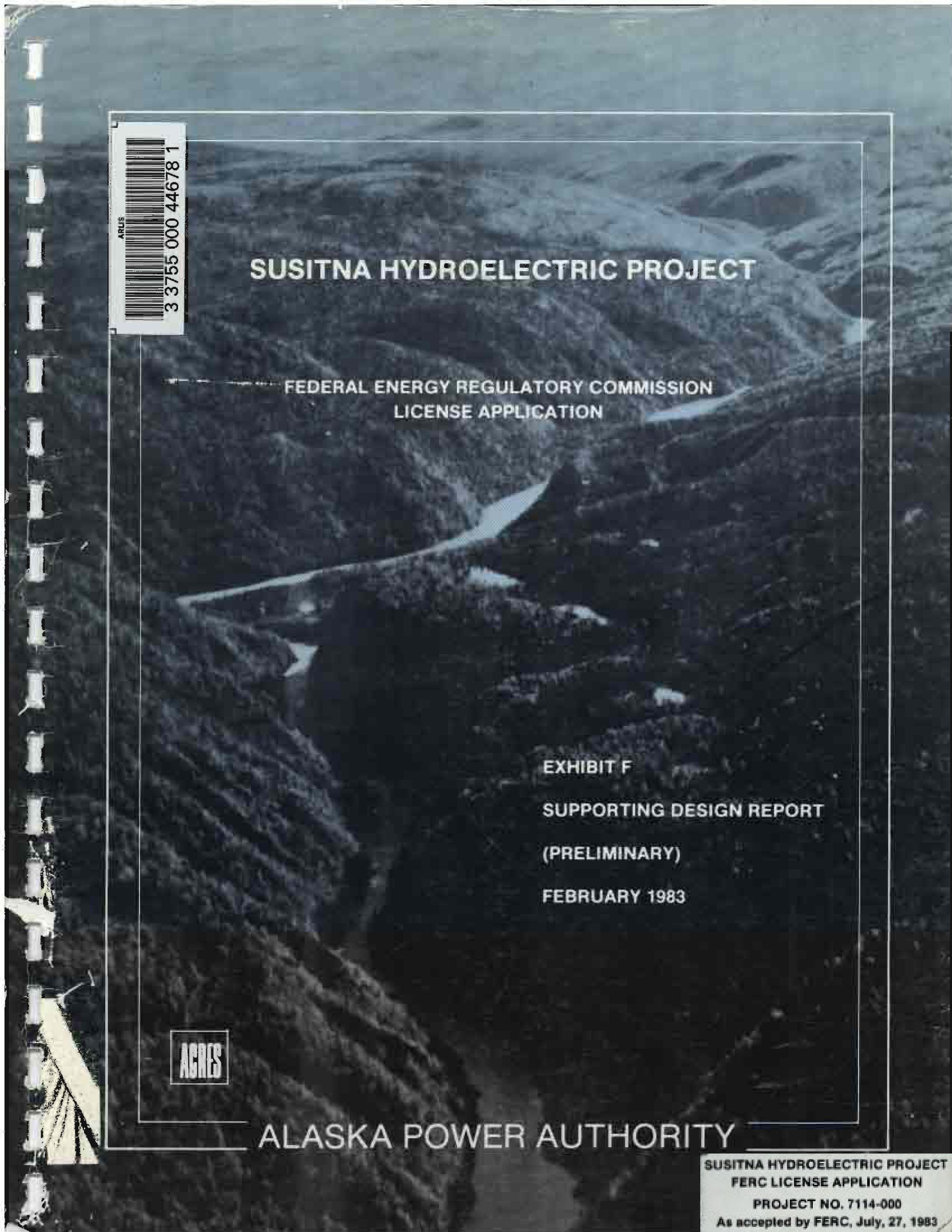

SUSITNA HYDROELECTRIC PROJECT

FEDERAL ENERGY REGULATORY COMMISSION
LICENSE APPLICATION

EXHIBIT F

SUPPORTING DESIGN REPORT

(PRELIMINARY)

FEBRUARY 1983

ACRES

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT
FERC LICENSE APPLICATION
PROJECT NO. 7114-000
As accepted by FERC, July, 27, 1983

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GENERAL

This document sets out the principal project parameters and design criteria for the Watana and Devil Canyon 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 adequate to fulfill their stated functions.

The report has been prepared as a main report with five reference volumes attached. This report has been collected as a design criteria document containing a summary of project parameters, design criteria, and codes and standards. The volumes included as reference volumes are independent reports produced as part of the feasibility and pre-license application preliminary design efforts. These volumes contain the detailed information necessary for review and independent evaluation of the project features.

The reports attached for direct reference are:

- 1980-81 Geotechnical Report; 3 Volumes and 1982 Supplemental Geotechnical Report (1);

- Feasibility Report, Volume 5, Appendix B, Design Development Studies (2);

- Final Report on Seismic Studies, February 1982 (3);

- Regional Flood Studies, December 1981 (4);

- Feasibility Report, Volume 4, Appendix A, Hydrological Studies (5); and

- Feasibility Report, Volume 6, Appendix C, Cost Estimate (6).

The report and reference volumes include information in addition to that required in the regulations. For clarity, the following cross-reference has been included. This reference directs the reader to the relevant portion of a reference volume for a specific topic.

WATANA

Topic	Direct Reference
Site Suitability Investigations	
- Previous Investigations	(1), Volume 1, Section 3
- Regional Geology	(1), Volume 1, Section 4
- Results of Geotechnical Investigations	(1), Volume 1, Section 6
Reservoir Rim Stability	(1), Volume 1, Appendix K
Boring Logs, Geology Reports	(1), Volume 1, Appendices B and D
Laboratory Test Reports	
Borrow Areas	(1), Volume 1, Section 6, Appendix F
Required Quantities of Construction Materials	(6)
Stability and Stress Analyses for Watana Dam	(2), Volume 5, Appendix B6
Basis for Seismic Loading	(3)
Basis for Spillway Design Flood	(4), (5) Appendix A3
Basis for Probable Maximum Flood	(5) Appendix A2

DEVIL CANYON

Topic	Direct Reference
Site Suitability Investigations	(1), Volume 1, Section 7
Reservoir Rim Stability	(1), Volume 1, Appendix K
Boring Logs, Geology Reports,	(1), Volume 1, Appendices C and E
Laboratory Test Reports	
Borrow Areas	(1), Volume 1, Section 7, Appendix G
Required Quantities of Construction Materials	(6)
Devil Canyon Stress Analyses	(2), Appendix B5
Stability of Abutment Slopes	(2), Appendix B5 Attachment 1
Basis for Seismic Loading	(3)
Basis for Spillway Design Flood	(4), (5) Appendix A3
Basis for Probable Maximum Flood	(5), Appendix A2

1 - PROJECT PARAMETERS

1 - PROJECT PARAMETERS

Item	Watana	Devil Canyon
River Flows		
Average flow (over 32 yrs of record)	7,990 cfs	9,080 cfs
Probable maximum flood inflow	326,000 cfs	346,000 cfs (routed through Watana) 362,000 cfs (unrouted)
Maximum flood inflow with return period of 1:10,000 yrs	156,000 cfs	161,000 cfs (unrouted) 165,000 cfs (after routing through Watana) (increase attributed to the assumed overlap of Watana peak outflow and peak flow from intermediate catchment)
Maximum flood inflow with return period of 1:25 yrs	76,000 cfs	37,800 cfs 85,000 cfs (unrouted)
Maximum flood inflow with return period of 1:50 yrs (unrouted)	87,000 cfs	39,000 cfs (after routing through Watana) 98,000 cfs (unrouted)
Normal maximum operating level	2,185 ft MSL	1,455 ft MSL
Average TWL	1,455 ft MSL	850 ft MSL
Minimum operating level	2,065 ft MSL	1,405 ft MSL
Area of reservoir at maximum operating level	38,000 acres	7,800 acres
Reservoir live storage	3.74×10^6 acre ft	0.35×10^6 acre ft

Item	Watana	Devil Canyon
Reservoir total storage	9.47×10^6 acre ft	1.9×10^6 acre ft
<u>Dam</u>		
Type	Rockfill	Concrete arch
Crest elevation	2,210 ft MSL at center 2,207 ft MSL at abutments	1,463 ft MSL (+3 ft parapet wall)
Crest length	4,100 ft	1,650 ft (arch dam including thrust blocks)
Height	885 ft above foundation at core	646 ft above foundation
Cut-off and foundation treatment	Core founded on rock, grout curtain and downstream drains	Founded on rock, grout curtain and downstream drains
Upstream slope	1V:2.4H	-
Downstream slope	1V:2H	-
Crest width	35 ft	20 ft
<u>Saddle Dam</u>	None	
Type		Earth/Rockfill
Crest Elevation		1472 ft MSL
Crest Length		950 ft
Height		245 ft
Cut-off and Foundation Treatment		Core founded on rock, grout curtain and downstream drains.
Upstream Slope		1V:2.4H
Downstream Slope		1V:2H
Crest Width		35 ft

Item	Watana	Devil Canyon
<u>Diversion</u>		
Cofferdam types	Rockfill	Rockfill
Cut-off and foundation	Founded on alluvium with slurry trench to rock	Founded on alluvium with grout curtain
Upstream cofferdam crest elevation	1,545 ft MSL	947 ft MSL
Downstream cofferdam crest elevation	1,472 ft MSL	898 ft MSL
Maximum pool level during construction	1,536 ft MSL	944 ft MSL
Water passages	2 concrete-lined tunnels, 38 ft dia.	1 concrete-lined tunnel, 30 ft dia.
Outlet structures	Low-level structure with high head slide closure gates	Low-level structure with high head slide closure gates
Diversion capacity	80,500 cfs	39,000 cfs
Final closure	Mass concrete plugs in line with dam grout curtain	Mass concrete plugs in line with dam grout curtain
Releases during impounding	6,000 cfs maximum via regulating gates in diversion plug	6,000 cfs maximum via low-level fixed cone valves
<u>Emergency Reservoir Drawdown</u>		
	Low level outlet tunnel	Fixed cone valves
Maximum capacity	30,000 cfs	38,500 cfs

Item	Watana	Devil Canyon
Outlet Facilities		
- capacity	24,000 cfs	38,500 cfs
- control struc.	Fixed cone valves	Fixed cone valves
- energy dissip.	Six 78" dia. fixed cone valves	3-90" dia., four 102" dia. fixed cone valves
Spillway		
Design Floods	Passes pmf preserving integrity of dam	Passes pmf preserving integrity of dam
	Passes routed 1:10,000-yr flood (156,000 cfs) with no damage to structures	Passes routed 1:10,000-yr flood (165,000 cfs) with no damage to structures
Main Spillway		
- capacity	120,000 cfs	123,000 cfs
- control struc.	Gated ogee crests	Gated ogee crests
- energy dissip.	Flip Bucket	Flip Bucket
- crest elev.	2,148 ft MSL	1,404 ft MSL
- gate sizes	3 - 49 ft H x 36 ft W	3 - 56 ft H x 30 ft W
Emergency Spillway		
- capacity	120,000 cfs	150,000 cfs
- type	Fuse plug	Fuse plug
- crest elev.	2200/2201.5	1464/1465.5
- chute width	310/200	200

Item	Watana	Devil Canyon
Power Intake		
Type	Massive concrete structure embedded in rock	Massive concrete structure embedded in rock
Number of intakes	6	4
Draw-off requirements	Multi-level	Multi-level
Drawdown	120 ft	50 ft
Maximum discharge/unit	3,870 cfs	3,670 cfs
Penstocks		
Type	Concrete-lined rock tunnels with downstream steel liner	Concrete-lined rock tunnels with down-stream steel liner
Number of penstocks	6	4
Diameter	17 ft conc/15 ft steel	20 ft conc/15 ft steel
Powerhouse		
Cavern size	455 ft x 74 ft x 126 ft	360 ft x 74 ft x 126 ft
Type	Underground	Underground
Transformer area	Separate gallery	Separate gallery
Control room & administration	Surface*	Underground
Access - vehicle - personnel	Rock tunnel Elevator from surface	Rock tunnel Elevator from surface
Power Plant		
Number of units	6	4
Nominal unit output**	170 MW at 659 ft net head	150 MW at 542 ft net head

Item	Watana	Devil Canyon
Turbines		
Rated net head	680 ft	590 ft
Rated full gate output	250,000 hp	205,000 hp
Rated discharge	3,490 ft^3/s	3,680 ft^3/s
Station output @ rated head		
- best gate	936 MW	510 MW
- full gate	1,098 MW	600 MW
Generator		
Type	Vertical synchronous	Vertical synchronous
Rated output (60°C)	190 MVA air-cooled	167 MVA air-cooled
Overload (80°C)	218 MVA	210 MVA
Power factor	0.9	0.9
Voltage	15 kV ± 5%	15 kV ± 5%
Frequency	60 Hz	60 Hz
Speed, rpm	225 rpm	225 rpm
Transformers	9 x 145 MVA 15/345 kV, single phase	12 x 70 MVA 15/345 kV, single phase
Tailrace		
Water passages	Two 34 ft dia. concrete-lined tunnels	One 38 ft dia. concrete-lined tunnel
Elevation of water passages	Below minimum tailwater	Below minimum tailwater
Surge	Single surge chamber	Single surge chamber
Tailwater elevations	See Fig. F.3	See Fig. F.4

*Area control center for both Watana and Devil Canyon plants.
**Based on a minimum reservoir level in peak demand month (December).

2 - PROJECT DESIGN DATA

2 - PROJECT DESIGN DATA

2.1 - Topographical Data

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

The hydrological data are based on records taken over a period of 30 years, supplemented by 2 years of records at site. Streamflows and respective drainage areas are extrapolated and adjusted to give a representative pattern of flows at the damsite. Flows are shown in Tables F.1 and F.2.

2.3 - Meteorological Data

Historical records of precipitation, temperature, and other climatic parameters are collected by NOAA 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.3 to F.5.

2.4 - Reservoir Data

Reservoir elevation, area and volume curves for Watana and Devil Canyon are given in Figures F.1 and F.2.

2.5 - Tailwater Elevations

Tailwater elevations plotted against flows are given in Figures F.3 and F.4.

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.4A.

The PMF was routed through Watana reservior and the peak outflow was 293,000 cfs. This flood routing reduced the peak inflow to Devil Canyon reservoir to 346,000 cfs. The routed peak outflow from Devil Canyon reservoir was 345,000 cfs.

The 10,000-year flood peak inflows are estimated to be 156,000 cfs at Watana, and 161,000 cfs (unrouted) and 165,000 cfs (routed) at Devil Canyon. The increase in the routed 10,000-year peak flow over the natural flood resulted because of the assumption of synchronization of routed flood peak and peak from the intervening area between the two developments.

The develoment of the PMF and Spillway design floods are presented in Appendix FC.

3 - CIVIL DESIGN CRITERIA

3 - CIVIL DESIGN CRITERIA

3.1 - Governing Codes and Standards

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

(a) General

- American National Standards Institute, ANSI A58.1;
- Uniform Building Code (UBC);
- Alaska State Building Construction Code; and
- Occupational Safety and Health Administration Standards (OSHA).

(b) Concrete

- American Concrete Institute - ACI Standard 318 (for reinforced concrete)
- American Concrete Institute - ACI Standard 301
- American Concrete Institute - ACI Standard 207 (for mass concrete)

(c) Structural Steel

- American Institute of Steel Construction, Steel Construction Manual.

3.2 - Design Loads

(a) Dead Loads:

Mass concrete	145	lbs/ft^3 (143 lbs/ft^3 when checking stability)
Reinforced concrete	150	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)

(b) Backfill Loads

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.

(c) Snow and Ice Loads

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

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

(d) Powerhouse Floor Loads

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

(e) Crane Loads

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

(f) Spillway Deck Loads

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

(g) Hydraulic Loads

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

(i) Uplift

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.

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

(h) <u>Seismic Loads</u>

See Reference No. 3.

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

(i) <u>Watana</u>

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.

(ii) <u>Arch Dam at Devil Canyon</u>

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 ratio - 0.10 of critical*.

Acceleration response spectrum - See Figure F.5.

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:

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

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 ft 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.

(i) Temperature and Thermal Loads

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

(j) Horizontal Ice Loads

The following horizontal ice load shall be considered to act at mid-depth of a 4 foot-thick ice cover at the water surface.

On the upstream gates .... 10.0 kips/lin ft

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

3.3 - Stability

(a) 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(d):

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

(b) 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:

- Stress at upstream face (parallel to slope);
- Stress at downstream face (parallel to slope);
- Location of resultant force;

- Sliding factor;
- Shear friction factor;
- Flotation factor of safety; and
- Overturning factor.

(i) Sliding Stability Analysis

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 (1):

$$F = \frac{(V-U)\tan \phi + cA}{P_w} \quad (1)$$

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

(c) Limiting Criteria, Safety Factors

(i) Concrete Gravity Structures

	Safety Factor			
Load Conditions	Sliding	Overturning*	Flotation	Compression
Normal	3 within concrete 4 within rock	Resultant within the center third**	1.5	3 on compressive strength of concrete 4 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 compres-ive strength of concrete 3.5 on compres-sive strength rock

Load Conditions	Sliding	Overturning	Flotation	Compression
Unusual (inc. 100-year return earthquate & PMF load case)	2 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

Note: * Opinions differ on the use of overturning safety factors. Acres policy 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.

** Safety factor implicitly greater than at least 1.5

(ii) 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 Main spillway gate structure Plate F13
Devil Canyon arch dam thrust blocks Plate F46
Devil Canyon main spillway gate structure Plate F55

(d) Loading Cases

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

(i) Intake and Outlet Structures

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

(ii) Powerhouse Structure (Surface structures, if applicable)

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

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:

  - o In all expansion and contraction joints
  - o In all vertical construction joints communicating with dry interior spaces; and
  - o In all horizontal construction joints communicating with dry interior spaces where the concrete thickness is less than 10 feet.

4 - GEOTECHNICAL DESIGN CRITERIA

4 - GEOTECHNICAL DESIGN CRITERIA

4.1 - Watana

(a) General

A detailed description of the geology and material properties for the Watana site are provided in reference document "1980-81 Geotechnical Report" and the "1982 Supplemental Geotechnical Report." 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 development of foundation preparation, treatment, material properties, and costs. The following tasks set forth the design considerations, parameters, and criteria for the Watana Dam and related structures.

(b) Dam Foundation Preparation and Treatment

(i) General

Rock foundations must meet the following criteria:

- The rock under the core must be nonerodible under the seepage gradients;

- Core material must be prevented from moving down into the foundation (e.g., into cracks or joints);

- Contact between the core and rock surface must have permeability no higher than that of the core; and

- Any seepage through the foundation must be controlled and discharged to avoid buildup of excessive seepage pressures under the structures.

(ii) Excavation Under the Core, Filters, and Shells

The core, filter, and shell portions of the dam will be founded on sound rock or concrete. All talus on the slopes, river alluvium, and weathered rock in the valley bottom and on the abutments will be removed. Estimated core foundation rock slopes will be on the average of 1H:2V below Elevation 1800 and 1H:1V above Elevation 1800. The cross cut slopes will be 1H:10V. Dental excavation over and above normal excavation will be performed in intensely sheared and altered zones. Under the core and filter, dental concrete will be placed as appropriate to provide a regular surface for fill placement.

(iii) Grouting

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

- Consolidation Grouting

The rock 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 would impede relief of seepage, so it will not be performed under the downstream blanket filter. 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.

- Curtain Grouting

Curtain grouting will be performed beneath the dam foundation to a maximum depth of 0.7H (where H is the maximum reservoir hydrostatic head at a particular location above the dam foundation) to a maximum depth of approximately 350 feet. Grouting will be carried out from a series of underground gallaries 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 using split spacing with the primary holes at 40-foot spacing. The secondary, tertiary, and quaternary holes will bring the final hole spacing to 5 feet if required.

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

Grout holes will be vertical and inclined at angles of 45 degrees to intersect the main joint sets. Additional grouting will also be performed as required in sheared and altered zones and poor quality rock 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.

- Contact Grouting

Contact grouting will be performed on concrete structures in contact with rock and behind all tunnel linings and tunnel plugs.

(iv) 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. Filters may be required in some of the drain holes to prevent washout of fine material.

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.

(v) Intake Structure

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.

(vi) 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. Since, however, these drains will be subject to freezing, it is proposed that a spillway drainage gallery be constructed at a depth of at least 30 feet 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.

(vii) 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 about 450 feet below reservoir level. The maximum hydraulic gradient along the buried channel for the edge of pool to Tsusena Creek will be approximately 9 percent.

Potential problems imposed by the Relict Channel are:

- Surface Flows - caused by settlement and resulting in a breaching of the reservoir rim.

- Subsurface Leakage - caused by low 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.

- Liquefaction - Filling of the reservoir resulting in saturation of material in relict channel that could result in liquefaction under seismic loading conditions causing a breach of the reservoir rim.

Remedial measures considered for the relict channel are:

- Lowering of the reservoir level to provide adequate freeboard to eliminate potential of settlement and surface flow.

- Placement of a downstream blanket to control the potential problem of piping.

- Long-term monitoring to determine rate of thaw of permafrost.

- Densification, in-place stabiliziation (i.e., grouting), or excavation and replacement of potentially liquifiable materials.

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

(c) Rock Slopes

(i) Design Methods

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.

(ii) Factor of Safety

Factors of safety employed in slope design for civil structures were:

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

(iii) Method of Analysis

Plane failures and two-plane wedge failures were analyzed on an equal angle stereogram (Hendron, 1971). 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

(d) Water Tunnels

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.

(e) Penstocks

The penstock tunnels will be concrete lined over their entire lengths, with steel linings placed just upstream of the powerhouse. Six penstocks, 17-feet in diameter, are proposed. Penstocks will be spaced 2.5 times the diameter, center to center.

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

(f) Caverns

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

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 sets, 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 1.5 times the largest cavern span.

(g) Watana Dam

(i) General

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

(ii) Design Criteria

To insure that the impervious core meets the earthquake resistant design, the following design features will be incorporated into the main dam cross section:

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

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

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

- Compacted river alluvium gravel will be used to construct the downstream outer shell, and compacted clean river alluvium gravel will be used to construct the upstream outer shell to minimize settlement and displacement that could be caused by a seismic event.

(iii) Freeboard and Static Settlement

The governing crest elevation excluding static and seismic settlement is 2205 feet.

The expected seismic settlement of 0.5 percent of the height of the dam will be incorporated in the design by locally steepening the slopes of the top of the dam to obtain 5 feet of additional freeboard at the maximum section and 2 feet of additional freeboard at the abutments.

(iv) Dam Cross Section

The typical cross section is shown in Plate F6. The impervious core slopes will be 1.0 horizontal to 4.0 vertically sloped upstream and downstream with a crest of 15 feet. Minimum core foundation will be 50 feet requiring flaring of the cross section at the left end of the embankment.

The upstream filter will have the following slopes:

- Fine filter zones will be 1.0 horizontal to 3.5 vertical sloped upstream on the reservoir side.

- Coarse filter zones will be 1.0 horizontal to 2.9 vertical sloped upstream on the reservoir side.

The downstream filter zones will have the following slopes:

- Fine filter zones will be 1.0 horizontal to 3.2 vertical sloped downstream on the tailwater side.

- Coarse filter zone will be 1.0 horizontal to 2.7 vertical sloped downstream on the tailwater side.

The upstream and downsteam filters are sized to provide protection against possible leakage through transverse cracks in the core that could occur as the result of settlement or displacement during a seismic event. The wide filter zones provide sufficient material for healing of any cracks in the core and the size of the downstream filter zones will ensure its capability to handle any abnormal leakage flows.

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

Slope protection on the upstream slope will consist of a 10-foot zone of oversized material up to 36 inches in diameter placed and compacted by suitable equipment.

The typical crest detail is shown in Plate F7. Because of the narrowing of the dam crest, the filter zones will be reduced in width and the upstream and downstream coarse filter replaced with carefully graded and selected shell materials above Elevation 2170. A layer of filter fabric is incorporated to protect the core material against damage from frost penetration and dessication, and to act as a coarse filter where required.

(v) Dam Material

- Core

The core material will be obtained from Borrow Site "D," which consists of a series of glacial tills separated by alluvial and lacustrine materials. Processing and blending will be necessary to provide the required moisture content and gradation and to remove any oversize material. However, information to date indicates this can be accomplished by selection of a vertical-face mining method and on-fill mixing and raking.

Material will be placed in 8-inch lifts at a maximum moisture content of 3 percent above optimum moisture content, and compacted to 95 percent of the maximum density obtained from the Modified Proctor Test (ASTM D698).

- Fine and Coarse Filters

Fine and coarse filter material can be obtained from Borrow Sites E, I, and J. Borrow Site E is the preferred primary borrow source for all the filter and alluvium fill material in the dam. The material will require processing to provide the proper gradations for the fine and coarse filters.

- Alluvium Fill Material

The alluvium fill can be obtained from Borrow Areas E, I, and J. The upstream shell of the dam will be constructed using processed river alluvium gravel with no more than 10 percent of the material less than 3/8-inch. The downstream shell will be constructed using unprocessed alluvium fill material, with mixing of a carefully controlled percentage of waste work from excavations.

- Riprap Material

The riprap material (slope protection) will be obtained from the oversize material from the various borrow areas, Quarry A, and any other rock excavations. The riprap material will be placed on the entire upstream slope, and in certain areas of the downstream slope of the dam as protection against wave overtopping and toe erosion.

(vii) Stability Analysis

Static and dynamic stability analyses have been performed to establish the upstream and downstream slopes of the Watana Dam. A summary of the stability analyses is outlined in Appendix FB. The analyses indicate stable slopes under all conditions for a 2.40 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 until final design analysis and investigations show steeper slopes are stable.

(h) Watana Emergency Spillway Fuse Plug

The earthfill fuse plug, with a crest elevation of 2201.5, is located in the upstream end of the unlined rock channel spillway. The 31.5 feet high fuse plug is designed to be eroded if overtopped by the reservoir and since the crest is lower than the dam embankment, the plug would be washed out prior to overtopping the main dam. Details of the fuse plug design are presented in Appendix FA.

Static and dynamic stability analyses have been performed to establish the fuse plug embankment slopes. The studies indicate the embankment slopes of 1:2.4 upstream and 1:1.5 downstream, are stable under all conditions of loading. A summary of the stability analyses is outlined in Appendix FB. The preliminary design is considered suitable for both operational stability and rapid failure if overtopped.

4.2 - Devil Canyon

(a) Foundation Preparation and Treatment

(i) Main Dam

The entire area under the dam will be excavated to sound, fresh rock. In addition, 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.

(ii) Grouting

- Consolidation Grouting

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

The consolidation holes will be at 10 feet spacing with depth ranging from 30 to 70 feet. The orientation of the consolidation holes will be such that they intersect the majority of discontinuities.

- Curtain Grouting

The extent of curtain grouting 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 scheme is proposed (Plate F46).

The grouting will be performed from tunnel galleries, the general arrangement of which is shown in 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 drainage curtains 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.

(iii) Drainage

- Dam

The grout galleries will be used for drainage. The drainage holes will be 3-inches in diameter and will follow a similar arrangement to the grout curtain.

The drainage 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 possible, drainage holes will be drilled from the galleries to prevent freezing. 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. Drainage holes will be drilled upward from the gallery wherever possible to provide the most effective drainage system. Drainage curtains drilled from upper and lower galleries will overlap by at least 10 feet. The drainage curtain will be drilled from the gallery and inclined about 10 degrees downstream from the vertical.

- Caverns

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.

- Rock Cuts

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.

(iv) Intake Structure

The foundation for the intake structure will be on sound, unweathered rock. 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.

(v) 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 under the structure and reduce uplift pressures.

The drainage/grouting gallery will be formed in the concrete rollway of the 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 beneatn 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.

Because of the susceptibility of the drains under the spillway slab to freezing, a drainage gallery will be constructed at a minimum depth of 30 feet 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 re- quired. 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.

(vi) Saddle Embankment Dam

- Foundation Excavation Preparation

  All overburden beneath the proposed saddle dam will be removed. The foundation area for the core and filters will be excavated to sound rock, while the rockfill shells will be excavated to competent rock. The final excavated foundation slopes will be no steeper than 1H:1V. The foundation will be regular in shape. 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.

- Grouting

  . Consolidation Grouting

  The rock under the 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.

  . Curtain Grouting

  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 left abutment, the curtain will extend under the fuse plug emergency spillway and continue 50 feet past the spillway.

  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 40-foot spacing. Using secondary, tertiary, and quaternary 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 on the left side of the dam will be between the dam and emergency spillway; 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 site, but isolated frozen lenses may occur, in which case thawing will be carried out prior to grouting.

- Drainage

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

The drainage 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.

(b) Rock Slopes and Foundation Design

(i) 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.

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

(ii) Devil Canyon Arch Dam

It is expected that the treated rock mass foundation modulus is in the range of $1x10^6$ psi to $3x10^6$ 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.

(iii) Spillway and Intake Structure Foundations

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.

(c) Tunnels and Penstocks

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 2.5 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.

(d) Caverns

(i) Support

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 80kips. The preferred type is a tensioned, resin-anchored, 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.

(ii) Drainage

Drainage will be provided for the walls and crown to prevent seepage from affecting their 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.

(iii) Spacing

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

(iv) Orientation

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.

(e) Devil Canyon Arch Dam

(i) Material and Thermal Properties

The material and thermal properties for the Devil Canyon arch dam are:

- unit weight of concrete - 150 lb/ft^3
- unit weight of water - 62.4 lb/ft^3

Static Properties

Concrete

- ultimate uniaxial compressive strength at 365 days .... 5000 psi
- allowable compressive stress .... 1250 psi
- sustained modulus of elasticity .... 3×10^6 psi
- allowable tensile stress .... 325 psi
- Poisson's ratio 0.2

Rock

- ultimate compressive strength .... 20,000 psi (unconfined)

- allowable compressive stress .... 5,000 psi

- static modulus of elasticity .... 2×10^6 psi

- Poisson's ratio 0.2

Dynamic Properties

Concrete

- uniaxial dynamic comrpessive strength .... 6,000 psi

- instantaneous modulus of elasticity .... 5×10^6 psi

- allowable linear rapid loading tensile strength .... 750 psi

- Poisson's ratio 0.2

Rock

- properties assumed as for static conditions.

Thermal Properties

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 .... -0.046 ft^2/hr

(ii) General Parameters

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

- normal maximum reservoir operating level ..Elevation 1455
- minimum reservoir operation level ....Elevation 1405
- dam crest elevation ....Elevation 1463
- minimum foundation level ....Elevation 820

(iii) 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. (See Ref. No. 2, Appendix B5).

The loads and conditions analyzed follow:

Static Loads

- self-weight of the dam;
- hydrostatic pressure from the reservoir;
- temperature changes; and
- ice load.

Dynamic Loads Caused by Seismic Events

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

Loading Combination

(a) Usual Load Combination

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 1455;
UL2 - Dam self weight + hydrostatic load with reservoir at EL 1405;
UL3 - As UL1 plus extreme winter temperature effects; and,
UL4 - As UL2 plus extreme winter temperature effects.

(b) Extreme Load Combination

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

EL1 - UL1 + extreme earthquake loading.

Results

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.

(f) Saddle Dam

(i) General

The design philosophy for the saddle dam is essentially the same as that for the main dam at Watana. The most significant difference is the use of rockfill in the shells instead of the river gravels used at Watana.

