APPENDIX S

Design Memorandum, Miner Flat Dam, February 1987

MORRISON-MAIERLE, INC.,
VOLUMES I THRU V OF VI

FEBRUARY 2007

KACZMAREK

DESIGN MEMORANDUM

MINER FLAT DAM

DECEMBER, 1986

[[REVISED FEBRUARY 1987]]

WHITE MOUNTAIN APACHE TRIBE

WHITERIVER, ARIZONA

VOLUME 1 OF 6

MORRISON-MAIERLE, INC.
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MINER FLAT DAM
DESIGN MEMORANDUM

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DESIGN MEMORANDUM
MINER FLAT DAM

SECTION AA - INTRODUCTION

AA.1 PURPOSE

The purpose of this design Memorandum is to provide the reviewing authorities and designers with a summary of the criteria to be used in preparing the final design, contract plans and specifications for the Miner Flat Dam. To provide the required design criteria the Memorandum has been assembled into multiple volumes containing site topography, project layouts, descriptions of local conditions, geological and foundation investigation, construction material requirements, hydrologic studies, operating data, dam studies and design criteria, dam access criteria, public facilities criteria, construction cost estimates, construction schedules and environmental studies.

AA.2 PROJECT DESCRIPTION

The Miner Flat Dam is a key unit described in the "Miner Flat Dam and Canyon Day Irrigation Project - February, 1986" report submitted by Morrison-Maierle, Inc. to the White Mountain Apache Tribe.

The Miner Flat Dam and resulting reservoir is being designed for the purpose of providing water and electrical power in the downstream Canyon Day Irrigation Project. In addition to the primary purpose of the dam additional recreational and tourist benefits will be realized by the Tribe.

The Miner Flat Dam Project will include the construction of the dam and outlet works, preparation for the reservoir, construction of the access roads to the dam, and hydropower facilities.

AA.3 ALTERNATE TYPES OF DAMS

The alternate type of dams considered in the report "Miner Flat Dam and Canyon Day Irrigation Project - February, 1986" include earth-fill, rock-fill, conventional concrete gravity and concrete thin arch. The report recommended that a conventional concrete gravity dam be provided for this site after eliminating the other alternatives for the following reasons.

Earth - Fill Dam - Investigations to locate suitable source of materials in the vicinity of the dam indicated that those materials are absent. A suitable borrow area of sufficient volume for an earth-fill dam was not readily available. Moreover, the high, narrow characteristics of the dam site would require a higher percentage of intensive construction practices than would be involved with a conventional earth-fill damsite. Therefore, earth-fill construction was eliminated from further consideration.

Rock - Fill Dam - A principal disadvantage of the rock-fill structure is the necessity to construct a spillway with crest elevation of 6,050 feet. This would permit 20 feet of free board before over-topping the crest of the dam at elevation 6,070. However, in spilling at elevation 6,050, 1,900 acre feet of active storage is lost below elevation 6,062, the crest elevation with other alternatives. Because storage is limiting at the Miner Flat damsite, the full utilization of the topographic potential is considered necessary.

Concrete Thin Arch Dam - Consideration was given to both constant radius and variable radius thin arch dams. The variable radius thin arch dam was selected for consideration on the basis that material requirements range from 80 to 90 per cent of the material requirements for a constant radius dam.

In the case of a thin arch dam, the structural section required to create the reservoir is substantially less than for the other dams. The structural mass is approximately 1/3 of the mass required for the concrete gravity dam and approximately 1/10 of the mass required for a rock-fill dam. A thin-arch dam, however relies upon the strength of the left and right abutments. It is these abutments that receive the force of the weight of the water on the upstream face of the thin arch dam. The integrity of a thin arch dam was subjected to question due to the nonuniformity of conditions on the left and right abutments of the damsite. The lateral extent of basalt on the right abutment exceeds several thousand feet, but the lateral extent on the left abutment, at the top of the dam, is less than 100 feet.

Unlike the concrete gravity dam, it would be necessary to construct a principal spillway to accommodate normal spills at the damsite. Over-topping of the dam would be an accepted practice during extremely rare floods, but would not generally be accepted as a normal operating procedure. This could become a relatively minor consideration if a thin arch concrete dam was constructed.

AA.4 SELECTED DAM TYPE

Two types of concrete gravity dams have been investigated in the preparation of this Design Memorandum. Detailed designs were made for a conventional concrete gravity dam and a roller compacted concrete (RCC) gravity dam to determine the most practical and economical gravity dam to be designed at this site. Preliminary designs, cost estimates and construction schedules for the conventional concrete and RCC dams have been included with this memorandum as back-up for the selection of the RCC dam. The RCC dam has been selected for final design to provide the most economical, practical, low maintenance, and safe dam for this site.

MINER FLAT DAM
DESIGN MEMORANDUM

SECTIONS A, B, AND C

SECTION A. GENERAL MAP

A general map A-1 has been prepared which shows the White River watershed, Miner Flat Dam, Canyon Day Irrigation Project, existing land lines, towns, transportation systems, public utilities and stream gaging stations. The location of sites for construction staging areas, borrow areas, disposal areas, archaeological sites, recreation areas and other features are shown on a more detailed map A-2 of the Miner Flat Dam and reservoir area. An archaeological evaluation of the site, prepared by Lyle M. Stone, Archaeological Research Services, Inc., Tempe, Arizona, should be reviewed for a more detailed investigation of the cultural resources at the project site.

SECTION B. GENERAL DESCRIPTION OF LOCAL CONDITIONS

The primary mode of transportation serving the Canyon Day Irrigation Project and Miner Flat Dam is Arizona Highway Number 73 which enters the project area from the west and traverses the central portion of the project area from west to east. Highway 73 connects the project lands with the community of Whiteriver, headquarters of the White Mountain Apache Tribe, and communities of Pinetop and Show Low beyond the northern boundary of the Fort Apache Indian Reservation.

Assuming that the majority of the construction materials and equipment will come to the project site from Phoenix, it is important to know load restrictions or other restrictions on Highway 60 as well as Highway 73. Material and equipment from the north via Highway 73 is also a possibility. Highway 60 has no special load restrictions and no permit or escort requirement for widths up to 14 feet. Over 14 feet width requires a class 'c' permit from the Arizona Department of Transportation, Structures Maintenance Division.

Highway 73 from Indian Pine south has the same restrictions as Highway 60. Highway 73 from Carrizo has no special load restrictions but anything over an 8 foot width requires a permit and over 10 feet wide requires escorts.

General aviation airport facilities are available at Whiteriver. A primary use has been as a base for slurry bombers for fires. Minimal fixed base operator services are available. Airport facilities for general aviation are also available at Springerville and St. Johns. Scheduled airline service is available at Show Low.

Presuming construction equipment and supplies would come primarily from Phoenix, the following freight rates for heavy transport illustrate freight costs:

42,000 lbs	-	$2.85/mi
50,000 lbs	-	3.10/mi
60,000 lbs	-	3.55/mi
70,000 lbs	-	4.10/mi
80,000 lbs	-	5.00/mi
90,000 lbs	-	5.75/mi
162,000 lbs	-	10.75/mi

The above rates do not include load/unloaded costs. There is also a state tax of 8¢/mi. for heavy transport.

The nearest railroad facility to the project site is presently the Apache Railroad at Showflake. Connection of this line to Santa Fe Rail is at Holbrook.

Since there are several communities within a reasonable commuting distance of the project site, it is not anticipated that a construction "camp" or temporary living quarters will be formally established at the site. Communities near the site do have residential rental available as well as mobile home court space. Residential rental is reported to be fairly scarce during the recreation/tourist season (May - September) but ample unit availability during the off season. Mobile home space is ample all year around.

Electrical power exists near the site via Navo-Patche Electrical Coop. There is a 69KV feed from McNary that goes to Whiteriver and a 14.4/24.9KV distribution is available at the project site. Ample power is available from McNary. Power rates would probably be available in two general categories.

(1) Large commercial-industrial capacity

A. + 50 KVA

a. on peak (6a.m. - 9p.m.)
demand $17/KW/month
energy 3.9071¢/KWH

b. off peak (9p.m. - 6a.m.)
demand $14/KW/month
energy 3.666 ¢/KWH

(2) small commercial capacity

a. customer charge $12.75/month

b. energy 9.14¢/KWH

Climate conditions at the Miner Flat Damsite, elevation 6,060 feet, lie in a mid-range of climatic conditions as reflected by the McNary and Whiteriver weather stations. The

Miner Flat Damsite is located at the transition between the Ponderosa Pine and Juniper-Pinyon vegetation zones.

At McNary 29 days is the mean number with minimum temperatures of 32 ° F. or below. In contrast to Whiteriver there are an additional 23 days of 32° F. and below in April and 13 days of 32° F. and below in May. June 10 is the normal date of the last freezing temperature in spring and September 27 is the normal date of the first freezing temperature in fall.

Daily temperatures average more than 60° F. in July and August. Early morning temperatures in the high 40's are expected, and temperatures in the high 70's or low 80's are reached by mid-afternoon. Temperatures above 90° F. are rare. In winter, temperatures at McNary are generally in the high teems in morning and reach the middle 40's by afternoon. Temperatures below zero are expected at least one or two days each winter.

In contrast to Whiteriver, about 80 per cent of the winter precipitation is snow. Accumulations of several feet of snow are expected during the cold and wetter periods of the winter. 6/

Independent analysis of climatic data at the McNary weather station was also undertaken. From 1943 through 1980, precipitation averaged 26.5 inches. During the months of September through March, average monthly precipitation ranged from 6.7 to 11 per cent of the average annual, and in May and June, monthly averages were 2.8 and 2.7 per cent of annual, respectively. As at Whiteriver, the July and August rainfall averages are the highest of the year with 13.0 and 14.6 per cent of the annual total occurring in those months.

The five year running average of precipitation at McNary for the period 1943 through 1980 ranged from 22.44 to 30.40 inches through 1978. The five year running averages ending in 1979 and 1980 were 32.74 and 33.31 inches, respectively. A marked increase in annual precipitation at Mcnary began in 1978.

Climatological data from the White River and McNary weather stations provide representative information for the majority of the White River watershed. As would be expected, precipitation increases with elevation, and temperature decreases with elevation. In the headwaters of the White River watershed in the vicinity of Mt. Baldy, normal annual precipitation amounts greater than 32 inches are expected. In the lower portions of the White River watershed near the confluence of the White River with the Black River, annual precipitation of 16 to 20 inches per year is expected. 7/

6/ U.S. Department of Commerce, Weather Bureau, Climatography of the United States, Climate of McNary, Arizona, period 1933-1972.

7/ Arizona Stated University, May 1975; Map of Normal Annual Precipitation, Arizona, State Climatologist and Laboratory of Climatology, (Tempe, AZ).

Winter temperatures are generally mild during the day and cold at night. Daytime temperatures rise to the middle or upper 50's and may rise into the 60's. Winter nighttime temperatures generally fall to about 20° F. Temperatures below zero are uncommon. During March the mean number of days of minimum temperatures below freezing is 21 days, and by April the mean number of days with low temperatures of 32° F. or below has declined to under 10 days. Freezing temperatures begin again in October with 4 days of 32° F. or below as the minimum temperature.

Independent analysis of precipitation data at Whiteriver for the period 1941 through 1980 was made. The average annual precipitation during the period of study was 18.3 inches. On the average, each month from September through March of each year provided from 6 to 10 per cent of the annual precipitation. Precipitation during the month of April averaged about 4 per cent, and precipitation during the months of May and June each averaged 2.6 per cent. Precipitation during the months of July and August, the principal months of the thunderstorm season averaged 15 and 18 per cent of the annual precipitation, respectively.

Five year running averages of precipitation at Whiteriver ranged from a low of 16.02 inches to a high of 18.88 inches for the period 1941 through 1978. The five year running average ending in 1979 and 1980 were 21.74 and 23.17 inches, respectively. The annual pattern of precipitation at Whiteriver between 1941 and 1978 was markedly consistent. Since 1978, a significant increase in precipitation has been recorded.

The weather station at McNary is located at elevation 7,320 feet and at a latitude of 34° 04' North. 5/ The McNary weather station is reflective of the climatic conditions in the central portion of the White River watershed. McNary is located in the Ponderosa Pine vegetation zone.

Wind data for Arizona is not especially abundant, existing in quantity only for the U.S. Weather Service airport offices at Winslow, Flagstaff, Prescott, Tuscon, Phoenix and Yuma. Fortunately, these six cities are quite widely scattered and contain roughly 75 per cent of the state's population.

It is felt that conditions at Winslow might be indicative of what might be expected at Miner Flat.

A brief summary of the wind conditions at Winslow is given in Table 3a. Because of the effect of terrain peculiarities on the wind at a point, it is not completely valid to extrapolate the data in Table 3a to Miner Flat. This is particularly true in the case of the wind direction, which is greatly affected by differential heating between mountains and valleys or between highlands and lowlands. Ordinarily, if no large-scale weather disturbances are present and the winds in the free atmosphere are light, the surface wind will blow upslope or upvalley during the day, when the air overlying the slope is heated more rapidly than that at the same elevation over the valley. At night, downslope

or downvalley winds are induced by the rapid radiational cooling of the air overlying the mountain slopes compared to the slow cooling of the free atmosphere air over the valleys. Normally the daytime upvalley winds are two or three times stronger than the nighttime downvalley winds.

Winslow lies in the windiest part of the state, with the possible exception of some of the deep narrow canyons of the plateau and central sections.

Average monthly records for McNary have been compiled from 1933 through 1972 and are listed in Table 3B.

TABLE 3A

WIND DATA FOR WINSLOW, BASED ON RECORD FOR JANUARY 1948 - DECEMBER 1955

Observation time	month	Wind Direction prevailing	Wind Direction constancy**	Percent of calms	Wind Speed* (Miles per Hour) average	Wind Speed* (Miles per Hour) maximum
0:20 a.m.	Jan	SW	46	14	6.2	30
	Apr	SW	64	7	8.3	42
	Jul	SW	65	11	7.1	21
	Oct	SW	50	22	4.8	20
3:20 a.m.	Jan	SE	42	16	5.9	29
	Apr	SW	48	16	6.6	27
	Jul	SW	44	18	5.0	19
	Oct	SE	43	19	4.8	35
6:20 a.m.	Jan	SE	52	12	5.9	28
	Apr	SE	50	10	7.0	25
	Jul	SE	54	17	5.2	23
	Oct	SE	57	14	5.1	25
9:20 a.m.	Jan	SE	55	12	7.9	30
	Apr	SE	42	10	10.3	68
	Jul	SE	41	18	5.9	18
	Oct	SE	61	13	7.8	26
12:20 p.m.	Jan	SE	40	9	9.9	50
	Apr	WSW	47	3	13.4	45
	Jul	NE	28	8	7.5	30
	Oct	SE	34	8	8.9	36
3:20 p.m.	Jan	SW	35	8	10.2	40
	Apr	WSW	58	3	16.3	58
	Jul	WSW	60	4	11.5	31
	Oct	WSW	36	5	9.9	38
6:20 p.m.	Jan	SW	49	13	7.3	35
	Apr	SW	62	#	15.3	46
	Jul	WSW	65	1	12.7	40
	Oct	Sw	50	9	7.1	22
9:20 p.m.	Jan	SW	58	13	6.3	35
	Apr	SW	78	3	10.0	33
	Jul	WSW	69	4	8.8	38
	Oct	SW	73	10	6.3	21

No data is available on the strongest wind speeds and gusts at the Winslow airport. However, one minute average speeds of at least 68 miles per hour and gusts of at least 70 miles per hour have occurred.

* Values given are the average and maximum one minute wind speeds.

** Values given are the percentage of the time that the wind blows within 45 degrees of the prevailing direction.

\# Less than one percent.

TABLE 3B

AVERAGE MONTHLY TEMPERATURES (F) FOR MCNARY

YEAR	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	ANNUAL
1933									61.2	50.4	39.3	36.4	
1934	32.2	37.3	43.8	47.5	55.2	56.6	65.7	63.8	56.4	50.4	38.4	33.7	48.4
1935	32.7	33.0	35.7	44.0	46.0	59.0	63.8	62.9	56.0	48.0	36.8	32.0	45.8
1936	30.5	30.3	35.4	45.6	52.9	61.0	64.2	63.7	56.4	48.0	45.3	33.8%	47.3
1937	21.2%	28.8	35.6%	45.2	54.0	64.3	68.5%	64.6	51.2%	39.8%	34.8	47.3	
1938	34.2	33.8%	37.6	44.5	51.5	59.8				49.0	35.8	34.2	
1939	28.4	22.7	36.1	46.6%	53.3%	60.0%	64.9	63.6	58.2				
1940		33.8	40.6	44.2%	53.0	61.6	64.4%	58.6	51.0	37.0			
1941				38.4	51.4	56.8	63.0	62.4	57.2		40.2	33.2	
1942	32.3	29.9	35.3	42.5	52.5	60.8	66.7	63.2	58.7	48.0	42.8	35.7	47.4
1943	32.0	36.6	40.2	49.2	53.8	60.7	67.0	65.2	60.2	48.6	40.3	32.4	48.9
1944	30.0	31.0	32.9	40.4	49.7	57.0	63.4	63.8	57.0	49.4	34.0	32.0	45.1
1945	29.8	34.4	32.6	41.6	50.1	56.2	65.4	65.0	57.4	49.3	37.3	28.2	45.6
1946	27.4	30.4	38.0	47.7	48.6	60.1	63.4	62.4	58.8	46.3	34.6	35.8	46.1
1947	26.6	36.6	38.8	42.1	52.8	56.4	65.2	62.8	59.6	49.1	32.6	28.6	45.9
1948	31.8	29.4	29.4	45.9	51.8	59.2	64.2	63.4	58.4	47.8	33.7	32.4	45.6
1949	23.4	27.0	36.2	43.4	50.0	57.4	62.5	60.6	58.7	44.1	42.5	27.6	44.5
1950	28.6		37.7	45.2	47.9	57.5	61.6	59.4	53.0	52.5	41.3	36.0	
1951	29.3	31.8	34.0	41.1	46.3%	58.3%	64.3	62.7	58.2		36.4%	29.9	
1952	32.4	31.3	30.0	45.4%	53.9	58.5	62.0			51.9	34.2%	29.7%	
1953	37.5%	32.2%	39.6%	43.5	49.8	59.2	66.3	62.0%	59.0	48.1	39.9	28.9	47.0
1954	32.4	38.0	36.0	48.0	52.2	58.7	65.7	61.9	59.0	51.3	41.7	32.3	48.1
1955	23.3	25.4	37.2	41.8	49.6	57.5	62.0	63.2	57.5	50.8	39.2	34.9	45.2
1956	34.9	24.0	38.5	42.2	52.3	62.7	62.7	61.0	61.1	50.1%	34.8	31.1	46.3
1957	32.4	39.2	38.3	41.7	49.1%	60.4%	65.5	62.9	58.4	48.3	34.5	36.8	47.3
1958	31.0	36.0	32.8	43.2	56.6	63.6	65.7	64.5	58.0	49.1	38.9	38.2%	48.1
1959	33.4	30.5	37.7	48.3	51.9	63.6	66.9	61.9	57.6	48.9	38.4	33.5	47.7
1960	24.8	26.7	41.2	46.1	51.1	62.2	65.8	64.6	59.6	47.1	40.5	30.3	46.7
1960	33.5	33.58	36.7	45.1	52.4	63.3	65.8	61.8	55.4	48.1	36.0	30.1	46.8
1962	28.2	34.8	30.9	48.8	50.7	58.2	64.3	64.4	58.7	49.7	42.2	34.9%	47.2
1963	27.7	36.5	36.7	44.8	55.3	58.7	66.6	62.1	60.3	53.1	40.0	33.6	48.0
1964	27.5	25.6	32.0	41.0	54.6	60.6	66.2	63.5	57.0	52.8	36.1	32.9	45.8
1965	32.6	29.7	33.4	42.1	49.4	55.4	64.0	63.4	56.6	50.4	42.1	33.2	46.0
1966	25.7	26.8	39.3	47.0%	55.6	61.0	68.0	64.4	58.1	48.5	42.4	32.4	47.4
1967	31.0	35.5	41.3	42.7	49.4	57.6	64.5	61.8	56.7	49.7	41.1	28.7	46.7
1968	30.9	36.7	37.0	41.3	51.3	60.7	62.7	58.3	55.7	50.0	37.5	28.1	45.9
1969	34.8	31.3	31.7	45.2	52.2	58.7	66.0	65.9	59.1	47.1	38.6	35.0	47.1
1970	30.4	37.4	34.3	39.9	53.2	61.4	66.3	64.1	56.8	45.1	41.6	33.3	47.0
1971	32.0	33.3	40.4	43.7	49.6	60.7	67.2	62.9	58.7	44.6	39.5	28.7	46.8
1972	33.4	36.0	45.3	46.0	52.2	59.7	65.8	64.0	58.7	49.9	33.8	29.3	47.8

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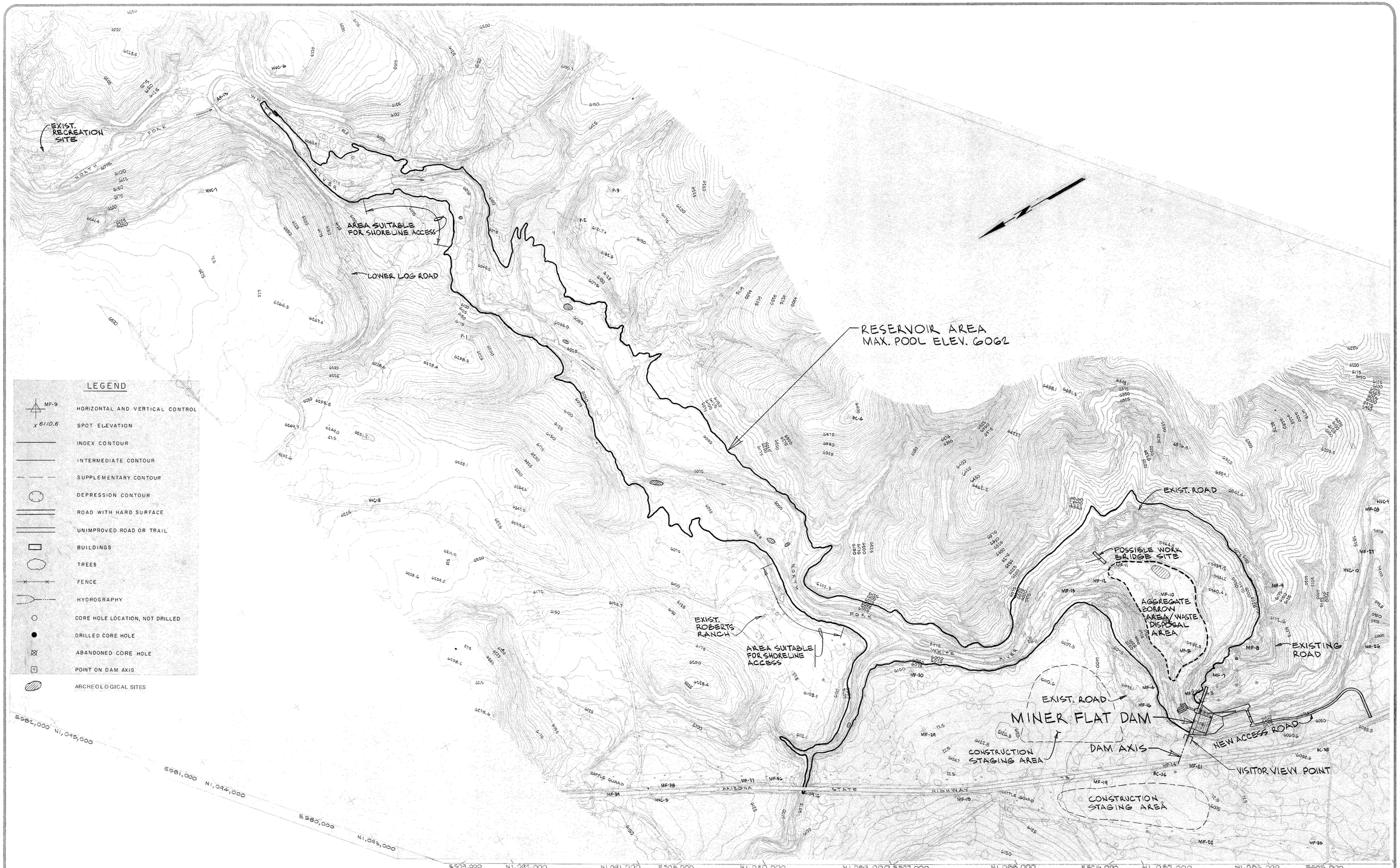

LEGEND

- MF-9 — HORIZONTAL AND VERTICAL CONTROL
- x 6110.6 — SPOT ELEVATION
- INDEX CONTOUR
- INTERMEDIATE CONTOUR
- SUPPLEMENTARY CONTOUR
- DEPRESSION CONTOUR
- ROAD WITH HARD SURFACE
- UNIMPROVED ROAD OR TRAIL
- BUILDINGS
- TREES
- FENCE
- HYDROGRAPHY
- CORE HOLE LOCATION, NOT DRILLED
- DRILLED CORE HOLE
- ABANDONED CORE HOLE
- POINT ON DAM AXIS
- ARCHEOLOGICAL SITES

SCALE: 1"= 400'

NO.	DESCRIPTION	DATE	BY
	REVISIONS		

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DRAWN BY: SJA
CHKD. BY: DJC
APPR. BY: CWK
DATE: 12-10-86
QA
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)

SHEET NUMBER
A2

SHEET TITLE
GENERAL MAP "B"

SHEET TITLE

GENERAL MAP "A"

PROJECT NUMBER 1780-14-02(34)

SHEET NUMBER A 1

MINER FLAT DAM

WHITE MOUNTAIN APACHE RESERVATION

NAVAJO COUNTY, ARIZONA

DRAWN BY: JMH
CHK'D. BY: DC
APPR. BY: CWK
DATE: 12-10-86
Q.A.
PEER REVIEW

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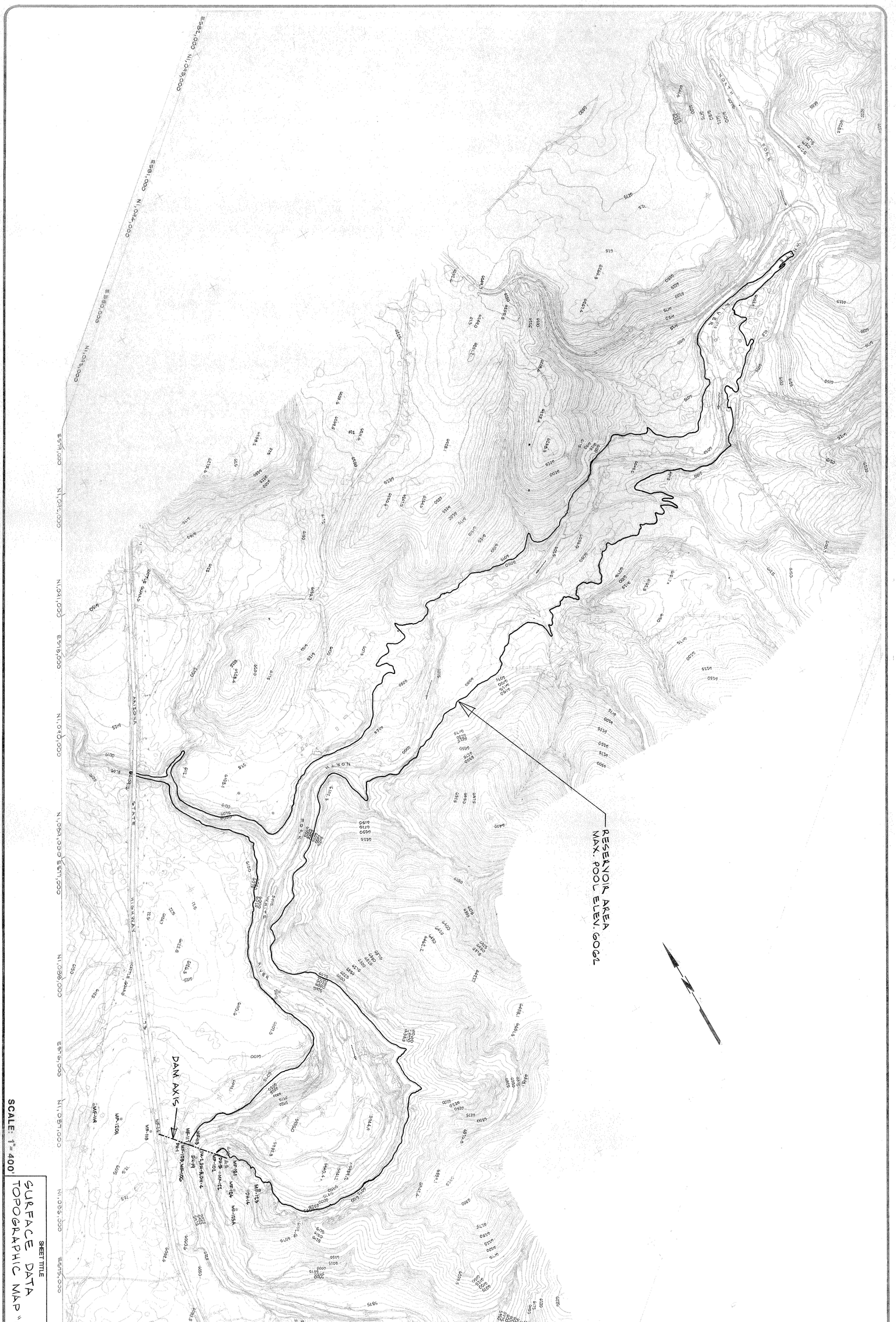
CORPORATE OFFICE HELENA, MONTANA
BRANCH OFFICES BILLINGS, MONTANA KALISPELL, MONTANA BOZEMAN, MONTANA RIVERTON, WYOMING

NO.	DESCRIPTION	DATE	BY.
REVISIONS			



SCALE: 1" = 400'

SHEET TITLE
SURFACE DATA
TOPOGRAPHIC MAP "B"

PROJECT NUMBER
1790-14-02(34)

SHEET NUMBER
C2

MINER FLAT DAM

WHITE MOUNTAIN APACHE RESERVATION

NAVAJO COUNTY, ARIZONA

DRAWN BY: CJA
CHK'D. BY: DC
APPR. BY: CNK
DATE: 12-10-86
QA
PEER REVIEW

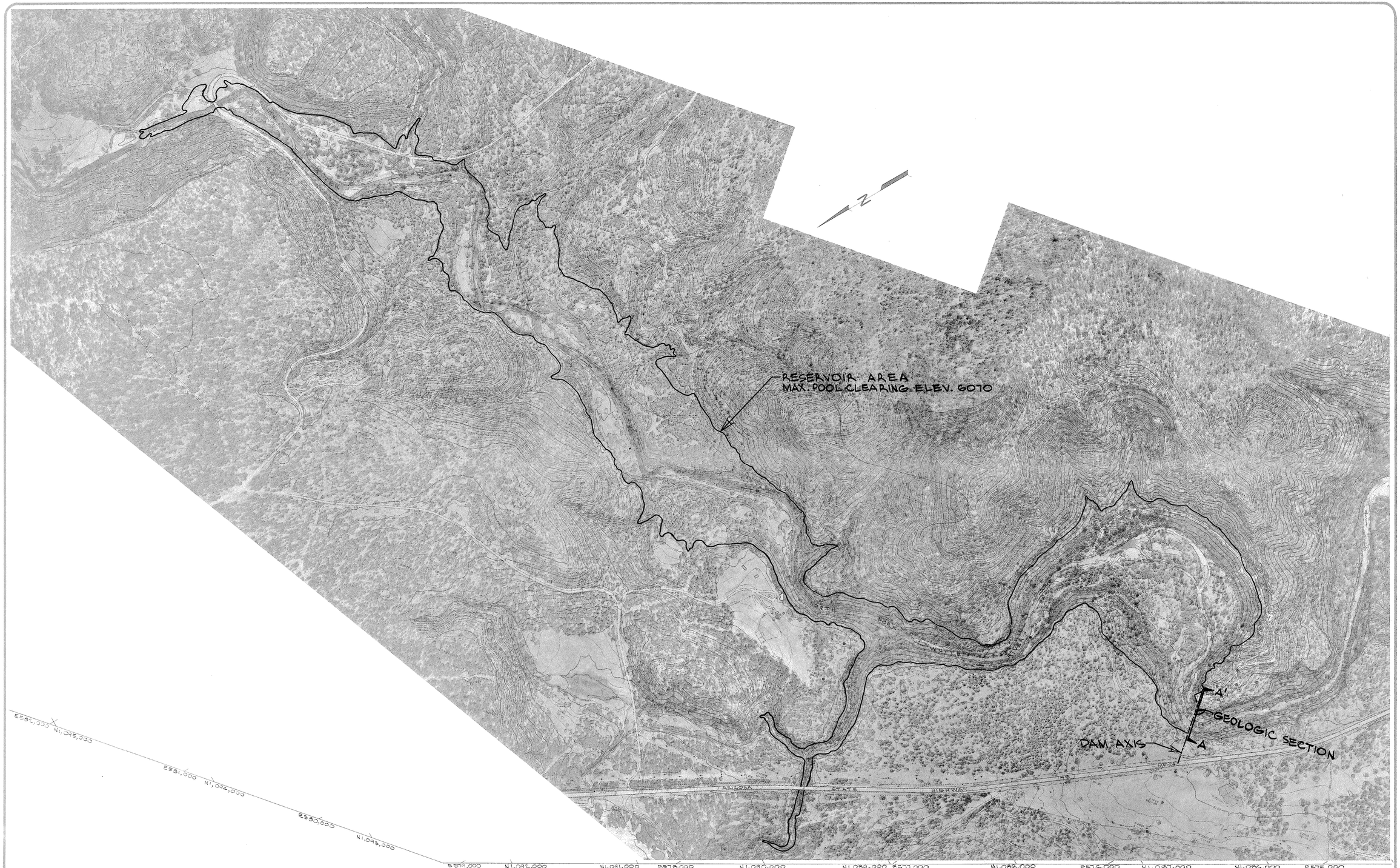
MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604

CORPORATE OFFICE
HELENA, MONTANA

BRANCH OFFICES
BILLINGS, MONTANA
KALISPELL, MONTANA
BOZEMAN, MONTANA
RIVERTON, WYOMING

NO.	DESCRIPTION	DATE	BY:

REVISIONS



SCALE: 1"=400'

SHEET TITLE
SURFACE DATA
TOPOGRAPHIC MAP "C"

NO.	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE, P.O. BOX 6147
HELENA, MONTANA 59604

CORPORATE OFFICE
BILLINGS, MONTANA

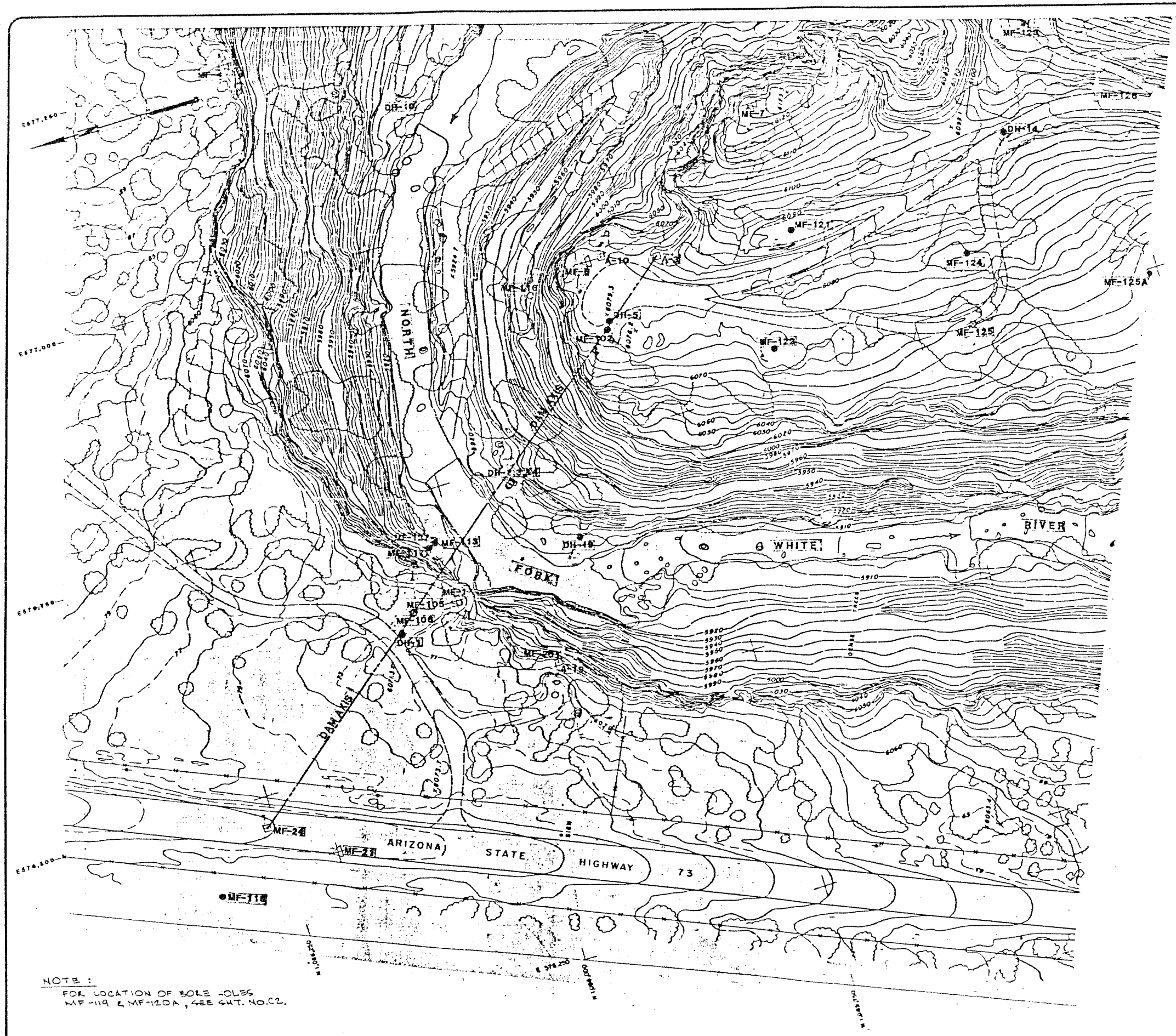
BRANCH OFFICES
BILLINGS, MONTANA
KALISPELL, MONTANA
BOZEMAN, MONTANA
BUTTE, MONTANA

DRAWN BY: CJA
CHK'D. BY: DC
APPR. BY: CMK
DATE: 12-10-96
Q.A.
PERS. REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(94)

SHEET NUMBER
C3



NOTE:
FOR LOCATION OF BORE HOLES MF-119 & MF-120A, SEE SHT. NO. C2.

LEGEND

- MF-9 HORIZONTAL AND VERTICAL CONTROL
- x6110.6 SPOT ELEVATION
- INDEX CONTOUR
- INTERMEDIATE CONTOUR
- SUPPLEMENTARY CONTOUR
- DEPRESSION CONTOUR
- ROAD WITH HARD SURFACE
- UNIMPROVED ROAD OR TRAIL
- BUILDINGS
- TREES
- FENCE
- HYDROGRAPHY
- CORE HOLE LOCATION, NOT DRILLED
- DRILLED CORE HOLE
- ABANDONED CORE HOLE
- POINT ON DAM AXIS

SCALE: 1"=50'

SHEET TITLE
SURFACE DATA
TOPOGRAPHIC MAP "A"

NO	DESCRIPTION	DATE	BY

REVISIONS

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604

DRAWN BY: CJA
CHK'D BY: DC
APPR BY: CJA
DATE: 12-10-82
Q.A.
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
C1

MINER FLAT

DESIGN MEMORANDUM

SECTION C

Geodetic Research, Inc. has prepared this section as a portion of the Design Memorandum. The summary is reproduced herein complete as received by Morrison-Maierle, Inc.

SUMMARY OF FIELD SURVEYS AND SURFACE DATA

PROPOSED MINER FLAT DAM SITE

FORT APACHE INDIAN RESERVATION, ARIZONA

NOVEMBER, 1985 THRU NOVEMBER, 1986

IN ACCORDANCE WITH MORRISON-MAIERLE, INC.
OCTOBER 10, 1985 LETTER OF AUTHORIZATION

Prepared by:
Elmer M. Clark
Professional Land Surveyor

Geodetic Research, Inc.
5753 South Kearney Street
Englewood, Colorado 80111-1526

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SUMMARY OF THE SURFACE DATA - FIELD SURVEYS

In accordance with the Morrison-Maierle, Inc. October 10, 1985 letter of authorization for the proposed Miner Flat Dam on the Fort Apache Indian Reservation, Arizona, Geodetic Research, Inc. (formerly Geotech Research, Inc.) completed the specified field surveys, photogrammetric mapping and certain authorized supplemental surveys as itemized below.

A. Field surveys establishing the Arizona State Plane Coordinate System at the proposed Miner Flat Dam Site and reservoir area.

B. Establish a primary, horizontal and vertical survey control net with semipermanent monuments as required for photogrammetric mapping, core hole drilling and preliminary construction.

C. Locate proposed core holes for grading and drilling and upon completion of the core holes, check the location of the core holes and determine the ground (or concrete) and pipe elevations.

D. Provide aerial photography for photogrammetric mapping.

E. Prepare a 1" = 400 ft. topographic map with 5.0 ft. contours for the proposed Miner Flat Dam Site and reservoir area.

F. Prepare a topographic map for the area adjacent to the proposed dam site and in the general construction area at a scale of 1" = 50 ft. with 2.0 ft. contour intervals.

G. Provide an aerial photographic map of the proposed Miner Flat Dam Site and reservoir area on a reproducible medium.

H. Complete field checks in the proposed dam site axis area.

I. Preliminary location of the major structures.

J. Provide critical elevations at the pump and delivery station and other points as required.

K. Provide black and white aerial photography from Canyon Day Irrigation Project upstream along the White River and the North Fork of the White River to Becker Creek.

L. Complete analytical aerotriangulation and digitized cross sections at approximately one half mile intervals along the White River and the North Fork of the White River from the Canyon Day Irrigation Project to Becker Creek.

All of the above work has been completed (except as noted in the following text), using professional surveying procedures and equipment for work of this nature.

A. FIELD SURVEYS ESTABLISHING THE ARIZONA STATE PLANE COORDINATE SYSTEM.

Prior to the commencement of the Miner Flat Dam Site, Geodetic Research, Inc. (formerly Geotech Research, Inc.) had established a second order horizontal control traverse 1/ and a vertical angle control traverse 2/ from U.S.G.S. stations near McNary, along Arizona State Highway 73 thence to the Canyon Day Irrigation Project. Final coordinates for this traverse were adjusted by the compass method and Arizona State Plane Coordinates (sea level datum) were modified with a combined grid and sea level factor of 0.999634924 to a mean elevation of 6,000.0 feet. 3/

Traverse stations #BC-34 and #BC-35 for the above traverse were located near the proposed Miner Flat Dam Site westerly of Arizona State Highway 73. These stations were recovered (undisturbed) and the distance between them checked within 0.04 feet.

All coordinates for the proposed Miner Flat Dam Site are based upon the above modified Arizona State Plane Coordinate system. The delta alpha angle between grid bearing and true bearing at traverse station #BC-34 is + 0° 08' 28", therefore, it was not practical to calculate a new coordinate system based upon a true north bearing with assumed coordinates.

1/ $\frac{\text{Traverse length = 135,926.963 ft.}}{\text{Closing error = 2.868 ft.}}$ = 1:47,391.046
Angular error 0° 00' 18" in 23 angles

2/ Maximum vertical distance (error) in closure, 1.0381 ft. in 28 stations (0.0381 ft. adjustment per station)

3/ The modified Arizona State Plane Coordinates may be reduced to sea level datum Arizona State Plane Coordinates by:
Modified "Y" coordinate X 0.999634924 = State "Y" coordinate
$\frac{\text{Modified "X" coordinate}}{1.00036521}$ + 182.538 = State "X" coordinate

B. ESTABLISH PRIMARY HORIZONTAL AND VERTICAL CONTROL.

All field surveys were made with a Wild T-1 or T-2 Theodolite in good adjustment. All horizontal and vertical angles were measured at least twice. All slope and horizontal distances were measured with a calibrated K&E Auto Ranger or new surveyor's chain divided to 1/100s of a foot.

All survey control traverses were closed or checked with double measurements and all triangulation included at least four elements of the triangle, i.e., two sides and two angles. Acceptable professional procedures were utilized at all times.

Preliminary calculations were completed in the field to check for closures. Upon completion of the field surveys, check calculations and final adjustments of all traverses and triangulation were completed and tabulated in the office. All horizontal distances for the primary control were checked within ± 0.18 feet. All elevations are based upon the adjusted elevations previously established at #BC-34 and #BC-35 (Item A and footnote 2). New elevations were established by differential leveling near the highway and by vertical angle closed traverses in the canyon and other rough areas. Vertical angle elevations were checked within ± 0.24 feet.

A diagram for the primary survey control net is included in Appendix 6 and a tabulated list of the primary control stations with their modified Arizona State Plane Coordinates, elevations and monument descriptions are included in Appendix 1.

All primary survey control stations are monumented with semi-permanent monuments ranging from 3 inch aluminum caps anchored with

rebars in concrete, spike and washers on the highway, capped rebars and uncapped rebars 10 inches to 18 inches in length, depending upon the terrain and material upon which the traverse station is located.

C. CORE HOLE LOCATIONS.

Prior to and during the road and site construction, nineteen core holes were located and ground elevations determined. Upon completion of the construction grading and drilling, surveys for final coordinates and ground (or concrete) and top of pipe elevations were determined for each core hole including eleven core holes drilled earlier. The horizontal position of all core holes were determined within ± 0.5 feet and ground or casing elevations were determined within ± 0.25 feet. Appendix 2 provides a tabulated list of the core holes, their modified Arizona State Plane Coordinates, the ground (or concrete) and the top of pipe elevations.

D. AERIAL PHOTOGRAPHY FOR PHOTOGRAMMETRIC MAPPING.

All aerial photography was flown with a Zeiss RMK-A 15/23 precision aerial mapping camera with a nominal 6 inch focal length Pleogon lens. The camera and lenses were calibrated by the Bureau of Standards, U.S. Geological Survey, February 28, 1984. All photography was flown with an approximate 60% forward lap and when required an approximate 30% side lap for adjacent flight lines. The black and white and color aerial photography for the 1" = 200' scale mapping was flown 6,000 feet above mean ground elevation resulting in an approximate 1" = 1,000' photo negative scale. The black and white aerial photography for the 1" = 50' topographic mapping at the dam site and the construction area was flown in three flight lines at an elevation of 1,500 feet above mean ground

elevation resulting in an approximate 1" = 250 ft. photo negative scale. Most of the aerial photography was flown on June 9, 1986 with one subsequent flight required due to air turbulence and cloudiness in the project area. Color aerial oblique photographs were also taken of the project area as shown in Appendix 3.

The aerial photography was flown to obtain the maximum sun angle and the best light in the canyons for the photogrammetric mapping. Prior to the aerial photography, most of the primary and the secondary horizontal and vertical survey control stations were located, set and paneled as required for the photogrammetric mapping.

E. RESERVOIR AREA TOPOGRAPHIC MAP.

All photogrammetric mapping is based upon the primary horizontal and vertical survey control net described in Item B above, supplemented by secondary control surveys and analytical aerotriangulation. The secondary control survey point coordinates are tabulated in Appendix 4. All map compilation was completed with a Wild AG-1 second order stereo compiler with a C factor of 1800. 4/

Using the 1" = 1,000 ft. aerial photography flown at 6,000 ft. above mean ground elevation and compiling 5.0 ft. contours, the map was compiled with a C factor of 1,200 which is substantially below the maximum allowable C factor of 1,800 for this Wild stereo compiler. Map accuracies should be well within the photogrammetric industry

4/ C Factor = $\frac{\text{Height of aerial photography above mean ground}}{\text{Contour interval}}$

standards. 5/

Map compilation for the reservoir and adjacent area was at a scale of 1" = 200 ft. with 5.0 ft. contours (with supplemental 2.5 ft. contours in the more level areas). This map was reduced to 1" = 400 ft., a copy of which is attached, marked Appendix 7.

Special effort was extended to compile all topographic detail visible in the aerial photography. The map title block was omitted and may be added later when the engineering drawings are completed.

F. DAM SITE AND CONSTRUCTION AREA TOPOGRAPHIC MAPS.

All photogrammetric mapping is based upon the primary horizontal and vertical survey control net described in Item B above, supplemented by secondary control surveys and analytical aerotriangulation. The secondary control survey point coordinates are tabulated in Appendix 4. All map compilation was completed with a Wild AG-1 second order stereo compiler with a C factor of 1800 (see footnote 4).

Using the 1" = 250 ft. aerial photography flown 1,500 ft. above mean ground elevation and compiling 2.0 ft. contours, this map was compiled with a C factor of 750 which is less than one half of the maximum allowable C factor of 1,800 for this Wild stereo compiler. Map accuracies should be well within the photogrammetric industry standards. 5/

5/ Generally accepted photogrammetric industry mapping standards in clear, visible areas on the photographs include:

a. Ninety percent of contours should be plotted to within one half contour interval accuracy.

b. The remaining ten percent of the contours should be plotted to within one contour interval accuracy.

c. Specific identifiable points that have clear lines of definition should be plotted to within one fortieth of the map scale.

Map compilation for the proposed Miner Flat Dam Site and construction area was at a scale of 1" = 50 ft. with 2.0 ft. contours (with supplemental 1.0 ft. contours in the more level areas). These maps (3) were reduced to a single map sheet at a scale of 1" = 100 ft. for convenience, a copy of which is attached, marked Appendix 8.

Special effort was extended to compile all topographic detail visible in the aerial photography. The map title block was omitted and may be added later when the engineering drawings are completed.

G. AERIAL ORTHOPHOTO OF THE PROPOSED MINER FLAT DAM SITE AND RESERVOIR AREA.

The specified aerial photography on a reproducible medium was compiled from the 1" = 1,000 ft. black and white aerial photography using a Northway-Gestalt GPM-IV first order analytical orthophoto instrument. The GMP-IV is an optical electronic scanner which enlarged and rectified the aerial photographs 2.5 X to a scale of 1" = 400 ft. with a resolution of ten lines per mm. This orthophoto scale, elevations and analytical aerotriangulation were based upon the primary horizontal and vertical survey control net in Item B above.

Upon completion of the orthophoto negative, a photographic composite was made utilizing the 1" = 400 ft. topographic map from Item E above and a 133 line/dot per inch half tone screen. This resulted in an accurate 1" = 400 ft. scale orthophoto with 5.0 ft. contour lines superimposed on a screened mylar which may be reproduced by standard blueline equipment.

The map title block was omitted and may be added later when the engineering drawings are completed. A copy of this orthophoto map is attached, marked Appendix 9.

H. COMPLETE FIELD SURVEY CHECKS ON THE PROPOSED MINER FLAT DAM SITE AXIS AREA.

Field surveys were completed on the primary axis profile for the proposed dam site and profiles downstream of the proposed dam site axis at 73 ft. and 140 ft. parallel offset lines. It was not practical to survey the almost vertical or overhanging cliffs on the right or left banks of the North Fork of the White River.

The field survey data and the profile data from the 1" = 50 ft. scale maps were plotted on 1 inch grid paper for a comparison which showed:

1. In areas with good ground visibility in the aerial photographs, i.e., along the top of the cliffs and the area adjacent to the North Fork of the White River, it was found that the field surveys and the topographic map profiles were well within photogrammetric mapping specifications (see footnote 5).

2. On the talus slope which includes large rock slabs along the left bank of the river, which is also covered with a dense undergrowth up to thirty feet high, it was found that the topographic mapping differed from the field survey by from 0 ft. to 5.0 ft. in most areas. However, in two areas the difference between the surveyed profile and the topographic mapping profile were from 7.0 ft. to 10.0 ft. for a short distance.

I. PRELIMINARY LOCATION OF THE MAJOR STRUCTURES.

The primary axis for the proposed Miner Flat Dam Site has been

located, reference points set and monumented and a field survey completed to check the 1" = 50 ft. topographic mapping. The alternate axis for the proposed Miner Flat Dam Site has also been located, reference points set and monumented and a field survey completed to check the 1" = 50 ft. topographic mapping (Item H above). Appendix 5 is a tabulated list of points set on the primary and alternate dam axis with the modified State Plane Coordinates for each point, the monument elevation and a description of the monument set.

Upon completion of the engineering drawings, the location of the major structures adjacent to the proposed dam site will be checked on the ground as required.

J. PROVIDE CRITICAL ELEVATIONS AT THE PUMP SITE, DELIVERY STATION AND OTHER CRITICAL POINTS AS REQUIRED.

This work has not been commenced pending completion of the engineering drawings.

K. BLACK AND WHITE AERIAL PHOTOGRAPHY FROM CANYON DAY IRRIGATION PROJECT UPSTREAM TO BECKER CREEK.

Black and white aerial photography was also flown on June 9, 1986 over the White River and the North Fork of the White River from the Canyon Day Irrigation Project to Becker Creek. The purpose of this aerial photography was to provide photo coverage for analytical aerotriangulation and a series of digitized cross sections. This aerial photography was flown with a 60% forward lap 6,000 ft. above the mean ground elevation which results in a photo negative scale of 1" = 1,000 ft.

L. ANALYTICAL AEROTRIANGULATION AND DIGITIZED CROSS SECTIONS.

Utilizing the black and white aerial photography in the above Item

K and U.S. Geological Survey topographic work sheets (on a mylar base) analytical aerotriangulation for each flight line was completed along the White River and the North Fork of the White River between the Canyon Day project and the proposed Miner Flat Dam Site. 6/ At 31 preselected points across the river(s), a series of digitized cross sections were recorded and transferred to computer $5\frac{1}{4}$ inch floppy disks.

The black and white aerial photography, the U.S.G.S. topographic work sheets and other data has been prepared and additional river cross sections can be digitized when required.

6/ The aerotriangulation was completed with a Wild BC-2 analytical compiler using Bundled Refinement Aerial Triangulation System (BRATS) software. The aerotriangulation flight line elevation residuals averaged 0.50 ft. which resulted in all points within the analytical triangulation area being well within the specified ± 1.0 ft. differential between cross sections.

PRIMARY SURVEY CONTROL COORDINATES AND ELEVATONS
JULY 30, 1986

JOB #: 85202 PROJECT: Miner Flat Dam Site, Arizona State, Zone E

STATION	NORTHING	EASTING	TRIG. ELEVATION MON.	GROUND	MONUMENT DESCRIPTION
BC-4	1,087,278.65	580,735.58	6,407.20'	6,406.8'	#5 rebar w/alum cap
BC-5	1,087,689.96	575,773.24	6,180.30'	6,180.3'	#5 rebar
BC-34	1,086,475.605	576,538.813	6,081.51'	6,082.1'	3" alum cap set in conc.
BC-34A	1,086,474.177	576,543.080	6,082.52'	6,082.5'	Spk. set in w'ly. edge of pvmnt.
BC-35	1,085,033.135	575,948.183	6,055.55'	6,056.3'	3" alum. cap set in conc.
BC-35A	1,085,053.392	575,975.590	6,057.60'	6,057.6'	Spk. set in w'ly. edge of pvmnt.
MF-1	1,086,015.495	576,634.435	6,072.00'	6,072.0'	60d spk. in rock
MF-2	1,086,326.994	578,113.691	6,102.23'	6,102.0	#5 rebar
MF-3	1,085,271.898	576,271.221	6,060.70'	6,059.9'	#5 rebar w/alum cap
MF-4	1,086,080.910	577,223.212	6,088.80'	6,088.3'	#5 rebar w/alum cap
MF-5	1,085,744.838	577,377.031	5,931.20'	5,931.2'	#5 rebar w/alum cap
MF-6	1,085,798.220	576,925.344	6,079.80'	6,079.6'	#5 rebar w/alum cap
MF-7	1,085,594.787	577,030.034	6,122.40'	6,122.4'	#5 rebar w/alum cap
MF-8	1,085,120.534	577,075.954	6,168.68'	6,167.5'	4" squared stump w/spk. & tin

PRIMARY SURVEY CONTROL COORDINATES AND ELEVATIONS
JULY 30, 1986

JOB #: 8502A PROJECT: Miner Flat Dam Site, Arizona State, Zone E

STATION	NORTHING	EASTING	ELEVATION TRIG. MON.	ELEVATION GROUND	MONUMENT DESCRIPTION
MF-9	1,084,717.318	577,457.604	6,239.45'	6,238.8'	#5 rebar
MF-10	1,085,639.537	577,907.943	5,944.58'	5,944.3'	#5 rebar
MF-11	1,085,862.748	578,340.194	5,947.02'	5,946.7'	#5 rebar
MF-12	1,086,116.974	578,318.504	5,952.66'	5,952.7'	70d spk. & tin
MF-13	1,086,405.852	578,398.946	5,957.30'	5,956.9'	#5 rebar
MF-14	1,086,597.637	578,100.301	6,103.61'	6,103.4'	#5 rebar
MF-15	1,086,323.298	578,069.094	6,102.49'	6,101.6'	#5 rebar
MF-16	1,086,272.556	577,145.491	6,089.90'	6,089.9'	#5 rebar
MF-17	1,085,573.198	578,670.739	6,018.86'	6,018.8'	#5 rebar
MF-18	1,088,148.244	577,241.897	6,126.84'	6,126.8'	3" alum cap set in conc.
MF-19	1,087,002.711	576,760.379	6,091.86'	6,092.6	3" alum cap set in conc.
MF-20	1,084,014.305	575,849.682	6,045.00'	6,045.0'	Spk in w'ly edge of hwy.
MF-21	1,086,199.211	576,425.642	6,079.49'	6,079.5'	Spk in w'ly edge of pvmt.
MF-22	1.086,479.953	575,772.730	6,089.48'	6,089.2'	#5 rebar
MF-23	1,086,777.148	575,081.553	6,126.13'	6,125.9'	#5 rebar
MF-24	1,086,258.544	576,468.582	6,080.10'	6,080.1'	Spk & wash center hwy.

PRIMARY SURVEY CONTROL COORDINATES AND ELEVATIONS
JULY 30, 1986

JOB #: 8502B PROJECT: Miner Flat Dam Site, Arizona State, Zone E

STATION	NORTHING	EASTING	ELEVATION TRIG. MON.	ELEVATION GROUND	MONUMENT DESCRIPTION
MF-25	1,085,821.464	576,452.817	6,066.71'	6,066.7'	X cut in top basalt cliff
MF-26	1,084,264.945	576,506.699	6,033.60'	6,033.6'	R/R spike
MF-27	1,083,774.245	577,310.999	5,868.75'	5,868.6'	#4 rebar on gravel bar in river
MF-28	1,083,556.017	577,630.826	5,867.29'	5,867.1'	#4 rebar
MF-29	1,088,125.141	577,914.500	6,117.00'	6,116.9'	#5 rebar
MF-30	1,087,948.066	578,439.977	6,102.60'	6,102.5'	#5 rebar
MF-35	1,085,523.830	575,250.784	6,082.27'	6,082.3'	#5 rebar in field
MF-36	1,089,605.628	578,223.425	6,121.66'	6,121.7'	#4 rebar 2.5' w'ly of e'ly R.O.W. fence
MF-37	1,089,708.534	578,275.319	6,123.98'	6,123.9'	#4 rebar 9.5' w'ly of e'ly R.O.W. fence
MF-38	1,090,491.546	578,670.180	6,137.48'	6,137.4'	#4 rebar on e'ly shoulder of highway
MF-39	1,090,957.734	578,905.271	6,151.17'	6,151.2'	PK nail on e'ly edge of pavement
MF-19A	1,086,999.036	576,768.030	6,093.55'	6,093.6'	Spk in w'ly edge of hwy.

CORE HOLE COORDINATES
JULY 30, 1986

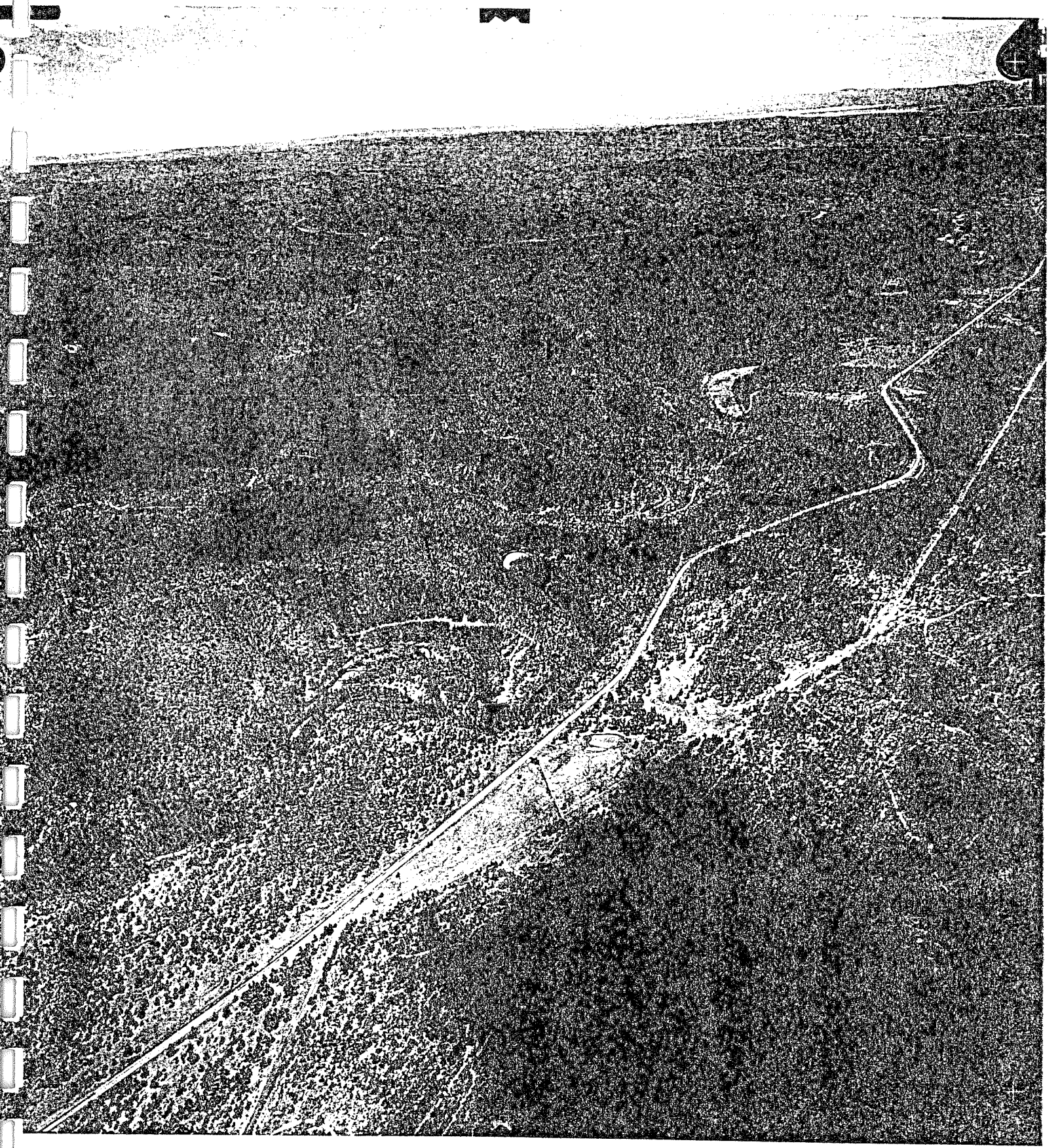
JOB #: 8502G PROJECT: Miner Flat Dam Site, Arizona State, Zone E

STATION	NORTHING	EASTING	TRIG. ELEVATIONS TOP OF GROUND OR CONCRETE	TRIG. ELEVATIONS TOP OF SURFACE PIPE OR PVC
DH-1	1,086,076.6	576,617.3	6,073.1'	--
DH-2	1,085,934.1	576,729.6	5,921.6	--
DH-3	1,085,932.3	576,731.1	5,921.6'	5,922.0'
DH-4	1,085,930.5	576,732.6	5,921.6'	--
DH-5	1,085,793.4	576,860.9	6,078.6'	6,079.7
DH-10	1,085,927.1	577,138.6	5,926.0'	--
DH-11	1,086,857.4	578,142.0	5,965.3'	5,966.7'
DH-12	1,088,693.7	578,775.2	6,107.1'	--
DH-13	1,088,366.4	578,958.6	6,102.0'	--
DH-14	1,085,359.4	576,925.5	6,096.3'	6,097.1'
DH-19	1,085,882.3	576,660.1	5,917.2'	5,919.3'
MF-102	1,085,797.9	576,852.9	6,078.2'	6,080.0'
MF-105	1,086,058.5	576,635.5	6,073.4'	6,075.6'
MF-106	1,086,059.7	576,634.4	6,073.4'	6,075.6'
MF-113	1,086,008.4	576,695.6	5,921.0'	5,922.8'
MF-116	1,085,859.6	576,908.5	5,968.7'	--
MF-117	1,086,021.3	576,686.0	5,920.6'	5,924.5' (?)

CORE HOLE COORDINATES
JULY 30, 1986

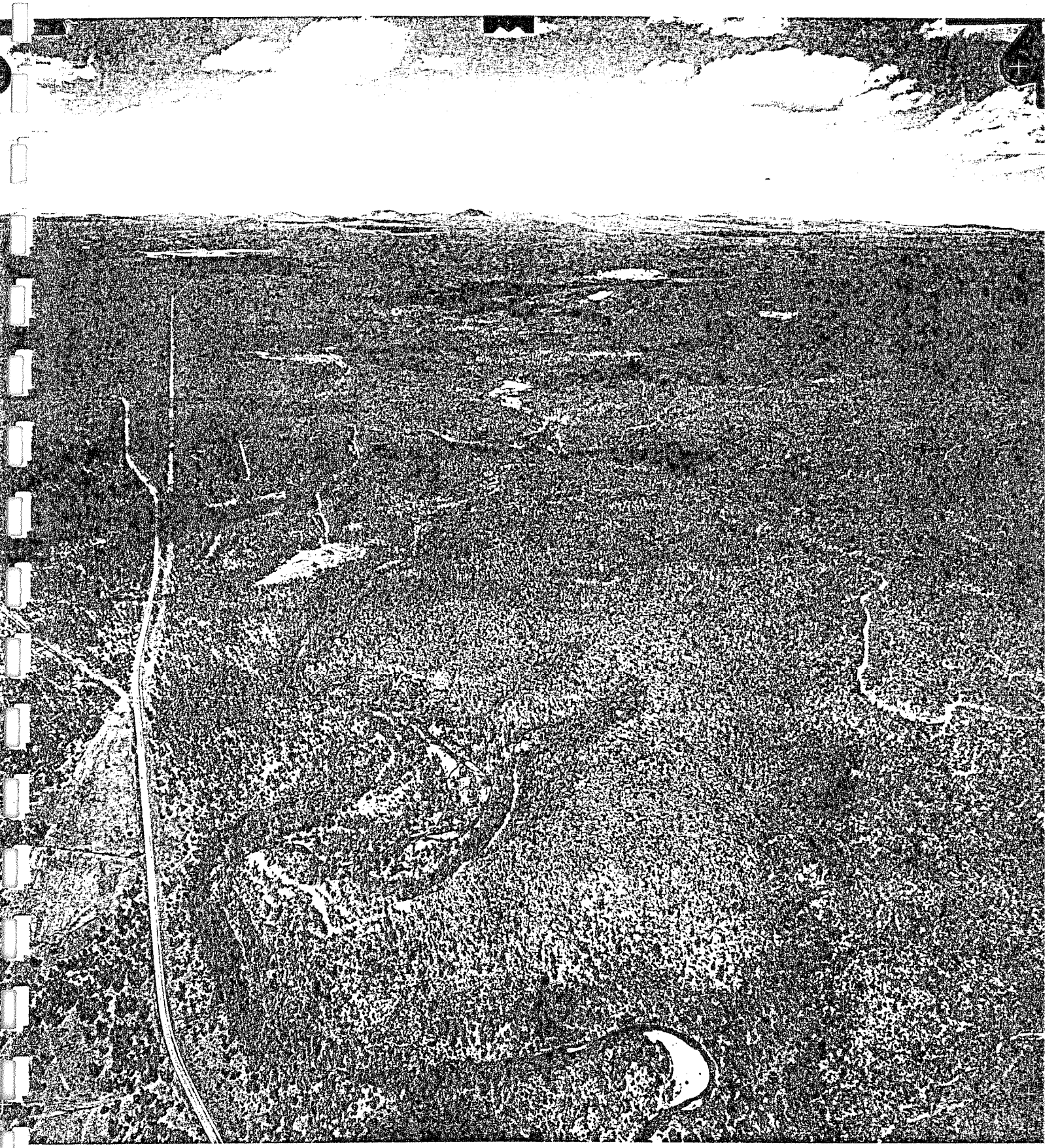
JOB #: 8502H PROJECT: Miner Flat Dam Site, Arizona State, Zone E

STATION	NORTHING	EASTING	TRIG. ELEVATIONS TOP OF GROUND OR CONCRETE	TRIG. ELEVATIONS TOP OF SURFACE PIPE OR PVC
MF-118	1,086,320.7	576,416.8	6,077.3'	6,079.1'
MF-119	1,086,816.2	576,078.1	6,094.6'	6,096.8'
MF-120	1,087,285.4	575,785.3	6,124.6'	--
MF-120A	1,086,564.7	576,232.7	6,081.4'	6,083.2'
MF-121	1,085,594.8	576,895.0	6,089.7'	6,091.8'
MF-122	1,085,644.9	576,785.0	6,074.3'	6,077.1'
MF-123	1,085,308.6	577,025.2	6,112.2 (abandoned)	--
MF-124	1,085,429.8	576,820.0	6,079.8'	6,082.0'
MF-125	1,085,444.8	576,750.0	6,070.4'	--
MF-125A	1,085,259.1	576,745.7	6,064.0'	6,066.2'
MF-126	1,085,260.0	576,930.0	6,100.0'	--
MF-127	1,086,018.9	576,698.6	(Abandoned)	--
MF-201	1,085,950.8	576,513.8	6,071.5'	--

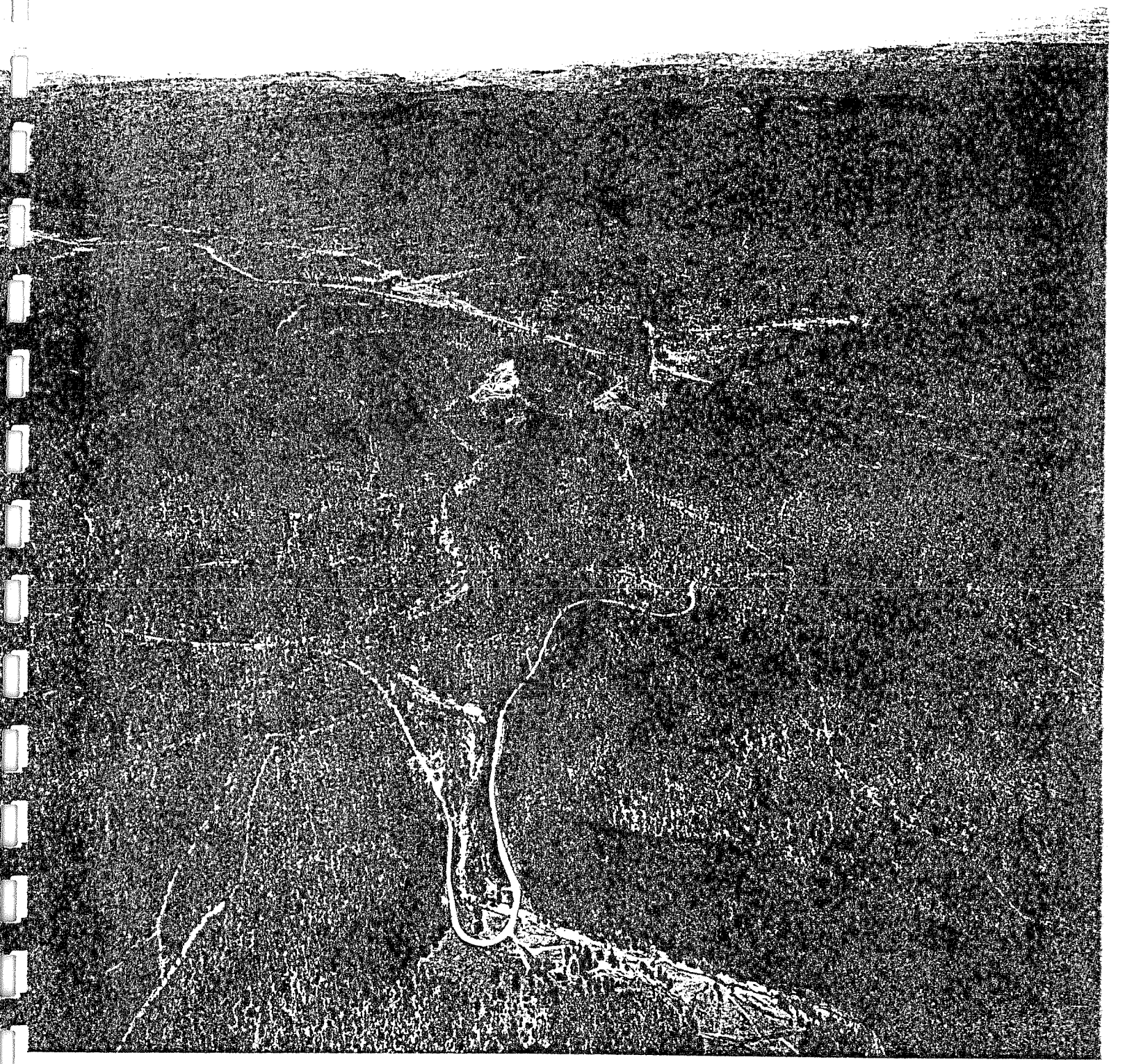


APPENDIX 3-1

PROPOSED MINER FLAT DAM SITE LOOKING TO THE SOUTHEAST



APPENDIX 3-2
PROPOSED MINER FLAT DAM SITE LOOKING TO THE NORTHEAST



APPENDIX 3-3

PROPOSED MINER FLAT DAM SITE
LOOKING FROM NORTH END OF RESERVOIR
SOUTHWESTERLY TO DAM SITE

SECONDARY SURVEY CONTROL COORDINATES AND ELEVATIONS
JULY 30, 1986

JOB #: 8502C PROJECT: Miner Flat Dam Site, Arizona State, Zone E

			TRIG. ELEVATION		MONUMENT
STATION	NORTHING	EASTING	MON.	GROUND	DESCRIPTION
AP-1A	1,087,098.595	575,880.482	6,109.85'	6,109.9'	60d spike
AP-2	1,085,302.050	576,939.874	6,098.38'	6,098.4'	30d spk & tin
AP-2A	1,087,185.551	575,621.297	6,126.41'	6,126.4	30d spk & tin
AP-3	1,085,433.305	576,764.979	--	--	1 x 2 stake
AP-3A	1,087,014.738	576,527.316	6,122.97'	6,123.0'	30d spk & tin
AP-10	1,084,999.239	574,682.782	6,076.48'	6,075.7'	30d spk in 30" diam stump
AP-11	1,090,208.978	580,354.026	6,145.51'	6,145.4'	#4 rebar
AP-12	1,090,556.349	580,827.072	6,178.34'	6,178.3'	30d spike
AP-13	1,090,866.513	586,158.290	6,075.91'	6,075.9' (5' NW'ly of SE cor)	SE cor. of bridge
AP-14	1,091,433.411	585,629.891	6,267.56'	6,267.5'	10" bridge 10-15' w'ly of basalt ledge
AP-15	1,091,435.831	585,312.550	6,271.03'	6,271.0'	30d spike
AP-16	1,091,395.217	585,035.376	6,271.01'	6,271.0'	30d spike
AP-17	1,091,259.228	584,769.887	6,268.02'	6,268.0'	30d spike
AP-18	1,091,071.020	584,515.154	6,266.46'	6,266.5'	30d spike
AP-19	1,091,401.296	584,256.021	6,266.17'	6,266.2'	30d spike
AP-20	1,091,569.930	584,308.360	6,267.74'	6,267.7'	30d spike

SECONDARY SURVEY CONTROL COORDINATES AND ELEVATIONS
JULY 30, 1986

JOB #: 8502D PROJECT: Miner Flat Dam Site, Arizona State, Zone E

STATION	NORTHING	EASTING	ELEVATION TRIG. MON.	ELEVATION GROUND	MONUMENT DESCRIPTION
AP-21	1,091,951.873	584,150.083	6,265.40'	6,265.4'	30d spike
AP-22	1,091,959.863	583,927.446	6,262.27'	6,262.3'	30d spike
AP-23	1,091,907.367	583,576.592	6,255.73'	6,255.7'	30d spike
AP-24	1,091,707.306	583,251.110	6,251.38'	6,251.4'	30d spike
AP-25	1,091,963.901	582,907.496	6,247.72	6,247.7'	30d spike
AP-26	1,091,574.942	582,696.525	6,241.14'	6,241.1'	30d spike
AP-27	1,090,973.308	581,510.421	6,208.62	6,208.6	P-K nail
AP-28	1,090,847.063	581,108.775	6,190.69'	6,290.7'	30d spike
AP-29	1,085,713.934	576,836.533	6,077.31'	6,077.3'	30d spk & tin
AP-30	1,085,595.670	576,791.024	6,077.03'	6,077.0'	30d spk & tin

HORIZONTAL & VERTICAL PHOTO CONTROL POINT COORDINATES
JULY 30, 1986

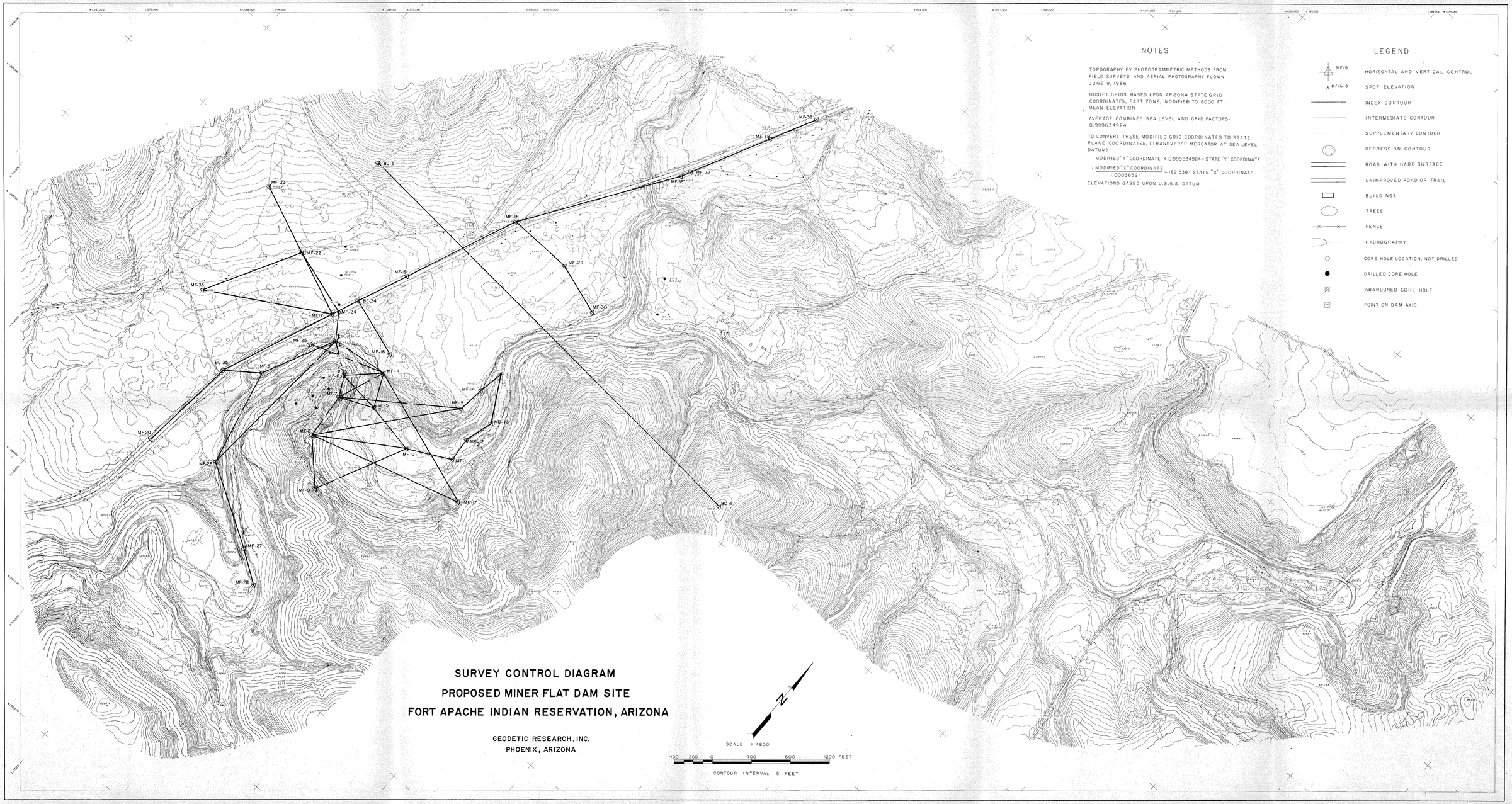
JOB #: 8502E PROJECT: Miner Flat Dam Site, Arizona State, Zone E

STATION	NORTHING	EASTING	TRIG. MON.	ELEVATION GROUND	MONUMENT DESCRIPTION
HVC-1	1,083,469.016	575,777.813	--	6,020.9'	W'ly edge of pavement
HVC-2	1,084,201.666	574,098.500	6,043.97'	6,044.0' (W of P/P)	Lg P/P at angle of P/L
HVC-3	1,084,220.895	574,078.045	6,048.64'	6,048.5'	#5 rebar 4' w'ly of sm P/P at P/L angle pt
HVC-4	1,089,063.091	575,365.415	6,388.37	6,388.2'	#5 rebar
HVC-5	1,090,591.271	578,642.860	6,139.06'	6,139.1'	#4 rebar 4.5' e'ly of W. ROW fence
HVC-6	1,090,273.552	586,110.993	6,245.97'	6,245.7'	#4 rebar 15-20' s'ly of basalt ledge
HVC-7	1,091,380.045	585,516.788	6,270.07'	6,270.0'	#4 rebar
HVC-8	1,091,515.817	582,295.066	6,227.47'	6,227.5'	#4 rebar
HVC-9	1,083,473.052	577,648.277	5,869.64'	5,869.6'	Bridge spike & tin
HVC-10	1,083,929.717	577,194.628	5,877.23'	5,877.2'	3" alum cap
HVC-11	1,086,356.613	578,362.098	5,958.18'	5,956.2'	top of rock 2.0' ab. gr.
HVC-12	1,086,481.439	578,340.252	--	5,959.1'	Gr. elev. 10' e'ly of lg. rock

MINER FLAT DAM AXIS COORDINATES
JULY 30, 1986

JOB #: 8502F PROJECT: Miner Flat Dam Site, Arizona State, Zone E

STATION	NORTHING	EASTING	TRIG. MON.	ELEVATION GROUND	MONUMENT DESCRIPTION
Primary Dam Axis					
A-1					
A-2					
A-3	1,085,729.699	576,909.786	--	--	#6 rebar w/ 3" alum cap
A-4	1,085,779.818	576,867.973	--	--	60d spike
A-5	1,085,804.006	576,847.794	--	--	60d spike
A-6	1,085,820.761	576,833.816	6,072.94'	6,072.9	Bridge spk
A-7					
A-8					
A-9	1,086,031.029	576,658.394	6,070.89'	6,070.9'	Temp. pt.
Secondary Dam Axis					
A-10	1,085,783.220	576,925.347	--	--	60d spike
A-11					
A-12					
A-13					
A-14					
A-15					
A-16					
A-17					
A-18					
A-19	1,085,938.150	576,544.911	6,066.08'	6,066.1'	X at top of cliff
A-20	1,085,941.846	576,535.835	--	--	40d spike
A-21	1,085,946.749	576,523.795	6,069.69'	6,069.7'	X on rock



SURVEY CONTROL DIAGRAM
PROPOSED MINER FLAT DAM SITE
FORT APACHE INDIAN RESERVATION, ARIZONA

GEODETIC RESEARCH, INC.
PHOENIX, ARIZONA

SCALE 1:4800

400 200 0 400 800 1200 FEET

CONTOUR INTERVAL 5 FEET

NOTES

TOPOGRAPHY BY PHOTOGRAMMETRIC METHODS FROM FIELD SURVEYS AND AERIAL PHOTOGRAPHY FLOWN JUNE 9, 1986

1000 FT. GRIDS BASED UPON ARIZONA STATE GRID COORDINATES, EAST ZONE, MODIFIED TO 6000 FT. MEAN ELEVATION

AVERAGE COMBINED SEA LEVEL AND GRID FACTOR: 0.999634924

TO CONVERT THESE MODIFIED GRID COORDINATES TO STATE PLANE COORDINATES, (TRANSVERSE MERCATOR AT SEA LEVEL DATUM):

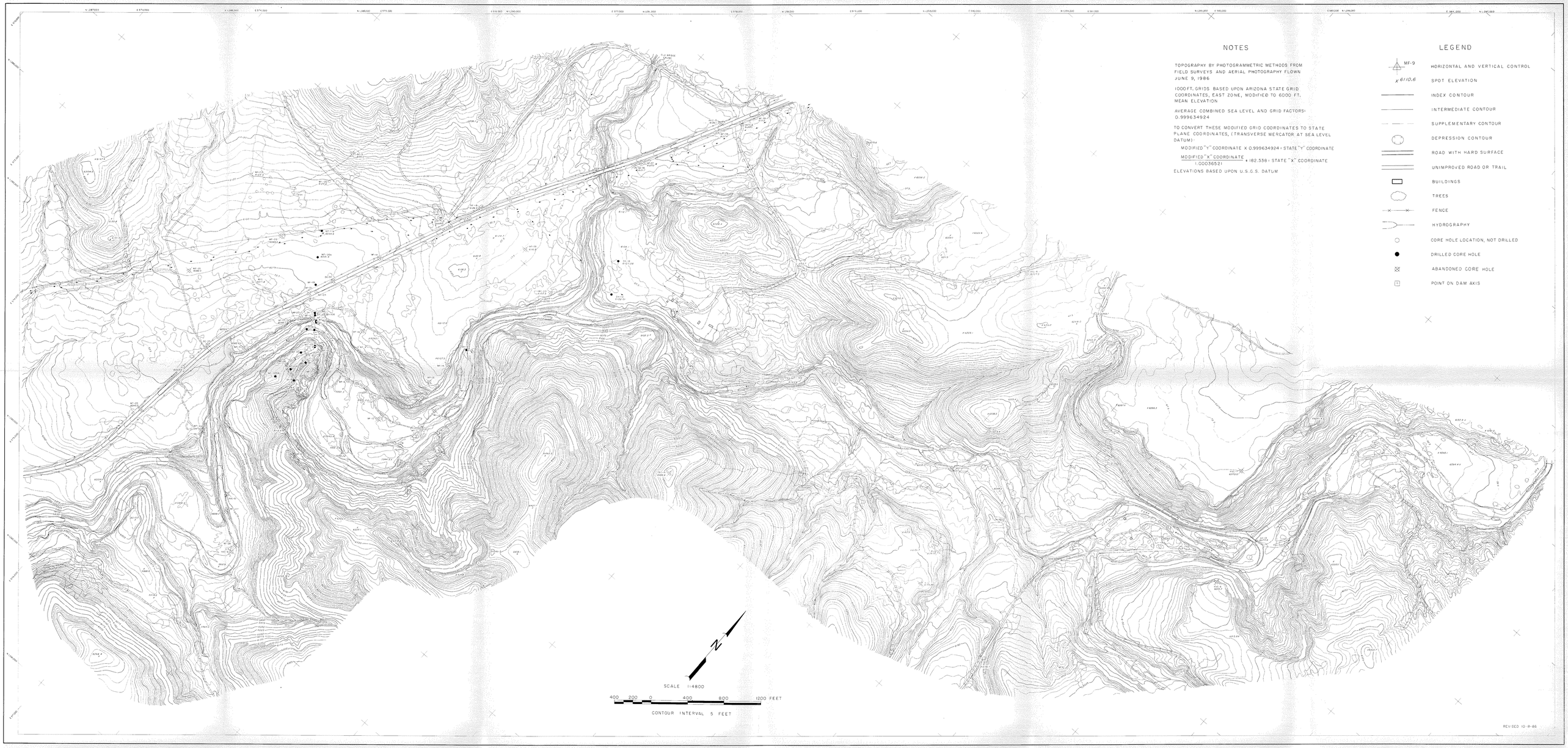
MODIFIED "Y" COORDINATE X 0.999634924 = STATE "Y" COORDINATE

$$-\frac{\text{MODIFIED "X" COORDINATE}}{1.00036521} + 182.538 = \text{STATE "X" COORDINATE}$$

ELEVATIONS BASED UPON U.S.G.S. DATUM

LEGEND

- MF-9 — HORIZONTAL AND VERTICAL CONTROL
- x 6110.6 — SPOT ELEVATION
- INDEX CONTOUR
- INTERMEDIATE CONTOUR
- SUPPLEMENTARY CONTOUR
- DEPRESSION CONTOUR
- ROAD WITH HARD SURFACE
- UNIMPROVED ROAD OR TRAIL
- BUILDINGS
- TREES
- FENCE
- HYDROGRAPHY
- CORE HOLE LOCATION, NOT DRILLED
- DRILLED CORE HOLE
- ABANDONED CORE HOLE
- POINT ON DAM AXIS



NOTES

TOPOGRAPHY BY PHOTOGRAMMETRIC METHODS FROM FIELD SURVEYS AND AERIAL PHOTOGRAPHY FLOWN JUNE 9, 1986

1000 FT. GRIDS BASED UPON ARIZONA STATE GRID COORDINATES, EAST ZONE, MODIFIED TO 6000 FT. MEAN ELEVATION

AVERAGE COMBINED SEA LEVEL AND GRID FACTORS: 0.999634924

TO CONVERT THESE MODIFIED GRID COORDINATES TO STATE PLANE COORDINATES, (TRANSVERSE MERCATOR AT SEA LEVEL DATUM):

MODIFIED "Y" COORDINATE X 0.999634924 + STATE "Y" COORDINATE

$$\frac{\text{MODIFIED "X" COORDINATE}}{1.00036521} + 182.538 + \text{STATE "X" COORDINATE}$$

ELEVATIONS BASED UPON U.S.G.S. DATUM

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- MF-9 — HORIZONTAL AND VERTICAL CONTROL
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- FENCE
- HYDROGRAPHY
- CORE HOLE LOCATION, NOT DRILLED
- DRILLED CORE HOLE
- ABANDONED CORE HOLE
- POINT ON DAM AXIS

N

SCALE 1:4800

400 200 0 400 800 1200 FEET

CONTOUR INTERVAL 5 FEET

REVISED 10-8-86

SCALE 1 INCH = 100 FEET

CONTOUR INTERVAL 2 FEET

LEGEND

- MF-9 — HORIZONTAL AND VERTICAL CONTROL
- x 6110.6 — SPOT ELEVATION
- INDEX CONTOUR
- INTERMEDIATE CONTOUR
- SUPPLEMENTARY CONTOUR
- DEPRESSION CONTOUR
- ROAD WITH HARD SURFACE
- UNIMPROVED ROAD OR TRAIL
- BUILDINGS
- TREES
- FENCE
- HYDROGRAPHY
- CORE HOLE LOCATION, NOT DRILLED
- DRILLED CORE HOLE
- ABANDONED CORE HOLE
- POINT ON DAM AXIS

NOTES

TOPOGRAPHY BY PHOTOGRAMMETRIC METHODS FROM FIELD SURVEYS AND AERIAL PHOTOGRAPHY FLOWN JUNE 9, 1986

250 FT. GRIDS BASED UPON ARIZONA STATE GRID COORDINATES, EAST ZONE, MODIFIED TO 6000 FT. MEAN ELEVATION

AVERAGE COMBINED SEA LEVEL AND GRID FACTORS= 0.999634924

TO CONVERT THESE MODIFIED GRID COORDINATES TO STATE PLANE COORDINATES, (TRANSVERSE MERCATOR AT SEA LEVEL DATUM):

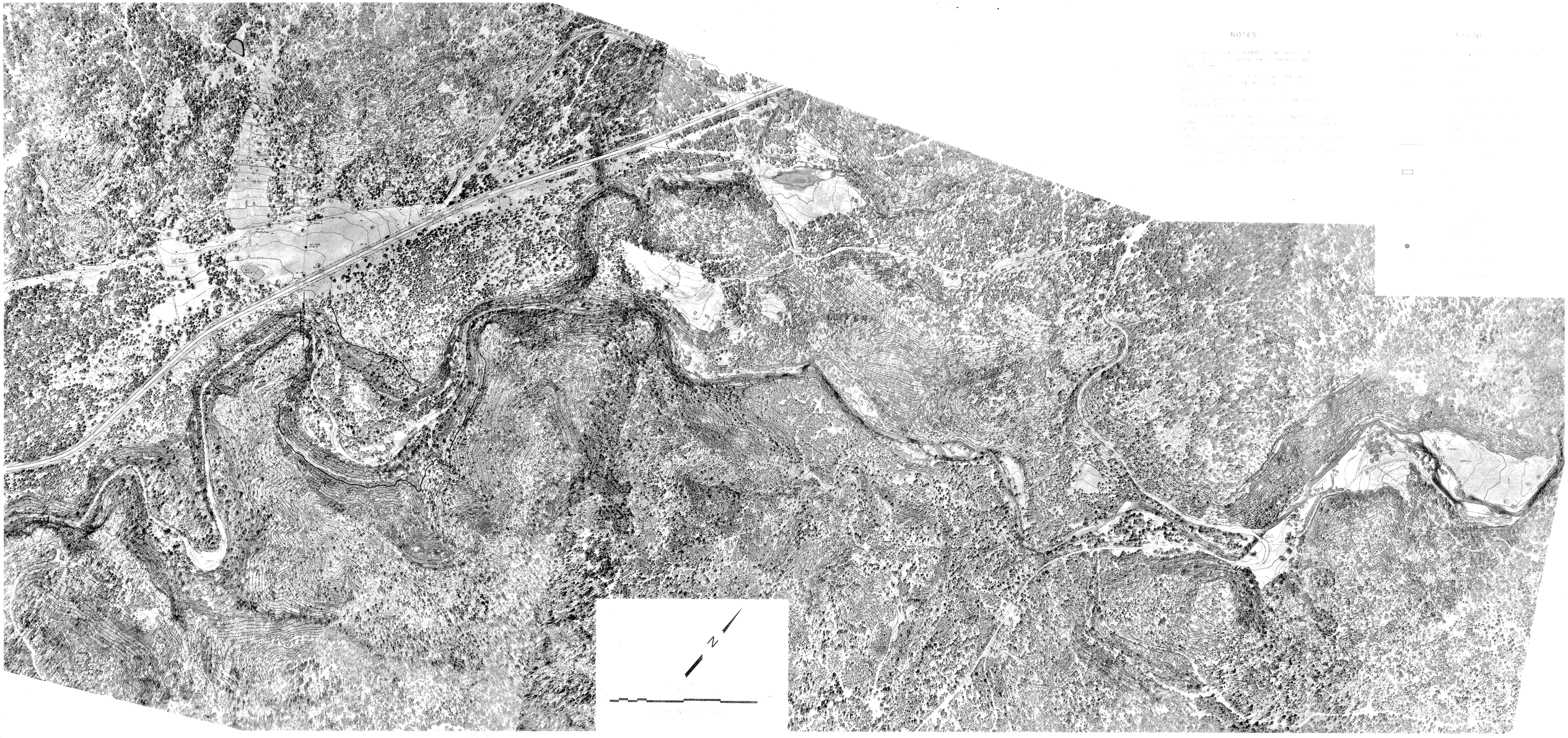
MODIFIED "Y" COORDINATE X 0.999634924 = STATE "Y" COORDINATE

$$\frac{\text{MODIFIED "X" COORDINATE}}{1.00036521} + 182.538 = \text{STATE "X" COORDINATE}$$

ELEVATIONS BASED UPON U.S.G.S. DATUM

ARIZONA STATE HIGHWAY 73

NORTH FORK WHITE RIVER


NOTES
LEGEND
N

APPENDIX S

Design Memorandum, Miner Flat Dam, February 1987

MORRISON-MAIERLE, INC.,
VOLUMES I THRU V OF VI

FEBRUARY 2007

017962-6002

DESIGN MEMORANDUM

MINER FLAT DAM

DECEMBER, 1986

[REVISED FEBRUARY 1987]

WHITE MOUNTAIN APACHE TRIBE

WHITERIVER, ARIZONA

VOLUME 2 OF 6

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604
CORPORATE OFFICE
HELENA, MONTANA
BRANCH OFFICES
BILLINGS, MONTANA
KALISPELL, MONTANA
BOZEMAN, MONTANA
RIVERTON, WYOMING

MINER FLAT DAM
DESIGN MEMORANDUM

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	B.	General Description of Local Conditions	AB - 1
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		General Map B	Sht A2
		Surface Data - Topog. Map A	Sht C1
		Surface DAta - Topog. Map B	Sht C2
		Surface Data - Topog. Map C	Sht C3
	C.	Surface Data by Geodetic Research, Inc.	
Two	D.	Foundation Data	
		D.1 - Geologic Data	
		D.2 - Foundation Stability Studies	
		D.3 - Seepage Model	
	Photos	Right Abutment	Fig D.1.1
		Left Abutment	Fig D.1.2
		Downstream View	Fig D.1.3
		Upstream View	Fig D.1.4
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		Foundation Data B	Sht D1A
		Bore Logs	Sht D2
		Bore Logs	Sht D3
		Bore Logs	Sht D4
		Bore Logs	Sht D5
		Bore Logs	Sht D6
	E.	Construction Material Data	
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MINER FLAT DAM
DESIGN MEMORANDUM

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	J.2	Dam Design and Stability Analysis	J - 3
	J.3	Dam Thermal Stress Studies	J - 13
	J.4	Dam Aggregate and Mix Design	J - 18
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\MINER FLAT DAM\INDEX

DESIGN MEMORANDUM

SECTION D

D.1 GEOLOGIC DATA

Geologic and engineering geologic field investigations have been conducted in the dam and reservoir area since February of 1982, principally by Mineral Systems, Inc. (MSI). The investigations included initial site mapping, geologic mapping, and preliminary core hole drilling in 1982. Site mapping, joint surveys, geophysical surveys, core drilling, in-situ permeability testing, and piezometer installation continued in 1983. Additional investigations including core drilling, in-situ permeabilty tests, piezometer installation, in-situ borehole deformation testing, and laboratory tests of materials strengths and properties were accomplished in 1985 and 1986. The information and data base resulting from the field investigations of the Miner Flat dam site are provided by three reports which are included by reference in this Design Memorandum. The three reports are:

1. "Engineering Geology and Mapping, Miner Flat Damsite and Reservoir, White Mountain Apache Tribe, Preliminary Report", Mineral Systems, Inc., August 1982.

2. "Preliminary Report, Engineering Geology, Miner Flat Dam Site, White Mountain Apache Reservation", Mineral Systems, Inc., November 1983.

3. "Preliminary Report, Engineering Geology, Miner Flat Dam Site, White Mountain Apache Reservation", Vols. I and II, Mineral Systems, Inc., July 1986.

The latter MSI report, dated July 1986, presents the most recent field investigations including information developed in response to the "Review and Evaluation of Dam Site Investigations" prepared by Harza Engineering Company. The earlier MSI reports prepared in 1982 and 1983 are included by reference because they include borehole logs, packer test results, and other data not reproduced in the 1986 MSI report.

Previous geologic investigations of the Miner Flat damsite and reservoir area consisted of a generalized regional geologic map of Navajo and Apache Counties, Arizona. The previous investigations did not present site specific data for the damsite and reservoir area. The 1986 MSI report provides geologic maps of the reservoir and damsite. The locations of foundation and abutment exploration drill holes and the locations of other types of geological and geophysical investigations of the dam site and reservoir area are shown on the geological maps. The 1986 MSI report also provides geologic cross sections through the dam site.

A series of color photographs are provided on Figures D.1.1 through D.1.4 to show the abutments at the damsite, the aggregate source area immediately upstream from the damsite, and the area immediately downstream from the damsite. Likewise, the drawings on Sheets D1 through D6 are presented to summarize herein the foundation geology, testing and sample locations, and drill hole logs.

FIGURE D.1.1 - RIGHT ABUTMENT

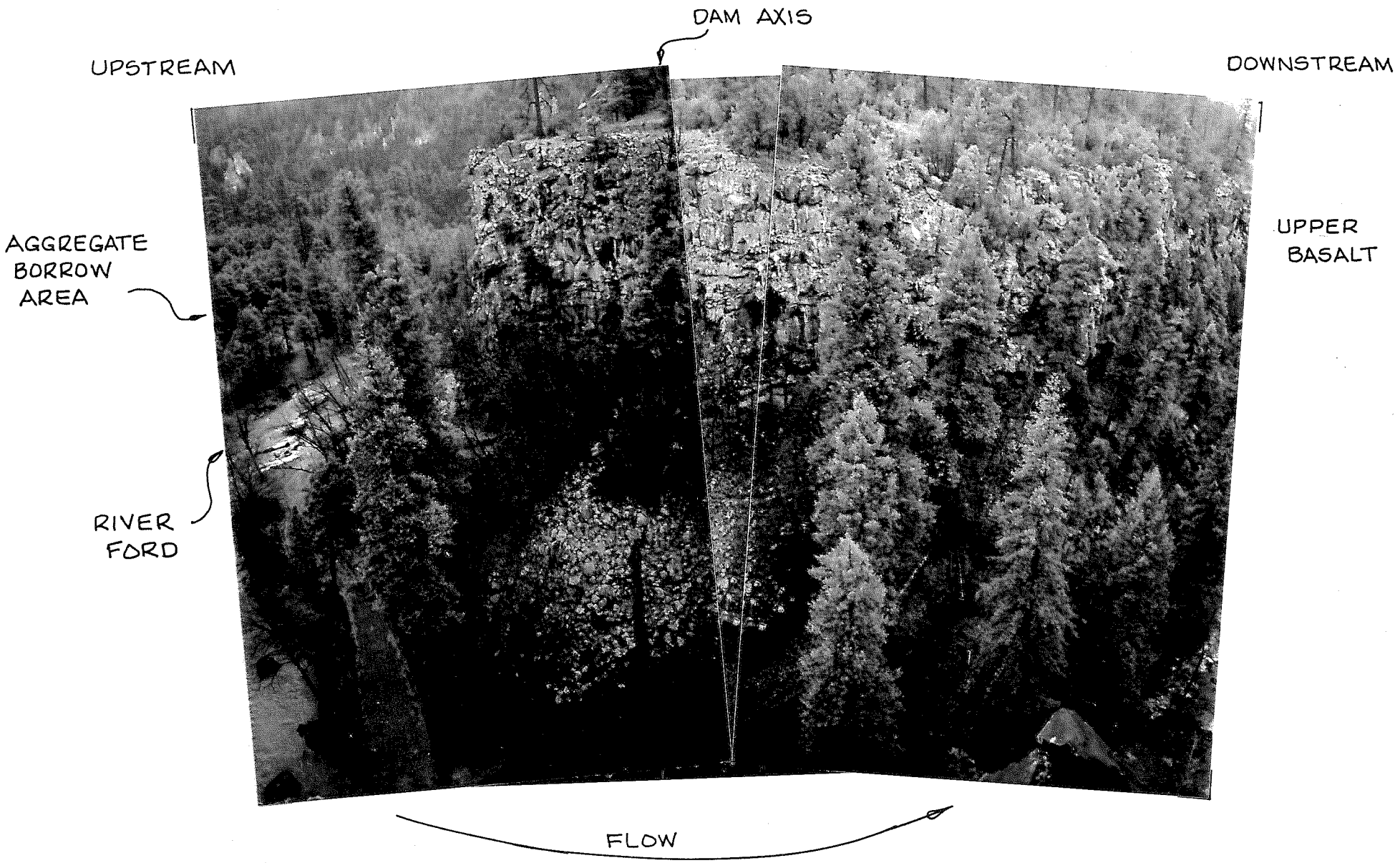


FIGURE D.1.2 – LEFT ABUTMENT

LEFT

RIGHT

FLOW

FIGURE D.1.3 - DOWNSTREAM

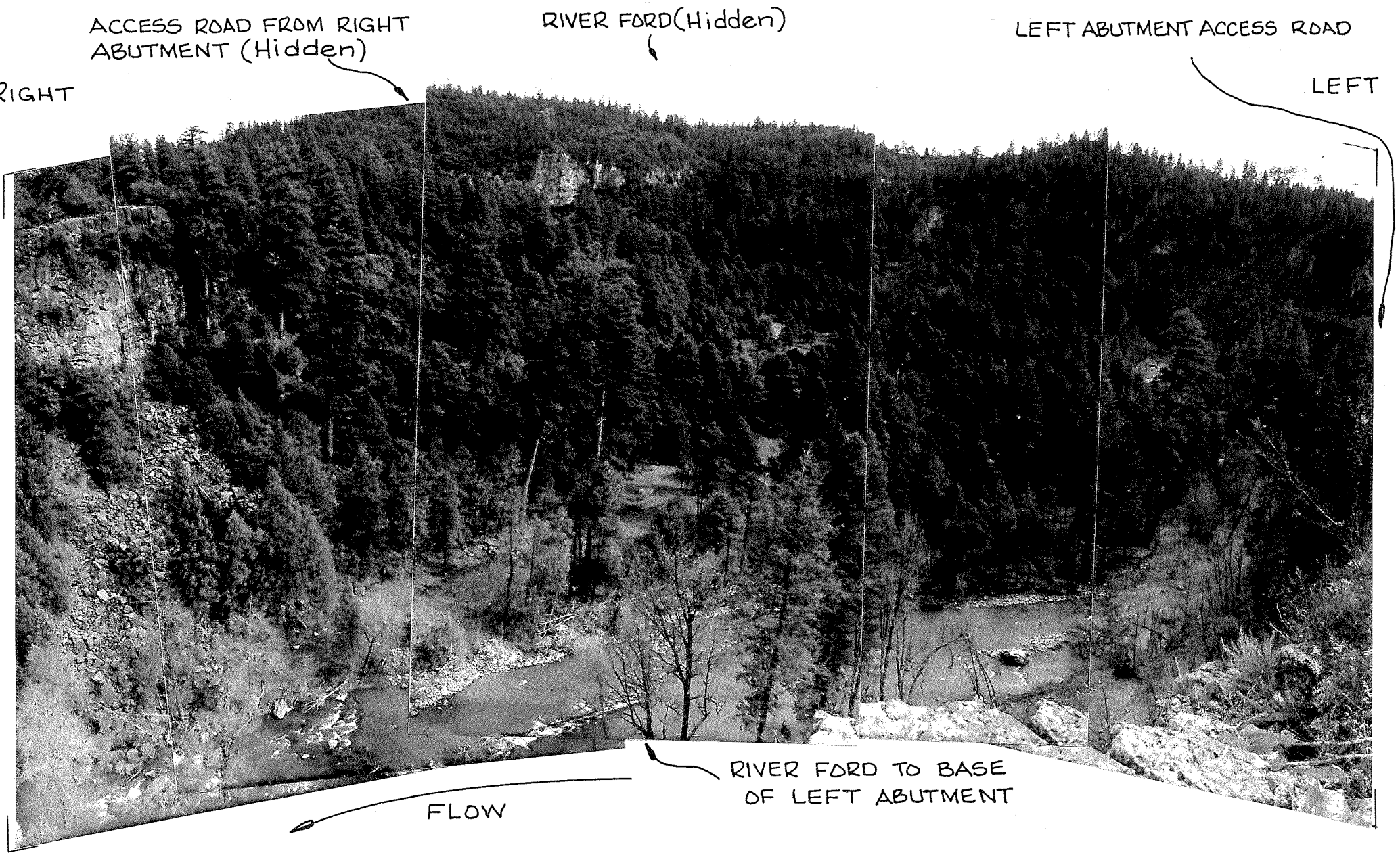


FIGURE D.1.4 - UPSTREAM (Aggregate Borrow/Rock Disposal Area)

EXPLANATION

af
Artificial fill

ALLUVIAL DEPOSITS

Qal
ALLUVIUM
(Boulders, cobbles, gravel, sand and silt along present stream channels.)

Qt_2
TERRACES
(Alluvium of boulders, cobbles, gravel, sand and silt; subscript relative age, 1 being the oldest.)

COLLUVIAL DEPOSITS

Qc
COLLUVIUM
(Derived from weathering of bedrock, letter symbols in parentheses are symbols for bedrock units from which colluvium derived.)

Qta
TALUS
(Letter symbols in parentheses are symbols for bedrock units from which talus derived.)

BEDROCK

QTg
QUATERNARY-TERTIARY GRAVEL
(No outcrops)

QTb
BASALT
(Massive to vesicular and scoriatious basalt)

Ps
SUPAI FORMATION
(Sandstone, fine-grained rounded, well sorted, cross-stratified, massive bedded. Lower part interbedded with siltstone)

70
Contact, showing dip
(Dashed where approximately located)

5
Strike and dip of beds

75
Strike and dip of joints

Strike of vertical joint

DH-3 5920.8
Drill Hole

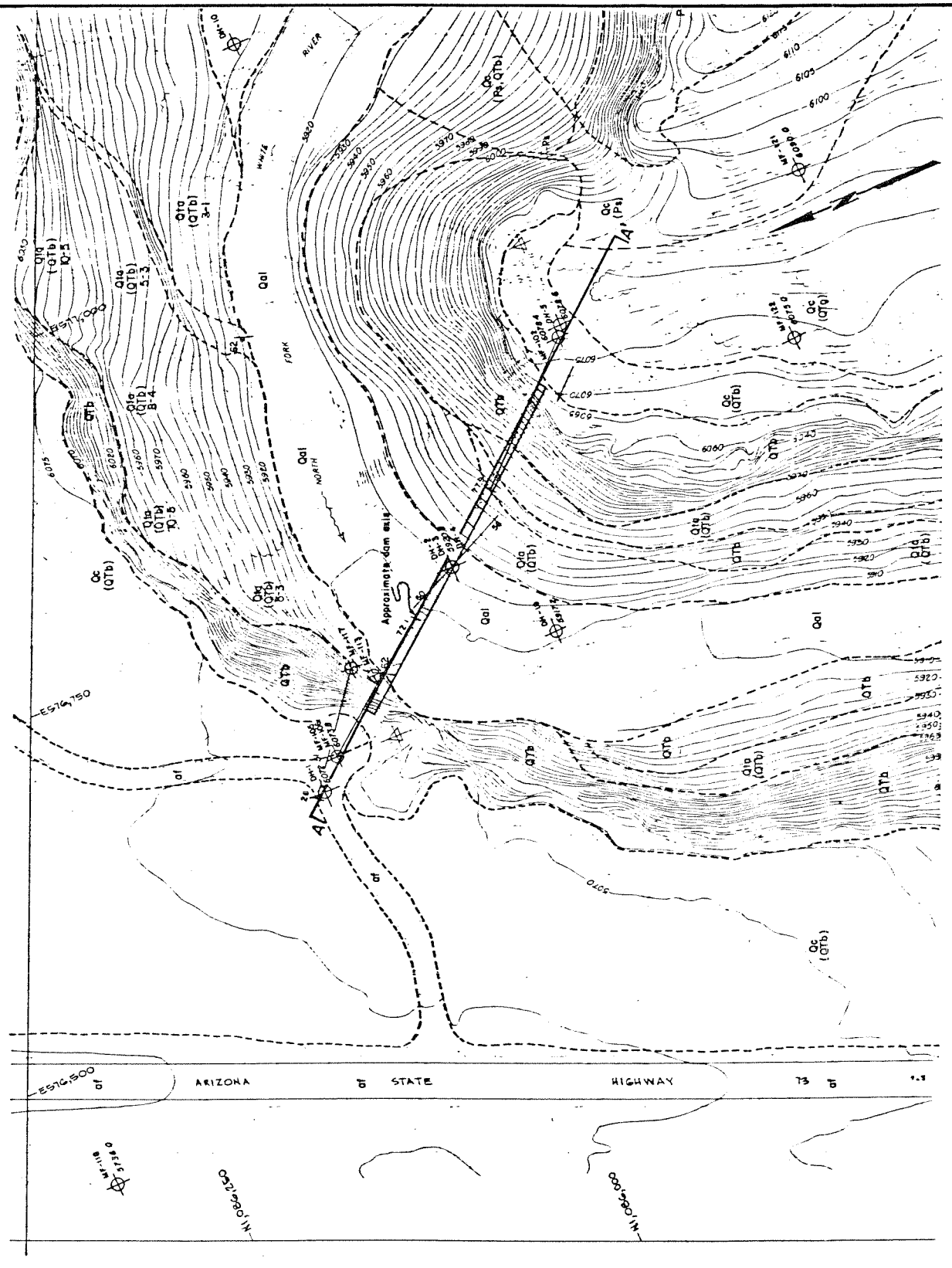
DH-2 60 1 5920.8
Surface drill hole with inclination

RA-2H
Line of detail survey

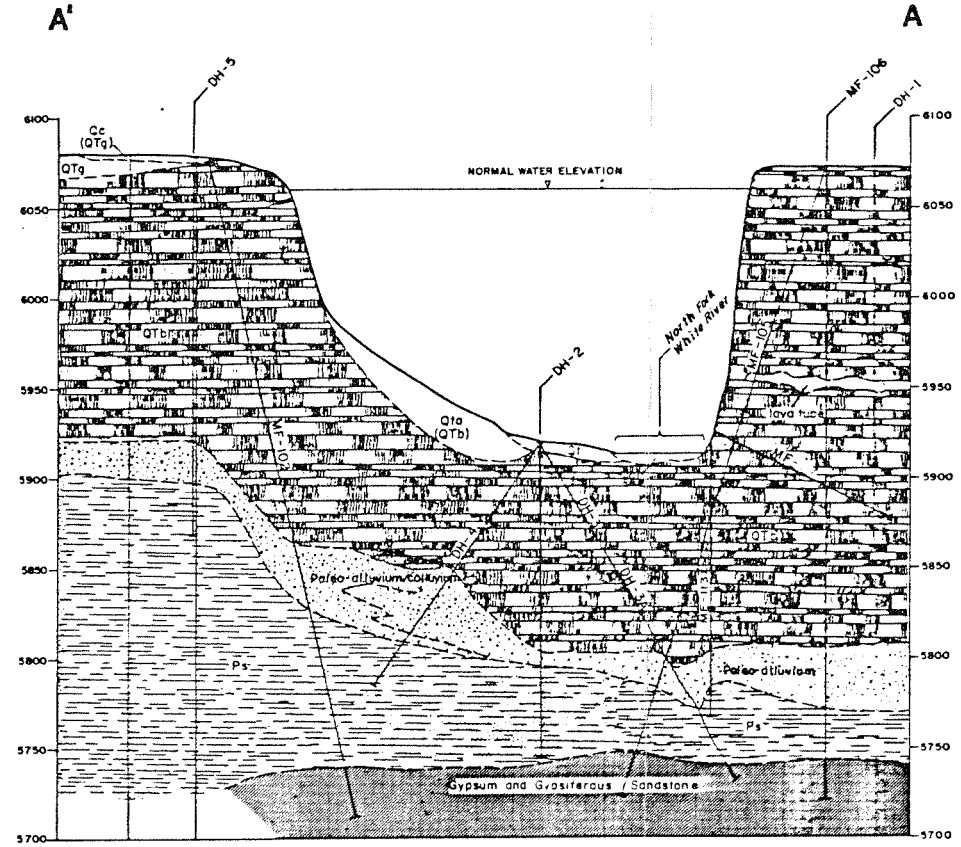
A A'
Line of section

Geology by Charles S. Robinson

Mineral Systems Inc
Golden, Colorado, June 1983



GEOLOGIC PLAN VIEW



GEOLOGIC SECTION A'-A

EXPLANATION

Qal
Alluvium

Qt
Terrace

Qc
Colluvium
(Bedrock source shown by symbol for rock unit)

Qta
Talus
(Bedrock source shown by symbol for rock unit)

QTg
Quaternary-Tertiary Gravel

QTb
Basalt

Ps
Supai Formation
(Showing gypsiferous units)

Contact

MF-105
Drill Hole

NO	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604
COMPOSITE OFFICES
HELENA, MONTANA
BILLINGS, MONTANA
GREAT FALLS, MONTANA
BOZEMAN, MONTANA
SPOKANE, WASHINGTON

DRAWN BY: CJA
CHKD. BY: DO
APPR. BY: C.N.K.
DATE: 12-10-86
QA
PEER REVIEW

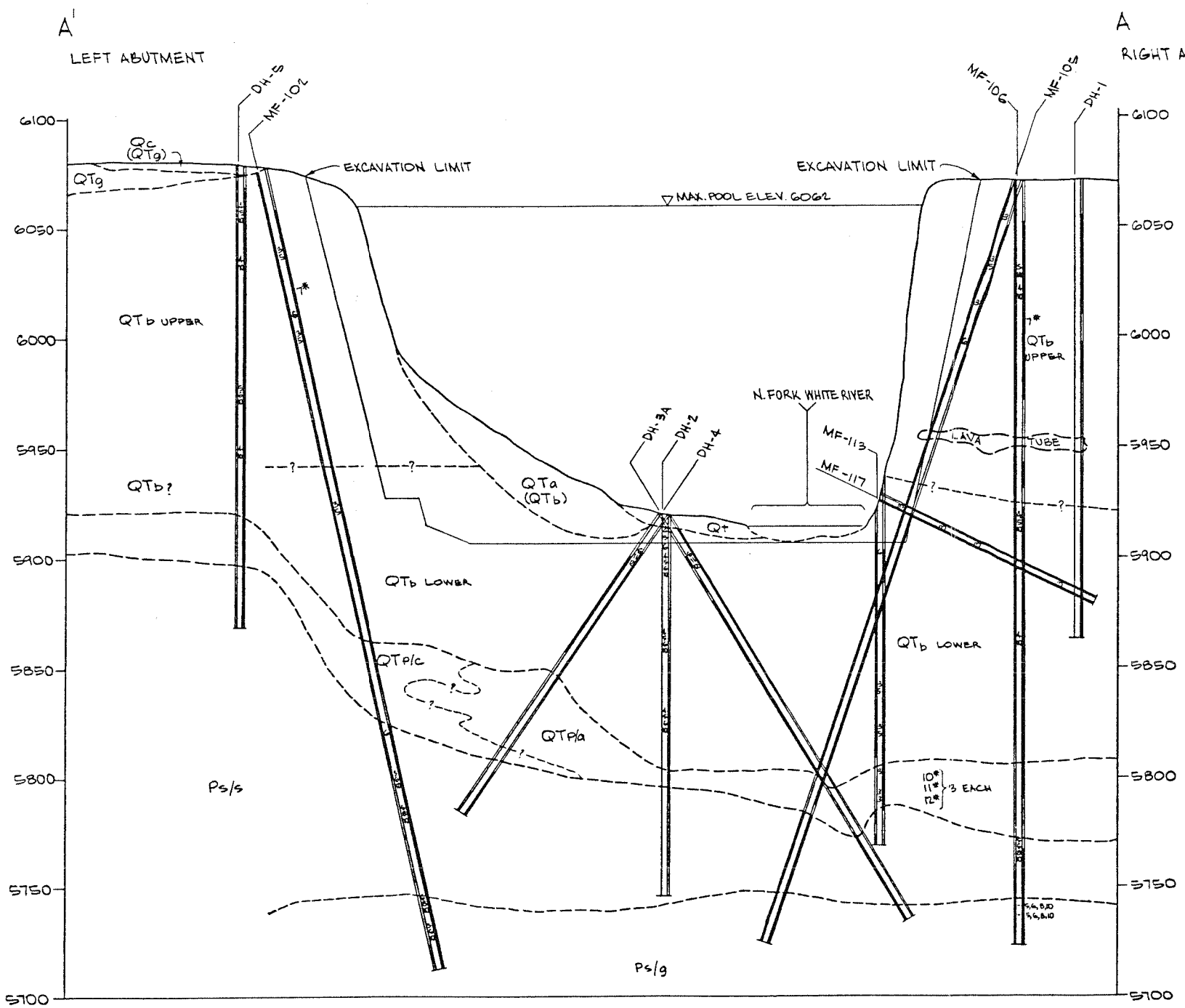
MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)

SHEET NUMBER
D1

SHEET TITLE
FOUNDATION DATA "A"

SCALE: 1"=50'



GEOLOGIC SECTION A'-A

EXPLANATION

Q+
TERRACE

Qc
COLLUVIUM
(BEDROCK, SOURCE SHOWN BY SYMBOL FOR ROCK UNIT)

Qta
TALUS
(BEDROCK, SOURCE SHOWN BY SYMBOL FOR ROCK UNIT)

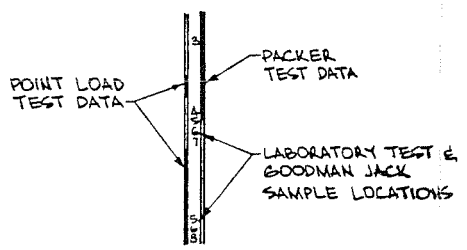
QTg
QUATERNARY-TERTIARY GRAVEL

QTb
BASALT
UPPER-NUMEROUS, COALESCING VESICULER & MASSIVE FLOWS
LOWER-SINGLE MASSIVE FLOW

QTp/
PALEO COLLUVIUM/ALLUVIUM
/c - PALEO COLLUVIUM
/a - PALEO ALLUVIUM

Ps/
SUPAI FORMATION
/s - SANDSTONE/SILTSTONE
/g - GYPSIFEROUS SANDSTONE/GYPSUM

TEST LOCATION CODING



FIELD TESTS
1. POINT LOAD
2. PACKER
3. GOODMAN JACK

LABORATORY TEST (NX CORE)
4. UNIAXIAL W/O ELASTIC PROPERTIES
5. UNIAXIAL W/ ELASTIC PROPERTIES
6. DIRECT SHEAR (CORE)
7. DIRECT SHEAR (REMOLDED)
8. TRIAXIAL (CORE)
9. TRIAXIAL (REMOLDED)
10. DENSITY
11. GRADATION
12. ATTERBERG LIMITS

NOTE:
7*, REMOLDED DIRECT SHEAR ON COMPOSITE SAMPLE OF FLOW BOUNDARY MATERIAL FROM HOLES MF-102, 105, & 106.
10*, 11*, & 12*, DENSITY, GRADATION & LIMITS ON NEARBY OUTCROP SAMPLE OF PALEO ALLUVIUM/COLLUVIUM.

SCALE: 1"=30'

SHEET TITLE
FOUNDATION DATA "B"

NO	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604
CORPORATE OFFICES
BILLINGS, MONTANA
BRANCH OFFICES
BUTTE, MONTANA
KALISPELL, MONTANA
BOZEMAN, MONTANA

DRAWN BY GJA
CH'D. BY DO
APPR. BY CWK
DATE 12-10-86
QA
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
D1A

MINER FLAT

DRILL HOLE NO. DH-1
Elevation: 4673'
Bearing: ∅
Date Started: 1/20/82
Drilled by: Western Technologies
Logged by: Charles H. Robinson
Inclination: 90°
Date Completed: 1/26/82

Depth (feet)	Lithologic Log	Description
10		Basalt, massive, very fine-grained, aphanitic, porphyritic, few vesicles, iron and clay stained fractures.
30		Basalt, vesicular, very fine-grained to aphanitic, vesicles filled with clay, iron and clay stained fractures.
50		Basalt, massive to slightly vesicular.
70		Basalt, vesicular, very fine-grained, aphanitic, porphyritic, with olivine.
90		Basalt, massive very fine-grained, aphanitic-porphyritic w/ olivine, hard to very hard.
110		Lava tube
130		Basalt, vesicular, very fine-grained, vesicles filled with silica, fractures stained with clay.
150		
170		Basalt, massive, very fine-grained, aphanitic-porphyritic with olivine phenocrysts. Locally fractured with silica filling and some clay staining.
190		
210		
212.0	E.O.H.	

MINER FLAT

DRILL HOLE NO. DH-2
Elevation: 5922'
Bearing: ∅
Date Started: 2/6/82
Drilled by: Western Technologies
Logged by: Charles H. Robinson
Inclination: 90°
Date Completed: 2/18/82

Depth (feet)	Lithologic Log	Description
		Silty sand. Alluvial terrace.
10		
30		
50		Basalt, massive, medium gray, very fine-grained, aphanitic-porphyritic with phenocrysts of olivine. Very hard, slightly fractured, fractures with stain of clay.
70		
90		
100.5		
	E.O.H.	

MINER FLAT

DRILL HOLE NO. DH-2 (Cont.)
Elevation: 5922'
Bearing: ∅°
Date Started: 8/2/83
Drilled by: Boyles Bros. Fred Barlois and Charles H. Robinson
Logged by:
Inclination: 90°
Date Completed: 8/6/83

Depth (feet)	Lithologic Log	Description
110		Basalt, massive, medium gray, very fine-grained, very hard.
		Basalt, very vesicular, scoracious, aphanitic, with incorporated sandstone and siltstone.
		Paleo-alluvium, cobbles and gravel in silty sand. Clasts of basalt, sandstone, siltstone.
130		
150		Sandstone, reddish brown to pale yellowish green, silty, very fine to fine-grained, slightly calcareous, soft.
170		
175.3	E.O.H.	

MINER FLAT

DRILL HOLE NO. DH-3
Elevation: 5922'
Bearing: 335°
Date Started: 2/19/82
Drilled by: Western Technologies
Logged by: Charles H. Robinson
Inclination: 60°
Date Completed: 2/21/82

Depth (feet)	Lithologic Log	Description
		Silty sand. Alluvial terrace.
10		
30		Basalt, massive, medium gray, very fine-grained, aphanitic-porphyritic, with olivine phenocrysts, very hard, slightly fractured, few fractures stained with clay.
44.5	E.O.H.	

MINER FLAT

DRILL HOLE NO. DH-4
Elevation: 5922'
Bearing: 156°
Date Started: 8/7/83
Drilled by: Boyles Bros.
Logged by: Charles H. Robinson
Inclination: 56°
Date Completed: 8/9/83

Depth (feet)	Lithologic Log	Description
		Colluvium, boulder and gravel with silty clayey sand.
10		
30		Basalt, massive, very fine-grained, aphanatic-porphyritic with olivine phenocrysts, very hard, slightly fractured with fractures stained with clay, some filled with silica.
50		
70		
90		Basalt, vesicular, scoracious or flow breccia.
110		Paleo-colluvium, sandstone gravel and cobbles in a sandy matrix.
130		
150		Sandstone, reddish brown, fine- to very fine-grained, silty, slightly calcareous.
161.5	E.O.H.	

MINER FLAT

DRILL HOLE NO. DH-5
Elevation: 5079'
Bearing: ∅
Date Started: 7/9/83
Drilled by: Boyles Bros.
Logged by: Pat Wiegand
Inclination: 90°
Date Completed: 7/12/83

Depth (feet)	Lithologic Log	Description
10		Basalt, massive, very fine-grained, aphanitic-porphyritic, fractures are stained with clay.
		Basalt, vesicular, very fine-grained to aphanitic-porphyritic with olivine phenocrysts, clay in vesicles and on fractures.
30		Basalt, vesicular, scoracious basalt at flow boundaries 21.8, 29.2, clay filled vesicules.
		Basalt, massive clay stained fractures.
50		Basalt, vesicular, clay filled fractures.
70		Basalt, massive, dark gray, very fine-grained, porphyritic.
		Basalt, vesicular, highly fractured, clay filled fractures and vesilces.
		Basalt, scoracious.
90		Basalt, massive, very fine-grained, aphanitic-porphyritic with olivine phenocrysts.
		Basalt, vesicular.
		Basalt, massive.
		Basalt, scoracious.
110		Basalt, flow breccia, scoracious.
		Basalt, vesicular, very fine-grained, aphanitic-porphyritic w/olivine phenocrysts, clay filled fractures.
130		Basalt, massive, very fine-grained, aphanitic-porphyritic with olivine phenocrysts.
150		Basalt, flow breccia, scoracious, weathered.
170		Paleo-colluvium, sandstone cobbles and gravel in a fine-grained sand matrix.
190		Sandstone, reddish brown, very fine-grained, silty, calcareous.
209.2	E.O.H.	

NO.	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604
CORPORATE OFFICE HELENA, MONTANA
BRANCH OFFICES BILLINGS, MONTANA KALISPELL, MONTANA GILLETTE, WYOMING RIVERTON, WYOMING

DRAWN BY: CJA
CHK'D. BY: DO
APP'D. BY: CWK
DATE: 12-10-86
QA
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)

SHEET NUMBER
D2

SHEET TITLE
FOUNDATION DATA "C"

MINER FLAT

DRILL HOLE NO. DH-14
Elevation: 6096'
Bearing: ∅
Date Started: 7/14/83

Drilled by: Boyles Bros.
Logged by: Pat Wiegand
Inclination: 90°
Date Completed: 7/21/83

Depth (feet)	Lithologic Log	Description
10		Quaternary-Tertiary gravel, cobbles and gravel in clayey, silty sand matrix.
30		
50		Basalt, massive, very fine-grained, aphanitic-porphyritic with olivine phenocrysts, fractures healed with silica or stained with clay.
70		Basalt, vesicular to scoracious, very fine-grained, aphanitic-porphyritic, fractures stained and filled with clay.
90		Basalt, massive, very fine-grained, aphanitic-porphyritid with olivine phenocrysts, fractures stained with clay.
110		Paleo-alluvium, cobbles and gravel of basalt, quartzite, volcanic and intrusive rocks in a silty sand matrix.
130		
150		Sandstone, reddish brown to medium brown, very fine-grained to fine-grained, silty, slightly calcareous.
170		
188.0		E.O.H.

MINER FLAT

DRILL HOLE NO. DH-15
Elevation: 5917'
Bearing: ∅
Date Started: 7/23/83

Drilled by: Boyles Bros.
Logged by: Pat Wiegand and Fred Barbis
Inclination: 90°
Date Completed: 7/26/83

Depth (feet)	Lithologic Log	Description
		Colluvium-Alluvium sand and gravel with cobbles of sandstone and basalt.
10		
30		
50		Basalt, massive, moderate to dark gray, very fine-grained, aphanitic-porphyritic with olivine phenocrysts, very hard, few fractures
70		
90		
110		Basalt, scoracious, flow breccia, aphanitic-porphyritic.
130		Paleo-alluvium, cobbles and gravel of quartzite, basalt, limestone, sandstone and igneous rocks in a sand matrix.
150		Sandstone, reddish brown, very fine-grained, silty, slightly calcareous.
170		Gypsum, white to grayish brown.
181.5		Sandstone breccia, silty, gypsiferous.
		E.O.H.

REVISIONS

NO	DESCRIPTION	DATE	BY

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE, P.O. BOX 6147
HELENA, MONTANA 59604
CORPORATE OFFICE HELENA, MONTANA
BRANCH OFFICES BILLINGS, MONTANA KALISPELL, MONTANA RAPID CITY, SOUTH DAKOTA RIVERTON, WYOMING

DRAWN BY: CJA
CHK'D BY: PO
APP'D BY: CWK
DATE: 12-10-86
Q.A.
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)

SHEET NUMBER
D3

SHEET TITLE
FOUNDATION DATA 'D'

DRILL HOLE NO. MF-102

Elevation: 6076 feet
Bearing: 320°
Date Started: 1/21/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 72°
Date Completed: 1/30/86

Depth (feet)	Description
	Fill and colluvium
	Basalt, aphanitic, porphyritic, olivine, massive
10	Basalt, vesicular, aphanitic, porphyritic, olivine phenocrysts. Clay-filled fractures, slightly weathered
30	Basalt, vesicular, scoracious basalt at flow boundry at 31.6' Clay-filled vesicules
50	Basalt, massive, very fine grained to aphanitic, porphyritic, olivine phenocrysts.
	Basalt, vesicular, clay-filled fractures
70	Basalt, massive
	Basalt, vesicular, highly fractured. Clay-filled fractures
	Basalt, massive, very fine grained
90	Basalt, scoracious, highly fractured, vesicular, aphanitic, clay-filled fractures and vesicules.
110	
130	Basalt, massive, very fine grained to aphanitic, porphyritic, olivine phenocrysts. Highly fractured to 139.8'. Clay-filled fractures
150	Fracture, clay-filled at 150'
170	
190	Basalt, flow breccia, scoracious, weathered
	Basalt, massive
210	Basalt, flow breccia
	Basalt, massive
230	Paleo-colluvium. Upper 2 feet basalt and sandstone boulders. Sandstone boulders <0.5 - 1.65 feet in diameter in sand matrix, moderately yellowish brown.
250	
270	Sandstone, moderately yellowish brown, fine gramed, silty, slighty calcareous.
	Sandstone, reddish to yellowish brown, silty and sandy siltstone, local brecciation.
290	
310	Sandstone, reddish brown, fine grained, silty, locally diagenetic breccia, white sandstone fragments.
330	
350	Sandstone, medium reddish brown, fine grained, with pale light gray silty sandstone, includes silty clay clasts. Gypsiferous, gypsum beds and veinlets to 0.4 feet thick. Locally sandstone brecciated
370	
375	E. O. H.

Flow boundary

Mineral Systems, Inc

DRILL HOLE NO. MF-105

Elevation: 6073 feet
Bearing: 140°
Date Started: 11/23/85

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 72°
Date Completed: 1/8/86

Depth (feet)	Description
	Colluvium, weathered from basalt
10	Basalt, vesicular, fine grained, porphyritic, phenocrysts of olivine. Vesicules 20.01 to 0.25 feet, clay filled, weathered.
30	Basalt, massive, fine grained to aphanitic, porphyritic.
	Basalt, vesicular. Vesicules clay filled
	Basalt, massive, very fine grained
50	Basalt, vesicular, horizontal fractures, olivine phenocrysts. Fractures clay filled or sand filled
70	Basalt, massive, porphyritic.
90	Basalt, vesicular, fine grained to aphanitic, porphyritic, fractured, vesicules to 0.2 feet. Becomes more massive with depth. Fractures and vesicules filled with sandy clay.
110	Basalt, massive, medium gray, fine grained to aphanitic, porphyritic. Fractures clay filled.
	Lava tube
130	Basalt, vesicular, locally ropey texture. Vesicules elongate parallel to flow banding. Vesicules and fractures clay filled. Some vesicules quartz filled.
150	
170	
190	Basalt, massive, medium gray to dark gray, very fine grained to aphanitic, porphyritic, phenocrysts of olivine, fractured. Fractures filled or stained with light brownish gray clay and/or soft olive green talc-like mineral.
210	
230	
250	
270	Basalt, vesicular, medium gray, vesicules to 0.1 feet, aphanitic. Fractures and vesicules clay filled.
290	Paleo-alluvium. Boulders, cobbles and gravel in silty sand. Clasts of sandstone, limestone, quartzite, granite and basalt.
310	Sandstone, silty, reddish orange and yellowish orange, fine to medium grained, poorly indurated, fractures tight
330	Sandstone, gypsiferous breccia
	Gypsum, sandy and silty
350	Sandstone, fine grained, silty gypsum 354.5-355.0. Gypsiferous, veinlets of gypsum to 0.05 feet. Locally gypsiferous sandstone breccia.
370	E. O. H.

Flow boundary

Mineral Systems, Inc

MINER FLAT

DRILL HOLE NO. MF-106

Elevation: 6072 feet
Bearing: 246.9°
Date Started: 11/17/85

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.4°
Date Completed: 12/5/85

Depth (feet)	Description
10	Basalt, massive, very fine grained, aphanitic, porphyritic. Few vesicules, clay on fractures.
	Basalt, vesicular, very fine grained to aphanitic, porphyritic, olivine phenocrysts. Clay on fractures.
30	Basalt, slightly vesicular, fine grained to aphanitic. Vesicules filled with clay or silica, clay stained fractures.
50	Basalt, massive to slightly vesicular
70	Basalt, vesicular, equigranular, olivine, fine grained to aphanitic. Vesicules filled with clay or silica, clay stained fractures.
90	
110	Basalt, massive, very fine grained to aphanitic, porphyritic, olivine.
	Open cavity
130	Basalt, vesicular, very fine grained to aphanitic. Vesicules < 0.01 to 0.1 ft. Filled with clay or silica.
150	
170	
190	Basalt, massive, very fine grained to aphanitic, porphyritic, phenocrysts of olivine. Locally fractured with fractures filled with clay or tight.
210	
230	
250	Basalt, vesicular, scoraceous, flow breccia.
270	Paleo-alluvium, boulders, cobbles and gravel in silty sand matrix. Clasts of basalt, granite, gneiss, chert, sandstone and limestone.
290	
310	
330	Sandstone, reddish orange and yellowish red, fine grained, silty, clayey, slightly calcareous, gypsiferous 341 - 352.8 ft
350	
357.8	E.O.H.

~~~~~ Flow boundary

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-113

Elevation: 5920 feet
Bearing: 211.1°
Date Started: 2/25/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.7°
Date Completed: 3/27/86

Depth (feet)	Description
	Fill
10	
30	
50	Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, phenocrysts of olivine, magnetic. Fractured, fractures tight or slightly clay stained.
70	
90	
110	Basalt, vesicular, scoraceous, aphanitic.
	Paleo-alluvium. Boulders, cobbles, gravel in silty sand. Clasts of basalt, granitic rock, chert, sandstone and limestone.
130	Sandstone, reddish brown, silty, very fine to fine grained.
150	
155	E. O. H.

Mineral Systems, Inc

MINER FLAT

DRILL HOLE NO. MF-117

Elevation: 5924 feet
Bearing: 308.7°
Date Started: 3/7/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 26°
Date Completed: 3/9/86

Depth (feet)	Description
10	
30	
50	Basalt, massive, medium dark gray, very fine grained to aphanitic, porphyritic. Phenocrysts of olivine to 1 mm. Locally fractured. Fractures filled with clay.
70	
90	
97.4	E. O. H.

Mineral Systems, Inc

SHEET TITLE

FOUNDATION DATA "E"

NO.	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE P.O. BOX 6147
HELENA, MONTANA 59604

CORPORATE OFFICE
BILLINGS, MONTANA
BRANCH OFFICES
KALISPELL, MONTANA
MISSOULA, MONTANA
BOZEMAN, MONTANA
BUTTE, MONTANA

DRAWN BY CJA
CHK'D BY DO
APP'D BY CJN/C
DATE 12-10-86
QA
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)

SHEET NUMBER
D4
~~~~~

MINER FLAT

DRILL HOLE NO. MF-118
Elevation: 6077 feet
Bearing: 243.8°
Date Started: 1/11/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.5°
Date Completed: 1/15/86

Depth (feet)	Lithologic Log	Description
		Colluvium, silty clay
10		Basalt, vesicular, dark gray, aphanitic, porphyritic, olivine phenocrysts, vesicules up to 0.15 feet. Clay on fractures.
		Basalt, massive, very fine grained to aphanitic, porphyritic.
30		Basalt, vesicular, very fine grained to aphanitic, slightly weathered. Paleosol 38.1 and 40.3. Clay filled fractures and vesicules.
50		Basalt, massive, medium gray, very grained, porphyritic, phenocrysts of olivine, slightly vesicular. Vesicules filled with clay.
70		
90		
110		Basalt, vesicular, medium to dark gray, aphanitic, porphyritic, phenocrysts of olivine. Vesicules filled with clay or silica. Paleosol at 90.3, 104 and 112. Fractures filled or stained with clay. Slightly weathered.
130		
150		
170		Basalt, massive, medium gray and dark brownish gray, very fine grained, porphyritic, phenacrysts of olivine. Locally fractured, fractures coated with clay.
190		
210		
230		
250		
270		Basalt, vesicular and scoraceous, aphanitic
290		Paleo-alluvium, boulders, cobbles and gravel in silty sand. Cobbles of basalt, granitic rocks, chert, quartzite, sandstone and limestone. Boulders to 1.35 feet.
310		
330		Sandstone, reddish and yellowish orange, fine grained, slightly cemented with $CaCo_3$, generally poorly consolidated. Locally diagenetic breccia.
340		E.O.H

~~~~~ – Flow boundary

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-119
Elevation: 6094 feet
Bearing: 231.9°
Date Started: 4/8/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.9°
Date Completed: 4/16/86

Depth (feet)	Lithologic Log	Description
10		Colluvium. Silty clay, yellowish brown, cobbles of quartzite
		Basalt, vesicular, medium gray, slightly weathered
30		Basalt, massive, medium gray, fine grained, porphyritic, olivine phenocrysts, slightly weathered.
50		Basalt, vesicular, aphanitic, porphyritic, olivine phenocrysts, moderate gray. Vesicules filled with clay and silica
		Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, olivine phenocrysts, fractures filled with silica and clay.
70		Basalt, vesicular, silica filled vesicules.
		Basalt, massive, slightly weathered
		Basalt, vesicular, scoraceous
90		Basalt, massive Flow boundary with 1.3 feet paleosol
		Basalt, vesicular, reddish gray, weathered.
110		Basalt, massive, medium gray, very fine grained, aphanitic, porphyritic, olivine phenocrysts.
		Basalt vesicular, reddish gray, aphanitic, silica and clay filled vesicules
130		Paleosol, reddish gray, silty and clayey silt with cobbles or gravel of basalt
		Basalt, vesicular, medium gray and grayish red, aphanitic, ropey flow banding.
150		
170		Basalt, massive, medium gray, very fine grained to anphanitic, porphyritic, phenoclasts of olivine. Local clay filled fractures
190		
210		Basalt, flow breccia, fragments of basalt in clay matrix.
230		Paleo-colluvium talus, reddish brown to reddish orange and light gray sandstone fragments to 0.5 foot in silty sand matrix
250		
270		Paleo-alluvium, reddish orange, cobbles and gravel of basalt, quartzite and sandstone in silty and clayey sand matrix. Limestone, quartzite and sandstone pebbles, slightly calcareous.
290		Sandstone, light yellowish gray, fine grained, friable to slightly calcareous cement, silty and clayey locally.
300		E.O.H.

~~~~~ – Flow boundary

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-120A
Elevation: 6082 feet
Bearing: 246°
Date Started: 4/18/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.5°
Date Completed: 4/21/86

Depth (feet)	Lithologic Log	Description
		Colluvium
10		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts.
30		Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, phenocrysts of olivine.
50		
70		Basalt, vesicular, medium gray and reddish gray, aphanitic, porphyritic, phyenocrysts of olivine. Vesicules and fractures filled with silica and clay.
90		Basalt, massive, reddish gray, olivine phenocrysts.
110		Basalt, vesicular, reddish gray, aphanitic, porphyritic, olivine phenocrysts.
130		Paleosol, medium gray to reddish gray basalt cobbles, gravel and sand in a dense clay matrix. Interflow alluvial deposit.
150		Basalt, vesicular, medium gray, aphanitic, porphyritic.
170		Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, olivine phenocrysts. Clay stain on fractures.
190		
210		
230		Basalt, vesicular, aphanitic, scoraceous and ropy, cobbles of basalt, clay and sand matrix
250		Paleosol alluvium. Cobbles and gravel of sandstone, basalt, quartz, quartzite and chert in reddish orange silty sand matrix.
270		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Basalt breccia 275.2-276, 276.7-276.9, 278-278.5 Clay matrix. vesicules filled with clay or silica.
290		Basalt, massive, medium gray very fine grained to aphanitic, porphyritic. At base basalt breccia
310		Paleo-alluvium, grayish orange, boulders, cobbles and gravel of sandstone, basalt, granitic rocks, metamorphic rocks, quartzite, limestone and chert in silty and clayey sand matrix.
330		
350		
351		E.O.H.

~~~~~ – Flow boundary

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-121
Elevation: 6090 feet
Bearing: 271°
Date Started: 2/10/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.8°
Date Completed: 2/28/86

Depth (feet)	Lithologic Log	Description
		Fill and colluvium
10		Colluvium, light gray to reddish orange. Sand and sandstone
30		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts, Massive 23.1-25 feet Vesicules filled with clay.
50		Basalt, massive, medium gray, very fine to aphanitic, porphyritic, olivine phenocrysts 0.3 ft paleosol at 38.1
		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts, vesicules filled with silica Paleosol at 55.0-55.2
70		Paleo colluvium, reddish gray, sandstone in silty sand matrix
90		Sandstone, light gray to reddish brown, very fine grained, silty, friable Locally thin lenses of sandy siltstone
110		
130		
150		
170		Sandstone, light reddish brown, fine grained, silty. Locally cross bedded with single layer of course grain to medium grain sand along cross beds. Friable
190		
191		E.O.H.

Mineral Systems, Inc

NO.	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604

DRAWN BY: CJA
CHK'D BY: DO
APPR. BY: CWK
DATE: 12-10-86
Q.A.
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)

SHEET NUMBER
D5

SHEET TITLE
FOUNDATION DATA "F"
~~~~~

MINER FLAT

DRILL HOLE NO. MF-122
Elevation: 6064 feet
Bearing: 278.5°
Date Started: 2/5/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.9°
Date Completed: 2/8/86

Depth (feet)	Lithologic Log	Description
		Colluvium, clayey sand.
10		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Silica and clay filled vesicules and fractures. Specular hematite?
30		Basalt, massive, medium gray, very fine grained, porphyritic, olivine phenocrysts.
50		Basalt, vesicular, medium gray, weathers reddish brown, aphanitic, porphyritic, olivine phenocrysts. Vesicules and veins filled with coarse basalt sand and clay.
70		Basalt, massive, medium gray, very fine grained, porphyritic, olivine phenocrysts. Vesicules filled with clay at 65 ft.
90		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Slightly weathered. Clay and silica filling vesicules.
110		Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic. Flow fractures.
130		Basalt, vesicular and scoraceous, medium gray, aphanitic, porphyritic, olivine phenocrysts. Sand size rock fragments between blocks of scoraceous basalt up to 1.5 ft long.
150		Sandstone, light reddish brown to medium brown, fine grained. Contact of basalt and sandstone sharp at 150.3 ft.. Less than 0.03 ft. of paleo-colluvium. Sandstone silty and friable
170		
175		E.O.H.

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-123
Elevation: 6113 feet
Bearing:
Date Started: 3/13/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 90°
Date Completed: 3/25/86

Depth (feet)	Lithologic Log	Description
10		Quaternary - Tertiary Gravel. Cobbles and gravel in clayey, silty sand. At 43 ft. 2-3 ft. dark carbonaceous sandstone. From 51 ft., cobbles, gravel and sand of white, fine grained sandstone.
30		
50		
70		Sandstone, light gray to pinkish gray, light pinkish orange, fine grained, silty, moderately indurated. Sandstone breccia at 74-76 ft.
90		
110		Sandstone, light gray to light pinkish gray, fine grained, silty, poorly indurated. Below 113 ft. interbedded reddish brown silty and clayey fine grained sandstone.
130		Sandstone, medium reddish brown, fine to medium grained, silty, some clay, poorly indurated.
150		Sandstone, light reddish to light yellowish brown, fine grained to very fine grained, silty, locally clayey, poorly indurated.
2075		E.O.H.

~~~~~ - Flow boundary

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-124
Elevation: 6081 feet
Bearing:
Date Started: 3/1/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 90°
Date Completed: 3/4/86

Depth (feet)	Lithologic Log	Description
10		Quaternary - Tertiary Gravel. Cobbles, gravel, sand and silt.
30		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Vesicules filled with silty clay.
50		Vesicular, clay filled flow boundary
70		Basalt, massive, medium gray, very fine grained, porphyritic, olivine phenocrysts. Vertical fractures, clay filling.
90		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Vesicules clay.
110		
130		Sandstone, light reddish orange, very fine grained, very friable. 99.4 - 101.3 only 2 pieces of sanstone recovered. Paleo-colluvium?
180		E.O.H.

~~~~~ - Flow boundary

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-125A
Elevation:
Bearing: 189.9°
Date Started: 3/5/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.9°
Date Completed: 3/7/86

Depth (feet)	Lithologic Log	Description
		Quaternary - Tertiary gravel
10		Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts.
30		Basalt, massive, medium gray, very fine grained, porphyritic
50		Basalt, vesicular, medium gray, very fine grained to aphanitic, porphyritic, olivine phenocrysts, slightly magnetic. Clay filled vesicules and fractures.
70		Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, olivine phenocrysts, slightly magnetic. Fractures vertical, stained with clay.
90		Basalt, vesicular, very fine grained to aphanitic, porphyritic Vesicules filled with clay.
110		Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, olive phenocrysts, slightly magnetic.
130		Basalt, vesicular, medium gray, aphanitic
170		Sandstone, pinkish gray to reddish orange, fine to very fine grained, silty, thinly bedded, poorly indurated
		E.O.H.

~~~~~ - Flow boundary

Mineral Systems, Inc.

NO.	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
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DRAWN BY: CJA
CHK'D BY: DO
APP'D BY: CWK
DATE: 12-10-86
Q.A. PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)

SHEET NUMBER
D6

SHEET TITLE
FOUNDATION DATA "G"
~~~~~

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT
TO
MORRISON - MAIERLE INC
ON
STABILITY ANALYSIS OF THE MINER FLAT DAM FOUNDATIONS NAVAJO COUNTY, ARIZONA

DISTRIBUTION:

25 copies - Morrison - Maierle Inc.
Helena, Montana

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Vancouver, B.C.

February, 1987

862-1622

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1.0 INTRODUCTION

Golder Associates has been retained by Morrison-Maierle Inc. of Helena, Montana to carry out a preliminary foundation stability analysis for the proposed Miner Flat Dam on the White Mountain Apache Reservation in Arizona. Authorization to proceed was received in a letter from Morrison-Maierle Inc. dated November 24, 1986. The project is in the preliminary engineering design phase and the foundation analysis is needed to complement similar studies on the dam structure being carried out by Morrison-Maierle Inc.

The proposed dam is located on the North Fork of the White River at Miner Flat, east of State Highway 73, about eight miles south of McNary, to the northeast of Pheonix, Arizona (Figure 1). The river valley at this location is steep-sided and formed by near vertically jointed columnar basalt. The proposed dam will consist of a 160 foot high Roller Compacted Concrete (RCC) gravity structure founded on the basalt. The basalt is separated from the underlying sandstone of the Supai Formation by up to 40 feet of paleo-soil which lies some 30 to 100 feet below the base of the valley at the dam site.

This report describes analyses carried out. Potential foundation stability problems are outlined, the basis of the geomechanical model is presented, and the selection of input parameters is discussed. The methods of analyses, the results and recommendations for further investigation, testing, and analyses to refine or clarify the conclusions of this study are also included.

2.0 STATEMENT OF PROBLEM

The proposed Miner Flat Dam is to be located in a steep canyon in the North Fork River. The walls and floor of the canyon consist of a

series of basalt flows. Site investigations have identified that between the basalts and the underlying sandstones is a paleo-soil layer. This layer is a mixture of boulders, gravels, sands and silts.

Review of this basic geologic model of the dam site has indicated that two potential foundation problems exist, as illustrated in Figure 2. These are sliding or shear failure along various planes of weakness within the foundation rocks or abutment slopes and excessive settlements along the base of the dam due to consolidation of the underlying paleo-soil.

2.1 Foundation Stability Against Sliding

Available geologic information indicated that three potential sliding surfaces might exist in the dam foundation, these being:

- sliding along the dam/basalt contact;
- sliding through a zone of low quality basalt, identified from the drillhole logs, that may contain adversely oriented joints or planes; and
- sliding through the paleo-soil layer with breakout through the basalt.

2.2 Foundation Stresses and Deformations

The paleo-soil layer varies in thickness in both the upstream/downstream direction and along the dam axis. Due to this variation, uneven consolidation and compaction of the paleo-soil under dam loads may occur leading to differential settlement both between the toe and heel of the dam and from one abutment to the other along the dam axis. Depending on the magnitude, such differential settlements could lead to the development of excessive stress in the dam, the underlying basalt and the paleo-soil.

2.3 Stability of Cut Slopes

The proposed construction methods indicate that relatively steep slopes will be excavated to prepare both dam abutments. Some of these slopes will be open only during construction, while others will be permanent. The presence of any adversely oriented planes of weakness and the development of high water pressures in these slopes after reservoir filling may cause slope instability.

2.4 Objectives

The objectives of the study have therefore been to assess the likely occurrance of the specific stability problems outlined above; that is to assess;

- the potential for sliding or shear failure in the foundation rocks;
- the deformation of the foundation rocks under dam and reservoir loading; and
- the stability of the proposed abutment excavations.

At this stage no consideration has been given to stability aspects of canyon walls either upstream or downstream of the dam, nor has consideration been given to other aspects of dam stability, such as resistance to overturning. These are addressed elsewhere.

3.0 DATA BASE

The engineering geology of the Miner Flat Dam site was previously investigated by Mineral Systems Inc. (MSI) who also carried out exploratory drilling, geologic mapping and preliminary field testing. The results of their work has been documented in various reports, as

discussed below. Subsequently, laboratory testing on selected core samples from the drilling program was undertaken by Morrison-Maierle Inc. (MMI).

Details of the topography, dam layout and proposed construction were provided to Golder Associates by MSI and MMI. Golder Associates assembled, reviewed and evaluated this data, and then constructed a geomechanical model of the site. Because some of the geotechnical parameters required as input to the analyses were poorly defined, it was necessary to use a range of values and assess the sensitivity of the dam stability to variations in those parameters.

3.1 Dam Layout and Loading

Details of the site topography and dam layout as summarized in Figure 3, were produced by MMI. A list of drawings supplied is given in Table 1.

TABLE 1

LIST OF MORRISON-MAIERLE INC. DRAWINGS OF PROPOSED MINER FLAT DAM SITE LAYOUT

	TITLE	SCALE	NUMBER	DATE
1.	Plan - R.C.C. Gravity Dam	1" to 20'	Unnumbered	Undated
2.	Section - R.C.C. Gravity Dam	1" to 20'	Unnumbered	Undated
3.	Upstream Elevation - R.C.C. Gravity Dam	1" to 20'	Unnumbered	Undated
4.	General Map "B"	1" to 400'	A2	12-10-86
5.	Surface Data - Topographic Map ""B"	1" to 400'	C2	12-10-86
6.	Surface Data - Topographic Map "C"	1" to 400'	C3	12-10-86
7.	Foundation Data "B" (Section A'-A)	1" to 30'	Unnumbered	Undated
	a. Untitled SECTION A	1" to 30'	Unnumbered	Undated
	b. Untitled SECTION B	1" to 30'	Unnumbered	Undated
	c. Untitled SECTION C	1" to 30'	Unnumbered	Undated

The details of external loads on the dam, shown in Figure 4, were provided in a memorandum from Morrison-Maierle dated 10 December, 1986.

3.2 Previous Reports

The results of the engineering geology investigation carried out by MSI are documented in the reports listed in Table 2.

These reports contain descriptions of general site conditions, geology and structure. The results of geologic mapping, line surveys, geophysical investigations and drilling and field testing programs are also provided. Preliminary assessments of engineering rock mass parameters to be used in analysis were discussed. Appendices attached to

these reports provided both the descriptive borehole logs and the engineering geology logs.

TABLE 2

LIST OF REPORTS BY MINERAL SYSTEMS INC. ON ENGINEERING GEOLOGY OF MINER FLAT DAM SITE

	Title	Date
1.	Preliminary Report Engineering Geology Miner Flat Damsite and Reservoir White Mountain Apache Reservation Arizona.	July, 1982
2.	Preliminary Report Engineering Geology Miner Flat Dam Site White Mountain Apache Indian Reservation Arizona.	November, 1983
3.	Preliminary Report Engineering Geology Miner Flat Dam Site White Mountain Apache Reservation Navajo County, Arizona Volume I and Volume II	July, 1986

No core photographs were available from the original core logging program carried out by MSI.

In December, 1986 MMI undertook selective relogging of the core and obtained core photographs. These photographs were forwarded to Golder Associates in a memorandum dated 8 January, 1987.

3.3 Geologic Profiles

Data from the engineering geology logs provided by MSI were used to construct a series of engineering geologic sections along the dam centre line, showing histograms of:

- core recovery
- rock quality designation (RQD)

- fracture index
- clay filled fracture occurrence
- uniaxial compressive strength
- permeability

These sections are given in Figures 5a through 5f, respectively.

Three typical foundation profiles showing geologic conditions beneath the dam were prepared by MSI. These profiles, oriented perpendicular to the dam axis, are referenced in plan on Figure 6 and are given in Figures 7a, 7b and 7c.

Geologic fabric of the basalt in the vicinity of the dam as determined from data collected from detailed line surveys is shown in Figure 8.

3.4 Field Testing

Preliminary field testing carried out by MSI consisted of point load testing on intact core specimens, in situ packer testing, and downhole Goodman Jack testing. The results of this testing were given in the MSI reports listed in Table 3.

3.4.1 Point Load Testing

Selected pieces of core from the drilling investigation were broken using point load testing equipment. Using the point load strength indices obtained, equivalent uniaxial compressive strengths were calculated. The results are summarized in Table 3 and plotted as a series of borehole histograms in Figure 5e.

TABLE 3

SUMMARY OF RESULTS OF POINT LOAD TESTS
FROM MINERAL SYSTEMS INC. INVESTIGATION OF MINER FLAT DAM SITE

	Basalt			Paleo-Soil		Sandstone	Gypsiferous Sandstone	
	Vertical Holes	Angle Holes	Low-Angle Holes	Vertical Holes	Angle Holes	Vertical Holes	Vertical Holes	Angle Holes
Maximum	42,109	36,547	36,547	957	10	13,983	8,263	6,594
Minimum	957	3,851	25,424	10	10	10	1,351	1,510
Average	23,780	22,863	32,308	35	10	2,654	4,018	3,284
Number of Samples Tested	255	58	17	64	10	60	7	12

* Equivalent Uniaxial Compressive Strength (psi)

3.4.2 Packer Testing

Packer tests were carried out to determine the hydraulic conductivity or permeability of the rock and paleo-soil formations. These tests were carried out at a constant head. Flow was measured at 5, 10 and 15 or 20 minute intervals.

The results of packer testing are plotted as histograms in Figure 5f and summarized in Table 4.

3.4.3 Goodman Jack Testing

A Goodman Jack was used to measure borehole wall deformation as a function of applied load. These measurements were used to calculate the in situ elastic deformation moduli (Young's Modulus, E) of both the basalt and paleo-soil.

The results of Goodman Jack testing are reproduced Table 5.

3.5 Laboratory Testing

Data from the results of the laboratory testing undertaken by MMI was reported to Golder Associates by memorandum. This data included:

- the results of uniaxial compressive strength testing (memorandum dated 17 December 1986); and
- the results of direct shear testing on saw cut samples of the basalt core and on a sample of flow boundary material (memorandum dated 16 January 1987).

Data on the strength and deformation properties of the Roller Compacted Concrete dam construction material were provided through MMI

TABLE 4

SUMMARY OF RESULTS OF PACKER TESTS
FROM MINERAL SYSTEMS INC. INVESTIGATION OF MINER FLAT DAM SITE

	Basalt			Paleo-Soil		Sandstone	Gypsiferous Sandstone	
	Vertical Holes	Angle Holes	Low-Angle Holes	Vertical Holes	Angle Holes	Vertical Holes	Vertical Holes	Angle Holes
Maximum	1.17×10^{-4}	5.71×10^{-4}	1.15×10^{-4}	3.54×10^{-4}	3.85×10^{-4}	1.73×10^{-3}	2.40×10^{-6}	8.72×10^{-5}
Minimum	5.20×10^{-8}	8.26×10^{-7}	7.47×10^{-5}	9.56×10^{-6}	1.06×10^{-4}	2.08×10^{-5}	1.55×10^{-6}	4.33×10^{-5}
Average	1.99×10^{-6}	5.11×10^{-5}	9.37×10^{-5}	5.92×10^{-5}	1.76×10^{-4}	2.87×10^{-4}	2.12×10^{-6}	6.55×10^{-5}
Number of Tests	13	18	3	8	3	10	2	3

* Hydraulic Conductivity of Boreholes (cm/sec)

TABLE 5

RESULTS OF GOODMAN JACK TEST FROM
MINERAL SYSTEMS INC. INVESTIGATION OF MINER FLAT DAM SITE

Hole Number	Rock Type	Top of Interval Tested (Feet)	Young Modulus (psi x 10^6) Extend E(near)	Extend E(far)	Refract E(near)	Refract E(far)
MF-102	Basalt	40.5	0.68	0.59	1.02	0.81
MF-102	Basalt	80.5	0.42	0.34	0.85	0.70
MF-102	Basalt	160.5	1.32	1.31	2.06	1.96
MF-105	Basalt	18.8	0.45	0.29	--	--
MF-105	Basalt	39.1	0.87	0.58	1.51	1.29
MF-105	Basalt	59.6	0.78	0.036	--	--
MF-113	Basalt	80.8	2.46	1.66	2.21	1.48
MF-113	Basalt	101.5	2.58	2.22	2.76	2.46
MF-113	Paleo-alluvium	121.5	0.16	0.07	0.46	ERR
MF-113	Paleo-alluvium	130.6	53.91	15.58	--	--
MF-113	Paleo-alluvium	133.0	0.03	0.35	--	--
MF-117	Basalt	10.0	0.94	0.57	0.93	0.87
MF-117	Basalt	30.0	1.49	0.72	1.51	1.06
MF-117	Basalt	50.0	0.99	1.02	1.39	1.12
MF-117	Basalt	90.2	1.13	0.74	1.62	1.86

by an independent consultant, E.K. Schrader, in a note dated Nov. 5/6/7, 1986.

3.5.1 Uniaxial Compressive Testing

The uniaxial compressive testing program was undertaken by the University of Arizona on behalf of MMI. This testing program was used to determine uniaxial compressive strength, density, elastic moduli and Poisson's ratio, for each of the samples tested. The test data are reproduced in Table 6. The uniaxial compressive strength data are also plotted as points on the dam axis section in Figure 5e.

3.5.2 Direct Shear Testing

A number of direct shear tests were done on saw cut surfaces from NX core samples of the basalt. In addition, a direct shear test was carried out on a composite sample of clay and basalt flow boundary material recovered from a number of drill holes. Each of the samples were tested at normal stresses of 50, 100, 150 and 200 psi. The saw cut tests were used to determine the base friction angle of the basalt. The composite sample was used to determine the residual friction angle of the flow boundary material.

The results of Direct Shear Testing are reproduced in Table 7.

TABLE 6

RESULTS OF UNIAXIAL COMPRESSIVE TESTING PROGRAM
UNDERTAKEN BY MORRISON-MAIERLE INC.

Sample	Rock Type	Drill Hole	Depth (ft)	Uniaxial Compressive Strength	Density (lbs/cu ft)	Elastic Constants	
						Elastic Mod (psi)	Poisson Ratio
U-2A	MAS BASALT	DH-2	16.8 - 26.1	15,737	178	4,501,078	.27
U-13A	MAS BASALT	DH-106	155.0 - 155.7	16,837	173	2,554,148	.19
U-1A	MAS BASALT	DH-2	6.9 - 16.8	50,400	178	5,533,514	.25
U-3A	MAS BASALT	DH-2	54.5 - 62.4	32,943	178	8,044,664	.42
U-15A	MAS BASALT	DH-106	158.9 - 159.7	18,962	176	4,324,483	.29
U-4A	MAS BASALT	DH-2	90.2 - 100.5	15,064	177	8,705,493	.40
U-14A	MAS BASALT	DH-106	156.2 - 157.4	18,104	174	4,036,184	.21
U-2B	MAS BASALT	DH-2	16.8 - 26.1	21,963	177	---	-
U-3B	MAS BASALT	DH-2	54.5 - 62.4	31,845	178	---	-
U-13B	MAS BASALT	DH-106	155.0 - 155.7	25,746	173	---	-
U-15B	MAS BASALT	DH-106	158.9 - 159.7	18,512	176	---	-
U-16	MAS BASALT	DH-106	215.0 - 218.9	22,450	181	---	-
U-7	MAS BASALT	DH-5	45.3 - 47.2	11,143	166	---	-
U-14B	MAS BASALT	DH-106	156.2 - 157.4	27,195	175	---	-
U-18	MAS BASALT	DH-2	6.9 - 16.8	45,075	178	---	-
U-10	VES BASALT	DH-106	41.1 - 43.0	10,569	149	7,002,875	.29
U-5	VES BASALT	DH-5	19.5 - 21.0	11,095	161	4,254,604	.30
U-11	VES BASALT	DH-106	50.6 - 52.5	15,133	171	---	-
U-12	VES BASALT	DH-106	118.4 - 120.2	6,833	134	---	-
U-8	VES BASALT	DH-5	104.7 - 106.2	6,494	125	---	-
U-9	VES BASALT	DH-5	132.0 - 132.9	6,530	143	---	-
U-6	VES BASALT	DH-5	23.5 - 24.4	3,218	132	---	-
U-17	SILTY SS	DH-106	306.8 - 309.8	1,588	151	521,081	.27
U-20	GYP SIL SS	DH-106	337.1 - 338.7	3,105	148	1,080,152	.13
U-22	GYP SS	DH-102	358.5 - 360.5	1,882	142	1,291,449	.07
U-18	GYP SS	DH-106	332.2 - 333.7	3,101	144	3,039,394	.16
U-19	GYP SIL SS	DH-106	336.8 - 336.8	2,715	153	765,127	.24

Notes:
MAS - massive
VES - vesicular
SS - sandstone
GYP - gypsiferous
SIL - silty

TABLE 7

RESULTS OF DIRECT SHEAR TESTS FROM
TESTING PROGRAM UNDERTAKEN BY MORRISON-MAIERLE INC.

SAMPLE	FRICTION ANGLE (degrees)
S1B - Saw Cut Basalt	36*
S2A - Saw Cut Basalt	35
S2B - Saw Cut Basalt	38*
S3A - Saw Cut Basalt	40
S3B - Saw Cut Basalt	33
S4A - Saw Cut Basalt	40*
S4B - Saw Cut Basalt	28
Upper Basalt	39.5**
Flow Boundary	23

* Base Friction Angle
** Residual Friction Angle

As part of the direct shear test program, Atterberg Limits were determined from small sample of clay joint infilling from the Upper Basalt flow boundaries. The results of this testing were,

liquid limit 65%
plastic limit 29%

3.5.3 RCC Dam Material

The strength and deformation properties of the RCC dam material provided by MMI, are reproduced in Table 8.

TABLE 8

PROPERTIES OF RCC DAM MATERIAL PROVIDED BY MORRISON-MAIERLE INC.

PROPERTY	Cohesion (psi)	Friction Angle (degrees)	Density lb/ft^3	Deformation Modulus (psi)
usual - 6 month	100	45	-	-
usual - 1 year	-	-	153	2.0×10^6
unusual - 1 year	115	50	-	-
extreme - 1 year	155	55	-	-

4.0 MATERIAL PROPERTIES

To develop a model of the dam foundation it was necessary to assign realistic geotechnical properties to each of the rock units and to the paleo-soil. Of particular importance was the determination of rock and paleo-soil strength and deformation parameters. The necessary parameters have been inferred from available geologic data, laboratory test results and from published data.

4.1 Summary of Geologic Conditions

The available data, indicates that for purposes of engineering analysis the foundation of the proposed Miner Flat Dam can be divided into three geologic units, a surface basalt layer, an underlying paleo-soil layer, and a basement consisting of sandstone and gypsiferous sandstone. The distribution of these units is shown on geologic cross-sections, Figure 7a) to 7c).

The ancestral valley of the North Fork of the White River was carved into sandstones of the Permian Age Supai Formation. These sandstones are pale reddish brown to yellowish brown, fine grained and well sorted. Beds within the sandstone range in thickness from less than an

inch to 15 feet. The thickest beds are massive and conspicuously cross bedded. The beds of the Supai Formation strike northeast to northwest and dip about 3 to 7 degrees to the east. Joints are spaced at intervals of 0.5 to 3 feet and typically strike at right angles or parallel to the strike of the beds. The sandstone joints dip at 60 to 90 degrees. At depth the sandstone becomes gypsiferous. The gypsiferous member and the sandstone above are considered as one unit for analysis purposes.

Along the walls of the ancestral valley, weathering products of the sandstones accumulated as colluvium, and flooding in the valley bottom deposited alluvium derived from erosion upstream. Subsequently basalt flowed down the ancestral valley pushing the colluvium and alluvium aside or sandwiching it against the surface of the underlying sandstone. The result of the basalt flow was the development of a paleo-soil which, in the vicinity of the dam foundation, ranges in thickness from less than an inch to as much as 40 feet.

The paleo-colluvium is thought to consist of weathered angular blocks of talus sandstone, or pieces of sandstone bound in matrix of silt and clay derived from slope wash deposits. The paleo-alluvium is thought to consist of clay, silt and sand, and gravel cobbles and boulders composed of igneous metamorphic or sedimentary rock. Since core recovery was very low in this material and the range of material types and possible grain size distribution may be variable, the paleo-colluvium and paleo-alluvium have been considered as one unit for analysis purposes. Observations of the paleo-soil in limited outcrop indicates that the material may stand in vertical cuts up to 30 feet high. The paleo-soil is believed to be dense. Hydraulic conductivities of 10^{-4} to 10^{-5} cm/sec were measured in packer tests.

The basalt, which will form the immediate foundation and both abutments of the dam, consists of a series of flows and is very dark grey to black, weathering medium to dark grey. It is fine grained to aphanitic and locally porphoritic and contains vesicules of less than 0.05 in to 0.15 in. The vesicules occur in bands of 1 inch to 1 foot wide. Individual flows range from about 20 to 80 feet in thickness and there is typically a thin clay seam of 0.1 to 4 inches between flows.

Discontinuities in the basalt are related to flow banding and joints are the result of basalt cooling. These joints have an average spacing of 1.8 feet and an average length of 3.9 feet. The jointing pattern is columnar, as shown in Figure 8, with predominantly horizontal and near vertical orientation. Generally the basalt is considered to be of good to very good quality.

4.2 General Approach to Parameter Selection

4.2.1 Parameters for Stability Analysis

The necessary input parameters for the analysis of the foundation stability against sliding are γ , the material unit weight, c, the cohesion and ϕ, the friction angle. Laboratory results were available from which base frictional strength of the basalts and the residual strength of the flow boundaries could be estimated. No direct data was available on the in situ strength of the basalt or sandstone rock masses. The paleo-soils were particularly difficult to describe in geotechnical terms. This was a result of poor recoveries and difficulties experienced in sampling. No paleo-soil material was therefore available for testing.

To estimate the in situ strength of the rock it was necessary to utilize the geologic data to rate the rock mass according to its quality and from this rating assess a range of rock mass strength parameters.

For the paleo-soils it was necessary to rely on available published data, and this resulted in selection of lower bound strength envelopes for analysis. For the RCC dam material, both the data supplied by MMI and data published in the proceedings of the the ASCE Symposium on Roller Compacted Concrete (Denver, Colorado, May, 1985) were utilized.

4.2.2 Parameters for Stress and Deformation Analysis

The necessary input parameters for the analysis of foundation stresses and deformations are Young's modulus of deformation, E, Poisson Ratio, ν , as well the c, ϕ, and γ, the strength and density parameters discussed in the previous subsection.

Some laboratory data for E and ν , for the intact basalt and sandstone were provided as were typical data for the RCC dam material. Little data however was available to determine the deformation parameters of the paleo-soil.

In order to arrive at values of deformation parameters for the rock mass, a rock mass classification method was used in conjunction with the results of the laboratory and in situ Goodman Jack testing.

Deformation parameters for the paleo soil were determined primarily from published data. This resulted in a wide range of values being considered.

4.3 Rock Mass Classification

4.3.1 Classification Systems

In recent years rock mass classification systems have been developed to make use of measurable parameters in an attempt to minimize judgemental bias and to permit a quantifiable rock mass rating. Two

widely used rock mass classification systems are the Q System (Barton et al, 1974) and the Rock Mass Rating System (Bieniawski, 1974, 1976). Bieniawski's CSIR RMR method was used for this study. The parameters required for input to determine an RMR value are;

- strength of the intact rock material
- drill core quality (RQD)
- spacing of joints
- condition of joints
- groundwater condition

Each of these five input parameters is assigned a rating based on the descriptions given in Table 9. The RMR value is determined by summing these assigned ratings.

4.3.2 Classification of Rock Units

The RMR classification values for the rock mass in the vicinity of the dam are plotted as histograms on the Geologic Section in Figure 9. These values were determined by assigning ratings from Table 9, to each of the core run intervals. The RQD, uniaxial compressive strength and fracture index parameters were obtained from the data plotted on Figures 5a to 5c. Generalized factors were assigned for joint and ground water conditions.

- Basalt

Based on the description of the joints provided in the MSI reports a joint condition rating of 20 and a ground water rating of 7 was assumed. The resulting RMR value for the basalt rock mass ranged from 50 to 90, though most values fell in the range 55 to 75. An average value of 65 was selected for analysis purposes.

ROCK MASS RATING CHART

TABLE 9

CSIR GEOMECHANICS CLASSIFICATION OF JOINTED ROCK MASSES

CLASSIFICATION PARAMETERS AND THEIR RATINGS

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Rock strength rating	R5	R4	R3	R2	R1 For this low range - uniaxial compressive test is preferred		
		Uniaxial compressive strength	>29 ksi	14.5 - 29 ksi	7.25 - 14.5 ksi	3.6 - 7.25 ksi	1.5-3.6 ksi	0.5 ksi	<0.5 ksi
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Fracture index frac./ft.		< 0.1	0.1 - 0.3	0.3 - 1.0	1.0 - 6.0	>6.0		
	Rating		30	25	20	10	5		
4	Condition of joints		>10 Very rough surfaces Not continuous No separation Hard joint wall rock	6 - 10 Slightly rough surfaces Separation < 1 mm Hard joint wall rock	2 - 6 Slightly rough surfaces Separation < 1 mm Soft joint wall rock	0 - 2 Slickensided surfaces or Gouge < 5 mm thick or Joints open 1 - 5mm Continuous joints	Soft gouge > 5mm thick or Joints open > 5mm Continuous joints		
	Rating		25	20	12	6	0		
5	Ground water	Inflow per 10m borehole length	None		< 25 litres/min.	25 - 125 litres/min.	> 125 litres/min.		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	OR 0		OR 0.0 - 0.2	OR 0.2 - 0.5	OR > 0.5		
		General conditions	OR Completely dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Rating		10		7	4	0		

ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 — 81	80 — 61	60 — 41	40 — 21	< 20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Modified from Bieniaski.

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

o Sandstone

Based on the description of joints provided, a joint condition rating of 12 and a ground water rating of 7 was assumed. The resulting RMR values for the sandstone rock mass ranged between 40 and 70, though most values fell between 45 and 65. An average value of 55 was selected for analysis purposes.

4.4 Strength Parameters

The dam and foundation can be regarded as a discontinuum consisting of a series of blocks of intact rock and roller compacted concrete separated by various planes of weakness. The behaviour of this system will be controlled by both the strength of the intact materials and the strength along planes of weakness (e.g. joints, flow boundaries, and contacts).

4.4.1 Rock Mass

An empirical rock mass failure criterion, developed by Hoek and Brown (1980), has been used to predict rock mass strength of the basalt and sandstone. The criterion consists of a number of empirical equations which allow strength envelopes to be constructed for different rock types and qualities. The empirical failure criterion, illustrated on Figure 10, is defined using the constants m and s according to the equation:

$$\sigma_1' = \sigma_3' + (m \sigma_c \sigma_3' + s \sigma_c^2)^{1/2}$$

where σ_1' and σ_3' are major and minor principal effective stress and σ_c is the uniaxial compresive strength of the rock.

The constants m and s can be determined using the equation

$$m/mi = \exp (RMR-100)/14$$
$$s = \exp (RMR-100)/6$$

where RMR is the rock mass rating and mi is the value of m for intact rock as described in Figure 10.

The basalt at the Miner Flats has been classified as a "good quality mass" of "fine grained polyminerallic igneous crystalline rock." The sandstone at the site has been classified as a "fair to good quality mass" of "arenaceous rock with strong crystals and poorly developed crystal cleavage."

Estimated strength envelopes for the basalt and sandstone are shown on Figure 11 for a range of RMR and UCS values. These envelopes represent peak strength conditions of in-place rock masses and assume that the rock mass is homogeneous and isotropic. Mid range values of c and ϕ, selected for analysis from these envelopes, are c = 140 psi, ϕ = 55° for the basalt, and c = 70 psi, ϕ = 40° for the sandstone. Field experience has shown that rock mass strengths calculated using m and s parameters defined by this failure criterion are conservative.

4.4.2 Paleo-Soil

Poor core recovery in the paleo-soil and the difficulty associated with sampling and testing materials of this type resulted in very little field data, and no laboratory data. The shear strength of the paleo-soil was therefore determined primarily from published data on the strength of compacted rockfill. As the paleo-soil consists of a broad range of particle sizes from boulders and cobbles to silt and clay, the rockfill analogy is considered a reasonable approximation.

It has been assumed that the paleo-soil consists predominantly of detrital material from the sandstone rock which existed at the surface prior to the basalt flows. The shear strength of compacted sandstone rockfill has been measured by Charles and Watt (1980). Their findings are reproduced in Figure 12a. From this graph an average value of $\emptyset$ = 35° and c = 14 psi is indicated for the range of stresses likely to develop within the paleo-soil zones. Other laboratory test data on the shear strength of rockfill has been published by Barton and Kjaernsli (1981). This data relates the effects of particle size, compressive strength of the intact rockfill material (σ_c), the in situ confining pressure (σ_n') and the particle roughness (R) to the friction angle $\emptyset$ using the formula:

$$\emptyset = R \log \left(\frac{s}{\sigma_n} \right) + \emptyset b$$

The factor s is an empirically determined equivalent strength parameter and is a function of average particle size and the unconfined compressive strength of the particle. Based on data presented by Barton and Kjaernsli for rockfill under confinement a value of = 0.25 was selected.

Using a base friction angle of 30° for sandstone, and a uniaxial compressive strengths ranging between 1000 and 2500 psi and, an equivalent roughness of about 8 (which corresponds to rough, subangular to angular colluvium material), a friction angle of approximately 35° is indicated within the range of normal stresses likely to develop within the sandstone. A probable range of friction angles for the paleo-soil, predicted by the above criterion, is plotted in Figure 12b.

4.4.3 Discontinuities and Flow Boundaries

The shear strength of uncemented discontinuities in rock is a result of two components. The first of these is termed the base friction angle, ϕ_b and refers to the frictional strength of a smooth surface. The base friction angle is inherent to the rock type and represents the minimum or residual strength for a discontinuity in that rock. The value of ϕ_b is not dependent on the scale of the test sample and the stress level in the direction normal to the surface.

The second component of the shear strength is the roughness caused by asperities along the discontinuity surface. At low normal stresses the effect of roughness is to cause dilation of the discontinuity during shear displacement, thereby increasing the angle of friction. At higher normal stresses, the mechanism of shear changes from dilation to shearing through the asperities. The failure envelope is curved and the total frictional strength is dependent on a third parameter, the stress level.

