3440

BEFORE THE
FEDERAL ENERGY REGULATORY COMMISSION

APPLICATION FOR LICENSE FOR MAJOR PROJECT

SUSITNA HYDROELECTRIC PROJECT

VOLUME 16

DRAFT

EXHIBIT F

HARZA-EBASCO
SUSITNA JOINT VENTURE

Alaska Power Authority

BEFORE THE
FEDERAL ENERGY REGULATORY COMMISSION

APPLICATION FOR LICENSE FOR MAJOR PROJECT

SUSITNA HYDROELECTRIC PROJECT
DRAFT LICENSE APPLICATION

VOLUME 16

EXHIBIT F
SUPPORTING DESIGN REPORT

November 1985

NOTICE

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

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

- (o) No change was made in this section, it remains the same as was presented in the July 29, 1983 License Application
- (*) Only minor changes, largely of an editorial nature, have been made
- (**) Major changes have been made in this section
- (***) This is an entirely new section which did not appear in the July 29, 1983 License Application

VOLUME COMPARISON

VOLUME NUMBER COMPARISON

LICENSE APPLICATION AMENDMENT VS. JULY 29, 1983 LICENSE APPLICATION

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LICENSE APPLICATION

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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	Temperature °F – Averages – Daily minimum	Temperature °F – Averages – Monthly	Temperature °F – Extremes – Highest	Temperature °F – Extremes – Date	Temperature °F – Extremes – Lowest	Temperature °F – Extremes – Date	Degree Days Base 65°F – Heating	Degree Days Base 65°F – Cooling	Precipitation in inches – Water equivalent – Total	Precipitation in inches – Water equivalent – Greatest in 24 hrs.	Precipitation in inches – Water equivalent – Date	Precipitation in inches – Snow, Ice pellets – Total	Precipitation in inches – Snow, Ice pellets – Greatest in 24 hrs.	Precipitation in inches – Snow, Ice pellets – Date	Relative humidity pct. – Hour 02 (local time)	Relative humidity pct. – Hour 08 (local time)	Relative humidity pct. – Hour 14 (local time)	Relative humidity pct. – Hour 20 (local time)	Wind – Resultant – Direction	Wind – Resultant – Speed mph	Wind – Average speed mph	Wind – Fastest mile – Speed mph	Wind – Fastest mile – Direction	Wind – Fastest mile – Date	Percent of possible sunshine	Average sky cover, tenths sunrise to sunset	Number of Days – Sunrise to Sunset – Clear	Number of Days – Sunrise to Sunset – Partly Clear	Number of Days – Sunrise to Sunset – Cloudy	Number of Days – Precipitation .01 inch or more	Number of Days – Snow, Ice pellets 1.0 inch or more	Number of Days – Thunderstorms	Number of Days – Heavy fog visibility 1/4 mile or loss	Number of Days – Temperature °F – Maximum – 90° (b) and above	Number of Days – Temperature °F – Maximum – 32° and below	Number of Days – Temperature °F – Minimum – 32° and below	Number of Days – Temperature °F – Minimum – 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	14.3	23.4	51	30	-3	13	1180	0	0.14	0.08	26	5.8	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	2.90	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	30.8	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	3	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		7.0	3	9	18	13	0	0	2	0	0	17	0	
OCT																							20	08	12														
YEAR																																							

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Normals, Means, And Extremes - through 1975#

Month	Temperature °F							Normal Degree Days Base 65°F		Precipitation in inches										
	Averages			Extremes						Water equivalent							Snow, Ice pallets			
	Daily maximum	Daily minimum	Monthly	Record Highest	Year	Record Lowest	Year	Heating	Cooling	Normal	Maximum monthly	Year	Minimum monthly	Year	Maximum $ in 24 hrs.	Year	Maximum monthly	Year	Maximum in 24 hrs.	Year
(a)				35		35					35		35		35		34		35	
JAN	7.9	-4.8	1.6	44	1945	-45	1971	1965	0	0.91	3.38	1948	0.09	1945	0.80	1948	64.8	1948	16.3	1973
FEB	13.5	-.4	6.6	45	1942	-45	1947	1635	0	1.23	4.31	1951	T	1950	2.79	1951	44.5	1951	28.0	1964
MAR	19.4	3.0	11.2	49	1961	-35	1971	1668	0	1.04	4.53	1946	0.07	1961	1.67	1946	59.1	1946	18.1	1946
APR	32.9	14.1	23.5	57	1956	-30	1944	1245	0	0.67	4.45	1966	0.06	1944	0.97	1963	28.7	1970	9.7	1963
MAY	45.7	29.1	37.4	76	1960	-14	1945	856	0	0.77	2.66	1966	0.04	1949	0.96	1946	17.4	1958	7.5	1946
JUN	58.0	39.9	49.0	89	1961	25	1947	480	0	2.19	4.45	1949	0.41	1942	2.22	1967	9.4	1974	8.7	1974
JUL	60.2	43.8	52.0	81	1961	32	1970	403	0	3.09	5.56	1959	1.17	1955	1.95	1948	9.7	1970	9.7	1970
AUG	56.0	41.1	48.6	81	1968	20	1955	508	0	3.30	6.33	1955	0.70	1941	2.10	1944	9.0	1955	6.0	1955
SEP	47.1	32.6	39.9	75	1957	6	1956	753	0	2.81	6.13	1965	0.29	1969	2.07	1944	21.5	1958	14.0	1955
OCT	30.4	17.5	24.0	59	1969	-15	1975	1271	0	1.62	3.79	1952	0.12	1967	1.24	1963	54.8	1970	12.6	1970
NOV	15.7	3.7	9.7	44	1962	-29	1948	1659	0	1.23	4.85	1952	0.06	1963	1.30	1964	75.1	1967	21.9	1970
DEC	9.2	-3.6	2.9	42	1969	-43	1961	1925	0	1.20	4.63	1951	0.24	1945	1.09	1967	50.7	1970	27.4	1970
YEAR	33.0	18.0	25.5	89	JUN 1961	-45	JAN 1971	14368	0	20.06	6.74	AUG 1944	T	FEB 1950	2.79	FEB 1951	75.1	NOV 1967	28.0	FEB 1964

Month	Relative humidity pct.				Wind					Pct. of possible sunshine	Mean sky cover, tenths sunrise to sunset	Mean Number of Days											Average station pressure mb
	Hour 02 (local time)	Hour 08 (local time)	Hour 14 (local time)	Hour 20 (local time)	$ Mean speed mph a	Prevailing direction	Fastest mile Speed mph	Fastest mile Direction	Fastest mile Year			Sunrise to Sunset Clear	Sunrise to Sunset Partly Clear	Sunrise to Sunset Cloudy	| Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog visibility 1/4 mile or loss	Temperatures °F Maximum (b) 90° and above	Temperatures °F Maximum 32° and below	Temperatures °F Minimum 32° and below	Temperatures °F Minimum 0° and below	Elev. 2405 feet m.s.l.
(a)	5	7	7	6	8	5	7	7			7	7	7	7	20	8	8	8	34	34	34	34	2
JAN	68	68	69	68	15.1	NE	44	05	1968		5.2	13	5	13	9	4	0	*	0	30	31	20	921.4
FEB	76	75	75	76	11.9	NE	46	07	1974		7.0	6	5	17	10	5	0	1	0	26	28	15	918.8
MAR	76	76	70	73	11.1	NE	48	10	1971		6.2	9	6	16	10	5	0	1	0	27	31	14	917.2
APR	80	75	65	75	7.6	NE	33	08	1971		7.2	5	7	18	7	4	0	1	0	13	30	3	922.9
MAY	83	70	58	67	7.7	W	28	07	1969		7.5	3	9	19	7	2	*	1	*	1	22	*	923.1
JUN	84	73	57	65	8.3	SW	29	22	1970		8.2	2	6	22	12	1	2	1	3	0	2	0	924.7
JUL	89	78	62	72	7.8	SW	30	23	1974		8.2	2	7	22	16	*	2	1	5	0	*	0	929.1
AUG	88	81	62	76	7.4	SW	31	22	1975		8.3	2	6	23	18	0	*	1	1	0	2	0	930.3
SEP	85	81	59	75	7.5	NE	32	23	1972		7.4	5	5	20	16	2	*	1	*	1	14	0	926.1
OCT	83	85	76	81	8.0	NE	35	23	1970		7.6	3	5	21	13	7	0	2	0	18	30	2	916.7
NOV	79	79	78	79	11.7	NE	39	25	1970		7.1	7	4	19	9	5	0	1	0	27	30	13	921.3
DEC	76	78	76	77	11.3	NE	44	11	1970		6.5	9	5	17	11	6	0	1	0	30	31	19	914.7
YEAR	81	76	67	74	9.7	NE	48	10	MAR 1971		7.2	68	70	227	138	41	5	12	9	173	251	86	922.0

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

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

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

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

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

TABLE F.2.3.2: SUMMARY OF CLIMATOLOGICAL DATA

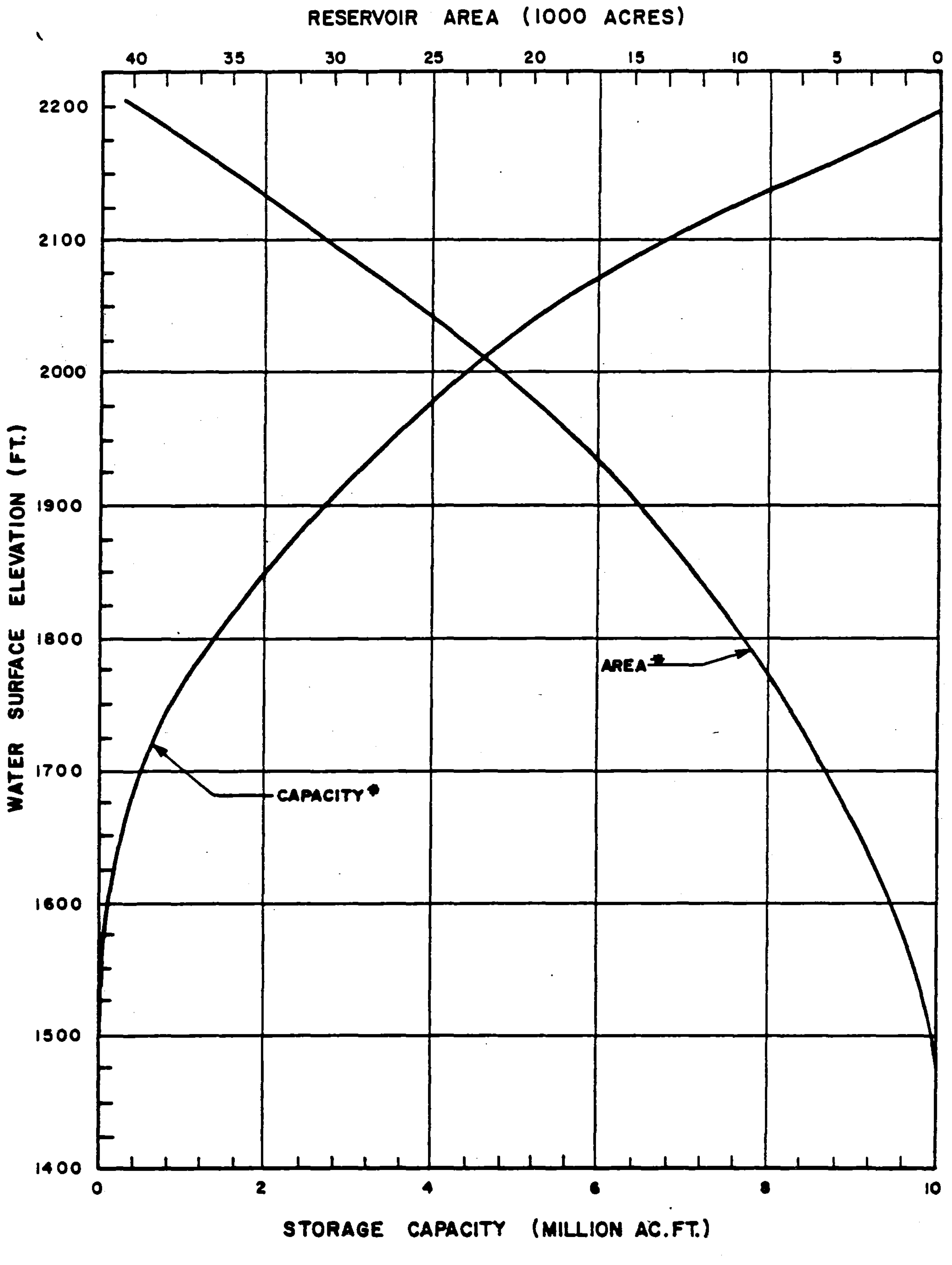
	Mean Monthly Precipitation (Inches)												
STATION	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
Anchorage	0.84	0.56	0.56	0.56	0.59	1.07	2.07	2.32	2.37	1.43	1.02	1.07	
Big Delta	0.36	0.27	0.33	0.31	0.94	2.20	2.49	1.92	1.23	0.56	0.41	0.42	11.44
Fairbanks	0.60	0.53	0.48	0.33	0.65	1.42	1.90	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	19.3
Talkeetna	1.63	1.79	1.54	1.12	1.46	2.17	3.48	4.89	4.52	2.54	1.79	1.71	28.64

