF INFLOW HYDROGRAPH

FIGURE F.2.6.1



MEAN RESPONSE SPECTRA AT THE DEVIL CANYON SITE FOR
SAFETY EVALUATION EARTHQUAKE

FIGURE F.3.2.1

APPENDIX F1

APPENDIX F1

This Appendix deleted.

APPENDIX F2

WATANA AND DEVIL CANYON EMBANKMENT STABILITY ANALYSES

APPENDIX F2
WATANA AND DEVIL CANYON EMBANKMENT STABILITY ANALYSES

1 - PRELIMINARY DESIGN (**)

1.1 - General (**)

This appendix presents the proposed embankment slope designs for Watana Stages I and III and the Devil Canyon Stage II embankments. The method of analysis and the safety factors comply with recommendations of the United States Army Corps of Engineers (COE 1982a, 1958). The stability studies have been conducted in sufficient detail to satisfy project feasibility.

Watana Dam Stages I and III have been analyzed. The cross section for analysis has been taken where it will be maximum height (700± feet for Stage I, and 885± feet for Stage III). The Devil Canyon Saddle Dam (Stage II) has not been independently evaluated because it has the same cross section and general foundation treatment as Watana. Therefore, because of the lower height of the Devil Canyon Saddle Dam (maximum 150± feet) its stability will be much less critical than for Watana, and higher stability factors of safety are to be expected.

Typical embankment cross sections for the three stages of Susitna development are presented in Figures F2.1, F2.2, and F2.3.

1.2 - Design Shear Strengths (***)

Design values are shown in the following tables below and on the individual figures. The tables are a resume of the materials which are of major influence in the stability analysis, together with their shear strengths. The design shear strengths are based primarily on interpretation of similar materials at other projects where extensive laboratory tests have been performed.

1.2.1 - Material Design Parameters (***)

(a) Impervious Core

Unit Weight (pcf)		Shear Strength
Moist,	m = 126	UU: cohesion, c = 1,500 psf Friction Angle, Ø = 0°
Saturated,	s = 130	CU: cohesion, c = 300 psf Friction Angle, Ø = 16.7°
Submerged,	sub = 67	CD: cohesion, c = 0 psf Friction Angle, Ø = 26.5°

(b) Rockfill and Filters

Unit Weight (pcf)		Shear Strength
Moist,	m = 130	UU: --
Saturated,	s = 140	CU: --
Submerged,	sub = 78	CD: cohesion, C = 0 psf Friction Angle, Ø = 38°

(c) Overburden Foundation

Unit Weight (pcf)		Shear Strength
Moist,	m = 125	CD: cohesion, C = 0 psf
Saturated,	s = 132	Friction Angle, Ø=32°
Submerged,	sub = 70	

(d) Bedrock Formation

Unit Weight (pcf)		Shear Strength
Moist,	m = 150	CD: cohesion, C = 40,000 psf
Saturated,	s = 150	
Submerged,	sub = 88	Friction Angle, Ø=38°

1.2.2 - Loading Conditions and Factors of Safety (F.S.) (***)

The following table is a summary of results from the static and earthquake (pseudo-static) stability analysis.

	Minimum Allowable FS1/		Watana - Stage I Min. Calculated FS				Watana - Stage III Min. Calculated FS			
			U/S Slope		D/S Slope		U/S Slope		D/S Slope	
Case	Static (S)	Earthquake (E)2/	S	E	S	E	S	E	S	E
End-of-Construction	1.3	1.0	1.97	1.30	1.54	1.09	1.52	1.04	1.58	1.13
Partial Pool Varying	1.5	1.0	1.84 (Critical Pool el. 1710)	1.20	--	--	1.54 (Critical Pool el. 1900)	1.05	--	--
Steady State Seepage at Normal Max. Pool	1.5	1.0	--	--	1.57	1.12	--	--	1.58	1.13
Rapid Drawdown Normal Max Pool to el. 1,800	1.0	--	1.78	--	--	--	1.26	--	--	--

1/ FS = Stability factor of safety.
2/ Seismic coefficient = 0.15.

1.3 - Method of Analysis (***)

The STABL computer program, which utilizes an adaptation of the Modified Bishop Method, was used to determine the location of critical failure surfaces for all embankment stability. Use of the STABL allowed many trial failure surfaces to be tested for both static and pseudo-static stability. The critical failure plane was found and the safety factory expressed as the ratio of available shear strength to that required for equilibrium. Circular and wedge-shaped trial failure surfaces were examined. Circular surfaces were found to yield the lower factors of safety for the downstream slope, and wedge-shaped surfaces were critical for the upstream slope because of the upstream inclination of the core. Only critical surface results are presented herein. Earthquake analyses considered a pseudo-static seismic coefficient of 0.15 (COE 1982a). As shown in Figure F2.14 the Susitna Project is located in Zone 4, which is a high risk area.

For each section analyzed, 50 randomly generated trial surfaces encompassing the entire range of potential failure surfaces were tested. The results presented in Figures F2.4 through F2.13 only show the ten most critical surfaces.

Dynamic stability was evaluated through a comparison of Watana Dam with similar dams in areas of high seismicity.

1.4 - Design Cases and Assumptions (***)

The critical conditions analyzed for failure in shear are listed in the following sections.

1.4.1 - End-of-construction Case (***)

Since placement moisture contents for the embankment are anticipated to be slightly in excess of optimum moisture, some pore pressure is likely to occur. However, for the rock shell design the inclined core is relatively narrow, thus confining the excess pore pressure to a zone just upstream of the center of the fill. The shear strength contolling the stability of the construction condition is the shear strength of the impervious core.

Both the upsteam and downstream slopes have been analyzed for slope stability immediately upon completion of construction, and prior to reservoir filling. Minimum allowable static and earthquake (pseudo-static) factors of safety of 1.3 and 1.0, respectively, have been considered. The steeper, downstream slope indicated the lower safety factor. A total stress analysis was performed. Stage I considered an unconsolidated undrained (UU) shear strength in the impervious core material, and moist unit weights throughout the embankment section. This loading condition conservatively models the embankment just at the end of the construction, when the fill has not yet had sufficient time to strengthen through the consolidation of the fill under its own weight, and the dissipation of excess pore pressures. Stage III considered consolidated drained (CD) shear strengths in the Stage I fill, and UU shear strength in the core Stage III impervious core fill. Moist unit weights were considered above the assumed elevation 1,900 reservoir level during Stage III construction, and submerged unit weights below.

The minimum post construction stability for Watana (Stages I and III) is shown in Section 1.2.2; the locations of critical failure surfaces are shown in Figures F2.4, F2.5, F2.9, and F2.10.

1.4.2 - Partial Pool Case (***)

The upstream slope was analyzed for minimum static and earthquake (pseudo-static) safety factors of 1.5 and 1.0 respectively, at the most critical reservoir pool elevations. The saturation line was assumed horizontal. Submerged weights were used below the saturation level and moist weights were used above the saturation line.

Four reservoir increments were studied for both Stage I and Stage III to determine the critical temporary reservoir level. For Stage I the temporary pool levels studied were elevations 1,600, 1,700, 1,800, and 1,900. For Stage III they were elevations 1,800, 1,900, 2,000, and 2,100. A plot of minimum factor of safety vs. pool level reveals the partial pool corresponding to the critical factor of safety.

The initial partial pool condition occurs after the end of construction when the fill is partially consolidated, but before complete reservoir filling and the establishment of steady state seepage. Construction case excess pore pressures are assumed to still be present. For Stage I consolidated undrained (CU) shear strength have been used in a total stress analysis, approximating this intermediate condition. However, Stage I fill would have completely consolidated and excess pore pressures dissipated by the time reservoir filling for Stage III begins. Therefore, Stage III analysis has considered consolidated drained (CD) shear strengths for Stage I fill (and Stage III pervious materials), and CU strengths for the Stage III impervious core.

The results of the partial pool case are summarized in Section 1.2.2. The critical pool occurs at el. 1,725 during Stage I filling, and at el. 1,900 in Stage III. The critical failure surfaces and pool determination are shown in Figures F2.6 and F2.11

1.4.3 - Steady State Seepage Case (***)

The downstream slope was analyzed for the steady seepage case. The normal maximum operative pool was selected as the most critical pool that will be maintained for a period long enough to develop steady seepage. Pools above this elevation do not remain long enough to saturate the embankment.

Steady state seepage is the long-term condition, achieved once a free-water line phreatic surface is established through the core and within the downstream filters and shell. By the time this condition takes place, all consolidation of the fill and dissipation of excess pore pressures will have occurred, and the consolidated drained (CD) strength of the fill material will govern the stability of the embankment.

The minimum long-term embankment slope stability is shown in Section 1.2.2; the locations of critical failure surfaces are shown in Figures F2.7 and F2.12. Slopes were designed for a minimum static factor of safety of 1.5, and a minimum earthquake (pseudo-static) factor of safety of 1.0.

1.4.4 - Rapid Drawdown Case (***)

The rapid drawdown analysis considered saturation of the embankment at the normal maximum operating elevation and drawdown to el. 1,800. It is assumed that the reservoir is above the normal maximum operating level for such a short time that the impervious embankment will not saturate and, therefore, sudden drawdowns from pools above this elevation are not applicable. The embankment slopes were designed for a minimum static safety factor of 1.0. The simultaneous occurrence of both an earthquake and rapid drawdown is considered highly improbable, and therefore a pseudo-static evaluation of the rapid drawdown case is not considered.

The rapid reservoir drawdown analysis applies only to the upstream embankment slope. The results of this analysis are presented in Section 1.2.2. Figures F2.8 and F2.13 show the locations of the critical failure planes.

The rapid drawdown condition has been conservatively evaluated by assuming that the reservoir can be lowered instantaneously from the maximum normal operating level to el. 1,800, which is the lowest intake level of the powerhouse intake structure. The drawdown analysis considers full consolidation of the fill at the time of drawdown, and an undrained condition in the impervious core immediately following drawdown. Hence, a consolidated undrained shear strength (CU) has been used in the total stress analysis. The weight of the core material above the lowered pool level at el. 1,800 increased from its pre-drawdown submerged unit weight, to a saturated unit weight. Hydrostatic uplift pressures along the failure surface through the core are determined from the saturated core outer surface. Because the rockfill would be free-draining, pore pressures would dissipate as the reservoir is drawn down, and an undrained condition would never be achieved.

Therefore, the drained strength (CD) for the rockfill is used in the analyses.

1.4.5 - Earthquake Case (***)

The earthquake case was checked by perfoming a pseudo-static analysis on each of the critical static analysis failure planes for the above cases, except sudden drawdown. This seismic analysis involved application of an additional horizontal force, acting in the direction of sliding of the potential failure mass. This force is equal to the total weight of the sliding mass times the seismic coefficient 0.15.

1.5 - Dynamic Stability Evaluation (***)

The dynamic stability was evaluated by comparing Watana Dam with similar dams located in areas of high seismicity. Dynamic analyses will be performed during final design. The performance and/or the results of dynamic analysis of the dam are summarized below for comparison with Watana Dam.

1.5.1 - Oroville Dam (***)

Oroville Dam (Seed 1979; Banerjee et al. 1979; State of California 1979). 1975 Earthquake; magnitude 5.7; epicentral distance 7.5 miles; focal depth 5.0 miles; a_{max} at dam crest = 0.13 g.

(a) Pertinent Data, and Observations at the Time of the Event (***)

The dam cross section has a slightly inclined impervious core, and shells of well-graded cobble, gravel and sand fill.

Height - 750 feet
Upstream Slopes - 2.2H:1V, 2.6H:1V and 2.75 H.1V
Downstream Slope = 2H:1V
Performance - No damage
Vertical Movement of the Crest = 0.03 feet
Horizontal Movement of Upstream Slope = 0.05 feet
Pore pressure increased in the core, and in an area within the upstream transition zone.

(b) Dynamic Re-evaluation, 1979 (***)

Dynamic analyses was performed to re-evaluate the dam for a near source maximum earthquake of magnitude 6.5 and a_{max} = 0.6 g.

The analyses indicates that in spite of areas of high pore pressure in the upstream shell, and the potential horizontal displacement of the dam of about 3 feet, the dam would be amply safe. There would be some likelihood of surface sloughing or insignificant movement along slopes at shallow depths near the crest. The minimum factor of safety with the high pore pressures would be reduced to 1.4 from 3.1 for normal operating conditions.

(c) Hypothetical Extreme Earthquake, Magnitude 8.25 (***)

This hypothetical study was made for the purpose of developing a better understanding of the performance of high embankment dams located near an epicentral region of great earthquakes. The results of the study indicate:

- The relatively high pore pressure zone in the upstream shell spreads over a significantly larger area within the upstream shell when compared with the similar area developed after a magnitude 6.5 earthquake.

- The minimum factor of safety with high pore pressure development reduced to 1.12 for the critical circle immediately after an earthquake of magnitude 8.25. The dam is dynamically stable and would not develop any massive slide in the upstream slope. The minimum factor of safety of 1.12 would be of a transient nature. The pore water pressure will dissipate in time and the dam will regain its pre-earthquake strength and stability factor of safety.

- The maximum horizontal displacements of the upstream slope after an earthquake of magnitude 8.25 would be in the order of 8 ft. The increase in strength caused by aging would reduce it to half the computed amount.

The conclusion was that a high dam, well-designed and built with suitable materials like Oroville Dam, would be able to safely withstand a near, extreme earthquake of 8.25 without significant damage, or danger of reservoir release.

1.5.2 - Miboro Dam (***)

Miboro Dam, Japan (Seed et al., 1977)
Kita-Muto Earthquake, 1961; Magnitude 7;
a = 0.1 g to 0.25 g at 20 km from epicenter.
a_{max} = 0.6 g at 10 km.

Dam Type - Rockfill
Height - 420 feet
Slopes - Upstream 2.5H:1V
Effect - No Damage
Settlement 1.2 inches
Horizontal Displacement 2.0 inches

1.5.3 - Cogoti Dam (***)

Cogoti Dam, Chile (Seed et al. 1977)
Chile Earthquake, 1943; Magnitude 8.3;
a max = 0.25 g to 0.5 g
Dam Type - Dumped rockfill with upstream concrete
Height - 275 feet
Effect - Crest settled 15 inches; minor rockslides on the 1.8H:1V; insignificant damage.

1.5.4 - La Honda Dam (***)

La Honda Dam, Venezuela (Kleiner et al. 1983) Dynamic stability analysis was performed, based on earthquake magnitude 8.25 occurring on Bocono Fault 12.4 miles from the dam site.

$$a_{max} = 0.50g$$

The embankment has an impervious central core of clayey sand, and shells of crushed sandstone.

Height ‾ 460 feet (140 meters)
Upstream slopes ‾ 3H:1V and 2.5H:1V
Downstream slope - 2.25H:1V
Result of Analysis: The dam will be safe with only insignificant damage. Small zones in the upstream shell indicate strain potential exceeding 5 percent. Vertical settlement of the crest would be on the order of 8.2 feet. Shallow sloughing of the upstream slope would likely occur.

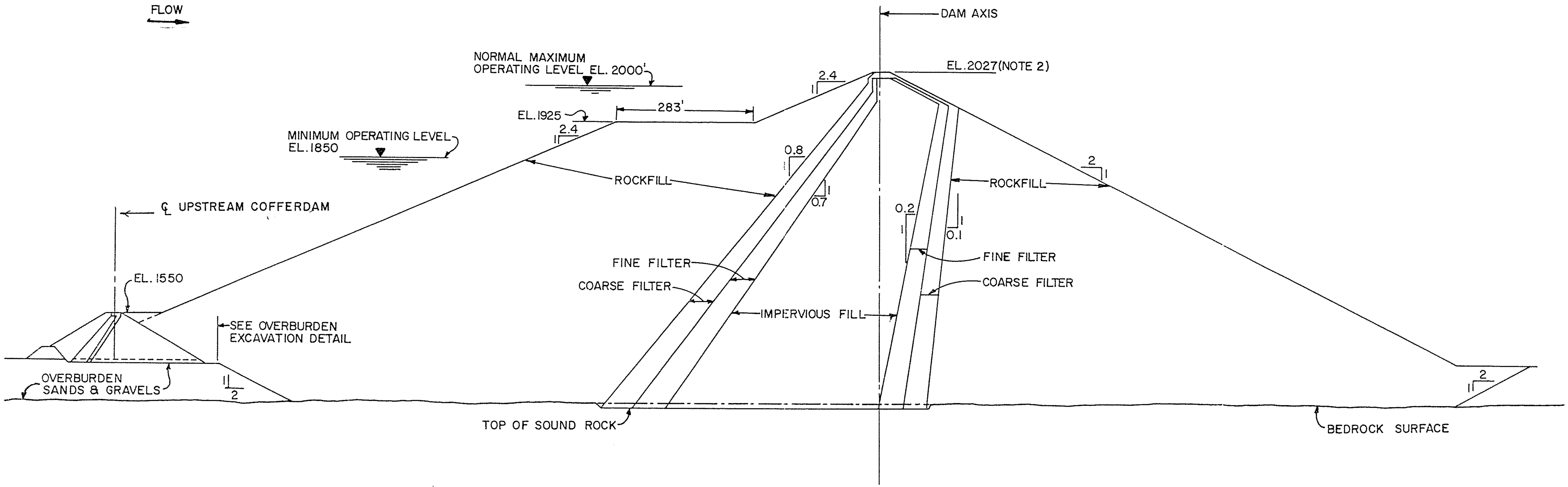
1.5.5 - Watana Dam (***)

Watana Dam is quite similar to the dams listed above, especially Oroville Dam. However, the shells of Watana would be constructed of rockfill, while the shells of Orovill were constructed of sand and gravel. The free-draining rockfill shells at Watana will tend to dissipate pore pressure more readily. However, settlements within the rockfill during strong ground motion would tend to be higher than in the sand and gravel of Oroville. These factors are somewhat compensating. Permanent deformations at the crest of Watana are anticipated to be of a similar magnitude as the deformations at Oroville Dam. Judging from the performance of Oroville Dam during the 1975 magnitude 5.7 earthquake, and subsequent dynamic stability analyses with magnitude 6.5 and extreme severe earthquake magnitude 8.25, Watana will be safe under strong seismic conditions.

1.6 - Conclusion (**)

The analyses indicate stable slopes under all loading conditions for Watana Stage I and Watana Stage III. Because of its lower height and identical cross section and foundation, the Devil Canyon Saddle Dam Stage II intuitively would also be stable under all loading condition.

FIGURES



NOTES:

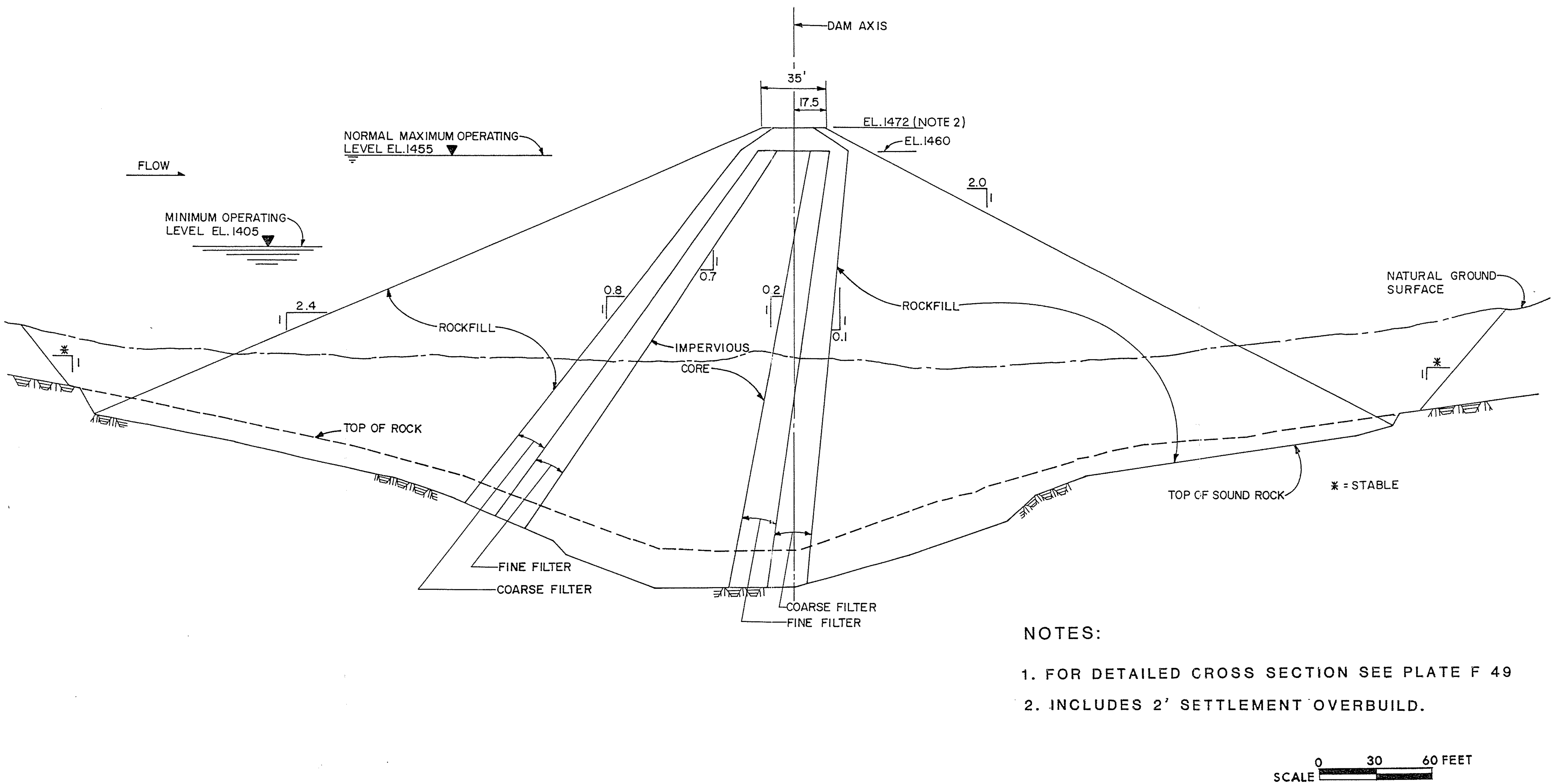
1. FOR DETAILED CROSS SECTION SEE PLATE F 7
2. INCLUDES 2' SETTLEMENT OVERBUILD

SCALE 0 100 200 FEET

WATANA DAM - STAGE I

AT MAXIMUM HEIGHT

FIGURE F2.1



NOTES:

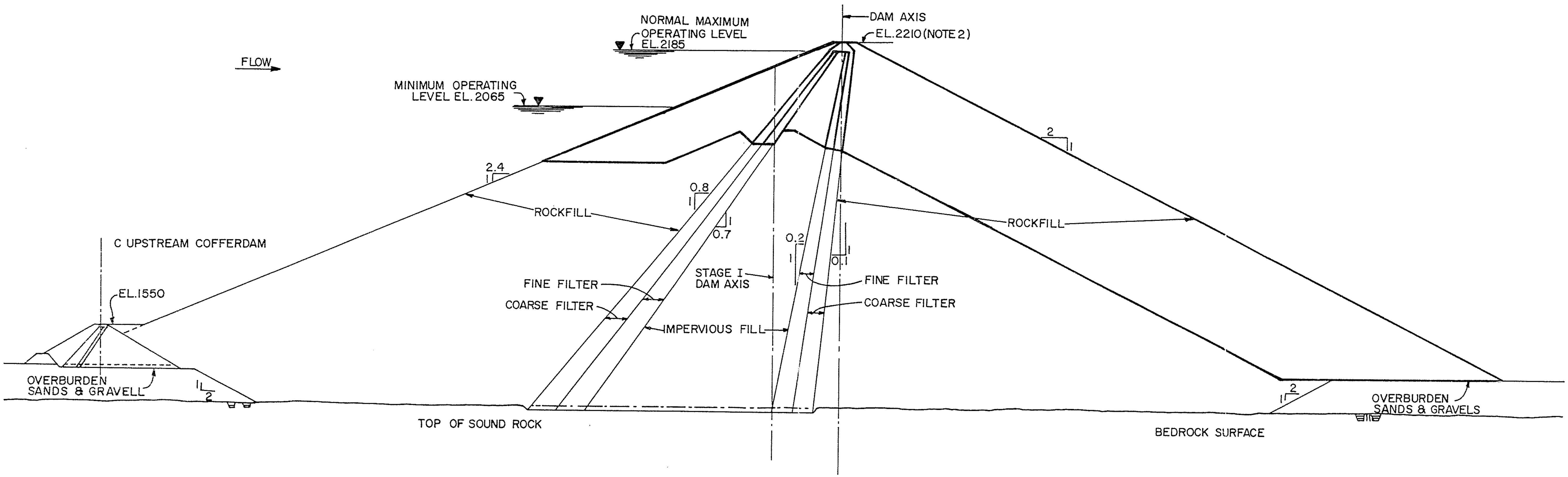
1. FOR DETAILED CROSS SECTION SEE PLATE F 49
2. INCLUDES 2' SETTLEMENT OVERBUILD.

DEVIL CANYON - STAGE II

SECTION THROUGH SADDLE DAM

AT MAXIMUM HEIGHT

FIGURE F2.2



NOTES:

1. FOR DETAILED CROSS SECTION SEE PLATE F 77
2. INCLUDES 5' SETTLEMENT OVERBUILD
3. STAGE III SHOWN WITH BOLD OUTLINE

SCALE 0 100 200 FEET

WATANA DAM - STAGE III

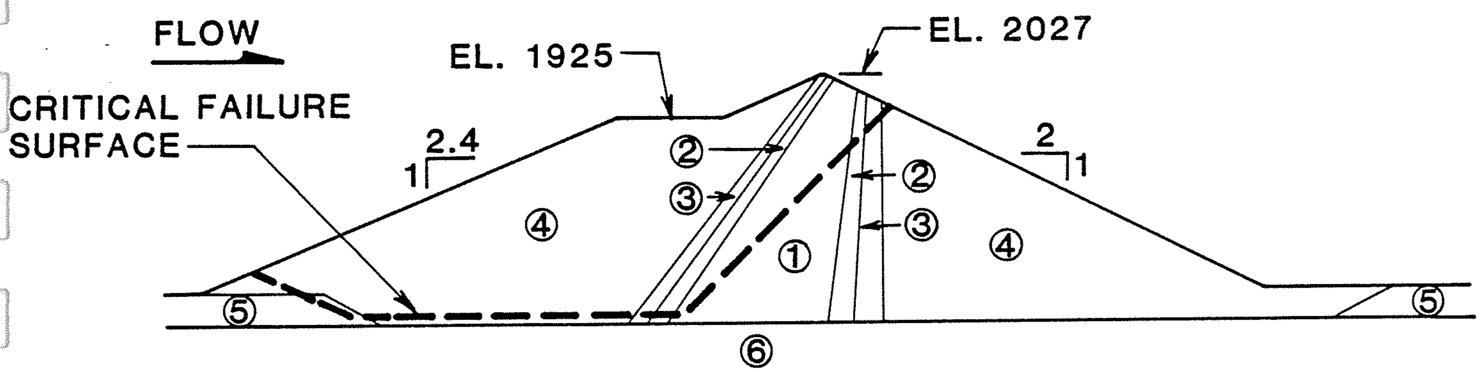
AT MAXIMUM HEIGHT

FIGURE F2.3

WATANA-STAGE I

SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.3	2.0
EARTHQUAKE (Seismic Coefficient=0.15)	1.0	1.3



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	UU: C=1,500 psf, $\phi=0^{\circ}$
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, $\phi=38^{\circ}$
OVERBURDEN FDN.	⑤	CD: C=0 psf, $\phi=32^{\circ}$
BEDROCK FDN.	⑥	CD: C= 40,000 psf, $\phi=38^{\circ}$

NOTE

MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

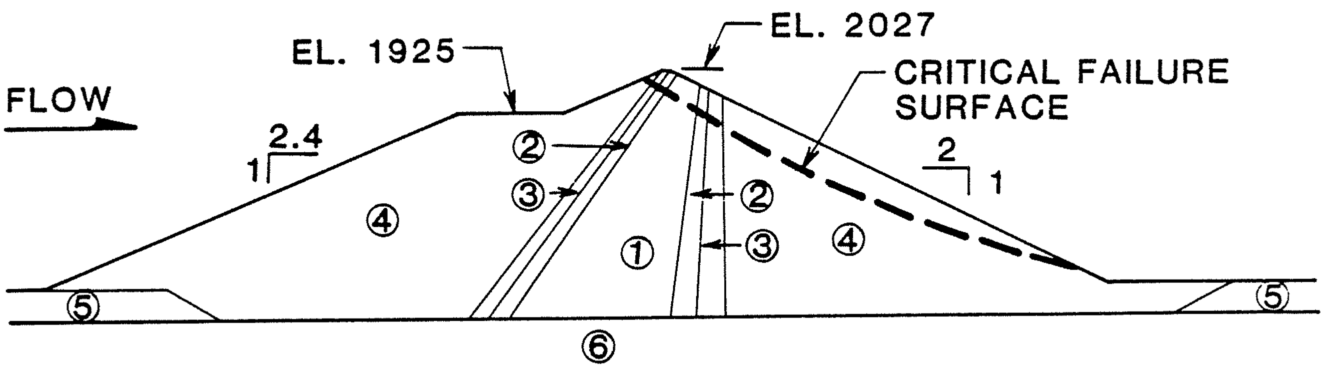
END-OF-CONSTRUCTION CASE
(UPSTREAM SLOPE)

FIGURE F2.