Barton and Choubey (1977) have proposed the following empirical relationship between the above three parameters and peak friction angle (ϕ_p along a discontinuity):

$$\phi_p = JRC.Log10(JCS/\sigma_n') + \phi_b,$$

where,

σ_n' = the normal stress acting across the joint

ϕ_b = the base friction angle for rock

JRC = the joint roughness coefficient

JCS = the uniaxial strength of the joint wall

From the results of laboratory testing, core logging and detailed line surveys, the following values were selected:

Base friction of basalt (ϕ_b) = 35° (see results of direct shear tests, Figure 9)

Joint roughness coefficient (JRC) = 7

Joint compressive strength = 15000 psi

At a normal stress across the joint of 50 psi, the estimate peak friction angle would be 52°; at 100 psi the peak friction angle would be 50°. Since the discontinuities are uncemented, a conservative value of c = 0, $\phi p = 50°$ was used for analyses. These strength parameters ignore any intact ridges which may occur along any potential failure surface or any steps in that surface.

Peak shear strength along flow boundaries was estimated at c = 0, $\phi = 40°$ based on the results of direct shear testing given in Table 7 and observed asperities and undulations along these surfaces.

4.4.4 Dam/Bedrock Contact

Shear strength along the dam/bedrock contact is a result of three components,

- rock/concrete friction properties
- rock/concrete adhesion value

- roughness and size of asperities along the rock surface

The first of these can be expressed as the base rock/concrete friction angle ϕ_{bc} which is similar to the base friction angle for uncemented discontinuities described in the previous subsection.

The second component of shear strength, the rock/concrete adhesion, can only be considered if cementing develops between the concrete and the rock. Since no test data is available to assign such a value, a conservative approach was taken and a value of zero was assumed.

The third component of the shear strength can be expressed as the roughness and asperity angle (i). Patton (1966) has demonstrated that the roughness and asperity angle (i) can be combined with the base friction angle (ϕ_{bc}) of the dam/bedrock contact to obtain an estimate of the peak frictional strength available along the contact.

Observations of the drill core, at the dam alignment and in the channel of the river made by MSI indicate that at least three orders of roughness and asperities exist. The first order asperites are on the scale of the drill core and have been termed the joint roughness effect. The second and third order asperities are of a scale greater than the drill core diameter and are inferred from observations in the river channel and on the erosion surface of the basalt flows. The second order asperites have been termed the waviness effect and the third asperites have been termed the step effect.

For failure to occur along the dam/bedrock interface the frictional properties between the concrete and rock must be exceeded and there must be shear through, or dilation over, the first order asperities (joint roughness), second order asperities (waviness) and third order asperi-

ties (steps) The roughness angle along the contact (i) is therefore a function of:

$$i = fn\ (\phi_j,\ \phi_w,\ \phi_s)$$

where ϕ_j is the effect of joint roughness, ϕ_w is the effect of waviness and ϕ_s is the effect of the steps.

The base friction angle ϕ_{bc} for the dam/bedrock contact has been estimated at 35° based on previous work by Robinson et al (1986). This work indicated that the value of ϕ_{bc} was approximately equal to the base friction angle ϕ_b of saw cut samples.

As discussed in the previous section, the joint roughness effect ϕ_j can be evaluated using the empirical relationship prepared by Barton and Choubey (1977 where

$$\phi_j = JRC\ log10\ (JCS/\ \sigma'_n)$$

As discussed previously the JRC is considered to be approximately 7 based on the reported results of detailed line survey and core logging data. The average uniaxial compressive strength of intact basalt is reported as 15,000 psi and the strength of the roller compacted concrete is estimated at 1,700 psi. Using a normal stress range of 50 to 100 psi and a JCS value of 1,700 psi, the joint roughness effect ϕ_j calculated from the above equation is approximately 9° to 11°.

The waviness and step effects have been estimated from MSI's field observations of the dam/bedrock contact angle. The reported observations indicate that the contact angle varies from horizontal to at least 80°. Since the proposed foundation excavation profile slopes downstream the step effect has been significantly reduced. A conservative estimate of the mean contact angle based on the MSI observations and the proposed

excavation profile, is 10°. Together these figures (ϕ_j + ϕ_w + ϕ_s) give a roughness angle (i) of about 20° and result in a probable peak friction angle, ϕp, of 55°.

During excavation, an evaluation of the size of joint blocks along the foundation should be carried out. Since any potential slip surface must either pass through the rock mass or dilate over the ridges formed by intact blocks, the step effect of intact joint blocks will, significantly increase the peak friction angle over the conservative estimate used here.

4.5 Deformation Parameters

4.5.1 Rock Mass

Rock mass deformation is influenced by both the deformation characteristics of intact rock and the deformation characteristics of discontinuities within the rock mass. The approach used to determine the modulus of deformation for the sandstone and basalt rock masses involved combining data from three separate sources. These were:

- measurement of deformation moduli from laboratory testing of intact rock samples;
- measurements of in-situ moduli using borehole Goodman Jack tests; and
- evaluation of in-situ moduli from empirical relationships between rock mass quality and published deformation moduli data.

Laboratory measurements of Young's Modulus and Poisson's Ratio have been carried out. The results of these measurements were given in Table 6. The measured deformation moduli for the basalt samples ranged between 2.55 x 10^6 psi and 8.71 x 10^6 psi and the measured Poisson Ratios ranged between 0.19 and 0.40. The average values for the basalt samples were 5.44 x 10^6 psi and 0.29 for the deformation modulus and

Poisson's Ratio respectively. The measured deformation modulus for the sandstone samples ranged between 0.52 x 10^6 psi and 3.04 x 10^6 psi with an average value of 1.34 x 10^6 psi, and the measured Poisson Ratio ranged between 0.07 and 0.27 with an average value of 0.17.

The results of borehole Goodman Jack Testing were reproduced in Table 5. The Goodman Jack Tests gave two sets of moduli, a loading stage (Extend) moduli and a rebound (Retract) moduli. A wide range of values were measured in the Basalt. Loading moduli ranging from 0.29 x 10^6 psi up to 2.58 x 10^6 psi were measured, while rebound moduli ranged from 0.7 x 10^6 psi up to 2.76 10^6 psi. The average moduli values for loading was 1.44 x 10^6 psi, and for rebound was 1.47 x 10^6 psi. Goodman Jack tests were not carried out in the sandstone.

The results of Goodman Jack testing in the basalt indicate values consistently lower than the laboratory test values. The construction of the Goodman Jack is such that when in a very hard material, such as the basalt, the stiff cylinders of the jack tend to generate line loads rather than full circumference loading which would occur in softer materials. This effect is difficult to quantify and therefore the jack results must be considered, at the least, to be approximate.

Empirical relationships between rock mass rating and in situ modulus for deformation have been presented by Bieniawski, (1978) and Serafim and Pereira (1983). These relationships are shown in Figure 13. The two charts shown, indicate that based on the RMR values calculated in Section 3, the in situ deformation modulus for the sandstone could range between 0.74 x 10^6 psi and 2 x 10^6 psi, while the in situ deformation modulus for the basalt could range between 2.9 x 10^6 psi and 4.35 x 10^6 psi.

The value of deformation modulus selected for the sandstone rock mass was 0.7 x 10^6 psi, which is the lower bound of the empirical range and approximately half the average laboratory measured value for intact specimens. A Poisson ratio of 0.25 was selected, corresponding approximately with the measured ratio in the laboratory.

The value of deformation modulus selected for the basalt rock mass was 3.6 x 10^6 psi which is about mid range for values determined empirically, slightly more than half the average laboratory measured value, and is slightly more than twice the values measured in situ with the Goodman Jack. A Poisson ratio of 0.25 was selected which corresponds approximately with measured ratios in the laboratory.

4.5.2 Paleo-Soil

Deformation of a material such as the paleo-soil is influenced by the in situ density of the material, the distribution of particle sizes within the material, the shape of those particles, and the deformation characteristics of the material constituting the larger particles. The amount of data available on these fundamental properties for the paleo-soil was very limited. For this reason it was necessary to rely almost entirely on published data on the deformability of rock fill. Consequently a wide range of possible deformation moduli values were considered.

Some of the most extensive testing of the deformability of soils of this type is that reported by Marsal (1973). Though the materials Marsal was working with may not be the same as those of which the paleo-soil consists, the values indicate a lower range of possible moduli for use in a sensitivity analysis. Marsal's data indicates that under initial loading conditions a reasonable lower bound deformation moduli for rockfill may be in the range of 7000 psi. This could be considered

an absolute lower bound value for the paleo-soil since it has subjected to a considerable initial load (pre-load) from the weight of overlying basalt. Marsal's data indicates that under reloading conditions a reasonable deformation moduli for rockfill may be in the range of 35,000 psi. In order to arrive at an absolute upper bound value for deformation moduli for the paleo-soil, the material could be considered as a very poor quality rock mass. Based on the work by Serafim and Pereira illustrated on Figure 13, a deformation modulus in the order of 0.8×10^6 psi would be reasonable for such a material.

The resulting values of deformation moduli used for the paleo-soil in the analyses are:

7,000 psi - lower bound
35,000 psi - mid range
800,000 psi - upper bound
3,600,000 psi - equivalent to basalt

A Poisson ratio value of 0.35 was selected for the paleo-soil.

A limited number of in situ borehole Goodman Jack tests were carried out in the paleo-soil as recorded in Table 6. Values of deformation modulus ranged widely from 0.07×10^6 psi to 53.9×10^6 psi. The limited number of these tests and the scattered results are inconclusive. Due to the lack of homogenity in the paleo-soil, uniform deformation of the borehole during a test and, test repeatability is unlikely. The deformation moduli interpreted from these tests have therefore not been used in selecting representative values.

4.6 Summary

Table 10 summarizes the estimated range of values for material properties.

5.0 ANALYSIS

Construction of the Miner Flats dam together with the impounding of the associated reservoir will result in a change in the state of stress within the foundation. Depending on the strength and deformation characteristics of the various foundation materials, these stress changes could result in a number of problems, including:

- o excessive shear stresses along adversely oriented discontinuties; such as joint surfaces or geologic contacts and;
- o generation of excessive differential settlements along the dam base.

It has been assumed that due to the high strength of basalts on which the dam will be founded that bearing capacity failure in the soil mechanics sense is not a problem. Specific analyses to calculate the factor of safety against such a mechanism have therefore not been carried out. In addition excavation of rock to develop the abutment walls could potentially give rise to slope stability problems.

The following methods have been used to evaluate these potential problems:

- o limit equilibrium solutions based on rigid block mechanics;
- o analytical solutions based on elastic theory; and
- o numerical solutions based on elastic theory.

TABLE 10

RANGE OF MATERIAL PROPERTIES

		Sandstone Rockmass	Basalt Rockmass	RC Concrete Dam	Paleo-Soil
c	high	100	200	175	14
(Cohesion)	mid	70	140	100	0
psi	low	20	70	35	0
Ø	high	45	65	55	38
(Friction)	mid	40	55	45	35
degrees	low	35	45	40	32
	high	150	175	-	145
(Unit Weight)	mid	145	165	153	140
pcf	low	140	160	-	135
E	high	2.1E6	4.5E6	2.0E6	80.0E4
(Modulus)	mid	0.7E6	3.6E6	0.8E6	3.4E4
psi	low	0.5E6	2.8E6	0.5E6	0.7E4
	high	0.30	0.30	-	0.50
(Poisson	mid	0.25	0.25	0.20	0.35
Ratio)	low	0.20	0.20	-	0.20

5.1 Methods

5.1.1 Limit Equilibrium Solutions

This type of analysis is based on rigid block mechanics and is ideally suited for evaluating the stability of rock slopes and the dam foundation against potential sliding failure.

The method of analysis used for both the dam foundation and the rock slopes was a two-dimensional, general failure surface, non-vertical slice technique developed by Sarma (1979). In this method the geometry of the sliding mass is defined by the co-ordinates of the corners of a number of three- or four-sided elements, while the phreatic surface is defined by the co-ordinates of its intersections with the slice sides. A closed form solution is then used to calculate the critical horizontal acceleration K_c required to induce a state of limiting equilibrium in the slope. The static factor of safety of the slope is then found by reducing the values of Tan ϕ and c, the cohesion to Tan ϕ/F and c/F (where F is the factor of safety) until K_c=0.

In order to determine whether the analysis is acceptable, a final check is carried out to assess whether all the effective normal stresses acting across the bases and sides of the slices are positive. If negative stresses are found, the slice geometry must be varied until these negative stresses are eliminated.

The Sarma analysis allows different shear strengths (defined by cohesion and angle of friction) to be specified for each slice base and side. In the analyses carried out at Miner Flats the interslice boundaries were assigned zero shear strength. These conditions were chosen to coincide with the standard method of analysis used by the U.S. Army Corps of Engineers (CEM 1110-2-2200, Gravity Dam Design). Water pressures acting on the sides and base of each slice were included in

the analysis. External forces due to water pressure in tension cracks or behind the dam were also incorporated.

5.1.2 Numerical Modelling

To model more precisely the foundation behaviour and, in particular the response of the paleo-soil horizon, finite element modelling has been used. In recent years, this method has become an increasingly powerful technique used in investigations of soil and rock mechanics problems. Its importance lies in its ability to solve statically indeterminate problems, principally the redistribution of stress in rock or soil mass, arising from deformations that take place due to excavation, or application of load.

Though it is realized that behaviour of the dam and foundation rocks is a three dimensional problem it was considered that for preliminary design purposes the problem could be simplified to two dimensions based on the assumption of plane strain. The plane strain condition specifies that strain in the Z-direction, ε_z , is zero. It follows from this assumption that the shear stress components τ_{zy} and τ_{xy} are also zero. From Hooke's Law:

$$\varepsilon_z = \frac{1}{E}\left\{\sigma_z - \nu(\sigma_x + \sigma_y)\right\}$$

since $\varepsilon_z = 0$

$$\sigma_z = \nu(\sigma_x + \sigma_y)$$

where E is the Young's Modulus and ν is the Poisson's Ratio.

The stress field, therefore, comprises the independent stress components σ_x , σ_y and τ_{xy}.

It is considered that the above assumption is valid for preliminary modelling the Miner Flats Dam and its underlying foundations. The z-axis of the model, equivalent to the length of the dam is greater than the x-axis or y-axis. The x-axis, equivalent to dam width, and the y-axis, equivalent to dam height, are approximately equal.

For the purposes of computation, the dam cross section is divided into a number of discrete elements. These elements are tied together at their corners and are constrained in deformation such that the joint between any two elements remains closed without contact stresses actually developing between them.

It can be shown that the force/displacement response of this mosaic of elements is identical to that of an elastic continum provided that the elements are small relative to the stress gradient across them. It follows that the size of element is immaterial in a uniformly stressed region. Large elements are sufficient where the variation in stress and displacement are gradual, that is, in locations remote from the excavation boundaries. In regions where the stress gradients are steep, the elements should be sufficiently small for the stress or displacement differences across them to be negligible.

Each element is subjected at its nodes to forces and displacements that are related to each other by the stress-strain characteristics of the element. At any one node, the sum of the individual element forces is equal to the external forces applied to the node, as a result of which the nodal displacements may be expressed in terms of the external nodal forces.

Since the nodal displacements and the external nodal forces can each be divided into two independent components, two such expressions can be found for each node. A set of simultaneous equations equal to

double the number of nodes and containing the same number of unknown components of displacement can, therefore, be set up.

Three of these equations are dependent, which correspond to the three rigid body degrees of freedom. Consequently, three components of displacement at selected nodes are to be equated to zero and the number of equations reduced by three. A solution of the remaining set of equations will yield the unknown components of nodal displacement whence the corresponding forces and stresses can be calculated.

5.1.3 Analytical Solutions

This type of analysis is based on closed form standard analytical elastic solutions such as those developed by Boussinesq and presented by Poulos and Davis (1978). The analysis procedure was used as a first approximation of potential foundation problems and as a check of the results from the finite element analyses.

5.2 Limit Equilibrium Analysis

5.2.1 Dam Foundation Stability

As discussed previously, three possible slip surfaces have been identified in the dam foundation. These slip surfaces are shown on Figure 14 and are identified as follows:

- slip surface 1 - along the dam/bedrock contact, from the heel to the toe of the dam;
- slip surface 2 - along a vertical tension crack at the upstream heel of the dam through a zone of potentially fractured basalt (defined by measured low RQDs) beneath the dam;
- slip surface 3 - through the paleo-soil layer which outcrops upstream of the dam, with breakout through intact basalt downstream of the dam toe.

Loading conditions for each slip surface are shown in Figure 14 and ranges for strength parameters used in the analyses of each slip surface are summarized in Table 11. Each analyses was carried out using a F.S.L. water elevation of 6082 feet, horizontal ice load of 5000 lbs per lineal foot and an effective horizontal silt load. Two different foundation drainage conditions were analyzed:

1. a uplift pressure varying linearly from F.S.L. at the heel of the dam to tailwater level at the toe of the dam, for the case where foundation drainage is inoperative.

2. an uplift pressure varying linearly from a value at the heel of the dam equal to 1/3 of the head difference (reservoir head minus tailwater head) to the tailwater value, for the case where a grout curtain and drainage system near the upstream heel of the dam is fully operative.

Each failure surface was also analyzed under a pseudo-static earthquake loading of 0.1 g horizontal and 0.05 g vertical. The hydrodynamic effect of the reservoir was modelled by increasing the water density. The earthquake loading analysis assumed that the foundation drainage and grouting measures were fully operative.

The results of the various analyses are given on Table 11. These results indicate that along the dam/foundation contact (slip surface 1), using estimated mid range values for strength (expected condition) along the contact of c = 0 psi and $\emptyset$ = 55°, the factor of safety against sliding failure is 2.20 for the condition with drains operable; 1.50 for the condition with drain inoperable; and, 1.65 under earthquake loading, assuming operable drains. The results also indicate that the factor of safety can be rapidly increased by increasing cohesion along the slip surface. This could be achieved by physically constructing a shear key or making due allowance for a cohesive bond between the dam and the foundation rocks.

TABLE 11

SARMA LIMIT EQUILIBRIUM STABILITY ANALYSIS OF
POTENTIAL SLIDING FAILURE IN DAM FOUNDATION

ANALYSIS	STRENGTH PARAMETERS				F.O.S.		
	Basalt		Paleo-Soil				
	Cohesion (psi)	Friction Ø (degrees)	Cohesion (psi)	Friction Ø (degrees)	Undrained	Drained	Earthquake*
Slip Surface 1							
Dam/Foundation Contact	0	40	-	-	0.90	1.40	1.05
	70	40	-	-	2.15	2.55	1.95
	0	55	-	-	1.50	2.20	1.65
	70	55	-	-	2.76	3.44	2.60
	0	65	-	-	2.30	3.30	2.50
	70	65	-	-	3.50	4.47	3.40
Slip Surface 2							
Low RQD Zone	0	40	-	-	1.05	1.44	1.04
	140	40	-	-	3.57	3.97	2.99
	0	55	-	-	1.79	2.46	1.76
	140	55	-	-	4.32	4.98	3.72
	0	65	-	-	2.68	3.68	2.65
	140	65	-	-	5.19	6.21	4.62
Slip Surface 3							
Paleo-Soil	0	40	0	35	1.03	1.19	0.89
	140	40	0	35	2.30	2.45	1.95
	0	55	0	35	1.34	1.46	1.18
	140	55	0	35	2.62	2.76	2.25
	0	55	0	40	1.45	1.61	1.26
	140	55	0	40	2.76	2.93	2.30

* Note: Combined With Drained Conditions

Analyses of slip through a potentially fractured zone below the dam (slip surface 2), using estimated lower bound strength values (expected worst case) of c = o and ϕ = 40°, indicate that the structure could be marginally stable under conditions where the drains became inoperative, or under earthquake loading. Using mid range strength values (expected condition) for the basalt of ϕ = 55°, c = 140 psi, the factor of safety against failure is 4.98 under the conditions where drainage is operative, 4.32 under condition of inoperative drains and approximately 3.72 under earthquake loading.

An analysis of a potental slip surface through the paleo-soil zone, with breakout through intact basalt (slip surface 3), was done using lower bound strength values (expected worst case) for the paleo-soil of zero cohesion and ϕ = 35° and estimated average strength values (expected condition) for the basalt of ϕ = 55°, c = 140 psi. The calculated factors of safety range from approximately 2.76 for the condition with fully operative drains, to 2.62 for the condition with inoperative drains, down to 2.25 under earthquake loading condition. The introduction of some cohesion and an increase in the friction angle of the paleo-soil will increase factors of safety; however additional sampling and testing of the paleosoil will be required to justify increasing these strength parameters.

5.2.2 Stability of Excavated Slopes

The left and right abutments of the dam will be excavated prior to construction of the dam. The proposed excavation line will result in the development of faces as steep as 80°. The basalts in the existing valley wall stand near vertical along the right abutment but are much flatter on the left abutment.

A study of the kinematics of failure of the left and right abutment slopes was carried out with the aid of the stereoplot shown on Figure 8. Though it was found that the majority of joints mapped dipped at angles steeper than the proposed slope angles, a number of potential failure geometries were identified. These included planar, wedge and toppling modes. Due to the steep dips, however, and the expected size of joints, such failures are anticipated to be of limited size.

In order to examine a possible worst case condition it was assumed that potential slip surfaces might develop from the crest of the abutments along steeply dipping cooling joints, through intact basalt to daylight along the zone of flow boundary material, identified during the drilling investigation. The geometry of these surfaces are shown in Figure 15.

Each abutment was analyzed using a range of strength parameters. Two different ground water conditions were assumed, namely:

- a fully drained condition with the piezometric surface at, or near, the elevation indicated during the field investigation.
- a condition of moderate water pressure under an artificially high piezometric surface.

The results of the limit equilibrium analyses of the excavated slopes are similar for both the right and left abutments and are given in Table 12. These results indicate that with the use of mid range strength parameters (expected condition) along the potential slip surface (ϕ flow boundary = 40°, zero cohesion; ϕ cooling joint = 40°, zero cohesion, and ϕ intact basalt = 55°, cohesion = 140 psi) the factor of safety is approximately 2.0 under fully drained conditions. An increase in piezometric level reduces the factor of safety to 1.40.

TABLE 12

SARMA LIMIT EQUILIBRIUM STABILITY ANALYSIS OF
POTENTIAL SLIDING FAILURE IN ABUTMENT SLOPES

Location	Friction Angle Flow Boundary (degrees)	Friction Angle Intact Basalt (degrees)	Cohesion Intact Basalt (psi)	Friction Angle Joints (degrees)	F.O.S.	
					Drained	Undrained
Left Abutment	40	40	0	40	1.02	0.75
	40	55	0	40	1.25	0.91
	40	55	140	40	2.01	1.65
	40	55	0	55	1.44	1.01
	40	55	140	55	2.41	1.85
Right Abutment	40	40	0	40	1.04	0.62
	40	55	0	40	1.29	0.75
	40	55	140	40	2.00	1.40
	40	55	0	55	1.48	0.83
	40	55	140	55	2.37	1.59

Minor slope stability problems such as individual block toppling or sliding may occur during slope excavation. The blocks of ground involved are expected to be small and are best stabilized during construction on an as need basis by measures such as rock bolting and meshing.

5.3 Finite Element Modelling

5.3.1 Description of Model

o Geometry and Mesh

The dam and foundation was modelled with the aid of a two dimensional finite element computer program (FES2D). This program used linear elastic plane strain quadralateral and triangular elements. The finite element mesh consisted of 687 nodal points and 743 elements.

Boundary conditions, external loadings, and the general layout of the mesh are shown in the inset of Figure 16. A detailed mesh "window" showing the dam and the foundation in the immediate vicinity of the dam is included on the figure with node and element numbers. The table in the top right hand corner of Figure 16 identifies nodes and elements which are referenced in the discussion of results. The mesh was drawn to allow for the analysis of three different sets of geologic foundation conditions, corresponding with those shown in profiles AA-AA', BB-BB' and CC-CC' on Figures 7a, 7b and 7c, respectively.

Within this overall model a number of different cases were examined. Changes were made in applied loads, and paleo-soil properties. Table 13 summarizes the parameters investigated in the individual cases. With one exception, all analyses were carried out on what was considered to be the most critical foundation geometry, that is a

TABLE 13

SUMMARY OF FINITE ELEMENT ANALYSIS RUNS

Case	Loads				Young's Modulus Paleo-Soil (psi)	Remarks
	Gravity	Water	Silt	Ice		
1	X	X	X	X	3.6×10^6	Full reservoir loads with paleo-soil Layer A at stiffness same as basalt.
2	X	X	X	X	8.0×10^5	Full reservoir loads with paleo-soil Layer A at upperbound stiffness.
3	X	X	X	X	3.5×10^4	Full reservoir loads with paleo-soil Layer A at mid range stiffness.
4	X	X	X	X	7.0×10^3	Full reservoir loads with paleo-soil Layer A at lower bound stiffness.
5	X	-	-	-	3.5×10^4	End of construction condition with paleo-soil Layer A at mid range stiffness.
6	X	-	-	-	7.0×10^3	End of construction condition with paleo-soil Layer A at lower bound stiffness.
7	X	X	X	X	3.5×10^4	Full reservoir loads with paleo-soil Layer B at mid range stiffness.

shallow, thick paleo-soil profile as represented by Profile A, Figure 16.

A single anlaysis was done using Profile B, Figure 16, in which the paleo-soil was both thinner and deeper. This profile is considered to represent the typical geologic conditions beneath the dam.

o Loading and Boundary Conditions

The loading and boundary conditions applied to the dam and rock mass are illustrated in Figure 16. Water loads were applied to the ground surface, upstream of the dam and to the upstream face of the dam. The elevation heads for these loads were 6082 ft for cases 1, 2, 3, 4 and 7. For cases 5 and 6 no water loads were incorporated, representing conditions at end of construction. In addition to water loads, an ice loading of 5000 lbs/ft was applied at the 6082 ft elevation and an effective silt loading was applied vertically to the ground surface, and horizontally against the dam.

It was assumed that residual tectonic forces existed within the foundation rock, a result of the depositional and erosional history of the area. Tectonic forces were therefore applied to the model in a horizontal direction and were assumed to be equivalent to the overburden load. In terms of stress that is:

$$\sigma_H = \sigma_V = \rho \cdot g \cdot h$$

where σ_H is the horizontal stress, σ_V is the vertical stress, g is the acceleration applied to the rock mass due to gravity, ρ the density of the rock and h the depth below ground surface.

The depth h was measured from the dam base. No account was therefore taken of the horizontal toe stresses that exist due to the proximity of the steep valley walls. This assumption is considered to give conservative results

No uplift water pressures were modelled beneath the dam. All the finite element analyses considered total, and not effective stresses. The influence of pore water pressure was incorporated in a subsequent manual calculations and analysis of the stress results in Section 5.3.3.

Boundary conditions imposed in the model, see Figure 16, are summarized as follows:

- o no vertical movement along the horizontal boundary at the base of the model was permitted. The bottom corners of the model were rigidly fixed;
- o no horizontal movement along the vertical boundaries at the upstream and downstream sides of the model were permitted; and
- o nodes and elements within the rest of the model were permitted freedom to move in any direction under response to stress conditions.

The application of external loads on the foundation in each analysis (water, silt, ice, and the dam) occurred simultaneously and instantaneoulsy. It is observed in the results that the dam material undergoes deformation due to self weight. These deformations would occur during construction as a matter of course but for purposes of simplifying the analysis this method was considered acceptable.

5.3.2 Deformation

Displacements along the base of the dam are largely dependent on the thickness, depth, and modulus of deformation selected for the paleo-soil layer. Plots showing the deformed shape of the dam as predicted by the modelling are provided in Figures 17a through 17g. These figures also give the calculated displacements for each of the cases analyzed. Table 14 provides a summary of displacement at the heel and toe of the dam. Shown on this table are total and differential settlements determined for the end of construction conditions, Layer A geometry; the full reservoir conditions using four different values of the deformation modulus in paleo-soil Layer A; and, for full reservoir conditions, Layer B.

The modelling results indicate that the construction of the dam over a soft paleo-soil layer of variable thickness (lower bound $E = 7 \times 10^3$ psi) could result in end of construction settlements at the base of the dam approaching one inch and end of construction differential settlements along the base of the dam approaching 1/2 inch. The effect of water, silt and ice loads (reservoir full condition) could cause an additional settlement at the heel of the dam of up to 3/4 inch and additional differential settlement along the base of the dam approaching 1/2 inch. The result is a net settlement of more than 2 inches and a differential settlement of more than 1 inch.

5.3.3 Stresses

The dam and reservoir system will impose stresses on the foundation rock and the paleo-soil layer. The distribution of stresses within the foundation will depend on the relative stiffness of the foundation materials and the magnitudes of the external loads imposed. The finite element analyses were carried out as total stress analyses and the

TABLE 14

SUMMARY OF CALCULATED VERTICAL DISPLACEMENTS
ALONG BASE OF DAM

CASE	Modulus of Deformation of:				
	paleosoil A 3.0×10^6 psi	paleosoil A 8×10^5 psi	paleosoil A 3.5×10^4 psi	paleosoil A 7×10^3 psi	paleosoil B 3.5×10^4 psi
	Inches	Inches	Inches	Inches	Inches
Settlement during construction at heel of dam.	0.15 (est)	0.19 (est)	0.48	1.41	0.23 (est)
Settlement during construction at toe of dam.	0.09 (est)	0.09 (est)	0.17	0.52	0.15 (est)
Differential settlement due to construction.	0.06	0.10	0.31	0.89	0.07
Settlement due to water loading heel of dam.	0.16	0.19	0.57	0.70	0.20
Settlement due to water loading toe of dam.	0.10	0.09	0.14	0.29	0.13
Differential settlement due to water loading.	0.07	0.10	0.43	0.41	0.08
Net settlement at heel of dam.	0.31	0.38	1.05	2.12	0.43
Net settlement at toe of dam.	0.19	0.18	0.31	0.81	0.28
Net differential settlement.	0.13	0.20	0.74	1.31	0.15

magnitude and orientation of maximum total principal stresses for each of the cases analyzed are shown on Figures 18a to 18g. The total principal stresses for each of the references elements are also shown on the figures.

The main reason for evaluating stresses distributions within the foundation was to assess the effects of the dam and reservoir loadings on the paleo-soil layer. A review of the stress tensors shown on Figures 18a through 18g indicate that the paleo-soil is well confined and that the magnitude of shear stresses developed within the paleo-soil are small.

The total stresses calculated in the finite element analysis have been used to determine factors of safety against failure of the paleo-soil. For the seven cases analyzed, the calculated factor of safety was 4 or greater for paleo-soil elements beneath the toe and heel of the dam.

Evaluation of effective stresses in the paleo-soil requires knowledge of the pore pressure distribution within the paleo-soil. Since data on anticipated pore pressure distributions after reservoir filling were not available, conservative estimates were made. It was assumed that at the end of construction the pore pressure in the paleo-soil Layer A would therefore be approximately 25 psi at element 495 beneath the heel of the dam, and that the pore pressure would drop to 20 psi at element 470 beneath the toe of the dam. Under reservoir full conditions it was estimated that the pore pressure in the paleo-soil layer would be approximately 60 psi at element 495 beneath the heel of the dam and that the pore pressure would drop to 45 psi at element 470 beneath the toe of the dam.

In Figure 19, Mohr circles, showing total and effective stresses for elements 495 and 470 beneath the heel and toe of the dam respectively, are plotted for both end of construction and reservoir full conditions. The circles lie well below the estimated strength envelope for the paleo-soil (see Section 4.4.2). Since it is anticipated that pore pressures in the paleo-soil will be lower than the values discussed above, the margin of safety against shear failure in the paleo-soil will be higher than shown on Figure 19.

5.4 Analytical Solutions

As a first approximation of possible stress distributions beneath the dam, a number of closed form analytical solutions were used. These assume that the foundation behaves as an isotropic homogenous mass.

The first case analyzed was at the end of construction condition with full dam weight on the foundation, but without reservoir loading (analagous to finite element cases 5 and 6). The dam was modelled as an infinite strip subjected to triangular loading. The resulting distribution of vertical stresses at the depth of the paleo-soil layer was very similar to that calculated by the finite element analyses.

The second case analyzed was also at the end of construction condition but this time the dam was modelled as a rectangular area subjected to triangular loading. At the center of the dam the elastic total vertical stress distribution was little changed from the case of the infinite strip. However, beneath the ends of the rectangle significantly lower total vertical stresses were calculated. At the depth of the paleo-soil layer total vertical stresses were approximately half those determined in the case of the infinite strip. End effects, which are not taken into account in the two-dimensional finite element anlaysis, are therefore significant.

The third case analyzed was intended to estimate the magnitude of stresses locked into the dam foundation as the result of downcutting of the river valley. The dam site was modelled as a homogeneous isotropic infinite mass subjected to a uniform geostatic load beneath a ground surface level coincident with the tops of the present valley walls. Downcutting of the valley and the resulting stress relief was modelled by removing a rectangularly loaded infinite strip from the valley floor. The results of the analysis indicate that for the steep walled, narrow valley modelled, stressed locked in beneath the edges of the valley floor are of similar magnitude to the anticipated stresses to be imposed by the proposed dam.

The results of these analytical solutions indicate that the assumptions made for the finite element analyses concerning geostatic stress level in the dam foundation and the assumed triangular loaded infinite strip configurations are conservative.

The three-dimensional effects of the river valley at the site will result in reduced stresses in the paleo-soil and consequently deformations of the paleo-soil and dam foundation will also be reduced.

6.0 DESIGN

6.1 Dam Stability

The various site investigation studies carried out at Miner Flats Dam have identified two zones in the foundation along which sliding or shear failure may be kinematically possible. These are:

1) a zone of low measured RQD within the basalt
2) the paleo-soil

In addition a third potential plane of weakness is respresented by the contact between the foundation rocks and the dam itself.

6.1.1 Zone of Low RQD

A number of holes drilled on the right abutment and at the dam mid point encountered low values of RQD. When the results obtained were plotted on a geologic section (see Figure 5b) it was possible to correlate the occurrance of these low RQD values between holes. This correlation was only possible however between holes on the right abutment and those at mid point. No correlation could be extended to the left abutment.

Downstream only a single borehole was available for correlation. Unfortunately due to the natural fall in rockhead, extrapolation resulted in the low RQD zone daylighting above this hole.

Several explanations exist for the low RQD's these include:

- o poor drilling practice resulting in drilling induced fractures;
- o the low RQD values are a result of near vertical fractures and threfore should not be correlated laterally; and
- o the presence of a low angle fracture zone.

A conservative approach has been adopted. It was assumed that that the low RQD's were due to the presence of low angle fracture zone. In addition it was assumed that this fracture zone extended over the entire footprint area of the dam and that a vertical release surface was available upstream. The resulting geometry was then analyzed.

The results (presented in Section 5.2.1) indicated that only under the most severe assumptions, of lower bound values for strength and a non-functioning drain system or earthquake loading, did the calculated factor of safety approach unity. On the basis of a more detailed examination of drill core carried by MMI subsequent to the analysis it would appear that the low RQD's were a result of poor drilling practice (as evidenced by grinding of core) and the presence of near vertical fractures. It would therefore appear that the lateral correlation of the low RQD's may not be justified.

Based therefore, on the results of a 'worst case' analysis and the findings of detailed core logging it is concluded that sliding or shear failure through a low RQD zone in the basalt has a sufficiently high factor of safety.

6.1.2 Paleo-Soil

The paleo-soil extends from sub-crop immediately upstream of the dam heel, and under the entire dam footprint. It is extremely variable, both in its depth beneath foundation level, its thickness and its consistency. Towards the left abutment the paleo-soil is approximately 35 ft thick and is some 40 ft below foundation level; 80 ft below foundation level and approximately 10 ft thick at dam mid point; and 60 ft below foundation level and approximately 20 ft thick toward the right abutment.

The soil profile found toward the left abutment was used for analysis; again, adopting a 'worse case' approach. In order to make a slip surface through the paleo-soil kinematically feasible, a downstream 'break-out' surface thorugh the basalts had to be assumed. From an examination fo rock fabric (see Figure 8) no joint surfaces with the required orientation have been encountered. The assumed 'break-through' surface will therefore be stepped, partially along existing joints and

partially through intact rock. This results in a high shear strength zone which will resist any down dip movement in the paleo-soil.

The results of the limit equilibrium analyses (see Section 5.2.1) and the finite element modelling (see Section 5.3) indicate that, even under extreme loading conditions, the calculated factor of safety is approximately 2 or greater. It is believed that this represents an adequate factor of safety, particularly as it is a result of analyzing 'worse case' geometry.

6.1.3 Dam-Foundation Contact

The contact between the dam and the rock foundation represents a surface along which sliding or shear failure may occur. Factors of safety were calculated using a range of strengths for the dam-foundation interface and using different loading conditions. The results indicate that based on the estimated strength along this contact the calculated factor of safety is in excess of 1.4 under extreme loading conditions (i.e. drains inoperative or earthquake loading) and in excess of 2.2 under normal operating conditions. It should be noted that the strength estimates were based on the assumption that the dam base had a relatively smooth profile and that no cohesion was developed. If the profile was more stepped, thus allowing for a greater degree of interlocking, the calculated factors of safety would be substantially increased. Similarly, if cohesion was developed along the contact, as has been demonstrated in recent testing carried by the U.S. Corps of Engineers at Willowdale Dam, the calculated factors of safety would be substantially increased.

6.2 Dam Settlement

The prediction of settlement along the base of the dam has been carried out with the aid of results obtained from two dimensional finite element modelling. Two foundation geometries were modelled. The first geometry (based on the geologic section given in Figure 7c) considered a paleo-soil layer approximately 10 ft thick and located some 80 ft below foundation level and is believed to represent the average or typical foundation geology. The second geometry modelled was based on the geologic section given in Figure 7a. It comprised a paleo-soil layer approximately 40 ft thick and was located some 35 ft beneath foundation level. This second profile is considered to represent a 'worse case' foundation geometry. Based on the results of the modelling (see Table 17) the predicted differential settlement between the toe and heel of the dam ranged between approximately 0.13 inches and 1.3 inches. The magnitude of the predicted settlement is substantially a function of the value assumed for the deformation modulus of the paleo-soil.

As a base case it was assumed that the paleo-soil had a deformation modulus equivalent to that of the basalt. Under this assumption the net differential settlement predicted was 0.13 inches. This value was similar to the predicated settlement for the deep thin paleo-soil geometry. It can be implied from this similarily that over the majority of the dam base where the paleo-soil is relatively deep and thin that differential settlements will be largely controlled by the dam geometry and the properties of the overlying basalt and should therefore be relatively small. Towards the left abutment, where the paleo-soil layer is thick and shallow, the properties of the layer will be the major influence on settlement. Under assumed 'worse case' conditions, that the paleo-soil is very soft, the predicted differential settlement between the heel and toe of the dam is approximately 1.3 inches. Under, however, the

assumed expected condition, the predicted differential settlements are approximately 0.75 inches.

To assist in planning construction procedures and to delineate areas that may require more investigation during detailed engineering, the area beneath the dam has been zoned, Figure 20. Each zone represents a particular settlement pattern and, as such, offers an approximate guide to the distribution of differential settlements across the dam base. Zone boundaries are based on the estimated depth to the paleo-soil from foundation level; while, settlement patterns within the zones are based on the results of the 2-D numerical modelling.

Settlements within Zones 1 and 2 (see Figure 20), where the paleo-soil is at depth, will be largely controlled by the properties of the basalt founding rocks. By contrast, settlement in Zone 4, where the paleo-soil is shallow, will be strongly influenced by the properties of the paleo-soil itself. Zone 3 represents a transition between Zone 2 and Zone 4.

Zone 1 coincides with the left and right abutments where the paleo-soil is generally in excess of 100 ft. below the dam/rock contact. Differential settlements are expected to be small ($\sim$ 0.1 inch).

Within Zone 2, the modelling results indicate that total settlements of up to 0.45 inch may occur along the dam heel, while up to 0.30 inch may occur along the dam toe; thus indicating a differential settlements of up to 0.15 inch between the heel and toe. Though, as yet, no modelling has been carried out parallel to the dam axis, little lateral differential settlement is expected within Zone 2 or between Zone 1 and Zone 2.

Calculated total settlements in Zone 4, (using expected values for paleo-soil properties) were 1.05 inch along the dam heel and 0.30 inch along the dam toe; resulting in a possible differential settlement between heel and toe of 0.75 inch. Zone 4 is located immediately adjacent to the left abutment. As settlement within Zone 4 takes place, it is believed that some stress transfer will take place from the "softer" Zone 4 foundation to the "stiffer" abutment foundation. Such a stress transfer will result in reduced total settlements and hence a decrease in differential settlement. However, as a result of the juxtaposition of "softer" and "stiffer" zones lateral differential settlements is expected to occur. Using the results of the numerical modelling (which because of 2-D nature are believed to represent a "worst" case) lateral differential settlements of up to 0.6 inch may occur between Zone 2 and Zone 4.

To enable more accurate predictions of differential settlements between the heels and toe of the dam and laterally more information is required on the geometry and properties of the paleo-soil underlying Zone 4. This should be addressed during the final detailed engineering studies.

It is believed that the very nature of the two-dimensional modelling has resulted in conservative predictions of values for settlement. This conservativeness is a result of the following:

- stress transfer along the contact between the dam and its abutments. This will reduce the load on the dam foundation;
- the effect of horizontal stress concentration (notch effect) at the base of the valley. This results in an additional clamping force across the dam foundation; and
- the assumption that the dam and the foundation rock are intimately connected (i.e. the assumption that the dam and foundation are a continuum).

These three factors would all tend to reduce the amount of settlement and hence the differential settlements that will occur over the dam base.

6.3 Slope Stability

The rocks that will be exposed during the excavation of left and right abutment slopes exhibit brittle rock characteristics. The stability in these rocks will therefore be controlled, by and large, by the orientation, spacing and size of discontinuities that exist within the rock mass.

The stability analyses has indicated that the calculated factors of safety against overall slope failure is adequate. However minor stability problems comprising sliding or toppling of small blocks are expected to occur on both abutment slopes. Due to the height of the slopes dislodgement of such blocks present a substantial hazard to men and equipment working below. Remedial measures must, therefore, be considered. These may include rock or cable bolting, mesh or the provision of catch benches. The use of smooth wall blasting techniques such as pre-split or cushion blasting will greatly assist in the control of local stability problems.

It is believed that the design of appropriate remedial measures is best accomplished during construction when the exposed rock mass can be fully inspected.

7.0 CONCLUSIONS AND RECOMMENATIONS

7.1 Conclusions

Aspects of foundation and abutment stability of the proposed Miner Flats Dam have been investigated with the aid of limit equilibrium

analyses and numerical modelling. Prior, however, to conducting these analyses appropriate input parameters had to be selected based on the results of the various site investigations and of laboratory testing.

The paleo-soil unit presented a particular problem. Due to sampling and testing difficulties little direct information was available for this unit. Published data had to be relied upon. Uncertainties associated with this approach therefore required that sensitivity analyses be conducted in order that the range of probable values for the input be investigated. Based on the results of these analyses the following conclusions were reached;

A) Sliding or Shear Failure

- Along Dam-Foundation Contact: Using mid range values for strength (the expected case) and normal loading conditions, the calculated factor of safety was 2.20; under assumed 'worst' case loading conditions the calculated factor of safety was 1.50. It is believed that these calculated factors of safety could be greatly increased by ensuring that a cohesive bond between the dam and the foundation rocks is developed, or, by providing a more irregular foundation profile, thus allowing substantial interlocking between the dam and the foundation rocks.

- Along Low RQD Zone: Based upon the results of a 'worst' case analysis and the findings of detailed core logging it is believed that a sufficient strength can be mobilized through the low RQD to essentially preclude a slip type of failure.

- Along/Through Paleo-soil: Based upon 'worst' case analyses, in terms of both assumed strength values and possible failure

geometries, it is believed that sufficient strength can be mobilized through the paleo-soil and the basalt to essentially preclude a slip type of failure. Inherent in this conclusion, however, is the assumption that the potential slip surface passes through the basalt mass and not along an existing plane of weakness.

B) Abutment Slopes

Planar, wedge, and toppling style failures are kinematically possible in both abutment slopes. Due to the attitude and size of joint surfaces, individual failures are expected to be of limited extent. Remedial measures will be necessary to ensure the safety of men and equipment working below these slopes.

c) Deformation of Dam Foundation

The results of the finite element modelling indicate the following foundation behaviour:

- The deformation of the dam foundation is sensitive to the depth, thickness and deformation modulus of the paleo-soil layer.
- The weight of the dam at end of construction causes settlement at the upstream heel of the dam, and rotation in the upstream direction.
- The weight of water within the reservoir depresses the base of the reservoir but the water pressure also causes rotation of the dam in the downstream direction.
- The paleo-soil is confined and is not subject to the development of excessive stress at either the end of the construction condition or their reservoir full condition.

- Predicted differential settlements along the dam base ranged from approximately 0.1 inch to 1.3 inches, depending on loading conditions and the assumed value for the deformation modulus for the paleo-soil. It is believed that these predictions are conservative due to the inherent limitations of the two-dimensional model used.

- Settlements are a function of dam geometry and paleo-soil thickness, depth and modulus. From available data these would appear to change gradually and therefore no sudden change in settlement is expected.

7.2 Recommendations

In deriving the above conclusions a number of assumptions were made, particularly with regard to paleo-soil properties and to foundation and abutment geology. The following recommendations therefore follow:

- little geologic information is available in the vicinity of the dam toe. Before construction at least three exploratory drill holes should be put down in this area to confirm or otherwise the predicted geology;

- the assumed deformation modulus for the paleo-soil is based on published data. An attempt should be made to more fully characterize this material in order to obtain better estimates of its properties;

- Smooth wall blasting methods should be used to excavate the abutment slopes in order to minimize vibration damage. As localized wedge and block failures are anticipated some form of remedial measure should be implemented; and

- As more detailed information becomes available the present geomechanical model should be up-dated and review analysis conducted.

Yours very truly,

GOLDER ASSOCIATES

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A. Moss

A. Rice, P. Eng.

EH/AM/AR/gg
862-1622

8.0 REFERENCES

Bieniawski, Z.T. Determining rock mass deformability: experience from case histories. Intl. J. Rock Mechanics and Mining Sciences, Vol. 15, 1978, pages 237-247.

Bieniawski, Z.T. Geomechanics classification of rock masses and its application in tunnelling. Proc. Third International Congress on Rock Mechanics, ISRM, Denver, Volume 11A, 1974, pages 27-32.

Bieniawski, Z.T. Rock mass classification in rock engineering, Proc. Symposium on Exploration for Rock Engineering, Johannesburg, Volume 1, 1976, pages 97-106.

Serafim, J.L. and Pereira, J.P. Consideration of the Geomechanics Classification of Bieniawski. Proc. Intl. Symp. on Engng. Geol. and Underground Construction. Lisbon, Portugal, 1983, pages 1133-44.

Hoek, E. and Brown, E.T. 1980, Underground Excavations in Rock, Institute of Mining and Metallurgy, London.

Hoek, E. Strength of Jointed Rock Masses. Geotechnique 33, No. 3, 1983, pages 187-223.

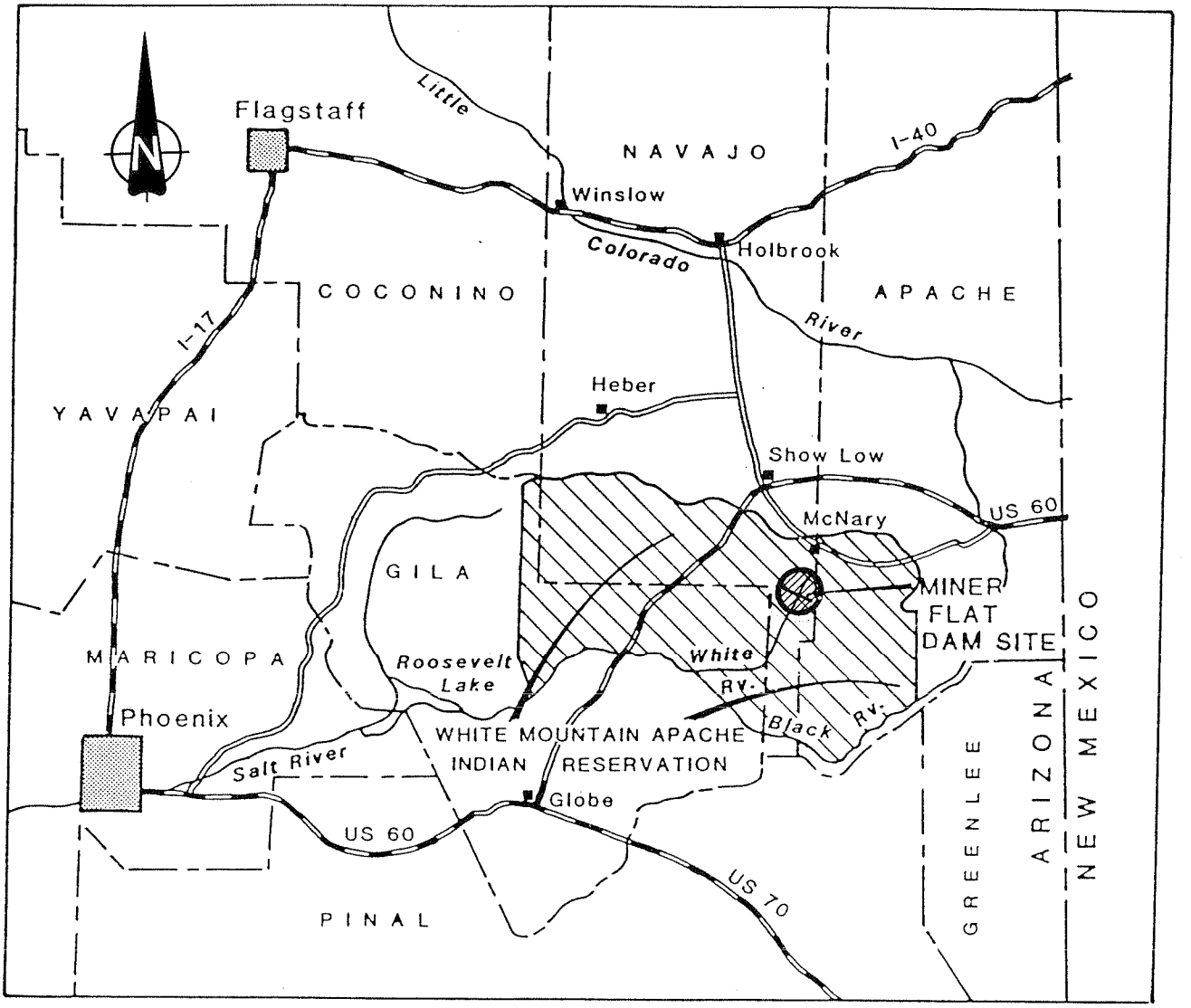
Charles, J.A. and Watt, K.S. The Influence of Confining Pressure on Shear Strength of Compacted Rockfill. Geotechnique 30, No. 4, 1980, pages 353-367.

Barton, N. and Kjaernsli, B. 1981 "Shear Strength of Rockfill". Journal of the Geotechnical Engineering Division, Proceedings, American Society of Civil Engineers, Vol. 107, G77, July 1981, pages 873-891.

Sarma, S.K. Stability Analysis of Embankments and Slopes. J. Geotechnical Engineering Division, Proceedings, American Society of Civil Engineers Vol. 105, G712, 1979, pages 1511-1524.

SITE LOCATION PLAN

Figure 1



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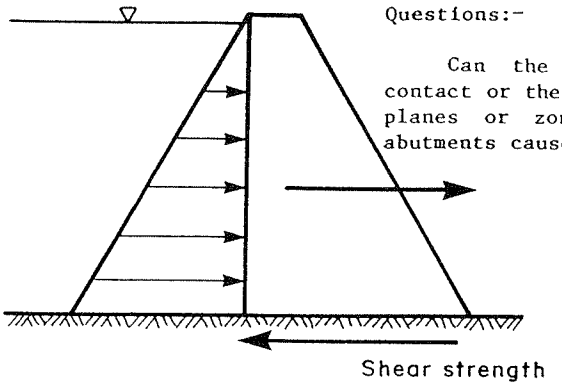
POTENTIAL FOUNDATION PROBLEMS

Figure 2

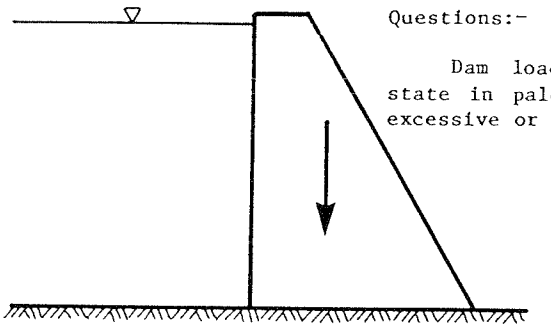
a) Stability



Questions:-

Can the strength along the dam/rock contact or the presence of adversely oriented planes or zones within the foundation or abutments cause instability?

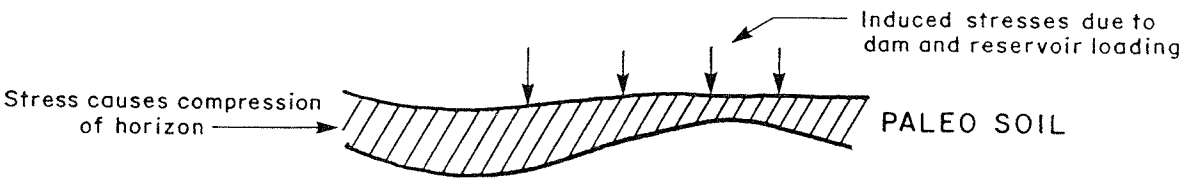
b) Settlement



Questions:-

Dam load results in change of stress state in paleosoil. Can this give rise to excessive or differential settlement?

BASALT



SANDSTONE

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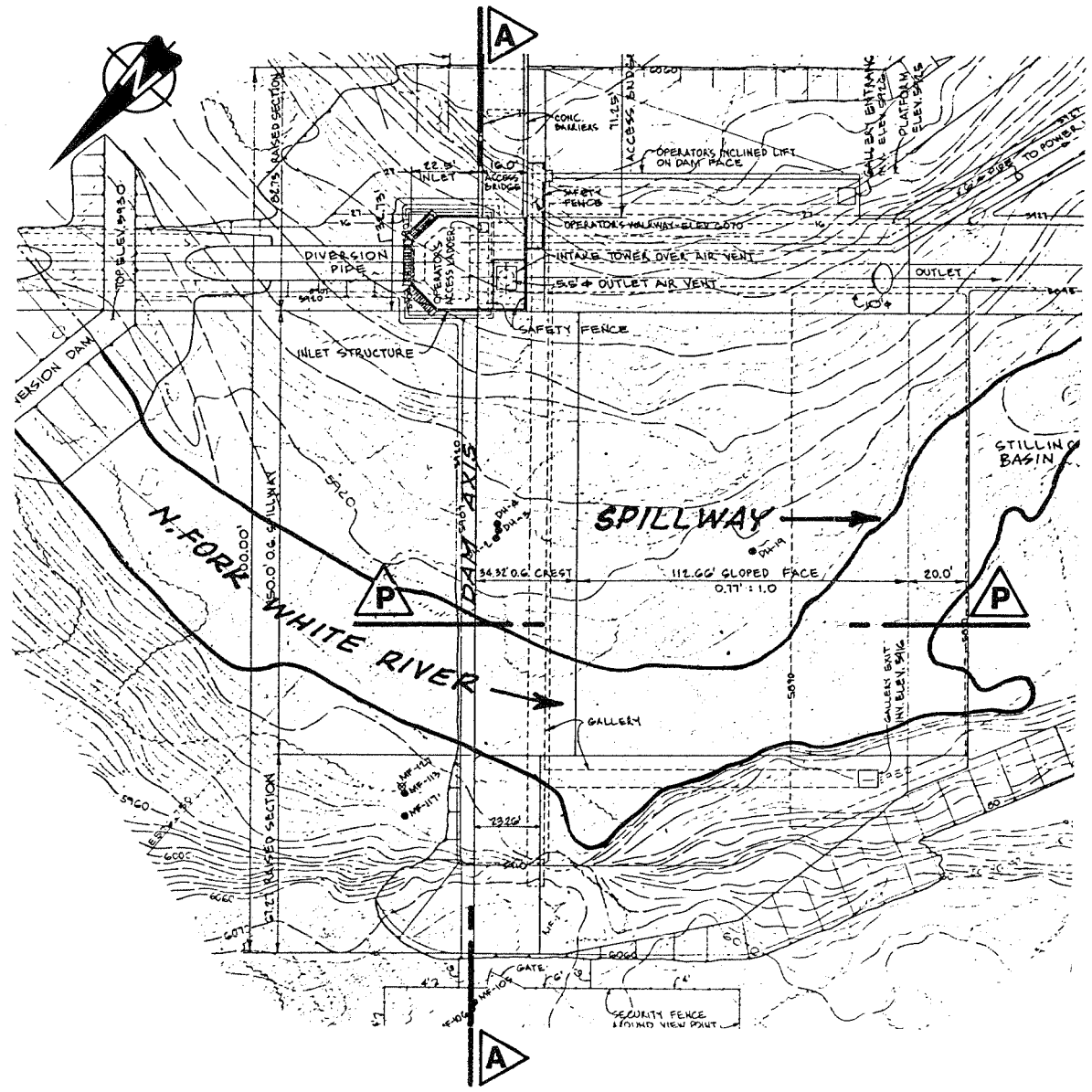
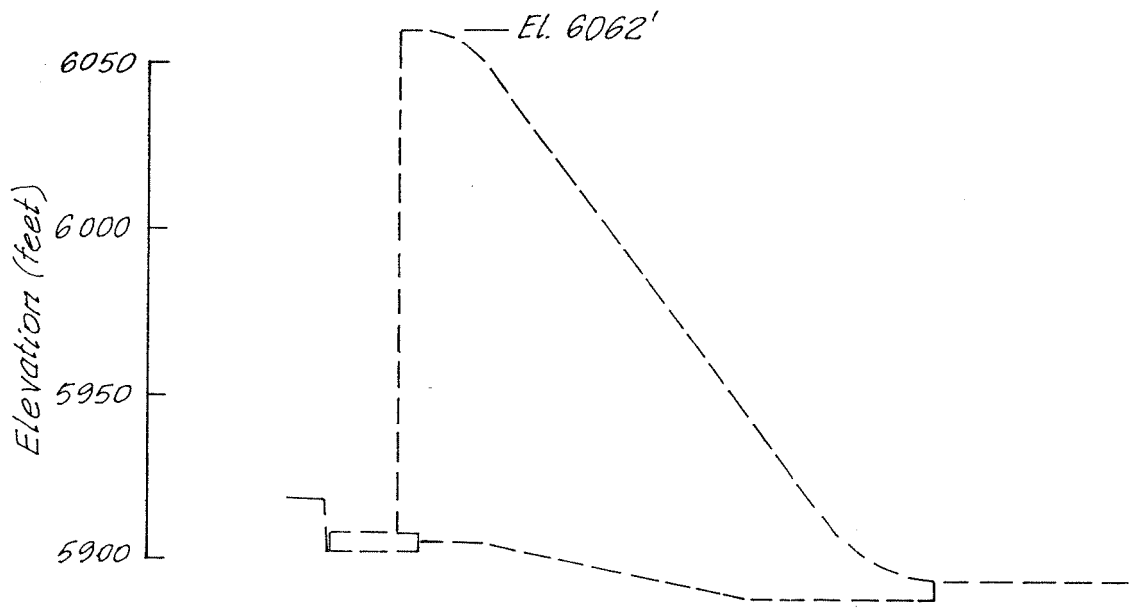
PROPOSED DAM LAYOUT

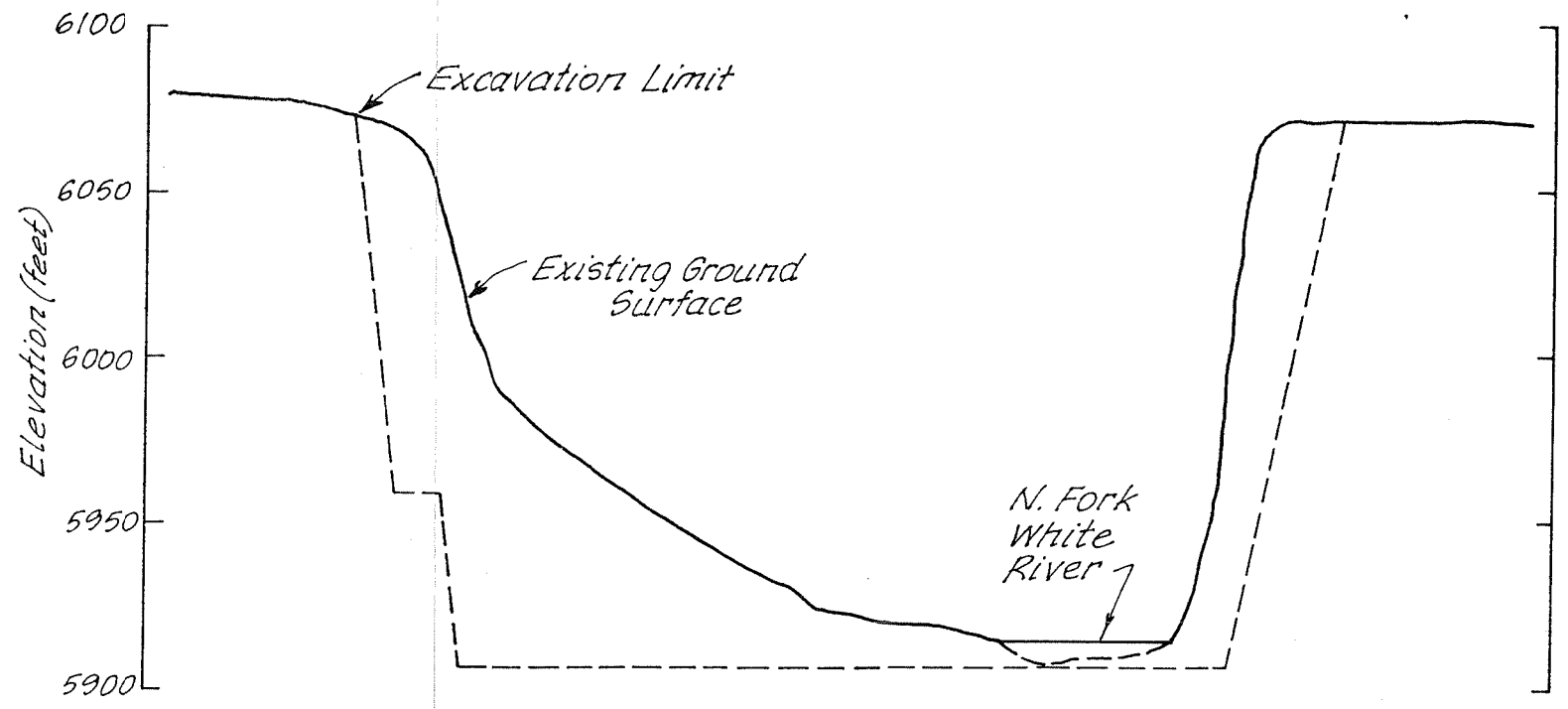
Figure 3



PLAN



DAM PROFILE P - P'



SECTION A - A'

Note: Geometric layout of proposed dam provided by Morrison-Maierle Inc. on drawings referenced in the text.

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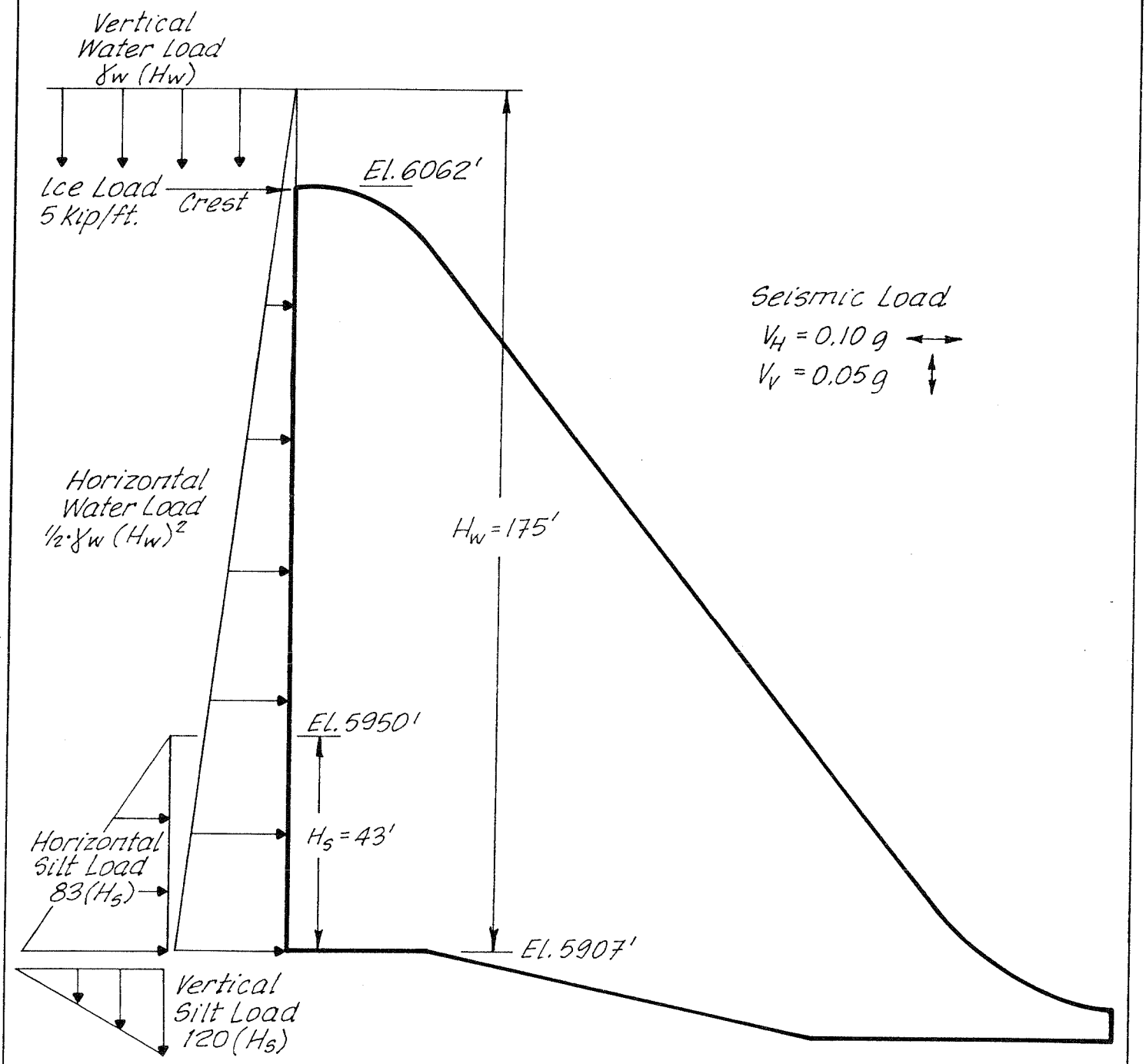
Scale: 1 inch to 60 feet.

BOUNDARY LOADING CONDITIONS

Figure 4



Note: Boundary loading conditions provided by Morrison & Maierle Inc.

Scale: 1 inch to 30 feet

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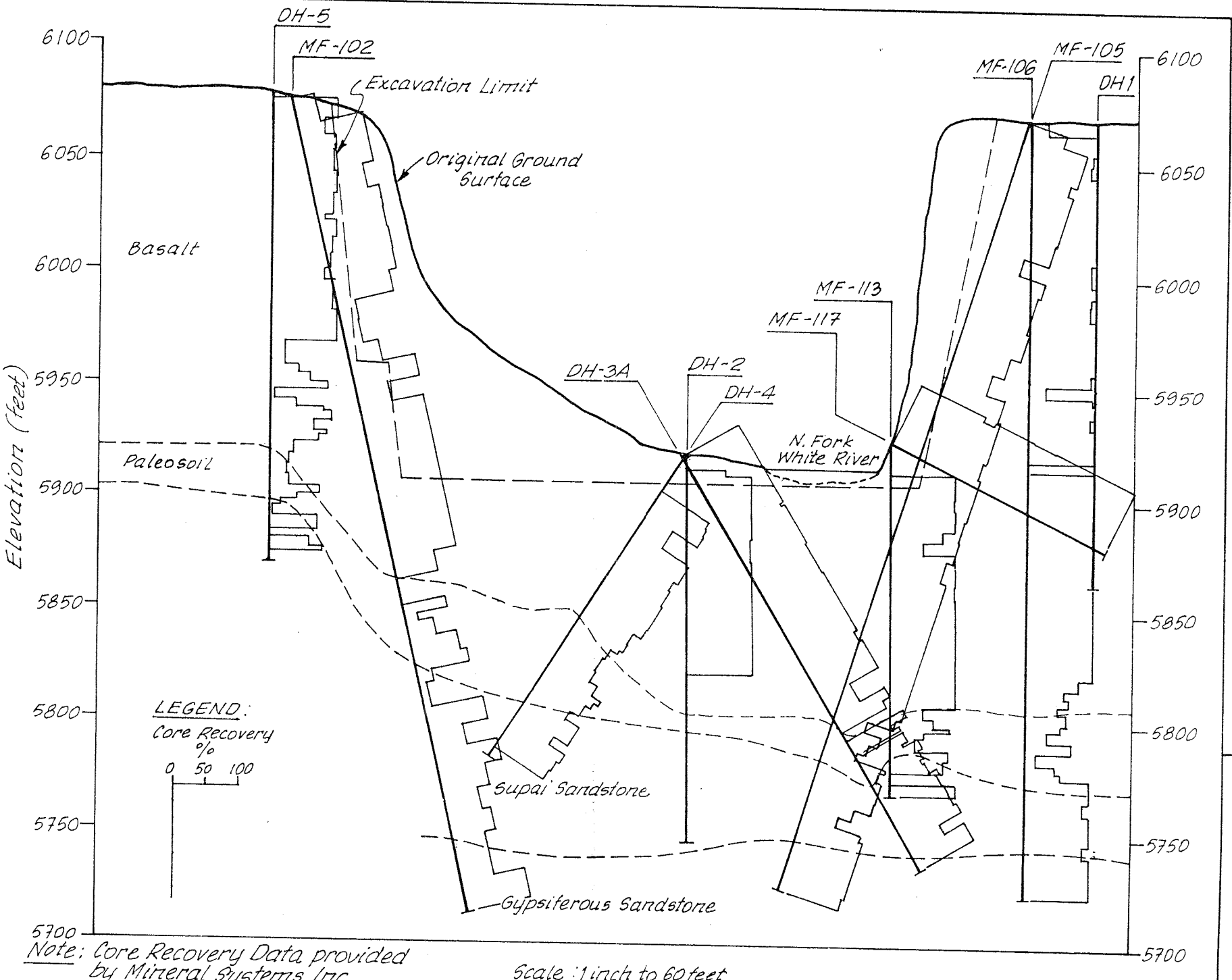
PROJECT NO. 862-1622 DRAWN S. REVIEWED DATE Jan. 187



Note: Core Recovery Data provided by Mineral Systems Inc.

Scale: 1 inch to 60 feet

GEOLOGIC SECTION WITH HISTOGRAMS OF CORE RECOVERY

Figure 5a

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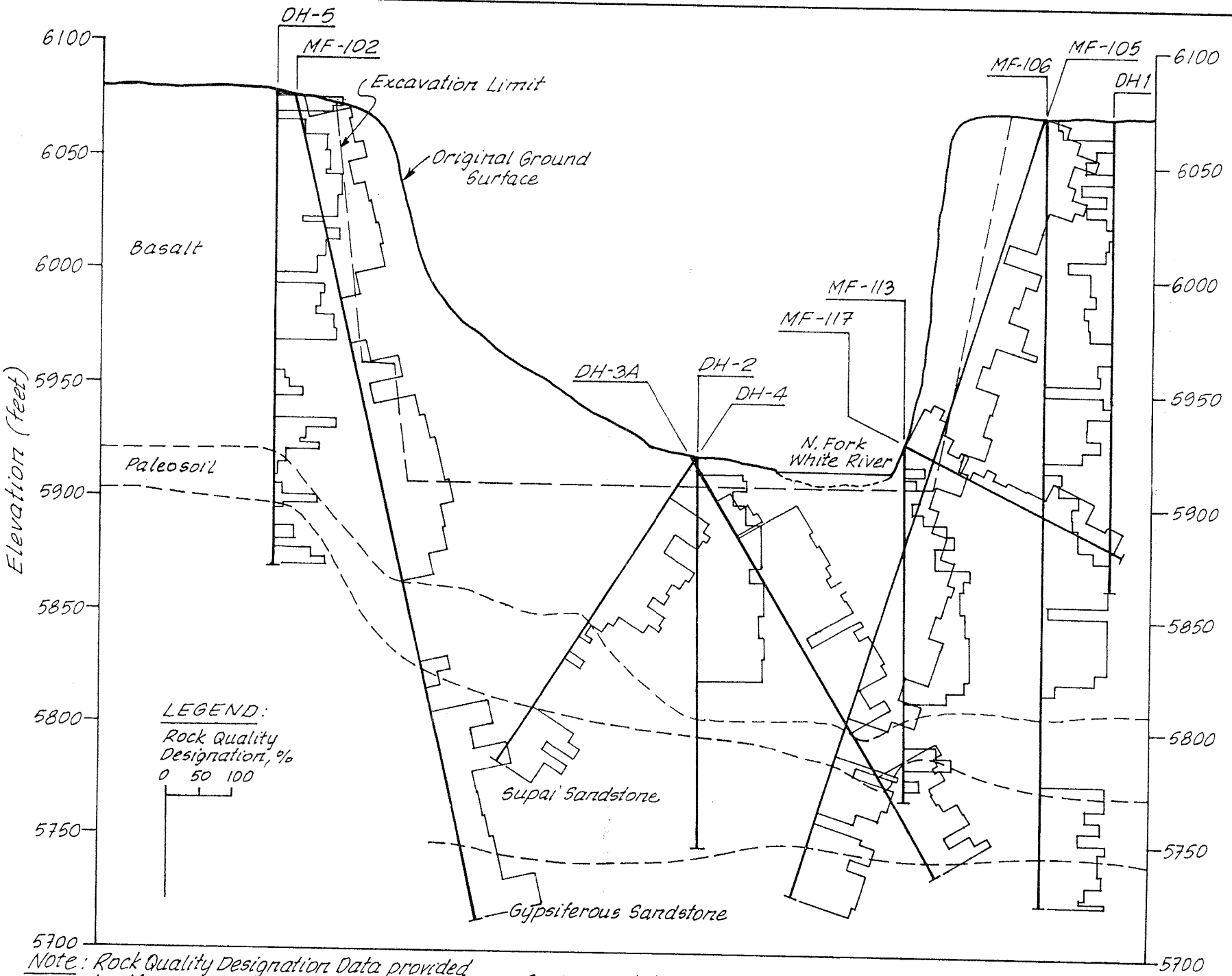

Note: Rock Quality Designation Data provided by Mineral Systems Inc.

Scale: 1 inch to 60 feet.

GEOLOGIC SECTION WITH HISTOGRAMS OF ROCK QUALITY DESIGNATION (RQD)

Figure 5b

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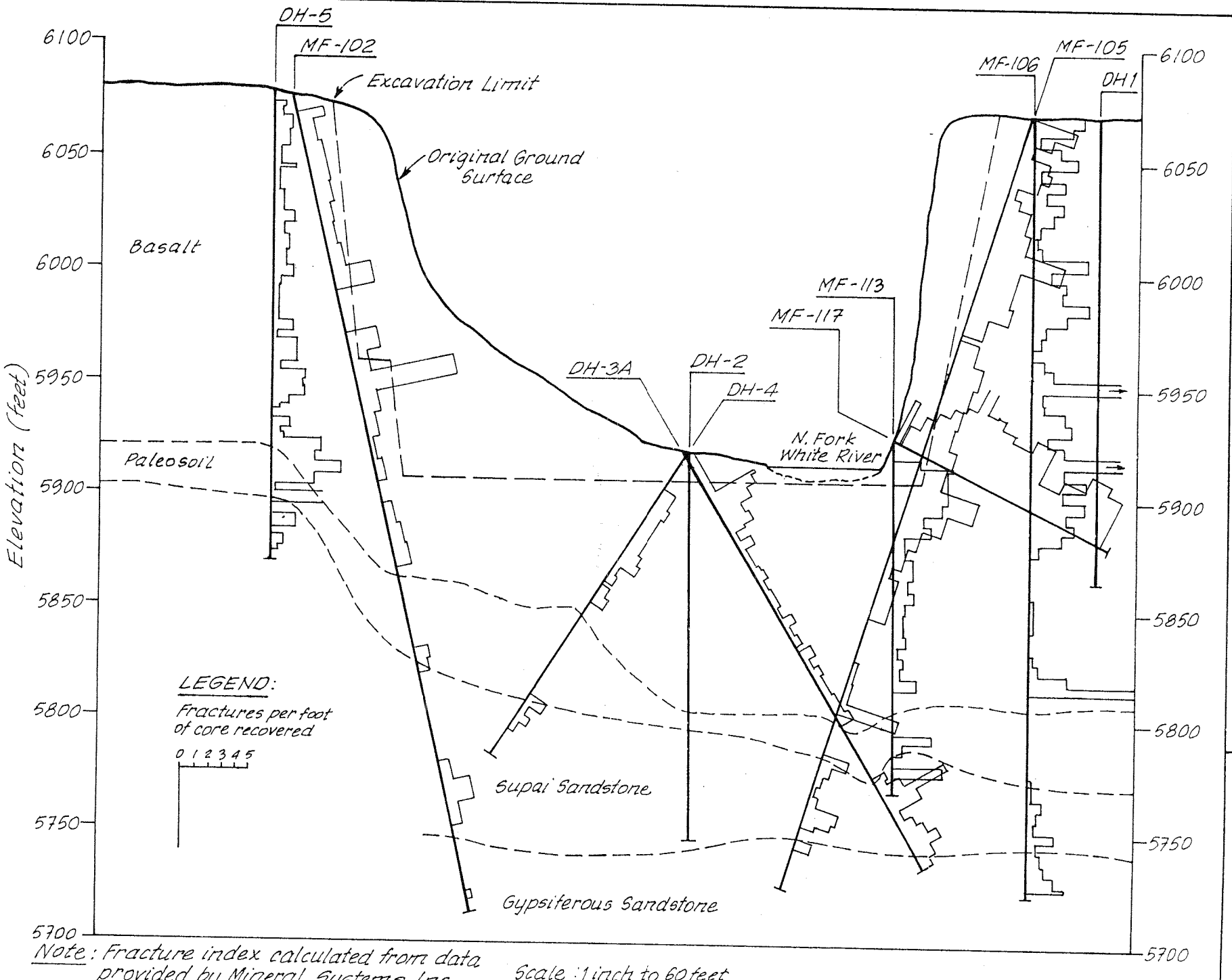

Note: Fracture index calculated from data provided by Mineral Systems Inc.

Scale: 1 inch to 60 feet.

GEOLOGIC SECTION WITH HISTOGRAMS OF FRACTURE INDEX

Figure 5c

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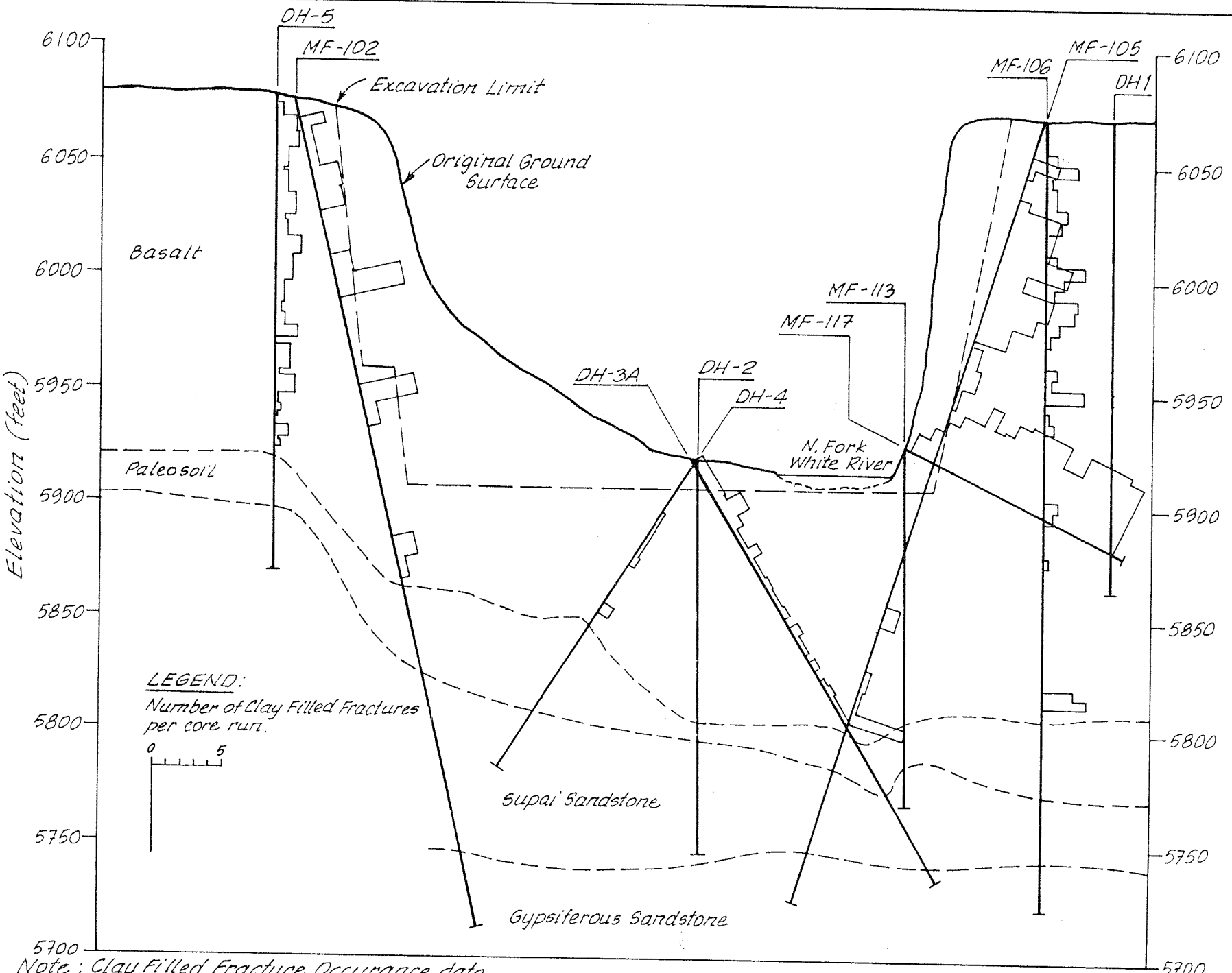

Note: Clay Filled Fracture Occurance data provided by Mineral Systems Inc.

Scale: 1 inch to 60 feet.

GEOLOGIC SECTION WITH HISTOGRAMS OF CLAY FILLED FRACTURE OCCURANCE

Figure 5d

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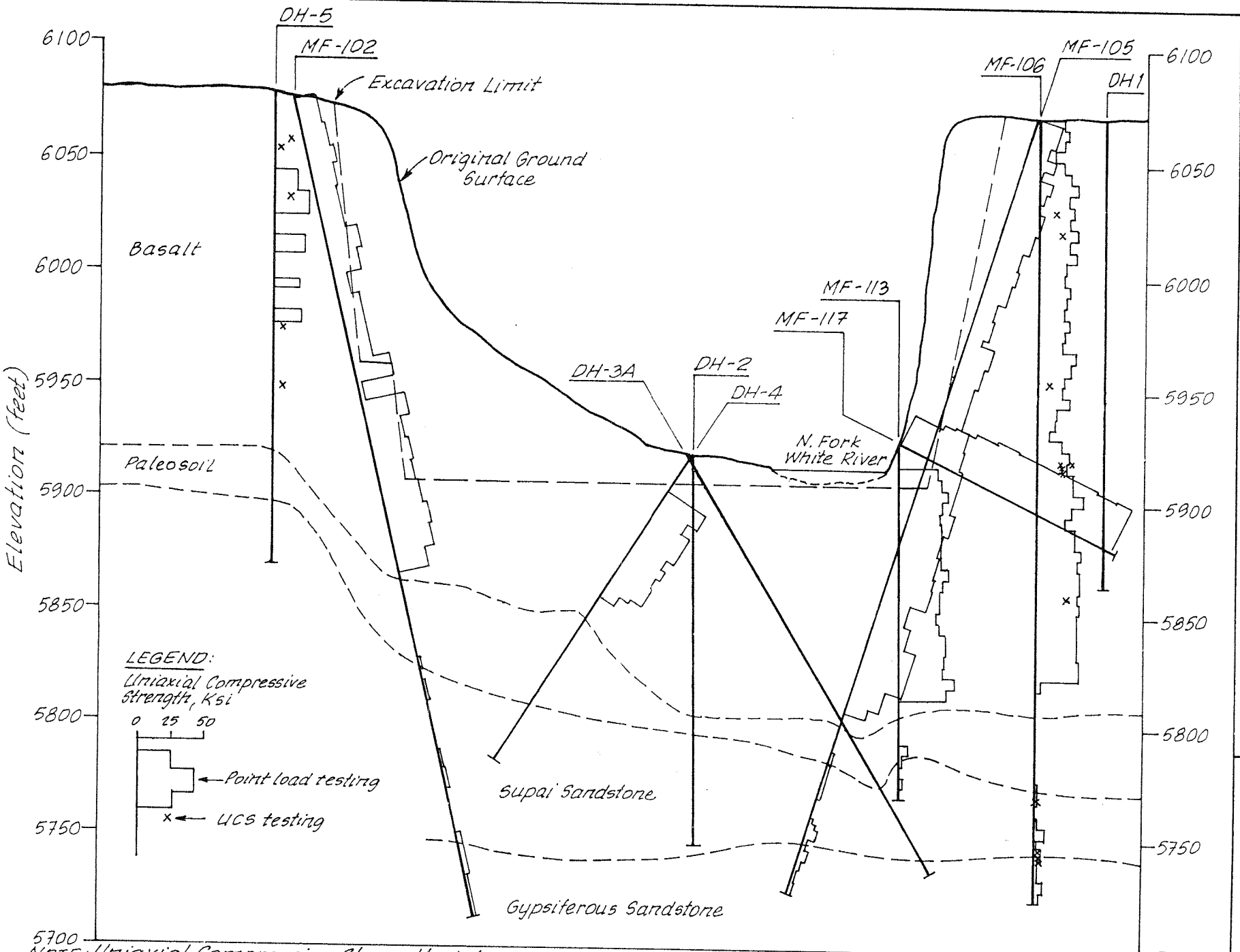

NOTE: Uniaxial Compressive Strength data from Point Load tests provided by Mineral Systems Inc.

Scale: 1 inch to 60 feet.

Uniaxial Compressive Strength data from UCS tests provided by Morrison - Maierle Inc.

GEOLOGIC SECTION WITH HISTOGRAMS OF UNIAXIAL COMPRESSIVE STRENGTH

Figure 5e

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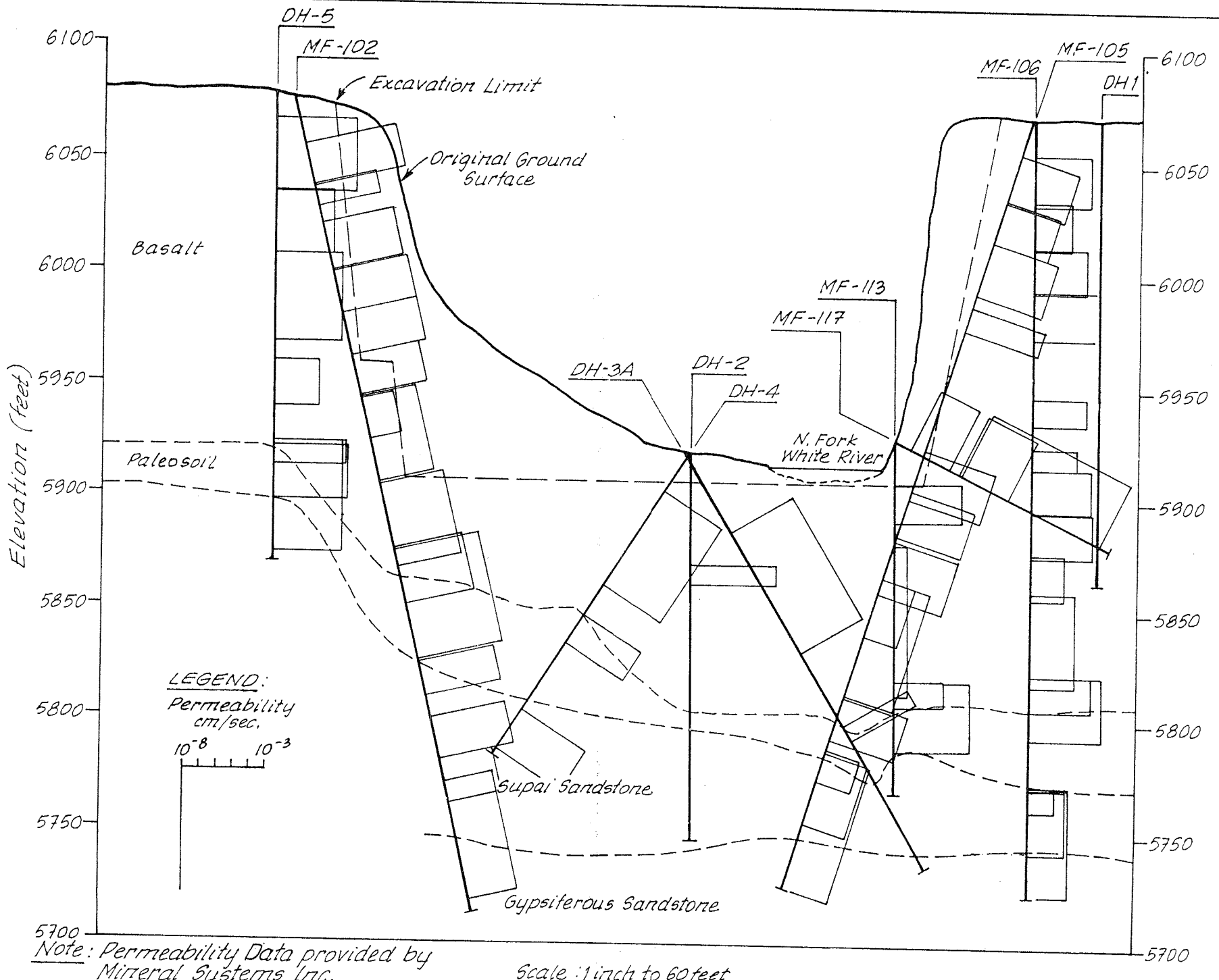

Note: Permeability Data provided by Mineral Systems Inc.

Scale: 1 inch to 60 feet.

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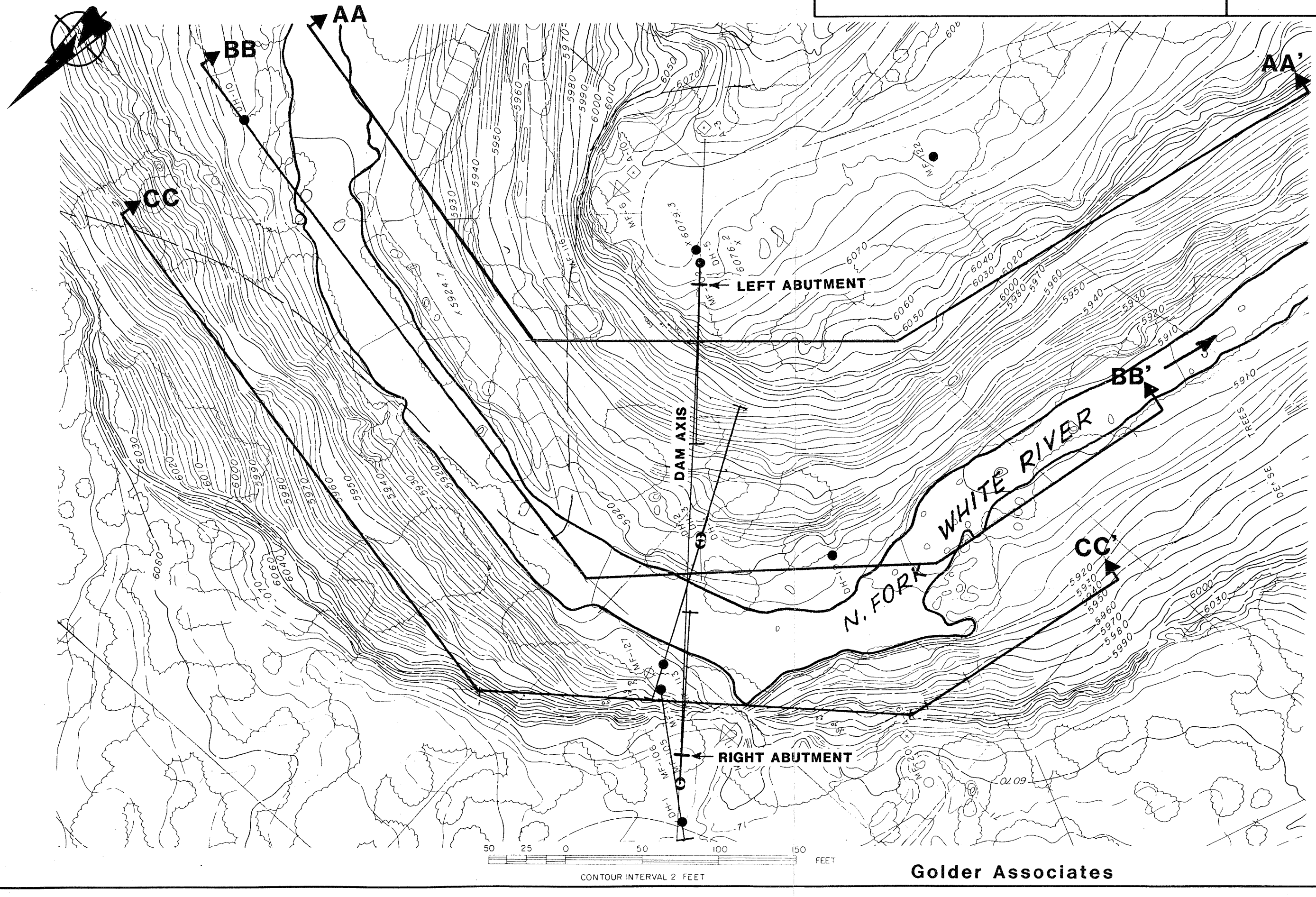
GEOLOGIC SECTION WITH HISTOGRAMS OF PERMEABILITY

Figure 5f

PLAN OF TYPICAL FOUNDATION PROFILES | Figure 6



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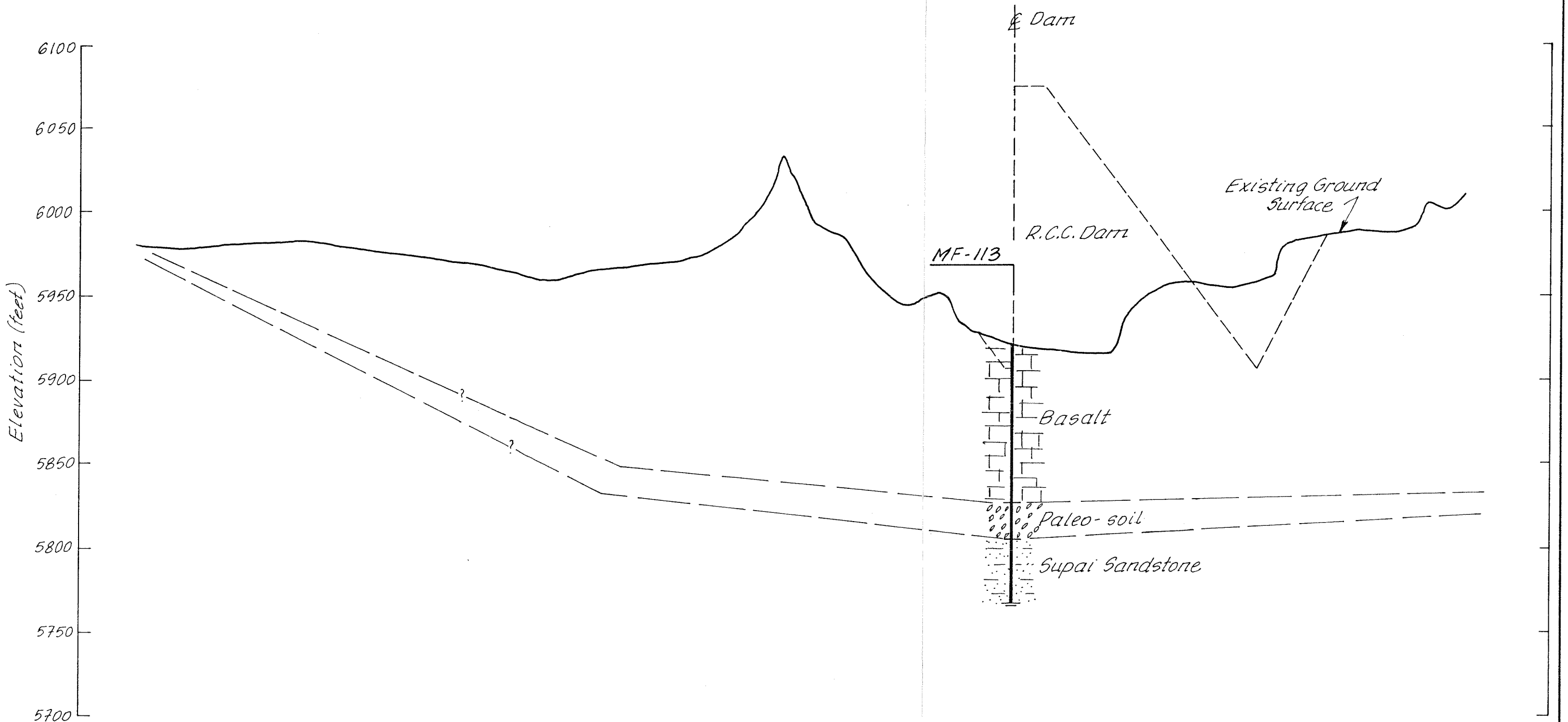
TYPICAL FOUNDATION PROFILE CC - CC' | **Figure 7c**



Note : Typical foundation profile prepared by Mineral Systems Incorporated.

Scale : 1 inch to 60 feet

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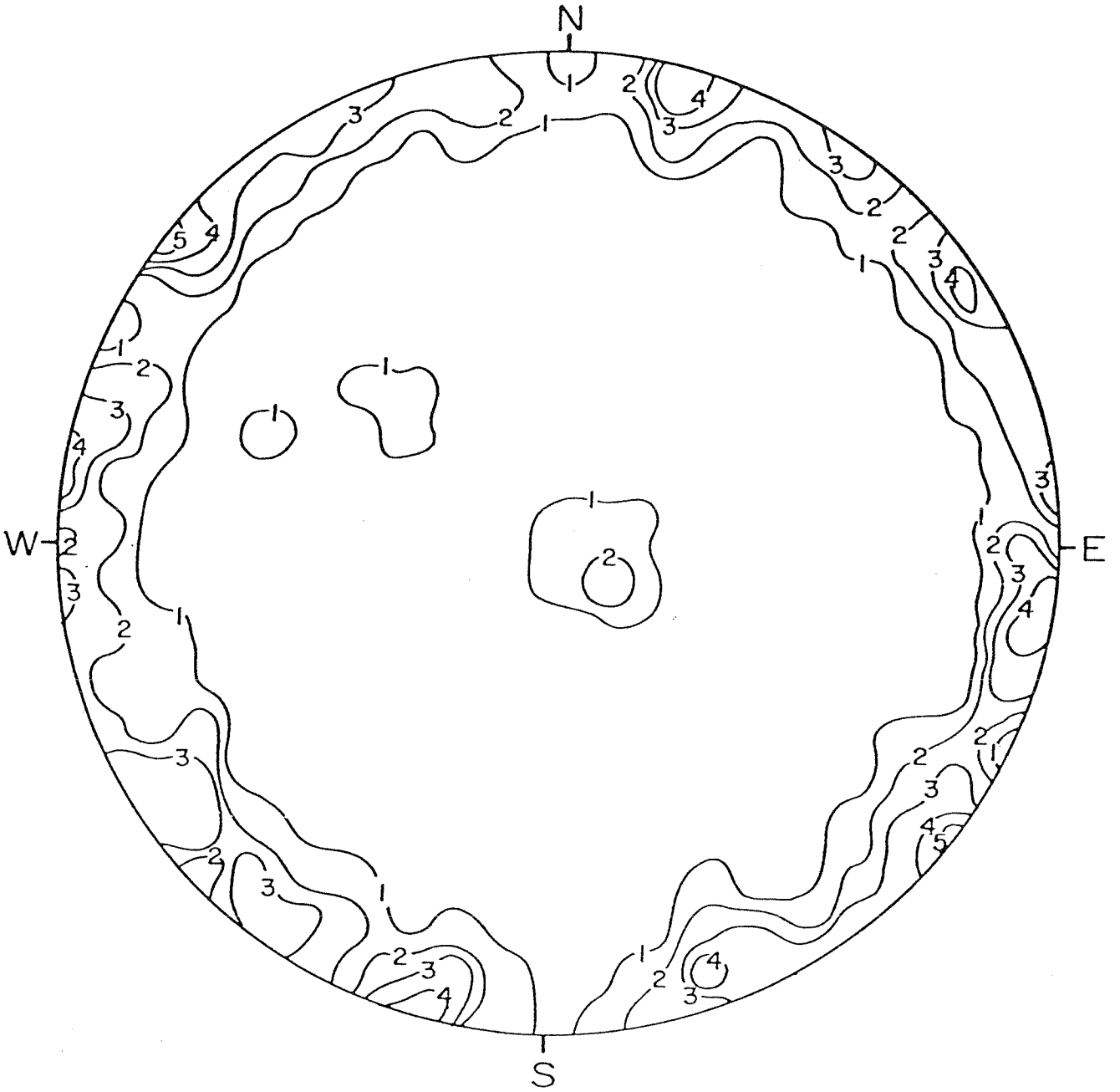
EQUAL AREA (LOWER HEMISPHERE)
PLOT OF DISCONTINUITIES (JOINTS) IN BASALT

Figure **8**



222 poles, Contour Interval 1 percent

Note: Discontinuity data obtained from detailed line surveys in the vicinity of the dam site by Mineral Systems Inc. (1983).

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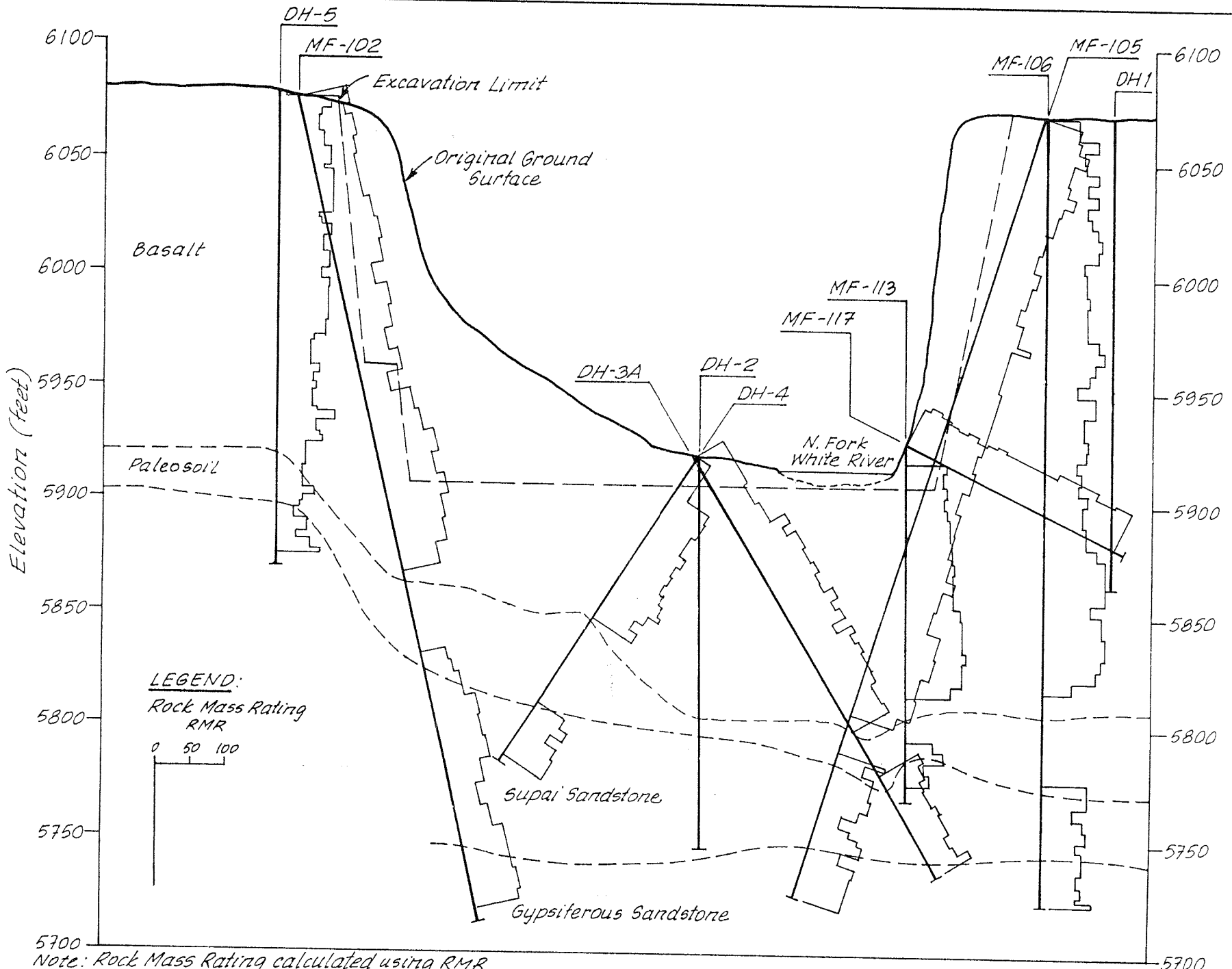

Note: Rock Mass Rating calculated using RMR relationship (Bieniawski, 1974, 1976).

Scale: 1 inch to 60 feet.

GEOLOGIC SECTION WITH HISTOGRAMS OF ROCK MASS RATING (RMR)

Figure 9

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APPROXIMATE RELATIONSHIP BETWEEN ROCK MASS QUALITY AND MATERIAL CONSTANTS

Disturbed rock mass m & s values — *undisturbed rock mass m & s values*

Emprircal failure criterion

$\sigma_1 = \sigma_3 + (m\sigma_c\sigma_3 + s\sigma_c^2)^{1/2}$

σ_1 = major principal stress
σ_3 = minor principal stress
σ_c = uniaxial compressive strength of intact rock, and
m, s are empirical constants

	CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS AND METAMORPHIC CRYSTALLINE ROCKS *amphibolite, gabbro, gneiss, granite, norite and quartz–diorite*
INTACT ROCK SAMPLES *Laboratory size specimens free from discontinuities.* CSIR rating RMR = 100 NGI rating Q = 500	m = 7.00 s = 1.00 *m = 7.00* *s = 1.00*	m = 10.00 s = 1.00 *m = 10.00* *s = 1.00*	m = 15.00 s = 1.00 *m = 15.00* *s = 1.00*	m = 17.00 s = 1.00 *m = 17.00* *s = 1.00*	m = 25.00 s = 1.00 *m = 25.00* *s = 1.00*
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints at 1 to 3m.* CSIR rating RMR = 85 NGI rating Q = 100	m = 2.40 s = 0.082 *m = 4.10* *s = 0.189*	m = 3.43 s = 0.082 *m = 5.85* *s = 0.189*	m = 5.14 s = 0.082 *m = 8.78* *s = 0.189*	m = 5.82 s = 0.082 *m = 9.95* *s = 0.189*	m = 8.56 s = 0.082 *m = 14.63* *s = 0.189*
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3 m.* CSIR rating RMR = 65 NGI rating Q = 10	m = 0.575 s = 0.00293 *m = 2.006* *s = 0.0205*	m = 0.821 s = 0.00293 *m = 2.865* *s = 0.0205*	m = 1.231 s = 0.00293 *m = 4.298* *s = 0.0205*	m = 1.395 s = 0.00293 *m = 4.871* *s = 0.0205*	m = 2.052 s = 0.00293 *m = 7.163* *s = 0.0205*
FAIR QUALITY ROCK MASS *Several sets of moderately weathered joints spaced at 0.3 to 1 m.* CSIR rating RMR = 44 NGI rating Q = 1	m = 0.128 s = 0.00009 *m = 0.947* *s = 0.00198*	m = 0.183 s = 0.00009 *m = 1.353* *s = 0.00198*	m = 0.275 s = 0.00009 *m = 2.030* *s = 0.00198*	m = 0.311 s = 0.00009 *m = 2.301* *s = 0.00198*	m = 0.458 s = 0.00009 *m = 3.383* *s = 0.00198*
POOR QUALITY ROCK MASS *Numerous weathered joints at 30 to 500mm with some gouge. Clean compacted waste rock.* CSIR rating RMR = 23 NGI rating Q = 0.1	m = 0.029 s=0.000003 *m = 0.447* *s = 0.00019*	m = 0.041 s=0.000003 *m = 0.639* *s = 0.00019*	m = 0.061 s=0.000003 *m = 0.959* *s = 0.00019*	m = 0.069 s=0.000003 *m = 1.087* *s = 0.00019*	m = 0.102 s=0.000003 *m = 1.598* *s = 0.00019*
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced at less than 50mm with gouge. Waste rock with fines.* CSIR rating RMR = 3 NGI rating Q = 0.01	m = 0.007 s=0.0000001 *m = 0.219* *s = 0.00002*	m = 0.010 s=0.0000001 *m = 0.313* *s = 0.00002*	m = 0.015 s=0.0000001 *m = 0.469* *s = 0.00002*	m = 0.017 s=0.0000001 *m = 0.532* *s = 0.00002*	m = 0.024 s=0.0000001 *m = 0.782* *s = 0.00002*

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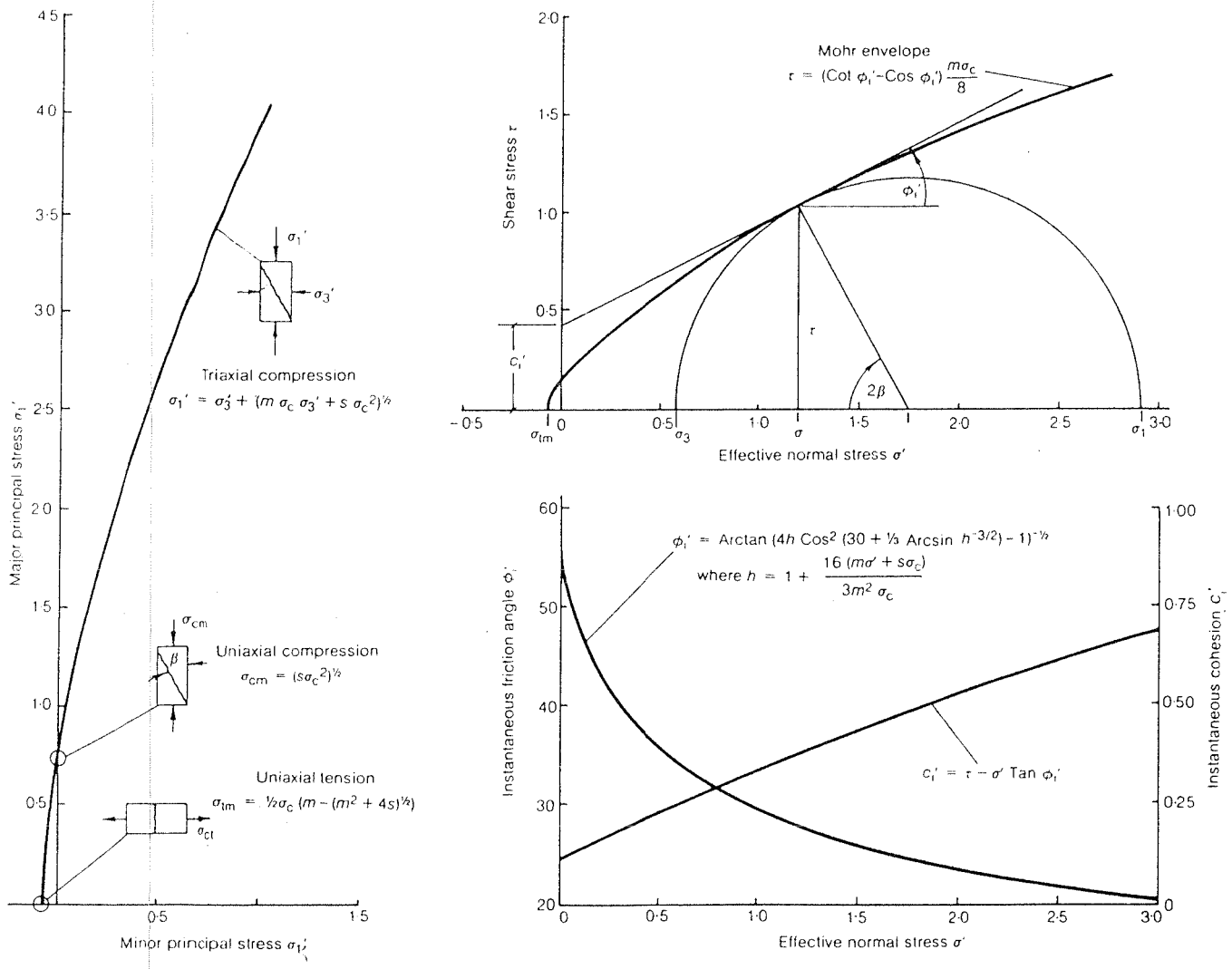
EMPIRICAL ROCK MASS FAILURE CRITERION

Figure 10



$m/m_i = \exp[(RMR - 100)/14]$

$s = \exp[(RMR - 100)/6]$

Note: Method of determining rock mass strength parameters using m and s values, proposed by Hoek and Brown (1980).

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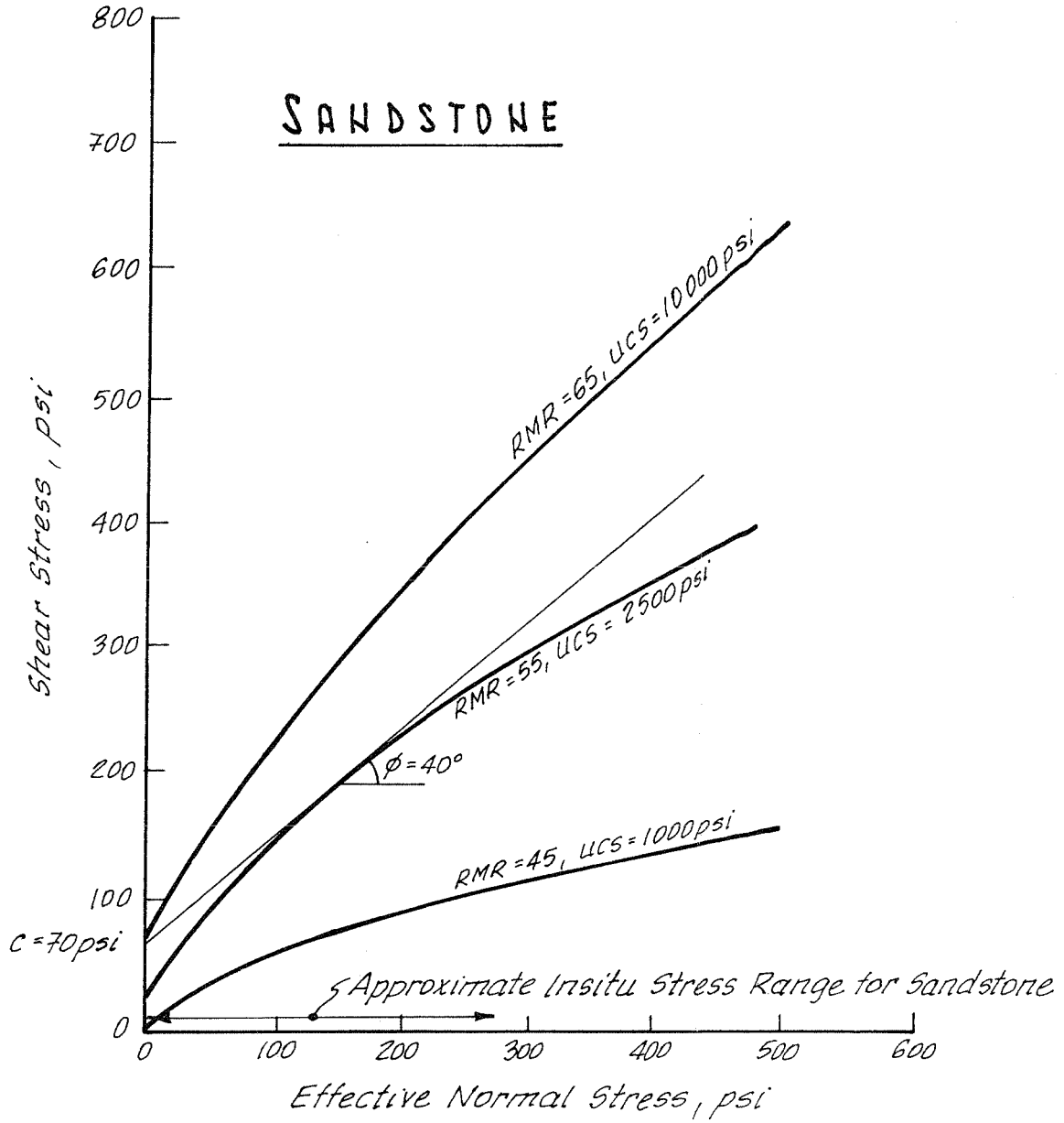
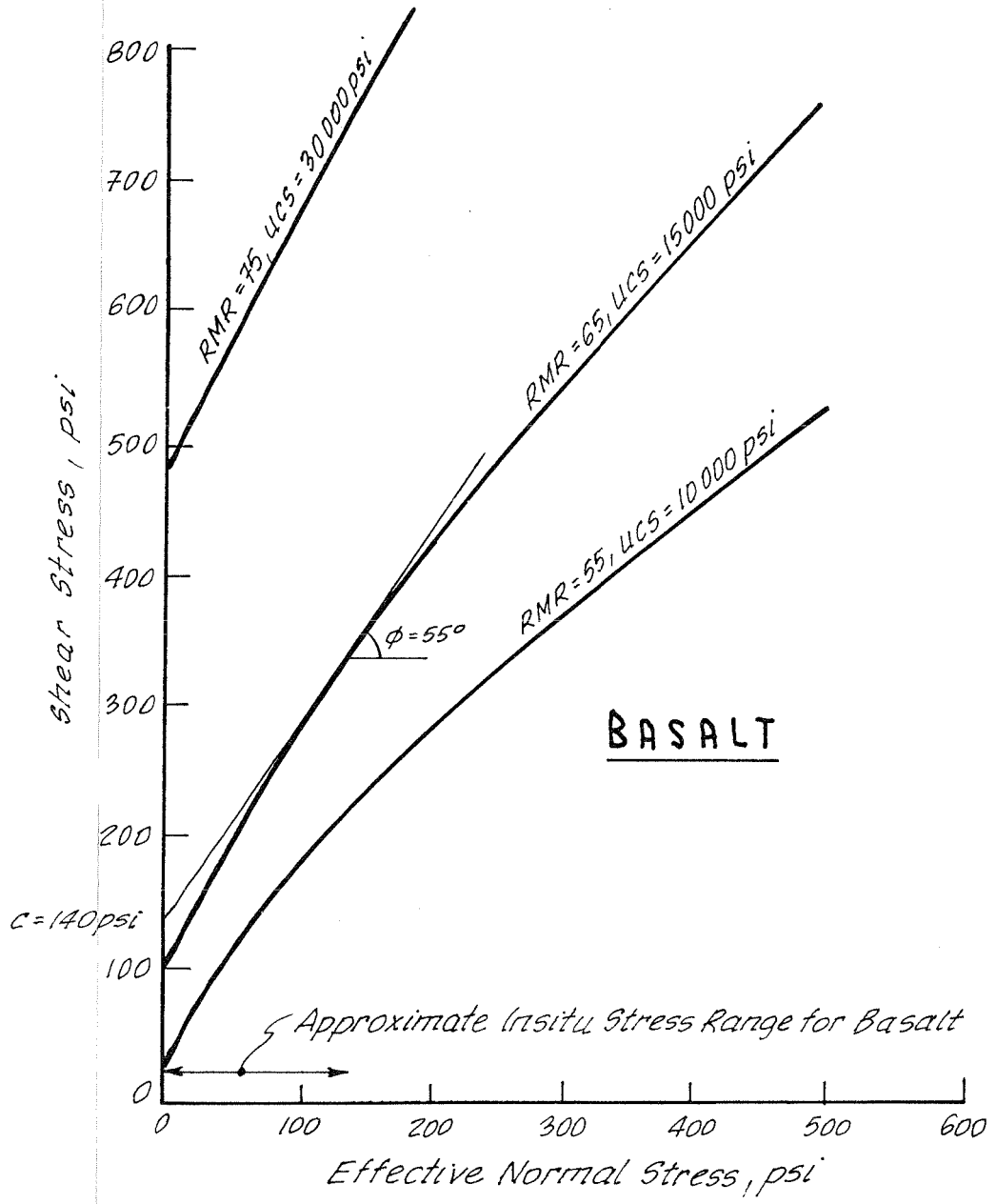
ROCK MASS STRENGTH PARAMETERS	Figure 11





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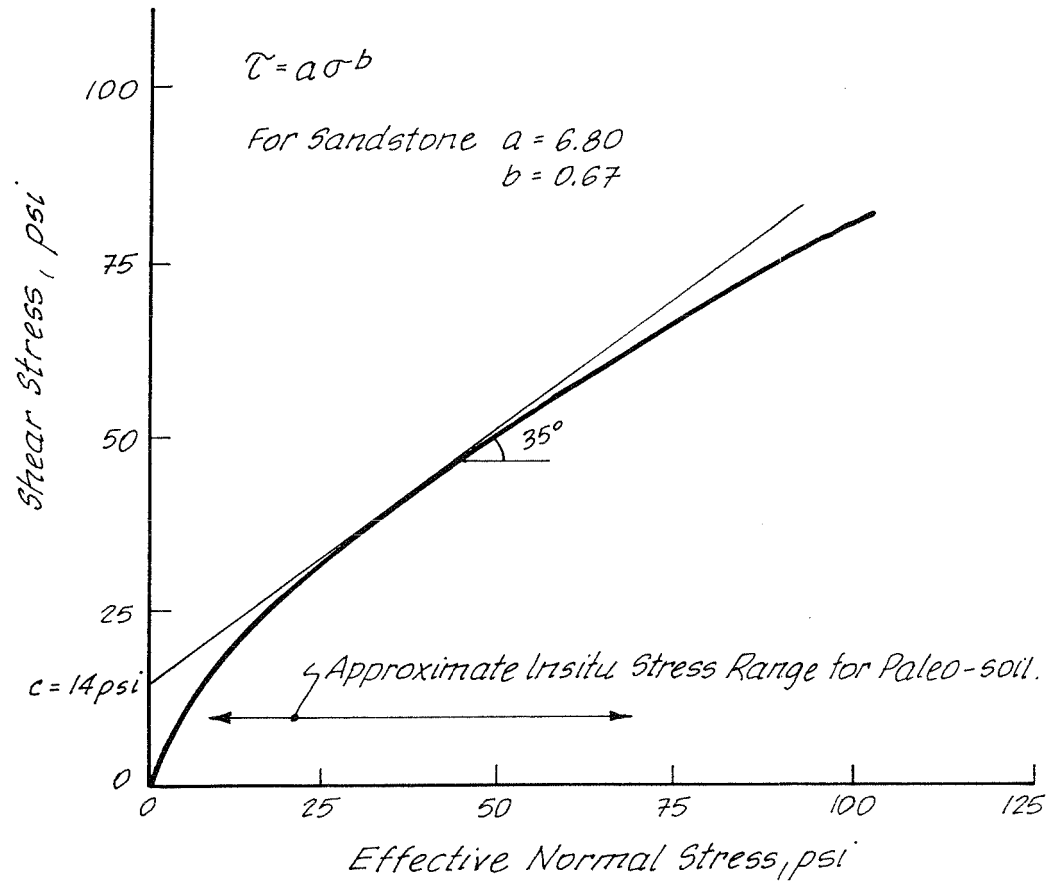
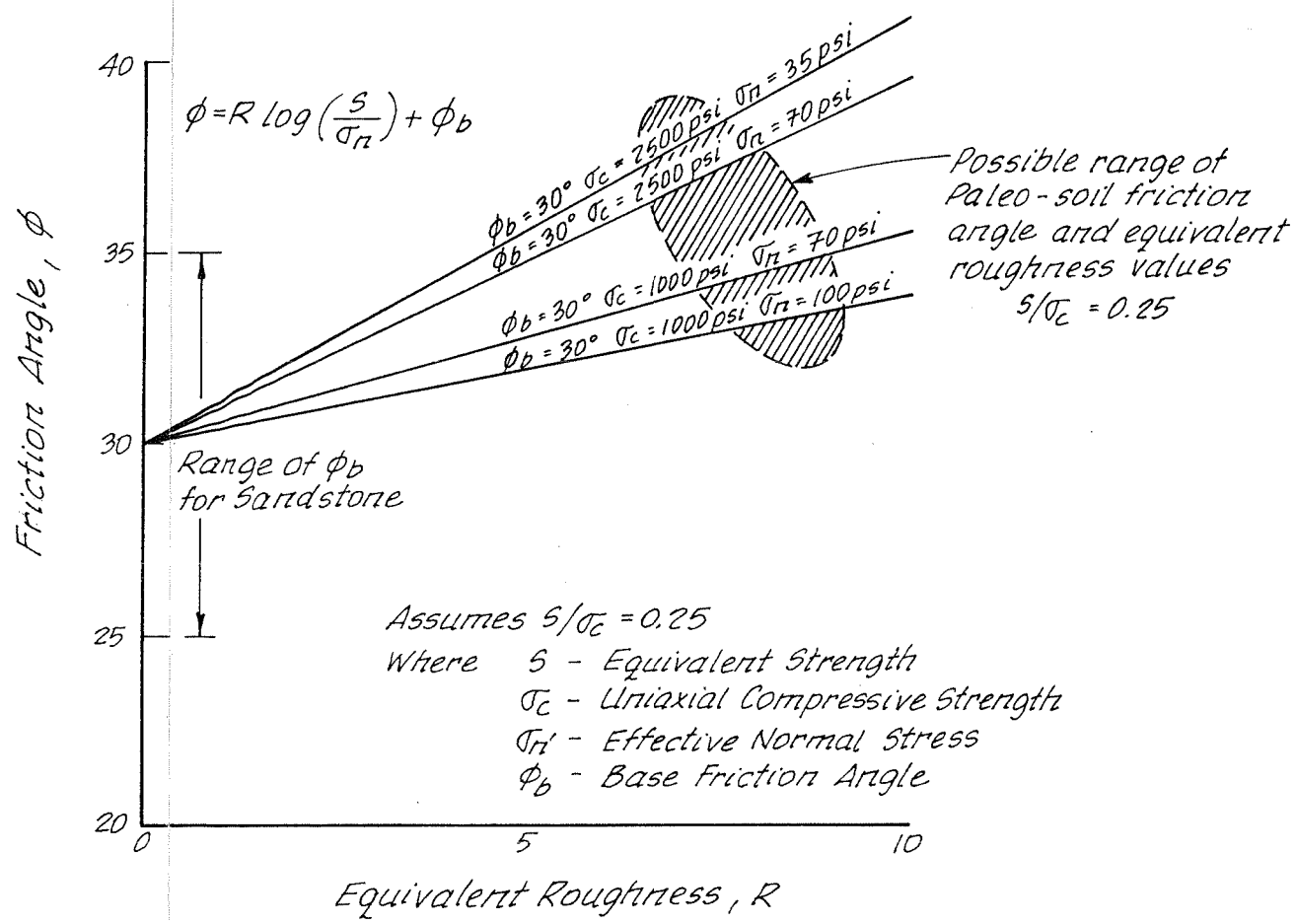
PALEO-SOIL STRENGTH PARAMETERS

Figure 12



a) Shear strength of compacted Sandstone Rockfill after Charles and Watt (1980).



b) Shear strength of Rockfill after Barton and Kjaernsli (1981).

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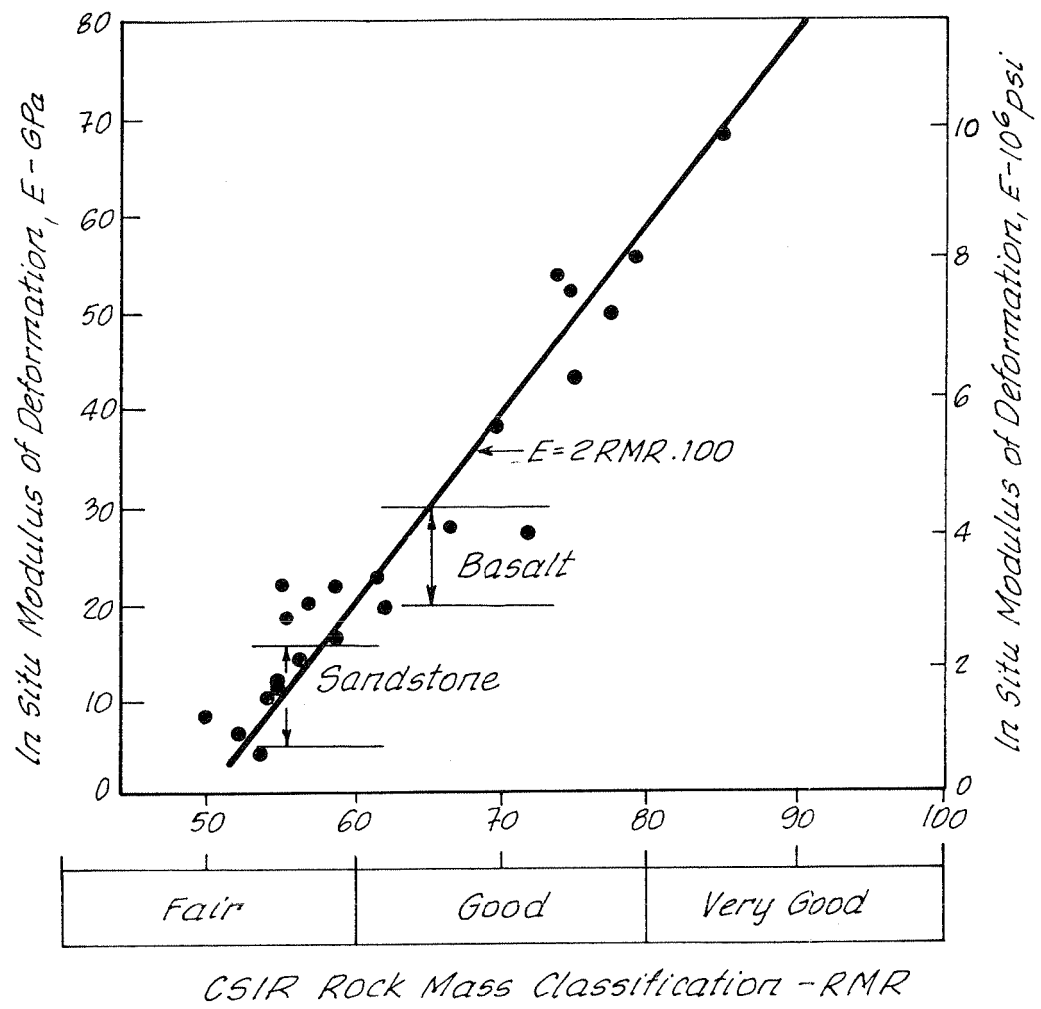
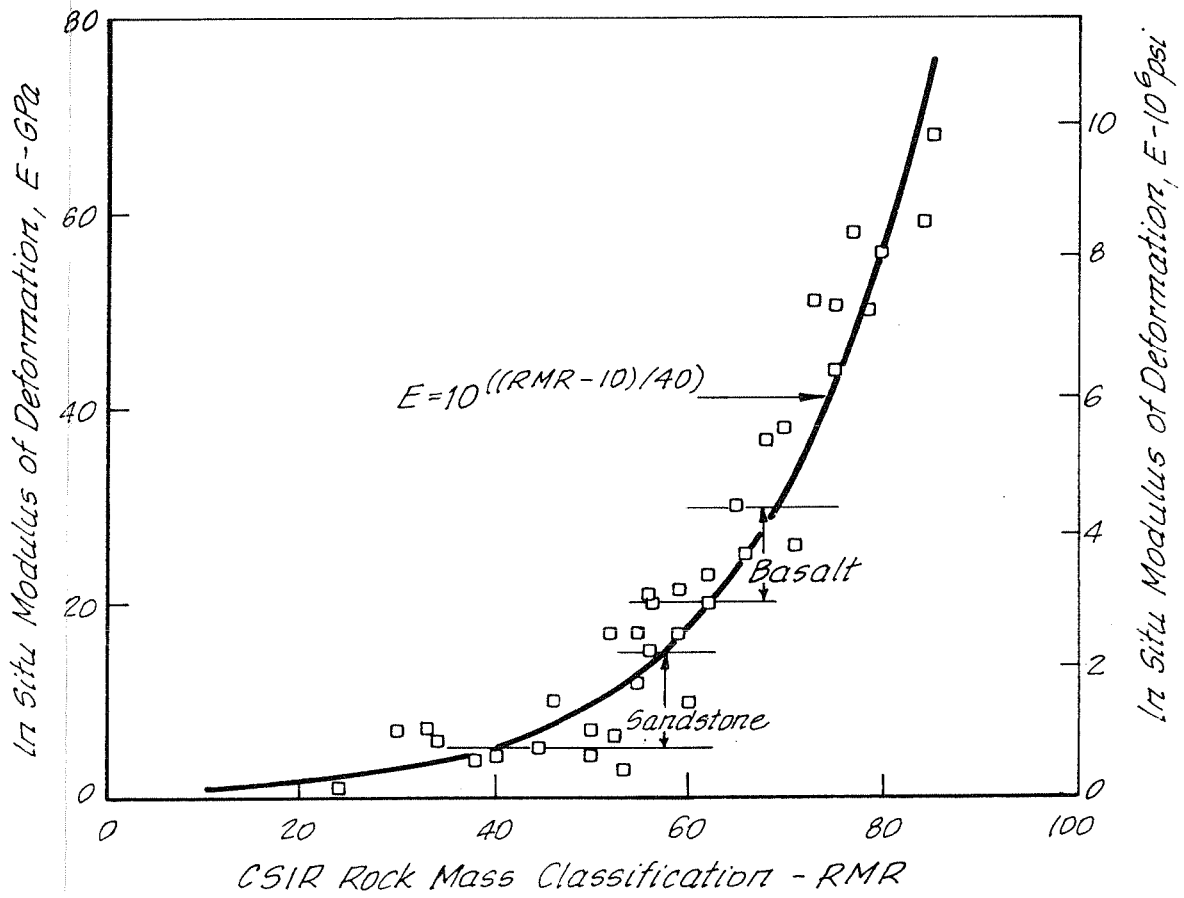


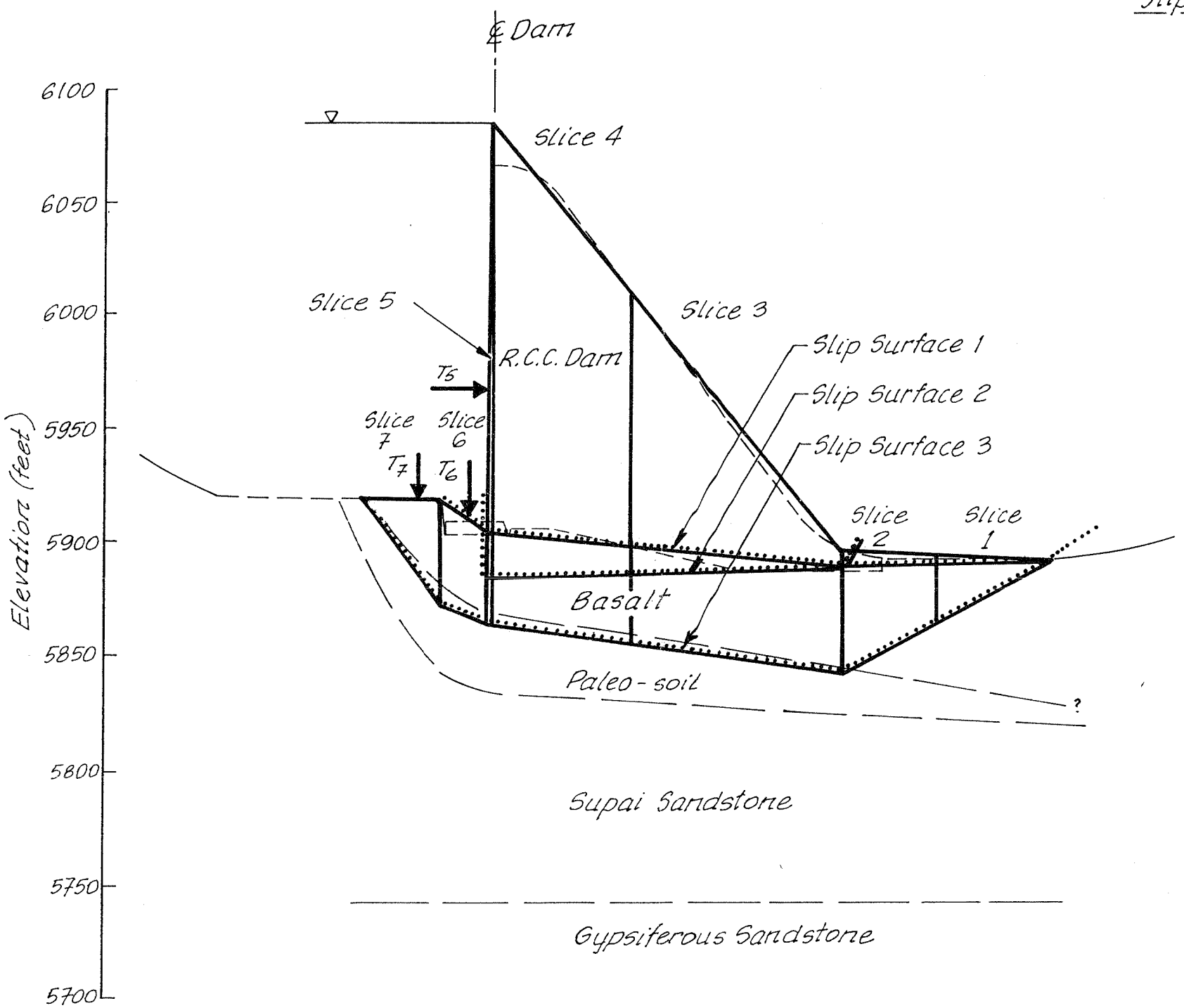
a) Relationship between rock mass quality and in situ modulus of deformation after Bieniawski (1978).



b) Relationship between rock mass quality and in situ modulus of deformation after Serafim and Pereira (1983).

SARMA ANALYSIS OF DAM FOUNDATION STABILITY

Figure 14

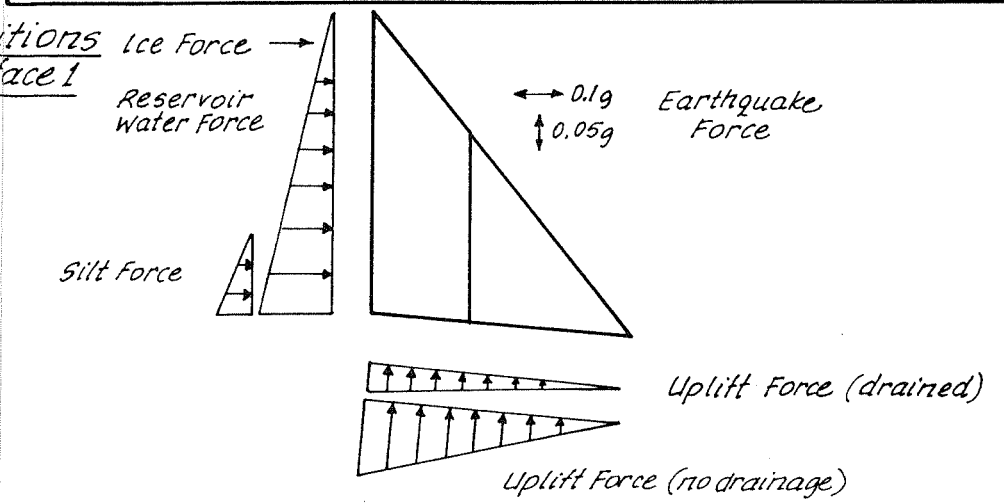


Scale: 1 inch to 60 feet

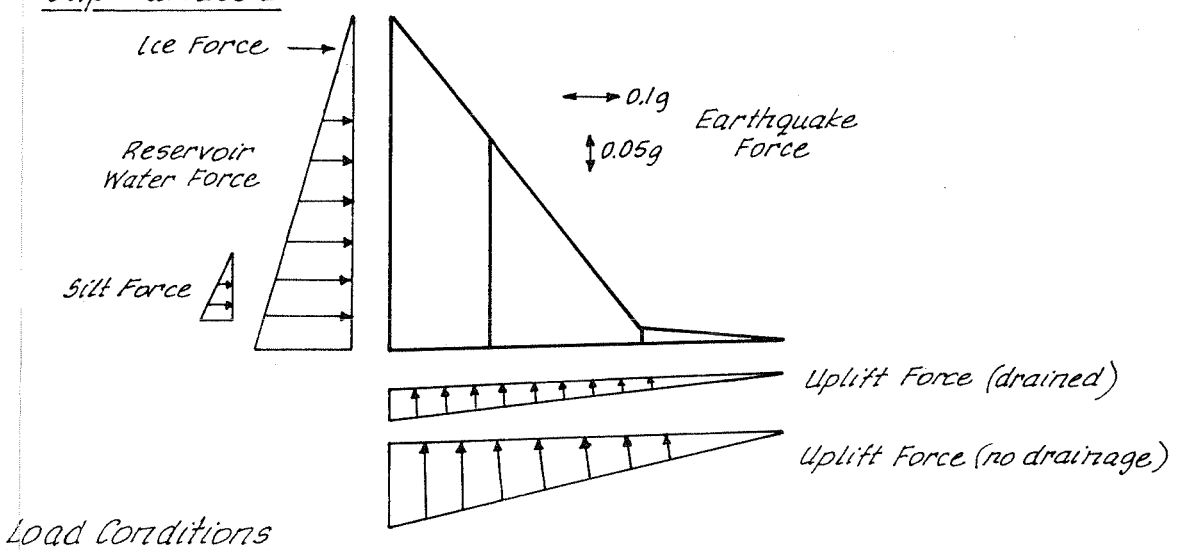
Notes:

T_5 - Horizontal silt and ice load
T_6 - Vertical water and silt load
T_7 - Vertical water and silt load

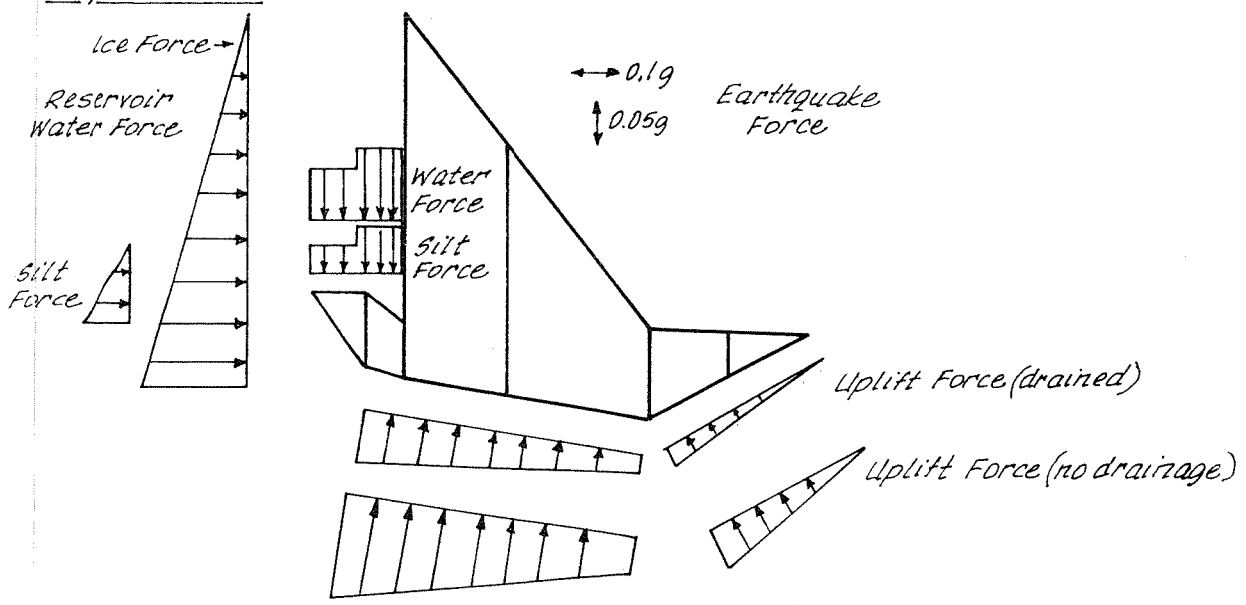
Load Conditions
Slip Surface 1



Load Conditions
Slip Surface 2



Load Conditions
Slip Surface 3



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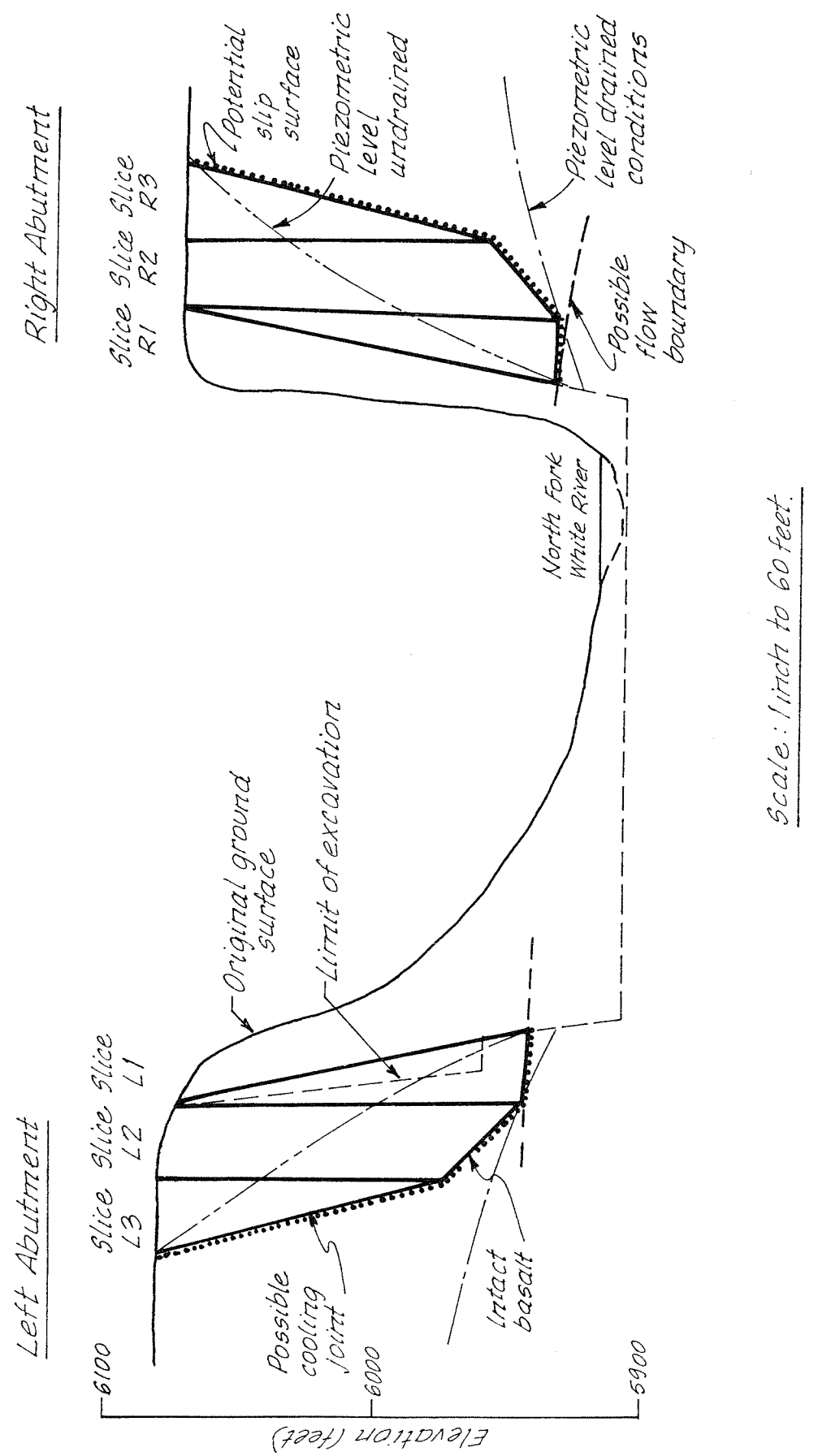
Note: Force Diagrams Not to Scale.

SARMA ANALYSIS OF EXCAVATED SLOPE STABILITY

Figure **15**



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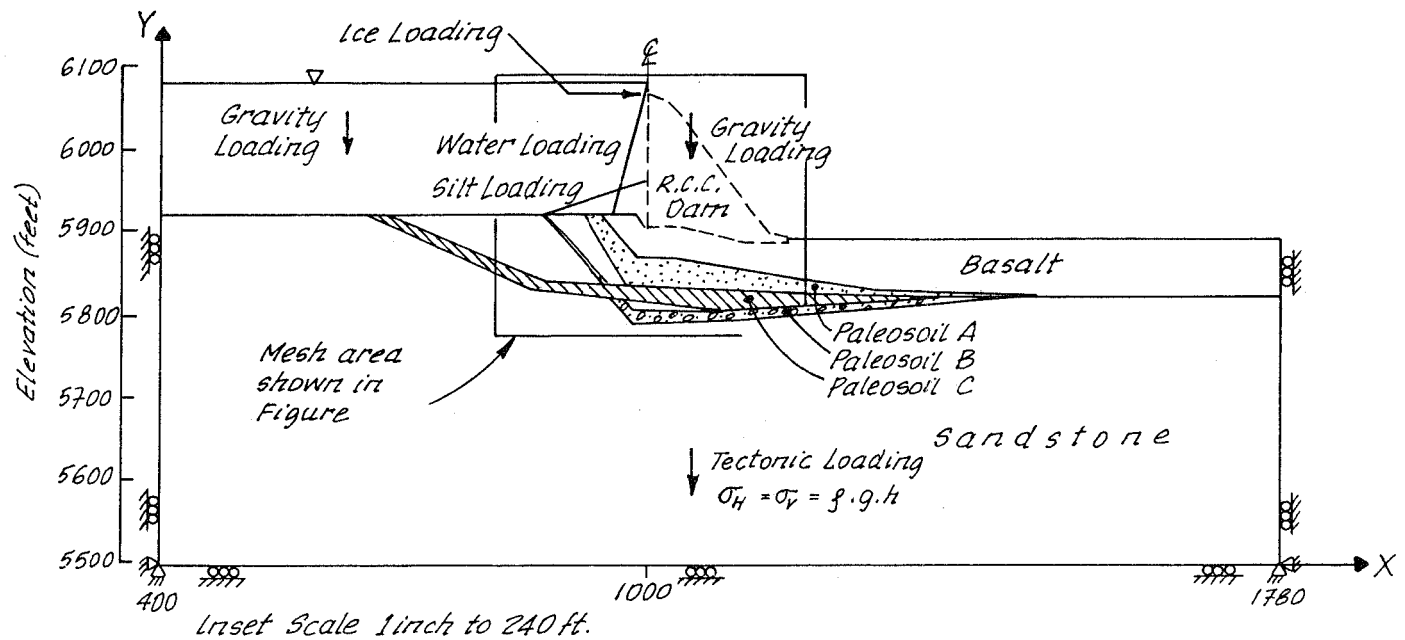
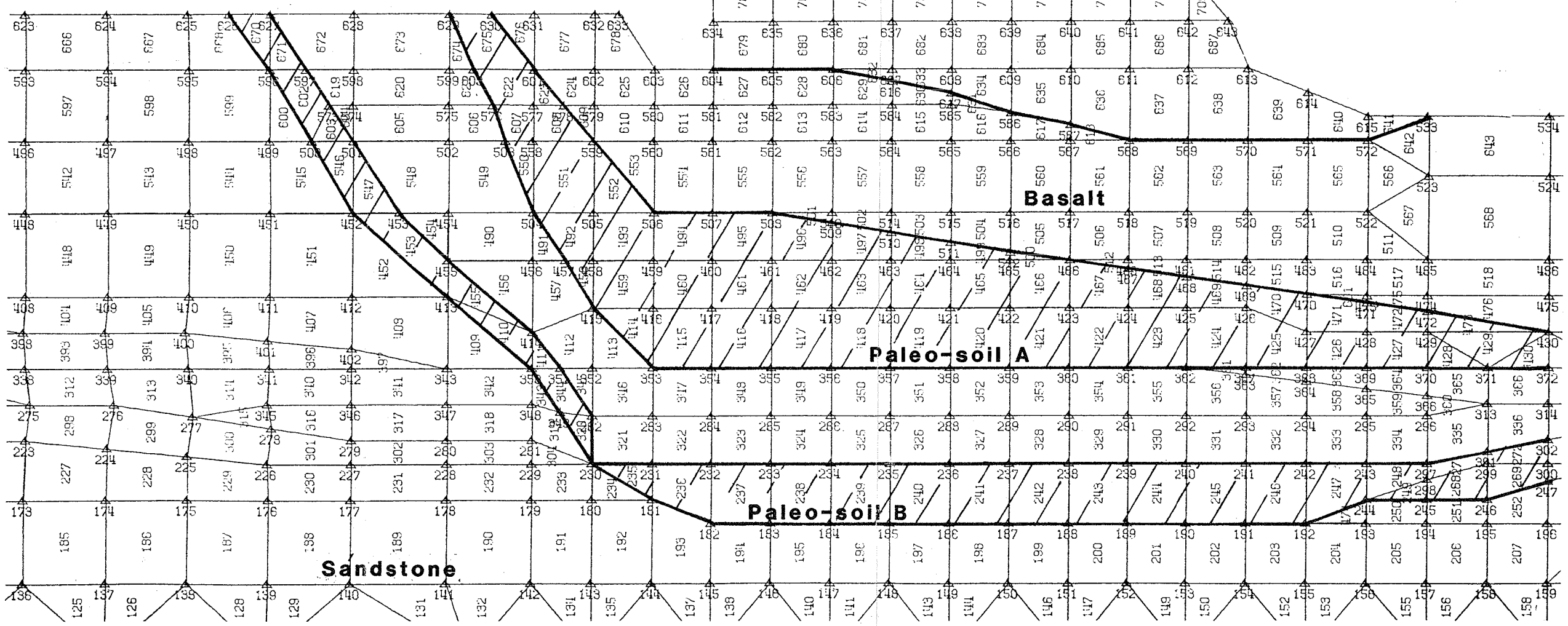
LINEAR FINITE ELEMENT MESH AND BOUNDARY LOAD CONDITIONS

Figure 16



Inset Scale 1 inch to 240 ft.

Note: Location of paleosoil layers A, B and C correspond to those shown in the typical foundation profiles, Figures 7a, 7b and 7c respectively.



Scale: 1 inch to 30 feet

REFERENCE NODES

	Upstream	Downstream
Spillway Crest	686	
Base of Dam	604	571
Top of Paleo-soil A	507	470
Base of Paleo-soil A	354	384
Top of Paleo-soil B	232	242
Base of Paleo-soil B	182	192

REFERENCE ELEMENTS

	Upstream	Downstream
Spillway Crest	743	
Base of Dam	679	639
Top of Basalt	627	564
Paleo-soil A	495	470
Paleo-soil B	237	246

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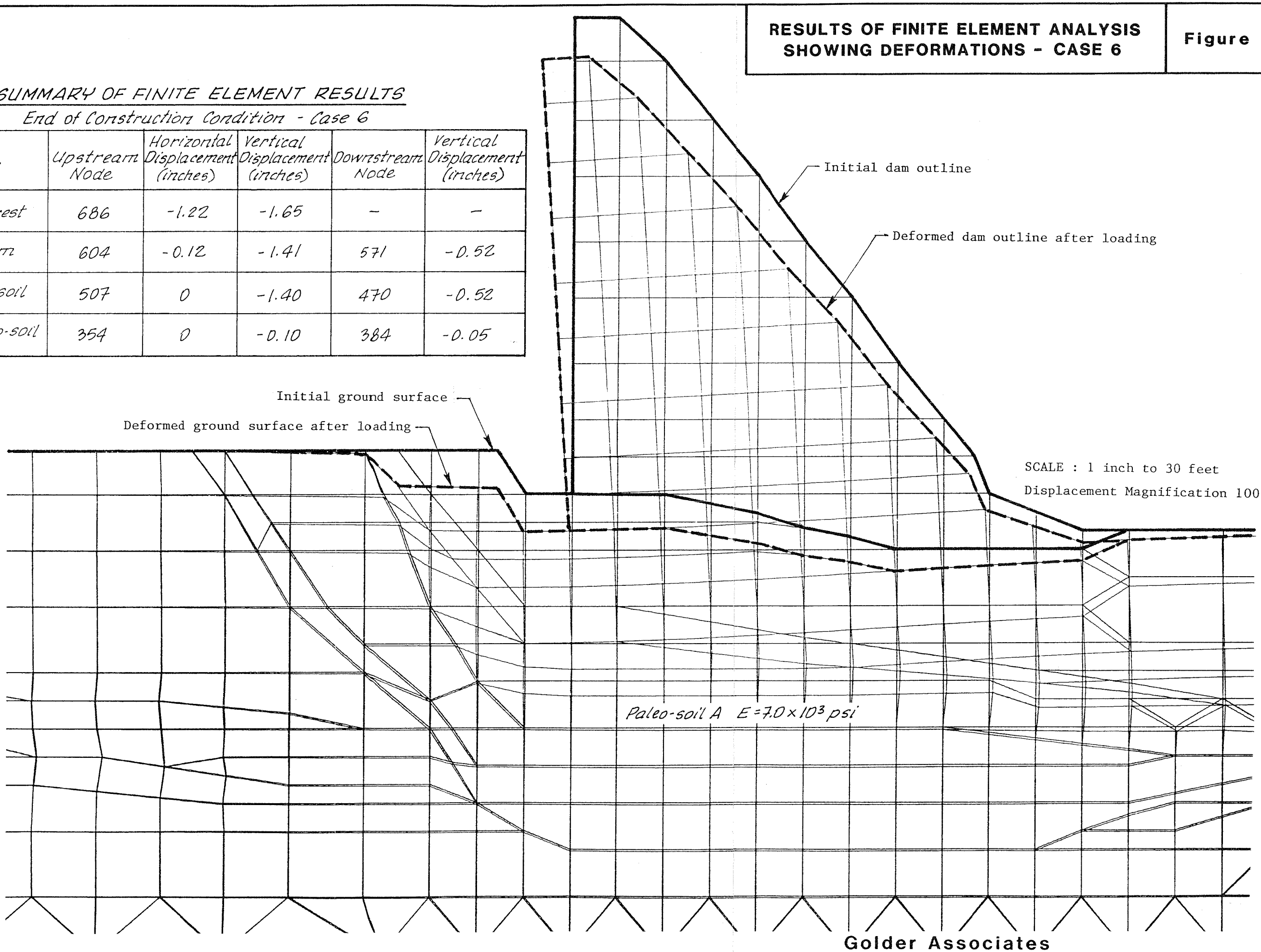
RESULTS OF FINITE ELEMENT ANALYSIS SHOWING DEFORMATIONS - CASE 6

Figure 17f

SUMMARY OF FINITE ELEMENT RESULTS

End of Construction Condition - Case 6

Location	Upstream Node	Horizontal Displacement (inches)	Vertical Displacement (inches)	Downstream Node	Vertical Displacement (inches)
Spillway Crest	686	-1.22	-1.65	-	-
Base of Dam	604	-0.12	-1.41	571	-0.52
Top of Paleo-soil	507	0	-1.40	470	-0.52
Base of Paleo-soil	354	0	-0.10	384	-0.05



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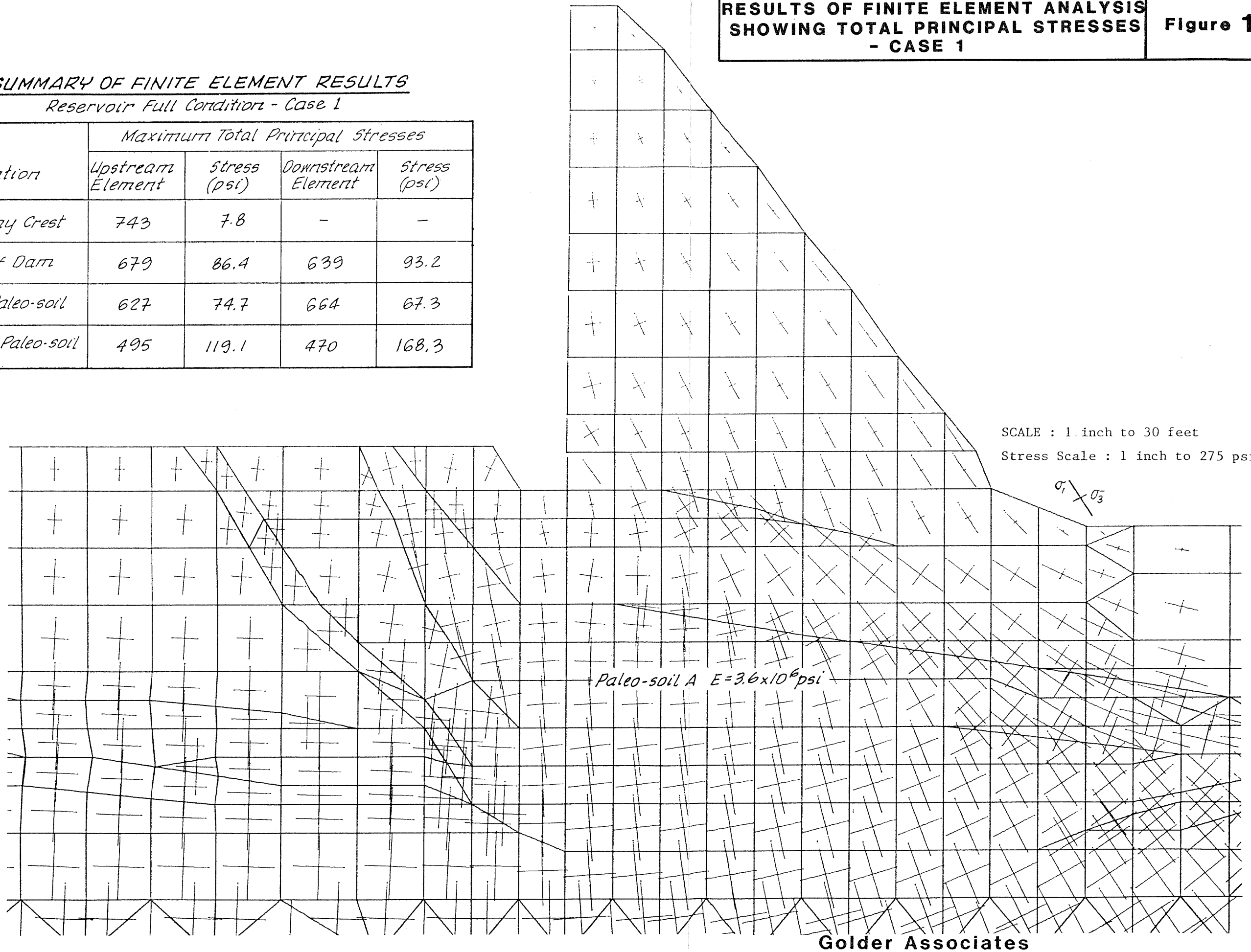
SUMMARY OF FINITE ELEMENT RESULTS

Reservoir Full Condition - Case 1

Location	Maximum Total Principal Stresses			
	Upstream Element	Stress (psi)	Downstream Element	Stress (psi)
Spillway Crest	743	7.8	-	-
Base of Dam	679	86.4	639	93.2
Top of Paleo-soil	627	74.7	664	67.3
Base of Paleo-soil	495	119.1	470	168.3

RESULTS OF FINITE ELEMENT ANALYSIS SHOWING TOTAL PRINCIPAL STRESSES - CASE 1

Figure 18a



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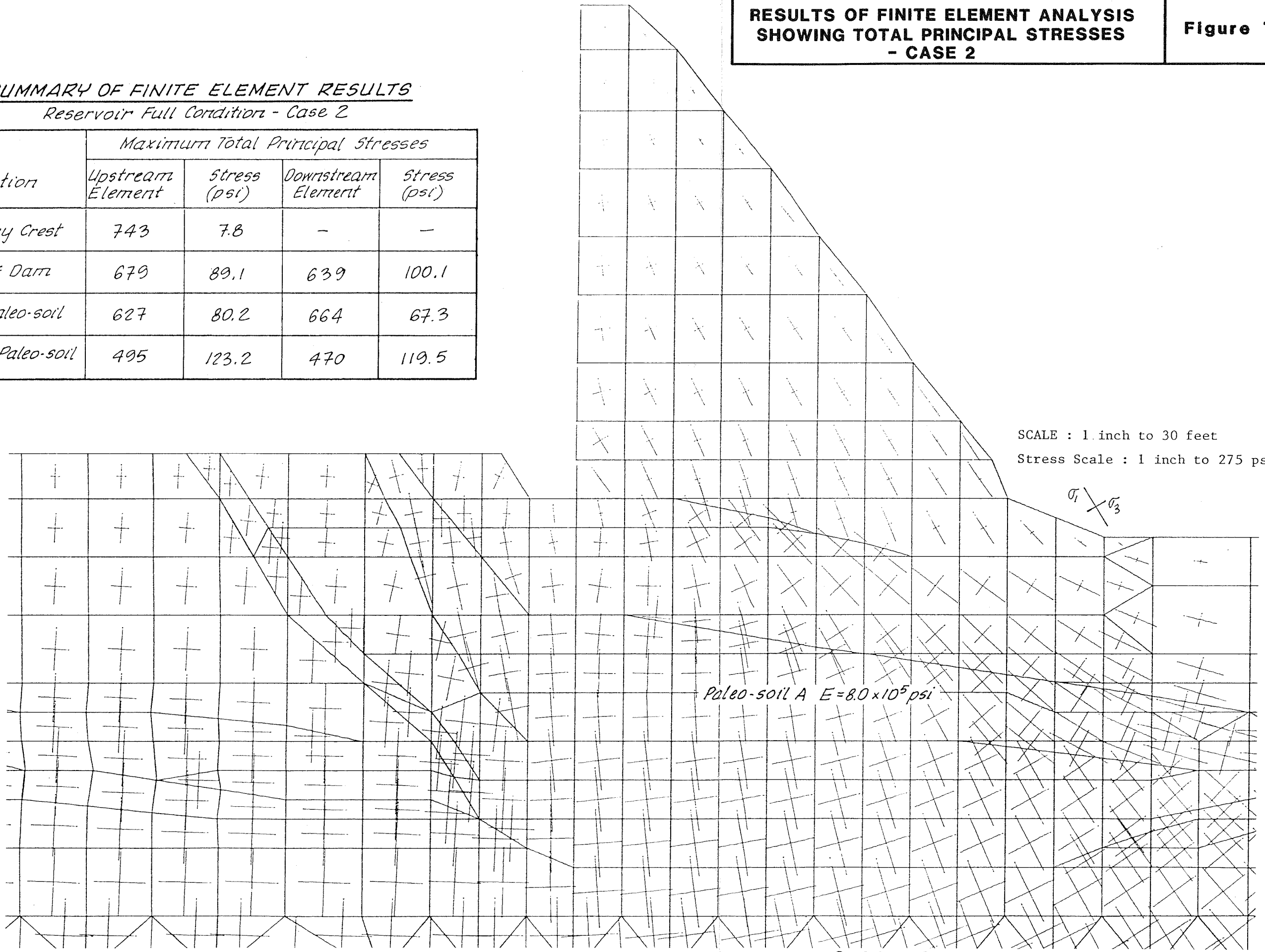
SUMMARY OF FINITE ELEMENT RESULTS

Reservoir Full Condition - Case 2

Location	Maximum Total Principal Stresses			
	Upstream Element	Stress (psi)	Downstream Element	Stress (psi)
Spillway Crest	743	7.8	–	–
Base of Dam	679	89.1	639	100.1
Top of Paleo-soil	627	80.2	664	67.3
Base of Paleo-soil	495	123.2	470	119.5



SCALE : 1 inch to 30 feet

Stress Scale : 1 inch to 275 psi

RESULTS OF FINITE ELEMENT ANALYSIS SHOWING TOTAL PRINCIPAL STRESSES - CASE 2

Figure 18b

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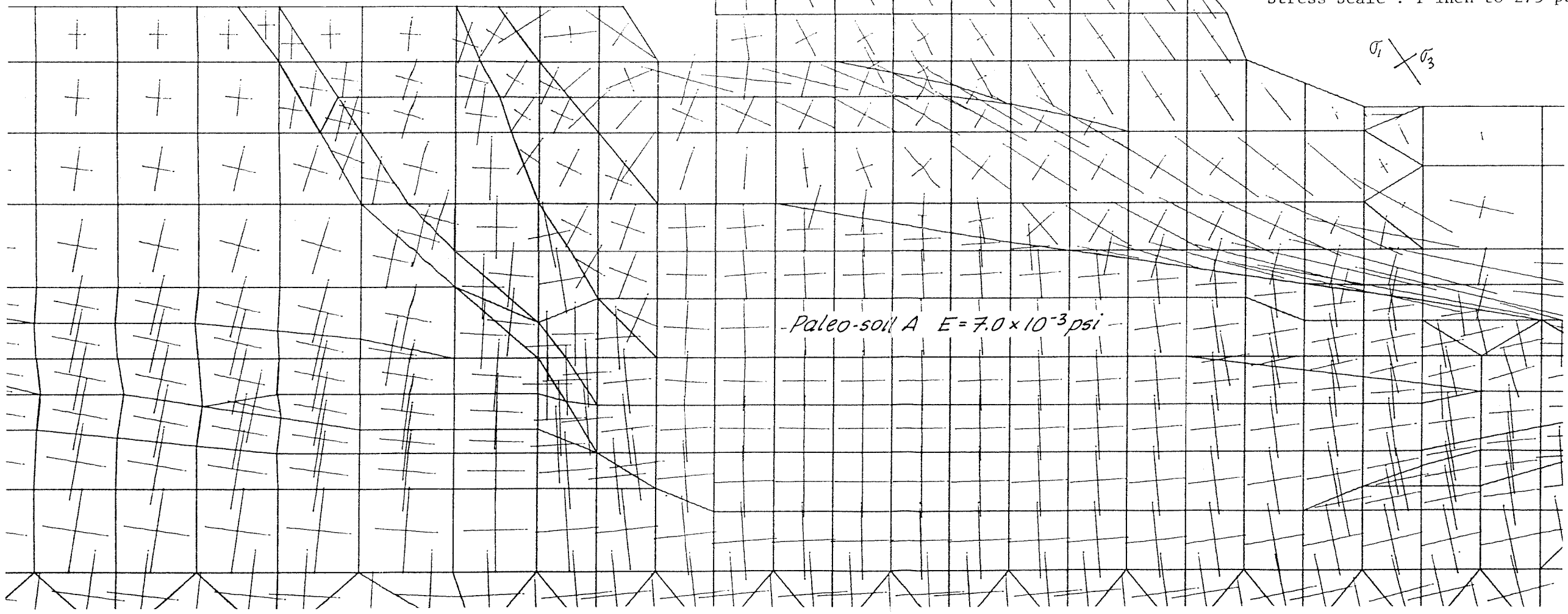
SUMMARY OF FINITE ELEMENT RESULTS

Reservoir Full Condition - Case 4

Location	Maximum Total Principal Stresses			
	Upstream Element	Stress (psi)	Downstream Element	Stress (psi)
Spillway Crest	743	7.8	–	–
Base of Dam	679	98.5	639	100.2
Top of Paleo-soil	627	205.5	664	67.3
Base of Paleo-soil	495	102.2	470	109.3



SCALE : 1 inch to 30 feet

Stress Scale : 1 inch to 275 psi

RESULTS OF FINITE ELEMENT ANALYSIS SHOWING TOTAL PRINCIPAL STRESSES – CASE 4

Figure 18d

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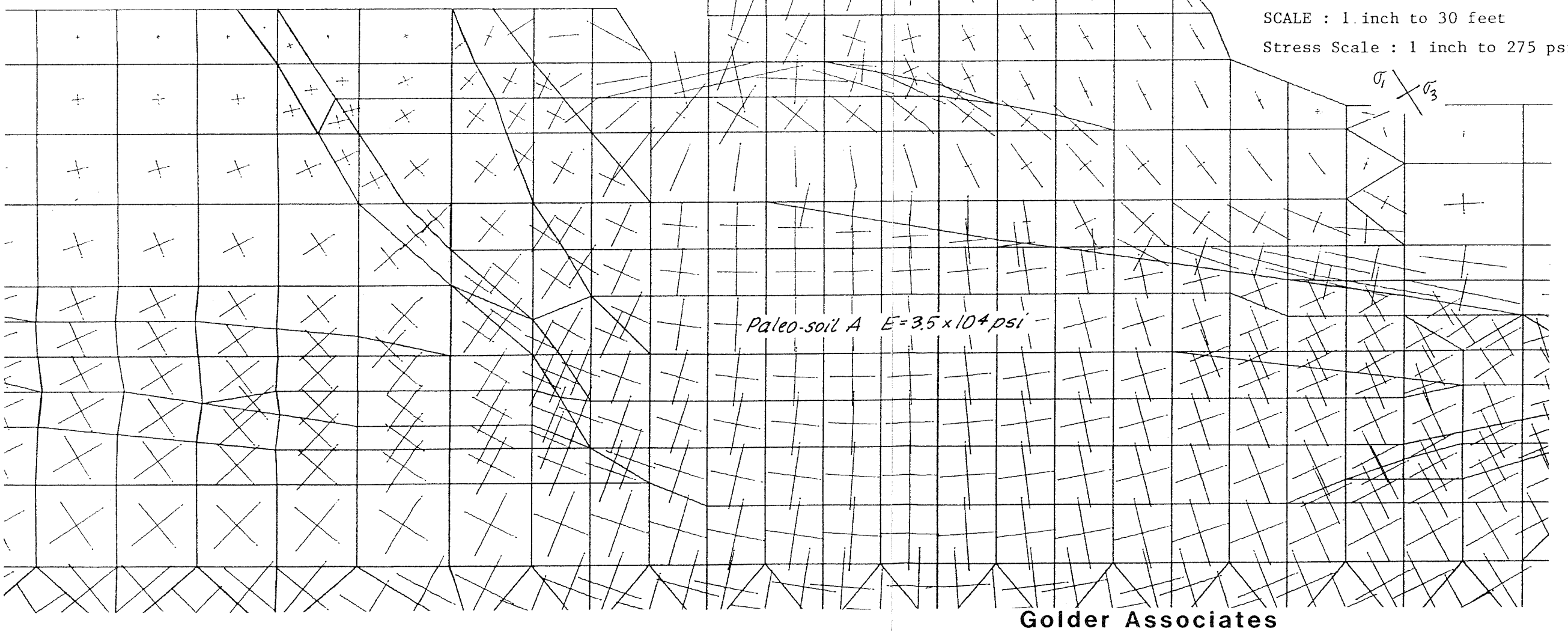
RESULTS OF FINITE ELEMENT ANALYSIS SHOWING TOTAL PRINCIPAL STRESSES – CASE 5

Figure 18e

SUMMARY OF FINITE ELEMENT RESULTS

End of Construction - Case 5

Location	Maximum Total Principal Stresses			
	Upstream Element	Stress (psi)	Downstream Element	Stress (psi)
Spillway Crest	743	6.8	—	—
Base of Dam	679	260.5	63.9	44.3
Top of Paleo-soil	627	314.0	664	7.1
Base of Paleo-soil	495	102.2	470	99.2



SCALE : 1 inch to 30 feet

Stress Scale : 1 inch to 275 psi

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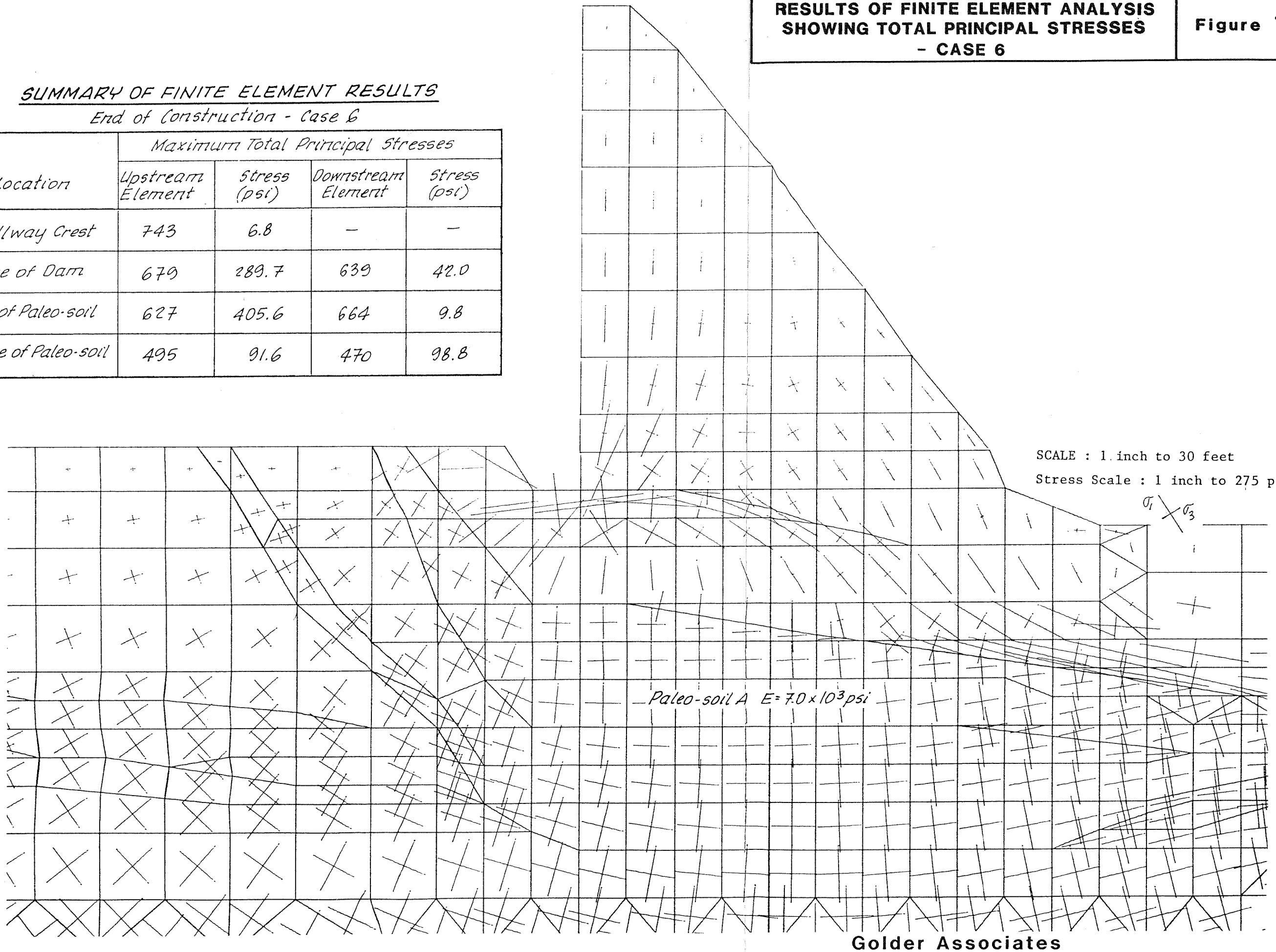
RESULTS OF FINITE ELEMENT ANALYSIS SHOWING TOTAL PRINCIPAL STRESSES - CASE 6

Figure 18f

SUMMARY OF FINITE ELEMENT RESULTS

End of Construction - Case 6

Location	Maximum Total Principal Stresses			
	Upstream Element	Stress (psi)	Downstream Element	Stress (psi)
Spillway Crest	743	6.8	–	–
Base of Dam	679	289.7	639	42.0
Top of Paleo-soil	627	405.6	664	9.8
Base of Paleo-soil	495	91.6	470	98.8



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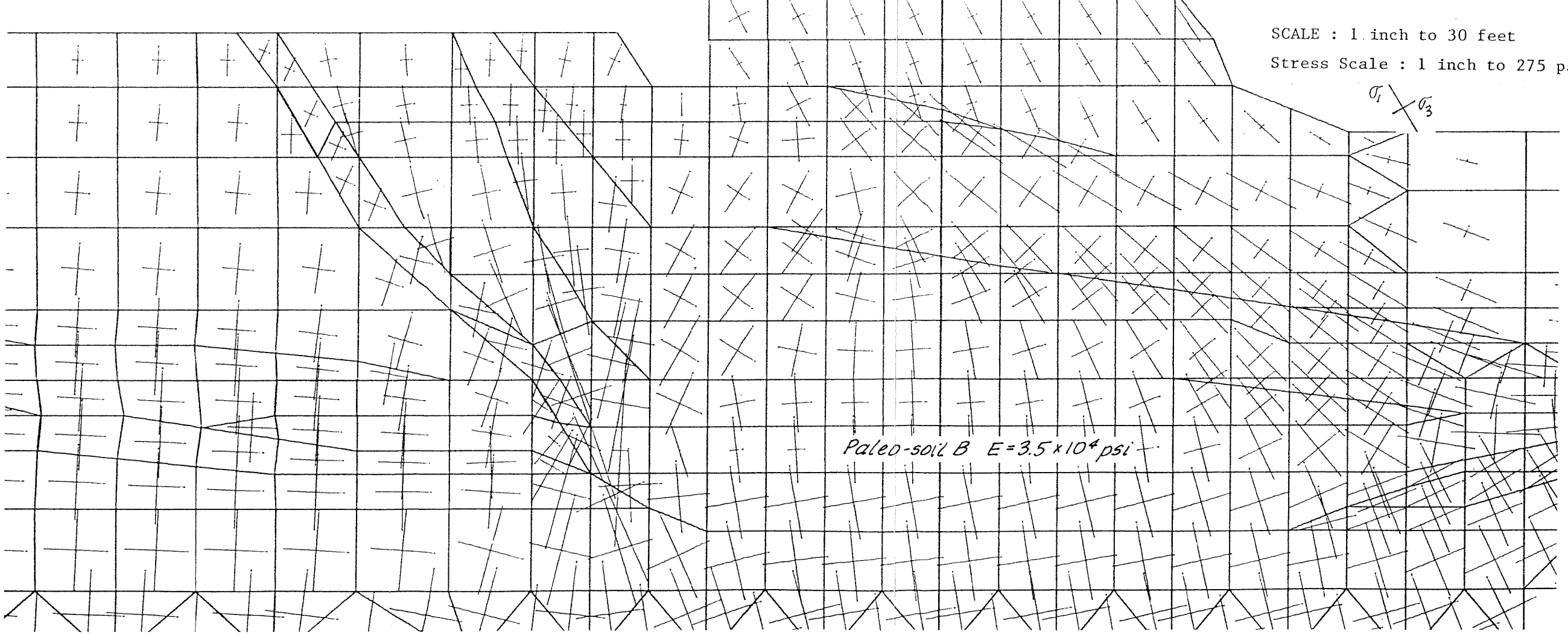
SUMMARY OF FINITE ELEMENT RESULTS

Reservoir Full Condition - Case 7

Location	Maximum Total Principal Stresses			
	Upstream Element	Stress (psi)	Downstream Element	Stress (psi)
Spillway Crest	743	7.8	–	–
Base of Dam	679	87.7	639	111.9
Top of Paleo-soil	627	78.1	664	67.3
Base of Paleo-soil	237	172.1	246	164.1

RESULTS OF FINITE ELEMENT ANALYSIS SHOWING TOTAL PRINCIPAL STRESSES – CASE 7

Figure 18g



SCALE : 1 inch to 30 feet

Stress Scale : 1 inch to 275 psi

σ_1 σ_3

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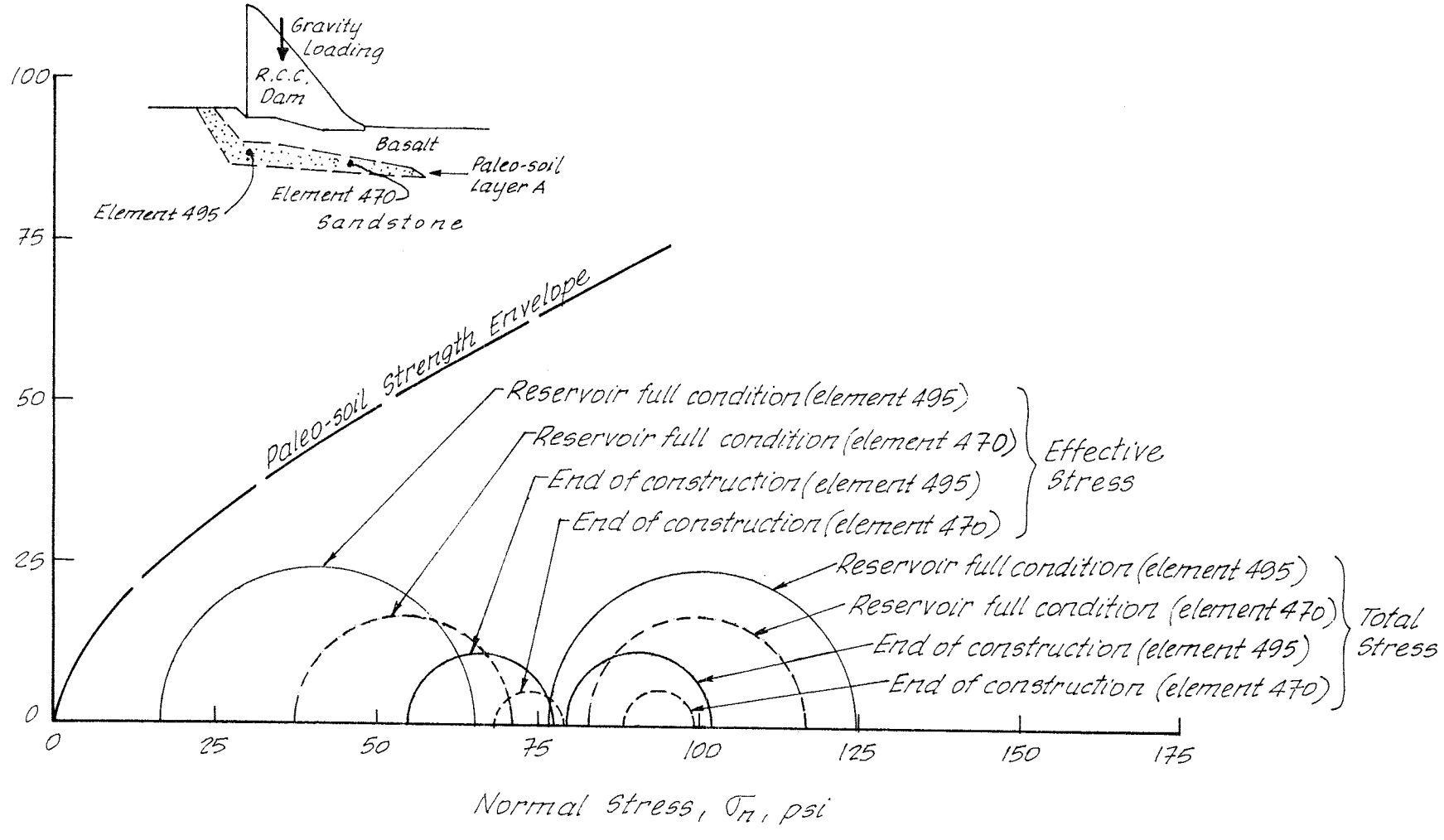

Note: Effective stress conditions calculated from results of Finite Element Analysis with paleo-soil layer A $E = 3.5 \times 10^4$ psi (cases 3 and 5) using estimated pore pressures.

