16646

Umetco Minerals Corporation

CELL 4 DESIGN

WHITE MESA PROJECT
BLANDING, UTAH

UMETCO MINERALS CORPORATION

Umetco Minerals Corporation

P.O. BOX 1029 • GRAND JUNCTION, COLORADO 81502
☎ (303) 245-3700

April 10, 1989

Mr. Edward F. Hawkins
Branch Chief - Uranium
Recovery Field Office, Region IV
Nuclear Regulatory Commission
Box 26325
Denver, Colorado 80226

Dear Mr. Hawkins:

Re: Cell 4 Design
White Mesa Project
Docket No. 40-8681 04008681210R

Please find enclosed three (3) copies of the response to the questions and concerns that you addressed in your March 15, 1989, letter to Mr. D. K. Sparling concerning the Cell 4 design for the White Mesa Project.

We would like to start construction on this project during the first week of May, 1989.

If you or your staff have any questions or need further information, please contact Curtis O. Sealy (303-245-3700).

Thank you for your timely consideration in this matter.

Very truly yours,

Curtis O. Sealy, P.E.

COS/sw

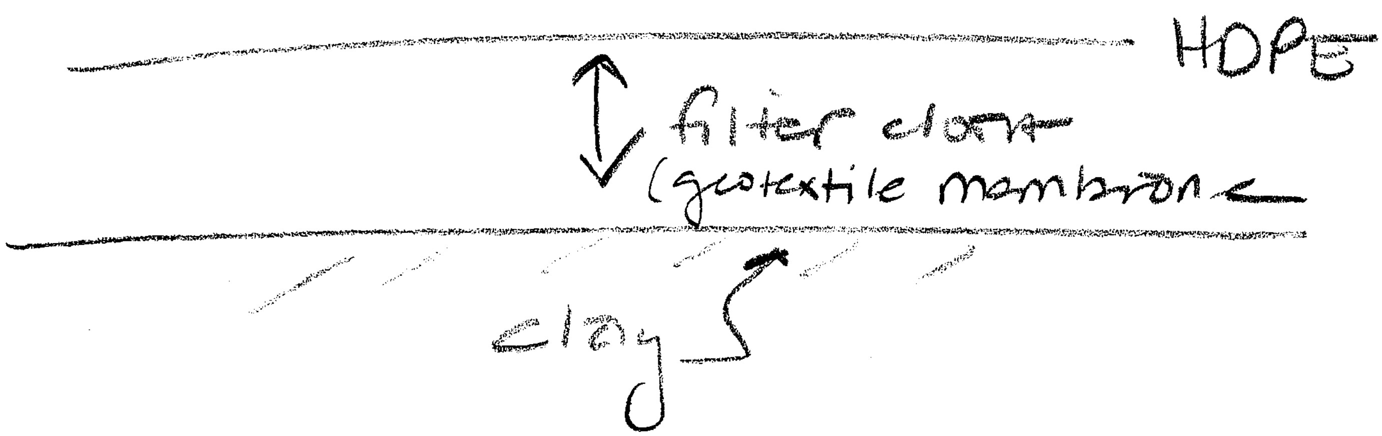
Enclosure

cc: Messrs. J. S. Hamrick
D. K. Sparling
Central Files

072400

Per RFH and HRR

3/16 - 1/4" Thick filter cloth



Cell 4
White Mesa Project

Docket No. 40-8681 04008681210R

NRC Statement:

1. The surface water design must be included in the design package. The design must establish sufficient freeboard in the cells to allow for settlement and wind wave runup, along with providing available storage capacity to store the entire inflow from a 6-hour Probable Maximum Precipitation (PMP) event. For design purposes, you should assume that the reservoir level at the beginning of the PMP event is at the maximum level expected during the design life of the cells. The diversion ditch design must also be included in the design package.

Response:

We have enclosed the general drainage map for the project with appropriate backup information from our consultant. (Letter from Western Engineers to Umetco dated April 5, 1989.)

Umetco will operate the cell to contain the PMP event of 10 inches with an additional 1.5 feet of freeboard to provide for wave runup due to wind. (See attached Drawing C4-2.)

WESTERN ENGINEERS INC. 1953
CONSULTING ENGINEERS / LAND SURVEYORS
2150 Hwy 6 & 50 Grand Junction CO 81505 · 303 242 5202

April 5, 1989

Mr. Curt Sealy
UMETCO Minerals Corporation
P.O. Box 1029
Grand Junction, CO 81502

RE: Cell 4 Site Drainage

Dear Curt:

Following is a summary of the White Mesa Uranium Mill Site drainage and its impact on the proposed Cell 4 - Phase A. Please refer to to sheet C4-6 - Cell 4 General Drainage Map.

The PMF event is based on the 6-hour PMP thunderstorm. This PMP is 10.0 inches.

Basin "A" and Basin "B" PMF runoff will be retained in Cell 1. Normal storm runoff from Basin "B" is retained in a 14 acre-foot retention pond. During the PMF the pond will overflow and drain into Cell 1. The Cell 1 flood pool volume is 180 acre-feet. This is between elevation 5615 (maximum operating pool level) and 5618.5 (the emergency spillway crest). Between the 180 acre-feet in Cell 1 and the 14 acre-foot pond on the mill site, there is adequate retention volume to retain the PMF from Basin "A" and Basin "B". The Cell 1 embankment crest is at 5620. This gives slightly over 1.5 feet of freeboard in Cell 1. Cell 1 elevations from D'Appalonia.

A drainage channel to intercept runoff above Cell 1 is not needed. Cell 1 has adequate PMF retention capacity without the diversion channel.

Basin "C" and Basin "D" PMF runoff will be retained in Cell 3. Normal storm runoff from Basin "C" will be retained in Cell 2. During the PMF, Cell 2 will overflow through the emergency spillway into Cell 3. The PMF retention volume required in Cell 3 is 141.5 acre-feet. This capacity is met with a 1.25 foot freeboard allowance if the normal operating level in Cell 3 remains at or below elevation 5603.75. This assumes the east end of the cell filled with tailings.

Cell 4 - Phase A is Drainage Basin "E". The tributary drainage area is 43.25 acres resulting in a PMF volume of 36.0 acre-feet. The normal operating level in the Cell will be set at elev. 5595.75. This allows for the required 1 foot flood pool while maintaining 1.25 feet of freeboard.

The freeboard requirements are based on a design procedure developed by the U.S.A.C.E. The procedure is summarized in the Handbook of Dam Engineering. Wind records for 1986, 1987, and 1988 taken at the Blanding White Mesa Mill Site were used to choose a design wind speed of 20 mph. While much stronger gusts probably occur at the site, they are generally of short duration and will not cause high waves. For example; an 80 mph wind over an effective fetch of 0.3 miles must be maintained for 6 minutes to result in maximum wave action (Figure 2.24 Handbook of Dam Engineering). The wave runup calculations are summarized below:

Wind Tide (Setup)	=	0.039 ft.
Wave Height	=	0.733 ft.
Wave Period	=	1.54 sec.
Wave Runup	=	1.17 ft (based on smooth surface)
Runup + Setup	=	1.21 ft.
Design Freeboard	=	1.25 ft.

The freeboard design and the PMP calculations are attached. I have also attached the mill site wind speed data. If you have any questions, please call.

Very truly yours,

WESTERN ENGINEERS, INC.

John M. Currier, P.E.
Staff Engineer

Table 6.3A.—Local-storm PMP computation, Colorado River, Great Basin and California drainages. For drainage average depth PMP. Go to table 6.3B if areal variation is required.

Drainage White Mesa Cell 4 Area < 1.0 (mi^2) (km^2)
Latitude 37° 35' Longitude 109° 35' Minimum Elevation 5600 (ft) (m)

Steps correspond to those in sec. 6.3A.

1. Average 1-hr 1-mi^2 (2.6-km^2) PMP for drainage [fig. 4.5]. 8.5 in. (mm)

2. a. Reduction for elevation. [No adjustment for elevations up to 5,000 feet (1,524 m): 5% decrease per 1,000 feet (305 m) above 5,000 feet (1,524 m)]. 97 %

   b. Multiply step 1 by step 2a. 8.25 in. (mm)

3. Average 6/1-hr ratio for drainage [fig. 4.7]. 1.20

	Duration (hr)									
	1/4	1/2	3/4	1	2	3	4	5	6	
4. Durational variation for 6/1-hr ratio of step 3 [table 4.4].	74	89	95	100	110	115	118	119	120	%
5. 1-mi^2 (2.6-km^2) PMP for indicated durations [step 2b X step 4].				8.25					9.9	in. (mm)
6. Areal reduction [fig. 4.9].										%
7. Areal reduced PMP [steps 5 X 6].				8.25					9.9	in. (mm)
8. Incremental PMP [successive subtraction in step 7].										in. (mm)
} 15-min. increments										

9. Time sequence of incremental PMP according to:

   Hourly increments [table 4.7]. ______ in. (mm)

   Four largest 15-min. increments [table 4.8]. ______ in. (mm)

UTETA CELL 4 DESIGN

WAVE RUNUP DESIGN.

REFERENCE: "HANDBOOK OF DAM ENGINEERING"

EDITED BY ALFRED R. GOLZE

SITE WIND SPEED: DESIGN 30 mph

SEE ATTACHED SITE WIND INFORMATION THAT shows Avg. hourly wind speed ≤ 11 mph for past three years

1). EFFECTIVE FETCH DISTANCE

α	Cos α	X_i	X_i cos α
0	1.0	2200	2200
6	.995	1900	1890
6	.995	1900	1890
12	.973	1700	1678
12	.973	1700	1678
18	.951	1500	1427
18	.951	1550	1474
24	.914	1400	1280
24	.914	1450	1325
30	.866	1300	1125
36	.809	1250	1011
42	.743	1200	892
	Σ 11.094		Σ 17970

$$F_{eff} = \frac{\Sigma X_i \cos \alpha}{\Sigma \cos \alpha} = \frac{17970}{11.094} = 1610 \text{ FT.} \quad (0.305)$$

[illegible]

2) WIND TIDE SETUP

$$S = \frac{V^2 F}{1400 D}$$

D = MEAN RESERVOIR DEPTH ALONG THE FETCH

D = 5.0' ASSUMED

$$S = \frac{(30)^2(.305 \text{ mile})}{(1400)(5)} = 0.039 \text{ FT}$$

3) WAVE CHARACTERISTICS

A) HEIGHT

$$\frac{g H_s}{V^2} = 0.0026 \left(\frac{gF}{V^2}\right)^{.47}$$

H_s = Avg height of 1/3 highest waves

$g = 32.2$

$V = 30$ mph = 44 fps

$F = 1610$ FT

$H_s = .733$ FT

B) PERIOD

$$\frac{g T}{V} = 0.45 \left(\frac{gF}{V^2}\right)^{.29}$$

$T = 1.54$ SEC

4) WAVE RUNUP

USE FIGURE 2.27 pg 142 HANDBOOK OF DAM ENGIN

R = Runup Measured vertically (FT)

H_o = height of specific wave

T = WAVE PERIOD

L_o = Length of wave [ft] = $5.12\ T^2$

Assume $H_o = H_s$

$L_o = 5.12 (1.54)^2 = 12.2$

$H_o/L_o = .233/12.2 = 0.060$

$R/H_o = 1.6$ (Fig 2.27 for 1:3 smooth slopes)

R = 1.173 FT

5) PMF FREEBOARD REQUIREMENT

FB = SETUP + WAVE RUNUP

= 0.039 FT + 1.173 FT

= 1.212 FT

FB = SAY 1.25 FT

6) NORMAL OPERATING SURFACE ELEVATION

MAX FLOOD POOL EL = 5698 - 1.25 = 5696.75

POOL SURFACE AREA @ 5696.75 = 39 ACRES

Pool SURFACE AREA @ 5695.75 = 38.6 ACRES

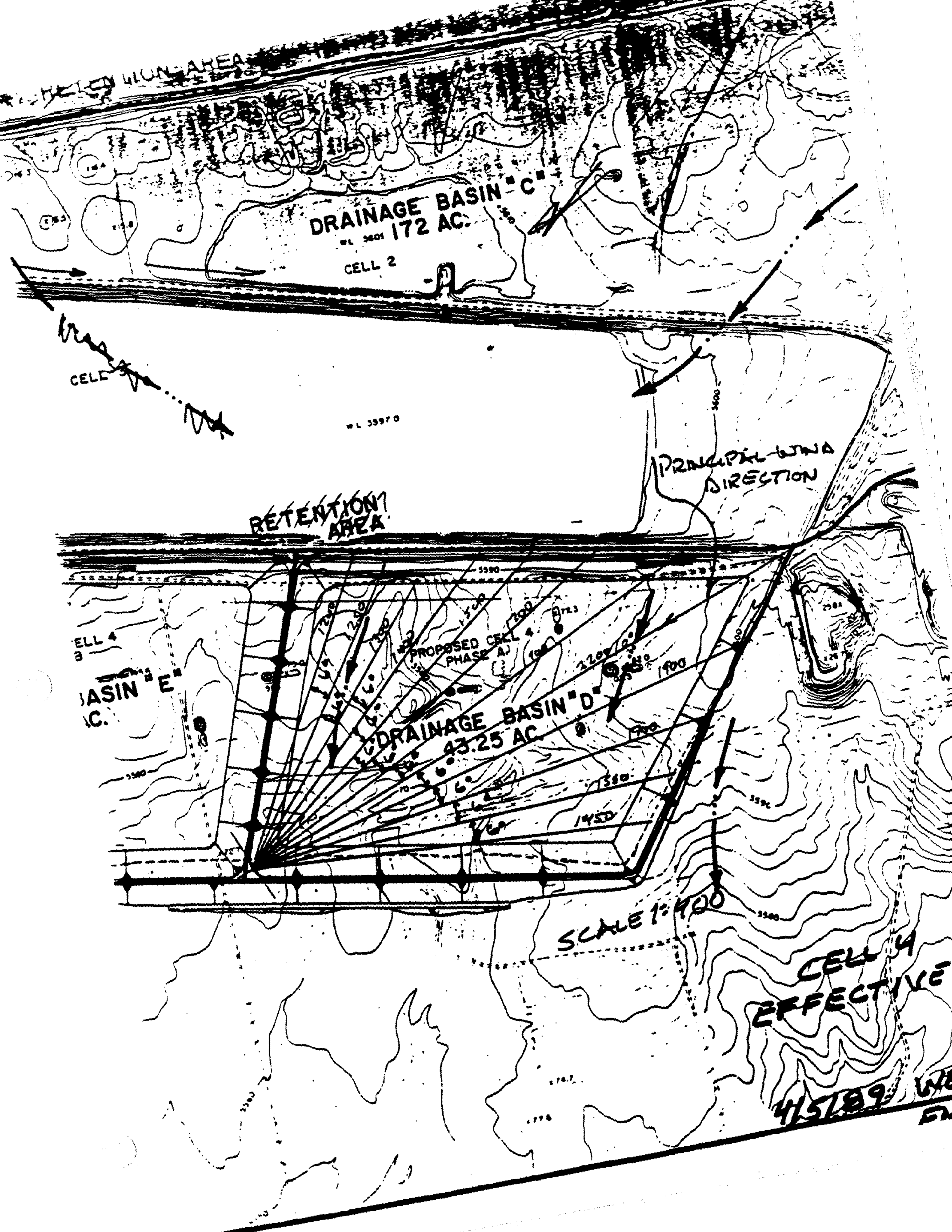
CAPACITY FROM 95.75 TO 96.75 = 38.8 AF

PMF Volume 36.0 AF

NORM. OPERATING LEVEL = CREST - FB - FLOOD POOL

= 5698 - 1.25' - 1.0

= 5695.75



DRAINAGE BASIN "C"
172 AC.
CELL 2
WL 3597.0
PRINCIPAL WIND DIRECTION
RETENTION AREA
PROPOSED CELL 4 PHASE A
DRAINAGE BASIN "D"
43.25 AC.
BASIN "E"
CELL 4
SCALE 1:400
CELL 4 EFFECTIVE
4/5/89

TABLE 3.2

FREQUENCY OF WINDS BY DIRECTION AND SPEED
FOR
JANUARY THROUGH JUNE 1988
UMETCO - BLANDING
TIME (MST): 0000-2400

DIRECTION	SPEED CLASS INTERVALS(M/S) 1<1.5	1.5< 3	3< 5	5< 8	8<11	>11	ALL	MEAN SPEED
N	1.07	6.85	4.52	.30	.03	.00	12.76	2.7
NNE	1.26	5.53	2.03	.30	.08	.00	9.20	2.5
NE	.99	3.09	.60	.22	.00	.00	4.90	2.3
ENE	.44	.63	.14	.16	.00	.00	1.37	2.3
E	.30	.77	.33	.08	.00	.00	1.48	2.4
ESE	.30	1.42	.79	.19	.00	.00	2.71	2.8
SE	.58	2.03	1.07	.74	.03	.00	4.44	3.2
SSE	.66	2.66	1.64	.63	.03	.00	5.61	3.0
S	.88	3.34	3.04	1.92	.68	.00	9.86	3.9
SSW	1.07	4.16	2.63	1.51	.58	.00	9.94	3.5
SW	.99	2.05	1.29	1.10	.16	.00	5.59	3.3
WSW	.25	1.04	.74	.66	.16	.00	2.85	3.8
W	.44	1.53	1.37	.85	.16	.00	4.35	3.6
WNW	.52	1.42	1.20	.82	.19	.00	4.16	3.7
NW	.49	1.64	2.35	2.33	.49	.00	7.31	4.4
NNW	.52	2.98	2.44	.74	.33	.00	7.01	3.4
ALL	10.73	41.16	26.18	12.54	2.93	.00	93.54	3.2

CALM (less than meter per second) = 6.5
PERIOD MEAN WIND SPEED = 3.1 M/S

ENECOTECH INC.
WIND 8/11/88

ENECOTECH

TABLE 3.1

FREQUENCY OF WINDS BY DIRECTION AND SPEED
FOR
JULY THROUGH DECEMBER 1988
UMETCO - BLANDING
TIME (MST): 0000-2400

DIRECTION	SPEED CLASS INTERVALS (M/S) 1<1.5	1.5< 3	3< 5	5< 8	8<11	>11	ALL	MEAN SPEED
N	1.01	7.42	4.03	.37	.00	.00	12.82	2.7
NNE	1.47	12.91	3.75	.64	.09	.00	18.86	2.5
NE	1.19	2.29	1.28	.92	.27	.00	5.95	3.2
ENE	.27	.92	.27	.27	.00	.00	1.74	2.9
E	.92	1.19	.27	.18	.00	.00	2.56	2.1
ESE	.46	1.47	.09	.09	.00	.00	2.11	2.1
SE	.82	2.47	.92	.64	.09	.00	4.95	3.0
SSE	.55	3.66	1.19	.00	.00	.00	5.40	2.4
S	.82	4.30	.92	.18	.00	.00	6.23	2.3
SSW	.73	4.21	1.28	.27	.00	.00	6.50	2.5
SW	.55	3.39	2.11	.37	.00	.00	6.41	2.9
WSW	.82	1.01	1.19	.73	.00	.00	3.75	3.2
W	.27	1.47	.64	.27	.00	.00	2.66	2.8
WNW	.18	1.56	.64	.18	.00	.00	2.56	2.7
NW	.73	2.29	2.01	1.01	.09	.00	6.14	3.4
NNW	1.10	2.93	1.92	.37	.00	.00	6.32	2.7
ALL	11.90	53.48	22.53	6.50	.55	.00	94.96	2.7

CALM (less than meter per second) = 5.0
PERIOD MEAN WIND SPEED = 2.6 M/S

ENECOTECH INC.
WIND 2/03/89

D\UMETCO\WDROSE.TBL

TABLE 3.2

FREQUENCY OF WINDS BY DIRECTION AND SPEED
FOR
JANUARY THROUGH JUNE 1987
UMETCO - BLANDING
TIME (MST): 0000-2400

SPEED CLASS INTERVALS(M/S)

DIRECTION	1<1.5	1.5< 3	3< 5	5< 8	8<11	>11	ALL	MEAN SPEED
N	0.64	3.29	1.25	0.16	0.00	0.00	5.34	2.5
NNE	0.86	7.48	3.13	0.54	0.10	0.00	12.11	2.8
NE	1.21	5.27	1.69	0.70	0.06	0.00	8.95	2.7
ENE	0.61	2.01	0.35	0.48	0.00	0.00	3.45	2.7
E	0.70	1.18	0.22	0.26	0.00	0.00	2.36	2.3
ESE	0.51	0.99	0.13	0.13	0.06	0.00	1.82	2.4
SE	0.86	2.52	0.96	0.42	0.13	0.00	4.89	2.8
SSE	0.73	3.07	2.01	0.89	0.22	0.00	6.93	3.2
S	1.05	4.15	1.79	0.80	0.26	0.00	8.05	3.0
SSW	0.61	2.94	2.20	1.50	0.48	0.00	7.73	3.8
SW	0.67	2.24	1.98	0.86	0.16	0.00	5.91	3.3
WSW	0.48	1.60	1.02	0.73	0.13	0.00	3.96	3.5
W	0.48	1.63	1.41	0.77	0.16	0.00	4.44	3.4
WNW	0.45	2.49	3.48	1.63	1.05	0.00	9.11	4.3
NW	0.58	1.98	1.57	0.35	0.16	0.00	4.60	3.1
NNW	0.45	1.53	0.70	0.16	0.03	0.00	2.88	2.6
ALL	10.89	44.35	23.90	10.38	3.00	0.00	92.52	3.1

CALM (less than meter per second) = 7.5
PERIOD MEAN WIND SPEED = 2.9 M/S

ENECOTECH INC.
WIND 7/23/87

ENECOTECH

TABLE 3.2

FREQUENCY OF WINDS BY DIRECTION AND SPEED
FOR
JULY THROUGH DECEMBER 1987
UMETCO - BLANDING
TIME (MST): 0000-2400

	SPEED CLASS INTERVALS(M/S)							
DIRECTION	1<1.5	1.5< 3	3< 5	5< 8	8<11	>11	ALL	MEAN SPEED
N	.66	2.64	1.17	.08	.00	.00	4.54	2.5
NNE	1.45	8.12	2.64	.08	.00	.00	12.28	2.4
NE	1.80	6.34	1.40	.66	.00	.00	10.20	2.4
ENE	.91	1.57	.58	.15	.00	.00	3.22	2.3
E	.51	.89	.20	.08	.00	.00	1.67	2.0
ESE	.48	.71	.23	.05	.00	.00	1.47	2.2
SE	.66	1.93	.74	.18	.00	.00	3.50	2.4
SSE	.74	3.58	1.37	.53	.00	.00	6.22	2.7
S	1.14	5.18	1.45	.43	.03	.00	8.22	2.5
SSW	1.09	5.10	2.77	1.62	.05	.00	10.63	3.1
SW	.63	3.43	1.93	1.17	.00	.00	7.16	3.2
WSW	.63	1.42	.56	.41	.00	.00	3.02	2.7
W	.61	2.46	1.83	.51	.05	.00	5.46	3.0
WNW	.41	2.13	2.08	.71	.18	.00	5.51	3.5
NW	.43	1.04	.89	.20	.00	.00	2.56	2.9
NNW	.46	1.27	.76	.13	.00	.00	2.61	2.6
ALL	12.61	47.81	20.58	6.98	.30	.00	88.28	2.7

CALM (less than meter per second) = 11.7
PERIOD MEAN WIND SPEED = 2.4 M/S

ENECOTECH INC.
WIND 2/08/88

TABLE 3.2

FREQUENCY OF WINDS BY DIRECTION AND SPEED
FOR
JANUARY THROUGH JUNE 1986
UMETCO - BLANDING
TIME (MST): 0000-2400

	SPEED CLASS INTERVALS(M/S)							MEAN
DIRECTION	1<1.5	1.5< 3	3< 5	5< 8	8<11	>11	ALL	SPEED
N	0.65	4.12	3.42	0.38	0.00	0.00	8.56	2.9
NNE	1.37	7.19	2.96	0.32	0.00	0.00	11.85	2.5
NE	1.35	5.25	0.97	0.62	0.03	0.00	8.21	2.4
ENE	1.08	1.45	0.30	0.00	0.00	0.00	2.83	1.8
E	0.75	1.02	0.27	0.03	0.00	0.00	2.07	2.0
ESE	0.83	0.94	0.30	0.05	0.00	0.00	2.13	2.0
SE	0.67	2.23	0.86	0.30	0.00	0.00	4.07	2.7
SSE	0.75	2.42	1.72	0.57	0.05	0.00	5.52	3.0
S	0.73	3.55	2.85	1.75	0.11	0.00	8.99	3.5
SSW	0.51	3.28	3.04	1.72	0.46	0.00	9.02	3.8
SW	0.27	2.42	2.34	1.51	0.48	0.00	7.03	4.0
WSW	0.05	1.05	1.13	1.40	0.19	0.00	3.82	4.6
W	0.27	1.05	0.94	1.08	0.22	0.00	3.55	4.2
WNW	0.22	0.94	0.83	0.35	0.11	0.00	2.45	3.6
NW	0.38	1.40	2.05	1.40	0.27	0.00	5.49	4.1
NNW	0.46	2.58	3.42	1.94	0.32	0.00	8.72	3.9
ALL	10.34	40.93	27.41	13.41	2.23	0.00	94.32	3.3

CALM (less than meter per second) = 5.7
PERIOD MEAN WIND SPEED = 3.1 M/S

ENECOTECH INC.
WIND 9/23/86

TABLE 3.2

FREQUENCY OF WINDS BY DIRECTION AND SPEED
FOR
JULY THROUGH DECEMBER 1986
UMETCO - BLANDING
TIME (MST): 0000-2400

SPEED CLASS INTERVALS(M/S)

DIRECTION	1<1.5	1.5< 3	3< 5	5< 8	8<11	>11	ALL	MEAN SPEED
N	1.13	5.98	2.32	.22	.00	.00	9.65	2.5
NNE	1.13	8.19	1.94	.43	.00	.00	11.70	2.6
NE	1.19	3.61	.92	.59	.00	.00	6.31	2.5
ENE	.59	1.08	.22	.05	.00	.00	1.94	2.1
E	.49	.86	.11	.00	.00	.00	1.46	1.9
ESE	.59	1.13	.54	.16	.00	.00	2.43	2.4
SE	.81	3.13	.75	.05	.00	.00	4.74	2.1
SSE	1.13	5.28	1.19	.05	.00	.00	7.65	2.2
S	1.40	3.50	2.43	.54	.00	.00	7.87	2.7
SSW	.75	3.83	1.56	.43	.00	.00	6.58	2.7
SW	.65	1.56	.70	.16	.00	.00	3.07	2.5
WSW	.49	1.08	.92	.16	.00	.00	2.64	2.8
W	1.08	2.32	1.67	.32	.05	.00	5.44	2.8
WNW	.49	3.23	2.43	1.13	.27	.00	7.56	3.5
NW	.65	3.83	1.19	.05	.00	.00	5.71	2.4
NNW	.59	3.88	1.08	.05	.00	.00	5.61	2.3
ALL	13.15	52.51	19.95	4.42	.32	.00	90.35	2.6

CALM (less than meter per second) = 9.6
PERIOD MEAN WIND SPEED = 2.4 M/S

ENECOTECH INC.
WIND 1/29/87

EnecoTech

Cell 4
White Mesa Project

Docket No. 40-8681 04008681210R

NRC Statement:

2. The settlement analysis referred to in the plan must be submitted. The results should substantiate that the magnitude of the total and differential settlement will not result in harmful cracking or dam instability.

Response:

The calculations for foundation settlement are attached.

Because of the partially saturated nature of the foundation soils, settlements should largely occur during construction.

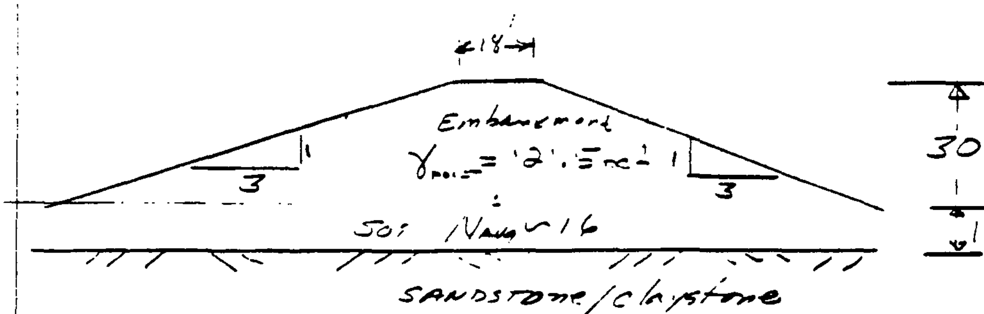
The type of construction proposed is similar to the other cells constructed for the project.

Surveys and observations (see attached letter and data) show that settlement of Cell 3 dike is negligible.

Monitoring of the new dikes for Cell 4 will be performed and evaluated regulatory as is presently being done of the existing cell dikes.

Problem statement: Compute settlement of Foundation Soils [illegible]

CELL 4 Dike at maximum section



Assume uniformly loaded area.

$P = \Delta\sigma_{on\ station} = 121.5 \times 30 = 3{,}645\ psf$

Soil [illegible] saturated ∴ use elastic methods to compute settlement of FDN Soils

$S_d = I_d\, P\, B\left(\frac{1-\mu^2}{E}\right)$

where S_d = settlement beneath ℄

I_d = Shape factor [illegible] L/B

B = width of loaded area ~ 198'

μ = Poissons Ratio ~ 0.5

E = young's modulus of soil

P = magnitude of Loading

Eq. (1)

$$S_d = C_d\, q\, B \left(\frac{1-\mu^2}{E}\right)$$

Determine C_d

For $H/B = 0/198 = 0.05$ $L/B \sim \frac{500}{10} \sim 50$

From Table 4.2 (Attached)

$C_d = 0.045$

Determine E

From Eq 4.5 $q_c/N \sim 2.75$

For N ... $q_c = 44$ tsf

where $E = 2 q_c$ Eq. 4.11 attached

$\therefore E = 2(44) = 88$ tsf or 176,000 psf

Substituting values into Eq. 1

$$S_d = C_d\, q\, B \left(\frac{1-\mu^2}{E}\right)$$

$$S_d = (.045)(3,645)(198)\left[\frac{1-0.3^2}{176,000}\right]$$

$\therefore S_{d_{MAX}} = 0.1384$ feet $\sim$ **1.7 inches** ←

Determination of Equivalent Young's Modulus In order to use the method it is necessary to estimate the stiffness in terms of an equivalent Young's modulus at various depths. It is suggested that this be done using the Dutch cone bearing capacity. The Dutch cone is a static-type penetrometer which has been used in Europe for more than 30 years. When provided with a sleeve to eliminate side friction it provides a convenient and rapid way to measure bearing capacity, and therefore strength, at various depths. In the case of cohesionless materials which have not been prestressed significantly to pressures above the in situ overburden pressure, the Dutch cone bearing capacity can be correlated with the Young's modulus (De Beer 1965; Webb, 1970). The relationship suggested by Schmertmann (1970), consistent with that of other investigators, is

$$E = 2q_c \tag{4.11}$$

in which q_c is the Dutch cone bearing capacity (usually expressed in kg/cm^2 or ton/ft^2).

Because the stiffness of cohesionless materials is more strongly influenced than strength by prestress effects, the correlation given in Eq. 4.11 is likely to underestimate the equivalent Young's modulus for overconsolidated cohesionless soils. Prestress is also likely to influence the strain distribution (Webb, 1971). However, the extent to which this is true is not known at this time, and no means is presently available for accounting on a routine basis for the influence of prestress.

The static Dutch cone bearing capacity is much more reliable than the standard penetration test, which is a measure of the dynamic bearing capacity, and whose results are fraught with experimental difficulties and uncertainty. Nonetheless, it is recognized that standard penetration resistance data may be available for sites when Dutch cone bearing capacity data are not. To permit the use of standard penetration data as a temporary expedient, Schmertmann (1970) recommends the conservative correlation between Dutch cone bearing capacity and standard penetration resistance given in Table 4.5. It is of course much more preferable to obtain directly the Dutch cone bearing capacity. However, when it is necessary to use the standard penetration resistance Schmertmann (1970) suggests that as many N values as possible be obtained, in order to minimize by averaging correlation errors associated with having only a few data.

The use of the method described is illustrated in the fol-

TABLE 4.5. CORRELATION BETWEEN DUTCH CONE BEARING CAPACITY q_c AND STANDARD PENETRATION RESISTANCE. (From Schmertmann, 1970.)

Soil Type	q_c/N*
Silts, sandy silts, slightly cohesive silt-sand mixtures	2.0
Clean, fine to med. sands and slightly silty sands	3.5
Coarse sands and sands with little gravel	5
Sandy gravel and gravel	6

*Units of q_c are kg/cm^2 or $tons/ft^2$; units of N are blows/ft.

Distributed Load at or Near the Surface of a Deep Layer When the foundation problem can be approximated as one or more uniformly distributed loads acting on circular or rectangular areas near the surface of a relatively deep stratum, the vertical settlement can be estimated by

$$S_d = C_d p B \left(\frac{1 - \mu^2}{E}\right) \qquad (4.2)$$

This expression, first given by Schleicher (1926), determines the vertical settlement of any point on the surface of an elastic half-space subjected to a uniformly distributed load over a circular or rectangular area. In this expression, S_d is the settlement of a point at the surface, p is the magnitude of the uniformly distributed load, B is a characteristic dimension of the loaded area as shown in Fig. 4.2, E is Young's modulus, and μ is Poisson's ratio for the elastic medium. The factor C_d is a parameter which accounts for the shape of the loaded area and the position of the point for which the settlement is being calculated. Values of C_d for several points on uniformly loaded rectangular areas of various shapes are given in Table 4.1.

Equation 4.2 can also be used to describe the vertical settlement of a concentrically loaded *rigid* circular or square area, such as a concrete footing, described by the boundary condition of uniform displacement rather than uniform loading. Only the form of C_d is different for the

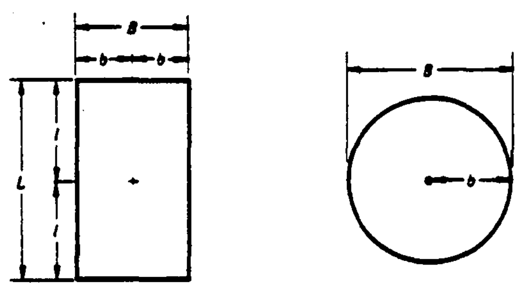


Fig. 4.2 Notation for loaded areas, shown in plan view.

From: Foundation Engineering Handbook
Winterkorn & Fang
Van Nostrand Reinhold Company 197

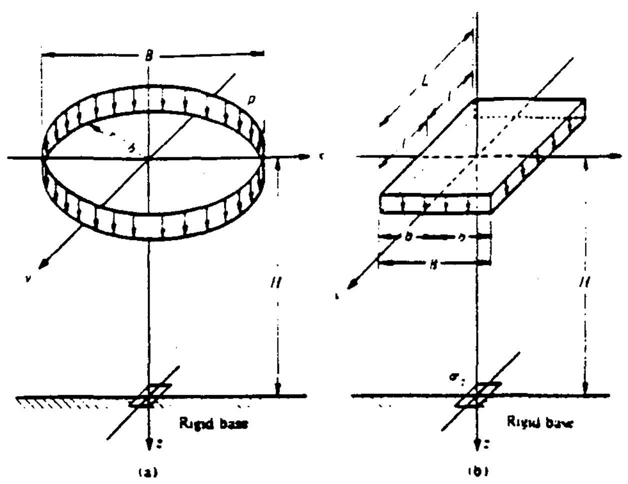


Fig. 4.3 Finite compressible material underlain by a rigid base. (a) Circular loaded area; (b) rectangular loaded area.

TABLE 4.2. VALUES OF THE SHAPE FACTOR C_d' FOR SETTLEMENT OF THE CENTER OF A UNIFORMLY LOADED AREA ON AN ELASTIC LAYER UNDERLAIN BY A RIGID BASE. (Modified from Egorov, 1958, as cited by Harr, 1966.)

	Circle	Rectangle						Infinite Strip
H/B	*Diameter = B*	*L/B* = 1	*L/B* = 1.5	*L/B* = 2	*L/B* = 3	*L/B* = 5	*L/B* = 10	*L/B* = ∞
0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.1	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
0.25	0.24	0.24	0.23	0.23	0.23	0.23	0.23	0.23
0.5	0.48	0.48	0.47	0.47	0.47	0.47	0.47	0.47
1.0	0.70	0.75	0.81	0.83	0.83	0.83	0.83	0.83
1.5	0.80	0.86	0.97	1.03	1.07	1.08	1.08	1.08
2.5	0.88	0.97	1.12	1.22	1.33	1.39	1.40	1.40
3.5	0.91	1.01	1.19	1.31	1.46	1.56	1.59	1.60
5.0	0.94	1.06	1.24	1.38	1.55	1.72	1.82	1.83
∞	1.00	1.12	1.36	1.52	1.78	2.10	2.53	∞

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March 4, 1982

Mr. Don Sparling, Mill Manager
Energy Fuels Nuclear, Inc.
White Mesa Uranium Millsite
Post Office Box 787
Blanding, Utah 84511

Surveying tailings management area, White Mesa Uranium Millsite, San Juan County, Utah.

Dear Mr. Sparling:

As a result of recent discussions concerning survey work conducted in the tailings management area of the White Mesa Uranium Millsite, I am writing in an attempt to summarize our involvement in the past, and state the condition of the various dikes, slopes and embankments as we have found them, during our field work.

It should be noted from the outset that survey work, for the construction of the tailings management system, has not been conducted on a periodic basis with a fixed time interval for the purpose of updating and maintaining control points; but rather it has been conducted only when requested. Additionally, survey points were never set at any location for the sole purpose of, or with the intent of, monitoring cell dike movement.

The initial survey work, to establish control points for the purpose of constructing the tailings management system, was commenced during July, 1979, at which time both horizontal and vertical control nets were established. The work on each cell or cell dike, progressed generally as follows:

CELL #1

July, 1979 - Horizontal control was set to establish centerline alignment and stationing on cell dike #1. Slope stakes were set to mark bottoms of slopes on the upstream and downstream sides.

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CELL #1 CONTINUED

October, 1979 - Changes in the engineering design of the dikes dictated that the soils be removed to a solid rock layer and that the dikes be built up from those layers to design elevations.

For this reason, after soils were removed, the centerline alignment and stationing of cell dike #1 was re-established and the slope stakes were reset to mark the bottoms of the slopes on the upstream and downstream sides.

November, 1979 - Cell dike #1 had been completed to near design elevation by construction crews. Cut and fill stakes were set at the crest of the dike to aid in the final grading process.

March, 1982 - Cell dike #1 was checked for alignment and grade and it was determined that, based on original control no movement had occurred. Periodic grading of the crest of the dike had taken place in order to maintain drainage and this was taken into consideration when checking crest elevations.

During surveying for the purpose of checking for movement, no cracks or distortions were sighted along the dike or along the slopes of the dike.

We were not involved in the construction of the inner portion of cell #1. This work was apparently completed by other survey crews.

CELL #2

July, 1979 - Horizontal control was set to establish centerline alignment and stationing on cell dike #2. Slope stakes were set to mark bottoms of slopes on the upstream and downstream sides.

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CELL #2 CONTINUED

October, 1979 - Changes in the engineering design of the dikes dictated that the soils be removed to a solid rock layer and that the dikes be built up from those layers to design elevations.

For this reason, after soils were removed, the centerline alignment and stationing of cell dike #2 was re-established and the slope stakes were reset to mark the bottoms of the slopes on the upstream and downstream sides.

November, 1979 - Cell dike #2 had been completed to near design elevation by construction crews. Cut and fill stakes were set at the crest of the dike to aid in the final grading process.

March, 1980 - Horizontal and vertical control was re-established at the crest of cell dike #2 for the purpose of establishing control at cell dike #3. It was determined that, based on original control, no movement had occurred. Periodic grading of the crest of the dike had taken place in order to maintain drainage and this was considered when checking crest elevations.

August, 1981 - Horizontal and vertical control was re-established at the crest of cell dike #2 for the purpose of determining earth quantities in cell #3. It was determined at that time, based on original control, that no movement had occurred. Again, periodic grading on the crest of the dike had been conducted to maintain drainage and this was taken into consideration when crest elevations were checked.

March, 1982 - Cell dike #2 was checked for alignment and elevation and it was determined at that time that no movement had occurred. Periodic grading along the crest of the dike had been conducted and this was a consideration when checking crest elevations.

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CELL #2 CONTINUED

During survey work conducted for the purpose of checking for movement, no cracks or distortions were sighted along the dike or along the slopes of the dike.

We were not involved in the construction of the inner portion of cell #2. This work was apparently completed by other survey crews.

CELL #3

July, 1979 - Horizontal control was set to establish centerline alignment and stationing on cell dike #3. Slope stakes were set to mark bottoms of slopes on the upstream and downstream sides.

November, 1979 - Centerline stakes were reset at cell dike #3.

April, 1980 - Removal of top soil for the construction of cell dike #3 had resulted in the destruction of most of the slope stakes originally set. Slope stakes were reset to mark the bottoms of the slopes on the upstream and the downstream sides.

August, 1981 - Cell dike #3 had been completed to near design elevation by construction crews. Horizontal and vertical control was established along the crest of the dike to aid in the final grading and to be used for control to determine earth quantities in cell #3.

September, 1981 - Survey work was conducted to determine earth quantities in cell #3. Outside perimeters of cell #3 were marked.

March, 1982 - Cell dike #3 was checked for alignment and elevation and it was determined that no movement had occurred since the original control had been set in August, 1981. Periodic grading of the crest of the dike had been conducted to maintain drainage and this was taken into consideration when checking crest elevations.

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In conclusion, during the time spent on the project by our field crews, no cracks, distortions, seepage, sinking or any other unusual conditions were sighted.

As you know, we are currently establishing permanent points along the crests of the existing cell dikes for the purpose of monitoring for movement. These points will offer a much more precise means of checking each dike, as well as aid in future survey work.

Should you require additional information concerning survey work conducted by our firm, please feel free to contact us at your convenience.

Sincerely yours,

Richard L. Johnston,
Land Surveyor

RLJ/ms

Richard L. Johnston 9-1-06

Richard L. Johnston
Land Surveyor
14201 County Road 20
Cortez, Colorado 81321

8/88

CELL DIKE MONITORING SYSTEM — CELL #3 DIKE

APPROXIMATE CENTERLINE STATION	10.00' LEFT OF CENTERLINE-POINT NUMBER	DISTANCE	ELEVATION	MOVEMENT LEFT OR RIGHT
0+00 (23)	65 *		5607.42	
		49.96		
0+50	66*		5607.69	
		49.89		
1+00	67*		5608.25	
		50.20		
1+50	68*		5608.50	
		50.09		
2+00	69*		5608.76	
		49.95		
2+50	70		5608.38	
		50.01		
3+00	71		5608.03	
		49.84		
3+50	72		5608.18	
		49.87		
4+00	73		5608.29	
		50.03		
4+50	74*		5608.11	
		50.16		
5+00	75		5607.92	
		49.78		
5+50	76		5607.96	
		50.06		
6+00	77*		5607.55	
		50.00		
6+50	78*		5607.54	
		49.92		
7+00	79		5607.67	
		50.07		
7+50	80		5607.56	
		50.04		
8+00	81		5607.80	
		50.01		
8+50	82		5607.78	
		49.92		
9+00	83		5607.95	
		49.98		
9+50	84		5607.98	
		50.22		
10+00	85		5607.92	

** Original point obliterated - replaced with new monitoring point

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8/88

CELL DIKE MONITORING SYSTEM — CELL #3 DIKE — PAGE 2

APPROXIMATE CENTERLINE STATION	10.00' LEFT OF CENTERLINE-POINT NUMBER	DISTANCE	ELEVATION	MOVEMENT LEFT OR RIGHT
10+00	85		5607.92	
		49.62		
10+50	86		5607.89	
		49.95		
11+00	87		5607.74	
		50.16		
11+50	88		5607.57	
		50.00		
12+00	89*		5608.09	
		50.00		
12+50	90		5608.24	
		49.96		
13+00	91*		5608.41	
		49.63		
13+50	92		5608.33	
		50.46		
14+00	93*		5608.13	
		49.76		
14+50	94*		5607.94	
		50.58		
15+00	95*		5607.81	
		49.35		
15+50	96*		5607.76	
		49.96		
16+00	97*		5607.83	
		50.07		
16+50	98		5608.31	
		49.85		
17+00	99		5608.72	
		50.04		
17+50	100*		5608.53	
		49.99		
18+00	101		5608.40	
		49.98		
18+50	102*		5607.89	
		50.17		
19+00	103*		5607.98	
		49.74		
19+50	104*		5607.99	
		50.73		
20+00	105		5608.13	

** Original point obliterated - replaced with new monitoring point

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CELL DIKE MONITORING SYSTEM CELL #3 DIKE PAGE 3

APPROXIMATE CENTERLINE STATION	10.00' LEFT OF CENTERLINE- POINT NUMBER	DISTANCE	ELEVATION	MOVEMENT LEFT OR RIGHT
20+00	105		5608.13	
		49.48		
20+50	106		5608.25	
		50.07		
21+00	107**		5608.02	
		50.21		
21+50	108**		5607.74	
		49.52		
22+00	109		5607.74	
		49.82		
22+50	110		5607.77	
		50.20		
23+00	111		5607.98	
		50.12		
23+50	112		5607.98	
		49.84		
24+00	113		5608.06	
		49.98		
24+50	114		5608.21	
		50.77		
25+00	115		5608.23	
		49.35		
25+50	116**		5608.09	
		49.87		
26+00	117		5607.79	
		50.06		
26+50	118		5607.78	
		50.09		
27+00	119		5607.77	
		50.16		
27+50	120		5608.17	
		49.82		
28+00	121		5608.15	
		50.17		
28+50	122		5608.23	
		50.08		
29+00	123		5608.11	
		49.63		
29+50	124		5608.10	
		50.78		
30+00	125		5607.64	

** Original point obliterated - replaced with new monitoring point

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CELL DIKE MONITORING SYSTEM CELL #3 DIKE PAGE 4

APPROXIMATE CENTERLINE STATION	10.00' LEFT OF CENTERLINE- POINT NUMBER	DISTANCE	ELEVATION	MOVEMENT LEFT OR RIGHT
30+00	125		5607.64	
		49.04		
30+50	126*		5607.84	
		50.13		
31+00	127*		5608.06	
		50.10		
31+50	128*		5608.18	
		49.92		
32+00	129		5608.12	
		50.00		
32+50	130*		5607.82	
		49.99		
33+00	131*		5607.63	
		49.97		
33+50	132*		5608.16	
		42.68		
33+91.8 (24)	133*		5608.11	

** Original point obliterated - replaced with new monitoring point

Avg 5608.00 ± .28

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CELL DIKE MONITORING SYSTEM   CELL #3 DIKE   PAGE 1   3/82

APPROXIMATE CENTERLINE STATION	10.00' LT OF CENTERLINE- POINT NO.	DISTANCE	ELEVATION
0+00 (23)	65		5607.60
		50.02	
0+50	66		5607.79
		49.94	
1+00	67		5608.39
		50.13	
1+50	68		5608.71
		50.04	
2+00	69		5608.80
		49.96	
2+50	70		5608.39
		49.98	
3+00	71		5608.04
		49.86	
3+50	72		5608.19
		49.89	
4+00	73		5608.29
		50.01	
4+50	74		5608.25
		50.17	
5+00	75		5607.93
		49.77	
5+50	76		5607.97
		50.06	
6+00	77		5607.58
		49.97	
6+50	78		5607.85
		49.94	
7+00	79		5607.69
		50.06	
7+50	80		5607.59
		50.03	
8+00	81		5607.81
		50.02	
8+50	82		5607.79
		49.92	
9+00	83		5607.96
		50.00	
9+50	84		5607.99
		50.23	
10+00	85		5607.92

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CELL DIKE MONITORING SYSTEM   CELL #3 DIKE   3/82

APPROXIMATE CENTERLINE STATION	10.00' LT OF CENTERLINE-POINT NO.	DISTANCE	ELEVATION
10+00	85		5607.92
		49.62	
10+50	86		5607.89
		49.95	
11+00	87		5607.75
		50.15	
11+50	88		5607.57
		50.05	
12+00	89		5608.11
		49.98	
12+50	90		5608.26
		49.96	
13+00	91		5608.44
		49.63	
13+50	92		5608.35
		50.47	
14+00	93		5608.16
		49.77	
14+50	94		5607.96
		50.42	
15+00	95		5607.89
		49.48	
15+50	96		5607.79
		49.94	
16+00	97		5607.87
		50.05	
16+50	98		5608.33
		49.86	
17+00	99		5608.74
		50.03	
17+50	100		5608.71
		49.98	
18+00	101		5608.42
		50.11	
18+50	102		5608.07
		50.12	
19+00	103		5608.11
		49.70	
19+50	104		5608.09
		50.73	
20+00	105		5608.12

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CELL DIKE MONITORING SYSTEM CELL #3 DIKE 3/82

APPROXIMATE CENTERLINE STATION	10.00' LT OF CENTERLINE-POINT NO.	DISTANCE	ELEVATION
20+00	105		5608.12
		49.45	
20+50	106		5608.26
		50.06	
21+00	107		5608.05
		50.21	
21+50	108		5607.77
		49.51	
22+00	109		5607.76
		49.84	
22+50	110		5607.79
		50.23	
23+00	111		5607.98
		50.11	
23+50	112		5607.99
		49.83	
24+00	113		5608.05
		49.99	
24+50	114		5608.21
		50.77	
25+00	115		5608.22
		49.34	
25+50	116		5608.17
		49.86	
26+00	117		5607.80
		50.04	
26+50	118		5607.76
		50.13	
27+00	119		5607.77
		50.17	
27+50	120		5608.17
		49.85	
28+00	121		5608.14
		50.14	
28+50	122		5608.23
		50.09	
29+00	123		5608.10
		49.62	
29+50	124		5608.10
		50.76	
30+00	125		5607.62

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CELL DIKE MONITORING SYSTEM   CELL #3 DIKE   PAGE 4   3/82

APPROXIMATE CENTERLINE STATION	10.00' LT OF CENTERLINE-POINT NO.	DISTANCE	ELEVATION
30+00	125		5607.62
		49.07	
30+50	126		5608.01
		50.05	
31+00	127		5608.20
		50.10	
31+50	128		5608.27
		49.96	
32+00	129		5608.10
		49.98	
32+50	130		5607.78
		49.91	
33+00	131		5608.11
		50.08	
33+50	132		5608.40
		42.68	
33+91.8 (24)	133		5608.55

7-10-86

NRC Statement:

3. The liquefaction analysis that determined the potential for liquefaction was "nil" - must be included in the Plan.

Response:

The foundation soils are partially saturated (i.e., have negative in-situ pore pressures). For liquefaction to occur, an excess hydrostatic pressure build-up due to application of cyclic shear stresses from strong ground motions would have to occur approximately equally to the applied confining pressure. Because of the absence of a water table, liquefaction is not deemed a problem at this site. However, we have provided computations assuming the water table to be at the surface and conservatively neglecting the confining effects of the embankment. The analysis is based on the following:

a) Seismic Risk Assessment - A Seismic Risk Assessment was performed by Dames & Moore in 1978. The assessment was updated by Dr. James Johnson of Mesa College in 1988 for the White Mesa Reclamation Plan. A copy of the Seismic Risk Assessment is enclosed.

b) Soil profiles developed by Dames & Moore, D'Appolonia and Chen & Associates.

c) Procedures developed by Seed & Adriss (1982) in "Ground Motions and Soil Liquefaction during Earthquakes". Earthquake Engineering Research Institute, Berkeley, California.

The attached computations show that liquefaction at the White Mesa Project site is not a problem.

I Seismic Risk Assessment from Reclamation [illegible]
USE MCE = 6.5

II MCE of 6.5 produces a ground acceleration of 0.07g ON Rock at site

III For a med. Stiff clay ~ 0.07g (see attached ground acceleration Curves fig (2.6-9))

IV Compute Cyclic Stress Ratio developed at site from MCE:

$$\tau_d = \frac{\tau_{av}}{\sigma_0'} = 0.65 \cdot \frac{a_{mce}}{g} \cdot \frac{\sigma_0}{\sigma_0'} \cdot R_d$$

where R_d = reduction Factor use 1.0
σ_0 = Total stress = 10 × 110 = 1100 psf
σ_0' = Effect. stress = 1110 - 10(62.4) = 476 psf

$$\therefore \tau_d = 0.65 \cdot \frac{0.07g}{g} \cdot \frac{1100}{476} = 0.105$$

V Compute Avg cyclic stress required to cause Liquefaction
Require Average std. penetration resistance of soils at use. Examination of soil Logs $N_{low} = 10$

modify penetration resistance Fig 47 attached
N = (1.6)(10) = 16

Use Fig 57 (attached) to compute Cyclic stress Ratio at site to cause Liquefaction

From Chart $\frac{\tau_{av}}{\sigma_v'} = 0.21$

VI Compute Factor of Safety Against Liquefaction

$$F_s = \frac{\tau_L}{\tau_d} = \frac{\text{Avg cyclic stress to cause Liquefaction}}{\text{Avg cyclic stress induced by earthquake}}$$

$$F_s = \frac{0.21}{0.105} = 2.0 \Leftarrow$$

A layer of Quaternary to Recent reddish-brown eolian silt and fine sand is spread over the surface of the project site. Most of the loess consists of subangular to rounded frosted quartz grains that are coated with iron oxide. Basically, the loess is massive and homogeneous, ranges in thickness from a dust coating on the rocks that form the rim cliffs to more than 20 feet (6 meters), and is partially cemented wth calcium carbonate (caliche) in light-colored mottled and veined accumulations which probably represent ancient immature soil horizons.

2.6.2.3 Structure [2.4.2.3]

The geologic structure at the project site is comparatively simple. Strata of the underlying Mesozoic sedimentary rocks are nearly horizontal; only slight undulations along the caprock rims of the upland are perceptible and faulting is absent. In much of the area surrounding the project site the dips are less than one degree. The prevailing regional dip is about one degree to the south. The low dips and simple structure are in sharp contrast to the pronounced structural features of the Comb Ridge Monocline to the west and the Abajo Mountains to the north.

Jointing is common in the exposed Dakota-Burro Canyon sandstones along the mesa's rim. More often than not, the primary joints are virtually parallel to the cliff faces and the secondary joints are almost perpendicular to the primary joints. Since erosion of the underlying weaker Brushy Basin mudstones removes both vertical and lateral support of the sandstone, large joint blocks commonly break away from the cliff leaving joint surfaces as the cliff face. Because of this, it is not possible to determine if the joints originated after the development of the canyons or if the joints influenced the developmment of canyons and cliffs. However, from a geomorphologic standpoint, it appears that the joints are related to the compaction of the underlying strata and, therefore, are sedimentary and physiographic features rather than tectonic in origin. Whatever the original cause, two sets of joint attitudes exist in the resistant sandstones adjacent to the west side of the project site. These sets range from N.10-18°E. and N.60-85°E. and nearly parallel the cliff faces.

2.6.3 Seismic Risk Assessment

2.6.3.1 General

This review utilized the environmental assessment completed in 1978 for the White Mesa Uranium Project by Dames & Moore (1978b). Information has been updated and procedures have been modified to conform to current requirements.

2.6.3.2 Seismic History of Region

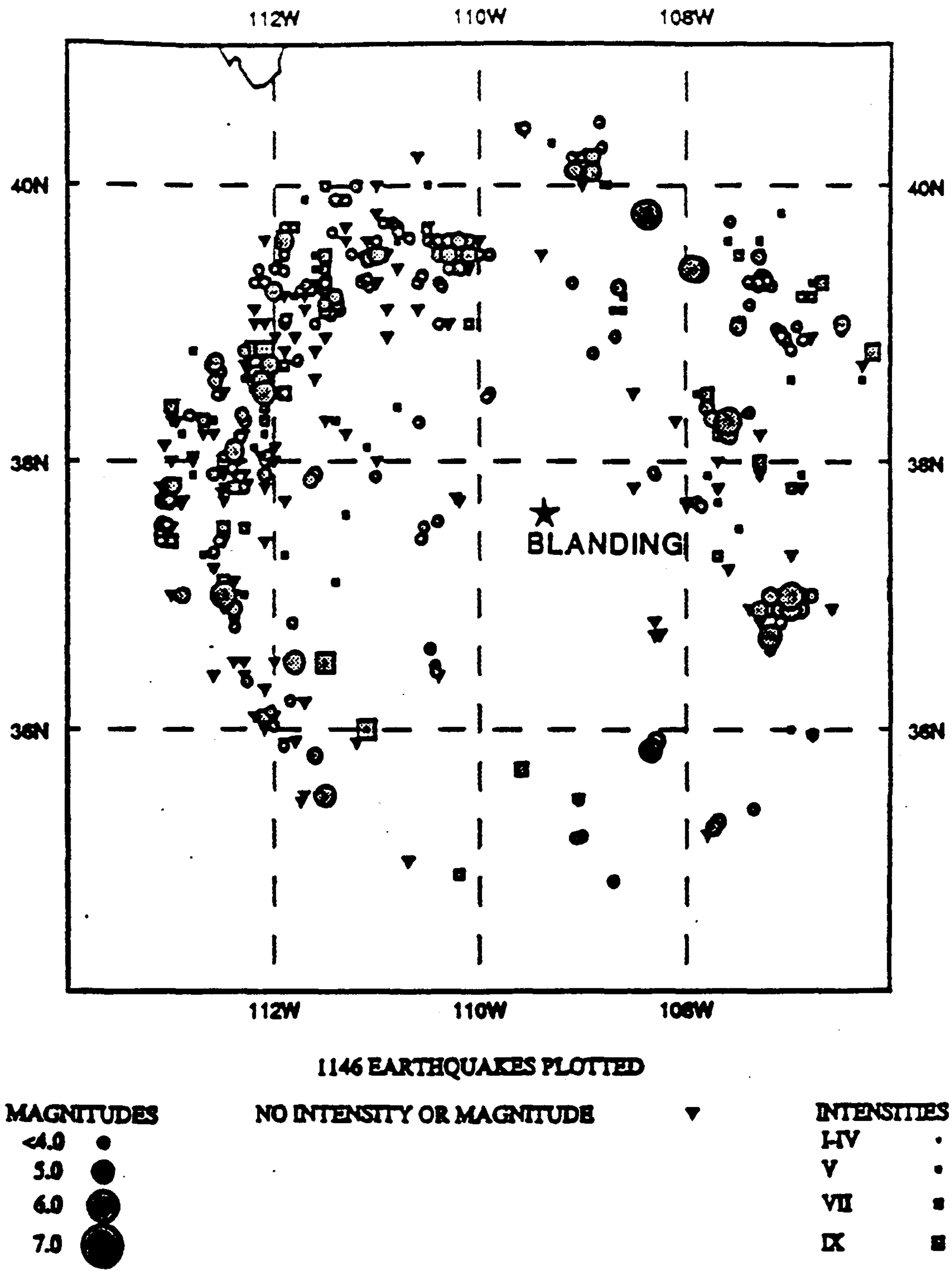
Because of the region's late settlement, the record of earthquake occurrences in the Colorado Plateau and surrounding regions dates back only 135 years. Documentation of the earlier events was based solely on newspaper reports that frequently recorded effects only in the more populated areas which may have been some distance from the epicenters. Not until the late 1950's was a seismograph network developed to properly locate and evaluate seismic events in this region (Simon, 1972).

The project area is within a relatively tectonically stable portion of the Colorado Plateau noted for its scarcity of historical seismic events. Conversely, the border between the Colorado Plateau and the Basin and Range Province and Middle Rocky Mountain Province some 155 to 240 miles (249 to 386 km) west and northwest, respectively, from the site is one of the most active seismic belts in the western United States.

The epicenters of historical earthquakes from 1853 through 1986 within a 200-mile (320-km) radius of the site are shown in Figure 2.6-3. More than 1146 events have occurred in the area, of which at least 45 were damaging; that is, having an intensity of VI or greater on the Modified Mercalli Scale. A description of the Modified Mercalli Scale is given in Table 2.6-3. All intensities mentioned herein refer to this table. Table 2.6-3 also shows a generalized relationship between Mercalli intensities and other parameters to which this review will refer. Since these relationships are frequently site specific, the table values should be used only for approximation and understanding.

Only 63 non-duplicative epicenters have been recorded within a 120 mile (200 km) radius of the project area (Figure 2.6-4). Of these, 50 had an intensity IV or less (or unrecorded) and two were recorded as intensity VI. The nearest event occurred in the Glen Canyon National Recreation Area approximately 38 miles (63 km) west-northwest of the project area. The next closest event occurred approximately 53 miles (88 km) to the northeast. Just east of Durango, Colorado, approximately 99 miles (159 km) due east of the project area, an event having local intensity of V was recorded on August 29, 1941 (Hadsell, 1968). It is very doubtful that these events would have been felt in the vicinity of Blanding.

Three of the most damaging earthquakes associated with the seismic belt along the Colorado Plateau's western border have occurred in the Elsinore-Richfield area about 168 miles (270 km) northwest of the project site. All were of intensity VIII. On November 13, 1901, a strong shock caused extensive damage from Richfield to Parowan. Many brick structures were damaged; rockslides were reported near Beaver. Earth cracks with the ejection of sand and water were reported, and some creeks increased their flow. Aftershocks continued for several weeks (von Hake, 1977). Following several weeks of small foreshocks, a strong earthquake caused major damage in the Monroe-Elsinore-Richfield area on September 29, 1921. Scores of chimneys were thrown down, plaster fell from ceilings, and a section of a new two-story brick wall collapsed at Elsinore's schoolhouse. Two days later, on October 1, 1921, another strong tremor caused additional damage to the area's structures. Large rockfalls occurred along both sides of the Sevier Valley and hot springs were discolored by iron oxides (von Hake, 1977). It is probable that these shocks may have been perceptible at the project site but they certainly would not have caused any damage.



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WHITE MESA PROJECT

SEISMICITY
320KM AROUND
BLANDING UTAH

JUNE, 1988 FIGURE 2.6-3

TABLE 2.8-3

MODIFIED MERCALLI SCALE, 1958 VERSION*

M§	Intensity	Effects	v,† cm/s	a‡ g
	I.	Not felt. Marginal and long-period effects of large earthquakes (for details see text).		
3	II.	Felt by persons at rest, on upper floors, or favorably placed.		
	III.	Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.		0.0035–0.007
4	IV.	Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wooden walls and frame creak.		0.007–0.015
	V.	Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.	1–3	0.015–0.035
5	VI.	Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc., off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken (visibly, or heard to rustle—CFR).	3–7	0.035–0.07
6	VII.	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments—CFR). Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.	7–20	0.07–0.15
	VIII.	Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.	20–60	0.15–0.35
7	IX.	General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. (General damage to foundations—CFR.) Frame structures, if not bolted, shifted off foundations. Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.	60–200	0.35–0.7
8	X.	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.	200–500	0.7–1.2
	XI.	Rails bent greatly. Underground pipelines completely out of service.		>1.2
	XII.	Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.	From Fig. 11.14	

NOTE: Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering (which has no connection with the conventional Class A, B, C construction).

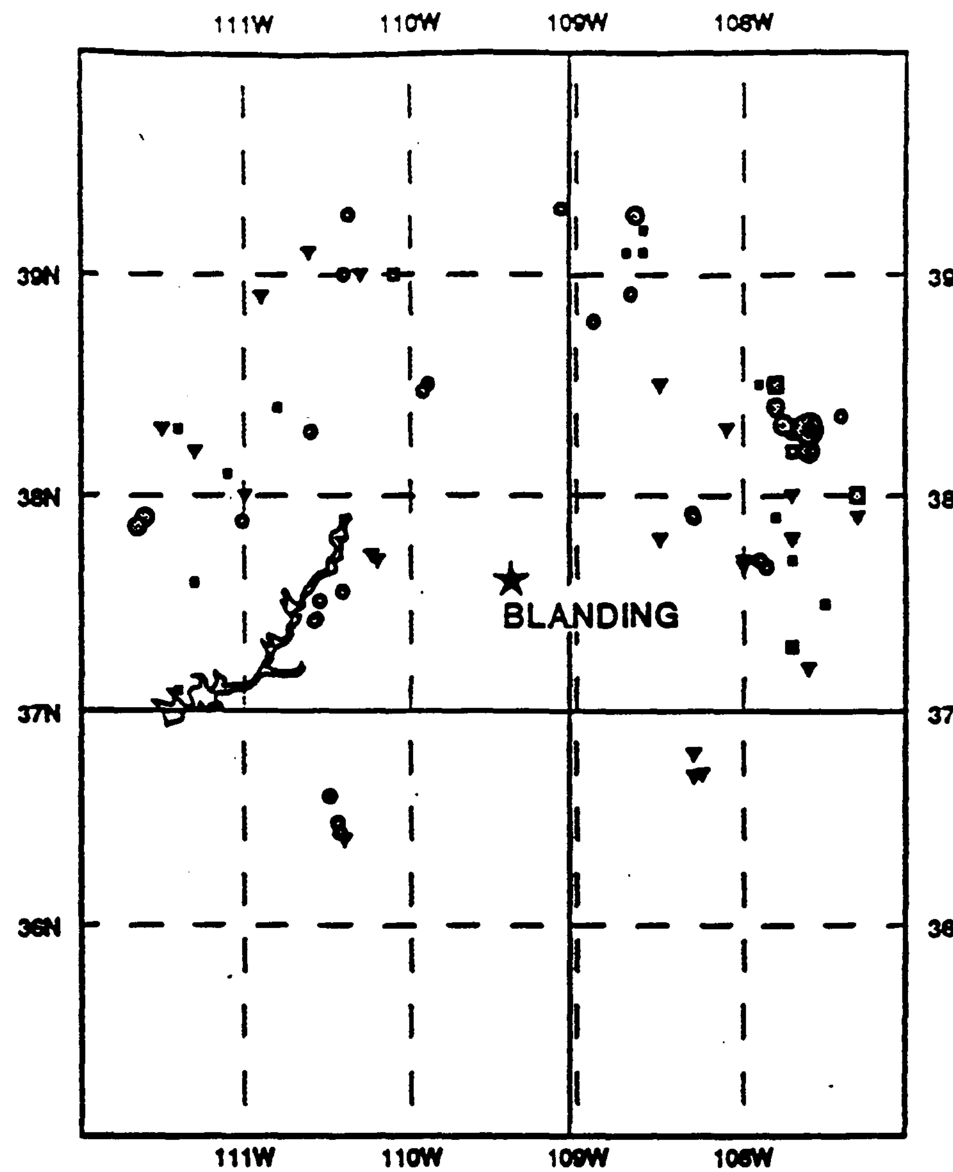
- Masonry A: Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.
- Masonry B: Good workmanship and mortar; reinforced, but not designed to resist lateral forces.
- Masonry C: Ordinary workmanship and mortar; no extreme weaknesses such as non-tied-in corners, but masonry is neither reinforced nor designed against horizontal forces.
- Masonry D: Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

*From Richter (1958).[1] Adapted with permission of W. H. Freeman and Company. by Hunt (1984).

†Average peak ground velocity, cm/s.

‡Average peak acceleration (away from source).

§Magnitude correlation.



103 EARTHQUAKES PLOTTED

MAGNITUDES
<4.0
5.0
6.0
7.0

NO INTENSITY OR MAGNITUDE

INTENSITIES
I-IV
V
VII
IX

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WHITE MESA PROJECT
SEISMICITY
200KM AROUND
BLANDING UTAH
JUNE, 1988 FIGURE 2.6-

Seven events of intensity VII have been reported in the area shown in Figure 2.6-3. Of these, only two are considered to have any significance with respect to the project site. On August 18, 1912, an intensity VII shock damaged houses in northern Arizona and was felt in Gallup, New Mexico, and southern Utah. Rock slides occurred near the epicenter in the San Francisco Mountains and a 50-mile (80 km) earth crack was reported north of the San Francisco Range (U. S. Geological Survey, 1970). Nearly every building in Dulce, New Mexico, was damaged to some degree when shook by a strong earthquake on January 22, 1966. Rockfalls and landslides occurred 10 to 15 miles (16 to 24 km) west of Dulce along Highway 17 where cracks in the pavement were reported (Hermann et al., 1980). Both of these events may have been felt at the project site but, again, would certainly not have caused any damage.

2.6.3.3 Relationship of Earthquakes to Tectonic Structures

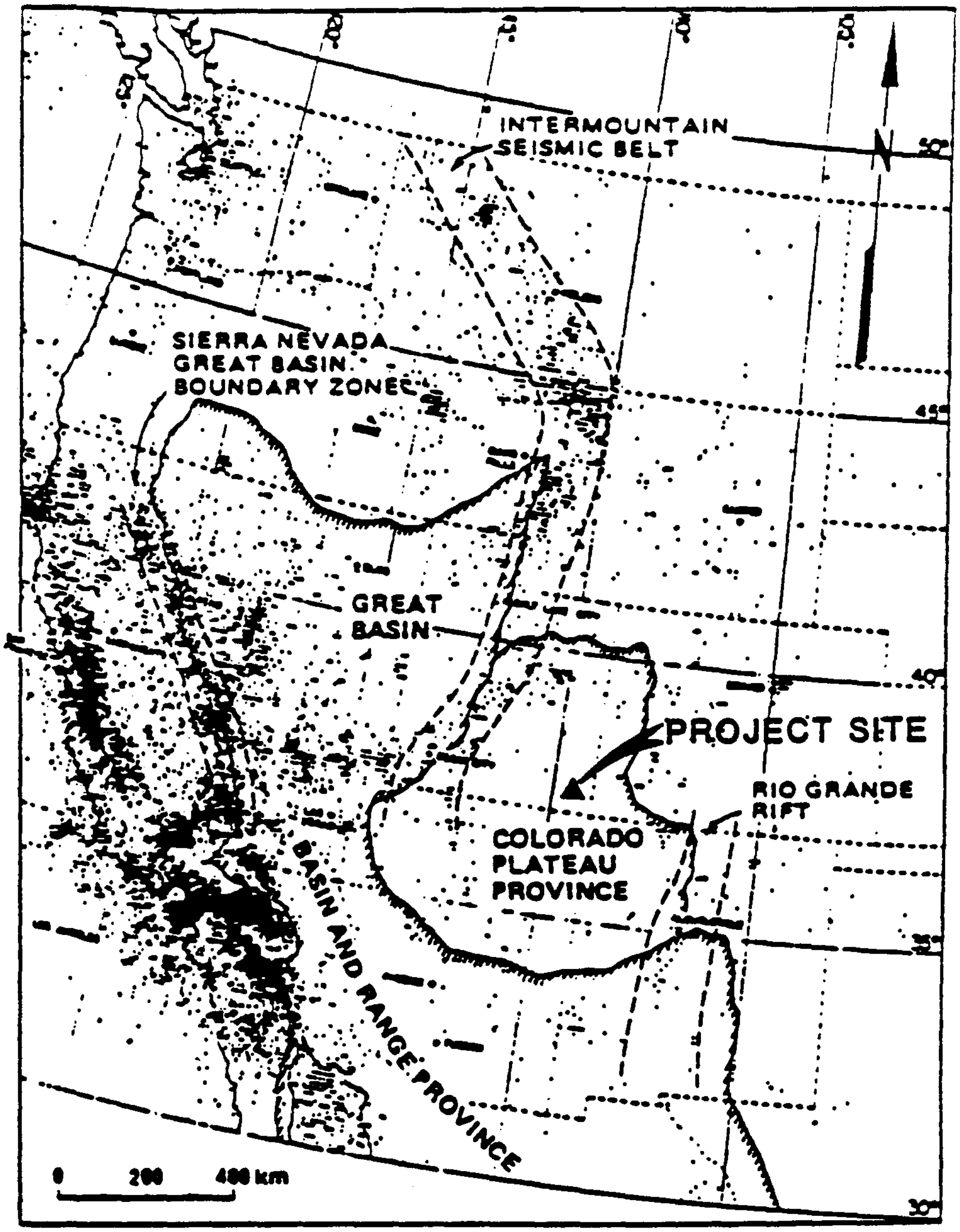
The majority of recorded earthquakes in Utah have occurred along an active belt of seismicity that extends from the Gulf of California, through western Arizona, central Utah, and northward into western British Columbia. The seismic belt is possibly a branch of the active rift system associated with the landward extension of the East Pacific Rise (Cook and Smith, 1967). This belt is the Intermountain Seismic Belt shown in Figure 2.6-5 (Smith, 1978).

It is significant to note that the seismic belt forms the boundary zone between the Basin and Range - Great Basin Provinces and the Colorado Plateau - Middle Rocky Mountain Provinces. This block-faulted zone is about 47 to 62 miles (75 to 100 km) wide and forms a tectonic transition zone between the relatively simple structures of the Colorado Plateau and the complex fault-controlled structures of the Basin and Range Province (Cook and Smith, 1967).

Another zone of seismic activity is in the vicinity of Dulce, New Mexico, near the Colorado border. This zone, which coincides with an extensive series of Tertiary intrusives, may also be related to the northern end of the Rio Grande Rift. This rift is a series of fault-controlled structural depressions extending southward from southern Colorado through central New Mexico and into Mexico. The rift is shown on Figure 2.6-5 trending north-south to the east of the project area.

Most of the events south of the Utah border of intensity V and greater are located within 50 miles (80 km) of post-Oligocene extrusives. This relationship is not surprising because it has been observed in many other parts of the world (Hadsell, 1968).

In Colorado, the Rio Grande Rift zone is one of three seismotectonic provinces that may contribute energy to the study area. Prominent physiographic expression of the rift includes the San Luis Valley in southern Colorado. The valley is a half-graben structure with major faulting on the eastern flank. Extensional tectonics is dominant in the area and very large earthquakes with recurrence intervals of several thousand years have been



Modified from Smith, 1978

SHOWS RELATIONSHIP OF THE COLORADO PLATEAU PROVINCE TO MARCANAL BELTS

UMETCO MINERALS CORPORATI
WHITE MESA PROJECT

SEISMICITY OF THE WESTER
UNITED STATES, 1950 TO 19

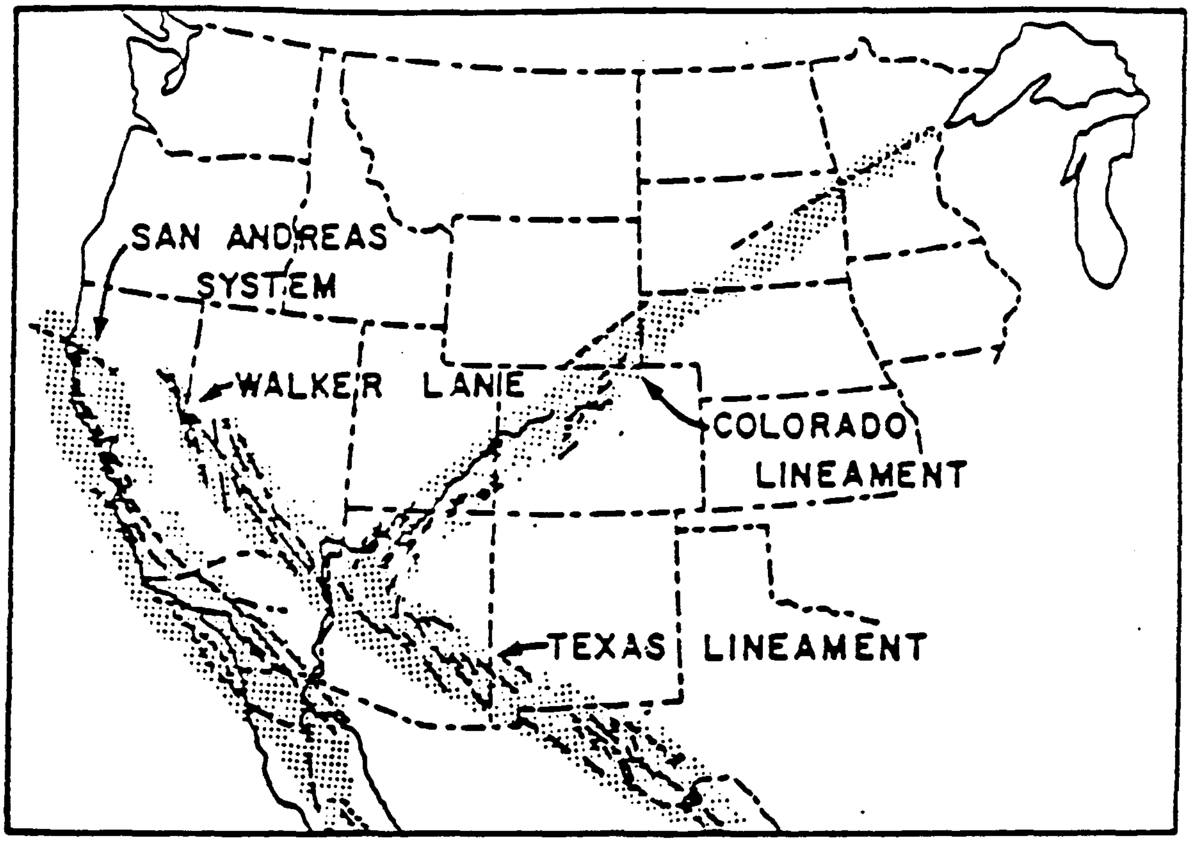
JUNE, 1988 FIGURE 2.6-5

projected (Kirkham and Rodgers, 1981). Mountainous areas to the west of the Rio Grande rift province include the San Juan Mountains. These mountains are a complex domical uplift with extensive Oligocene and Miocene volcanic cover. Many faults are associated with the collapse of the calderas and apparently have not moved since. Faults of Neogene age exist in the eastern San Juan Mountains that may be related to the extension of the Rio Grande rift. Numerous small earthquakes have been felt or recorded in the western mountainous province despite an absence of major Neogene tectonic faults (Kirkham and Rodgers, 1981).