WATANA-STAGE I

SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.3	1.5
EARTHQUAKE (Seismic Coefficient=0.15)	1.0	1.1



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	UU: C=1,500 psf, $\phi=0^{\circ}$
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, $\phi=38^{\circ}$
OVERBURDEN FDN.	⑤	CD: C=0 psf, $\phi=32^{\circ}$
BEDROCK FDN.	⑥	CD: C= 40,000 psf, $\phi=38^{\circ}$

NOTE

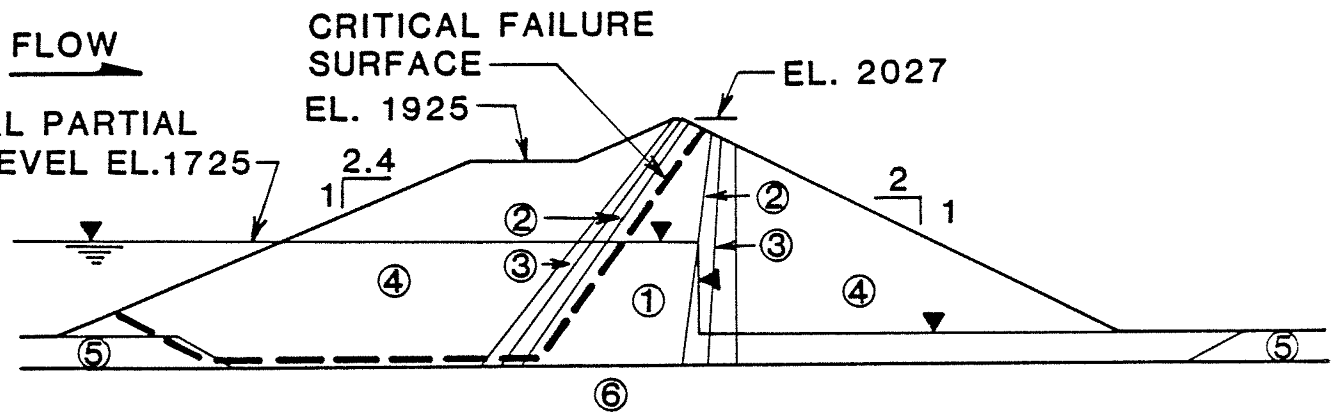
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

END-OF-CONSTRUCTION CASE
(DOWNSTREAM SLOPE)

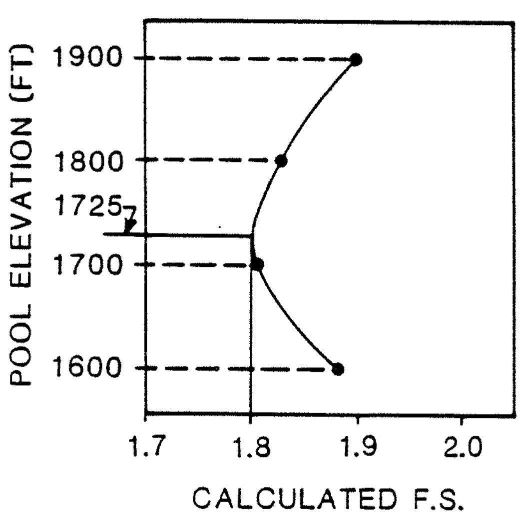
FIGURE F2.

WATANA-STAGE I
SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.5	1.8
EARTHQUAKE (Seismic Coefficient=0.15)	1.0	1.2



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	CU: C= 300 psf, Ø=16.7
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, Ø=38°
OVERBURDEN FDN.	⑤	CD: C=0 psf, Ø=32°
BEDROCK FDN.	⑥	CD: C= 40,000 psf, Ø=38°



NOTE

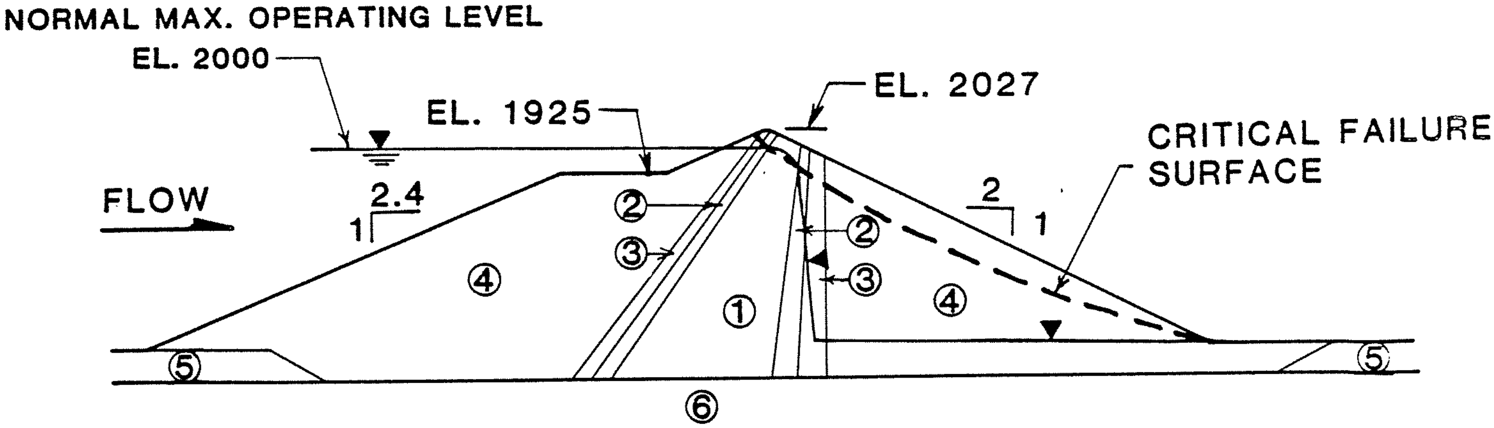
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

PARTIAL POOL CASE
(UPSTREAM SLOPE)

FIGURE 5

WATANA-STAGE I
SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.5	1.6
EARTHQUAKE (Seismic Coefficient=0.15)	1.0	1.1



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	CD: C=0 psf, ϕ=26.5°
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, ϕ=38°
OVERBURDEN FDN.	⑤	CD: C=0 psf, ϕ=32°
BEDROCK FDN.	⑥	CD: C=40,000 psf, ϕ=38°

NOTE

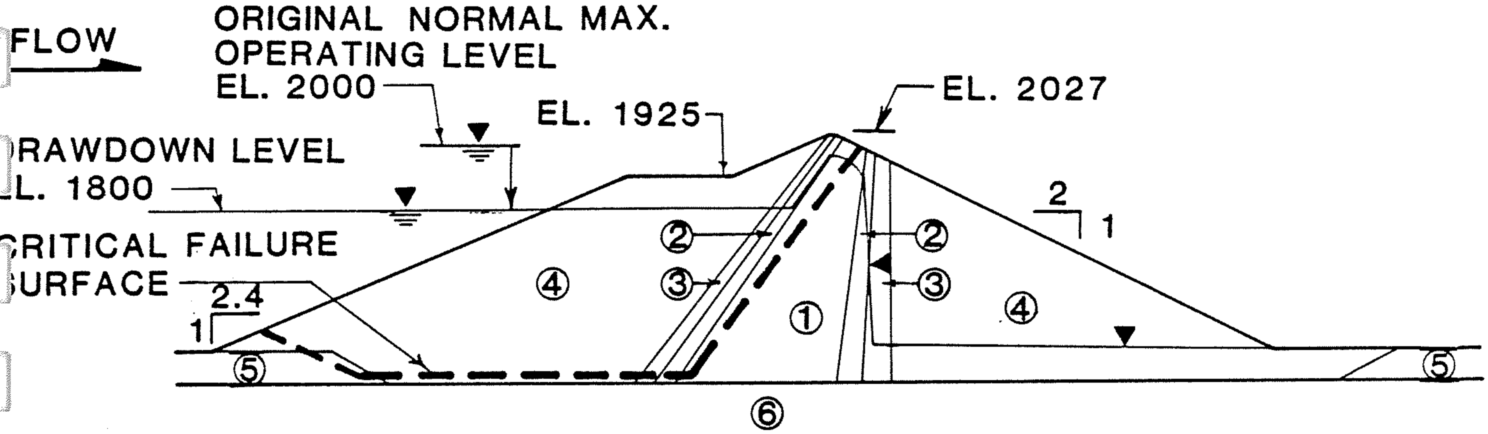
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

STEADY STATE SEEPAGE CASE
(DOWNSTREAM SLOPE)

FIGURE F2.

WATANA-STAGE I
SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.0	1.8



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	CU: C=300 psf, Ø=16.7°
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, Ø=38°
OVERBURDEN FDN.	⑤	CD: C=0 psf, Ø=32°
BEDROCK FDN.	⑥	CD: C= 40,000 psf, Ø=38°

NOTE

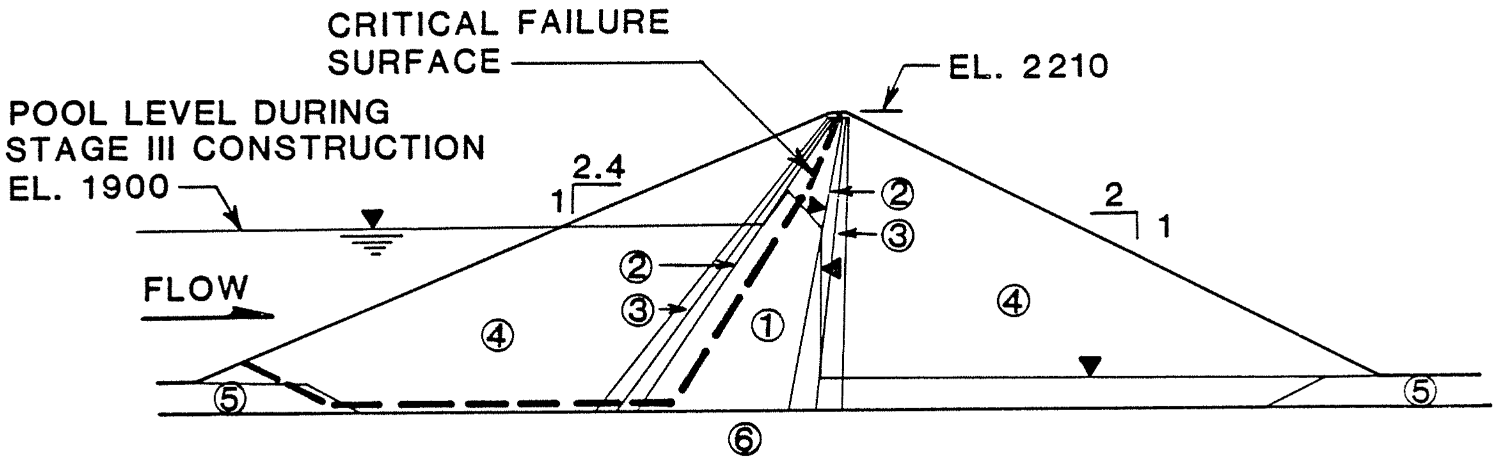
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

RAPID DRAWDOWN CASE
(UPSTREAM SLOPE)

FIGURE 52

WATANA-STAGE III
SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.3	1.5
EARTHQUAKE (Seismic Coefficient=0.15)	1.0	1.0



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	STAGE I EMBANKMENT CD: C=0 psf, ϕ=26.5°
		STAGE III EMBANKMENT UU: C=1,500 psf, ϕ=0°
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, ϕ=38°
OVERBURDEN FND.	⑤	CD: C=0 psf, ϕ=32°
BEDROCK FND.	⑥	CD: C=40,000 psf, ϕ=38°

NOTE

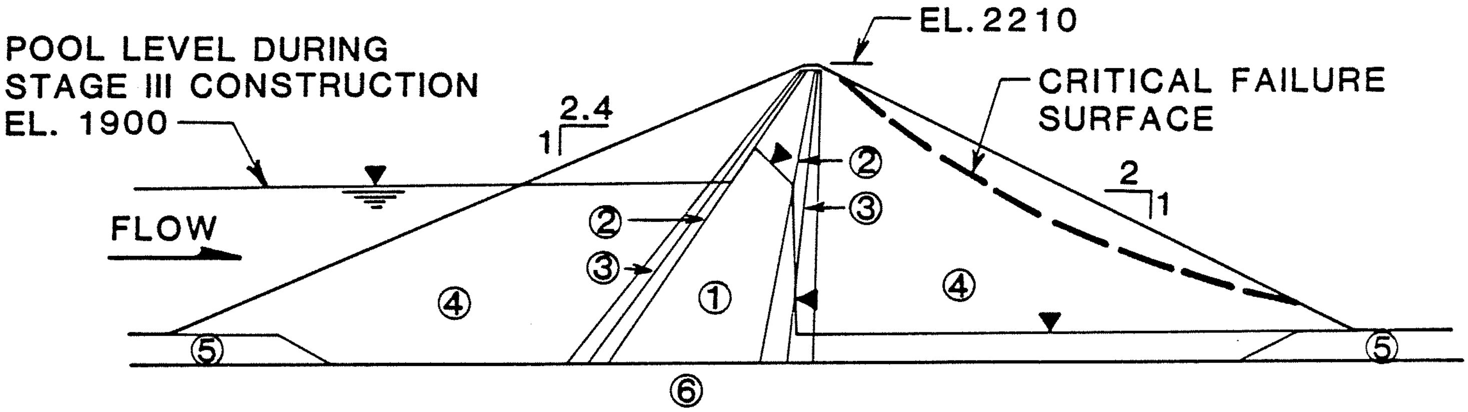
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

END-OF-CONSTRUCTION CASE
(UPSTREAM SLOPE)

FIGURE F2.

WATANA-STAGE III
SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.3	1.6
EARTHQUAKE (Seismic Coefficient=0.15)	1.0	1.1



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	STAGE I EMBANKMENT CD: C=0 psf, ϕ=26.5° STAGE III EMBANKMENT UU: C=1,500 psf, ϕ=0°
ROCKFILL AND FILTERS	② ③ ④	CD: C=0 psf, ϕ=38°
OVERBURDEN FND.	⑤	CD: C=0 psf, ϕ=32°
BEDROCK FND.	⑥	CD: C=40,000 psf, ϕ=38°

NOTE

MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

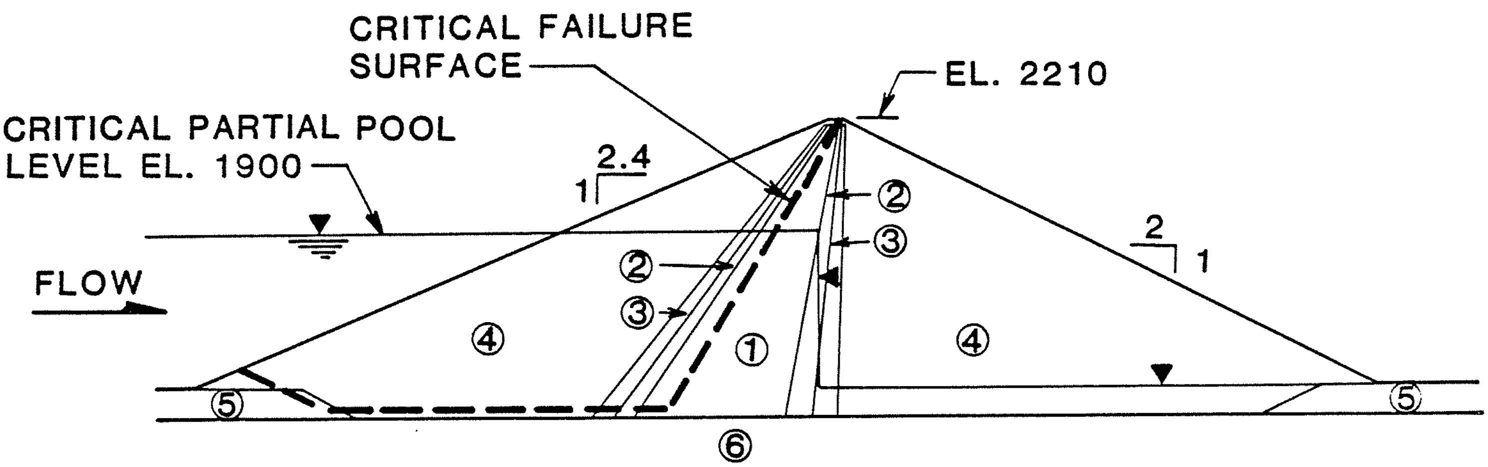
END-OF-CONSTRUCTION CASE
(UPSTREAM SLOPE)

FIGURE F2.1(

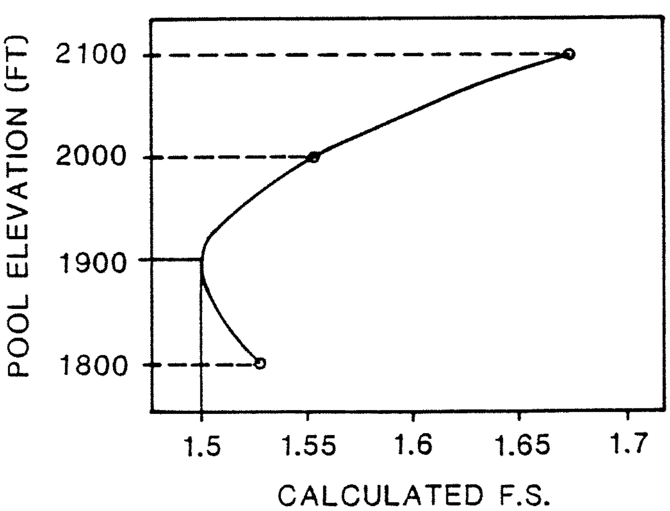
WATANA-STAGE III

SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.5	1.5
EARTHQUAKE (Seismic Coefficient=0.15)	1.0	1.1



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	STAGE I EMBANKMENT CD: C=0 psf, ϕ=26.5°
		STAGE III EMBANKMENT CU: C=300 psf, ϕ=16.7°
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, ϕ=38°
OVERBURDEN FND.	⑤	CD: C=0 psf, ϕ=32°
BEDROCK FND.	⑥	CD: C=40,000 psf, ϕ=38°



NOTE

MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

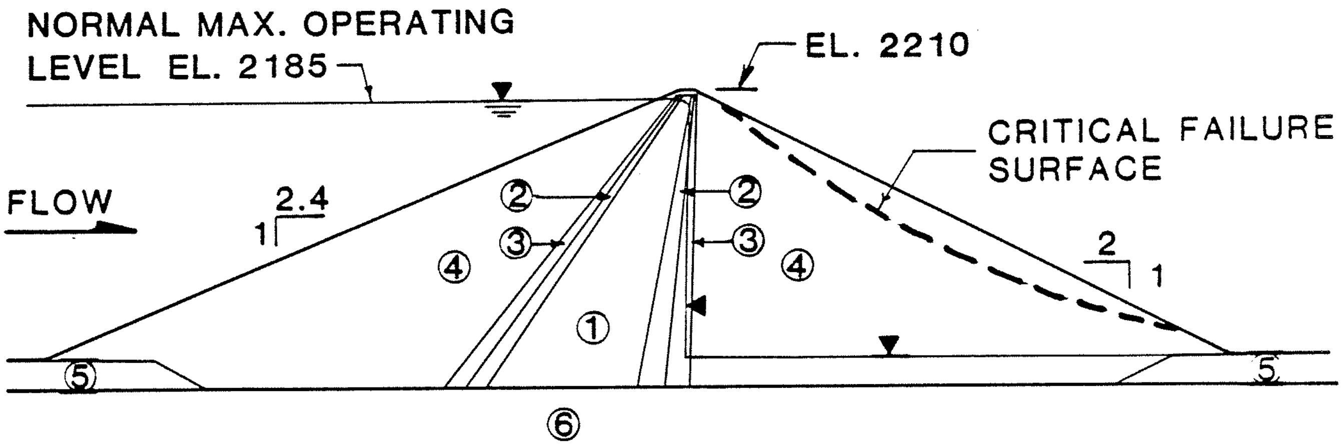
PARTIAL POOL CASE
(UPSTREAM SLOPE)

FIGURE E2.11

WATANA-STAGE III

SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.5	1.6
EARTHQUAKE (Seismic Coefficient=0.15)	1.0	1.1



MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	CD: C=0 psf, Ø=26.5°
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, Ø=38°
OVERBURDEN FDN.	⑤	CD: C=0 psf, Ø=32°
BEDROCK FDN.	⑥	CD: C=40,000 psf, Ø=38°

NOTE

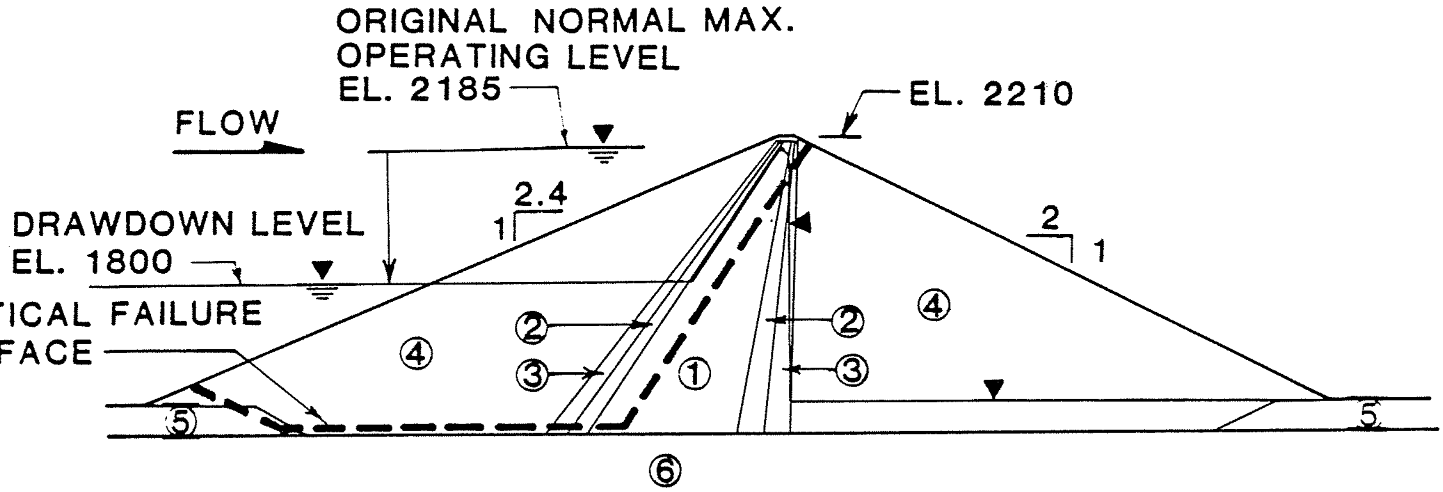
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

STEADY STATE SEEPAGE CASE
(DOWNSTREAM SLOPE)

WATANA-STAGE III

SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.0	1.3

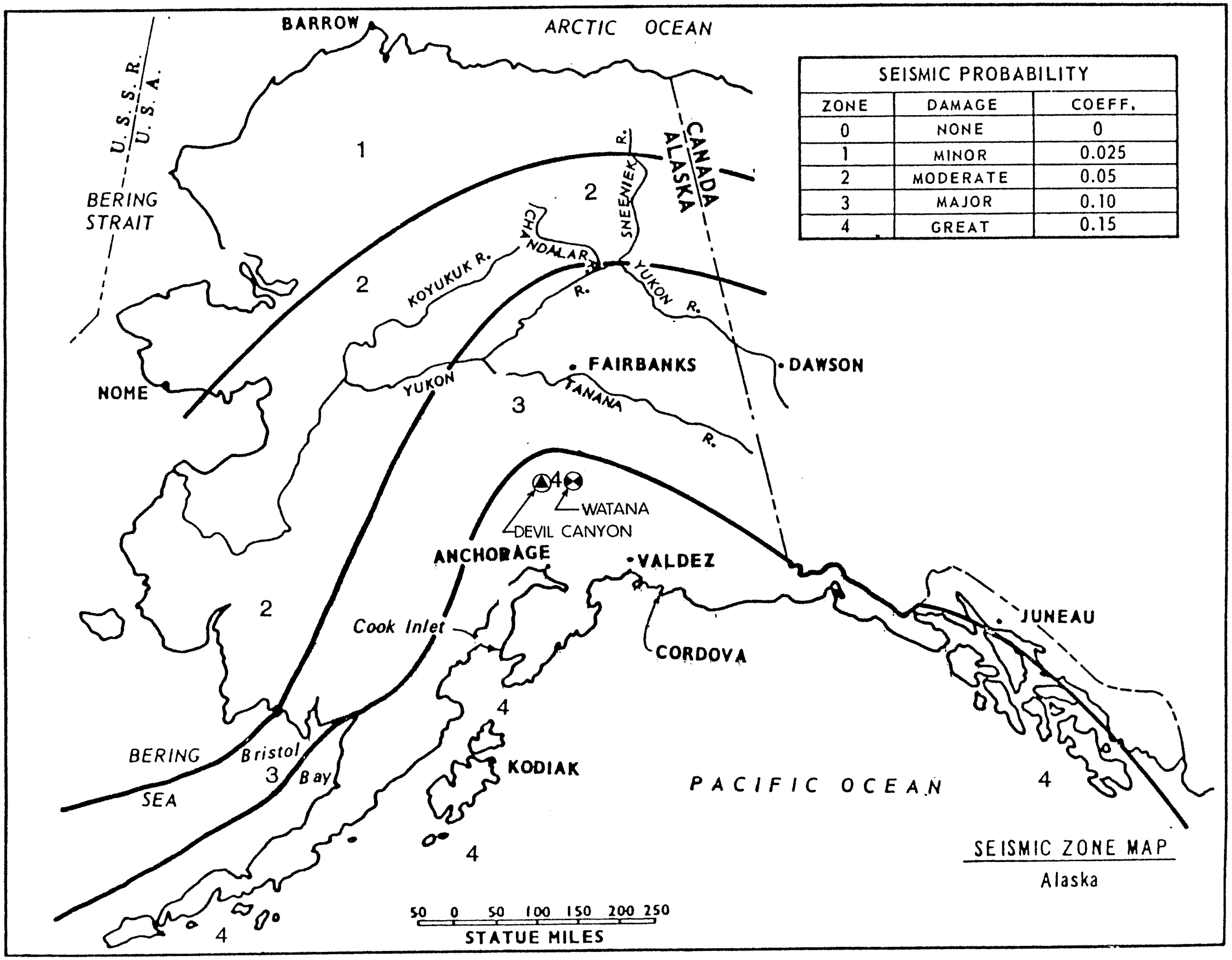


MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	CU: C=300 psf, Ø=16.7°
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, Ø=38°
OVERBURDEN FDN.	⑤	CD: C=0 psf, Ø=32°
BEDROCK FDN.	⑥	CD: C= 40,000 psf, Ø=38°

NOTE

MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

RAPID DRAWDOWN CASE
(UPSTREAM SLOPE)



SEISMIC PROBABILITY		
ZONE	DAMAGE	COEFF.
0	NONE	0
1	MINOR	0.025
2	MODERATE	0.05
3	MAJOR	0.10
4	GREAT	0.15

SEISMIC ZONE MAP
Alaska

FIGURE F2.14

APPENDIX F3

SUMMARY AND PMF AND SPILLWAY DESIGN FLOOD ANALYSES

APPENDIX F3
SUMMARY OF PMF AND SPILLWAY DESIGN FLOOD ANALYSES

1 - INTRODUCTION (**)

The natural PMF peaks at the Watana and Devil Canyon damsites are estimated to be 326,000 cubic feet per second (cfs) and 362,000 cfs, respectively. The routed peak inflows to Devil Canyon are estimated to be 358,000 cfs and 339,000 cfs in Stages II and III. The natural 10,000 year flood peaks are estimated to be 174,000 cfs and 184,000 cfs at Watana and Devil Canyon. Using the 95 percent one-sided upper confidence limits, the 10,000-year floods are estimated to be 240,000 cfs and 262,000 cfs. The 10,000-year events were not routed through the reservoirs because the total capacities of the spillways at the 50 year flood surcharge pool in combination with the outlet works are greater than the 95 percent one sided upper confidence limit estimates, and so the floods could be passed without additional surcharging.

