UMGRJ080A01 V02D004
PRELIMINARY FINAL VOL 4 OF 5
BOX:RU 16 DOC:08/01/1990
SURF000525

Uranium Mill Tailings Remedial Action Project (UMTRAP)

Grand Junction, Colorado

GRJ-PH-II

Calculations

Preliminary Final

Volume IV

August 1990

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

GJT 605.02.01

Um/ORJ 8.A.1
Vol 4

UMTRA PROJECT

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SHELF COPY

Attachment
Letter 91-3050-388
May 9, 1991

PHASE II CONSTRUCTION
GRAND JUNCTION
CALCULATION INDEX

Calculation No.	Title
	VOLUME I
05-622-01-00	Dewatering
05-626-01-03	Tailings Excavation - Tailings Pile Limits and Quantities
05-626-02-04	Tailings Excavation - Off-Pile Excavation Limits and Quantities
05-628-01-00	Site Drainage - Hydrology Parameters
05-631-01-01	Access Road - Culvert at Indian Creek
05-631-02-01	Access and Haul Roads - Culvert Placement and Protection at Indian Creek
05-631-03-01	Access and Haul Roads - Culvert Outlet Revision
	VOLUME II
05-633-01-01	Site Grading - Restoration Quantity for Grand Junction Processing Site
05-504-01-02	Erosion Protection - Top and Sideslopes of Tailings Embankment
05-504-02-00	Erosion Protection - Time of Concentration, Cheney Disposal Site Embankment
05-504-07-00	Permanent Site Drainage - Off-Pile Drainage Swale
05-504-04-00	Disposal Site Drainage During Construction - Retention Basin and Emergency Spillway
05-504-05-02	Riprap Toe Protection

PHASE II CONSTRUCTION
GRAND JUNCTION
CALCULATION INDEX

Calculation No.	Title
	VOLUME III
05-505-02-02	Rock Quality for the Erosion Protection - Cheney Disposal Site
05-505-03-02	Availability and Suitability of Materials
05-654-01-02	Hydrogeology - Dewatering
05-654-02-00	Slurry Trench at Processing Site - Quantities
05-654-03-00	Hydrogeology - Slurry Trench Seepage Windows
05-655-01-00	Surface Water Runoff Accumulation and Discharge
05-666-01-02	Construction Sequence
05-667-04-02	Quantity Estimate Summary - Phase II Construction
05-670-01-05	Radon Barrier Design - Thickness
05-670-02-03	Radon Barrier Design - Average Ra-226 Concentrations
	VOLUME IV
05-670-05-03	Embankment Design - Material Properties
05-670-06-02	Embankment Design - Settlement and Cover Cracking
	VOLUME V
05-670-07-05	Embankment Design - Slope Stability
05-670-08-01	Embankment Design - Drain Layer/Bedding Layer
05-670-09-02	Embankment Design - Depth of Frost Penetration
05-670-11-00	Radon Barrier - Ra-226 Concentrations in DOE Compound
05-670-12-00	Embankment Design - Paleochannel Remediation
05-670-13-00	Embankment Design - Infiltration

MKE DOCUMENT NO. 5025-GRJ-C-01-01013-02

Calculation Cover Sheet

Contract No. 5025 Discipline ESCI Calc. No. 05-670-05-00 03 No. of Sheets 300

Project: UMTRA - Grand Junction

Feature: Embankment Design

Item: Material Properties

Sources of Data: See Attached Sheets

Sources of Formulae & References: See Attached Sheets

Preliminary Calc. ☐ Final Calc. ☒ Supersedes Calc. No. 05-670-05-00

Rev. No.	Revision	Calculation By	Date	Checked By	Date	Approved By	Date
3	ADDITIONAL DATA	H. LUBIS	6/21/90	S.E. Botsford	6-25-90	P. Sircar	6-27-90
2	Additional Data	S.E. Botsford	2/9/89	RM Brady	2-9-89	P. Sircar	2/17/89
1	Updated Soils Data	P. Sircar	2/4/88	WYLin	2/5/88	WYL for JL	2/17/88
0		P. Sircar	7-13-87	S.E. Botsford	7/13/87	[illegible]	7/13/87

Form 42-110

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project: UMTRA - GRJ	Contract No. 5025-16	Sheet	
Feature: EMBANKMENT DESIGN	Designed H.L.	File No.	
Item: MATERIAL Properties	Checked SHB	Date 6/19/90	Date 6-19-90

REVISION 3

Purpose

The purposes of Revision 3 are:

1. To design the new tailings embankment. The old embankment was moved to the western end of the site

2. To include the latest material properties tests in the design calculation.

3. To include the required data of clean-fill dike. It is planned to place 2 different clean-fill dikes on the side slope, such as:
   a. Below Existing Ground (Compacted Mancos Shale)
   b. Above Existing Ground (-1" Material)

REVISION 2

Revision 2 was necessitated by the following:

1. A vegetative cover was planned for the top slope (see Reference 13). This required data for routing soil.

2. The primary radon barrier material was to be from the Layer 3 foundation soils, and not the Layer 1 foundation soils as previously planned.

Project	UMTRA-GRJ	Contract No. 5025	File No.
Feature	Embankment Design	Designed PS	Date 7-9-87
Item	Soil Properties	Checked SHB	Date 7/13/87

References

1. U.S. Dept of Energy, "Remedial Action Plan and Site Conceptual Design for Stabilization of the Inactive Uranium Mill Tailings at Grand Junction, Colorado," Draft, June 1986
   - MKE Doc. No. 5025-GRJ-R-03-00226-00 (Draft RAP, Vol. 1)

2. Nelson, J.D. and Wardwell, R.E., "Characterization of Inactive Uranium Mill Tailings Sites: Grand Junction, Colorado," prepared for DOE, Albequerque, New Mexico by Colorado State University, Fort Collins, Colorado, July 1980
   - MKE Doc. No. 5025-GRJ-R-03-00555-00

3. UMTRA-GRJ, "Evaluation of Borrow Material at Cheney Reservoir Site", IOC from R.W. Hendrks to T.R. Walker,
   - MKE Doc. No. 5025-GRJ-I-01-00111

4. Grand Junction Soil Tests - IOC from T.G. Myers of MKF to T.R. Walker of MKE, dated October 16, 1986.
   - MKE Doc. No. 5025-GRJ-I-02-00355-00
   (Chen's results on radon barrier are enclosed

5. UMTRA Design Procedures Manual
   - MKE Doc. No. 4005-GEN-Q-01-00571-02

6. Report from Rogers & Associates, Salt Lake City, Utah, dated December 16, 1986 (for radon barrier diffusion coefficients
   - MKE Document No. 5025-GRJ-L-09-00436-00

7. "Radon Barrier Design — Thickness",
— MKE Calc. No. 05-670-01-~~00~~03

8. "Embankment Design — Settlement and Cover Cracking"
— MKE Calc. No. 05-670-~~03~~-00
06-01
Rev 3. P.S. 6-26-
H.L. 6/26/

9. "Embankment Design — Slope Stability"
— MKE Calc. No. 05-670-04-00

10. "Erosion Protection, Bedding and Radon Barrier — Availability and Suitability" 05-505-03-00
— MKE Calc. No. ~~05-118-03-00~~
H.L. 6/26/00
P.S. 6/26/00

11. Colorado State University Report
(MKE Doc. No. 5025-GRJ-L-09-00052-00)

12. UMTRA Design Procedures Manual, REV. 7, JAN. 1989, MKE DOC. 4005-GEN-Q-01-06596-00

13. "Embankment Cover Design — Vegetative Cover"
MKE Calc. No 05-670-10-00

14. BOWLES, J.E., "FOUNDATION ANALYSIS AND DESIGN", 3RD EDITION, Mc. GRAW HILL COMPANY, 1982

15. WINTERKORN, H.F., FANG, H.Y., "FOUNDATION ENGINEERING HANDBOOK", VAN NOSTRAND REINHOLD COMPANY

16. NAVFAC DM-7.1, "SOIL MECHANICS — DESIGN MANUAL 7.1", DEPARTMENT OF THE NAVY NAVAL FACILITIES ENGINEERING COMMAND, MAY 1982

REFERENCES CONT'D

17. Design Standards Embankment Dams No. 13, Chapter 4 Static Stability Analysis, USBR, Engineering Research Center, Denver, Colorado, Aug. 1987

18. Rock Quality for Erosion Protection - Cheney Disposal Site, MKE Calc. No. 05-505-02-00

19. Embankment Design - Drain layer, MKE Calc. No. 05-670-09-01

20. Laboratory Analysis of Soil Hydraulic Properties of the GRJ main tailings and Vicinity Properties, Deliv. Order 005, by DBS&A, MKE DOC. 3885-GRJ-R-02-04329-00

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I Purposes

The purposes of these computations are to establish pertinent geotechnical material properties for use in the design of the tailings embankment at the Cheney Reservoir Disposal Site. Thereby, computations could be made to assess the stability and deformation of the tailings embankment. The properties of the following materials will be required:

- Properties for the relocated tailings from the Processing site, which will form the embankment (Ref 1,2)
- Properties for the proposed radon barrier and erosion protection material (References 4, 6, 7, 8)
- Properties for the in-situ foundation soils at the Cheney Reservoir Disposal Site (Ref 1,3)
- Properties for Rooting Soil SHS 2-17-89

The design evaluation process will involve assessment of

- Stability of the embankment-foundation system under static and seismic loading conditions (Calc. No. 05-670-04-00)
- Settlement of the embankment and potential for cracking of the radon barrier (Calc. No. 05-670-03-00)
- Adequacy/effectiveness of the radon barrier and the erosion protection material to perform their desired functions (See Calc. Nos. 05-670-01-~~00~~ 03 and ~~05-659-01-01~~ 05-505-03-00)

REV. 3 H.L. 6/26/91
P.S. 6/26/91

II Evaluation and Summary

Contaminated uranium mine tailings will be excavated from the main pile, including under the pile, the northeast storage area, the mill buildings settling Ponds 1, 2, and 3 and the vicinity properties. Estimates in RAP indicate an approximate 4.1 million cubic yards (including 0.39 million cy of vicinity properties) of contaminated material including 1.2 million cy of vicinity properties to be relocated at the Cheney Reservoir disposal site

All the contaminated material will be disposed of as a tailings embankment in the Cheney Reservoir disposal site Present plans call for the an excavation of approximately 10 feet in depth over a 60 acre area and to be overlain by the tailings from all sources. The tailings embankment will we 60 feet high and with the 'usual' 5:1 slopes. The areas from which contaminated soils are excavated wil be restored with clean soil to adjacent grade.

Existing data from draft RAP and other sources (see References) were reviewed and re-evaluated if necessary, as presented in the following pages. The data includes soil borings, soil test results, water level data etc. These data have been utilized to develop the pertinent design parameters, which are summarized in Sheets 3.

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project: UMTRA - GRAND JUNCTION
Feature: EMBANKMENT DESIGN
Item: MATERIAL PROPERTIES

Contract No. 5025-16
Designed H. L.
Checked P. S.

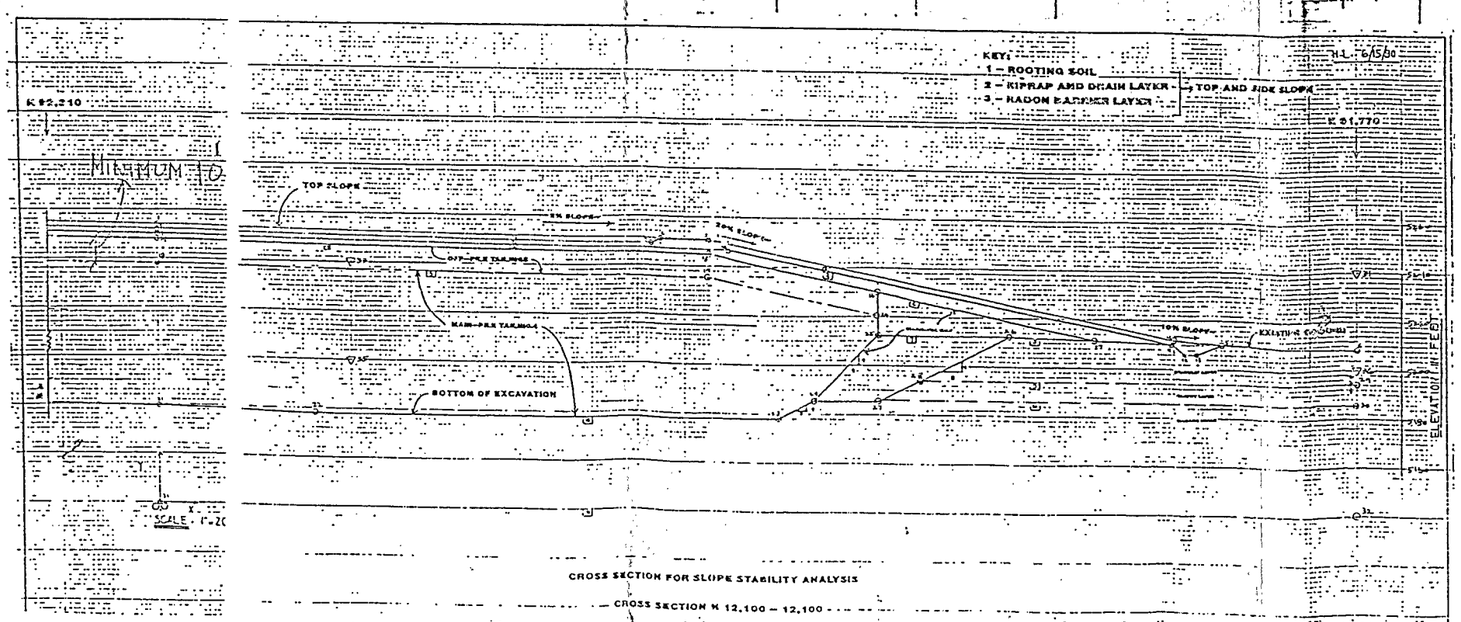
Sheet 3
File No.
Date 6/20/90
Date 6/27/90

REV. 3

SUMMARY OF DESIGN PARAMETERS

LAYER No	DESCRIPTION	IN-PLACE DRY DENSITY γ_d, PCF	IN-PLACE DENSITY γ_t, PCF	IN-PLACE MOIST. W, %	SATURATED DENSITY γ_{SAT}	SATURATED MOIST. W_{SAT}	SHORT TERM STATIC C, PSF	SHORT TERM SEISMIC Ø, DEGREE	LONG TERM STATIC C, PSF	LONG TERM STATIC Ø, DEGREE	LONG TERM SEISMIC C, PSF	LONG TERM SEISMIC Ø, DEGREE	CONSOLIDATION P_p, KSF	C_v, ft²/hr	C_c	C_R	e	PERMEAB k, cm/sec
1	ROOTING SOIL	84.8	95.0	12.0	—	—	S_u = 250 psf		0	20°	S_u = 250 psf		—	—	—	—	—	—
2	RIPRAP | DRAIN LAYER	125.0	125.0	—	125.0	0	0	38°	0	38°	0	38°	—	—	—	—	—	—
3	RADON BARRIER 100% COMPACTION	113.7	130.6	14.9	135.2	18.9	S_u = 750 psf		280	28°	S_u = 750 psf		2.2	0.026	0.178	—	0.526	5.02×10^{-8}
4	OFF-PILE TAILINGS 95% COMPACT.	111.0	124.8	12.4	132.1	19.0	S_u = 4000 psf*		0*	30°	0	18.5°	2.2	2.3×10^{-5}	0.23	0.007	0.437	$2.0\times10^{-}$
5	MAIN-PILE TAILINGS 90% COMPACT.	86.5	105.5	22	117.2	35.5	0 18.5°	25°	0	30°	0	18.5°	1.0	1.0×10^{-3}	0.2	0.02	0.97	$7.0\times10^{-}$
6	CLEAN-FILL DIKE ABOVE GRND.	116.1	130.3	12.3	137.0	18.0	0	25	0	31°	0	25°	—	—	—	—	—	—
7	CLEAN-FILL DIKE BELOW GRND.	103.5	122.6	18.5	128.3	23.9	175	0	0	20°	0	20°	—	—	—	—	0.328	
8	LAYER 1 TOP SOIL	83.4	95	12	115.4	38.0	S_u = 500 psf		0	30°	S_u = 500		—	—	—	—	1.050	—
9	LAYER 2 GRAVELLY LAYER	130	130	7.0	142	9.5	0	35	0	35°	0	35°	—	—	—	—	—	7.0×10^{-5}
10	LAYER 3 CLAYEY LAYER	107.4	117.3	9.2	130	21	S_u = 1500 psf		0	29°	0	10°	6.0	2.3×10^{-5}	0.116	0.015	0.568	5.7×10^{-5} TO 4.4×10^{-6}
11	LAYER 4 MANCOS SHALE	133.7	145.0	10.3	145.0	10.3	S_u = 5000 psf		0	35°	S_u = 5000 psf						0.293	Approx. 1×10^{-7}
12	LAYER 5 GENERAL FILL	107.2	120.7	18.5	129.8	21	280	28°	280	28°	280	28°	—	—	—	—	0.572	1.4×10^{-6} TO 5.6×10^{-7}

* For off-pile tailings greater than 12.5 thickness on the top layer, use C=0, Ø=18.5° for short term static/seismic



CROSS SECTION FOR SLOPE STABILITY ANALYSIS

CROSS SECTION K 12,100 – 12,100

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project _____ Contract No. _____ Sheet _____ File No. _____
Feature _____ Designed _____ Date _____
Item _____ Checked _____ Date _____

GRAND JUNCTION PROCESSING SITE

Introduction:

Contaminated materials primarily consist of 2 categories

A. Materials from Main Pile, including tailings, sub pile material
Extensive data exists for this material (some are presented as part of this evaluation). All test results are summarized in Sheets 24 and 25.

- Boring logs
- Index Tests, including in-place and compaction densities and moisture contents, grain size distributions and Atterberg limits. These tests are summarized in
- Strength tests including direct shear and consolidated undrained triaxial tests.
- Consolidation Tests
- Permeability tests

B. Materials from outside the Main Pile, including material from settling ponds and vicinity property. Data base on these materials is limited to boring logs from the 3 pond areas, reported by Bendix Corporation (Doc. No. 5025-GRJ-R-09-00355-00, Appendix G-microfiche) and to some laboratory testing on vicinity properties performed by Western Engineering recently. Additional Data were gathered which are discussed in Sheets 71-94.

Rev. 3 H.L. 6/26/90 P.S. 6/26/90

Tailings Materials To Be Placed at the Disposal Site

Contaminated tailings materials from the processing site and vicinity will be excavated and transported to the Cheney Reservoir Disposal site where an embankment will be built of the excavated contaminated material. There is a distinct level difference in radiation between the main pile and offsite areas. Thus, as presently planned, the offsite material will be placed nearer the top of the embankment to reduce the thickness of the radon barrier.

The major portion of the available soils data is concentrated at the main pile location. The available soils data from other locations is limited to

4 borings in Pond 1 area
3 borings in Pond 2 area
8 borings in Pond 3 area
} done by Bendix

These borings in the pond areas (5025-GRJ-R-09-00355-00) indicate the soils in the upper 10-15 feet to be clays (CL and CH), and thus, probably more closely related to slimes than to sands or very sandy sand-slimes in the main pile area. Also additional data, discussed in Sheets 71-94. Note: Offsite materials shall occupy a minimum of 10 feet of the top of the embankment.

REV. 3 H.L. 6/26/90
P.S. 6/26/90

My analysis of the sections (Sheets 7 through 23) in the light of the boring logs within the main pile area indicates the following:

1. There appears to be a predominance of 'sand' as opposed to a 'sand-slime' in the tailings pile. Thus, the mixture in the resultant embankment will likely be between a 'sand' and a 'sand-slime'.

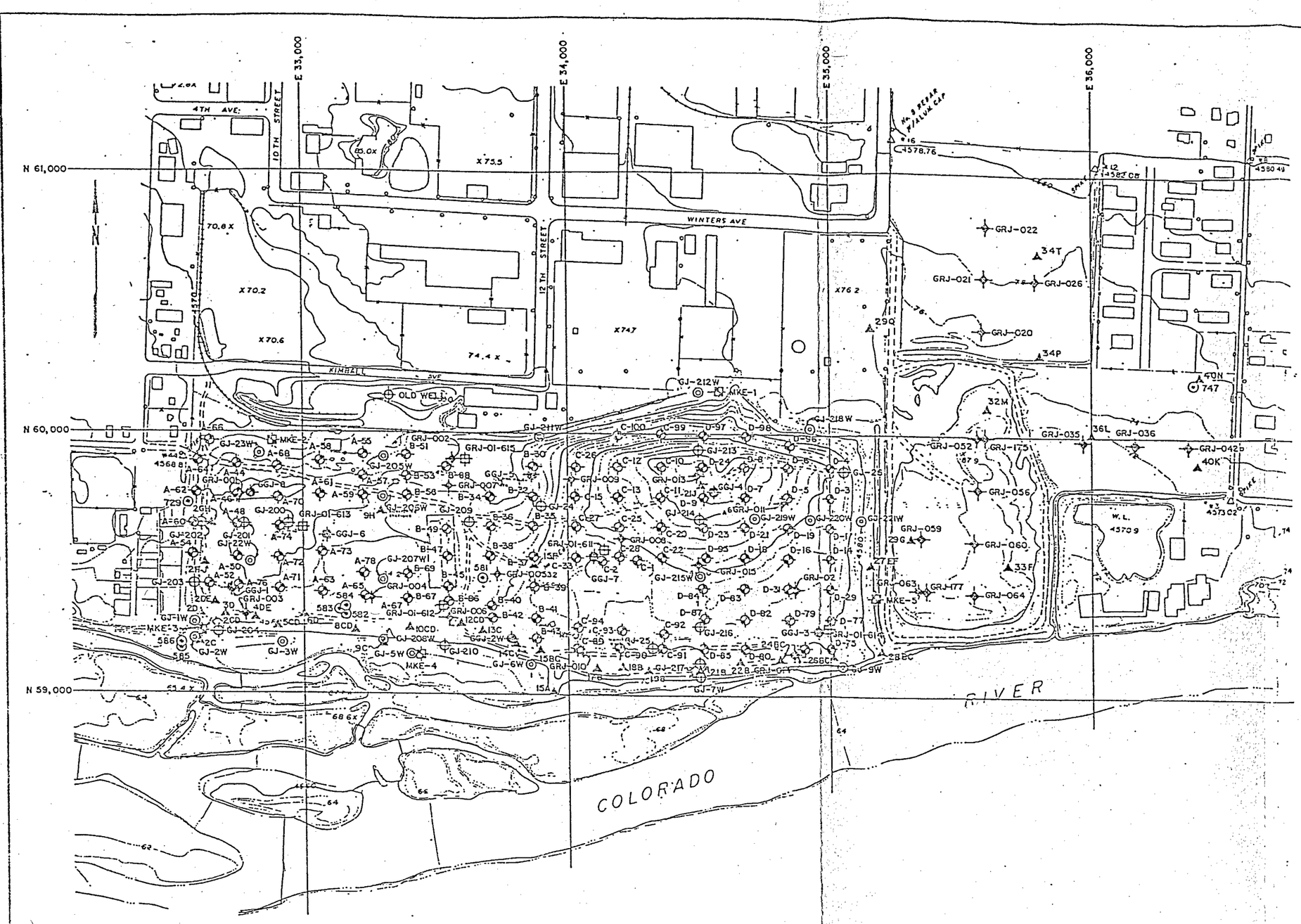
2. There is some correlation between the blowcounts and the type of material, particularly the presence of slimes. Blow counts less than 13, and definitely less than 10 and under are mostly slimes.

3. An approximately 700' x 400' area at the bottom of the pile in the southeast and appears to be slimes. The slimes are usually soft and nearly saturated due to the proximity to groundwater.

4. The ratio in volume is about 10 to 1 of sand and sand-slime tailings to slime tailings. The estimated volume of slime tailings at the processing site is about 0.26 million cubic yards

Area of Soft slimes at Bottom of Tailings pile
Area of slimes Tailings thicker than 5 feet
KIMBALL AVE
12TH STREET
N
SCALE
FEET
Boring Location Plan
UMTRA/GRJ
by MDL 7/1/86
ck'd WYL 2/4/88

PS 2/5/88
WYL 2/5/88
7a



NOTES:

1. GROUND WATER DATA AND SUBSURFACE EXPLORATION LOGS ARE AVAILABLE FROM THE CONTRACTOR.
2. JACOBS MONITOR WELLS NUMBERED 581 THROUGH 585 CORRESPOND TO FIELD DESIGNATION GWGJ-1 THROUGH GWGJ-6 RESPECTIVELY.
3. EXISTING CONTOURS AND FEATURES SHOWN ARE THOSE WHICH EXISTED PRIOR TO PHASE I-A CONSTRUCTION.

REFERENCE DRAWINGS:

GRJ-PS-10-0208	CONSTRUCTION FACILITIES AND SITE DRAINAGE
GRJ-PS-10-0209	SLURRY TRENCH PLAN
GRJ-PS-10-0211	CONTAMINATED MATERIAL EXCAVATION PLAN SHEET 1 OF 2
GRJ-PS-10-0212	CONTAMINATED MATERIAL EXCAVATION PLAN SHEET 2 OF 2

LEGEND:

4580	EXISTING SITE FEATURES & CONTOURS
N61,000	CONSTRUCTION GRID COORDINATE
GJ-200	AUGER BORINGS DRILLED BY COLORADO STATE UNIVERSITY (1980)
A-66, B-51, C-26, D-27	AUGER BORINGS DRILLED BY SERGENT, HAUSKINS & BECKWITH (1981)
GGJ-1	AUGER BORINGS DRILLED IN 1982, MONITORING WELLS 1982.
GJ-205W	MONITOR WELLS INSTALLED BY COLORADO STATE UNIVERSITY (1980)
724	MONITOR WELL INSTALLED BY JACOBS ENGINEERING GROUP, INC. (1985)
GRJ-01-611	TEST PITS DUG BY JACOBS ENGINEERING GROUP, INC. (1985)
MKE-1	TEST PITS DUG BY MORRISON-KNUDSEN ENGINEERS, INC. (1985)
GRJ-001	AUGER BORINGS DRILLED BY JACOBS ENGINEERING GROUP (1984)
	OLD WELL WEST OF MILL
2C	AUGER BORINGS DRILLED BY LINCOLN DEVORE (1987)

SCALE 200 0 200 400 FEET

FINAL REVIEW		
E&D MANAGER	CHIEF ENGINEER	QA MANAGER

NO.	DATE	REVISIONS	BY	CK	E&D MGR	CHIEF ENG	FAC REV	DOE APP
0		ISSUED FOR FINAL REVIEW	–	–	–	–		

U. S. DEPARTMENT OF ENERGY
ALBUQUERQUE, NEW MEXICO

DESIGNED | DRAWN HBC | CHECKED | INSPECTED | RECOMMENDED

GRAND JUNCTION PROCESSING SITE
GRAND JUNCTION, COLORADO
PHASE II CONSTRUCTION

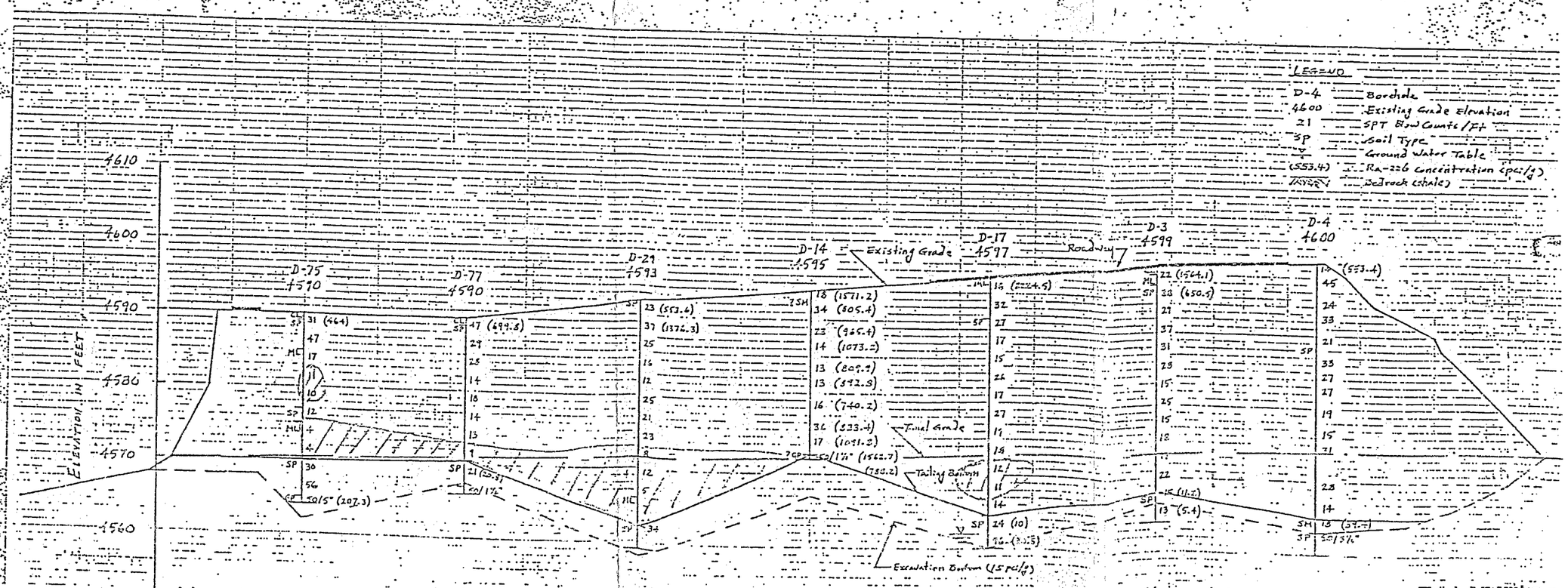
BORINGS AND TEST PITS LOCATION PLAN

APPROVED | DATE | DOE PROJECT ENGINEER | DATE

MORRISON-KNUDSEN ENGINEERS, INC.
ULTRA PROJECT
180 MONTGOMERY ST. SAN FRANCISCO CA 94105

PROJECT NO. DE-AC04-83AL18796
DRAWING NO. GRJ-PS-10-0217
REV. 8

P.S 2/4/88
WYL 2/16/88
Slimes



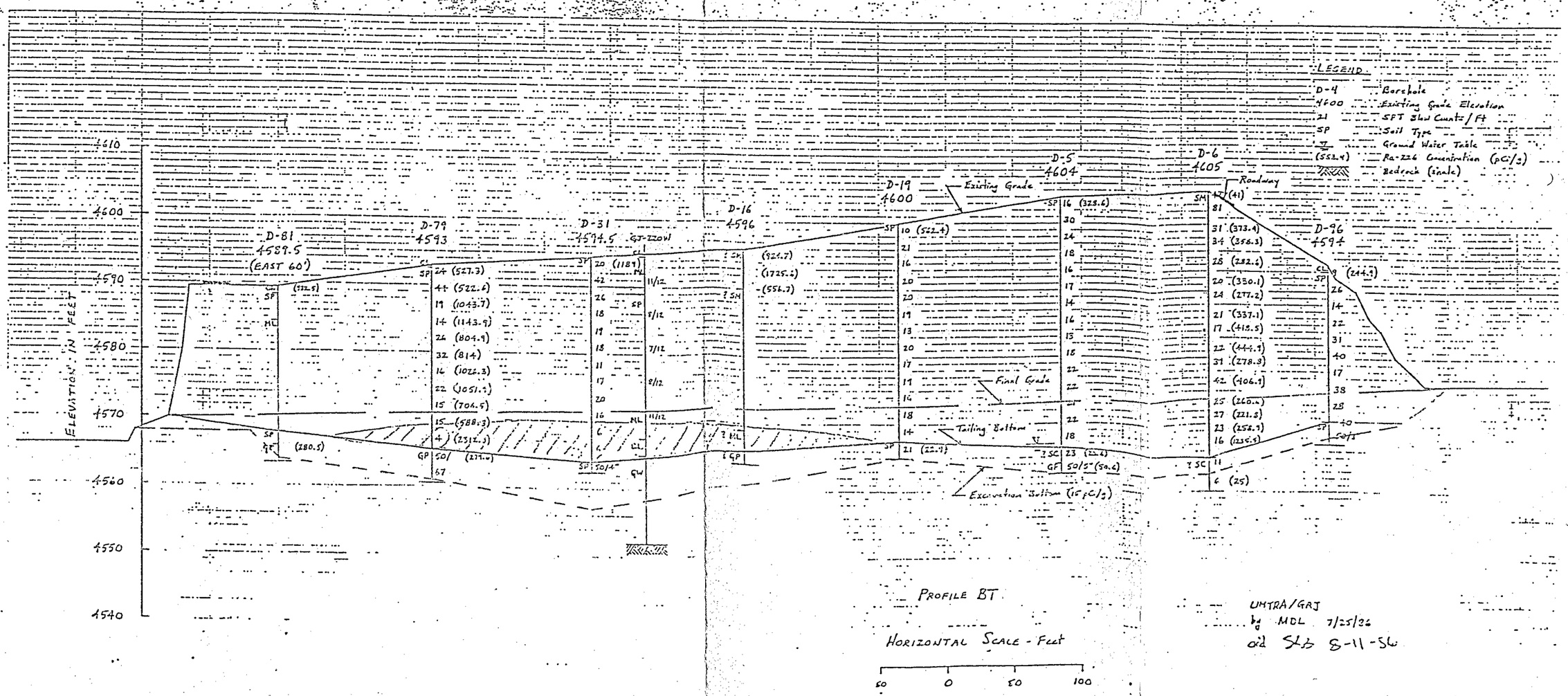
PROFILE AT

HORIZONTAL SCALE - Feet

50 0 50 100

UMTRA/GRJ
by MDL 7/25/86
ckd GFS 8-11-86

P.S 2/4/88



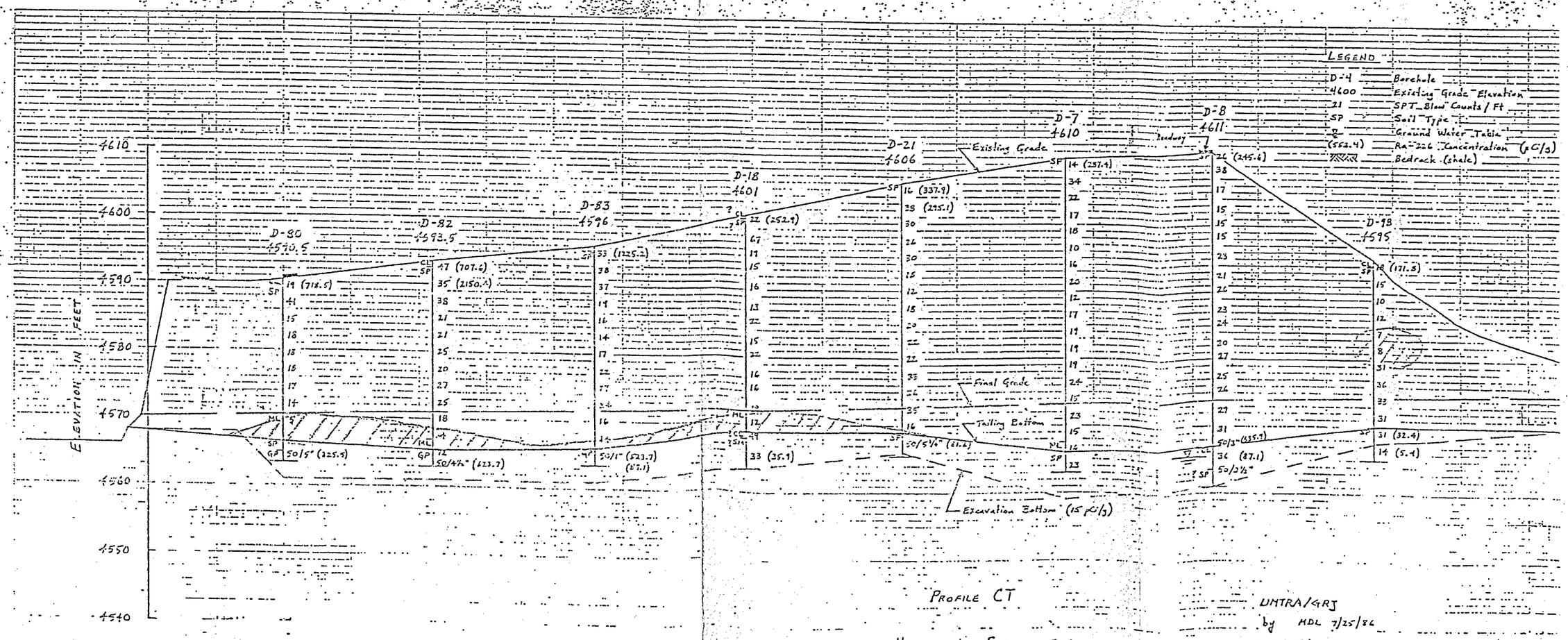
PROFILE BT

HORIZONTAL SCALE - Feet

50 0 50 100

UMTRA/GRJ
by MDL 7/25/86
old SLB 8-11-86

PS 2/4/88

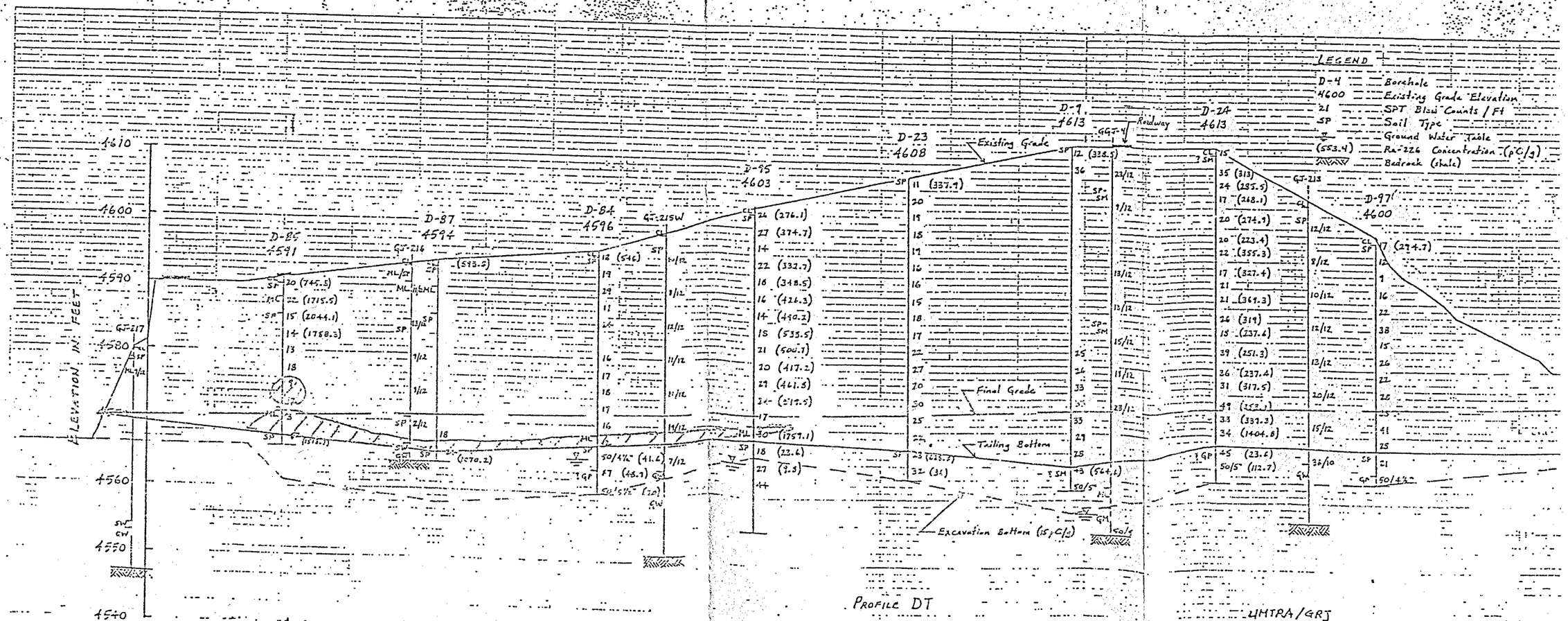


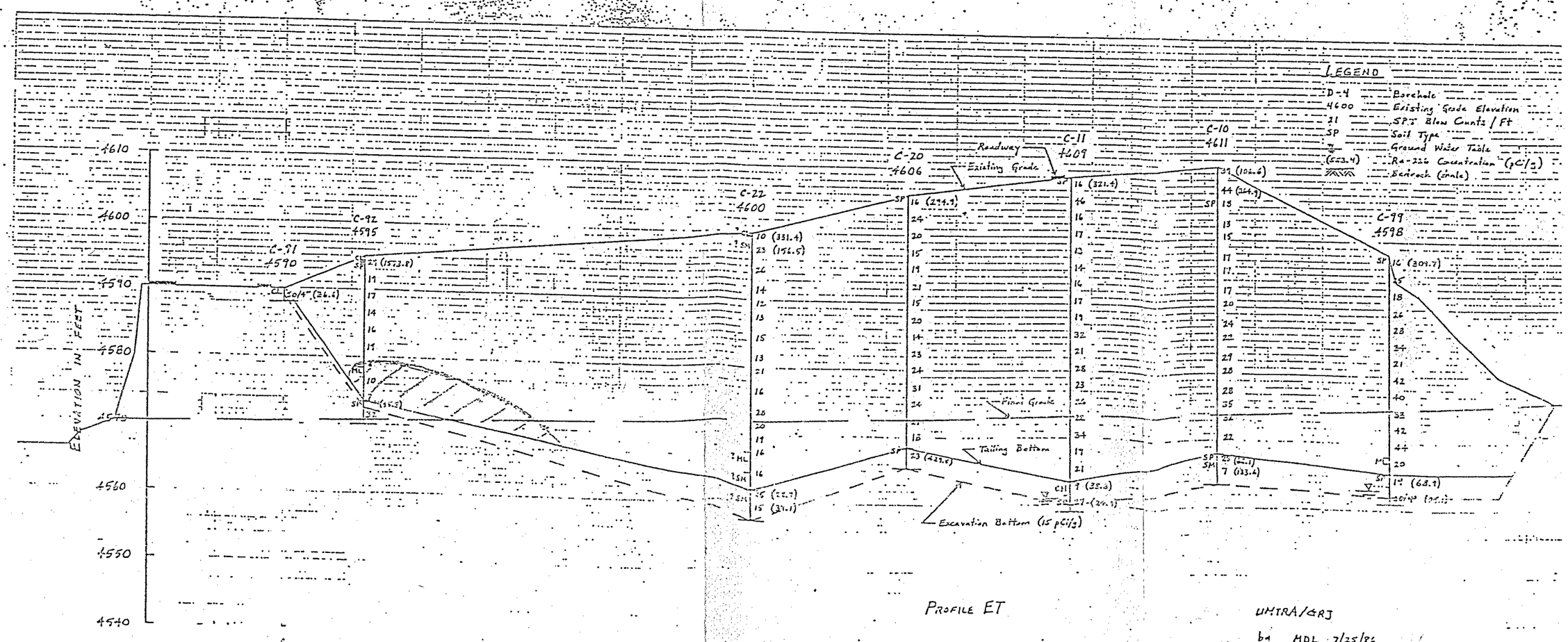
PROFILE CT

HORIZONTAL SCALE - Feet

50 0 50 100

UMTRA/GRJ
by MDL 7/25/86
CKD SJS 5-11-86


LEGEND
D-4 Borehole
4600 Existing Grade Elevation
21 SPT Blow Counts / Ft
SP Soil Type
Ground Water Table
(553.4) Ra-226 Concentration (pCi/g)
Bedrock (shale)
ELEVATION IN FEET
4610
4600
4590
4580
4570
4560
4550
4540
D-85
4591
D-87
4594
D-84
4596
D-95
4603
D-23
4608
D-7
4613
D-24
4613
D-97
4600
Existing Grade
Final Grade
Tailing Bottom
Excavation Bottom (15 pCi/g)
Roadway
GJ-215W
GJ-216
GJ-217
GJ-213
GGJ-4
(545.2)
(570.2)
(1715.5)
(2044.1)
(1768.3)
(1757.1)
(564.1)
(1404.8)
(112.7)
(41.4)
(48.7)
(23.4)
(7.5)
50/4½"
50/5"
50/1½"
50/4
50/5
PROFILE DT
HORIZONTAL SCALE - FEET
50 0 50 100
UMTRA/GRJ
by MDL 7/25/86
ckd SJS 5-11-86



PROFILE ET

HORIZONTAL SCALE - Feet

50 0 50 100

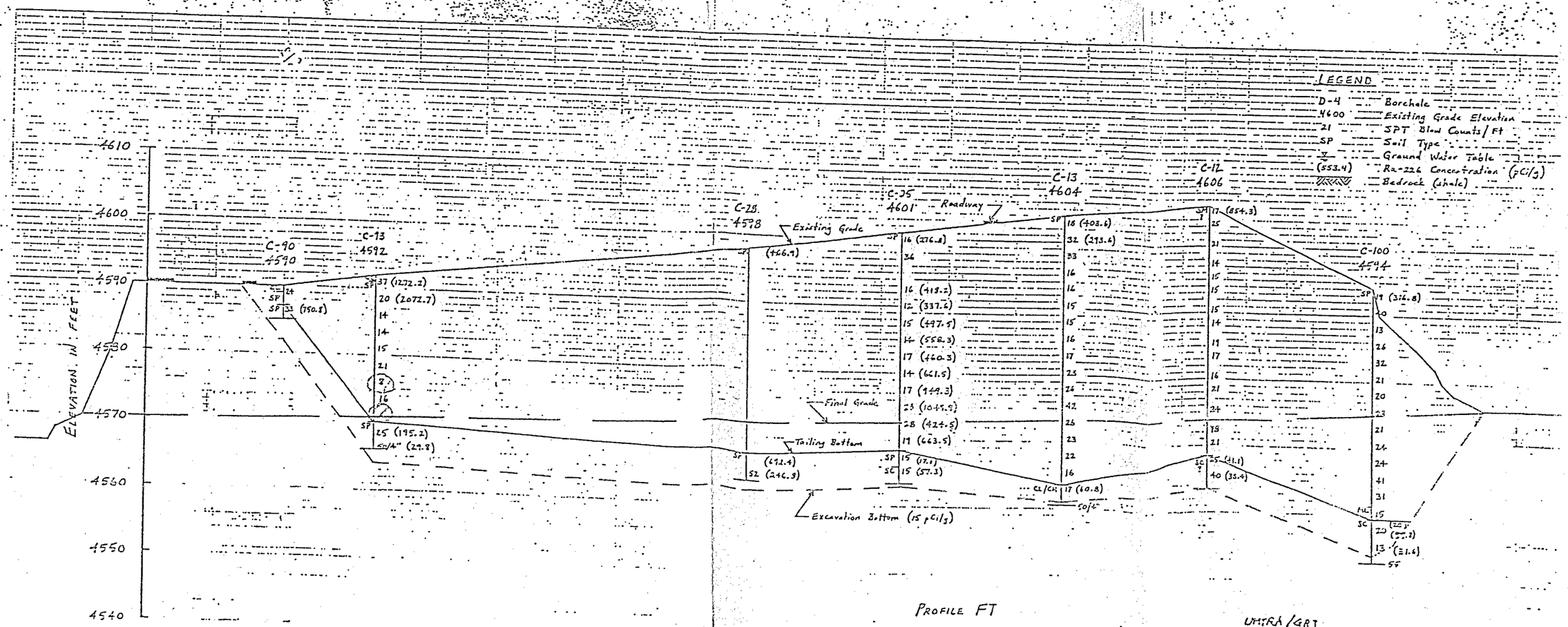
UMTRA/GRJ

by MDL 7/25/86

ck'd 3-11 SL

PS 2/4/88

PS 2/4/88



PROFILE FT

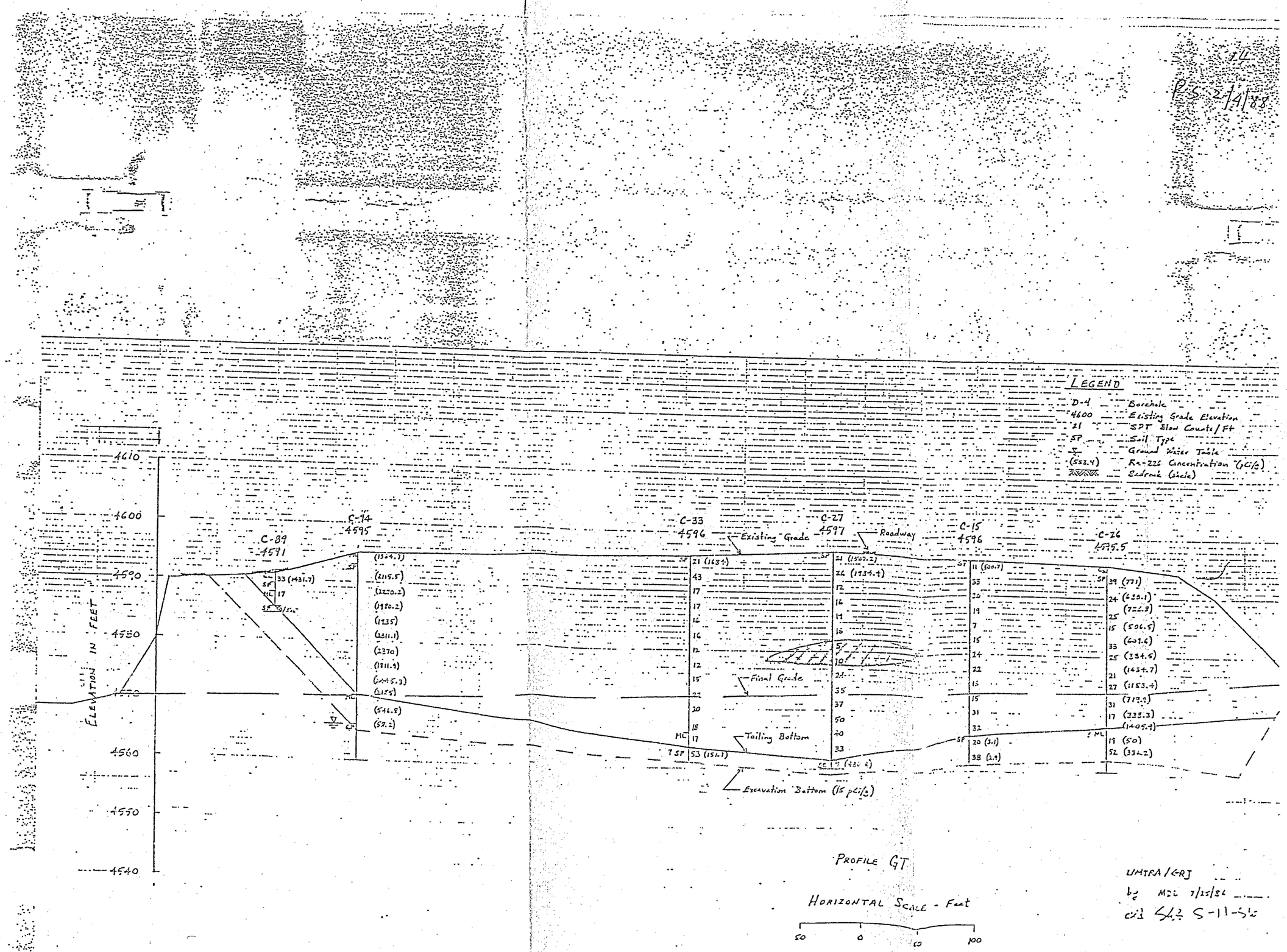
HORIZONTAL SCALE - Feet

50 0 50 100

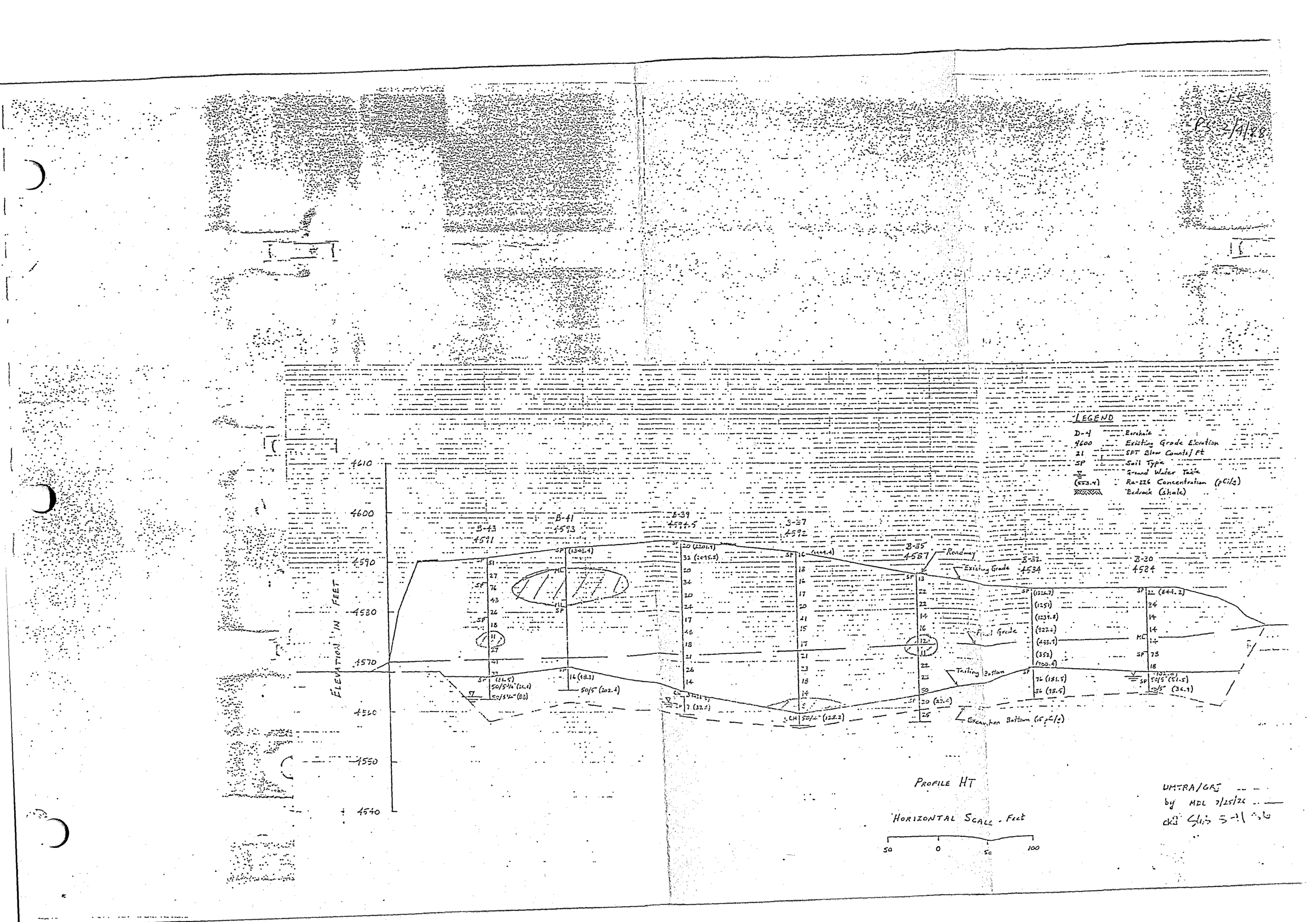
UMTRA/GRJ
by MDL 7/25/86
ckd 5-11-86

44

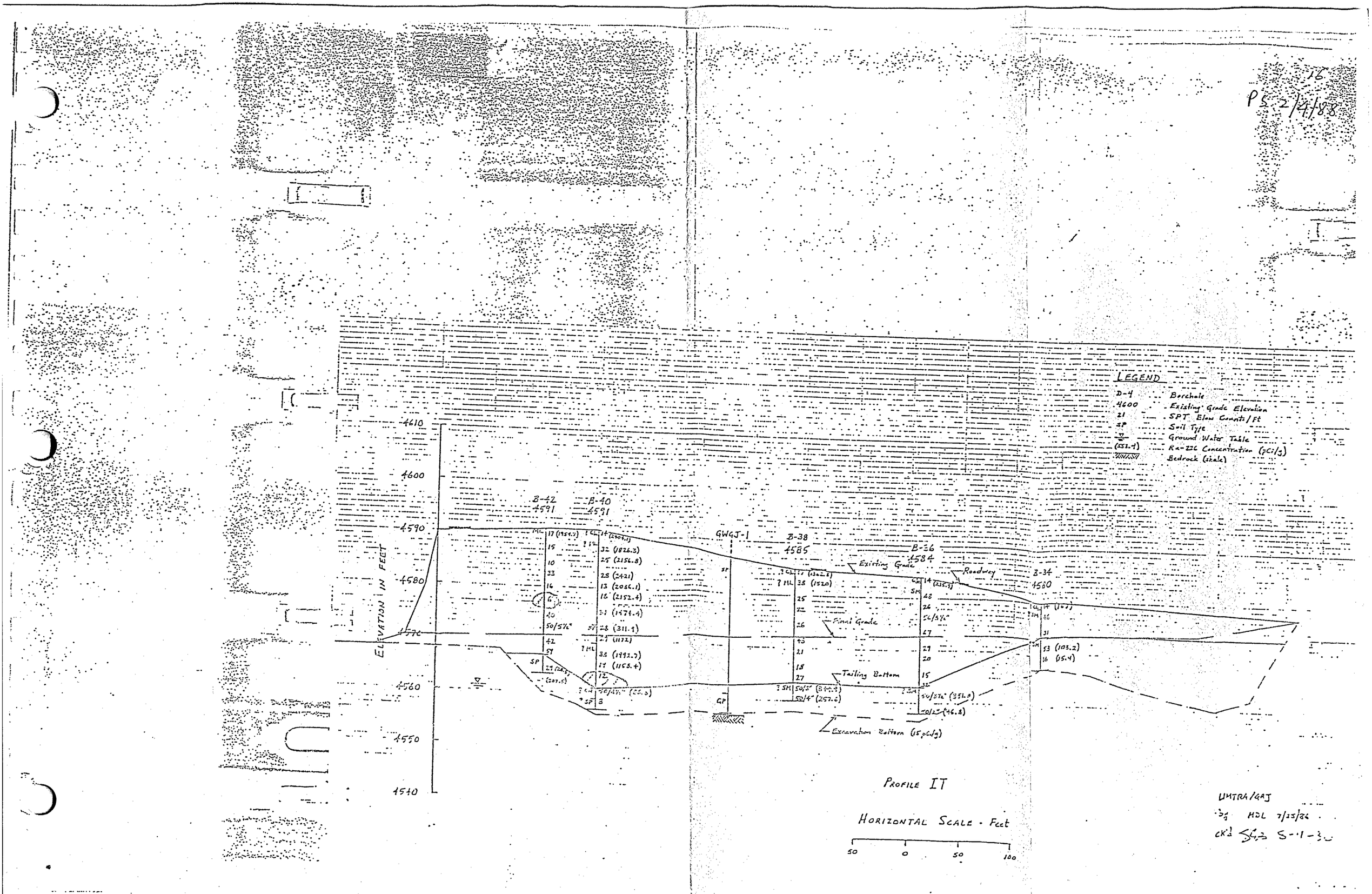
PS 2/19/88



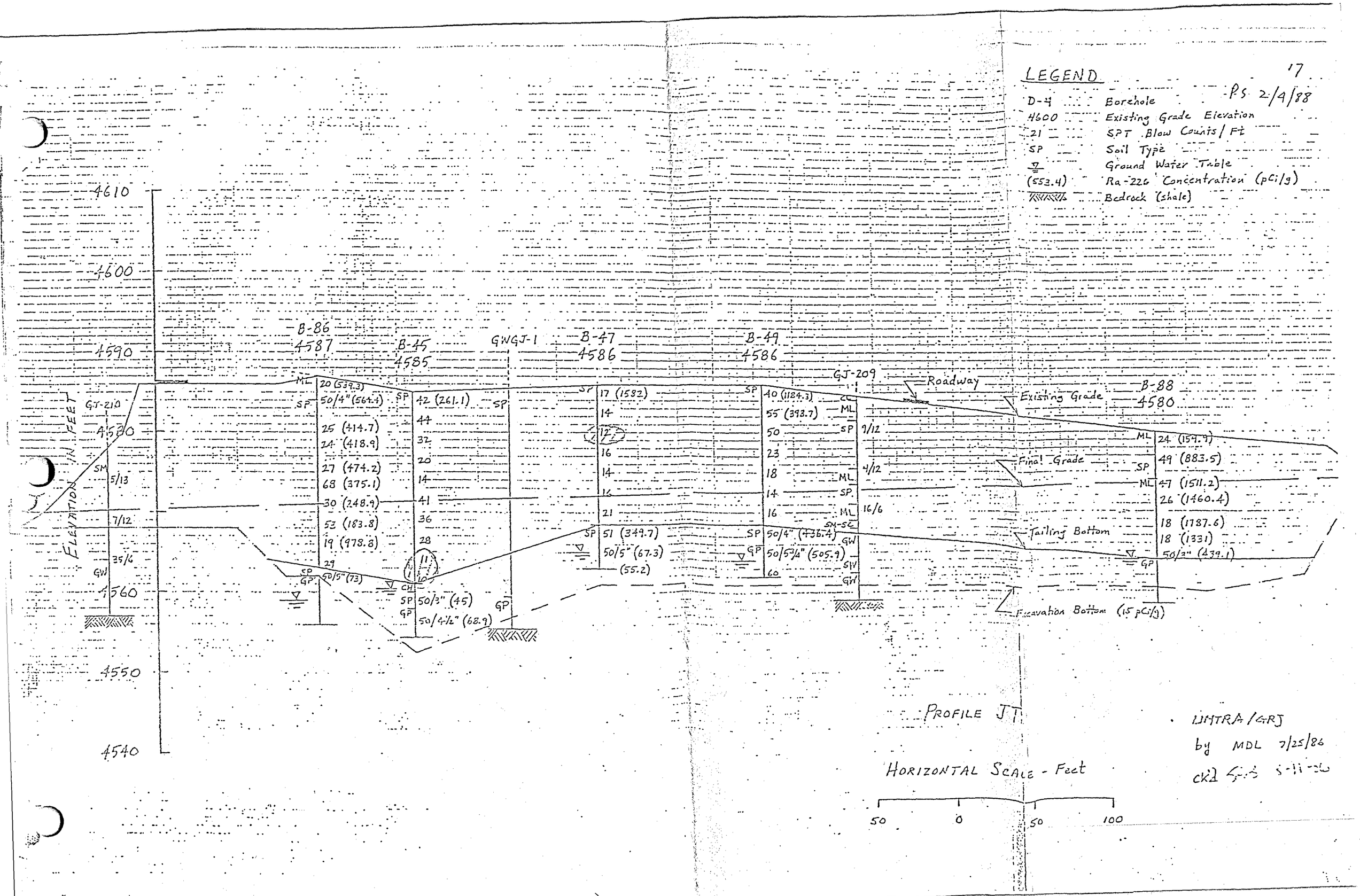
UMTRA/GRJ
by MJL 7/25/86
ckd SLZ 5-11-86