The third seismotectonic province in Colorado, that of the Colorado Plateau, extends into the surrounding states to the west and south. In Colorado, the major tectonic element that has been recurrently active in the Quaternary is the Uncompahgre uplift. Both flanks are faulted and earthquakes have been felt in the area. The faults associated with the Salt Anticlines are collapsed features produced by evaporite solution and flowage (Cater, 1970). Their non-tectonic origin and the plastic deformation of the salt reduces their potential for generating even moderate-sized earthquakes (Kirkham and Rodgers, 1981).

Case and Joesting (1972) have called attention to the fact that regional seismicity of the Colorado Plateau includes a component added by basement faulting. They inferred a basement fault trending northeast along the axis of the Colorado River through Canyonlands. This basement faulting may be part of the much larger structure that Hite (1975) examined and Warner (1978) named the Colorado lineament (Figure 2.6-6). This 1300-mile (2100-km) long lineament that extends from northern Arizona to Minnesota is suggested to be a Precambrian wrench-fault system formed some 2.0 to 1.7 billion years before present. While it has been suggested that the Colorado lineament is a source zone for larger earthquakes ($m = 4$ to 6) in the west-central United States, the observed spatial relationship between epicenters and the trace of the lineament does not prove a causal relation (Brill and Nuttli, 1983). In terms of contemporary seismicity, the lineament does not act as a uniform earthquake generator. Only specific portions of the proposed structure can presently be considered seismic source zones and each segment exhibits seismicity of distinctive activity and character (Wong, 1981). This is a reflection of the different orientations and magnitudes of the stress fields along the lineament. The interior of the Colorado Plateau forms a tectonic stress province, as defined by Zoback and Zoback (1980), that is characterized by generally east-west tectonic compression. Only where extensional stresses from the Basin and Range province of the Rio Grande rift extend into the Colorado Plateau would the Colorado lineament in the local area be suspected of having the capability of generating a large magnitude earthquake (Wong, 1984). At the present time, the well defined surface expression of regional extension is far to the west and far to the east of the project area.

Recent work by Wong (1984) has helped define the seismicity of the whole Colorado Plateau. He called attention to the low level (less than $M_L = 3.6$) but high number (30) of earthquakes in the Capitol Reef area from 1978 to 1980 that were associated with the Waterpocket fold and the Cainville monocline,



SOURCE: WARNER, 1978

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COLORADO LINEAMENT

JUNE, 1988 FIGURE 2.6-5

two other major tectonic features of the Colorado Plateau. Only five earthquakes in the sequence were of M_L greater than 3, and fault plane solutions suggest the swarm was produced by normal faulting along northwest-trending Precambrian basement structures (Wong, 1984). The significance of the Capitol Reef seismicity is its relatively isolated occurrence within the Colorado Plateau and its location at a geometric barrier in the regional stress field (Aki, 1979). Stress concentration that produces earthquakes at bends or junctures of basement faults as indicated by this swarm may be expected to occur at other locations in the Colorado Plateau Province. No inference that earthquakes such as those at Capitol Reef are precursors for larger subsequent events is implied.

2.6.3.4 Potential Earthquake Hazards to Project

The project site is located in a region known for its scarcity of recorded seismic events. Althought the seismic history for this region is barely 135 years old, the epicentral pattern, or fabric, is basically set and appreciable changes are not expected to occur. Most of the larger seismic events in the Colorado Plateau have occurred along its margins rather than in the interior central region. Based on the region's seismic history, the probability of a major damaging earthquake occurring at or near the project site is very remote. Studies by Algermissen and Perkins (1976) indicate that southeastern Utah, including the site, is in an area where there is a 90 per cent probability that a horizontal acceleration of four per cent gravity (0.04g) would not be exceeded within 50 years.

Minor earthquakes, not associated with any seismic-tectonic trends, can presumably occur randomly at almost any location. Even if such an event with an intensity as high as VI should occur at or near the project site, horizontal ground accelerations would not exceed 0.10g but would probably range between 0.05 an 0.09g (Coulter et al., 1973; Trifunac and Brady, 1975). These magnitudes of ground motion would not pose significant hazards to the existing and proposed facilities at the Project Site.

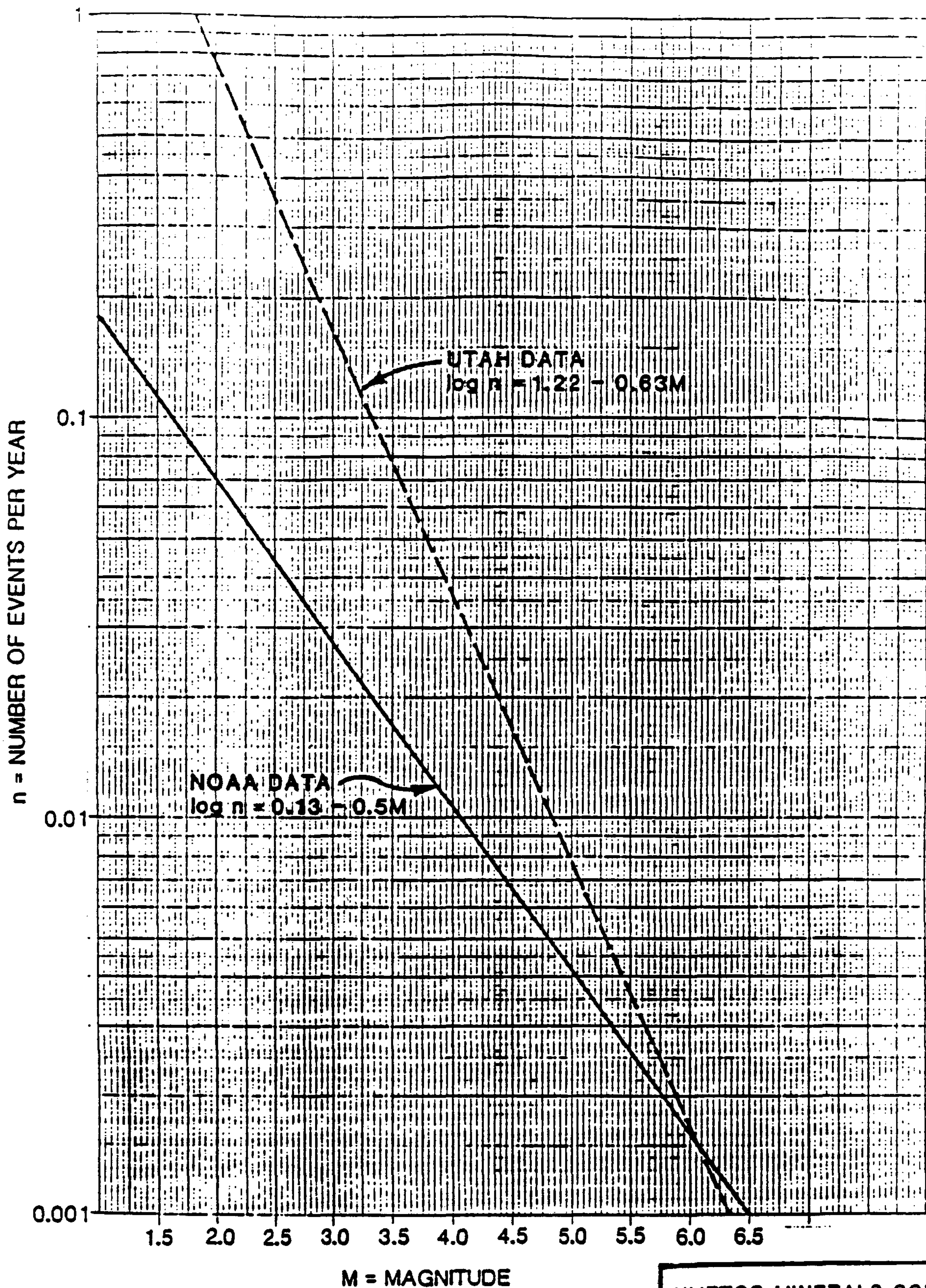
In addition to general estimates of earthquake hazards, such as those offered by Dames and Moore (1978b), a more detailed analysis of the relationship between the project area and regional seismicity was performed. As can be seen in Figure 2.6-3, a map based on the seismologic data base from the National Geophysical Data Center of the National Oceanic and Atmospheric Administration (NOAA 1988), many events occur within the Intermountain Seismic Belt and within the Rio Grande rift. Since the Colorado Plateau Province, and particularly the Blanding basin portion, in which the project site lies, is a distinctly different tectonic province, the historical sample chosen for magnitude/frequency estimates was limited to a radius of about 120 miles (200 km) from the project. This sample included a region which is more representative of the seismicity of the Colorado Plateau.

The map in Figure 2.6-4 shows a plot of earthquake epicenters within 124 miles (200 km) of Blanding. In comparing this map with Figure 2.6-3, it is apparent that the project site is located in an area of very low seismic flux compared to the marginal areas to both east and west. Using a sample of 46 events ranging in magnitude from 2 to 5.5 and covering a time span of 135 years, the calculated yearly rates for each magnitude range were normalized to an area of 6,823 square miles (17,671 km^2) [47-mile (75-km) radius]. A Poisson distribution was assumed and a linear regression produced the best fit line of magnitude vs. frequency shown in Figure 2.6-7. The form of $\log(n) = b + aM$ (where n is the expected number of events per year of magnitude M) was used. The sample of 46 events included all non-duplicative epicenters with reported magnitude or intensity. Intensities were converted to magnitudes where necessary using the equation $M = 2.1 + 0.5I$ as recommended by Krinitsky and Chang (1975). The resulting recurrence intervals of events of a specific magnitude are considerably longer than for the same size event in the more active zones to the east and west but are more realistic of the Blanding area (Arabasz, Smith and Richins, 1979; Kirkham and Rodgers, 1981). The magnitude/frequency relationship determined is not intended to represent true rates but is only a conservative estimate based on samples which are representative of only the area near the White Mesa project site.

The historical records from both NOAA and Utah data sources list only five events which have occurred within 62 miles (100 km) of the study site. The largest of these relatively near events was a magnitude 4.0 event on July 22, 1986, that occurred 61.5 miles (99 km) west of the site. In the 124 miles (200 km) radius data set the largest event is one of magnitude 5.5 on October 11, 1960, at a distance of 112 miles (180 km) from the project.

The nearest fault to the site identified on maps by Andersen and Miller (1979) as youthful and therefore presumably potentially active is 25 miles (57 km) north of the project site (Figure 2.6-8). The fault, as mapped, has a well defined length of about 1.9 miles (3 km) and a possible total length of 6.8 miles (11 km). The fault is identified by Anderson and Miller as a suspected Quaternary fault, but may not be the result of tectonic activity. The evidence for Quaternary movement on this fault is not strong and it appears to be a discrete extension of the South fault of the Shay graben. No other part of the surface expression of the Shay graben structure gives any indication that the fault's traces are anything other than erosionally produced fault-line scarps. Nevertheless, it is appropriate to use this mapped suspected Quaternary fault for estimates for seismic design criteria since it is the nearest fault or fault segment to the project area that has been mapped as young. Additional discussion of the faulting in the Shay and Verdure grabens (Figure2.6-8) is included below.

Estimates of the maximum credible earthquake from this fault 35 miles (57 km) north of the project assuming rupture of 1.9 miles (3 km) and 6.7 miles (11 km), as well as a one-half total length ruture 3.4 miles (5.5 km) have been made using the relationships discussed in Slemmons (1977). These are summarized in Tables 2.6-4 and 2.6-5. Determinative assumptions included the expected length of surface rupture, type of fault movement, and regional location. This least squares approach to the regression analysis was well validated by the studies of Bonilla et al. (1984).



COVERS AREA WITHIN A 75 KM
RADIUS OF THE PROJECT SITE

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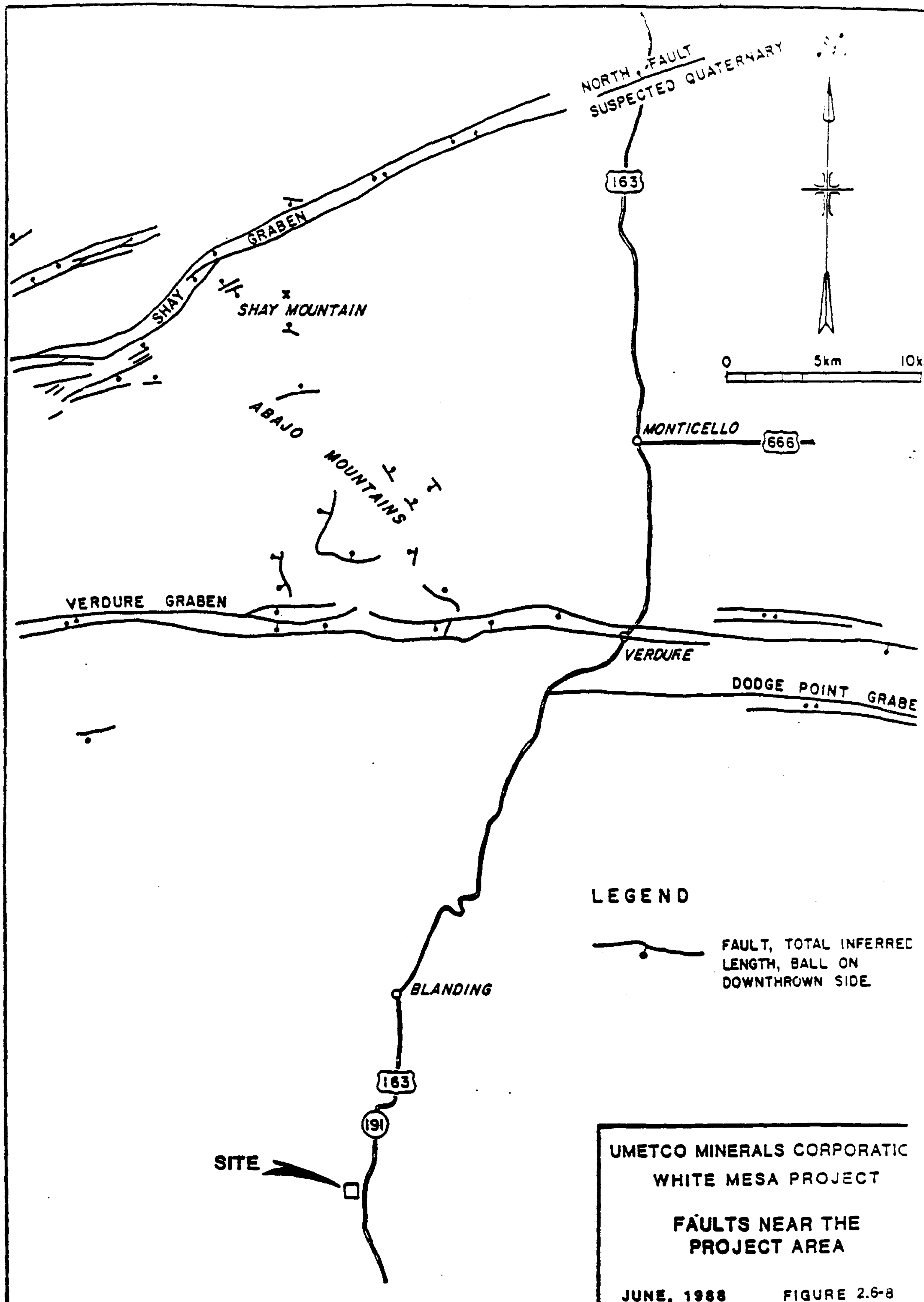
WHITE MESA PROJECT

MAGNITUDE / FREQUENCY RELATIONSHIP

JUNE, 1988 FIGURE 2.6-7



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FAULTS NEAR THE PROJECT AREA

JUNE, 1988 FIGURE 2.6-8

Table 2.6-4

Maximum Fault Displacement (D)
of
North Fault (Figure 2.6-8)

Parameter	a	b	D_L = 3 km	D_L = 5.5 km	D_L = 11 km
NA	-4.270	1.036	0.08 m	0.14 m	0.29 m
A	-4.375	1.014	0.14 m	0.26 m	0.53 m
A+C	-2.898	0.705	0.36 m	0.55 m	0.89 m

Relationship: Log D = a + b Log L

From: Slemmons (1977)

Table 2.6-5

Maximum Magnitude (M)
of
North Fault (Figure 2.6-8)

Relationship	Parameter	a	b	M_L = 3 km	M_L = 5.5 km	M_L = 11 km
	NA	-0.146	1.504	5.08	5.48	5.93
M = a + b Log L	A	1.845	1.151	5.85	6.15	6.50
	A+C	2.042	1.121	5.94	6.23	6.57
	NA	6.745	0.995	5.65	5.90	6.21
M = a + b Log D	A	6.827	1.050	5.93	6.21	6.54
	A+C	6.757	1.226	6.21	6.44	6.69
	NA	3.510	0.701	5.18	5.53	5.97
M = a + b Log LD	A	4.551	0.530	5.94	6.22	6.55
	A+C	3.691	0.707	5.84	6.15	6.51
	NA	4.808	0.420	5.35	5.66	6.05
M = a + b Log LD^2	A	5.568	0.299	6.10	6.34	6.61
	A+C	4.752	0.459	5.94	6.23	6.56
			MEAN	5.75	6.04	6.39
MAXIMUM MAGNITUDE (MEAN + S)				6.11	6.37	6.66

NA = North America
A = Normal Movement
A + C = Combined Normal and Oblique Movement

From: Slemmons (1977)

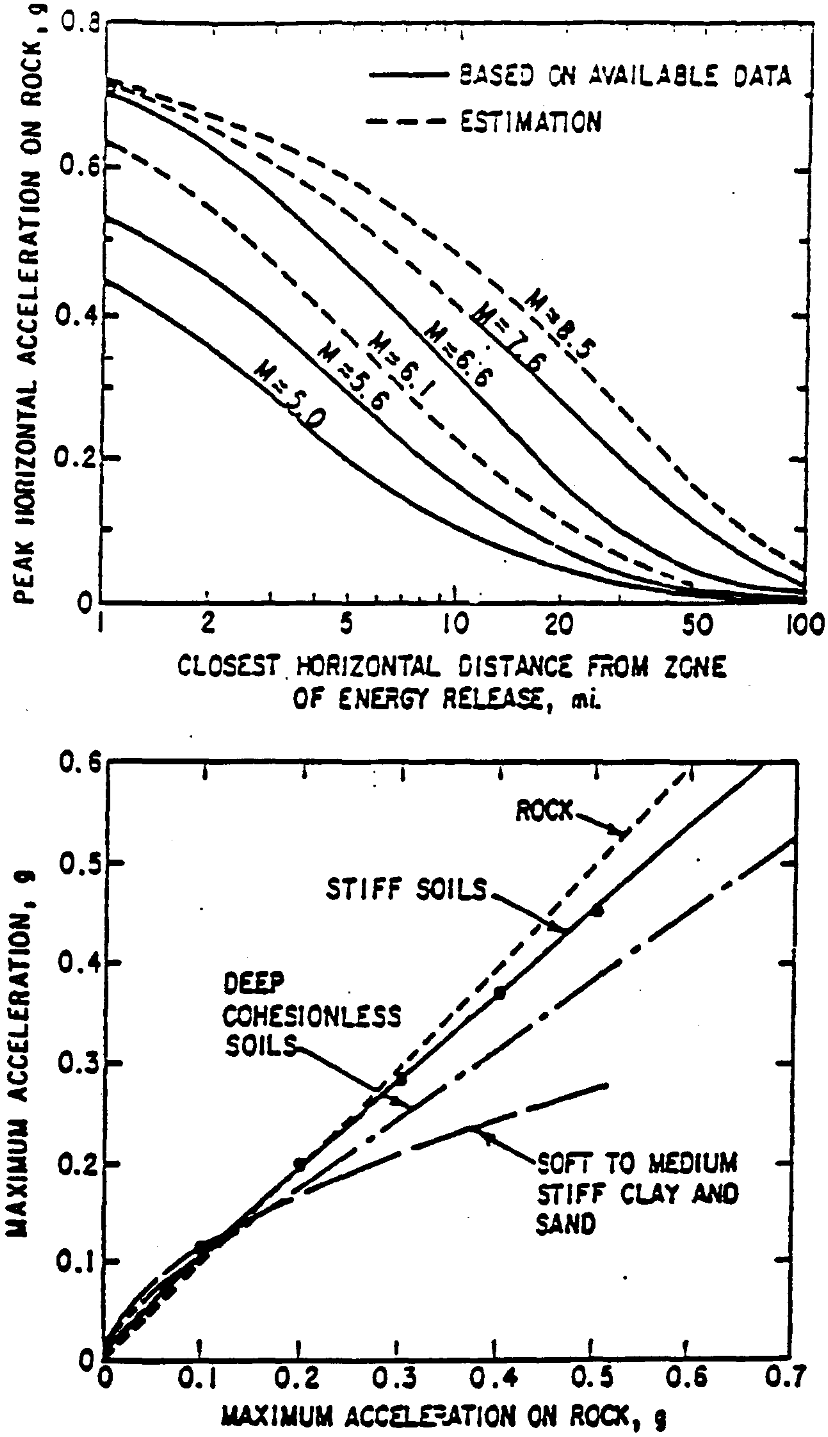
The maximum estimated magnitude of this fault assuming a 3.4 Miles (5.5 km) rupture length is 6.4. This is consistent with estimates of maximum magnitude for the Colorado Plateau by Kirkham and Rodgers (1981) and by Andersen and Miller (1979). Based on the estimated historical magnitude/frequency relationships, this maximum earthquake would have a recurrence interval of about 1000 years. Because the historical magnitude/frequency relationship estimated was not constrained at its upper right limit (say to a value of M 6.5), it is reasonable to deduce that recurrence-intervals determined are shorter (more conservative) than their actual rate of recurrence.

Ground motions at the White Mesa project site resulting from an maximum credible earthquake (MCE) on the fault discussed can be estimated from attenuation curves established by Seed and Idriss (1982). The graph in Figure 2.6-9 indicates an estimated peak horizontal acceleration at a distance of 57 km from the epicenter would be 0.07g.

Note that conservative estimates were used by assuming an estimated magnitude of 6.4 based on the mean plus one standard deviation and assuming the earthqake would initiate at a point on the fault closest 35 miles (57 km) to the project site.

The MCE and possible horizontal acceleration estimate have a very low probability of occurrance but would be used for seismic design parameters for a worst case scenario based on this mapped fault.

Faults associated with the Verdure graben are even closer to the White Mesa Project than the fault discussed above but they show no evidence of recent surface displacement. These faults are shown on the map on Figure 2.6-8 in spite of not being shown on the Quaternary Fault Map of Utah because of their surface expression and similarity in trend and structure to Shay graben to the north (Anderson and Miller, 1979; Woodward-Clyde Consultants, 1982). During micro-seismic monitoring from 1979 through 1982, Wong (1984) recorded only two events associated with these east-west grabens. Both of the micro-earthquakes (M_L 0.6 and M_L 0.7) were in the Abajo Mountains near the western end of the southern fault trace of the Shay graben. Data resolution did not permit these very small events to be assigned to the south fault and they may well be related to the old intrusions. However, this is the only indication found that any of these structures may be active. Detailed field study and mapping of the Verdure graben show Quaternary pediment gravels and alluvium overlying the graben in several places and Witkind (1964) indicated that the south fault was in igneous contact with the Rocky Trail laccolith with no slickensides present (Woodward-Clyde Consultants, 1982). Stream courses cross the faults of the graben with no deflection or gradient change. The implication is that these faults are old and may be of Oligocene age and related to the period of laccolithic intrusion. It is also possible, as many authors believe, that these fault structures are the result of salt flowage in underlying formations and not produced by tectonic stresses (Kirkham and Rodgers, 1981). Since the regional stress field in this area is approximately east-west, it is difficult to see how the normal faulting of the Verdure graben or the Shay graben and its possible extension that forms the small fault north of Monticello could be



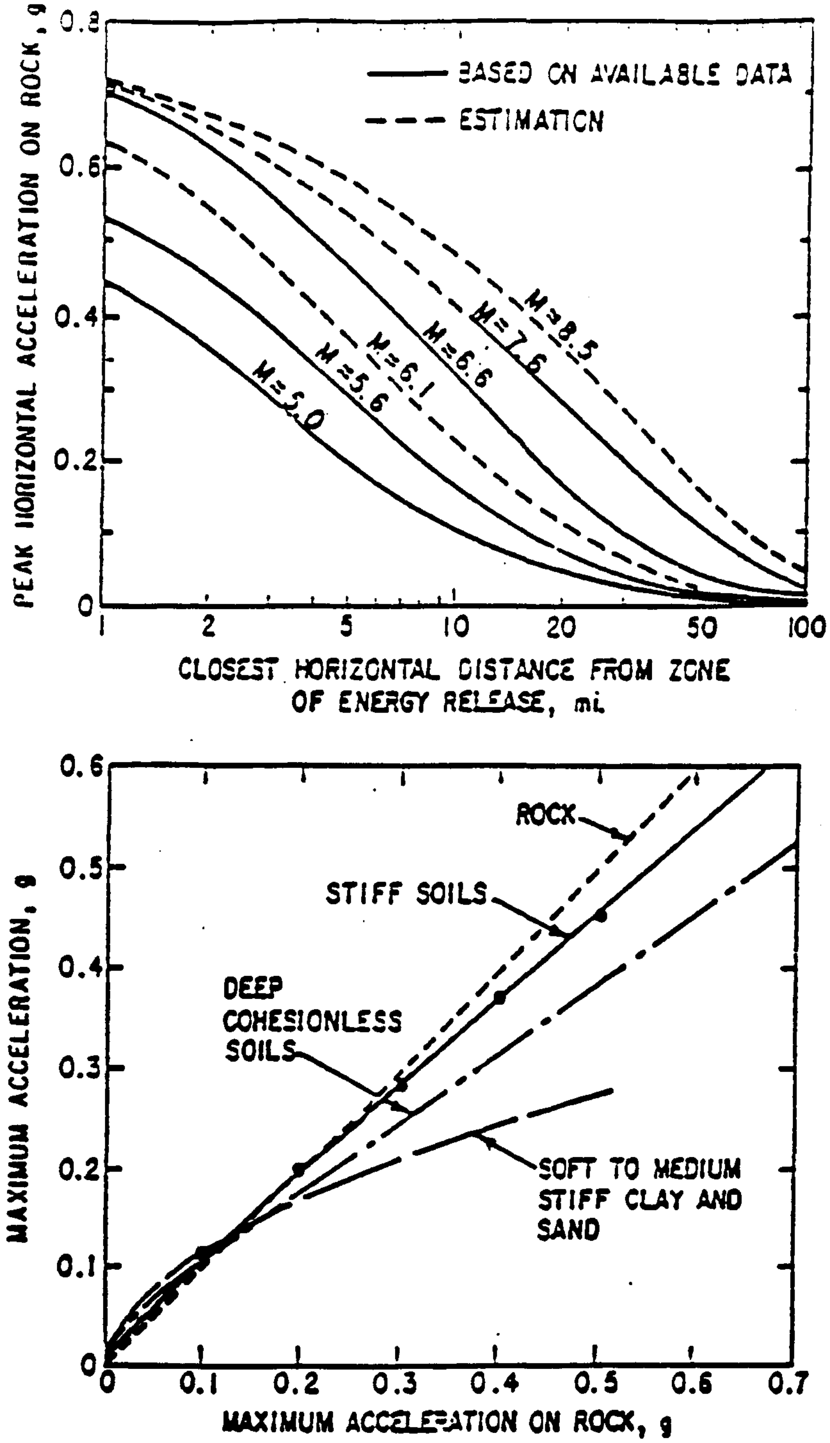
(From Seed and Idriss, 1982)

UMETCO MINERALS CORPORATIO

WHITE MESA PROJECT

GROUND ACCELERATION CURVES

JUNE, 1988 FIGURE 2.6-9



(From Seed and Idriss, 1982)

UMETCO MINERALS CORPORATI

WHITE MESA PROJECT

GROUND ACCELERATION CURVES

JUNE, 1988 FIGURE 2.6-

GROUND MOTIONS AND SOIL LIQUEFACTION DURING EARTHQUAKES

H. BOLTON SEED
I.M. IDRISS

EARTHQUAKE ENGINEERING RESEARCH INSTITUTE

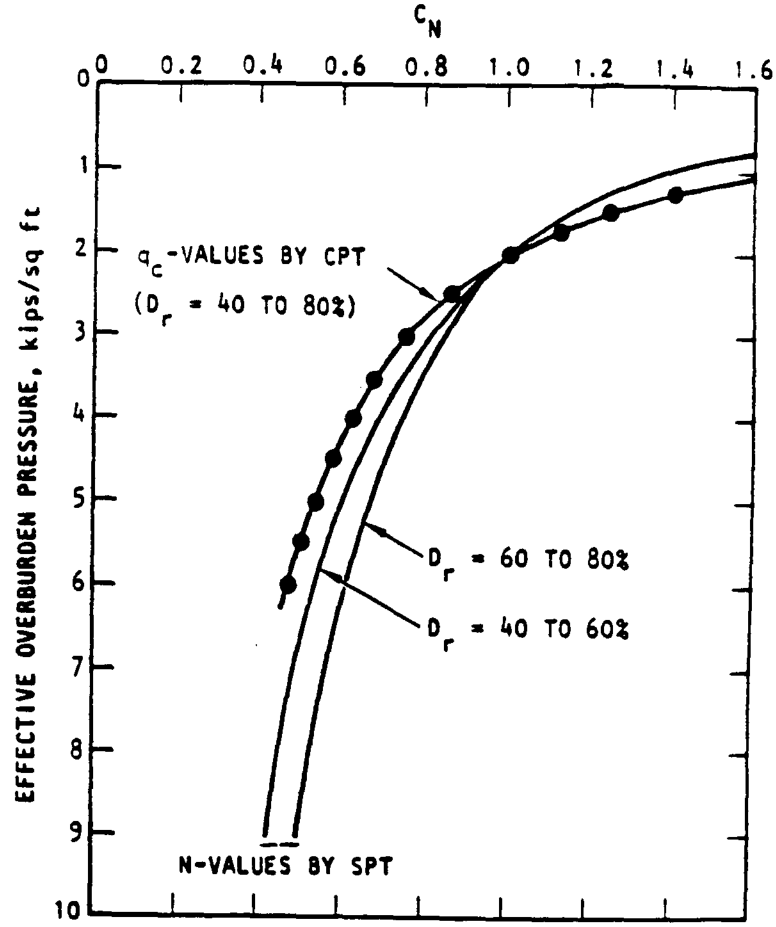
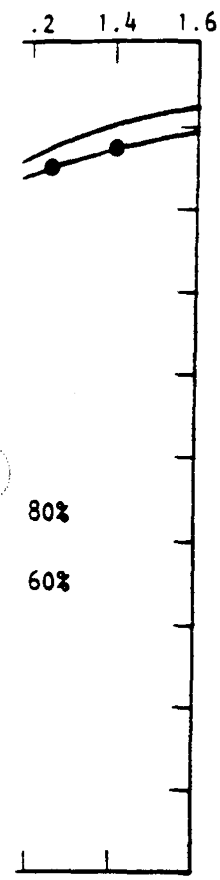


Figure 47. Chart for values of C_N.



Interestingly, all of these factors affect the cyclic loading characteristics of sands and the penetration resistance in the same general way. Changes in the factors that tend to increase the cyclic loading resistance also tend to increase the penetration resistance. Thus it is more meaningful to use penetration resistance directly as an index of liquefaction resistance than to use an individual factor such as relative density. More recent correlations of field data have recognized this fact and used penetration resistance directly rather than some other property. It would seem to make little difference whether the test used were the Standard Penetration Test (SPT) or the Cone Penetration Test (CPT) since either is likely to be a good indicator of cyclic loading liquefaction characteristics. However, because of its widespread use in areas where earthquakes have occurred, the Standard Penetration Test has provided virtually all the field test data available for field liquefaction studies.

Since the standard penetration resistance, N, measured in the field actually reflects the influence of the soil properties and the effective confining pressure, it has been found desirable to eliminate the influence of confining pressure by using a normalized penetration resistance N_1, where N_1 is the measured penetration of the soil under an effective overburden pressure of 1 ton per sq. ft. The value of N_1 for any sand can be determined from the measured value N from the relationship

$$N_1 = C_N \cdot N$$

where C_N is a function of the effective overburden pressure at the depth where the penetration test was conducted. Values of C_N may be read off from the chart shown in Fig. 47, which is based on studies conducted at the Waterways Experiment Station (Bieganousky and Marcuson, 1976; Marcuson and Bieganousky, 1977).

In principle there is no reason why other index characteristics (such as possible electrical properties, shear wave velocity or other in-situ test data) can not be correlated with cyclic loading characteristics determined in the field. However, there is very little field test data available to establish good correlations of field performance with any soil characteristics other than the

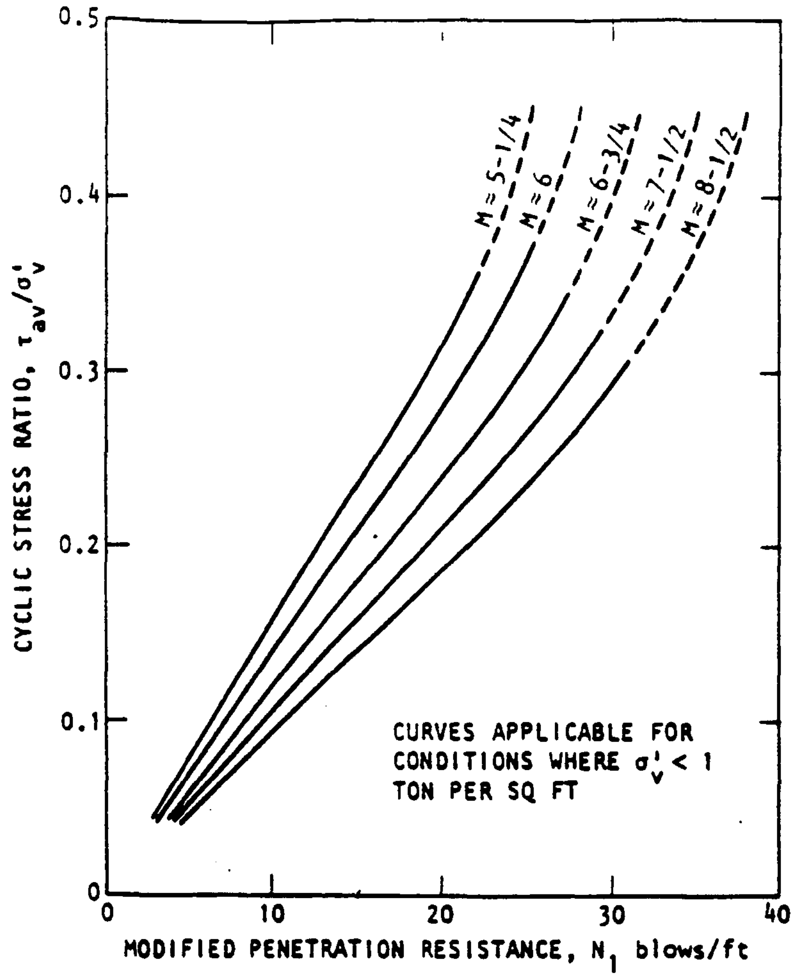


Figure 57. Chart for evaluation of liquefaction potential for sands for different magnitude earthquakes.

NRC Statement:

4. As only one cell will be constructed initially, the structural stability of the outside embankment curve should be addressed separately. Conventional analyses assume that the modeled section is supported on both sides.

Response:

We have enclosed a letter from Western Engineers addressing this statement.

Please also note Figure 3.4-1 of the Cell 4 design package. The stability analysis assumes that the synthetic liner has completely failed and that a phreatic surface (steady state seepage conditions) has developed within the embankment, adding to the conservatism of the analysis.

WESTERN ENGINEERS, INC. CONSULTING ENGINEERS / LAND SURVEYORS
2150 Hwy 6 & 50, Grand Junction, CO 81505 · 303 242 5202

March 22, 1989

Mr. Curt Sealy
UMETCO Minerals Corporation
P.O. Box 1029
Grand Junction, CO 81502

RE: Blanding Cell 4 Stability Analysis

Dear Curt:

In the NRC's cursory review of the Cell 4 design the following comment was made:

> As only one cell will be constructed initially, the structural stability of the outside embankment curve should be addressed separately. Conventional analyses assume that the modeled section is supported on both sides.

For the sake of conservatism, the stability analysis that was performed ignored the effects of side friction on the critical slide mass. Therefore, with respect to our evaluation, a change in the dike geometry laterally bounding the analyzed section would have no effect. This means that, using the same methods of analyses the only change in safety factor would be due to differing embankment geometry such as height and slopes. The embankment slopes around the curve are identical to the rest of the dike. The height at the curve is less than at the section analyzed which will result in an increase in factor of safety at the curve.

Very truly yours,

WESTERN ENGINEERS, INC.

John M. Currier, P.E.
Staff Engineer

CELL 4 DESIGN

TAILINGS MANAGEMENT SYSTEM

WHITE MESA PROJECT

BLANDING, UTAH

Umetco Minerals Corporation

AUGUST, 1988

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 SUMMARY OF TAILINGS CELL 4 DESIGN

1.1 Cell 4 Description

This document presents Umetco Minerals Corporation's design of proposed tailings Cells 4-A and 4-B. These cells will be constructed adjacent to the existing Cell 3. The Project Site is shown on Figure 1.1-1.

The configuration of Cells 4-A and 4-B and their location in the Tailings Management System are shown on Drawing C4-1. Umetco will defer construction of Cell 4-B until conditions appropriate to phased disposal dictate its construction. Cell 4-A construction will commence as soon as possible in 1989. References to "Cell 4" in this document shall mean both Cells 4-A and 4-B. The analytical results and engineering studies are applicable to both cells.

Cells 4-A and 4-B have been sized to each have a volume of approximately 1150 acre feet with a phased final surface area of 40 acres each. Each cell will be in compliance with 40 CFR Part 61. Cell 4 has been designed as a below-grade repository similar to the previous cells in the Tailings Management System. The impoundment will be contained on the downstream side by a homogenous earth dike. Groundwater protection has been provided for by use of a synthetic liner system in conjunction with a leak detection system and a slimes drain. Groundwater monitoring will be provided as described in Section 5.3.

1.2 Report Organization

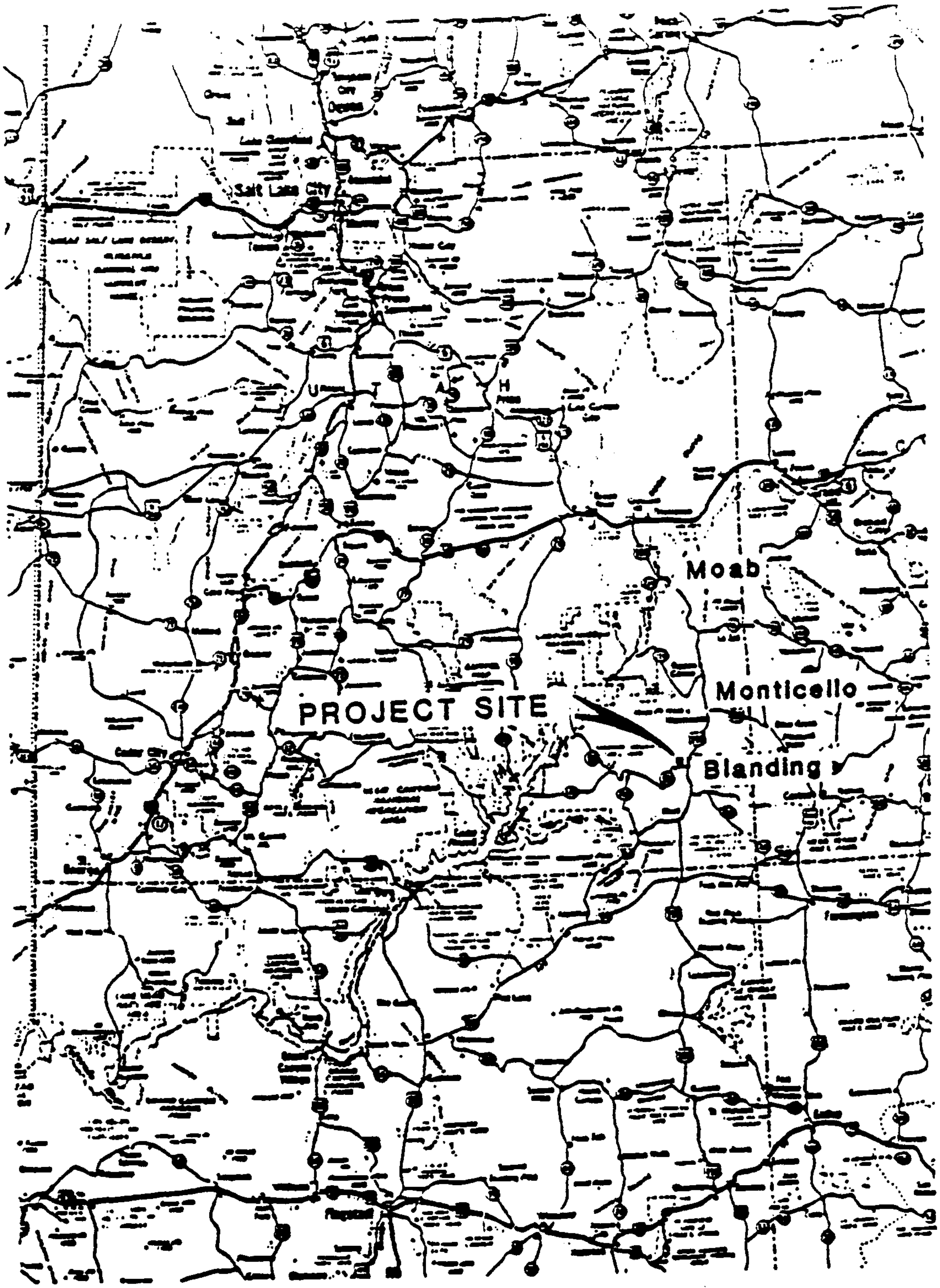
A brief description of the geology, subsurface conditions and groundwater is given in Section 2.0 Site Characteristics.

Additional information on site characteristics is given in Appendix A. Section 1.0 "Regional Geology" and Section 2.0 "Blanding Site Geology" were reproduced from the Environmental Report (Dames & Moore, 1978b) with minor changes in syntax. Section 1.3 "Seismic Risk Assessment" is a new study prepared for this report by Dr. James B. Johnson, Professor of Geology, Mesa College, Grand Junction, Colorado. Section 2.0 "Surface Water" and Section 3.0 "Groundwater" were reproduced from the Final Environmental Statement (US NRC 1979) with minor changes in syntax. Recent information on groundwater (Section 3.3) and archeological sites (Section 4.1) is included in Appendix A.

The design of Cell 4 is presented in Section 3.0. Design Criteria (Section 3.1), Site Preparation (Section 3.2), Embankment (Section 3.3), Stability Considerations (Section 3.4), and Impoundment Features including the synthetic lining and leak detection and slimes drain systems are given in Section 3.5. Construction is covered in Section 4.0. The Plans and Specifications are given in Attachment I and the Quality Plan for Quality Control/Quality Assurance is given in Attachment II.

Cell 4 operations are presented in Section 5.0. Tailings management (Section 5.1), health physics (Section 5.2) and monitoring (Section 5.3) are described.

Cell 4 closure is covered in Section 6.0.



0 20 40 60 80 100

SCALE IN FEET

UMETCO MINERALS CORPORATION
WHITE MESA PROJECT

VICINITY MAP

JUNE, 1988 FIGURE 1.1-1

2.0 SITE CHARACTERISTICS

2.1 Geology

The original geological studies of the area (Dames & Moore, 1978b) have been reviewed by Dr. James Johnson of Mesa College. In addition, Dr. Johnson updated the seismic history of the region, assessed the relationship of earthquake to tectonic structures and performed a site seismic risk evaluation. The original geological studies and Dr. Johnson's recent work are presented in Appendix A.

2.2 Subsurface Conditions

Extensive geotechnical investigations have been conducted in the area of the tailings management cells and surrounding area by various consulting firms as follows:

Dames & Moore	29 Borings	January 1978
Chen & Associates	75 Borings	July 1978
Chen & Associates	53 Borings	January 1979
D'Appolonia	12 Test Pits	May 1981

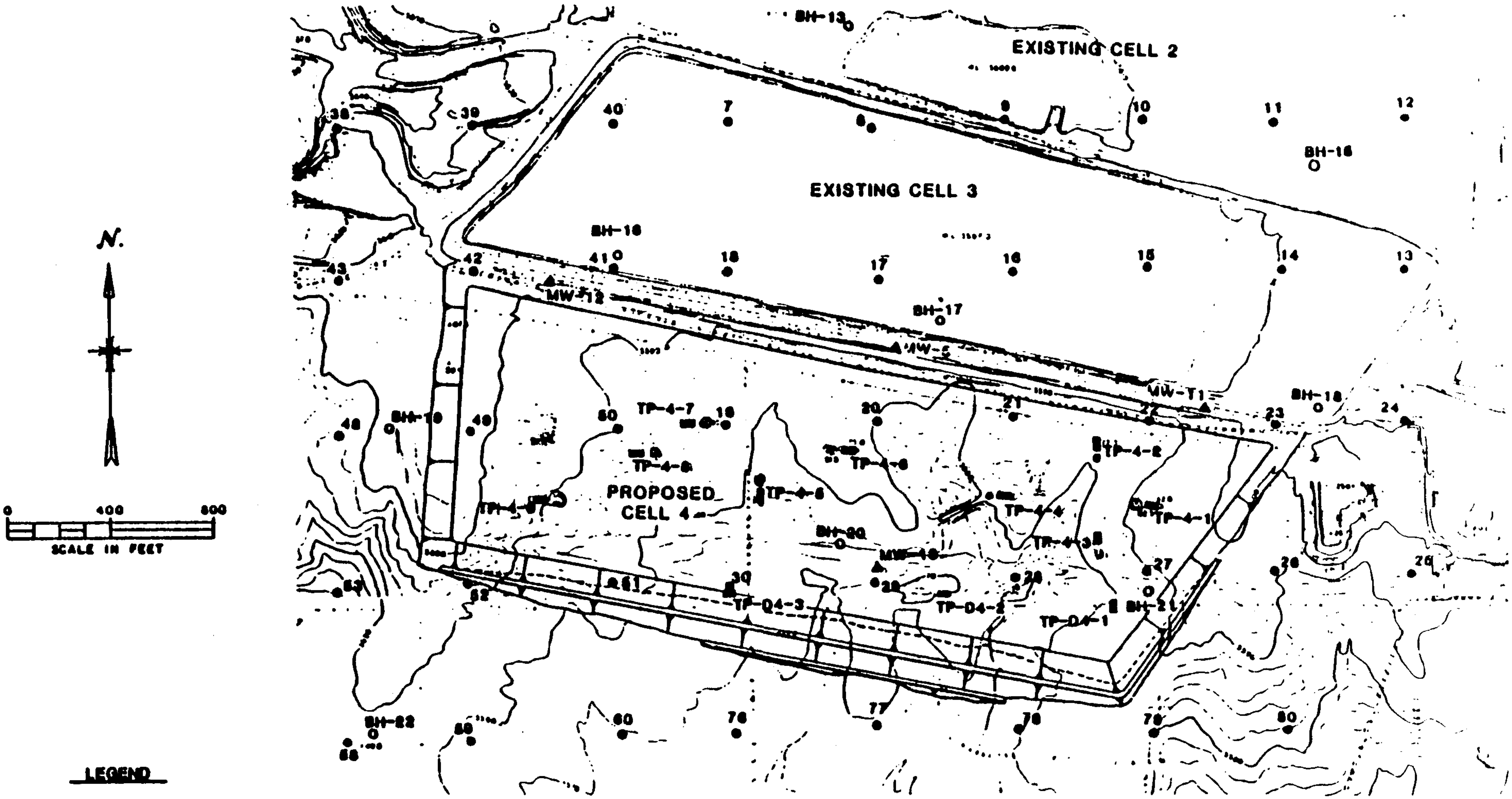
The logs of the borings and test pits are presented in Appendix B of this report with summaries of the geotechnical data. The location of the borings and test pits in the vicinity of Cell 4 are shown on Figure 2.2-1. The subsurface conditions in the Cell 4 area consist of fine silty sand to sandy to clayey silts mixed with silty to sandy clays (eolian and alluvial materials) overlying bedrock. The following paragraphs briefly describe the subsurface conditions within the project area.

2.2.1 Overburden Soils

Two general soil types comprise the overburden soils. They are sand-silt soils and moderately plastic clays and silts which are described as follows:

Sand-Silt Soils

These soils typically contain 26 to 70 percent passing the No. 200 sieve. Atterberg Limits performed on the silty materials in the vicinity of Cell 4 indicate liquid limit values of 19 to 28 with plasticity index values ranging from 2 to 6. The sands are slightly silty to silty and are non-plastic. A sample of the silty soil from test boring 19 (Chen and Associates, 1978) at a depth of 0 to 3 feet indicated a permeability of 3.4×10^{-8} cm/sec (0.035 feet/year) when compacted to approximately 94 percent of maximum Standard Proctor Dry Density (ASTM D-698). A sample of silty sand to sandy silts from boring 19 (Dames & Moore, 1978a) at a depth of 4 feet has a remolded permeability of 6.8×10^{-5} cm/sec (70.1 feet/year). This sample was also compacted to approximately 95% of maximum Standard Proctor Dry Density.



LEGEND

▲ MONITORING WELL

● CHEN AND ASSOCIATES DRILL HOLES

○ DAMES AND MOORE DRILL HOLES

▮ TEST PITS D'APPOLONIA

Note: Exterior boundaries shown for clarity. For actual footprint see Sheet C4-1.

UMETCO MINERALS CORPORATION
WHITE MESA PROJECT

EXPLORATORY HOLES PLAN

JUNE, 1988 FIGURE 2.2-1

The materials have only moderate strength characteristics when compacted and tested under saturated conditions (Dames & Moore, 1978a).

These soils are slightly calcareous to highly calcareous with $CaCO_3$ content varying from 1.7 to 29.7 percent (see Table 2.2-1).

Moderately Plastic Clays and Silts

The second broad category of overburden soils in the project area is moderately plastic clays and silts. These soils also include the severely weathered portion of the upper claystone bedrock.

In the project area, these soils were found to have moderate to high liquid limits (26 to 60) with plastiscity indices varying from 9 to 30. A sample of sandy clay from test boring 49 at a depth of 5 to 7 feet had a remolded permeability of 3.2×10^{-7} cm/sec (0.33 feet/year) (Chen and Associates, 1978). This sample compacted to 95% of maximum Standard Proctor Dry Density represents the upper limit of permeability expected for this type of material.

A sample collected from Test Pit 4-1 indicated the weathered claystone at a depth of 10 to 10.5 feet to have a $CaCO_3$ content of 9.5 percent (see Table 2.2-1) (D'Appolonia, 1981).

2.2.2 Bedrock

The depth to bedrock is variable and ranges from approximately 3.5 to 13 feet below the existing ground surface.

The bedrock found in the area of Cell 4 is predominantly sandstone of the Dakota Sandstone Formation. However, claystones and claystone sandstones lenses and layers were found dispersed intermittently within the sandstones. The sandstones are soft to moderately hard at the surface and are only slightly cemented. The cementation with corresponding hardness generally increases with depth but poorly cemented zones occur within the Formation. The upper portion of the claystone bedrock is highly weathered and has the characteristics of a plastic clay.

Two sets of joints were found exposed in the bedrock in the cliffs west of the site. The joint sets range from N10°-18°E to N60°-85°E. The joint sets approximately parallel the cliff face.

Losses of drilling fluids occurred randomly in some of the drill holes indicating that permeable zones exist within the Bedrock Formation.

TABLE 2.2-1

Calcium Carbonate Content of Soils

Test Pit No.	Depth (Feet)	Soil or Bedrock Type	% CaCO3
4-1	5 - 5.5	Sandy Silt	3.8
4-1	10 - 10.5	Weathered Claystone	9.5
4-2	2 - 5	Silty Sand	2.3
4-3	2 - 3.3	Silty Sand	21.6
4-4	3 - 3.5	Silty Sand	1.7
4-5	3 - 5	Silty Sand	2.9
4-6	4 - 4.5	Silty Sand	3.0
4-7	3.5 - 6	Silty Sand	29.7
4-8	3 - 6	Silty Sand	7.2
4-9	4 - 8	Silty Sand	24.0

Note: Data from D'Appolonia (1981).

Twenty-five in-situ packer tests performed by Dames & Moore (1978a) in the bedrock formation shows a permeability range of 0 to 79 feet per year with an average value of about 6 feet per year. Two tests showed extremely high permeability values. One packer test in boring B-3 at a depth of 30 feet showed a permeability value of 568 feet per year. The other test (boring B-19 at a depth of 45 feet) showed a permeability value of 943 feet per year. The former value is probably the result of near surface weathering while the latter value shows the results of secondary permeability (effects of joints) within the formation.

2.2.3 Groundwater

The movement of groundwater that occurs at shallow depths in the Dakota sandstone and Burro Canyon Formation at the mill site is believed to be perched water and confined to isolated zones within White Mesa. Water quality data and phreatic elevations tend to support this hypothesis.

Beneath the shallow isolated water-bearing zone is the Brushy Basin Member of the Morrison Formation which is an aquiclude. There are also locally impermeable lenses in the base of the Burro Canyon Formation.

Borings and monitor wells indicate that the depth to the true groundwater table varies between 70 and 110 feet below Cell 4.

The gradient is to the south southwest. Based on a gradient of one foot per 100 feet, the shallow groundwater movement below the tailings site was calculated to be approximately 0.0025 to 0.01 foot (0.08 to 0.3 cm) per year toward the south-southwest (Dames & Moore 1978b, Section 2.6.1.3). The indicated gradient under the tailings cells along this line based on current data is 1.6 feet per 100 feet. Additional discussion of groundwater is given in Section 3.0 in Appendix A.

Perched water conditions may exist locally at various depths due to lenses and layers of claystones present within the sandstones. However, these perched water conditions should not affect construction of Cell 4.

3.0 DESIGN

3.1 Design Criteria

Cell 4 has been designed in general conformance to the other containment structures comprising the White Mesa Tailings Management System. The following documents supplied the criteria and basis for design.

- Environmental Protection Agency, 40 CFR Part 61, National Emission Standards for Hazardous Air Pollutants; Standards for Radon-222 Emissions from Licensed Uranium Mill Tailings.

- United States Nuclear Regulatory Commission Rules and Regulations, Title 10, Chapter 1; Part 40, Code of Federal Regulations

- Regulatory Guide 3.11, Design, Construction, and Inspection of Embankment Retention Systems for Uranium Mills, Revision 2, 1977

In addition, guidelines and concepts established in the "Engineers Report, Tailings Management System" prepared by D'Appolonia (Report No. RM78-682) dated June 1979 were utilized in the Cell 4 design.

3.2 Site Preparation

The foundation for the embankment will consist of the existing natural soils. Topsoil, vegetative matter, etc., will be removed from the entire embankment footprint prior to construction. The exposed surface will then be inspected to detect presence of any soft, loose areas, sand lenses or highly calcareous soils. If found, these materials will be removed and replaced with properly compacted soils approved by the Engineer.

The entire embankment footprint will then be scarified to a depth of 8 inches adjusted for moisture and compacted to at least 95 percent of maximum Standard Proctor Dry Density (ASTM D-698) within ± 2 percent of Optimum Moisture Content.

3.2.1 Impoundment Area

The excavations for the impoundment will be placed on a 3(H) to 1(V) slope. Excavations will be made by conventional earthmoving equipment where possible. Areas that cannot be removed in this manner will be blasted using low velocity explosives. Blasting will only be performed when the Engineer is present to inspect the resulting excavation. Overbreak areas will be removed of loose material and replaced with compacted siltly soils.

The bottom of the impoundment will be prepared by removing topsoil and vegetative matter from the subgrade. The exposed surface will be constructed to the lines and grades shown on the plans. Proof-rolling with heavy equipment will be performed over the entire bottom to detect soft or loose areas. These areas, if found, will be removed to depths as determined by the Engineer and replaced with ML or CL soils compacted to at least 95 percent of maximum Standard Proctor Dry Density (ASTM D-698) within $\pm$ 2 percent of optimum moisture content. Any fill required to achieve the desired grade within the impoundment will also be compacted to this criteria.

3.3 Embankment

The embankment will be built using on-site soils or soils obtained from the nearby borrow sources. Materials for use in the embankment will classify as CL or ML soils under the Unified Soil Classification System.

The side slopes of the embankment will be 3(H) to 1(V) and the top width will be a minimum of 18 feet. Where the embankment exceeds a height of 25 feet, a stablization berm will be constructed as shown on Drawing C4-2.

The embankment (see Plans and Specifications, Attachment I) will be constructed in 12-inch loose layers and compacted to at least 95 percent of maximum Standard Proctor Dry Density (ASTM D-698) within $\pm$ 2 percent of optimum moisture content.

Highly calcareous soils (containing visible white material) will not be used in the construction of the embankments.

3.4 Stability Considerations

3.4.1 Embankment Stability Analysis

The dike stability analysis was performed using the "STABR" computer model developed by Guy LeFebre at the University of California, Berkely.

The program calculates factors of safety by the Ordinary Method of Slices and Bishop's Modified Method for circular slip surfaces tangent to any horizontal level or through a specified point. The program can also search for the critical circular slip surface with the minimum factor of safety. Seismic forces can be introduced to pseudostatically analyze the effects of the earthquake ground motions.

For this analysis, Bishop's Modified Method was used and three horizontal tangent levels were examined. The automatic search feature was also used to determine the critical circular slip surface.

A factor of safety of 1.0 indicates that the slope would be at incipient failure. For this analysis a factor of safety of 1.5 or more under static conditions indicates an acceptable level of stability along the analyzed circular surface.

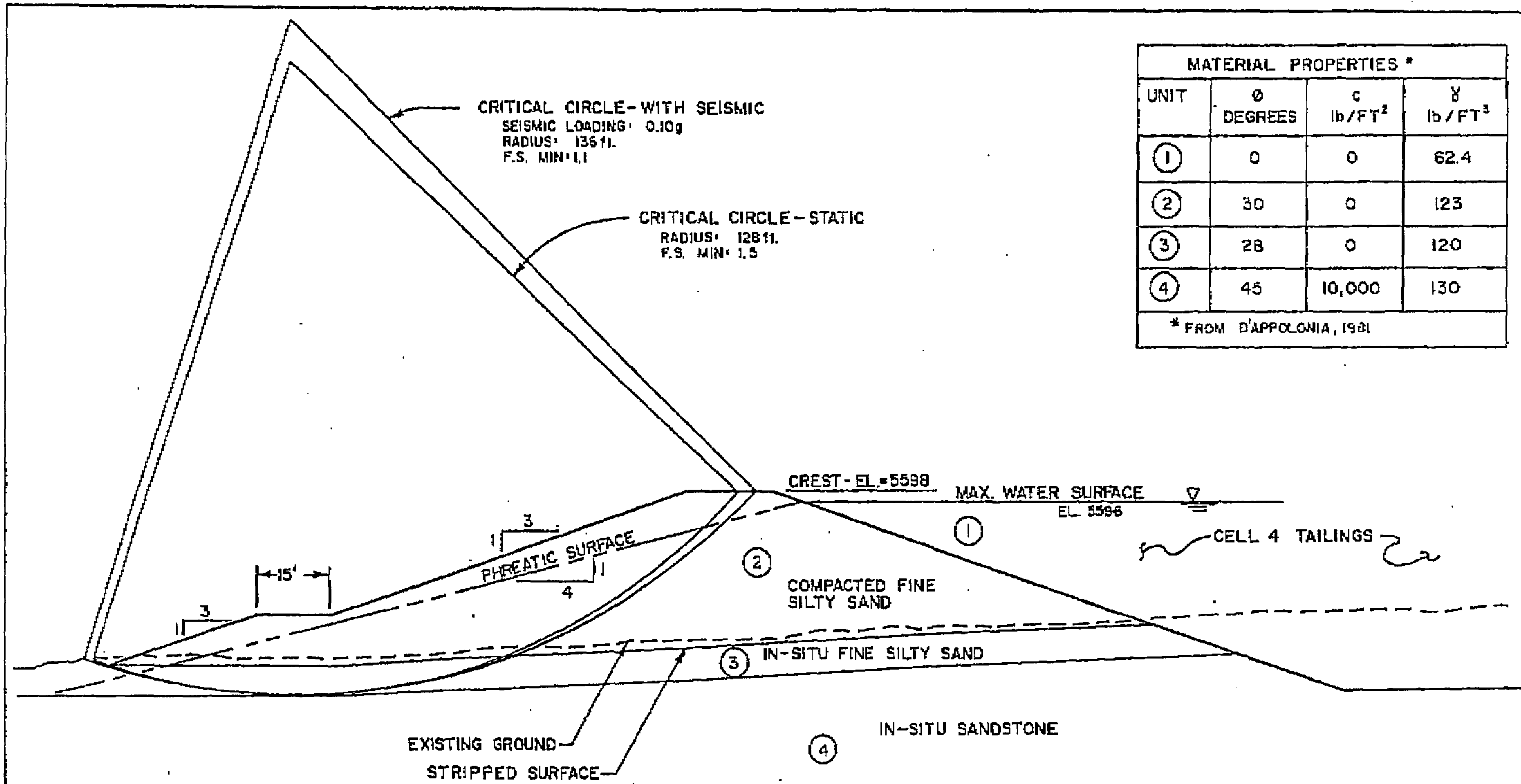
The acceptable factor of safety under seismic loading conditions (maximum credible earthquake 0.10 g) is 1.0. The maximum seismic loading of 0.10g was assumed to occur at the center of the slice.

The stability analysis assumes that the tailings are saturated and are completely fluid. It also assumes that the cell liner has completely failed and that the steady state seepage condition has been reached. Under actual operating conditions it is highly improbable that these conditions could exist. Therefore, the stability analysis produces results that are considered to be extremely conservative.

Two embankment sections were analyzed for static and dynamic stability. Material properties used in the analyses are shown in Figure 3.4-1 and are those used previously by D'Appolonia in their analysis of Cell 3 Embankments (D'Appolonia, 1981).

Figure 3.4-1 is a section through the highest portion of the Cell 4 dike. The maximum height is 31 feet. A 15-foot wide bench has been added on the downstream side of the dike to improve the stability. Using this embankment configuration in the analysis, the minimum factor of safety under static conditions is 1.5. Applying a seismic loading of 0.10g to simulate dynamic conditions, the analysis produces a minimum pseudostatic factor of safety of 1.1.

Figure 3.4-2 shows a dike section with a maximum height above the stripped surface (prepared subgrade) of 25 feet. The dike has 3(H) to 1(V) faces and an 18-foot crest width. The results of the analyses indicate a minimum static factor of safety of 1.5 and a minimum pseudostatic safety factor of 1.1 for a 0.10g lateral loading.



MATERIAL PROPERTIES *			
UNIT	Ø DEGREES	c lb/FT²	γ lb/FT³
(1)	0	0	62.4
(2)	30	0	123
(3)	28	0	120
(4)	45	10,000	130

* FROM D'APPOLONIA, 1981

NOTES:

1. ANALYSIS ASSUMES:
   a. COMPLETELY SATURATED TAILINGS (FLUID)
   b. COMPLETELY FAILED LINER
   c. STEADY STATE SEEPAGE CONDITIONS
2. ANALYSIS BASED ON BISHOPS MODIFIED METHOD FOR CIRCULAR SLIP SURFACES.
3. SEISMIC FORCE IS APPLIED AT CENTER OF SLICE

CELL 4 DIKE

MAXIMUM HEIGHT SECTION

SCALE: 1" = 20'

UMETCO MINERALS CORPORATION

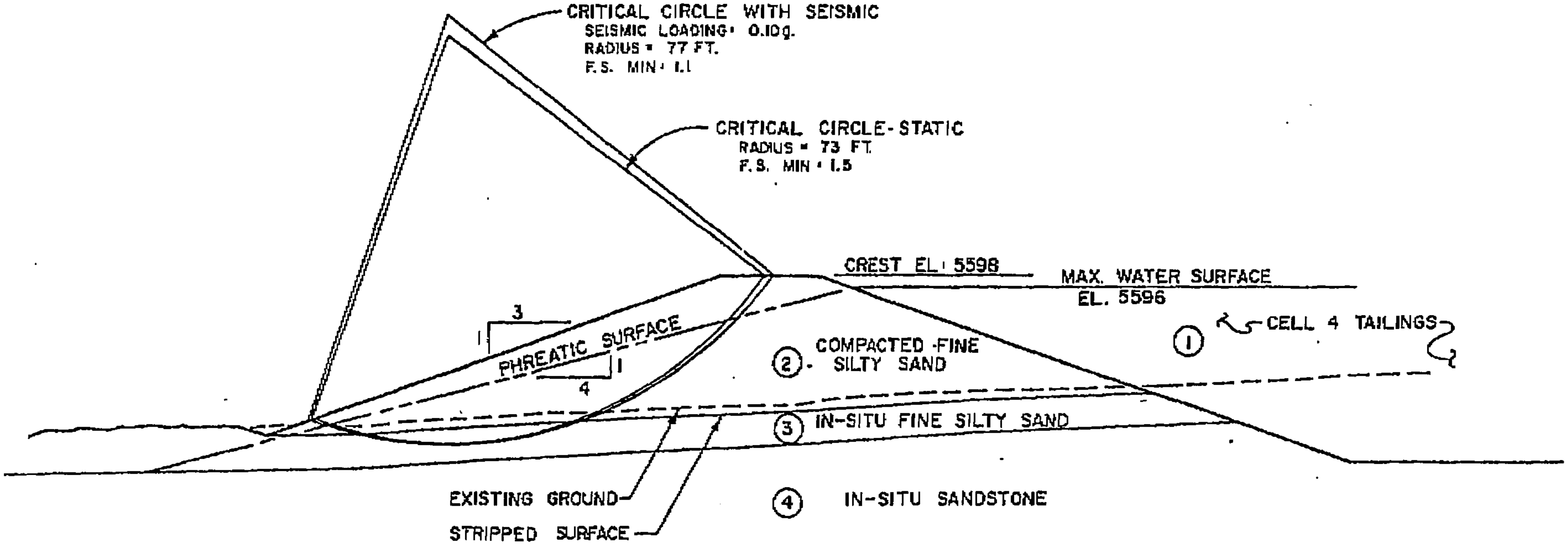
WHITE MESA PROJECT

STABILITY ANALYSIS

- CELL 4 -

MAXIMUM HEIGHT SECTION

JUNE, 1988 FIGURE 3.4-1



NOTE:
REFER TO FIGURE 3.4-1
FOR MATERIAL PROPERTIES
AND OTHER NOTES

CELL 4 DIKE
25-FT. SECTION
SCALE: 1" = 20'

UMETCO MINERALS CORPORATION
WHITE MESA PROJECT
STABILITY ANALYSIS
- CELL 4 -
25-FOOT HIGH SECTION

JUNE, 1988 FIGURE 3.4-2

The results indicate the Cell 4 embankment to be stable during operating conditions even under loadings induced by the Maximum Credible Earthquake (MCE).

3.4.2 Foundation Settlement

Settlement analysis indicates the foundation soils will settle on the order of one to two inches due to the weight of the embankment. The majority of this settlement is expected to occur during construction.

3.4.3 Embankment Settlement

The embankment will settle under self-weight loading and due to compaction. It is anticipated that embankment settlement will be on the order of two inches and that approximately 90 percent of this settlement will occur during construction.

3.4.4 Liquefaction Potential

The potential for foundation or embankment materials to experience liquefaction under the long-term predicted seismic loadings is nil. Tailings placed within the Cell could liquefy as a result of severe seismic shock. This condition is similar to the state of the tailings immediately after deposition from the slurry line. The disposal cell is an engineered earth structure and is designed to contain any liquefied tailings.

3.4.5 Erosion Potential

The downstream face of the embankment will be placed on a 3(H) to 1(V) slope. Erosion protection will be maintained by active intervention as needed. Daily inspections are conducted on the Tailings Management System. Any signs of erosion will be reported on the daily inspection form and any deteriorated areas will be repaired as required.

3.5 Impoundment Features

3.5.1 Leak Detection System

A leak detection (underdrain) system will be installed beneath the synthetic liner and will consist of 4-inch diameter perforated HDPE or PVC pipe in the configuration shown on Drawing Nos. C4-4 and C4-5. The pipe will be embedded in granular material (crushed sandstone) in trenches in the clayey base with a nominal depth of 12 inches and with a 40 mil HDPE or 30 mil PVC lining. The openings of the drainage pipe will be compatible with the size of the granular drainage material. Filter criteria will be used to size the openings. The drain will connect to a 12-inch diameter HDPE or PVC access pipe as shown Drawing Nos. C4-4 and C4-5.

3.5.2 Lining System

A synthetic lining system will be constructed on the sides and bottom of the impoundment. The lining will consist either of 30 mil PVC (the same lining as Cells 1, 2, and 3) or 40 mil HDPE.

The bottom of Cell 4 will have a nominal 12-inch thick clayey base. The sub-base of the side slopes of the impoundment will be rolled to provide a smooth surface prior to construction of the liner.

If a PVC lining is constructed, a 12-inch soil layer will be placed over the liner for protection against ultraviolet light and operational traffic. If HDPE lining is used, no soil cover will be provided prior to deposition of tailings. However, earth covering and protective earth ramps will be installed as needed for operations.

3.5.3 Slimes Drain System

A slimes drain (overdrain) will be installed on top of the synthetic liner as shown on Drawing Nos. C4-4 and C4-5. This drain will consist of a series of HDPE or PVC pipes embedded in granular drainage material (crushed sandstone). The drain will connect to a 12-inch HDPE or PVC access pipe at the location shown on the plans.

4.0 CONSTRUCTION

4.1 Construction Material Sources

It is anticipated that all materials will be obtained from on-site. Soils classifying as CL-ML materials are available from within the impoundment area, as well as sandstone to be processed for drainage material. In addition, stockpiled materials from construction of the previous cells are available, if needed. It is expected the construction will generate approximately 800,000 excess cubic yards of random fill and 1,000,000 cubic yards of rock which can be used in the reclamation of the facility.

4.2 Plans and Specifications

Plans and Specifications for the construction of Cell 4 have been prepared for this project and are presented in Attachement I.

4.3 Quality Control/Quality Assurance

4.3.1 General

The "Quality Plan for Construction Activities - White Mesa Project, Blanding, Utah" (referred to as the Quality Plan), given in Attachment II, describes how the Construction Quality Control/Quality Assurance (QC/QA) activities are implemented.

This Quality Plan includes the following:

1. QC/QA objectives, definitions, methodology and activities.

2. Organizational structure including duties and qualifications of personnel and information flow.

3. Surveys, inspections, sampling and testing including methods and procedures, frequency and type and qualifications of personnel.

4. Changes and corrective actions includes authority of personnel and methodology.

5. Documentation requirements, including personnel and forms.

4.3.2 Quality Control

Quality Control ensures that the construction proceeds in accordance with the Plans and Specifications approved by the Lead Agency and shall be provided by the Quality Control Officer or his designated representative.

4.3.3 Quality Assurance

Quality Assurance consisting of audits and cross checks on the Quality Control functions shall be provided by the Quality Assurance Officer or his designaged representative.

The Quality Assurance Office shall implement the program by performing the following functions:

1. Prequalify Quality Control personnel.

2. Quality Control Technician training.

3. Verifications of effectiveness of Quality Control personnel.

4. Verification of test procedures and results by spot retest.

5. Equipment checks.

6. Review of calculations and documentation.

5.0 CELL 4 OPERATIONS

5.1 Tailings Management

Cell 4 will follow the "final grade" method that is currently being used in Cell 3. During initial operations, tailings will be discharged into the bottom of the cell in a manner that will maintain liner integrity, either by discharging on the earthen work pads or on other suitable materials. Discharge points around the periphery of the cell will be used as necessary so that complete coverage of the cell bottom is achieved during this initial operational phase. This layer will be saturated or remain covered by liquids.

Tailings will then be discharged in the east end of Cell 4 until the tailings surface is at final grade. The final grade surface will then be advanced towards the middle downstream corner of the cell. Discharge points will be likewise employed in other areas of the cell, working the final grade surface towards the slimes pool area. Cell 4 will be operated with sufficient freeboard to contain the PMP event.

The final grade method allows the maximum amount of solution to drain from the sands, thus stabilizing the sands in the shortest amount of time, As the sand surface at final elevation stabilizes, the first layer of the final cover will be applied. This will limit exposed tailings surfaces as per ALARA principles.

When the cell is full, the slimes drain system will be activated and solutions will be pumped to Cell 1-I for evaporation. The dewatered Cell 4 will then be subject to final reclamation activities.

During the operational life of the cell, solutions will be recycled via the pond return pumping system. This sytem consists of a floating pump barge and pumps. Pond solutions will be pumped back to the mill for maximum practical use. At the end of operations for Cell 4, tailings will be placed to top of the synthetic liner. Provisions will be made to handle the PMP event.

Cell 4 will be covered by the operational procedures, inspection programs, and ALARA activities as specified by the Source Material License.

5.2 Health Physics

Personnel (Umetco and contractors) will be monitored and protected during the Cell 4 construction work as follows:

1. Badges - All personnel working at the construction site whether directly concerned with this project or not, will be provided with and required to wear TLD badges. Badge recordings and filings will be in accordance with 10 CRF 20.101 and 10 CFR 20.202(a). If records show any personnel exposed in excess of 25 percent of CFR 20 limits, they will be reassigned to work in low exposure areas and conditions causing the exposures to exceed the 25 percent level will be investigated.

2. Programs and Procedures - The programs and procedures for the monitoring and protection of persons will be in accordance with License Conditions 11, 36 through 40. At the end of shift or before leavidng the restricted area, persons working in the area will be required to monitor themselves with an alpha survey meter. If alpha contamination on skin or clothes exceeds 1000 dpm/100 cm^2, decontamination by washing, showering or vacuum cleaning will be carried out before leaving the restricted area.

5.3 Monitoring

The monitoring program required by the Source Material License will be followed with any changes resulting from the addition of Cell 4.