(ii) Dam Cross Section

The central vertical core will be protected by fine and coarse filters on both upstream and downstream slopes and supported by gravel and rockfill shells. The core will have a crest width of 15 feet and side slopes of 1H:4V 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 gravel or rockfill, processed to remove fine material in order to minimize pore pressure generation and ensure rapid dissipation during and after a seismic event. Since pore pressures cannot develop in unsaturated sections of the downstream shell, the material in that zone will be unprocessed rockfill from surface or underground excavations.

Protection on the upstream slope will consist of a 10-foot layer of riprap.

(iii) Dam Material

No source of material suitable for the core of the saddle dam has been identified closer than the borrow areas at Watana (Sites D and H). The current proposal is to use Site D for core material for the saddle dam. The in-place volume of core material is 306,000 cubic yards.

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. Surplus material from Site G will be used in the upstream shell. The balance of the shell material will be rockfill obtained primarily from the excavations for the spillways. 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, but it is proposed to make up any shortfall by deepening and extending the emergency spillway cut. If, however, the excavated rock is found unsuitable for construction material, that Quarry Site will be utilized as a primary rock source.

(iv) Stability Analysis

Special precautions have been taken to ensure stability under earthquake loading by the use of processed free draining gravel and 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.

Static and dynamic 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 4.1(g)(vii). However, further analyses will be required for the Devil Canyon saddle dam.

(g) Devil Canyon Emergency Spillway Fuse Plug

The earthfill plug, with a crest elevation of 1465.5 is located in the upstream end of the unlined rock channel spillway. The 31.5 feet high fuse plug is designed to be eroded if overtopped by the reservoir and since the crest is lower than the dam embankment, the plug would be washed out prior to overtopping the main dam.

Details of the fuse plug for Watana Dam are outlined in Appendix FA. The designs for Watana and Devil Canyon are identical.

Static and dynamic stability analyses have been performed to establish the fuse plug embankment slopes. The studies indicate the embankment slopes of 1:2.4 upstream and 1:1.5 downstream are stable under all conditions of loading. A summary of the stability analyses is outlined in Appendix FB. The preliminary design is considered suitable for both operational stability and rapid failure if overtopped.

5 - HYDRAULIC DESIGN CRITERIA

5 - HYDRAULIC DESIGN CRITERIA

	Watana	Devil Canyon
5.1 - River Flows		
Average annual flow	7,990 cfs	9080 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.	293,000 cfs	345,000 cfs (routed through Watana Reservoir)
Project Design: 1:10,000-year flood inflow. Derived from annual flood series frequency analysis. Reservoirs assumed at normal maximum operating level.	156,000 cfs	165,000 cfs (routed through Watana Reservoir)
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.	31,000 cfs	39,000 cfs
Diversion Design: 1:50-year flood (routed) at Watana 1:25-year flood at Devil Canyon. Annual flood series frequency analysis. Devil Canyon diversion assumes normal power operation and storage at Watana.	80,500 cfs	39,000 cfs
5.3 - Reservoir Levels		
Normal Maximum Operating Level:	2185 ft MSL	1455 ft MSL
Minimum Reservoir Level:	2065 ft MSL	1405 ft MSL
Maximum Reservoir Level:	2201 ft MSL	1466 ft MSL
PMF surcharge level	2201 ft MSL	1466 ft MSL
1:10,000-year surcharge level	2193.5 ft MSL	1455 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. Outlet operational when Watana reservoir level exceeds Elevation 2185.5.	2193 ft MSL	1455 ft MSL
Allowable reservoir surcharge above normal maximum operating level.	8.5 ft	--

5.5 - Reservoir Parameters

	Watana	Devil Canyon
Reservoir area at normal maximum operating level.	38,000 acres	7800 acres
Reservoir Live Storage: (Storage between normal maximum and minimum reservoir levels)	3,740,000 acre-ft	350,000 acre-ft
Reservoir Total Storage: (At normal maximum operating level)	9,470,000 acre-ft	1,090,000 acre-ft

5.6 - Freeboard (Hydraulic Considerations)

	Watana	Devil Canyon
Allowance for wave height and run up	5 ft	3 ft
Allowance for restricted discharges and reduced nitrogen supersaturation above normal maximum operation level	8 ft	0 ft

5.7 - Criteria

(a) Spillways

(i) Capacity

- Pass PMF while maintaining the integrity of the main water retaining structures. 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 operated only for short duration is acceptable.

- Pass routed 1:50-year flood without elevating nitrogen supersaturation levels above 116 percent.

(ii) Chute

- Maximum velocity 90 ft/s without aeration.

(iii) Energy Dissipation

- Minimum radius of flip bucket greater than 7 x depth of design flow.

Plunge Pool

(iv) Diversion

Minimum release during impounding	Determined by downstream flow constraints. Range of 1000 to 19,000 cfs.

(v) Reservoir Levels

Normal maximum operating limit	Maximum elevation of 2250 MSL at Watana. Level should be as low as possible. Economic benefits of any level over 2000 MSL must be clearly demonstrated.
Minimum operating level	As close to normal maximum as possible. Economic benefits of any level lower than 2100 MSL must be clearly demonstrated.
Average minimum operating level	As close to 2150 MSL or higher. Economic benefit of any level lower than 2150 MSL must be clearly demonstrated.

(vi) Reservoir Operation

Downstream discharge during operation	On a daily basis, discharge from the most downstream structure should be constant; this can be accomplished through base-load operation or a reregulation structure. Reservoir operation should provide flows consistent with downstream flow mitigation plan.

6 - EQUIPMENT DESIGN CODES AND STANDARDS

6 - EQUIPMENT DESIGN CODES AND STANDARDS

6.1 - Design Codes and Standards

(a) Turbines

- ASME Boiler and Pressure Vessel Code, Section VIII, Pressure Vessels;
- ANSI Standard B49.1;
- ANSI Standard 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.

(b) Gate Equipment

- 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;* and
- ANSI Standard B31.1.*

(c) Valve Equipment

- ASME Boiler and Pressure Vessel Code, Section VIII, Pressure Vessels.

(d) Crane Equipment

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

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* Hydraulic hoist design.

(e) Elevators

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

(f) Mechanical Systems

	Applicable Systems**
- 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

(a) Turbines

(i) Operation

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 objectionable surges in power, detrimental vibrations or objectionable noises. The turbines will be designed for continuous operation at maximum runaway speed.

(ii) Stresses

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:

**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 .... 2000 lb/in^2 tension
  10,000 lb/in^2 compression
- Bronze bearings .... 3000 lb/in^2
- Babbitt bearings .... 500 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

  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 8000-hour operating period will not exceed 0.1 times the discharge diameter of the runner (in ft).

(b) Gate Equipment

(i) Gates and Guides

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 on a flat track, hertz contact stresses (compressive) will not exceed 250 times BHN (in lb/in^2). For flat wheels or rollers on a flat track, the load (in lb) per inch width of roller contact will not exceed 1600 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.

(ii) Hoists

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 2000 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.

(c) Valves

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.

(d) Trashracks

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

(e) 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 (b) above for extreme loading on wire rope hoists.

(f) Mechanical Systems

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

(a) Diversion Control Gates

Fixed roller 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 rollers for transfer of hydrostatic loads to the guides after diversion closure when the head increases as the reservoir is impounded.

(b) 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:

- Passing required releases during reservoir impoundment; and
- 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 energy dissipation. The gates will be designed to operate at full or partial opening for heads up to the low operating level of the outlet facilities. 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.

(c) Trashracks (Watana)

Course trashracks 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 trashracks are as follows:

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

(d) Diversion Tunnel Stoplogs

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 - Main Spillway

(a) Spillway Gates

The spillway gates will be fixed wheel vertical lift gates operated by a double drum wire rope hoist mounted on a tower and bridge structure. The hoist housing will be enclosed and heated.

Provision will be made for installation of heaters in the gates and guides.

(b) Stoplogs

A set of stoplog guides will be provided upstream from each spillway gate to permit 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.

6.5 - Outlet Facilities

(a) 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. In sizing the valve, the cylindrical gate opening will be assumed to be restricted to about 80 percent of its theoretical maximum to prevent possible vibration.

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.

(b) Ring Follower Gates

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

- Relieve the hydrostatic load on the valve when it is not in operation;
- Permit inspection and maintenance of the valve; and
- 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.

(c) 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 the single tunnel, the gates will have fixed hoists. A gantry crane will be used to handle the gates at Devil Canyon.

(d) 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) .... 12 ft/s
Spacing .... 0.1 x valve size (approximate)
Design differential head .... 40 ft (approximate)

6.6 - Power Intake

(a) 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) .... 5 ft/s (approx.)
Bar spacing .... maximum spacing not to exceed minimum distance between runner blades
Design differential head .... 20 ft (approx.)

(b) Intake Gates

Fixed wheel or roller type vertical lift gates will be installed at the entrance to each penstock. The gates will be used to permit dewatering of the penstock 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 fixed hoists.

(c) Intake Bulkhead Gates

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.

(d) Water Level Shutters

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

(a) Turbines

The turbines will be vertical shaft Francis type directly connected to synchronous generators. The turbines will have steel spiral cases and concrete elbow draft tubes.

The turbine capacity will be established on the basis of the minimum reservoir level in December (the peak demand month). At Watana, the unit output in December with minimum reservoir level will be 170 MW. At Devil Canyon, the output will be 150 MW with minimum December reservoir level.

The preliminary turbine data have been established as follows:

	Watana	Devil Canyon
Number	6	4
Head (net)		
- rated	680 ft	590 ft
- maximum	725 ft	603 ft
- minimum	600 ft	541 ft
Power at rated speed	250,000 hp	225,000 hp
Synchronous speed	225 rpm	225 rpm
Specific Speed	32.4	35.0

The design or rated head of the turbines for both Watana and Devil Canyon will be the weighted average operating (net) head.

The specific speed of the turbines will be selected within current experience with respect to head.

(b) Powerhouse Cranes

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

- Installation of the turbines, generators, and other equipment; and
- 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.

(c) Draft Tube Gates

Draft tube gate guides will be provided at the end of each draft tube to permit 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.

(d) Miscellaneous Mechanical Equipment

Miscellaneous mechanical equipment will include:

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

(e) Mechanical Services

The mechanical services within the powerhouse will include:

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

- Compressed air system

  . service area system
  . tailwater depression air
  . governor air
  . circuit breaker air

- Oil storage and handling

  . transformer oil system
  . governor and lubricating oil system

- Drainage and dewatering system

  . clearwater drainage
  . unit dewatering and filling system
  . sanitary drainage system

- Heating and ventilating system

6.8 - Tailrace Tunnels

(a) 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, where there will be two tailrace tunnels, stoplog guides and stoplogs will be provided for the tunnel intake (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.

APPENDIX FA

APPENDIX FA - WATANA EMERGENCY SPILLWAY

1 - Selection of Spillway Design

1.1 - Introduction

The basic criteria for the Emergency Spillway are that it shall not come into operation until the reservoir elevation reaches Elevation 2200 and that the capacity of the spillway shall be for the PMF routed through the reservoir. In addition, there must be sufficient freeborad on the main dam and relict channel saddle dam when the emergency spillway is passing the PMF as safeguard against overtopping the dam or breaching the reservoir rim.

A number of alternative arrangements and designs were considered for the Watana site. It was concluded that the most appropriate alternative, considering project economics, safety and operational reliability, was to utilize two spillways. The gate-controlled service spillway would carry flows in excess of the capacity of the power plant and all flows up to the critical reservoir level at Elevation 2200. The emergency spillway would come into operation at that elevation and be designed to carry the PMF.

The following alternative emergency spillway designs were considered:

- Uncontrolled open-cut channel;
- Gated spillway; and
- Open-cut channel with fuse plug.

These alternatives are discussed below.

1.2 - Uncontrolled Open-Cut Channel

An uncontrolled open-cut channel has the obvious advantage of security. The only risks are cut slope slides into the channel but such material would be removed by regular maintenance and, in any event, would be unlikely to seriously impede PMF discharges.

However, to maintain the criteria stated above, the spillway sill would have to be at Elevation 2200. To minimize the height of the main dam, the depth over the sill at PMF flow would have to be as small as possible. At the same time, maximum reservoir level is limited by the saddle elevation in the relict channel. An open cut channel to satisfy both criteria would have to be very wide (in excess of 1000 feet), would involve extensive excavation and would encroach into the relict channel, where special measures would be necessary to ensure stability of the spillway under PMF flows.

Alternatively, the spillway sill cold be lowered to incorporate the spillway cut in sound rock, but this could only be achieved by lowering normal operating levels in the reservoir which would adversely affect the project economics.

An uncontrolled open-cut spillway was, therefore, considered unsuitable for the Watana site.

1.3 - Controlled Open Cut Spillway

To satisfy all criteria requires that the emergency spillway not come into operation until the reservoir level reaches Elevation 2200 but that once that level has been reached, the PMF flood will not result in significant surcharging above that elevation.

Two methods of achieving these requirements were studied; a gated spillway and a fuse plug.

(a) Gated Spillway

The gated spillway alternative was considered and costed and proved to be significantly more expensive than a fuse plug in construction and operating costs. Gates were considered less desirable than either an open cut or a fuse plug from a safety, reliability and maintenance standpoint. Mechanical gates would require long-term maintenance and service throughout the project lifetime. In addition, the gates would require operator action and the effectiveness of the spillway could be jeopardized by human error.

(b) Fuse Plug Closure

Fuse plug closure was considered most desirable from a realiability standpoint, both because it is a "passive" system that does not require operator action to function, and it does not have the electrical and mechanical risks of failure of a gated installation.

From an initial cost standpoint, the fuseplug spillway has a lower cost than an open-cut spillway due to the greater flow depth, which is achieved following wash-out of the plug, with resultant order-of-magnitude reduction in channel excavation requirements. The earth-fill fuse plug has significantly lower construction costs than a gated installation. Annual maintenance costs for a fuse plug in an open-cut spillway would be lowest, with maintenance limited to minor repair of face materials, annual checks to ensure the crest and pilot channel elevations are maintained, and application of herbicides to prevent development of plant growth on the downstream face of the plug.

Because the emergency spillway is never intended to be used (reservoir operation is designed to safely pass all floods up to the PMF event without emergency spillway use), the capitalized replacement cost of the plug after use does not come into consideration.

1.4 - Recommended Emergency Spillway

Based on the above discussions, the fuse plug spillway was determined to be the most economic, safe and reliable design for the Watana arrangement.

2 - FUSE PLUG DESIGN

The selected fuse plug design was the result of four primary criteria:

- High seismic stability;
- High static stability;
- High resistance to failure by seepage; and
- Low resistance to failure by overtopping.

The fuse plug is primarily a dam, since under normal operating conditions, the spillway approach channel will be flooded and there will be up to 15 feet of water acting against the fuse plug. The selected section (Exhibit F, Plate F18) was developed under the first three criteria along the same lines as the main dam, to ensure security during annual pool fluctuations from base (Elevation 2170) to Elevation 2185, and flood surcharging to Elevation 2194. At Elevation 2194, with design wave of 6 feet, the security of the structure was considered adequate, but some form of floating breakwater may be advisable to reduce wave action within the approach channel.

To insure rapid failure by overtopping, several variations were made in the design from that of the main dam. The material in the downstream shell is designated as select, washed and sorted gravel. Removal of cohesive properties will ensure rapid erosion. The clean nature of this material also serves to drain any seepage which may occur through the core during normal operation. The core is inclined with the base upstream to induce rapid collapse by undermining when the downstream shell material is eroded when water overtops the plug.

The selected shell particle size was based on a requirement for rapid erosion while still maintaining a pervious shell that will readily drain precipitation and seepage and so minimize buildup of ice in the interior of the plug. The crushed stone and riprap layers were selected to provide the necessary protection from erosion by rainfall, snow-slump erosion, and reservoir wave and ice scouring.

The mode of failure of the fuse plug is as follows:

- Flood filling of the pilot channel at Elevation 2200;

- Headwall erosion at the downstream fine filter cap, resulting in gullying of the fine filter;

- Progressive gullying of the downstream shell and filter;

- Failure of the core cap, either by gullying in the pilot channel area or by underminng by washout of the downstream filters and shell. (These failures could occur simultaneously but, if the core is frozen, its failure would probably be the result of undermining); and

- Progressive undermining failure of the core and filter due to washout of the downstream shell when the whole plug is overtopped concurrent with lateral erosion of the plug from the pilot channel area.

The riprap and upstream shell material are not expected to restrict flow, because, by the time erosion extends to the upstream riprap contact, a hydraulic head of 30 to 32 feet will be eroding the plug on a gradient of approximately 25 percent. This slope will be more than adequate to erode all particle sizes in the plug and wash them out of the spillway channel.

As protection against delayed erosion of the pilot channel, general overflow of the plug will occur at Elevation 2200, causing general gullying and washout of the downstream face.

3 - FUSE PLUG OPERATIONAL PROCEDURES

The following operational and maintenance procedures are anticipated to provide added security to the fuse plug design:

- Routine maintenance to ensure crest elevation and pilot channel are maintained in a clear, clean, level state and at the proper elevations;

- Annual herbicide application to ensure no cohesive or deep-rooted vegetation will grow on the fuse plug;

- Standard operational procedure to move excavation equipment to the fuse plug at any time pool level exceeds a pre-set elevation. The equipment would be standard project maintenance equipment such as a backhoe or dragline and would be used to accelerate crest breaching or remove any blockages as necessary if the fuse plug were overtopped; and

- A contingency plan for explosive demolition utilizing pre-placed vertical pipe explosive chambers. The explosive demolition would be a part of the state emergency preparedness plan, and personnel and explosives could be put on standby at the damsite if the combined hydrologic, reservoir level and meteorologic conditions necessary for a major (greater than 10,000 year flood) flood are present or threaten.

4 - SUMMARY

In summary, it is considered that a fuse plug design is preferable to a gated outlet facility, in lieu of a viable open-cut emergency spillway configuration. The design is considered to be suitable, at the preliminary design level, both for long-term operational stability and resistance to premature failure; and to assure rapid failure if overtopped. The possibility of frozen moisture in the plug has been reduced by assuring an extremely free-draining material in the shells and filters, so that aside from a thin surficial snow-filled layer, the plug will be in a surface-saturated moisture condition at the worst, and in most areas, dryer than that. This limited amount of moisture will rapidly thaw under the effect of the overflowing water and is not expected to significantly retard the progressive failure mechanism.

TABLES

TABLE F.1: PRE-PROJECT FLOW AT WATANA (CFS)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
1	4719.9	2083.6	1168.9	815.1	641.7	569.1	680.1	8655.9	16432.1	19193.4	16913.6	7320.4	6648.1
2	3299.1	1107.3	906.2	808.0	673.0	619.8	1302.2	11649.8	18517.9	19786.6	16478.0	17205.5	7733.7
3	4592.9	2170.1	1501.0	1274.5	841.0	735.0	803.9	4216.5	25773.4	22110.9	17356.3	11571.0	7776.1
4	6285.7	2756.8	1281.2	818.9	611.7	670.7	1382.0	15037.2	21469.8	17355.3	16681.6	11513.5	8035.2
5	4218.9	1599.6	1183.8	1087.8	803.1	638.2	942.6	11696.8	19476.7	16983.6	20420.6	9165.5	7400.4
6	3859.2	2051.1	1549.5	1388.3	1050.5	886.1	940.8	6718.1	24881.4	23787.9	23537.0	13447.8	8719.3
7	4102.3	1588.1	1038.6	816.9	754.8	694.4	718.3	12953.3	27171.8	25831.3	19153.4	13194.4	9051.0
8	4208.0	2276.6	07.0	1373.0	1189.0	935.0	945.1	10176.2	25275.0	19948.9	17317.7	14841.1	8381.0
9	6034.9	2935.9	2258.5	1480.6	1041.7	973.5	1265.4	9957.8	22087.8	19752.7	18843.1	5978.7	7769.4
10	3668.0	1729.5	1115.1	1081.0	949.0	694.0	885.7	10140.6	18329.6	20493.1	23940.4	12466.9	8011.0
11	5165.5	2213.5	1672.3	1400.4	1138.9	961.1	1069.9	13044.2	13233.4	19506.1	19323.1	16085.6	7954.0
12	6049.3	2327.8	1973.2	1779.9	1304.8	1331.0	1965.0	13637.9	22784.1	19839.8	19480.2	10146.2	8602.9
13	4637.6	2263.4	1760.4	1608.9	1257.4	1176.8	1457.4	11333.5	36017.1	23443.7	19887.1	12746.2	9832.9
14	5560.1	2508.9	1708.9	1308.9	1184.7	883.6	776.6	15299.2	20663.4	28767.4	21011.4	10800.0	9277.7
15	5187.1	1789.1	1194.7	852.0	781.6	575.2	609.2	3578.8	42841.9	20082.8	14048.2	7524.2	8262.7
16	4759.4	2368.2	1070.3	863.0	772.7	807.3	1232.4	10966.0	21213.0	23235.9	17394.1	16225.6	8451.5
17	5221.2	1565.3	1203.6	1060.4	984.7	984.7	1338.4	7094.1	25939.6	16153.5	17390.9	9214.1	7374.4
18	3269.8	1202.2	1121.6	1102.2	1031.3	889.5	849.7	12555.5	24711.9	21987.3	26104.5	13672.9	9095.7
19	4019.0	1934.3	1704.2	1617.6	1560.4	1560.4	1576.7	12826.7	25704.0	22082.8	14147.5	7163.6	8032.2
20	3135.0	1354.9	753.9	619.2	607.5	686.0	1261.6	9313.7	13962.1	14843.5	7771.9	60.0	4912.3
21	2403.1	1020.9	709.3	636.2	602.1	624.1	986.4	9536.4	14399.0	18410.1	16263.8	7224.1	6114.6
22	3768.0	2496.4	1687.4	1097.1	777.4	717.1	813.7	2857.2	27612.8	21126.4	27446.6	12188.9	8588.5
23	4979.1	2587.0	1957.4	1570.9	1491.4	1366.0	1305.4	15973.1	27429.3	19820.3	17509.5	10955.7	8963.4
24	4301.2	1977.9	1246.5	1031.5	1000.2	873.9	914.1	7287.0	23859.3	16351.1	18016.1	8099.7	7112.0
25	3056.5	1354.7	931.6	786.4	689.9	627.3	871.9	12889.0	14780.6	15971.9	13523.7	9786.2	6313.7
26	3088.8	1474.4	1276.7	1215.8	1110.3	1041.4	1211.2	11672.2	26689.2	23430.4	15126.6	13075.3	8402.7
27	5679.1	1601.1	876.2	757.8	743.2	690.7	1059.8	8938.8	19994.0	17015.3	18393.5	5711.5	6834.8
28	2973.5	1926.7	1687.5	1348.7	1202.9	1110.8	1203.4	8569.4	31352.8	19707.3	16807.3	10613.1	8232.6
29	5793.9	2645.3	1979.7	1577.9	1267.7	1256.7	1408.4	11231.5	17277.2	18385.2	13412.1	7132.6	6992.2
30	3773.9	1944.9	1312.6	1136.8	1055.4	1101.2	1317.9	12369.3	22904.8	24911.7	16670.7	9096.7	8183.7
31	6150.0	3525.0	2032.0	1470.0	1233.0	1177.0	404.0	10140.0	00.0	26740.0	18000.0	11000.0	8907.9
32	6458.0	3297.0	1385.0	1147.0	971.0	889.0	1103.0	10406.0	17017.0	27840.0	31435.0	12026.0	9580.4
MAX	6458.0	3525.0	2258.5	1779.9	1560.4	1560.4	1965.0	15973.1	42841.9	28767.4	31435.0	17205.5	9832.9
MIN	2403.1	1020.9	709.3	619.2	602.1	569.1	609.2	2857.2	13233.4	14843.5	7771.9	4260.0	4912.3
MEAN	4513.1	2052.4	1404.8	1157.3	978.9	898.3	1112.6	10397.6	22912.9	20778.0	18431.4	10670.4	7985.9

TABLE F.2: PRE-PROJECT FLOW AT DEVIL CANYON (CFS)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
1	5758.2	2404.7	1342.5	951.3	735.7	670.0	802.2	10490	18468.6	21383.4	18820.6	7950.8	7537.8
2	3652.0	1231.2	1030.8	905.7	767.5	697.1	1504.6	13218.5	19978.5	21575.9	18530.0	19799.1	8615.9
3	5221.7	2539.0	1757.5	1483.7	943.2	828.2	878.5	4989.5	30014.2	24861.7	19647.2	13441.1	8918.0
4	7517.6	3232.6	1550.4	999.6	745.6	766.7	1531.8	17758.3	25230.7	19184.0	19207.0	13928.4	9356.4
5	5109.3	1921.3	1387.1	1224.2	929.7	729.4	1130.6	15286.0	188.1	19154.1	24061.6	11579.1	866.9
6	4830.4	2506.8	1868.0	1649.1	1275.2	1023.6	1107.4	8390.1	28081.9	26212.8	24959.6	13989.2	9707.4
7	1647.9	1788.6	1206.6	921.7	893.1	852.3	867.3	15979.0	31137.1	29212.0	2609.8	16495.8	10608.2
8	5235.3	2773.8	1986.6	1583.2	1388.9	1105.4	1109.0	12473.6	28415.4	22109.6	19389.2	18029.0	9668.7
9	7434.5	3590.4	2904.9	1792.0	1212.2	1085.7	1437.4	11849.2	24413.5	21763.1	21219.8	6988.8	8866.8
10	4402.8	1999.8	1370.9	1316.9	1179.1	877.9	1119.9	13900.9	21537.7	23390.4	28594.4	15329.6	9649.6
11	6060.7	2622.7	2011.5	1686.2	1340.2	1112.8	1217.8	14802.9	14709.8	21739.3	22066.1	18929.9	9084.4
12	7170.9	2759.9	2436.6	2212.0	1593.6	1638.9	2405.4	16030.7	27069.3	22880.6	21164.4	12218.6	10021.3
13	5459.4	2544.1	1978.7	1796.0	1413.4	1320.3	1613.4	12141.2	40679.7	24990.6	22241.8	14767.2	10946.5
14	6307.7	2696.0	1896.0	1496.0	1387.4	958.4	810.9	17697.6	24094.1	32388.4	22720.5	11777.2	10431.8
15	5998.3	2085.4	1387.1	978.0	900.2	663.8	696.5	4046.9	47816.4	21926.0	15585.8	8840.0	9250.7
16	5744.0	2645.1	1160.8	925.3	828.8	866.9	1214.4	12267.1	24110.3	26195.7	19789.3	18234.2	9555.5
17	6496.5	1907.8	1478.8	1478.4	1278.7	1187.4	1619.1	8734.0	30446.3	18536.2	20244.6	10844.3	8697.0
18	3844.0	1457.9	1364.9	1357.9	1268.3	1089.1	1053.7	14435.5	27796.4	25081.2	30293.0	15728.2	10460.4
19	4885.3	2203.5	1929.7	1851.2	1778.7	1778.7	1791.0	14982.4	29462.1	24871.0	16090.5	8225.9	9175.4
20	3576.7	1531.8	836.3	686.6	681.8	769.6	1421.3	10429.9	14950.7	15651.2	8483.6	4795.5	5352.1
21	2866.5	1145.7	810.0	756.9	708.7	721.8	1046.6	10721.6	17118.9	21142.2	18652.8	8443.5	7063.9
22	4745.2	3081.8	2074.8	1318.8	943.6	866.8	986.2	3427.9	31031.0	22941.6	30315.9	13636.0	9657.2
23	5537.0	2912.3	2312.6	2036.1	1836.4	1 59.8	1565.5	19776.8	31929.8	21716.5	18654.1	11884.2	10199.0
24	4638.6	2154.8	1387.0	1139.8	1128.6	955.0	986.7	7896.4	26392.6	17571.8	19478.1	8726.0	7738.3
25	3491.4	1462.9	997.4	842.7	745.9	689.5	949.1	15004.6	16766.7	17790.0	15257.0	11370.1	7160.5
26	3506.8	1619.4	1486.5	1408.8	1342.2	1271.9	1456.7	14036.5	30302.6	26188.0	17031.6	15154.7	9609.6
27	7003.3	1853.0	1007.9	896.8	876.2	825.2	1261.2	11305.3	22813.6	18252.6	19297.7	6463.3	7705.5
28	3552.4	2391.7	2147.5	1657.4	1469.7	1361.0	1509.8	11211.9	35606.7	21740.5	18371.2	11916.1	9438.8
29	6936.3	3210.8	2371.4	1867.9	1525.0	1480.6	1597.1	11693.4	18416.8	20079.0	15326.5	8080.4	7765.1
30	4502.3	2324.3	1549.4	1304.1	1203.6	1164.7	1402.8	13334.0	24052.4	27462.8	19106.7	10172.4	9023.0
31	6900.0	3955.0	2279.0	1649.0	1383.0	1321.0	1575.0	11377.0	26255.0	30002.0	20196.0	12342.0	9994.5
32	7246.0	3699.0	1544.0	1287.0	1089.0	997.0	1238.0	11676.0	17741.0	31236.0	35270.0	12762.0	10577.9
MAX	7517.6	3955.0	2904.9	2212.0	1836.4	1778.7	2405.4	19776.8	47816.4	32388.4	35270.0	19799.1	10946.5
MIN	2866.5	1145.7	810.0	686.6	681.8	663.8	696.5	3427.9	14709.8	15651.2	8483.6	4795.5	5352.1
MEAN	5311.8	2382.9	1652.0	1351.9	1146.9	1041.8	1281.5	12230.2	25938.4	23100.9	20709.0	12276.3	9084.4

TABLE F.3: TYPICAL NOAA CLIMATE DATA RECORD

Meteorological Data For 1976

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

Month	Temperature °F – Averages – Daily maximum	Daily minimum	Monthly	Extremes – Highest	Date	Lowest	Date	Degree days Base 65 °F – Heating	Cooling	Precipitation in inches – Water equivalent – Total	Greatest in 24 hrs.	Date	Snow, Ice pellets – Total	Greatest in 24 hrs.	Date	Relative humidity, pct. – Hour 02 (Local time)	Hour 08	Hour 14	Hour 20	Wind – Resultant – Direction	Speed m.p.h.	Average speed m.p.h.	Fastest mile – Speed m.p.h.	Direction	Date	Percent of possible sunshine	Average sky cover, tenths, sunrise to sunset	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 ¼ mile or less	Temperature °F – Maximum – (b) 90° and above	32° and below	Minimum – 32° and below	0° and below	Average station pressure mb – Elev. 2405 feet m.s.l.
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				28	23	30		6.0	11	4	16	12	7	0	2	0	29	31	20	
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				31	07	23		3.9	17	4	8	7	6	0	0	0	27	29	24	
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					35	07	17		8.0	4	4	23	11	8	0	0	0	31	31	15	
APR	36.3	14.5	25.4	51	30	-3	15	1180	0	0.14	0.08	26	9.8	3.1	26		68						20	08	14		6.2	8	8	14	3	2	0	0	0	8	30	2	
MAY	43.6	29.4	36.5	54	2	17	7	878	0	2.98	1.90	8	8.7	2.6	8		69						17	24	18		7.5	5	6	20	7	4	0	0	0	0	27	0	
JUN	60.6	40.9	50.8	74	27	34	8	420	0	0.51	0.30	30	0.0	0.0			69						18	22	17		6.9	6	8	16	4	0	0	0	1	0	0	0	
JUL	62.1	43.6	52.9	76	23	33	6	368	0	1.05	0.33	23	0.0	0.0			81						29	23	27		8.1	3	7	21	14	0	0	1	4	0	0	0	
AUG	62.8	41.8	52.3	78	2	31	29	383	0	0.96	0.20	7	0.0	0.0			80						20	26	7						13	0			5	0	1	0	
SEP	49.8	31.7	40.8	59	14	16	30	718	0	1.59	0.48	9	0.4	0.3	20		76						25	25	19		7.6	3	9	18	13	0	0	2	0	0	17	0	
OCT																							20	08	12														
YEAR																																							