LEGEND
D-4 Borehole
4600 Existing Grade Elevation
21 SPT Blow Counts/Ft
SP Soil Type
Ground Water Table
(553.4) Ra-226 Concentration (pCi/g)
Bedrock (shale)
ELEVATION IN FEET
4610
4600
4590
4580
4570
4560
4550
4540
B-43
4591
B-41
4593
B-39
4594.5
B-37
4592
B-35
4587
B-32
4584
B-30
4584
Roadway
Existing Grade
Final Grade
Tailing Bottom
Excavation Bottom (16 pCi/g)
PROFILE HT
HORIZONTAL SCALE - Feet
50
0
50
100
UMTRA/GRS
by MDL 7/25/86
ck'd

16
PS 2/4/88


LEGEND
D-4 Borehole
4600 Existing Grade Elevation
21 SPT Blow Counts/Ft
SP Soil Type
Ground Water Table
(153.4) Ra-226 Concentration (pCi/g)
Bedrock (shale)
ELEVATION IN FEET
4610
4600
4590
4580
4570
4560
4550
4540
B-42
4591
B-40
4591
GWGJ-1
B-38
4585
B-36
4584
B-34
4580
Existing Grade
Roadway
Final Grade
Tailing Bottom
Excavation Bottom (15 pCi/g)
PROFILE IT
HORIZONTAL SCALE - Feet
50
0
50
100
UMTRA/GAJ
HDL 7/25/86
CK'd

P.S. 2/4/88

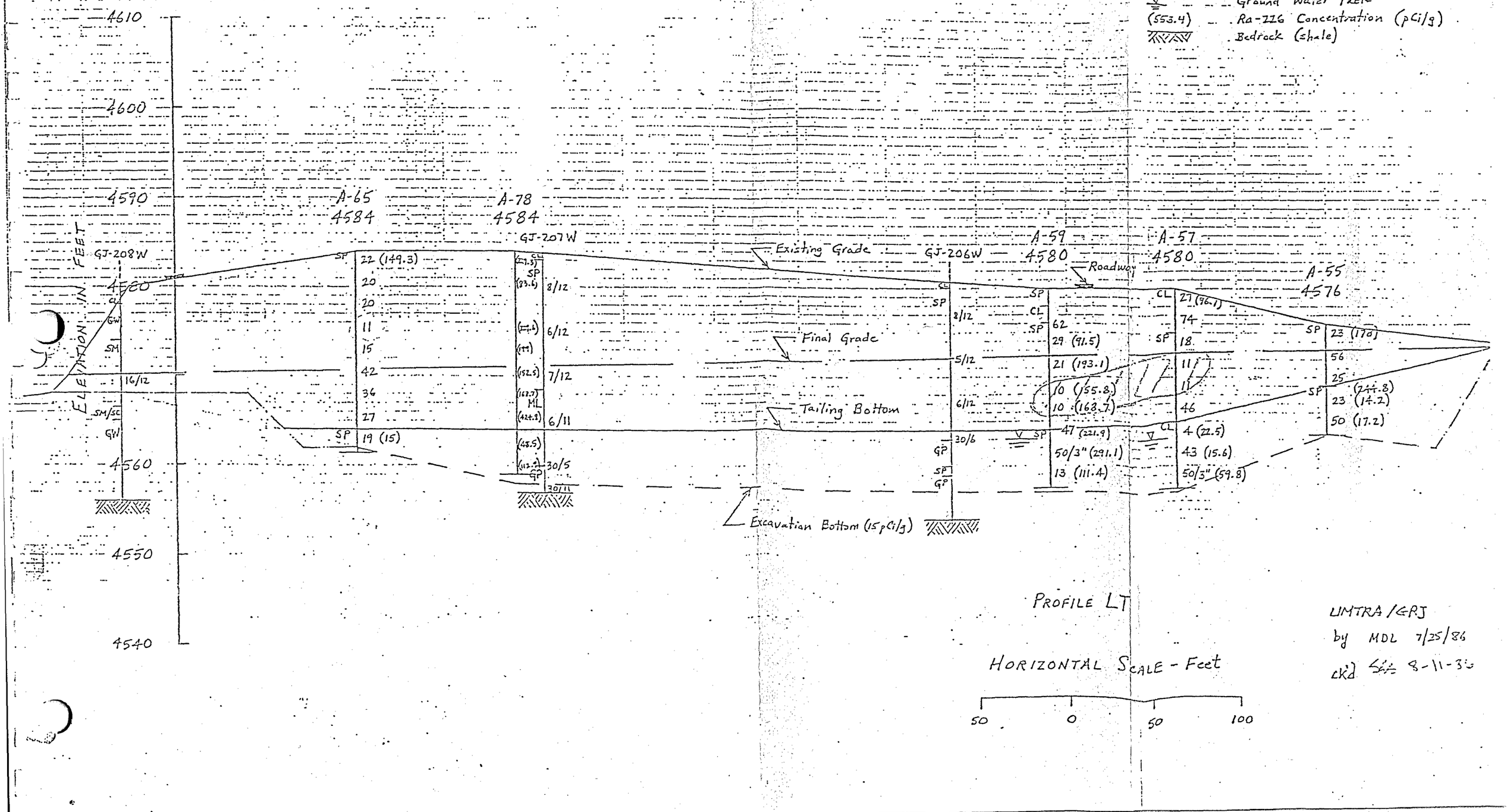


UMTRA/GRJ
by MDL 7/25/86
ckd 5-11-86

PS 2/4/88

LEGEND

D-4	Borehole
4600	Existing Grade Elevation
21	SPT Blow Counts/Ft
SP	Soil Type
▽	Ground Water Table
(553.4)	Ra-226 Concentration (pCi/g)
▨	Bedrock (shale)



PROFILE LT

HORIZONTAL SCALE - Feet

50 0 50 100

UMTRA/GRJ

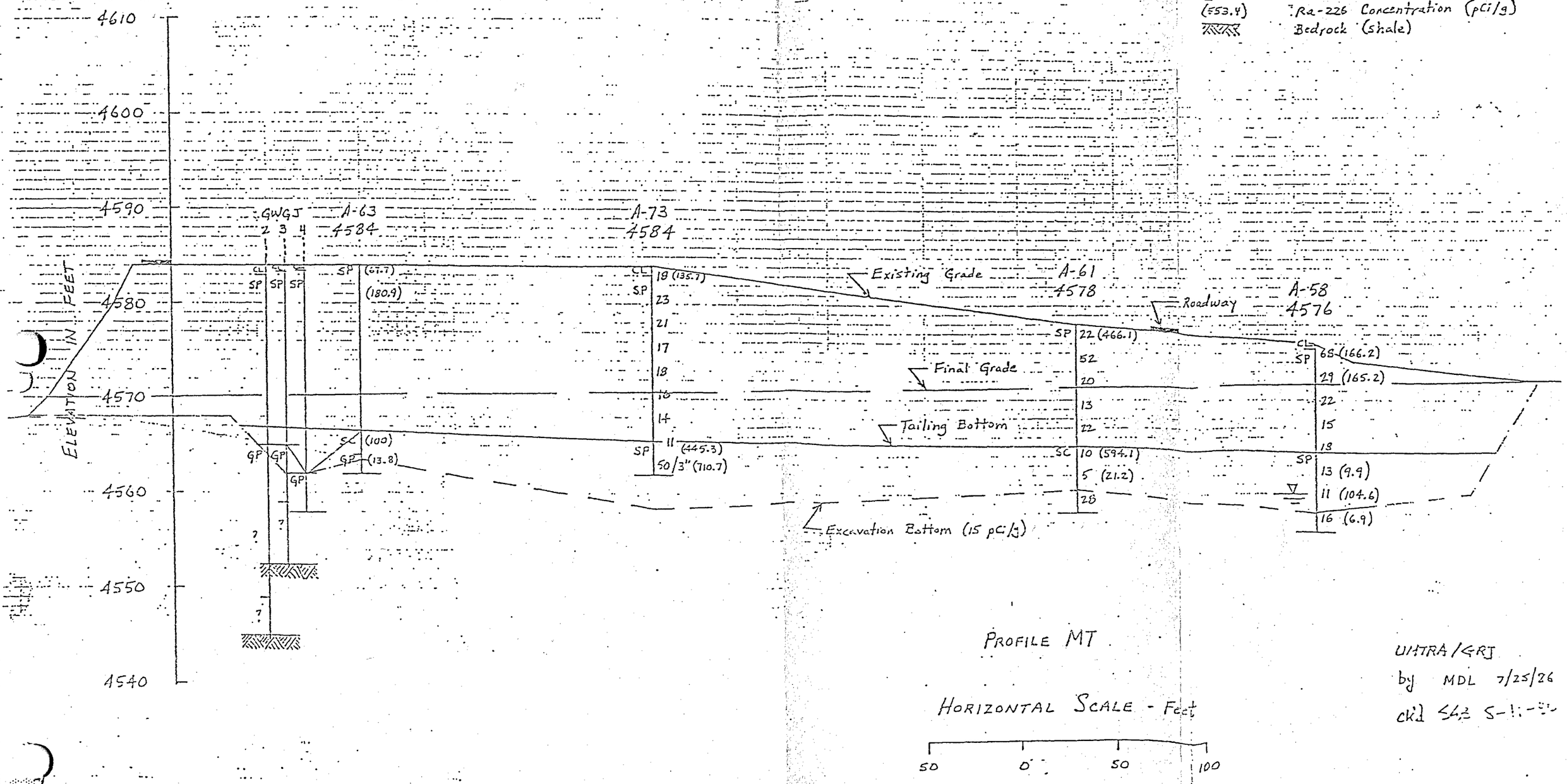
by MDL 7/25/86

ckd 8-11-86

PS 2/4/88

LEGEND

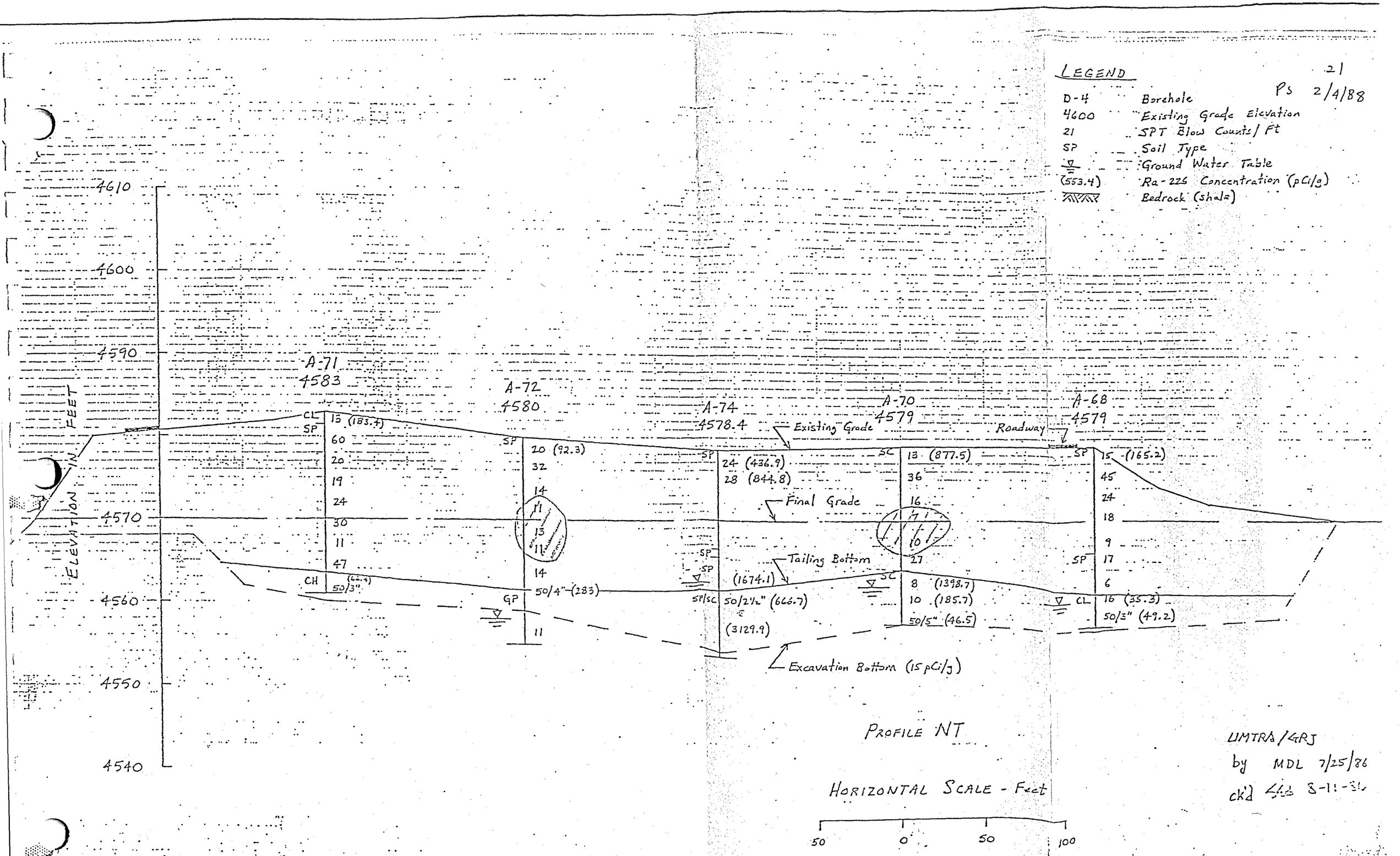
D-4	Borehole
4600	Existing Grade Elevation
21	SPT Blow Counts / Ft
SP	Soil Type
▽	Ground Water Table
(553.4)	Ra-226 Concentration (pCi/g)
	Bedrock (Shale)



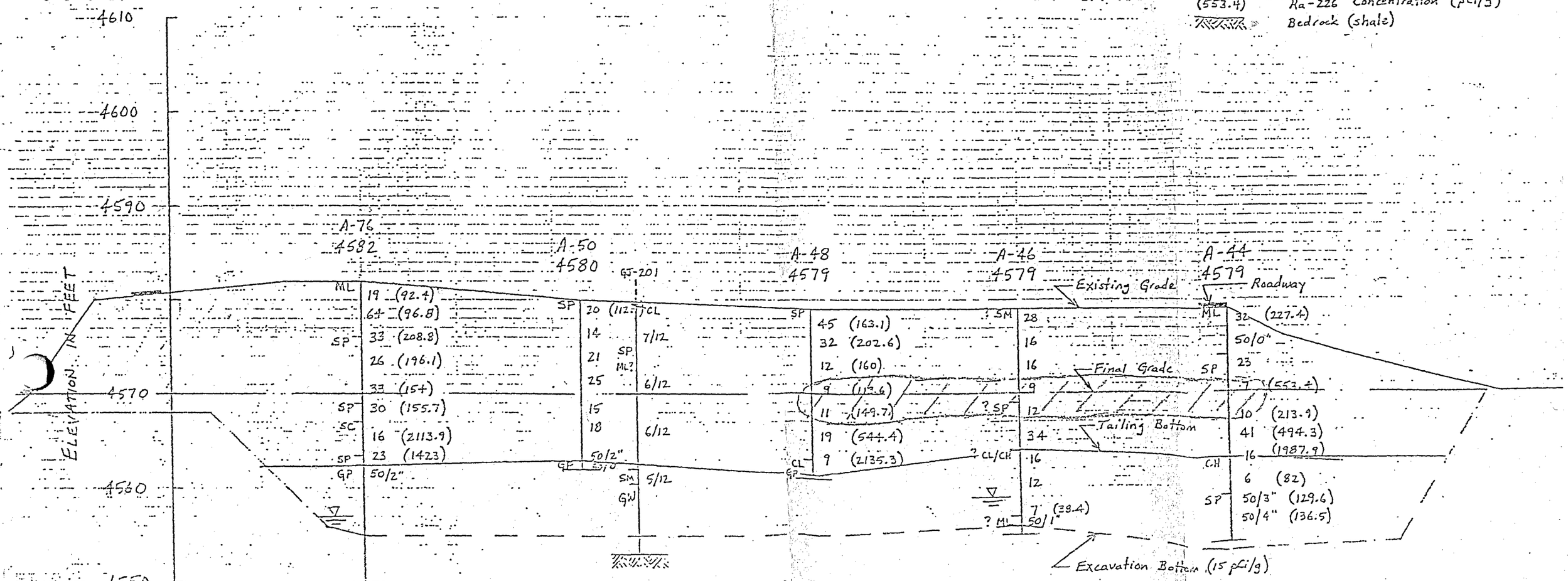
PROFILE MT

HORIZONTAL SCALE - Feet

ULTRA/GRJ
by MDL 7/25/26
ck'd SGB 5-1:-:-


LEGEND
D-4 Borehole
4600 Existing Grade Elevation
21 SPT Blow Counts / Ft
SP Soil Type
Ground Water Table
(553.4) Ra-226 Concentration (pCi/g)
Bedrock (shale)
21
PS 2/4/88
ELEVATION IN FEET
4610
4600
4590
4570
4560
4550
4540
A-71
4583
A-72
4580
A-74
4578.4
A-70
4579
A-68
4579
Existing Grade
Roadway
Final Grade
Tailing Bottom
Excavation Bottom (15 pCi/g)
CL
SP
60
20
19
24
30
11
47
CH
50/3"
15 (183.4)
(62.4)
20 (92.3)
32
14
13
11
14
50/4" (283)
GP
11
SP
24 (436.9)
28 (844.8)
(1674.1)
SP/SC
50/2½" (666.7)
(3129.9)
SC
13 (877.5)
36
16
10
27
8 (1398.7)
10 (185.7)
50/5" (46.5)
SP
15 (165.2)
45
24
18
9
17
6
CL
16 (35.3)
50/3" (49.2)
PROFILE NT
HORIZONTAL SCALE - Feet
50
0
50
100
UMTRA/GRJ
by MDL 7/25/86
ck'd 8-11-86

PS 2/4/88

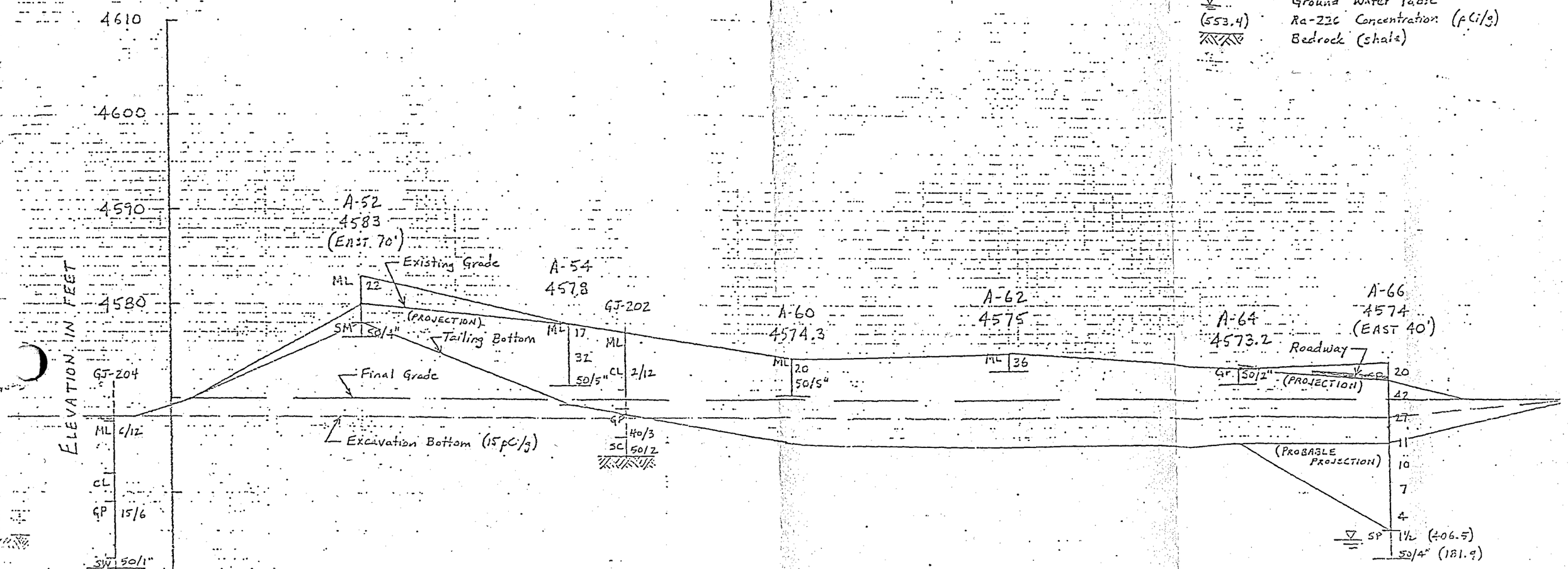


UMTRA/GRJ
by MDL 7/25/86
ck'd 3-11-86

PS 2/4/88

LEGEND

D-4	Borehole
4600	Existing Grade Elevation
21	SPT Blow Counts/Ft
SP	Soil Type
▽	Ground Water Table
(553.4)	Ra-226 Concentration (pCi/g)
▨	Bedrock (shale)



PROFILE PJ

HORIZONTAL SCALE - Feet

50 0 50 100

LIMTRA/GRJ
by MDL 7/25/86
ck'd SLL 8-11-86

PS 2/4/88 WYL 2/4/88 24

SUMMARY OF SOIL TEST RESULTS

Job No. 5025 Project Name UMTRA / GRJ Feature Grand Junction Processing Site Date

Hole or Trench Number	Sample Number	Depth (ft) From	Depth (ft) To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γ_d	Compaction Test	Compaction Optimum w	Compaction Optimum γ_d	Shear Strength Test	Shear Strength Initial w	Shear Strength Initial γ_d	Shear Strength C (ld)	Shear Strength ϕ (ld)	Permeability γ_d (sat)	Permeability k	Consolidation C_v	Consolidation C_c	Consolidation e_o	Notes
I. Sand	Tailings	(Less	than 30%	passing #200 sieve)																						
GRJ-01 -611		3.5	7.0	SM	0	85	15			2.69	3.1		SP	17.4	101.0											
-612		7.5	11.0	SP/SM	0	89	11			2.69	2.9		SP	15.6	99.6											
-613		0.5	4.0	SM	0	81	19			2.70	8.7		SP	16.1	103.5											
-615		4.5	8.0	SM	0	75	25			2.71	11.2		SP	17.3	103.8											
GJ-213B		0	3.0	SM	0	86	14			2.70			SP	13.0	109.2	DS		98.3		0 / 30°	99.8 (NA)	1.4×10^{-3}	fast	0.10	0.73	Remolded to density of 90% Comp as ASTM D-698
-214B		0	3.0	SP	0	93	7			2.70			SP	10.8	110.0						99.8 (NA)	1.1×10^{-3}				Remolded to density of 90% Comp. as ASTM D-698
GJ-419	Undisturb	14.0	16.0	SM	0	71	29			2.70	5.4	86.8														
GJ-22	Undisturb	10.2		SP	0	94 86	6				3.9	98.2									98.2 (NA)	1.1×10^{-2}			0.72	Sat = 18%
GJ-201	Undisturb	14.0		SM	0	77	23				40.6	80.0									80.0 (NA)	3.6×10^{-5}			1.11	Sat = 99%
GJ-207	Undisturb	4.0		SP/SM	0	93	7				2.5	84.6									84.6 (NA)	4.3×10^{-4}			0.99	Sat = 7%
	Undisturb	19.0		SM	0	85	15				37.5	74.1									74.1 (NA)	1.2×10^{-4}			1.27	Sat = 79%
GJ-209	Undisturb	4.0		SM	0	95	5														99.6 (NA)	1.1×10^{-4}			0.70	Sat = 16%
GJ-214	Undisturb	14.0		SP	0	96	4				3.5	99.6														
GJ-220	Undisturb	19.0		SM	0	88	12				9.0	89.4									89.4 (NA)	1.3×10^{-4}			0.89	Sat = 28%
GGJ-3	Undisturb	1.0	3.0	SM	0	83	17				5.9	100.9									100.9 (NA)	2.0×10^{-3}				
-4	Undisturb	13.0	15.0	SM/SP	0	95	5				4.5	86.4									86.4 (NA)	1.0×10^{-3}				
	Undisturb	43.0	45.0	SM/SP	0	95	5				9.7	91.4									91.4 (NA)	8.0×10^{-4}				
-6	Undisturb	8.0	10.0	SM/SP	0	95	5				3.0	91.4									91.4 (NA)	6.0×10^{-3}				
-7	Undisturb	18.0	20.0	SP	0	96	4				3.2	91.0									91.0 (NA)	3.0×10^{-3}				
				Average	0	88	12			2.70	9.7	89.5	SP MD	15.0 10.1	104.5 115.4											
Additional Investigations by MKE (See Sheet 75)																										
9H		5	20'		0	86	14	N	P				SP	14.1	105.0											
21J		5	20		0	88	12	N	P				SP	13.3	108.0											
21J		20	45		0	84	16	N	P				SP	13.2	109.2	$\overline{CU}$	14.1	98.0	0.152 0	34.6 20.5						

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

(from Reference 20)

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
MAIN-PILE TAILINGS — H.L. 6/19/9
SWB 6/22/90

Sh. 24A
REV. 3

SUMMARY OF SOIL TEST RESULTS

Job No 5025-16 Project Name UMTRA – GRAND JUNCTION Feature PROCESSING SITE Date 6/13/90

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γ_d	Compaction Test	Compaction Optimum w	Compaction Optimum γ_d	Shear Strength Test	Shear Strength Initial w	Shear Strength Initial γ_d	Shear Strength C	Shear Strength ϕ	Permeability γ_d	Permeability k	Consolidation C	Consolidation C	Consolidation K	Notes
MAIN-PILE TAILINGS (REF: MKE DOC. 3885-GRJ-R-02-04329-00 – LABORATORY ANALYSIS OF SOIL HYDRAULIC PROPERTIES OF THE GRJ MAIN TAILINGS AND VICINITY PROPERTIES DELIV. ORDER 005 BY DES & A)																										
GRJ-01 500(A)	5-1			SP	0	95	5						SP	10.6	102.3											
"	5-2			"	0	95	5																			
GRJ-01 501(B)	5-3			SM	0	76	24						SP	19.1	101.1											
"	5-4			"	0	80	20																			
GRJ-01 502(C)	5-5			SM-SP	0	86	14						SP	16.1	108.6											
"	5-6			"	0	83	17																			
GRJ-01 503(D)	5-7			SP	0	92	8						SP	12.7	111.1											
"	5-8			"	0	89	11																			
GRJ-01 601	5-9			SP	0	90	10						SP	12.1	118											
"	5-10			"	0	91	9																			
GRJ-01 602	5-11			SM	0	79	21						SP	16.6	108.6											
"	5-12			"	0	75	25																			
GRJ-01 603	5-13			SM	0	72	28						SP	18.1	107.4											
"	5-14			"	0	73	27																			
AVERAGE =					0	84	16						SP	15	108.2											

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

SUMMARY OF SOIL TEST RESULTS

PS 2/4/88 WYL 2/5/88 25

Job No 5025 Project Name UMTRA / GRJ Feature Grand Junction Processing Site Date

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γ_d	Compaction Test	Compaction Optimum w	Compaction Optimum γ_d	Shear Strength Test	Initial w	Initial γ_d	C c/b	ϕ c'/b'	Permeability γ_d (sat)	Permeability k	Consolidation C_v	Consolidation C_c	Consolidation e_o	Notes
II Sand-slime Tailings (30% to 70% Passing #200 Sieve)																										
GRJ-01 -611		0.5	3.5	ML	0	46	54	-		2.75	24.3		SP	28.0	75.1											
GJ-217B		0	3.0	SM	0	70	30	38	18	2.77			SP	18.8	101.0											
GJ-417	Undisturb	15.0	17.0	ML	0	32	68			2.77						CU	37.2	64.0	0.7	0.4 / 30.2						
																			18.5°							
GJ-216	Undisturb	14.0		SM	0	59	41				23.6	100.0									100.0 (NA)	4.4×10^{-4}			0.64	Sat=93%
GJ-220	Undisturb	24.0		SM	0	63	37				20.0	91.8									91.8 (64.0%)	2.6×10^{-4}			0.84	
				Average	0	54	46			2.76	22.6	95.9	SP	23.4	88.1											
III Slime Tailings (More than 70% Passing #200 Sieve)																										
GRJ-01 -614		5.0	7.0	ML	0	15	85			2.80	33.3		SP	42.1	72.8											
GJ-209B		0	3.0	ML	0	30	70	37	3	2.76			SP	28.6	83.8	DS		75.4		0 / 45.5°	74.9 (NA)	9.3×10^{-5}	fast	0.24	1.30	
-216B		0	3.0	ML	0	9	91	46	5	2.70			SP	32.7	80.0											
GJ-421	Undisturb	8.0	10.0	ML	0	2	98			2.79						CU	62.3	60.6	0.1 / 21.9°	0.1 / 30.2°	60.6 (90.9%)	4.2×10^{-6}				
GJ-24	Undisturb	6.5		ML	0	18	82				41.6	65.5									65.5 (NA)	4.6×10^{-4}			1.65	Sat = 68%
GJ-200	Undisturb	14.0		ML	0	10	90														93.9 (NA)	1.3×10^{-6}			0.79	Sat = 72%
GJ-209	Undisturb	14.0		ML	0	20	80				22.5	93.9														
GJ-220	Undisturb	14.0		ML	0	22	78				31.0	88.3									58.3 (NA)	6.3×10^{-5}			1.89	Sat = 44%
GGJ-2	Undisturb	13.0	15.0	ML	0	5	95				37.0	104.1														
-7	Undisturb	33.0	34.0	ML				40	10		39.2															
				Average	0	15	85	41	6		37.4	80.5	SP	39.5	78.9											
IV Mixed Tailings (Man Made)																										
GJ-16M		80% GJ-213B (Sand) 20% GJ-209B SM				84	16			2.71			SP	12.9	109.2	CU		109.2		0 / 43°	98.6 (NA)	8.5×10^{-4}				
-17M		60% GJ-213B (Sand-Slimes) 40% GJ-209B SM				68	32			2.73			SP	16.1	106.5	DS ? CU		95.3 106.5		0 / 45° 1.44 / 33.5°	93.6 (NA)	4.8×10^{-4}	fast	0.134	0.79	
-18M		40% GJ-213B Sand-Slimes 60% GJ-209B SM				60	40			2.76			SP	19.0	99.0	CU		99.0		0 / 44°	99.2 (NA)	5.6×10^{-4}				

* Visual Classification

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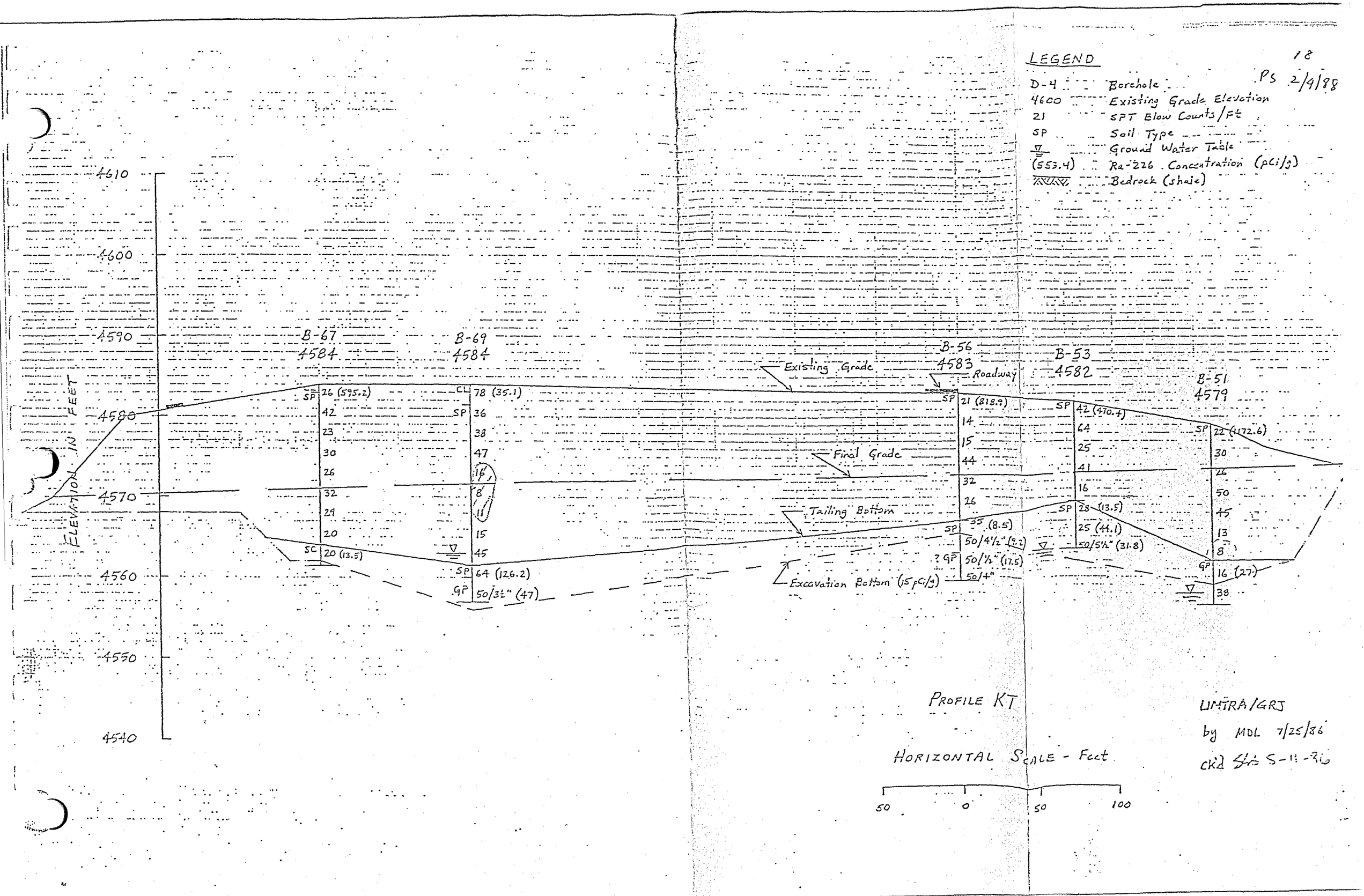
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained


LEGEND
D-4 Borehole
4600 Existing Grade Elevation
21 SPT Blow Counts/Ft
SP Soil Type
Ground Water Table
(553.4) Ra-226 Concentration (pCi/g)
Bedrock (shale)
18
PS 2/4/88
ELEVATION IN FEET
4610
4600
4590
4580
4570
4560
4550
4540
B-67
4584
B-69
4584
B-56
4583
B-53
4582
B-51
4579
Existing Grade
Roadway
Final Grade
Tailing Bottom
Excavation Bottom (15 pCi/g)
SP 26 (595.2)
42
23
30
26
32
29
20
SC 20 (13.5)
CL 78 (35.1)
SP 36
38
47
18
8
11
15
45
SP 64 (126.2)
GP 50/3½" (47)
SP 21 (818.9)
14
15
44
32
26
SP 35 (8.5)
50/4½" (9.2)
? GP 50/½" (17.5)
50/4"
SP 42 (470.4)
64
25
41
16
SP 28 (13.5)
25 (44.1)
50/5½" (31.8)
SP 22 (172.6)
30
26
50
45
13
8
GP 16 (27)
38
PROFILE KT
HORIZONTAL SCALE - Feet
50
0
50
100
UMTRA/GRJ
by MDL 7/25/86
ck'd 5-11-86

Main-pile Tailings

REVISION 3

MAIN-PILE TAILINGS

Based on Grain Size Analysis ~~previously~~ recently completed on Sh. 24A, and compared with data on Sh. 24 and 25 (Previous Tests), it is reasonable to use the pertinent data from previous calculation

Also, present plans are as follows:

(a) A minimum of 10-ft of the top of the tailings embankment shall consist of ponds area material.

(b) The rest of the embankment shall consist of both main-pile tailings, and ponds area material. For the purposes of settlement and stability analyses, the entire ponds area material will be assumed at the top (≈25'). This is conservative, since ponds area material has greater compressibility and lower strength than the main tailings pile material.

'Designated Average Tailings Material' from file

It is expected that considerable mixing will take place between the 3 broad categories of tailings, e.g. sands, sand-slimes and slimes. A cursory examination of the boring logs indicate a majority (over 50 percent) of the tailings may be classed as 'sands'. Therefore, the resultant material in the tailings embankment will most likely be dominated by material which will fall in the sand-slimes zone, biased towards the coarser end of the category. For the purposes of material characterization, the 'designated average material' is a sand-slime with 40 percent passing the #200 sieve. Thus, the sample GJ-18M most closely represents the 'designated average material.'

Other-Contaminated Material

Additional contaminated material from settling ponds and vicinity property will be included in the tailings embankment and will be placed at the top of the relocated tailings material in the tailings embankment at the disposal site. Details of their properties will be discussed in sheets 71 thru 94.

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project: UMTRA/GRJ
Feature: Embankment Design
Item: Soil Properties

Contract No. 5025
Designed: PS
Checked: WYL

Sheet 27
File No.
Date 2/4/88
Date 2/5/88

Data of Density vs Moisture Content

Sample ID	Depth (ft)	-#200		Saturation @ $(\gamma_d)_{max}$ & OMC	$(\gamma_d)_{max}$	O.M.C.
611-001	0.5-3.5	54		59.8	75.1	28.0
611-002	3.5-7.0	15		70.5	101.0	17.4
612-001	7.5-11.0	11		61.2	99.6	15.6
613-001	0.5-4.0	19		69.3	103.5	16.1
614-001	5.0-7.0	85	slime	84.1	72.8	42.1
615-001	4.5-8.0	25		74.6	103.8	17.3
GJ-213B	0-3.0	14		64.6 (a)	109.2	13.0
GJ-209B	0-3.0	70	slime	74.8 (a)	83.8	28.6
GJ-214B	0-3.0	7		54.8 (a)	110.0	10.8
GJ-216B	0-3.0	91	slime	78.2 (a)	80.0	32.7
GJ-217B	0-3.0	30		73.6/75.9 (a)	101.0	18.8
GJ-16M	0-3.0	16		64.2 (a)	109.2	12.9
GJ-17M	0-3.0	32		72.0 (a)	106.5	16.1
GJ-18M	0-3.0	40		70.9 (a)	99.0	19.0

(a) Computed using specific gravity of 2.70 for sand and 2.76 for others

Avg. Saturation = 69.4% ≈ 70%

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Item: Soil Properties

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Sheet 28
File No.
Date 2/4/83
Date 2/4/88

CLAY CONTENT IN TAILINGS

Sample No	-5μ	-2μ	-#200	-#200/-5.
611 (0.5-3.5)	18	14	54	3.0
611 (3.5-7.0)	6	4	15	2.5
612 (7.5-11.0)	5	4	11	2.2
613 (0.5-4.0)	8	4	19	2.4
614 (5.0-7.0)	26	15	85	3.3
615 (4.5-8.0)	12	8	25	2.1
417 (15.0-17.0)	20	12	68	3.4
419 (14.0-16.0)	14	10	29	2.1
421 (8.0-10.0)	19	14	98	
GJ-24 (6.5)	40	19	82	
GJ-200 (14)	17	10	90	
GJ-209 (14)	12		80	
GJ-22 (10.2)	7		12	
GJ-209B (0-3.0)	26	15	70	2.69
GJ-216B (0-3.0)	60	25	91	1.5
GJ-217B (0-3.0)	17	14	30	1.8
GJ-17M (0-3.0)	10		32	3.2
GJ-18M (0-3.0)	14		40	2.9

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Item: Soil Properties

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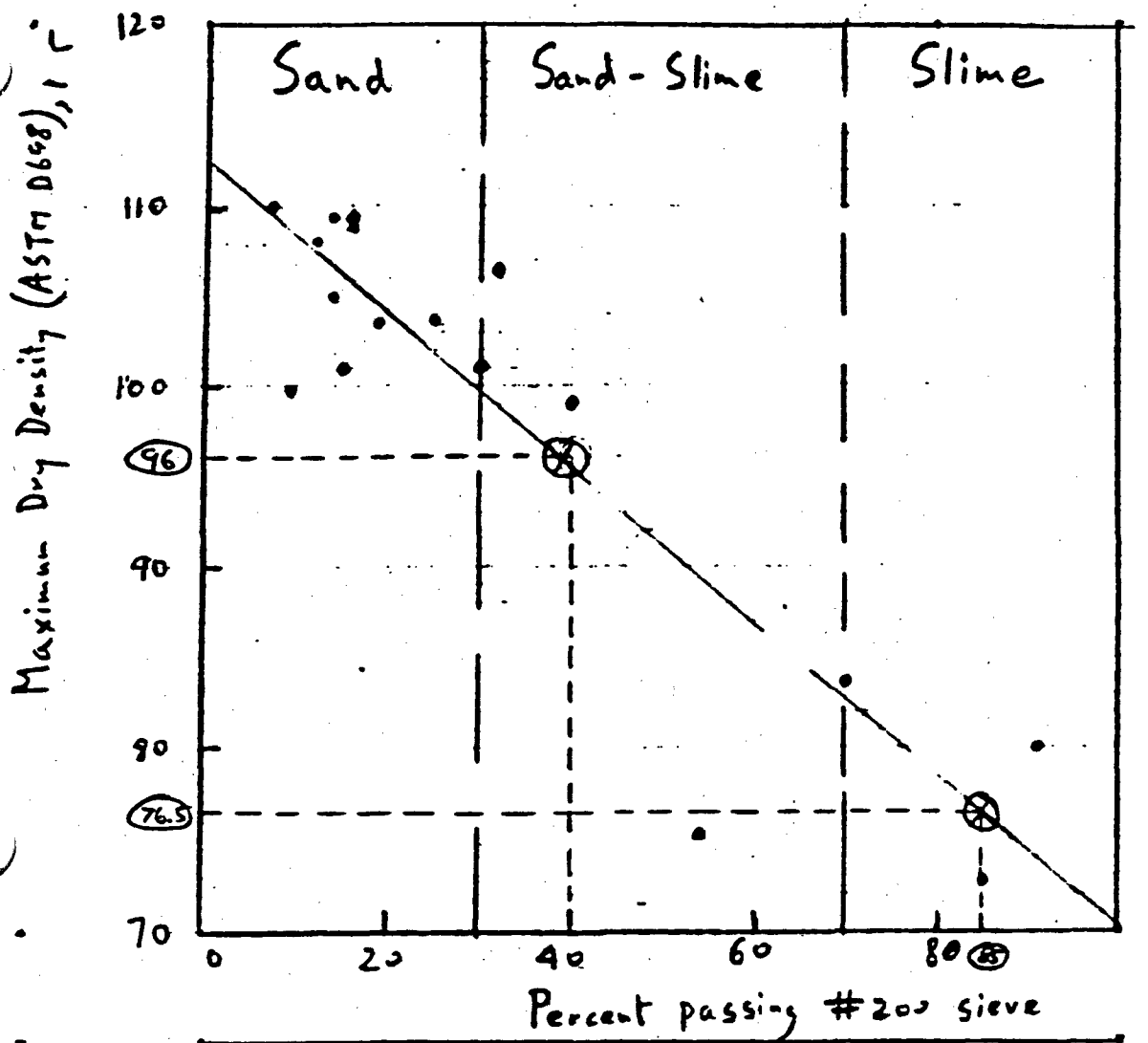
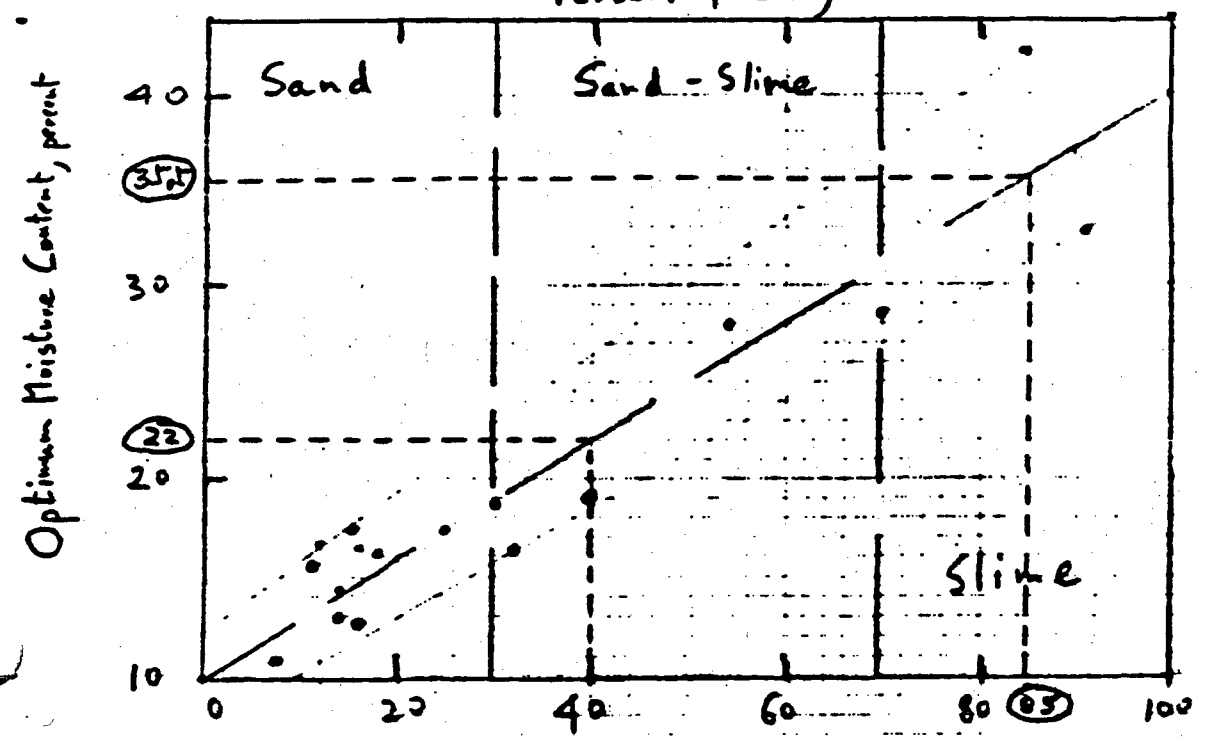
Sheet 29
File No.
Date 2/9/88
Date 2/5/88

Shown in Sheets 30 and 31 are plots of (a) maximum dry density (Standard Proctor), (b) Optimum Moisture Content, (c) degree of saturation and (d) specific gravity with percent passing #200 sieve. Some obvious patterns emerge.

PS 2/4/88
WYL 2/5/88





PS 2/4/88

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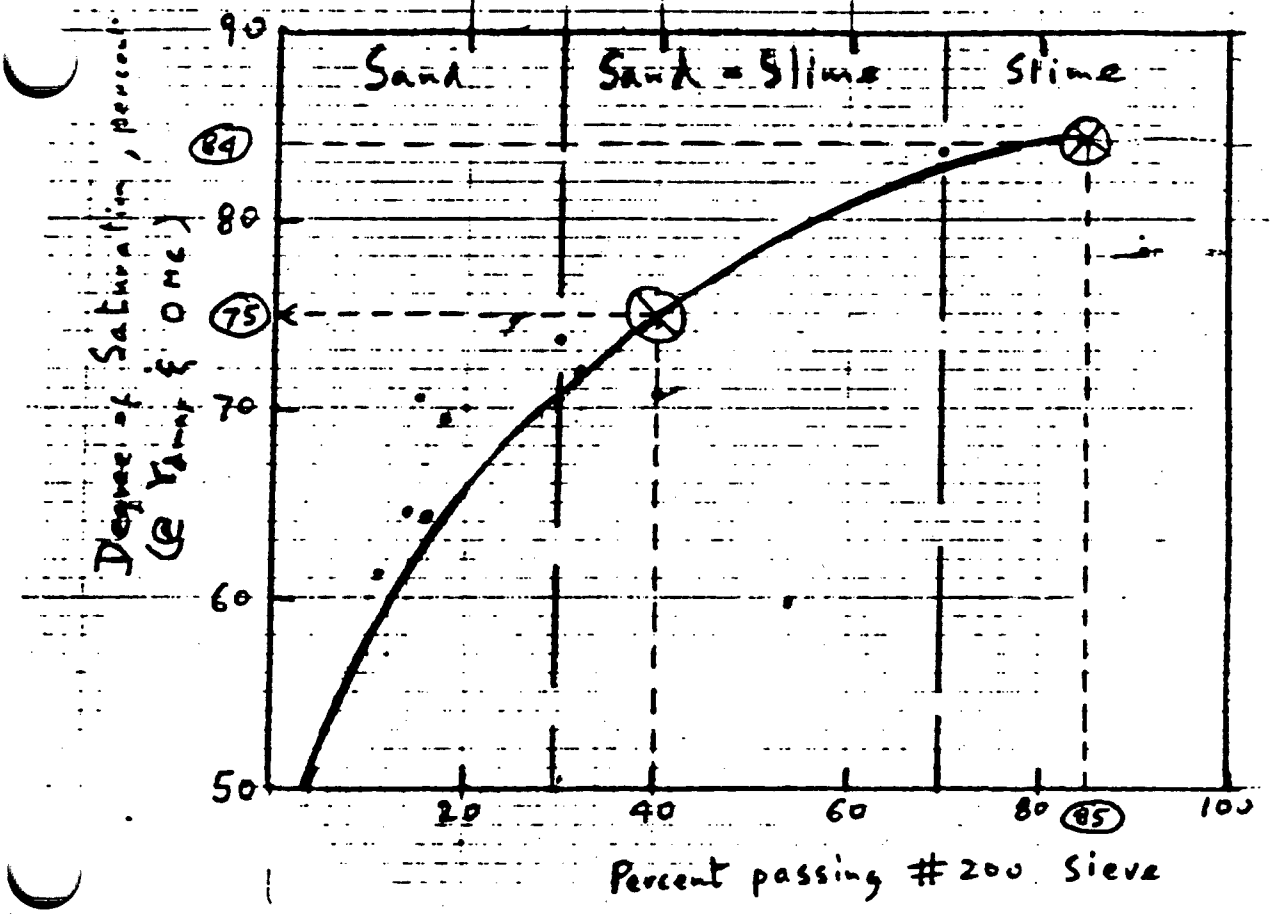
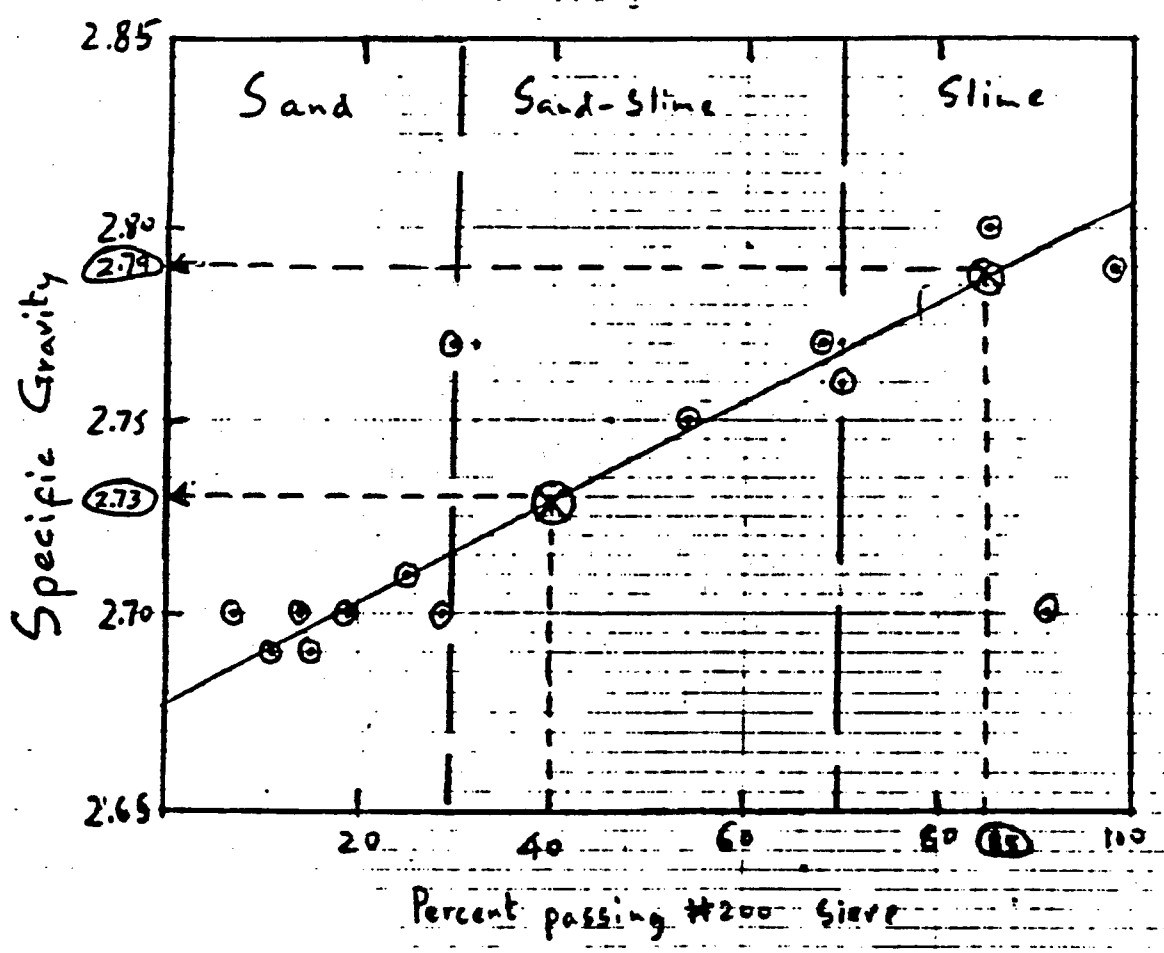




In-Place density, Specific Gravity and Moisture Contents of Main Pile Tailings

Plots in Sheets 30 indicate that for material for which 40 % passes #200 sieve,

Maximum Dry Density = 96 pcf

Optimum Moisture Content = 22 %

Degree of Saturation = 75 % (Sheet 31)

∴ For 90 % Compaction,

Bulk Density, $\gamma = 0.9 \times 96 \times 1.22 = 105.4$ pcf

(Sample 18M, for which 40% passes #200 sieve:
$\gamma = 99 \times 0.9 \times 1.19 = 106.0$ pcf)

Thus, use design value of $\gamma = \underline{105 \text{ pcf}}$

Also, use design specific = $\underline{2.73}$ (from Sheet 31)

For saturated conditions

$\gamma_d = \frac{G}{1+e}\gamma_w$ $\quad e = \frac{\gamma_w G}{\gamma_d} - 1 = \frac{62.4 \times 2.73}{96 \times 0.90} - 1 = 0.97$

$G w_{sat} = S e$ $\quad w_{sat} = \frac{S e}{G} = \frac{100\% \times 0.97}{2.73} = 35.5\%$

$\gamma_t = \frac{G + Se}{1+e}\gamma_w$ $\quad \gamma_t = \frac{2.73 + 100\% \times 0.97}{1 + 0.97} \times 62.4 = 117.2$ pcf

Strength Properties

Purpose and Background

The purpose of these computations is to come up with appropriate design shear strength properties for the tailings material from the main pile. The tailings material will be relocated to the disposal site and placed and compacted at 90% compaction as per ASTM D-698. Strength data available to us, are as follows:

- Direct Shear Tests on Compacted Samples (Sheets 40 & 41)

- Consolidated Undrained (CU) Triaxial Tests with Pore Pressure Measurements on Remolded Samples (Sheets 43 thru 53)

- Consolidated Undrained (CU) Triaxial Tests with Pore Pressure Measurements on Undisturbed Samples (Sheets 54 and 55)

Summary of Evaluation

Design Shear Strength Properties

	Main Pile	
Undrained	$c = 0$; $\phi = 25°$	$\gamma_t = 105$ pcf
Drained	$c = 0$; $\phi = 35°$	$\gamma_t = 105$ pcf

Evaluation Procedure And Commentary

Main Pile Tailings

As mentioned above, the ratio of sand and sand-slime tailings to slimes tailings is about 10 to 1 in volume. There appears to be a predominance of sand to sand-slimes in the tailings pile, thus the mixture in the proposed tailings embankment will likely be between a sand and a sand-slimes. The designated average material is a sand-slime with 40% passing the #200 sieve.

Based on CSU report (Ref 11), the tailings have a high content of absorbed salts. These salts may provide some cementation and also form a strongly attracted, rough surface on all particle faces, beyong the angularity. This rough surface increases the shear resistance beyong that normally associated with sand of this type.