6.0 CELL 4 CLOSURE

In addition to requiring an impermeable liner for protection against liquid migration, compliance with EPA standards (U.S. NRC 1985) requires the following:

- Free liquids must be removed from the tailings.
- The pile must be stabilized to a bearing capacity sufficient to support a final cover.
- The surface impoundment must be covered with a final cover which will:
  - Provide long-term minimization of migration of liquids through the impoundment.
  - Function with minimum maintenance.
  - Promote drainage and minimize erosion or abrasion of the final cover.
  - Accommodate settling and subsidence.
  - Have a permeability less than or equal to the permeability of any bottom liner system or natural subsoil present.

Cell 4 will be closed and reclaimed upon cessation of operations to meet the above requirements. A Reclamation Plan approved by the U. S. Nuclear Regulatory Commission will be implemented.

7.0 REFERENCES

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Dames and Moore, 1978a. Site Selection and Design Study - Tailing Retention and Mill Facilities, White Mesa Uranium Project, January 17, 1978.

Dames and Moore, 1978b. Environmental Report, White Mesa Uranium Project, San Juan County, Utah, January 20, 1978, revised May 15, 1978.

D'Appolonia Consulting Engineers, Inc., 1979. Engineers Report, Tailings Management System, White Mesa Uranium Project, Blanding, Utah.

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U. S. Nuclear Regulatory Commission, 1977. Regulatory Guide 3.11, Design, Construction, and Inspection of Embankment Retention Systems for Uranium Mills, Revision 2, 1977.

U. S. Nuclear Regulatory Commission, 1979. Final Environmental Statement - White Mesa Uranium Project, NUREG-0556.

U. S. Nuclear Regulatory Commission, 1985. Standard Review Plan for UMTRA Title I Mill Tailings - Remedial Action Plans, Division of Waste Management.

U. S. Nuclear Regulatory Commission, 1987. 10 CFR 10, Appendix A.

ATTACHMENT I

SPECIFICATION NO. C4-1

PLANS AND SPECIFICATIONS

FOR

CELL 4 CONSTRUCTION

TAILINGS MANAGEMENT SYSTEM

WHITE MESA PROJECT
BLANDING, UTAH

PREPARED BY

UMETCO MINERALS CORPORATION
GRAND JUNCTION COLORADO

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	(signature)

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Page No.

1.0 INTENT OF SPECIFICATIONS

The Specifications presented in this document cover the construction of the Cell 4 tailings impoundment, including the embankment, leak detection system, the synthetic liner and the slimes drain system for the White Mesa Project Tailings Management System, Blanding, Utah.

2.0 ENVIRONMENTAL QUALITY PROTECTION

2.1 General

The Work shall be carried out in compliance with applicable statutes, rules and regulations, licenses, and permits.

2.2 Land

During construction, care shall be exercised to preserve the natural landscape and to prevent any unnecessary destruction, scarring or defacing of the natural surroundings in the vicinity of the Work. Temporary diversion ditches, berms, silt fences, etc., may be required to prevent erosion and sediment transport from the construction area.

2.3 Water

Construction activities shall be performed by methods that will prevent entrance or accidental spillage of pollutants into nearby gullies, washes and underground water sources.

2.4 Air

Reasonable and practical efforts shall be made to operate construction equipment to minimize emissions of air contaminants.

Fugitive dust from unpaved haul roads and other areas of heavy vehicle use shall be controlled by sprinkling, dust suppression agents, or by vehicle speed limits. If due to unusual circumstances, sprinkling and/or dust suppression agents are not fully effective in controlling excessive fugitive dust emission, vehicle speeds on unpaved haul roads shall be limited to 20 mph.

Storage and handling of flammable and combustible liquids and provisions for fire prevention shall be in accordance with local and State regulations.

2.5 Archeological Clearance

The archeological sites in the project area are shown on Drawing B-1 in Appendix A. If any construction activity is going to impact within 100 feet of any of these sites, the following steps will be taken:

1. For an undetermined archeological site, a determination will be made as to whether the site is contributing.

2. Contributing sites will be excavated in accordance with License Condition 15.

3.0 SITE PREPARATION

3.1 Clearing, Grubbing and Stripping

The area of Cell 4 shall be cleared of all existing vegetation and the root systems removed by grubbing.

The entire area to be occupied by Cell 4 shall be stripped to a sufficient depth to remove all unsuitable materials. The unsuitable materials to be removed by stripping shall include all debris, topsoil, and vegetative matter, including stumps and roots. All perishable and objectionable materials that are unsuitable for use in permanent construction, or that might interfere with the proper bonding of the embankment with the foundation or the proper compaction of the embankment, shall be removed. Any boulders or large rocks which interfere with achieving the desired rough grade or with placement of fill materials shall be removed.

Materials from clearing, grubbing and stripping shall be separated into topsoil, combustible matter including stumps, roots, etc., and rubble including boulders, rock, etc. The Engineer shall determine locations for stockpiling and/or the disposition of these materials.

3.2 Topsoil

Topsoil is defined as that material having a significant organic content which will readily support vegetation and is approximately 12 inches thick at this site. Previous areas stripped only need to be removed of vegetation. All topsoil in the area of Work shall be stripped to its full depth, where appropriate, and stockpiled. Topsoil shall be reused in reclamation work.

4.0 EARTHWORK

4.1 General

The required earthwork for Cell 4 consists of excavation, preparation of the Cell 4 bottom, preparation of the foundations for the embankment, and placement of the earthfill for the embankments. The earthwork shall be performed as shown on Drawing Nos. C4-1 and C4-2 in accordance with the Specifications.

4.2 Excavation

4.2.1 General

Excavation for Cell 4 shall be made to the lines, grade and dimensions shown on Drawing Nos. C4-1 and C4-2. The alignments and excavation lines on the drawings are subject to change as may be found necessary to adapt the foundation for the embankments to the conditions disclosed by the excavation and as approved by the Engineer. Accurate trimming of the slopes of the excavation for the embankments will not be required. The excavation shall conform as closely as practical to the established lines and grades. The finished foundation contours against which the embankments are to be placed shall be cleaned of all loose, soft and disintegrated materials including removal of all such materials from pockets, and depressions of the foundation. Sharp irregularities shall be reduced to provide satisfactory foundation contours.

All necessary precautions shall be taken to preserve the material below and beyond the lines of all excavations in the soundest possible condition. Where required to complete the Work, all excess excavation and overexcavation shall be refilled with suitable materials acceptable to the Engineer as specified herein.

Rock excavation shall be achieved by mechanical means unless blasting of hard lenses or areas of rock is approved by the Engineer. A detailed blasting plan must be submitted to the Engineer prior to issuance of approval for blasting. Blasting will be permitted only after adequate provision has been made for the protection of persons, the work, and public and private property. Furthermore, all blasting shall be conducted in accordance with applicable Federal, State, and local laws and regulations.

The excavated surface in areas to be covered by a synthetic lining shall be free from all loose earth and rock fragments over 6 inches in size, roots, vegetation, or other foreign material. The excavated surface shall also be free from sharp breaks in slope and shall be fairly smooth with no pieces or fragments protruding more than 4 inches from the general plane of excavation.

4.2.2 Excavated Materials

Materials from excavation shall be separated as far as practicable into materials that meet the specifications for fills, sandstone for crushing, synthetic liner cover, and reclamation cover and into unsuitable materials.

Excavated material which will be suitable for dike construction or other fills when dry, shall be taken from the excavation, dried, and then placed in the fill area. Material excavated as rock which breaks down rapidly so it does not meet the requirements for rockfill material shall be considered soil and stockpiled accordingly. Materials for lining bedding or cover may be temporarily placed in approved stockpile areas during excavation and later removed for use as required.

Materials to be used as reclamation cover shall be placed in stockpile areas designated by the Engineer.

Where practical, suitable materials shall be excavated separately from unsuitable materials. All materials removed from all excavations which are considered unsuitable shall be disposed of as follows.

Excavated materials shall be considered unsuitable for use in fills or as synthetic lining bedding or cover if they have expansive properties, are highly calcareous (contain visible white material), or have other unsuitable properties. Materials with these properties may be mixed with other material and used as suitable material only with the approval of the Engineer. Unsuitable materials not mixed shall be stockpiled separately from suitable materials. These materials shall be considered suitable for use as reclamation cover.

Excavated materials containing rubbish or other foreign material shall be considered unsuitable for any use and shall be disposed of as directed by the Engineer.

4.3 Preparation of Foundation

After the topsoil has been removed, the exposed surface shall be proof-rolled with heavily loaded pneumatic-tired equipment. Any soils found to be deflecting excessively under the moving wheel loads shall be removed.

Prior to the placement of any fill, the stripped areas shall be inspected by the Engineer for wet materials, soft spots, small local zones or pockets of soft silts or clays, or other unsuitable materials. Areas of unsuitable materials shall be overexcavated. The determination of unsuitable materials shall be made by the Engineer.

All cavities, depressions and irregularities found within the area to be covered by the embankments and within Cell 4 and which extend below or beyond the established lines of excavation, shall be filled with embankment materials compacted as specified for the embankment.

The foundation for the earthfill shall be prepared by leveling, moistening, scarifying, and compacting so that the surface materials of the foundation will be stable and will provide a satisfactory bonding surface with the first layer of earthfill as specified for the subsequent layers of earthfill.

Where fill is to be placed on natural slopes steeper than one vertical to seven horizontal, the existing slope shall be benched prior to placing fill. The width of any bench should not be greater than 25 feet or less than 10 feet. The width of each bench should be maintained within the specified limits, and the height of the cut face varied in accordance with the slope of the natural ground surface. The height of cut at the face should not exceed 5 feet. The slope of the temporary cut face should be no steeper than one vertical to one horizontal. All benches should be sloped at a minimum of one percent away from the cut face to maintain proper drainage.

The fills shall be constructed to the lines, grades, and cross sections indicated on Drawing Nos. C4-1 and C4-2.

The surface in areas on the bottom of Cell 4 to be covered by a clayey base and synthetic lining shall be free from loose earth, ruts, sharp breaks in slope, roots, vegetation or other foreign material, and all cobbles or rock fragments protruding from the final smooth surface.

All areas that do not met these requirements shall be corrected to the satisfaction of the Engineer before construction of the leak detection system (Section 5.0).

4.4 Embankment

4.4.1 General

The embankment shall be constructed to the lines, grades and cross sections indicated on Drawing Nos. C4-1 and C4-2.

No brush, roots, sod or other perishable, unsuitable materials or frozen soils shall be placed in the embankments. The embankments shall be maintained, in an approved manner, including maintaining surfaces free of weeds or other vegetation, until final completion of the Work.

4.4.2 Materials

The earthfill embankment materials shall consist of silt and clay earth materials excavated from the construction area or imported from the Borrow Area. Cobbles and rock fragments having maximum dimensions of more than 12 inches shall not be placed in the earthfill. Should cobbles or rock fragments of such a size be found in otherwise acceptable earthfill materials, they shall be removed and be placed on the rubble stockpile or may be placed on the downstream faces of the embankments.

All materials for use in embankment construction shall be approved by the Quality Control Officer or his designated representative.

4.4.3 Placement and Compaction

No earthfill shall be placed until the preparation of the foundation has been approved by the Quality Control Officer or his designated representative. The embankments shall be constructed as shown on Drawing Nos. C4-1 and C4-2 in accordance with the Specifications.

The distribution and gradation of the materials throughout the earthfill shall be such that the fill shall as far as practicable be free of lenses, pockets, streaks or layers of material differing substantially in texture, gradation, or moisture content from the surrounding material. Successive loads of material shall be dumped on the earthfill so as to produce the best practical distribution of material.

Particles larger than 6 inches, but less than 12 inches in maximum dimensions, shall be worked into the fill in such a manner as will disintegrate friable material and orient and distribute resistant particles to effect a compacted well-knit mass with spaces between larger particles thoroughly choked with compacted finer materials. To aid in accomplishing this, material containing more than 20 percent (by volume) of particles exceeding 6 inches in maximum dimensions, shall be spread in lifts not exceeding 12 inches in thickness (loose measure), and tracked with at least four passes of the treads of a crawler type tractor which, by means of sufficient overlap, will assure complete coverage of an entire layer by the tractor treads. Second and subsequent passes of the treads shall not be made until each pass, as defined above, is completed. If the size and content of resistant particles in the fill material precludes proper compaction, the material shall be disposed of or mixed with finer materials before placement.

The materials shall be placed in the earthfill in continuous approximately horizontal layers not more than one foot in loose thickness. The earthfill shall be compacted to at least 95 percent of maximum Standard Proctor Dry Density (ASTM D-698). The embankment shall be constructed continuously and approximately horizontally for the entire width and length of the embankment. Difference in elevation between any cross section shall not be greater than two feet.

Upon placement and compaction of a lift of fill material, the surface shall be scarified to a depth of two inches prior to the placement of the next lift unless the compaction equipment leaves a surface sufficiently roughened to bond the two lifts.

For uniformity, a minimum of three passes of a sheepsfoot or segmented wheel roller in the 20- to 30-ton class shall be required on each lift. If the surface of the prepared foundation or the compacted surface of any layer of earthfill is too dry or smooth to bond properly with the layer of material to be placed thereon, it shall be moistened and/or reworked with a harrow, scarifier, or other suitable equipment to a sufficient depth to provide a satisfactory bonding surface before the next succeeding layer of earthfill is placed. If the compacted surface of any layer of earthfill in place is too wet due to precipitation for proper compaction of the earthfill material to be placed thereon, it shall be removed; allowed to dry; or be reworked with harrow, scarifier, or other suitable equipment to reduce the moisture content to the required level. It shall then be recompacted to the earthfill requirements.

No materials shall be placed in the embankments when either the materials, the foundation, or the embankment on which it is to be placed is frozen or when ambient temperatures do not permit the placement and compaction of the materials to the specified density without developing frost lenses in the fill. The side surface of the embankment which forms the sub-base for the synthetic liner shall be rolled smooth with steel wheel rollers.

4.4.4 Moisture and Density Control

As far as practicable, the materials shall be brought to the proper moisture content before excavation or moisture shall be added to the material by sprinkling on the earthfill, and each layer of the earthfill shall be conditioned so that the moisture content is uniform as far as practicable throughout the layer prior to and during compaction. The moisture content of the compacted earthfill shall be within the limits of 2 percent dry and 2 percent wet of the standard optimum moisture content. Material that is not within the specified limits after compaction shall be rejected and shall be reworked until the moisture content is between the specified limits. Reworking may include removal, reharrowing, reconditioning, rerolling, or combinations of these procedures.

Density control of compacted earthfill shall be such that the compacted material represented by samples having a dry density less than 95 percent of its maximum Standard Proctor Dry Density shall be rejected. Such rejected material shall be reworked as necessary and rerolled until a dry density equal to or greater than 95 percent of its maximum Standard Proctor Dry Density is obtained.

To determine that the moisture content and dry density requirements of the compacted fill are being met, field and laboratory tests shall be made at specified intervals on samples taken from the embankment as specified in Section 4.4.5 Frequency of Quality Control Tests.

4.4.5 Frequency of Quality Control Tests

The frequency of the Quality Control Tests for earthwork shall be as follows:

1. The frequency of the field density and moisture tests shall be not less than one test for each 1,000 cubic yards of placed fill material.

2. There shall be at least one field density and moisture test for each lift of fill material placed.

3. Gradations and Atterberg limits of compacted fill materials shall be performed at a frequency of not less than each 5,000 cubic yards of compacted earthfill.

4. Frequency of laboratory standard Proctor compaction tests shall be such that maximum densities are determined for the range of materials being placed in the fill; however, frequency of compaction tests shall not be less than one test for each 10,000 cubic yards of compacted fill material.

4.5 Clayey Base

4.5.1 General

The clayey base shall be a minimum of 12 inches compacted thickness placed on the bottom of Cell 4. The clayey base shall be placed to the approximate surveyed lines and grades.

4.5.2 Materials

The clayey base shall consist of uncontaminated earth materials excavated from the construction area or imported from the Borrow area. Cobbles and rock fragments having a dimension of more than 6 inches shall be removed from the earthfill. The clayey base material shall have at least 30 percent by dry weight passing the No. 200 sieve. That portion of the clay base material passing the No. 40 sieve shall classify as either a ML, CL or CH under the Unified Soil Classification System.

All materials for use in the clayey base shall be approved by the Quality Control Office or his designated representative.

4.5.3 Placement and Compaction

The foundation shall be approved by the Quality Control Officer or his designated representative prior to placement of the clayey base.

The clayey base shall be placed and compacted to at least 95 percent of standard Proctor maximum density (ASTM D-698). The same requirements/ procedures for placement and compaction (Section 4.4.3) and for moisture and density control (Section 4.4.4), as the embankment earthfill in Section 4.4, shall be used for the placement and compaction of the clayey base except the moisture content of the clayey base shall be within the limits of 1 percent dry and 3 percent wet of the standard optimum moisture content.

The integrity of the clayey base shall be maintained prior to installation of the synthetic liner. This will be accomplished by sprinkling and rolling as required.

4.5.4 Frequency of Quality Control Tests for Clayey Base

The frequency of the Quality Control Tests for earthwork shall be as follows:

1. The frequency of the field density and moisture tests shall be not less than one test for each 500 cubic yards of placed clayey fill material.

2. There shall be at least one field density and moisture test for each list of clayey fill material placed.

3. Gradations and Atterberg limits of compacted clayey fill materials shall be performed at a frequency of not less than each 5,000 cubic yards of placed clayey base.

4. Frequency of laboratory standard Proctor compaction tests shall be such that maximum densities are determined for the range of clayey materials being placed in the fill; however, frequency of compaction tests shall not be less than one test for each 10,000 cubic yards of compacted clayey fill material.

4.6 Work Area Drainage

To protect the surface of the fill, the top of all fill areas shall be crowned and sealed at the end of each working day to minimize the infiltration of water in the event of rainfall.

As interim protection of the cut and fill slopes, adequate surface drainage shall be provided at both the top and bottom of slopes to intercept and conduct runoff from the developed areas and to reduce saturation and erosion of the slopes.

5.0 LEAK DETECTION SYSTEM

5.1 General

A leak detection system (underdrain) shall be constructed below the synthetic liner. This underdrain shall consist of 4-inch perforated pipe embedded in granular materials in trenches excavated into the clayey base and lined with 30 mil HDPE or PVC. Items of work included in this section include providing materials, excavating trenches, installing the HDPE or PVC lining, and placing drain pipes and drainage materials. The leak detection system shall be constructed to the approximate grades and dimensions shown on Drawing Nos. C4-4 and C4-5. The nominal 24-inch width of the drainage trenches shall be adjusted to accommodate the placement of the HDPE liner according to the manufacturer's recommendations.

5.2 Materials

5.2.1 Lining

The leak detection trenches shall be lined with a minimum 40 mil HDPE or 30 mill PVC as shown on Drawing No. C4-3.

5.2.2 Drainage Pipe

Drainage pipe shall consist of Schedule 40 PVC (ASTM D-2241) or HDPE pipe (ASTM D-3034). The pipe shall be provided with openings designed according to standard filter criteria. See Drawing No. C4-5.

5.2.3 Access Pipe

The access pipe shall be constructed of 12-inch diameter HDPE or PVC pipe with a minimum wall thickness of 0.5-inch (nominal dimensions).

5.2.4 Drainage Material

Suitable drainage material shall consist of crushed sandstone meeting the requirements shown on Drawing No. C4-5. In addition, suitable bedding material shall contain no highly calcareous soils (containing visible white material), rubble, frozen and foreign materials or pieces three inches in size or larger.

5.3 Excavation of Trenches

The alignments and excavation lines shown on the drawings are subject to changes as may be found necessary to adapt to conditions disclosed by the excavation and as required for the HDPE or PVC liner installation. Trench excavations shall be made to the lines and relative grades shown on Drawing No. C4-4. Prior to installing the trench linings, the leak detection trenches shall be graded to a surface tolerance of plus or minus 0.1 foot over a 10-foot distance and sharp protrusions shall be removed.

5.4 Installation of Synthetic Liner

Prior to placement of HDPE or PVC pipe and drainage material, a 40 mil HDPE or 30 mil PVC liner shall be placed in the drain trenches as shown on Drawing No. C4-3. At locations where the PVC pipe penetrates the synthetic liner, the liner shall be sealed with a boot and clamp in a manner consistent with the manufacturer's recommendations.

5.5 Placement of Drain Pipe and Drainage Material

An approximate 2-inch layer of drainage material shall be placed in the lined trench to provide bedding for the drain pipes. The drain pipe shall be centrally placed in the lined trench to provide bedding for the drain pipes. The drain pipe shall be centrally placed in the trench on top of the bedding with the slots facing down.

The drainage and access pipes shall be installed as shown on Drawing Nos. C4-4 and C4-5. The HDPE and PVC pipes shall be joined together with manufacturer and Engineer approved methods and procedures. Appropriate fittings, including end caps and T-sections, shall be used as required to join pipe sections together.

The drainage pipes shall be placed so that they slope to the drainage pipe access pipe.

The drainage material shall be placed to minimize segregation and facilitate compaction. Placement of the drainage materials shall be performed in such a manner to avoid damage to the drain pipes.

6.0 SYNTHETIC LINER

6.1 General

This specification covers the design, supply, fabrication and installation of synthetic liners as shown on Drawing Nos. C4-3 and C4-5. The liner is to be delivered to and installed in the new Cell 4 impoundment and in the trenches for the leak detection drains.

6.2 Performance Requirements

1. The liner shall resist both the chemical action of the liquids and the physical action of the tailings stored in the ponds as well as the effects of the environment, including but not limited to, extreme temperatures, ultraviolet radiation and variable wind conditions.

2. The sources of constituents in the liquids are from the processing of uranium ore. The composition of the aqueous liquids which will be pumped to the cell with the tailings are expected to have the typical concentrations given in Table 6.2-1.

3. The ponds shall perform at any degree of fullness from near empty to 100 percent full.

4. Liners shall be suitable for a design ambient temperature range of minus 30° F to plus 110° F. Liner materials shall withstand the higher Liner temperatures resulting from exposure to sunlight.

5. The service life of the Liners shall be at least 22 years.

6.3 Service Conditions

The Liner shall be suitable for installation in an unprotected outdoor location.

The Liner will be in service 24 hours per day, 365 days per year. The only time the ponds wil be completely empty will be at the start of operations.

TABLE 6.2-1

TYPICAL CELLS LIQUIDS COMPOSITION

Ion	Cell 1-I Grams/Liter	Cell 3 Grams/Liter
V	0.27	0.21
U	0.105	0.115
Na	9.7	5.9
NH_3	7.8	13.9
C_1	8.0	5.1
SO_4	190.0	180.0
Cu	0.74	0.40
Ca	0.63	0.64
Mg	0.79	5.40
Al	2.30	2.30
Mn	0.14	0.08
Zn	1.20	0.59
Mo	0.24	0.05
pH	0.70	0.82
As	0.44	0.22
Se	N.D.	N.D.

6.4 Design Criteria

1. The Liners shall be installed in Cell 4 for which the subgrade has been prepared and in the leak detection trenches for which the earthwork has been prepared by Owner. The dimensions of Cell 4 are given on Drawing Nos. C4-1 and C4-2.

2. The tailings slurry will be introduced into the ponds by pumping and by gravity flow through HDPE piping laid over the berms and upon the interior pond slopes. The Liner shall be reinforced at inlet points to prevent wear, puncture, and/or other damage. Where earth berms are constructed to provide a working area, no reinforcing is required.

3. Discharge of liquids to other cells may occur by pumping or by gravity flow. Liquids may also be recycled to the Mill.

4. The methods and standards of joining and sealing the Liner sections to form a watertight Liner shall be specified by the Seller in the quotation.

6.5 Materials

Seller shall warrant that the synthetic liner material supplied and that the thickness meets the requirements of the Specifications.

6.5.1 HDPE Liner

The Liner shall be of high density polyethylene (HDPE) with a minimum thickness of 40 mils on the bottom and on the slopes of Cell 4. Liner thickness in the Specifications are nominal thicknesses. Actual thicknesses shall be within the manufacturer's tolerances.

6.5.2 Alternative Liners

An alternative PVC synthetic Liner with a minimum thickness of 30 mils may be proposed in lieu of HDPE lining materials. Alternate Liners shall meet Performance Requirements, Service Conditions and Design Criteria in accordance with these Specifications.

6.6 Liner Quality

1. The synthetic Liner shall be manufactured of new, first quality products designed and manufactured specifically for the purpose of tailings slurry containment.

2. The Liner materials shall be so produced as to be free of holes, blisters, undispersed raw materials, or any contamination by foreign matter. Any such defect shall be repaired using the extrudate welding technique in accordance with the manufacturer's recommendations.

3. Seller shall submit Certification from the Liner manufacturer documenting that the Liner meets all the physical property specifications for the intended application. In addition, the Seller shall certify that the material meets the minimum requirements for the properties presented in Table 6.1-2 for HDPE liners. If alternate Liners are proposed, the Seller shall submit minimum property requirements of the Liner proposed for review and approval.

6.7 Items of Work

Work by Seller shall include but not be limited to the following:

1. Fabrication of one (1) Liner for the Proposed Cell 4 and liners for the leak detection drains, including all hardware and accessories pipe and reinforcing of the Liner for the inlet piping. See Drawing Nos. C4-1, C4-2, C4-3, and C4-5.

2. Delivery of the Liner material to the Owner's Plant Site near Blanding, Utah.

3. Review and acceptance of Owner's earthwork and Liner installation drawings.

4. Inspection and acceptance of Owner's earthwork prior to Liner installation.

5. Furnishing of equipment, materials, supervision and labor for the Liner uncrating, handling and installation, including joining of the Liner material to form a continuous Liner covering the sides and bottoms of the impoundment and backfilling liner anchoring trenches.

6. Repair of any holes or blemishes detected before filling.

7. All required Quality Control per Section 6.9 of these Specifications.

8. The Liner installer shall provide Umetco with layout drawings of the proposed Liner placement pattern and seams prior to project commencement. The drawings shall indicate the panel configuration and locations of seams.

9. The Liner installer shall provide data on installation details for compensating for expansion and contraction due to temperature fluctuations (minus 30°F to plus 110°F).

6.8 Items of Work Not Included

Owner shall perform the following:

1. Cell earthwork including excavation.

2. Construction of embankment.

TABLE 6.1-2

Material Properties
for
HDPE Liners

Property	Test Method
Gauge (Nominal)	
Thickness, Mils (Minimum)	ASTM D1593
Specific Gravity (Minimum)	ASTM D792
Minimum Tensile Properties (Each Direction)	ASTM D638
1. Tensile Strength at Break (Pounds/Inch Width)	
2. Tensile Strength at Yield (Pounds/Inch Width)	
3. Elongation at Break (Percent)	
4. Elongation at Yield (Percent)	
5. Modulus of Elasticity (Pounds/Square Inch)	ASTM D882
Tear Resistance to Soil (Pounds Minimum)	ASTM D1004 Die C
Low Temperature, F	ASTM D746 Procedure B
Dimensional Stability (Each Direction, Percent Change Maximum)	ASTM D1204 212 F 15 Minutes
Resistance to Soil Burial (Percent Change Maximum in Relation to Original Value)	ASTM D3038 (As Modified in Appendix A)
1. Tensile Strength at Break and Yield	
2. Elongation at Break and Yield	
3. Modulus of Elasticity	
Environmental Stress Crack (Minimum, Hours)	ASTM D1693 (As modified in Appendix A)

3. Placement of the prepared subgrade.

4. Excavation of liner anchoring trenches and trenches for the leak detection drains.

5. Supply and installation of inlet piping and safety ropes.

6.9 Quality Control

The Seller shall provide the Quality Control functions for installation of the Liner. Quality Assurance will be provided by Umetco.

6.9.1 Field Seams

6.9.1.1 Destructive Tests

1. Field Fabricated Startup Seam - Seller shall provide a representative seam fabricated from the same sheet material and using the same seaming methods as those recommended by the synthetic Liner manufacturer. The startup seam shall be no less than 20 feet (3 m) in length and shall be provided at the start of each day's or shift's seaming. Random samples for shear and peel testing shall be cut from the startup seam. The startup seam shall be allowed to cure or age properly before testing in accordance with manufacturer's directions.

2. Field Cut Out - A minimum of one 2-foot (0.61 m) long section of the fabricated seam per 500 feet of seam shall be cut from the installed Lining. The cutout section shall be wide enough to accommodate peel and shear testing. Random specimens for peel and shear testing shall be cut from the sample. The resulting hole shall be patched with an oval-shaped piece of sheet material and shall be seamed in accordance with the manufacturer's instructions. The cutout seam shall be allowed to cure or age properly before testing in accordance with the manufacturer's recommendations.

3. The integrity of the field seams shall be determined in accordance with applicable methods in ASTM D-4437-84 Standards.

4. The location of the seams, the machine used, the operator, weather conditions and any problems shall be logged on a shift basis.

6.9.1.2 Nondestructive Tests

A Quality Control Technician employed by Seller shall make a visual inspection of all field seams. In addition, the following test method shall be employed by the Seller.

Vacuum Box Testing - All field seams shall be inspected for unbonded areas by applying a vacuum to a soaped section of seam. This nondestructive test shall be performed in accordance with the method given in ASTM D-4437-84 Standards.

6.9.1.3 Repairs

All defective field seams as detected by both destructive and nondestructive tests shall be marked and repaired in accordance with manufacturer's recommendations.

6.10 Quality Control Reports

6.10.1 Material

The test reports, material properties sheets and Quality Control certificates required in Section 6.6 shall be supplied to Umetco by the installer and reviewed by Umetco prior to commencement of Liner installation.

6.10.2 Field Installation Reports

The installer shall provide Umetco with reports of the following:

a) Changes in layout drawings.

b) Results of test seams.

c) Welding data.

d) Nondestructive tests results.

e) Destructive tests results.

f) Repair data.

6.10.3 As-Built Drawing

Upon completion of the project, the installer shall provide Umetco with a reproducible original of the drawings showing the panel location number and seam location number, patches and destructive test samples.

6.10.4 Warranty

The Liner installer shall guarantee the Liner to be free of defects for a period of 22 years after installation. These warranties shall be provided to Umetco upon completion of the project.

6.10.5 Acceptance

The installed Liner shall be accepted by Umetco when:

a) The Liner installation is complete.

b) All Quality Control documentation is submitted.

c) As-built drawings are received.

d) Warranties are received.

7.0 SLIMES DRAIN SYSTEM

7.1 General

A slimes drain system shall be constructed above the synthetic liner. The slimes drain shall consist of 4-inch perforated pipe embedded in granular material as shown on Drawing Nos. C4-4 and C4-5.

7.2 Materials

7.2.1 Drainage Pipe

Drainage pipe shall consist of Schedule 40 PVC (ASTM D-2241) or HDPE pipe (ASTM D-3034). The pipe shall be provided with openings designed according to standard filter criteria. See Drawing C4-5.

7.2.2 Access Pipe

The access pipe shall be constructed of 12-inch diameter HDPE or PVC pipe with a minimum wall thickness of 0.5-inch (nominal dimensions).

7.2.3 Drainage Material

Suitable drainage material shall consist of crushed sandstone meeting the requirements shown on Drawing No. C4-5. In addition, suitable bedding material shall contain no highly calcareous soils (containing visible white material), rubble, frozen and foreign materials or pieces three inches in size or larger.

7.3 Installation of Drainage and Access Pipes

The drainage and access pipes shall be installed as shown on Drawing Nos. C4-4 and C4-5. The HDPE or PVC pipes shall be joined together with manufacturer and Engineer approved methods and procedures. Appropriate fittings, including end caps and T-sections, shall be used as required to join pipe sections together.

The drainage pipes shall be placed on the completed cell surface so that they slope to the drainage pipe access pipe.

7.4 Drainage Material Placement

The thickness of the drainage material shall be at least 12 inches when placed in a compacted condition. Material shall be placed in one lift and rolled with four passes of a 10-ton vibratory roller or its equivalent. Care shall be taken not to damage the perforated pipe.

The drainage material shall be compacted to 65 percent relative density (ASTM-D2049) as defined by:

$$D_D = (E_1 - E_N)/(E_1 - E_D) \text{ percent}$$

where:

D_D = relative density in Percent

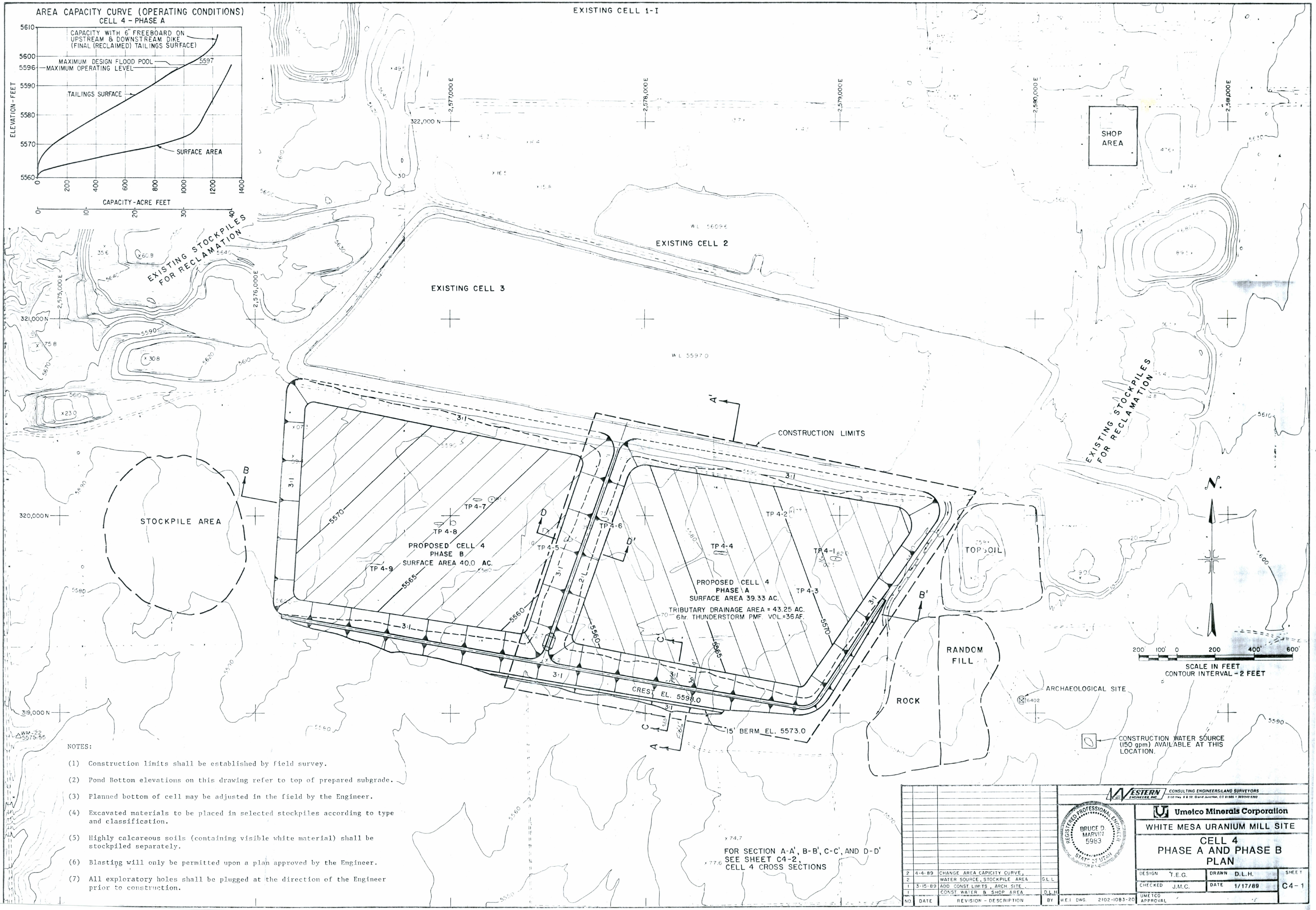
E_1 = void ratio of the granular soil in its loosest state (minimum dry density)

E_D = void ratio of the granular soil in its densest state (maximum dry density)

E_N = void ratio of the soil in its natural state

8.0 DRAWING LIST

Drawing No.	Title
C4-1	Cell 4 Plan
CR-2	Cell 4 Cross Sections
CR-3	Typical Cell Liner Details
CR-4	Leak Detection and Slimes Drain Plans
CR-5	Leak Detection and Slimes Drain Details



NOTES:

(1) Construction limits shall be established by field survey.

(2) Pond Bottom elevations on this drawing refer to top of prepared subgrade.

(3) Planned bottom of cell may be adjusted in the field by the Engineer.

(4) Excavated materials to be placed in selected stockpiles according to type and classification.

(5) Highly calcareous soils (containing visible white material) shall be stockpiled separately.

(6) Blasting will only be permitted upon a plan approved by the Engineer.

(7) All exploratory holes shall be plugged at the direction of the Engineer prior to construction.

NO	DATE	REVISION - DESCRIPTION	BY
2	4-4-89	CHANGE AREA CAPACITY CURVE, WATER SOURCE, STOCKPILE AREA	G.L.L.
1	3-15-89	ADD CONST. LIMITS, ARCH. SITE, CONST. WATER & SHOP AREA	D.L.H.

WESTERN ENGINEERS, INC. CONSULTING ENGINEERS/LAND SURVEYORS

REGISTERED PROFESSIONAL ENGINEER BRUCE D. MARVIN 5983 STATE OF UTAH

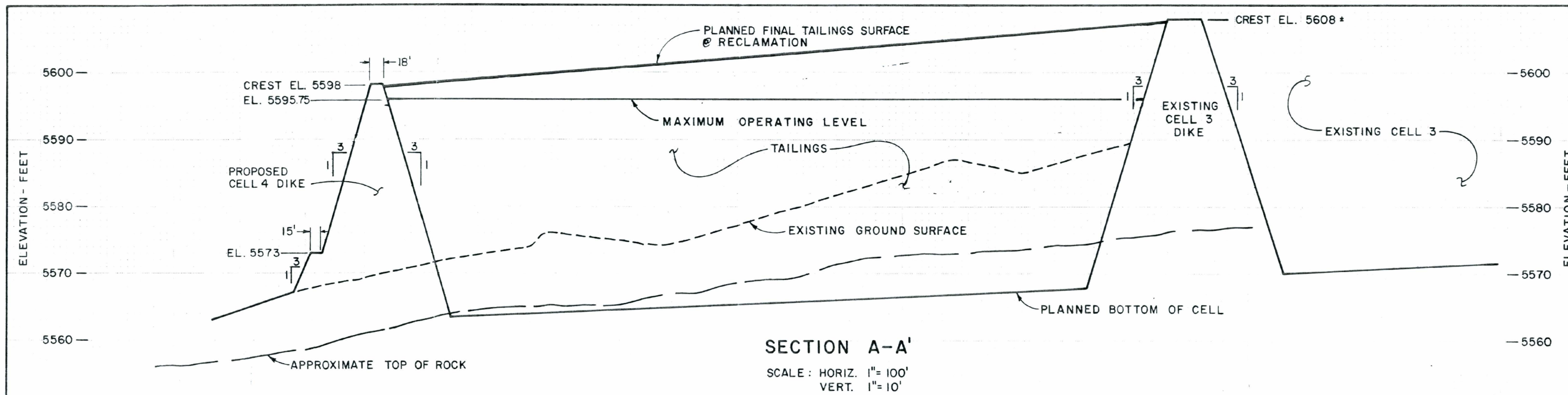
Umetco Minerals Corporation

WHITE MESA URANIUM MILL SITE

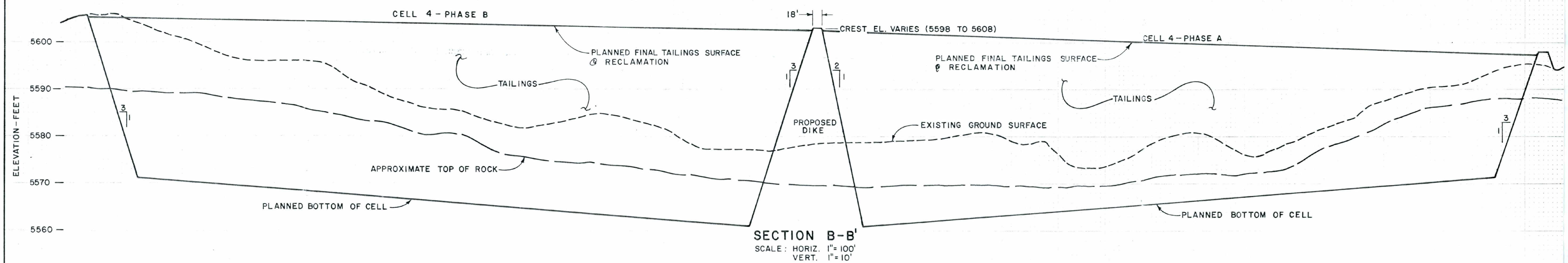
CELL 4
PHASE A AND PHASE B
PLAN

DESIGN T.E.G.	DRAWN D.L.H.	SHEET
CHECKED J.M.C.	DATE 1/17/89	C4-1
UMETCO APPROVAL		

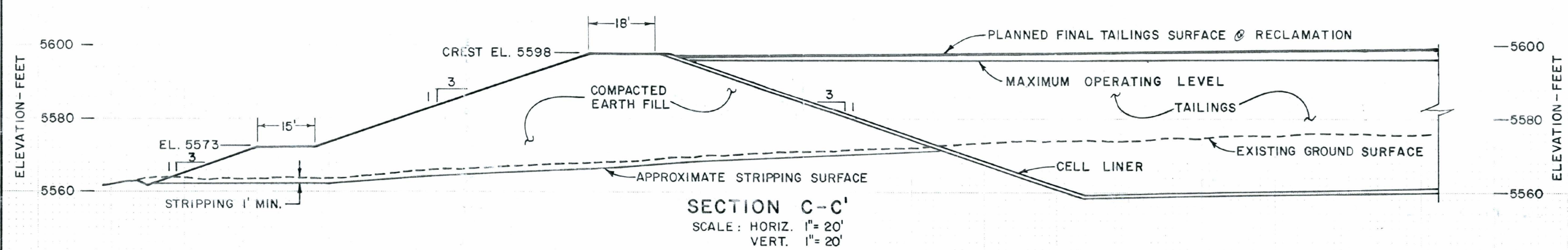
W.E.I. DWG. 2102-1083-20



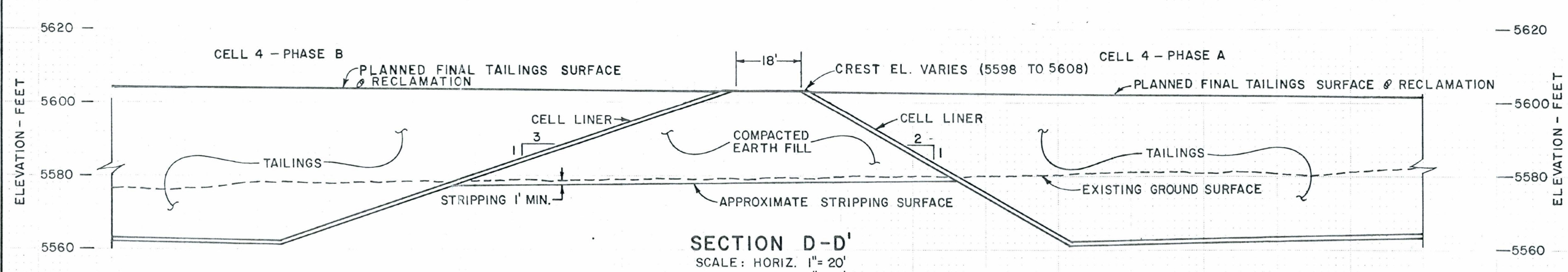
SECTION A-A'
SCALE: HORIZ. 1"= 100' VERT. 1"= 10'



SECTION B-B'
SCALE: HORIZ. 1"= 100' VERT. 1"= 10'



SECTION C-C'
SCALE: HORIZ. 1"= 20' VERT. 1"= 20'



SECTION D-D'
SCALE: HORIZ. 1"= 20' VERT. 1"= 20'

NOTE:

(1) Embankment material to classify as CL or ML under the Unified Soil Classification System. Highly calcareous soils (containing visible white material) shall not be used in embankment construction.

(2) Embankment fill to be placed in 12-inch, loose lifts or less; and compacted to at least 95% Maximum Standard Proctor Dry Density (ASTM D-698) to within ±2% of optimum moisture content.

(3) A camber of 0.5 feet to be provided on top of the embankment at section of maximum fill. Camber to be reduced at other sections in proportion to height of fill.

(4) Cell 4 - Phase B shall be constructed or under construction before tailings are placed in Cell 4 - Phase A above the planned interim tailings surface. A spillway constructed in the proposed dike between Cell 4 - Phase A and Cell 4 - Phase B will channel storm runoff to Cell 4 - Phase B as tailings are added up to the Planned Final Tailings Surface in Cell 4 - Phase A.

FOR LOCATIONS OF SECTIONS SEE SHEET C4-1, CELL 4 PLAN

NO.	DATE	REVISION - DESCRIPTION	BY
1	4-5-89	REV MAX OPER. LEVEL ELEV	RWQ

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REGISTERED PROFESSIONAL ... BRUCE D. MARVIN 5983 STATE OF UTAH

Umetco Minerals Corporation

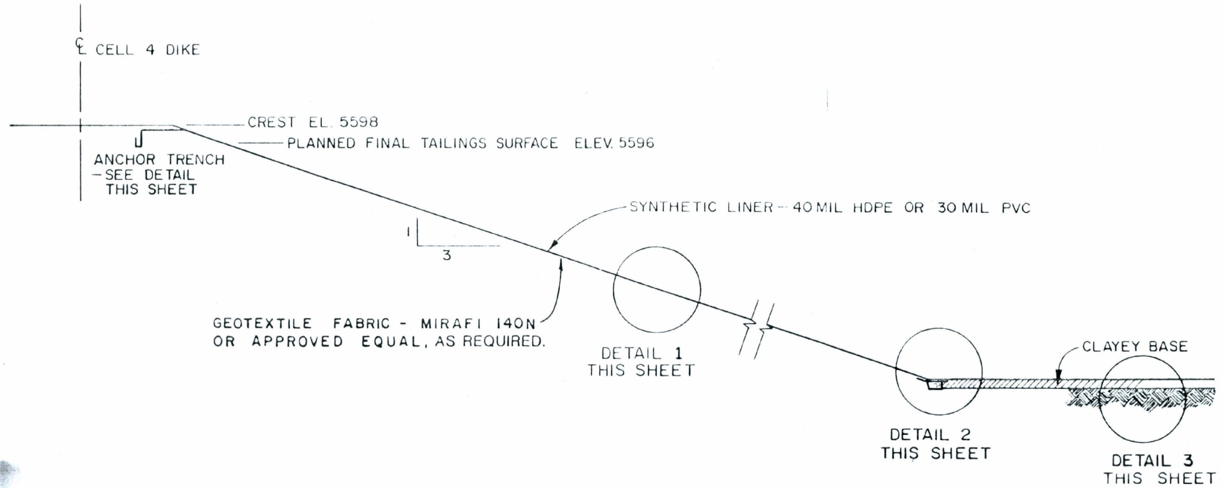
WHITE MESA URANIUM MILL SITE

CELL 4
CROSS SECTIONS

DESIGN	T.E.G.	DRAWN	D.L.H.
CHECKED	J.M.C.	DATE	1/17/89
UMETCO APPROVAL			

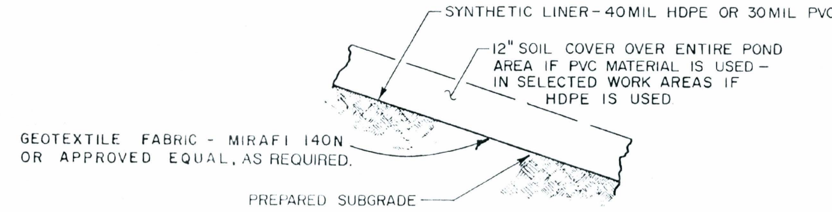
SHEET C4-2

W.E.I. DWG. 2102-1083-21



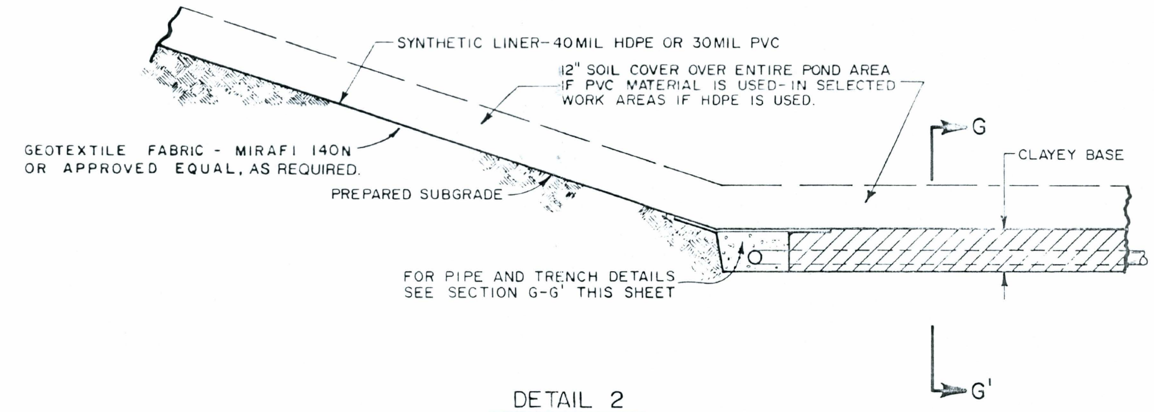
TYPICAL LINER INSTALLATION

10' 5' 0 10' 20'
SCALE IN FEET



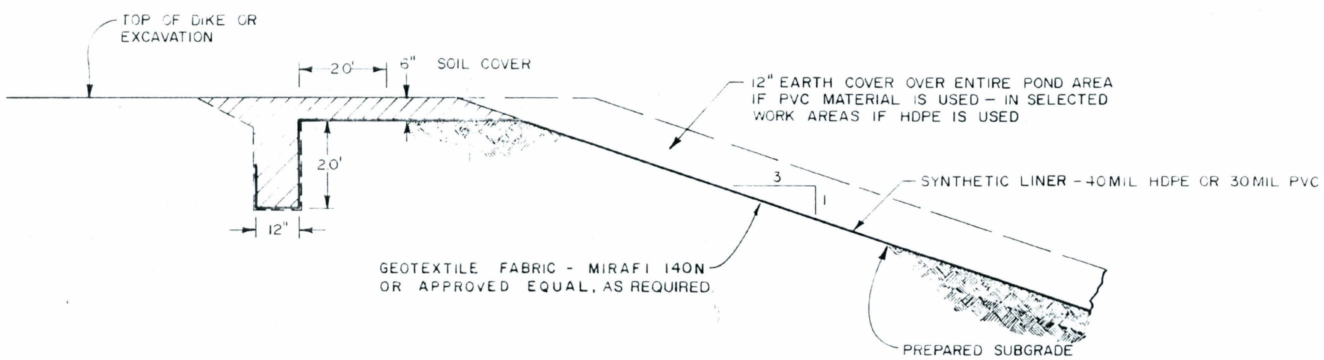
DETAIL 1

1' 6" 0 1' 2' 3' 4' 5'
SCALE IN FEET



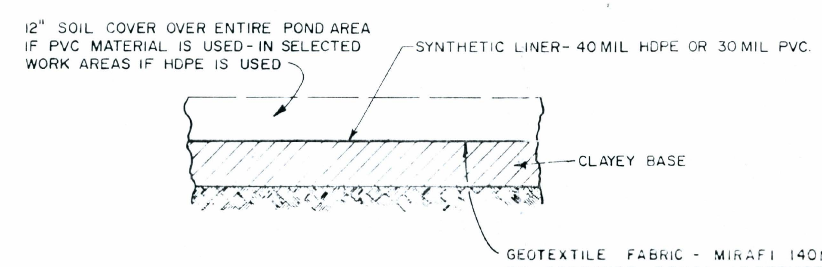
DETAIL 2

1' 6" 0 1' 2' 3' 4' 5'
SCALE IN FEET



TYPICAL ANCHOR TRENCH DETAIL

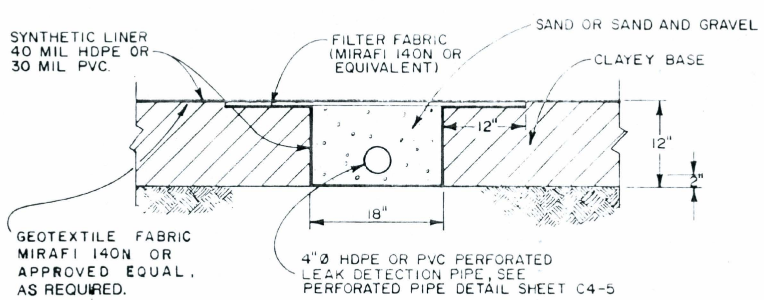
1' 6" 0 1' 2' 3' 4' 5'
SCALE IN FEET



DETAIL 3

TYPICAL LINER INSTALLATION ON
CELL BOTTOM

1' 6" 0 1' 2' 3' 4' 5'
SCALE IN FEET



SECTION G-G'

1' 6" 0 1' 2'
SCALE IN FEET

NOTES:

(1) A 12-inch soil cover or tailings shall be placed over entire liner (except in the area of the slimes drain) if PVC is used, otherwise, in selected work areas if HDPE is used.

(2) Liner to be reinforced with an additional layer of liner beneath entry pipes anchored to top of berms.

(3) All bedding and drainage material (crushed sandstone) shall classify as SM or SP in the Unified Soil Classification System and have a maximum size of 3/4 inch.

(4) Geotextile shall be non-woven and have a minimum fabric weight of 4 oz/yard. Geotextile shall be MIRAFI 140N or approved equal.

WESTERN ENGINEERS, INC. — CONSULTING ENGINEERS/LAND SURVEYORS

REGISTERED PROFESSIONAL ENGINEER — BRUCE D. MARVIN 5983 — STATE OF UTAH

Umetco Minerals Corporation

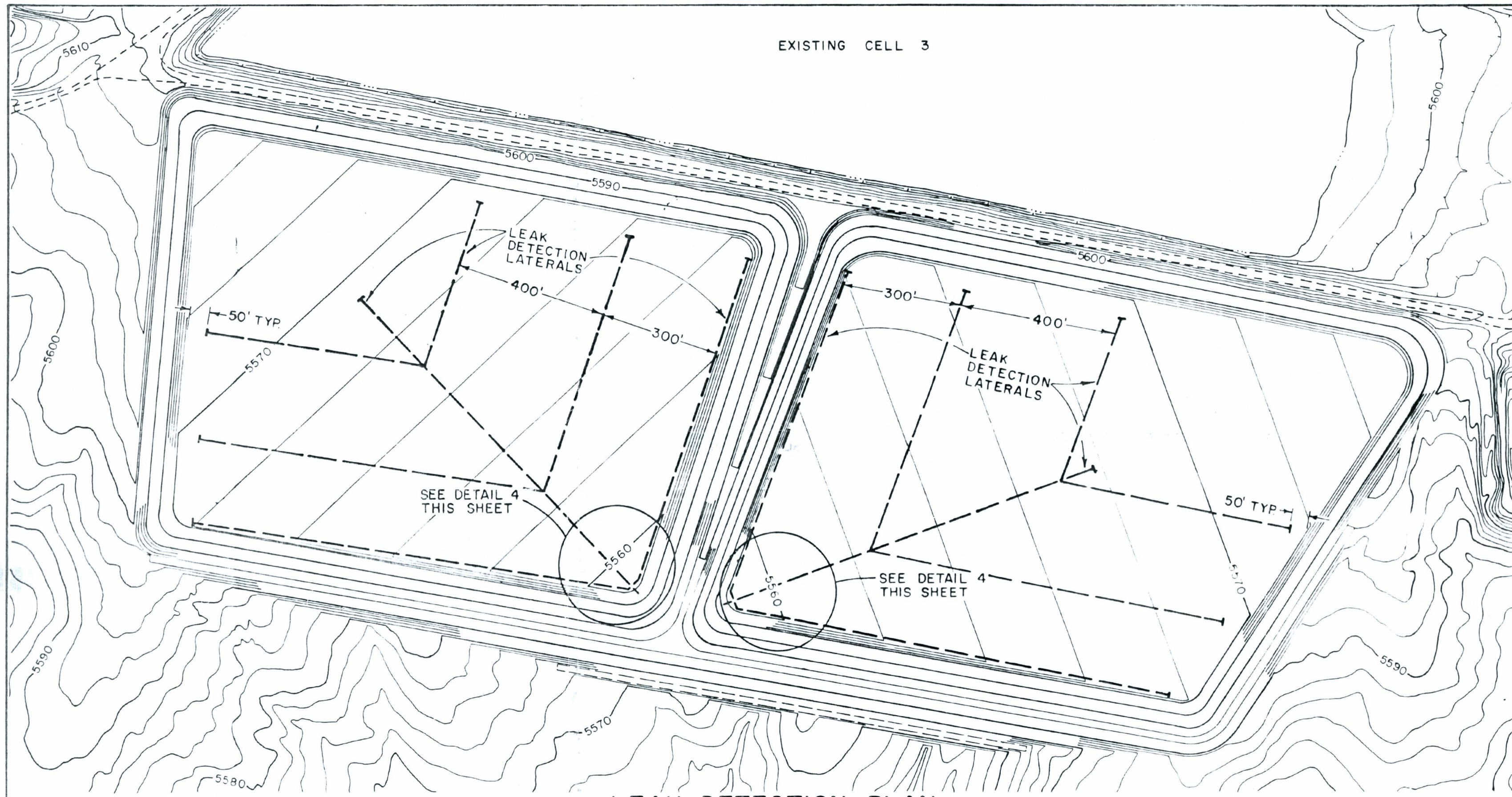
WHITE MESA URANIUM MILL SITE

TYPICAL
CELL LINER DETAILS

NO.	DATE	REVISION - DESCRIPTION	BY

DESIGN	T.E.G.	DRAWN	D.R.H.	SHEET
CHECKED	J.M.C.	DATE	1/17/89	C4-3
UMETCO APPROVAL	[signature] P.E.			

W.E.I. DWG. NO. 2102-1083-22

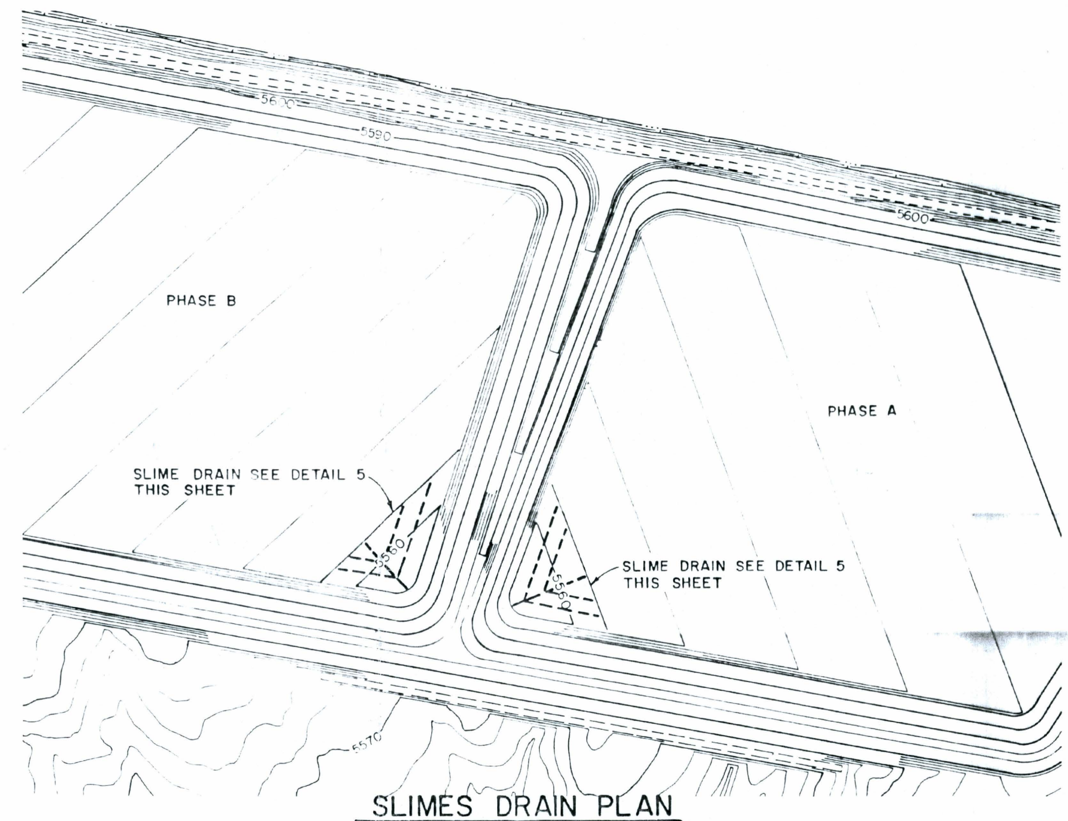


LEAK DETECTION PLAN

PROPOSED CELL 4

200' 100' 0 200' 400' 600'

SCALE IN FEET

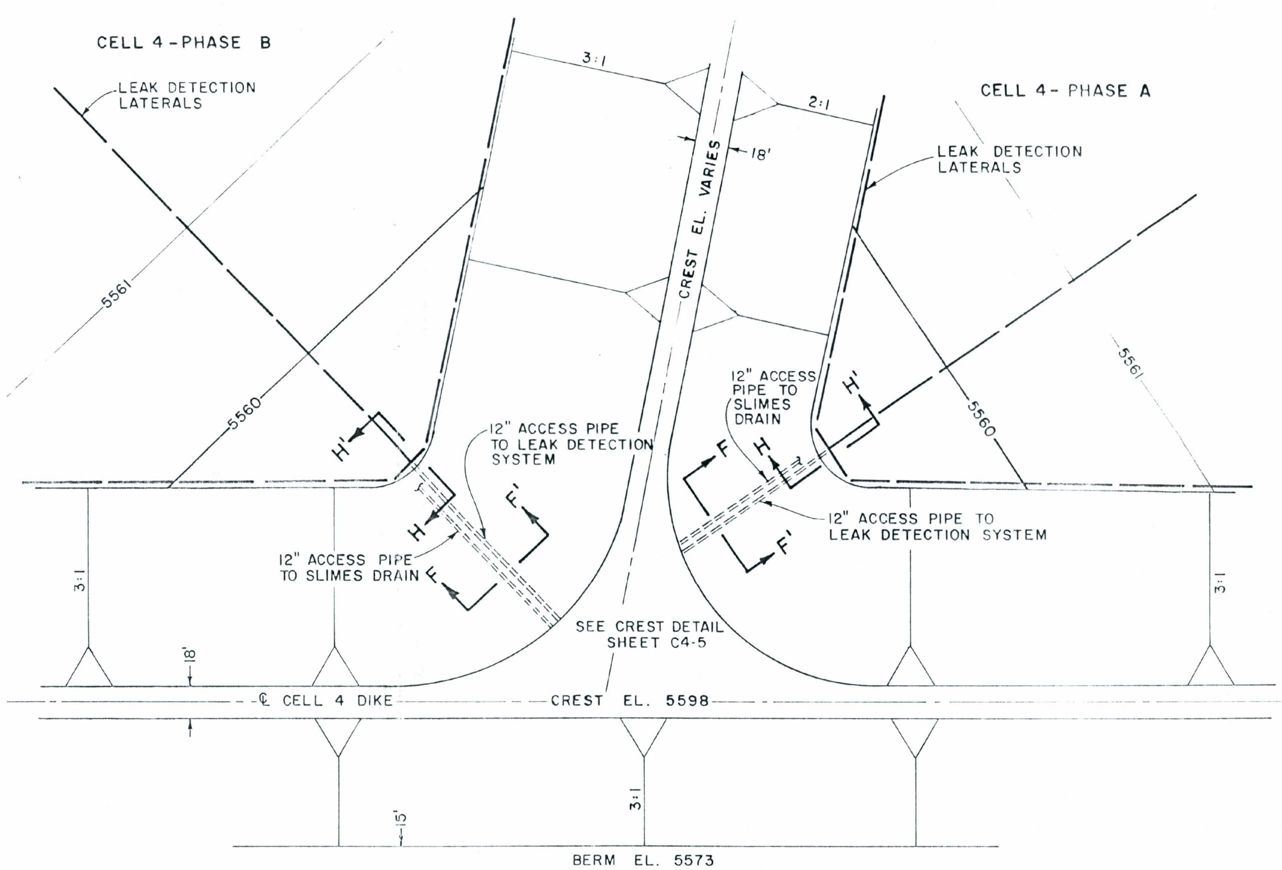


SLIMES DRAIN PLAN

PROPOSED CELL 4

200' 100' 0 200' 400' 600'

SCALE IN FEET

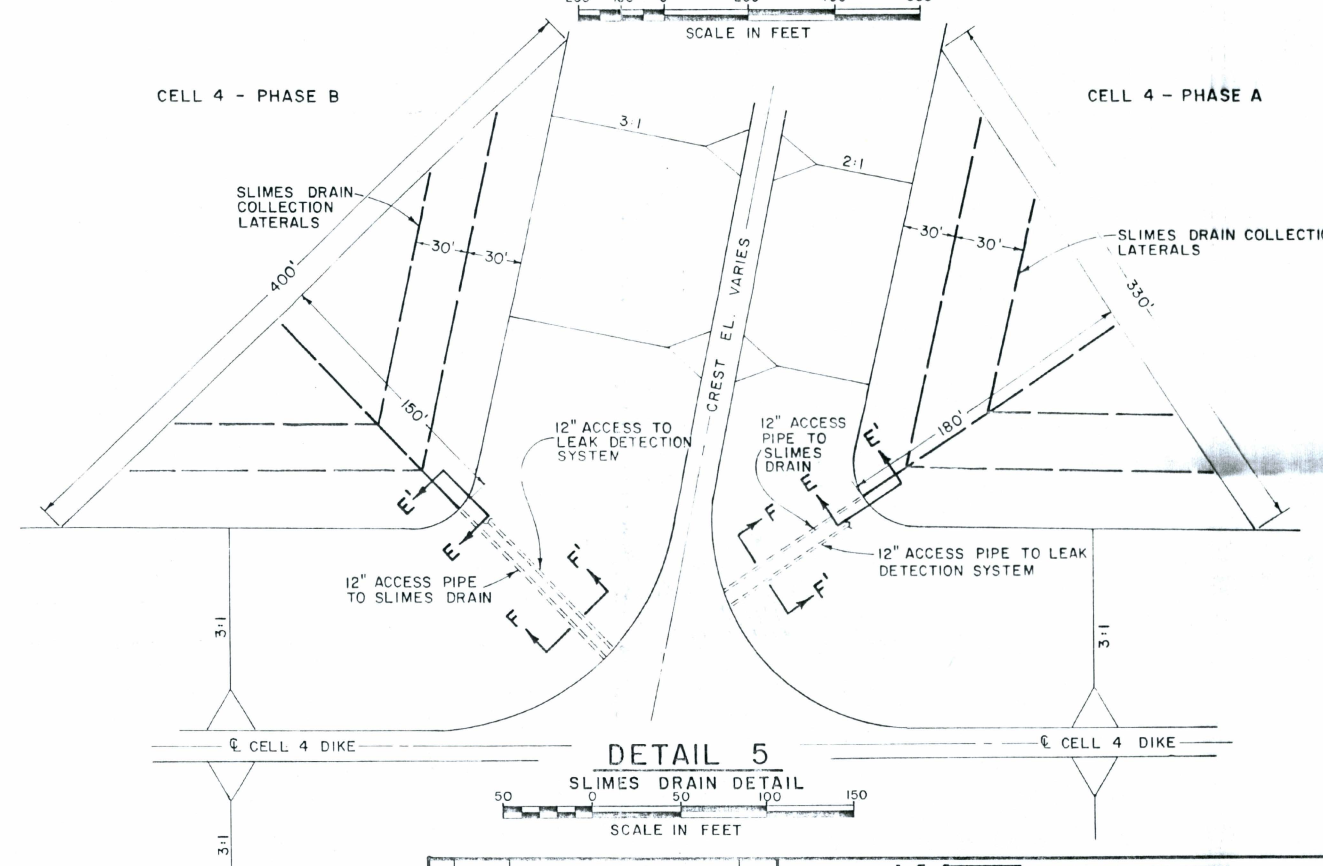


DETAIL 4

LEAK DETECTION SYSTEM DETAIL

50' 0 50' 100' 150'

SCALE IN FEET



DETAIL 5

SLIMES DRAIN DETAIL

50 0 50 100 150

SCALE IN FEET

FOR SECTIONS E-E', F-F', AND H-H' SEE SHEET C4-5, LEAK DETECTION AND SLIMES DRAIN DETAILS.

NO	DATE	REVISION - DESCRIPTION	BY

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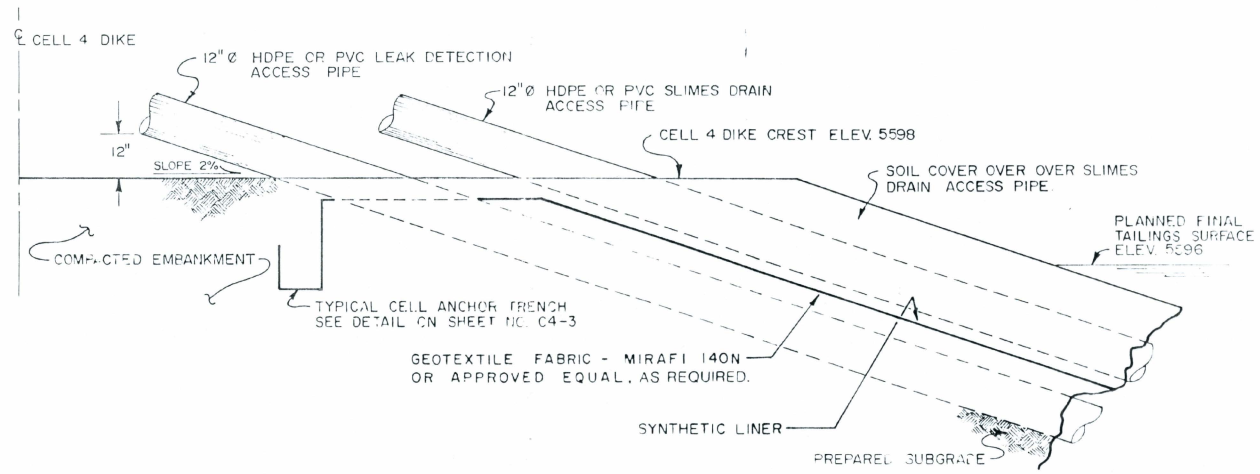
Umetco Minerals Corporation

WHITE MESA URANIUM MILL SITE

LEAK DETECTION AND SLIMES DRAIN PLANS

DESIGN T.E.G.	DRAWN R.W.Q. D.L.H.	SHEET	
CHECKED J.M.C.	DATE 1/17/89	C4-4	
UMETCO APPROVAL			

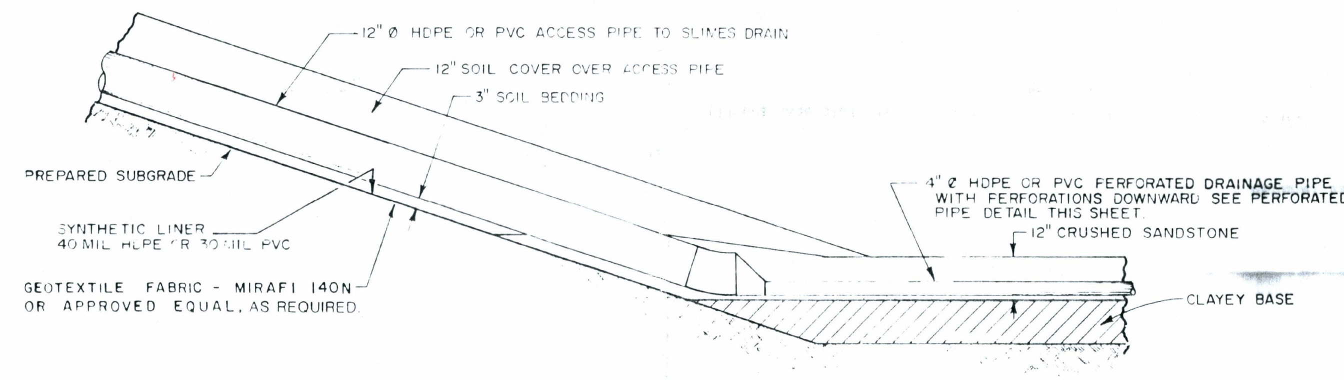
W.E.I. DWG. 2102-1083-23



CRESTDETAIL

ACCESS PIPES

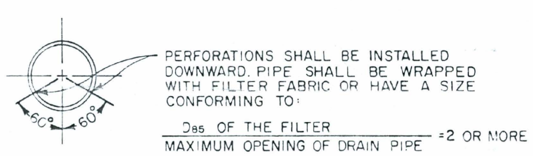
SCALE IN FEET



SECTION E-E'

ACCESS PIPE TO SLIMES DRAIN

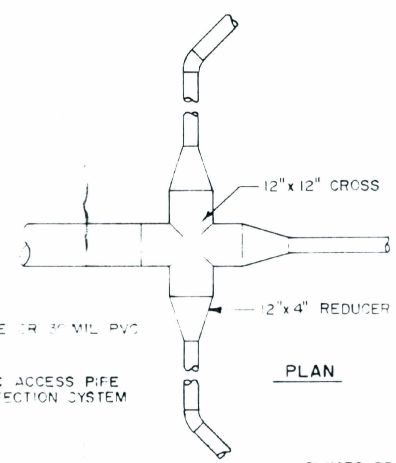
SCALE IN FEET



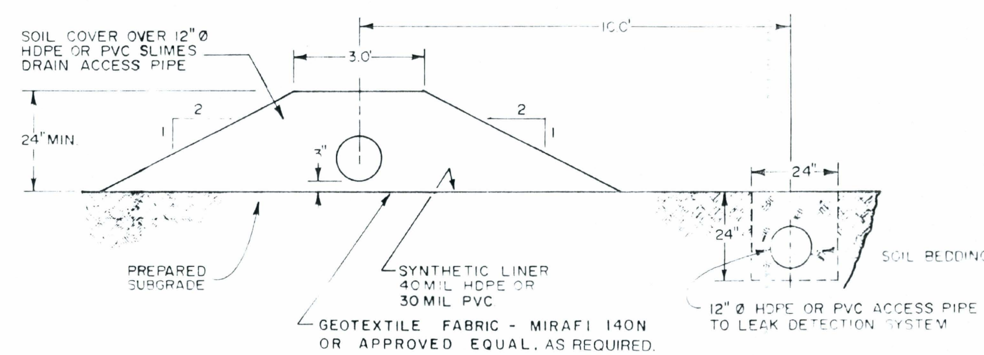
PERFORATED PIPE DETAIL

SLIMES DRAIN AND LEAK DETECTION SYSTEM.