Normals, Means, And Extremes - THROUGH 1975#

Month	Temperatures °F – Normal – Daily maximum	Daily minimum	Monthly	Extremes – Record highest	Year	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 pellets – Maximum monthly	Year	Maximum in 24 hrs.	Year	Relative humidity pct. – Hour 02 (Local time)	Hour 08	Hour 14	Hour 20	Wind – @ Mean speed m.p.h.	Prevailing direction	Fastest mile – Speed m.p.h.	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 ¼ mile or less	Temperatures °F – Max. – (b) 90° and above	32° and below	Min. – 32° and below	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
J	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
F	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	48	07	1974		7.0	6	5	17	10	5	0	1	0	26	28	15	918.8
M	19.4	3.0	11.2	49	1961	-35	1971	1668	0	1.04	4.53	1946	0.07	1961	1.67	1946	39.1	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
A	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
M	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
J	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
J	60.2	43.8	52.0	81	1961	32	1970	403	0	3.09	5.58	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
A	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
S	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	1959	85	81	59	75	7.5	NE	32	23	1972		7.4	5	5	20	16	2	*	1	*	1	14	0	924.1
O	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	5	5	21	13	7	0	2	0	18	30	2	916.7
N	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	11.3	NE	39	25	1970		7.1	7	4	19	9	5	0	1	0	27	30	13	921.3
D	9.2	-3.4	2.9	42	1969	-43	1961	1925	0	1.20	4.63	1951	0.24	1949	1.09	1967	90.7	1970	27.4	1970	76	78	76	77	12.7	NE	44	11	1970		8.5	9	5	17	11	6	0	1	0	30	31	19	914.7
YR	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.4: SUMMARY OF CLIMATOLOGICAL DATA

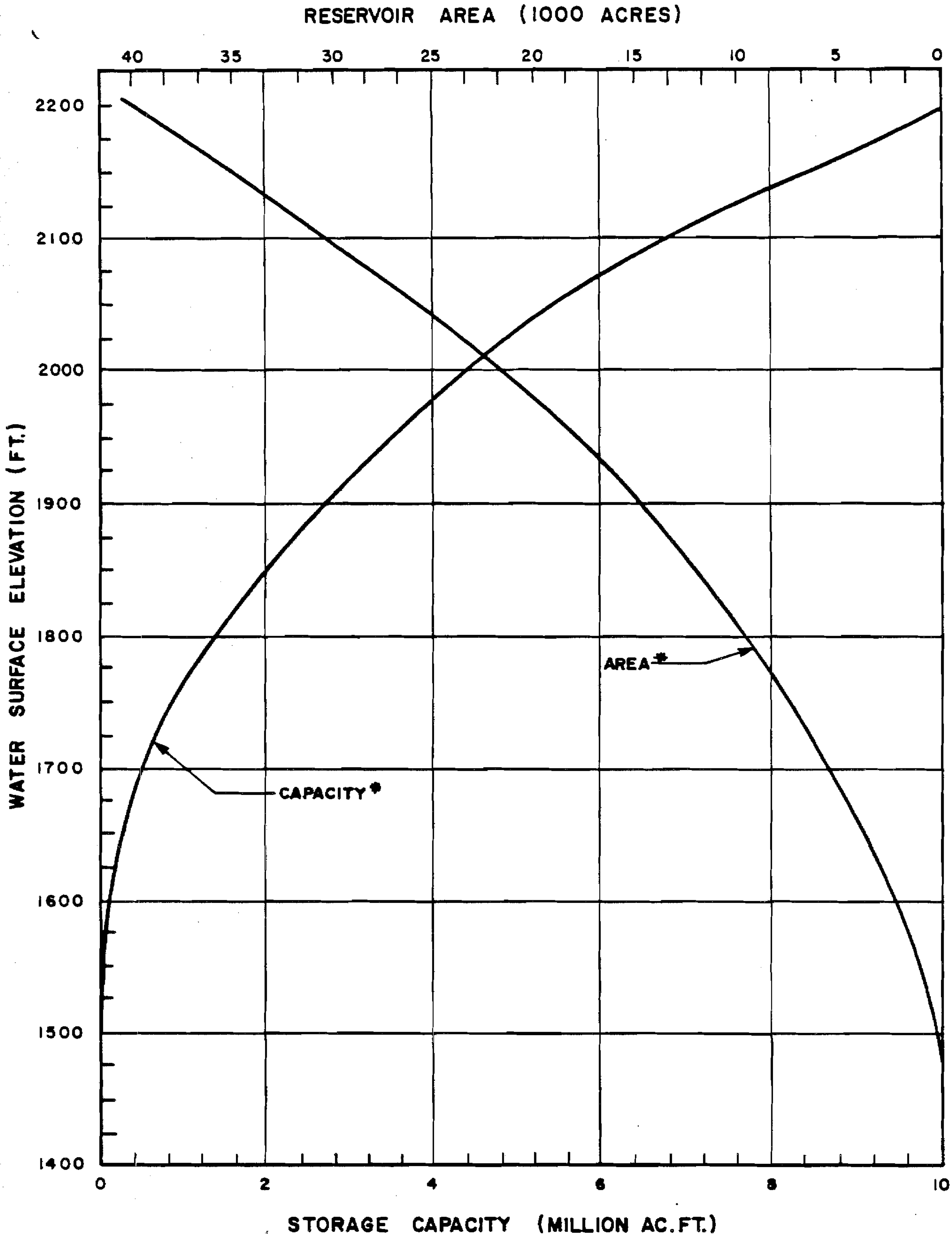
MEAN MONTHLY PRECIPITATION (Inches)													
STATION	JAN	FEB	MAR	APR	MAY	JUN	JULY	AUG	SEPT	OCT	NOV	DEC	ANNUAL
Anchorage	0.84	0.56	0.56	0.56	0.59	1.07	2.07	2.32	2.37	1.43	1.02	1.07	
Big Delta	0.36	0.27	0.33	0.31	0.94	2.20	2.49	1.92	1.23	0.56	0.41	0.42	11.44
Fairbanks	0.60	0.53	0.48	0.33	0.65	1.42	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	19.03
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)													
Anchorage	11.8	17.8	23.7	35.3	46.2	54.6	57.9	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 Park	- 2.7	4.8	11.5	26.4	40.8	51.5	54.2	50.2	40.8	23.0	8.9	- 0.1	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.5: 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

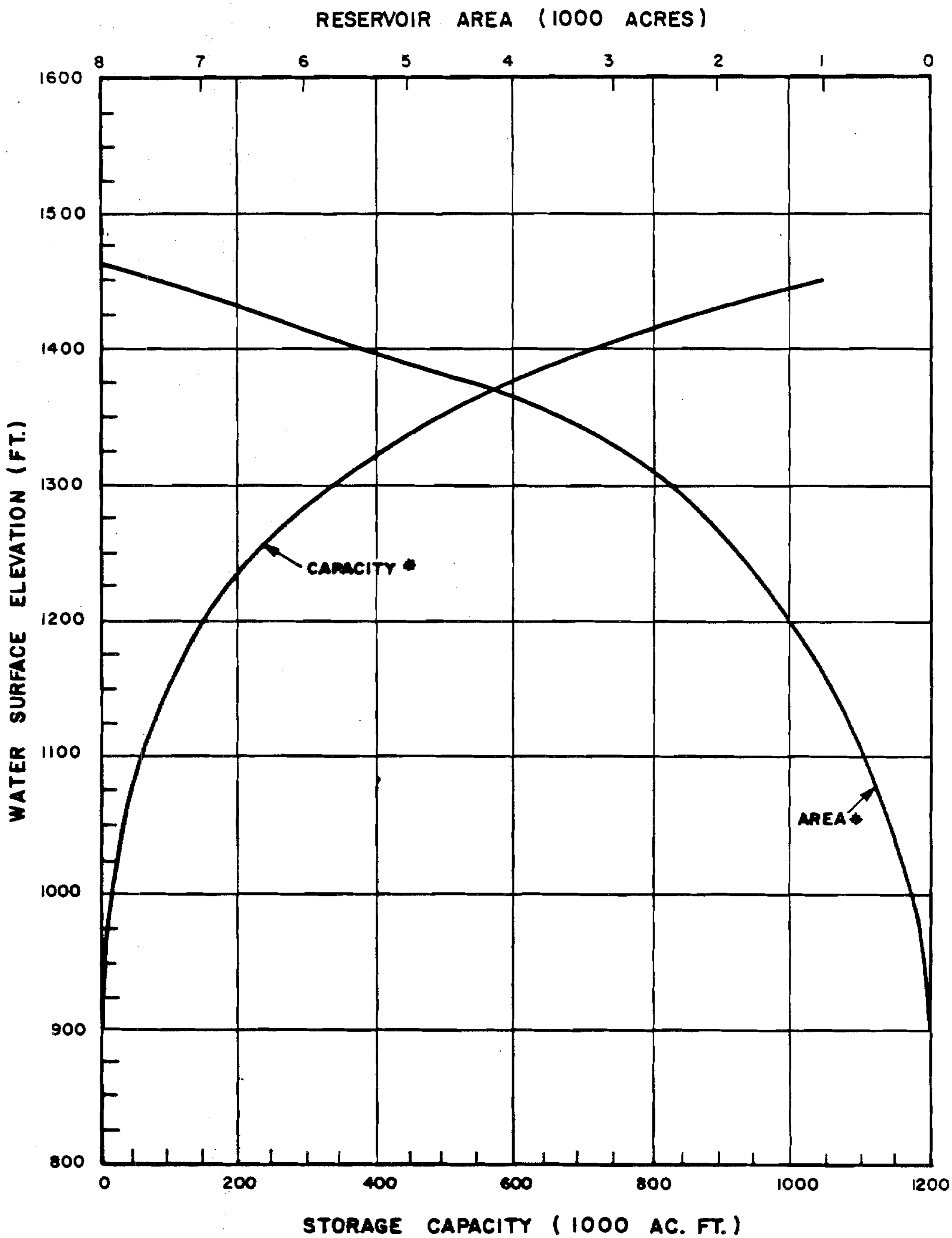
FIGURES



AREA AND CAPACITY CURVES
WATANA RESERVOIR

*WITHOUT RESERVOIR SILTATION

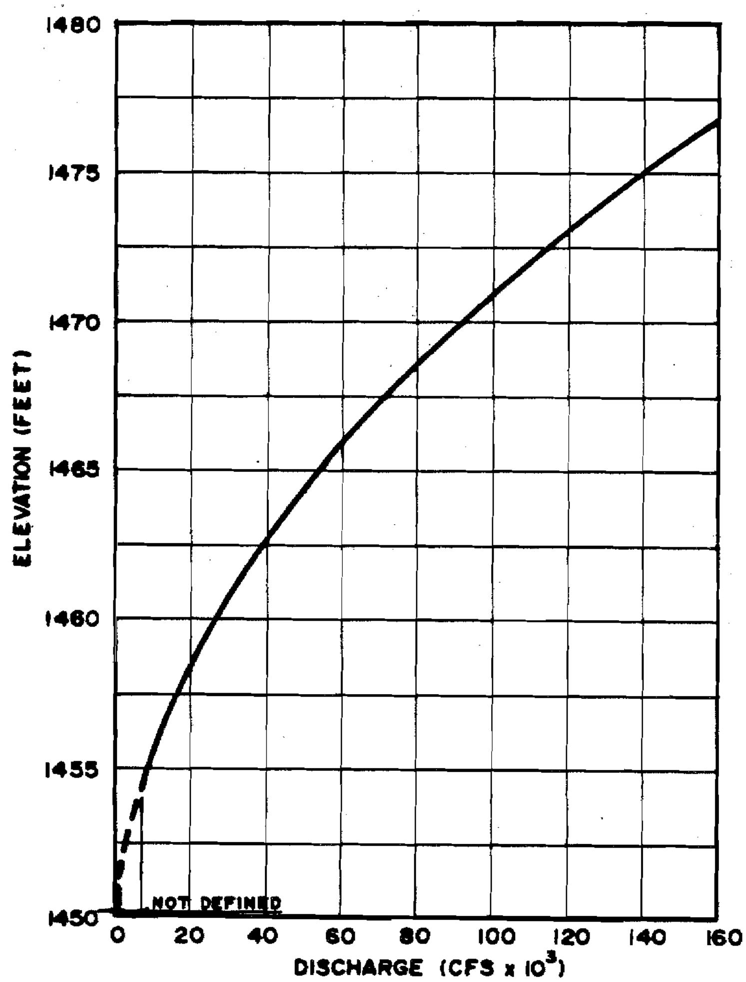
FIGURE F.1



AREA AND CAPACITY CURVES
DEVIL CANYON RESERVOIR

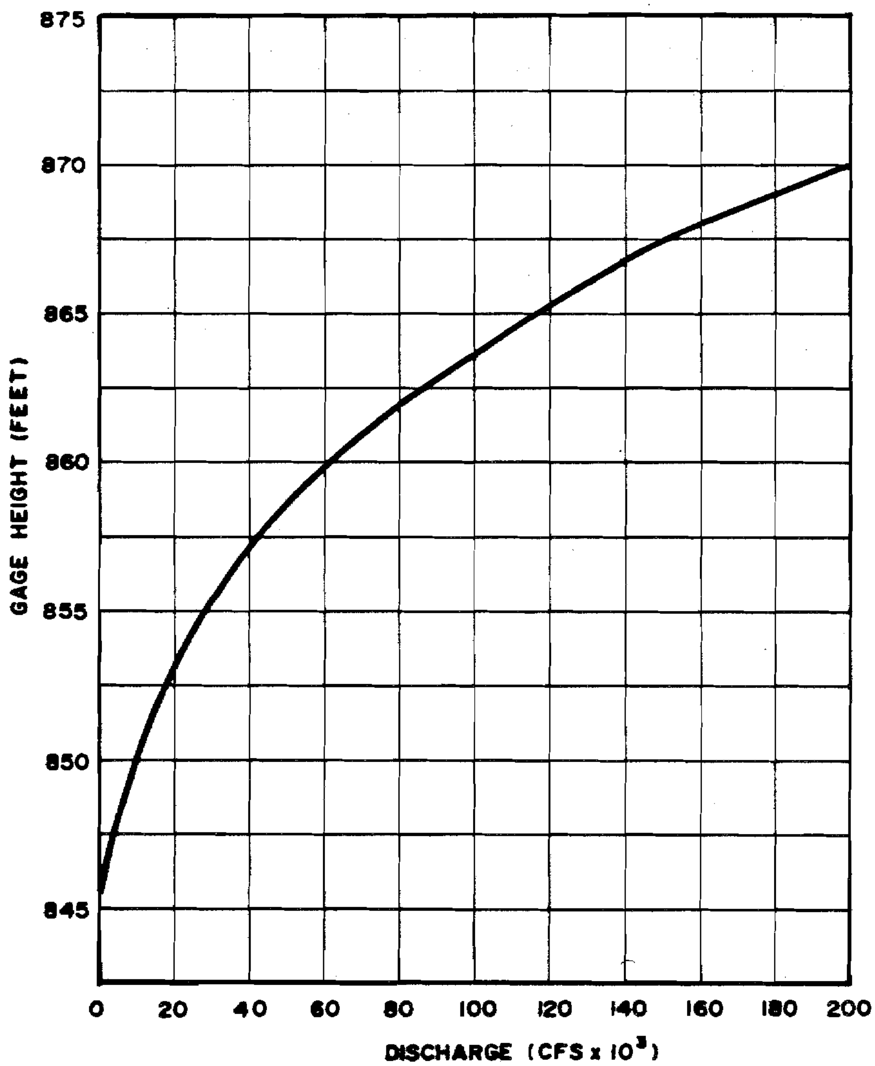
*WITHOUT RESERVOIR SILTATION

FIGURE F.2



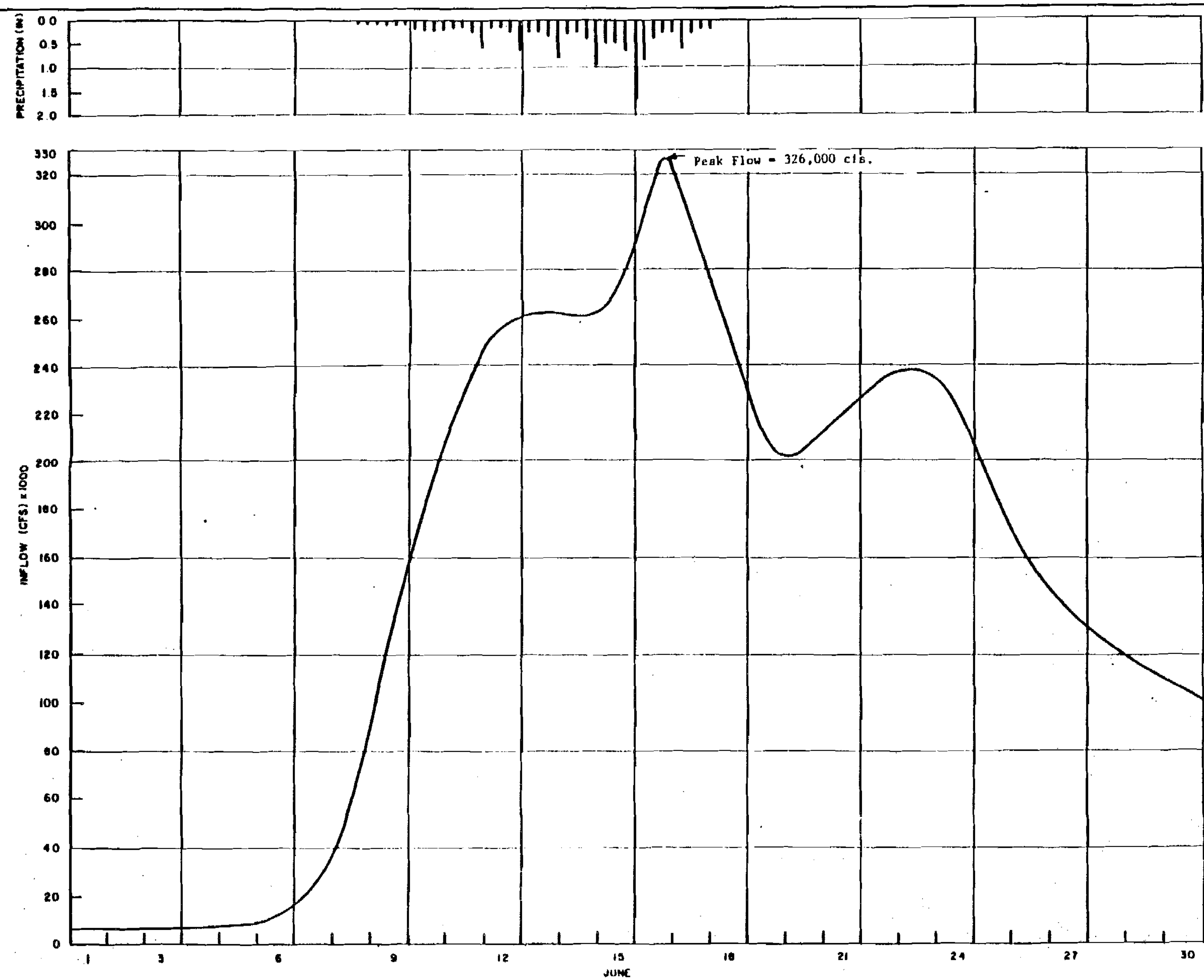
WATANA TAILWATER RATING

FIGURE F.3



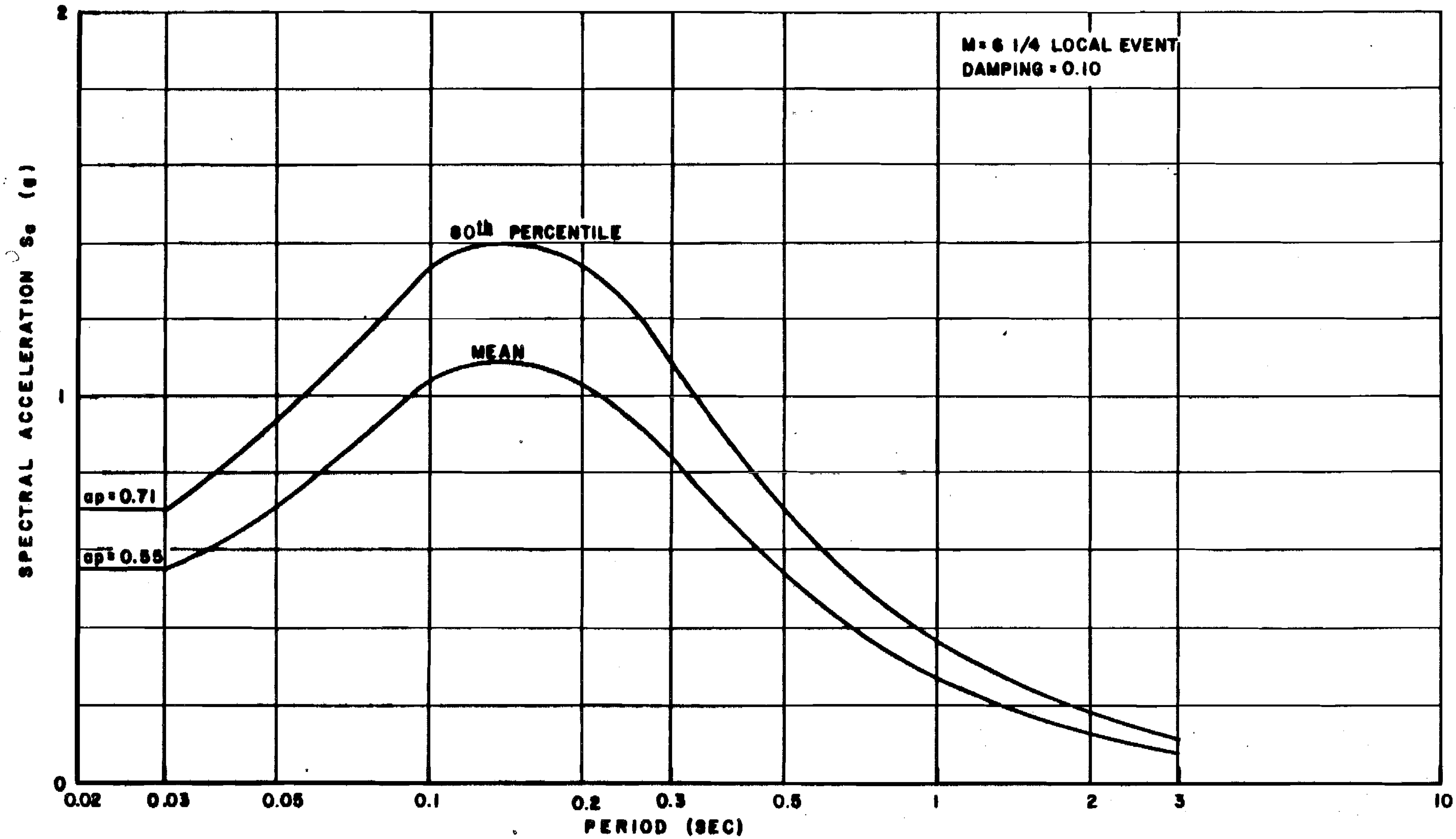
DEVIL CANYON
TAILWATER RATING
(TAILRACE TO PORTAGE CREEK)

FIGURE F.4



WATANA PMF INFLOW HYDROGRAPH

FIGURE E4A



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

FIGURE F.5

APPENDIX FB

APPENDIX FB WATANA AND DEVIL CANYON EMBANKMENT STABILITY ANALYSES

1 - Preliminary Design

1.1 - General

Stability analyses for the Watana Main Dam and the Devil Canyon Saddle Dam embankments have been conducted in sufficient detail to satisfy project feasibility. The following paragraphs summarize these evaluations along with the spillway fuse plug embankments for both dams.

1.2 - Watana Main Dam and Devil Canyon Saddle Dam

Although only the Watana main dam maximum cross-section has been analyzed, the safety factors also apply to the Devil Canyon Saddle Dam, which has the same configuration but a much lower height. The embankment design (cross-section and foundation treatment) is essentially the same for both embankments (figures FB-1 and FB-2). The quoted safety factors derived from the ±830 foot high dam are conservative for the ±150 high saddle dam.

a. Methodology

The static analyses were performed using the STABL computer program developed to handle general slope stability problems by adaptation of the Modified Bishop method, and FEADAM, a finite element program for static analysis of earth and rockfill dams, to determine the initial stresses in the dam during normal operating conditions.

The dynamic analyses were performed using the QUAD 4 finite element program which incorporates strain-dependent shear modulus and damping parameters.

b. Static Analysis

Loading Conditions and Factors of Safety

Case	Required Minimum Factor of Safety	Calculated Factor of Safety	
		U/S Slope	D/S Slope
Construction	1.3	2.0	1.7
Normal Maximum Operating	1.5	2.0	1.7
Maximum Reservoir Drawdown	1.0	1.8	1.7
Maximum Reservoir Level During PMF	1.3	2.0	1.7

c. Seismic Stability Evaluation

The safety factor evaluation of the embankment seismic stability was based on a comparison of available shear strength to the earthquake induced shear stresses. A shear stress exceedance ratio was utilized to represent an indication of the embankment stability. This ratio is considered to represent a factor of safety against reaching a strain level of 5 percent for a particular element within the dam cross section. In this type analysis, a ratio less than 1.0 indicates satisfactory performance. Experience on embankments, which have been subjected to earthquake loading indicates that if the strain within the dam is less than 5 percent, the earthquake had little or no effect on the stability of the dam. Experience also indicates that the integrity of the dam is not compromised if the strain exceeds 5 percent at some locations. The effect of larger strains depends on the extent and location of the occurence. Localized shear stress exceedance adjacent to exterior slopes or near the crest are to be expected and do not indicate overall dam stability problems.

During the period of earthquake shaking very little dissipation of excess pore pressures is expected in the impervious core; therefore, the stability should be evaluated on the basis of undrained conditions. In the case of the shells the analysis has utilized both drained and undrained conditions. As there should be negligible build up of pore pressures in these high permeability granular materials, stability should be evaluated on the basis of drained conditions. Evaluations of the plots of the drained and undrained shear stress exceedance for a soft and stiff core on Figures FB-4, FB-5, FB-6 and FB-7 indicate only limited zones of shear stress exceedance adjacent to the toe of the upstream shell, near the crest and in the surface layer of the downstream shell. These are localized zones indicating the dam is safe, as the overall stability is not affected by the seismic action.

d. <u>Conclusions</u>

Static and dynamic analyses confirm the stability of the upstream and downstream slopes of the proposed cross-sections of the Watana dam shown on Figure FB-1. 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.

1.3 - Spillway Fuse Plug Embankments

The emergency spillway fuse plug embankments utilize exterior slopes and fill materials similar to the dam embankments (Figure FB-1 and FB-2).

a. <u>Methodology</u>

The stability studies for the fuse plug embankments have been carried out using a computer programmed Morgenstern-Price method of analysis. The static analyses have been investigated for loading conditions simulating the construction case, normal maximum operation (steady seepage) and maximum reservoir drawdown (sudden drawdown). The seismic analysis utilized only the latter two cases. Newmark (reference 1), Makdisi, and Seed (reference 2) methods were used in a simplified permanent deformation analysis.

b. Static Analysis

Loading Conditions and Factors of Safety

Case	Required Minimum Factor of Safety	Calculated Factor of Safety U/S Slope
Construction	1.3	1.75
Normal Maximum Operation	1.5	2.10
Rapid.Reservoir Drawdown	1.0	2.28

c. Seismic Stability Evaluation

Loading Conditions and Factors of Safety
(Pseudo - Static Analysis)

Case	Required Minimum Factor of Safety	Calculated Factor of Safety U/S Slope
Normal Maximum Operation	1.0	1.04
Rapid Reservoir Drawdown	1.0	1.19

d. Simplified Permanent Deformation Analyses

Loading Conditions and Crest Settlement
Resulting From Seismic Shaking

Condition (prior to earthquake)	Settlement (inches) Newmark	Settlement (inches) Makdisi & Seed
Steady Seepage	8.5	3.3
Rapid Drawdown	2.1	1.8

e. Conclusions

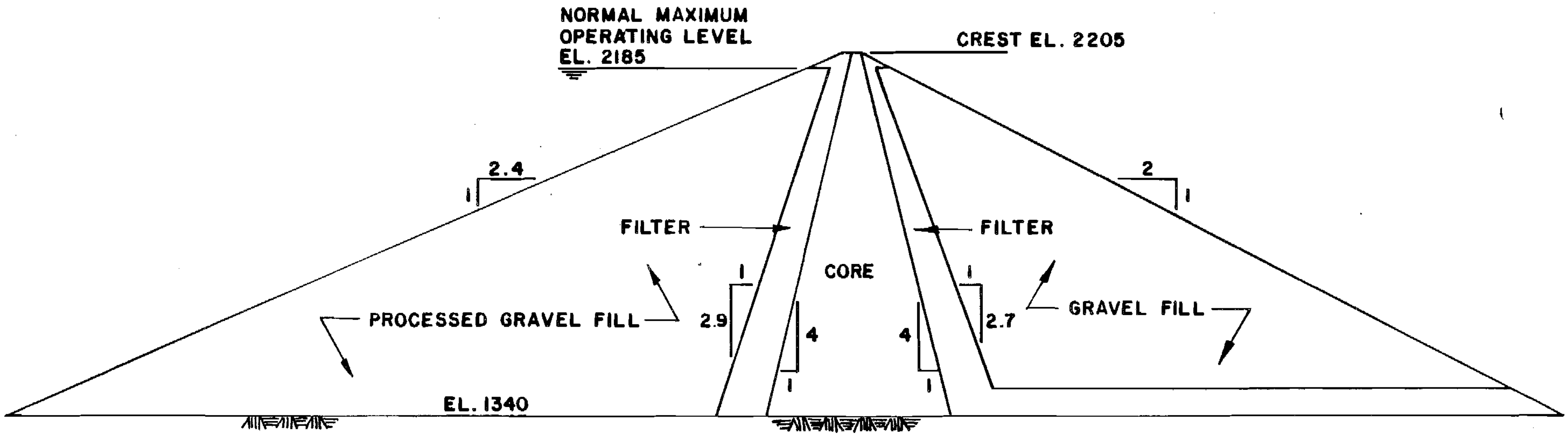
The calculated factors of safety indicate slope stability under static loading conditions. A preliminary simplified embankment response analysis carried out using the seismic safety evaluation earthquake (maximum credible earthquake) as base excitation, indicate the embankment is safe.