Test results indicate that effective shear strength parameters of 100% standar compaction for CU tests are about 10 degree in friction angle lower than for direct shear tests for corresponding data, see sheet 39. Thus, CU tests are considered, for conservatism, and yet, reality.

Besides, the leaching process has a slight effect on a friction angle for the sand tailings, see sheet 41.

Drained or Effective Shear Strength Parameters

Test results indicate effective shear strength parameters for 100 % Standard Compaction of CU tests are lower than for corresponding data for direct shear tests. See Sheet 39. Thus, CU tests are considered, for conservatism.

Effective shear strength parameters for undisturbed tailings (Sheets 54 and 55) are for samples at around 70 % of Standard Proctor Compaction for samples with 32 % and 97 % passing #200 sieves for CU tests with pore pressure measurements.

Note, that in all cases, the lowest value of ϕ' for $c'=0$ is used. This is conservative, and yet, realistic

The value of ϕ' is thus interpolated for 90% Standard compaction (as will be in the field). as follows:

$\phi' = 33°$ @ 70 percent compaction (Sample 417-15.0'-17.0')
$\phi' = 34$ @ 90 percent compaction (Sample 21J)
$\phi' = 42°$ @ 100 percent compaction (Sample 17M)

Use $\phi' = 35°$ for the Design

This is conservative, and will account for leaching effect

(ii) Undrained or Total Shear Strength Parameters

The same 2 samples were used for interpolation of total stress strengths from CU tests.

At 70 percent compaction, c=0.7 ksf and ϕ = 18.3° (Sample 417,15)

At 90 percent Compaction c=0.95 ksf and ϕ = 20.5° or c = 0 ksf and ϕ = 25° (Sample 21J)

At 100 percent Compaction c=3.6 ksf and ϕ = 35° (Sample 17M)

Use c=0, ϕ = 25° for Design ---- conservative

The CU tests on compacted samples are evaluated in Sheets through . The following test results were available to us:

A. Stress Paths and Effective Mohr envelopes for the 3 samples tested. (Sheets 43, 46 and 49)

B. Effective Principal Stress Ratio versus Axial Strain for the 3 samples tested (Sheets , and)

C. Deviator Stress versus Axial Strain for the 3 samples tested (Sheets 44, 47 and 50).

The 3 samples tested are GJ-16M, GJ-17M and GJ-18M, all at 100 % of ASTM D698 compaction.

Based on the above data, values of principal stresses at failure are computed both for total and effective stresses, . Mohr circles based on these computed principal stresses are drawn and the resulting Mohr envelopes (e.g. ϕ and c for total stress and ϕ' and c' for effective stresses) are developed. The tacit assumption made here is that for both the total and effective stress conditions, the failure occurs at the same point, e.g. at the point of maximum effective stress ratio.

It is notable that the maximum deviator stress for all the tests increase significantly beyond the value at the maximum effective principal stress ratio. This is reflective of the

dilating nature of the soil samples (the increase cannot be attributed to bulging of samples, since the strains are still small). Mohr circles and Mohr envelopes under total stress conditions, using the maximum deviator stress as the 'failure' criterion are developed in Sheets and . Some extrapolations of the stress-strain data had to be made for the value of the maximum deviator stress in most cases. As noted on most cases, a significantly higher envelope is obtained by using the maximum deviator stress as the 'failure' criterion, as opposed to the maximum effective principal stress ratio. The latter criterion is thereby selected, since it is more conservative

Drained (Effective)

h. Shear Strength Characteristics (Compacted Sample

A. Direct Shear Test Results (90% Compaction ASTM D-698)

Sample No.	γ_d, 90% Std. Proctor	ϕ	Percent passing #200 sie
GJ-209B (Slime)	95.4 (.9 x 83.8)	41°-45° (51°-53°)	70
GJ-17M (Sand-slime)	95.9 (.9 x 106.5)	41°-43° (53°-54°)	32
GJ-213B (Sand)	98.3 (.9 x 109.2)	29°-30° (51° for dry) (43°-45°-57° dry)	14

(ϕ values in parentheses are for 100% compaction)

B. Consolidated-Undrained (CU) Test Results (100% Compaction ASTM D-698)

GJ-16M (80% sand, 20% slimes)	109.2	$\phi' = 43°$	16
GJ-17M (60% sand, 40% slimes)	106.5	$\phi' = 42°$	32
GJ-18M (40% sand, 60% slimes)	99.0	$\phi' = 44°$	40

Thus, there appears to be little effect of grain size. Also, the ϕ' value for CU tests seem to be lower than for direct shear tests for the same density by about 10 degrees

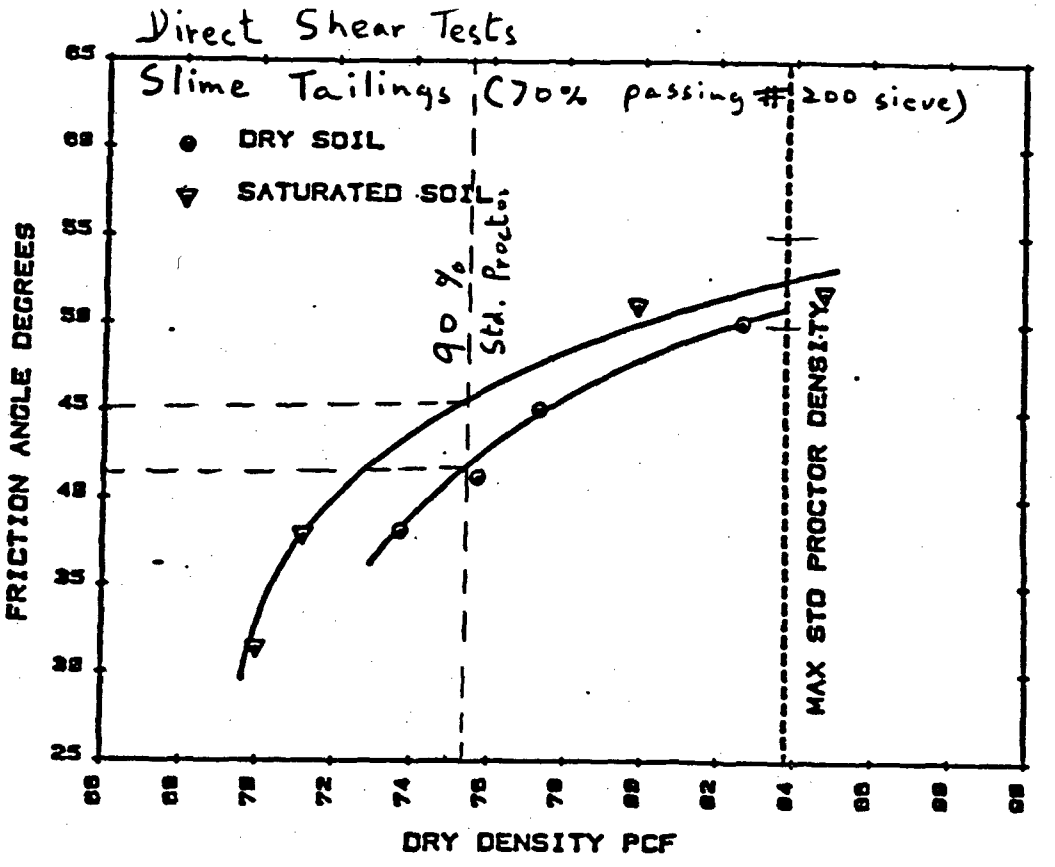


Figure 16B. Angle of Internal Friction vs. Density: Field Sample GJ-209B (Predominantly Slimes).

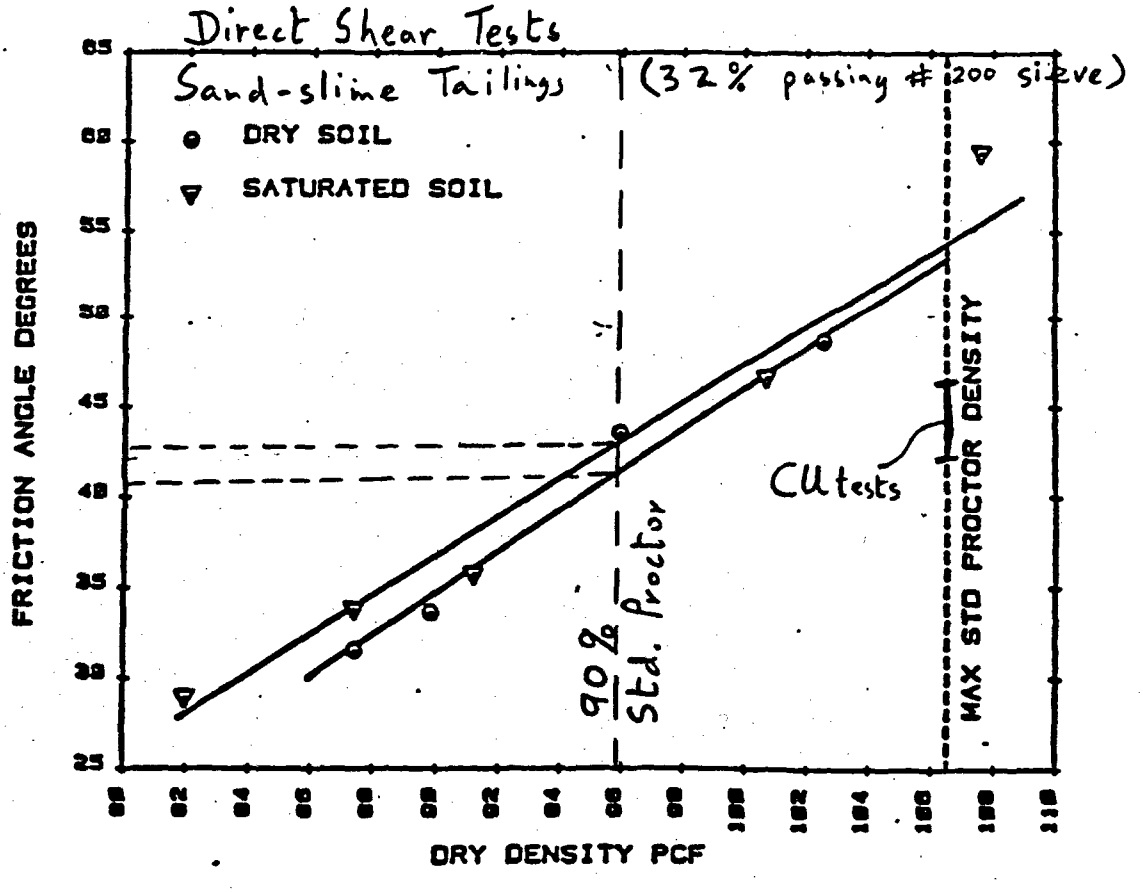


Figure 16C. Angle of Internal Friction vs. Density: Laboratory Sample GJ-17M (Sand-Slime Mixture).

4/28/87 67

WYL 2/5/88

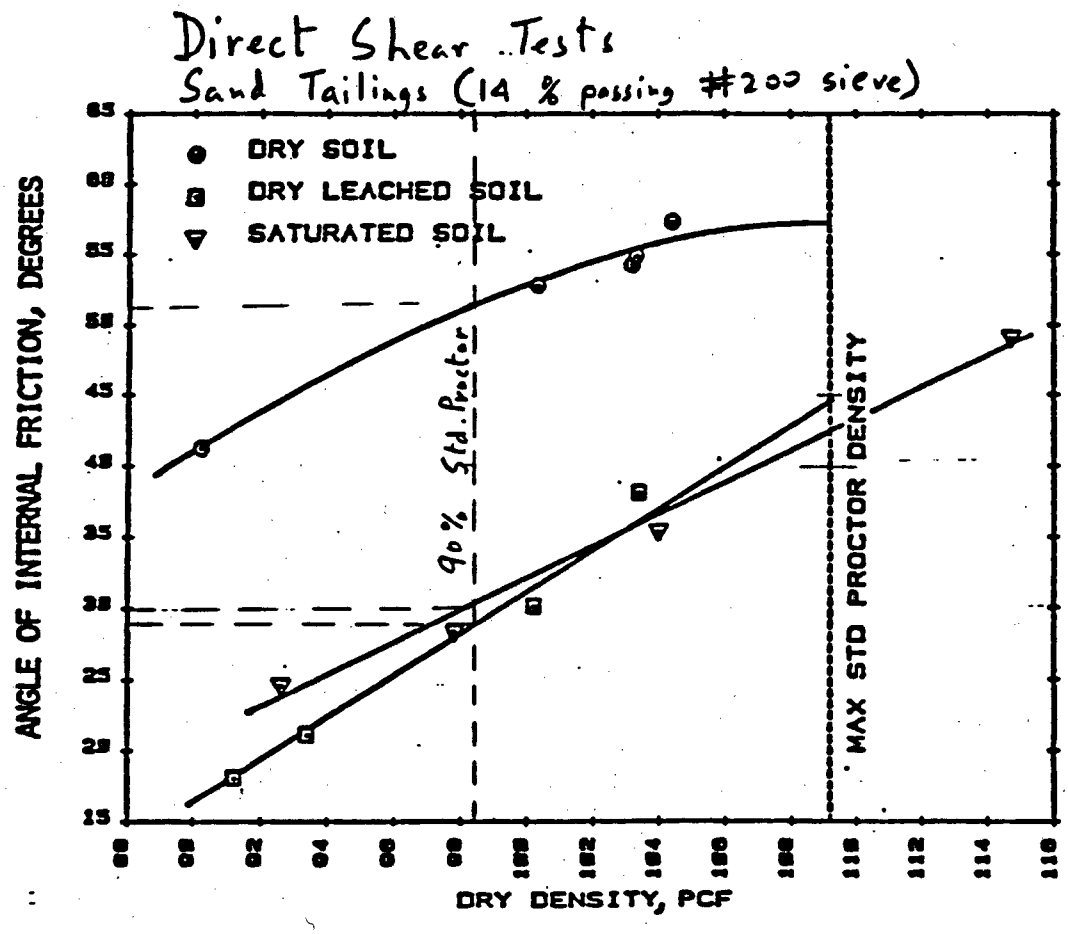
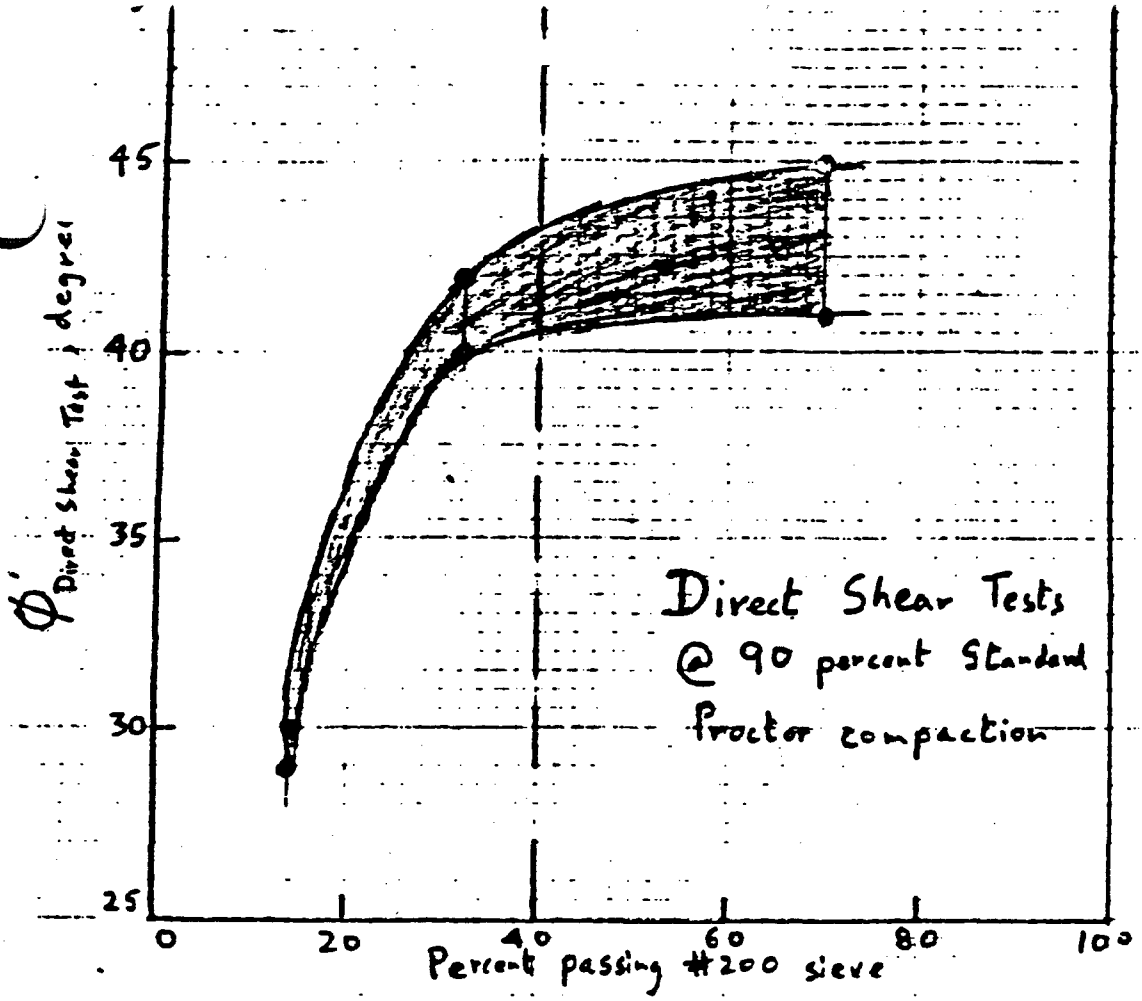
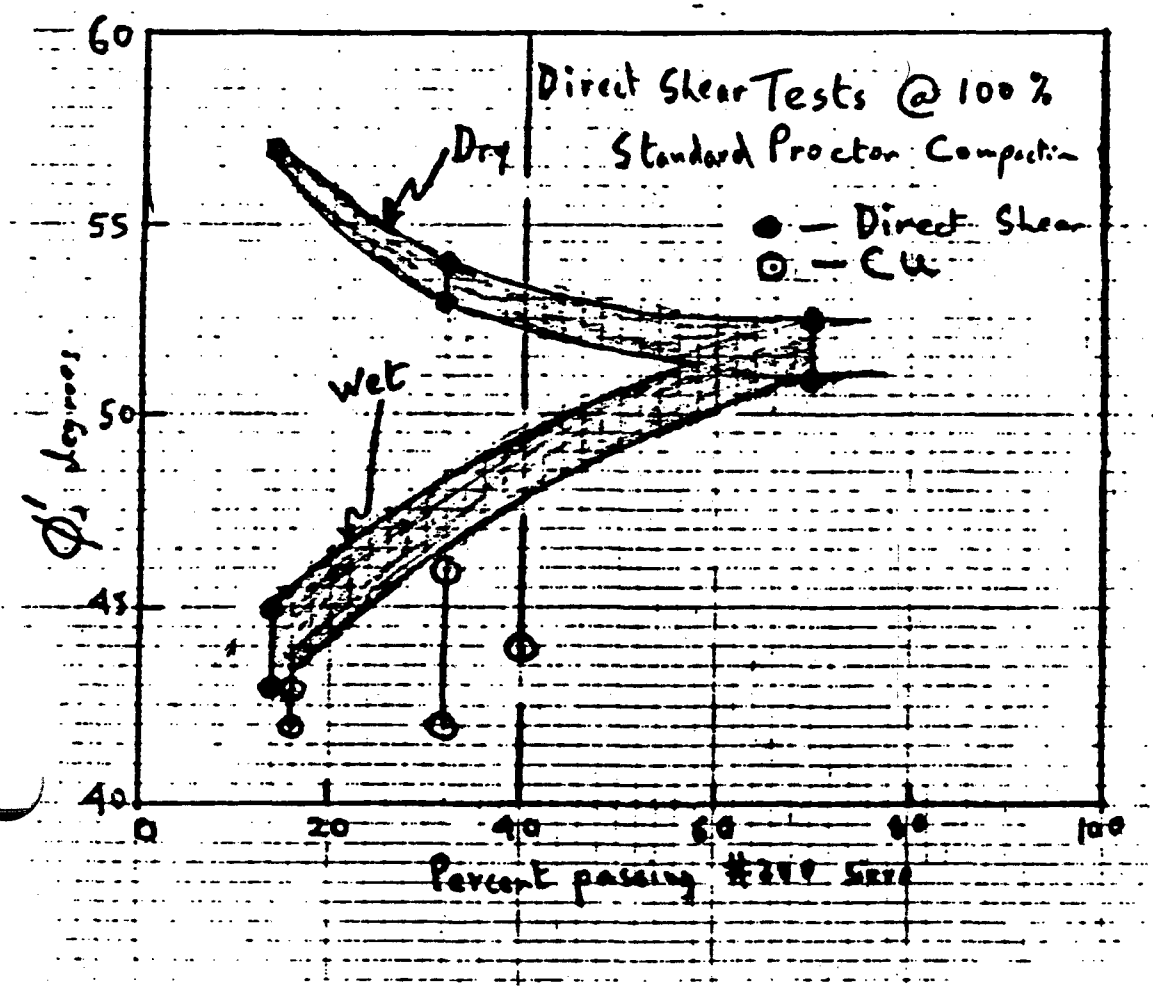


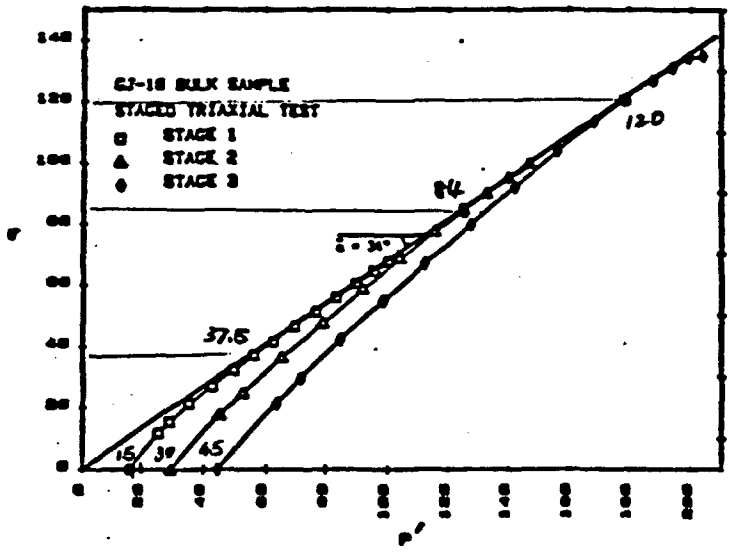
Figure 16A. Angle of Internal Friction vs. Density: Field sample GJ-213B (Beach Sand).

PS 2/4/88
WYL 2/5/88

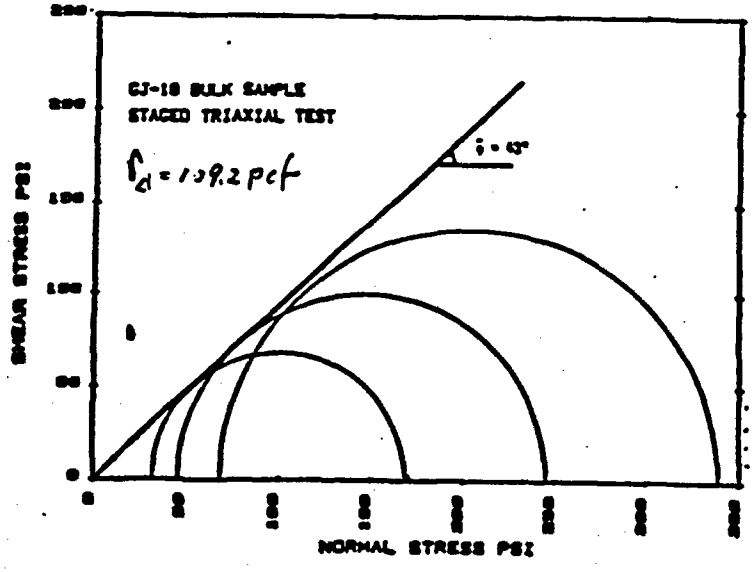




PS 2/4/88
WYL 2/5/88



i) Stress Paths



ii) Failure Envelope

Figure 17A. Stage Triaxial Test Results: GJ-16M (80% Sand/20% Slimes)

PS 2/4/88
WYL 2/5/88

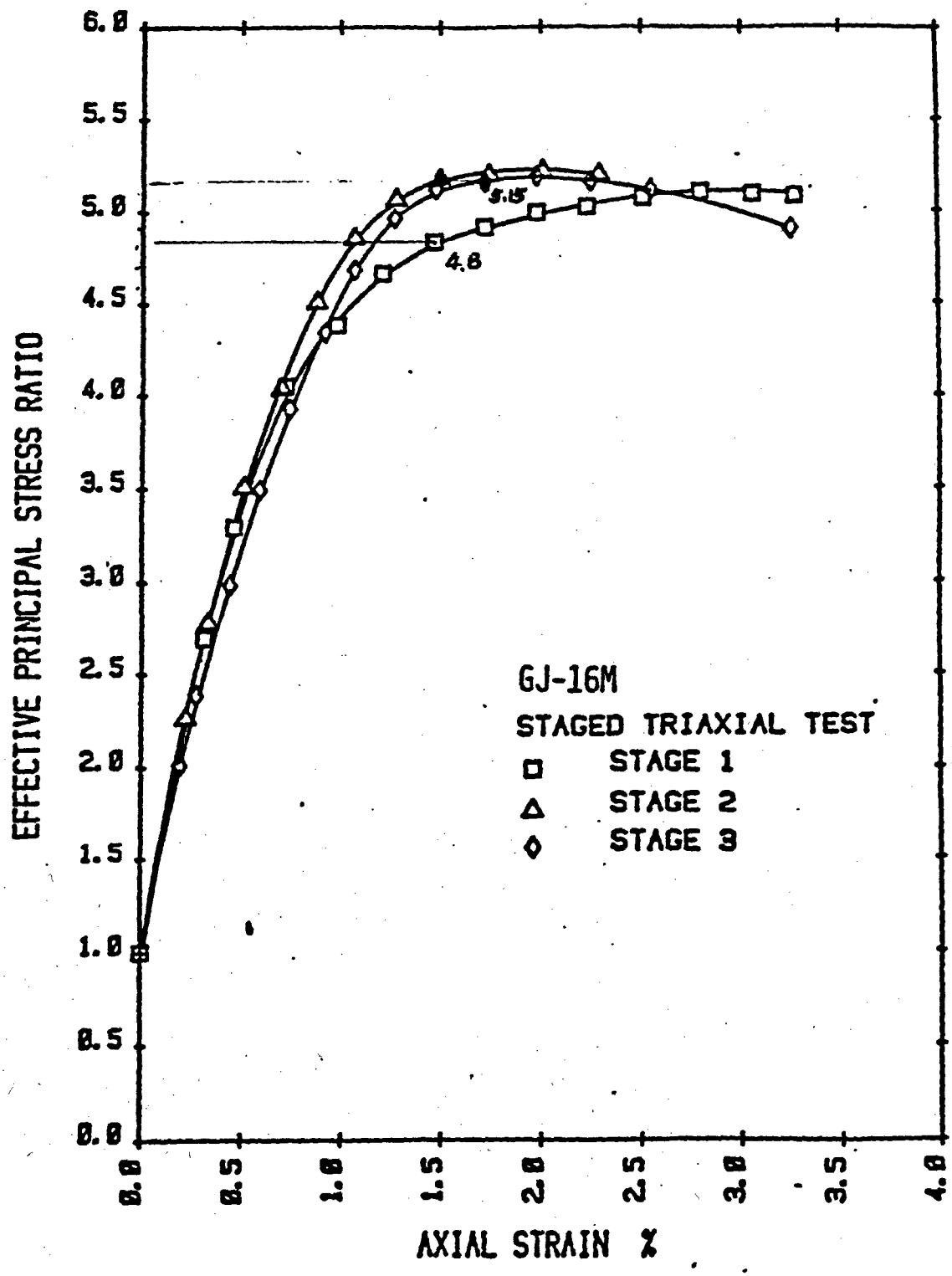


FIGURE B-2: EFFECTIVE PRINCIPAL STRESS RATIO VS. AXIAL STRAIN, GJ-16M

PS 2/4/88
WYL 2/5/88

45

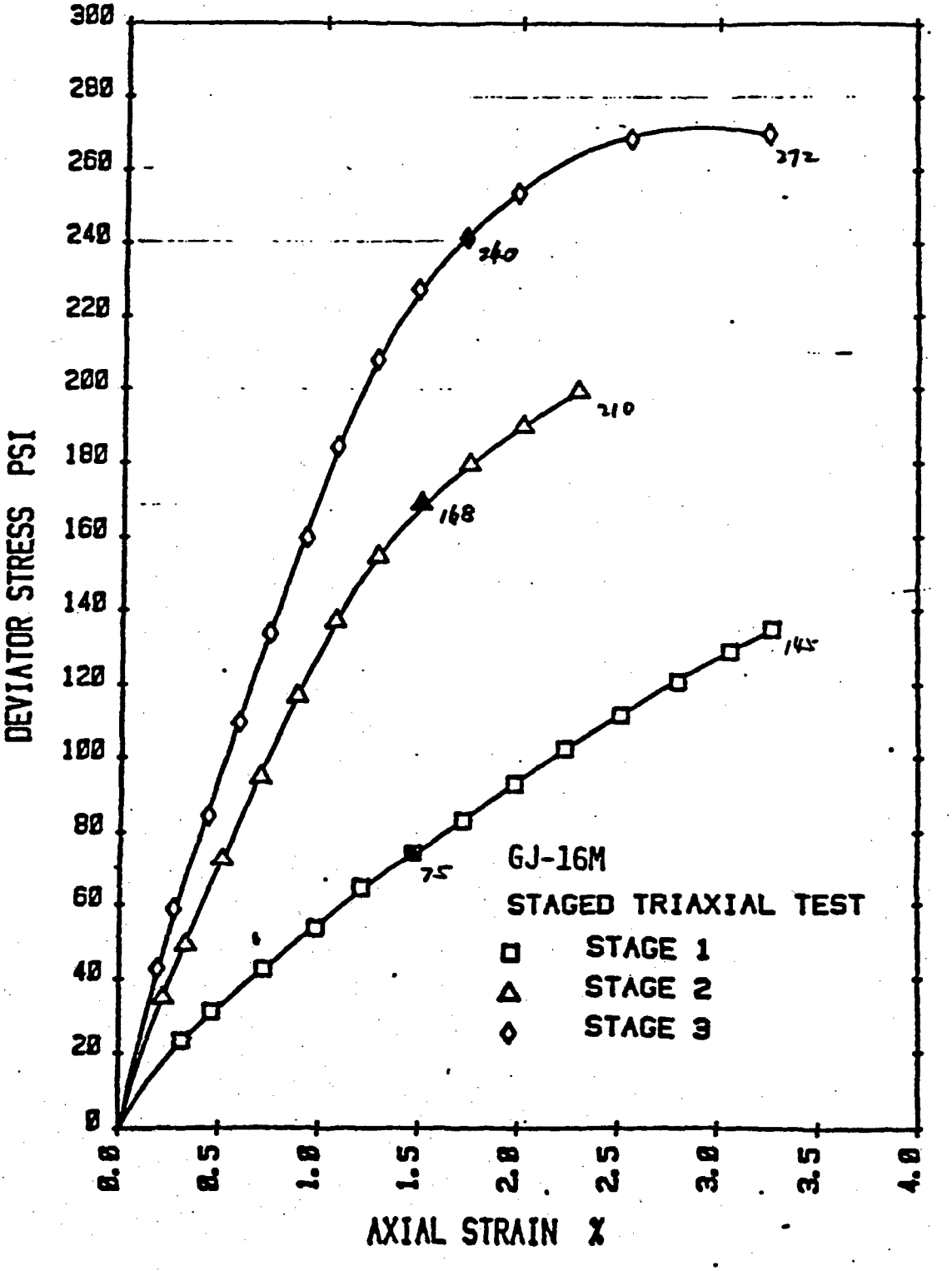
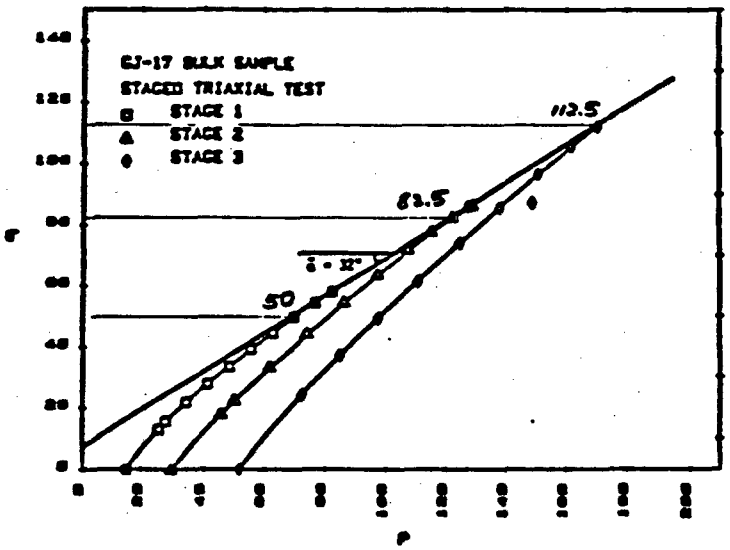
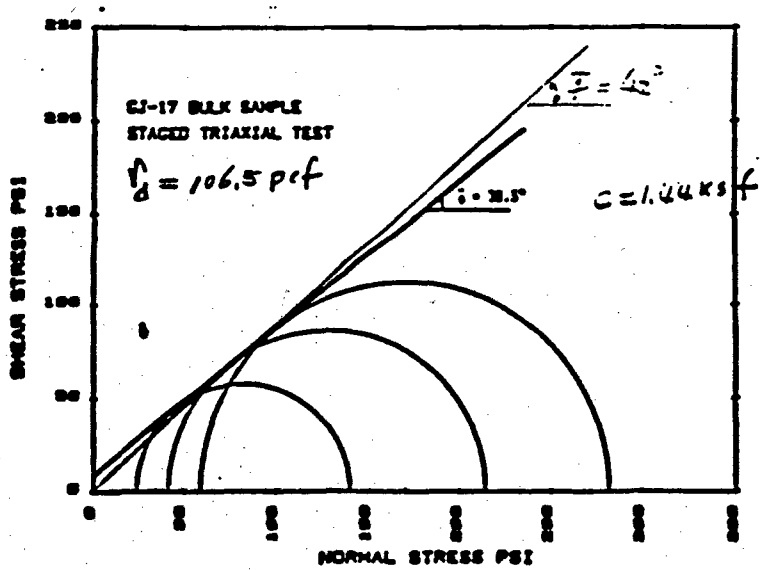


FIGURE B-1: DEVIATOR STRESS VS. AXIAL STRAIN, GJ-16M

RS 2/4/88
WYL 2/5/88



i) Stress Paths



ii) Failure Envelope

Figure 17B. Stage Triaxial Test Results: GJ-17M (60% Sand/40% Slimes)

PS 2/4/88
WYL 2/5/88

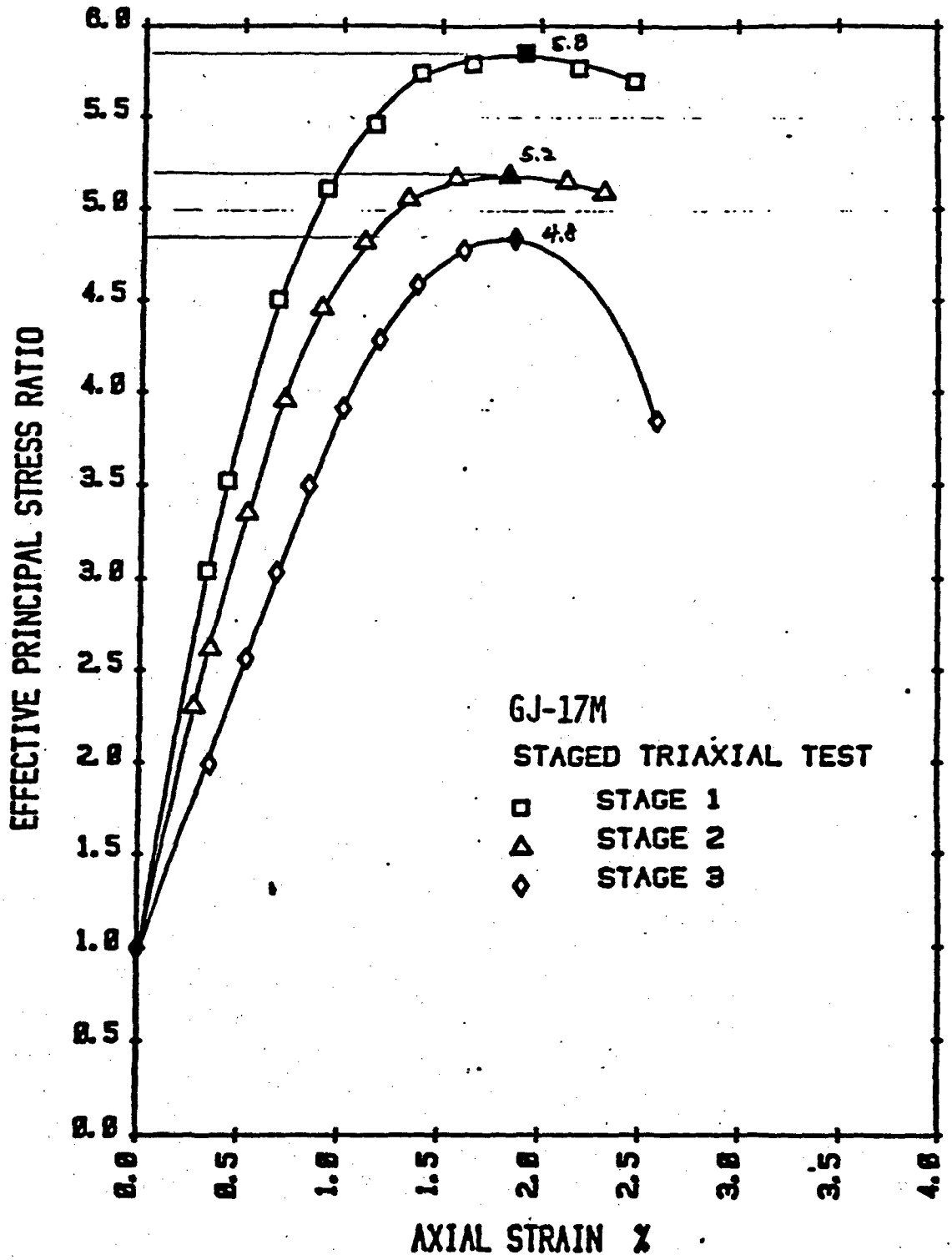


FIGURE B-4: EFFECTIVE PRINCIPAL STRESS RATIO VS. AXIAL STRAIN, GJ-17M

48

PS 2/4/88
WYL 2/5/88

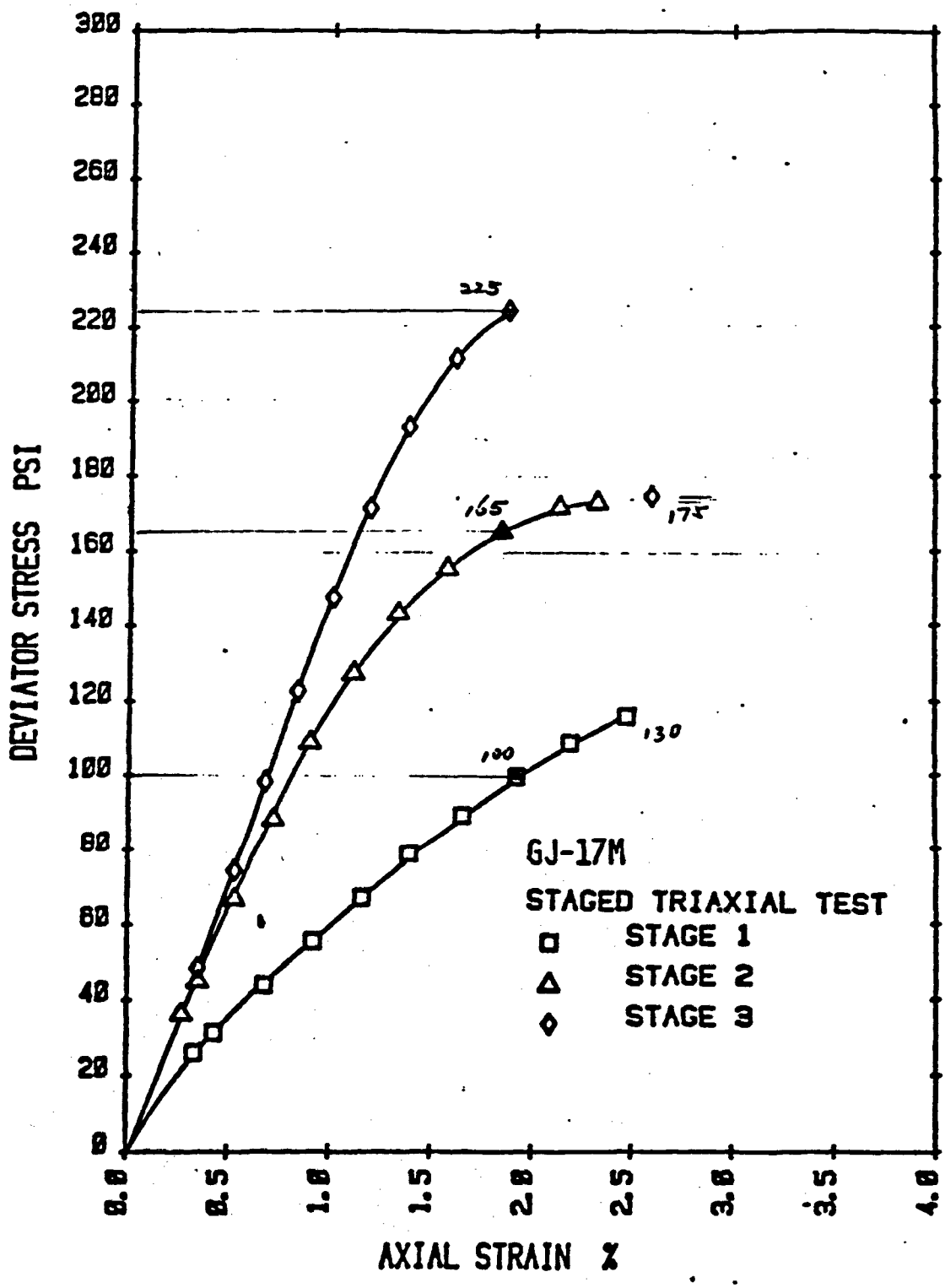
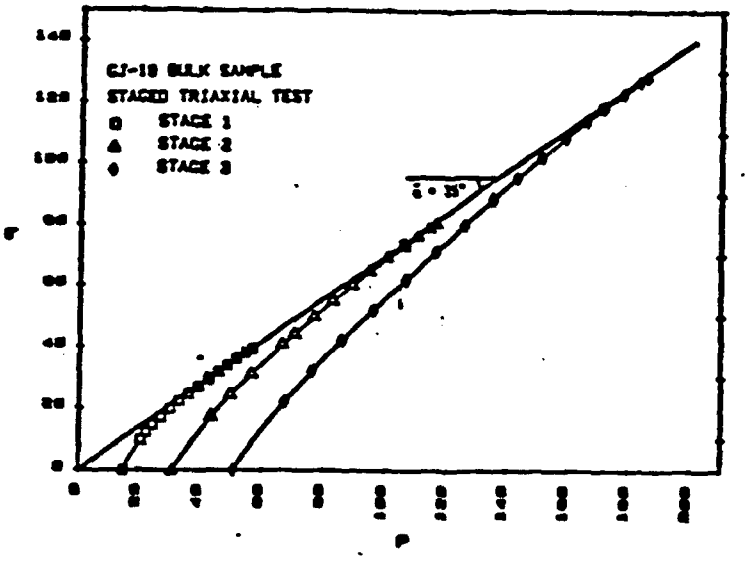


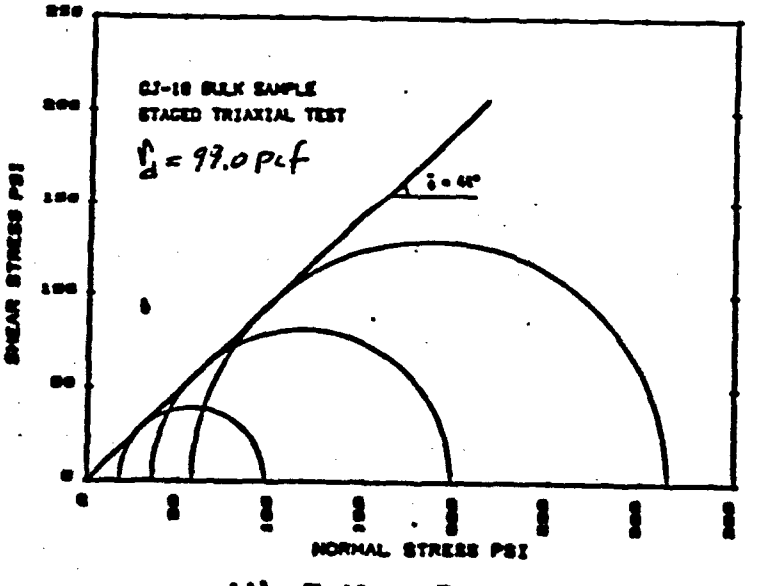
FIGURE B-3: DEVIATOR STRESS VS. AXIAL STRAIN, GJ-17M

S.d
6C/4C

PS 2/4/88
WYL 2/5/88



i) Stress Paths



ii) Failure Envelope

Figure 17C. Stage Triaxial Test Results: GJ-18M (40% Sand/60% Slimes)

BS 2/4/80
WYL 2/5/88

40

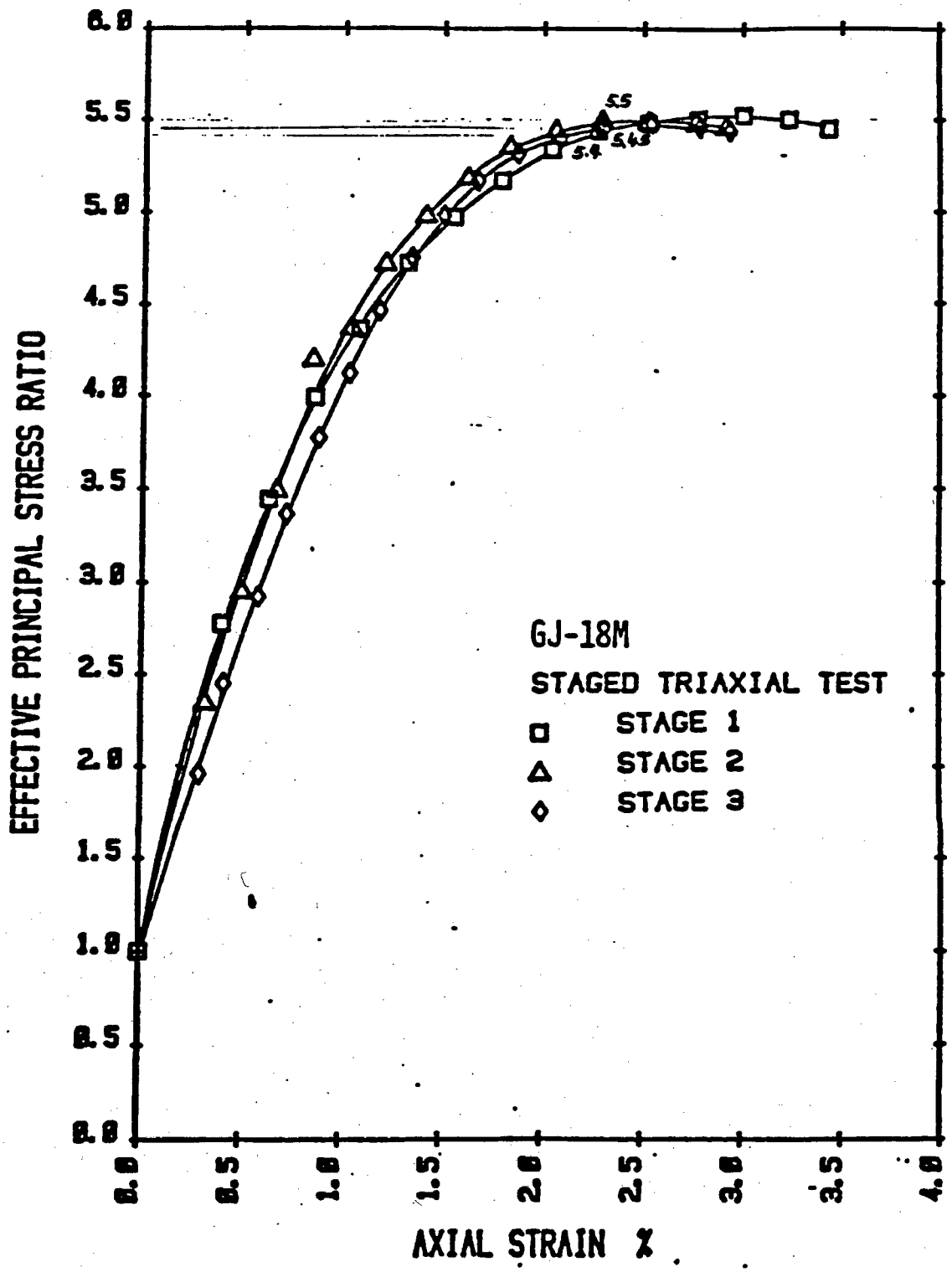


FIGURE B-6: EFFECTIVE PRINCIPAL STRESS RATIO VS. AXIAL STRAIN, GJ-18M

PS 2/4/88
WYL 2/5/88

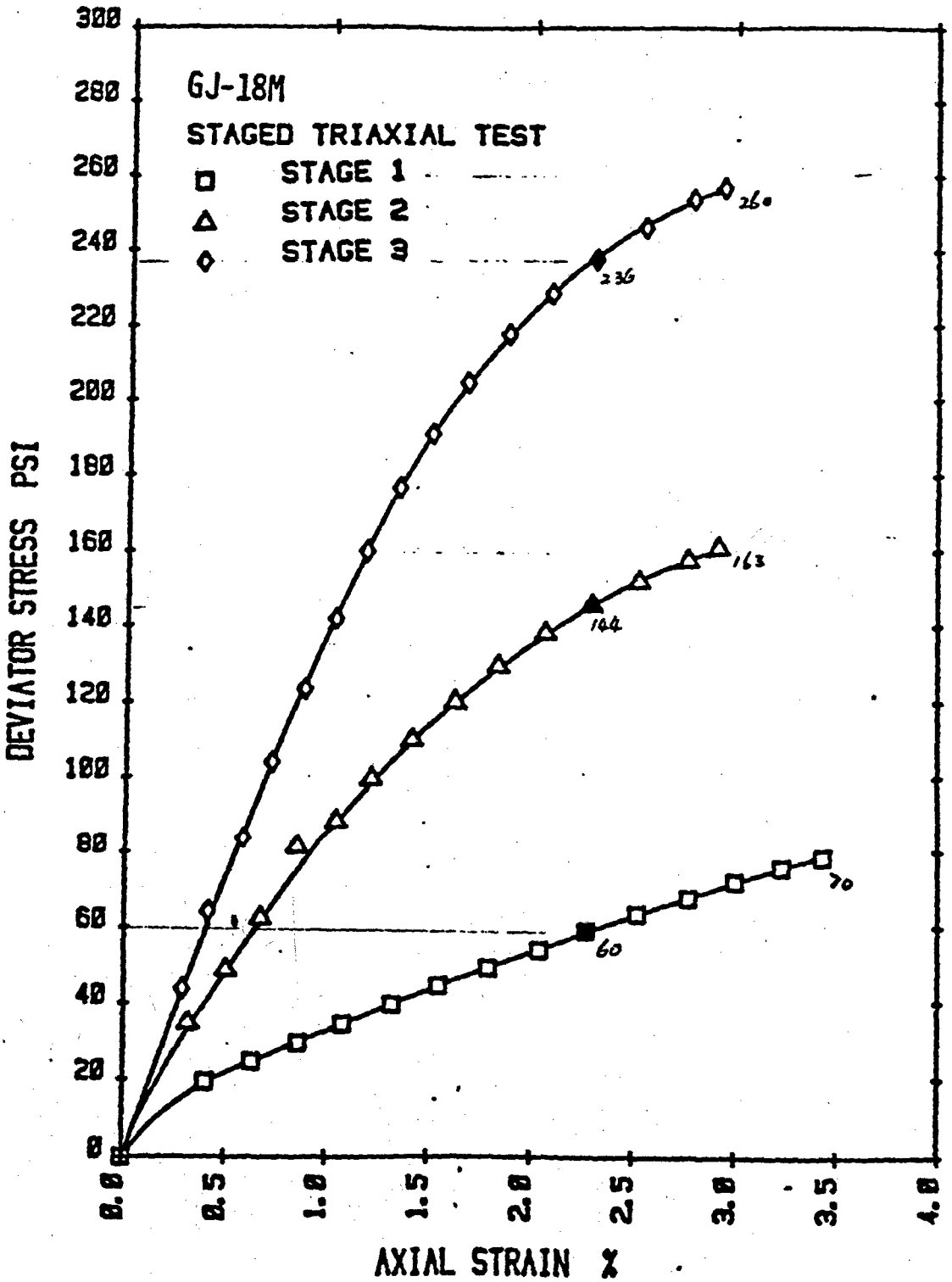


FIGURE B-5: DEVIATOR STRESS VS. AXIAL STRAIN, GJ-18M.