– NOT TO SCALE –



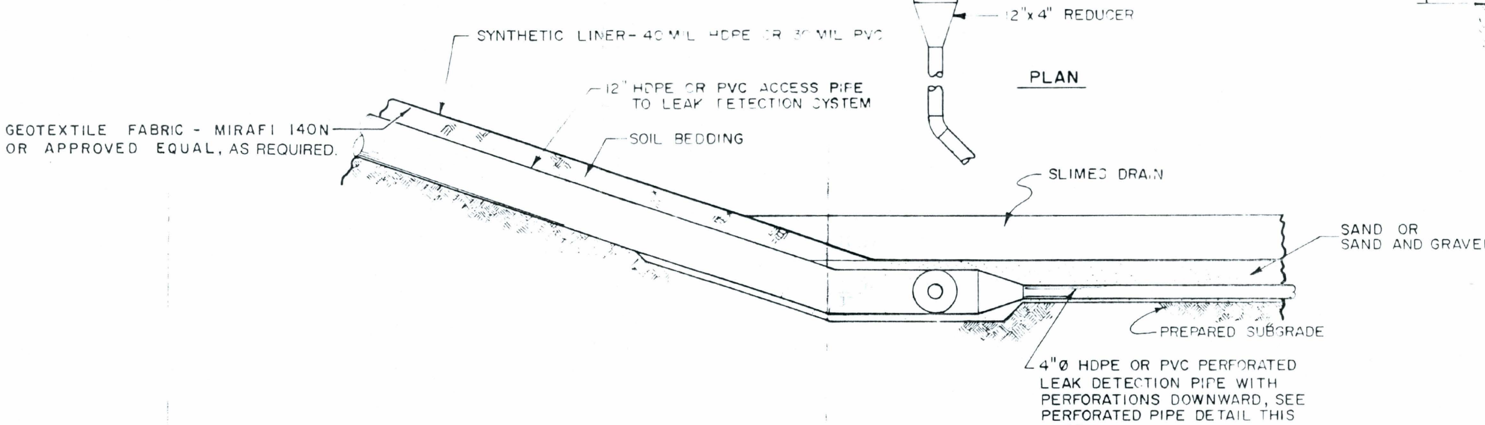
PLAN



SECTION F-F'

ACCESS PIPES ON EMBANKMENT SLOPES

SCALE IN FEET



SECTION H-H'

ACCESS PIPE TO LEAK DETECTION

SCALE IN FEET

Notes:

(1) Drainage pipe shall be Schedule 40 PVC (ASTM D-3034) or HDPE (ASTM D-2241).

(2) All HDPE pipe shall be "Driscopipe" or approved equal.

(3) All bedding and drainage material (crushed sandstone) shall classify as SM or SP in the Unified Soil Classification System and have a maximum size of 3/4 inch.

FOR LOCATIONS OF SECTIONS SEE SHEET C4-4, LEAK DETECTION AND SLIMES DRAIN PLANS

NO.	DATE	REVISION - DESCRIPTION	BY

WESTERN ENGINEERS, INC. CONSULTING ENGINEERS/LAND SURVEYORS

Umetco Minerals Corporation

WHITE MESA URANIUM MILL SITE

LEAK DETECTION AND SLIMES DRAIN DETAILS

DESIGN	T.E.G.	DRAWN	D.R.H.	SHEET
CHECKED	J.M.C.	DATE	1/17/89	C4-5
UMETCO APPROVAL				

W.E.I. DWG. NO. 2102-1083-24

ATTACHMENT II

QUALITY PLAN NO. QP-GEN-1-C4WM

QUALITY PLAN

FOR

CONSTRUCTION ACTIVITIES

WHITE MESA PROJECT

BLANDING, UTAH

PREPARED BY
UMETCO MINERALS CORPORATION
GRAND JUNCTION, COLORADO

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	[signature], P.E.

CONFIDENTIALITY

This Quality Plan No. QP-GEN-1-C4WM is considered to be proprietary information developed by Umetco Minerals Corporation.

INDEX

PART II ORGANIZATIONAL STRUCTURE

PART III SURVEYS, INSPECTIONS, SAMPLING AND TESTING

PART IV CHANGES AND CORRECTIVE ACTIONS

PART V DOCUMENTATION

PART VI QUALITY CONTROL PROCEDURES

PART I GENERAL

1.0 SCOPE OF QUALITY PLAN

The Quality Plan for Construction Activities hereinafter referred to as the Quality Plan describes how the Construction Quality Control/Quality Assurance (QC/QA) activities are implemented.

This Quality Plan includes the following:

(1) Organizational Structure.

(2) Surveys, Inspections, Sampling and Testing.

(3) Changes and Corrective Actions.

(4) Documentation Requirements.

2.0 QUALITY PLAN OBJECTIVES

The objectives of the Quality Plan are as follows:

(1) Quality Control: Quality Control verifies that the construction is in accordance with the Plans and Specifications.

(2) Quality Assurance: Quality Assurance provides the cross-checks and auditing function on Quality Control.

(3) Monitoring: Monitoring provides the required information and data to evaluate the effects of Construction Activities.

3.0 DEFINITIONS

Compliance Report A report prepared by the QC Officer upon completion of a Construction Segment. A Compliance Report requires the approval of the Site Manager and the Regulatory Project Manager. Any subsequent Construction Segment that is dependent upon successful completion of a specific Construction Segment cannot be initiated until a Compliance Report is prepared and approved for the previous dependent Construction Segment. Compliance Reports are to be completed on Form No. F-23 which is attached in Part V.

Construction Task A basic construction feature of a Construction Project involving a specific Construction Activity.

Construction Segment An essential construction component consisting of one or more Construction Tasks of a Construction Project. Construction Segments require a Compliance Report upon completion to verify this project component has been constructed in accordance with the Plans and Specifications.

Construction Project The total authorized/approved Project that requires several Construction Segments to complete.

Design Change Changes made in a Construction Project that alters or changes the intent of the Plans and Specifications. Design changes require approval of the Design Engineer, the Site Manager or his designated representative and the Regulatory Project Manager. Design Changes are to be reported on Form No. F-26 which is attached in Part V.

Field Change Changes made during construction to fit field conditions that do not alter the intent of the Plans and Specifications. Field Changes require approval of the Site Manager or his designated representative. Field Changes are to be reported on Form No. F-25 which is attached in Part V.

Final Construction Report A report prepared by the Site Manager or his designated representative upon completion of a Construction Project. This report shall be submitted to the Regulator.

4.0 QUALITY CONTROL/QUALITY ASSURANCE

4.1 Methodology

4.1.1 Flow of Activities

Figure 1 shows the general relationships of Quality Control and Quality Assurance activities in the performance of the Construction Activities for a given work area. The Quality Control Activities implemented with standardized QC procedures, provide the necessary tests and observations for the construction, sampling and monitoring process. Quality Assurance audits and reviews will provide oversight of the QC Activities.

4.1.2 Compliance Reports

For each project, the Quality Plan requires a Compliance Report at the successful completion of a Construction Segment. The Construction Tasks making up a Construction Segment shall be determined to be in compliance with the Plans and Specifications by the QC Officer. A Compliance Report will then be prepared by the QC Officer and submitted to the Site Manager and the Regulatory Project Manager for approval, before the next dependent phase of construction can begin. The Site Manager and Regulatory Project Manager shall review Quality Control Data, Quality Assurance documentation and review their own observations before approving the Compliance Report.

After the Construction Project has been completed, a Final Construction Report shall be prepared by the Site Manager or his designated representative for submittal to the Regulator for approval.

4.2 Quality Control

4.2.1 General

Quality Control (QC) will be conducted by the Quality Control Officer or his designated representative hereinafter referred to as the QC Officer. The QC Officer will implement the QC Program.

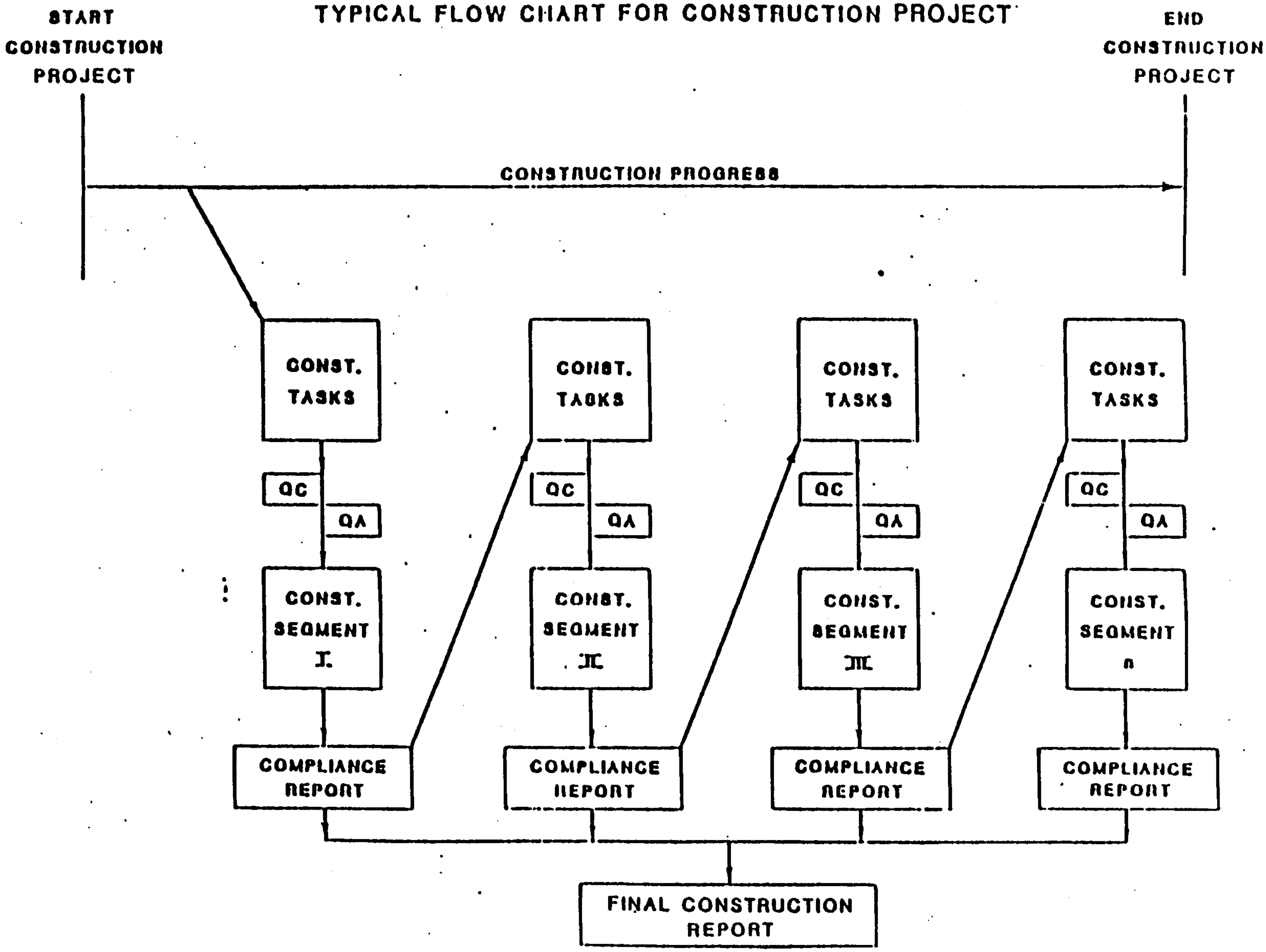
4.2.2 Quality Control Activities

Quality Control requirements for a Construction Project are presented in the Specifications.

The Quality Control Activities will be implemented with standardized Quality Control Procedures. The Quality Control Procedures include Field Sampling, Testing, Observation and Monitoring Procedures, and Laboratory Testing Procedures. The Quality Control Procedures are listed and are included in Part VI (see Table III).

FIGURE 1

TYPICAL FLOW CHART FOR CONSTRUCTION PROJECT



4.3 Quality Assurance

4.3.1 General

Quality Assurance (QA) will be conducted by the Quality Assurance Officer or his designated representative hereinafter referred to as the QA Officer. The QA Officer will implement the QA Program.

The QA Officer shall coordinate the QA program with the QC Officer and Regulatory Project Manager. The QA Officer will perform specific QA Activities on an as-needed basis.

4.3.2 Quality Assurance Activities

The QA functions shall be implemented by the QA Officer by performing the following activities.

4.3.2.1 Prequalification of QC Personnel

Each QC Personnel shall be prequalified by a QA Officer who is a knowledgeable specialist in the area of qualification. The QA Officer shall determine the areas of expertise of the respective technician and maintain a QA file on the technician. Areas of competency will be identified and training needs noted for the respective technician.

4.3.2.2 Verification of Effectiveness of QC Program

The effectiveness of the QC Program will be verified by the QA Officer by performing the following audits:

(1) Test and Sampling Procedures. Test procedures will be be audited on a quarterly basis by appropriate specialists. This will entail direct observation of test methods and sampling, and performing random duplicate tests.

(2) Equipment. Equipment will be inspected and checked intermittently. Calibration Certificates will be verified and maintained in the files.

(3) Calculations and Documentation. Calculations from tests and monitoring will be spot checked randomly from the files. Documentation will be checked for accuracy and completeness.

4.4 Documentation

Each QA Activity and Audit will be documented in writing. Audit Reports will be prepared by the QA Officer, submitted to the Site Manager and kept in the White Mesa Project Files and made available for review by the Regulatory Project Manager.

5.0 MONITORING

Monitoring functions fall under the responsibilities of the Quality Control Officer. Scheduled monitoring and observations shall be made at the intervals required in the Plans and Specifications by Quality Control Field Technicians under the direction of the QC Officer. Monitoring records shall be reviewed by the QC Officer and shall be made available for review by the Regulatory Project Manager. The QA Officer shall audit monitoring records on an unscheduled basis. Monitoring records originals shall be maintained in the White Mesa Project Files.

PART II ORGANIZATIONAL STRUCTURE

1.0 SCOPE

The following items are covered in this Section:

(1) A description of the Quality Control Organization.

(2) The classification, qualifications, duties, responsibilities and authority of personnel.

(3) The individual who shall be responsible for overall management at the site for Quality Control.

(4) The specific authority and responsibility of all other personnel regarding the Quality Plan.

(5) A program for information flow among workers, construction management and inspectors about various QC/QA, and health and safety requirements.

2.0 ORGANIZATION

A schematic diagram of the organization for implementation of the Quality Plan is shown on Fig. 2. The Site Manager, the Quality Control Officer, the Quality Assurance Officer, and the Regulatory Project Manager play major roles.

3.0 DUTIES AND QUALIFICATIONS OF PERSONNEL

3.1 Site Manager

3.1.1 Duties, Responsibilities and Authority

The Site Manager shall oversee the Construction Project and shall be responsible for the conduct, direction and supervision of the Work. As shown on the organizational chart, the Site Manager shall have ultimate responsibility for all construction and QC/QA Activities. He shall appoint all personnel. He shall interact as required with the QA Officer, the QC Officer and the Regulatory Project Manager.

3.1.2 Qualifications

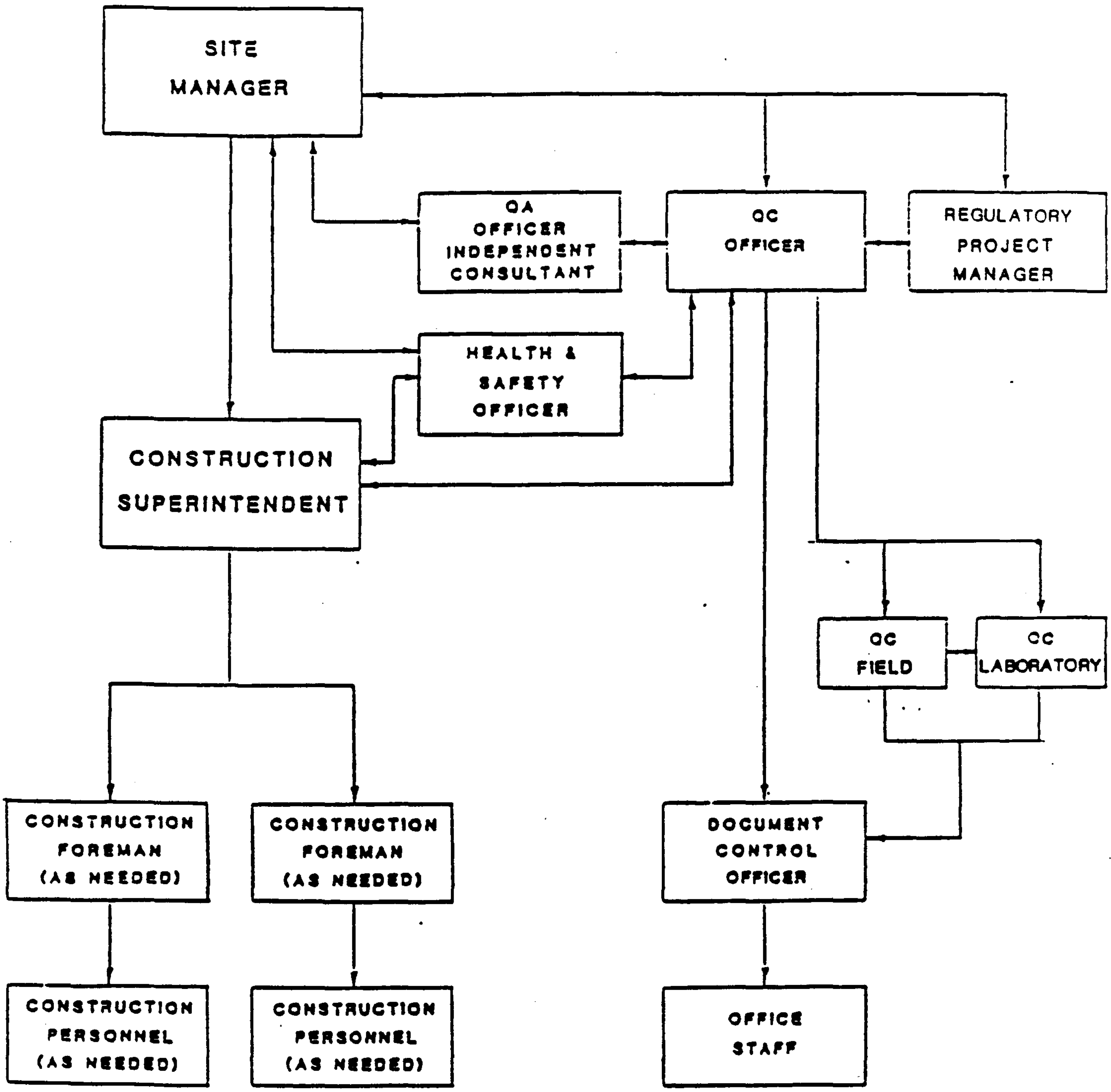
The Regulator shall be notified in writing of the name, title and qualifications of the Site Manager.

3.2 Designated Representative for Site Manager

In the absence of the Site Manager, his designated representative shall assume the duties of the Site Manager. The Regulator shall be notified in writing of the name and qualifications of the Designated Representative for the Site Manager.

FIGURE 2

QC/QA ORGANIZATION CHART



3.3 Quality Control Officer

3.3.1 Duties, Responsibilities and Authority

The Quality Control Officer shall be responsible for overall implementation and management of the Quality Control Program for the Construction Project. He shall supervise Field and Laboratory Quality Control Technicians, and shall coordinate with the Document Control Manager, the Office Staff and the Health and Safety Officer. He shall have specific authority and responsibility with regard to all other personnel for the Quality Plan. The Quality Control Officer shall have the authority to reject work or material, to require removal or placement, to specify and require appropriate corrective actions if it is determined that the Quality Control/Quality Assurance, personnel, instructions, controls, tests, records, are not conforming to the Plans and Specifications. He will be the main liaison between Umetco and the Regulatory Project Manager. The Quality Control Officer's signature is required on all Compliance Reports (CR's) required in the Specifications.

He will be familiar with the existing White Mesa Facilities, and Quality Control/Quality Assurance methodology. As Quality Control Officer, his responsibilities will include the following:

(1) Provide overall surveillance of Quality Control requirements.

(2) Be familiar with all documents, requirements, equipment and procedures relating to project construction.

(3) Provide and document Quality Control Technician training.

(4) Evaluate and approve all reports.

(5) Assure schedules are met and adequately documented.

(6) Schedule data reduction activities.

(7) Arrange consultation with additional staff, the QA Officer, Site Manager, and/or Regulatory Project Manager to help find solutions to unsolved problems.

(8) Identify invalid, unacceptable, or unusable data.

(9) Take corrective action if Quality Control procedures indicate the construction is not meeting the requirements of the Specifications.

(10) Assure all documentation is complete, accurate, and up to date.

(11) Interact and cooperate with QA Personnel.

3.3.2 Qualifications

The Regulator shall be notified in writing of the name, title and qualifications of the Quality Control Officer.

3.4 Designated Representative for Quality Control Officer

In the absence of the Quality Control Officer, his designated representative shall assume the duties of the Quality Control Officer. In addition, the designated representative may be assigned some of the duties, responsibilities and authority of the Quality Control Officer. The Regulator shall be notified in writing of the name and qualifications of the Designated Representative for the Quality Control Officer.

3.5 Quality Assurance Officer

3.5.1 Duties

The Quality Assurance Officer, an independent consultant and/or Umetco/UCC expert, shall implement the Quality Assurance functions which includes prequalification of QC Personnel, verification of test procedures and results by spot retests, equipment checks, and review of calculations and documentation and Compliance Reports (CR's). He should be familiar with the construction process and be qualified in construction testing.

His responsibilities will include the following:

(1) Be familiar with all documents, requirements, equipment and procedures relating to project construction.

(2) Certify that the QC Officer is qualified to conduct the various test and monitoring procedures and observations, and document same.

(3) Through spot checks, retests, equipment checks and review of calculations and documentation verify test procedures, monitoring and observations are being performed correctly and accurately in accordance with the Specifications.

(4) Consult with the QC Officer, and the Site Manager to help solve problems.

(5) Prepare QA reports for review by the Site Manager and Regulatory Project Manager.

3.5.2 Qualifications

The Regulator shall be notified in writing of the name, company and qualifications of the Quality Assurance Officer.

3.6 Designated Representative of the Quality Assurance Officer

In the absence of the Quality Assurance Officer, his designated representative shall assume the duties of the Quality Assurance Officer. In addition, certain specialists may be designated to assume some of the duties of the Quality Assurance Officer. The Regulator shall be notified in writing of the name and qualifications of the Designated Representative for the Quality Assurance Officer.

3.7 NRC Project Manager

3.7.1 Duties

The Regulatory Project Manager shall represent the Regulator's interests in the Construction Project, playing an overview role in reviewing procedures, personnel qualifications, equipment, calculations and documentation. The Regulatory Project Manager's approval signature is required on all Compliance Reports (CR's) required in the Specifications.

3.7.2 Qualifications

The Regulator shall notify Umetco/UCC of the name, title and qualifications of the Regulatory Project Manager.

3.8 QC Technicians

3.8.1 Duties

The QC Technicians for implementation of the Quality Plan shall be classified as follows:

(1) Construction Quality Control Technicians - Field.

(2) Construction Quality Control Technicians - Laboratory.

A Quality Control Technician may be qualified for and perform the duties in more than one classification.

3.8.2 Qualifications

The QC Officer shall supervise or may appoint a supervisor for each classification to provide scheduling, oversee equipment calibrations, enforce documentation requirements and provide for preliminary document review. The number of Technicians in each classification will depend on the project needs as the work progresses.

The Construction Quality Control Technicians shall satisfactorily complete a training program and receive on-the-job training as required under the direction of the QC Officer.

A procedure verification program shall be implemented by the QA Officer for all Construction Quality Control Technicians.

3.9 <u>Personnel Designations</u>

The Site Manager or his designated representative will be referred to as the Site Manager.

The Quality Control Officer or his designated representative will be referred to as the QC Officer.

The Quality Assurance Officer or his designated representative will be referred to as the QA Officer.

4.0 PROGRAM FOR INFORMATION FLOW

4.1 Review of Documents

The Plans and Specifications for the Construction Project describe the Work to be performed. The Plans and Specifications describe the QC/QA and Monitoring requirements.

These documents shall be reviewed in depth by Umetco personnel including the Quality Control Officer and Site Manager.

4.2 Information Flow

4.2.1 Umetco Information Flow

As shown on the Organization Chart (Figure 2), the Construction Superintendent gives instructions to the Construction Foremen who supervise the construction workers. The Construction Superintendent may directly supervise all or some of the construction workers.

The QC Officer monitors the construction work and completes the forms and reports as given in the Quality Control Procedures. He ensures that all key personnel received the required information.

Part IV Changes and Corrective Actions outlines the procedure for implementing changes and corrective actions.

4.2.2 Information Flow to Regulator

All reports of sampling, tests, inspections and construction records will be maintained in the White Mesa Project files. These documents shall be available to the Regulatory Project Manager at all times. The Regulatory Project Manager shall have the right to inspect and reproduce any documents as needed.

A list of the required reports is shown on Table I. These reports will be kept in the White Mesa Project Files.

TABLE I

REQUIRED REPORTS

Report Type	Frequency	Originator	Approval
Construction Activities	Daily during Contruction	QC Technician	QC Officer
Sampling, Field and Laboratory Testing	Report for each respective test	QC Technician	QC Officer
*Compliance Report	Upon completion of Construction Segment	QC Officer	Site Manager, Regulatory Project Mgr
*Final Construction Report	After completion of the Construction Project	QC Officer Site Manager	Regulatory Project Mgr

* Reports to be submitted to the Regulator

PART III SURVEYS, INSPECTIONS, SAMPLING AND TESTING

1.0 SCOPE

The following items are covered in this Section:

(1) Methods and procedures for surveys, inspections, sampling and testing during various construction tasks.

(2) The necessary qualifications of individuals performing surveys, inspections, sampling and testing.

(3) The number and type of surveys, inspections and/or tests to be conducted.

2.0 METHODS AND PROCEDURES

2.1 Quality Control Procedures

Quality Control Procedures have been written to meet the following objectives:

(1) To describe the equipment, calibration and methods/procedures to be followed in performing surveys, sampling and testing.

(2) To describe the procedures to observe construction activities.

(3) To describe the procedures for monitoring.

A list of Quality Control Procedures is given in Part IV (Table III).

All Quality Control Procedures for sampling, testing, and monitoring shall be conducted by the QC Officer and/or Quality Control Technicians. The results shall be reviewed and approved by the QC Officer before being delivered to the Document Control Officer for reproduction, distribution, and filing.

All Boundary Surveys shall be made and documented by a Registered Land Surveyor. Construction surveys shall be made and documented by appropriately trained Quality Control Field Technicians.

3.0 FREQUENCY AND TYPE

The number and type of survey, observations, inspections and/or tests are specified in the Plans and Specifications.

PART IV CHANGES AND CORRECTIVE ACTIONS

1.0 SCOPE

The methodology for dealing with changes and corrective actions is detailed in this Section.

2.0 AUTHORITY OF PERSONNEL

The Site Manager and/or the Quality Control Officer shall have the authority to reject material or work, to require removal or replacement, to specify and require appropriate actions if it is determined that the Quality Control/Quality Assurance, personnel, instructions, controls, tests, records are not conforming to the Plans and Specifications.

3.0 METHODOLOGY

3.1 Field and Design Changes

Changes in locations or alignments of construction features that do not alter design concepts shall be approved by the Site Manager or his designated representative with the agreement of the Regulatory Project Manager. These changes will require a Field Change Order (Form F-25).

Changes in design concepts shall be approved and documented by the Design Engineer, shall be approved by the Site Manager, and shall be submitted to the Regulator for approval. These changes will require a Design Change Order (Form F-26).

All changes will be recorded in the Final Construction Report including "as-built" Drawings for the Work.

3.2 Corrective Actions

The Quality Control Officer shall require corrective actions if tests and observations indicate the Work is not conforming to the intent of the Plans and Specifications. Appropriate corrective actions shall be determined by reviewing pertinent Quality Control Records. Contemplated corrective actions shall be brought to the attention of the Site Manager, the Construction Superintendent and the Regulatory Project Manager.

PART V DOCUMENTATION

1.0 SCOPE

Documentation requirements shall include the following:

(1) The identification of the person who has authority to provide for the submittal and/or storage of all survey, test and inspection reports.

(2) Specification of reporting requirements, forms, formats, and distribution of reports.

(3) A description of record keeping to document construction methods and results, surveys, sampling, testing and inspection of construction. Samples of forms and records shall be included.

(4) Documentation of corrective actions.

2.0 PERSONNEL

2.1 <u>Document Control Officer</u>

2.1.1 Duties

The Document Control Officer shall be appointed by the Site Manager. His responsibilities shall include:

(1) Maintaining permanent files for the Construction Project. All tests, surveys, monitoring and report originals shall be maintained in the project files.

(2) Instituting and overseeing data reproduction and distribution. A distribution list shall be prepared for each project number and shall be reviewed and approved by the QC Officer.

3.0 FORMS

All test results, sampling, surveys, and monitoring shall be documented on the forms for those particular procedures where applicable. Specific surveys require a notebook prepared for data recording. Each Construction Quality Control Field Technician shall complete a Construction Activities Report for each day's work. Forms shall be completed so that all important data are recorded. Data required on all forms and notebooks includes project number, date, technician's signature, and the signature of the supervisor or his designee, who has reviewed and approved the work. The Document Control Officer shall return all incomplete forms to the appropriate supervisor to be properly filled out.

A list of forms is given in Table II. Forms F-23, F-25 and F-26 are included in this section. The other forms are attached to the respective Quality Control Procedures.

TABLE II
LIST OF FORMS

Form No.	Title
F-1	Construction Activities Report
F-2	Excavation Observation Report
F-3	Fill Observation and Testing
F-4	Soil Sampling Log
F-5	Field Density Test (Sand Cone, Balloon)
F-6	Moisture-Density Worksheet
F-7	Laboratory Compaction Test
F-8	Moisture-Density Relationship -1
F-9	Moisture-Density Relationship -2
F-10	Gradation Analysis with Hydrometer Worksheet
F-11	Gradation Analysis Worksheet
F-12	Gradation Test Results
F-13	Atterberg Limits 1-Point Worksheet
F-14	Atterberg, -200, and Density Worksheet
F-15	Atterberg Limits 3-Point Worksheet
F-16	Specific Gravity Worksheet
F-17	Summary of Laboratory Tests
F-18	Liner Seam Destructive Tests
F-19	Liner Seam Nondestructive Tests
F-23	Compliance Report
F-25	Field Change Order
F-26	Design Change Order

Form No. F-23

COMPLIANCE REPORT

Project No. ______________ Date ________________

Construction Segment ____

____

Drawing No. ______________

Specification No. _____________

Description of Completed Construction Segment

____

____

____

____

____

____

____

____

____

____

____

____

By: QC Officer ______________________________________

Approvals

Site Manager ____

Regulatory Project Manager _____________________________

Form No. F-25

FIELD CHANGE ORDER

Project No. ______________ Date ________________

Drawing No. ______________

Specification No. ____________

Design feature

____

____

____

____

Modifications

____

____

____

____

Reason

____

____

____

____

Initiated by: ________________________________

Approved by: ________________________________

Site Manager

Form No. F-26

DESIGN CHANGE ORDER

Project No. ______________ Date ________________

Drawing No. ______________

Specification No. ____________

Design feature

____

____

____

Change in design

____

____

____

____

Reason

____

____

____

Initiated by: ______________________________________

Approvals:

Site Manager ______________________________________

Regulatory Project Manager ___________________________

Design Engineer ____________________________________

PART VI QUALITY CONTROL PROCEDURES

1.0 GENERAL

The Quality Control Procedures listed in Table III are included in this Section.

TABLE III
LIST OF QUALITY CONTROL PROCEDURES

Procedure No.	Title
QC-1-WM	In-Place Density Tests
QC-2-WM	Laboratory Standard Compaction Test
QC-3-WM	Determining the Specific Gravity of Soils
QC-4-WM	Soil Classification for Engineering Purposes
QC-5-WM	Soils Finer than No. 200 Sieve
QC-6-WM	Particle Size Analysis
QC-7-WM	Moisture Content of Soils
QC-9-WM	Sampling of Aggregates and Soils
QC-10-WM	Observation of Excavation
QC-11-WM	Observation of Fill Placement
QC-12-WM	Observation of Reclamation
QC-13-WM	Monitoring of Erosion Monuments
QC-14-WM	Monitoring of Piezometers
QC-15-WM	Monitoring of Surface Movement Monuments
QC-16-WM	Monitoring of Temporary Settlement Plates
QC-17-WM	Monitoring Sediment Control Barriers
QC-19-WM	HDPE Liner Seam Integrity
QC-26-WM	Determining Clay and Random Fill Thickness
QC-27-WM	Preparation of Riprap Reference Stockpiles
QC-28-WM	Determining Riprap Thickness for Reclamation Covers and Channels

PROCEDURE QC-1-04WM

QUALITY CONTROL PROCEDURE
FOR
IN-PLACE DENSITY TESTS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	[signature], P.E.

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1.0 EQUIPMENT

1.1 Nuclear Gauge Method

(1) Gauge - Instrument containing all electronic modules, rechargeable battery packs and nuclear sources.

(2) Reference Standard Block - This block serves to establish the standard counts against which all measurements are proportional; second, it serves as a known repeatable reference for calibration.

(3) Drill Rod - Used with a hammer to punch a hole required for direct transmission measurements.

(4) Scraper Plate - Functions as a guide for the drill rod and aids in smoothing the test site and removing the drill rod.

(5) AC and DC Battery Chargers - The AC charger will operate from 115 or 230 volt power. The DC Charger will operate when plugged into a 12 volt, negative ground, vehicle cigarette lighter.

1.2 Sand Cone Method

(1) Density Apparatus - Jar with volume of approximately 1 gallon, metal cone detachable from jar, square metal plate with flanged center hole.

(2) Sand - clean, dry, uniform, uncemented, durable and free-flowing.

(3) Balances - (a) with a capacity of 10 kg accurate to 2g below 7000g and 3g above 7000g, (b) with a capacity of 2000g accurate to 0.1%.

(4) Drying Equipment - Oven as specified in ASTM Method D-2216.

(5) Miscellaneous Equipment - Digging tools, small brush, plastic bags or airtight container.

1.3 Rubber Balloon Method

(1) Calibrated Vessel - A calibrated vessel designed to contain a liquid within a relatively thin, flexible, elastic membrane (rubber balloon) for measuring the volume of the test hole under the conditions of this method. The apparatus shall be equipped so that an externally controlled pressure

or partial vacuum can be applied to the contained liquid. It shall be of such weight and size that it will not cause distortion of the excavated test hole and adjacent test area during the performance of the test. Provision shall be made for placing weights (surcharge) on the apparatus. There shall be a volume indicator for determining to the nearest 0.00025 cu. ft. any change in volume of the test hole. The flexible membrane shall be of such size and shape as to fill the test hole completely without wrinkles or folds when inflated within the test hole, and its strength shall be sufficient to withstand such pressure as is necessary to ensure complete filling of the test hole.

The description and requirements given in this Section are intended to be nonrestrictive. Any apparatus using a flexible (rubber) membrane and liquid that can be used to measure the volume of a test hole in soil under the conditions of this method to an accuracy within 1.0 percent is satisfactory.

(2) Balances - A balance or scale of approximately 20 kg capacity accurate to 1.0g (or 35 lb capacity accurate to 0.01 lb) and a balance of 1600g capacity accurate to 0.1g.

(3) Drying Apparatus - A stove, oven or other apparatus proven suitable for drying soil or moisture samples.

(4) Miscellaneous Equipment - Small pick, chisels, or spoons for digging test holes; plastic bags, buckets with lids, or other suitable metal containers that can be closed for retaining the soil taken from the test holes; thermometer for determining temperature of water; small paint brush.

1.4 Drive Cylinder Method

(1) Density Apparatus - The essential elements of the apparatus consist of calibrated drive cylinders, a detachable driving head and a drive hammer. Any size of drive cylinders and driving apparatus which will give an accuracy within one percent when checked by the sand cone or other acceptable in-place density method is satisfactory.

(2) Balances - Any standard laboratory balance with a capacity equal to the mass of soil and cylinder and sensitive to 0.1g is satisfactory for obtaining the net mass of the sample.

(3) Drying Equipment - A stove or oven for drying moisture samples.

(4) Miscellaneous Equipment - A shovel is required for digging out the cylinders after they have been driven into the soil and a steel straightedge with one edge sharpened is needed to cut the ends of the sample flush with the cylinder. Seamless tin cans with lids, or other suitable containers, for retaining the moisture samples are necessary, as well as miscellaneous small tools.

2.0 METHODOLOGY

2.1 Objective

The objective of this test is to determine the density of in-place soils/aggregates/tailings/sludges or a combination of these materials.

2.2 Test Procedures

The tests shall be conducted in accordance with the Standard Procedures referenced below. The percentage of maximum dry density shall be calculated based on the maximum dry density obtained from a compaction test performed on that material type. Compaction tests shall be performed for each material type tested and at intervals in accordance with the Specifications.

2.2.1 Nuclear Gauge Method

ASTM D-2922-81 (Density)
ASTM D-3017-78 (Moisture)
AASHTO T-238-79 (Density)
AASHTO T-239-76 (Moisture)

2.2.2 Sand Cone Method

ASTM D-1556-82
AASHTO T-191

2.2.3 Rubber Balloon Method

ASTM D-2167-66 (1972)
AASHTO T-205-64 (1982)

2.2.4 Drive Cylinder Method

AASHTO T-204-64 (1982)

2.3 References

(1) Annual Book of ASTM Standards Volume 04.08
(2) AASHTO Part 2, Tests

3.0 CALIBRATION

3.1 Nuclear Gauge Method

The nuclear gauge shall be calibrated daily or in accordance with the manufacturer's recommendations.

3.2 Sand Cone Method

The density equipment, sand and balances shall be calibrated at least monthly.

3.3 Water Balloon Method

Density equipment and balances shall be calibrated at least monthly.

3.4 Drive Cylinder Method

Balances for moisture and density determinations shall be calibrated at least monthly.

4.0 REPORTING

4.1 Forms

- F-3 Fill Observation and Testing (for Nuclear Density Test Data)
- F-5 Field Density Tests (Sand Cone and Water Balloon)
- F-6 Moisture-Density Worksheet

4.2 Records

Completed reports shall be maintained in the White Mesa Project Files.

F-3 FILL OBSERVATION & TESTING

Project No. ____________

Technician ____________ Date ____________

Approved By ____________ Daily Report No. ____________

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY MAXIMUM DRY DENSITY (pcf)	LABORATORY OPTIMUM MOISTURE CONTENT (%)	FIELD DRY DENSITY (pcf)	FIELD MOISTURE CONTENT (%)	PERCENT COMPACTION	SOIL TYPE

SPECIFICATION COMPACTION & MATERIAL

TYPE AND NUMBER OF EARTH MOVING UNITS

TYPE AND NUMBER OF COMPACTION UNITS

NUMBER OF PASSES THICKNESS OF LIFT

METHOD OF ADDING MOISTURE

☐ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

F-5 FIELD DENSITY TEST (Sand cone, Balloon)

Technician ______________________ Project No. ______________________

Approved By ______________________ Date ______________________

Sample No. ______________________

Description of Soil ______________________

Laboratory Data from Field Test

Sand-cone method	Balloon method
Wt. of wet soil + can ____________	Wt. of wet soil + can ____________
Wt. of can ____________	Wt. of can ____________
Wt. of wet soil, W' ____________	Wt. of wet soil, W' ____________
Wt. of wet soil + pan ____________	Wt. of wet soil + pan ____________
Wt. of dry soil + pan ____________	Wt. of dry soil + pan ____________
Wt. of pan ____________	Wt. of pan ____________
Wt. of dry soil ____________	Wt. of dry soil ____________
Water content, w% ____________	Water content, w% ____________

Field Data

Sand-cone method	Balloon method
Type of sand used ____________	Correction factor CF = ____________
Unit wt. of sand, γ_{sand} = ____________ pcf	Final scale reading ____________ cu ft
Wt. of jug + cone before use ____________ lb	Initial scale reading ____________ cu ft
Wt. of jug + cone after use ____________ lb	Vol. of hole, V_h' ____________ cu ft
Wt. of sand used (hole + cone) ____________ lb	Vol. of hole = V_h' (CF) ____________ cu ft
Wt. of sand in cone (from calibration) ____________ lb	
Wt. of sand in hole, W ____________ lb	
Vol. of hole, $V_h = W/\gamma_{sand}$ = ____________ cu ft	

Density of Soil

Wet density $\gamma_{wet} = W'/V_h$ = ____________ pcf Dry density $\gamma_{dry} = \gamma_{wet}/(1 + w)$ = ____________ pcf

TECHNICIAN ____________________ PROJECT NO. ____________________

APPROVED B ____________________ DATE ____________________

F-6 MOISTURE & DENSITY WORKSHEET

IDENTIFICATION		TEST RESULTS				MOISTURE DETERMINATION				DENSITY DETERMINATION				
SAMPLE NO.	SAMPLE TYPE	% MOISTURE	DRY DENSITY PCF	UNIF. CLASS	DESCRIPTION	DISH NO.	WT. OF WET SOIL AND DISH	WT. OF DRY SOIL AND DISH	WT. OF DISH	SAMPLE LENGTH IN.	SAMPLE DIAMETER IN.	WT. OF WET SOIL AND TARE	WT. OF TARE	WT. OF WET SOIL

PROCEDURE QC-2-04WM

QUALITY CONTROL PROCEDURE
FOR
LABORATORY STANDARD COMPACTION TEST

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
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1.0 EQUIPMENT

(1) Cylindrical molds, 4-inch and 6-inch diameters.
(2) Rammer, either manual or mechanical, able to drop 12 inches and weighing 5.5 lb.
(3) Balance or scale, a) with a capacity of 20 kg, accurate to 1.0g, and b) with a capacity of 1000g, accurate to 0.01g
(4) Oven, 110 ±5°C.
(5) Straightedge.
(6) Sieves: 3-inch, 3/4-inch, and No. 4.
(7) Mixing tools: mixing pan, spoon or spatula.
(8) Moisture containers.

2.0 METHODOLOGY

2.1 Objective

The objective is to determine the relationship between the material moisture content and density when compacted in a mold of a given size with a 5.5 lb. (2.49 kg) rammer dropped from a height of 12 inches (305 mm) to determine maximum density and optimum moisture content for each soil type encountered in fill placement operations.

2.2 Test Procedure

The procedure for performing this test shall be in accordance with ASTM Method D-698-78.

2.3 References

ASTM Volume 04.08 D-698-78.

3.0 REPORTING

3.1 Forms

F-7 Laboratory Compaction Test
F-8 Moisture-Density Relationships -1
F-9 Moisture-Density Relationships -2

3.2 Records

Completed Reports shall be maintained in the White Mesa Project Files.

F-7 LABORATORY COMPACTION TEST
WORKSHEET

TECHNICIAN ____________________ PROJECT NO. ____________________
APPROVED BY ____________________ DATE ____________________

SAMPLE NO. ____________________
SAMPLE DESCRIPTION ____________________ COLOR ____________________
ASTM D 698-78 ☐ ASTM D 1557-78 ☐ METHOD: A B C D OTHER ____________________

TEST DATA

POINT NUMBER	1	2	3	4	5	6	7
WT. OF SOIL AND TARE							
AMOUNT OF ADDED WATER, WT.							
WT.OF SOIL, WATER & TARE							
AMOUNT OF WATER ADDED, VOL.							
WT. OF MOLD AND WET SOIL							
WT. OF MOLD							
WT.OF WET SOIL							
WET DENSITY PCF							
DISH NUMBER							
WT. OF DISH AND WET SOIL							
WT. OF DISH AND SOIL							
WT. OF DISH							
WT. OF WATER							
WT. OF DRY SOIL							
MOISTURE CONTENT %							
DRY DENSITY PCF							

AT WHICH POINT WAS SOIL PUMPING ? ______

AT WHICH POINT WAS SOIL BLEEDING ? ______

OPTIMUM MOISTURE CONTENT ____________ %

MAXIMUM DRY DENSITY ____________ pcf

ROCK CORRECTION DATA

(WEIGHTS ARE PER POINT)

+#4 __________ %= __________ lb.

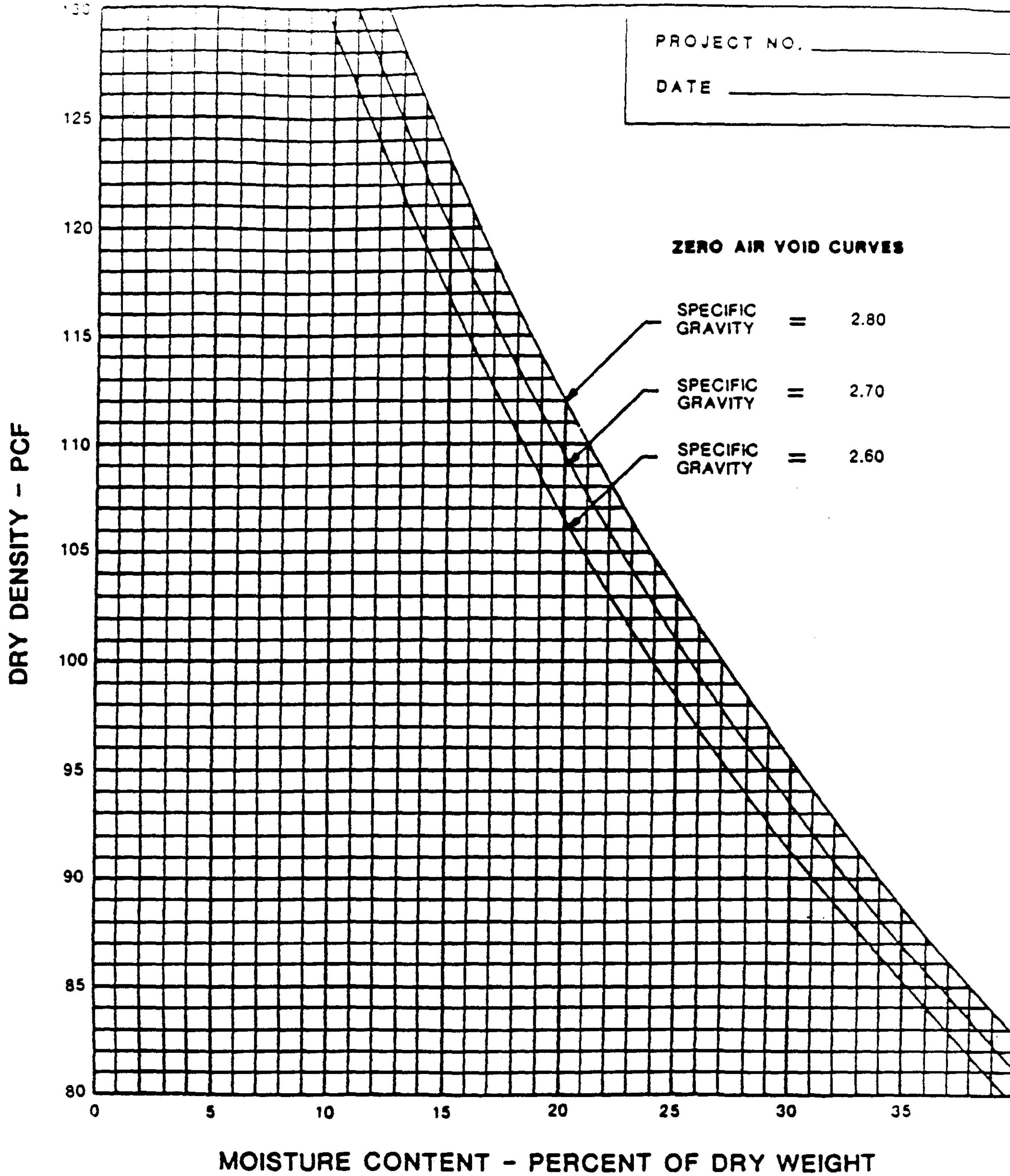
-#4 __________ %= __________ lb.

100%= __________ lb.

COMMENTS: ____

PROJECT NO. ____________

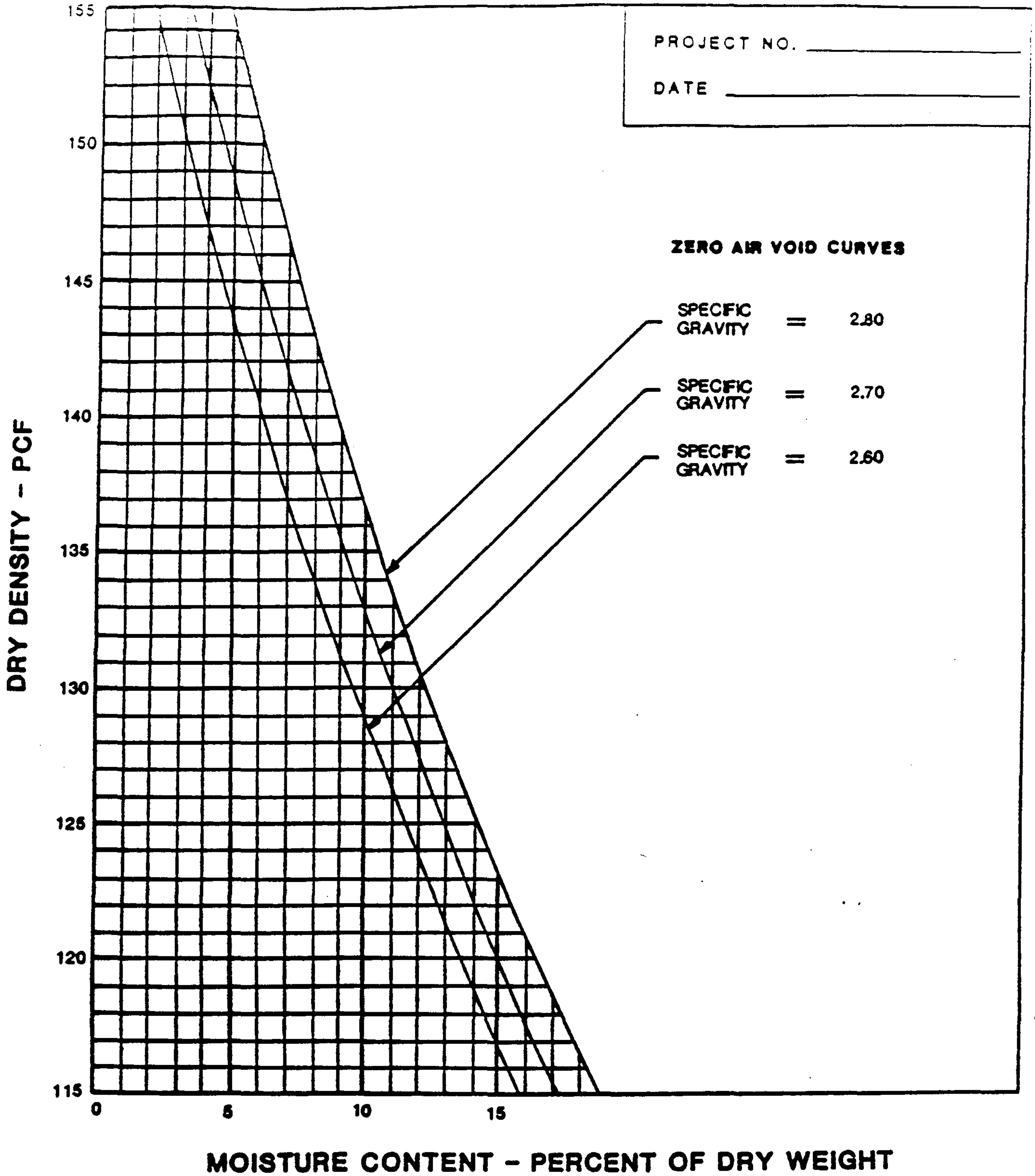
DATE ____________



LOCATION :	**F-8 MOISTURE-DENSITY RELATIONSHIPS - 1**
SAMPLE NO:	
MATERIAL DESCIPTION:	
MAX. DRY DENSITY : ______ PCF OPT. MOIST. CONTENT : ______ %	PROCEDURE:
LIQUID LIMIT : ______ PLASTICITY INDEX : ______	TECHNICIAN:
GRAVEL : ______ % SAND : ______ % SILT AND CLAY (-200) : ______ %	APPROVED BY:

PROJECT NO. ______________

DATE ______________



LOCATION :	**F-9 MOISTURE-DENSITY RELATIONSHIPS - 2**
AMPLE NO:	
MATERIAL DESCIPTION:	
MAX. DRY DENSITY : ______ PCF OPT. MOIST. CONTENT : ______ %	PROCEDURE :
LIQUID LIMIT : ______ PLASTICITY INDEX : ______	TECHNICIAN:
GRAVEL : ____ % SAND : ____ % SILT AND CLAY (-200) : ____ %	APPROVED BY:

QUALITY CONTROL PROCEDURE
FOR
DETERMINING THE
SPECIFIC GRAVITY OF SOILS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
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1.0 EQUIPMENT AND MATERIALS

1.1 Equipment

(1) Pycnometer (100 ml to 500 ml volumetric flask, Class "A")
(2) Balance, sensitive to 0.01g
(3) Thermometer, range 0-50°C
(4) Magnetic stirrer
(5) De-airing apparatus (vacuum pump)
(6) Beaker and cover.

1.2 Materials

(1) Distilled water.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to determine the specific gravity of soils by means of a pycnometer.

2.2 Procedure

Tests are to be run in accordance with the procedure outlined in ASTM D-854-83.

2.3 References

(1) ASTM D-854-83, Annual Book of ASTM Standards, Volume 04.08.

3.0 REPORTING

3.1 Laboratory Form

F-16 Specific Gravity Test Worksheet

3.2 Report Form

F-17 Summary of Laboratory Test Results

3.3 Records

Completed reports shall be maintained in the White Mesa Project Files.

TECHNICIAN ______________________ **F-16 SPECIFIC GRAVITY** PROJECT NO. ______________________

APPROVED BY ______________________ **WORKSHEET** DATE ______________________

SPECIFIC GRAVITY OF SOILS (G_s) ASTM D-854		TRIAL 1	TRIAL 2	TRIAL 1	TRIAL 2	TRIAL 1	TRIAL 2
RUN BY							
SAMPLE NO.							
DESCRIPTION							
PREP. DISH							
FLASK NO.							
TEMPERATURE OF WATER AND SOIL, T, °C							
DISH NO.							
DISH + DRY SOIL, g							
DISH, g							
DRY SOIL, g	A						
FLASK + WATER @ T, °C, g	B						
A + B, g							
FLASK + WATER + SOIL, g	C						
DISPLACED WATER, (A + B) - C, g							
CORRECTION FACTOR FOR TEMP. T, °C	K						
$G_s = (A \cdot K) \div (A + B - C)$							
G_s, TRIAL 1 - G_s, TRIAL 2*							
AVERAGE G_s							

REMARKS ____

*The difference between G_s values for the two trials should be ≤ 0.050.

F-17 SUMMARY OF LABORATORY TEST RESULTS

TECHNICIAN ____________________ PROJECT NO. ____________________

APPROVED BY ____________________ DATE ____________________

SAMPLE NO.	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		SOIL OR BEDROCK TYPE	COMMENTS
			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)		

PROCEDURE QC-4-04WM

QUALITY CONTROL PROCEDURE
FOR
SOIL CLASSIFICATION FOR ENGINEERING PURPOSES

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	[signature], P.E.

INDEX

Page No.

1.0 EQUIPMENT AND MATERIALS

1.1 Equipment

(1) Sample preparation apparatus - see ASTM D-421 or D-2217.
(2) Particle size analysis apparatus - see ASTM D-422 or D-1140.
(3) Liquid limit apparatus - see ASTM D-4318.
(4) Plastic limit apparatus - see ASTM D-4318.
(5) Classification chart.

1.2 Materials

(1) No. 40 Sieve.
(2) Moisture containers.
(3) Balance, accurate to 0.01g.
(4) Oven, 100 $\pm 5^{o}$c.

2.0 METHODOLOGY

2.1 General

The objective is to determine the classification of soils for engineering purposes in accordance with the Unified Soils Classification System based on particle size and Atterberg limits (liquid limit and plasticity index) of the soil.

2.2 Test Procedures

Analysis to be performed in accordance with ASTM D-2487-83. Other procedures are as follows:

(1) Sample preparation ASTM D-421-85, or
ASTM D-2217-85

(2) Particle size analysis ASTM D-422-63, or
ASTM D-1140-54

(3) Liquid limit ASTM D-4318-84

(4) Plastic limit ASTM D-4318-84.

Based on the test results, the soil shall be classified according to the Unified Soil Classification System.

2.3 <u>References</u>

Annual Book of ASTM Standards, Volume 04.08, D-2487-83. Also the following ASTM standard procedures:

(1) D-421-85
(2) D-2217-85
(3) D-422-63
(4) D-1140-54
(5) D-4318-84

3.0 REPORTING

3.1 <u>Forms</u>

F-10 Gradation Analysis with Hydrometer Worksheet
F-11 Gradation Analysis Worksheet
F-12 Gradation Test Results
F-13 Atterberg Limits 1-Point Worksheet
F-14 Atterberg, -200, Moisture and Density Worksheet
F-15 Atterberg Limits 3-Point Worksheet
F-17 Summary of Laboratory Test Results

3.3 <u>Records</u>

Completed reports shall be maintained at the White Mesa Project Files.

WORKSHEET

TECHNICIAN ____________ PROJECT NO ____________

APPROVED BY ____________ DATE ____________

SAMPLE NO. ____________

VISUAL DESCRIPTION: ____________

SAMPLE PREPERATION

RUN BY ______ SIEVING TIME ______

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED						
DRY WT. PASSING						
% OF TOTAL PASSING						
	W% = ______					

SAMPLE WEIGHTS	WET	DR
TOTAL SAMPLE	______	______
RETAINED ON NO. 4	______	______
PASSING NO. 4	______	______

SIEVE AND HYDROMETER ANALYSIS

RUN BY ______ SIEVING TIME ______

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)			
16			
30 (40)			
50			
100			
200			
PAN			—
TOTAL			—

FACTOR = $\frac{W\%}{W}$ = ____________ = ____________

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO SAMPL
DISH NO.				
WT. WET SOIL AND DISH				
WT. DRY SOIL AND DISH				
WT. DISH				
WT. OF DRY SOIL		______ = W		
% MOISTURE				

HYDROMETER ANALYSIS

RUN BY ______

CYLINDER NO. ____________ SPECIFIC GRAVITY ____________ DISPERSING AGENT ____________

DISH NO. ____________ DATE ____________ AMOUNT ____________ ml DATE CALIB ____________

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—		—	—	
	STOP MIX	—	—	—	—		—	—	
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

GRAVEL ____________ % SAND ____________ % CLAY-SLIT ____________ %. STORAGE LOCATION ____________

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

ECHNICIAN ______________ PPROVED BY ______________

F-11 GRADATION ANALYSIS
WORKSHEET

PROJECT NO. ______________ DATE ______________

AMPLE NO.					
ESCRIPTION*					
JN BY	RUN BY	RUN BY	RUN BY	RUN BY	RUN BY
ISH NO.					
RY SOIL & DISH					
200 SOIL & DISH					
ISH WT.					
RY SOIL WT.					

EIVE IZE		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS
5																		
3																		
1-1/2																		
3/4																		
3/8																		
#4																		
#8																		
#16																		
#30																		
#50																		
#100																		
#200																		
AN																		

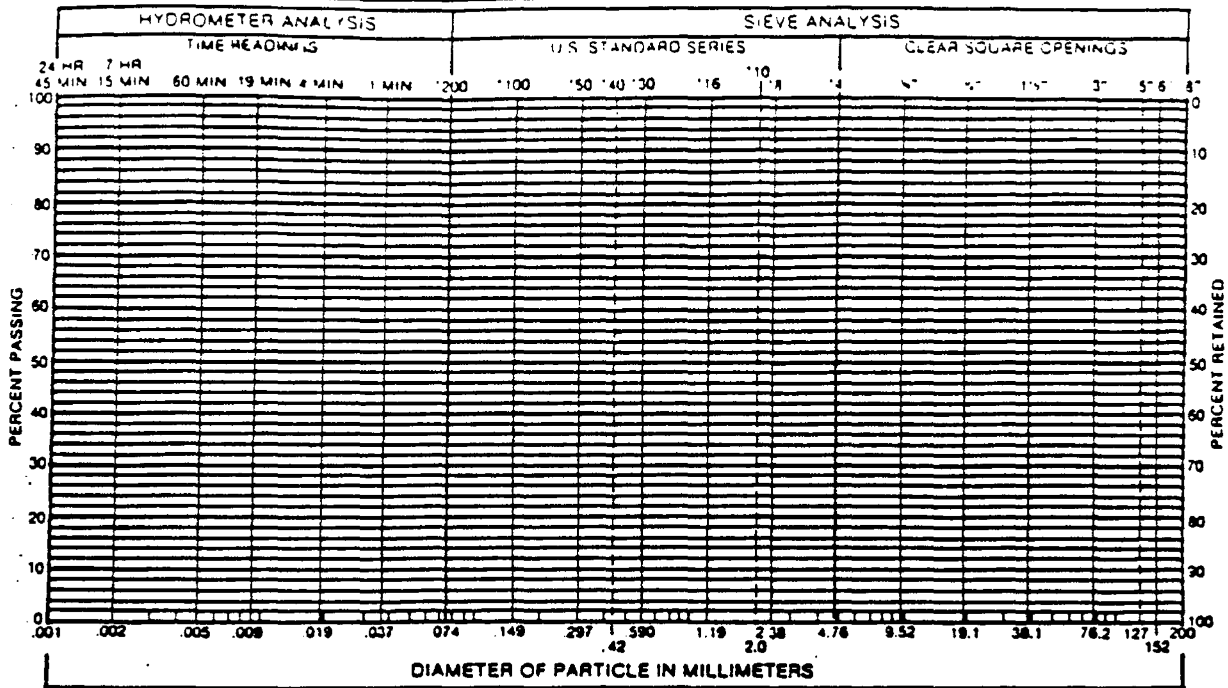
RAVEL %					
AND %					
LAY-SILT %					
EMARKS:					

REPLACE WITH #10 & #40 SEIVE WHEN REQUESTED

F-12 GRADATION TEST RESULTS

TECHNICIAN ______________________ PROJECT NO. ______________________

APPROVED BY ______________________ DATE ______________________

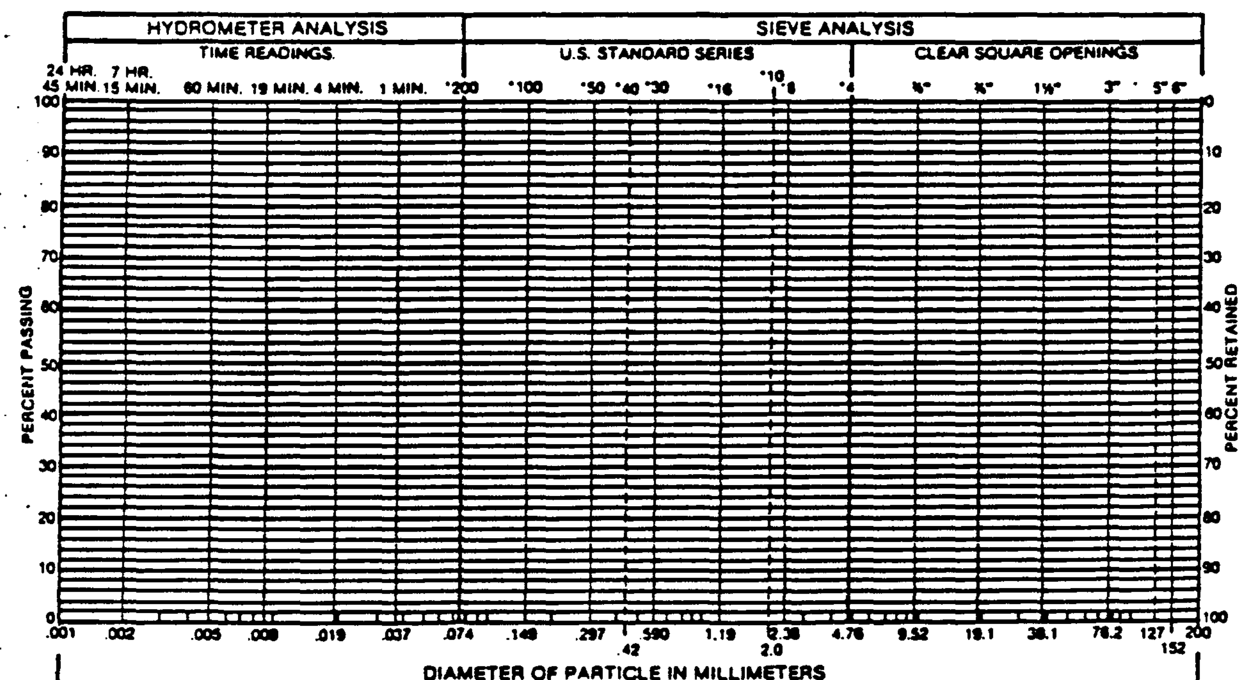


CLAY TO SILT	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM



CLAY TO SILT	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

TECHNICIAN: ______________________

APPROVED BY: ______________________

F-13 ATTERBERG LIMITS 1-POINT WORKSHEET

PROJECT NO. ______________________

DATE ______________________

HOLE / DEPTH	/		/		/		/		/	
SAMPLE NO. / RUN BY	/		/		/		/		/	
PREP. DISH / TRAY LOCAT.	/		/		/		/		/	
NO. OF BLOWS	P.L.		P.L.		P.L.		P.L.		P.L.	
DISH NO.										
WT. WET SOIL & DISH										
WT. DRY SOIL & DISH										
WT. OF DISH										
WT. OF WATER										
WT. OF DRY SOIL										
WATER CONTENT Wn										
LIQUID LIMIT LL										
PLASTIC INDEX PI										
CLASSIFICATION										

HOLE / DEPTH	/		/		/		/		/	
SAMPLE NO. / RUN BY	/		/		/		/		/	
PREP. DISH / TRAY LOCAT.	/		/		/		/		/	
NO. OF BLOWS	P.L.		P.L.		P.L.		P.L.		P.L.	
DISH NO.										
WT. WET SOIL & DISH										
WT. DRY SOIL & DISH										
WT. OF DISH										
WT. OF WATER										
WT. OF DRY SOIL										
WATER CONTENT Wn										
LIQUID LIMIT LL										
PLASTIC INDEX PI										
CLASSIFICATION										

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN ______________________ PROJECT NO. ______________________
APPROVED BY ______________________ DATE ______________________

SAMPLE NO. ______________________
SAMPLE DESCRIPTION ______________________ COLOR ______________________
____

ATTERBERG LIMITS PL LL
PREP. DISH __________ RUN BY __________

	PL	LL
NO. OF BLOWS	———	
DISH NO.		
WT. OF WET SOIL & DISH		
WT. OF DRY SOIL & DISH		
WT. OF DISH		
WT. OF WATER		
WT. OF DRY SOIL		
WATER CONTENT		

LIQUID LIMIT, LL __________
PLASTIC INDEX, PI __________

-200
RUN BY __________

DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 __________ %

MOISTURE CONTENT
RUN BY __________

DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

MOISTURE CONTENT __________ %

DENSITY
RUN BY __________

LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY __________ PCF

REMARKS: ______________________
____
____

F-15 ATTERBERG LIMITS 3-POINT WORKSHEET

TECHNICIAN ______________________ PROJECT NO. ______________________

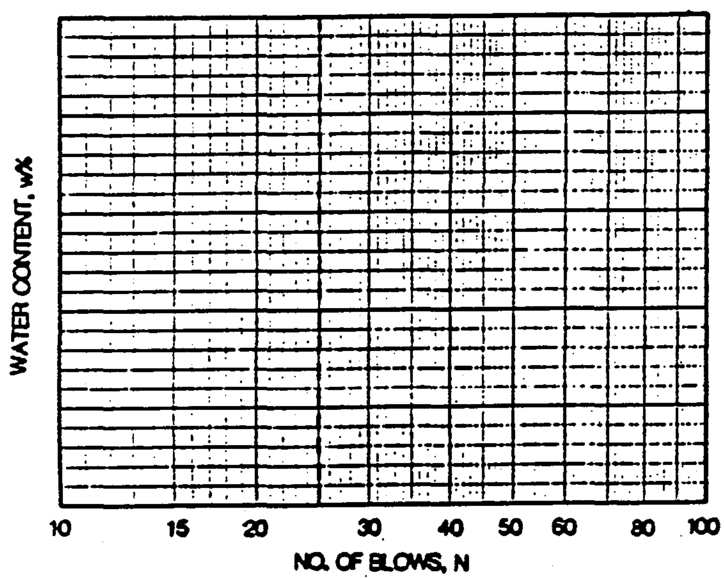
APPROVED BY ______________________ DATE ______________________

SAMPLE NO. ______________________

DESCRIPTION ______________________

LIQUID LIMIT DETERMINATION

TRAY LOCATION					
DISH NO.					
WT. WET SOIL & DISH					
WT. DRY SOIL & DISH					
WT. OF DISH					
WT. OF WATER					
WT. OF DRY SOIL					
WATER CONTENT, w%					
NO. OF BLOWS, N					



LIQUID LIMIT, LL ____________

PLASTIC LIMIT, PL ____________

PLASTICITY INDEX, PI ____________

PLASTIC LIMIT DETERMINATION

TRAY LOCATION					
DISH NO.					
WT. OF WET SOIL & DISH					
WT. OF DRY SOIL & DISH					
WT. OF DISH					
WT. OF WATER					
WATER CONTENT, w% - PL					

F-17 SUMMARY OF LABORATORY TEST RESULTS

TECHNICIAN ______________________ PROJECT NO. ______________________

APPROVED BY ______________________ DATE ______________________

SAMPLE NO.	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		SOIL OR BEDROCK TYPE	COMMENTS
			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)		

PROCEDURE QC-5-C4WM

QUALITY CONTROL PROCEDURE
FOR
SOILS FINER THAN NO. 200 SIEVE

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	[signature], P.E.

INDEX

1.0 EQUIPMENT

Equipment

(1) U.S. Standard Sieve: No. 40, No. 200.
(2) Containers.
(3) Balance, accurate to 0.1g.
(4) Oven, 100 $\pm 5^{o}C$.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to determine the total amount of materials finer than the U.S. Standard No. 200 sieve contained in soil, aggregate or soil aggregate mixture.

2.2 Test Procedure

The tests are to be performed in accordance with the procedure outlined in ASTM D-1140-54.

2.3 References

Annual Books of ASTM Standards, ASTM Procedure D-1140-54 Volume 04.08.

3.0 REPORTING

3.1 Forms

F-11 Gradation Analysis Worksheet
F-14 Atterberg, -200, Moisture and Density Worksheet
F-17 Summary of Laboratory Test Results

3.2 Records

Completed reports shall be maintained in the White Mesa Project Files.

F-11 GRADATION ANALYSIS WORKSHEET

ECHNICIAN ____________________ PROJECT NO. ____________________

PPROVED BY ____________________ DATE ____________________

AMPLE NO.						
ESCRIPTION						
UN BY ______	RUN BY ______	RUN BY ______	RUN BY ______	RUN BY ______	RUN BY ______	

ISH NO.						
RY SOIL & DISH						
00 SOIL & DISH						
ISH WT.						
RY SOIL WT.						

EIVE IZE		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS
6																		
3																		
1-1/2																		
3/4																		
3/8																		
#4																		
#8																		
#16																		
#30																		
#50																		
#100																		
#200																		
AN																		

RAVEL %						
AND %						
LAY-SILT %						
EMARKS:						

REPLACE WITH #10 & #40 SEIVE WHEN REQUESTED

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN: ______________________ PROJECT NO. ______________________
APPROVED BY ______________________ DATE ______________________

SAMPLE NO. ______________________
SAMPLE DESCRIPTION ______________________ COLOR ______________
____

ATTERBERG LIMITS

PREP. DISH ____________ RUN BY ____________

	PL	LL
NO. OF BLOWS	——	
DISH NO.		
WT. OF WET SOIL & DISH		
WT. OF DRY SOIL & DISH		
WT. OF DISH		
WT. OF WATER		
WT. OF DRY SOIL		
TER CONTENT		

LIQUID LIMIT, LL ________
PLASTIC INDEX, PI ________

-200

RUN BY ____________

DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 ____________ %

MOISTURE CONTENT

RUN BY ____________

DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

MOISTURE CONTENT ________ %

DENSITY

RUN BY ____________

LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY ____________ PCF

REMARKS: ______________________

F-17 SUMMARY OF LABORATORY TEST RESULTS

TECHNICIAN ______________________ PROJECT NO. ______________________

APPROVED BY ______________________ DATE ______________________

SAMPLE NO.	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		SOIL OR BEDROCK TYPE	COMMENTS
			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)		

QUALITY CONTROL PROCEDURE
FOR
PARTICLE SIZE ANALYSIS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	Curtis O. Seely, P.E.

INDEX

1.0 EQUIPMENT AND MATERIALS

1.1 Equipment

(1) Balance, accurate to 0.1g.
(2) Stirring apparatus with baffled dispersion cup.
(3) Hydrometer, 152H.
(4) Sedimentation cylinder, 1000 ml.
(5) Thermometer, accurate to ±0.5°C.
(6) U.S. Standard Sieves: 3-inch, 2-inch, 1-1/2-inch, 1-inch, 3/4-inch, 3/8-inch, No. 4, No. 8, No. 16, No. 20, No. 30, No. 40, No. 50, No. 60, No. 100, No. 140, No. 200.
(7) Covered container.
(8) Timing device.
(9) Oven, 110 ±5°C.

1.2 Materials

(1) Dispersing agent: 4% solution $Na(PO_3)$ ("Calgon" is not acceptable).
(2) Distilled water.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to quantitatively determine the distribution of particle sizes of soils, aggregates and soil-aggregate mixtures. The distribution of particle sizes larger than a No. 200 sieve are determined by screening. The distribution of particles smaller than a No. 200 sieve are determined by hydrometer analysis.

2.2 Test Procedures

Particle size analysis shall be performed in accordance with the procedures outlined in ASTM D-422-63.

2.3 References

Annual Book of ASTM Standards, ASTM Procedure D-422-63, Volume 04.08.

3.0 REPORTING

3.1 Forms

F-10 Gradation Analysis with Hydrometer Worksheet
F-11 Gradation Analysis Worksheet
F-12 Gradation Test Results

3.2 Records

Completed reports shall be maintained in the White Mesa Project Files.

WORKSHEET

TECHNICIAN: ______________________ PROJECT NO ______________

APPROVED BY. ______________________ DATE ______________

SAMPLE NO. ______________________

VISUAL DESCRIPTION: ______________________

SAMPLE PREPERATION

RUN BY______ SIEVING TIME______

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED						
DRY WT. PASSING						
% OF TOTAL PASSING						
	W% = ______					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	______	______
RETAINED ON NO. 4	______	______
PASSING NO. 4	______	______

SIEVE AND HYDROMETER ANALYSIS

RUN BY______ SIEVING TIME______

FACTOR = $\frac{W\%}{W}$ = ______________ = ______________

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)			
16			
30 (40)			
50			
100			
200			
PAN			——
TOTAL			——

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO SAMPLE
DISH NO.				
WT. WET SOIL AND DISH				
WT. DRY SOIL AND DISH				
WT. DISH				
WT. OF DRY SOIL		______ = W		
% MOISTURE				

HYDROMETER ANALYSIS

RUN BY______

CYLINDER NO. ____________ SPECIFIC GRAVITY ____________ DISPERSING AGENT: ____________

DISH NO. ____________ DATE ____________ AMOUNT ____________ ml DATE CALIB. ____________

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER
	START MIX	——	——	——	——		——	——
	STOP MIX	——	——	——	——		——	——
	0.5 min							0.050 mm
	1.0 min							0.037 mm
	4.0 min							0.019 mm
	19 min							0.009 mm
	60 min							0.005 mm
	7h 15 min							0.002 mm
	25h 45 min							0.001 mm

GRAVEL ____________% SAND ____________% CLAY-SLIT ____________%. STORAGE LOCATION ____________

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

ECHNICIAN ______ PROJECT NO. ______

APPROVED BY ______ DATE ______

F-11 GRADATION ANALYSIS
WORKSHEET

AMPLE NO.						
ESCRIPTION						
UN BY ______	RUN BY ______	RUN BY ______	RUN BY ______	RUN BY ______	RUN BY ______
ISH NO.						
RY SOIL & DISH						
200 SOIL & DISH						
ISH WT.						
RY SOIL WT.						