2 - PROBABLE MAXIMUM FLOOD (PMF)

2.1 - Calibration of SSARR Model (o)

In the derivation of PMF, the rainfall-runoff relationships, snowmelt criteria and routing of runoff excess through watershed and channel system, were defined by Streamflow Synthesis and Reservoir Regulations (SSARR) watershed model (COE 1972).

The model was calibrated by U.S. Army Corps of Engineers (COE 1975, 1979) for the Susitna River basin above Gold Creek, a stream gaging station located about 12 miles downstream from the Devil Canyon damsite (Figure F3.1).

The model determines runoff excess from average basin precipitation, snowmelt, evapotranspiration, deep percolation and soil moisture replenishment, and uses flow separation techniques to temporarily store this excess as surface storage, sub-surface storage and groundwater storage to provide time delay effect. The basic routing scheme is provided in the User's Manual for the Model (COE 1972). Figure F3.2 provides a schematic representation of the basic elements of the SSARR model.

The drainage area of the basin above Susitna River at Gold Creek is about 6,160 square miles (mi^2). The basin was divided in 13 relatively homogeneous sub-basins. Flows from these sub-basins were combined and routed downstream to derive the flows at specified locations including those where observed flood hydrographs were available. Figure F3.3 shows a schematic layout of the sub-basins. The figure also shows the drainage area of each sub-basin.

The COE selected the spring floods of 1964 and 1972 and the summer floods of 1967 and 1971 for the model calibration. The calibration was performed by comparing daily observed and simulated flood hydrographs at four stream gaging stations - Susitna River at Gold Creek, near Cantwell and near Denali, and Maclaren River near Paxson (see Figure F3.3). Daily precipitation or snow water equivalent data observed at Summit, Trims Camp, Paxson, Gulkana or Gracious House (see Figure F3.1 for locations) were used. The relationships between parameters in the model and initial values of the parameters were estimated initially based on hydrologic characteristics of each sub-basin. The estimated relationships and initial values were then progressively changed until the simulated flows were within acceptable limits of observed flows. Table F3.1 shows the comparsion of observed and simulated flood peaks. The simulated and observed hydrographs are shown on Figure F3.4 through F3.10. The derived relationships between the model parameters are shown on Figures F3.11 through F3.17.

The input data and calibration procedures used by the COE were reviewed and a few discrepancies in data input were identified. The model calibration was checked by removing these discrepancies. As a result, relationships between the parameters were revised in two cases (see Figures F3.11 and F3.14) using the floods of August 1967 and June 1972 and corresponding daily rainfall data. It was realized that the initial values of the model parameters were not very sensitive except for a few days at the beginning of simulation period. The calibrated relationships between the parameters were tested for their validity by using the 1971 flood. Figures F3.18 through F3.26 show the simulated and observed hydrographs. Table F3.2 lists the curve numbers of the parametric relationships and other pertinent data used for each sub-basin. Elevation-area relationships for the sub-basins are given in Table F3.3.

2.2 - Probable Maximum Precipitation (PMP) (**)

The PMP's for the basins above Watana and Devil Canyon were estimated from the analysis of the following six historic storms by storm maximization:

August 22-28, 1955
July 28 - August 3, 1958
August 19-25, 1959
August 9-17, 1967
August 4-10, 1971
July 25-31, 1980

(a) Storm Isohyetal Pattern (**)

Precipitation pattern in the Susitna basin is greatly affected by orography. Therefore, it was necessary to develop isohyetal patterns for each storm to define variation in precipitation over the basin. This was done by isopercental technique discussed below.

The isopercental technique requires a base isohyetal pattern, usually mean annual or mean seasonal precipitation pattern. For the purpose of these analyses, the isohyetal pattern of July 1980 storm was used as a base map. The July 1980 storm pattern was well-defined because the storm was recorded at a number of gages within and in the vicinity of the basin.

The ratios of the total storm precipitation of a given storm to the July 1980 storm were derived and plotted at each station where data were available for both storms. Isopercental lines were drawn based on these ratios. The ratios on these lines were then multiplied by the July 1980 pattern to yield values to draw isohyetal map for the given storm. The resulting isohyetal patterns are shown on Figures F3.27 through F3.32.

(b) Storm Maximization (**)

The maximization factor for each storm was determined as the ratio between the maximum precipitable water and the precipitable water available during the storm. The maximum precipitable water was computed using 50-year return period maximum 12-hour persisting dewpoint temperatures. These temperatures were derived from dewpoint temperatures recorded at Anchorage for the months of May through September. The actual storm dewpoint temperatures were derived by examining the temperatures prior to the storm occurrence. The maximization factors are listed in the following table.

MAXIMIZATION FACTORS

	Storm Dewpoint at 1,000 mb		Max. Dewpoint at 1,000 mb		
Storm	Temp.	Precip. Water	Temp.	Precip. Water	Max. Factor
August 1955	47	18.3	59.5	34.1	1.86
July-August 1958	50	21.0	60.0	35.2	1.66
August 1959	48	18.9	59.5	34.1	1.80
August 1967	46	17.6	60.0	35.2	2.00
August 1971	49	19.9	60.0	35.2	1.77

PMP. Average precipitation over the basin above Watana was computed using the isohyetal pattern developed for six storms (Figure F3.27 through F3.32). These precipitation amounts were multiplied by the maximization factors resulting in maximized total precipitation given in the following table.

MAXIMIZED PRECIPITATION

Storm	Maximized Total Precipitation
August 1955	7.03
July-August 1958	4.96
August 1959	6.82
August 1967	12.54
August 1971	9.04

The August 1967 storm resulted in the largest maximized precipitation amount if it were to occur also in August. However, snowmelts in August would be negligible compared to those in late spring and early summer. Therefore, the storm was assumed to occur in June with a lower maximization factor, estimated to be 1.4. This provided an average basin PMP of 8.7 inches above Watana site. The PMP for the basin above Devil Canyon was computed by adding the sub-basin between the two sites to 8.8 inches.

(c) Temporal Precipitation Pattern (**)

The August 1967 storm has a duration of 10 days. Daily distribution of basin average precipitation was computed using daily storm precipitation observed at stations within and surrounding the basin. This distribution was used for PMP.

The daily precipitation amounts were arranged sequentially so that critical flood conditions are produced at the dam sites. This was done by assuming that the largest 24-hour precipitation occurs on the eighth day of the PMP storm. The second largest occurs on the seventh an third largest on the ninth day. The entire pattern is shown in the following table:

TEMPORAL PATTERN OF PMP

Daily Precipitation Ranking[1/]

	Storm Duration									
Daily Precipitation Ranking[1/]	10	9	8	7	6	4	2	1	3	5

1/ "1" is largest and "10" is smallest.

Daily precipitation was further distributed into 50 percent, 20 percent, 15 percent and 15 percent values for each respective 6-hour period. The 6-hour precipitation was distributed in ascending order for each day up to the ninth day, while the ninth and tenth day's 6-hourly precipitation was distributed in descending order. The following table gives the 6-hourly distribution pattern for the PMP over the drainage basin above Watana.

2.3 - Snowmelt Criteria (o)

An analysis of major historical floods indicated that snowmelt contributes a major part of the floods. Therefore, to insure adequate snowmelt contribution to the PMF, it was assumed that the snowpack is unlimited for glacial sub-basins (10 and 210). The snowpack for other sub-basins was estimated to be large enough to ensure a substantial residual snowpack during the storm period. The estimates were based on maximum recorded data at stations in and around the Susitna basin. The following table gives the estimated initial snowpack for each sub-basin.

6-HOURLY DISTRIBUTION PATTERN

Day	Hour	PMP	Day	Hour	PMP	Day	Hour	PMP
1	6	.00	5	6	.12	9	6	.59
	12	.00		12	.12		12	.24
	18	.01		18	.16		18	.17
	24	.01		24	.40		24	.17
2	6	.04	6	6	.16	10	6	.40
	12	.04		12	.16		12	.17
	18	.04		18	.21		18	.12
	24	.05		24	.54		24	.12
3	6	.13	7	6	.19			
	12	.13		12	.19			
	18	.13		18	.26			
	24	.13		24	.65			
4	6	.10	8	6	.32			
	12	.32		12	.32			
	18	.15		18	.43			
	24	.35		24	1.08			

INITIAL SNOWPACK FOR PMF

Sub-basin	Snowpack	Sub-basin	Snowpack
10	99	330	33
20	81	340	27
80	35	380	59
180	32	480	57
210	99	580	48
220	62	680	48
280	30		

The temperature sequences prior to, during, and after PMP are shown on Figure F3.33. Temperatures through May are assumed at 32°F to ensure the snowpack is ripening, but yielding little or no snowmelt runoff; following that, a sudden increase in temperature is assumed. This temperature gradient is based on maximum one to seven day temperature rises observed for the period of records at Anchorage and Talkeetna. During the PMP storm, the temperatures are lowered. After the most significant precipitation has fallen, temperatures are increased again.

2.4 - Occurrence of Snowmelt and PMP Storm (o)

The snowmelt starts on June 3 based on the adapted temperature sequences (Figure F3.33). The PMP storm is assumed to occur between June 8 and 17. This provides a 5-day period between start of PMP and start of snowmelt. This time interval was considered adequate for combination of floods resulting from PMP and snowmelt.

2.5 - Antecedent Conditions (**)

The amount of soil moisture present at the on-set of PMP and snowmelt significantly controlled the amount of water available for runoff including its distribution as surface, subsurface, and and baseflow components. Relatively moist soil conditions were assumed for each sub-basin. The following table gives the initial values used for the model parameters.

2.6 - PMF (***)

The calibrated relationships of the model parameters shown in Figures F3.11 through F3.17, and the initial values of parameters shown in the following table, were used to derive the PMF hydrographs at the dam sites. The resulting inflow peaks are 326,000 cfs for Watana site and 362,000 cfs for Devil Canyon site (without Watana). Figures F3.34 and F3.35 show the inflow hydrographs at the two sites.

INITIAL VALUES OF SSARR MODEL PARAMETERS

			Runoff		
Sub-Basin	Soil Moisture	Baseflow Infiltration Index	Surface	Sub-Surface	Base-Flow
10	8	.03	10	30	60
20	4	.03	10	50	60
80	4	.03	5	10	70
180	4	.03	7	10	108
210	8	.03	10	10	10
220	4	.03	10	10	60
280	4	.03	4	10	70
330	4	.03	18	0	0
340	4	.03	18	20	120
380	4	.03	8	20	130
480	4	.03	16	30	420
580	4	.03	5	10	260
680	4	.03	4	10	140

The U.S. Army Corps of Engineers (COE 1965a) indicates that the standard project flood (SPF) serves the following primary purposes:

> "Represents a 'standard' against which the degree of protection finally selected for a project may be judged and compared with protection provided at similar projects in other localities. The SPF estimate must reflect a generalized analysis of flood potentialities in a region, as contrasted to an analysis of flood records at the specific locality that may be misleading because of the inadequacies of records or abnormal sequences of hydrologic events during the period of stream flow observation.
>
> Represent the flood discharge that should be selected as the design flood for the project, or approached as nearly as practicable in consideration of economic or other governing limitations, where some small degree of risk can be accepted but an unusually high degree of protection is justified by hazards to life and high property values within the area to be protected. Estimates completed to date indicate that SPF flood discharges flood discharges are generally equal to 40 to 60 percent of 'maximum probable' floods for the same basins.
>
> The Maximum Probable (or Maximum Possible) Flood estimates are applicable to projects where consideration is to be given to virtually complete security against potential floods. Applications of such estimates are usually confined to the determination of spillway requirements for high dams, but in unusual cases may constitute the design flood for local protection works where an exceptionally high degree of protection is advisable and economically obtainable."

Additionally, the same publication goes on to state that:

> "Estimates completed to date indicate that SPF discharges based on detailed studies usually equal 50 to 60 percent of the maximum probable (or 'maximum possible') flood for the same basin; a ratio of 50 percent is considered representative of average conditions. Inasmuch as computation of maximum probable flood estimates are normally required as the basis of design of spillways for high dams, it is convenient to estimate the SPF for reservoir projects as equal to 50 percent of the maximum probable flood hydrograph to avoid the preparation of a separate SPF estimate (see paragraph 1-05 and 3-02 d regarding SPF series). Accordingly, this convention is acceptable for reservoir projects in general. The rule may also be applied in estimating SPF hydrographs for basins outside of the region and range of areas covered by generalized charts present herein where maximum probable flood estimates based on detailed hydrometeor logical investigations have been completed. Where snow melt or extreme ranges in topography are major factors to be taken into consideration, it is appropriate to estimate the maximum probable flood hydrograph for the basin by considering optimum combinations of critical flood-producing factors and assuming the SPF hydrographs is equal to 50 percent of the maximum probable discharges. This approximation is based on the conclusion that critical conditions can be determined from analyses of meteorological and topographic influences, whereas a substantial period of hydro-meteorological records are required to determine appropriate combinations of flood producing factors meeting SPF specifications."

In accordance with these criteria and criteria presented by the U.S. Committee on Large Dams (USCOLD 1970) the Watana and Devil Canyon spillways have been designed to pass the PMF in combination with the outlet works without overtopping the dams.

Additionally, the 10,000-year flood and the 95 percent one-sided upper confidence level have been computed and the capacity of the spillways and outlet works have been found capable of passing these discharges without surcharging the reservoir above the 50-year flood pool level.

The 10,000 year flood peak on the Susitna River at Gold Creek and its 95 percent one-sided upper confidence level were estimated to be 190,000 cfs and 270,000 cfs, respectively. The estimates at Watana damsite are 174,000 cfs and 248,000 and at Devil Canyon damsite are 184,000 cfs and 262,000 cfs. The peak flows at Gold Creek were estimated from the station record of 34 years. The peaks at the damsites were estimated by multiplying the Gold Creek values by the square root of the drainage area ratios. The mean estimates of the 10,000 year flood are greater than 50 percent of the PMF peaks. The 95 percent one-sided upper confidence level values are greater than 70 percent of the PMF peaks.

The combined spillway and outlet facility capacities at Watana at the the 50-year flood surcharge pool level during Stages I, II and III are 290,000 cfs, 280,000 cfs and 250,000 cfs, respectively. The corresponding capacity at Devil Canyon during Stages II and III is 282,000 cfs. These capacities are far in excess of the mean estimates of the 10,000-year flood, exceed the 95 percent one-sided upper-confidence-level values and exceed the guidelines of the U.S. Army Corps of Engineers for standard project floods (COE 1965a). Since the spillways also have the capacity to pass the PMF without overtopping the dam, the spillway and outlet facilities are considered to have a sufficient capacity to ensure the safety of the project.

2.7 - Design Floods

(This section deleted)

TABLES

TABLE F3.1: COE CALIBRATION RESULTS COMPARISON OF SIMULATED AND OBSERVED MAXIMUM DAILY DISCHARGE

	Observed		Simulated		Percent
	Discharge	Date	Discharge	Date	Difference
A Susitna River at Gold Creek					
May 19 to June 25, 1964	85,900	Jun. 7	80,500	Jun. 5	-6.3
July 1 to August 31, 1967	76,000	Aug. 15	78,800	Aug. 16	+3.7
May 6 to September 30, 1971	66,300	Jun. 12	53,000	Jun. 11	-20.1
	77,700	Aug. 10	74,100	Aug. 12	-4.6
May 2 to September 30, 1972	70,700	Jun. 17	60,800	Jun. 17	-14.0
	26,400	Sep. 14	32,300	Sep. 15	+22.4
B Susitna River nr. Cantwell					
May 19 to June 25, 1964	49,100	Jun. 7	51,100	Jun. 4	-4.1
July 1 to August 31, 1967	36,400	Aug. 15	36,600	Aug. 16	+0.1
May 6 to September 30, 1971	24,000	Jun. 23	32,600	Jun. 23	-35.8
	36,000	Aug. 9	44,000	Aug. 11	+22.2
May 2 to September 30, 1972	37,600	Jun. 17	37,800	Jun. 17	+0.5
	21,000	Sep. 14	22,800	Sep. 15	+8.6
C Susitna River nr. Denali					
May 19 to June 25, 1964	16,000	Jun. 7	17,200	Jun. 4	-7.5
July 1 to August 31, 1967	No record		16,000	Aug. 16	
May 6 to September 30, 1971	17,600	Jun 27	17,300	Jun. 24	-1.7
	33,400	Aug. 10	31,500	Aug. 11	-5.7
May 2 to September 30, 1972	14,700	Jun. 16	20,300	Jun. 17	+38.1
	5,690	Sep. 13	15,300	Sep. 13	+16.9
D Maclaren River nr. Paxson					
May 19 to June 25, 1964	6,400	Jun. 7	6,230	Jun. 4	-2.7
July 1 to August 31, 1967	7,280	Aug. 14	7,290	Aug. 15	0
May 6 to September 30, 1971	5,520	Jun. 25	5,430	Jun. 25	-1.6
	8,100	Aug. 11	7,980	Aug. 12	-1.5
May 2 to September 30, 1972	6,680	Jun. 16	7,780	Jun. 16	-16.5
	3,980	Sep. 13	2,950	Sep. 12	-25.9

TABLE F3.2: SUB-BASIN WATERSHED CHARACTERISTICS INPUT FOR SSARR MODEL

Sub-basin Identification Number	10	20	80	180	210	220	280	330	340	380	480	580	680
Drainage area, mi^2	221	694	312	477	44	232	307	48	1047	735	1045	628	345
Number of Surface Routing Phases	4	4	4	4	3	4	4	3	8	3	4	4	4
Surface Storage Time (hr)	6	8	3	3	6	5	3	15	10	3	8	8	8
Number of Sub-Surface Routing Phases	4	4	4	4	3	4	4	1	8	4	4	4	4
Sub-Surface Storage Time (hr)	12	20	8	8	12	20	8	0	48	8	15	15	15
Number of Baseflow Routing Phases	4	5	5	5	3	5	5	1	8	4	5	5	5
Baseflow Storage Time, (hr)	24	156	156	156	24	156	156	0	200	96	156	156	156
Baseflow Infiltration Index Time (hr)	100	100	100	100	100	75	100	100	100	100	100	100	100
Table No. for PPT vs. KE (Figure F3.15)	5001	5001	5001	5001	5001	5001	5001	5001	5001	5001	5001	5001	5001
Table No. QGEN vs. SCA (Figure F3.16)	6004	6006	6006	6006	6004	6006	6006	6006	6006	6006	6006	6006	6006
Table No. for Month vs ETI (Figure F3.14)	4009	4008	4008	4008	4009	4008	4008	4008	4008	4008	4008	4008	4008
Table No. for SMI vs ROP (Figure F3.11)	1015	1018	1018	1018	1015	1018	1018	1022	1021	1018	1020	1020	1020
Table No. for BII vs BFP (Figure F3.12)	2017	2011	2009	2009	2017	2012	2009	2009	2009	2009	2009	2009	2009
Maximum Percent of Runoff to Baseflow	10	10	9	9	10	10	10	9	9	10	9	9	9
Table No. for RGS vs. RS (Figure F3.13)	3009	3008	3008	3008	3009	3003	3008	3008	3008	3008	3008	3008	3008
Table No. for QGEN vs MELTR (Figure F3.17)	7011	7005	7010	7010	7009	7005	7010	7010	7010	7010	7005	7005	7005
Rain Freez. Temp. (°F)	35	35	35	35	35	35	35	35	35	35	35	35	35
Base Temp. for Degree - Day (°F)	32	32	32	32	32	32	32	32	32	32	32	32	32
Lapse Rate (°F/1000 ft)	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3

TABLE F3.3: SUB-BASIN ELEVATION-AREA RELATIONSHIP

Sub-basin 10

Elevation, ft	2800	3000	4000	5000	6000	7000	8000	9000	13,820
Percent area below	0	4.5	17.7	35.9	61.1	84.8	96.1	99.8	99.9

Sub-basin 20

Elevation, ft	2440	3000	4000	5000	6000	7000	8000	9000	10,000	13,820
Percent area below	0	27.7	53.2	81.3	92.8	97.1	98.4	98.9	99.8	99.9

Sub Basin 80

Elevation, ft	2370	3000	4000	5000	6000	6100
	0	35.9	74.4	97.1	99.7	99.9

Sub-basin 180

Elevation, ft	2350	3000	4000	5000	6000	6100
Percent area below	0	35.0	82.0	96.4	96.5	99.9

Sub-basin 210

Elevation, ft	3150	4000	5000	6000	7000	8000	8850
Percent area below	0	10.9	24.1	67.2	96.0	99.8	99.9

Sub-basin 220

Elevation, ft	2860	3000	4000	5000	6000	7000	8000	8850
Percent area below	0	8.2	50.5	80.1	94.9	98.6	99.8	99.9

Sub-basin 280

Elevation, ft	2350	3000	4000	5000	5275
Percent area below	0	49.8	96.7	96.8	99.9

Sub-basin 330

Elevation, ft	2361	2363
Percent area below	0	99.9

Sub-basin 340

Elevation, ft	2100	3000	4000	5000	5275
Percent area below	0	68.7	95.2	99.8	99.9

Sub-basin 380

Elevation, ft	1910	2000	3000	4000	5000	6000	7000	7770
Percent area below	0	2.0	15.6	49.1	78.4	96.0	99.8	99.9

Sub-basin 480

Elevation, ft	1450	2000	3000	4000	5000	6000	7000	7200
Percent area below	0	3.0	27.7	68.3	91.1	98.9	99.8	99.9

Sub-basin 580

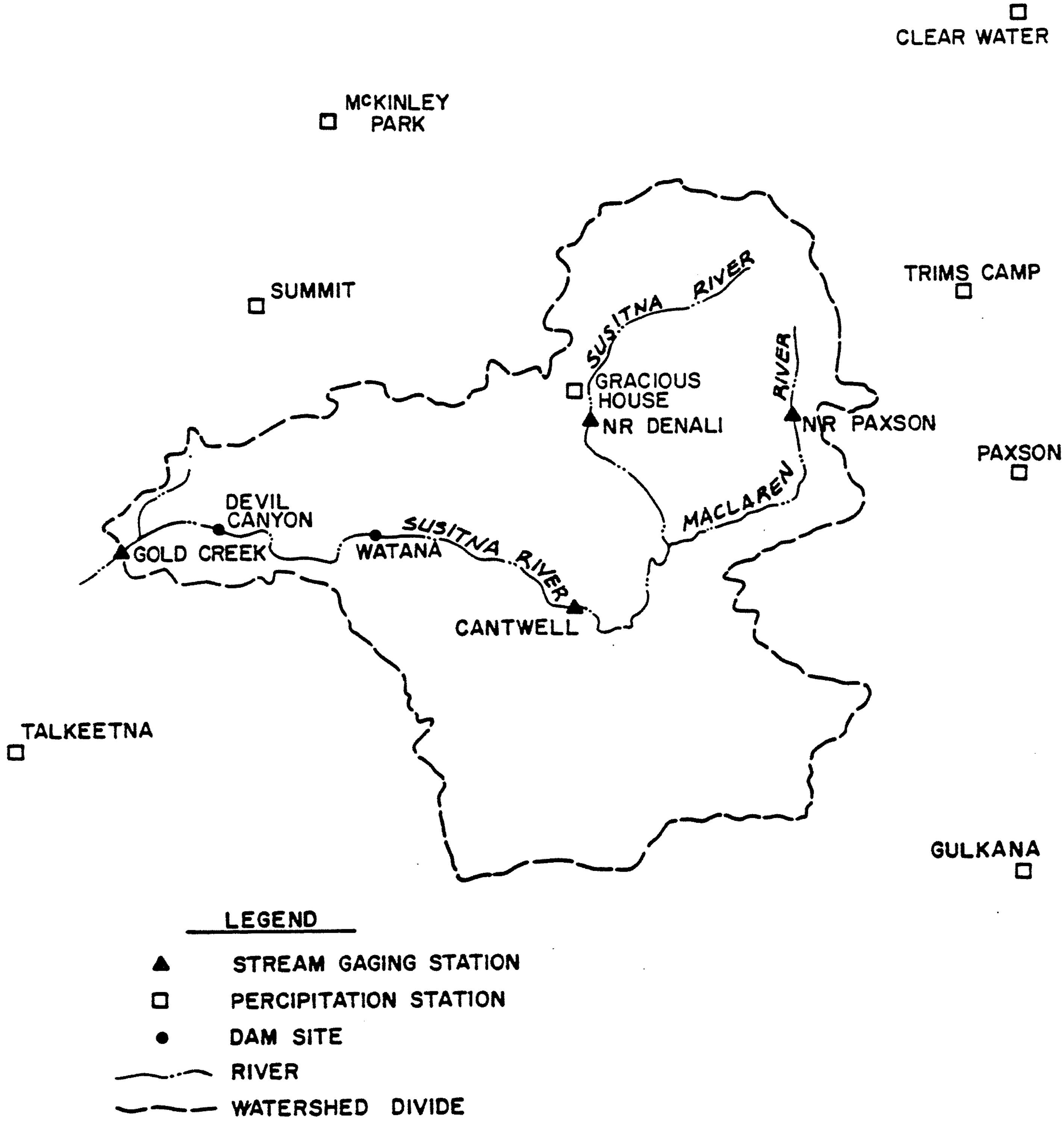
Elevation, ft	910	1000	2000	3000	4000	5000	6000	6910
Percent area below	0	2.0	8.4	44.1	79.5	96.2	99.8	99.9