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Feature: Embankment Design
Item: Material Properties

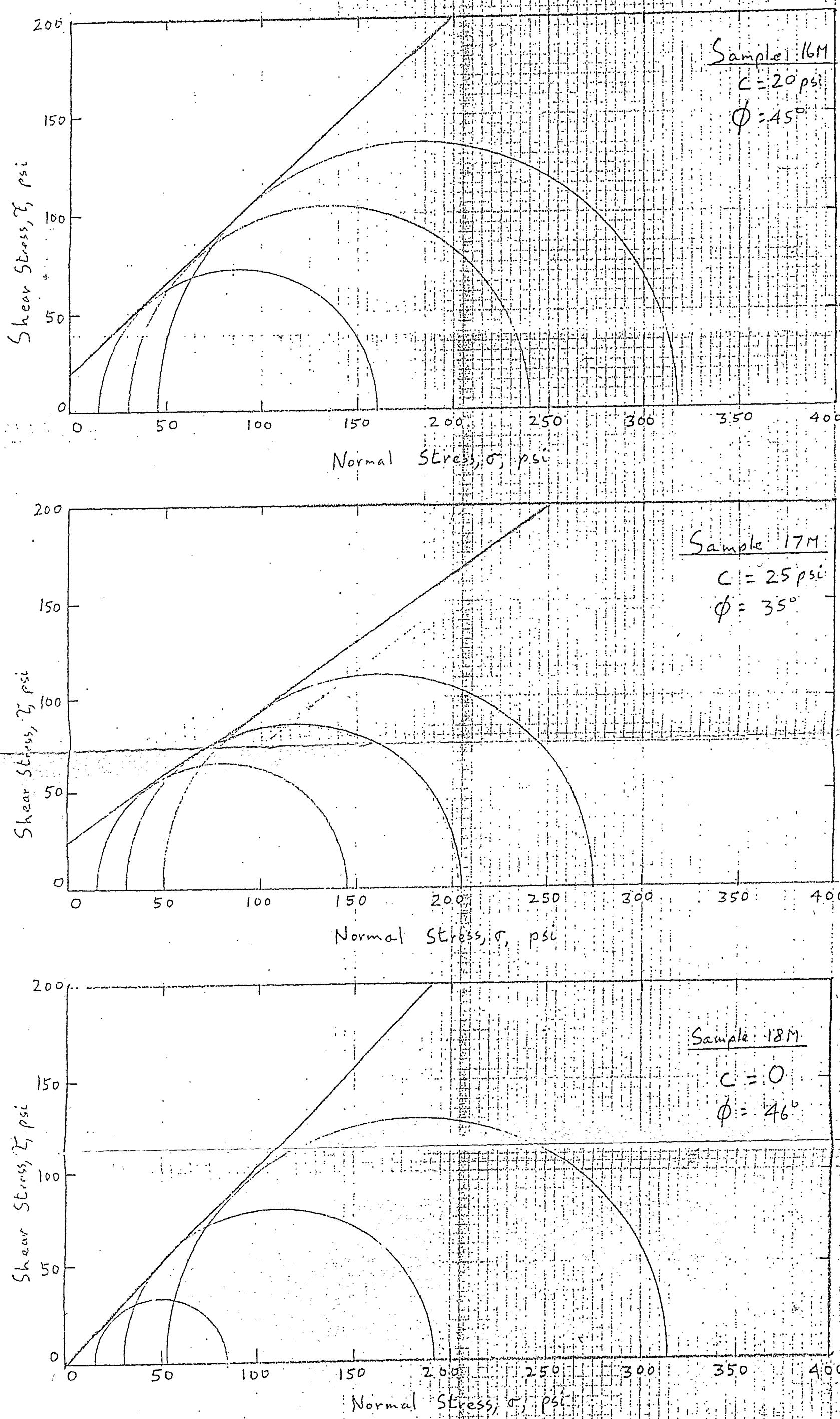
Contract No. 5025
Designed: PS
Checked: WYL

Sheet 52
File No.
Date 2/4/88
Date 2/5/88

Determine c, ϕ values for Cu tails on Compacted tailings

Sample No.	Percent passing #200 sieve	σ_{3c}	$(\sigma_1-\sigma_3)_f$	ϕ	c
16M	16	15	145	45°	20 psi
		30	210		
		45	272		
17M	32	15	130	35°	25 psi
		30	175		
		50	225		
18M	40	15	70	46°	0 psi
		30	163		
		53	260		

The values of σ_{3c} and $(\sigma_1-\sigma_3)_f$ are determined from the p-q data on sheets . ϕ and c are determined from Mohr Circle: the next sheet



PS 2/4/88
WWL 2/4/88

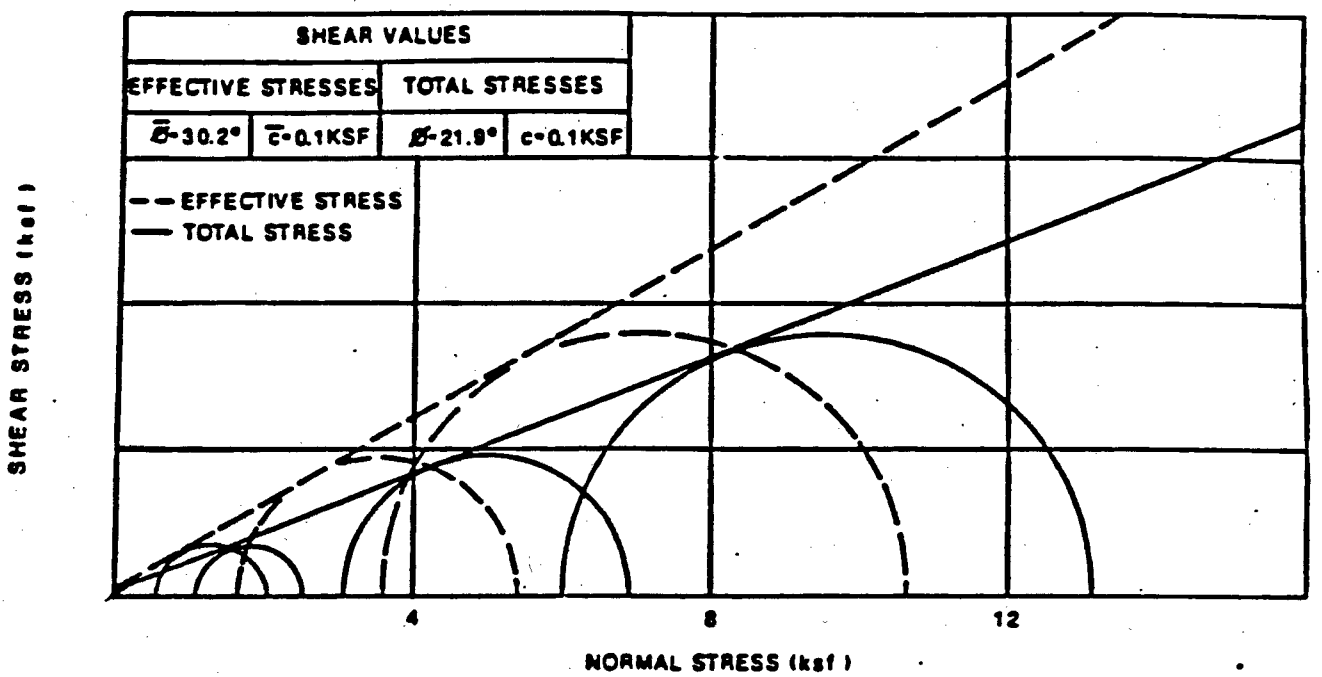
Fig. 3

CU Test Results on Compacted Tailings

(all samples compacted to 100 percent Standard Proctor density)

PS 2/4/88 SK

WYL 2/5/88



STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	SOIL DESCRIPTION
1	60.6	62.3	SILT-CLAY, SANDY
2	62.8	61.8	
3	64.7	58.9	

LOCATION NO.	DEPTH (FT.)	SAMPLE TYPE
421	8.0-10.0	SHELBY

Percent passing #200 sieve = 98%

(Figure E.6.11 – RAP)

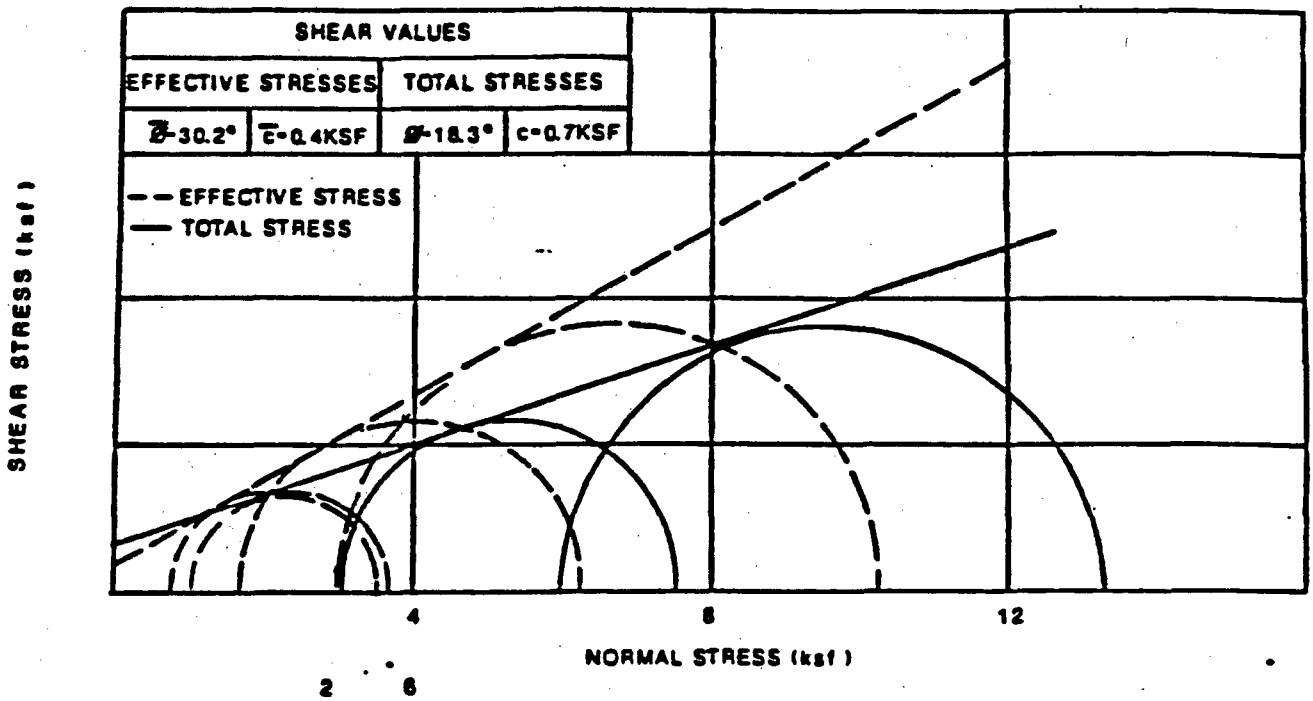
Degree of Compaction ≈ 85 to 91 % ($\gamma_{d_{max}}$ = 71 pcf)

FIGURE D.4.6

SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION

PS 2/1/88 ±5

WYL 2/5/88



STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	SOIL DESCRIPTION
1	64.0	37.2	CLAY-SILT, SANDY
2	64.5	36.6	
3	67.0	54.4	

LOCATION NO.	DEPTH (FT.)	SAMPLE TYPE
417	15.0-17.0	SHELBY

Percent passing #200 sieve = 32 %

(Figure E.6.9 – RAP)

∴ Degree of Compaction = 65 to 70% (γ_d = 98 pcf)

FIGURE D.4.5

SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION

Chen & Associates
Consulting Geotechnical Engineers

Job No. 1 343 87

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

SAMPLE LOCATION		OPT. MOISTURE CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (psf)	SPECIFIC GRAVITY	SOIL OR BEDROCK TYPE
HOLE	DEPTH (feet)			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
BH-~~6~~-29-G	0-20	12.5	116.3	16	26	58	32	18		2.67	Sandy lean clay with gravel
BH-~~6~~-29-G	20-30	11.6	117.9	31	24	45	32	18		2.68	Clayey gravel with sand
9H	5-8	14.1	105.0	0	86	14		NP			Silty sand
21J	5-20	13.3	108.0	0	88	12		NP			Poorly graded sand with silt
21J	20-45	13.2	109.2	0	84	16		NP			Silty sand
33F-B	0-20	13.2	116.2	15	33	52	28	15		2.71	Sandy lean clay with gravel

PS 2/4/88
WYL 2/5/88
JSG

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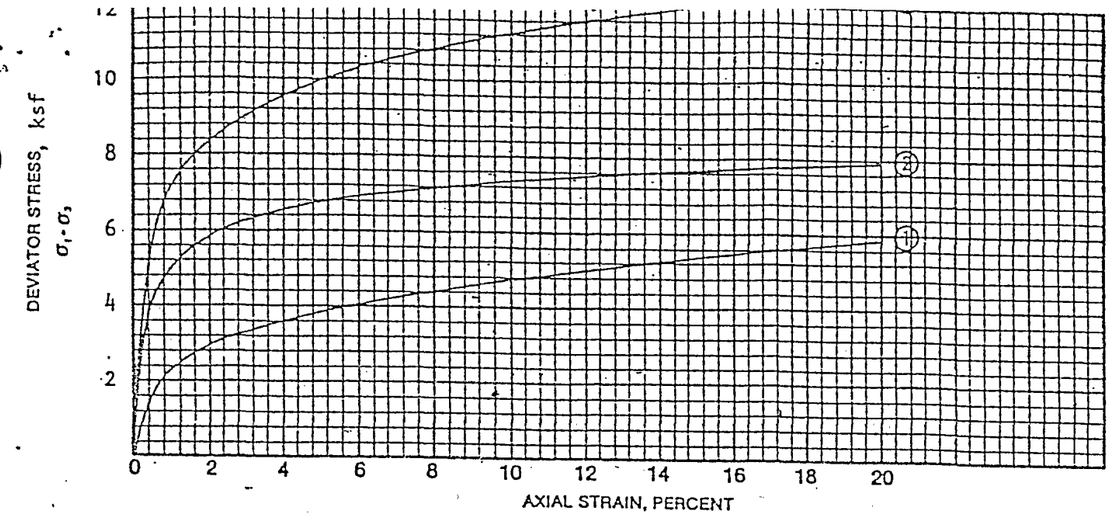
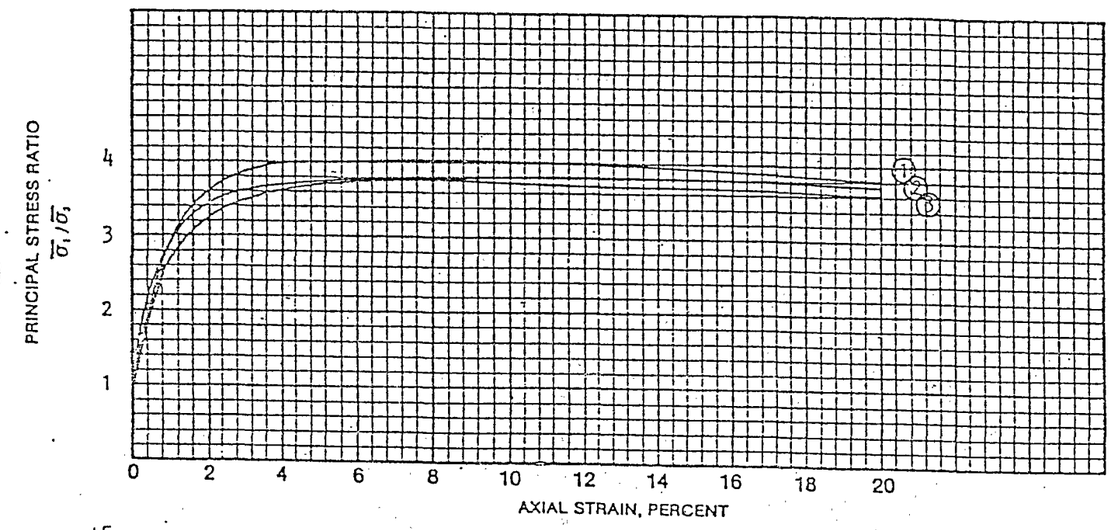
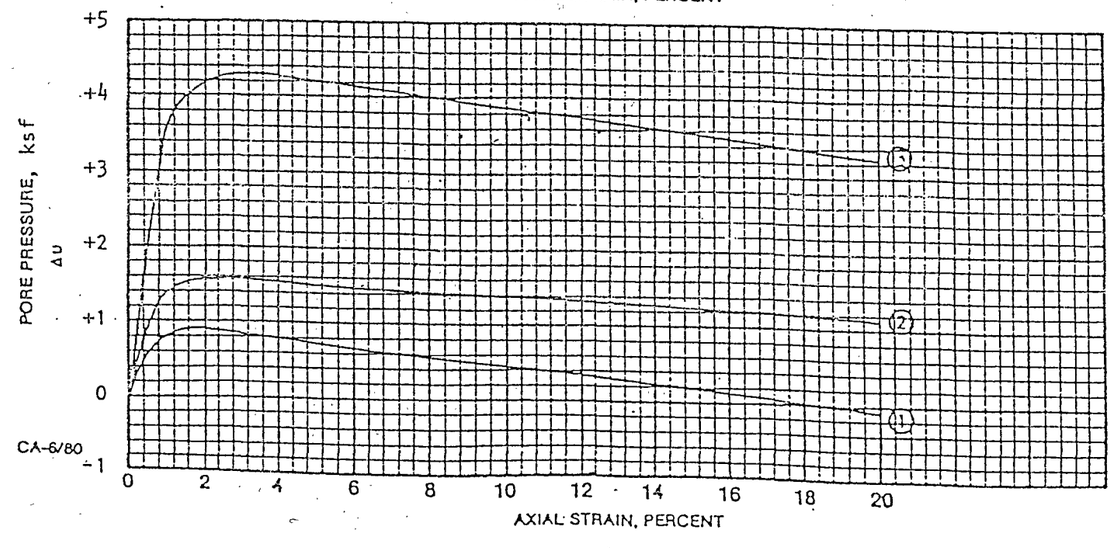






CA-6/80

SHEAR STRENGTH OF SOIL IN TRIAXIAL COMPRESSION

SSb

PS 2/4/88

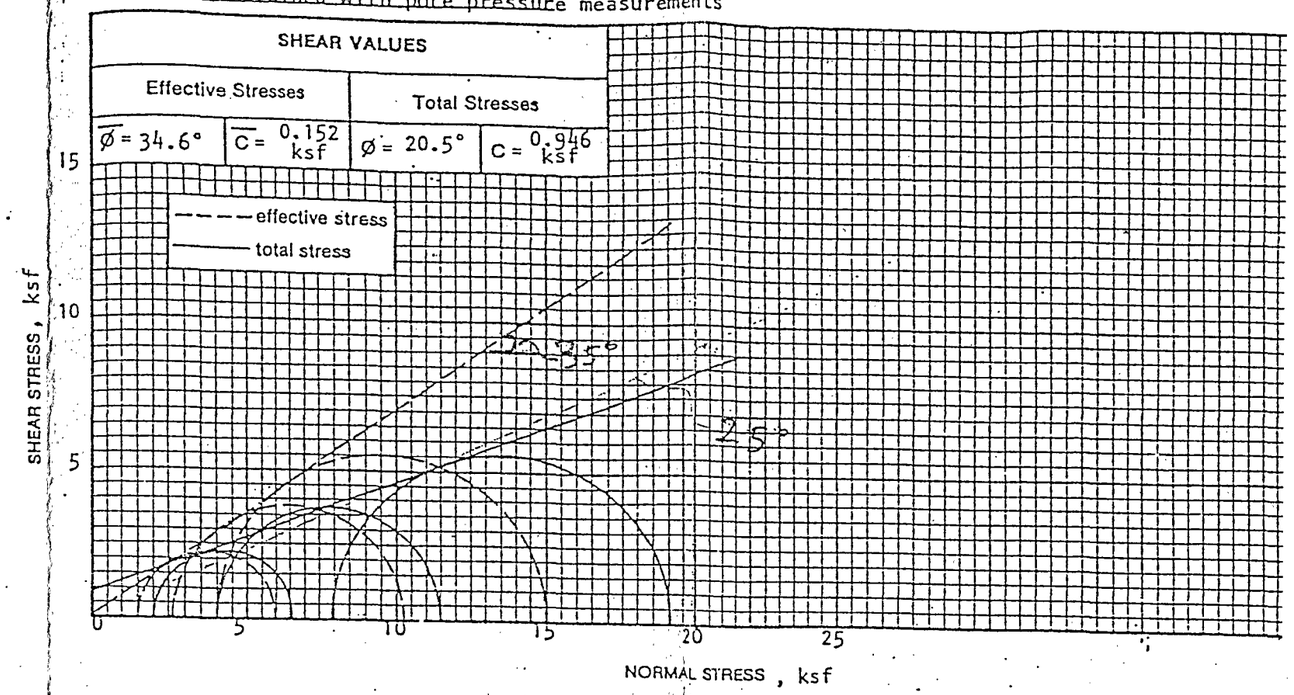
WYL 2/5/88

Job No. 1 343 87

Date June 17, 1987

Type of Test Saturated, consolidated, undrained with pore pressure measurements

SHEAR VALUES			
Effective Stresses		Total Stresses	
$\bar{\phi}$ = 34.6°	$\bar{c}$ = 0.152 ksf	ϕ = 20.5°	c = 0.946 ksf



Specimen Number	Specimen Location		Initial Specimen Data					Soil Description
	Boring Number	Depth (Ft)	Sample Type	Length (in)	Diameter (in)	Dry Density (P.C.F.)	Moisture Content (%)	
1	21J	20-45	Remolded	3.921	1.875	98.0	13.7	silty sand
2	"	"	"	3.927	1.875	97.3	14.3	" "
3	"	"	"	3.937	1.875	97.4	14.1	" "

Specimen Number	"B" Parameter	VALUES PLOTTED AT MAXIMUM PRINCIPAL STRESS RATIO							Remarks
		Total Confining Stress σ_3	Total Axial Stress σ_1	Deviator Stress $\sigma_1-\sigma_3$	Effective Lateral Stress $\bar{\sigma}_3$	Effective Axial Stress $\bar{\sigma}_1$	Pore Pressure μ	Percent Strain ϵ%	
1	0.95	2.016	6.551	4.535	1.498	6.033	0.518	9.005	All units in ksf
2	0.96	3.974	11.559	7.585	2.678	10.264	1.296	14.009	" "
3	0.99	7.934	18.982	11.048	3.974	15.022	3.960	9.005	" "

Remarks:

Fig. 14

Consolidation Properties

Settlement Property Computations

One-dimensional compression curves for recompacted tailings materials are presented in sheet 61 through 63. The curves also include the effects of wetting on compression

Assessment of settlement will require

- Compression Index, C_c
- Recompression Index, C_r
- Past preconsolidation pressure (P_p), separating compression and recompression
- Coefficient of Consolidation, C_v, to assess the time rate of settlement
- Effect of wetting on settlement

The properties are examined in the following pages and design values for the 'average material' determined, which is summarized below

For tailings materials

$C_c = 0.20$

$C_r = 0.02$

$P_p = 1$ ksf

$C_v = 0.001\ ft^2/sec$

Consolidation Properties for main pile materials

Determination of Compression Index, C_c, for Design

The compression index, C_c for the compacted tailings from the 1-D compression tests are plotted on Pg. 65 vs. degree of compaction. The C_c vs degree of compaction @ 90% compaction vs -#200 sieve are plotted on Pg 66. For a material with 40% passing #200 sieve, $C_c = 0.17$. Use $C_c = 0.20$ for design

conservative

Determination of Recompression Index, C_r, for Design

Values of C_r are determined from test results and tabulated on page 58. A value of 0.020 is recommended for design. This value represents that for sands and sand-slimes @ near 90% compaction for the saturated case.

Determination of Design Maximum Past Pressure

An examination of the compression test curves indicate the presence of a pre-consolidation pressure (maximum past pressure). For the 'design average material', the maximum past pressure at placement may be taken as 1 KSF.

TABLE 5

One-Dimensional Compression Test Results

Sample	Test	Initial Dry Density γ_D, pcf	% of Std Proctor Max Density	Initial Water Content, w,%	Initial Void Ratio, e	Initial Saturation S,%	Water Application	Wetting Collapse $\frac{\Delta H}{H}$, %	Total Deformation 0-17,600 psf $\frac{\Delta H}{H}$, %	C_r	Consolidation Index, C_c
GJ-213B (sand)	1	90.0	82.4	6.0	0.87	0.18	Initial	9.3	14	19.1	0.12
	2	99.1	90.8	11.4	.70	0.44	Initial	0.5	14	6.1	.020 0.09
GJ-209B (slimes)	1	57.8	69.0	28.9	1.98	0.40	End	8.7	70	34.1	.008 0.53
	2	69.4	82.8	28.9	1.48	0.54	End	1.6	70	14.7	.010 0.32
	3	81.3	97.0	35.2	1.11	0.81	Initial	1.0	70	15.7	.034 0.17
	4	87.3	104.2	35.2	0.97	1.00±	Initial	-	70	8.4	.015 0.10
GJ-17M (sand-slimes)	1	87.9	82.5	17.2	0.94	0.50	Initial	1.3	32	17.8	0.20
	2	95.3	89.5	19.8	0.79	0.69	Initial	0.7	32	8.6	.015 0.13
	3	99.3	93.2	18.8	0.72	0.72	End	1.3	32	9.6	.009 0.14

Notes:
1) All samples settled quickly upon load application.
2) Some creep observed.
3) Reloaded at 4-10 hour increments after deformation stabilized.
4) Samples cured in consolidation ring one hour before initial loading.

PS 2/4/88
WYL 2/5/88

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Determination of Coefficient of Consolidation, C_v, for use in settlement computations in the tailings embankment

Purpose: To aid in the determination of time rate of consolidation settlement, C_v is required.

Formula:

$$C_v = \frac{K\,(1+e_o)}{a_v\,\gamma_w}$$

where e = void ratio (e_o = initial void ratio)

γ_w = unit weight of water = .0624 kcf

$a_v = -\frac{\Delta e}{\Delta p}$ = slope of e-log p curve

p = vertical applied stress

k = permeability

Assumptions: (1) The 'average' material in the embankment is assumed to be equivalent to sample GJ-18M (0-3 ft), which has 40 percent passing #200

(2) The stress range of interest to us is 1 ksf to 4 ksf (upto 40 ft of depth)

(3) Permeability, k = 5×10^{-5} cm/sec.

$$\frac{a_v}{a_v} = -\frac{de}{dp} = \left(-\frac{de}{d(\log p)}\right) \cdot \frac{d(\log p)}{dp}$$

$$= C_c \left(\frac{1}{2.303} \cdot \frac{d(\ln p)}{dp}\right)$$

$$= \frac{1}{2.303} \cdot C_c \cdot \frac{1}{p}$$

Assuming a 'conservative' value of $C_c = 0.20$
$p = 2$ ksf (avg.)

$$a_v = \frac{1}{2.303} \times 0.20 \times \frac{1}{2} = 0.0434 \text{ (for } p = 2 \text{ ksf)}$$

$$= \frac{1}{2.303} \times 0.20 \times \frac{1}{4} = 0.0217 \text{ (for } p = 4 \text{ ksf)}$$

$$C_v = \frac{K(1+e_0)}{a_v \gamma_w}$$

$$= \frac{5 \times 10^{-5} \times \frac{1}{30.48} \times (1+0.8)}{0.0434 \times .0624} = 0.0011 \text{ ft}^2/\text{sec for } p=2$$

$(\approx 1.0 \text{ cm}^2/\text{sec})$

$= 0.0022 \text{ ft}^2/\text{sec for } p=4$

$(\approx 2.0 \text{ cm}^2/\text{sec})$

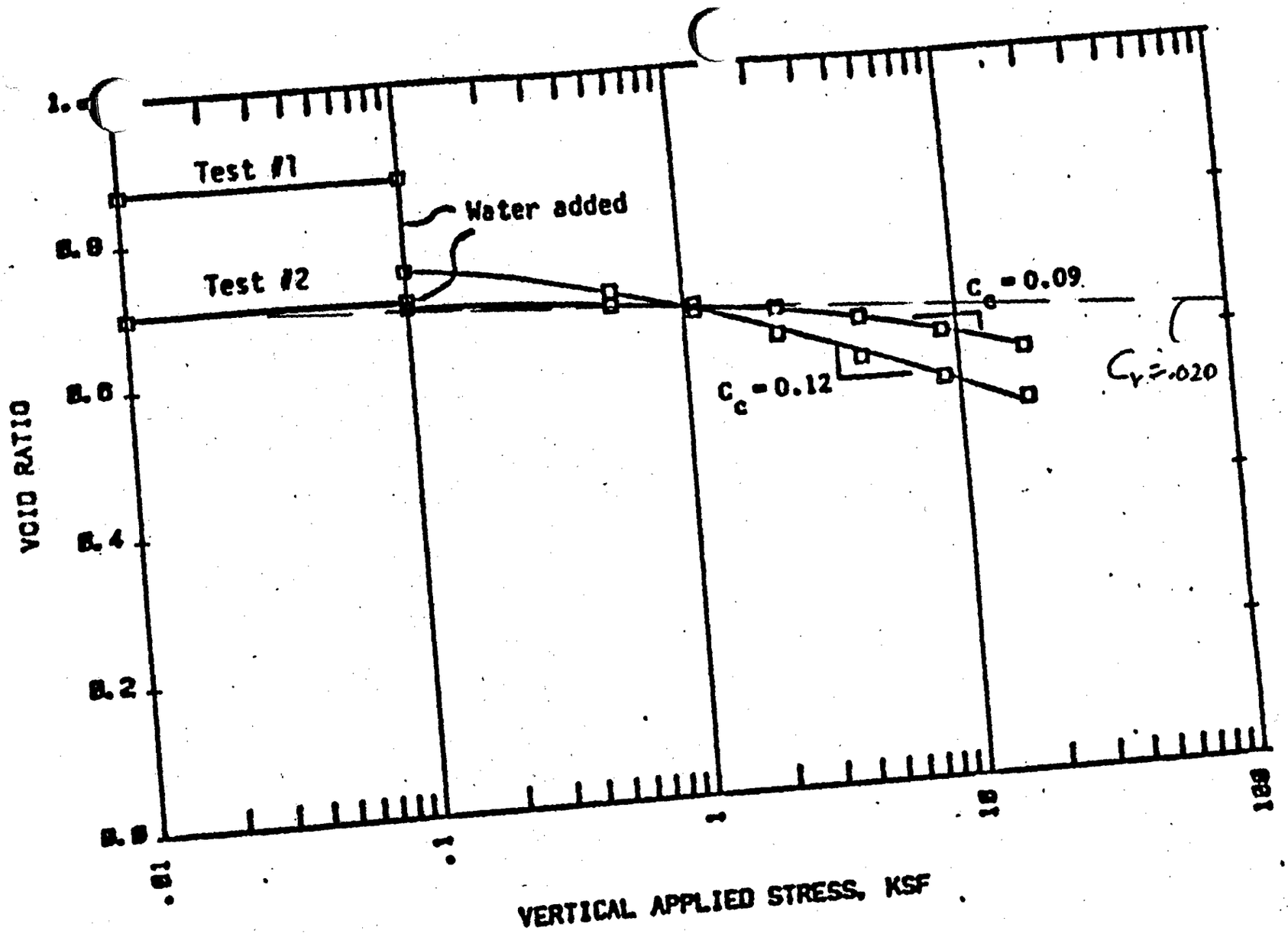
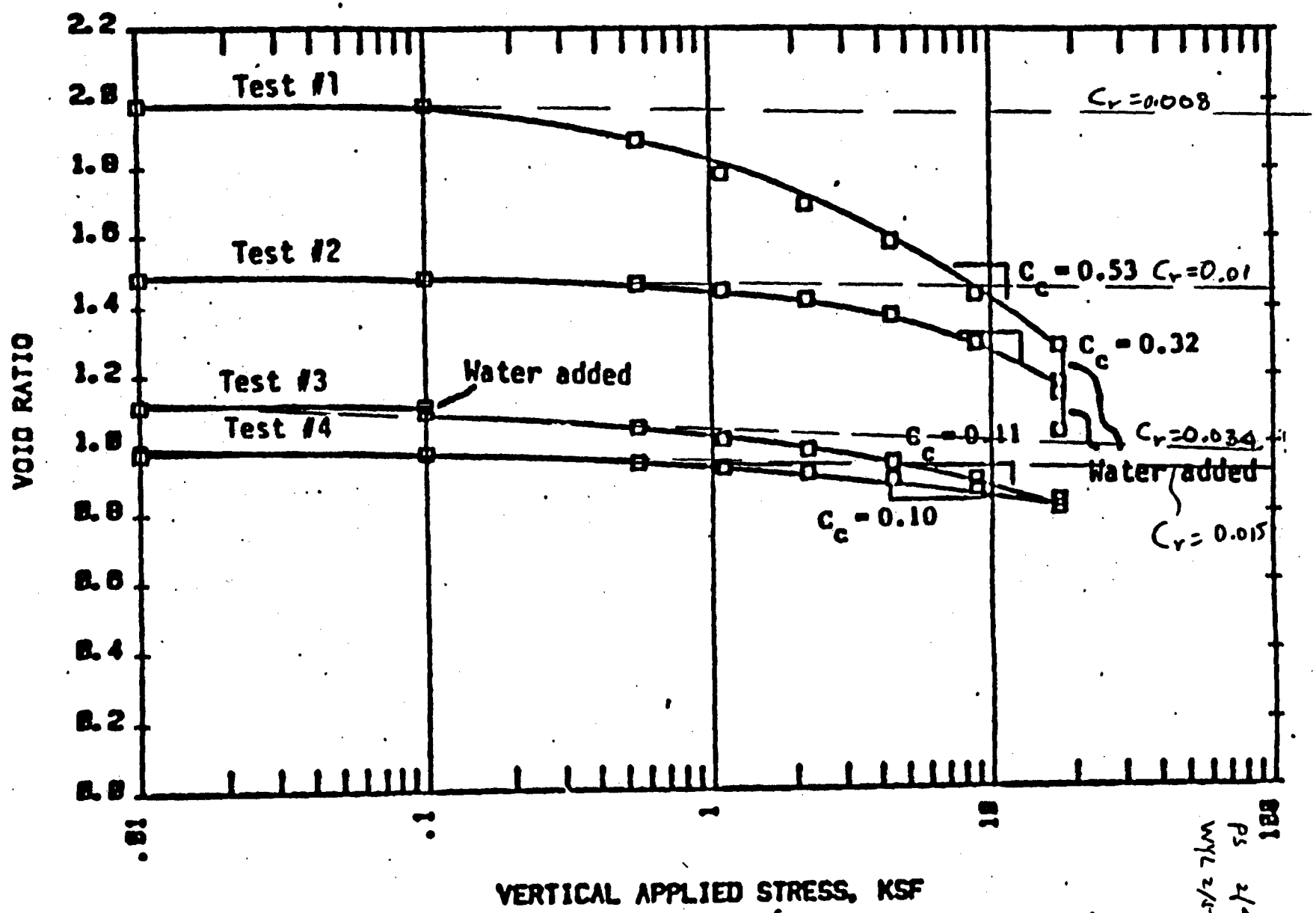


Figure 15A. One Dimensional Compression Curves: Field Sample GJ-213B (Beach Sand).

WYL 2/4/88
PS 2/4/88



Field Sample GJ-209B (Predominantly Slimes)

WJL 2/17/88
PS 2/19/88

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Figure 15B. One Dimensional Compression

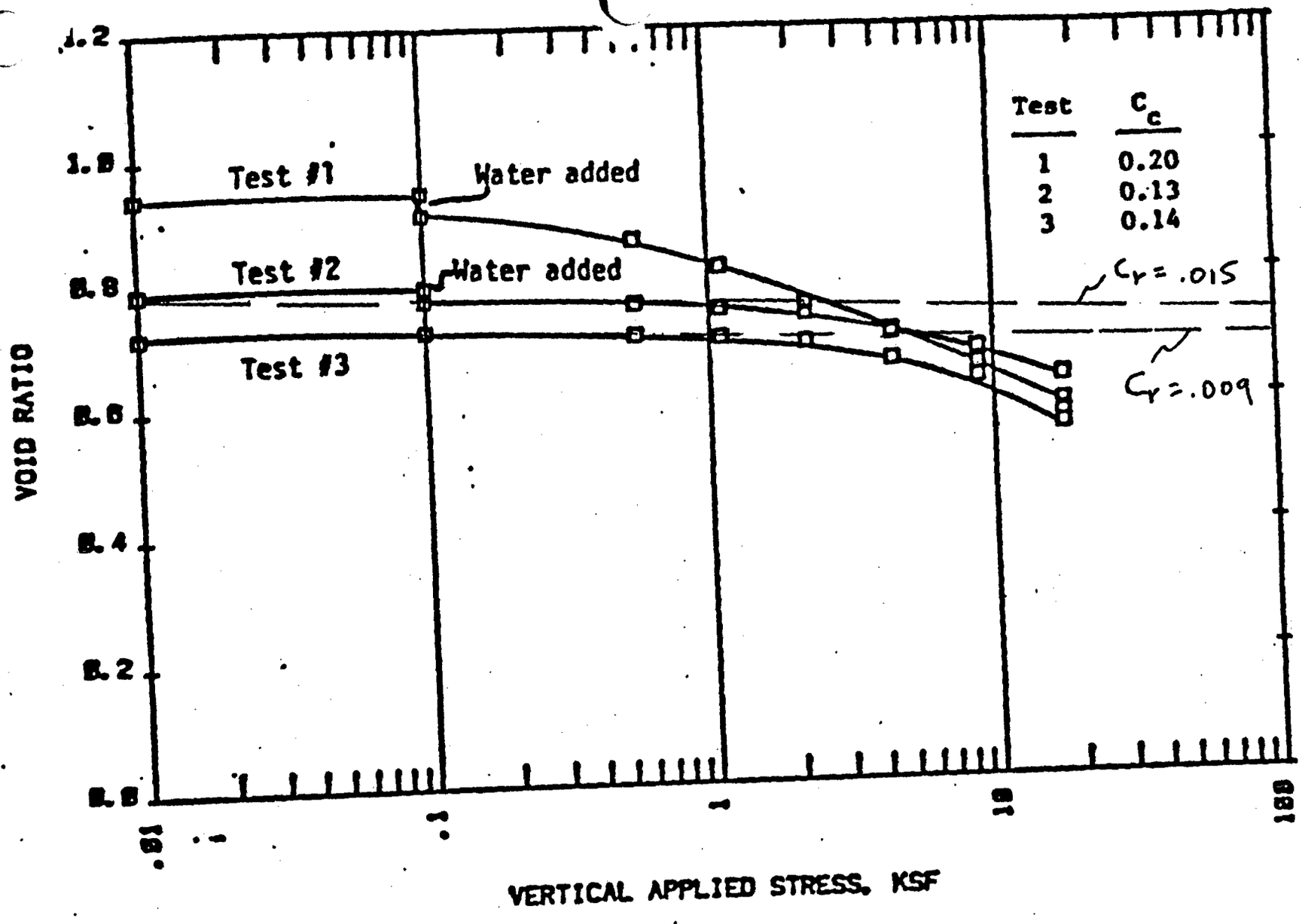
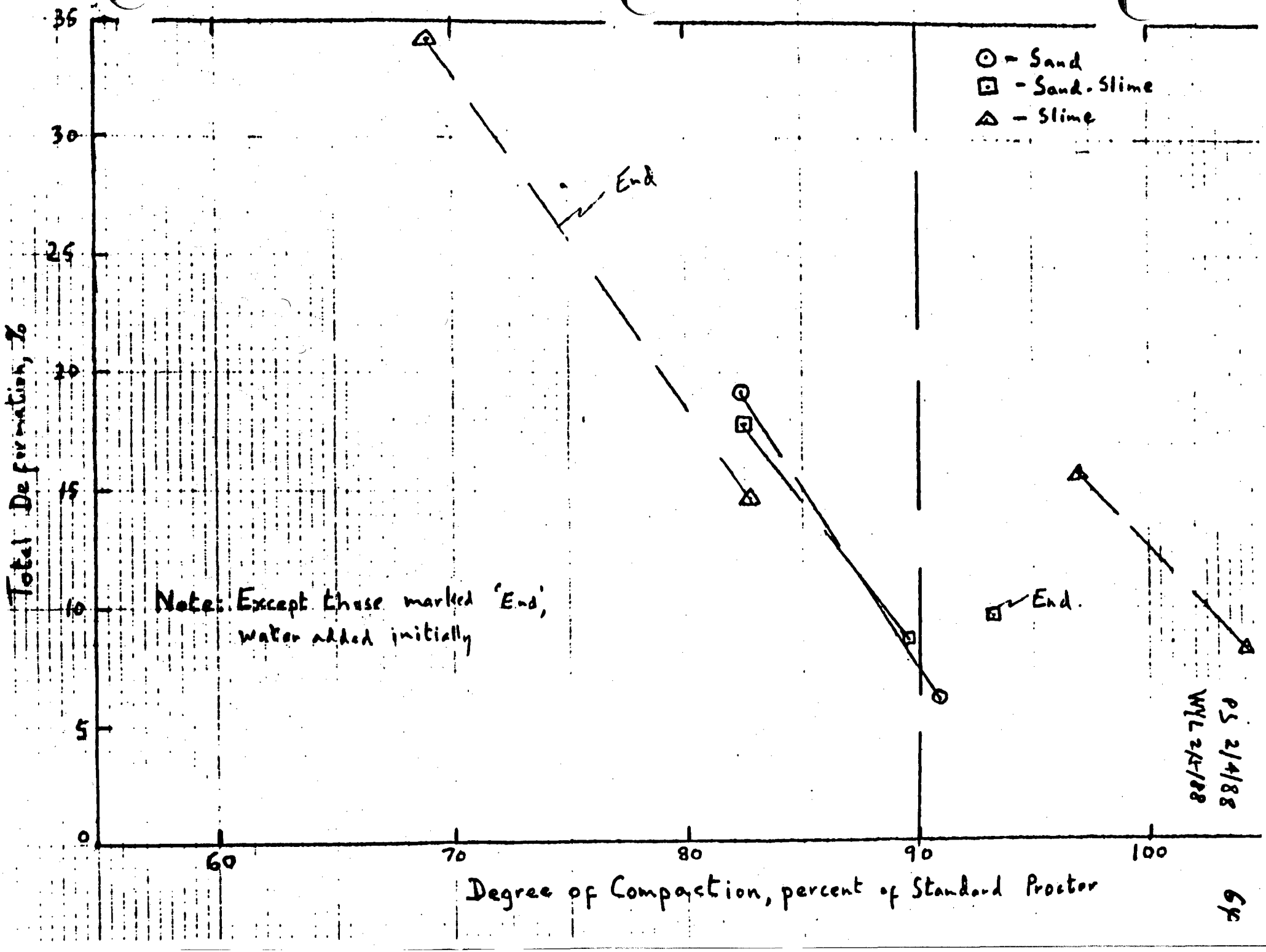
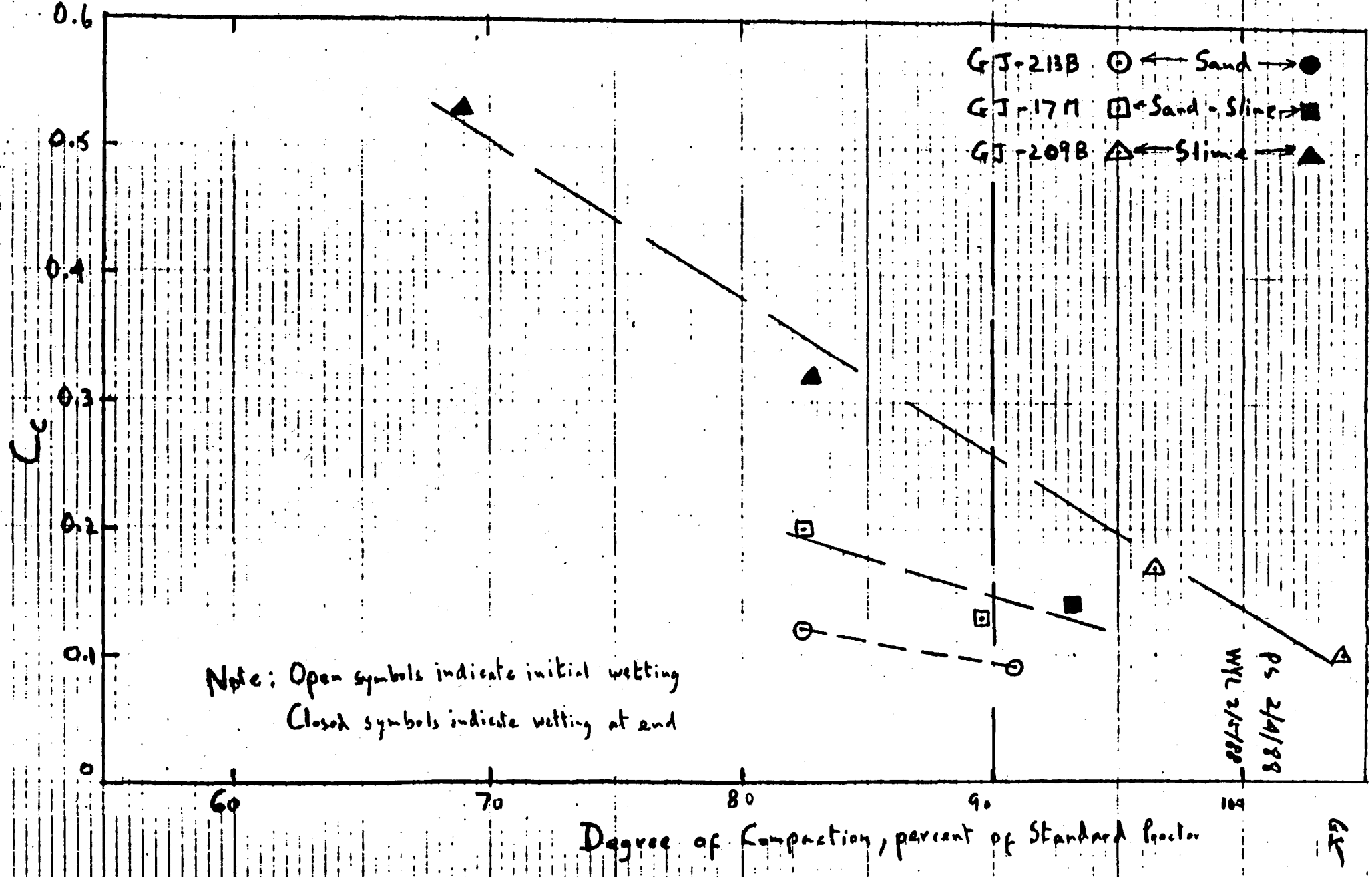
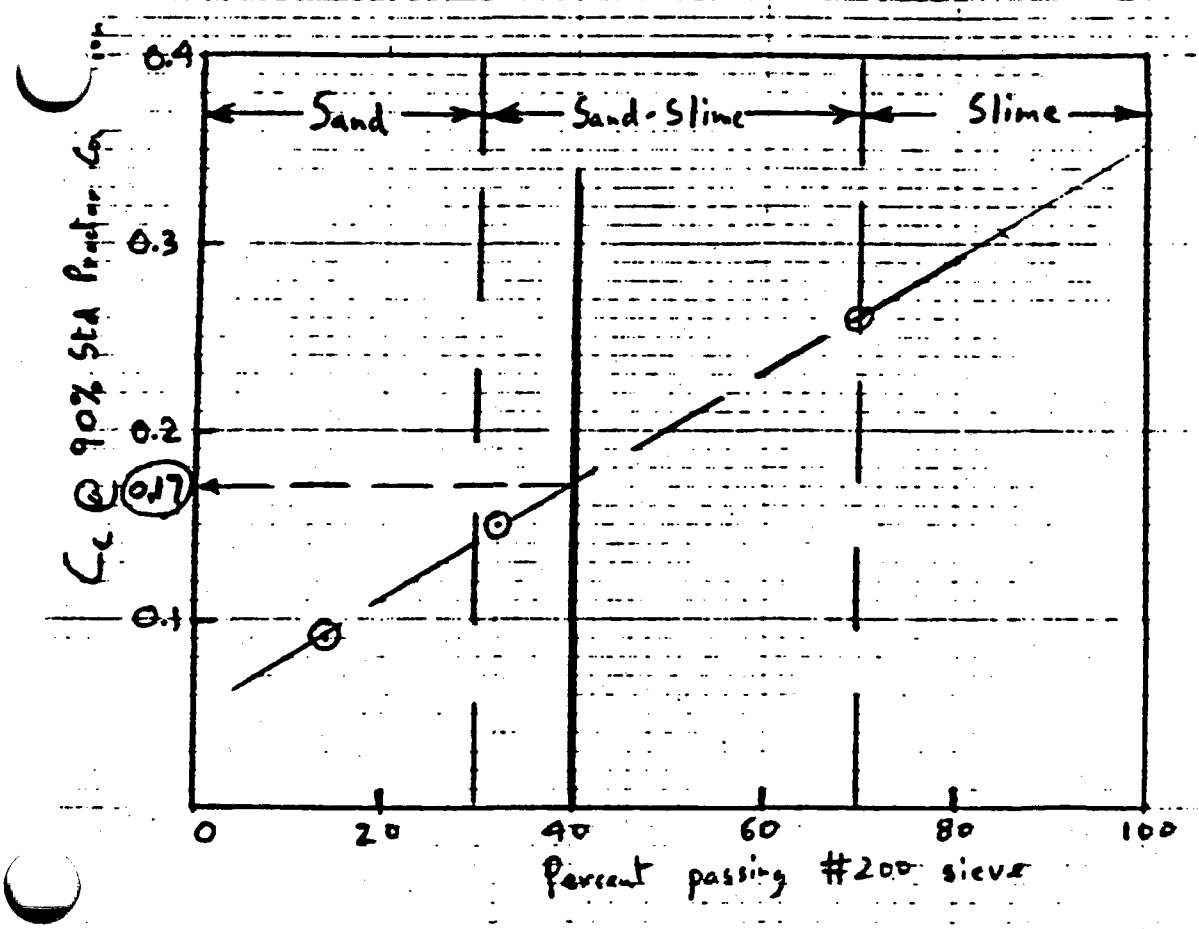


Figure 15C. One Dimensional Compression Curves: Laboratory Sample GJ-17M (Sand-Slime Mixture).


Total Deformation, %
Degree of Compaction, percent of Standard Proctor
0
5
10
15
20
25
30
35
60
70
80
90
100
⊙ - Sand
⊡ - Sand - Slime
△ - Slime
End
End.
Note: Except those marked 'End', water added initially
PS 2/4/88
WYL 2/4/88


GJ-213B ← Sand →
GJ-17M ← Sand - Slime →
GJ-209B ← Slime →
Note: Open symbols indicate initial wetting
Closed symbols indicate wetting at end
C_v
0.6
0.5
0.4
0.3
0.2
0.1
0
60
70
80
90
100
Degree of Compaction, percent of Standard Proctor
WYL 2/17/88
PS 2/4/88

PS 2/4/88
WYL 2/5/88



INTERNATIONAL ENGINEERING COMPANY, INC.

Project _____ Contract No. _____ Sheet _____ File No. _____
Feature _____ Designed _____ Date _____
Item _____ Checked _____ Date _____

Permeability

Purpose

The purpose of these computations is to come up with appropriate design inplace permeability values for the embankment materials, e.g. the compacted tailings from the main pile, ponds and vicinity property.

Available Data

Available data consist of saturated permeability tests on
— Undisturbed Samples (See Sheet 68)
— Remolded Samples — 3 fabrics (See Sheet 69)

These data are obtained from Colorado State Univ. Report (MKE Document No. 5025-GRJ-R-03-00055-00)

Evaluation Process and Commentary

The measured permeabilities are plotted against percent passing #200 sieve, as shown on Sheet 70. As is to be expected, the permeability decreases with increasing percentage of fines (material passing #200 sieve).

The in-situ material will have a structure similar to that of fabric C. The in-place materials in the embankment will have a structure similar to fabric A. The comparison for effect of fabric indicate that in 5 of 6 cases, fabric A has the lowest permeability value and in all cases, the highest degree of compaction. The pattern of permeabilities for permeabilities on undisturbed samples is unclear.

Summary

Design Permeability (cm/sec) — Main Pile Material (40% passing #200) = 7.0×10^{-5} cm/sec

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PS 2/4/88
WYL 2/5/88

TABLE 6

Permeabilities of Saturated Undisturbed Samples

Boring	Depth	Dry Density γ_d	%-#200 Sieve	Permeability, k, cm/sec	% Compaction
GJ- 22	10.2'	98.2	5.9	1.1×10^{-2}	89
GJ- 24	6.5'	65.5	82.0	4.6×10^{-5}	83
GJ-201	14.0'	80.0	23.0	3.6×10^{-5}	78
GJ-207	4.0'	84.6	7.0	4.3×10^{-4}	77
GJ-207	19.0'	74.1	15.2	1.2×10^{-4}	70
GJ-209	14.0'	93.9	79.8	1.3×10^{-5}	115 (?)
GJ-214	14.0'	99.6	4.2	1.1×10^{-4}	90
GJ-216	14.0'	100.0	41.4	4.4×10^{-4}	105
GJ-220	14.0'	58.3	78.0	6.3×10^{-5}	75
GJ-220	19.0'	89.4	12.0	1.3×10^{-4}	83
GJ-220	254.0	91.8	36.7	2.6×10^{-4}	86

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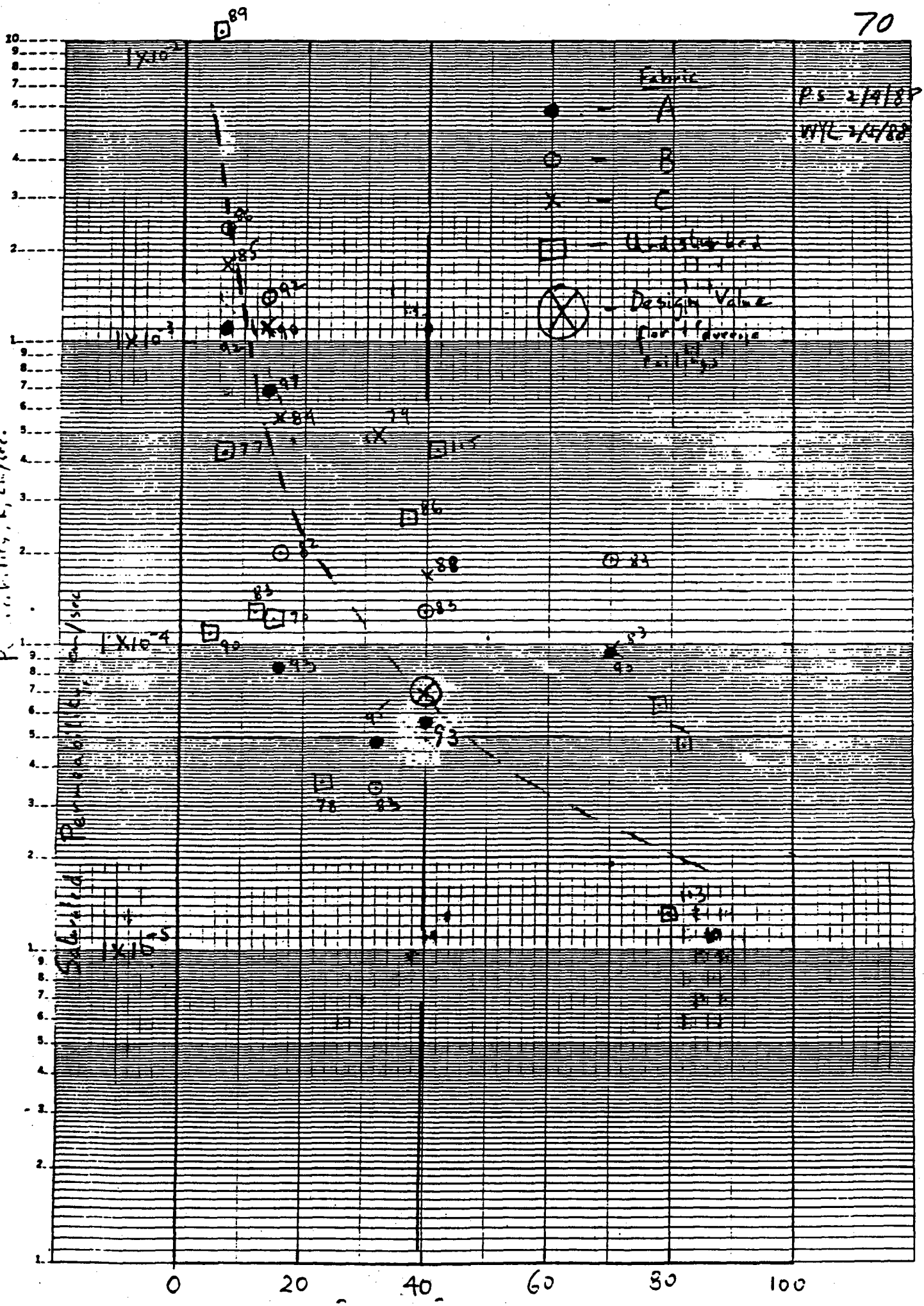
PL 2/4/85
WYL 2/5/88

TABLE 7

Permeability of Saturated Remolded Field and Laboratory Mixtures

Sample			% <150	Dry Density[1] γ_d, pcf	Compact. %	Permeability k, cm/sec
Field Samples						
GJ-213B	1)	Fabric A[2]	14	104.8	97	6.9×10^{-4}
	2)	Fabric B		99.8	92	1.4×10^{-3}
	3)	Fabric C		97.3	90	1.1×10^{-3}
GJ-209B	1)	Fabric A	70	74.9	90	9.3×10^{-5}
	2)	Fabric B		68.6	83	1.9×10^{-4}
	3)	Fabric C		69.1	83	9.6×10^{-5}
GJ-214B	1)	Fabric A	7.2	99.8	92	1.1×10^{-3}
	2)	Fabric B		94.2	86	2.4×10^{-3}
	3)	Fabric C		93.0	85	1.8×10^{-3}
Laboratory Mixed Samples						
GJ-16M	1)	Fabric A	16	98.6	93	8.5×10^{-5}
	2)	Fabric B		87.3	82	2.0×10^{-4}
	3)	Fabric C		88.6	84	5.6×10^{-4}
GJ-17M	1)	Fabric A	32	93.6	95	4.8×10^{-5}
	2)	Fabric B		81.1	83	3.4×10^{-5}
	3)	Fabric C		77.4	79	4.8×10^{-4}
GJ-18M	1)	Fabric A	40	89.2	93	5.6×10^{-5}
	2)	Fabric B		79.2	83	1.3×10^{-4}
	3)	Fabric C		84.2	88	1.7×10^{-4}

Note: 1) Final density after wetting collapse
~~2) See text for fabric description~~
2) Fabric A – dynamically compacted
B – placed loose or lightly compacted; subsequent densification by overburden or underdrainage
C – grain by grain deposition through series of screens; effect similar to hydraulic fill



PS 2/4/88
WYL 2/4/88

K+E SEMI-LOGARITHMIC 46 6013 KEUFFEL & ESSER CO. MADE IN U.S.A.

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project _____ Contract No. _____ Sheet _____ File No. _____
Feature _____ Designed _____ Date _____
Item _____ Checked _____ Date _____

Off-Pile Tailings and Vicinity Properties

REVISION 3

OFF-PILE TAILINGS (POND AREA MATERIALS

Based on Grain Size Analysis completed recently on sheet 73A, and compared with previous Grain Size tests on sheet 73, use Geotechnical Properties from the previous calculation.

Also, off-pile tailings will be compacted to 95 pct of ASTM D-698 (and not 100 pct, as per Revision 2). In addition, off-pile tailings shall occupy a minimum of 10 feet of the embankment top (see Sheet 3). The rest of the embankment shall include both mainpile and off-pile materials.

PROPERTIES OF POND AREA MATERIALS

Introduction

The Draft RAP did not have any information regarding the contaminated pond area soils and vicinity property materials except the soil boring logs reported in the Bendix report (MKE. Doc. No. 5025-GRJ-R-03-00555-00. The boring logs indicated soils to be primarily sandy and silty clays, e.g. CL and CH material. Thereby, it was assumed that these materials would exhibit the properties for slimes. An 85 percent value of passing through #200 sieve was adopted

A field investigation and laboratory testing program were perform to better define the properties of these materials. Three soil borings were drilled in the area, and a total of three bucket samples recovered. The following tests were performed in the laboratory.

- Grain-Size Analyses, including hydrometer analyses
- Liquid and Plastic Limits
- UU tests on samples compacted at 90, 95 and 100 percent Standard Proctor
- CU tests on samples compacted at 90 percent Standard Proctor
- Consolidation tests - - - - - - 90 percent - - -
- Permeability tests - - - - - - 90 percent - - -

The results of all these tests are summarized on the next sheet.

The soils did not reflect the presence of slime type material. In the following pages, design values of relevant parameters are developed. The soils are primarily sandy, gravelly clays. The design values are summarized in Sheet 72.

Finally, it should be noted that the properties are based on only limited data, and new material from the vicinity project areas are continuously being deposited. It is hard to say however, at this time to what degree the properties developed herein represent the properties for the entire soils layer.

Summary of Design Soil Properties

The design soil properties developed in the following pages are presented below.

Dry Density, γ_d	=	~~116.8 pcf~~	111.0 pcf
Optimum moisture content	=	~~12.4 pct~~	12.4 pct
Specific Gravity	=	~~2.69~~	2.69
Moist Density	=	~~131.3 pcf~~	124.8 pcf
Saturated Density	=	~~135.8 pcf~~	132.1 pcf

REV. 3
H.L. 6/26/90
P.S. 6/26/90

Short Term (Undrained Shear Strength)

C = 800 psf

ϕ = 0

Long Term (Drained or Effective Shear Strength)

REV. 3
H.L. 6/23/90
P.S. 6/26/90

	STATIC	SEISMIC
C'	= 0	$C = 0$
ϕ'	= 30°	$\phi = 18.5°$

from sheet 83

Consolidation Characteristics

$C_{c\varepsilon}$ = 0.160

$C_{r\varepsilon}$ = 0.005

C_v = 2.31×10^{-5} ft²/sec = 0.0214 cm²/sec

P_p = 2.2 ksf

Permeability = 2×10^{-6} cm/sec

SUMMARY OF SOIL PROPERTIES FOR PONDS AREA MATERIALS

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project: UMTRA - GRJ
Feature:
Item:

Contract No. 5025 | Sheet | File No. | Designed PS | Date 1-27-88 | Checked WYL | Date 2/5/88

Sample Identification	Laboratory Classification	Mechanical Analysis: Gravel	Sand	Fines	Atterberg Limits: LL	PI	Specific Gravity G	Natural: W %	γ_d	Compaction: Test	Optimum W	Optimum γ_d	Shear Strength: Test	Initial W	Initial γ_d	C	ϕ	Permeability: γ_d	k	Consolidation: C_c	C_{RC}	E	Notes
BH 29-G (0-20')	Sandy Lean Clay with gravel	16	26	58	32	18	2.67				12.5	116.3	UU	13.9	104.7	0.210 Ksf		164.1	7.4×10^{-4} cm/sec	0.160	0.005	2.2 Ksf	
"													UU	12.8	110.0	0.415 Ksf							
"													UU	12.4	116.1	0.84 Ksf							
"													CU	12.9 12.6 12.5	104.2 104.6 104.7	0.149 Ksf 0.110 Ksf	30.3° (Effective) 17.7° (Total)						
BH 29-G (20-30')	Clayey gravel with sand	31	24	45	32	18	2.68				11.6	117.9	UU	11.2	106.4	0.225 Ksf							
"													UU	11.7	111.6	0.510 Ksf							
"													UU	11.6	118.1	1.15 Ksf							
"													U										
BH 33-F (0'-20')	Sandy Lean Clay with gravel	15	33	52	28	15	2.71				13.2	116.2	UU	12.5	104.5	0.210 Ksf		104.2	2.0×10^{-6} cm/sec.				
"													UU	13.2	110	110.9 Ksf							
"													UU	13.0	116.4	0.875 Ksf							
"																							

*Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

Notes: (1) All UU tests were saturated before shearing
(2) The C & ϕ values for CU tests are those presented by Chen & Assoc. Our interpretation may be different

UMTRA – GRJ
5025-16
MATERIAL PROPERTIES
VICINITY PROPERTIES – H.L. 6/19/91
SHB 6/22/90

Sh. 73A
REV. 3

SUMMARY OF SOIL TEST RESULTS

Job No 5025-16 Project Name UMTRA – GRAND JUNCTION Feature PROCESSING SITE Date 6/13/90

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis – Gravel	Sand	Fines	Atterberg Limits LL	PI	Specific Gravity G	Natural w %	Natural γd	Compaction Test	Optimum w	Optimum γd	Shear Strength Test	Initial w	Initial γd	C	φ	Permeability γd	k	Consolidation Cc	Cc	Pc	Notes
VICINITY PROPERTIES (REF: MKE DOC. 5025-GRJ-R-02-04128-00)																										
VP WEST TOP	1			SM/SC	22	48	30																			
VP MID. NORTH	2			"	26	46	28																			
VP MID. EAST	3			"	28	33	39																			
VP EAST	4			"	28	34	38																			
VP SOUTH EAST	5			"	21	31	48																			
AVERAGE =					25	38	37																			
VICINITY PROPERTIES (REF: LAB. ANALYSIS OF SOIL HYDRAULIC PROPERTIES OF THE GRJ MAINTAILINGS AND V.P. MAT. DELIVERY NO. 005, MKE DOC. 3885-GRJ-R-02-04327-00)																										
RS-3	5-15				0	35	65						SP	13.6	117.4											
"	5-16				0	33	67																			
RS-5	5-17				0	31	69						SP	14.6	115.5											
"	5-18				0	31	69																			
RS-6	5-19				0	25	75						SP	15.1	116.1											
"	5-20				0	25	75																			
RS-7	5-21				0	27	73						SP	13.9	115.5											
"	5-22				0	28	72																			
RS-10	5-23				0	42	58						SP	14.3	114.2											
"	5-24				0	41	59																			
AVERAGE =					0	32	68						SP	14.3	115.7											

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

SP = ASTM D698-78 (A)

Density, Moisture Content and Specific Gravity

(a) Dry Density (at 100 % Standard Proctor Compaction) = 116.3 for 29-G (0-20')
117.9 for 29-G (20-30')
116.2 for 33-F (0-20')

(Note: Based on stability requirements, a ~~100~~ 95 percent compaction is recommended)

∴ Average Dry Density = $\frac{116.3+117.9+116.2}{3}$ = 116.8 pcf
Max.