EIVE IZE		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS		CUM. WT. RET	% PASS
5																		
3																		
1-1/2																		
3/4																		
3/8																		
#4																		
#8																		
#16																		
#30																		
#50																		
#100																		
#200																		
AN																		

RAVEL %						
SAND %						
CLAY-SILT %						
REMARKS:						

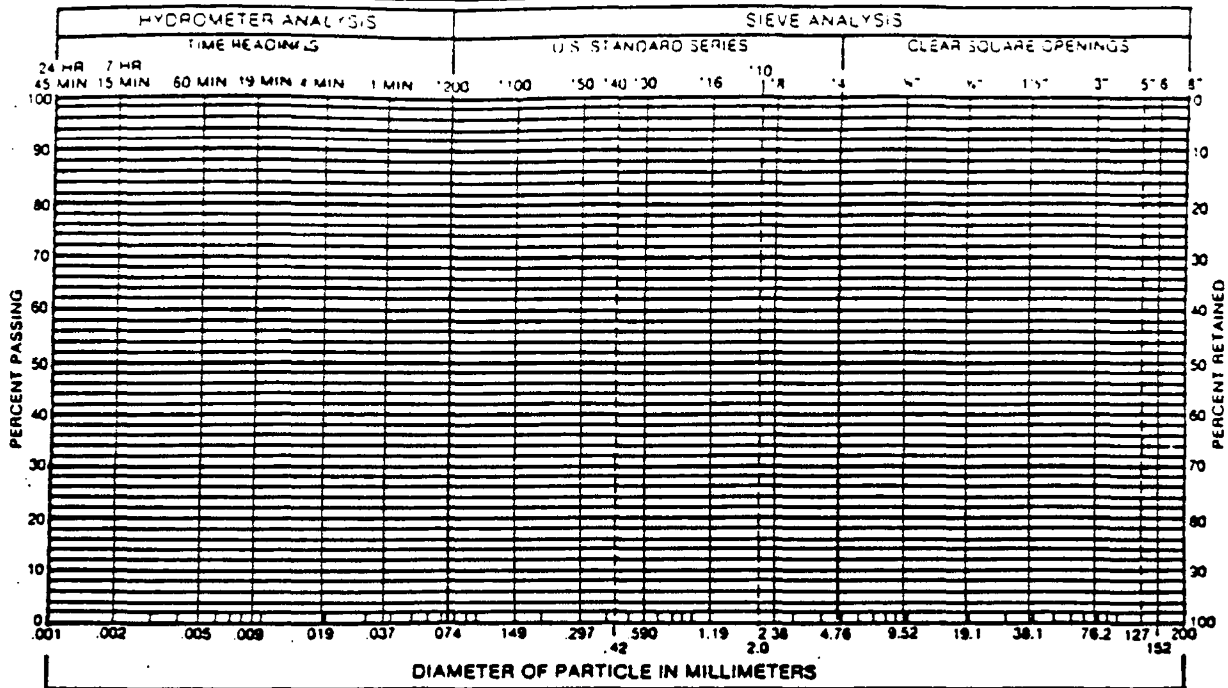
REPLACE WITH #10 & #40 SEIVE WHEN REQUESTED

DA-3-79

F-12 GRADATION TEST RESULTS

TECHNICIAN ______________________ PROJECT NO. ______________________

APPROVED BY ______________________ DATE ______________________

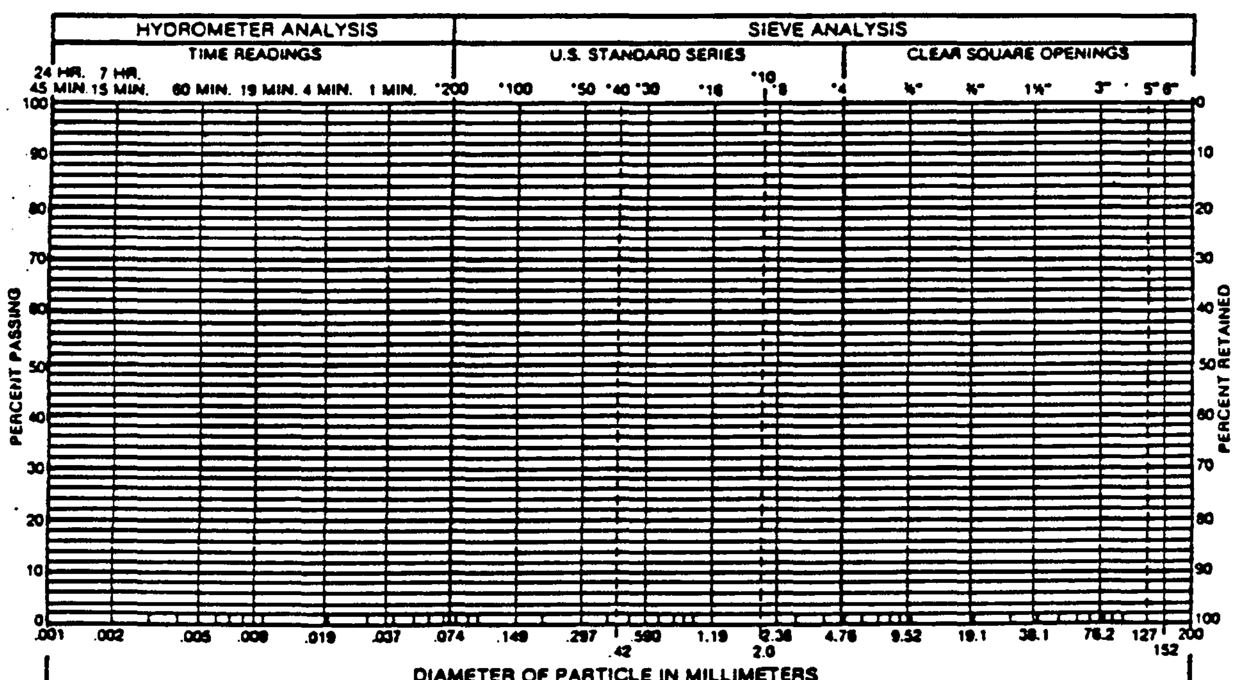


CLAY TO SILT	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM



CLAY TO SILT	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

QUALITY CONTROL PROCEDURE
FOR
MOISTURE CONTENT OF SOILS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	P.E.

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Page No.

1.0 EQUIPMENT AND MATERIALS

Equipment

(1) Balance, accurate to 0.01g.
(2) Moisture containers.
(3) Oven, 100 ±5°C.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to determine the laboratory moisture content of soils, aggregates and soil-aggregate mixtures.

2.2 Test Procedures

The tests shall be performed in accordance with the procedures outlined in ASTM D-2216-80.

2.3 References

Annual Book of ASTM Standards, ASTM Procedure D-2216-80, Volume 04.08.

3.0 REPORTING

3.1 Forms

F-6 Moisture and Density Worksheet
F-17 Summary of Laboratory Test Results

3.2 Records

Completed reports shall be maintained in the White Mesa Project Files.

TECHNICIAN ______________

APPROVED ______________

F-6 MOISTURE & DENSITY WORKSHEET

PROJECT NO. ______________

DATE ______________

IDENTIFICATION		TEST RESULTS				MOISTURE DETERMINATION				DENSITY DETERMINATION				
SAMPLE NO.	SAMPLE TYPE	% MOISTURE	DRY DENSITY PCF	UNIF. CLASS	DESCRIPTION	DISH NO.	WT. OF WET SOIL AND DISH	WT. OF DRY SOIL AND DISH	WT. OF DISH	SAMPLE LENGTH IN.	SAMPLE DIAMETER IN.	WT. OF WET SOIL AND TARE	WT. OF TARE	WT. OF WET SOIL

F-17 SUMMARY OF LABORATORY TEST RESULTS

TECHNICIAN ______________________ PROJECT NO. ______________________

APPROVED BY ______________________ DATE ______________________

SAMPLE NO.	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		SOIL OR BEDROCK TYPE	COMMENTS
			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)		

PROCEDURE QC-9-04WM

QUALITY CONTROL PROCEDURE
FOR
SAMPLING OF AGGREGATES AND SOILS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	[signature], P.E.

INDEX

1.0 EQUIPMENT AND MATERIALS

1.1 Equipment

Shovels, scoops, buckets and brushes for sampling.

1.2 Materials

Plastic and cloth bags, tags and markers for samples and sample identification.

2.0 SAMPLING METHODOLOGY

2.1 Objectives

The objective is to provide standard sampling procedures for obtaining samples of soils, aggregates and soil-aggregate mixtures from stockpiles, truck loads and borrow areas.

2.2 Test Procedure

Sampling shall be done in accordance with the standardized procedures outlined in ASTM D-75-82.

2.3 References

ASTM Procedure D-75-82, Annual Book of ASTM Standards, Volume 04.08.

3.0 REPORTING

3.1 Forms

F-1 Construction Activities Report
F-4 Soil Sampling Log

3.2 Records

Completed daily reports shall be maintained at the White Mesa Project Files.

F-1 CONSTRUCTION ACTIVITIES REPORT

Project No. ______________________

Date ______________________

Technician ______________________

Daily Report No. ______________________

Approved By ______________________

WEATHER CONDITIONS AND TEMPERATURE ______________________

EQUIPMENT ______________________

____

CONSTRUCTION ACTIVITIES ______________________

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OBSERVATIONS

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F-4 SOIL SAMPLING LOG

SAMPLE NO. ____________

DATE ____________

SAMPLED BY ____________

PROJECT NO. ____________

DELIVERED TO LABORATORY

DATE ____________

LOCATION ____________

(EXAMPLE: STOCKPILE, ____________

BORROW AREA, TRUCK, ____________

FILL) ____________

DEPTH ____________

SAMPLE TYPE ____________

(EXAMPLE: LARGE BULK ____________

SAMPLE, DRIVE CYLINDER, ____________

ECT.) ____________

VISUAL CLASSIFICATION ____________

INTENDED USE ____________

(EXAMPLE: CLAYEY BORROW, ____________

RANDOM FILL, ____________

ETC.) ____________

TESTING PROGRAM ____________

(EXAMPLE: STANDARD COMPACTION TEST, ____________

ATTERBERG LIMITS, ____________

ETC.) ____________

QUALITY CONTROL PROCEDURE
FOR
OBSERVATION OF EXCAVATION

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	Curtis O. Sealy, P.E.

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1.0 EQUIPMENT

Tape measure, notebook(s), hand level.

2.0 METHODOLOGY

2.1 Objectives

The objectives are to monitor excavation by visual observation, measurement and other means, and to compare the construction with the requirements of the Specifications.

2.2 Procedure

The location, dimensions, slopes and grades of excavations shall be documented. Soils excavated and exposed in excavations shall be described in accordance with the Unified Soils Classification system. Excavation of raffinate crystals, tailings, contaminated soils, scrap and neutralized sludge shall be monitored in accordance with the appropriate Specifications. Preparation of areas to receive compacted fill shall be observed for stripping of organics and other deleterious materials and the condition of the soil or rock surfaces prior to fill placement shall be noted. Volumes of excavated soils shall be estimated by tabulating load counts for materials removed.

2.3 Frequency of Observations

Observation of excavations shall be conducted on a routine basis established by the appropriate Specifications as the construction proceeds.

2.4 References

(1) 1986 Annual Book of ASTM Standards, Volume 04.08.
(2) Geotechnical Quality Control: Low Level Radioactive Waste and Uranium Mill Tailings Disposal Facilities, Johnson, Spigolon, and Lutton, 1983, NUREG/CR-3356.

3.0 REPORTING

3.1 Forms

Each Technician shall complete the following forms describing observations of construction activities. Unusual conditions shall be brought to the attention of the QC Officer.

F-1 Construction Activities Report
F-2 Excavation Observation Report

3.2 Records

Completed Daily Reports shall be maintained in the White Mesa Project Files.

F-1 CONSTRUCTION ACTIVITIES REF .RT

Project No. ______________

Date ______________

Technician ______________

Daily Report No. ______________

Approved By ______________

WEATHER CONDITIONS AND TEMPERATURE ______________

EQUIPMENT ______________

____

CONSTRUCTION ACTIVITIES ______________

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OBSERVATIONS

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F-2 EXCAVATION OBSERVATION REPORT

Project No. ______________________

Date ______________________

Daily Report No. ______________________

Technician ______________________

Approved By ______________________

EXCAVATION FOR: ______________________

____

LOCATION: ______________________

____

EXCAVATION BASE ELEVATION(S): ______________________

____

MATERIALS EXPOSED IN BASE(S) OF EXCAVATION(S): ______________________

____

REMARKS: ______________________

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QUALITY CONTROL PROCEDURE
FOR
OBSERVATION OF FILL PLACEMENT

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
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1.0 EQUIPMENT

Tape measure, notebooks, hand level.

2.0 METHODOLOGY

2.1 Objectives

The objectives are to monitor placement of earth or rock fills by visual observation, measurement and other means, and to compare the construction with the requirements of the Specifications.

2.2 Procedure

Location, dimensions, slopes and grades of placed fill soils shall be noted. Placed fill soils shall be described in accordance with the Unified Soil Classification system. Classifications, supported by laboratory testing, shall be compared to materials required by the Specifications. During fill placement, thickness of fill lifts, uniformity of soils, compactive effort and other construction details shall be noted. Volumes of placed fill shall be estimated by tabulating load counts for materials handled.

2.3 Frequency of Observations

Observation of fill placement shall be conducted on a periodic basis as the construction proceeds.

2.4 References

(1) 1986 Annual Book of ASTM Standards, Volume 04.08.
(2) Geotechnical Quality Control: Low Level Radioactive Waste and Uranium Mill Tailings Disposal Facilities, Johnson, Spigolon, and Lutton, 1983, NUREG/CR-3356.

3.0 REPORTING

3.1 Forms

Each Technician shall complete the following forms describing observations of construction activities. Unusual conditions shall be brought to the attention of the QC Officer.

F-1 Construction Activities Report
F-3 Fill Observation and Testing

3.2 Records

Completed Daily Reports shall be maintained in the White Mesa Project Files.

F-1 CONSTRUCTION ACTIVITIES REPORT

Project No. ______________

Date ______________

Technician ______________

Daily Report No. ______________

Approved By ______________

WEATHER CONDITIONS AND TEMPERATURE ______________

EQUIPMENT ______________

____

CONSTRUCTION ACTIVITIES ______________

____

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OBSERVATIONS

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F-3 FILL OBSERVATION & TESTING

Project No. ____________

Technician ____________

Date ____________

Approved By ____________

Daily Report No. ____________

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		

SPECIFICATION COMPACTION & MATERIAL

TYPE AND NUMBER OF EARTH MOVING UNITS

TYPE AND NUMBER OF COMPACTION UNITS

NUMBER OF PASSES THICKNESS OF LIFT

METHOD OF ADDING MOISTURE

☐ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S)
AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

QUALITY CONTROL PROCEDURE FOR OBSERVATION OF FINAL RECLAMATION

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
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1.0 EQUIPMENT AND MATERIALS

Tape measure and notebook.

2.0 METHODOLOGY

2.1 Objectives

The objective of this procedure is to monitor the stability of the Final Reclamation Cover. The stability of the cover will be qualitatively evaluated based upon the extent of erosion and the development of vegetation on the cover.

2.2 Procedure

A visual reconnaissance of the final cover will be made. Any erosion features such as potholes or sinkholes, gullies or rills resulting from surface runoff or other erosional agents should be noted and described by location and dimension. The development of vegetation shall also be noted and described relative to natural, undisturbed terrain in the area.

3.0 FREQUENCY OF OBSERVATIONS

The final reclamation cover and vegetation shall be inspected on an annual basis over the monitoring period or as required in the Specifications.

4.0 REPORTING

4.1 Field Reports

A notebook of field monitoring shall be used to record monitoring entries. Information recorded will include date, area, and observer's name and title, as well as pertinent observation data.

4.2 Records

The notebook of final cover monitoring data shall be maintained in the White Mesa Project Files.

QUALITY CONTROL PROCEDURE
FOR
MONITORING OF EROSION MONUMENTS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
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1.0 EQUIPMENT

(1) Erosion Monuments shall be installed at locations shown on the Plans.
(2) Tape measure, notebook.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to estimate the amount and rate of gully migration through the use of Erosion Monuments.

2.2 Procedure

Pertinent observations of the gully shall be described and recorded. The monuments shall be observed visually and surveyed, if necessary, during the long-term monitoring program. From these observations, the rate of gully migration shall be determined and the potential impact on the integrity of the disposal area assessed.

2.3 Monitoring Frequency

Monitoring of the Erosion Monuments shall be incorporated into the Long-Term Monitoring Program.

3.0 REPORTING

3.1 Field Reports

A notebook of field monitoring of the erosion monuments shall be used to record monitoring entries. The information shall include date of observation and monument number. The general condition of the ground surface in the vicinity of each monument shall be noted and recorded.

3.2 Records

Records of erosion monitoring shall be maintained in the White Mesa Project Files.

QUALITY CONTROL PROCEDURE
FOR
MONITORING OF PIEZOMETERS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
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1.0 EQUIPMENT

(1) Electric Tape - M-Scope - Manufactured by Slope Indicator Company, Model 51453, or equal.
(2) Tape measure.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to measure the water (phreatic) levels in existing and newly installed piezometers in accordance with the Specifications. The data collected will be used to aid in estimating tailings pile settlement. The monitoring data will also be used to determine if the reclamation is operating as proposed.

2.2 Procedure

Phreatic levels in the piezometers shall be measured by lowering the sensor on the end of the electric tape into the piezometer casing. The sensor is lowered until it contacts a conductive liquid. The circuit in the electric tape is then completed activating a speaker noise, diode light emission, or meter deflection on the electric tape. The depth to the phreatic level is then read using the 5-foot interval markings on the electric tape and a tape measure to measure the distance from the 5-foot intervals to where the cable intersects the plane elevation at the top of the piezometer casing. Depth of the phreatic levels shall be measured to the nearest 1/4-inch.

2.3 Monitoring Frequency

Frequency of piezometer monitoring shall be in accordance with the Specifications.

3.0 REPORTING

3.1 Field Data

A piezometer monitoring notebook shall be prepared for recording piezometer monitoring data.

3.2 Records

The piezometer monitoring data shall be maintained in the White Mesa Project Files.

QUALITY CONTROL PROCEDURE
FOR
MONITORING OF SURFACE MOVEMENT MONUMENTS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	Curtis O. Seely, P.E.

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1.0 EQUIPMENT

(1) Survey equipment itemized below or approved equal:
Wilde T2AE Theodolite, one second precision,
Wilde NA2 erected image level,
Wilde GST-20 Legs, and
Auto Ranger EDM.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to accurately locate the horizontal and vertical location of the Surface Movement Monument so that if movement occurs, the direction and magnitude of the movement can be determined by resurveying the monuments.

2.2 Procedure

The Surface Movement Monuments shall be surveyed from established control points approved by the Site Manager. The survey equipment shall be set up over the control points to survey the location and elevation of movement monuments. The survey theodolite shall be set up over a control station and backsighted to another control station. From this, the surveyor shall use the angle right method to determine the bearing, from that station to each visible monument. This procedure shall be repeated at another control site, covering the same monuments, to provide two bearings whose intersection point is the location of the monument. The other method used shall be the EDM or Electronic Distance Measurement using radio waves. Elevations shall be determined through angle measurements, distance and geometry. Horizontal and vertical coordinates shall be recorded to the nearest 0.01 feet.

2.3 Calibration

The survey equipment shall be maintained and calibrated in accordance with the manufacturer's recommendations. Calibration of readings is monitored by observing each monument at least four times and keeping closure angles at less than five seconds.

2.4 Survey Frequency

The Surface Movement Monuments shall be surveyed in accordance with the Specifications.

2.5 References

(1) Surveying Theory and Practice, Sixth Edition, 1981, Davis, Foote, Anderson and Mikhail.

3.0 REPORTING

3.1 Field Data

Field Data notebooks containing raw survey data shall be maintained at the Project Office.

3.2 Records

Records of reduced survey monument coordinates and observations shall be maintained in the White Mesa Project Files.

QUALITY CONTROL PROCEDURE
FOR
MONITORING OF TEMPORARY SETTLEMENT PLATES

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
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1.0 EQUIPMENT AND MATERIALS

Survey equipment itemized below or approved equipment:
Wilde NA2 erected image level
Wilde GST-20 tripod.

2.0 METHODOLOGY

The purpose of installing and monitoring the Temporary Settlement Plates is to obtain consolidation data of the tailings due to the placement of surcharge materials on the tailings. The Temporary Settlement Plates shall be surveyed from previously established elevation control points approved by the QC Officer. Elevation readings of the settlement plates are to be determined to the nearest 0.01 foot.

3.0 CALIBRATION

The survey level shall be maintained and calibrated in accordance with the manufacturer's recommendations.

4.0 SURVEY FREQUENCY

The Temporary Settlement Plates shall be surveyed monthly for a period of 3 months. The readings shall then be evaluated by the QC Officer to determine the frequency of future readings. In any case, readings shall be taken every 6 months until the placement of the final cover is started. At that time, permanent settlement monuments will be installed on the Tailings Piles.

5.0 RECORDS AND REPORTS

Field data notebooks containing field survey data shall be maintained at the Project Office.

Data records of reduced Temporary Settlement Plate elevations shall be maintained in the White Mesa Project Files.

Graphical records showing relative and cumulative settlements shall be reported and maintained in the White Mesa Project Files.

PROCEDURE QC-17-04WM

QUALITY CONTROL PROCEDURE
FOR
MONITORING SEDIMENT CONTROL BARRIERS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2 /88	For Submittal to Lead Agency	Curtis O. Sealy, P.E.

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1.0 PERSONNEL

Monitoring of Sediment Control Barriers shall be performed by the QC Personnel.

2.0 METHODOLOGY

2.1 Materials and Installation

Initial inspections shall be performed to verify that the Sediment Control Barriers are properly constructed according to the Plans and Specifications.

2.2 Monitoring

Inspections shall be made to check Sediment Control Barriers for performance and for sediment buildup. The Sediment Control Barriers shall be cleaned whenever the depth of sediment at the center of the Sediment Control Barrier exceeds 18 inches.

3.0 FREQUENCY

The Sediment Control Barriers shall be inspected for sediment buildup and performance after a precipitation event which exceeds one-half inch during a 24-hour period, or at least once a year.

4.0 RECORDS

The QC Officer shall confirm in writing that the Sediment Control Barriers have been constructed in accordance with the Drawings and Specifications.

Written records shall be made of each inspection of Sediment Control Barriers and maintained in the White Mesa Project Files.

Written records of dates of Sediment Control Barrier cleanout shall be maintained in the White Mesa Project Files.

PROCEDURE QC-19-04WM

QUALITY CONTROL PROCEDURE
FOR
HDPE LINER SEAM INTEGRITY

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	Curtis O. Sealy, P.E.

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1.0 EQUIPMENT

1.1 Destructive Testing

(1) Field laboratory equipped with a constant rate of extension power driven tension testing machine accurate to within +1% for measuring tensional force. The machine shall be equipped with a chart recorder with distance separated on one axis and applied force on the other axis of the coordinates. The capacity of the machine shall be such that the required testing force is more than 15 percent of the machine capacity and less than 85 percent of the machine capacity. The testing machine shall have grips capable of holding the test specimens without slipping during performance of the tests. The preferred rate of extension for the machine for testing shall be 2 inches per minute.

1.2 Nondestructive Testing

(1) Vacuum box equipped with a vacuum gage, a clear glass view panel in the top, and a soft rubber gasket on the periphery of the open bottom. The vacuum box shall be equipped with a gasoline or electricity-driven vacuum pump capable of applying vacuum between 4 and 8 inches of mercury. A vacuum box similar to the Series A100 Straight Seam Tester as supplied by the American Parts and Service Company, 2201 West Commonwealth Avenue, P.O. Box 702, Alhambra, California, 91802, shall be satisfactory.

(2) A soap and water solution for application to the field seams prior to applying vacuum with the vacuum box.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to determine the integrity of field seams used in the joining of sheets of High Density Polyethylene Liners (HDPE).

2.2 Test Procedures

(1) The testing shall be conducted in accordance with the standardized procedures described in ASTM D-4437-84, ASTM D-413-82, and ASTM D-816-82. Preparation of samples for testing shall be in accordance with ASTM D-618-61.

(2) Destructive testing shall include peel test and the shear test described in the above-referenced standards. In both procedures, a failed test shall occur when the weld fails. A passing test shall occur when the sheeting fails.
(3) Frequency of destructive testing of test welds and field seams shall be in accordance with the Specifications. Samples for testing from both test welds and field seams shall be labeled identifying the operator, the welding machine used, the date and shift. Field seam samples shall also be labeled with the location in the field seam from which the sample was taken.
(4) All field seams shall be inspected visually.
(5) All field seams except linings for ditches and trenches for the leak detection drains shall be tested by the Vacuum Box Test described in the above-referenced standards, and shall be performed on 100 percent of field seams.
(6) All defective field seams as detected by both destructive and nondestructive tests including visual inspection shall be marked and repaired in accordance with the manufacturer's recommendations.
(7) All repaired areas shall be retested.

2.3 References

(1) ASTM D-4437-84, Annual Book of ASTM Standards, Volume 09.01
(2) ASTM D-413-82, Annual Book of ASTM Standards, Volume 09.01
(3) ASTM D-816-82, Annual Book of ASTM Standards, Volume 09.01
(4) ASTM D-618-61, Annual Book of ASTM Standards, Volumes 10.01 and 10.02.

3.0 REPORTING

3.1 Forms

F-18 Liner Seam Destructive Tests
F-19 Liner Seam Nondestructive Tests

3.2 Records

Completed test report forms shall be maintained in the White Mesa Project Files.

F-18 LINER SEAM DESTRUCTIVE TESTS

TECHNICIAN____________________ PROJECT NO.____________________

APPROVED BY____________________ DATE____________________

LINER MATERIAL____________________ THICKNESS____________________

WELD NO:____________________

LOCATION:____________________

WELDING MACHINE NO:____________________

OPERATOR:____________________

SAMPLED BY:____________________

OBSERVATIONS:____

____

____

SAMPLE SEAM TEST RESULTS

SPECIMEN	PEEL RESULTS (PASS/FAIL)	SHEAR RESULTS (PASS/FAIL)
1	__________	
2	__________	
3		__________
4		__________

F-19 LINER SEAM NONL_STRUCTIVE TESTS

TECHNICIAN: ____________ TEST METHOD: VISUAL ____________ PROJECT NO: ____________

APPROVED BY: ____________ VACUUM BOX ____________ DATE ____________

WELD NO.	LOCATION	LEAK DETECTED (Yes/No)	RETEST OF REPAIRED LEAK (Yes/No)

QUALITY CONTROL PROCEDURE
FOR
DETERMINING CLAY AND RANDOM FILL THICKNESS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	Curtis O. Sealy, P.E.

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1.0 EQUIPMENT

(1) Survey Method:
Survey equipment itemized below or approved equal.
(a) Wilde T2AE Theodolite, one second precision.
(b) Wilde NA2 Erected Image level.
(c) Wilde GST-20 legs.
(d) Auto Ranger EDM.

(2) Direct Measurement:
(a) Chalk, bentonite, lime or other material readily discernible from cover soils.
(b) Tape measure.
(c) Straight edge at least 10 feet in length.

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to determine the thickness of clay and random fill layers.

2.2 Procedure

(1) Survey Method

Using established control points approved by the Site Manager, lines and grades of the reclamation cover shall be surveyed during cover placement. Cover thickness shall be determined by comparing the surveyed configuration prior to and after cover placement. Intervals between cover layer surveys shall be determined by the Site Manager. Prior to measuring an elevation at the control point, all loose material shall be removed from the surface of the previous layer with a shovel to expose a firm surface.

(2) Direct Method
(a) At predetermined locations, prior to cover placement, an approximate 1/4-inch thick reference layer of material (chalk, lime, bentonite, geotextile fabric or approved alternate), readily discernible from the cover soils, shall be placed over a 50 foot by 50 foot square area.

(b) After the cover layer of clay or random fill is placed, a pit shall be excavated through the cover to the reference layer.

(c) The thickness of the cover shall be measured by taping the distance normal to the straight edge placed across the top of the pit to the reference layer exposed in the pit. Measurements shall be made to the nearest inch. Prior to placing the straight edge across the pit, all loose surficial soils shall be removed with a shovel to expose a firm base.

2.3 Calibration

The survey equipment shall be maintained and calibrated in accordance with the manufacturer's recommendations.

2.4 Survey and Measurement Frequency

Survey and direct measurement frequency during cover placement shall be determined by the Site Manager with the approval of the Regulatory Project Manager.

2.5 References

(1) Surveying Theory and Practice, Sixth Edition, 1981, Davis, Foote, Anderson and Mikhail.

3.0 REPORTING

3.1 Field Data

Field data notebooks containing raw survey data shall be maintained in the White Mesa Project Office. Direct measurement data shall be systematically recorded and incorporated in As-Built Documents.

3.2 Records

Records of reduced survey data and direct measurements shall be maintained in the White Mesa Project Files.

QUALITY CONTROL PROCEDURE
FOR
PREPARATION OF RIPRAP REFERENCE STOCKPILES

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
0	8/2/88	For Submittal to Lead Agency	[signature], P.E.

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1.0 EQUIPMENT

Tape measures, screens, front end loader, screening equipment and truck scale(s).

2.0 METHODOLOGY

2.1 Objective

The objective of this procedure is to prepare reference stockpiles of riprap for each of the riprap types required for construction. Riprap shall be visually compared to the reference stockpile to evaluate compliance with the required gradings.

2.2 Procedure

2.2.1 Specified Riprap Gradings

Riprap gradings are given in the Specifications. More than one type of riprap grading may be called for in the Specifications.:

2.2.2 Reference Stockpiles

(1) Two Reference Stockpiles shall be prepared for each riprap type. The stockpiles shall be prepared combining riprap of the various sizes. The sizes shall be separated from quarry produced riprap by screening or by sizing using a tape measure. One reference stockpile shall be prepared using the maximum size and the largest permissible sizes for the D_{50} and D_{15} percentages. The second stockpile for the same type shall be prepared using the smallest permissible sizes for the D_{50} and D_{15} percentages. Once the gradings have been determined, the materials comprising the reference stockpiles shall be weighed for each stockpile and the density determined on a "pounds per cubic yard" basis.

2.2.3 Stockpile Sizes and Locations

The reference stockpiles shall be equal in size to at least the size of the average truck load capacity to be used for the construction. The stockpiles shall be located at the source so that comparisons of quarried material to the reference stockpiles can be made before the riprap leaves the source site.

2.2.4 Stockpile Segregation

The riprap sizes forming the stockpiles shall be thoroughly mixed using a front end loader in such a manner as to limit segregation. The stockpiles shall be remixed or replaced as necessary during the construction period.

3.0 REPORTING

3.1 Field Data

Records of the gradings for the reference stockpiles shall be made during stockpile preparation. The gradings and weights shall be reviewed and approved by the Quality Control Officer and Site Manager.

3.2 Records

Records of the reference stockpile gradings shall be maintained in the White Mesa Project Files.

PROCEDURE QC-28-04WM

QUALITY CONTROL PROCEDURE
FOR
DETERMINING RIPRAP THICKNESS FOR RECLAMATION COVERS AND CHANNELS

FOR

UMETCO MINERALS CORPORATION
WHITE MESA MILL
BLANDING, UTAH

REV. NO.	DATE OF ISSUE	PURPOSE OF ISSUE	ISSUE APPROVED BY
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1.0 EQUIPMENT

Tape measure and truck scales.

2.0 METHODOLOGY

2.1 Objective

The purpose of this procedure is to determine the quantity of riprap material to be placed in a given area to achieve the required design thickness for reclamation covers and channels where required.

2.2 Procedures

2.2.1 Layout Method

For a given work area to receive riprap, the subgrade surface shall be measured by taping parallel to the ground surface. The pattern and size of the area shall be determined by that which best fits the foundation conditions and construction operations. The surface area to receive riprap shall be computed on a square foot basis.

2.2.2 Computation of Riprap Quantity

The quantity of riprap to be placed in the measured work area shall be computed as follows:

(1) Determine type of riprap required.
(2) Calculate the number of cubic yards of riprap required to cover the measured work area to the design depth using the following formula:

$$CY = \frac{D \times L \times W}{27}$$

Where: CY = Cubic Yards of material required for given work area
D = Design depth in feet
L = Length of work area in feet
W = Width of work area in feet

(3) Calculate tons of riprap required in work area using the following formula:

$$T = CY \times DRS$$

Where: T = Tons of material required for work area
CY = Cubic Yards of riprap required in work area
DRS = Average density of reference stockpile in tons per cubic yard.

(4) Calculate the number of truck loads required for the work area using the formula:

$$TL = T/TF$$

Where: TL = Number of uniform truck loads required for the work area
T = Tons of material required for work area
TF = Truck factor for a truck size and riprap type in tons per truck load

2.3 Calibration

The trucks delivering riprap to the work area shall be calibrated by visual inspection by the loader operator to determine that the volume of material in each truck is relatively uniform and the gradation is similar to the reference stockpiles. Trucks having appreciably less material shall be counted separately as a different truck type for factoring.

2.4 Verification of Riprap Quantity

The amount of riprap placed in a measured work area shall be verified by the following:

(1) For a given work area, a minimum of one truck type shall be selected at random for each day riprap is placed and shall be weighed. This weight shall be the daily truck factor (DTF).

(2) If the DTF is within 5 percent of the TF for the truck size, the correct truck loads have been delivered to the measured work area.

(3) If the DTF is less than 5 percent of the TF, then the additional truck loads required for a given work area shall be determined as follows for each truck type:

$$\text{Additional truck loads} = TL \times [TF/DTF - 1]$$

(4) If the DTF is greater than 5 percent of the TF, then the excess truck loads in the given work area shall be determined as follows:

Excess truck loads = TL x (1- TF/DTF)

(5) Excess material can be utilized by correcting the work area dimensions to the appropriate size for the truck loads delivered.

2.5 <u>Truck Factor Determination</u>

2.5.1 Equipment

Truck Scale.

2.5.2 Objective

To determine an average loaded weight for each size truck with each type of riprap for use in estimating truck loads required to achieve the design thickness of riprap.

2.5.3 Procedure

2.5.4 Initial Truck Factor

Utilizing the reference stockpile materials, fill each size truck with the riprap material to a point considered as full and consistent with anticipated construction methods. The truck is then weighed and the net weight recorded for each truck size and riprap type.

The riprap material should be placed back into the reference stockpile. The same truck shall then be reloaded or loaded from the other reference stockpile for the same riprap type or a different truck of the same size loaded and the procedure repeated.

The truck factor can be estimated based on one truck net weight or an average.

The truck factor shall be calculated as:

TF = TTW/TT

Where: TF = Truck Factor for each truck size and each riprap type in tons per truck load.
TTW = Total net weight of all trucks weighed for each truck size and riprap type.
TT = Total number of trucks weighed of each truck size and riprap type

2.5.5 Construction Truck Factor

As construction progresses, the initial Truck Factor shall be modified by including the truck net weights measured during construction. By doing so, the Truck Factor will provide a better basis for estimating since current data will be used.

For each truck size and riprap type, the Truck Factor shall be periodically recalculated using the Daily Truck net weights measured during construction, initial truck weights and the formula in Section 2.5.4.

3.0 REPORTING

3.1 Field Data

Work areas and area measurements, truck weights (riprap), truck counts (amount of riprap placed in a given work area) and truck weight verification shall be recorded on appropriate forms and field data notebooks.

3.2 Records

Records of work areas, area measurements, riprap quantity calculations, truck weights, truck factors, truck counts and calculations of the amount of riprap placed on the measured work area (thickness) shall be maintained in the White Mesa Project Files.

CELL 4 DESIGN

TAILINGS MANAGEMENT SYSTEM

WHITE MESA PROJECT

BLANDING, UTAH

APPENDIX A

Umetco Minerals Corporation

AUGUST, 1988

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1.0 GEOLOGY

The site is near the western margin of the Blanding Basin in southeastern Utah and within the Monticello uranium-mining district. Thousands of feet of multi-colored marine and non-marine sedimentary rocks have been uplifted and warped, and subsequent erosion has carved a spectacular landscape for which the region is famous. Another unique feature of the region is the wide-spread presence of unusually large accumulations of uranium-bearing minerals.

1.1 Regional Geology [2.4.1]*

1.1.1 Physiography [2.4.1.1]

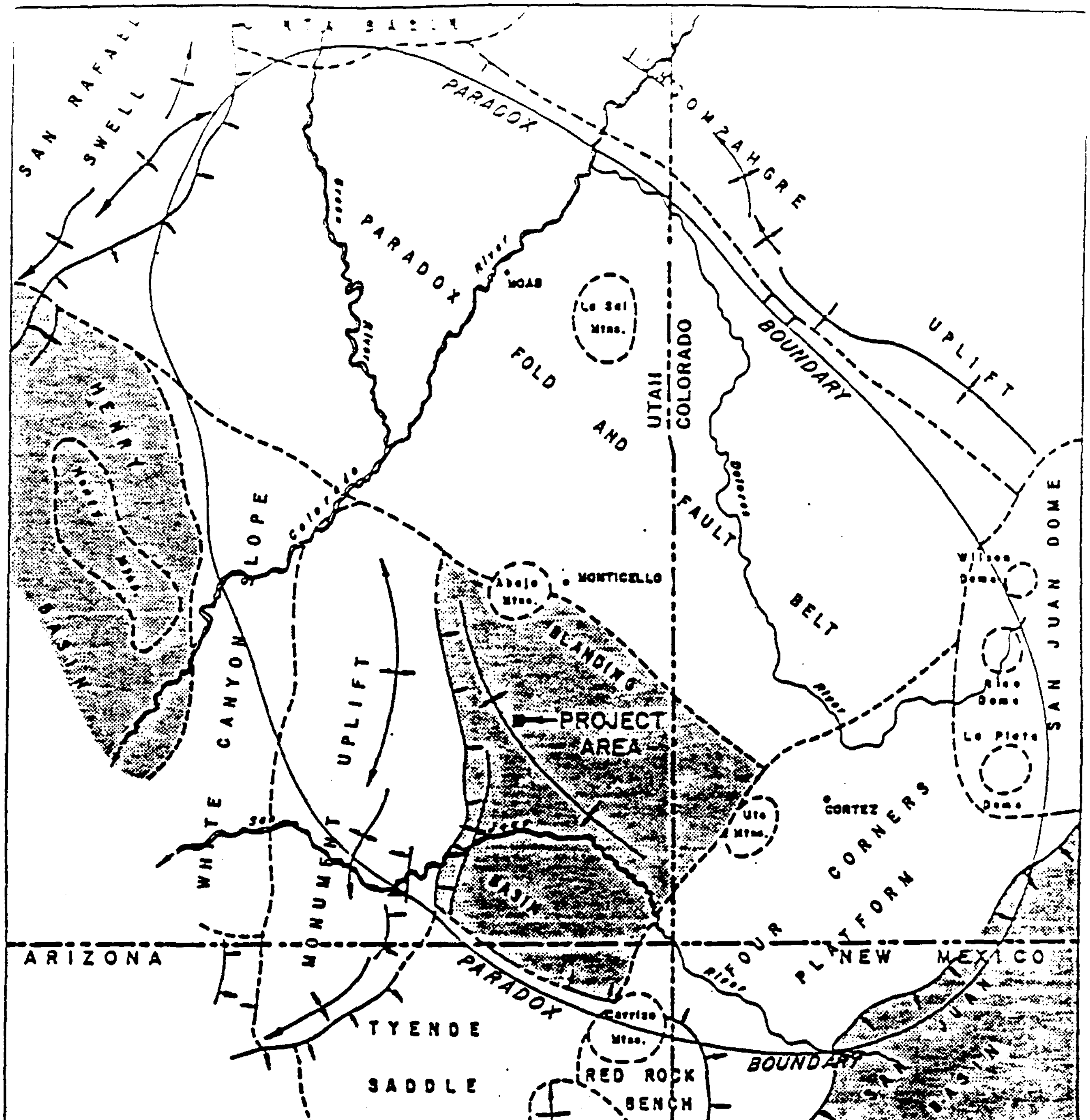
The project site is within the Canyon Lands section of the Colorado Plateau physiographic province. To the north, this section is distinctly bounded by the Book Cliffs and Grand Mesa of the Uinta Basin; western margins are defined by the tectonically controlled High Plateaus section, and the southern boundary is arbitrarily defined along the San Juan River. The eastern boundary is less distinct where the elevated surface of the Canyon Lands section merges with the Southern Rocky Mountain province.

Canyon Lands has undergone epeirogenic uplift and subsequent major erosion has produced the region's characteristic angular topography reflected by high plateaus, mesas, buttes, structural benches, and deep canyons incised into relatively flat-lying sedimentary rocks of pre-Tertiary age. Elevations range from approximately 3000 feet (914 meters) in the bottoms of the deeper canyons along the southwestern margins of the section to more than 11,000 feet (3353 meters) in the topograhically anomalous laccolithic Henry, Abajo and La Sal Mountains to the northeast. With the exception of the deeper canyons and isolated mountain peaks, an average elevation in excess of 500 feet (1524 meters) persists over most of the Canyon Lands section.

On a more localized regional basis, the project site is located near the western edge of the Blanding Basin, sometimes referred to as the Great Sage Plain (Eardly, 1958), lying east of the north-south trending Monument Uplift, south of the Abajo Mountains and adjacent to the northwesterly-trending Paradox Fold and Fault Belt (Figure A-1). Topographically, the Abajo Mountains are the most prominent feature in the region, rising more than 4000 feet (1219 meters) above the broad, gently rolling surface of the Great Sage Plain.

The Great Sage Plain is a structural slope, capped by the resistant Burro Canyon Formation and the Dakota Sandstone, almost horizontal in an east-west direction but descends to the south with a regional slope of about 2000 feet (610 meters) over a distance of nearly 50 miles (80 kilometers). Though not as deeply or intricately dissected as other parts of the Canyon Lands, the plain is cut by numerous narrow and vertical-walled south-trending valleys 100 to more than 500 feet (30 to 152+ meters) deep. Waters from the intermittent streams that drain the plain flow southward to the San Juan River, eventually joining the Colorado River and exiting the Canyon Lands section through the Grand Canyon.

*From Environmental Report (Dames & Moore 1978b).



(After Shoemaker, 1956; Kelley, 1958)

EXPLANATION

BOUNDARY OF TECTONIC DIV.

MONOCLINE, SHOWING TRACE OF AXIS AND DIRECTION OF DIP

ANTICLINE, SHOWING TRACE OF AXIS AND DIRECTION OF PLUNGE

SYNCLINE, SHOWING TRACE OF AXIS AND DIRECTION OF PLUNGE

N.

25 0 25 50 75

SCALE IN MILES

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TECTONIC INDEX MAP

JUNE, 1988 FIGURE A-1

1.1.2 Rock Units [2.4.1.2]

The sedimentary rocks exposed in southeastern Utah have an aggregate thickness of about 6000 to 7000 feet (1829 to 2134 meters) and range in age from Pennsylvania to Late Cretaceous. Older unexposed rocks are known mainly from oil well drilling in the Blanding Basin and Monument Uplift. These wells have encountered correlative Cambrian to Permian rock units of markedly differing thicknesses but averaging over 5000 feet (1524 meters) in total thickness (Witkind, 1964). Most of the wells drilled in the region have bottomed in the Pennsylvanian Paradox Member of the Hermosa Formation. A generalized stratigraphic section of rock units ranging in age from Cambrian through Jurassic and Triassic(?), as determined from oil-well logs, is shown in Table A-1. Descriptions of the younger rocks, Jurassic through Cretaceous, are based on field mapping by various investigators and are shown in Table A-2.

Paleozoic rocks of Cambrian, Devonian and Mississippian ages are not exposed in the southeastern Utah region. Most of the geologic knowledge regarding these rocks was learned from the deeper oil wells drilled in the region, and from exposures in the Grand Canyon to the southwest and in the Uinta and Wasatch Mountains to the north. A few patches of Devonian rocks are exposed in the San Juan Mountains in southwestern Colorado. These Paleozoic rocks are the result of periodic transgressions and regressions of epicontinental seas and their lithologics reflect a variety of depositional environments.

In general, the coarse-grained feldspathic rocks overlying the Precambrian basement rocks grade upward into shales, limestones and dolomites that dominate the upper part of the Cambrian. Devonian and Mississippian dolomites, limestones and interbedded shales unconformably overlie the Cambrian strata. The complete absence of Ordovician and Silurian rocks in the Grand Canyon, Uinta Mountains, southwest Utah region and adjacent portions of Colorado, New Mexico and Arizona indicates that the region was probably epeirogenically positive during these times.

The oldest stratigraphic unit that crops out in the region is the Hermos Formation of Middle and Late Pennsylvanian age. Only the uppermost strata of this formation are exposed, the best exposure being in the canyon of the San Juan River at the "Goosenecks" where the river traverses the crest of the Monument uplift. Other exposures are in the breached centers of the Lisbon Valley, Moab and Castle Valley anticlines. The Paradox Member of the Hermosa Formation is sandwiched between a relatively thin lower unnamed member consisting of dark-gray shale siltstone, dolomite, anyhydrite, and limestone, and an upper unnamed member of similar lithology but having a much greater thickness. Composition of the Paradox Member is dominantly a thick sequence of interbedded salt (halite), anhydrite, gypsum, and black shale. Surface exposures of the Paradox in the Moab and Castle Valley anticlines are limited to contorted residues of gypsum and black shale.

Conformably overlying the Hermosa is the Pennsylvanian and Permian(?) Rico Formation, composed of interbedded reddish-brown arkosic sandstone and gray marine limestone. The Rico represent a transition zone between the predominantly marine Hermosa and the overlying continental Cutler Formation of Permian age.

TABLE A-1

GENERALIZED STRATIGRAPHIC SECTION OF SUBSURFACE ROCKS BASED ON OIL-WELL LOGS

(After Stokes, 1954; Witkind, 1964; Huff and Lesure, 1965; Johnson and Thordarson, 1966)

Era	Age	Stratigraphic Unit	Thickness* (ft)	Description
MESOZOIC		Glen Canyon Group:		
MESOZOIC	Jurassic and Triassic(?)	Navajo Sandstone	300-400	Buff to light gray, massive, cross-bedded, friable sandstone
MESOZOIC	Triassic(?)	Kayenta Formation	100-150	Reddish-brown sandstone and mudstone and occasional conglomerate lenses
MESOZOIC	Triassic	Wingate Sandstone	250-350	Reddish-brown, massive, cross-bedded, fine-grained sandstone
MESOZOIC		Chinle Formation:		
MESOZOIC		Undivided	600-700	Variegated claystone with some thin beds of siltstone and limestone
MESOZOIC		Moss Back Member	0-100	Light colored, conglomeratic sandstone and conglomerite
MESOZOIC		Shinarump Member	0-20	Yellowish-gray, fine to coarse-grained sandstone; conglomeratic sandstone and conglomerate
		Unconformity		
MESOZOIC	Middle(?) and Lower Triassic	Moenkopi Formation	50-100	Reddish-brown mudstone and fine-grained sandstone
		Unconformity		
PALEOZOIC	Permian	Cutler Formation:		
PALEOZOIC		Organ Rock Member	0-600	Reddish-brown, sandy mudstone
PALEOZOIC		Cedar Mesa Sandstone Member	1100-1400	Reddish-brown, massive, fine to medium-grained sandstone
PALEOZOIC	Pennsylvanian and Permian(?)	Rico Formation	450	Red and gray calcareous, sandy shale; gray limestone and sandstone
PALEOZOIC	Pennsylvanian	Hermosa Formation:		
PALEOZOIC		Upper Member	1000-1200	Gray, massive limestone; some shale and sandstone
PALEOZOIC		Paradox Member	1200	Halite, anhydrite, gypsum, shale, and siltstone
PALEOZOIC		Lower Member	200	Limestone, siltstone, and shale
		Unconformity		
PALEOZOIC	Mississippian	Leadville Limestone	500	White to tan sucrose to crystalline limestone
PALEOZOIC	Devonian	Ouray Limestone	100	Light gray and tan, thin-bedded limestone and dolomite
PALEOZOIC		Elbert Formation	200	Gray and brown dolomite and limestone with thin beds green shale and sandstone
		Unconformity		
PALEOZOIC	Cambrian	Ophir Formation and Tintic Quartzite	600	Gray and brown limestone and dolomite, feldspathic sandstone and arkose

* To convert feet to meters, multiply by 0.3048. Average thickness given if range is not shown.

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TABLE A-2

GENERALIZED STRATIGRAPHIC SECTION OF EXPOSED ROCKS IN THE PROJECT VICINITY

(After Haynes et al., 1962; Witkind, 1964; Huff and Lesure, 1965)

ERA	SYSTEM	SERIES (Age)	STRATIGRAPHIC UNIT		THICKNESS* (ft)	LITHOLOGY
CENOZOIC	QUATERNARY	Holocene to Pleistocene	Alluvium		2-25+	Silt, sand and gravel in arroyos and stream valleys.
CENOZOIC	QUATERNARY	Holocene to Pleistocene	Colluvium and Talus		0-15+	Slope wash, talus and rock rubble ranging from cobbles and boulders to massive blocks fallen from cliffs and outcrops of resistant rock.
CENOZOIC	QUATERNARY	Holocene to Pleistocene	Loess		0-22+	Reddish-brown to light-brown, unconsolidated, well-sorted silt to medium-grained sand; partially cemented with caliche in some area; reworked partly by water.
			Unconformity			
MESOZOIC	CRETACEOUS	Upper Cretaceous	Mancos Shale		0-11(?)	Gray to dark-gray, fissile, thin-bedded marine shale with fossiliferous sandy limestone in lower strata.
MESOZOIC	CRETACEOUS	Upper Cretaceous	Dakota Sandstone		30-75	Light yellowish-brown to light gray-brown, thick bedded to cross-bedded sandstone, conglomeratic sandstone; interbedded thin lenticular gray carbonaceous claystone and impure coal; local course basal conglomerate.
			Unconformity			
MESOZOIC	CRETACEOUS	Lower Cretaceous	Burro Canyon Formation		50-150	Light-gray and light-brown, massive and cross-bedded conglomeratic sandstone and interbedded green and gray-green mudstone; locally contains thin discontinuous beds of silicified sandstone and limestone near top.
			Unconformity(?)			
MESOZOIC	JURASSIC	Upper Jurassic	Morrison Formation	Brushy Basin Member	200-450	Variegated gray, pale-green, reddish-brown, and purple bentonitic mudstone and siltstone containing thin discontinuous sandstone and conglomerate lenses.
MESOZOIC	JURASSIC	Upper Jurassic	Morrison Formation	Westwater Canyon Member	0-250	Interbedded yellowish- and greenish-gray to pinkish-gray, fine- to coarse-grained arkosic sandstone and greenish-gray to reddish-brown sandy shale and mudstone.
MESOZOIC	JURASSIC	Upper Jurassic	Morrison Formation	Recapture Member	0-200	Interbedded reddish-gray to light brown fine- to medium-grained sandstone and reddish-gray silty and sandy claystone.
MESOZOIC	JURASSIC	Upper Jurassic	Morrison Formation	Salt Wash Member	0-350	Interbedded yellowish-brown to pale reddish-brown fine-grained to conglomeritic sandstones and greenish- and reddish-gray mudstone.
			Unconformity			
MESOZOIC	JURASSIC	Upper Jurassic	San Rafael Group	Bluff Sandstone	0-150+	White to grayish-brown, massive, cross-bedded, fine- to medium-grained eolian sandstone.
MESOZOIC	JURASSIC	Upper Jurassic	San Rafael Group	Summerville Formation	25-125	Thin-bedded, ripple-marked reddish-brown muddy sandstone and sandy shale.
MESOZOIC	JURASSIC	Upper Jurassic	San Rafael Group	Entrada Sandstone	150-180	Reddish-brown to grayish-white, massive, cross-bedded, fine- to medium-grained sandstone.
MESOZOIC	JURASSIC	Middle Jurassic	San Rafael Group	Carmel Formation	20-100+	Irregulary bedded reddish-brown muddy sandstone and sandy mudstone with local thin beds of brown to gray limestone and reddish- to greenish-gray shale.
			Unconformity			

*To convert feet to meters, multiply feet by 0.3048.

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Two members of the Cutler probably underlying the region south of Blanding are, in ascending order, the Cedar Mesa Sandstone and the Organ Rock Tongue. The Cedar Mesa is a white to pale reddish-brown, massive, cross-bedded, fine-to medium-grained eolian sandstone. An irregular fluvial sequence of reddish-brown fine-grained sandstones, shaly siltstones and sandy shales comprise the Organ Rock Tongue.

The Moenkopi Formation, of Middle(?) and Lower Triassic age, unconformably overlies the Cutler strata. It is composed of thin, evenly-bedded, reddish- to chocolate-brown, ripple-marked, cross-laminated siltstone and sandy shales with irregular beds of massive medium-grained sandstone.

A thick sequence of complex continental sediments known as the Chinle Formation unconformably overlies the Moenkopi. For the purpose of making lithology correlations in oil wells this formation is divided into three units: the basal Sinarump Member, the Moss Back Member and an upper undivided thick sequence of variegated reddish-brown, reddish- to greenish-gray, yellowish-brown to light-brown betonitic claystones, mudstones, sandy siltstones, fine-grained sandstones, and limestones. The basal Shinarump is dominantly a yellowish-grey, fine- to coarse-grained sandstone, conglomeratic sandstone and conglomerate characteristically filling ancient stream channel scours eroded into the Moenkopi surface. Numerous uranium deposits have been located in this member in the White Canyon mining district to the west of Comb Ridge. The Moss Back is typically composed of yellowish- to greenish-grey, fine- to medium-grained sandstone, conglomeratic sandstone and conglomerate. It commonly comprises the basal unit of the Chinle where the Shinarump was not deposited, and in a like manner, fills ancient stream channels scoured into the underlying unit.

In the Blanding Basin the Glen Canyon Group consists of three formations which are, in ascending order, the Wingate Sandstone, the Kayenta and the Navajo Sandstone. All are conformable and their contacts are gradational. Commonly cropping out in sheer cliffs, the Late Triassic Wingate Sandstone is typically composed of buff to reddish-brown, massive, cross-bedded, well-sorted, fine-grained quartzose sandstone of eolian origin. Late Triassic(?) Kayenta is fluvial in origin and consists of reddish-brown, irregularly to cross-bedded sandstone, shaly sandstone and, locally, thin beds of limestone and conglomerate. Light yellowish-brown to light-gray and white, massive, cross-bedded, friable, fine- to medium-grained quartzose sandstone typifies the predominatnly eolian Jurassic and Triassic(?) Navajo Sandstone.

Four formations of the Middle to Late Jurassic San Rafael Group unconformably overly the Navajo Sandstone. These strata are composed of alternating marine and non-marine sandstones, shales and mudstones. In ascending order, the formations are the Carmel Formation, Entrada Sandstone, Summerville Formation, and Bluff Sandstone. The Carmel usually crops out as a bench between the Navajo and Entrada Sandstones. Typically reddish-brown muddy sandstone and sandy mudstone, the Carmel locally contains thin beds of brown to gray limestone and reddish- to greenish-gray shale. Predominantly eolian in origin, the Entrada is a massive cross-bedded fine- to medium-grained sandstone ranging in color from reddish-brown to grayish-white that crops out in cliffs or hummocky slopes. The Summerville is composed of regular thin-bedded, ripple-marked, reddish-brown muddy sandstone and sandy shale of marine origin and forms steep to gentle slopes above the Entrada. Cliff-forming Bluff Sandstone is present only in the southern part of the Monticello district thinning northward and pinching out near Blanding. It is a white to grayish-brown, massive, cross-bedded eolian sandstone.

In the southeastern Utah region the Late Jurassic Morrison Formation has been divided in ascending order into the Salt Wash, Recapture, Westwater Canyon, and Brushy Basin Members. In general, these strata are dominantly fluvial in origin but do contain lacustrine sediments. Both the Salt Wash and Recapture consist of alternating mudstone and sandstone; the Westwater Canyon is chiefly sandstone with some sandy mudstone and claystone lenses, and the heterogeneous Brushy Basin consists of variegated bentonitic mudstone and siltstone containing scattered thin limestone, sandstone and conglomerate lenses. As strata of the Morrison Formation are the oldest rocks exposed in the project area vicinity and are one of the two principal uranium-bearing formations in southeast Utah, the Morrison, as well as younger rocks, is described in more detail in Section 1.2.2.

The Early Cretaceous Burro Canyon Formation rests unconformably(?) on the underlying Brushy Basin Member of the Morrison Formation. Most of the Burro Canyon consists of light-colored, massive, cross-bedded fluvial conglomerate, conglomerate sandstone and sandstone. Most of the conglomerates are near the base. Thin, even-bedded, light-green mudstones are included in the formation and light-grey thin-bedded limestones are sometimes locally interbedded with the mudstones near the top of the formation.

Overlying the Burro Canyon is the Dakota Sandstone of Upper Cretaceous age. Typical Dakota is dominantly yellowish-brown to light-gray, thick-bedded, quartzitic sandstone and conglomeratic sandstone with subordinate thin lenticular beds of mudstone, gray carbonaceous shale and, locally, thin seams of impure coal. The contact with the underlying Burro Canyon is unconformable whereas the contact with the overlying Mancos Shale is gradational from the light-colored sandstones to dark-grey to black shaly siltstone and shale.

Upper Cretaceous Mancos Shale is exposed in the region surrounding the project vicinity but not within it. Where exposed and weathered, the shale is light-gray or yellowish-gray, but is dark, to olive-gray where fresh. Bedding is thin and well developed; much of it is laminated.

Quaternary alluvium within the project vicinity is of three types: alluvial silt, sand and gravels deposited in the stream channels; colluvium deposits of slope wash, talus, rock rubble and large displaced blocks on slopes below cliff faces and outcrops of resistant rock; and alluvial and windblown deposits of silt and sand, partially reworked by water, on benches and broad upland surfaces.

1.1.3 Structure and Tectonics [2.4.1.3]

According to Shoemaker (1954 and 1956), structural features within the Canyon Lands of southeastern Utah may be classified into three main categories on the basis of origin or mechanism of the stress that created the structure. These three categories are: (1) structures related to large-scale regional uplifting or downwarping (epeirogenic deformation) directly related to movements in the basement complex (Monument Uplift and the Blanding Basin); (2) structures resulting from the plastic deformation of thick sequences of evaporite deposits, salt plugs and salt anticlines, where the structural expression at the surface is not reflected in the basement complex (paradox Fold and Fault Belt); and (3) structures that are formed in direct response to stresses induced by magmatic intrusion including local laccolithic domes, dikes and stocks (Abajo Mountains).

Each of the basins and uplifts within the project area region is an asymetric fold usually separated by a steeply dipping sinuous monocline. Dips of the sedimentary beds in the basins and uplifts rarely exceed a few degrees except along the monocline (Shoemaker, 1956) where, in some instances, the beds are nearly vertical. Along the Comb Ridge monocline, the boundary between the Monument Uplift and the Blanding Basin, approximately 8 miles (12.9 kilometers) west of the project area, dips in the Upper Triassic Wingate sandstone and in the Chinle Formation are more than 40 degrees to the east.

Structures in the crystalline basement complex in the central Colorado Plateau are relatively unknown but where monoclines can be followed in Precambrian rocks they pass into steeply dipping faults. It is probable that the large monoclines in the Canyon Lands section are related to flexure of the layered sedimentary rocks under tangential compression over nearly vertical normal or high-angle reverse faults in the more rigid Precambrian basement rocks (Kelley, 1955; Shoemaker, 1956; Johnson and Thordarson, 1966).

The Monument Uplift is a north-trending, elongated, upwarped structure approximately 90 miles (145 kilometers) long and nearly 35 miles (56 kilometers) wide. Structural relief is about 3000 feet (914 meters) (Kelley, 1955). Its broad crest is slightly convex to the east where the Comb Ridge monocline defines the eastern boundary. The uniform and gently descending western flank of the uplift crosses the White Canyon slope and merges into the Henry Basin (Figure A-1).

East of the Monument Uplift, the relatively equidimensional Blanding Basin merges almost imperceptibly with the Paradox Fold and Fault Belt to the north, the Four Corners Platform to the southeast and the Defiance Uplift to the south. The basin is a shallow feature with approximately 700 feet (213 meters) of structural relief as estimated on top of the Upper Triassic Chinle Formation by Kelley (1955), and is roughly 40 to 50 miles (64 to 80 kilometers) across. Gentle folds within the basin trend westerly to northwesterly in contrast to the distinct northerly orientation of the Monument Uplift.

Situated to the north of the Monument Uplift and Blanding Basin is the most unique structural feature of the Canyon Lands section, the Paradox Fold and Fault Belt. This tectonic unit is dominated by northwest trending anticlinal folds and associated normal faults covering an area about 150 miles (241 kilometers) long and 65 miles (104 kilometers) wide. These anticlinal structures are associated with salt flowage from the Pennsylvanian Paradox Member of the Hermosa Formation and some show piercement of the overlying younger sedimentary beds by plug-like salt intrusions (Johnson and Thordarson, 1966). Prominent valleys have been eroded along the crests of the anticlines where salt piercements have occurred or collapses of the central parts have resulted in intricate systems of step-faults and grabens along the anticlinal crests and flanks.

The Abajo Mountains are located approximately 20 miles (32 kilometers) north of the project area on the more-or-less arbitrary border of the Blanding Basin and the Paradox Fold and Fault Belt (Figure A-1). These mountains are laccolithic domes that have been intruded into and through the sedimentary rocks by several stocks (Witkind, 1964). At least 31 laccoliths have been identified. The youngest sedimentary rocks that have been intruded are those of Mancos Shale of Late Cretaceous age. Based on this and other vague and inconclusive evidence, Witkind (1964) has assigned the age of these intrusions to the Late Cretaceous or early Eocene.

Nearly all known faults in the region of the project area are high-angle normal faults with displacements on the order of 300 feet (91 meters) or less (Johnson and Thordarson, 1966). The largest known faults within a 40-mile (64-kilometer) radius around Blanding are associated with the Shay graben on the north side of the Abajo Mountains and the Verdure graben on the south side. Respectively, these faults trend northeasterly and easterly and can be traced for approximate distances ranging from 21 to 34 miles (34 to 55 kilometers) according to Witkind (1964). Maximum displacements reported by Witkind on any of the faults is 320 feet (98 meters). Because of the extensions of Shay and Verdure fault systems beyond the Abajo Mountains and other geologic evidence, the age of these faults is Late Cretaceous or post-Cretaceous and antedate the laccolithic intrusions (Witkind, 1964).

A prominent group of faults is associated with the salt anticlines in the Paradox Fold and Fault Belt. These faults trend northwesterly parallel to the anticlines and are related to the salt emplacement. Quite likely, these faults are relief features due to salt intrusion or salt removal by solution (Thompson, 1967). Two faults in this region, the Lisbon Valley fault associated with the Lisbon Valley salt anticline and the Moab fault at the southeast end of the Moab anticline have maximum vertical displacements of at least 5000 feet (1524 meters) and 2000 feet (609 meters), respectively, and are probably associated with breaks in the Precambrian basement crystalline complex. It is possible that zones of weakness in the basement rocks represented by faults of this magnitude may be responsible for the beginning of salt flowage in the salt anticlines, and subsequent solution and removal of the salt by ground water caused collapse within the salt anticlines resulting in the formation of grabens and local complex block faults (Johnson and Thordarson, 1966).

The longest faults in the Colorado Plateau are located some 155 to 210 miles (249 to 338 kilometers) west of the project area along the western margin of the High Plateau section. These faults have a north to northeast echelon trend, are nearly vertical and downthrown on the west in most places. Major faults included in this group are the Hurrican, Toroweap-Sevier, Paunsaugunt, and Paradise faults. The longest fault, the Toroweap-Sevier, can be traced for about 240 miles (386 kilometers) and may have as much as 3000 feet (914 meters) of displacement (Kelley, 1955).

From the later part of the Precambrian until the middle Paleozoic the Colorado Plateau was a relatively stable tectonic unit undergoing gentle epeirogenic uplifting and downwarping during which seas transgressed and regressed, depositing and then partially removing layers of sedimentary materials. This period of stability was interrupted by northeast-southwest tangential compression that began sometime during late Mississippian or early Pennsylvanian and continued intermittently into the Triassic. Buckling along the northeast margins of the shelf produced northwest-trending uplifts, the most prominent of which are the Uncompahgre and San Juan Uplifts, sometimes referred to as the Ancestral Rocky Mountains. Clearly, these positive features are the earliest marked tectonic controls that may have guided many of the later Laramide structures (Kelley, 1955).

Subsidence of the area southwest of the Uncompahgre Uplift throughout most of the Pennsylvanian led to the filling of the newly formed basin with an extremely thick sequence of evaporites and associated interbeds which comprise the Paradox Member of the Hermosa Formation (Kelly, 1958). Following Paradox deposition, continental and marine sediments buried the evaporite sequence as epeirogenic movements shifted shallow seas across the region during the Jurassic, Triassic and much of the Cretaceous. The area underlain by the Paradox Member in eastern Utah and western Colorado is commonly referred to as the Paradox Basin (Figure A-1). Renewed compression during the Permian initiated the salt anticlines and piercements, and salt flowage continued through the Triassic.

The Laramide orogeny, lasting from Late Cretaceous through Eocene time, consisted of deep-seated compressional and local vertical stresses. The orogeny is responsible for a north-south to northwest trend in the tectonic fabric of the region and created most of the principal basins and uplifts in the eastern-half of the Colorado Plateau (Grose, 1972; Kelley, 1955).

Post-Laramide epeirogenic deformation has occurred throughout the Tertiary; Eocene strata are flexed sharply in the Grand Hogback monocline, fine-grained Pliocene deposits are tilted on the flanks of the Defiance Uplift, and Pleistocene deposits in Fisher Valley contain three angular unconformaties (Shoemaker, 1956).

1.2 Blanding Site Geology

1.2.1 Physiography and Topography [2.4.2.1]

The project site is located near the center of White Mesa, one of the many finger-like north-south trending mesas that make up the Great Sage Plain. The nearly flat upland surface of White Mesa is underlain by resistant sandstone caprock which forms steep prominent cliffs separating the upland from deeply entrenched intermittent stream courses on the east, south and west.

Surface elevations across the project site range from about 5550 to 5650 feet (1692 to 1722 meters) and the gently rolling surface slopes to the south at a rate of approximately 60 feet per mile (18 meters per 1.6 kilometer).

Maximum relief between the mesa's surface and Cottonwood Canyon on the west is about 750 feet (229 meters) where Westwater Creek joins Cottonwood Wash. These two streams and their tributaries drain the west and south sides of White Mesa. Drainage on the east is provided by Recapture Creek and its tributaries. Both Cottonwood Wash and Recapture Creeks are normally intermittent streams and flow south to the San Juan River. However, Cottonwood Wash has been known to flow perennially in the project vicinity during wet years.

1.2.2 Rock Units [2.4.2.2]

Only rocks of Jurassic and Cretaceous ages are exposed in the vicinity of the project site. These include, in ascending order, the Upper Jurassic Salt Wash, Recapture, Westwater Canyon, and Brushy Basin Members of the Morrison Formation; the Lower Cretaceous Burro Canyon Formation; and the Upper Cretaceous Dakota Sandstone. The Upper Cretaceous Mancos Shale is exposed as isolated remnants along the rim of Recapture Creek valley several miles southeast of the project site and on the eastern flanks of the Abajo Mountains some 20 miles (32 kilometers) north but is not exposed at the project site. However, patches of Mancos Shale may be present within the project site boundaries as isolated buried remnants that are obscurred by a mantle of alluvial windblown silt and sand.

The Morrison Formation is of particular economic importance in southeast Utah since several hundred uranium deposits have been discovered in the basal Salt Wash Member (Stokes, 1967).

In most of eastern Utah, the Salt Wash Member underlies the Brushy Basin. However, just south of Blanding in the project vicinity the Recapture Member replaces an upper portion of the Salt Wash and the Westwater Canyon Member replaces a lower part of the Brushy Basin. A southern limit of Salt Wash deposition and a northern limit of Westwater Canyon deposition has been recognized by Haynes et al. (1972) in Westwater Canyon approximately 3 to 6 miles (4.8 to 9.7 kilometers), respectively, northwest of the project site. However, good exposures of Salt Wash are found throughout the Montezuma Canyon area 13 miles (21 kilometers) to the east.

The Salt Wash Member is composed dominantly of fluvial fine-grained to conglomeratic sandstones, and interbedded mudstones. Sandstone intervals and usually yellowish-brown to pale reddish-brown while the mudstones are greenish- and reddish-gray. Carbonaceous materials ("trash") vary from sparse to abundant. Cliff-forming massive sandstone and conglomeratic sandstone in discontinuous beds make up to 50 percent or more of the member. According to Craig et al. (1955), the Salt Wash was deposited by a system of braided streams flowing generally east and northeast. Most of the uranium-vanadium deposits are located in the basal sandstones and conglomeratic sandstones that fill stream-cut scour channels in the underlying Bluff Sandstone, or where the Bluff Sandstone has been removed by pre-Morrison erosion, in similar channels cut in the Summerville Formation. Mapped thicknesses of this member range from 0 to approximately 350 feet (0-107 meters) in southeast Utah. Because the Salt Wash pinches out in a southerly direction in Recapture Creek 3 miles (4.8 kilometers) northwest of the project site and does not reappear until exposed in Montezuma Canyon, it is not known for certain that the Salt Wash actually underlies the site.

The Recapture Member is typically composed of interbedded reddish-gray, white, and light-brown fine- to medium-grained sandstone and reddish-gray, silty and sandy claystone. Bedding is gently to sharply lenticular. Just north of the project site, the Recapture intertongues with and grades into the Salt Wash and the contact between the two cannot be easily recognized. A few spotty occurrences of uraniferous mineralization are found in sandstone lenses in the southern part of the Monticello district and larger deposits are known in a conglomeratic sandstone facies some 75 to 100 miles (121 to 161 kilometers) southeast of the Monticello district. Since significant ore deposits have not been found in extensive outcrops in more favorable areas, the Recapture is belived not to contain potential resources in the project site (Johnson and Thordarson, 1966).

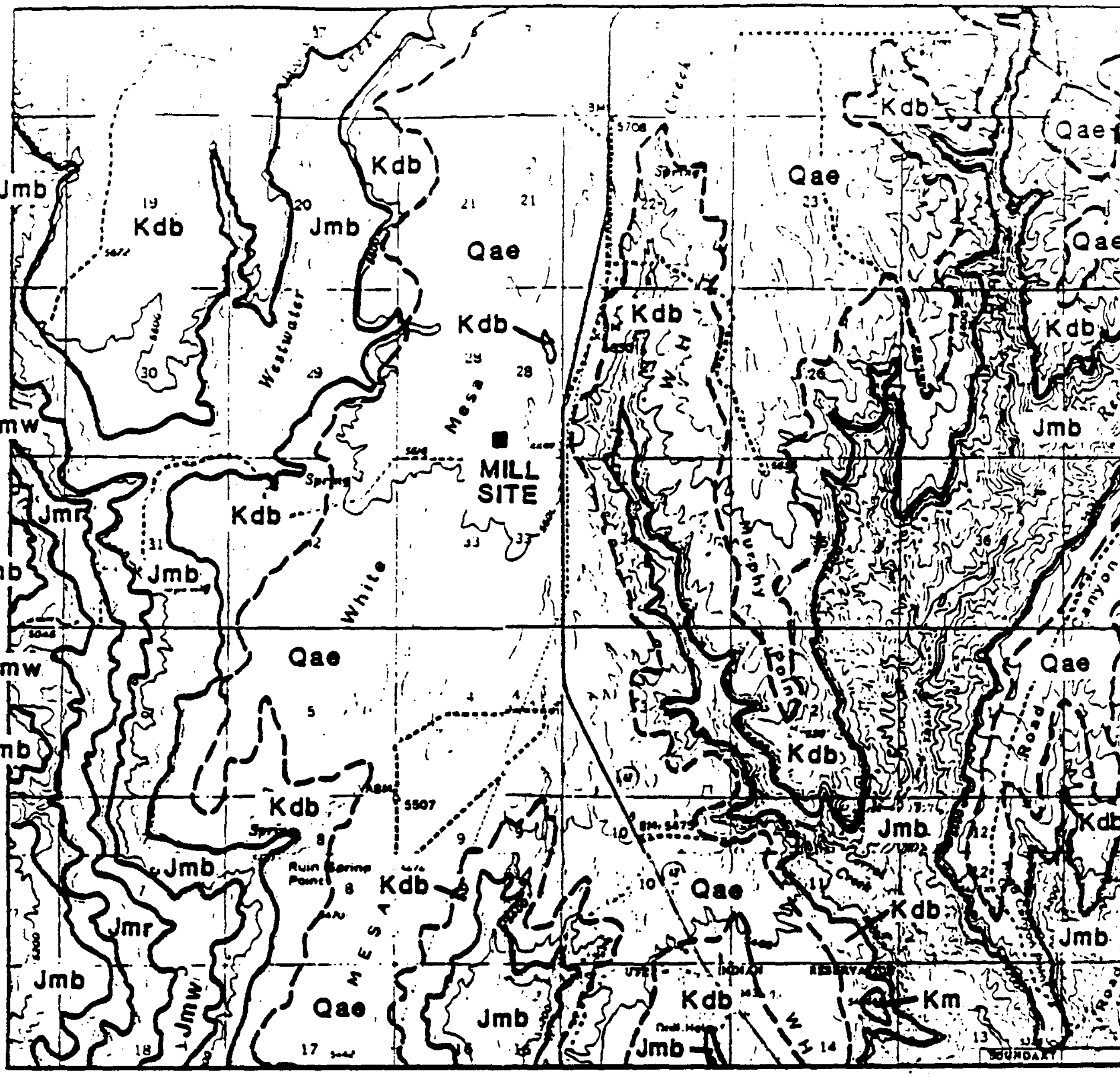
Just north of the project site, the Westwater Canyon Member intertongues with and grades into the lower part of the overlying Brushy Basin Member. Exposures of the Westwater Canyon in Cottonwood Wash are typically composed on interbedded yellowish- and greenish-gray to pinkish-gray, lenticular, fine- to coarse-grained arkosic sandstone and minor amounts of greenish-gray to reddish-brown sandy shale and mudstone. Like the Salt Wash, the Westwater Canyon Member is fluvial in origin, having been deposited by streams flowing north and northwest, coalescing with streams from the southwest depositing the upper part of the Salt Wash and the lower part of the Brushy Basin (Huff and Lesure, 1965). Several small and scattered uranium deposits in the Westwater Canyon are located in the extreme southern end of the Monticello district. Both the Recapture Member and the Westwater Canyon contain only traces of carbonaceous materials, are believed to be less favorable host rocks for uranium deposition (Johnson and Throdarson, 1966) and have very little potential for producing uranium reserves.

The lower part of the Brushy Basin is replaced by the Westwater Canyon Member in the Blanding area but the upper part of the Brushy Basin overlies this member. Composition of the Brushy Basin is dominantly variegated bentonitic mudstone and siltstone. Bedding is thin and regular and usually distinguished by color variations of gray, pale-green, reddish-brown, pale purple, and maroon. Scattered linticular thin beds of distinctive green and red chert-pebble conglomeratic sandstone are found near the base of the member, some of which contain uranium-vanadium mineralization in the southernmost part of the Monticello district (Haynes et al., 1972). Thin discontinuous beds of limestone and beds of grayish-red to greenish-black siltstone of local extent suggest that much of the Brushy Basin is probably lacustrine in origin.

For the most part, the Great Sage Plain owes its existence to the erosion of resistant sandstones and conglomerates of the Lower Cretaceous Burro Canyon Formation. This formation unconformably(?) overlies the Brushy Basin and the contact is concealed over most of the project area by talus blocks and slope wash. Massive, light-gray to light yellowish-brown sandstone, conglomeratic sandstone and conglomerate comprise more than two-thirds of the formation's thickness. The conglomerate and sandstone are interbedded and usually grade from one to the other. However, most of the conglomerate is near the base. These rocks are massive cross-bedded units formed by a series of interbedded lenses, each lens representing a scour filled with stream-deposited sediments. In places the formation contains greenish-gray lenticular beds of mudstone and claystone. Most of the Burro Canyon is exposed in the vertical cliffs separating the relatively flat surface of White Mesa from the canyons to the west and east. In some places the resistant basal sandstone beds of the overlying Dakota Sandstone are exposed at the top of the cliffs, but entire cliffs of Burro Canyon are most common. Where the sandstones of the Dakota rest on sandstones and conglomerates of the Burro Canyon, the contact between the two is very difficult to identify and most investigators map the two formations as a single unit (Figure A-2). At best, the contact can be defined as the top of a silicified zone in the upper part of the Burro Canyon that appears to be remnants of an ancient soil that formed during a long period of weathering prior to Dakota deposition (Huff and Lesure, 1965).