The anticipated effects on the fuse plug embankment caused by the seismic safety evaluation earthquake are modest. The estimated permanent displacement of the crest could be up to a maximum of 8.5 inches.

References

1. Newmark, N.M., "Effects of Earthquakes on Dams and Embankments", Geotechnique, Volume XV, No. 2, 1965.

2. Makdisi, F.I., and Seed, H.B., "Simplified Procedure for Estimating Dam and Embankments Earthquake Induced Deformations", ASCE Journal Geotechnical Engineering Division, Volume 104, GT 7, 1978.



NOTE:
FOR DETAILED CROSSECTION SEE PLATE 9
IN VOLUME 3 OF FEASIBILITY REPORT.

WATANA DAM

MAXIMUM CROSS SECTION

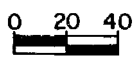
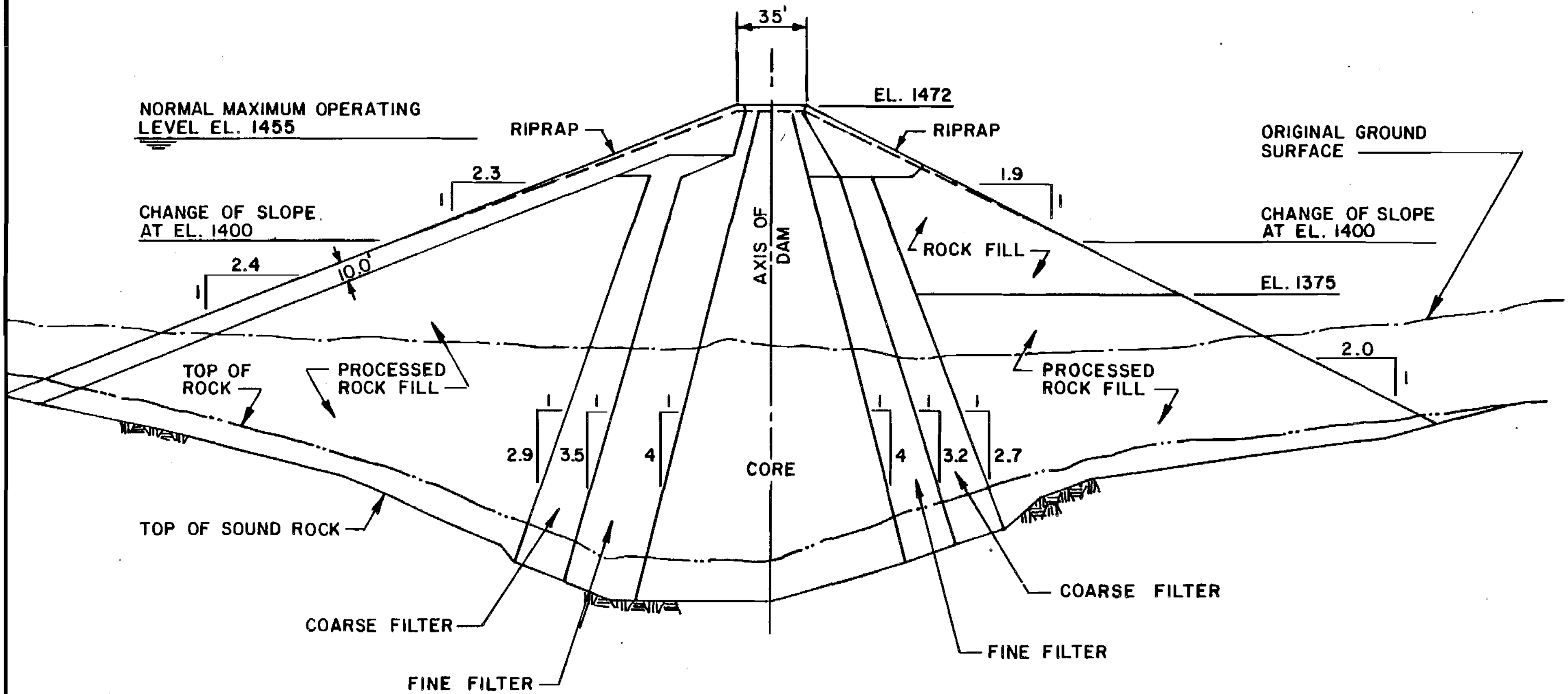
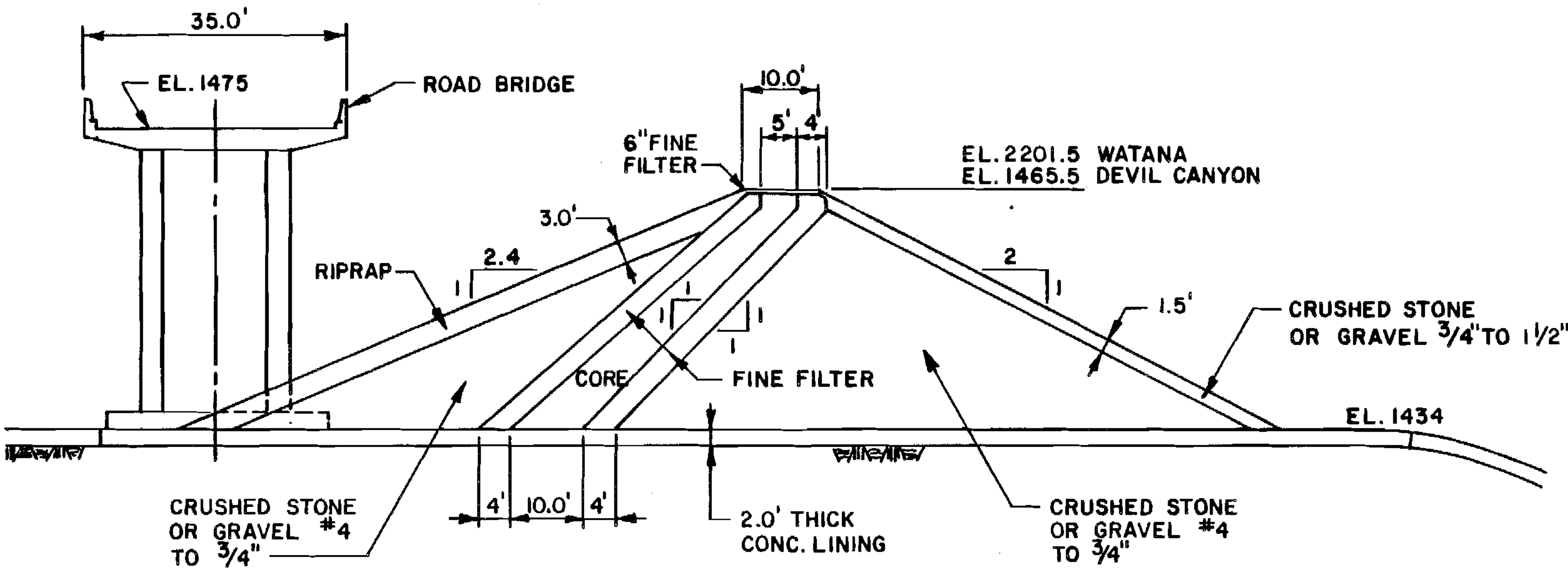


FIGURE FB - 1



DEVIL CANYON
SECTION THROUGH SADDLE DAM
AT MAXIMUM HEIGHT

FIGURE FB - 2



DEVIL CANYON AND WATANA
TYPICAL SECTION THROUGH FUSE PLUG

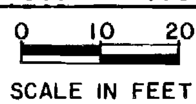
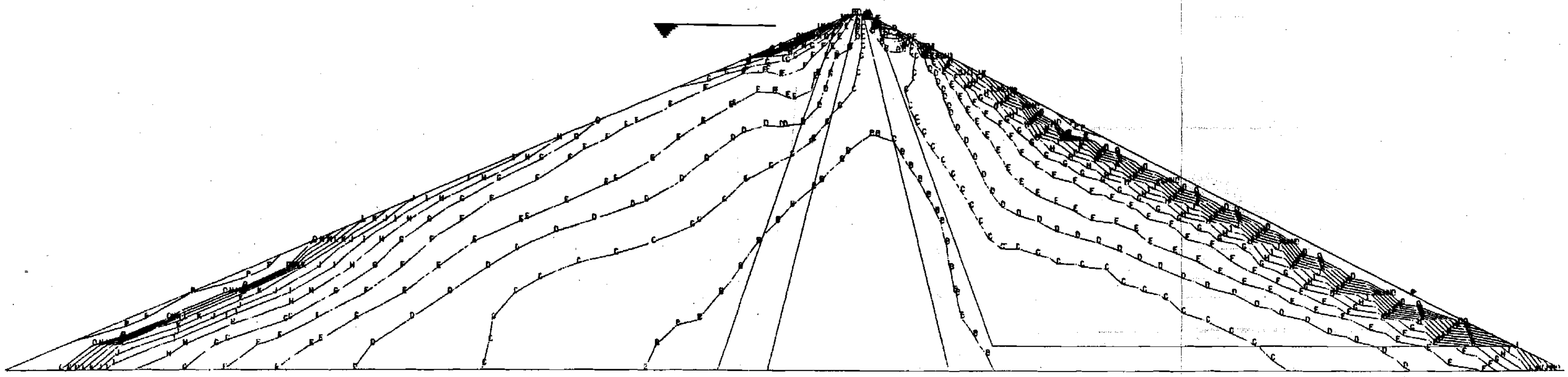


FIGURE FB - 3

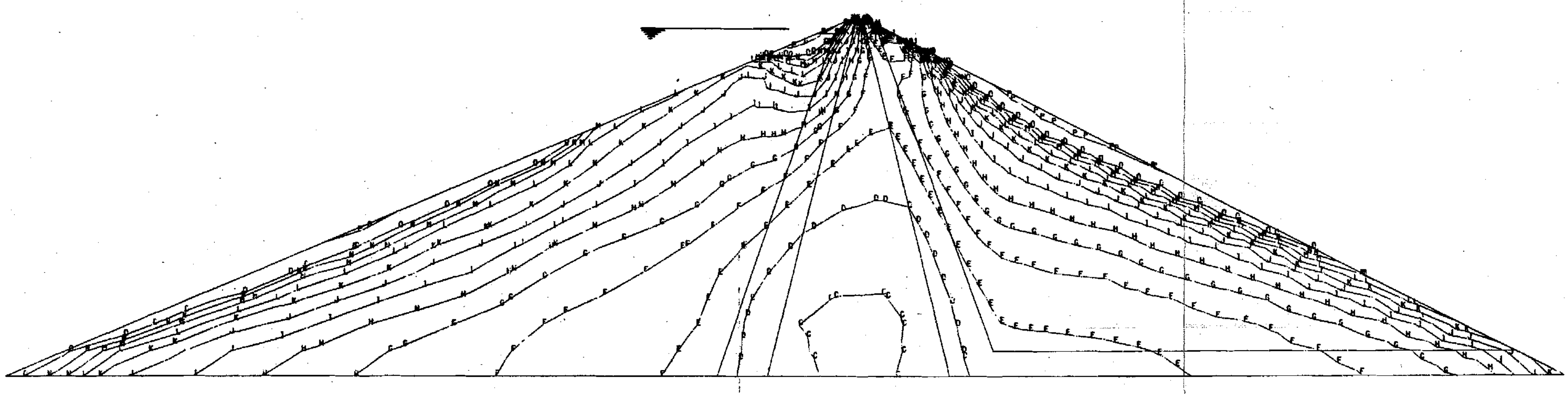


SYMBOL	A	B	C	D	E	F	G	H	I	J
VALUE	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
SYMBOL	K	L	M	N	O	P	Q	R	S	
VALUE	1.1	1.2	1.3	1.4	1.5	2.0	4.0	6.0	8.0	

DYNAMIC RUN SOFT CORE

DRAINED SHEAR STRESS EXCEEDANCE (TAUdeff/TAUfd)

FIGURE FB - 4



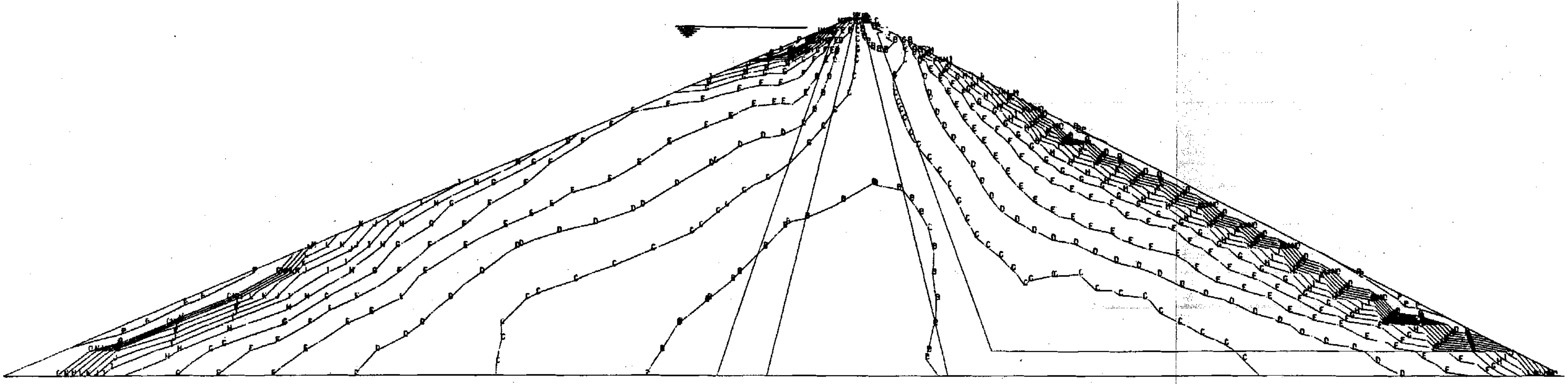
SYMBOL	A	B	C	D	E	F	G	H	I	J
VALUE	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
SYMBOL	K	L	M	N	O	P	Q	R	S	
VALUE	1.1	1.2	1.3	1.4	1.5	2.0	4.0	6.0	8.0	

DYNAMIC RUN SOFT CORE

UNDRAINED SHEAR STRESS EXCEEDANCE (TAU d max / TAU c)

FIGURE FB - 5

ACRES



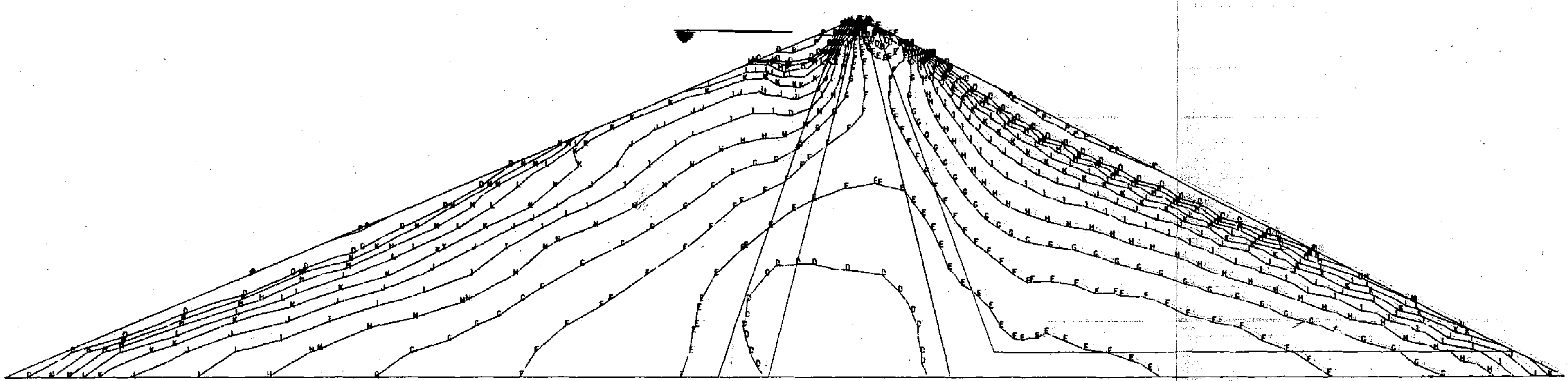
SYMBOL	A	B	C	D	E	F	G	H	I	J
VALUE	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
SYMBOL	K	L	M	N	O	P	Q	R	S	
VALUE	1.1	1.2	1.3	1.4	1.5	2.0	4.0	6.0	8.0	

DYNAMIC RUN STIFF CORE

DRAINED SHEAR STRESS EXCEEDANCE (TAUdeff/TAUfd)

FIGURE FB - 6

ACRES



SYMBOL	A	B	C	D	E	F	G	H	I	J
VALUE	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
SYMBOL	K	L	M	N	O	P	Q	R	S	
VALUE	1.1	1.2	1.3	1.4	1.5	2.0	4.0	6.0	8.0	

DYNAMIC RUN STIFF CORE

UNDRAINED SHEAR STRESS EXCEEDANCE (TAUdmax/TAUc)

FIGURE FB - 7

ACRES

APPENDIX FC

APPENDIX FC - SUMMARY OF PMF AND SPILLWAY DESIGN FLOOD ANALYSES

Introduction

The inflow PMF peaks are estimated to be 326,000 cubic feet per second (cfs) for Watana, and 346,000 cfs (routed through Watana) and 362,000 cfs (unrouted through Watana) for Devil Canyon. The 10,000-year flood peaks are estimated to be 156,000 cfs at Watana, and 161,000 cfs (unrouted) and 165,000 cfs (routed) at Devil Canyon. The increase in the routed 10,000-year peak flow over the natural flood resulted because of the synchronization of routed flood peak and peak from the intervening area between the two developments. The major work tasks performed to derive the PMF and 10,000-year flood peaks are summarized below. Figures and tables are provided to supplement the summary.

Probable Maximum Flood (PMF)

1 - Calibration of SSARR Model

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[1/].

The model was calibrated by U.S. Army Corps of Engineers (COE)[2/] for the Susitna River basin above Gold Creek, a stream gaging station located about 12 miles downstream from the Devil Canyon damsite (Figure FC-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[1/]. Figure FC-2 provides a schematic representation of the basic elements of the SSARR model.

[1/] U.S. Army Corps of Engineers, September 1972 (revised June 1975): Program Description and User Manual for SSARR Model, Program 724-KJ-G0010, Portland, Oregon.

[2/] U.S. Army Corps of Engineers: Interim Feasibility Report, South Central Rainbelt Area, Alaska, Appendix I, Part 1, Section A, 1975, and Supplemental Feasibility Report, 1979.

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 FC-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 FC-3). Daily precipitation or snow water equivalent data observed at Summit, Trims Camp, Paxson, Gulkana or Gracious House (see Figure FC-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 1 shows the comparsion of observed and simulated flood peaks. The simulated and observed hydrographs are shown on Figure FC-4 through FC-10. The derived relationships between the model parameters are shown on Figures FC-11 through FC-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 FC-11 and FC-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 FC-18 through FC-26 show the simulated and observed hydrographs. Table 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 3.

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

Table 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 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 FC-15)	5001	5001	5001	5001	5001	5001	5001	5001	5001	5001	5001	5001	5001
Table No. QGEN vs. SCA (Figure FC-16)	6004	6006	6006	6006	6004	6006	6006	6006	6006	6006	6006	6006	6006
Table No. for Month vs ETI (Figure FC-14)	4009	4008	4008	4008	4009	4008	4008	4008	4008	4008	4008	4008	4008
Table No. for SMI vs ROP (Figure FC-11)	1015	1018	1018	1018	1015	1018	1018	1022	1021	1018	1020	1020	1020
Table No. for BII vs BFP (Figure FC-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 FC-13)	3009	3008	3008	3008	3009	3003	3008	3008	3008	3008	3008	3008	3008
Table No. for QGEN vs MELTR (Figure FC-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

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 FC-27 through FC-32.

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 Table 4.

Table 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
Percent area below	0	35.9	74.4	97.1	99.7	99.9

Sub-basin 180						
Elevation, ft	2350	3000	4000	5000	6000	6200
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

TABLE 4

MAXIMIZATION FACTORS

Storm	Storm Dewpoint at 1,000 mb		Max. Dewpoint at 1,000 mb		Max. Factor
	Temp. (°F)	Precip. Water (mm)	Temp. (°F)	Precip. Water (inch)	
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 percipitation over the basin above Watana was computed using the isohyetal pattern developed for six storms (Figure FC-27 through FC-32). These precipitation amounts were multiplied by the maximization factors resulting in maximized total precipitation given in Table 5.

TABLE 5

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.

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 and third largest on the ninth day. The entire pattern is shown in Table 6.

TABLE 6

TEMPORAL PATTERN OF PMP

	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. Table 7 gives the 6-hourly distribution pattern for the PMP over the drainage basin above Watana.

3 - Snowmelt Criteria

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. Table 8 gives the estimated initial snowpack for each sub-basin.

TABLE 7

6-HOURLY DISTRIBUTION PATTERN

Day	Hour	PMP (inch)	Day	Hour	PMP (inch)	Day	Hour	PMP (inch)
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	.10		12	.32			
	18	.15		18	.43			
	24	.35		24	1.08			

TABLE 8

INITIAL SNOWPACK FOR PMF

Sub-basin	Snowpack (in.)	Sub-basin	Snowpack (in.)
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 FC-33. Temperature 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.

4 - Occurrence of Snowmelt and PMP Storm

The snowmelt starts on June 3 based on the adapted temperature sequences (Figure FC-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.

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 baseflow components. Relatively moist soil conditions were assumed for each sub-basin. Table 9 gives the initial values used for the model parameters.

6 - PMF

The calibrated relationships of the model parameters shown in Figures FC-11 through FC-17 and the initial values of parameters shown in Table 9 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 FC-34 and FC-35 show the inflow hydrographs at the two sites.

TABLE 9

INITIAL VALUES OF SSARR MODEL PARAMETERS

Sub-basin	Soil Moisture (in)	Baseflow Infiltration Index (in/day)	Runoff: Surface (cfs)	Runoff: Sub-Surface (cfs)	Runoff: Base-flow (cfs)
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

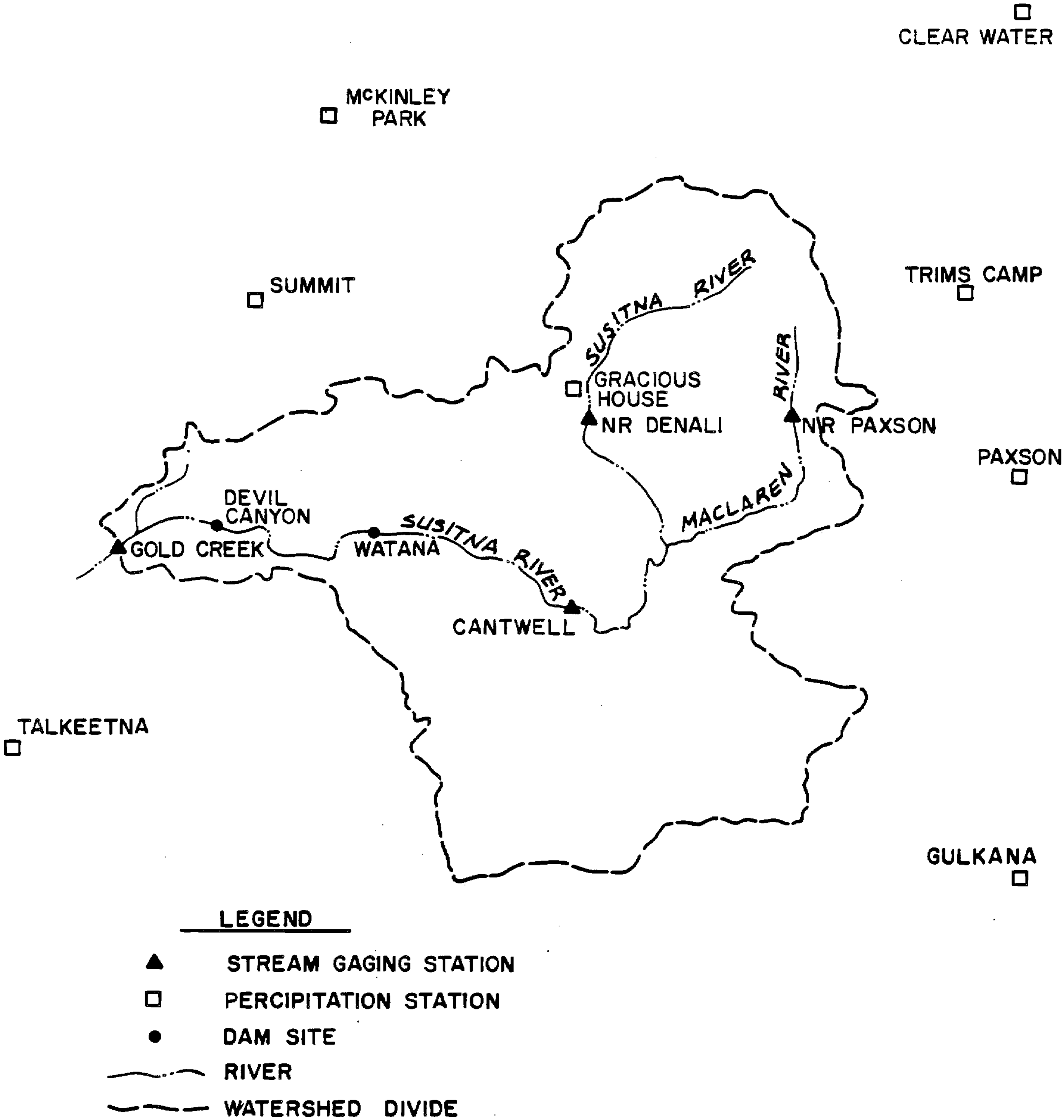
7 - Design Floods

The main spillway of Watana and Devil Canyon developments are designed to safely pass floods of 10,000-year return period. The estimated flood peaks for 10,000-year flood are 156,000 cfs and 165,000 cfs, respectively. Figures FC-34 and FC-35 also show the design flood hydrographs. In case of Devil Canyon development, the inflow hydrograph is composed of flood outflow from Watana and the natural flood flows from the intervening area between Watana and Devil Canyon. This is based on the assumption that Watana dam will be constructed first.

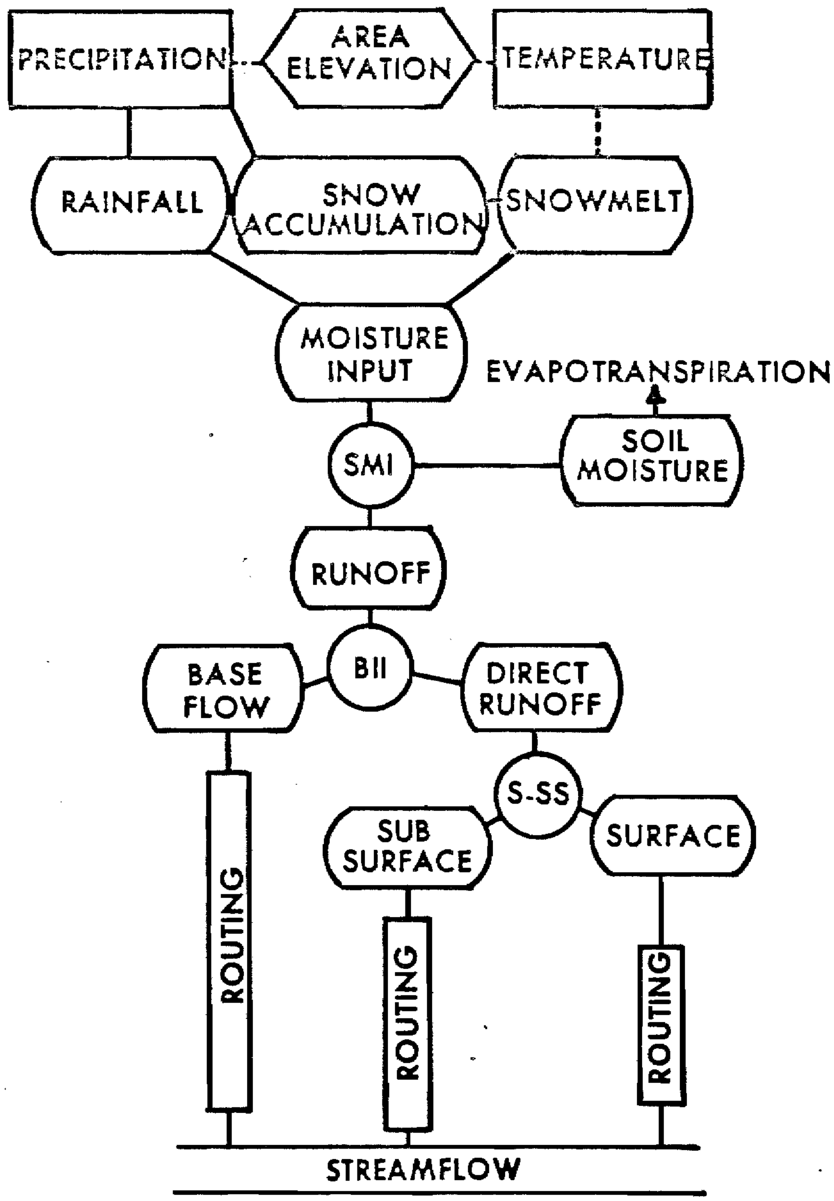
The flood hydrographs were derived using 10,000-year flood peak (annual series) and 1-, 3-, 7-, 15- and 30-day flood volumes. The flood peak and volumes frequency curves were developed for the Susitna River at Gold Creek and transposed to the dam sites. The procedures used to develop the frequency curves and transposition factors are discussed in a report by R & M Consultants, Inc.[3/].

[3/] R & M Consultants, Inc., December 1981. Regional Flood Study Task 3, Hydrology, prepared for Acres American Incorporated Anchorage, Alaska.