Sub-basin 680

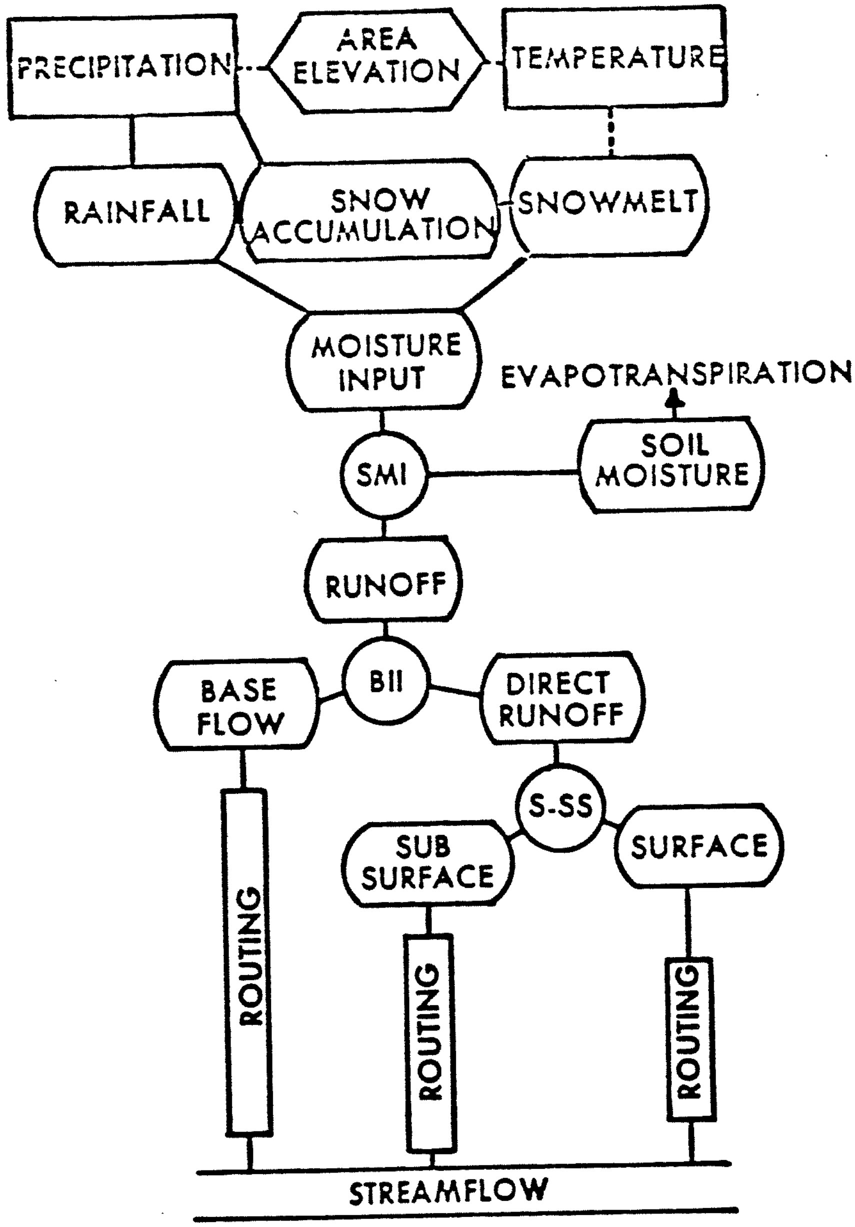
Elevation, ft	677	1000	2000	3000	4000	5000	6000	6018
Percent area below	0	3.2	26.1	51.0	80.9	97.1	99.8	99.9

FIGURES

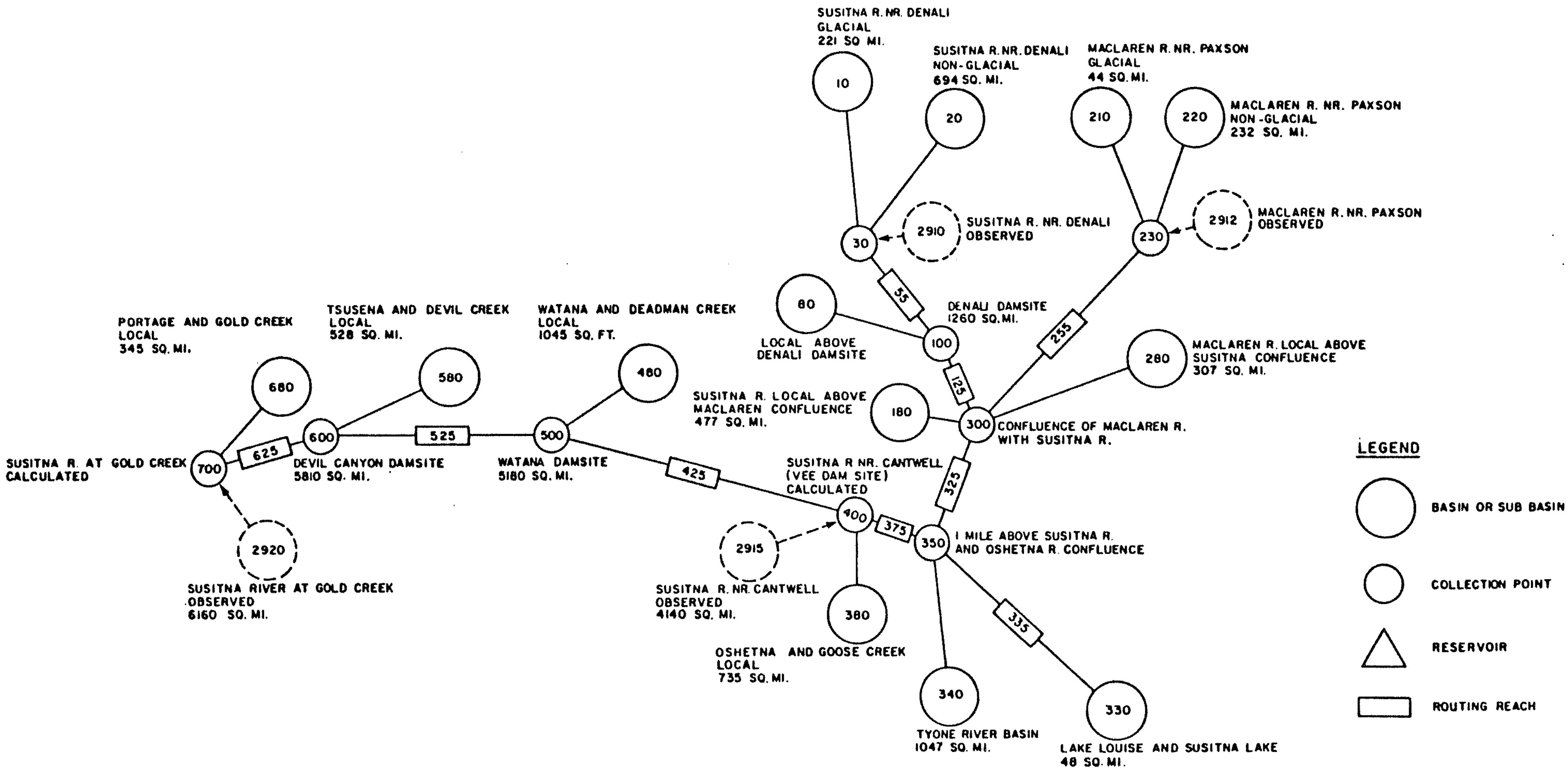
Figure F3.1



SUSITNA RIVER BASIN
ABOVE GOLD CREEK



SSARR WATERSHED MODEL

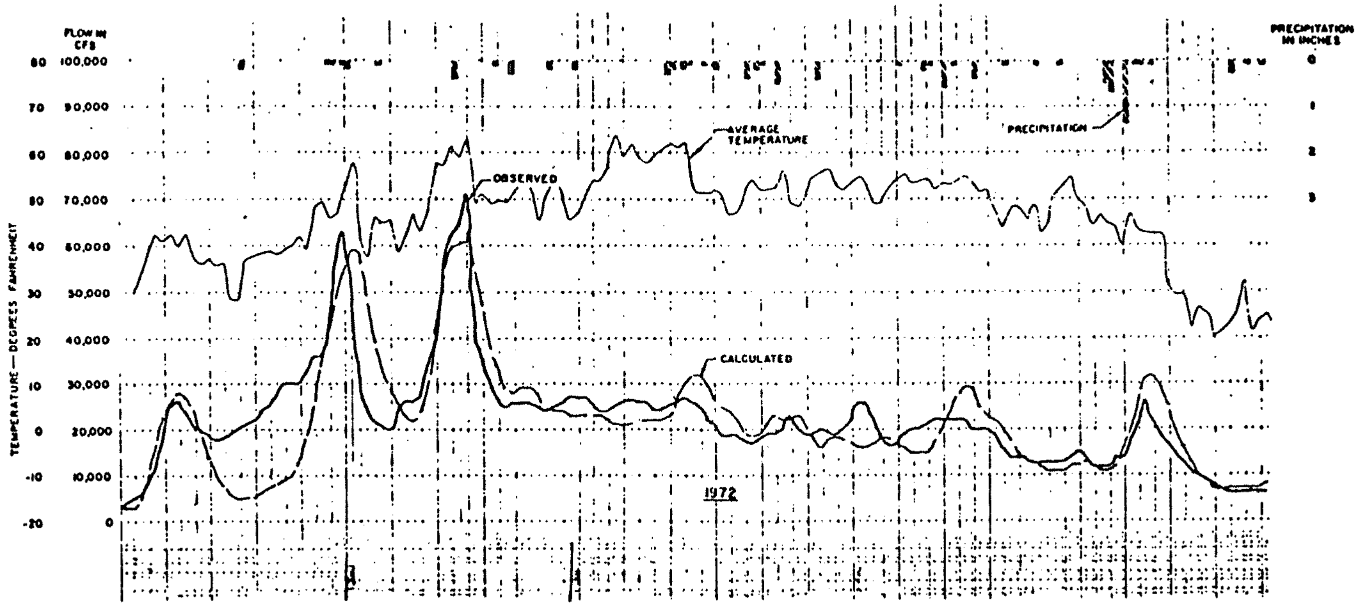


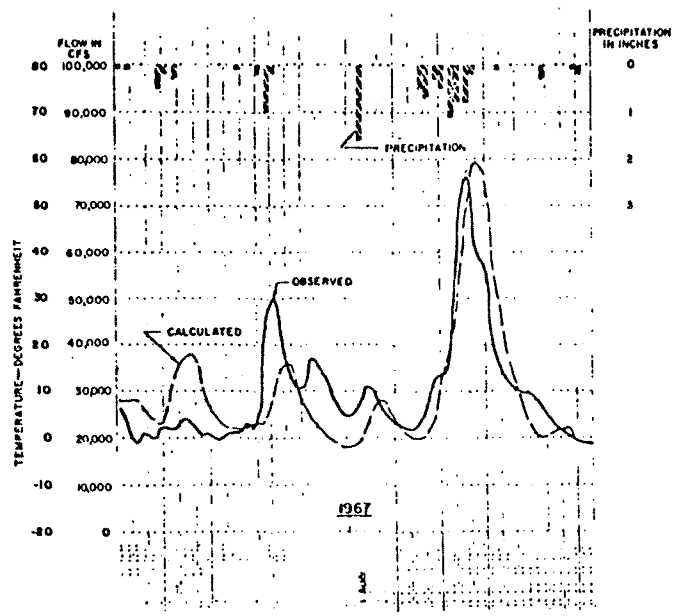
SCHEMATIC DIAGRAM OF SSARR COMPUTER MODEL

REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY
REPORT, 1975 APPENDIX I PART I

Figure F3.3

ACRES



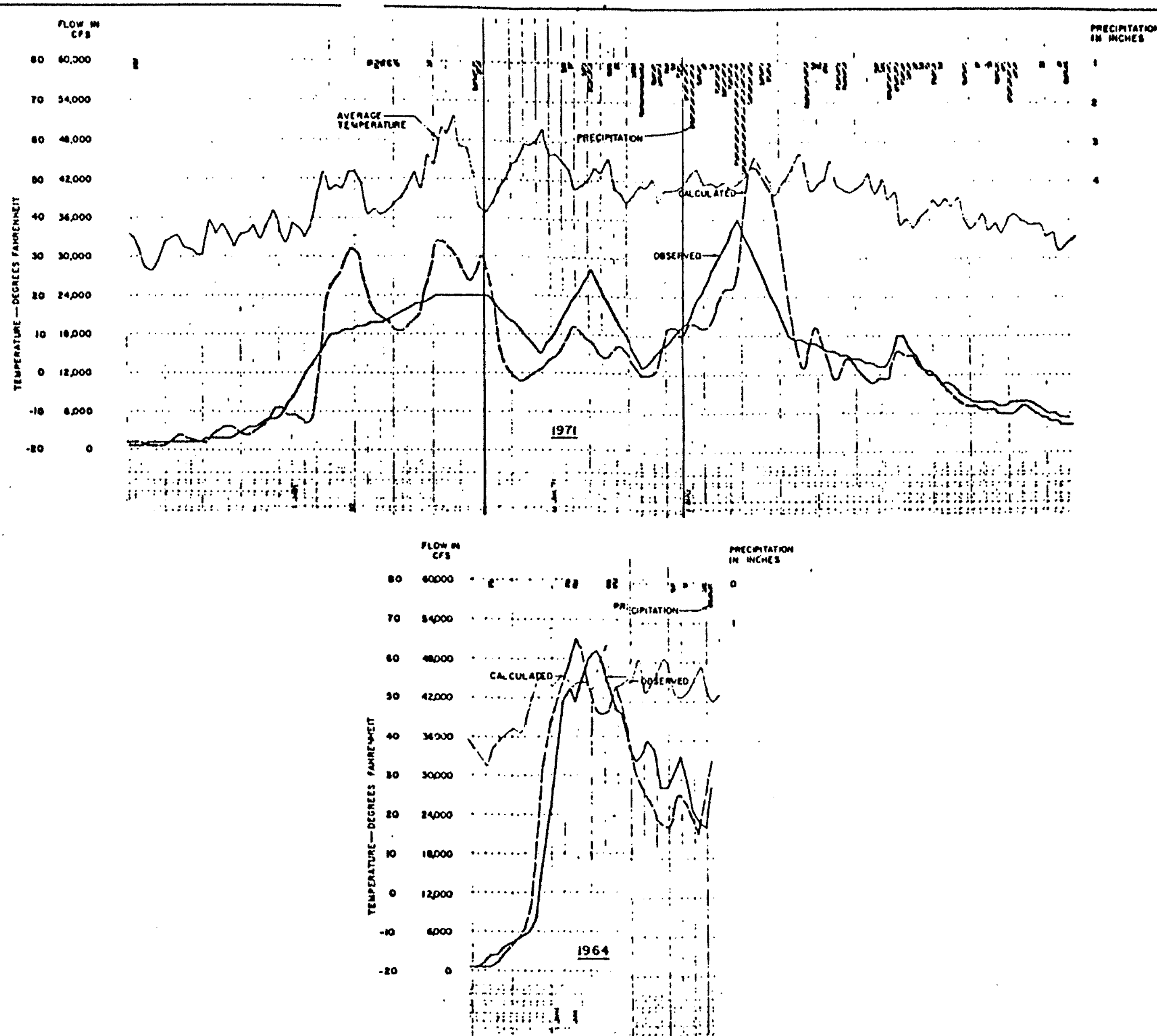


REFERENCE:
U S ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH : SUSITNA RIVER AT GOLD CREEK, 1967, 1972

Figure F3.4

ACRES

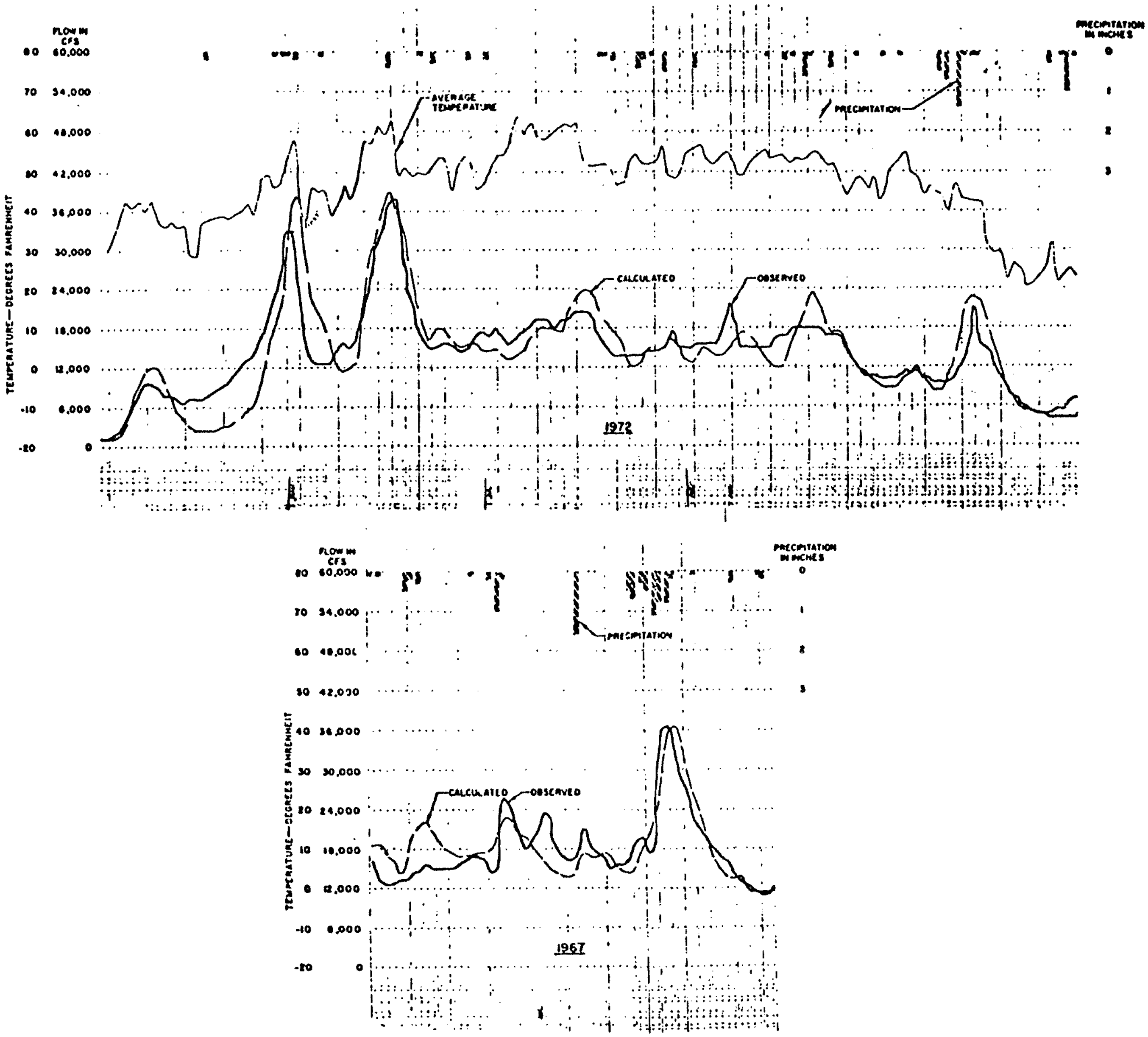


REFERENCE.
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH : SUSITNA RIVER NEAR CANTWELL, 1964, 1971

Figure F3.5

ACRES

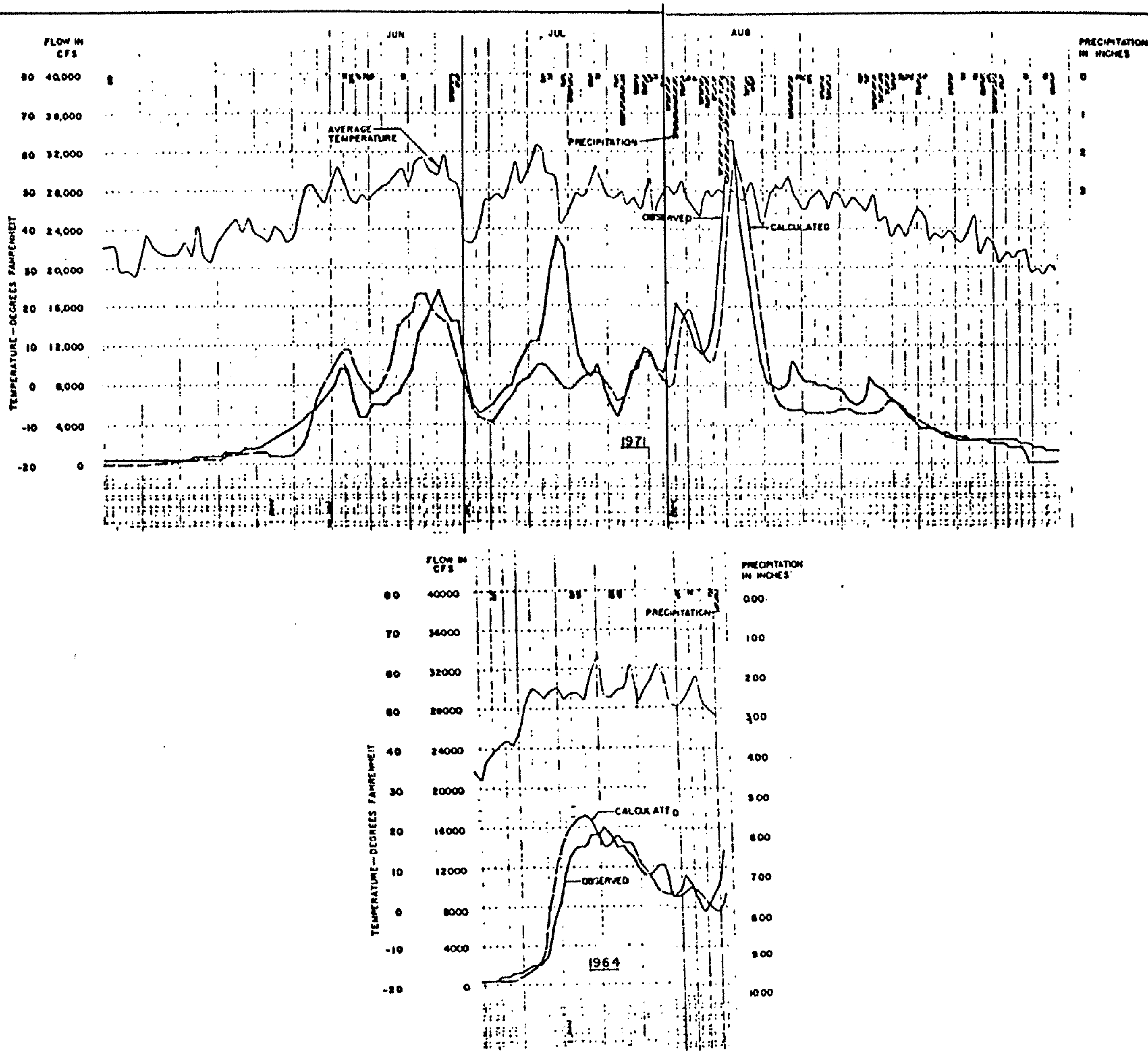


REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH : SUSITNA RIVER NEAR CANTWELL, 1967, 1972

Figure F3.6

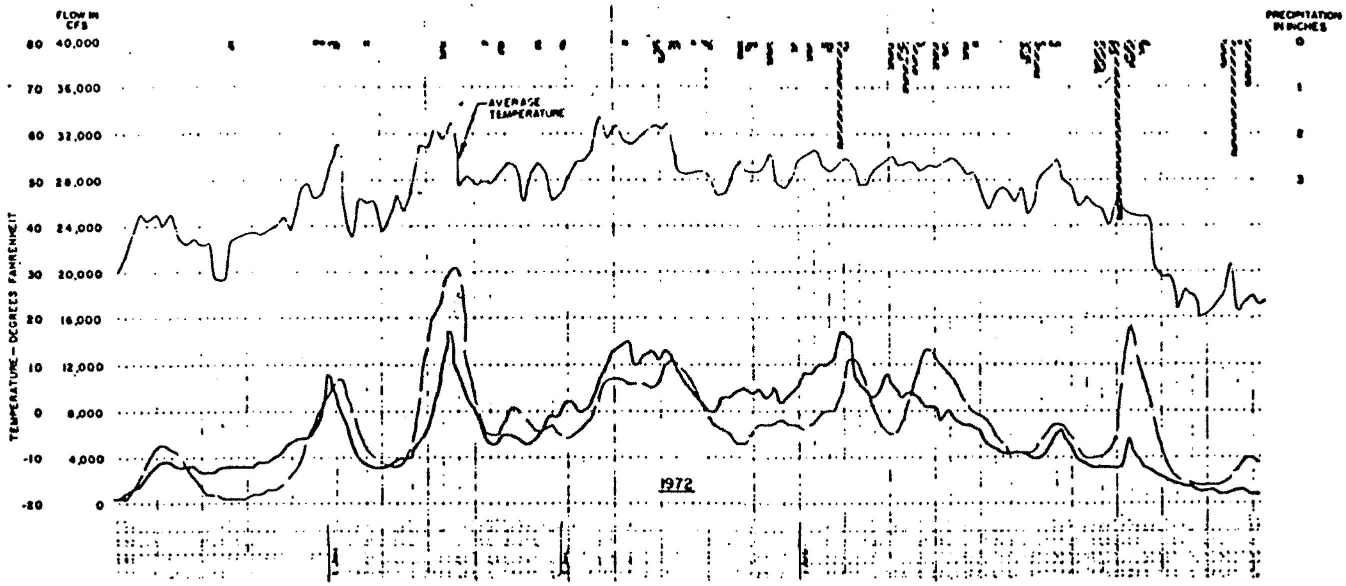
ACRES



REFERENCE.
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH SUSITNA RIVER NEAR DENALI, 1964, 1971