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

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

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

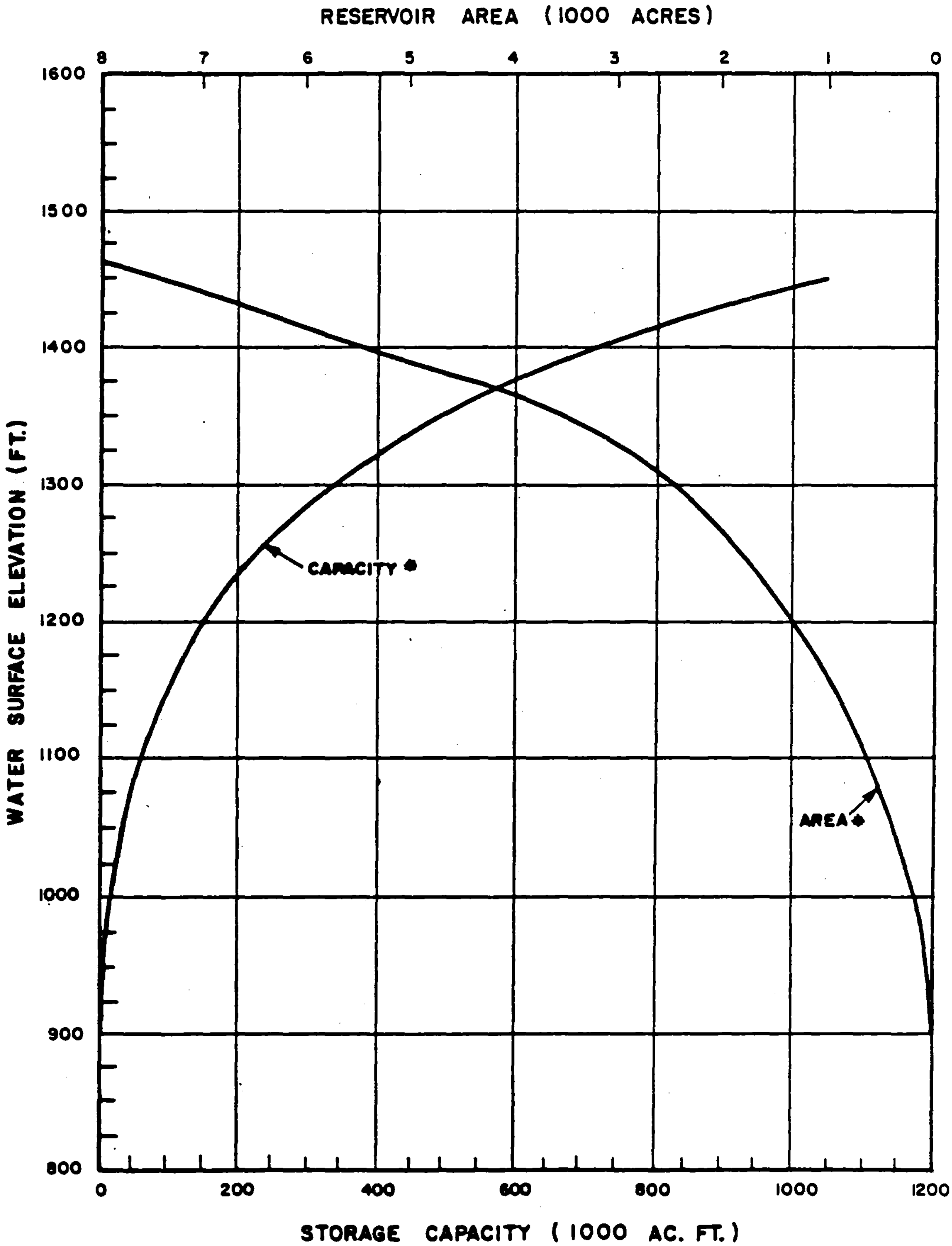
FIGURES



AREA AND CAPACITY CURVES
WATANA RESERVOIR

*WITHOUT RESERVOIR SILTATION

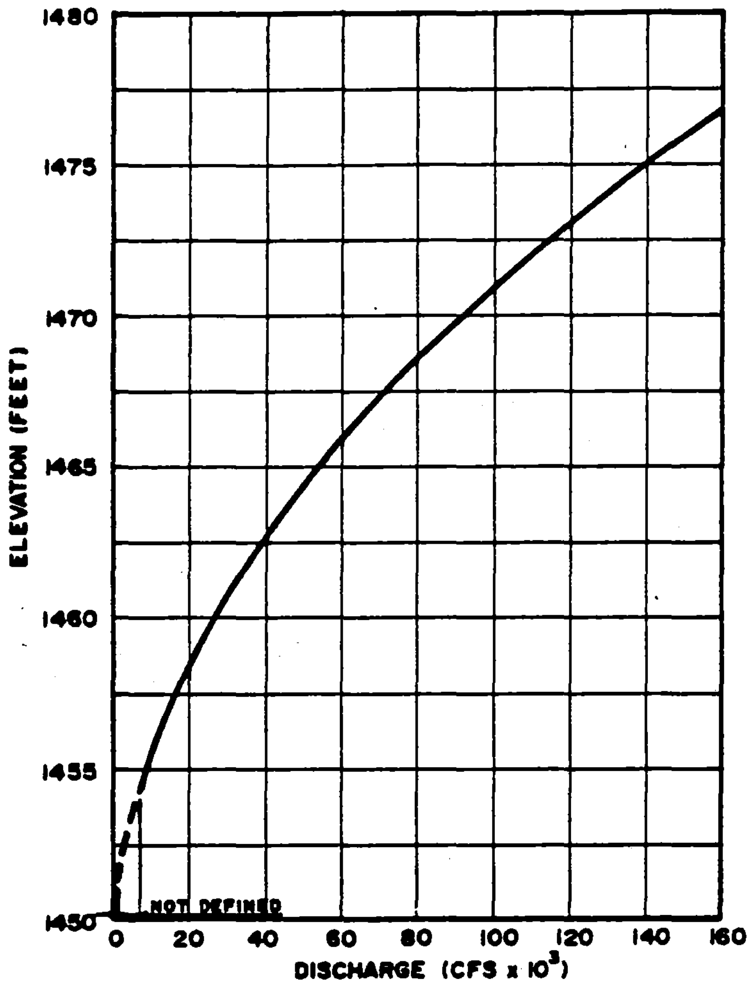
FIGURE F.2.4.1



AREA AND CAPACITY CURVES
DEVIL CANYON RESERVOIR

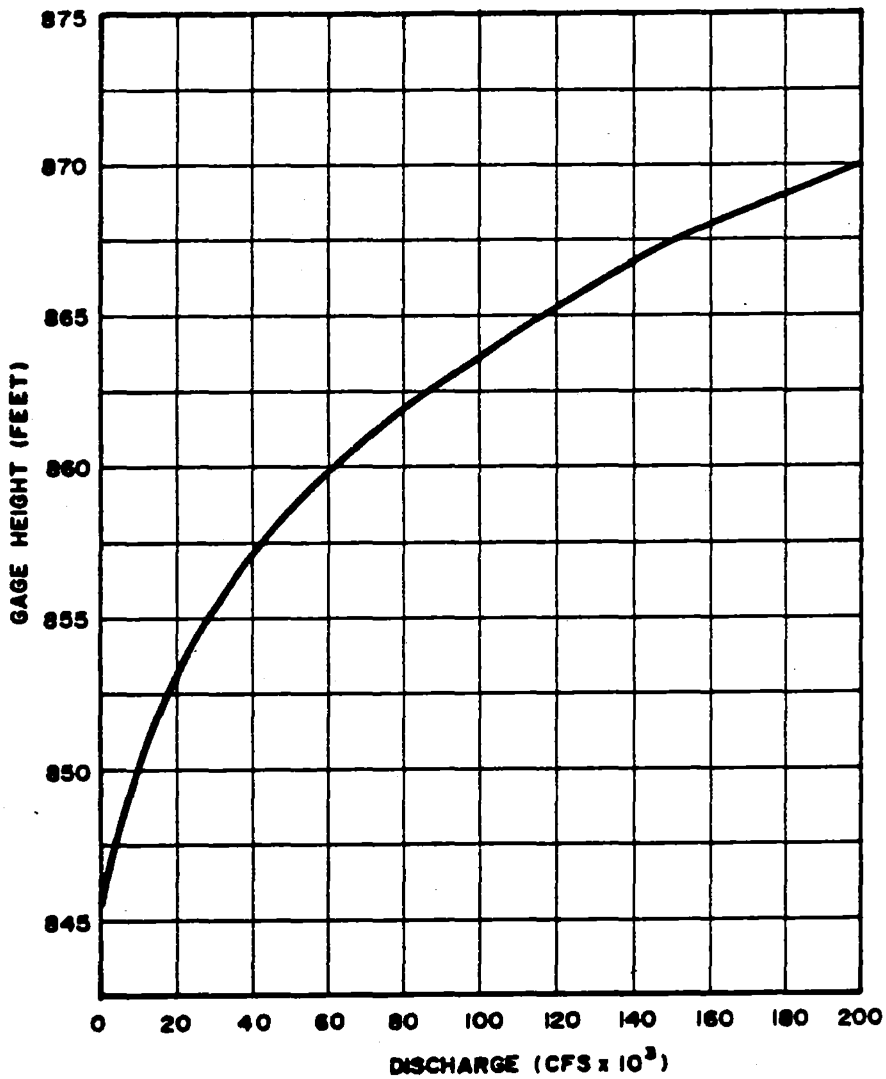
*WITHOUT RESERVOIR SILTATION

FIGURE F.2.4.2



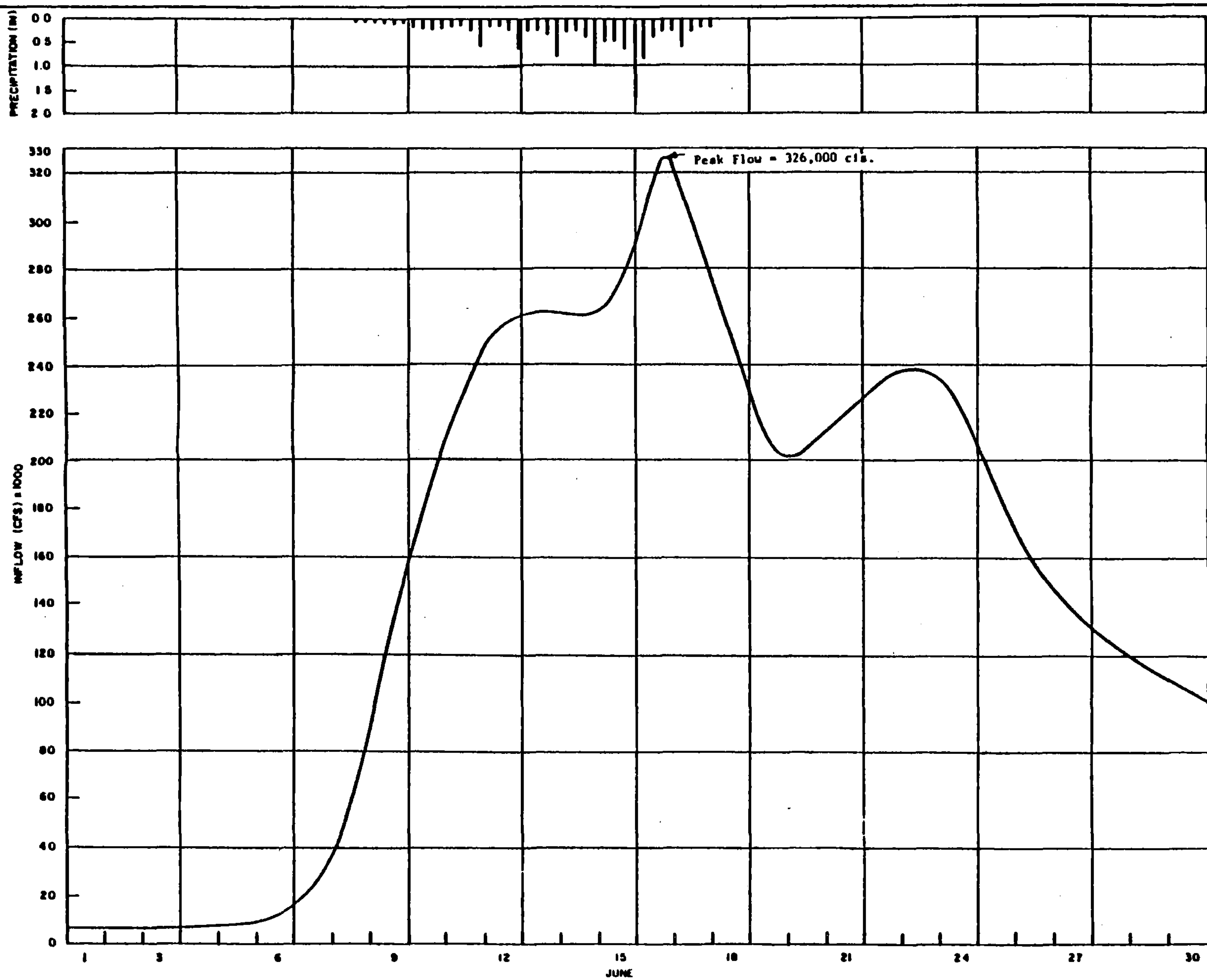
WATANA TAILWATER RATING

FIGURE F.2.5.1



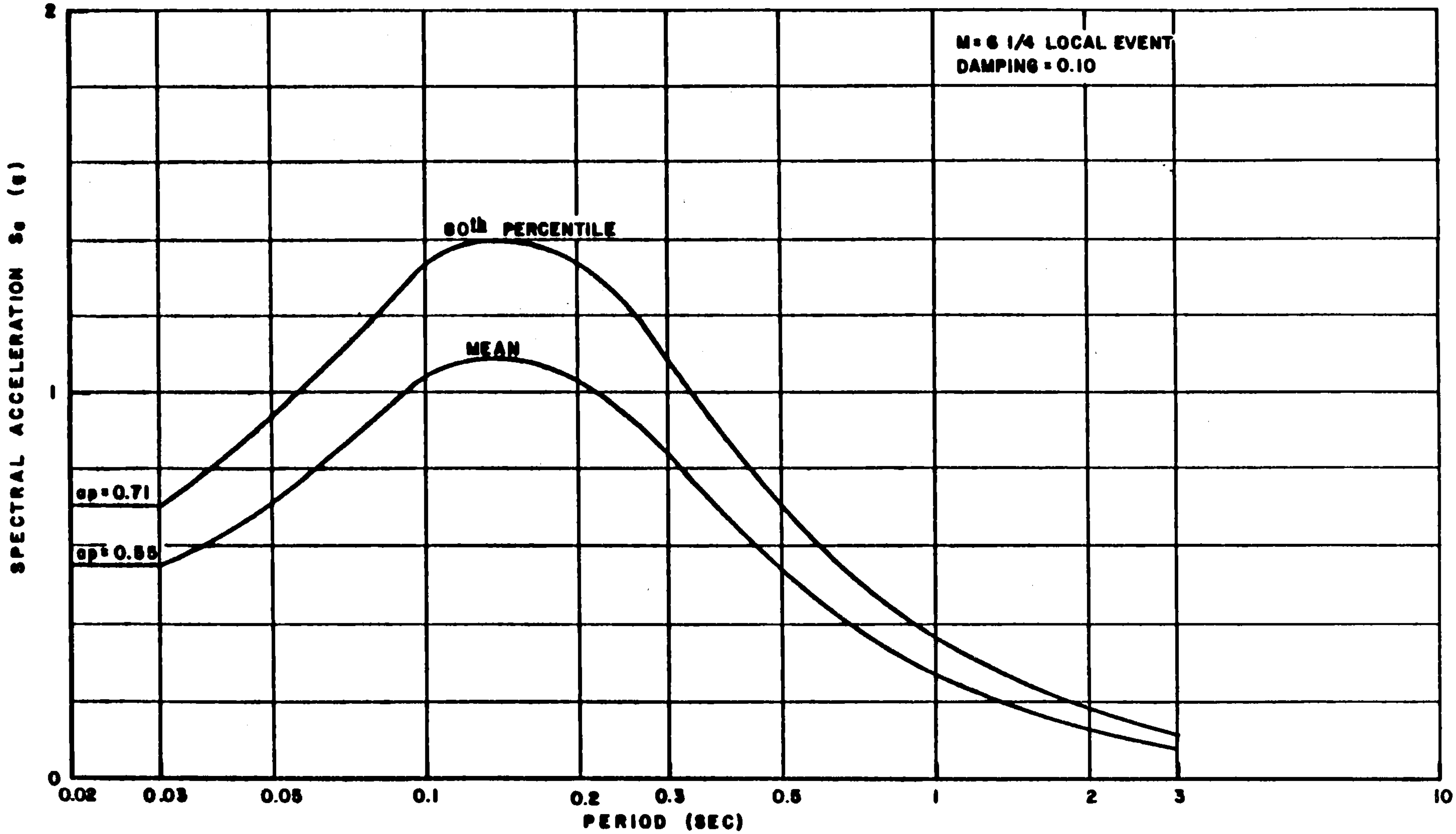
DEVIL CANYON
TAILWATER RATING
(TAILRACE TO PORTAGE CREEK)

FIGURE F.2.5.2



WATANA PMF INFLOW HYDROGRAPH

FIGURE F.2.6.1



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

FIGURE F.3.2.1

APPENDIX F1

APPENDIX F1

This Appendix deleted.

APPENDIX F2
WATANA AND DEVIL CANYON EMBANKMENT STABILITY ANALYSES

APPENDIX F2
WATANA AND DEVIL CANYON EMBANKMENT STABILITY ANALYSES

1 - PRELIMINARY DESIGN (**)

1.1 - General (**)

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

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

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

1.2 - Design Shear Strengths (***)

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

1.2.1 - Material Design Parameters (***)

(a) Impervious Core

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

(b) Rockfill and Filters

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

(c) Overburden Foundation

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

(d) Bedrock Formation

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

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

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

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

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

1.3 - Method of Analysis (***)

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

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

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

1.4 - Design Cases and Assumptions (***)

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

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

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

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

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

1.4.2 - <u>Partial Pool Case</u> (***)

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

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

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

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

1.4.3 - <u>Steady State Seepage Case</u> (***)

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

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

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

1.4.4 - Rapid Drawdown Case (***)

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

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

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

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

1.4.5 - Earthquake Case (***)

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

1.5 - Dynamic Stability Evaluation (***)

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

1.5.1 - Oroville Dam (***)

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

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

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

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

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

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

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

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

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

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

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

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

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

1.5.2 - Miboro Dam (***)

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

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

1.5.3 - Cogoti Dam (***)

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

1.5.4 - La Honda Dam (***)

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

$$a_{max} = 0.50g$$

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

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

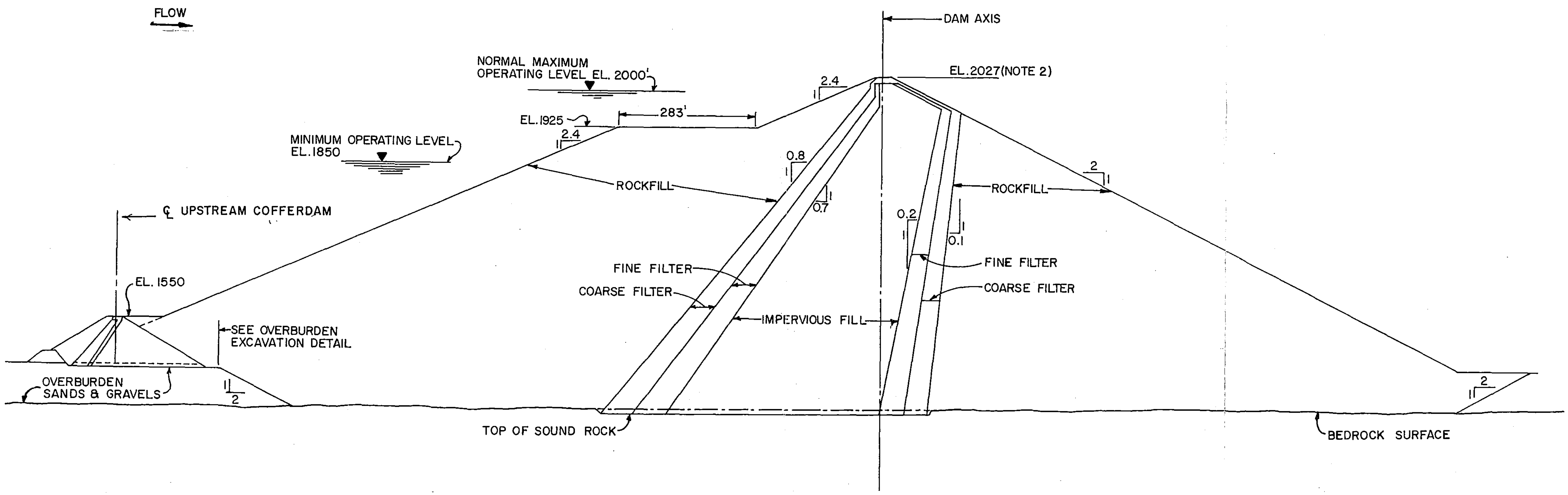
1.5.5 - Watana Dam (***)

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

1.6 - Conclusion (**)

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

FIGURES



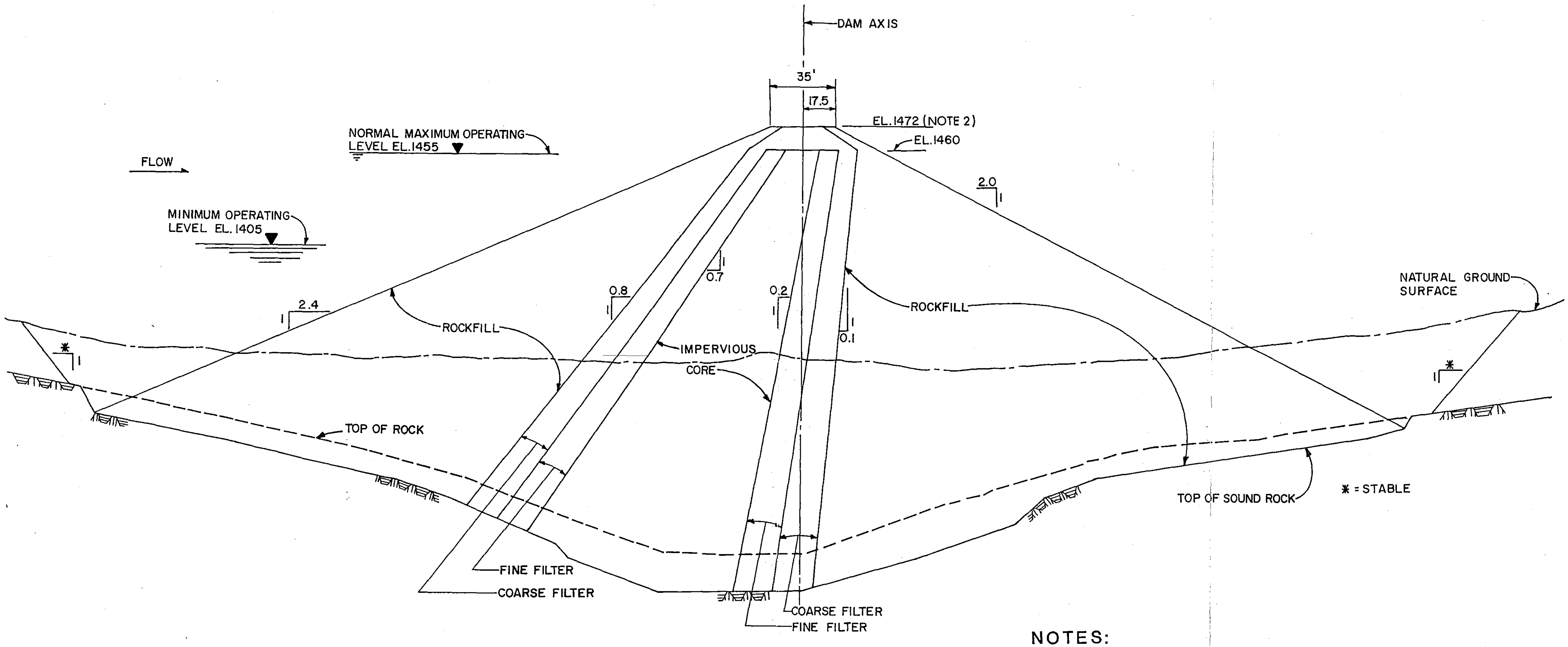
NOTES:

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

WATANA DAM - STAGE I

AT MAXIMUM HEIGHT

FIGURE F2.1



NOTES:

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

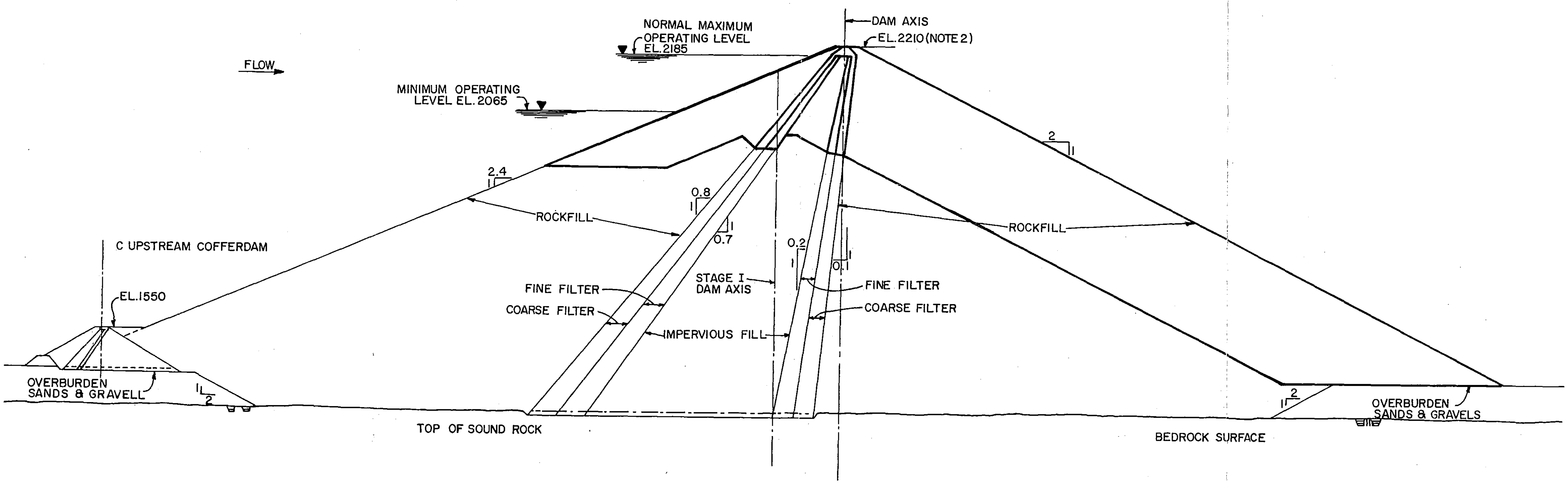
SCALE 0 30 60 FEET

DEVIL CANYON - STAGE II

SECTION THROUGH SADDLE DAM

AT MAXIMUM HEIGHT

FIGURE F2.2



NOTES:

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

SCALE 0 100 200 FEET

WATANA DAM - STAGE III

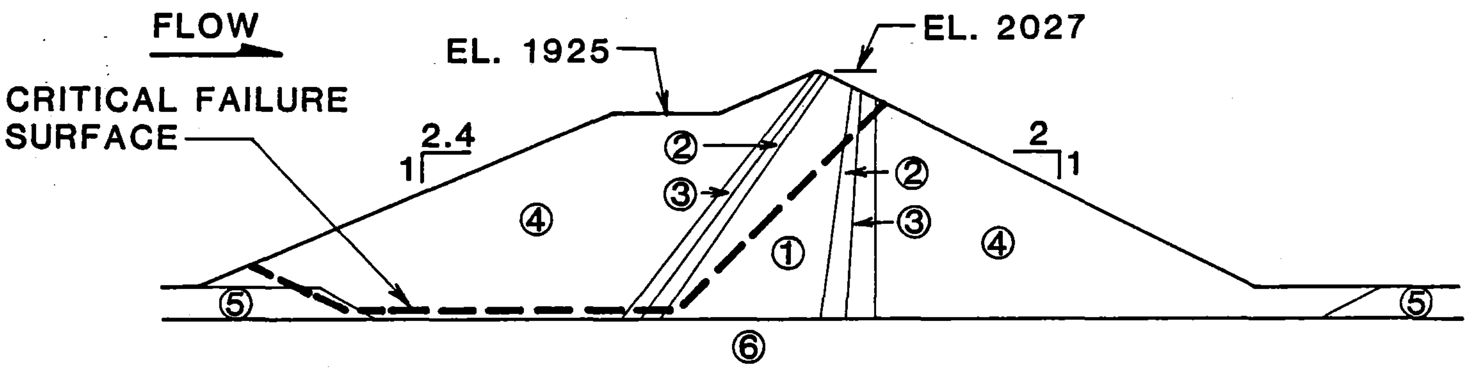
AT MAXIMUM HEIGHT

FIGURE F2.3

WATANA-STAGE I

SLOPE STABILITY-FACTOR OF SAFETY

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



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

NOTE

MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

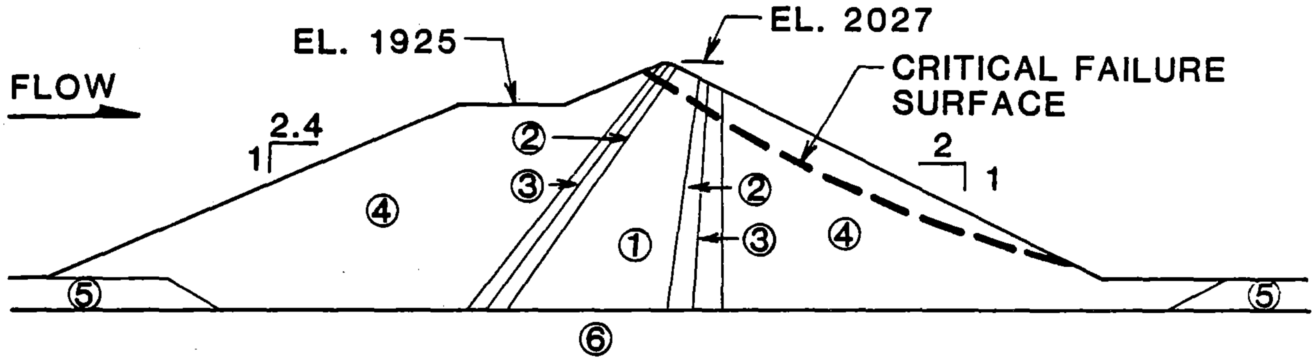
END-OF-CONSTRUCTION CASE
(UPSTREAM SLOPE)

FIGURE F2.4

WATANA-STAGE I

SLOPE STABILITY-FACTOR OF SAFETY

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



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

NOTE

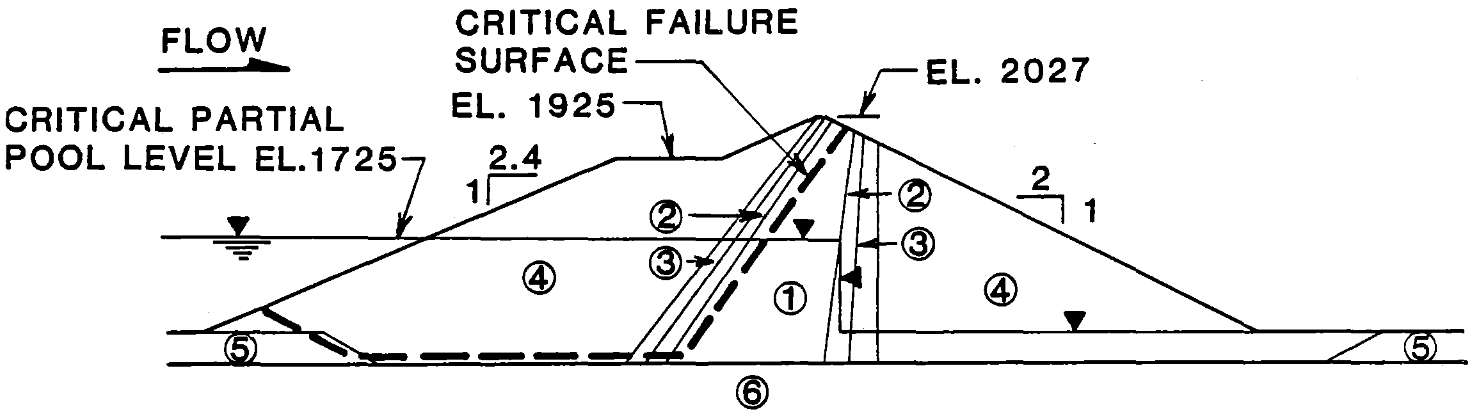
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

END-OF-CONSTRUCTION CASE
(DOWNSTREAM SLOPE)

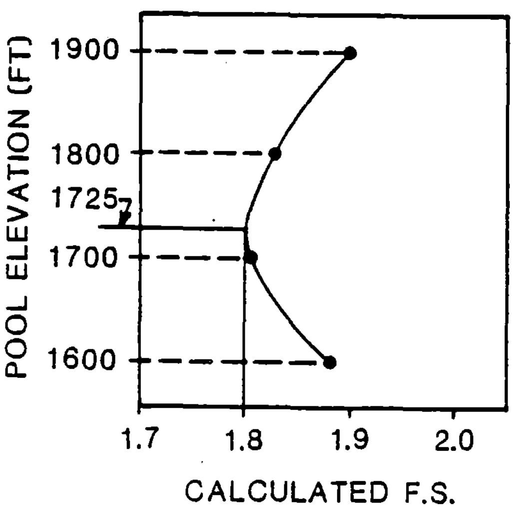
FIGURE F2.5

WATANA-STAGE I
SLOPE STABILITY-FACTOR OF SAFETY

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



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



NOTE

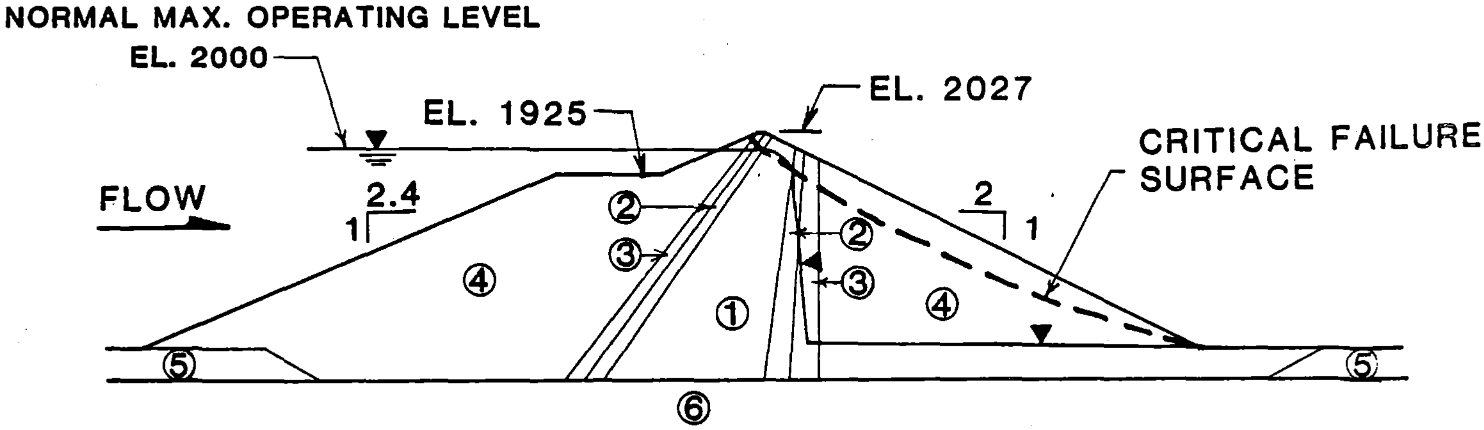
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