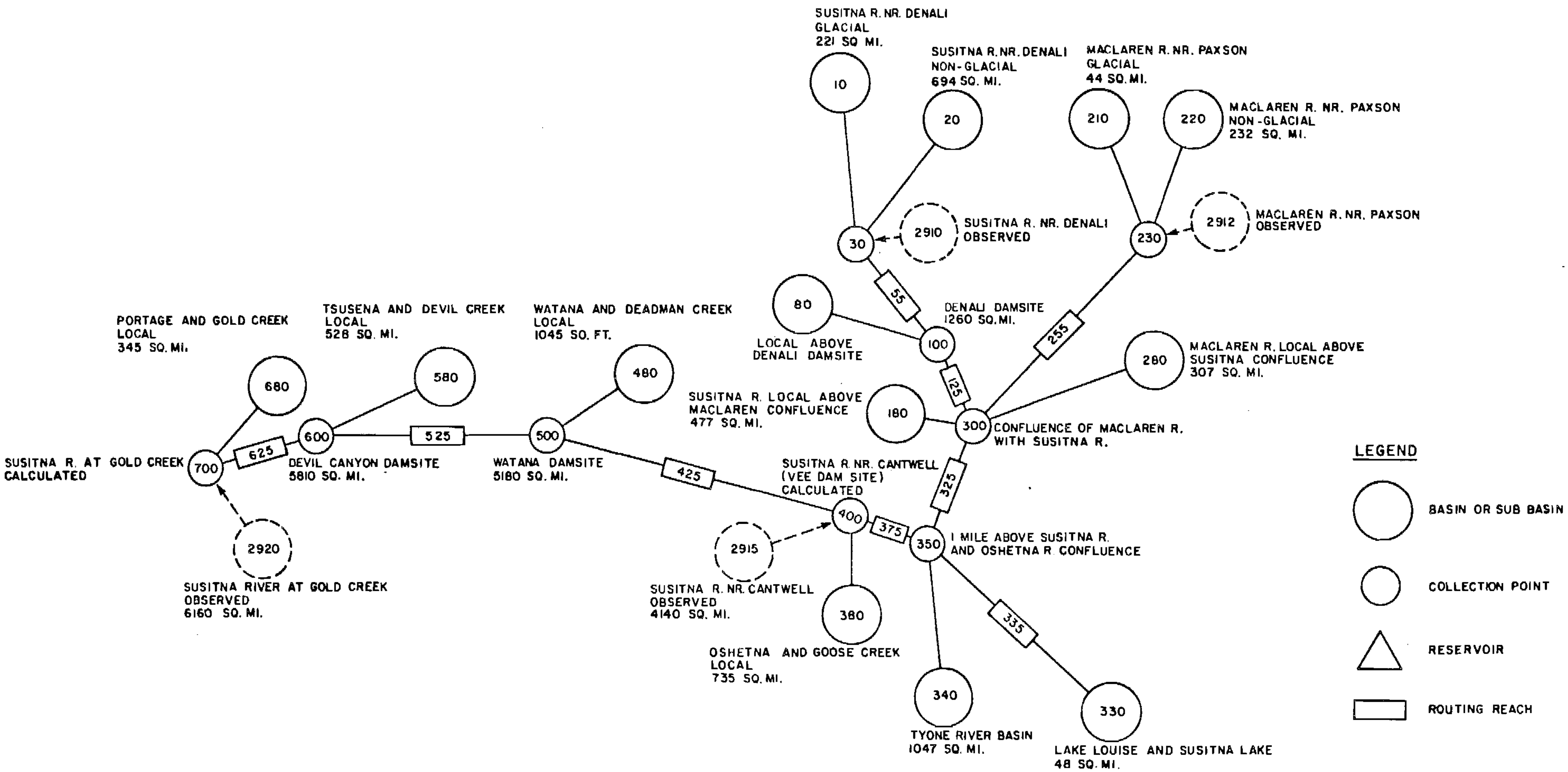
FIGURE FC-1



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 FC-3

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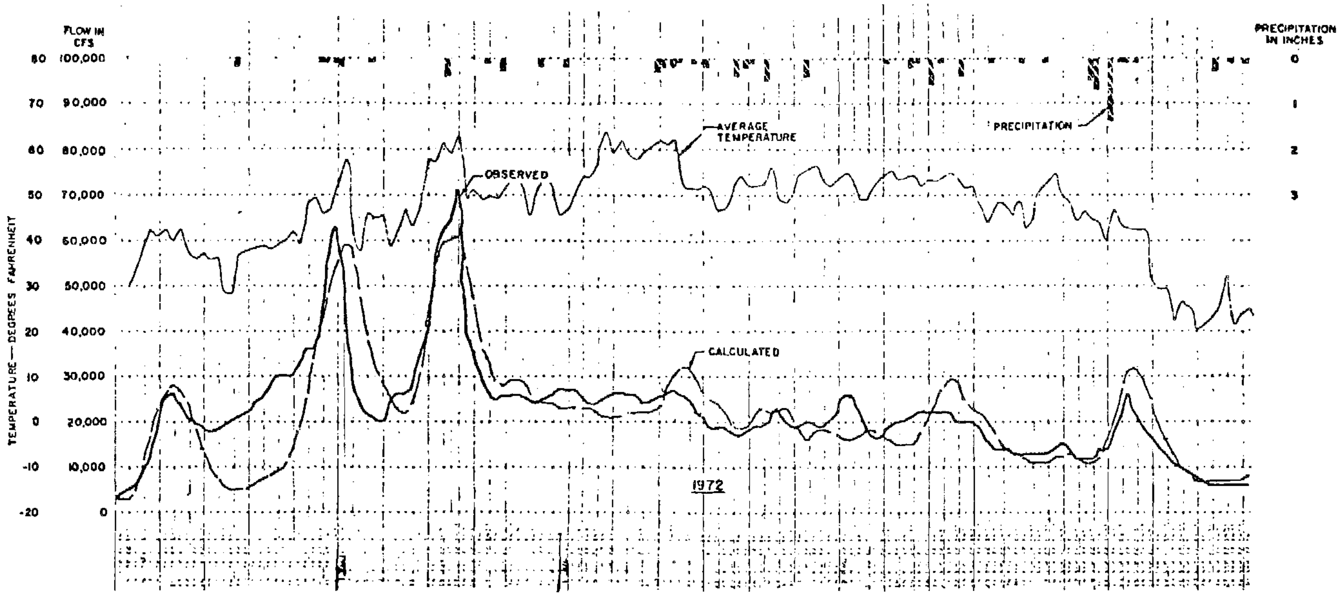

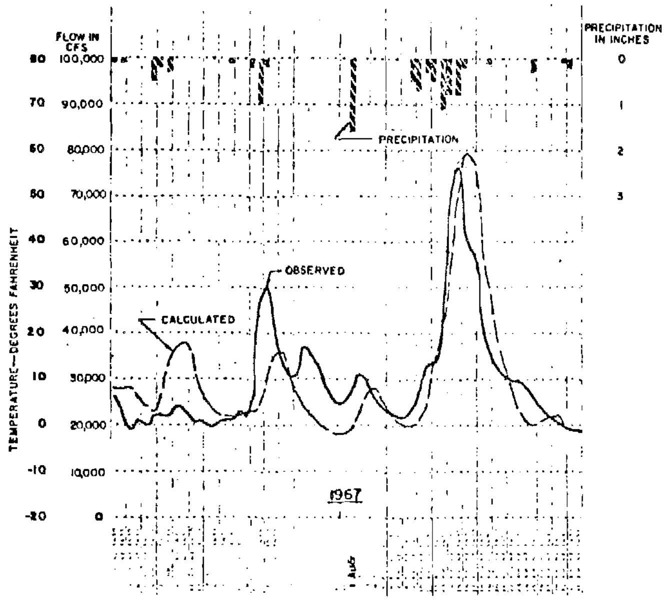

REFERENCE:
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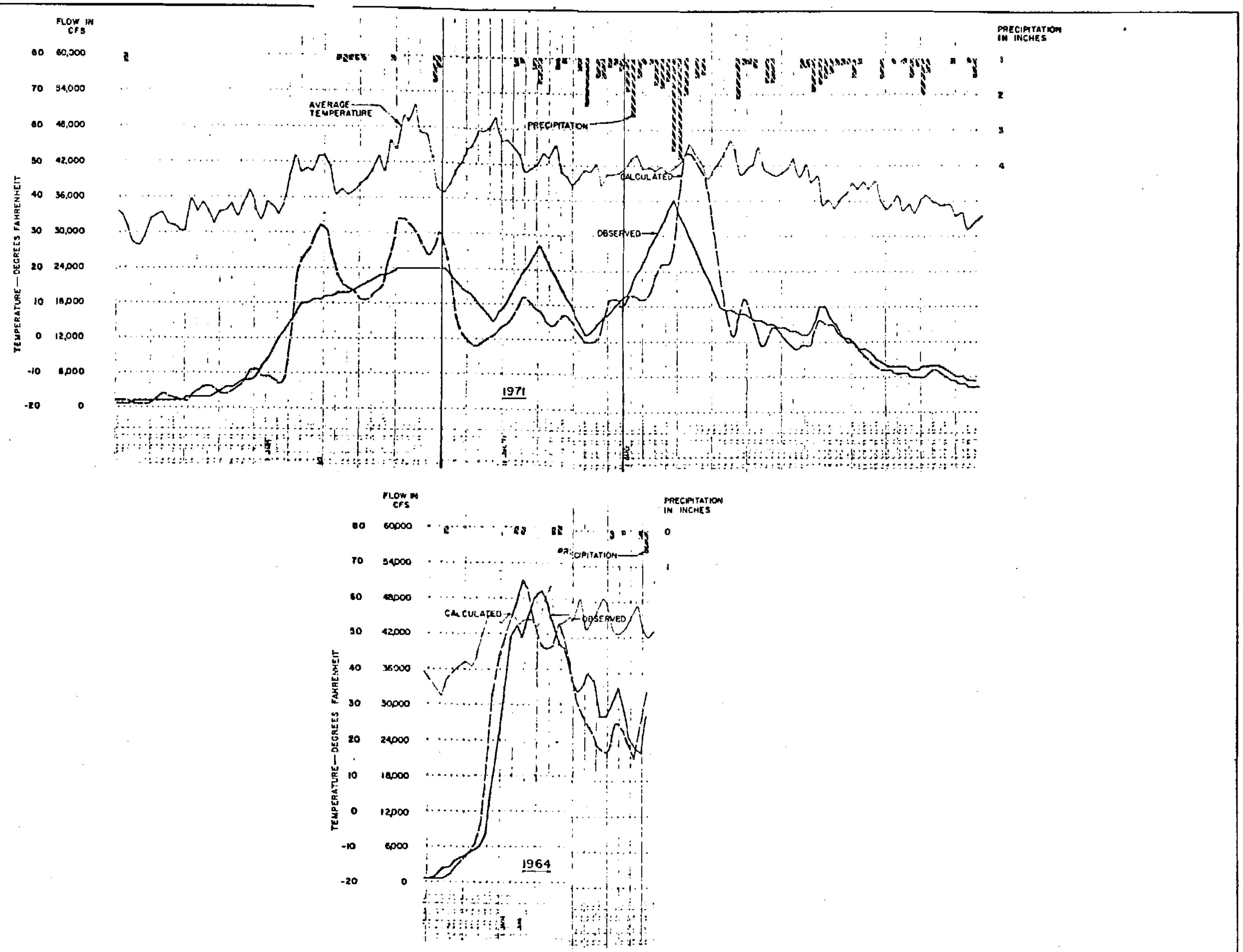
HYDROGRAPH : SUSITNA RIVER AT GOLD CREEK, 1967, 1972

FIGURE FC-4

ACRES

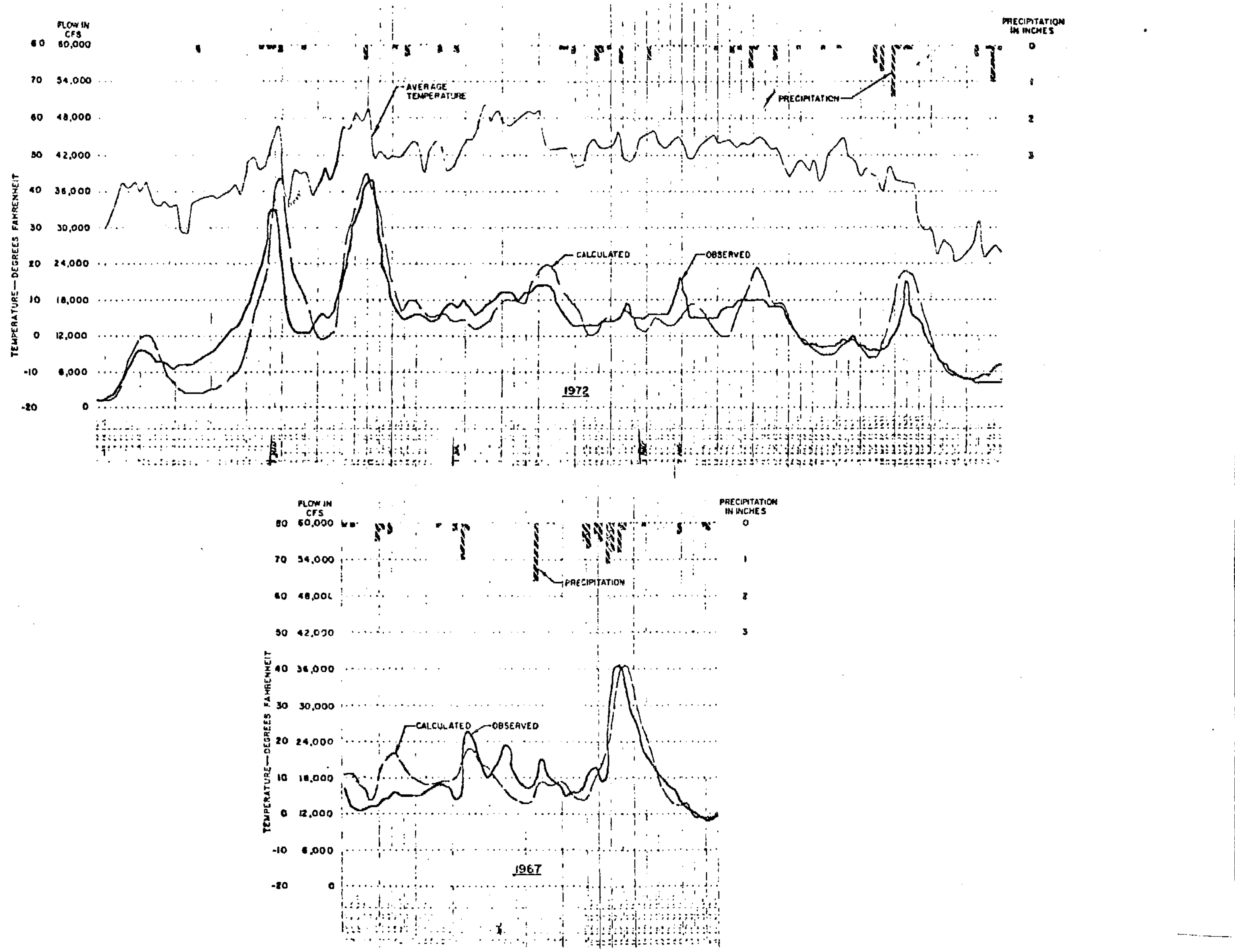


REFERENCE.
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH : SUSITNA RIVER NEAR CANTWELL, 1964, 1971

FIGURE FC-5

ACRES



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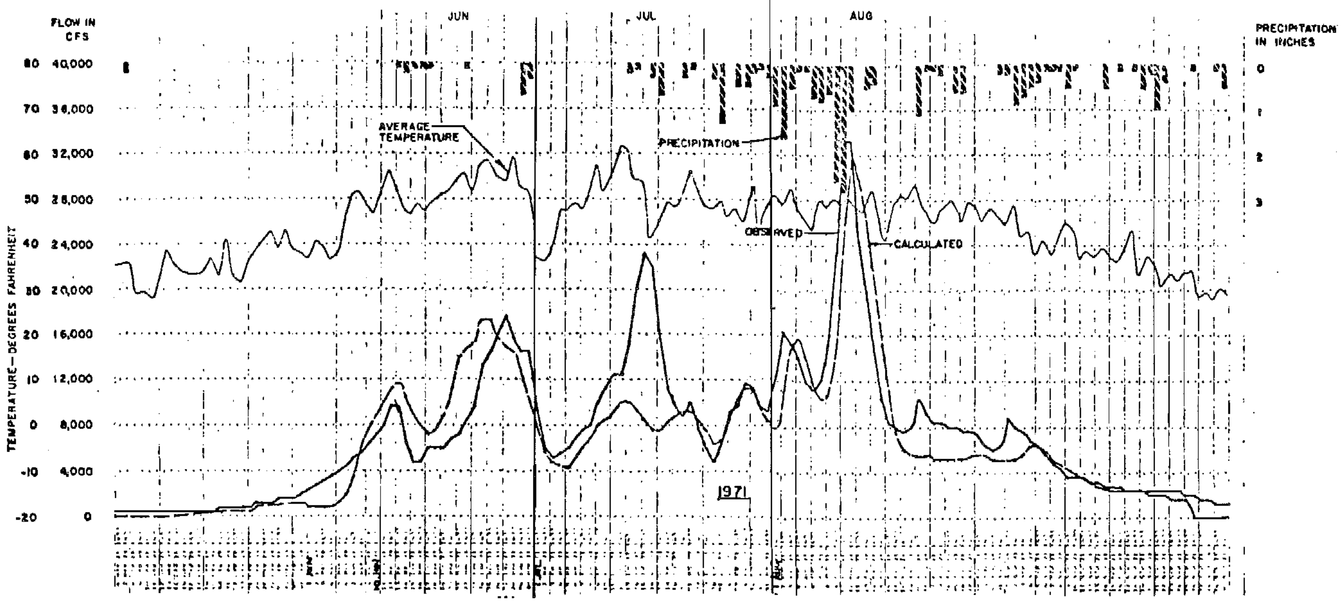
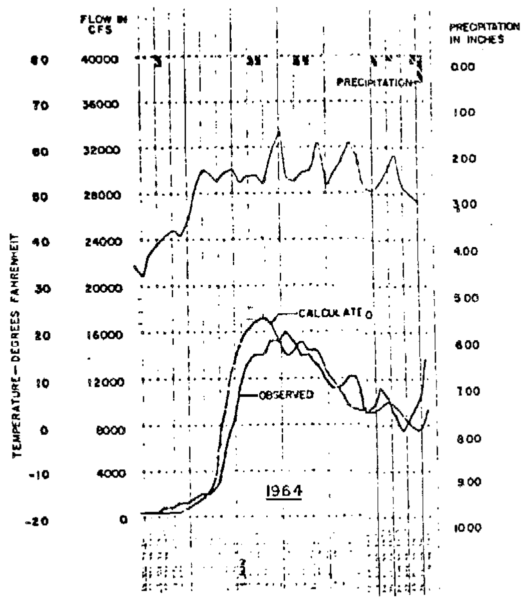
HYDROGRAPH : SUSITNA RIVER NEAR CANTWELL, 1967, 1972

FIGURE FC-6

ACRES



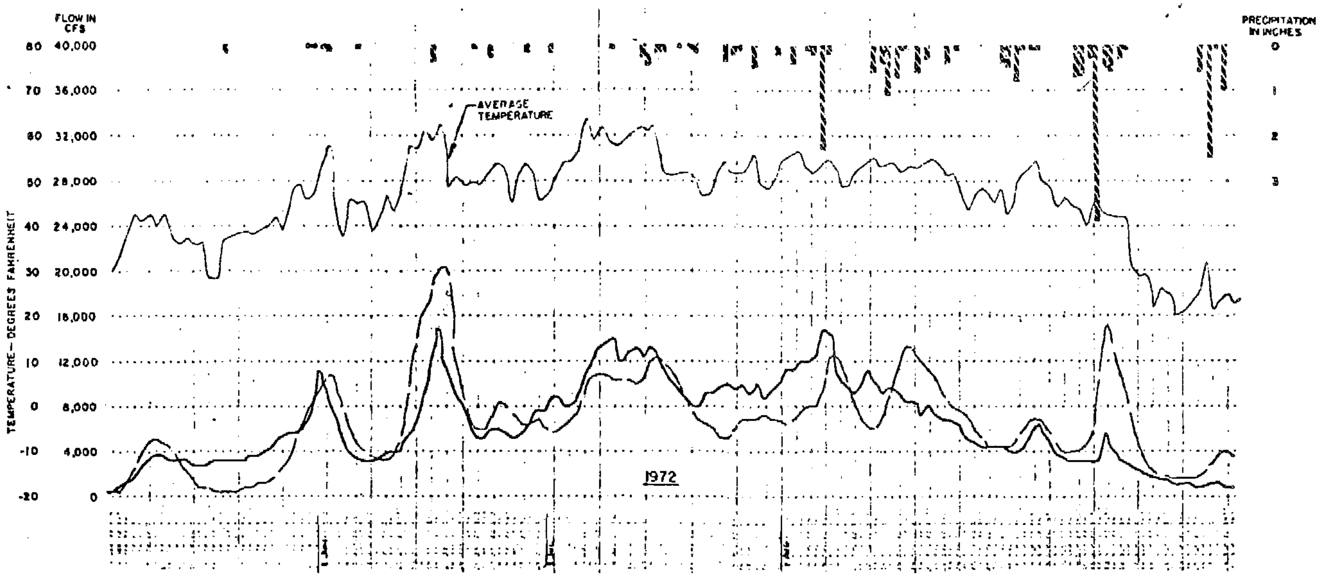


REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH SUSITNA RIVER NEAR DENALI, 1964, 1971

FIGURE FC-7

ACRES

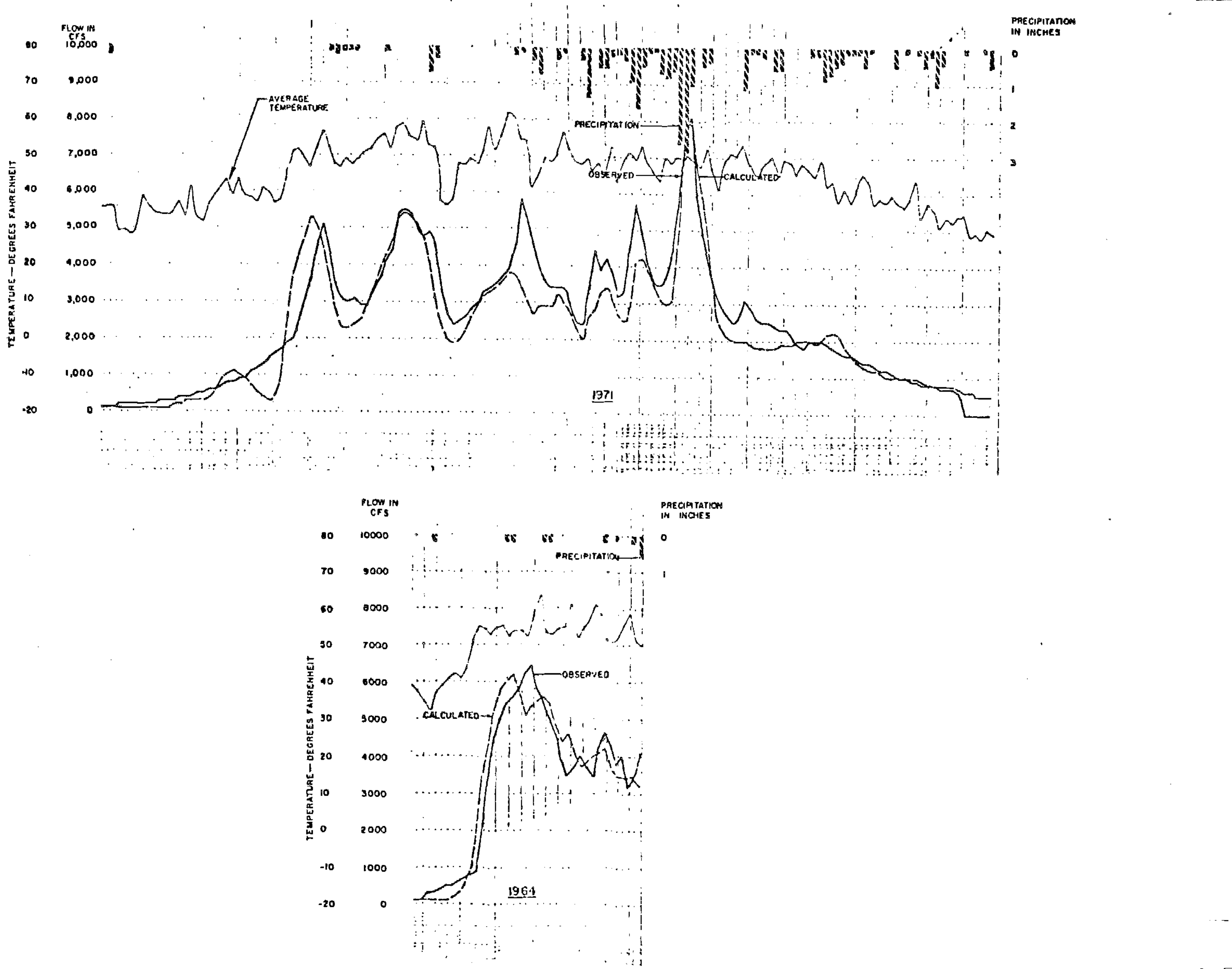


REFERENCE.
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH SUSITNA RIVER NEAR DENALI, 1972

FIGURE FC - 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 FC - 9

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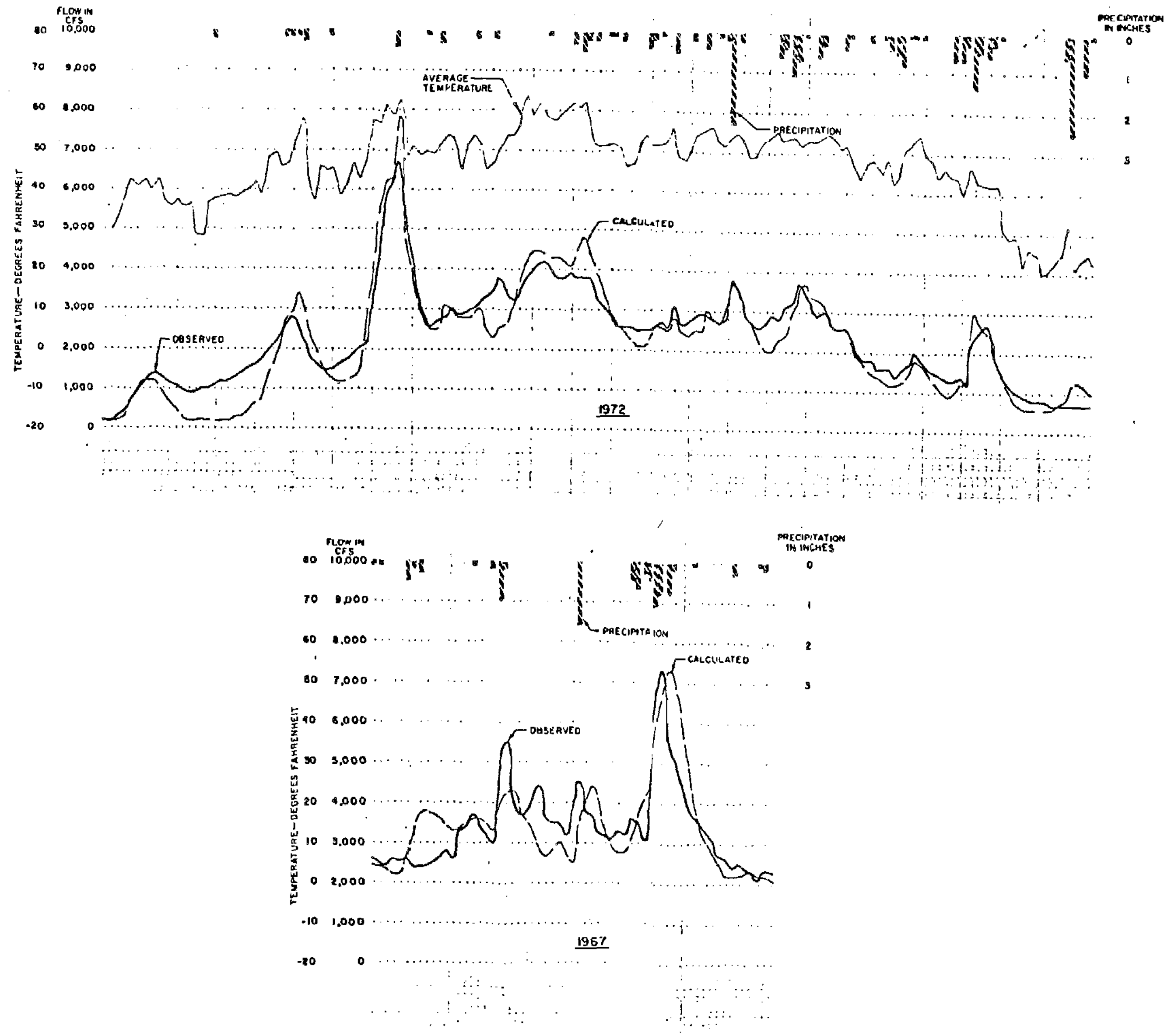

REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH MACLAREN RIVER NEAR PAXSON, 1967, 1972

FIGURE FC-10

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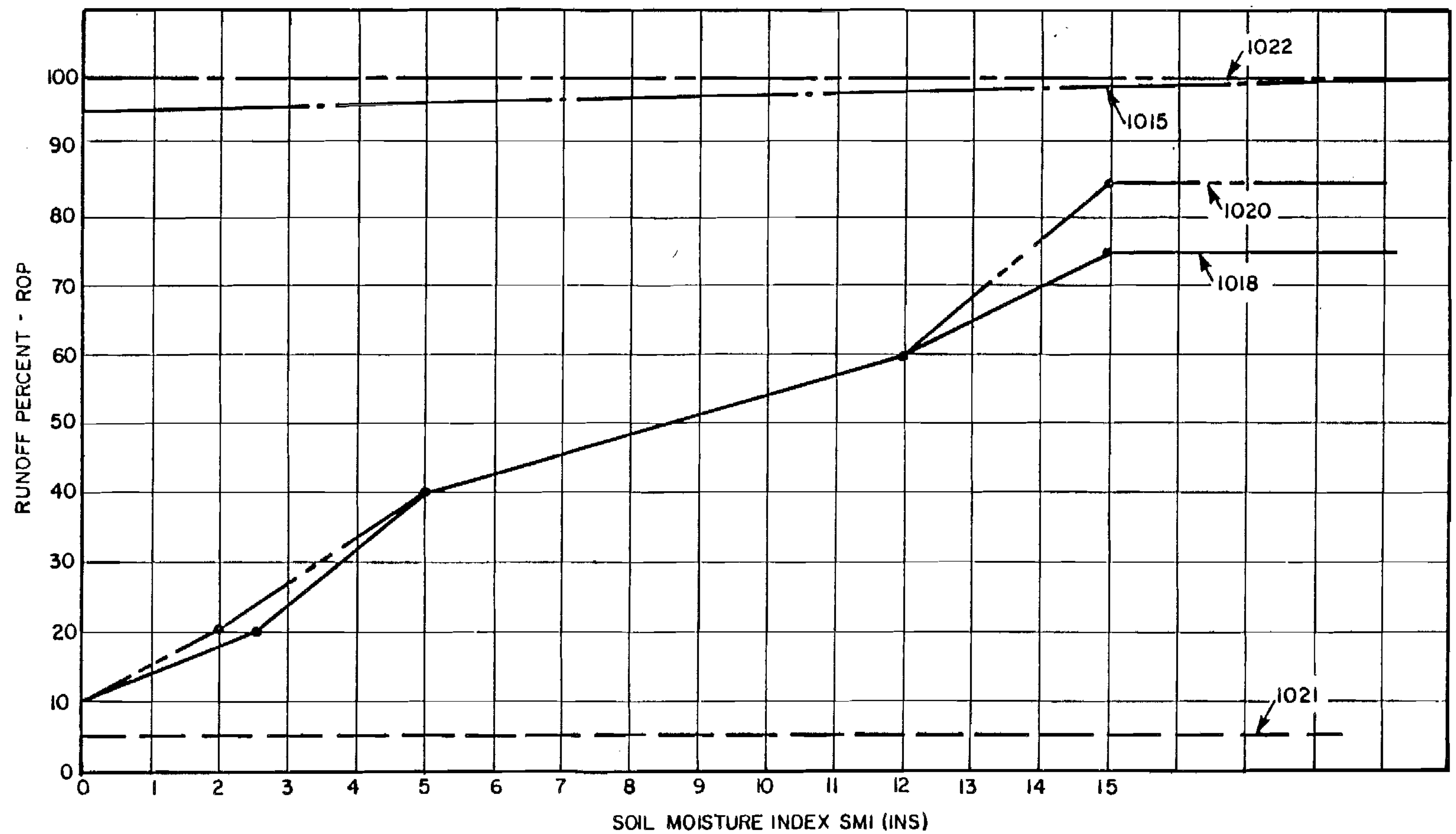