STRESS CONDITIONS IN PALEO-SOIL LAYER A

Figure **19**

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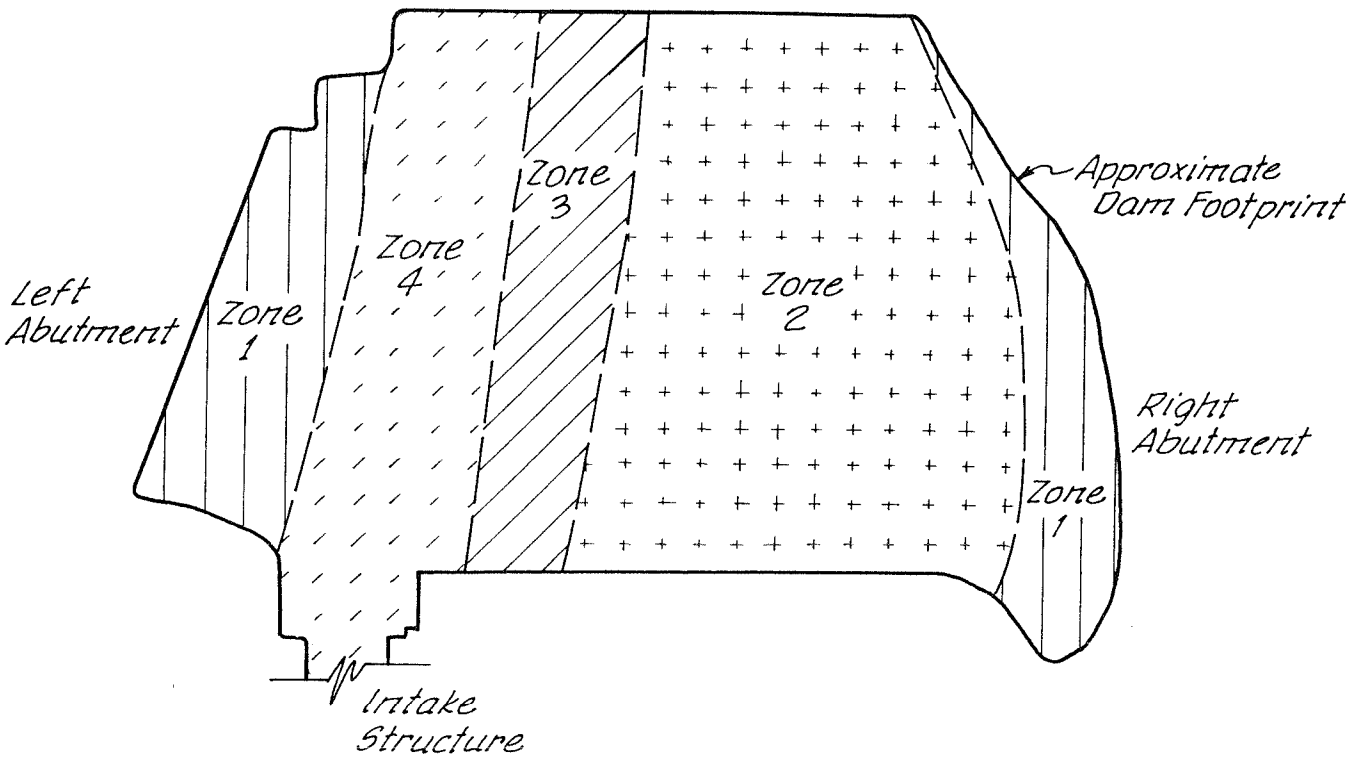
APPROXIMATE DISTRIBUTION OF SETTLEMENT

Figure **20**



Zone 1 - Paleo-soil >100 ft. deep;
Predicted Differential Settlement heal to toe ~0.1 inch (estimate)

Zone 2 - Paleo-soil 80 to 100 ft. deep;
Predicted Differential Settlement heal to toe ~0.15 inch (from modelling)

Zone 3 - Paleo-soil 60 to 80 ft. deep;
Predicted Differential Settlement heal to toe ~0.40 inch (estimate)

Zone 4 - Paleo-soil 30 to 60 ft. deep;
Predicted Differential Settlement heal to toe ~0.75 inch (from modelling)

Not to Scale.

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The damsite is located on bedrock in an area of low seismic activity and is not considered to be subject to significant seismic loading. The detailed site mapping indicates that faults are not present in the mapped area. The minimum seismic accelerations of 0.10g horizontal and 0.05g vertical as recommended by the U.S. Bureau of Reclamation are considered appropriate.

The geologic units involved in the dam foundation analysis and identifiable as having differing physical and engineering properties are:

Upper Basalt - A series of relatively thin (about 10 to 40 feet thick), subhorizontal, vesicular to massive, coalescing basalt flows form the abutments of the dam beginning at an elevation approximately 25 feet above foundation grade and extending to the top of the abutments. The flows exhibit typical, pervasive, subvertical thermal jointing and interflow boundaries with 0.1-4 inches of fine-grained, granular infilling. The Upper Basalt flows are relatively fresh with no indication of significant strength loss from weathering processes.

The Upper Basalt flows will not be significantly loaded by the dam but will form the abutments on either side of the structure. As such, their shear strength and permeability properties are required parameters for evaluation of abutment stability whereas the deformation properties are of lesser importance. Direct shear testing of sawn surfaces in the NX core was used to measure the shear strength of clean basalt discontinuities. Interflow boundaries were subjected to separate testing procedures. Ten uniaxial tests (including four with elastic properties) were used to provide information on intact rock strength and intact deformation characteristics in the Upper Basalt flows. Density values were obtained from the uniaxial samples for assessment of intact rock density. Five Goodman Jack tests were completed in the Upper Basalt and were compared to the uniaxial modulus data to help assess rock mass deformation characteristics.

Packer testing in the vertical and subvertical drill holes indicated generally low in-situ formation permeability. Occasional zones of circulation loss were encountered during drilling. The major one of these zones was interpreted as a lava tube encountered within the right abutment. Packer testing of the lava tube zone resulted in a minor "blow out" of water from the nearby cliff face before establishing any other significant flow. The lava tube is interpreted to be a blind cavity and is not considered indicative of a significant "plumbing system" within the Upper Basalt.

Abutment grouting or additional excavation may be required if significant open jointing and flow boundaries are encountered during foundation excavation. Present planning is for a minimum 20 feet of basalt to be excavated from the abutments. The 20 feet of rock removal is considered to be an appropriate depth of excavation to remove basalt significantly effected by weathering, by stress relief opening of columnar jointing, or by erosion along flow boundaries.

Abutment stability analysis assumes full reservoir head with linear pressure drop. The fracture orientation and fracture

spacing in the Upper Basalt flows indicates that subhorizontal gravity relief drains can be used successfully to control seepage pore pressures in the basalt abutments.

Other engineering and construction considerations presented by the abutment rocks include preparation of the abutment slopes. Controlled blasting will be essential to construction of the unbenched cut slopes of the foundation excavation in order to reduce blast damage to the abutment rock and to mitigate rockfall potential. Blasting will be evaluated on a performance basis with 85-90% of presplit barrels visible on the finished slope. Mitigation procedures to reduce rockfall hazard during construction include scaling, spot bolting and chain-link mesh.

Flow Boundaries - The flow boundaries between the basalt flows in the Upper Basalt present discontinuities important to the analysis of block sliding in the abutments and to the potential for piping of fines from the contact zones. Typically, the flow boundaries contain 0.1-4 inches of fine-grained, granular infilling. Although flow boundaries were detected in the Upper Basalt flows at some locations, flow boundary interbeds between the flows were not detected in the foundation and abutment area of the proposed dam.

The NX core drilling did not recover samples of interflow boundaries materials suitable for direct shear testing of intact core. Accordingly, a composite bulk sample of fines and basalt recovered from the flow boundaries in drill holes MF-102, MF-105, and MF-106 was remolded and consolidated in a shear box where it was subjected to direct shear testing in a saturated condition. The results of the direct shear tests of the bulk sample were used to conservatively approximate the shear strength along the interflow boundaries.

Atterberg Limits tests were performed on a composite sample of clay infilling recovered in NX core from flow boundaries. The results were used in the evaluation of the potential influence of this material on shear strength and its general susceptibility to piping. The location of the flow boundaries above the dam's foundation grade, the minimal thickness of the flow boundaries, and the lack of spatial continuity of the boundaries indicates that the potential for piping of interflow material is not a significant threat to the structural integrity of either the gravity dam or the abutments.

Lower Basalt - A single, subhorizontal, massive basalt flow forms the immediate foundation of the dam from approximately 25 feet above upstream foundation grade to a range of approximately 40 to 100 feet below the upstream foundation grade. The Lower Basalt is quite variable in thickness with a subhorizontal upper contact and a lower contact that follows the ancestral pre-basalt valley. The lower contact is characterized by a vesicular to scoriacious flow breccia zone generally less than 10 feet thick.

The Lower Basalt is a single flow without significant subhorizontal discontinuities or evidence of weathering induced strength reduction. Structurally, it is dominated by the typical subvertical thermal jointing common to basalt flows. Packer testing, core logging and outcrop inspection indicate the subvertical joints are typically clean and tight at depth. The Lower Basalt

forms the immediate foundation of the dam. The compressive strength and deformation characteristics of the Lower Basalt are provided by point load and RQD data in conjunction with 21 uniaxial tests (13 with elastic properties), seven direct shear tests on saw cut surfaces, and seven in-situ Goodman Jack tests. Rock density values were obtained from the uniaxial test samples.

An anomalous zone of low RQD values with highly variable core recovery and high intact rock strength was encountered at a depth of approximately 20 feet immediately below the upstream foundation grade. As indicated on the engineering geologic logs of holes penetrating the zone, (Mineral Systems Inc., 1983; Mineral Systems Inc., 1986) the rock reportedly exhibits increased jointing subparallel to the core axis and joints contain "clay filling" & "clay staining".

Detailed re-examination of the rock core samples from the zone of low RQD's revealed the low RQD's are the result of the rock coring practices. Evidence of the effects of the coring procedures include grinding of the core samples, breakage of the core, and slipped samples. The core samples further indicate that the presence of subvertical fractures aligned generally parallel to the core significantly aggravated the difficulties of obtaining intact rock core. The effects of the drilling and coring in conjunction with the near vertical fractures clearly induced considerable fracturing and breakage of the core with the result that core recovery was variable and low RQD's were observed.

Accordingly, the low RQD's and variable core recovery observed in the zone are attributed to a combination of stress relieved vertical columnar joints and drilling induced fractures. Although the zone of low RQD in the lower basalt cannot be correlated laterally into the left abutment, the evidence suggests that the zone may be laterally continuous across the width of the valley floor cut into the stratified basalt sequence at the site. This conclusion is consistent with the interpretation that the fact the vertical fractures interfered with the coring process indicates stress release and enlargement of the fractures due to unloading caused by entrenchment of the valley into the basalts. Accordingly, the stability analysis presented in the following section of this design memorandum assumes a conservatively worst case for the zone of low RQD's in which it is analyzed as a low angle fracture zone extending over the entire footprint area of the dam. Under this worst case analysis it is demonstrated that an adequate factor of safety exists for sliding or shear failure through a low angle fracture zone of low RQD in the basalt foundation.

Paleo Colluvium/Alluvium - A layer of unconsolidated material is present between the Lower Basalt flow and the underlying Supai Formation bedrocks. The thickness of the basalt overlying the unconsolidated material ranges from about 40 feet near the left abutment to 80 feet near the dam mid point and about 60 feet near the right abutment. The thickness of the unconsolidated material ranges from about 35 feet near the left abutment to 10 feet at the dam mid point and 20 feet thick near the right abutment.

The unconsolidated material represents the surficial deposits which were present in the ancestral valley of the North Fork of the White River at the time basalt flows began to fill the ances-

tral valley. Accordingly, the unconsolidated materials buried by the basalts which filled the ancestral river valley include fluvially deposited silty sand and gravel deposited along the ancestral river channel and flood plain, fine-grained silt and sand deposits emplaced as colluvial foot slope deposits along the valley margins, and sandstone rock rubble in a fine-grained silty sand matrix deposited as talus at the base of cliffs on the ancestral valley walls. The unconsolidated deposits buried by the basalts are collectively referred to herein as Paleo Colluvium/Alluvium.

The NX core barrels used in the coring program did not recover intact samples of the Paleo Colluvium/Alluvium materials. The partial samples recovered and the response of the equipment during coring of the Paleo Colluvium/Alluvium interval indicate that the Paleo Colluvium/Alluvium consists of coarse gravel, cobbles, and boulders in a dense matrix of silty sand. Examination of outcrops of the Paleo Colluvium/Alluvium downstream from the proposed damsite support this interpretation. The only field data developed regarding the strength parameters of the Paleo Colluvium/Alluvium was completion of three Goodman Jack tests in drill hole MF-113. The results of the tests are not considered to be representative or reliable. Laboratory data regarding the materials properties of the Paleo Colluvium/Alluvium was not obtained due to the lack of samples for analysis.

Core holes reamed through the Paleo Colluvium/Alluvium indicated that although the materials are essentially unindurated, they stand well in the borehole walls, are slowly permeable, and are relatively dense and compact. The geologic conditions at the site indicate that the Paleo Colluvium/Alluvium was subjected to considerable preloading by from the weight of the basalt before the contemporary river valley was entrenched into the basalt. Thus the Paleo Colluvium/Alluvium materials under the dam foundation are anticipated to be preconsolidated. The in-situ permeability tests and the recovered material indicates the Paleo Colluvium/Alluvium is sandy in nature and apparently without significant clay or open work gravel components. As such, and considering it has already carried a load greater or equal to that from the dam, significant time-dependent settlement problems are considered highly unlikely. Compression of the unit is expected to occur during construction under elastic conditions rather than due to permeability controlled consolidation by release of pore pressure.

Based on the foregoing considerations plus the limited field data from the boreholes and from the examination of the Paleo Colluvium/Alluvium outcrops downstream from the damsite, the Paleo Colluvium/Alluvium is considered to be analogous to compacted rockfill material. Accordingly, a range of values for shear strength and deformation parameters was determined for the Paleo Colluvium/Alluvium principally from published data for compacted rockfill.

Because direct information for the materials properties of the Paleo Colluvium/Alluvium was not available, a wide range of presumptive values obtained from the published literature was used in the stability analysis to provide predictions based on the conservatively worst case conditions probable. The stability analysis presented in the following section of this memorandum

indicates a factor of safety of 2 or greater for a sliding or shear failure through the Paleo Colluvium/Alluvium under worst case conditions.

Analysis of total settlements using the range of published values assigned to the Paleo Colluvium/Alluvium indicates that under worst case conditions, lateral differential settlement related to the Paleo Colluvium/Alluvium in the foundation will be a design consideration. Clearly, more information on the geometry and material properties of the Paleo Colluvium/Alluvium underlying the foundation is required and will be obtained during the final engineering studies. If the final engineering studies indicate that a potential for excessive differential settlement actually exists due to the properties of the Paleo Colluvium/Alluvium, then remedial measures such as consolidation grouting or cast-in-place piles will be evaluated.

The Paleo Alluvium/Colluvium presents a potential path for seepage losses from the reservoir. The potential seepage paths include seepage under the dam through the buried sediments of the ancestral river valley as well as seepage through the left abutment through the buried sediments on the ancestral valley wall. Geologic relationships indicate that seepage through the left abutment Paleo Colluvium/Alluvium could daylight on the valley wall downstream from the dam whereas seepage through the main paleo channel would daylight approximately one mile downstream from the dam. The potential seepage losses and seepage pressures associated with the Paleo Colluvium/Alluvium are discussed in detail in Section D.3 herein. Finite difference modeling of seepage through the abutments and buried paleo channel under the basalt as presented in Section D.3 indicates that seepage losses will not be significant.

The outcrop of the Paleo Colluvium/Alluvium has not been mapped immediately downstream of the reservoir. However, it is presumed the unit conforms to the base of the basalt and daylights along the basal contact of the basalt. As such, the Paleo Alluvium/-Colluvium is is confined beneath basalt in the left abutment for at least 820 feet downstream from the dam where, buried beneath Supai talus, a "window" through the basalt may occur (Figure 3, Mineral Systems Inc., 1986). With the exception of the potential "window", the main channel of the buried Paleo Colluvium/Alluvium should not daylight for approximately a mile downstream from the dam (Mineral Systems Inc., 1986, p. 43). The seepage analysis presented in Section D.3 herein indicates that even if the Paleo Colluvium/Alluvium material is susceptible to piping, the hydraulic gradient of the phreatic seepage surfaces simulated by the digital seepage model is considerably less than the gradient generally associated with the onset of piping conditions. If subsequent studies or experience indicate a significant potential for piping, for example at the "window" through the basalt downstream from the left abutment, a series of filter protected gravity drain holes through the basalt or a filter blanket drain at the outcrop will mitigate the piping problem.

Supai Sandstone/Siltstone - The Supai Formation is the lowermost geologic unit mapped or penetrated by drill holes at the proposed damsite. The surface outcrops and uppermost part of the Supai Formation penetrated by the drill holes consists of interbedded fine-grained sandstone and siltstone. The sandstone and silt-

stone forms the ridge behind the left abutment and comprises approximately 30 to 90 feet of the thickness of the bedrocks investigated by coring beneath the axis of the damsite. The upper contact of the interbedded sandstone and siltstone is the erosional surface formed by the ancestral river valley, approximately 80 to 135 feet below the heel of the dam.

The interbedded sandstone and siltstone of the Supai Formation does not present any unusual or limiting design considerations for the foundation performance. The depth of the unit below foundation grade considered with the confined nature and history of loading of the unit indicate it will not negatively impact the project design or construction. Shear strength and deformation parameters of the interbedded sandstone and siltstone were determined by five direct shear tests (two on saw cut surfaces and three on intact samples), four uniaxial compressive tests (two with elastic properties), and four Goodman Jack tests in the sample intervals subjected to uniaxial testing.

Supai Gypsiferous Sandstone/Gypsum - The deepest unit investigated beneath the dam is a facies of the Supai Formation composed of fine-grained gypsiferous sandstone with lenses of gypsum. This unit has a gradational, subhorizontal upper contact approximately 160 feet below the heel of the dam.

For the same reasons given for the interbedded sandstone and siltstone facies above, the gypsum bearing facies is considered to not significantly impact the foundation performance. A general suite of tests consisting of seven uniaxial compression tests (four with elastic properties), four direct shear on core, and two direct shear tests on saw cut surfaces were completed on samples from the gypsiferous sandstone facies in the Supai Formation. All o the Supai Formation beneath the dam foundation is saturated and and in-situ permeability tests of the gypsiferous sandstone facies indicate a low permeability (approximately 10^{-6} cm/sec). As such, the potential for dissolution of gypsum is considered to be insignificant.

D.2 FOUNDATION STABILITY STUDIES

The present design is based on a preliminary engineering geologic assessment of the site. The site geology does not present exceedingly difficult or unusual foundation problems and is well suited to a concrete gravity dam of either conventional concrete or roller compacted concrete construction.

A simple foundation shape is maintained in order to eliminate stress concentrations in the dam and allow efficient excavation. For the conventional concrete option it avoids narrow monoliths with small base areas, small areas of difficult concrete placement and torsion on monoliths. For the joint-free, roller compacted concrete option it avoids small areas of difficult placement.

The amount of dental concrete required is expected to be small. Controlled blasting procedures will be required to excavate the foundation with dental concrete used for voids that will not be filled during normal placement. For the roller compacted concrete option a high slump, bedding mix will be used at all interfaces between foundation rock and the dam. This technique, using procedures tested and proven on other projects, results in excellent dam to rock bonding.

Foundation and abutment stability studies were accomplished by Golder Associates and are presented in the report titled, "Report to Morrison-Maierle, Inc. on Stability Analysis of the Miner Flat Dam Foundations, Navajo County, Arizona", which is presented herein beginning on the following page. Data for the stability studies were provided from the Mineral Systems, Inc. field investigations and from independent laboratory tests of materials properties provided to Golder Associates by Morrison-Maierle, Inc. Summaries of the field and laboratory data are provided in the Golder Associates report with a narrative description of the stability problem, the analytical methods applied to the problem, and conclusions and recommendations.

DESIGN MEMORANDUM

SECTION D

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D.3 SEEPAGE MODELS

1.1. INTRODUCTION

Numerical simulation of seepage flows by means of digital modeling was used to evaluate potential seepage conditions in the foundation and abutments of the Miner Flat Dam site. The potential seepage path evaluated in the foundation of the dam was a buried paleoalluvial channel consisting of unconsolidated deposits of alluvium of the ancestral pre-basalt North Fork of the White River. Potential seepage paths evaluated in the abutments of the dam included fractured basalt in the right abutment; and basalt, paleocolluvium, and the Supai Formation in the left abutment.

1.2. MODELING METHODOLOGY

The modeling methodology selected to simulate seepage flows and pore pressures in the foundation and abutment materials at the Miner Flat Dam is a finite difference model. The model employs conventional iterative alternating implicit solution of simultaneous partial differential equations governing nonsteady state, two-dimensional groundwater flow between model nodes. The program code used for the Miner Flat Dam seepage simulations is a widely available groundwater flow simulation program referred to as the Prickett-Lonnquist Aquifer Simulation Model (PLASM). The mathematical model was initially prepared by the Illinois State Water Survey (Prickett and Lonnquist, 1971). The original PLASM code of Prickett and Lonnquist (1971) was retained in the Miner Flat Dam models and modified with a number of subroutines to facilitate input and output data management.

Other modifications to the PLASM code were limited to provisions to deal with dewatered nodes and nodes that were in an initially dewatered or dry condition. Aquifer transmissivities

in the model were based on the product of the permeability and the differential head along the permeability vectors. Because the differential head between dry nodes is zero, the transmissivity between dry nodes will always remain at zero under the original PLASM code. Consequently, dry nodes will always remain dry because water cannot flow into an area of zero transmissivity. Accordingly, minor reprogramming of the original PLASM code was required to revise this particular aspect of the original model.

Selection of a finite difference model for numerical simulation of seepage flow in the buried channel at the Miner Flat Dam was based on consideration of several factors:

1. Digital modeling provided considerable advantages over steady state and transient flow net analyses by responding to the complex three-dimensional distribution of boundary conditions at the site.

2. Laborious construction of transient flow nets was avoided by the application of digital modeling with the result that both transient and steady state seepage conditions were determined rapidly for a wide range of input parameters.

3. The model selected is well documented and easily acquired.

4. The model is a valid numerical model of proven credibility.

5. The model is easily adapted to most computers.

6. The model output may be modified to a number of easily used formats.

7. Input parameters required for the model are based on information that is normally provided by conventional field investigations and which does not require unusual or difficult field tests and measurements.

8. Input parameters may be readily modified over a range of values to test the sensitivity of the seepage analysis to the accuracy of the field measurements of aquifer permeability and to assumptions in extrapolation of data from drill hole locations to the rest of the model.

1.2.1. Input/Output Parameters

Input parameters for the Miner Flat Dam seepage models included 11 variables as follows:

1. X and Y coordinates defining the node positions and dimensions.
2. Permeability (hydraulic conductivity) in the X and Y direction (anisotropic as appropriate) based on packer tests of permeability conducted in boreholes in the field.
3. Transmissivity vectors in the X and Y directions to provide anisotropic transmissivity as required.
4. An unconfined aquifer storage coefficient.
5. A confined aquifer storage coefficient.
6. Initial head (pore pressure) in the aquifer.
7. The elevation of the bottom of the aquifer (bottom of the seepage zone).
8. The elevation of the top of the aquifer (top of the seepage zone, i.e., elevation of the base of the confining bed, if any).

Model output parameters consisted of pore pressure (head), rate of seepage flow, change in pore pressure, and an index map of node conditions (i.e., confined, unconfined, dry, sink, boundary, etc.).

The input variables for the seepage model of the buried paleoalluvial channel were based on a 12 by 24 or 288 node array for a total of 3,168 discrete input values. The overall grid dimensions were 2,018.75 feet wide by 16,477.35 feet long. Input variables defining the coordinates of the nodes in the model were based on a nonuniform grid spacing designed to include a reservoir area, the dam, and the area downstream from the dam. The nonuniform grid was designed specifically to provide reasonable definition of the controlling boundary conditions in the foundation, abutments, and downstream areas. Values for the input variables were defined by superimposing the model grid over the

geologic maps and cross sections prepared by Mineral Systems, Inc. as presented in "Preliminary Report - Engineering Geology Miner Flat Dam Site, White Mountain Apache Reservation".

The model of seepage through the abutments of the dam site utilized an 18 by 17 array consisting of 306 nodes for a total of 3,366 discrete input values. The overall dimensions of the abutment seepage model were 4,280 feet parallel to the axis of the proposed dam and 3,580 feet perpendicular the the alignment of the proposed dam axis. The arrays used to define the model nodes and boundary conditions for the model of the paleoalluvial channel are shown on Figure 1-1 and those for the model of seepage through the abutment rocks are shown on Figure 1-16. Geologic information provided by Mineral Systems Inc. was used in defining the input variables to the abutments seepage model.

1.2.2. Boundary Conditions

Geologic conditions identified by the geotechnical investigations of the site and presented in the Mineral Systems, Inc. report which were regarded as significant controlling factors in designing the modeling grids and establishing the boundary conditions in the two seepage models included geologic contacts between different materials, thicknesses of the various materials, elevations of geologic contacts, and field measurements of hydraulic conductivities of the various geologic materials. Terrain data provided by the 1 inch to 200 feet topographic maps with 5-foot contour intervals provided additional definition of boundary conditions influencing seepage paths and hydraulic gradients.

The thickness of the paleoalluvium penetrated by drill holes on a cross section along the dam axis was used to define the the seepage zone or aquifer thickness of the paleoalluvial buried

channel. Basalt overlying the buried channel and bedrocks bounding the bottom and sides of the buried channel were assumed to be impermeable boundaries to groundwater movement. Both the buried channel seepage model and the abutments seepage model included a vertical zone of unconfined permeable paleocolluvium in the left abutment of the damsite.

The unconfined paleocolluvium zone in the left abutment provided direct hydraulic communication between the reservoir area upstream from the dam and the river valley downstream from the dam so that a seepage path was present in the left abutment from the reservoir to the downstream area in the left abutment in both models. The seepage path was assumed to have a minimum thickness of 10 feet normal to the geologic contacts and was assigned the same permeability as the paleocolluvium in the buried channel model. In the abutments seepage model the paleocolluvial seepage path in the left abutment was assumed to be the entire width and thickness of the nodes defining the seepage path and therefore ranged in width from 150 to 270 feet or substantially thicker and wider than the actual field conditions. The paleocolluvial channel representations in the two seepage models was unconfined just as is the paleocolluvial interface between the basalt and Supai Formation in the left abutment shown on the Mineral Systems, Inc. cross sections.

In the buried channel seepage model, a wedge paleocolluvial material, assumed to be 10 feet thick normal to the geologic contacts, was projected into the Miner Flat Dam reservoir area from the buried paleoalluvial channel beneath the basalt. The paleocolluvial wedge provided a direct seepage path from the surface water reservoir into the buried paleocolluvial channel under the basalt foundation at the dam site. The buried channel

seepage model carried the buried paleoalluvial channel downstream from the Miner Flat Dam to a location identified by Mineral Systems, Inc. where the paleoalluvial channel daylights into the river valley and would therefore discharge seepage at the land surface downstream from the dam.

The wedge of paleocolluvial material, 10 feet thick normal to the geologic contacts, was used to provide hydraulic communication in the buried channel seepage model between the buried paleoalluvial channel and the wall or floor of the river channel downstream from the dam. Therefore, if the pore pressure head elevation in the paleoalluvium downstream from the dam increased above the elevation of the interface between the basalt and the underlying Supai Formation exposed in the valley wall downstream from the dam, the model allowed seepage water to flow from the buried channel into the North Fork of the White River.

1.3. BURIED PALEOALLUVIAL CHANNEL SEEPAGE MODEL

Design of the nonuniform grid for the buried channel seepage model was based on consideration of the previously described hydrogeologic boundary conditions that determine the potential gradients of seepage paths in the paleocolluvium and paleoalluvium at the Miner Flat Dam site, assuming that the stratified basalt and bedrocks act as essentially impervious strata confining significant seepage flows to the unconsolidated prebasalt colluvium and alluvium. Field measurements of average hydraulic conductivities in the materials indicated an average value of 5.11×10^{-5} cm/sec for the basalt, or an order of magnitude less permeability than the average value of 1.76×10^{-4} for the paleoalluvium. Grid spacings were selected to provide reasonable definition of the geohydrologic boundary conditions indicated by the Mineral Systems, Inc. geotechnical investigations and

resulted in the following coordinate spacings for the buried channel seepage model:

Coordinate Interval	X Coordinate Spacings (feet)	Y Coordinate Spacings (feet)
1 - 2	506.25	3,844.34
2 - 3	337.50	3,844.34
3 - 4	225.00	2,562.89
4 - 5	150.00	1,708.59
5 - 6	100.00	1,139.06
6 - 7	100.00	759.38
7 - 8	100.00	506.25
8 - 9	100.00	337.50
9 -10	100.00	225.00
10 -11	100.00	150.00
11 -12	100.00	100.00
12 -13 (x-Bndy)	100.00	100.00
13 -14 Total	2,018.75	100.00
14 -15		100.00
15 -16		100.00
16 -17		100.00
17 -18		100.00
18 -19		100.00
19 -20		100.00
20 -21		100.00
21 -22		100.00
22 -23		100.00
23 -24		100.00
24 - y-Bndy		100.00
		Total 16,477.35

1.3.1. **Input Variables**

Transmissivity values used in the buried channel seepage flow simulations were based on the field measurements of an average value of hydraulic conductivity of 1.76×10^{-4} cm/sec as reported in the geotechnical report by Mineral Systems, Inc. The actual values of hydraulic conductivity used in the seepage flow simulations for the paleoalluvium ranged from 1.00×10^{-4} cm/sec to 1.00×10^{-2} cm/sec, i.e., the order of magnitude of the field measurements in one simulation and 100 times the order of magnitude of the field measurements in the second seepage flow simulation. The range of hydraulic conductivities used was employed to test the sensitivity of the model to the accuracy of the field measurements of hydraulic conductivity.

Transmissivity in the model is the product of the saturated

thickness times the permeability. The model automatically recalculated a new transmissivity for each node as water levels in the seepage zone changed. Thus, transmissivity values were continuously adjusted during the seepage flow simulations to reflect the ongoing changes in the pore pressure in the seepage zone.

An unconfined aquifer storage coefficient of 0.01 was used for the paleoalluvial material in the buried channel seepage model based on the assumption that the unconfined storativity of the seepage zone in the paleoalluvium was approximately equal to an assumed value of effective porosity of 1.0 percent in the paleoalluvial materials. Field measurements of the paleoalluvial material porosity were not available, however, sensitivity testing of the porosity assumption indicated that the assumption had little effect on the final seepage flows and pressures indicated by the model. Similar assumptions and sensitivity testing were used to arrive at a presumptive value of 10^{-4} for storativity for confined seepage flow conditions in the seepage zone. The model compared the pore pressure elevation to the elevation of the base of the confining layer for each node in order to assign an unconfined or confined storage coefficient value as appropriate during the seepage flow simulations.

1.3.2. Simulation Concepts

Initial pore pressures in the paleoalluvium seepage zone were established by setting the head elevations in the aquifer equal to the gradient of the North Fork of the White River. A series of transient flow simulations were then conducted to arrive at a simulation of steady state flow in the paleoalluvial aquifer without the presence of the dam and reservoir. The dam and reservoir were then superimposed instantaneously upon the steady state flow condition, with a full reservoir, and subse-

quent transient flow simulations were accomplished until seepage flow from the full reservoir reached steady state flow conditions as indicated by the absence of significant head change at nodes in the model. Steady state seepage flow from the full reservoir was used as a measure of potential seepage pore pressures.

The elevations of the top and bottom of the paleoalluvial seepage zone materials were obtained by superimposing the model grid over a series of geologic cross sections and profiles prepared from the Mineral Systems, Inc. geologic maps and cross section data combined with the topographic map contour elevation data for the study site. The top and bottom elevations define the thickness of the seepage flow path. Moreover, the relationship between the pore pressure head elevation and the elevation at the base of the confining layer, if any, determines if the model uses an unconfined or a confined aquifer storage coefficient. The topographic elevation of the surface outcrop of the paleocolluvial wedge in the left abutment and in the river valley downstream from the dam is the threshold elevation at which seepage pore pressure in the paleoalluvial seepage zone will cause seepage discharge from the valley walls into the White River downstream from the dam.

Figure 1-1 is a schematic of the seepage flow model at the end of one year of seepage flow simulation. The following designations are used on Figure 1-1 to show aquifer and boundary conditions in the model after one year of seepage flow from a full reservoir behind the Miner Flat Dam:

1. C = confined aquifer node - This symbol indicates that the pore pressure in the paleoalluvium/paleocolluvium is at an elevation head in excess of the basal elevation of the overlying basalt and seepage flow is occuring as confined groundwater flow which is exerting an uplift pressure on the overlying basalt and/or the dam.

MINER FLAT DAM WITH RESERVOIR, AFTER 4-YEARS SIMULATION

NODE DESIGNATIONS

CONFINED AQUIFER NODE = C, UNCONFINED AQUIFER NODE = U
DEWATERED AQUIFER NODE = DRY, NON-AQUIFER NODE = ----

#	1	2	3	4	5	6	7	8	9	10	11	12
24	----	SOURCE	SOURCE	SOURCE	SOURCE	SOURCE	SOURCE	SOURCE	SOURCE	----	----	----
23	----	C	C	C	C	C	C	C	SOURCE	----	----	----
22	----	C	C	C	C	C	C	C	SOURCE	----	----	----
21	----	C	C	C	C	C	C	C	SOURCE	----	----	----
20	----	C	C	C	C	C	C	C	SOURCE	----	----	----
19	----	C	C	C	C	C	C	C	SOURCE	----	----	----
18	----	C	C	C	C	C	C	C	C	----	----	----
17	----	C	C	C	C	C	C	C	C	C	----	----
16	----	C	C	C	C	C	C	C	C	C	----	----
15	----	C	C	C	C	C	C	C	C	C	C	----
14	----	C	C	C	C	C	C	C	C	C	C	----
13	----	C	C	C	C	C	C	C	C	C	C	----
12	----	C	C	C	C	C	C	C	C	C	C	SINK
11	----	C	C	C	C	C	C	C	C	C	C	SINK
10	----	C	C	C	C	C	C	C	C	C	C	SINK
9	----	C	C	C	C	C	C	C	C	C	C	SINK
8	----	C	C	C	C	C	C	C	C	C	C	SINK
7	----	C	C	C	C	C	C	C	C	C	C	SINK
6	----	C	C	C	C	C	C	C	C	C	C	SINK
5	----	C	C	C	C	C	C	C	C	C	U	SINK
4	----	U	C	C	C	C	C	C	C	C	U	SINK
3	----	U	C	C	C	C	C	C	C	C	U	SINK
2	----	U	C	U	U	U	U	U	U	U	U	SINK
1	----	SINK	SINK	SINK	SINK	SINK	SINK	SINK	SINK	SINK	SINK	SINK

DAM

Fig. 1-1: Schematic of Seepage Flow Model

2. U = unconfined aquifer node - This symbol indicates that seepage flow is taking place in the paleoalluvium/paleocolluvium, however, the pore pressure has not caused the phreatic surface (potentiometric surface) to rise to the basal elevation of the overlying basalt and seepage flow is occuring without exerting an uplift pressure on the basalt and/or dam.

3. DRY = dewatered aquifer node - This symbol indicates a node in which the paleoalluvial/paleocolluvial material is entirely drained. This condition exists in the model only as an initial condition in the wedge of paleocolluvium bewteen the buried paleoalluvium and the river valley and dissappears during the flow simulations as the seepage pressure elevation rises in the buried paleoalluvial channel seepage path and seepage flows over the threshold through the paleocolluvium nodes and into the row of sinks representing the river valley.

4. --- = nonaquifer node - This symbol indicates a node with zero permeability and zero transmissivity, i.e., a no flow boundary within the model. No flow boundaries are used to represent the right side of the paleoalluvial channel and to represent the portion of the left reservoir bank and left abutment consisting of the Supai Formation.

5. SOURCE = reservoir or source - This symbol indicates the area within the model which supplies water to simulate either the steady state source of recharge to the paleoalluvial seepage path from the river during pre-dam conditions or the steady state source of recharge to the paleoalluvial seepage path from the Miner Flat reservoir. These nodes are assigned essentially infinite storativity values in the model.

6. SINK = discharge location - This symbol indicates the location of a node which has been assigned essentially infinite storativity in the model and which elevation is set low enough that seepage always flows from adjacent nodes into this type of node anytime there is water available in the adjacent nodes. The sink nodes are used to represent the White River valley downstream from the proposed dam in order to provide a place for seepage flow to discharge laterally from the paleoalluvial channel into the river valley when seepage pore pressures become sufficient to support such discharge. The sink nodes are also used across row 1 of the model to simulate the location at which the paleoalluvial channel is intersected by the river valley downstream from the damsite and to which confined or unconfined seepage flow in the paleoalluvial channel must discharge.

As shown on Figure 1-1, the five nodes in the model located within the foundation area of the proposed Miner Flat Dam are (6,17), (6,18), (7,17), (7,18), and (8,18). The grid spacing in the vicinity of the dam and reservoir is 100 by 100 feet, includ-

ing the nodes within the foundation area.

1.3.3. Seepage Flow Simulations

The number of time increments and the duration of each time increment required to reach steady state seepage flow from the continuously full surface water reservoir condition was established by trial and error transient seepage flow simulations. The transient seepage flow simulations indicated that when the order of magnitude of the average measured field hydraulic conductivity of 1.00×10^{-4} cm/sec was used as an input variable, essentially steady state seepage flow was established by the end of four years of seepage in the paleoalluvial materials.

As shown on Figure 1-2, seepage losses from the reservoir through the paleoalluvial materials were about seven gallons per minute. Seven gallons per minute amounts to about 11.3 acre-feet per year of seepage losses. The rate of seepage loss as determined by the model is related directly and linearly to the hydraulic conductivity of the seepage path materials. Accordingly, if it is assumed for the sake of sensitivity testing that the field measurement values of hydraulic conductivity for the paleoalluvial materials were increased of two orders of magnitude by a multiple of 100, the rate of seepage under steady state conditions would be about 700 gpm or 1129 acre-feet per year as shown on Figure 1-3. The estimated seepage losses shown on Figures 1-2 and 1-3 represent only the steady state seepage losses through the paleoalluvial materials and are not indicative of bank loss or bank storage rates for the entire reservoir.

Surface water hydrology analysis for the watershed indicates that an average of 50,000 to 60,000 acre-feet per year of water will be spilled over the dam annually. In view of this fact, it is concluded that annual seepage losses of 1,129 acre-feet to the

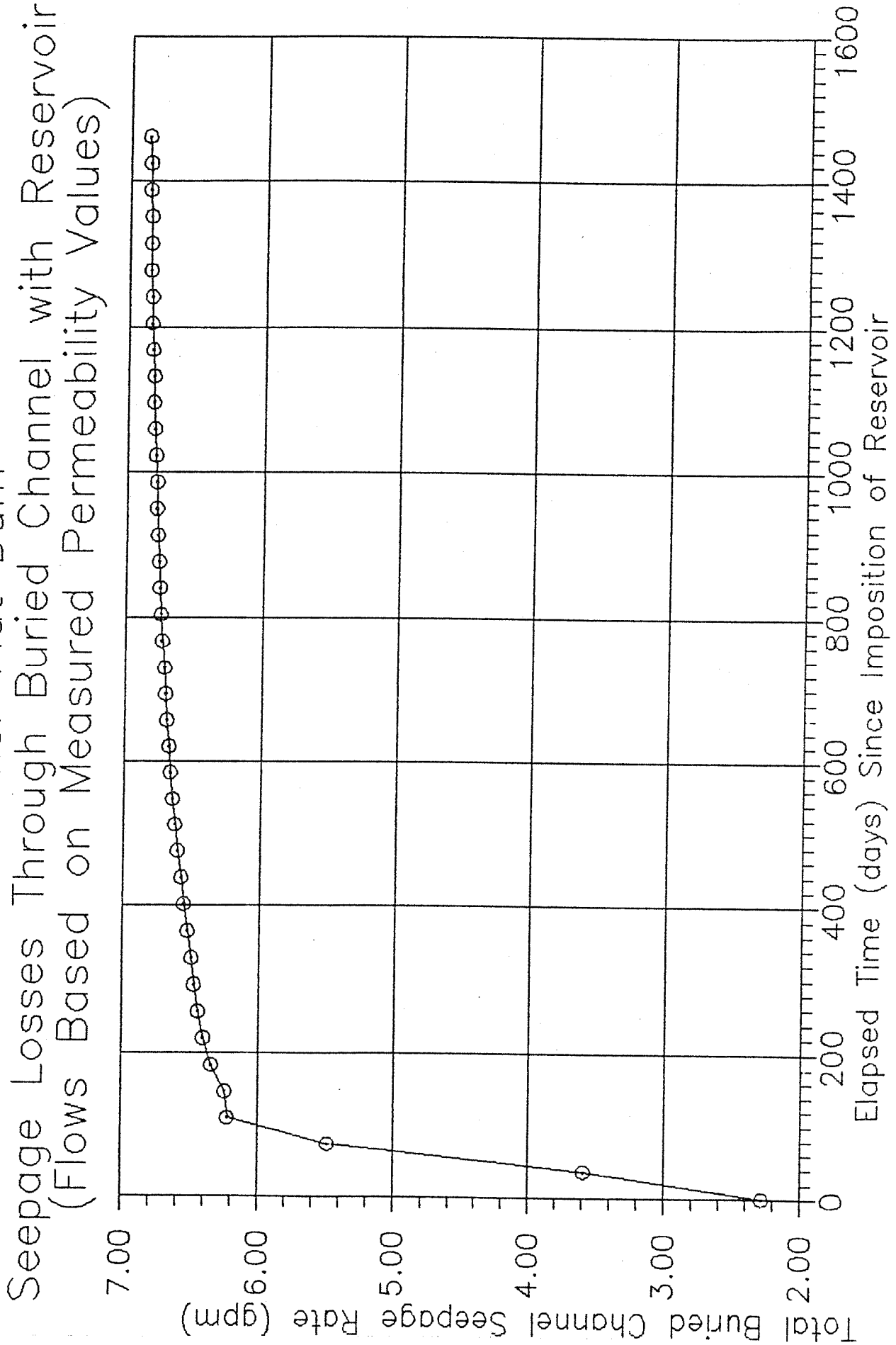


Fig. 1-2: Paleoalluvial seepage losses at field permeability.

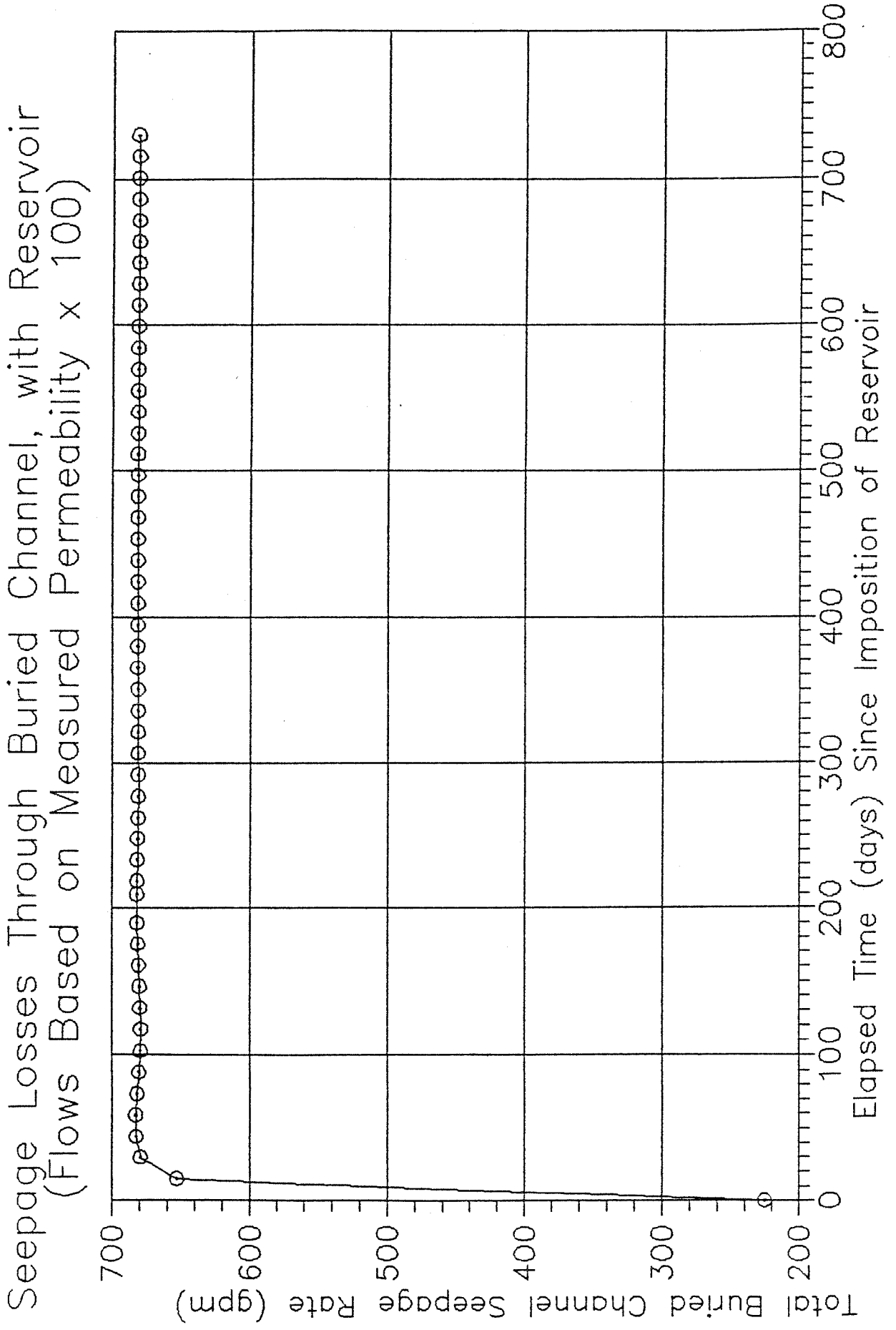


Fig. 1-3: Paleoalluvial seepage losses at 100 X permeability.

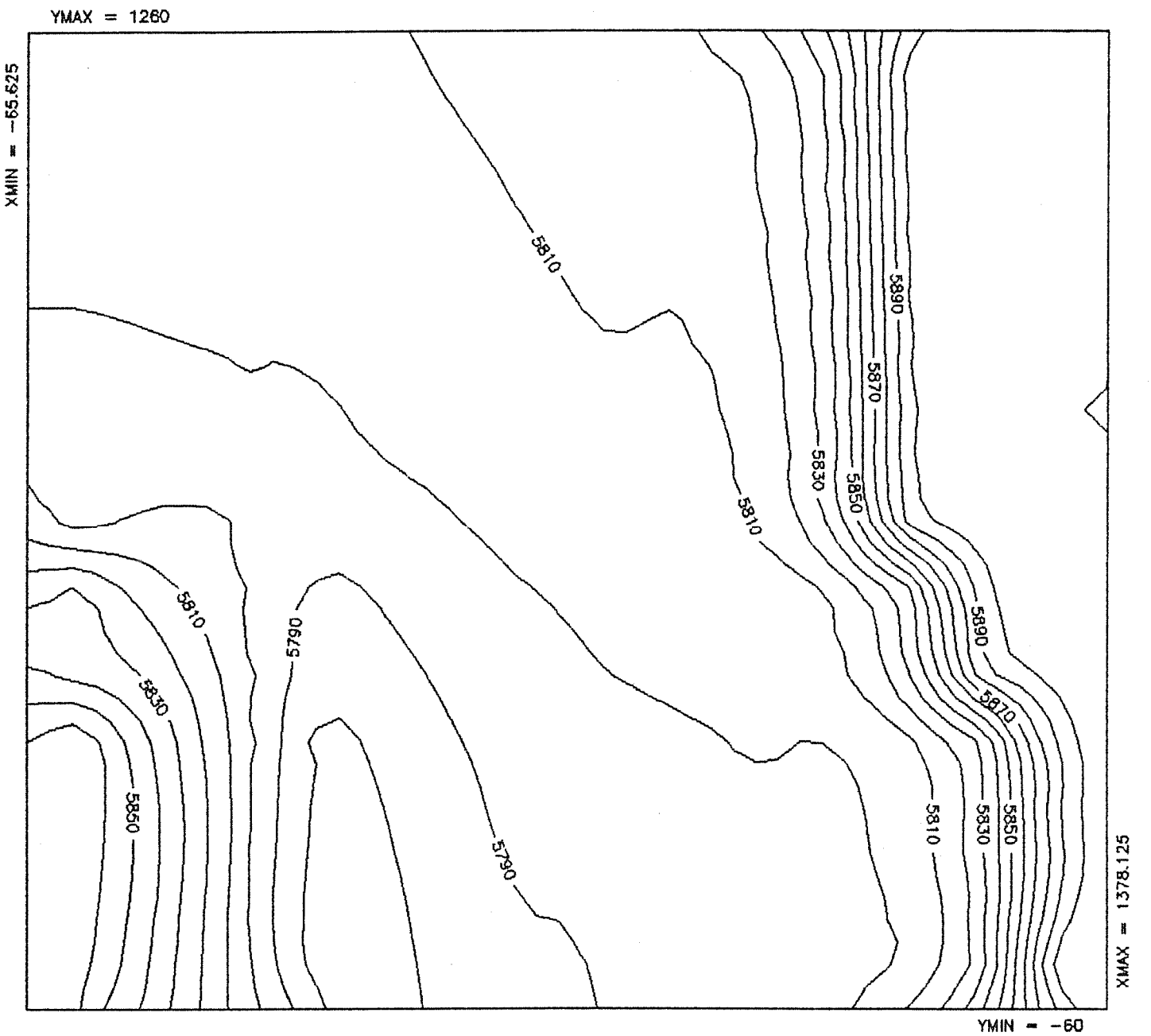
buried paleoalluvial channel will not be significant compared to the reservoir storage and the available water supply.

1.3.4. **Uplift Pressures**

The finite difference model used to simulate seepage flow through the paleoalluvial buried channel at the Miner Flat Dam provides the same information as provided by conventional flow net analysis. The model determines the pore pressure and seepage rates at known locations as a product of the permeability and the net head change from node to node in the model. Thus, the finite difference model was used as a flow net in the seepage analysis. However, whereas a conventional flow net analysis is conducted along a single vertical slice of the seepage flow path, the finite difference model provided an evaluation of the distribution of head and the associated seepage flow quantity on an areal basis takeing into consideration the effects of boundary conditions which would not appear on a simplistic line of section flow net. Accordingly, the finite difference modeling methods used, provided a better evaluation of uplift pressure distribution and seepage flow quantities than would be obtained by conventional flow net analysis.

Output from the Miner Flat Dam seepage flow model was utilized as input for a series of computer generated contour maps of pore pressure distribution and uplift pressure distribution related to steady state seepage in the paleoalluvial buried channel. The elevations on the surface of the buried paleoalluvium (i.e., the elevation of the base of the overlying basalt layer where present) as used in the seepage model are shown on Figure 1-4.

Computer generated maps on Figures 1-5 through 1-8 show the phreatic surface (potentiometric surface) in the buried channel



Upper Surface Elevation of Buried Gravel Channel

Fig. 1-4: Elevations on top of buried channel.

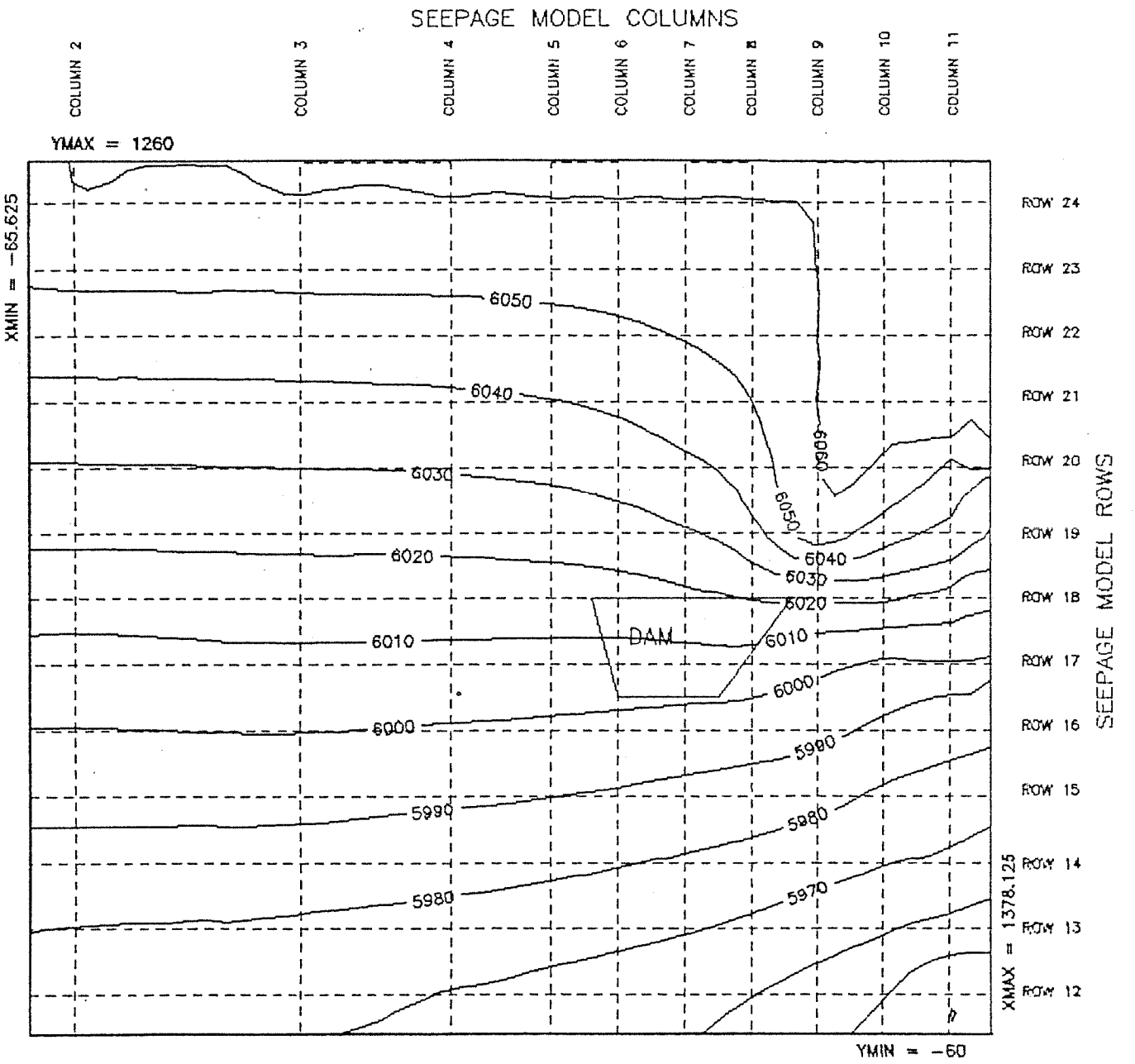


Fig. 5: Buried channel head (ft) at 1 year.

Fig. 1-5: Pore pressure elevation head after one year.

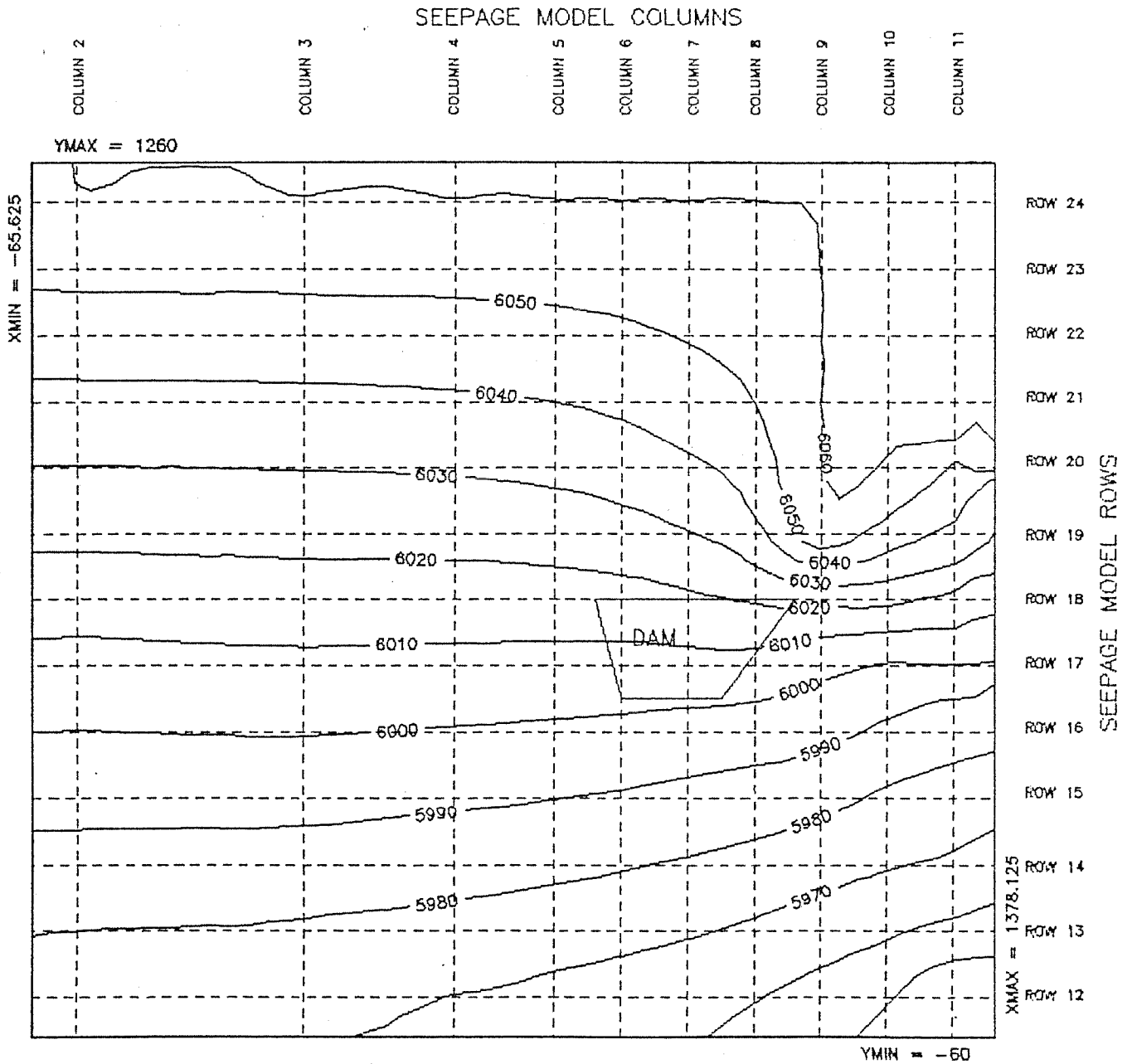


Fig. 6: Buried channel head (ft) at 2 years.

Fig. 1-6: Pore pressure elevation head after two years.

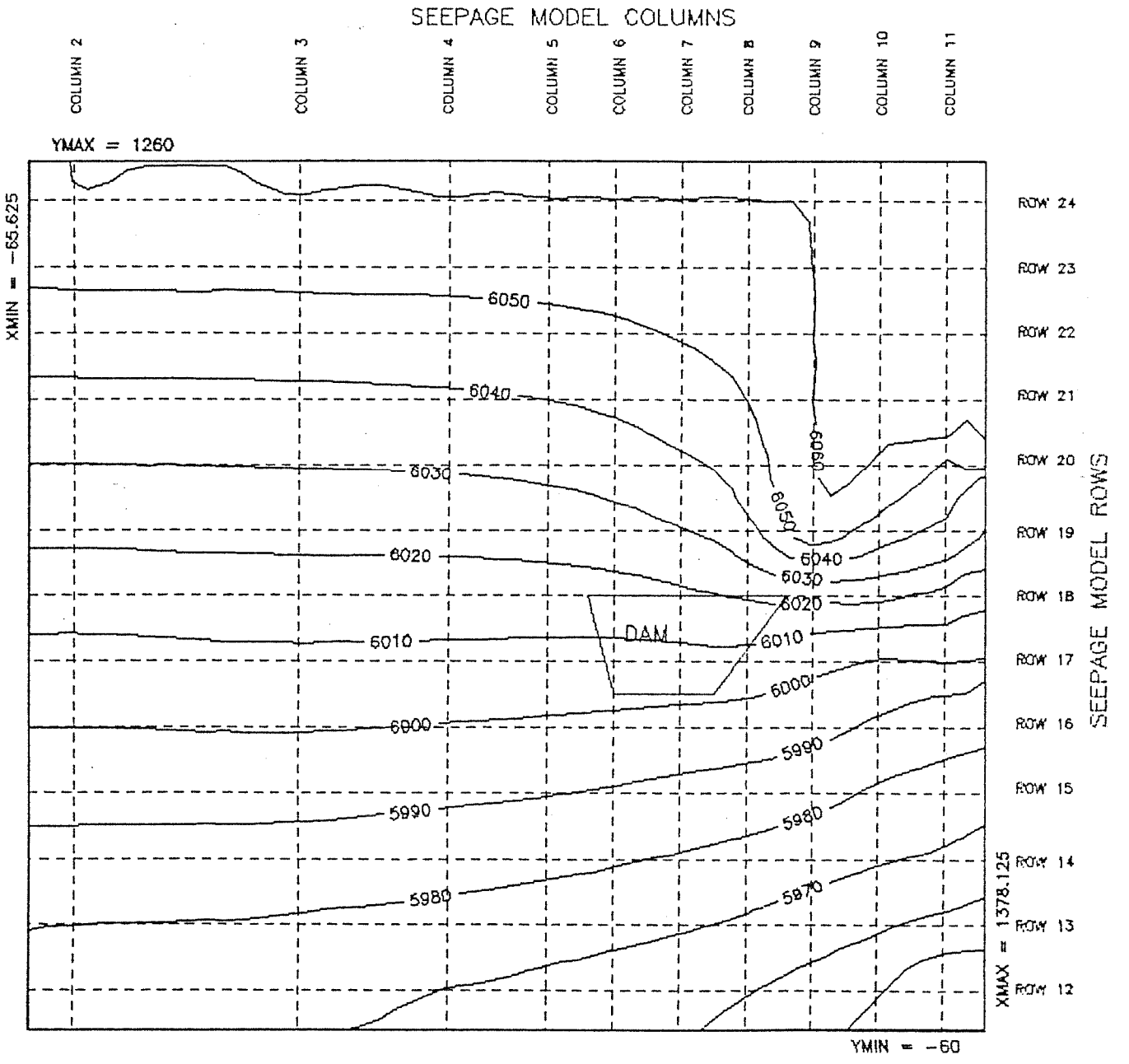


Fig. 7: Buried channel head (ft) at 3 years.

Fig. 1-7: Pore pressure elevation head after three years.

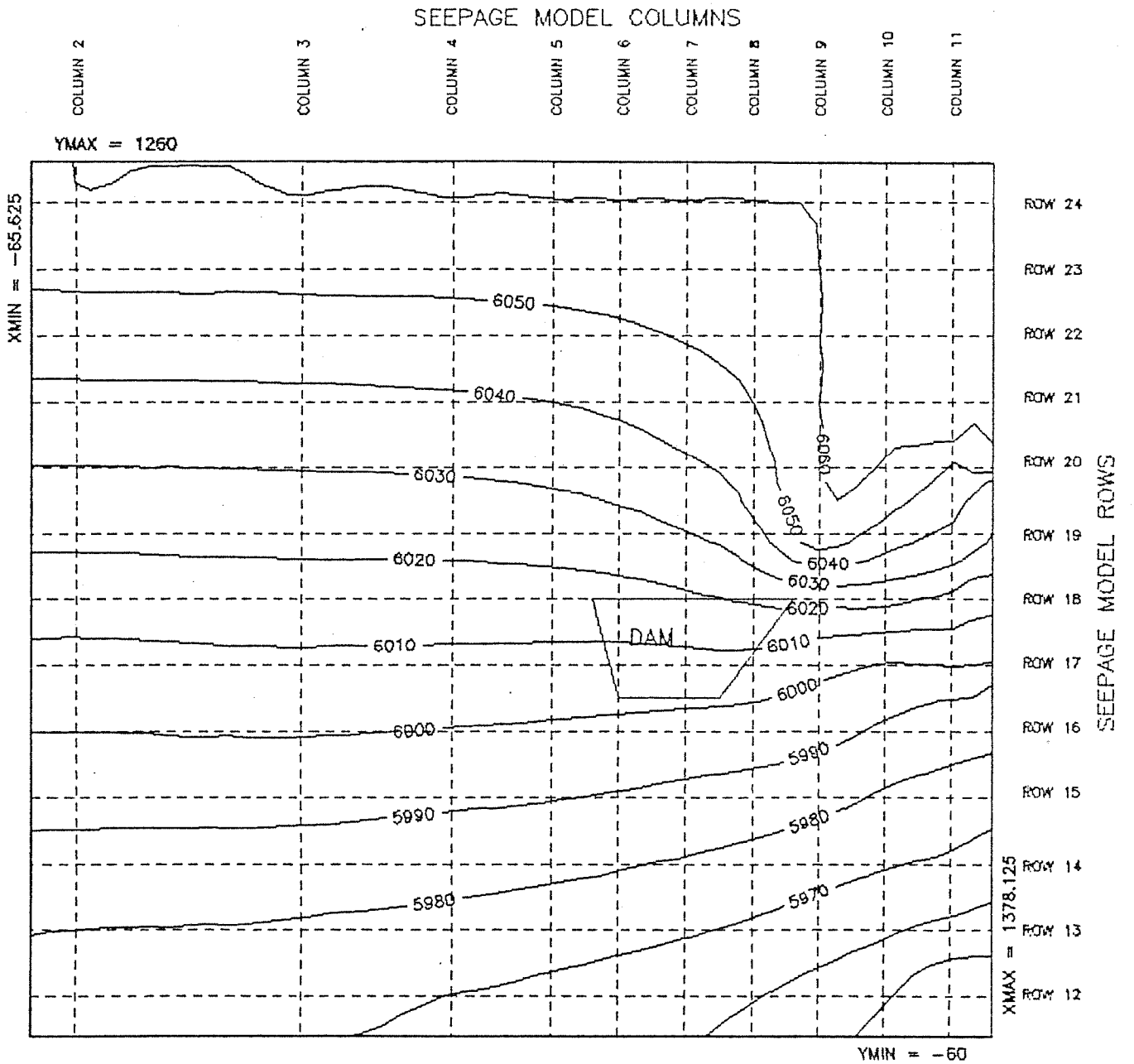


Fig. 8: Buried channel head (ft) at 4 years.

Fig. 1-8: Pore pressure elevation head after four years.

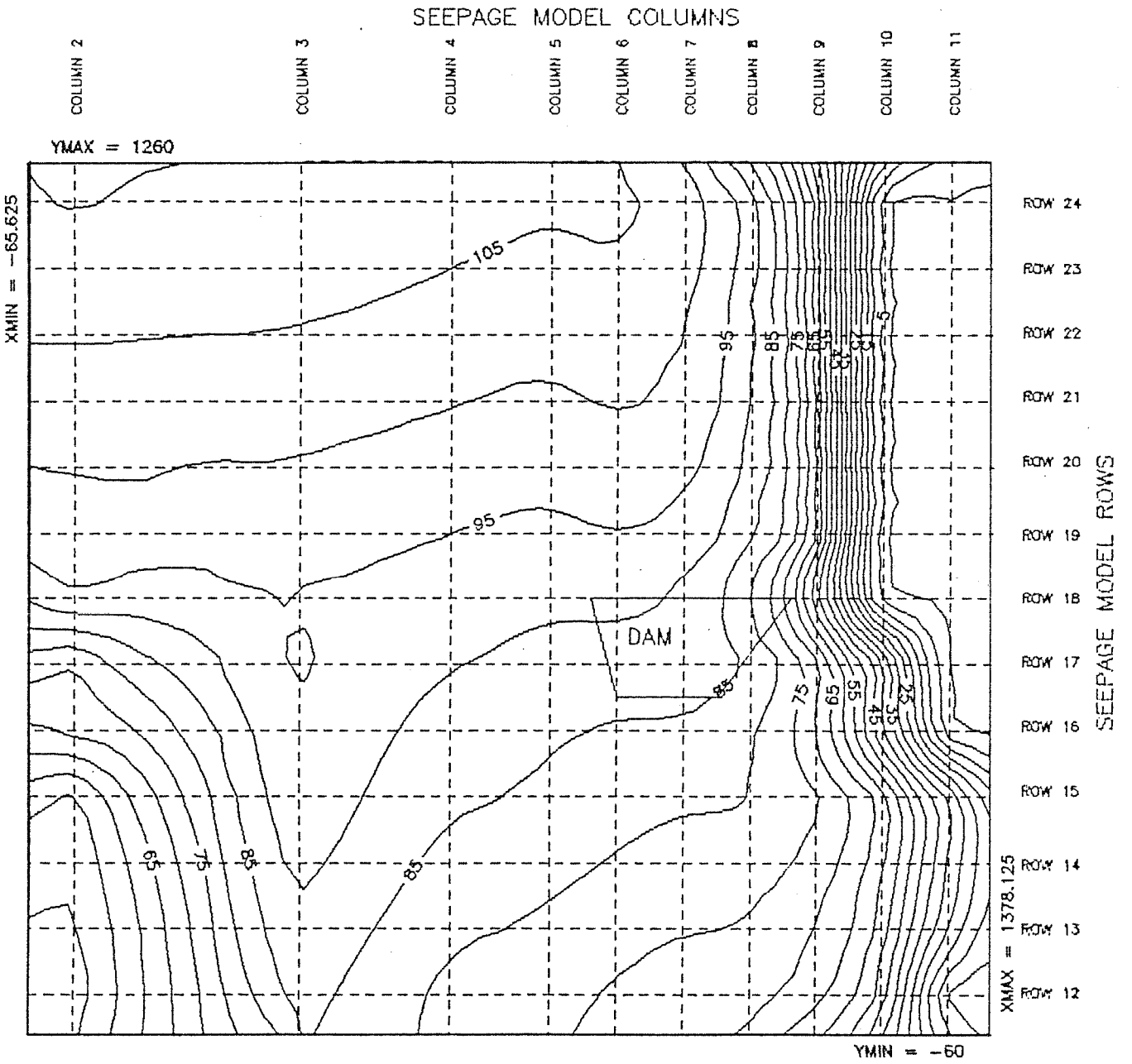
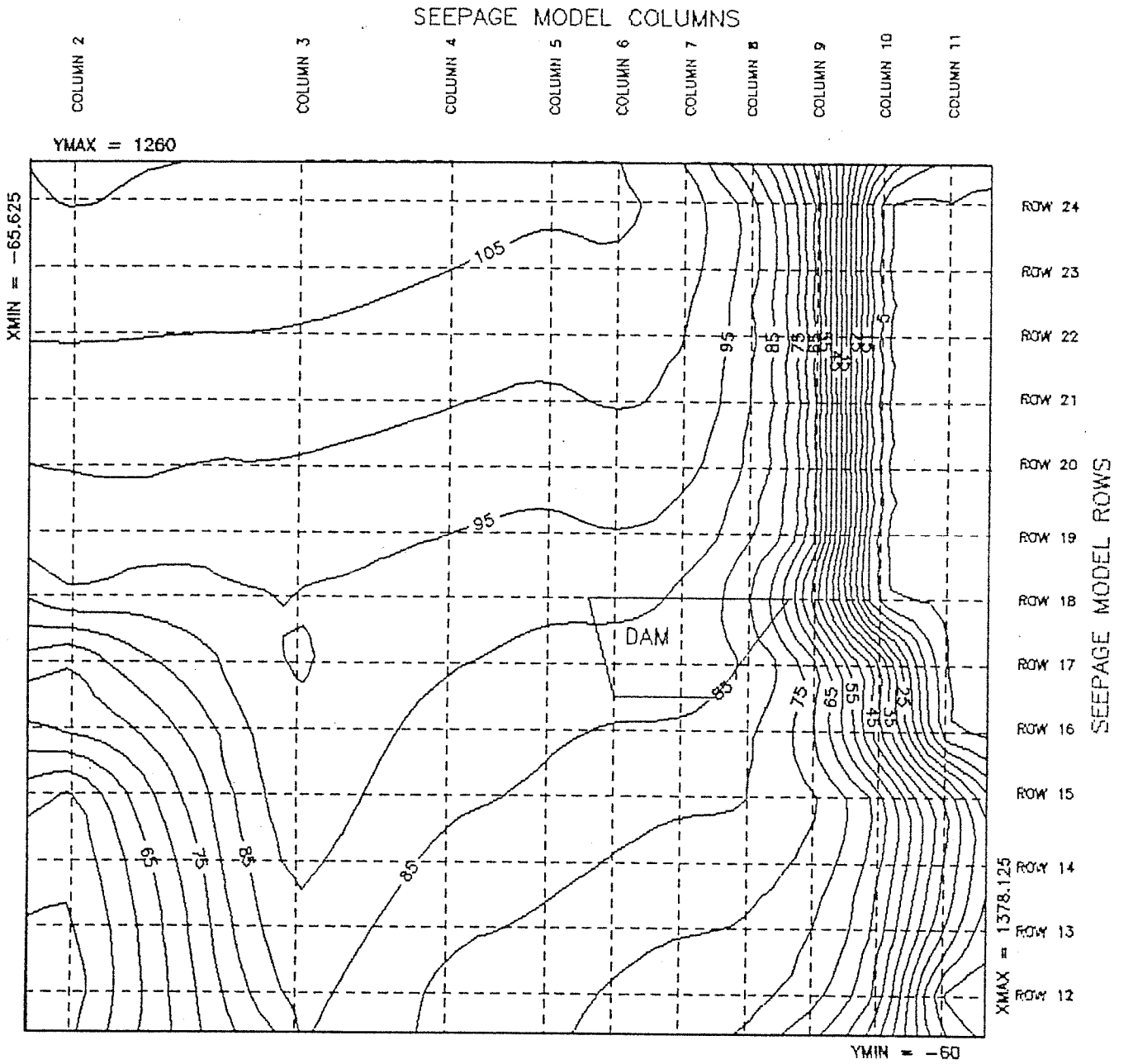


Fig. 9: Buried channel uplift pressure (psi) at 1 year.

Fig. 1-9: Uplift pressure distribution (psi) after one year.



g. 10: Buried channel uplift pressure (psi) at 2 years.

Fig. 1-10: Uplift pressure distribution (psi) after two years.

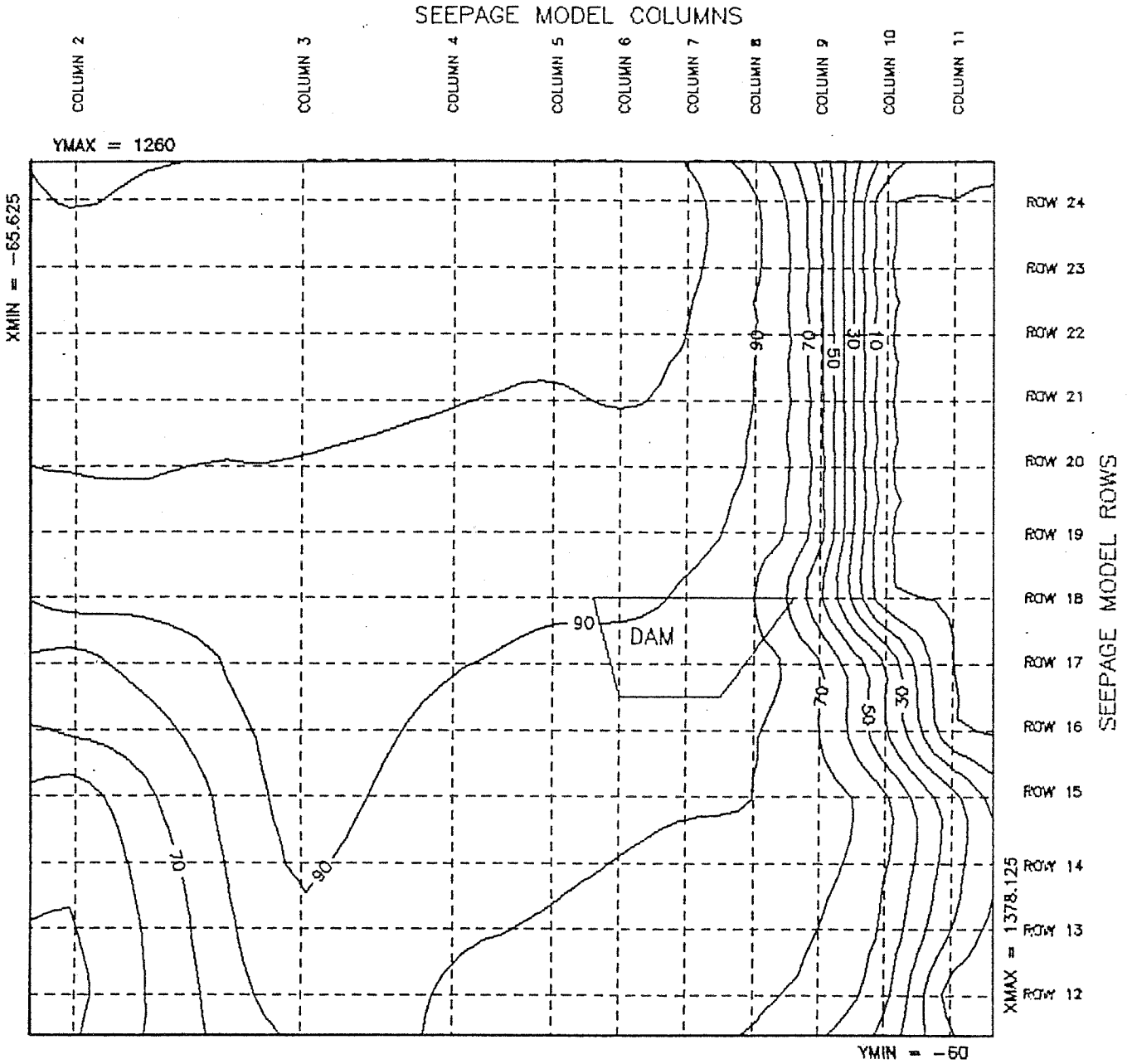


Fig. 11: Buried channel uplift pressure (psi) at 3 years.

Fig. 1-11: Uplift pressure distribution (psi) after three years.

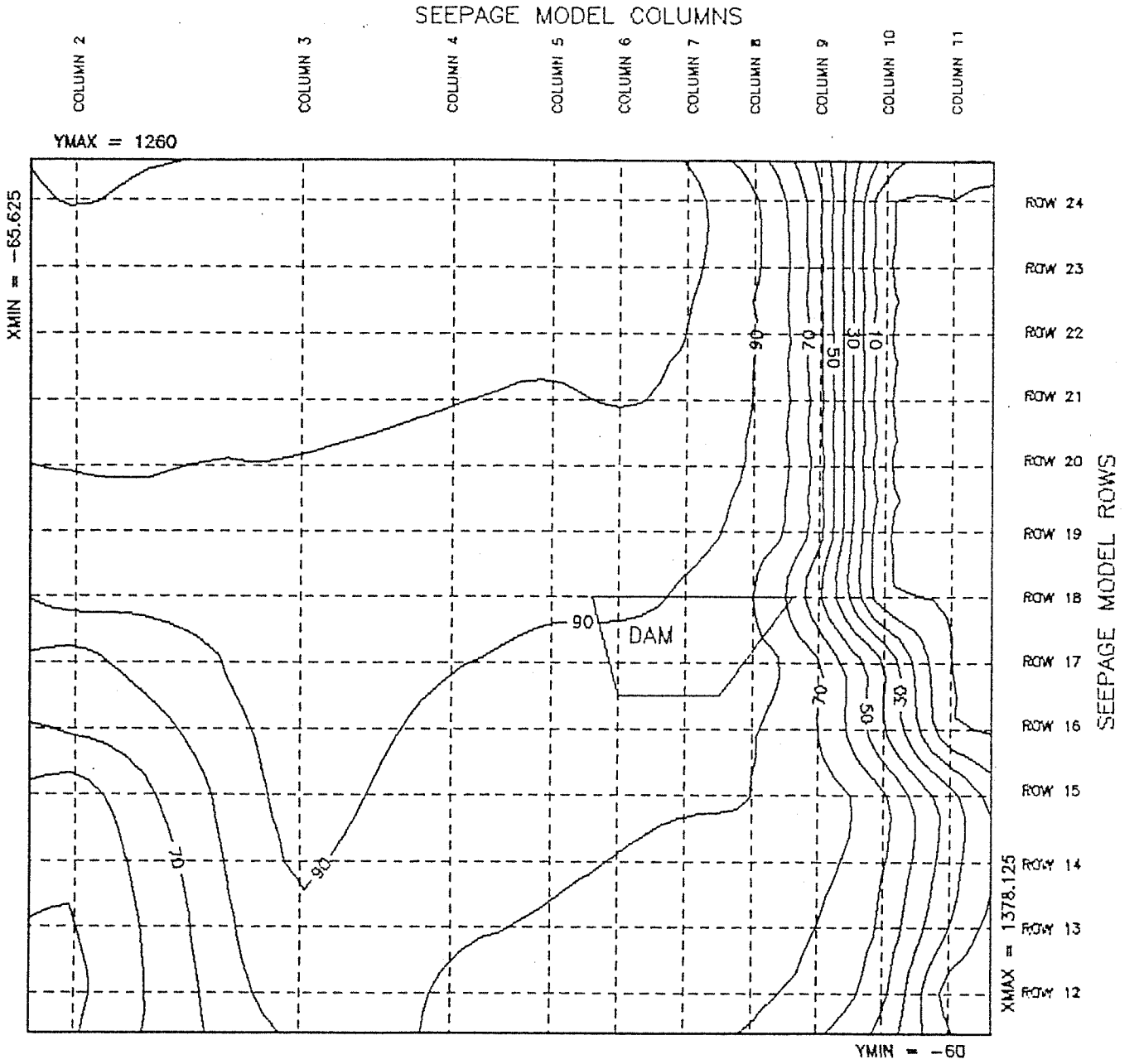


Fig. 12: Buried channel uplift pressure (psi) at 4 years.

Fig. 1-12: Uplift pressure distribution (psi) after four years.

at one, two, three, and four years of simulation of continuous seepage flow, respectively. Computer generated maps on Figures 1-9 through 1-12 show the distibution of uplift pressures in the buried channel as calculated from the elevation difference between the top of the buried channel (base of the overlying basalt) and the pore pressure head (potentiometric surface).

The uplift pressures indicated by the buried channel seepage model were compared to structural pressure by looking at specific nodes in the model that fall within the area of the foundation of the dam. Uplift pressure under the basalt in the floor of the river channel downstream from the dam was also compared to rock overburden pressure exerted by the basalt underlying the river channel at the downstream toe of the dam. The comparison of uplift pressure from the buried paleoalluvial channel and the associated paleocolluvium to the pressure load of the proposed concrete gravity dam and the basalt overlying the buried seepage paths is shown on Table 1-1. A net pressure profile from the upstream face of the dam to the downstream river channel is shown on Figure 1-13.

The data presentated on Table 1-1 are from model nodes that lie either under the dam or in the river channel downstream from the dam. Node (7,18) is located at the upstream face of the dam as are all of the foundation area nodes on model row 18. Several nodes are presented as fractional locations between nodes in the seepage model in order to conform to the locations of calculated structural pressures for the concrete gravity dam. Pore pressure data for the fractional locations are extrapolated from adjacent nodes in the model. The node at (7,16) is defined on Table 1-1 as being in the "apron" of the dam. The apron as used on Table 1-1 is an area of rock excavation in the basalt at the toe of the

MINER FLAT DAM FOUNDATION AND DOWNSTREAM RIVER CHANNEL

ROCK DENSITY = 200 PCF
WATER DENSITY = 62.4 PCF

	DAM NODES (X,Y)	CONFINING BASE ELEVATION (FEET)	FOUNDATION OR CHANNEL ELEVATION (FEET)	ROCK OVERBURDEN THICKNESS (FEET)	PHREATIC SURFACE ELEVATION (FEET)	STRUCTURAL BEARING PRESSURE (PSI)	ROCK OVERBURDEN PRESSURE (FEET)	TOTAL LOAD (PSI)
DAM	(7,18)	5812.00	5907	95.00	6017.84	100.7	131.9	232.6
DAM	(7,17.28)	5805.72	5907	101.28	6009.63	92.3	140.7	233.0
DAM	(7,17.22)	5805.20	5895	89.80	6008.95	96.7	124.7	221.4
DAM	(7,16.80)	5802.93	5885	82.07	6004.41	62.8	114.0	176.8
TOE	(7,16.375)	5802.20	5885	82.80	6000.10	130.3	115.0	245.3
APRON	(7,16)	5801.55	5895	93.45	5996.29	.0	129.8	129.8
CHANNEL	(7,15)	5800.84	5909	108.16	5987.24	.0	150.2	150.2
CHANNEL	(7,14)	5799.11	5907	107.89	5979.21	.0	149.8	149.8

	DAM NODES (X,Y)	SEEPAGE PRESSURE UPLIFT (PSI)	TOTAL LOAD (PSI)	NET UPLIFT "-" = LIFT "+" = LOAD (PSI)
DAM	(7,18)	-89.2	232.6	143.4
DAM	(7,17.28)	-88.4	233.0	144.6
DAM	(7,17.22)	-88.3	221.4	133.1
DAM	(7,16.80)	-87.3	176.8	89.5
TOE	(7,16.375)	-85.8	245.3	159.5
APRON	(7,16)	-84.4	129.8	45.4
CHANNEL	(7,15)	-80.8	150.2	69.4
CHANNEL	(7,14)	-78.0	149.8	71.8

Table 1-1: Net pressure distribution at 200 PCF basalt density.

dam and does not include a concrete structure to provide structural load or pressure. Structural pressure at the toe of the dam is greater than at other locations under the dam due to the rotational forces exerted on the dam under full hydrostatic and hydrodynamic load. The structural pressures are based on an assumed water elevation of 6,062 feet, no tailwater, and ice at the crest of the dam and are the minimum structural pressures for a full reservoir condition.

The rock density used on Table 1-1 represents the upper limit of a range of rock density initially assumed for the lower basalt layer at the dam site which forms the foundation of the dam and provides the confining layer over the buried seepage path and to which the seepage pressures are theoretically transmitted. The assumed rock density of 200 pounds per cubic foot (PCF) used on Table 1-1 was based on the office estimate of the density of a solid core of hard, non-vesicular basalt from the foundation area. The actual result of the approximate density measurement of the core was 204.5 PCF. Subsequent laboratory analysis of 21 samples of the lower basalt indicated a range of rock density of 173 to 181 PCF and an average rock density of 177 PCF for the 21 samples. The samples analysed in the laboratory included discontinuities in the rocks due to fractures and vesicles. The core hole logs from the foundation of the dam as presented in the series of geotechnical reports by Mineral Systems, Inc. indicate that the basalt at and below the foundation elevation is relatively massive and dense with no significant zones of vesicularity, contact scoria, or flow breccia to significantly decrease the density of the rock mass or provide significant discontinuities in the structural and hydraulic integrity of the mass. Accordingly, the average rock density of 177 PCF for the lower basalt

is a reasonable representation of the foundation rock density.

A separate analysis of the net pressure distribution at the foundation area and in the downstream river channel is shown on Table 1-2 and Figure 1-14. The purpose of the analysis shown on Table 1-2 and Figure 1-14 is to determine the hypothetical basalt density in the foundation area at which the combined basalt rock overburden load and structural pressure would equal the seepage uplift pressure at some location in the foundation area. As shown on Table 1-2 and Figure 1-14, the hypothetical density of the basalt in the foundation area must be reduced to 130 PCF before the combined weight of the structure and underlying basalt is diminished to the extent that the structural and overburden pressures are decreased to equal the seepage uplift pressure.

The hypothetical basalt density of 130 PCF, as compared to the measured density of 177 PCF, is required to decrease theoretical load pressure enough to equal the uplift pressure shown by the seepage model. This requires that the basalt density in the foundation be reduced to 73 percent of the density of the measured core, that is, a 27 percent reduction in basalt density. Inspection of the various core logs, inspection of the core, and evaluation of the geologic and geotechnical information regarding the massive basalt strata constituting the foundation rock at the dam site indicates that it is totally improbable and unreasonable to anticipate that discontinuities of any kind might exist in the foundation basalt to the extent that the rock density might be reduced by 27 percent. Accordingly, the conclusion that 177 PCF is a reasonable rock density for the foundation area includes a large factor of safety in regards to seepage uplift pressures in that the rock density of 177 PCF would have to be reduced to something less than 130 PCF before seepage uplift pressure would

MINER FLAT DAM FOUNDATION AND DOWNSTREAM RIVER CHANNEL

ROCK DENSITY = 130 PCF
WATER DENSITY = 62.4 PCF

	DAM NODES (X,Y)	CONFINING BASE ELEVATION (FEET)	FOUNDATION OR CHANNEL ELEVATION (FEET)	ROCK OVERBURDEN THICKNESS (FEET)	PHREATIC SURFACE ELEVATION (FEET)	STRUCTURAL BEARING PRESSURE (PSI)	ROCK OVERBURDEN PRESSURE (FEET)	TOTAL LOAD (PSI)
DAM	(7,18)	5812.00	5907	95.00	6017.84	100.7	85.8	186.5
DAM	(7,17.28)	5805.72	5907	101.28	6009.63	92.3	91.4	183.7
DAM	(7,17.22)	5805.20	5907	89.80	6008.95	96.7	81.1	177.8
DAM	(7,16.80)	5802.93	5895	82.07	6004.41	62.8	74.1	136.9
TOE	(7,16.375)	5802.20	5885	82.80	6000.10	130.3	74.8	205.1
APRON	(7,16)	5801.55	5895	93.45	5996.29	.0	84.4	84.4
CHANNEL	(7,15)	5800.84	5909	108.16	5987.24	.0	97.6	97.6
CHANNEL	(7,14)	5799.11	5907	107.89	5979.21	.0	97.4	97.4

	DAM NODES (X,Y)	SEEPAGE PRESSURE UPLIFT (PSI)	TOTAL LOAD (PSI)	NET UPLIFT "-" = LIFT "+" = LOAD (PSI)
DAM	(7,18)	-89.2	186.5	97.3
DAM	(7,17.28)	-88.4	183.7	95.4
DAM	(7,17.22)	-88.3	177.8	89.5
DAM	(7,16.80)	-87.3	136.9	49.6
TOE	(7,16.375)	-85.8	205.1	119.3
APRON	(7,16)	-84.4	84.4	.0
CHANNEL	(7,15)	-80.8	97.6	16.9
CHANNEL	(7,14)	-78.0	97.4	19.4

Table 1-2: Net pressure distribution at 130 PCF basalt densi

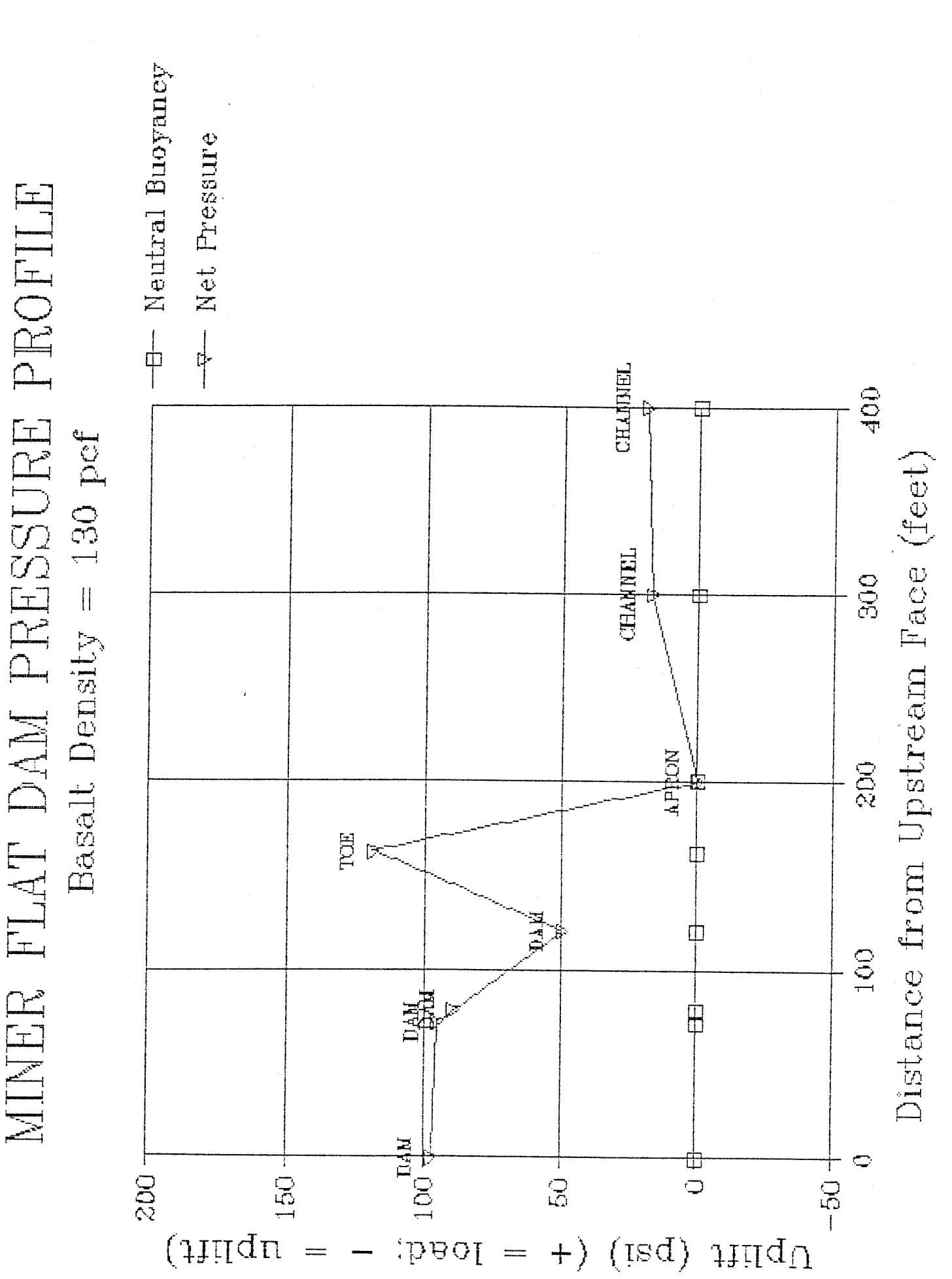


Fig. 1-14: Net pressure distribution profile at 130 PCF basalt density

exceed the load pressure exerted by the combination of the structure and the confining basalt.

A worst case assumption is shown on Table 1-3 and Figure 1-15 where it is assumed that the confined seepage pressure is transmitted directly to the base of the impermeable concrete gravity dam without exerting any pressure against the basalt overlying the buried paleoalluvium and paleocolluvium. This is, of course, a very unrealistic assumption in that it reduces the effective density of the rock in the foundation to zero pounds per cubic foot. The only way this could occur physically would be for the basalt permeability to be such that it was not confining the seepage in the buried pre-basalt channel. If this were the case, the pore pressures profile in the foundation would have a steeper gradient and decrease more rapidly from the upstream face to the toe of the dam than in the present model.

Accordingly, the worst case assumption presented on Table 1-3 and Figure 1-15 represents unrealistically high seepage uplift pressure conditions. Even so, when all of the seepage uplift pressure is transmitted to the concrete gravity dam with no rock overburden pressure, there is only a portion of the upstream to downstream cross section of the dam where seepage pressures from the model exceed the structural pressure of the dam. In other words, the mass of the dam taken as a whole and expressed as structural pressure, continues to exceed the seepage uplift pressure even when the foundation rock density is equated to zero density.

MINER FLAT DAM FOUNDATION AND DOWNSTREAM RIVER CHANNEL

ROCK DENSITY = 200 PCF
WATER DENSITY = 62.4 PCF

	DAM NODES (X,Y)	CONFINING BASE ELEVATION (FEET)	FOUNDATION OR CHANNEL ELEVATION (FEET)	ROCK OVERBURDEN THICKNESS (FEET)	PHREATIC SURFACE ELEVATION (FEET)	STRUCTURAL BEARING PRESSURE (PSI)	ROCK OVERBURDEN PRESSURE (FEET)	TOTAL LOAD (PSI)
DAM	(7,18)	5812.00	5907	95.00	6017.84	100.7	131.9	232.6
DAM	(7,17.28)	5805.72	5907	101.28	6009.63	92.3	140.7	233.0
DAM	(7,17.22)	5805.20	5895	89.80	6008.95	96.7	124.7	221.4
DAM	(7,16.80)	5802.93	5885	82.07	6004.41	62.8	114.0	176.8
TOE	(7,16.375)	5802.20	5885	82.80	6000.10	130.3	115.0	245.3
APRON	(7,16)	5801.55	5895	93.45	5996.29	.0	129.8	129.8
CHANNEL	(7,15)	5800.84	5909	108.16	5987.24	.0	150.2	150.2
CHANNEL	(7,14)	5799.11	5907	107.89	5979.21	.0	149.8	149.8

	DAM NODES (X,Y)	SEEPAGE PRESSURE UPLIFT (PSI)	TOTAL LOAD (PSI)	NET UPLIFT "-" = LIFT "+" = LOAD (PSI)	SEEPAGE PRESSURE UPLIFT (PSI)	STRUCTURAL BEARING PRESSURE (PSI)	NET UPLIFT "-" = LIFT "+" = LOAD (PSI)
DAM	(7,18)	-89.2	232.6	143.4	-89.2	100.7	11.5
DAM	(7,17.28)	-88.4	233.0	144.6	-88.4	92.3	3.9
DAM	(7,17.22)	-88.3	221.4	133.1	-88.3	96.7	8.4
DAM	(7,16.80)	-87.3	176.8	89.5	-87.3	62.8	-24.5
TOE	(7,16.375)	-85.8	245.3	159.5	-85.8	130.3	44.5
APRON	(7,16)	-84.4	129.8	45.4	-84.4	.0	(N/A)
CHANNEL	(7,15)	-80.8	150.2	69.4	-80.8	.0	(N/A)
CHANNEL	(7,14)	-78.0	149.8	71.8	-78.0	.0	(N/A)

Table 1-3: Net pressure distribution at zero PCF basalt density.

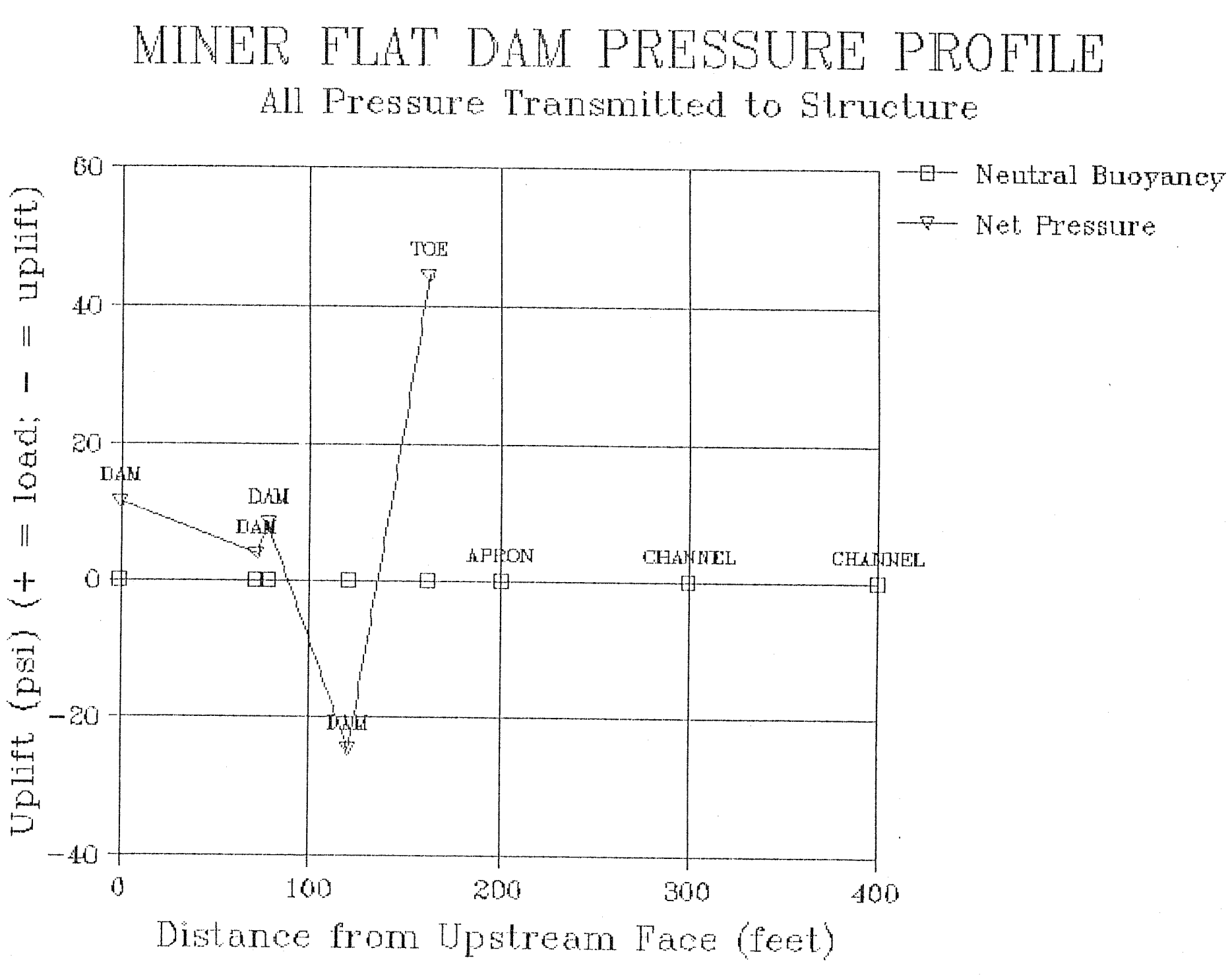


Fig. 1-15: Net pressure distribution profile at zero PCF basalt density.

1.3.5. Conclusions

The 288 node finite difference model used to simulate seepage quantities and pore pressure in the buried pre-basalt paleoalluvial channel at the proposed Miner Flat Dam site indicates that seepage losses through a confined seepage path of the geometry indicated by the geotechnical field investigations would be less than seven gallons per minute or 11.3 acre-feet per year at the measured hydraulic conductivity of a magnitude of 1.00×10^{-4} cm/sec.

Seepage losses calculated by the model are directly proportional to the hydraulic conductivity measured in the field. If the values of hydraulic conductivity measured in the field were erroneously low due to core hole wall damage, plugging of the paleoalluvial interstices by core hole fluid invasion, or other factors adversely affecting the field tests, the seepage quantity calculated by the seepage model would be reduced accordingly. Simulation of seepage flow quantities using a presumed value of hydraulic conductivity 100 times greater than the values measured in the field results in seepage quantities approaching 700 gpm or 1,129 acre-feet per year.

The finite difference seepage model is essentially a three-dimensional flow net analysis and provides the same type of information as a conventional flow net analysis, i.e., distribution of uplift pressure and seepage flow quantity based on differences in head (net head) from one location to another. Seepage uplift pressures determined by flow net analysis, whether it be by conventional two-dimensional flow net construction along a line of section or by finite difference digital modeling, are dependent only upon the shape of the flow net and the distribution of equipotential lines. The permeability assigned to a flow

net or digital model affects only the quantity of seepage flowing through the model but does not affect the shape of the flow net or the distribution of the pressures within the flow net.

Therefore, the uplift pressures generated by the seepage flow model using the field measurements of hydraulic conductivity at 1.00×10^{-4} cm/sec are identical to those using a presumed hydraulic conductivity of any reasonable range of values desired. The generation of calculated uplift pressures in the model is not influenced by the selection of a value of hydraulic conductivity.

Uplift pressures due to seepage in the buried pre-basalt paleoalluvial and paleocolluvial deposits as determined by application of the seepage flow model do not exceed the reasonable range of foundation overburden and structural pressures calculated for the foundation area and downstream river channel. The analysis shows that if the overburden pressure of foundation rock confining the buried pre-basalt seepage path is reduced to zero, there would be a localized zone of buoyancy under part of the concrete gravity dam structure at full reservoir with no tailwater, but that the overall mass of the structure would still exert a positive net downward force in excess of the uplift pressures. The analysis supports the conclusion that potential seepage pressures in the buried pre-basalt materials do not adversely affect the feasibility of the proposed dam site.

1.4. ABUTMENTS SEEPAGE MODEL

Essentially the same finite difference modeling techniques were used to simulate seepage flows through the abutments of the Miner Flat Dam site as were used to simulate seepage flows through the buried paleoalluvial channel. The nonuniform grid delineating the node positions for the abutments seepage model is shown on Figure 1-16. Dimensions for the node array as shown on

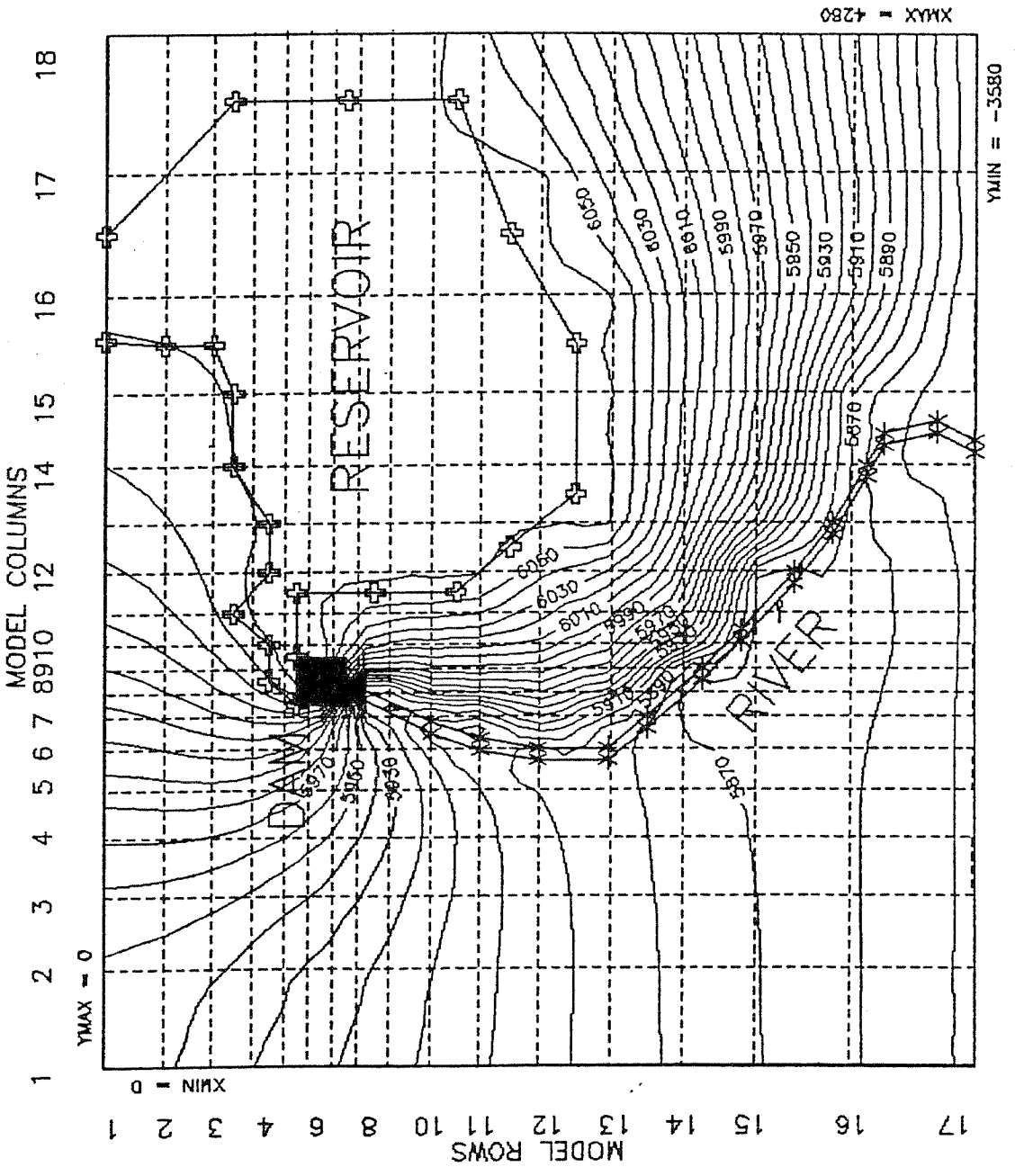


FIGURE 1-16: ABUTMENT SEEPAGE SURFACE

Figure 1-16 are as follows:

Coordinate Interval	X Coordinate Spacings (feet)	Y Coordinate Spacings (feet)
1 - 2	400	240
2 - 3	300	200
3 - 4	240	170
4 - 5	200	130
5 - 6	170	100
6 - 7	130	100
7 - 8	100	100
8 - 9	100	130
9 -10	100	170
10 -11	130	200
11 -12	170	240
12 -13	200	300
13 -14	240	300
14 -15	300	300
15 -16	400	400
16 -17	500	500
17 -18	600	---
Totals	4,280	3,580

Initial water levels in the abutments seepage model were set at the elevation of the water surface of the White River where the river crossed each row of nodes except in the reservoir area where a water surface elevation of 6,062 feet was used. Aquifer bottom elevations were assumed to be 50 feet lower than the initial water level. Water levels in the sink nodes used to represent the river downstream from the dam were set two feet or more deeper than the river elevation at each row of nodes. Accordingly, flow in the model was always to the sinks representing the river.

No-flow boundary conditions were automatically established by the program code for all exterior boundaries of the model. Hydraulic conductivity values for the different geologic materials were the average values presented in the Mineral Systems Inc. report. Although the program code for the model was written to select confined or unconfined storativity values as appropriate, all of the seepage flow through the abutments took place under unconfined flow conditions with the top of the seepage zone

or aquifer being set equal to the land surface.

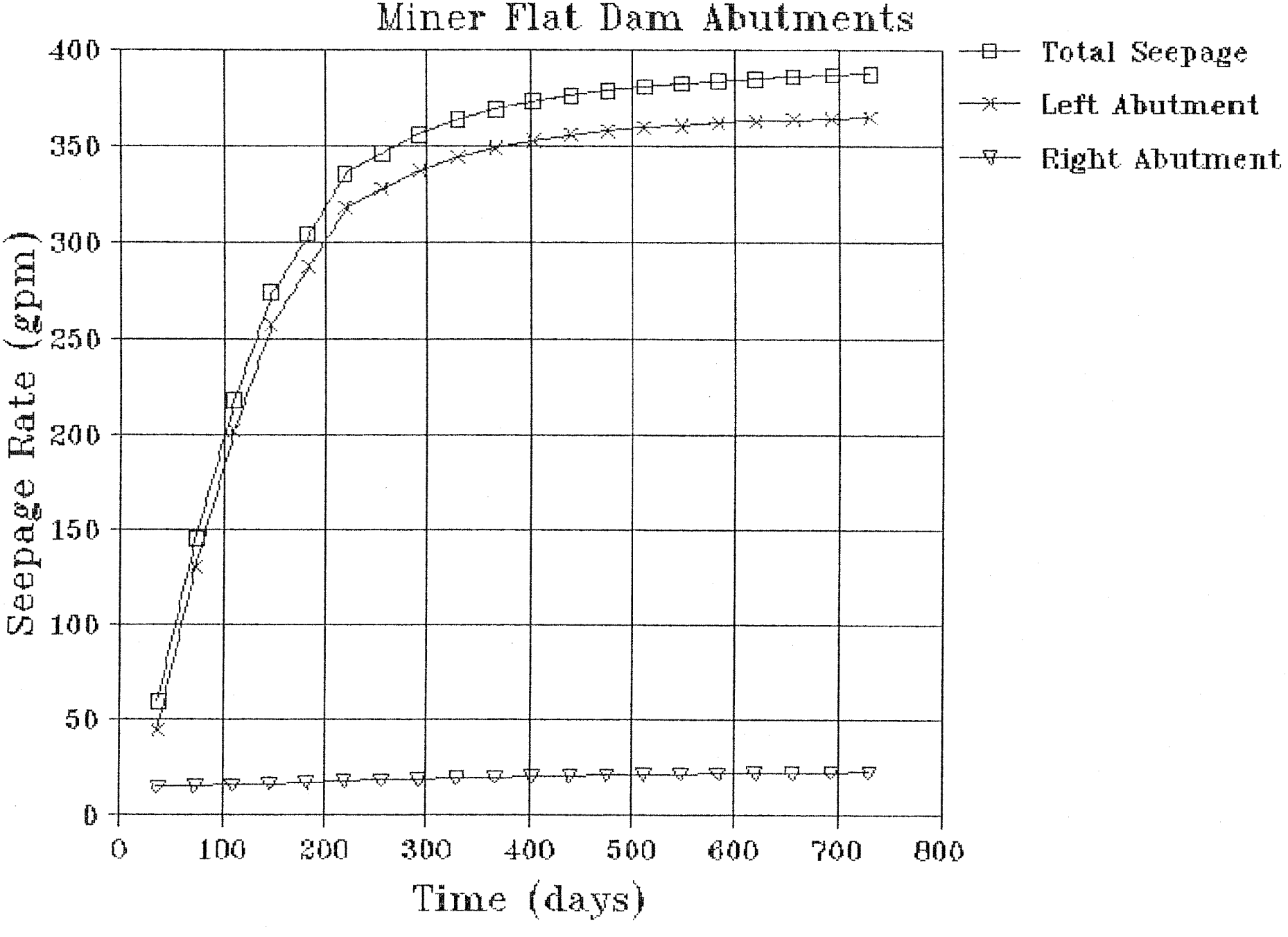
Values of hydraulic conductivity for the various types of geologic materials present in the abutments were obtained from the Mineral Systems Inc. report. Values of storativity for the unconfined conditions were assumed. Values of hydraulic conductivity and storativity used in the abutments seepage model are summarized on Table 1-4.

Table 1-4: Abutments seepage model aquifer parameters.

Geologic Material	Hydraulic Conductivity (gpd/ft^2)	Coefficient of Storage
Paleoalluvium	3.6050	0.15
Supai Formation	7.9734	0.10
Basalt	3.3930	0.05

Operation of the abutments seepage model from initial conditions consisted of instantaneous imposition of the full reservoir upon the steady state conditions prevailing during pre-Miner Flat Dam Conditions. The lessening of the rate of increase in the seepage flows from the reservoir through the right abutment shown on Figure 1-17 indicates that essentially steady state seepage flows were in effect through the left abutment after 438 days of a two-year long flow simulation of seepage from full reservoir conditions. In view of the fact that it is not anticipated that the reservoir will remain at full storage for continuous periods of one year or more, a duration of 365 days of continuously full storage in the reservoir was selected for modeling seepage through the abutments.

FIG. 1-17: SEEPAGE RATE



The rate of seepage for the right and left abutments is shown on Table 1-5 in terms of 36.5-day increments. The increasing rates of seepage shown on Table 1-5 correspond to the seepage rate curves shown on Figure 1-17 and reflect the buildup of seepage from the time of instantaneous imposition of the reservoir onto the unsaturated reservoir site to seepage conditions at the end of 365 days of continuously full reservoir conditions.

Table 1-5: Abutment seepage rates.

Elapsed Time (days)	Right Abutment Seepage Rate (gpm)	Left Abutment Seepage Rate (gpm)	Combined Abutment Seepage Rate (gpm)
36.5	14.83	44.73	59.56
73.0	15.12	130.50	145.62
109.5	15.69	202.38	218.07
146.0	16.38	257.64	274.02
182.5	17.07	287.31	304.38
219.0	17.72	317.78	335.50
255.5	18.32	327.45	345.78
292.0	18.87	337.07	355.94
328.5	19.36	344.03	363.39
365.0	19.81	349.02	368.83

As shown on Table 1-5, the combined seepage rate from the left and right abutments of the dam site at the end of one year is about 369 gpm or about 4,450 acre-feet per year, assuming a full reservoir for the entire year. In view of the predicted annual spills of 50,000 to 60,000 acre-feet over the spillway, the simulated seepage loss of 4,450 acre-feet per year is negligable.

A number of factors cause the simulation of seepage flows in the abutments of the dam site to be conservatively large. One factor is the fact that it was assumed that the reservoir was full to the maximum storage elevation of 6,062 feet for the entire year of seepage simulation. In reality, the storage pool will be below an elevation of 6,062 feet much of the time. A

second factor is the no-flow external boundary conditions in the model. For example, seepage flowing from the reservoir towards column 18 in the model (Figure 1-16) encounters a barrier to flow before an equilibrium gradient is established. This forces the water to flow towards row 17 in the model, thus creating higher seepage surface elevations and more seepage discharge to the river than would occur without the presence of the no-flow boundary adjacent to column 18. Similar increases in seepage surface elevations and seepage flow occur in the right abutment due to the presence of no-flow boundaries at the top and left side of the seepage model.

A final factor increasing the rate of seepage flow to the river downstream from the dam is that the sink nodes representing the river in the model are set at elevations much lower than the actual river elevation in a number of instances to equalize initial heads in the sinks and prevent internodal transfers of water between sink nodes. Consequently, the differential head between the sink nodes and the adjacent abutment nodes is greater in the model than the actual differential head would be between the river and the abutment rocks in reality. The greater differential head in the model results in greater seepage flows to the sinks representing the river. Accordingly, the rates of simulated seepage flows through the abutments of the dam site are conservatively large due to several factors.

1.5. SUMMARY

The seepage models demonstrate that uplift pressures predicted in the buried paleoalluvial channel under the basalt rock foundation of the dam will not exceed the overall structural and rock overburden pressures exerted by the dam and basalt overlying the channel. If it is conservatively assumed that all

of the uplift pressure from the buried channel is transmitted directly to the dam without any confining pressure being exerted by the rock overburden in the foundation, a small area is identified in the footprint area of the dam where uplift pressures exceed the structural pressure; however, the overall or net structural pressure of the dam is still in excess of the net uplift pressure, even when the weight of the rock in the dam foundation is ignored.

Seepage flow losses through the buried paleoalluvial channel were approximately 11.3 acre-feet per year in a four-year simulation of seepage flow assuming full reservoir conditions. Four-years of seepage simulation were required to obtain steady-state flow conditions in the buried channel starting from initial steady state flow conditions related to the river elevation without the reservoir. Accordingly, the assumption of a continuously full reservoir for a period of four years provides a conservatively high estimate of potential seepage losses in the buried channel within the order of magnitude of hydraulic conductivities measured in the paleoalluvial materials in the field.

Field measured values of hydraulic conductivity in the buried channel materials were arbitrarily assumed to be 100 times more permeable than indicated by the field measurements. The result of this assumtion was to increase the annual seepage losses simulated by the model to 1,129 acre-feet per year. Seepage uplift pressures in the buried channel did not change as a result of the assumed increase in hydraulic conductivity.

Flow simulations of seepage through the rocks above river level elevation in the abutments of the dam site were likewise conservatively large and resulted in prediction of about 369 gpm or 4,450 acre-feet per year of total seepage loss through the

right and left abutments. Combined seepage losses predicted for the buried channel and the abutments totaled 5,579 acre-feet per year under the worst case assumptions used in the seepage simulations. In comparison, surface water hydrologic analysis of the dam and reservoir site predicts that annual spills at the site will average in the range of 50,000 to 60,000 acre-feet per year. This means that the excess surface water supply which the dam and reservoir do not have enough capacity to regulate will far exceed the conservatively worst case seepage losses indicated by the models.

REFERENCES

Prickett, T.A., and Lonnquist, C.G., 1971, Selected digital computer techniques for groundwater resource evaluation: Illinois State Water Survey Bulletin 55, 62 p.

DESIGN MEMORANDUM

SECTION E

E.1 CONCRETE AGGREGATE

Adequate quantities of suitable concrete aggregate are available at the dam site for the construction of the envisioned concrete gravity dam. The potential aggregate sources are:

* Approximately 200,000 cubic yards of gravel deposits immediately upstream from the dam

* Approximately 135,000 cubic yards of basalt excavated from the dam foundation.

These quantities far exceed the total concrete volumes of 141,040 cubic yards for the conventional concrete dam and 127,645 cubic yards for the rolled concrete dam. As indicated in Section J.4 for the roller compacted concrete option, 50-70% of the aggregate will come from the basalt foundation excavation and 30-50% from the gravel source.

The source area for the gravel is a river terrace deposit located immediately upstream of the dam on the west bank of the river and is accessed by the existing drill access road from State Highway 73 as shown on the "General Map B", Sheet A2. The basalt available from the foundation excavation is accessed from upstream and will be processed, stockpiled and excess disposed in the gravel borrow area indicated above.

Photographs of the gravel borrow and rock disposal area are included in Section D, Figure D1.3. As visible in the photographs, the area is timbered and will require clearing prior to use.

Quantity assessment and preliminary material testing for the aggregate is included in the following attached report and memos:

* "Preliminary Report, Engineering Geology, Miner Flat Dam Site, White Mountain Apache Reservation", Mineral Systems, Inc., November 1983

* Memorandum, Aggregate Test Pit & Petrographic Analysis for E. Schrader, December 8, 1986.

* Memorandum, Results of Alkali Reactivity & L.A. Abrasion Tests, MF-102, December 5, 1986.

At this point, because the decision between conventional mass concrete and roller compacted concrete (RCC) has yet to be made and the high degree of confidence in the adequacy of available

aggregate for either technique, detailed mix designs for both options have not been prepared. It is intended that the final mix designs be prepared early in the project as indicated in Section J.4.

E.2 COMMERCIAL CONCRETE PLANTS

The dam site is located in a rural setting. The availability of existing commercial concrete plants is limited to the "Ready Mix" variety used for residential and light commercial structures.

The four plants within a hauling distance of approximately 30 miles from the dam site were surveyed and the results are presented below:

OPERATOR	DIST (mi)	FIXED BATCH PLANT	PORT. BATCH PLANT	NO. TRUCKS	OWN AGG. SOURCE	AGG. CRUSH/ SCREEN PLANT	AGG. WASH PLANT
Fort Apache Mat. Whiteriver, AZ	8	1	0	4	yes*	yes*	yes*
Show Low Rd. Mx. Show Low, AZ	30	2	1	9	no	no	no
A&A Ready Mix Show Low, AZ	30	1	1	5	no	no	no
Pierce Const. Show Low, AZ	30	1	0	2	no	yes	no

* Located 20 miles from batch plant & 12 miles from dam site.

Figure E.2.1 "Summary of Commercial Concrete Plants"

E.3 ACCESS ROADS

Site access is presently difficult with the existing exploration drill roads affording the only vehicle and equipment access. These roads are indicated on the "General Map B", Sheet A2. Good access is available to the right abutment rim from the adjacent State Highway 73. Access to the aggregate source area, upstream diversion area and left abutment will require upgrading the existing drill road from the right abutment and constructing at least one work bridge over the river.

Upgrading the access road from the right abutment rim to the aggregate source area will require both alignment and width alterations. The present road alignment is suitable for exploration equipment and light vehicles but a tight, steep corner included in the road at the top the first section above the aggregate source area must be removed.

The bridge will access the existing drill road from river grade upstream of the dam to the rim of the left abutment. This road, constructed primarily through Supai Formation sandstone, is currently subject to sloughing of both cut and fill slopes and must be upgraded for construction use. The alignment is suitable for construction equipment but widening, drainage improvements and slope stabilization will be required. Surfacing may be required for all weather use.

Other crossings may be required to access the base of the abutments from the upstream direction. At present, river fords are used to cross to the left abutment rim access road, the base of the left abutment and from the base of the left abutment to the base of the right abutment.

There is no current access from downstream of the dam site to either abutment rim or river grade. A permanent downstream access road for daily use is designed as shown on "Access Road Plan & Profile", Sheet J5. This road will be useful for construction. A permanent service road intended for intermittent use is designed to access the left abutment rim and is shown on "East Side Access Road To Top Of Dam", Sheets J6 & J7. This road is for post-reservoir use and will not be directly useful during construction.

The right abutment rim and the pasture area immediately to the west across the highway are covered with a layer of soft clayey soil. Drill access roads crossing these areas required repeated applications of locally available cinder surfacing in order to remain passable during wet weather. A drill access road was successfully constructed across the pasture area through the use of road stabilization fabric overlaid with 6 inches of cinder surfacing. The road was subsequently reclaimed with the removal of the cinders and stabilization fabric.

As these areas are designated as construction staging sites their use under all weather conditions will require surfacing. A source of volcanic cinder surfacing is located adjacent to State Highway 73 approximately 8 miles north of the dam site.

E.4 TIMBER PRODUCT AVAILABILITY

The area supports a softwood timber industry producing yard and structural lumber. Plywood and stress-graded lumber are not locally produced. The timber products required for the construction of the envisioned dam are available either from local producers or suppliers.

MORRISON-MAIERLE, INC.

MEMO

TO: FILE: MINER FLAT DAM, JOB #1740-14-02-03(30)

FROM: DALE ORTMAN

SUBJECT: AGGREGATE TEST PIT & PETROGRAPHIC ANALYSIS FOR E. SCHRADER

DATE: DECEMBER 8, 1986

As requested by Ernie Schrader a single test pit was dug and logged on November 7, 1986 at the proposed aggregrate source site. The purpose of the pit was to obtain a sample of material for preliminary petrographic testing in order to supply Mr. Schrader with data necessary to his initial concrete mix estimate for the rolled concrete dam alternative.

The test pit was dug to the depth limit of the Case 580 back hoe contracted for the work (Approximately 11 feet) and encountered groundwater at approximately 10 feet. The materials encountered are similar to those indicated in the previous test pits done by Mineral Systems Inc. (Mineral Systems Inc, 1983). The test pit was located adjacent to the earlier test pits as indicated on the attached map.

An approximately 500 pound bulk sample of 6 inch minus material was collected from the test pit. The sample was sieved at a testing laboratory in Tucson and representative samples of the 1 1/2 minus fractions were shipped to the following laboratory for petrographic analysis.

Construction Technology Laboratories
5420 Old Orchard Road
Skokie, IL 60077

Attn: Don Campbell

The results of the gradation and petrographic analyses are attached. It should be noted that material larger than 6 inch was excluded from the sample.

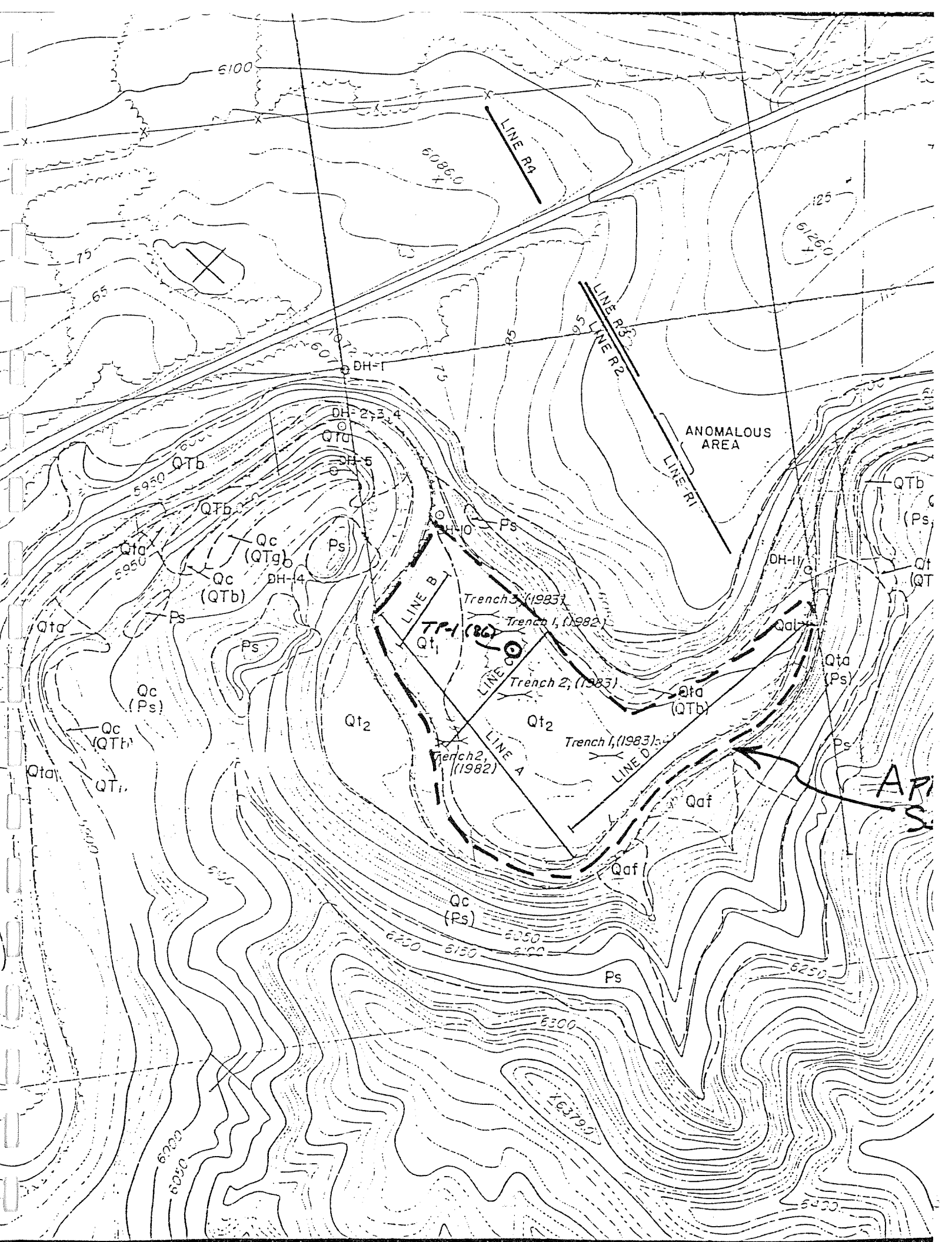
TEST PIT LOG

Location: Miner Flat Dam
Material: Aggregrate
Equip: Case 580

Test Pit No: TP-1(86)
Date: 11/7/86
Logged by: Dale Ortman

Depth Interval	Description
0 - 0.5 feet	Topsoil, Dk. Brown, Fine Sand some Silt, Roots & Organic material, moist
0.5 - 3.0	Sand trace Silt, med-fine, Lt. Brown, trace roots, moist
3.0 - 11.0	Sand & Gravel, approx. 20% cobbles & boulders 6"-2', damp (3-6 ft), moist (6-10 ft), groundwater encountered at 10 ft.

E.O.H. 11.0 ft.	



6100
LINE R4
6086.0
125
6126.0
75
65
LINE R3
LINE R2
LINE R1
ANOMALOUS AREA
DH-1
DH-2,3,4
Qta
DH-5
QTb
5950
QTb
Qc (QTg)
Qta
Qc (QTb)
DH-14
Ps
DH-10
Ps
LINE B
Trench 3, (1983)
Trench 1, (1982)
TP-1 (86)
Qt1
LINE C
Trench 2, (1983)
Qta (QTb)
Qt2
Qt2
Trench 1, (1983)
Trench 2, (1982)
LINE A
LINE D
Qal
DH-11
Qta (Ps)
Ps
Qaf
Qaf
Qc (Ps)
Qc (QTb)
Qta
QT1
Qc (Ps)
6050
6200
6150
6100
Ps
6250
6300
6000
6050
6400

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING • HYDROLOGY

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ROBERT L. FREW
ALLON C. OWEN, JR., P.E.

November 11, 1986

Morrison-Maierle, Inc.
910 Helena Avenue
P.O. Box 6147
Helena, Montana 59604

SHB Job No. C86-6504

Project: Miner Flat Dam

Lab No. 1009 Bulk sample delivered to SHB on 11/10/86, by Mr. Dale Ortman, P.E.

SIEVE ANALYSIS

Sieve Size	Percent Passing
6.0 inch	100
3.0 inch	84
2.0 inch	79
1.5 inch	71
1.0 inch	61
3/4 inch	54
1/2 inch	46
3/8 inch	41
No. 4	32
No. 8	29
No. 10	28
No. 16	26
No. 30	23
No. 40	19
No. 50	14
No. 100	7
No. 200	3.2

Tom L. Romero
Lab & Field Supervisor

Technical Services Report

to

MORRISON-MAIERLE, INC.

EXAMINATION OF AGGREGATE SAMPLES FROM MINER FLAT DAM PROJECT

Submitted

by

CONSTRUCTION TECHNOLOGY LABORATORIES
5420 Old Orchard Road
Skokie, Illinois 60077

DECEMBER 10, 1986

CTL a Division of the PORTLAND CEMENT ASSOCIATION
construction technology laboratories
5420 Old Orchard Road, Skokie, Illinois 60077-4321 • Phone 312/965-7500

TECHNICAL SERVICES REPORT

Project No.: CR-M318 Date: December 10, 1986

Re: Petrographic Examination of Fine and Coarse Aggregate Samples, Miner Flat Dam Project

One sieved sample of coarse aggregate and one unsieved sample of fine aggregate were received November 13, 1986, from Mr. D. Ortman, Morrison-Maierle, Inc., Helena, Montana. The coarse aggregate sample was comprised of five sieved fractions identified, respectively, as 1"-retained, 3/4"-retained, 1/2"-retained, 3/8"-retained, and No. 4-retained. The fine aggregate sand sample was identified as minus No. 4. Mr. Schrader, Consultant, Wallawalla, Washington, requested petrographic examination of both samples to determine the aggregate constituents. Results of the examination were reported by telephone to Mr. Schrader.

FINDINGS AND CONCLUSIONS

1. The coarse aggregate sample is a natural gravel comprised mainly of basalt, orthoquartzite, and metaquartzite. Minor amounts of sandstone, graywacke, siltstone, chert, limestone, and mud lumps are evident. The fine aggregate sample is a natural sand comprised mainly of basalt, feldspar, quartz, and mud lumps. The minor constituents

include orthoquartzite, metaquartzite, sandstone, graywacke, siltstone, chert, and magnetite. Mud lumps and free particles of rock-forming minerals are more abundant in the fine aggregate compared to the coarse aggregate. Tables 1 and 2 list aggregate constituents by number and weight percent.

2. Mud lumps are soft, friable, poorly consolidated mixtures of sand, silt and clay. Mud lumps are estimated to comprise 5.0% by weight of the coarse aggregate sample and 9.1% by weight of the fine aggregate sample. Friable coatings of similar material occur on some of the non-friable constituents of the aggregates.
3. Aggregate particles are rounded to subangular. The majority of particles are hard to moderately hard and dense. Particles of weathered rocks and minerals are concentrated in the finest sieve fractions, passing the No. 100 sieve.
4. Material passing the No. 200 sieve (finer than 0.075 mm diameter) consists mainly of feldspar at various stages of weathering, weathered rock fragments, quartz, mud lumps, iron oxides (magnetite, hematite, and limonite), and calcite.

DISCUSSION

Constituents of the aggregate samples are described in the ensuing discussion. Basalt is dark gray, brown to reddish brown, hard volcanic rock of fine grained plagioclase,

pyroxene, and variable amounts of magnetite, brown glass and olivine. In addition, basalt contains variable amounts of secondary alteration minerals of hematite, limonite and iddingsite. Massive, trachytic, vesicular, tuffaceous and porphyritic varieties of basalt were observed. Dacite is a light gray, hard volcanic rock of mainly fine-grained feldspar, and comprises less than 5% by weight of the fine and coarse aggregate samples.

Orthoquartzite and metaquartzite are light gray tan to red rocks comprised of quartz and small amounts of feldspar, magnetite, hematite, calcite and chert. Particles of these rocks are generally hard and dense; however, approximately 10 to 20% of orthoquartzite and metaquartzite particles are moderately soft and porous. Orthoquartzite generally has rounded to subangular, sand-sized quartz grains in an interstitial matrix of fine grained quartz or quartz plus iron oxides. In contrast, metaquartzite generally has a mosaic of interlocking irregular grains of quartz.

Chert is a hard, dense rock with a waxy luster and ranges from gray, tan, brown, blue to red. Chert is comprised of microcrystalline quartz, chalcedony and minor amounts of other minerals, namely hematite, magnetite and calcite. Veining, laminations, and concretionary or concentric structures are evident in many chert particles.

Sandstone and graywacke are gray, tan, brown to red sedimentary rocks comprised of angular to rounded, mainly sand-sized grains of minerals and rocks. The most common

grains are quartz, feldspar, chert and iron oxides (magnetite and hematite). Siltstone consists of similar grains which are mainly silt-sized. Sandstone, graywacke and siltstone are hard to moderately hard, dense rocks and component grains are set in a fine-grained matrix comprised of various combinations of quartz, iron oxides, calcite and clayey minerals.

Limestone is a light gray, moderately hard rock comprised of fine- to coarse-grained calcite. Minor amounts of quartz, chert and clayey minerals were observed with calcite.

Mud lumps are soft, friable, brown to tan mixtures of sand, silt and clay-sized grains of feldspar, quartz, chert, volcanic rocks and other rock and mineral types. Mud lumps are poorly consolidated and readily disintegrate upon exposure to water or when rubbed between fingers.

The fine aggregate sample also contains free particles of the following minerals: quartz, feldspar at various stages of weathering, calcite, magnetite and hematite. Small accessory amounts of granitic rocks (granite and granodiorite) and metamorphic rocks (schist, phyllite and hornfels) were observed in the aggregate samples.

METHODS OF TESTS

The fine aggregate sample was sieved and weighed in accordance with ASTM C 136-84a, "Standard Method for Sieve Analysis of Fine and Coarse Aggregates."

The sieved fine and coarse aggregate samples were examined in accordance with ASTM C 295-85, "Standard Practice for

Petrographic Examination of Aggregates for Concrete." Over 150 particles were examined and identified in each sieve fraction. However, the 1-inch sieve fraction of the coarse aggregate was also examined and identified even though it contained less than 150 particles. Particles from the No. 8 and coarser sieves were examined using a stereomicroscope at magnifications up to 45X. Particles from the No. 16 and finer sieves, as well as representative rock-types of aggregates, were examined by thin-section methods using a polarized-light microscope at magnifications up to 250X. Material retained in the pan fraction (passing the No. 200 sieve) was too fine to be counted, and constituents by relative abundance were estimated.

P. A. Studemeister

P. A. Studemeister
Research Petrographer
Concrete Materials/
Technical Services Department

PAS/mv

CR-M318

Attachments

Copy to--
D. C. Sikes
A. A. Alonzo

TABLE 1 – RESULTS OF PARTICLE COUNTS FOR COARSE AGGREGATE SAMPLE, MINER FLAT DAM PROJECT

Constituents	Composition of Fractions Retained on Sieves										Weight % in Coarse Aggregate
	1 inch[a] (25 mm)		3/4 inch (19.0 mm)		1/2 inch (12.5 mm)		3/8 inch (9.5 mm)		No. 4 (4.75 mm)		
	Number of particles	(%)	Number of particles	(%)	Number of particles	(%)	Number of particles	(%)	Number of particles	(%)	
Basalt	46	(54.0)	136	(48.0)	184	(46.7)	256	(54.6)	214	(53.9)	51.5
Orthoquartzite and Metaquartzite	13	(15.3)	45	(15.9)	76	(19.3)	67	(14.3)	54	(13.6)	15.7
Sandstone and Graywacke	5	(5.9)	18	(6.4)	33	(8.4)	25	(5.3)	10	(2.5)	5.7
Siltstone	2	(2.4)	40	(14.1)	18	(4.6)	25	(5.3)	23	(5.8)	6.1
Chert	3	(3.5)	18	(6.4)	23	(5.8)	37	(7.9)	44	(11.1)	6.8
Limestone	10	(11.8)	15	(5.3)	18	(4.6)	13	(2.8)	6	(1.5)	5.6
Mud Lumps	2	(2.4)	7	(2.5)	27	(6.8)	25	(5.3)	33	(8.3)	5.0
Miscellaneous[b]	4	(4.7)	4	(1.4)	15	(3.8)	21	(4.5)	13	(3.3)	3.6
TOTAL:	85	(100.0)	283	(100.0)	394	(100.0)	469	(100.0)	397	(100.0)	100.0
Weight % retained on sieve (excluding pan fraction)		25.6		18.2		20.7		12.9		22.6	100.0

NOTES:

(a) Insufficient number of particles (less than 150) was present to perform a complete examination of this sieve fraction in accordance with ASTM C 295-85.

(b) Miscellaneous constituents include granitic rocks (granite and granodiorite), dacite, ironstone, hornfels, and quartz.

TABLE 2 – RESULTS OF PARTICLE COUNTS FOR FINE AGGREGATE SAMPLE, MINER FLAT DAM PROJECT

Constituents	Composition of Fractions Retained on Sieves												Weight % in Fine Aggregate
	No. 8 (2.36 mm)		No. 16 (1.18 mm)		No. 30 (0.6 mm)		No. 50 (0.3 mm)		No. 100 (0.15 mm)		No. 200 (0.075 mm)		
	Number of particles	(%)	Number of particles	(%)	Number of particles	(%)	Number of particles	(%)	Number of particles	(%)	Number of particles	(%)	
Basalt	90	(34.5)	109	(59.1)	65	(26.5)	41	(15.6)	16	(8.3)	12	(5.9)	20.9
Dacite	10	(3.8)	3	(1.6)	6	(2.5)	14	(5.3)	6	(3.1)	0	(0)	3.3
Orthoquartzite and Metaquartzite	21	(8.1)	8	(4.4)	14	(5.7)	8	(3.1)	3	(1.5)	0	(0)	3.6
Sandstone and Graywacke	15	(5.8)	8	(4.4)	20	(8.2)	8	(3.1)	1	(0.5)	0	(0)	3.3
Siltstone	22	(8.4)	2	(1.1)	12	(4.9)	9	(3.4)	7	(3.6)	5	(2.4)	4.0
Chert	27	(10.3)	19	(10.3)	12	(4.9)	17	(6.5)	9	(4.7)	5	(2.4)	6.3
Limestone and Calcite	4	(1.5)	0	(0)	2	(0.8)	3	(1.2)	3	(1.5)	7	(3.4)	1.4
Ironstone	3	(1.1)	0	(0)	5	(2.0)	4	(1.5)	4	(2.1)	6	(3.0)	1.7
Quartz	7	(2.7)	2	(1.1)	13	(5.3)	44	(16.8)	18	(9.3)	3	(1.5)	8.6
Feldspar	16	(6.1)	7	(3.8)	58	(23.7)	80	(30.5)	80	(41.5)	95	(46.3)	28.0
Magnetite	0	(0)	0	(0)	0	(0)	3	(1.2)	15	(7.8)	21	(10.2)	3.3
Mud Lumps	38	(14.6)	18	(9.8)	27	(11.0)	11	(4.2)	16	(8.3)	31	(15.1)	9.1
Miscellaneous (a)	8	(3.1)	8	(4.4)	11	(4.5)	20	(7.6)	15	(7.8)	20	(9.8)	6.5
TOTAL:	261	(100.0)	184	(100.0)	245	(100.0)	262	(100.0)	193	(100.0)	205	(100.0)	100.0
Weight % retained on sieve (excluding pan fraction)	14.4		9.0		12.5		30.1		23.8		10.2		100.0

NOTES:

(a) Miscellaneous constituents include schist, phyllite, hornfels, granitic rocks (granite and granodiorite), claystone, hematite and other minerals.

MORRISON-MAIERLE, INC.

MEMO

TO: FILE: MINER FLAT DAM, JOB # 1740-14-02-03(30)

FROM: DALE ORTMAN

SUBJECT: RESULTS OF BASALT ALKALI REACTIVITY & L.A. ABRASION TESTS, MF-102

DATE: DECEMBER 5, 1986

In order to assess the preliminary suitability of basalt excavated from the dam foundation for use as concrete aggregate a sample has been submitted to a testing laboratory for Alkali Reactivity testing (ASTM C-289) and L.A. Abrasion testing (ASTM C-131). The attached results indicate the basalt is potentially as good as the upstream gravel source and is considered suitable as a source of concrete aggregate.

The sample is a composite of NX basalt core from drill hole MF 102 in the left abutment. The drill hole is directly adjacent to the proposed excavation and the sample was taken from the hole interval corresponding to the foundation excavation.

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING • HYDROLOGY
B DWAINE SERGENT P E
LAWRENCE A HANSEN PH D P E
RALPH E WEEKS P G
DARRELL BUFFINGTON P E
DONALD VAN BUSKIRK P G
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DALE V BEDENKOP P E
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DONALD G METZGER P G
JONATHAN A CRYSTAL P E
ROBERT D BOOTH P E
NORMAN H WETZ P E
ROBERT L FREW
ALLON C OWEN JR P E

December 4, 1986

Morrison-Maierle, Inc.
P. O. Box 6147
Helena, Montana 59604

SHB Job No. LT86-3447
Report No. 1

Attention: Dale Ortman, P.E.

Re: Miner Flat Dam/Hole MF 102

Gentlemen,

Transmitted herewith are the results of a potential reactivity test performed by Arizona Testing Laboratories on sample submitted. Results of L. A. Abrasion test performed in our laboratory is also attached.

Should any questions arise concerning this report, please do not hesitate to call.

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

By
Dale V. Bedenkop, P.E.

Copies: Addressee (2)

Arizona Testing Laboratories
817 West Madison Street □ Phoenix, Arizona 85007 □ 602/254-6181

For: Sergent, Hauskins & Beckwith
3940 West Clarendon
Phoenix, Arizona 85019

Date: November 19, 1986

Lab. No.: 0412

Sample: Aggregate

Received: 11/12/86

Submitted by: Same

Marked: LT 86-3447
W/O #1, Lab. #1
Your P.O. #L-4745

REPORT OF LABORATORY TESTS

Potential Reactivity C 289:

Dissolved Silica, mm/L	49
Reduction in Alkalinity, mm/L	35

Note: This material is not potentially reactive.

Respectfully submitted,
ARIZONA TESTING LABORATORIES

Claude E. McLean, Jr.

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

ENGINEERING ANALYSIS • PHYSICAL TESTING • QUALITY CONTROL • FIELD EXPLORATION

REPORT ON LABORATORY TESTS

DATE _____

PROJECT Miner Flat Dam JOB NO. LT86-3447

LOCATION - LAB NO. 6-3447-1

CLIENT Morrison-Maierle, Inc. ADDRESS P. O. Box 6147 Helena, MT 59604

SOURCE OF SAMPLE Hole MF 102

MATERIAL Vesicular Basalt

ASTM C-131
LOS ANGELES ABRASION

Grading A

Rev. 500

Loss 27.6%

APPENDIX S

Design Memorandum, Miner Flat Dam, February 1987

MORRISON-MAIERLE, INC.,
VOLUMES I THRU V OF VI

FEBRUARY 2007

DESIGN MEMORANDUM

MINER FLAT DAM

019842-0002

DECEMBER, 1986

(REVISED FEBRUARY 1987)

WHITE MOUNTAIN APACHE TRIBE

WHITERIVER, ARIZONA

VOLUME 3 OF 6

MORRISON-MAIERLE, INC.
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BOZEMAN, MONTANA
RIVERTON, WYOMING

MINER FLAT DAM
DESIGN MEMORANDUM

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ENGINEERING GEOLOGY MINER FLAT DAM SITE
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

Prepared for

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July, 1986

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PRELIMINARY REPORT
ENGINEERING GEOLOGY
MINER FLAT DAM SITE AND RESERVOIR
WHITE MOUNTAIN APACHE RESERVATION
ARIZONA

INTRODUCTION

The proposed dam on the North Fork of the White River at Miner Flat is east of State Highway 73, about eight miles south of McNary and about eleven miles north of the White Mountain Apache Tribal headquarters at Whiteriver, Arizona (fig. 1). Access to the site, and to the dam abutments, is by a pilot road that connects with State Highway 73 at Miner Flat.

The dam site was selected based on topography. A lava flow in the valley of the North Fork of the Whiteriver has been bisected by the Whiteriver forming steep almost vertical cliffs of about 170 feet in height. To determine the feasibility of the site, geotechnical investigations, including geologic mapping of the dam site and reservoir and drilling of three holes for core, were conducted in February and March, 1982. Based on the geotechnical work conducted in 1982, a program for additional geotechnical study was conducted in 1983. This program included detailed geologic mapping of the dam site; core drilling of ten additional holes, logging of core and testing of drill holes, geophysical investigation of the thickness of the basalt flow and gravel deposits, and bulk sampling and testing of the gravel deposits.

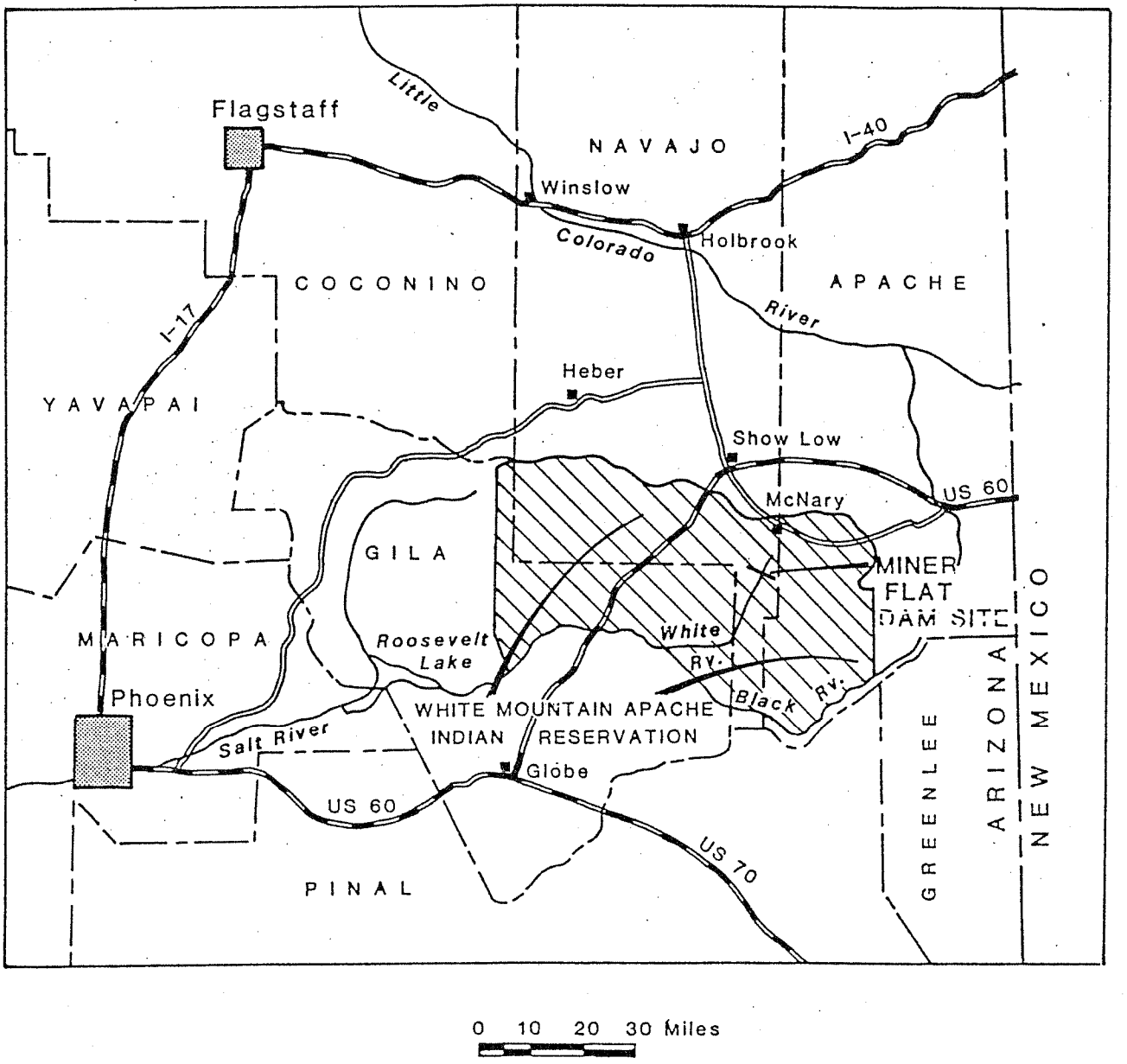


FIGURE 1.--**INDEX MAP OF THE WHITE MOUNTAIN APACHE INDIAN RESERVATION SHOWING LOCATION OF THE MINER FLAT DAM SITE**

Recommendations for additional investigations were made as a result of the 1983 investigations. This additional work, which consisted of core drilling 13 holes, testing of drill holes, and logging and testing of core were carried out in late 1985 and early 1986. This report presents the data obtained during the 1985-1986 investigations.

The geotechnical investigations were conducted by or under the supervision of Mineral Systems, Inc., under contract to Morrison-Maierle, Inc. of Helena, Montana. The preparation of the topographic maps and the surveying of the locations of the drill holes was done by Geotech Research, Inc. of Englewood, Colorado. Drilling during 1985-1986 was done by Kelmine Exploration, Inc. of Denver, Colorado.

GENERAL GEOLOGY

Miner Flat is the top of a basalt flow that flowed down an ancestrial valley of the North Fork of the White River. The area physiographically is in the transition zone between the Colorado Plateau (Hunt, 1956) and the Basin and Range Province (Moore, 1968). It is about eight miles south of a conspicuous topographic feature, the Mogollon Rim, which forms the southern limit of the Colorado Plateau. The geology of the area has not been previously studied in detail. The regional geology is shown on the geologic maps of Gila, Navajo and Apache Counties (Arizona Bureau of Mines, 1959, 1960) and the general geology is described by Moore (1968). The nearest detailed geologic studies are those of McKay (1972) of the Show Low Quadrangle about 15 miles to the northwest. Merrill and Pewe (1977) describe the Late Cenezoic geology of the White Mountains about 20 miles to the west. Condit (1983), in partial fulfillment of the requirements for Doctors Degree at the University of New Mexico, has mapped and dated the basalt flows along the North Fork of the White River to the north of the dam site.

The bedrock of the Miner Flat dam and reservoir site is the Supai Formation; which consists predominantly of sandstone at the site; and a series of basalt flows, which are capped by a partially indurated gravel. Overlying the bedrock are surficial deposits, which include colluvium (including talus), alluvium, alluvial fans and terrace deposits. The geology of the Miner Flat dam site and the reservoir are shown on Figures 2 and 3.

BEDROCK UNITS

The bedrock units are exposed in the cliffs along the North Fork of the White River. A valley was cut by the river into the Supai Formation. This valley was filled with a series of basalt lava flows. The river then deposited gravel on the basalt flow before cutting a channel through the margin of the flows and the Supai Formation.

Supai Formation

The Supai Formation crops out in steep cliffs south of the dam axis and locally in cliffs along the north and south sides of the reservoir. The Supai Formation is of Permian age. In the reservoir area only sandstone of the Limestone and Siltstone Member (McKay, 1972) crops out. At the top of the Limestone and Siltstone Member is a sandstone unit 50 to 70 feet thick. This is the unit that is exposed in much of the Miner Flat area. Below this unit in the area (but not exposed) is about 110 feet of siltstone and sandstone.

The sandstone is pale reddish brown to yellowish brown, fine grained, and well sorted. The beds range in thickness from less than an inch to 15 feet. The thickest beds are conspicuously cross-bedded and

massive. Near the base of the outcrop east of the dam axis beds of siltstone are exposed. The siltstone is reddish brown and sandy. The beds are less than 1 to 2 feet thick, parallel laminated with the laminae general 0.1 to 1 inch thick. The siltstones are gradational with over and underlying beds of sandstone.

Basalt

The basalt crops out and forms conspicuous cliffs at the dam site and along the north side of the reservoir. The flows have filled an ancestral valley of the North Fork of the White River and the surface of the basalt slopes to the south.

The basalt is very dark gray to black, weathering medium to dark gray. It is fine-grained to aphanitic and locally porphyritic. Phenocrysts of plagioclase feldspar and olivine, generally less than 1 mm in size, are conspicuous locally. The basalts are vesicular, with the vesicules of less than 1 mm to 30 mm. The vesicules occur in bands of 1 inch to 1 foot wide. Individual flows range from about 20 to 80 feet in thickness. In drilling it was noted that there was typically a thin clay seam of less than 0.1 to 4 inches between flows. Individual flows are not continuous and cannot be mapped throughout the reservoir

area. The flows moved down the valley in a series of tongues. A tongue would move some distance, stop, and then be partially or completely engulfed by a younger flow. A preliminary study of the flow banding shows banding at many dip angles; generally, however, the dips were all down the gradient of the valley.

The basalts flows of the White Mountains at the headwaters of the North Fork of the White River have been studied by Condit (1983). He has shown that there were a sequence of flows originating in the White Mountains that flowed down valleys to the south and west. He has dated these flows. The ages range from about 0.5 to 2.05 million years B.P. (before the present).

Paleocolluvium/Paleoalluvium

The basalt flowed down a stream valley. In the valley along the sides was colluvium derived from the weathering of the sandstone of the Supai Formation. In the bottom of the valley was alluvium deposited by the stream that eroded the valley. As the basalt flowed down the valley it pushed the colluvium aside and the basalt is now in contact with weathered sandstone, or the basalt flowed over the colluvium or alluvium often incorporating fragments of sandstone or boulders, cobbles, gravel sand and silt from the alluvium into the base of the basalt flow. The thickness of the paleocolluvium/paleoalluvium ranges from less than an inch to as much as about 69 feet.

Quaternary/Tertiary Gravel

Quaternary/Tertiary gravel deposits are classified with the bedrock deposits because they are locally partially indurated, or cemented, and could be considered as sandstones or conglomerates, and they are considerably older than the alluvial deposits along the river and in the terraces. These gravels crop out along the road to the Lower Log Road bridge at the eastern end of the reservoir area (fig. 2) and underly extensive colluvial covered areas north and south of the reservoir. The Quaternary/Tertiary gravel unit consists of stratified but poorly sorted lenses of sand and gravel that include cobbles and boulders. The clasts consist predominantly of well rounded igneous and metamorphic rocks. Correlation of this unit with other gravels, or interbasalt formations, as described by Merrill and Pewe (1977) to the east, or those mapped by McKay (1972) to the northwest, has not been made.

STRUCTURE

The structure of the bedrock is related to the uplift of the Colorado Plateau to the north and the cooling of the lava flows. The beds of the Supai Formation strike northeast to northwest and dip about 3° - 7° to the east. The low angle of dip, and the crossbedding, make the determination of dip difficult. Joints in the Supai Formation are spaced at intervals of 0.5 to 3 feet. Typically the joints strike at right angels or parallel to the strike of the beds, and dip from 60° to 90°. The gentle folding, and the development of joints in the Supai Formation, are probably related to the vertical uplift of the Colorado Plateau in middle or late Tertiary time (Hunt, 1956).

The discontinuities in the basalt are related to the flow banding and joints as a result of cooling of the basalt. When the basaltic lava cooled, the volume decreased, and joints formed. They form at right angles to the cooling surface. As most of the flows were essentially tabular bodies, the cooling surfaces were the base and the top of the flows, and the joints are near vertical. Joints have developed along flow boundaries as a result of stress relief along cliffs from erosion. Figure 4 is a contour diagram of discontinuities in the basalt in the vicinity of the proposed dam based on the data collected along the detail line studies. This diagram shows that the discontinuities strike in all directions and are near vertical -- typical of columnar jointed basalts.

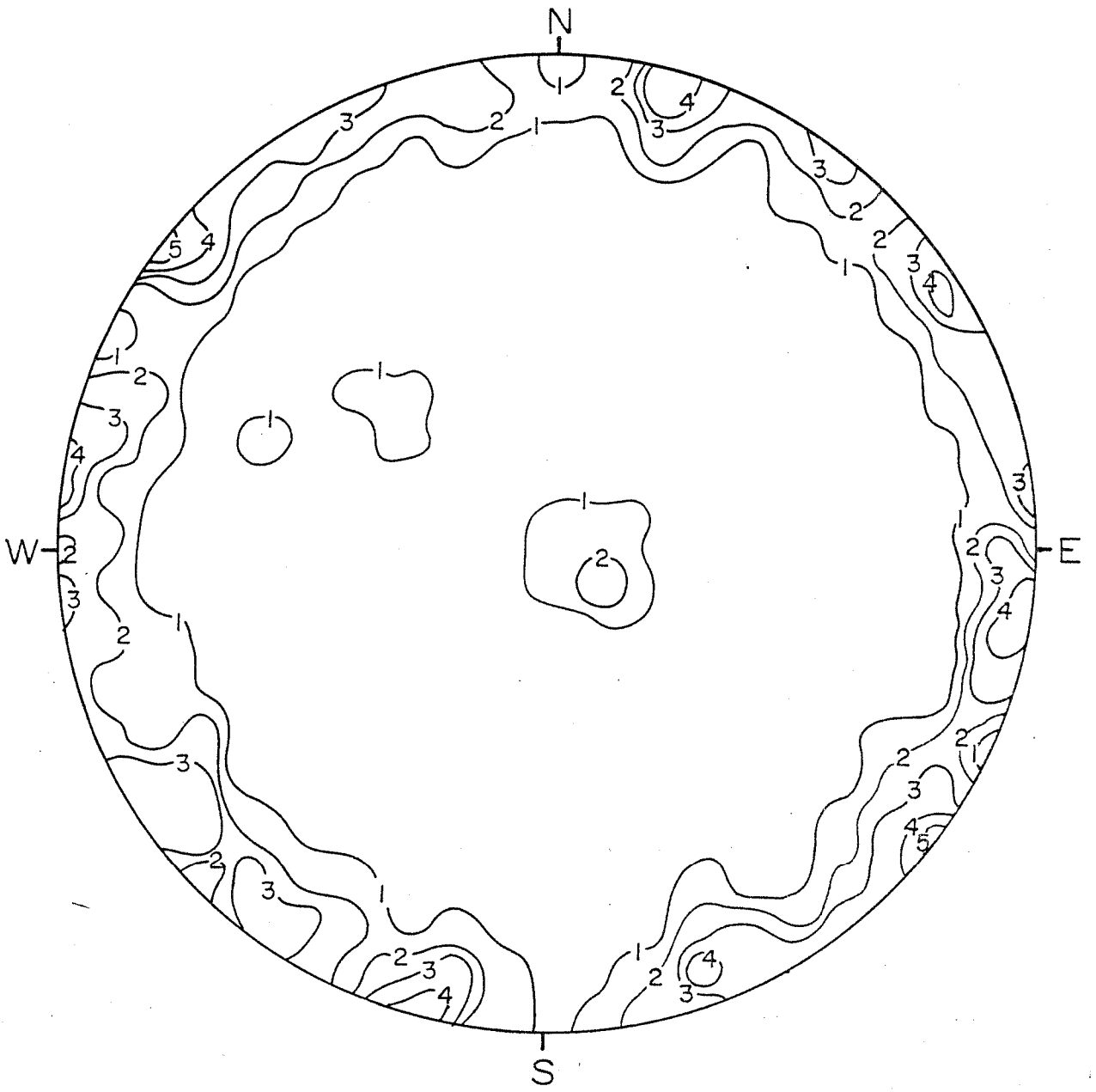


FIGURE 4

EQUAL AREA PLOT (LOWER HEMISPHERE)
OF DISCONTINUITIES (JOINTS) IN BASALT
MINER FLAT DAM SITE

(222 poles, Contour interval 1. percent)

SURFICIAL DEPOSITS

Surficial deposits are derived from the in-place weathering of bedrock and the transportation of weathered material or older surficial deposits. In the Miner Flat area, the surficial deposits include colluvial deposits, including talus deposits, and alluvial deposits, including alluvium, alluvial fans and terrace deposits.

Colluvial Deposits

The Colluvial deposits include the material derived from the inplace weathering of bedrock and talus from the weathering of cliffs.

Colluvium: -- Colluvium is weathered material from the bedrock and reflects the nature of the underlying bedrock. On steep slopes, it does move down slope under the force of gravity. On the geologic maps, Figures 2 and 3, the formation from which the colluvium is derived is indicated by the letter symbol for that formation in parentheses. The colluvium from the basalt consists of silt and clay, reddish brown to dark brown, with subangular sand, gravel, cobbles and boulders composed of basalt. The colluvium derived from the Supai Formation consists of

sand with angular pieces of gravel and cobbles of sandstone. Colluvium derived from the Quaternary/Tertiary gravels consist of cobbles and gravel in a sandy matrix that grades downward in sand and gravel with cobbles. There has been a concentration of the coarser material at the surface. On steep slopes, underlain by two formations, the colluvium on the slope will consist of a mixture of the two types of colluvium. The colluvium ranges in thickness from 0 to at least 50 feet.

Talus: -- Talus, a form of colluvium, consists of blocks of bedrock that have become detached from a cliff and fallen to the base of the cliff. Talus deposits occur below the basalt and sandstone cliffs along the river up and down stream of the dam site. The blocks of basalt and sandstone in the talus range from sand-size to boulders more than 10 feet in maximum dimensions. The talus deposits are estimated to reach a thickness of at least 50 feet.

Alluvial Deposits

Alluvial deposits are those deposits formed by running water. They include alluvium, terrace deposits and alluvial fans.

Alluvium: -- Recent alluvium occurs along the channel of the North Fork of the White River. The material consists of silt, sand, gravel and cobbles derived from pre-existing surficial deposits and transported and deposited by the present river. The sand and gravel are primarily of older metamorphic and igneous rocks, derived from the older Tertiary gravels (McKay, 1972) with a few pieces of basalt. The cobbles are composed of older metamorphic and igneous rocks, basalt, sandstone and limestone. The sedimentary rocks were derived locally from the Supai Formation.

Terrace Deposits: -- The terrace deposits are alluvial deposits formed by the river when it flowed at a higher elevation than at present. These deposits are 4 to 10 feet above the present alluvium along the river, and extend upriver throughout the reservoir area (fig. 2). They are composed of silt, sand, gravel and cobbles similar in composition to those in the alluvial deposits, except that there are more cobbles of sandstone and limestone in the terrace deposits. On top of the terrace deposits are over-bank food deposits of fine sand and silt 0.5 and 2 feet thick.

Alluvial Fans: -- Alluvial fans are formed where tributary steams enter a main valley and there is an abrupt change in gradient. They

are most common along the south side of the river where small streams have cut narrow and steep channels into the Supai Formation. The material in the alluvial fans consists primarily of silt and sand. They do contain some gravel and cobble size pieces of sandstone derived from the steep bedrock slopes bordering the channels.

GEOTECHNICAL INVESTIGATIONS

The geotechnical investigations conducted in 1982 (Robinson, 1982) were primarily for the purpose of determining the feasibility of constructing a dam near Miner Flats, and to select an approximate alignment. The 1982 study consisted of preparing a preliminary geologic map of the dam and reservoir area; drilling, logging, and testing three holes drilled for core approximately along the suggested axis of the dam; and digging two test pits, collecting samples, and running engineering analyses to evaluate the potential gravel resources. The purpose of the 1983 geotechical investigations was to obtain specific data for dam design. The geotechnical investigations (Mineral Systems, 1983) included preparing a detailed (1:600; 1 inch = 50 ft.) geologic outcrop map of the dam area (fig. 3) and completing the geologic outcrop map (fig. 2; 1:4800, 1 inch = 400 ft.) of the reservoir area; making five detail line surveys; making a seismic traverse across the basalt flow to determine the thickness; drilling, logging and testing eight holes drilled for core; installing piezometers to monitor groundwater levels in six of the drill holes; conducting a materials investigation that consisted of digging and sampling three test pits in the alluvial deposits upstream of the proposed dam, drilling three holes to determine the depth to bedrock, running four seismic traverse lines across the deposits, and conducting engineering tests on the samples from the test pits. Based on this work it was possible to establish a Rock Mass Classification for the dam foundation, give an estimate of the amount and

types of material immediately available for construction and to prepare recommendations for further geotechnical investigations for specific dam structures and appurtenant structures.

The geotechnical investigation in 1985-1986 was directed towards further defining the geology of the dam axis and abutment areas, and determining the engineering properties of the rock. Thirteen holes were drilled for core (fig. 2). The core was logged and tested with a point load tester. Packer tests were run on selected intervals in the drill holes and piezometers installed in twelve holes. Goodman jacking tests were run at selected intervals in four holes. All holes accessible were surveyed.

DRILLING

Holes were drilled and cored in the area of the Miner Flat Dam site to determine the engineering properties of the rock and to determine the hydrology of the foundation materials and the bedrock in the reservoir area. Thirteen holes were drilled using NX wire line equipment with double-tube core barrels. The core was logged and point-load tested. Packer tests were conducted to determine the permeability of selected intervals of the drill hole and Goodman jack tests were made to determine the in-situ elastic properties. Piezometers to measure groundwater levels were installed in twelve holes. One hole was lost as a result of running sand. The locations of the drill holes are shown on figures 2 and 3.

Core Logging

An engineering geologic log was made of the core after each run. In addition to standard notations as hole number, location, date, driller, etc., the following parameters were recorded:

Depth

Weathering/Alteration

Rock Type

R.Q.D. (Rock Quality Designation)

Total Core Recovery

Dip of Fractures

Strength (Point Load Test or Cohesive Soil Classification)

SCR (Selected Core Recovery)

The code system for the engineering logging of the core is included as Appendix B. The data from the logging form is encoded into a computer which justifies and prints out the data. Appendix B includes the Engineering Geologic logs of the 1985-1986 drill holes. Logs of the 1982 and 1983 drill holes were previously furnished (Robinson, 1982; Mineral Systems, 1983).

More detailed logs of each run are kept in a separate notebook. The logs include time of run, any drilling problems, as loss of circulation, additives, if any, and detailed geologic descriptions.

Graphic logs, including summary descriptions of rock units and details as to piezometer completions are given in Appendix A.

Testing

During the drilling operations, as the core was logged, selected intervals were subjected to point-load testing. As a drill hole was advanced the hydraulic conductivity, or permeability, of selected intervals of the rock in the walls of the drill hole was measured by packer tests. Also, as the hole was advanced, the in-situ elastic moduli of the wall rock was measured using a Goodman jack. After completion of the drill holes piezometers were installed, and developed, for measurement of groundwater levels.