The Upper Cretaceous Dakota Sandstone disconformably overlies the Burro Canyon Formation. Locally, the disconformity is marked by shallow depressions in the top of the Burro Canyon filled with Dakota sediments containing angular to sub-rounded rock fragments probably derived from Burro Canyon strata (Witkind, 1964) but the contact is concealed at the project site. The Dakota is composed predominantly of pale yellowish-brown to light gray, massive, intricately cross-bedded, fine- to coarse-grained quartzose sandstone locally well-cemented with silica and calcite; elsewhere it is weakly cemented and friable. Scattered throughout the sandstone are lenses of conglomerate, dark-gray carbonceous mudstones and shale and, in some instances, impure coal. In general, the lower part of the Dakota is more conglomeratic and contains more cross-bedded sandstone than the upper part which in normally more thinly bedded an marine-like in appearance. The basal sandstones and conglomerates are fluvial in origin, whereas the carbonaceous mudstones and shales were probably deposited in back water areas behind beach ridges in front of the advancing Late Cretaceous sea (Huff and Lesure, 1965). The upper sandstones probably represent littoral marine deposits since they grade upward into the dark-gray siltstones and marine shales of the Mancos Shale.

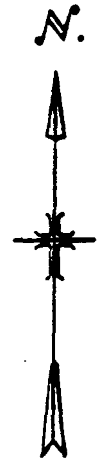
The Mancos shale is not exposed in the project vicinity. The nearest exposures are small isolated remnants resting conformably on Dakota Sandstone along the western rim above Recapture Creek 4.3 to 5.5 miles (6.9 to 8.9 kilometers) southeast of the project site. Additional exposures are found on the eastern and southern flanks of the Abajo Mountains approximately 16 to 20 miles (26 to 32 kilometers) to the north. It is possible that thin patches of Mancos may be buried at the project site but are obscurred by the mantle of alluvial windblown silt and sand covering the upland surface. The Upper



REFERENCES: GEOLOGY, IN PART, AFTER HAYNES ET AL., 1962. BASE MAP PREPARED FROM PORTIONS OF THE BLANDING, BRUSHY BASIN WASH, BLUFF, AND MONTEZUMA CREEK U.S.G.S. 15-MINUTE TOPOGRAPHIC QUADRANGLES.

EXPLANATION

Qae	LOESS
Km	MANCOS SHALE
Kdb	DAKOTA AND BURRO CANYON FORMATIONS (UNDIFFERENTIATED)
Jmb	MORRISON FORMATION: BRUSHY BASIN MEMBER
Jmw	WESTWATER CANYON MEMBER
Jmr	RECAPTURE MEMBER
▬ ‒ ‒	CONTACT, DASHED WHERE APPROXIMATE



SCALE IN FEET

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GEOLOGIC MAP OF PROJECT AREA

JUNE, 1988 FIGURE A-2

Cretaceous Mancos shale is of marine origin and consists of dark- to olive-gray shale with minor amounts of gray, fine-grained, thin-bedded to blocky limestone and siltstone in the lower part of the formation. Bedding in the Mancos is thin and well developed, and much of the shale is laminated. Where fresh, the shale is brittle and fissile and weathers to chips that are light- to yellowish-gray. Topograhic features formed by the Mancos are usually subdued and commonly displayed by low rounded hills and gentle slopes.

A layer of Quaternary to Recent reddish-brown eolian silt and fine sand is spread over the surface of the project site. Most of the loess consists of subangular to rounded frosted quartz grains that are coated with iron oxide. Basically, the loess is massive and homogeneous, ranges in thickness from a dust coating on the rocks that form the rim cliffs to more than 20 feet (6 meters), and is partially cemented wth calcium carbonate (caliche) in light-colored mottled and veined accumulations which probably represent ancient immature soil horizons.

1.2.3 Structure [2.4.2.3]

The geologic structure at the project site is comparatively simple. Strata of the underlying Mesozoic sedimentary rocks are nearly horizontal; only slight undulations along the caprock rims of the upland are perceptible and faulting is absent. In much of the area surrounding the project site the dips are less than one degree. The prevailing regional dip is about one degree to the south. The low dips and simple structure are in sharp contrast to the pronounced structural features of the Comb Ridge Monocline to the west and the Abajo Mountains to the north.

Jointing is common in the exposed Dakota-Burro Canyon sandstones along the mesa's rim. More often than not, the primary joints are virtually parallel to the cliff faces and the secondary joints are almost perpendicular to the primary joints. Since erosion of the underlying weaker Brushy Basin mudstones removes both vertical and lateral support of the sandstone, large joint blocks commonly break away from the cliff leaving joint surfaces as the cliff face. Because of this, it is not possible to determine if the joints originated after the development of the canyons or if the joints influenced the developmment of canyons and cliffs. However, from a geomorphologic standpoint, it appears that the joints are related to the compaction of the underlying strata and, therefore, are sedimentary and physiographic features rather than tectonic in origin. Whatever the original cause, two sets of joint attitudes exist in the resistant sandstones adjacent to the west side of the project site. These sets range from N.10-18°E. and N.60-85°E. and nearly parallel the cliff faces.

1.3 Seismic Risk Assessment

1.3.1 General

This review utilized the environmental assessment completed in 1978 for the White Mesa Uranium Project by Dames & Moore (1978b). Information has been updated and procedures have been modified to conform to current requirements.

1.3.2 Seismic History of Region

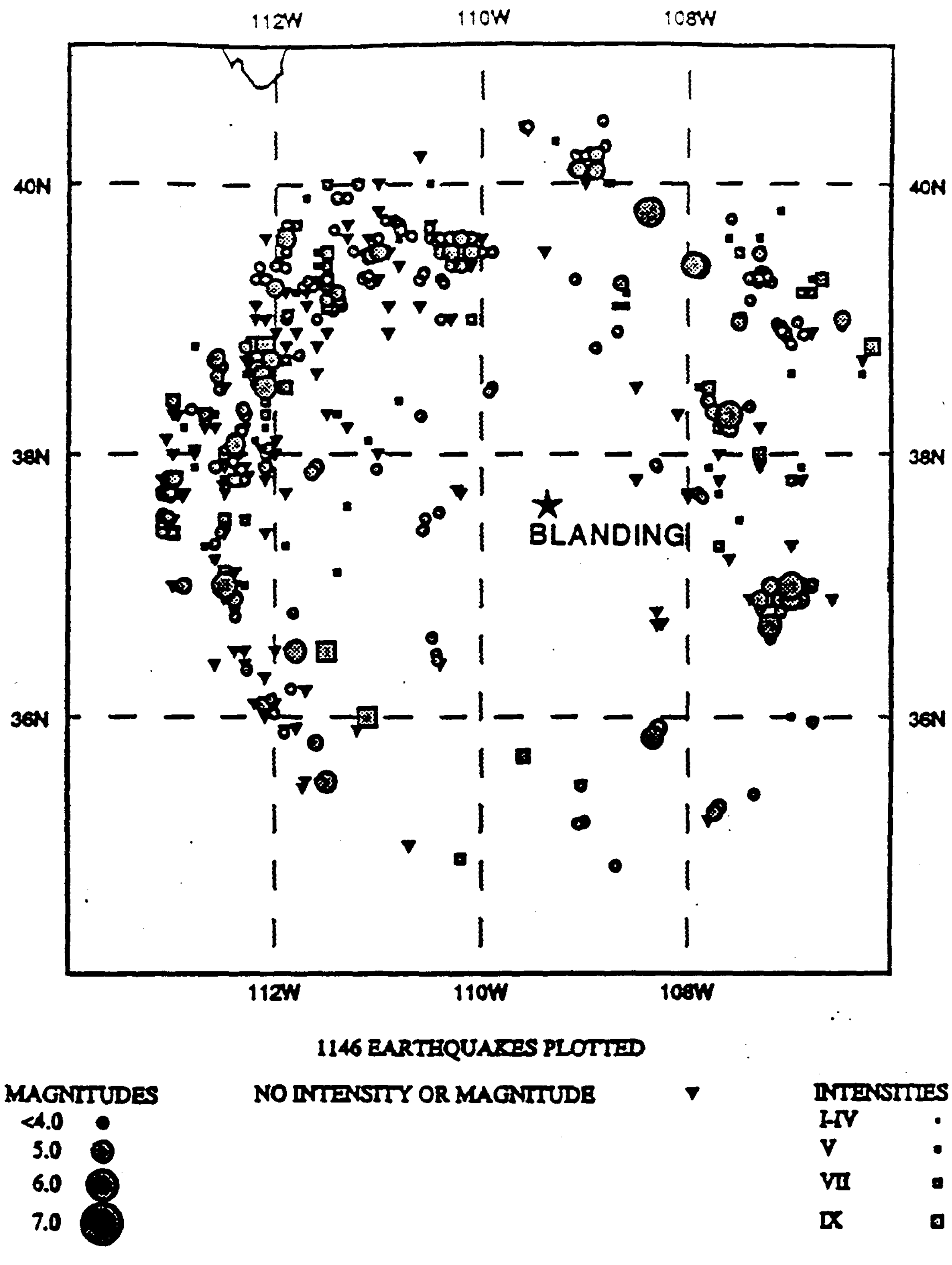
Because of the region's late settlement, the record of earthquake occurrences in the Colorado Plateau and surrounding regions dates back only 135 years. Documentation of the earlier events was based solely on newspaper reports that frequently recorded effects only in the more populated areas which may have been some distance from the epicenters. Not until the late 1950's was a seismograph network developed to properly locate and evaluate seismic events in this region (Simon, 1972).

The project area is within a relatively tectonically stable portion of the Colorado Plateau noted for its scarcity of historical seismic events. Conversely, the border between the Colorado Plateau and the Basin and Range Province and Middle Rocky Mountain Province some 155 to 240 miles (249 to 386 km) west and northwest, respectively, from the site is one of the most active seismic belts in the western United States.

The epicenters of historical earthquakes from 1853 through 1986 within a 200-mile (320-km) radius of the site are shown in Figure A-3. More than 1146 events have occurred in the area, of which at least 45 were damaging; that is, having an intensity of VI or greater on the Modified Mercalli Scale. A description of the Modified Mercalli Scale is given in Table A-3. All intensities mentioned herein refer to this table. Table A-3 also shows a generalized relationship between Mercalli intensities and other parameters to which this review will refer. Since these relationships are frequently site specific, the table values should be used only for approximation and understanding.

Only 63 non-duplicative epicenters have been recorded within a 120 mile (200 km) radius of the project area (Figure A-4). Of these, 50 had an intensity IV or less (or unrecorded) and two were recorded as intensity VI. The nearest event occurred in the Glen Canyon National Recreation Area approximately 38 miles (63 km) west-northwest of the project area. The next closest event occurred approximately 53 miles (88 km) to the northeast. Just east of Durango, Colorado, approximately 99 miles (159 km) due east of the project area, an event having local intensity of V was recorded on August 29, 1941 (Hadsell, 1968). It is very doubtful that these events would have been felt in the vicinity of Blanding.

Three of the most damaging earthquakes associated with the seismic belt along the Colorado Plateau's western border have occurred in the Elsinore-Richfield area about 168 miles (270 km) northwest of the project site. All were of intensity VIII. On November 13, 1901, a strong shock caused extensive damage from Richfield to Parowan. Many brick structures were damaged; rockslides were reported near Beaver. Earth cracks with the ejection of sand and water were reported, and some creeks increased their flow. Aftershocks continued for several weeks (von Hake, 1977). Following several weeks of small foreshocks, a strong earthquake caused major damage in the Monroe-Elsinore-Richfield area on September 29, 1921. Scores of chimneys were thrown down, plaster fell from ceilings, and a section of a new two-story brick wall



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SEISMICITY
320KM AROUND
BLANDING UTAH

JUNE, 1988 FIGURE A-3

TABLE A-3

MODIFIED MERCALLI SCALE, 1958 VERSION*

M§	Intensity	Effects	v,† cm/s	a‡
	I.	Not felt. Marginal and long-period effects of large earthquakes (for details see text).		
3	II.	Felt by persons at rest, on upper floors, or favorably placed.		
	III.	Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.		0.0035–0.007
4	IV.	Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wooden walls and frame creak.		0.007–0.015
	V.	Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.	1–3	0.015–0.035
5	VI.	Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc., off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken (visibly, or heard to rustle—CFR).	3–7	0.035–0.07
6	VII.	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments—CFR). Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.	7–20	0.07–0.15
	VIII.	Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.	20–60	0.15–0.35
7	IX.	General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. (General damage to foundations—CFR.) Frame structures, if not bolted, shifted off foundations. Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.	60–200	0.35–0.7
8	X.	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.	200–500	0.7–1.2
	XI.	Rails bent greatly. Underground pipelines completely out of service.		>1.2
	XII.	Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.	From Fig. 11.14	

NOTE: Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering (which has no connection with the conventional Class A, B, C construction).

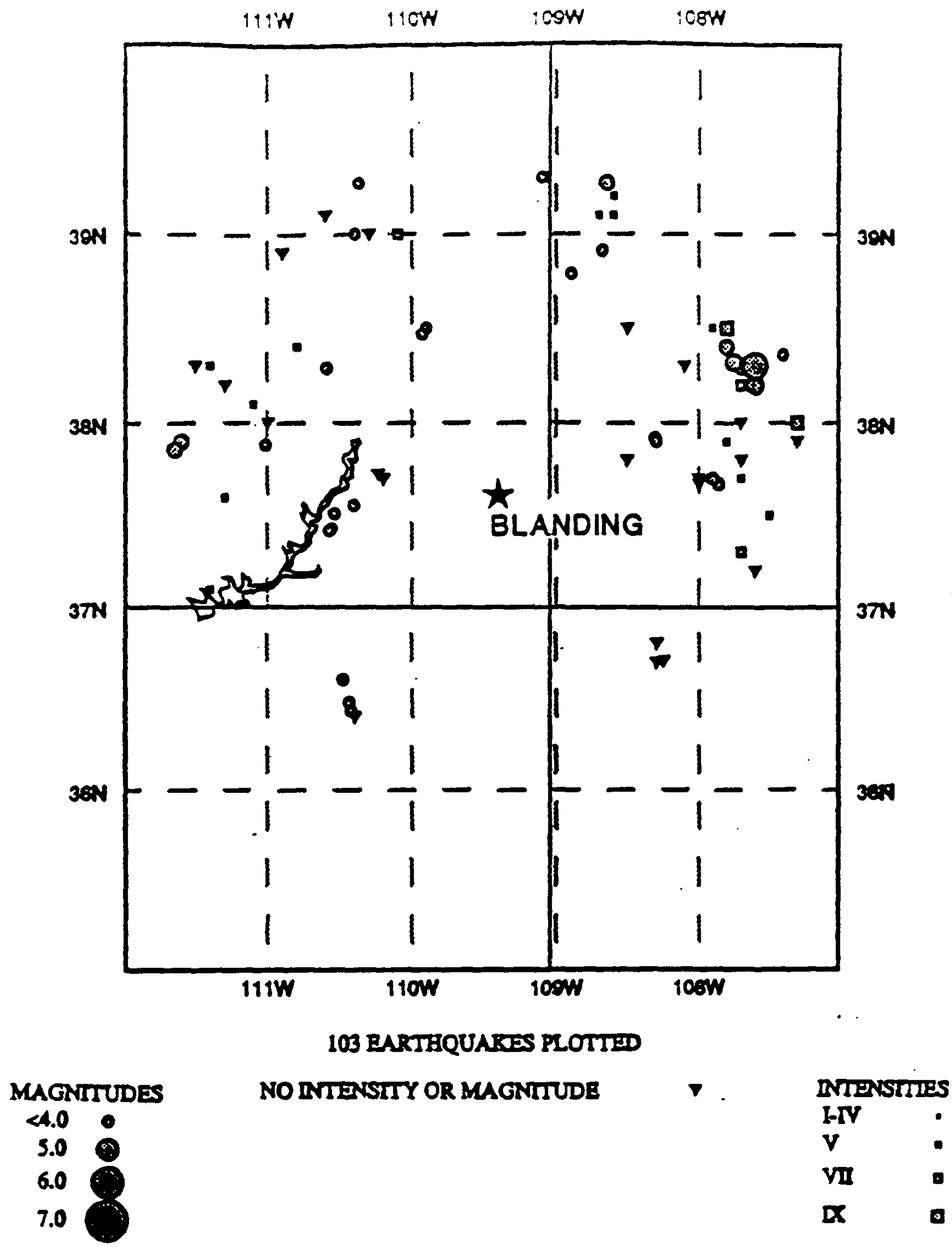
- Masonry A: Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.
- Masonry B: Good workmanship and mortar; reinforced, but not designed to resist lateral forces.
- Masonry C: Ordinary workmanship and mortar; no extreme weaknesses such as non-tied-in corners, but masonry is neither reinforced nor designed against horizontal forces.
- Masonry D: Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

*From Richter (1958).[1] Adapted with permission of W. H. Freeman and Company. by Hunt (1984).

†Average peak ground velocity, cm/s.

‡Average peak acceleration (away from source).

§Magnitude correlation.



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SEISMICITY
200KM AROUND
BLANDING UTAH

JUNE, 1988 FIGURE A-4

collapsed at Elsinore's schoolhouse. Two days later, on October 1, 1921, another strong tremor caused additional damage to the area's structures. Large rockfalls occurred along both sides of the Sevier Valley and hot springs were discolored by iron oxides (von Hake, 1977). It is probable that these shocks may have been perceptible at the project site but they certainly would not have caused any damage.

Seven events of intensity VII have been reported in the area shown in Figure A-3. Of these, only two are considered to have any significance with respect to the project site. On August 18, 1912, an intensity VII shock damaged houses in northern Arizona and was felt in Gallup, New Mexico, and southern Utah. Rock slides occurred near the epicenter in the San Francisco Mountains and a 50-mile (80 km) earth crack was reported north of the San Francisco Range (U. S. Geological Survey, 1970). Nearly every building in Dulce, New Mexico, was damaged to some degree when shook by a strong earthquake on January 22, 1966. Rockfalls and landslides occurred 10 to 15 miles (16 to 24 km) west of Dulce along Highway 17 where cracks in the pavement were reported (Hermann et al., 1980). Both of these events may have been felt at the project site but, again, would certainly not have caused any damage.

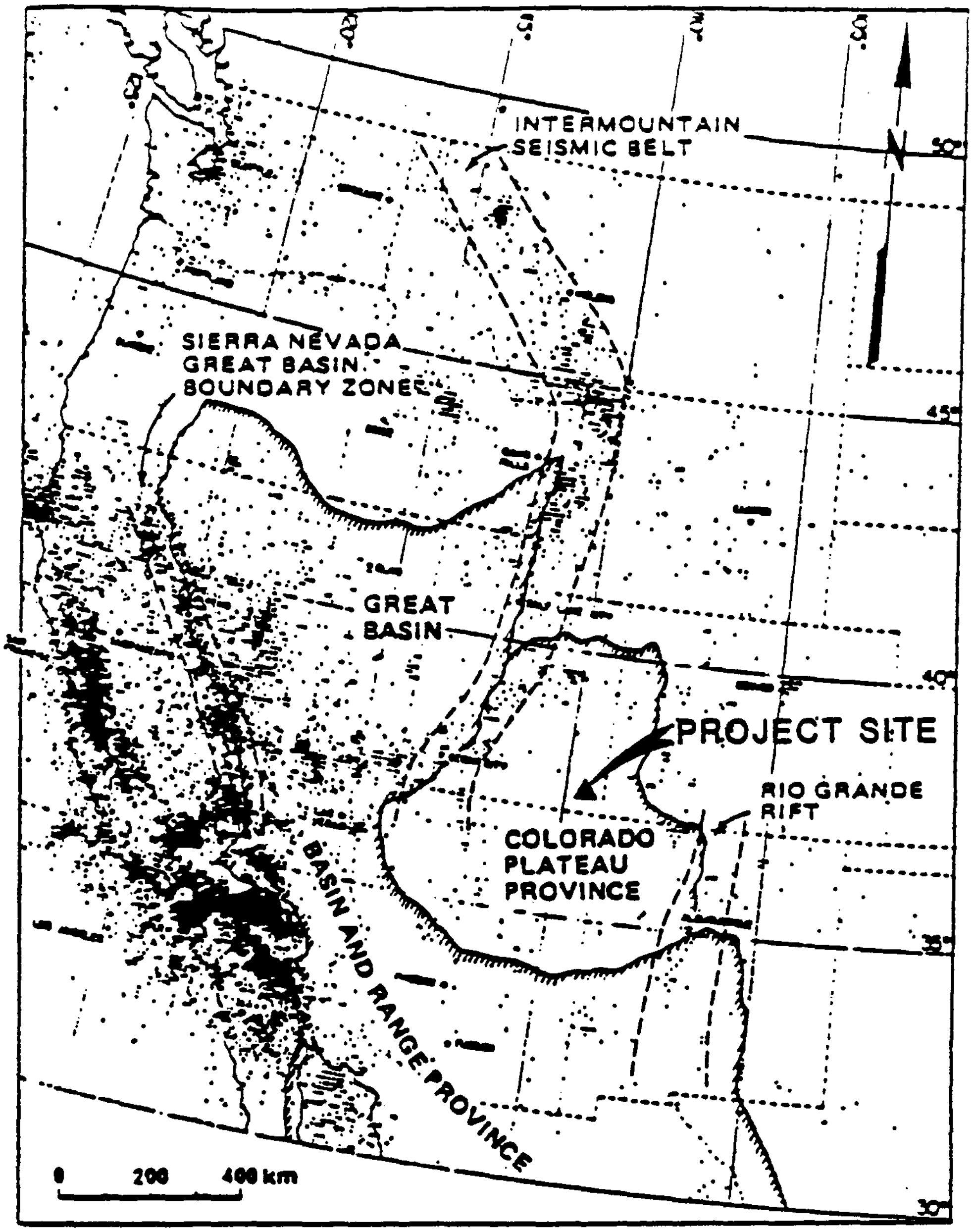
1.3.3 Relationship of Earthquakes to Tectonic Structures

The majority of recorded earthquakes in Utah have occurred along an active belt of seismicity that extends from the Gulf of California, through western Arizona, central Utah, and northward into western British Columbia. The seismic belt is possibly a branch of the active rift system associated with the landward extension of the East Pacific Rise (Cook and Smith, 1967). This belt is the Intermountain Seismic Belt shown in Figure A-5 (Smith, 1978).

It is significant to note that the seismic belt forms the boundary zone between the Basin and Range - Great Basin Provinces and the Colorado Plateau - Middle Rocky Mountain Provinces. This block-faulted zone is about 47 to 62 miles (75 to 100 km) wide and forms a tectonic transition zone between the relatively simple structures of the Colorado Plateau and the complex fault-controlled structures of the Basin and Range Province (Cook and Smith, 1967).

Another zone of seismic activity is in the vicinity of Dulce, New Mexico, near the Colorado border. This zone, which coincides with an extensive series of Tertiary intrusives, may also be related to the northern end of the Rio Grande Rift. This rift is a series of fault-controlled structural depressions extending southward from southern Colorado through central New Mexico and into Mexico. The rift is shown on Figure A-5 trending north-south to the east of the project area.

Most of the events south of the Utah border of intensity V and greater are located within 50 miles (80 km) of post-Oligocene extrusives. This relationship is not surprising because it has been observed in many other parts of the world (Hadsell, 1968).



Modified from Smith, 1978

SHOWS RELATIONSHIP OF THE COLORADO PLATEAU PROVINCE TO MARCANAL BELTS

UMETCO MINERALS CORPORATION

WHITE MESA PROJECT

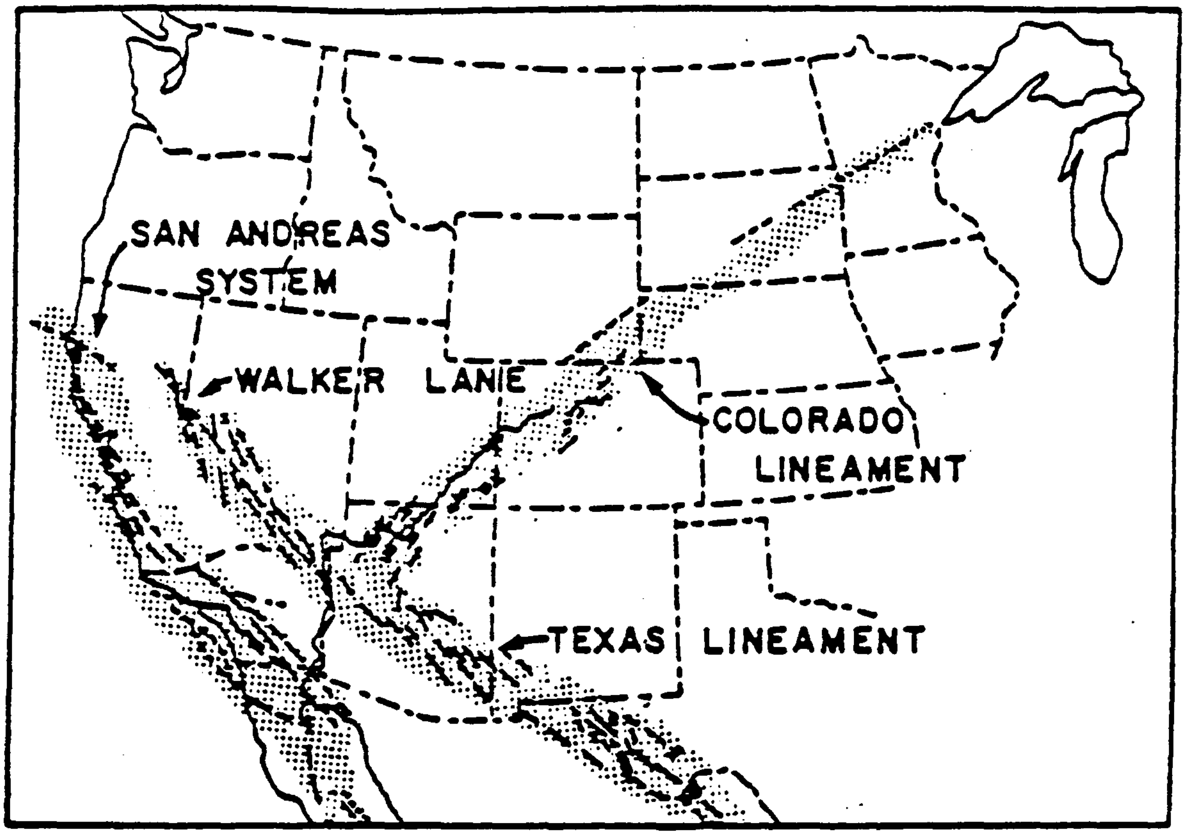
SEISMICITY OF THE WESTERN UNITED STATES, 1950 TO 1976

JUNE, 1988 FIGURE A-5

In Colorado, the Rio Grande Rift zone is one of three seismotectonic provinces that may contribute energy to the study area. Prominent physiographic expression of the rift includes the San Luis Valley in southern Colorado. The valley is a half-graben structure with major faulting on the eastern flank. Extensional tectonics is dominant in the area and very large earthquakes with recurrence intervals of several thousand years have been projected (Kirkham and Rodgers, 1981). Mountainous areas to the west of the Rio Grande rift province include the San Juan Mountains. These mountains are a complex domical uplift with extensive Oligocene and Miocene volcanic cover. Many faults are associated with the collapse of the calderas and apparently have not moved since. Faults of Neogene age exist in the eastern San Juan Mountains that may be related to the extension of the Rio Grande rift. Numerous small earthquakes have been felt or recorded in the western mountainous province despite an absence of major Neogene tectonic faults (Kirkham and Rodgers, 1981).

The third seismotectonic province in Colorado, that of the Colorado Plateau, extends into the surrounding states to the west and south. In Colorado, the major tectonic element that has been recurrently active in the Quaternary is the Uncompahgre uplift. Both flanks are faulted and earthquakes have been felt in the area. The faults associated with the Salt Anticlines are collapsed features produced by evaporite solution and flowage (Cater, 1970). Their non-tectonic origin and the plastic deformation of the salt reduces their potential for generating even moderate-sized earthquakes (Kirkham and Rodgers, 1981).

Case and Joesting (1972) have called attention to the fact that regional seismicity of the Colorado Plateau includes a component added by basement faulting. They inferred a basement fault trending northeast along the axis of the Colorado River through Canyonlands. This basement faulting may be part of the much larger structure that Hite (1975) examined and Warner (1978) named the Colorado lineament (Figure A-6). This 1300-mile (2100-km) long lineament that extends from northern Arizona to Minnesota is suggested to be a Precambrian wrench-fault system formed some 2.0 to 1.7 billion years before present. While it has been suggested that the Colorado lineament is a source zone for larger earthquakes ($m = 4$ to 6) in the west-central United States, the observed spatial relationship between epicenters and the trace of the lineament does not prove a causal relation (Brill and Nuttli, 1983). In terms of contemporary seismicity, the lineament does not act as a uniform earthquake generator. Only specific portions of the proposed structure can presently be considered seismic source zones and each segment exhibits seismicity of distinctive activity and character (Wong, 1981). This is a reflection of the different orientations and magnitudes of the stress fields along the lineament. The interior of the Colorado Plateau forms a tectonic stress province, as defined by Zoback and Zoback (1980), that is characterized by generally east-west tectonic compression. Only where extensional stresses from the Basin and Range province of the Rio Grande rift extend into the Colorado Plateau would the Colorado lineament in the local area be suspected of having the capability of generating a large magnitude earthquake (Wong, 1984). At the present time, the well defined surface expression of regional extension is far to the west and far to the east of the project area.



SOURCE: WARNER, 1978

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COLORADO LINEAMENT

JUNE, 1988 FIGURE A-6

Recent work by Wong (1984) has helped define the seismicity of the whole Colorado Plateau. He called attention to the low level (less than $M_L = 3.6$) but high number (30) of earthquakes in the Capitol Reef area from 1978 to 1980 that were associated with the Waterpocket fold and the Cainville monocline, two other major tectonic features of the Colorado Plateau. Only five earthquakes in the sequence were of M_L greater than 3, and fault plane solutions suggest the swarm was produced by normal faulting along northwest-trending Precambrian basement structures (Wong, 1984). The significance of the Capitol Reef seismicity is its relatively isolated occurrence within the Colorado Plateau and its location at a geometric barrier in the regional stress field (Aki, 1979). Stress concentration that produces earthquakes at bends or junctures of basement faults as indicated by this swarm may be expected to occur at other locations in the Colorado Plateau Province. No inference that earthquakes such as those at Capitol Reef are precursors for larger subsequent events is implied.

1.3.4 Potential Earthquake Hazards to Project

The project site is located in a region known for its scarcity of recorded seismic events. Althought the seismic history for this region is barely 135 years old, the epicentral pattern, or fabric, is basically set and appreciable changes are not expected to occur. Most of the larger seismic events in the Colorado Plateau have occurred along its margins rather than in the interior central region. Based on the region's seismic history, the probability of a major damaging earthquake occurring at or near the project site is very remote. Studies by Algermissen and Perkins (1976) indicate that southeastern Utah, including the site, is in an area where there is a 90 per cent probability that a horizontal acceleration of four per cent gravity (0.04g) would not be exceeded within 50 years.

Minor earthquakes, not associated with any seismic-tectonic trends, can presumably occur randomly at almost any location. Even if such an event with an intensity as high as VI should occur at or near the project site, horizontal ground accelerations would not exceed 0.10g but would probably range between 0.05 an 0.09g (Coulter et al., 1973; Trifunac and Brady, 1975). These magnitudes of ground motion would not pose significant hazards to the existing and proposed facilities at the Project Site.

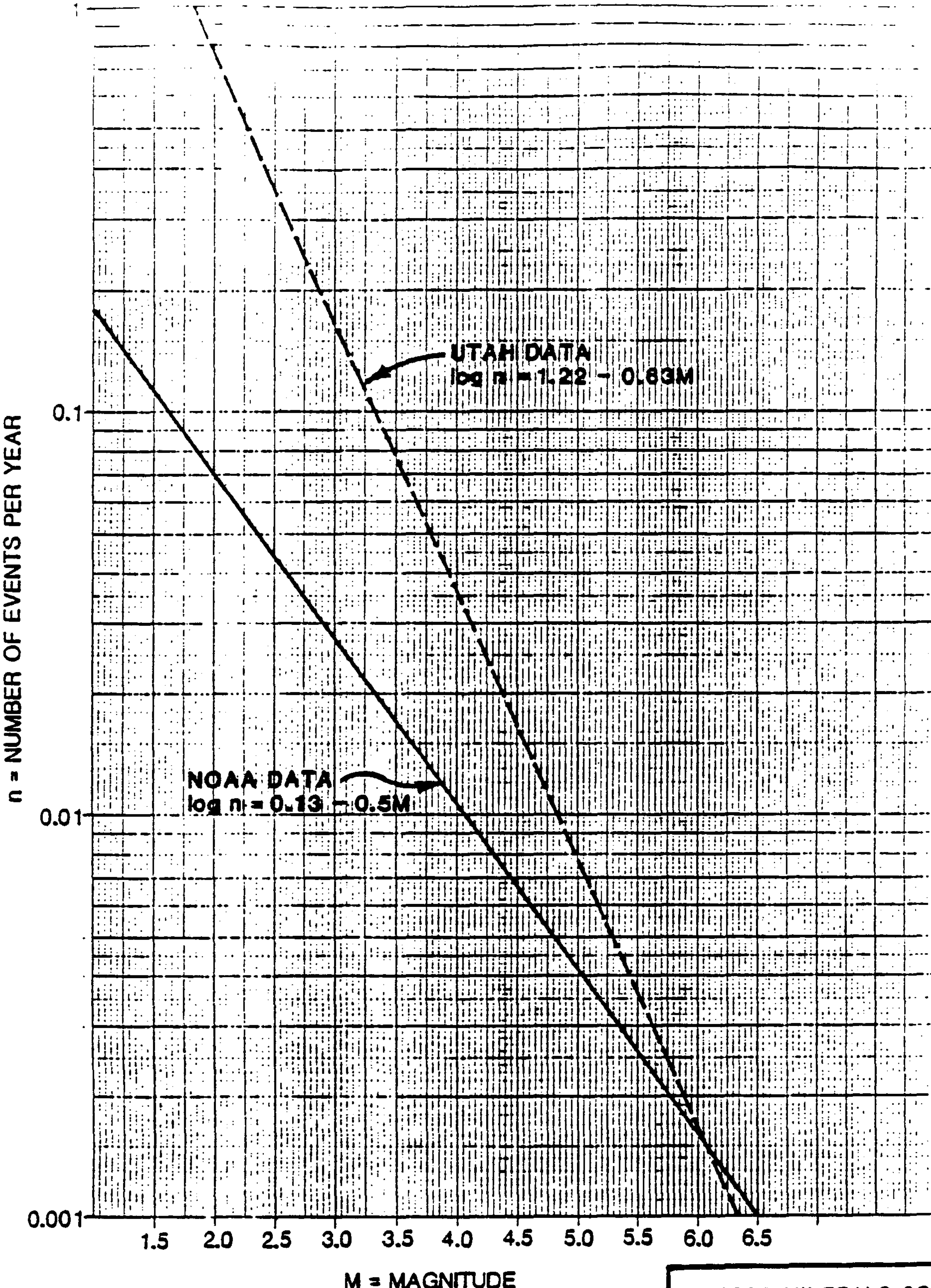
In addition to general estimates of earthquake hazards, such as those offered by Dames and Moore (1978b), a more detailed analysis of the relationship between the project area and regional seismicity was performed. As can be seen in Figure A-3, a map based on the seismologic data base from the National Geophysical Data Center of the National Oceanic and Atmospheric Administration (NOAA 1988), many events occur within the Intermountain Seismic Belt and within the Rio Grande rift. Since the Colorado Plateau Province, and particularly the Blanding basin portion, in which the project site lies, is a distinctly different tectonic province, the historical sample chosen for magnitude/frequency estimates was limited to a radius of about 120 miles (200 km) from the project. This sample included a region which is more representative of the seismicity of the Colorado Plateau.

The map in Figure A-4 shows a plot of earthquake epicenters within 124 miles (200 km) of Blanding. In comparing this map with Figure A-3, it is apparent that the project site is located in an area of very low seismic flux compared to the marginal areas to both east and west. Using a sample of 46 events ranging in magnitude from 2 to 5.5 and covering a time span of 135 years, the calculated yearly rates for each magnitude range were normalized to an area of 6,823 square miles (17,671 km^2) [47-mile (75-km) radius]. A Poisson distribution was assumed and a linear regression produced the best fit line of magnitude vs. frequency shown in Figure A-7. The form of $\log(n) = b + aM$ (where n is the expected number of events per year of magnitude M) was used. The sample of 46 events included all non-duplicative epicenters with reported magnitude or intensity. Intensities were converted to magnitudes where necessary using the equation $M = 2.1 + 0.5I$ as recommended by Krinitsky and Chang (1975). The resulting recurrence intervals of events of a specific magnitude are considerably longer than for the same size event in the more active zones to the east and west but are more realistic of the Blanding area (Arabasz, Smith and Richins, 1979; Kirkham and Rodgers, 1981). The magnitude/frequency relationship determined is not intended to represent true rates but is only a conservative estimate based on samples which are representative of only the area near the White Mesa project site.

The historical records from both NOAA and Utah data sources list only five events which have occurred within 62 miles (100 km) of the study site. The largest of these relatively near events was a magnitude 4.0 event on July 22, 1986, that occurred 61.5 miles (99 km) west of the site. In the 124 miles (200 km) radius data set the largest event is one of magnitude 5.5 on October 11, 1960, at a distance of 112 miles (180 km) from the project.

The nearest fault to the site identified on maps by Andersen and Miller (1979) as youthful and therefore presumably potentially active is 25 miles (57 km) north of the project site (Figure A-8). The fault, as mapped, has a well defined length of about 1.9 miles (3 km) and a possible total length of 6.8 miles (11 km). The fault is identified by Anderson and Miller as a suspected Quaternary fault, but may not be the result of tectonic activity. The evidence for Quaternary movement on this fault is not strong and it appears to be a discrete extension of the South fault of the Shay graben. No other part of the surface expression of the Shay graben structure gives any indication that the fault's traces are anything other than erosionally produced fault-line scarps. Nevertheless, it is appropriate to use this mapped suspected Quaternary fault for estimates for seismic design criteria since it is the nearest fault or fault segment to the project area that has been mapped as young. Additional discussion of the faulting in the Shay and Verdure grabens (Figure A-8) is included below.

Estimates of the maximum credible earthquake from this fault 35 miles (57 km) north of the project assuming rupture of 1.9 miles (3 km) and 6.7 miles (11 km), as well as a one-half total length ruture 3.4 miles (5.5 km) have been made using the relationships discussed in Slemmons (1977). These are summarized in Tables A-4 and A-5. Determinative assumptions included the expected length of surface rupture, type of fault movement, and regional location. This least squares approach to the regression analysis was well validated by the studies of Bonilla et al. (1984).



COVERS AREA WITHIN A 75 KM RADIUS OF THE PROJECT SITE

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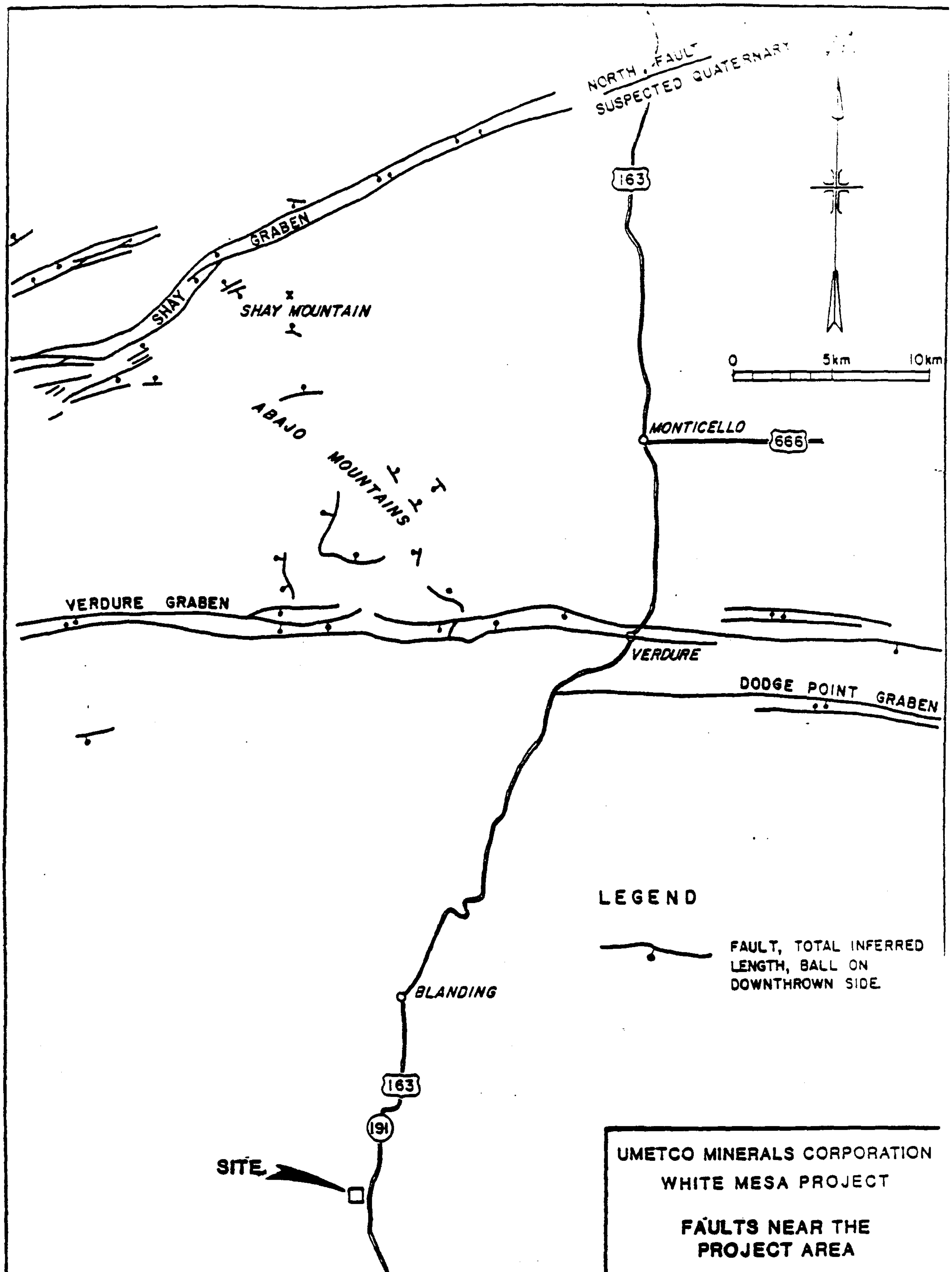
MAGNITUDE / FREQUENCY RELATIONSHIP

JUNE, 1988 FIGURE A-7



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FAULTS NEAR THE PROJECT AREA

JUNE, 1988 FIGURE A-8

Table A-4

Maximum Fault Displacement (D)
of
North Fault (Figure A-8)

Parameter	a	b	D = 3 km	D = 5.5 km	D = 11 km
NA	-4.270	1.036	0.08 m	0.14 m	0.29 m
A	-4.375	1.014	0.14 m	0.26 m	0.53 m
A+C	-2.898	0.705	0.36 m	0.55 m	0.89 m

Relationship: Log D = a + b Log L

From: Slemmons (1977)

Table A-5

Maximum Magnitude (M)
of
North Fault (Figure A-8)

Relationship	Parameter	a	b	M = 3 km	M = 5.5 km	M = 11 km
	NA	-0.146	1.504	5.08	5.48	5.93
M = a + b Log L	A	1.845	1.151	5.85	6.15	6.50
	A+C	2.042	1.121	5.94	6.23	6.57
	NA	6.745	0.995	5.65	5.90	6.21
M = a + b Log D	A	6.827	1.050	5.93	6.21	6.54
	A+C	6.757	1.226	6.21	6.44	6.69
	NA	3.510	0.701	5.18	5.53	5.97
M = a + b Log LD	A	4.551	0.530	5.94	6.22	6.55
	A+C	3.691	0.707	5.84	6.15	6.51
	NA	4.808	0.420	5.35	5.66	6.05
M = a + b Log LD^2	A	5.568	0.299	6.10	6.34	6.61
	A+C	4.752	0.459	5.94	6.23	6.56
			MEAN	5.75	6.04	6.39
MAXIMUM MAGNITUDE (MEAN + S)				6.11	6.37	6.66

NA = North America
A = Normal Movement
A + C = Combined Normal and Oblique Movement

From: Slemmons (1977)

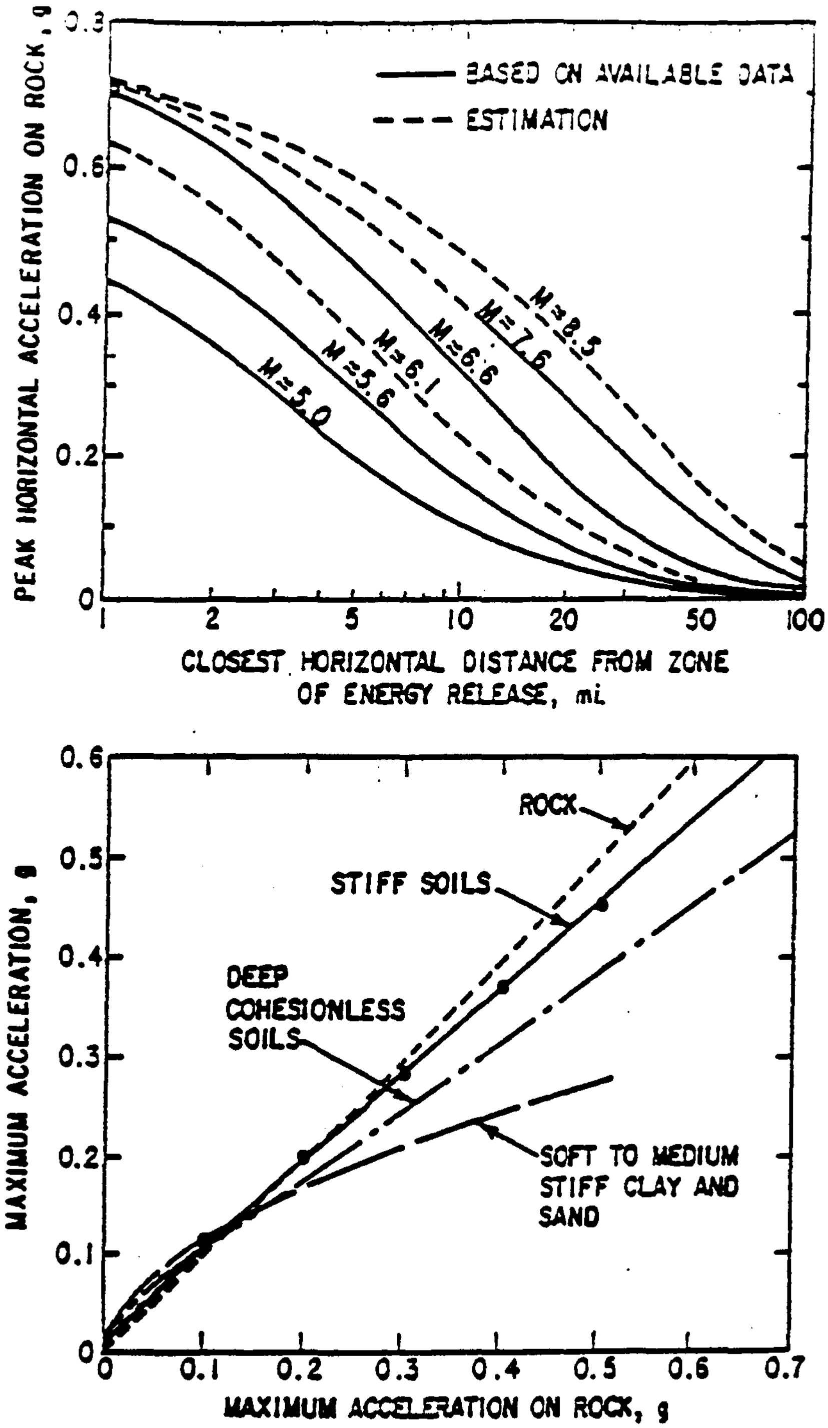
The maximum estimated magnitude of this fault assuming a 3.4 Miles (5.5 km) rupture length is 6.4. This is consistent with estimates of maximum magnitude for the Colorado Plateau by Kirkham and Rodgers (1981) and by Andersen and Miller (1979). Based on the estimated historical magnitude/frequency relationships, this maximum earthquake would have a recurrence interval of about 1000 years. Because the historical magnitude/frequency relationship estimated was not constrained at its upper right limit (say to a value of M 6.5), it is reasonable to deduce that recurrence-intervals determined are shorter (more conservative) than their actual rate of recurrence.

Ground motions at the White Mesa project site resulting from an maximum credible earthquake (MCE) on the fault discussed can be estimated from attenuation curves established by Seed and Idriss (1982). The graph in Figure A-9 indicates an estimated peak horizontal acceleration at a distance of 57 km from the epicenter would be 0.07g.

Note that conservative estimates were used by assuming an estimated magnitude of 6.4 based on the mean plus one standard deviation and assuming the earthquake would initiate at a point on the fault closest 35 miles (57 km) to the project site.

The MCE and possible horizontal acceleration estimate have a very low probability of occurrence but would be used for seismic design parameters for a worst case scenario based on this mapped fault.

Faults associated with the Verdure graben are even closer to the White Mesa Project than the fault discussed above but they show no evidence of recent surface displacement. These faults are shown on the map on Figure A-8 in spite of not being shown on the Quaternary Fault Map of Utah because of their surface expression and similarity in trend and structure to Shay graben to the north (Anderson and Miller, 1979; Woodward-Clyde Consultants, 1982). During micro-seismic monitoring from 1979 through 1982, Wong (1984) recorded only two events associated with these east-west grabens. Both of the micro-earthquakes (M_L 0.6 and M_L 0.7) were in the Abajo Mountains near the western end of the southern fault trace of the Shay graben. Data resolution did not permit these very small events to be assigned to the south fault and they may well be related to the old intrusions. However, this is the only indication found that any of these structures may be active. Detailed field study and mapping of the Verdure graben show Quaternary pediment gravels and alluvium overlying the graben in several places and Witkind (1964) indicated that the south fault was in igneous contact with the Rocky Trail laccolith with no slickensides present (Woodward-Clyde Consultants, 1982). Stream courses cross the faults of the graben with no deflection or gradient change. The implication is that these faults are old and may be of Oligocene age and related to the period of laccolithic intrusion. It is also possible, as many authors believe, that these fault structures are the result of salt flowage in underlying formations and not produced by tectonic stresses (Kirkham and Rodgers, 1981). Since the regional stress field in this area is approximately east-west, it is difficult to see how the normal faulting of the Verdure graben or the Shay graben and its possible extension that forms the small fault north of Monticello could be



(From Seed and Idriss, 1982)

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GROUND ACCELERATION CURVES

JUNE, 1988 FIGURE A-9

produced by tectonics stresses (Wong, 1984). If the faults are non-tectonic they are not likely to be capable of generating earthquakes larger than magnitude 4 or 5. As mentioned before, probabilistic maps by Algermissen and Perkins (1976) indicate there is a 90 percent probability that horizontal acceleration will not exceed 0.04 g in 50 years in the project area. This is in keeping with the projected MCE from the fault north of Monticello that presents some weak geologic evidence of Quaternary movement. As discussed above, this fault and its MCE should be used for seismic design at the White Mesa Project.

2.0 SURFACE WATER (Section 2.6.1)*

2.1 Surface Water Description (Section 2.6.1.1)

The mill site is located on White Mesa, a gently sloping (1% SSW) plateau that is physically defined by the adjacent drainages which have cut deeply into regional sandstone formations. There is a small drainage area of approximately 62 acres (25 ha) above the site that could yield surface runoff to the site. Runoff from the project area is conducted by the general surface topography to either Westwater Creek, Corral Creek, or to the south into an unnamed branch of Cottonwood Wash. Local porous soil conditions, topography, and low average annual rainfall [11.8 inches (30 cm)] cause these streams to be intermittently active, responding to spring snowmelt and local rainstorms (particularly thunderstorms). Surface runoff from approximately 384 acres (155 ha) of the project site drains westward and is collected by Westwater Creek, and runoff from another 384 acres (155 ha) drains east into Corral Creek. The remaining 713 acres (289 ha) of the southern and southwestern portions of the site drain indirectly into Cottonwood Wash (Dames & Moore, 1978b, p. 2-143). The site and vicinity drainages carry water only on an intermittent basis. The major drainages in the project vicinity are depicted in Figure A-10 and their drainages tabulated in Table A-6. Total runoff from the site (total yield per watershed area) is estimated to be less than 0.5 inch (1.3 cm) annually (Dames & Moore, 1978b, p. 2-143).

There are no perennial surface waters on or in the vicinity of the project site. This is due to the gentle slope of the mesa on which the site is located, the low average annual rainfall of 11.8 inches (29.7 cm) per year at Blanding (Dames & Moore, 1978b, p. 2-168), local soil characteristics and the porous nature of local stream channels. Prior to construction, three small ephermeral catch basins were present on the site to the northwest and northeast of the scale house.

Corral Creek is an intermittent tributary to Recapture Creek. The drainage area of that portion of Corral Creek above and including drainage from the eastern portion of the site is about 5 square miles (13 km^2). Westwater Creek is also an intermittent tributary of Cottonwood Wash. The Westwater Creek drainage basin covers nearly 27 square miles (70 km^2) at its confluence with Cottonwood Wash 1.5 miles (2.5 km) west of the project site. Both Recapture Creek and Cottonwood Wash are similarly intermittently active,

*From Final Environmental Statement (US NRC 1979)

Figure A-10. Drainage map of the vicinity of the White Mesa Project.

<u>Source:</u> Dames & Moore (1978b), Plate 2.6-5

Table A-6

Drainage Areas of Project Vicinity and Region

Basin description	Drainage area	
	km²	sq miles
Corral Creek at confluence with Recapture Creek	15.0	5.8
Westwater Creek at confluence with Cottonwood Wash	68.8	26.6
Cottonwood Wash at USGS gage west of project site	≤531	≤205
Cottonwood Wash at confluence with San Juan River	≤860	≤332
Recapture Creek at USGS gage	9.8	3.8
Recapture Creek at confluence with San Juan River	≤518	≤200
San Juan River at USGS gage downstream of Bluff, Utah	≤60,000	≤23,000

Source: Dames & Moore (1978b), Table 2.6-3 although they carry water more often and for longer periods of time due to their larger watershed areas. They both drain to the south and are tributaries of the San Juan River. The confluences of Recapture Creek and Cottonwood Wash with the San Juan River are approximately 18 miles (29 km) south of the project site. The San Juan River, a major tributary for the upper Colorado River, has a drainage of 23,000 square miles (60,000 km^2) measured at the USGS gauge to the west of Bluff, Utah (Dames & Moore, 1978b, p. 2-130).

Storm runoff in these streams is characterized by a rapid rise in the flow rates, followed by rapid recession primarily due to the small storage capacity of the surface soils in the area. For example, on August 1, 1968, a flow of 20,500 cfs (581 m^3/sec) was recorded in Cottonwood Wash near Blanding. The average flow for that day, however, was only 4340 cfs (123 m^3/sec). By August 4, the flow had returned to 16 cfs (0.5 m^3/sec) (Dames & Moore, 1978b, p. 2-135). Monthly streamflow summaries are presented in Figure A-11 for Cottonwood Wash and Recapture Creek. Flow data are not available for the two smaller water courses closest to the project site, Corral Creek and Westwater Creek, because these streams carry water infrequently and only in response to local heavy rainfall and snowmelt, which occurs primarily in the months of April, August, and October. Flow typically ceases in Corral and Westwater Creeks within 6 to 48 hours after precipitation or snowmelt ends.

2.2 Surface Water Quality (Section 2.6.1.2)

Sampling of surface water quality in the project vicinity began in July 1977 and continued through March 1978. Baseline data describe and evaluate existing conditions at the project site and vicinity. Sampling of the temporary on-site surface waters (two catch basins) has been attempted but without success because of the lack of naturally occurring water in these basins. The basin to the northeast of the mill site has been filled with well water to serve as a nonpotable water source during construction of office and laboratory buildings in conjunction with the mill (approximately six months). This water has not been sampled but presumably reflects the poor quality associated with local groundwater. Sampling of ephemeral surface waters in the vicinity has necessitated correlation with major precipitation events as these water courses are normally dry at other times.

The locations of the surface water sample sites are presented in Figure A-12. The water quality values obtained for these sample sites are given in Dames & Moore (1978b) Table 2.6-7 and U.S. NRC (1979) Table 2.22. Water quality samples were collected during the spring at several intermittently active streams that drain the project area. These streams include Westwater Creek (S1R, S9) Corral Creek below the small irrigation pond (S3R), the junction of Corral Creek and Recapture Creek (S4R), and Cottonwood Creek (S8R). Samples were also taken from a surface pond southeast of the mill (S5R). No samples were taken at S2R on Corral Creek or at the small wash (S6R) located south of the site.

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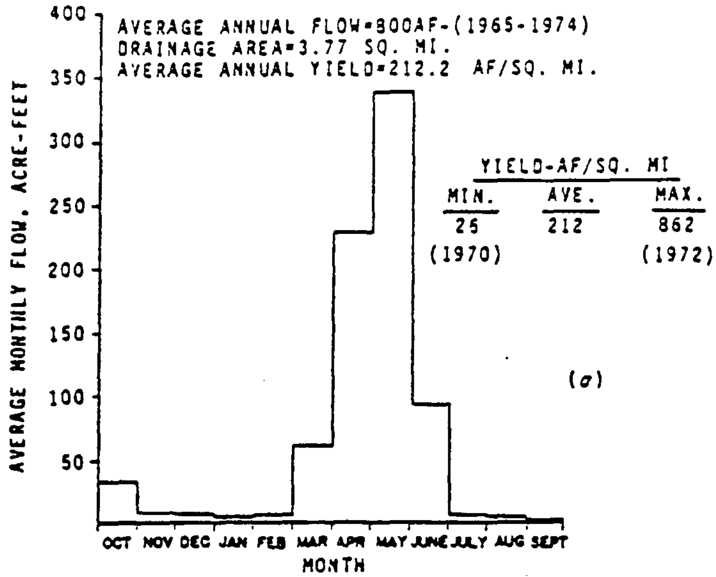


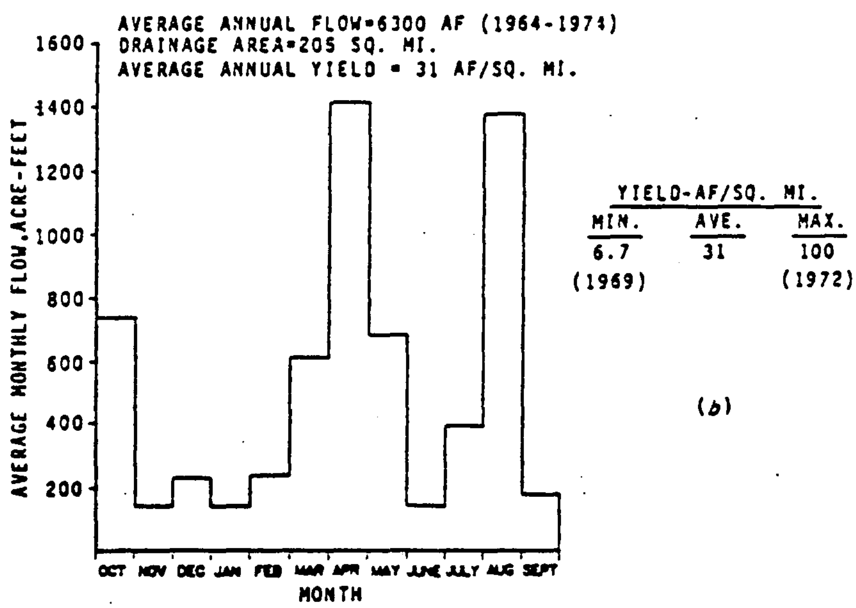


Figure A-11. Streamflow summary in the Blanding, Utah vicinity. (a) Upper portion of the watershed near the headwaters of Recapture Creek near Blanding at 7200 feet MSL; USGS gauge 09378630. (b) Cottonwood Wash about 7 miles (11 km) southwest of Blanding, Utah at 5138 feet MSL; USGS gauge 09378700. Source: Adapted from Dames & Moore (1978b), Plate 2.6-6.

ES-4589

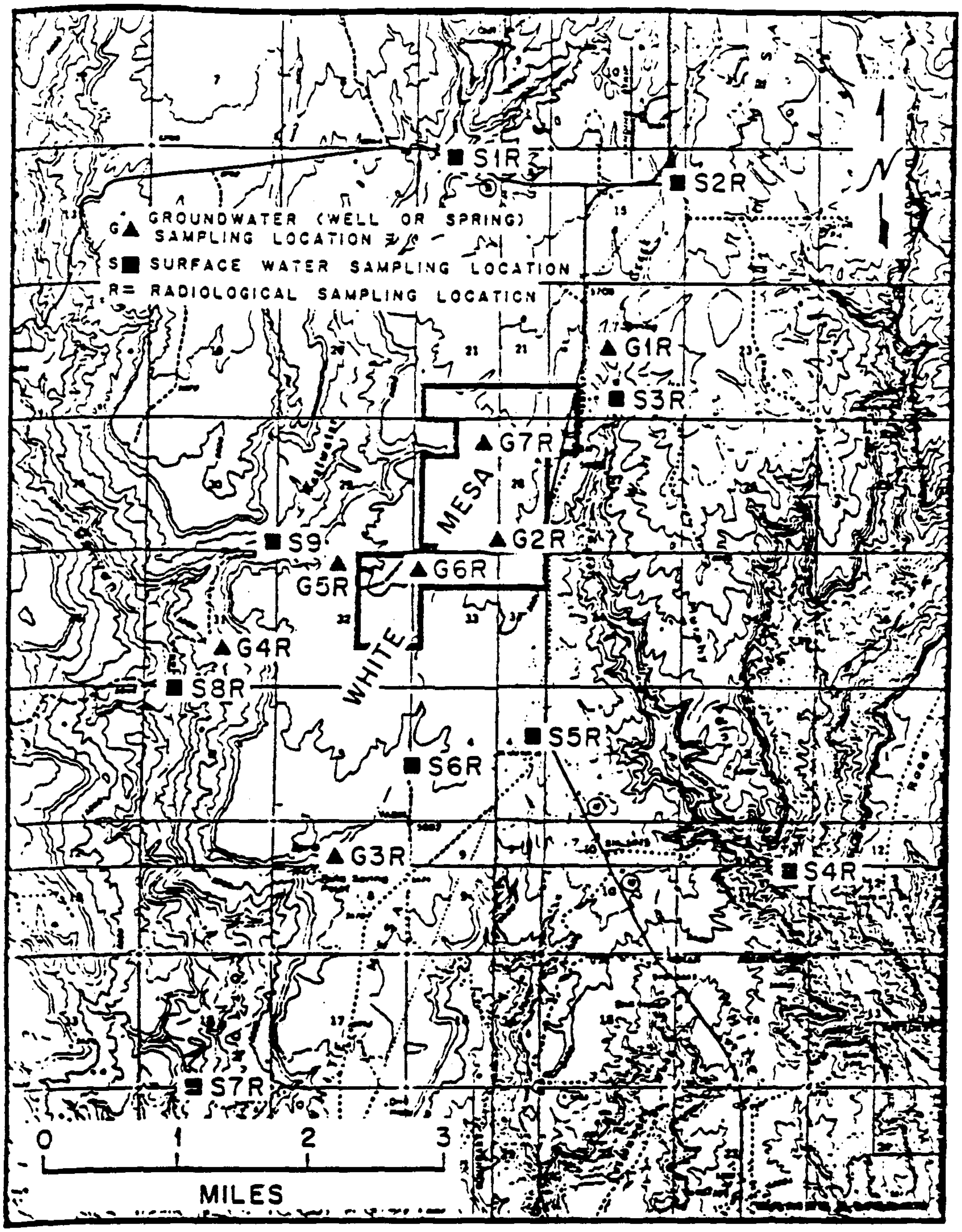


Figure A-12. Preoperational water quality sampling stations in the White Mesa project vicinity.

Source: Dames & Moore (1978b) Plate 2.6-10.

Surface water quality in the vicinity of the mill is generally poor. Waters in Westwater Creek (S1R and S9) were characterized by high total dissolved solids (TDS; mean of 674 mg/liter) and sulfate levels (mean 117 mg of SO_4 per liter). The waters were typically hard (total hardness measured as $CaCO_3$; mean 223 mg/liter) and had an average pH of 8.25. Estimated flow rates for Westwater Creek averaged 0.3 fps (0.08 m/sec) at the time of sampling.

Samples from Cottonwood Creek (S8R) were similar in quality to Westwater Creek water samples, although the TDS and sulfate levels were lower (TDS averaged 264 mg/liter; SO_4 averaged 40 mg/liter during heavy spring flow conditions [80 fps (24 m/sec) streamflow].

The concentrations of TDS increased downstream in Corral Creek, averaging 3180 mg/liter at S3R and 6660 mg/liter (one sample) at S4R. Total hardness averaged in excess of 2000 mg/liter, and pH values were slightly alkaline. Estimated flows in Corral Creek were typically less than 0.1 fps (0.03 m/sec) during sampling.

The spring sample collected at the surface pond south of the project site (S5R) indicated a TDS concentration of less than 300 mg/liter. The water was slightly alkaline with moderate dissolved sulfate levels averaging 42 mg/liter.

During heavy runoff, the concentration of total suspended solids in these streams increased sharply to values in excess of 1500 mg/liter (U.S. NRC 1979 Table 2.22)

High concentrations of certain trace elements were measured in some sampling areas. Levels of mercury (total) were reported as high as 0.002 mg/liter (S3R, 7/25/77; S8R, 7/25/77). This level is 40 times the EPA recommended limit for the protection of freshwater aquatic life (0.05 ug/liter). Total iron measured in the pond (S5R, 11/10/77) was 9.4 mg/liter, over nine times the EPA recommended limit of 1 mg/liter for the protection of aquatic life. These values appear to reflect groundwater quality in the vicinity and are probably due to evaporative concentration and not due to human perturbation of the environment.

3.0 GROUNDWATER

3.1 Hydrogeology (Section 2.6.2)

A generalized section of the stratigraphic and water-bearing units in southeastern Utah is shown in Figure A-13. Recharge of these aquifers occurs from seasonally variable rainfall infiltrating along the flanks of the Abajo, Henry, and La Sal Mountains and along the flanks of folds. Recharge water also originates from precipitation on the flat lying beds where it percolates into the groundwater region along joints.

In the White Mesa area, 39 groundwater appropriations (applications for water wells) were on file in 1978 with the Utah State Engineers Office for wells lying within a 5-mile (8-km) radius of the project site. All but one of these wells produce from the Dakota and Morrison Formations. Thirty-five of these are for wells which are actually constructed (Dames & Moore 1978b, Table 2.6-1). Most of these wells produce less than 10 gpm (55 m^3/day) and are used for domestic, irrigation, and stockwatering purposes. The remaining

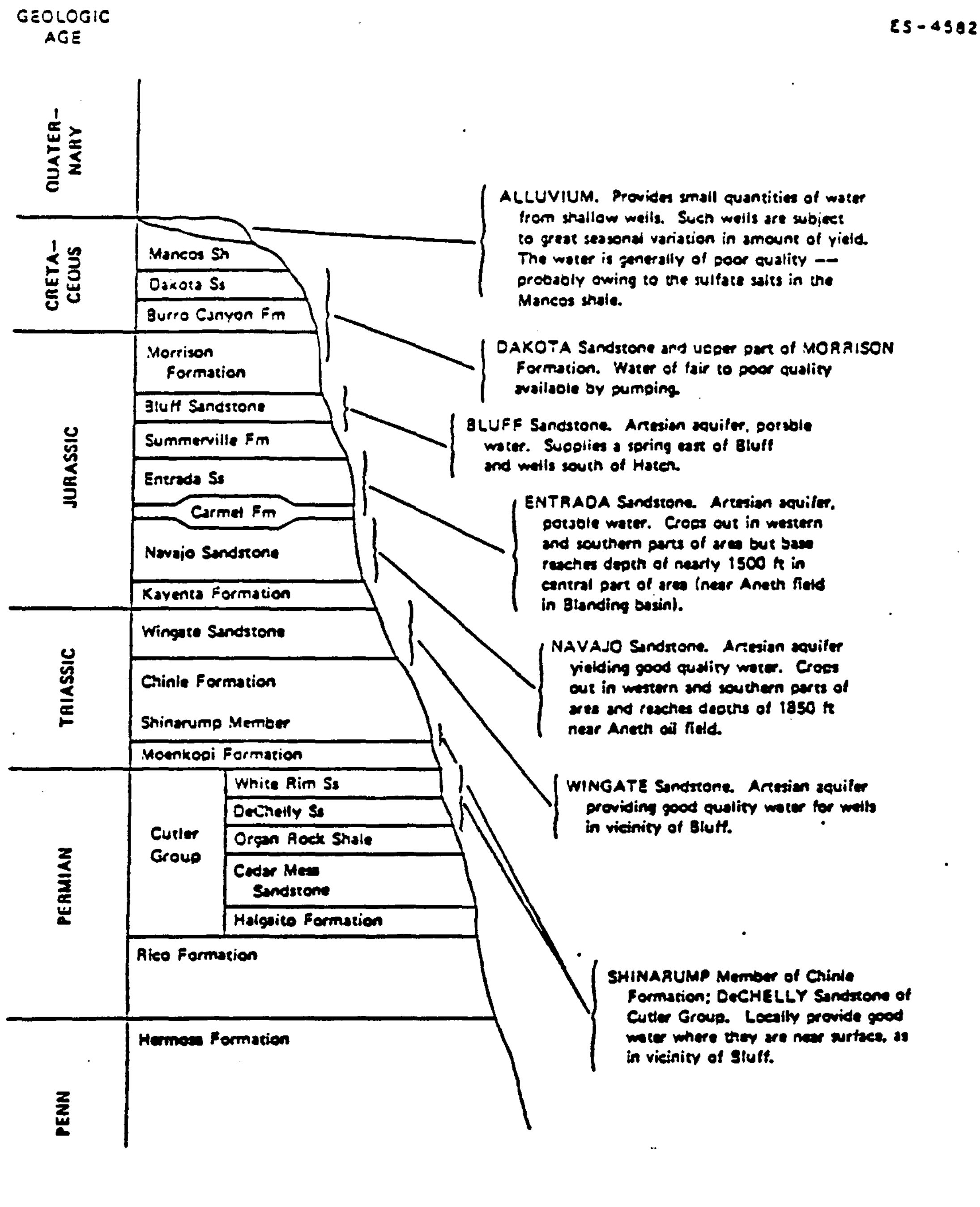


Figure A-13. Generalized stratigraphic section showing freshwater-bearing units in southeastern Utah.

Source: Dames & Moore (1978b) Plate 2.6-1.

well, which was drilled to a depth of 1800 feet (548 m) by Energy Fuels Nuclear, withdraws water from the Navajo Sandstone. The majority (31) are hydrologically upgradient or cross gradient with respect to the project site. The remaining four wells (three on-site and one offsite, south) are on the project site. Two of the on-site wells were located in the area of the proposed tailings impoundment and were completely plugged with bentonite and/or another suitable clay.

As is the case throughout most of the Four Corners region, the Blanding area depends largely on groundwater for its water supply. A porous soil, underlain by the Dakota Sandstone on top of a regional aquiclude (the Brushy Basin Member of the Morrison Formation), provides the Blanding area with a near surface source of groundwater. This situation is somewhat uncommon in the highly dissected south-central portion of the Colorado Plateau.

The Dakota Sandstone on White Mesa has been completely isolated by erosion; consequently, all recharge to this formation comes from precipitation and irrigation on the mesa. No irrigation occurs close to the mill site, and normal precipitation is only 12 inches (30 cm) per year, most of which reenters the atmosphere as evapotranspiration (i.e., it does not penetrate the soils over the Dakota). The Dakota is the underlying bedrock under the tailings impoundment and has a permeability coefficient from 5 to 10 feet (1.5 to 3 m) per year (Dames & Moore 1978b, Section 4.2.4.1 and Appendix H). Jointing occurs in the formation but is probably not fully penetrating. An aquiclude, the Brushy Basin Member of the Morrison Formation, underlies the Dakota Sandstone, which accounts for the groundwater retained in the lower portion of the Dakota.

In the immediate vicinity, only the Dakota Sandstone and the Salt Wash Member (including the Westwater Member) are significant aquifers. The Entrada and Navajo Formations contain larger quantities of water, but their depth prohibits common exploitation for domestic water supplies.

Comb Ridge and the Abajo Mountains are significant areas of recharge of the Salt Wash and deeper aquifers. General gradients of groundwater movement in these aquifers follow the regional structure, and the water discharges ultimately in the vicinity of the San Juan River.

Because the Brushy Basin Member acts as an aquiclude to the Salt Wash Member in the uplands, the primary recharge areas for this aquifer are Brushy Basin Wash to the northwest of Blanding, Cottonwood Creek to the west and southwest of the town, and the upper reaches of Montezuma Creek, especially along Dodge and Long canyons.

Several permeability tests were conducted at the mill and tailings retention sites. The results of these tests show a hydraulic conductivity of 5 to 10 feet (1.5 to 3 m) per year. The shallow groundwater movement at the mill site is estimated to be about 0.01 to 0.02 foot (0.3 to 0.6 cm) per year toward the south-southwest and the shallow groundwater movement at the tailings site is about 0.0025 to 0.01 foot (0.08 to 0.3 cm) per year in the same direction.

3.2 Groundwater Quality (Section 2.6.2)

Table A-7 is a tabulation of groundwater quality of the Navajo Sandstone aquifer. The TDS range from 244 to 1110 mg/liter in three samples taken over a period from January 27, 1977, to May 4, 1977. High iron (0.57 mg/liter) concentrations are found in the Navajo Sandstone. The U. S. Environmental Protection Agency recommends 0.3 mg of dissolved iron per liter for drinking water. Feltis (1966) noted that the total dissolved solids in the alluvium and at shallow depths in the Dakota Sandstone, the Burro Canyon Formation, and the Morrison Formation range from 300 to 2000 mg/liter.

Groundwater from local springs and wells at locations shown in Figure A-12 has been sampled. Total dissolved solids ranged from about 700 to 3300 mg/liter. Standards for public drinking water were frquently exceeded for sulfate, selenium, iron, and arsenic. The waters are suitable for stock and wildlife use.

3.3 Current Groundwater Characteristics

3.3.1 Groundwater Movement

The movement of groundwater occurring at shallow depths in the Dakota sandstone and Burro Canyon Formation at the mill site is believed to be confined to isolated zones within White Mesa. Water quality data and phreatic elevations tend to support this hypothesis.

Beneath the shallow isolated water-bearing zone is the Brushy Basin Member of the Morrison formation which is an aquiclude. There are also locally impermeable lenses in the base of the Burro Canyon Formation.

Based on a gradient of one foot per 100 feet, the shallow groundwater movement at the tailings site was calculated to be approximately 0.0025 to 0.01 foot (0.08 to 0.3 cm) per year toward the south-southwest (Dames & Moore 1978b, Section 2.6.1.3).