PARTIAL POOL CASE
(UPSTREAM SLOPE)

FIGURE F2.6

WATANA-STAGE I
SLOPE STABILITY-FACTOR OF SAFETY

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



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

NOTE

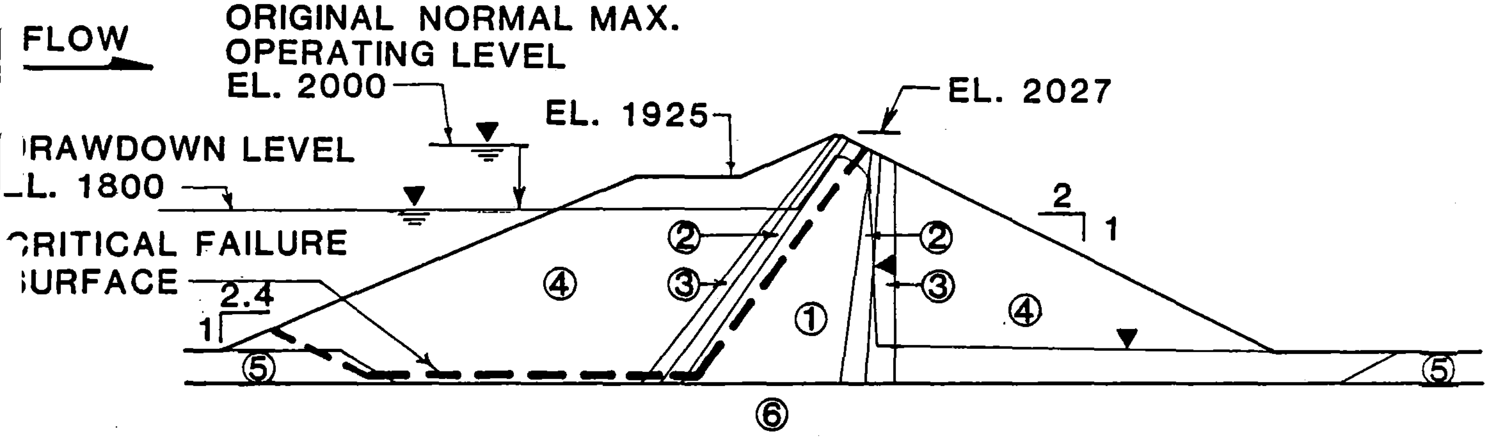
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

STEADY STATE SEEPAGE CASE
(DOWNSTREAM SLOPE)

FIGURE F2.7

WATANA-STAGE I
SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.0	1.8



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

NOTE

MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

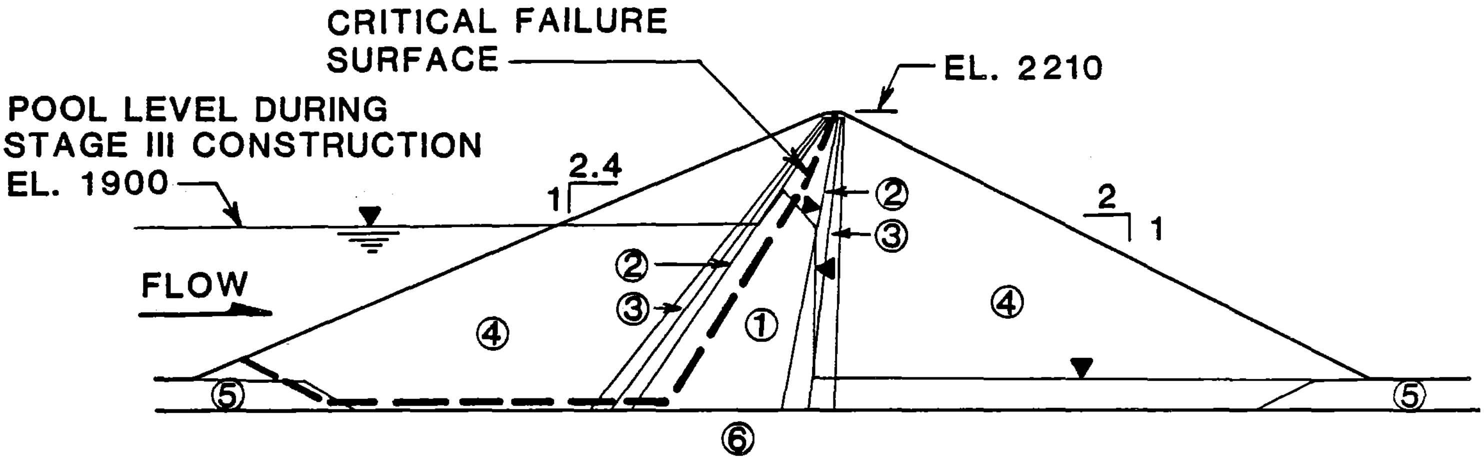
RAPID DRAWDOWN CASE
(UPSTREAM SLOPE)

FIGURE F2.8

WATANA-STAGE III

SLOPE STABILITY-FACTOR OF SAFETY

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



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

NOTE

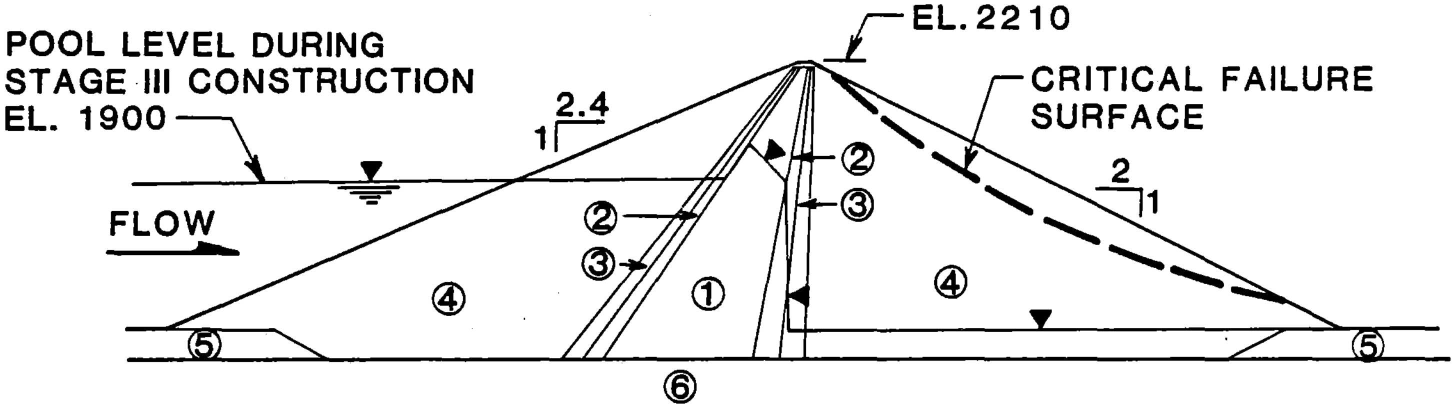
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

END-OF-CONSTRUCTION CASE
(UPSTREAM SLOPE)

FIGURE F2.9

WATANA-STAGE III
SLOPE STABILITY-FACTOR OF SAFETY

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



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

NOTE

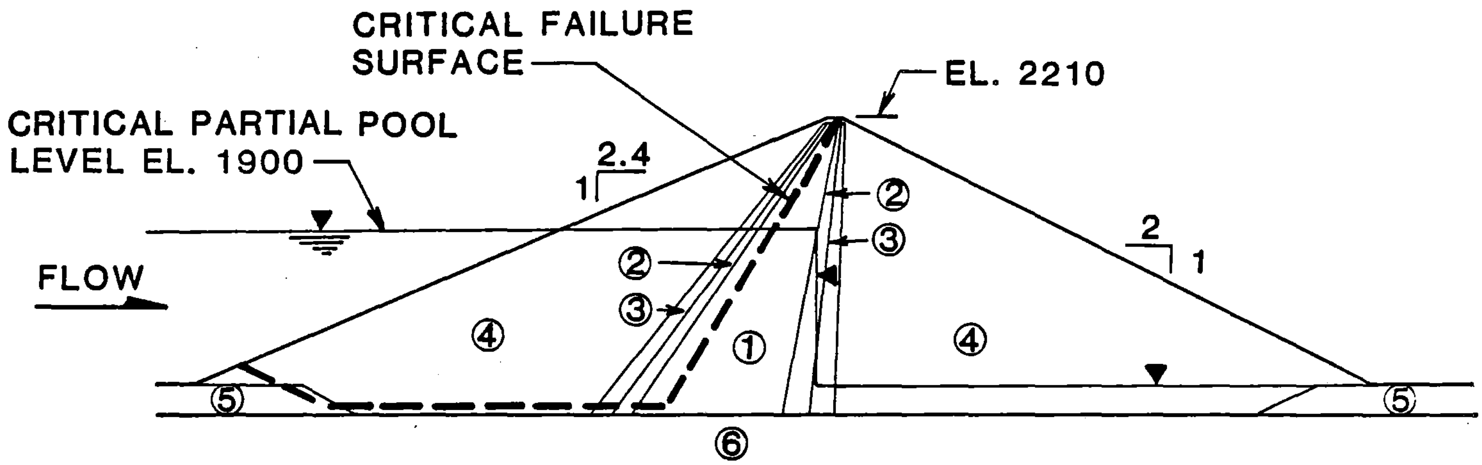
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

END-OF-CONSTRUCTION CASE
(UPSTREAM SLOPE)

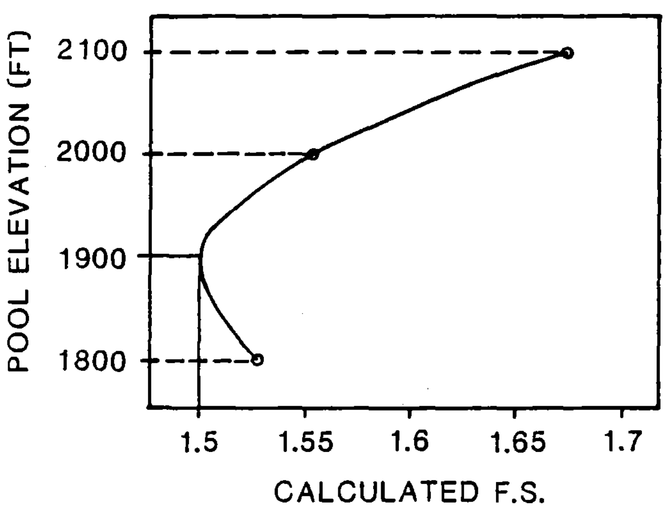
FIGURE F2.10

WATANA-STAGE III
SLOPE STABILITY-FACTOR OF SAFETY

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



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



NOTE

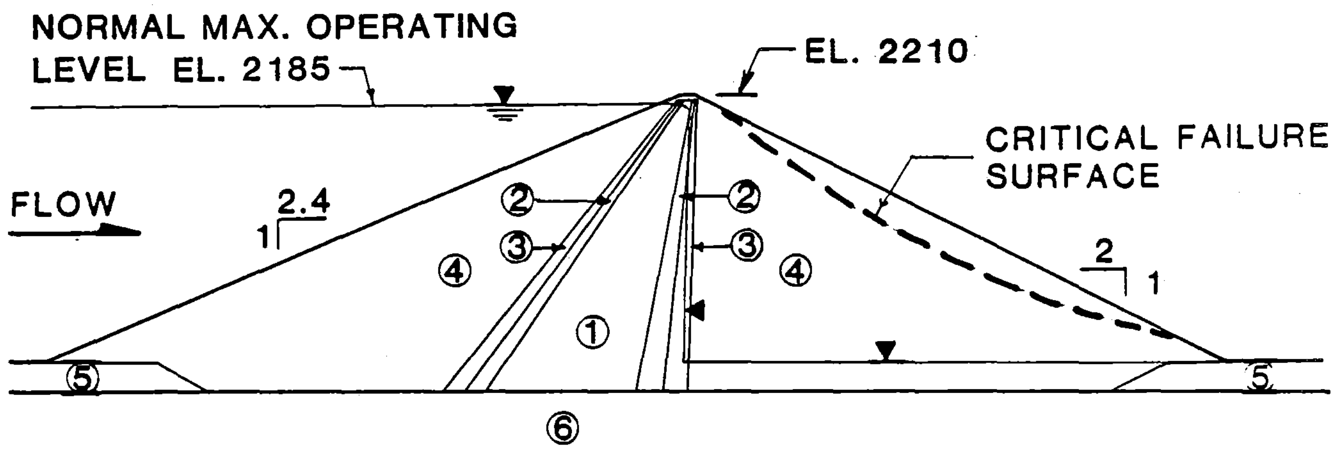
MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

PARTIAL POOL CASE
(UPSTREAM SLOPE)

FIGURE F2.11

WATANA-STAGE III
SLOPE STABILITY-FACTOR OF SAFETY

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



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

NOTE

MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

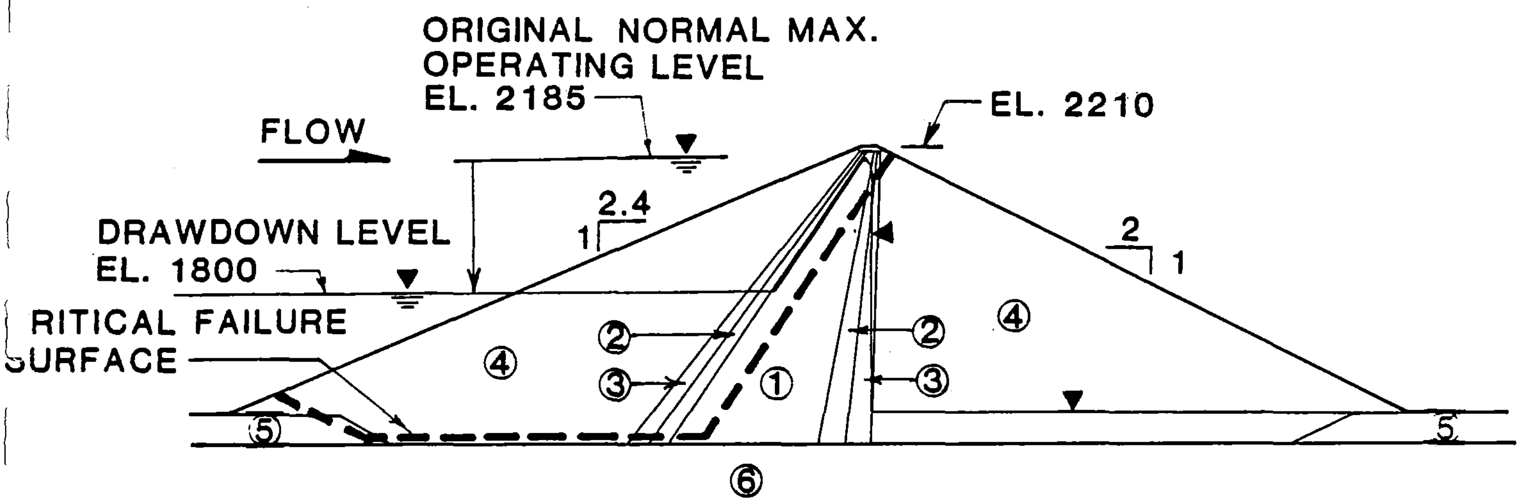
STEADY STATE SEEPAGE CASE
(DOWNSTREAM SLOPE)

FIGURE E2.12

WATANA-STAGE III

SLOPE STABILITY-FACTOR OF SAFETY

CONDITION	ALLOWABLE	CALCULATED
STATIC	1.0	1.3



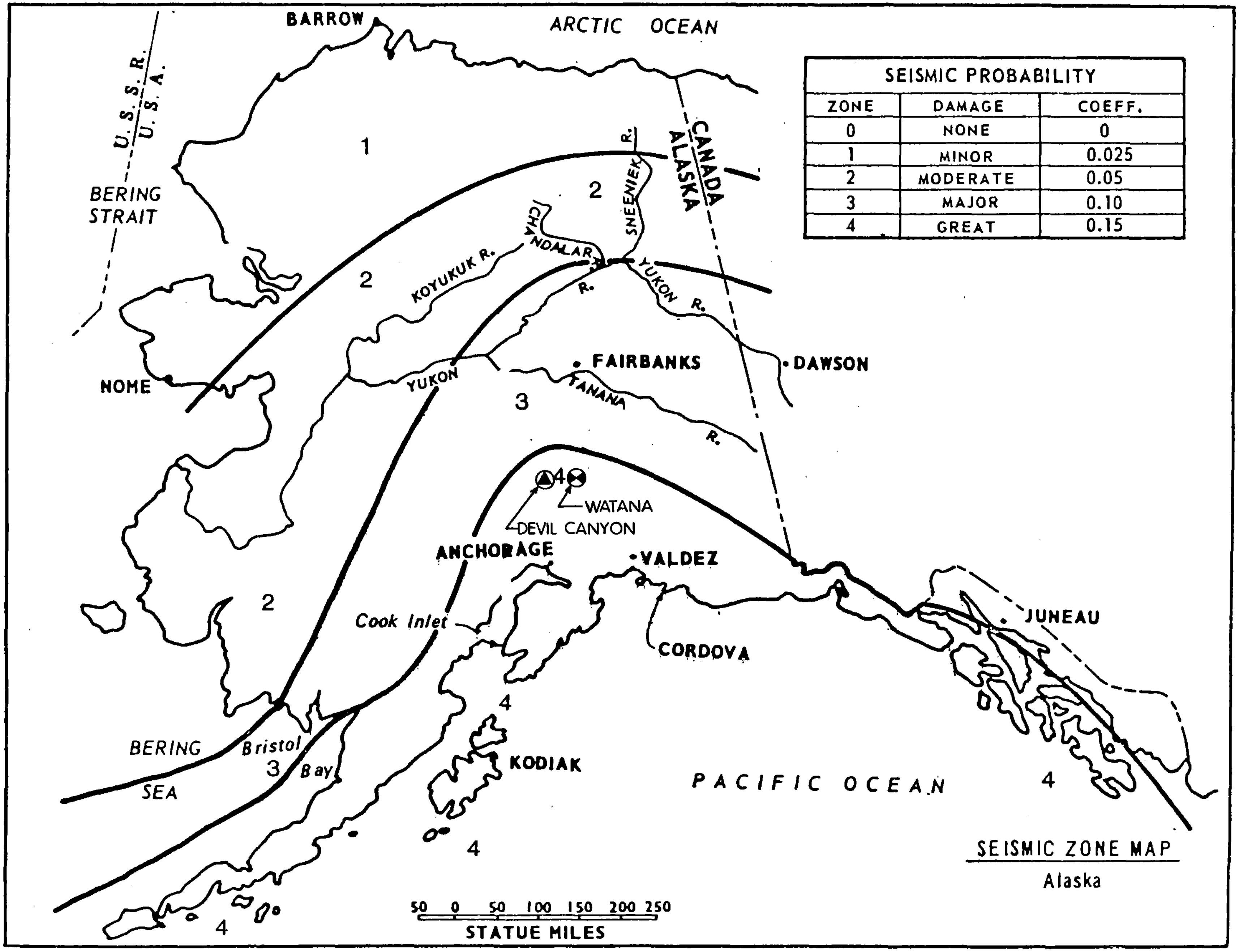
MATERIAL		SHEAR STRENGTH USED IN ANALYSIS
IMPERVIOUS CORE	①	CU: C=300 psf, Ø=16.7°
ROCKFILL AND FILTERS	②,③,④	CD: C=0 psf, Ø=38°
OVERBURDEN FDN.	⑤	CD: C=0 psf, Ø=32°
BEDROCK FDN.	⑥	CD: C= 40,000 psf, Ø=38°

NOTE

MATERIAL DESIGN PARAMETERS ARE DISCUSSED IN SECTION 1.2

RAPID DRAWDOWN CASE
(UPSTREAM SLOPE)

FIGURE F2.13



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

SEISMIC ZONE MAP
Alaska

SOURCE: COE 1982

FIGURE F2.14

APPENDIX F3

SUMMARY AND PMF AND SPILLWAY DESIGN FLOOD ANALYSES

APPENDIX F3
SUMMARY OF PMF AND SPILLWAY DESIGN FLOOD ANALYSES

1 - INTRODUCTION (**)

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

2 - PROBABLE MAXIMUM FLOOD (PMF)

2.1 - Calibration of SSARR Model (o)

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

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

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

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

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

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

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

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

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

(a) Storm Isohyetal Pattern (**)

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

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

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

(b) Storm Maximization (**)

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

MAXIMIZATION FACTORS

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

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

MAXIMIZED PRECIPITATION

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

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

(c) Temporal Precipitation Pattern (**)

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

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

TEMPORAL PATTERN OF PMP

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

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

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

2.3 - Snowmelt Criteria (o)

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

6-HOURLY DISTRIBUTION PATTERN

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

INITIAL SNOWPACK FOR PMF

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

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

2.4 - Occurrence of Snowmelt and PMP Storm (o)

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

2.5 - Antecedent Conditions (**)

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

2.6 - PMF (***)

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

INITIAL VALUES OF SSARR MODEL PARAMETERS

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

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

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

Additionally, the same publication goes on to state that:

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

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

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

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

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

2.7 - Design Floods

(This section deleted)

TABLES

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

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

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

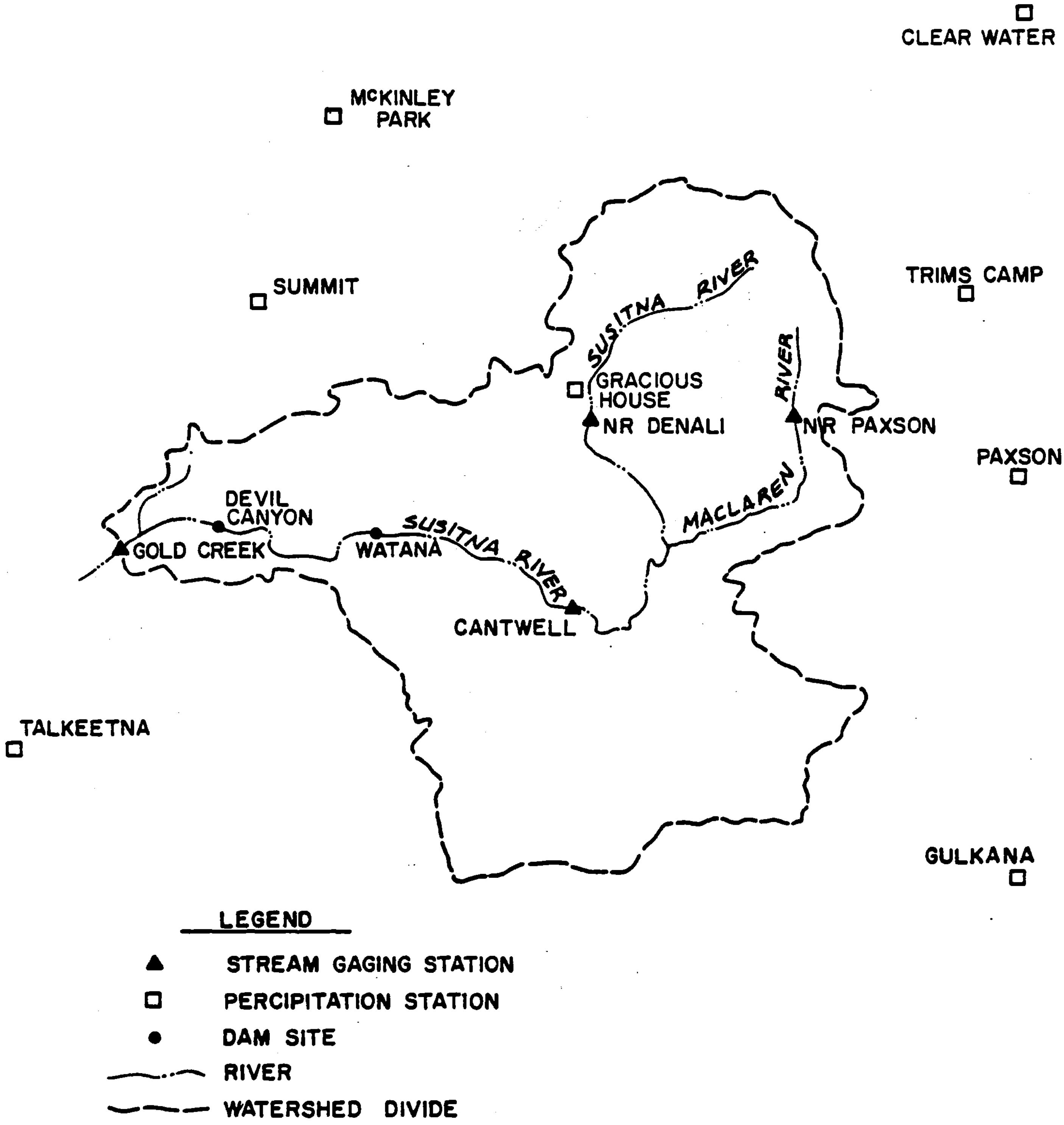
Sub-basin Identification Number	10	20	80	180	210	220	280	330	340	380	480	580	680
Drainage area, mi^2	221	694	312	477	44	232	307	48	1047	735	1045	628	345
Number of Surface Routing Phases	4	4	4	4	3	4	4	3	8	3	4	4	4
Surface Storage Time (hr)	6	8	3	3	6	5	3	15	10	3	8	8	8
Number of Sub-Surface Routing Phases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