Point-Load Tests and Strength Estimates

From each core run one or more samples of rock were selected for point-load testing or estimating rock strength. The point-load test is a method for measuring the strength of rock specimens in the field using portable equipment. Selected pieces of core with length to diameter ratios of greater than 1:5 are broken by the application of a concentric load using a pair of conical platens. A point-load strength index Is is obtained. The Point-Load Strength Index (Is) is expressed by:

$$Is = \frac{P \quad 2.36}{D^2}$$

where P is the failure load and D is the distance between the platens and 2.36 is a gauge factor. It has been determined that a reasonable correlation exists between the Point Load Index (Is) and the uniaxial compressive strength of materials (Bieniawski, 1975) and is:

$$\sigma c = Is \quad (14 + 0.175D)$$

where σc is the uniaxial compressive strength. The constant (14 + 0.175D) is a conversion factor from metric to English units and for core diameter.

All of the core recovered was not suitable for point-load testing. Some was too soft or friable. For these sections of core the classification for cohesive soil and rock, as proposed by Hoek and Brown (1980, p.98) was used to estimate the uniaxial compressive strength.

The strength of different rock types as determined from the point-load tests, or estimated from the classification of cohesive rock and soil are summarized in the following table. The strengths are classified as to the angle of the core hole. In general, the basalt flows and the beds of sedimentary rock were about horizontal, so the point-load tests on core in vertical holes was parallel to the flows or bedding, and the test in low-angle holes near right angles to the flows or bedding. A graphic presentation of the strength is shown on the logs in Appendix B. Figure 5 shows by histograms the range and average strength of the different units tested.

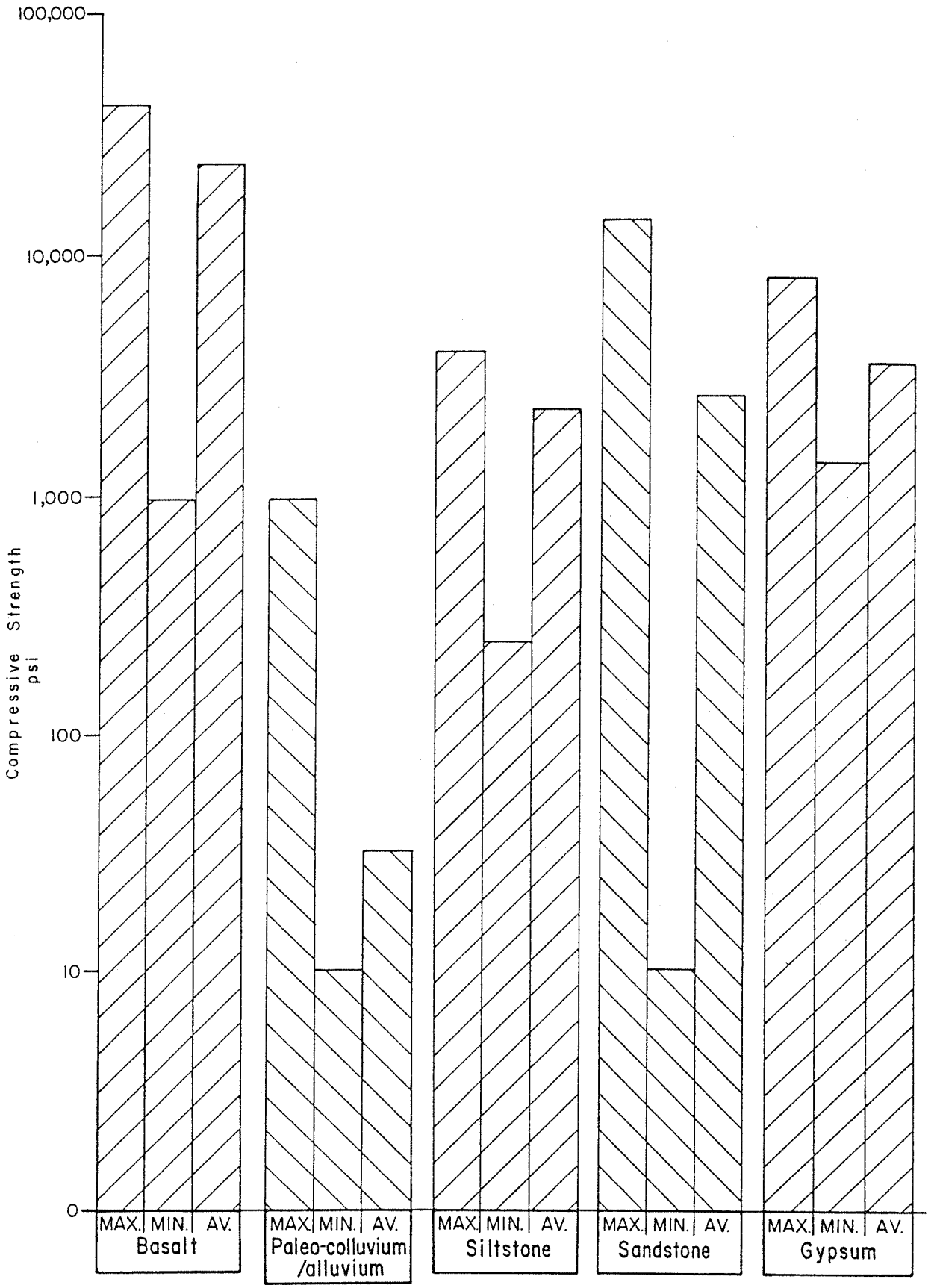


Figure 5 -Histograms showing strength of rock, Miner Flat dam site.

RESULT OF POINT-LOAD TESTS

Miner Flat Dam Site, 1985-1986 (pounds per square inch)

	Basalt			Paleoalluvium/ colluvium		Supai Formation				
						Siltstone		Sandstone	Gypsum	
	Vertical Holes	Angle Holes	Low-Angle Holes	Vertical Holes	Angle Holes	Vertical Holes	Angle Holes	Vertical Holes	Vertical Holes	Angle Holes
Maximum	42,109	36,547	36,547	957	10	13,983	5,403	13,983	8,263	6,594
Minimum	957	3,851	25,424	10	10	239	636	10	1,351	1,510
Average	23,780	22,863	32,308	35	10	2,705	1,629	2,654	4,018	3,284
Number of Samples Tested	255	58	17	64	10	28	18	60	7	12

It was proposed, and is recommended, that representative samples of the core be submitted to a material testing laboratory for determination of uniaxial compressive strengths. The purpose would be to establish the correlation between field and laboratory values for rock units at the proposed dam site.

Packer Tests

Packer Tests were conducted in each drill hole to determine the hydraulic conductivity, or permeability of the rock formations encountered in the drill holes. In general the following procedure was followed.

The hole was drilled for 50 feet into the formation to be tested, or to the bottom of the formation to be tested, and the hole washed until the return water was clear. A wireline inflatable packer was set at 20 feet from bottom of hole and the flow measured at a suitable pressure in relation to the maximum hydrostatic head that would be obtained for a full reservoir. If the flow was less than 20 gpm (gallons per minute), the packer was moved up the hole in 20 foot increments until a flow of about 20 gpm was achieved, or the entire hole was tested or the test interval overlapped a previously tested interval. If the initial flow exceeds 10 gpm then, the tested interval is reduced to the bottom 10 feet of the hole, and the upper part of the hole is tested by using two packers separated by 10 foot intervals. In some instances, because of geology or drilling problems, the hole was drilled to final depth and the double packer system used to test intervals of the hole that were considered significant.

Ideally a complete packer test consisted of five constant head flow vs. time tests. At each pressure, the pressure would be held constant and the flow measured each minute for 5 minutes, then at 10 minutes, and

at the highest pressure at an additional 15 or 20 minutes. The pressures used were:

0.33P Where P = 1 lb/ft reservoir head at the test section

0.66P

P Not to exceed 200 psi

0.50P

0.25P

The average permeability was calculated for specific test sections within each drill hole. The permeability coefficient K is defined by the U.S. Bureau of Reclamation (1974) as:

170 ft.

$$K = \frac{Q}{2\,LH} \ln \frac{L}{r}$$

Where

Q = average flow rate

L = length of test section

H = effective head of water acting on test section

3.250 in

r = radius of drill hole

The following table summarizes the results of the packer tests. The basic data are given in Appendix D.

HYDRAULIC CONDUCTIVITY IN CORE HOLES

Miner Flat Dam Site, 1985-1986 (centimeters per second)

	Basalt			Paleoalluvium/ colluvium		Supai Formation				
						Siltstone		Sandstone	Gypsum	
	Vertical Holes	Angle Holes	Low-Angle Holes	Vertical Holes	Angle Holes	Vertical Holes	Angle Holes	Vertical Holes	Vertical Holes	Angle Holes
Maximum	1.17×10^{-4}	5.71×10^{-4}	1.15×10^{-4}	3.54×10^{-4}	3.85×10^{-4}	1.50×10^{-3}	3.85×10^{-4}	1.73×10^{-3}	2.40×10^{-6}	$8.72 \times 10^{-}$
Minimum	5.20×10^{-8}	8.26×10^{-7}	7.47×10^{-5}	9.56×10^{-6}	1.06×10^{-4}	3.33×10^{-7}	1.25×10^{-6}	2.08×10^{-5}	1.55×10^{-6}	$4.33 \times 10^{-}$
Average	1.99×10^{-6}	5.11×10^{-5}	9.37×10^{-5}	5.92×10^{-5}	1.76×10^{-4}	3.74×10^{-5}	1.55×10^{-5}	2.87×10^{-4}	2.12×10^{-6}	$6.55 \times 10^{-}$
Number of Tests	13	18	3	8	3	11	4	10	2	3

A graphic presentation of hydraulic conductivity is shown in Appendix C.

Goodman Jack Tests

The Goodman jack is a borehole probe for measurement of borehole wall deformation as a function of an applied load. Data from the load-deformation measurements can be used to calculate the in-situ elastic moduli (Youngs modulus, E) of the rock. The probe is designed to operate in NX size (3.000-inch) holes. Deformation pressure is transmitted to the rock through hydraulic actuated movable plates. Two LVDT displacement transducers mounted at each end of the movable plates measure the deformation. The system also includes a portable solid-state indicator for measuring displacement, a hydraulic pump, pressure gauge, hydraulic hose and electric cable. The following table summarizes the results of Goodman Jack tests:

RESULTS GOODMAN JACK TESTS

Hole Number	Rock Type	Top Of Interval Tested (Feet)	Youngs Modulus			
			Extend		Refract	
			E(near)	E(far)	E(near)	E(far)
MF-102	Basalt	40.5	0.68	0.59	1.02	0.81
MF-102	Basalt	80.5	0.42	0.34	0.85	0.70
MF-102	Basalt	160.5	1.32	1.31	2.06	1.96
MF-105	Basalt	18.8	0.45	0.29	--	--
MF-105	Basalt	39.1	0.87	0.58	1.51	1.29
MF-105	Basalt	59.6	0.78	.036	--	--
MF-113	Basalt	80.8	2.46	1.66	2.21	1.48
MF-113	Basalt	101.5	2.58	2.22	2.76	2.46
MF-113	Paleo-alluvium	121.5	0.16	0.07	0.46	ERR
MF-113	Paleo-alluvium	130.6	53.91	15.58	--	--
MF-113	Paleo-alluvium	133.0	0.03	0.35	--	--
MF-117	Basalt	10.0	0.94	0.57	0.93	0.87
MF-117	Basalt	30.0	1.49	0.72	1.51	1.06
MF-117	Basalt	50.0	0.99	1.02	1.39	1.12
MF-117	Basalt	90.2	1.13	0.74	1.62	1.86

A graphic illustration of the lithology, results of the point-load tests and the results of the Goodman Jack tests are given in Appendix C.

It was proposed, and is recommended, that representative samples of the core be submitted to a materials testing laboratory for triaxial compressive testing and the determination of laboratory elastic properties. This would allow a correlation of the field and laboratory tests for the rock units at the Miner Flat dam site.

Piezometers

Piezometers have been installed in most of the holes drilled. In the 1985-1986 program the piezometers are of 1.5 inch PVC pipe. Figure 6 is a schematic diagram of a typical piezometer installation. Slotted PVC pipe was set opposite that section of the drill hole to be monitored. The section of the hole with the slotted PVC was filled with sand. A cement grout seal consisting of 1-bag of cement mixed with enough water to make it pourable--approximately 30 feet--was placed above the sand. The casing, and the space between the PVC pipe and the casing, were cemented to two feet below the bedrock. The piezometers were developed by blowing out the water in the piezometer until the return was clear.

The following table lists the drill holes in which piezometers have been installed in the Miner Flat dam area and measured water levels.

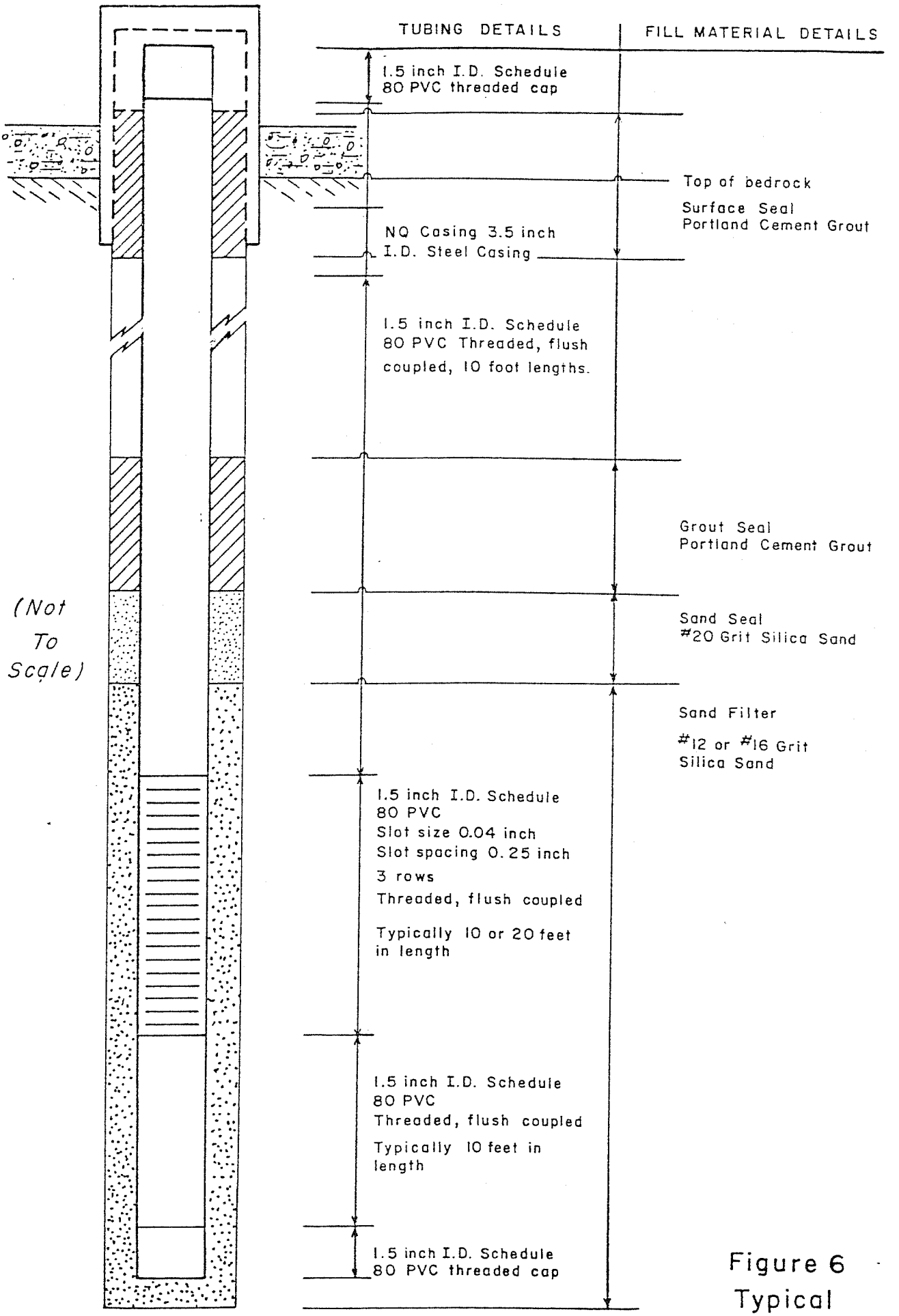


Figure 6
Typical
Piezometer Installation

Mineral Systems, Inc.

29-May-86

MINER FLAT DAM SITE: PIEZOMETERS and WATER LEVELS

HOLE	INSTALLED	DEVELOPED	MEASURED1	DEPTH1	ELEVATION1	MEASURED2	DEPTH2	ELEVATION2	MEASURED3	DEPTH3	ELEVATION3
DH-1	-	-		NA	NA		NA	NA		NA	NA
DH-2	06-Aug-83	-	21-Aug-83	0.4	5921.2		NA	NA	23-Apr-86	3.8	5917.8
DH-3	-	-		NA	NA		NA	NA		NA	NA
DH-3A		-		NA	NA		NA	NA		NA	NA
DH-4		-	21-Aug-83	0.8	5921.0		NA	NA		NA	NA
DH-5	12-Jul-83	-	21-Aug-83	157.8	5920.9		NA	NA		NA	NA
DH-10	06-Jun-83	-	21-Aug-83	4.1	5922.0	03-Mar-86	2.9	5923.1	23-Apr-86	2.9	5923.1
DH-11	06-Jun-83	-	21-Aug-83	8.0	5957.3	03-Mar-86	7.4	5957.9	23-Apr-86	7.4	5957.9
DH-12		-	21-Aug-83	96.4	6014.1		NA	NA		NA	NA
DH-13		-	21-Aug-83	76.7	6030.3		NA	NA		NA	NA
DH-14	22-Jul-83	-	21-Aug-83	173.9	5922.4		NA	NA		NA	NA
DH-19	26-Jul-83	-	21-Aug-83	-4.4	5921.6		NA	NA		NA	NA
MF-102	04-Feb-86	24-Mar-86	03-Feb-86	166.3	5915.8	03-Mar-86	165.3	5916.8	23-Apr-86	165.1	5917.0
MF-105	10-Jan-86	24-Mar-86	03-Feb-86	158.0	5923.4	03-Mar-86	159.8	5921.7	23-Apr-86	158.4	5923.0
MF-106	06-Dec-85	24-Mar-86	03-Feb-86	148.4	5925.4	03-Mar-86	150.0	5923.8	23-Apr-86	149.0	5924.8
MF-113	07-Mar-86	24-Mar-86		NA	NA	03-Mar-86	2.4	5918.1	23-Apr-86	-5.0	5925.5
MF-117	12-Mar-86	-		NA	NA		NA	NA		NA	NA
MF-118	15-Jan-86	24-Mar-86	03-Feb-86	138.0	5939.0	03-Mar-86	138.4	5938.6	23-Apr-86	138.8	5938.2
MF-119	17-Apr-86	23-Apr-86		NA	NA		NA	NA	23-Apr-86	30.2	6063.5
MF-120A	22-Apr-86	23-Apr-86		NA	NA		NA	NA	23-Apr-86	26.1	6055.9
MF-121	23-Feb-86	24-Mar-86		NA	NA	03-Mar-86	168.4	5921.4	23-Apr-86	168.1	5921.7
MF-122	08-Feb-86	24-Mar-86		NA	NA	03-Mar-86	153.2	5920.8	23-Apr-86	152.8	5921.2
MF-123	-	-		NA	NA		NA	NA		NA	NA
MF-124	04-Apr-86	23-Apr-86		NA	NA		NA	NA	23-Apr-86	159.8	5920.8
MF-125A	07-Apr-86	23-Apr-86		NA	NA		NA	NA	23-Apr-86	153.2	5911.8

Drill Hole Surveying

Most of the holes drilled for core in 1985-1986 were surveyed using an Owl Model 1275 digital borehole survey instrument. The survey data and graphic plots of the drill holes are given in Appendix F.

ENGINEERING GEOLOGY

The purpose of the 1985-1986 geotechnical investigations were to better define the distribution, engineering properties, and hydrology of the geologic materials in and adjacent to the proposed alignment for the Miner Flat dam. The work was done by drilling and coring holes, and subjecting the holes and core to testing. As a result of the drilling, the geologic maps (figs. 2 and 3) have been modified from those prepared in 1982 and 1983. The locations of drill holes are shown on figures 2 and 3. Geologic cross sections have been prepared from the geologic maps and the logging and surveying of the drill holes. Figure 7 is a geologic section along the dam axis at a scale of 1:1200 (1 inch = 100 feet). Figures 8 and 9 show geologic sections through the left abutment ridge at a scale of 1:600 (1 inch = 50 feet).

GEOLOGIC UNITS

The dam proposed at Miner Flat is to be a gravity concrete dam. The dam will be founded on basalt. The reservoir will be over surficial deposits that overlie the Supai Formation, the paleocolluvium/ paleoalluvium, the basalt and the Quaternary/Tertiary gravels. The distribution, hydrologic characteristics, and engineering properties of these units will influence the design and construction of the dam and reservoir.

Supai Formation

The upper part of the Supai Formation forms much of the bedrock under the reservoir and the east bank (fig. 2). A Supai Formation ridge forms part of the left abutment of the dam (fig. 3) overlying the Supai Formation is the paleocolluvium/paleoalluvium or basalt.

The Supai Formation consists for the most part of fine-grained sandstone. In the lower part of the section, for the most part below the reservoir, are interlayed sandy siltstone, gypsiterous sandstone and gypsum. The beds are friable to slightly indurated. Locally the lower part of a bed may be cemented with calcium carbonate, or gypsum, making them relatively hard. The uniaxial compressive strength, as determined from point-load tests, or estimated, ranged from about 14,000 to 10 psi. The hydraulic conductivity as determined by packer tests in drill holes ranged from 1.53×10^{-3} to 3.33×10^{-7} cm/sec.

Paleocolluvium/Paleoalluvium

Paleocolluvium (including talus) developed on the Supai Formation along the valley that was eroded into the Supai Formation prior to the floor of the basalt. The paleocolluvium/paleoalluvium are locally exposed between the basalt and Supai Formation in the walls of the present valley (fig. 2).

The paleocolluvium/paleoalluvium as exposed and intersected by drill holes ranged in thickness from 0 to almost 69 feet. In general, this material did not core well because of the range in rock types that compose the material and the range in sorting. The paleocolluvium could consist of only weathered Supai sandstone; angular blocks of sandstone that probably represented talus slopes or pieces of sandstone in silt and clay representative of slope-wash deposits. The paleoalluvium includes clay, silt and sand, and gravel, cobbles and boulders composed of igneous, metamorphic or sedimentary rocks from older formations, or basalt. The materials were poorly consolidated and core recovery was low. It was not possible to obtain representative samples for point-load tests. By placing the packer at the base of the overlying basalt, however, it was possible to determine the hydraulic conductivity. The hydraulic conductivity ranged from 1.06×10^{-4} to 9.56×10^{-6} cm/sec.

Basalt

The basalt will be the foundation of the dam and form both abutments. The basalt occurs on either side of the modern stream, but the mass of the basalt is to the west. The basalt filled a paleo-channel the axis of which is west of the present river (fig. 7). The flows came down the paleochannel in a series of tongues. From the coring and logging several different flows have been identified at the dam site. At the base is typically a massive flow. Most of the massive flow is fine-grained to aphanitic and slightly porphyritic with phenocrysts to 1mm of olivine. At the base will be a zone of less than a half a foot to 3 feet thick of scoraceous and vesicular basalt that has incorporated sand, pieces of sandstone, basalt fragments, or pebbles and cobbles from the underlying paleocolluvium/paleoalluvium. The upper part of the basalt is composed of a series of flows including vesicular and scoraceous basalt. Typically a thin paleosol of brown silty clay occurs between the individual flows. In drill hole 120A, west of the dam site, a 30 + foot paleosol, or paleoalluvium occurred between flows.

The engineering properties of the basalt have been extensively tested. In summary, the strength ranged from about 42,000 to 957 psi (Appendix F), the hydraulic conductivity from 1.15×10^{-4} to 5.20×10^{-8} cm/sec (Appendix D) and Youngs Moduli (E) from 0.34 to 2.76 (Appendix E).

Quaternary/Tertiary Gravel

Overlying the basalt flows at the dam site is a deposit of silt, sand, gravel and cobbles of unknown age. These deposits were formed by stream flowing on, and eroding the basalts. Near the head of the reservoir, on the east and west sides (fig. 2) equivalent gravels are overlain by younger basalt. The gravels are poorly consolidated and coring is difficult. Most of the fines are washed away, and only pieces of the pebbles and cobbles are recovered. From exposures along Highway 73 near Post Office Creek it may be seen that the deposits consist mostly of thin bedded sand and silt with local lenses of gravel and cobbles. On weathered surfaces, as on the Supai Formation ridge east of the left abutment, there is a lag concentration of the gravel and cobbles.

Channels were eroded into the basalt and the Supai Formation by the streams that deposited the Quaternary/Tertiary gravels. Figure 9 shows cross-sections along the Supai Formation ridge east of the left abutment of the dam. Based on the drilling, a channel filled with gravel deposits has been defined. The base of the channel is at an elevation about 6010 feet, which is about 50 feet below the maximum reservoir level.

STRUCTURE

The sedimentary rock units and the basalt flows are almost horizontal. The geologic structures that will influence the design and construction of the dam are the joints and the flow boundaries. No faults were identified in the area of the dam or reservoir.

Joints are not conspicuous in the sedimentary rocks. Because of the extensive cover (colluvium) there are only limited exposures of the sedimentary rocks, and these are the more massive units. In general, the joints are parallel or at right angles to the strike of the beds and dip steeply.

Columnar jointing, typical of basalt flows, occur in the area (fig. 4). The joints are near vertical. The joints in one flow do not extend into the joints in overlying or underlying flows. Joints do occur along the flow boundaries. These joints are near horizontal. Typically the joints, and flow boundaries in the basalt are filled, or their surfaces coated, with a fine silty clay.

GEOHYDROLOGY

The movement of water in the area of the dam and reservoir will be determined by the permeability of the bedrock. In order to evaluate the probable movement numerous packer tests were conducted in the drill holes (Appendix D).

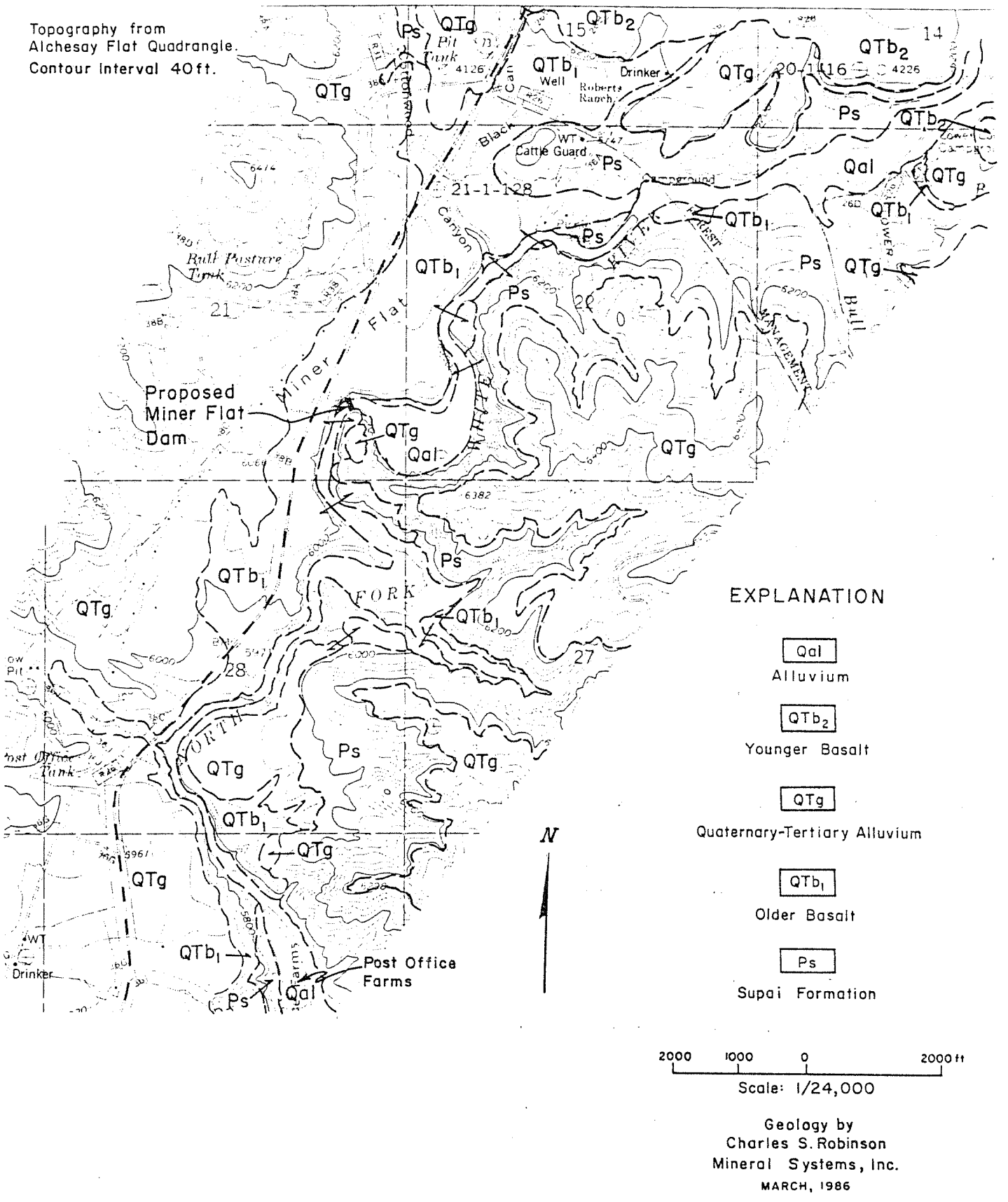
The permeability of the bedrock will depend upon the fracture permeability and the granular permeability. There are few fractures in the sedimentary rocks, and the permeability is primarily a granular permeability. The columnar jointing and the jointing along the flow boundaries in the basalt will be the primary permeability in the basalt. This permeability is limited, as shown by the packer tests. Below the zone of weathering in the basalt, or behind the joints opened by weathering and stress relief in the basalt cliffs, there is little interconnection between the joints and, as a result, the permeability is very low.

Of particular importance in the study of the Miner Flat dam and reservoir was the relation of the paleocolluvium/paleoalluvium to the dam and reservoir. These were the deposits on which the basalt was emplaced. From the geologic mapping it was known that the paleocolluvium/paleoalluvium would intersect the reservoir. It was considered possible that these deposits could serve as hydraulic channels from the reservoir.

Figure 10 is a map that shows the distribution of the basalt in the Miner Flat reservoir area. At the base of the basalt is the paleocolluvium/paleoalluvium. At an assumed maximum reservoir elevation of 6060 feet, it may be seen that water from the reservoir could infiltrate the paleocolluvium/paleoalluvium. Based on the drilling and the geologic mapping it was possible to contour the base of the basalt flow and identify the approximate center of the channel down which the basalt flowed. Water from the reservoir that infiltrates the paleocolluvium/paleoalluvium will move towards the center of the old channel, and then down the old channel.

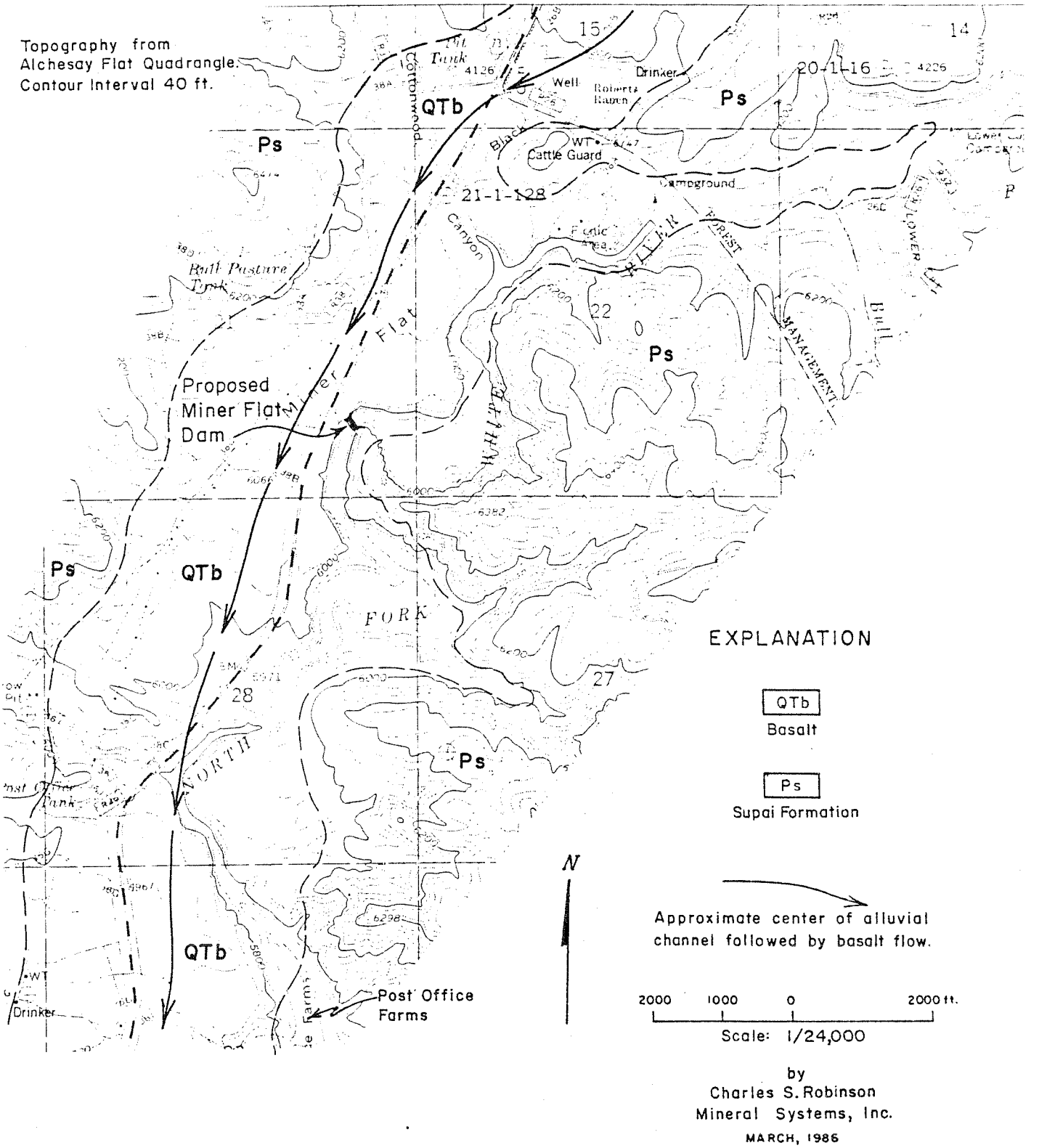
To determine where reservoir water that infiltrated the paleocolluvium/paleoalluvium might reappear, a reconnaissance geologic map was made of the entire Miner Flat dam and reservoir area (fig. 11). From that geologic map, a map showing the approximate limits of the Miner Flat basalt at the time of its emplacement was prepared (fig. 12). From these maps it may be seen that the reservoir water could not get out from under the basalt until at that point where the lower contact of the basalt crosses the North Fork of the White River just north of Post Office Farms.

Every effort was made to test the hydraulic conductivity of the paleocolluvium/paleoalluvium during the drilling program. Seven tests were conducted during the 1983 program and 11 tests in 1985-1986. The hydraulic conductivity ranged from 1.5×10^{-3} to 9.56×10^{-6} cm/sec, with an average of 1.20×10^{-4} cm/sec. With hydraulic conductivities this low, and a path length of more than one mile, the leakage from the reservoir through the paleocolluvium/paleoalluvium will not be significant.



Reconnaissance Geologic Map of the Miner Flat Dam Area, White Mountain Apache Reservation, Arizona

Figure 11



Map showing approximate location of Miner Flat basalt at time of emplacement. White Mountion Apache Reservation, Arizona

Figure 12

RECOMMENDATIONS

The geotechnical investigations of the Miner Flat dam site to date have defined the general suitability of the site for a concrete gravity dam and appurtenant structures. The following recommendations for additional geotechnical investigation are to establish more data for specific design of specific features.

DRILLING

Additional holes should be drilled for core, logged, tested and surveyed as have been the previous holes. Along the dam axis a 150 foot horizontal hole should be drilled, parallel to the dam axis about one third to one half way up the right abutment. A similar hole should be drilled on the left abutment, and an inclined hole should be drilled near the base of the left abutment. The holes in the left abutment should be drilled through the basalt, the paleocolluvium/paleoalluvium and into the sandstone. Short drill holes should be drilled into bedrock to below design grade in the area of all appurtenant structures. This would include at least three holes along the diversion tunnel, or cut, two holes at the power house, two holes along the line of the

proposed penstocks, and at least one hole in the stilling basin. The purpose of those holes would be to determine the depth of bedrock and the competency of the bedrock.

One or more holes should be drilled in the Quaternary/Tertiary gravel filled channel east of the left abutment. The shape of this channel needs to be better defined to design the cut off structure that will be required.

LABORATORY TESTING

Core samples of the different rock types drilled, and those to be drilled should be submitted to a materials testing laboratory for determination of uniaxial and triaxial compressive strengths. These laboratory data could then be correlated with the field measurements to define the in-situ engineering properties of the rock. The discontinuities in the rock, particularly the joints in the basalt, vertical and horizontal, should be subjected to direct shear tests. Based on these data, and the field data, a rock mass classification could be developed for the foundation rock for the dam.

DAM ANALYSES

Four major analyses have yet to be done; a final rock mass classification of the rock that will be involved in the construction, an analyses of the groundwater hydrology on reservoir filling, a slope-stability analyses of the cliffs downstream of the dam and above the power house, and an analyses of erosion in/or below the stilling basin. The program recommended above, and previous work, should give adequate data to make such analyses.

VOLUME II

PRELIMINARY REPORT

ENGINEERING GEOLOGY MINER FLAT DAM SITE

WHITE MOUNTAIN APACHE RESERVATION

NAVAJO COUNTY, ARIZONA

Prepared for

Morrison-Maierle, Inc.
Helena, Montana

by

Mineral Systems, Inc.
Golden, Colorado

July, 1986

VOLUME II

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Appendix E. -- Basic data, Goodman Jack tests

Appendix F. -- Basic data, drill hole surveys

APPENDIX A

Lithologic Logs of Drill Holes

MINER FLAT

DRILL HOLE NO. MF-102
Elevation: 6076 feet
Bearing: 320°
Date Started: 1/21/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 72°
Date Completed: 1/30/86

Depth (feet)	Lithologic Log	Description
		Fill and colluvium
10		Basalt, aphanitic, porphyritic, olivine, massive
		Basalt, vesicular, aphanitic, porphyritic, olivine phenocrysts. Clay-filled fractures, slightly weathered.
30		Basalt, vesicular, scoracious basalt at flow boundry at 31.6' Clay-filled vesicules
50		Basalt, massive, very fine grained to aphanitic, porphyritic, olivine phenocrysts.
		Basalt, vesicular, clay-filled fractures
70		Basalt, massive
		Basalt, vesicular, highly fractured. Clay-filled fractures
		Basalt, massive, very fine grained
90		Basalt, scoracious, highly fractured, vesicular, aphanitic, clay-filled fractures and vesicules.
110		
130		Basalt, massive, very fine grained to aphanitic, porphyritic, olivine phenocrysts. Highly fractured to 139.8'. Clay-filled fractures
150		Fracture, clay-filled at 150'
170		
190		
		Basalt, flow breccia, scoracious, weathered
		Basalt, massive
210		Basalt, flow breccia
		Basalt, massive
230		Paleo-colluvium. Upper 2 feet basalt and sandstone boulders. Sandstone boulders <0.5-1.65 feet in diameter in sand matrix, moderately yellowish brown.
250		
270		Sandstone, moderately yellowish brown, fine grained, silty, slighty calcareous.
290		Sandstone, reddish to yellowish brown, silty and sandy siltstone, local brecciation.
310		Sandstone, reddish brown, fine grained, silty, locally diagenetic breccia, white sandstone fragments.
330		
350		Sandstone, medium reddish brown, fine grained, with pale light gray silty sandstone, includes silty clay clasts. Gypsiferous, gypsum beds and veinlets to 0.4 feet thick. Locally sandstone brecciated
370		
375		E. O. H.

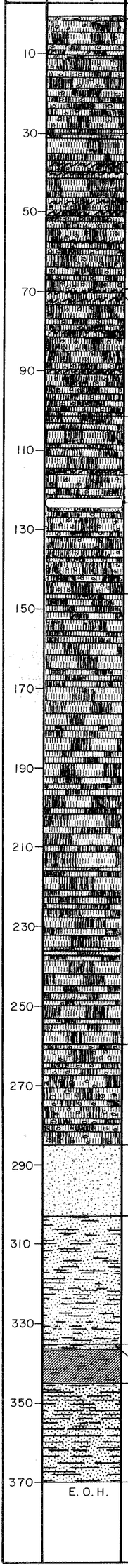
~~~~~ -Flow boundary

Mineral Systems, Inc.
~~~~~

MINER FLAT

DRILL HOLE NO. MF-105
Elevation: 6073 feet
Bearing: 140°
Date Started: 11/23/85

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 72°
Date Completed: 1/8/86



Description

Colluvium, weathered from basalt

Basalt, vesicular, fine grained, porphyritic, phenocrysts of olivine. Vesicules 20.01 to 0.25 feet, clay filled, weathered.

Basalt, massive, fine grained to aphanitic, porphyritic.

Basalt, vesicular. Vesicules clay filled.

Basalt, massive, very fine grained

Basalt, vesicular, horizontal fractures, olivine phenocrysts. Fractures clay filled or sand filled.

Basalt, massive, porphyritic.

Basalt, vesicular, fine grained to aphanitic, porphyritic, fractured, vesicules to 0.2 feet. Becomes more massive with depth. Fractures and vesicules filled with sandy clay.

Basalt, massive, medium gray, fine grained to aphanitic, porphyritic. Fractures clay filled.

Lava tube

Basalt, vesicular, locally ropey texture. Vesicules elongate parallel to flow banding. Vesicules and fractures clay filled. Some vesicules quartz filled.

Basalt, massive, medium gray to dark gray, very fine grained to aphanitic, porphyritic, phenocrysts of olivine, fractured. Fractures filled or stained with light brownish gray clay and/or soft olive green talc-like mineral.

Basalt, vesicular, medium gray, vesicules to 0.1 feet, aphanitic. Fractures and vesicules clay filled.

Paleo-alluvium. Boulders, cobbles and gravel in silty sand. Clasts of sandstone, limestone, quartzite, granite and basalt.

Sandstone, silty, reddish orange and yellowish orange, fine to medium grained, poorly indurated, fractures tight.

Sandstone, gypsiferous breccia

Gypsum, sandy and silty

Sandstone, fine grained, silty gypsum 354.5-355.0. Gypsiferous, veinlets of gypsum to 0.05 feet. Locally gypsiferous sandstone breccia.

∿∿∿∿∿ – Flow boundary

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-106
Elevation: 6072 feet
Bearing: 246.9°
Date Started: 11/17/85

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.4
Date Completed: 12/5/85

Description
Basalt, massive, very fine grained, aphanitic, porphyritic. Few vesicules, clay on fractures.
Basalt, vesicular, very fine grained to aphanitic, porphyritic, olivine phenocrysts. Clay on fractures.
Basalt, slightly vesicular, fine grained to aphanitic. Vesicules filled with clay or silica, clay stained fractures.
Basalt, massive to slightly vesicular
Basalt, vesicular, equigranular, olivine, fine grained to aphanitic. Vesicules filled with clay or silica, clay stained fractures.
Basalt, massive, very fine grained to aphanitic, porphyritic, olivine.
Open cavity
Basalt, vesicular, very fine grained to aphanitic. Vesicules < 0.01 to 0.1 ft. Filled with clay or silica.
Basalt, massive, very fine grained to aphanitic, porphyritic, phenocrysts of olivine. Locally fractured with fractures filled with clay or tight.
Basalt, vesicular, scoraceous, flow breccia.
Paleo-alluvium, boulders, cobbles and gravel in silty sand matrix. Clasts of basalt, granite, gneiss, chert, sandstone and limestone.
Sandstone, reddish orange and yellowish red, fine grained, silty, clayey, slightly calcareous, gypsiferous 341 - 352.8 ft.

~~~~~ - Flow boundary

Mineral Systems, Inc.
~~~~~

MINER FLAT

DRILL HOLE NO. MF-113
Elevation: 5920 feet
Bearing: 211.1°
Date Started: 2/25/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.7°
Date Completed: 3/27/86

Depth (feet)	Lithologic Log	Description
		Fill
10		
30		
50		Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, phenocrysts of olivine, magnetic. Fractured, fractures tight or slightly clay stained.
70		
90		
110		
		Basalt, vesicular, scoraceous, aphanitic.
130		Paleo-alluvium. Boulders, cobbles, gravel in silty sand. Clasts of basalt, granitic rock, chert, sandstone and limestone.
150		Sandstone, reddish brown, silty, very fine to fine grained.
155	E.O.H.	

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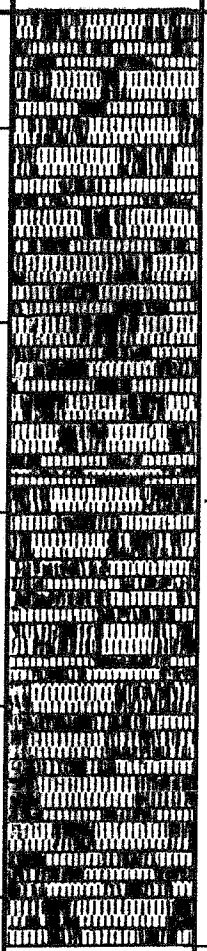
MINER FLAT

DRILL HOLE NO. MF-117
Elevation: 5924 feet
Bearing: 308.7°
Date Started: 3/7/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 26°
Date Completed: 3/9/86

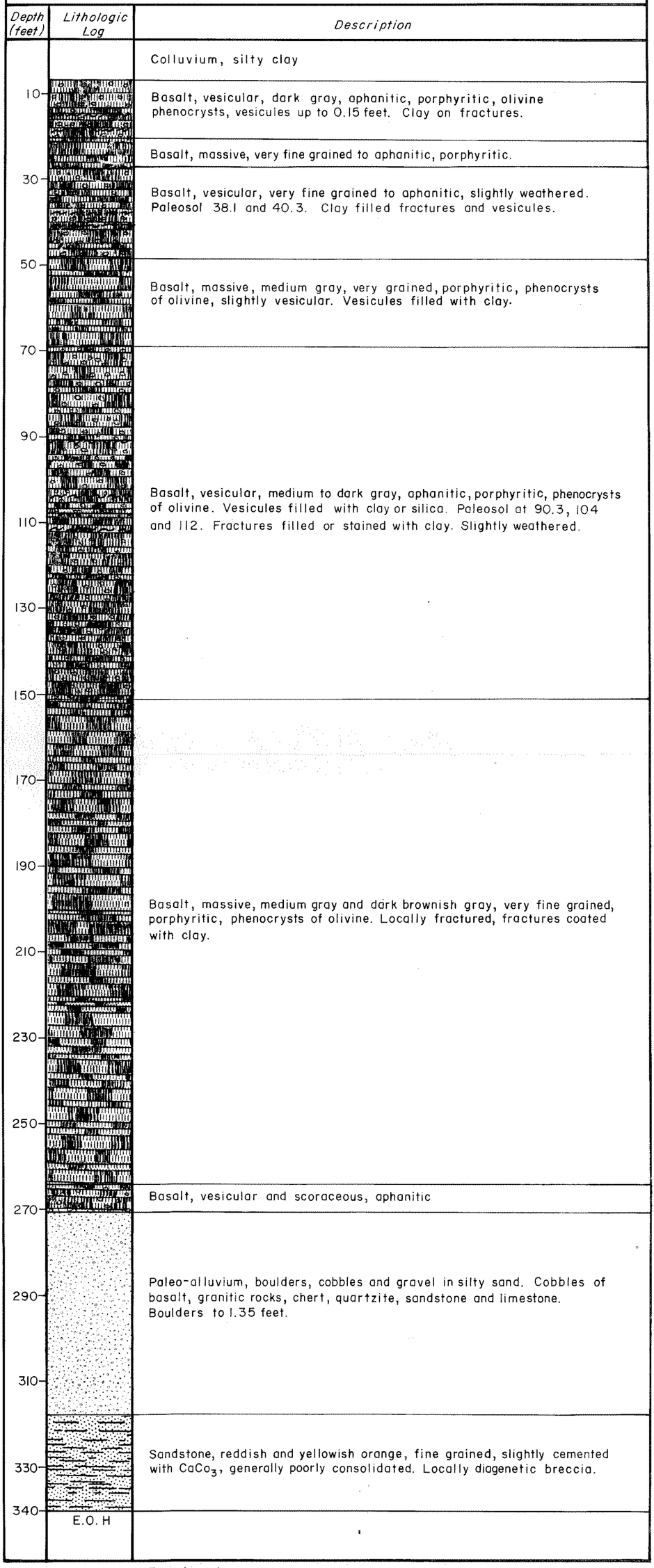
Depth (feet)	Lithologic Log	Description
10		Basalt, massive, medium dark gray, very fine grained to aphanitic, porphyritic. Phenocrysts of olivine to 1 mm. Locally fractured. Fractures filled with clay.
30		
50		
70		
90		
97.4	E. O. H.	

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-118
Elevation: 6077 feet
Bearing: 243.8°
Date Started: 1/11/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.5°
Date Completed: 1/15/86



~~~~~ – Flow boundary

Mineral Systems, Inc.
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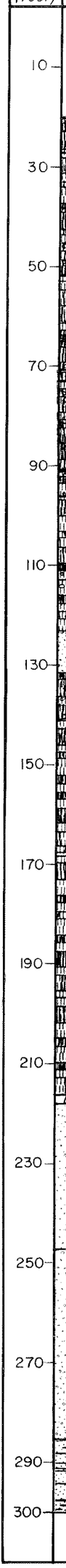
MINER FLAT

DRILL HOLE NO. MF-119
Elevation: 6094 feet
Bearing: 231.9°
Date Started: 4/8/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.9°
Date Completed: 4/16/86



Depth (feet)	Description
10	Colluvium. Silty clay, yellowish brown, cobbles of quartzite
	Basalt, vesicular, medium gray, slightly weathered
30	Basalt, massive, medium gray, fine grained, porphyritic, olivine phenocrysts, slightly weathered.
	Basalt, vesicular, aphanitic, porphyritic, olivine phenocrysts, moderate gray. Vesicules filled with clay and silica.
50	Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, olivine phenocrysts, fractures filled with silica and clay.
	Basalt, vesicular, silica filled vesicules.
70	Basalt, massive, slightly weathered
	Basalt, vesicular, scoraceous
	Basalt, massive Flow boundary with 1.3 feet paleosol
90	Basalt, vesicular, reddish gray, weathered.
	Basalt, massive, medium gray, very fine grained, aphanitic, porphyritic, olivine phenocrysts.
110	Basalt vesicular, reddish gray, aphanitic, silica and clay filled vesicules
	Paleosol, reddish gray, silty and clayey silt with cobbles or gravel of basalt.
130	Basalt, vesicular, medium gray and grayish red, aphanitic, ropey flow banding.
150	
170	Basalt, massive, medium gray, very fine grained to anphanitic, porphyritic, phenoclasts of olivine. Local clay filled fractures
190	
210	
	Basalt, flow breccia, fragments of basalt in clay matrix.
230	Paleo-colluvium talus, reddish brown to reddish orange and light gray sandstone fragments to 0.5 foot in silty sand matrix.
250	
270	Paleo-alluvium, reddish orange, cobbles and gravel of basalt, quartzite and sandstone in silty and clayey sand matrix. Limestone, quartzite and sandstone pebbles, slightly calcareous.
290	Sandstone, light yellowish gray, fine grained, friable to slightly calcareous cement, silty and clayey locally.
300	E.O.H.

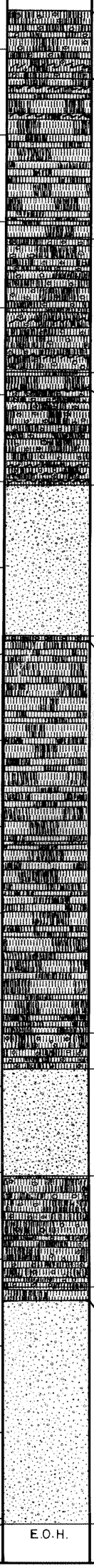
~~~~~ — Flow boundary

Mineral Systems, Inc.
~~~~~

MINER FLAT

DRILL HOLE NO. MF-120A
Elevation: 6082 feet
Bearing: 246°
Date Started: 4/18/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.5°
Date Completed: 4/21/86



Depth (feet)	Description
	Colluvium
10	Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts.
30	Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, phenocrysts of olivine.
50	
70	Basalt, vesicular, medium gray and reddish gray, aphanitic, porphyritic, phyenocrysts of olivine. Vesicules and fractures filled with silica and clay.
90	Basalt, massive, reddish gray, olivine phenocrysts.
	Basalt, vesicular, reddish gray, aphanitic, porphyritic, olivine phenocrysts.
110	Paleosol, medium gray to reddish gray basalt cobbles, gravel and sand in a dense clay matrix. Interflow alluvial deposit.
130	
150	Basalt, vesicular, medium gray, aphanitic, porphyritic.
170	Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, olivine phenocrysts. Clay stain on fractures.
190	
210	
230	Basalt, vesicular, aphanitic, scoraceous and ropy, cobbles of basalt, clay and sand matrix.
250	Paleosol alluvium. Cobbles and gravel of sandstone, basalt, quartz, quartzite and chert in reddish orange silty sand matrix.
270	Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Basalt breccia 275.2-276, 276.7-276.9, 278-278.5 Clay matrix. vesicules filled with clay or silica.
290	Basalt, massive, medium gray very fine grained to aphanitic, porphyritic. At base basalt breccia.
310	Paleo-alluvium, grayish orange, boulders, cobbles and gravel of sandstone, basalt, granitic rocks, metamorphic rocks, quartzite, limestone and chert in silty and clayey sand matrix.
330	
350	
351	E.O.H.

~~~~~ – Flow boundary

Mineral Systems, Inc.
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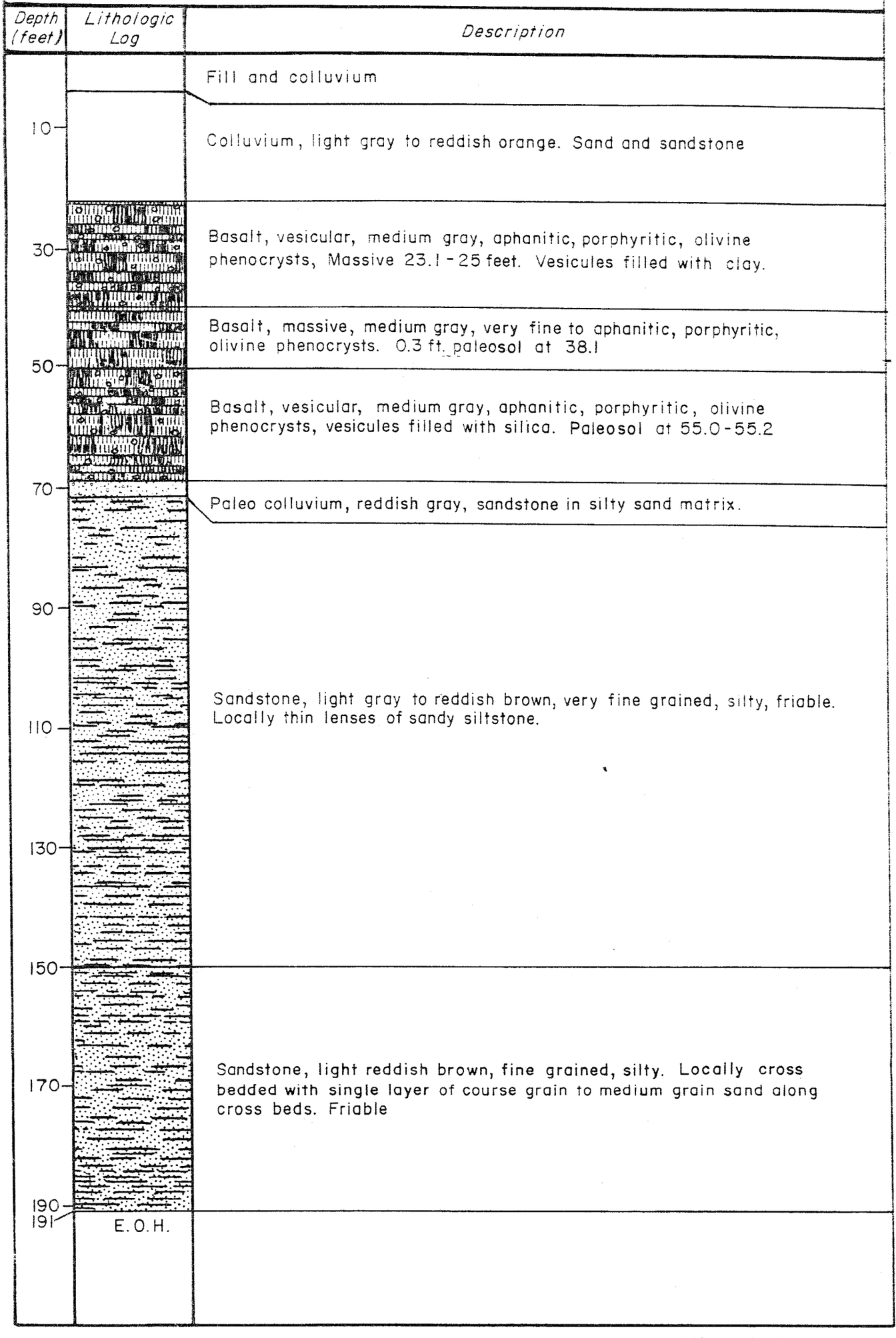
MINER FLAT

DRILL HOLE NO. MF-121
Elevation: 6090 feet
Bearing: 271°
Date Started: 2/10/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.8°
Date Completed: 2/28/86

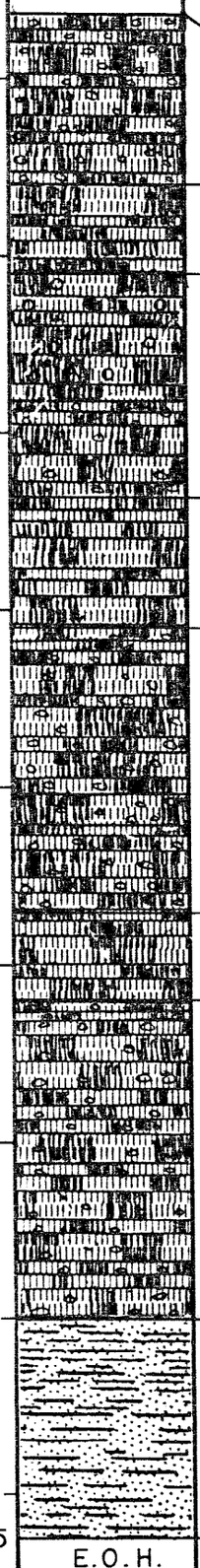


Mineral Systems, Inc

MINER FLAT

DRILL HOLE NO. MF-122
Elevation: 6064 feet
Bearing: 278.5°
Date Started: 2/5/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.9°
Date Completed: 2/8/86



Depth (feet)	Description
	Colluvium, clayey sand
10	Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Silica and clay filled vesicules and fractures. Specular hematite?
30	Basalt, massive, medium gray, very fine grained, porphyritic, olivine phenocrysts.
50	Basalt, vesicular, medium gray, weathers reddish brown, aphanitic, porphyritic, olivine phenocrysts. Vesicules and veins filled with coarse basalt sand and clay.
70	Basalt, massive, medium gray, very fine grained, porphyritic, olivine phenocrysts. Vesicules filled with clay at 65 ft.
90	Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Slightly weathered. Clay and silica filling vesicules.
110	Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic. Flow fractures.
130	Basalt, vesicular and scoraceous, medium gray, aphanitic, porphyritic, olivine phenocrysts. Sand size rock fragments between blocks of scoraceous basalt up to 1.5 ft. long.
150	Sandstone, light reddish brown to medium brown, fine grained. Contact of basalt and sandstone sharp at 150.3 ft.. Less than 0.03 ft. of paleo-colluvium. Sandstone silty and friable
170	
175	E.O.H.

Mineral Systems, Inc.

MINER FLAT

DRILL HOLE NO. MF-123
Elevation: 6113 feet
Bearing:
Date Started: 3/13/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 90°
Date Completed: 3/25/86

Depth (feet)	Lithologic Log	Description
10		
30		Quaternary - Tertiary Gravel. Cobbles and gravel in clayey, silty sand. At 43ft. 2-3 ft. dark carbonaceous sandstone. From 51 ft., cobbles, gravel and sand of white, fine grained sandstone.
50		
70		Sandstone, light gray to pinkish gray, light pinkish orange, fine grained, silty, moderately indurated. Sandstone breccia at 74-76 ft.
90		
110		Sandstone, light gray to light pinkish gray, fine grained, silty, poorly indurated. Below 113ft. interbedded reddish brown silty and clayey fine grained sandstone.
130		Sandstone, medium reddish brown, fine to medium grained, silty, some clay, poorly indurated.
150		Sandstone, light reddish to light yellowish brown, fine grained to very fine grained, silty, locally clayey, poorly indurated.
207.5	E.O.H.	

~~~~~ Flow boundary

Mineral Systems, Inc.
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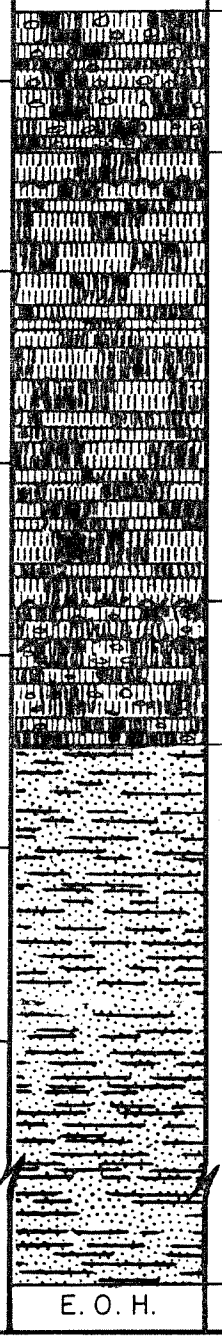
MINER FLAT

DRILL HOLE NO. MF-124
Elevation: 6081 feet
Bearing:
Date Started: 3/1/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 90°
Date Completed: 3/4/86



Depth (feet)	Description
10	Quaternary - Tertiary Gravel. Cobbles, gravel, sand and silt.
30	Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Vesicules filled with silty clay.
50	Vesicular, clay filled flow boundary
70	Basalt, massive, medium gray, very fine grained, porphyritic, olivine phenocrysts. Vertical fractures, clay filling.
90	Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts. Vesicules clay.
110	Sandstone, light reddish orange, very fine grained, very friable. 99.4 - 101.3 only 2 pieces of sanstone recovered. Paleo-colluvium?
130	
180	E. O. H.

~~~~~-Flow boundary

Mineral Systems, Inc.
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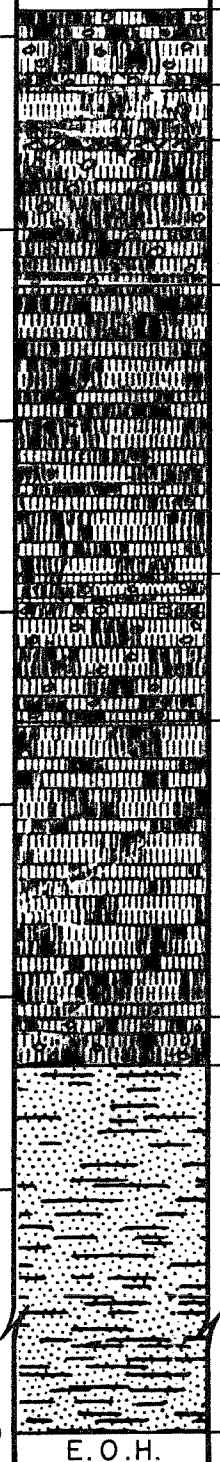
MINER FLAT

DRILL HOLE NO. MF-125A
Elevation:
Bearing: 189.9°
Date Started: 3/5/86

Drilled by: Kelmine Exploration
Logged by: Charles H. Robinson
Inclination: 89.9°
Date Completed: 3/7/86