Figure F3.7

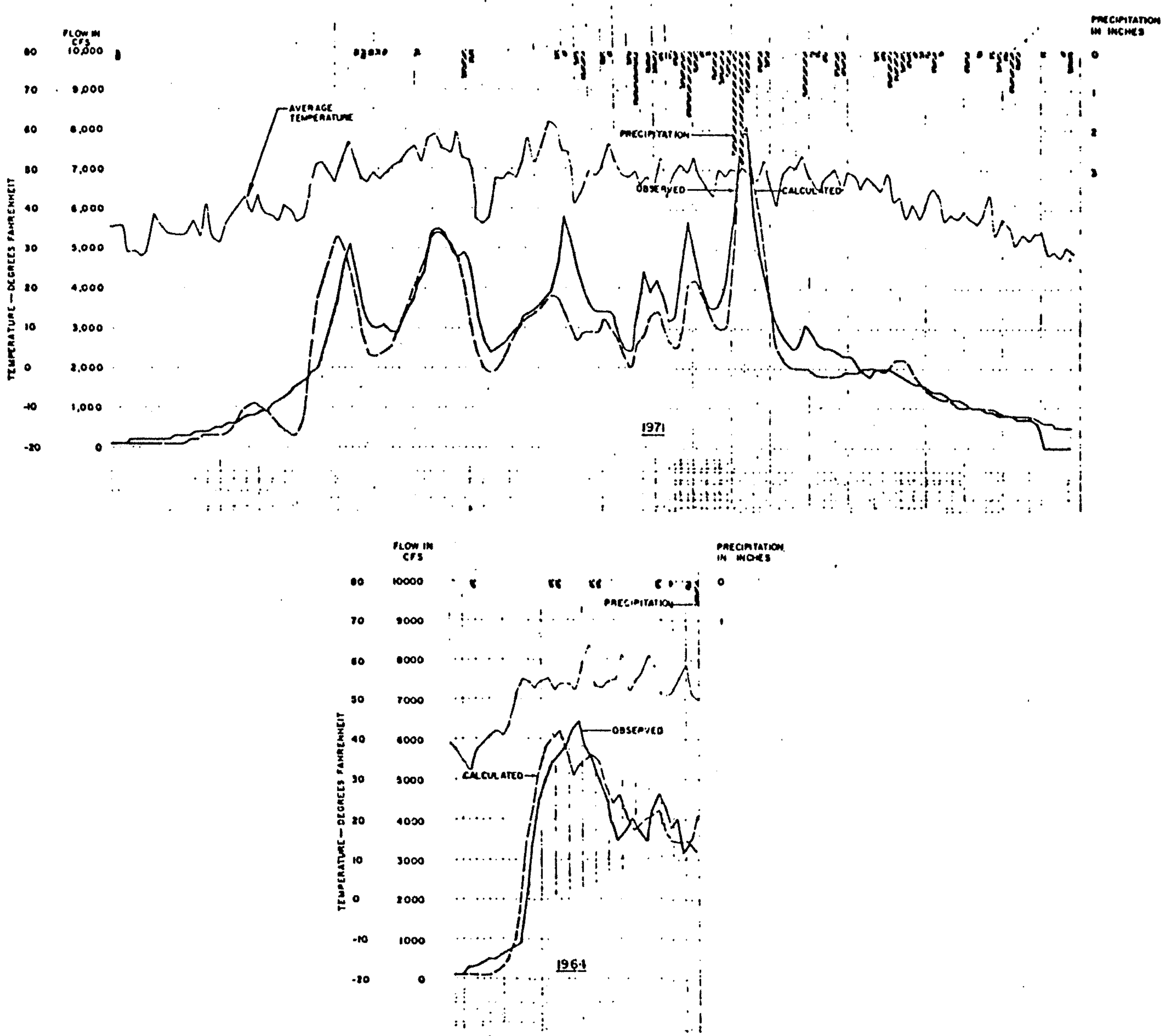


REFERENCE
U S ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY
REPORT, 1975 APPENDIX I PART I

HYDROGRAPH SUSITNA RIVER NEAR DENALI, 1972

Figure F3.8

ACRES

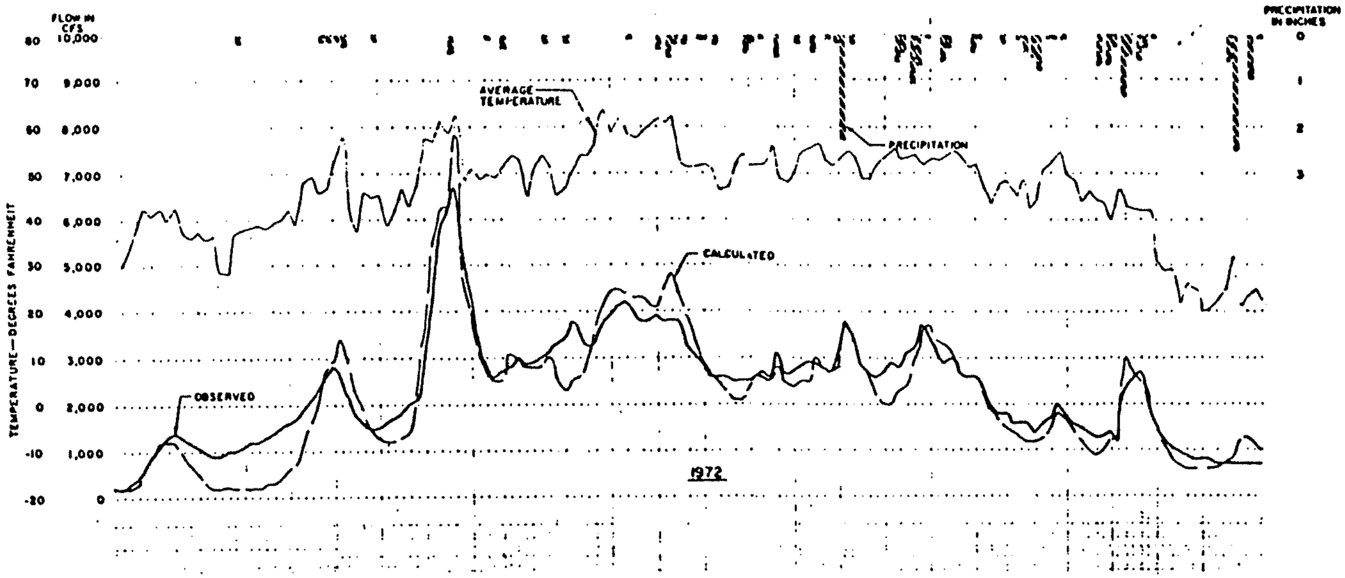


REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH: MACLAREN RIVER NEAR PAXSON, 1964, 1971

Figure F3.9

ACRES



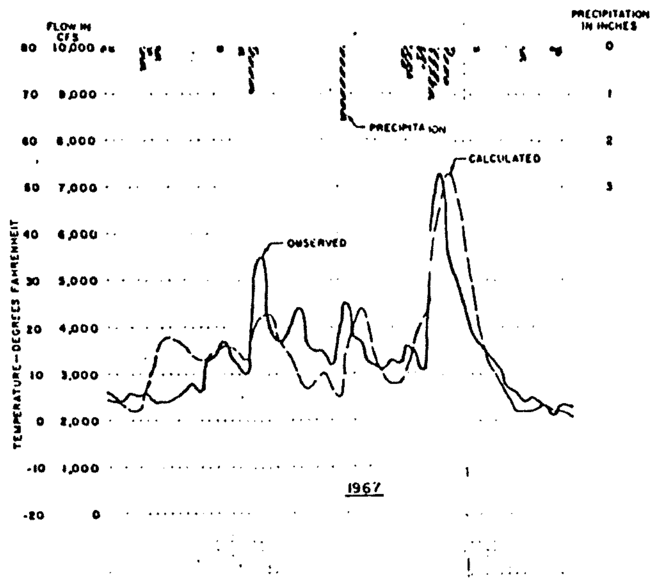
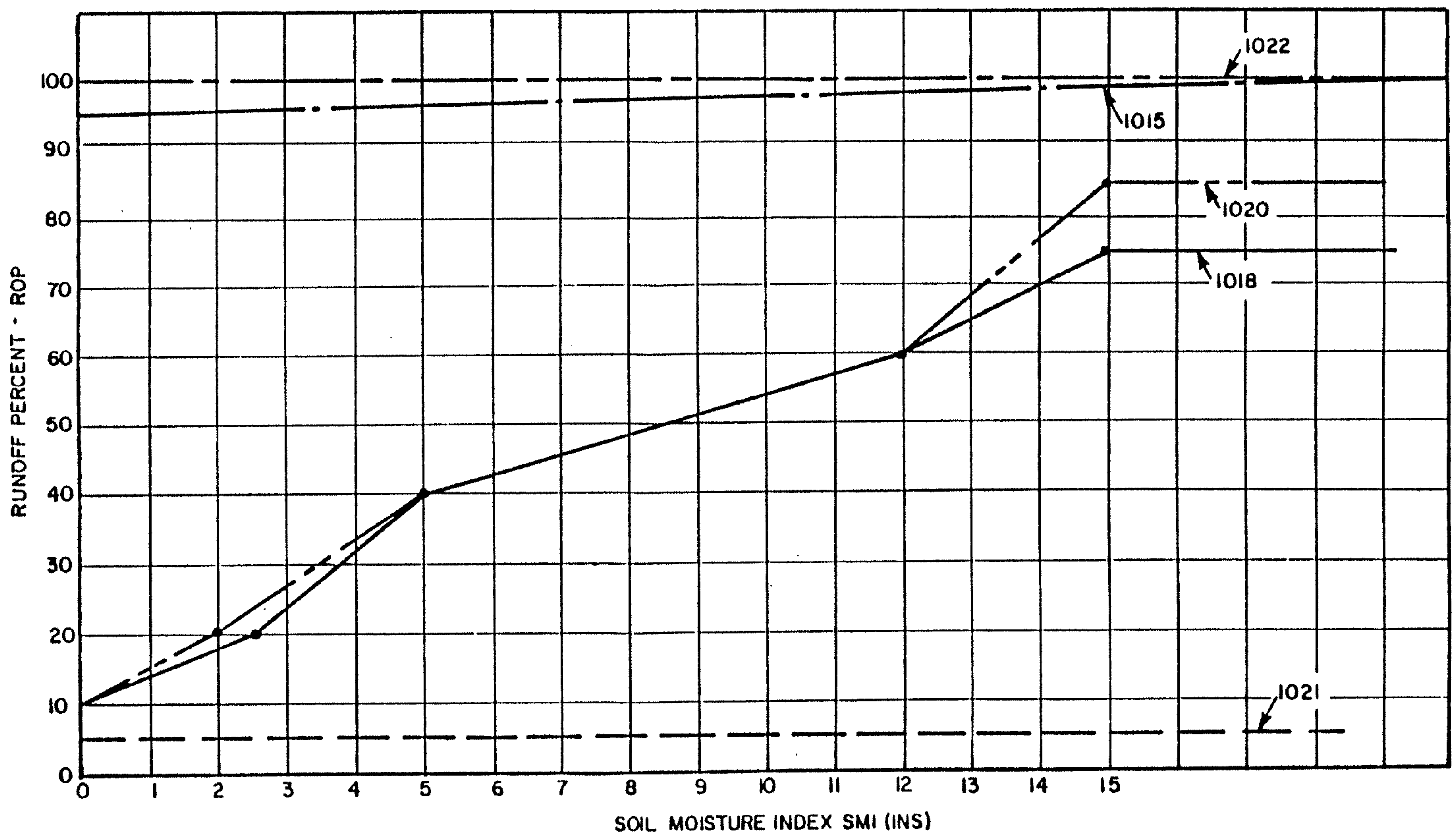


Figure F3.10

HYDROGRAPH MACLAREN RIVER NEAR PAXSON, 1967, 1972

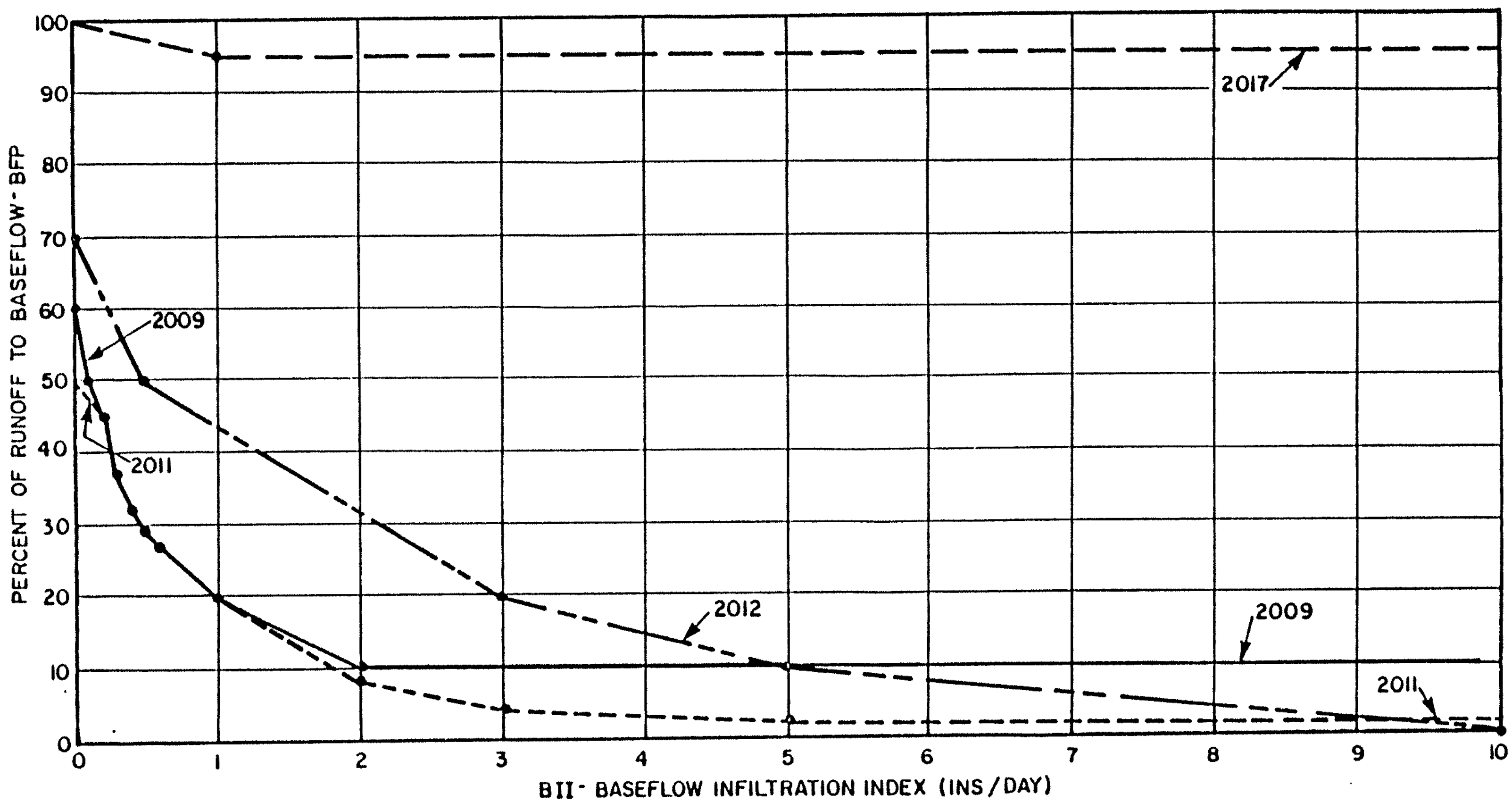
REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I



SSARR MODEL SMI VS ROP

Figure F3.11

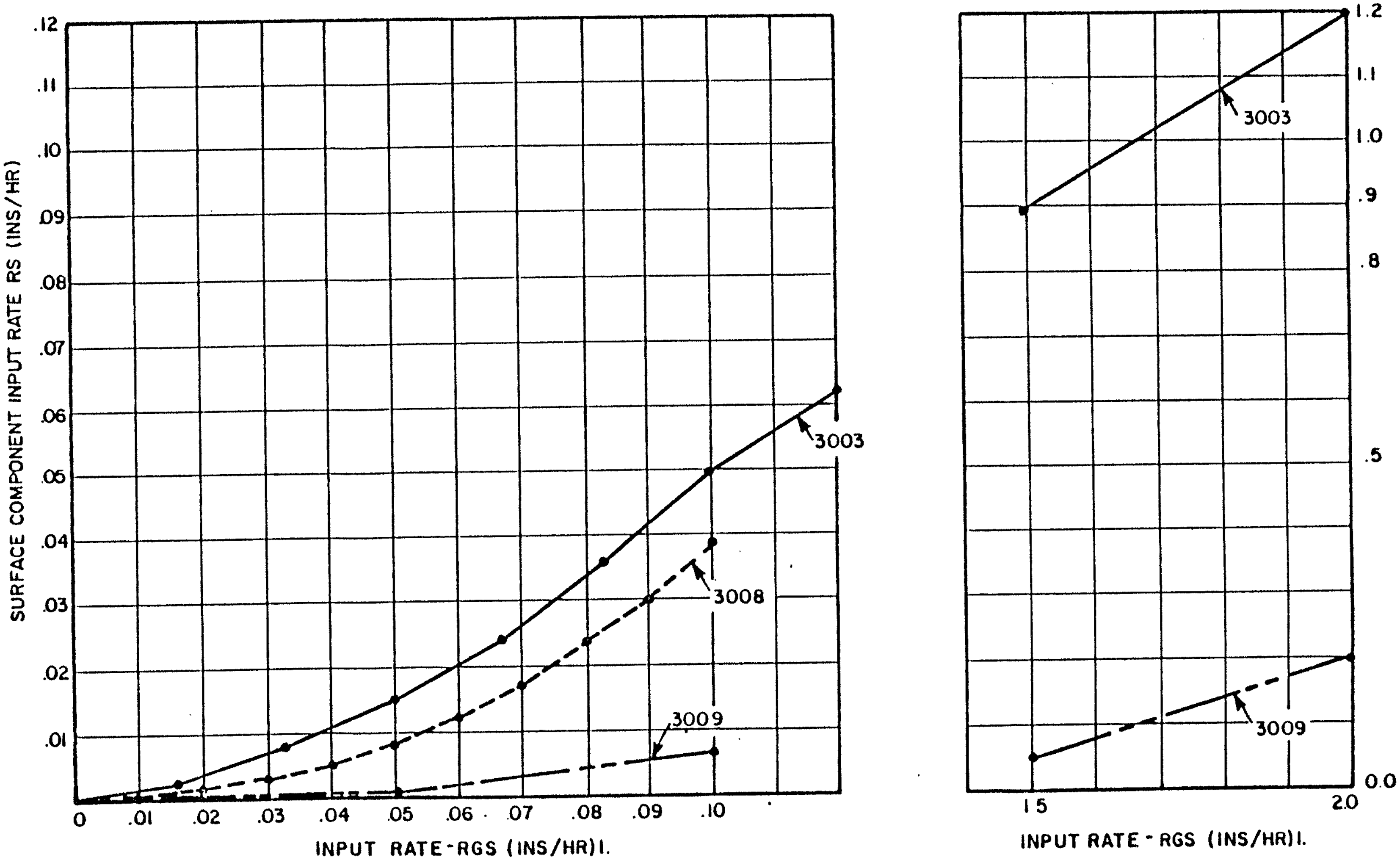
ACRES



SSARR MODEL BII VS BFP

Figure F3.12

ACRES



SSARR MODEL RGS VS RS

Figure F3.13

ACRES

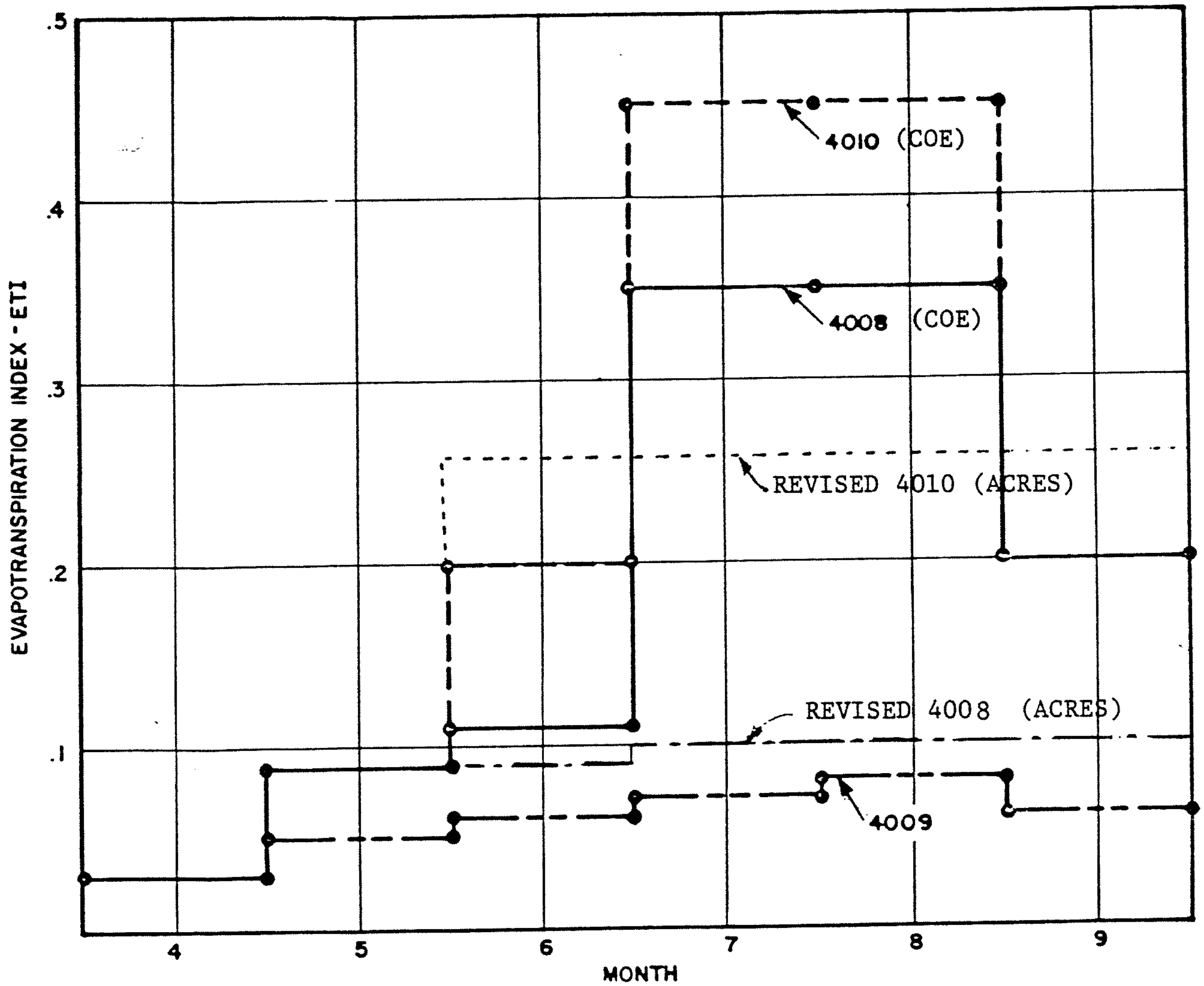


Figure F3.14

SSARR MODEL MONTH VS ETI

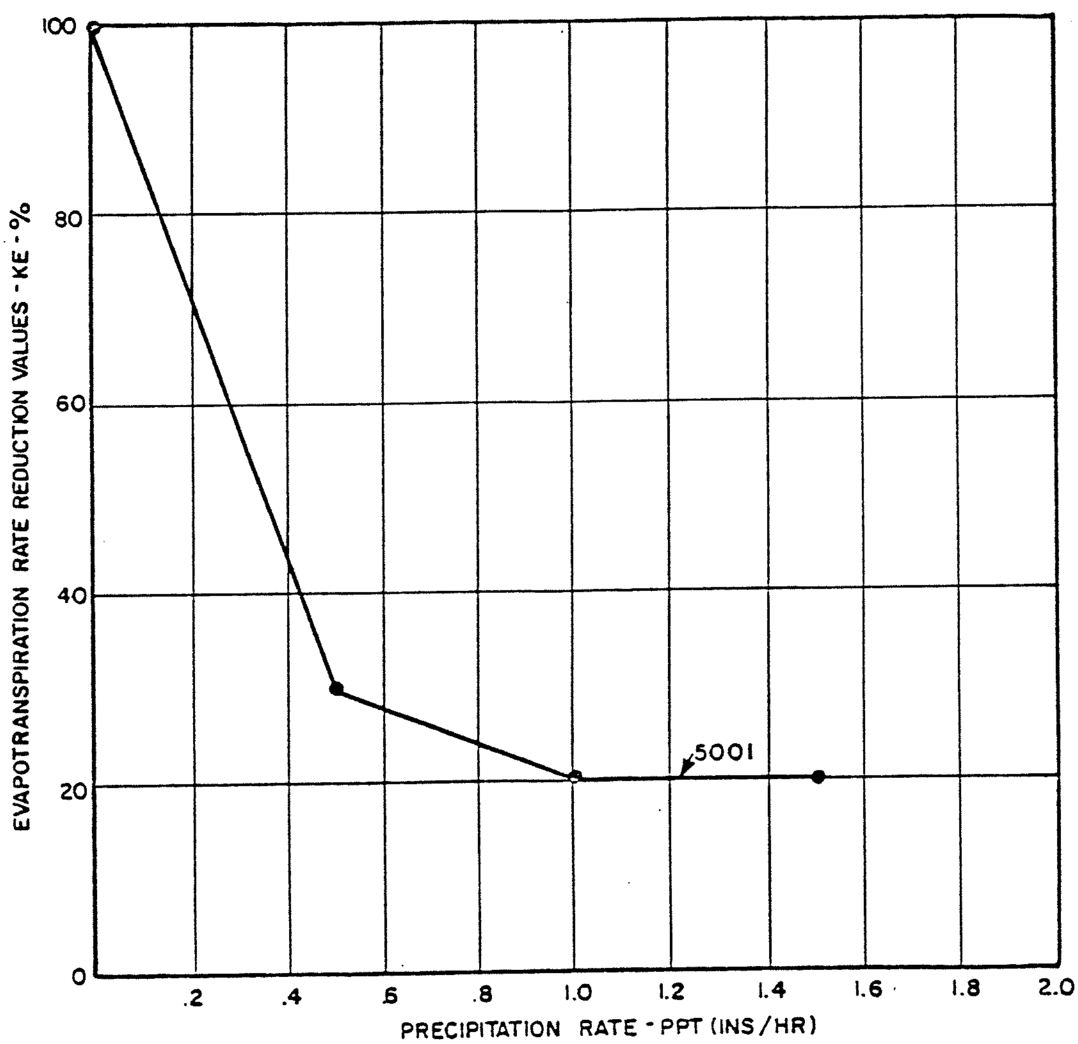
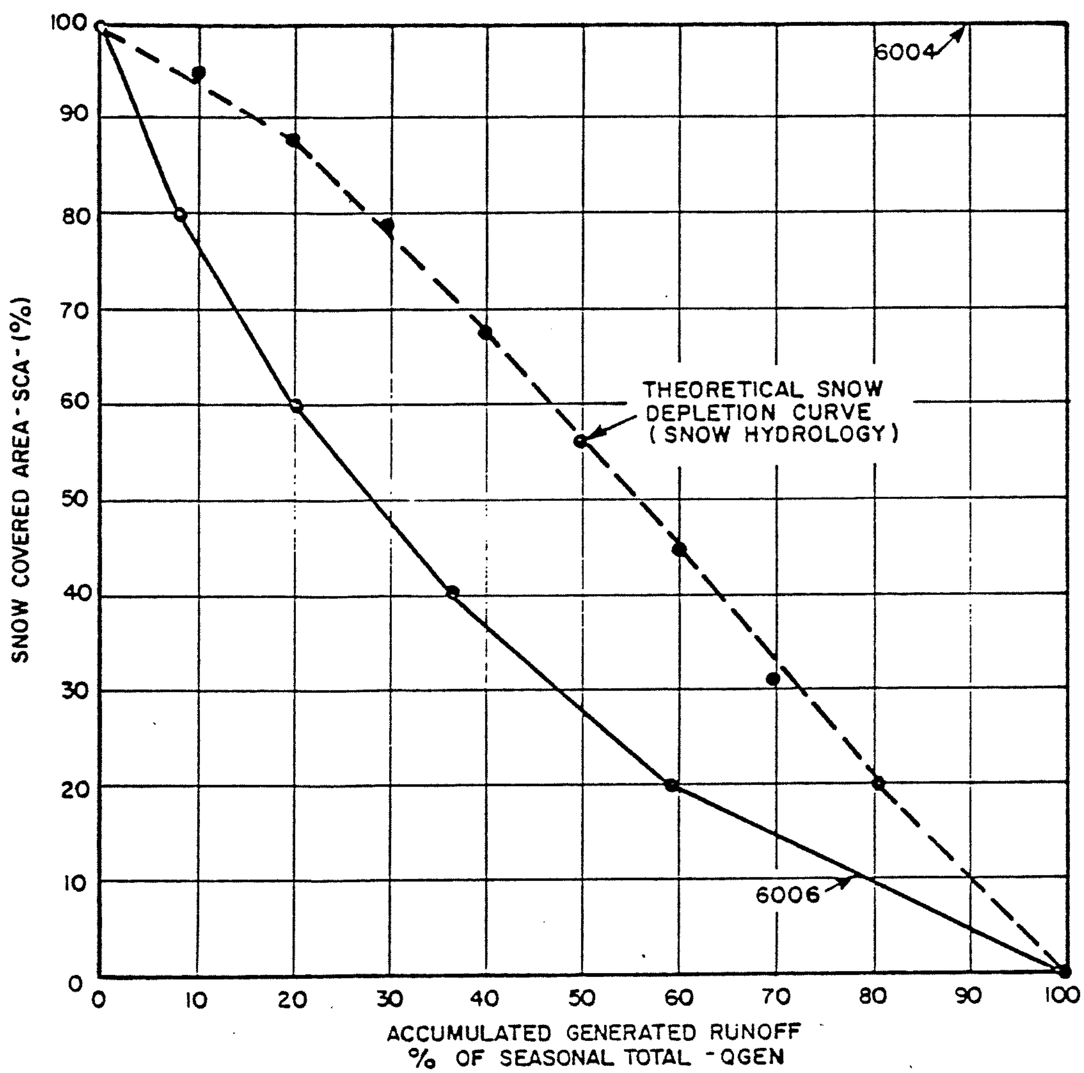