(b) Optimum Moisture Content = 12.5% for 29-G (0-20')
11.6% for 29-G (20-30')
13.2% for 33-F (0-20')

∴ Average Optimum Moisture Content = $\frac{12.5+11.6+13.2}{3}$ = 12.4 pct

(c) Specific Gravity = $\frac{2.67+2.68+2.71}{3}$ = 2.69

See Sheet 74A

(d) Average Moist Density, γ = 116.8 x 1.124 = 131.3 pcf at 100% compaction and optimum m/c

(e) Average Saturated Density,

Void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.69 \times 62.4}{116.8} - 1 = 0.437$

$\gamma_{ave} = \frac{G_s \gamma_w + e \gamma_w}{1+e} = \frac{2.69 \times 62.4 + 0.437 \times 62.4}{1.437} = 135.8$ pcf

$w_{sat} = \frac{Se}{G} = \frac{100\% \times 0.437}{2.69} = 16.2\%$

Revision 3

Since inclusion of clean fill dikes, requirements of stability are met with only 95 percent of standard compaction:

(c) Average Dry Density = .95 x 116.8 = 111.0 pcf

(d) Average Moist Density = 111.0 x 1.124 = 124.8 pcf

(e) Average Saturated Density.

$$\text{Void ratio, } e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.69 \times 62.4}{111.0} - 1 = 0.512$$

$$\gamma_{sat} = \frac{G_s \gamma_w + e \gamma_w}{1+e} = \frac{2.69 \times 62.4 + 0.512 \times 62.4}{1.512} = 132.1 \text{ pcf}$$

$$w_{sat} = \frac{Se}{G_s} = \frac{0.512}{2.69} = 19.0 \text{ pct}$$

Chen & Associates
Consulting Geotechnical Engineers

Job No. 1 343 87

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

SAMPLE LOCATION		OPT. MOISTURE CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO 200 SIEVE	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (psf)	SPECIFIC GRAVITY	SOIL OR BEDROCK TYPE
HOLE	DEPTH (feet)			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
BH-6-29	0-20	12.5	116.3	16	26	58	32	18		2.67	Sandy lean clay with gravel
BH-6-29	20-30	11.6	117.9	31	24	45	32	18		2.68	Clayey gravel with sand
9H	5-~~8~~20'	14.1	105.0	0	86	14		NP			Silty sand
21J	5-20	13.3	108.0	0	88	12		NP			Poorly graded sand with silt
21J	20-45	13.2	109.2	0	84	16		NP			Silty sand
33F-B	0-20	13.2	116.2	15	33	52	28	15		2.71	Sandy lean clay with gravel

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WYL 2/5/88

CA-2-79

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PS 2/4/88

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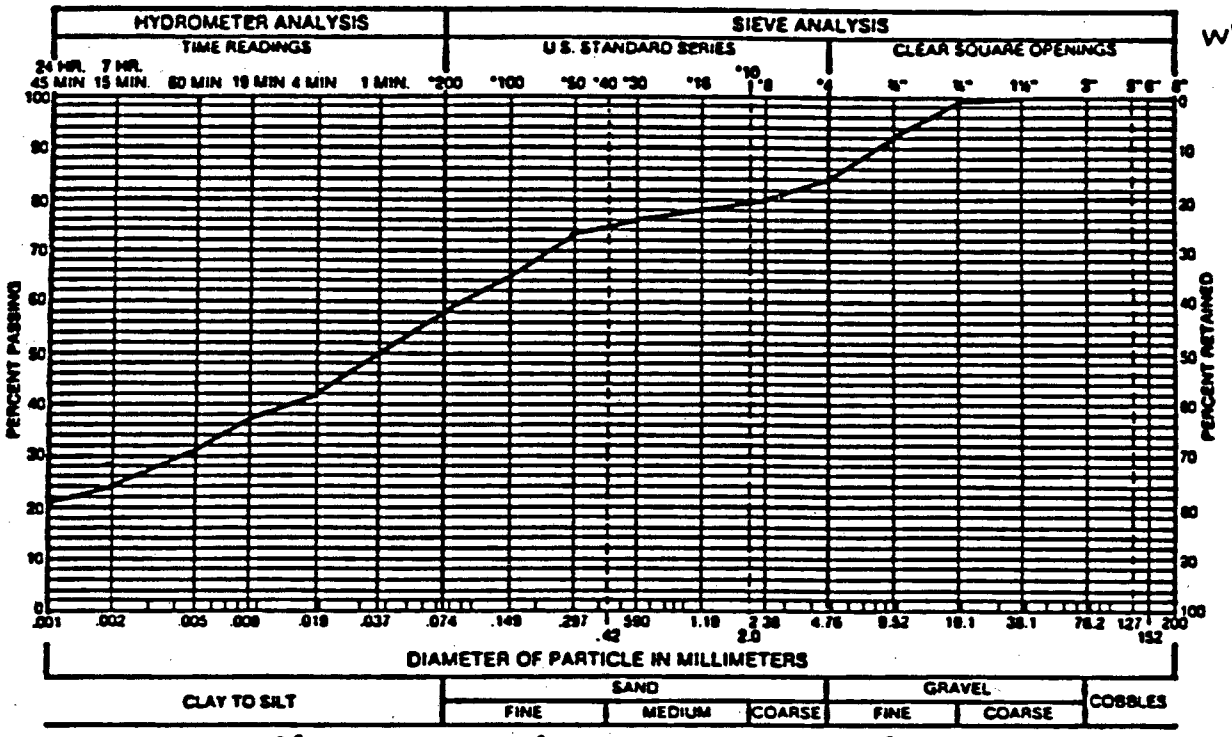
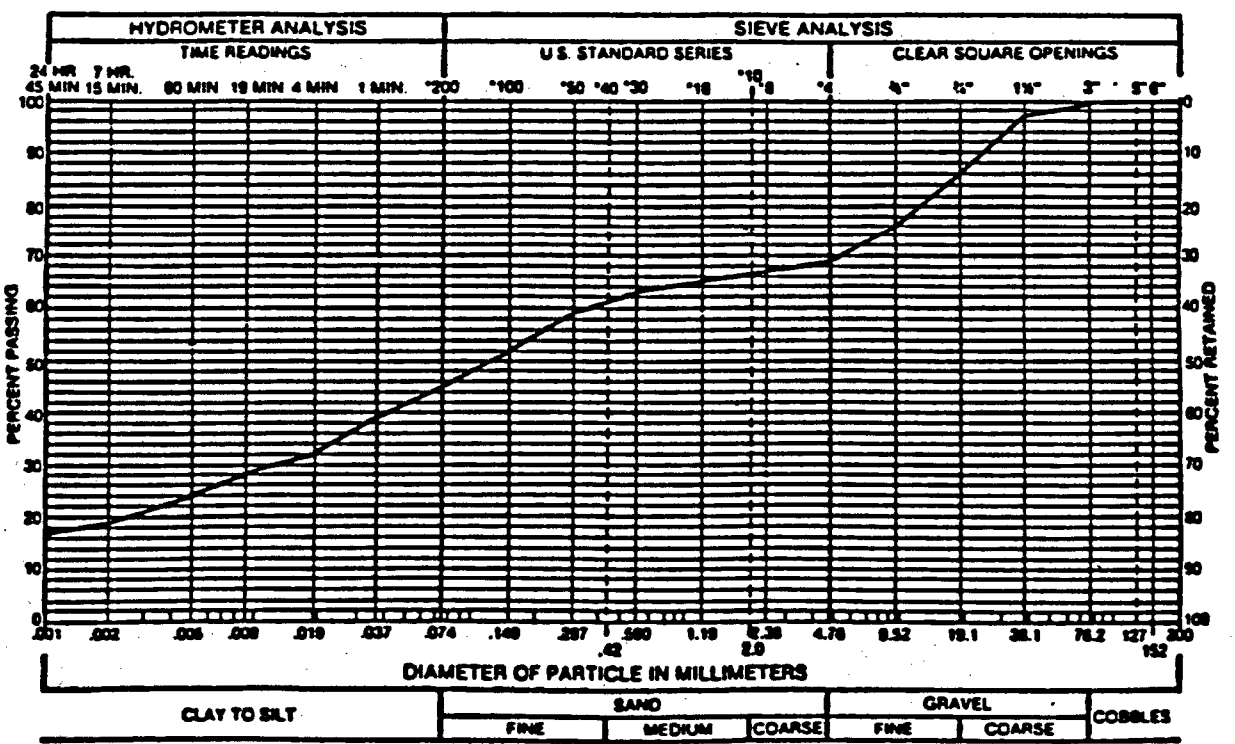


GRAVEL 16 % SAND 26 % SILT AND CLAY 58 %

LIQUID LIMIT 32 % PLASTICITY INDEX 18 %

SAMPLE OF Sandy lean clay with gravel FROM Hole BH6 ~~29~~ 29-G at depth 0-20'



GRAVEL 31 % SAND 24 % SILT AND CLAY 45 %

LIQUID LIMIT 32 % PLASTICITY INDEX 18 %

SAMPLE OF Clayey gravel with sand FROM Hole BH6 ~~29~~ 29-G at depth 20'-30'

1 343 87

GRADATION TEST RESULTS

Fig. 1

CA-2-79

MKE - Grand Junction

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PS 2/4/88
WYL 2/5/88

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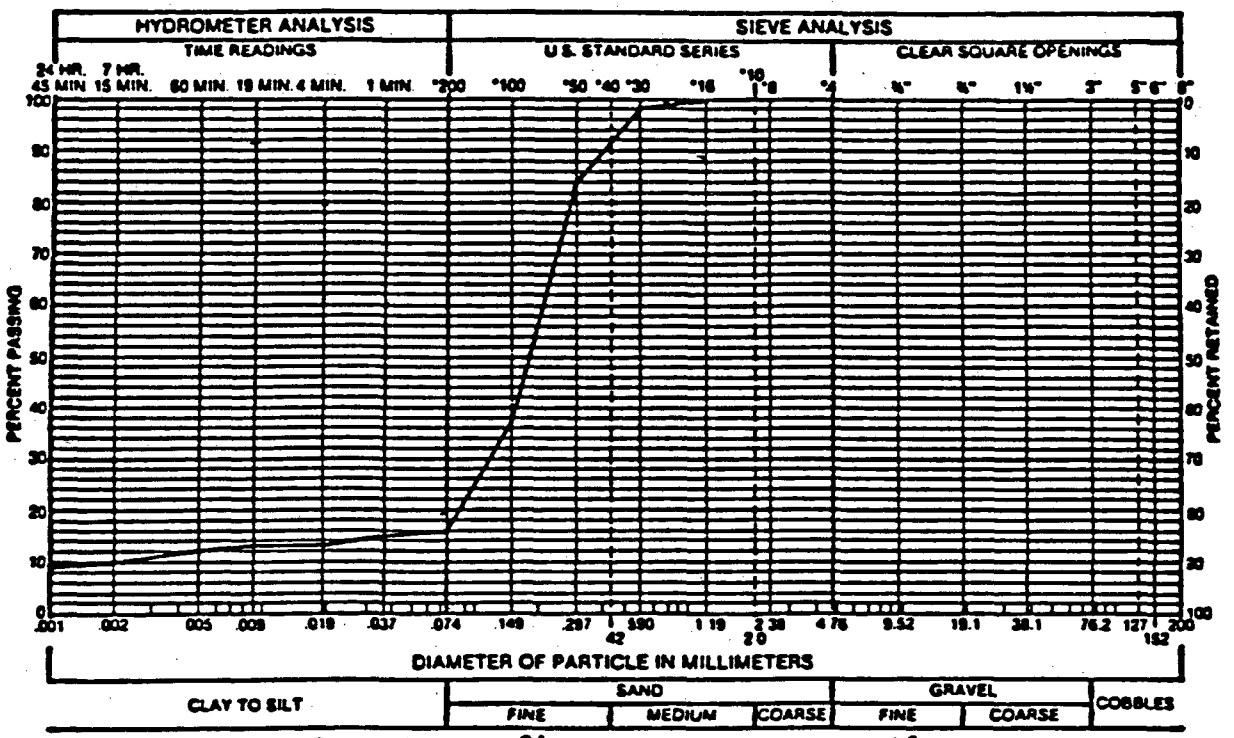
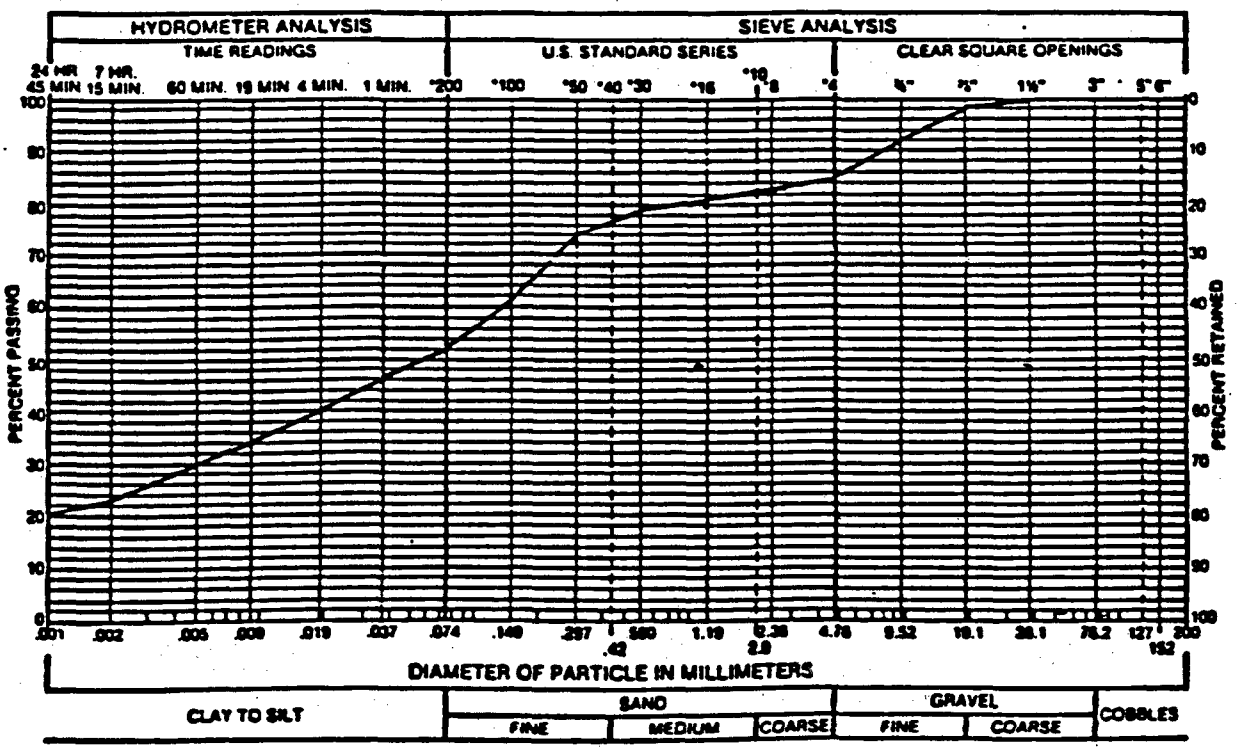


GRAVEL 0 % SAND 84 % SILT AND CLAY 16 %

LIQUID LIMIT % PLASTICITY INDEX Nonplastic

SAMPLE OF Silty sand FROM Hole 21J at depth 20'-45'



GRAVEL 15 % SAND 33 % SILT AND CLAY 52 %

LIQUID LIMIT 28 % PLASTICITY INDEX 15 %

SAMPLE OF Sandy lean clay with gravel FROM Hole 33F-B at depth 0-20'

1 342 97

GRADATION TEST RESULTS

Fig. 3

Shear Strength Characteristics

The UU test results are presented on Sheets 80 through 83. The shear strength versus degree of compaction is plotted on the next sheet. The influence of degree of compaction on shear strength is evident.

~~Preliminary analyses (MKE Calc. No. 054670-04-00 slope stability) indicate that a factor of safety of 1.06 is obtained if the ponds area material is assigned an undrained shear strength of 600 psf. This requires a compaction of at least 96 to 98 percent compaction. A 100 percent compaction is recommended. The resultant shear strength ranges from 840 psf to 1150 psf. Use S_u=800 psf for design~~

Only one CU test has been run. This test was run on material at 90 percent compaction, e.g. the compaction level originally planned. No tests are available at ~~100~~ 95 percent compaction level. In the test (see Sheet 83), the values of ϕ' and ϕ interpreted by us are 33.5° and 18.5°, respectively. Use ϕ'=30° for long term static stability analysis.

Revision 3 — H.L. 6/27/90
P.S. 6/27/90

Short Term, static & Seismic	Long-term static	Long Term Seismic
Upto 12.5', S_u=400 psf	c=0, ϕ=30°	c=0, ϕ=18.5°
Below 12.5', ϕ=18.5°, c=0		

For explanation, see Sheet 78A

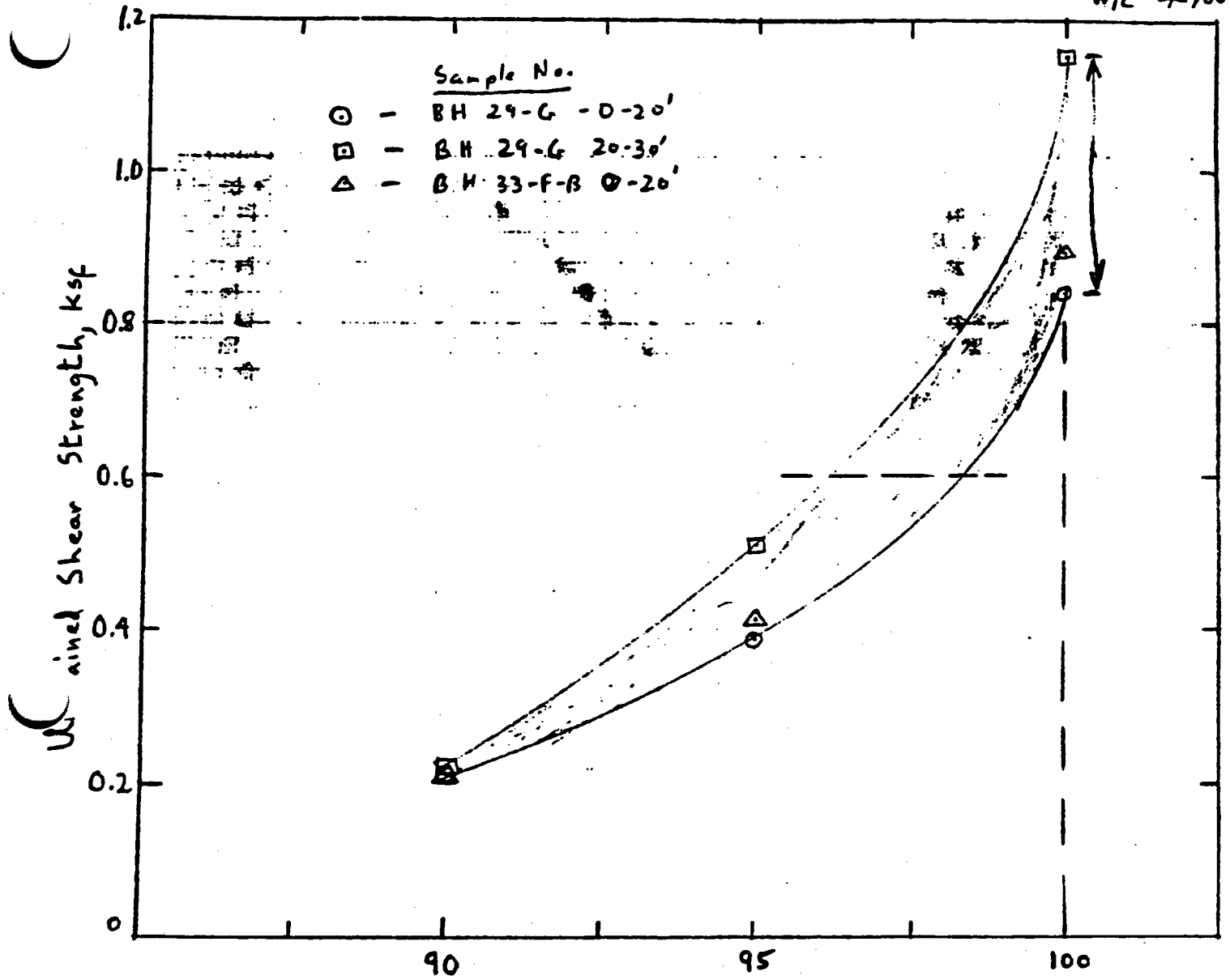
The worst condition from the standpoints of settlement and slope stability are when all the ponds area material is placed on top. Since that operation will take at least 1 yr, using Reference 5, Figure 7.3,

$$\frac{m^2 t}{C_v} = \frac{(25)^2 \times 1}{2.3 \times 10^{-5} \times 86400 \times 365} = 0.86$$

This indicates that over the course of the year, essentially the full layer will almost be consolidated under its own weight, since actually it is a double drained situation.

However, prior to placement of the radon barrier, rainfall could saturate the material, which then would not immediately consolidate. Thus, for part of the layer, UU strengths could apply. It is assumed that 1/2 of the layer will be unconsolidated, e.g. UU upto 12.5 ft depth and CU below that. For long-term seismic, it is reasonable to assume the entire layer consolidated, since water addition is limited by the radon barrier. Thus, CU strengths will apply.

PS 1/27/88
WYL 2/5/88



Notes: (1) The degree of compaction is based on

(2) All samples were saturated and then subjected to a confining pressure of 1000 psf.

UU Test Results

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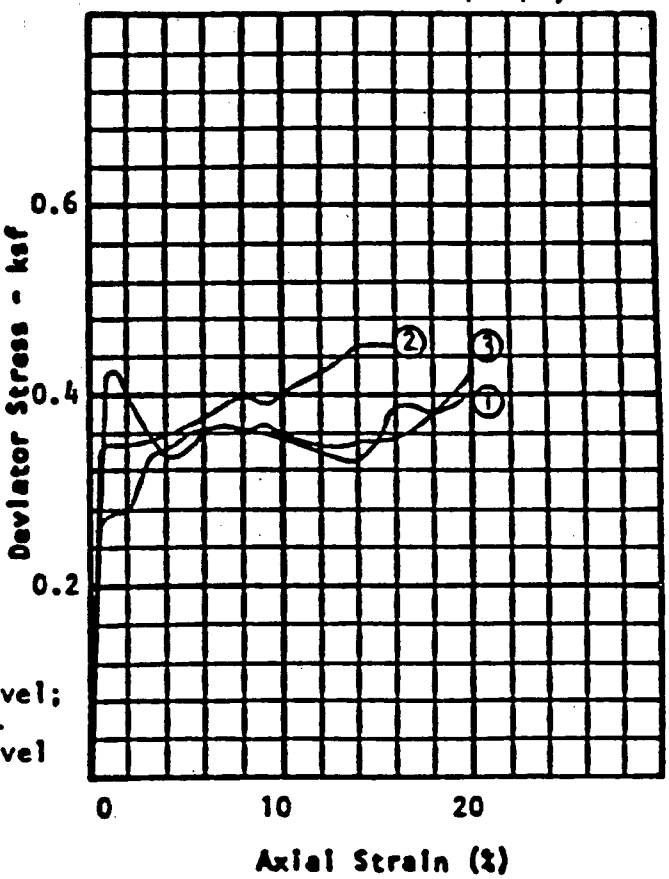
WYL 2/5/88

TEST NUMBER	①	②	③	4
LOCATION Hole No.	33F-B	BHG-29	BHG-29	
Depth-ft.	0-20	20-30	0-20	
HEIGHT - INCH	3.928	3.944	3.946	
DIAMETER - INCH	1.875	1.874	1.875	
WATER CONTENT - %	13.4	11.2	12.5	
DRY DENSITY - pcf	104.7	106.4	104.5	
$\sigma_1 - \sigma_3$	0.420	0.460	0.420	
σ_1 - ksf	1.414	1.444	1.414	
σ_3 - ksf	0.994	0.994	0.994	



TYPE OF SPECIMEN: Remolded

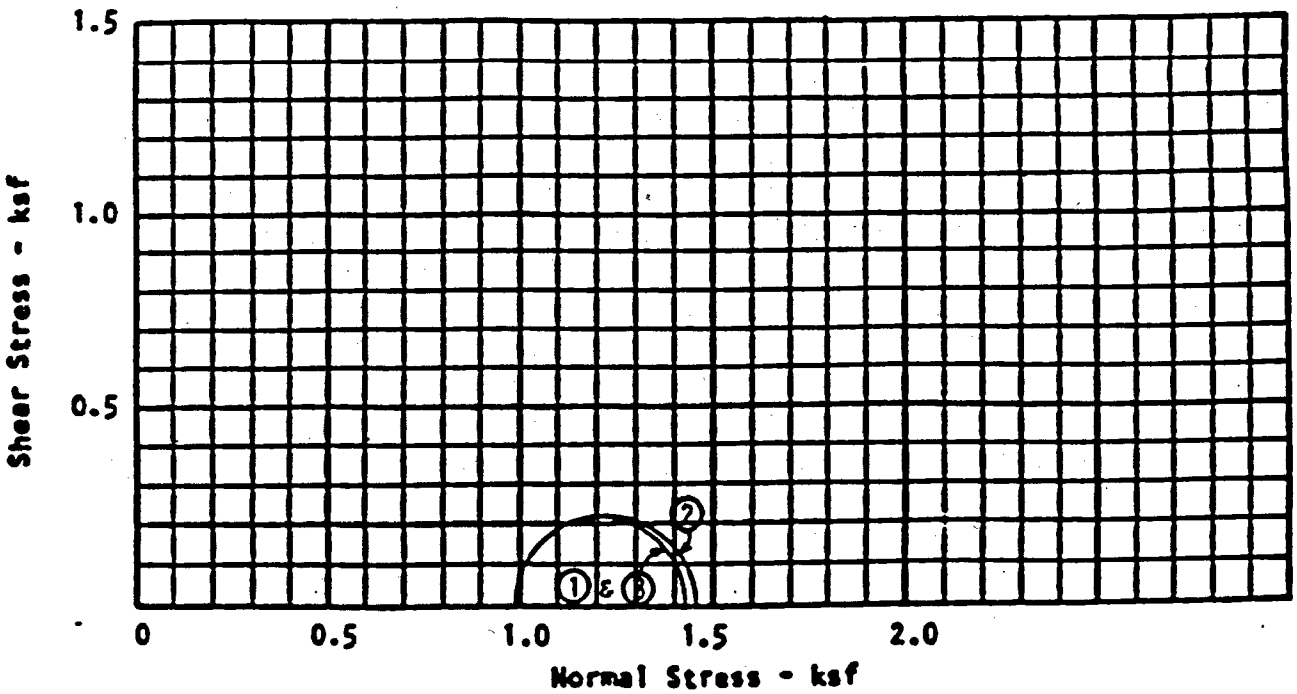
SOIL DESCRIPTION: ① Sandy lean clay with gravel; ② Clayey gravel with sand; ③ Sandy lean clay with gravel

TYPE OF TEST: Saturated, Unconsolidated, Undrained

TAN ϕ

ϕ

COHESION - ksf



1 343 87

TRIAXIAL SHEAR TEST RESULTS

Fig. 12

8-1

PS 2/4/88
WYL 2/4/88

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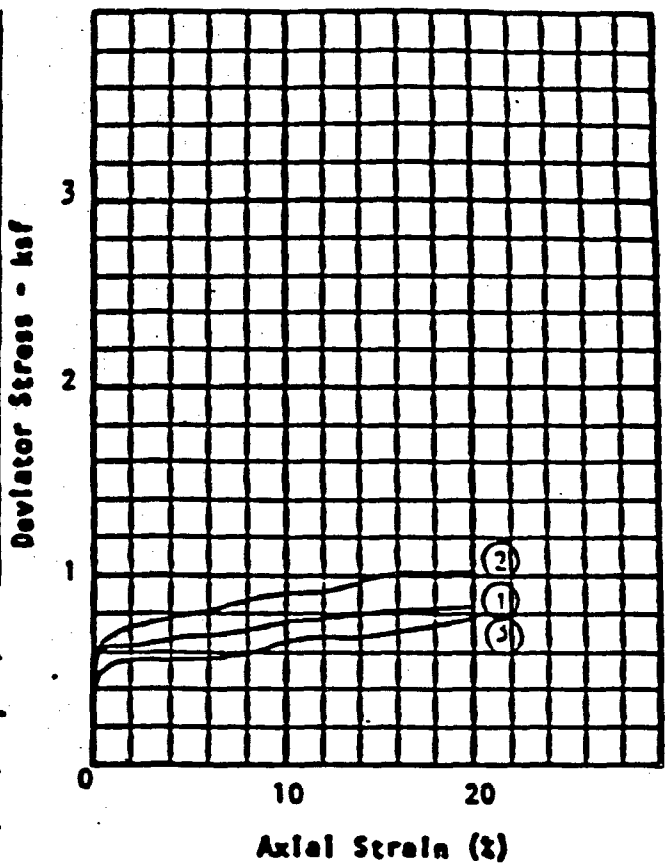
TEST NUMBER	1	2	3	4
LOCATION Hole No.	BG-29	BHG-29	33F-8	
Depth - Feet	0-20	20-30	2-29	
HEIGHT - INCH	3.923	3.929	3.926	
DIAMETER - INCH	1.880	1.880	1.875	
WATER CONTENT - %	12.8	11.7	13.2	
DRY DENSITY - pcf	110.0	111.6	110.9	
$\sigma_1 - \sigma_3$	0.83	1.02	0.77	
σ_1 - ksf	1.82	2.01	1.76	
σ_3 - ksf	0.99	0.99	0.99	



TYPE OF SPECIMEN: Remolded - #4 material*

SOIL DESCRIPTION: Silty clay/Clayey silt

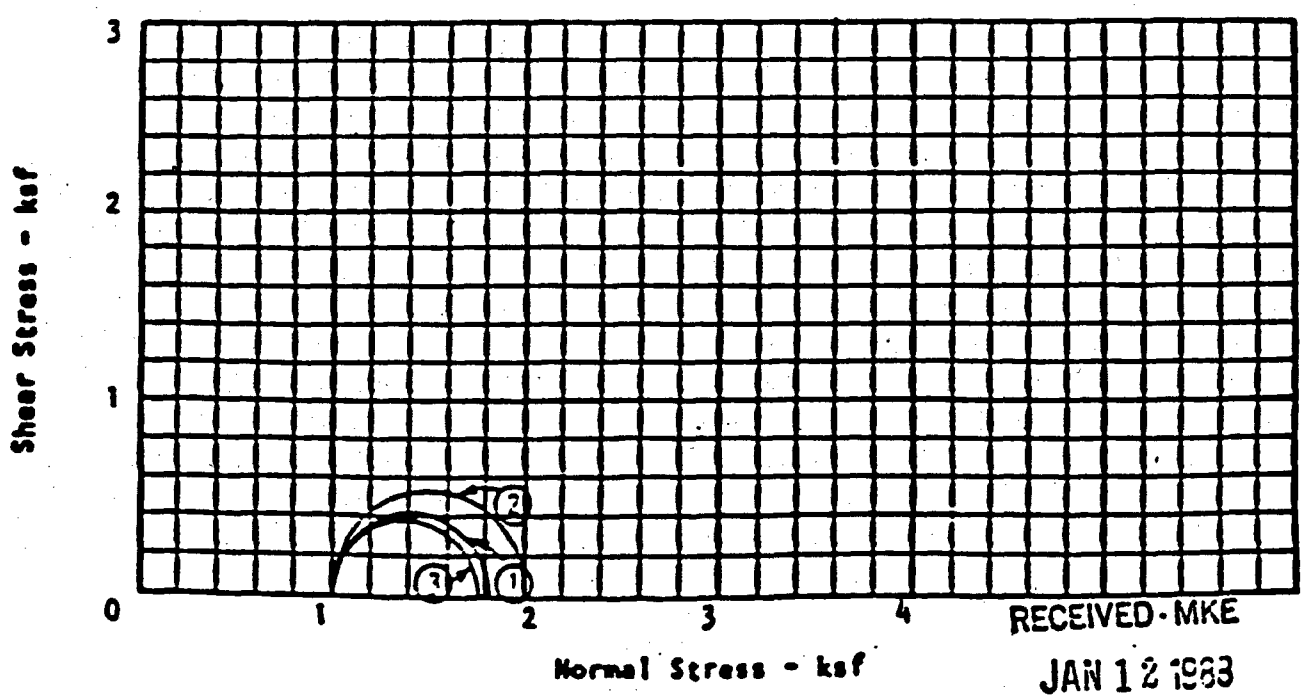
TYPE OF TEST: Saturated, Unconsolidated, Undrained

*95% Compaction

TAN ϕ

ϕ

COHESION - ksf



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1 343 87

TRIAXIAL SHEAR TEST RESULTS

Fig. 1

82

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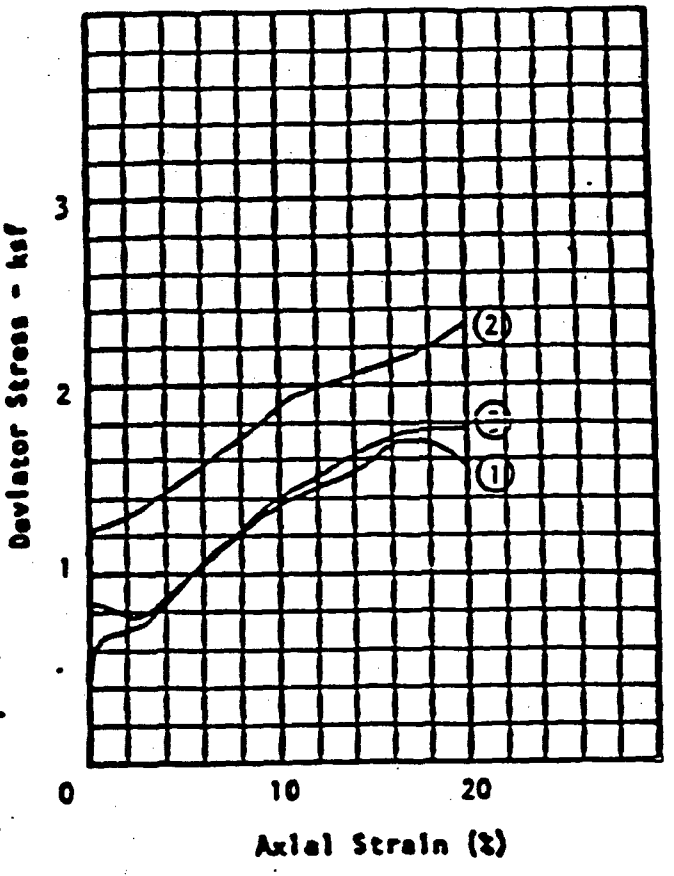
PS 2/4/88
WYL 2/4/88

TEST NUMBER	1	2	3	4
LOCATION Hole No.	BHG-29	BHG-29	33F-B	
Depth - Feet	0-20	20-30	2-29	
HEIGHT - INCH	3.885	3.912	3.925	
DIAMETER - INCH	1.880	1.880	1.878	
WATER CONTENT - %	12.4	11.6	13.0	
DRY DENSITY - pcf	116.1	118.1	116.4	
$\sigma_1 - \sigma_3$	1.68	2.30	1.75	
σ_1 - ksf	2.67	3.29	2.74	
σ_3 - ksf	0.99	0.99	0.99	



TYPE OF SPECIMEN: Remolded - #4 material*

SOIL DESCRIPTION: Silty clay/Clayey silt

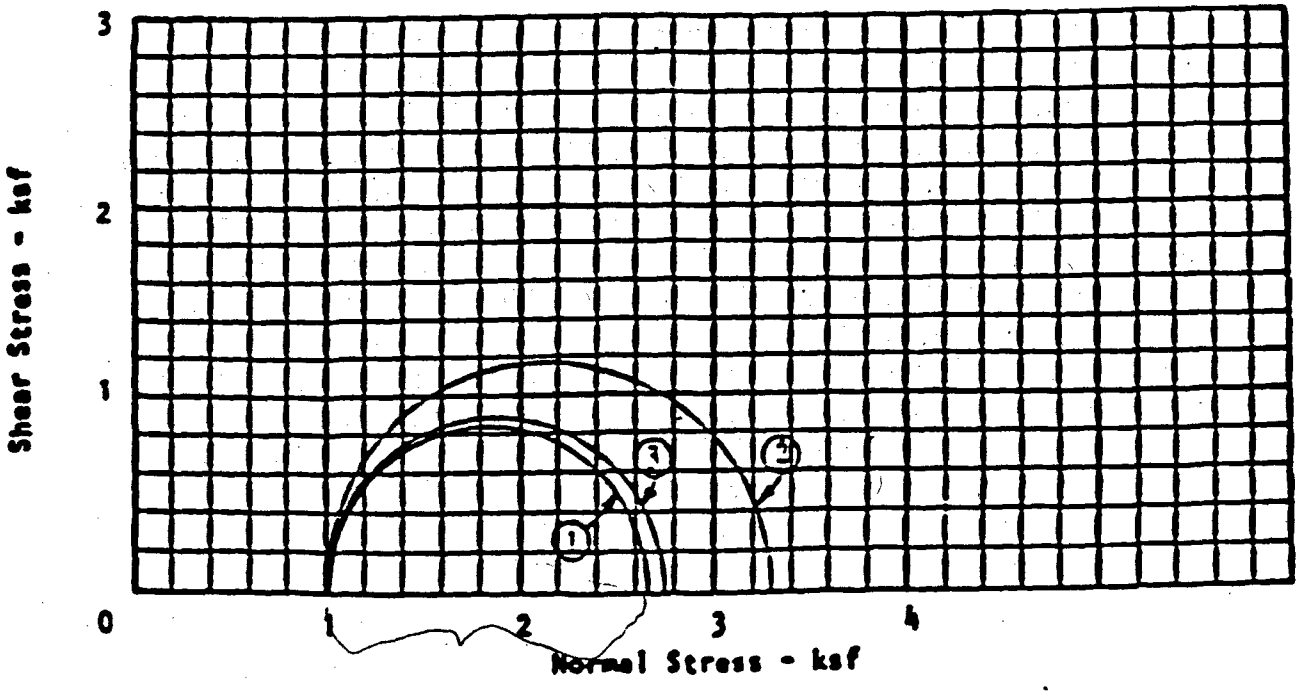
TYPE OF TEST: Saturated, Unconsolidated, Undrained

*100% Compaction

TAN ϕ

ϕ

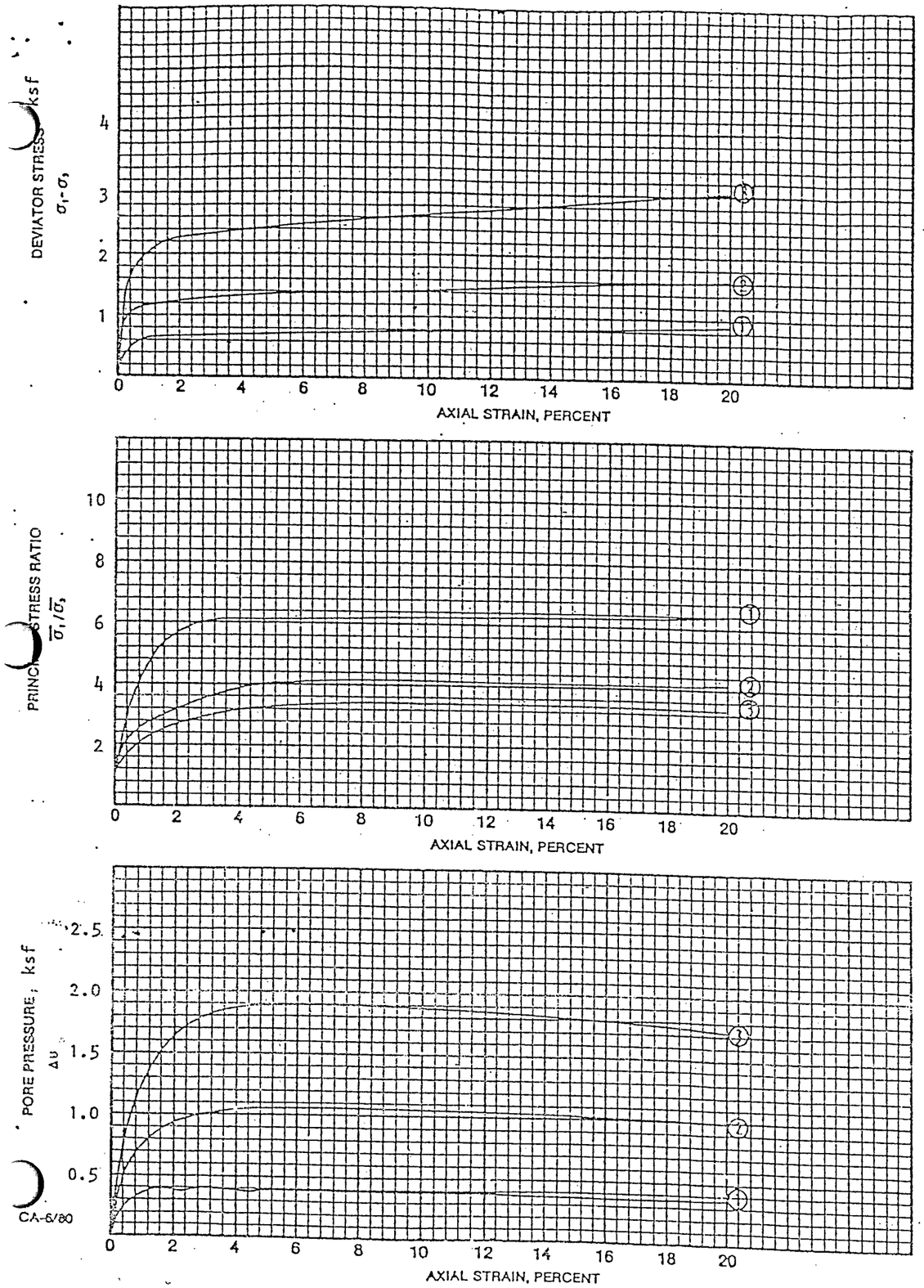
COHESION - ksf



1 343 87

Fig. 2

TRIAXIAL SHEAR TEST RESULTS



CA-6/80

Job No. 1 343 87
Date June 8, 1987
Type of Test Saturated, consolidated, undrained with pore pressure measurements

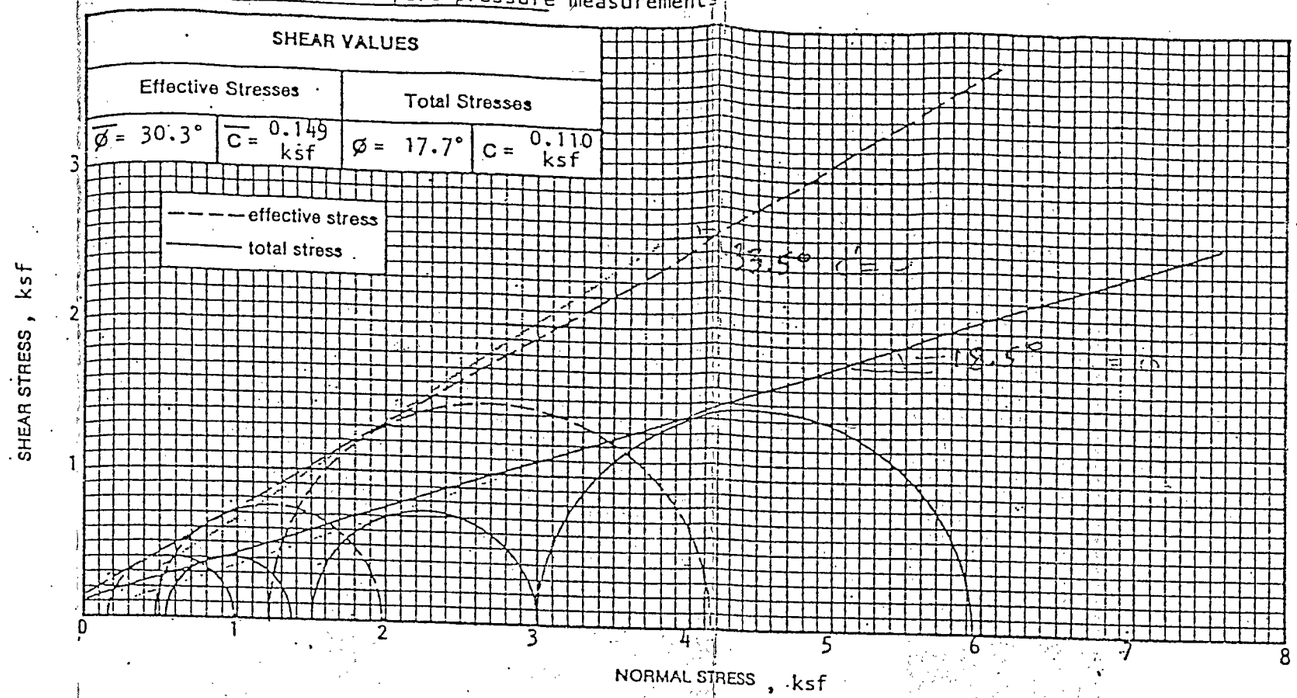
FHWA - Grand Junction

SHEAR STRENGTH OF SOIL IN TRIAXIAL COMPRESSION

83

PS 2/4/88
WYL 2/5/88

SHEAR VALUES			
Effective Stresses		Total Stresses	
$\bar{\phi}$ = 30.3°	$\bar{C}$ = 0.149 ksf	ϕ = 17.7°	C = 0.110 ksf



Specimen Number	Specimen Location		Initial Specimen Data					Soil Description
	Boring Number	Depth (Ft)	Sample Type	Length (in)	Diameter (in)	Dry Density (P.C.F.)	Moisture Content (%)	
1	BHG-29	0-20	Remolded	3.943	1.875	104.2	12.9	Sandy lean clay with gravel
2	"	"	"	3.938	1.875	104.6	12.6	" " "
3	"	"	"	3.938	1.875	104.7	12.5	" " "

Specimen Number	"B" Parameter	VALUES PLOTTED AT MAXIMUM PRINCIPAL STRESS RATIO							Remarks
		Total Confining Stress σ_3	Total Axial Stress σ_1	Deviator Stress $\sigma_1-\sigma_3$	Effective Lateral Stress $\bar{\sigma}_3$	Effective Axial Stress $\bar{\sigma}_1$	Pore Pressure μ	Percent Strain ϵ%	
1	0.95	0.533	1.363	0.830	0.158	0.989	0.374	14.012	Units in ksf
2	0.97	1.498	2.999	1.501	0.475	1.977	1.022	14.001	" " "
3	0.96	2.995	5.940	2.945	1.195	4.140	1.800	15.994	" " "

Remarks:

Fig. 13

Consolidation Characteristics

Consolidation characteristics are developed from the single consolidation test performed. The following values were caculated from the starin - log p plot on the following page, and are recommended as design values:

$P_p = 2.2$ ksf

$C_{c\varepsilon} = 0.160$ $\quad (C_c = 0.160 \times 1.437 = 0.230)$

$C_{r\varepsilon} = 0.005$ $\quad (C_r = 0.005 \times 1.437 = 0.0072)$

Coefficient of Consolidation, C_v, was computed from the time-settlement curves in the consolidation test, presented on sheets 85 through 92. The C_v values are computed on each page. The values are as follows:

Stress	C_v (ft^2/min.)
500	0.74×10^{-3}
1,000	1.74×10^{-3}
2,000	$(1.47$ to $1.70) \times 10^{-3}$
4,000	0.96×10^{-3}
8,000	1.74×10^{-3}

Average $= 1.39 \times 10^{-3}$ ft^2/min $= 2.31 \times 10^{-5}$ ft^2/sec $= 0.214$ cm^2/s

Use $C_v = 2.3 \times 10^{-5}$ ft^2/sec. for design

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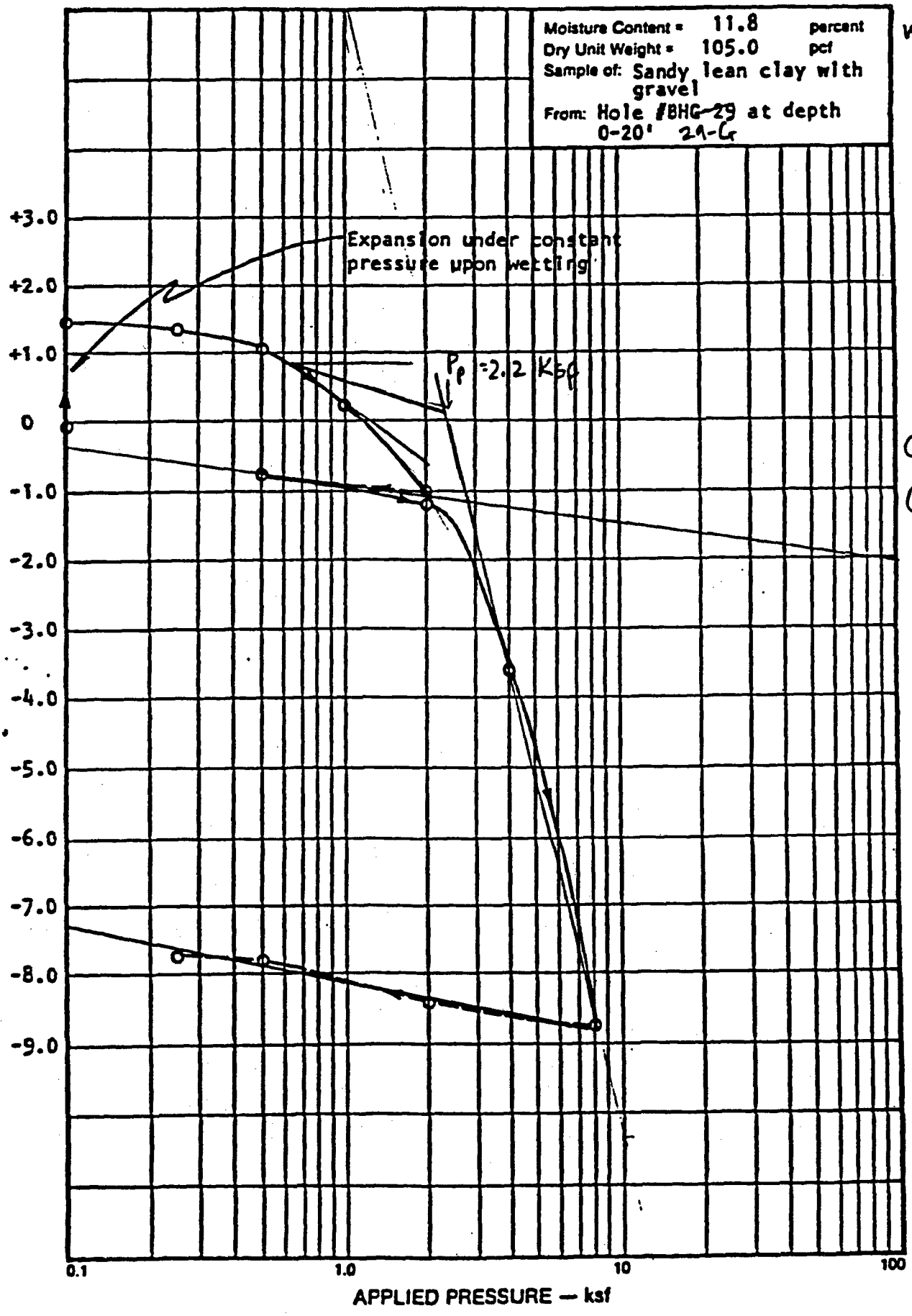
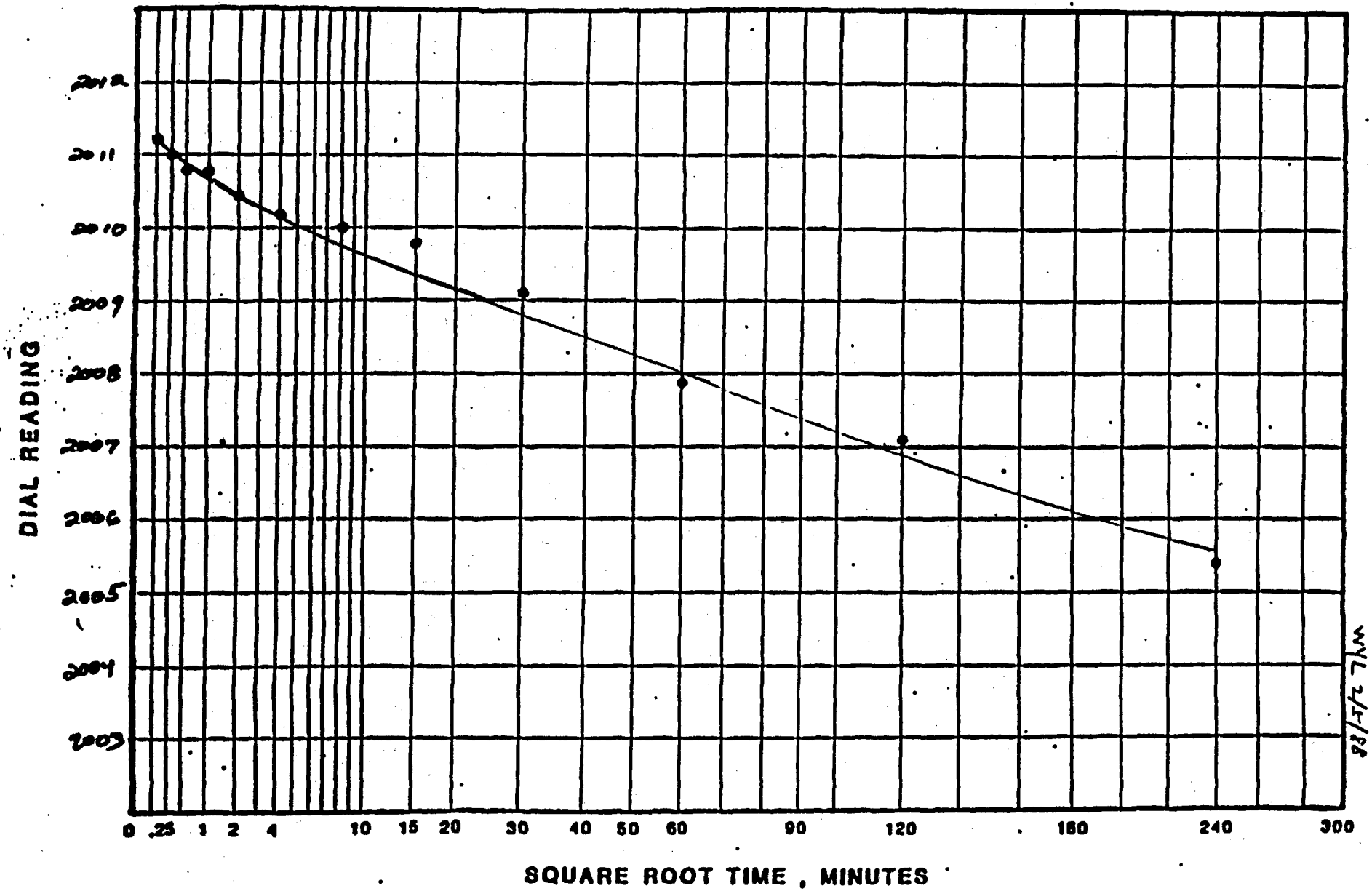
PS 2/4/88
WYL 2/5/88



P_p = 2.2 ksf
$C_{c\varepsilon}$ = 0.16
$C_{r\varepsilon}$ = .005

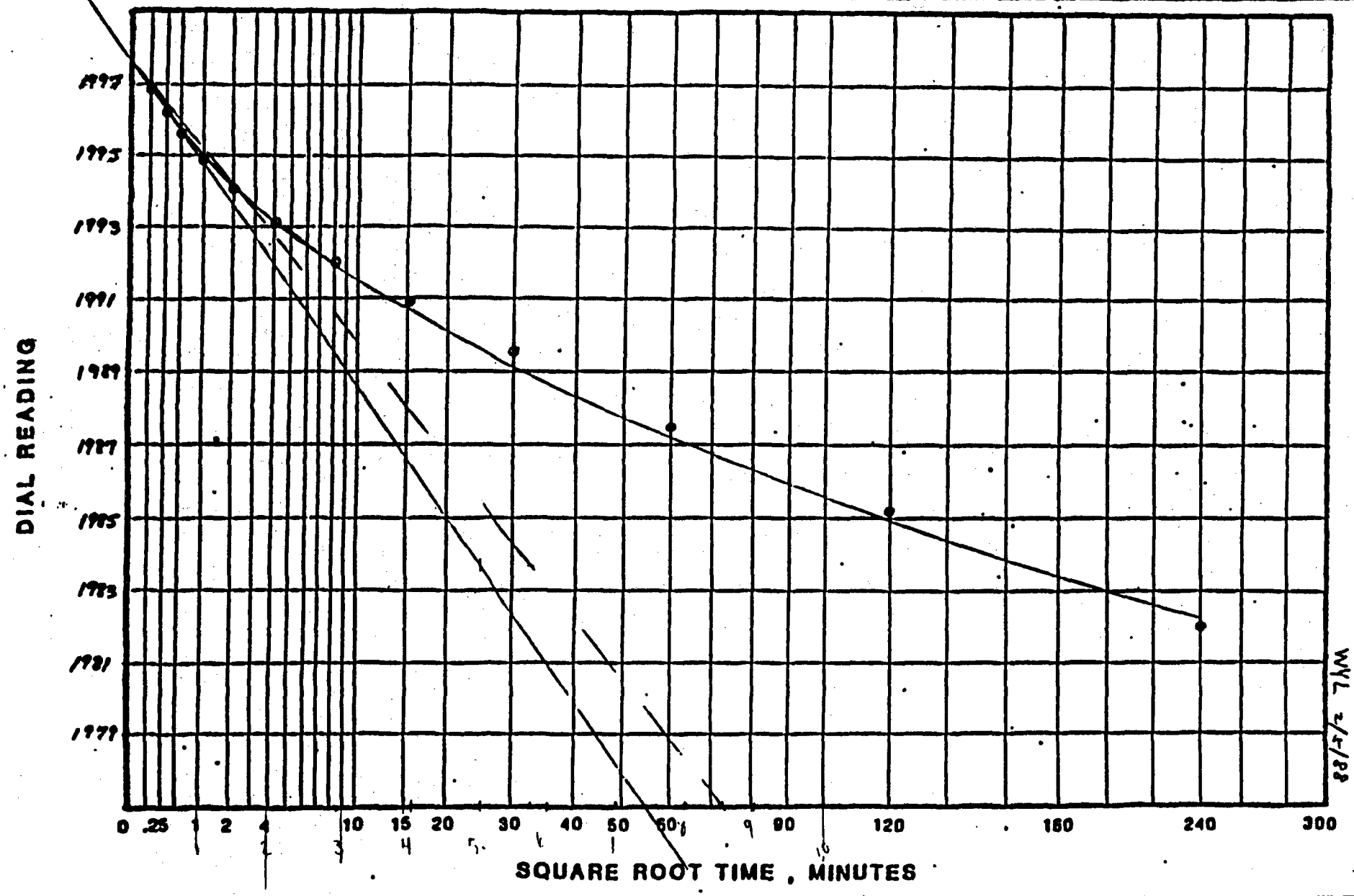


WJL 2/5/88
PS 2/4/88

HOLE BH6-29	DEPTH 0-20	SAMPLE NO.	LOAD STRESS 250 PSF	TIME SETTLEMENT CURVE
c_v	cm^2/sec	s_v	K_{avg} cm/sec	chen and associates, inc.
d_{50}		d_{100}		

$v = 1.41 \times 10^{-4}$ ft/min = 11, cm/sec.