Depth (feet)	Description
	Quaternary - Tertiary gravel
10	Basalt, vesicular, medium gray, aphanitic, porphyritic, olivine phenocrysts.
	Basalt, massive, medium gray, very fine grained, porphyritic.
30	Basalt, vesicular, medium gray, very fine grained to aphanitic, porphyritic, olivine phenocrysts, slightly magnetic. Clay filled vesicules and fractures.
50	Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, olivine phenocrysts, slightly magnetic. Fractures vertical, stained with clay.
70	Basalt, vesicular, very fine grained to aphanitic, porphoritic. Vesicules filled with clay.
90	Basalt, massive, medium gray, very fine grained to aphanitic, porphyritic, olive phenocrysts, slightly magnetic.
110	Basalt, vesicular, medium gray, aphanitic.
130	Sandstone, pinkish gray to reddish orange, fine to very fine grained, silty, thinly bedded, poorly indurated.
170	E.O.H.

~~~~~~-Flow boundary

Mineral Systems, Inc.
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APPENDIX B

Engineering Geologic Logs of Drill Holes

APPENDIX B

CODING SYSTEM FOR ENGINEERING GEOLOGIC LOG OF CORE

The following form was used to log the core from the drill holes. The headings across the top are self explanatory. An explanation for the columns is as follows:

Depth: -- Depth at bottom of hole at the completion of each run usually reported by driller.

Weathering/Alteration: -- This is a record of type of weathering or alteration of the rock and the intensity, Table 11, Appendix B.

Rock Type: -- This is the genetic name for the type of rock in the core. If more than one type of rock encountered in a run, the run is logged by rock type. Two letter system used. Table 3. Mineral modifiers are used to further define the rock. Table 4. The grain size is defined on table 5. Texture bedding and sorting are defined by Table 6.

R.Q.D.: -- Is the combined length of core pieces greater than 4-inches long expressed as a percentage of the total core run.

Total Core Recovery: -- Length of core recovered.

Dip of Fractures: -- The number of fractures in the run form angles of 90°-70°, 70°- 50°, and 30°-0° as measured from the core axis, are recorded. Original logging used 90°-60°, 60°-40°, 40°-20 ° and 20°-0°. The roughness is measured using the scale in Appendix A. Alteration or

filling along the fracture is defined on Tables 7, 8 and 9, Appendix B. The type or genetic classification of fractures is given in Table 10.

Strength: -- Selected samples of rock core were tested in the field with a point load tester. It has been found that a reasonable correlation exists between the Point Load Index and the Uniaxial Compress Strength of Material (Bieniawski, 1974).

The Point Load Index (Is) is:

$$IS = \frac{P}{D^2}$$

where P is the failure load and D is the distance between platens. The uniaxial compressive strength σ_c is:

$$\sigma_c = 24 \text{ Is}$$

The gauge reading on the point load tester is recorded in the initial logging.

SCR: -- Selected core recovery. The total length of core in a run greater in length than 1 foot expressed as a percentage of the total run.

Comments: -- Notes that may be significant in the interpretation of the engineering properties of the rock or test results are recorded.

MINER FLAT DAM SITE

TABLE 1: ROCK MASS WEATHERING/ALTERATION

CODE	DESCRIPTION
	UNWEATHERED/UNALTERED
A	ARGILLIC
Q	SILICIFICATION
B	CALCAREOUS
W	WEATHERED
	NO DESCRIPTION

MINER FLAT DAM SITE

TABLE 2: INTENSITY

CODE	DESCRIPTION
F	FRESH
S	SLIGHTLY
M	MODERATELY
H	HIGHLY
C	COMPLETELY
R	RESIDUAL SOIL

MINER FLAT DAM SITE

TABLE 3: GENETIC ROCK TYPE

CODE	DESCRIPTION	
BA	BASALT	IGNEOUS
SS	SANDSTONE	SEDIMENTARY
SI	SILTSTONE	SEDIMENTARY
SM	SILTY SANDSTONE	SEDIMENTARY
GY	GYPSUM/ANHYDRITE	SEDIMENTARY
PA	PALEO-ALLUVIUM	SEDIMENTARY
PC	PALEO-COLLVIUM	SEDIMENTARY
FI	FILL	SEDIMENTARY
CO	COLLUVIUM	SEDIMENTARY
TG	TERTIARY GRAVEL	SEDIMENTARY

MINER FLAT DAM SITE

TABLE 4: MINERAL MODIFIERS

CODE	DESCRIPTION
	NO MINERAL MODIFER
O	OLIVINE
Q	QUARTZ
M	SILT
S	SAND
G	GYPSUM
C	CLAY
	NO MINERAL MODIFER
F	IRON
R	GRAVEL
B	COBBLES
O	BOULDERS
A	CARBONATE

MINER FLAT DAM SITE

TABLE 5: GRAIN SIZE

CODE	DESCRIPTION	
A	APHANITIC	IGNEOUS
B	BOULDER	SEDIMENTARY
C	COBBLES	SEDIMENTARY
G	GRAVEL	SEDIMENTARY
V	VERY FINE	IGNEOUS AND METAMORPHIC
F	FINE	IGNEOUS AND METAMORPHIC
M	MEDIUM	IGNEOUS AND METAMORPHIC
R	COURSE	IGNEOUS AND METAMORPHIC
P	VERY COURSE	IGNEOUS AND METAMORPHIC
	NO GRAIN SIZE	SEDIMENTARY

MINER FLAT DAM SITE

TABLE 6: TEXTURE, BEDDING, SORTING

CODE	DESCRIPTION	
V	VESICULAR	IGNEOUS
M	MASSIVE	IGNEOUS
A	APHANITIC	IGNEOUS
P	PORPHYRTIC	IGNEOUS
B	BEDDED	SEDIMENTARY
T	THINNLY BEDDED	SEDIMENTARY
L	LAMINATED	SEDIMENTARY
R	BRECCIATED	IGNEOUS
	NO TEXTURE	IGNEOUS
S	SCORACIOUS	IGNEOUS
X	CROSS BEDDED	SEDIMENTARY

MINER FLAT DAM SITE

TABLE 7: FRACTURE CHARACTERISTICS

CODE	DESCRIPTION
T	TIGHT
O	OPEN
S	STAINED
F	FILLED
H	HEALED
	NO CHARACTERISTICS

MINER FLAT DAM SITE

TABLE 8: STAIN TYPES

CODE	DESCRIPTION
I	IRON
C	CLAY

MINER FLAT DAM SITE

TABLE 9: FILLING TYPES

CODE	DESCRIPTION
Q	QUARTZ
C	CLAY
	NO FILLING TYPES
R	CRUSHED ROCK
G	GYPSUM
F	IRON

MINER FLAT DAM SITE

TABLE 10: GENETIC DISCONTINUITY CLASSIFICATION

CODE	DESCRIPTION
SJ	SINGLE JOINT
JS	JOINT SET
FB	FLOW BOUNDRY
CT	CONTACT
BD	BEDDING

MINER FLAT DAM SITE

TABLE 11: STRENGTH

CODE	DESCRIPTION
S1	VERY SOFT SOIL/ROCK
S2	SOFT SOIL/ROCK
S3	FIRM SOIL/WEAK ROCK
S4	STIFF SOIL
S5	VERY STIFF SOIL
R1	VERY WEAK ROCK
R2	WEAK ROCK
R3	MEDIUM STRONG ROCK
R4	STRONG ROCK
R5	VERY STRONG ROCK

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-102

SITE:	MINER FLAT DAM SITE	NORTHING:	1,085,798.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	LEFT ABUTEMENT	EASTING:	576,853.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-102	ELEVATION:	6,078.40	START DEPTH:	0.00
START DATE:	01/21/86	INCLINATION:	77.9	LOGGED BY:	CHR
COMPLETION DATE:	01/30/86	BEARING:	319.3	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
8.0	WF	BAD VP	10.0	33.8	0	0			0	0			0	0			0	0			17,089	0.0	‡
12.7	WS	BAD VP	74.5	100.0	5	4	FC	SJ	0	0			0	0			4	3	FC	SJ	17,089	32.3	‡
23.3	WF	BAD VV	68.4	97.2	6	3	T	SJ	0	0			0	0			3	3	FC	SJ	19,419	67.5	‡
31.2	WF	BAD VM	83.0	98.7	6	4	T	SJ	0	0			0	0			3	4	FC	SJ	22,215	83.0	‡
41.8	WF	BAD VV	52.9	82.4	7	4	FC	SJ	0	0			0	0			3	4	FC	SJ	18,642	39.1	‡
51.0	WF	BAD VM	71.5	100.0	3	3	T	SJ	6	3	FC	SJ	0	0			2	3	FC	SJ	17,089	29.7	‡ #
61.0	WF	BAD VM	74.5	98.0	0	0			5	3	SC	SJ	0	0			3	2	SC	SJ	17,399	37.1	‡
71.0	WF	BAD VM	78.7	100.0	0	0			5	3	SC	SJ	0	0			2	2	SC	SJ	26,721	59.3	‡
81.0	WF	BAD VM	72.3	98.5	0	0			2	3	SC	SJ	0	0			7	3	FC	SJ	22,526	53.0	‡
91.0	WS	BAD VV	25.3	87.5	12	4	FC	SJ	5	3	FC	SJ	0	0			8	3	FC	SJ	10,875	14.7	‡
101.0	WS	BAD VS	0.0	28.0	0	0			0	0			0	0			0	0			18,642	0.0	‡
111.0	WS	BAD VS	0.0	26.0	0	0			0	0			0	0			0	0			R3	0.0	
121.0	WF	BADQVM	21.2	50.0	2	3	T	SJ	4	3	T	SJ	0	0			5	3	T	SJ	17,089	0.0	‡
131.0	WF	BADQFM	64.1	100.0	3	3	T	SJ	0	0			4	3	T	SJ	5	4	T	SJ	29,517	49.2	
139.8	WF	BADQFM	11.5	48.3	11	3	SC	SJ	7	4	FC	SJ	6	4	FC	SJ	8	3	FC	SJ	4,661	0.0	‡
150.1	WF	BAD FM	75.7	100.0	6	4	T	SJ	0	0			2	3	T	SJ	5	3	FC	SJ	34,954	47.8	‡
160.3	WF	BADQPM	84.8	100.0	4	3	T	SJ	2	3	T	SJ	0	0			2	3	SC	SJ	27,963	84.8	‡ #
170.6	WF	BADQPM	94.5	98.1	0	0			0	0			0	0			2	2	T	SJ	30,294	94.5	
180.7	WF	BADQPM	93.1	99.5	4	5	SC	SJ	2	4	SC	SJ	0	0			0	0			28,740	87.3	
190.8	WF	BADQPM	100.0	100.0	0	0			0	0			0	0			0	0			31,071	100.0	‡
200.9	WF	BADQPM	77.2	100.0	0	0			0	0			4	3	SC	SJ	2	2	FC	SJ	34,178	77.2	‡ #
211.0	WF	BADQVV	74.3	100.0	0	0			1	4	SC	SJ	0	0			8	2	FC	SJ	32,624	74.3	‡ #
220.4	WF	BADQVV	43.7	64.9	4	3	T	SJ	0	0			0	0			3	2	FC	SJ	24,856	17.0	‡
230.6	WR	PCSCGM	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	‡
232.6	WR	PCSCGM	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	‡
237.7	WR	PCSBFM	0.0	68.6	0	0			0	0			0	0			0	0			S2	0.0	‡
241.0	WR	PCSBFM	0.0	19.1	0	0			0	0			0	0			0	0			S2	0.0	‡
249.1	WR	PCSBFM	0.0	42.6	0	0			0	0			0	0			0	0			S2	0.0	
257.9	WR	PCSBFM	0.0	74.4	0	0			0	0			0	0			0	0			S2	0.0	‡
263.9	WS	SSS FM	40.5	83.3	4	2	T	SJ	0	0			0	0			1	2	T	SJ	2,641	28.3	‡
269.9	WS	SS AFM	18.8	23.8	1	2	T	SJ	0	0			0	0			0	0			932	18.8	‡

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-102

SITE:	MINER FLAT DAM SITE	NORTHING:	1,085,798.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	LEFT ABUTEMENT	EASTING:	576,853.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-102	ELEVATION:	6,078.40	START DEPTH:	0.00
START DATE:	01/21/86	INCLINATION:	77.9	LOGGED BY:	CHR
COMPLETION DATE:	01/30/86	BEARING:	319.3	PAGE:	2

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
281.0	WF	SS AFM	0.0	10.8	0	0			0	0			0	0			0	0			2,330	0.0	‡
291.0	WF	SSMAFR	85.8	91.5	0	0			0	0			0	0			0	0			777	85.8	‡
301.0	WF	SSMAFR	50.0	52.0	0	0			0	0			0	0			0	0			621	50.0	‡
311.0	WF	SSMAFR	100.0	100.0	0	0			0	0			0	0			0	0			1,165	100.0	‡ #
321.0	WF	SSM FR	38.2	71.7	5	4	FR	SJ	0	0			0	0			0	0			2,175	38.2	
331.0	WF	SSMCFR	40.3	54.4	8	2	T	SJ	3	2	T	SJ	0	0			0	0			699	36.2	‡
341.0	WF	SSM FM	44.2	67.5	4	2	T	SJ	0	0			0	0			1	2	T	SJ	854	44.2	‡
361.0	WF	SSMGFR	50.0	50.0	0	0			0	0			0	0			0	0			4,195	50.0	‡
371.0	WF	SSMGFR	97.5	100.0	0	0			0	0			0	0			0	0			1,787	97.5	‡ #
375.0	WF	SSMGFR	100.0	100.0	0	0			1	2	T	SJ	0	0			0	0			2,253	100.0	‡ #

DEPTH ---------- COMMENT ----------

= TOTAL CORE RECOVERY > 100%

8.0 BOREHOLE ON LEFT ABUTEMENT DRILLED ALONG AXIS OF PROPOSED DAM.
12.7 TOP OF ROCK AT DEPTH OF 4.5 FEET. FLOW BOUNDRY AT 9.9 FEET.
23.3 MEDIUM GRAY OLIVINE BASALT; SLIGHTLY VESICULAR TO MASSIVE.
31.2 FLOW BOUNDRY AT 20 FEET; LOST DRILLING FLUID CIRCULATION AT 20 FEET.
41.8 SCORACIOUS BASALT (FLOW BOUNDRY) AT 31.6 FEET.
51.0 MASSIVE; VERY FINE-GRAINED; PORPHYRITIC; OLIVINE BASALT.
61.0 FLOW BOUNDRY AT 54.2-55 FEET.
71.0 VESICULAR BASALT TO 65 FEET; MASSIVE BASALT BELOW 65 FEET.
81.0 FLOW BOUNDRY 73.7-75 FEET.
91.0 SCORACIOUS BASALT; FLOW BOUNDRY BELOW 83 FEET.
101.0 FLOW BOUNDRY; GRAVEL TO COBBLE SIZED PIECES OF SCORACIOUS BASALT WITH CLAY.
121.0 FLOW BOUNDRY TO 117.2 FEET; MASSIVE BASALT BELOW 119.5 FEET.
139.8 INTENSELY FRACTURED
150.1 MASSIVE BASALT
160.3 QUARTZ FILLING OF FLOW BANDING; FLOW BANDING AT 20 DEGREES TO CORE AXIS.
190.8 ONE PIECE OF CORE 10.1 FEET LONG.
200.9 FRACTURE BELOW 198.6 PARALLEL TO CORE AXIS REDUCES RQD&SCR.
211.0 SCORACIOUS FLOW BRECCIA 204-208.2 FEET.
220.4 MASSIVE BASALT TO 215 FEET; FLOW BRECCIA WITH CLAY MATRIX BELOW 215 FEET.

DEPTH	---------- COMMENT ----------
	# = TOTAL CORE RECOVERY > 100%
230.6	NO RECOVERY; ROCK TYPE BASED ON MATERIAL BELOW.
232.6	DRILLING INDICATES UNCONSOLIDATED MATERIAL. PALEO-COLLUVIUM
237.7	SANDSTONE COBBLES UP TO 0.5 FEET IN DIAMETER.
241.0	RECOVERED ONE SANDSTONE COBBLE 0.5 FEET IN DIAMETER.
257.9	SANDSTONE BOULDER 1.65 FEET IN DIAMETER.
263.9	PALEO-COLLUVIUM/SANDSTONE CONTACT APPROX. 259 FEET.
269.9	MODERATE YELLOWISH BROWN;SANDSTONE;FINE-GRAINED;SLIGHTLY CALCAREOUS.
281.0	VERY SOFT & POORLY INDURATED SANDSTONE. HYDRAULIC COMMUNICATION WITH DH-4.
291.0	MEDIUM REDDISH BROWN & PALE YELLOWISH ORANGE;MOTTLED SILTY SANDSTONE BRECCIA
301.0	WATER LEVEL AT 166.45 FEET AT 9:00 AM.
311.0	ONE PIECE OF CORE 10.2 FEET LONG.
331.0	CLAYEY SILTY SANDSTONE TO SANDY SILTSTONE.
341.0	MEDIUM REDDISH BROWN TO PALE BLUEISH GRAY SILTY SANDSTONE WITH CLAY CLASTS.
361.0	GYPSIFEROUS SANDSTONE AT 347.5; BRECCIATED
371.0	GYPSUM VEINLETS UP TO 0.03 FEET THICK.
375.0	END OF HOLE

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-105

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,058.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	RIGHT ABUTEMENT	EASTING:	576,636.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-105	ELEVATION:	6,078.40	START DEPTH:	0.00
START DATE:	11/23/85	INCLINATION:	72	LOGGED BY:	CHR
COMPLETION DATE:	01/08/86	BEARING:	140	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
5.8	WS	BAO VP	39.7	60.3	8	3	SI	SJ	0	0			0	0			4	3	SI	SJ	18,642	0.0	*
11.2	WS	BAO VP	61.1	100.0	9	3	SI	SJ	0	0			0	0			2	2	SI	SJ	18,642	20.2	
16.3	WF	BAO VP	95.9	98.4	2	3	T	SJ	1	3	SI	SJ	0	0			0	0			15,225	95.9	*
21.2	WS	BAO VV	63.7	100.0	6	6	FC	SJ	0	0			0	0			5	4	FC	SJ	21,749	0.0	* #
26.0	WF	BAO VV	62.3	87.5	8	3	SC	SJ	1	3	T	SJ	0	0			2	4	FC	SJ	15,225	26.0	
31.0	WF	BAO PV	75.0	98.6	5	3	SC	SJ	0	0			2	3	SC	SJ	2	3	SC	SJ	25,322	44.4	*
36.0	WF	BAO PV	100.0	100.0	0	0			0	0			0	0			2	3	SC	SJ	21,905	99.0	*
40.0	WF	BAO PV	85.0	100.0	4	4	FC	SJ	0	0			0	0			0	0			20,973	53.3	*
50.2	WF	BAO PV	50.1	99.0	8	3	FC	SJ	0	0			4	3	T	SJ	9	3	FC	SJ	25,322	19.1	*
59.5	WF	BAO PV	47.3	95.9	11	4	FC	SJ	0	0			5	3	SC	SJ	6	3	FC	SJ	23,148	17.5	*
68.0	WF	BAO PV	6.4	55.3	15	3	FC	SJ	0	0			5	3	FC	SJ	7	3	FC	SJ	18,953	0.0	*
75.2	WF	BAO PV	65.8	100.0	8	3	SC	SJ	0	0			7	3	FC	SJ	4	3	FC	SJ	20,196	47.9	*
85.0	WS	BAOFFP	49.5	93.4	12	3	FC	SJ	2	3	FC	SJ	4	3	FC	SJ	6	3	FC	SJ	18,642	34.0	*
92.5	WF	BAO FO	42.1	92.3	9	4	FC	SJ	4	3	FC	SJ	0	0			7	2	FC	SJ	12,894	17.3	*
102.8	WS	BAOFFB	73.8	98.5	12	4	FC	SJ	0	0			0	0			3	3	T	SJ	14,758	50.0	*
105.1	WF	BAOSVP	97.8	97.8	0	0			0	0			0	0			0	0			10,875	97.8	*
115.0	WF	BAOSPV	36.4	83.3	8	3	T	SJ	1	3	SC	SJ	0	0			3	5	FC	SJ	21,594	36.4	*
125.0	WS	BAO PV	26.2	65.0	11	4	SC	SJ	0	0			0	0			4	3	FC	SJ	18,642	25.8	*
135.0	WS	BAO PV	62.5	96.0	5	4	SC	SJ	8	3	SC	SJ	0	0			6	3	FC	SJ	15,535	10.0	*
145.0	WF	BAO VP	59.5	99.2	17	4	SC	FB	0	0			0	0			1	5	FC	SJ	15,535	40.0	*
155.0	WF	BAOQVM	73.5	100.0	12	3	SC	SJ	3	3	T	SJ	1	4	SC	SJ	1	3	T	SJ	23,303	60.0	* #
163.2	WF	BAOQVM	41.6	94.9	8	3	T	SJ	4	3	T	SJ	3	3	T	SJ	3	3	SC	SJ	24,080	27.1	
173.3	WF	BAOQVM	54.4	98.1	7	4	SC	SJ	7	3	T	SJ	0	0			6	3	SC	SJ	27,963	15.4	
183.5	WF	BAOQVM	15.7	98.8	18	3	T	SJ	5	4	T	SJ	16	3	T	SJ	13	3	SC	SJ	27,342	0.0	
193.8	WF	BAOQVM	31.3	97.1	10	4	T	SJ	9	4	T	SJ	4	3	T	SJ	9	3	SC	SJ	31,381	12.3	*
203.9	WF	BAOQVM	55.6	100.0	6	4	T	SJ	7	4	T	SJ	0	0			4	2	SC	SJ	28,585	36.7	
214.0	WF	BAOQVM	78.7	89.4	7	3	T	SJ	1	4	T	SJ	0	0			2	3	SC	SJ	31,071	39.6	
218.8	WF	BAOQVM	89.6	100.0	0	0			0	0			0	0			1	4	T	SJ	31,847	89.6	
229.1	WF	BAOQVM	84.8	97.9	5	5	T	SJ	2	3	T	SJ	0	0			4	2	T	SJ	32,624	69.0	
239.2	WF	BAOQVM	77.2	100.0	7	5	T	SJ	0	0			0	0			5	2	FC	SJ	20,973	45.5	* #
249.4	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			26,410	100.0	*

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-105

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,058.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	RIGHT ABUTEMENT	EASTING:	576,636.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-105	ELEVATION:	6,078.40	START DEPTH:	0.00
START DATE:	11/23/85	INCLINATION:	72	LOGGED BY:	CHR
COMPLETION DATE:	01/08/86	BEARING:	140	PAGE:	2

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
259.6	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			35,731	100.0	‡
269.9	WF	BAOQVM	61.7	98.3	0	0			0	0			0	0			2	5	FC	SJ	35,731	61.7	‡
280.0	WF	BAOQVM	100.0	100.0	2	3	T	SJ	0	0			0	0			1	3	FC	SJ	23,303	100.0	‡
285.0	WS	BAO VV	64.0	92.0	6	2	FC	SJ	5	3	FC	SJ	0	0			10	2	FC	SJ	14,914	0.0	‡
290.0	WF	PARBGB	0.0	50.0	0	0			0	0			0	0			0	0			S2	0.0	‡
295.0	WF	PARBGB	0.0	78.0	0	0			0	0			0	0			0	0			S2	0.0	‡
300.0	WF	PARBGB	0.0	62.0	0	0			0	0			0	0			0	0			S2	0.0	
303.3	WF	PARBGB	0.0	51.5	0	0			0	0			0	0			0	0			S2	0.0	
305.0	WM	SSMCFM	95.9	100.0	0	0			0	0			0	0			0	0			1,554	100.0	
310.0	WM	SSMSFM	32.6	90.0	0	0			0	0			9	2	T	SJ	0	0			777	22.0	‡
315.0	WM	SSMCFL	97.0	100.0	4	2	T	SJ	0	0			0	0			0	0			2,020	47.0	
320.0	WM	SSMCFL	100.0	100.0	2	2	T	SJ	0	0			0	0			0	0			1,398	100.0	#
325.0	WM	SSMCFL	92.0	96.6	1	4	T	SJ	0	0			0	0			0	0			1,398	32.0	
330.0	WM	SISCVL	57.4	73.6	0	0			2	3	T	SJ	0	0			0	0			854	57.4	
335.1	WM	SSMCFB	0.0	52.4	4	3	T	SJ	0	0			2	2	T	SJ	0	0			699	0.0	
337.7	WF	SSMCVB	78.8	94.2	0	0			0	0			2	2	T	SJ	3	1	FC	SJ	2,486	78.8	
340.0	WF	GYSGFM	100.0	100.0	2	2	T	SJ	0	0			0	0			0	0			5,282	0.0	#
345.0	WF	GYMSFM	100.0	100.0	0	0			0	0			0	0			0	0			6,447	100.0	‡ #
350.0	WF	SMGCVB	70.0	100.0	0	0			0	0			0	0			6	3	T	SJ	1,864	56.0	‡
355.0	WF	GYSGMM	100.0	100.0	0	0			0	0			0	0			0	0			5,204	100.0	‡
360.0	WF	SSSGFM	100.0	100.0	0	0			0	0			0	0			0	0			3,029	100.0	‡
365.0	WF	SSMGVB	100.0	100.0	0	0			0	0			0	0			0	0			1,476	100.0	‡
370.0	WF	SSMGVB	96.0	96.0	0	0			0	0			0	0			0	0			2,641	96.0	‡

DEPTH ---------- COMMENT ----------

= TOTAL CORE RECOVERY > 100%

5.8	TOP OF ROCK APPROXIMATELY .75 TO 1.0 FEET
16.3	INCREASE IN VESICLES. VESICLES UP TO .25X.15 FEET.
21.2	JOINTS AND SOME VESSICLES FILLED WITH SANDY-SILTY CLAY.
31.0	FILM OF CLAY ON FRACTURE SURFACES.
36.0	MASSIVE BASALT
40.0	FLOW BOUNDRY 38.1 TO 40.0 FEET.

DEPTH	---------- COMMENT ----------
	# = TOTAL CORE RECOVERY > 100%
50.2	MASSIVE BASALT 42-48;FLOW BOUNDRY 48-50.2
59.5	VESSICULAR TO 57.5;FLOW BOUNDRY AT 57.5 WITH SANDY CLAY FILLED FRACTURES.
68.0	POOR RECOVERY/ASSUMED FLOW BOUNDRY
75.2	FLOW BOUNDRY AT 72 FEET
85.0	SLIGHTLY WEATHERED;FLOW BOUNDRY AT 80; 100% WATER LOSS AT 82-83.
92.5	PALEO-SOL AT 90 FEET; 0.25 FEET RECOVERED.
102.8	FLOW BANDING 10 DEGREES TO CORE AXIS AT 99 FEET.
105.1	STRENGTH MEASURED PARALLEL TO FLOW BANDING.
115.0	1 CLAY FILLED VERTICAL FRACTURE 108-110.3 FEET.
125.0	100% CIRCULATION LOSS AT 116.5; LAVA TUBE 122-125 FEET.
135.0	VESICULAR BASALT
145.0	FLOW BANDED BASALT 20 DEGREES TO CORE AXIS;VESICULAR TO "ROPEY TEXTURE"
155.0	FLOW BANDED "ROPEY TEXTURE" TO 147 FEET; 147 MASSIVE BASALT.
193.8	MASSIVE BASALT; HARD TO VERY HARD.
239.2	OLIIVE GREEN "TALC LIKE" FRACTURE FILLING.
249.4	ONE PIECE OF CORE 10.2 FEET LONG.
259.6	ONE PIECE OF CORE 10.2 FEET LONG.
269.9	1 VERTICAL FRACTURE 260.8-269.9 FEET.
280.0	CONTACT WITH FLOW BOUNDRY BRECCIA AT 279.9 FEET.
285.0	SCORACIOUS BASALT FRAGMENTS WITH CLAY MATRIX ; BRECCIA
290.0	PALEO-ALLUVIUM WITH LIMESTONE;QTZITE;CHERT;GRANITIC GRAVEL AND COBBLES.
295.0	BASALT BOULDER 293.5-295 FEET.
303.3	CONTACT WITH SUPAI SANDSTONE AT 303.3
305.0	MEDIUM REDDISH ORANGE AND PALE ORANGISH YELLOW SILTY SANDSTONE.
315.0	MEDIUM REDDISH ORANGE AND PALE ORANGISH YELLOW; MOTTLED; SILTY SANDSTONE .
325.0	POORLY INDURATED
330.0	MEDIUM REDDISH ORANGE SANDY SILTSTONE; POORLY INDURATED.
337.7	GYPSIFEROUS SANDSTONE BRECCIA.
340.0	CONTACT WITH GYPSUM AT 337.7
345.0	SILTY SANDSTONE INTERBED 343.4-344.4
350.0	GYPSIFEROUS SANDY SILTSTONE
355.0	SOLID GYPSUM
360.0	GYPSIFEROUS SANDSTONE
365.0	GYPSIFEROUS SILTY SANDSTONE.
370.0	END OF HOLE

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-106

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,059.60	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	RIGHT ABUTEMENT	EASTING:	576,634.60	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-106	ELEVATION:	6,073.40	START DEPTH:	0.00
START DATE:	11/13/86	INCLINATION:	89.4	LOGGED BY:	CHR
COMPLETION DATE:	12/07/85	BEARING:	246.9	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM	
5.0	WF	BAO PV	16.4	27.0	3	2	SI	SJ	0	0			0	0			2	3	SI	SJ	16,467	0.0	†	
8.9	WF	BAO PV	24.4	100.0	4	3	T	SJ	0	0			4	4	SI	SJ	3	3	FQ	SJ	20,973	0.0	†	#
14.0	WF	BAO PV	98.0	100.0	0	0			0	0			0	0			2	4	SC	SJ	17,866	98.0	†	#
19.0	WF	BAO AV	74.6	94.0	6	5	SC	SJ	0	0			0	0			3	4	FC	SJ	10,875	62.6	†	
24.1	WS	BAO AV	57.8	91.6	8	8	FC	SJ	0	0			4	6	SC	SJ	3	4	FC	SJ	17,089	49.4	†	
29.2	WF	BAO VP	100.0	100.0	1	1	SC	SJ	0	0			0	0			1	2	FC	SJ	22,915	93.1	†	#
34.2	WF	BAO VP	52.6	98.0	5	4	SI	SJ	0	0			3	4	SC	SJ	3	6	FC	SJ	26,876	25.4	†	
39.2	WF	BAO VP	92.0	100.0	2	3	FC	SJ	1	3	T	SJ	0	0			1	6	FC	SJ	21,283	40.0		#
44.4	WF	BAO VP	60.4	97.5	7	5	FC	SJ	0	0			3	4	SC	SJ	1	6	SC	SJ	24,856	24.2		
49.4	WF	BAO VP	92.0	100.0	5	8	FC	SJ	2	4	T	SJ	0	0			0	0			21,749	58.0		#
54.3	WF	BAOQVM	100.0	100.0	1	3	SC	SJ	0	0			0	0			1	2	SI	SJ	26,876	100.0		#
59.4	WF	BAOQVM	95.5	97.5	0	0			0	0			1	3	SI	SJ	0	0			22,060	95.5	†	
64.4	WF	BAO VP	92.0	100.0	0	0			0	0			0	0			3	3	FC	SJ	27,187	92.0	†	#
69.5	WF	BAO VP	33.3	92.7	8	5	FC	SJ	0	0			5	4	T	SJ	6	4	FC	SJ	19,574	24.5		
74.6	WF	BAO VP	68.6	93.1	4	4	FC	SJ	0	0			0	0			2	4	FC	SJ	17,866	61.8	†	
79.4	WF	BAO VP	70.2	100.0	7	4	SC	SJ	0	0			3	6	T	SJ	2	3	FC	FB	22,215	37.1	†	#
84.7	WF	BAO AV	76.4	94.9	8	3	FC	SJ	3	4	FC	SJ	0	0			0	0			20,507	46.6	†	
89.8	WF	BAO AV	59.8	92.7	10	4	FC	FB	5	3	FC	SJ	0	0			5	2	FC	SJ	18,021	0.0	†	
94.9	WF	BAO AV	66.9	100.0	7	3	SC	SJ	2	4	SC	SJ	0	0			4	3	FC	SJ	22,526	55.5		
100.0	WF	BAOQVA	68.6	98.2	4	6	SC	SJ	0	0			0	0			1	5	FC	SJ	18,798	50.4	†	
104.9	WF	BAOQVP	98.8	100.0	3	4	SC	SJ	0	0			0	0			0	0			17,089	96.3		#
110.0	WF	BAOQVP	100.0	100.0	0	0			0	0			0	0			0	0			27,653	100.0		
115.1	WF	BAOQVP	81.0	99.0	3	3	FC	SJ	0	0			0	0			1	3	FC	SJ	23,303	72.5		
120.1	WF	BAOQVP	85.0	95.0	4	4	SC	SJ	1	2	SC	SJ	0	0			2	4	FC	SJ	18,642	34.2	†	
125.4	WF	BAO VP	0.0	25.5	11	3	FC	SJ	0	0			0	0			3	3	FC	SJ	17,866	0.0	†	
129.9	WF	BAO AV	89.6	98.4	3	4	FC	SJ	0	0			0	0			0	0			22,526	74.7		
134.8	WF	BAO AV	100.0	100.0	3	4	SC	SJ	0	0			0	0			1	3	T	SJ	13,205	92.2		#
140.0	WF	BAO AV	66.7	93.1	11	3	T	FB	0	0			0	0			1	4	FC	SJ	12,117	0.0	†	
145.0	WS	BAO AV	65.4	100.0	12	4	SC	FB	0	0			0	0			3	4	FC	SJ	7,768	0.0	†	
150.1	WF	BAOQAV	86.3	99.0	4	6	SC	FB	0	0			0	0			2	5	T	SJ	17,399	40.6	†	
155.0	WF	BAOQAV	58.2	100.0	7	5	SC	SJ	0	0			3	4	T	SJ	2	4	T	SJ	23,303	0.0		#

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-106

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,059.60	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	RIGHT ABUTEMENT	EASTING:	576,634.60	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-106	ELEVATION:	6,073.40	START DEPTH:	0.00
START DATE:	11/13/85	INCLINATION:	89.4	LOGGED BY:	CHR
COMPLETION DATE:	12/07/85	BEARING:	246.9	PAGE:	2

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
160.0	WF	BAQQVP	3.0	3.0	3	4	T	SJ	0	0			0	0			0	0			22,060	3.0	
165.0	WF	BAQ VP	28.0	100.0	7	4	T	SJ	0	0			0	0			5	5	SC	SJ	26,876	0.0	‡
170.0	WF	BAQ VP	23.4	100.0	9	3	T	SJ	5	3	T	SJ	0	0			7	5	SC	SJ	26,565	0.0	
175.1	WF	BAQ VP	53.9	100.0	8	4	T	SJ	0	0			0	0			4	5	FC	SJ	34,954	19.6	#
180.0	WF	BAQ VP	43.9	100.0	11	3	T	SJ	6	3	T	SJ	0	0			3	5	FC	SJ	25,167	0.0	#
185.0	WF	BAQ VP	45.4	100.0	5	4	T	SJ	4	3	SC	SJ	0	0			4	4	SC	SJ	20,196	31.4	
190.0	WF	BAQQVP	54.0	100.0	5	4	T	SJ	0	0			4	4	T	SJ	7	4	T	SJ	27,963	21.8	
195.0	WF	BAQQVP	55.0	99.0	3	3	T	SJ	5	4	SC	SJ	0	0			3	4	SC	SJ	27,497	20.0	
200.0	WF	BAQQVP	76.6	100.0	0	0			0	0			2	4	T	SJ	1	3	FC	SJ	32,779	65.0	‡ #
204.9	WF	BAQQVP	100.0	100.0	0	0			0	0			0	0			0	0			30,138	100.0	#
210.0	WF	BAQQVM	98.6	98.6	0	0			0	0			0	0			0	0			32,158	98.6	
215.0	WF	BAQQVM	100.0	100.0	0	0			0	0			0	0			0	0			31,226	100.0	
220.0	WF	BAQQVM	98.8	99.6	0	0			0	0			0	0			0	0			29,828	98.8	
225.0	WF	BAQQVM	8.8	100.0	0	0			0	0			0	0			1	4	T	SJ	31,226	88.0	#
230.0	WF	BAQQVM	100.0	100.0	1	3	T	SJ	0	0			0	0			0	0			30,138	100.0	#
235.0	WF	BAQQVM	100.0	100.0	1	3	T	SJ	0	0			0	0			0	0			29,828	100.0	
240.0	WF	BAQQVM	99.0	99.4	0	0			0	0			0	0			0	0			29,517	99.0	
245.0	WF	BAQQVM	100.0	100.0	0	0			0	0			0	0			0	0			29,517	100.0	#
250.0	WF	BAQQVM	100.0	100.0	0	0			1	3	T	SJ	0	0			0	0			32,003	100.0	
255.0	WF	BAQQVM	80.0	100.0	2	4	T	SJ	0	0			1	4	FQ	SJ	0	0			31,692	80.0	‡
260.0	WF	BA GR	19.4	80.0	6	4	FC	SJ	0	0			0	0			5	3	FC	SJ	2,796	0.0	‡
263.4	WF	BA GR	0.0	58.8	10	3	FC	FB	6	3	FC	FB	0	0			0	0			R2	0.0	‡
267.4	WF	PAS CB	0.0	66.3	0	0			0	0			0	0			0	0			S2	0.0	‡
270.9	WF	PAMSCB	0.0	48.6	0	0			0	0			0	0			0	0			S2	0.0	
275.9	WF	PAMSCB	0.0	64.0	0	0			0	0			0	0			0	0			S2	0.0	
279.1	WF	PASCBB	0.0	53.1	0	0			0	0			0	0			0	0			S2	0.0	
284.3	WF	PASCBB	0.0	32.7	0	0			0	0			0	0			0	0			S2	0.0	
288.8	WF	PASCBB	0.0	70.0	0	0			0	0			0	0			0	0			S2	0.0	
294.7	WF	PASCBB	0.0	16.9	0	0			0	0			0	0			0	0			S2	0.0	
297.9	WF	PASCBB	0.0	46.9	0	0			0	0			0	0			0	0			S2	0.0	
301.0	WF	PASCBB	0.0	54.8	0	0			0	0			0	0			0	0			S2	0.0	

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-106

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,059.60	DRILLING METHOD:	NB3 TRIPLE TUBE CORE
LOCATION:	RIGHT ABUTEMENT	EASTING:	576,634.60	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-106	ELEVATION:	6,073.40	START DEPTH:	0.00
START DATE:	11/13/85	INCLINATION:	89.4	LOGGED BY:	CHR
COMPLETION DATE:	12/07/85	BEARING:	246.9	PAGE:	3

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
311.8	WM	SBMCFB	98.0	98.0	1	2	T	SJ	0	0			1	2	T	SJ	0	0			2,641	87.0	
317.0	WM	SBMCFB	68.8	87.5	0	0			0	0			3	2	T	SJ	2	2	T	SJ	932	27.5	
322.0	WM	SISCVB	92.2	100.0	0	0			0	0			0	0			3	3	T	SJ	1,709	61.0	#
327.0	WM	SBMCVB	57.4	98.2	0	0			2	3	FO	SJ	0	0			0	0			7,302	57.4	‡
330.6	WM	SBM VB	100.0	100.0	0	0			0	0			7	2	T	SJ	0	0			1,329	100.0	#
335.9	WF	GY VM	86.7	86.7	3	2	T	BD	0	0			0	0			0	0			5,437	86.7	‡
341.0	WF	SBMGFB	86.5	98.1	5	2	T	BD	0	0			0	0			0	0			2,020	75.0	‡
346.0	WF	SBMCFB	95.0	100.0	0	0			6	2	T	SJ	0	0			0	0			2,409	95.0	‡
351.0	WF	SBMGFB	68.0	100.0	5	2	T	SJ	0	0			0	0			5	6	FO	SJ	8,078	43.0	‡ #
352.8	WF	SBB FB	61.1	100.0	0	0			0	0			5	3	T	SJ	0	0			3,729	0.0	#
355.0	WF	GY FM	100.0	100.0	0	0			0	0			0	0			0	0			4,505	100.0	‡ #

DEPTH	COMMENT
	# = TOTAL CORE RECOVERY > 100%
5.0	0-3.5 FEET COLLUVIUM
8.9	8.4-8.6 FEET VERTICAL FRACTURE
14.0	ONE PIECE OF CORE 4.3 FEET LONG
18.0	15.0 FEET VESICULAR
24.1	20 FEET FLOW BOUNDRY; VERY VESICULAR WITH SILICA FILLINGS
29.2	DECREASE IN THE NUMBER OF VESICLES WITH DEPTH
34.2	30.1-33.0 FEET VERTICAL FRACTURE
59.4	MASSIVE; 4.21 FEET STICK OF CORE
64.4	62 FEET FLOW BOUNDRY
74.8	74 FEET FLOW BOUNDRY
79.4	76-77 FEET SLIGHT WEATHERING: FLOW BOUNDRY
84.7	80.1 FEET FLOW BOUNDRY WITH SILICA FILLED VESICLES
89.9	FLOW BOUNDRY
100.0	VERTICAL FRACTURE 1.3 FEET LONG
100.0	VERTICAL FRACTURE 1.3 FEET LONG
120.1	119.7 FEET 100% WATER LOSS
125.4	119.7-121.5 FEET OPEN LAVA TUBE
142.0	STRENGTH PARALLEL TO FLOW BANDING

DEPTH	---------- COMMENT ----------
	# = TOTAL CORE RECOVERY > 100%
145.0	STRENGTH TO FLOW BANDING; FLOW BOUNDRIES
150.1	147.2 FEET BOTTOM OF BASALT FLOW
195.0	DRILLING FLUID COMING UP DH-1
20[illegible].0	1 FRACTURE @ 20 DEGREES 1/4 INCH THICK AND FILLED WITH CLAY
255.0	QUARTZ FILLED FRACTURE AT 60 DEGREES
260.0	253 FEET FLOW BRECCIA
263.4	FLOW BOUNDRY
267.4	PALEO- ALLUVIUM
302.0	CONTACT WITH SILTY SANDSTONE
327.0	330.1
335.8	330.9 CONTACT WITH GYPSUM
341.0	GYPSIFEROUS SANDSTONE
346.0	GYPSUM INCLUSIONS (INTRUSIONS ?)
351.0	SANDSTONE BRECCIA HEALED WITH GYPSUM
355.0	END OF HOLE

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-113

SITE:	MINER FLAT DAM SITE	NORTHING:	0	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	RIGHT ABUTEMENT	EASTING:	0	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-113	ELEVATION:	5,920.50	START DEPTH:	0.00
START DATE:	02/25/86	INCLINATION:	89.7	LOGGED BY:	CHR
COMPLETION DATE:	02/27/86	BEARING:	211.1	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
9.1	WR	FISBRM	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	*
13.4	WS	BAO VM	30.2	100.0	6	4	T	SJ	0	0			0	0			3	5	SC	SJ	30,294	0.0	*
14.3	WS	BAO VM	0.0	100.0	2	5	T	SJ	1	4	T	SJ	0	0			1	4	T	SJ	31,071	0.0	
19.5	WS	BAOQVM	16.3	99.6	12	4	T	SJ	3	4	T	SJ	0	0			5	5	T	SJ	32,313	0.0	*
24.5	WS	BAO VM	25.4	100.0	8	4	T	SJ	5	4	SC	SJ	0	0			6	4	SC	SJ	37,285	0.0	
29.6	WS	BAO VM	8.4	96.3	5	4	T	SJ	10	4	SC	SJ	0	0			6	4	SC	SJ	32,624	0.0	*
34.6	WF	BAO VM	28.0	100.0	3	5	T	SJ	8	4	SC	SJ	0	0			7	4	SC	SJ	31,071	0.0	
39.7	WF	BAOQVM	62.2	98.0	0	0			6	4	T	SJ	0	0			3	4	T	SJ	32,313	52.9	*
44.7	WF	BAOQVM	74.2	80.0	4	4	T	SJ	0	0			0	0			3	4	T	SJ	32,003	42.2	*
54.8	WF	BAOQVM	43.6	50.3	7	3	T	SJ	0	0			0	0			0	0			31,071	30.4	
59.9	WF	BAOQVM	100.0	100.0	7	3	T	SJ	0	0			0	0			0	0			36,352	0.0	
65.0	WF	BAOQVM	88.2	100.0	2	3	T	SJ	3	3	T	SJ	0	0			2	3	T	SJ	32,624	65.9	
70.0	WF	BAOQVM	84.0	98.0	5	3	T	SJ	0	0			0	0			0	0			30,760	37.0	
75.1	WF	BAOQVM	84.7	100.0	6	4	T	SJ	1	3	T	SJ	0	0			0	0			30,760	29.0	
80.0	WF	BAOQVM	91.8	100.0	4	3	T	SJ	0	0			0	0			0	0			27,963	68.6	
85.0	WF	BAOQVM	93.0	100.0	3	2	T	SJ	0	0			0	0			0	0			30,294	70.0	
90.0	WF	BAOQVM	98.0	100.0	1	2	T	SJ	2	2	T	SJ	0	0			0	0			34,799	68.0	
95.0	WF	BAOQVM	100.0	100.0	4	2	T	SJ	0	0			0	0			0	0			35,265	63.6	#
100.0	WF	BAOQVM	100.0	100.0	1	2	T	SJ	0	0			0	0			0	0			33,401	100.0	*
105.0	WF	BAOQVM	100.0	100.0	3	2	T	SJ	0	0			0	0			0	0			32,624	89.0	
110.0	WF	BAOQVM	91.4	100.0	0	0			2	2	T	SJ	0	0			0	0			41,168	91.4	*
115.0	WF	BAOQVM	64.0	99.0	8	2	T	SJ	0	0			0	0			0	0			34,178	33.4	*
120.0	WM	BAO VS	0.0	50.0	0	0			0	0			0	0			0	0			R2	0.0	*
125.0	WR	PAGRRB	0.0	67.4	0	0			0	0			0	0			0	0			S2	0.0	*
126.4	WR	PAGRRB	0.0	92.9	0	0			0	0			0	0			0	0			S2	0.0	*
129.3	WR	PAGRRB	0.0	51.7	0	0			0	0			0	0			0	0			S2	0.0	*
135.2	WR	PAGRRB	0.0	37.3	0	0			0	0			0	0			0	0			S2	0.0	*
140.0	WS	SSMSFM	47.9	89.6	5	4	T	SJ	4	3	T	SJ	0	0			3	3	T	SJ	5,437	47.9	*
145.0	WS	SSMCVR	72.6	83.0	1	2	T	SJ	1	2	T	SJ	0	0			0	0			1,243	72.6	*
150.0	WS	SSMCVR	0.0	0.0	0	0			0	0			0	0			0	0			R2	0.0	*
155.0	WS	SSMSFR	21.4	100.0	13	2	T	SJ	0	0			0	0			5	2	T	SJ	1,476	0.0	*

DEPTH	---------- COMMENT ----------
	# = TOTAL CORE RECOVERY > 100%
9.1	BOREHOLE AT RIVER LEVEL AT RIGHT ABUTEMENT. 0-9.1 FILL USED TO BUILD PAD.
13.4	MEDIUM GRAY OLIVINE BASALT; PORPHYRITIC; SLIGHTLY MAGNETIC.
19.5	SLIGHT CLAY STAINING ON FRACTURE SURFACES.
29.6	BASALT IS SLIGHTLY WEATHERED AND MASSIVE WITH NO VESSICLES.
39.7	OLIVINE BASALT WITH QUARTZ; NO STAINING OR ALTERATION OF JOINT SURFACES.
44.7	DEPTH OF WEATHERING ALONG JOINTS APPROX. 35 FEET.
100.0	ONE FRACTURE 90 DEGREES TO CORE AXIS.
110.0	NOTE POINT LOAD TEST VALUE! VERY HIGH
115.0	VESICULAR TO SCORACIOUS BASALT (FLOW BOUNDRY) AT 114.7 FEET.
120.0	SCORACIOUS BASALT TO 117 FEET CONTACT WITH PALEO-ALLUVIUM.
125.0	PALEO-ALLUVIUM; SAND;GRAVEL;COBBLES&BOULDERS OF BASALT;SANDSTONE;LIMESTONE.
126.4	PALEO-ALLUVIUM WITH CHERT AND GRANITIC COBBLES; BASALT BOULDERS TO 1.0 FEET.
129.3	PALEO-ALLUVIUM WITH QUARTZITE GRAVEL AND COBBLES.
135.2	PALEO-ALLUVIUM/SANDSTONE CONTACT AT APPROX. 134 FEET.
140.0	WHITE TO REDDISH BROWN MOTTLED SILTY SANDSTONE; FINE-GRAINED.
145.0	REDDISH BROWN SILTY CLAYEY SANDSTONE BRECCIA WITH MNOX STAINED FRAGMENTS.
150.0	NO RECOVERY ROCK TYPE ASSUMED FROM PREVIOUS RUN.
155.0	REDDISH BROWN SILTY SANDSTONE BRECCIA; END OF HOLE.

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-117

SITE:	MINER FLAT DAM SITE	NORTHING:	0	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	RIGHT ABUTEMENT	EASTING:	0	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-117	ELEVATION:	5,924.50	START DEPTH:	0.00
START DATE:	03/06/86	INCLINATION:	26	LOGGED BY:	CHR
COMPLETION DATE:	03/09/86	BEARING:	309.5	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	RGH	ALT	TYPE	70 - 50 NO	RGH	ALT	TYPE	50 - 30 NO	RGH	ALT	TYPE	30 - 0 NO	RGH	ALT	TYPE	STRENGTH (psi)	SCR (%)	CM
2.8	WS	BAOQVM	64.3	100.0	8	4	SC	SJ	0	0			0	0			2	3	SC	SJ	26,410	0.0	‡
3.3	WS	BAOQVM	90.0	90.0	0	0			0	0			0	0			0	0			24,856	0.0	‡
8.5	WS	BAOQVM	75.2	100.0	4	4	SC	SJ	0	0			0	0			4	3	SC	SJ	25,633	19.8	‡ #
13.2	WS	BAOQVM	59.6	100.0	4	3	SC	SJ	4	3	SC	SJ	0	0			5	3	SC	SJ	24,856	23.8	#
18.2	WS	BAOQVM	41.6	100.0	6	3	SC	SJ	7	3	SC	SJ	0	0			4	3	SC	SJ	27,963	0.0	#
23.4	WS	BAOQVM	19.4	100.0	9	3	SC	SJ	7	3	SC	SJ	0	0			6	3	SC	SJ	30,760	0.0	#
28.0	WS	BAOQVM	28.3	100.0	10	4	SC	SJ	3	4	SC	SJ	0	0			9	4	SC	SJ	30,294	0.0	#
32.8	WS	BAOQVM	0.0	100.0	18	3	T	SJ	10	3	T	SJ	0	0			12	3	SC	SJ	33,245	0.0	‡
37.5	WF	BAOQVM	12.6	92.6	12	3	T	SJ	5	3	SC	SJ	0	0			5	3	SC	SJ	33,401	0.0	
41.9	WF	BAOQVM	26.1	100.0	9	3	T	SJ	7	3	SC	SJ	0	0			7	3	SC	SJ	35,420	0.0	#
48.3	WF	BAOQVM	14.5	100.0	12	4	T	SJ	6	4	SC	SJ	0	0			10	4	SC	SJ	35,420	0.0	
58.2	WF	BAOQVM	21.2	100.0	16	3	SC	SJ	13	3	SC	SJ	0	0			9	3	SC	SJ	34,488	0.0	
67.6	WF	BAOQVM	22.6	97.3	14	3	T	SJ	20	3	SC	SJ	0	0			11	3	SC	SJ	34,954	0.0	
77.6	WF	BAOQVM	50.5	99.5	7	3	SC	SJ	9	3	SC	SJ	0	0			5	3	SC	SJ	35,731	23.5	
87.6	WF	BAOQVM	52.1	99.5	8	3	SC	SJ	17	3	SC	SJ	0	0			5	3	SC	SJ	35,731	0.0	‡
97.4	WF	BAOQVM	21.6	100.0	14	3	T	SJ	19	3	SC	SJ	0	0			9	3	SC	SJ	33,245	0.0	#
105.0	WF	BAOQVM	30.5	100.0	10	3	SC	SJ	12	3	SI	SJ	0	0			5	3	SC	SJ	35,110	0.0	‡ #

DEPTH ---------- COMMENT ----------

\# = TOTAL CORE RECOVERY > 100%

2.8 BOREHOLE DRILLED TO CROSS AXIS OF DAM APPROX. 30 FEET INTO ABUTEMENT.

3.3 MEDIUM GRAY OLIVINE BASALT WITH QUARTZ;PORPHYRTIC WITH OLIVINE PHENOCRYSTS.

8.5 SLIGHTLY MAGNETIC

32.8 INTENSELY FRACTURED; ALL PIECES SMALLER THAN 0.35 FEET.

87.6 OCCASIONAL FLOW BANDING 20 DEGREES TO CORE AXIS (IE.HORIZONTAL).

105.0 END OF HOLE; HOLE MAKES 6 GALLONS PER MINUTE AFTER HOLE COMPLETION.

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-118

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,321.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	PALEO-CHANNEL	EASTING:	576,417.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-118	ELEVATION:	6,077.00	START DEPTH:	0.00
START DATE:	1/11/86	INCLINATION:	89.5	LOGGED BY:	CHR
COMPLETION DATE:	1/15/86	BEARING:	243.8	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
8.7	WR	COMCVM	0.0	55.7	0	0			0	0			0	0			0	0			S3	0.0	*
10.0	WS	BAO VV	69.2	100.0	4	6	T	SJ	0	0			0	0			0	0			9,321	0.0	*
19.7	WS	BAO VV	65.2	100.0	10	4	T	SJ	4	3	T	SJ	0	0			4	3	T	SJ	7,768	33.9	*
29.9	WF	BAO VV	63.7	84.9	11	4	T	SJ	2	3	FC	SJ	0	0			5	3	FC	SJ	21,439	39.2	*
40.3	WS	BAO VV	74.0	98.6	10	4	T	SJ	0	0			0	0			4	3	FC	SJ	13,982	42.8	*
48.2	WF	BAO VV	52.4	92.8	10	3	FC	SJ	0	0			0	0			4	2	FC	SJ	21,749	33.7	*
58.8	WS	BAO VM	58.5	94.5	8	4	SC	SJ	4	3	SC	SJ	0	0			4	4	SC	SJ	26,410	32.5	
69.0	WF	BAOQVM	89.4	100.0	4	3	SC	SJ	1	3	T	SJ	0	0			1	3	FC	SJ	24,701	83.0	*
79.4	WF	BAOQVM	67.3	100.0	8	4	SC	SJ	2	6	T	SJ	0	0			2	2	FC	SJ	20,196	52.4	*
89.8	WF	BAOQVM	55.2	93.3	9	3	SC	SJ	6	3	FC	SJ	0	0			4	2	FC	SJ	10,409	0.0	*
100.1	WS	BAOQVV	13.9	66.7	8	4	SC	SJ	8	3	FC	SJ	0	0			10	2	FC	SJ	10,875	10.1	*
110.7	WF	BAOQVV	64.3	91.8	6	3	SC	SJ	2	4	FC	SJ	0	0			8	3	FC	SJ	17,866	54.2	*
121.1	WF	BAOQVM	39.4	95.4	11	4	FC	SJ	2	3	SC	SJ	0	0			5	4	FC	SJ	22,992	27.9	*
131.0	WM	BAO VV	55.9	92.6	6	4	SC	SJ	3	3	SC	SJ	0	0			9	3	FC	SJ	16,467	37.6	*
141.0	WF	BAO VV	75.5	100.0	8	4	T	SJ	3	3	T	SJ	0	0			3	3	SC	SJ	13,516	56.5	*
151.0	WF	BAOQVM	79.2	100.0	11	4	T	SJ	1	4	T	SJ	0	0			1	3	T	SJ	9,321	63.3	* #
161.0	WF	BAOQVM	54.0	100.0	7	3	T	SJ	1	3	T	SJ	0	0			4	4	T	SJ	24,235	49.1	*
171.0	WF	BAOQVM	61.4	100.0	6	4	T	SJ	4	4	T	SJ	0	0			3	4	FC	SJ	24,080	31.0	
181.0	WF	BAOQVM	71.5	100.0	12	4	SC	SJ	2	6	SC	SJ	0	0			3	4	FC	SJ	37,285	53.3	
191.0	WF	BAOQVM	44.0	100.0	8	3	T	SJ	9	4	T	SJ	0	0			5	3	FC	SJ	37,595	25.4	
201.0	WF	BAOQVM	62.7	100.0	7	3	T	SJ	3	3	T	SJ	0	0			6	3	SC	SJ	35,731	36.8	
211.0	WF	BAOQVM	100.0	100.0	5	3	T	SJ	0	0			0	0			0	0			38,838	84.5	*
221.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			41,168	100.0	
231.0	WF	BAOQVM	25.0	100.0	0	0			0	0			0	0			1	4	FC	SJ	34,954	25.0	*
241.0	WF	BAOQVM	74.0	100.0	0	0			1	3	FC	SJ	0	0			2	4	T	SJ	33,090	74.0	
251.0	WF	BAOQVM	99.0	100.0	0	0			0	0			0	0			1	3	T	SJ	36,508	99.0	*
261.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			1	4	T	SJ	37,595	100.0	#
271.0	WF	BAOQVV	42.3	60.0	5	3	SC	SJ	0	0			0	0			4	3	FC	SJ	27,187	34.2	*
278.0	WR	PASGFB	0.0	50.0	0	0			0	0			0	0			0	0			S2	0.0	*
279.4	WR	PASGFB	0.0	35.7	0	0			0	0			0	0			0	0			S2	0.0	
285.1	WR	PASGFB	0.0	67.2	0	0			0	0			0	0			0	0			S2	0.0	*

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-118

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,321.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	PALEO-CHANNEL	EASTING:	576,417.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-118	ELEVATION:	6,077.00	START DEPTH:	0.00
START DATE:	1/11/86	INCLINATION:	89.5	LOGGED BY:	CHR
COMPLETION DATE:	1/15/86	BEARING:	243.8	PAGE:	2

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
288.3	WR	PASGFB	0.0	89.1	0	0			0	0			0	0			0	0			S2	0.0	
289.1	WR	PASGFB	0.0	87.5	0	0			0	0			0	0			0	0			S2	0.0	
290.2	WR	PASGFB	0.0	45.5	0	0			0	0			0	0			0	0			S2	0.0	
295.6	WR	PASGFB	0.0	46.3	0	0			0	0			0	0			0	0			S2	0.0	
297.9	WR	PASGFB	0.0	7.4	0	0			0	0			0	0			0	0			S2	0.0	‡
303.4	WR	PASGFB	0.0	70.0	0	0			0	0			0	0			0	0			S2	0.0	‡
308.4	WR	PASGFB	0.0	80.0	0	0			0	0			0	0			0	0			S3	0.0	‡
309.7	WR	PASGFB	0.0	53.8	0	0			0	0			0	0			0	0			S2	0.0	
314.0	WR	PASGFB	0.0	74.4	0	0			0	0			0	0			0	0			S2	0.0	
319.3	WR	PASGFB	0.0	67.9	0	0			0	0			0	0			0	0			S3	0.0	‡
324.2	WS	SSMCFM	100.0	100.0	0	0			0	0			0	0			0	0			777	100.0	‡ #
329.5	WF	SSMCFR	33.0	60.4	4	3	T	SJ	1	2	T	SJ	0	0			0	0			1,243	33.0	‡
334.7	WF	SSMSFR	0.0	51.9	5	3	T	SJ	0	0			0	0			3	3	T	SJ	3,418	0.0	
340.0	WF	SSMSFR	0.0	0.0	0	0			0	0			0	0			0	0			R2	0.0	‡

DEPTH ---------- COMMENT ----------

= TOTAL CORE RECOVERY > 100%

8.7 BOREHOLE DRILLED TO INVESTIGATE PALEO-CHANNEL; SILTY CLAY WITH SOME SAND.
10.0 COLLUVIUM/BASALT CONTACT AT 8.7 FEET.
19.7 DARK GRAY TO BLACK OLIVINE BASALT; SLIGHTLY WEATHERED; VESICULAR.
29.9 100% LOSS OF DRILLING FLUID CIRCULATION AT 14 FEET. FLOW BOUNDRIES AT 14.2
40.3 FLOW BOUNDRIES AT 18.8 & 27.5-28 & PALEOSOL 38.1-38.3 & 40.1-40.3.
48.2 CLAY FILLED FRACTURES AT 20 DEGREES TO CORE AXIS.
69.0 FEW VESICLES; SOME FILLED WITH QUARTZ AND/OR CLAY.
79.4 69-70 FEET NEARLY VERTICAL FRACTURE FILLED WITH CLAY 0.02 FEET THICK.
89.8 MULTIPLE FLOW BOUNDRIES; VESICULAR BASALT WITH CLAY AND QUARTZ FILLING.
100.1 CLAY PALEOSOL AT 90.3.
110.7 PALEOSOL AT 104 FEET.
121.1 PALEOSOL AT 112 FEET.
131.0 VISICULAR AND FLOW BANDED; MODERATELY WEATHERED.
141.0 DEGREE OF WEATHERING DECREASES.
151.0 FLOW BANDING TO 147 FEET. STRENGTH ACROSS FLOW BANDING.

DEPTH	---------- COMMENT ----------
	# = TOTAL CORE RECOVERY > 100%
161.0	MEDIUM GRAY OLIVINE BASALT WITH QUARTZ; MASSIVE.
211.0	MASSIVE BASALT; HARD.
231.0	ONE UNDULATING FRACTURE AT APPROX. 0 DEGREES FROM 223.5-230.6 FEET WITH CLAY
251.0	ONE FRACTURE AT 10 DEGREES TO CORE AXIS.
271.0	MASSIVE BASALT TO 264.5; CONTACT WITH SCORACIOUS TO VESICULAR BASALT.
278.0	PALEO-ALLUVIUM; GRAVEL AND COBBLES OF QUARTZITE;CHERT AND BASALT IN A SANDY SILT
285.1	PALEO-ALLUVIUM AS ABOVE WITH BASALT;QURATZITE AND SANDSTONE COBBLES.
297.9	SILTY SAND WITH GRAVEL AND COBBLES OF BASALT;QUARTZ AND QUARTZITE.
303.4	PARTIALLY CEMENTED PALEO-ALLUVIUM.
308.4	WELL CEMENTED PALEO-ALLUVIUM 304-305.
319.3	SLIGHTLY TO MODERATELY CEMENTED TO 318.0. WEATHERED SUPAI BEDROCK AT 318.0.
324.2	REDDISH AND YELLOWISH ORANGE SILTY SANDSTONE WITH SOME CLAY.
329.5	MEDIUM GRAY SILTY SANDSTONE BRECCIA
340.0	CORE BARREL MISLATCH; ROCK TYPE ASSUMED FROM PREVIOUS RUN. END OF HOLE.

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-119

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,816.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	PALEO-CHANNEL	EASTING:	576,078.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-119	ELEVATION:	6,093.70	START DEPTH:	0.00
START DATE:	4/ 8/86	INCLINATION:	89.81	LOGGED BY:	CHR
COMPLETION DATE:	4/16/86	BEARING:	235.1	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
20.0	WR	CDGCVM	0.0	40.0	0	0			0	0			0	0			0	0			S3	0.0	*
25.3	WS	BAO VV	56.6	82.1	7	4	T	SJ	0	0			1	4	SC	SJ	0	0			19,108	0.0	*
26.8	WS	BAO VV	62.0	100.0	2	3	T	SJ	0	0			0	0			0	0			18,332	0.0	*
32.3	WS	BAO VM	77.6	92.7	5	3	T	SJ	1	3	T	SJ	0	0			0	0			15,535	29.6	*
36.0	WS	BAO VM	100.0	100.0	0	0			0	0			0	0			0	0			13,671	100.0	* #
41.0	WF	BAOQVV	52.0	100.0	7	4	FC	SJ	2	3	FC	SJ	0	0			2	3	FC	SJ	12,117	32.6	*
46.0	WF	BAOQVV	48.0	100.0	9	3	FC	SJ	2	4	FC	SJ	2	3	FC	SJ	4	3	FC	SJ	24,235	21.0	*
51.0	WF	BAO VV	0.0	94.0	7	4	FC	SJ	8	4	FC	SJ	0	0			4	3	FC	SJ	19,264	0.0	*
56.0	WF	BAOQVM	87.0	100.0	4	4	SC	SJ	0	0			0	0			1	3	FC	SJ	21,749	87.0	*
61.0	WF	BAOQVM	87.6	100.0	4	4	T	SJ	0	0			0	0			1	3	FC	SJ	24,856	62.0	*
66.0	WS	BAO VV	36.0	81.0	6	4	FC	SJ	0	0			0	0			5	4	FC	SJ	17,866	0.0	*
71.0	WS	BAOQVM	83.0	100.0	7	4	SC	SJ	0	0			0	0			0	0			20,196	60.0	
76.0	WS	BAOQVV	74.4	100.0	7	4	SC	SJ	0	0			0	0			0	0			17,089	50.0	* #
81.0	WS	BAOQVV	68.4	100.0	6	4	SC	SJ	3	4	FC	SJ	0	0			0	0			19,419	50.0	* #
86.0	WF	BAOQVM	72.0	100.0	7	4	SC	SJ	2	4	SC	SJ	0	0			0	0			19,419	50.0	*
91.0	WM	BAOQVV	66.0	100.0	8	4	FC	SJ	0	0			4	4	FC	SJ	0	0			2,330	36.0	*
96.0	WM	BAOQVV	71.0	98.0	5	4	SC	SJ	0	0			1	3	FC	SJ	2	3	FC	SJ	17,399	30.4	*
101.0	WM	BAOQVM	96.0	100.0	3	3	SC	SJ	0	0			0	0			0	0			24,856	96.0	*
106.0	WF	BAOQVM	100.0	100.0	0	0			2	3	T	SJ	0	0			0	0			30,294	100.0	*
111.0	WF	BAOQVM	100.0	100.0	2	3	T	SJ	1	3	T	SJ	0	0			0	0			26,721	100.0	
116.0	WF	BAOQVM	63.4	100.0	4	3	T	SJ	1	3	SC	SJ	0	0			2	3	SC	SJ	27,187	56.4	*
121.0	WM	BAOQVV	47.4	90.0	6	4	FC	SJ	2	3	FC	SJ	0	0			3	4	FC	SJ	15,846	0.0	*
126.0	WM	BAOQVV	38.4	78.0	3	3	SC	SJ	0	0			0	0			0	0			15,535	0.0	*
131.0	WH	BAO VV	0.0	55.4	20	3	FC	SJ	0	0			0	0			10	3	FC	SJ	S3	0.0	*
136.0	WM	BAOQVV	29.0	100.0	7	4	FC	SJ	5	4	FC	SJ	0	0			5	4	FC	SJ	8,234	0.0	*
141.0	WS	BAOQVV	100.0	100.0	2	3	T	SJ	0	0			0	0			0	0			16,778	71.8	*
146.0	WF	BAOQVM	94.0	100.0	4	3	T	SJ	1	3	T	SJ	0	0			0	0			21,128	59.4	
150.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			23,303	100.0	*
155.1	WF	BAO VM	100.0	100.0	3	3	T	SJ	2	3	T	SJ	0	0			0	0			23,614	77.5	
160.3	WF	BAOQVM	88.8	100.0	2	3	T	SJ	3	3	T	SJ	0	0			0	0			27,187	79.2	
165.5	WF	BAOQVM	93.3	100.0	2	3	T	SJ	1	3	T	SJ	0	0			0	0			27,963	93.3	

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-119

SITE:	MINER FLAT DAM SITE	NORTHING:	1,086,816.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	PALEO-CHANNEL	EASTING:	576,078.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-119	ELEVATION:	6,093.70	START DEPTH:	0.00
START DATE:	4/ 8/86	INCLINATION:	89.81	LOGGED BY:	CHR
COMPLETION DATE:	4/16/86	BEARING:	235.1	PAGE:	2

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
170.7	WF	BAQQVM	80.8	100.0	5	3	T	SJ	0	0			0	0			1	2	FC	SJ	27,963	50.0	#
173.0	WF	BAQQVM	100.0	100.0	3	3	T	SJ	1	3	T	SJ	0	0			0	0			27,031	46.5	#
176.0	WF	BAQQVM	67.0	73.3	2	3	T	SJ	1	3	FC	SJ	0	0			0	0			26,410	0.0	
181.0	WF	BAQQVM	100.0	100.0	5	3	T	SJ	2	3	T	SJ	0	0			0	0			27,963	20.6	
186.0	WF	BAQQVM	87.4	100.0	1	3	T	SJ	0	0			1	3	FC	SJ	0	0			29,517	87.4	
191.0	WF	BAQQVM	85.4	100.0	1	3	T	SJ	0	0			0	0			2	3	FC	SJ	30,449	74.0	
196.0	WF	BAQQVM	99.0	100.0	2	4	T	SJ	0	0			0	0			0	0			29,206	86.4	#
201.0	WF	BAQQVM	100.0	100.0	0	0			0	0			0	0			0	0			29,517	100.0	‡ #
206.0	WF	BAQQVM	97.0	99.0	0	0			0	0			0	0			1	3	HC	SJ	28,274	97.0	‡
211.0	WF	BAQQVM	83.6	100.0	0	0			0	0			0	0			2	3	HC	SJ	31,692	68.0	‡ #
216.0	WF	BAQQVM	100.0	100.0	4	2	T	SJ	0	0			0	0			4	5	HC	SJ	25,167	81.0	
221.0	WR	PCSOFM	19.4	70.0	4	4	FC	SJ	0	0			0	0			0	0			S2	0.0	‡
224.0	WR	PCSOFM	0.0	60.7	0	0			0	0			0	0			0	0			S2	0.0	‡
229.0	WR	PCSOFM	0.0	42.0	0	0			0	0			0	0			0	0			S2	0.0	
234.0	WR	PCSOFM	0.0	43.4	0	0			0	0			0	0			0	0			S2	0.0	‡
239.8	WR	PCSCFM	0.0	37.1	0	0			0	0			0	0			0	0			S2	0.0	
245.2	WR	PCSCFM	0.0	63.9	0	0			0	0			0	0			0	0			S2	0.0	
250.7	WR	PCSOFM	0.0	49.1	0	0			0	0			0	0			0	0			S2	0.0	‡
256.0	WR	PASBRB	0.0	56.6	0	0			0	0			0	0			0	0			S2	0.0	‡
261.0	WR	PCSOFM	0.0	55.0	0	0			0	0			0	0			0	0			S2	0.0	‡
266.0	WR	PASGFM	0.0	50.0	0	0			0	0			0	0			0	0			S2	0.0	‡
271.0	WR	PCSOFM	0.0	40.0	0	0			0	0			0	0			0	0			S2	0.0	‡
276.0	WR	PASGFM	0.0	37.0	0	0			0	0			0	0			0	0			S2	0.0	‡
281.0	WR	PAGCRM	0.0	38.0	0	0			0	0			0	0			0	0			S2	0.0	‡
291.0	WF	SSSCFM	22.6	47.3	4	2	T	SJ	0	0			5	2	T	SJ	0	0			4,971	0.0	‡
292.4	WS	SSCAFR	92.9	92.9	0	0			0	0			0	0			0	0			4,661	0.0	‡
296.9	WS	SSCAFR	46.7	91.6	4	3	FC	SJ	3	4	FC	SJ	0	0			1	6	FC	SJ	13,671	34.9	‡
300.0	WS	SSCAFR	0.0	0.0	0	0			0	0			0	0			0	0			R3	0.0	‡

DEPTH ---------- COMMENT ----------

= TOTAL CORE RECOVERY > 100%

20.0 COLLUVIUM/TERTIARY GRAVEL. YELLOWISH ORANGE SILTY CLAY WITH COBBLES OF QUARTZITE.

DEPTH	---------- COMMENT ----------
	# = TOTAL CORE RECOVERY > 100%
25.3	PALE TO MEDIUM GRAY OLIVINE BASALT;VESICULAR WITH CLAY FILLING VESSICLES.
26.8	VESSICLES BECOMING SMALLER AND FEWER.
32.3	MASSIVE OLIVINE BASALT. FLOW BANDING 80-90 DEGREES TO CORE AXIS.
36.0	ONE PIECE OF CORE 3.75 FEET LONG.
41.0	FLOW BOUNDRY AT 38.2. CLAY FILLING FRACTURES; CLAY & QUARTZ FILLING SOME VESSICLES
46.0	VESICULAR BASALT
51.0	FLOW BOUNDRIES AT 47 AND 50 FEET.
56.0	MASSIVE AT 51.1; SOME VESSICLES AND FRACTURES WITH QUARTZ FILLING.
61.0	NEARLY VERTICAL FRACTURE AT 59.2 FEET FILLED WITH DENSE CLAY .25 INCHES THICK.
66.0	SCORACIOUS TO VESSICULAR BASALT 62.5-65 FEET; FLOW BOUNDRY.
76.0	SCORACIOUS TO VESICULAR BASALT 72.7- 75 FEET; FLOW BOUNDRY. QUARTZ FILLING .
81.0	FLOW BOUNDRY AT 80.2 FEET.
86.0	FLOW BOUNDRY AT 84 FEET.
91.0	PALEOSOL/FLOW BOUNDRY AT 86-87.3 FEET. MODERATELY WEATHERED BASALT BELOW.
96.0	ROPEY TEXTURED FLOW BANDING AT 94 FEET 20-40 DEGREES TO CORE AXIS.
101.0	GRAYISH RED OLIVINE BASALT WITH QUARTZ;MODERATELY WEATHERED;MASSIVE.
106.0	MEDIUM GRAY MAASSIVE BASALT.
116.0	BECOMING VESICULAR WITH QUARTZ FILLING SOME VESICLES.
121.0	FLOW BOUNDRY AT 118.5. MODERATELY WEATHERED.
126.0	PALEOSOL AT 123.2;SANDY CLAEY SILT WITH GRAVEL AND COBBLES OF BASALT.
131.0	GRAYISH RED BASLAT PALEOSOL.
136.0	SCORACIOUS AND ROPEY TEXTURED BASALT;MODERATELY TO HIGHLY WEATHERED.
141.0	BECOMING LESS WEATHERED AND LESS VESICULAR.
150.0	ONE PIECE OF CORE 4.0 FEET LONG.
201.0	ONE PIECE OF CORE 5.1 FEET LONG.
206.0	NEARLY VERTICAL FRACTURE HEALED WITH OVERCONSOLIDATED CLAY 202.7-206.5 FEET.
211.0	ONE FRACTURE AT 10 DEGREES TO CORE AXIS HEALED WITH CLAY AT 209.4-210.8 FEET
221.0	BASALT FLOW BRECCIA TO 217.3. PALEO-COLLUVIUM; SANDSTONE TALUS.
224.0	POOR RECOVERY OF PALEO-COLLUVIUM.
234.0	SANDSTONE TALUS BLOCKS UP TO 0.5 FEET.
250.7	SANDSTONE TALUS BLOCKS UP TO 3 FEET.
256.0	PALEO-ALLUVIUM WITH GRAVEL AND COBBLES OF CHERT; QUARTZITE AND STANDSTONE.
261.0	PALEO-COLLUVIUM WITH BASALT COBBLES AND SANDSTONE BOULDERS.
266.0	PALEO-ALLUVIUM/PALEO-COLLUVIUM WITH QUARTZITE GRAVEL AND SANDSTONE BOULDERS.
271.0	PALEO-COLLUVIUM WITH SANDSTONE COBBLES AND BOULDERS.
276.0	PALEOCOLLUVIUM/ALLUVIUM WITH SANDSTONE BOULDERS AND QUARTZITE & LIMESTONE GRAVEL
281.0	PALEO-ALLUVIUM WITH LIMESTONE AND QUARTZITE GRAVEL AND SANDSTONE COBBLES.

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-120A

SITE:	MINER FLAT DAM SITE	NORTHING:	0	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	PALEO-CHANNEL	EASTING:	0	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-120A	ELEVATION:	6,082.00	START DEPTH:	0.00
START DATE:	4/18/86	INCLINATION:	89.5	LOGGED BY:	CHR
COMPLETION DATE:	4/21/86	BEARING:	250.4	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
8.5	WR	COCRVM	0.0	23.5	0	0			0	0			0	0			0	0			S3	0.0	*
11.8	WS	BAO VV	63.6	100.0	4	4	T	SJ	0	0			2	3	FC	SJ	1	3	FC	SJ	21,749	33.3	*
17.1	WS	BAO VV	33.8	92.5	5	4	SC	SJ	3	4	SC	SJ	3	3	FC	SJ	1	3	FC	SJ	20,817	0.0	*
22.3	WS	BAOQVM	67.9	96.2	4	3	FC	SJ	3	3	FC	SJ	1	3	SC	SJ	1	3	SC	SJ	18,642	56.3	*
26.0	WS	BAOQVM	29.7	96.5	1	3	SC	SJ	4	4	SC	SJ	0	0			2	3	SC	SJ	21,749	0.0	*
30.5	WS	BAOQVV	100.0	100.0	1	3	SC	SJ	3	3	SC	SJ	0	0			0	0			18,642	100.0	#
35.6	WS	BAOQVM	55.9	100.0	2	3	T	SJ	3	3	T	SJ	0	0			5	4	T	SJ	17,866	33.3	* #
40.8	WS	BAO VV	40.4	100.0	9	3	SC	SJ	6	3	FC	SJ	5	4	FC	SJ	6	3	FC	SJ	34,178	0.0	
46.0	WS	BAO VM	57.7	80.4	6	3	T	SJ	3	4	SC	SJ	0	0			0	0			27,963	24.8	
51.0	WF	BAO VM	62.4	100.0	7	4	T	SJ	0	0			3	3	FC	SJ	2	4	FC	SJ	23,303	34.6	* #
56.0	WF	BAOQVM	84.0	100.0	3	3	T	SJ	2	3	T	SJ	0	0			0	0			22,526	43.4	*
57.2	WS	BAOQVV	100.0	100.0	0	0			0	0			0	0			0	0			20,196	100.0	*
61.0	WM	BAOQVM	72.6	98.4	7	3	FC	SJ	0	0			0	0			0	0			18,953	0.0	*
66.0	WM	BAOQVV	68.8	100.0	8	3	SC	SJ	0	0			0	0			0	0			23,769	58.6	*
71.0	WM	BAOQVM	96.0	100.0	6	4	T	SJ	0	0			0	0			0	0			25,167	68.0	
76.0	WM	BAOQVV	86.8	98.0	6	4	FC	SJ	0	0			0	0			0	0			19,730	31.0	*
81.0	WM	BAOQVV	79.4	100.0	3	4	FC	SJ	3	3	FC	SJ	0	0			0	0			15,225	67.6	*
86.0	WM	BAOQVV	39.4	100.0	8	3	FC	SJ	0	0			1	3	FC	SJ	4	4	FC	SJ	7,768	0.0	* #
91.0	WS	BAOQVM	73.4	100.0	4	4	SC	SJ	2	3	SC	SJ	2	3	FC	SJ	2	3	FC	SJ	25,167	52.6	* #
96.0	WS	BAOQVV	43.0	100.0	8	3	FC	SJ	4	3	FC	SJ	0	0			0	0			15,535	0.0	*
101.0	WS	BAOQVV	72.0	96.0	4	3	SC	SJ	1	3	SC	SJ	0	0			2	3	SC	SJ	24,546	0.0	*
106.0	WM	BAOQVS	56.2	98.0	6	4	FC	SJ	4	3	FC	SJ	0	0			3	3	FC	SJ	13,982	47.6	*
111.0	WM	BAOQVS	32.4	82.0	10	3	FC	SJ	0	0			0	0			5	3	FC	SJ	18,332	22.0	*
116.0	WC	PACBVB	0.0	68.2	0	0			0	0			0	0			0	0			S2	0.0	*
121.0	WR	PASBFB	12.0	58.0	0	0			0	0			0	0			0	0			S2	0.0	
122.4	WR	PASBFB	0.0	78.6	0	0			0	0			0	0			0	0			S2	0.0	
126.0	WR	PASBFB	0.0	72.2	0	0			0	0			0	0			0	0			S2	0.0	
131.0	WR	PASBFB	0.0	40.0	0	0			0	0			0	0			0	0			S2	0.0	
136.0	WR	PASBFB	0.0	46.0	0	0			0	0			0	0			0	0			S2	0.0	
141.0	WR	PASBFB	0.0	52.0	0	0			0	0			0	0			0	0			S2	0.0	
146.0	WR	PASBFB	0.0	40.0	0	0			0	0			0	0			0	0			S2	0.0	

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-120A

SITE:	MINER FLAT DAM SITE	NORTHING:	0	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	PALEO-CHANNEL	EASTING:	0	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-120A	ELEVATION:	6,082.00	START DEPTH:	0.00
START DATE:	4/18/86	INCLINATION:	89.5	LOGGED BY:	CHR
COMPLETION DATE:	4/21/86	BEARING:	250.4	PAGE:	2

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
151.0	WM	BAO VM	38.0	98.0	0	0			5	4	SC	SJ	0	0			4	4	SC	SJ	35,731	0.0	*
154.5	WF	BAOQVV	88.6	88.6	0	0			0	0			0	0			0	0			34,954	88.6	*
159.7	WF	BAOQVM	70.2	100.0	5	4	T	SJ	3	4	T	SJ	0	0			1	4	T	SJ	30,760	0.0	*
164.8	WF	BAOQVM	92.2	98.0	7	4	T	SJ	0	0			0	0			0	0			30,760	31.8	
166.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			27,963	100.0	*
171.0	WF	BAOQVM	70.0	100.0	5	3	T	SJ	0	0			1	3	T	SJ	3	3	T	SJ	31,847	56.0	*
176.0	WF	BAOQVM	80.2	100.0	5	4	T	SJ	0	0			0	0			1	4	T	SJ	37,285	53.4	*
181.0	WF	BAOQVM	82.4	94.0	2	4	T	SJ	4	4	T	SJ	0	0			1	4	T	SJ	34,178	50.4	
186.0	WF	BAOQVM	63.6	100.0	4	4	T	SJ	0	0			2	2	T	SJ	1	3	T	SJ	31,071	37.4	
191.0	WF	BAOQVM	0.0	100.0	14	4	T	SJ	0	0			0	0			4	3	T	SJ	29,517	0.0	
196.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			30,760	100.0	*
201.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			32,935	100.0	*
206.0	WF	BAOQVM	100.0	100.0	2	2	T	SJ	0	0			0	0			0	0			33,401	100.0	
211.0	WF	BAOQVM	90.0	100.0	2	3	T	SJ	2	2	T	SJ	0	0			0	0			31,071	59.0	
216.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			31,847	100.0	
221.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			31,071	100.0	
226.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			31,071	100.0	
231.0	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			30,760	100.0	
236.0	WF	BAOQVM	100.0	100.0	0	0			0	0			1	4	T	SJ	1	3	T	SJ	36,042	100.0	*
241.0	WC	BAO VR	56.0	92.0	0	0			0	0			0	0			0	0			R1	0.0	*
246.0	WH	BAO VS	61.0	90.0	3	3	SC	SJ	2	3	SC	SJ	0	0			1	3	FC	SJ	4,661	0.0	*
251.0	WR	PAS FB	0.0	18.0	0	0			0	0			0	0			0	0			S2	0.0	*
256.0	WR	PASGFB	0.0	6.0	0	0			0	0			0	0			0	0			S2	0.0	
261.0	WR	PASGFB	0.0	38.0	0	0			0	0			0	0			0	0			S1	0.0	
266.0	WR	PASGFB	0.0	84.0	0	0			0	0			0	0			0	0			S2	0.0	*
271.0	WR	PASGFB	0.0	89.0	0	0			0	0			0	0			0	0			R1	0.0	*
276.0	WM	BAOQVV	70.6	99.0	4	3	SC	SJ	4	3	FC	SJ	0	0			0	0			32,003	20.6	*
281.0	WM	BAOQVV	73.2	97.6	5	4	SC	SJ	0	0			3	4	FC	SJ	2	3	SC	SJ	35,731	23.0	*
286.0	WF	BAOQVV	98.2	100.0	1	4	T	SJ	2	3	SC	SJ	0	0			1	3	SC	SJ	38,527	89.2	* *
291.0	WF	BAOQVM	90.0	98.0	2	3	T	SJ	1	3	T	SJ	0	0			0	0			36,508	90.0	*
296.0	WF	BAOQVM	88.0	100.0	1	3	FC	SJ	1	3	SC	SJ	0	0			1	3	SC	SJ	37,285	54.0	*

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-120A

SITE:	MINER FLAT DAM SITE	NORTHING:	0	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	PALEO-CHANNEL	EASTING:	0	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-120A	ELEVATION:	6,082.00	START DEPTH:	0.00
START DATE:	4/18/86	INCLINATION:	89.5	LOGGED BY:	CHR
COMPLETION DATE:	4/21/86	BEARING:	250.4	PAGE:	3

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
301.0	WF	BAOQVM	46.0	74.0	1	4	T	SJ	0	0			0	0			0	0			34,954	46.0	I
306.0	WR	PASGRB	0.0	64.0	0	0			0	0			0	0			0	0			S3	0.0	I
311.0	WR	PAGBRB	0.0	45.0	0	0			0	0			0	0			0	0			S3	0.0	I
316.0	WR	PACBRB	0.0	48.0	0	0			0	0			0	0			0	0			S3	0.0	I
321.0	WR	PASBRB	0.0	38.0	0	0			0	0			0	0			0	0			S3	0.0	
326.0	WR	PASBRB	0.0	78.0	0	0			0	0			0	0			0	0			S3	0.0	I
331.0	WR	PASRRB	0.0	38.0	0	0			0	0			0	0			0	0			S3	0.0	
336.0	WR	PASRRB	0.0	68.0	0	0			0	0			0	0			0	0			S3	0.0	
341.0	WR	PASRRB	0.0	84.0	0	0			0	0			0	0			0	0			R1	0.0	I
346.0	WC	SMSCVM	0.0	69.0	0	0			0	0			0	0			0	0			R1	0.0	I
351.0	WC	SMSCVM	66.0	66.0	0	0			0	0			0	0			0	0			R1	66.0	I

DEPTH	COMMENT
8.5	BASALT/TERTIARY GRAVEL COLLUVIUM; SILTY CLAY WITH BASALT BOULDERS.
11.8	MEDIUM GRAY OLIVINE BASALT;SLIGHTLY WEATHERED TO FRESH.
17.1	FLOW BOUNDRY/PALEOSOL AT 14 FEET;FLOW BANDING NEARLY VERTICAL.
22.3	PALEOSOL AT 18-18.3; SCORACIOUS 18.3-18.6 FEET.
26.0	PALEOSOL AT 25 FEET.
35.6	MEDIUM GRAY OLIVINE BASALT WITH QUARTZ;MASSIVE.
51.0	FLOW BANDING FROM 20-90 DEGREES FROM CORE AXIS.
56.0	VESICULAR BELOW 54 FEET.
61.0	MODERATE REDDISH BROWN OLIVINE BASALT WITH QUARTZ;VESICULAR WITH CLAY&QUARTZ
66.0	MODERATELY WEATHERED;QUARTZ CRYSTALS FILLING SOME VESSICLES.
76.0	FLOW BOUNDRY AT 73.2 FEET.
81.0	FLOW BOUNDRY AT 80.5 FEET.
86.0	REDDISH GRAY VESUCULAR TO SCORACIOUS BASALT.
91.0	FLOW BOUNDRY AT 89.7.
96.0	FLOW BANDING AT 45 DEGREES TO CORE AXIS.
101.0	FLOW BOUNDRY AT 96.7 FEET WITH 0.6 FEET CLAY PALEOSOL.
106.0	SCORACIOUS BASALT;FLOW BOUNDRY AT 102 FEET WITH CLAY; SAND&GRAVEL 102-103.6.
111.0	FLOW BOUNDRIES AT 107.5;109.5;110.5. DENSE CLAY AND SAND.
116.0	GRAY TO RED PALEO-ALLUVIUM; CLAY;SAND;GRAVEL AND COBBLES OF BASALTIC ORIGIN.
151.0	REDISH GRAY TO MEDIUM GRAY VESICULAR TO MASSIVE BASALT.
154.5	ONE PIECE OF CORE 3.1 FEET LONG.
159.7	SOME FRACTURES HEALED WITH QUARTZ.
166.0	ONE PIECE OF CORE 1.2 FEET LONG.
171.0	MEDIUM GRAY OLIVINE BASALT WITH QUARTZ; MASSIVE;SLIGHTLY MAGNETIC.

DEPTH	---------- COMMENT ----------
196.0	ONE PIECE OF CORE 5.0 FEET LONG.
201.0	ONE PIECE OF CORE 5.0 FEET LONG.
236.0	MASSIVE BASALT; 6 FRACTURES IN 45 FEET OF CORE.
241.0	BASALT BRECCIA WITH SAND AND COBBLES AT 237.8 FEET.
246.0	SCORACIOUS BASALT BRECCIA.
266.0	POORLY INDURATED AT 265 FEET.
271.0	POORLY TO MODERATELY INDURATED PALEO-ALLUVIUM CONGLOMERATE.
276.0	MEDIUM GRAY OLIVINE BASALT WITH QUARTZ;VESICULAR;BASALT BRECCIA AT 275.2 FEET.
281.0	BASALT BRECCIA WITH CLAY MATRIX AT 276.7-276.9;278.0-278.25
286.0	VESICULAR TO MASSIVE BASALT
291.0	SLIGHTLY VESICULAR BASALT WITH QUARTZ FILLING SOME VESSICLES.
296.0	FLOW BANDING 30 DEGREES TO CORE AXIS AT 294 FEET.
301.0	MASSIVE BASALT TO 298.3 FEET;BASALT BRECCIA BELOW.
306.0	PALEO-ALLUVIUM;GRAYISH ORANGE PINK SAND;SILT WITH GRAVEL.
311.0	PALEO-ALLUVIUM;GRAVEL OF CHERT;SANDSTONE;LIMESTONE;CHERT;QUARTZITE;GRANITICS
316.0	PALEO-ALLUVIUM;SIGNIFICANT CLAY RECOGNIZED IN DRILL CUTTINGS.
326.0	PALEO-ALLUVIUM;SANDSTONE BOULDERS UP TO 0.9 FEET IN DIAMETER.
341.0	PALEO-ALLUVIUM;WELL CEMENTED WITH $CaCO_3$.
346.0	COMPLETELY WEATHERED SANDY CLAYEY SILTSTONE
351.0	END OF HOLE.

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-121

SITE:	MINER FLAT DAM SITE	NORTHING:	1,085,645.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	SUPAI RIDGE	EASTING:	576,785.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-121	ELEVATION:	6,075.00	START DEPTH:	0.00
START DATE:	2/ 9/86	INCLINATION:	89.9	LOGGED BY:	CHR
COMPLETION DATE:	2/12/86	BEARING:	278.5	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
11.5	WR	COSRFM	0.0	47.5	0	0			0	0			0	0			0	0			S3	0.0	*
15.5	WR	COSRFM	0.0	80.8	0	0			0	0			0	0			0	0			S3	0.0	
17.9	WR	COSRFM	0.0	66.7	0	0			0	0			0	0			0	0			S2	0.0	
23.1	WR	COSRFM	32.7	61.0	0	0			2	3	T	SJ	1	2	T	SJ	0	0			17,866	0.0	*
28.2	WS	BAO VM	51.6	93.7	7	3	SC	SJ	0	0			0	0			5	3	FC	SJ	20,973	22.5	*
30.0	WF	BAO VV	71.1	100.0	2	4	FC	SJ	0	0			0	0			0	0			14,758	71.1	#
40.1	WF	BAO VM	84.0	100.0	4	4	SC	SJ	0	0			0	0			0	0			24,856	64.7	*
50.3	WF	BAO VM	80.2	98.7	0	0			0	0			8	3	SC	SJ	0	0			26,099	62.0	
60.6	WF	BAOQVM	78.9	97.4	0	0			6	3	SC	SJ	5	3	FC	SJ	0	0			21,749	39.1	*
70.4	WF	BAOQVV	31.3	77.6	10	3	SC	SJ	0	0			0	0			5	3	FC	SJ	19,419	10.7	*
77.2	WF	SSMSVM	42.6	84.1	6	2	T	SJ	0	0			4	2	T	SJ	3	4	T	SJ	1,864	0.0	*
81.0	WF	SSMSFB	69.7	100.0	5	2	T	SJ	0	0			0	0			2	2	T	SJ	1,165	42.1	#
89.7	WF	SSS FX	30.2	85.6	16	2	T	SJ	2	2	T	SJ	0	0			2	2	T	SJ	1,787	0.0	*
100.2	WF	SSS FX	57.4	90.2	5	2	T	SJ	0	0			10	2	T	SJ	2	2	SI	SJ	466	34.4	*
106.5	WF	SSS FX	92.9	92.9	1	2	T	SJ	0	0			0	0			1	2	T	SJ	1,787	92.9	*
111.0	WF	SSMSFX	25.1	73.3	8	2	T	SJ	0	0			0	0			1	2	T	SJ	2,486	0.0	*
121.0	WF	SSMSFB	58.2	82.6	13	2	T	SJ	0	0			3	2	T	SJ	0	0			1,942	32.5	*
128.6	WF	SSS FL	6.6	59.2	13	2	T	SJ	0	0			0	0			4	2	T	SJ	1,165	0.0	*
137.7	WF	SSS FB	34.1	78.0	4	2	T	SJ	0	0			5	2	T	SJ	2	2	T	SJ	466	15.5	*
147.3	WF	SSS FB	48.1	91.7	7	2	T	SJ	5	2	T	SJ	0	0			0	0			1,243	26.5	*
155.0	WF	SSS FB	35.6	96.8	12	2	T	SJ	3	2	T	SJ	0	0			8	3	T	SJ	544	18.8	
164.8	WF	SSS FB	60.5	89.4	9	3	T	SJ	0	0			0	0			4	3	T	SJ	1,087	40.8	*
175.0	WF	SSS FX	53.4	90.8	11	2	T	SJ	3	2	T	SJ	0	0			3	2	T	SJ	1,787	15.1	*
185.0	WF	SSS FX	11.0	37.2	11	2	T	SJ	0	0			0	0			3	2	T	SJ	1,010	11.0	
191.0	WF	SSS FX	51.7	96.3	9	2	T	SJ	0	0			0	0			2	2	T	SJ	1,709	18.8	*

DEPTH ---------- COMMENT ----------

= TOTAL CORE RECOVERY > 100%

11.5 COLLUVIUM;WHITE TO REDDISH ORANGE SAND AND SANDSTONE TALUS BLOCKS.
23.1 SUPAI COLLUVIUM TO 20.5; VESICULAR TO MASSIVE BASALT;FLOW BANDED 60 DEGREES.
28.2 MEDIUM GRAY OLIVINE BASALT; MASSIVE TO VESICULAR.
40.1 VESICULAR BASALT;PALEOSOL AT 34.5 & 38.6; 0.3 FEET OF CLAY RECOVERED.

DEPTH	---------- COMMENT ----------
	# = TOTAL CORE RECOVERY > 100%
60.6	OLIVINE QUARTZ BASALT;SANDY CLAY PALEOSOL 0.2 FEET THICK AT 55.5 FEET.
70.4	BASALT TO 67 FEET; PALEO-COLLUVIUM 67-68.8;SANDSTONE BELOW 68.8 FEET.
77.2	WHITE TO REDDISH BROWN SANDSTONE TO SILTY SANDSTONE;FRIABLE.
89.7	CROSS BEDDED QUARTZ SANDSTONE.
100.2	IRON STAINED JOINTS 30 DEGREES TO CORE AXIS AT 98 FEET.
106.5	PALE YELLOW SANDSTONE WITH MINOR CaCO3.
111.0	MEDIUM REDDISH BROWN SILTY SANDSTONE;FINE-GRAINED;MEDIUM-GRAINED WITH DEPTH.
121.0	WHITE SANDSTONE;FINE- TO MEDIUM-GRAINED FRIABLE TO 117 FEET.'
128.6	MEDIUM REDDISH BROWN SANDSTONE;FINE-GRAINED;VERY FRIABLE.
137.7	POORLY INDURATED
147.3	PINK SANDSTONE AND SAND; VERY POORLY INDURATED.
164.8	PALE REDDISH BROWN AND WHITE SANDSTONE;FRIABLE.
175.0	CROSS BEDDED SANDSTONE.
191.0	END OF HOLE.

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-122

SITE:	MINER FLAT DAM SITE	NORTHING:	1,085,645.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	SUPAI RIDGE	EASTING:	576,785.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-122	ELEVATION:	6,075.00	START DEPTH:	0.00
START DATE:	2/ 5/86	INCLINATION:	89.9	LOGGED BY:	CHR
COMPLETION DATE:	2/ 7/86	BEARING:	278.5	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM	
3.0	WR	COSCMM	0.0	25.0	0	0			0	0			0	0			0	0			S3	0.0	‡	
8.0	WS	BAO VM	22.0	74.0	4	2	T	SJ	0	0			0	0			1	3	T	SJ	17,089	0.0	‡	
12.3	WS	BAOQVM	95.3	100.0	4	2	SC	SJ	3	2	SC	SJ	0	0			2	3	SC	SJ	19,419	34.9	‡	#
17.4	WS	BAO VV	33.9	98.0	7	4	SC	SJ	2	4	SC	SJ	0	0			3	4	SC	SJ	17,866	25.5	‡	
22.7	WS	BAO VV	2.6	90.6	12	3	FC	SJ	0	0			0	0			4	3	FC	SJ	20,196	0.0	‡	
27.8	WF	BAOQVM	36.1	97.1	8	3	SC	SJ	3	3	SC	SJ	0	0			5	3	FC	SJ	27,963	0.0	‡	
32.5	WF	BAOQVM	89.8	100.0	4	3	T	SJ	2	3	SC	SJ	0	0			2	2	SC	SJ	28,274	51.1		#
37.5	WM	BAO VV	45.4	87.2	5	3	FC	SJ	8	3	FC	SJ	0	0			2	3	FC	SJ	17,089	0.0	‡	
47.0	WF	BAOQVV	45.7	86.5	12	4	FC	SJ	5	3	FC	SJ	0	0			8	3	FC	SJ	19,419	24.8	‡	
57.2	WF	BAO VV	40.6	67.6	8	3	FC	SJ	3	3	FC	SJ	0	0			3	3	FC	SJ	10,564	25.6		
65.2	WF	BAO VM	75.0	95.1	10	3	SC	SJ	0	0			1	2	T	SJ	3	3	FC	SJ	15,535	65.1	‡	
76.0	WF	BAO VV	71.5	94.0	10	4	SC	SJ	0	0			2	3	SC	SJ	4	3	FC	SJ	18,642	28.9		
86.4	WS	BAO VV	55.3	96.2	12	4	T	SJ	0	0			4	3	FC	SJ	2	3	FC	SJ	16,312	28.8	‡	
96.9	WF	BAO VV	66.3	95.7	9	4	T	SJ	0	0			2	4	FC	SJ	4	3	FC	SJ	13,982	36.8		
106.6	WF	BAOQVM	57.7	97.9	10	3	SC	SJ	3	3	SC	SJ	0	0			2	4	FC	SJ	22,526	36.7	‡	
118.3	WF	BAOQVM	37.8	76.9	6	3	T	SJ	2	3	SC	SJ	4	3	SC	SJ	3	3	FC	SJ	34,178	29.4	‡	
123.0	WS	BAO VR	8.9	51.1	8	3	FC	SJ	0	0			0	0			0	0			7,768	0.0	‡	
130.1	WS	BAO VR	0.0	14.1	0	0			0	0			0	0			0	0			R2	0.0	‡	
136.8	WF	BAO VV	52.5	87.3	6	3	SC	SJ	3	3	SC	SJ	0	0			3	3	FC	SJ	29,983	19.4	‡	
139.2	WF	BAO VV	0.0	99.6	0	0			0	0			0	0			0	0			R2	0.0		
143.7	WF	BAO VS	40.0	84.4	6	3	FC	SJ	0	0			2	3	FC	SJ	2	3	FC	SJ	8,078	0.0	‡	
154.1	WF	BAO VS	31.0	63.5	10	3	FC	SJ	0	0			0	0			5	3	FC	SJ	2,330	12.3	‡	
164.8	WF	SSS FM	8.6	49.3	13	3	T	SJ	0	0			0	0			0	0			2,563	0.0	‡	
175.0	WF	SSS FM	81.1	99.0	9	3	T	SJ	0	0			0	0			2	3	T	SJ	R2	67.6	‡	

DEPTH ---------- COMMENT ----------

= TOTAL CORE RECOVERY > 100%

8.0 BASALT/TERTIARY GRAVEL COLLUVIUM;SILTY CLAYEY SAND WITH COBBLES OF BASALT.
8.0 CONTACT OF COLLUVIUM WITH BASALT AT 3.0 FEET.
12.3 MEDIUM GRAY OLIVINE BASALT WITH QUARTZ;VESICULAR AT 10 FEET.
17.4 ROPEY TEXTURE; LOST DRILL FLUID CIRCULATION AT 14 FEET.
22.7 FLOW BOUNDRY

DEPTH	---------- COMMENT ----------
	# = TOTAL CORE RECOVERY > 100%
27.8	MASSIVE BASALT BELOW 24 FEET.
37.5	FLOW BOUNDRY AT 35 FEET.
47.0	FLOW BOUNDRY WITH PALEOSOL AT 41 FEET.
65.2	MASSIVE BASALT TO 65 FEET.
86.4	VESICULAR BASALT
106.6	MASSIVE BASALT AT 104.5 FEET.
118.3	MASSIVE BASALT TO 114.2. SCORACIOUS BASALT TO BASALT BRECCIA BELOW 114.2.
123.0	FLOW BRECCIA; SCORACIOUS FRAGMENTS CEMENTED WITH CLAY.
130.1	CORE BARREL MISLATCH; SCORACIOUS BASALT FRAGMENTS RECOVERED.
136.8	SCORACIOUS TO MASSIVE BASALT.
143.7	SCORACIOUS BASALT.
154.1	SCORACIOUS BASALT TO 150.3. PALE RED TO PINK; FINE-GRAINED SANDSTONE.
164.8	AT CONTACT <.25 FEET OF PALEO-COLLUVIUM. MODERATE BROWN SANDSTONE.
175.0	PALE PINKISH YELLOW SANDSTONE; FRIABLE;>75% QUARTZ SAND. END OF HOLE.

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-123

SITE:	MINER FLAT DAM SITE	NORTHING:	1,085,307.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	SUPAI RIDGE	EASTING:	577,018.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-123	ELEVATION:	6,113.10	START DEPTH:	0.00
START DATE:	3/13/86	INCLINATION:	90	LOGGED BY:	CHR
COMPLETION DATE:	3/22/86	BEARING:	0	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
53.0	WR	TGSBRB	0.0	39.4	0	0			0	0			0	0			0	0			S3	0.0	*
58.0	WS	SSS FM	20.0	61.0	5	3	T	SJ	0	0			4	6	T	SJ	8	3	T	SJ	7,379	0.0	*
58.8	WS	SSS FM	0.0	100.0	4	2	T	SJ	0	0			0	0			0	0			12,739	0.0	*
64.7	WF	SSS FM	32.4	64.7	9	4	T	SJ	0	0			8	4	T	SJ	5	3	T	SJ	4,350	0.0	*
68.5	WF	SSS FM	0.0	35.8	4	4	T	SJ	0	0			5	3	T	SJ	3	3	T	SJ	7,457	0.0	
70.4	WF	SSSCVR	0.0	48.9	3	2	T	SJ	0	0			0	0			2	3	T	SJ	5,826	0.0	*
76.2	WF	SSMCFR	0.0	58.6	19	3	T	SJ	0	0			3	3	T	SJ	3	2	T	SJ	7,146	0.0	*
80.4	WF	SSS FM	11.0	52.4	5	3	T	SJ	0	0			0	0			2	2	T	SJ	10,564	0.0	*
84.6	WF	SSS FM	0.0	59.5	5	3	T	SJ	0	0			0	0			6	2	T	SJ	7,768	0.0	
89.8	WF	SSS FM	50.6	100.0	8	2	T	SJ	0	0			3	2	T	SJ	3	2	T	SJ	9,010	50.6	*
95.2	WS	SSS FM	33.5	95.2	10	2	T	SJ	0	0			6	2	SI	SJ	5	2	SI	SJ	2,408	20.4	*
100.7	WS	SSS FM	20.7	93.6	6	2	T	SJ	0	0			6	2	SI	SJ	11	2	SI	SJ	3,029	0.0	*
106.2	WS	SSS FM	39.1	92.2	7	2	T	SJ	0	0			5	2	SI	SJ	6	2	SI	SJ	1,787	39.1	*
111.8	WS	SSS FM	55.7	91.1	5	2	T	SJ	0	0			2	2	T	SJ	3	2	SI	SJ	1,942	37.5	*
121.9	WS	SSMCFB	62.6	86.3	10	2	T	SJ	0	0			7	2	T	SJ	6	2	T	SJ	1,320	53.6	*
131.9	WF	SSS FB	23.3	68.2	25	2	T	SJ	0	0			3	2	T	SJ	5	2	T	SJ	699	18.5	*
141.9	WF	SSMSFB	6.5	17.5	4	2	T	SJ	0	0			0	0			3	2	T	SJ	777	0.0	*
151.9	WF	SSMCVB	22.0	42.5	7	2	T	SJ	0	0			3	2	T	SJ	2	2	T	SJ	1,320	18.0	
161.9	WF	SSMCVM	0.0	.2	0	0			0	0			0	0			0	0			R1	0.0	*
171.9	WF	SSMSVM	26.0	33.0	2	2	T	SJ	0	0			0	0			2	2	T	SJ	R1	26.0	*
181.9	WF	SSMSVM	0.0	0.0	0	0			0	0			0	0			0	0			R1	0.0	*
190.6	WF	SSM VB	73.0	92.0	5	2	T	SJ	0	0			0	0			7	2	T	SJ	R1	73.0	*
196.8	WF	SSS VB	12.6	33.9	5	2	T	SJ	0	0			0	0			3	2	T	SJ	R1	0.0	
202.9	WF	SSS FB	45.9	73.8	9	2	T	SJ	0	0			0	0			3	2	T	SJ	R1	45.9	
207.0	WF	SSS FM	0.0	70.7	11	2	T	SJ	0	0			0	0			0	0			R1	0.0	*

DEPTH	COMMENT
53.0	TERTIARY GRAVEL OF SAND SILT CLAY COBBLES AND GRAVEL OF GRANITICS;SANDSTONE.
58.0	TOP OF BEDROCK AT 53 FEET. WHITE TO PALE YELLOWISH ORANGE SANDSTONE.
58.8	SANDSTONE; FINE-GRAINED;POORLY LITHIFIED OR INDURATED.
64.7	MODERATE ORANGE PINK TO WHITE SANDSTONE; FINE-GRAINED.
70.4	SANDSTONE/CLAY BRECCIA.
76.2	SANDSTONE AND SILTY CLAYEY SANDSTONE BRECCIA.
80.4	MOTTLED;PALE ORANGISH PINK;WHITE AND MODERATE REDDISH BROWN SANDSTONE.

DEPTH	COMMENT
89.8	WHITE SANDSTONE;FINE-GRAINED;POORLY INDURATED.
95.2	IRON STAIN ON SOME FRACTURES.
100.7	MODERATELY INDURATED;IRON STAINING ON SOME FRACTURES.
106.2	MODERATE IRON STAINING.
111.8	WHITE SANDSTONE; FINE- TO MEDIUM-GRAINED;POORLY TO MODERATELY INDURATED.
121.9	WHITE SANDSTONE TO 113. INTERBEDDED SANDSTONE AND SILTY SANDSTONE;SOME CLAY.
131.9	MODERATE REDDISH BROWN SANDSTONE; FINE- TO MEDIUM-GRAINED;POORLY INDURATED.
141.9	SILTY SANDSTONE;LENTICULAR TO BEDDED;POORLY INDURATED.
161.9	VERY POOR RECOVERY.
171.9	MODERATE YELLOWISH ORANGE TO PALE REDDISH BROWN SILTY SAND(STONE).
181.9	NO RECOVERY! ROCK TYPE ASSUMED FROM ADJACENT ROCKS.
190.6	MODERATE REDDISH BROWN;YELLOWISH ORANGE AND WHITE MOTTLED SAND(STONE).
207.0	HOLE CAVED;LOST DRILL STRING; END OF HOLE!

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-124

SITE:	MINER FLAT DAM SITE	NORTHING:	1,085,430.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	SUPAI RIDGE	EASTING:	576,820.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-124	ELEVATION:	6,080.60	START DEPTH:	0.00
START DATE:	4/ 1/86	INCLINATION:	90	LOGGED BY:	CHR
COMPLETION DATE:	4/ 4/86	BEARING:	0	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
23.0	WR	TGSGRB	0.0	11.3	0	0			0	0			0	0			0	0			S3	0.0	‡
24.7	WS	BAO VV	0.0	100.0	3	4	T	SJ	0	0			2	3	T	SJ	0	0			8,544	0.0	‡
28.0	WS	BAO VV	40.9	79.4	3	5	T	SJ	0	0			0	0			5	4	FC	SJ	13,671	0.0	‡
29.9	WS	BAOQVV	0.0	73.7	3	4	T	SJ	0	0			0	0			3	4	FC	SJ	18,953	0.0	
35.1	WS	BAO VV	19.8	69.8	11	4	T	SJ	0	0			0	0			6	3	SC	SJ	23,303	0.0	
40.3	WF	BAOQVV	71.9	100.0	4	3	T	SJ	0	0			2	3	SC	SJ	3	3	T	SJ	21,749	47.1	‡
45.4	WF	BAOQVM	46.1	72.9	3	4	FC	SJ	0	0			2	3	T	SJ	0	0			21,439	29.4	‡
50.2	WF	BAOQVV	89.2	100.0	6	4	SC	SJ	0	0			0	0			0	0			13,982	45.2	‡ #
55.4	WF	BAOQVM	41.2	94.6	5	3	SC	SJ	0	0			4	3	SC	SJ	4	3	SC	SJ	24,856	28.8	
60.1	WF	BAOQVM	93.6	100.0	0	0			1	3	SC	SJ	0	0			0	0			22,992	93.6	‡ #
65.1	WF	BAOQVM	87.0	98.0	1	3	T	SJ	0	0			0	0			1	3	T	SJ	22,371	87.0	
70.2	WF	BAOQVM	100.0	100.0	0	0			0	0			0	0			0	0			19,419	100.0	‡
75.3	WF	BAOQVM	94.1	98.4	3	3	SC	SJ	0	0			0	0			4	3	SC	SJ	19,264	65.1	‡
80.0	WF	BAOQVM	34.9	100.0	5	2	T	SJ	0	0			6	2	SC	SJ	3	3	FC	SJ	22,060	22.3	#
85.0	WF	BAOQVM	55.6	92.6	6	3	T	SJ	0	0			0	0			1	3	SC	SJ	17,866	22.2	‡
90.9	WS	BAO VV	64.4	87.6	6	3	T	SJ	0	0			0	0			4	3	FC	SJ	R3	47.5	‡
96.0	WS	BAOQVM	100.0	100.0	2	3	T	SJ	0	0			0	0			2	3	SC	SJ	25,245	68.6	‡ #
99.4	WS	BAO VV	67.6	100.0	5	3	SC	SJ	2	3	SC	SJ	0	0			2	2	SC	SJ	14,914	0.0	‡ #
101.3	WC	SSS FM	0.0	100.0	0	0			0	0			0	0			0	0			8,700	0.0	‡ #
106.0	WC	SSS FM	0.0	8.5	0	0			0	0			0	0			0	0			S2	0.0	‡
111.0	WC	SSS FM	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	‡
116.0	WS	SSS FM	8.6	58.0	4	2	T	SJ	0	0			3	2	T	SJ	0	0			R1	0.0	‡
121.0	WC	SSS FM	0.0	3.0	0	0			0	0			0	0			0	0			S2	0.0	
126.0	WM	SSS FB	0.0	54.0	8	2	T	SJ	0	0			3	2	T	SJ	2	2	T	SJ	2,020	0.0	‡
131.1	WC	SSS FB	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	‡
136.0	WS	SSS FB	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	
141.0	WC	SSS FB	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	
146.0	WF	SSS FB	67.0	93.2	6	2	T	SJ	0	0			6	2	T	SJ	0	0			1,243	67.0	‡
151.0	WF	SSS FB	31.2	75.4	7	2	T	SJ	0	0			5	2	T	SJ	1	2	T	SJ	1,243	24.0	‡
156.0	WC	SSS FB	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	
161.0	WF	SSS FB	37.0	76.0	7	2	T	SJ	0	0			2	2	T	SJ	2	2	T	SJ	932	0.0	

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-124

SITE:	MINER FLAT DAM SITE	NORTHING:	1,085,430.00	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	SUPAI RIDGE	EASTING:	576,820.00	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-124	ELEVATION:	6,080.60	START DEPTH:	0.00
START DATE:	4/ 1/86	INCLINATION:	90	LOGGED BY:	CHR
COMPLETION DATE:	4/ 4/86	BEARING:	0	PAGE:	2

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
166.0	WF	SSS FB	39.0	98.0	5	2	T	SJ	0	0			6	2	T	SJ	4	2	T	SJ	R1	20.0	#
171.0	WF	SSS FB	0.0	50.0	4	2	T	SJ	2	2	T	SJ	3	2	T	SJ	0	0			R1	0.0	
176.0	WF	SSS FB	9.0	50.0	2	2	T	SJ	0	0			3	2	T	SJ	0	0			R1	0.0	
180.0	WC	SSS FB	0.0	0.0	0	0			0	0			0	0			0	0			S2	0.0	#

DEPTH ---------- COMMENT ----------

= TOTAL CORE RECOVERY > 100%

23.0 TERTIARY GRAVEL OF CLAY SILT SAND GRAVEL AND COBBLES OF QUARTZITE;QUARTZ .
24.7 CONTACT WITH BASALT AT 23 FEET. VESICULAR BASALT.
28.0 MEDIUM GRAY OLIVINE BASALT. VERTICAL FRACTURES FILLED WITH SILTY CLAY.
40.3 1 TO 3.5 FOOT MASSIVE BASALT INTERVALS.
45.4 MASSIVE BASALT TO 41.7 FEET. CLAY FILLED FLOW BOUNDRY TO 43 FEET.
50.2 MASSIVE BASALT TO 46 FEET. LARGE VESICLES (0.05 FEET) BELOW 46 FEET.
60.1 MASSIVE BASALT; FLOW BANDING 60 DEGREES TO CORE AXIS.
70.2 MASSIVE BASALT; ONE PIECE OF CORE 5.1 FEET LONG.
75.3 FRACTURES 'STAINED' WITH CLAY.
90.9 FLOW BOUNDRY AT 88 FEET.
180.0 NO RECOVERY OF CORE. END OF HOLE.

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-125A

SITE:	MINER FLAT DAM SITE	NORTHING:	0	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	SUPAI RIDGE	EASTING:	0	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-125A	ELEVATION:	0	START DEPTH:	0.00
START DATE:	4/ 5/86	INCLINATION:	90	LOGGED BY:	CHR
COMPLETION DATE:	4/ 6/86	BEARING:	0	PAGE:	1

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
7.0	WR	TGSBRB	0.0	3.6	0	0			0	0			0	0			0	0			S2	0.0	*
10.0	WS	BAD VV	76.7	100.0	6	4	T	SJ	0	0			0	0			0	0			9,010	0.0	* *
14.9	WS	BAD VM	98.0	100.0	4	4	T	SJ	1	3	T	SJ	0	0			0	0			19,574	65.3	* *
20.3	WS	BAD VM	88.9	96.3	4	4	SI	SJ	0	0			0	0			0	0			20,973	44.1	*
25.5	WS	BAD VV	51.9	100.0	5	4	T	SJ	0	0			1	3	SC	SJ	4	5	FC	SJ	14,603	21.5	*
30.7	WS	BAD VV	62.3	99.0	5	3	SC	SJ	0	0			0	0			4	3	FC	SJ	17,089	36.2	*
36.0	WF	BADQVM	66.4	99.1	5	3	SC	SJ	0	0			0	0			3	3	FC	SJ	21,749	44.2	*
41.0	WF	BADQVM	100.0	100.0	2	3	T	SJ	1	3	T	SJ	0	0			1	3	FC	SJ	20,817	74.2	
46.0	WF	BADQVM	98.0	98.0	0	0			0	0			0	0			0	0			20,507	98.0	*
51.0	WF	BADQVM	56.6	100.0	3	3	T	SJ	1	3	T	SJ	0	0			2	3	T	SJ	25,633	25.0	* *
56.0	WF	BADQVM	100.0	100.0	1	3	T	SJ	0	0			0	0			1	2	HQ	SJ	23,924	100.0	* *
61.0	WF	BADQVM	97.6	100.0	1	3	SC	SJ	2	3	SC	SJ	0	0			0	0			25,167	97.6	
66.0	WF	BADQVM	100.0	100.0	1	5	T	SJ	0	0			0	0			0	0			24,701	92.2	* *
71.0	WF	BADQVV	8.8	96.0	8	3	FC	SJ	5	3	FC	SJ	0	0			4	6	FC	SJ	18,642	0.0	*
76.0	WM	BAD VV	12.0	100.0	7	3	FC	SJ	8	3	FC	SJ	0	0			6	4	FC	SJ	10,098	0.0	*
81.0	WS	BADQVV	86.4	98.0	6	4	FC	SJ	0	0			0	0			0	0			12,428	56.2	*
86.0	WS	BADQVM	95.6	100.0	3	3	SC	SJ	0	0			0	0			0	0			20,196	85.0	*
91.0	WS	BADQVM	93.0	100.0	3	4	T	SJ	0	0			0	0			0	0			19,885	93.0	*
96.0	WF	BADQVM	100.0	100.0	0	0			0	0			0	0			0	0			18,642	100.0	*
101.0	WF	BADQVM	97.0	100.0	0	0			0	0			1	3	FC	SJ	0	0			15,535	97.0	*
111.0	WF	BADQVM	47.2	49.4	1	4	T	SJ	0	0			1	3	SC	SJ	0	0			18,332	47.2	*
116.0	WF	BADQVM	42.0	80.0	0	0			0	0			0	0			0	0			20,507	20.0	*
121.0	WS	SSMCFB	0.0	48.4	9	2	T	SJ	0	0			0	0			0	0			1,787	0.0	*
126.0	WF	SSS FM	0.0	73.0	0	0			0	0			0	0			7	2	T	SJ	S2	0.0	*
131.0	WF	SSS FM	0.0	72.2	8	2	T	SJ	0	0			4	2	T	SJ	3	2	T	SJ	R1	0.0	*
136.0	WF	SSS FM	0.0	6.0	0	0			0	0			0	0			0	0			R1	0.0	*
141.0	WF	SSS FM	0.0	62.0	0	0			0	0			0	0			0	0			1,709	0.0	*
146.0	WF	SSS FM	0.0	6.0	0	0			0	0			0	0			0	0			R1	0.0	
151.0	WF	SSS FB	0.0	46.6	0	0			0	0			0	0			0	0			R1	0.0	*
156.0	WF	SSS FB	66.4	78.0	0	0			0	0			2	2	T	SJ	0	0			R1	58.0	
161.0	WF	SSS FB	35.4	66.0	0	0			0	0			6	2	T	SJ	0	0			R1	20.0	

ENGINEERING GEOLOGIC LOG
BOREHOLE: MF-125A

SITE:	MINER FLAT DAM SITE	NORTHING:	0	DRILLING METHOD:	NQ3 TRIPLE TUBE CORE
LOCATION:	SUPAI RIDGE	EASTING:	0	DRILLING CONTRACTOR:	KELMINE EXPLORATION
BOREHOLE:	MF-125A	ELEVATION:	0	START DEPTH:	0.00
START DATE:	4/ 5/86	INCLINATION:	90	LOGGED BY:	CHR
COMPLETION DATE:	4/ 6/86	BEARING:	0	PAGE:	2

DEPTH (ft.)	WEATH/ ALT	ROCK TYPE	RQD (%)	TOT CORE RECOVERY (%)	DIP OF FRACTURES TO CORE AXIS: 90 - 70 NO	90 - 70 RGH	90 - 70 ALT	90 - 70 TYPE	70 - 50 NO	70 - 50 RGH	70 - 50 ALT	70 - 50 TYPE	50 - 30 NO	50 - 30 RGH	50 - 30 ALT	50 - 30 TYPE	30 - 0 NO	30 - 0 RGH	30 - 0 ALT	30 - 0 TYPE	STRENGTH (psi)	SCR (%)	CM
166.0	WF	SSS FB	20.0	72.0	7	2	T	SJ	6	2	T	SJ	0	0			0	0			R1	20.0	
170.0	WF	SSS FB	0.0	77.5	0	0			5	2	T	SJ	6	2	T	SJ	0	0			R1	0.0	#

DEPTH ---------- COMMENT ----------

= TOTAL CORE RECOVERY > 100%

7.0 TERTIARY GRAVEL OF GRAVEL AND COBBLES IN A SANDY CLAYEY SILT.
10.0 CONTACT WITH BASALT AT 7.0 FEET. MEDIUM GRAY OLIVINE BASALT;VESICULAR.
14.9 FLOW BANDING AT 12 FEET.
20.3 MEDIUM GRAY OLIVINE BASALT; MASSIVE
25.5 VESICULAR AND FLOW BOUNDRY AT 20.8 FEET.CLAY FILLED VERTICAL FRACTURE 22-24.
30.7 VESICULAR BASALT;SLIGHTLY MAGNETIC
36.0 MASSIVE BASALT
46.0 MEDIUM GRAY OLIVINE BASALT WITH QUARTZ;MASSIVE.ONE PIECE OF CORE 4.9 FEET.
51.0 VERTICAL FRACTURES DECREASE 'RQD' AND 'SCR'.
56.0 FRACTURE AT 10 DEGREES TO CORE AXIS HEALED WITH QUARTZ.
66.0 VESICULAR BELOW 65.5 FEET.
71.0 FLOW BOUNDRY BASALT STARTING AT 66 FEET.
76.0 MEDIUM GRAY VESICULAR BASALT TO 73 FEET. 'PALEO-SURFACE' STARTING AT 73.
81.0 MEDIUM REDDISH GRAY OLIVINE BASALT;VESICULAR;FLOW BANDING AT 10-20 DEGREES.
86.0 MOSTLY MASSIVE BASALT.
91.0 FLOW BANDING AT 80 DEGREES TO CORE AXIS.
96.0 MEDIUM GRAY OLIVINE BASALT WITH QUARTZ;MASSIVE;SLIGHTLY MAGNETIC.
101.0 ONE FRACTURE IN 10 FEET OF CORE.
111.0 MINOR FLOW BANDING
116.0 VESICULAR AT 111.7 FEET. SANDSTONE CONTACT AT 112.2 FEET.
121.0 PALE REDDISH ORANGE SANDSTONE;FINE-GRAINED;SILTY & CLAYEY INTERBEDS.
126.0 SAND(STONE) FINE- TO VERY FINE-GRAINED;VERY POORLY INDURATED.
131.0 GRAYISH PINK SAND(STONE);>75% QUARTZ SAND.
136.0 VERY POOR RECOVERY
141.0 MODERATE REDDISH ORANGE SAND(STONE);FINE-GRAINED;VERY POORLY INDURATED.
151.0 THINLY BEDDED SAND(STONE).
170.0 MODERATE REDDISH ORANGE SAND(STONE). END OF HOLE.

APPENDIX C

Histograms of Geologic Logs,

Fracture Distribution by Dip,

Fracture Frequence, and

Hydraulic Conductivity

MINER FLAT DAM SITE: MF-102: ENGINEERING GEOLOGIC LOG

DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
8.0		++	.	++	++	17,479	6,070.6
12.7		++++++	++	++	++	17,479	6,066.0
23.3		++++++	++	++++	++++	19,863	6,055.6
31.2		++++++	++++	++++	++++++	22,723	6,047.9
41.8		++++	++	++	++++	19,068	6,037.5
51.0		++++++	++++	++	++++	17,479	6,028.5
61.0		++++++	++++	++	++++	17,797	6,018.8
71.0		++++++	++++	++++	++++++	27,331	6,009.0
81.0		++++++	++++	++++	++++++	23,041	5,999.2
91.0		++++	++	++	++++	11,123	5,989.4
101.0		++	.	.	++++++	19,068	5,979.7
111.0		++	.	.	.	0	5,969.9
121.0		++++	++	.	++++	17,479	5,960.1
131.0		++++++	++++	++++	++++++	30,191	5,950.3
139.8		++++	++	.	++	4,767	5,941.7
150.1		++++++	++++	++++	++++++	35,753	5,931.7
160.3		++++++	++++	++++	++++++	28,602	5,921.7
170.6		++++++	++++++	++++++	++++++	30,986	5,911.6
180.7		++++++	++++++	++++++	++++++	29,397	5,901.7
190.8		++++++	++++++	++++++	++++++	31,780	5,891.9
200.9		++++++	++++	++++	++++++	34,958	5,882.0
211.0		++++++	++++	++++	++++++	33,369	5,872.1
220.4		++++	++++	+++	++++++	25,424	5,862.9
230.6		.	.	.	.	0	5,853.0
232.6		.	.	.	.	0	5,851.0
237.7		++++	.	.	.	0	5,846.0
241.0		+++	.	.	.	0	5,842.8
249.1		++++	.	.	.	0	5,834.9
257.9		++++	.	.	.	0	5,826.3
263.9		++++	++++	++++	++	2,701	5,820.4
269.9		++++	+++	+++	.	953	5,814.5
281.0		++	.	.	++	2,384	5,803.7
291.0		++++++	++++++	++++++	.	795	5,793.9
301.0		++++	++++	++++	.	636	5,784.1
311.0		++++++	++++++	++++++	+	1,192	5,774.4
321.0		++++	++++	++++	++	2,225	5,764.6
331.0		++++	++++	++++	.	715	5,754.8
341.0		++++	++++	++++	.	874	5,745.0
351.0		++++	++++	++++	+	1,907	5,735.2
361.0		++++++	++++++	++++++	++++	4,290	5,725.5
371.0		++++++	++++++	++++++	+	1,827	5,715.7
375.0		++++++	++++++	++++++	++	2,304	5,711.8

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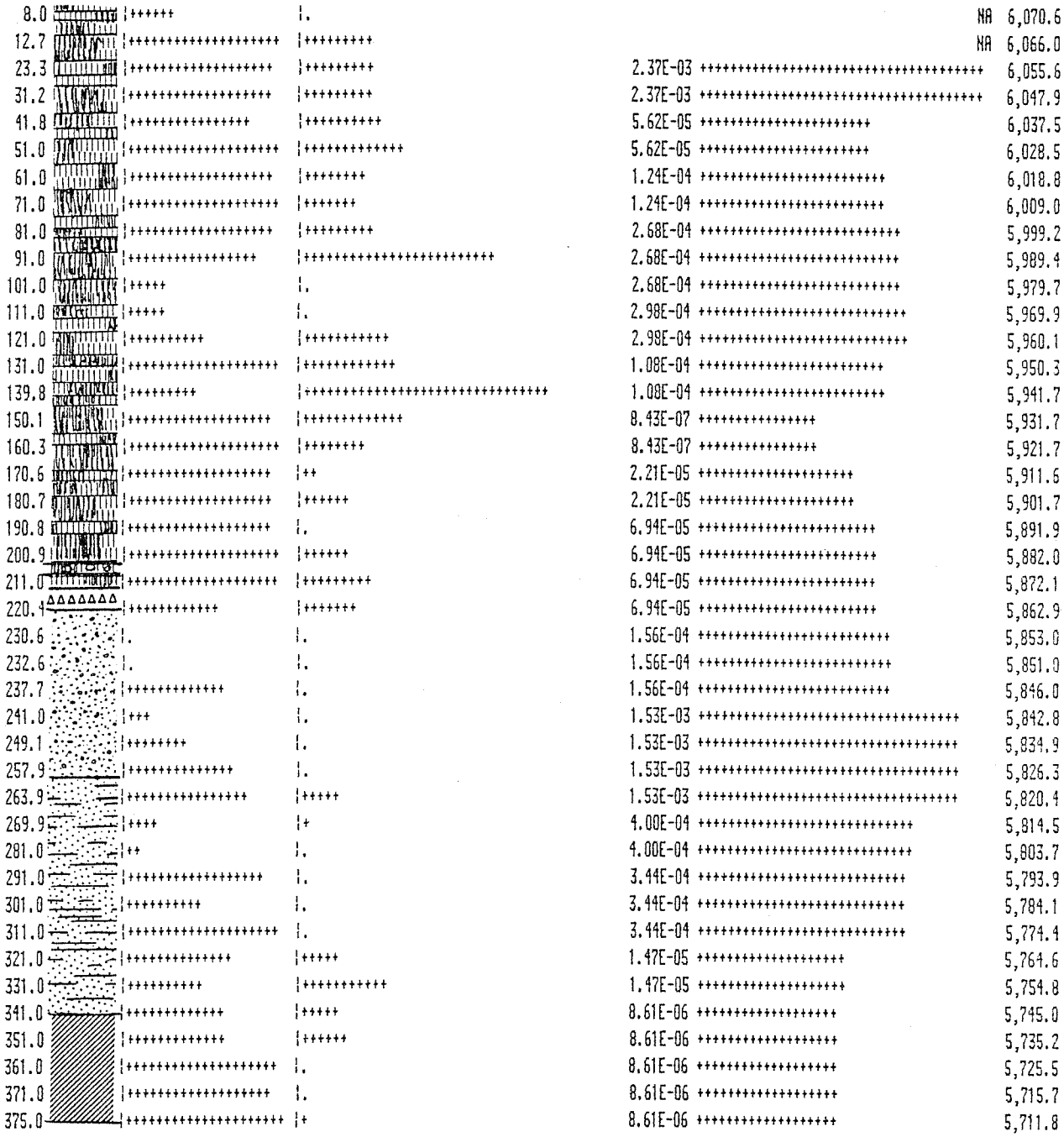
MINER FLAT DAM SITE: MF-102: FRACTURE LOG

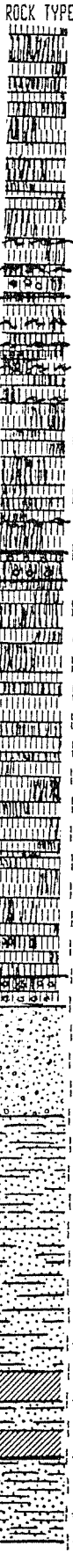
DEPTH	RECOVERY 0% 100%	NUMBER OF FRACTURES 90-70 0 10	70-50 0 10	50-30 0 10	30-0 0 10 20	ELEV. (feet)
8.0	++++++	.	.	.	.	6,070.6
12.7	++++++++++++++++++++	+++++	.	.	++++	6,066.0
23.3	++++++++++++++++++++	++++++	.	.	+++	6,055.6
31.2	++++++++++++++++++++	+++++++	.	.	+++	6,047.9
41.8	++++++++++++++++	+++++++	.	.	+++	6,037.5
51.0	++++++++++++++++++++	+++++	++++++	.	++	6,028.5
61.0	++++++++++++++++++++	.	+++++	.	+++	6,018.8
71.0	++++++++++++++++++++	.	+++++	.	++	6,009.0
81.0	++++++++++++++++++++	.	++	.	+++++++	5,999.2
91.0	++++++++++++++++	++++++++++++	+++++	.	++++++++	5,989.4
101.0	+++++	.	.	.	.	5,979.7
111.0	+++++	.	.	.	.	5,969.9
121.0	++++++++++	++	++++	.	+++++	5,960.1
131.0	++++++++++++++++++++	+++	.	++++	+++++	5,950.3
139.8	+++++++++	++++++++++	+++++++	++++++	++++++++	5,941.7
150.1	++++++++++++++++++++	++++++	.	++	+++++	5,931.7
160.3	++++++++++++++++++++	++++	++	.	++	5,921.7
170.6	++++++++++++++++++++	.	.	.	++	5,911.6
180.7	++++++++++++++++++++	++++	++	.	.	5,901.7
190.8	++++++++++++++++++++	.	.	.	.	5,891.9
200.9	++++++++++++++++++++	.	.	++++	++	5,882.0
211.0	++++++++++++++++++++	.	+	.	++++++++	5,872.1
220.4	++++++++++++	++++	.	.	+++	5,862.9
230.6	.	.	.	.	.	5,853.0
232.6	.	.	.	.	.	5,851.0
237.7	++++++++++++++	.	.	.	.	5,846.0
241.0	+++	.	.	.	.	5,842.8
249.1	++++++++	.	.	.	.	5,834.9
257.9	+++++++++++++++	.	.	.	.	5,826.3
263.9	++++++++++++++++	++++	.	.	+	5,820.4
269.9	++++	+	.	.	.	5,814.5
281.0	++	.	.	.	.	5,803.7
291.0	++++++++++++++++++++	.	.	.	.	5,793.9
301.0	++++++++++	.	.	.	.	5,784.1
311.0	++++++++++++++++++++	.	.	.	.	5,774.4
321.0	+++++++++++++++	+++++	.	.	.	5,764.6
331.0	++++++++++	++++++++	+++	.	.	5,754.8
341.0	++++++++++++	++++	.	.	+	5,745.0
351.0	++++++++++++++	++++	.	.	++	5,735.2
361.0	++++++++++++++++++++	.	.	.	.	5,725.5
371.0	++++++++++++++++++++	.	.	.	.	5,715.7
375.0	++++++++++++++++++++	.	+	.	.	5,711.8

MINER FLAT DAM SITE: MF-102: FRACTURES and HYDRAULIC CONDUCTIVITY



25-Jun-86

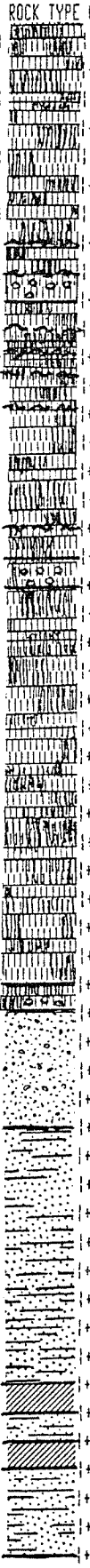
MINER FLAT DAM SITE: MF-105: ENGINEERING GEOLOGIC LOG



DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
5.8		++++++++++	+++++++	.	++++++++++++++++++	19,068	6,068.3
11.2		++++++++++++++++++++	++++++++++++	++++	++++++++++++++++++++	19,068	6,063.1
16.3		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	+++++++++++++++	15,572	6,058.3
21.2		++++++++++++++++++++	++++++++++++	.	++++++++++++++++++++	22,246	6,053.6
26.0		++++++++++++++++++	++++++++++++	+++++	+++++++++++++++	15,572	6,049.1
31.0		++++++++++++++++++	+++++++++++++++	++++++++	++++++++++++++++++++++++++	25,901	6,044.3
36.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++	22,405	6,039.5
40.0		++++++++++++++++++++	++++++++++++++++	++++++++++	++++++++++++++++++++	21,452	6,035.7
50.2		++++++++++++++++++	++++++++++	+++	++++++++++++++++++++++++	25,901	6,026.0
59.5		++++++++++++++++++	+++++++++	+++	++++++++++++++++++++++	23,676	6,017.2
68.0		++++++++++	+	.	++++++++++++++++++	19,386	6,009.1
75.2		++++++++++++++++++	++++++++++++++	+++++++++	++++++++++++++++++	20,657	6,002.2
85.0		++++++++++++++++++	+++++++++	++++++	++++++++++++++++++	19,068	5,992.9
92.5		++++++++++++++++++	++++++++	+++	++++++++++++	13,189	5,985.8
102.8		++++++++++++++++++	+++++++++++++++	++++++++++	+++++++++++++++	15,096	5,976.0
105.1		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++	11,123	5,973.8
115.0		+++++++++++++++++	+++++++	+++++++	++++++++++++++++++++	22,087	5,964.4
125.0		++++++++++++++	+++++	+++++	++++++++++++++++++	19,068	5,954.8
135.0		++++++++++++++++++	++++++++++++	++	+++++++++++++++	15,890	5,945.3
145.0		++++++++++++++++++	++++++++++	++++++++	+++++++++++++++	15,890	5,935.8
155.0		++++++++++++++++++++	+++++++++++++++	++++++++++++	++++++++++++++++++++	23,835	5,926.3
163.2		++++++++++++++++++	++++++++	+++++	++++++++++++++++++++++++	24,630	5,918.5
173.3		++++++++++++++++++	++++++++++	+++	++++++++++++++++++++++++++	28,602	5,908.9
183.5		++++++++++++++++++	+++	.	++++++++++++++++++++++++	27,967	5,899.2
193.8		++++++++++++++++++	++++++	++	++++++++++++++++++++++++++++++	32,098	5,889.4
203.9		++++++++++++++++++++	++++++++++	+++++++	++++++++++++++++++++++++++++	29,238	5,879.7
214.0		+++++++++++++++++	+++++++++++++++	+++++++	++++++++++++++++++++++++++++++	31,780	5,870.1
218.8		++++++++++++++++++	+++++++++++++++++	++++++++++++++++	++++++++++++++++++++++++++++++	32,575	5,865.6
229.1		++++++++++++++++++	+++++++++++++++++	++++++++++++++	++++++++++++++++++++++++++++++	33,369	5,855.8
239.2		++++++++++++++++++++	+++++++++++++++	+++++++++	++++++++++++++++++++	21,452	5,846.1
249.4		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++++	27,013	5,836.4
259.6		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++++++++++++++	36,547	5,826.7
269.9		++++++++++++++++++	++++++++++++	++++++++++++	++++++++++++++++++++++++++++++++++	36,547	5,816.9
280.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++	23,835	5,807.3
285.0		++++++++++++++++++	++++++++++++	.	+++++++++++++++	15,255	5,802.6
290.0		++++++++++	.	.	.	0	5,797.8
295.0		+++++++++++++++	.	.	.	0	5,793.0
300.0		++++++++++++	.	.	.	0	5,788.3
303.3		++++++++++	.	.	.	0	5,785.1
305.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	+	1,589	5,783.5
310.0		++++++++++++++++++	++++++	++++	.	795	5,778.8
315.0		++++++++++++++++++	++++++++++++++++++	+++++++++	++	2,066	5,774.0
320.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	+	1,430	5,769.2
325.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	+	1,430	5,764.5
330.0		+++++++++++++++	++++++++++++	++++++++++++	.	874	5,759.7
335.1		++++++++++	.	.	.	715	5,754.9
337.7		++++++++++++++++++	+++++++++++++++	+++++++++++++++	++	2,542	5,752.4
340.0		++++++++++++++++++	++++++++++++++++++	.	+++++	5,403	5,750.2
345.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	++++++	6,594	5,745.5
350.0		++++++++++++++++++++	+++++++++++++++	++++++++++++	+	1,907	5,740.7
355.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	+++++	5,323	5,735.9
360.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	+++	3,099	5,731.2
365.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	+	1,510	5,726.4
370.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++	2,701	5,721.7

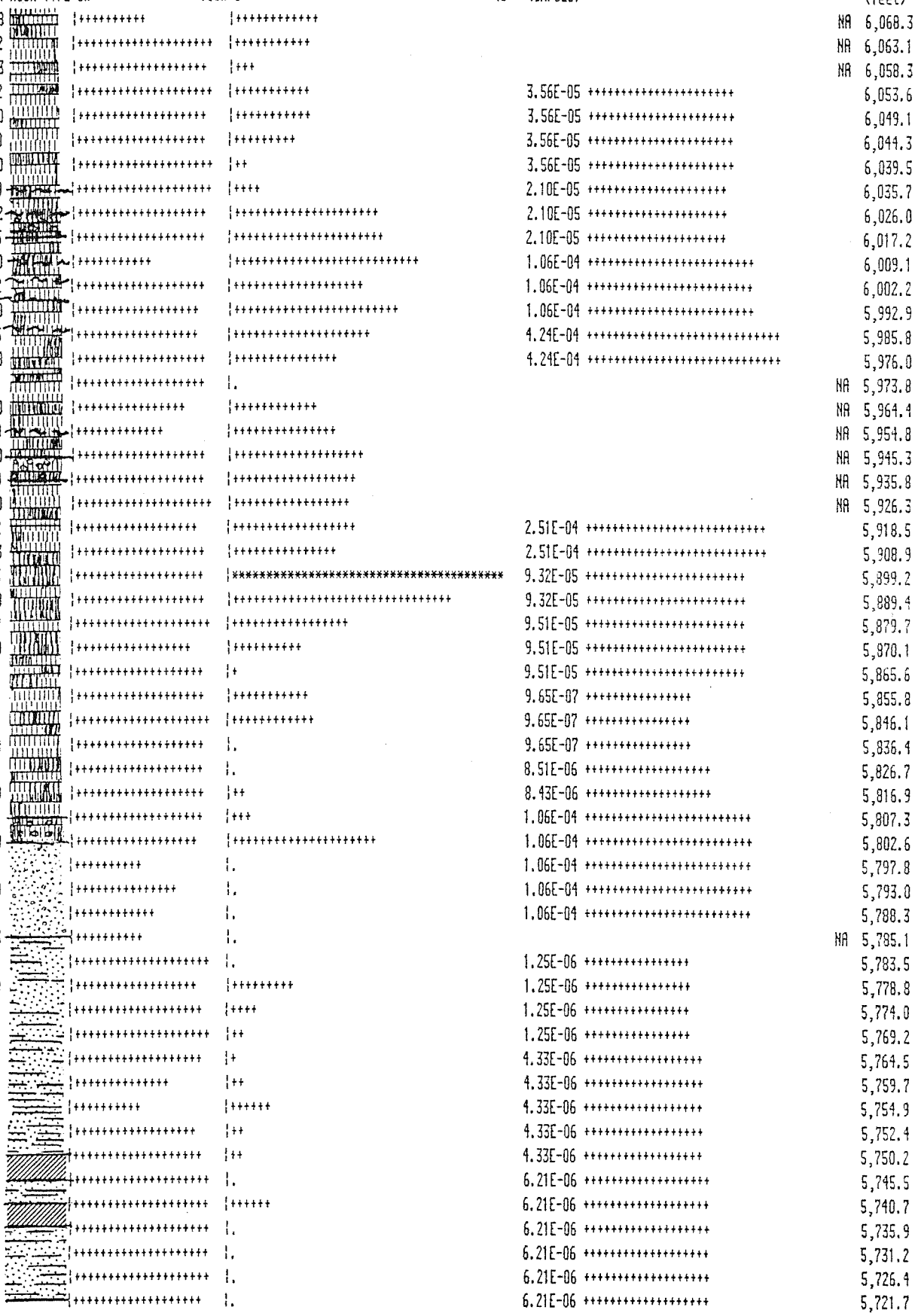
25-Jun-86

MINER FLAT DAM SITE: MF-105:FRACTURE LOG



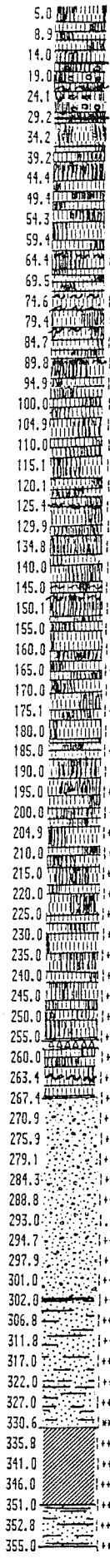
DEPTH	RECOVERY 0% 100%	NUMBER OF FRACTURES 90-70 0 10 20	70-50 0 10 20	50-30 0 10 20	30-0 0 10 20	ELEV. (feet)
5.8	++++++++++	++++++++	.	.	++++	6,068.3
11.2	++++++++++++++++++++	+++++++++	.	.	++	6,063.1
16.3	++++++++++++++++++++	++	+	.	.	6,058.3
21.2	++++++++++++++++++++	++++++	.	.	+++++	6,053.6
26.0	++++++++++++++++++++	++++++++	+	.	++	6,049.1
31.0	++++++++++++++++++++	+++++	.	++	++	6,044.3
36.0	++++++++++++++++++++	.	.	.	++	6,039.5
40.0	++++++++++++++++++++	++++	.	.	.	6,035.7
50.2	++++++++++++++++++++	++++++++	.	++++	++++++++++	6,026.0
59.5	++++++++++++++++++++	++++++++++	.	+++++	++++++	6,017.2
68.0	++++++++++	++++++++++++++++	.	+++++	+++++++	6,009.1
75.2	++++++++++++++++++++	++++++++	.	+++++++	++++	6,002.2
85.0	++++++++++++++++++	++++++++++++	++	++++	++++++	5,992.9
92.5	++++++++++++++++++	+++++++++	++++	.	+++++++	5,985.8
102.8	++++++++++++++++++++	++++++++++++	.	.	+++	5,976.0
105.1	++++++++++++++++++++	.	.	.	.	5,973.8
115.0	++++++++++++++++	++++++++	+	.	+++	5,964.4
125.0	++++++++++++	++++++++++	.	.	++++	5,954.8
135.0	++++++++++++++++++++	+++++	++++++++	.	++++++	5,945.3
145.0	++++++++++++++++++++	++++++++++++++++++	.	.	+	5,935.8
155.0	++++++++++++++++++++	++++++++++++	+++	+	+	5,926.3
163.2	++++++++++++++++++	++++++++	++++	+++	+++	5,918.5
173.3	++++++++++++++++++	++	+++++++	.	++++++	5,908.9
183.5	++++++++++++++++++++	++++++++++++++++++	+++++	++++++++++++++++	++++++++++++	5,899.2
193.8	++++++++++++++++++++	++++++++++	+++++++++	++++	+++++++++	5,889.4
203.9	++++++++++++++++++++	++++++	+++++++	.	++++	5,879.7
214.0	++++++++++++++++++	+++++++	+	.	++	5,870.1
218.8	++++++++++++++++++++	.	.	.	+	5,865.6
229.1	++++++++++++++++++++	+++++	++	.	++++	5,855.8
239.2	++++++++++++++++++++	+++++++	.	.	+++++	5,846.1
249.4	++++++++++++++++++++	.	.	.	.	5,836.4
259.6	++++++++++++++++++++	.	.	.	.	5,826.7
269.9	++++++++++++++++++++	.	.	.	++	5,816.9
280.0	++++++++++++++++++++	++	.	.	+	5,807.3
285.0	++++++++++++++++++	++++++	+++++	.	++++++++++	5,802.6
290.0	++++++++++	.	.	.	.	5,797.8
295.0	++++++++++++++++	.	.	.	.	5,793.0
300.0	++++++++++++	.	.	.	.	5,788.3
303.3	++++++++++	.	.	.	.	5,785.1
305.0	++++++++++++++++++++	.	.	.	.	5,783.5
310.0	++++++++++++++++++	.	.	+++++++++	.	5,778.8
315.0	++++++++++++++++++++	++++	.	.	.	5,774.0
320.0	++++++++++++++++++++	++	.	.	.	5,769.2
325.0	++++++++++++++++++++	+	.	.	.	5,764.5
330.0	++++++++++++++++	.	++	.	.	5,759.7
335.1	++++++++++	++++	.	++	.	5,754.9
337.7	++++++++++++++++++	.	.	++	.	5,752.4
340.0	++++++++++++++++++	++	.	.	.	5,750.2
345.0	++++++++++++++++++++	.	.	.	.	5,745.5
350.0	++++++++++++++++++++	.	.	.	++++++	5,740.7
355.0	++++++++++++++++++++	.	.	.	.	5,735.9
360.0	++++++++++++++++++++	.	.	.	.	5,731.2
365.0	++++++++++++++++++++	.	.	.	.	5,726.4
370.0	++++++++++++++++++++	.	.	.	.	5,721.7

MINER FLAT DAM SITE: MF-105: FRACTURES and HYDRAULIC CONDUCTIVITY



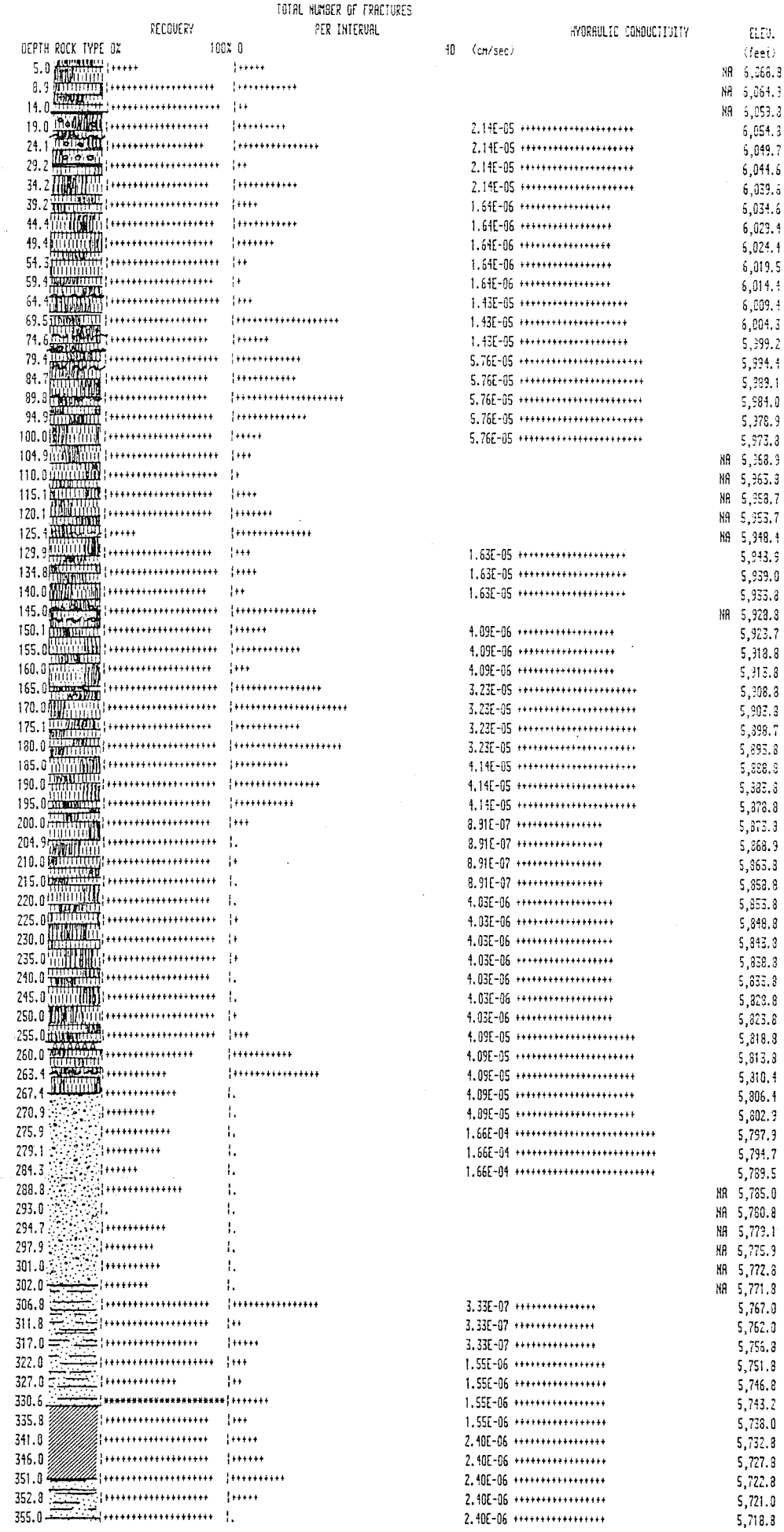
25-Jun-86

MINER FLAT DAM SITE: MF-106: ENGINEERING GEOLOGIC LOG



DEPTH	ROCK TYPE	RECOVERY 0%	RECOVERY 100%	RQD 0%	RQD 100%	SCR 0%	SCR 100%	STRENGTH 0	STRENGTH 40,000 psi	ELEV. (feet)
5.0		+++++	+++	.		.		+++++++++++++++	16,844	6,068.8
8.9		++++++++++++++++++	++++	.		.		++++++++++++++++++++	21,152	6,064.9
14.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++	18,274	6,059.8
19.0		++++++++++++++++	++++++++++++++	++++++++++++		++++++++++++		++++++++++	11,123	6,054.8
24.1		++++++++++++++++	++++++++++	++++++++++		++++++++++		++++++++++++++++	17,479	6,049.7
29.2		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++++	23,438	6,044.6
34.2		++++++++++++++++++	++++++++++	+++++		++++++++		++++++++++++++++++++++++++	27,490	6,039.6
39.2		++++++++++++++++++	++++++++++++++++++	++++++++		++++++++++		++++++++++++++++++++	21,769	6,034.6
44.4		++++++++++++++++++	++++++++++++	++++		++++++++++++		++++++++++++++++++++++++	25,424	6,029.4
49.4		++++++++++++++++++	++++++++++++++++++	++++++++++++		++++++++++++		++++++++++++++++++++	22,246	6,024.4
54.3		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++++++++	27,490	6,019.5
59.4		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++	22,564	6,014.4
64.4		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++++++++	27,808	6,009.4
69.5		++++++++++++++++++	++++++	++++		++++++++++++++++++		++++++++++++++++++	20,022	6,004.3
74.6		++++++++++++++++++	++++++++++++++	++++++++++++		++++++++++++		++++++++++++++++	18,274	5,999.2
79.4		++++++++++++++++++	++++++++++++++	+++++++		++++++++++++++++++		++++++++++++++++++++	22,723	5,994.4
84.7		++++++++++++++++++	++++++++++++++++	+++++++++		++++++++++++++++++		++++++++++++++++++	20,975	5,989.1
89.8		++++++++++++++++++	++++++++++++	.		++++++++++++++++++		++++++++++++++++++	18,433	5,984.0
94.9		++++++++++++++++++	++++++++++++++	++++++++++		++++++++++		++++++++++++++++++++++	23,041	5,978.9
100.0		++++++++++++++++++	++++++++++++++	++++++++++		++++++++++		++++++++++++++++++	19,227	5,973.8
104.9		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++	17,479	5,968.9
110.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++++++++	28,284	5,963.8
115.1		++++++++++++++++++	++++++++++++++++	++++++++++++++		++++++++++++++		++++++++++++++++++++	23,835	5,958.7
120.1		++++++++++++++++++	++++++++++++++++	++++++		++++++++++++++		++++++++++++++++++	19,068	5,953.7
125.4		+++++	.	.		++++++++++++++		++++++++++++++++++	18,274	5,948.4
129.9		++++++++++++++++++	++++++++++++++++++	++++++++++++++		++++++++++++++		++++++++++++++++++++	23,041	5,943.9
134.8		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++		++++++++++++	13,507	5,939.0
140.0		++++++++++++++++++	++++++++++++++	.		++++++++++++++		++++++++++++	12,394	5,933.8
145.0		++++++++++++++++++	++++++++++++++	.		++++++++++++++		+++++++	7,945	5,928.8
150.1		++++++++++++++++++	++++++++++++++++	++++++++		++++++++++++++		++++++++++++++++	17,797	5,923.7
155.0		++++++++++++++++++	++++++++++	.		++++++++++++++		++++++++++++++++++++	23,835	5,918.8
160.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++		++++++++++++++++++++	22,564	5,913.8
165.0		++++++++++++++++++	+++++	.		++++++++++++++		++++++++++++++++++++++++	27,490	5,908.8
170.0		++++++++++++++++++	++++	.		++++++++++++++		++++++++++++++++++++++++	27,172	5,903.8
175.1		++++++++++++++++++	++++++++++	+++		++++++++++++++		++++++++++++++++++++++++++++++++++	35,753	5,898.7
180.0		++++++++++++++++++	++++++++	.		++++++++++++++		++++++++++++++++++++	25,742	5,893.8
185.0		++++++++++++++++++	++++++++	++++++		++++++++++++++		++++++++++++++++	20,657	5,888.8
190.0		++++++++++++++++++	++++++++++	++++		++++++++++++++		++++++++++++++++++++++	28,602	5,883.8
195.0		++++++++++++++++++	++++++++++	++++		++++++++++++++		++++++++++++++++++++++	28,125	5,878.8
200.8		++++++++++++++++++	++++++++++++++++	++++++++++++++		++++++++++++++		++++++++++++++++++++++++++++	33,528	5,873.8
204.9		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++	30,827	5,868.9
210.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++++	32,893	5,863.8
215.0		++++++++++++++++++	++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++	31,939	5,858.8
220.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++	30,509	5,853.8
225.0		++++++++++++++++++	++++++++++++++++	++++++++++++++++		++++++++++++++++		++++++++++++++++++++	31,939	5,848.8
230.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++	30,827	5,843.8
235.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++	30,509	5,838.8
240.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++	30,191	5,833.8
245.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++	30,191	5,828.8
250.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++		++++++++++++++++++		++++++++++++++++++++++	32,734	5,823.8
255.0		++++++++++++++++++	++++++++++++++++	++++++++++++++++		++++++++++++++++		++++++++++++++++++++++	32,416	5,818.8
260.0		++++++++++++++++	+++	.		.		++	2,860	5,813.8
263.4		++++++++++	.	.		.		.	0	5,810.4
267.4		++++++++++++	.	.		.		.	0	5,806.4
270.9		+++++++++	.	.		.		.	0	5,802.9
275.9		++++++++++++	.	.		.		.	0	5,797.9
279.1		++++++++++	.	.		.		.	0	5,794.7
284.3		++++++	.	.		.		.	0	5,789.5
288.8		++++++++++++++	.	.		.		.	0	5,785.0
293.0		.	.	.		.		.	0	5,780.8
294.7		++++++++++	.	.		.		.	0	5,779.1
297.9		+++++++++	.	.		.		.	0	5,775.9
301.0		++++++++++	.	.		.		.	0	5,772.8
302.0		++++++++	.	.		.		.	0	5,771.8
306.8		++++++++++++++++++	++++++++++++	+++++				+	1,700	5,767.0
311.8		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++				++	2,701	5,762.0
317.0		++++++++++++++++++	++++++++++++	+++++				.	953	5,756.8
322.0		++++++++++++++++++	++++++++++++++++++	++++++++++++				+	1,748	5,751.8
327.0		++++++++++++	++++++++++++	++++++++++++				+++++++	7,468	5,746.8
330.6		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++				+	1,351	5,743.2
335.8		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++				+++++	5,562	5,738.0
341.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++				++	2,066	5,732.8
346.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++				++	2,463	5,727.8
351.0		++++++++++++++++++	++++++++++++++	++++++++				++++++++	8,263	5,722.8
352.8		++++++++++++++++++	++++++++++++	.				+++	3,814	5,721.0
355.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++				++++	4,608	5,718.8

DEPTH	ROCK TYPE	RECOVERY 0% 100%	NUMBER OF FRACTURES 90-70 0 10 20	70-50 0 10 20	50-30 0 10 20	30-0 0 10 20	ELEV. (feet)
5.0		++++	+++	.	.	++	6,068.8
8.9		++++++++++++++++++	++++	.	++++	+++	6,064.9
14.0		++++++++++++++++++	.	.	.	++	6,059.8
19.0		++++++++++++++++	++++++	.	.	+++	6,054.8
24.1		++++++++++++++++	++++++++	.	++++	+++	6,049.7
29.2		++++++++++++++++++	+	.	.	+	6,044.6
34.2		++++++++++++++++	+++++	.	+++	+++	6,039.6
39.2		++++++++++++++++++	++	+	.	+	6,034.6
44.4		++++++++++++++++	+++++++	.	+++	+	6,029.4
49.4		++++++++++++++++	+++++	++	.	.	6,024.4
54.3		++++++++++++++++++	+	.	.	+	6,019.5
59.4		++++++++++++++++	.	.	+	.	6,014.4
64.4		++++++++++++++++++	.	.	.	+++	6,009.4
69.5		++++++++++++++++	++++++++	.	+++++	+++++++	6,004.3
74.6		++++++++++++++++	++++	.	.	++	5,999.2
79.4		++++++++++++++++++	+++++++	.	+++	++	5,994.4
84.7		++++++++++++++++	++++++++	+++	.	.	5,989.1
89.8		++++++++++++++++	++++++++++	+++++	.	+++++	5,984.0
94.9		++++++++++++++++++	+++++++	++	.	++++	5,978.9
100.0		++++++++++++++++++	++++	.	.	+	5,973.8
104.9		++++++++++++++++++	+++	.	.	.	5,968.9
110.0		++++++++++++++++++	+	.	.	.	5,963.8
115.1		++++++++++++++++++	+++	.	.	+	5,958.7
120.1		++++++++++++++++++	++++	+	.	++	5,953.7
125.4		+++++	++++++++++	.	.	+++	5,948.4
129.9		++++++++++++++++++	+++	.	.	.	5,943.9
134.8		++++++++++++++++++	+++	.	.	+	5,939.0
140.0		++++++++++++++++	+	.	.	+	5,933.8
145.0		++++++++++++++++++	++++++++++	.	.	+++	5,928.8
150.1		++++++++++++++++++	++++	.	.	++	5,923.7
155.0		++++++++++++++++++	+++++++	.	+++	++	5,918.8
160.0		++++++++++++++++++	+++	.	.	.	5,913.8
165.0		++++++++++++++++++	+++++++	++++	.	+++++	5,908.8
170.0		++++++++++++++++++	+++++++++	+++++	.	+++++++	5,903.8
175.1		++++++++++++++++++	++++++++	.	.	++++	5,898.7
180.0		++++++++++++++++++	++++++++++	++++++	.	+++	5,893.8
185.0		++++++++++++++++++	++	++++	.	++++	5,888.8
190.0		++++++++++++++++++	+++++	.	++++	+++++++	5,883.8
195.0		++++++++++++++++++	+++	+++++	.	+++	5,878.8
200.0		++++++++++++++++	.	.	++	+	5,873.8
204.9		++++++++++++++++++	.	.	.	.	5,868.9
210.0		++++++++++++++++	+	.	.	.	5,863.8
215.0		++++++++++++++++++	.	.	.	.	5,858.8
220.0		++++++++++++++++	.	.	.	.	5,853.8
225.0		++++++++++++++++++	.	.	.	+	5,848.8
230.0		++++++++++++++++++	+	.	.	.	5,843.8
235.0		++++++++++++++++++	+	.	.	.	5,838.8
240.0		++++++++++++++++	.	.	.	.	5,833.8
245.0		++++++++++++++++++	.	.	.	.	5,828.8
250.0		++++++++++++++++++	.	+	.	.	5,823.8
255.0		++++++++++++++++++	++	.	+	.	5,818.8
260.0		++++++++++++++	++++++	.	.	+++++	5,813.8
263.1		++++++++++	++++++++++	++++++	.	.	5,810.4
267.4		++++++++++++	.	.	.	.	5,806.4
270.9		+++++++++	.	.	.	.	5,802.9
275.9		++++++++++++	.	.	.	.	5,797.9
279.1		++++++++++	.	.	.	.	5,794.7
284.3		++++++	.	.	.	.	5,789.5
288.8		++++++++++++++	.	.	.	.	5,785.0
293.0		.	.	.	.	.	5,780.8
294.7		++++++++++	.	.	.	.	5,779.1
297.9		+++++++++	.	.	.	.	5,775.9
301.0		++++++++++	.	.	.	.	5,772.8
302.0		++++++++	.	.	.	.	5,771.8
306.8		++++++++++++++++++	+++++++++	+	.	++++++	5,767.0
311.8		++++++++++++++++++	+	.	+	.	5,762.0
317.0		++++++++++++++++	.	.	+++	++	5,756.8
322.0		++++++++++++++++++	.	.	.	+++	5,751.8
327.0		++++++++++++	.	++	.	.	5,746.8
330.6		++++++++++++++++++++	.	.	+++++++	.	5,743.2
335.8		++++++++++++++++++	+++	.	.	.	5,738.0
341.0		++++++++++++++++++	+++++	.	.	.	5,732.8
346.0		++++++++++++++++++	.	++++++	.	.	5,727.8
351.0		++++++++++++++++++	+++++	.	.	+++++	5,722.8
352.8		++++++++++++++++++	.	.	+++++	.	5,721.0
355.0		++++++++++++++++++	.	.	.	.	5,718.8

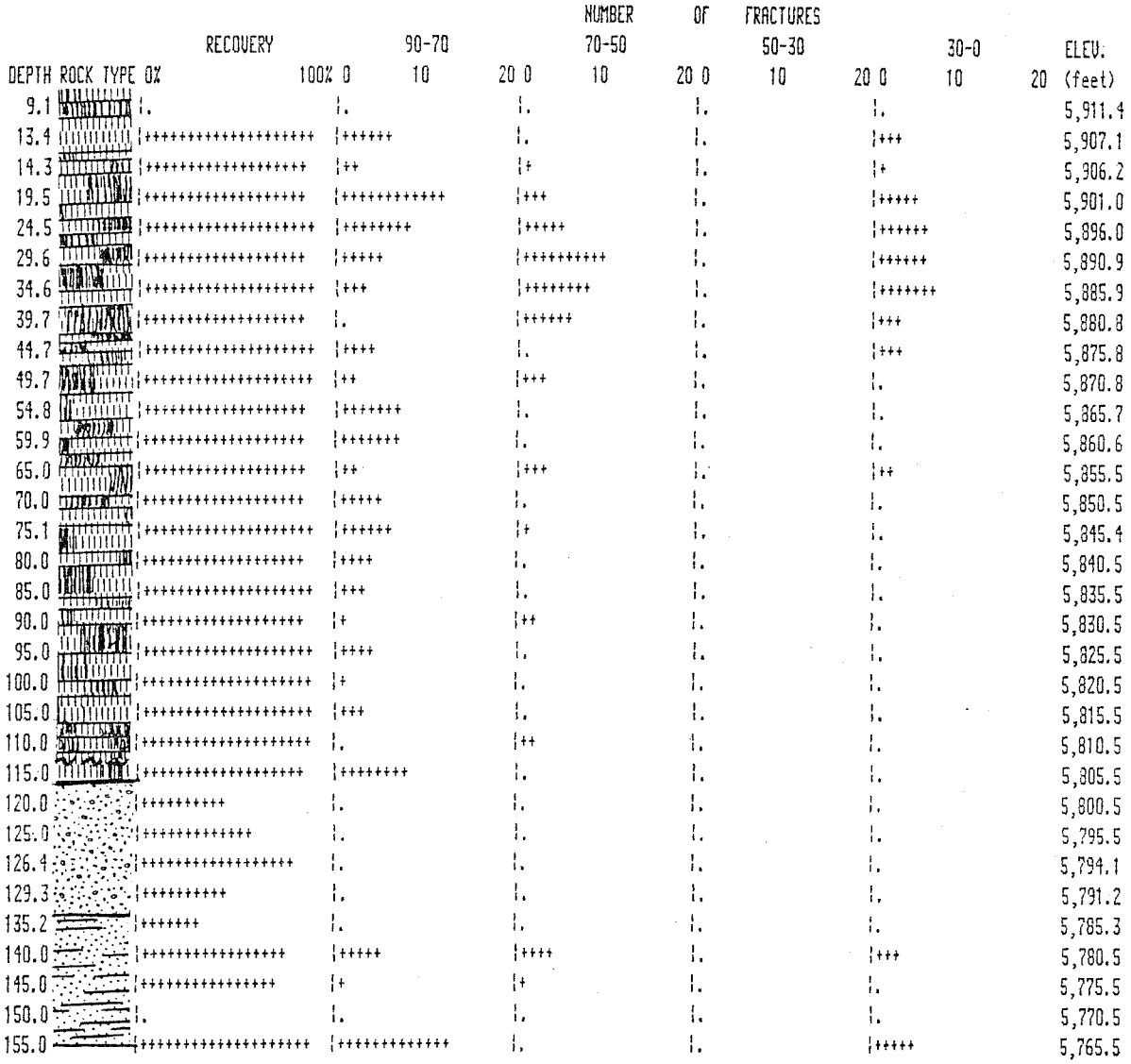

TOTAL NUMBER OF FRACTURES
RECOVERY
PER INTERVAL
HYDRAULIC CONDUCTIVITY
ELEV.
DEPTH ROCK TYPE 0% 100% 0 40 (cm/sec) (feet)
5.0 NA 6,068.8
8.9 NA 6,064.3
14.0 NA 6,059.8
19.0 2.14E-05 6,054.8
24.1 2.14E-05 6,049.7
29.2 2.14E-05 6,044.6
34.2 2.14E-05 6,039.6
39.2 1.64E-06 6,034.6
44.4 1.64E-06 6,029.4
49.4 1.64E-06 6,024.4
54.3 1.64E-06 6,019.5
59.4 1.64E-06 6,014.4
64.4 1.43E-05 6,009.4
69.5 1.43E-05 6,004.3
74.6 1.43E-05 5,999.2
79.4 5.76E-05 5,994.4
84.7 5.76E-05 5,989.1
89.8 5.76E-05 5,984.0
94.9 5.76E-05 5,978.9
100.0 5.76E-05 5,973.8
104.9 NA 5,968.9
110.0 NA 5,963.8
115.1 NA 5,958.7
120.1 NA 5,953.7
125.1 NA 5,948.7
129.9 1.63E-05 5,943.9
134.8 1.63E-05 5,939.0
140.0 1.63E-05 5,933.8
145.0 NA 5,928.8
150.1 4.09E-06 5,923.7
155.0 4.09E-06 5,918.8
160.0 4.09E-06 5,913.8
165.0 3.23E-05 5,908.8
170.0 3.23E-05 5,903.8
175.1 3.23E-05 5,898.7
180.0 3.23E-05 5,893.8
185.0 4.14E-05 5,888.8
190.0 4.14E-05 5,883.8
195.0 4.14E-05 5,878.8
200.0 8.91E-07 5,873.8
204.9 8.91E-07 5,868.9
210.0 8.91E-07 5,863.8
215.0 8.91E-07 5,858.8
220.0 4.03E-06 5,853.8
225.0 4.03E-06 5,848.8
230.0 4.03E-06 5,843.8
235.0 4.03E-06 5,838.8
240.0 4.03E-06 5,833.8
245.0 4.03E-06 5,828.8
250.0 4.03E-06 5,823.8
255.0 4.09E-05 5,818.8
260.0 4.09E-05 5,813.8
263.4 4.09E-05 5,810.4
267.4 4.09E-05 5,806.4
270.9 4.09E-05 5,802.9
275.9 1.66E-04 5,797.9
279.1 1.66E-04 5,794.7
284.3 1.66E-04 5,789.5
288.8 NA 5,785.0
293.0 NA 5,780.8
294.7 NA 5,779.1
297.9 NA 5,775.9
301.8 NA 5,772.0
302.0 NA 5,771.8
306.8 3.33E-07 5,767.0
311.8 3.33E-07 5,762.0
317.0 3.33E-07 5,756.8
322.0 1.55E-06 5,751.8
327.0 1.55E-06 5,746.8
330.6 1.55E-06 5,743.2
335.8 1.55E-06 5,738.0
341.0 2.40E-06 5,732.8
346.0 2.40E-06 5,727.8
351.0 2.40E-06 5,722.8
352.8 2.40E-06 5,721.0
355.0 2.40E-06 5,718.8

25-Jun-86

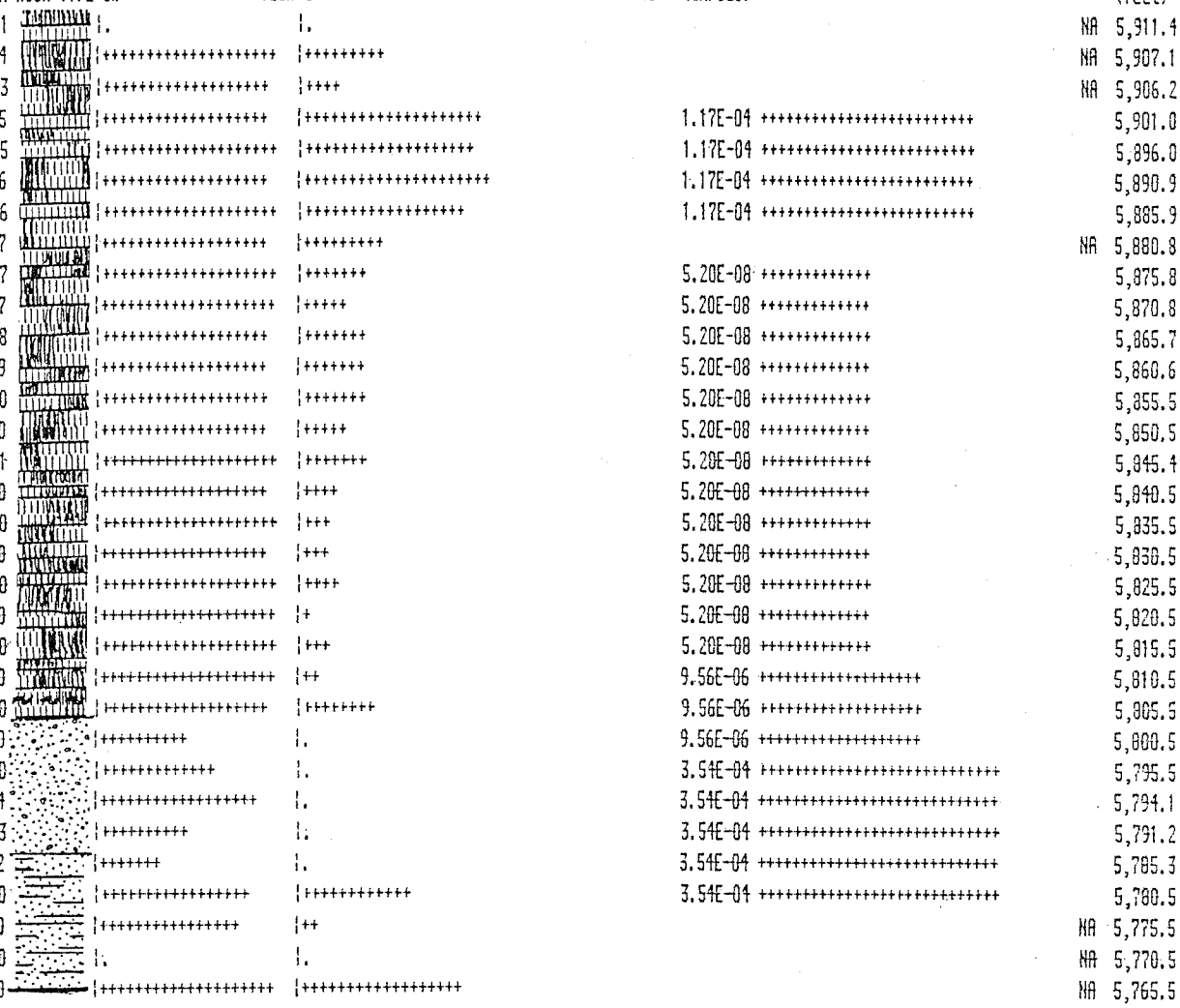
MINER FLAT DAM SITE: MF-113: ENGINEERING GEOLOGIC LOG

DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
9.1						0	5,911.4
13.4						30,986	5,907.1
14.3						31,780	5,906.2
19.5						33,051	5,901.0
24.5						38,136	5,896.0
29.6						33,369	5,890.9
34.6						31,780	5,885.9
39.7						33,051	5,880.8
44.7						32,734	5,875.8
49.7						32,416	5,870.8
54.8						31,780	5,865.7
59.9						37,183	5,860.6
65.0						33,369	5,855.5
70.0						31,462	5,850.5
75.1						31,462	5,845.4
80.0						28,602	5,840.5
85.0						30,986	5,835.5
90.0						35,594	5,830.5
95.0						36,071	5,825.5
100.0						34,164	5,820.5
105.0						33,369	5,815.5
110.0						42,109	5,810.5
115.0						34,958	5,805.5
120.0						0	5,800.5
125.0						0	5,795.5
126.4						0	5,794.1
129.3						0	5,791.2
135.2						0	5,785.3
140.0						5,562	5,780.5
145.0						1,271	5,775.5
150.0						0	5,770.5
155.0						1,510	5,765.5

MINER FLAT DAM SITE: MF-113: FRACTURE LOG



MINER FLAT DAM SITE: MF-113: FRACTURES and HYDRAULIC CONDUCTIVITY



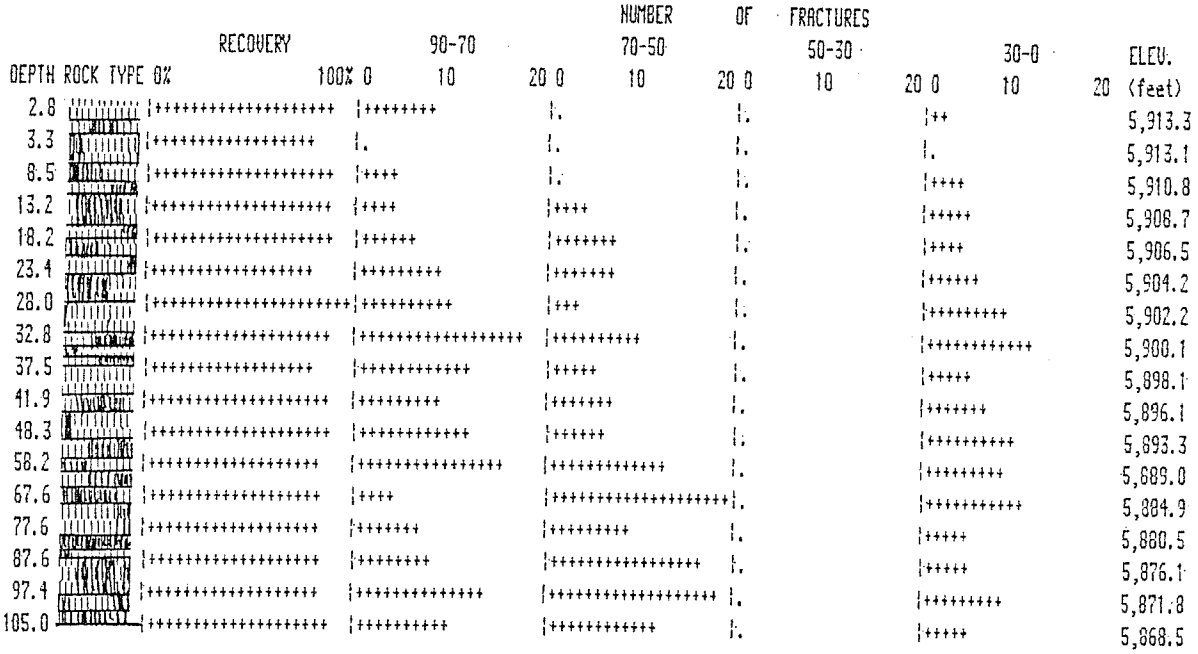
25-Jun-86

MINER FLAT DAM SITE: MF-117: ENGINEERING GEOLOGIC LOG

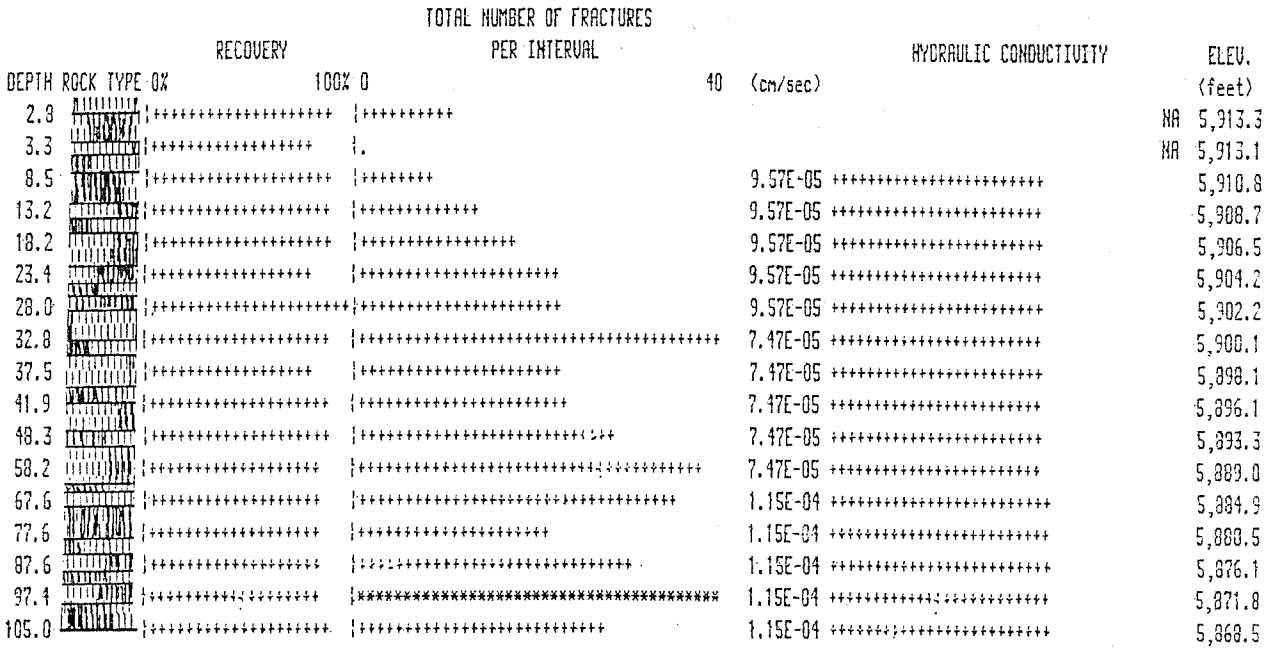
DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
2.8		++++++++++++++++++++	++++++++++++	.	++++++++++++++++++++++++++++	27,013	5,913.3
3.3		++++++++++++++++++	++++++++++++++++++++	.	++++++++++++++++++++++++	25,424	5,913.1
8.5		++++++++++++++++++++	++++++++++++++++	+++	++++++++++++++++++++++++++	26,219	5,910.8
13.2		++++++++++++++++++++	++++++++++++	++++	++++++++++++++++++++++++	25,424	5,908.7
18.2		++++++++++++++++++++	++++++++	.	++++++++++++++++++++++++++++	28,602	5,906.5
23.4		++++++++++++++++++	+++	.	++++++++++++++++++++++++++++++	31,462	5,904.2
28.8		++++++++++++++++++++++	+++++	.	++++++++++++++++++++++++++++	30,986	5,902.2
32.8		++++++++++++++++++++	.	.	++++++++++++++++++++++++++++++++	34,005	5,900.1
37.5		++++++++++++++++++	++	.	++++++++++++++++++++++++++++++++	34,164	5,898.1
41.9		++++++++++++++++++++	+++++	.	++++++++++++++++++++++++++++++++++	36,229	5,896.1
48.3		++++++++++++++++++++	++	.	++++++++++++++++++++++++++++++++	35,594	5,893.3
58.2		++++++++++++++++++	++++	.	++++++++++++++++++++++++++++++++	35,276	5,889.0
67.6		++++++++++++++++++++	++++	.	++++++++++++++++++++++++++++++++	35,753	5,884.9
77.6		++++++++++++++++++	++++++++++	++++	++++++++++++++++++++++++++++++++++	36,547	5,880.5
87.6		++++++++++++++++++++	++++++++++	.	++++++++++++++++++++++++++++++++++	36,547	5,876.1
97.4		++++++++++++++++++++	++++	.	++++++++++++++++++++++++++++++++	34,005	5,871.8
105.0		++++++++++++++++++++	++++++	.	++++++++++++++++++++++++++++++++	35,912	5,868.5

25-Jun-86

MINER FLAT DAM SITE: MF-117: FRACTURE LOG

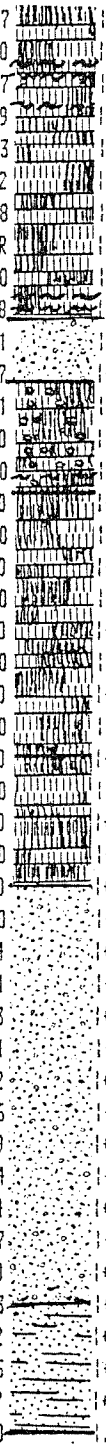


MINER FLAT DAM SITE: MF-t17: FRACTURES and HYDRAULIC CONDUCTIVITY



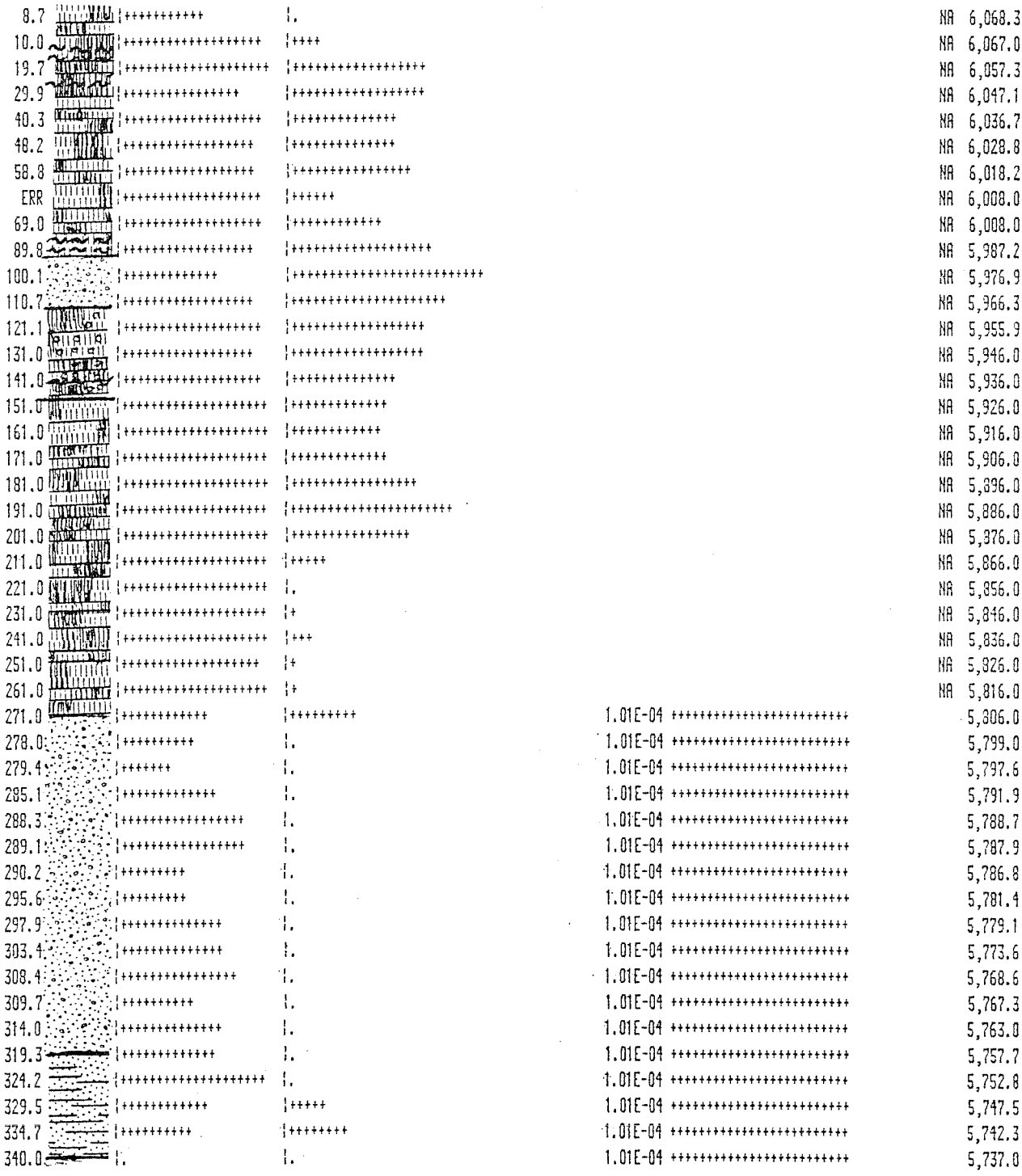
DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
8.7		++++++++++	.	.	.	0	6,068.3
10.0		++++++++++++++++++	++++++++++++++	.	+++++++++	9,534	6,067.0
19.7		++++++++++++++++++++	++++++++++++++	++++++	+++++++	7,945	6,057.3
29.9		++++++++++++++++	++++++++++++	+++++++	++++++++++++++++++++	21,928	6,047.1
40.3		++++++++++++++++++	+++++++++++++++	++++++++	+++++++++++++++	14,301	6,036.7
48.2		++++++++++++++++++	++++++++++	++++++	++++++++++++++++++++	22,246	6,028.8
58.8		++++++++++++++++++	++++++++++	++++++	++++++++++++++++++++++++++++	27,013	6,018.2
ERR		++++++++++++++++++	++++++++++++++++	++++++++++++++++	++++++++++++++++++++++++	25,265	ERR
69.0		++++++++++++++++++	++++++++++++++	++++++++++	++++++++++++++++++++	20,657	6,008.0
89.8		++++++++++++++++++	++++++++++	.	++++++++++	10,646	5,987.2
100.1		++++++++++++++	++	++	++++++++++	11,123	5,976.9
110.7		++++++++++++++++++	++++++++++++	++++++++++	++++++++++++++++++	18,274	5,966.3
121.1		++++++++++++++++++++	+++++++	+++++	++++++++++++++++++++++	23,517	5,955.9
131.0		++++++++++++++++++	++++++++++	+++++++	++++++++++++++++	16,844	5,946.0
141.0		++++++++++++++++++	++++++++++++++++	++++++++++	++++++++++++	13,824	5,936.0
151.0		++++++++++++++++++++	++++++++++++++++	++++++++++++	+++++++++	9,534	5,926.0
161.0		++++++++++++++++++++	++++++++++	+++++++++	++++++++++++++++++++++++	24,789	5,916.0
171.0		++++++++++++++++++++	++++++++++++	++++++	++++++++++++++++++++++++	24,630	5,906.0
181.0		++++++++++++++++++++	++++++++++++++++	++++++++++	++	38,136	5,896.0
191.0		++++++++++++++++++++	++++++++	+++++	++	38,454	5,886.0
201.0		++++++++++++++++++++	++++++++++++	+++++++	++++++++++++++++++++++++++++++++++	36,547	5,876.0
211.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++	++	39,725	5,866.0
221.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	**42,109		5,856.0
231.0		++++++++++++++++++++	+++++	+++++	++++++++++++++++++++++++++++++++++	35,753	5,846.0
241.0		++++++++++++++++++++	++++++++++++++	++++++++++++++	++++++++++++++++++++++++++++++++	33,846	5,836.0
251.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++++++++++++++++	37,342	5,826.0
261.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++++++++++++++++++++	38,454	5,816.0
271.0		++++++++++++	++++++++	++++++	++++++++++++++++++++++++++	27,808	5,806.0
278.0		++++++++++	.	.	.	0	5,799.0
279.4		+++++++	.	.	.	0	5,797.6
285.1		++++++++++++++	.	.	.	0	5,791.9
288.3		++++++++++++++++	.	.	.	0	5,788.7
289.1		++++++++++++++++	.	.	.	0	5,787.9
290.2		+++++++++	.	.	.	0	5,786.8
295.6		+++++++++	.	.	.	0	5,781.4
297.9		+++++++++++++++	.	.	.	0	5,779.1
303.4		+++++++++++++++	.	.	.	0	5,773.6
308.4		++++++++++++++++	.	.	.	0	5,768.6
309.7		++++++++++	.	.	.	0	5,767.3
314.0		+++++++++++++++	.	.	.	0	5,763.0
319.3		++++++++++++++	.	.	.	0	5,757.7
324.2		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	.	795	5,752.8
329.5		++++++++++++	++++++	++++++	+	1,271	5,747.5
334.7		++++++++++	.	.	+++	3,496	5,742.3
340.0		.	.	.	.	0	5,737.0

MINER FLAT DAM SITE: MF-118: FRACTURE LOG

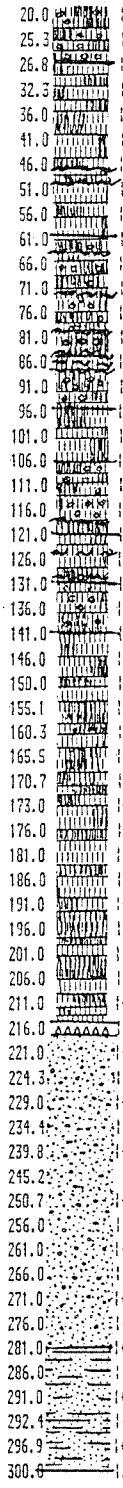


DEPTH	ROCK TYPE	RECOVERY 0% 100%	NUMBER OF FRACTURES 90-70 0 10	70-50 0 10	50-30 0 10	30-0 0 10 20	ELEV. (feet)
8.7		++++++++++	.	.	.	.	6,068.3
10.0		++++++++++++++++++	++++	.	.	.	6,067.0
19.7		++++++++++++++++++	++++++++++	++++	.	++++	6,057.3
29.9		++++++++++++++++	++++++++++	++	.	+++++	6,047.1
40.3		++++++++++++++++++	++++++++++	.	.	++++	6,036.7
48.2		++++++++++++++++++	++++++++++	.	.	++++	6,028.8
58.8		++++++++++++++++++	++++++++	++++	.	++++	6,018.2
ERR		++++++++++++++++++	++++	+	.	+	6,008.0
69.0		++++++++++++++++++	++++++++	++	.	++	5,997.6
89.8		++++++++++++++++++	+++++++++	++++++	.	++++	5,987.2
100.1		++++++++++++++	++++++++	++++++++	.	++++++++++	5,976.9
110.7		++++++++++++++++++	++++++	+++++++	.	++++++++	5,966.3
121.1		++++++++++++++++++	++++++++++	++	.	+++++	5,955.9
131.0		++++++++++++++++++	++++++	+++	.	+++++++++	5,946.0
141.0		++++++++++++++++++	++++++++	+++	.	+++	5,936.0
151.0		++++++++++++++++++	++++++++++	+	.	+	5,926.0
161.0		++++++++++++++++++	+++++++	+	.	++++	5,916.0
171.0		++++++++++++++++++	++++++	++++	.	+++	5,906.0
181.0		++++++++++++++++++	++++++++++	++	.	+++	5,896.0
191.0		++++++++++++++++++	++++++++	+++++++++	.	+++++	5,886.0
201.0		++++++++++++++++++	+++++++	+++	.	++++++	5,876.0
211.0		++++++++++++++++++	+++++	.	.	.	5,866.0
221.0		++++++++++++++++++	.	.	.	.	5,856.0
231.0		++++++++++++++++++	.	.	.	+	5,846.0
241.0		++++++++++++++++++	.	+	.	++	5,836.0
251.0		++++++++++++++++++	.	.	.	+	5,826.0
261.0		++++++++++++++++++	.	.	.	+	5,816.0
271.0		++++++++++++	+++++	.	.	++++	5,806.0
278.0		++++++++++	.	.	.	.	5,799.0
279.4		+++++++	.	.	.	.	5,797.6
285.1		++++++++++++++	.	.	.	.	5,791.9
288.3		++++++++++++++++	.	.	.	.	5,788.7
289.1		++++++++++++++++	.	.	.	.	5,787.9
290.2		+++++++++	.	.	.	.	5,786.8
295.6		+++++++++	.	.	.	.	5,781.4
297.9		+++++++++++++++	.	.	.	.	5,779.1
303.4		+++++++++++++++	.	.	.	.	5,773.6
308.4		++++++++++++++++	.	.	.	.	5,768.6
309.7		++++++++++	.	.	.	.	5,767.3
314.0		+++++++++++++++	.	.	.	.	5,763.0
319.3		++++++++++++++	.	.	.	.	5,757.7
324.2		++++++++++++++++++++	.	.	.	.	5,752.8
329.5		++++++++++++	++++	+	.	.	5,747.5
334.7		++++++++++	+++++	.	.	+++	5,742.3
340.0		.	.	.	.	.	5,737.0

MINER FLAT DAM SITE: MF-118: FRACTURES and HYDRAULIC CONDUCTIVITY



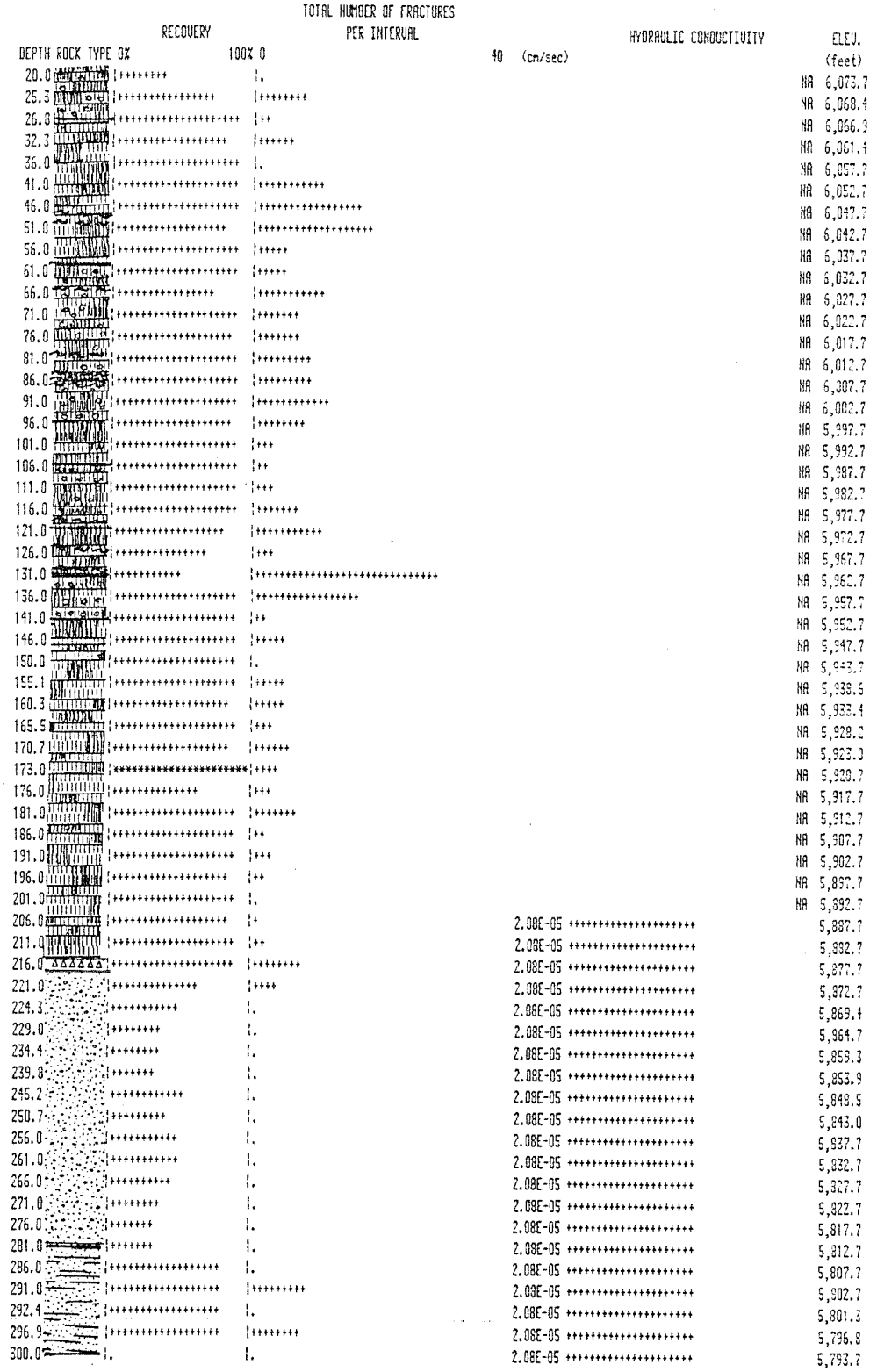
25-Jun-86 MINER FLAT DAM SITE: MF-119: ENGINEERING GEOLOGIC LOG



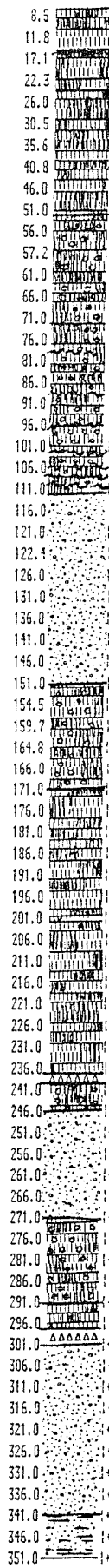
DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
20.0		++++++++	.	.	.	0	6,073.7
25.3		++++++++++++++++	++++++++++	.	++++++++++++++++++	19,545	6,068.4
26.8		++++++++++++++++++++	++++++++++++	.	++++++++++++++++++	18,750	6,066.9
32.3		++++++++++++++++++	++++++++++++++++	+++++	++++++++++++++	15,890	6,061.4
36.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++	++++++++++++	13,983	6,057.7
41.0		++++++++++++++++++++	+++++++++	++++++	++++++++++++	12,394	6,052.7
46.0		++++++++++++++++++++	+++++++++	++++	++++++++++++++++++++++++	24,789	6,047.7
51.0		++++++++++++++++++	.	.	++++++++++++++++++	19,704	6,042.7
56.0		++++++++++++++++++++	++++++++++++++++	++++++++++++++++	++++++++++++++++++++	22,246	6,037.7
61.0		++++++++++++++++++++	++++++++++++++++	++++++++++++	++++++++++++++++++++++++	25,424	6,032.7
66.0		++++++++++++++++	+++++++	.	++++++++++++++++++	18,274	6,027.7
71.0		++++++++++++++++++++	++++++++++++++++	++++++++++++	++++++++++++++++++++	20,657	6,022.7
76.0		++++++++++++++++++	++++++++++++++	++++++++++	++++++++++++++++	17,479	6,017.7
81.0		++++++++++++++++++++	++++++++++++	+++++	++++++++++++++	15,572	6,012.7
86.0		++++++++++++++++++++	++++++++++++++	++++++++++	++++++++++++++++++	19,863	6,007.7
91.0		++++++++++++++++++++	++++++++++++++	+++++++	++	2,384	6,002.7
96.0		++++++++++++++++++	++++++++++++++	++++++	+++++++++++++++++	17,797	5,997.7
101.0		++++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++++	25,424	5,992.7
106.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++++++++++++	30,986	5,987.7
111.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++++++++	27,331	5,982.7
116.0		++++++++++++++++++++	++++++++++++	++++++++++	++++++++++++++++++++++++++	27,808	5,977.7
121.0		++++++++++++++++++	+++++++++	.	++++++++++++++++	16,208	5,972.7
126.0		++++++++++++++++	++++++++	.	++++++++++++++	15,890	5,967.7
131.0		++++++++++	.	.	.	0	5,962.7
136.0		++++++++++++++++++++	+++++	.	++++++++	8,422	5,957.7
141.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++	++++++++++++++++	17,161	5,952.7
146.0		++++++++++++++++++++	++++++++++++++++++	++++++++++	++++++++++++++++++++	21,611	5,947.7
150.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	23,835	5,943.7
155.1		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++	++++++++++++++++++++++	24,153	5,938.6
160.3		++++++++++++++++++	++++++++++++++++++	++++++++++++++++	++++++++++++++++++++++++++	27,808	5,933.4
165.5		++++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++++++	28,602	5,928.2
170.7		++++++++++++++++++	++++++++++++++++	++++++++++	++++++++++++++++++++++++++	28,602	5,923.0
173.0		++++++++++++++++++++	++++++++++++++++++++	+++++++++	++++++++++++++++++++++++++	27,649	5,920.7
176.0		++++++++++++++	++++++++++++++	.	++++++++++++++++++++++++++	27,013	5,917.7
181.0		++++++++++++++++++++	++++++++++++++++++++	++++	++++++++++++++++++++++++++	28,602	5,912.7
186.0		++++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++++++++	30,191	5,907.7
191.0		++++++++++++++++++++	++++++++++++++++++	++++++++++++++	++++++++++++++++++++++++++++	31,145	5,902.7
196.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++++++++	29,873	5,897.7
201.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++++++++++	30,191	5,892.7
206.0		++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++++++++	28,920	5,887.7
211.0		++++++++++++++++++++	++++++++++++++++	++++++++++++	++++++++++++++++++++++++++++	32,416	5,882.7
216.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++	++++++++++++++++++++	25,742	5,877.7
221.0		++++++++++++++	+++	.	.	0	5,872.7
224.3		++++++++++	.	.	.	0	5,869.4
229.0		++++++++	.	.	.	0	5,864.7
234.4		++++++++	.	.	.	0	5,859.3
239.8		+++++++	.	.	.	0	5,853.9
245.2		++++++++++	.	.	.	0	5,848.5
250.7		+++++++++	.	.	.	0	5,843.0
256.0		++++++++++	.	.	.	0	5,837.7
261.0		++++++++++	.	.	.	0	5,832.7
266.0		+++++++++	.	.	.	0	5,827.7
271.0		++++++++	.	.	.	0	5,822.7
276.0		+++++++	.	.	.	0	5,817.7
281.0		+++++++	.	.	.	0	5,812.7
286.0		++++++++++++++++++	.	.	.	0	5,807.7
291.0		++++++++++++++++++	++++++++	.	+++++	5,085	5,802.7
292.4		++++++++++++++++++	++++++++++++++++++	.	++++	4,767	5,801.3
296.9		++++++++++++++++++	+++++++++	++++++	++++++++++++	13,983	5,796.8
300.0		.	.	.	.	0	5,793.7

DEPTH	ROCK TYPE	RECOVERY 0% 100%	90-70 NUMBER OF FRACTURES 0 10 20	70-50 0 10 20	50-30 0 10 20	30-0 0 10 20	ELEV. (feet)
20.0		++++++++	.	.	.	.	6,073.7
25.3		++++++++++++++++	+++++++	.	+.	.	6,068.4
26.8		++++++++++++++++++++	++	.	.	.	6,066.9
32.3		++++++++++++++++++	+++++	+	.	.	6,061.4
36.0		++++++++++++++++++++	.	.	.	.	6,057.7
41.0		++++++++++++++++++++	+++++++	++	.	++	6,052.7
46.0		++++++++++++++++++++	++++++++	++	++	++++	6,047.7
51.0		++++++++++++++++++	+++++++	++++++++	.	++++	6,042.7
56.0		++++++++++++++++++++	++++	.	.	+	6,037.7
61.0		++++++++++++++++++++	++++	.	.	+	6,032.7
66.0		++++++++++++++++	++++++	.	.	+++++	6,027.7
71.0		++++++++++++++++++++	+++++++	.	.	.	6,022.7
76.0		++++++++++++++++++	+++++++	.	.	.	6,017.7
81.0		++++++++++++++++++++	++++++	+++	.	.	6,012.7
86.0		++++++++++++++++++++	+++++++	++	.	.	6,007.7
91.0		++++++++++++++++++++	++++++++	.	++++	.	6,002.7
96.0		++++++++++++++++++	+++++	.	+	++	5,997.7
101.0		++++++++++++++++++++	+++	.	.	.	5,992.7
106.0		++++++++++++++++++++	.	++	.	.	5,987.7
111.0		++++++++++++++++++++	++	+	.	.	5,982.7
116.0		++++++++++++++++++++	++++	+	.	++	5,977.7
121.0		++++++++++++++++++	++++++	++	.	+++	5,972.7
126.0		++++++++++++++++	+++	.	.	.	5,967.7
131.0		++++++++++	++++++++++++++++++++	.	.	++++++++++	5,962.7
136.0		++++++++++++++++++++	+++++++	+++++	.	+++++	5,957.7
141.0		++++++++++++++++++++	++	.	.	.	5,952.7
146.0		++++++++++++++++++++	++++	+	.	.	5,947.7
150.0		++++++++++++++++++++	.	.	.	.	5,943.7
155.1		++++++++++++++++++++	+++	++	.	.	5,938.6
160.3		++++++++++++++++++	++	+++	.	.	5,933.4
165.5		++++++++++++++++++++	++	+	.	.	5,928.2
170.7		++++++++++++++++++	+++++	.	.	+	5,923.0
173.0		++++++++++++++++++++	+++	+	.	.	5,920.7
176.0		++++++++++++++	++	+	.	.	5,917.7
181.0		++++++++++++++++++++	+++++	++	.	.	5,912.7
186.0		++++++++++++++++++++	+	.	+	.	5,907.7
191.0		++++++++++++++++++++	+	.	.	++	5,902.7
196.0		++++++++++++++++++	++	.	.	.	5,897.7
201.0		++++++++++++++++++++	.	.	.	.	5,892.7
206.0		++++++++++++++++++	.	.	.	+	5,887.7
211.0		++++++++++++++++++++	.	.	.	++	5,882.7
216.0		++++++++++++++++++++	++++	.	.	++++	5,877.7
221.0		++++++++++++++	++++	.	.	.	5,872.7
224.3		++++++++++	.	.	.	.	5,869.4
229.0		++++++++	.	.	.	.	5,864.7
234.4		++++++++	.	.	.	.	5,859.3
239.8		+++++++	.	.	.	.	5,853.9
245.2		++++++++++	.	.	.	.	5,848.5
250.7		+++++++++	.	.	.	.	5,843.0
256.0		++++++++++	.	.	.	.	5,837.7
261.0		++++++++++	.	.	.	.	5,832.7
266.0		+++++++++	.	.	.	.	5,827.7
271.0		++++++++	.	.	.	.	5,822.7
276.0		+++++++	.	.	.	.	5,817.7
281.0		+++++++	.	.	.	.	5,812.7
286.0		++++++++++++++++++	.	.	.	.	5,807.7
291.0		++++++++++++++++++	++++	.	+++++	.	5,802.7
292.4		++++++++++++++++++	.	.	.	.	5,801.3
296.9		++++++++++++++++++	++++	+++	.	+	5,796.8
300.0		.	.	.	.	.	5,793.7

25-Jun-86 MINER FLAT DAM SITE: MF-119: FRACTURES and HYDRAULIC CONDUCTIVITY

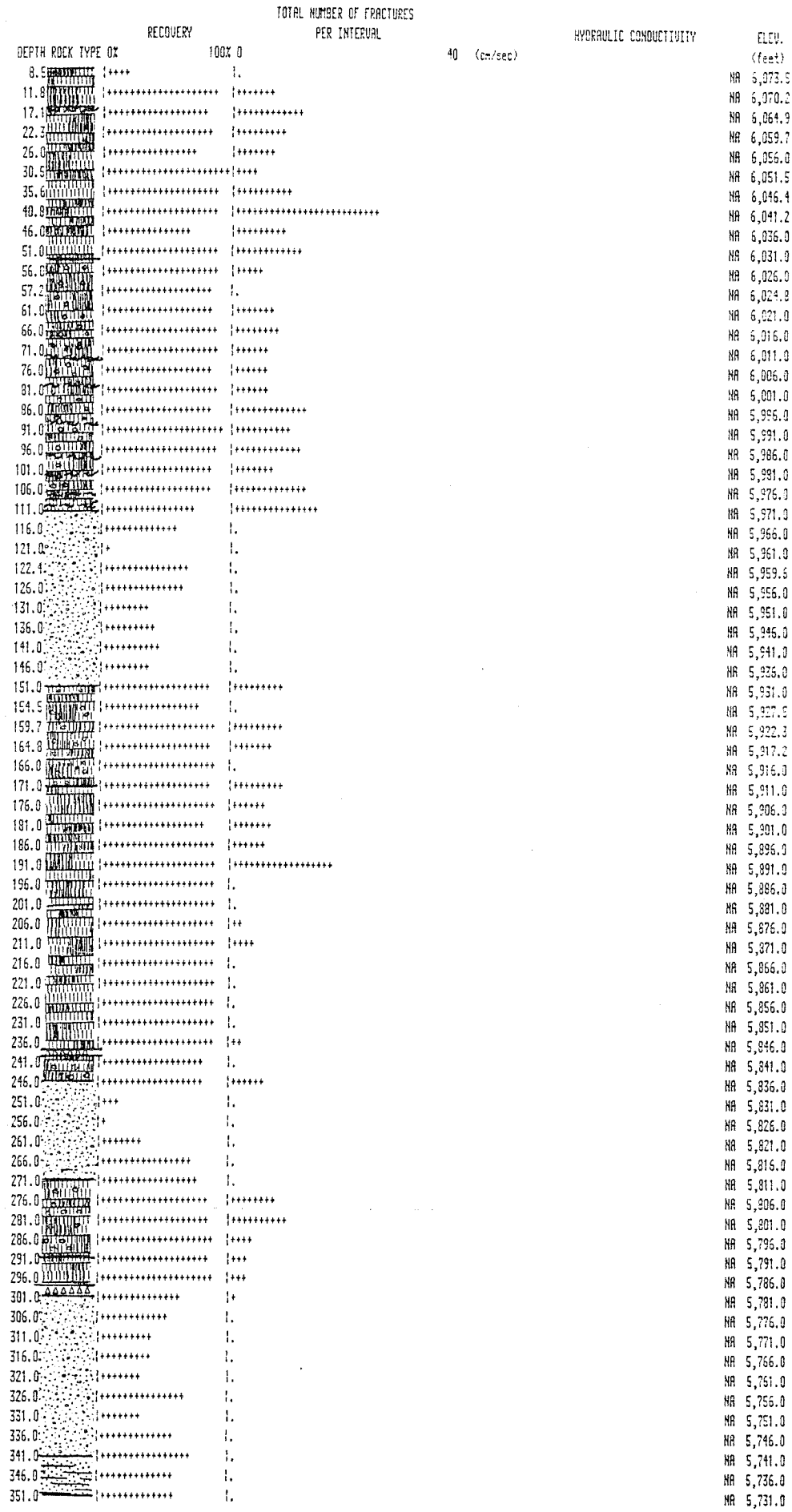


MINER FLAT DAM SITE: MF-120A: ENGINEERING GEOLOGIC LOG



DEPTH	ROCK TYPE	RECOVERY 0% 100%	ROD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
8.5						0	6,073.5
11.8						22,246	6,070.2
17.1						21,293	6,064.9
22.3						19,068	6,059.7
26.0						22,246	6,056.0
30.5						19,068	6,051.5
35.6						18,274	6,046.4
40.8						34,958	6,041.2
46.0						28,602	6,036.0
51.0						23,835	6,031.0
56.0						23,041	6,026.0
57.2						20,657	6,024.8
61.0						19,386	6,021.0
66.0						24,312	6,016.0
71.0						25,742	6,011.0
76.0						20,180	6,006.0
81.0						15,572	6,001.0
86.0						7,945	5,996.0
91.0						25,742	5,991.0
96.0						15,890	5,986.0
101.0						25,106	5,981.0
106.0						14,301	5,976.0
111.0						18,750	5,971.0
116.0						0	5,966.0
121.0						0	5,961.0
122.4						0	5,959.6
126.0						0	5,956.0
131.0						0	5,951.0
136.0						0	5,946.0
141.0						0	5,941.0
146.0						0	5,936.0
151.0						36,547	5,931.0
154.5						36,753	5,927.5
159.7						31,462	5,922.3
164.8						31,462	5,917.2
166.0						28,602	5,916.0
171.0						32,575	5,911.0
176.0						38,136	5,906.0
181.0						34,958	5,901.0
186.0						31,780	5,895.0
191.0						30,191	5,891.0
196.0						31,462	5,886.0
201.0						33,687	5,881.0
206.0						34,164	5,876.0
211.0						31,780	5,871.0
216.0						32,575	5,866.0
221.0						31,780	5,861.0
226.0						31,780	5,856.0
231.0						31,462	5,851.0
236.0						37,024	5,846.0
241.0						0	5,841.0
246.0						4,767	5,836.0
251.0						0	5,831.0
256.0						0	5,826.0
261.0						0	5,821.0
266.0						0	5,816.0
271.0						0	5,811.0
276.0						32,734	5,806.0
281.0						36,547	5,801.0