Figure F3.15

SSARR MODEL PPT VS KE

ACRES



SSARR MODEL QGEN VS SCA

Figure F3.16

ACRES

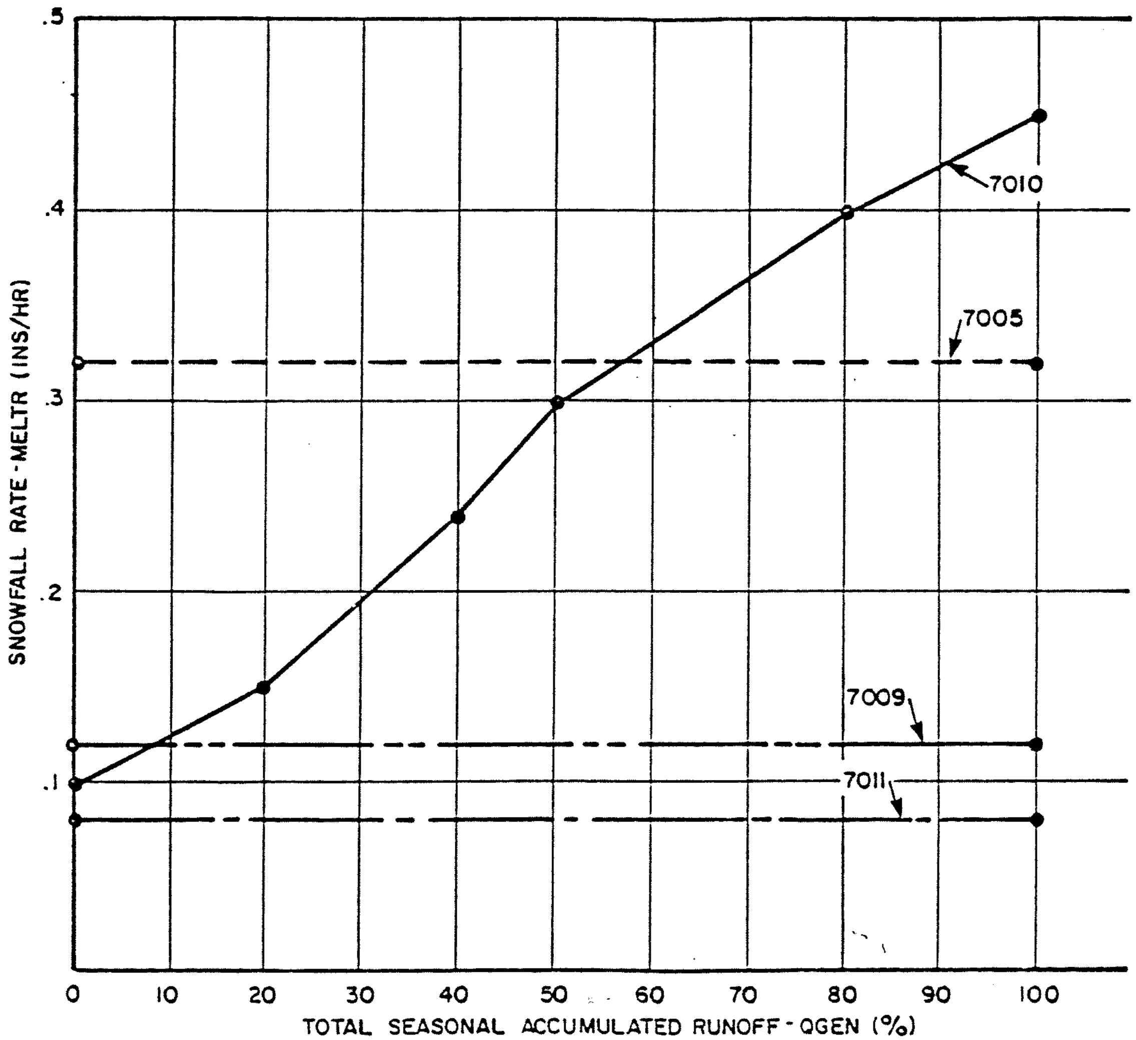


Figure F3.17

SSARR MODEL QGEN VS MELTR

ACRES

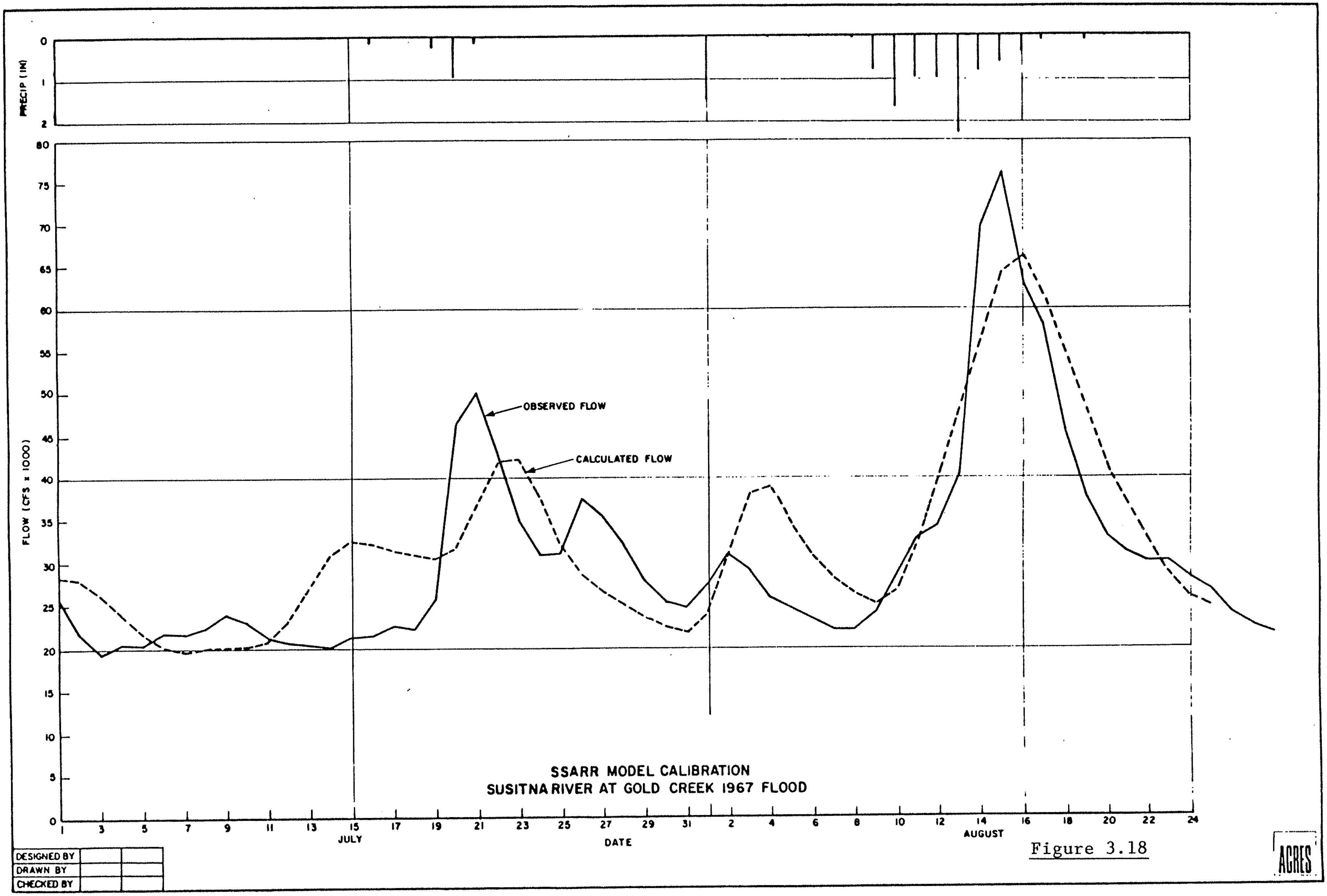
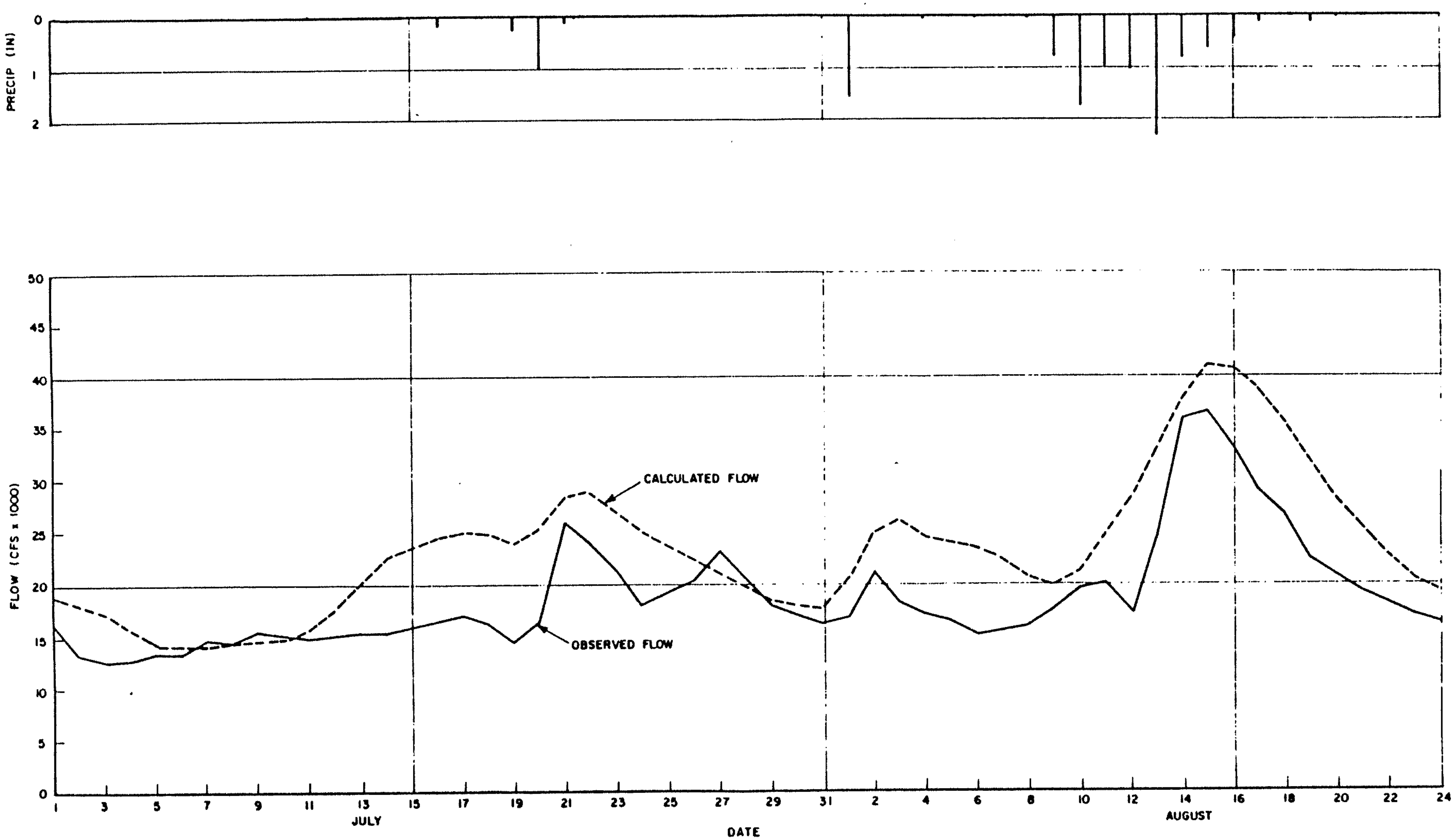

PRECIP (IN)
FLOW (CFS x 1000)
OBSERVED FLOW
CALCULATED FLOW
SSARR MODEL CALIBRATION
SUSITNA RIVER AT GOLD CREEK 1967 FLOOD
JULY
DATE
AUGUST
DESIGNED BY
DRAWN BY
CHECKED BY
ACRES

Figure 3.18



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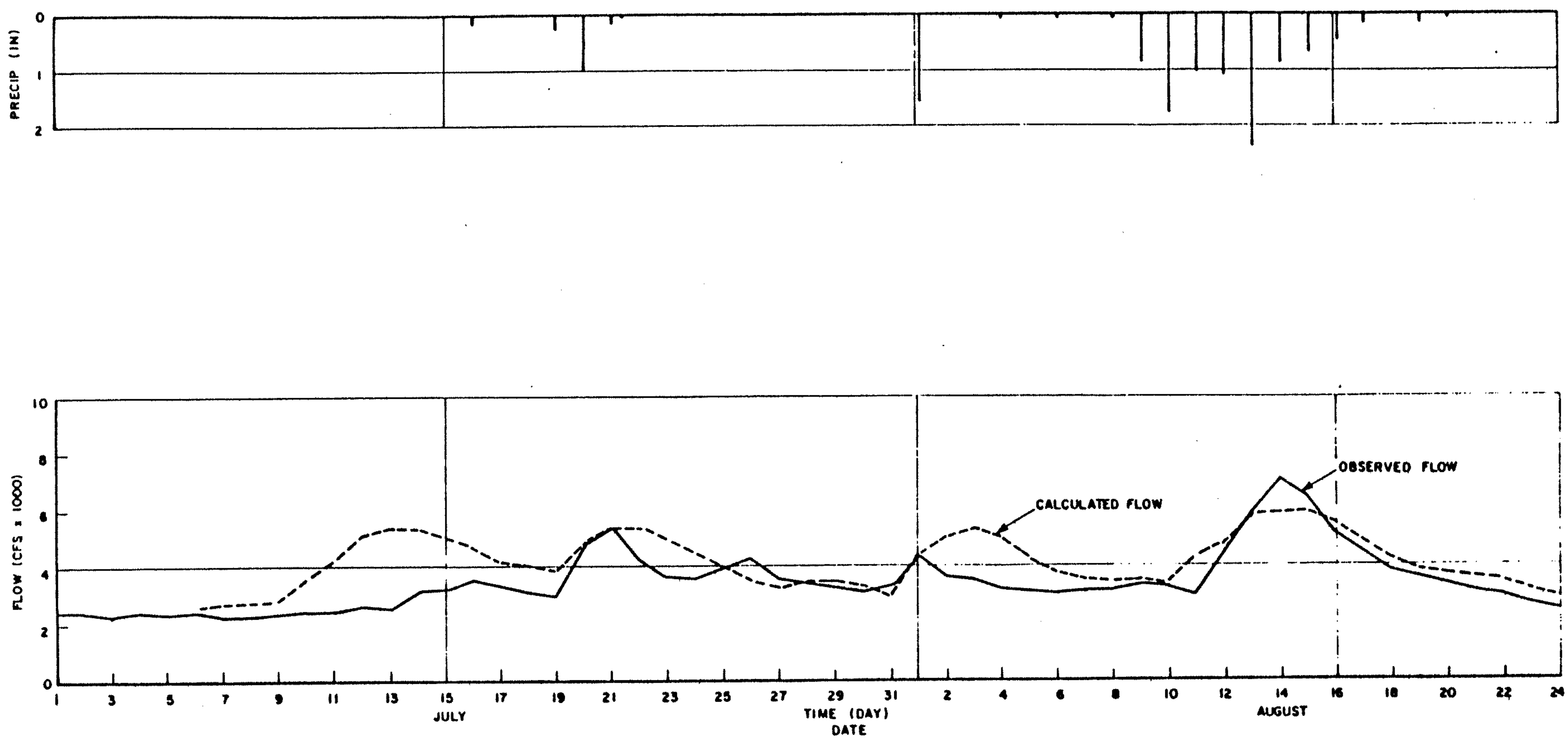
SSARR MODEL CALIBRATION
SUSITNA RIVER NEAR CANTWELL 1967 FLOOD

Figure F3.19

ACRES



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SSARR MODEL CALIBRATION
MACLAREN RIVER NEAR PAXSON 1967 FLOOD

Figure F3.20

ACRES

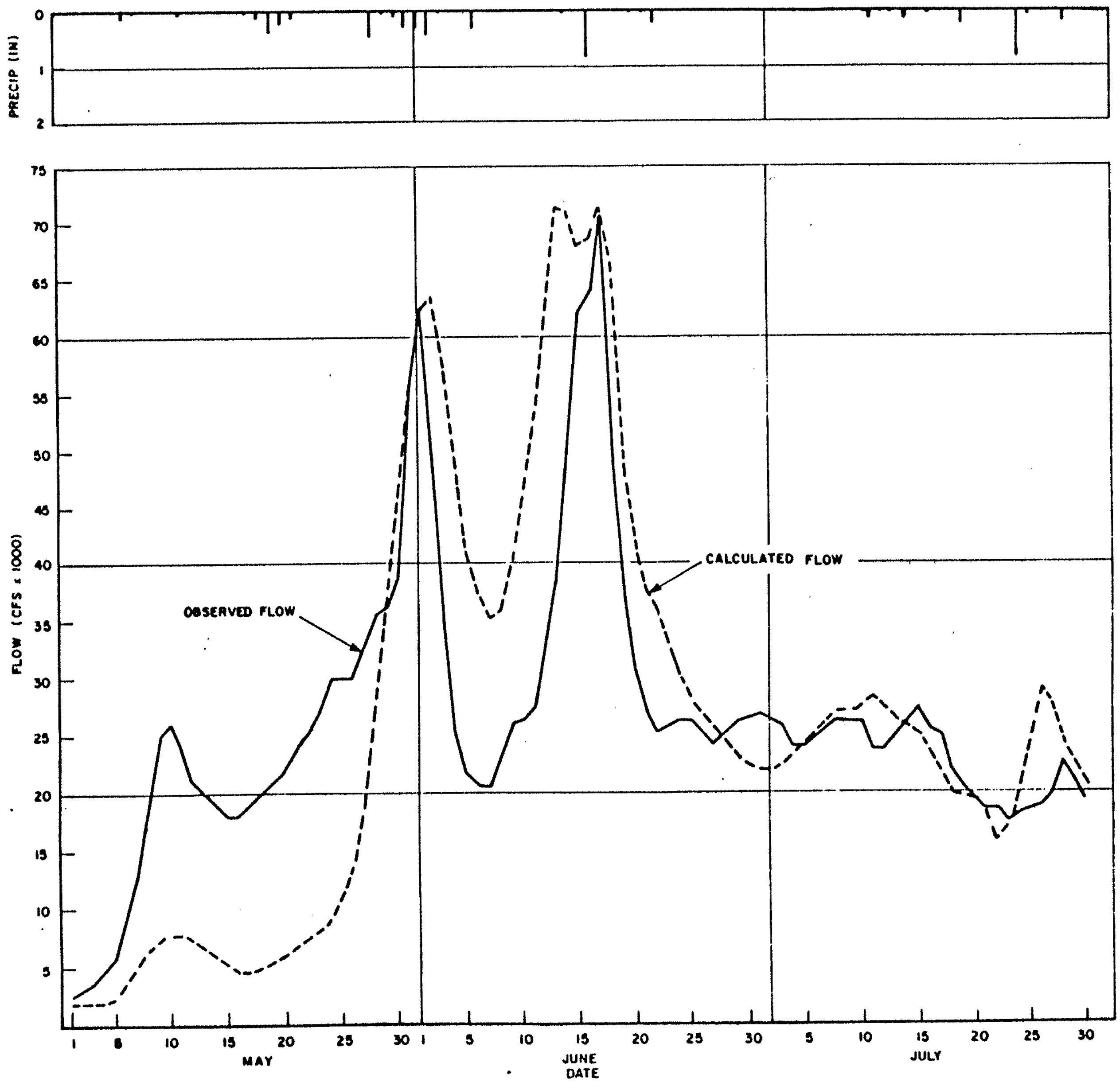

Figure F3.21

SSARR MODEL CALIBRATION
SUSITNA RIVER AT GOLD CREEK 1972 FLOOD

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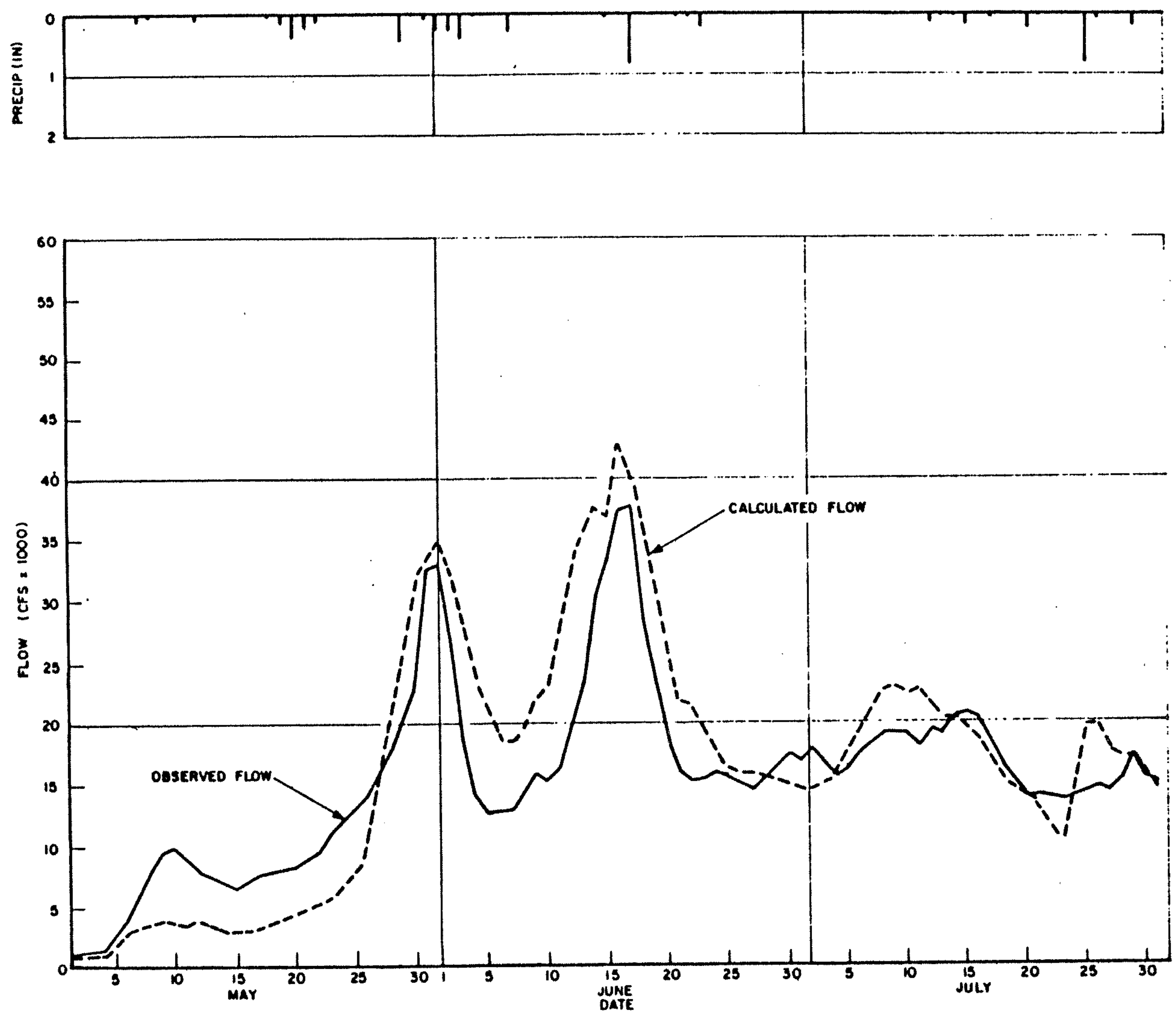
ACRES



SSARR MODEL CALIBRATION
SUSITNA RIVER NEAR CANTWELL 1972 FLOOD

Figure F3.22

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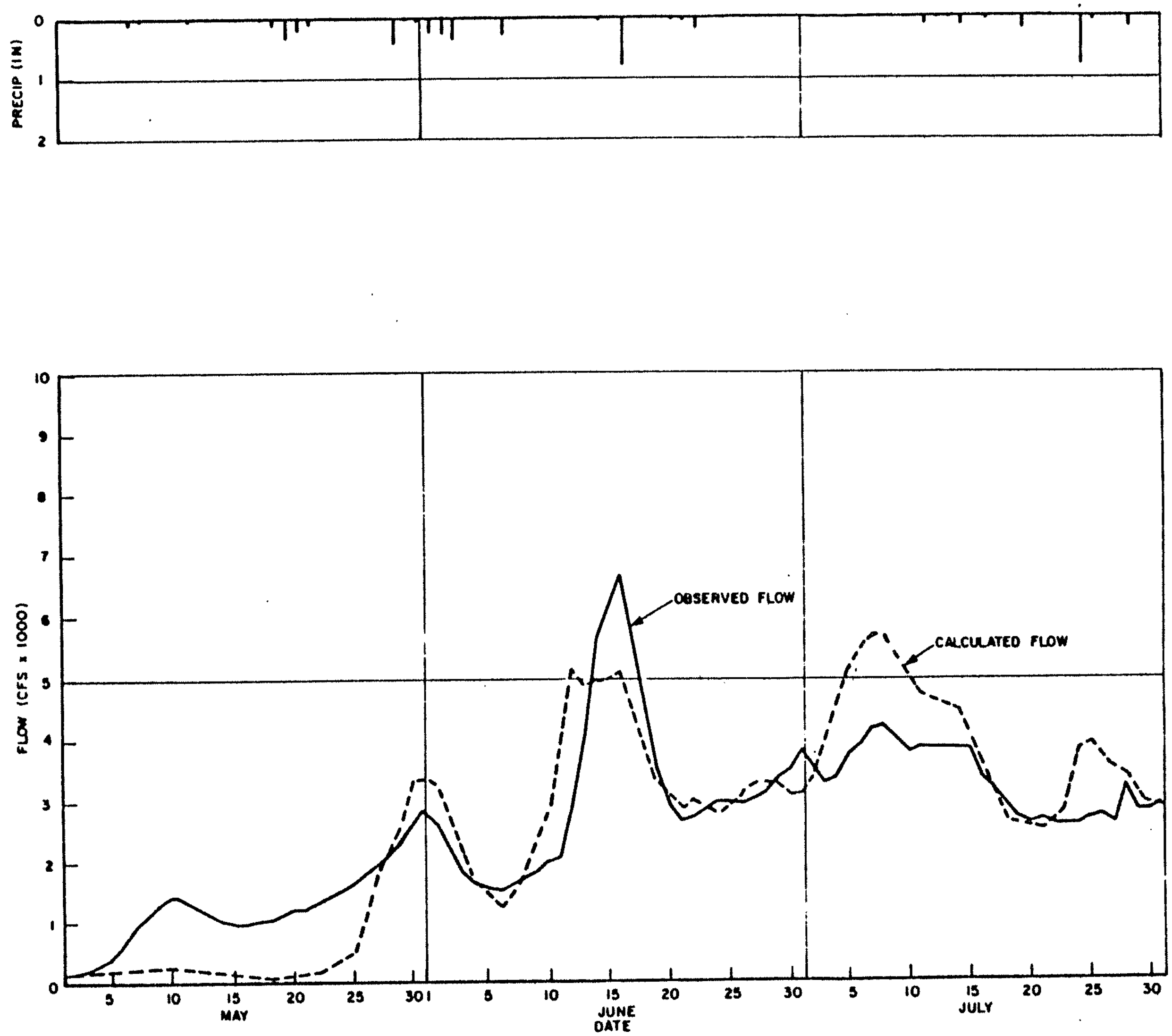
ACRES