Groundwater levels have been shown to follow a line drawn between Bore Hole 3 and Monitor Well 3 which trends from the northeast corner of the mill site to the southwest corner. Figure A-14 shows the phreatic levels measured by Dames & Moore (1978b) in bore holes (BH) 3, 9, 12, 19, and 28 in late 1977 and the phreatic elevations averaged over three years (1985-1987) in monitor wells (MW) 1, 2, 3, 4, 5, 11, 12, and 13. The line drawn between BH 3 and MW 3 is west 146 degrees from north. The indicated gradient under the tailings cells along this line is 1.6 feet per 100 feet.

3.3.2 Current Groundwater Quality at the Mill Site

Pre-existing groundwater quality at the mill site combined with the nature of the tailings solutions can be used to assess the effects, if any, from mill operations. The chemical composition of the tailings solutions were estimated by Dames & Moore (1978b) based on laboratory test work. The U. S. Nuclear Regulatory Commission (NRC) had samples of solutions from Cells 1-I and 3 analyzed, with a sample date of July 14, 1987 (U. S. NRC 1987b). Table A-8 shows both sets of data.

Table A-7

Water quality of groundwater in the project vicinity. Zero values (0.0) are below detection limits.

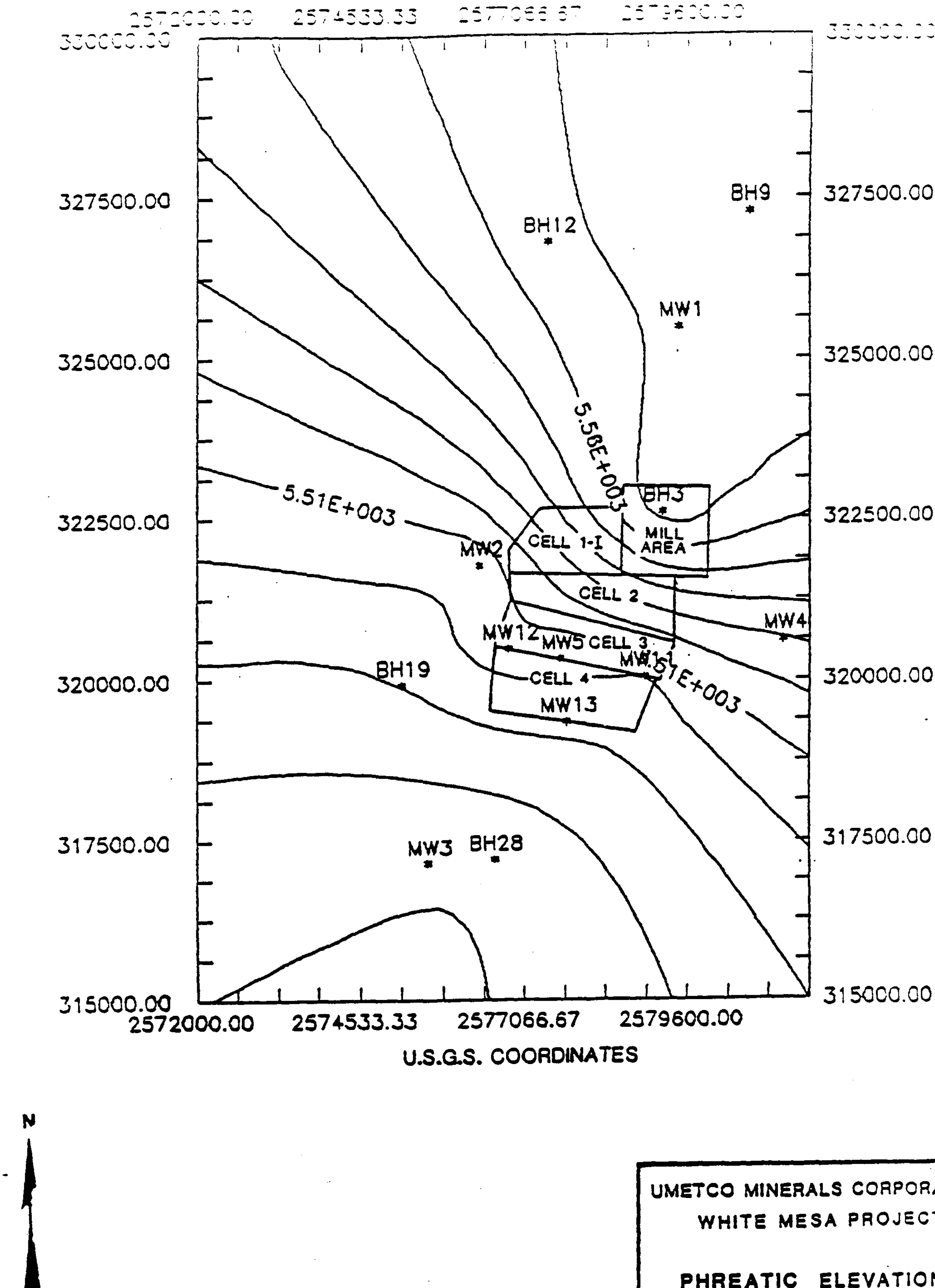
Parameter	Blanding mill site well in Navajo Sandstone, G2R				
	1/27/77[a]	5/4/77[c]	7/25/77	12/05/77	3/23/78
Field specific conductivity, µmhos/cm			400		310
Field pH			6.9		7.6
Dissolved oxygen					—
Temperature °C			22.2		11
Estimated flow, m^3 day (gpm)			109 (20)		—
Determination, mg/liter					
pH	8.0	7.9	7.7	7.9	8.16
TDS (at 180°C)	244	246	1110	446	216
Redox potential			220	220	211
Alkalinity (as $CaCO_3$)	189	180	224	185	187
Hardness, total (as $CaCO_3$)	196		208	196	177
Carbonate (as CO_3)	0.0		0 . 0	0.0	0.0
Aluminum, dissolved			<0.01	<0.1	<0.1
Ammonia (as N)	0.0		<0.1	<0.1	0.16
Arsenic, total	0.014		<0.01	<0.01	0.007
Barium, total	<0.0		0.13	<0.1	0.15
Boron, total	0.040		<0.1	0.11	<0.1
Cadmium, total	0.0		0.004	<0.02	<0.006
Calcium, dissolved	51	49	51	57	112
Chloride	0.0	50	<1	2	4
Sodium, dissolved	8.0		5.3	23	13
Silver, dissolved	0.0		<0.002	0.010	0.006
Sulfate, dissolved (as SO_4)	24	17	17	83	26.7
Vanadium, dissolved			<0.002	0.16	0.005
Manganese, dissolved	0.020		0.03	0.03	0.03
Chromium, total	0.0		0.02	<0.05	0.02
Copper, total	0.0		0.005	<0.010	0.005
Fluoride, dissolved	0.17	0.1	0.22	0.2	0.2
Iron, total	0.54		0.61	0.35	2.1
Iron, dissolved			0.57	0.30	2.3
Lead, total	0.0		0.02	<0.05	<0.06
Magnesium, dissolved	17	19	18	15	21
Mercury, total	0.0	0 . 0	0.002	<0.00002	0.00002
Molybdenum, dissolved			<0.01	0.010	0.004
Nitrate (as N)	0.05	0.12	<0.05	<0.06	<0.05
Phosphorus, total (as P)	0.03		<0.01	<0.02	0.03
Potassium, dissolved	3.0		3.2	2.8	2.4
Selenium, dissolved	0.0		0.05	0.014	<0.005
Silica, dissolved (as SiO_2)	12	5.8	12	6	8
Strontium, dissolved			0.67	0.5	0.60
Uranium, total (as U)			<0.002	0.16	<0.002
Uranium, dissolved (as U)			<0.002	0.031	<0.002
Zinc, dissolved	0.0		0.39	0.007	0.12
Total organic carbon				1.1	16
Chemical oxygen demand				<1	66
Oil and grease				1.0	1
Total suspended solids				6	1940
Determination (pCi/liter)					
Gross alpha ± precision		7	10.2 ± 2.8	1.6 ± 1.3	1.9 ± 1.4
Gross beta ± precision		<20	73 ± 19	8 ± 8	9 ± 8
Ra-226 ± precision			0.1 ± 0.3	0.6 ± 0.4	0.3 ± 0.3
Th-230 ± precision			0.7 ± 2.7	0.3 ± 0.6	0.1 ± 0.4
Pb-210 ± precision			1.0 ± 2.0	0.7 ± 2.1	0.0 ± 4.0
Po-210 ± precision			0.0 ± 0.3	0.0 ± 0.8	0.0 ± 0.6

[a] The spring in Corral Creek, Station No. G1R, was tested on July 25, 1977, and again on November 10, 1977. Because of the low flow, the spring could not be located.

[b] Utah State Division of Health Analysis, Lab No. 77081.

[c] Partial analysis by Hazen Research, Inc., Sample No. HRI-11503.

Source: Adapted from Dames & Moore (1978b), Table 2.6-6, and "Supplemental Report, Baseline Water Quality Environmental Report, White Mesa Uranium Project," June 29, 1978.



UMETCO MINERALS CORPORATION

WHITE MESA PROJECT

PHREATIC ELEVATIONS

JUNE, 1988 FIGURE A-14

TABLE A-8

CELLS LIQUIDS COMPOSITION

Ion	Tailings Liquor (Dames & Moore) Grams/Liter	Cell 1-I (NRC) Grams/Liter	Cell 3 (NRC) Grams/Liter
V	0.24	0.27	0.21
U	0.0025	0.105	0.115
Na	4.90	9.7	5.9
NH_3	0.065	7.8	13.9
C_1	3.05	8.0	5.1
SO_4	82.2	190.0	180.0
Cu	1.62	0.74	0.40
Ca	0.48	0.63	0.64
Mg	4.06	0.79	5.40
Al	4.26	2.30	2.30
Mn	4.58	0.14	0.08
Zn	0.09	1.20	0.59
Mo	0.007	0.24	0.05
pH	1.8-2.0	0.70	0.82
As		0.44	0.22
Se		ND	ND

A generalized stratigraphic sequence for wells is as follows:

Formation	Depth, Feet
Soil	0 to 20
Dakota Formation	70
Burro Canyon Formation	85
Brushy Basin Member	630
Salt Wash Member	730
Bluff Formation	1,010
Summerville Formation	1,180
Entrada Formation	1,280
Carmel Formation	1,440
Navajo Sandstone	1,870
Kayenta Formation	1,955
Wingate Formation	2,213

The five Supply Wells draw water from the Navajo Formation at rates from 50 to 250 gallons per minute. There are two types of Supply Wells, production and culinary. The production wells are not involved in groundwater monitoring. There is only one culinary well operating at any time, and it is monitored at the same frequency as the shallow monitor wells with the analysis of the same suite of constituents. Supply Well 1 has dried up and Supply Well 2 is now the culinary well.

The monitor wells are listed in Table A-9 and the locations are shown on Drawing No. C-1 in the Appendix B. The groundwater indicator parameters are given in Table A-10 (U.S. NRC 1988).

TABLE A-9

Monitor Wells

Well	Casing Elevation	Ground Elevation	Water Elevation	Well Depth
MW1	5,648.2	5,645.7	5,571.9	188.8
MW2	5,613.5	5,611.4	5,503.1	122.9
MW3	5,555.3	5,553.0	5,471.6	93.7
MW4	5,622.6	5,620.9	5,529.0	123.3
MW5	5,609.3	5,608.8	5,502.1	138.0
MW11	5,611.1	5,608.3	5,499.8	133.7
MW12	5,609.5	5,608.1	5,508.3	129.6
MW13	5,570.4	5,568.7	5,492.5	114.8
Culinary Well	5,628.8	5,628.0	4,978.0	1,860.0
Alternate (Well 2)	5,652.5	5,650.4	4,997.4	1,885.0

Table A-10

GROUNDWATER INDICATOR PARAMETERS*

	MW-1	MW-2/MW-3 Compliance Wells
Lower Limit of Detection		
Arsenic, mg/l	0.005	0.005
Selenium, mg/l	0.001	0.001
Chloride, mg/l	1.0	1.0
Background		
Arsenic, mg/l	0.001	
Selenium, mg/l	0.001	
Chloride, mg/l	15	
Threshold Values		
Arsenic, mg/l		0.0125
Selenium, mg/l		0.0025
Chloride, mg/l		Constant Delta**

*U.S. NRC (1988)
**Umetco (1987)

Sampling the 9 monitor wells (including the culinary well) indicate the following. In general, Monitor Well 1 has the lowest concentrations and Monitor Well 3 has the highest concentrations. Data from both these wells were used to establish background concentrations. The "constant delta" method was developed for chlorides (Umetco 1987, U.S. NRC 1988).

No trends are apparent in the data, although arsenic and selenium values are elevated in every monitor well. The selenium concentration in the cell liquids is negligible (see Table A-8). Thus the high arsenic and selenium concentrations in the groundwater must be from natural sources. Chlorides, sulfates, and pH should trend together in response to tailings solutions. These parameters are not trending in any of the monitor wells.

4.0 ARCHEOLOGICAL SITES

4.1 Current Status of Excavations

Archeological investigations for the entire mill site and for Cells 1-I through Cell 4 were completed with the issuance of four separate reports covering 30 sites (excluding re-investigations). (Lindsay 1978, Nielson 1979, Casjens et al 1980, and Agenbroad et al 1981).

The sites reported as excavated are as follows:

6380	6394	6437
6381	6395	6684
6384	6396	6685
6385	6397	6686
6386	6403	6697
6387	6404	6698
6388	6420	6699
6391	6429	6754
6392	6435	6757
6393	6436	7754

The sites remaining to be excavated are:

6379	6441	7658	7690
6382	6443	7659	7691
6405	6444	7660	7693
6408	6445	7661	7696
6421	6739	7665	7700
6427	6740	7668	7752
6430	7653	7675	7876
6431	7655	7684	8014
6432	7656	7687	
6439	7657	7689	

The approximate locations of these sites are shown on Drawing No. B-1 in Appendix A.

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MAPS

CELL 4 DESIGN

TAILINGS MANAGEMENT SYSTEM

WHITE MESA PROJECT
BLANDING, UTAH

APPENDIX B

Umetco Minerals Corporation

AUGUST, 1988

APPENDIX B

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SECTION 1

FIGURE C-1

Boring and Test Pit Locations

MAPS

chen and associates, inc.
CONSULTING ENGINEERS

SOIL & FOUNDATION ENGINEERING
96 S. ZUNI • DENVER, COLORADO 80223 • 303/744-7105
1924 EAST FIRST STREET • CASPER, WYOMING 82601 • 307/234-2126

SECTION 2

Extracted Data From

SOIL PROPERTY STUDY
EARTH LINED TAILINGS RETENTION CELLS
WHITE MESA URANIUM PROJECT
BLANDING, UTAH

Prepared for:

ENERGY FUELS NUCLEAR, INC.

PARK CENTRAL
1515 ARAPAHOE STREET
DENVER, COLORADO 80202

Job No. 16,406

July 18, 1978

TABLE OF CONTENTS

Table No.

Figure No.

Note: The attached logs have been redrawn

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Test Hole	Depth (Ft.)	NATURAL		Maximum Dry Density	Optimum Moisture Content	ATTERBERG LIMITS		GRADATION ANALYSIS			REMOLDED		PERMEABILITY		Specific Gravity	Soil Type
		Moisture Content (%)	Dry Density (pcf)	(pcf)	(%)	Liquid Limit (%)	Plasticity Index (%)	Maximum Size	Passing #200 (%)	Less than 2 μ (%)	Dry Density (pcf)	Moisture Content (%)	ft./yr.	cm./sec.		
2	0-5			117.5	10.8	20	3	#16	58	19	111.6	16.4	0.57	5.5×10^{-7}		Sandy Silt
3	7-8	7.2				21	6	#16	62							Sandy Clayey Silt
5	7½-10			104.1	18.5	33	8	3/4 In.	56	12	102.1	22.0	0.085	8.2×10^{-8}	2.65	Calcareous Silty Clay
6	1-2	10.3				25	7	#16	77							Sandy Clayey Silt
6	8½-9	6.1				27	8	#4	70							Sandy Clay
8	5-5½	13.1					NP	3/4 In.	62							Calcareous Sandy Silt
9	0-1	8.1					NP	#16	53							Sand - Silt
10	4-6½					24	10	#4	73							Sandy Clay
11	5½-6½	14.0				26	6	#16	65							Siltstone Claystone
12	2-5			101.0	20.6	53	35	#16	88	59	95.0	18.3	0.068	6.6×10^{-8}	2.67	Weathered Claystone
13	7-8	13.1				39	13	#8	84							Calcareous Silt Clay
14	1-2	19.3				40	21	#4	89							Weathered Claystone
15	1½-4½			106.8	19.0	26	8	3/8 In.	65	27	103.4	18.0	0.012	1.2×10^{-8}	2.64	Red Calcareous
17	2-3	11.4				19	4	#8	59							Sandy Clay
19	0-3			117.5	12.8	23	6	#16	70		109.9	12.4	0.035	3.4×10^{-8}		Sandy Silt
22	1-2	13.2				26	10	#4	73							Sandy Clayey Silt
23	1-3					48	24	#30	87							Sandy Clay
23	6-3					61	30	#30	96							Weathered Claystone
25	1-3½	13.3				26	9	#4	57							Claystone
26	4½-5	15.3				41	20	#4	91							Sandy Clay
28	0-2	12.7				28	10	3/8 In.	72							Weathered Claystone
29	2-3	8.5				19	2	#16	59							Sandy Clay
32	8-8½	5.6				23	6	#30	73							Sandy Silt
37	0-4			118.8	11.5	23	5	#8	72		110.5	11.5	0.63	6.1×10^{-7}		Sandy Clayey Silt
38	5-7			111.0	16.7	29	14	3/8 In.	69		102.4	17.9	0.041	4.0×10^{-8}		Sandy Clayey Silt
40	4-5½			110.0	16.2	26	9	#8	64	27	106.4	16.4	0.017	1.6×10^{-8}	2.65	Sandy Clay Sandy Clay

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Test Hole	Depth (Ft.)	NATURAL Moisture Content (%)	NATURAL Dry Density (pcf)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	ATTERBERG LIMITS Liquid Limit (%)	ATTERBERG LIMITS Plasticity Index (%)	GRADATION ANALYSIS Maximum Size	GRADATION ANALYSIS Passing #200 (%)	GRADATION ANALYSIS Less than 2 μ (%)	REMOLDED Dry Density (pcf)	REMOLDED Moisture Content (%)	PERMEABILITY ft./yr.	PERMEABILITY cm./sec.	Specific Gravity	Soil Type
40	9-9½	6.8				22	8	3/8 In.	60							Sandy Clay
42	13½-14½	7.6				26	10	3/8 In.	73							Sandy Clay
43	11-12	12.1				41	22	#4	86							Claystone
43	13½-16½			110.0	16.9	40	24	3/8 In.	85	44	104.1	15.8	0.024	2.3×10^{-8}	2.62	Claystone
44	6½-7	7.5				30	11	3/8 In.	79							Calcareous Sandy Clay
46	0-2	12.3				22	6	#16	76							Sandy Clayey, Silt
✓48	5-5½					30	9	3/8 In.	65							Sandy Clay
✓49	5-7			110.7	15.6	25	9	#16	71		105.2	13.9	0.33	3.2×10^{-8}		Sandy Clay
✓49	14-15					28	5	#8	55							Calcareous Sandy Silt
54	0-2	12.1				23	9	#8	64							Sandy Clay
55	5-5½	7.8				28	14	#30	71							Sandy Clay
55	9½-10½					28	13	#4	71							Sandy Clay
✓58	5½-6	12.5				35	11	#4	75							Sandy, Silty, Clay
61	0-1	11.5				21	4	#16	75							Sandy Silt
62	11-11½	8.1					NP	1 In.	34							Calcareous Sand & Silt
63	4-6					30	14	#8	68							Sandy Clay
65	1-2	9.0					NP	#16	44							Silty Sand
68	7½-8	8.6				28	13	#8	67							Sandy Clay
70	3½-4½	16.4				27	4	1½ In.	46							Calcareous Sand & Silt
72	0-2	12.2				22	8	#16	59							Sandy Clay
75	10-11	12.4				41	25	#4	75							Weathered Claystone
75	12-14					45	22	#16	93							Claystone

TABLE II

LABORATORY PERMEABILITY TEST RESULTS

Sample	Soil Type	Compaction: Dry Density (pcf)	Compaction: Moisture Content (%)	Compaction: % of ASTM D698	Surcharge Pressure (psf)	Permeability (Ft/Yr)	Permeability (Cm/Sec)
TH 2 @ 0'-5'	Sandy Silt	111.6	16.4	95	500	0.57	5.5×10^{-7}
TH 5 @ 7½'-10'	Calcareous Silty Clay	102.1	22.0	101	500	0.085	8.2×10^{-8}
TH 12 @ 2'-5'	Weathered Claystone	95.0	18.3	94	500	0.068	6.6×10^{-8}
TH 15 @ 1½'-4½'	Calcareous Sandy Clay	103.4	18.0	97	500	0.012	1.2×10^{-8}
TH 19 @ 0'-3'	Sandy, Clayey Silt	109.9	12.4	94	500	0.035	3.4×10^{-8}
TH 37 @ 0'-4'	Sandy, Clayey Silt	110.5	11.5	93	500	0.63	6.1×10^{-7}
TH 38 @ 5'-7'	Sandy Clay	102.4	17.9	92	500	0.041	4.0×10^{-8}
TH 40 @ 4'-5½'	Sandy Clay	106.4	16.4	97	500	0.017	1.6×10^{-8}
TH 43 @ 13½-16½'	Claystone	104.1	15.8	95	500	0.024	2.3×10^{-8}
TH 49 @ 5'-7'	Sandy Clay	105.2	13.9	95	500	0.33	3.2×10^{-7}

TABLE III

RESULTS OF ATTERBERG LIMITS

SAMPLE	SOIL TYPE	PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	SHRINKAGE RATIO
2 @ 0 - 5'	Sandy Silt	58	20	17	17.	1.81
5 @ 7½ - 10'	Calcareous Silty Clay	56	33	25	25	1.62
15 @ 1½-4½'	Calcareous Sandy Clay	65	26	18	17.5	1.76
19 @ 0-3'	Sandy, Clayey Silt	70	23	17	18	1.80
26 @ 4½-5'	Weathered Claystone	91	41	21	12	1.90
38 @ 5 - 7'	Sandy Clay	69	29	15	14	1.89

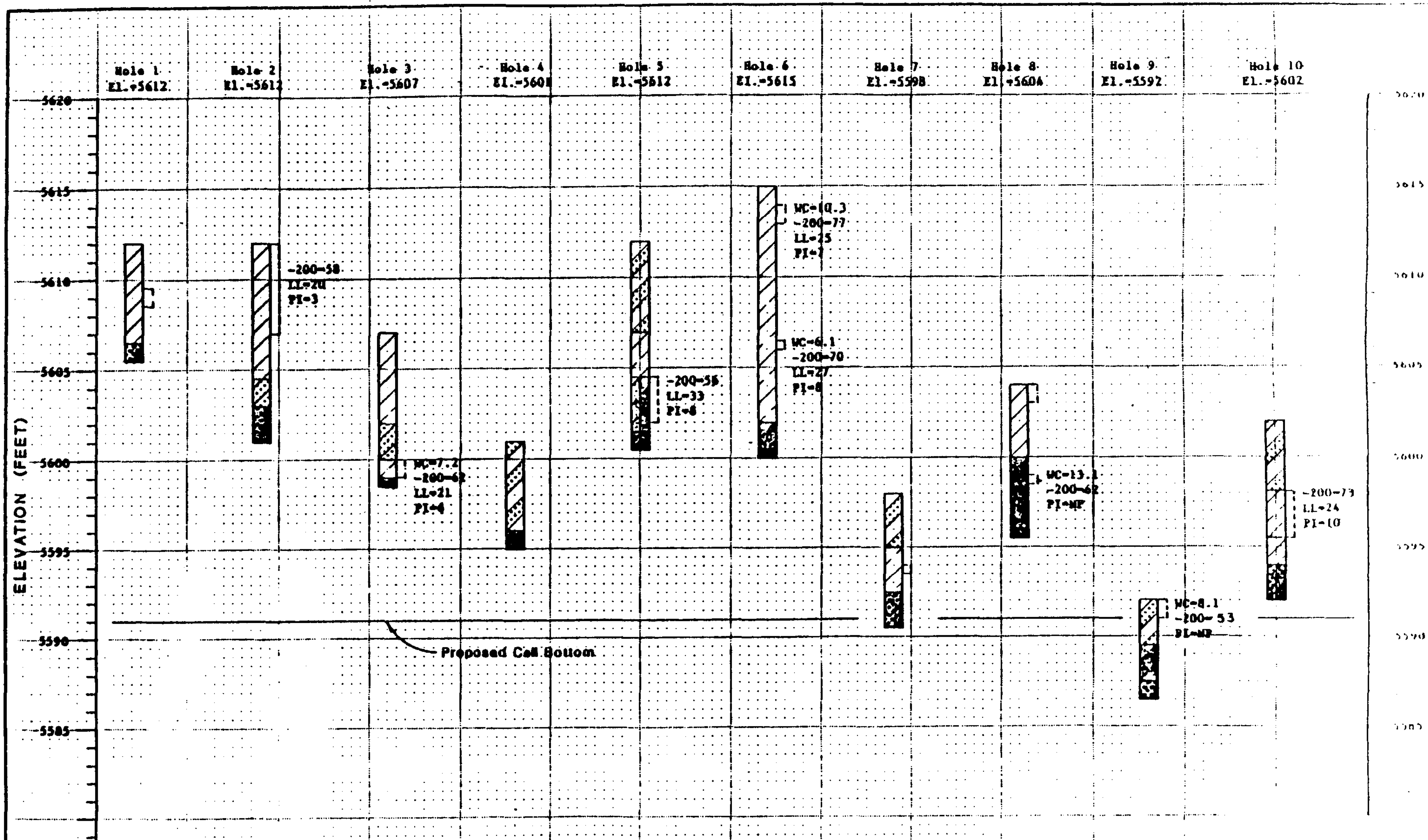


FIGURE 2

LOGS OF EXPLORATORY BORINGS

(CHEN & ASSOCIATES)

WHITE MESA PROJECT

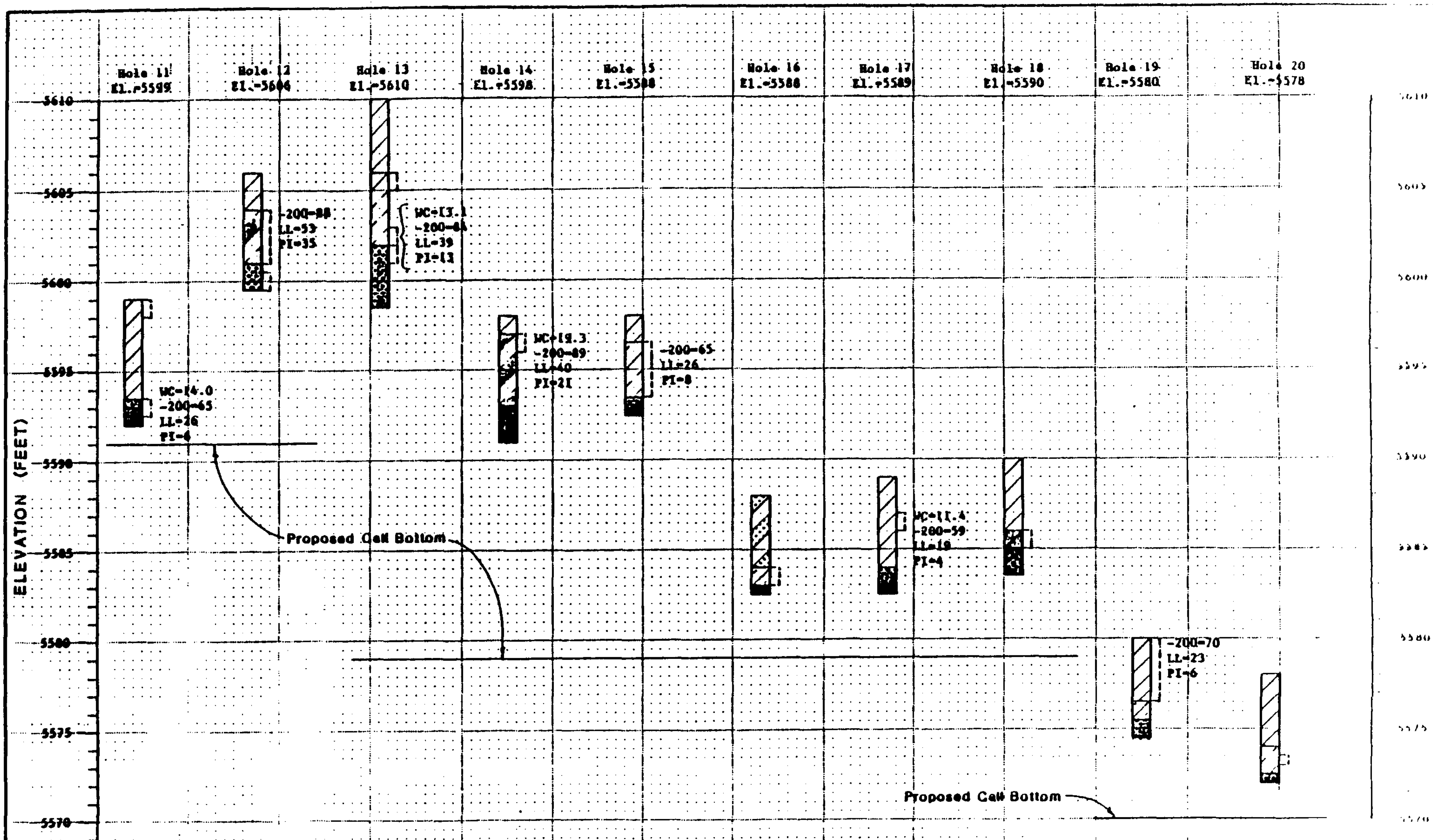


FIGURE 3
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

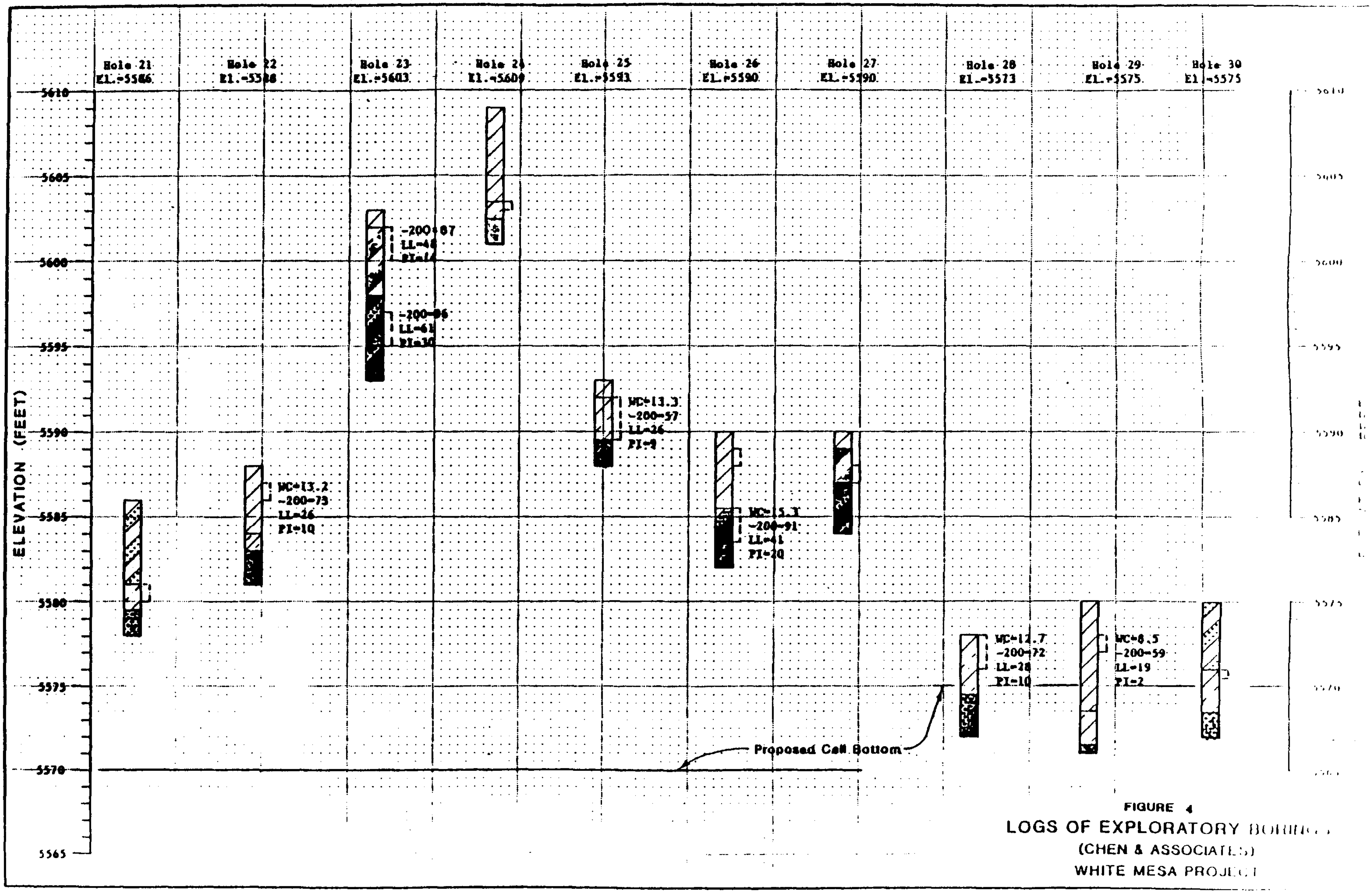


FIGURE 4
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

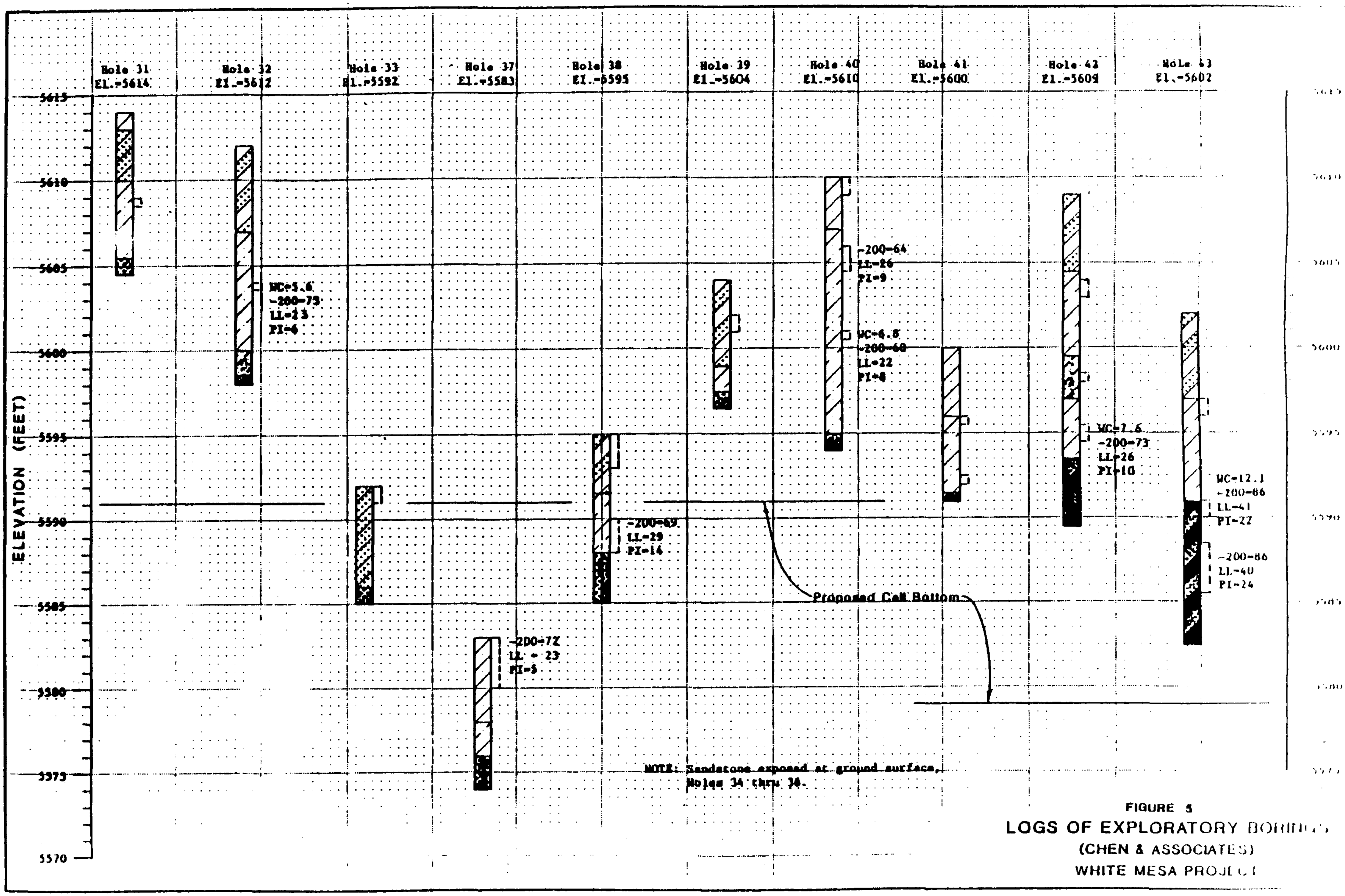


FIGURE 5
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

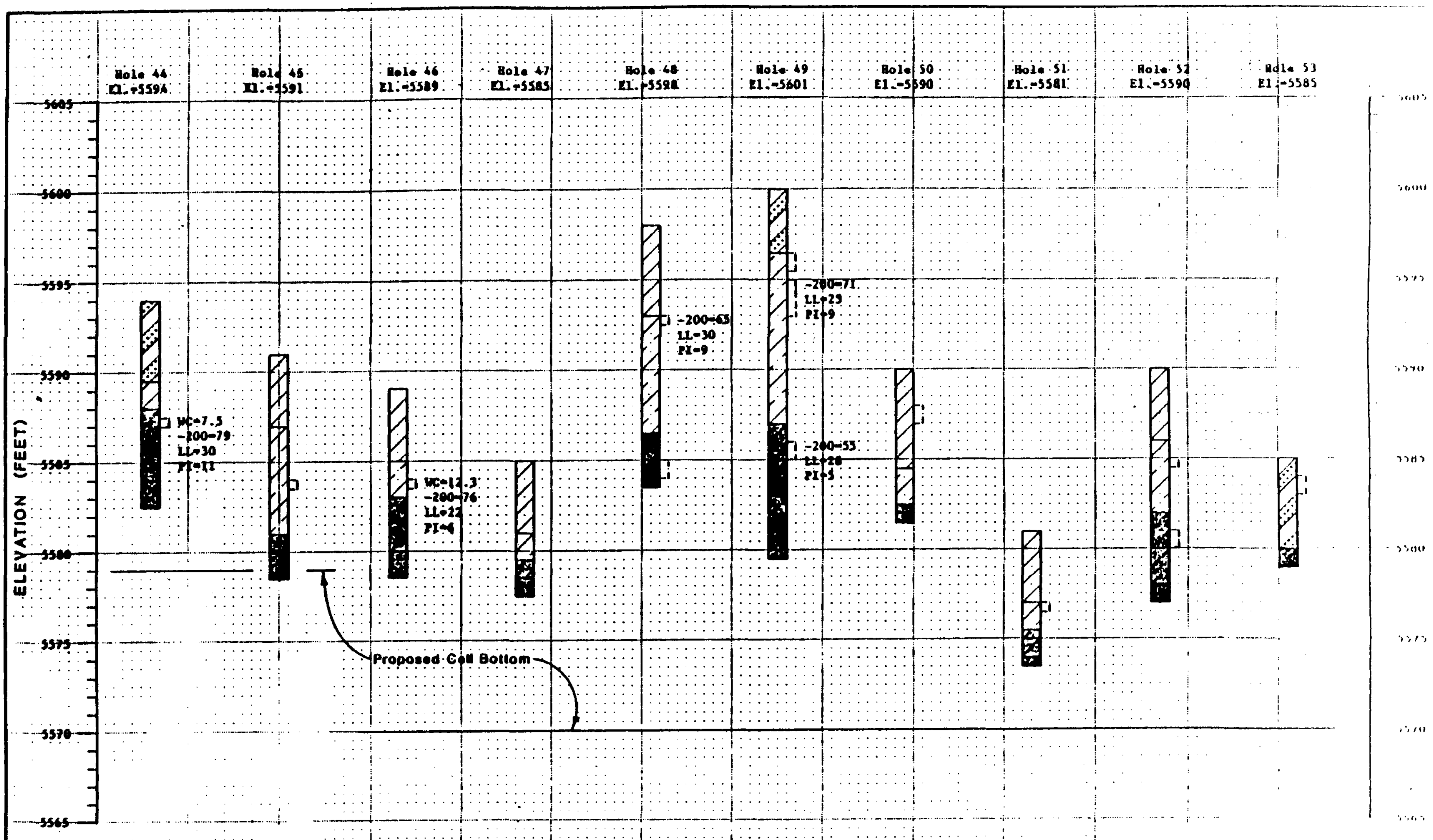


FIGURE 6
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

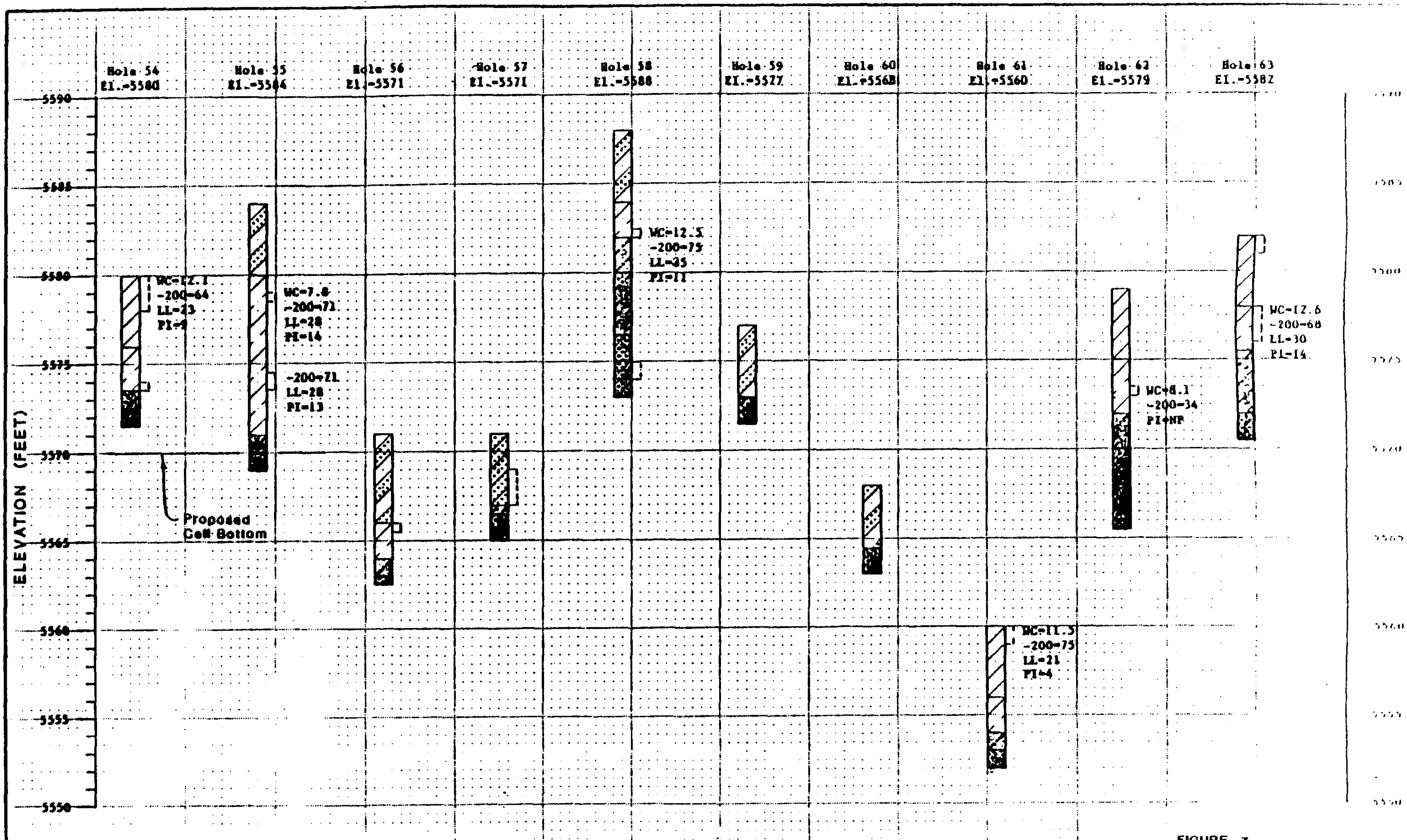


FIGURE 7

LOGS OF EXPLORATORY BORINGS

(CHEN & ASSOCIATES)

WHITE MESA PROJECT

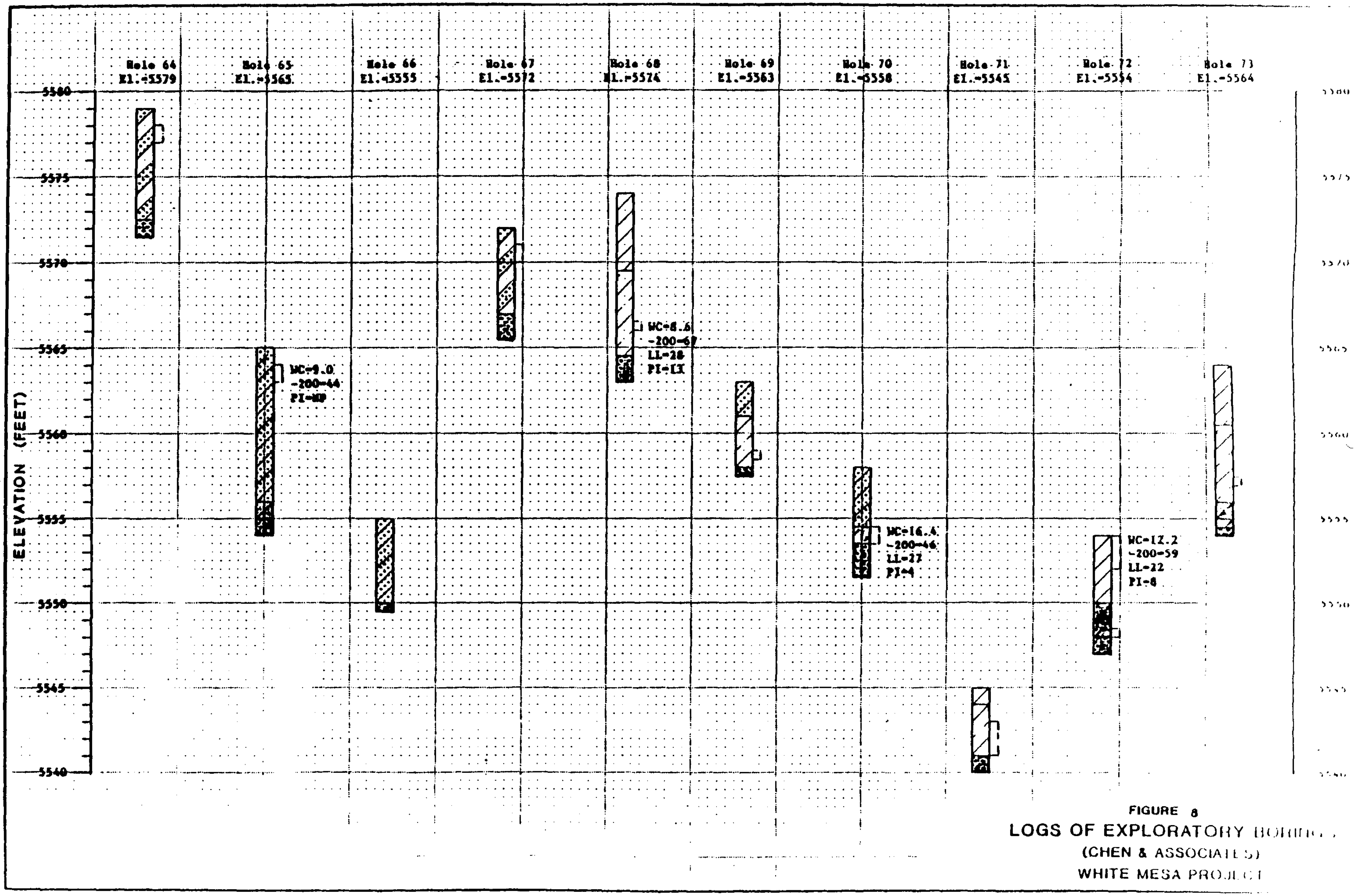
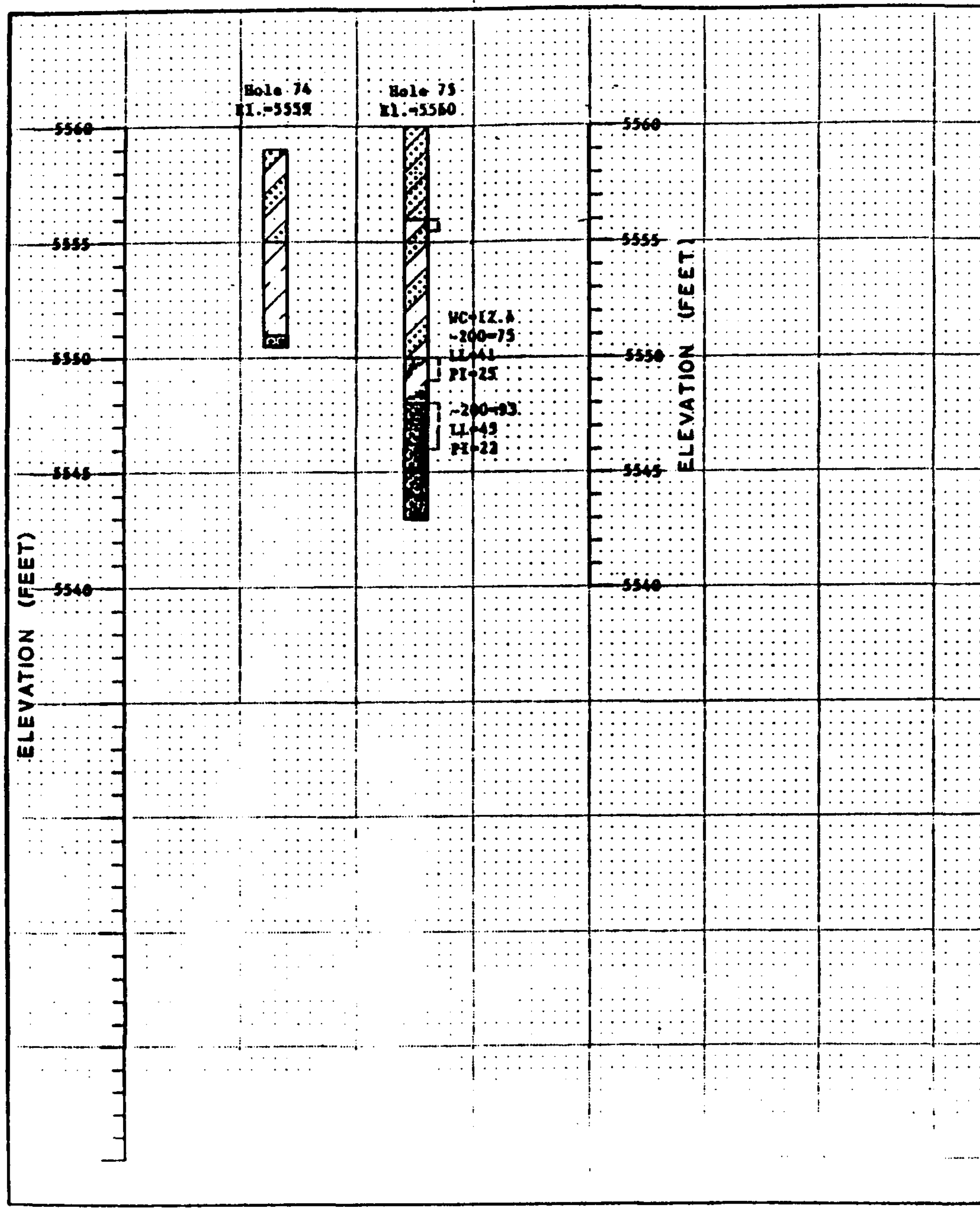


FIGURE 8
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT



Legend:

- Sand (SM), silty, fine to medium grained, approximately 40-50% silt, slightly moist to moist, reddish brown.
- Sand, silty to sandy silt (SM-ML), fine to medium grained, approximately 50-60% silt, slightly moist to moist, reddish brown.
- Silt (ML), sandy, approximately 60-70% silt, fine to medium sand size, slightly calcareous with depth, slightly moist to moist, reddish brown to light brown.
- Clay, silty to sandy silt (CL-ML), approximately 60-75% low to non-plastic fines, fine to medium sand size, slightly to moderately calcareous with depth, slightly moist, light brown.
- Clay (CL), highly calcareous, sandy to silty, approximately 50-75% low plasticity fines, scattered very hard lenses/layer, dry to slightly moist, light tan to white.
- Weathered claystone (CL-CH), approximately 90% medium to high plasticity fines, moist, gray-brown.
- Claystone, bedrock, slightly moist, greenish gray.
- Claystone-sandstone bedrock, lightly cemented, roughly stratified, fine to medium grained, greenish gray.
- Sandstone bedrock, well cemented with depth, fine to medium grained, tan to gray.
- Disturbed auger sample.

NOTES:

(1) Test holes were drilling on May 17 and 18, 1978 with a 12-inch, single-flight, power auger.
(2) Elevations are approximate and taken from contours shown on Fig. 1.
(3) No free water was found in test holes at the time of drilling.
(4) WC = Water Content (%);
-200 = Percent Passing No. 200 Sieve;
LL = Liquid Limit (%);
PI = Plasticity Index (%)
NP = Non-Plastic.

FIGURE 9

LOGS OF EXPLORATORY BORINGS

(CHEN & ASSOCIATES)

WHITE MESA PROJECT

chen and associates, inc.
CONSULTING ENGINEERS

SOIL & FOUNDATION ENGINEERING • 96 S. ZUNI • DENVER, COLORADO 80223 • 303/744-7105

SECTION 3

Extracted Data From

SOIL PROPERTY STUDY
PROPOSED TAILINGS RETENTION CELLS
WHITE MESA URANIUM PROJECT
BLANDING, UTAH

Prepared for:

ENERGY FUELS NUCLEAR, INC.
1515 ARAPAHOE STREET
DENVER, COLORADO 80202

Job No. 17,130

January 23, 1979

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Note: The attached logs have been redrawn.

CHEN AND ASSOCIATES

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	TRIAXIAL SHEAR TESTS		PERCENT PASSING NO. 200 SIEVE	SOIL TYPE
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		DEVIATOR STRESS (PSF)	CONFINING PRESSURE (PSF)		
76	0 - 1	4.5		21	5				78	Sandy silt
	9.5 - 10	4.4			NP				26	Silty, gravelly sand
77	7.5 - 8	8.6		30	15				71	Sandy clay
79	0 - 1	4.1		20	5				83	Sandy silt
	5 - 5.5	5.5			NP				41	Calcareous sandy clay
80	4.5 - 7			39	20				78	Calcareous sandy clay
	8 - 8.5	10.1		40	20				86	Weathered claystone
81	3 - 4	6.3		26	8				64	Silty, sandy clay
83	4 - 6			24	7				64	Sandy, clayey silt
84	0 - 2			18	2				65	Sandy silt
	9 - 9.5	2.7			NP				27	Silty sand
86	8 - 8.5	2.6			NP				12	Sandstone
87	0 - 1	3.1		16	1				61	Sandy silt
89	0 - 3			21	5				66	Sandy silt
90	8 - 8.5	12.9		35	15				61	Weathered claystone
92	0 - 1	5.9		21	5				80	Sandy silt
94	5 - 5.5	13.7		27	10				68	Sandy clay
95	6 - 7			23	5				62	Sandy silt
96	0 - 2	5.2		21	4				79	Sandy silt
	8.5 - 9.5			32	6				66	Calcareous sandy clay
98	0 - 1	3.8		20	5				74	Sandy silt
	4 - 4.5	17.8		49	25				76	Weathered claystone
99	8 - 9.5			40	20				89	Weathered claystone

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TABLE I

SUMMARY OF LABORATORY TEST RESULTS

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	TRIAXIAL SHEAR TESTS DEVIATOR STRESS (PSF)	TRIAXIAL SHEAR TESTS CONFINING PRESSURE (PSF)	PERCENT PASSING NO. 200 SIEVE	SOIL TYPE
99	11 - 12	13.5		26	10				73	Claystone
100	0 - 1			17	NP				44	Silty sand
	5.5 - 6	12.0			NP				61	Sandstone-siltstone
102	6.5 - 7	16.7		30	8				79	Calcareous sandy cla
	13.5 - 14	9.5		23	6				87	Claystone-siltstone
103	10 - 10.5	7.0		28	12				57	Sandy clay
104	8 - 8.5	9.2		33	9				70	Calcareous sandy cl
105	0 - 1	5.4		22	6				77	Sandy silt
	6.5 - 7	4.5			NP				86	Sandy silt
106	5 - 5.5	10.4		28	6				59	Claystone-sandstone
107	7.5 - 9				NP				23	Sandstone
108	0 - 1	4.0		18	3				69	Sandy silt
	9.5 - 10	9.9		38	16				93	Claystone
109	4 - 5			25	7				75	Sandy, clayey silt
111	9 - 9.5	5.8		25	10				53	Claystone
113	5 - 8			40	20				84	Weathered claystone
	10.5 - 11			24	10				54	Claystone-sandstone
114	0 - 2			22	6				58	Sandy, clayey silt
115	4.5 - 6				NP				58	Calcareous
116	0 - 3			22	5				72	Sandy silt
	7 - 8			24	10				42	Claystone-sandstone
117	1 - 2	10.6		25	5				77	Sandy silt
118	0 - 2			25	6				77	Sandy silt

CHEN AND ASSOCIATES

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	TRIAXIAL SHEAR TESTS		PERCENT PASSING NO. 200 SIEVE	SOIL TYPE
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		DEVIATOR STRESS (PSF)	CONFINING PRESSURE (PSF)		
118	6.5 - 8.5			40	20				89	Weathered claystone
119	4.5 - 5	10.0		26	12				68	Sandy clay
120	1 - 2			25	8				69	Sandy, clayey silt
	5 - 5.5	15.5		29	10				78	Sandy clay
	11 - 11.5	11.6		42	24				90	Claystone
122	4 - 6			25	8				66	Sandy, silty clay
	14.5 - 15	6.4		26	8				66	Sandy clay
123	1 - 3			23	7				71	Sandy, clayey silt
124	4.5 - 5	6.0		23	7				69	Sandy, clayey silt
125	0 - 1	3.8		22	6				67	Sandy silt
127	5 - 6			54	34				89	Claystone
128	6 - 8			41	24				90	Claystone

T/ . II

LABORATORY PERMEABILITY TEST RESULTS

Sample	Classification	Compaction: Dry Density (pcf)	Compaction: Moisture Content (%)	Compaction: % of ASTM D698	Surcharge Pressure (psf)	Permeability Ft./Yr.	Permeability Cm/Sec
TH 80 @ 4½-7'	Calcareous sandy clay -200=78; LL=39; PI=20	100.2	19.4	96	500	0.81	7.8×10^{-7}
TH 84 @ 0-2'	Sandy silt -200=65; LL=18; PI=2	113.8	11.7	96	500	4.45	4.3×10^{-6}
TH 96 @ 8½-9½'	Calcareous sandy clay -200=66; LL=32; PI=6	96.9	20.7	97	500	1.55	1.5×10^{-6}
TH 96 @ 8½-9½'	Calcareous sandy clay	95.7	20.3	96	500	26.90*	2.6x10 [illegible]
TH 99 @ 8-9½'	Weathered claystone -200=89; LL=40; PI=20	99.8	18.5	95	500	0.22	2.1x10
TH 100 @ 0-1'	Very silty sand -200=44; PI=NP	117.5	9.7	98	500	0.38	3.7x10
TH 114 @ 0-2'	Sandy, clayey silt -200=58; LL=22; PI=6	112.4	12.9	95	500	0.60	5.8x10
TH 120 @ 1-2'	Sandy, clayey silt -200=69; LL=24; PI=6	108.2	14.7	95	500	0.11	1.1x10
TH 122 @ 4-6'	Sandy, silty clay -200=66; LL=25; PI=8	108.8	15.5	96	500	0.43	4.2x10
TH 123 @ 1-3'	Sandy, clayey silt -200=71; LL=23; PI=7	110.9	12.6	95	500	0.56	5.4x10
TH 128 @ 6-7'	Claystone -200=89; LL=41; PI=24	92.4	23.9	93	500	0.12	1.[illegible]x10
TH 128 @ 6-7'	Claystone -200=89; LL=41; PI=4	93.1	22.1	94	500	0.52*	[illegible]x10

* 1.5 pH sulfuric acid liquor used during percolation test interval.

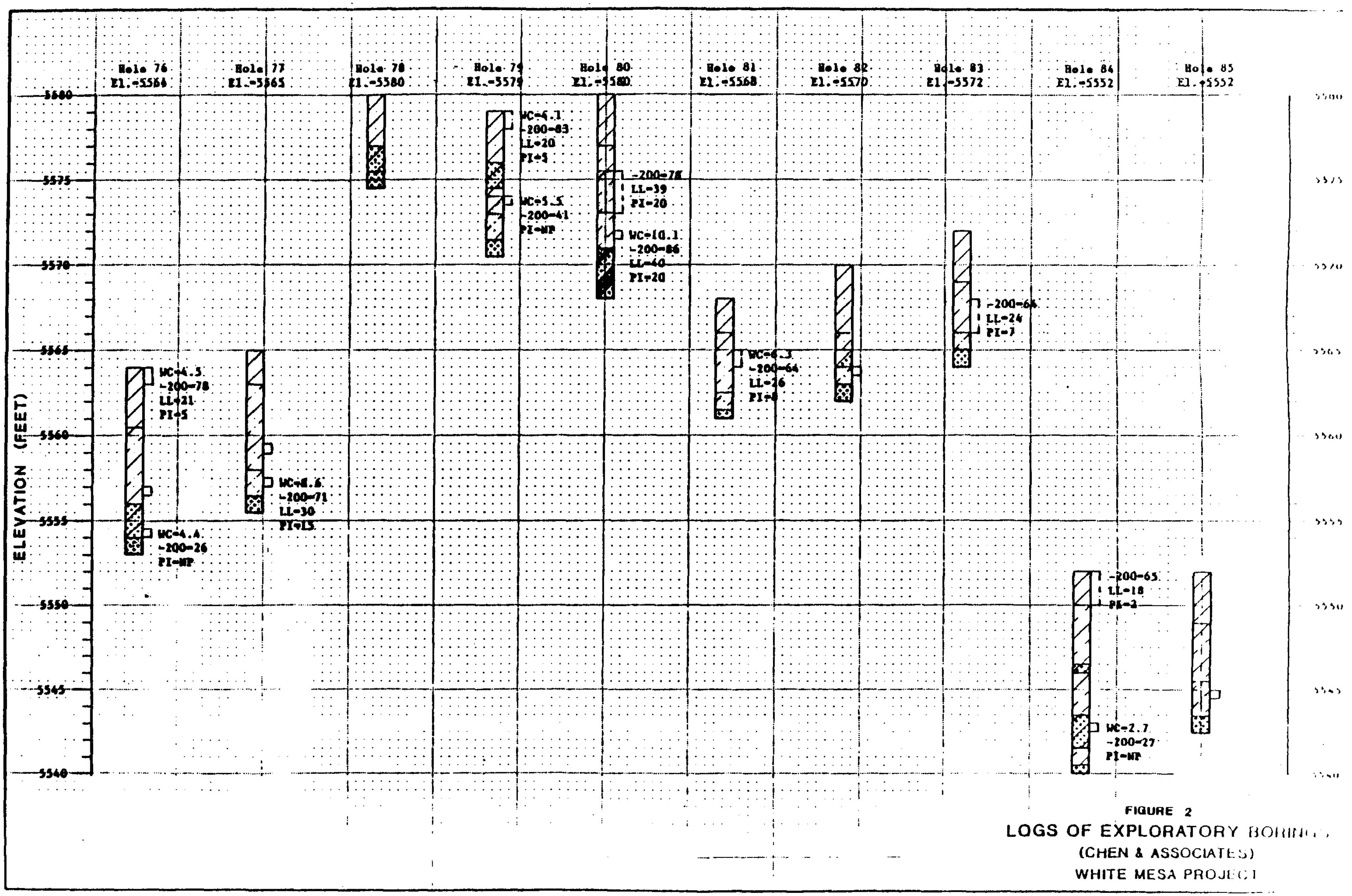

FIGURE 2
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

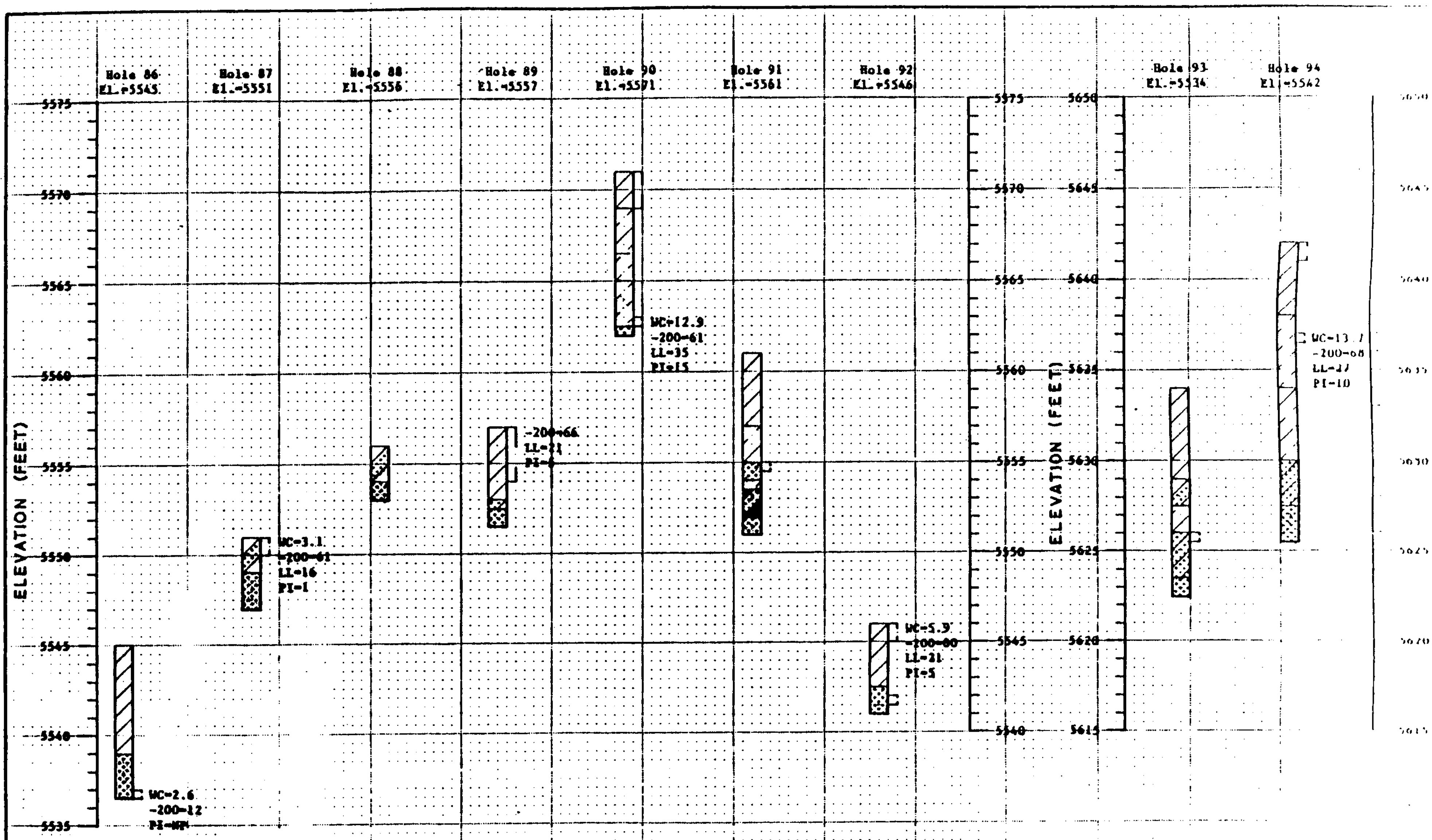

FIGURE 3
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

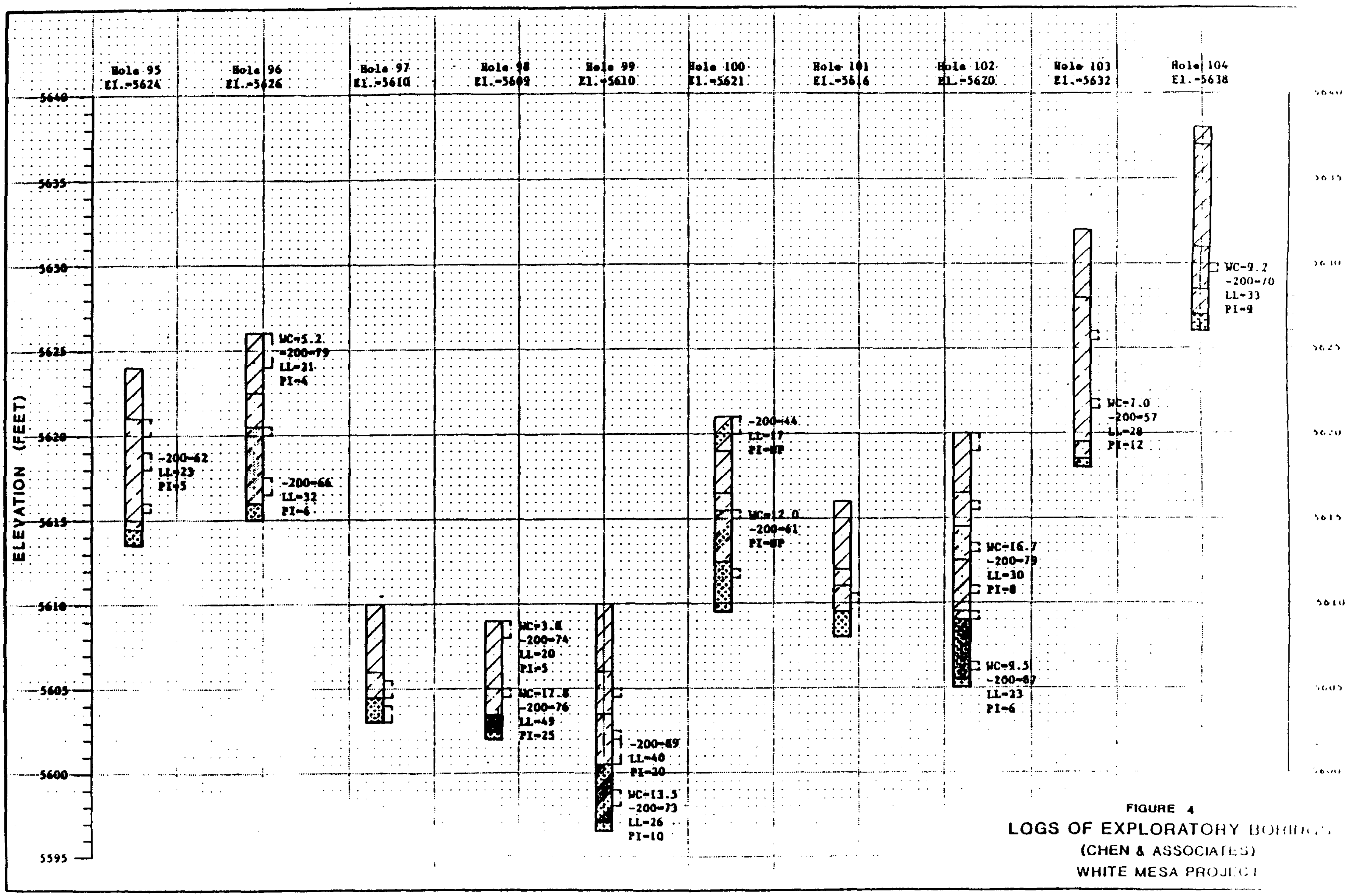

FIGURE 4
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

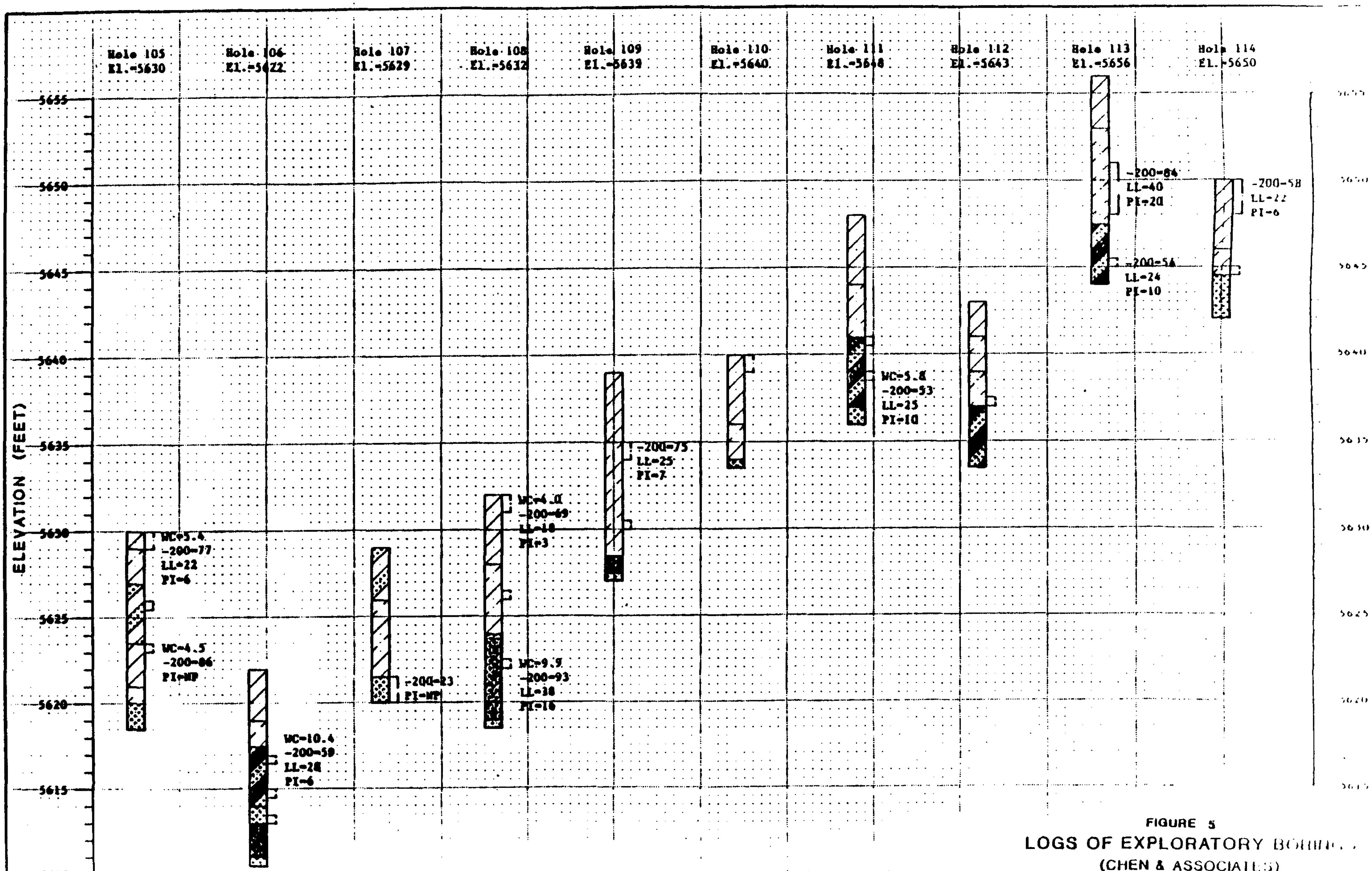

FIGURE 5
LOGS OF EXPLORATORY BORI...
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

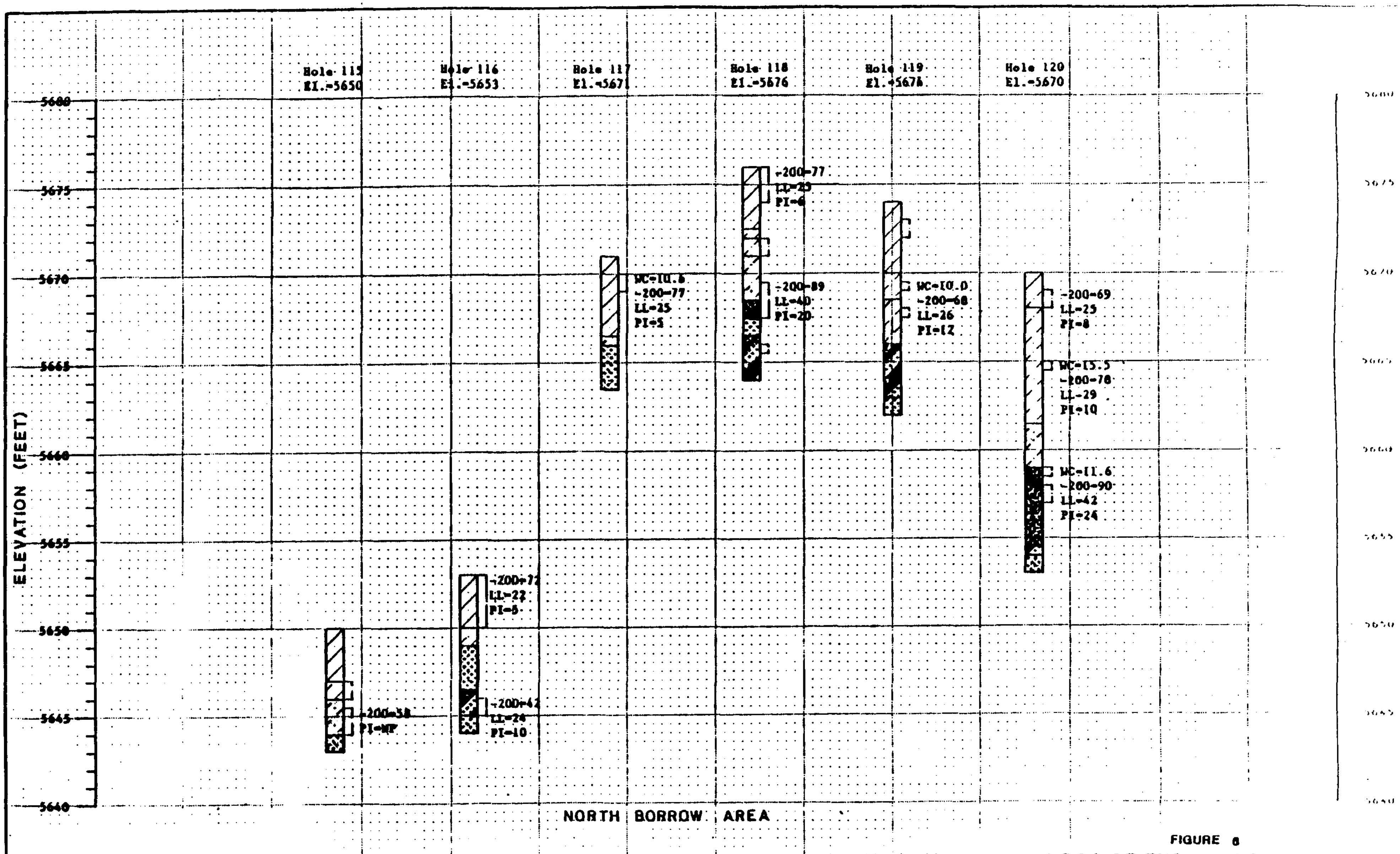

FIGURE 6
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

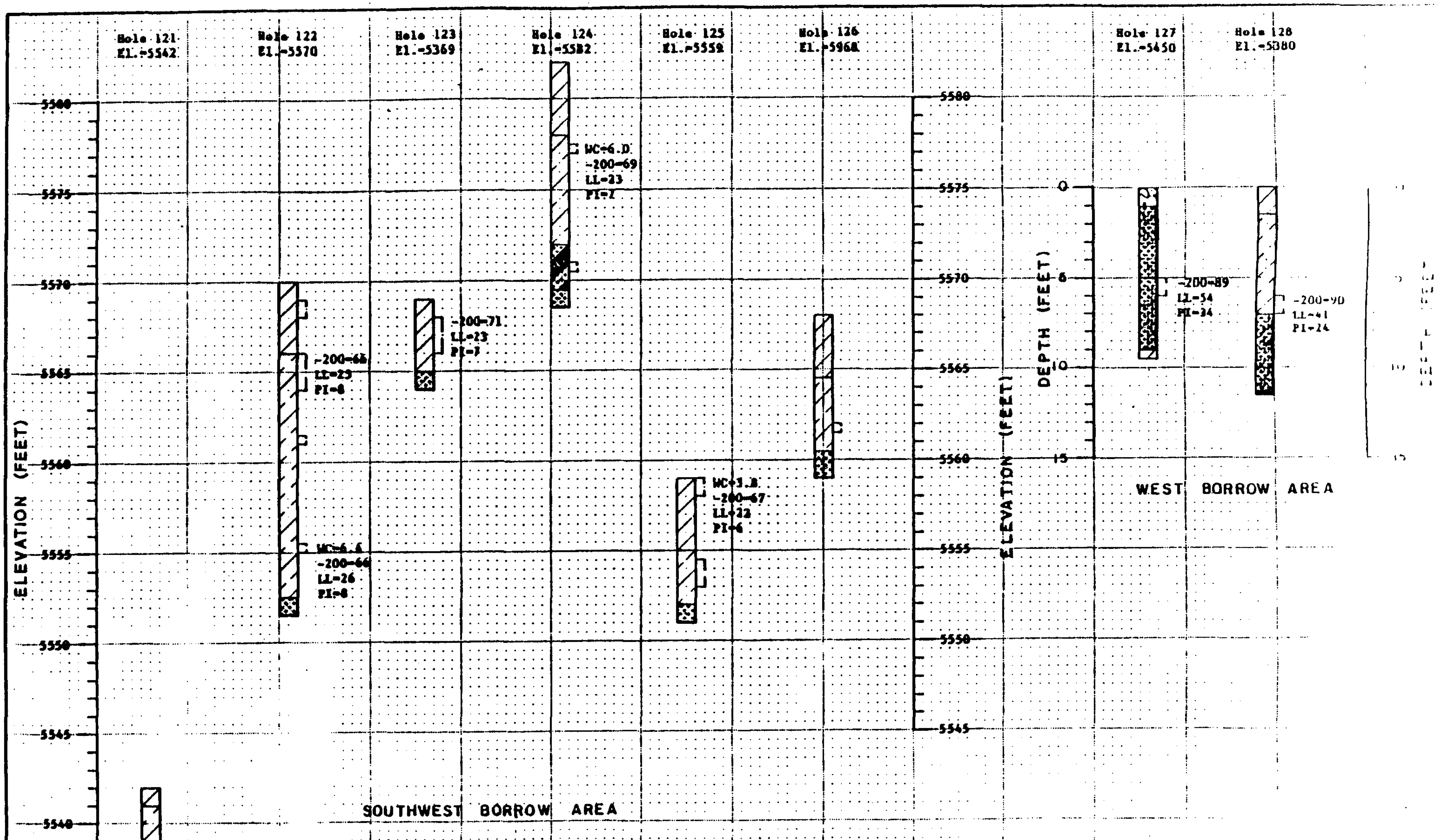

FIGURE 7

LOGS OF EXPLORATORY BORINGS

(CHEN & ASSOCIATES)

WHITE MESA PROJECT

LEGEND:

Silt (ML), sandy, approximately 60-70% silt, fine to medium sand size, slightly calcareous with depth, slightly moist to moist, reddish brown to light brown.