286.0						39,407	5,796.0
291.0						37,342	5,791.0
296.0						38,136	5,786.0
301.0						35,753	5,781.0
306.0						0	5,776.0
311.0						0	5,771.0
316.0						0	5,766.0
321.0						0	5,761.0
326.0						0	5,756.0
331.0						0	5,751.0
336.0						0	5,746.0
341.0						0	5,741.0
346.0						0	5,736.0
351.0						0	5,731.0

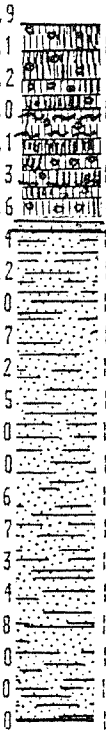
DEPTH	ROCK TYPE	RECOVERY 0% 100%	NUMBER OF FRACTURES 90-70 0 10 20	70-50 0 10 20	50-30 0 10 20	30-0 0 10 20	ELEV. (feet)
8.5		++++	.	.	.	.	6,073.5
11.8		++++++++++++++++++++	++++	.	++	+	6,070.2
17.1		++++++++++++++++++	+++++	+++	+++	+	6,064.9
22.3		++++++++++++++++++	++++	+++	+	+	6,059.7
26.0		++++++++++++++++	+	++++	.	++	6,056.0
30.5		++++++++++++++++++++++	+	+++	.	.	6,051.5
35.6		++++++++++++++++++++	++	+++	.	+++++	6,046.4
40.8		++++++++++++++++++++	+++++++++	++++++	+++++	++++++	6,041.2
46.0		++++++++++++++++	++++++	+++	.	.	6,036.0
51.0		++++++++++++++++++++	+++++++	.	+++	++	6,031.0
56.0		++++++++++++++++++++	+++	++	.	.	6,026.0
57.2		++++++++++++++++++	.	.	.	.	6,024.8
61.0		++++++++++++++++++	+++++++	.	.	.	6,021.0
66.0		++++++++++++++++++++	++++++++	.	.	.	6,016.0
71.0		++++++++++++++++++++	++++++	.	.	.	6,011.0
76.0		++++++++++++++++++	++++++	.	.	.	6,006.0
81.0		++++++++++++++++++++	+++	+++	.	.	6,001.0
86.0		++++++++++++++++++	++++++++	.	+	++++	5,996.0
91.0		++++++++++++++++++++	++++	++	++	++	5,991.0
96.0		++++++++++++++++++++	++++++++	++++	.	.	5,986.0
101.0		++++++++++++++++++	++++	+	.	++	5,981.0
106.0		++++++++++++++++++	++++++	++++	.	+++	5,976.0
111.0		++++++++++++++++	++++++++++	.	.	+++++	5,971.0
116.0		++++++++++++	.	.	.	.	5,966.0
121.0		+	.	.	.	.	5,961.0
122.4		++++++++++++++++	.	.	.	.	5,959.6
126.0		++++++++++++++	.	.	.	.	5,956.0
131.0		++++++++	.	.	.	.	5,951.0
136.0		+++++++++	.	.	.	.	5,946.0
141.0		++++++++++	.	.	.	.	5,941.0
146.0		++++++++	.	.	.	.	5,936.0
151.0		++++++++++++++++++	.	+++++	.	+++	5,931.0
154.5		++++++++++++++++	.	.	.	.	5,927.5
159.7		++++++++++++++++++++	+++++	+++	.	+	5,922.3
164.3		++++++++++++++++++	+++++++	.	.	.	5,917.7
166.0		++++++++++++++++++++	.	.	.	.	5,916.0
171.0		++++++++++++++++++	+++++	.	+	+++	5,911.0
176.0		++++++++++++++++++++	+++++	.	.	+	5,906.0
181.0		++++++++++++++++++	++	++++	.	+	5,901.0
186.0		++++++++++++++++++++	++++	.	++	.	5,896.0
191.0		++++++++++++++++++++	++++++++++++++	.	.	++++	5,891.0
196.0		++++++++++++++++++++	.	.	.	.	5,886.0
201.0		++++++++++++++++++++	.	.	.	.	5,881.0
206.0		++++++++++++++++++++	++	.	.	.	5,876.0
211.0		++++++++++++++++++++	++	++	.	.	5,871.0
216.0		++++++++++++++++++++	.	.	.	.	5,866.0
221.0		++++++++++++++++++++	.	.	.	.	5,861.0
226.0		++++++++++++++++++++	.	.	.	.	5,956.0
231.0		++++++++++++++++++++	.	.	.	.	5,851.0
236.0		++++++++++++++++++++	.	.	+	+	5,846.0
241.0		++++++++++++++++++	.	.	.	.	5,841.0
246.0		++++++++++++++++++	+++	++	.	+	5,836.0
251.0		+++	.	.	.	.	5,831.0
256.0		+	.	.	.	.	5,826.0
261.0		+++++++	.	.	.	.	5,821.0
266.0		++++++++++++++++	.	.	.	.	5,816.0
271.0		++++++++++++++++	.	.	.	.	5,911.0
276.0		++++++++++++++++++++	++++	++++	.	.	5,806.0
281.0		++++++++++++++++++++	+++++	.	+++	++	5,801.0
286.0		++++++++++++++++++++	+	++	.	+	5,796.0
291.0		++++++++++++++++++++	++	+	.	.	5,791.0
296.0		++++++++++++++++++++	+	+	.	+	5,786.0
301.0		++++++++++++++	+	.	.	.	5,781.0
306.0		++++++++++++	.	.	.	.	5,776.0
311.0		+++++++++	.	.	.	.	5,771.0
316.0		+++++++++	.	.	.	.	5,766.0
321.0		+++++++	.	.	.	.	5,761.0
326.0		++++++++++++++++	.	.	.	.	5,756.0
331.0		+++++++	.	.	.	.	5,751.0
336.0		++++++++++++++	.	.	.	.	5,746.0
341.0		++++++++++++++++++	.	.	.	.	5,741.0
346.0		++++++++++++++	.	.	.	.	5,736.0
351.0		++++++++++++++	.	.	.	.	5,731.0


TOTAL NUMBER OF FRACTURES
PER INTERVAL
RECOVERY
HYDRAULIC CONDUCTIVITY
DEPTH ROCK TYPE 0% 100% 0 40 (cm/sec) ELEV. (feet)
8.5 NA 6,073.5
11.8 NA 6,070.2
17.1 NA 6,064.9
22.3 NA 6,059.7
26.0 NA 6,056.0
30.5 NA 6,051.5
35.6 NA 6,046.4
40.8 NA 6,041.2
46.0 NA 6,036.0
51.0 NA 6,031.0
56.0 NA 6,026.0
57.2 NA 6,024.8
61.0 NA 6,021.0
66.0 NA 6,016.0
71.0 NA 6,011.0
76.0 NA 6,006.0
81.0 NA 6,001.0
86.0 NA 5,996.0
91.0 NA 5,991.0
96.0 NA 5,986.0
101.0 NA 5,981.0
106.0 NA 5,976.0
111.0 NA 5,971.0
116.0 NA 5,966.0
121.0 NA 5,961.0
122.4 NA 5,959.6
126.0 NA 5,956.0
131.0 NA 5,951.0
136.0 NA 5,946.0
141.0 NA 5,941.0
146.0 NA 5,936.0
151.0 NA 5,931.0
154.5 NA 5,927.5
159.7 NA 5,922.3
164.8 NA 5,917.2
166.0 NA 5,916.0
171.0 NA 5,911.0
176.0 NA 5,906.0
181.0 NA 5,901.0
186.0 NA 5,896.0
191.0 NA 5,891.0
196.0 NA 5,886.0
201.0 NA 5,881.0
206.0 NA 5,876.0
211.0 NA 5,871.0
216.0 NA 5,866.0
221.0 NA 5,861.0
226.0 NA 5,856.0
231.0 NA 5,851.0
236.0 NA 5,846.0
241.0 NA 5,841.0
246.0 NA 5,836.0
251.0 NA 5,831.0
256.0 NA 5,826.0
261.0 NA 5,821.0
266.0 NA 5,816.0
271.0 NA 5,811.0
276.0 NA 5,806.0
281.0 NA 5,801.0
286.0 NA 5,796.0
291.0 NA 5,791.0
296.0 NA 5,786.0
301.0 NA 5,781.0
306.0 NA 5,776.0
311.0 NA 5,771.0
316.0 NA 5,766.0
321.0 NA 5,761.0
326.0 NA 5,756.0
331.0 NA 5,751.0
336.0 NA 5,746.0
341.0 NA 5,741.0
346.0 NA 5,736.0
351.0 NA 5,731.0

MINER FLAT DAM SITE: MF-121: ENGINEERING GEOLOGIC LOG

DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
11.5	Qc	+	.	.	.	0	6,078.3
15.5		+	.	.	.	0	6,074.3
17.9		+	.	.	.	0	6,071.9
23.1		+	+	.	+	18,274	6,066.7
28.2		+	+	+	+	21,452	6,061.6
30.0		+	+	+	+	15,096	6,059.8
40.1		+	+	+	+	25,424	6,049.7
50.3		+	+	+	+	26,695	6,039.5
60.6		+	+	+	+	22,246	6,029.2
70.4		+	+	+	+	19,863	6,019.4
77.2		+	+	.	+	1,907	6,012.6
81.0		+	+	+	+	1,192	6,008.8
89.7		+	+	.	+	1,827	6,000.1
100.2		+	+	+	.	477	5,989.6
106.5		+	+	+	+	1,827	5,983.3
111.0		+	+	.	+	2,542	5,978.8
121.0		+	+	+	+	1,986	5,968.8
128.6		+	+	.	+	1,192	5,961.2
137.7		+	+	+	.	477	5,952.1
147.3		+	+	+	+	1,271	5,942.5
155.4		+	+	+	.	556	5,934.4
164.8		+	+	+	+	1,112	5,925.0
175.0		+	+	+	+	1,827	5,914.8
185.0		+	+	+	+	1,033	5,904.8
191.0		+	+	+	+	1,748	5,898.8

MINER FLAT DAM SITE: MF-121: FRACTURE LOG

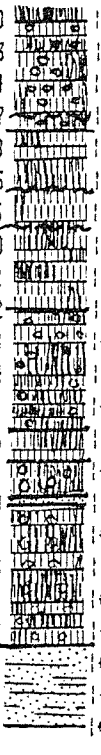


DEPTH	ROCK TYPE	RECOVERY 0% 100%	NUMBER OF FRACTURES 90-70 0 10 20	70-50 0 10 20	50-30 0 10 20	30-0 0 10 20	ELEV. (feet)
11.5		+++++++++	.	.	.	.	6,078.3
15.5	Qc	++++++++++++++++	.	.	.	.	6,074.3
17.9		+++++++++++++	.	.	.	.	6,071.9
23.1		++++++++++++	.	++	+	.	6,066.7
28.2		++++++++++++++++++	+++++++	.	.	+++++	6,061.6
30.0		++++++++++++++++++++++	++	.	.	.	6,059.8
40.1		++++++++++++++++++++	++++	.	.	.	6,049.7
50.3		++++++++++++++++++++	.	.	++++++++	.	6,039.5
60.6		++++++++++++++++++++	.	++++++	+++++	.	6,029.2
70.4		++++++++++++++++	++++++++++	.	.	+++++	6,019.4
77.2		++++++++++++++++	++++++	.	++++	+++	6,012.6
81.0		++++++++++++++++++++	+++++	.	.	++	6,008.8
89.7		++++++++++++++++++	++++++++++++++++	++	.	++	6,000.1
100.2		++++++++++++++++++	+++++	.	++++++++++	++	5,989.6
106.5		++++++++++++++++++	+	.	.	+	5,983.3
111.0		+++++++++++++++	++++++++	.	.	+	5,978.8
121.0		++++++++++++++++	++++++++++++++	.	+++	.	5,968.8
128.6		++++++++++	++++++++++++++	.	.	++++	5,961.2
137.7		+++++++++++++++	++++	.	+++++	++	5,952.1
147.3		++++++++++++++++++	+++++++	+++++	.	.	5,942.5
155.4		++++++++++++++++++	++++++++++++	+++	.	++++++++	5,934.4
164.8		++++++++++++++++++	+++++++++	.	.	++++	5,925.0
175.0		++++++++++++++++++	++++++++++	+++	.	+++	5,914.8
185.0		+++++++	++++++++++	.	.	+++	5,904.8
191.0		++++++++++++++++++	+++++++++	.	.	++	5,898.8

MINER FLAT DAM SITE: MF-121: FRACTURES and HYDRAULIC CONDUCTIVITY

DEPTH	ROCK TYPE	RECOVERY 0%	RECOVERY 100%	TOTAL NUMBER OF FRACTURES PER INTERVAL 0	40	HYDRAULIC CONDUCTIVITY (cm/sec)	ELEV. (feet)
11.5		++++++++++	.			NA	6,078.3
15.5	Qc	++++++++++++++++	.			NA	6,074.3
17.9		++++++++++++++	.			NA	6,071.9
23.1		++++++++++++	+++			NA	6,066.7
28.2		++++++++++++++++++	++++++++++++			NA	6,061.6
30.0		++++++++++++++++++++++	++			NA	6,059.8
40.1		++++++++++++++++++++	++++			NA	6,049.7
50.3		++++++++++++++++++++	++++++++			NA	6,039.5
60.6		++++++++++++++++++++	++++++++++++			NA	6,029.2
70.4		++++++++++++++++	++++++++++++++++		3.29E-03	++	6,019.4
77.2		++++++++++++++++	++++++++++++++		7.02E-04	++++++++++++++++++++++++++++++	6,012.6
81.0		++++++++++++++++++++	+++++++		7.02E-04	++++++++++++++++++++++++++++++	6,008.8
89.7		++++++++++++++++++	++++++++++++++++++++		1.92E-04	++++++++++++++++++++++++++	6,000.1
100.2		++++++++++++++++++	++++++++++++++++++		1.34E-03	++++++++++++++++++++++++++++++++++	5,989.6
106.5		++++++++++++++++++	++		1.50E-03	++++++++++++++++++++++++++++++++++	5,983.3
111.0		++++++++++++++	+++++++++		1.50E-03	++++++++++++++++++++++++++++++++++	5,978.8
121.0		++++++++++++++++	++++++++++++++++		1.42E-03	++++++++++++++++++++++++++++++++++	5,968.8
128.6		++++++++++	++++++++++++++++		1.42E-03	++++++++++++++++++++++++++++++++++	5,961.2
137.7		++++++++++++++++	++++++++++			NA	5,952.1
147.3		++++++++++++++++++	++++++++++++			NA	5,942.5
155.4		++++++++++++++++++	++++++++++++++++++++++++		2.19E-03	++++++++++++++++++++++++++++++++++	5,934.4
164.8		++++++++++++++++++	++++++++++++++		2.19E-03	++++++++++++++++++++++++++++++++++	5,925.0
175.0		++++++++++++++++++	++++++++++++++++			NA	5,914.8
185.0		+++++++	++++++++++++++			NA	5,904.8
191.0		++++++++++++++++++	++++++++++			NA	5,898.8

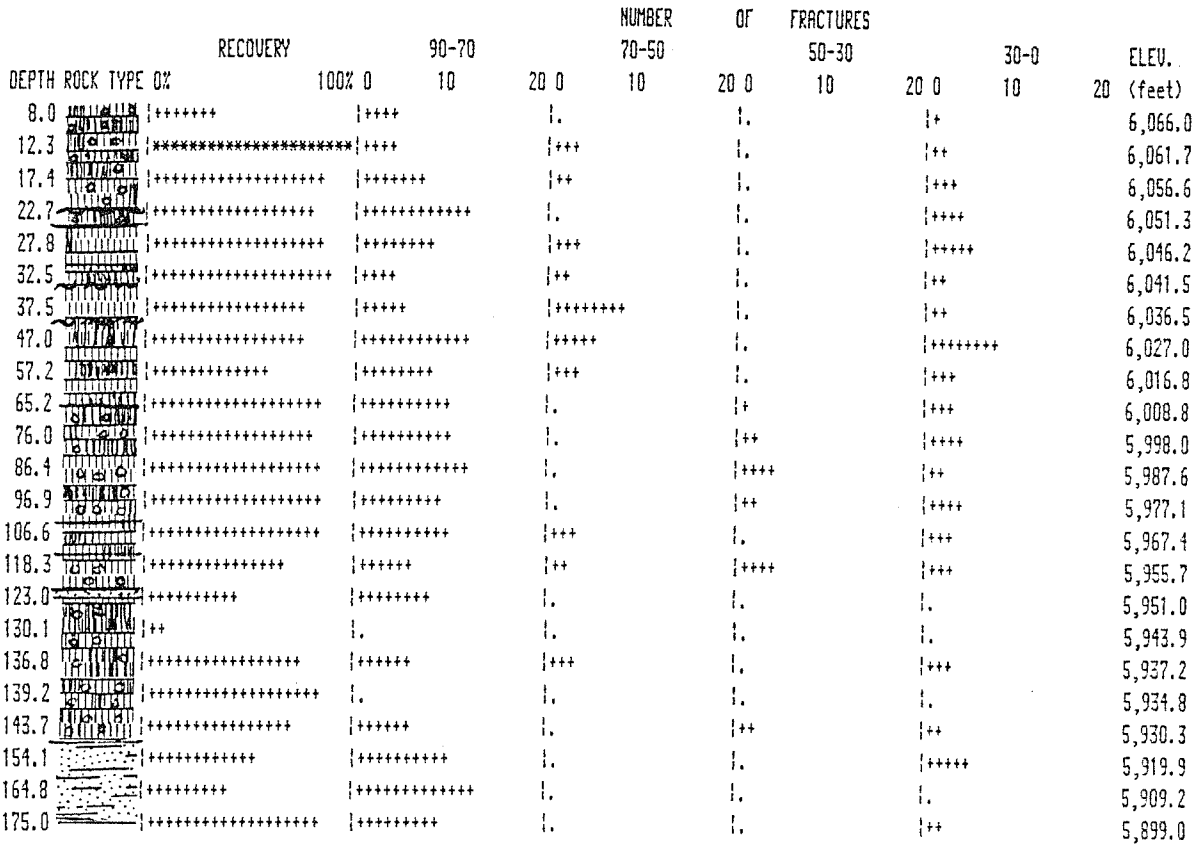
MINER FLAT DAM SITE: MF-122: ENGINEERING GEOLOGIC LOG



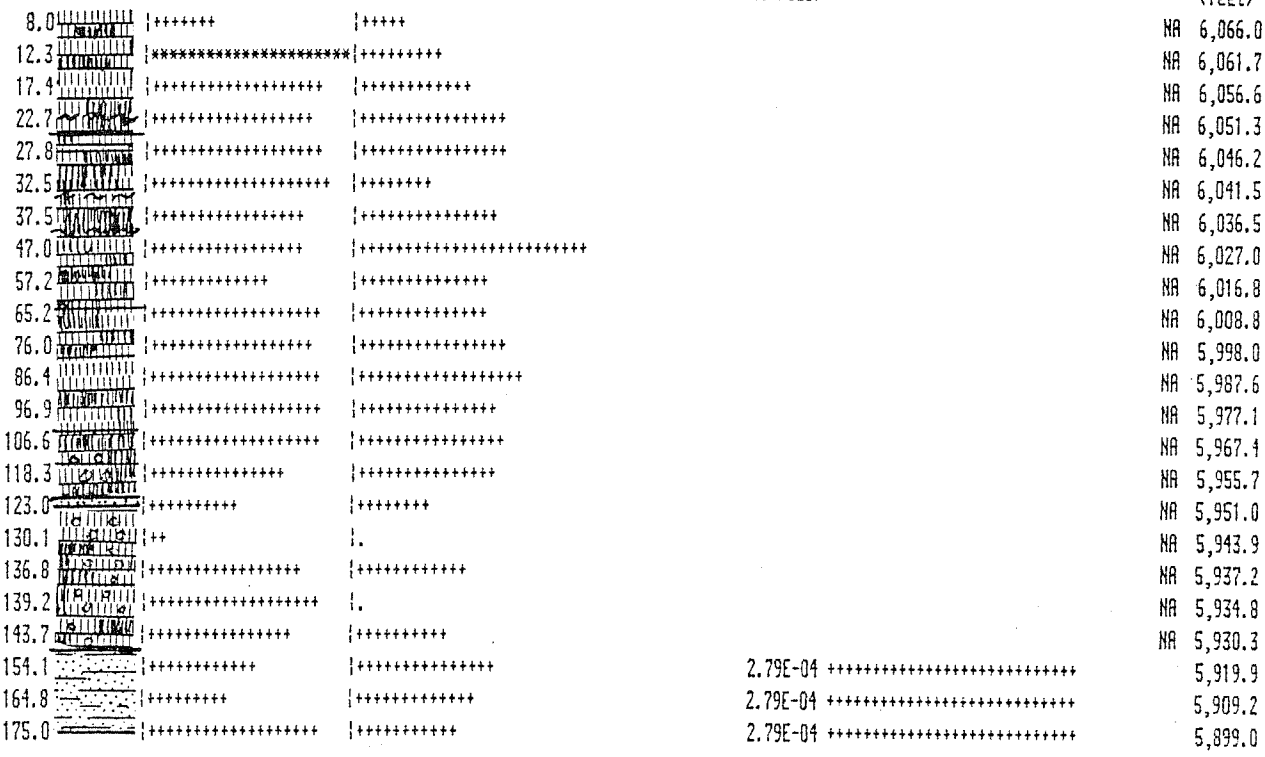
DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
8.0		++++++++	++	.	++++++++++++++++	17,479	6,066.0
12.3		**********	++++++++++++++++++	++++++	++++++++++++++++++++	19,863	6,061.7
17.4		++++++++++++++++++++	++++++	+++++	++++++++++++++++++	18,274	6,056.6
22.7		++++++++++++++++++	+++++	.	++++++++++++++++++++	20,657	6,051.3
27.8		++++++++++++++++++++	+++++++	.	++++++++++++++++++++++++++++	28,602	6,046.2
32.5		++++++++++++++++++++++	++++++++++++++++	++++++++++	++++++++++++++++++++++++++++	28,920	6,041.5
37.5		++++++++++++++++++	+++++++++	.	++++++++++++++++++	18,909	6,036.5
47.0		++++++++++++++++++	+++++++++	++++	++++++++++++++++++++	19,863	6,027.0
57.2		++++++++++++++	++++++++	+++++	++++++++++	10,805	6,016.8
65.2		++++++++++++++++++++	++++++++++++++++	++++++++++++++	++++++++++++++++	15,890	6,008.8
76.0		++++++++++++++++++	++++++++++++++	+++++	++++++++++++++++++	19,068	5,998.0
86.4		++++++++++++++++++++	++++++++++++	+++++	++++++++++++++++	16,685	5,987.6
96.9		++++++++++++++++++++	++++++++++++++	+++++++	++++++++++++++	14,301	5,977.1
106.6		++++++++++++++++++++	++++++++++++	+++++++	++++++++++++++++++++++	23,041	5,967.4
118.3		++++++++++++++++	+++++++	+++++	++++++++++++++++++++++++++++++++++	34,958	5,955.7
123.0		++++++++++	+	.	+++++++	7,945	5,951.0
130.1		++	.	.	.	0	5,943.9
136.8		++++++++++++++++++	++++++++++	+++	++++++++++++++++++++++++++++++	30,668	5,937.2
139.2		++++++++++++++++++++	.	.	.	0	5,934.8
143.7		++++++++++++++++++	++++++++	.	++++++++	8,263	5,930.3
154.1		++++++++++++++	++++++	++	++	2,384	5,919.9
164.8		+++++++++	+	.	++	2,622	5,909.2
175.0		++++++++++++++++++++	++++++++++++++++++	++++++++++++++	.	0	5,899.0

25-Jun-86

MINER FLAT DAM SITE: MF-122: FRACTURE LOG



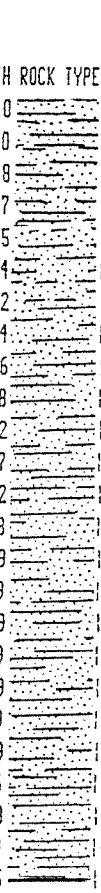
MINER FLAT DAM SITE: MF-122: FRACTURES and HYDRAULIC CONDUCTIVITY



MINER FLAT DAM SITE: MF-123: ENGINEERING GEOLOGIC LOG

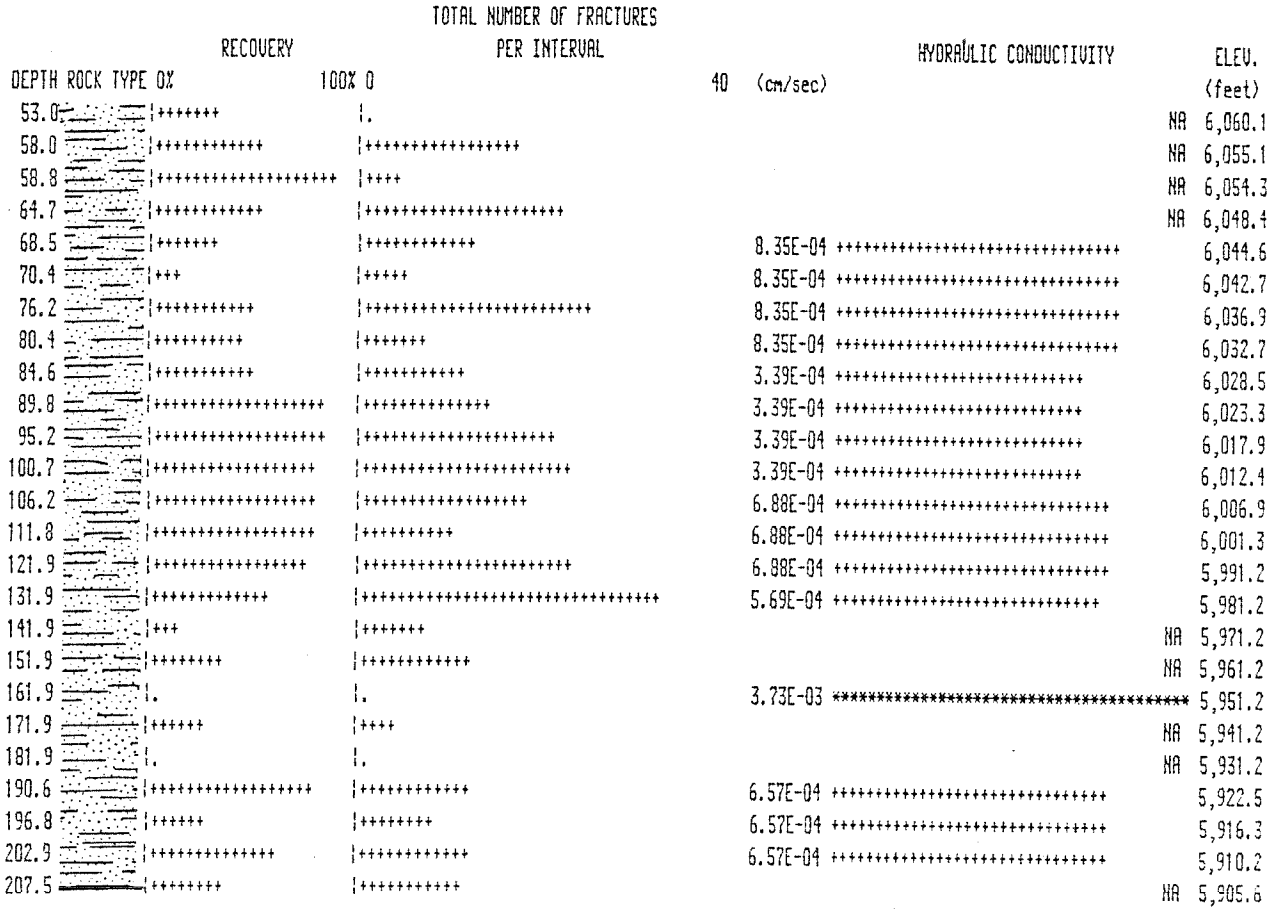
DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi	ELEV. (feet)
53.0					0	6,060.1
58.0					7,548	6,055.1
58.8					13,030	6,054.3
64.7					4,449	6,048.4
68.5					7,627	6,044.6
70.4					5,959	6,042.7
76.2					7,309	6,036.9
80.4					10,805	6,032.7
84.6					7,945	6,028.5
89.8					9,216	6,023.3
95.2					2,463	6,017.9
100.7					3,099	6,012.4
106.2					1,827	6,006.9
111.8					1,986	6,001.3
121.9					1,351	5,991.2
131.9					715	5,981.2
141.9					795	5,971.2
151.9					1,351	5,961.2
161.9					0	5,951.2
171.9					0	5,941.2
181.9					0	5,931.2
190.6					0	5,922.5
196.8					0	5,916.3
202.9					0	5,910.2
207.5					0	5,905.6

MINER FLAT DAM SITE: MF-123: FRACTURE LOG



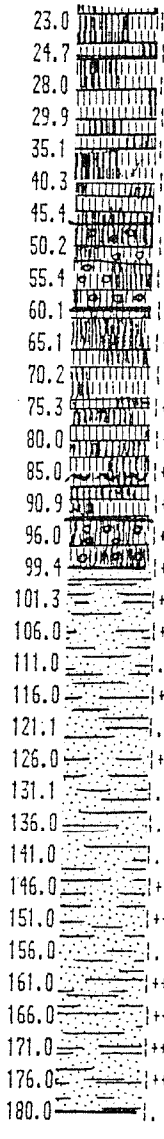
DEPTH	ROCK TYPE	RECOVERY 0%	100%	NUMBER OF FRACTURES 90-70 0	10	20	70-50 10	50-30 20 0	10	30-0 20 0	10	20	ELEV. (feet)
53.0		+++++++		.		.		.		.			6,060.1
58.0		++++++++++++		+++++		.		++++		++++++++			6,055.1
58.8		++++++++++++++++++++		++++		.		.		.			6,054.3
64.7		++++++++++++		+++++++++		.		++++++++		+++++			6,048.4
68.5		+++++++		++++		.		+++++		+++			6,044.6
70.4		+++		+++		.		.		++			6,042.7
76.2		++++++++++++		++++++++++++++++++		.		+++		+++			6,036.9
80.4		++++++++++		+++++		.		.		++			6,032.7
84.6		++++++++++++		+++++		.		.		++++++			6,028.5
89.8		++++++++++++++++++		++++++++		.		+++		+++			6,023.3
95.2		++++++++++++++++++		++++++++++		.		++++++		+++++			6,017.9
100.7		+++++++++++++++++		++++++		.		++++++		++++++++++			6,012.4
106.2		+++++++++++++++++		+++++++		.		+++++		++++++			6,006.9
111.8		+++++++++++++++++		+++++		.		++		+++			6,001.3
121.9		+++++++++++++++++		++++++++++		.		+++++++		++++++			5,991.2
131.9		++++++++++++		**********		.		+++		+++++			5,981.2
141.9		+++		++++		.		.		+++			5,971.2
151.9		++++++++		+++++++		.		+++		++			5,961.2
161.9		.		.		.		.		.			5,951.2
171.9		++++++		++		.		.		++			5,941.2
181.9		.		.		.		.		.			5,931.2
190.6		++++++++++++++++++		+++++		.		.		+++++++			5,922.5
196.8		++++++		+++++		.		.		+++			5,916.3
202.9		++++++++++++		+++++++++		.		.		+++			5,910.2
207.5		++++++++		++++++++++		.		.		.			5,905.6

MINER FLAT DAM SITE: MF-123: FRACTURES and HYDRAULIC CONDUCTIVITY



25-Jun-86

MINER FLAT DAM SITE: MF-124: ENGINEERING GEOLOGIC LOG



DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 100%	40,000 psi	ELEV. (feet)
23.0		++	.	.	.	0	6,057.6
24.7		++++++++++++++++++++	.	.	++++++++	8,740	6,055.9
28.0		++++++++++++++++	++	.	++++++++++++++	13,983	6,052.6
29.9		++++++++++++++	.	.	++++++++++++++++++	19,386	6,050.7
35.1		++++++++++++++	+++	.	++++++++++++++++++++++++	23,835	6,045.5
40.3		++++++++++++++++++++	+++++++++++++++	+++++++++	++++++	6,356	6,040.3
45.4		++++++++++++++	+++++++++	+++++	++++++++++++++++++++	21,928	6,035.2
50.2		++++++++++++++++++++++	++++++++++++++++++	+++++++++	++++++++++++++	14,301	6,030.4
55.4		++++++++++++++++++	++++++++	+++++	++++++++++++++++++++++++++	25,424	6,025.2
60.1		++++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++	23,517	6,020.5
65.1		++++++++++++++++++++	++++++++++++++++++	++++++++++++++++++	++++++++++++++++++++++	22,882	6,015.5
70.2		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++++	++++++++++++++++++	19,863	6,010.4
75.3		++++++++++++++++++	++++++++++++++++++	++++++++++++++	++++++++++++++++++	19,704	6,005.3
80.0		++++++++++++++++++++	++++++	++++	++++++++++++++++++++	22,564	6,000.6
85.0		++++++++++++++++++	++++++++++++	++++	++++++++++++++++++	18,274	5,995.6
90.9		++++++++++++++++++	++++++++++++	+++++++++	.	0	5,989.7
96.0		++++++++++++++++++++	++++++++++++++++++++	++++++++++++++	++++++++++++++++++++++++	24,232	5,984.6
99.4		++++++++++++++++++++	++++++++++++++	.	++++++++++++++++	15,255	5,981.2
101.3		++++	.	.	++++++++	8,898	5,979.3
106.0		+	.	.	.	0	5,974.6
111.0		.	.	.	.	0	5,969.6
116.0		++++++++++++	+	.	.	0	5,964.6
121.1		.	.	.	.	0	5,959.5
126.0		++++++++++++	.	.	++	2,066	5,954.6
131.1		.	.	.	.	0	5,949.5
136.0		.	.	.	.	0	5,944.6
141.0		.	.	.	.	0	5,939.6
146.0		++++++++++++++++++	++++++++++++++	++++++++++++++	+	1,271	5,934.6
151.0		++++++++++++++++	++++++	++++	+	1,271	5,929.6
156.0		.	.	.	.	0	5,924.6
161.0		++++++++++++++++	+++++++	.	.	953	5,919.6
166.0		++++++++++++++++++++	+++++++	++++	.	0	5,914.6
171.0		++++++++++	.	.	.	0	5,909.6
176.0		++++++++++	+	.	.	0	5,904.6
180.0		.	.	.	.	0	5,900.6

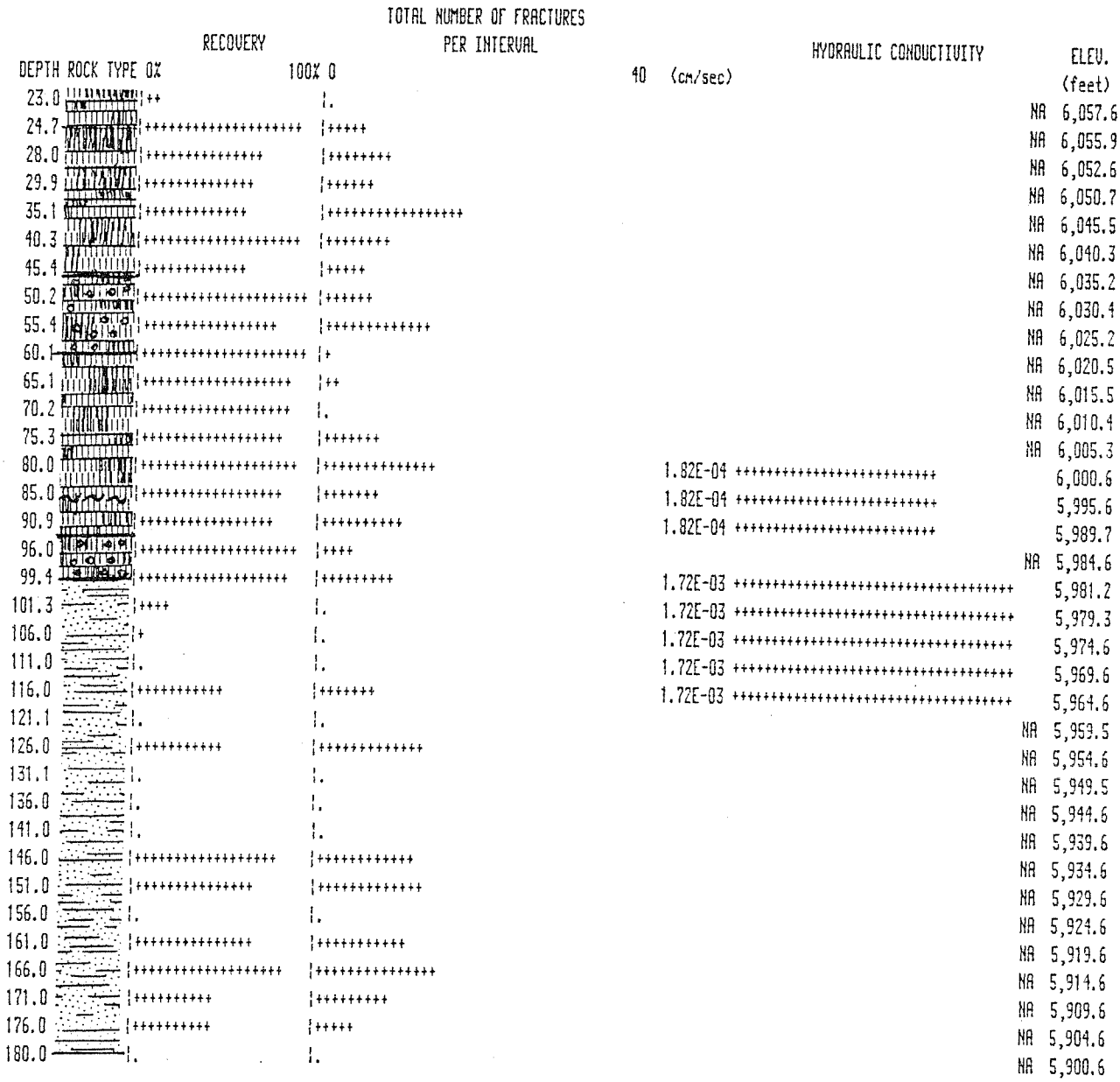
25-Jun-86

MINER FLAT DAM SITE: MF-124: FRACTURE LOG

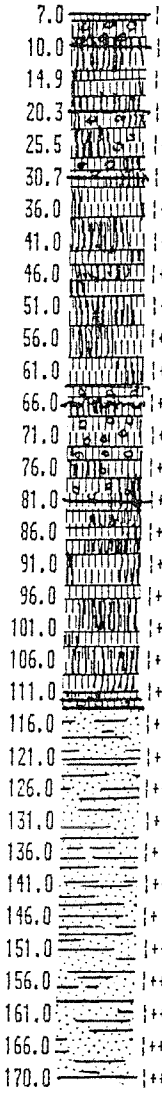
DEPTH	ROCK TYPE	RECOVERY 0% 100%	NUMBER OF FRACTURES 90-70 0 10 20	70-50 0 10 20	50-30 0 10 20	30-0 0 10 20	ELEV. (feet)
23.0		++	.	.	.	.	6,057.6
24.7		++++++++++++++++++++	+++	.	++	.	6,055.9
28.0		++++++++++++++++	+++	.	.	+++++	6,052.6
29.9		++++++++++++++	+++	.	.	+++	6,050.7
35.1		++++++++++++++	++++++++++++	.	.	++++++	6,045.5
40.3		++++++++++++++++++++	++++	.	++	++	6,040.3
45.4		++++++++++++++	+++	.	++	.	6,035.2
50.2		++++++++++++++++++++	++++++	.	.	.	6,030.4
55.4		++++++++++++++++++	+++++	.	++++	++++	6,025.2
60.1		++++++++++++++++++++	.	+	.	.	6,020.5
65.1		++++++++++++++++++++	+	.	.	+	6,015.5
70.2		++++++++++++++++++++	.	.	.	.	6,010.4
75.3		++++++++++++++++++	+++	.	.	++++	6,005.3
80.0		++++++++++++++++++++	+++++	.	++++++	+++	6,000.6
85.0		++++++++++++++++++	++++++	.	.	+	5,995.6
90.9		++++++++++++++++++	++++++	.	.	++++	5,989.7
96.0		++++++++++++++++++++	++	.	.	++	5,984.6
99.4		++++++++++++++++++++	+++++	++	.	++	5,981.2
101.3		++++	.	.	.	.	5,979.3
106.0		+	.	.	.	.	5,974.6
111.0		.	.	.	.	.	5,969.6
116.0		++++++++++	++++	.	+++	.	5,964.6
121.1		.	.	.	.	.	5,959.5
126.0		++++++++++	++++++++	.	+++	++	5,954.6
131.1		.	.	.	.	.	5,949.5
136.0		.	.	.	.	.	5,944.6
141.0		.	.	.	.	.	5,939.6
146.0		++++++++++++++++++	++++++	.	++++++	.	5,934.6
151.0		++++++++++++++++	+++++++	.	+++++	+	5,929.6
156.0		.	.	.	.	.	5,924.6
161.0		++++++++++++++++	+++++++	.	++	++	5,919.6
166.0		++++++++++++++++++++	+++++	.	++++++	++++	5,914.6
171.0		++++++++++	++++	++	+++	.	5,909.6
176.0		++++++++++	++	.	+++	.	5,904.6
180.0		.	.	.	.	.	5,900.6

25-Jun-86

MINER FLAT DAM SITE: MF-124: FRACTURES and HYDRAULIC CONDUCTIVITY



MINER FLAT DAM SITE: MF-125A: ENGINEERING GEOLOGIC LOG



DEPTH	ROCK TYPE	RECOVERY 0% 100%	RQD 0% 100%	SCR 0% 100%	STRENGTH 0 40,000 psi		ELEV. (feet)
7.0						0	6,058.0
10.0						9,216	6,055.0
14.9						20,022	6,050.1
20.3						21,452	6,044.7
25.5						14,937	6,039.5
30.7						17,479	6,034.3
36.0						22,246	6,029.0
41.0						21,293	6,024.0
46.0						20,975	6,019.0
51.0						26,219	6,014.0
56.0						24,471	6,009.0
61.0						25,742	6,004.0
66.0						25,265	5,999.0
71.0						19,068	5,994.0
76.0						10,329	5,989.0
81.0						12,712	5,984.0
86.0						20,657	5,979.0
91.0						20,339	5,974.0
96.0						19,068	5,969.0
101.0						15,890	5,964.0
106.0						17,479	5,959.0
111.0						18,750	5,954.0
116.0						20,975	5,949.0
121.0						1,827	5,944.0
126.0						0	5,939.0
131.0						0	5,934.0
136.0						0	5,929.0
141.0						1,748	5,924.0
146.0						0	5,919.0
151.0						0	5,914.0
156.0						0	5,909.0
161.0						0	5,904.0
166.0						0	5,899.0
170.0						0	5,895.0

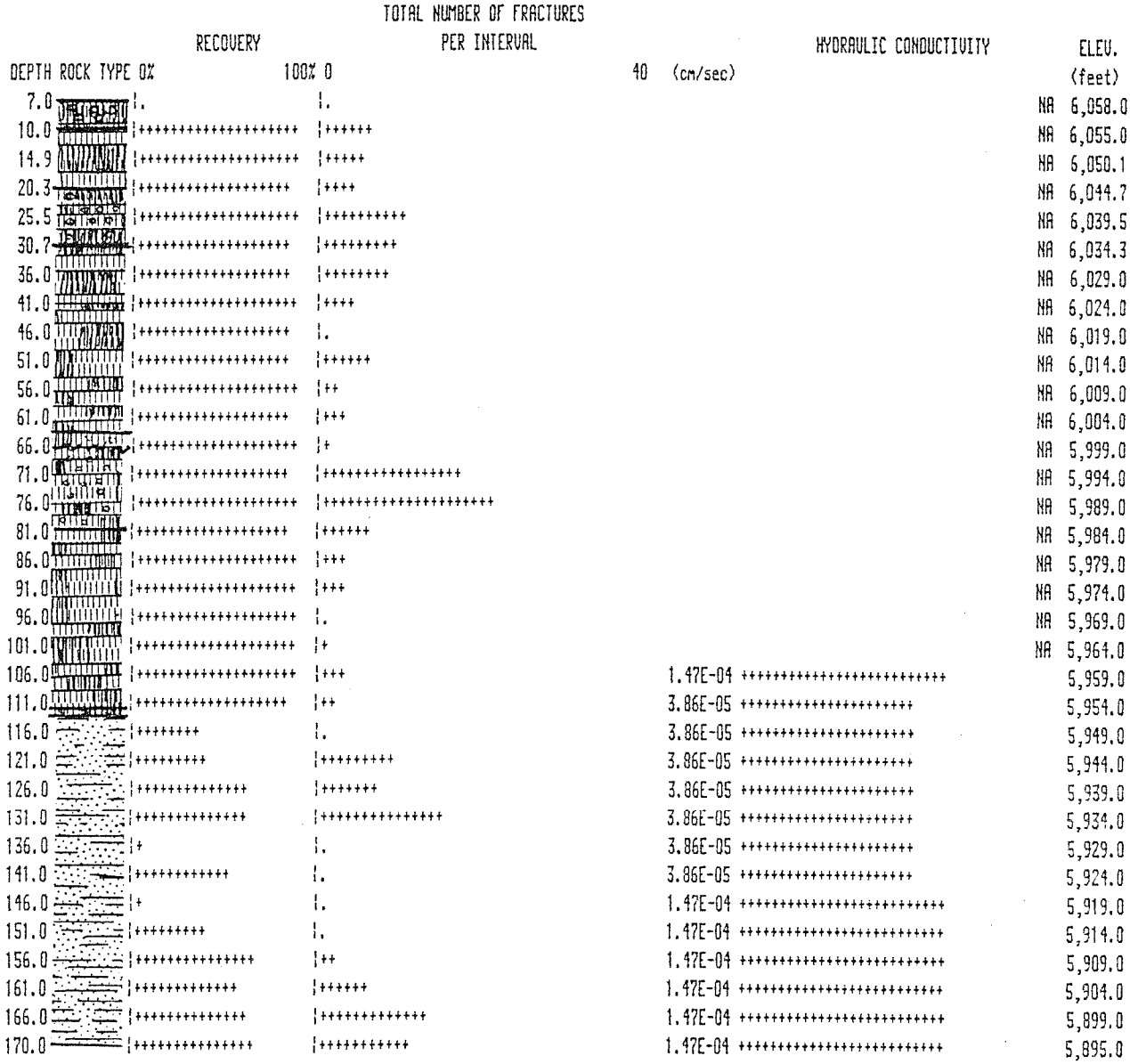
25-Jun-86

MINER FLAT DAM SITE: MF-125A: FRACTURE LOG

DEPTH	RECOVERY 0% 100%	NUMBER OF FRACTURES 90-70 0 10 20	70-50 0 10 20	50-30 0 10 20	30-0 0 10 20	ELEV. (feet)
7.0						6,058.0
10.0	++++++++++++++++++++	++++++				6,055.0
14.9	++++++++++++++++++++	++++	+			6,050.1
20.3	++++++++++++++++++++	++++				6,044.7
25.5	++++++++++++++++++++	+++++		+	++++	6,039.5
30.7	++++++++++++++++++++	+++++			++++	6,034.3
36.0	++++++++++++++++++++	+++++			+++	6,029.0
41.0	++++++++++++++++++++	++	+		+	6,024.0
46.0	++++++++++++++++++++					6,019.0
51.0	++++++++++++++++++++	+++	+		++	6,014.0
56.0	++++++++++++++++++++	+			+	6,009.0
61.0	++++++++++++++++++++	+	++			6,004.0
66.0	++++++++++++++++++++	+				5,999.0
71.0	++++++++++++++++++++	++++++++	+++++		++++	5,994.0
76.0	++++++++++++++++++++	+++++++	++++++++		++++++	5,989.0
81.0	++++++++++++++++++++	++++++				5,984.0
86.0	++++++++++++++++++++	+++				5,979.0
91.0	++++++++++++++++++++	+++				5,974.0
96.0	++++++++++++++++++++					5,969.0
101.0	++++++++++++++++++++			+		5,964.0
106.0	++++++++++++++++++++		++	+		5,959.0
111.0	++++++++++++++++++++	+		+		5,954.0
116.0	++++++++					5,949.0
121.0	+++++++++	+++++++++				5,944.0
126.0	+++++++++++++++				+++++++	5,939.0
131.0	+++++++++++++++	++++++++		++++	+++	5,934.0
136.0	+					5,929.0
141.0	++++++++++++					5,924.0
146.0	+					5,919.0
151.0	+++++++++					5,914.0
156.0	++++++++++++++++			++		5,909.0
161.0	++++++++++++++			++++++		5,904.0
166.0	+++++++++++++++	+++++++	++++++			5,899.0
170.0	++++++++++++++++		+++++	++++++		5,895.0

25-Jun-86

MINER FLAT DAM SITE: MF-125A: FRACTURES and HYDRAULIC CONDUCTIVITY



APPENDIX D

Basic Data, Packer Tests

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-102

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
MF-102	22.0	41.8	4.3		165.3	-77.9	22-Jan-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
20	17.500	17.400	17.333	17.425	17.500	NA	NA	NA	17.43	22.6	2.14E-03	ROCK TYPE:
25	19.300	19.250	19.200	19.200	19.260	NA	NA	NA	19.24	21.6	2.47E-03	BASALT
30	20.700	20.650	20.733	20.625	20.600	20.650	20.613	20.625	20.65	22.6	2.53E-03	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	35.5	NA	
										AVERAGE		
										2.37E-03	(cm/sec)	

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	40.9	51.0	4.3		165.3	-77.9	23-Jan-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
32	0.850	0.950	0.950	0.975	0.933	0.980	NA	NA	0.94	123.0	2.12E-05	ROCK TYPE:
48	3.450	3.500	3.617	3.763	3.880	4.398	4.833	5.475	4.11	156.6	7.27E-05	BASALT
24	4.300	4.300	4.333	4.250	4.240	4.040	4.000	NA	4.21	101.0	1.15E-04	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	49.2	NA	
										AVERAGE		
										5.62E-05	(cm/sec)	

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	59.1	81.0	4.3		165.3	-77.9	23-Jan-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
22	3.500	3.318	3.258	3.288	3.240	3.140	NA	NA	3.29	121.3	7.51E-05	ROCK TYPE:
44	7.200	7.100	7.067	5.300	7.060	7.120	NA	NA	6.81	165.0	1.14E-04	BASALT
66	11.600	11.600	11.400	11.400	11.360	11.320	11.280	11.200	11.40	199.5	1.58E-04	
33	9.000	8.600	8.367	8.250	8.180	8.080	NA	NA	8.41	134.8	1.73E-04	
										AVERAGE		
										1.24E-04	(cm/sec)	

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	81.5	101.0	4.3		165.3	-77.9	24-Jan-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
26	8.900	8.750	8.633	8.537	8.520	8.490	NA	NA	8.64	138.6	1.73E-04	ROCK TYPE:
52	18.200	18.150	18.133	18.100	18.120	18.150	18.160	18.240	18.16	149.7	3.36E-04	BASALT
43	16.700	16.700	16.667	16.625	16.700	16.600	16.600	16.610	16.65	138.8	3.32E-04	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	93.5	NA	
										AVERAGE		
										2.68E-04	(cm/sec)	

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
81.3	121.0	4.3	165.3	-77.9	24-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	11.500	11.650	11.667	11.575	11.260	11.250	NA	NA	11.48	146.3	2.17E-04	ROCK TYPE:
60	20.500	20.400	20.400	20.400	20.400	20.380	20.367	20.380	20.40	161.5	3.50E-04	BASALT
45	18.200	18.200	18.167	18.125	18.140	18.100	18.047	18.000	18.12	143.4	3.50E-04	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	103.2	NA	
											AVERAGE	
											2.98E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
121.2	139.8	4.3	165.3	-77.9	24-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	2.700	2.650	2.667	2.663	2.670	2.670	NA	NA	2.67	199.7	3.70E-05	ROCK TYPE:
40	18.800	18.500	18.333	18.350	18.320	18.100	17.867	17.750	18.25	159.7	3.17E-04	BASALT
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	131.9	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	131.9	NA	
											AVERAGE	
											1.08E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
141.5	160.3	4.3	165.3	-77.9	25-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
90	0.100	0.125	0.125	0.113	0.120	0.100	0.103	0.090	0.11	359.7	8.43E-07	ROCK TYPE:
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	151.8	NA	BASALT
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	151.8	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	151.8	NA	
											AVERAGE	
											8.43E-07	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
140.7	180.7	4.3	165.3	-77.9	25-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	1.250	1.200	1.183	1.169	1.140	1.165	NA	NA	1.18	230.4	1.42E-05	ROCK TYPE:
60	2.500	2.475	2.467	2.450	2.430	2.390	NA	NA	2.45	298.7	2.27E-05	BASALT
90	3.450	3.400	3.433	3.475	3.500	3.530	3.500	3.460	3.47	366.8	2.62E-05	
45	2.600	2.900	2.750	2.688	2.670	2.615	NA	NA	2.70	263.8	2.84E-05	
											AVERAGE	
											2.21E-05	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
181.5	220.4	4.3	165.3	-77.9	25-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	6.500	6.400	6.200	6.150	6.140	6.080	NA	NA	6.25	262.1	6.60E-05	ROCK TYPE:
60	8.400	8.300	8.300	8.275	8.280	8.280	NA	NA	8.31	325.5	7.06E-05	BASALT
90	10.300	10.150	10.133	10.125	10.100	9.980	9.887	9.835	10.06	388.5	7.17E-05	
45	7.400	7.350	7.333	7.400	7.450	7.340	NA	NA	7.38	293.7	6.96E-05	
											AVERAGE	
											6.94E-05	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
211.7	237.7	4.3	165.3	-77.9	26-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	6.000	6.150	6.333	6.450	6.520	6.740	NA	NA	6.37	227.0	7.77E-05	ROCK TYPE:
60	16.000	16.200	16.233	16.250	16.200	16.500	16.687	16.825	16.36	252.3	1.80E-04	COLLUVIUM
80	20.000	20.350	20.333	20.250	20.260	20.270	NA	NA	20.24	271.6	2.06E-04	
45	16.500	16.350	16.267	16.250	16.260	16.200	NA	NA	16.30	218.0	2.07E-04	
											AVERAGE	
											1.56E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
212.8	263.9	4.3	165.3	-77.9	27-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
20	29.000	29.000	29.000	29.000	29.000	NA	NA	NA	29.00	52.6	1.53E-03	ROCK TYPE:
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	165.9	NA	COLLUVIUM
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	165.9	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	165.9	NA	
											AVERAGE	
											1.53E-03	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
260.5	281.0	4.3	165.3	-77.9	27-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
15.7	19.100	19.200	19.300	19.300	19.200	18.900	18.800	18.700	19.06	131.8	4.00E-04	ROCK TYPE:
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	165.9	NA	SANDSTONE
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	165.9	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	165.9	NA	
											AVERAGE	
											4.00E-04	(cm/sec)

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-102

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
291.5	311.0	4.3	165.3	-77.9	28-Jan-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	18.500	18.150	18.100	18.075	18.060	NA	NA	NA	18.18	171.1	2.94E-04	ROCK TYPE:
30	20.400	20.600	20.433	20.300	20.400	26.350	NA	NA	21.41	146.9	4.03E-04	SANDSTONE
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	165.9	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	165.9	NA	
											AVERAGE	
											3.44E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
311.3	331.0	4.3	165.3	-77.9	29-Jan-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	1.300	1.375	1.433	1.450	1.440	1.405	NA	NA	1.40	234.8	1.65E-05	ROCK TYPE:
60	NA	NA	NA	NA	NA	NA	NA	NA	NA	304.5	NA	GYPSUM/
90	2.200	2.188	2.125	2.112	2.090	1.985	1.957	1.940	2.07	372.9	1.54E-05	SANDSTONE
45	1.275	1.275	1.217	1.175	1.160	1.165	NA	NA	1.21	269.6	1.24E-05	
											AVERAGE	
											1.47E-05	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
321.6	375.0	4.3	165.3	-77.9	29-Jan-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	0.800	0.775	0.767	0.756	0.780	0.805	NA	NA	0.78	235.1	9.19E-06	ROCK TYPE:
60	1.350	1.350	1.333	1.325	1.290	1.180	NA	NA	1.30	304.2	1.19E-05	GYPSUM/
90	1.500	1.500	1.483	1.475	1.470	1.445	1.400	1.370	1.46	373.4	1.08E-05	SANDSTONE
45	0.400	0.400	0.435	0.462	0.480	0.550	NA	NA	0.45	269.8	4.66E-06	
											AVERAGE	
											8.61E-06	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
105	17.2	40.0	4.2	158.0	-72.1	08-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
15	0.650	0.650	0.633	0.625	0.620	0.610	NA	NA	0.63	66.0	2.35E-05	ROCK TYPE:
20	1.050	1.050	1.050	1.050	1.050	1.045	NA	NA	1.05	77.4	3.35E-05	BASALT
30	2.400	2.325	2.300	2.300	2.280	2.350	2.230	2.220	2.30	99.6	5.71E-05	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	31.4	NA	
											AVERAGE	
											3.56E-05	(cm/sec)

	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	39.3	59.5	4.2	158.0	-72.1	08-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
15	0.400	0.375	0.383	0.363	0.360	0.345	NA	NA	0.37	85.8	1.05E-05	ROCK TYPE:
30	1.050	0.988	1.033	1.025	1.020	1.025	NA	NA	1.02	120.3	2.09E-05	BASALT
45	2.050	2.050	2.067	2.050	2.060	2.010	2.037	1.975	2.04	154.3	3.26E-05	
22	1.100	1.125	1.133	1.125	1.120	1.085	NA	NA	1.12	101.8	2.71E-05	
											AVERAGE	
											2.10E-05	(cm/sec)

	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	59.5	85.00	5.00	158.0	-72.1	13-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
25	4.200	4.150	4.167	4.175	4.180	4.160	NA	NA	4.17	127.9	8.05E-05	ROCK TYPE:
50	7.500	7.500	7.467	7.450	7.400	7.350	NA	NA	7.43	178.1	1.03E-04	BASALT
75	10.867	10.800	10.767	10.775	10.760	10.750	10.747	10.750	10.78	223.9	1.19E-04	
37	7.700	7.700	7.667	7.650	7.620	7.570	NA	NA	7.64	147.4	1.28E-04	
											AVERAGE	
											1.06E-04	(cm/sec)

	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	90.8	102.80	5.00	158.0	-72.1	13-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	18.000	18.250	17.900	17.750	17.700	17.895	NA	NA	17.90	104.2	4.24E-04	ROCK TYPE:
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	97.1	NA	BASALT
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	97.1	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	97.1	NA	
											AVERAGE	
											4.24E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
159.5	183.5	5.0	160.6	-72.1	16-Dec-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	19.700	19.500	19.567	19.650	19.760	10.66	NA	NA	17.83	175.8	2.51E-04	ROCK TYPE:
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	168.2	NA	BASALT
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	168.2	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	168.2	NA	
										AVERAGE		
										2.51E-04	(cm/sec)	

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
179.5	203.5	5.0	158.0	-72.1	18-Dec-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	6.800	6.650	6.667	6.381	6.760	6.930	NA	NA	6.70	247.4	6.69E-05	ROCK TYPE:
50	12.500	12.500	12.533	12.500	12.560	NA	NA	NA	12.52	271.8	1.14E-04	BASALT
25	9.700	9.750	9.71	9.725	9.700	9.690	9.700	9.715	9.71	226.1	1.06E-04	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	187.2	NA	
										AVERAGE		
										9.32E-05	(cm/sec)	

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
201.5	229.1	5.0	158.0	-72.1	18-Dec-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	6.400	6.350	6.400	6.381	6.420	6.570	NA	NA	6.42	270.8	5.86E-05	ROCK TYPE:
45	12.500	12.350	12.433	12.375	12.420	12.440	NA	NA	12.42	283.3	1.08E-04	BASALT
60	13.917	16.750	16.667	16.700	16.900	16.750	NA	NA	16.28	296.8	1.36E-04	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	209.9	NA	
										AVERAGE		
										9.51E-05	(cm/sec)	

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
221.5	249.4	5.0	158.0	-72.1	18-Dec-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	0.250	0.250	0.225	0.225	0.230	0.230	NA	NA	0.24	298.3	1.95E-06	ROCK TYPE:
60	0.275	0.237	0.217	0.188	0.155	0.080	NA	NA	0.19	367.6	1.29E-06	BASALT
90	0.062	0.063	NA	0.088	0.090	NA	0.037	0.040	0.06	437.0	3.57E-07	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	229.1	NA	
										AVERAGE		
										9.65E-07	(cm/sec)	

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-105

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
222.0	280.0	5.0	158.0	-72.1	18-Dec-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	0.750	0.875	0.950	0.988	1.010	1.055	NA	NA	0.94	313.0	7.41E-06	ROCK TYPE:
60	1.350	1.350	1.333	1.300	1.280	1.255	NA	NA	1.31	382.1	8.48E-06	BASALT
90	1.600	1.600	1.617	1.625	1.630	1.645	1.650	1.653	1.63	451.2	8.91E-06	
45	1.250	1.275	1.283	1.275	1.260	NA	NA	NA	1.27	347.4	9.02E-06	
											AVERAGE	
											8.43E-06	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
278.6	300.0	5.0	158.0	-72.1	07-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	4.600	4.600	4.700	4.800	5.680	7.450	NA	NA	5.31	218.9	5.99E-05	ROCK TYPE:
60	14.600	14.700	14.733	14.925	15.040	15.360	NA	NA	14.89	250.5	1.47E-04	GRAVEL
90	19.800	19.650	19.700	19.675	19.545	17.600	19.320	19.005	19.53	289.5	1.67E-04	
45	0.833	0.727	13.600	13.650	13.600	13.520	NA	13.450	8.48	244.8	8.56E-05	
											AVERAGE	
											1.06E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
305.0	320.0	5.0	158.0	-72.1	07-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
90	0.200	0.200	0.200	0.175	0.180	0.175	0.167	0.168	0.18	363.2	1.25E-06	ROCK TYPE:
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	155.4	NA	SANDSTONE
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	155.4	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	155.4	NA	
											AVERAGE	
											1.25E-06	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
306.5	340.0	5.0	158.0	-72.1	07-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
60	0.525	0.525	0.542	0.550	0.540	0.510	NA	NA	0.53	293.9	4.47E-06	ROCK TYPE:
90	0.675	0.638	0.633	0.625	0.630	0.595	0.575	0.565	0.62	363.2	4.20E-06	SANDSTONE
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	155.4	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	155.4	NA	
											AVERAGE	
											4.33E-06	(cm/sec)

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-105

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
306.4	370.0	5.0	158.0	-72.1	08-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	1.000	0.950	0.850	0.713	0.680	0.530	NA	NA	0.79	224.5	8.66E-06	ROCK TYPE:
60	0.700	0.545	0.658	0.650	0.640	0.610	NA	NA	0.63	293.9	5.33E-06	SANDSTONE
90	0.800	0.775	0.767	0.763	0.770	0.760	0.74	0.728	0.76	363.1	5.19E-06	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	155.4	NA	
											AVERAGE	
											6.21E-06	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
MF-106	16.4	39.2	4.2		148.4	-89.35	19-Nov-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
12	0.50	0.45	0.33	0.35	0.34	0.32	0.31	NA	0.37	59.7	1.54E-05	ROCK TYPE:
20	0.80	0.75	0.67	0.65	0.60	0.54	NA	NA	0.67	78.1	2.11E-05	BASALT
30	2.00	1.50	1.60	1.38	1.30	1.21	1.18	1.17	1.42	100.9	3.47E-05	
15	0.60	0.53	0.50	0.48	0.46	0.44	NA	NA	0.50	66.6	1.86E-05	
											AVERAGE	
											2.14E-05	(cm/sec)

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	37.6	59.4	4.2		148.4	-89.35	19-Nov-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
15	NA	NA	NA	NA	NA	NA	NA	NA	NA	87.3	NA	ROCK TYPE:
30	0.00	0.03	0.03	0.03	0.04	NA	NA	NA	0.02	122.0	4.46E-07	BASALT
50	0.38	0.34	0.33	0.31	0.30	0.29	NA	NA	0.32	168.2	4.77E-06	
25	0.10	0.10	0.10	0.10	0.09	0.08	NA	NA	0.09	110.4	2.09E-06	
											AVERAGE	
											1.64E-06	(cm/sec)

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	58.7	79.4	4.2		148.4	-89.35	20-Nov-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
25	0.00	NA	NA	NA	NA	NA	NA	NA	NA	131.0	NA	ROCK TYPE:
50	0.85	0.83	0.80	NA	0.78	0.77	NA	NA	NA	188.7	NA	BASALT
75	1.75	1.59	1.52	1.49	1.47	1.42	1.42	1.42	1.51	246.0	1.51E-05	
37	1.15	0.90	0.83	0.80	0.78	0.75	NA	NA	0.87	158.5	1.35E-05	
											AVERAGE	
											1.43E-05	(cm/sec)

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	78.7	100.0	4.2		148.4	-89.4	20-Nov-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	0.77	0.73	0.67	0.64	0.61	0.58	NA	NA	0.66	162.7	1.01E-05	ROCK TYPE:
60	4.50	4.00	3.83	3.73	3.66	3.51	NA	NA	3.94	228.9	4.26E-05	BASALT
90	14.80	14.75	15.33	16.13	16.40	17.09	17.20	NA	15.96	251.7	1.57E-04	
45	11.00	11.60	11.50	11.42	11.40	11.25	NA	NA	11.36	171.9	1.63E-04	
											AVERAGE	
											5.76E-05	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
127.3	140.0	4.2	148.4	-89.35	21-Nov-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
33	0.65	0.65	0.58	0.61	0.61	0.58	NA	NA	0.61	214.0	7.09E-06	ROCK TYPE:
66	2.60	3.45	4.37	5.25	6.08	NA	NA	NA	4.35	286.4	3.75E-05	BASALT
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	137.8	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	137.8	NA	
											AVERAGE	
											1.63E-05	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
150.7	160.0	4.2	148.4	-89.35	21-Nov-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	0.17	0.18	0.15	0.14	0.13	0.14	NA	NA	0.15	228.8	1.62E-06	ROCK TYPE:
60	0.35	0.36	0.37	0.36	0.36	0.36	NA	NA	0.36	298.1	2.98E-06	BASALT
90	1.15	1.13	1.10	1.10	1.11	1.11	1.11	1.12	1.11	367.2	7.50E-06	
45	0.85	0.83	0.82	0.81	0.80	NA	NA	NA	0.82	263.3	7.70E-06	
											AVERAGE	
											4.09E-06	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
159.6	180.0	4.2	148.4	-89.35	24-Nov-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	1.00	1.08	1.08	1.10	1.10	1.12	NA	NA	1.08	243.0	1.10E-05	ROCK TYPE:
60	2.80	2.80	2.83	2.85	2.85	2.88	NA	NA	2.84	310.9	2.25E-05	BASALT
90	8.90	9.10	9.30	9.45	9.60	10.03	10.33	10.57	9.66	363.2	6.57E-05	
45	7.30	7.20	7.20	7.20	7.20	NA	NA	NA	7.22	267.4	6.67E-05	
											AVERAGE	
											3.23E-05	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
179.3	200.0	4.2	148.4	-89.35	24-Nov-85

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	2.20	2.10	2.10	2.08	2.06	2.01	NA	NA	2.09	262.2	1.97E-05	ROCK TYPE:
60	5.00	5.00	5.00	5.00	5.00	4.98	NA	NA	5.00	327.3	3.77E-05	BASALT
90	10.00	10.00	10.00	10.00	10.12	10.36	9.81	10.57	10.11	381.3	6.55E-05	
45	7.00	7.00	7.00	7.05	7.04	7.02	NA	NA	7.02	287.8	6.03E-05	
											AVERAGE	
											4.14E-05	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN	DATE
198.9	220.0	4.2	148.4	-89.35	25-Nov-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	NA	NA	NA	NA	NA	NA	NA	NA	NA	282.9	NA	ROCK TYPE:
60	NA	NA	NA	NA	NA	NA	NA	NA	NA	352.2	NA	BASALT
90	0.20	0.18	0.16	0.16	0.16	0.13	0.12	0.11	0.15	421.5	8.91E-07	
45	NA	NA	NA	NA	NA	NA	NA	NA	NA	317.6	NA	
											AVERAGE	
											8.91E-07	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
216.7	260.0	4.2	148.4	-89.35	25-Nov-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	0.55	0.52	0.51	0.50	0.49	0.46	NA	NA	0.51	311.8	4.81E-06	ROCK TYPE:
60	0.60	0.65	0.63	0.63	0.63	0.61	NA	NA	0.62	381.0	4.05E-06	BASALT
90	0.88	0.88	0.87	0.88	0.86	0.85	0.84	0.84	0.86	450.3	4.72E-06	
45	0.50	0.45	0.50	0.49	0.48	0.47	NA	NA	0.48	346.4	3.44E-06	
											AVERAGE	
											4.03E-06	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
254.9	270.9	4.2	148.4	-89.35	02-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	0.95	0.98	1.02	1.04	1.05	1.06	NA	NA	1.01	221.7	1.13E-05	ROCK TYPE:
60	3.70	3.70	3.77	3.78	3.80	3.80	NA	NA	3.76	288.2	3.22E-05	GRAVEL
90	11.00	11.15	11.20	11.25	11.38	12.13	12.38	12.58	11.63	333.7	8.62E-05	
45	8.83	8.64	8.57	8.58	9.00	NA	NA	NA	8.72	241.2	8.94E-05	
											AVERAGE	
											4.09E-05	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
255.1	284.3	4.2	148.4	-89.35	02-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	12.31	12.50	12.53	12.47	12.48	11.60	NA	NA	12.32	191.9	1.59E-04	ROCK TYPE:
60	15.30	15.25	15.30	15.32	15.30	15.34	15.35	15.51	15.34	245.2	1.55E-04	GRAVEL
80	19.50	19.60	19.67	19.75	19.80	20.00	NA	NA	19.72	262.2	1.86E-04	
45	NA	NA	NA	NA	NA	NA	NA	NA	NA	256.5	NA	
											AVERAGE	
											1.66E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN	DATE
306.5	317.0	4.2	148.4	-89.35	04-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	0.15	0.13	0.09	0.07	0.06	0.03	NA	NA	0.09	221.9	9.61E-07	ROCK TYPE:
60	NA	NA	0.01	0.01	0.02	NA	NA	NA	0.01	291.2	1.16E-07	SANDSTONE
90	NA	NA	NA	NA	NA	NA	NA	NA	NA	360.5	NA	
45	NA	NA	NA	NA	NA	NA	NA	NA	NA	256.5	NA	
										AVERAGE		
										3.33E-07	(cm/sec)	

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
305.8	335.8	4.2	148.4	-89.35	04-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	0.08	0.13	0.14	0.15	0.15	0.15	NA	NA	0.13	221.9	1.47E-06	ROCK TYPE:
60	0.40	0.26	0.18	0.20	0.18	0.17	NA	NA	0.23	291.2	1.96E-06	SANDSTONE
90	0.25	0.20	0.18	0.19	0.18	0.18	0.17	0.16	0.19	360.5	1.29E-06	
45	NA	NA	NA	NA	NA	NA	NA	NA	NA	256.5	NA	
										AVERAGE		
										1.55E-06	(cm/sec)	

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
307.5	355.0	4.2	148.4	-89.35	05-Dec-85

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	0.35	0.48	0.37	0.34	0.31	0.25	NA	NA	0.35	221.9	3.88E-06	ROCK TYPE:
60	0.30	0.23	0.22	0.21	0.21	0.19	NA	NA	0.22	291.2	1.91E-06	SANDSTONE
90	0.25	0.30	0.30	0.30	0.31	0.29	0.28	0.27	0.29	360.5	1.97E-06	
45	0.29	0.25	0.23	0.23	0.22	0.20	NA	NA	0.23	256.5	2.26E-06	
										AVERAGE		
										2.40E-06	(cm/sec)	

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-113

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE				
MF-113	16.9	34.6	3.5		-4.5	-89.7	26-Feb-86				
PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)
23	1.450	1.350	1.317	1.288	1.270	1.190	NA	NA	1.31	51.8	7.67E-05 ROCK TYPE:
46	3.050	2.900	2.817	2.788	2.770	2.690	NA	NA	2.84	103.6	8.29E-05 BASALT
70	7.400	7.400	7.433	7.525	7.600	7.980	8.400	8.615	7.79	148.4	1.59E-04
35	4.600	4.600	4.567	4.550	4.560	4.520	NA	NA	4.57	75.5	1.83E-04
										AVERAGE	
										1.17E-04	(cm/sec)

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE				
	45.0	115.0	3.5		-4.5	-89.7	26-Feb-86				
PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)
30	NA	NA	NA	NA	NA	NA	NA	NA	NA	152.8	NA ROCK TYPE:
60	NA	NA	NA	NA	NA	NA	NA	NA	NA	222.1	NA BASALT
90	NA	NA	NA	NA	NA	NA	NA	NA	0.01	291.4	5.20E-08
45	NA	NA	NA	NA	NA	NA	NA	NA	NA	187.4	NA
										AVERAGE	
									LESS THAN	5.20E-08	(cm/sec)

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE				
	107.6	120.0	3.5		-4.5	-89.7	27-Feb-86				
PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)
30	0.350	0.325	0.292	0.288	0.280	0.240	NA	NA	0.30	68.3	1.31E-05 ROCK TYPE:
60	0.700	0.650	0.633	0.625	0.600	0.545	NA	NA	0.63	137.5	1.38E-05 PALEO-
90	0.950	0.925	0.883	0.835	0.810	0.725	0.670	0.628	0.80	206.8	1.18E-05 ALLUVIUM
45	0.150	0.137	0.142	0.138	0.125	0.109	NA	NA	0.13	102.9	3.93E-06
										AVERAGE	
										9.56E-06	(cm/sec)

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE				
	108.2	140.0	3.5		-4.5	-89.7	27-Feb-86				
PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)
30	9.400	9.050	8.900	8.700	8.540	8.080	NA	NA	8.78	52.8	5.03E-04 ROCK TYPE:
60	12.700	12.700	12.533	12.450	12.300	11.740	NA	NA	12.40	107.2	3.50E-04 PALEO-
90	16.600	16.250	16.267	16.250	16.320	16.000	15.720	15.500	16.11	156.2	3.12E-04 ALLUVIUM
45	8.600	8.500	8.400	8.350	8.300	8.140	NA	NA	8.38	88.8	2.86E-04
										AVERAGE	
										3.54E-04	(cm/sec)

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-117

BOREHOLE:	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
MF-117	8.0	28.0	-1.0	35	-26	08-Mar-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
40	0.550	0.550	0.533	0.525	0.520	0.505	NA	NA	0.54	99.2	1.48E-05	ROCK TYPE:
60	2.600	3.425	3.767	4.025	4.168	4.574	NA	NA	3.60	142.8	6.92E-05	BASALT
90	12.300	13.150	13.767	14.325	14.800	16.430	NA	NA	13.67	178.0	2.11E-04	
50	12.800	12.750	12.767	12.750	12.740	12.590	NA	NA	12.76	90.2	3.88E-04	
											AVERAGE	
											9.57E-05	(cm/sec)

	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	33.0	58.2	-1.0	-39.9075	-26	09-Mar-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	1.600	1.550	1.517	1.488	1.450	1.370	NA	NA	1.50	50.3	8.16E-05	ROCK TYPE:
60	3.250	3.150	3.100	3.048	3.020	2.960	NA	NA	3.09	118.1	7.18E-05	BASALT
90	5.000	4.950	4.950	4.975	4.980	4.960	4.920	4.910	4.96	184.3	7.38E-05	
45	2.200	2.200	2.200	2.212	2.220	2.230	NA	NA	2.21	84.4	7.19E-05	
											AVERAGE	
											7.47E-05	(cm/sec)

	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	34.4	105.0	-1.0	-39.9075	-26	09-Mar-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	2.450	2.400	2.367	2.338	2.320	2.270	NA	NA	2.36	49.6	1.30E-04	ROCK TYPE:
60	4.800	4.700	4.667	4.675	4.680	4.600	NA	NA	4.69	115.6	1.11E-04	BASALT
90	7.050	7.000	6.967	6.950	6.920	6.840	6.827	6.770	6.92	179.7	1.06E-04	
45	3.500	3.425	3.400	3.413	3.420	3.445	NA	NA	3.43	83.0	1.14E-04	
											AVERAGE	
											1.15E-04	(cm/sec)

27-Jun-86 MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-118

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
MF-118	266.0	324.2	5.0		138.4	-89.5	15-Jan-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
28	12.400	12.550	12.657	12.625	12.680	NA	NA	NA	12.63	176.6	8.17E-05	ROCK TYPE:
50	18.500	18.850	19.100	19.250	19.360	19.650	NA	NA	19.24	187.2	1.17E-04	GRAVEL
40	19.000	19.000	19.333	19.500	19.600	NA	NA	NA	19.29	163.8	1.34E-04	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	143.4	NA	
											AVERAGE	
											1.09E-04	(cm/sec)

27-Jun-86 MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-119

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE					
MF-119	206.7	300.0	3.5		155.1	-89.9	15-Jan-86					
PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	NA	NA	NA	NA	NA	0.010	NA	NA	0.01	227.9	3.37E-08	ROCK TYPE:
63	6.100	5.875	5.714	5.663	5.580	5.380	NA	NA	5.72	297.4	1.47E-05	PALEO-
92	11.900	12.143	12.233	12.325	12.400	12.810	13.120	13.275	12.53	340.1	2.83E-05	COLLUVIUM/
50	7.500	7.400	7.417	7.425	7.380	7.330	NA	NA	7.41	263.0	2.16E-05	SANDSTONE
											AVERAGE	
											2.08E-05	(cm/sec)

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-121

BOREHOLE:	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
MF-121	64.0	70.0	4.6	168.2	-89.8	23-Feb-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
20	17.500	17.500	17.533	17.600	17.600	17.860	NA	NA	17.60	45.7	2.69E-03	ROCK TYPE:
40	22.000	21.500	21.667	21.750	21.800	21.700	NA	NA	21.74	53.4	2.84E-03	PALEO-
60	26.000	26.500	26.333	26.500	26.400	26.500	26.467	26.450	26.39	48.2	3.82E-03	COLLUVIUM
30	21.000	21.000	21.000	21.250	21.200	21.100	NA	NA	21.09	36.7	4.01E-03	
											AVERAGE	
											3.29E-03	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	71.9	81.0	4.3	168.2	-89.8	12-Feb-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
25	5.500	5.500	5.533	5.600	5.640	6.020	NA	NA	5.63	133.3	2.95E-04	ROCK TYPE:
50	17.800	17.600	NA	17.950	18.060	18.500	NA	NA	17.98	120.9	1.04E-03	SANDSTONE
75	NA	NA	NA	NA	NA	NA	NA	NA	NA	254.0	NA	
37	8.000	17.700	17.667	17.650	17.700	17.824	17.920	17.990	16.56	102.7	1.13E-03	
											AVERAGE	
											7.02E-04	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	91.4	111.0	4.3	168.2	-89.8	12-Feb-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	17.800	18.100	18.233	18.450	18.600	19.220	NA	NA	18.40	95.8	1.34E-03	ROCK TYPE:
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	105.5	NA	SANDSTONE
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	105.5	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	105.5	NA	
											AVERAGE	
											1.34E-03	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	98.0	111.0	4.3	168.2	-89.8	12-Feb-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	19.300	19.200	19.167	19.350	19.500	19.850	NA	NA	19.39	90.2	1.50E-03	ROCK TYPE:
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	108.8	NA	SANDSTONE
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	108.8	NA	
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	108.8	NA	
											AVERAGE	
											1.50E-03	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
80.0	86.0	4.3	168.2	-89.8	23-Feb-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	3.000	2.950	2.900	2.887	2.849	2.680	NA	NA	2.88	156.1	1.29E-04	ROCK TYPE:
60	4.600	4.550	4.500	4.425	4.330	4.080	NA	NA	4.41	223.3	1.38E-04	SANDSTONE
80	5.250	5.250	5.200	5.100	5.060	4.900	4.933	7.460	5.39	267.4	1.41E-04	
40	11.700	11.650	11.600	11.625	11.600	11.570	NA	NA	11.62	149.9	5.42E-04	
											AVERAGE	
											1.92E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
120.0	126.0	4.3	168.2	-89.8	23-Feb-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	19.000	19.000	19.000	19.000	19.000	18.900	NA	NA	18.98	112.4	1.18E-03	ROCK TYPE:
60	25.000	25.200	25.000	NA	25.000	25.083	NA	NA	25.06	119.5	1.47E-03	SANDSTONE
90	28.000	29.000	29.333	29.500	29.600	29.700	29.800	29.800	29.34	136.8	1.50E-03	
45	23.000	23.500	23.333	23.250	23.400	23.200	NA	NA	23.28	104.5	1.56E-03	
											AVERAGE	
											1.42E-03	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
160.0	166.0	4.3	168.2	-89.8	22-Feb-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	16.500	16.500	16.500	20.750	33.440	NA	NA	NA	20.74	135.9	1.07E-03	ROCK TYPE:
60	22.000	22.000	22.333	22.439	45.600	NA	NA	NA	26.87	138.2	1.36E-03	SANDSTONE
90	27.857	27.500	27.667	34.750	55.800	41.900	37.267	27.950	35.09	98.1	2.50E-03	
45	25.000	25.000	28.667	31.500	50.600	NA	NA	NA	32.15	35.5	6.33E-03	
											AVERAGE	
											2.19E-03	(cm/sec)

TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
162.0	191.0	4.3	168.2	-89.8	21-Feb-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
40	34.000	34.000	34.000	33.750	34.000	34.000	NA	NA	33.96	4.0	5.98E-02	ROCK TYPE:
60	37.000	37.000	37.333	37.250	37.200	37.300	NA	NA	37.18	3.1	8.31E-02	SANDSTONE
90	27.857	36.500	38.333	39.000	39.600	41.444	NA	NA	37.12	380.0		

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-122

BOREHOLE:	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
MF-122	150.0	175.0	4.3	152.8	-89.9	08-Feb-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	18.500	18.550	18.633	18.675	18.700	19.000	19.200	NA	18.79	158.0	2.73E-04	ROCK TYPE:
38	20.500	20.500	20.333	20.375	20.400	20.450	20.467	20.500	20.43	164.3	2.85E-04	PALEO-
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	157.1	NA	COLLUVIUM
0	NA	NA	NA	NA	NA	NA	NA	NA	NA	157.1	NA	
											AVERAGE	
											2.79E-04	(cm/sec)

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-123

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
MF-123	67.8	80.4	5.3	)	208	-90	20-Mar-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
20	12.400	12.200	12.250	12.500	12.660	12.970	NA	NA	12.50	94.7	5.23E-04	ROCK TYPE:
40	20.500	20.300	20.200	20.250	20.260	20.250	NA	NA	20.29	92.3	8.71E-04	SANDSTONE
60	25.000	25.000	25.000	25.000	25.000	25.000	25.133	25.250	25.05	98.1	1.01E-03	
30	20.000	19.500	19.667	19.750	19.600	19.600	NA	NA	19.69	73.8	1.06E-03	
											AVERAGE	
											8.35E-04	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	80.6	100.7	5.3	)	208	-90	20-Mar-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
20	6.500	6.350	6.333	6.250	6.200	6.040	NA	NA	6.28	134.1	1.86E-04	ROCK TYPE:
40	8.300	8.400	8.433	8.450	8.440	8.560	NA	NA	8.43	174.0	1.92E-04	SANDSTONE
60	20.200	20.650	20.867	20.950	20.960	21.870	22.193	22.290	21.25	147.6	5.70E-04	
30	17.500	17.600	17.500	17.425	17.400	17.160	NA	NA	17.43	106.2	6.51E-04	
											AVERAGE	
											3.39E-04	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	102.6	121.9	5.3	)	208	-90	21-Mar-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	9.100	9.500	10.500	12.500	13.900	17.240	NA	NA	12.12	157.8	3.05E-04	ROCK TYPE:
60	26.400	26.500	26.667	26.500	26.600	26.600	NA	NA	26.54	121.9	8.63E-04	SANDSTONE
90	31.000	31.500	31.333	31.250	31.200	31.300	NA	NA	31.26	140.7	8.81E-04	
45	25.000	25.000	25.000	25.000	25.000	24.900	NA	NA	24.98	102.2	9.68E-04	
											AVERAGE	
											6.88E-04	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
	123.0	126.0	5.3	)	208	-90	21-Mar-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	14.600	14.550	14.567	14.600	14.560	14.370	NA	NA	14.54	150.7	3.82E-04	ROCK TYPE:
60	19.500	19.350	19.267	19.250	19.280	19.280	NA	NA	19.32	181.1	4.23E-04	SANDSTONE
90	27.000	27.500	27.667	28.000	28.200	28.900	NA	NA	27.88	157.7	7.00E-04	
45	24.000	23.800	23.800	23.750	23.720	23.590	NA	NA	23.78	101.6	9.27E-04	
											AVERAGE	
											5.69E-04	(cm/sec)

TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
153.0	159.0	5.3	>	208	-90	00-Jan-00

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
30	30.000	29.500	29.667	29.750	29.800	29.800	NA	NA	29.75	26.8	4.39E-03	ROCK TYPE:
60	34.000	34.500	34.667	34.500	34.615	NA	NA	NA	34.46	31.9	4.28E-03	SANDSTONE
90	38.000	38.000	37.667	37.750	37.800	NA	NA	NA	37.84	51.2	2.93E-03	
45	32.000	31.500	31.667	31.750	31.600	NA	NA	NA	31.70	35.6	3.53E-03	
											AVERAGE	
										LESS THAN	3.73E-03	(cm/sec)

TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE
183.0	202.9	5.3	>	208	-90	22-Mar-86

PRESSURE (psi)	TIME (min) 1	2	3	4	5	10	15	20	AVERAGE FLOW (gpm)	HEAD (feet)	HYDRAULIC CONDUCTIVITY (cm/sec)	
40	25.000	25.000	25.000	25.000	25.000	25.200	NA	NA	25.03	170.9	5.80E-04	ROCK TYPE:
73	30.000	30.000	30.333	30.250	30.400	30.300	NA	NA	30.21	194.0	6.17E-04	SANDSTONE
103	35.000	35.500	35.333	35.250	35.400	35.500	NA	NA	35.33	201.6	6.94E-04	
58	30.000	30.000	30.333	30.250	30.200	30.200	NA	NA	30.16	159.9	7.47E-04	
											AVERAGE	
										LESS THAN	6.57E-04	(cm/sec)

BOREHOLE:	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE					
MF-124	78.0	90.9	5.3		159.75	-90	02-Apr-86					
									AVERAGE		HYDRAULIC	
PRESSURE				TIME	(min)				FLOW	HEAD	CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
25	14.750	13.800	12.833	12.000	11.260	8.476	NA	NA	12.19	118.1	4.01E-04	ROCK TYPE:
50	8.000	7.750	7.733	7.625	7.600	7.360	NA	NA	7.68	193.3	1.55E-04	BASALT
80	12.800	12.900	12.233	12.100	12.000	11.770	11.700	11.865	12.17	245.2	1.93E-04	
40	4.300	4.200	4.217	4.200	4.220	4.190	NA	NA	4.22	178.4	9.20E-05	
											AVERAGE	
											1.82E-04	(cm/sec)

	TOP	BOTTOM	GAUGE HT.		H2O LEVEL	INCLIN.	DATE					
	0.0	0.0	5.3		159.75	-90	02-Apr-86					
	GS	NG.							AVERAGE		HYDRAULIC	
PRESSURE				TIME	(min)				FLOW	HEAD	CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
30	9.500	9.000	NA	6.950	6.500	5.855	NA	NA	7.71	62.6	4.80E-04	ROCK TYPE:
68	24.000	22.500	22.667	23.000	23.200	23.500	NA	NA	23.14	59.6	1.51E-03	SANDSTONE
96	29.000	28.500	28.667	28.571	28.600	28.700	29.000	29.450	28.81	69.5	1.61E-03	
54	25.000	24.500	24.667	24.750	24.800	24.800	NA	NA	24.75	12.9	7.46E-03	
											AVERAGE	
											1.72E-03	(cm/sec)

MINER FLAT DAM SITE: HYDRAULIC CONDUCTIVITIES: BOREHOLE MF-125A

BOREHOLE:	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
MF-125A	108.0	141.1	6.5	153.17	-90	06-Apr-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
33	1.350	1.350	1.283	1.238	1.190	0.840	NA	NA	1.21	207.0	1.07E-05	ROCK TYPE:
63	4.500	4.400	4.267	4.150	NA	4.000	NA	NA	4.26	272.8	2.85E-05	PALEO-
93	16.000	15.700	15.667	15.750	15.800	15.700	15.533	15.400	15.69	297.7	9.61E-05	CONTACT
45	9.300	9.250	9.233	9.125	9.080	8.680	NA	NA	9.11	218.3	7.61E-05	
											AVERAGE	
											3.86E-05	(cm/sec)

	TOP	BOTTOM	GAUGE HT.	H2O LEVEL	INCLIN.	DATE
	107.7	170.0	6.5	153.17	-90	07-Apr-86

PRESSURE	TIME (min)								AVERAGE FLOW	HEAD	HYDRAULIC CONDUCTIVITY	
(psi)	1	2	3	4	5	10	15	20	(gpm)	(feet)	(cm/sec)	
31	9.700	NA	10.300	10.425	10.480	10.644	NA	NA	10.31	210.1	8.95E-05	ROCK TYPE:
65	19.500	19.400	19.433	19.375	19.400	19.300	NA	NA	19.40	237.0	1.49E-04	SANDSTONE
100	26.000	26.500	26.333	26.500	26.600	26.600	26.733	26.816	26.51	256.8	1.88E-04	
45	19.000	19.500	NA	19.750	19.608	19.600	NA	NA	19.49	190.1	1.87E-04	
											AVERAGE	
											1.47E-04	(cm/sec)

APPENDIX E

Basic Data, Goodman Jack Tests

GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

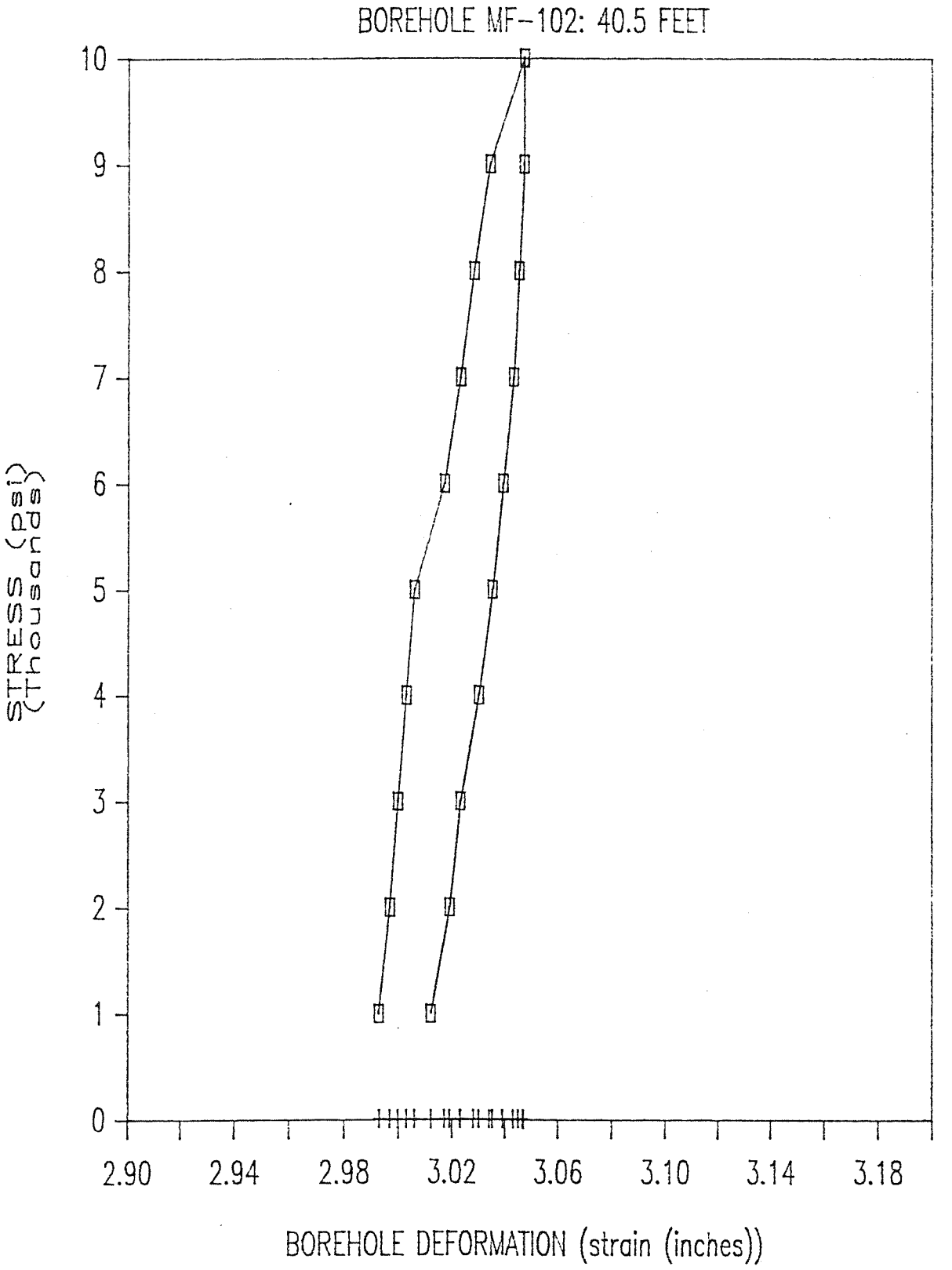
MF-102	80.5 FT		DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
1,000	-0.007	-0.008	2.994	2.993			0.25	0.26
2,000	-0.003	-0.002	2.998	2.999			0.82	0.59
3,000	0.017	0.036	3.018	3.035			0.17	0.09
4,000	0.028	0.051	3.029	3.050			0.31	0.23
5,000	0.036	0.061	3.037	3.059			0.42	0.35
6,000	0.046	0.072	3.047	3.070			0.34	0.31
7,000	0.054	0.081	3.055	3.079			0.43	0.38
8,000	0.062	0.089	3.062	3.087			0.43	0.42
9,000	0.068	0.096	3.068	3.094	AVERAGE	DEFORMATION	0.58	0.48
10,000	0.075	0.106	3.075	3.104	(10^6 psi)	MODULUS	0.50	0.33
10,000	0.075	0.106	3.075	3.104	0.42	0.34	RETRACT	
9,000	0.068	0.096	3.068	3.094			0.50	3.3E-01
8,000	0.071	0.104	3.071	3.102			1.16	3.3E-01
7,000	0.069	0.102	3.069	3.100			1.74	4.2E-01
6,000	0.065	0.098	3.065	3.096			0.87	1.7E+00
5,000	0.061	0.093	3.062	3.091			0.87	8.3E-01
4,000	0.057	0.089	3.058	3.087			0.86	6.7E-01
3,000	0.052	0.084	3.053	3.082			0.69	8.4E-01
2,000	0.046	0.077	3.047	3.075	AVERAGE	RECOVERY	0.57	6.8E-01
1,000	0.037	0.066	3.038	3.064	(10^6 psi)	MODULUS	0.38	4.9E-01
					0.85	0.70		

MF-102	160.5 FEET		DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
1,000	-0.011	-0.011	2.989	2.990			0.38	0.33
2,000	-0.006	-0.006	2.995	2.995			0.66	0.70
3,000	-0.003	-0.002	2.998	2.999			1.10	0.88
4,000	-0.001	0.000	3.000	3.001			1.65	1.76
5,000	0.001	0.002	3.002	3.003			1.65	1.76
6,000	0.003	0.005	3.004	3.005			1.65	1.17
7,000	0.005	0.007	3.006	3.007			1.66	1.76
8,000	0.007	0.010	3.008	3.010			1.66	1.17
9,000	0.010	0.012	3.011	3.012	AVERAGE	DEFORMATION	1.11	1.76
10,000	0.012	0.014	3.013	3.014	(10^6 psi)	MODULUS	1.67	1.76
10,000	0.012	0.014	3.013	3.014	1.32	1.31	RETRACT	
9,000	0.012	0.014	3.013	3.014			ERR	ERR
8,000	0.011	0.014	3.012	3.014			ERR	ERR
7,000	0.010	0.013	3.011	3.013			3.34	ERR
6,000	0.008	0.011	3.009	3.011			3.33	3.52
5,000	0.006	0.009	3.007	3.009			1.66	1.76
4,000	0.004	0.007	3.005	3.007			1.66	1.76
3,000	0.002	0.004	3.003	3.005			1.66	1.76
2,000	-0.001	0.002	3.000	3.003	AVERAGE	RECOVERY	1.66	1.17
1,000	-0.006	-0.002	2.995	2.999	(10^6 psi)	MODULUS	1.10	1.76
					2.06	1.96		

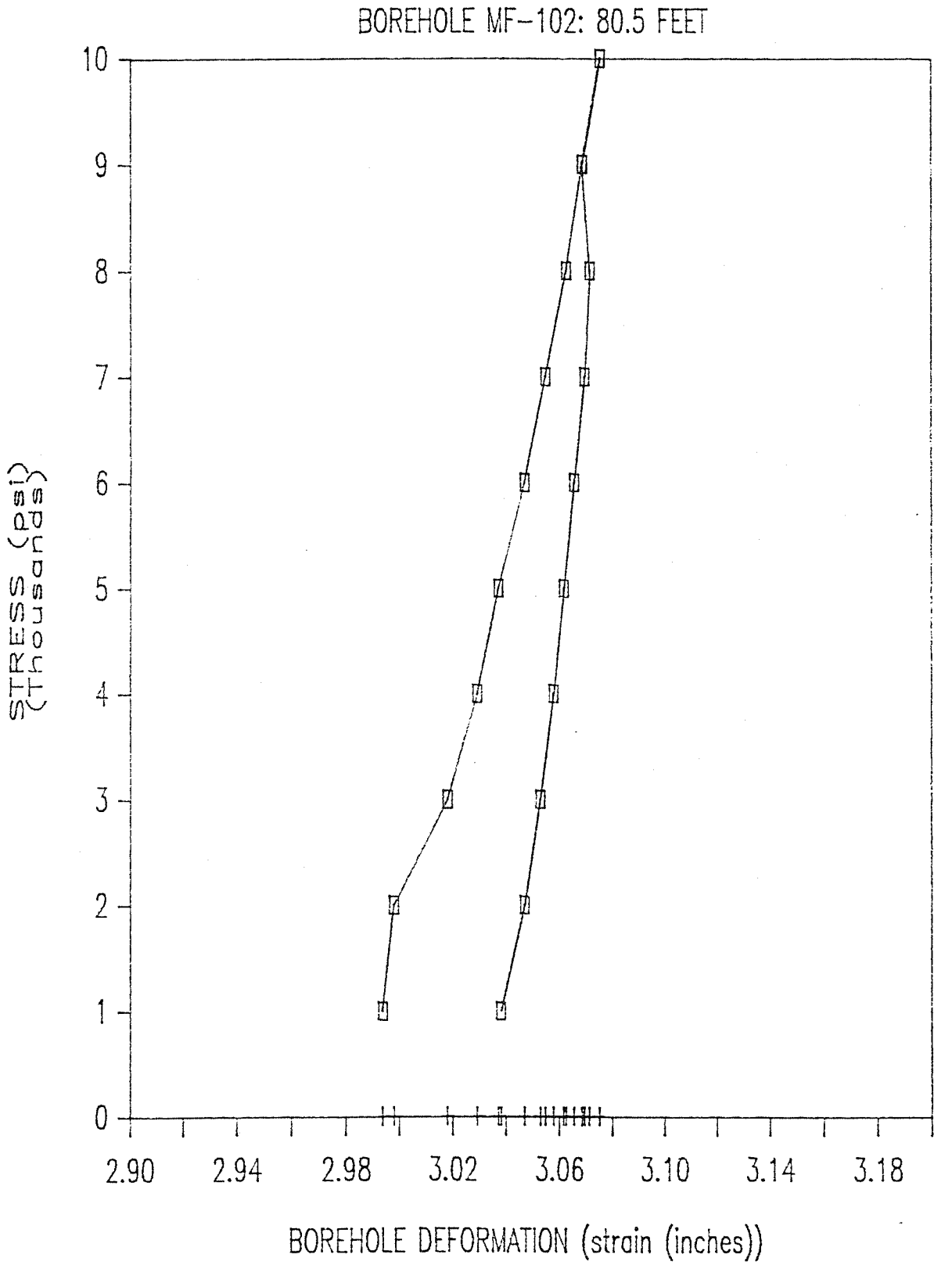
GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

MF-102 40.5 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
1,000	-0.008	-0.014	2.993	2.987			0.27	0.46
2,000	-0.004	-0.009	2.997	2.992			0.82	0.70
3,000	-0.001	-0.005	3.000	2.996			1.10	0.88
4,000	0.002	-0.001	3.003	3.000			1.10	0.88
5,000	0.005	0.003	3.006	3.004			1.10	0.88
6,000	0.016	0.015	3.017	3.015			0.30	0.29
7,000	0.022	0.022	3.023	3.022			0.56	0.50
8,000	0.027	0.028	3.028	3.028			0.67	0.59
9,000	0.033	0.036	3.034	3.035	AVERAGE	DEFORMATION	0.56	0.44
10,000	0.046	0.048	3.047	3.047	(10^6 psi)	MODULUS	0.26	0.29
10,000	0.046	0.048	3.047	3.047	0.68	0.59	RETRACT	
9,000	0.046	0.048	3.047	3.047			ERR	ERR
8,000	0.044	0.045	3.045	3.044			ERR	ERR
7,000	0.042	0.042	3.043	3.041			1.71	1.16
6,000	0.038	0.038	3.039	3.037			1.71	1.16
5,000	0.034	0.034	3.035	3.033			0.85	0.87
4,000	0.029	0.027	3.030	3.027			0.85	0.87
3,000	0.022	0.018	3.023	3.018			0.68	0.50
2,000	0.018	0.013	3.019	3.013	AVERAGE	RECOVERY	0.48	0.39
1,000	0.011	0.004	3.012	3.005	(10^6 psi)	MODULUS	0.84	0.70
					1.02	0.81		

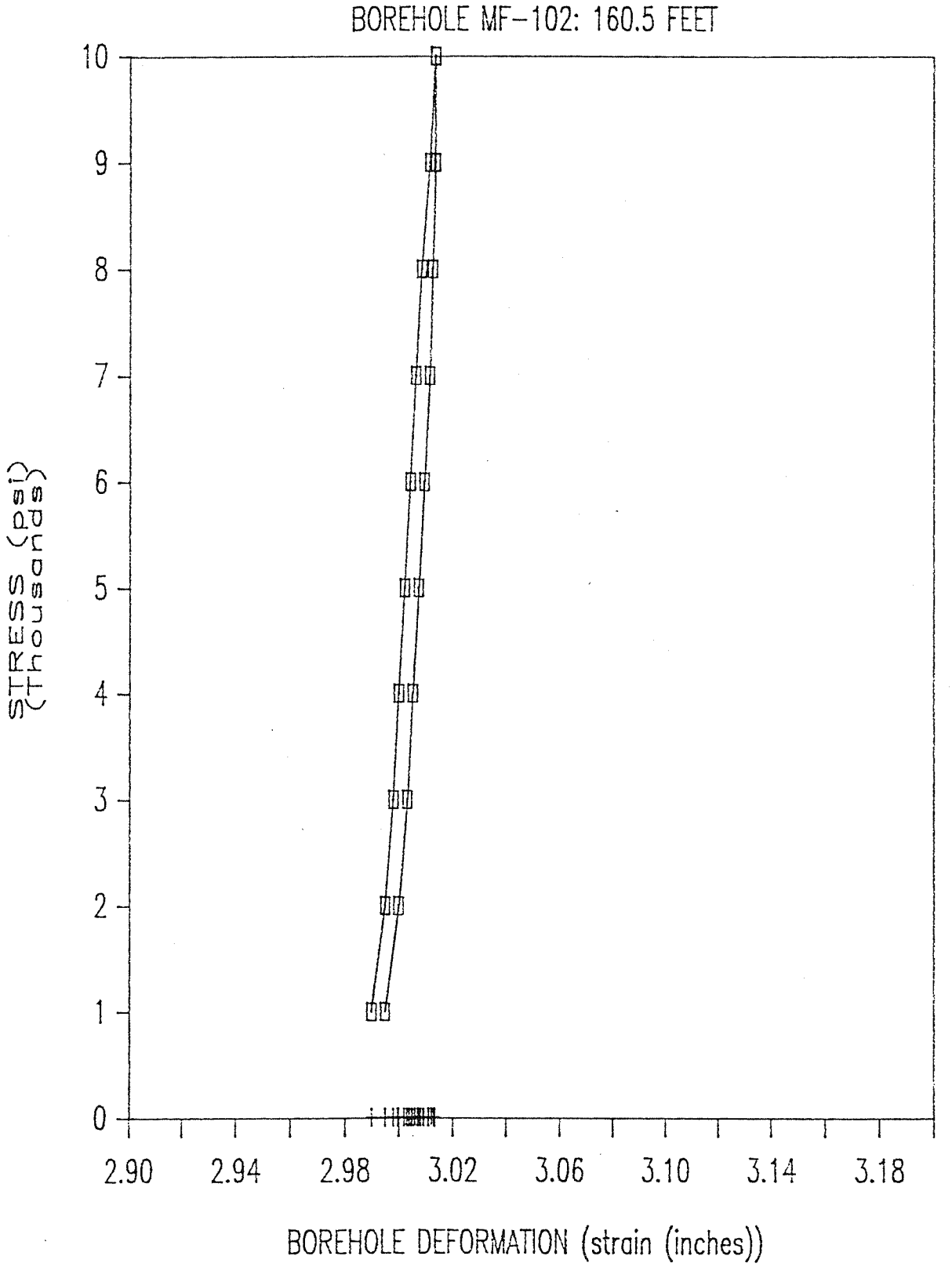
DEFORMATION MODULAE: MINER FLAT DAM



DEFORMATION MODULAE: MINER FLAT DAM



DEFORMATION MODULAE: MINER FLAT DAM



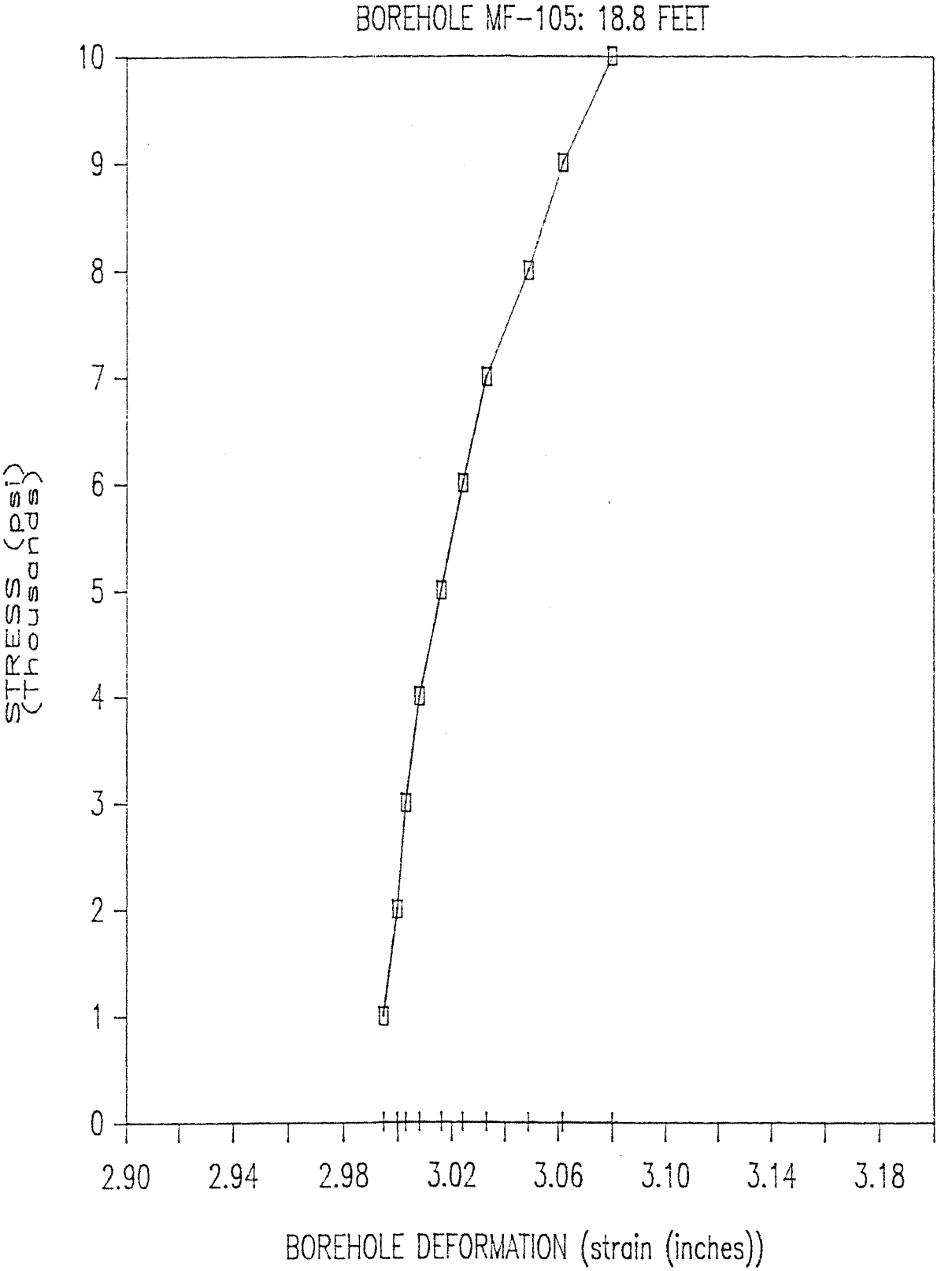
GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

MF-105 18.8 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
1,000	-0.006	-0.011	2.995	2.990			0.23	0.33
2,000	-0.001	-0.004	3.000	2.997			0.66	0.50
3,000	0.002	0.003	3.003	3.004			1.10	0.50
4,000	0.007	0.013	3.008	3.013			0.66	0.35
5,000	0.015	0.027	3.016	3.027			0.42	0.25
6,000	0.023	0.047	3.024	3.046			0.42	0.17
7,000	0.032	0.080	3.033	3.078			0.38	0.10
8,000	0.048	0.096	3.049	3.094			0.21	0.21
9,000	0.061	0.107	3.062	3.105	AVERAGE	DEFORMATION	0.26	0.30
10,000	0.080	0.127	3.080	3.126	(10^6 psi)	MODULUS	0.18	0.16
					0.45	0.29		

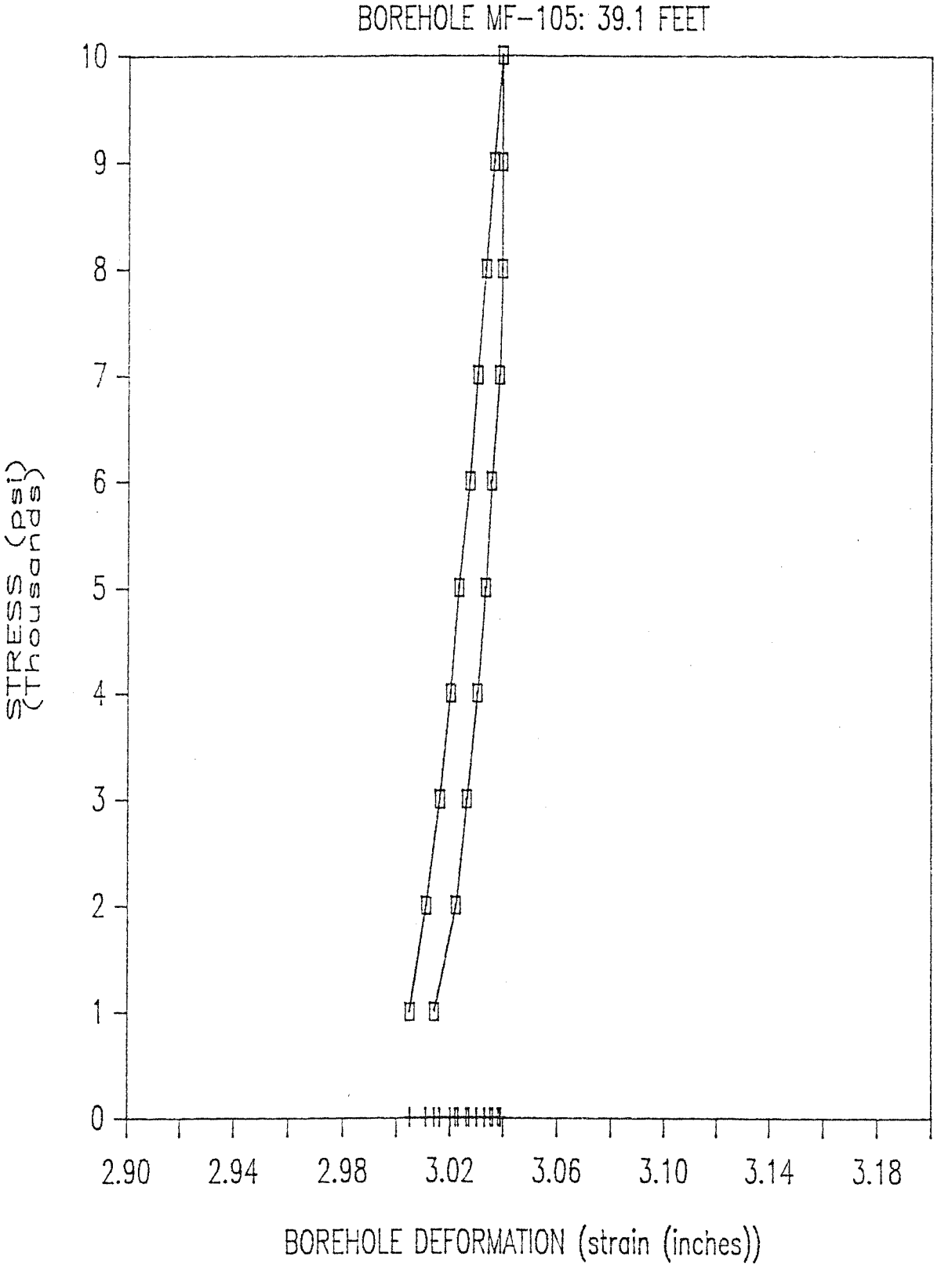
MF-105 39.1 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
1,000	0.004	0.021	3.005	3.021			0.14	0.08
2,000	0.010	0.030	3.011	3.029			0.55	0.39
3,000	0.015	0.036	3.016	3.035			0.67	0.58
4,000	0.019	0.042	3.020	3.041			0.84	0.58
5,000	0.022	0.047	3.023	3.046			1.12	0.70
6,000	0.026	0.053	3.027	3.052			0.84	0.58
7,000	0.029	0.058	3.030	3.057			1.13	0.69
8,000	0.032	0.063	3.033	3.061			1.13	0.69
9,000	0.035	0.068	3.036	3.066	AVERAGE	DEFORMATION	1.13	0.69
10,000	0.038	0.072	3.039	3.070	(10^6 psi)	MODULUS	1.13	0.86
10,000	0.038	0.072	3.039	3.070	0.87	0.58	RETRACT	
9,000	0.038	0.072	3.039	3.070			ERR	ERR
8,000	0.038	0.072	3.039	3.070			ERR	ERR
7,000	0.037	0.070	3.038	3.068			ERR	ERR
6,000	0.034	0.068	3.035	3.066			3.40	1.71
5,000	0.032	0.065	3.033	3.063			1.13	1.72
4,000	0.029	0.062	3.030	3.060			1.70	1.15
3,000	0.025	0.059	3.026	3.057			1.13	1.15
2,000	0.021	0.055	3.022	3.054	AVERAGE	RECOVERY	0.84	1.15
1,000	0.013	0.049	3.014	3.048	(10^6 psi)	MODULUS	0.84	0.87
					1.51	1.29		

MF-105 59.6 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
1,000	0.002	0.022	3.003	3.022			0.15	0.08
2,000	0.006	0.028	3.007	3.028			0.83	0.59
3,000	0.009	0.035	3.010	3.034			1.11	0.50
4,000	0.013	0.042	3.014	3.041			0.83	0.50
5,000	0.017	0.050	3.018	3.049			0.84	0.44
6,000	0.021	0.060	3.022	3.058			0.84	0.35
7,000	0.025	0.070	3.026	3.068			0.84	0.34
8,000	0.030	0.079	3.031	3.077			0.68	0.38
9,000	0.034	0.093	3.035	3.091	AVERAGE	DEFORMATION	0.85	0.24
10,000	0.038	0.110	3.039	3.108	(10^6 psi)	MODULUS	0.85	0.20
					0.78	0.36		

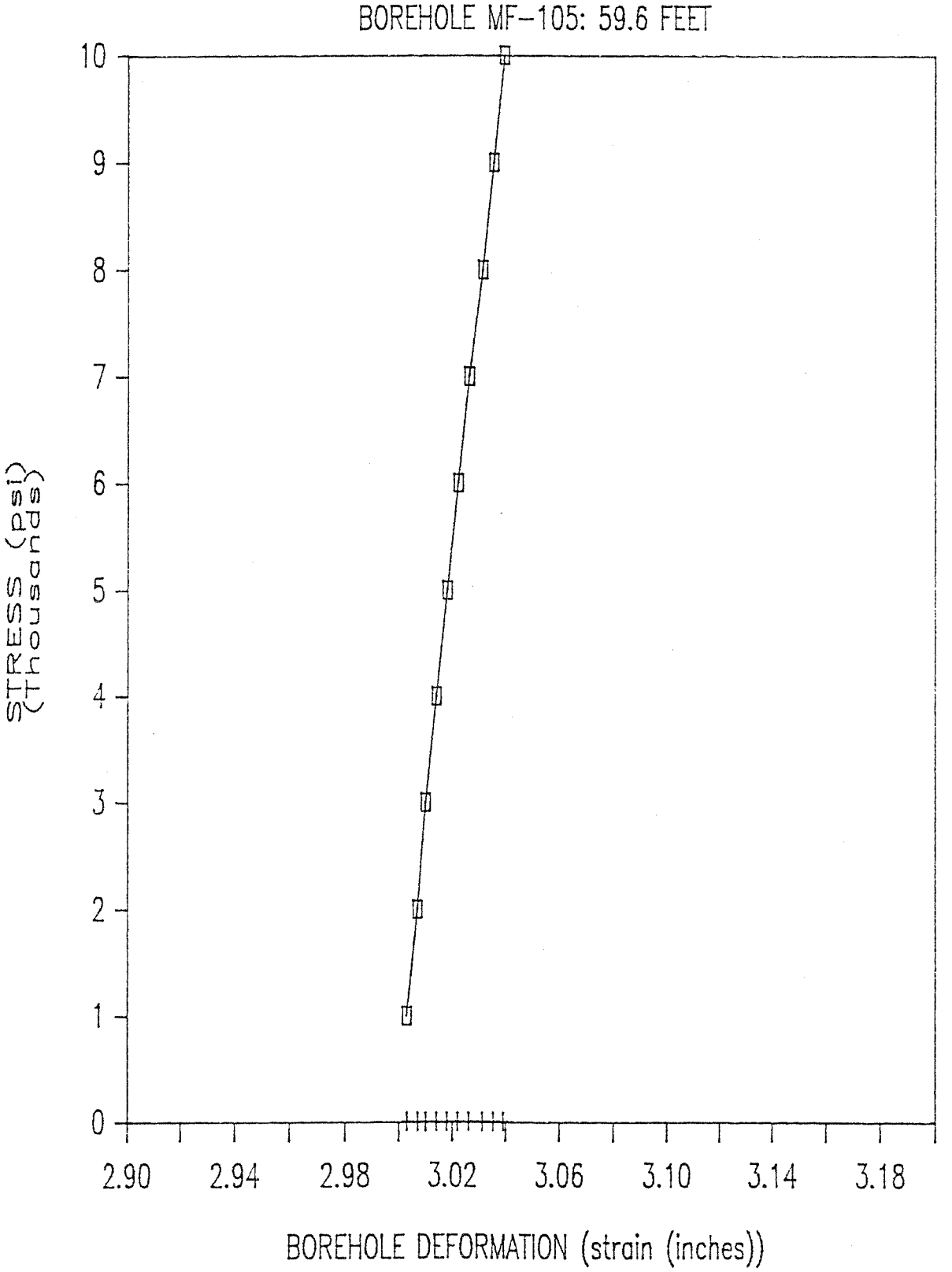
DEFORMATION MODULAE: MINER FLAT DAM



DEFORMATION MODULAE: MINER FLAT DAM



DEFORMATION MODULAE: MINER FLAT DAM



GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

MF-113 133.0 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
500	-0.022	-0.030	2.978	2.972				
600	-0.012	-0.029	2.988	2.973			0.03	0.35
700	-0.003	-0.028	2.998	2.974			0.04	0.35
800	0.010	-0.027	3.011	2.975			0.03	0.35
900	0.019	-0.026	3.020	2.976			0.04	0.35
800	0.022	-0.026	3.023	2.976			NA	NA
700	0.022	-0.026	3.023	2.976			NA	NA
600	0.022	-0.026	3.023	2.976			NA	NA
500	0.022	-0.027	3.023	2.975	AVERAGE	DEFORMATION	NA	NA
					(10^6 psi)	MODULUS	NA	NA
					0.03	0.35		

MF-113 130.6 FEET							EXTEND	
PRESSURE	NEAR	FAR	DIANEAR	DIAFAR			E(near)	E(far)
500	-0.007	0.160	2.994	3.161			123.96	9.31
600	0.002	0.220	3.003	3.230			36.64	4.88
700	0.009	0.280	3.010	3.307			47.42	4.35
800	0.018	0.360	3.019	3.428			37.12	2.80
900	0.035	0.500	3.036	3.702			19.85	1.23
1,000	0.047	0.600	3.048	3.960			28.43	1.30
1,500	0.103	0.111	3.102	3.109			31.15	-1.98
1,600	0.111	0.123	3.109	3.122			44.42	27.29
1,700	0.118	0.135	3.116	3.134			50.95	26.88
1,800	0.121	0.139	3.119	3.138			119.17	79.78
2,000	0.132	0.166	3.129	3.168	AVERAGE	DEFORMATION	65.21	23.11
2,200	0.138	0.184	3.135	3.188	(10^3 psi)	MODULUS	119.98	33.45
2,000	0.138	0.184	3.135	3.188	53.91	15.58	RETRACT	
1,800	0.138	0.184	3.135	3.188			ERR	ERR
1,700	0.138	0.184	3.135	3.188			ERR	ERR
1,600	0.138	0.184	3.135	3.188			ERR	ERR
1,500	0.138	0.184	3.135	3.188			ERR	ERR
1,000	0.138	0.185	3.135	3.189			ERR	ERR
900	0.137	0.185	3.134	3.189			ERR	ERR
800	0.137	0.185	3.134	3.189			ERR	-0.30
700	0.136	0.185	3.133	3.189	AVERAGE	RECOVERY	0.36	ERR
600	0.136	0.185	3.133	3.189	(10^3 psi)	MODULUS	ERR	ERR
500	0.135	0.184	3.132	3.188	NA	NA	0.36	ERR

GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

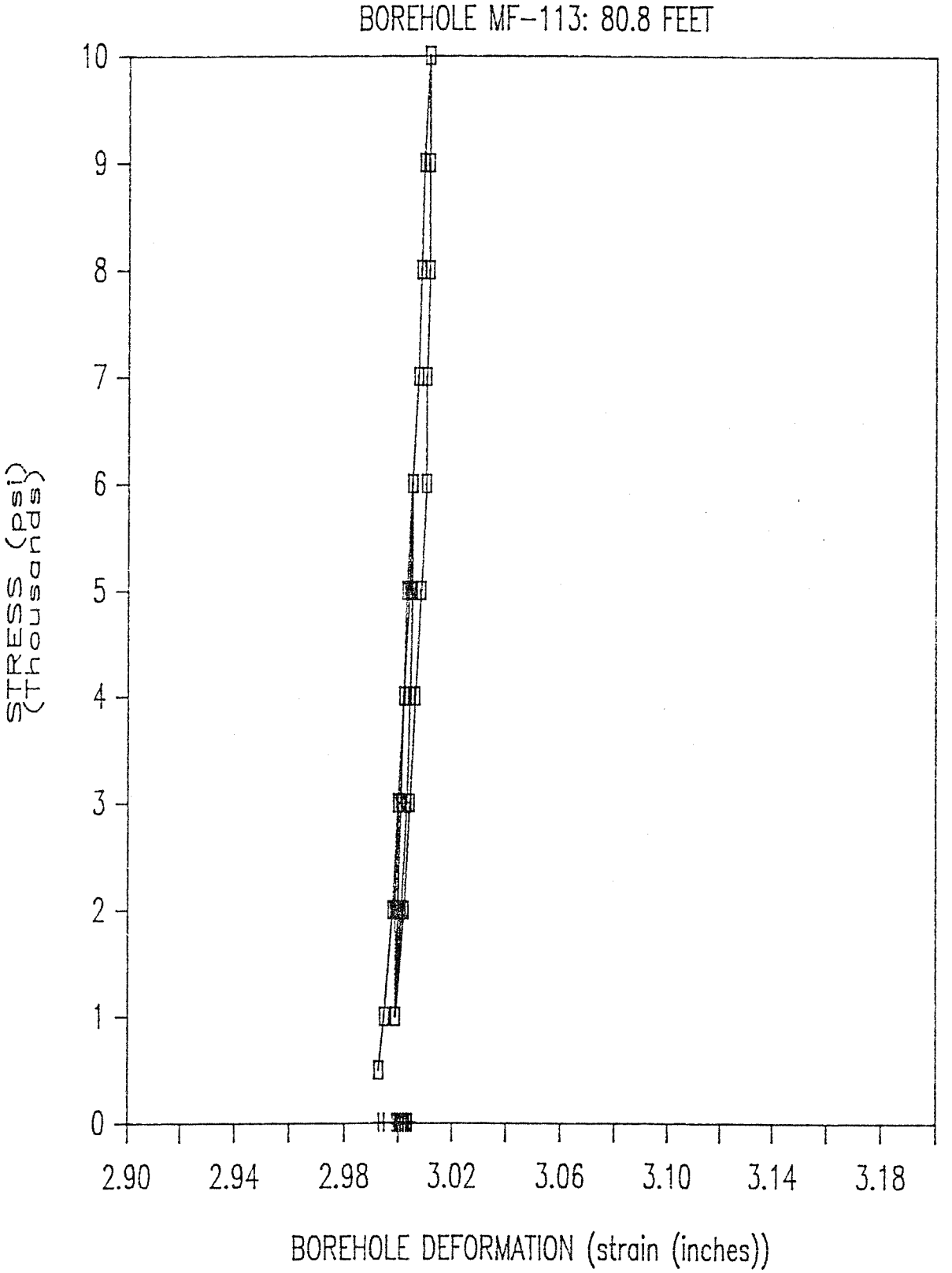
MF-113 121.5 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
500	-0.003	0.002	2.998	3.003			0.10	0.07
600	-0.001	0.007	3.000	3.007			0.16	0.07
700	0.000	0.013	3.001	3.013			0.33	0.06
800	0.002	0.021	3.003	3.021			0.17	0.04
900	0.005	0.026	3.006	3.026			0.11	0.07
1,000	0.008	0.031	3.009	3.030			0.11	0.07
1,500	0.016	0.058	3.017	3.057			0.21	0.06
1,600	0.019	0.064	3.020	3.062			0.11	0.06
1,800	0.023	0.068	3.024	3.066	AVERAGE	DEFORMATION	0.17	0.17
2,000	0.027	0.086	3.028	3.084	(10^6 psi)	MODULUS	0.17	0.04
2,000	0.027	0.086	3.028	3.084	0.16	0.07	RETRACT	
1,800	0.026	0.086	3.027	3.084			0.68	ERR
1,600	0.025	0.086	3.026	3.084			0.68	ERR
1,500	0.024	0.086	3.025	3.084			0.34	ERR
1,000	0.019	0.086	3.020	3.084			1.68	ERR
900	0.018	0.086	3.019	3.084			0.07	ERR
800	0.016	0.085	3.017	3.083			0.34	ERR
700	0.015	0.085	3.016	3.083			0.17	0.34
600	0.013	0.082	3.014	3.080	AVERAGE	RECOVERY	0.33	ERR
500	0.011	0.079	3.012	3.077	(10^6 psi)	MODULUS	0.17	0.11
					0.46	ERR		

MF-113 101.5 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
500	-0.005	-0.010	2.996	2.991			SET	JACK
1,000	-0.005	-0.010	2.996	2.991			ERR	ERR
2,000	-0.003	-0.007	2.998	2.994			1.65	1.17
3,000	-0.001	-0.004	3.000	2.997			1.65	1.17
4,000	0.000	-0.002	3.001	2.999			3.30	1.76
5,000	0.001	0.000	3.002	3.001			3.31	1.76
6,000	0.003	0.001	3.004	3.002			1.65	3.52
7,000	0.004	0.003	3.005	3.004			3.31	1.76
8,000	0.006	0.004	3.007	3.005			1.66	3.52
9,000	0.007	0.006	3.008	3.006	AVERAGE	DEFORMATION	3.32	1.76
10,000	0.008	0.007	3.009	3.007	(10^6 psi)	MODULUS	3.32	3.52
10,000	0.008	0.007	3.009	3.007	2.58	2.22	RETRACT	
9,000	0.008	0.007	3.009	3.007			ERR	ERR
8,000	0.008	0.007	3.009	3.007			ERR	ERR
7,000	0.007	0.007	3.008	3.007			ERR	ERR
6,000	0.006	0.006	3.007	3.006			3.32	ERR
5,000	0.004	0.004	3.005	3.005			3.32	3.52
4,000	0.003	0.003	3.004	3.004			1.66	1.76
3,000	0.001	0.001	3.002	3.002			3.31	3.52
2,000	0.000	-0.001	3.001	3.000	AVERAGE	RECOVERY	1.65	1.76
1,000	-0.002	-0.005	2.999	2.996	(10^6 psi)	MODULUS	3.31	1.76
					2.76	2.46		

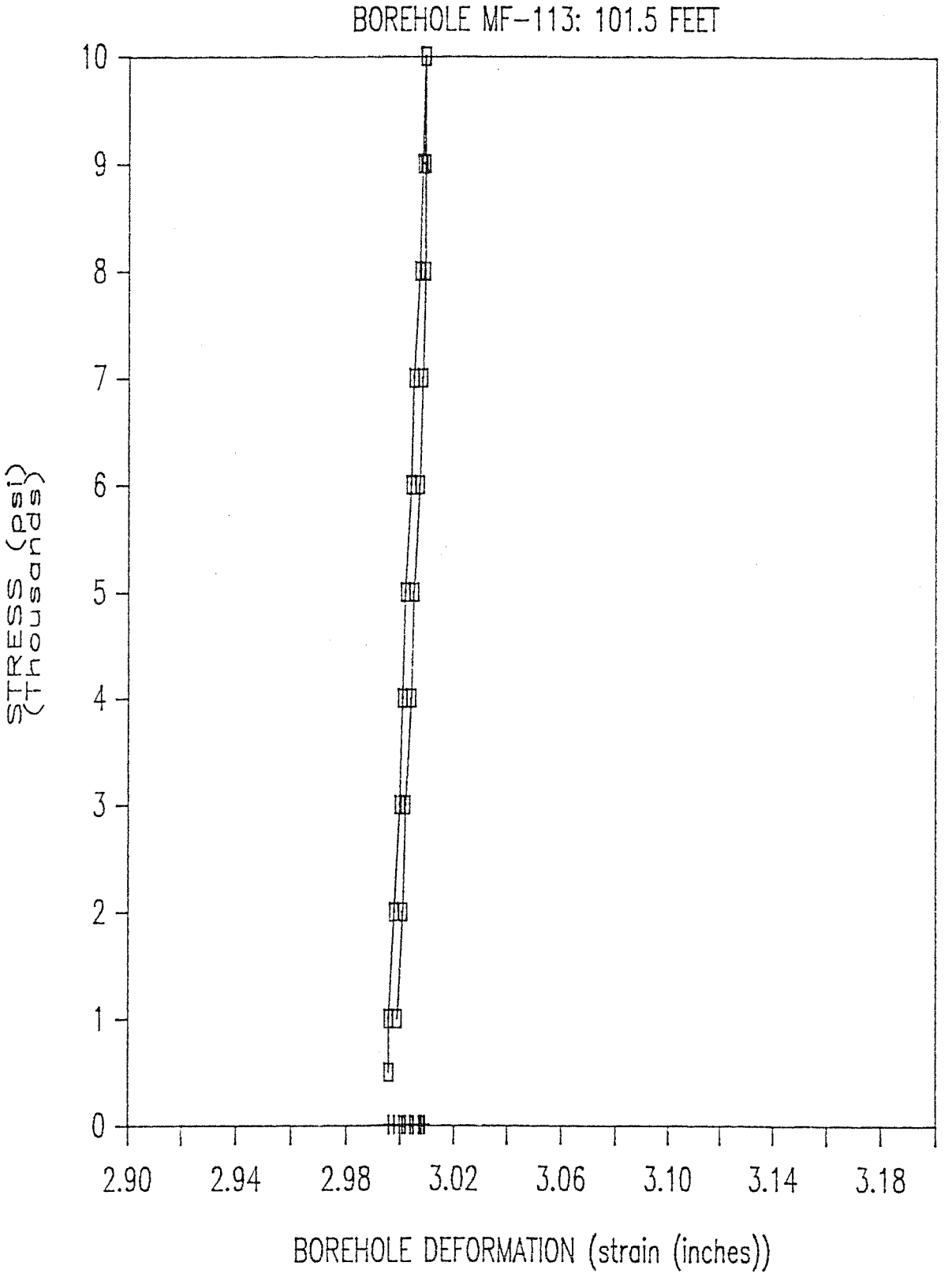
GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

MF-113 80.8 FEET PRESSURE	NEAR	FAR	DIAMETER NEAR	DIAMETER FAR			EXTEND E(near) SET	EXTEND E(far) JACK
500	-0.008	-0.019	2.993	2.982			0.82	0.88
1,000	-0.006	-0.017	2.995	2.984			1.10	0.88
2,000	-0.003	-0.013	2.998	2.988			1.65	1.17
3,000	-0.001	-0.010	3.000	2.991			ERR	ERR
2,000	-0.001	-0.010	3.000	2.991			3.30	1.17
1,000	-0.002	-0.013	2.999	2.988			ERR	ERR
2,000	-0.002	-0.013	2.999	2.988			3.30	0.88
3,000	-0.001	-0.009	3.000	2.992			1.65	1.17
4,000	0.001	-0.006	3.002	2.995			3.31	1.76
5,000	0.002	-0.004	3.003	2.997			1.66	1.76
6,000	0.004	-0.002	3.005	2.999			ERR	ERR
5,000	0.004	-0.002	3.005	2.999			3.31	ERR
4,000	0.003	-0.002	3.004	2.999			3.31	1.76
3,000	0.002	-0.004	3.003	2.997			1.65	1.17
2,000	0.000	-0.007	3.001	2.994			1.65	0.70
1,000	-0.002	-0.012	2.999	2.989			3.30	3.52
2,000	-0.001	-0.011	3.000	2.990			3.30	1.17
3,000	0.000	-0.008	3.001	2.993			3.31	1.17
4,000	0.001	-0.005	3.002	2.996			1.65	1.76
5,000	0.003	-0.003	3.004	2.998			3.31	1.76
6,000	0.004	-0.001	3.005	3.000			1.66	3.52
7,000	0.006	0.000	3.007	3.001			3.32	1.76
8,000	0.007	0.002	3.008	3.003			3.32	3.52
9,000	0.008	0.003	3.009	3.004			1.66	1.76
10,000	0.010	0.005	3.011	3.005			ERR	ERR
9,000	0.010	0.005	3.011	3.005			ERR	ERR
8,000	0.010	0.005	3.011	3.005			3.33	ERR
7,000	0.009	0.005	3.010	3.005			ERR	3.52
6,000	0.009	0.004	3.010	3.005	AVERAGE	DEFORMATION	1.66	1.76
5,000	0.007	0.002	3.008	3.003	(10^6 psi)	MODULUS	1.66	1.76
4,000	0.005	0.000	3.006	3.001	2.46	1.66	1.66	1.17
3,000	0.003	-0.003	3.004	2.998			1.65	0.88
2,000	0.001	-0.007	3.002	2.994	AVERAGE	RECOVERY	1.10	0.88
1,000	-0.002	-0.011	2.999	2.990	(10^6 psi)	MODULUS		
					2.21	1.48		

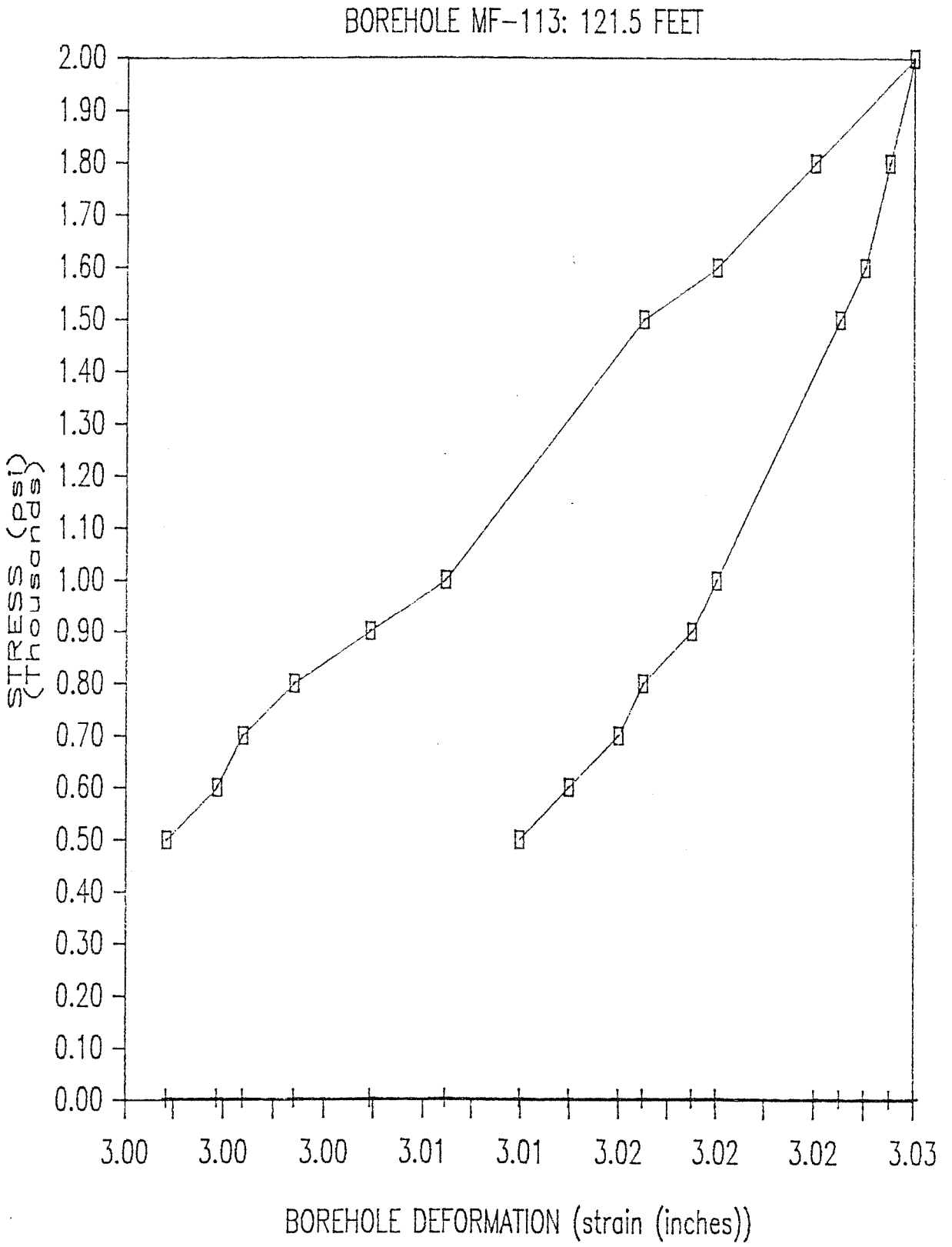
DEFORMATION MODULAE: MINER FLAT DAM



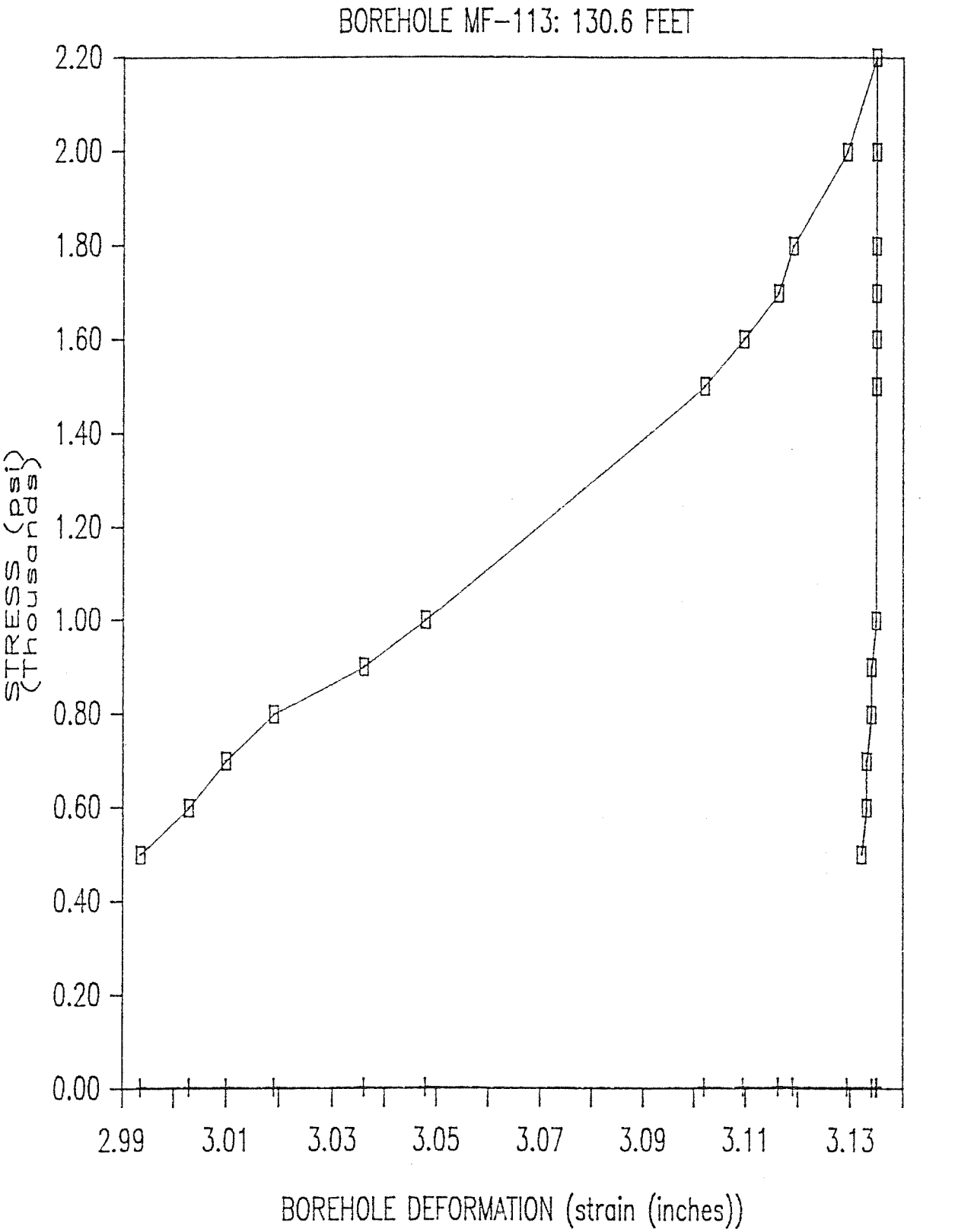
DEFORMATION MODULAE: MINER FLAT DAM



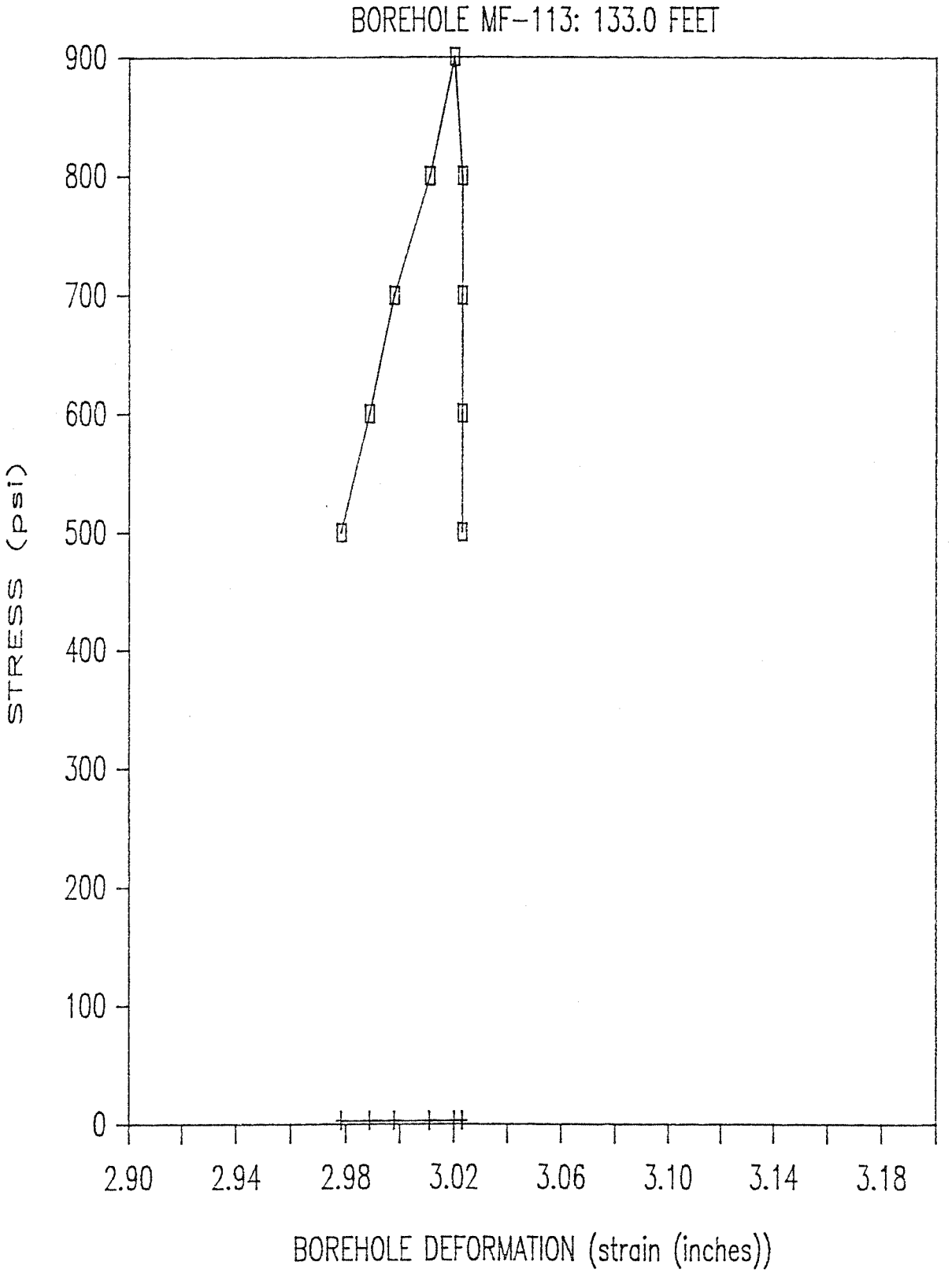
DEFORMATION MODULAE: MINER FLAT DAM



DEFORMATION MODULAE: MINER FLAT DAM



DEFORMATION MODULAE: MINER FLAT DAM



GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

MF-117 10.0 FEET PRESSURE	NEAR	FAR	DIAMETER NEAR	DIAMETER FAR			EXTEND E(near) SET	E(far) JACK
500	0.014	0.000	3.015	3.001			0.84	0.59
1,000	0.016	0.003	3.017	3.004			0.84	0.50
2,000	0.020	0.010	3.021	3.010			1.12	0.39
3,000	0.023	0.019	3.024	3.019			1.68	ERR
2,000	0.021	0.019	3.022	3.019			0.84	0.59
1,000	0.017	0.013	3.018	3.013			1.12	1.76
2,000	0.020	0.015	3.021	3.015			1.12	0.70
3,000	0.023	0.020	3.024	3.020			1.12	0.44
4,000	0.026	0.028	3.027	3.028			1.13	0.35
5,000	0.029	0.038	3.030	3.037			0.85	0.29
6,000	0.033	0.050	3.034	3.049			ERR	ERR
5,000	0.033	0.050	3.034	3.049			1.69	3.48
4,000	0.031	0.049	3.032	3.048			0.85	0.87
3,000	0.027	0.045	3.028	3.044			1.12	0.50
2,000	0.024	0.038	3.025	3.037			0.67	0.35
1,000	0.019	0.028	3.020	3.028			1.68	1.75
2,000	0.021	0.030	3.022	3.029			1.12	0.70
3,000	0.024	0.035	3.025	3.034			1.12	0.58
4,000	0.027	0.041	3.028	3.040			1.13	0.70
5,000	0.030	0.046	3.031	3.045			1.13	0.58
6,000	0.033	0.052	3.034	3.051			0.85	0.49
7,000	0.037	0.059	3.038	3.057			0.57	0.31
8,000	0.043	0.070	3.044	3.068			0.07	0.24
9,000	0.090	0.084	3.089	3.082			0.22	0.42
10,000	0.106	0.092	3.105	3.090			ERR	ERR
9,000	0.106	0.092	3.105	3.090			ERR	ERR
8,000	0.106	0.092	3.105	3.090			1.18	ERR
7,000	0.103	0.092	3.102	3.090			0.89	ERR
6,000	0.099	0.092	3.098	3.090	AVERAGE	DEFORMATION	0.88	1.12
5,000	0.095	0.089	3.094	3.087	(10^6 psi)	MODULUS	0.88	0.56
4,000	0.091	0.083	3.090	3.081	0.94	0.57	0.70	0.57
3,000	0.086	0.077	3.086	3.075			0.70	0.38
2,000	0.081	0.068	3.081	3.066	AVERAGE	RECOVERY	0.70	0.29
1,000	0.076	0.056	3.076	3.055	(10^6 psi)	MODULUS		
					0.93	0.87		

GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

MF-117 30.0 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
500	0.014	0.017	3.015	3.017			SET	JACK
1,000	0.017	0.022	3.018	3.022			0.56	0.35
2,000	0.018	0.032	3.019	3.031			3.35	0.35
3,000	0.021	0.047	3.022	3.046			1.12	0.23
2,000	0.020	0.041	3.021	3.040			3.36	0.58
1,000	0.018	0.037	3.019	3.036			1.68	0.87
2,000	0.019	0.038	3.020	3.037			3.35	3.50
3,000	0.021	0.042	3.022	3.041			1.68	0.87
4,000	0.024	0.051	3.025	3.050			1.12	0.39
5,000	0.027	0.060	3.028	3.058			1.12	0.38
6,000	0.030	0.069	3.031	3.067			1.13	0.38
5,000	0.030	0.069	3.031	3.067			ERR	ERR
4,000	0.028	0.069	3.029	3.067			1.69	ERR
3,000	0.025	0.067	3.026	3.065			1.13	1.72
2,000	0.022	0.062	3.023	3.060			1.12	0.69
1,000	0.019	0.052	3.020	3.051			1.12	0.35
2,000	0.020	0.052	3.021	3.051			3.36	ERR
3,000	0.023	0.057	3.024	3.056			1.12	0.69
4,000	0.025	0.062	3.026	3.060			1.68	0.69
5,000	0.028	0.066	3.029	3.064			1.13	0.86
6,000	0.031	0.071	3.032	3.069			1.13	0.69
7,000	0.034	0.077	3.035	3.075			1.13	0.57
8,000	0.038	0.084	3.039	3.082			0.85	0.49
9,000	0.042	0.089	3.043	3.087			0.85	0.68
10,000	0.047	0.097	3.048	3.095			0.68	0.42
9,000	0.047	0.097	3.048	3.095			ERR	ERR
8,000	0.046	0.097	3.047	3.095			3.42	ERR
7,000	0.043	0.097	3.044	3.095			1.14	ERR
6,000	0.041	0.096	3.042	3.094	AVERAGE	DEFORMATION	1.71	3.35
5,000	0.038	0.094	3.039	3.092	(10^6 psi)	MODULUS	1.14	1.68
4,000	0.034	0.090	3.035	3.088	1.49	0.72	0.85	0.84
3,000	0.031	0.086	3.032	3.084			1.13	0.84
2,000	0.027	0.079	3.028	3.077	AVERAGE	RECOVERY	0.85	0.49
1,000	0.023	0.067	3.024	3.065	(10^6 psi)	MODULUS	0.84	0.29
					1.51	1.06		

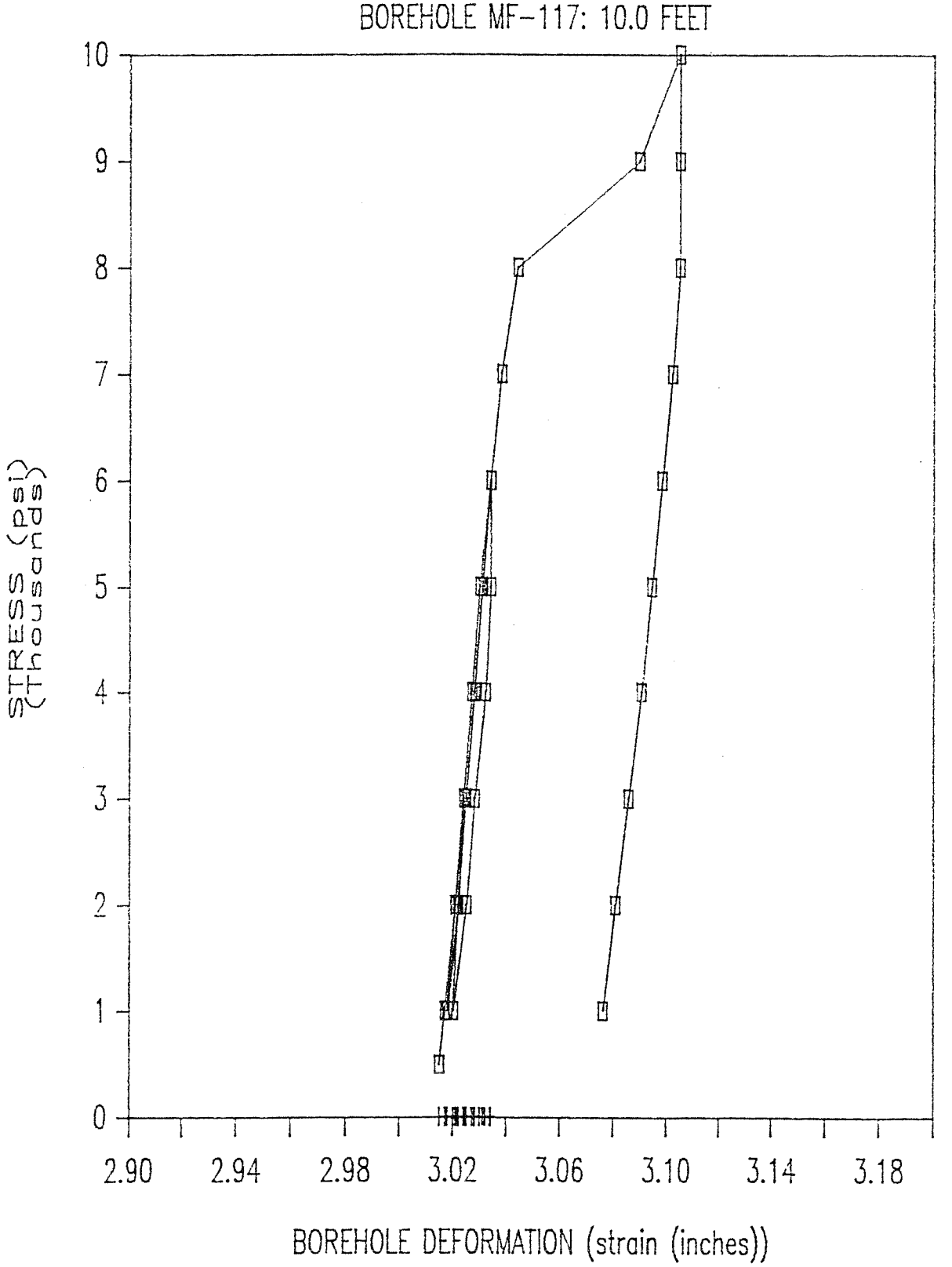
GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

MF-117 50.0 FEET			DIAMETER	DIAMETER			EXTEND	
PRESSURE	NEAR	FAR	NEAR	FAR			E(near)	E(far)
500	0.003	0.000	3.004	3.001			SET	JACK
1,000	0.005	0.002	3.006	3.003			0.83	0.88
2,000	0.008	0.005	3.009	3.005			1.11	1.17
3,000	0.011	0.010	3.012	3.010			1.11	0.70
2,000	0.010	0.010	3.011	3.010			3.33	ERR
1,000	0.007	0.006	3.008	3.006			1.11	0.88
2,000	0.009	0.008	3.010	3.008			1.66	1.76
3,000	0.012	0.011	3.013	3.011			1.11	1.17
4,000	0.016	0.016	3.017	3.016			0.84	0.70
5,000	0.021	0.021	3.022	3.021			0.67	0.70
6,000	0.026	0.026	3.027	3.026			0.67	0.70
5,000	0.025	0.026	3.026	3.026			3.37	ERR
4,000	0.023	0.024	3.024	3.024			1.68	1.76
3,000	0.020	0.021	3.021	3.021			1.12	1.17
2,000	0.016	0.017	3.017	3.017			0.84	0.88
1,000	0.012	0.012	3.013	3.012			0.84	0.70
2,000	0.015	0.014	3.016	3.014			1.11	1.76
3,000	0.018	0.018	3.019	3.018			1.12	0.88
4,000	0.021	0.021	3.022	3.021			1.12	1.17
5,000	0.024	0.024	3.025	3.024			1.12	1.17
6,000	0.027	0.027	3.028	3.027			1.12	1.17
7,000	0.031	0.030	3.032	3.029			0.85	1.17
8,000	0.035	0.034	3.036	3.033			0.85	0.83
9,000	0.040	0.039	3.041	3.038			0.68	0.70
10,000	0.045	0.043	3.046	3.042			0.68	0.87
9,000	0.045	0.043	3.046	3.042			ERR	ERR
8,000	0.043	0.043	3.044	3.042			1.71	ERR
7,000	0.040	0.041	3.041	3.040			1.14	1.74
6,000	0.037	0.038	3.038	3.037	AVERAGE	DEFORMATION	1.14	1.16
5,000	0.034	0.036	3.035	3.035	(10^6 psi)	MODULUS	1.13	1.75
4,000	0.031	0.032	3.032	3.031	0.99	1.02	1.13	0.88
3,000	0.027	0.029	3.028	3.028			0.85	1.17
2,000	0.022	0.024	3.023	3.024	AVERAGE	RECOVERY	0.67	0.70
1,000	0.018	0.019	3.019	3.019	(10^6 psi)	MODULUS	0.84	0.70
					1.39	1.12		

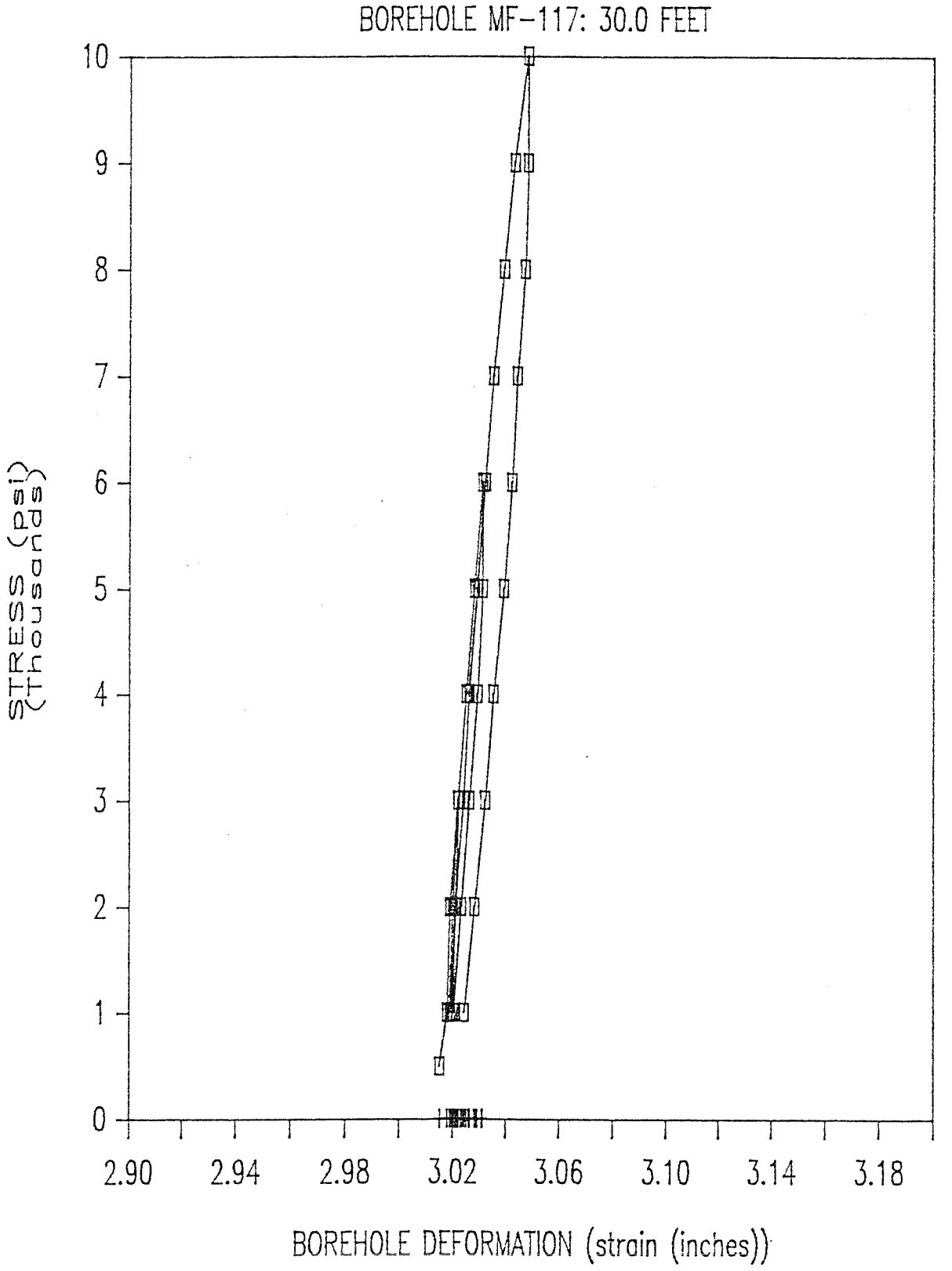
GOODMAN JACK: DEFORMATION MODULI: MINER FLAT DAM SITE

MF-117 PRESSURE	90.2 FT NEAR	FAR	DIAMETER NEAR	DIAMETER FAR			EXTEND E(near)	E(far)
1,000	0.000	0.003	3.001	3.004			0.16	0.14
2,000	0.003	0.009	3.004	3.009			1.10	0.59
3,000	0.006	0.014	3.007	3.014			1.11	0.70
4,000	0.010	0.022	3.011	3.022			0.83	0.44
5,000	0.014	0.027	3.015	3.027			0.83	0.70
6,000	0.017	0.031	3.018	3.030			1.12	0.88
7,000	0.020	0.036	3.021	3.035			1.12	0.70
8,000	0.026	0.039	3.027	3.038			0.56	1.17
9,000	0.027	0.043	3.028	3.042	AVERAGE	DEFORMATION	3.38	0.87
10,000	0.030	0.046	3.031	3.045	(10^6 psi)	MODULUS	1.13	1.16
10,000	0.030	0.046	3.031	3.045	1.13	0.74	RETRACT	
9,000	0.030	0.046	3.031	3.045			ERR	ERR
8,000	0.030	0.046	3.031	3.045			ERR	ERR
7,000	0.028	0.046	3.029	3.045			1.69	ERR
6,000	0.027	0.045	3.028	3.044			3.38	ERR
5,000	0.025	0.043	3.026	3.042			1.69	3.48
4,000	0.022	0.041	3.023	3.040			1.12	1.74
3,000	0.020	0.038	3.021	3.037			1.68	1.74
2,000	0.017	0.035	3.018	3.034	AVERAGE	RECOVERY	1.12	1.16
1,000	0.012	0.030	3.013	3.029	(10^6 psi)	MODULUS	0.67	1.17
					1.62	1.86		

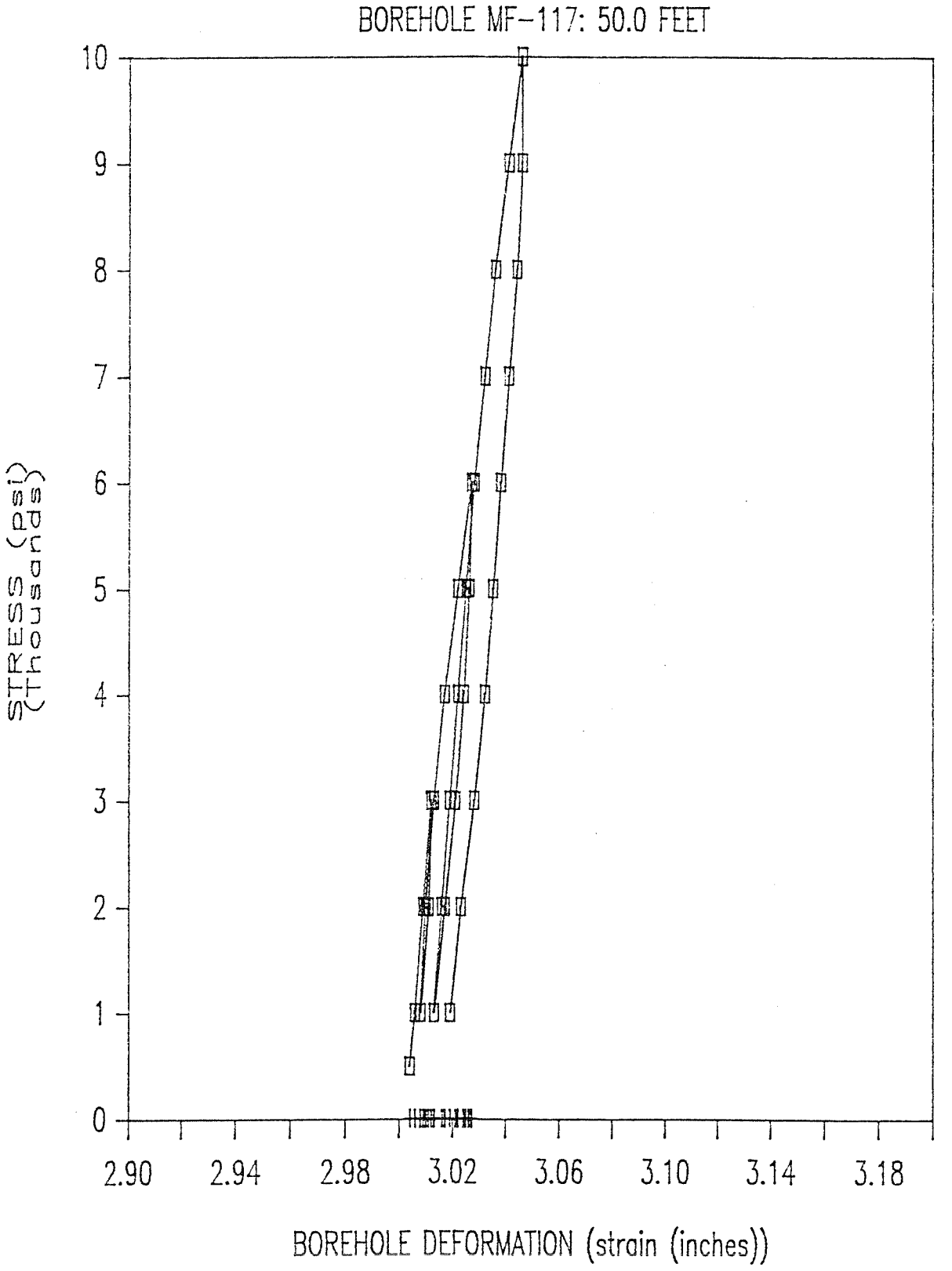
DEFORMATION MODULAE: MINER FLAT DAM



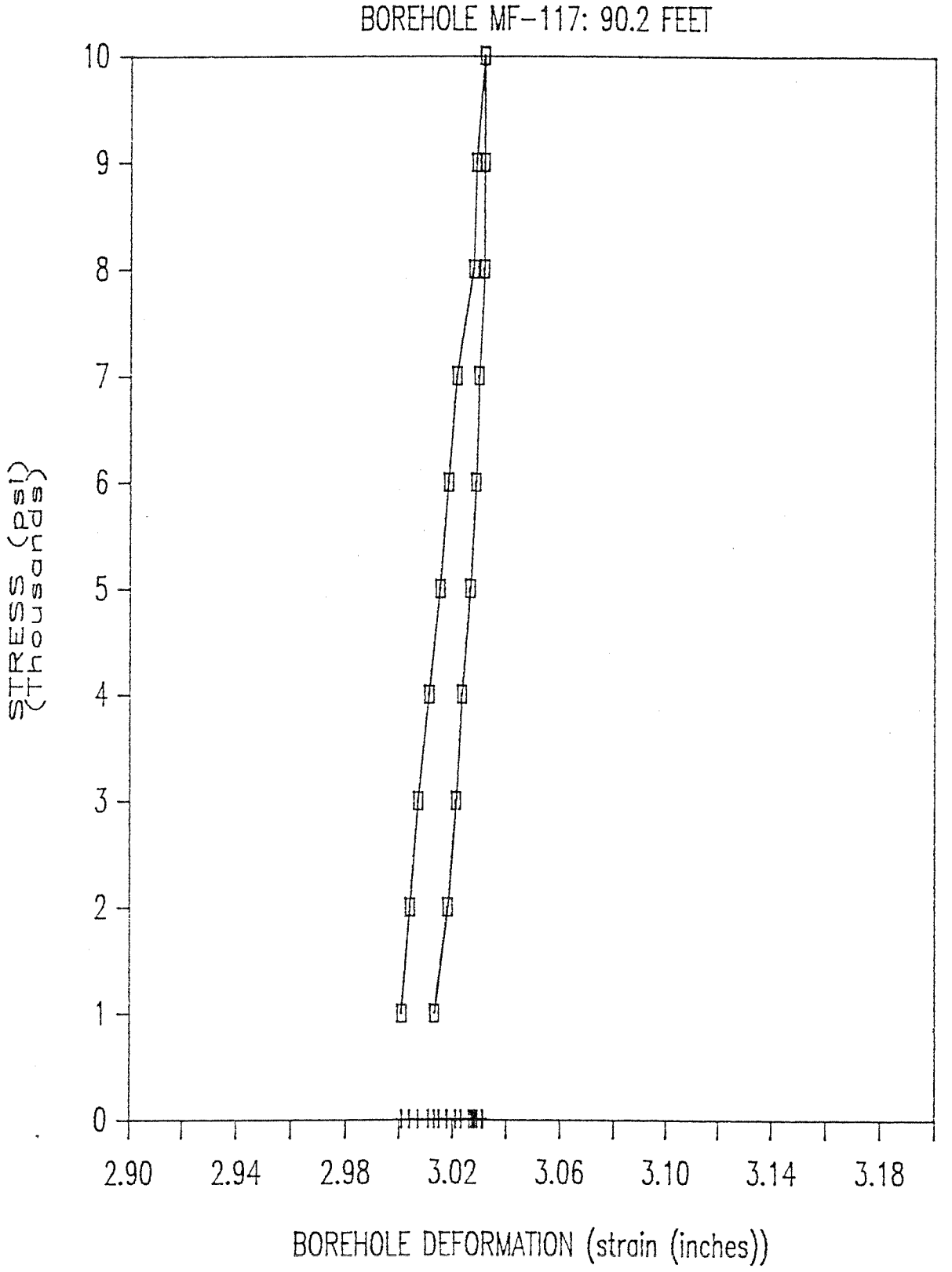
DEFORMATION MODULAE: MINER FLAT DAM



DEFORMATION MODULAE: MINER FLAT DAM



DEFORMATION MODULAE: MINER FLAT DAM



APPENDIX F

Basic Data Drill Hole Surveys

BOREHOLE COORDINATE DATA

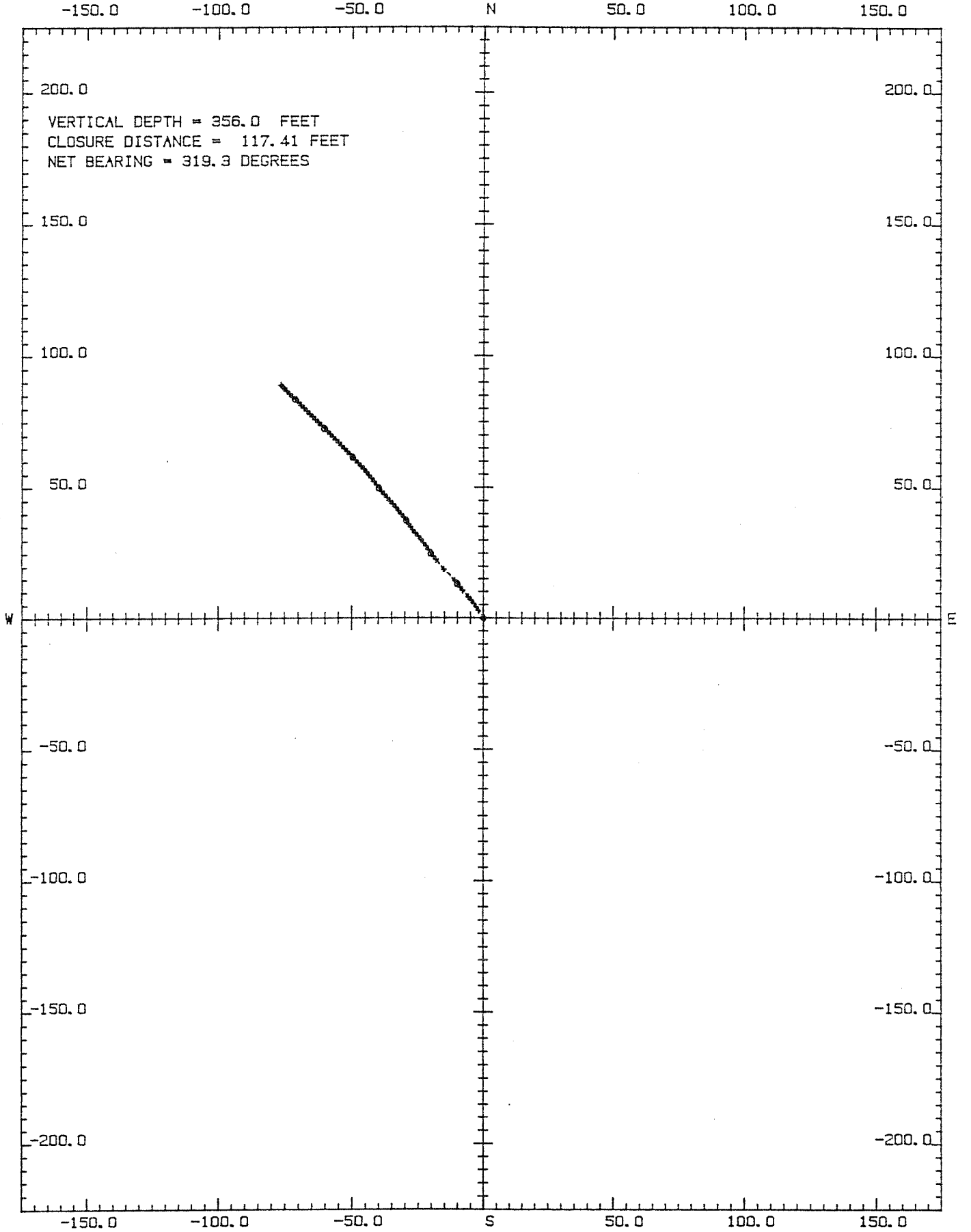
MINER FLAT DAM SITE
BOREHOLE: MF-102

NORTHING: 1,085,798.00 LOGGED BY: CHR FILE: MFS102
EASTING: 576,853.00 DATE: 1/29/86 PAGE: 1
ELEVATION: 6,078.40

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
0.0	20.0	320	1,085,798.00	576,853.00	6,078.40
10.0	19.5	328	1,085,800.73	576,851.02	6,068.99
15.0	19.4	323	1,085,802.10	576,850.07	6,064.27
20.0	19.4	322	1,085,803.41	576,849.06	6,059.56
25.0	19.4	323	1,085,804.73	576,848.05	6,054.84
30.0	19.4	319	1,085,806.02	576,847.01	6,050.13
40.0	19.4	319	1,085,808.53	576,844.83	6,040.69
45.0	19.2	320	1,085,809.78	576,843.76	6,035.97
50.0	18.5	325	1,085,811.06	576,842.77	6,031.24
55.0	18.4	317	1,085,812.29	576,841.78	6,026.50
75.0	18.4	318	1,085,816.95	576,837.51	6,007.52
90.0	18.2	323	1,085,820.58	576,834.52	5,993.28
95.0	18.4	323	1,085,821.83	576,833.58	5,988.53
100.0	18.5	319	1,085,823.06	576,832.58	5,983.79
105.0	18.3	327	1,085,824.32	576,831.63	5,979.05
110.0	18.7	323	1,085,825.61	576,830.72	5,974.30
115.0	18.3	326	1,085,826.90	576,829.80	5,969.56
120.0	19.2	318	1,085,828.17	576,828.81	5,964.83
125.0	19.1	321	1,085,829.41	576,827.75	5,960.10
130.0	19.0	318	1,085,830.65	576,826.69	5,955.38
135.0	19.0	321	1,085,831.89	576,825.63	5,950.65
140.0	10.0	327	1,085,832.89	576,824.88	5,945.83
145.0	19.0	326	1,085,833.93	576,824.19	5,941.00
150.0	19.0	324	1,085,835.26	576,823.26	5,936.27
155.0	19.0	322	1,085,836.56	576,822.28	5,931.54
160.0	19.1	321	1,085,837.84	576,821.26	5,926.82
165.0	19.0	321	1,085,839.11	576,820.23	5,922.09
170.0	19.0	319	1,085,840.35	576,819.19	5,917.36
175.0	19.0	319	1,085,841.58	576,818.12	5,912.64
180.0	19.0	319	1,085,842.81	576,817.05	5,907.91
185.0	19.0	319	1,085,844.04	576,815.98	5,903.18
190.0	19.0	319	1,085,845.27	576,814.92	5,898.45
195.0	18.9	319	1,085,846.49	576,813.85	5,893.73
200.0	18.0	324	1,085,847.73	576,812.87	5,888.98
205.0	18.0	324	1,085,848.98	576,811.96	5,884.23
210.0	18.1	321	1,085,850.21	576,811.01	5,879.47
215.0	18.0	322	1,085,851.42	576,810.05	5,874.72
220.0	17.9	323	1,085,852.64	576,809.11	5,869.96
225.0	17.9	320	1,085,853.84	576,808.16	5,865.20
230.0	18.0	318	1,085,855.01	576,807.14	5,860.45

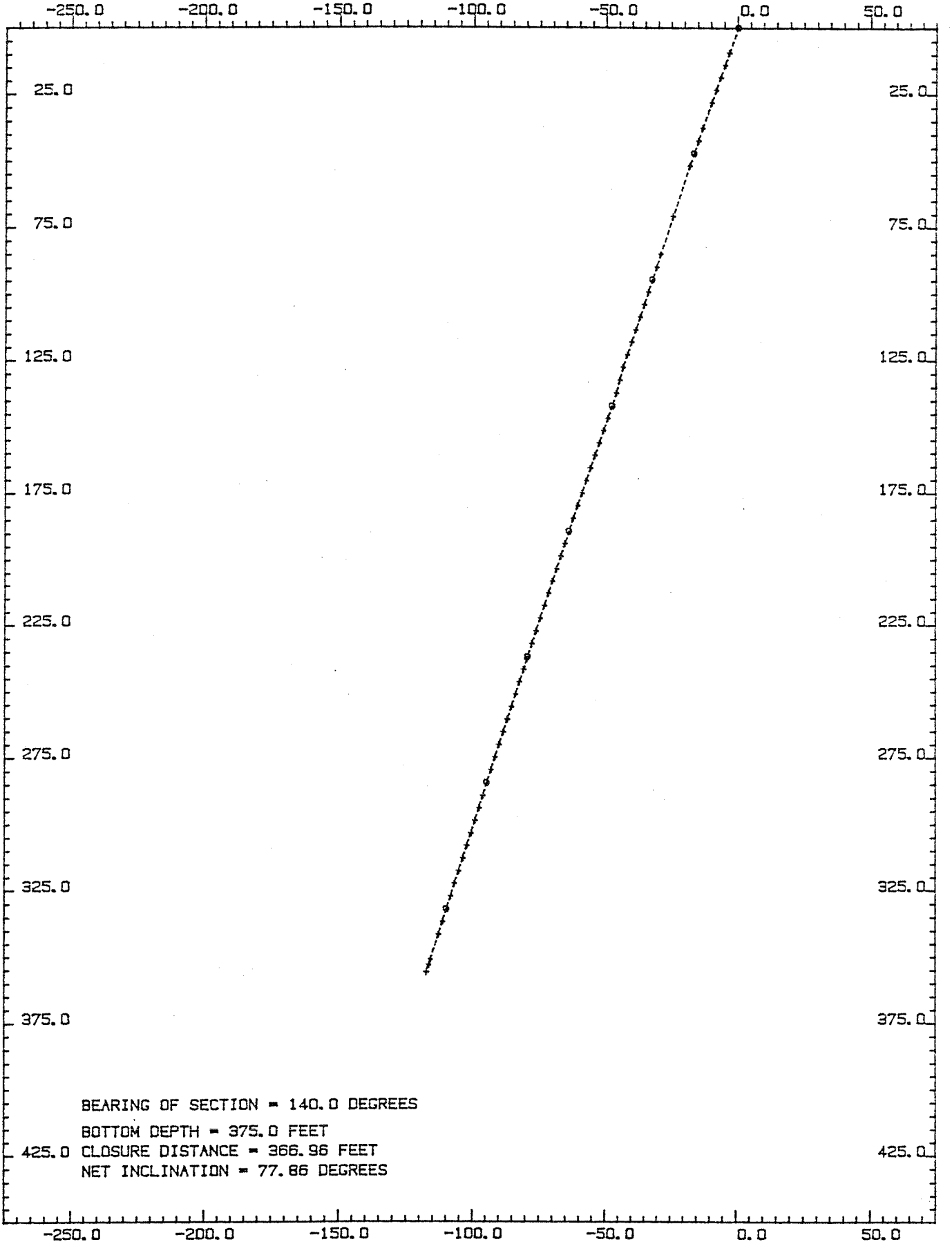
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-102

COLLAR NORTHING: 1,085,798.00 COLLAR ELEVATION: 6,078.4 NORTH SCALE: 1 in = 50.0 ft
COLLAR EASTING: 576,859.00 LOGGED BY: CHR DATE: 1/29/86 EAST SCALE: 1 in = 50.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-102

COLLAR NORTHING: 1,085,798.00 COLLAR ELEVATION: 6,078.4 HORIZ. SCALE: 1 in = 50.0 ft
COLLAR EASTING: 576,853.00 LOGGED BY: CHR DATE: 1/29/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

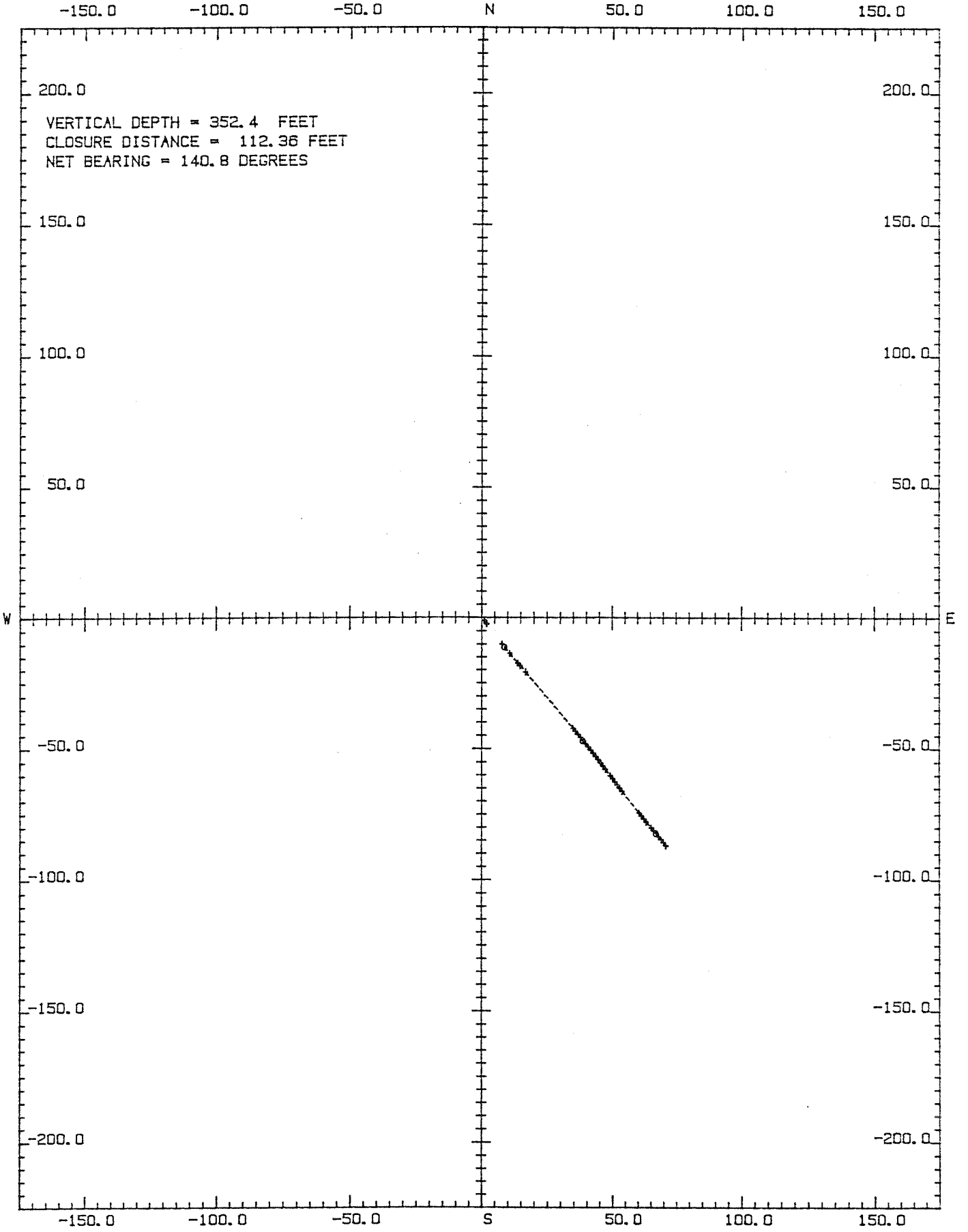
MINER FLAT DAM SITE
BOREHOLE: MF-105

NORTHING: 1,086,058.00 LOGGED BY: CHR FILE: MFS105
EASTING: 576,636.00 DATE: 1/8/86 PAGE: 1
ELEVATION: 6,073.40

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
10.0	18.5	142	1,086,056.75	576,636.98	6,063.66
15.0	18.8	144	1,086,055.47	576,637.94	6,058.92
45.0	18.4	141	1,086,047.88	576,643.76	6,030.49
50.0	18.4	139	1,086,046.67	576,644.77	6,025.74
60.0	18.3	141	1,086,044.26	576,646.80	6,016.25
75.0	18.2	140	1,086,040.64	576,649.79	6,002.01
80.0	18.2	140	1,086,039.44	576,650.79	5,997.26
90.0	18.2	141	1,086,037.03	576,652.78	5,987.76
180.0	18.0	139	1,086,015.61	576,670.74	5,902.21
185.0	18.0	141	1,086,014.43	576,671.74	5,897.46
190.0	17.9	141	1,086,013.23	576,672.71	5,892.70
194.9	17.9	141	1,086,012.06	576,673.65	5,888.04
200.0	17.9	140	1,086,010.85	576,674.65	5,883.18
205.0	18.0	140	1,086,009.67	576,675.64	5,878.43
210.0	17.9	140	1,086,008.49	576,676.63	5,873.67
215.0	17.9	141	1,086,007.31	576,677.61	5,868.91
220.0	17.9	141	1,086,006.11	576,678.58	5,864.15
225.0	17.9	142	1,086,004.91	576,679.53	5,859.40
230.0	17.9	142	1,086,003.70	576,680.48	5,854.64
235.0	17.9	143	1,086,002.48	576,681.42	5,849.88
240.0	17.8	142	1,086,001.26	576,682.35	5,845.12
245.0	17.8	142	1,086,000.06	576,683.29	5,840.36
255.0	17.7	142	1,085,997.66	576,685.17	5,830.84
260.0	17.7	142	1,085,996.46	576,686.10	5,826.07
265.0	17.7	143	1,085,995.25	576,687.03	5,821.31
270.0	17.6	143	1,085,994.04	576,687.94	5,816.54
275.0	17.6	143	1,085,992.83	576,688.85	5,811.78
280.0	17.5	143	1,085,991.63	576,689.76	5,807.01
315.0	17.5	139	1,085,983.46	576,696.38	5,773.63
320.0	17.4	140	1,085,982.32	576,697.35	5,768.86
325.0	17.4	140	1,085,981.17	576,698.31	5,764.09
330.0	17.3	139	1,085,980.04	576,699.28	5,759.32
340.0	17.3	139	1,085,977.79	576,701.23	5,749.77
345.0	17.4	140	1,085,976.66	576,702.20	5,745.00
350.0	17.1	139	1,085,975.53	576,703.16	5,740.22
355.0	17.2	139	1,085,974.42	576,704.13	5,735.44
360.0	17.5	142	1,085,973.27	576,705.08	5,730.67
365.0	17.5	141	1,085,972.09	576,706.01	5,725.90
370.1	17.5	142	1,085,970.89	576,706.97	5,721.04

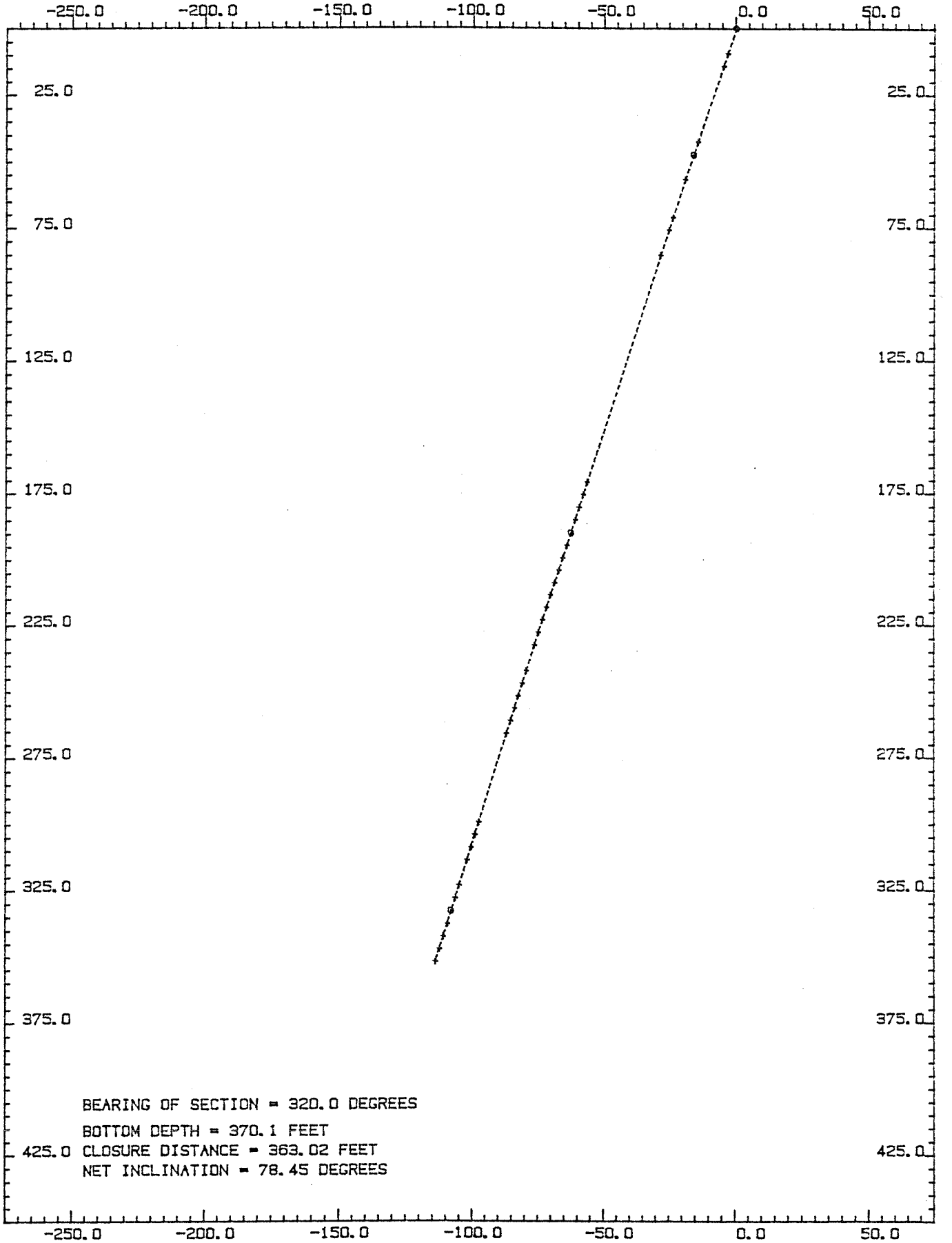
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-105

COLLAR NORTHING: 1,086,058.00 COLLAR ELEVATION: 6,073.4 NORTH SCALE: 1 in = 50.0 ft
COLLAR EASTING: 576,636.00 LOGGED BY: CHR DATE: 1/8/86 EAST SCALE: 1 in = 50.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-105

COLLAR NORTHING: 1,086,058.00 COLLAR ELEVATION: 6,073.4 HORIZ. SCALE: 1 in = 50.0 ft
COLLAR EASTING: 576,636.00 LOGGED BY: CHR DATE: 1/8/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

MINER FLAT DAM SITE
BOREHOLE: MF-106

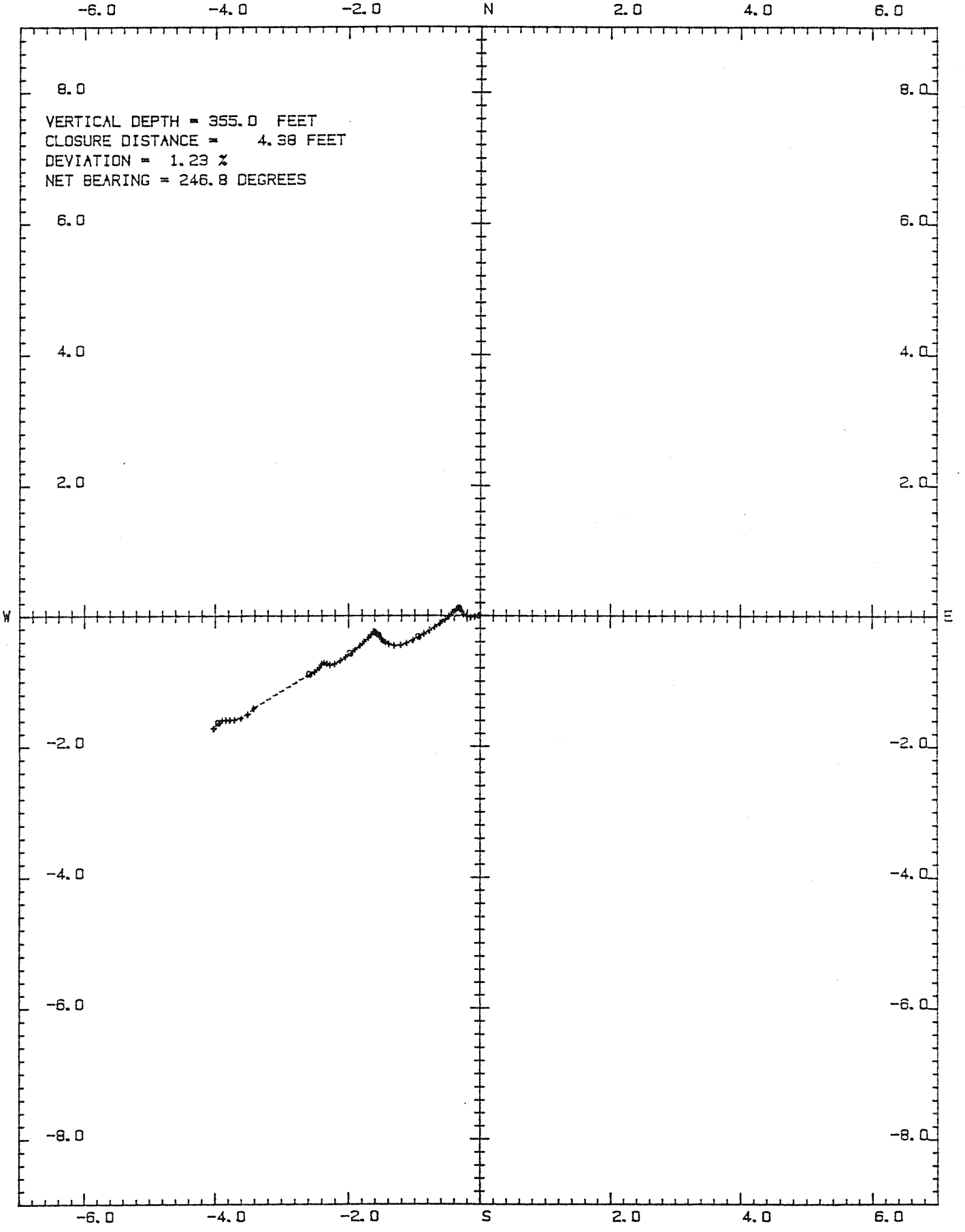
NORTHING: 1,086,059.63 LOGGED BY: CHR FILE: MFS106
EASTING: 576,634.61 DATE: 11/25/85 PAGE: 1
ELEVATION: 6,073.40

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
5.0	.1	324	1,086,059.63	576,634.61	6,068.40
10.0	.1	234	1,086,059.63	576,634.60	6,063.40
14.9	.5	260	1,086,059.63	576,634.58	6,058.50
20.0	.7	253	1,086,059.62	576,634.53	6,053.40
25.0	.8	271	1,086,059.61	576,634.46	6,048.40
30.0	.7	289	1,086,059.62	576,634.40	6,043.40
35.0	.6	322	1,086,059.65	576,634.35	6,038.40
40.0	.5	324	1,086,059.69	576,634.32	6,033.40
45.0	.4	327	1,086,059.72	576,634.30	6,028.40
50.0	.3	325	1,086,059.74	576,634.28	6,023.40
54.9	.1	234	1,086,059.75	576,634.27	6,018.50
60.0	.9	233	1,086,059.73	576,634.24	6,013.40
65.0	.9	225	1,086,059.67	576,634.18	6,008.40
70.1	.9	227	1,086,059.62	576,634.12	6,003.30
75.0	1.0	232	1,086,059.57	576,634.06	5,998.40
80.0	1.1	233	1,086,059.51	576,633.99	5,993.41
85.0	1.1	236	1,086,059.45	576,633.91	5,988.41
90.0	1.1	237	1,086,059.40	576,633.83	5,983.41
95.0	1.1	240	1,086,059.35	576,633.75	5,978.41
100.0	1.1	241	1,086,059.30	576,633.66	5,973.41
105.0	1.2	239	1,086,059.25	576,633.58	5,968.41
110.0	1.2	250	1,086,059.21	576,633.48	5,963.41
114.8	1.1	249	1,086,059.18	576,633.39	5,958.61
120.0	1.1	281	1,086,059.17	576,633.30	5,953.41
125.0	.9	297	1,086,059.19	576,633.21	5,948.41
130.0	.5	304	1,086,059.22	576,633.16	5,943.41
135.0	.4	297	1,086,059.24	576,633.13	5,938.41
140.0	.5	325	1,086,059.27	576,633.10	5,933.41
144.9	.4	329	1,086,059.30	576,633.08	5,928.51
150.0	.4	319	1,086,059.33	576,633.06	5,923.41
154.9	.3	317	1,086,059.35	576,633.04	5,918.51
160.0	.2	304	1,086,059.37	576,633.02	5,913.41
165.0	.3	299	1,086,059.38	576,633.00	5,908.41
169.9	.3	229	1,086,059.38	576,632.98	5,903.51
174.9	.8	220	1,086,059.34	576,632.95	5,898.52
179.5	.9	226	1,086,059.29	576,632.90	5,893.92
184.9	1.0	226	1,086,059.23	576,632.84	5,888.52
189.7	1.0	228	1,086,059.17	576,632.78	5,883.72
195.0	1.1	230	1,086,059.11	576,632.70	5,878.42
200.0	1.0	231	1,086,059.05	576,632.63	5,873.42

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
205.0	1.1	234	1,086,058.99	576,632.56	5,868.42
209.8	1.1	235	1,086,058.94	576,632.48	5,863.62
214.9	1.2	240	1,086,058.89	576,632.40	5,858.52
220.0	.6	297	1,086,058.87	576,632.33	5,853.42
225.0	.5	294	1,086,058.89	576,632.29	5,848.42
230.0	.5	293	1,086,058.91	576,632.25	5,843.42
235.0	.8	214	1,086,058.89	576,632.21	5,838.42
240.0	.8	214	1,086,058.83	576,632.17	5,833.42
244.8	1.4	231	1,086,058.77	576,632.10	5,828.62
250.0	.6	276	1,086,058.73	576,632.03	5,823.43
315.0	1.3	222	1,086,058.22	576,631.19	5,758.44
320.1	1.5	229	1,086,058.13	576,631.10	5,753.34
325.0	1.3	249	1,086,058.07	576,631.00	5,748.44
330.0	1.0	266	1,086,058.05	576,630.91	5,743.44
335.0	.7	269	1,086,058.04	576,630.83	5,738.44
340.0	.7	270	1,086,058.04	576,630.77	5,733.44
345.0	.6	269	1,086,058.04	576,630.72	5,728.44
350.0	1.3	216	1,086,057.99	576,630.66	5,723.44
354.5	1.4	222	1,086,057.91	576,630.59	5,718.94
355.0	.7	247	1,086,057.91	576,630.58	5,718.44

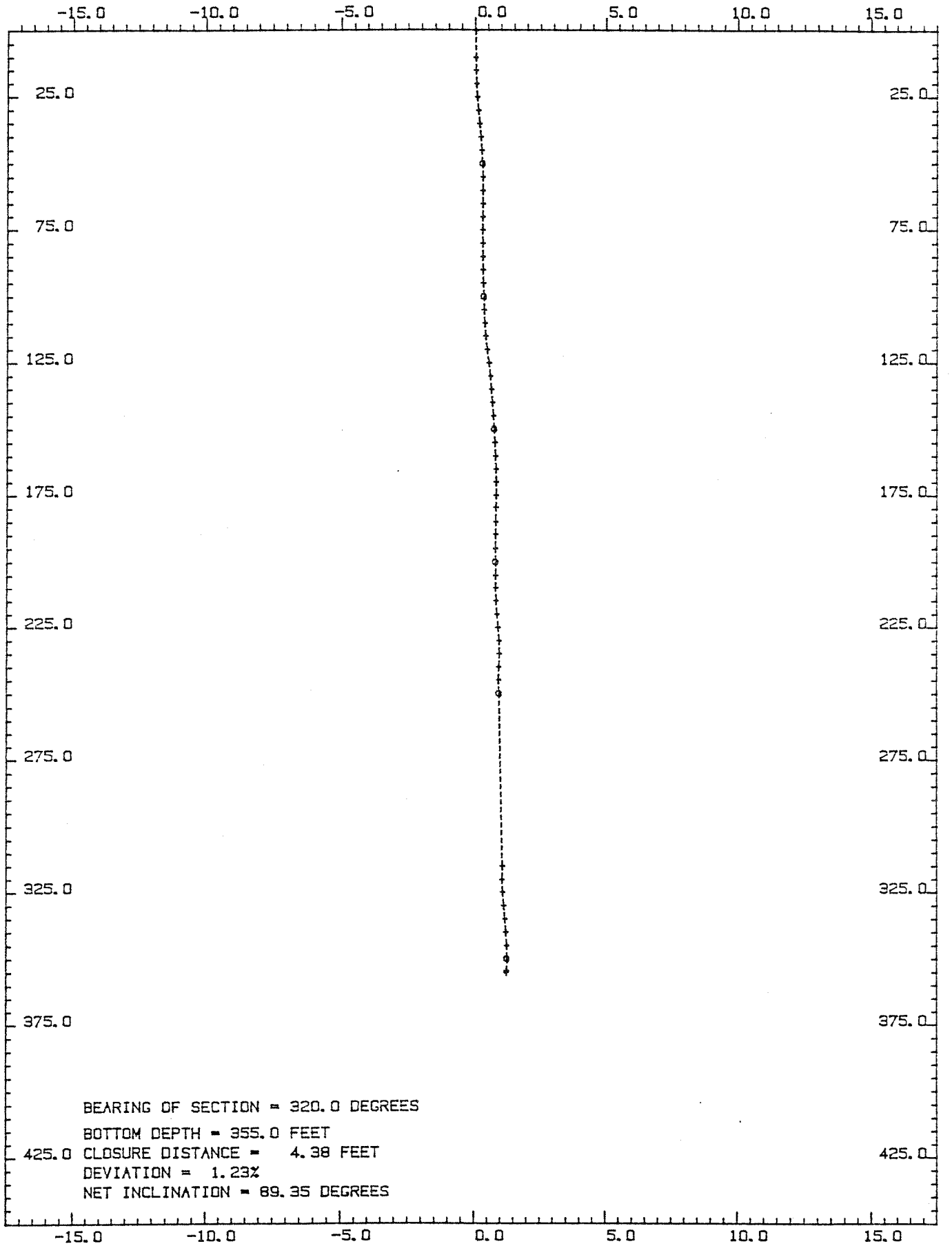
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-106

COLLAR NORTHING: 1,086,059.63 COLLAR ELEVATION: 6,073.4 NORTH SCALE: 1 in = 2.0 ft
COLLAR EASTING: 576,634.61 LOGGED BY: CHR DATE: 11/25/85 EAST SCALE: 1 in = 2.0 ft



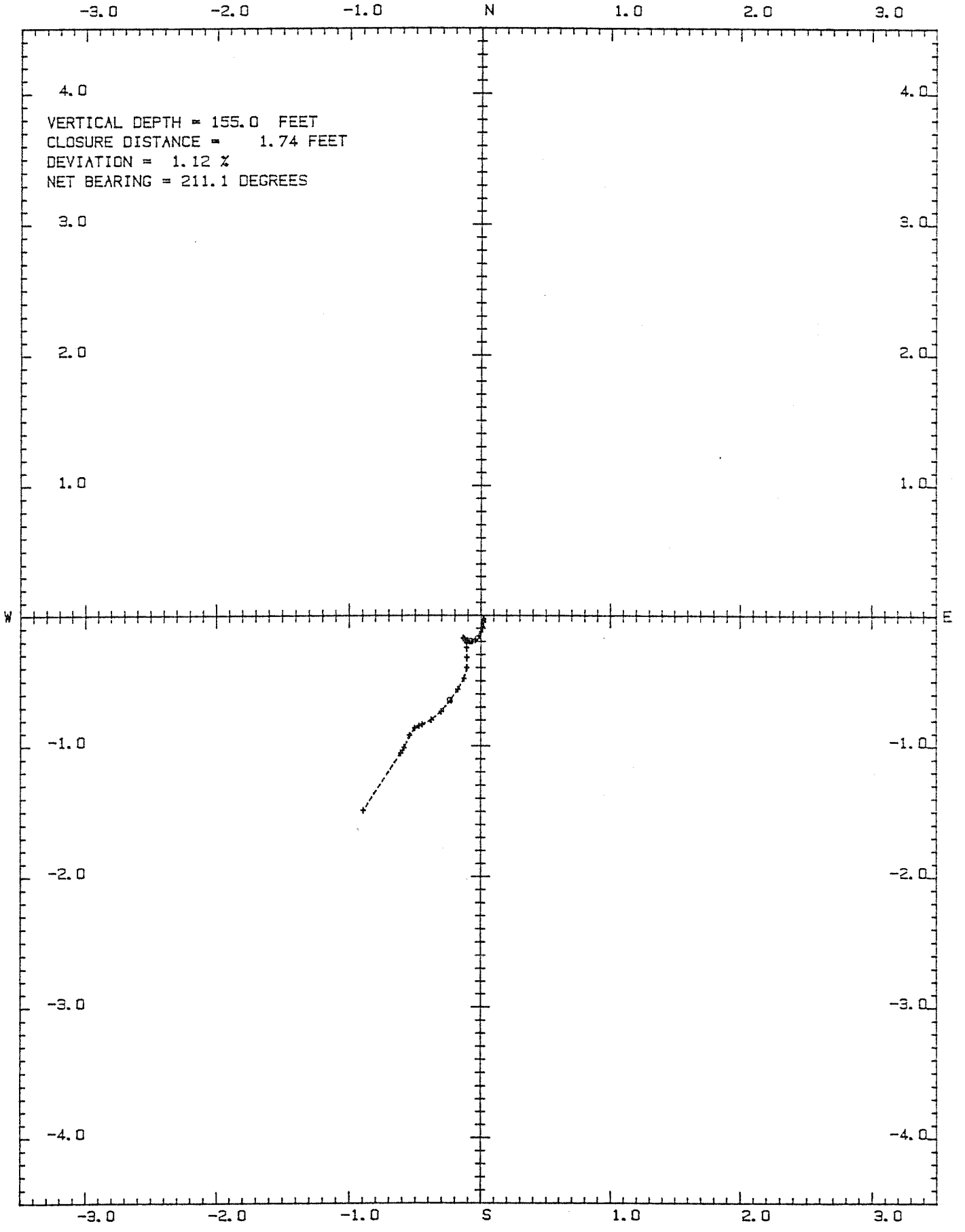
MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-106

COLLAR NORTHING: 1,086,059.63 COLLAR ELEVATION: 6,073.4 HORIZ. SCALE: 1 in = 5.0 ft
COLLAR EASTING: 576,634.61 LOGGED BY: CHR DATE: 11/25/85 VERTICAL SCALE: 1 in = 50.0 ft



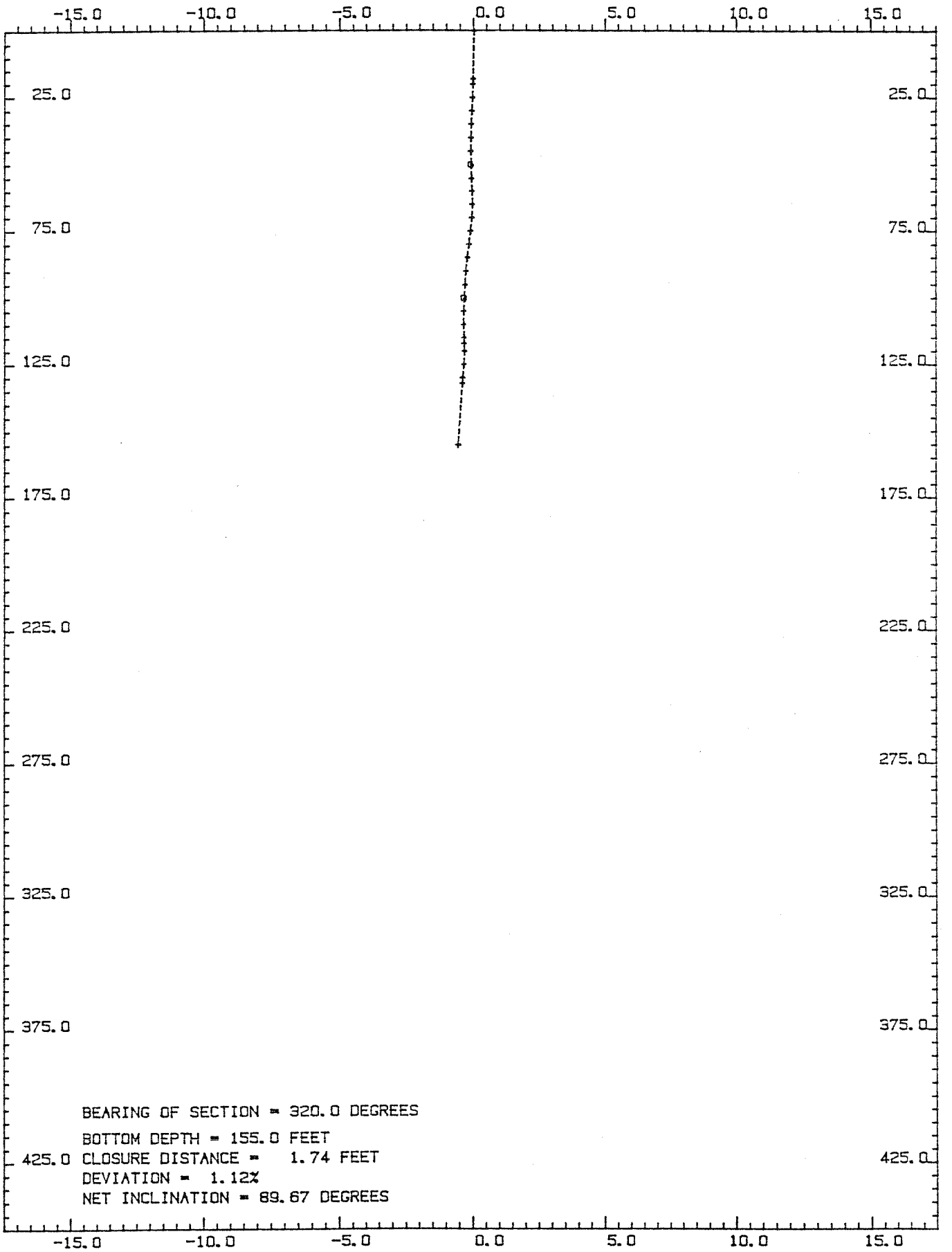
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-113

COLLAR NORTHING: 1,086,019.00 COLLAR ELEVATION: 5,920.5 NORTH SCALE: 1 in = 1.0 ft
COLLAR EASTING: 576,698.00 LOGGED BY: CHR DATE: 2/28/86 EAST SCALE: 1 in = 1.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-113

COLLAR NORTHING: 1,086,019.00 COLLAR ELEVATION: 5,920.5 HORIZ. SCALE: 1 in = 5.0 ft
COLLAR EASTING: 576,698.00 LOGGED BY: CHR DATE: 2/28/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

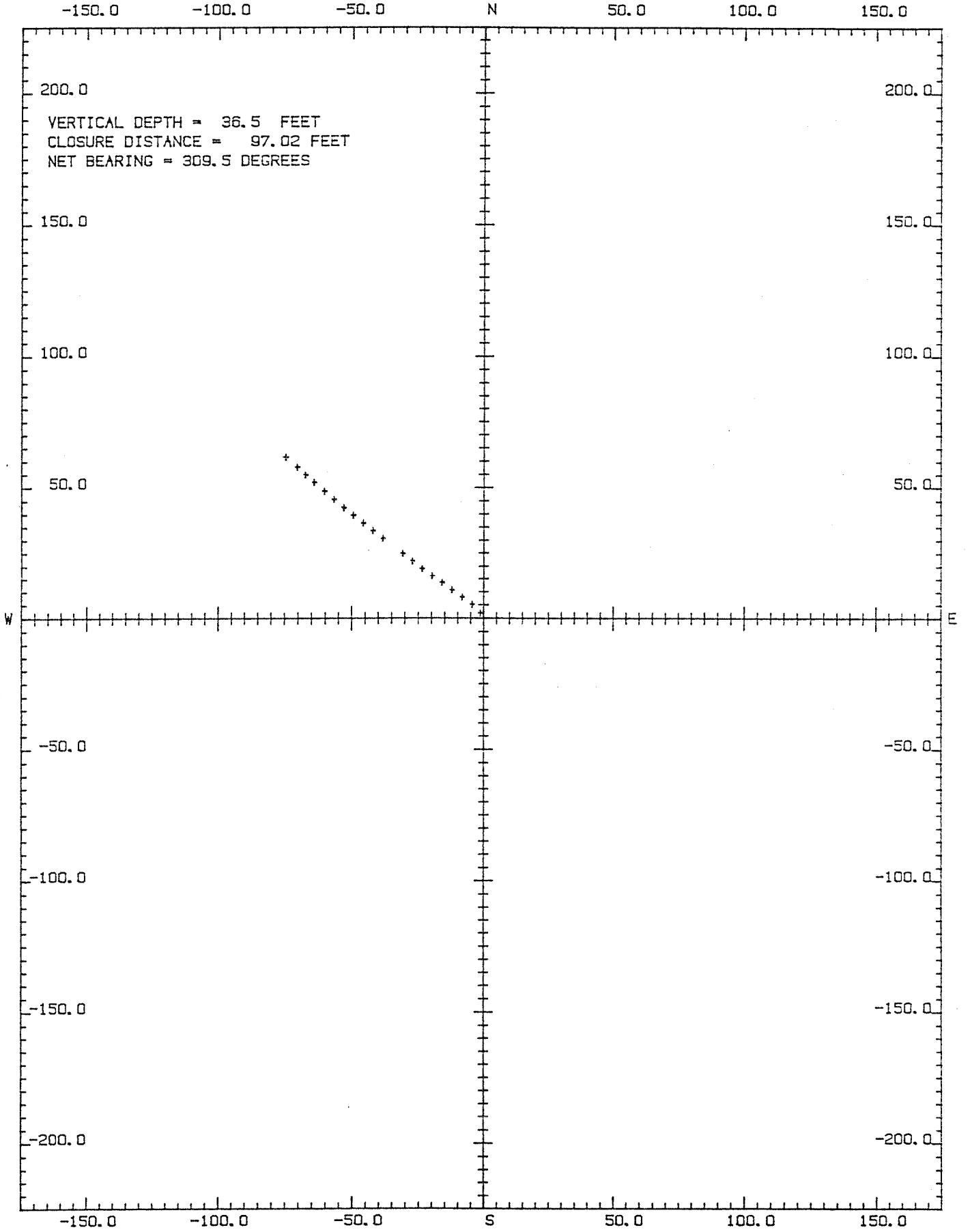
MINER FLAT DAM SITE
BOREHOLE: MF-117

NORTHING: 1,086,025.00 LOGGED BY: CHR FILE: MFS117
EASTING: 576,695.00 DATE: 3/10/86 PAGE: 1
ELEVATION: 5,924.80

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
5.0	70.2	322	1,086,026.85	576,693.55	5,921.45
10.0	71.3	307	1,086,030.13	576,690.21	5,919.80
15.0	71.3	305	1,086,032.92	576,686.38	5,918.20
20.0	71.5	306	1,086,035.67	576,682.52	5,916.61
25.0	71.2	306	1,086,038.45	576,678.69	5,915.01
30.0	71.0	306	1,086,041.23	576,674.86	5,913.39
34.9	71.0	308	1,086,044.02	576,671.16	5,911.79
40.0	70.8	308	1,086,046.99	576,667.37	5,910.12
45.0	70.7	308	1,086,049.89	576,663.65	5,908.48
55.0	70.6	308	1,086,055.70	576,656.21	5,905.16
60.0	70.5	309	1,086,058.64	576,652.52	5,903.50
65.0	70.6	309	1,086,061.60	576,648.86	5,901.83
70.0	70.6	310	1,086,064.60	576,645.22	5,900.17
74.9	70.5	309	1,086,067.54	576,641.65	5,898.54
80.0	70.5	311	1,086,070.63	576,637.97	5,896.84
85.0	70.4	311	1,086,073.72	576,634.42	5,895.16
90.4	70.2	312	1,086,077.09	576,630.61	5,893.34
94.9	70.2	312	1,086,079.93	576,627.46	5,891.82
99.4	70.5	313	1,086,082.79	576,624.34	5,890.31
105.5	70.2	312	1,086,086.67	576,620.10	5,888.26

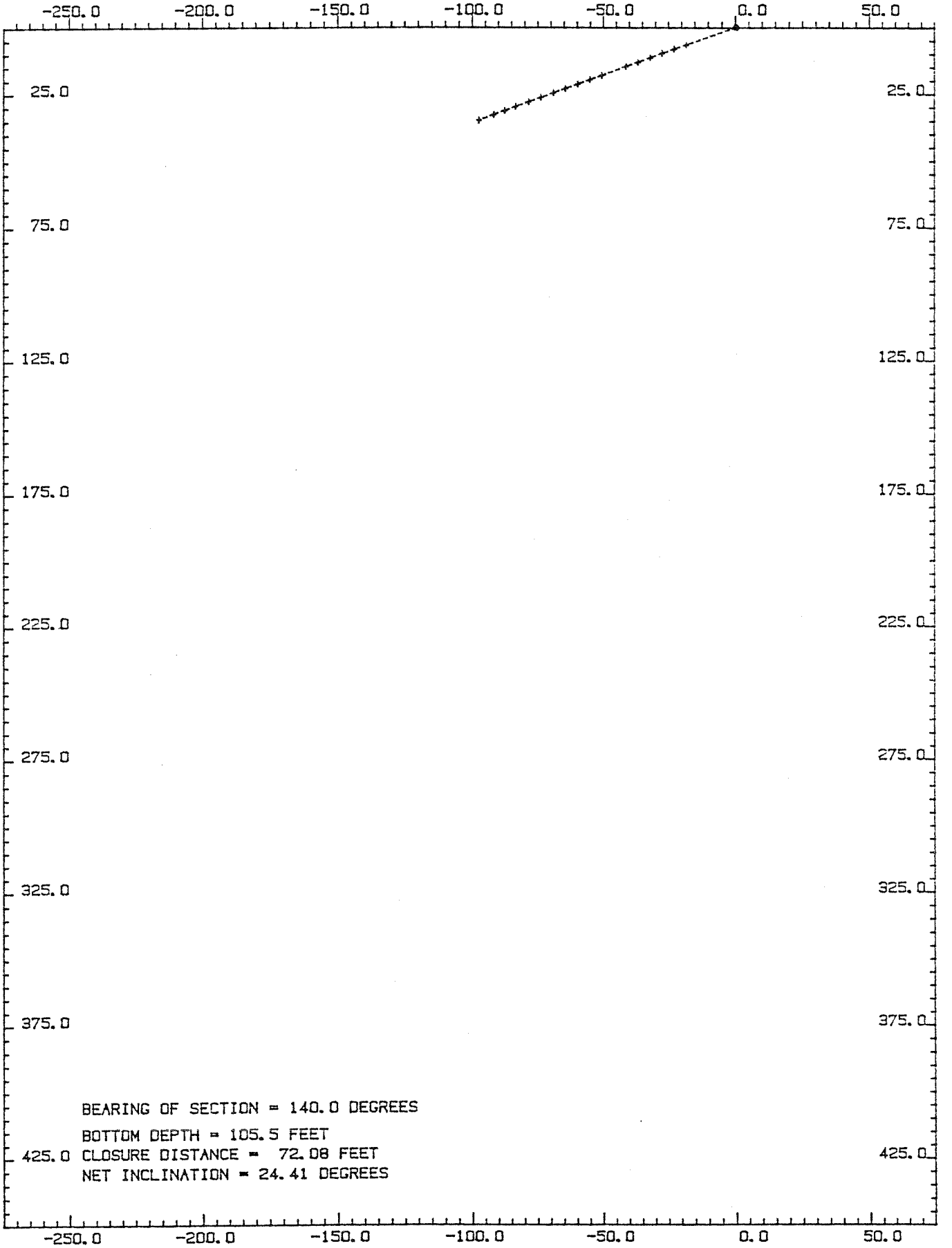
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-117

COLLAR NORTHING: 1,086,025.00 COLLAR ELEVATION: 5,924.8 NORTH SCALE: 1 in = 50.0 ft
COLLAR EASTING: 576,695.00 LOGGED BY: CHR DATE: 3/10/86 EAST SCALE: 1 in = 50.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-117

COLLAR NORTHING: 1,086,025.00 COLLAR ELEVATION: 5,924.8 HORIZ. SCALE: 1 in = 50.0 ft
COLLAR EASTING: 576,695.00 LOGGED BY: CHR DATE: 3/10/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

MINER FLAT DAM SITE
BOREHOLE: MF-118

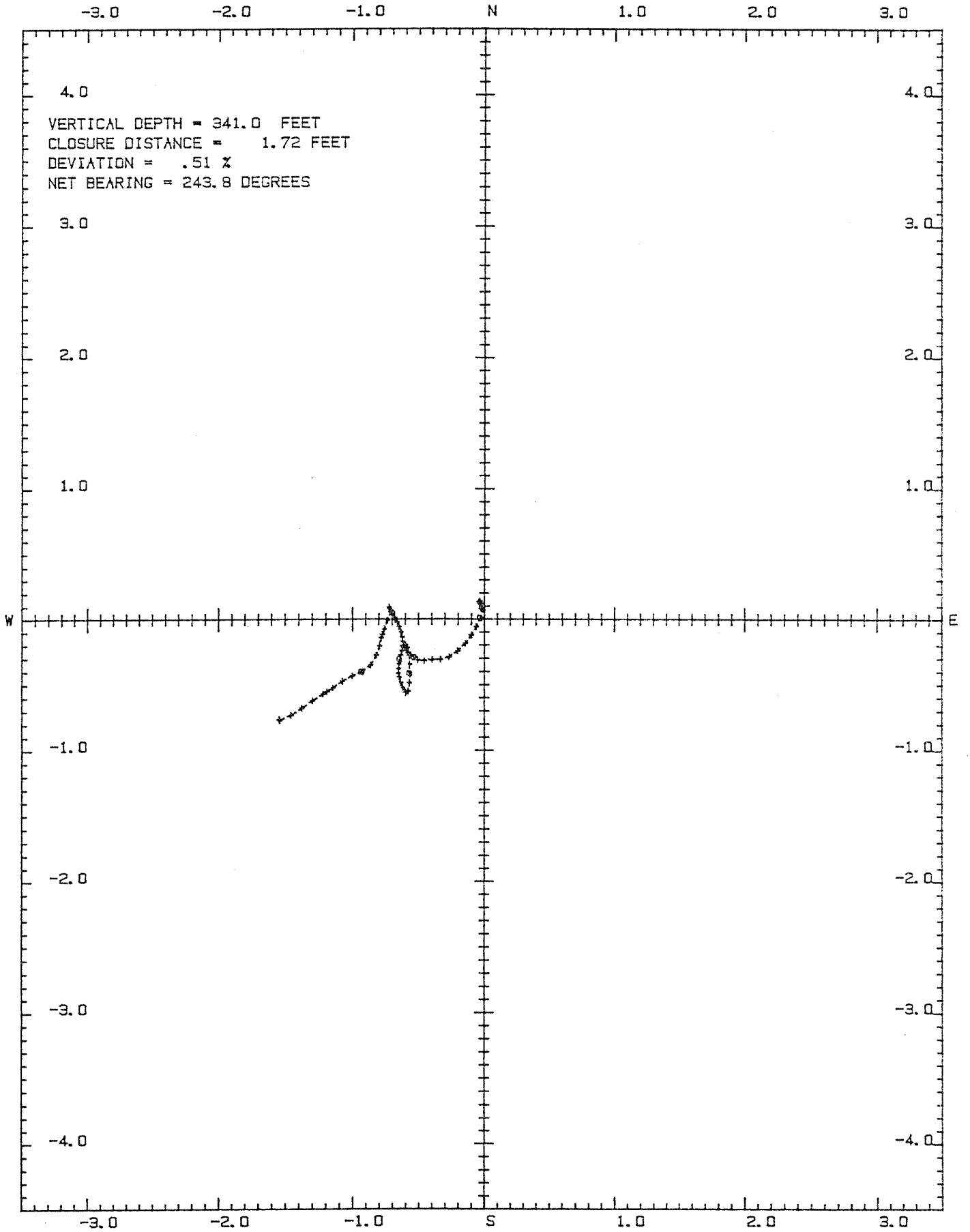
NORTHING: 1,086,321.00 LOGGED BY: CHR FILE: MFS118
EASTING: 576,417.00 DATE: 1/5/86 PAGE: 1
ELEVATION: 6,077.00

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
20.0	.5	342	1,086,321.08	576,416.97	6,057.00
25.0	.3	336	1,086,321.12	576,416.96	6,052.00
30.0	.2	1	1,086,321.14	576,416.96	6,047.00
35.0	.2	119	1,086,321.14	576,416.96	6,042.00
40.0	.4	154	1,086,321.12	576,416.98	6,037.00
45.0	.7	186	1,086,321.07	576,416.98	6,032.00
50.0	.9	203	1,086,321.01	576,416.96	6,027.00
55.0	.8	206	1,086,320.94	576,416.93	6,022.00
60.0	.9	214	1,086,320.88	576,416.90	6,017.00
65.0	.9	218	1,086,320.81	576,416.85	6,012.00
70.0	1.0	232	1,086,320.76	576,416.79	6,007.00
75.0	.9	239	1,086,320.71	576,416.72	6,002.00
80.0	.7	275	1,086,320.69	576,416.66	5,997.01
85.0	.7	262	1,086,320.69	576,416.60	5,992.01
90.0	.7	265	1,086,320.68	576,416.54	5,987.01
95.0	.5	292	1,086,320.69	576,416.49	5,982.01
100.0	.3	323	1,086,320.71	576,416.46	5,977.01
105.0	.4	299	1,086,320.73	576,416.44	5,972.01
110.0	.3	321	1,086,320.74	576,416.41	5,967.01
115.0	.3	331	1,086,320.77	576,416.40	5,962.01
120.0	.3	324	1,086,320.79	576,416.38	5,957.01
125.0	.2	85	1,086,320.80	576,416.39	5,952.01
130.0	.1	43	1,086,320.80	576,416.40	5,947.01
135.0	.6	159	1,086,320.78	576,416.41	5,942.01
140.0	.8	172	1,086,320.72	576,416.42	5,937.01
145.0	.8	182	1,086,320.65	576,416.43	5,932.01
150.0	.8	178	1,086,320.58	576,416.43	5,927.01
155.0	.8	184	1,086,320.51	576,416.43	5,922.01
160.0	.7	194	1,086,320.45	576,416.42	5,917.01
165.0	.5	328	1,086,320.44	576,416.40	5,912.01
170.0	.5	334	1,086,320.48	576,416.38	5,907.01
175.0	.5	341	1,086,320.52	576,416.36	5,902.01
180.0	.4	344	1,086,320.55	576,416.35	5,897.01
185.0	.4	357	1,086,320.59	576,416.34	5,892.01
190.0	.4	3	1,086,320.62	576,416.34	5,887.01
195.0	.4	6	1,086,320.66	576,416.34	5,882.01
200.0	.4	14	1,086,320.69	576,416.35	5,877.01
205.0	.4	13	1,086,320.73	576,416.36	5,872.01
210.0	.4	18	1,086,320.76	576,416.37	5,867.01
215.0	.4	1	1,086,320.79	576,416.37	5,862.01

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
220.0	.4	355	1,086,320.83	576,416.37	5,857.01
225.0	.4	357	1,086,320.86	576,416.37	5,852.01
230.0	.5	338	1,086,320.90	576,416.36	5,847.01
235.0	.5	337	1,086,320.94	576,416.34	5,842.01
240.0	.4	337	1,086,320.98	576,416.33	5,837.01
245.0	.5	333	1,086,321.01	576,416.31	5,832.01
250.0	.4	333	1,086,321.05	576,416.29	5,827.01
255.0	.5	320	1,086,321.08	576,416.27	5,822.01
260.0	.3	112	1,086,321.09	576,416.27	5,817.01
265.0	.7	187	1,086,321.06	576,416.28	5,812.01
270.0	.9	206	1,086,320.99	576,416.26	5,807.01
275.0	.8	196	1,086,320.92	576,416.23	5,802.01
278.0	.8	202	1,086,320.88	576,416.22	5,799.01
280.0	.7	188	1,086,320.86	576,416.21	5,797.01
285.0	.8	195	1,086,320.79	576,416.20	5,792.01
290.0	.9	201	1,086,320.72	576,416.17	5,787.01
295.0	1.1	219	1,086,320.65	576,416.13	5,782.02
300.0	.9	249	1,086,320.60	576,416.06	5,777.02
305.0	.9	243	1,086,320.57	576,415.99	5,772.02
310.0	1.0	238	1,086,320.53	576,415.92	5,767.02
315.0	1.0	235	1,086,320.48	576,415.85	5,762.02
318.0	1.0	235	1,086,320.45	576,415.80	5,759.02
320.0	1.0	238	1,086,320.43	576,415.78	5,757.02
325.0	1.1	237	1,086,320.38	576,415.70	5,752.02
330.0	1.1	234	1,086,320.32	576,415.62	5,747.02
335.0	1.1	237	1,086,320.27	576,415.54	5,742.02
340.0	1.0	255	1,086,320.23	576,415.46	5,737.02
341.0	1.1	32	1,086,320.24	576,415.45	5,736.02

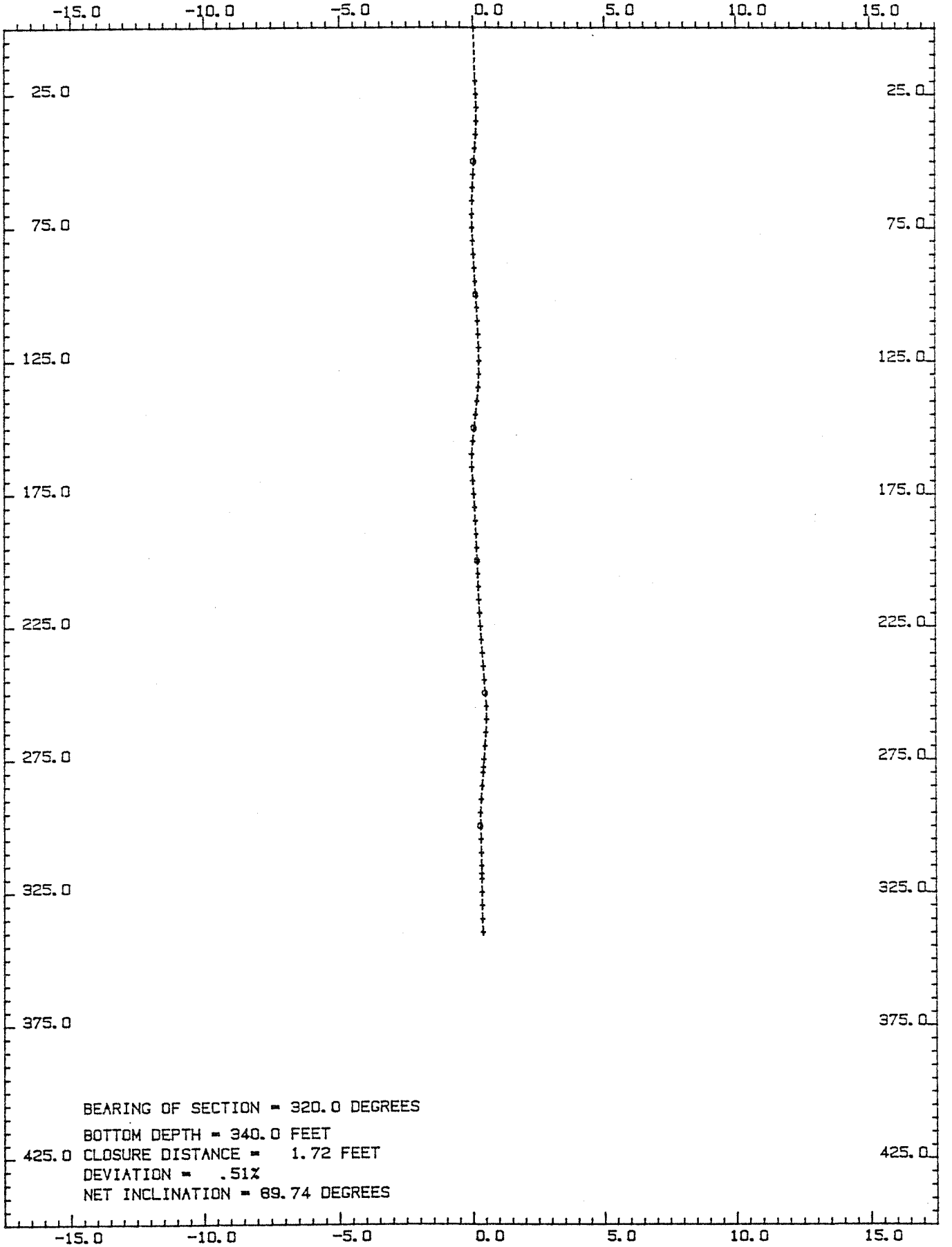
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-118

COLLAR NORTHING: 1,086,321.00 COLLAR ELEVATION: 6,077.0 NORTH SCALE: 1 in = 1.0 ft
COLLAR EASTING: 576,417.00 LOGGED BY: CHR DATE: 1/5/86 EAST SCALE: 1 in = 1.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-118

COLLAR NORTHING: 1,086,321.00 COLLAR ELEVATION: 6,077.0 HORIZ. SCALE: 1 in = 5.0 ft
COLLAR EASTING: 576,417.00 LOGGED BY: CHR DATE: 1/5/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

MINER FLAT DAM SITE
BOREHOLE: MF-119

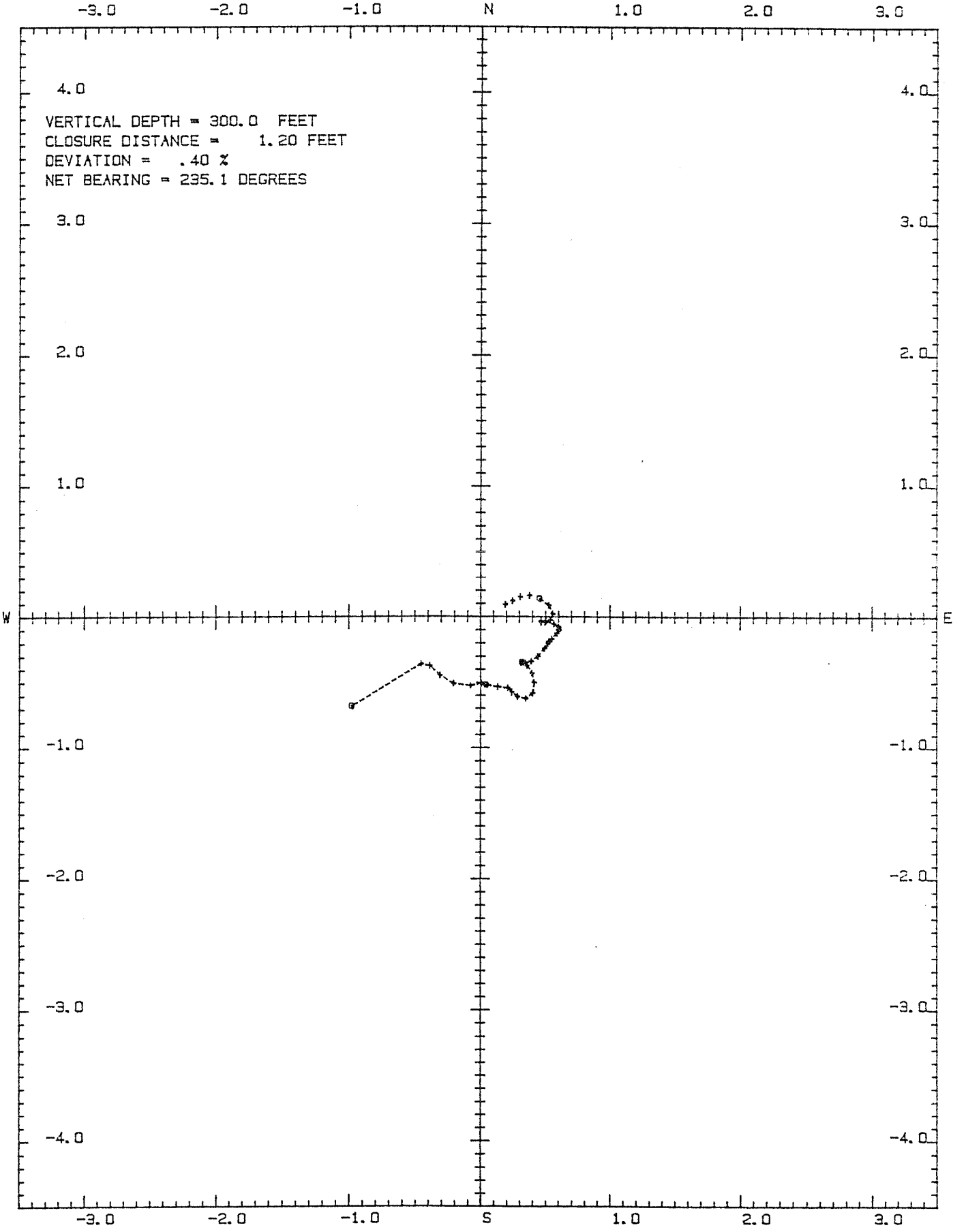
NORTHING: 1,086,816.00 LOGGED BY: CHR FILE: MFS119
EASTING: 576,078.00 DATE: 4/17/86 PAGE: 1
ELEVATION: 6,093.70

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
30.0	.8	64	1,086,816.09	576,078.19	6,063.70
35.0	.7	58	1,086,816.12	576,078.25	6,058.70
40.0	.8	67	1,086,816.15	576,078.30	6,053.70
45.0	.9	96	1,086,816.16	576,078.37	6,048.70
50.0	1.0	116	1,086,816.14	576,078.45	6,043.70
55.0	1.0	138	1,086,816.09	576,078.52	6,038.70
60.0	.8	178	1,086,816.02	576,078.55	6,033.70
65.0	.5	241	1,086,815.98	576,078.53	6,028.71
70.0	.5	256	1,086,815.96	576,078.49	6,023.71
75.0	.2	301	1,086,815.96	576,078.47	6,018.71
80.0	.2	48	1,086,815.97	576,078.46	6,013.71
85.0	.7	104	1,086,815.97	576,078.50	6,008.71
90.0	.8	116	1,086,815.94	576,078.56	6,003.71
95.0	.2	155	1,086,815.92	576,078.60	5,998.71
100.0	.1	161	1,086,815.91	576,078.60	5,993.71
105.0	.8	212	1,086,815.88	576,078.58	5,988.71
110.0	.6	237	1,086,815.83	576,078.54	5,983.71
115.0	.2	194	1,086,815.81	576,078.52	5,978.71
120.0	.1	207	1,086,815.80	576,078.52	5,973.71
125.0	.9	218	1,086,815.76	576,078.49	5,968.71
130.0	.9	221	1,086,815.70	576,078.44	5,963.71
135.0	.6	247	1,086,815.66	576,078.39	5,958.71
140.0	.5	266	1,086,815.65	576,078.34	5,953.71
145.0	.2	299	1,086,815.65	576,078.31	5,948.71
150.0	.3	98	1,086,815.65	576,078.32	5,943.71
155.0	.8	137	1,086,815.63	576,078.36	5,938.71
160.0	.8	156	1,086,815.57	576,078.39	5,933.71
165.0	.9	176	1,086,815.50	576,078.41	5,928.71
170.0	1.0	199	1,086,815.42	576,078.40	5,923.71
175.0	.9	269	1,086,815.38	576,078.35	5,918.71
180.0	.7	302	1,086,815.39	576,078.28	5,913.71
185.0	.6	315	1,086,815.43	576,078.24	5,908.71
190.0	.4	316	1,086,815.46	576,078.21	5,903.71
195.0	1.5	266	1,086,815.47	576,078.13	5,898.71
200.0	.7	310	1,086,815.48	576,078.04	5,893.71
205.0	.4	243	1,086,815.49	576,078.00	5,888.72
210.0	1.6	262	1,086,815.47	576,077.92	5,883.72
215.0	1.5	293	1,086,815.49	576,077.79	5,878.72
220.0	1.3	311	1,086,815.55	576,077.68	5,873.72
225.0	1.1	317	1,086,815.63	576,077.61	5,868.72

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
230.0	.9	239	1,086,815.64	576,077.54	5,863.72
300.0	.1	232	1,086,815.32	576,077.02	5,793.73

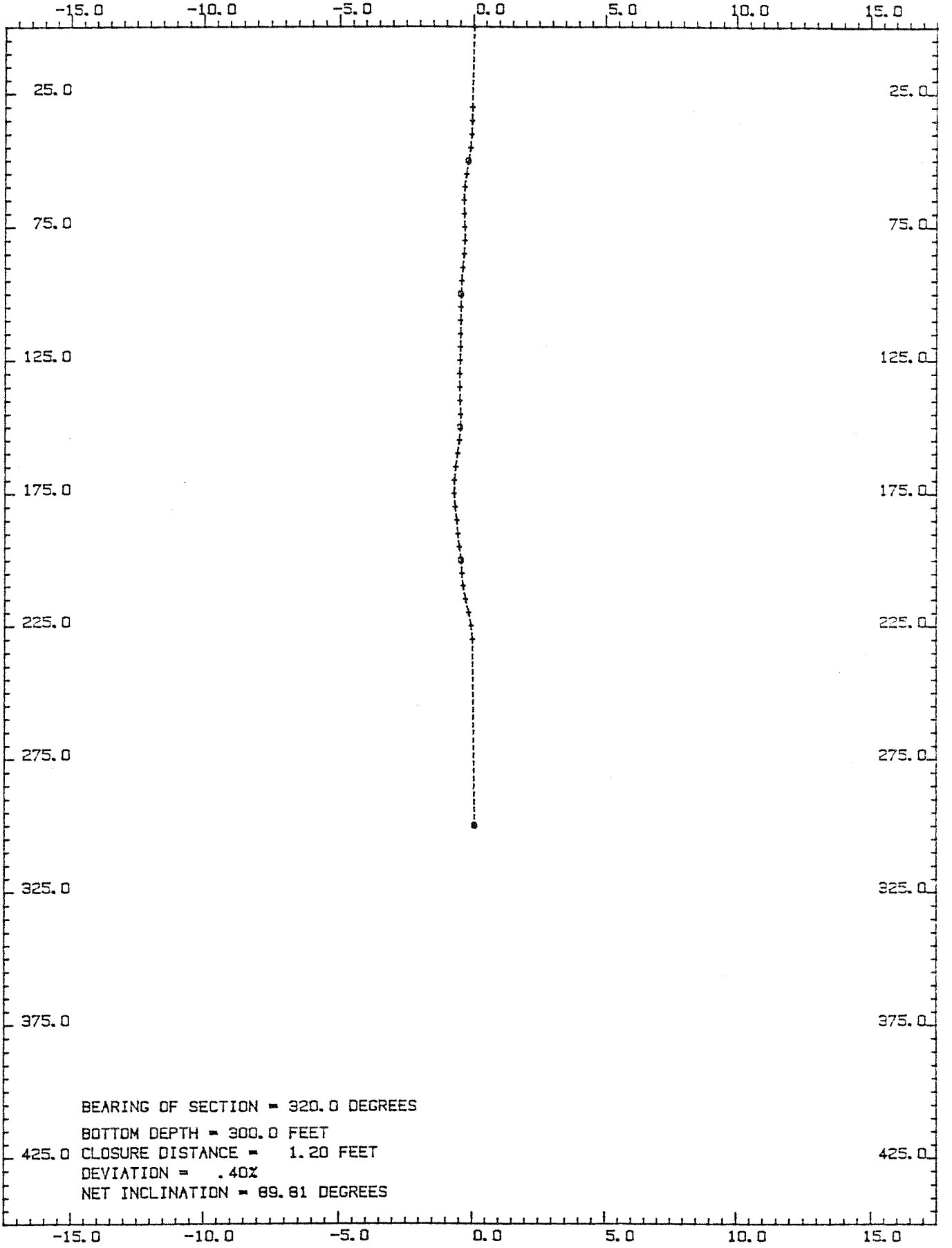
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-119

COLLAR NORTHING: 1,086,816.00 COLLAR ELEVATION: 6,093.7 NORTH SCALE: 1 in = 1.0 ft
COLLAR EASTING: 576,078.00 LOGGED BY: CHR DATE: 4/17/86 EAST SCALE: 1 in = 1.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-119

COLLAR NORTHING: 1,086,816.00 COLLAR ELEVATION: 6,093.7 HORIZ. SCALE: 1 in = 5.0 ft
COLLAR EASTING: 576,078.00 LOGGED BY: CHR DATE: 4/17/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

MINER FLAT DAM SITE
BOREHOLE: MF-120A

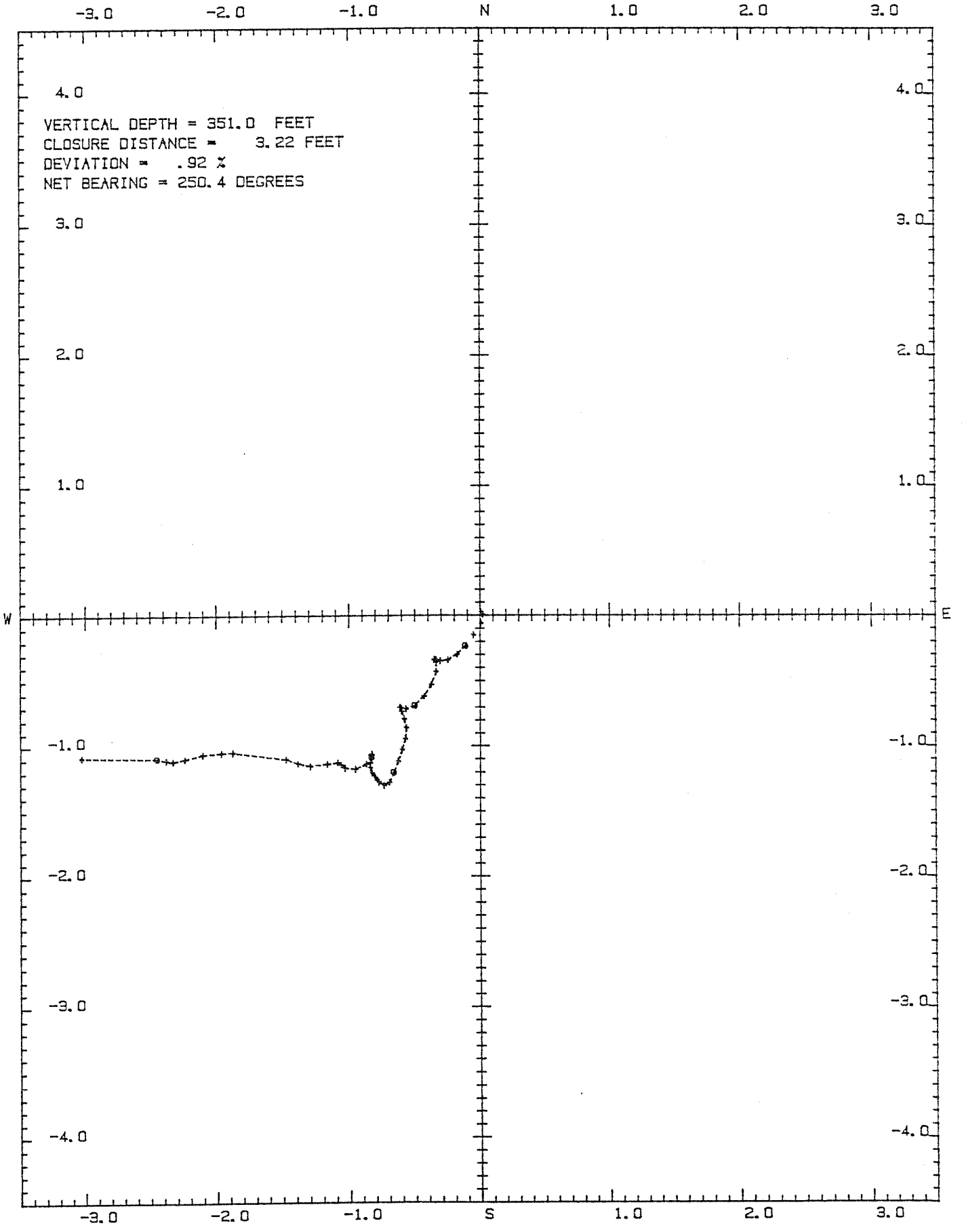
NORTHING: 1,086,568.00 LOGGED BY: CHR FILE: MFS120A
EASTING: 576,247.00 DATE: 4/21/86 PAGE: 1
ELEVATION: 6,082.00

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
30.0	.1	15	1,086,568.03	576,247.01	6,052.00
35.0	.6	151	1,086,568.01	576,247.02	6,047.00
40.0	1.2	211	1,086,567.94	576,247.01	6,042.00
45.0	1.3	216	1,086,567.85	576,246.95	6,037.00
50.0	1.1	220	1,086,567.77	576,246.88	6,032.00
55.0	1.0	230	1,086,567.70	576,246.82	6,027.00
60.0	.9	249	1,086,567.66	576,246.75	6,022.00
65.0	.5	285	1,086,567.65	576,246.69	6,017.01
70.0	.3	283	1,086,567.66	576,246.66	6,012.01
75.0	0.0	283	1,086,567.66	576,246.64	6,007.01
80.0	.6	148	1,086,567.64	576,246.66	6,002.01
85.0	1.2	195	1,086,567.57	576,246.66	5,997.01
90.0	1.2	207	1,086,567.47	576,246.62	5,992.01
95.0	1.2	216	1,086,567.38	576,246.57	5,987.01
100.0	1.1	235	1,086,567.31	576,246.50	5,982.01
105.0	.6	275	1,086,567.29	576,246.43	5,977.01
110.0	.3	301	1,086,567.29	576,246.39	5,972.01
115.0	.1	122	1,086,567.30	576,246.39	5,967.01
120.0	.7	163	1,086,567.27	576,246.40	5,962.01
125.0	.7	160	1,086,567.21	576,246.42	5,957.01
130.0	.9	173	1,086,567.14	576,246.43	5,952.01
135.0	1.1	196	1,086,567.06	576,246.42	5,947.01
140.0	1.0	196	1,086,566.97	576,246.40	5,942.01
145.0	1.1	201	1,086,566.88	576,246.37	5,937.01
150.0	1.0	204	1,086,566.80	576,246.33	5,932.01
155.0	.9	204	1,086,566.72	576,246.30	5,927.02
160.0	.6	293	1,086,566.70	576,246.26	5,922.02
165.0	.5	321	1,086,566.72	576,246.22	5,917.02
170.0	.5	323	1,086,566.76	576,246.20	5,912.02
175.0	.5	329	1,086,566.79	576,246.17	5,907.02
180.0	.5	350	1,086,566.83	576,246.16	5,902.02
185.0	.4	20	1,086,566.87	576,246.16	5,897.02
190.0	.4	9	1,086,566.91	576,246.17	5,892.02
195.0	.3	6	1,086,566.94	576,246.17	5,887.02
200.0	.7	191	1,086,566.92	576,246.17	5,882.02
205.0	1.0	230	1,086,566.86	576,246.13	5,877.02
210.0	1.2	260	1,086,566.82	576,246.04	5,872.02
215.0	.7	306	1,086,566.83	576,245.97	5,867.02
220.0	.2	308	1,086,566.86	576,245.94	5,862.02
225.0	.5	306	1,086,566.87	576,245.91	5,857.02

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
230.0	1.5	249	1,086,566.86	576,245.83	5,852.02
235.0	1.5	275	1,086,566.85	576,245.71	5,847.02
240.0	.7	306	1,086,566.87	576,245.62	5,842.02
245.0	1.5	283	1,086,566.90	576,245.53	5,837.02
270.0	.4	255	1,086,566.95	576,245.12	5,812.03
275.1	1.6	271	1,086,566.95	576,245.04	5,806.93
280.0	1.7	259	1,086,566.94	576,244.90	5,802.03
285.0	1.6	253	1,086,566.90	576,244.76	5,797.03
290.0	.5	279	1,086,566.88	576,244.67	5,792.04
295.0	.7	279	1,086,566.89	576,244.62	5,787.04
300.0	.9	284	1,086,566.91	576,244.55	5,782.04
351.0	.5	246	1,086,566.92	576,243.97	5,731.04

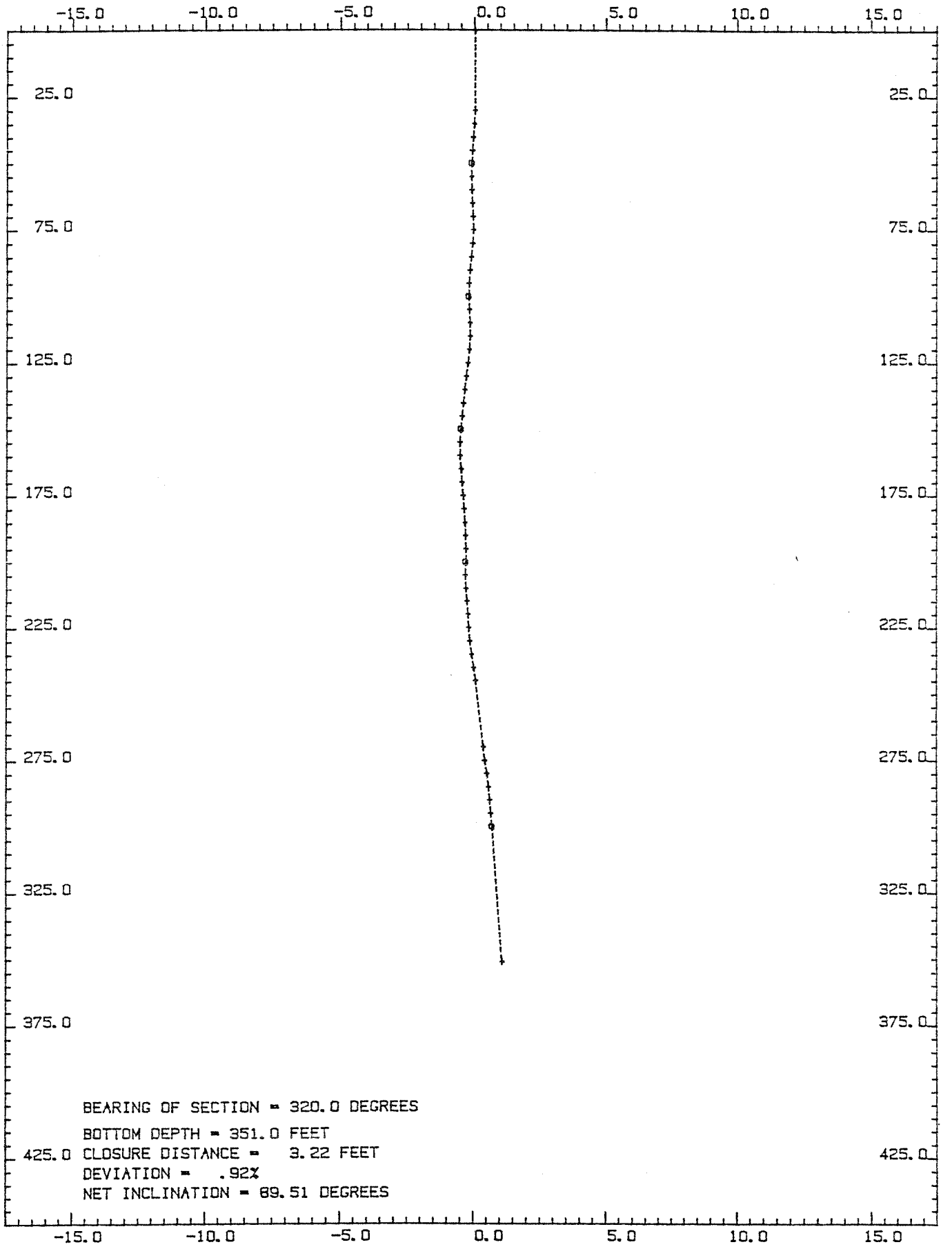
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-120A

COLLAR NORTHING: 1,086,568.00 COLLAR ELEVATION: 6,082.0 NORTH SCALE: 1 in = 1.0 ft
COLLAR EASTING: 576,247.00 LOGGED BY: CHR DATE: 4/21/86 EAST SCALE: 1 in = 1.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-120A

COLLAR NORTHING: 1,086,568.00 COLLAR ELEVATION: 6,082.0 HORIZ. SCALE: 1 in = 5.0 ft
COLLAR EASTING: 576,247.00 LOGGED BY: CHR DATE: 4/21/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

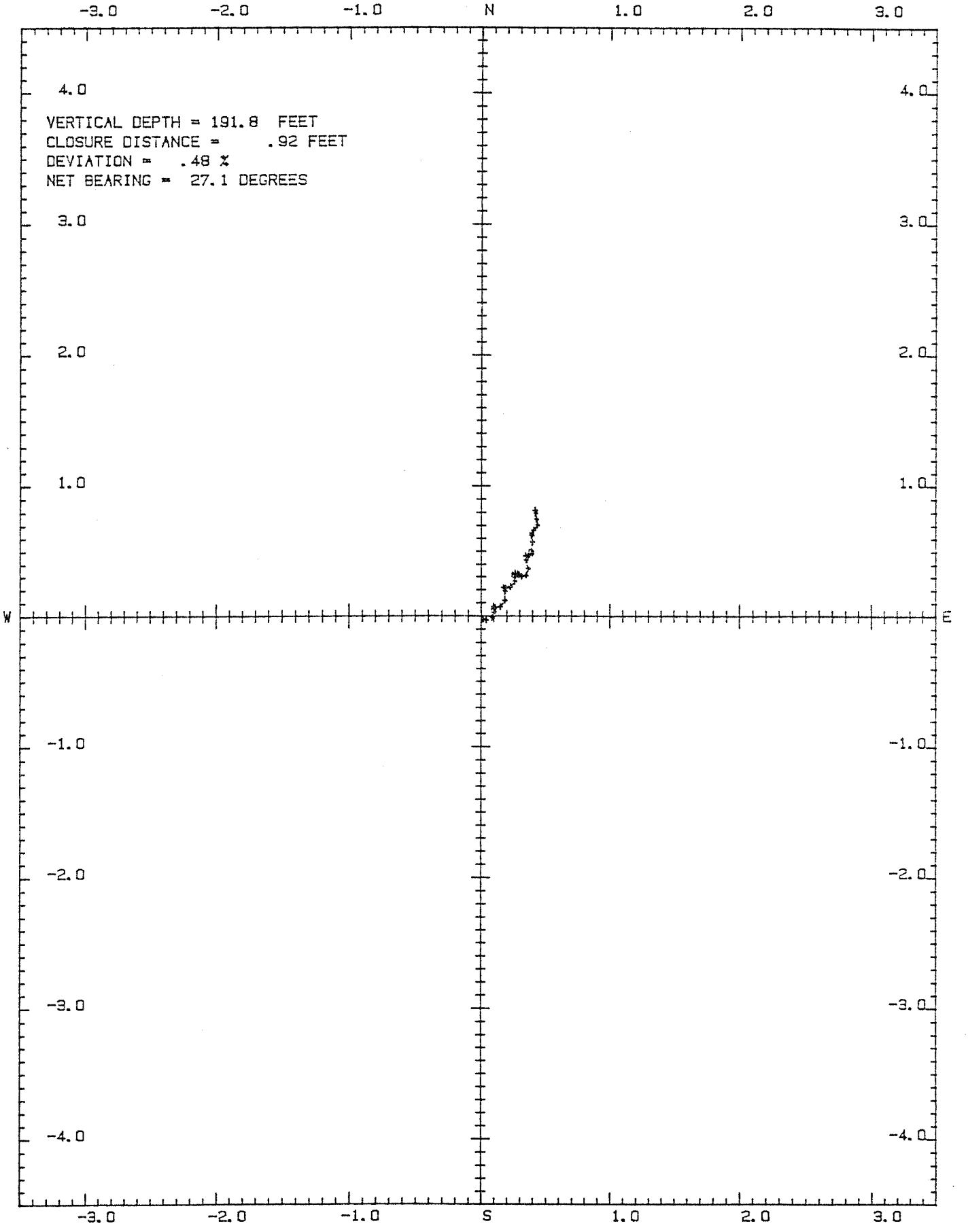
MINER FLAT DAM SITE
BOREHOLE: MF-121

NORTHING: 1,085,595.00 LOGGED BY: CHR FILE: MFS121
EASTING: 576,895.00 DATE: 2/21/86 PAGE: 1
ELEVATION: 6,090.20

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
35.0	.1	148	1,085,594.97	576,895.02	6,055.20
40.0	.5	95	1,085,594.97	576,895.04	6,050.20
45.0	.7	48	1,085,594.99	576,895.08	6,045.20
50.0	.8	349	1,085,595.04	576,895.10	6,040.20
55.0	.1	150	1,085,595.07	576,895.10	6,035.20
60.0	.3	105	1,085,595.07	576,895.11	6,030.20
65.0	.6	67	1,085,595.07	576,895.15	6,025.20
70.0	.9	17	1,085,595.12	576,895.18	6,020.20
75.0	.8	340	1,085,595.19	576,895.18	6,015.20
80.0	.1	155	1,085,595.22	576,895.17	6,010.20
85.0	.3	88	1,085,595.22	576,895.19	6,005.20
90.0	.6	70	1,085,595.22	576,895.23	6,000.20
95.0	.8	17	1,085,595.27	576,895.26	5,995.20
100.0	.4	315	1,085,595.31	576,895.25	5,990.20
105.0	.4	92	1,085,595.32	576,895.26	5,985.20
110.0	.1	144	1,085,595.32	576,895.28	5,980.20
115.0	.2	117	1,085,595.31	576,895.29	5,975.20
120.0	.3	110	1,085,595.30	576,895.31	5,970.20
125.0	.6	62	1,085,595.31	576,895.35	5,965.20
130.0	.9	349	1,085,595.36	576,895.36	5,960.20
135.0	.8	345	1,085,595.44	576,895.35	5,955.20
140.0	.1	127	1,085,595.47	576,895.34	5,950.21
145.0	.4	104	1,085,595.46	576,895.36	5,945.21
150.0	.9	14	1,085,595.49	576,895.39	5,940.21
155.0	1.0	357	1,085,595.58	576,895.40	5,935.21
160.0	.2	316	1,085,595.63	576,895.39	5,930.21
165.0	.3	115	1,085,595.63	576,895.39	5,925.21
170.0	.6	331	1,085,595.64	576,895.39	5,920.21
175.0	.6	91	1,085,595.67	576,895.41	5,915.21
180.0	.9	0	1,085,595.70	576,895.43	5,910.21
185.0	.2	309	1,085,595.75	576,895.43	5,905.21
190.0	.9	0	1,085,595.79	576,895.42	5,900.21
191.8	.5	349	1,085,595.82	576,895.42	5,898.41

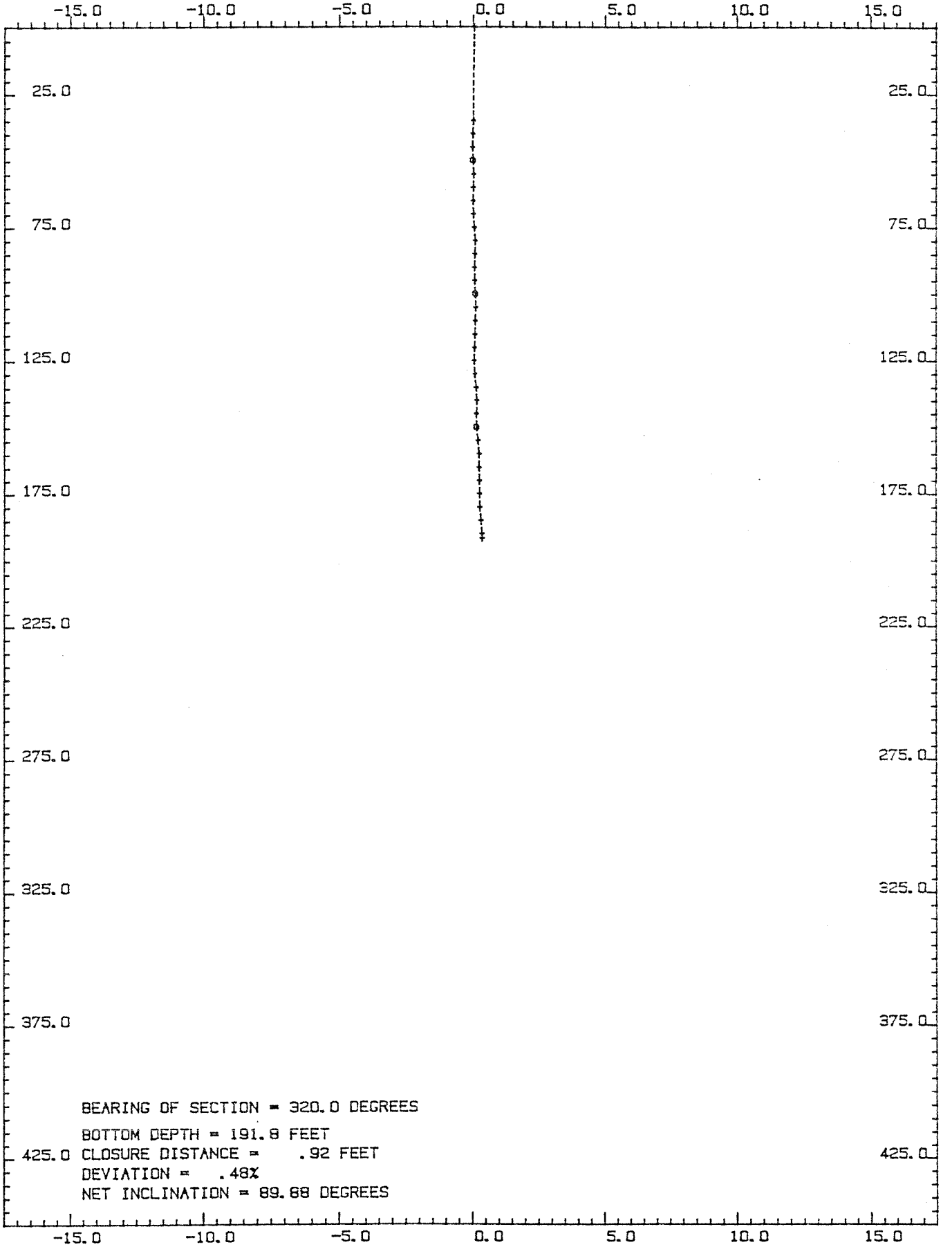
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-121

COLLAR NORTHING: 1,085,595.00 COLLAR ELEVATION: 6,090.2 NORTH SCALE: 1 in = 1.0 ft
COLLAR EASTING: 576,895.00 LOGGED BY: CHR DATE: 2/21/86 EAST SCALE: 1 in = 1.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-121

COLLAR NORTHING: 1,085,595.00 COLLAR ELEVATION: 6,090.2 HORIZ. SCALE: 1 in = 5.0 ft
COLLAR EASTING: 576,895.00 LOGGED BY: CHR DATE: 2/21/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

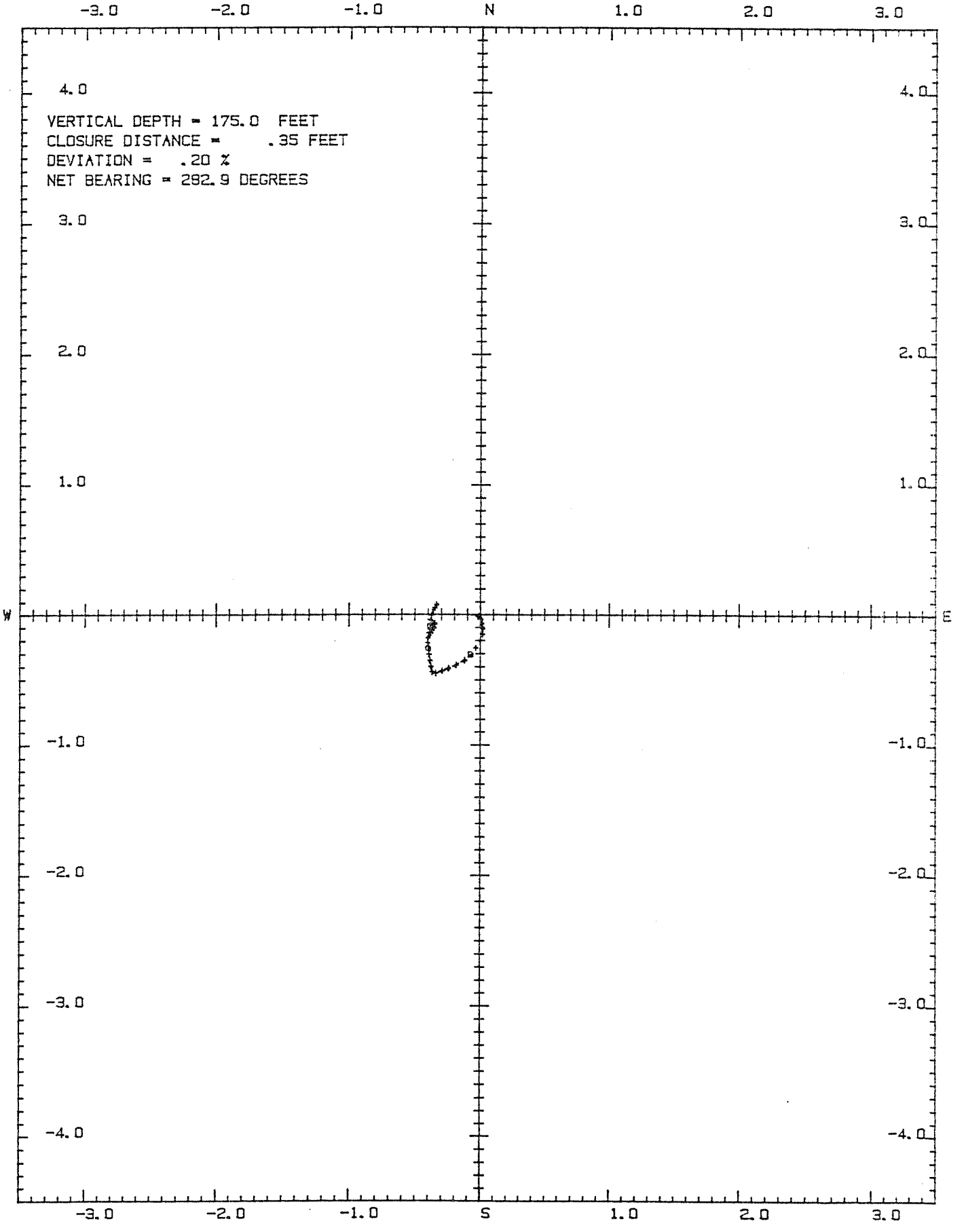
MINER FLAT DAM SITE
BOREHOLE: MF-122

NORTHING: 1,085,645.00 LOGGED BY: CHR FILE: MFS122
EASTING: 576,785.00 DATE: 2/8/86 PAGE: 1
ELEVATION: 6,075.00

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
5.0	.4	233	1,085,644.99	576,784.99	6,070.00
10.0	.2	74	1,085,644.98	576,784.98	6,065.00
15.0	.2	144	1,085,644.98	576,784.99	6,060.00
20.0	.3	154	1,085,644.96	576,785.00	6,055.00
24.9	.4	171	1,085,644.93	576,785.01	6,050.10
30.0	.5	173	1,085,644.89	576,785.02	6,045.00
35.0	.5	195	1,085,644.85	576,785.02	6,040.00
40.0	.7	205	1,085,644.80	576,785.00	6,035.00
45.0	.8	218	1,085,644.74	576,784.96	6,030.00
50.0	.7	216	1,085,644.69	576,784.92	6,025.00
55.0	.8	236	1,085,644.65	576,734.88	6,020.00
60.0	.9	247	1,085,644.61	576,784.81	6,015.00
65.0	.6	243	1,085,644.58	576,784.75	6,010.00
70.0	.6	253	1,085,644.57	576,784.70	6,005.00
75.0	.6	244	1,085,644.55	576,784.65	6,000.00
80.0	.5	348	1,085,644.56	576,784.63	5,995.00
85.0	.5	349	1,085,644.60	576,784.62	5,990.00
90.0	.6	350	1,085,644.65	576,784.61	5,985.00
95.0	.5	354	1,085,644.69	576,784.60	5,980.00
100.0	.5	352	1,085,644.74	576,784.60	5,975.00
105.0	.5	357	1,085,644.78	576,784.59	5,970.01
110.0	.4	11	1,085,644.82	576,784.60	5,965.01
115.0	.5	18	1,085,644.86	576,784.61	5,960.01
120.0	.4	25	1,085,644.89	576,784.62	5,955.01
125.0	.4	24	1,085,644.93	576,784.63	5,950.01
130.0	.3	149	1,085,644.93	576,784.65	5,945.01
135.0	.6	211	1,085,644.90	576,784.64	5,940.01
140.0	.5	211	1,085,644.85	576,784.62	5,935.01
145.0	.5	4	1,085,644.86	576,784.61	5,930.01
150.0	.6	4	1,085,644.91	576,784.61	5,925.01
155.0	.5	9	1,085,644.95	576,784.62	5,920.01
160.0	.5	9	1,085,645.00	576,784.62	5,915.01
165.0	.4	32	1,085,645.03	576,784.63	5,910.01
168.0	.5	37	1,085,645.05	576,784.65	5,907.01
175.0	.1	278	1,085,645.08	576,784.66	5,900.01

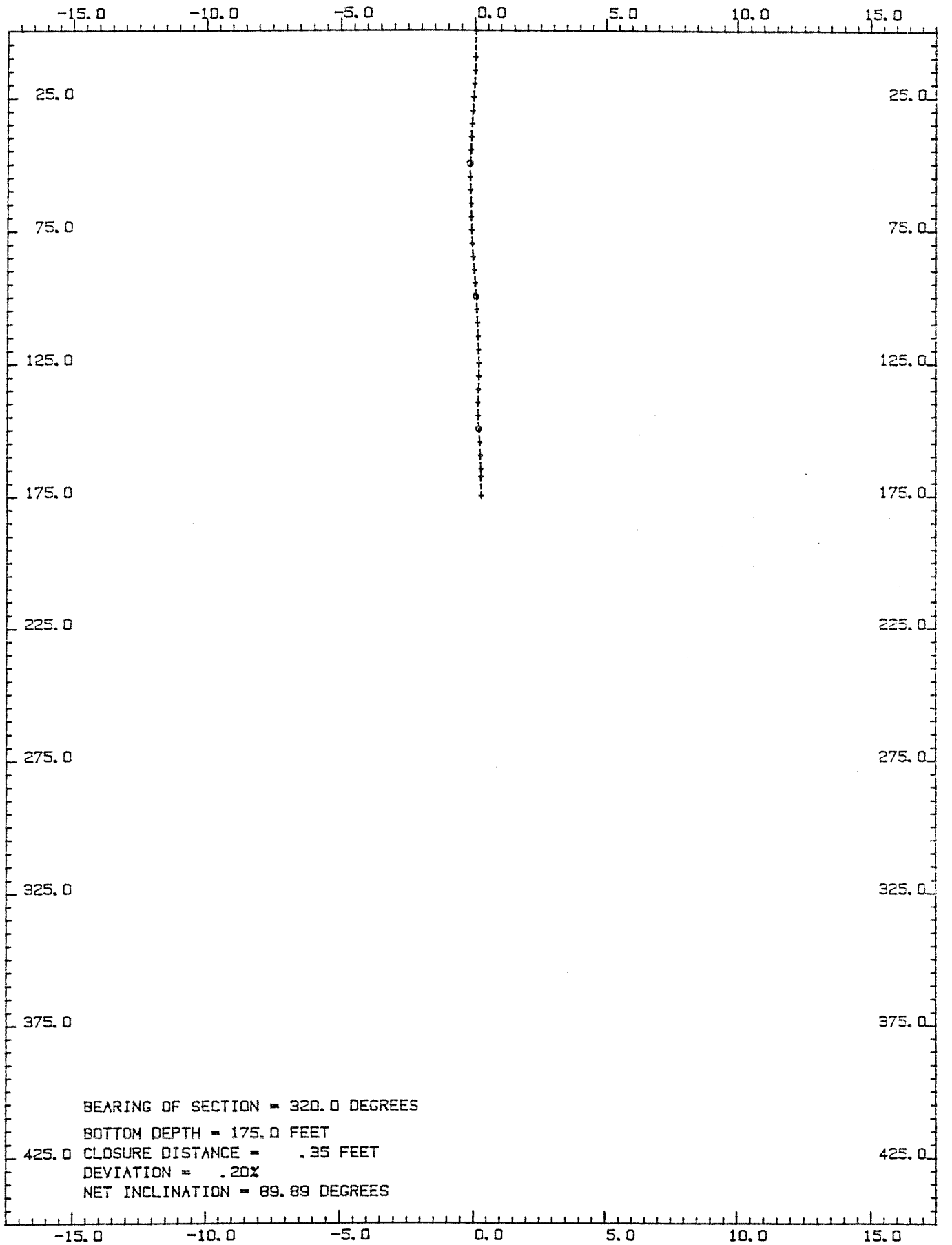
MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-122

COLLAR NORTHING: 1,085,645.00 COLLAR ELEVATION: 6,075.0 NORTH SCALE: 1 in = 1.0 ft
COLLAR EASTING: 576,785.00 LOGGED BY: CHR DATE: 2/8/86 EAST SCALE: 1 in = 1.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-122

COLLAR NORTHING: 1,085,645.00 COLLAR ELEVATION: 6,075.0 HORIZ. SCALE: 1 in = 5.0 ft
COLLAR EASTING: 576,785.00 LOGGED BY: CHR DATE: 2/8/86 VERTICAL SCALE: 1 in = 50.0 ft



BOREHOLE COORDINATE DATA

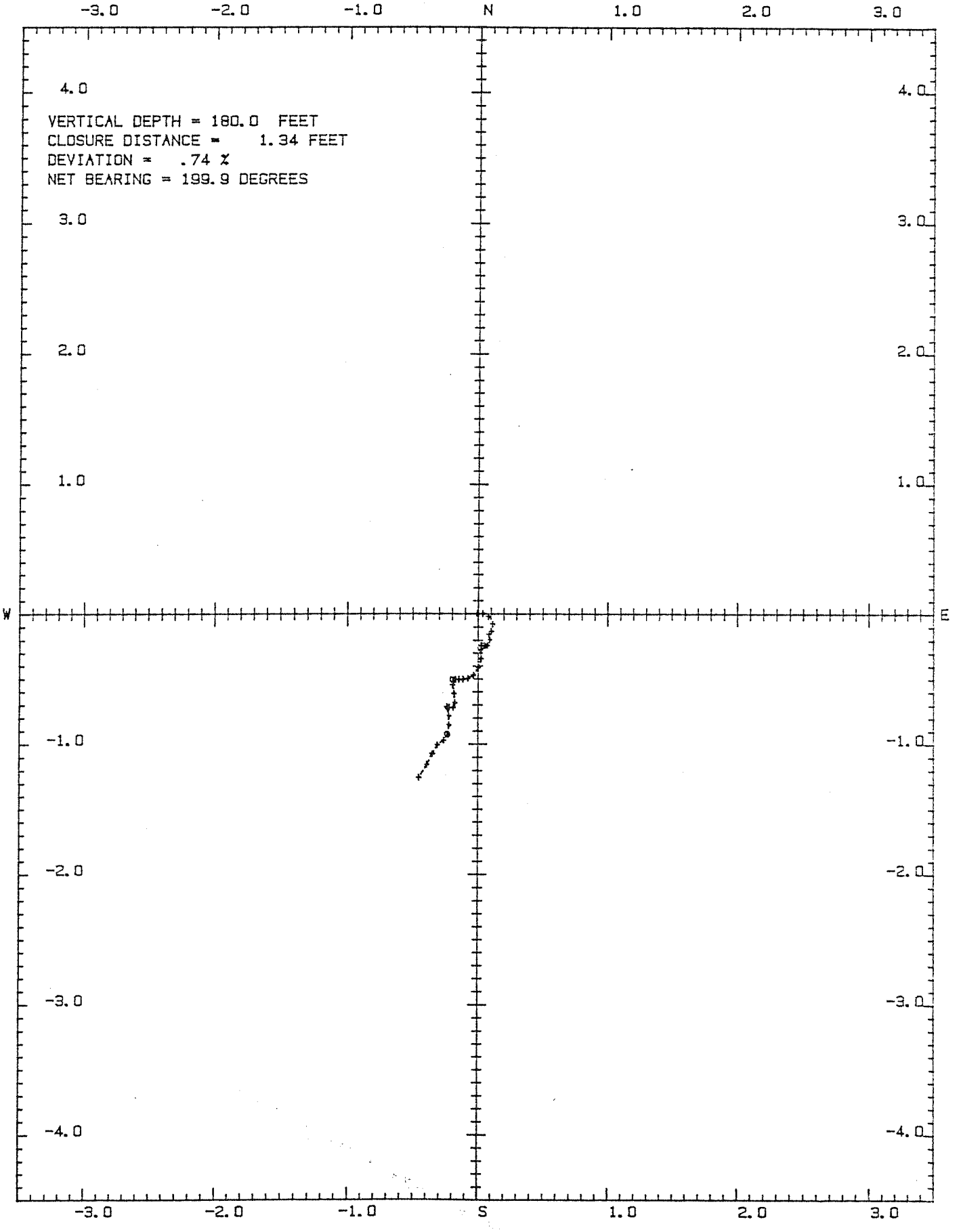
MINER FLAT DAM SITE
BOREHOLE: MF-125A

NORTHING: 0.00 LOGGED BY: CHR FILE: MFS125A
EASTING: 0.00 DATE: 4/7/86 PAGE: 1
ELEVATION: 6,070.00

CABLE DEPTH	INCLIN	DIRECTION	NORTH STATION	EAST STATION	ELEVATION
5.4	.2	273	.00	-.01	6,064.60
10.0	.4	84	.00	-.00	6,060.00
15.0	.5	99	.00	.04	6,055.00
20.0	.7	134	-.02	.08	6,050.00
25.0	.8	164	-.08	.11	6,045.00
30.0	.7	221	-.13	.10	6,040.00
35.0	0.0	283	-.16	.08	6,035.00
40.0	.9	171	-.20	.09	6,030.00
45.0	.5	254	-.24	.07	6,025.00
50.0	.2	295	-.24	.04	6,020.00
55.0	.3	264	-.24	.02	6,015.00
60.0	.8	163	-.28	.02	6,010.00
65.0	.8	191	-.34	.02	6,005.00
70.0	.9	206	-.41	.00	6,000.00
75.0	.7	223	-.47	-.04	5,995.00
80.0	.5	261	-.50	-.08	5,990.00
85.0	.4	261	-.50	-.12	5,985.00
90.0	.3	275	-.50	-.15	5,980.00
95.0	.3	268	-.50	-.18	5,975.00
100.0	.2	243	-.51	-.20	5,970.00
105.0	.8	170	-.55	-.20	5,965.01
110.0	.8	171	-.62	-.19	5,960.01
115.0	.8	179	-.69	-.18	5,955.01
120.0	.3	269	-.72	-.19	5,950.01
125.0	.4	272	-.72	-.22	5,945.01
130.0	.1	319	-.72	-.24	5,940.01
135.0	.5	154	-.73	-.24	5,935.01
140.0	.8	179	-.79	-.23	5,930.01
145.0	.8	179	-.86	-.23	5,925.01
150.0	.9	198	-.93	-.24	5,920.01
155.0	.5	254	-.97	-.27	5,915.01
160.0	.9	220	-1.01	-.32	5,910.01
165.0	.9	200	-1.08	-.35	5,905.01
170.0	1.2	212	-1.16	-.40	5,900.01
180.0	.1	199	-1.26	-.46	5,890.01

MINER FLAT DAM SITE
PLAN OF BOREHOLE: MF-125A

COLLAR NORTHING: 0.00 COLLAR ELEVATION: 6,070.0 NORTH SCALE: 1 in = 1.0 ft
COLLAR EASTING: 0.00 LOGGED BY: CHR DATE: 4/7/86 EAST SCALE: 1 in = 1.0 ft



MINER FLAT DAM SITE
CROSS SECTION OF BOREHOLE: MF-125A

COLLAR NORTHING: 0.00 COLLAR ELEVATION: 6,070.0 HORIZ. SCALE: 1 in = 5.0 ft
COLLAR EASTING: 0.00 LOGGED BY: CHR DATE: 4/7/86 VERTICAL SCALE: 1 in = 50.0 ft

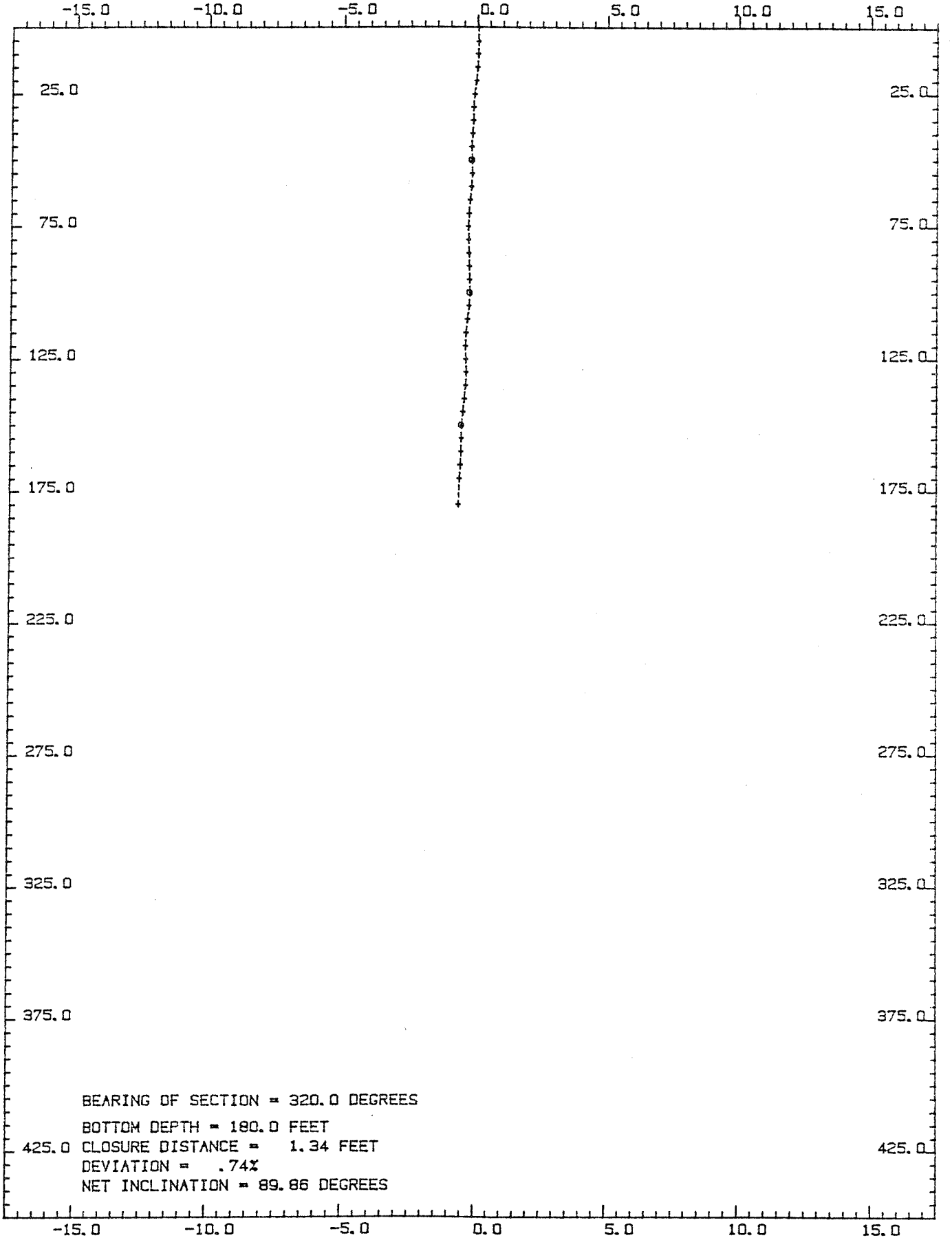


FIGURE 2
GEOLOGIC MAP
OF THE
MINER FLAT DAM RESERVOIR AREA
WHITE MOUNTAIN APACHE INDIAN RESERVATION
NAVAJO COUNTY, ARIZONA

EXPLANATION

af
Artificial fill

ALLUVIAL DEPOSITS

Qal
ALLUVIUM
(Boulders, cobbles, gravel, sand and silt along present stream channels.)

Qt_2
TERRACES
(Alluvium of boulders, cobbles, gravel, sand and silt; subscript relative age, 1 being the oldest.)

COLLUVIAL DEPOSITS

Qc
COLLUVIUM
(Derived from weathering of bedrock; letter symbols in parentheses are symbols for bedrock units from which colluvium derived.)

Qta
TALUS
(Letter symbols in parentheses are symbols for bedrock units from which talus derived.)

BEDROCK

QTg
QUATERNARY-TERTIARY GRAVEL
(No outcrops)

QTb
BASALT
(Massive to vesicular and scoriatious basalt.)

Ps
SUPAI FORMATION
(Sandstone, fine-grained rounded, well sorted, cross-stratified, massive bedded. Lower part interbedded with siltstone.)

70
Contact, showing dip
(Dashed where approximately located)

5
Strike and dip of beds

75
Strike and dip of joints

Strike of vertical joint

DH-3 5920.8
Drill Hole

DH-2 60 5920.8
Surface drill hole with inclination

RA-2H
Line of detail survey

A A'
Line of section

Geology by Charles S. Robinson

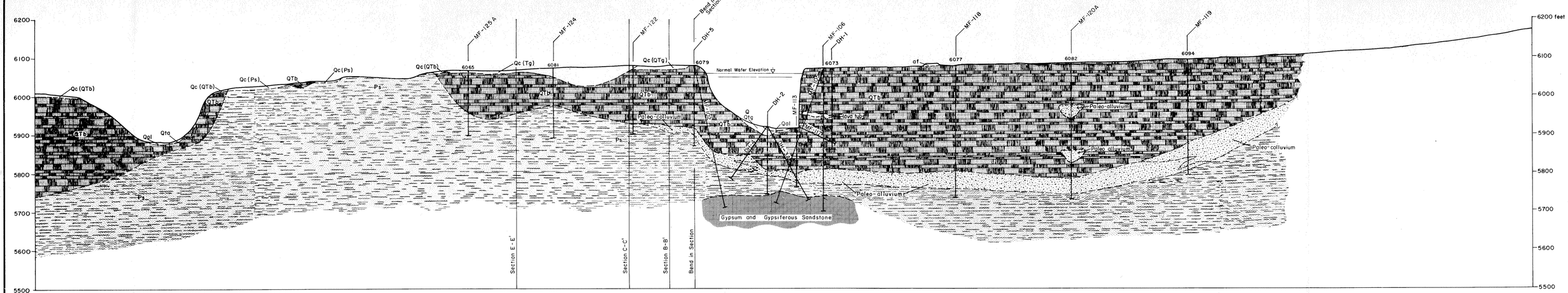
Mineral Systems, Inc.

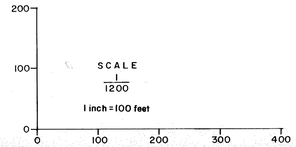
Golden, Colorado, June 1983

FEET

GEOLOGIC MAP OF THE MINER FLAT DAM SITE
WHITE MOUNTAIN APACHE INDIAN RESERVATION
NAVAJO COUNTY, ARIZONA

FIGURE 3

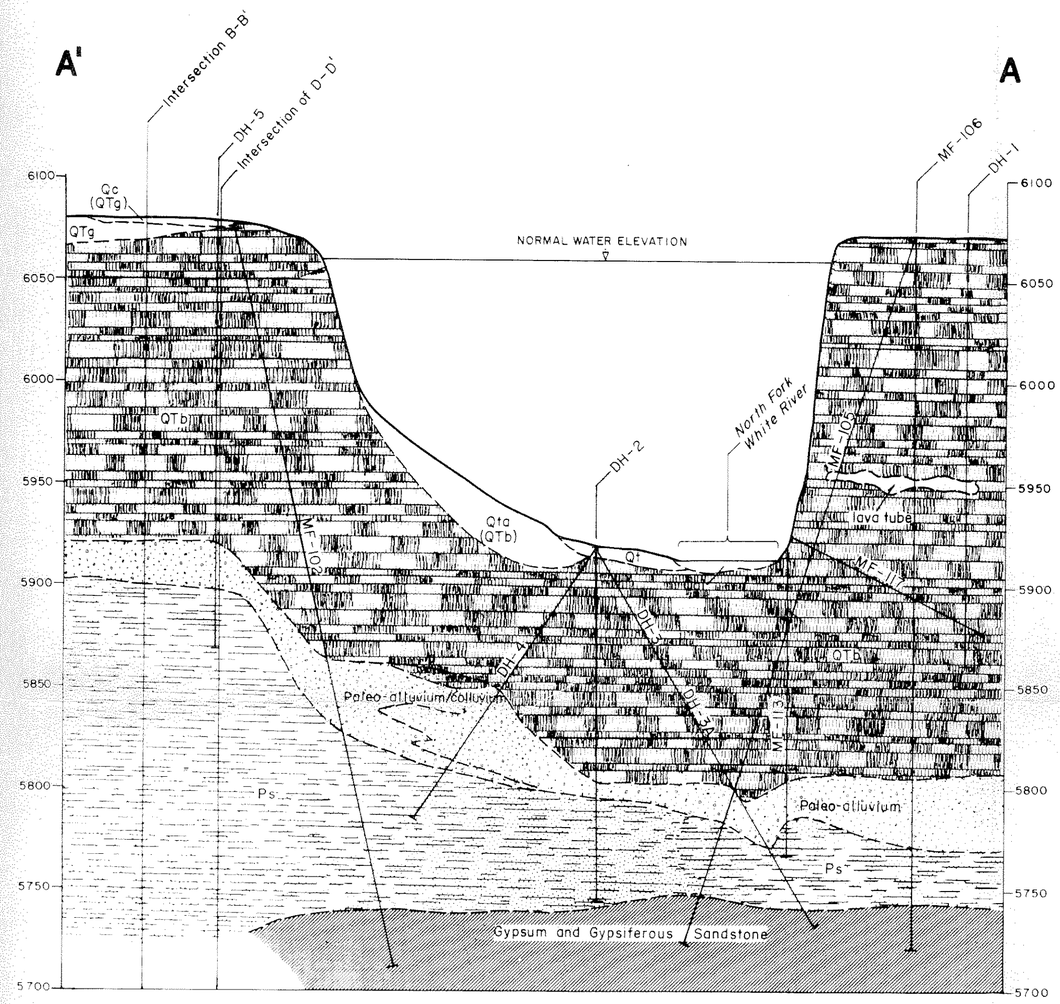




PRELIMINARY GEOLOGIC SECTION
MINER FLAT DAM SITE
WHITE MOUNTAIN APACHE INDIAN RESERVATION
NAVAJO COUNTY, ARIZONA

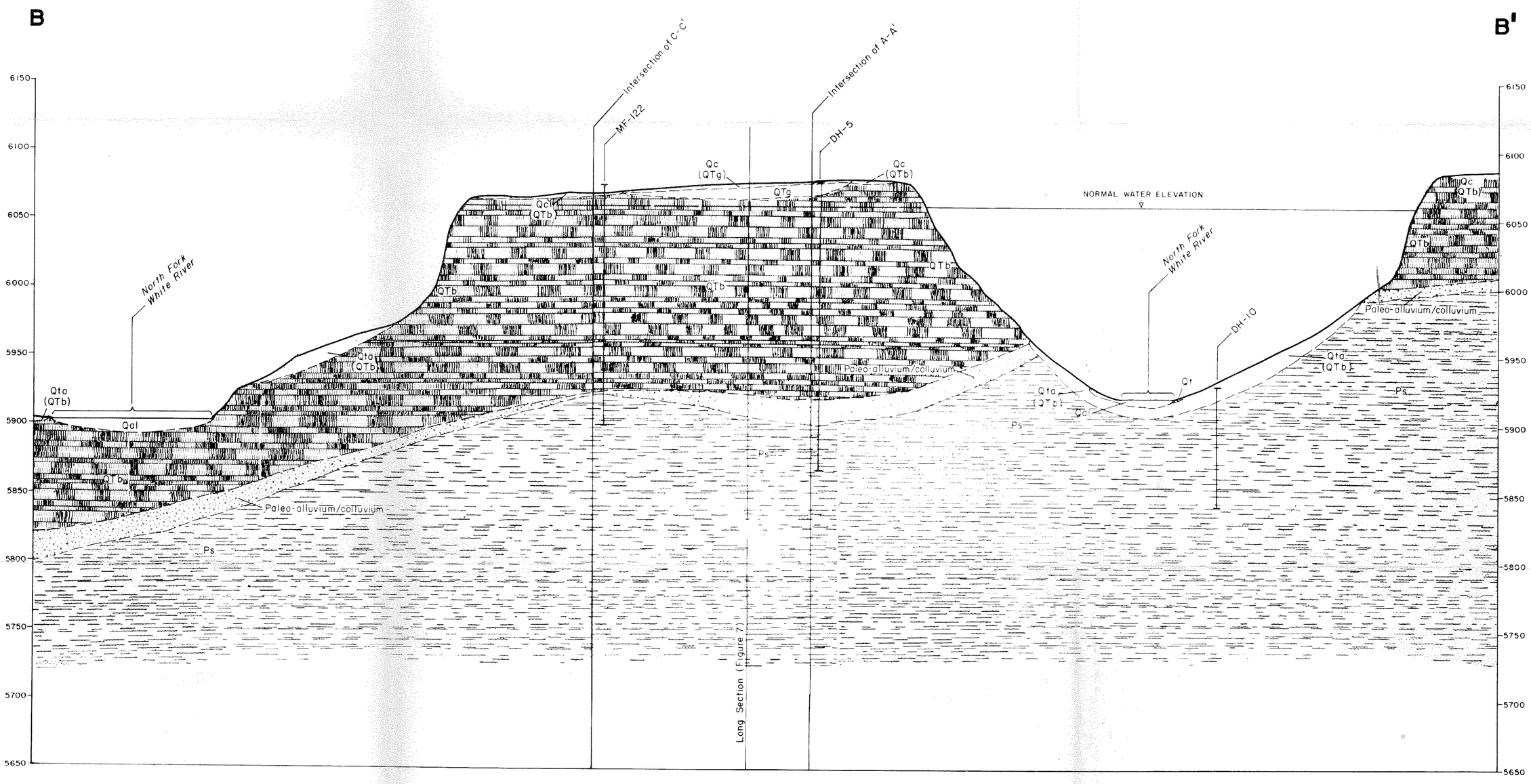
Figure 7

MINERAL SYSTEMS, INC.
Golden, Colorado
February, 1986



EXPLANATION

- Qal — Alluvium
- Qt — Terrace
- Qc — Colluvium (Bedrock source shown by symbol for rock unit)
- Qta — Talus (Bedrock source shown by symbol for rock unit)
- QTg — Quaternary-Tertiary Gravel
- QTb — Basalt
- Ps — Supai Formation (Showing gypsiferous units)
- Contact
- MF-105 — Drill Hole

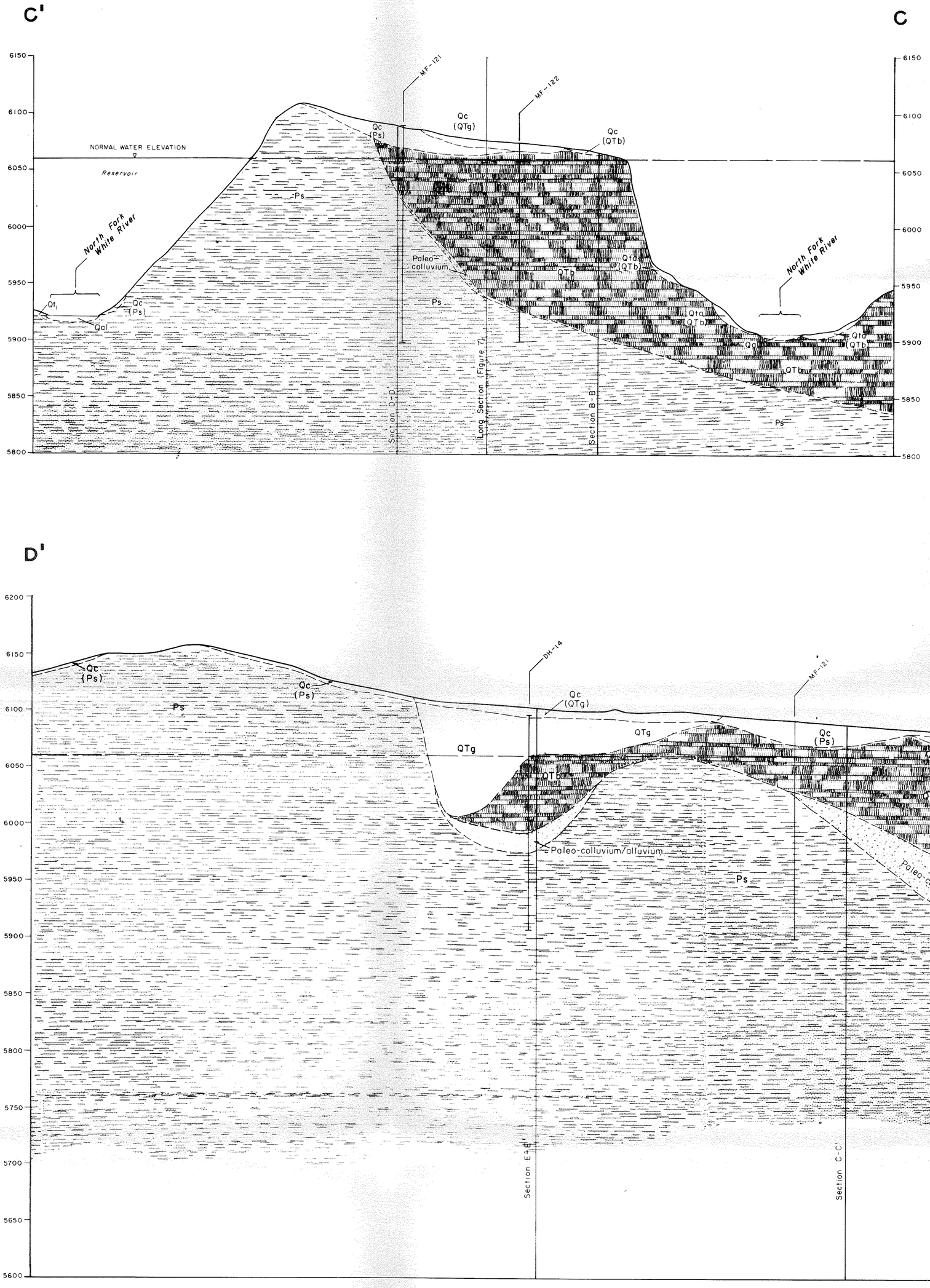


HORIZONTAL AND VERTICAL SCALE
1 INCH = 50 FEET
0 50 100 150 FEET

GEOLOGIC SECTIONS A-A' AND B-B'
MINER FLAT DAM SITE
WHITE MOUNTAIN APACHE INDIAN RESERVATION
NAVAJO COUNTY, ARIZONA

MINERAL SYSTEMS, INC
1986

Figure 8



GEOLOGIC SECTIONS C-C', D-D' AND E-E'
MINER FLAT DAM SITE
WHITE MOUNTAIN APACHE INDIAN RESERVATION
NAVAJO COUNTY, ARIZONA

Figure 9

Mineral Systems, Inc.
Golden, Colorado

1986

LEGEND

- BC-4 / 6406.20 — HORIZONTAL AND VERTICAL CONTROL STATION
- 093 / 5082.10 — ANALYTICAL AEROTRIANGULATION STATION
- 6247.0 — SPOT ELEVATION
- 6300 — INDEX CONTOUR
- INTERMEDIATE CONTOUR
- SUPPLEMENTARY CONTOUR
- DEPRESSION CONTOUR
- ROAD WITH HARD SURFACE
- UNIMPROVED ROAD OR TRAIL
- BUILDINGS
- TREES
- FENCE
- HYDROGRAPHY

NOTES

TOPOGRAPHY BY PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY FLOWN 11-4-80.

500-FT. GRIDS BASED UPON ARIZONA STATE GRID COORDINATES, EAST ZONE, MODIFIED TO 6000 MEAN ELEVATION.

ELEVATIONS BASED UPON U.S.G.S. DATUM

EXPLANATION

- 6060 — Limit of reservoir
- 6000 / 5900 — Contoured on base of basalt (Contour interval 100 feet)
- Direction of water infiltration from reservoir.
- Axis of basalt filled channal
- Limit of basalt

400 0 400 800

CONTOUR INTERVAL 10 FEET

CONTOUR MAP OF THE BASE OF THE BASALT FLOW

MINER FLAT DAM AND RESERVOIR SITE
WHITE MOUNTAIN APACHE INDIAN RESERVATION
NAVAJO COUNTY, ARIZONA

Figure 10

TOPOGRAPHIC MAP
MINER FLAT DAMSITE
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO CO., ARIZONA

GEODETIC ENGINEERS, INC
Englewood, Colorado

DATE: JAN., 1982
FILE NO: 82202

APPENDIX S

Design Memorandum, Miner Flat Dam, February 1987

MORRISON-MAIERLE, INC.,
VOLUMES I THRU V OF VI

FEBRUARY 2007

DESIGN MEMORANDUM

MINER FLAT DAM

DECEMBER, 1986

[REVISED FEBRUARY 1987]

WHITE MOUNTAIN APACHE TRIBE

WHITERIVER, ARIZONA

VOLUME 4 OF 6

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604
CORPORATE OFFICE
HELENA, MONTANA
BRANCH OFFICES
BILLINGS, MONTANA
KALISPELL, MONTANA
BOZEMAN, MONTANA
RIVERTON, WYOMING

MINER FLAT DAM
DESIGN MEMORANDUM

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	J.2	Dam Design and Stability Analysis	J - 3
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		Conventional Concrete Dam-Section	Sht J4
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		Eastside Access Road	Sht J6
		Eastside Access Road	Sht J7
Six		1986 Environmental Assessment by Joe C. Elliott	

DESIGN MEMORANDUM

SECTION F

F. WATER FOR CONSTRUCTION PURPOSES

The North Fork White River has sufficient water supply to provide the necessary water during construction. Principal water uses during construction will be make up water for the rollcrete plant and make up water for the aggregate washing and screening plant. Other uses of water for dust control and construction support will be negligible.

It is anticipated that the average rate of rollcrete production will be 120,000 cubic yards in a 30 day period or an average maximum production rate of 4,000 cubic yards per day. The rollcrete mix will require 150 pounds of water per cubic yard (about 18 gallons per cubic yard) which works out to a constant 24-hour per day water requirement of about 50 gpm. If the 24-hour demand rate is to be supplied in a lesser period of time such as 8 hours to meet the construction schedule, the average maximum instantaneous demand for water from the North Fork White River will be 150 gpm of consumptive use.

Flow records indicate that the North Fork White River at the proposed dam site will easily satisfy a 150 gpm requirement. In addition, if the surface flow of the North Fork White River decreases such that the 150 gpm instantaneous demand cannot be satisfied by surface flow, test pits excavated into the unconsolidated alluvium along the river indicate that pumping from sumps excavated into the alluvium will easily satisfy the 150 gpm requirement from the groundwater flow in the alluvium. Additional water supply requirements for a washing and screening plant are essentially nonconsumptive and should easily be pro-

vided by the surface water flows. Make up water requirements for the aggregate plant can be substantially reduced by recirculation of plant water through a series of settling ponds.

Accordingly, the peak make up water requirements for construction purposes are estimated to be two times the average maximum daily 8-hour requirement for the rollcrete makeup water, or 300 gpm, which is 0.67 cubic feet per second. A demand of 0.67 cubic feet per second is a very small demand compared to the low flows in the North Fork White River. Moreover, if the surface flow of the North Fork White River were to completely dry up, the groundwater flow in the alluvium adjacent to the river will supply the anticipated construction water demands.

\MINER FLAT DAM\CONSTWTR

DESIGN MEMORANDUM
HYDROLOGY AND HYDRAULICS

SECTIONS G, H, AND I

G. HYDROLOGIC ANALYSIS

1. BASIN CHARACTERISTICS

DRAINAGE AREA

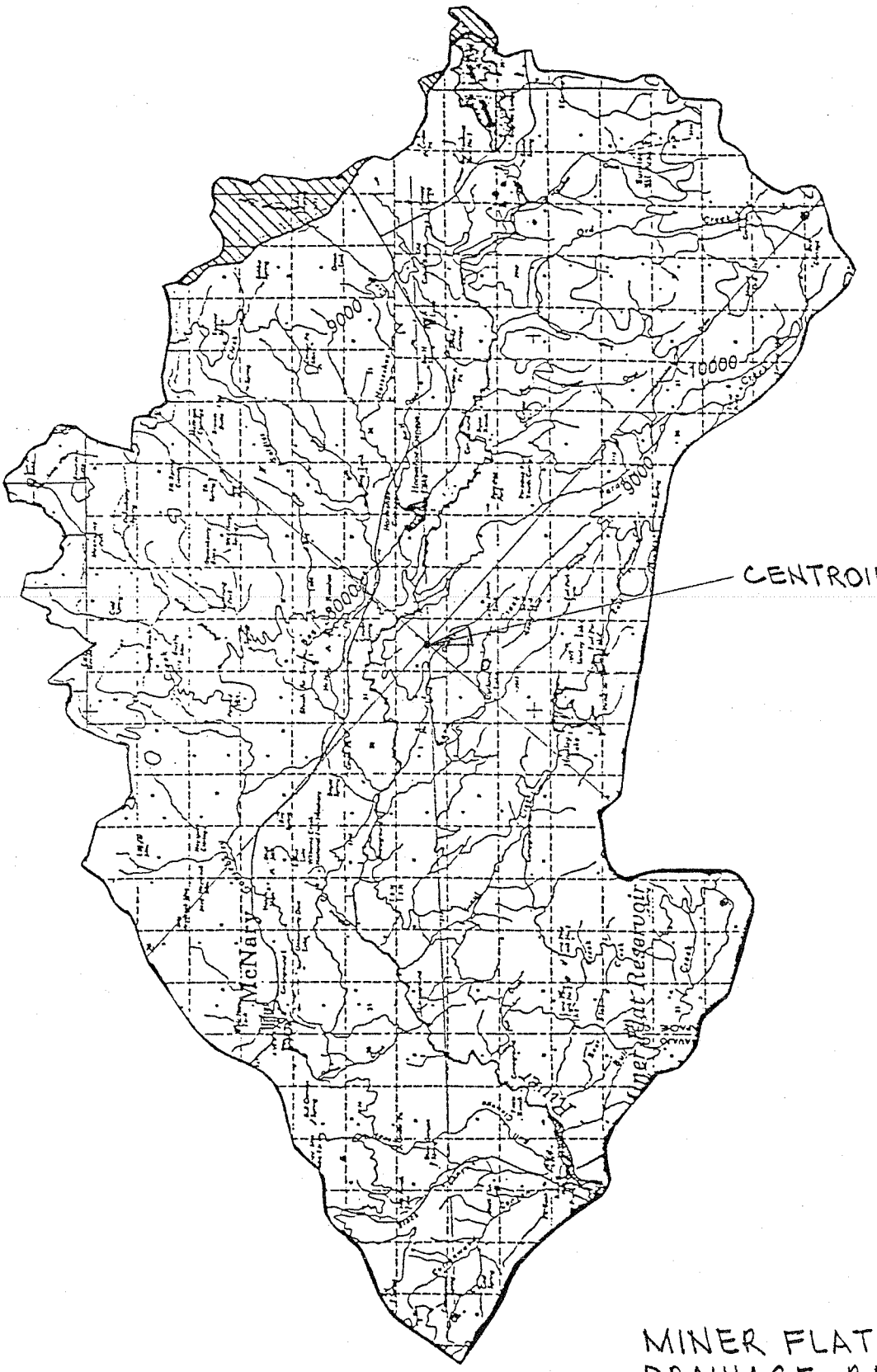
The drainage area at the dam site was determined using 7 1/2 minute U.S. Geological Survey quadrangle mapping. The total drainage area equals 238.5 square miles. The drainage basin shape is ideal for a hydrologic analysis using a single stream course as shown of the following Figure. Subbasins were not used due to the basin shape, however, elevation bands were used for the snowmelt computation.

ELEVATION BANDS

Elevation bands within the basin were determined using 7 1/2 minute USGS quadrangle mapping. The areas between each band are shown on the following Table.

BAND	SQUARE MILES
Below 6000'	0.1
6-7000	32.9
7-8000	60.9
8-9000	88.9
9-10,000	42.2
10-11,000	13.1
Above 11,000	0.5
Total	238.5 square miles

Note that the average basin elevation is 8280 feet.



MINER FLAT DAM
DRAINAGE BASIN

STREAM LENGTH AND SLOPE

The length of the longest stream course in the basin was determined using 7 1/2 minute USGS quadrangle mapping and equals 33.05 miles or 174,500 feet.

The stream course length from the dam site to the centroid of the basin was determined using a cutout of the basin and hanging a plumb line at different locations. The stream length to the centroid was determined to equal 22.04 miles or 116,400 feet.

The high point along the longest stream course equals 9,460 feet and the minimum elevation at the dam site equals 5,910 feet. The average stream slope equals 107 feet per mile.

UPSTREAM RESERVOIRS

Two large reservoirs lie upstream of the proposed dam site, Hawley and Sunrise. Available Dam Safety Investigations were requested from the the U.S. Bureau of Indian Affairs in Phoenix. A SEED report was obtained for Sunrise Dam. This report is not a detailed dam safety investigation. A SEED report was not obtained for Hawley Lake. At this time there has not been a Dam Safety Inspection of either structure with sufficient detail to determine the spillway design flood and adequacy. Therefore, it is not known whether these projects would be overtopped during a PMF and fail.

The failure of either upstream dam would impact Miner Flat Dam. There is sufficient channel reach between the projects to provide an undetermined amount of channel routing of the potential dambreak peaks. Refer to the dambreak section for the magnitude of routing of a dambreak at Miner Flat Dam to the town of Whiteriver.

RAINFALL RECORDS

Long-term rainfall records are available for McNary. Records are presented on the following Table for McNary which show the monthly distribution, minimum, maximum and averages.

MCNARY PRECIPITATION
WEATHER STATION

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL	MASS PLOT	5-YEAR AVERAGE	PERCENT LONG TERM	MASS PLOT	5-YEAR AVERAGE
1943	6.19	.04	4.10	2.54	1.04	3.28	.75	.62	.47	1.45	4.42	2.07	26.97	26.97	--	1.02	1.02	--
1944	2.04	.06	1.73	.80	3.49	2.18	1.90	.62	.00	2.23	4.10	3.27	22.42	49.39	--	.85	1.88	--
1945	.79	2.63	2.40	1.64	1.09	4.29	.67	.09	.34	3.02	4.42	.91	22.29	71.68	--	.85	2.72	--
1946	2.06	.07	3.72	2.02	1.10	2.05	2.16	.00	.00	3.54	4.80	5.32	26.84	98.52	--	1.02	3.74	--
1947	2.15	3.25	1.89	.89	1.26	.21	.18	3.72	.21	1.63	8.66	2.92	26.97	125.49	25.10	1.02	4.77	.95
1948	3.53	1.94	1.39	.32	3.23	2.64	.15	.00	.46	3.83	2.27	1.67	21.43	146.92	23.99	.81	5.58	.91
1949	3.38	.05	4.89	4.35	2.00	1.68	.73	.49	1.52	7.49	3.09	2.74	32.61	179.53	26.03	1.24	6.82	.99
1950	2.25	.25	2.23	1.26	.80	1.50	.26	.20	.77	1.92	1.50	1.74	14.68	194.21	24.51	.56	7.38	.93
1951	.25	.46	.00	2.53	1.42	1.90	2.62	.60	.00	4.10	5.43	1.17	20.48	214.69	23.23	.78	8.16	.88
1952	3.30	1.95	4.58	6.53	1.11	5.11	3.72	.00	1.03	2.56	2.93	2.63	35.45	250.14	24.93	1.35	9.50	.95
1953	.00	3.21	2.06	.89	1.34	3.65	2.03	.23	.81	3.48	3.87	.00	21.57	271.71	24.96	.82	10.32	.95
1954	.32	1.51	1.15	2.25	.72	8.29	.00	.75	.53	4.80	4.80	2.35	27.47	299.18	23.93	1.04	11.37	.91
1955	.87	.53	.96	3.53	1.56	.12	.01	.30	2.58	4.76	5.46	.39	21.07	320.25	25.21	.80	12.17	.96
1956	1.44	2.05	3.24	3.27	2.42	.11	2.05	.00	.24	3.31	1.89	.56	20.58	340.83	25.23	.78	12.95	.96
1957	1.46	.00	.66	9.07	1.24	2.11	1.60	1.81	.63	3.75	4.30	.29	26.92	367.75	23.52	1.02	13.97	.89
1958	3.55	1.89	.53	.27	4.16	5.99	2.30	.31	1.18	1.43	4.96	3.41	29.98	397.73	25.20	1.14	15.11	.96
1959	2.42	1.75	.30	1.17	1.68	.00	.93	.00	.94	1.83	5.88	1.15	18.05	415.78	23.32	.69	15.80	.89
1960	5.63	1.98	4.76	3.14	.81	1.76	.60	1.30	.37	.87	1.73	1.23	24.18	439.96	23.94	.92	16.72	.91
1961	3.63	.52	1.51	1.25	.65	3.47	.31	.07	.38	2.67	4.70	1.78	21.14	461.10	24.05	.80	17.52	.91
1962	4.52	1.16	3.77	3.09	2.28	1.14	.06	.37	.74	2.87	1.41	4.42	25.83	486.93	23.84	.98	18.50	.91
1963	2.05	2.09	1.26	1.40	2.60	1.88	.72	.00	.00	3.08	5.97	1.95	23.00	509.93	22.44	.87	19.38	.85
1964	1.50	2.42	.28	1.12	.53	1.96	1.44	.00	.04	5.16	4.87	4.75	24.07	534.00	23.64	.91	20.29	.90
1965	.37	1.99	1.56	2.92	3.99	2.97	3.63	.68	.88	6.54	2.07	1.81	29.41	563.41	24.69	1.12	21.41	.94
1966	1.08	4.76	9.26	1.49	1.70	.51	.27	.28	.53	2.88	1.82	2.90	27.48	590.89	25.96	1.04	22.45	.99
1967	.90	1.49	3.05	1.07	.24	3.45	.54	.96	1.22	6.35	5.71	2.44	27.42	618.31	26.28	1.04	23.49	1.00
1968	.17	1.76	10.73	2.16	1.10	3.62	1.55	.21	.55	5.01	3.72	.53	31.11	649.42	27.90	1.18	24.68	1.06
1969	.95	2.05	4.68	3.64	2.41	2.50	.37	1.60	.04	3.77	3.50	1.02	26.53	675.95	28.39	1.01	25.68	1.06
1970	1.54	2.04	1.12	.31	.43	3.88	2.23	.51	.41	3.64	3.61	4.05	23.77	699.72	27.26	.90	26.59	1.04