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DRAWN BY		
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SSARR MODEL CALIBRATION
MACLAREN RIVER NEAR PAXSON 1972 FLOOD

Figure F3.23

ACRES

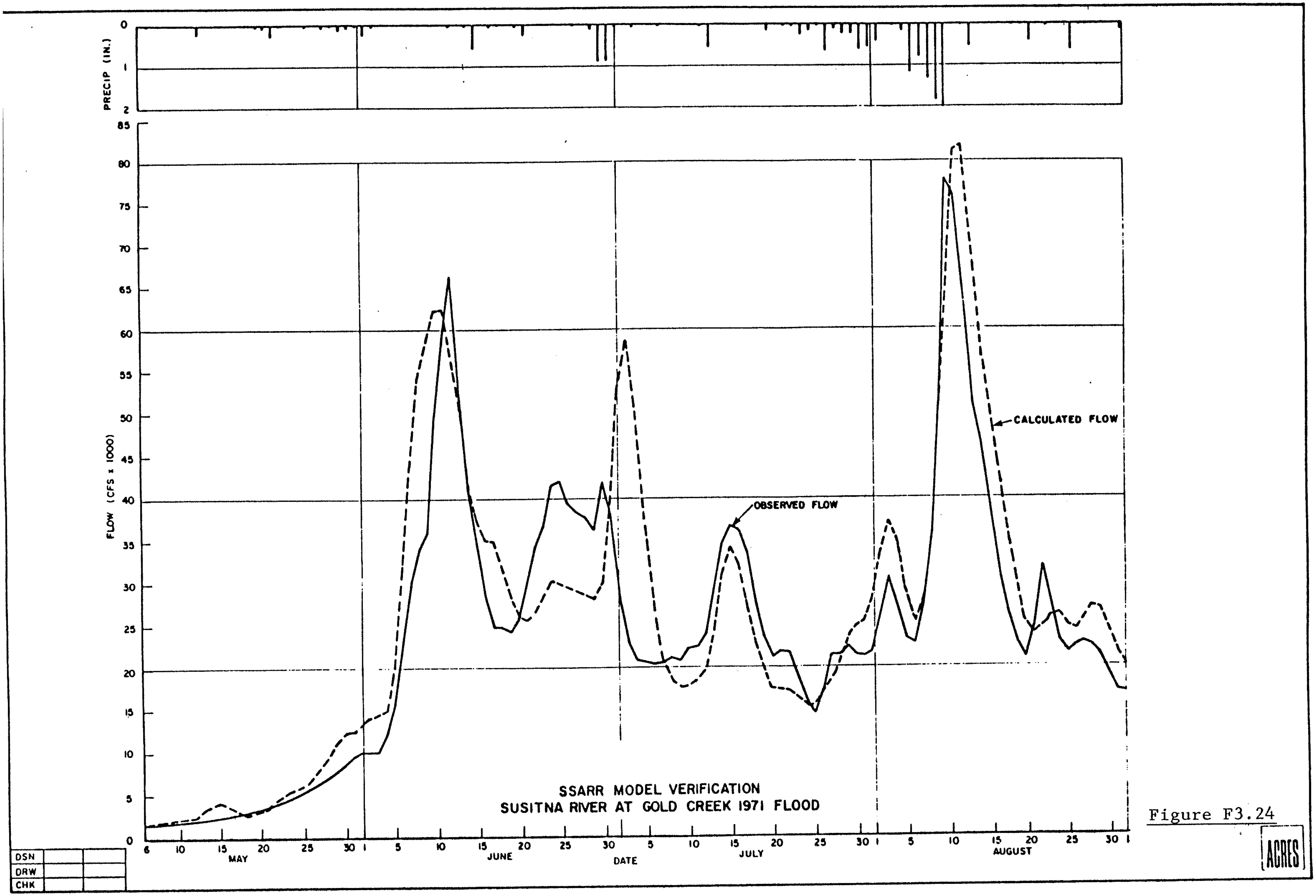
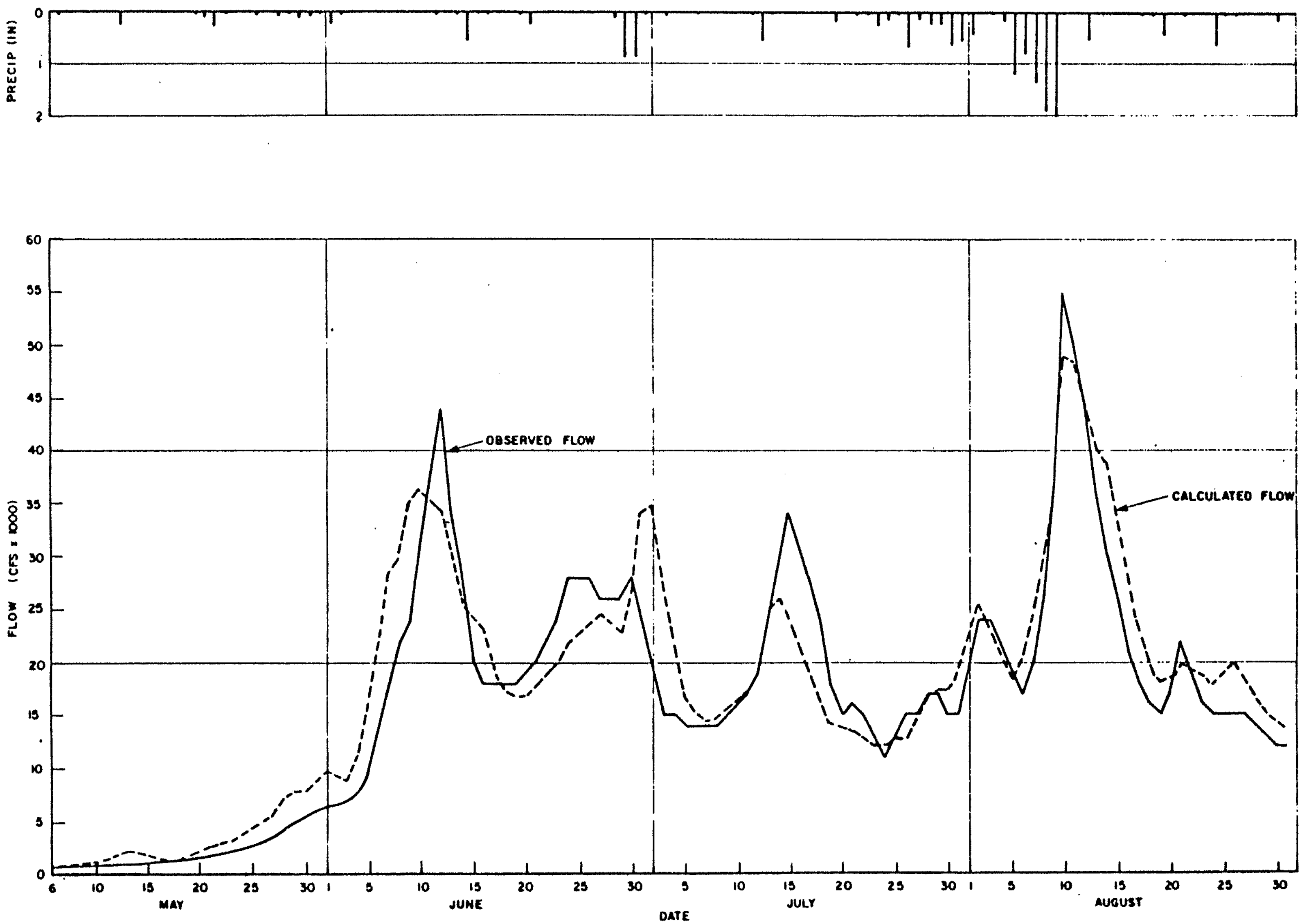


Figure F3.24

ACRES



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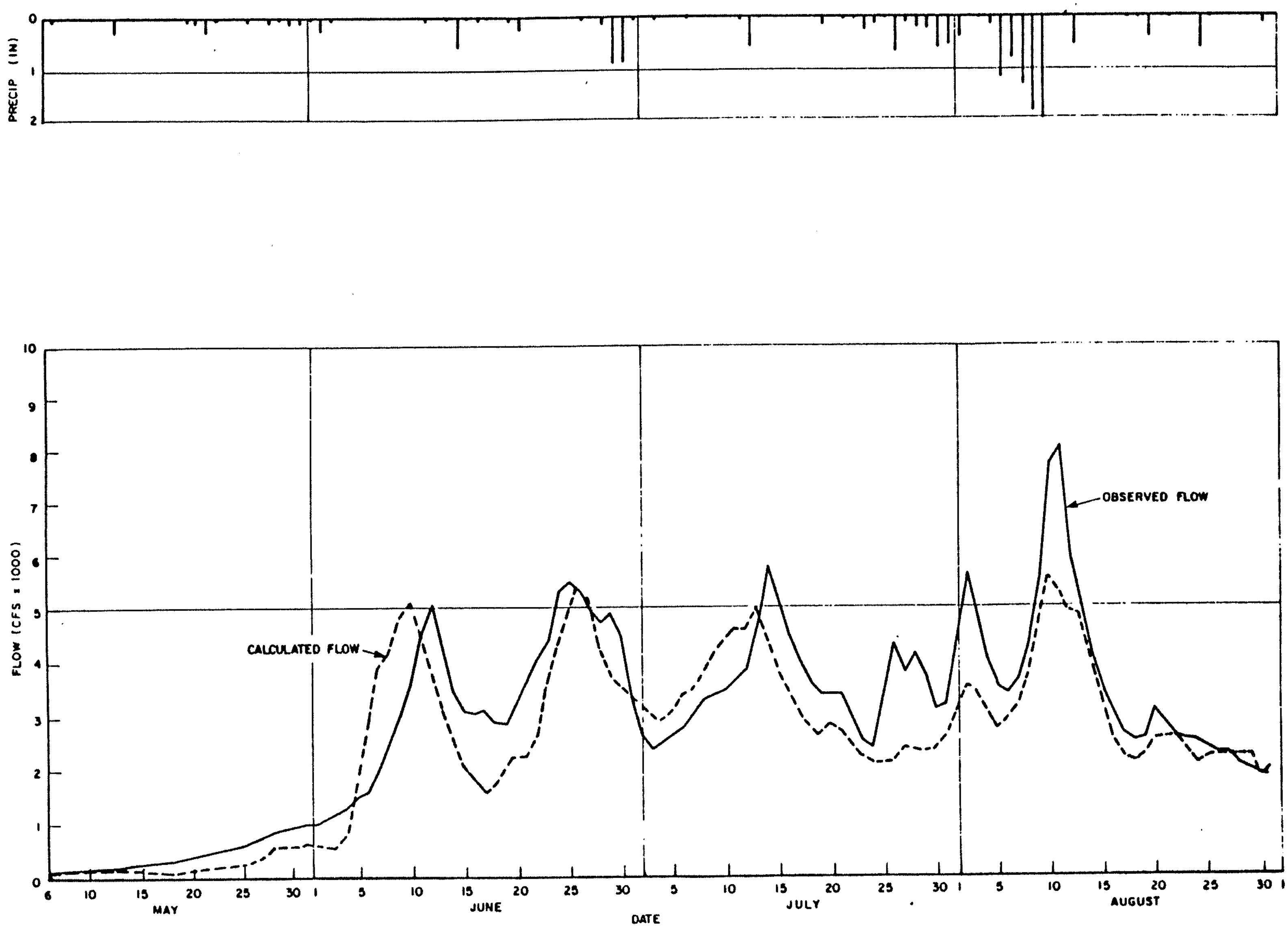
SSARR MODEL VERIFICATION
SUSITNA RIVER NEAR CANTWELL 1971 FLOOD

Figure F3.25

ACRES



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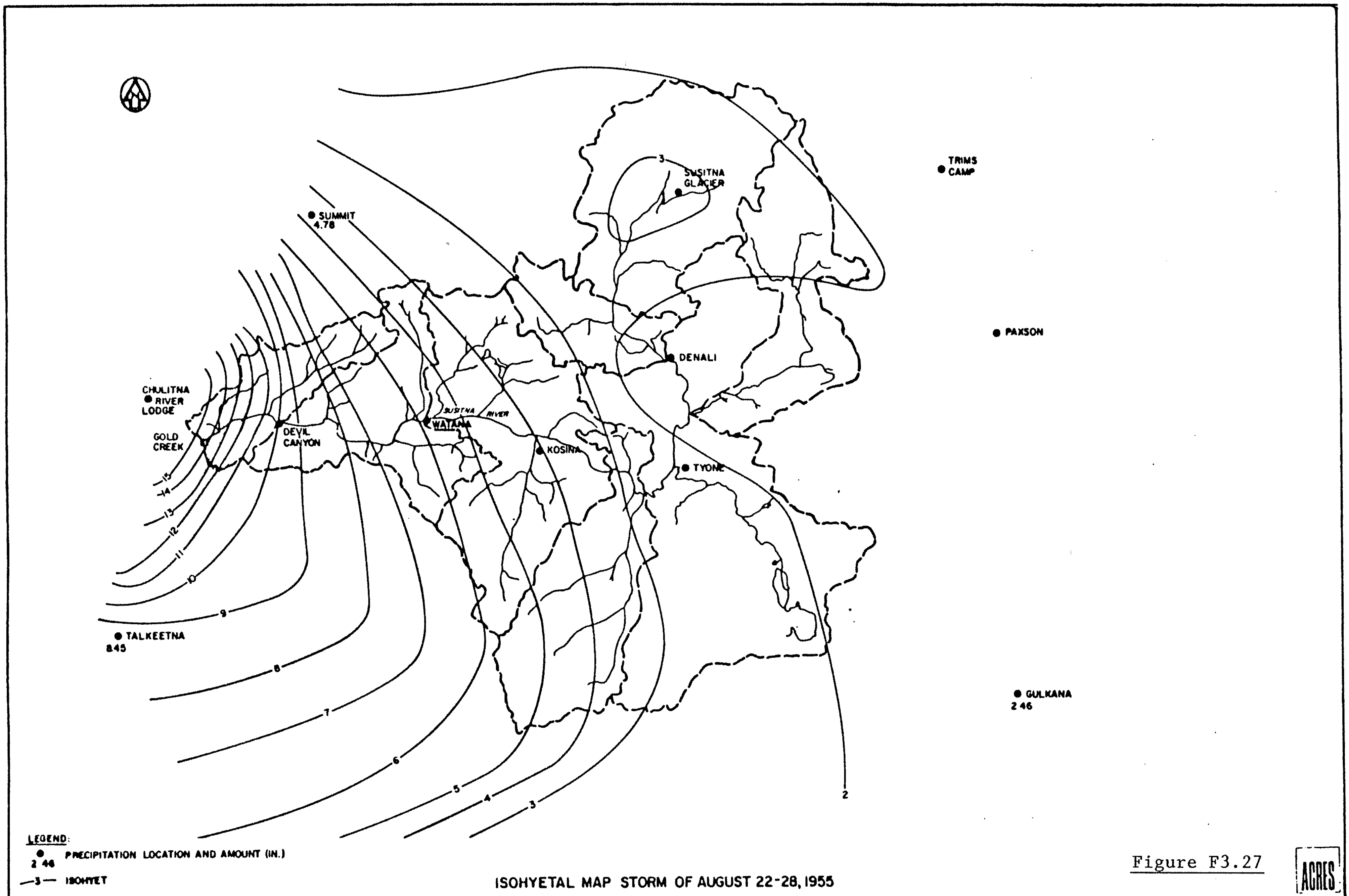
SSARR MODEL VERIFICATION
MACLAREN RIVER NEAR PAXSON 1971 FLOOD

Figure F3.26

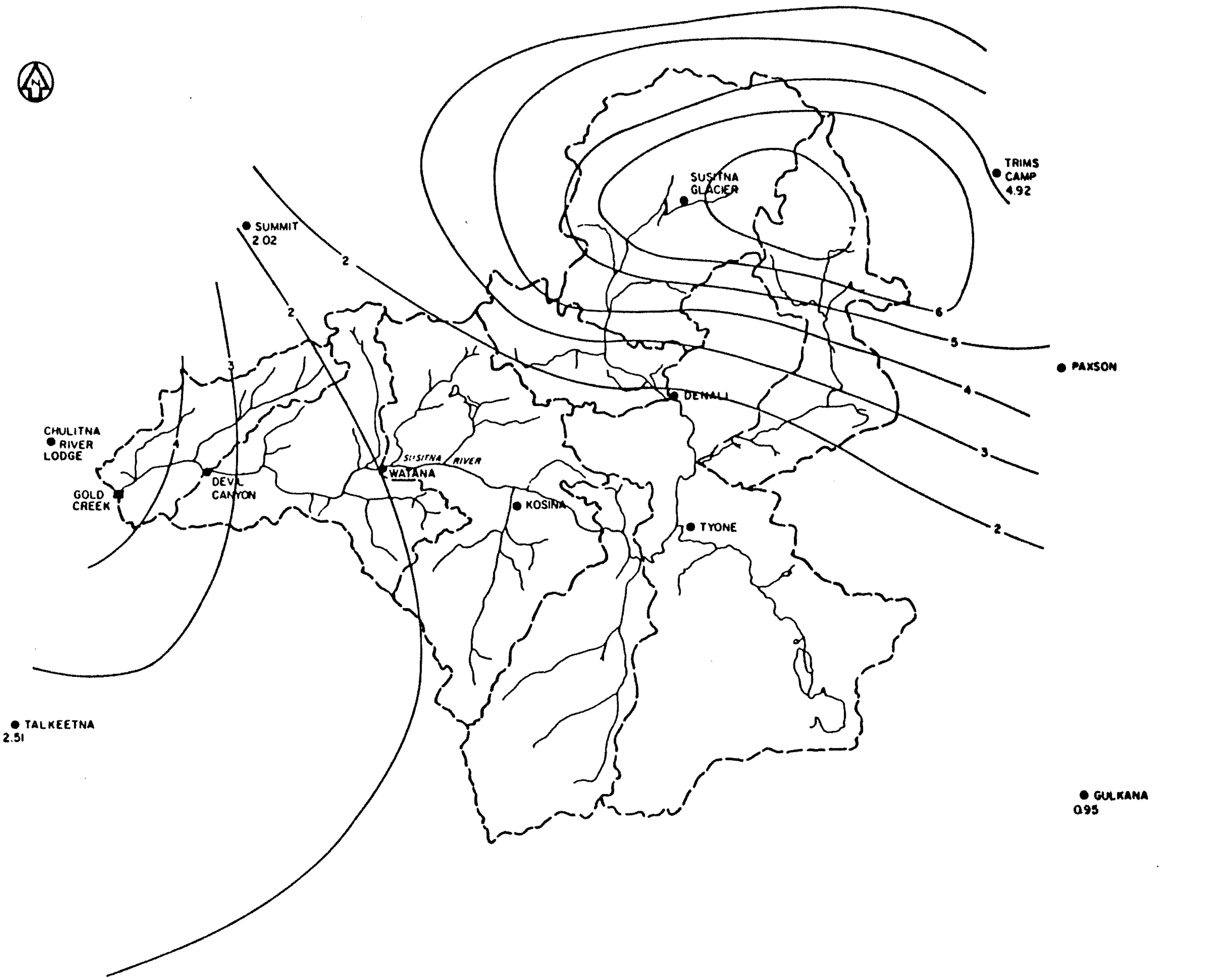
ACRES



ISOHYETAL MAP STORM OF AUGUST 22-28, 1955

Figure F3.27

ACRES

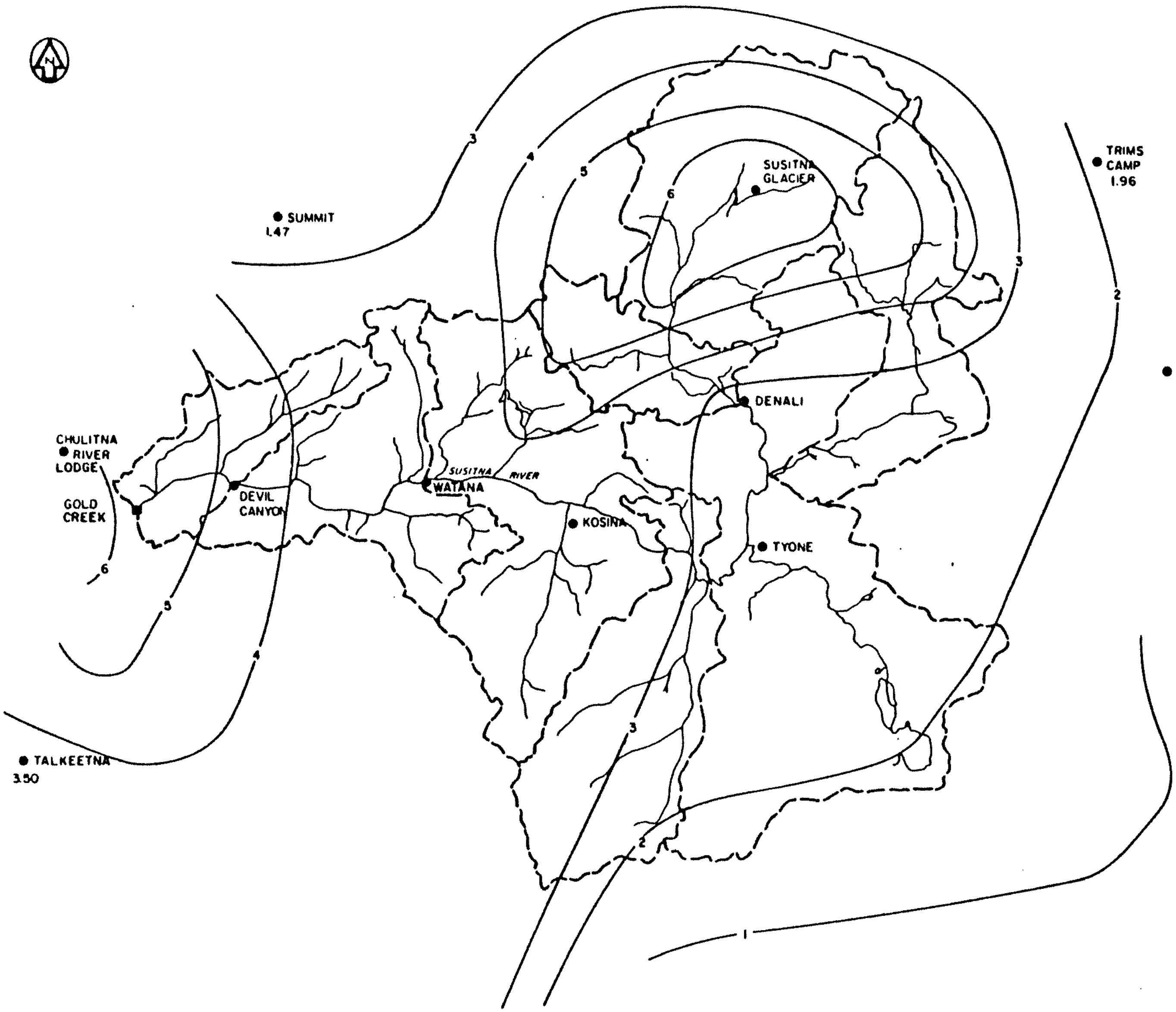


LEGEND
● 2.51 PRECIPITATION LOCATION AND AMOUNT (IN.)
—3— ISOHYET

ISOHYETAL MAP STORM OF JULY 28 - AUGUST 3, 1958

Figure F3.28

ACRES



LEGEND:

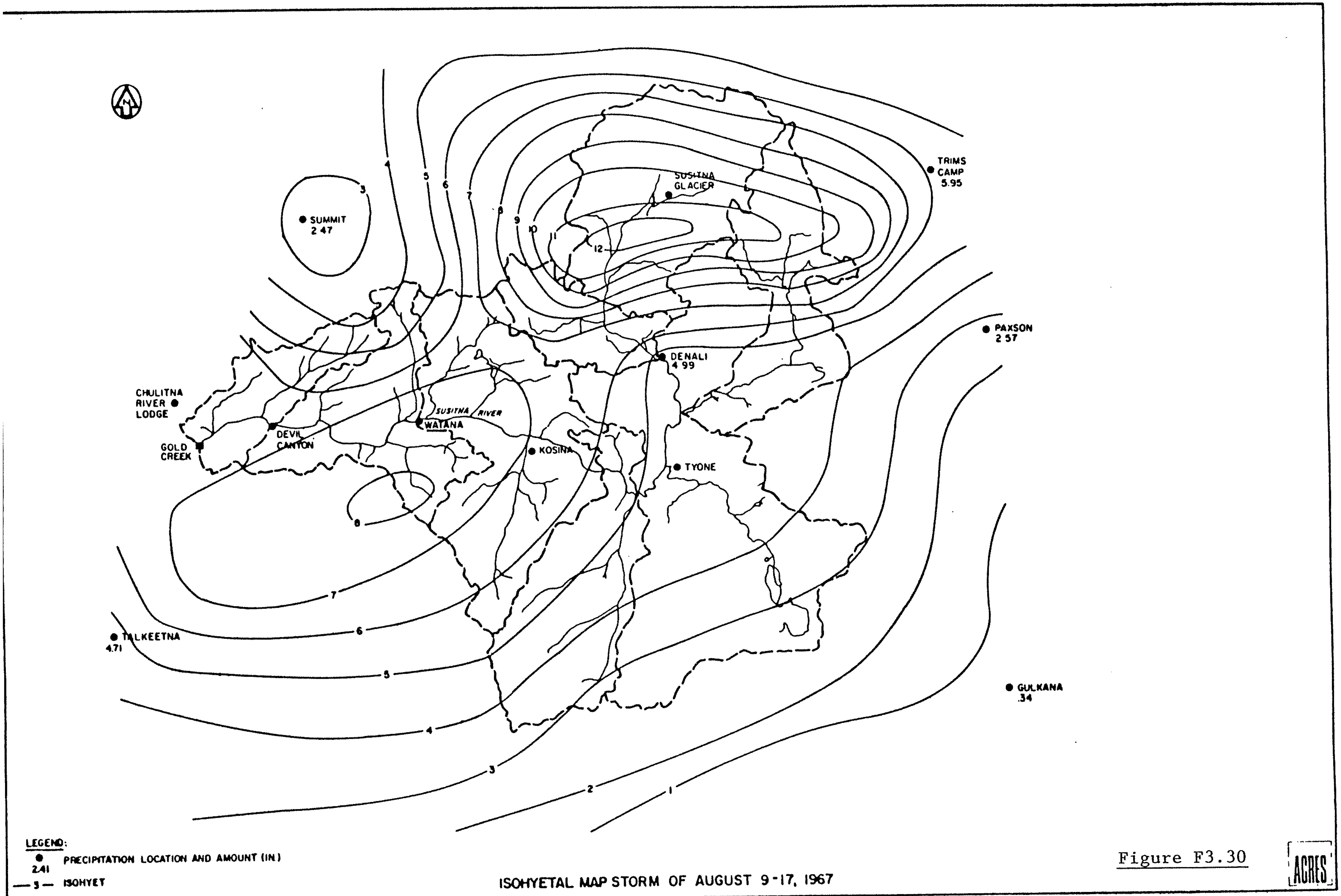
● 3.50 PRECIPITATION LOCATION AND AMOUNT (IN)