$C_v = \frac{0.848\ H^2}{t_{90}} = \frac{(0.848)(1/12)/4}{2\ min} = 7.0 \times 10^{-4}$ ft²/min



P.S. 2/4/88
WYL 2/5/88

87

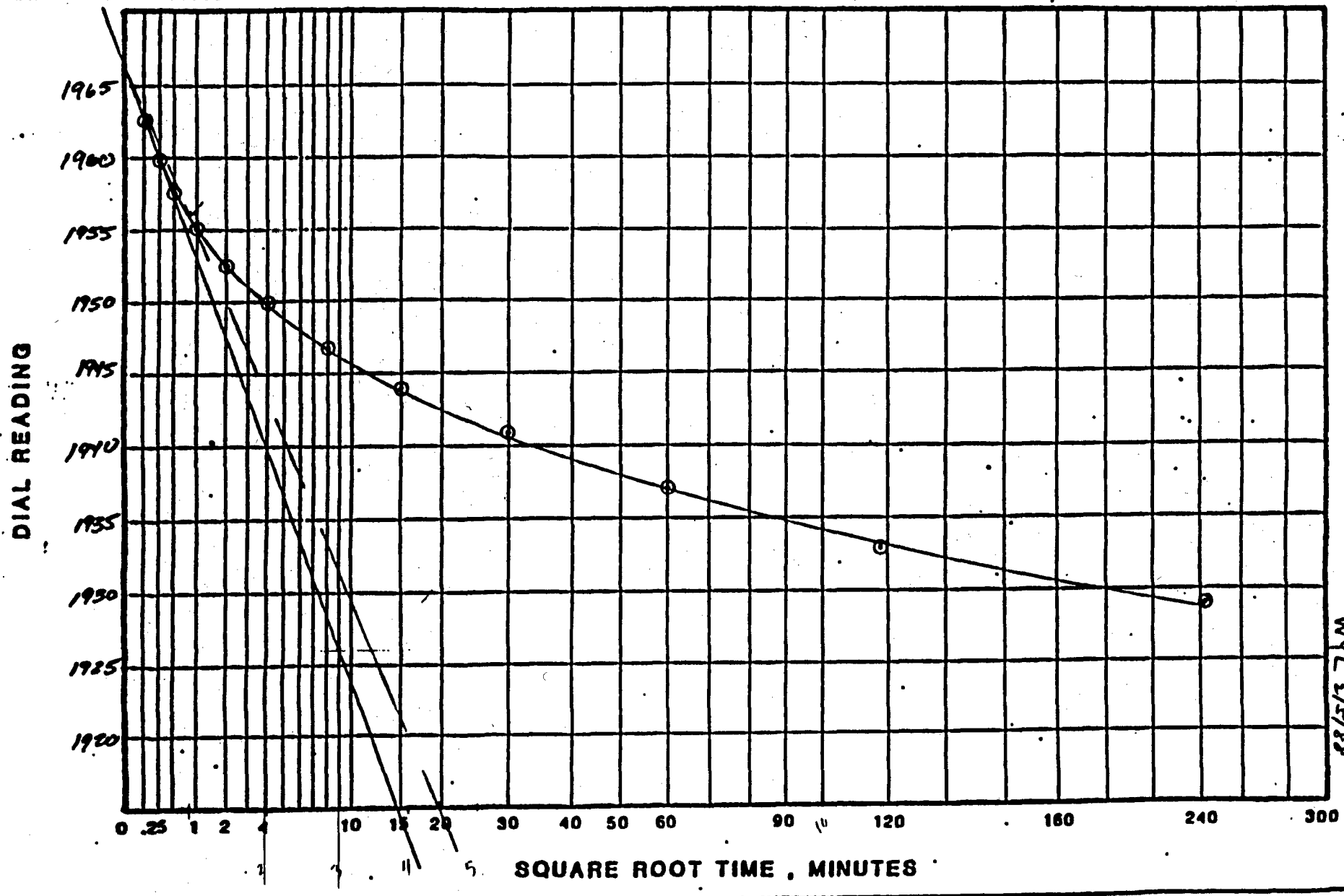
HOLE BH 6-19	DEPTH 0-20	SAMPLE NO. ____	LOAD STRESS 500 PSF
c_v ____	cm²/sec	a_v ____	K_{avg} ____ cm/sec
d_{90} ____		d_{100} ____	

TIME SETTLEMENT CURVE

chen and associates, inc.

t_{90} 12 t_{91} .85 n

$$C_v = \frac{0.848\,H^2}{t_{90}} = \frac{0.848\,(1/12)^2/4}{0.85} = 1.73\times10^{-3}\ ft^2/min = 0.0268\ cm^2/sec$$



WYL 2/4/88

P5

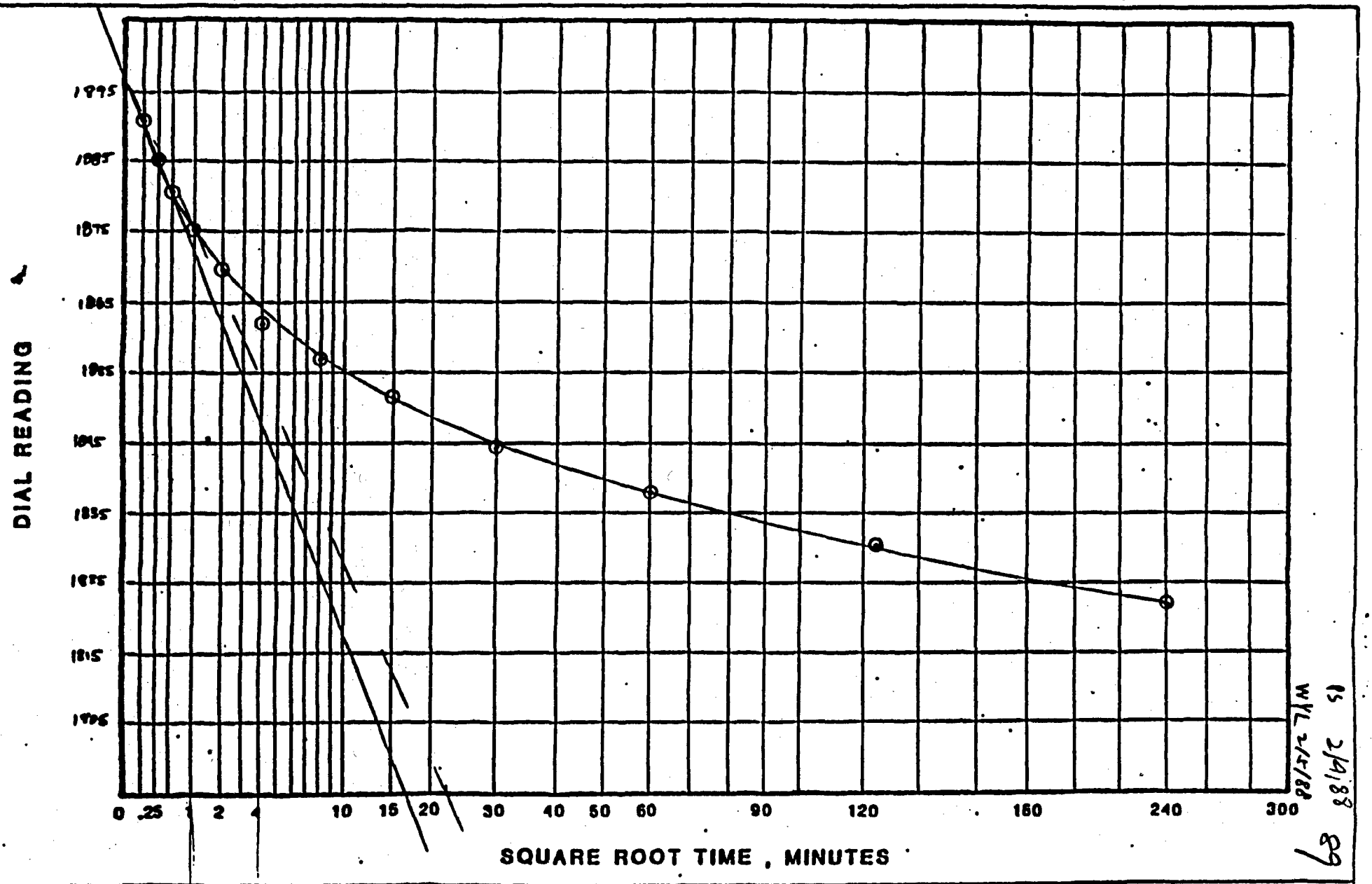
88

HOLE BH6-29	DEPTH 0-20	SAMPLE NO. ____	LOAD STRESS 1000 PSF
c_v ____	cm^2/sec	a_v ____	K_{avg} ____ cm/sec
d_{90} ____		d_{100} ____	

TIME SETTLEMENT CURVE

chen and associates, inc.

$$C_v = \frac{0.848\,(1/12)^2/4}{1\ \text{min}} = 1.47\times10^{-3}\ \text{ft}^2/\text{min} = 0.0228\ \text{cm}^2/\text{sec}$$



WL 2/5/88 2/9/88

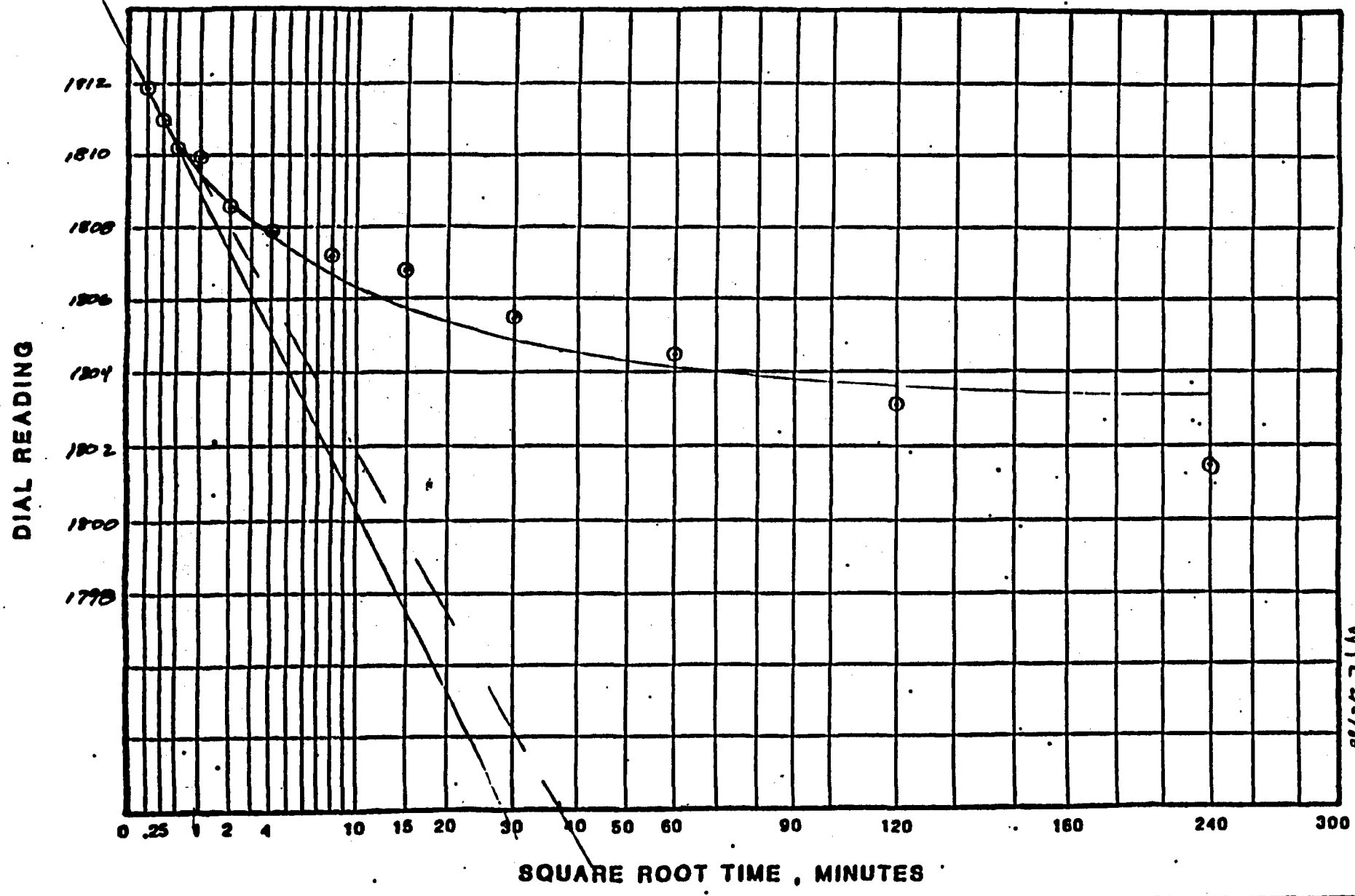
HOLE BH6-29	DEPTH 0-20	SAMPLE NO. ____	LOAD STRESS 2000 PSF
c_v ____	cm²/sec	a_v ____	K_{avg} ____ cm/sec
d_{90} ____		d_{100} ____	

TIME SETTLEMENT CURVE

chen and associates, inc.

$t_{90} = 0.87$ min

$C_v = \frac{0.848\,(1/12)^2/4}{0.87} = 1.70 \times 10^{-3}\ ft^2/min = 0.0264\ cm^2/sec$



BS 2/4/89
WYL 2/5/88

90

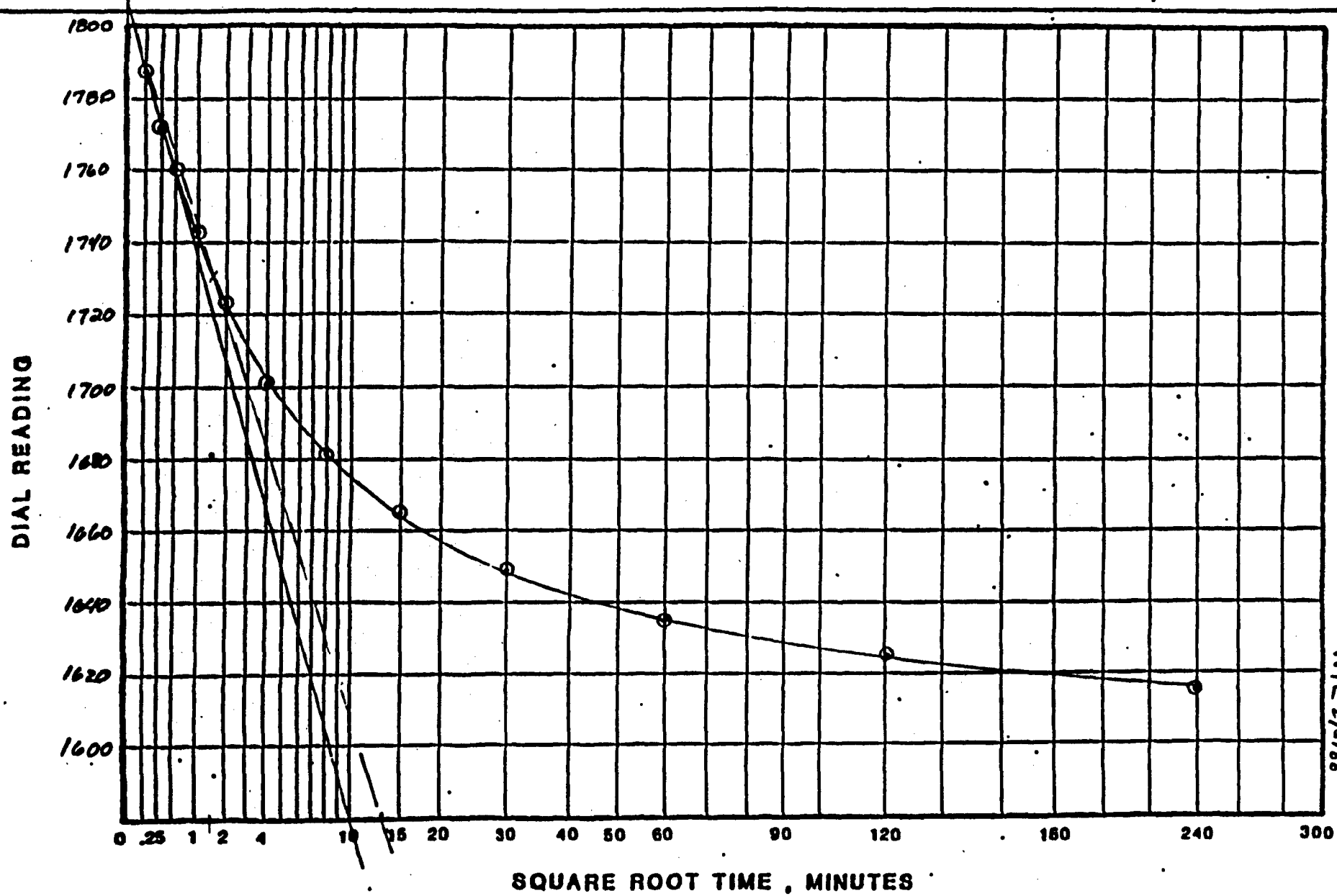
HOLE BH6-29	DEPTH 0-20	SAMPLE NO. ____	LOAD STRESS 2000 ~~kg~~ PSF
c_v ____	cm^2/sec	e_v ____	K_{avg} ____ cm/sec
d_{50} ____		d_{100} ____	

TIME SETTLEMENT CURVE

chen and associates, inc.

VI. 1.C. 90° 9'

$$C_v = \frac{0.848\ (1/12)^2 H}{1.54\ \text{min}} = 7.57 \times 10^{-4}\ ft^2/min = 0.0148\ cm^2/sec$$



WYL 2/4/88 PS 2/4/88

HOLE BH6-29 DEPTH 0-20 SAMPLE NO. _____ LOAD STRESS 4000 PSF

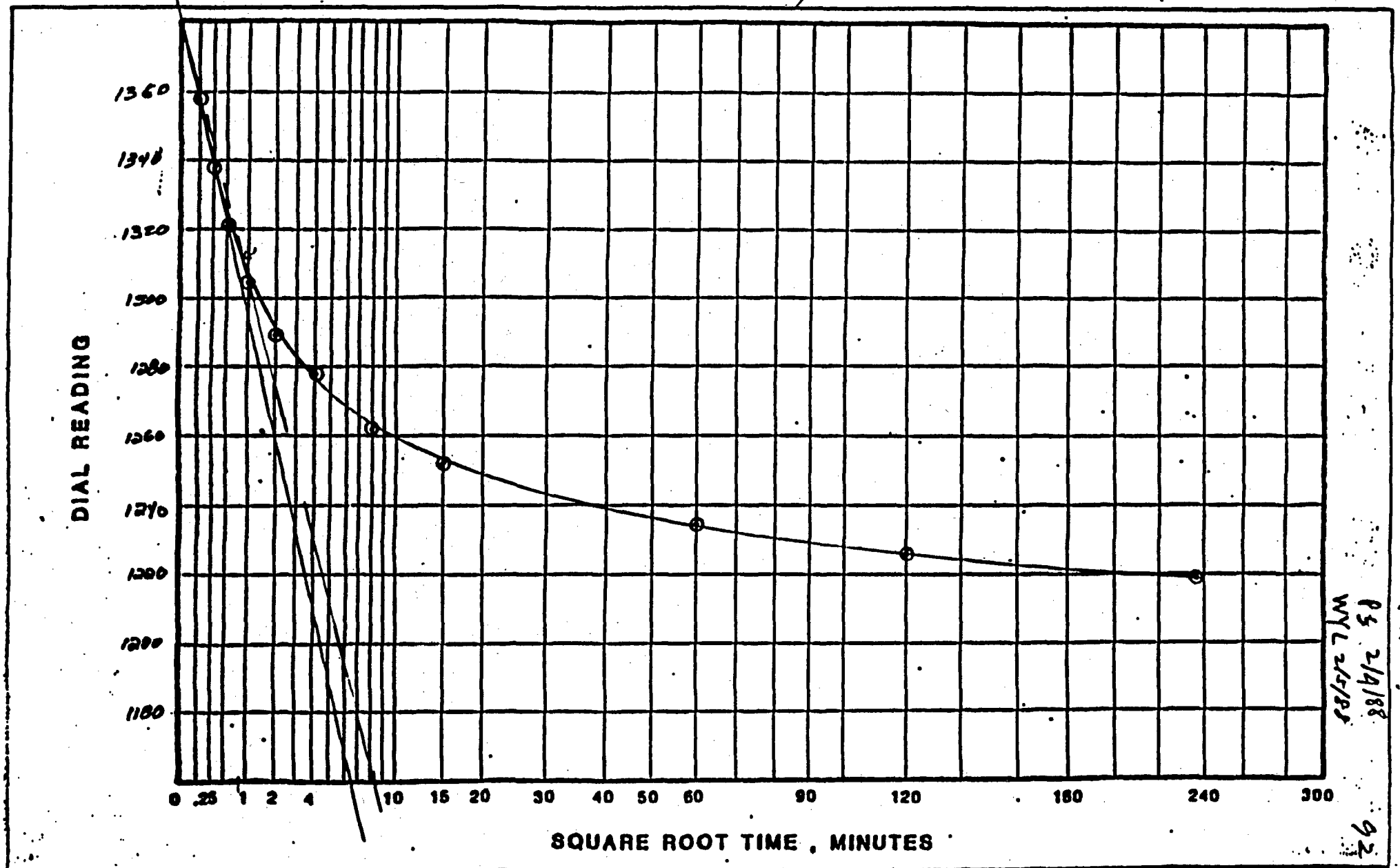
c_v _____ cm^2/sec a_v _____ K_{avg} _____ cm/sec

d_{50} _____ d_{100} _____

TIME SETTLEMENT CURVE

chen and associates, inc.

$$C_v = \frac{0.848\,(1/12)^2/4}{0.846} = 1.74 \times 10^{-3}\ ft^2/min = 0.0269\ cm^2/sec$$



PS 2/4/88
WYL 2/5/88

92

HOLE BH 6-29	DEPTH 0-20	SAMPLE NO. _____	LOAD STRESS 8000 PSF
c_v 29-6 cm²/sec		a_v _____	K_{avg} _____ cm/sec
d_{50} _____		d_{100} _____	

TIME SETTLEMENT CURVE

chen and associates, inc.

Re-evaluation of C_v for Ponds Area Material

The initial points of the time-settlement curves on Sheets 86 through 92 does not include the initial dial reading (e.g. t= Therefore the extrapolation of the tangent at 0 time appeared somewhat suspect. In addition, the value did not agree well with indirect evaluations based on permeability, C_c etc, such as discussed on Sheet 60. Thus, time-settlement curves were replotted and re-evaluated from dial readings. The replots are presented on Sheets 92B through 92F, and the dial readings are presented on Sheets through .

An examination of these curves indicate that a very major part of the settlement has usually taken place by the time the first reading (@ 0.1 min) is taken. This leads us to believe that it may have already started curving prior to the first reading. Except for the 4-Ksf results, for all others, an average value is about 0.25 min, i.e. 15 seconds.

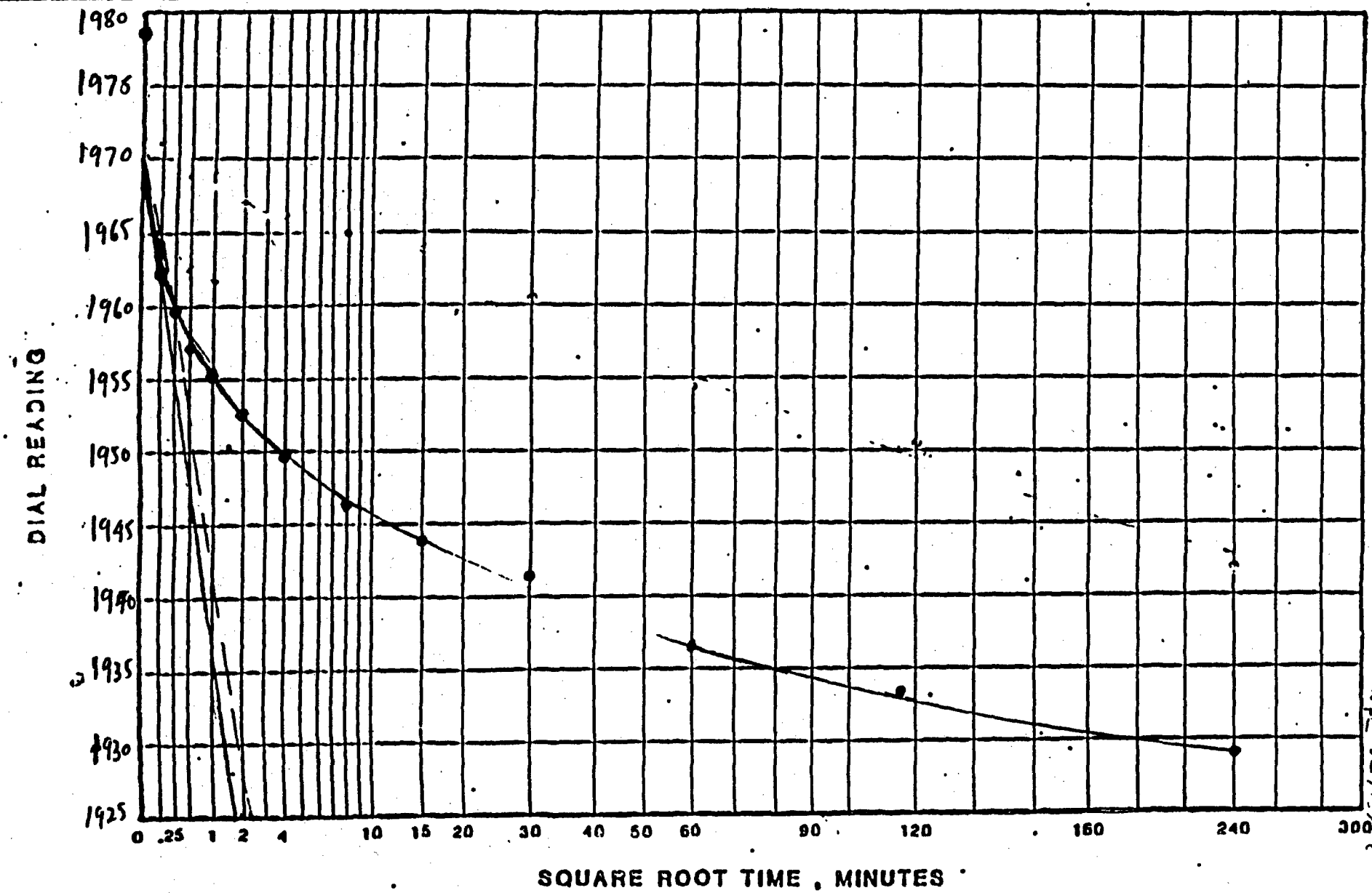
$$C_v = \frac{0.848\left(\frac{0.75}{12\times 2}\right)^2}{.15} = 5.52\times 10^{-5}\ ft^2/sec = .0513\ cm^2/sec$$

Calculating indirectly, using concepts of Sheet 60

$$C_v = \frac{K\,a_v}{(1+e_o)\gamma_w} = \frac{1\times 10^{-6}\times\frac{1}{30.48}\times(1.437)}{\frac{1}{2.303}(.018)\times\frac{1}{1}\times .0624} \cong 1\times 10^{-4}\ ft^2/sec$$

$\therefore$ Use $5.5\times 10^{-5}\ ft^2/sec$.

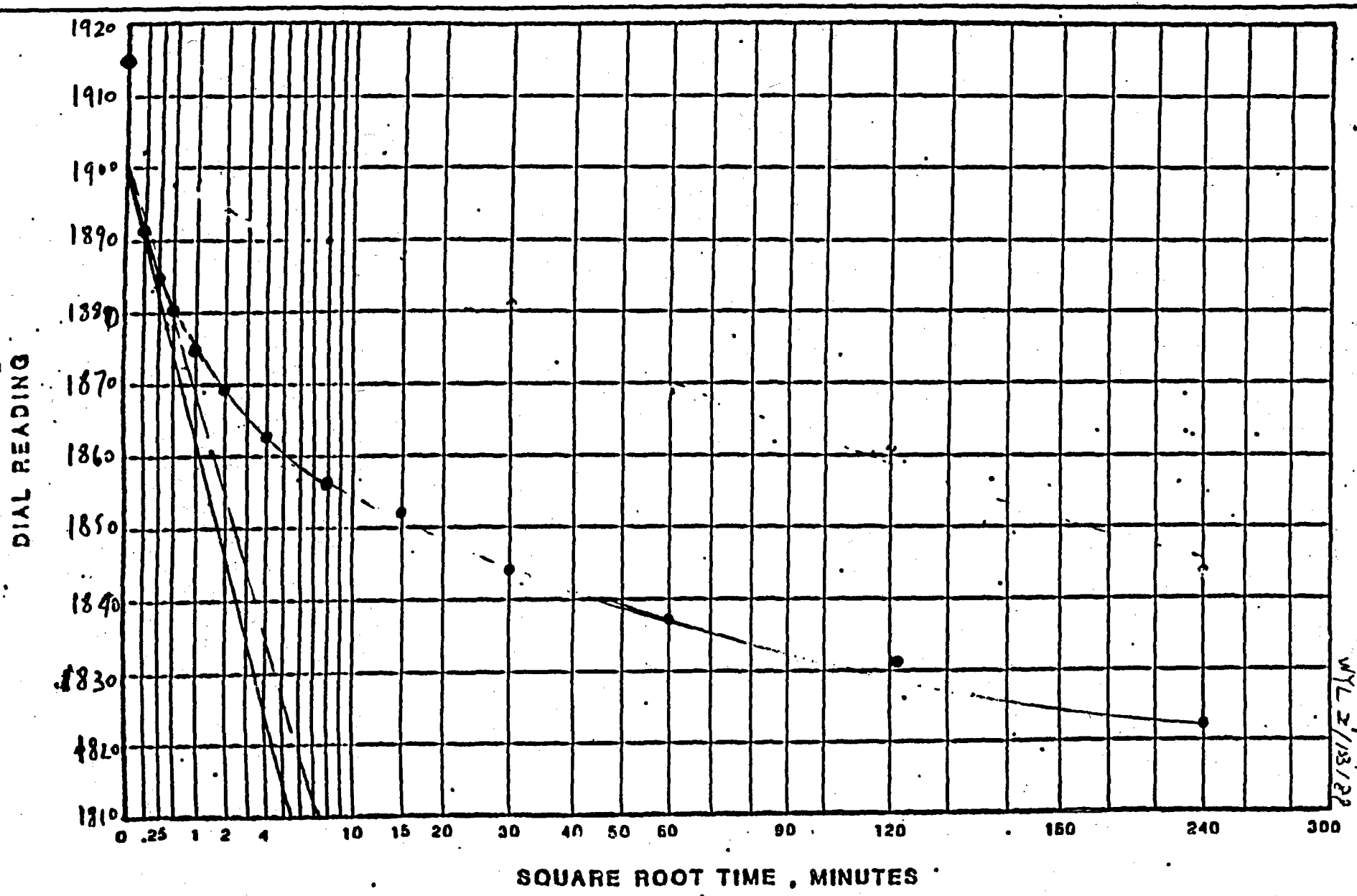
t_{90} = 0.25 min.



Pg. 2/13/53

HOLE B116-29	DEPTH 0-20	SAMPLE NO. ____	LOAD STRESS 7000 PSF	TIME SETTLEMENT CURVE
c_v ____	cm^2/sec	σ_v ____	K_{avg} ____ cm/sec	

$t_{90} = 0.25$ min



PS 2/13/85
WYL 2/13/22

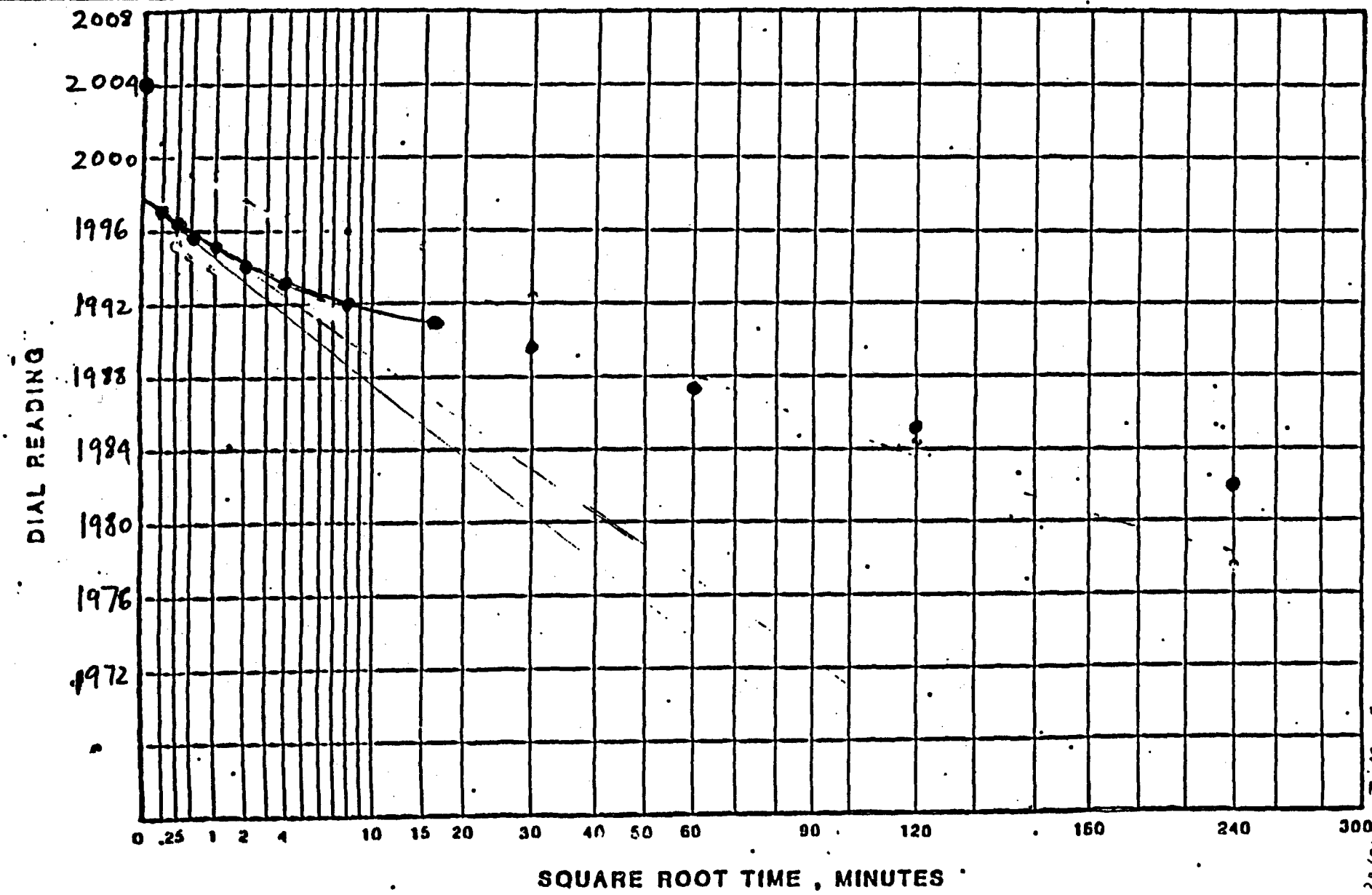
HOLE B116-29 DEPTH 0-20 SAMPLE NO. _____ LOAD STRESS 2000 PSF

c_v _____ cm²/sec a_v _____ K_{avg} _____ cm/sec

TIME SETTLEMENT CURVE

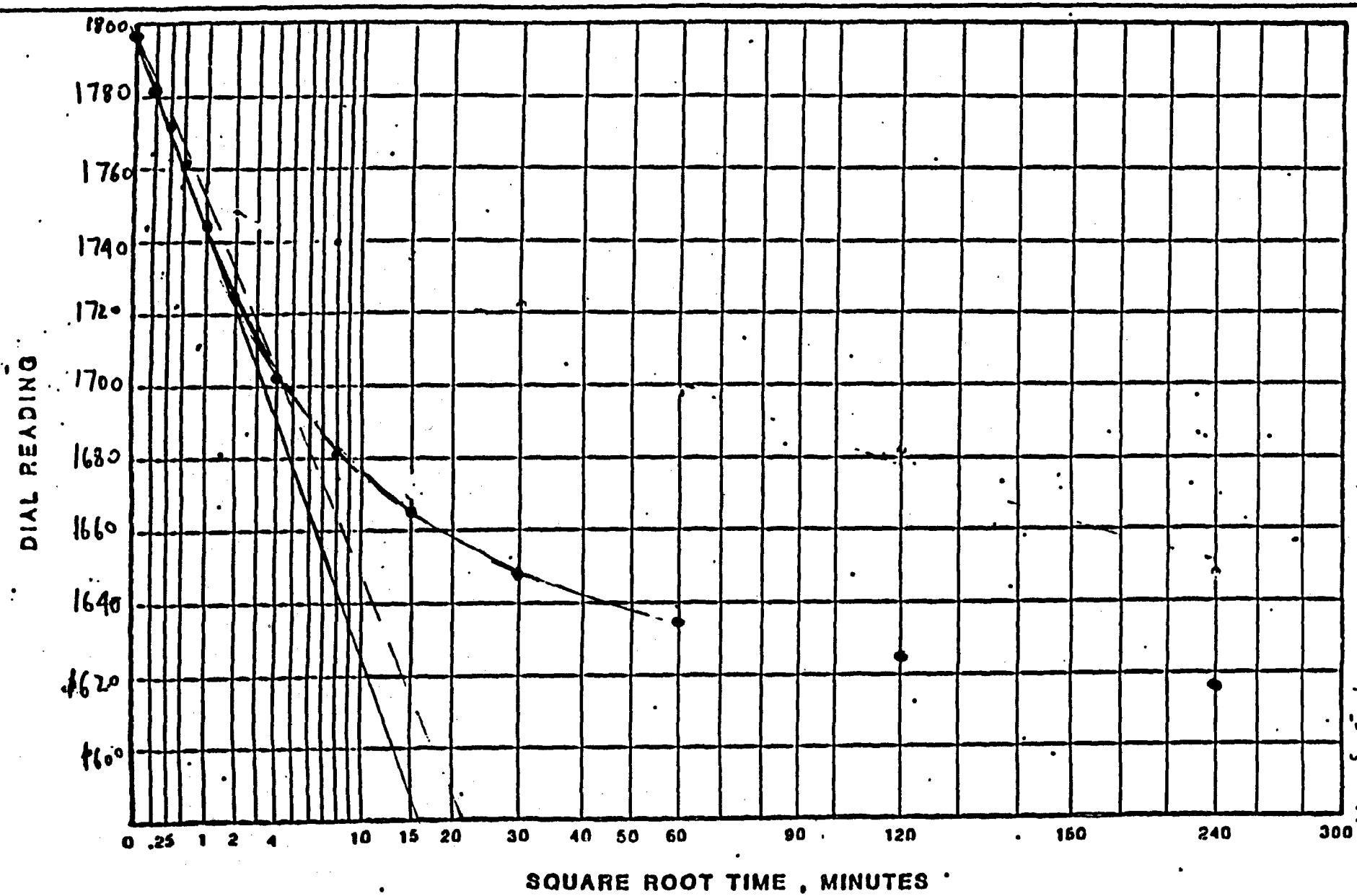
92

$t_{90} = 0.5$ min.



PS 2/13/84

HOLE B116-29	DEPTH 0-20	SAMPLE NO.	LOAD STRESS 500 PSF	TIME SETTLEMENT CURVE
c_v	cm^2/sec	σ_v'	K_{avg} cm/sec	



HOLE B116-29 DEPTH 0-20 SAMPLE NO. _____ LOAD STRESS 4 000 PSF

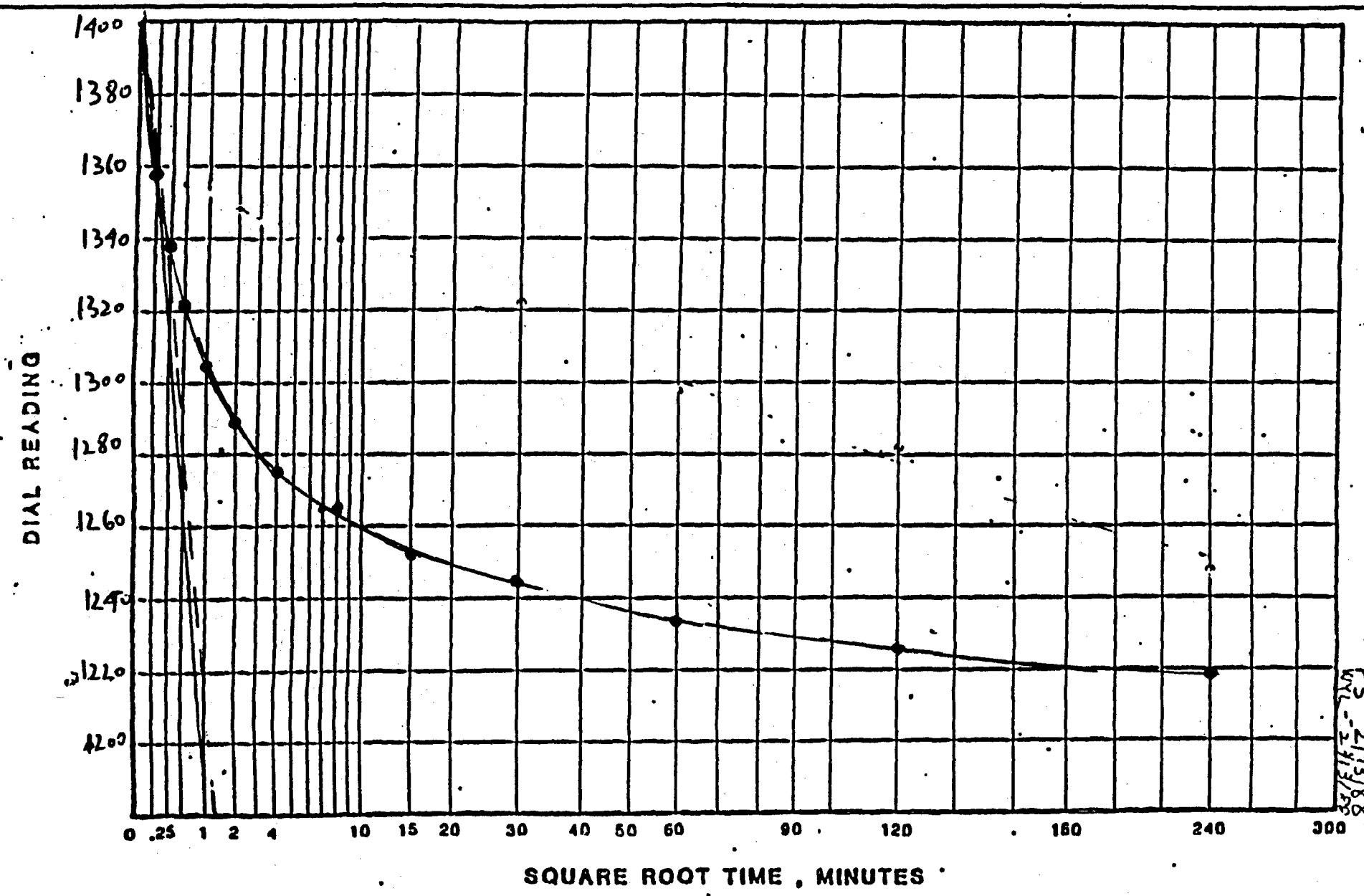
c_v _____ cm^2/sec σ_v' _____ K_{avg} _____ cm/sec

TIME SETTLEMENT CURVE

PS 2/13/87

MYL 2/13/87

$t_{90} = 0.25$ min.



PS - 2/13/88

HOLE B116-29 DEPTH 0-20 SAMPLE NO. _____ LOAD STRESS 8000 PSF

c_v _____ cm^2/sec a_v _____ K_{avg} _____ cm/sec

TIME SETTLEMENT CURVE

Permeability

The results of the two permeability tests performed are presented on the next sheet. A value of 2.0×10^{-4} cm/sec. is recommended for design.

9K

PS 2/4

WYL 2/5/88

Chen & Associates, Inc.

Table I

Summary of Permeability Test Results Job No. 1 343 87

Sample No.	Depth Feet	Initial Moisture Content, %	Initial Dry Density, pcf	Initial Compaction, %	Final Moisture Content, %	Final Dry Density, pcf	Coefficient of Permeability, cm/sec
BH6-29	0-20	9.8	104.1	89.5	21.8	109.2	7.4×10^{-7}
33F-B	0-20	10.6	104.2	89.7	21.6	109.6	2.0×10^{-6}

RECEIVED-MKE
SEP 8 1987
UMTRA-S.F.

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project _____ Contract No. _____ Sheet _____ File No. _____
Feature _____ Designed _____ Date _____
Item _____ Checked _____ Date _____

CHENEY DISPOSAL SITE

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project
Feature Embankment Design
Item

Contract No.
Designed
Checked WYL

Sheet
File No.
Date
Date 2/5/88

Introduction

In-Situ Foundation Soils

Boring and test pits in the proposed disposal site (Cherry Reservoir) revealed a highly weathered shale bedrock (Mancos Shale), overlain by silty clay to silty gravel [illegible] approximately 30 feet thick. The terrain is generally sloping 2.5 % towards the west, a trend apparently [illegible]

REV. 3 H.L. 6/16/93 ~~The excavation plan [illegible] in Sheet 6-22-89 [illegible]~~

A [illegible] all the [illegible] and [illegible] the disposal site are [illegible] SA-

[illegible] drawn [illegible]

[illegible] 103, the [illegible] plan [illegible] 99

REV. 3 H.L. 6/16

Geological Cross-section is shown [illegible] 99B

An examination of the sections reveals [illegible]

[illegible] embankment

REV. 3, SEE Sheet 96 H.L. 6/16/90

Layer 1 — Sandy silt to silty clay

Layer 2 — Silty gravel

Layer 3 — Very stiff hard silt

Layer 4 — Silty gravel (sometimes absent)

Layer 5 — Very weathered shale (fissile)

REV. 3 H.L. 6/16/90 ~~The tailings pile will be founded on either layer 2 or layer 3.~~

Our limited investigations indicate that the upper layers are alluvial deposits, whereas the lower layers are weathered residual soils from Mancos Shale. The transition takes place somewhere in Layer 3, where both 'transported' clays and remnants of shale are evidenced. Usually, the former overlies the latter.

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project JMTRA - [illegible]
Feature EMBANKMENT DESIGN
Item MATERIAL PROPERTIES

Contract No. [illegible]
Designed H.L.
Checked [illegible]

Sheet [illegible]
File No.
Date [illegible]
Date [illegible]

REVISION 3

The soil or rock types in REVISION 3 are:

LAYER 1 – Top Soil (Source of material [illegible] Roadbed Soil and Restoration)

LAYER 2 – Gravelly Layer (Source of Riprap and -1" material)

LAYER 3 – Clayey Layer (Source of Reservoir Barrier Material)

LAYER 4 – Mancos Shale Layer (Source of Clean-Fill Dike)

LAYER 5 – General Fill – Mixed [illegible] (clay and sand)

(NOTE: The stockpile [illegible] provided by [illegible] contractors (see [illegible]) is shown Sheet [illegible])

The available soil test results are summarized by layers on the following sheets. Pertinent design properties are [illegible] in the following [illegible] summarized on sheet 3.

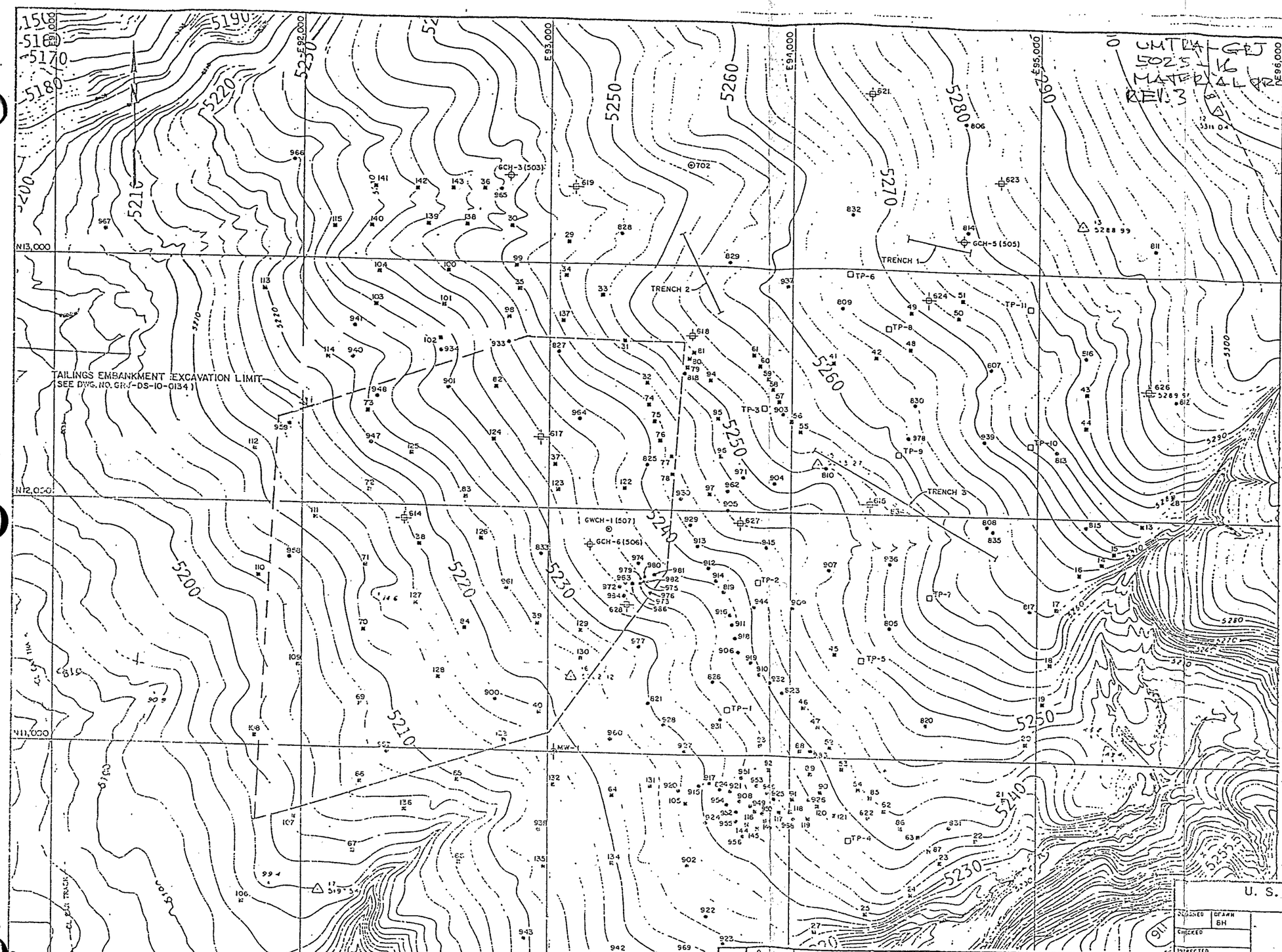
L.[illegible]

Foundation Deformations

Of all the layers in the foundation, the Layer 3 hard clays are the most significant from the standpoint of deformation (settlement) under the load from the newly placed embankment. The granular layers (Layer 2) ~~and Layer 4~~ ~~are~~ is essentially incompressible; in addition, the settlement will occur relatively rapidly, such that all the settlements will have taken place while the embankment is placed. The bedrock is also essentially incompressible. Layer 1 is only 2 to 3 ft thick, and is absent (excavated out) in most of the foundation.

REV. 3 01.0.93 H.L.

The pertinent soil properties for the computation of deformations in Layer 3 are presented in the appropriate section.



BOREHOLES & TEST PITS	COORDINATES	
GWCH-2 (508)	N14,170	E96,180
GWCH-3 (509)	N14,335	E94,140
GCH-1 (501)	N15,240	E95,360
GCH-2 (502)	N14,280	E94,160
GCH-4 (504)	N14,140	E96,240
613	N15,270	E95,100
701	N14,460	E95,020
26	N10,170	E93,985
935	NOT DRILLED	
985	NOT DRILLED	

UMTRA-GJ
50251-16
MATERIAL PROPERTIES
REV. 3
H-L. 6/16/92
SJB 6-22-90

NOTE:

1. GROUND WATER DATA AND SUBSURFACE EXPLORATION LOGS ARE AVAILABLE FROM THE CONTRACTOR.
2. BOREHOLES AND TEST PITS NOT SHOWN ON DRAWING ARE LISTED ON TABLE ABOVE.