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

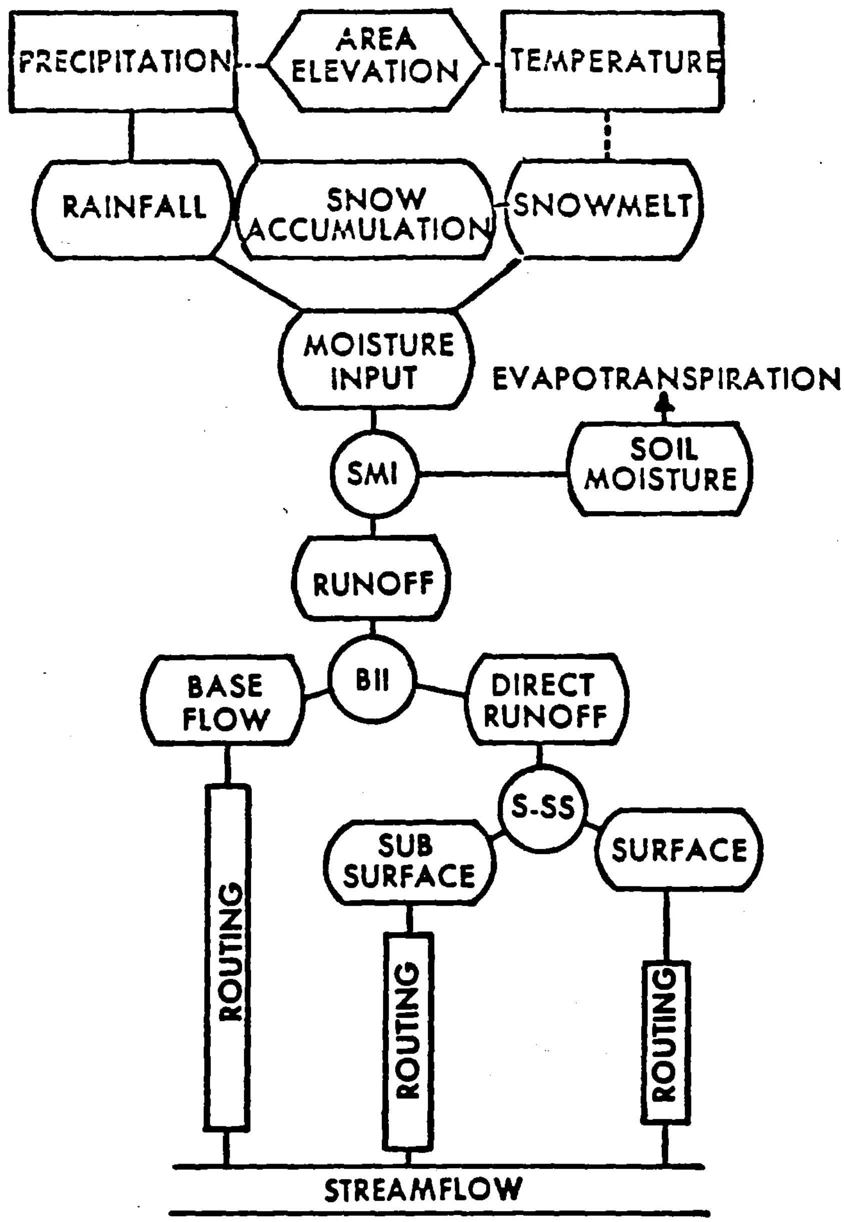
Sub-basin 10										
Elevation, ft	2800	3000	4000	5000	6000	7000	8000	9000	13,820	
Percent area below	0	4.5	17.7	35.9	61.1	84.8	96.1	99.8	99.9	
Sub-basin 20										
Elevation, ft	2440	3000	4000	5000	6000	7000	8000	9000	10,000	13,820
Percent area below	0	27.7	53.2	81.3	92.8	97.1	98.4	98.9	99.8	99.9
Sub Basin 80										
Elevation, ft	2370	3000	4000	5000	6000	6100				
	0	35.9	74.4	97.1	99.7	99.9				
Sub-basin 180										
Elevation, ft	2350	3000	4000	5000	6000	6100				
Percent area below	0	35.0	82.0	96.4	96.5	99.9				
Sub-basin 210										
Elevation, ft	3150	4000	5000	6000	7000	8000	8850			
Percent area below	0	10.9	24.1	67.2	96.0	99.8	99.9			
Sub-basin 220										
Elevation, ft	2860	3000	4000	5000	6000	7000	8000	8850		
Percent area below	0	8.2	50.5	80.1	94.9	98.6	99.8	99.9		
Sub-basin 280										
Elevation, ft	2350	3000	4000	5000	5275					
Percent area below	0	49.8	96.7	96.8	99.9					
Sub-basin 330										
Elevation, ft	2361	2363								
Percent area below	0	99.9								
Sub-basin 340										
Elevation, ft	2100	3000	4000	5000	5275					
Percent area below	0	68.7	95.2	99.8	99.9					
Sub-basin 380										
Elevation, ft	1910	2000	3000	4000	5000	6000	7000	7770		
Percent area below	0	2.0	15.6	49.1	78.4	96.0	99.8	99.9		
Sub-basin 480										
Elevation, ft	1450	2000	3000	4000	5000	6000	7000	7200		
Percent area below	0	3.0	27.7	68.3	91.1	98.9	99.8	99.9		
Sub-basin 580										
Elevation, ft	910	1000	2000	3000	4000	5000	6000	6910		
Percent area below	0	2.0	8.4	44.1	79.5	96.2	99.8	99.9		
Sub-basin 680										
Elevation, ft	677	1000	2000	3000	4000	5000	6000	6018		
Percent area below	0	3.2	26.1	51.0	80.9	97.1	99.8	99.9		

FIGURES

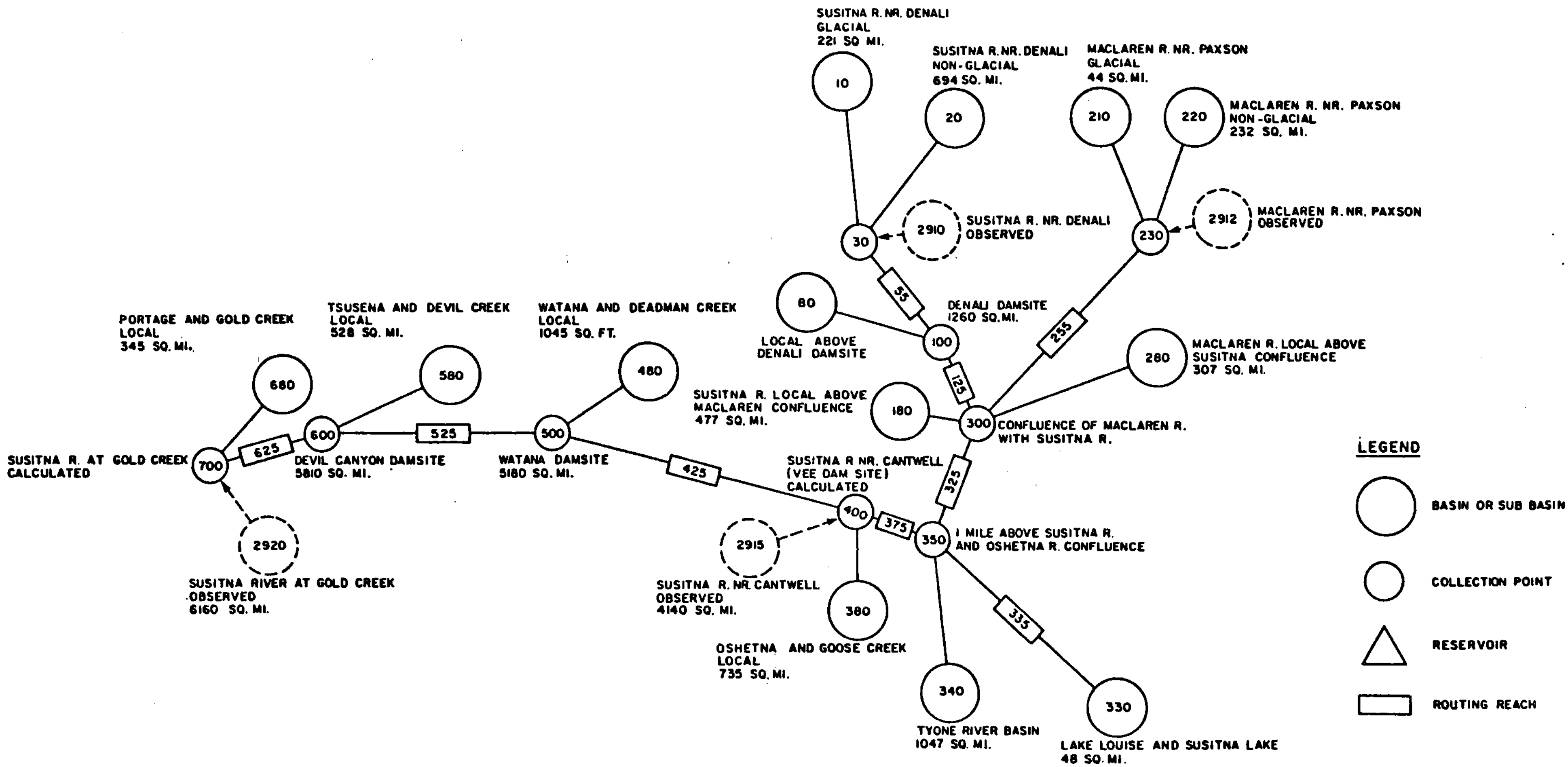
Figure F3.1



SUSITNA RIVER BASIN
ABOVE GOLD CREEK



SSARR WATERSHED MODEL

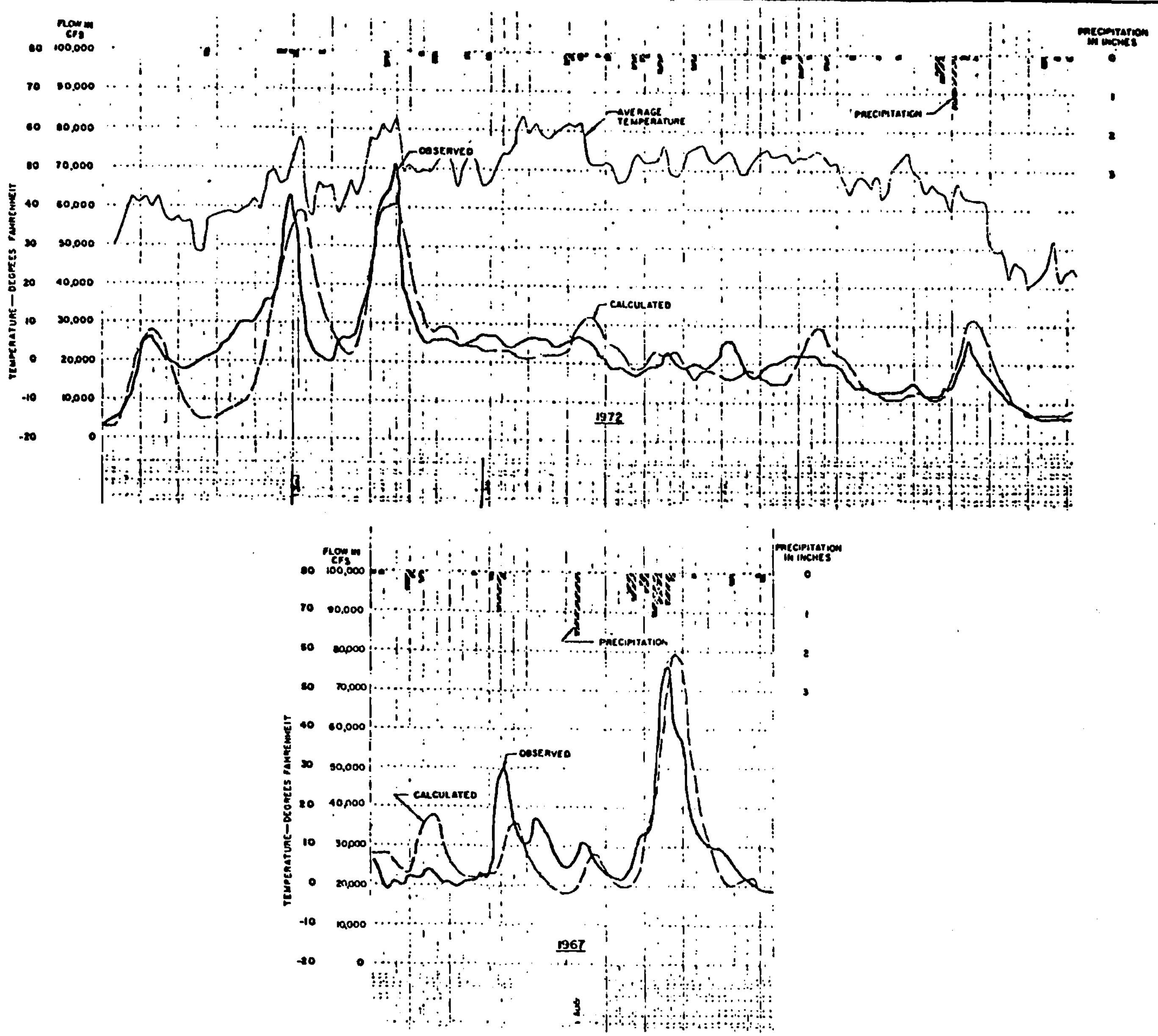


SCHEMATIC DIAGRAM OF SSARR COMPUTER MODEL

REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY
REPORT, 1975 APPENDIX 1 PART 1

Figure F3.3

ACRES

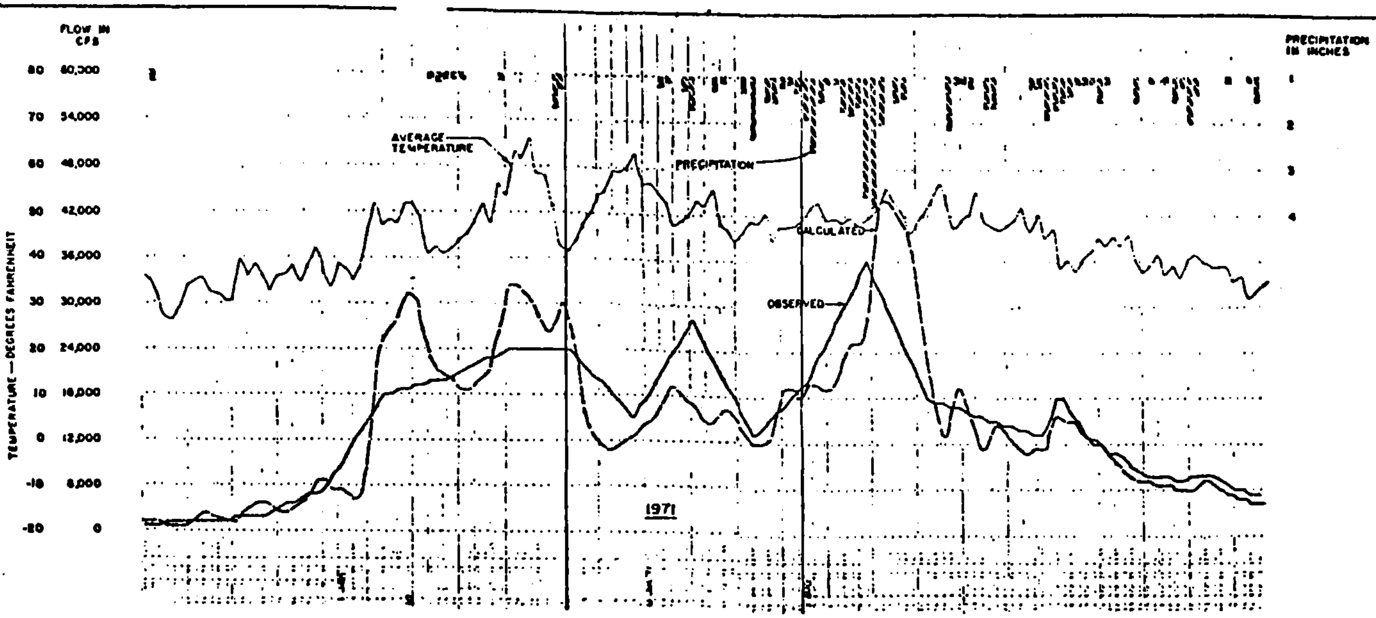


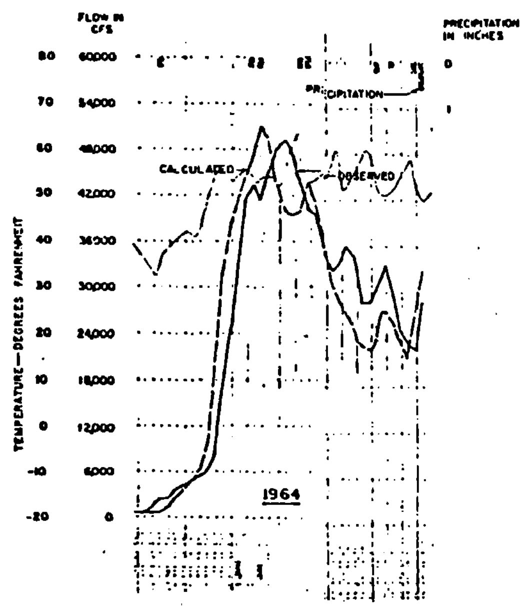
REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH : SUSITNA RIVER AT GOLD CREEK, 1967, 1972

Figure F3.4

ACRES



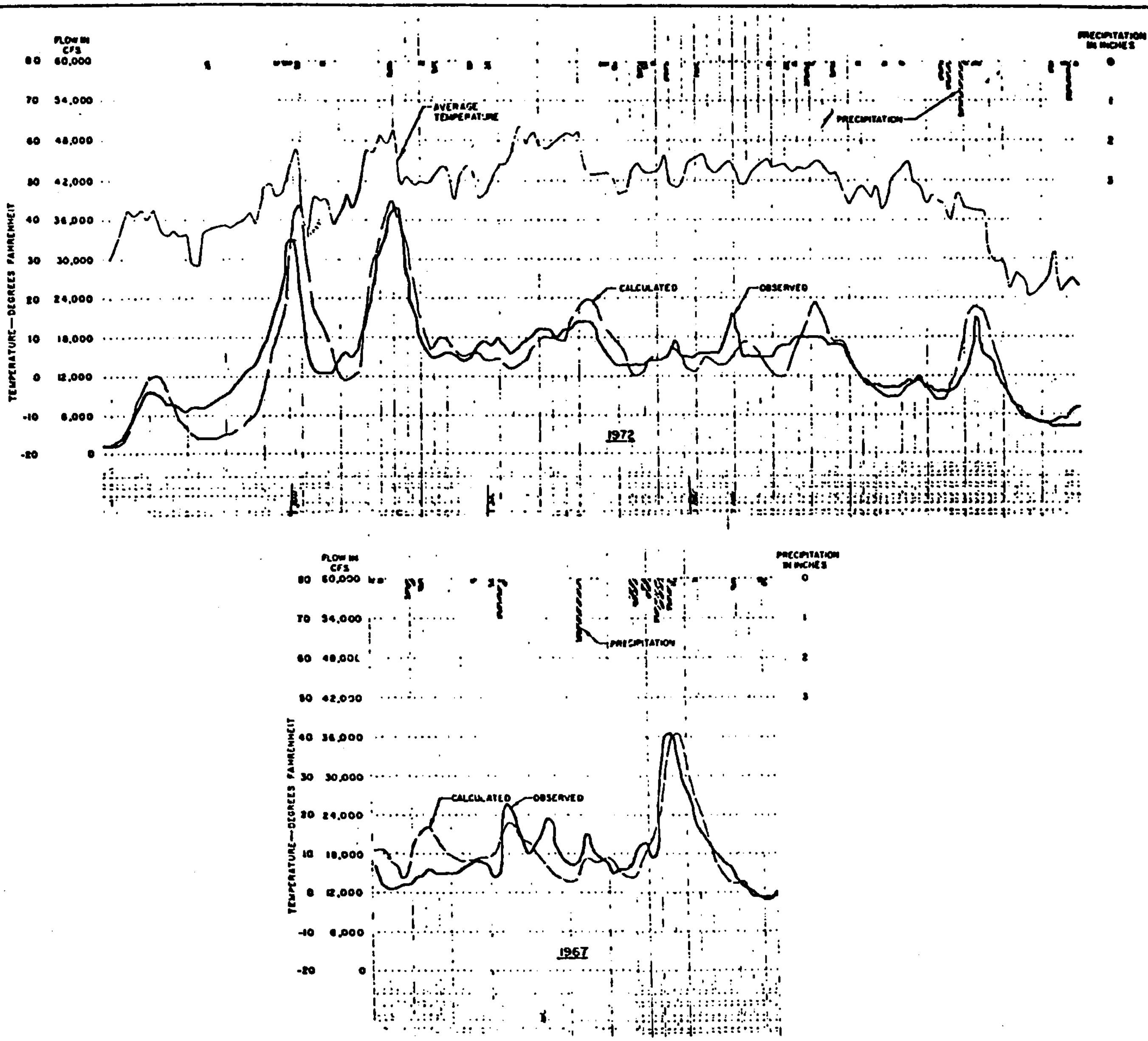


REFERENCE.
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH : SUSITNA RIVER NEAR CANTWELL, 1964, 1971