SSARR MODEL SMI VS ROP

FIGURE FC - 11

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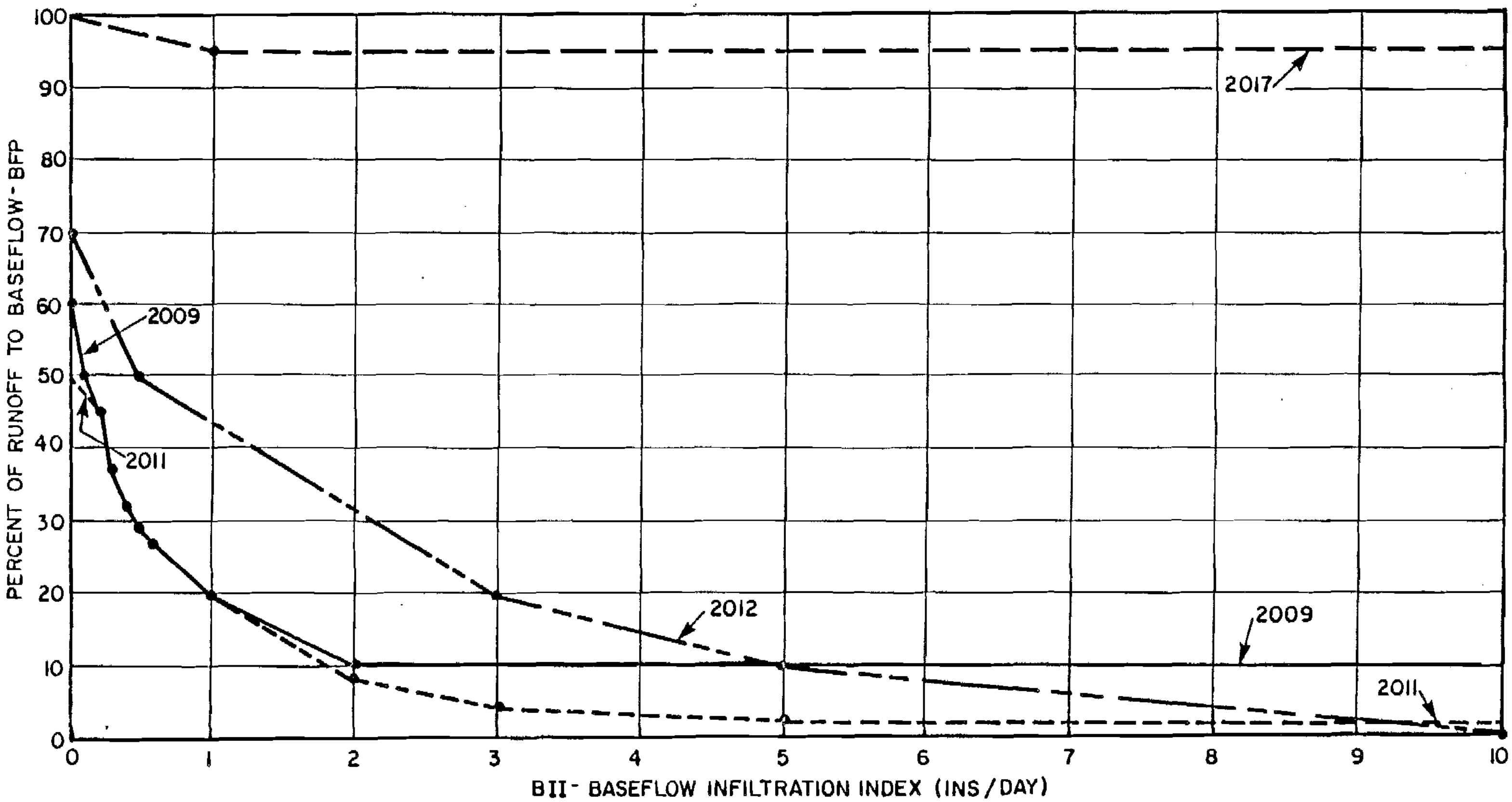

SSARR MODEL BII VS BFP

FIGURE FC-12

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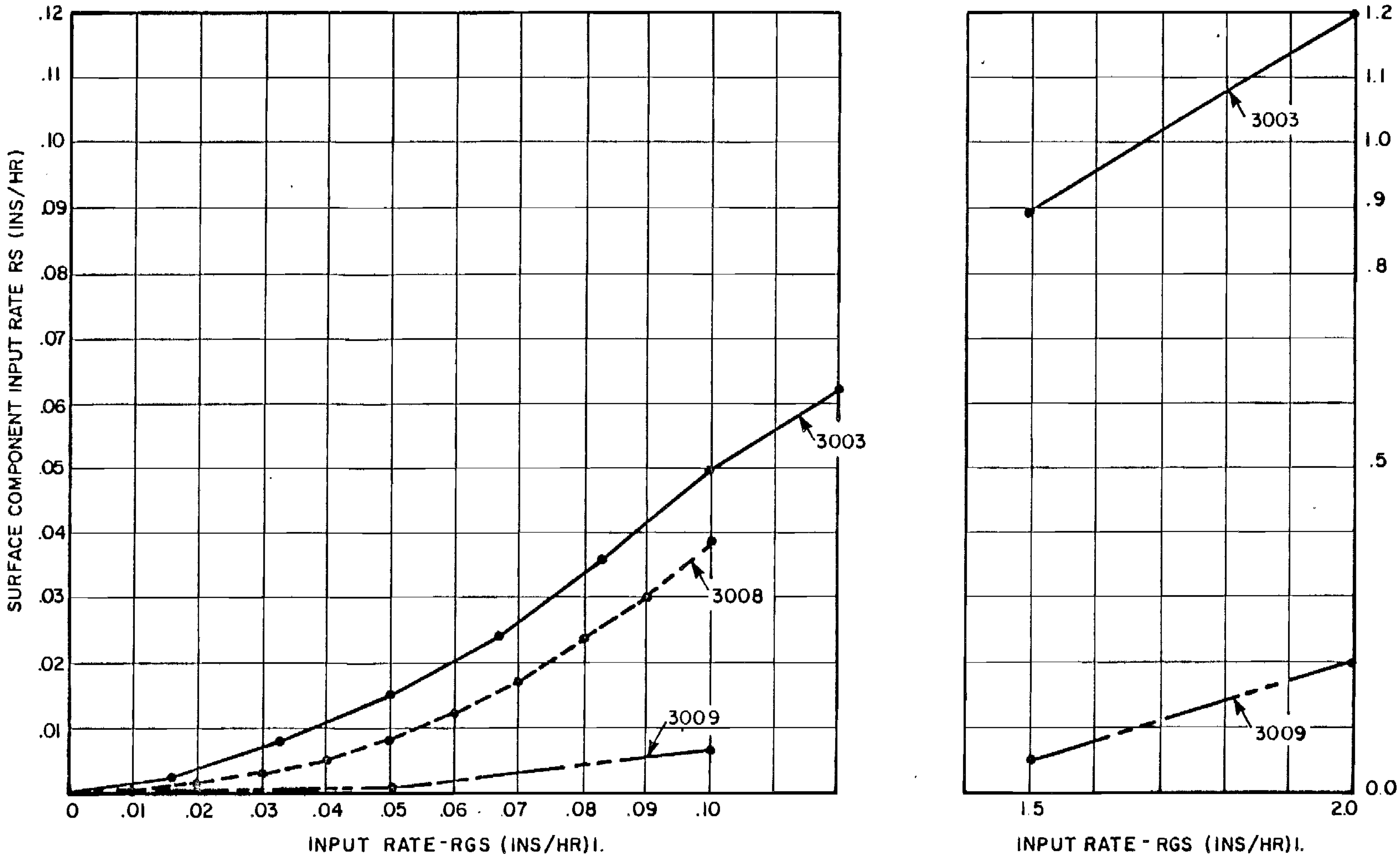

SSARR MODEL RGS VS RS

FIGURE FC – 13

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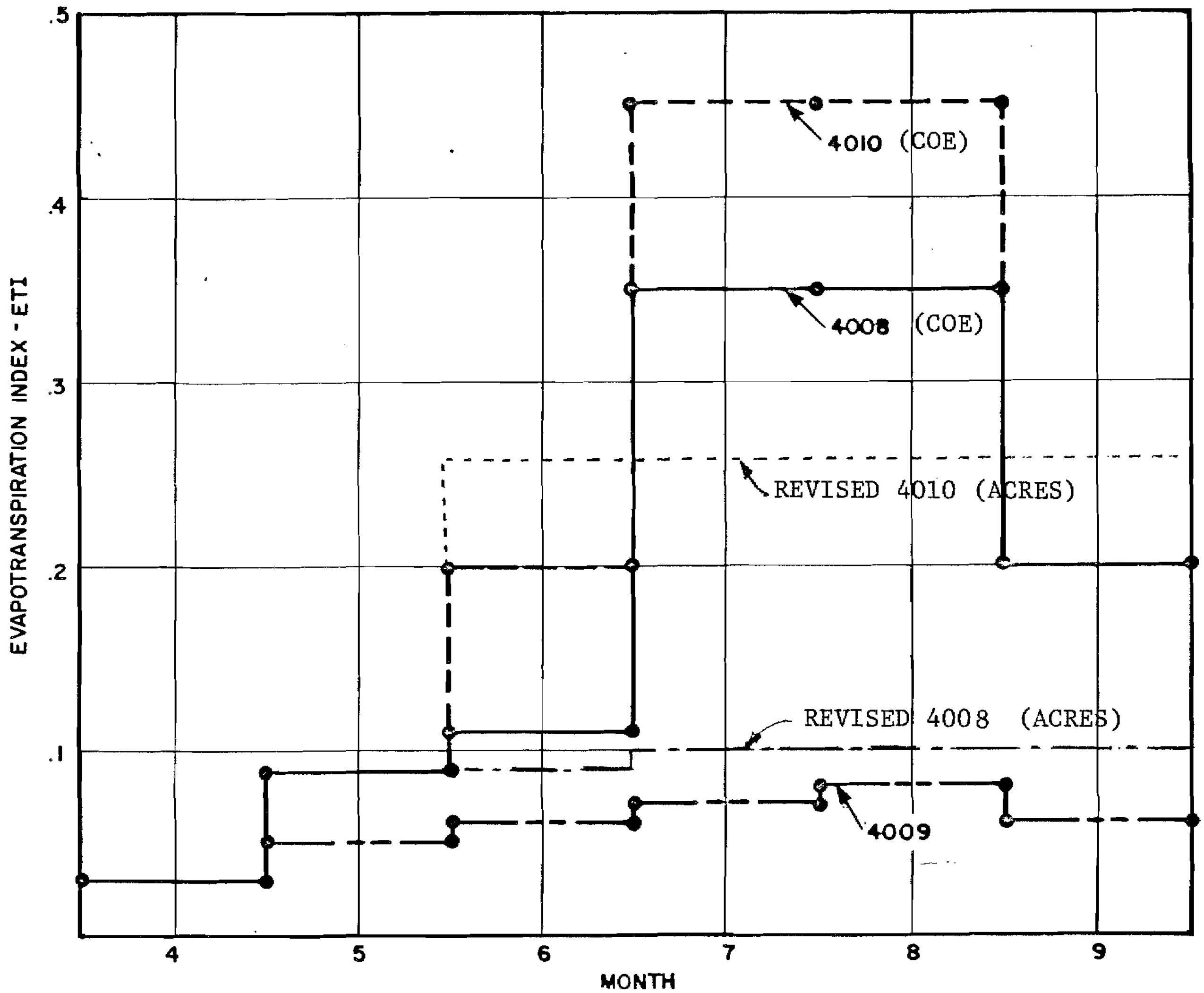


SSARR MODEL MONTH VS ETI FIGURE FC - 14

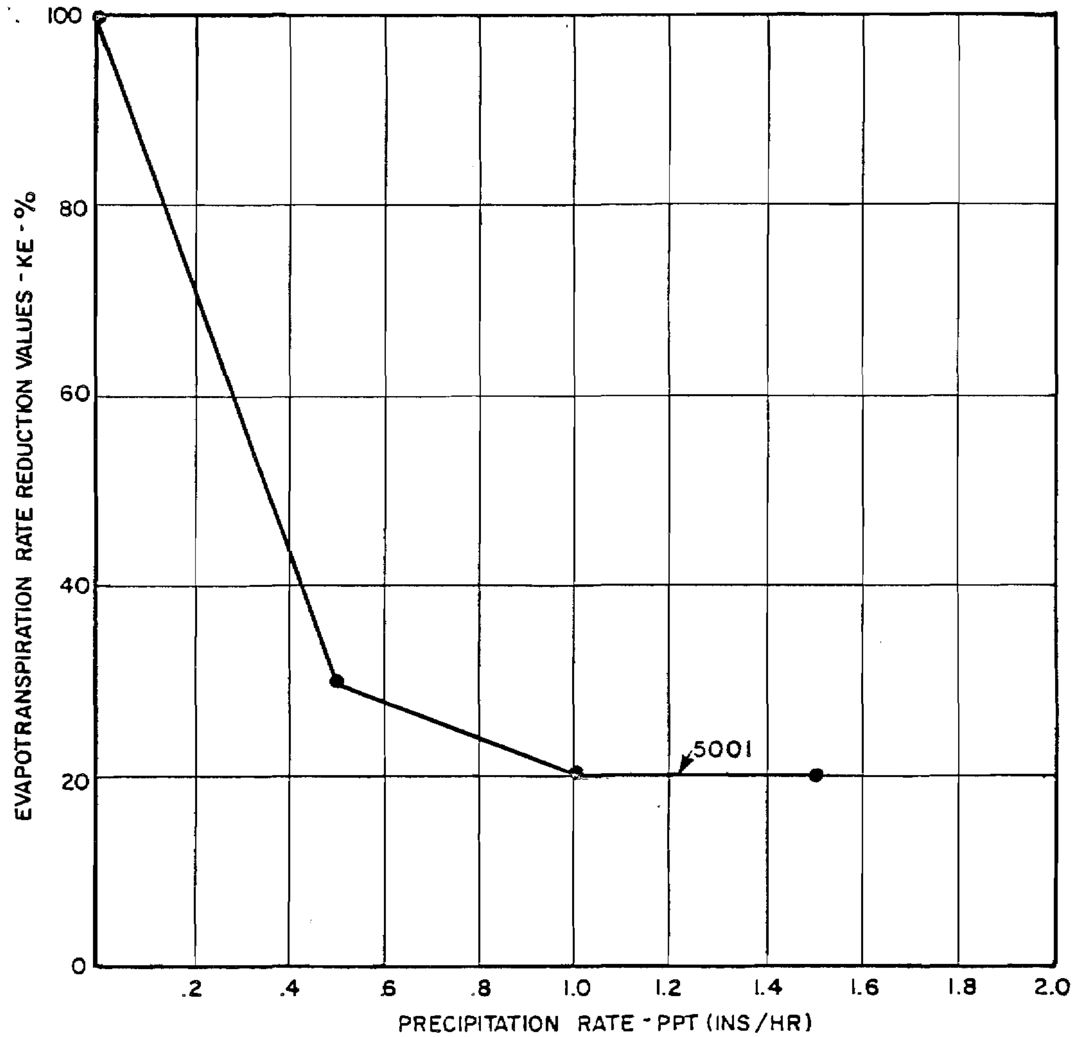
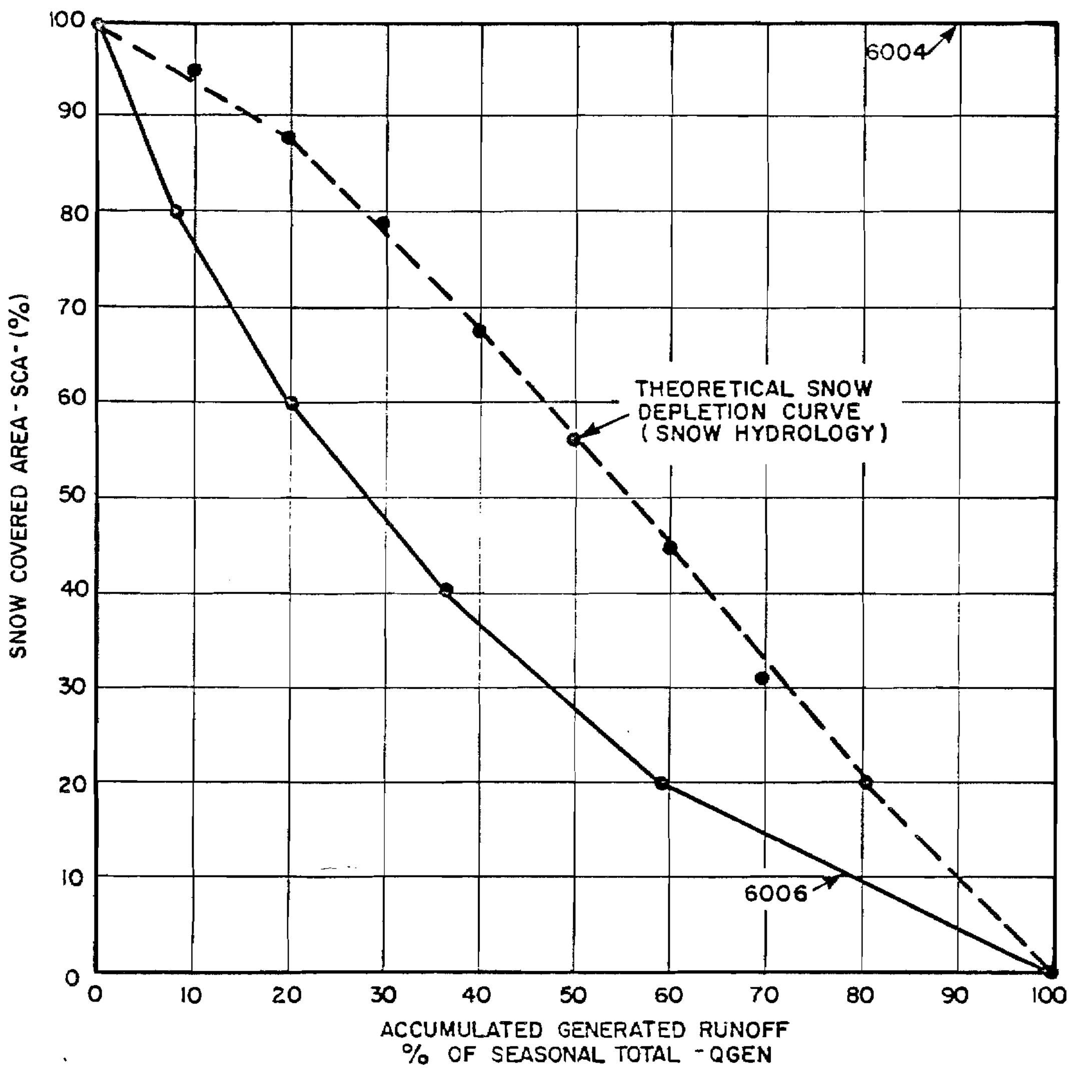


FIGURE FC - 15

SSARR MODEL PPT VS KE



SSARR MODEL QGEN VS SCA

FIGURE FC-16

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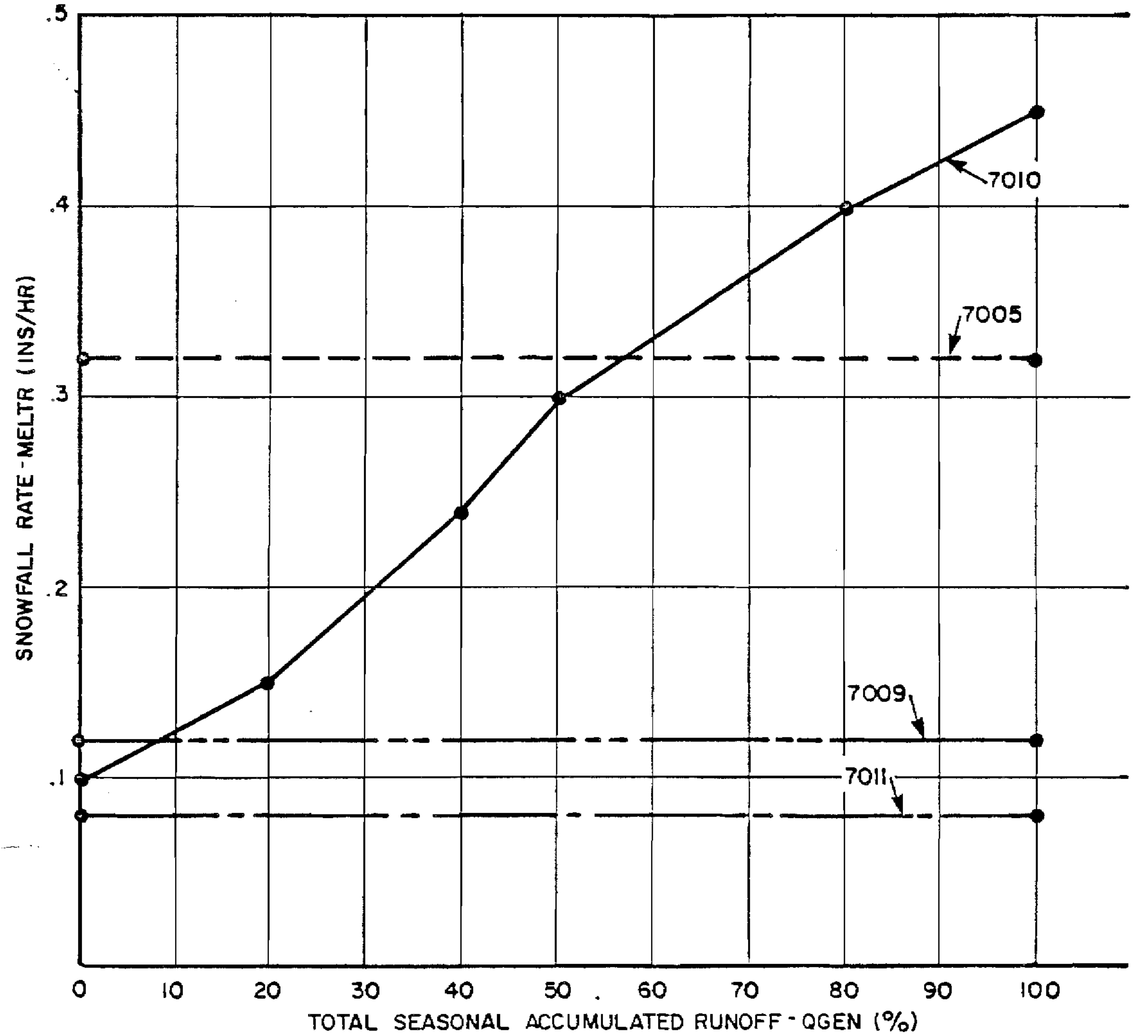

FIGURE FC - 17

SSARR MODEL QGEN VS MELTR

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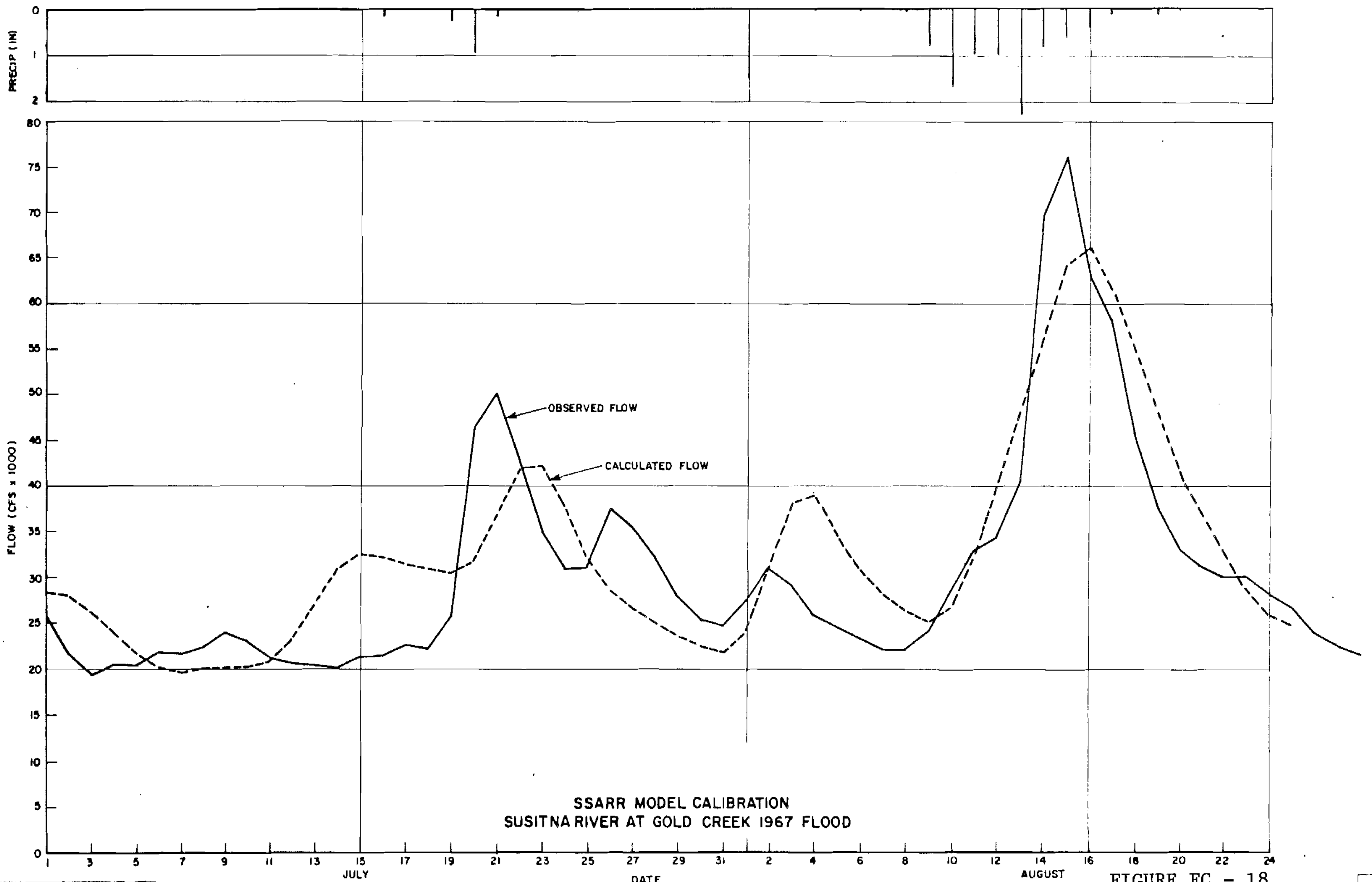

SSARR MODEL CALIBRATION
SUSITNA RIVER AT GOLD CREEK 1967 FLOOD

FIGURE FC - 18

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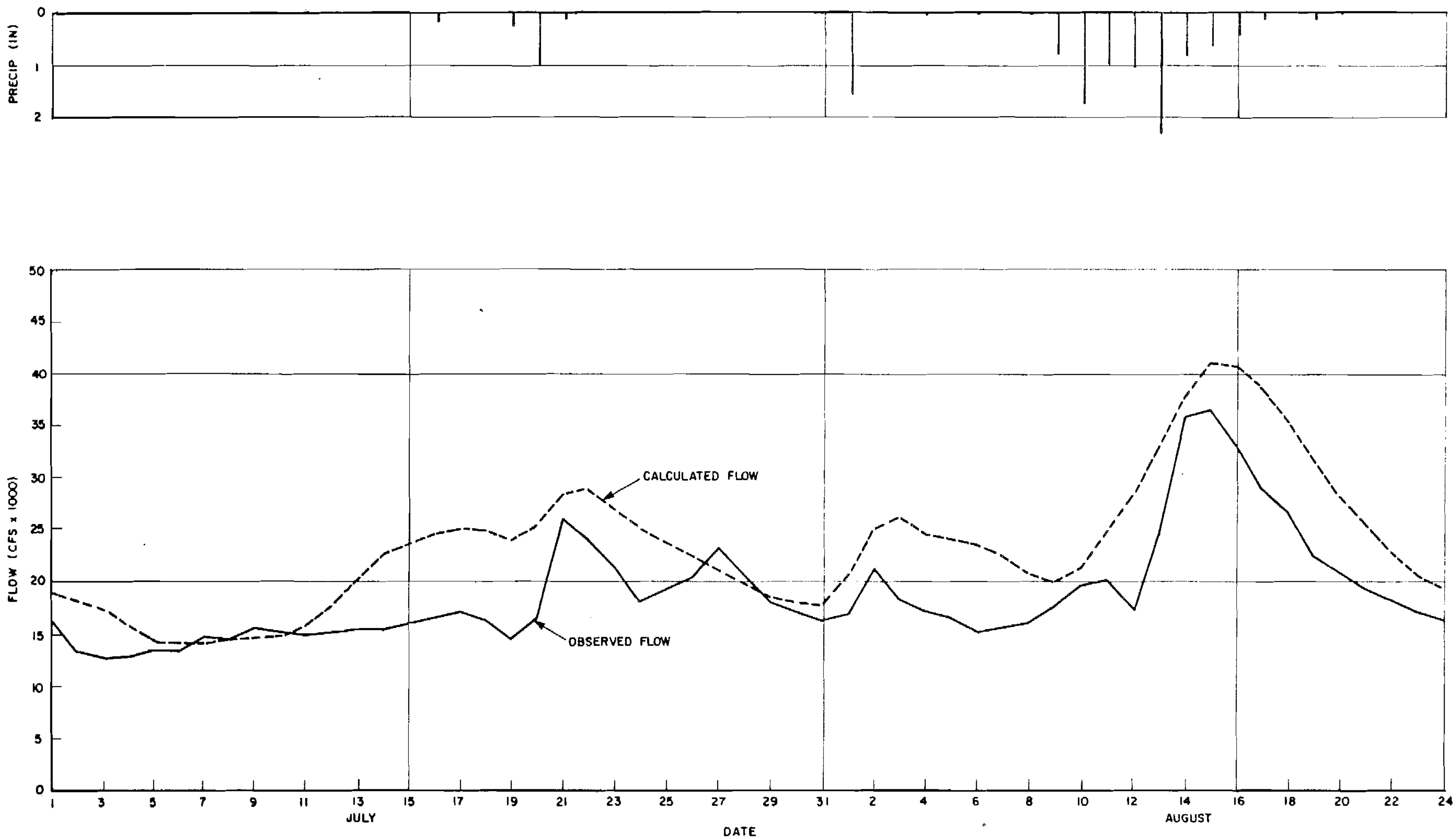