1971	1.93	.50	2.18	.87	.95	.68	1.26	.02	.00	1.84	6.87	3.55	20.65	720.37	25.90	.78	27.37	.98
1972	7.51	2.14	4.55	.09	.12	.00	.07	.60	5.26	2.01	2.67	1.69	26.71	747.08	25.75	1.01	28.39	.98
1973	8.15	3.06	2.73	1.92	3.54	9.33	1.21	3.20	1.42	4.29	1.55	.26	40.66	787.74	27.66	1.54	29.93	1.05
1974	.11	3.06	.26	5.35	.61	2.13	.40	.15	.00	2.46	3.88	2.52	20.93	808.67	26.54	.80	30.73	1.01
1975	8.28	.58	1.08	2.84	3.64	4.38	2.47	.50	.00	4.08	2.39	2.08	32.32	840.99	28.25	1.23	31.95	1.07
1976	.34	2.64	1.61	.86	3.56	3.08	2.84	1.58	.55	7.62	3.57	3.13	31.38	872.37	30.40	1.19	33.15	1.16
1977	1.99	.77	.25	3.95	.57	2.46	1.55	.65	1.46	4.46	3.33	2.93	24.37	896.74	29.93	.93	34.07	1.14
1978	2.63	1.33	.95	4.04	6.26	7.57	1.21	1.23	.41	2.54	2.72	1.49	32.38	929.12	28.28	1.23	35.30	1.07
1979	1.72	8.31	9.24	7.00	1.14	5.07	.55	4.37	1.43	.87	3.03	.51	43.24	972.36	32.74	1.64	36.95	1.24
1980	.78	1.55	1.63	9.46	8.08	3.37	1.08	.00	.00	2.38	4.70	2.16	35.19	1007.55	33.31	1.34	38.28	1.27
TOTAL	91.78	67.84	102.29	101.30	75.07	110.52	46.42	28.02	27.40	130.52	146.60	79.79	1007.55					
YEARS	38	38	38	38	38	38	38	38	38	38	38	38	38					
AVERAGE	2.42	1.79	2.69	2.67	1.98	2.91	1.22	.74	.72	3.43	3.86	2.10	26.51447					
PER CENT	9.11%	6.73%	10.15%	10.05%	7.45%	10.97%	4.61%	2.78%	2.72%	12.95%	14.55%	7.92%	100.00%					
1943-1978	53.70	31.80	48.97	56.25	40.72	58.49	28.82	12.16	14.12	76.32	93.53	48.53	563.41					
	23	23	23	23	23	23	23	23	23	23	23	23	23					
	2.33	1.38	2.13	2.45	1.77	2.54	1.25	.53	.61	3.32	4.07	2.11	24.50					
	9.53%	5.64%	8.69%	9.98%	7.23%	10.38%	5.12%	2.16%	2.51%	13.55%	16.60%	8.61%	100.00%					
1966-1978	35.58	26.18	42.45	28.59	25.13	43.59	15.97	11.49	11.85	50.95	45.34	28.59	365.71					
	13	13	13	13	13	13	13	13	13	13	13	13	13					
	2.74	2.01	3.27	2.20	1.93	3.35	1.23	.88	.91	3.92	3.49	2.20	28.13					
	9.73%	7.16%	11.61%	7.82%	6.87%	11.92%	4.37%	3.14%	3.24%	13.93%	12.40%	7.82%	100.00%					

SNOWPACK RECORDS

Available snowpack records were obtained from the Soil Conservation Service. Note that several of the snow courses in the upper basin were discontinued in later years. The stations used were: Hawley Lake, Mt. Ord, Smith Cienaga, Sunrise Summit, Wilson Lake, Forest Dale, McNary and Milk Ranch. These were selected with the criteria of proximity to the study basin and similar elevations and exposures.

STREAMFLOW RECORDS

Monthly streamflow records were obtained from the USGS for the following gages and sites:

SITE	GAGE #	DRAINAGE AREA
White River Near McNary	4910	66 square miles
White River Near Fort Apache	4940	632
East Fork White River	4924	38.8

A transfer relationship was used to determine the streamflows at the dam site and shown on the following Table. The average annual stream flow at the Miner Flat dam site is estimated at 139,700 acre-feet. The majority of this runoff would occur during March, April and May or the spring snowmelt period.

FINAL1.MFD

NORTH FORK WHITE RIVER NEAR MCNARY, STATION NUMBER 4910
DRAINAGE AREA: 66 Square Miles

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
1958	1,230	1,380	1,190	845	900	2,040	14,530	14,050	5,470	1,320	1,220	2,230	46,405
1959	2,380	1,230	1,020	940	976	1,180	1,640	1,050	592	644	2,590	1,190	15,432
1960	805	1,250	1,040	1,350	1,100	7,550	7,280	7,180	3,580	1,020	827	584	33,566
1961	684	644	712	615	692	1,200	3,190	1,920	758	619	1,120	1,250	13,404
1962	781	972	906	764	1,960	1,630	16,270	11,460	4,610	1,940	1,020	1,120	43,433
1963	1,270	1,140	887	835	1,800	3,050	3,880	3,440	941	710	1,940	3,870	23,763
1964	1,400	1,220	902	688	696	1,060	5,270	3,560	1,240	1,150	1,990	3,790	22,966
1965	1,730	1,160	972	2,180	1,650	2,380	12,500	10,880	7,670	4,410	3,920	1,640	51,092
1966	1,120	2,520	3,270	2,250	1,390	6,360	11,050	9,240	3,210	1,950	2,390	2,050	46,800
1967	1,270	1,210	1,090	1,050	1,250	1,830	1,780	1,450	1,070	1,450	5,020	4,340	22,810
1968	1,200	861	922	738	851	3,190	9,650	9,480	6,030	2,190	2,650	1,180	38,942
1969	889	801	833	1,010	962	2,070	9,580	8,140	4,210	2,520	2,750	2,740	36,505
1970	1,370	1,270	1,060	738	754	1,550	5,040	6,900	2,620	1,350	2,170	2,990	27,812
1971	1,760	1,010	950	780	724	1,750	1,920	1,700	886	661	1,740	1,840	15,721
1972	6,010	2,940	2,590	1,970	1,500	3,280	2,180	1,880	1,410	1,060	1,070	952	26,842
1973	5,460	2,410	1,640	1,230	873	1,360	11,640	27,900	17,460	4,900	2,080	1,080	78,033
1974	932	960	1,030	902	764	2,660	3,180	3,200	1,240	1,020	1,240	747	17,875
1975	1,810	1,240	817	662	655	2,760	7,090	12,240	6,210	1,630	1,200	1,330	37,644
1976	845	762	499	778	1,130	1,960	5,360	7,790	2,860	1,690	1,630	1,270	26,574
1977	1,210	762	817	829	704	1,310	3,610	2,390	1,170	896	1,580	994	16,272
1978	1,330	1,030	1,090	938	908	5,750	10,900	8,870	3,790	1,130	1,110	766	37,612
1979	673	1,180	6,050	2,680	1,480	3,360	15,950	20,770	15,680	3,220	1,930	1,060	74,033
1980	760	889	831	718	1,300	2,590	7,140	11,250	8,540	2,260	2,950	1,860	41,088
TOTAL	36,919	28,841	31,118	25,490	25,019	61,870	170,630	186,740	101,247	39,740	46,137	40,873	794,624
NO. YEARS	23	23	23	23	23	23	23	23	23	23	23	23	23
AVERAGE	1,605	1,254	1,353	1,108	1,088	2,690	7,419	8,119	4,402	1,728	2,006	1,777	34,549
PER CENT	4.65%	3.63%	3.92%	3.21%	3.15%	7.79%	21.47%	23.50%	12.74%	5.00%	5.81%	5.14%	100.00%

WHITE RIVER NEAR FORT APACHE - NORTH FORK WHITE RIVER NEAR MCNARY - EAST FORK WHITE RIVER
DRAINAGE AREA: 527.2 Square Miles

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
1958	882	1,660	1,618	1,078	3,066	18,820	35,740	18,630	1,320	-26	1,930	2,510	87,228
1959	3,130	1,565	1,187	894	865	1,809	1,830	4	-356	676	6,460	858	18,922
1960	1,900	7,860	6,545	8,570	5,640	28,630	19,000	7,570	1,460	-267	893	110	87,911
1961	573	678	990	971	990	2,199	5,070	590	-177	-277	2,274	961	14,842
1962	604	1,857	2,901	3,993	13,830	10,320	45,940	11,360	1,060	324	-185	1,816	93,820
1963	1,034	1,849	1,525	1,341	4,580	9,350	9,740	1,970	-412	-1,085	3,760	4,000	37,652
1964	936	1,322	928	808	765	1,337	9,300	1,930	-74	1,117	2,530	5,490	26,389
1965	1,711	733	1,190	12,110	7,410	13,280	40,710	19,230	3,930	4,520	4,480	1,380	110,684
1966	1,062	5,410	21,550	12,880	4,090	29,570	22,550	8,280	1,250	998	3,450	2,500	113,590
1967	1,347	1,069	1,987	2,017	923	2,700	3,490	620	115	3,410	14,730	7,580	39,988
1968	1,622	1,393	2,791	5,582	16,039	23,060	33,750	19,700	1,840	290	4,180	1,560	111,807
1969	1,103	1,424	1,132	7,890	4,188	10,270	29,820	12,130	1,390	1,160	2,950	4,070	77,527
1970	3,535	2,152	1,750	1,707	1,588	4,910	10,790	6,660	1,620	636	2,470	4,050	41,868
1971	960	852	965	1,374	1,545	2,960	2,660	1,200	-146	665	8,860	4,340	26,235
1972	14,520	5,830	13,820	7,790	2,839	4,330	2,090	230	1,280	1,347	1,662	265	56,003
1973	13,230	5,280	7,790	7,240	8,447	22,080	60,620	84,280	7,800	3,810	2,390	844	223,811
1974	711	1,071	1,299	2,236	1,931	6,410	5,720	2,500	308	-229	180	531	22,668
1975	3,310	2,620	2,305	2,232	2,694	17,660	34,680	24,040	4,170	2,800	1,768	1,640	99,919
1976	980	1,179	1,844	1,263	6,462	5,200	13,960	8,740	1,560	3,990	4,270	1,950	51,398
1977	1,302	1,227	1,403	1,729	1,621	2,238	7,060	2,630	471	3,920	3,430	1,693	28,724
1978	2,190	1,421	565	1,215	6,662	56,180	34,330	13,580	2,120	891	2,880	897	122,931
1979	1,117	10,090	34,390	13,950	13,270	30,460	59,370	38,030	9,700	2,190	2,300	578	215,445
1980	527	1,178	1,194	5,570	40,140	23,980	55,240	43,510	11,850	4,430	3,750	2,120	193,489
TOTAL	58,286	59,720	111,669	104,440	149,585	327,753	543,460	327,414	52,079	35,290	81,412	51,743	1,902,851
NO. YEARS	23	23	23	23	23	23	23	23	23	23	23	23	23
AVERAGE	2,534	2,597	4,855	4,541	6,504	14,250	23,629	14,235	2,264	1,534	3,540	2,250	82,733
PER CENT	3.06%	3.14%	5.87%	5.49%	7.86%	17.22%	28.56%	17.21%	2.74%	1.85%	4.28%	2.72%	100.00%

NORTH FORK WHITE RIVER AT MINER FLAT DAM
DRAINAGE AREA: 238 Square Miles

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
1958	1,518	4,250	3,560	2,480	4,790	23,050	57,330	43,740	9,750	2,060	4,070	6,960	163,558
1959	3,402	3,650	2,880	2,430	2,420	3,600	4,590	1,880	606	2,110	13,390	3,020	43,978
1960	1,425	10,270	8,460	11,190	7,750	40,090	32,300	20,750	7,480	1,400	2,390	1,160	144,665
1961	871	2,060	2,350	2,060	2,210	4,170	10,480	4,290	1,180	826	4,220	3,140	37,857
1962	978	3,500	4,660	5,520	17,500	13,590	70,600	30,760	8,660	3,240	1,630	3,780	164,418
1963	1,607	3,810	3,020	2,830	7,760	14,280	17,420	8,540	1,200	240	7,630	9,860	78,197
1964	1,705	3,470	2,450	1,970	1,910	3,030	16,460	8,210	1,940	3,010	5,830	11,090	61,075
1965	2,288	2,460	2,780	15,630	10,280	17,370	58,140	38,480	16,540	11,320	10,760	4,030	190,078
1966	1,467	9,800	27,040	16,960	6,520	40,010	39,520	23,580	6,320	3,900	7,310	5,880	188,307
1967	1,710	2,890	3,990	3,620	2,750	5,560	6,910	3,590	2,020	6,520	23,870	15,030	78,460
1968	1,729	2,840	4,430	7,340	18,840	29,190	48,690	37,080	12,100	4,170	9,610	3,830	179,849
1969	1,249	2,860	2,640	10,360	6,300	14,480	46,150	28,040	8,300	4,760	7,610	9,090	141,839
1970	2,524	4,420	3,890	3,100	2,950	7,770	18,280	19,870	5,960	2,750	6,330	9,750	87,594
1971	2,073	2,430	2,460	2,650	2,850	5,840	5,960	4,140	1,390	1,800	12,620	8,030	52,243
1972	10,749	11,180	18,410	11,090	5,230	10,060	5,830	3,260	3,810	3,230	3,460	1,990	88,299
1973	9,778	9,300	10,660	9,670	10,460	25,780	79,120	127,400	35,480	11,520	5,870	2,650	337,688
1974	1,164	2,550	2,830	3,650	3,220	10,480	11,130	8,310	2,310	1,380	2,480	1,900	51,404
1975	2,890	5,130	3,720	3,450	4,040	23,250	45,900	44,510	15,020	5,730	3,730	4,340	161,710
1976	1,165	2,430	2,940	2,550	8,390	8,160	22,070	22,710	6,160	7,170	7,550	4,280	95,575
1977	1,635	2,540	2,700	3,110	2,860	4,220	13,400	7,600	2,550	5,490	6,310	3,470	55,885
1978	2,045	3,020	2,170	2,760	9,000	68,270	53,830	30,410	8,730	2,810	5,020	2,160	190,225
1979	1,038	12,520	43,950	18,690	16,520	37,370	86,140	71,390	35,360	7,620	6,360	2,510	339,468
1980	932	2,760	2,820	7,210	45,250	29,680	70,730	67,860	29,800	8,920	8,780	5,210	279,952
TOTAL	55,942	110,140	164,810	150,320	199,800	439,300	820,980	656,400	222,666	101,976	166,830	123,160	3,212,324
NO. YEARS	23	23	23	23	23	23	23	23	23	23	23	23	23
AVERAGE	2,432	4,789	7,166	6,536	8,687	19,100	35,695	28,539	9,681	4,434	7,253	5,355	139,666
PER CENT	1.74%	3.43%	5.13%	4.68%	6.22%	13.68%	25.56%	20.43%	6.93%	3.17%	5.19%	3.83%	100.00%

FLOOD PEAKS VERSUS RECURRENCE INTERVAL

A flood peak versus recurrence interval relationship was developed for the dam site using stream flow records from Gages 4940, 4924, 4910, and 4908. These records were analyzed using log-Pearson methodology as presented in the Water Resources Bulletin 17-B with the following results:

		RECURRENCE INTERVAL, years					
GAGE	DA(sq.mi.)	2-	5-	10-	50-	100-	500-Year
4940	632	3280	5874	8055	14,290	17,595	27,064
4910	66	400	730	1015	1,850	2,300	3,615
4924	38.8	283	546	786	1,565	2,020	3,463
4908	39	183	266	325	469	535	704

Gages 4940 and 4924 have 27 years of record. Gages 4910 and 4908 have respectively; 38 and 13 years of record. Because of the short period of record availabel for 4908, it was not used directly.

Historic flood peaks in the region have been directly affected by the mean basin elevation. This affect has been reported in the ADOT-RS-15(121) Report for Methods for Estimating the Magnitude and Frequency of Floods in Arizona. This report concluded that basins with average elevations greater than 7500 feet should be analyzed separately as a high elevation group.

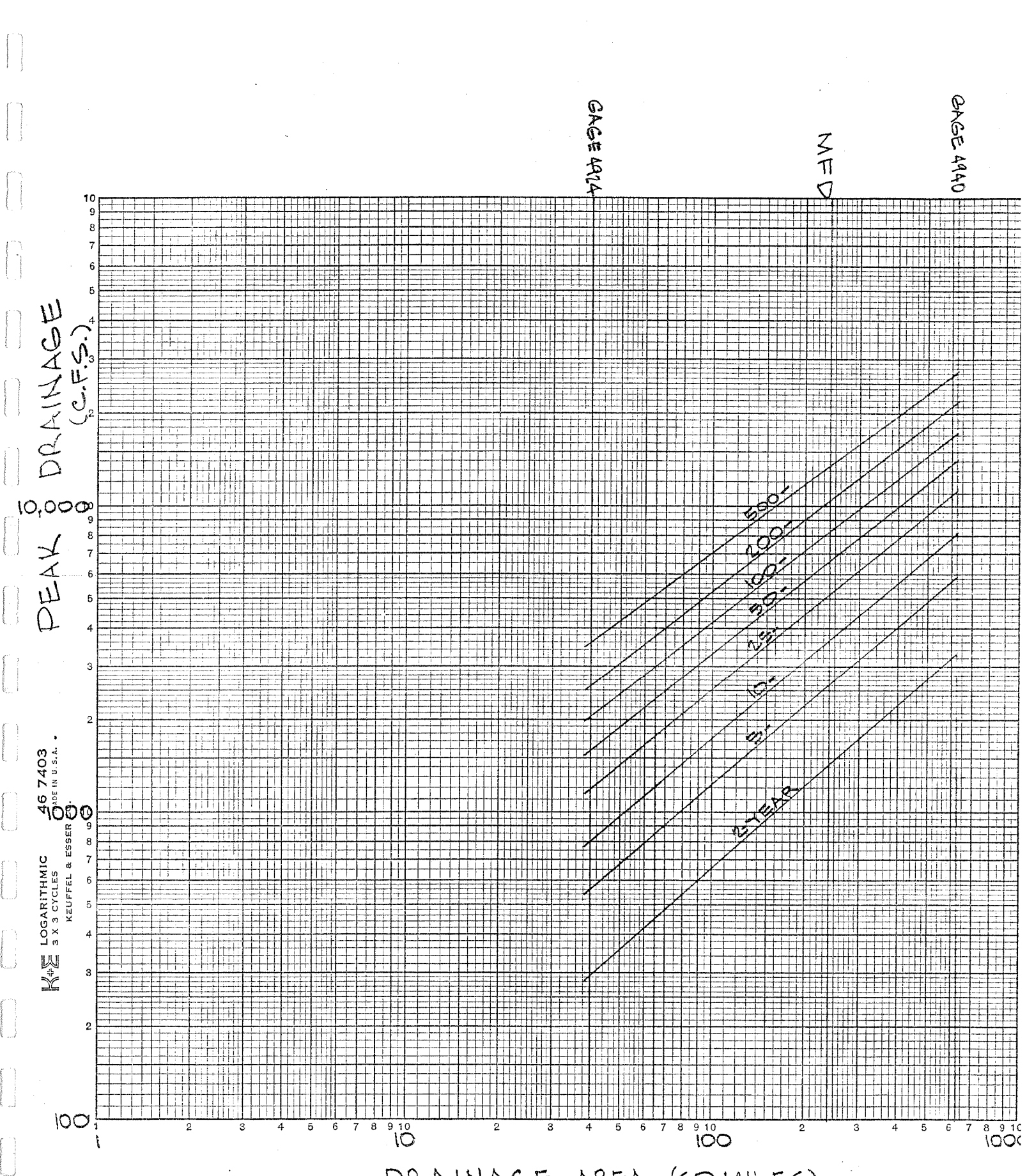
Historic flood events from these basins have been many times less than peaks from comparable basin areas at lower elevations. This is mostly explained by the major historic flood events which have occured in the winter or early spring and had temperatures warm enough to keep rain falling at the lower elevations. This produced rainfall and snowmelt runoff and significant runoff events. During the same events, the precipitation fell as snow at higher elevation and produced much less runoff on a per square mile basis.

This is demonstrated by the records at gages 4924 and 4908 which have equal drainage areas but lie at different elevations. The elevation respectively are; 6050 and 8370 feet. The 100-year flood at 4924 is approximately 4 times that of 4908. This ratio is also found in the results of the ADOT report.

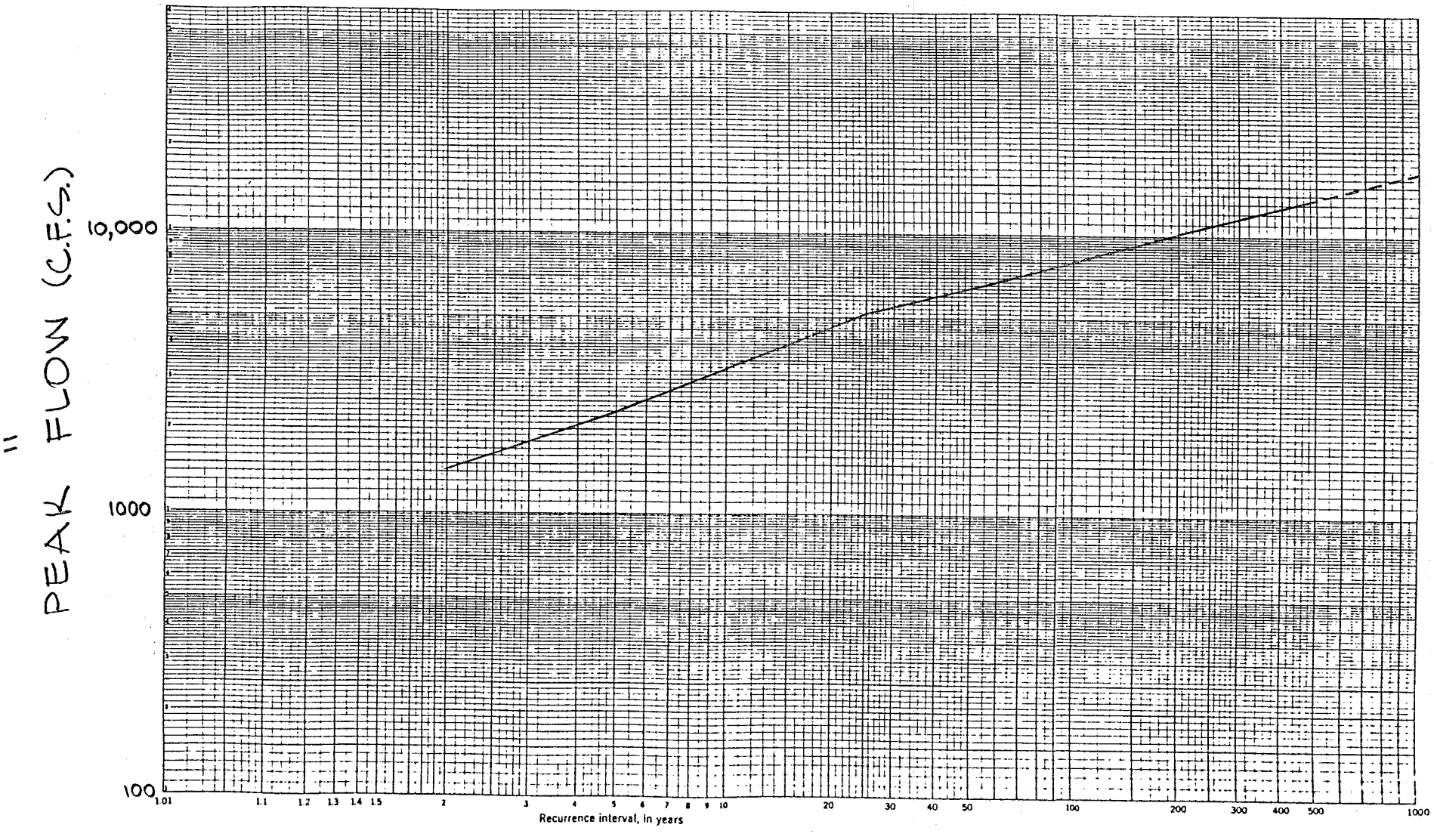
Because there is a significant drainage area at the Miner Flat Dam site which lies below 7500 (62.5 square miles), the transfer relationship for the dam site is based upon gages 4940 and 4924.

These results above were transferred to the dam site using a logarithmic transfer relationship as shown on the following Figure and Table. Routing through the reservoir was not considered.

	RECURRENCE INTERVAL, years					
SITE	2-	5-	10-	50-	100-	500-Year
MINER FLAT	1420	2600	3300	6500	8200	13,500


GAGE 4914
MFD
GAGE 4940
PEAK DRAINAGE (C.F.S.)
10,000
500-
200-
100-
50-
25-
10-
5-
2-YEAR
DRAINAGE AREA (SQ. MILES)
1
10
100
1000
LOGARITHMIC
3 X 3 CYCLES
KEUFFEL & ESSER
46 7403
MADE IN U.S.A.

FLOOD PEAKS


PEAK FLOW (C.F.S.)
10,000
1000
100
1.01
1.1
1.2
1.3
1.4
1.5
2
3
4
5
6
7
8
9
10
20
30
40
50
100
200
300
400
500
1000
Recurrence interval, in years

FLOOD PEAKS

HISTORIC FLOOD EVENTS

Flood peaks of record occured during two separate storm events during 1978. The first event occurred March 1st to 5th and resulted in a peak of 7,020 at Gage 4940. The second event occurred December 17th to 22nd with peaks of 14,600 cfs at Gage 4940 and 1,060 at Gage 4910. This event produced a 75-year flood at Gage 4940. The peak at gage 4910 was not as significant due to the higher elevation basin at that gage. Much of precipatation falling at the higher elevations occured as snow.

The December event developed the peaks of record in the White River basin. This event is shown on the following Figure. The estimated peak at the dam site using a log transfer relationship is 6,000 cfs or approximately equal to a 40-year flood.

ACCUMULATED PRECIPITATION, INCHES
0
2
4
6
FLOW (C.F.S.)
16000
14000
12000
10000
8000
6000
4000
2000
0
WHITE R.
1200
2200
0800
1800
0400
1400
2400
1000
2000
0600
1600
0200
1200
2200
24
48
72
96
108
132
12/17
12/18
12/19
12/20
12/21
12/22
TIME (HOURS)
DEC. 17-22

SNYDER'S TIME TO PEAK

The time to peak for the basin at gaging station 4940 was developed using the December 1978 event and the optimization routine in the HEC-1 program. The Snyder time to peak of the December 1978 event was determined to be 6.5 hours with Cp equal to 0.44.

The stream length at this gage equals 64.8 miles and the length to the centroid equals 32.7 miles. The average stream slope equals 78.5 feet per mile.

Snyder equation:

$$Tp = Ct\ (LLc)^{0.33}$$

The value of Ct was determined to be 0.50 and is comparable to results determined by Snyder for Western streams. The value of Cp (0.44) is also comparable to Snyder's findings in Western streams.

Using this Ct value with the basin parameters at the dam site produces a basin lag time of 4.4 hours. This is similar to values from other empirical equations. A lag time of 4.4 hours is equal to an average velocity of 11.0 feet per second.

Hydraulic studies of 25 miles of the stream channel indicate a travel time of 3.3 hours for the PMF event or an average velocity of 11.1 feet per second. The flood travel times level off as significant events spill into the overbanks. The time to peak developed from the 1978 event and transfered to the Miner Flat dam site is considered to be representative of a large event such as the PMF.

2. PMP DESIGN STORMS

DATA SOURCE

Probable Maximum Precipitation amounts were developed from Hydrometeorological Report No. 49. PMP amounts were developed on a monthly basis for general and local storms. An August general storm event produced both the greatest 6- and 72-hour amounts respectively 7.21 and 23.22 inches.

The six-hour thunderstorm event developed a storm total of 5.5 inches. The areal reduction due to the size of the basin reduced the thunderstorm rainfall to a point where it did not become the worst case 6-hour rainfall. This eliminated the local thunderstorm event from further evaluation.

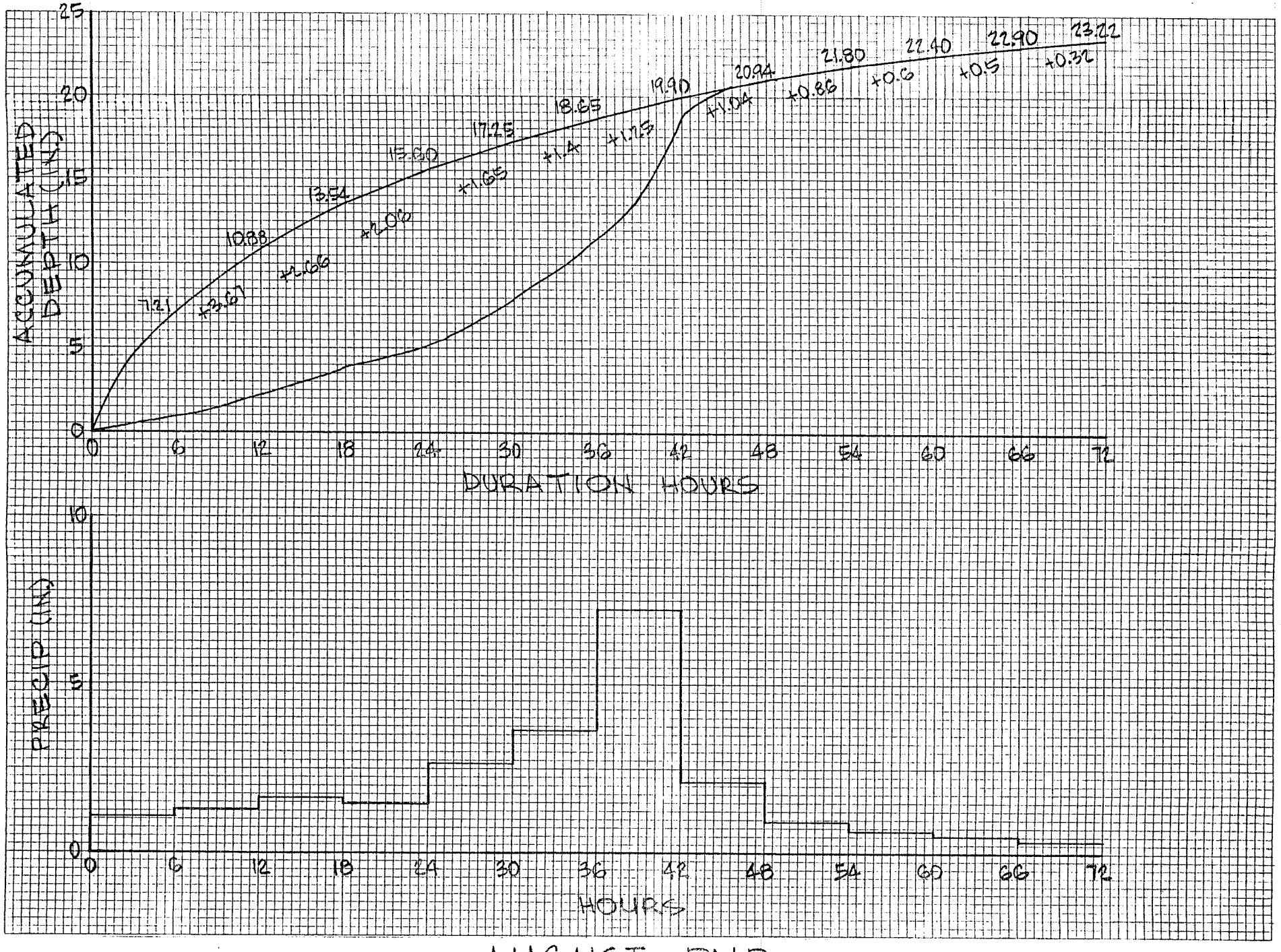
The 72-hour April PMP amount equals 16.87 inches or less than the August event. Because the basin accumulates a significant snowpack, the April event was evaluated with smowmelt conditions.

AUGUST PMP EVENT

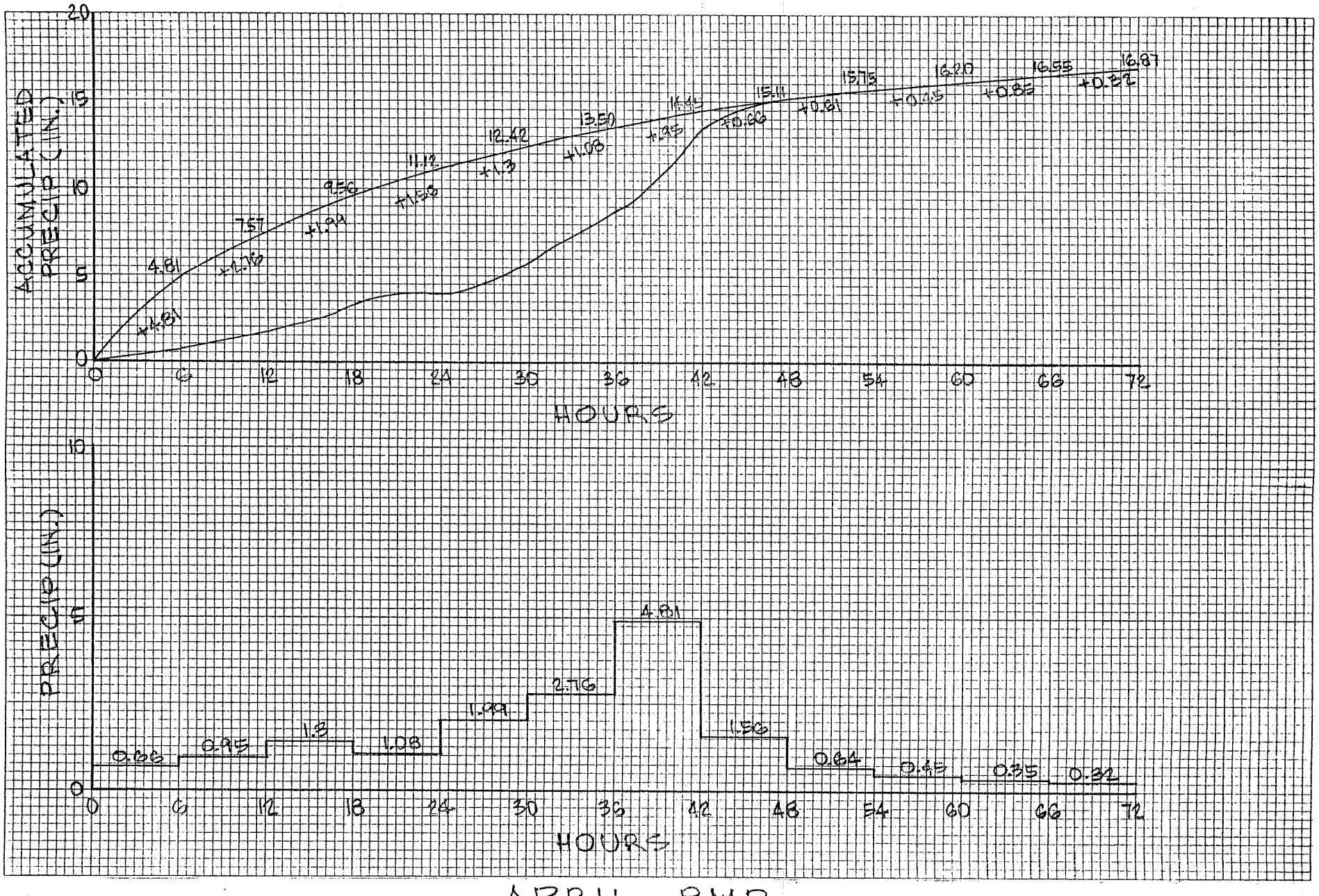
The August design storm event is shown on the following Figure. Note that the accumlated amounts for the HMR depth-duration curve and the "S" storm event distribution are shown. The storm events were distributed using the following distribution of 6-hour rainfall increments; 8, 7, 5, 6, 4, 2, 1, 3, 9, 10, 11, 12.

APRIL PMP EVENT

The April design storm event is shown on the following Figure. Note that the accumlated amounts for the HMR depth-duration curve and the "S" storm event distribution are shown. The storm events were distributed using the following distribution of 6-hour rainfall increments; 8, 7, 5, 6, 4, 2, 1, 3, 9, 10, 11, 12.


ACCUMULATED DEPTH (INCHES)
25
20
15
10
5
0
0
6
12
18
24
30
36
42
48
54
60
66
72
DURATION HOURS
7.21
10.88
13.54
15.00
17.25
18.65
19.90
20.94
21.80
22.40
22.90
23.22
+3.67
+2.66
+2.06
+1.65
+1.4
+1.25
+1.04
+0.86
+0.6
+0.5
+0.32
PRECIP (INCHES)
10
5
0
HOURS

AUGUST PMP


ACCUMULATED PRECIP (IN.)
20
15
10
5
0
0
6
12
18
24
30
36
42
48
54
60
66
72
HOURS
4.81
+4.81
7.57
+2.76
9.56
+1.99
11.12
+1.56
12.42
+1.3
13.50
+1.08
14.45
+0.95
15.11
+0.66
15.75
+0.64
16.20
+0.45
16.55
+0.35
16.87
+0.32
PRECIP (IN.)
10
5
0
0.66
0.95
1.3
1.08
1.99
2.76
4.81
1.56
0.64
0.45
0.35
0.32
HOURS

APRIL PMP

3. DESIGN EVENTS

DAM SAFETY CRITERIA

Storage of the Miner Flat Reservoir at the spillway crest elevation of 6062 feet equals approximately 8,260 acre-feet. The hydraulic height of the dam at this reservoir elevation equals 6062 - 5910 or 152 feet. Therefore, the structure falls into the commonly accepted criteria for a large classification.

The town of Whiteriver lies approximately 14 miles downstream. There are numerous other residences, campgrounds, and a Fish Hatchery between the dam site and Whiteriver. The failure of the dam could result in the loss of life and substantial property damage. Therefore, the project would fall under criteria for a high hazard structure.

A structure with a large size and high hazard classification must pass the Probable Maximum Flood. The stability design is based upon the criteria that the dam will not fail when overtopped by the PMF event which causes considerable ovetopping of the dam and inundation of the downstream area. The chute spillway on the dam face is capable of passing 12,300 cfs at elevation 6070 feet or greater than the 100-year event.

BASIN CONDITIONS FOR PMF EVENT

BASE FLOW

A flow of 120 cfs was used to represent base flow from the basin prior to the PMF event.

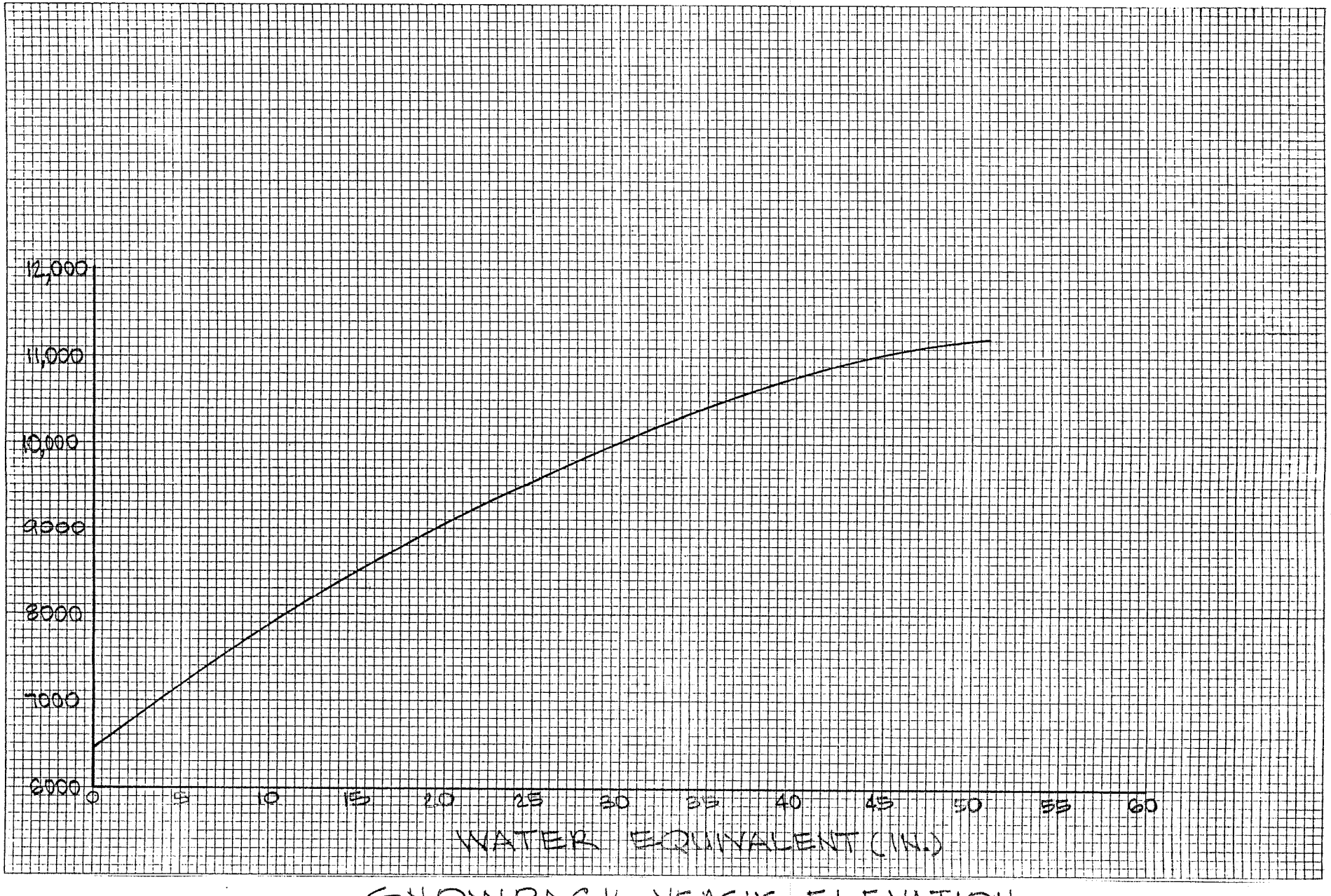
INFILTRATION LOSSES

Infiltration losses were determined using the December 1978 flood event at Gage 4940. The HEC-1 exponential loss rate function and a uniform loss rate were determined using the optimization routine of HEC-1. The uniform loss rate from this analysis equaled 0.147 inches per hour. This loss rate was reduced to 0.10 inches per hours as a worst case for the August PMF event. This was done to represent antecedent moisture conditions from a prior event and the reduced infiltration expected from the higher elevations found in the Miner Flat Dam basin.

An April PMF event would most likely occur with substantial snow cover conditions and frozen soil profiles. Therefore, loss rates for soil infiltration was set equal to zero for this event.

SNOWMELT CONDITIONS

Snowpack conditions for an April PMF were established at the maximum snowpack of record which occured during the winter of 1978 and 1979. The water equivalent of the snowpack on April 1st of 1979 ranged from 50 inches at elevations above 11,000 feet to 3 inches at elevations between 6 to 7,000 feet.



SNOWPACK VERSUS ELEVATION

The melt coeficient method was used to predict snowmelt during the April event. A melt coeficient of 0.16 was used to represent a mature spring snowpack. Drainage through the snowpack was maximized. A daily temperature pattern ranging from 60 degrees at noon to 40 degrees at 6 a.m. was used. None of the precipitation was allowed to fall as snow.

SPILLWAY DESIGN EVENT

Results of the August and April PMF events show flood peaks of respectively; 113,000 and 98,000 cfs. Because of the magnitude of PMF volume, there is insignificant reservoir routing. Both events will grossly overtop the dam and allow the resevoir elevations to rise for August and April events respectively: 6086.2 and 6084.5 feet. Maximum storage at these flows are respectively: 13, 580 and 10,950 acre-feet.

DAMBREAK ANALYSIS

- DAM FAILURE

A total dam failure was assumed which would completely remove the structure over a period of 0.5 hours. This case was assumed to develop the worst case dambreak flood wave. This case was assumed to develop the dambreak analysis. A concrete gravity dam would not be expected to fail in a manner this severe.

- ROUTING DATA

Thirty-one cross-sections were obtained for a channel reach 134,000 feet downstream of the dam to a point approximately 5 miles downstream of Canyon Day. These were developed from aerial photogrammetry and have a vertical accuracy of + or - one-half foot.

Storage versus flow relationships were developed using the HEC-2 water surface profiles model. Routing characteristics were developed for a channel reach using 3 to 6 cross-sections. The routing parameters was input into the HEC-1 model using storage, flow and elevation data for each reach.

- RESULTS

A dambreak at the peak of the August PMF event would result in a peak release of 570,000 cfs just below the site. This flood peak would take 1.5 hours to travel to the town of Whiteriver and would have a peak of 210,000 cfs. Flood depths would be on the order of 30 feet. The difference in stage between a PMF event at Whiteriver and a PMF with the failure of the dam would range from 3 to 5 feet.

A dambreak with the reservoir at the spillway elevation of 6062 feet and a base flow of 1000 cfs would result in a dambreak peak of 275,000 cfs. This flood peak would take 1.5 hours to travel to Whiteriver and would arrive with a flood peak of 50,000 cfs. This peak at Whiteriver would be greater than the peak of record at that point. The flood depths would be on the order of 18 feet. The floodplain would range from 600 to 1200 feet wide.

H. RESERVOIR DATA

1. SOURCE

Reservoir surface area and storage data was developed from mapping with a contour interval of 5 feet and at a scale of 1"=50 feet. Reservoir storage was determined using the conic method in the HEC-1 program with surface areas measured at each contour interval.

Storage at the dam crest elevation of 6062 feet is 8,620 acre-feet. Elevation versus surface area and storage are summarized on the following Tables and Figure.

The reservoir area will be cleared to an elevation of 6070 feet or 8 feet above the spillway crest elevation. Clearing to this elevation will limit the problems associated with floating debris to that which would occur naturally. A log boom will be located approximately 300 feet upstream from the spillway crest to detain floating debris. Periodic maintenance will be required to clean the debris collected.

The maximum fetch length for wind across the reservoir to the dam face is 0.43 miles. A 64 mile per hour design wind would develop wave heights of 1.9 feet and cause a wave runup of approximately 5 feet on a 2H:1V bank. Wind setup would be minor with less than 0.009 feet estimated.

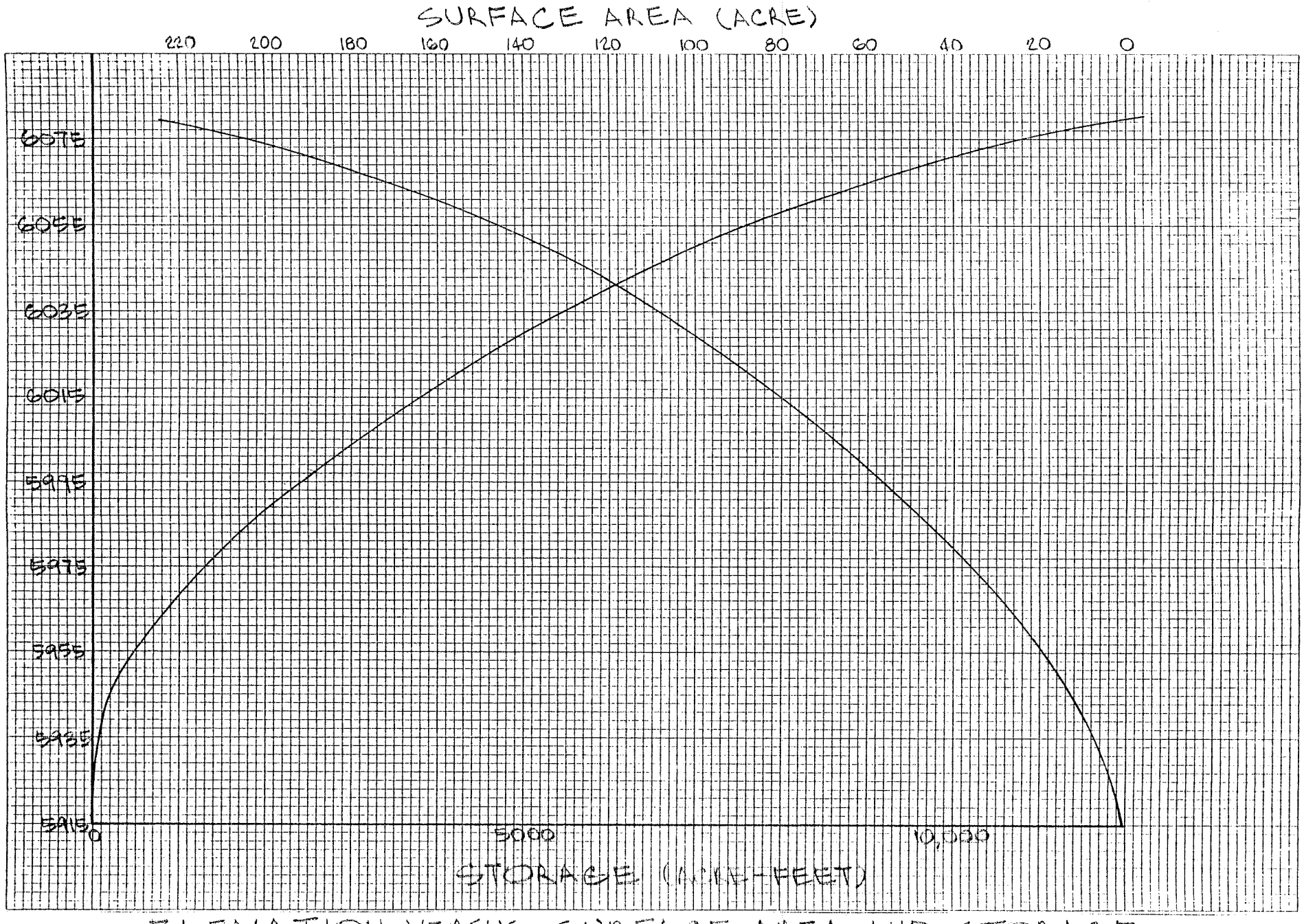
Normal freeboard with the resevoir at the spillway crest elevation of 6062 feet is 8 feet and is adequate for wave action. Note that the entire dam crest can be overtopped during a severe flood event.

SURFACE AREA VERSUS ELEVATION

Elevation (feet)	Surface Area (acres)
5915.00	0
5925.00	1.8
5930.00	3.3
5940.00	15.4
5950.00	29.1
6000.00	56.6
6050.00	126.1
6060.00	158.1
6065.00	176.0
6070.00	191.0
6075.00	212.6
6080.00	223.6

STORAGE VERSUS ELEVATION

Elevation (ft)	Storage (Acre-feet)
5915.00	0.0
5925.00	6.00
5930.00	18.49
5940.00	104.26
5950.00	323.21
6000.00	2429.14
6050.00	6881.66
6060.00	8299.60
6062.00	8622.80
6064.00	8960.21
6065.00	9134.35
6066.00	9311.83
6068.00	9676.00
6070.00	10052.63
6072.00	10443.85
6074.00	10851.91
6075.00	11062.36
6076.00	11276.07
6078.00	11710.04
6080.00	12152.82



ELEVATION VERSUS SURFACE AREA AND STORAGE

I. SPILLWAY AND OUTLET WORKS OPERATION

1. SPILLWAY

The spillway configuration consists of two components: 1) a 150 foot wide ogee crest serving as the principal spillway and 2) the remainder of the dam crest (150 feet) which will be overtopped during a severe event. The spillway crest elevation is 6062 feet and the dam crest is at elevation 6070 feet.

The ogee crest spillway rating table is shown on the following page. The rating was developed using COE design procedure with a design head of 20 feet. The capacity at the parapet wall crest of 6070 feet is 12,400 cfs. This capacity is greater than the 100-year event.

Hydraulic rating of the dam crest and the surrounding overbank areas was developed using the HEC-2 water surface profiles program. Note that the blocked dam crest section would function as a broad-crested weir.

Discharges during a PMF event would overtop the entire dam crest and reach into the overbank areas. Flows would be directed against the abutments. Because of the rock abutments, this is not considered a threat to the dam stability but would cause damages to the downstream structures.

SPILLWAY OPERATION

Operation of the spillway can be minimized , if desired to control nitrogen saturation, by use of the outlet works. The outlet capacity is sufficient to routinely discharge events up to a 2-year flood. Floods greater than a 2-year event can be allowed to use the spillway. Flows greater than the outlet capacity are generally less than 2 days in duration. Using this operating criteria would minimize nitrogen saturation problems to very brief periods occurring at a return period greater than 2-years.

A gaging station would be located downstream of the stilling basin and future powerhouse addition. This station would include a continuous recorder to provide streamflow records.

2. OUTLET

Maximum discharge of the 10 foot diameter outlet conduit is approximately 4,000 cfs or a 10-year flood event. Demand for air downstream of the gate was established at 6,900 cubic feet per second using findings in the COE Technical Report HL 84-2 and based upon recent hydraulic studies at Libby Dam, Montana.

MINER FLAT DAM

OGEE CREST RATING CURVE

CREST EL.	6062
DESHD	20
LENGTH	152
PDPTH	146
NPIERS	0

WSE	HE	HE/DESHD	CC	EC	COFQ	KP	KA	ZEFFEL	Q
6062	0	0	3.1000	.0000	3.1000	.123	-.008	152.00	0
6064	2	.1	3.2050	.0059	3.2428	.101	.023	152.00	1,394
6066	4	.2	3.3200	.0090	3.3799	.082	.045	152.00	4,110
6068	6	.3	3.4150	.0114	3.4933	.063	.062	152.00	7,804
6070	8	.4	3.5200	.0135	3.6157	.046	.074	152.00	12,436

NOTE: REMAINDER OF DAM CREST AND OVERBANK AREA BECOME EFFECTIVE ABOVE THIS ELEVATION

6072	10	.5	3.6170	.0155	3.7302	.034	.081	152.00	17,930
6074	12	.6	3.7100	.0174	3.8406	.026	.089	152.00	24,267
6076	14	.7	3.8000	.0191	3.9471	.017	.093	152.00	31,427
6078	16	.8	3.8800	.0208	4.0438	.009	.097	152.00	39,338
6080	18	.9	3.9430	.0224	4.1225	.003	.099	152.00	47,854
6082	20	1	4.0000	.0241	4.1963	0	.1	152.00	57,050
6084	22	1.1	4.0450	.0260	4.2596	-.006	.1	152.00	66,810
6086	24	1.2	4.0700	.0281	4.3038	-.012	.1	152.00	76,916
6088	26	1.3	4.0900	.0307	4.3474	-.013	.1	152.00	87,605

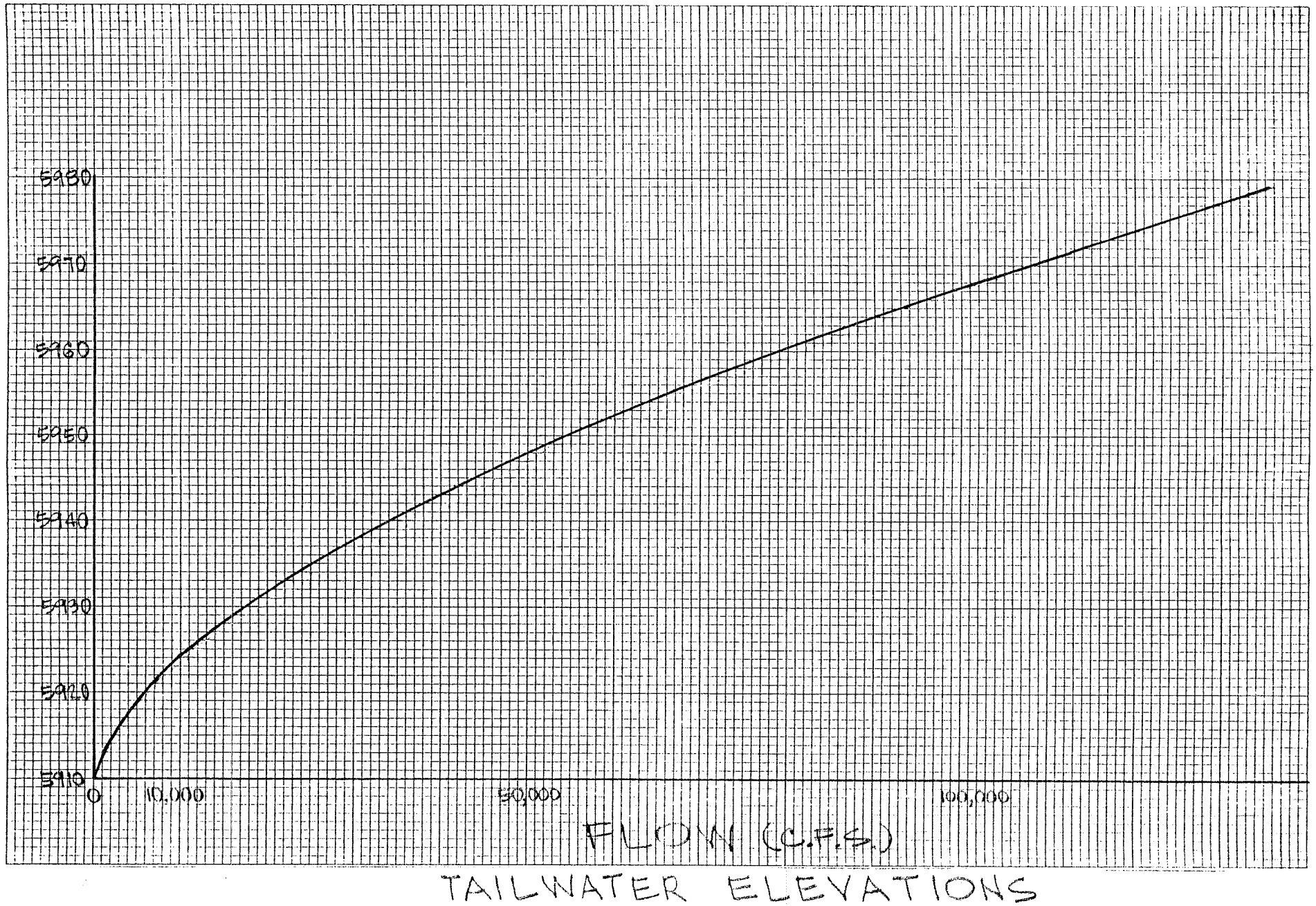
HYDROPOWER OUTLET

A separate 6 foot diameter penstock would serve the future powerhouse addition. The powerhouse would require a discharge of 190 cfs with reservoir elevations at the spillway crest (operating head of 150 feet) for a capacity of 2 megawatts. This flow equals a velocity of 7 feet per second in the penstock. Penstock velocities are being held to a minimum in consideration of friction losses and the potential for water hammer. The affect of water hammer will be a major factor due to the length of the penstock and the need to allow for sudden shutdown of the hydropower plant. Sudden shutdown of the hydropower plant has the potential to produce pressures in the range of 400 psi and will require a pipe wall thickness of at least 0.7 inch.

The hydropower facility will be designed and constructed during a later phase of the project not included in this work. It will be located on the left bank, approximately 295 feet downstream of the dam crest. Access to the plant from the highway will follow the roadway traversing the canyon area and across a bridge to the site area. This access route will be available for floods up to a 500-year frequency. The hydropower plant will require daily observation by a caretaker. It is recommended that remote sensing be provided to inform the caretaker at his residence of problems at the hydropower plant. The hdyropower plant will be remote to the caretaker's residence. It is recommended that it be automated to include shutdown of the plant and the control tower gate.

3. TAILWATER ELEVATIONS

Tailwater elevations are shown on the following Table and Figure. Elevations during a 2-year flood (1330 cfs) would equal 5913.6 feet or a depth in the stilling basin of 18.6 feet. During a 500-year event the elevation 5925.2 feet or a depth of 30.2 feet. Tailwater elevation during a PMF event would be 5972 feet or a depth of 77 feet. Significant inundation damage will occur to the proposed powerhouse during a PMF event. Events up to a 100-year flood will not threaten the powerhouse with finished floor elevation at or above elevation 5925 feet.



TAILWATER ELEVATIONS

TAILWATER ELEVATIONS

Flow	Elevation
1330	5913.6
3375	5916.8
6000	5919.9
7300	5921.2
11750	5925.0
14000	5926.7
25000	5933.8
40000	5941.3
52500	5948.5
67500	5954.9
105000	5969.0
135000	5979.0

4. STILLING BASIN

The stilling basin will consist of an area of approximately 90 feet in length by 200 feet in width with an invert elevation of 5895 feet. Tailwater depth at low flows will be approximately 15 feet.

During the 500-year flood event discharges down the spillway will equal 90 cfs per foot of width at a velocity of 82 feet per second and a depth of 1.13 feet. The Froude number equals 13.6. Procedures found in the USBR Engineering Monograph No. 25 indicate for a Type II basin that a minimum tailwater depth of 19 feet and a basin length of 81 feet would be required. Note that the basin would not include energy dissipation blocks. Additional tailwater depth and basin length is used to offset the elimination of blocks. Actual tailwater depth is on the order of 30 feet. The basin end sill is approximately 15 feet in height. The proposed basin does not follow the Type II configuration exactly but is expected to give performance equal to a basin with an end sill and without blocks on the floor.

The rock invert is judged to be resistant to erosion and will maintain the downstream invert elevations and the stilling basin end sill. Additionally, if a PMF event were to occur, the flows over the canyon walls would not cause failure of the dam due to flow over the rock abutments.

5. INTAKE TOWER

The intake tower will use 2 primary gates: 1) an 8'x8' roller gate on the upstream end of the outlet conduit and 2) a 5'x5' roller gate on the hydropower penstock. These gates will be operated by hydraulic controls and will use an electronic position indicator. The hydropower penstock will remain closed until the hydroplant is added at a later phase of the project.

The intake tower will function as a wet well with 3 roller gates mounted on the outside wall or upstream face at three different

elevations. These secondary gates have two purposes: 1) to serve as emergency gates and allow dewatering of the wet well and 2) to control the temperature of releases from the reservoir.

Release temperatures can be controlled by opening the selected secondary gate or by a combination of secondary gates open to provide a mixture of temperatures. The bottom gate will have an invert elevation of 5910 feet and an opening of 8'x 8'. The two remaining gates will have 5'x 5'openings at invert elevations of 6057 and 6027 feet. These gates can be operated to provide for withdrawals from one temperature regime (reservoir elevation) or a mixture of 2 or 3 regimes. These gate elevations will allow intake of water from a wide range of reservoir elevations. The totall depth at the intake structure is normally 150 feet. The reservoir will be operated so that the tower structure is normally full of water (wet well), with the rate of discharge controlled by the primary gates on the outlet and penstock.

6. COFFER DAM AND STREAM DIVERSION DURING CONSTRUCTION

The hydraulic capacity of the coffer dam is 1400 cfs or approximately equal to a 2-year flood event. The construction of the base of the dam which will require an estimated period of at least 60 days.

Failure of the coffer dam during a flood event would cause a dam break wave which would dissipate significantly in the first 5 miles. Note that the coffer dam crest is at elevation 5930 feet and reservoir storage at that elevation is approximately 20 acre-feet. When the construction of the dam rises to this elevation or above, the concrete gravity section will control the maximum reservoir elevation during a flood which exceeds the diversion capacity.

The temporary coffer dam and stream diversion through the 10 foot diameter outlet has a capacity approximately equal to a 2-year flood event. The foundation preparation and initial placement of rollcrete or concrete will occur during the late winter and early spring months as shown on the construction schedule. This period normally produces the peak runoff for the season.

Therefore, the stream diversion capacity could be exceeded during the period of construction when the dam foundation is being prepared and the initial concrete lifts are placed. If the coffer dam diversion capacity were exceeded during the early phases, the foundation area would be inundated and flood damages would occur. Damages would include clean-up of the foundation area and faces between concrete lifts. It is recommended that the coffer dam be of rockfill construction with an impermeable membrane lining to control seepage. A damaging failure of the coffer dam would be unlikely under most circumstances due to the limited storage capacity and rockfill construction.

Several options were investigated to reduce the flood inundation and damage potential during the period of foundation preparation and early concrete placement. A twin or parallel outlet conduit

was eliminated due to the expense of construction estimated at approximately $225,000. Additional coffer dam height could be used to increase the capacity slightly. A large increase from the 2-year to a 10-year capacity would require approximately 60 feet in head and would not be feasible for a coffer dam.

The approach recommended is realistically describe the temporary stream diversion capacity and limitations and require the contractor to provide a stream diversion plan addressing flood events in excess of the capacity. It is suggested that the contractor may elect to provide for; 1) additional capacity using a temporary open channel or pipe or 2) increase his bid price for foundation preparation and concrete placement to include a contingency for damages and repetitive effort due to flood inundation.

A draft stream diversion specification follows:

BID ITEM __, DIVERSION AND CARE OF STREAM

(1) This item shall consist of all work necessary for diverting all stream flows of the White River during the period of construction. The Contractor shall be responsible for the temporary diversion of all stream flows to protect the temporary and permanent works of improvement.

A coffer dam and 10 foot diameter pipe as shown on the plans will provide a maximum unobstructed flow capacity of 1400 cfs. This capacity is approximately equal to a 2-year flood capacity and could be expected to be exceeded on the average once every two years. There is a significant probability that the temporary stream diversion capacity provided by the coffer dam and 10 foot diameter pipe could be exceeded during the period of construction.

Flood peaks at selected recurrence intervals which could be expected on the White River at the construction site are shown below. A 2-year flood peak, on the average, could be expected to have a 1 in 2 chance of occuring in any given year. A 5-year flood would have a 1 in 5 chance of occuring in any given year. Note that nature does not necessarily follow a normal distribution of flood events. Flood events of all recurrence intervals have been known to occur back to back at random locations in all areas of the world. There is no way to predict the flood events to be expected during the construction period.

FLOOD PEAKS AT MINER FLAT DAM, (cfs)

RECURRENCE INTERVAL	2-YEAR	5-YEAR	10-YEAR	50-YEAR	100-YEAR	500-YEAR
FLOOD PEAK	1,420	2,600	3,300	6,500	8,200	13,500

(2) The Contractor will be required to submit a plan for approval by the Engineer for the temporary diversion of stream flows of the White River at the construction site using the coffer dam and 10 foot diameter pipe shown on the plans.

(3) The Contractor will be required to submit a plan for approval by the Engineer for the temporary diversion of flood flows of the White River at the construction site in excess of the coffer dam and 10 foot diameter pipe capacity.

Acceptable concepts may include additional pipe capacity, open channel overchute, or a do nothing additional alternative with acknowlegement and acceptance of the potential for flood damage risk by the Contractor.

8. DOWNSTREAM WARNING PLAN

It is recognized that a downstream warning plan will be required for the project. The plan is not provided for in the scope of work for the dam design and will be provided for under a future contract and scope of work. This plan will include the dambreak flood profile, inundation mapping and travel times for a reach approximately 25 miles downstream of the project. The warning plan will be provided to the local administrator of disaster and emergency services and law enforcement agencies for use in the event of a dambreak.

9. OPERATION AND MAINTENANCE PLAN

It is recognized that an operation and maintenance manual will be required for the project. This manual is not provided for in this scope of work for the dam design and will be done under a future contract and scope of work. It will include a reservoir operation plan, flood release table, and maintainence and operation guidelines for the project including inspection guidelines, outlet gates and controls, concrete repairs, clearing of reservoir debris, stream gage operation, road maintainence, and tram hoist maintenance.

The following computations are not submitted as part of this Design Memorandum, but will be available if requested.

- WORKSHEETS FOR AUGUST, APRIL, THUNDERSTORM PMP
- HEC-1 OPTIMIZATION OF DECEMBER 1978 EVENT
- AUGUST PMF HEC-1 OUTPUT
- APRIL PMF HEC-1 OUTPUT
- DAMBREAK HEC-1 OUTPUT

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APPENDIX S

Design Memorandum, Miner Flat Dam, February 1987

MORRISON-MAIERLE, INC.,
VOLUMES I THRU V OF VI

FEBRUARY 2007

DESIGN MEMORANDUM

MINER FLAT DAM

DECEMBER, 1986

WHITE MOUNTAIN APACHE TRIBE

WHITERIVER, ARIZONA

VOLUME 5 OF 6

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MINER FLAT DAM
DESIGN MEMORANDUM

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DESIGN MEMORANDUM

SECTION J

J.1 DAM DESIGN - GENERAL

A conservative approach has been taken in design. The concept is to provide a very stable, permanent, simple, and low maintenance dam without complex construction.

Although a rock fill dam could be used, it has unique problems resulting from the vally shape, there was concern early in exploration about the extent of sound foundation rock, and a major separate spillway structure could be required.

An embankment or fill dam was considered unacceptable because of the lack of suitable materials, the valley shape, and its potential for erosion.

A concrete gravity dam is most desireable because of permanence, low maintenance, available good sources for aggregate, the ability to put the spillway over the structure, and the valley shape. However, as discussed in the cost and schedule sections J.6 and J.7, it would be expensive if built by conventional methods. Fortunately then is an alternate method that can be used to place mass concrete in a gravity dam. When applied to Miner Flat, this reduces the estimated conventionally built concrete gravity dam cost form about $15 million to $9 million. The resulting structure is still a very stable mass concrete gravity dam. The construction method is called "Roller Compacted Concrete" or "RCC". This procedure uses heavy earthmoving equipment to transport and place the material in a very efficient, rapid, and economical manner. RCC has recently been used very successfully to save time and money on a number of projects throughout the world. Within the United States it has become a common dam construction method. This includes projects by both the U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation.

The RCC Dam will require normal construction preparations, but only about two months for it to then be raised in a "Round the clock" continuous operation without verticle joints typically needed with conventional placement methods. This is technically and practically very achievable within the controls stipulated in the construction specifications of Section J.8.

Even with the savings achieved by using RCC, it has been possible to keep stresses to very low levels (lower than in the conventional approach), add extra mass, and build redundency into the design which assures better performance with higher factors of safety. Some of this could be deleted, but for the modest premium in cost, it is considered appropriate for this project. For example, internal drains have been included throughout the entire structure, a special bedding mix is included near the upstream face between layers of the RCC to provide extra strength and water tightness, and a very heavy duty permanent independent

water barrier is provided at the upstream face. Even without the barrier, the dam design is safe and complete. With it, seepage will be minimal or none existent, and factors of safety for stability increases substantially.

The water barrier is described in detail in the specifications of Section J.8. It is a thick, continuous, flexible sheet of PVC or Hi Density Polyethylene. In essence, it is a water stop from the top to the bottom of the dam and from abutment to abutment.

The upstream face of the dam will consist of attractive, high quality precast concrete panels that protect the RCC and water barrier, and act as forms during construction. This technique has worked very well in previous projects.

The downstream face and top of the dam will be constructed using durable conventional concrete formulated to have minimal shrinkage. This will be placed monolithically with the RCC so there is no distinctive joint between them. This technique has also been used successfully at other projects. It is discussed further in Section J.5. The result will be conventional concrete fully encasing an economical mass of stable Roller Compacted Concrete.

The detailed preliminary designs for a conventional concrete and RCC dome have been made for this Design Memorandum to provide sufficient details for realistic cost estimates, construction schedules, and to provide the basis for final designs. The preliminary designs are the basis for the drawings included in Section J.15.

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DESIGN MEMORANDUM

SECTION J

J.2 DAM DESIGN AND STABILITY ANALYSIS

The following information and summaries are provided as design criteria for final design and to show methodology in arriving at the recommended dam sections. Detailed computations and computer runs have not been encluded in this Memorandum due to the volume of those computations. All computations have been retained by the designers for back-up to this Memorandum and may be reviewed if required.

DESIGN AND LOADING CONDITIONS EVALUATED

IN GENERAL:

United States Bureau of Reclamation Design Criteria were used.

USBR "Design of Gravity Dams" 1976
USBR "Design of Small Dams" 1977

R = Roller Compacted Placement Method used in construction
C = Conventional Placement Method used in construction
V = Verticle
H = Horizontal
Y = Yes
N = No

Empty Load Conditions:
- Minimum required factor of safety = 3
- No reservoir
- Deadload of concrete mass
- Dam base at elevation 5907
- Dam cost at elevation 6062 (spillway)
- No silt
- No tailwater
- Wind pressure (horizontal) against the downstream face = 1.6 psf
- No uplift pressure
- No earthquake

Usual Load Conditions:
- Minimum required factor of safety = 3
- Normal Reservoir Evaluation of 6062 (Spillway Crest)
- Dead Load of Concrete Mass
- Dam Base at Elevation 5907
- Dam Crest at Elevation 6062 (Spillway)
- Silt - Accumulation to Elevation 5950
- Silt - Produces an effective horizontal equivalent of a fluid in mass at 85 pcf
- Tailwater at elevation 5912
- Applicable uplift pressures applied along lift joint surfaces
- No earthquake

Unusual Load Conditions:	Minimum required factor of safety = 2 Reservoir level at PMF of 6082 Dead Load of Concrete Mass Dam Base at Elevation 5907 Dam Crest at Elevation 60602 (Spillway) Silt Accumulation at elevation 5950 Silt produces an effective horizontal equivalent of a fluid mass at 85 pcf Tailwater at elevation 5979 Applicable uplift pressures applied along L ft joint surfaces No earthquake
Extreme Load Condition:	Minimum required factor of safety = 1 Normal reservior elevation at 6062 (spillway crest) Dead Load of concrete mass Dam base at elevation 6907 Dam crest at elevation 6062 (spillway) Silt accumulated to elevation 6960 Silt produces an effective horzontal equivalant of a fluid mass at 85 pcf Tailwater at elevation 6912 Applicable uplift pressures with and without drains openable Earthquake horizontal acceleration = 0.10 x gravity Earthquake vertical acceleration - 0.05 x gravity
Elevations Studied:	Stresses and/or factors of safety were determined at approximately 10 foot increments (9.8425 feet exact) in elevation starting at the assumed foundation of 5907 and progressing up to the spillway crest elevation of 6062.
Approach:	Two approaches were used to assist in development and selections of the final design sections for both the conventional and roller compacted gravity dams. One method used specific values for shear strength (along the joint interface), compressive strength, density, and tensile strength. The resulting factors of safety were then calculated and compared to the minimum required factors of safety for the different load conditions. The other method set the factors of safety at their minimum acceptable levels and then calculated the material properties or strengths necessary to achieve them. The different situations evaluated are summarized in the following tabulations. Properties used for the

different concretes to determine the resulting factors of safety are as shown below, or are indicated on the table when a different strength was studied. A very conservative approach was used in estimated strengths. Where bedding mix, internal drains, and a membrane were included in various designs studied, the efficiency at which they were considered is shown in the summary. For example, the upstream membrane is not required for stability (efficiency = 0%). If considered, it provides extraordinary factors of safety where fully effective (efficiency = 100%). The drains and bedding mix were considered to reduce uplift to varying degrees ranging from 0% to 67% of the difference between the reservoir and tailwater head pressure. Again they are not fully necessary and result in very substantial factors of safety where considered.

MATERIAL PROPERTIES FOR ROLLER COMPACTED CONCRETE

PROPERTY	RCC		BEDDING MIX	
	Probable	Used in Computations	Probable	Used in Computations
Density	153+ pcf	153 pcf	153 pcf	153 pcf
Tensile Strength (usual - 6 mo±)	35 psi	0 psi	160 psi	0 psi
Tensile Strength (unusual - 1 yr+)	40 psi	0 psi	185 psi	0 psi
Tensile Strength (extreme - 1 yr+)	55 psi	0 psi	240 psi	0 psi
Shear (usual - 6 mo+)	100 psi @ 45°	80 psi @ 40°	200 psi @ 45°	80 psi @ 40°
Shear (unusual - 1 yr+)	115 psi @ 50°	90 psi @ 45°	230 psi @ 50°	90 psi @ 45°
Shear (extreme - 1 yr+)	155 psi @ 55°	120 psi @ 45°	300 psi @ 55°	120 psi @ 45°
Compression (usual - 6 mo±)	1600 psi	1400 psi	2000 psi	1400 psi
Compression (unusual - 1 yr+)	1800 psi	1600 psi	2400 psi	1600 psi
Compression (extreme - 1 yr+)	2200 psi	1600 psi	2800 psi	1600 psi

MATERIAL PROPERTIES FOR CONVENTIONAL MASS CONCRETE

PROPERTY	Conventional Mass Concrete	
	Probable	Used in Computations
Density	153 pcf	153 pcf
Tensile Strength (usual - 1 yr)	150 psi	0 psi
Tensile Strength (unusual - 1 yr+)	175 psi	0 psi
Tensile Strength (extreme - 1 yr+)	233 psi	0 psi
Shear (usual - 1 yr)	250 psi @ 45°	250 psi @ 45°
Shear (unusual - 1 yr+)	300 psi @ 50°	300 psi @ 45°
Shear (extreme - 1 yr+)	400 psi @ 55°	400 psi @ 45°
Compression (usual - 1 yr)	2500 psi	1600 psi
Compression (unusual - 1 yr+)	3000 psi	1800 psi
Compression (extreme - 1 yr+)	3800 psi	1800 psi

*unless otherwise noted.

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DESIGNS AND LOADING CONDITIONS INVESTIGATED

Case	Placement Method	Loading Conditions	Downstream Slope H:1.0	Upstream Face	Gallery	Membrane		Found Drain		Internal Drains		Bedding		Grout Curtain	
						Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency
1	R	Usual	.80	V	Y	N	0%	Y	67%	Y	67%	Y	67%	Y	0%
2	R	Usual	.80	V	Y	N	0%	Y	67%	Y	67%	Y	67%	Y	0%
3	R	Extreme	.80	V	Y	N	0%	Y	67%	Y	67%	Y	67%	Y	0%
4	R	Empty	.80	V	Y	N	0%	Y	67%	Y	67%	Y	67%	Y	0%
5	R	Usual	.75	V	Y	N	0%	Y	67%	Y	67%	Y	67%	Y	0%
6	R	Unusual	.75	V	Y	N	0%	Y	67%	Y	67%	Y	67%	Y	0%
7	R	Extreme	.75	V	Y	N	0%	y	67%	Y	67%	Y	67%	Y	0%
8	R	Empty	.75	V	Y	N	0%	Y	67%	Y	67%	Y	67%	Y	0%
9	R	Usual	.80	V	Y	N	0%	Y	67%	N	0%	Y	67%	Y	0%
10	R	Unusual	.80	V	Y	N	0%	Y	67%	N	0%	Y	67%	Y	0%
11	R	Extreme	.80	V	Y	N	0%	Y	67%	N	0%	Y	67%	Y	0%
12	R	Usual	.80	V	Y	N	0%	Y	67%	Y	67%	N	0%	Y	0%
13	R	Usual	.80	V	Y	N	0%	Y	67%	Y	67%	N	0%	Y	0%
14	R	Extreme	.80	V	Y	N	0%	Y	67%	Y	67%	N	0%	Y	0%
15	R	Usual	.80	V	N	N	0%	N	0%	N	0%	N	0%	N	0%
16	R	Unusual	.80	V	N	N	0%	N	0%	N	0%	N	0%	N	0%
17	R	Extreme	.80	V	N	N	0%	N	0%	N	0%	N	0%	N	0%
18	R	Usual	.80	Y	N	Y	90%	N	0%	N	0%	N	0%	N	0%
19	R	Unusual	.80	Y	N	Y	90%	N	03%	N	0%	N	0%	N	0%
20	R	Extreme	.80	Y	N	Y	90%	N	0%	N	0%	N	0%	N	0%
21	R	Usual	.75	V	N	Y	90%	N	0%	N	0%	N	0%	N	0%
22	R	Unusual	.75	V	N	Y	90%	N	0%	N	0%	N	0%	N	0%
23	R	Extreme	.75	V	N	Y	90%	N	0%	N	0%	N	0%	N	0%
24	R	Usual	.75	V	Y	N	0%	N	0%	N	0%	Y	67%	N	0%
25	R	Unusual	.75	V	Y	N	0%	N	0%	N	0%	Y	67%	N	0%
26	R	Extreme	.75	V	Y	N	0%	N	0%	N	0%	Y	67%	N	0%
27	R	Usual	.75	V	Y	N	0%	Y	67%	Y	67%	N	0%	N	0%
28	R	Unusual	.75	V	Y	N	0%	Y	67%	Y	67%	N	0%	N	0%
29	R	Extreme	.75	V	Y	N	0%	Y	67%	Y	67%	N	0%	N	0%
30	R	Usual	.72	V	Y	N	0%	Y	67%	Y	67%	N	0%	N	0%
31	R	Unusual	.72	V	Y	N	0%	Y	67%	Y	67%	N	0%	N	0%
32	R	Extreme	.72	V	Y	N	0%	Y	67%	Y	67%	N	0%	N	0%
33	R	Empty	.72	V	Y	N	0%	Y	67%	Y	67%	Y	67%	N	0%
34	R	Usual	.72	V	Y	N	0%	Y	67%	Y	67%	Y	67%	N	0%
35	R	Unusual	.72	V	Y	N	0%	Y	67%	Y	67%	Y	67%	N	0%
36	R	Extreme	.72	V	Y	N	0%	Y	67%	Y	67%	Y	67%	N	0%

DESIGNS AND LOADING CONDITIONS INVESTIGATED
(continued)

Case	Placement Method	Loading Conditions	Downstream Slope H:1.0	Upstream Face	Gallery	Membrane		Found Drain		Internal Drains		Bedding		Grout Curtain	
						Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency
37	R	Usual	.72	V	Y	Y	0%	Y	67%	Y	67%	Y	67%	Y	0%
38	R	Unusual	.72	V	Y	Y	0%	Y	67%	Y	67%	Y	67%	Y	0%
39	R	Extreme	.72	V	Y	Y	0%	Y	67%	Y	67%	Y	67%	Y	0%
40	R	Usual	.80	V	Y	Y	0%	Y	67%	Y	67%	Y	67%	Y	0%
41	R	Unusual	.80	V	Y	Y	0%	N	67%	N	67%	N	67%	N	0%
42	R	Extreme	.80	V	Y	Y	0%	N	67%	N	67%	N	67%	N	0%
43	R	Empty	.80	V	Y	Y	0%	N	67%	N	67%	N	67%	N	0%

* 37 through 43 used friction angle = 45°, shear Cohesion = 80 psi, tensile strength (Bedding) = 15 psi; Compressive Strength = 1750 psi.

DESIGNS AND LOADING CONDITIONS INVESTIGATED
(continued)

Case	Placement Method	Loading Conditions	Downstream Slope	Upstream Face	Gallery	Internal Drains Y/N	Internal Drains Efficiency	Age	F'c (psi)	Shear F't (psi)	Shear C (psi)
44	C	Usual	.72	V	N	N	0%	6 mo.	1600	0	250
45	C	Unusual	.72	V	N	N	0%	1 yr+	1800	0	300
46	C	Extreme	.72	V	N	N	0%	1 yr+	1800	0	400
47	C	Usual	.75	V	N	N	0%	6 mo.	1600	0	250
48	C	Unusual	.75	V	N	N	0%	1 yr+	1800	0	300
49	C	Usual	.75	V	N	N	0%	6 mo.	1600	0	250
50	C	Unusual	.75	V	N	N	0%	1 yr+	1800	50	300
51	C	Usual	.72	V	Y	Y	67%	6 mo	1600	0	250
52	C	Unusual	.72	V	Y	Y	67%	1 yr+	1800	o	300
53	C	Extreme	.72	V	Y	Y	67%	1 yr+	1800	0	400
54	C	Empty	.72	V	Y	Y	67%	6 mo.	1600	0	250

DESIGNS AND LOADING CONDITIONS INVESTIGATED
(continued)

Case	Placement Method	Loading Conditions	Downstream Slope	Upstream Face	Gallery	Internal Drains Y/N	Internal Drains Efficiency	Age	F'c (psi)	F't (psi)	Shear C (psi)
55	C	Usual	.72	V	Y	Y	67%	6 mo.	1600	0	200
56	C	Empty	.72	V	Y	Y	67%	6 mo.	1600	0	200
57	C	Unusual	.72	V	Y	Y	67%	1 yr+	1800	0	225
58	C	Extreme	.72	V	Y	Y	67%	1 yr+	1800	0	300
59	C	Usual	.72	V	Y	N	0%	6 mo.	1600	100	200
60	C	Unusual	.72	V	Y	N	0%	1 yr+	1800	117	225
61	C	Extreme	.72	V	Y	N	0%	1 yr+	1800	156	300
62	C	Usual	.72	V	Y	N	0%	6 mo	1600	50	200
63	C	Unusual	.72	V	Y	N	0%	1 yr+	1800	53	225
64	C	Extreme	.72	V	Y	N	0%	1 yr+	1800	78	300
65	C	Unusual	.72	V	Y	N	0%	1 yr+	1800	58	225
66	C	Unusual	.72	V	Y	N	0%	1 yr+	1800	0	225

DESIGNS AND LOADING CONDITIONS INVESTIGATED
(continued)

Case	Placement Method	Loading Conditions	Downstream Slope H:1.0	Upstream Face	Gallery	Membrane		Found Drain		Internal Drains		Bedding		Grout Curtain	
						Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency	Y/N	Efficiency
67	R	Usual	.77	V	Y	Y	0%	Y	67%	Y	67%	Y	0%	Y	0%
68	R	Unusual	.77	V	Y	Y	0%	Y	67%	Y	67%	Y	0%	Y	0%
69	R	Extreme	.77	V	Y	Y	0%	Y	67%	Y	67%	Y	0%	Y	0%
70	R	Empty Res	.77	V	Y	Y	0%	Y	67%	Y	67%	Y	67%	Y	0%
71	R	Unusual	.77	V	Y	Y	0%	Y	67%	Y	67%	Y	67%	Y	0%
72	R	Unusual	.77	V	Y	Y	0%	Y	67%	Y	67%	Y	67%	Y	0%
73	R	Extreme	.77	V	Y	Y	0%	Y	67%	Y	67%	Y	67%	Y	0%
74	R	Usual	.77	V	Y	Y	90%	Y	67%	Y	0%	Y	0%	Y	0%
75	R	Unusual	.77	V	Y	Y	90%	Y	67%	Y	0%	Y	0%	Y	0%
76	R	Extreme	.77	V	Y	Y	90%	Y	67%	Y	0%	Y	0%	Y	0%
101	R	Empty	.77	V		Y	0%	Y	67%	Y	67%	Y	0%	Y	0%
102	R	Usual	.77	V		Y	0%	Y	67%	Y	67%	Y	0%	Y	0%
103	R	Unusual	.77	V		Y	0%	Y	67%	Y	67%	Y	0%	Y	0%
104	R	Extreme	.77	V		Y	0%	Y	0%	Y	0%	Y	0%	Y	0%

\LOADCOND.MFD

J.3 DAM THERMAL STRESS STUDIES

GENERAL

A major factor in any Mass Concrete Dam is the internal stress and related cracking potential due to temperature changes. There are essentially two problems:

(1) Long term internal tensile stresses within the mass or body of the dam. These relate to contraction of the mass as it undergoes slow long term cooling from the elevated early temperatures because of heat generation during the hydration process. When the contraction is resisted or restrained by things such as the foundation or portions of the structure that do not cool and contract, a resulting tensile stress develops.

(2) Tensile stresses that develop near the surface if it is subjected to rapid cooling such as during a cold night while the interior just below the surface remains warm. This stress is removed when the surface warms again, for example, when the sun shines on it the next morning. This is often referred to as thermal shock.

In both cases, cracking will develop if the tensile stress from thermal contraction exceeds the tensile strength or strain capacity at that time. In the case of thermal shock, the concrete may be at a very early age and consequently have a low strength. In the case of the long term deep internal stresses, the strength can be much higher and it can be relieved somewhat by creep.

The "Stress Coefficient" is a convenient tool to help evaluate thermal stresses. It is a material property value unique to each mix and applicable to the time period during which the temperature change takes place. Specifically, the stress coefficient tells how much tensile stress (psi) will develop in a given restrained mix for every degree of temperature drop that it undergoes during the time period under question. It considers creep, the modulus of elasticity, and the coefficient of thermal expansion. For the aggregate materials and probable mix proportions at miner Flat, stress coefficients similar to the following can be expected for concrete subjected to the start of thermal loading at seven (7) days.

STRESS COEFFICIENTS

Roller Compacted Concrete	5 - 7 psi/F degree
Conventional Mass Concrete	10 - 12
Spillway Facing	15 - 18
Conventional Concrete (C.I.P.)	
Upstream Face	15 - 18

These values are consistent with similar mixes of similar material composition at other projects. The values will be verified for the exact mix designs used at Miner Flat when they are finalized.

Thermal stresses are thus a function of the material properties and the quantity or extent of the temperature drop. The temperature change that will occur depends on many factors for each set of circumstances. These include:

1. Adiabatic heat rise generated from cement hydration;
2. Ambient Conditions (wind, temperatures and their fluctuation from season to season as well as within each day);
3. The rate of concrete placement;
4. The temperature of the mix when it is placed;
5. The effect of surrounding materials such as foundation rock, forms, and insulation;
6. Delays in construction due to rain or other circumstances;
7. The lift height or depth of each concrete placement;
8. The time between placements.

For the Miner Flat Project, an adiabatic rise of 37.3 F degrees was applied to the conventional mass concrete and a rise of 26.0 F degrees was used in the Roller Compacted Concrete.

A proven finite element analysis method for determining temperatures and their rate of change was made for both the conventional mass concrete and the RCC dams. The computor program takes into account the influencing factors listed above. Based on results of these studies, controls for constructions were established that minimize thermal stresses to tolerable levels for the designs recommended. These in turn provided the basis for the cost estimate and a realistic schedule as discussed in Sections J.6 and J.7. The Controls are "in-line" with controls used in the past on projects of a similar nature with similar materials and under similar environments.

For the RCC dam, the construction controls are detailed in the enclosed preliminary specifications (Section J.8). A sample of resulting temperatures at selected points of interest in the dam is shown in the following Figure # J.3A.

For the Conventional Concrete, the following controls were used in the thermal study.

- Placement allowed from March 15 to November 15
- Surface Insulation used as needed to prevent excessive thermal shock in the fall
- Maximum placing temperature 50° - 55° F
- Five foot lift heights
- Five days between placements.

A sample of resulting temperatures at selected points of interest in the dam is shown in Figure # J.3B.

It is worth noting that the precast concrete panels used in the upstream face of the recommended RCC dam design provides permanent insulation to the RCC mass and reduces the "thermal shock" to which the dam mass is subjected. This benefit is not realized in the conventional mass concrete dam. It is also worth noting that minor and shallow "thermal shock" cracking on the top of the dam and at the downstream face (but not transverse to the spillway flow) is a tolerable condition.

\MINERFLATDAM\STRSSTUD.MFD

ROLLER COMPACTED CONCRETE

120 LBS. TYPE II CEMENT PER CUBIC YARD
PLACING SCHEDULE AS DISCUSSED IN SECTION J.3

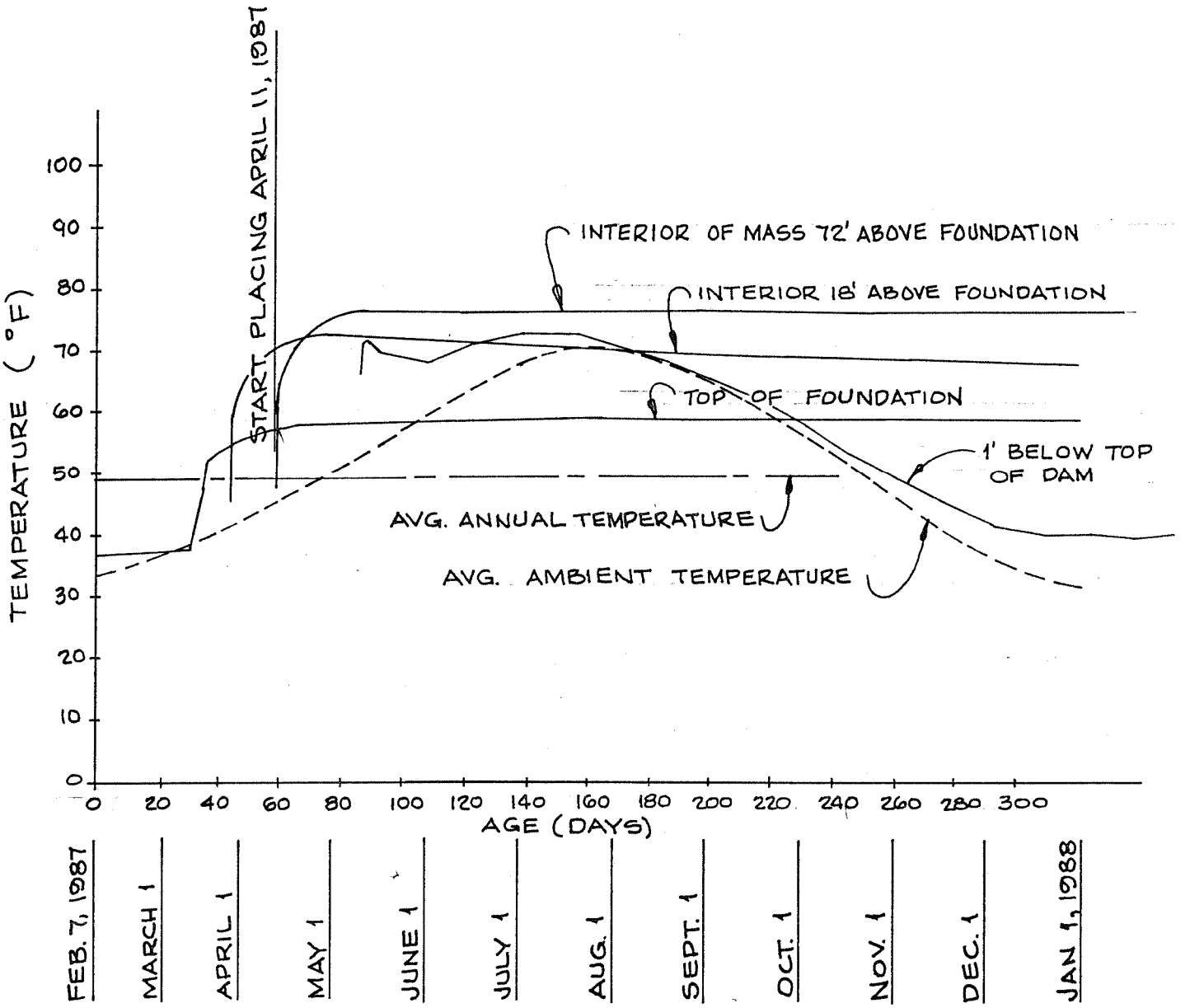

FIGURE J.3A

CONVENTIONAL MASS CONCRETE

185 LBS. TYPE II CEMENT PER CUBIC YARD
80 LBS. FLY ASH PER CUBIC YARD
TEMPERATURE OF MIX WHEN PLACED=50°F
5 DAYS BETWEEN LIFTS

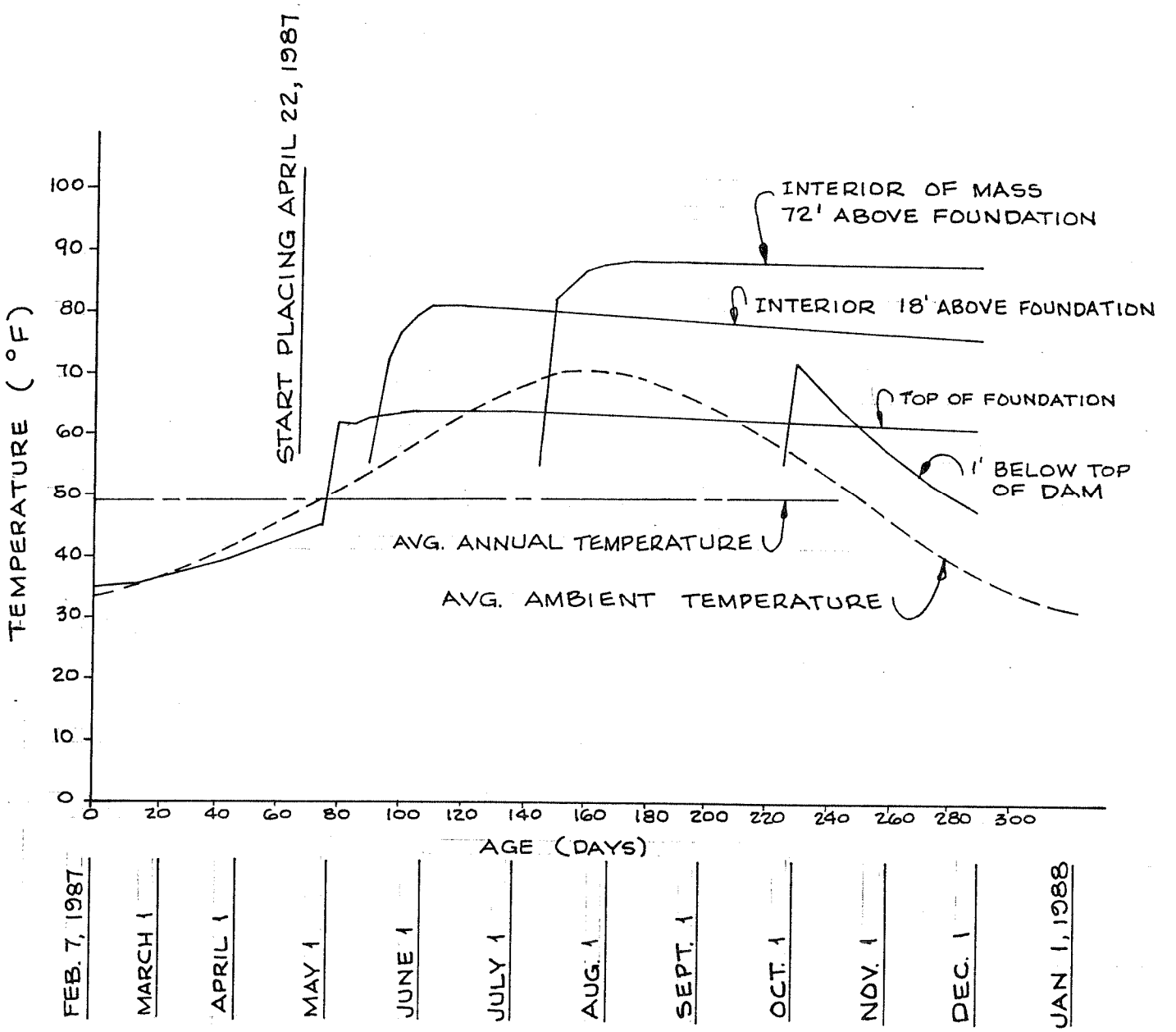

FIGURE J.3B

J.4 DAM AGGREGATES AND MIX DESIGNS

Site explorations indicate that both the gravel deposits upstream of the dam and rock from required foundation excavation can be used to make the suitable RCC aggregate. With additional processing, these sources could be used to make conventional concrete aggregate. If a Conventional Mass Concrete dam or an RCC dam requiring high strength were built, a substantial processing facility with classifiers, washing and settling ponds would be necessary. For the RCC dam as designed, a simple high production operation without washing can be used to make the RCC aggregate. A smaller, separate plant can be used to make the remaining small amount of conventional concrete aggregate for facing, walls, fill, etc. or it could be obtained from a commercial producer meeting ASTM Specifications.

Results of aggregate source investigations including petrographic reports are contained in Section E. The materials are typically hard, sound, and dense, although there are small quantities of friable particles in the gravels. The rock and the greatest constituent of the gravels are good quality basalt. The materials are similar in description and physical appearance to those used or tested at other projects which produced very excellent RCC. For example, at Willow Creek Dam, RCC aggregates were produced from a blend of about 70% excavated basalt and 30% silty sandy gravels. A URUGUA-I quartzitic sands and a sandy silt material blended with quarried basalt rock produced excellent aggregate. For these materials, no washing was necessary and, in fact, it was found that including the non-plastic fines helped the overall RCC quality. A similar operation is anticipated at Miner Flat for RCC aggregate, where probably about 50% to 70% of the material will be basalt from required rock excavation, 20% to 30% will be from gravel borrow, and 0% to 10% will be from a local borrow source of sand and/or silt. There is adequate quantity of required excavation and borrow to produce the aggregates necessary. The gradation limits and construction controls for RCC aggregates at Miner Flat are detailed in the preliminary specifications of Section J.8. Conventional Concrete Aggregate will meet normal ASTM C-33 Specification Requirements.

There is enough experience with lean RCC mixes using similar aggregate sources to confidently expect that RCC of the quality required at Miner Flat can be produced without difficulty and at a low cement factor. This experience includes several mixes at Willow Creek Dam, a number of them for URUGUA-I, and mixes for Zintel Canyon dam. Similarly, experience has also shown that other aggregates of poor quality require higher cement or cement plus fly ash rates in order to achieve reasonable strengths.

The design strength level used for Miner Flat is only 1400 psi at 1 year, but an even lower level would still provide all

necessary factions of safety. Nonetheless, the intent is to provide a 1 year average strength of 1800 psi in order to conservatively account for mix variability and possible construction problems. Strength levels used in the stability analysis and that are anticipated are given in Section J.2. It is expected that 1800 psi can be achieved with about 120 pounds of Type II Cement per cubic yard or less. At this cement content, the anticipated adiabatic heat rise will be about 26 F degrees and internal thermal cracking of the mass should be avoided.

If the aggregate were questionable, if there was little experience with similar materials, if there was insufficient remaining time to make mix designs, or if the required strength level were high, it would be appropriate to have already made preliminary mix designs. However, that is not the case. The construction contract will be arranged so that the exact mix design can be determined and provided during the course of early construction work, and so that it can be adjusted if necessary. This includes the options of using pozzolan for a portion of some of the cement. This approach is not uncommon in dam construction.

During the first phases of site construction, samples of representative aggregates will be provided to the Engineer by the contractor for the purpose of mix designs. This will allow adequate time to develop optimum mix designs for economy and quality, make adjustments and, if necessary, remake the mixes. It also allows for enough time to obtain test results through six months age. Tests will be made at various ages for various cement contents such as 80, 100, 120, 150, and 170 pounds of cement per cubic yard. Ages tested will probably include 2, 7, 14, 28, 56, 90, 180 and 365 days. Tests to verify the design assumptions will include at least compression, modulus of elasticity, and indirect tensions. From this data and the results of similar mixes from other projects, other material properties can be extrapolated. This approach has been successfully used at other RCC projects.

\MINERFLATDAM\AGG&MX.DSN

J.5 RCC DAM CONSTRUCTION - SPILLWAY FACING

The facing concrete in the spillway section presents a special problem. The surface of the facing must be extremely smooth and free of defects. For that reason, we feel the formwork for the facing concrete must be internally supported. That is, it cannot have any ties through the face of the spillway. We have included a conceptual sketch (Figure J.5A) of one forming system that was successfully used on the Copperfield River Dam in Australia. That spillway has been used on more than one occasion and has performed without any problems.

The basic concept is to imbed approximately 3" diameter steel pipes into the facing concrete. These pipes are approximately 6 feet in length. Immediately before a pour, the pipes are lubricated with a heavy grease. There is a bracket that attaches to the pipes and extends out over the face of the dam to which the form is attached. The supporting bracket must have enough strength to permit the forms to kick against it, since that is the only means of support. Beneath the form, there is a scaffold provided. The scaffold provides a little additional weight and, since there are usually people on the scaffold during the pour work, it also helps hold down the bottom of the form, which has a tendency to want to float. As soon as the facing concrete has achieved enough strength, approximately five to six hours after a pour, a jack is attached to the pipes. The bolt that attaches the bracket to the pipe is loosened. The pipes are then jacked up 1 1/2 feet in preparation of the next lift. The jack is then lowered and attached to the supporting bracket. The forms are moved away from the face of the concrete by means of adjusting bolts that attach the forms to the brackets. The form is then raised to the elevation of the next pour.

With the scaffold permanently attached to the forms, as soon as the form is removed, the finishers can finish the concrete, remove any defects, and then immediately apply curing compound. The concrete is still green enough that it finishes very well. The pipes extend three lifts into the dam so that they are bearing in concrete that has acquired some amount of strength.

The spillway will be placed without contraction joints and without reinforcing steel. Although the facing mix will be formulated with the intent to eliminate shrinkage cracks, some tight cracking may still result from the combination of temperature changes and drying. These cracks would be very tight and follow directly over the drain holes left by the temporary form support pipes. They would not be transverse to the flow path and would terminate at the drain. In fact, this condition developed at Copperfield Dam where the same technique was previously used. That spillway has operated without incident.

If the velocity head from the spillway were to force water into a crack, the pressure would immediately relieve itself into the drain hole. The drains also act to prevent the potential build-up of pressure within the RCC behind the facing if for some reason there were to be a seepage path there.

\MINERFLATDAM\SPLWYFC.CON

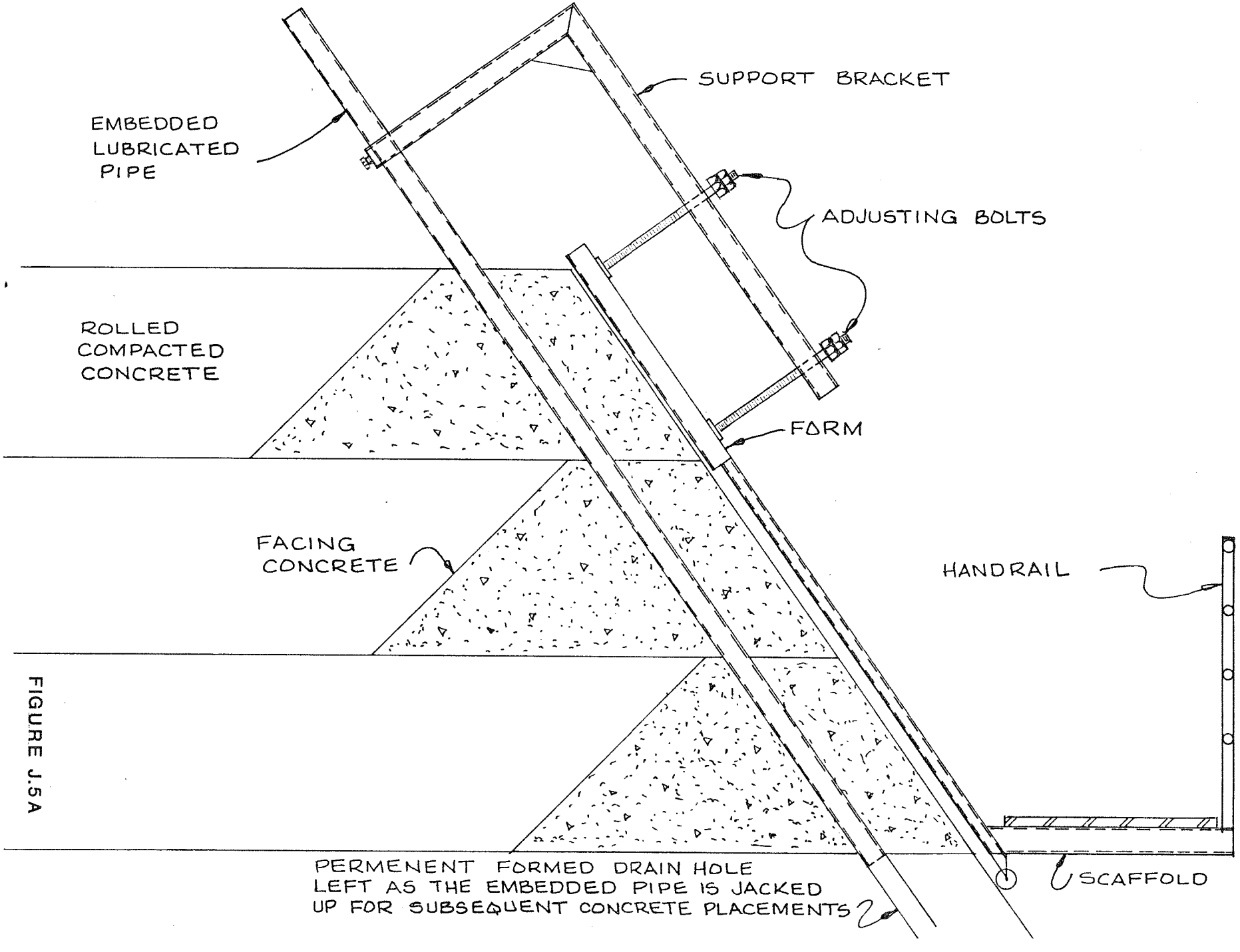
SUPPORT BRACKET
EMBEDDED LUBRICATED PIPE
ADJUSTING BOLTS
ROLLED COMPACTED CONCRETE
FORM
FACING CONCRETE
HANDRAIL
SCAFFOLD
PERMENENT FORMED DRAIN HOLE LEFT AS THE EMBEDDED PIPE IS JACKED UP FOR SUBSEQUENT CONCRETE PLACEMENTS

FIGURE J.5A

J.6. RCC DAM CONSTRUCTION - SCHEDULE

A construction schedule for the RCC Dam has been prepared based on the design rates of material placement, anticipated construction seasons, air temperatures, and construction methods used in developing the cost estimates.

The following Figures are self explanitory and critical in developing the RCC Construction Schedule. Those Figures are as follows:

Figure J.6A	-	Useable RCC Aggregate Stockpile Size
J.6B	-	Crusher Production
J.6C	-	RCC Mixer Production
J.6D	-	RCC Production (Dam Height)
J.6E	-	RCC Production (Volume Placed)
J.6F	-	RCC Dam Schedule

\MINER FLAT DAM\RCCDAM.STR

USEABLE RCC AGGREGATE STOCKPILE SIZE
STOCKPILE SIZE (TONS x 1000)
250
200
150
100
50
0
1 SEPT 87
1 OCT 87
1 NOV 87
1 DEC. 87
1 JAN 88
1 FEB 88
1 MAR. 88
1 APRIL 88
1 MAY 88
1 JUNE 88
1 JULY 88
DAYS
0
50
100
150
200
250
300
350
SEPT. 12 - START CRUSHER
SEPT. 21 - START USEABLE STOCKPILE
63°F
51°F = TEMP INTO STOCKPILE
43° F.
RCC AGG = 237,477 TON
AVG. RCC PLACE TEMP=49°F
START RCC APRIL 11, 1988
AVG. AGG. STOCKPILE TEMP. ~ 51°F
51°F
53°F
57°F
FINISH RCC MAY 30

FIGURE J.6A

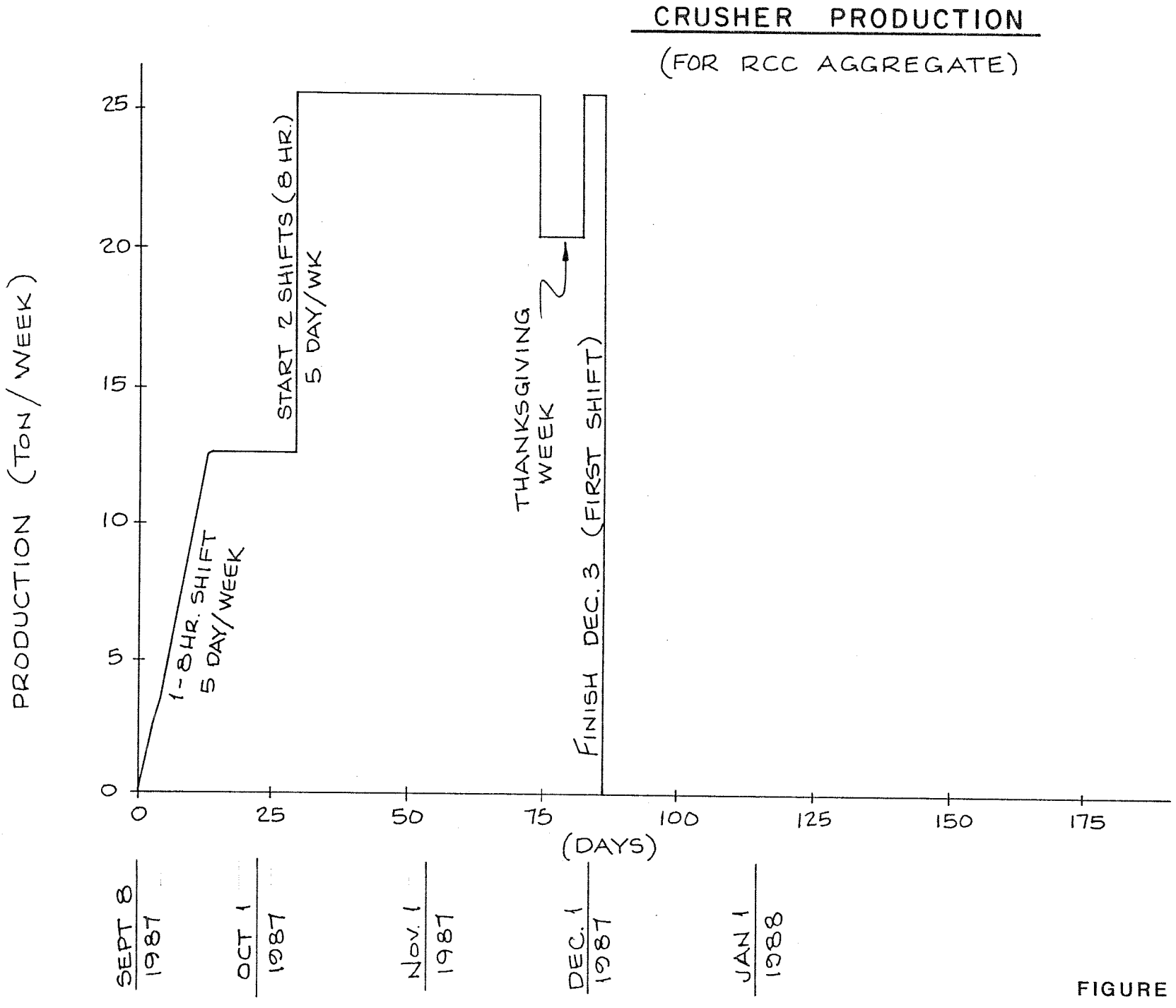
CRUSHER PRODUCTION
(FOR RCC AGGREGATE)
PRODUCTION (TON/WEEK)
0
5
10
15
20
25
1-8HR. SHIFT
5 DAY/WEEK
START 2 SHIFTS (8 HR.)
5 DAY/WK
THANKSGIVING
WEEK
FINISH DEC. 3 (FIRST SHIFT)
0
25
50
75
100
125
150
175
(DAYS)
SEPT 8
1987
OCT 1
1987
NOV. 1
1987
DEC. 1
1987
JAN 1
1988

FIGURE J.6B

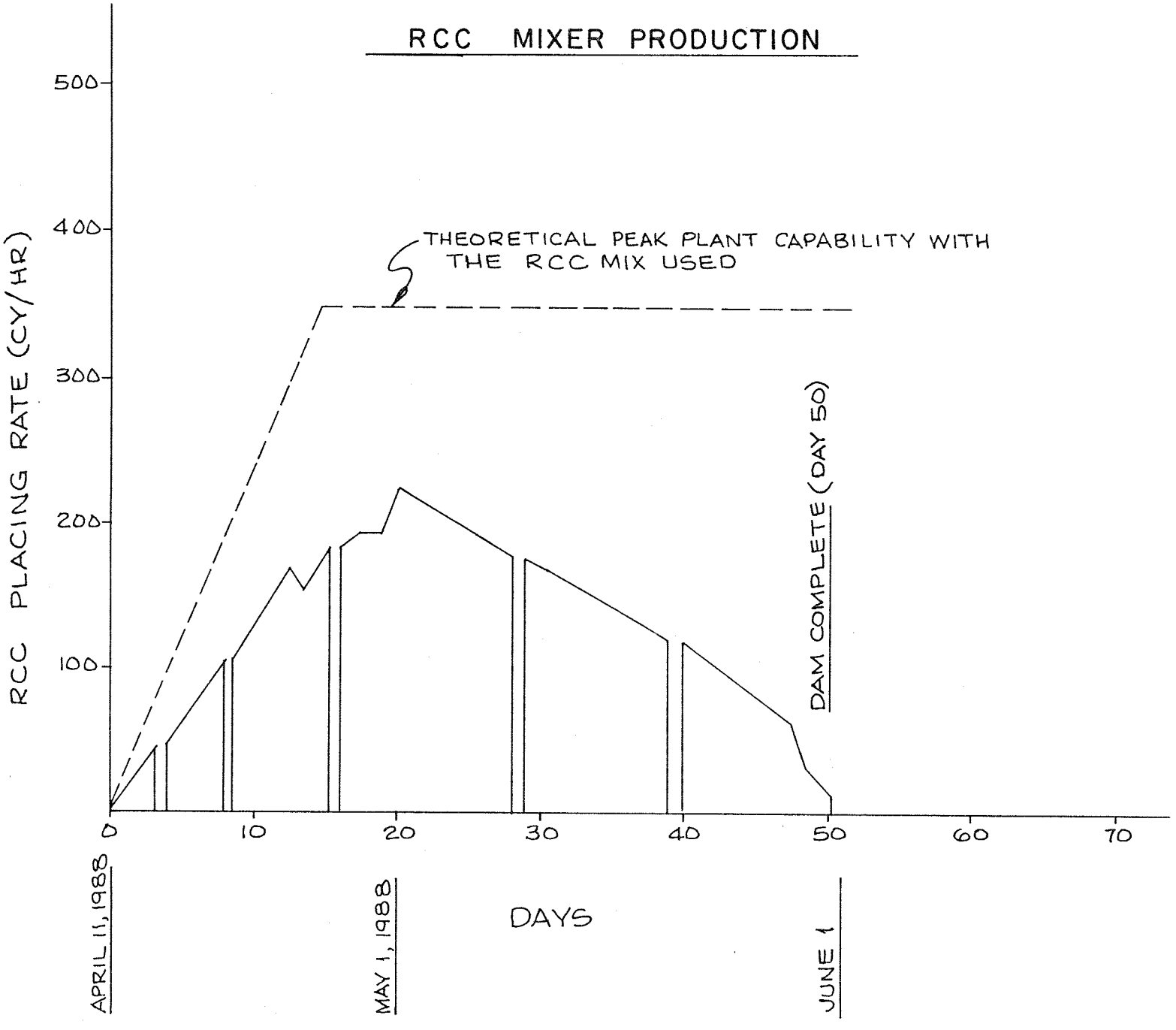
RCC MIXER PRODUCTION
RCC PLACING RATE (CY/HR)
500
400
300
200
100
THEORETICAL PEAK PLANT CAPABILITY WITH THE RCC MIX USED
DAM COMPLETE (DAY 50)
0
10
20
30
40
50
60
70
DAYS
APRIL 11, 1988
MAY 1, 1988
JUNE 1

FIGURE J.6C

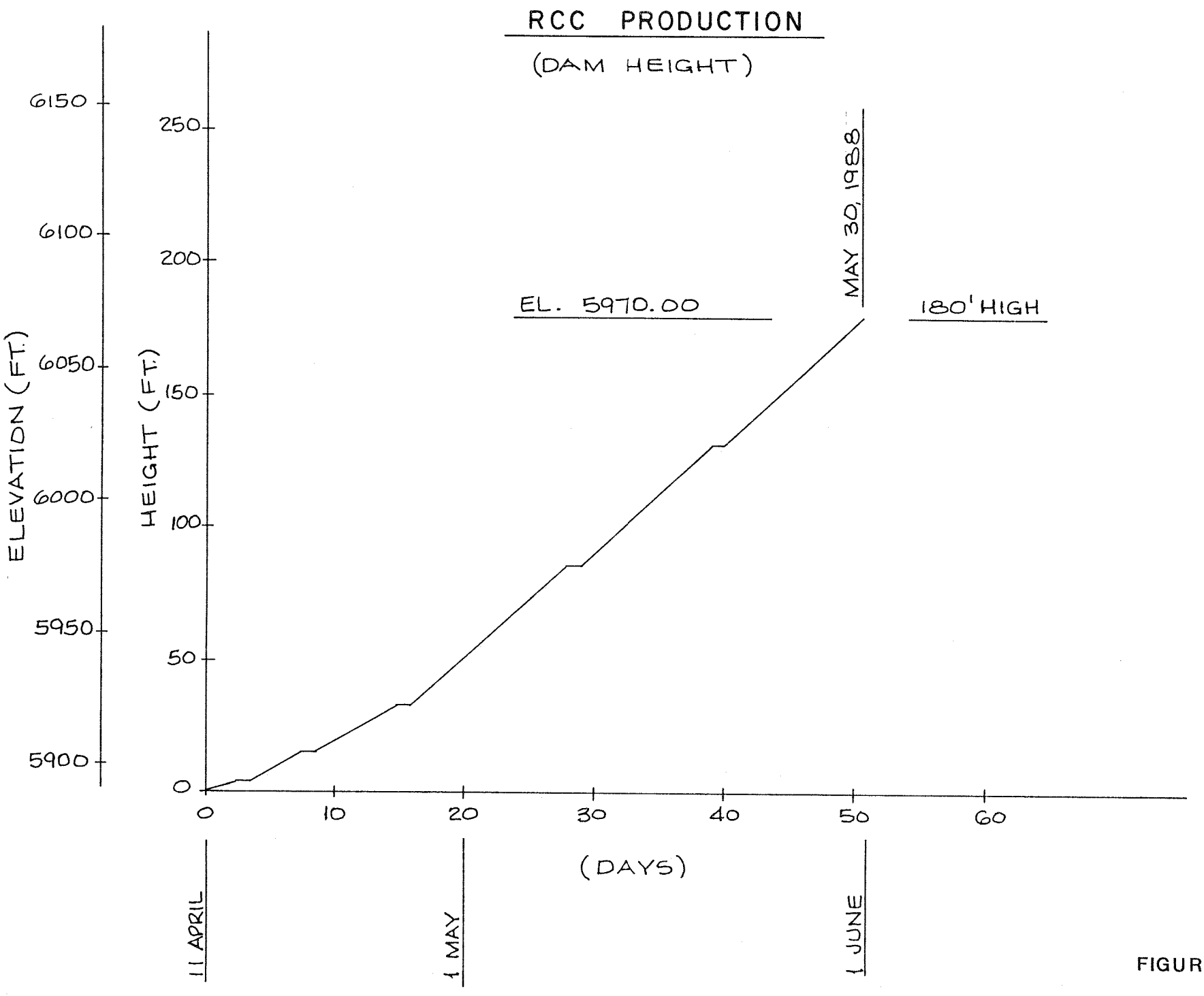
RCC PRODUCTION
(DAM HEIGHT)
ELEVATION (FT.)
HEIGHT (FT.)
6150
6100
6050
6000
5950
5900
250
200
150
100
50
0
EL. 5970.00
MAY 30, 1988
180' HIGH
0
10
20
30
40
50
60
(DAYS)
11 APRIL
1 MAY
1 JUNE

FIGURE J.6D

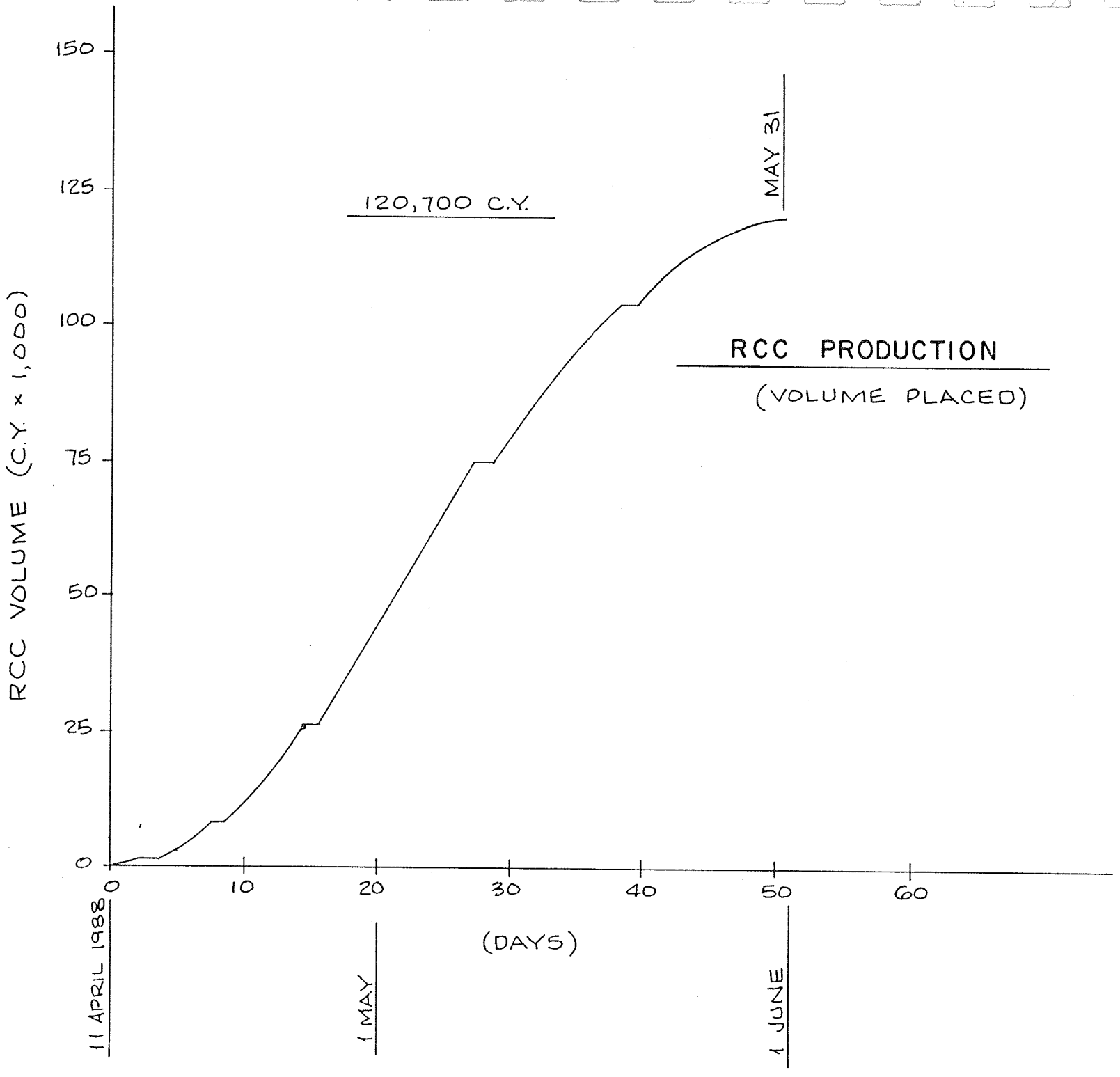
RCC PRODUCTION
(VOLUME PLACED)
120,700 C.Y.
MAY 31
RCC VOLUME (C.Y. x 1,000)
150
125
100
75
50
25
0
0
10
20
30
40
50
60
(DAYS)
11 APRIL 1988
1 MAY
1 JUNE

FIGURE J.6E

DESIGN MEMORANDUM
RCC DAM CONSTRUCTION SCHEDULE

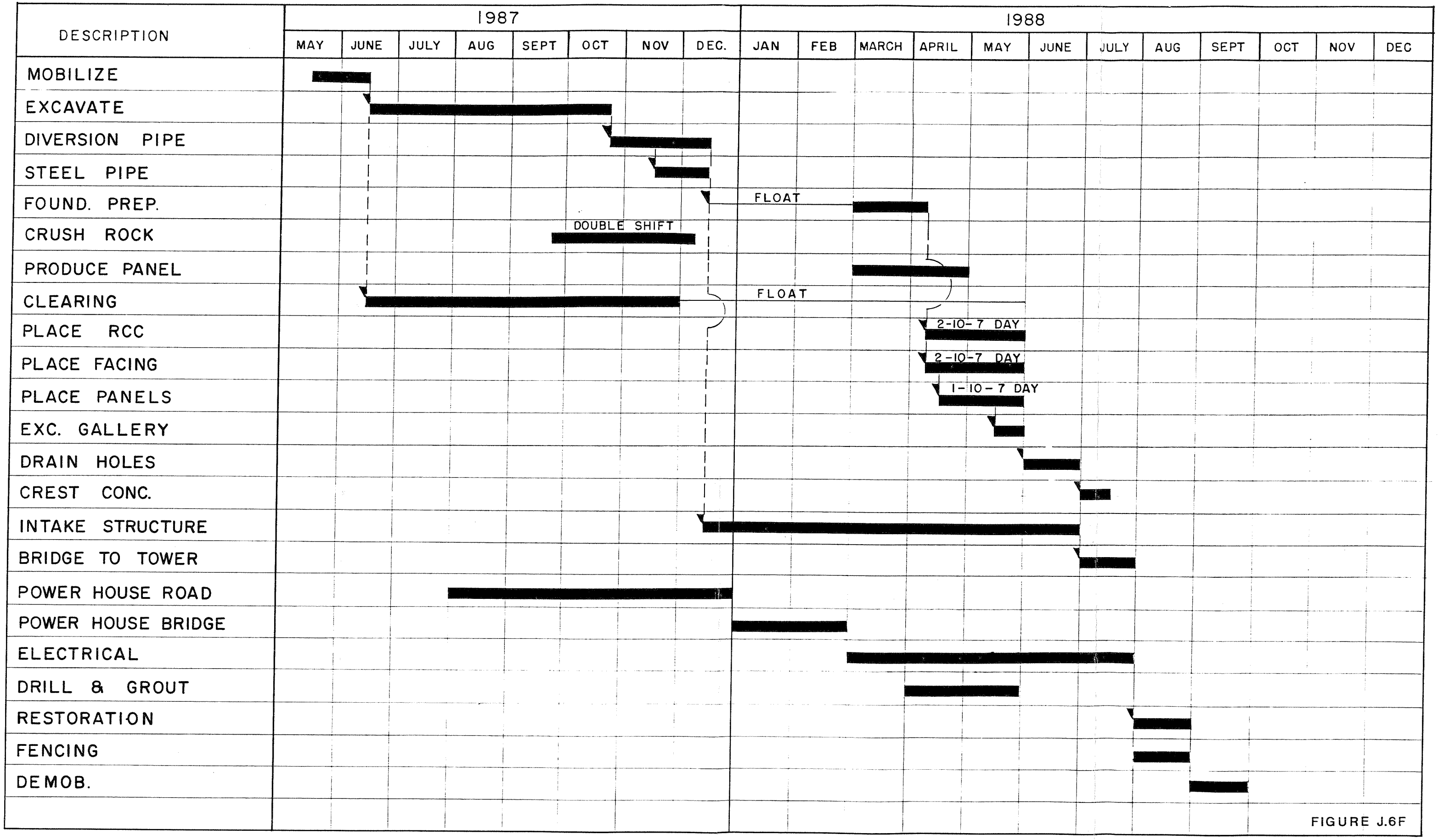

FIGURE J.6F

J.7 DAM COST ESTIMATES

INTRODUCTION

The Cost Estimate for Miner Flat Dam was prepared after an in-depth analysis of climactic conditions, scheduling considerations and logistics. Each major item of work was broken down into its component parts. The cost of each component was arrived at by applying the appropriate labor, equipment and materials costs. All production rates used in the RCC Estimate were very conservative.

Union labor rates with a 19.3% payroll burden were used.

Equipment rates were from the Cost Reference Guide for Construction Equipment published by the Equipment Guide Book Company of Palo Alto, California.

Material costs were from local suppliers.

The project layout, for purposes of the Cost Estimate. The project was laid out as follows:

1. Approximately 73,000 cubic yards of rock excavation was hauled to the crushing plant located 1,000 feet ± upstream from the dam.

2. Approximately 73,000 cubic yards of gravel was excavated from the canyon bottom to blend with the shot rock to produce required aggregates.

3. The aggregates were stockpiled on the upper right abutment.

4. The pug mill for roller compacted concrete and the conventional concrete plant were located on the upper right abutment as near the C8L as possible.

5. Both RCC and conventional concrete were delivered from the plant by conveyer and from the top of the canyon to the bottom by a spiral chute.

6. The RCC was transported on the dam by a 988B front end loader.

7. Conventional concrete was transported on the dam by transit mix truck.

8. The upstream face panels were set by crane from the canyon floor. A large crane was used so it could also hoist equipment to and from the dam for repairs as required since no other access to the dam crest was provided.

9. The office shop and carpenter yard are also located on the upper right abutment.

MAJOR COST ITEMS:

1. Excavation is the first major cost item to be discussed. The canyon is very deep and narrow making access difficult. The side walls will need to be pre-split, but are too high to take in one lift. After the first shot, it will be difficult to get back to the work area for subsequent lifts. Sequencing would probably require excavating the left abutent, moving the stream to the left side, excavating the right abutment, moving the stream back to the right side and then installing the outlet pipes and making a permanent diversion. Due to access problems and the fact that during the blasting operations, care must be taken to keep the stream flowing. Production rates are conservative. One stream crossing was provided for.

2. RCC production is low because it is always controlled by the downstream facing concrete. Since the dam is being constructed during cold weather, the facing concrete will not set quickly enough to allow for more than three lifts per day, which limits RCC production to a maximum of 250 cubic yards per hour and an average of only 118 cubic yards per hour. Limitations on production permits the utilization of a fairly small plant, 350 cubic yards per hour. The crew sizes remain virtually the same as would be the case for twice the production.

3. The quantity of facing concrete for each lift is fairly small, but there must be a forming crew, a placing crew and finishers at all times, as these functions are concurrent. Since these people are not as productive as they would be on a larger project, the unit prices are higher than might be anticipated.

4. About the same situation exists for the upstream facing panels. The maximum number of panels required per shift is 15, as opposed to a maximum of 60 per shift on past projects. The panel crew is working only a single shift, but nonetheless, due to the limited productivity, the unit costs are still a bit high. Even though all of these crews are not as productive as a person would like for them to be, since all of the operations are concurrent, it is still necessary to maintain separate RCC downstream facing concrete and panel crews.

SHIFTS OF WORK

The project is scheduled to use a single eight (8) hour per day, five (5) day per week shift except as follows:

1. The crusher is scheduled to work two (2) eight (8) hour shifts five (5) days per week.

2. The RCC and downstream facing is scheduled to work two (2) ten (10) hour shifts seven (7) days per week and the upstream facing panels is scheduled to work one (1) ten (10) hour shift seven (7) days per week.

ADVANTAGES OF RCC AS OPPOSED TO CONVENTIONAL MASS CONCRETE

If the contract were awarded in mid May in 1987 it is possible to complete the RCC structure by December 1, 1987. The following items would be necessary to achieve that schedule:

1. The Contractor would need to work two (2) ten (10) hour shifts seven (7) days a week on the excavation and mobilize immediately after award.

2. The contractor would have to make diversion no later than August 1, 1987.

3. Production of aggregates would be during June, July and August. Crushing during the hottest time of the year has some slight adverse effects on the temperatures in the structure but not enough to cause any alarm.

4. RCC placement would start about September 1st and finish by the end of October. The small ice plant might possibly be required for the facing concrete during the early stages of the work but probably not.

With the conventional mass concrete dam, on the other hand, we do not see any way of starting placement sooner than mid March of 1988. Due to thermal considerations a minimum of 180 days are required to construct the conventional concrete dam. Some potential problems that could cause further delays are as follows:

1. There are five (5) Monoliths across the dam. A lift cannot be placed on top of a previous lift sooner than the fifth day. During the colder part of the year, if the set time is retarded enough that a delay in the time the clean up crew can get back on the pour or the time the forming crew can get back on the pour, would probably cause the loss of one days time since there is no alternative place to work.

2. Placement will not be permitted from November 15 to March 15, so a minor delay problems could conceivably force the completion on the conventional structure into 1989.

3. A large batch plant with an ice plant is required to maintain acceptable production rates on the conventional mass structure. At least two (2) large cranes having over one hundred tons capacity are required for the placement. The volumes in each pour are relatively small in this structure, but none the less we still require some very large sophisticated equipment which is going to limit the number of bidders on the conventional dam.

The RCC alternative is the best choice for this location. We have applied a 15% contingency to each estimate but realistically a 10% contingency for the RCC and a 25% contingency for the conventional concrete would be more realistic.

MINER FLAT DAM

Bid Summary Roller Compacted Concrete

Description	Quantity	Adjusted Bid Unit Price	Amount
Excavation	126,874 C.Y.	9.50	1,205,303
Foundation Prep.	3,000 S.Y.	24.50	73,500
Dental Conc.	100 C.Y.	100.00	10,000
Bedding mix	2,500 C.Y.	60.00	150,000
R.C.C.	120,702 C.Y.	23.00	2,776,146
Facing Concrete	5,091 C.Y.	225.00	1,145,475
O.G. Concrete	359 C.Y.	250.00	89,750
Crest Non-Overflow	189 C.Y.	200.00	37,800
Face Panels	40,384 S.F.	12.00	484,608
Concrete AD	1,262 C.Y.	220.00	277,640
Reinforcing	69,110 LBS.	0.50	34,555
Guard Fence	110 L.F.	14.00	1,540
Gallery	Lump Sum	-0-	60,000
Misc. Metal	Lump Sum	-0-	300,000
Diversion Dam	Lump Sum	-0-	15,000
Clearing	165 Acre	2,000.00	330,000
Outlet Pipe	303 L.F.	750.00	227,250
Power House Pipe	150 L.F.	1,500.00	225,000
Bridge to Tower	Lump Sum	-0-	100,000
Bridge to Power House	Lump Sum	-0-	180,000
Drain Holes From Crest	3,050 L.F.	10.00	30,500
Drain Holes From Gallery	1,325 L.F.	30.00	39,750
Drill and Grout	930 L.F.	30.00	27,900
Electrical	Lump Sum	-0-	150,000
Security Fence	Lump Sum	-0-	30,000
Mobilization	Lump Sum	-0-	895,000
Instrumentation	Lump Sum	-0-	40,000
Restoration	Lump Sum	-0-	14,000
TOTAL BID			8,950,717

MINER FLAT DAM

Bid Summary Conventional Cast in Place Concrete

Description	Quantity	Adjusted Bid Unit Price	Amount
Excavation	138,000 C.Y.	9.50	1,311,000
Foundation Prep.	3,000 S.Y.	24.50	73,500
Dental Conc.	100 C.Y.	100.00	10,000
Mass Concrete	141,040 C.Y.	71.00	10,013,840
O.G. Concrete	359 C.Y.	250.00	89,750
Concrete AD	1,262 C.Y.	220.00	277,640
Reinforcing Steel	69,110 LBS.	0.50	34,555
Guard Fence	110 L.F.	14.00	1,540
Gallery	Lump Sum	-0-	150,000
Misc. Metal	Lump Sum	-0-	300,000
Diversion Dam	Lump Sum	-0-	15,000
Clearing	165 Acre	2,000.00	330,000
Outlet Pipe	303 L.F.	750.00	227,250
Power House Pipe	150 L.F.	1,500.00	225,000
Bridge to Tower	Lump Sum	-0-	100,000
Bridge to Power House	Lump Sum	-0-	180,000
Drain Holes From Crest	3,050 L.F.	10.00	30,500
Drain Holes From Gallery	1,325 L.F.	30.00	39,750
Drill and Grout	930 L.F.	30.00	27,900
Electrical	Lump Sum	-0-	150,000
Security Fence	Lump Sum	-0-	30,000
Mobilization	Lump Sum	-0-	1,500,000
Instrumentation	Lump Sum	-0-	40,000
Restoration	Lump Sum	-0-	14,000
TOTAL BID			15,171,225

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J.8 RCC DAM SPECIFICATIONS

A set of RCC Dam Specifications has been prepared for this Design Memorandum to provide details not covered in the previous articles for the construction of the Miner Flat RCC Dam. The specifications may be modified during the final design and used as a portion of the final design specifications.

\MINER FLAT DAM\J-8.MFD

Section ___

UPSTREAM IMPERVIOUS BARRIER

1. GENERAL

The work covered by this section consists of materials and installation for the impervious barrier (liner) at the upstream face of the dam. The liner shall be attached to the rear face of the upstream precast panels. The purpose of the barrier is to prevent seepage into the dam by providing a permanent, durable, waterproof barrier from abutment to abutment and from the top of the dam to the base of the dam. The barrier will be located between the panels and the roller compacted concrete (RCC). See Section ___ "Roller Compacted Concrete."

2. MATERIAL

2.1 Qualifications and Support

The supplier of the liner shall be a well established manufacturer and supplier specializing in this material. The company shall have at least 5 years experience with the basic product supplied,

and with its use in hydrostatic water retaining structures. If supply is through or by a company located outside of the U.S.A. the company shall have at least 3 years of successful construction trade experience with the type of product to be supplied. The Supplier shall designate a highly qualified senior technical person as the responsible liasison for fabrication, shipping, installation (including field welds and repairs), any necessary field training, overall quality control, and coordination. He shall be available for consultation if needed and shall be on-site during at least the first three days of placement where joint welding of the liner is performed.

2.2 Composition and Properties

The liner may be polyvinyl chloride or high density polethylene. It shall have a minimum sheet thickness of 0.080 inch (80 mil) or 2 mm. The liner shall have sufficient flexability to be field welded and lapped as required in the field at temperatures ranging from 35 to 95 degrees F. Premolded corners may be used to facilitate installation of the added liner sheet near the foundation and abutment. The liner shall have long term resistance to deterioration under the alkali environment of damp concrete in the dam. It shall be resistant to degradation from

organic and bacterial growth. The liner shall be resistant to through punctures and tears during handling and installation, and from placement and compaction of the bedding mix and roller compacted concrete. The liner shall meet the following physical property requirements.

TEST STANDARD	TEST DESCRIPTION	METRIC	U.S.
ASTM D 1505 or 792	Specific Gravity (min)	0.94	0.94
ASTM D 638 Type IV @ Speed C	Ultimate Tensile Strength (min)	3.54 Kg/mm width	200 width
ASTM D 638 Type IV	Elongation @ Break	600%	600%
ASTM D1004 Die C	Tear Resistance (min)	27 Kg	60 lbs
ASTM D 1203 (A)	Volatile Loss (max)	0.2%	0.2%
FTMS 101B	Puncture Resistance (min)	159 Kg	350 pou
ASTM D 751(A), #Proc 1	Hydrostatic Resistance	25 Kg/Cm2	500 psi
ASTM D 256, Method B	Notched Impact Resistance	25x10-5	14x10-5
	Thermal Expansion (max)	per degrees C	per deg F
ASTM D 696	Water Absorption (max)	0.1%	0.1%

Prior to the contractor placing any order for material, the manufacturer shall provide an affadavit guaranteeing that material supplied for this project will be tested and will meet all of the requirements specified.

2.3 Size

The liner shall be provided in sheets that are at least the panel size and that will allow for heat welding the liner from one panel to the next around the perimeter of each panel. The anticipated nominal panel size is __ft x __ft. A plan to provide one factory weld in one direction to make up the required liner size may be submitted for review and consideration. The submittal must be complete, address the issue of intersecting field and factory splices, and guarantee that the factory welds will be tested prior to shipping and be 100% water tight under a persistant pressure of at least 120 psi.

3. INSTALLATION

3.1 Onto Precast Panels

The liners shall be cast integrally with the precast panels so that they remain attached to the panels during handling an erection. An acceptable method of attachment shall be provided so that concrete for the precast panels can be cast directly onto the liner or so that the liner can be placed onto the top of the fresh

mix. The liner shall remain well attached to the hardened panel during all operations including steam curing, form stripping, handling, storage, erection, joint welding, and placement of the dam concrete. Possible methods of attachment include "T" sections which are extruded as an integral part of the liner, studs thermally attached to liner without damaging it, epoxy bonding, and strips that attach to the liner and contain filaments which embedded into the concrete.

3.2 Field Welds

Joint strips and welds shall have the same qualities after welding as specified for the parent sheet itself. Welds shall be done by heat equipment specifically designated for fusing together sheets of the liner into one water tight section. Welding shall be done only by personnel trained and qualified by the Suppier. The welds shall provide at least 1 inch of lap from one sheet of liner to the next or to the weld strip. All welds and any questionable surfaces in the liner shall be tested with an approved electrical holiday or flaw detector. Any defective weld, repair, or liner shall be repaired by additional welds and may require welding of a new patch over the defective area.

3.3 Anchor Rods

Where anchor rods for the precast panels extend through the liner, the hole shall be sealed with flexible Aquatepoxy paste or approved equal, and with a washer and nut as shown on the drawings. The hole in the liner may be up to 2 times the diameter of the anchor bar. The washer shall have a diameter of 4 to 5 times the diameter of the bar. The procedure used shall be to first dip the end of the anchor bar into the epoxy, then thread it into the panel, then tighten the washer and nut so that a pressure seal is accomplished and so that the epoxy squeezes out from the spaces between the membrane and washer, the washer and nut, and the nut and bar. All work shall be finished well within the pot life or useable time of the epoxy under the temperature and conditions during placement.

Section ___

PRECAST PANELS

1. GENERAL

Except as otherwise allowed, specified, or supplemented by this SECTION: "PRECAST PANELS" all precast concrete and related work shall conform to the requirements of SECTION "CONCRETE". The precast concrete facing panels for the dam may be cast on site or in an acceptable established off-site facility naomrally engaged in commercial precast concrete manufacturing. If the precast panels are cast on site, the Contractor shall establish a specific casting area complete with casting beds, lifting and handling equipment, batch plant, cure and protection equipment, placing equipment, lighting, proper drainage, storage areas, etc. If the site facility does not reliably and consistently produce precast concrete of the required quality and at the necessary production rates, the Contractor shall replace or supplement the on-site casting plant with an "off-site" facility normally engaged in commercial production of precast concrete. At least 85 percent of the precast facing panels and related hardware, including all panels and hardware required for the placement of roller compacted concrete (RCC) to Elevation 6,062, shall be in storage on site prior

to the start of production RCC placement.

2. COMPOSITION

Precast concrete may use high early strength portland cement. The concrete shall contain an air-entraining admixture, water, normal weight fine aggregate, and coarse aggregate to a maximum size of 1 inch. A water-reducing admixture conforming to ASTM C 494 may be used.

3. AGGREGATES

Aggregates for precast concrete shall come from the approved site sources or from an established and approved precast plant having a history of quality construction and durable materials. Aggregates from "off-site" shall have demonstrate a satisfactory service record under similar conditions to those which they will be subjected to under the design of this contract. Aggregates shall not be used without prior approval. If, in the opinion of the Engineer an adequate service record or adequate laboratory testing has not been demonstrated, the contractor shall perform all tests required by ASTM C 33 through a laboratory meeting the

requirements of ASTM E 329 and submit this data to the Engineer for review at least 15 days prior to the first intended use of the materials. Established gradations for precast concrete will be allowed to vary slightly from those listed in SECTION "CONCRETE", providing that the gradations actually used have a history of excellent quality contrete construction. If a variation from the gradations of SECTION "CONCRETE" is desired, the Engineer shall be notified in writing and his concurrence shall be obtained prior to casting the concrete. The maximum permissible size of aggregate for precast concrete shall be 1 inch.

4. QUALITY

Mix designs for precast wall panels made with conventional concrete shall obtain design compressive strengths of 4,000 psi at 90 days, with a minimum design compressive strength of 1,000 psi prior to form stripping and of 2800 psi prior to shipping. The mix designs and strengths shall be the Contractor's responsibility. Compressive strength tests are required for all precast concrete. Strengths shall be determined by field cylinders cast and cured with the precast item they represent.

The Contractor shall stipulate his proposed cure procedure (i.e., steam, water, etc.), at the time the mix designs are submitted.

5. PLACING

Concrete placement for precast construction shall be as specified in SECTION "CONCRETE" except that internal vibration is not required for sections thinner than 7 inches if sufficient external vibration is used. Established mechanical placing equipment used to manufacture precast concrete under factory conditions shall be used. Additional finishing after consolidation of the concrete will not be necessary, unless irregularities or imperfections of the surface occur which require hand touchup with wood floats. The Contractor shall provide rounds on exposed corners and edges with an edging tool or by forming.

6. CURING AND PROTECTION

All concrete shall be cured and protected by an approved method or combination of methods as specified in SECTION "CONCRETE", or by steam curing. Steam curing, if used, must comply with the following requirements: Initial application of steam shall be

started as soon as the concrete has attained its initial set. The steam shall be at 100 percnet relative humidity and shall not be applied directly to the concrete. Application shall be at such rate that the air temperature adjacent to the concrete and within the steam enclosure does not increase at a rate exceeding 40 degrees F per hour. A curing temperature of 110 to 140 degrees F shall then be maintained until the specified strength of concrete has been attained, after which the air temperature adjacent to the concrete and within the steam enclosure shall be decreased at a rate not exceeding 40 degrees F per hour until the temperature is within 20 degrees of the ambient air to which the concrete will be exposed. During the time of temperature drop or when drying shrinkage could occur, the concrete panel shall be kept free of restraint which could cause cracking.

7. FORMS

7.1 Materials

Forms shall be of wood, steel, or other approved material. The type, size, shape, quality, and strength of all materials of which the forms are made shall be subject to approval and the requirements specified in SECTION "CONCRETE".

7.2 Construction

Forms shall be true to line and grade, mortar, tight and sufficiently rigid to prevent objectionable deformation under load. The form surfaces shall be smooth, free from irregularities, dents, sags, or holes when used for permanently exposed faces. All form removals shall be accomplished in such a manner as to prevent injury to the concrete.

8. HANDLING

Points of support used in handling, transportation, storage, and erection of all precast members shall be as close as practical to the final points of load, support, and bearing. Care shall be taken during handling to prevent cracking and damage. Members damaged beyond safe usability during handling, storage, or erection shall be replaced by the Contractor at his expense. Lifting eyes and similar expedients for construction and handling may be used only if specifically approved on shop drawings. The Contractor shall be responsible for stress calculations and assuring that the use of such handling expedients does not damage the precast items. Occasional minor cracks and corner or edge

spalls found in precast concrete after being set in place will be tolerated. However, consistently damaged precast concrete facing panels as determined by the Engineer shall be removed and replaced at the Contractor's expense.

9. QUALITY CONTROL

The Contractor shall establish and maintain quality control to assure compliance with contract requirements and shall maintain records of his quality control for all operations including but not limited to the following:

1) Inspection of materials as delivered to the jobsite for damage.

2) Storage and handling of materials.

3) All requirements for QUALITY CONTROL OF SECTION "CONCRETE".

SECTION ___

ROLLER COMPACTED CONCRETE (RCC)

1. GENERAL

The work covered by this section consists of furnishing all plant, material and equipment, and performing all labor for the manufacturing, transporting, placing, compacting, and curing of roller compacted concrete (RCC). Roller-compacted concrete is a combination of crushed rock and/or natural sand, gravel, and/or soil having a controlled gradation to which cementing materials such as cement or cement and pozzolan is added. The materials are blended with water to damp consistency which can be hauled in dump type trucks or delivered with a conveyor, spread with earth moving equipment, and compacted with a vibratory roller.

This specification has been deliberately prepared to be general in nature and adaptable to adjustments that may become neccesary during construction.

2. APPLICABLE PUBLICATIONS

Except as otherwise allowed, specified, or supplemented by this specification section, roller compacted concrete

(RCC) is subject to the Standards and Publications listed in the Specification Section for "CONVENTIONAL CONCRETE". In addition, the following publications which do not form a part of this specification may be used for general background information.

"Roller-Compacted Concrete", ACI Committee 207 report, Journal of the American Concrete Institute, July - August 1980.

Schrader, E.K. "Roller-Compacted Concrete" The Military Engineer, September - October 1977. Also reprinted by the Portland Cement Association.

Chao, P.C. and Johnson, H.J., "Rollercrete Useage at Tarbela Dam", Concrete International, November 1979.

"Willow Creek Dam Concrete Report" U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, Washington, Volumes 1 and 2, 1984 update.

Roller-Compacted Concrete, Symposium Volume of the American Society of Civil Engineers, May 1985.

3. COMPOSITION

3.1 General.

Roller-compacted concrete will be composed of portland cement, pozzolan (if required), aggregates, and water. Water shall meet the standard chemical requirements for ready mix concrete as established in ASTM C 94, except that water with a higher concentration of non-clayey, non-aggressive, solid particles such as silts will be allowed.

3.2 Mix Designs.

The roller-compacted concrete mix design proportions will be determined by the engineer. The mix design(s) will be based on laboratory tests of aggregate materials obtained from the project source, and on cementitious materials representative of those to be used in the project.

The general criteria for mix design(s) used in the dam will be (1) to provide adequate strength to meet structural design loads with normal or above normal factors of safety; (2) to minimize internal heat rise from hydration and the subsequently developed stress or crack potential; (3) to maximize stress relaxation through creep and elastic properties; and (4) to provide a constructable mix.

The anticipated RCC mix design(s) are approximated below based on experience at other projects and limited preliminary information on the expected aggregate source (weights are based on saturated surface dry aggregate). The actual mix design(s) will be determined from testing. It is the intent to utilize a single RCC mix throughout the dam. However, if this cannot practically and economically meet the design criteria, zoned areas of different mixes will be used as necessary between different elevations.

(POUNDS PER CUBIC YARD)

MIX	PRIMARY USE	AGGREGATE SIZE	CEMENT	POZZOLAN	WATER	AGGREGATE
1	All mass including the non-over flow sections	3 in.	120	0	150	3935
2	Areas requiring higher strength to meet design criteria or construction needs (if necessary)	3 in.	170	0	155	3875

Minor adjustments in exact mix proportions (such as the added moisture required to obtain optimum compaction during cool/damp weather as compared to warm/wet weather, and the exact ratios of aggregate proportions from different size groups to obtain the correct overall gradation) shall be the contractor's responsibility. Continual routine monitoring shall be used as the basis for adjustments. The cement content shall not be adjusted without written approval or direction from the engineer. Cement adjustments will only be allowed or directed after development of sufficient supporting test results indicating justification for the adjustment.

3.3 Samples for Mix Designs.

Prior to July 13, 1987 representative production samples of aggregates, cement, and pozzolan proposed for this project shall be delivered by the contractor to the laboratory. Samples of aggregates shall be taken under the supervision of a materials/concrete representative of the engineer. Samples of materials other than aggregates shall be representative of those proposed for the project and shall be submitted accompanied by manufacturer's test reports indicating compliance with applicable specification requirements. All materials shall be identified with a tag. Minimum quantities of of materials required shall be as follows:

MATERIAL	QUANTITY
Each size group of RCC aggregate	30 ton
Cement (In barrels or sealed sacks)	1 ton
Pozzolan (In barrels or sealed sacks)	1 ton

4. CEMENTING MATERIALS

4.1 General

Portland cement (and pozzolan if needed) shall be the cementing material used in RCC. It shall be furnished in bulk to the jobsite. Cement shall conform to the requirements of ASTM C 150. If used, pozzolan shall be ASTM C 618, Class F or equivalent. Class C pozzolan may be used if the particular pozzolan proposed has no detrimental reactivity with the aggregates being used and if the 7-day heat of hydration for a blend of 65 percent cement plus 35 percent pozzolan by volume does not exceed the heat generated by the cement with no substitution. Natural pozzolan meeting the requirements of fly ash is also acceptable.

4.2 Acceptance Requirements and Tests.

The source or sources of cement and pozzolan shall consistently supply material with similar chemical and physical properties. The supplier shall routinely check the chemical and physical properties of the cementitious materials for conformance with the

referenced standards. Materials not meeting the standard requirements shall not be forwarded to the project by the supplier. Copies of all test results representing material sent to the project shall be forwarded immediately to the project. Cement (and pozzolan, if used) may be subjected to check testing by the contractor or engineer from samples obtained at the mill, at transfer points, and at the project as necessary.

4.3 Transportation of Bulk Cement and Pozzolan.

When bulk cement is not unloaded from primary carriers directly into weathertight storage silos at the concrete plant, transportation from the railhead, mill, or intermediate storage to the batching plant shall be accomplished in adequately designed weathertight trucks, conveyors, or other means which will completely and thoroughly protect the cement from exposure to moisture.

4.4 Temperature of Cement.

The temperature of cement when delivered to the jobsite shall not exceed 150 degrees F. The temperature of air (if that process is used) to

transport cement into storage containers or silos shall not exceed 160 degrees F. The temperature of the air will be determined by measuring the temperature on the outside of the transport pipe with a surface thermometer.

4.5 Storage.

Immediately upon receipt at the concrete plant, cement and pozzolan shall be stored in a dry, weathertight and properly ventilated structure. All storage facilities shall be subject to approval and shall permit easy access for inspection and identification. Sufficient cement shall be in storage to complete at least 4 hours of placement at the typical production rate being experienced. In order that cement may not become unduly aged after delivery, the Contractor shall use any cement which has been stored at the site for 60 days or more before using cement and pozzolan of lesser age.

4.6 Source.

The Contractor shall notify the engineer of the source or sources from which the cement and pozzolan

(if used) will be obtained in advance of the time when production RCC placing with materials from those sources is expected to begin.

5. AGGREGATE

5.1 General.

Aggregate shall be produced from the designated sources and quarry areas. During daily production the material being used to produce aggregate shall consist of at least 50% quarried basalt or basalt from required excavation, and at least 30% natural gravel from the designated borrow area. The contractor may supplement these materials with borrow from approved commercial or site sources of sand, silts, or other blends. Unless otherwise specifically stated, the gradation limits given below and the percentages referred to apply to the total aggregate weight used in a unit volume of RCC, including all size groups. They do not apply to the weight of aggregate in any one size group. The resulting allowable range of material passing or retained on any sieve size is broader than would typically be required for a conventional concrete aggregate.

5.2 Production Schedule.

Not more than 10% of the aggregate shall be produced before September 28, 1987, and at least 90% shall be manufactured and stockpiled prior to April 11, 1988. All RCC aggregate shall be produced before April 25, 1988. Payment will be made for stockpiled aggregate.

5.3 Gradations and Stockpiles

The Contractor shall stockpile aggregates in two size groups. A separate stockpile for blend sand or silt will not be required but may be used.

Group I - 100% passing the 4 inch sieve and at least 97 percent retained on the 3/4 inch sieve.

Group II - 98 percent passing the 1 inch sieve.

The Contractor's aggregate production and stockpiles shall be such that when (by weight) the Group I and Group II material (plus any other size group or blend material developed by the Contractor) is combined, the resulting gradation will be within the tabulated limits.

In general, it is expected that approximately 45 percent of the total aggregate production will be Group I, and 55 percent will be Group II. The Contractor shall be responsible to provide all necessary extra aggregates to allow for overbuild, stockpile bases, and materials used for his

convenience. In order to insure reasonable consistency within each stockpile, samples for the combined gradation test will be obtained from various parts of the stockpiles.

The gradation band has been widened to the maximum extent possible. Any gradation within the band and having the same basic graphed shape as indicated by the graphed gradation band will be acceptable. However, the gradation will not be allowed to go from near the maximum allowed amount passing one sieve to near the minimum amount allowed on the subsequent sieve, or vice versa.

U.S. STANDARD SIEVE	PERCENT FINER BY WEIGHT
4 inch	100
3 inch	98-100
2 inch	85-95
1-1/2 inch	74-89
1 inch	62-76
3/4 inch	55-68
3/8 inch	41-54
4	32-45
8	25-37
16	19-30
30	14-24
50	10-19
100	6-14
200	4-10

The maximum permissible amount passing the No. 200 sieve shall be determined by the following table which is dependent upon the plasticity of all of the fines (washed sample) passing the No. 40 or No. 50 sieve. Experience has shown that results are similar for either sieve. The size used can be based on convenience of testing at the discretion of the engineer.

LIQUID LIMIT	PLASTIC INDEX	MAX % PASSING NO. 200
0-25	0-5	10.0
0-25	5-10	9.0
0-25	10-15	4.0
0-25	15-20	3.0
0-25	20-25	1.5
25-35	0-5	9.0
25-35	5-10	8.0
25-35	10-15	6.5
25-35	15-20	5.0
25-35	20-25	1.5
35-45	0-5	8.5
35-45	5-10	5.5
35-45	10-15	4.0
35-45	15-20	2.0
35-45	20-25	1.5
45-55	0-5	5.5
45-55	5-10	5.0
45-55	10-15	3.5
45-55	15-20	3.0
45-55	20-25	1.5

The plasticity index will not be allowed to exceed 25 and the liquid limit will not be allowed to exceed 55 as determined on all materials passing the No. 40 or No. 50 sieve from washed aggregate. Routine daily testing of "non suspect" material may be done by dry sieving. At least once per week during aggregate production, (and for all "suspect material") the washed gradation shall be determined.

5.4 Particle Shape.

The amount of flat and elongated particles as determined by CRD C 119 shall not exceed 40 percent on any individual sieve size nor a weighted average of 30 percent total for all the sieve sizes in the total gradation. These amounts are increased from values typically used for conventional concrete, but have been found to be acceptable limits for roller compacted concrete.

5.5 Washing.

Aggregates used in other RCC projects have been produced from natural deposits and also from a combination of silty overburden with shot rock excavated from quarry areas. Washing of the aggregates at most jobs was not necessary, and in

fact was found to be not desirable. It is expected that a properly designed production plant for processing aggregates will not require washing. However, some moisture spray may be necessary for dust control on conveyor belts.

5.6 Storage.

Aggregates shall be stored in stockpiles at the designated storage areas near the dam. Separators such as timbers or boards shall be used between adjacent stockpiles (if separate stockpiles are used) to prevent contamination and intermixing between the stockpiles. The Contractor shall be responsible to provide a system which reliably and consistently stockpiles the aggregates and later allows withdrawal of the aggregates from the stockpiles without contamination or segregation. The system shall also provide for intermixing or blending of aggregates delivered to and withdrawn from any individual stockpile. Segregated and/or contaminated aggregates that are not representative of the stockpiles and which do not result in the specified combined gradation will not be allowed in production of RCC. The Contractor's aggregate

handling system may utilize conveyors, reclaim tunnels, face loading by front end loaders, or other acceptable methods.

5.7 Plant Layout.

The Contractor shall submit a layout and schematic drawing with a narrative description of his planned aggregate production, locations and sizes of stockpiles, transportation methods, and storage procedures to the engineer for review. Anticipated peak production capacity, normal operational rates, and storage area volumes shall be indicated.

6.1 General.

The Contractor shall select the type of concrete "plant" to be used and shall determine its layout. The "plant" includes all necessary mixers, volumetric or weigh method controls, storage bins, feed systems, and discharge mechanisms. Prior to the start of production RCC placement the Contractor shall submit to the engineer for information a layout and schematic drawing of the concrete plant

together with a narrative description giving its anticipated peak capacity and normal production rate. The submittal shall also include a narrative description and a layout of the anticipated methods of handling aggregates. The anticipated equipment for mixing, transporting, and placing RCC shall also be included in the submittal. The Contractor shall have the concrete plant on site, in-place, and in operating condition at least 14 days prior to the start of production RCC mixing and placing. The plant may be the conventional mass concrete batch type, a continuous mix rotating tilted drum, or a twin shaft paddle type continuous mix "pug mill." The plant shall be capable of routinely and consistently producing well mixed large aggregate and relatively dry RCC of the type to be used on the project. The mix plant or an equivalent shall have demonstrated satisfactory performance with similar mixes prior to being used in producing RCC at the project. Until the placement is completed to Elevation 6065, the plant shall have a minimum combined peak capacity sustainable for at least 1 hour of at least 735 tons or approximately 350 cubic yards (compacted) per hour using the designated jobiste mix design and aggregate proportions or ratios. For concrete placed above elevation 6065, the plant capacity may be reduced

to 400 tons per hour. If two mix designs are used in construction of the project, the plant shall be capable of simultaneously delivering two different mix designs from separate mixers or of automatically (or semi-automatically) switching from one mix design to another in the same mixer.

6.2 Batch Type Plant.

6.2.1 Bins and Silos. Separate bins or compartments shall be provided for each size group of aggregate. Separate silos shall be provided for bulk portland cement and for pozzolan (if used). The silos and compartments shall be of ample size and so constructed that the various materials will be maintained separated under all working conditions. All compartments containing cement (and pozzolan) shall be separated from each other by a free draining air space. The aggregate bins shall have steep side slopes, large gate openings, and be capable of handling the aggregate in a damp condition without choking.

6.2.2 Weigh Batchers. Aggregate shall be weighed in separate weigh batchers with individual scales.

Bulk cement (and pozzolan) shall each be weighed on a separate scale in a separate weigh batcher. Water may be measured by weight or by volume. If measured by weight, it shall not be weighed cumulatively with another ingredient. The weigh batchers shall be arranged so as to permit the convenient addition or removal of material.

6.2.3 Water Batcher. A suitable water measuring device shall be provided which will be capable of measuring the mixing water within the specified requirements for each batch. The mechanism for delivering water to the mixers shall be free from leakage. The filling and discharge valves for the water batcher shall be interlocked so that the discharge valve cannot be opened before the filling valve is used, a suitable strainer shall be provided ahead of the metering device.