Figure F3.5

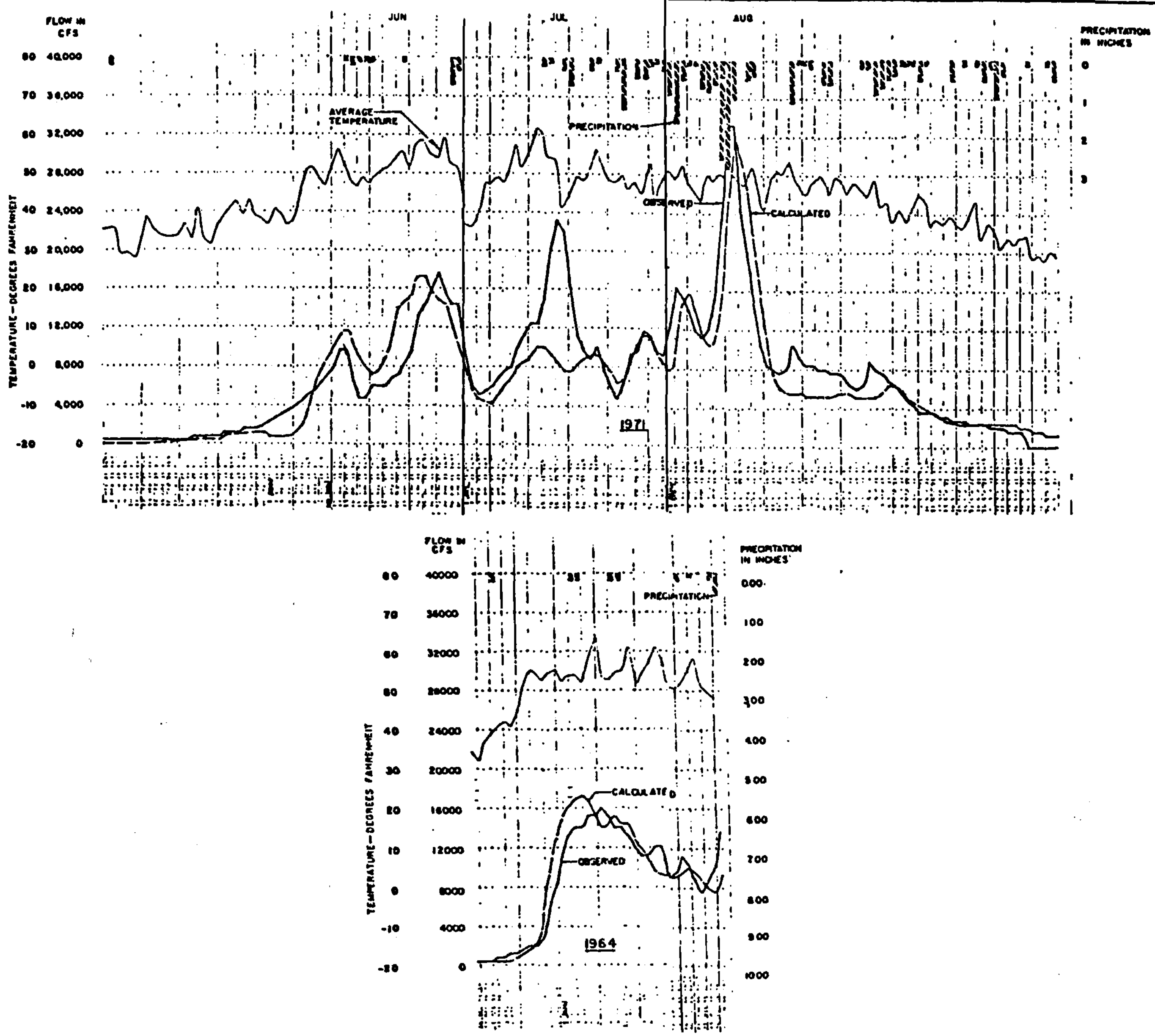
ACRES



HYDROGRAPH : SUSITNA RIVER NEAR CANTWELL, 1967, 1972

Figure F3.6

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

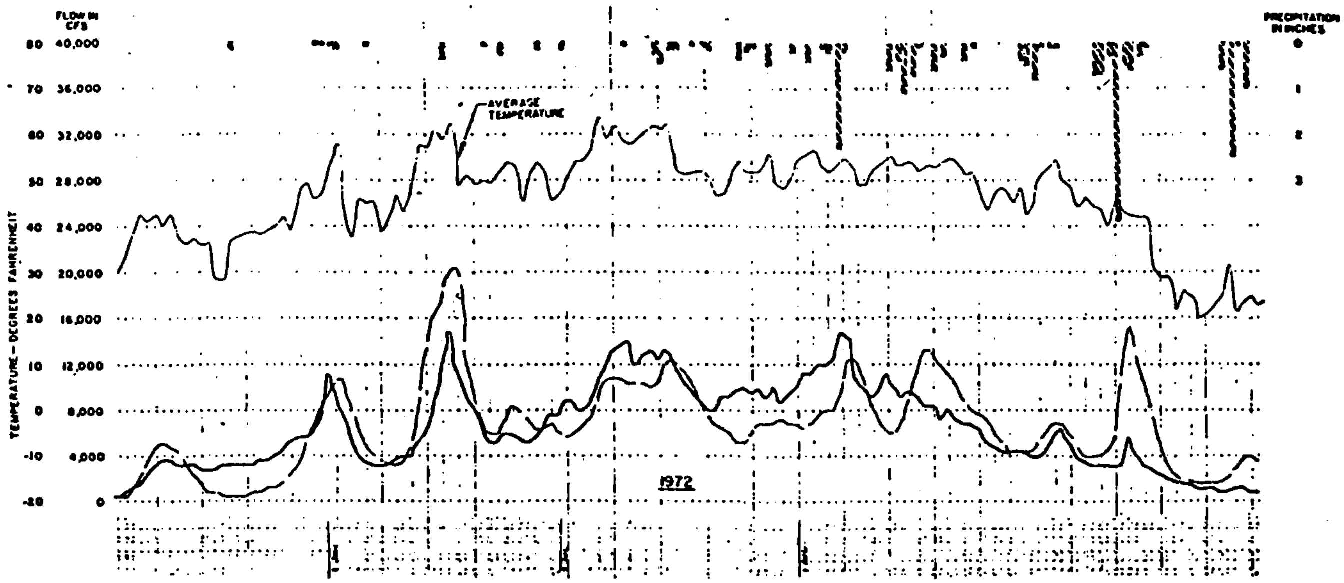


REFERENCE.
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH SUSITNA RIVER NEAR DENALI, 1964, 1971

Figure F3.7

ACRES

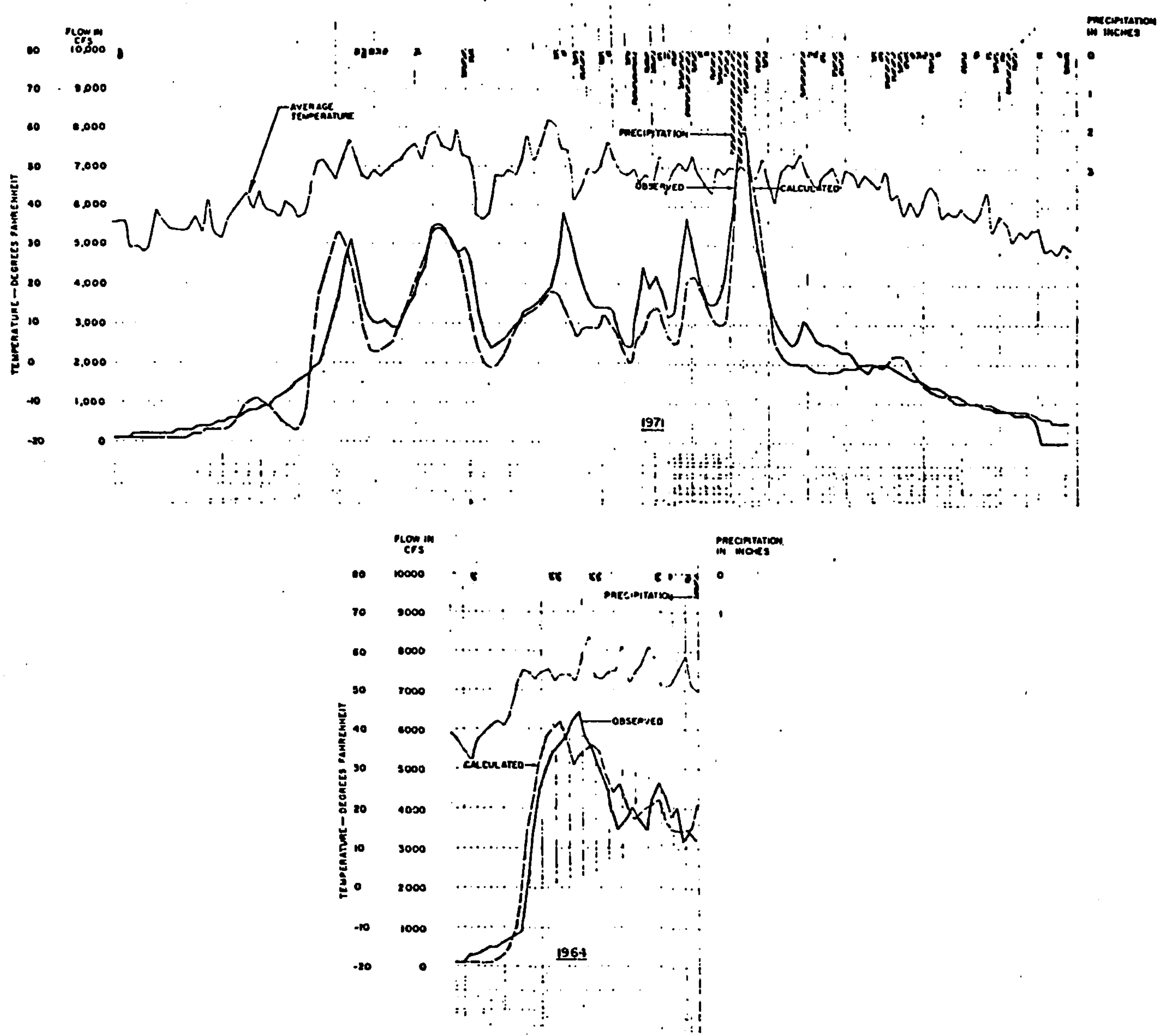


REFERENCE
U S ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH SUSITNA RIVER NEAR DENALI, 1972

Figure F3.8

ACRES

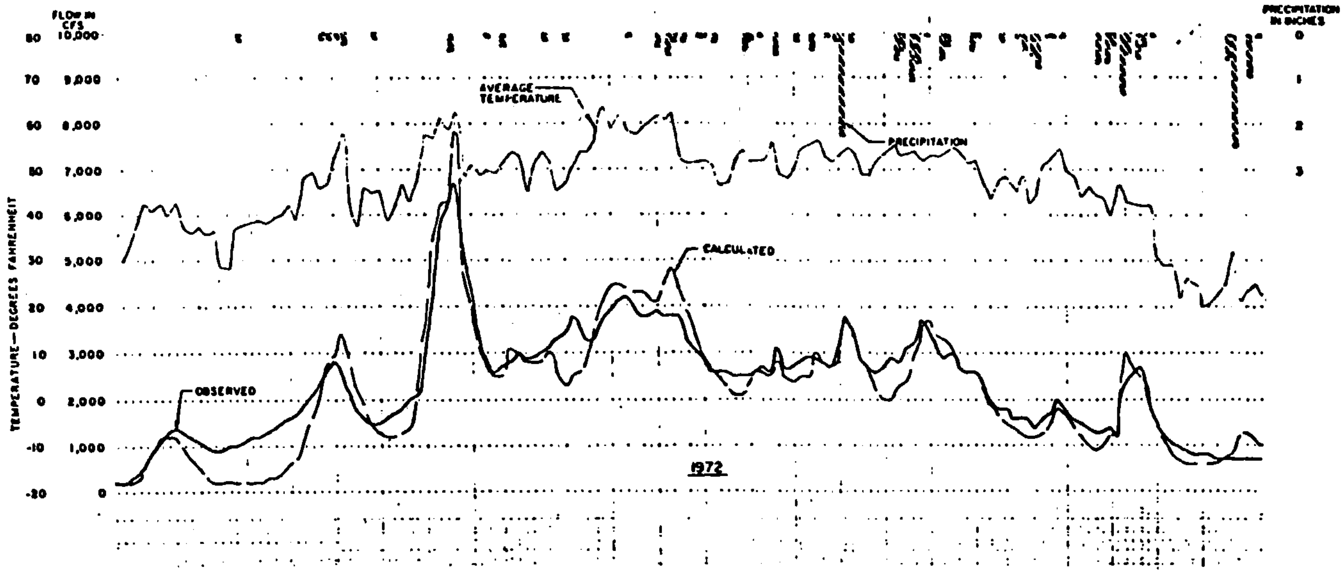


REFERENCE:
U.S. ARMY CORPS OF ENGINEERS INTERIM FEASIBILITY REPORT, 1975 APPENDIX I PART I

HYDROGRAPH: MACLAREN RIVER NEAR PAXSON, 1964, 1971

Figure F3.9

ACRES



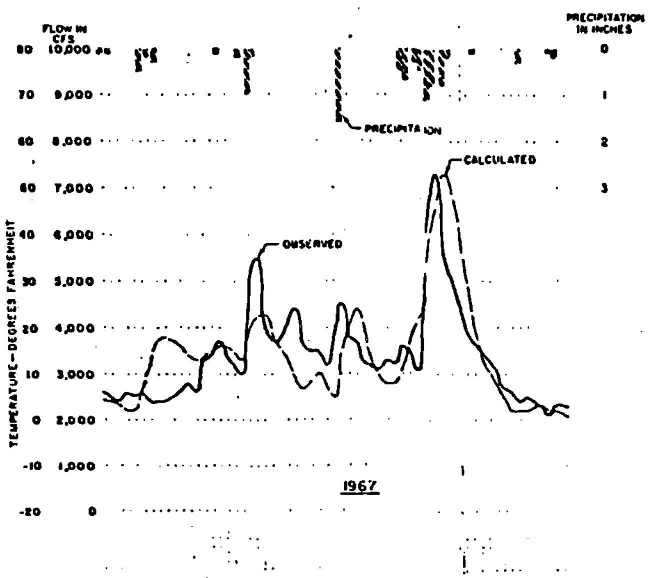
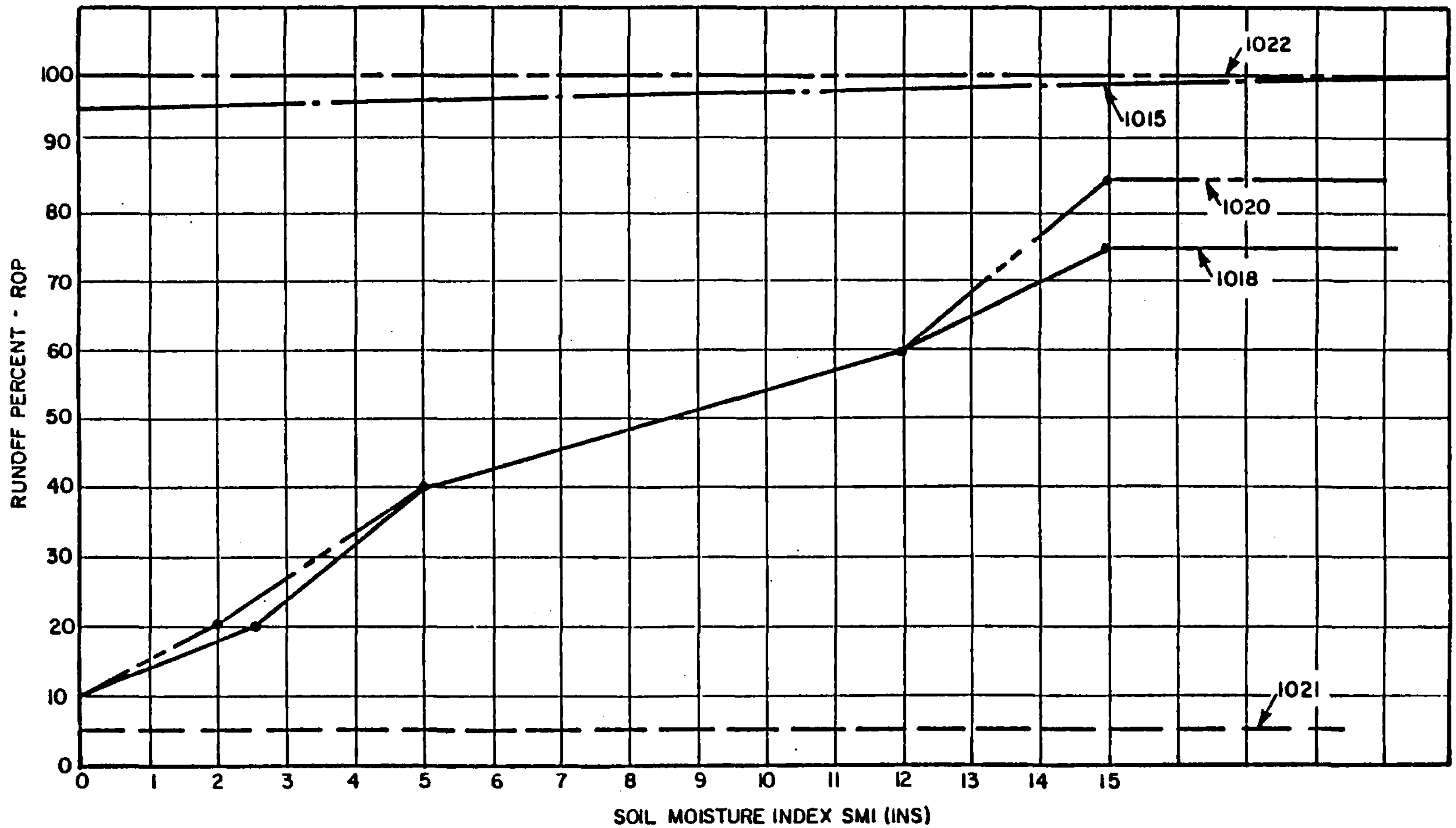