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SSARR MODEL CALIBRATION
SUSITNA RIVER NEAR CANTWELL 1967 FLOOD

FIGURE FC - 19

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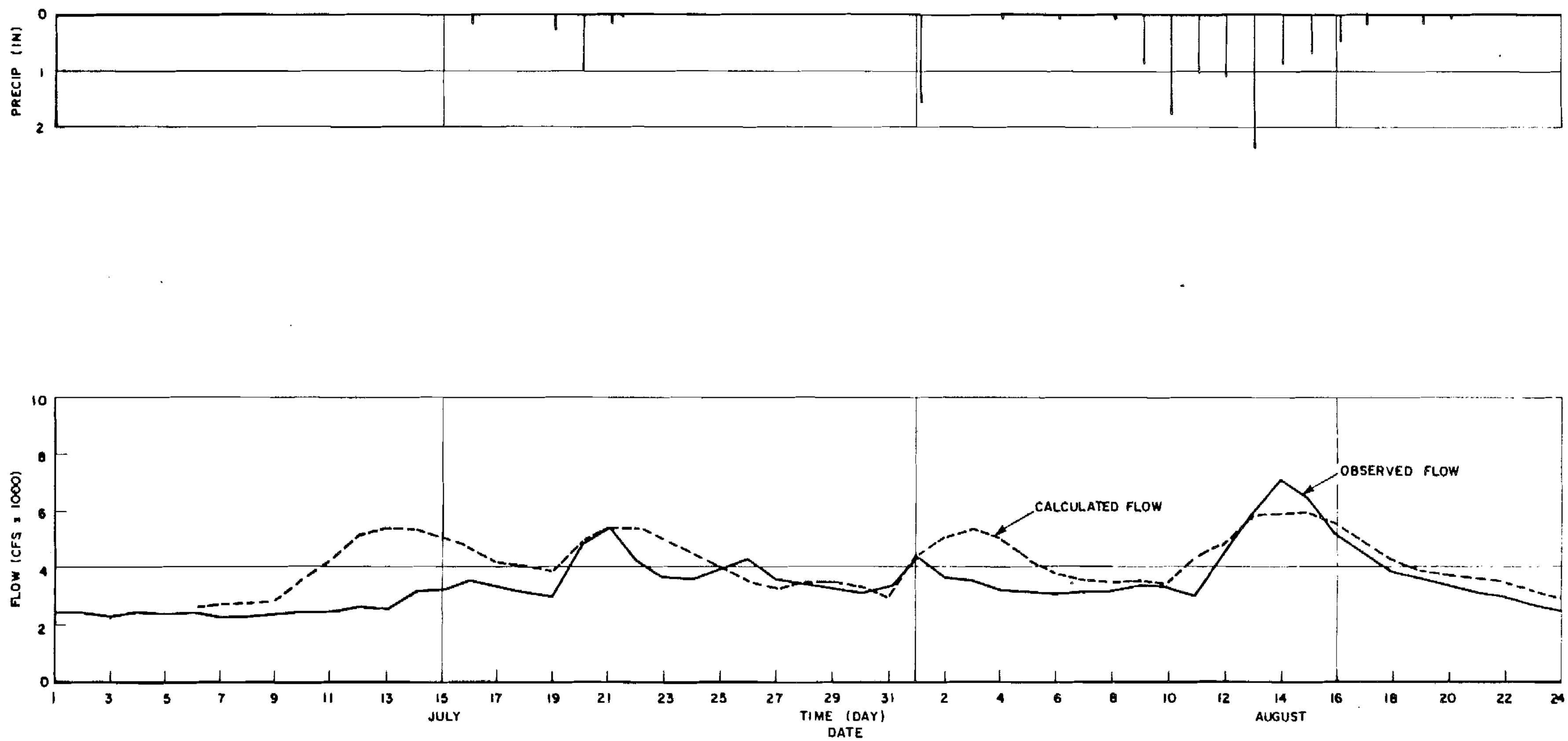

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SSARR MODEL CALIBRATION
MACLAREN RIVER NEAR PAXSON 1967 FLOOD

FIGURE FC - 20

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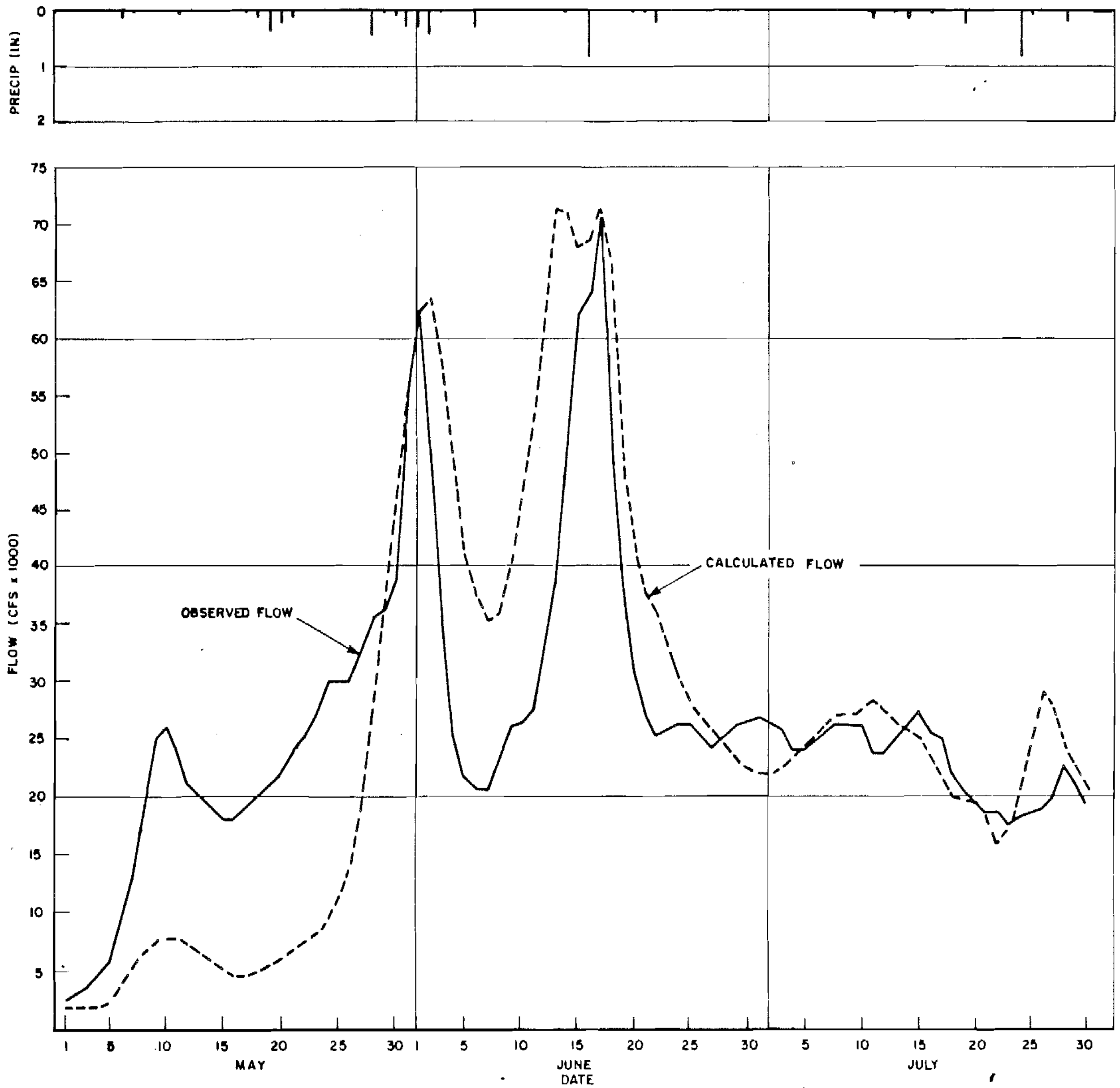

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FIGURE FC - 21

SSARR MODEL CALIBRATION
SUSITNA RIVER AT GOLD CREEK 1972 FLOOD

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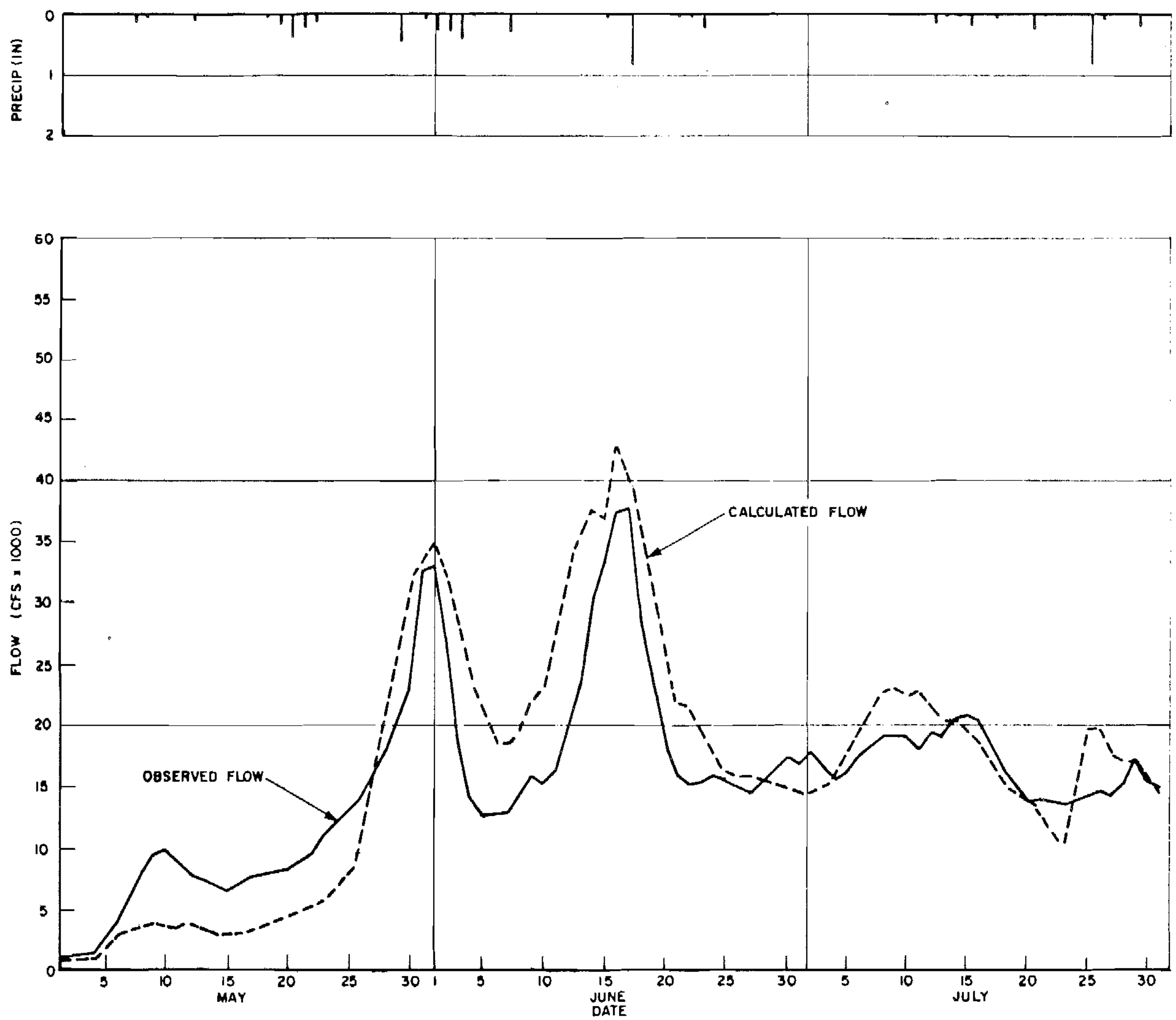

FIGURE FC - 22

SSARR MODEL CALIBRATION
SUSITNA RIVER NEAR CANTWELL 1972 FLOOD

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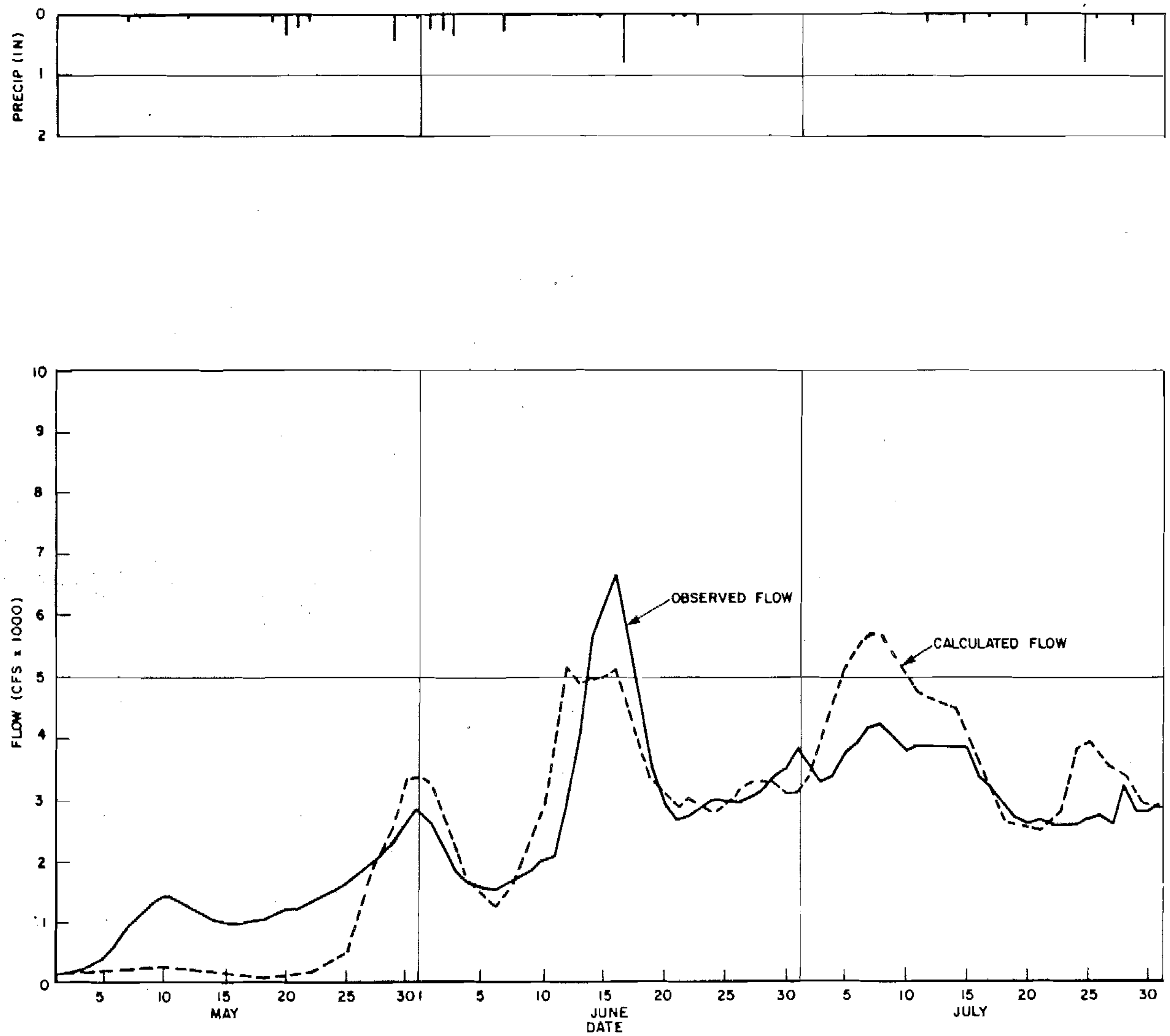

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SSARR MODEL CALIBRATION
MACLAREN RIVER NEAR PAXSON 1972 FLOOD

FIGURE FC - 23

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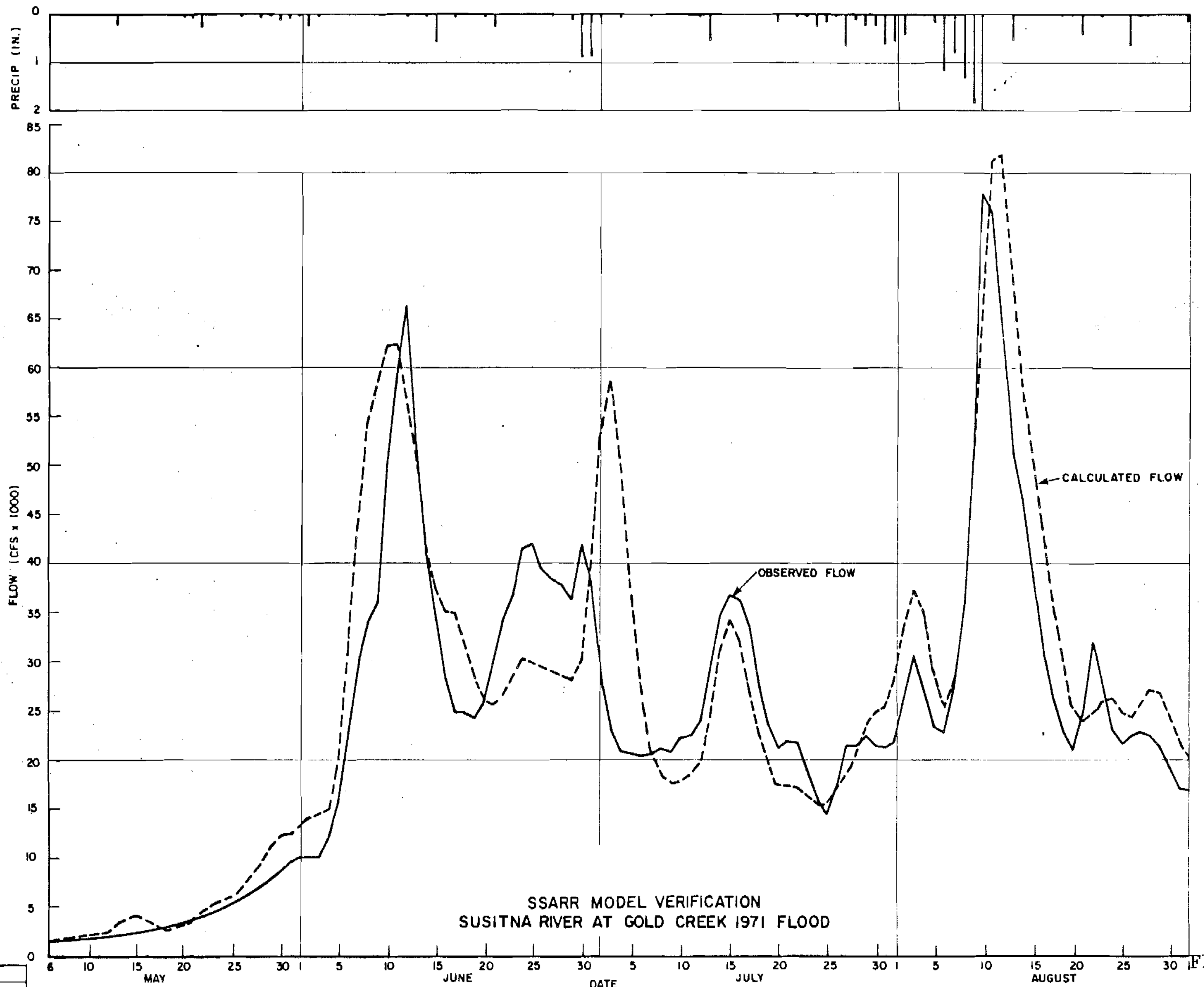

FIGURE FC-24

DSN		
DRW		
CHK		

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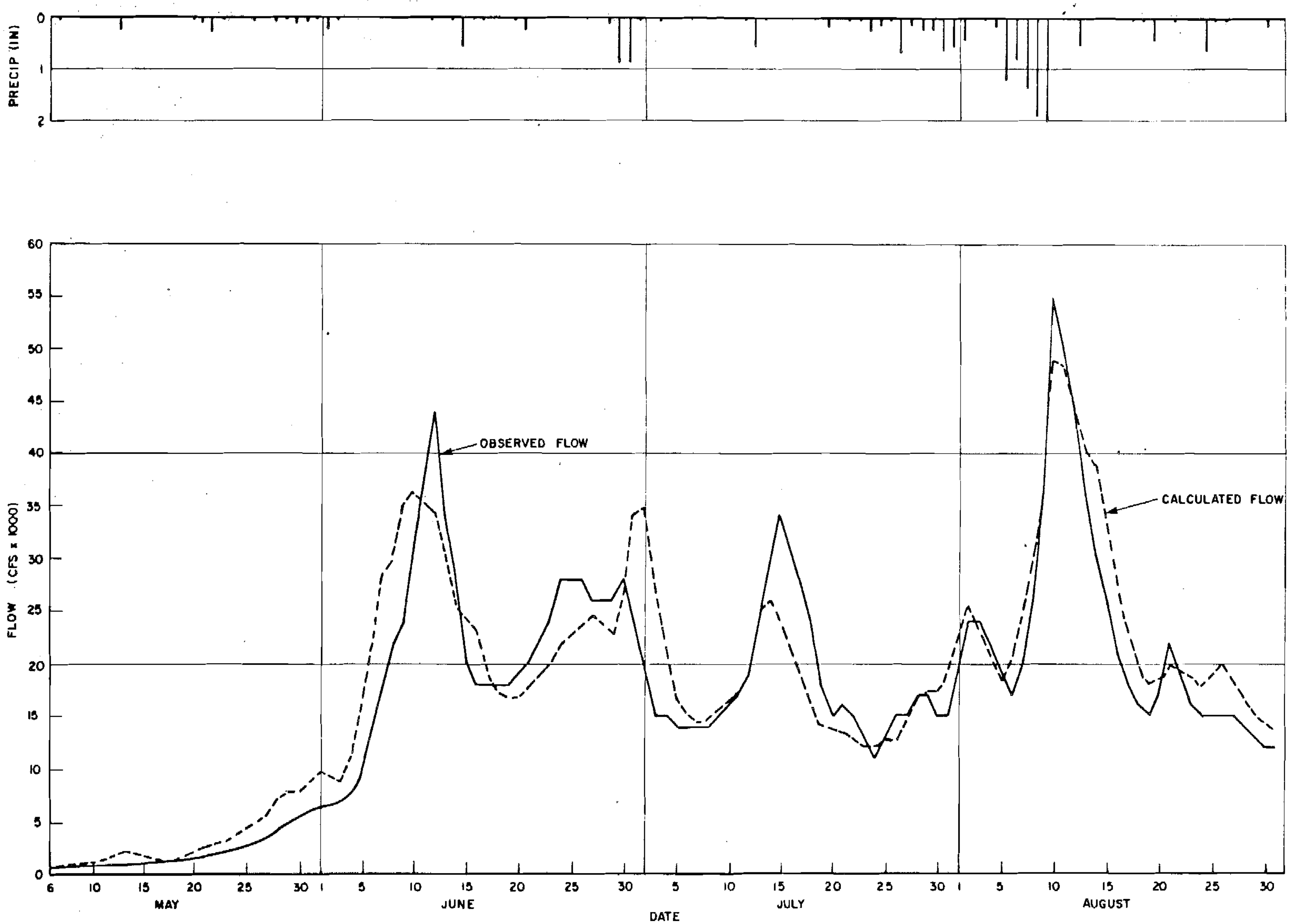

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SSARR MODEL VERIFICATION
SUSITNA RIVER NEAR CANTWELL 1971 FLOOD

FIGURE FC-25

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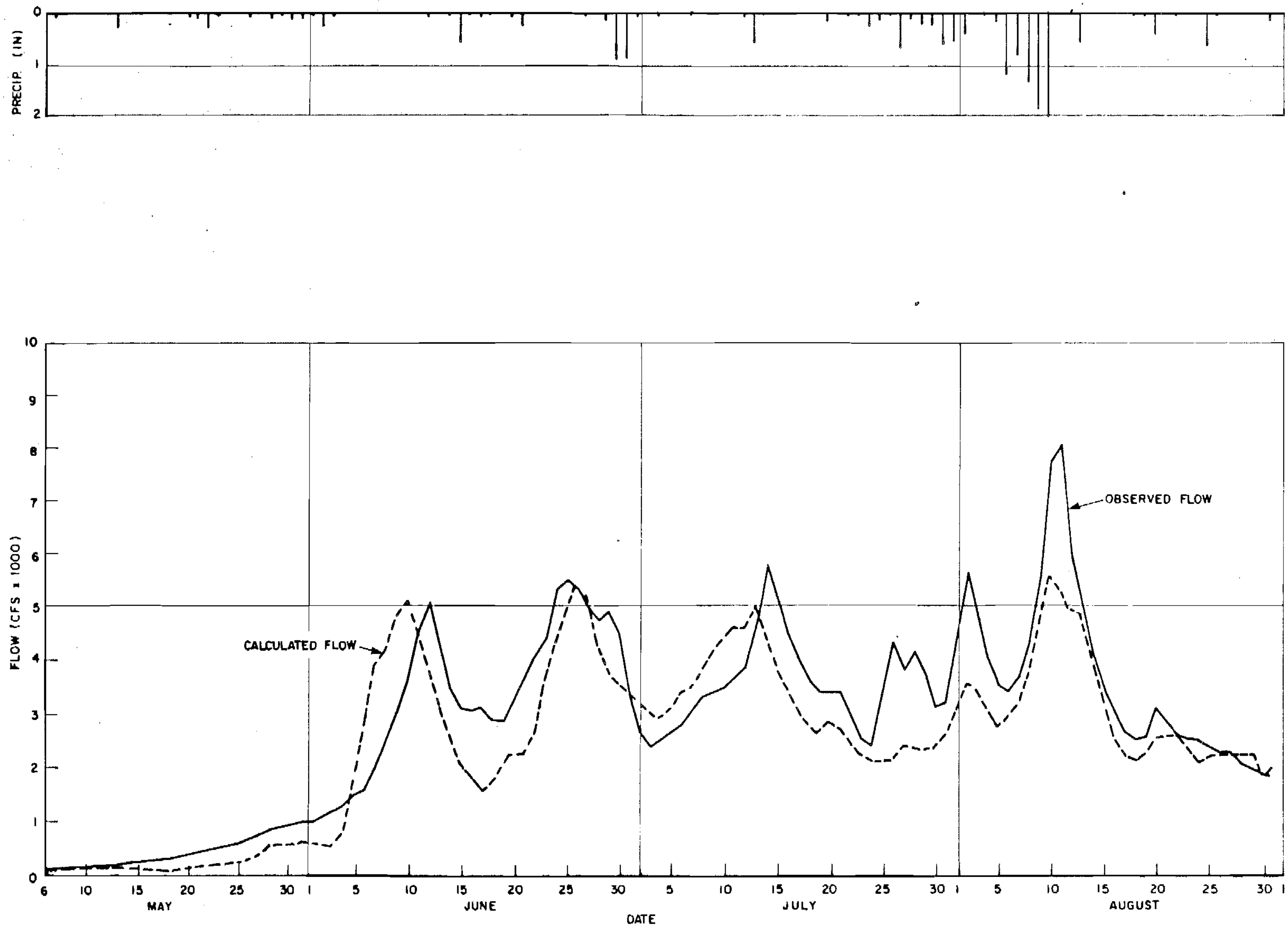

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FIGURE FC - 26

SSARR MODEL VERIFICATION
MACLAREN RIVER NEAR PAXSON 1971 FLOOD

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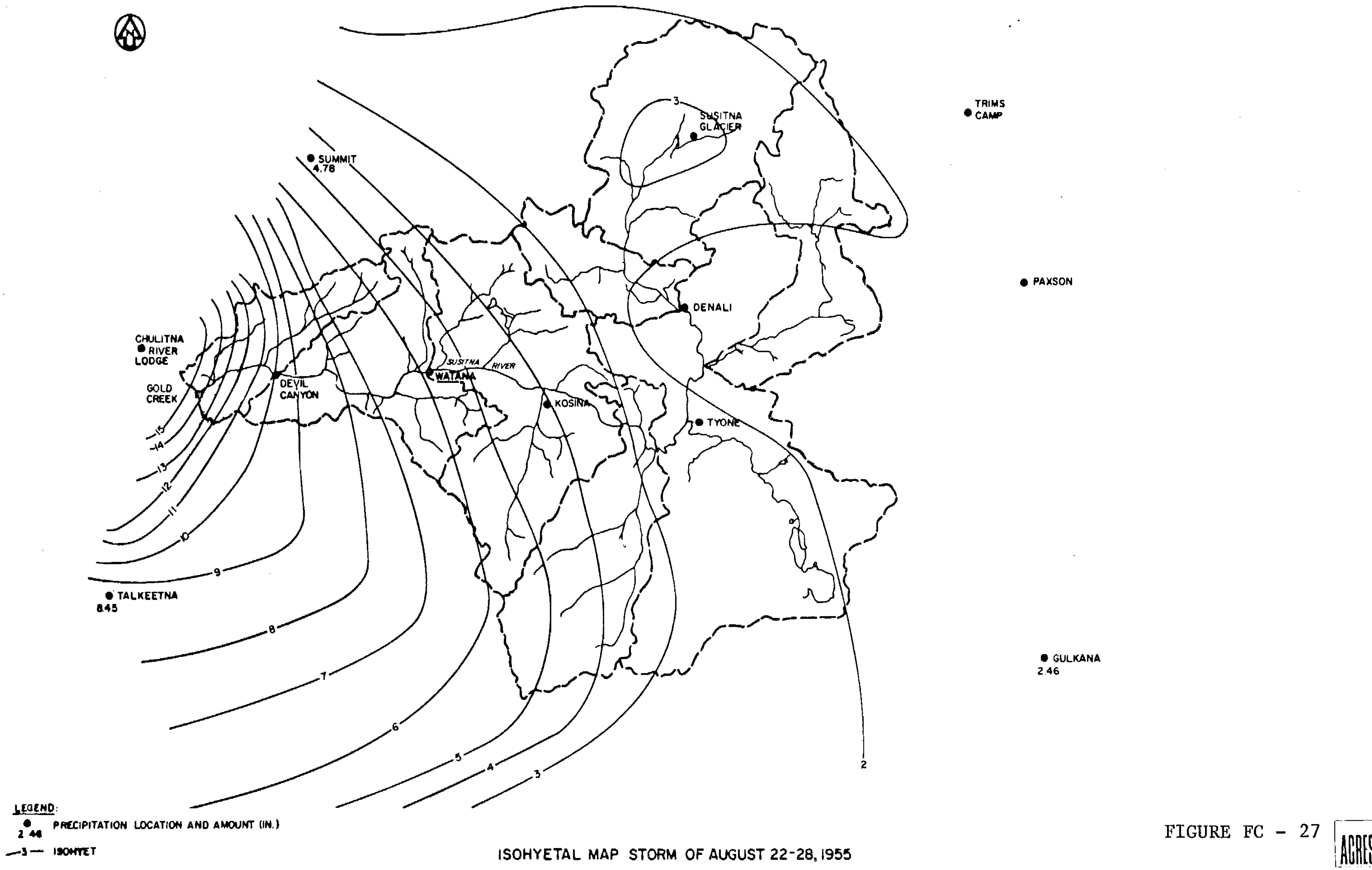
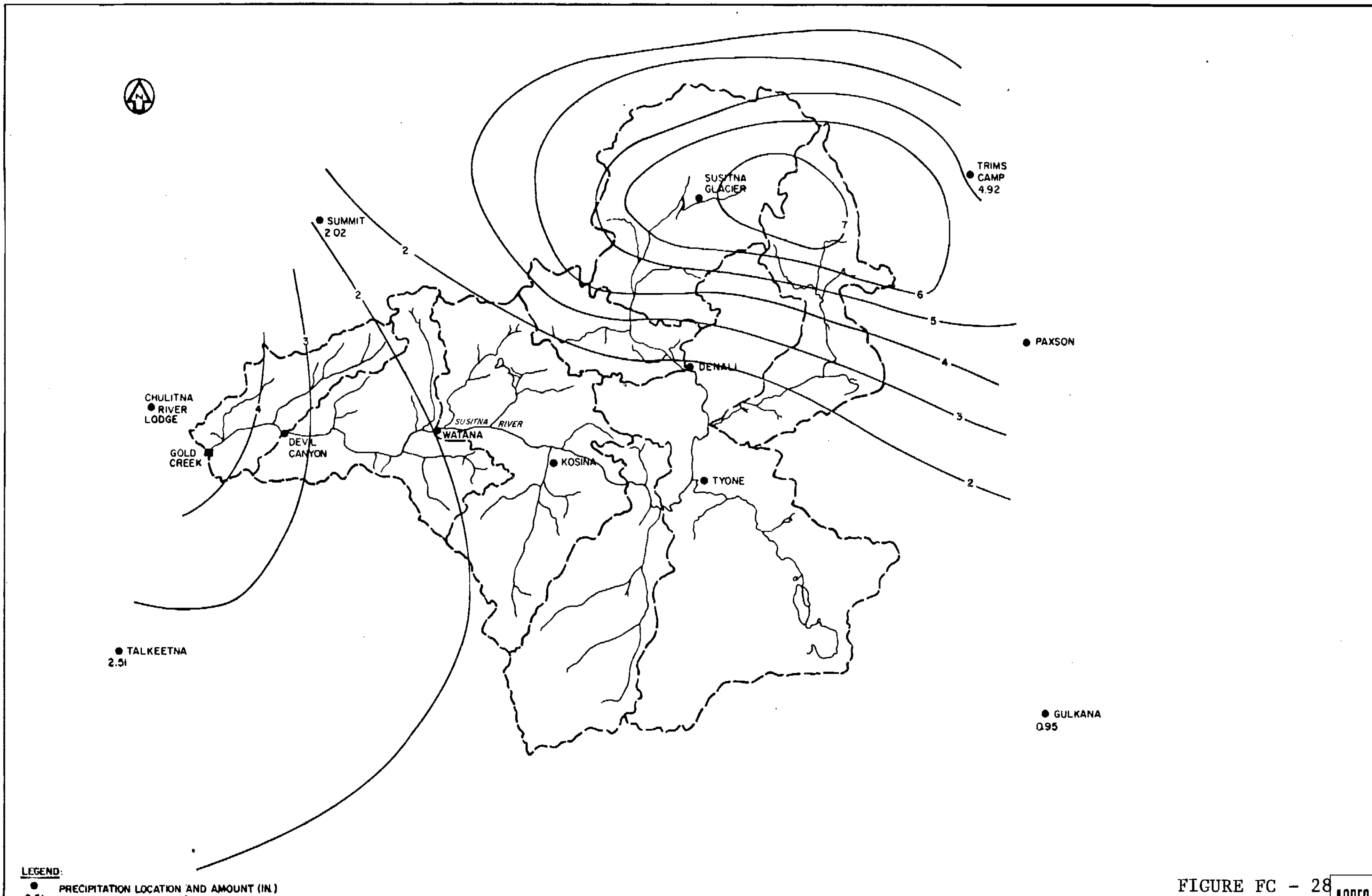