6.2.4 Gob Hopper. If necessary to control segregation and assure effective utilization of the mixer, a gob hopper or holding device of at least twice the volume of the largest hauling vehicle shall be provided for RCC temporarily accumulated after mixing

and while waiting to be loaded into a hauling vehicle for transport.

6.2.5 Moisture Control. The plant shall be capable of ready adjustment to compensate for the varying moisture content of the aggregates and to change the weights of the materials being batched if necessary.

6.2.6 Scales. Adequate facilities shall be provided for the accurate measurement and control of each of the materials entering each batch of concrete. The weighing equipment shall have an accuracy within 0.2 percent of scale capacity. The Contractor shall provide standard test weights and any other auxiliary equipment required for checking the operating performance of each scale or other measuring device. Tests shall be made prior to the start up of RCC placing and at least monthly thereafter. Each weighing unit shall include a visible springless dial which shall indicate the scale load at all stages of the weighing operation or shall include a beam scale with a beam balance indicator which will show the scale in balance at zero load and at any beam setting. The indicator shall have an over and under travel equal to at least

5 percent of the capacity of the beam. The weighing equipment shall be arranged so that the plant operator can conveniently observe all dials and indicators.

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6.2.7 Batching Operation and Accuracy. The measuring operation of each material shall start when actuated by one or more starting mechanisms and stop automatically when the designated weight or volume of each material has been reached. They shall be interlocked in such a manner that the discharge device cannot be actuated until the indicated quantity of each material is within the applicable tolerance. These control requirements can be satisfied by a semi-automatic batching system as defined in the Concrete Plant Manufacturers Bureau (CPBM) "Concrete Plant Standards", with interlocks as described above or by an "automatic batching system" as defined in the "Concrete Plant Standards". The plant shall be arranged so as to facilitate the inspection of all operations at all times. Delivery of materials from the batching equipment shall be within the following limits of accuracy on any individual batch and shall be within 1% accuracy of the

average of any 100 consequentive batches of the same mix design.

MATERIAL	PERCENT
Pozzolan (if used)	Plus-or-minus 3%
Cement	Plus-or-minus 3%
Water	Plus-or-minus 4%
All Aggregate	Plus-or-minus 4%

6.2.8 Mixers for Batch Type Plants. Stationary batch type mixers shall be the tilting horizontal-shaft or vertical-shaft type capable of routinely and consistently mixing the coarse RCC mix without excessive maintenance. The mixers shall not be charged in excess of 95 percent of the capacity recommended by the manufacturer for conventional concrete. Mixers shall be capable of combining the materials into a uniform mixture without spillage and of discharging this mixture without segregation. Mixers shall be provided with an acceptable device to lock the discharge mechanism until the required mixing time has elapsed. The mixers shall be operated at the drum or mixing blade speed designated by the manufacturer or approved for RCC. If no uniformity test data are available, the initial

mixing time for each batch after all materials are in the mixer shall be 1 minute for mixers having a capacity of 3-cubic yards or less. For mixers of larger capacities, the minimum mixing time shall be increased by 20 seconds for each additional cubic yard or fraction thereof of concrete mixed. These mixing periods are predicated on proper control of the speed or rotation of the mixer drum or blades, and on proper introduction of the materials into the mixer. The mixing time shall be increased when such increase is necessary to secure the required uniformity and consistency of the concrete, or when the average variability index of three series of test samples of concrete is less than any of the following uniformity requirements when tested in accordance with the test procedure described in the Appendix to this Section. The mixing time may be reduced when tests indicate that the concrete still meets all the following uniformity requirements:

TEST	VARIABILITY INDEX, MIN
Water content of full mix, % of weight	75
Coarse aggregate content of concrete (Plus No. 4),% by weight	80
Unit weight of air-free mortar, lb/cy	85
Cement content of full mix, % by weight	70

When the Contractor proposes to reduce the mixing time, a set(three) of uniformity tests shall be made at the reduced mixing time to determine whether the reduced mixing time will produce RCC which meets the requirement of these specifications. Each time a decrease or increase in mix time is indicated by test results, the mix time for the following day will be changed by 10 seconds and another series of tests will be run. This process may be repeated until the minimum mix time that still provides acceptable material is established. Mixers shall be maintained in satisfactory operating condition, and mixer drums shall be kept free of hardened concrete. Mixer blades

shall be replaced when worn down more than 20 percent of their depth. Should any mixer at any time produce unsatisfactory results, its use shall be promptly discontinued until it is repaired. Suitable facilities shall be provided for obtaining representative samples of concrete for uniformity tests. All necessary platforms, tools, and equipment for obtaining samples shall be furnished by the Contractor.

NOTES:

(1) The variability index will be based on result of samples taken from the placement after spreading and immediately before compaction. This will include the beneficial or detrimental effect of additional handling and remixing from hauling, transferring and spreading.

(2) Charging procedures of the mixer can have a significant effect on the mixer performance, and shall be considered part of the mixer proficiency test.

(3) The RCC mix in loose form contains no paste slurry, but does have considerable "bulking". Mixers

may not be able to be loaded to their full rated capacity without spilling or poor mixer performance. Establishing the maximum acceptable batch size will be part of the mixer proficiency test purpose.

6.2.9 Sampling Facilities. Suitable facilities and labor shall be provided for obtaining representative samples of materials as they enter the mixer, are discharged from gob hoppers, and from the placement area after spreading but before compaction. All necessary tools, platforms, and equipment for obtaining samples shall be furnished. Approximately 400 pounds of material will be required for each sample.

6.3 Continuous Mix Plant

6.3.1 General. Continuous mix plants shall be capable of producing concrete of the same quality and uniformity as would be produced in a conventional batch type plant.

6.3.2 Bins and Silos. Separate feed bins or compartments shall be provided for each size group of aggregate. Separate silos shall be provided for bulk portland cement and pozzolan (if used). The silos and compartments shall be of ample size and so constructed that the various materials will be maintained separate under all working conditions. All compartments containing cement (and pozzolan) shall be separated from each other by a free draining air space. The aggregate bins shall have steep side slopes, large gate openings, and be capable of handling the aggregate in a damp condition without choking.

6.3.3 Cement and Aggregate Feed. Cement and aggregates shall be uniformly, continously, and simulaneously fed at the appropriate ratios for the mix design desired into the mixer by belt or other acceptable method. Aggregate feed may be accomplished onto a single belt from feed bins for the various size groups through openings at the bottom of the bins.

Each opening shall be provided with a gate that can be locked at the necessary opening size to provide the correct feed rate. The bins shall be kept sufficiently full and shall be of sufficient size to insure a uniform flow of aggregate at an essentially constant rate. It is expected that particular attention may be needed to insure a continuous flow of the Group II aggregate (and blend sand if used) if it is very damp and contains a high content of fines. Cement (and pozzolan) shall be fed continuously in a positive manner which can be controlled by adjusting the belt speed or feed rate. Special attention shall be given to the cement (and pozzolan) feed equipment so that it consistently and uniformly delivers materials even at very low (90 lb/cy) dosage rates. The feed shall be capable of gradual adjustment while in operation. Continuously correcting automated gate openings and/or belt-feed rates by electronic feedback of weight sensor units on the belts shall be used if operation without them is not capable of providing the accuracy, consistency, and quality required by these specifications.

6.3.4 Water Dispenser. A suitable water dispenser shall be provided which is capable of dispensing the mix water within the specified requirements. The mechanism for delivering water to the mixer(s) shall be free from leakage. The meter may be an in-line volumetrically activated flow meter, but it shall be adapted to read or convert the weight of water being added in pounds per minute to pounds per cubic yard of concrete. The valve shall be capable of gradual adjustment during the mixing process to compensate for varying moisture contents in the aggregates. The valve shall be automatically controlled so that it will close if cement and/or aggregate stops entering the mixer at the required rate. This control may be bypassed for cleaning operations.

6.3.5 Gob Hopper. If necessary to control segregation and assure effective utilization of the mixer, a gob hopper or holding device of at least twice the volume of the largest hauling vehicle shall be provided for RCC temporarily accumulated after mixing and while waiting to be loaded into a hauling vehicle for transport.

6.3.6 Operation and Accuracy. The intent of operation for continuous mix plants is that they be operated as much as possible in a continuous manner at one set of feed rates. Shut downs and startups during RCC production placing shall be kept to an absolute minimum. The plant equipment shall be designed, calibrated, and operated so that all materials simultaneously begin feeding into the mixer at the correct rates when the mixer is started, and all materials simultaneously stop feeding into the mixer when it is stopped. No lag or lead time between materials will be allowed at the point where they enter the mixer. After material feeds are shut down, all remaining active material in the mixer shall be discharged. At the first startup after each daily or major shutdown (more than 30 minutes) the first cubic yard of RCC produced by the mixer shall be wasted. If a uniform mix of the required proportions is not being discharged from the mixer at that time, the material shall continue to be wasted until consistent material of the specified proportions is discharged.

Delivery of materials as they are discharged from the mixer and from any gob hoppers shall be within the following limits of accuracy:

MATERIAL	PERCENT
Pozzolan (if used)	Plus-or-minus 3%
Cement	Plus-or-minus 3%
Water	Plus-or-minus 4%
All Aggregate	Plus-or-minus 4%

6.3.7 Mixers for Continuous Type Mix Plants.

Continuous mix plants shall be a rotating tilted drum or twin shaft paddle type pug mill capable of routinely and consistently producing well mixed large aggregate and dry RCC of the type to be used on the project. The mixer (or equivalent) shall have demonstrated satisfactory performance with similar RCC mixes prior to being used in production RCC at the project, or will require a conditional approval by the engineer for the start of production. Mixers shall not be charged in excess of the capacity recommended by the manufacturer for RCC. Mixers shall be capable

of combining the materials into a uniform mixture and of discharging this mixture without segregation. The minimum mix retention time in any continuous mix plant shall be 45 seconds unless mixer tests show that a suitable product can be achieved at lower retention times. Higher mix times will be required if needed to provide adequate mixing. Uniformity tests will be based on results of three random test samples from the placement during the same shift. A method of adjusting the mix time such as by adjusting the angle of tilt of a continuous mix drum shall be provided. The mixing periods are predicated on proper control of the speed of rotation of the mixer drum or paddles, and on proper introduction of the materials into the mixer. The mixing time will be increased when such increase is necessary to secure the required uniformity and consistency of the concrete, or when the average variability index of a set (three) of test samples of concrete is less than any of the following uniformity requirements when tested in accordance with the test procedure described in the Appendix to this Section. The mixing time may be reduced when tests indicate that

the concrete still meets all the following uniformity requirements:

TEST	VARIABILITY INDEX, MIN.
Water content of full mix, % of weight	75
Coarse aggregate content of concrete (Plus No. 4), % by weight	80
Unit weight of air-free mortar, (lb/cy)	85
Cement content of full mix, % by weight	70

When the Contractor proposes to reduce the mixing time, a set (three) of uniformity tests shall be made at the reduce mixing time to determine whether the reduced mixing time will produce RCC which meets the requirements of these specifications. The mixers shall be maintained satisfactory operating condition, and mixer drums shall be kept free of hardened concrete. Mixer blades and pugmill paddles shall be replaced when worn down more than 25 percent of their depth. Should any mixer at any time produce unsatisfactory results, its use shall be promptly discontinued until it is

repaired. Each time a decrease or increase in mix time is indicated by test results, the mix time for the following day will be increased or decreased by 5 seconds and another series of tests shall be run. This process can be repeated until the minimum mix time that still provides acceptable material is established.

Notes:

a. The variability index will be based on results of samples taken from the placement after spreading and immediately before compaction. This will include the beneficial or detrimental effect of additional handling and remixing from hauling, transferring and spreading.

b. Charging procedures into the mixer can have a significant effect on the mixer performance and shall be considered part of the mixer proficiency test.

c. The RCC mixing loose form contains no paste slurry, but does have considerable "bulking". Mixers may not be able to be loaded to their full rated capacity without spilling or poor mixer performance. This will be part of the mixer proficiency test.

6.3.8 Sampling Facilities. Suitable facilities and labor shall be provided for obtaining representative samples of materials as they enter the mixer, are discharged from the mixer, are discharged from gob hoppers, and from the placement after spreading but before compacting. All necesssary platforms, tools, and equipment for obtaining samples shall be furnished. Approximately 400 pounds of material will be required for each sample.

CONVEYING/TRANSPORTING

7.1 General. Concrete shall be conveyed from mixer to placement as rapidly as practicable, by methods which control segregation, contamination, and drying. Methods and equipment for handling, hauling, and depositing the mix shall be submitted the Engineer at least three weeks prior to production opperations If necessary the Contractor shall provide baffles at the end of conveyors and within hoppers to limit free falls, and at other locations that otherwise cause excessive segregation. In general, equipment will NOT be allowed to track mud or other contamination

onto previously placed RCC. This may require using clean crushed rock on haul roads, washing tires of vehicles prior to driving onto the RCC, and other special measures. Unavoidable localized contamination such as at the entrance of access or haul roads onto the RCC may occur but must be cleaned from the RCC prior to placing the next lift. The total length of time from the start of mixing until completion of compaction shall not exceed 40 minutes under any circumstances.

7.2 Temporary Storage Containers. Interim storage at gob hoppers (such as at a central dispatch point on the dam) shall be provided when vehicles are used for hauling, and when direct conveyor systems do not otherwise provide continuous unsegrated delivery to the final placement location. The gob hoppers shall be constructed with adequate capacity (at least twice the largest hauling vehicle load) so that the mixing sequence normally is not stopped or slowed during production if the hauling vehicles are delayed. Gob hoppers shall be

constructed with side slopes and gates that allow for the free flow of RCC without segregation or choking. Telephone or radio communication shall be provided between gob hoppers, the batch plant, and the placement site. If more than one RCC mix is used, gob hoppers shall be emptied of all of one mix before being filled with a mix of a different design.

7.3 Belts. Conveyor belts shall be operated at high speeds which meet production requirements and do not segregate materials. All belts shall be continuously protected to prevent drying by the wind and sun, and overwetting from rain. The equipment shall be designed for low maintenance continuous operation with clean return belt surfaces using no slump low cement factor large aggregate mass concrete, RCC, and structural concrete. Vertical lifts of the RCC mix may be accomplished with bucketed conveyors. Conveyor belt systems shall be designed by personnel fully experienced with belt delivery of mass concrete and familiar with material similar to large aggregate low cement factor RCC. The anticipated system

shall be submitted for prior to the start of RCC production operation.

7.4 Chutes. Unless specifically authorized in writing, chutes will not be permitted. The contractor may propose a chute or "controlled drop" system which may be given conditional initial approval by the engineer, and which will be given conditional approval only after satisfactory field demonstration under sustained production conditions.

7.5 Hauling Vehicles. RCC may be hauled using trucks, large front end loaders, or scrapers. Trucks should be the bottom-dump type, except that end-dump trucks may be used to deposit RCC in confined areas, and trucks with special tailgates or drop control methods which demonstrate an ability to handle dumping without excessive segregation may be used. Experience has shown that conventional end dumps have a tendency to cause segregation at the edge of the deposited material. Any segregation that results from the vertical drop when the truck bed is tipped shall be removed by hand and/or effective reworking during spreading. Hauling vehicles

shall be maintained in good operating condition and will not be allowed to spill or drip oil, grease, or other visible and obvious contamination onto the RCC. All hauling vehicles shall be operated in a manner which precludes tight turns, sudden stops, or other procedures that damages previously compacted RCC. If a surface is damaged by vehicle operation (tight turns, driving over a wet area that pumps, etc.) the damaged surface whall be cleaned or the damaged material shall be removed.

8. PLACING AND SPREADING

8.1 General. It is the intent of this specification to construct the RCC mass in as nearly a continuous operation as is practical. Above elevation 5900, the rate of rise shall be at least 2 ft per day, but not over 6 ft per day. The Contractor shall work at least 7 days per week, two shifts per day during placement of RCC. Placement of RCC shall begin by April 11, 1988, and all RCC in the dam structure shall be completely placed to elevation 6063 before June 6, 1988. All RCC shall be completed by June 17, 1988.

NOTE: Meeting these dates and production rates (including the aggregate production requirements of subparagraph: "Production Schedule" of paragraph "Aggregate" is necessary to obtain the desired in-place material properties and to minimize cracking potential from internally developed thermal stresses. If the contractor fails to meet these requirements, an engineering analysis will be made showing what if any, additional restrictions and requirements must be enforced in order to produce a similar quality final in-place material. The Contractor shall comply with these restrictions and requirements and shall be responsible for all related costs. The special restrictions and requirements will depend on a variety of conditions and technical factors that exist at that time. This includes Contractor's actual start date for RCC placement and his RCC production rate, the time of the year and weather conditions, the ambient and mix temperatures, the stockpiled temperature of the aggregate, and the location within the dam that is effected. The additional restrictions and requirements may include but are not necessarily limited to reduced or accelerated placement rates, use of surface insulation, use of chilled mix water, and placing only during specified hours of the day.

8.2 Weather. Roller compacted concrete shall not be placed when ambient temperatures drop to below 32

degrees F, except that if the surface of the compacted RCC and the temperature of the mix itself stays above 35 degree F, the engineer will permit placing of RCC during temporary periods if the ambient temperature remains above 25 degrees F. If the ambient temperature drops to below 30 degrees F and the surface of any RCC less than seven days old drops to below 35 degrees F, the surface shall be covered with heavy tarps, blankets, straw or other acceptable temporary protection until after the ambient temperature raises to above 32 degrees F. RCC shall not be placed during heavy rains (more than 0.2 inch per hour or 0.03 inch in six minutes as defined by the U.S. Weather Bureau Glossary of Meteorology). Depending on the

Contractor's equipment and placing method, placing may have to be suspended during light rains. Experience has shown that placing RCC during a temporary light mist can usually be accomplished with care, but that if hauling vehicles rather than conveyors are used for delivery; it is relatively easy to damage the surface and subsequently have to clean it. Production shall stop when free surface water begins to accumulate on the compacted RCC or when pumping, tracking or other unacceptable damage begins to develop. If unusual adverse weather such as heavy rain is forecast to occur during placment, an interruption in placing operations for that time period shall be planned.

8.3 Layout of the Placement Area.

It is the intent of this contract to raise the dam at essentially the same level across its entire area, except for the slight upward incline in the downstream direction as indicated on the drawings. After all RCC has been placed to the spillway crest level, RCC may be placed on either side of the spillway training walls independently. As nearly as is practical the Contractors shall expose at one time the surfaces

of only one layer, the preceding layer, and the succeeding layer. One additional layer may be exposed under special circumstances such as crossing the gallery and conduit areas. As placement of a lift progresses the exposed edges shall be kept "live" by progressively placing out from them. Whenever a cold joint at any edge of any lift does occur, it shall be located at least 10 feet from the location of other cold joints that may have previously occurred in the same direction. The joint shall be prepared as required by contract for "cold joints" prior to resumption of RCC placement. No cold joint shall be allowed along the edge of a lift in the upstream-downstream direction for more than one-third of the upstream/downstream dimension of the dam at that elevation.

8.4 Depositing.

Roller-compacted concrete shall be deposited at the location at which it is to be spread. For truck and scraper delivery, depositing will generally be accomplished with a dump-spread action while the vehicle is moving. When it is necessary to dump RCC in a pile from rear end dumps, the pile shall be deposited

on top of the fresh RCC layer being advanced, not on the old layer being covered. If belt delivery is used, the belt shall discharge in a manner that does not cause unacceptable segregation.

8.5 Spreading.

Within 10 minutes of deposition, the mix shall be spread into an even layer that will, after compaction, be a nominal 18 inches thick. However, a thicker layer up to 24 inches can be used if the Contractor demonstrates that it can be spread and compacted without segregation and with a consistent average density similar to that achieved for the 18 inch thickness. Where RCC is spread onto or into bedding mix, the RCC mix shall be spread and compacted within three hours of the time the bedding mix was batched, prior to the time that it begins to set or dry from exposure, and within 40 minutes of when the bedding mix was first deposited. Spreading shall be accomplished with tracked equipment supplemented if desired by a grader. Tracked equipment for spreading shall be limited to a maximum D-7 Caterpillar or equivalent size. Dozers shall have hydraulic tilt and/or angle capability of the blade. In

general tracked spreading equipment shall operate only on uncompacted material and will not be allowed to crab or turn on compacted RCC. A front end loader with operator shall be available to assist with deposition and spreading of materials as needed in confined areas, at irregular foundation conditions, and at other locations as needed. Spreading shall be accomplished in a manner that does not cause segregation. It is expected that this will necessitate spreading two successive layers approximately 9 inches deep before compacting the material as one lift 18 inches deep. If large aggregate revels, rolls or segregates to the outside edge of a spread layer of RCC, it shall be reblended into the RCC or removed. Experience has shown that this may occur when the mix becomes drier than desired and/or when care is not used during dumping and spreading. Large aggregate that rolls to the outside edge of a spread layer can be picked up by laborers with flat shovels and be broadcast over the uncompacted surface so that it is compacted into it without segregation by the roller. All equipment shall be maintained in good operating conditions which does not leak or drip oil, grease, or other obvious visible contamination onto the fill. No concrete shall be placed on a previous layer which is suspect and is being considered for testing or rejection.

9. COMPACTION

9.1 General.

Within 10 minutes of spreading, each layer of RCC shall be compacted with a minimum of four passes of a self-propelled double drum or eight passes of a self propelled single drum vibratory compactor. (A round trip is two passes). A minimum average wet density per lift of 153 pounds per cubic foot shall be achieved. This is the density used in the dam stability analysis. It also is approximately 98% of the practical achievable density using good practice and equipment, and it is approximately 95% of the theoretical air free density. No individual reading of less than 150 pounds per cubic foot will be allowed. This is approximately 93% of the theoretical air free density. The largest size equipment specified below and capable of physical and practical operation in the area to be compacted shall be used. Large width self-propelled vibratory rollers shall be used in open areas. Small walk behind vibratory rollers and/or hand-guided power tampers shall be used for compaction at the abutments and in any areas that cannot be reached with the drum of a large vibratory roller. Rollers shall not be operated in the vibratory mode until they are moving. All compacting

equipment shall be kept in good operating condition at all times and will not be allowed to drip or spill oil or other obvious visible contamination onto the RCC. The edge of all compacted layers against which adjacent RCC is not placed within 25 minutes shall be broken down or trimmed, and rolled so that the edge is thoroughly compacted and does not contain loose segregated aggregate.

9.2 Large Self Propelled Rollers. Self-propelled vibratory rollers shall be of the single or double drum type. They shall transmit a dynamic impact to the surface through a smooth steel drum by means of revolving weights, eccentric shafts, or other equivalent methods. The compactor shall have a gross weight of not less than 20,000 lbs and shall produce a minimum dynamic force of at least 475 lbs per inch of drum width at the operating frequency which is used during construction. The roller shall have a vibrating frequency of at least 1500 cpm (cycles per minute). The roller drum shall be between 4 and 5-1/2 feet in diameter and 5-1/2 to 8 feet in width. The roller shall be operated at speeds not exceeding 1-1/2 miles per hour. The engine driving the eccentric mass shall have a

rating of not less than 125 horsepower. Within the range of operational capability of the equipment, variations to the frequency and speed of operation which result in maximum density at the fastest production rate will be allowed. At least one self-propelled vibratory roller in good operating condition and meeting these requirements shall be maintained full time (with operator) on the placement area at all times during production placement. Standby replacement equipment shall be available for functional operation on the placement within 30 minutes time.

9.3 Power Tampers and Small Rollers.

Power tampers and small vibratory rollers similar to the Dynapac CM20, Dynapac LC70, and Case Model W100, which can operate within a few inches of a veritcal face shall be used to compact the RCC adjacent to form work, near the abutment face, up to the downstream face, and at other areas where the large vibratory rollers specified above cannot manuever. The dynamic force produced by small rollers shall be at least 250 lbs/in of drum width for each drum of a doubledrum unit and at least 350 lbs/in of drum width for a single-drum unit. Rammers

shall be similar to the Dynapac LC70 and shall develop a force per blow of at least 3500 pounds per square foot. At least one small roller, one rammer, and one plate tamper in good condition shall be on the dam. The amount of rolling and tamping required shall be whatever is necessary for the particular equipment to provide an average density of at least 97 percent of the average density that is attained with the large self-propelled vibratory roller. It is expected that this normally will not require more than six passes of any combination of small vibratory roller and compactor. It is also expected that some very localized and acceptable void space between large aggregate particles may occur immediately adjacent to the upstream face precast panels. This would only be apparent if the panels are removed. At least one small roller and three tampers in good operating condition shal be maintained on the placement area during all production operations. Standby replacement equipment shall be available for functional operation on the placement within 30 minutes time. Not more than six passes of any combination of vibratory roller or compactor should be required.

9.4 Compaction at Interface with Conventional Concrete Facing.

Contractor shall be responsible for achieving an intimate contact between the conventional concrete facing mix and the RCC with no discernable joint line by whatever method is most suitable and economical to the Contractor. Acceptability will be determined by horizontally drilled cores. Cores shall be drilled from the formed face of the facing mix and at least 12 inches into the RCC. The total depth of core hole shall be 3 ft. If the Contractor can demonstrate by a test section (including cores) that he can achieve consolidation of the face concrete with a flowable mix by using a high range water reducing admixtrue and the vibration of the rollers, then internal consolidation of the conventional concrete facing mix will not be necessary. Otherwise consolidation shall be achieved by the vibratory rollers supplemented with internal vibration.

10. JOINTS

10.1 General.

It is the intent of this contract to place the entire roller-compacted concrete mass with sufficient continuity so that it hardens and acts as one monolithic block without discontinuous joints or potential planes of separation. The length of time between placement of successive layers of RCC which can be tolerated is dependent upon temperature. Joint quality is also dependent upon cleanliness and surface moisture. When the time limits between successive placements of RCC layers exceeds those described below, a cold joint will be considered to have occurred, and the procedures described below for cold joints shall be followed. Joint surfaces shall be kept in a clean, uncontaminated, and continuously moist condition until placement of the succeeding concrete. Compacted RCC surfaces to receive bedding mix at the upstream face shall be especially clean. RCC surfaces at the upstream half of the dam shall be essentially uncontaminated where the RCC is placed on them. The downstream half of each layer shall also be kept clean

and damp, but occasional isolated rocky or small (one square foot) contaminated areas will not be reason to stop placing. An adequate number of operable nozzles from water trucks or a water distribution line which provide overlapping water mists shall be maintained at the placement area at all times. They shall be used as necessary to prevent joint surfaces from drying and shall be supplemented as necessary by mists from hand-held hoses to reach inaccessible areas. The mist or spray shall not be applied in a channeled or pressurized manner that erodes the fresh RCC surface. It also shall not be applied at a rate which causes ponding at the surface. Trucks shall not be allowed to drip obviously visible oil or other contaminant on the RCC surface. At least one person 24 hours per day, seven days per week shall be on duty on the placement with the sole responsibility of operating the water system to maintain the entire surface moist but not overwatered. He may be allowed to perform routine maintenance of nozzles and move hoses only if these activities do not prevent him from fully accomplishing his responsibility of keeping the entire exposed surface in a continuously damp condition.

10.2 Cold Joint Classification.

Cold joints will be classified as needing a Type I or Type II treatment based on the following criteria. If a joint does not fall into the category of Type I or II, it shall not be considered as a cold joint and no special treatment is required unless the Contractor fails to meet other requirements of this specification such as keeping the joint, and preventing damage to the surface by equipment operations.

Type I: More than 2000 degree hours have passed before placement of the successive layer of RCC, but not more than 54 hours have elapsed. Degree hours shall be determined by accumulating the average temperature in degrees Fahrenheit at the lift surface during each one-hour increment after the surface has been compacted. A continuous clock-type temperature recording device as commonly used in precast concrete construction or other suitable method shall be used for determining and recording the time and temperatures. At least two clocks shall be running and recording at all times during the period that joints have not yet reached the 2000 degree hour or 54-hour cutoff point.

Type II: More than 54 hours have passed before placement of the successive layer.

10.3 Type I Cold Joint Treatment

Cold joints that fall into the category of Type I shall be clean at the time of placement of subsequent concrete on them. No dry areas or ponded water will be allowed. A nominal 1 inch thickness of conventional concrete bedding mix shall be spread over the upstream third of the lift joint surface before placement of the next RCC layer. No additional payment shall be made for any bedding mix or its constituents unless the cold joint was directed by the Engineer for his convenience. The bedding mix shall be spread so that the entire surface area requiring the bedding mix is covered with some of the mix, the maximum thickness determined by dividing the volume used by the area covered is between 3/4 and 1-3/4 inch.

10.4 Type II Cold Joint Treatment:

Cold joints that fall into the category of Type II shall be prepared for the subsequent lift by removing all laitance, loose debris, and contaminants. The cleaning procedure shall expose but not undercut the aggregate. At this stage of maturity, water jetting will be used if air alone does not satisfactorily prepare the surface. After preparation the surface shall be left in a damp condition and treated as a Type I joint, except that the bedding mix shall be spread over the upstream 35% of the joint surface. No additional payment shall be made for any joint preparation, cleaning, or placement of any bedding mix or its constituents unless the cold joint was directed by the engineer.

10.5 Upstream Joint Treatment.

All horizontal joints shall receive bedding mix near the upstream face as indicated on the drawings. The surface to receive this bedding shall be cleared by air blowing prior to placing the bedding mix.

10.6 Horizontal Joints Between Facing Mix Layers.

The top surface of each layer (lift) of conventional concrete facing mix (spillway, training walls, etc.) shall be kept continuously moist until placement of the next layer. The surface shall be cleaned by air blowing prior to placing the next lift, supplemented as necessary by hand picking or shovel scraping to remove loose and/or segregated material along the interface of the RCC and conventional concrete at the top surface. If the conventional mix has set for more than 24 hours and developed extensive hard laitance that is not removed by air blowing, water jetting shall be used in combination with air to clean the surface.

11. REINFORCING STEEL BARS INCLUDING ANCHOR BARS

11.1 General.

Reinforcing steel and anchor bars if used in the RCC shall conform to the requirements of Section "Conventional Concrete," except that placement shall be as modified herein.

11.2 Anchor Bars.

Holes for anchors may be drilled by diamond-core or rotary-percussion equipment. Anchors shall be grouted in place by filling the hole with grout, forcing the anchor bar into the hole, vibrating the bar by holding a concrete vibrator against it until no further settlement of the grout occurs, and protecting the bar from further movement or disturbance for a least the next 6 hours. The bar shall be inserted into the grout and vibrated within 35 minutes after initial mixing of the grout. Prior to grouting, each hole shall be washed until return water from a line running to the bottom of the hole is clean, followed by blowing with the air until the water is expelled from the hole. Cleaned holes shall be plugged to prevent contamination before grouting.

11.3 Placing RCC at Steel Bars.

Reinforcing steel, including the bent legs of anchor bars, shall be located with not less than 1 inch nor more than 4 inch clear distance between the bar and the surface of the RCC below it.

12. GALLERY

12.1 General.

The gallery shall be constructed by using one of the following schemes, the details of which shall be the Contractor's responsibility and shall be submitted for information: (1) precast gallery segments, (2) removable rigid forms against which the RCC is placed, (3) stay-in-place precast-interlocking panels similar to the upstream face panel construction, held in place with straps or anchor bars embedded in the RCC and with a ceiling section made of precast slabs, and (4) using a noncementing fill (such as the RCC mix without cement) as a temporary filler in the gallery area, and then removing it after the RCC has gained sufficient strength to be self-supporting. Regardless of which procedure is used, the bottom of the gallery shall be the exposed surface of an RCC lift, and shall be sloped to drain. In no case will an invert surface be allowed to pond more than 3 inches of water if flooded.

12.1 Precast Gallery Segments.

Precast sections shall be designed by the Contractor to carry the full load of a vibratory roller over the first

lift of fresh RCC above the ceiling section with a sector of safety of 4, and shall be designed to carry the vibrating load of subsequent compaction without deflection that could damage the previously placed layers. Temporary internal bracing to carry vertical loads until after the first three lifts of RCC are placed above the gallery section shall be supplied by the Contractor. The design shall be the responsibility of the Contractor and submitted to the Engineer. The bottom of the gallery shall be the exposed surface of the RCC lift upon which the gallery segments are placed.

12.3 Noncementing Fill Methods.

The gallery section may be constructed by placing a nomcementitious fill in the area where the gallery is to be located, compacting it in layers at the same time that the adjacent RCC is compacted, and later removing the fill. Boards, styrofoam panels or other separators shall be used between the surfaces of the RCC and noncementitious fill to prevent the majority of fill material from bonding to the RCC. Details of how this procedure will be followed, what the noncementitious fill will consist of, and how the fill will be removed

later, shall be submitted for information. An acceptable system shall be established to assure that no noncementing material is placed elsewhere in the structure. In order to visually distinguish this mix from the standard RCC mixes, and to help facilitate later excavation, either Type I or Type II aggregate, or sand shall be used. Excavation of the gallery fill shall not start until the RCC has gained sufficient strength (estimated to be 25 days) to be self-supporting and until at least 30 feet of RCC have been placed above the gallery section. As soon as the strength and cover requirements have been met, the Contractor shall begin the removal of the noncementitious material so as to facilitate any grout and drain hole drilling and other work which should be completed prior to raising the reservoir. Blasting will not be permitted in removal of material from the gallery. As it progresses, excavation shall include removal of separators, scaling for removal of loose aggregate projecting into the gallery area, and trimming of sharp edges of RCC which may have formed at the top or bottom of the separator boards. Scaling may be accomplished with hand tools and small chipping guns. It is intended only to remove loose and semi-detached material and sharp projections which constitute a safety hazard. It

is not the intent to leave a smooth or attractive finish or to do any extensive structural shaping.

13. CURING AND PROTECTION

13.1 General.

The surface of all RCC layers upon which subsequent concrete will be placed shall be kept continously damp and at a temperature above 35 degrees F until it is covered with the next layer. The final top surface of RCC in the spillway and nonoverflow sections shall be continuously moist cured until covered with conventional concrete when it is required, or until an age of 60 days when it is not to be covered with conventional concrete. Curing compounds will not be permitted on the RCC. The exposed face of all other RCC surfaces (such as the downstream face of the dam) will not require cure or protection. The surfaces of RCC layers upon which subsequent layers will be placed shall be protected from erosion by heavy rain and damage by water trucks. Any surface that is damaged by erosion that undercuts coarse aggregates shall be treated as a Type II joint.

14. PRECAST FACING PANELS

14.1 General.

If used, precast panels shall comply with the requirements of Technical Section "Precast Panels." As much as possible, large rollers shall be used for compaction near the panels. Hand-operated tampers and rollers shall be used for compaction adjacent to the panels and where large rollers cannot safely maneuver. Minor chips and occasional hairline cracks that occur accidentally in the panels during erection and construction will be tolerated. Panels with major spalls or cracks shall be replaced.

14.2 Anchors.

Tie-back anchor bars shall be located with at least 2 inches clear space above or below horizontal joints of RCC layers. The end of the bar shall be dipped in "aquatepoxy" paste as produced by American Chemical Company prior to being threaded into the panel insert. The washer and nut shall be tightened against the impervious liner on the back of the panel before the

epoxy sets and so that it fills the void space and seals the washer to the membrane.

14.3 Joints Between Panels.

Joints between precast panels may be shimmed up to 1/4 inch in any direction for alignment. Shims for horizontal joints shall have a bearing area of at least 4 square inches. Prior to placing concrete against the panels, the impervious membranes at adjacent panels shall be heat welded in the field with a lap splice as shown on the drawings. All heat welding shall be done by trained personnel approved by the Supplier using mechanical heat splicing equipment specifically designed for that purpose. Any weld suspected of not being fully fused shall be redone or tested using a holiday detector.

15. CONVENTIONAL CONCRETE FACING

15.1 General.

Except as modified or supplemented herein, conventional concrete for the spillway face, training walls, and any

other location where conventional concrete is to be placed monolithic with the RCC, shall comply with the requirements of Section "Conventional Concrete."

15.2 Mix Proportions.

The exact mix design shall be developed and proven by tests. Materials used in the tests shall be representative of those to be used in the dam. The mix design shall conform to ASTM C94 and shall be within the following general guidelines:

Maximum aggregate size	1-1/2 inch
Maximum amount of 1-1/2 Agg.	20%
Initial air content	4%-10%
Air content after 30 minutes	4%-7%
Water reducing admixture	(hi range) or (hi and normal range)
Maximum water-cement ratio	0.45
Maximum water content	220 Lb/cy
Minimum cement content	375 Lb/cy
Design strength (90 days)	4000 psi
Retarder	optional

All admixtures except the high range water reducer (super plasticizer) shall be added to the mix at the batch plant.

15.3 Placing Procedure.

The high range water reducer shall be added to the mix at the placement location when the area is ready to receive concrete and when RCC is being readied for adjacent placement. The high range water reducer shall be forced from a pressurized tank into the mixer of the mixer truck. A predetermined quantity of admixture, appropriate for the batch size, shall be used. The concrete shall then be thoroughly mixed and immediately discharged. Additional admixture may be added by hand on occasion if necessary to further improve workability. Extra water shall not be added at the placement location, except for that amount necessary to briefly (five seconds by truck hose) flush or rinse down the back of the mixer drum after addition of superplasticizer. The concrete shall be discharged against the form. The RCC shall immediately be spread into the conventional facing mix, and compaction shall immediately follow that. Timing shall be arranged so that the RCC is compacted into the facing mix as it begins to stiffen from slump loss, but before it begins to achieve an initial set. This normally will require compaction within 15 to 40 minutes after addition of the superplasticizer. Compaction will be by large roller as

much as is possible, supplemented by hand-guided tampers. It is anticipated that the facing mix will have a slump in the range of 1/2 to 2 inch prior to addition of the superplasticizer and 4 to 8 inches immediately after mixing it with the superplasticizer. It is also anticipated (and desired) for the facing mix to rapidly lose slump (but not set) so that the RCC can be spread into it and it will tend to support compaction equipment. Internal vibration shall supplement the compaction equipment as necessary to insure full consolidation of the facing mix. The compaction procedure shall effectively force the RCC into the facing so that the two mixes hydrate into a monolithic mass.

15.4 Finishing the Facing Mix.

The facing mix of the spillway shall be kept continually moist starting immediately after the form is pulled away from it. As soon as it is practical after the form is raised, the surface shall be finished to remove any imperfections and ridges, and to fill any holes with "dry-pack" mortar. Immediately after finishing, the surface shall be coated with curing compound.

16. FOUNDATION PREPARATION

16.1 General.

Prior to placing roller compacted concrete on the foundation and against abutments, the exposed rock shall be cleaned, filled with dental concrete (if necessary), shaped (if necessary), and prepared. No foundation or abutment area will be covered with any concrete, bedding mix, or dental fill until it has been accepted by the designated field representative. A bedding mix consisting of a lean conventional wet mix shall be placed at the interface between the rock foundation and RCC mixes.

16.2 Bedding Mix.

The bedding mix shall be the same between the foundation or abutment and the RCC and wherever required between layers of RCC. The bedding mix shall be retarded so that its initial set time exceeds 3 hours at 95 degrees F. It may be retempered within this time period to add moisture lost by evaporation. The bedding mix shall be spread so that all of the surface to receive bedding is covered with some material, and so that the average

thickness over rock foundations does not exceed about 2 inches. The RCC shall be spread over the bedding and compacted into it before it reaches its initial set and within 45 minutes of when it was discharged. The exact mix porportions shall be developed by the Engineer. Materials used in the tests shall be representative of those to be used in the dam. The mix design shall be within the following general guidelines:

Slump	5 - 9 inch
Maximum aggregate size	1 or 1-1/2 inch
Maximum amount of coarse Agg.	55%
Air content	0%-10%
Water reducing admixture	Required
Minimum cement content	400 Lbs/cy
Design strength (90 day)	2000 psi
Retarder	As required to achieve the required delayed set time

Admixtures may be reintroduced into the mix to maintain its workability. The mix should be a very workable and highly sanded mix with some coarse aggregate. It shall be proportioned so that it does not segregate.

16.2 Shaping and Filling.

Shaping by minor rock excavation (trimming) of obtrusive high points and overhangs, and by filling with

conventional dental fill concrete of depressions into which RCC cannot be thoroughly compacted, will be required. Depending on the location, size, shape, and rock quality, trimming may necessitate any one or a combination of the following methods: mechanical ripping, scaling by hand pry bars, jackhammers, surface charges, or small shots of stemmed dynamite in drilled holes. If any large areas or volumes of unacceptable quality or shape rock are encountered, they shall be treated as ROCK EXCAVATION before receiving the foundation preparation required herein. Dental fill concrete may require rough face forming. Vibration and curing of bedding mixes and for dental concrete placed for filling voids in the foundation and placed for panel leveling pads are not required.

16.4 Final Cleanup.

Prior to placing any concrete or bedding mix, the surface shall be cleaned of loose, unkeyed, and deteriorated rock; all mud, silt accumulations, vegetation, grease and spilled oils; and frozen materials, accumulations of gravels, sands, and loose rock fragments; laitance that may have accumulated on dental concrete, and any other detrimental material. It

is expected that most of this can be accomplished with air blowing, high-volume water washing, and/or air/water jetting using equipment normally designed for this purpose and used in large scale foundation cleanups. A clean sound surface is required, but it is not the intent of this specification to require a perfect and polished surface with no exposed rock having visible fractures. All surfaces upon which RCC or any bedding mix is placed shall be damp and at a surface temperature in excess of 35 degree F. The Contractor shall have available adequate equipment for necessary air, air/water, and pressure water jetting of the foundation.

17. SHAPING AND CLEANUP OF THE UNFORMED DOWNSTREAM FACE

17.1 General.

During construction, spillage over the downstream face is expected to occur which will result in an accumulation of loose, uncompacted RCC at the face and along the toe. It will probably have little or no cemented value. During construction, the unformed downstream face shall be routinely trimmed with hand tools to provide a relatively uniform surface and to

improve its appearance. However, it will remain rough and not fully compacted. During or after construction, the face shall be cleared of protrusions and obvious major discontinuities that may exist. It shall then be dragged with a caterpillar-track or other suitable device. Loose gravel or buildup at the toe of the dam which is not advantageous from an aesthetic and/or drainage standpoint shall be removed. Suitable material removed during construction may be used for haul roads or the Contractor's convenience.

18. INSTRUMENTAION

18.1 General.

The Contractor shall be prepared to install instrumentation such as thermal couples and Carlson strain gages (crack meters) if required by the Engineer. The locations and type of instruments will be estimated in general prior to construction, but exact locations and types of instruments will be determined as the job progresses. All instrumentation will be available in sufficient time to be installed at its desired location without delaying construction progress.

19. TEST SECTION

19.1 General.

The Contractor shall construct a test section more than one panel high and containing at least 175 cubic yards of RCC prior to placing RCC in the dam. The test section shall include various techniques and materials to be used in construction of the dam such as the precast panels, joint cleaning, compaction, density testing, bedding mix, facing mix, etc. The test section shall serve as a practice, training, and orientation area, and it may be used to help evaluate the practical effectiveness of various construction methods and pieces of equipment. It will also serve as a practice area for inspection. The Engineer will closely monitor activities during construction of the test fill and provide an informal critique and review session afterward for all those involved, including supervision, inspection, engineering, and craft personnel.

20. TOLERANCES

20.1 General.

Except as supplemented or modified below, tolerances shall be as required in the specification sections for "CONVENTIONAL CONCRETE" and (if applicable) "PRECAST CONCRETE."

20.2 Specific Requirements.

(1) Offsets between adjacent precast facing panels shall not exceed 3/8 inch in finished construction.

(2) Gradual variation of erected precast facing panel lines shall not exceed 1 inch in 10 feet, 2 inch in 30 feet, and 3 inch in 100 feet.

(3) Offsets of adjacent precast gallery segments or panels (if used) shall not exceed 1-1/2 inch.

(4) Variation from the lines and grades of the gallery walls and ceiling from that shown on the contract drawings shall not exceed plus or minus 6 inch (2 inch for the floor), except that tolerances at the gallery

entrances shall be kept within the limits necessary for doorways to fit and function properly.

(5) Allowable gradual overbuild of the downstream face of the dam shall be limited to 5 feet. Underbuild will not be allowed.

(6) Variation in the downstream face of the dam from a straight line shall not exceed 1 foot in 40 feet, 2 feet in 100 feet, and 3 feet in 200 feet.

(7) The thickness of individual compacted layers of RCC shall be within plus or minus 2 inches of that stipulated.

(8) The elevation of the surfaces of roller compacted concrete layers upon which subsequent concrete is placed shall not vary by more than 6 inches from the design elevation, except that the elevation of the top three lifts shall be within 2 inches of that shown on the drawings.

21. QUALITY CONTROL

21.1 General

The Contractor shall establish and maintain an effective quality control program with the Engineer for the roller compacted concrete. It will be the means of ensuring compliance with contract requirements and of maintaining records of control, including all tests and inspections, their findings, and the remedial actions taken when necessary.

The Contractor's program will be established and run under the supervision of a full time RCC Quality Control Engineer who will review and approve all activities concerning the production of materials, planning and scheduling of construction activities for placing RCC, and the running and evaluation of RCC tests. The RCC Quality Control Engineer shall work closely with the Engineer and shall keep the Engineer advised of planned construction procedures, the placing schedule, the testing schedule, and results obtained from tests. The Contractor will designate an RCC Quality Control Engineer for each shift and may have several personnel in the various

areas of activity monitoring, testing, and reporting. All information and test results will be made available to the Engineer. The Engineer will be permitted full access to all construction and Contractor quality control activities.

The RCC Quality Control Program shall include but not be limited to the following: aggregate manufacture and gradation, moisture, batching requirements and mix proportions at the batch plant, mix delivery, compaction, joints, insuring adequate materials are on hand, embedded items, erection of precast facings, and all other tests and inspections required by these specifications. The program shall be similar to that described in the following paragraphs.

21.2 Aggregate Gradations.

21.2.1 Testing: At least once during each shift that concrete is placed and once during each shift that aggregates are produced, gradations shall be checked for each aggregate size used or produced, and

for the combined gradation of all aggregates batched at the designated proportions. A recheck sample is required for any combined gradation test out of specifications. The location from which samples are taken may be selected by the Contractor providing that they give an accurate indication of gradations of materials as they enter the mixer. However, provisions must be made for accurate sampling of aggregates on feed belts to the concrete plant.

21.2.2 Action Required: Whenever a test result is outside of the specification limits, the recheck sample shall be taken. If the recheck sample is outside of the specification limits, the process shall be considered out of control, and positive steps shall be taken by the Contractor to rectify the situation. The Engineer and the Contractor's Quality Control Engineer will jointly decide if production and placement of concrete shall be stopped at that time. Except in extreme cases that

represent a serious design concern, construcion will be permitted to continue during the shift after the gradation problem was identified and while it is being corrected. It is expected that the problem will be corrected by the end of the second shift after it is identified. The Contractor will be responsible for all costs incurred as a result of stopping any concreting operations due to out-of-specification materials.

21.3 Aggregate Moisture Determination.

21.3.1 Testing: At least once during each day of placement for each aggregate size used, moisture content determinations shall be made in accordance with ASTM C 566 (ASTM C 70 where appropriate for fine aggregate if it is stockpiled separately). The location from which the sample is selected may be determined by the Contractor providing that it is typical of materials entering the mixer.

21.3.2 Action Required: When moisture content determinations indicate a change in water entering the mix with the aggregates, the placement foreman shall be contacted to see if a corresponding adjustment in water added at the mixer is necessary to obtain adequate compaction at the placement site and improve placing conditions.

21.4 Particle Shape.

21.4.1 Testing: During the period of initial aggregate production and initial stockpiling, frequent tests shall be made in accordance with CRD-C 119 to determine the amount of flat and elongated particles. After it has been established that a problem does not exist, and if production procedures remain constant, continued testing shall be required only once per week.

21.4.2 Action: Two consecutive failures in the same sieve size or for the overall gradation shall require action to be taken

to correct the deficiency. Except in extreme cases that represent a serious design concern, aggregate production will be permitted to continue during the shift after the problem was identified and while it is being corrected. It is expected that the problem will be corrected by the end of second shift after it was identified. Materials produced "out of spec" shall be blended with materials "in spec" so that the composite as used in RCC meets specification requirements.

21.5 Material Finer than the No. 200 Sieve.

21.5.1 Testing: During the period of initial aggregate production and stockpiling, frequent tests shall be made to determine the amount of material passing the No. 200 sieve for the combined gradation requirement. Washing of the coarse aggregates shall be used to accumulate all minus 200 material in the combined gradation. Liquid limit and plasticity indexes shall be determined for the minus

50 materials. After it has been established that a problem does not exist, and if production remain constant, continued testing will be required once per week.

21.5.2 Action: When testing for material finer than the No. 200 sieve indicates excessive quantity or plasticity, corrective action shall be taken to correct the deficiency. Except in extreme cases, aggregate production will be permitted to continue during the shift after the problem was identified and while it is being corrected. It is expected that the problem will be corrected by the end of the second shift after it was identified. Materials produced "out of spec" shall be blended with materials "in spec" so that the composite as used in RCC meets specification requirements.

21.6.1 Aggregate Quantities.

21.6.1 Monitoring: The Contractor shall accurately monitor and record the quantity

of each aggregate produced and used during each shift. A record of the cumulative amount of each aggregate shall be maintained. Quantities shall be based on saturated surface dry (SSD) moisture conditions.

21.7 Concrete Plant Control.

21.7.1 Routine Control and Reporting. When the concrete plant is operating, the amount of all constituent materials including cement, pozzolan, each size of aggregage, water and admixtures shall be continuously controlled. The aggregate weights and amount of water to compensate for free moisture in the aggregates shall be adjusted as necessary. A daily report shall be prepared indicating the type and source of cement used during that day, aggregate size groups used, required mix proportions per cubic yard each mix design used, the amount of water as free moisture (above SSD) in each size of aggregate, and the aggregate weights per cubic yard for

each mix design of concrete made during plant operation. The report shall include the total amount of each material used per shift for each mix.

21.8 Scales (if used) for Weight Batching.

21.8.1 Tests and Checking: The accuracy of scales shall be checked by test weights prior to the startup of concreting operations. Rechecks shall be made at least every 60 shifts of operation thereafter. Such tests shall also be made whenever there are variations in properties of the RCC that could result from batching errors. The accuracy of each batching device when weigh batching procedures are used shall be checked frequently during the startup of operations by noting and recording the required weight and the weight actually batched.

21.8.2 Action Required: Whenever either the weighing accuracy or batching accuracy is found not to comply with specification requirements, the plant shall not be operated until necessary adjustments or

repairs have been made.

21.9 Volumetric Feed Calibrations.

21.9.1 Tests and Checking: The accuracy of volumetric feeds shall be checked by collecting all material delivered during a unit of time to the mixer and also by washout tests of material exiting from the mixer. Suitable methods and equipment shall be provided for obtaining and handling samples at the concrete plant. The weight of material corresponding to a standard time interval, and the resulting proportions of materials per cubic yard shall be determined. The accuracy of volumetric feeds shall be determined at least three times during checkout of the concrete plant prior to production operations and RCC placement. Rechecks shall be made at least every 60 shifts of operation thereafter and whenever there are variations in the properties or control

of RCC that could be the result of volumetric feed errors. The sample shall be of sufficient size to give accurate determinations. This may require weights in excess of 400 pounds per item checked.

21.10 Testing Concrete Mixers.

21.10.1 General: Fresh concrete shall be sampled and tested for compliance with the specifications at the placing location. The Contractor shall provide a method of readily obtaining representative RCC samples from the plant and gobhopper locations.

21.10.2 Mixer Performance: A complete mixer performance test of three different batches of concrete or runs through a volumetric plant shall be made on each mix prior to the start of concrete placing. Additional tests may be made at any time to support a Contractor's request for reduction of mixing time. Whenever mixer adjustments

are necessary because of failure of a mixer to comply, the mixer shall be retested after adjustment.

21.11 Temperature.

21.11.1 Testing: Near the start and end of each shift, at least one test of temperature shall be made at the batch plant and at the placement on randomly selected batches of each mix design of concrete used per shift of placement. Additional tests shall be made when rapid set time or workability loss is reported by the placing foreman or Engineer, or when hot or cold weather problems occur. The temperature of air and concrete shall be recorded during the period of cure and cold weather protection when those restrictions are applicable.

21.11.2. Action Required: Temperatures shall be included as standard data in the quality control reports.

21.12 Moisture Content.

21.12.1 Tests and Checking: At least once during each 4 hours of production placement at the batch plant, and once every 4 hours at the placement site, the moisture content shall be determined on the RCC mix. Moistures determined at the placement shall be made using a nuclear gage in the direct transmission mode immediately after compaction. The probe shall be driven to the full depth of the lift for each reading. The gage shall be calibrated against oven-dry samples of each mix design used. The calibration shall be verified to oven dry materials at least once per 20 shifts. At least three tests shall be made in different areas of each layer of RCC placed. The placing foreman shall continuously monitor the apparent effectiveness of compaction equipment from a visual standpoint, and shall notify the batch plant whenever the mix becomes too dry or too wet.

21.12.2 Action Required: Whenever moisture content tests indicate a significant change from what has been established as the optimum batching and placing moisture, the placing foreman shall be notified and the behavior of the mix under conditions at that time shall be observed. Whenever the placing foreman observes a condition of moisture which begins to consistently allow the vibratory rollers to sink excessively in the mix, cause excessive paste to develop at the surface, or leave an open appearing unconsolidated surface, an adjustment shall be made in the mix water added at the plant and the adjustment shall be noted.

21.13 Cement.

Samples of the RCC mix will be obtained from the placement area for determination of cement contents using a chemical calcium analyzer or other acceptable procedure. All testing will be done jointly with personnel from both the Contractor and Engineer.

21.14 Concrete Specimens for Compressive Strength Tests.

The Engineer will furnish molds for nominal 6 x 12 inch RCC test cylinders. Forms for cylinders will be rigid reusable type with disposable liners. The Contractor (with the assistance of the Engineer)

will make all test samples, transport them, cure them, test them, and extract them from the molds.

21.15 Density.

21.15.1 Tests and Checking: At least once every 2 hours during placement, the density of RCC after compaction shall be determined with a nuclear density gage previously calibrated against each mix design used. At least two double probe nuclear gages in good working condition shall be available, and at least one gage shall be on the placement area at all times. The Contractor will provide and operate the gage. Each layer of RCC shall be tested by the nuclear gage in at least six separate locations for density. The direct transmission mode shall be used. The gage shall penetrate the full depth of the lift. eject

Readings shallbe taken in each quadrant of a circle made by rotating the gage 90 degrees around the transmission probe after each reading. The probe shall be driven to the full depth of the RCC layer for each reading, and may also be driven to lesser depths for supplemental information.

21.15.2 Action Required. Whenever the nuclear gage indicates a wet density of less than ___ Lbs per cubic yards, a retest shall be made. If the retest indicates incomplete compaction, additional rolling shall be immediately provided, and a determination shall be made as to whether the lower density resulted from insufficient passes of the roller or a change in the mix properties. If the mix properties have changed, adjustments such as increasing or decreasing the moisture content shall be made at the batch plant. If the problem persists and if the lower density is the result of incomplete rolling, the operator shall be notified and the Engineer may require removal of the incompletely compacted material. If the

same operator repeatedly rolls less than the required number of passes, he shall be replaced with a different operator.

21.16 Compaction Equipment

21.16.1 Tests and Checking. Before any compactor is used in RCC construction, it shall be checked for correct dimensions, weight, and vibratory capacity. At least once per 12 shifts of use, a spot recheck of frequency shall be made. At least once per each shift of placement for the first 5 days of operation by any new operator, his performance shall be spot checked for the correct number of passes, correct spread, coverage of the area being rolled, and good rolling practice. Thereafter, spot checks shall be made on each operator at least weekly, and the results shall be noted in the quality control reports.

21.16.2 Action Required. Compaction equipment not meeting the physical dimensions and weights required shall be removed from the project. Any roller having improper frequency shall be corrected before being used for RCC compaction. Roller operators running at speeds in excess of specification requirements shall be immediately notified and shall correct any noted improper practices or be replaced by another operator.

21.17 Dumping and Spreading.

21.17.1 Tests and Checking. The placing foreman or other designated representative shall continually observe and monitor dumping and spreading operations to insure that they are done in a manner that minimizes segregation. Each layer of RCC shall be routinely checked in its spread condition for evenness and thickness so that smooth, even, compacted layers within the specified thickness tolerances result after compaction. A laser or optical level shall be used in conjunction with hand-carried

vertical rods to determine the elevation of (and subsequently the thickness) of each layer. It shall be the type that emits a rotating or constant light beam in a fixed plane.

21.17.2 Action Required. Whenever thickness checks on uncompacted RCC indicate an excess or shortage of material, the layer shall be immediately bladed off or supplemented as needed. Whenever the thickness or elevation exceeds the allowable tolerances, the Engineer shall be notified and determine with the Contractor what corrective action, if any, is necessary.

21.18 Preparation for Concrete Placement.

21.18.1 General. Foundations, construction joints, forms, and embedded items shall be inspected in sufficient time prior to each concrete placement by the Contractor in order to assure that the area is ready to receive concrete. Forms and facing panels shall be checked closely for condition,

support, alignment, and dimensions. The placing foreman shall be the first level of supervision of all placing operations. The RCC Quality Control Engineer shall determine that the correct mix design of concrete is placed in each location. The designated quality control man shall be responsible for measuring and recording concrete temperatures, ambient temperatures, weather conditions, time of placement, yardage placed, and method of placement. The placing foreman shall not permit placing to begin until he has verified that an adequate numer of vibratory rollers and spreading equipment of the right size, in working order, and with competent operators are available.

21.19 Curing, Protection, and Joint Surfaces.

21.19.1 Moist Curing. At least every four hours, around the clock, 7 days per week, an inspection shall be made of all areas subject to moist curing and joint protection. The surface moisture condition shall be noted and recorded. If an

isolated area has been allowed to dry, that area shall be considered as improperly cured. The Contractor shall IMMEDIATELY wet the area and take positive steps to insure that the problem does not reoccur.

21.19.2 Protection. At least every eight hours, around the clock, 7 days per week, an inspection shall be made of all areas subject to cold-weather protection or protection against damage. Deficiencies shall be noted. During removal of cold-weather protection, measurement of concrete and ambient temperature shall be made at least every two hours.

21.20.1 General. Concrete Plant control records and all results (both passing and failing) of tests conducted at the project site shall be recorded daily and shall be formally transmitted to a designated representative of the Engineer within two days after the end of each weekly reporting period. These requirements do not relieve the Contractor of the obligation to report certain

failures immediately as required in the preceding paragraphs. Such reports of failures and the action taken shall be confirmed in writing in the routine reports.

MIXER AND PLACING PERFORMANCE TEST PROCEDURE FOR ROLLER COMPACTED CONCRETE

1. SCOPE

1.1 The procedures described here are used to evaluate how thoroughly roller compacted concrete (RCC) is mixed prior to compaction but after it has been placed during construction. They determine water content, cement content, and coarse aggregate content for the full mix, and unit weight of the air-free mortar. By comparison of results from three samples taken throughout a shift of placing, the ability of the mixing and placing procedure to provide consistent material is determined.The Engineer may authorise deletion of the cement test portion of the procedure or use of an alternate method.

2. GENERAL

2.1 Mixer performance tests are used to establish minimum allowed mix times for conventional concrete or to determine the variability of a mix after a specified duration of mixing. Normally, the procedure consists of mixing a specified batch size for the designated length of time, obtaining samples of material from different locations in the mixer, and comparing results of tests on each sample to see how much they vary.

2.2 The procedure usually follows Corps of Engineers Test Method CRD-C 55 or the method allowed in ASTM C-94, Annex A1. A brief description of each test follows:

CRD-C 55 is based on three samples taken from a single batch. It checks for water content, cement content, unit weight of the air-free mortar, and coarse aggregate content. It requires about 1 week before results from all of the tests are analyzed and data by the chemical method of cement determination on hardened mortar is available. Most of the testing is performed on the mortar portion of the mix after after sieving from the full mix.

ASTM C-94 is based on two samples taken from a single batch. It tests for slump, air content, coarse aggregate content, unit weight of the 1-1/2 inch minus portion of the mix, compressive strength at 7 days, and unit weight of the air-free mortar. It requires 1 week before all results are available, and it does not use the full mix in all of the test procedures.

2.3 Each of these procedures (ASTM & CRD) were developed for conventional concrete mixes, and each checks only the effectiveness of the mixer itself. Blending of roller compacted concrete materials can be significantly affected during transporting, rehandling, and spreading operations. Generally this can be expected to provide additionl beneficial mixing, although with improper practices, it could adversely cause segregation. This additional mixing can be used to advantage by taking samples from the placing area at differnt times during a shift rather than taking them from a single batch in the mixer. Using this technique, different batches of concrete are intermixed and the composite effect of "batch-to-batch" variability as well as "within batch" variability is taken into account. Accordingly, the maximum permissible variability of the tested samples is increased from typical values used when only within batch testing is done.

2.4 An additional significant advantage of sampling from the placement instead of taking samples from one batch at the mixer is that fresh samples are used in the tests. In CRD-C 55, three samples are obtained at the same time for testing. In practice, the third sample may be hours old before it is subjected to the tests, and results can be adversely affected.

2.5 Another major advantage of obtaining samples from the placement is its compatibility with continuous mixers.

2.6 The procedure described here also uses a rapid chemical analysis for determining the cement content, and results of the entire test procedure are normally available by the start of the day following the tests.

3. SAMPLING

3.1 After an agreed initial and conservative mix time has been established, a shift of placement shall be permitted at that mix time while teting is performed. If no guidance is provided in the contract documents for an initial mix time, it shall be 1 minute for mixers with a capacity of 3 cubic yards. For larger mixers, the time shall be increased by 15 seconds for each additional cubic yard or fraction thereof. The mix time shall begin after all ingredients have entered the mixer and shall stop after the discharge cycle begins. For continuous mix plants, the mix time applies to equivalent discharge rates in cubic yards per minute.

3.2 During the shift of placement, samples shall be obtained during the first, second, and third portions of the shift. However, the first sample shall not be taken until after the plant has been in operation for at least the first 5 percent of the shift. The second sample shall not be obtained until after testing on the first sample has progressed to the point that it will be essentially completed and so that full attention can be given to the second sample as soon as it arrives at the laboratory. Similarly, the third sample shall not be obtained until testing on the second sample is essentially complete. All testing (except waiting for final drying where applicable) shall be completed within 2 hours after obtaining each sample.

3.3 The samples shall be obtained at random under the direction of the laboratory chief. They shall be excavated from the approximate center of a zone of RCC that has been spread for compaction by production methods, but prior to compaction. The sample size shall be approximately 5 cubic feet or 650 pounds. It shall include essentially all material through the full depth of the spread RCC layer. A recommended method of obtaining the sample is with a 1-ton 4x4 pickup truck and front-end loader assisted by two laborers with shovels. The sample shall be covered with a tarp and protected from wind, rain, sun, etc. while being transported. The sample shall be kept in, and all testing shall be done in, a protected environment out of the wind and sun and within a temperature range of 40 to 80 F degrees.

4. WATER CONTENT

4.1 Obtain approximately 1/2 cubic foot or 75 pounds of the full RCC mix and determine the sample weight to the nearest 0.1 pound. This sample should be taken and weighed as soon as possible after the bulk material is delivered to the laboratory. After weighing, spread the RCC material loosely in a series of pans that will fit into an oven. Be careful not to lose any of the sample. Preferably the sample should be initially weighed in the drying pans.

4.2 Dry the sample to constant weight, keeping it loosely separated to facilitate drying. Initial drying for 1 to 6 hours shall be accomplished by simple exposure to the dry atmosphere of an enclosed room, the sun, a breeze from an electric fan, etc. After initial drying, separate the material with a trowel or hoe to assure that it is still loosely separated and not hydrating into a solid mass. Then dry the

sample to constant weight in an oven at 230 plus or minus 9 F degrees. Drying for 12 hours in the oven after the initial drying period is usually sufficient to obtain constant moisture.

4.3 Compute the total moisture content as follows:

$$P = \frac{W - D}{D} \times 100$$

Where:

P = The total moisture content in the mix (%)

W = The weight of the sample prior to drying (lbs)

D = The weight of the sample after drying (lbs)

5. UNIT WEIGHT OF AIR-FREE MORTAR

5.1 Sieve approximately 1 cubic foot or 150 pounds of the full RCC mix sample over a 2-inch screen. The sieving action shall be vigorous enough so that lumps of pasty material, mortar, and smaller aggregate fall through the screen. This can be accomplished by raking the sample back and forth over the screen as it is suspended above a platform or wheelbarrow. The retained coarse aggregate particles should not be washed and will have a slight coating of cement paste with fines. Discard the retained coarse aggregate.

5.2 Compact the material passing the 2-inch screen into a minimum 1/4 cubic foot pressure-type air pot and determine the compacted weight of the sample. Compaction should be accomplished in three lifts, compacting each lift with a pneumatic tamper prior to placing the next lift. The pneumatic tamper shall be equivalent to an Ingersoll-Rand Model SPG-30 with a 5-3/4 inch lock-type butt. The butt shall have a flat (not eliptical) striking surface, but the outside edge may be rounded. As the last lift is compacted in the mold, a helper should simultaneously add material so that the final compacted surface is essentially level with the top of the container. (A collar that fits over the container is recommended to allow initial overfilling of the loose material and to protect the edge of the container from damage during compaction.)

5.3 Determine the air content of the compacted sample using the ASTM C-138 (Gravimetric Method) or ASTM C-231 (Pressure Method).

5.4 Wash the entire air-content test sample over a No. 4 sieve so that all material including paste and fines adhering to the plus No. 4 material is removed. Waste the minus No. 4 material. Obtain the saturated surface dry weight of the aggregate from the air content test sample that is retained on the No. 4 sieve. Towels and a fan will expedite drying the aggregate back to a saturated surface-dry condition.

5.5 Calculate the unit weight of air-free mortar as follows:

$$M = \frac{b - c}{V - \left(\frac{V \times A}{100} + \frac{c}{G \times W} \right)}$$

Where:

M = Unit weight of air-free mortar (lb/cu ft)

b = Weight of the concrete sample compacted into the air content test container (lb)

c = Saturated surface-dry weight of aggregate from the air content test container that is retained on the No. 4 sieve (lb)

V = Volume of the air content test container (cu ft)

A = Air content of the test sample (%)

G = Specific gravity of the coarse aggregate

W = Unit weight of water (62.3 lb/cu ft)

6. PERCENTAGE OF COARSE AGGREGATE

6.1 Obtain approximately 1-1/2 cubic feet or 200 pounds of the full mix RCC sample and accurately determine its weight. Wash the sample to remove all material passing the No. 4 sieve, including all coatings on the coarse aggregate particles. To facilitate the washing operation, the weighted sample should be washed in smaller increments of approximately 20 to 30 pounds each, and a series of nested screens should be used above the

No. 4 screen to avoid overloading. Discard everything passing the No. 4 sieve. Save everything retained on the No. 4 sieve.

6.2 Dry all washed material retained on the No. 4 sieve to a saturated surface-dry condition and obtain the weight. Towels and an electric fan will expedite drying the aggregate back to a saturated surface-dry condition.

6.3 Compute the percentage of coarse aggregate (as percentage of the total mix by weight) as follows:

$$C = \frac{W - w}{W} \times 100$$

Where:

C = Percentage of the full mix that is coarse aggregate (%)

W = Weight of the full mix sample that is taken for sieving (lbs)

w = Weight of saturated surface-dry aggregate retained on the No. 4 sieve (lbs)

7. CEMENT CONTENT

7.1 Cement contents of each sample shall be determined for the full mix using the calcium analyzer and suspension tank (washing machine) of the concrete quality monitor (CQM). The CQM as used for conventional concrete is described in Publication TR M-293 titled "Corps of Engineers Concrete Quality Monitor: Operations Guide," dated May 1981. It is available from the U. S. Army Corps of Engineers Engineer Research Laborotory, Champaign, Illinois. Modifications for use with RCC include adding Calgon to the wash water, using a larger sample, and providing more sieves to facilitate the washing operation. Extra care must be taken and extra effort is normally required when using roller compacted concrete as compared to conventional concrete because of the typially lower cement factors and higher fines content.

7.2 Fill the suspension tank with tap water to the 10-gallon mark on the side of the tank.

7.3 Provide a nest of the following stainless steel sieves above the suspension tank: 3/4 inch, No. 4, No. 30, No. 50, No. 100. (The sieves may be modified to best suit the gradation of the RCC and to facilitate washing, but the bottom sieve must be the No. 100).

7.4 Turn on the recirculating pump for the suspension tank and add 1/2 pound of Calgon Water Softener to the water. Be careful to keep the recirculating hose above the screens. Do NOT lose any of the recirculated water. If the hose slips from above the screens and water is lost or pumped out of the system at any time, the test procedure shall be restarted.

7.5 Obtain approximately 8 pounds of the full RCC mix and place it into one or more polyethylene open-top tubs. Special attention must be given to obtaining representative samples, especially if they contain large aggregate in excess of 1-1/2 inches. Accurately determine the total sample weight.

7.6 Transfer the sample to the nest of sieves over the suspension tank, taking care not to overload the sieves or restrict the flow of water through them. Depending on the gradation of the mix and the amount of fines, it may be advantageous to wash about 4 pounds of the sample at a time. Carefully wash the aggregate, using water from the recirculating pump hose. It may be necessary to individually hand rub or scrub the aggregate particles with a fine wire bristle brush to remove all adhering cement and fines. After material retained on the top sieve has been washed, the material and sieve may be removed so that washing can proceed for the material retained on the next sieve, etc. Special care shall be taken to thoroughly rinse the finer aggregates that cannot be individually scrubbed. Rubbing them over the screen will help.

7.8 Obtain a sample of the recirculated water from the suspension tank. With the recirculating pump operating, insert a stirring spoon and vigorously stir the 10 gallon solution. Immediately extract a sample of the solution with a 30 ml syringe pipet. Place the suspended material in a 500 ml Erlenmayer flask. Refill the syringe pipet with 5 percent nitric acid and then discharging it into the contents of the Erlenmeyer flask. While discharging the acid solution from the syringe pipet, shake it occasionally to ensure that all cement that settled out when the samples was taken has dissolved and is flushed out with the acid

solution. Use a volumetric flask to add 250 ml of top water to the Erlenmeyer flask.

7.9 Put a magnetic stirring bar in the Erlenmeyer flask and place it on a magnetic stirrer. Turn on the stirring motor and stir for at least 3 minutes before taking the first sample from the flask for analysis. Continue stirring until the last sample has been taken from the flask.

7.10 Turn the calcium analyzer power switch to "on."

7.11 (This step is required each time the calcium analyzer cuvette is filled with new potassium hydroxide solution. A single cuvette filling is sufficient for 15 to 20 readings.) Fill the cuvette to the indicated mark with 1.0 N (Normal) potassium hydroxide and 100 µl (Eppendorf) of reconstituted calcein reagent. Put the cuvette in the analyzer, add 100 (Eppendorf) of calcium standard solution and push the titration button to condition the cuvette for analysis.

7.12 Begin the analysis by placing the m Eq/mg% toggle switch on the mg% position and adding 100 µl (Eppendorf) of the calcium standard to the cuvette. Press the titration button. Record the result and repeat the test by adding another 100 sample. Continue repeating until consecutive results are less than 1.5 percent apart. Push the calibration button and run an additional 100 sample of the calcium standard to ensure that the calcium standard readout value is 10 plus or minus 0.1 mg percent.

7.13 Determine the strength of the cement solution in the Erlenmeyer flask by analyzing a 20 µl sample in the calcium analyzer. Repeat by adding more 20 µl samples from the Erlenmeyer flask until the values are less than 1.5 percent apart. If the calcium meter readings are less than 5, the sample size and basis for calibration may be increased to a 100 sample.

7.14 Determine the cement content by entering the calibration graph with the stabilized calcium analyzer reading. If the sample size (approximately 8 lbs) is different from the size on which the calibration chart is based, the readings should be mathematically adjusted by proportioning to correct for the difference.

7.15 (This procedure is necessary prior to running any production tests.) Establish a calibration graph indicating the cement content in pounds per cubic yard as a function of the calcium analyzer reading (sample attached). This is

accomplished by making small batches of thoroughly mixed RCC in the laboratory using accurately weighed ingredients ingredients and variable cement contents. For each mix, the calcium analyzer reading corresponding to the known cement factor is determined. The points are plotted and a straight line graph is drawn.

7.15.1 The batch size should be not less than 2-1/2 cubic feet mixed in a mixer of at least 3 cubic feet capacity. Dampen the mixer drum prior to mixing, preblend the weighed materials (except water) by shovel on a hard floor prior to adding them to the mixer. Add the dry materials intermittently with the water while the drum is rotating. Use the last of the water to rinse down the inside edge of the mixer drum while it is rotating. After all ingredients have been added, mix for 3 minutes at the flattest drum angle that does not spill material out of the mixer. Rest for 3 minutes and scrape the inside surfaces of the mixer drum and blades. Mix again for 2 minutes and discharge onto a damp hard floor. Scrape all materials from within the mixer drum and remix the material on the floor by shovel. Obtain a representative sample and run the calcium analyzer test on the mix. It is desirable to use exactly the same sample size (8 pounds) for each calibration test. If different sample sizes are used, the results must be mathematically adjusted to normalize them to the same initial sample size on which the calibration chart is based.

7.15.2 As a minimum, four mixes shall be used to establish the calibration curve for each cement factor and for each set of mix proportions used in production. For each mix, all batch quantities except cement shall remain constant. One mix shall be at a cement factor of zero (none) pounds per cubic yard. One mix shall be at the designated cement factor. One mix shall be at approximately 15 percent less than the designated cement factor. One mix shall be at approximately 15 percent more than the designated cement factor.

8. REPORT

8.1 A report including the variability index for each test shall be prepared following the format shown on the attached sample sheet.

ROLLER COMPACTED CONCRETE

MIXING AND PLACING PERFORMANCE

PROJECT _____ DATE _____

RESPONSIBLE OWNER/ENGINEER REPRESENTATIVE _____

MIX DESIGNATION _____ AGGREGATE _____

BATCH SIZE _____ _____

MIXING DURATION _____

TEST RESULTS

	Sample 1	Sample 2	Sample 3	Variability Index*	Minimum Allowed	Result
Cement (kg/m3)	74.8	70.0	61.1	81.7	70	Pass
Water (%)	8.2	8.0	7.7	93.9	75	Pass
Unit Weight of Air-Free Mortar (kg/m3)	80.8	79.8	79.7	98.6	85	Pass
Coarse Aggregate	48	44	52	84.6	80	Pass

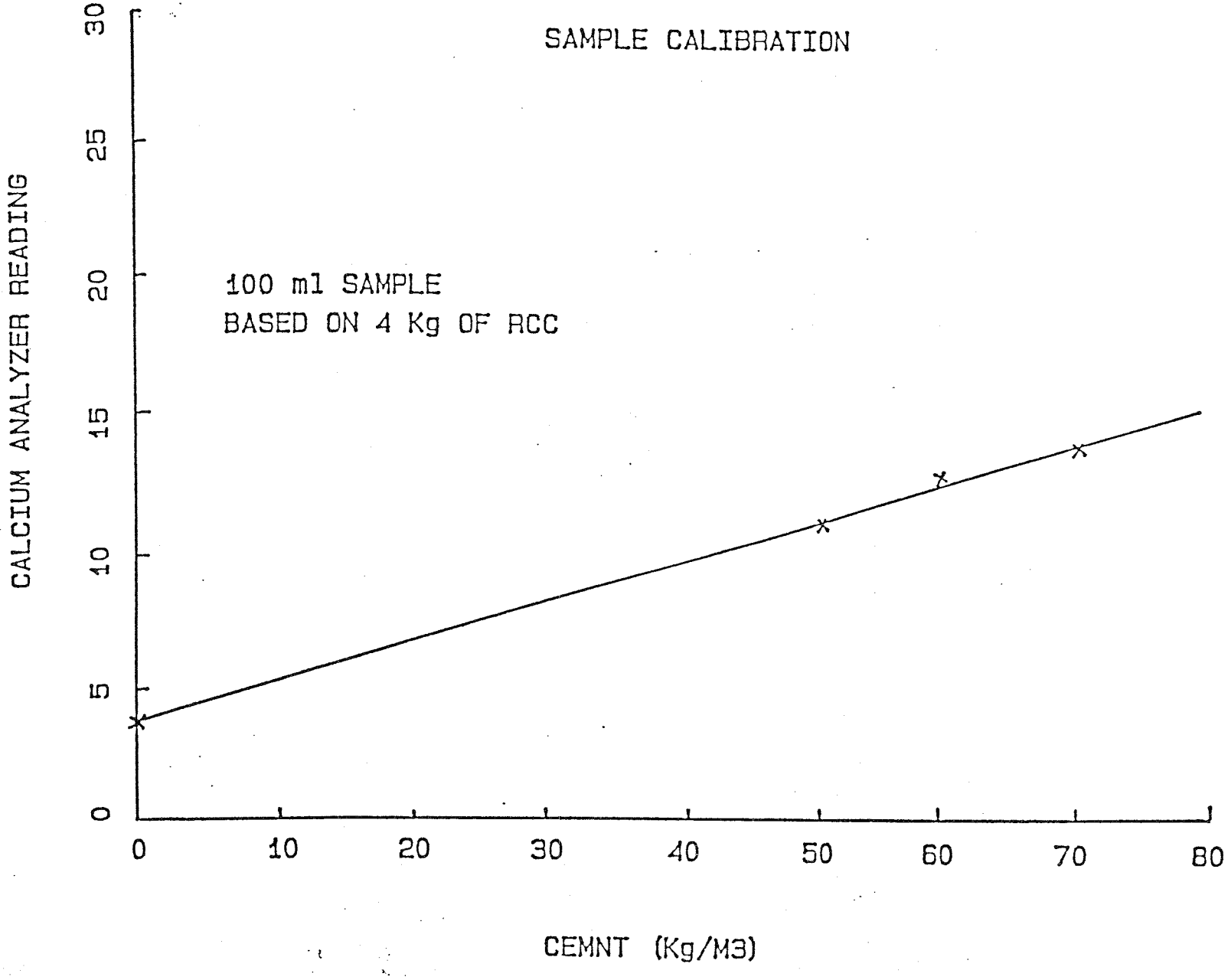
* (smaller value/largest value) x 100

ADDITIONAL INFORMATION

Compacted Unit Weight (kg/m3)	2446	2400	2373
Air Content (%)	0.9	1.0	1.4

REMARKS

Test passed. Contractor permitted to operate the next shift at 80 seconds mix time if testing is performed again during that shift.


SAMPLE CALIBRATION
100 ml SAMPLE
BASED ON 4 Kg OF RCC
CALCIUM ANALYZER READING
0
5
10
15
20
25
30
0
10
20
30
40
50
60
70
80
CEMNT (Kg/M3)

J.9 DAM ACCESS ROADS

Power Plant Access

Power plant access is provided by a 20' X 120' bridge from the plant to the west side of the river and an 18' wide gravel surfaced road from the bridge to State Highway 73. The road is 1500 feet long with a maximum grade of 12%. After leaving the bridge, the road immediately begins to ascend the nearly vertical rock walls of the west bank requiring approximately 34,000 C.Y. of rock excavation. Guard rail will be required for approximately 1000 feet.

The selected alternative will provide quick access from the power plant to the state highway which is maintained year-round by ADOT.

This alternative was selected after a review of other possible alternatives including:

1. Following the river downstream to a point where the rock walls are not so steep. This alternative was not selected because it was found that, on either side of the river, significant rock cuts and/or fills would still be required to follow the river bank. Maximum grades would be 14% and the roadway length would be increased significantly.

2. Eliminating the new bridge by bringing the access road up the east bank of the river. This alternative would require a similar amount of rock excavation and would require a total roadway length of approximately 6 miles (possibly along the existing Lower Log Road using the existing bridge) to a maintained, all-weather road (S.H. 73). Any savings gained by elimination of the new bridge would be offset by the increased roadway length.

3. Constructing a bridge over the spillway of the dam then descending the rock walls on the east side of the river. This alternative was eliminated by the high cost of constructing a bridge over the spillway.

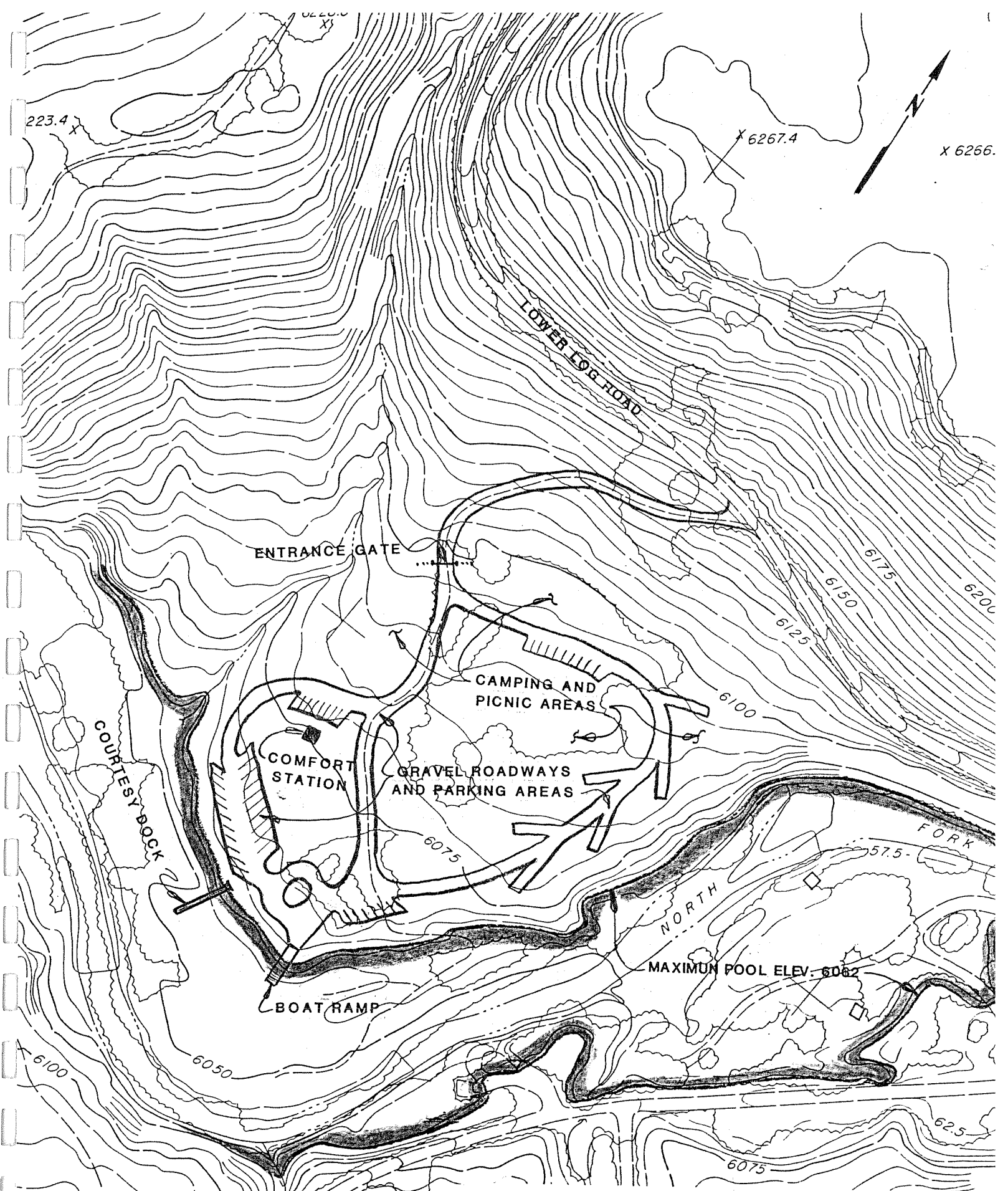
East Side Access Road to Top of Dam

Access for infrequent travel to the top of the dam on the east side will be provided by constructing some new road and by using the existing Lower Log Road. Gravel surfacing would be added. Driving distance from the top of the dam to State Highway 73 is approximately 6 miles.

J.10 RECREATION FACILITIES

Recreation facilities are keyed to probable reservoir access. Several possible points of shoreline access have been illustrated on General Map "B" (Sheet No. A2). The majority of the shoreline is virtually inaccessible due to very steep abutting terrain. However, there are several areas that do have terrain that will afford enough area for development of road ramps, courtesy docks, picnic areas and associated parking. There is some possibility that a few overnight camping spots could be constructed if they are desired. Although initial construction may call for gravel surfacing of roads and parking areas, substantial future use may necessitate payment on roads and parking.

A sketch of a "Typical Recreation Area Layout" (Figure J.10A) has been included to illustrate one possible development at an area of shoreline access.



MINER FLAT DAM

FIGURE J.10A

TYPICAL RECREATION AREA LAYOUT

SCALE: 1" 200'

J.11 PUBLIC SAFETY AND VISITOR FACILITIES

To provide for public safety at the dam, the visitor view point, dam crest access and dam access roads shall have six foot and four foot security fences along with locked control gates. Warning signs shall be installed in these areas to assist in controlling public access. The location of the fencing and gates are shown on Drawings No. J1 and J5.

The Visitor Facility shall consist of one view point located at the right abutment of the dam. This shall be a fenced area with graveled parking for approximately ten cars. Access from the highway will be along an existing road as shown on Drawings Nos. A2 and J1. The view point road will be improved with a gravel surface and adequate drainage facilities.

J.12 RESERVOIR CLEARING

The reservoir area below Elevation 6062 feet as shown on Drawing No. C-3 shall be cleared of all trees, stumps, and brush five feet or more in height, regardless of diameter, and two inches or more in diameter, regardless of height.

Trees and stumps in the reservoir area shall either be uprooted or cut off so that the maximum allowable stump height shall be six inches as measured on the uphill side of the stump. Brushy in the reservoir area shall be cut off approximately flush with the ground level.

All down timber, branches, and other floatable and combustible material five feet or more in length, regardless of diameter, and two inches or more in diameter, regardless of length, shall be cleared.

No trees shall be cut outside of areas mentioned above without specific approval, and all trees designated by the contracting authority shall be protected from damage by the contractor's construction operations.

J.13 HYDROPOWER FACILITY

Hydropower requirements for the Miner Flat Dam remain essentially the same as detailed in the report "Miner Flat Dam and Canyon Day Irrigation Project - February, 1986" and as reiterated herein with minor changes.

To provide more efficient and economical control of water flow into the powerhouse a separate six foot diameter supply pipe is to be provided in lieu of combining the ten foot diameter outlet pipe with the power supply and associated complex control valves as was suggested in the 1986 Report.

The power plant will be designed to provide an annual electrical power supply (Million Watt Hours) of approximately 5,220 MWH which is 88 MWH in excess of that required for the Canyon Day Irrigation Project.

The Miner Flat damsite is situated immediately to the east of existing 14.4 and 69 kilovolt (KV) transmission lines that parallel highway 73 between Pinetop and Whiteriver. These existing transmission lines are less than 5,000 feet to the west of the damsite. Consequently, only minor construction is necessary to interconnect the proposed hydropower facilities with the transmission system of the local electrical cooperative. Minor amounts of hydroelectric generation will be used at the damsite for operation of the outlet controls and lighting.

J.14 CONSTRUCTION COST ESTIMATE SUMMARY

The following is a Construction Cost Summary for the Miner Flat Dam Project based on the preceding preliminary design, final design criteria, construction schedules and drawings developed for the Memorandum.

These construction cost estimates do not include costs of construction inspection, testing and quality control. Those requirements will be performed by the owner or his contracted representatives.

1. Power Plant Access Road
(Drawing J5)

Unit	Description	Quantity	Unit Price	Amount	Totals
C.Y.	Unclass. Excavation (Incl. Borrow)	35,000	$10.00	$350,000	
L.F.	18" CMP Culvert	32	25.00	800	
L.F.	36" CMP Culvert	40	45.00	1,800	
Ton	Crushed Top Surfacing, Ty, B, Gr.2	1,378	8.00	11,024	
L.F.	Guard Rail	925	12.00	11,100	
Each	Guard Rail End Anchor	2	600.00	1,200	
L.F.	Metal Gate	25	16.00	400	
Each	Signing	10	300.00	3,000	
				379,324	
	Contingency of 20%			75,865	
					$455,189

2. East Side Access Road To Top Of Dam
(Length = 9,000', Width = 18)
(Drawing J6 and J7)

Unit	Description	Quantity	Unit Price	Amount	Totals
C.Y.	Unclass. Excavation	67,500	$ 3.00	$202,500	
L.F.	18" CMP Culvert	400	25.00	10,000	
Ton	Crushed Top Surfacing	4,500	8.00	36,000	
L.F.	Guard Rail	500	12.00	6,000	
Each	Signing	3	300.00	900	
				255,400	
	Contingency of 20%			51,080	
					$306,480

3. Recreation Site
(Section J.10)

Unit	Description	Quantity	Unit Price	Amount	Totals
1	24 Car Parking Lot w/Trailer Spaces	LS	$ -	$ 6,000	
1	Entrance Gate	LS	-	1,500	
1	Concrete Boat Ramp	LS	-	4,500	
1	Floating Dock and Landing	LS	-	9,000	
1	Double Latrine and Holding Tank	LS	-	5,000	
1	Well Pump Controls	LS	-	6,000	
Each	Fire Rings	10	250.00	2,500	
1	Post Barriers	LS	-	2,000	
Each	Picnic Tables	10	250.00	2,500	
					$ 39,000

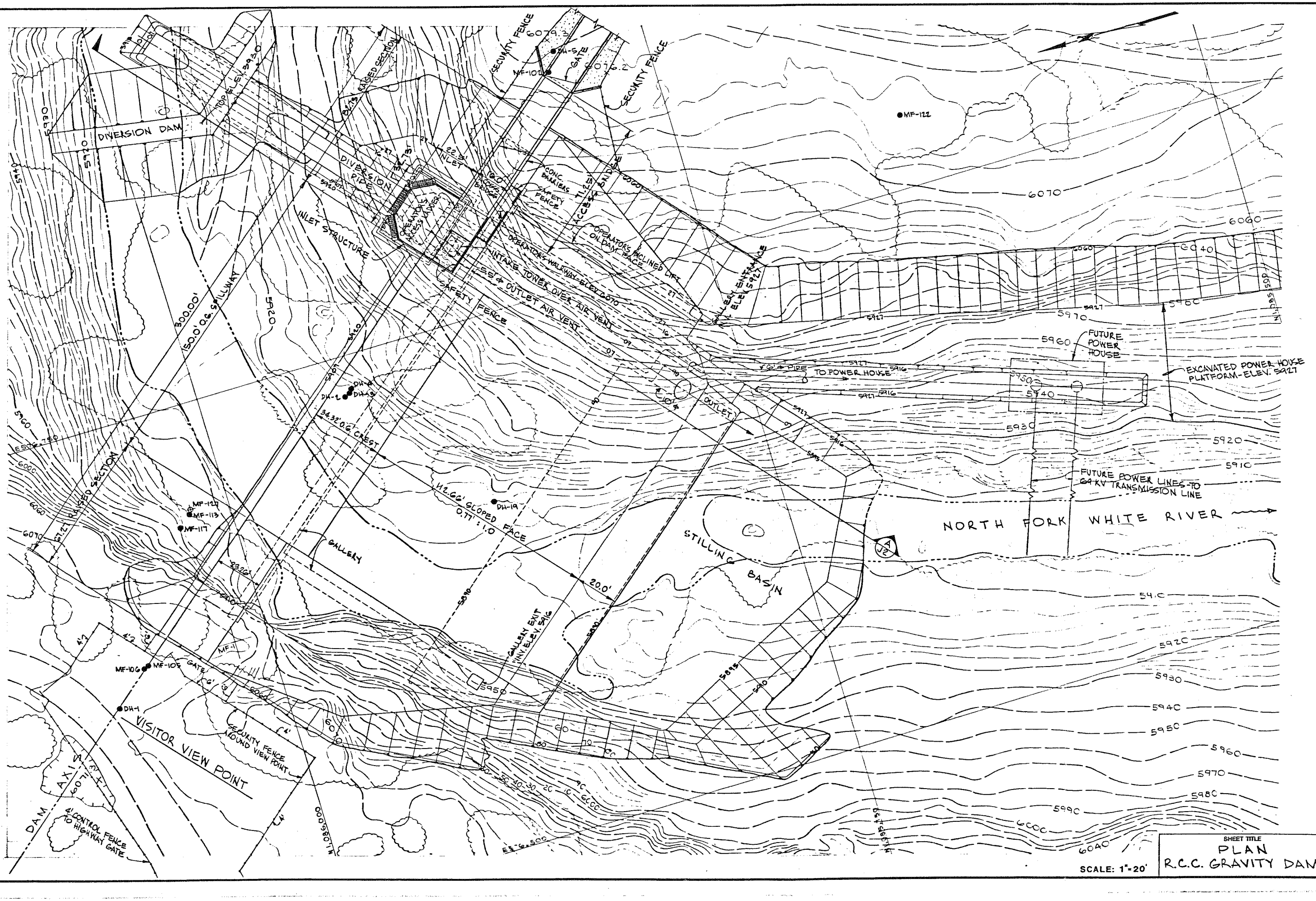
4. Visitor Viewing Area
(Section J.11)
(Drawing A2 and J1)

Unit	Description	Quantity	Unit Price	Amount	Totals
C.Y.	Excavation	160	$ 4.00	$ 640	
C.Y.	1-1/2"Aggregate Surface	160	18.00	2,880	
Each	16' Gates (4' high)	2	750.00	1,500	
Each	4' Pedestrian (6' high)	1	100.00	100	
L.F.	4' Fence	430	8.00	3,440	
L.F.	6' Fence	70	12.00	840	
L.F.	18" C.M.P. Culvert	30	20.00	600	
					$ 10,000

5. Hydropower Facility
(Section J.13)
(Drawing J1)

Unit	Description	Quantity	Unit Price	Amount	Totals
L.S.	Turbine	1	$800,000	$800,000	
L.S.	Switchyard	1	80,000	80,000	
L.S.	Station Electrical	1	235,000	235,000	
L.S.	Miscellaneous	1	57,000	57,000	
Miles	Electrical Interconnection	1	15,000	15,000	
L.S.	Valving	1	50,000	50,000	
				$1,237,000	
	Contingency at 20%			247,400	
					$1,484,400

Total Costs for Items 1, 2, 3, 4 & 5	=	$2,295,069
Total Costs for RCC Dam (Section J.7)	=	$8,950,717
Total RCC Dam Project Construction Cost	=	$11,245,786
Total Costs for Items 1, 2, 3, 4 &5	=	$ 2,295,069
Total Cost for Conventional Concrete Dam (Section J.7)	=	$15,171,225
Total Conventional Concrete Dam Project Construction Cost	=	$17,466,294



NO.	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE, P.O. BOX 6147
HELENA, MONTANA 59604
CORPORATE OFFICE
HELENA, MONTANA
BRANCH OFFICES
BILLINGS, MONTANA
KALISPELL, MONTANA
BOZEMAN, MONTANA
MISSOULA, MONTANA

DRAWN BY: CJA
CHK'D BY: BJK
APPR. BY: CNM
DATE: 12-10-86
QA
PEER REVIEW

MINER FLAT DAM

WHITE MOUNTAIN APACHE RESERVATION

NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)

SHEET NUMBER
J1

SHEET TITLE
PLAN
R.C.C. GRAVITY DAM

DIVERSION DAM
TOP EL. 5930
5910
5920
300.00' 0.G. SPILLWAY
150.0' 0.G. SPILLWAY
RAISED SECTION
6113
SECURITY FENCE
6079.3
DH-5
GATE
6071.2
MF-104
MF-122
6070
6060
6040
5960
5940
5920
5910
5930
5940
5950
5960
5970
5980
6000
6020
6040
6060
6070
INLET STRUCTURE
DIVERSION PIPE
22.5' INLET
CONC. BARRIERS
SAFETY FENCE
ACCESS BRIDGE
OPERATOR'S ACCESS LADDER
OPERATOR'S WALKWAY-ELEV. 6070
OPERATOR'S INCLINED DECK ON DAM FACE
INTAKE TOWER OVER AIR VENT
5.5'ø OUTLET AIR VENT
SAFETY FENCE
GALLERY ENTRANCE ELEV. 5926
OUTLET
16'ø
7'ø PIPE TO POWER HOUSE
FUTURE POWER HOUSE
EXCAVATED POWER HOUSE PLATFORM-ELEV. 5927
FUTURE POWER LINES TO 69 KV TRANSMISSION LINE
NORTH FORK WHITE RIVER
N 1°25' E
N 88°55' W
ESTIMATED 7.50
RAISED SECTION
MF-123
MF-119
MF-117
MF-106
MF-105
MF-1
DH-1
DH-2
DH-3
DH-19
GATE
34.33'0.G. CREST
42.22' SLOPED FACE
0.77 : 1.0
GALLERY
GALLERY EXIT WINDOW ELEV. 5916
STILLING BASIN
20.0'
VISITOR VIEW POINT
SECURITY FENCE AROUND VIEW POINT
DAM AXIS
6' CONTROL FENCE TO HIGHWAY GATE
N 10°55' W
ESTIMATED 6.50
SCALE: 1"=20'

NO.	DESCRIPTION	DATE	BY:
	REVISIONS		

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
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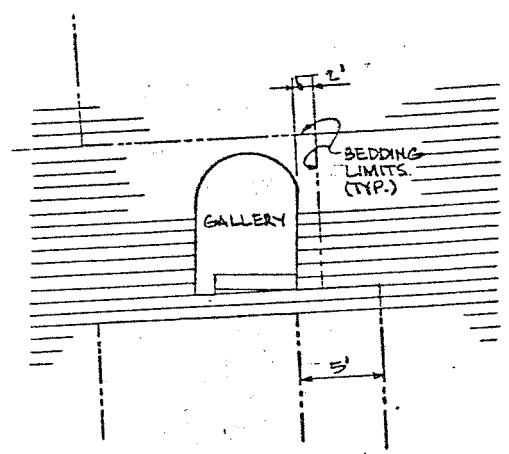
BILLINGS, MONTANA
BUTTE, MONTANA
BOZEMAN, MONTANA
KALISPELL, MONTANA

DRAWN BY: CJA
CHK'D BY: BJK
APPR. BY: CWMS
DATE: 12-10-86
QA
PEER REVIEW

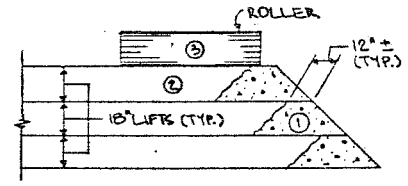
MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
J1

SHEET TITLE
PLAN
R.C.C. GRAVITY DAM

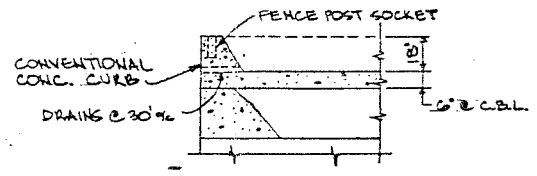


GALLERY BEDDING LIMITS

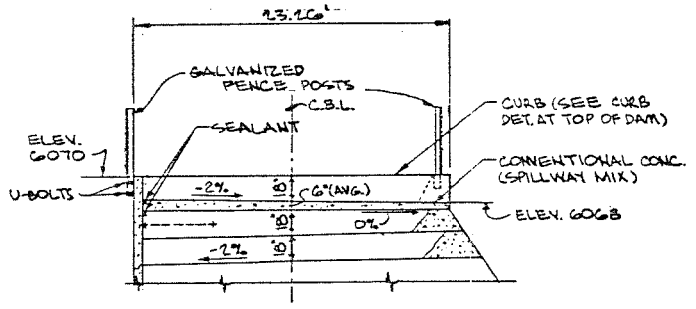


① PLACE CONVENTIONAL CONCRETE
② PLACE R.C.C.
③ COMPACT R.C.C. INTO CONVENTIONAL CONCRETE BEFORE EITHER MIX SETS. SUPPLEMENT CONSOLIDATION OF THE CONVENTIONAL CONCRETE WITH INTERNAL VIBRATION (SEE SPEC'S)

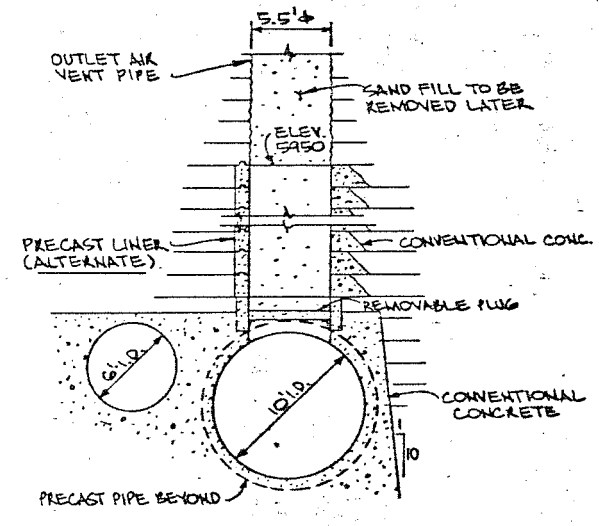
LIFT DETAIL AT FACE



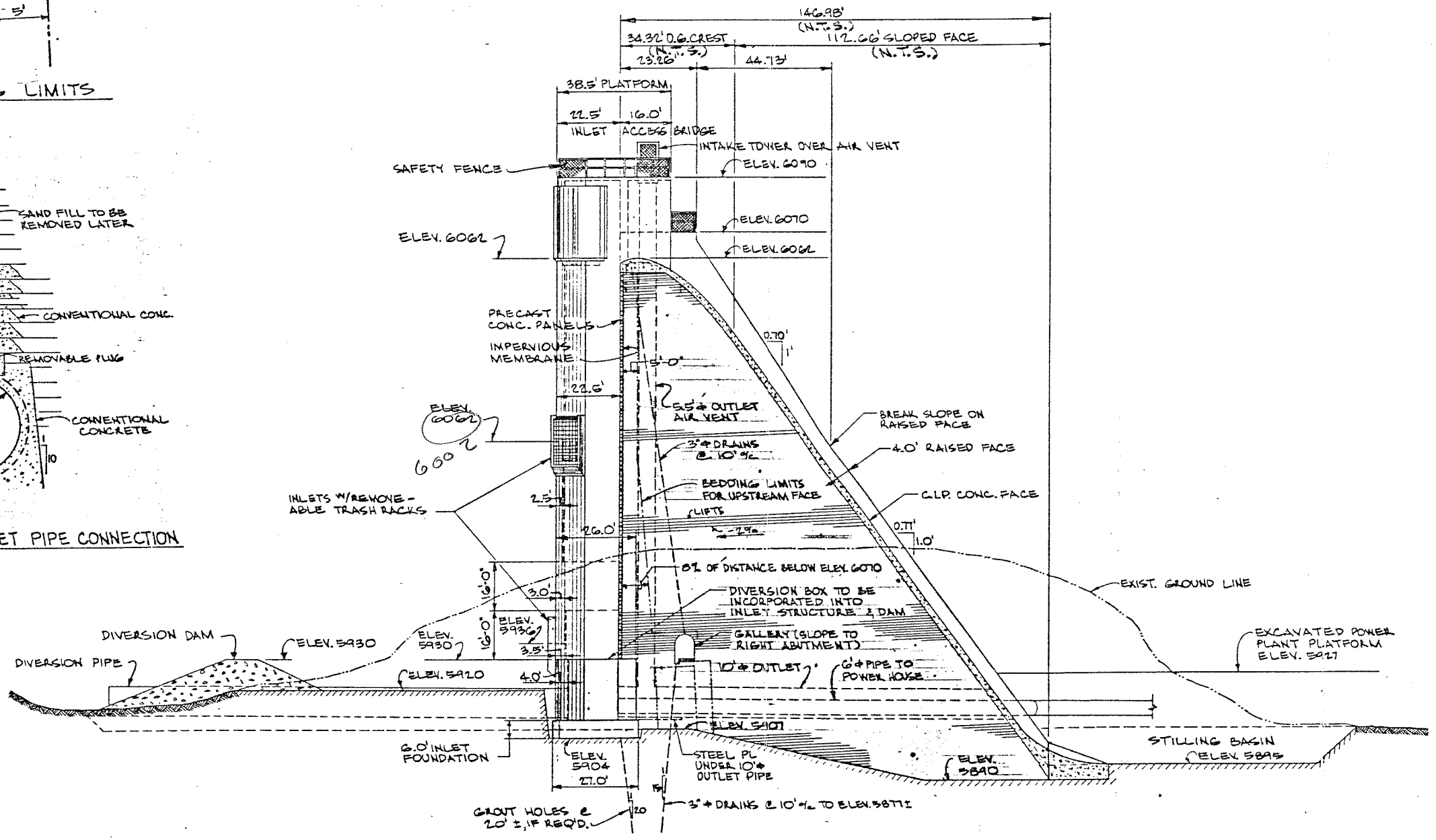
CURB DETAIL AT TOP OF DAM



TOP OF NON-OVERFLOW DETAIL



AIR VENT PIPE TO OUTLET PIPE CONNECTION



SECTION A/J2

SCALE: 1" = 20'

SHEET TITLE
SECTION
R.C.C. GRAVITY DAM

NO.	DESCRIPTION	DATE	BY
	REVISIONS		

MORRISON-MAIERLE, INC.
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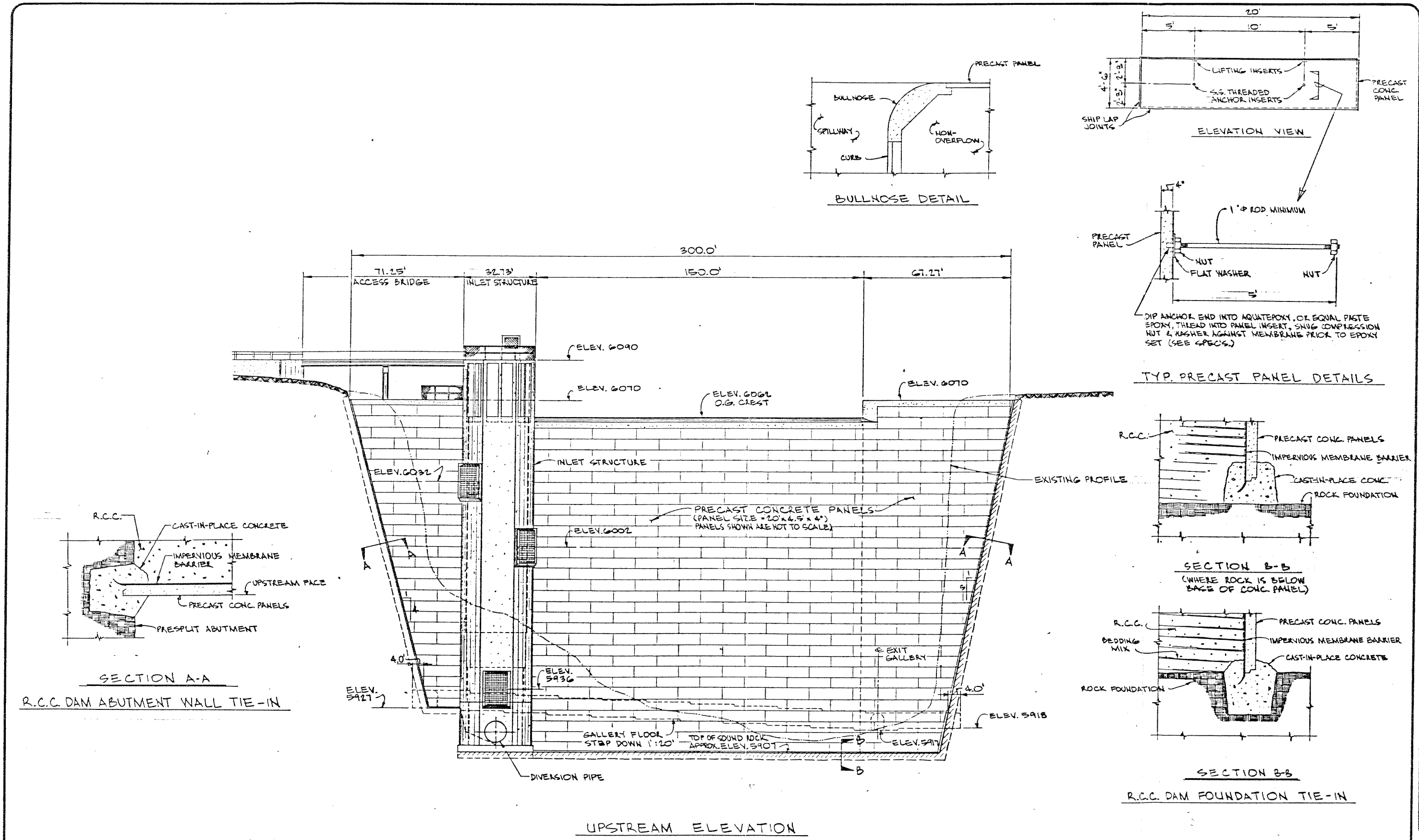
HELENA, MONTANA 59604

DRAWN BY: CJA
CHK'D. BY: BK
APPR. BY: CWK
DATE: 12-10-82
Q.A.
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
J2



SCALE: 1" = 20'

SHEET TITLE
UPSTREAM ELEVATION
R.C.C. GRAVITY DAM

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE P.O. BOX 6147
HELENA, MONTANA 59604

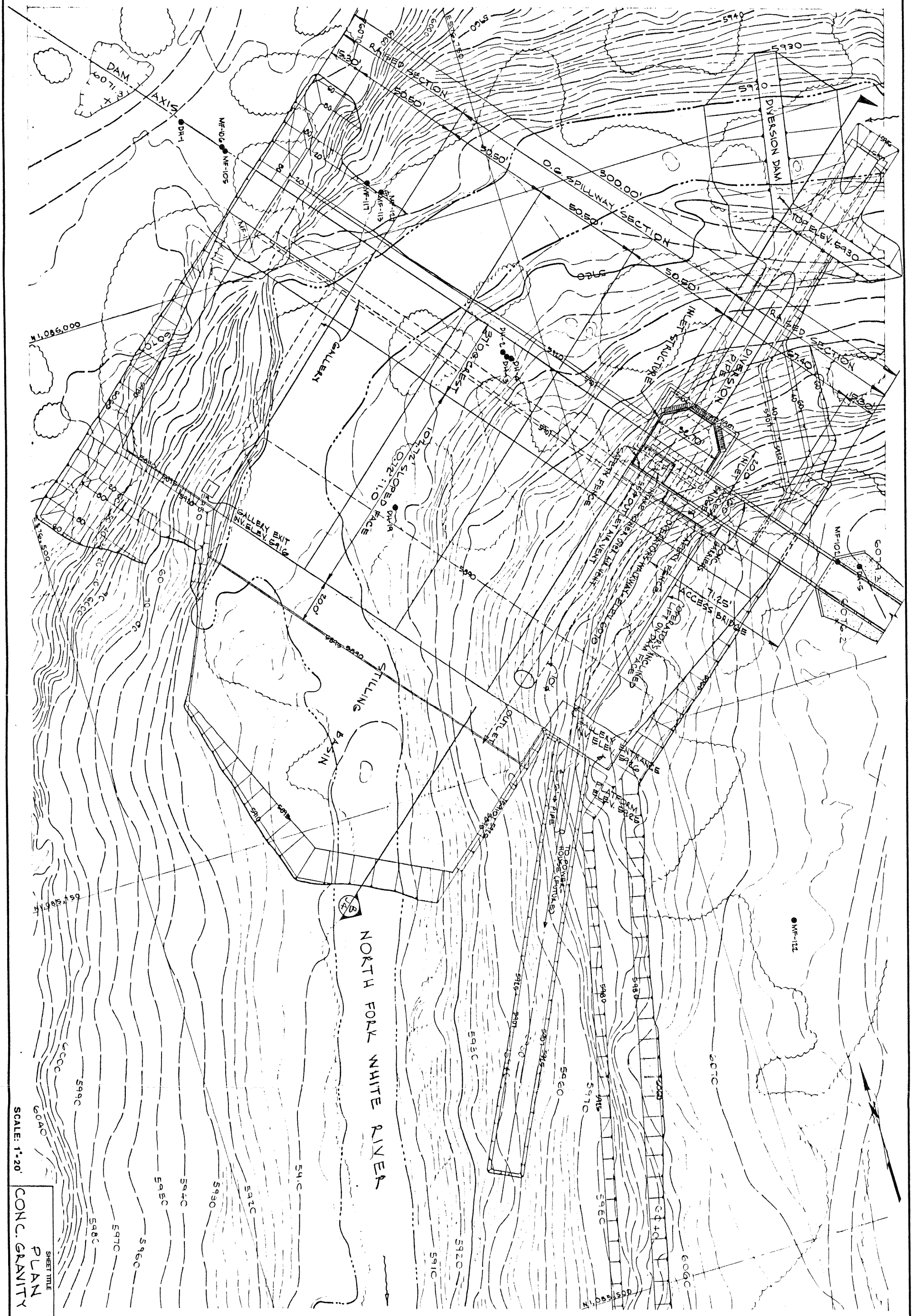
NO	DESCRIPTION	DATE	BY

REVISIONS

DRAWN BY: CJA
CHK'D BY: BJK
APPR BY: CVNR
DATE: 12-10-86
Q.A.
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
J2A



SCALE: 1"=20'

SHEET TITLE

PLAN
CONC. GRAVITY DAM

PROJECT NUMBER 1780-14-02(34)

SHEET NUMBER J3

MINER FLAT DAM

WHITE MOUNTAIN APACHE RESERVATION

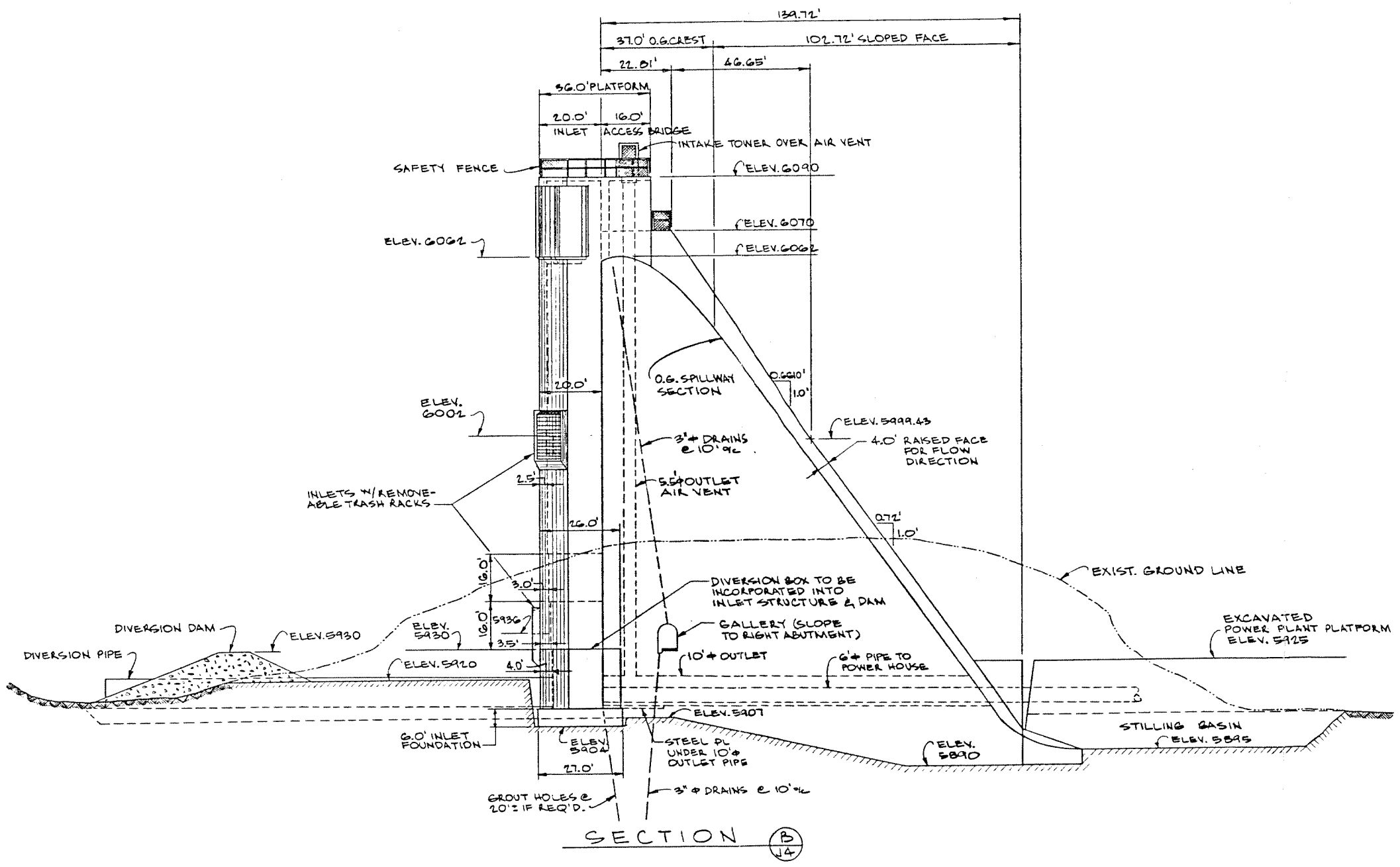
NAVAJO COUNTY, ARIZONA

DRAWN BY: CJA
CHK'D. BY: BK
APPR. BY: CWK
DATE: 12-10-86
Q.A.
PEER REVIEW

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604
CORPORATE OFFICE HELENA, MONTANA
BRANCH OFFICES BILLINGS, MONTANA KALISPELL, MONTANA BOZEMAN, MONTANA RIVERTON, WYOMING

NO	DESCRIPTION	DATE	BY

REVISIONS



SECTION B/J4

SCALE: 1"=20'

SHEET TITLE
SECTION
CONC. GRAVITY DAM

NO	DESCRIPTION	DATE	BY

REVISIONS

MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604

DRAWN BY: CJA
CHK'D BY: BK
APPR BY: CVNK
DATE: 12-10-86
Q.A.
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
J4

STA. 25 + 47
EXIST. TEL LINE
X-ING ℄

PARKING AREA
(FUTURE)
POWERHOUSE

11+00 BEGIN NEW ACCESS ROAD

NEW 20'x120' ACCESS BRIDGE

NORTH FORK WHITE RIVER

℄ BASE LINE

℄ NEW ACCESS RD.

DITCH BLOCK

P.C. 21+82.3
P.T. 22+65.0

P.C. 18+48.3
P.T. 19+34.2

P.C. 16+80.0
P.T. 17+80.0

P.C. 14+01.0
P.T. 14+99.1

P.C. 20+82.1
P.T. 21+55.2

P.C. 23+19.1
P.T. 23+65.2

P.I. 12+64.2
Δ = 80°48'
D = -
T = 31.9'
L = 52.7'
R = 37.5'

P.I. 14+99.1
Δ = 12°09'
D = 12°00'
T = 50.8'
L = 101.3'
R = 477.5'

P.I. 16+95.8
Δ = 22°26'
D = 12°
T = 94.7'
L = 186.9'
R = 477.5'

P.I. 20+15.3
Δ = 23°08'
D = 15°
T = 78.2'
L = 154.2'
R = 381.97'

P.I. 23+19.1
Δ = 113°02'
D = 57°17'45"
T = 151.2'
L = 197.3'
R = 100.0'

EX. TEL LINE TO BE REMOVED & BURIED

NEW GATE

25+82.2 END NEW ACCESS ROAD MATCH EX.

ARIZONA STATE HIGHWAY 73

NOTE: EXISTING OVERHEAD TELEPHONE LINE TO BE BURIED DURING CONSTRUCTION PHASE.

STA. 11+00 BEGIN NEW ACCESS ROAD
EL. 5925.1

STA. 16+50
NEW 36" x 36'
PIPE CR

STA. 21+50
NEW 24" x 32'
PIPE CR

℄ GRND. PROFILE

℄ RDWY. PROFILE

+3.00 %

100' V.C.

+12.00 %

300' V.C.

+2.30 %

25+82.0 END ACCESS ROAD MATCH EX. ELEV. 6045.0

TYPICAL SECTION

21.0'
18.0'
2.5' MIN.
1.5'
9.0'
9.0'
10.0' MIN
1.5'
3:1
3:1
0.5' CRUSHED GRAVEL

* TRANSITION TO 24.0' ~ 21+55 TO 22+00
24.0' = STA. 22+00 TO 25+82

PLAN & PROFILE
SCALE: 1" 50' HORIZ. 1" 20' VERT.

NO.	DESCRIPTION	DATE	BY
	REVISIONS		

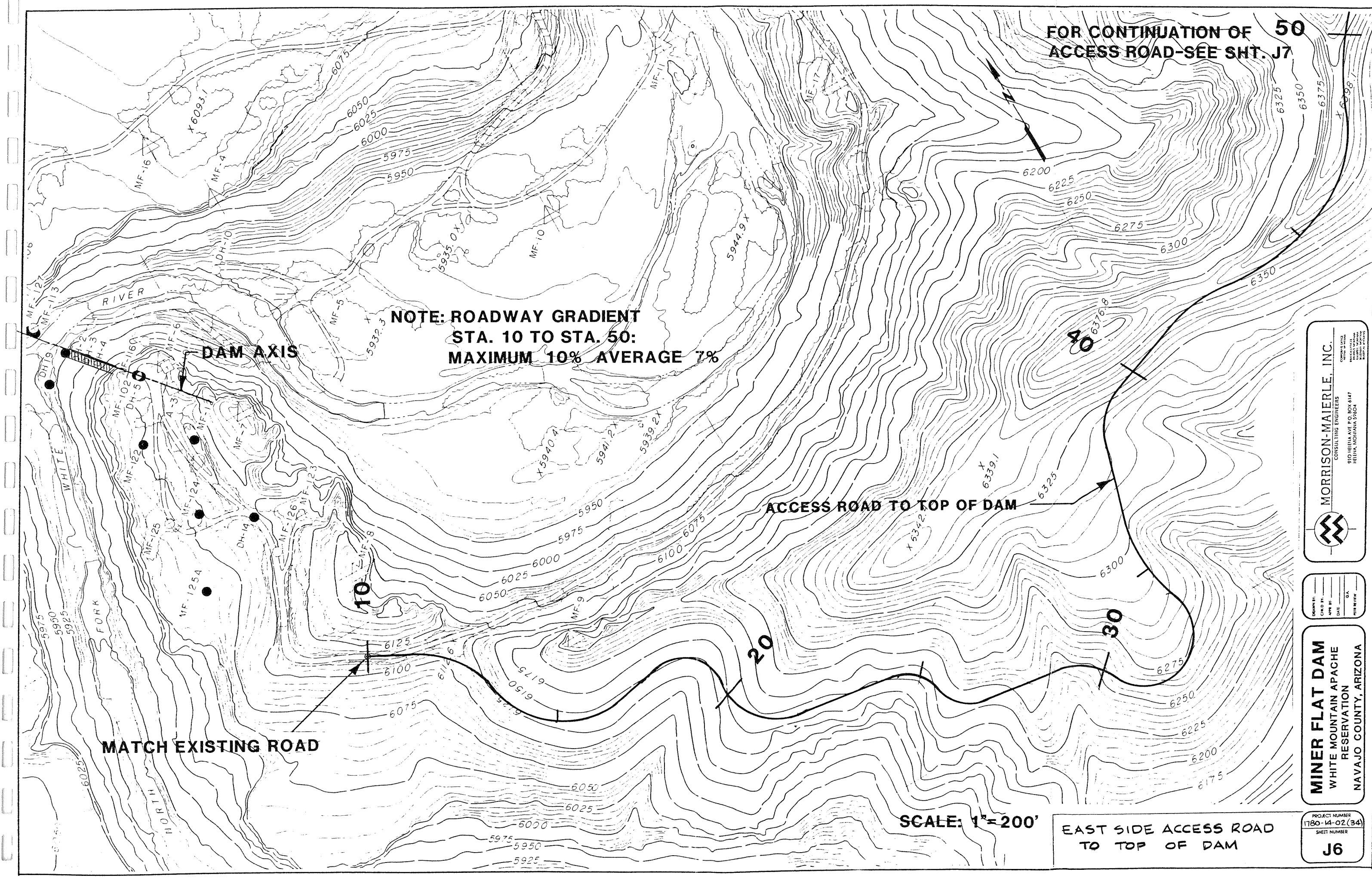
MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE. P.O. BOX 6147
HELENA, MONTANA 59604

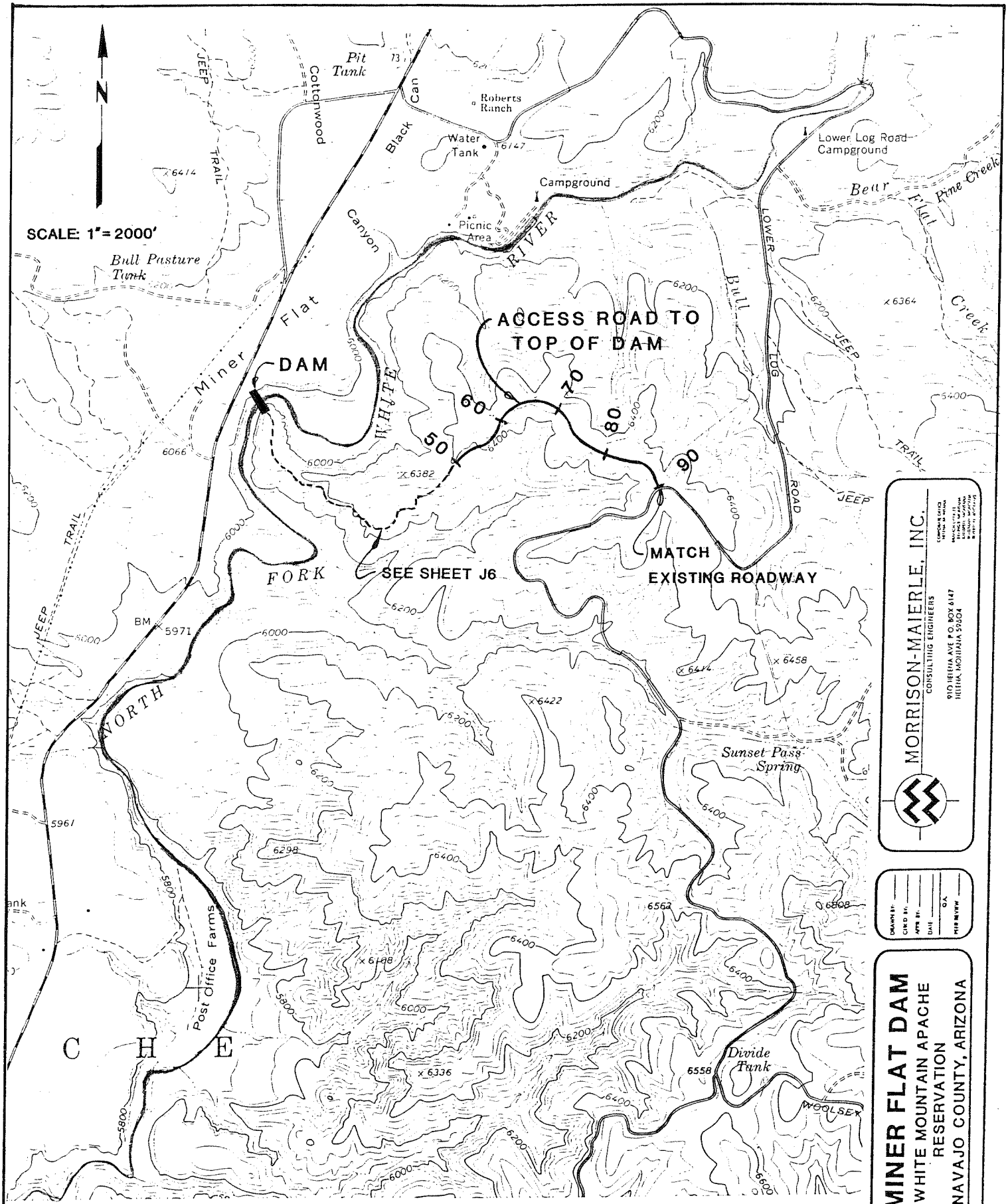
CORPORATE OFFICE
HELENA, MONTANA
BRANCH OFFICES
BILLINGS, MONTANA
KASPELL, MONTANA
KALISPELL, MONTANA
RIVERTON, WYOMING

DRAWN BY: KHL
CHK'D BY: RJP
APP'D BY: CWK
DATE: 12-10-80
Q.A.
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
J5


FOR CONTINUATION OF ACCESS ROAD-SEE SHT. J7
50
NOTE: ROADWAY GRADIENT
STA. 10 TO STA. 50:
MAXIMUM 10% AVERAGE 7%
DAM AXIS
ACCESS ROAD TO TOP OF DAM
MATCH EXISTING ROAD
10
20
30
40
RIVER
WHITE
FORK
NORTH
SCALE: 1"=200'
EAST SIDE ACCESS ROAD
TO TOP OF DAM
MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
910 HELENA AVE P.O. BOX 6147
HELENA, MONTANA 59604
MINER FLAT DAM
WHITE MOUNTAIN APACHE
RESERVATION
NAVAJO COUNTY, ARIZONA
PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
J6



MORRISON-MAIERLE, INC.
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910 HELENA AVE P.O. BOX 6147
HELENA, MONTANA 59604

DRAWN BY:
CHKD BY:
APP'D BY:
DATE:
QA
PEER REVIEW

MINER FLAT DAM
WHITE MOUNTAIN APACHE RESERVATION
NAVAJO COUNTY, ARIZONA

NOTE:
ROADWAY GRADIENT STA. 50 TO STA. 90:
MAXIMUM=5% AVERAGE=2%

EAST SIDE ACCESS ROAD TO TOP OF DAM

PROJECT NUMBER
1780-14-02(34)
SHEET NUMBER
J7