Sand, silty to sandy silt (SM-ML), fine to medium grained, approximately 50-60% silt, slightly moist to moist, reddish brown.

Sand (SM), silty, fine to medium grained, approximately 30-50% silt, some scattered gravel, slightly moist, reddish brown.

Clay, silty to sandy silt (CL-ML), approximately 60-75% low to non-plastic fines, fine to medium sand size, slightly to moderately calcareous with depth, slightly moist, light brown.

Clay (CL), sandy, approximately 60-75% low to medium plastic fines, fine to medium sand size, slightly calcareous, slightly moist, reddish brown.

Clay (CL), highly calcareous, sandy to silty, approximately 50-75% low plasticity fines, scattered very hard lenses/layer, dry to slightly moist, light tan to white.

Weathered claystone (CL-CH), approximately 75-90% medium to high plasticity fines, slightly moist to moist, gray-brown to greenish.

Claystone bedrock, slightly moist, greenish gray to dark gray.

Siltstone bedrock, well-cemented, very hard, gray.

Claystone-sandstone bedrock, lightly cemented, generally grading coarser with depth, fine to medium grained, slightly moist, greenish gray.

Sandstone-siltstone bedrock, lightly cemented, slightly moist, gray-brown.

Sandstone bedrock, fairly clean to silty and clayey, well cemented with depth, fine to medium grained, scattered conglomerate lenses/layers, slightly moist to dry, tan to gray.

Disturbed auger sample.

NOTES:

(1) Test holes were drilled on September 19 through 21, 1978, with a 12-inch, single-flight, power auger.
(2) Elevations are approximate and taken from contours shown n Fig. 1.
(3) No free water was found in the test holes at the time of drilling.
(4) WC = Water Content (%);
-200 = Percent Passing No. 200 Sieve;
LL = Liquid Limit (%);
PI = Plasticity Index (%);
NP = Nonplastic.

FIGURE 8
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

SECTION 1

Extracted Data From

REPORT
SITE SELECTION AND DESIGN STUDY
TAILING RETENTION AND MILL FACILITIES
WHITE MESA URANIUM PROJECT
BLANDING, UTAH
FOR ENERGY FUELS NUCLEAR, INC.

Dames and Moore

January 17, 1978

09973-015-14

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MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICTY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

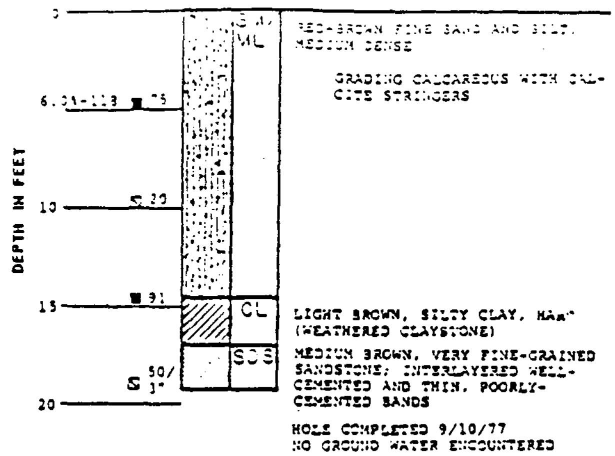
SOIL CLASSIFICATION CHART

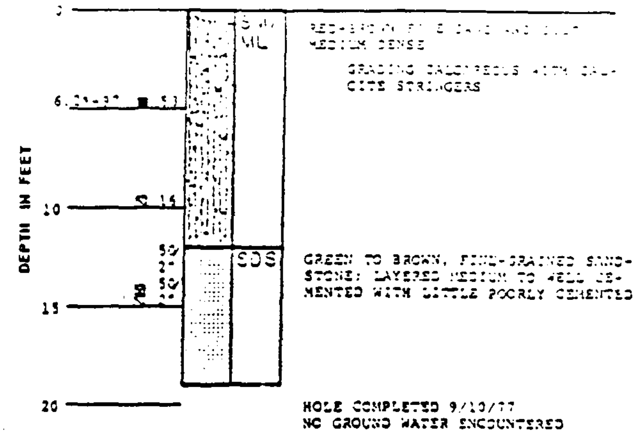
	SDS	SANDSTONE		SLN	SILTSTONE
	CLS	CLAYSTONE		CGL	CONGLOMERATE

GRAPHIC LOG SYMBOLS FOR ROCK

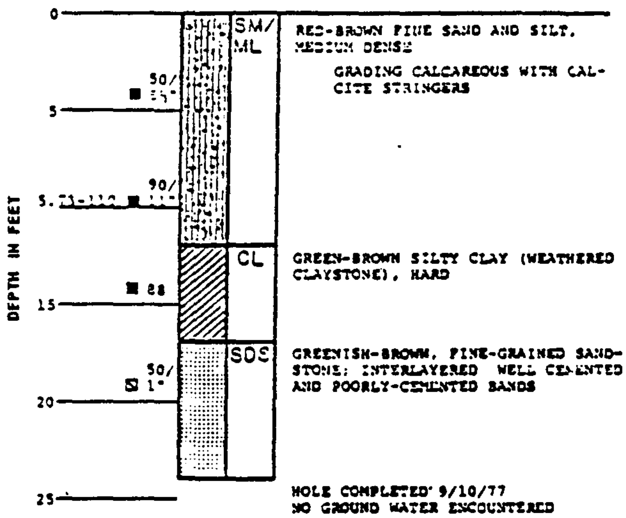
UNIFIED SOIL CLASSIFICATION SYSTEM AND GRAPHIC LOG SYMBOLS

DAMES & MOORE

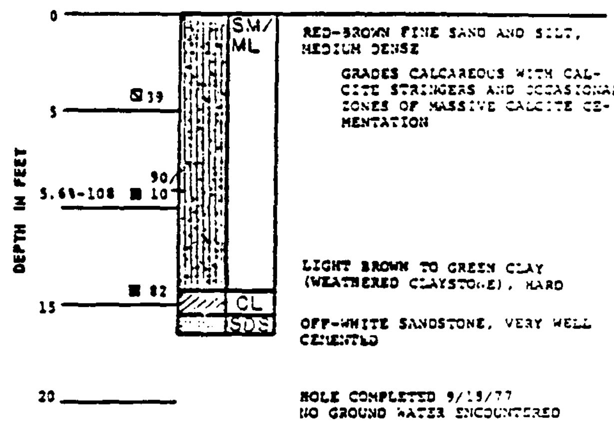




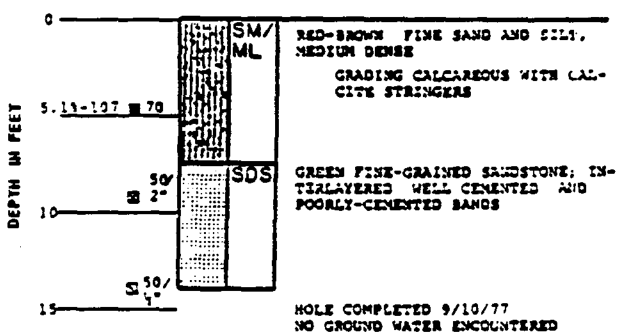
BORING NO. 2
EL. 5634.3 FT.



BORING NO. 6
EL. 5633.5 FT.



BORING NO. 4
EL. 5623.2 FT.



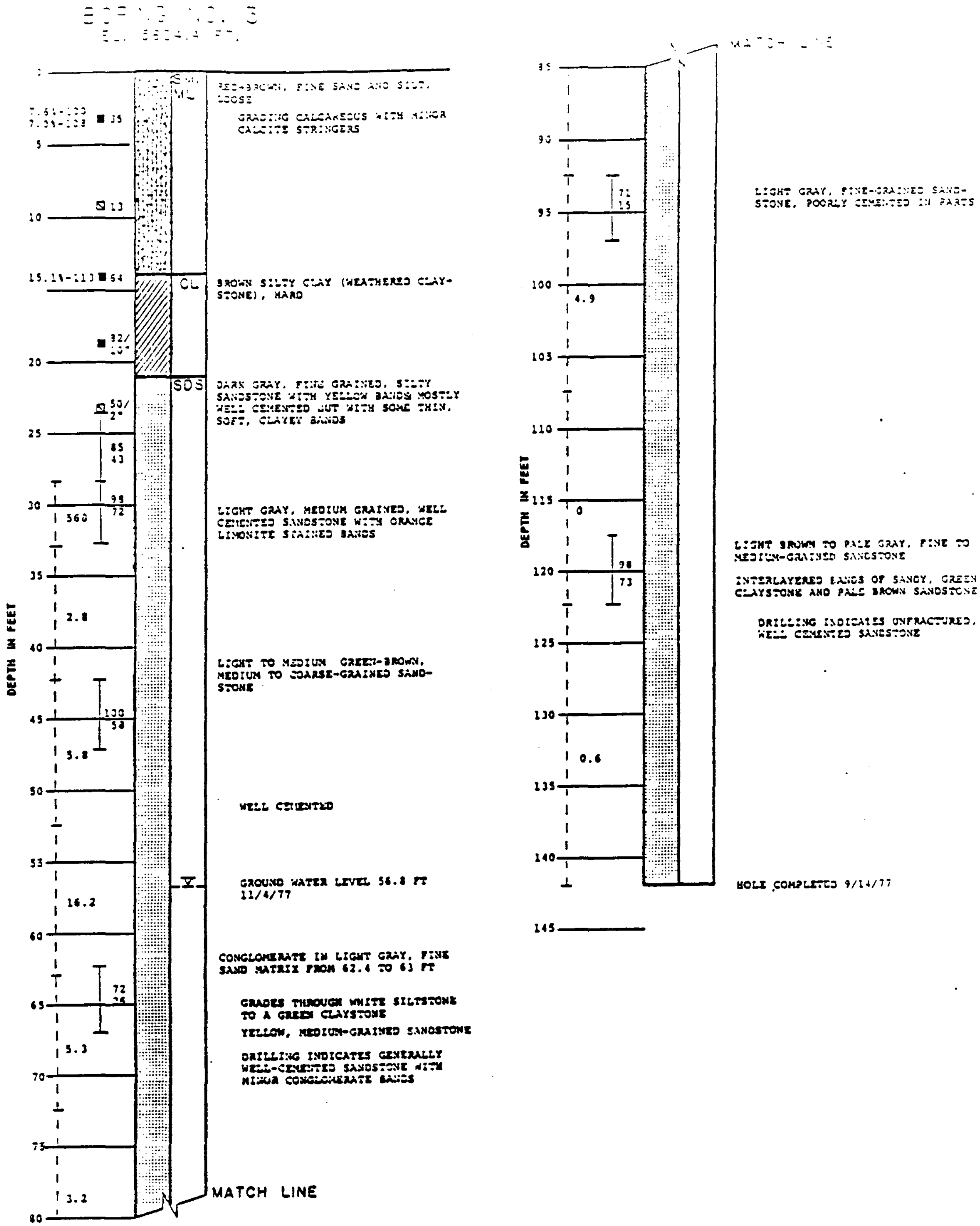
KEY

A-B ■ C	INDICATES DEPTH AT WHICH UNDISTURBED SAMPLE WAS EXTRACTED USING DAMES & MOORE SAMPLER
▣ C	INDICATES DEPTH AT WHICH DISTURBED SAMPLE WAS EXTRACTED USING DAMES & MOORE SAMPLER
□ C	INDICATES SAMPLE ATTEMPT WITH NO RECOVERY
⧄ C	INDICATES DEPTH AT WHICH DISTURBED SAMPLE WAS EXTRACTED USING STANDARD PENETRATION TEST SAMPLER
A	FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
B	DRY DENSITY EXPRESSED IN LBS/CU FT
C	BLOWS/FT OF PENETRATION USING A 140-LB HAMMER DROPPING 30 INCHES
D E	INDICATES NC CORE RUN
D	PERCENT OF CORE RECOVERY
E	RQD*
F	INDICATES PACKER TEST SECTION
F	PERMEABILITY MEASURED BY SINGLE PACKER TEST IN FT/YR
NA	NOT APPLICABLE (USED FOR RQD IN CLAYS OR MECHANICALLY FRACTURED ZONES)
NOTE:	ELEVATIONS PROVIDED BY ENERGY FUELS NUCLEAR, INC.

* ROCK QUALITY DESIGNATION -- PERCENTAGE OF CORE RECOVERED IN LENGTHS GREATER THAN 4 INCHES

LOG OF BORINGS

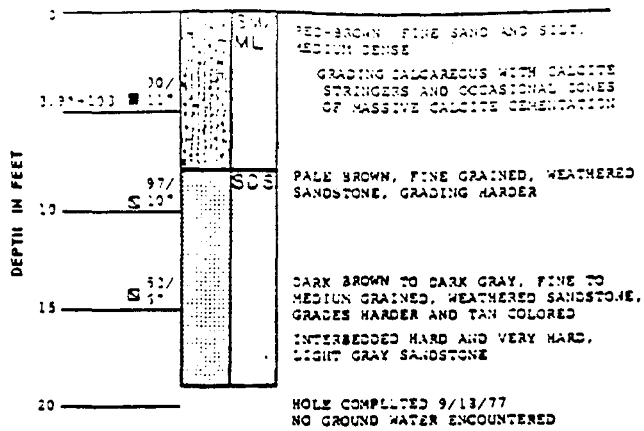
DAMES & MOORE

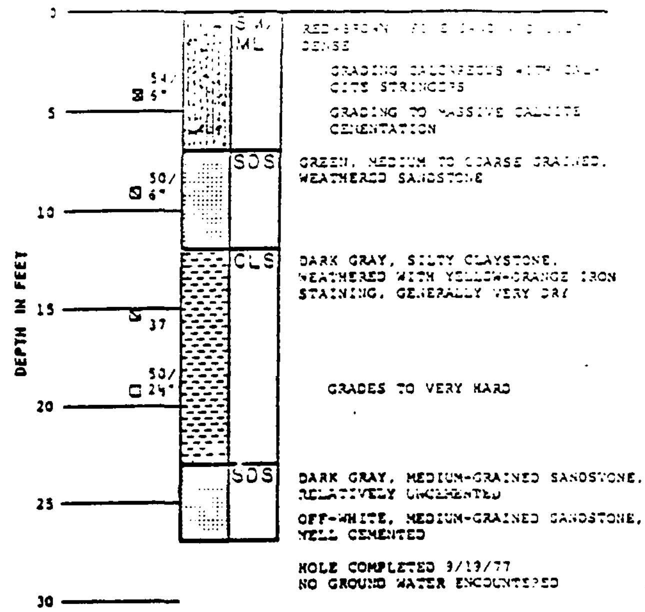


LOG OF BORINGS

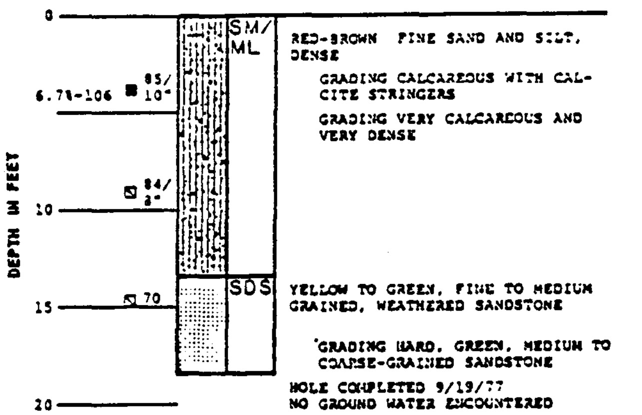
DAMES & MOORE

PLATE A-4

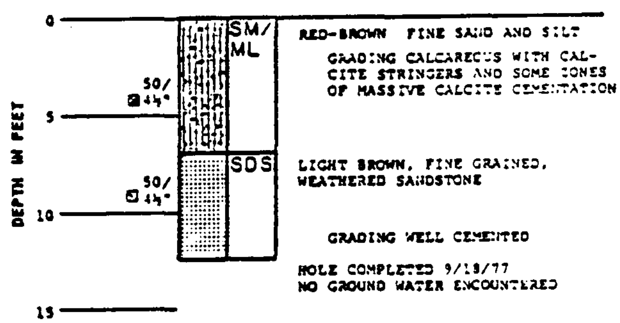




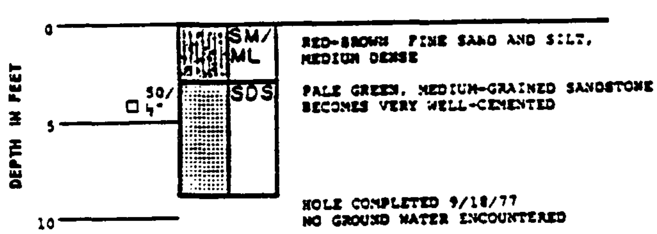
BORING NO. 10
EL. 5690.9 FT.



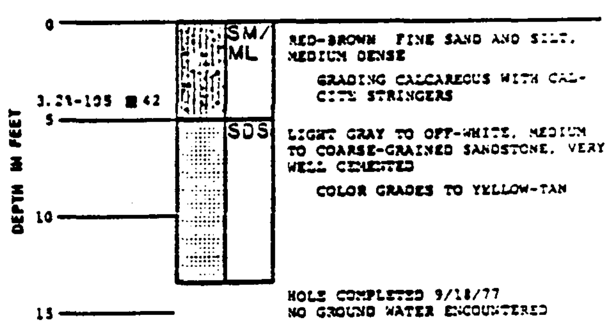
BORING NO. 11
EL. 5677.8 FT.



BORING NO. 13
EL. 5602.4 FT.

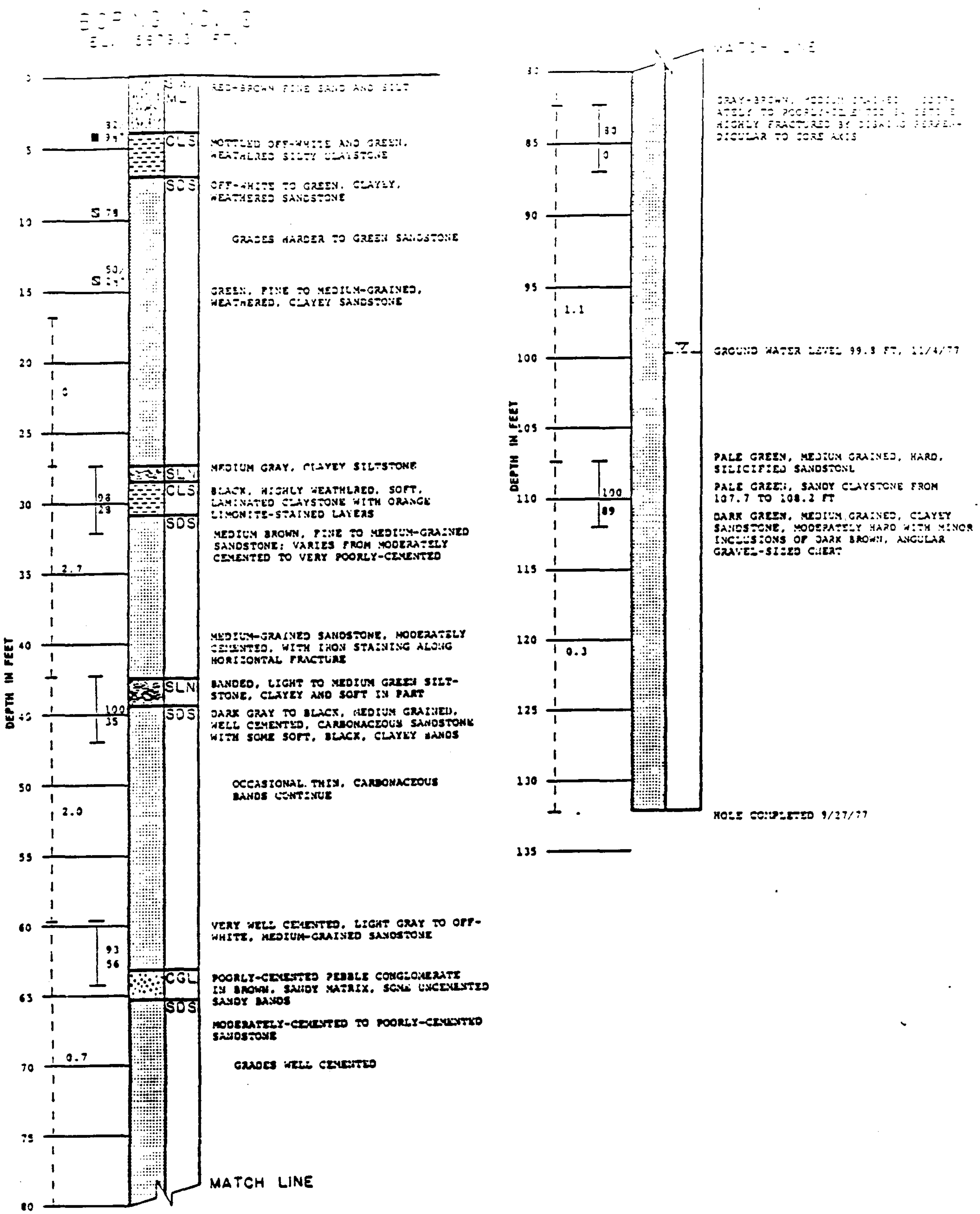


BORING NO. 14
EL. 5597.5 FT.



LOG OF BORINGS

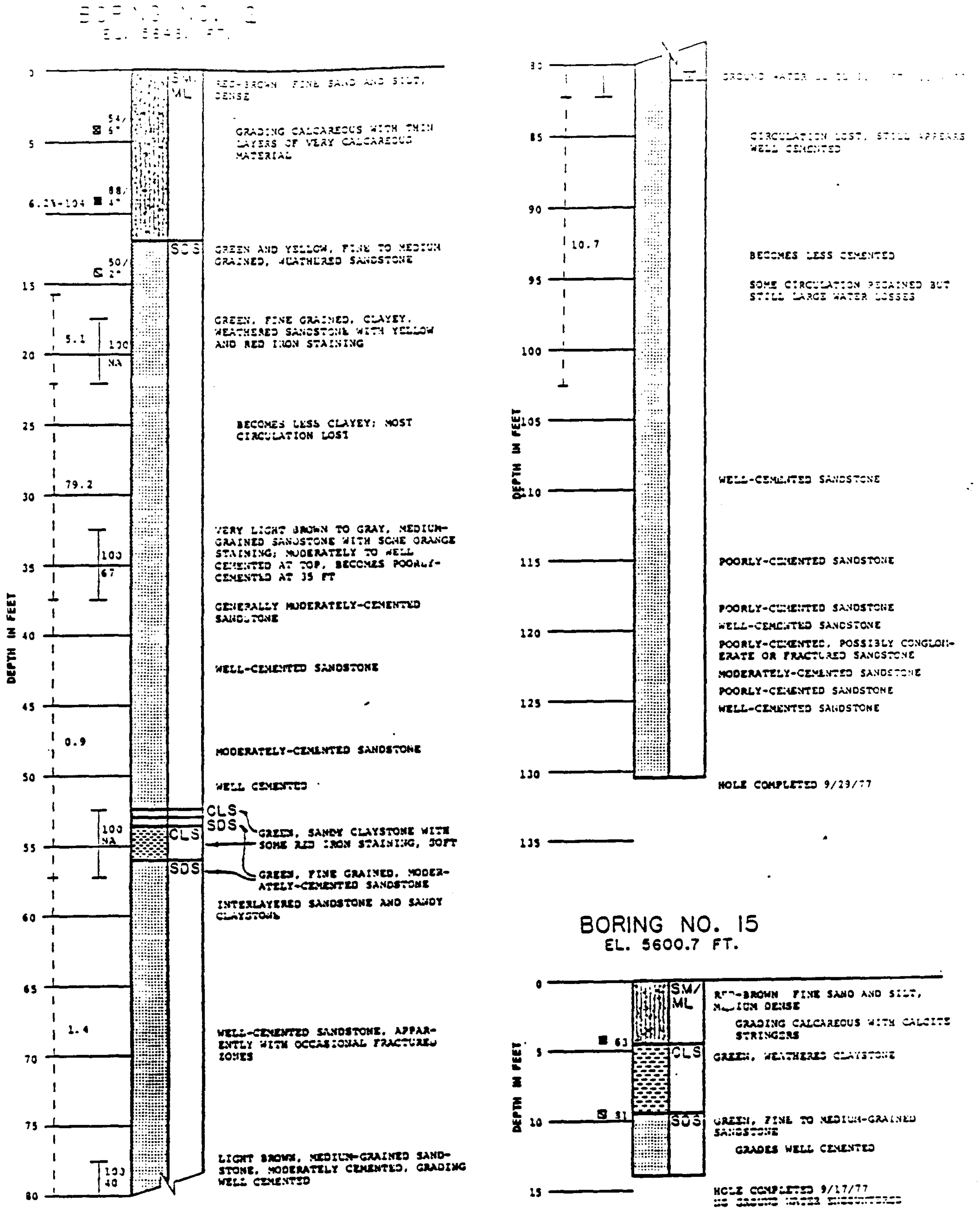
DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE

PLATE A-6

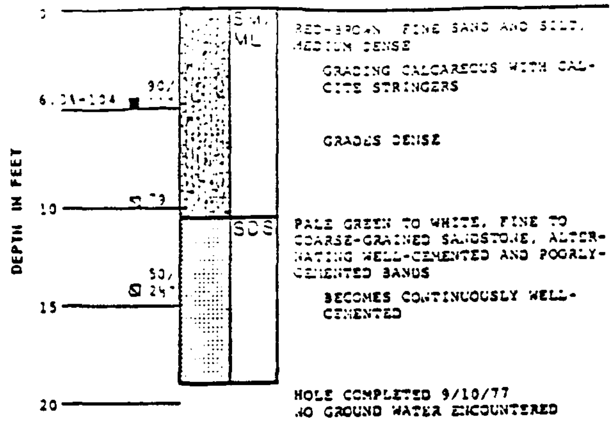


LOG OF BORINGS

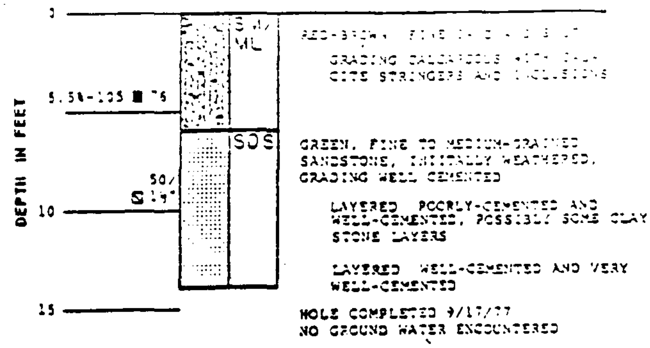
DAMES & MOORE

PLATE A-7

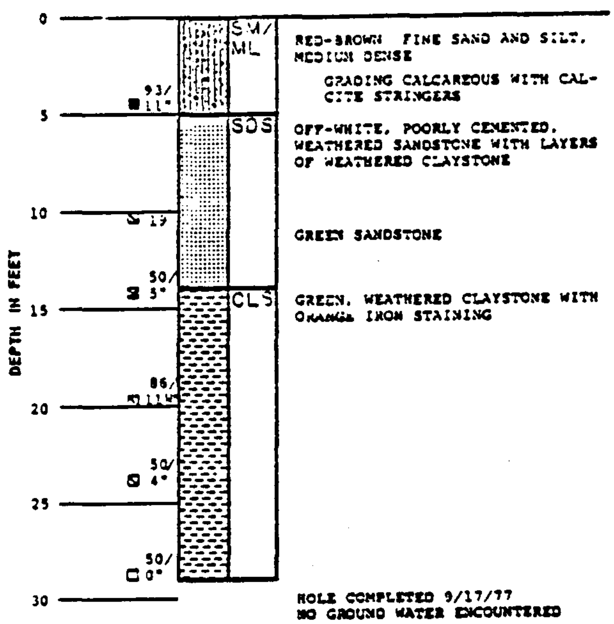
BORING NO. [illegible]
EL. [illegible] FT.



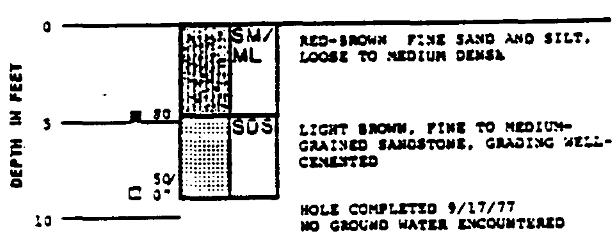
BORING NO. [illegible]
EL. [illegible] FT.



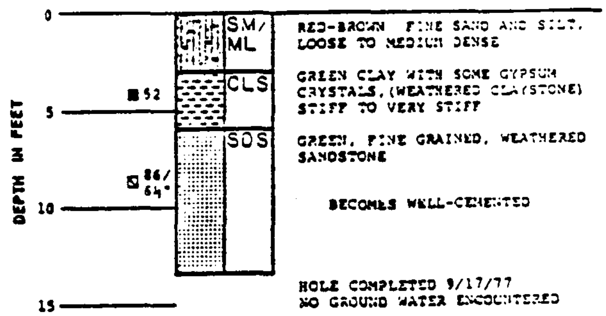
BORING NO. 18
EL. 5608.5 FT.



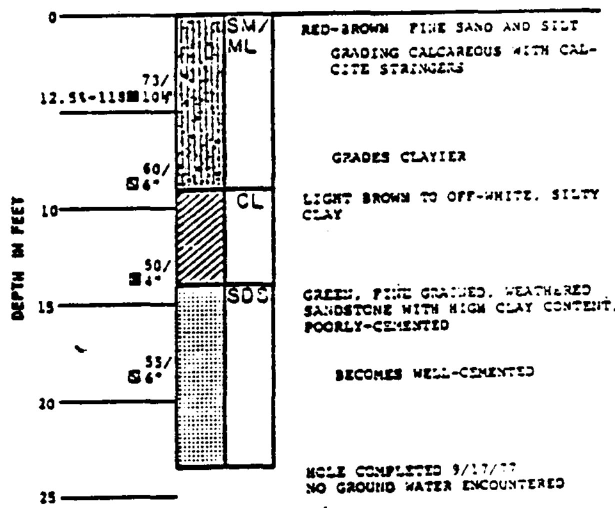
BORING NO. 20
EL. 5570.4 FT.



BORING NO. 21
EL. 5584.5 FT.



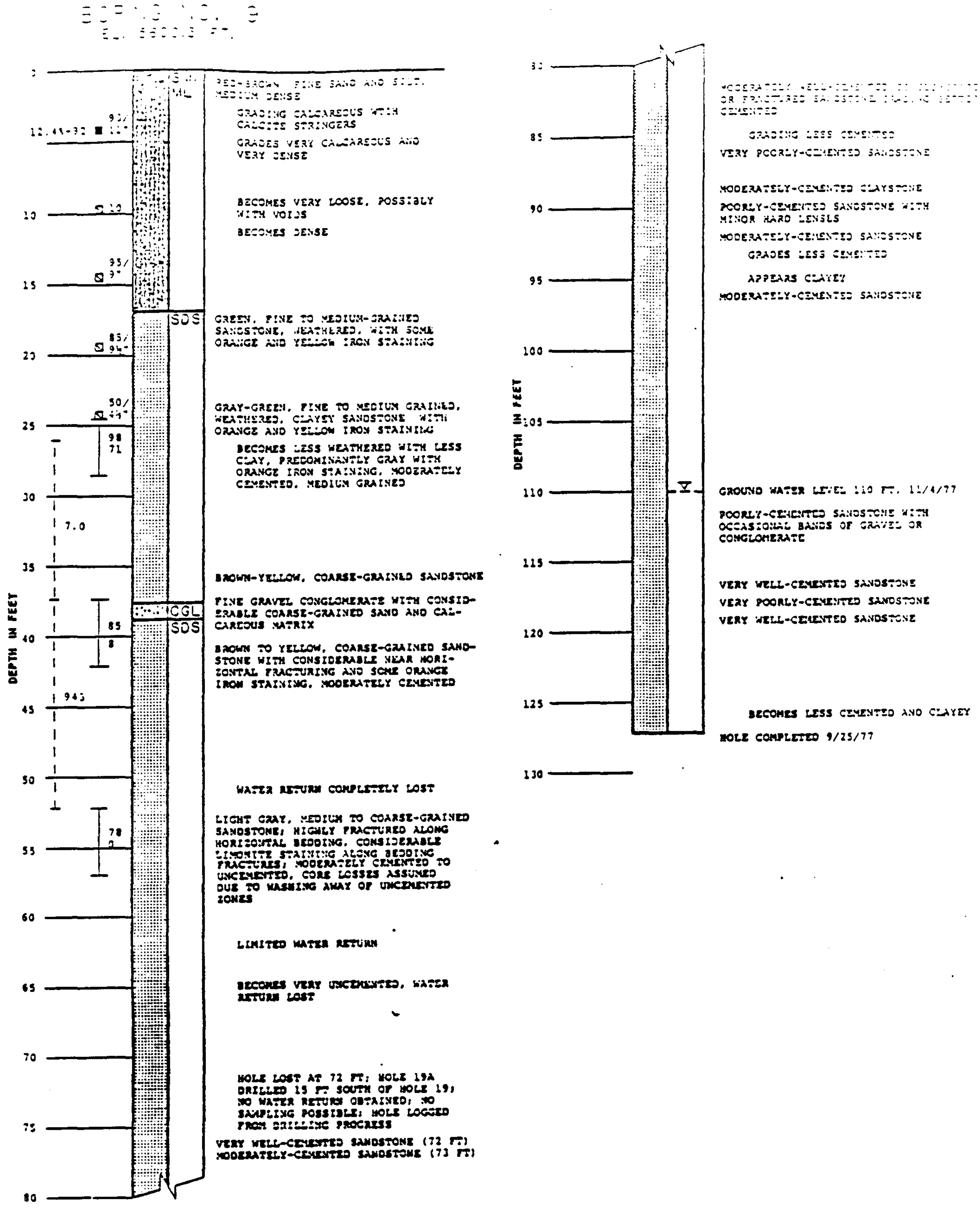
BORING NO. 22
EL. 5585.3 FT.



LOG OF BORINGS

DAMES & MOORE

PLATE A-8

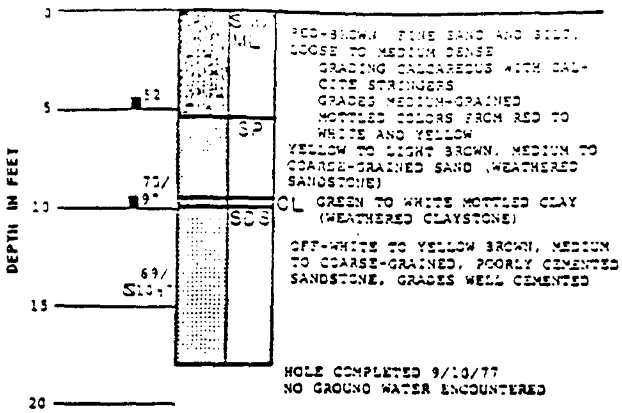


LOG OF BORINGS

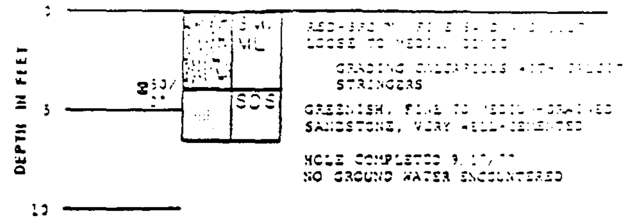
DAMES & MOORE

PLATE A-9

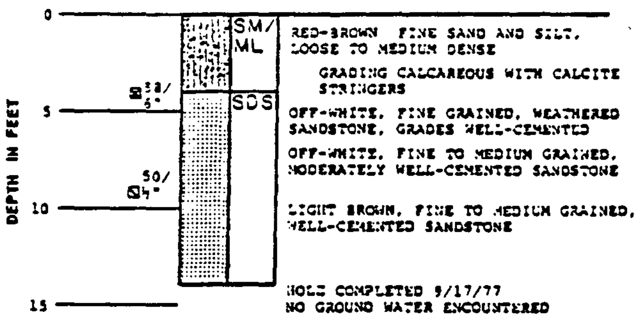
BORING NO. 23
EL. [illegible] FT.



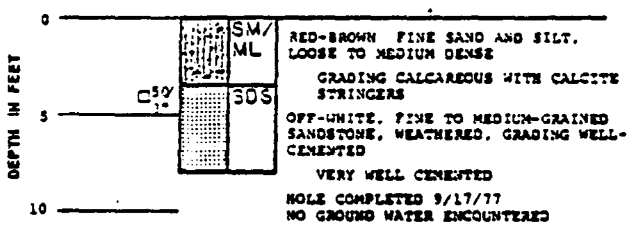
BORING NO. [illegible]



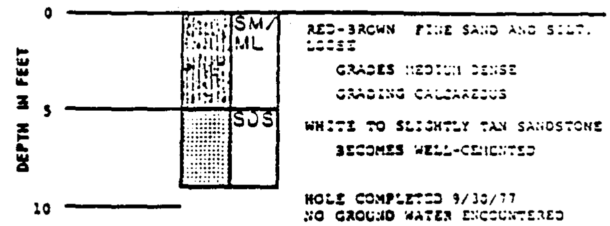
BORING NO. 24
EL. 5573.4 FT



BORING NO. 26
EL. 5578.3 FT.

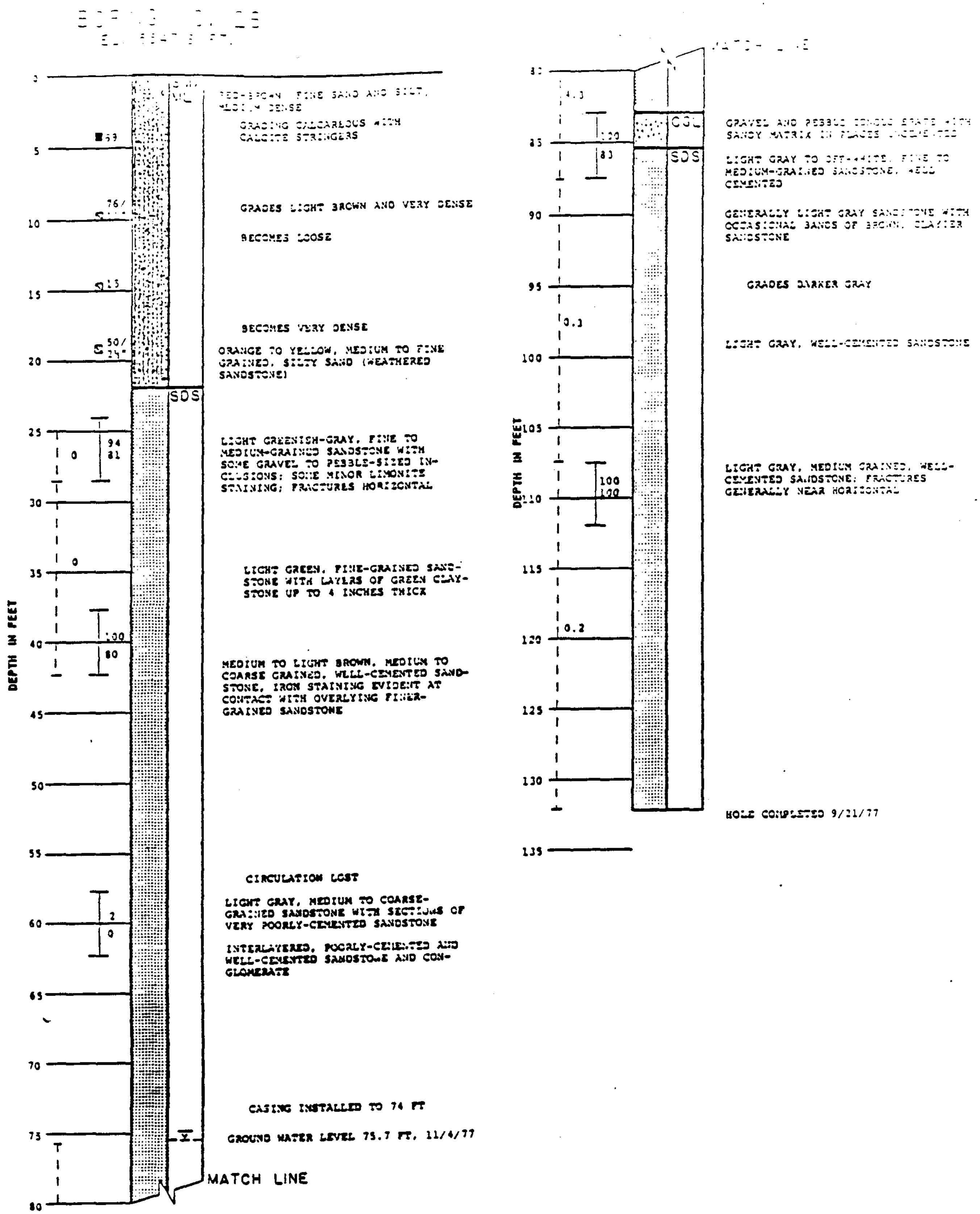


BORING NO. 29
EL. 5655.0 FT. (APPROX)



LOG OF BORINGS

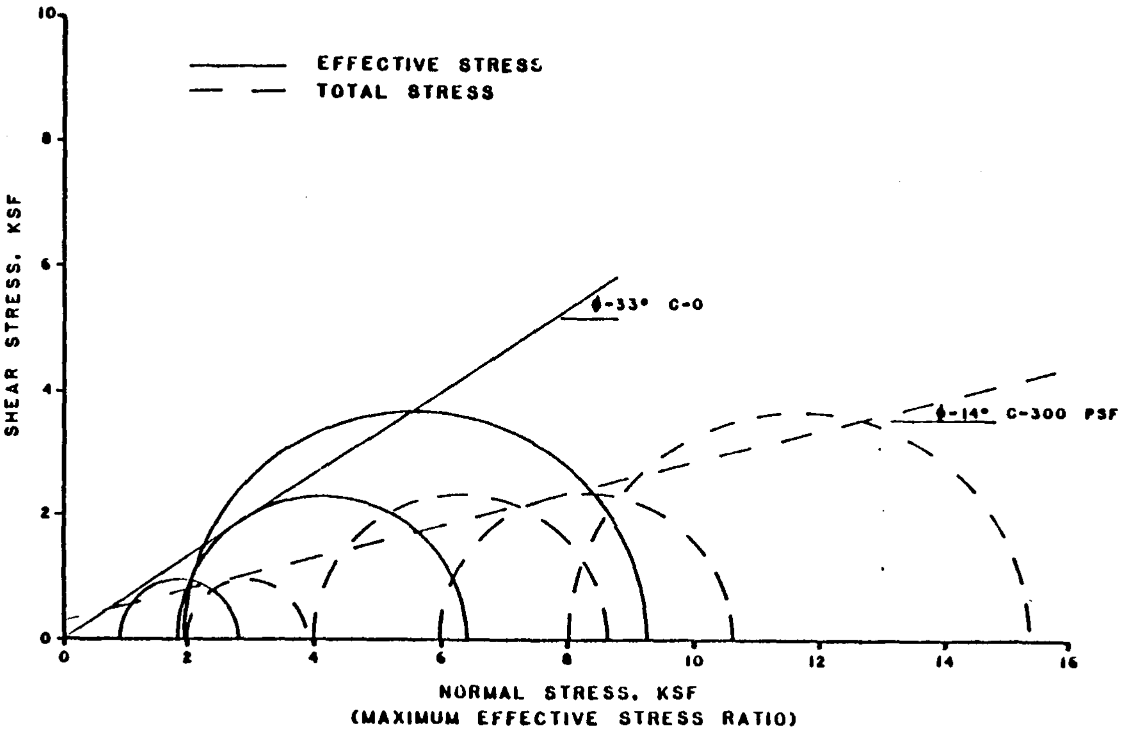
DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE

TRIAXIAL COMPRESSION TESTS ON SILTY FINE SAND COMPACTED TO 95% OF AASHTO T-99 MAXIMUM DRY DENSITY



		(1)		(2)		(3)		( )	
KEY		(1)		(2)		(3)		[illegible]	
BORING		19A		19A		19A		[illegible]	
SAMPLE		1		2		3		[illegible]	
DEPTH (FEET)		Bulk		Bulk		Bulk		[illegible]	
INITIAL	w, %	13.3		13.2		13.3		[illegible]	
	γ_d, PCF	111.1		111.2		111.1		[illegible]	
	e_o	0.529		0.527		0.528		[illegible]	
	% S %	68 %		68 %		68 %		[illegible]	
FINAL	w, %	18.2		17.4		16.7		[illegible]	
	γ_d, PCF	114.7		117.1		117.0		[illegible]	
	e_f	0.481		0.450		0.451		[illegible]	
	% S %	100 %		100 %		100 %		[illegible]	
BACK PRESSURE (PSI)		86.1		72.2		165		[illegible]	
STRAIN RATE (INCHES/MINUTE)		.000833		.000833		.000833		[illegible]	
STRESS CONDITION		PEAK $\sigma_1 - \sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1 - \sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1 - \sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1 - \sigma_3$	MAX
TOTAL STRESS	ε, %	13.49	5.56	20.00	10.98	17.64	8.51	[illegible]	[illegible]
	TIME TO FAIL (MIN)	960	396	1421	780	1261	[illegible]	[illegible]	[illegible]
	σ_3, KSF	2.00	2.00	4.00	4.00	6.00	[illegible]	[illegible]	[illegible]
	$\sigma_1 - \sigma_3$	2.98	1.88	5.68	4.57	5.12	[illegible]	[illegible]	[illegible]
	σ_1, KSF	4.98	3.88	9.68	8.57	11.12	[illegible]	[illegible]	[illegible]
	$\frac{1}{2}(\sigma_1 - \sigma_3)$	1.24	0.94	2.84	[illegible]	2.53	[illegible]	[illegible]	[illegible]
	$\frac{1}{2}(\sigma_1 + \sigma_3)$	13.24	2.92	6.84	[illegible]	8.59	[illegible]	[illegible]	[illegible]
	u, KSF	0.72	1.07	1.61	2.19	3.84	[illegible]	[illegible]	[illegible]
	A, $u/(\sigma_1 - \sigma_3)$	0.29	0.57	0.28	[illegible]	0.75	[illegible]	[illegible]	[illegible]
EFFECTIVE STRESS	ε, %	13.49	5.56	20.00	10.98	17.64	8.51	[illegible]	[illegible]
	TIME TO FAIL (MIN)	960	396	1421	780	1261	[illegible]	[illegible]	[illegible]
	$\bar{\sigma}_3$, KSF	1.28	0.93	2.39	[illegible]	2.16	[illegible]	[illegible]	[illegible]
	$\bar{\sigma}_1 - \bar{\sigma}_3$	2.48	1.88	5.68	[illegible]	5.12	[illegible]	[illegible]	[illegible]
	$\bar{\sigma}_1$, KSF	3.76	2.81	8.07	[illegible]	7.28	[illegible]	[illegible]	[illegible]
	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$	1.24	0.94	2.84	[illegible]	2.53	[illegible]	[illegible]	[illegible]
	$\frac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3)$	2.52	1.87	5.23	[illegible]	4.75	[illegible]	[illegible]	[illegible]
	u, KSF	0.72	1.07	1.61	2.19	3.84	[illegible]	[illegible]	[illegible]
	A, $u/(\bar{\sigma}_1 - \bar{\sigma}_3)$	0.29	0.57	0.28	[illegible]	0.75	[illegible]	[illegible]	[illegible]
	$\bar{\sigma}_1/\bar{\sigma}_3$	2.94	3.01	3.38	[illegible]	3.36	[illegible]	[illegible]	[illegible]

TRIAXIAL COMPRESSION TEST REPORT

TYPE OF TEST CONSOLIDATED - UNDRAINED [illegible] WITH PORE PRESSURE MEASUREMENT

TYPE MATERIAL COMPACTED CORE

SAMPLE DESCRIPTION

CLASSIFICATION REDDISH - BROWN CLAYEY [illegible]

LIQUID LIMIT – PLASTIC LIMIT – SPECIFIC GRAVITY, G_s [illegible]

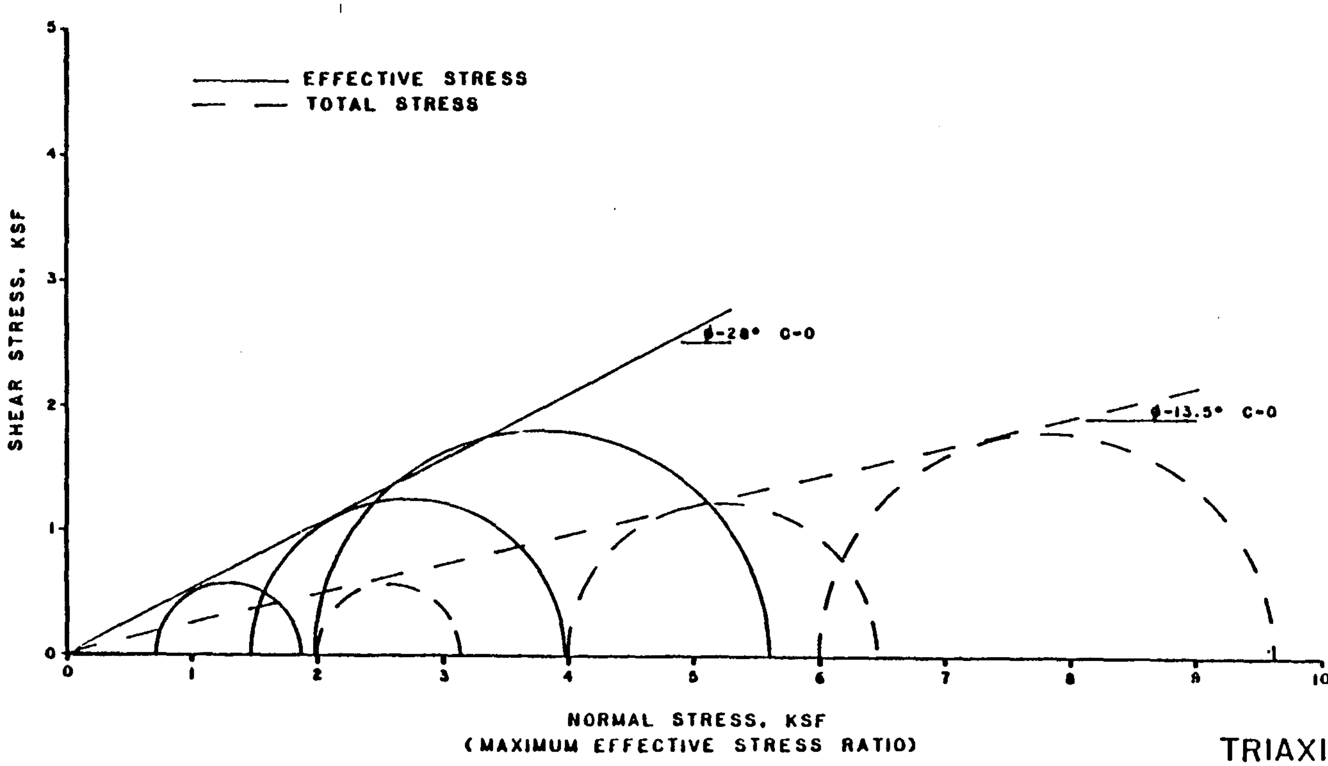
PROJECT ENERGY FUELS

LOCATION DENVER

JOB NO. 9773-015-12 PREPARED BY KY [illegible]

CHECKED BY KY [illegible]

MULTI PHASE TRIAXIAL COMPRESSION TESTS ON SILTY FINE SAND AT NATURAL DENSITY



KEY		(1)		(2)		(3)	
BORING		14		14		14	
SAMPLE		1		1		1	
DEPTH (FEET)		4'		4'		4'	
INITIAL	w, %	3.2		19.7		[illegible]	
	γ_d, PCF	104.7		108.6		[illegible]	
	e_o	.5803		.5227		[illegible]	
	S	.12		1.0		1.0	
FINAL	w, %	19.7		17.7		[illegible]	
	γ_d, PCF	108.6		112.6		[illegible]	
	e_f	.5227		.4693		[illegible]	
	S	1.0		1.0		1.0	
BACK PRESSURE (PSI)							
STRAIN RATE (INCHES / MINUTE)							
STRESS CONDITION		PEAK $\sigma_1 - \sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1 - \sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1 - \sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$
TOTAL STRESS	ε, %	1.5	2.5	1.0	1.5	1.0	[illegible]
	TIME TO FAIL (MIN)	15	25	10	15	10	[illegible]
	σ_3, PSF	2000	2000	4000	4000	6000	6000
	$\sigma_1 - \sigma_3$	1171	1160	2484	2467	3704	[illegible]
	σ_1, PSF	3171	3160	6484	6467	9704	[illegible]
	$\frac{1}{2}(\sigma_1 - \sigma_3)$	585	580	1242	1234	1852	[illegible]
	$\frac{1}{2}(\sigma_1 + \sigma_3)$	2585	2580	5242	5234	7852	[illegible]
	u, PSF	1166	1282	2318	2506	3441	[illegible]
	A, $u/(\sigma_1 - \sigma_3)$	.996	1.11	.933	1.02	.929	[illegible]
EFFECTIVE STRESS	ε, %	1.5	2.5	1.0	1.5	1.0	[illegible]
	TIME TO FAIL (MIN)	15	25	10	1500	10	[illegible]
	$\bar{\sigma}_3$, PSF	834	718	1682	1494	2559	[illegible]
	$\bar{\sigma}_1 - \bar{\sigma}_3$	1171	1160	2484	2467	3704	[illegible]
	$\bar{\sigma}_1$, PSF	2005	1878	4166	3961	[illegible]	[illegible]
	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$	585	580	1242	1234	1852	[illegible]
	$\frac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3)$	1420	1298	2924	2729	[illegible]	[illegible]
	u, PSF	1166	1282	2318	2506	[illegible]	[illegible]
	A, $u/(\bar{\sigma}_1 - \bar{\sigma}_3)$	.956	1.11	.933	1.02	[illegible]	[illegible]
	$\bar{\sigma}_1/\bar{\sigma}_3$	2.40	2.62	2.48	2.65	[illegible]	[illegible]

TRIAXIAL COMPRESSION TEST REPORT

TYPE OF TEST TX-CU-PP

TYPE MATERIAL Brn Silt & Fi Sand

SAMPLE DESCRIPTION

CLASSIFICATION SM/ML

LIQUID LIMIT N/A PLASTIC LIMIT N/A SPECIFIC GRAVITY, G_s [illegible]

PROJECT Energy Fuels

LOCATION Blanding, UT.

JOB NO. [illegible] PREPARED BY LWC [illegible]

CHECKED BY

SECTION 5

TEST PIT LOGS

From

Engineer's Report

Second Phase Design – Cell 3 Tailings Management System

White Mesa Uranium Project
Blanding, Utah

Energy Fuels Nuclear, Inc.
Denver, Colorado

TABLE OF CONTENTS

DATE BEGAN: [illegible]
DATE FINISHED: [illegible] 81
GROUND SURFACE EL.: ~5594'

TEST PIT NO. [illegible]

N ~[illegible] E~[illegible]

FIELD ENGINEER [illegible]
CHECKED BY R. [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS
	1.0		Medium stiff red sandy silt. Dry. Roots to depth 1.5'	SAMPLE NO. 1, DEPTH 10"-12", TYPE JAR
	2.0		Very stiff red-white sandy silt. Dry. Very calcareous 2.8'	2, 22"-24", JAR
	3.0		Soft-medium stiff red sandy silt. Moist, w/streaks of white. Slightly calcereous	
5590	4.0			3, 4'6", JAR
	5.0			A, 5'-5'6", BAG; 3.8% $CaCO_3$
	6.0			
	7.0			
5586	8.0			4, 8'4", JAR
	9.0		9.0'	
	10.0		Very soft greenish gray claystone. Moist, weathered	5, 10'4", JAR; Consistency of soil but with intact layering
	11.0			B, 10'-10'6", BAG; 9.55% $CaCO_3$
5582	12.0			

DATE BEGAN [illegible]
DATE FINISHED [illegible]
GROUND SURFACE ELEV. ~5594'

TEST PIT NO [illegible]

N ~319,690' E~2,578,206'

FIELD ENGINEER [illegible]
CHECKED BY [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS
	13.0		"Same as above" 13.5'	
	14.0		Soft yellowish brown sandstone. Weathered	Intact, Easy rippable
	15.0			Ripping becomes difficult
5578	16.0		16.0'	
			Bottom of test pit 16.0'	

DATE BEGAN [illegible]
FINISHED [illegible]
GROUND SURFACE EL. ~5583'

TEST PIT NO. 4-2

N ~319,950' E ~2,577,955'

FIELD ENGINEER [illegible]
CHECKED BY [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS: SAMPLE NO.	DEPTH	TYPE
	1.0		Red sandy silt. Dry w/ roots to depth			
	2.0		2.0'			
			Medium dense red silty sand. Moist.	A 2.3% $CaCO_3$	2'-5'	BAG
5580	3.0			1	3'	JAR
	4.0					
	5.0		5.7'			
	6.0		Soft yellowish brown sandstone	2	5'8"	JAR
5576	7.0		7.0'			
			Bottom of test pit 7.0'			

DATE BEGAN: [illegible]

DATE FINISHED: [illegible] 81

GROUND SURFACE EL.: ~5582'

TEST PIT NO. 4-3

N ~319,552' E ~2,577,950'

FIELD ENGINEER: [illegible]

CHECKED BY: R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				SAMPLE NO.	DEPTH	TYPE
	1.0		Medium dense red sandy silt. Dry. Roots to depth 1.3'			
5580	2.0		Medium dense red-white silty sand. Very calcereous 3.3'	1	1'8"	JAR
	3.0			A 21.6% $CaCO_3$	2'-3'4"	BAG
5578	4.0		Soft greenish gray to yellowish brown claystone-sandstone. Weathered 5.0'	2	3'9"	JAR
	5.0		Bottom of test pit 5.0'			

DATE BEGAN: [illegible]

DATE FINISHED: [illegible]

GROUND SURFACE EL: ~5581'

TEST PIT NO. [illegible]

N~[illegible] E~2,577,530'

FIELD ENGINEER: [illegible]

CHECKED BY: [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS: SAMPLE NO.	DEPTH	TYPE
5580	1.0		Medium dense red sandy silt. Dry. Roots to depth — 1.5'			
	2.0		Dense red-white silty sand. Slightly calcareous. Dry — 2.7'	1	2'1"	JAR
	3.0		Medium dense red silty sand. Moist	A 1.7% $CaCO_3$	3'	BAG
	4.0					
5576	5.0					
	6.0			2	6'	JAR
	7.0					
	8.0		8.0'			
5572	9.0		Soft yellowish brown sandstone. Weathered			
	10.0					
	11.0					
	12.0		12.0'			
			Bottom of test pit 12.0'			

DATE BEGAN [illegible]

DATE FINISHED: [illegible]

GROUND SURFACE EL.: ~5578'

TEST PIT NO. [illegible]

N ~[illegible] E ~[illegible]

FIELD ENGINEER [illegible]

CHECKED BY [illegible]

ELEV (FEET)	DEPTH (FEET)	DESCRIPTION	SAMPLE NO.	DEPTH	TYPE
	1.0	Medium stiff red sandy silt. Dry. Roots to depth 1.5'			
5576	2.0–3.0–4.0	Dense red silty sand. Dry. Slightly calcareous 3.8'	1	2'	JAR
			A 2.9% $CaCO_3$	3'-5'	BAG
	5.0	Medium dense red silty sand. Moist 5.0'	2	4'	JAR
5572	6.0–7.0	Soft yellowish brown sandstone 7.0'			
		Bottom of test pit 7.0'			

DATE BEGAN: [illegible]
FINISHED: [illegible]
GROUND SURFACE EL.: ~5579'

TEST PIT NO. 4-6

N ~[illegible]' E ~[illegible]'

FIELD ENGINEER: R. [illegible]
CHECKED BY: R. [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				SAMPLE NO.	DEPTH	TYPE
5578	1.0		Medium stiff red sandy silt			
	2.0		2.0'			
			Dense red-white silty sand. Dry. Very calcareous			
	3.0			1	3'	JAR
			3.7'			
	4.0		Medium dense red silty sand. Moist	A 3% $CaCO_3$	4'	BAG
				2	4.5'	JAR
5574	5.0					
	6.0					
	7.0					
	8.0		8.0'			
			Soft yellowish brown sandstone 8.5'			
			Bottom of test pit 8.5'			

DATE BEGAN [illegible]

DATE FINISHED: [illegible]

GROUND SURFACE EL: ~558[illegible]'

TEST PIT NO [illegible]

N [illegible]' E [illegible]'

FIELD ENGINEER [illegible]

CHECKED BY R. [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				SAMPLE NO.	DEPTH	TYPE
	1.0 2.0		Medium stiff red sandy silt. Dry. Roots to depth 2.0'	1	2'	JAR
	3.0		Dense redish-white silty sand. Very dry. Chalky. Very calcareous 3.0'			
5580	4.0 5.0 6.0		Medium dense red-white silty sand. Moist. Very calcareous 6.0'	2 A 29.7% $CaCO_3$ 3	4' 3'6"-6' 6'	JAR BAG JAR
5576	7.0 8.0 9.0		Dense redish white silty sand. Very dry. Chalky. Very calcareous 9.0'			
	10.0		Soft yellowish brown sandstone 10.0'			
			Bottom of test pit 10.0'			

DATE BEGAN [illegible]

DATE FINISHED: [illegible]

GROUND SURFACE EL.: ~[illegible]'

TEST PIT NO [illegible]

N ~319,918' E ~2,576,056'

FIELD ENGINEER [illegible]

CHECKED BY [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS: SAMPLE NO.	DEPTH	TYPE
5588	1.0		Medium stiff red sandy silt. Dry. Roots to depth 1.7'			
	2.0		Dense red silty sand. Dry. Some with white streaks 2.7'	1	2'	JAR
	3.0 4.0		Loose red silty sand. Moist. Some white streaks	A 7.2% $CaCO_3$	3'-6'	BAG
5584	5.0		5.0'	2	4'5"	JAR
	6.0 7.0 8.0		Dense whitish red silty sand. Moderately calcareous	3	6'	JAR
5580	9.0		9.0'			
	10.0		Soft yellowish brown sandstone 10.0'			
			Bottom of test pit 10.0'			

DATE BEGAN: [illegible]
FINISHED: 1/15/81
GROUND SURFACE EL.: ~5594'

TEST PIT NO. 4-9

N ~[illegible]' E ~[illegible]'

FIELD ENGINEER: [illegible]
CHECKED BY: [illegible]

ELEV (FEET)	DEPTH (FEET)	DESCRIPTION	SAMPLE NO.	DEPTH	TYPE
	1.0	Medium stiff red sandy silt. Dry. Roots to depth — 1.7'			
	2.0–3.0	Dense red silty sand. Dry. White streaks. Slightly calcareous — 3.0'	1	2'	JAR
5590	4.0	Stiff redish white sandy silt. Dry. Very calcareous — 4.3'	2	4'	JAR
	5.0–6.0	Medium dense red silty sand. Moist. Some white streaks. Slightly calcareous — 6.0'	A 24% $CaCO_3$	4'-8'	BAG
5586	7.0–8.0	Dense redish white silty sand. Chalky. Very calcareous. Dry — 8.0'	3	7'	JAR
	9.0	Very soft yellowish brown sandstone. Weathered			
	10.0–11.0	Medium hard yellowish brown sandstone — 11.0'			
		Bottom of test pit 11.0'			

DATE BEGAN: [illegible]

DATE FINISHED: [illegible]

GROUND SURFACE EL: ~ 5577'

TEST PIT NO. D4-1

N ~[illegible] E~[illegible]

FIELD ENGINEER: [illegible]

CHECKED BY: [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	SAMPLE NO.	DEPTH	TYPE
	1.0		Stiff red sandy silt. Dry. Roots to depth — 1.3'			
	2.0		Dense red silty sand. Dry. — 1.9'			
5574	3.0		Medium dense red silty sand. Moist	1	2'4"	JAR
	4.0		— 4.3'	ST-1	3'-5'	3" Shelby
				2	4'2"	JAR
	5.0		Very soft greenish gray claystone. Moist			
	6.0					
5570	7.0		— 7.0'			
	8.0		Soft yellowish brown sandstone — 8.0'	3	7'2"	JAR
			Bottom of test pit 8.0'			

DATE BEGAN: [illegible]
DATE FINISHED: [illegible]
GROUND SURFACE EL.: ~ [illegible]

TEST PIT NO. [illegible]

N ~ [illegible] E ~ [illegible]

FIELD ENGINEER [illegible]
CHECKED BY [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	SAMPLE NO.	DEPTH	TYPE
	1.0		Medium stiff red sandy silt. Dry. Roots to depth			
5570	2.0		2.0'			
	3.0		Dense red silty sand. Dry. Streaks of white 3.0'	1	3'	JAR
	4.0		Medium dense red silty sand. Moist. Streaks of white			
	5.0					
5566	6.0			ST-1	5'-7'	3" Shelby
				2	6'	JAR
				ST-2	6'-8'	3" Shelby
	7.0		7.0'			
	8.0		Dense red silty sand. Dry	3	8'	JAR
	9.0		9.75'			
5562	10.0		Soft yellowish brown sandstone			
	11.0		11.5'			
			Bottom of test pit 11.5'			

DATE BEGAN: [illegible]

FINISHED: 1/15/81

GROUND SURFACE EL.: ~5578'

TEST PIT NO D4-3

N ~319,430' E~2,576,325'

FIELD ENGINEER [illegible]

CHECKED BY [illegible]

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS
	1.0		Medium stiff red sandy silt. Dry. Roots to depth 1.0'	
			Dense red silty sand. Dry. Some roots 1.7'	
	2.0, 3.0		Dense red silty sand. Dry. Streaks of white. Slightly calcareous 3.3'	
5574	4.0, 5.0, 6.0		Medium dense red silty sand. Moist 6.0'	
	7.0		Soft yellowish brown sandstone 7.0'	
			Bottom of test pit 7.0'	

SAMPLE NO.	DEPTH	TYPE
1	2'	JAR
ST-1	3'6"-5'6"	3"Shelby
2	5'	JAR

SECTION 6

TABLE C-1

SUMMARY OF LABORATORY TEST RESULTS

CELL 4

ble C-1

SUMMARY OF BORATORY TEST RESULTS

Cell 4

TECHNICIAN ______________________ PROJECT NO. ______________________

APPROVED BY ______________________ DATE ______________________

SAMPLE NO.	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO 200 SIEVE	ATTERBERG LIMITS		SOIL OR BEDROCK TYPE	COMMENTS
			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
TH 19 @ 0-3					70	23	6	Sandy clayey silt	
TH 23 @ 1-3					87	48	24	Weathered claystone	
TH 23 @ 6-8					96	61	30	Claystone	
TH 25 @ 1-3½	13.13				57	26	9	Sandy clay	
TH 26 @ 4½-5	15.3				91	41	20	Weathered claystone	
TH 28 @ 0-2	12.7				72	28	10	Sandy clay	
TH 29 @ 2-3	8.5				59	19	2	Sandy clayey silt	
TH 48 @ 5-5½					65	30	7	Sandy clay	
TH 49 @ 5-7					71	25	9	Sandy clay	
TH 49 @ 14-15					55	28	5	Calc. sandy silt	
TH 58 @ 5½-6	12.5				75	35	11	Sandy clay	
TH 76 2 0-1	4.5				78	21	5	Sandy silt	
TH 76 @ 9½-10	4.4				26	NP	--	Silty, gravelly sand	
TH 77 @ 7½-8	8.6				71	30	15	Sandy clay	
TH 79 @ 0-1	4.1				83	20	5	Sandy silt	
TH 79 @ 5-5½	5.5				41	NP	---	Calc. silty sand	