—3— ISOHYET

ISOHYETAL MAP STORM OF AUGUST 19-25, 1959

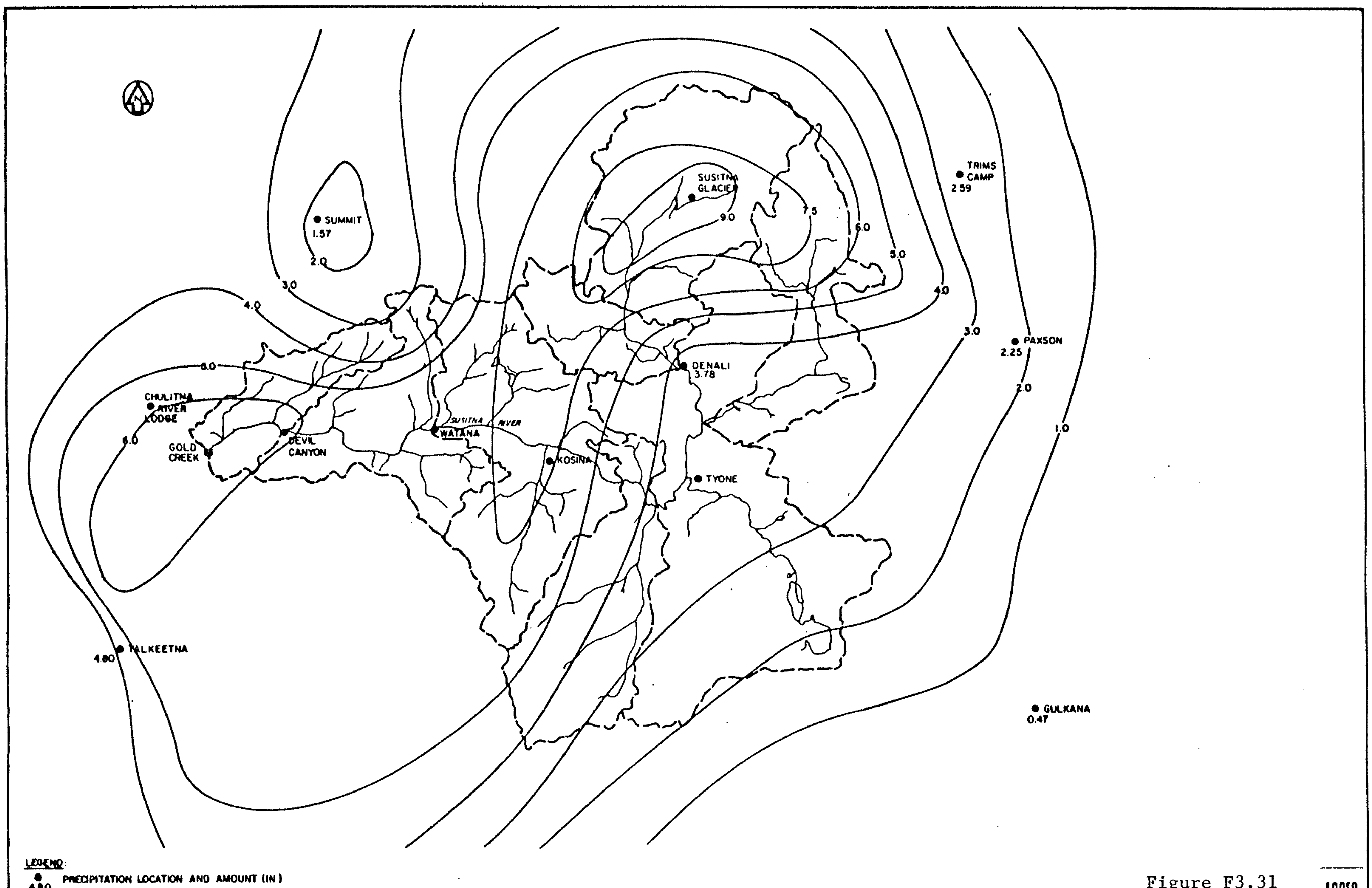
Figure F3.29

ACRES



ISOHYETAL MAP STORM OF AUGUST 9-17, 1967

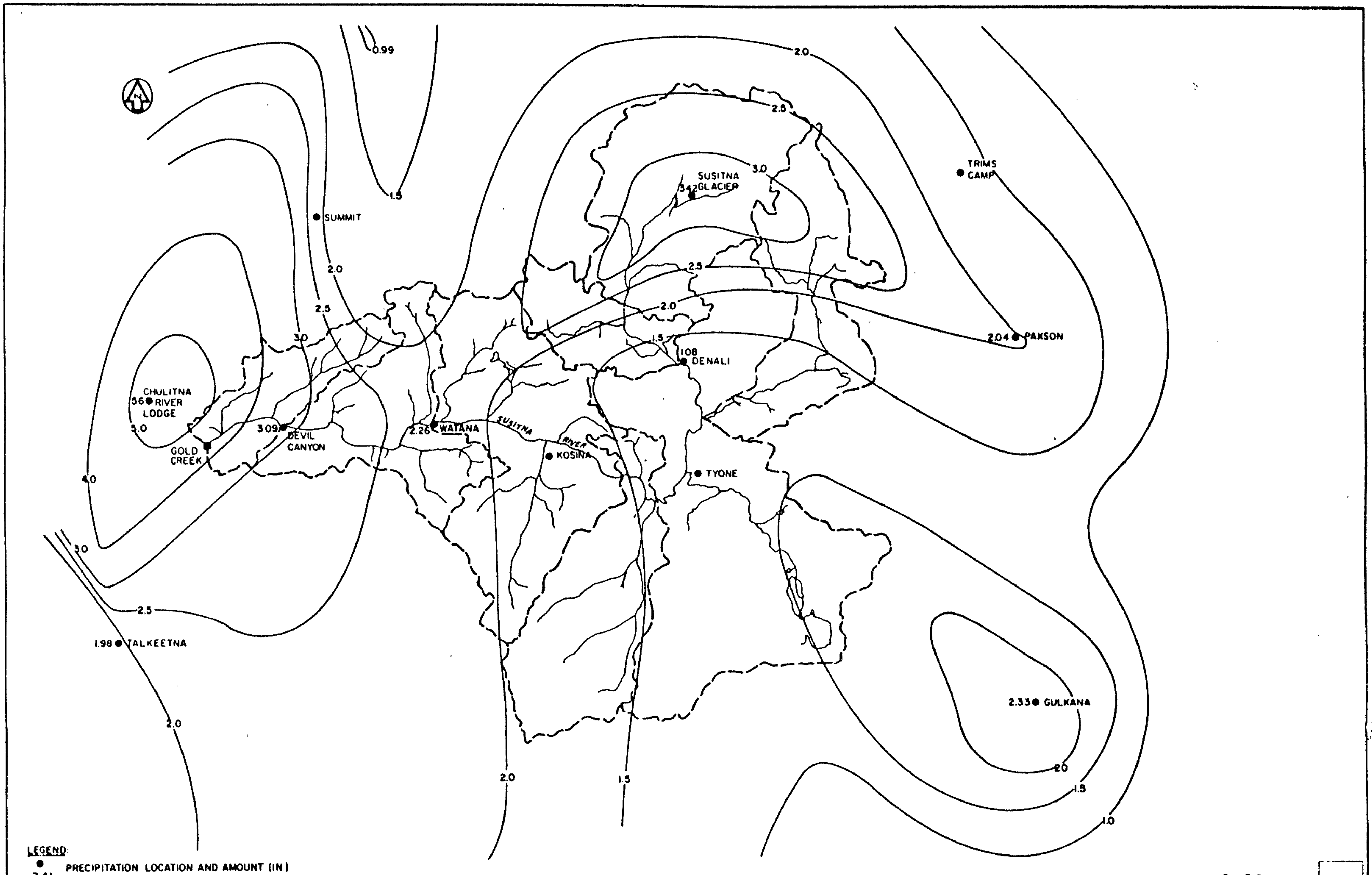
Figure F3.30



ISOHYETAL MAP STORM OF AUGUST 4-10, 1971

Figure F3.31

ACRES



ISOHYETAL MAP STORM OF JULY 25-31, 1980

Figure F3.32

ACRES

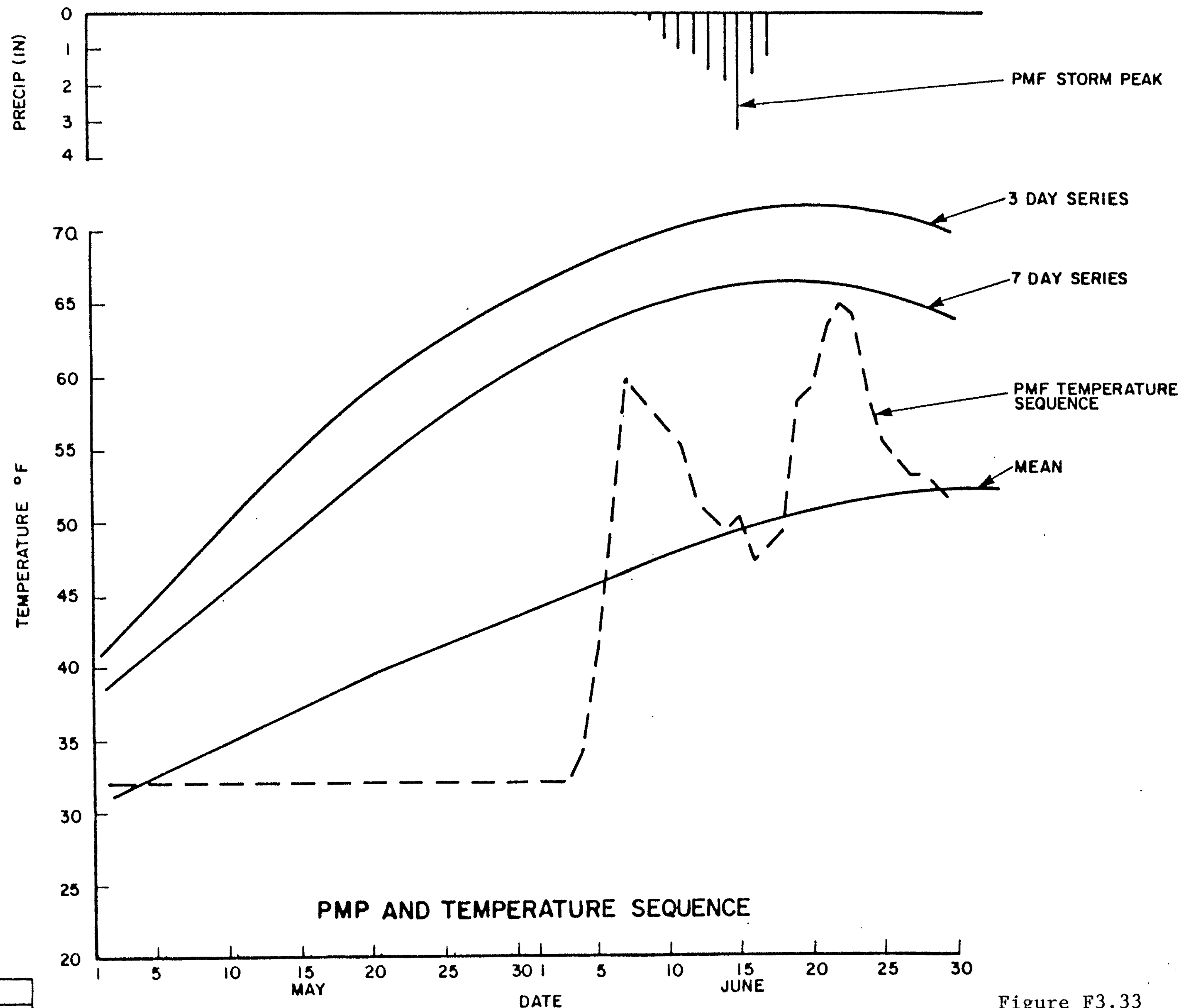


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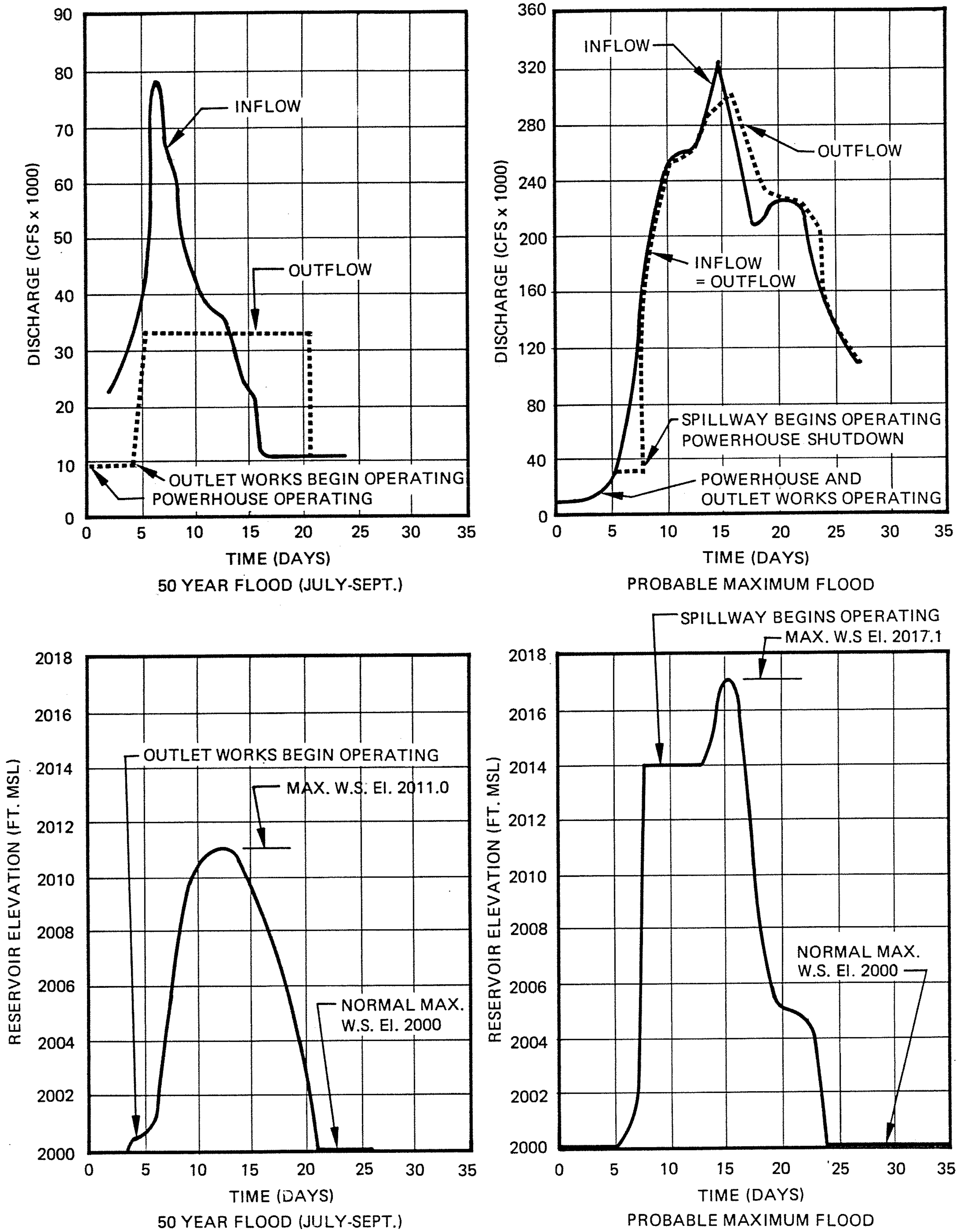
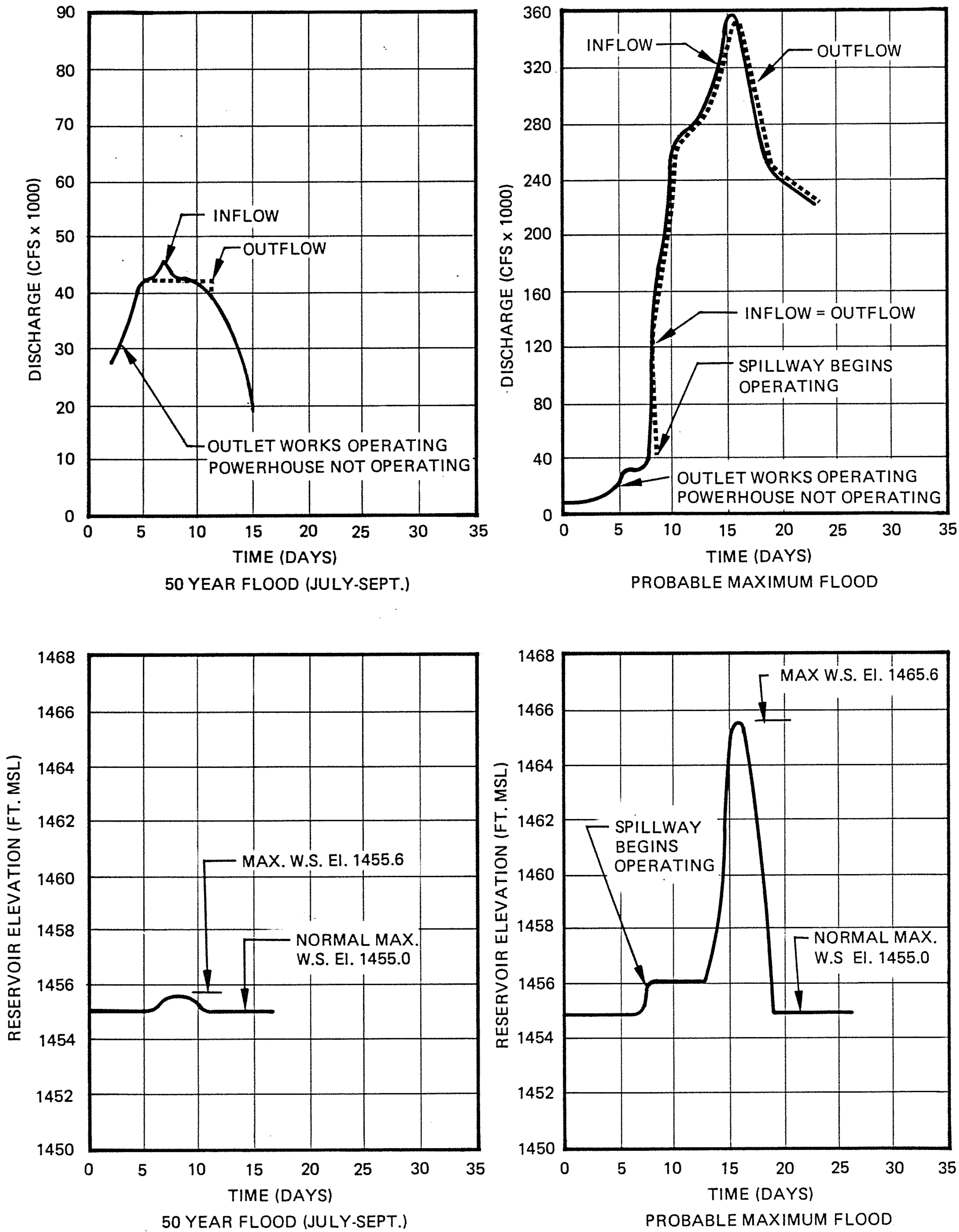
Figure F3.33

ACRES



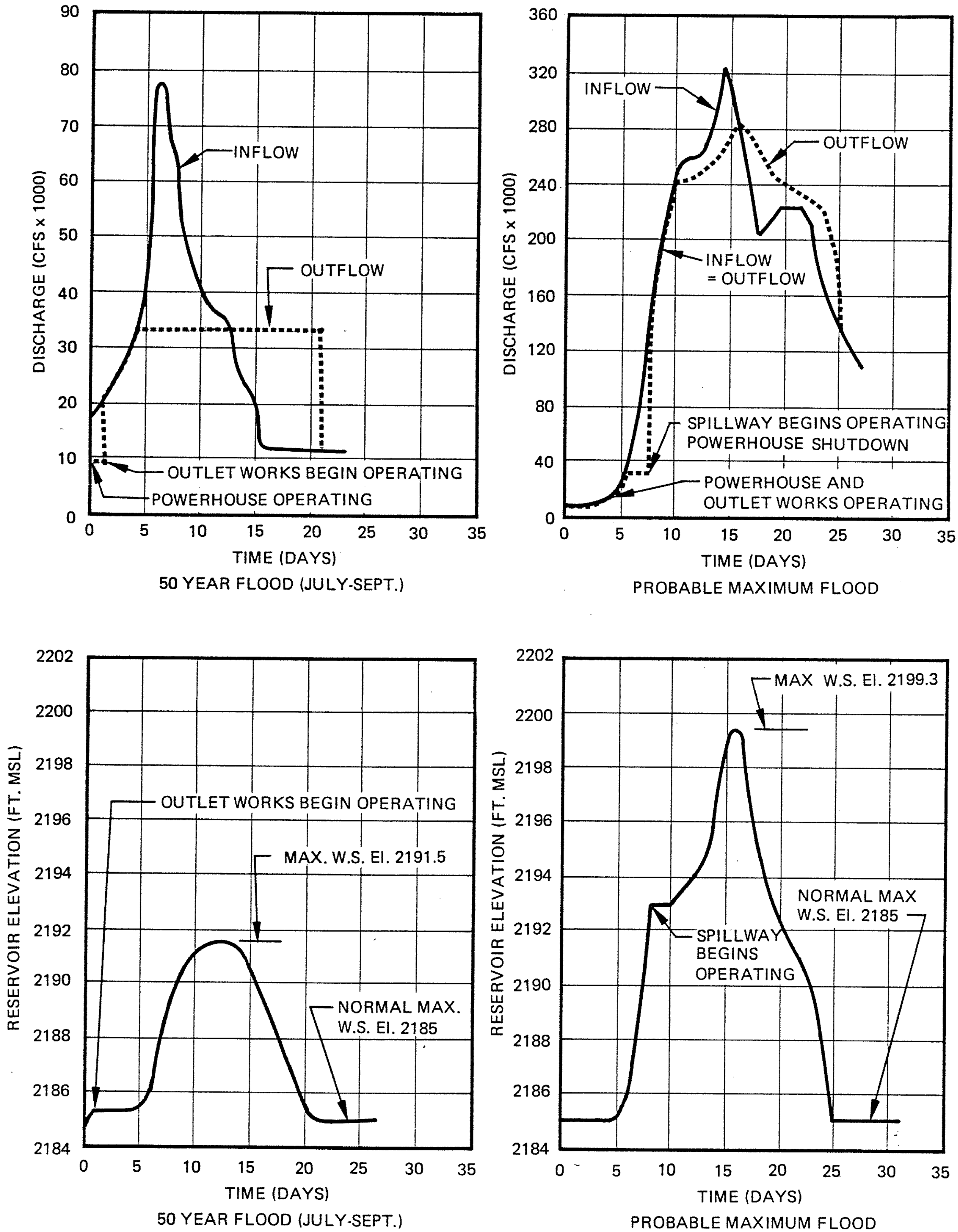
WATANA STAGE I
FLOOD DISCHARGES AND RESERVOIR SURFACE ELEVATIONS

FIGURE F3.34

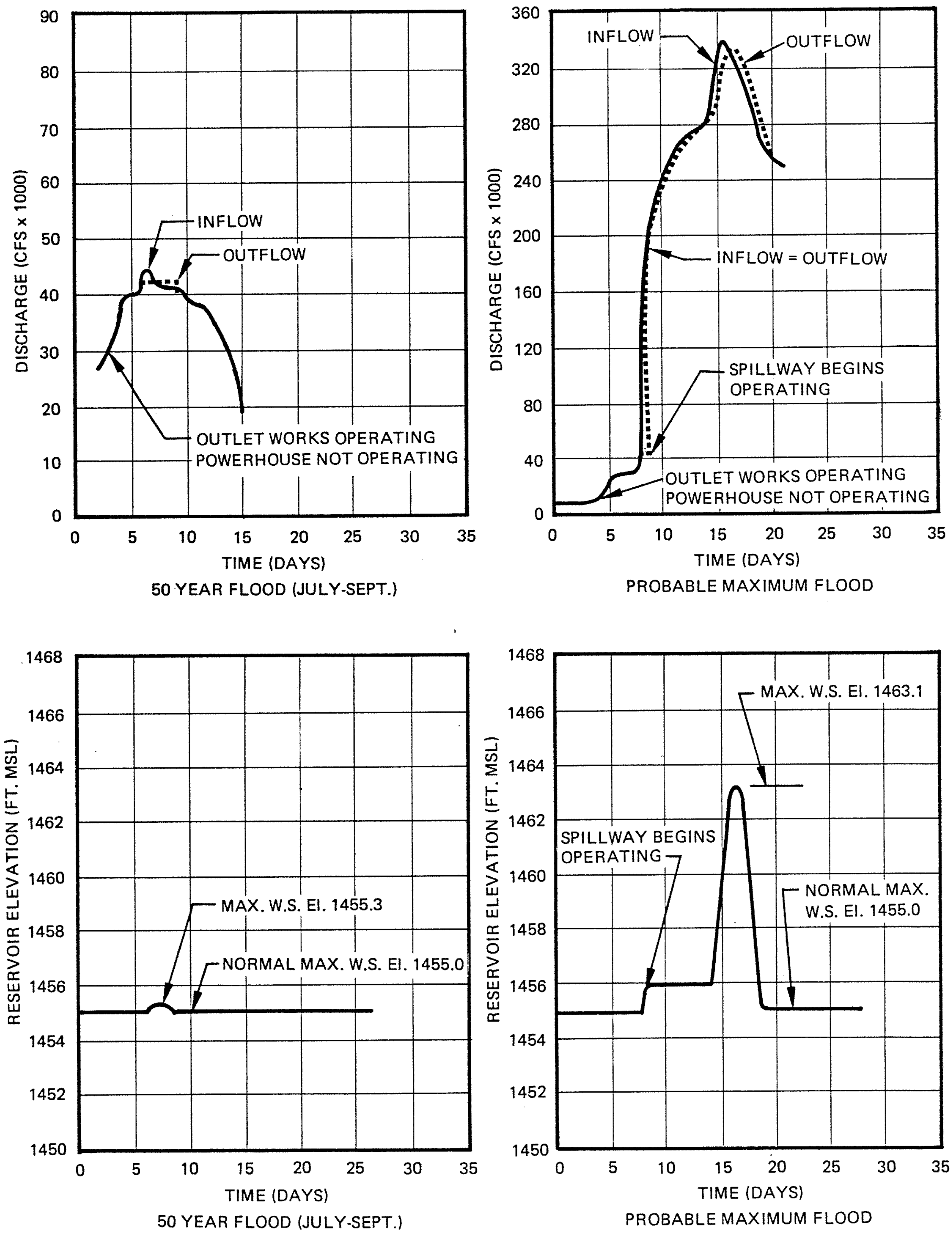


DEVIL CANYON STAGE II
FLOOD DISCHARGES AND RESERVOIR SURFACE ELEVATIONS

FIGURE F3.35



WATANA STAGE III
DISCHARGES AND RESERVOIR SURFACE ELEVATIONS



DEVIL CANYON STAGE III

FLOOD DISCHARGES AND RESERVOIR SURFACE ELEVATIONS

FIGURE E3.37