Figure F3.10

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

HYDROGRAPH MACLAREN RIVER NEAR PAXSON, 1967, 1972

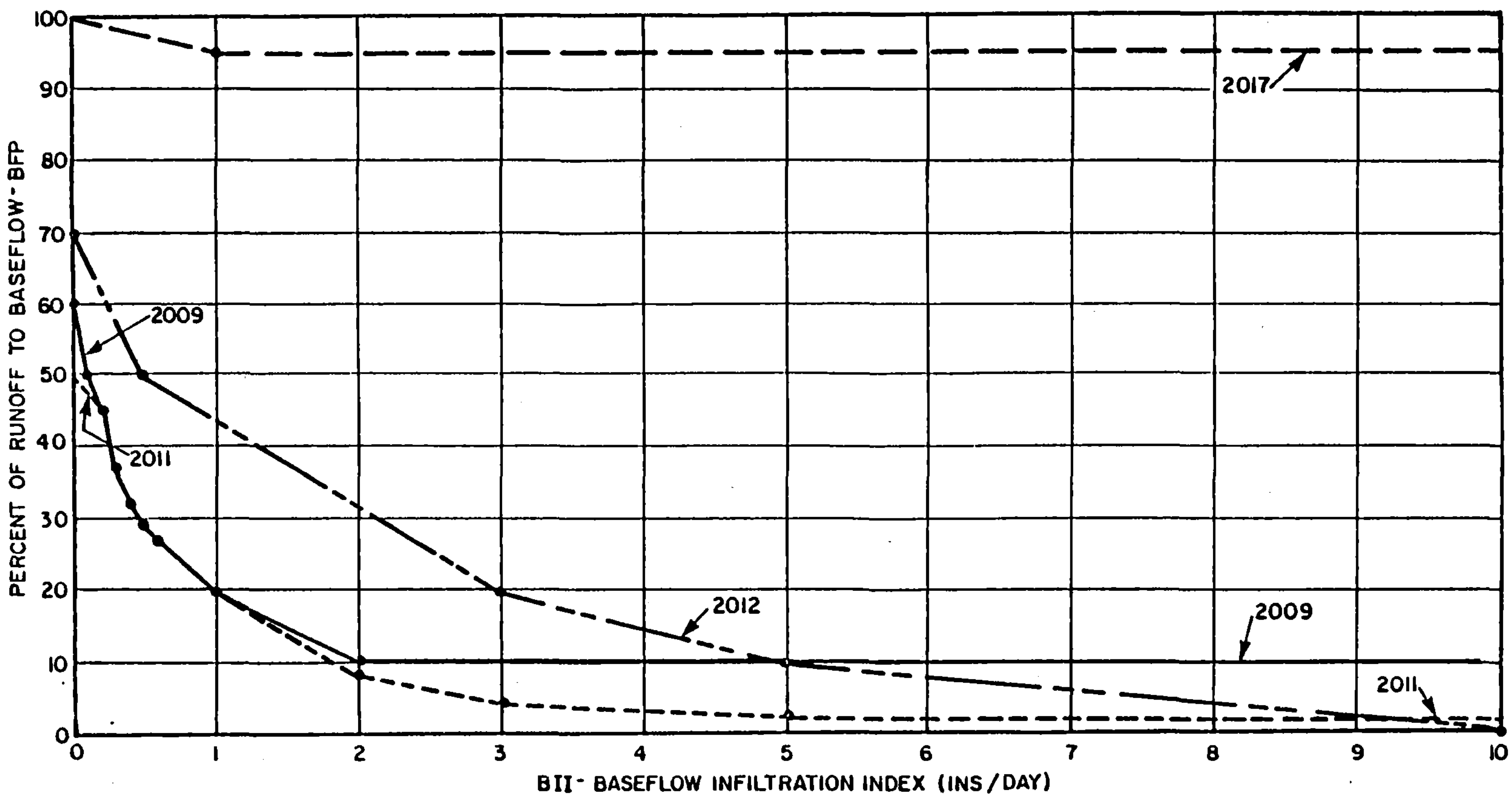
ACRES



SSARR MODEL SMI VS ROP

Figure F3.11

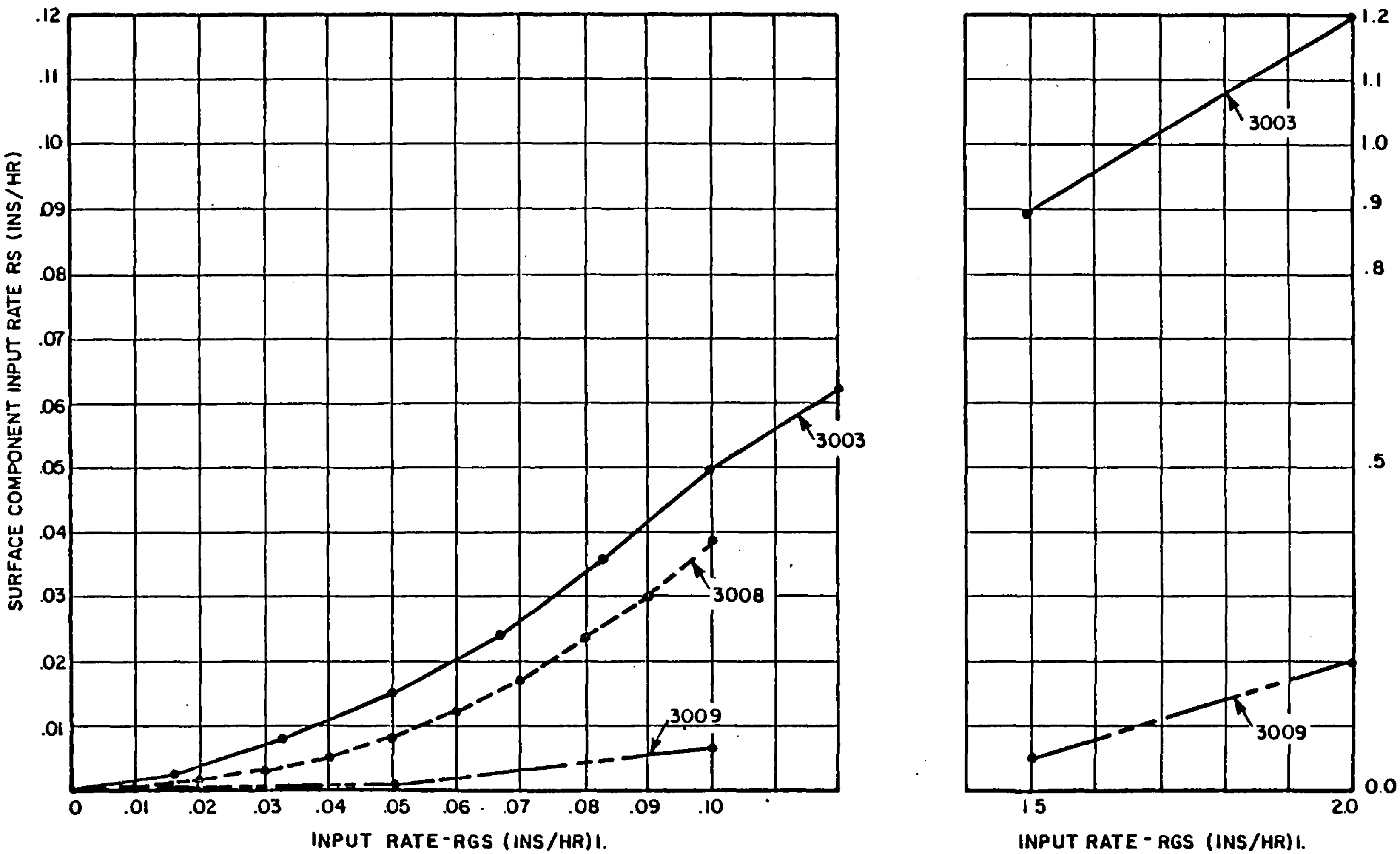
ACRES



SSARR MODEL BII VS BFP

Figure F3.12

ACRES



SSARR MODEL RGS VS RS

Figure F3.13

ACRES

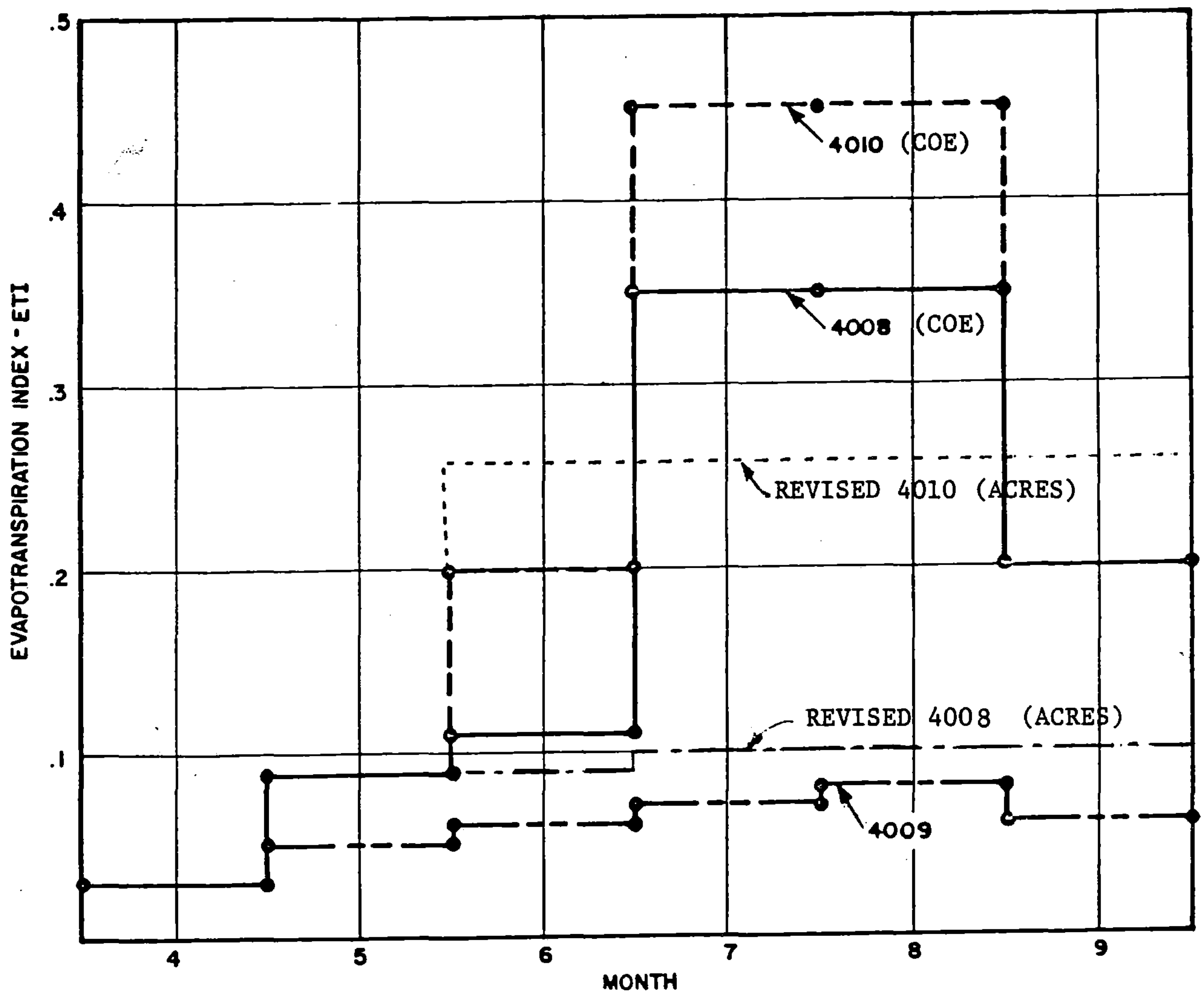


Figure F3.14

SSARR MODEL MONTH VS ETI

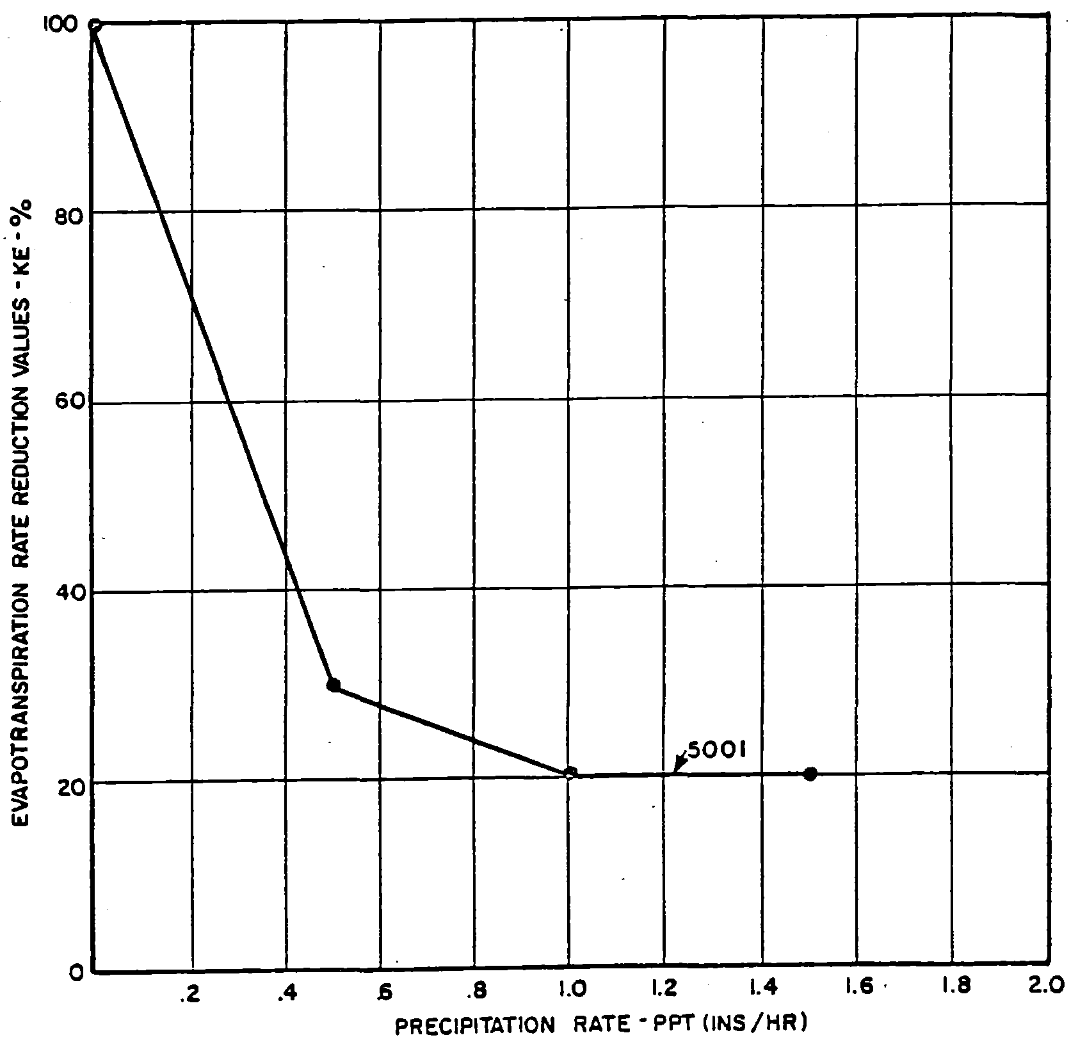
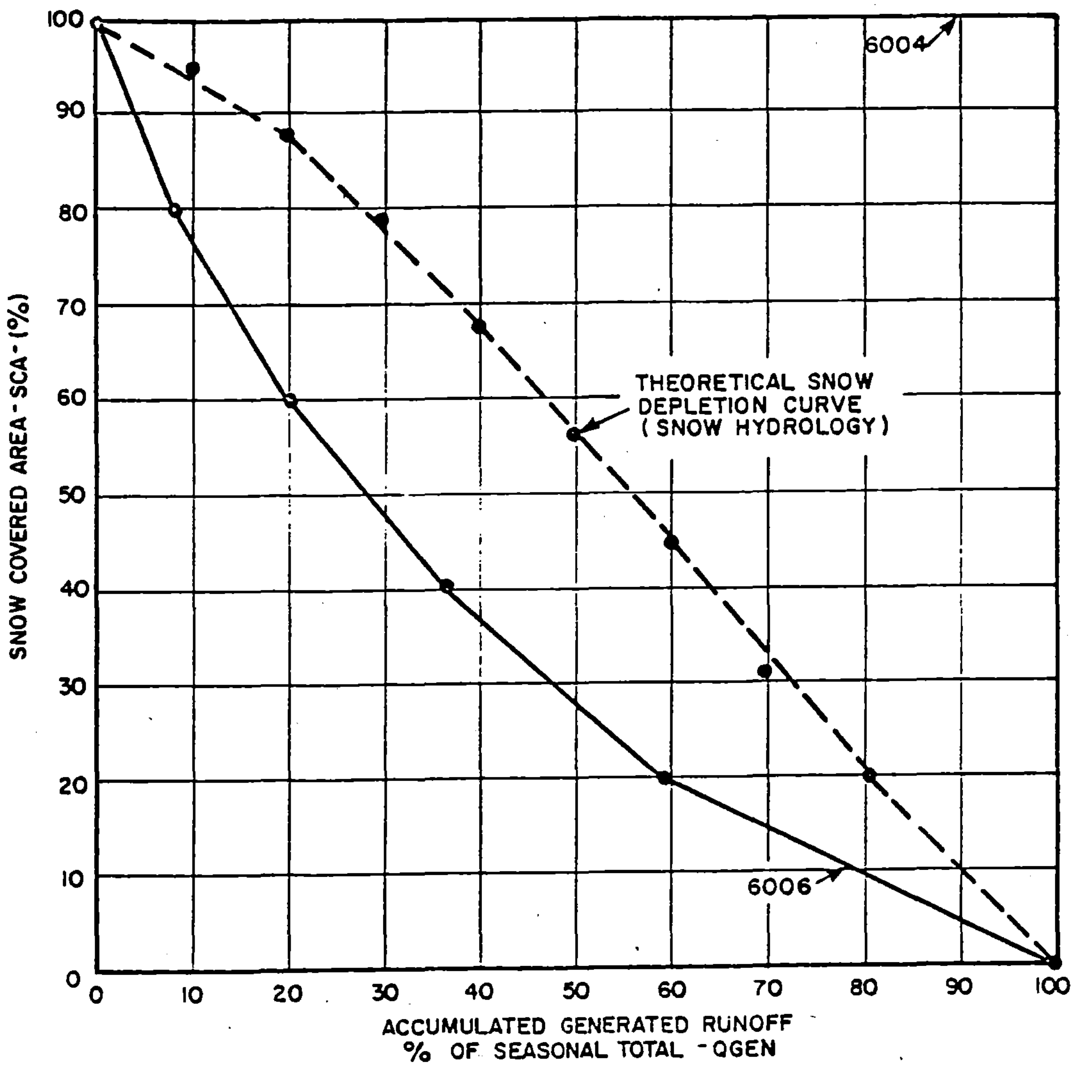


Figure F3.15

SSARR MODEL PPT VS KE

ACRES



SSARR MODEL QGEN VS SCA

Figure F3.16

ACRES

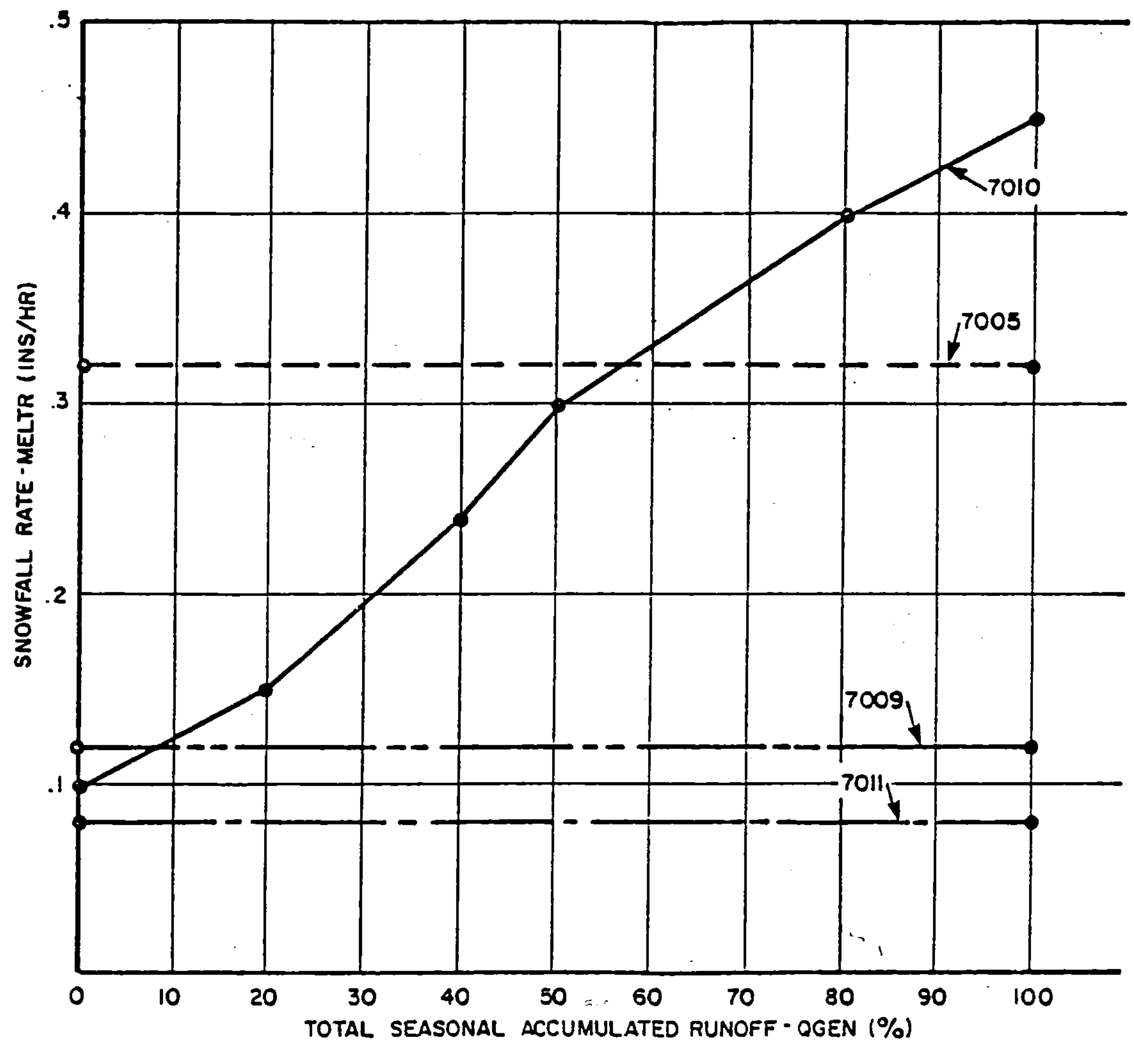
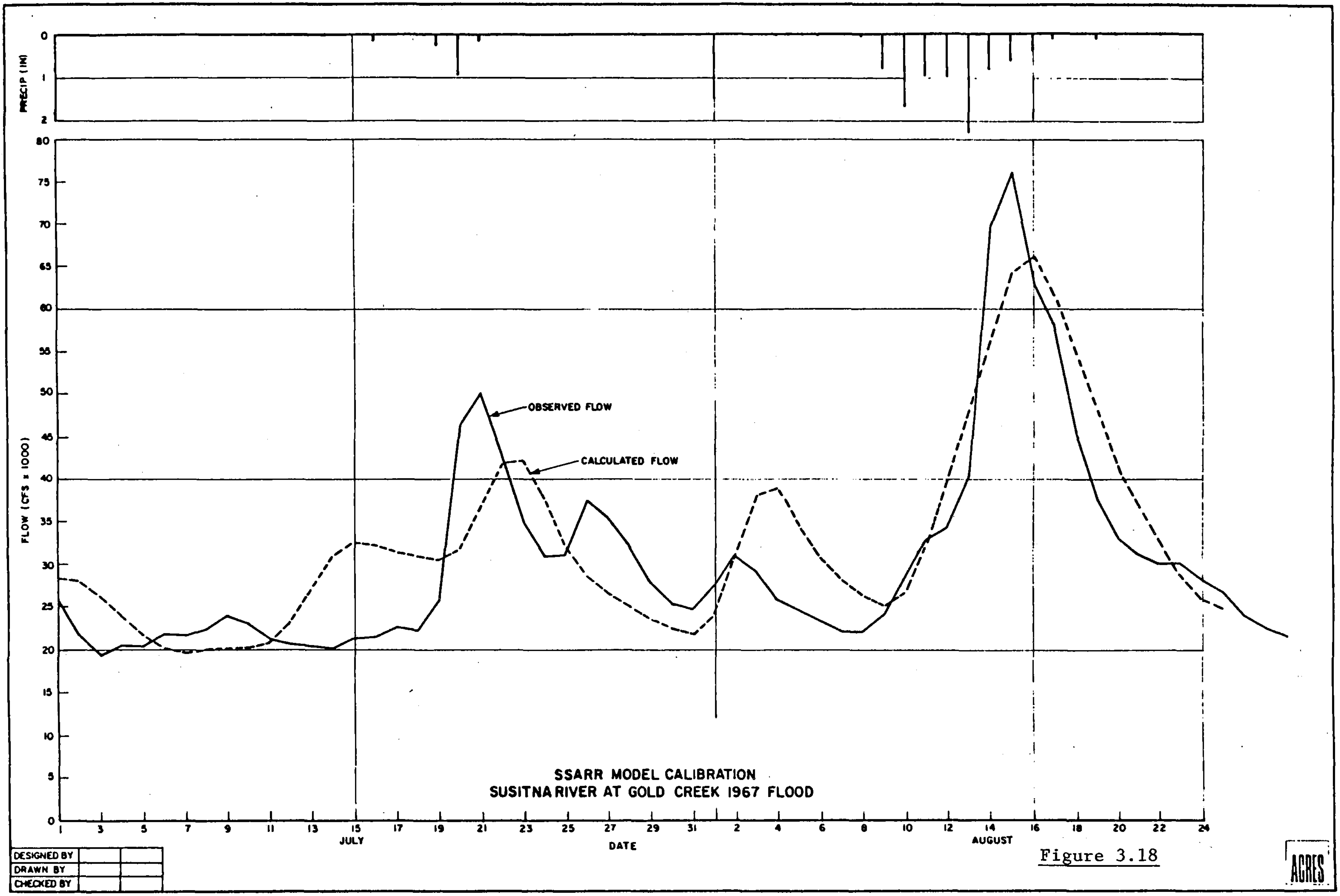