REFERENCE DRAWINGS:

GRJ - DS - 10-0125 CONSTRUCTION FACILITIES AND SITE DRAINAGE

GRJ - DS - 10-0134 TAILINGS EMBANKMENT AREA EXCAVATION PLAN

LEGEND:

5240 — EXISTING SITE FEATURES AND CONTOURS
N12,000 — CONSTRUCTION GRID COORDINATE
GCH-6 — BORINGS DRILLED BY NUS IN 1982
GWCH-3 — MONITOR WELLS INSTALLED IN 1982
701 — MONITOR WELLS INSTALLED BY TAC IN 1985
616 — TEST PITS DUG BY TAC IN 1984
617 — TEST PITS DUG BY LINCOLN DEVORE/MKE IN 1985
TP-5 — TEST PITS DUG BY WESTERN ENG./MKE IN 1987
MW-1 — EXISTING WATER SUPPLY WELL BY MKE IN 1987
41 — TEST PIT BY TAC IN 1989
930 — WELL-BOREHOLE BY TAC IN 1985

200 0 200 400
SCALE FEET

NO.	DATE	REVISIONS	BY	CK	E&D MGR.	CHIEF ENG.	CA MGR.	DOE APP
1		SUPERSEDES DWG. NO. GRJ-DS-10-0125, REV. 0. REVISED SITE LOCATION						

U. S. DEPARTMENT OF ENERGY
ALBUQUERQUE, NEW MEXICO

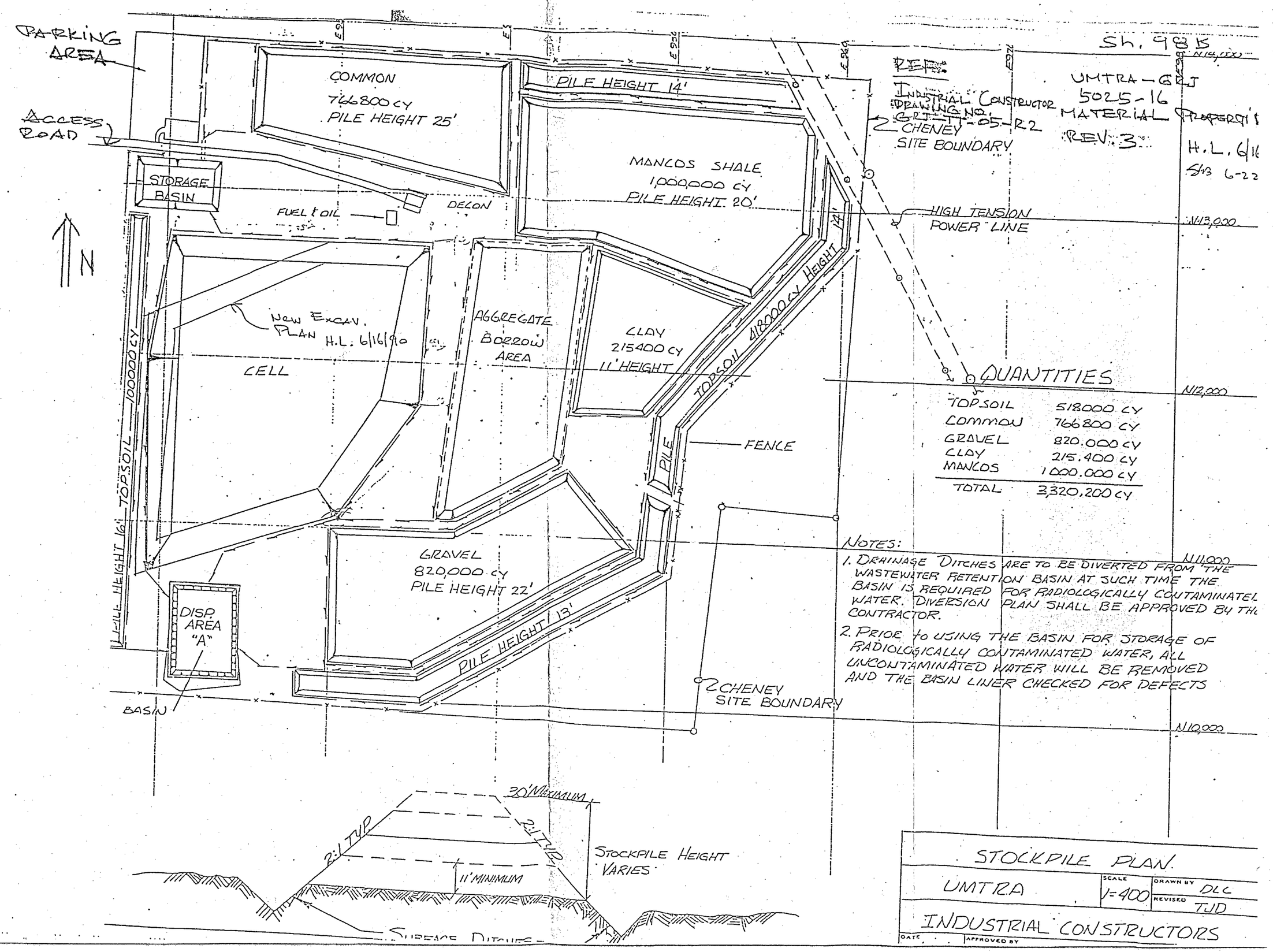
DESIGNED / DRAWN BH
CHECKED
INSPECTED
RECOMMENDED
APPROVED — DATE — DOE PROJECT ENGINEER — DATE

CHENEY RESERVOIR DISPOSAL SITE
GRAND JUNCTION, COLORADO
PHASE II CONSTRUCTION

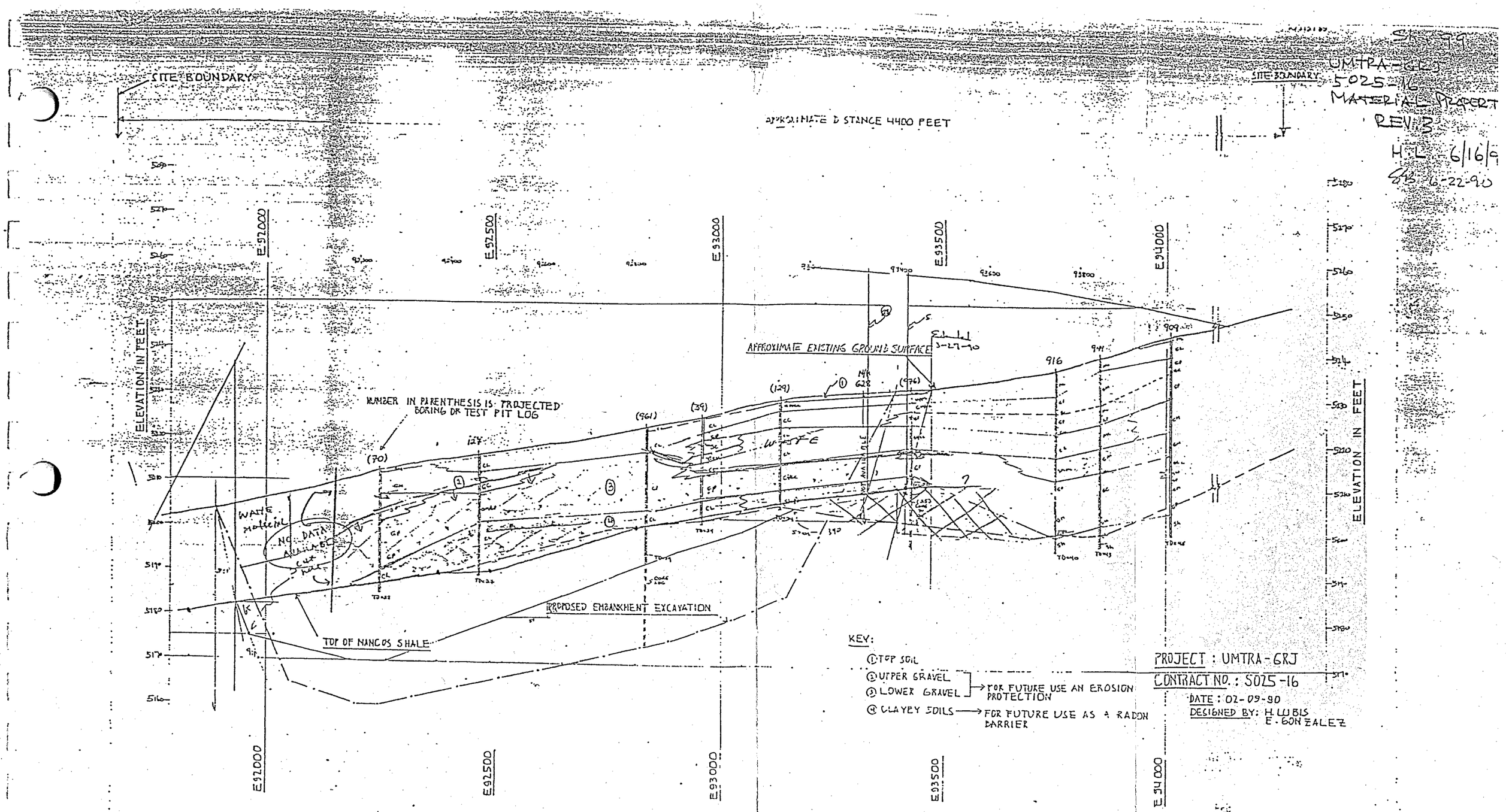
BORINGS AND TEST PITS LOCATION PLAN

MORRISON-KNUDSEN ENGINEERS, INC.
UMTRA PROJECT
180 HOWARD ST. SAN FRANCISCO, CA 94105

PROJECT NO. DE-AC04-83AL18796
DRAWING NO. GRJ-DS-10-0223
REV. 1


PARKING AREA
ACCESS ROAD
COMMON
766800 CY
PILE HEIGHT 25'
PILE HEIGHT 14'
STORAGE BASIN
FUEL & OIL
DECON
MANCOS SHALE
1,000,000 CY
PILE HEIGHT 20'
N
NEW EXCAV. PLAN H.L. 6/16/90
CELL
AGGREGATE BORROW AREA
CLAY
215400 CY
11' HEIGHT
TOPSOIL 418000 CY HEIGHT
14'
100000 CY
HEIGHT 16' TOPSOIL
FENCE
PILE
GRAVEL
820,000 CY
PILE HEIGHT 22'
PILE HEIGHT 13'
DISP. AREA "A"
BASIN
CHENEY SITE BOUNDARY
HIGH TENSION POWER LINE
REF:
INDUSTRIAL CONSTRUCTOR DRAWING NO. GRTT-05-R2
SITE BOUNDARY
Sh. 9815
UMTRA-GJ
5025-16
MATERIAL PROPERTIES
REV. 3
H.L. 6/16
N14,000
N13,000
N12,000
N11,000
N10,000
QUANTITIES
TOPSOIL 518000 CY
COMMON 766800 CY
GRAVEL 820,000 CY
CLAY 215,400 CY
MANCOS 1,000,000 CY
TOTAL 3,320,200 CY
NOTES:
1. DRAINAGE DITCHES ARE TO BE DIVERTED FROM THE WASTEWATER RETENTION BASIN AT SUCH TIME THE BASIN IS REQUIRED FOR RADIOLOGICALLY CONTAMINATED WATER. DIVERSION PLAN SHALL BE APPROVED BY THE CONTRACTOR.
2. PRIOR TO USING THE BASIN FOR STORAGE OF RADIOLOGICALLY CONTAMINATED WATER, ALL UNCONTAMINATED WATER WILL BE REMOVED AND THE BASIN LINER CHECKED FOR DEFECTS
30' MAXIMUM
2:1 TYP
2:1 TYP
11' MINIMUM
STOCKPILE HEIGHT VARIES
SURFACE DITCHES
STOCKPILE PLAN
UMTRA
SCALE 1=400
DRAWN BY DLC
REVISED TJD
INDUSTRIAL CONSTRUCTORS
DATE
APPROVED BY

Sh. 99
UMTRA-GRJ
5025-16
MATERIAL PROPERT
REV. 3
H.L. 6/16/9
6-22-90

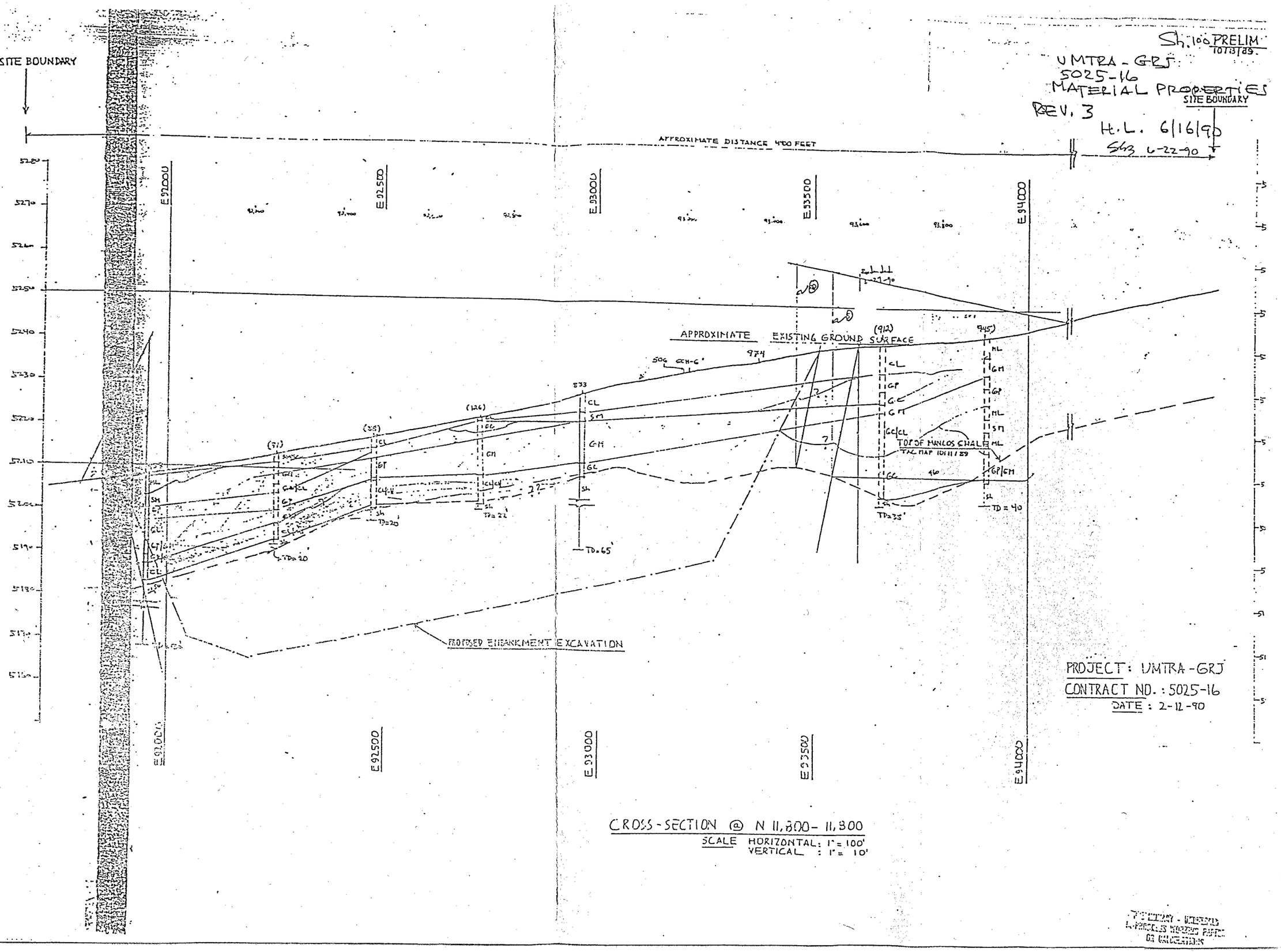


CROSS-SECTION @ N 11,600 - 11,600
SCALE HORIZONTAL : 1" = 100'
VERTICAL : 1" = 10'

→ SEE EXCAVATION PLAN ON Sh. 98 FOR CROSS SECTION LOCATION

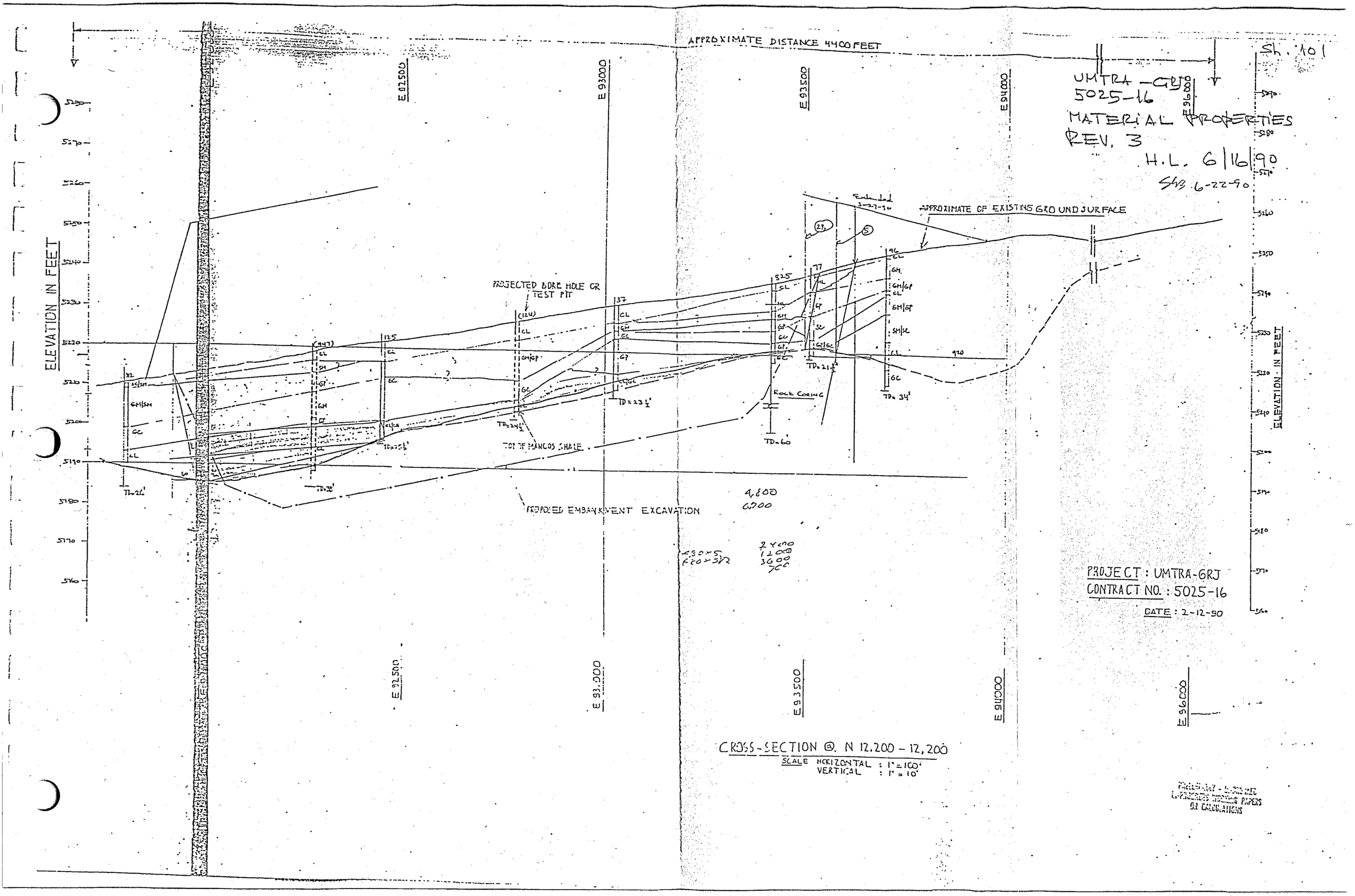
Sh. 100 PRELIM
10/13/89

UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
REV. 3
H.L. 6/16/90
SKB 6-22-90



CROSS-SECTION @ N 11,800-11,300

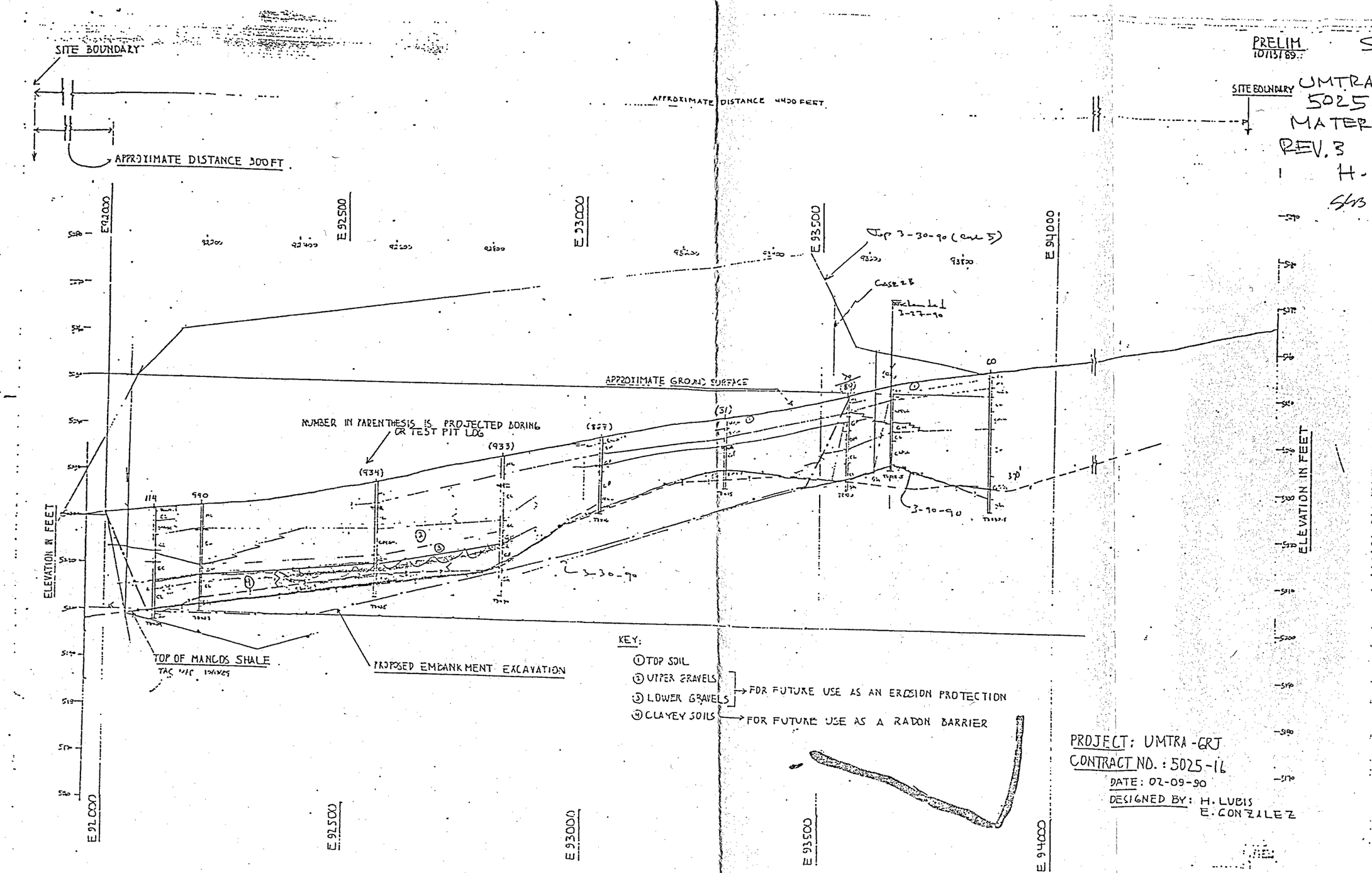
SCALE HORIZONTAL: 1" = 100'
VERTICAL : 1" = 10'


Sh. 101
UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
REV. 3
H.L. 6/16/90
SHB 6-22-90
APPROXIMATE DISTANCE 4400 FEET
E 92500
E 93000
E 93500
E 94000
E 96000
ELEVATION IN FEET
APPROXIMATE OF EXISTING GROUND SURFACE
PROJECTED BORE HOLE OR TEST PIT
TOP OF MANCOS SHALE
PROPOSED EMBANKMENT EXCAVATION
ROCK CORING
TD=60'
TD=34'
TD=23½'
TD=24½'
TD=21½'
4,800
6,000
24,000
12,000
30,000
700
PROJECT : UMTRA-GRJ
CONTRACT NO. : 5025-16
DATE : 2-12-90
CROSS-SECTION @ N 12,200 - 12,200
SCALE HORIZONTAL : 1" = 100'
VERTICAL : 1" = 10'
PRELIMINARY - UNCHECKED
EXPERIMENTAL WORKING PAPERS
OR CALCULATIONS

PRELIM 10/13/89

Sh. 102

SITE BOUNDARY UMTRA-GRJ 5025-16 MATERIAL PROPER REV. 3 6/16/90 H.L. 6/13 6-22-90

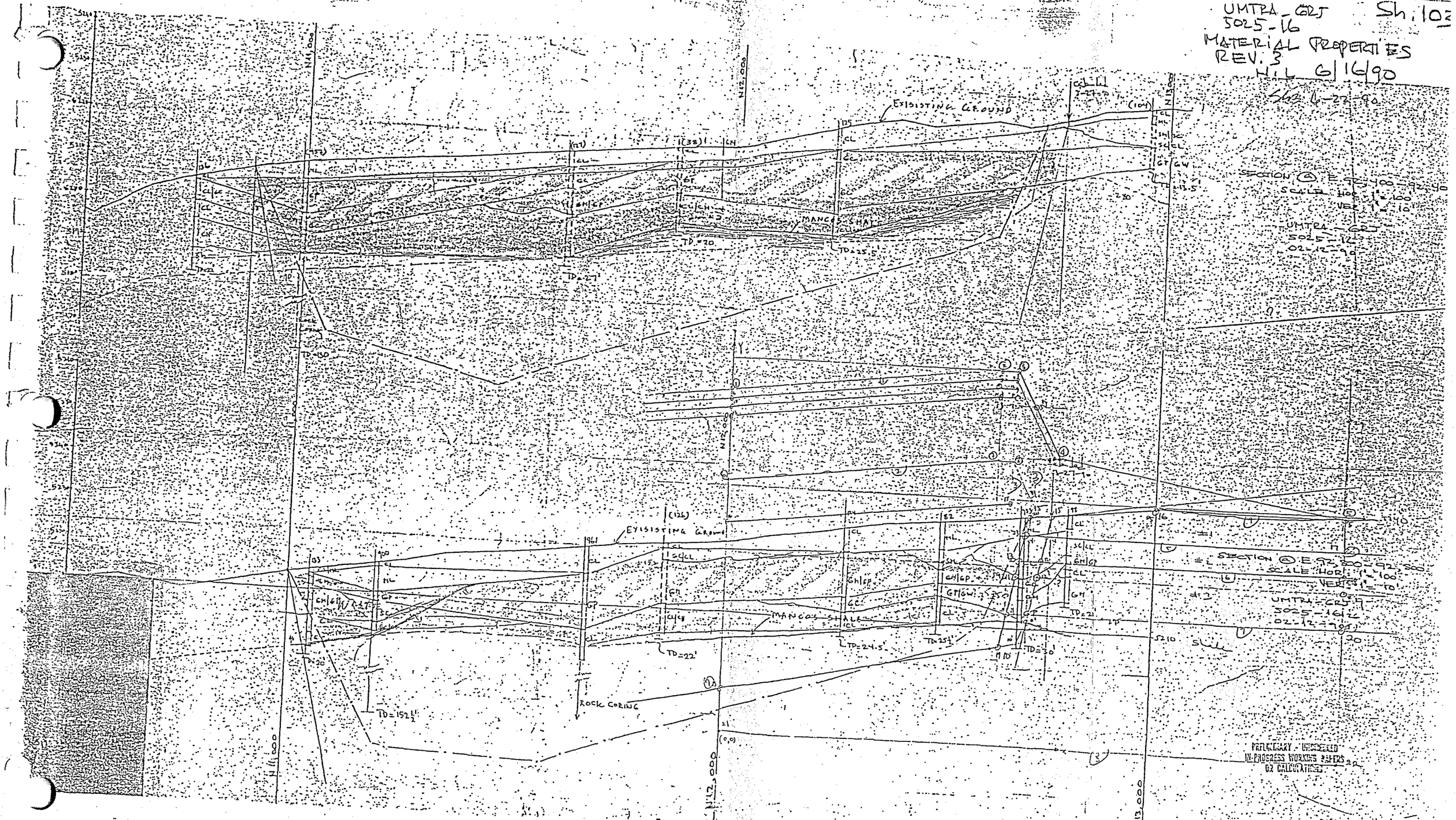


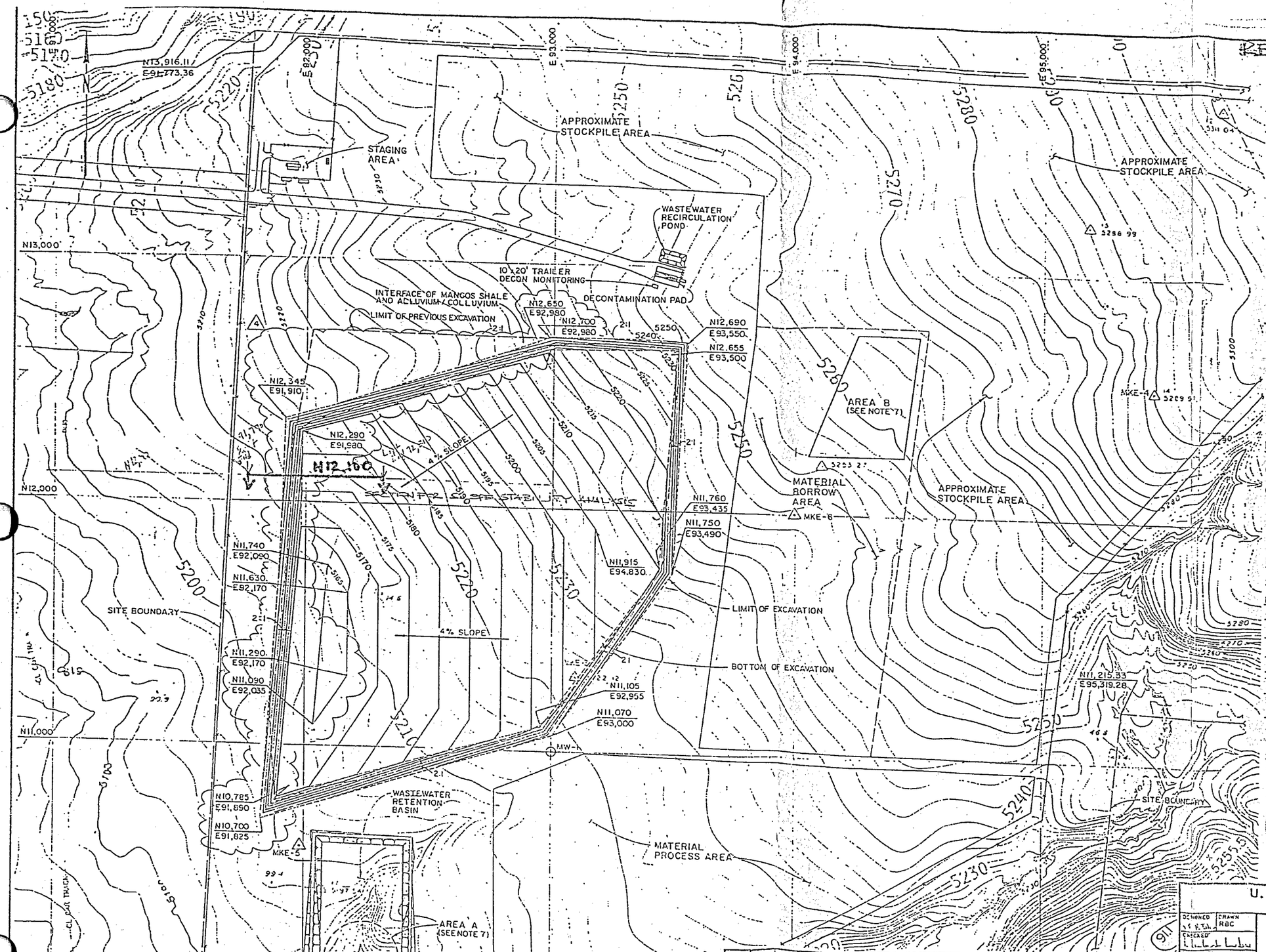
PROJECT: UMTRA-GRJ
CONTRACT NO.: 5025-16
DATE: 02-09-90
DESIGNED BY: H. LUBIS
E. GONZALEZ

CROSS-SECTION @ N 12,600 - 12,600

SCALE: HORIZONTAL : 1" = 100'
VERTICAL : 1" = 10'

UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
REV. 3
H.L. 6/16/90
Sh. 102
Sh. 4-22-90





REV. 3 UMTRA-GRJ -5025-16 H.L. 6/15

NOTES: MATERIAL PROPERTIES

1. ALL UNCONTAMINATED MATERIAL EXCAVATED FROM THE SITE DURING CONSTRUCTION SHALL BE PLACED TEMPORARILY WITHIN THE AREAS INDICATED WITH STABLE SLOPES NO STEEPER THAN 2(H) TO 1(V). MAXIMUM HEIGHT OF THE PILE SHALL BE 30 FEET.
2. THE SUBCONTRACTOR SHALL PROVIDE SUMP AND PUMP TO DRAIN THE PONDED NON-CONTAMINATED WATER FROM RAINSTORM AND SNOW MELT FROM THE TAILINGS EMBANKMENT AREA INTO THE NATURAL DRAIN.
3. THE SUBCONTRACTOR SHALL NOT EXCAVATE INTO THE MANCOS SHALE PRIOR TO THE ISSUANCE OF SUCH INSTRUCTIONS BY THE CONTRACTOR.
4. (DELETED)
5. THE LOCATION OF THE INTERFACE OF MANCOS SHALE WITH ALLUVIUM/COLLUVIUM IS APPROXIMATE ONLY AND IS BASED ON SOIL BORING LOGS. ACTUAL LOCATION MAY VARY IN THE FIELD.
6. THE DESIGNATED MATERIAL BORROW AREA SHALL NOT BE USED AS STOCKPILE AREA WITHOUT THE CONTRACTOR'S APPROVAL.
7. THE SUBCONTRACTOR MAY SPOIL OVERSIZE ROCK FROM THE DISPOSAL CELL EXCAVATION INTO THE DRAINAGE GULLY UNDER THE PROPOSED WASTEWATER RETENTION BASIN (AREA A) IF, AND AS AVAILABLE. IF THIS AREA IS NOT AVAILABLE, THE SUBCONTRACTOR SHALL TEMPORARILY STOCKPILE THE OVERSIZE MATERIAL IN THE DESIGNATED AREA B. HE SHALL DISPOSE OF THIS MATERIAL AS REQUIRED BY THE CONTRACTOR.
8. LOCATIONS OF WASTEWATER RECIRCULATION POND AND DECONTAMINATION PAD ARE APPROXIMATE.
9. A WORKING BENCH APPROXIMATELY 20' WIDE SHALL BE PLACED BELOW THE TRANSITION BETWEEN MANCOS SHALE AND ALLUVIUM. THE SIDESLOPE IN THE MANCOS MAY VARY AS APPROVED BY THE CONTRACTOR.

REFERENCE DRAWINGS:

GRJ-DS-10-0125, CONSTRUCTION FACILITIES AND SITE DRAINAGE
GRJ-DS-10-0223, BORINGS AND TEST PITS LOCATION PLAN

LEGEND:

- EXISTING SITE FEATURES AND CONTOURS
- N 12,000 — CONSTRUCTION GRID COORDINATE
- 5250 — CONTOURS OF EXCAVATION
- TEMPORARY CHAIN LINK FENCE AND GATE
- TEMPORARY WIRE FENCE
- ACCESS ROAD
- EXCAVATION
- EMBANKMENT
- MKE-3 — EXISTING PERMANENT SURVEY MONUMENT
- MW-1 — EXISTING WATER SUPPLY WELL

QA APPROVED FOR QUALITY REQUIREMENTS

200 0 200 400
SCALE FEET

NO	DATE	REVISIONS	BY	CK	END MGR.	CHIEF ENG.	QA MGR	DOE APV.
4	2-2-90	REVISED WEST SIDE TAILINGS EMBANKMENT EXCAVATION PLAN						
3	12-20-89	REVISED TAILINGS EMBANKMENT EXCAVATION PLAN. LOCATION OF STAGING AREA B DELETED NOTE 4. ADDED AREA B & NOTE 9						
2	11-17-89	SUPERSEDED DWG. NO GRJ-DS-10-0134, REV. 1 AND REVISED SITE LOCATION						
1	9-15-88	REVISED TAILINGS EXCAVATION AREA, GRADING TEMPORARY WIRE FENCE AND STOCKPILE AREA	1 FOR	SIGNATURES, SEE			SUPERSEDED	
0	7-15-88	ISSUED FOR CONSTRUCTION	DWG.	REV. 11				

QA MK-F

U. S. DEPARTMENT OF ENERGY
ALBUQUERQUE, NEW MEXICO

DESIGNED | DRAWN RBC | CHECKED | INSPECTED | RECOMMENDED | APPROVED | DATE 1/19/90

CHENEY RESERVOIR DISPOSAL SITE
GRAND JUNCTION, COLORADO
PHASE II CONSTRUCTION

TAILINGS EMBANKMENT AREA EXCAVATION PLAN

DOE PROJECT ENGINEER | DATE 2-2-90

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY
UMTRA PROJECT
180 HOWARD ST., SAN FRANCISCO, CA 94105

PROJECT NO. DE-AC04-83AL18796

DRAWING NO. GRJ-DS-10-0134 REV. 4

Layer 1 Soils – Silty Clays to Clayey Silts – In-situ

Introduction

The surficial soils at the disposal site consist of 2 to 3 feet of silty clays to clayey silts, which are reasonably dry. Test results from Drift Prep on this layer are presented in Sheet 104. Additional test results are presented on Sheet 105, as radon barrier material. The soil properties are developed below. Test results are summarized on the next sheet.

Design Soil Properties

Dry Density, γ_d = 83.4 pcf

In-situ Density γ = 93 pcf

In-situ moisture content = 11.3 pct

Saturated Density = 115 pcf

Shear Strength = S_u 500 psf (short term loading, including seismic loading)

= $\phi = 30°$, $c = 0$ (long term static loading)

REV. 3:

Due to the fact that the proposed embankment was moved to the western end of the site within the same location, the design soil properties of layer 1 can still be used in Revision 3. It is expected that these soils will be used as a rooting medium and restoration materials for disposal and processing site.

H.L. 6/18/90
SJB 6-22-90

SUMMARY OF SOIL TEST RESULTS

Job No 5025 — Project Name UMTRA – Grand Junction — Feature Cheney Reservoir Site — Date 2/5/88 WYL — 2/4/88 PS

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γ_d	Compaction Test	Compaction Optimum w	Compaction Optimum γ_d	Shear Strength (1) Test	Shear Strength Initial w	Shear Strength Initial γ_d	Shear Strength C/ϕ	Shear Strength C'/ϕ'	Permeability (1) γ_d (sat.)	Permeability (1) k cm/sec	Consolidation (1) C	Consolidation (1) CR/RR	Consolidation (1) e_0	Notes
LAYER NO. 1																										
GCH-1		0.2	2.8	CL			78	41	19		10.2	78.3														
GCH-2		0.2	1.5	CL/ML			61	37	14		14.5	85.3														
GCH-4		0.5	2.5	ML/SM			49	40	18		9.6	84.9														
GCH-6		0	1.8	CL/ML			67	24	8		14.5	85.1														
GBH-2		0	4.2	CL				35	16	2.72	9.7		SP	21.5	105.0											
614		0.5	1.5	CL	0	20	80	37	20	2.73	12.2		SP	19.2	103.4	UU	21.0	98.5	1.3/0		98.8 (100%)	8.1×10^{-8}		0.13/0.02		Water retention at 15 bars of pressure for 95% standard proctor on sample = 17.6 %
																CU	20.9	98.9	0/20.0°	0/30.0°	101.6 (100%)	2.1×10^{-8}				
702		3.0	4.0	CL	0	30	70																			
702		8.0	9.0	SM/ML	0	54	46																			
Additional Data From Testpits																										
TP-3		0	4.5		12	16	72	45	25	2.75	9.9		SP	20.5	106.0							1.19×10^{-7}				clay content %
4		0	2.5		10	17	73	35	16	2.72	10.1		SP	15.5	111.4						105.6	2.05×10^{-8}				30
6		0	3.0		7	20	73	41	22	2.78	14.9		SP	19.4	106.9							1.56×10^{-7}				22
8		0	3.0		26	38	36	33	9		6.5		SP	13.8	115.7											24
10		0	2.5		9	13	78	43	26	2.73	12.7		SP	17.1	109.0							9.71×10^{-8}				0
																										22
Average					9	26	65	37	18	2.74	11.3	83.4		18.1	108.2							8.24×10^{-8}		0.13/0.02		

Note:

(1) indicated 95% compaction ASTM D698

CR = Virgin Compression Ratio $CR = \frac{C_c}{1+e_0}$

RR = Recompression Ratio $RR = \frac{C_r}{1+e_0}$

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

Note: Material from Layer 1 will be used as Rooting Soil. See data for Rooting Soil in Sheets thru . The grain-size distribution and compaction data for this layer are presented there. This material was originally planned for use as radon barrier.

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Project UMTRA-GRJ	Contract No. 5025	Sheet 106	
Feature Embankment Design	Designed PS	File No.	Date 2/4/88
Item Soil Properties	Checked WYL	Date 2/5/88	

Specific Gravity, Density and Moisture Content
From Summary of Soil Test Results (Sheet 105) H.L. 6/18/90

(a) Specific Gravity = 2.74

(b) Dry density, avg. = 83.4 pcf

(c) Avg. in-situ moisture content = 11.3 pct

∴ (d) Insitu Density = 83.4 × 1.113 = 92.8 pcf Use 93 pcf

(e) Saturated Density:

Void Ratio, $e = \frac{62.4 \times 2.74}{83.4} - 1 = 1.05$

$\therefore w_{sat} = e/2.74 = \frac{1.05}{2.74} = 0.38 = 38$ pct

$\therefore \gamma_{sat} = 83.4 \times 1.38 = 115.4$ pcf Use 115 pcf

Shear Strength Properties of Layer 1 Soils

$LL_{ave} = 37$
$PI_{ave} = 18$
$(\gamma_d)_{ave} = 83.4$

(See Sheet 105) H.L. 6/18/90

$\therefore PL_{ave} = 19$. (Note: PL = LL − PI)

Using $G_s = 2.74$

Void ratio, $e = \frac{G_s \gamma_w}{(\gamma_d)_{ave}} - 1 = \frac{2.74 \times 62.4}{83.4} - 1 = 1.05$

If saturated, w dis. content, $w = \frac{e}{G_s} = \frac{1.05}{2.74} \approx 38.3$, i.e. $\approx LL$

$\therefore LI \approx 1.0$

Using sensitivity = 3.

$\therefore P_p' = 0.15$ tsf $= 0.3$ ksf (Ref. 16, Page 142) = 300 psf H.L. 6/18/90

$(S_u/p)_{NC} = 0.11 + .0037 \times 18 = 0.177$ (Skempton Eq., Ref. 16, page 141)

$(OCR)_{ave} = \frac{300}{115 \times 1} = 2.6$. $\therefore S_u = 115 \times 0.177 \times (2.6)^{0.8} = 43.7$ psf (Ref. 14, p. 57)

This value is very low, since blow counts range from 24 to 50/6". Thus, the soil is at least very stiff due to cementation. Conservatively, use $S_u = 500$ psf, i.e. barely medium stiff. Use this for short term loading, including all seismic loadings.

For long-term static loading, use $\phi' = 30°$, $c' = 0$, corresponding to PI = 18 (see next Sheet)

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(FROM: Ref. 14, page 57)

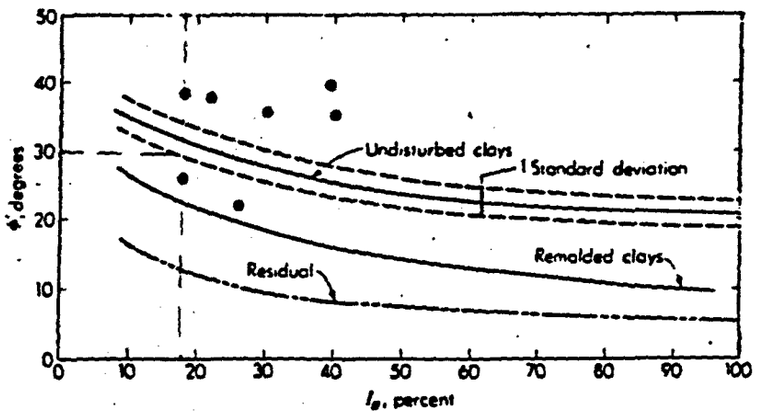

Figure 2-20 Correlation between ϕ' and plasticity index I_p for normally consolidated clays. Approximately 80 percent of data falls within one standard deviation. Typical extreme scatter values are shown. [*Data from several sources: DM-7, Ladd et al. (1977), Bjerrum and Simons (1960), Kanja and Wolle (1977).*] (Foundation Analysis And Design, Bowles)

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WYL 2/5/88

Table E.4.2 Summary of classification test results

Location ID	Sample depth (ft)	USCS classification	% passing #200 sieve	Atterberg limits (percent) LL	PL	PI
614[a]	0.5-1.5	CL	80	37	17	20
615[a]	3.5-5.0	~~SP-GP~~ GP	11	--	--	--
	7.0-8.0	CL	77	46	21	25
701-02	3-4	CL/GC	54	NA	NA	NA
701-07	15-16	CL	85	37	17	20
701-10	23-24	CL	59	NA	NA	NA
702-02	3-4	CH	75	NA	NA	NA
702-04	8-9	~~GC~~ SM/ML	46	NA	NA	NA
702-07	15-16	CL	44	NA	NA	NA
702-09	20-21	CL	85	41	20	21
702-11	25-26	~~SC~~ GC	14	NA	NA	NA
GCH-1[b]	0.2-2.8	CL	~~NA~~ 78	41	22	19
	33.0-33.5	CL		32	15	17
GCH-2[b]	0.2-15	CL-ML	~~NA~~ 61	37	23	14
	8.5-10.0	~~CL-ML~~ SM/ML		22	14	8
GCH-4[b]	0.5-2.5	~~CL~~ ML-SM	~~NA~~ 49	40	22	18
	13.0-15.0	~~CL-ML~~ GM		22	18	4
	43.0-44.0	CL		34	18	16
GCH-6[b]	0-1.8	CL-ML	~~NA~~ 67	24	16	8
	23.0-24.0	~~CL-ML~~ CL/GC		23	18	5
GBH-2[b]	0-4.2	CL	NA	35	19	16
	4.2-7.2	SC		29	19	10
	0-7.2	CL		--	--	--

[a]Sampled and reported by TAC.
[b]Reported by NUS, 1983.
NA - Not available.

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PC 2/-/88
WYL 2/5/88

Table E.4.3 Summary of specific gravity, in-situ moisture content, and in-situ dry density results

Location ID	Sample depth (ft)	USCS classification	Specific gravity	In-situ moisture (%)	In-situ density (pcf)
614[a]	0.5 - 1.5	CL	2.73	12.2	
615[a]	7.0 - 8.0	CL		14.2	
GCH-1[b]	0.2 - 2.8	CL		10.2	78.3
	33.0 - 33.5	CL		10.3	
GCH-2[b]	0.2 - 1.5	CL-ML		14.5	85.3
	8.5 - 10.0	~~CL-ML~~ SM/ML		5.3	
GCH-4[b]	0.5 - 2.5	~~CL~~ ML/SM		9.6	84.9
	13.0 - 15.0	~~CL-ML~~ GM		6.0	
	43.0 - 44.0	CL		10.4	
GCH-6[b]	0 - 1.8	CL-ML		14.5	85.1
	23.0 - 24.0	~~CL-ML~~ CL/SC		5.0	
GBH-2[b]	0 - 4.2	CL	2.72	9.7	
	4.2 - 7.2	SC	2.78	9.7	

[a]Sampled and reported by TAC.
[b]Reported by NUS, 1983.

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WYL 2/5/88

Table E.5.3 Summary of moisture-density results

Location ID	Sample depth (ft)	USCS classification	In-situ moisture (%)	Maximum dry density (pcf)	Optimum moisture (%)	Saturation percent at optimum moisture and maximum density
614-001[a]	0.5 - 1.5	CL	12.2	103.4	19.2	80.9
GBH-2[b]	0 - 4.2	CL	9.7	105	21.5	94.8
GBH-2[b]	4.2 - 7.2	SC	9.7	111	18.0	88.9

[a]Sampled and reported by TAC.
[b]Reported by NUS, 1983.

Table E.1.1 Summary of laboratory results for undisturbed soil samples from the Cheney Reservoir site

Borehole[a]	Sample depth (feet)	Description	USCS classification[b]	Moisture content (%)	Wet unit weight (lb/ft^3)	Percent passing through No. 200 sieve	Atterberg limits[c] (%)		
							LL	PL	PI
GCH-1 501	0.2-2.8	Silty clay	CL	10.2	86.3	78	41	22	19
GCH-1	33-34	Silty clay	CL	10.3	--	--	32	15	17
GCH-2 502	0.2-1.5	Sandy silt	CL-ML	14.5	97.7	61	37	23	14
GCH-2	8.5-10	Sandy silt	CL-ML	5.3	--	--	22	14	8
GCH-4 504	0.5-2.5	~~Silty clay~~ Sandy silt	CL ML-CM	9.6	93.0	49	40	22	18
GCH-4	13-15	Sandy silt	CL-ML	6.0	--	--	22	18	4
GCH-4	43-44	Clay shale	CL	10.4	--	--	34	18	16
GCH-6 506	0-1.8	Sandy silt	CL-ML	14.5	97.4	67	24	16	8
GCH-6	23-24	Sandy silt	CL-ML	5.0	--	--	23	18	5

[a]See Figure E.1.1 for location.
[b]Class in the Unified Soil Classification System. Key: SM, silty sands, sand-silt mixtures; CL, inorganic clays of low to medium plasticity, and gravelly, sandy, silty, and lean clays; ML, inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
[c]Atterberg limits: LL = liquid limit; PL = plastic limit; PI = plastic index.

Table E.5.4 Summary of saturated hydraulic conductivity for radon barrier materials

Sample ID	Sample depth (ft)	USCS class.	Dry density (pcf)	Percent[a] compaction	Final saturation (%)	Effective consolidation stress (psi)	Total press. head (ft)	Hydraulic conductivity (cm/sec)
614-001	0.5-1.5	CL	98.8	95	100	10	11.6	8.1×10^{-8}
614-001	0.5-1.5	CL	101.6	98	100	20	15.4	2.1×10^{-8}

[a]Percent compaction based on standard Proctor maximum dry density, ASTM D698.

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Table E.7.2 In-situ permeability tests at Cheney Reservoir

Location[a]	Depth interval (ft)	Permeability (cm/sec)
501	12-22	4×10^{-6}
	21-31	1×10^{-6}
	31-40	$<1 \times 10^{-7}$
	40-50	$<1 \times 10^{-7}$
502	10-14	2×10^{-4}
	20-31	3×10^{-6}
	36-46	$<1 \times 10^{-7}$
	41-51	$<1 \times 10^{-7}$
503	8-14	5×10^{-5}
	14-28	1×10^{-7}
	30-40	$<1 \times 10^{-7}$
	40-50	1×10^{-6}
504	10-18	3×10^{-5}
	22-34	5×10^{-6}
	35-44	6×10^{-5}
	44-56	$<5 \times 10^{-8}$
505	8-18	1×10^{-5}
	18-28	9×10^{-6}
	23-35	1×10^{-5}
	38-51	1×10^{-7}
506	10-19	6×10^{-5}
	21-29	1×10^{-4}
	29-40	6×10^{-5} to 1×10^{-4}
	40-51	1×10^{-5} to 5×10^{-5}

[a]Boring locations are shown in Figure E.7.1.

Ref. NUS, 1983.

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PS 2/4/88
WYL 2/5/88

Table E.5.5 Summary of capillary moisture results

Location ID-	Sample depth (ft)	Remolded moisture content (%)	Remolded dry density (pcf)	Water retention (%)	
				Bars of pressure	Result
614-001	0.5-1.5	21.0	98.7	-0.3	24.47
				-0.5	23.74
				-1.0	22.38
				-3.0	21.02
				-5.0	20.19
				-7.0	19.55
				-10.0	18.24
				-15.0	17.59

Table E.5.6 Specific gravity, absorption, sodium sulfate, and Los Angeles abrasion results

Sample ID	Sample depth (ft)	Specific gravity		Absorption (%)	Sodium sulfate soundness (percent loss)	Los Angeles abrasion (percent loss)	
		Apparent	Bulk			Fine	Coarse
614-002	2.5-3.5	2.56	2.47	1.4	NA	15.6	20.3
615-001	3.5-5.0	2.73	2.65	1.1	10.1	NA	25.1

NA - Not available.

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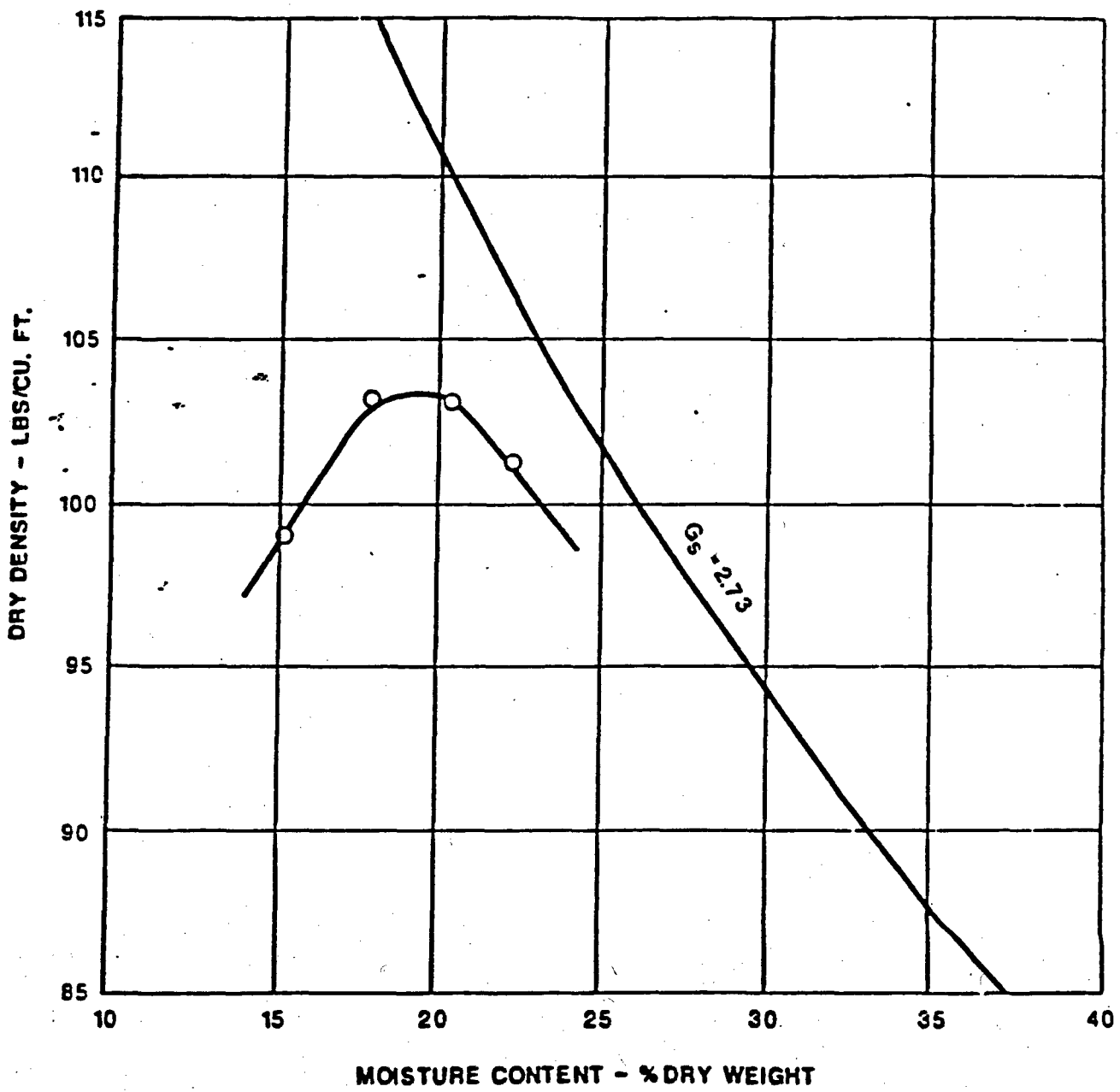
WYL 2/5/88



LEGEND			
SAMPLE GRJ-03	DEPTH INTERVAL (FT.)	OPTIMUM MOISTURE CONTENT (%)	MAXIMUM DRY DENSITY LBS/CU FT.
614	0.5 - 1.5	19.2	103.4

FIGURE E.5.1
SUMMARY OF MOISTURE-DENSITY RELATIONSHIP TESTS

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PS 2/4/88
WYL 2/4/88

Table E.5.3 Summary of moisture-density results

Location ID	Sample depth (ft)	USCS classification	In-situ moisture (%)	Maximum dry density (pcf)	Optimum moisture (%)	Saturation percent at optimum moisture and maximum density
614-001[a]	0.5 - 1.5	CL	12.2	103.4	19.2	80.9
GBH-2[b]	0 - 4.2	CL	9.7	105	21.5	94.8
GBH-2[b]	4.2 - 7.2	SC	9.7	111	18.0	88.9

[a]Sampled and reported by TAC.
[b]Reported by NUS, 1983.

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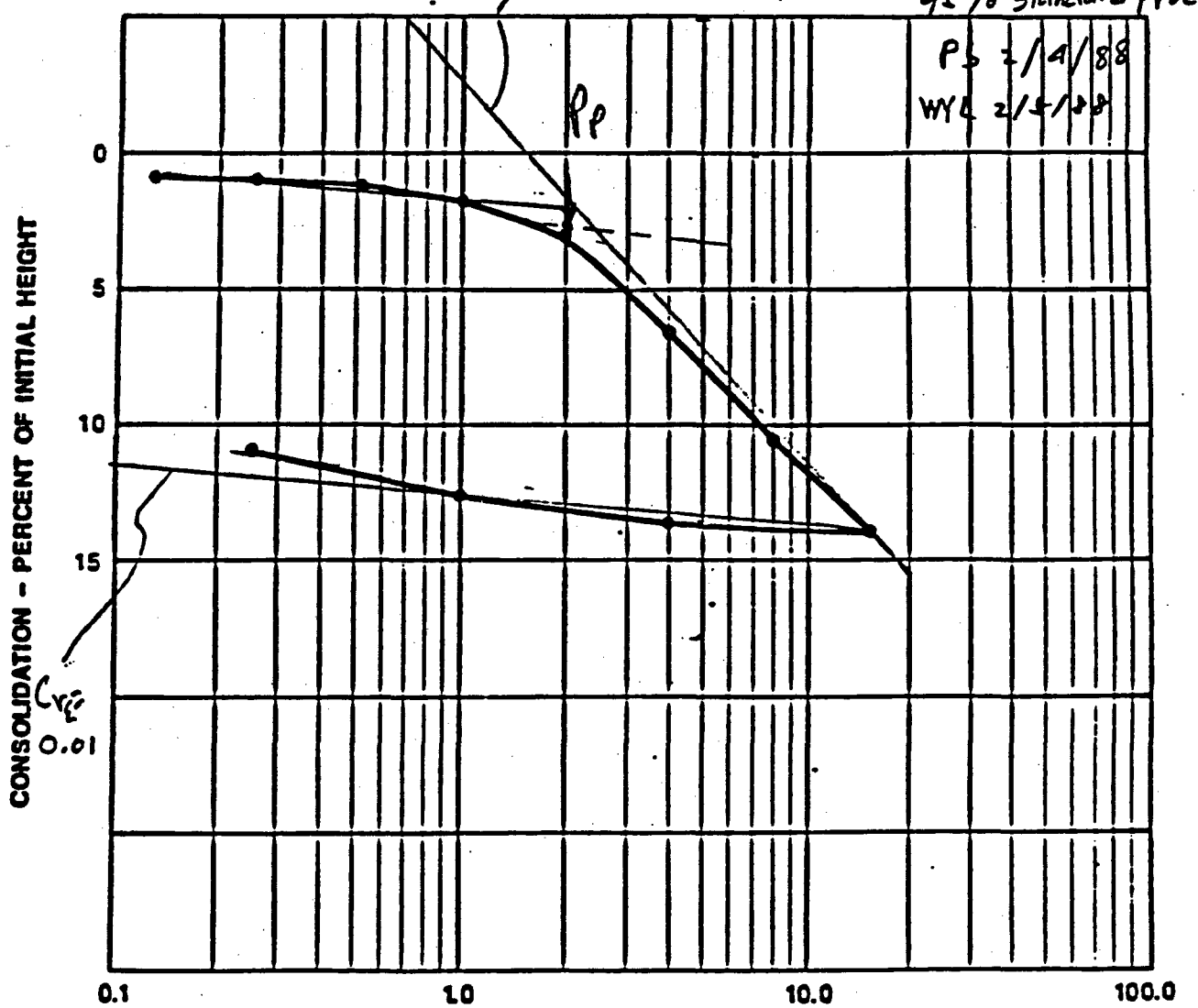
Remolded Sample

95% Standard Proctor

$C_{cE} = 0.14$

P_p



$C_{rE} = 0.01$

Remolded Sample

LL = 37; PI = 20

LEGEND						
SAMPLE GRJ-03	DEPTH INTERVAL (FT.)	DRY DENSITY PCF LBS./CU. FT.		MOISTURE CONTENT (%)		UNIFIED SOIL CLASSIFICATION
		INITIAL	FINAL	INITIAL	FINAL	
614	0.5-1.5	97.8	109.2	21.1	21.1	CL

Assume $G_s = 2.75$; $\therefore e_0 = \frac{62.4 \times 2.75}{97.8} - 1 = 0.7546$

$\therefore C_c = C_{cE} \times (1+e_0) = 0.14 \times 1.7546 \approx 0.25$

$C_r = C_{rE} \times (1+e_0) = 0.01 \times 1.7546 \approx 0.018$

FIGURE E.5.2

SUMMARY OF CONSOLIDATION TESTS

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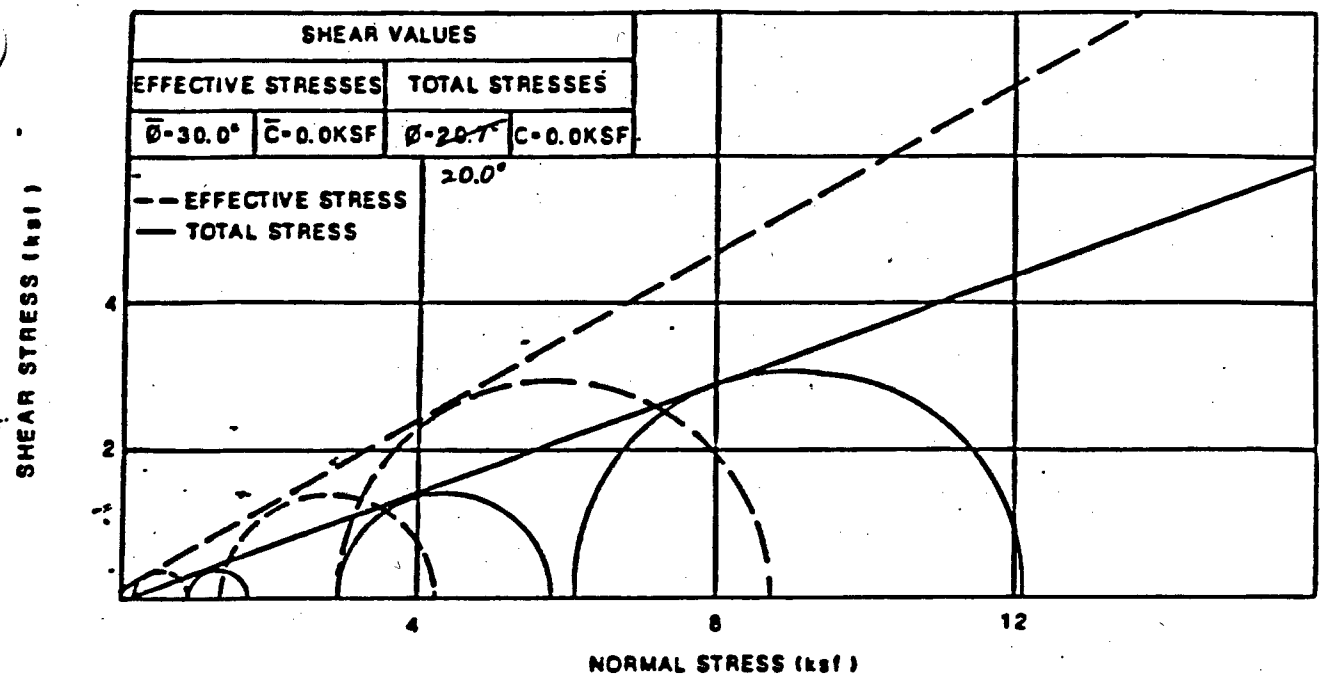
WYL 2/5/88



STAGE NO.	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	SOIL DESCRIPTION
1	98.9	20.9	CLAY, SILTY, SANDY
2	100.5	25.5	
3	104.9	22.9	

LOCATION NO.	DEPTH (FT.)	SAMPLE TYPE
GRJ-03 614	0.5-1.5	REMOLD- (95% standard comp.)