ISOHYETAL MAP STORM OF AUGUST 22-28, 1955

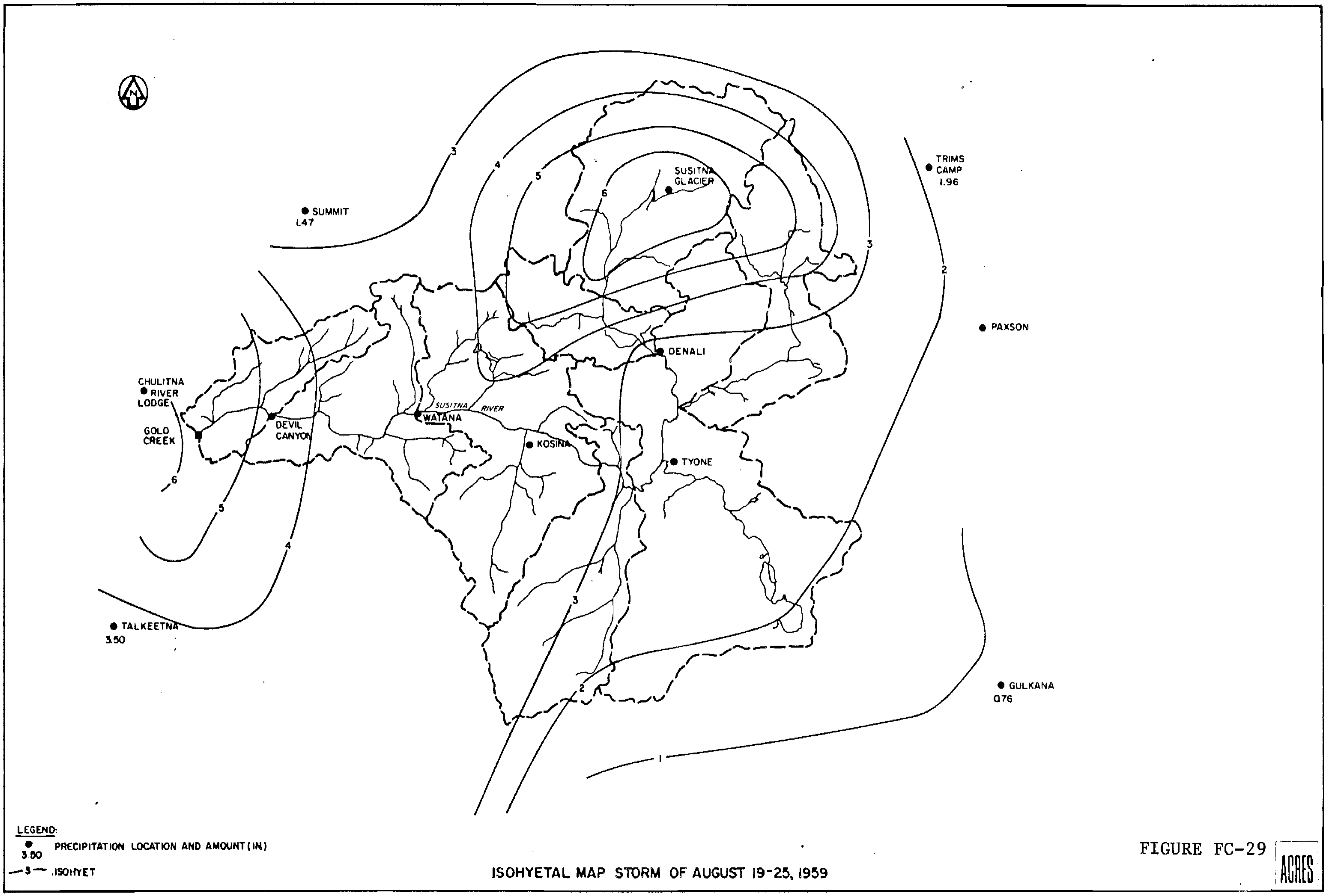
FIGURE FC - 27

ACRES



ISOHYETAL MAP STORM OF JULY 28 - AUGUST 3, 1958

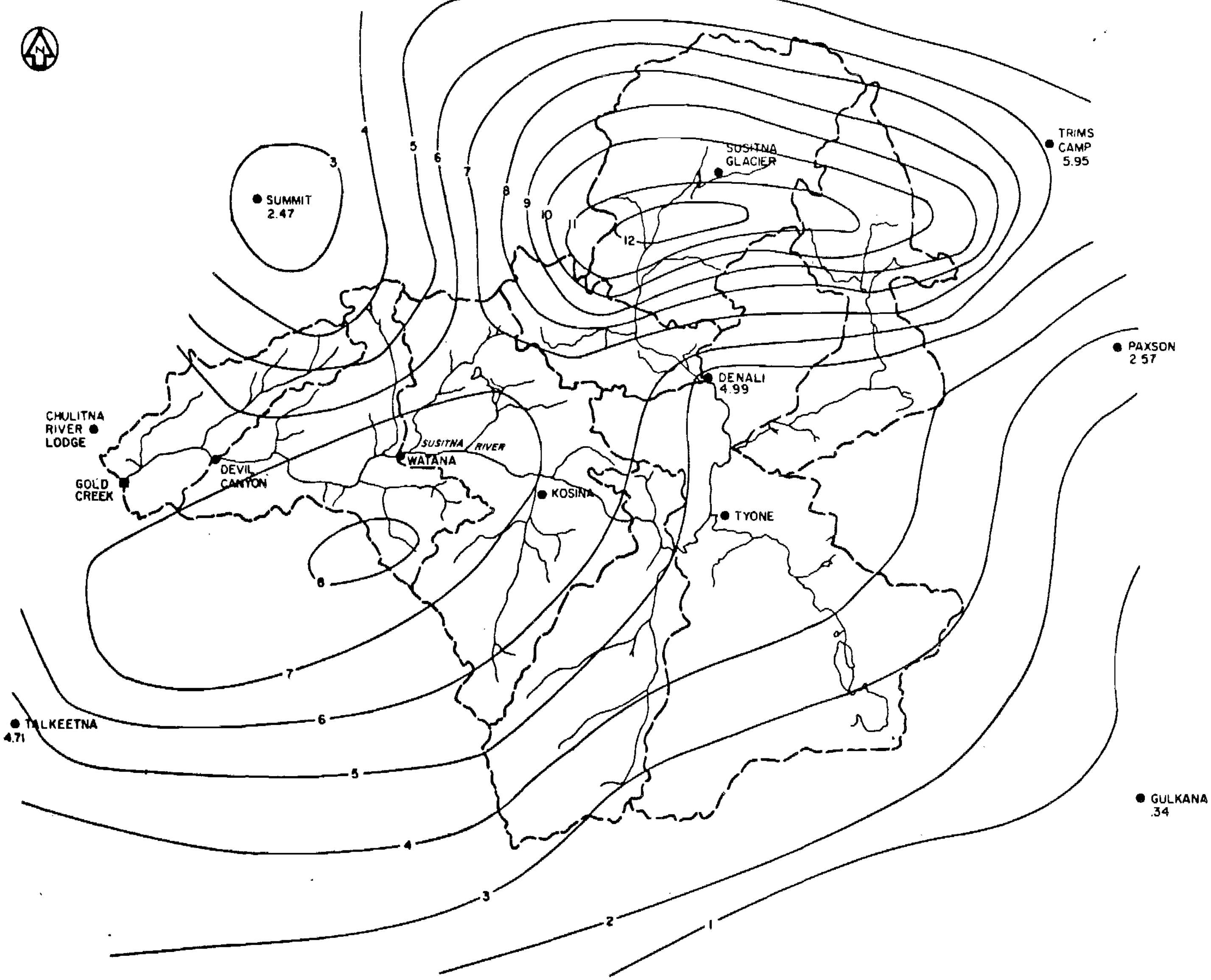
FIGURE FC - 28



ISOHYETAL MAP STORM OF AUGUST 19-25, 1959

FIGURE FC-29

ACRES



LEGEND:

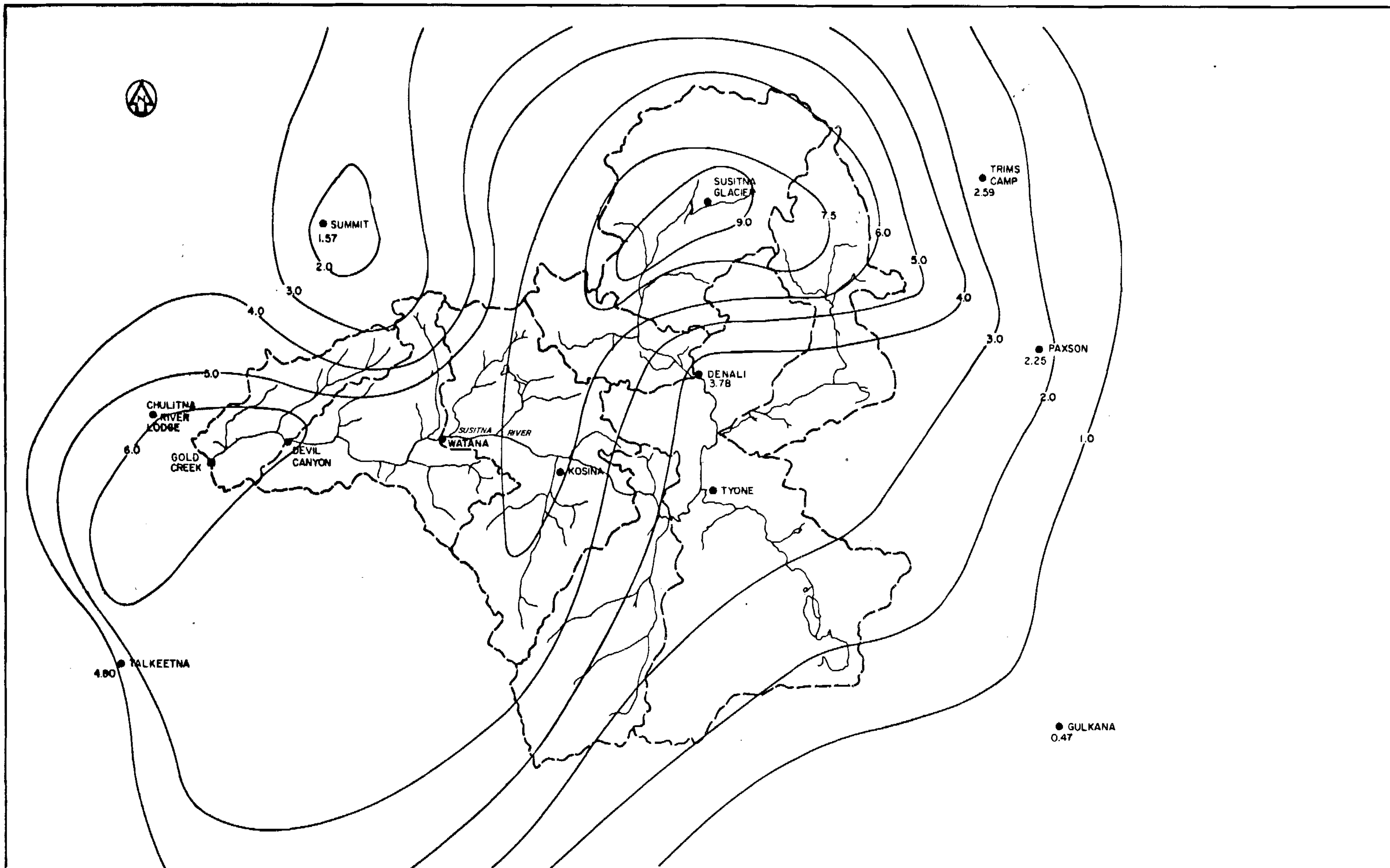
● 2.41 PRECIPITATION LOCATION AND AMOUNT (IN.)

—3— ISOHYET

ISOHYETAL MAP STORM OF AUGUST 9-17, 1967

FIGURE FC- 30

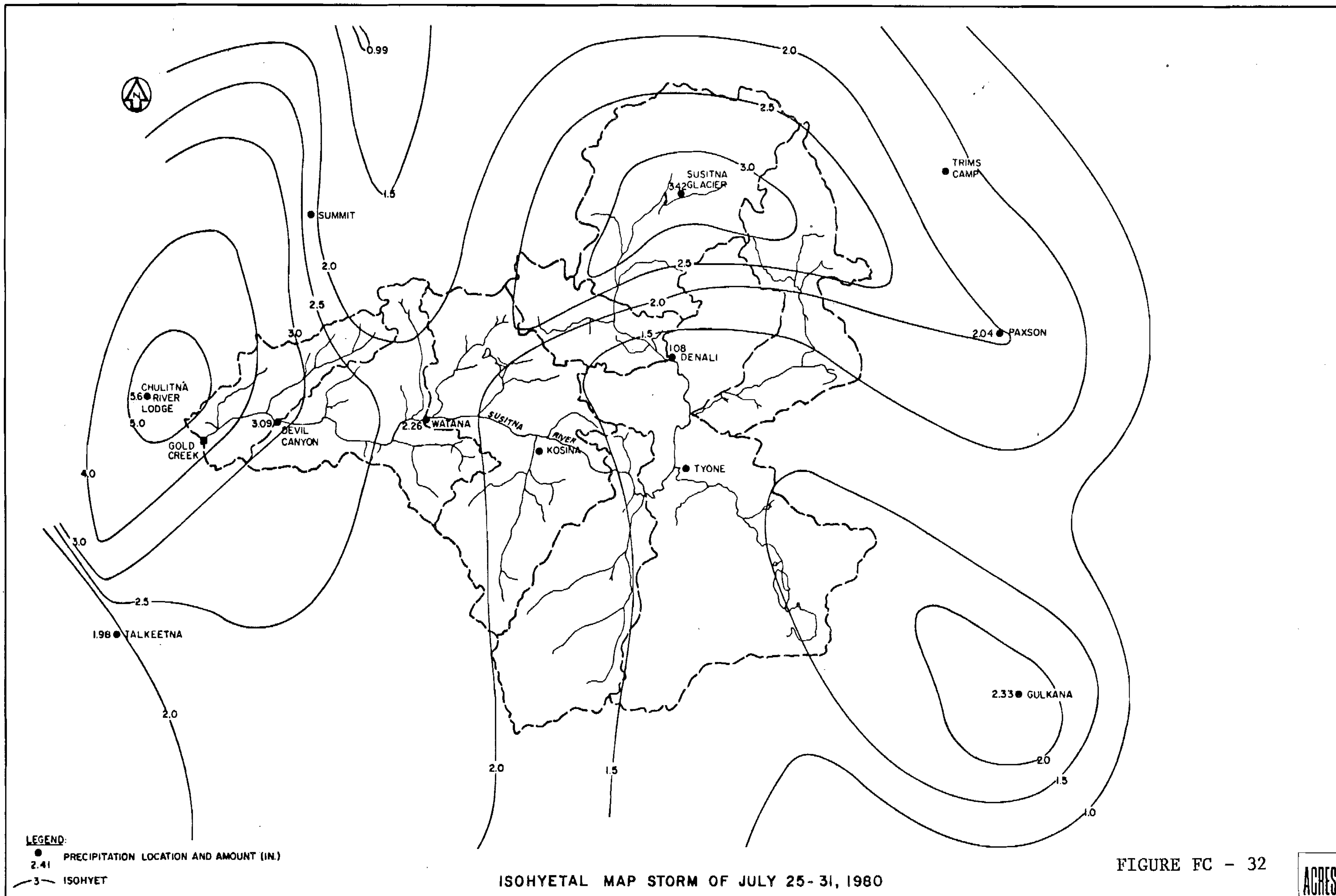
ACRES



ISOHYETAL MAP STORM OF AUGUST 4-10, 1971

FIGURE FC - 31

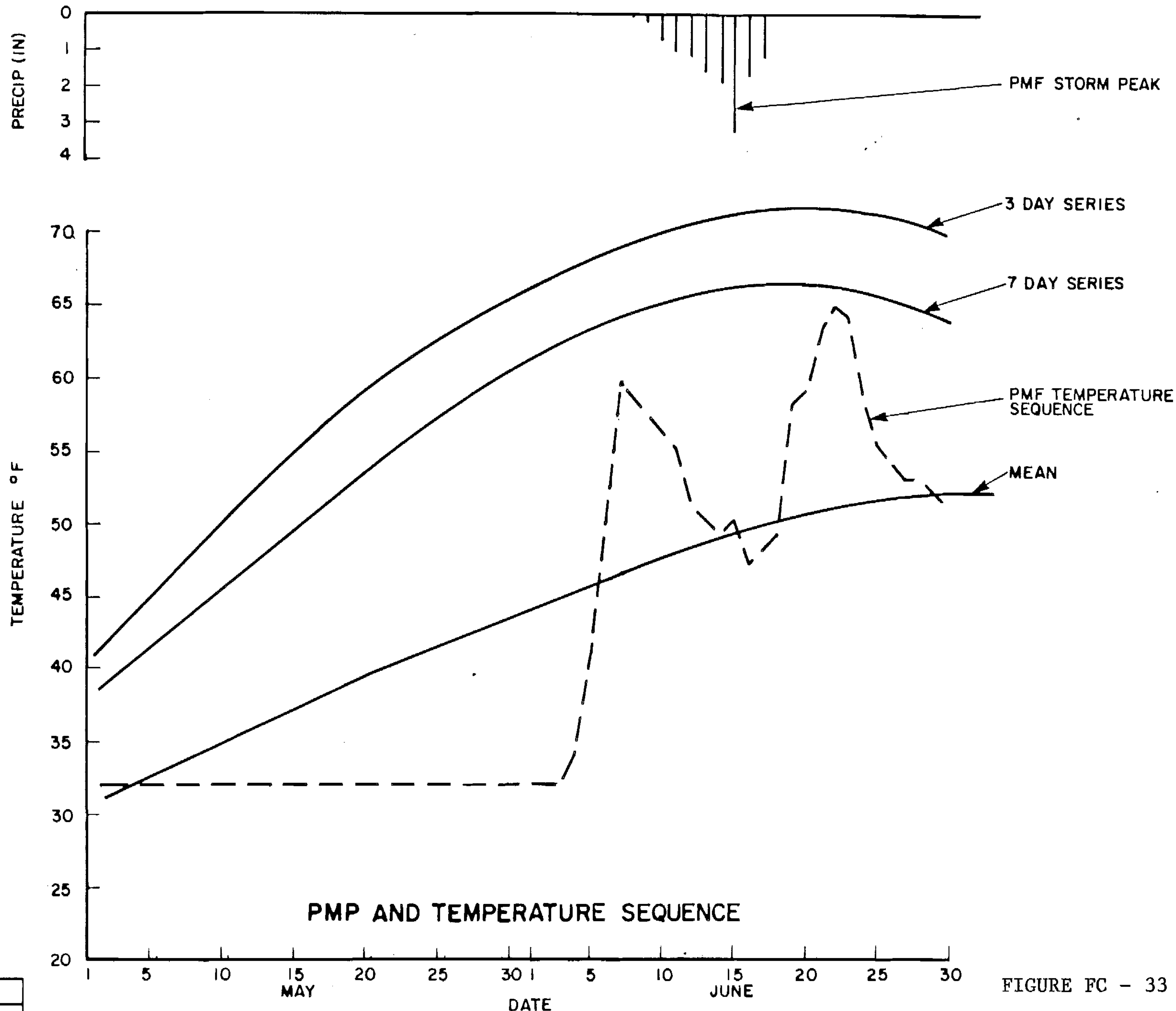
ACRES



ISOHYETAL MAP STORM OF JULY 25-31, 1980

FIGURE FC - 32

ACRES



PMP AND TEMPERATURE SEQUENCE

FIGURE FC - 33

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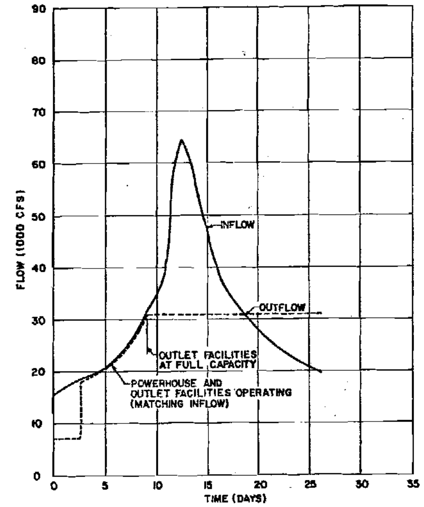
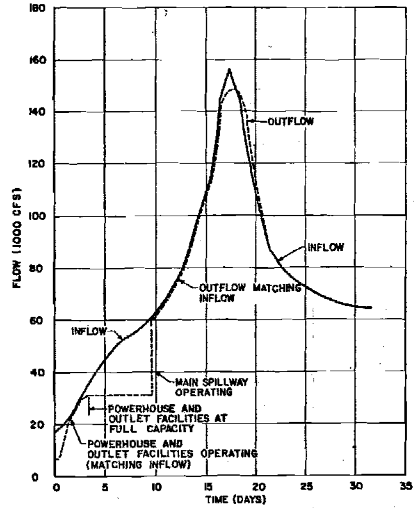


1:50 YEAR FLOOD
(SUMMER)



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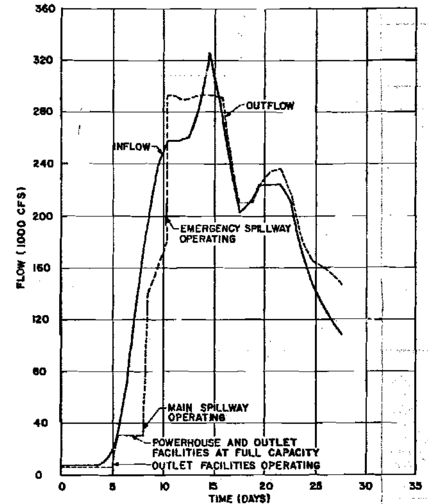
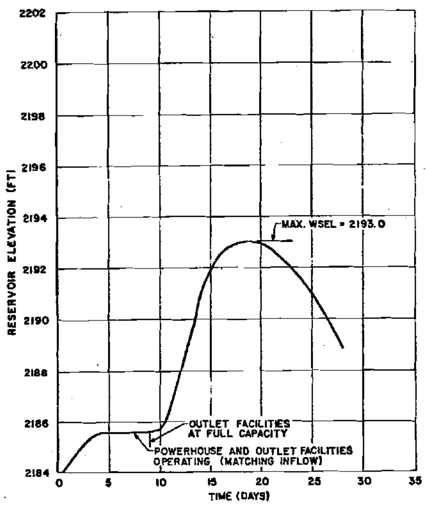


PROBABLE MAXIMUM FLOOD



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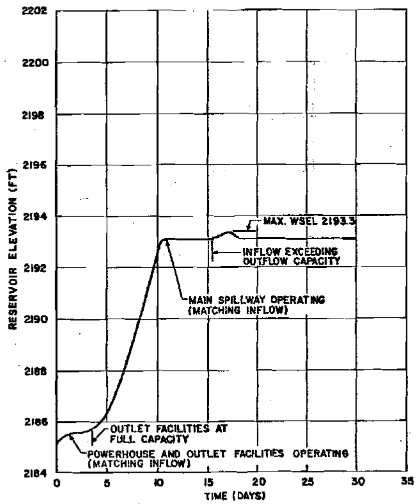
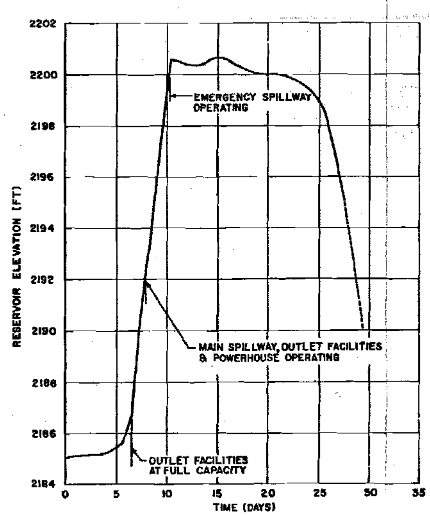
(SUMMER)



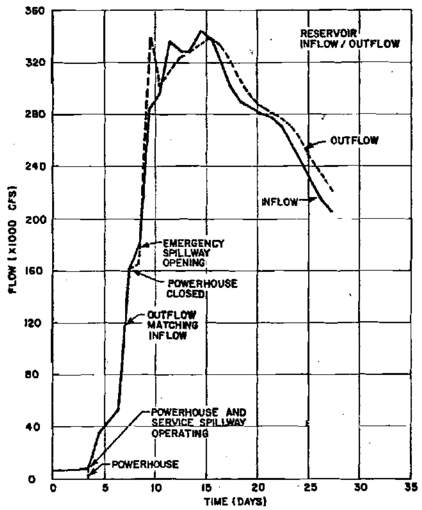
1:10,000 YEAR FLOOD



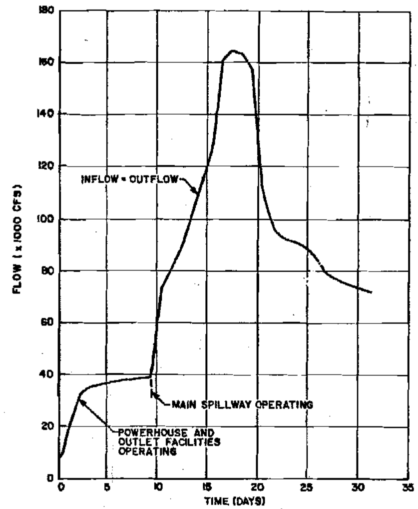
PROBABLE MAXIMUM FLOOD

WATANA
HYDROLOGICAL DATA - SHEET 1

FIGURE FC - 34

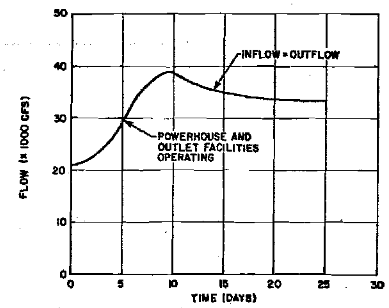


PROBABLE MAXIMUM FLOOD



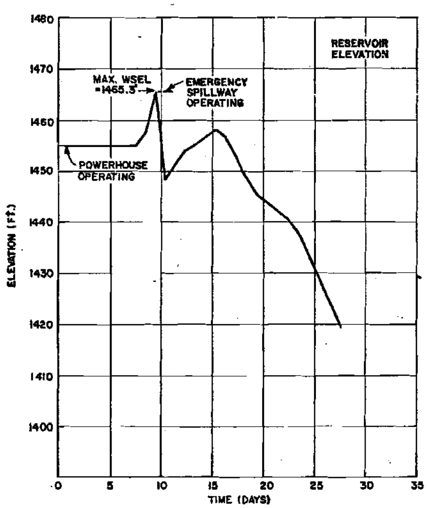
RESERVOIR ROUTING

1:10,000 YR. FLOOD

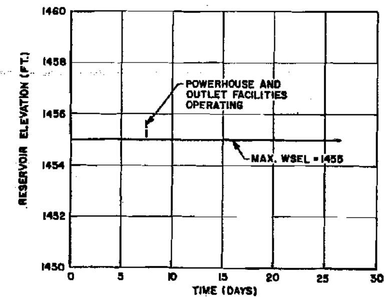


RESERVOIR ROUTING

1:50 YR. SUMMER FLOOD

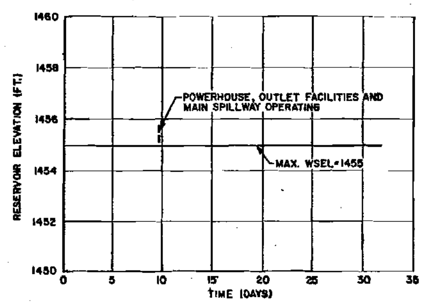


PROBABLE MAXIMUM FLOOD



RESERVOIR ROUTING

1:50 YR. SUMMER FLOOD



RESERVOIR ROUTING

1:10,000 YR. FLOOD

DEVIL CANYON
HYDROLOGICAL DATA - SHEET I

FIGURE FC - 35