Figure F3.17

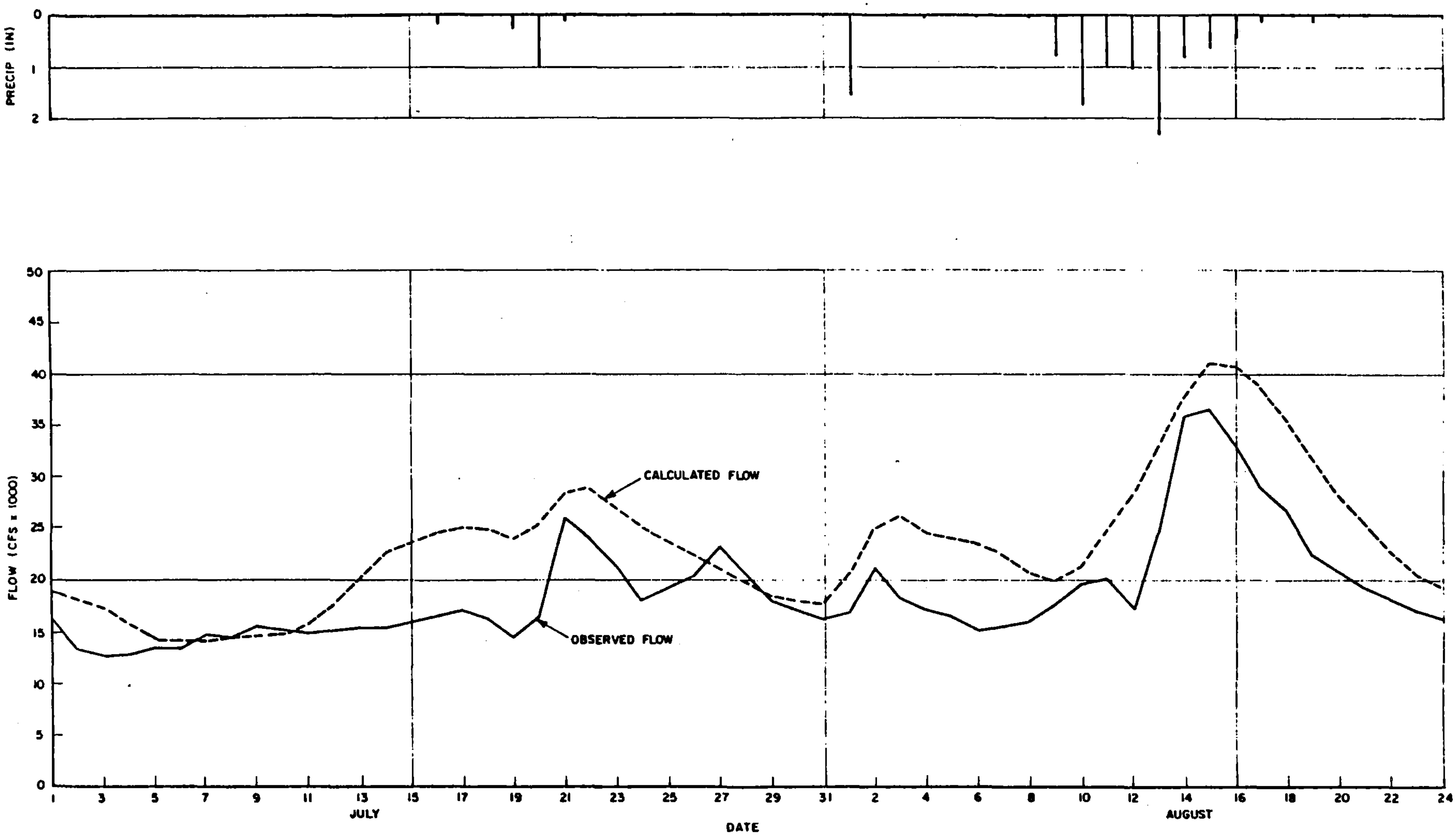
SSARR MODEL QGEN VS MELTR

ACRES



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

Figure 3.18

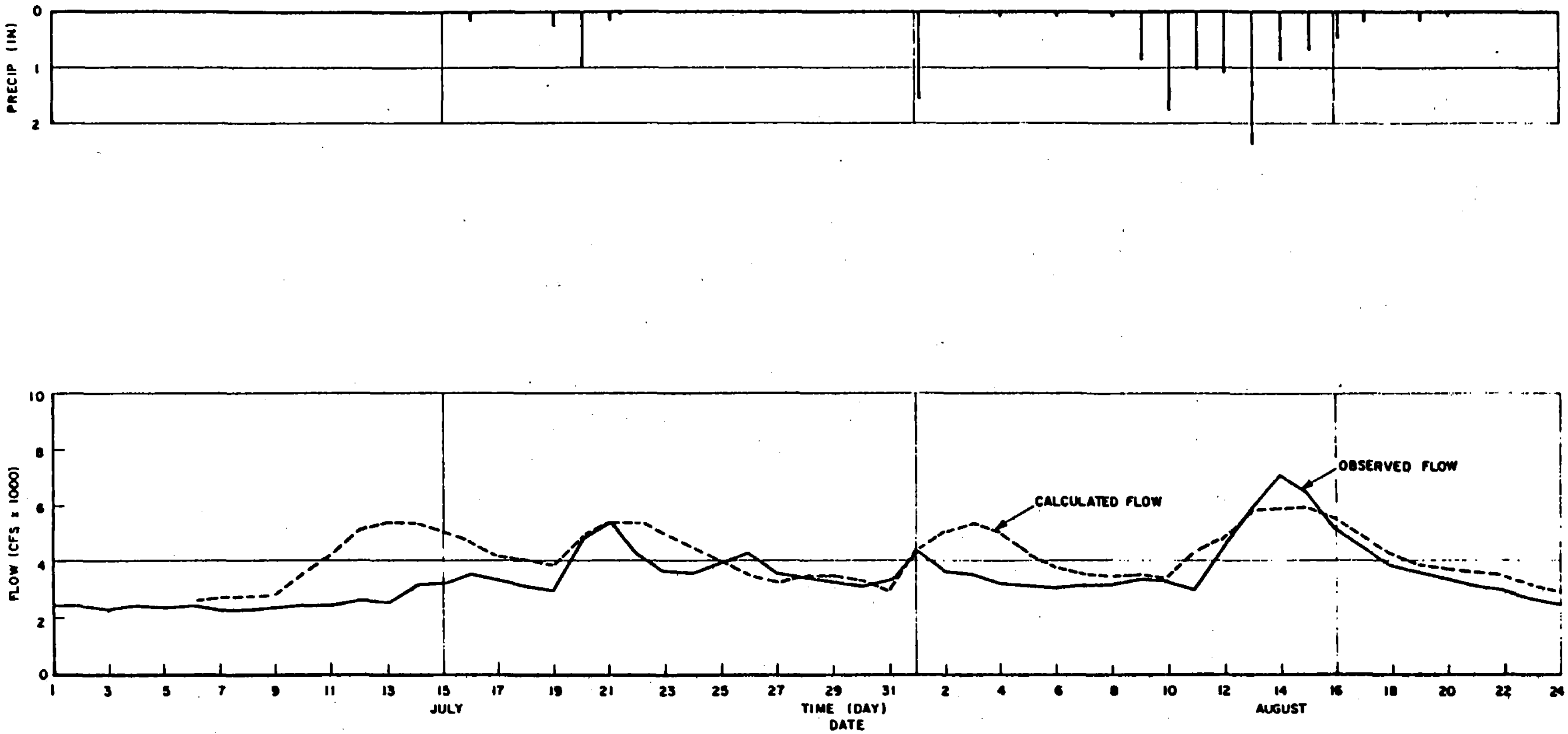


DESIGNED BY		
DRAWN BY		
CHECKED BY		

SSARR MODEL CALIBRATION
SUSITNA RIVER NEAR CANTWELL 1967 FLOOD

Figure F3.19

ACRES



DESIGNED BY		
DRAWN BY		
CHECKED BY		

SSARR MODEL CALIBRATION
MACLAREN RIVER NEAR PAXSON 1967 FLOOD

Figure F3.20

ACRES

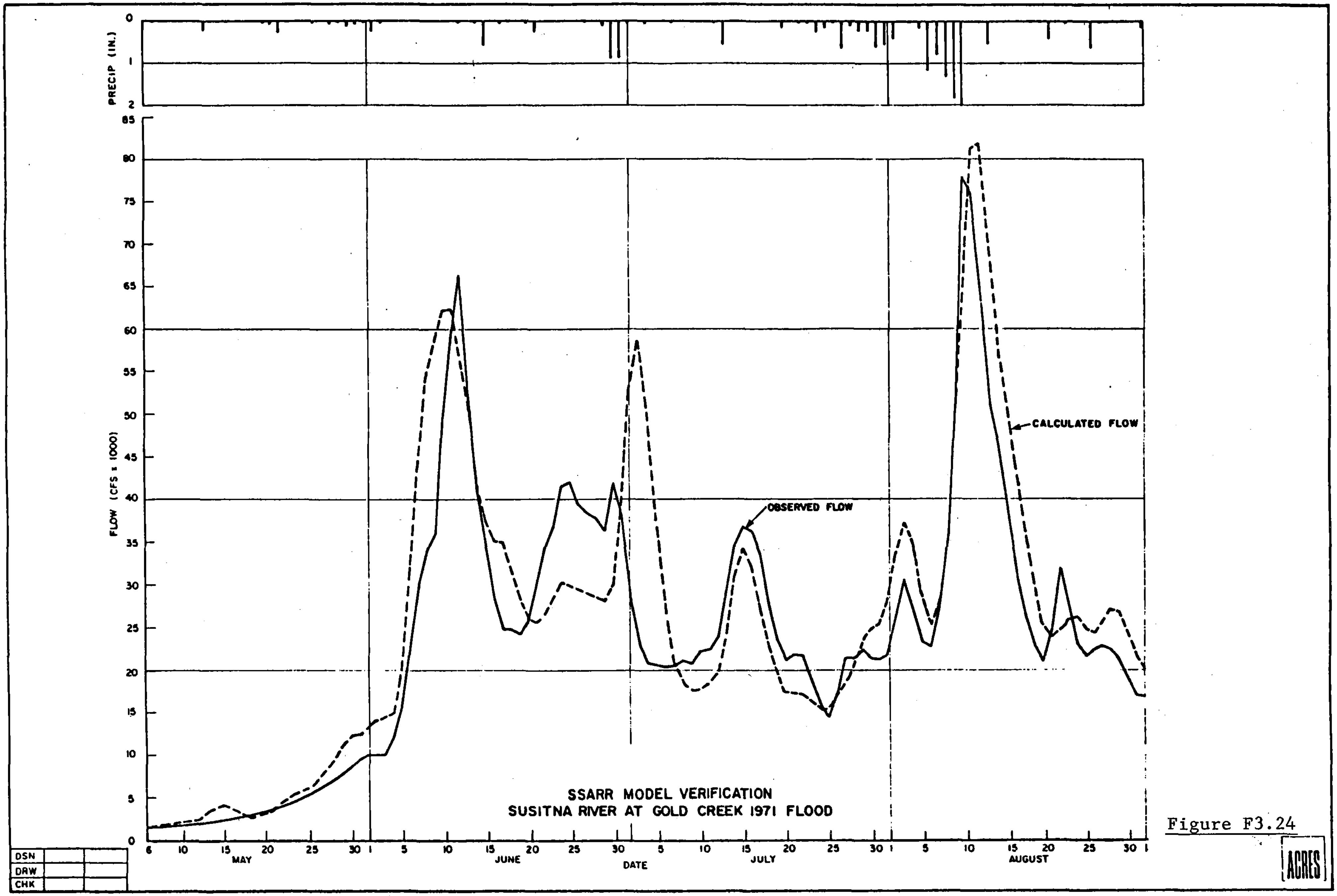
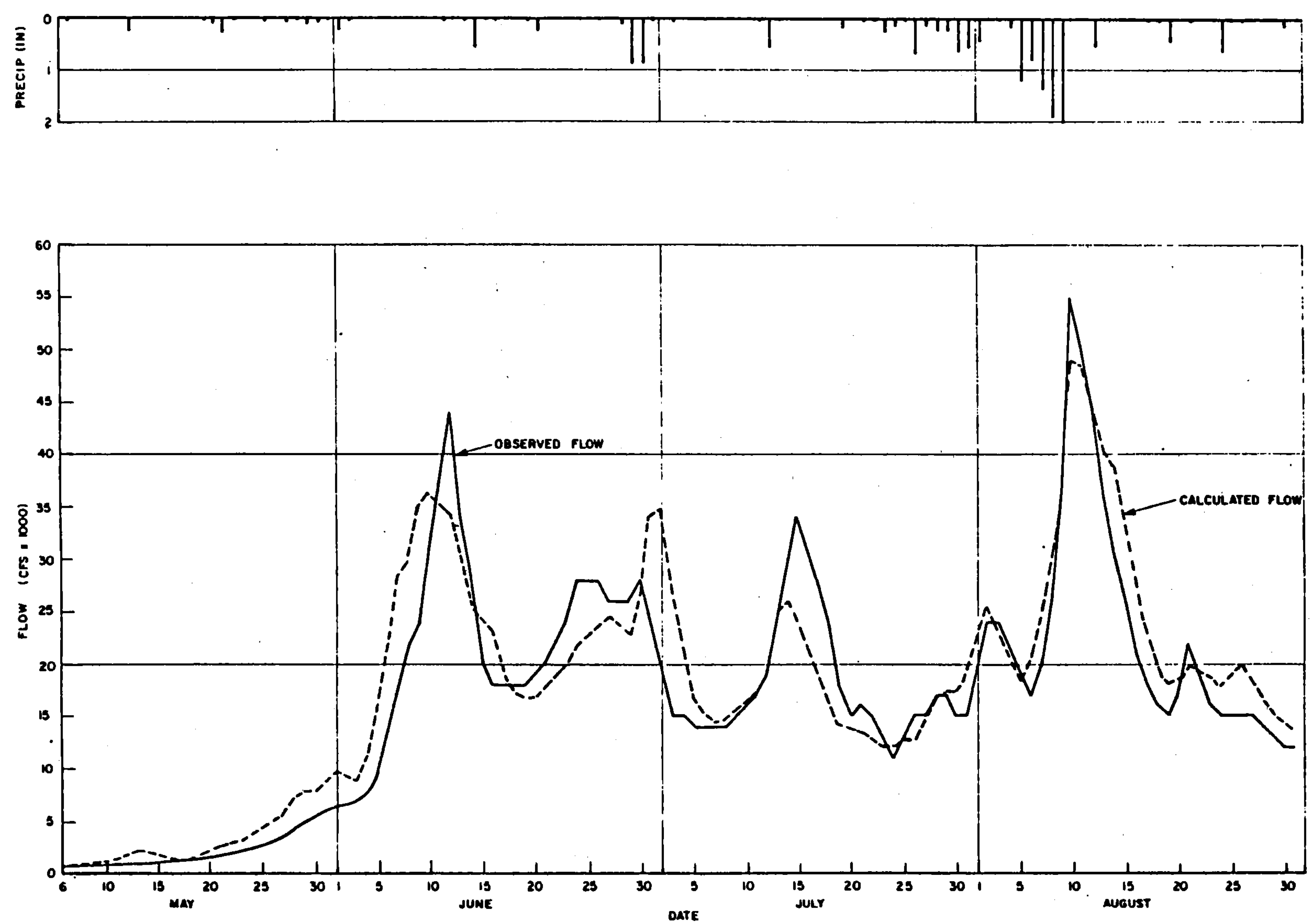


Figure F3.24

ACRES

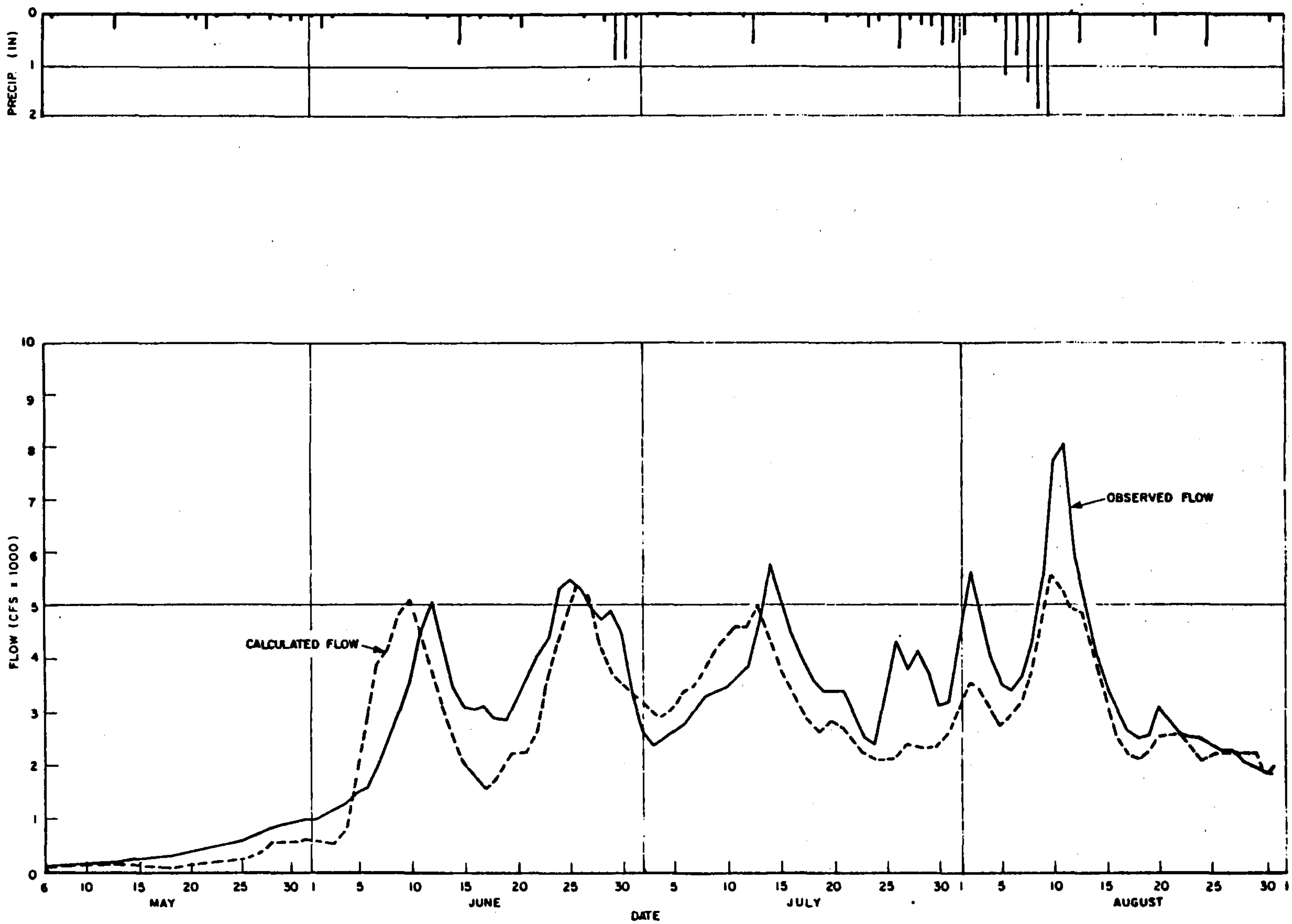


DESIGNED BY		
DRAWN BY		
CHECKED BY		

SSARR MODEL VERIFICATION
SUSITNA RIVER NEAR CANTWELL 1971 FLOOD

Figure F3.25

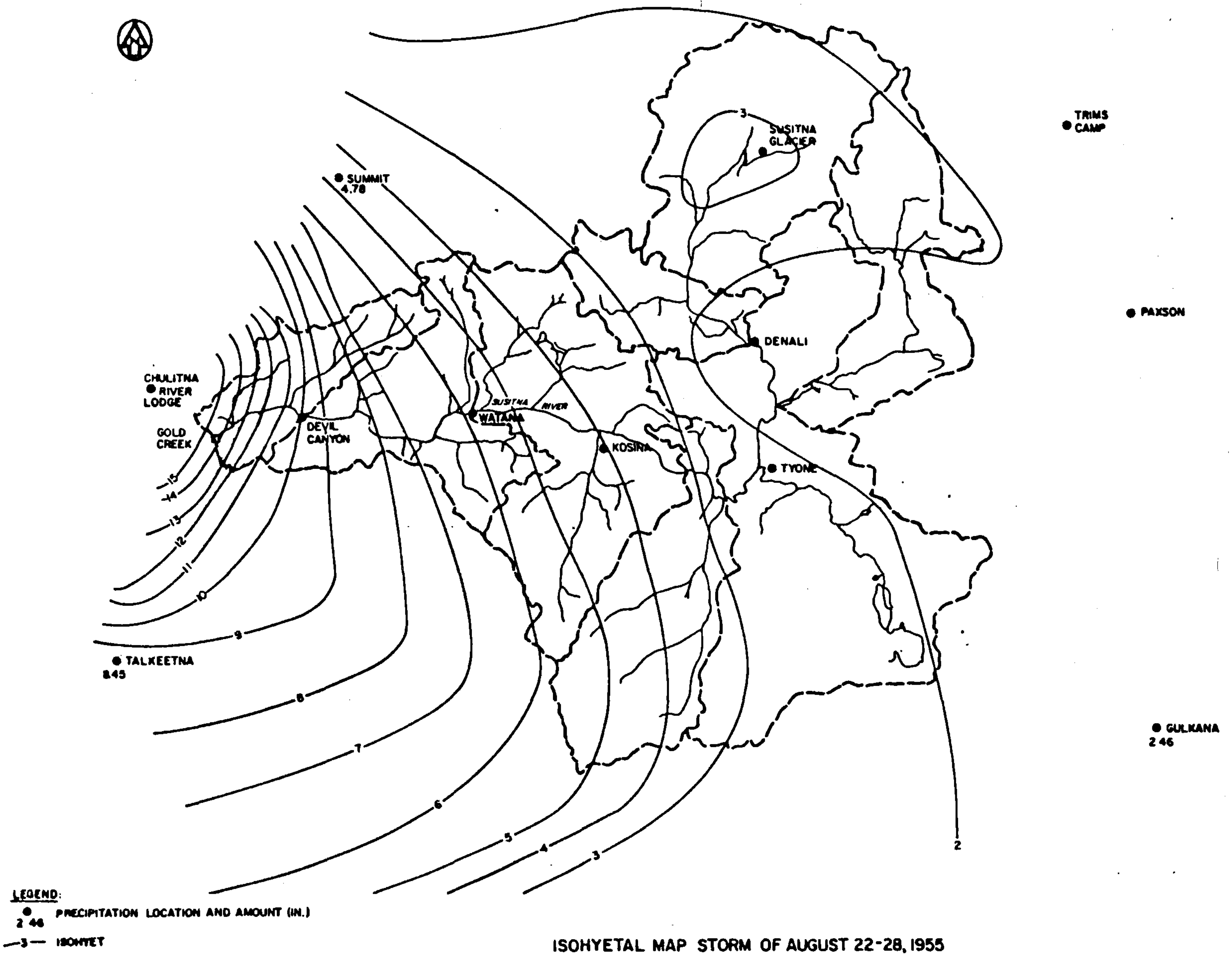
ACRES



DESIGNED BY		
DRAWN BY		
CHECKED BY		

SSARR MODEL VERIFICATION
MACLAREN RIVER NEAR PAXSON 1971 FLOOD

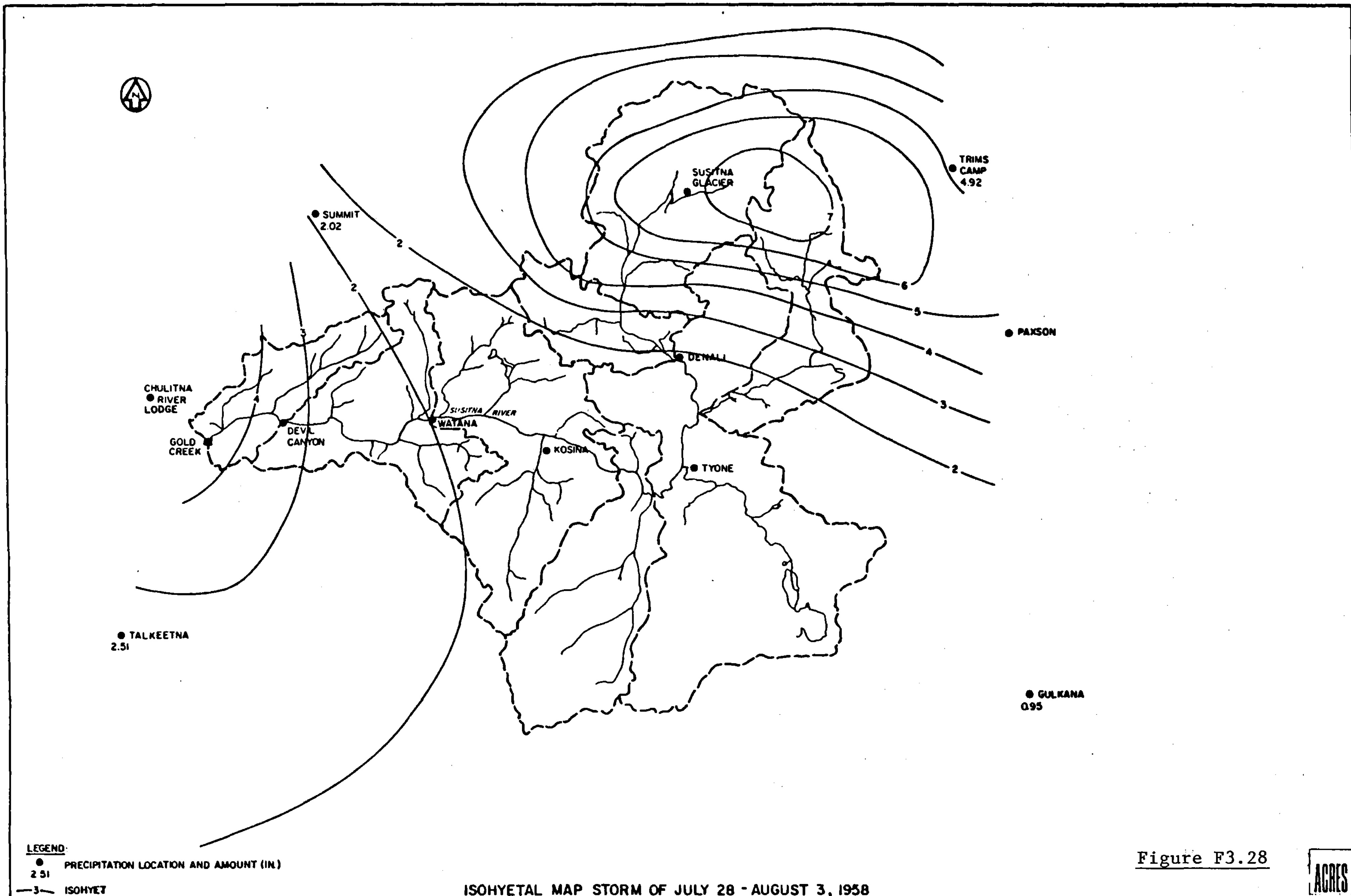
Figure F3.26



ISOHYETAL MAP STORM OF AUGUST 22-28, 1955

Figure F3.27

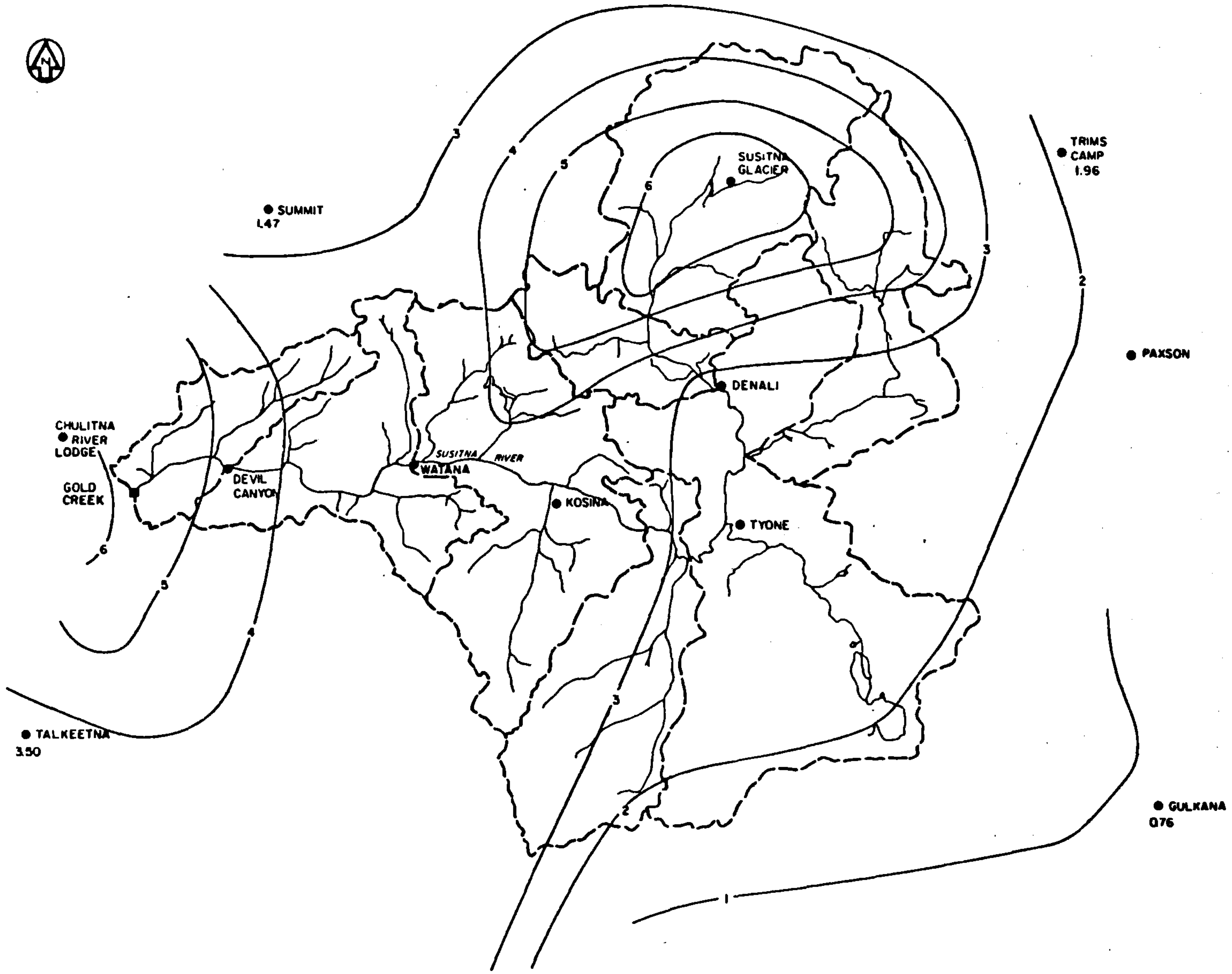
ACRES



ISOHYETAL MAP STORM OF JULY 28 - AUGUST 3, 1958

Figure F3.28

ACRES



LEGEND:

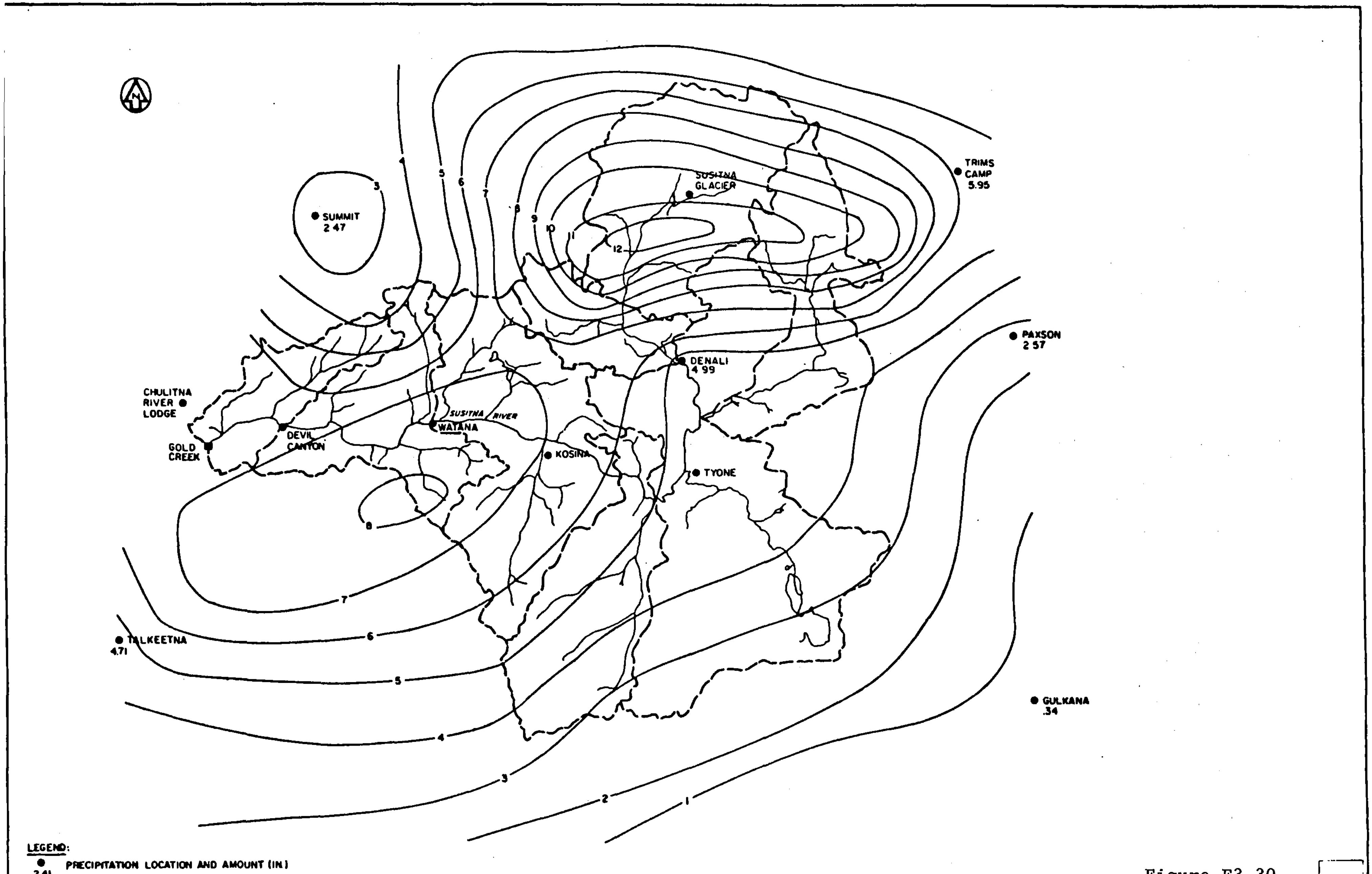
● 3.50 PRECIPITATION LOCATION AND AMOUNT (IN.)

—3— ISOHYET

ISOHYETAL MAP STORM OF AUGUST 19-25, 1959

Figure F3.29

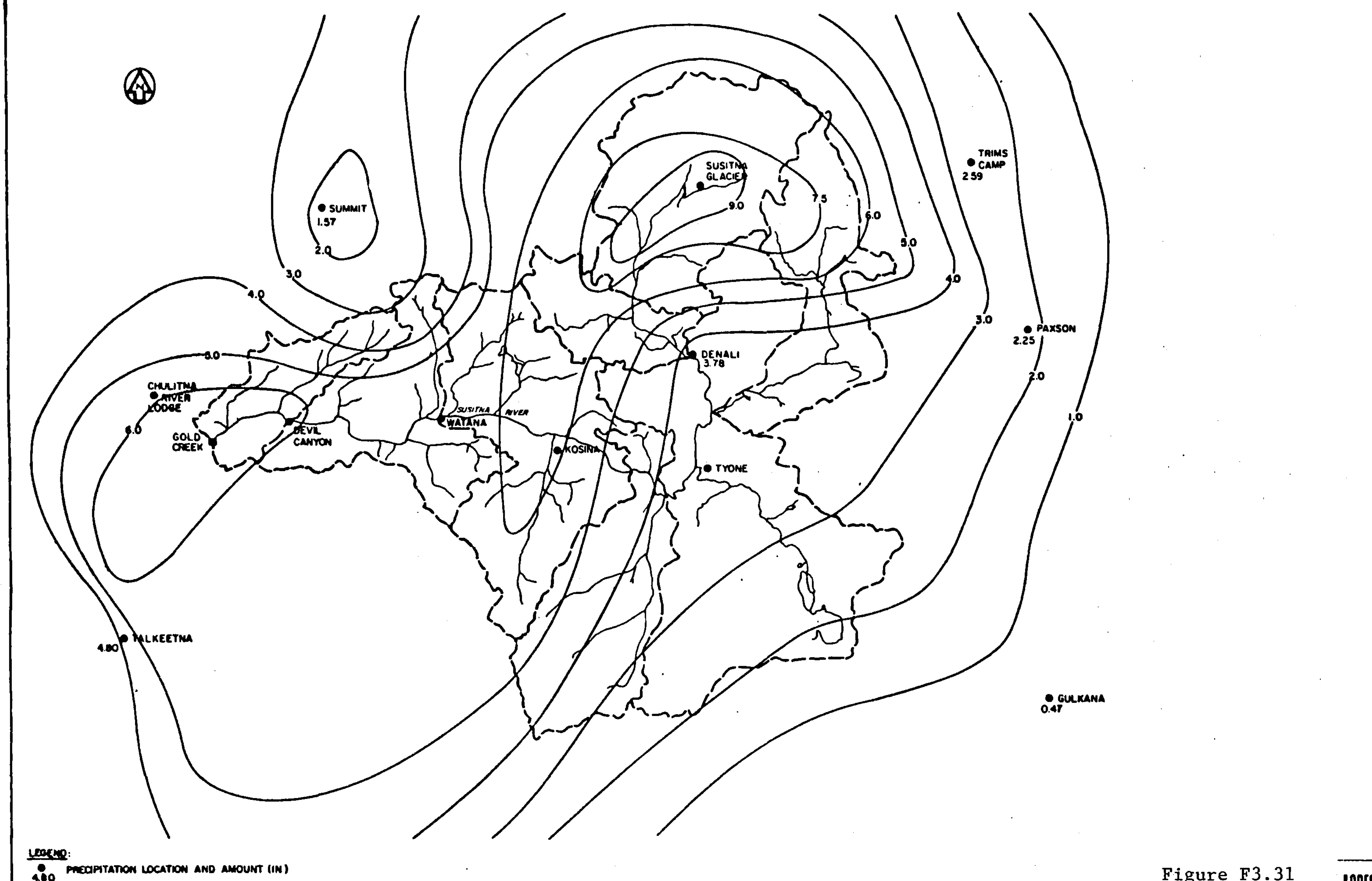
ACRES



ISOHYETAL MAP STORM OF AUGUST 9-17, 1967

Figure F3.30

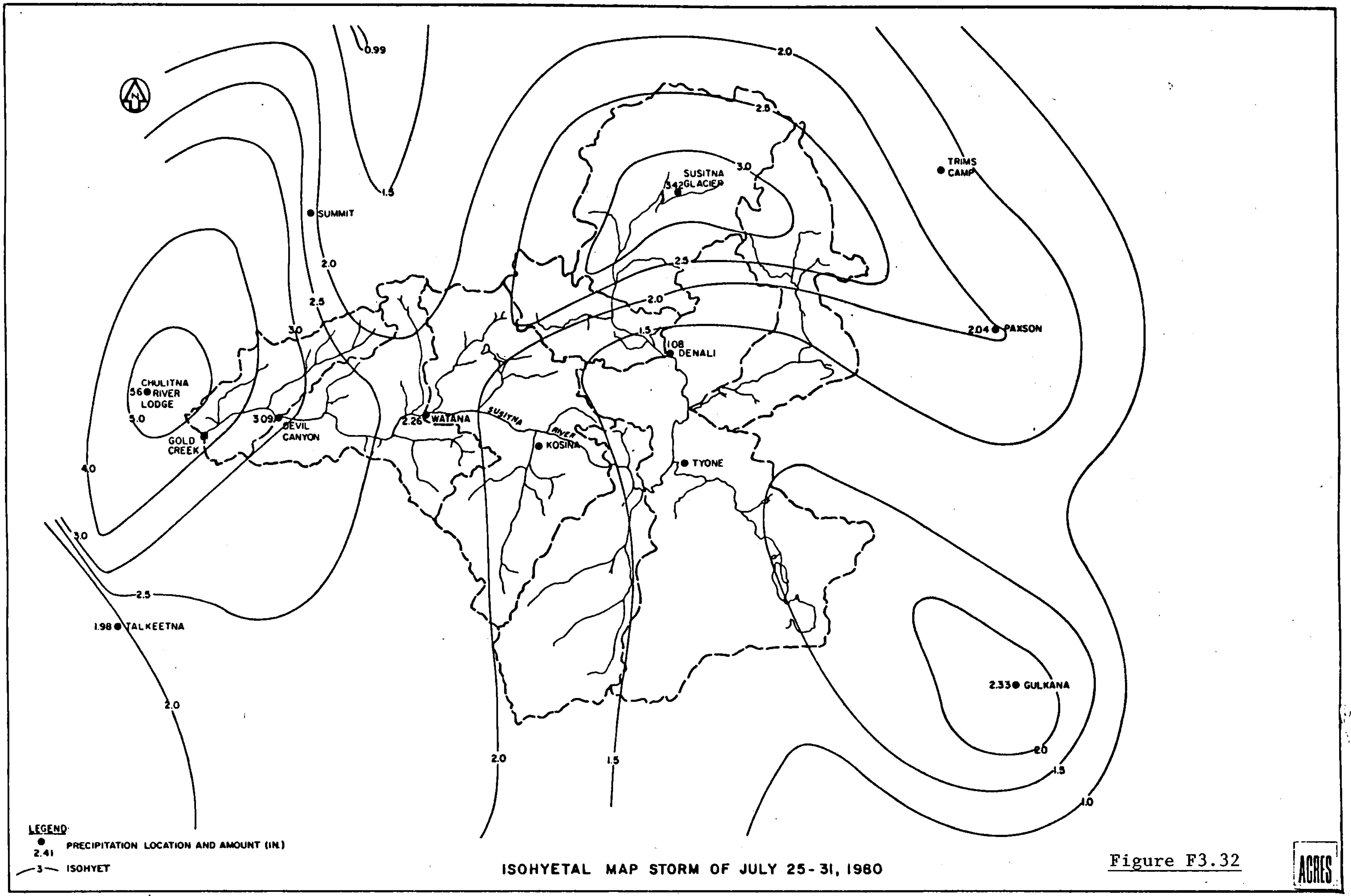
ACRES



ISOHYETAL MAP STORM OF AUGUST 4-10, 1971

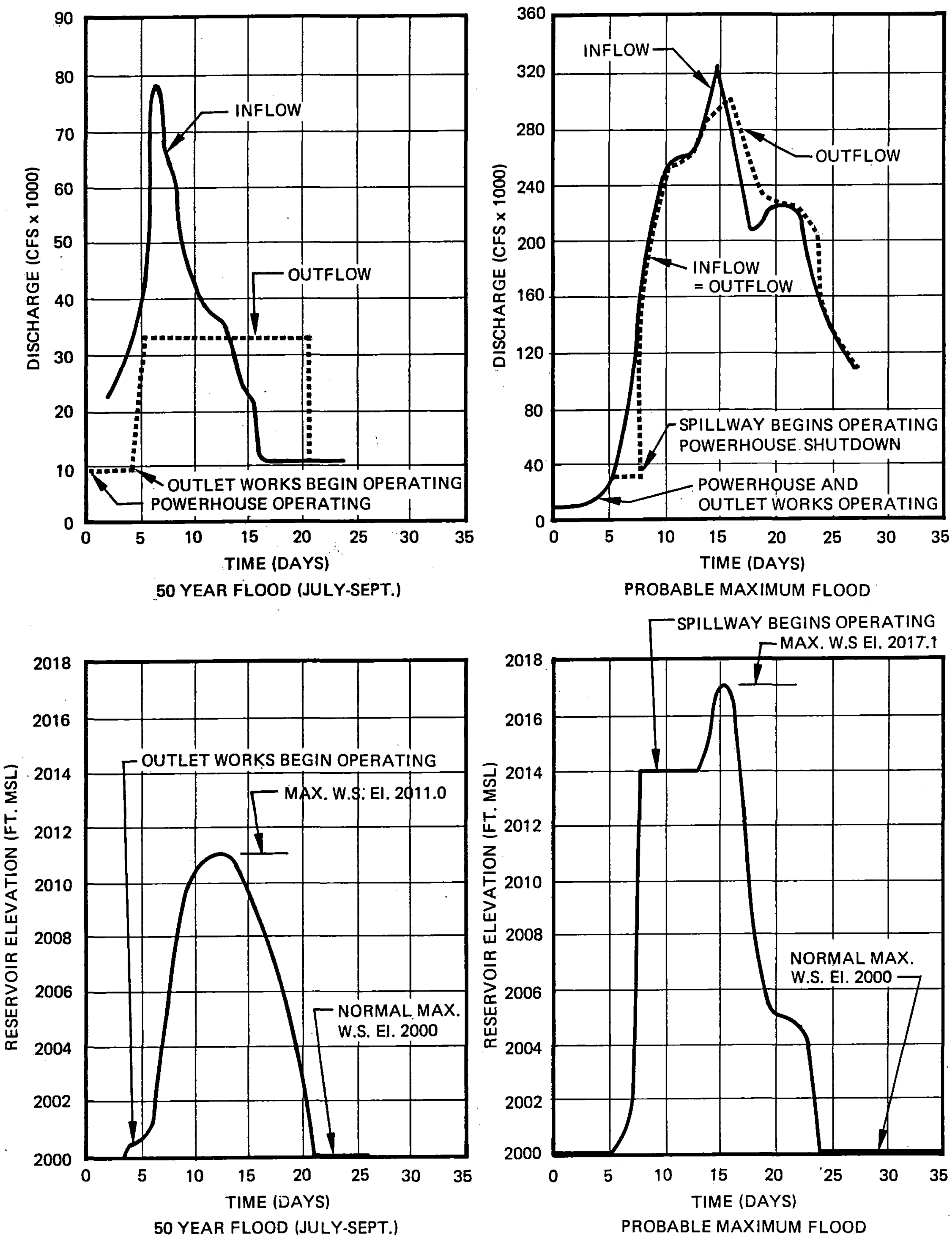
Figure F3.31

ACRES



ISOHYETAL MAP STORM OF JULY 25-31, 1980

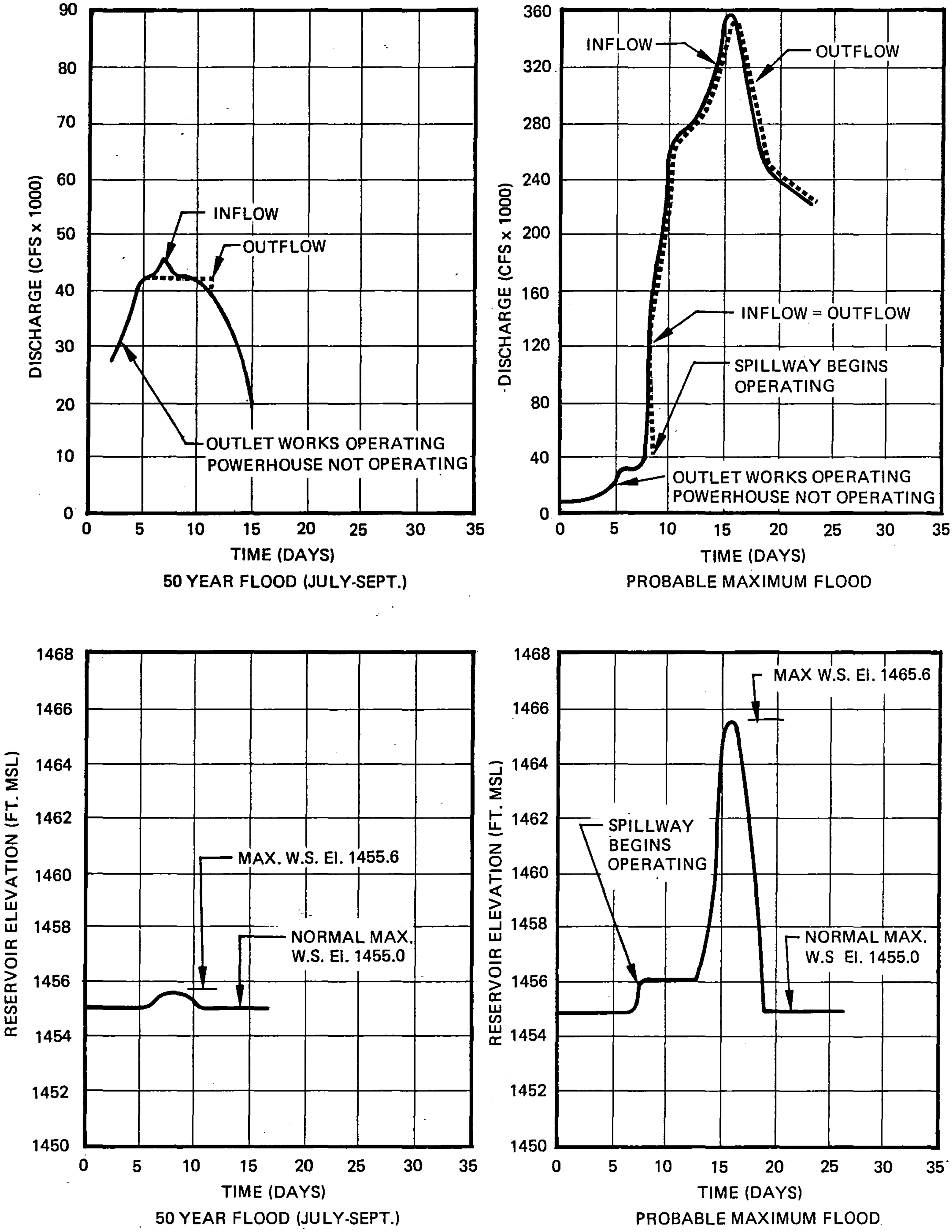
Figure F3.32



WATANA STAGE I

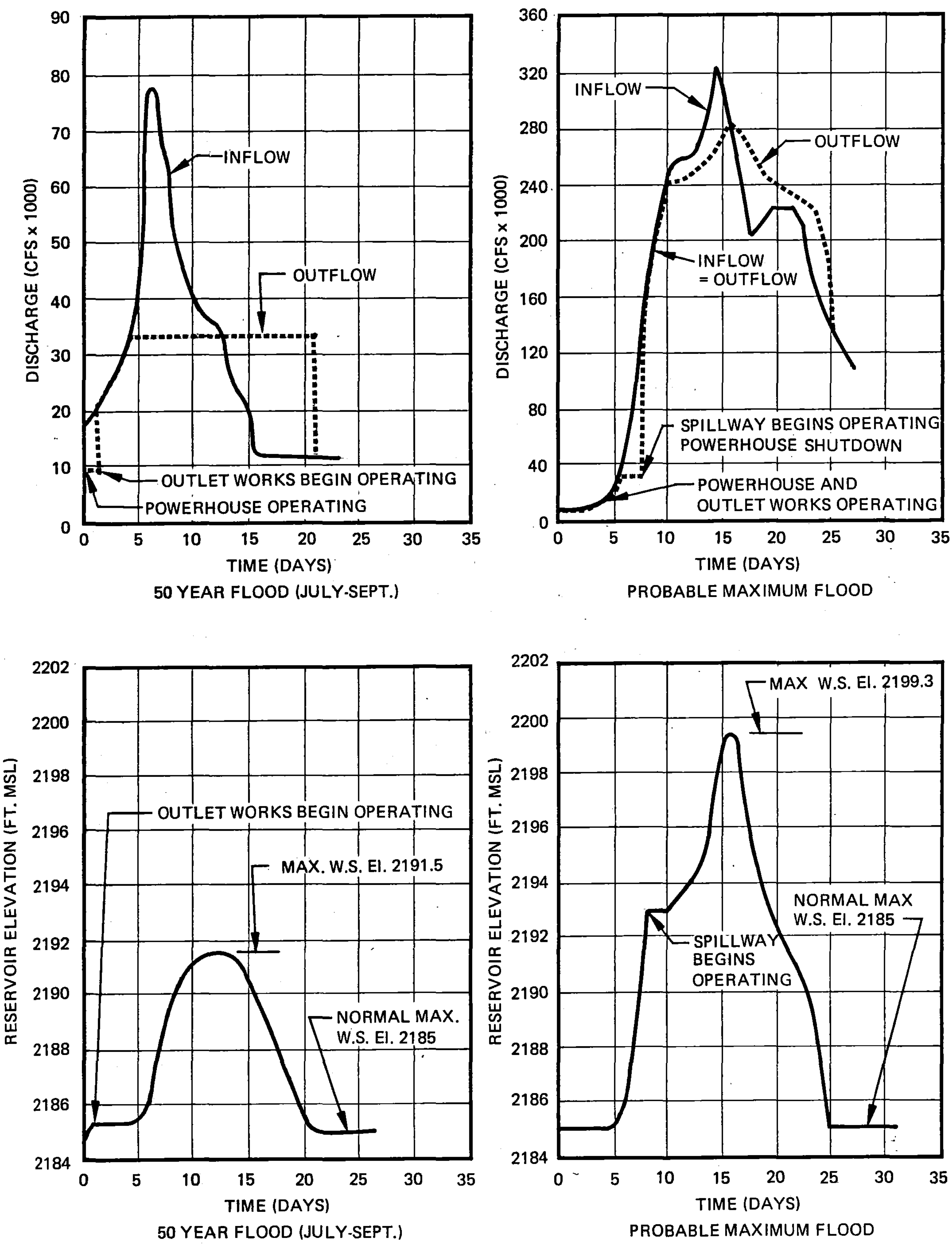
FLOOD DISCHARGES AND RESERVOIR SURFACE ELEVATIONS

FIGURE F3.34



DEVIL CANYON STAGE II
FLOOD DISCHARGES AND RESERVOIR SURFACE ELEVATIONS

FIGURE F3.35



WATANA STAGE III
DISCHARGES AND RESERVOIR SURFACE ELEVATIONS

FIGURE F3.36