FIGURE E.5.3

SHEAR STRENGTH OF SOIL IN STAGED TRIAXIAL COMPRESSION

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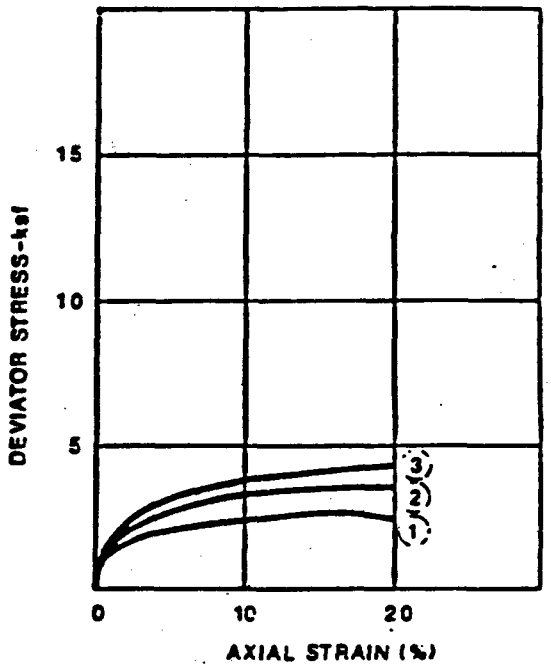
WYL 2/5/88

TEST NUMBER	1	2	3	4
LOCATION SAMPLE NO. DEPTH FT.	← CR.1-05 → ← 614 → ←0.5-1.5→			
WEIGHT-INCH	3.995	4.002	3.966	
DIAMETER-INCH	1.924	1.924	1.926	
WATER CONTENT-%	21.0	21.2	21.3	
DRY DENSITY-pcf	98.5	98.5	98.5	
σ_1 σ_3 -ksf	2.60	3.69	4.19	
σ_3 -ksf	1.01	3.02	6.05	
σ_1 -ksf	3.61	6.71	10.24	



TYPE OF SPECIMEN: Remold

SOIL DESCRIPTION: Clay, sandy, silty

TYPE OF TEST: Unsaturated
Unconsolidated
Undrained

TAN Ø

Ø

COHESION-ksf

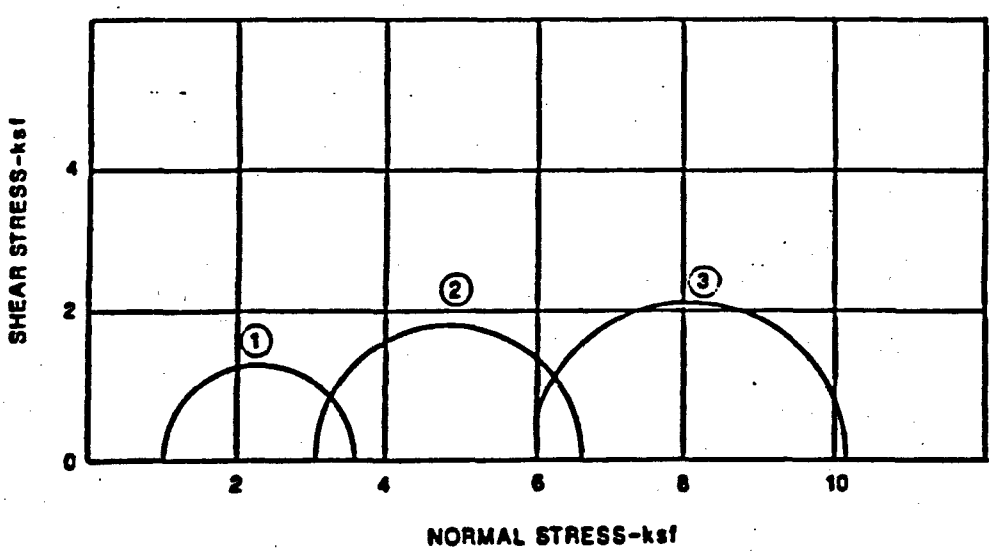


FIGURE E.5.4
TRIAXIAL SHEAR TEST RESULTS

REVISION 3

LAYER 2 GRAVELLY LAYER — In-situ

The gravelly layer was investigated by test pits excavated in August through September 1989. The materials from layer 2 will be used as a source of riprap and -1" material. Five large scale sieve tests were performed to investigate suitability of the materials. The -1" material was also tested since it is expected to be used as a reserved material for radon barrier. Twenty additional gradation tests were performed by MKF in April 1990. The soil test results are summarized on sheet 123. The rocks in this layer range up to 3' to 4', and are primarily basaltic and well-graded. It is expected that the rocks will be used as a riprap layer, and will be very free draining; and therefore undrained strengths and consolidation characteristics are not needed.

Presented on next sheet are typical ϕ angles for various rocks. The basalt rocks range from 37° to 39°. Due to the large fine portion presente in the samples, a conservative value of ϕ of 35° is chosen.

DESIGN PARAMETERS

ASSUMED: $\gamma_{dry} = 130$ pcf

$\gamma_{in-situ} = 130$ pcf

From Sh. 122, assigned $\phi = 35°$, $c = 0$

From Sh. 123 $SG = 2.6$

$$e = \frac{2.6 \times 62.4}{130} - 1 = 0.248$$

$$w_{sat} = \frac{Se}{SG} = \frac{100\% \times 0.248}{2.6} = 9.5\%$$

$$\gamma_{sat.} = \frac{G\gamma_w + e\gamma_w}{\quad} = \frac{2.6 \times 62.4 + 0.248 \times 62.4}{\quad} = 147$$

UMTRA-GJ 5025-16 Sh. 12
MATERIAL PROPERTIES
FROM: REF. 15, page 264
4B622

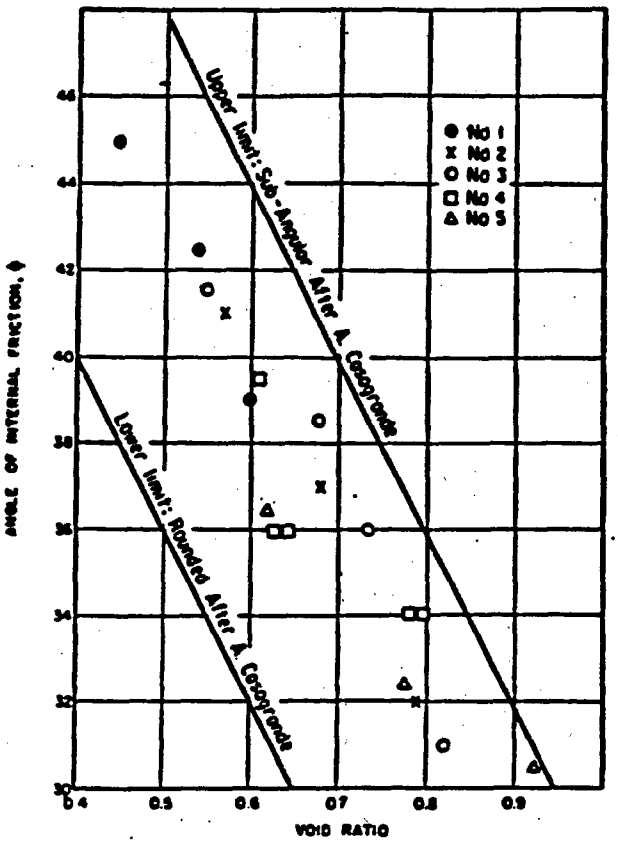


Fig. 7.27 Angle of internal friction vs. void ratio. (After Zolkov and Wiseman, 1965.)

intermediate sized material (12-inch-diameter specimen) had angles of internal friction 1 to 1-½ degrees lower than the smallest material.

Pertinent data from the large rockfill tests including tests reported by Marsal (1965, 1967a and 1967b) are compiled in Table 7.4. The table lists values of angle of internal friction, axial strain at failure and volumetric strain at failure for those tests conducted at a confining pressure of 350 psi for which information was available. It may be observed that for the dredger tailings, which contain rounded particles, the values of axial and volumetric strains at failure are 6.5 and 1.5 percent, respectively, while for all of the angular materials these values are greater than 13 and 5.5 percent, respectively. The values of the volumetric strain at failure for all of the angular materials for which data were available ranged between 5.5–13 percent. Among these materials granitic gneiss and shale showed the greatest volumetric compression at failure. The volumetric compression at failure for the rest of the angular materials, however, ranged between 5.5–6.5 percent.

Seed and Lee (1966) demonstrated that the critical confining pressure (the value of confining pressure for which there is no volume change at failure) is a significant characteristic of a soil and that the undrained strength of noncavitating samples (where the negative pressure does not exceed one atmosphere) can be determined with a satisfactory degree of accuracy from a knowledge of the critical confining pressure of the soil and the angle of friction corresponding to constant volume conditions. Values of the critical confining pressure for the rockfill materials are estimated and listed in Table 7.4. As can be seen the critical confining pressure for a rockfill material composed of rounded particle, i.e., dredger tailings, is substantially greater than that for the rockfill materials composed of

TABLE 7.4. STRENGTH AND DEFORMATION CHARACTERISTICS OF LARGE ROCKFILL SPECIMENS. Confining Pressure = 350 psi.

Dam or Place	*Material*	*Gradation*	*Particle Shape*	$\phi°$	ϵ_{if}, *percent*	ϵ_{vf}, *percent*	$\sigma_{3\,crit}$, *psi*	*Reference*
Oroville	Dredger tailings	Well-graded 6" fines	Rounded	43	6.5	1.5	120	Marachi et al., 1969
Pinzandaran	Sand and gravel (dry)	Well-graded 8"-fines	Rounded	39	8	4.7	60	Marsal, 1967b
San Francisco	Basalt	Well-graded 8"-¼"	Angular	39	15	6	60	Marsal, 1967a
San Francisco	Basalt	Well-graded 3"-¼"	Angular	38				Marsal, 1967b
San Francisco	Basalt	Poorly-graded 6"-fines	Angular	37	20	6.5	40	Marachi et al., 1969
Malpaso	Conglomerate (dry)	Well-graded 8"-fines	Angular	37	13	4.5	20	Marsal, 1967b
El Infiernillo	Silicified conglomerate (dry)	Poorly-graded 8"-fines	Angular	36.5	14	5.5	30	Marsal, 1965a
Pyramid	Argillite	Poorly-graded 6"-fines	Angular	36.5	20	5.5	25	Marachi et al., 1969
El Infiernillo	Diorite (dry)	Poorly-graded 8"-fines	Angular	35	15	10	25	Marsal, 1967b
El Granero	Shale*	Well-graded 8"-¼"	Angular	35	> 14	> 10	10	Marsal, 1967b
El Granero	Shale*	Poorly-graded 8"-¾"	Angular	33	> 14	> 10	5	Marsal, 1967b
Mica	Granitic gneiss*	Well-graded 8"-fines	Angular	32	> 14	6	20	Marsal, 1967a
Mica	Granitic gneiss*	Poorly-graded 8"-1½"	Angular	25	14	10	5	Marsal, 1967a

*Tests were not carried to failure.

UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - (2A) - - - -
-1" MATERIAL (2B)

H.L. 6/26/90 123
SKB 6-22-90

SUMMARY OF SOIL TEST RESULTS

Job No 5025-16 Project Name UMTRA - GRAND JUNCTION Feature CHENEY DISPOSAL SITE Date 6/11/90

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Sand	Fines	Atterberg Limits LL	PI	Specific Gravity G	Natural W %	γ_d	Compaction Test	Optimum W	Optimum γ_d	Shear Strength Test	Initial W	Initial γ_d	C	ϕ	Permeability γ_d	k	Consolidation C	C_c	R_c	Notes
TP# 70, 71, 72, 73, 114, 100, 101, 102	#1.			GW	49	28	23			2.62 2.64 2.60																
TP# 122, 123, 124, 125	#2			"	47	31	22			2.65 2.65 2.64																
TP# 122, 123, 124, 125, 127, 128, 133	#3			"	44	29	27			2.67 2.71 2.66																
TP# 126, 127, 128	#4			"	57	29	14			2.63 2.59 2.61																
TP# 131, 132, 133, 134, 135	#5			"	42	39	19			2.63 2.64 2.72																
AVERAGE =					48	31	21			2.64																
-1" MATERIAL																										
TEST PILE #1					19	46	35	23.5	0.9				A/C	15.9/12.5	114.3/120.8											
TEST PILE #2					19	44	37	22.1	2.2				A/C	13.5/11.3	117.8/124.6											
TEST PILE #3					15	44	41	25.2	7.4				A/C	17.4/14.2	113.2/119.2											
TEST PILE #4					23	52	25	23.4	6.1				A/C	14.5/11.4	118.2/123.4											
TEST PILE #5					19	52	29	22.9	4.7				A/C	14.6/12.3	117.3/122.9											
									*	A: ASTM D-698-A C: ASTM D-698-C																
AVERAGE =					19	48	33	23.4	4.3				* A/C	15.2/12.3	116.2/122.2											
ADDITIONAL 20 GRADATION RESULTS (DATA FROM: MKE DOC. 5025-GRJ-R-02-04233-00)																										
	EP-G 01			GW	LARGER THAN 1": SMALLER THAN 1":					58% 42%																
	EP-G-02			"			"			67% 33%																
	EP-G-03			"			"			68% 32%																
	EP-G-04			"			"			65% 35%																
	EP-G-05			"			"			64% 36%																
	EP-G-06			"			"			68% 32%																
	EP-G-07			"			"			66% 34%																

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER — 2A
FILL MATERIAL — 2B

124
H.L 6/26/90
6-22-90

SUMMARY OF SOIL TEST RESULTS

Job No 5025-16 Project Name UMTRA- GRAND JUNCTION Feature CHENEY DISPOSAL SITE Date 6/11/90

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural W %	Natural γ_d	Compaction Test	Compaction Optimum W	Compaction Optimum γ_d	Shear Strength Test	Shear Strength Initial W	Shear Strength Initial γ_d	Shear Strength C	Shear Strength ϕ	Permeability γ_d	Permeability k	Consolidation C	Consolidation C_c	Consolidation R	Notes
CONTINUATION OF 20 GRADATION RESULTS (DATA FROM: MKE DOC 5025-GRJ-R-02-04233-00)																										
	EP-G-08			GW*	LARGER THAN 1": 72% SMALLER THAN 1": 28%																					
	EP-G-09			"			"			76% 24%																
	EP-G-10			"			"			70% 30%																
	EP-G-11			"			"			55% 45%																
7	EP-G-12			"			"			67% 33%																
0	EP-G-13			"			"			62% 38%																
7	EP-G-14			"			"			68% 32%																
5	EP-G-15			"			"			72% 28%																
8	EP-G-16			"			"			72% 28%																
8	EP-G-17			"			"			64% 36%																
	EP-G-18			"			"			61% 39%																
	EP-G-19			"			"			68% 32%																
	EP-G-20			"			"			69% 31%																

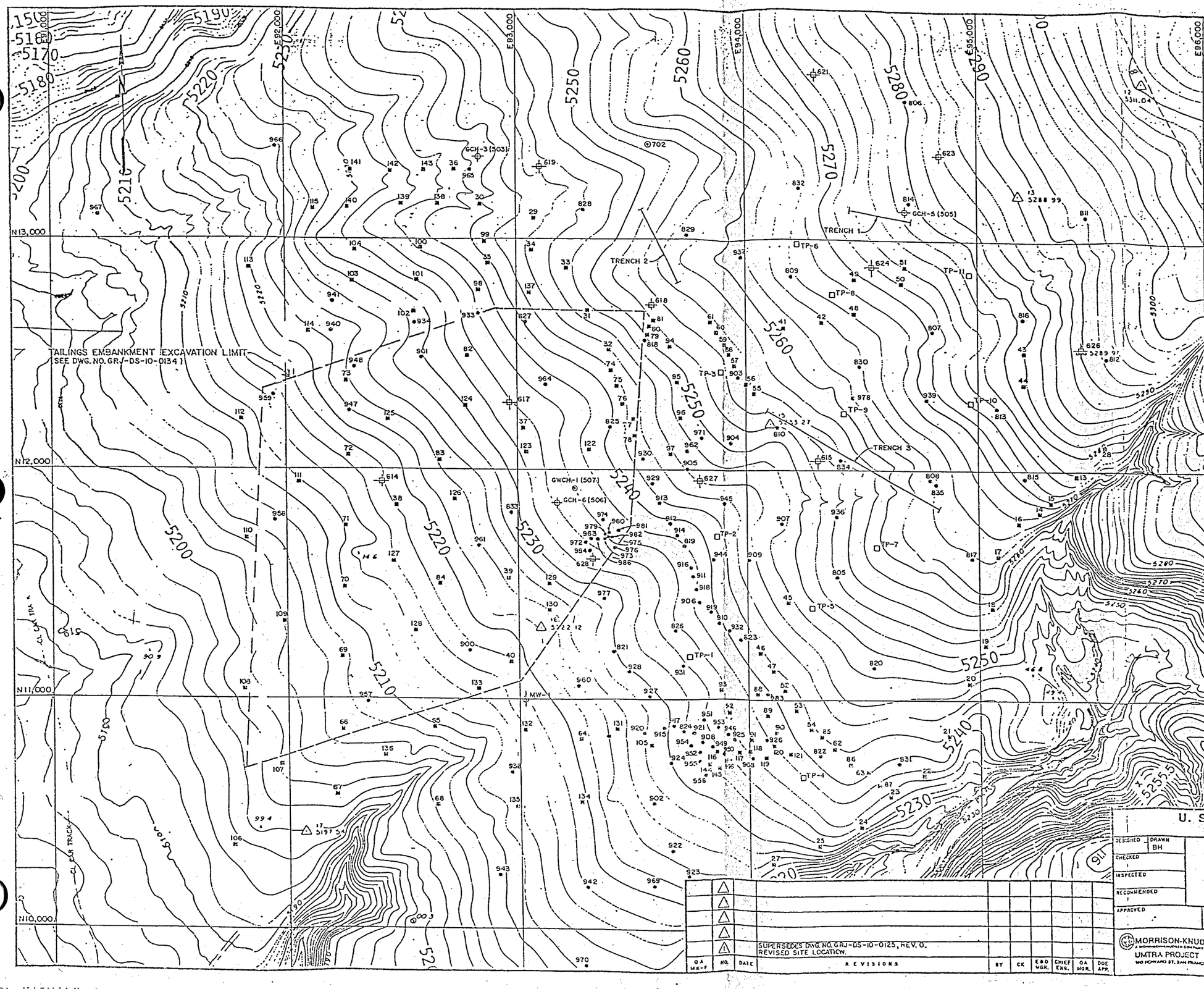
* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Sieve - Size Test

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

125



BOREHOLES & TEST PITS	COORDINATES	
GWCH-2 (508)	N14,170	E96,180
GWCH-3 (509)	N14,335	E94,140
GCH-1 (501)	N15,240	E95,360
GCH-2 (502)	N14,280	E94,160
GCH-4 (504)	N14,140	E96,240
613	N15,270	E95,100
701	N14,460	E95,020
26	N10,170	E93,985
935	NOT DRILLED	
985	NOT DRILLED	

NOTE:

1. GROUND WATER DATA AND SUBSURFACE EXPLORATION LOGS ARE AVAILABLE FROM THE CONTRACTOR.
2. BOREHOLES AND TEST PITS NOT SHOWN ON DRAWING ARE LISTED ON TABLE ABOVE.

REFERENCE DRAWINGS:

GRJ - DS - 10 - 0125 CONSTRUCTION FACILITIES AND SITE DRAINAGE

GRJ - DS - 10 - 0134 TAILINGS EMBANKMENT AREA EXCAVATION PLAN

LEGEND:

5240 EXISTING SITE FEATURES AND CONTOURS

N12,000 CONSTRUCTION GRID COORDINATE

GCH-6 BORINGS DRILLED BY NUS IN 1982

GWCH-3 MONITOR WELLS INSTALLED IN 1982

701 MONITOR WELLS INSTALLED BY TAC IN 1985

616 TEST PITS DUG BY TAC IN 1984

617 TEST PITS DUG BY LINCOLN DEVORE/MKE IN 1985

TP-5 TEST PITS DUG BY WESTERN ENG./MKE IN 1987

MW-1 EXISTING WATER SUPPLY WELL BY MKE IN 1987

41 TEST PIT BY TAC IN 1989

930 WELL-BOREHOLE BY TAC IN 1989

SAMPLES SOURCE LOCATION
<GRAVELLY LAYER>

SAMPLE #1: TP #70, #71, #72, #73, #110, #100, #101, #102

SAMPLE #2: TP #122, 123, 124, 125

SAMPLE #3: TP #122, 123, 124, 125, 127, 128, 133

SAMPLE #4: TP #126, 127, 128

SAMPLE #5: TP #131, 132, 133, 134, 135

200 0 200 400
SCALE FEET

U. S. DEPARTMENT OF ENERGY
ALBUQUERQUE, NEW MEXICO

CHENEY RESERVOIR DISPOSAL SITE
GRAND JUNCTION, COLORADO
PHASE II CONSTRUCTION

BORINGS AND TEST PITS LOCATION PLAN

DESIGNED | DRAWN BH | CHECKED | INSPECTED | RECOMMENDED | APPROVED | DATE | DOE PROJECT ENGINEER | DATE

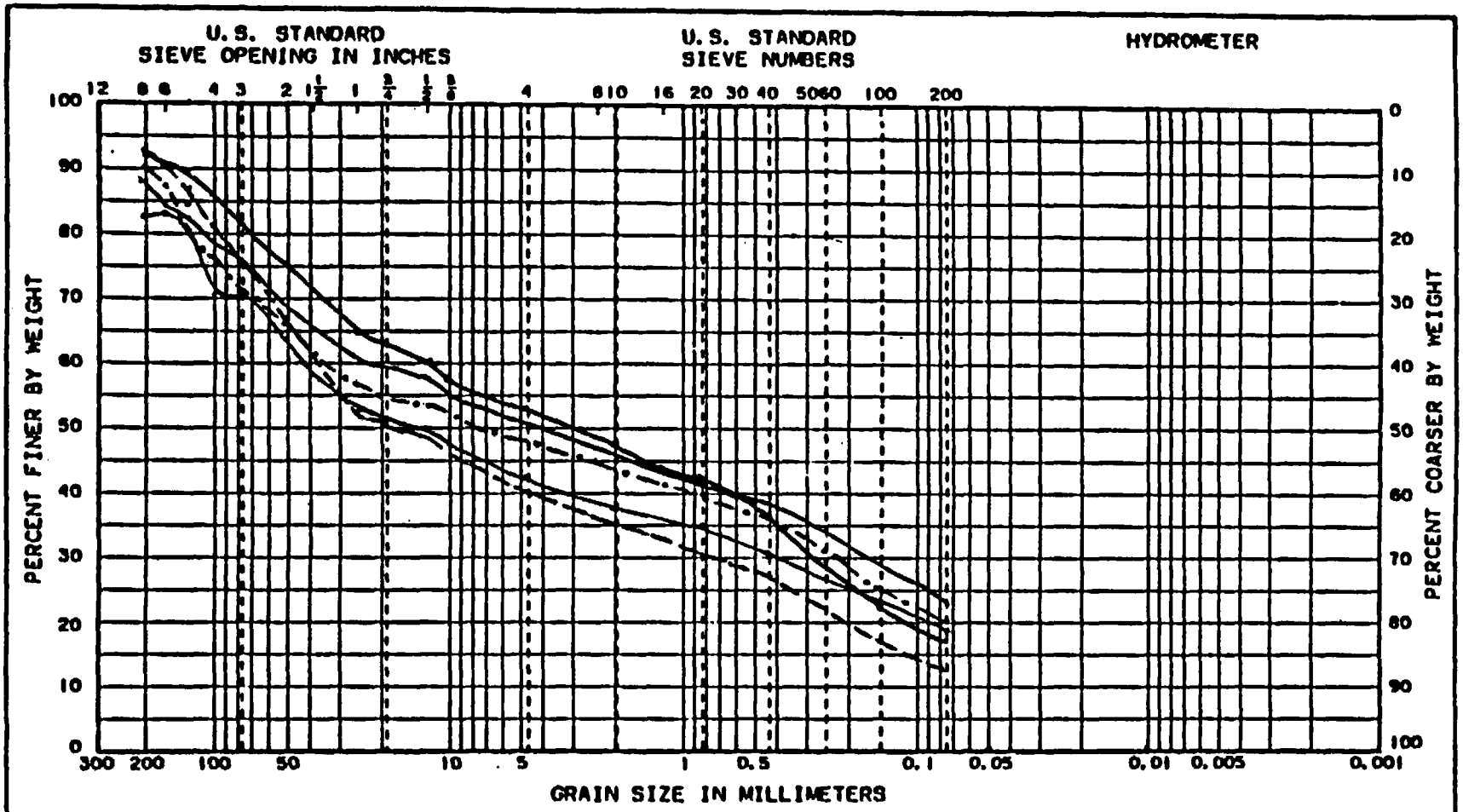
MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY
UMTRA PROJECT
180 MONTGOMERY ST. SAN FRANCISCO, CA 94105

PROJECT NO. DE-AC04-83AL18796

DRAWING NO. GRJ-DS-10-0223 REV. 1

QA MK-F	NO.	DATE	REVISIONS	BY	CK	E&D MGR.	CHIEF ENG.	QA MGR.	DOE APP.
	1		SUPERSEDES DWG. NO. GRJ-DS-10-0125, REV. 0. REVISED SITE LOCATION.						

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY



SAMPLE NO.	ELEV. OR DEPTH	CLASSIFICATION	NAT WX	LL	PL	PI
—— ①		GW				
—·— ②		GW				
—— ③		GW				
— — ④		GW				
—— ⑤		GW				

PROJECT UMTRA - GRJ
JOB NO. 5025 - 16
AREA CHENEY SITE
HOLE NO.
DATE 12/19/89

CAB110

DATA FROM: MKE DOC. 5025-GRJ-R-09-03589-00

GRAIN SIZE ANALYSIS

UMTRA - GRJ
5025-16 MATERIAL PROPERTIES
GRAVELLY LAYER (2A)

126

6-22-9

SAMPLE #1, Cheney Reservoir, LD #71177J
M.K. FERGUSON, P.O. #3050-511-10002

SAMPLE LOCATION

T.P.#	DEPTH
70	
71	
72	
73	
114	
100	
101	
102	

TOTAL DRY SAMPLE WEIGHT - 18,152 pounds
AVERAGE % MOISTURE..-1" - 3.47%

SIEVE SIZE	% EACH SIEVE	% PASSING	DELETERIOUS MAT. % of TOTAL
18"	7.38	92.62	
8"	7.33	82.29	
6"	1.81	83.48	
5"	2.6	80.88	
4"	4.12	71.76	0.16% Sedim. Rocks
3"	4.56	72.2	0.32% Sedim. Rocks
2	8	64.2	0.71% Sedim. Rocks
1"	11.75	52.45	1.03% Sedim. Rocks
3/4"		52.19	
1/2"		49.67	
3/8"		47.52	
#4		43.53	
#10		38.08	
#20		34.51	
#40		31.47	
#100		24.23	
#200		19.51	

2.58% Sulfate and clay lumps in the +1" material, included in the -1" sieve.

SIEVE SIZE	SPECIFIC GRAVITY Bulk	SSD	Appar.	ABSORP. %	SULFATE SOUND. % Loss	L.A. ABRASION % LOSS after 100 Rev	500 Rev
1" -3"	2.559	2.617	2.717	2.28%	3.48%	5.9%	26.1%
3"-8"	2.597	2.642	2.719	1.72%	2.84%	5.1%	21.8%
8"-18"	2.568	2.6	2.654	1.26%	1.91%	5.0%	23.9%

Notes:

ASTM C-88 SODIUM SULFATE SOUNDNESS Loss after 5 Cycles
ASTM C-131 LOS ANGELES ABRASION
ASTM C-127 AGGREGATE SPECIFIC GRAVITY & ABSORPTION

SAMPLE #2 Cheney Reservoir Site, L.D. #71177J
M.K. FERGUSON, P.O. #3050-511-10002

SAMPLE LOCATION	
T.P.#	DEPTH
122	12'-18'
123	10.5'-14'
124	15'-18.5'
125	16'-20'

TOTAL DRY SAMPLE WEIGHT - 16298 pounds
AVERAGE % MOISTURE..-1" - 5.94%

SIEVE SIZE	% EACH SIEVE	% PASSING	DELETERIOUS MAT. % of TOTAL
18"	8.47	91.53	
	3.00		
8"	1.2	90.33	0.01% Sedi. Rocks
6"	3.14	87.19	0.05% Sedi. Rocks
5	6.38	80.81	0.19% Sedi. Rocks
4	4.59	76.22	0.21% Sedi. Rocks
3	4.95	71.27	0.35% Sedi. Rocks
2	4.64	66.63	0.51% Sedi. Rocks
1	8.24	58.38	0.82% Sedi. Rocks
3/4"		55.46	
1/2"		54.18	
3/8		52.43	
#4		48.63	
#10		44.49	
#20		40.46	
#40		36.03	
#100		25.45	
#200		19.91	

1.95% Sulfate and clay lumps in the +1 material, included in the -1" Sieve.

SIEVE SIZE	SPECIFIC GRAVITY Bulk	SSD	Appar.	ABSORP. %	SULFATE SOUND. % Loss	L.A. ABRASION % LOSS after 100 Rev	500 Rev
1"-3"	2.586	2.649	2.759	2.42%	3.08%	5.5%	24.5%
3"-8"	2.603	2.654	2.743	1.96%	2.50%	5.4%	23.0%
8"-18"	2.589	2.637	2.719	1.86%	2.19%	5.3%	22.1%

Notes:

ASTM C-88 SODIUM SULFATE SOUNDNESS Loss after 5 Cycles
ASTM C-131 LOS ANGELES ABRASION
ASTM C-127 SPECIFIC GRAVITY & ABSORPTION

UMTRA-GRJ
5025-16
MATERIAL PROP.
GRAVELLY LAYER - 2A

SAMPLE #3, Cheney Reservoir, LD #71177J
M.K. FERGUSON, P.O. #3050-511-10002

TOTAL DRY SAMPLE WEIGHT - 19656 pounds
AVERAGE % MOISTURE..-1" - 4.90%

SAMPLE LOCATION		SIEVE SIZE	% EACH SIEVE	% PASSING	DELETERIOUS MAT. % of TOTAL
T.P.#	DEPTH				
122	2.5'-7.0'				
122	7'-12'				
123	6'-10.5'	18	4.93	95.07	
124	5'-15'		2.06		
125	8'-12'	8"	6.38	88.69	0.13% Sedi. Rocks
127	4'-9'				
128	2'-7'	6"	4.08	84.61	0.17% Sedi. Rocks
133	1.5-3.5'	5"	2.11	82.5	0.08% Sedi. Rocks
133	3.5-11'	4"	3.16	79.34	0.16% Sedi. Rocks
		3	3.34	76	0.20% Sedi. Rocks
		2	7.16	68.84	0.38% Sedi. Rocks
		1	8.46	60.37	0.47% Sedi. Rocks
		3/4		59.46	
		1/2		57.59	
		3/8"		55.06	
		#4		51.44	
		#10		46.12	
		#20		41.66	
		#40		35.15	
		#100		29.4	
		#200		23	

0.41% Sulfate and clay lumps in the +1" material, included in the -1" Sieve

SIEVE SIZE	SPECIFIC GRAVITY Bulk	SSD	Appar.	ABSORP. %	SULFATE SOUND. % Loss	L.A. ABRASION % LOSS after 100 Rev	500 Rev
1"-3"	2.612	2.665	2.757	2.01%	3.47	6.9%	27.1%
3"-8"	2.663	2.711	2.798	1.82%	2.34	5.9%	25.0%
8"-18	2.615	2.66	2.737	1.71%	2.52	5.8%	25.0%

Notes:

ASTM C-88 SODIUM SULFATE SOUNDNESS Loss after 5 Cycles
ASTM C-131 LOS ANGELES ABRASION
ASTM C-127 AGGREGATE SPECIFIC GRAVITY & ABSORPTION

UMTRA - GRJ
5025-16
MATERIAL PROP.
GRAVELLY LAYER - (2A)

SAMPLE #4, Cheney Reservoir, LD #71177J
M.K. FERGUSON, P.O. #3050-511-10002

TOTAL DRY SAMPLE WEIGHT - 17462 pounds
AVERAGE % MOISTURE..-1 - 4.0%

SAMPLE LOCATION

T.P.#	DEPTH
126	4'-14'
127	9'-15'
128	9'-16'

SIEVE SIZE	% EACH SIEVE	% PASSING	DELETERIOUS MAT. % of TOTAL
18	2	98	
	1.00		
8	4.65	93.35	
6	3.14	90.21	
5"	3.57	86.64	0.11% Sedi. Rocks
4"	5.36	81.28	0.17% Sedi. Rocks
3"	4.81	76.47	0.21% Sedi. Rocks
2"	10.37	66.1	0.65% Sedi. Rocks
1"	13.68	52.42	0.97% Sedi. Rocks
3/4"		50.85	
1/2"		48.07	
3/8"		45.76	
#4		40.47	
#10		35.17	
#20		31.29	
#40		27.21	
#100		16.67	
#200		12.27	

1.75% Sulfate and clay lumps in the +1" material, included in the -1" Sieve

SIEVE SIZE	SPECIFIC GRAVITY Bulk	SSD	Appar.	ABSORP. %	SULFATE SOUND. % Loss	L.A. ABRASION % LOSS after 100 Rev	500 Rev
1"-3"	2.57	2.631	2.738	2.39%	2.57	6.7%	26.6%
3"-8"	2.547	2.594	2.674	1.87%	2.27	5.9%	25.0%
8"-18"	2.561	2.611	2.697	1.97	2.22	5.5%	23.0%

Notes:

ASTM C-88 SODIUM SULFATE SOUNDNESS Loss after 5 Cycles
ASTM C-131 LOS ANGELES ABRASION
ASTM C-127 AGGREGATE SPECIFIC GRAVITY & ABSORPTION

UMTRA-GRJ
5025-16
MATERIAL PROP.
GRAVELLY LAYER - 2A

SAMPLE #5, Cheney Reservoir, LD #71177J
M.K. FERGUSON, P.O. #3050-511-10002

TOTAL DRY SAMPLE WEIGHT - 21216 pounds
AVERAGE % MOISTURE..-1 - 4.35%

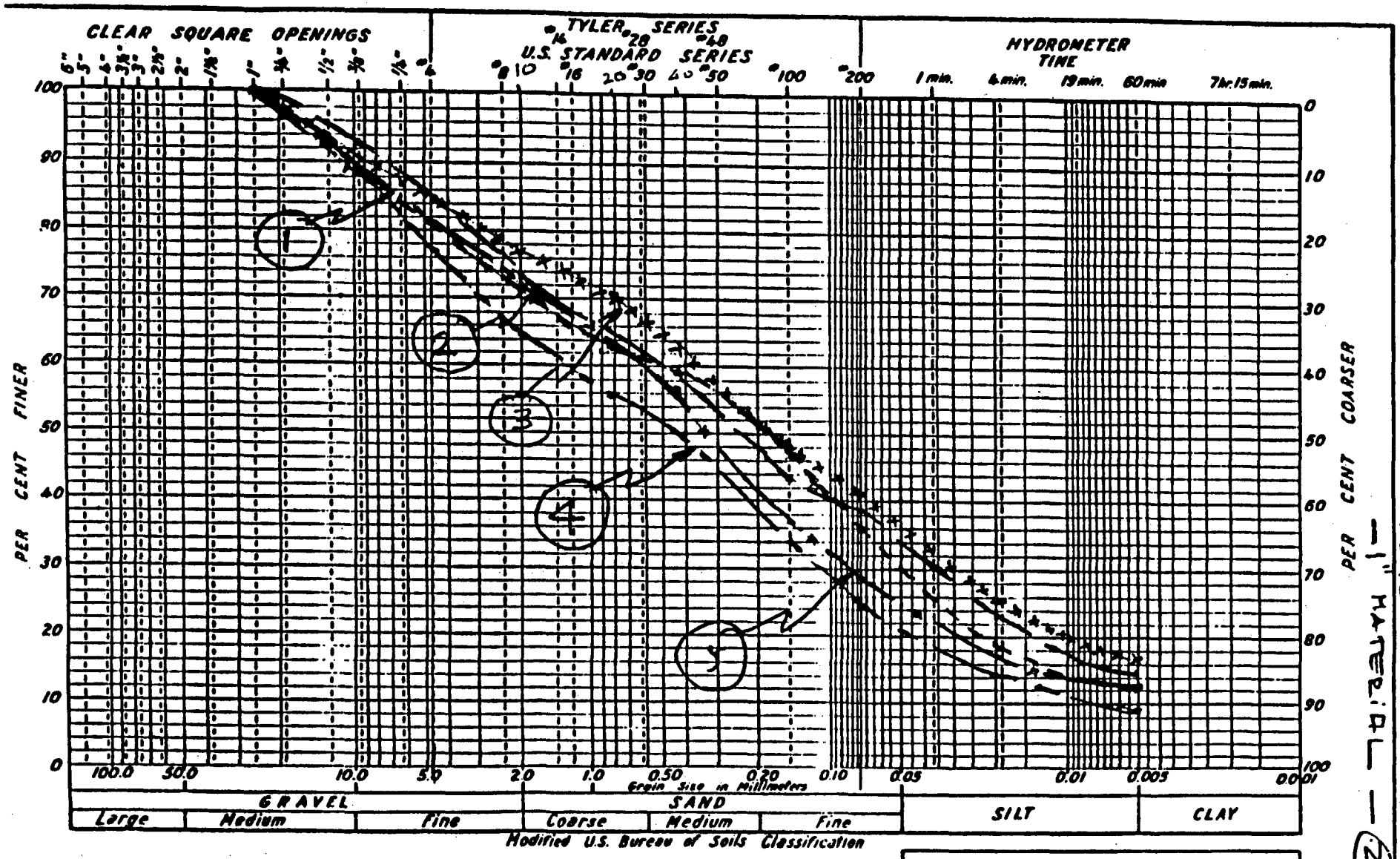
SAMPLE LOCATION T.P.#	DEPTH	SIEVE SIZE	% EACH SIEVE	% PASSING	DELETERIOUS MAT. % of TOTAL
133	3.5'-11'				
133	15'-18'				
132	10'-18'	18"	1.46	98.54	
131	11.5'-17'		1.38		
134	2'-5'	8"	5.58	92.96	
134	5'-8'				
135	21'-27'	6"	1.62	91.34	
135	9'-13'	5"	1.7	89.64	
		4"	3.64	86	0.14% Sedi. Rocks
		3"	4.41	81.59	0.24% Sedi. Rocks
		2"	6.26	75.33	0.45% Sedi. Rocks
		1"	10.53	64.8	0.91% Sedi. Rocks
		3/4		62.86	
		1/2		60.26	
		3/8		57.48	
		#4		53.2	
		#10		47.17	
		#20		41.99	
		#40		35.95	
		#100		22.67	
		#200		16.85	

1.17% Sulfate and clay lumps in the +1" material, included in the -1" Sieve

SIEVE SIZE	SPECIFIC GRAVITY Bulk	SSD	Appar.	ABSORP. %	SULFATE SOUND. % Loss	L.A. ABRASION % LOSS after 100 Rev	500 Rev
1"-3"	2.558	2.628	2.752	2.77%	4.26	6.2%	25.9%
3"-8"	2.595	2.64	2.717	1.73%	2.66	5.3%	23.0%
8"-18	2.679	2.72	2.792	1.51%	2.08	4.5%	20.6%

Notes:
ASTM C-88 SODIUM SULFATE SOUNDNESS Loss after 5 Cycles
ASTM C-131 LOS ANGELES ABRASION
ASTM C-127 AGGREGATE SPECIFIC GRAVITY & ABSORPTION

— 1" Mat'l.



UNIT 24 - GEJ
5025-16
MATERIAL PROPERTIES
-1" MATERIAL — 28

Sh 6-22-90

133

JANUARY 9, 90

MK-FERGUSON COMPANY
A MORRISON KNUDSEN COMPANY

MKE DOC. 5025-GRJ-I-02-03796-00

INTER-OFFICE CORRESPONDENCE

DATE: December 29, 1989

TO: F. J. Feliz

FROM: R. E. Cooney

LOCATION: San Francisco, CA - MKES

LOCATION: Albuquerque, NM - 3050

SUBJECT: Grand Junction, Phase II
Soil Testing (P.O. #10002)

Enclosed are soil testing results by Lincoln-DeVore. These are for the 1"-minus material left after the large-scale sieve analysis, per your IOC to T. G. Myer of October 27, 1989 (#5025-GRJ-I-01-00621-00).

Please advise Tom Jennings if these results are acceptable for the purpose of authorizing payment to Lincoln-DeVore.

REC:TPJ:kja

Enclosure

cc w/o enc.: R. E. Cooney
T. P. Jennings
J. G. Pepin
File

Attachment: MKE DOC. 5025-GRJ-R-09-03797-00

RECEIVED-MKE
JA[illegible]
UMTRA-S.F.

3807K

STA001

MKE DOC. 5025-GRJ-R-09-03797-00

PROJECT Cheney Reservoir - Grand Junction

CLIENT V.K. Ferguson

SAMPLE LOCATION Test pile #1

SOIL TYPE Sample scalped on #4 sieve

RECEIVED MKE
JAN 02 1990
UMTRA S.F.

TEST NO. 71177-1 #1

DATE 12-1-89

TEST BY DPW

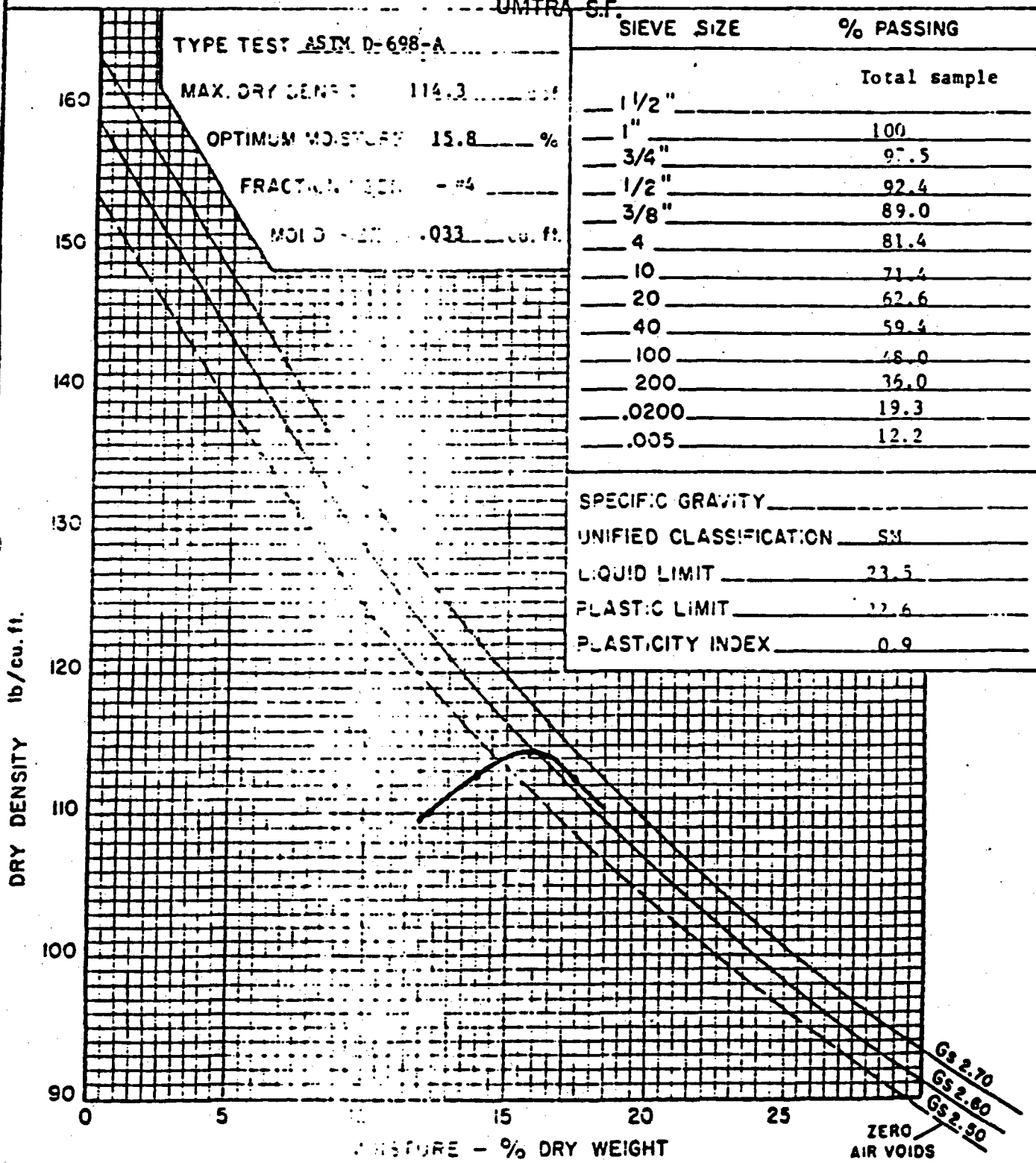
TYPE TEST ASTM D-698-A

MAX. DRY DENSITY 114.3 pcf

OPTIMUM MOISTURE 15.8 %

FRACTION USED - #4

MOLD VOL. .033 cu. ft.



SIEVE SIZE	% PASSING
	Total sample
1 1/2"	
1"	100
3/4"	97.5
1/2"	92.4
3/8"	89.0
4	81.4
10	71.4
20	62.6
40	59.4
100	48.0
200	36.0
.0200	19.3
.005	12.2

SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SM

LIQUID LIMIT 23.5

PLASTIC LIMIT 22.6

PLASTICITY INDEX 0.9

MOISTURE – DENSITY RELATION

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir- Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #2

SOIL TYPE Sample scalped on #4 sieve

TEST No. 71177-J #2

DATE 12-1-89

TEST BY [illegible]

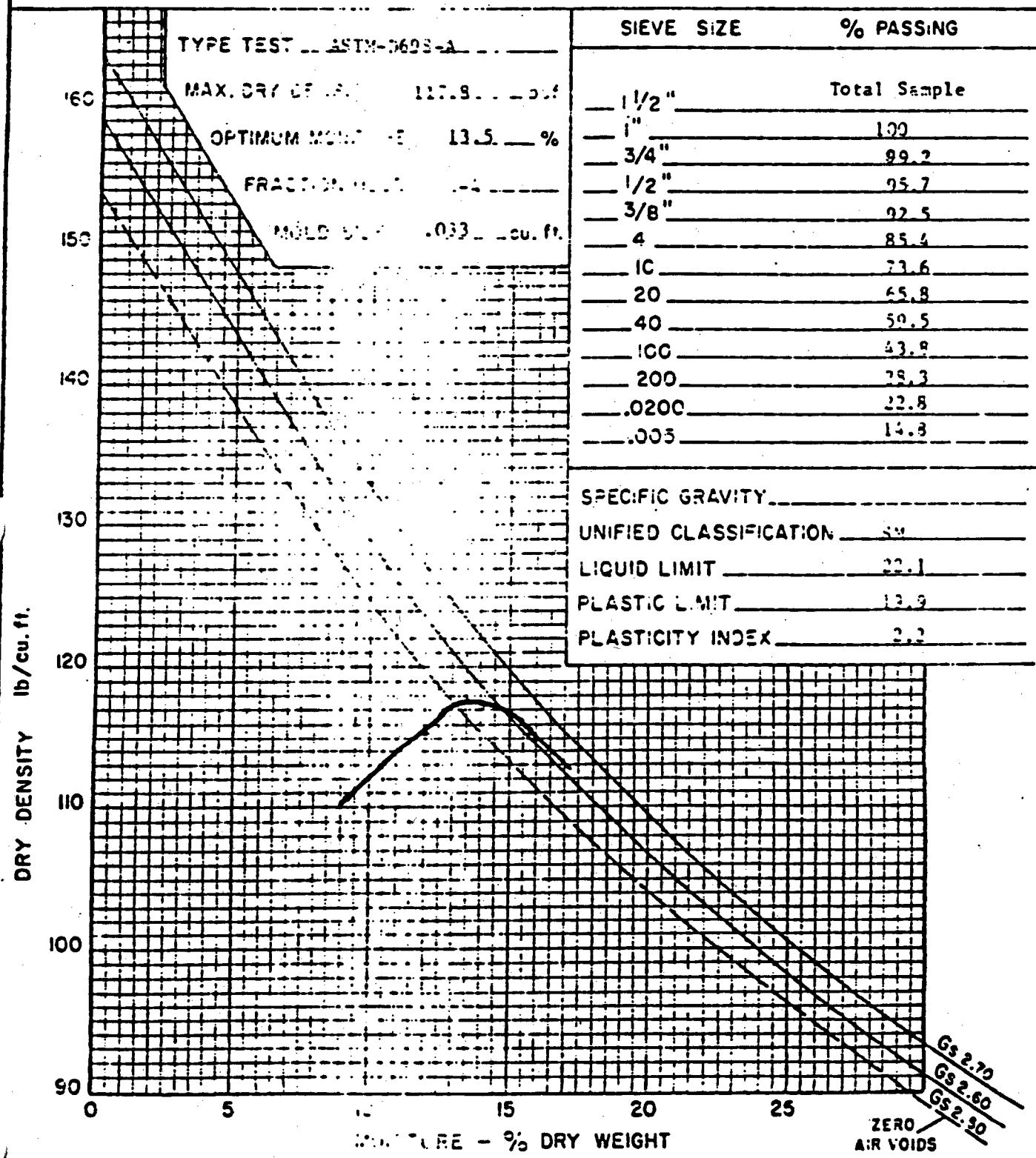
TYPE TEST ASTM-D698-A

MAX. DRY DENSITY 117.8 pcf

OPTIMUM MOISTURE 13.5 %

FRACTION USED [illegible]

MOLD SIZE .033 cu. ft.



SIEVE SIZE	% PASSING
	Total Sample
1 1/2"	
1"	100
3/4"	99.2
1/2"	95.7
3/8"	92.5
4	85.4
10	73.6
20	65.8
40	50.5
100	43.8
200	38.3
.0200	22.8
.005	14.8

SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SM

LIQUID LIMIT 22.1

PLASTIC LIMIT 19.9

PLASTICITY INDEX 2.2

MOISTURE – DENSITY RELATION

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir - Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #3

SOIL TYPE Sample scalped on #4 sieve

TEST NO. 71177-J #3

DATE 12-1-89

TEST BY DPW

TYPE TEST ASTM D-698-A

MAX. DRY DENSITY 113.2 pcf

OPTIMUM MOISTURE 17.4 %

FRACTION USED -4

MOLD SIZE .033 cu. ft.

SIEVE SIZE	% PASSING
1 1/2"	Total Sample
1"	100
3/4"	97.4
1/2"	93.7
3/8"	90.4
4	84.2
10	77.2
20	70.6
40	63.7
100	47.6
200	41.2
.0200	25.8
.005	16.4

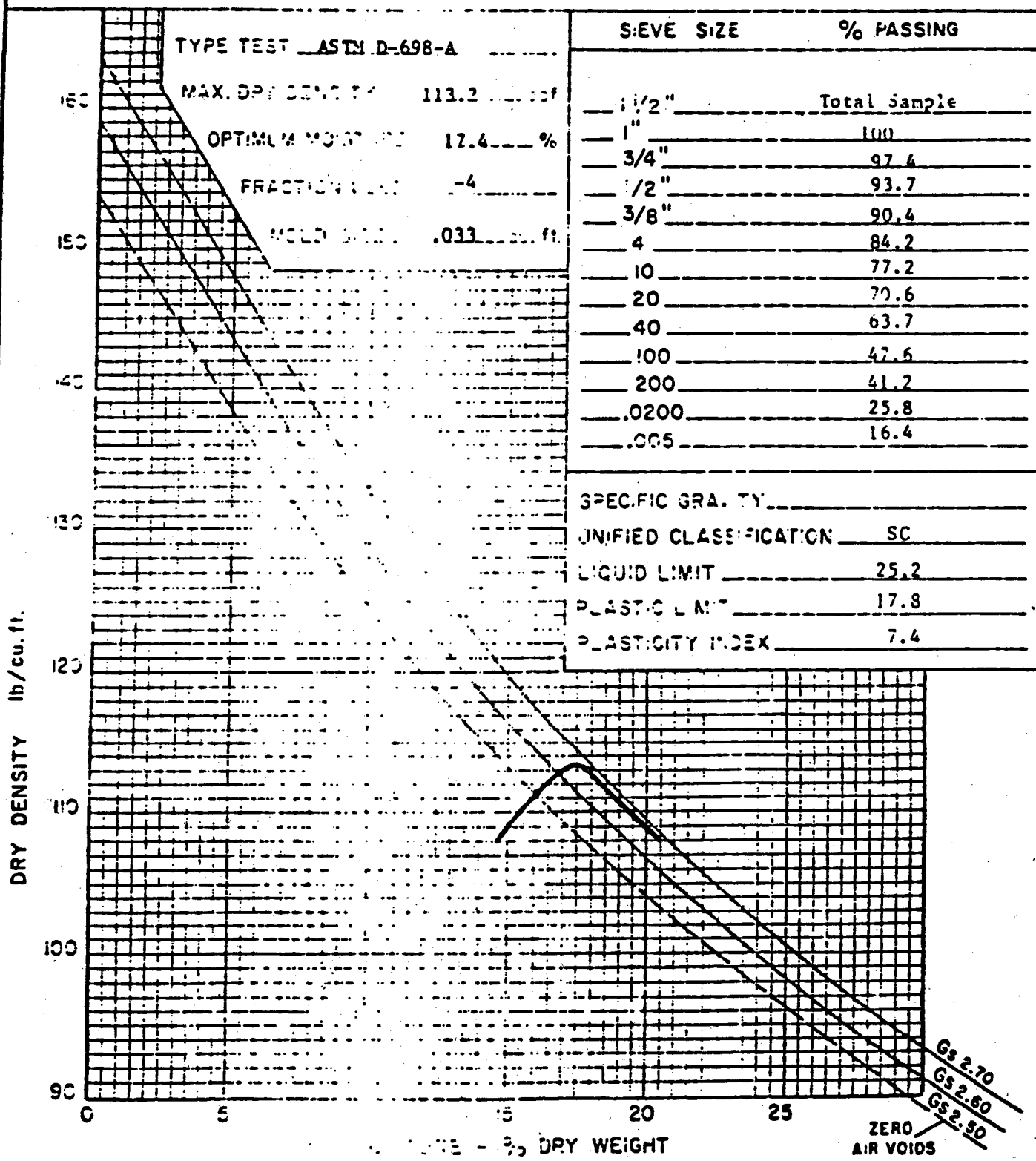
SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SC

LIQUID LIMIT 25.2

PLASTIC LIMIT 17.8

PLASTICITY INDEX 7.4



MOISTURE — DENSITY RELATIONS

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir - Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #4

SOIL TYPE Sample scalped on #4 sieve

TEST NO. 71177-1 #4

DATE 12-1-89

TEST BY DPW

TYPE TEST ASTM D-698-A

MAX. DRY DENSITY 119.2 pcf

OPTIMUM MOISTURE 14.5 %

FRACTION -4

MOLD SIZE .033 cu. ft.

SIEVE SIZE	% PASSING
1 1/2"	Total Sample
1"	100
3/4"	99.1
1/2"	94.3
3/8"	89.5
4	77.7
10	64.9
20	55.6
40	49.7
100	34.1
200	24.9
.0200	14.2
.005	9.5

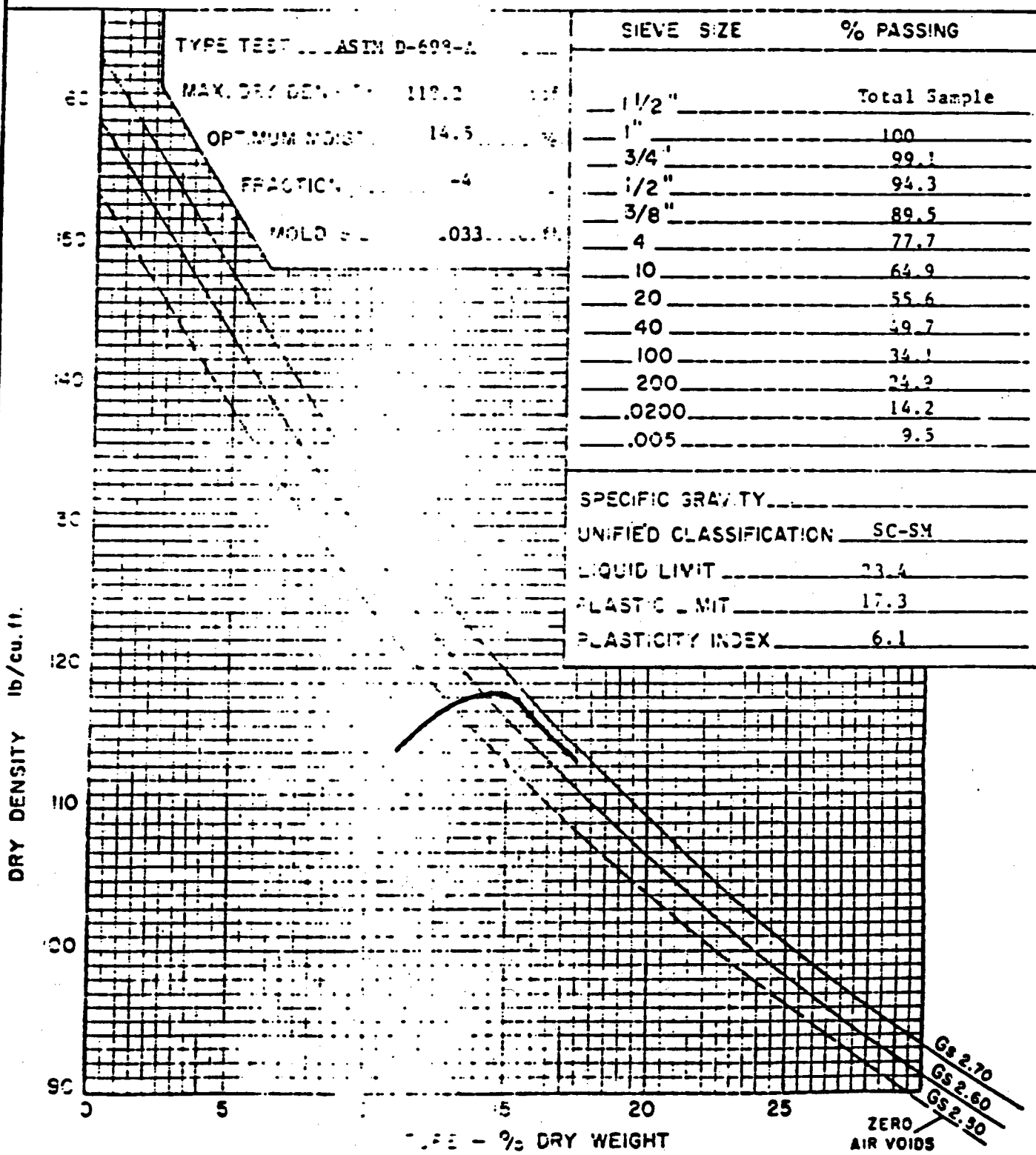
SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SC-SM

LIQUID LIMIT 23.4

PLASTIC LIMIT 17.3

PLASTICITY INDEX 6.1



MOISTURE - DENSITY RELATION

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir - Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #5

SOIL TYPE Sample scalped on #4 sieve

TEST NO. 71177-J #5

DATE 12-1-89

TEST BY CMB

TYPE TEST ASTM-D-698-A

MAX. DRY DENSITY 117.3 pcf

OPTIMUM WC 14.6 %

FRACTION -4

WEIGHT 0.33 cu. ft.

SIEVE SIZE	% PASSING
	Total Sample
1 1/2"	
1"	100
3/4"	97.2
1/2"	92.8
3/8"	88.9
4	81.0
10	72.0
20	64.2
40	56.5
100	36.2
200	29.3
.0200	17.6
.005	12.4

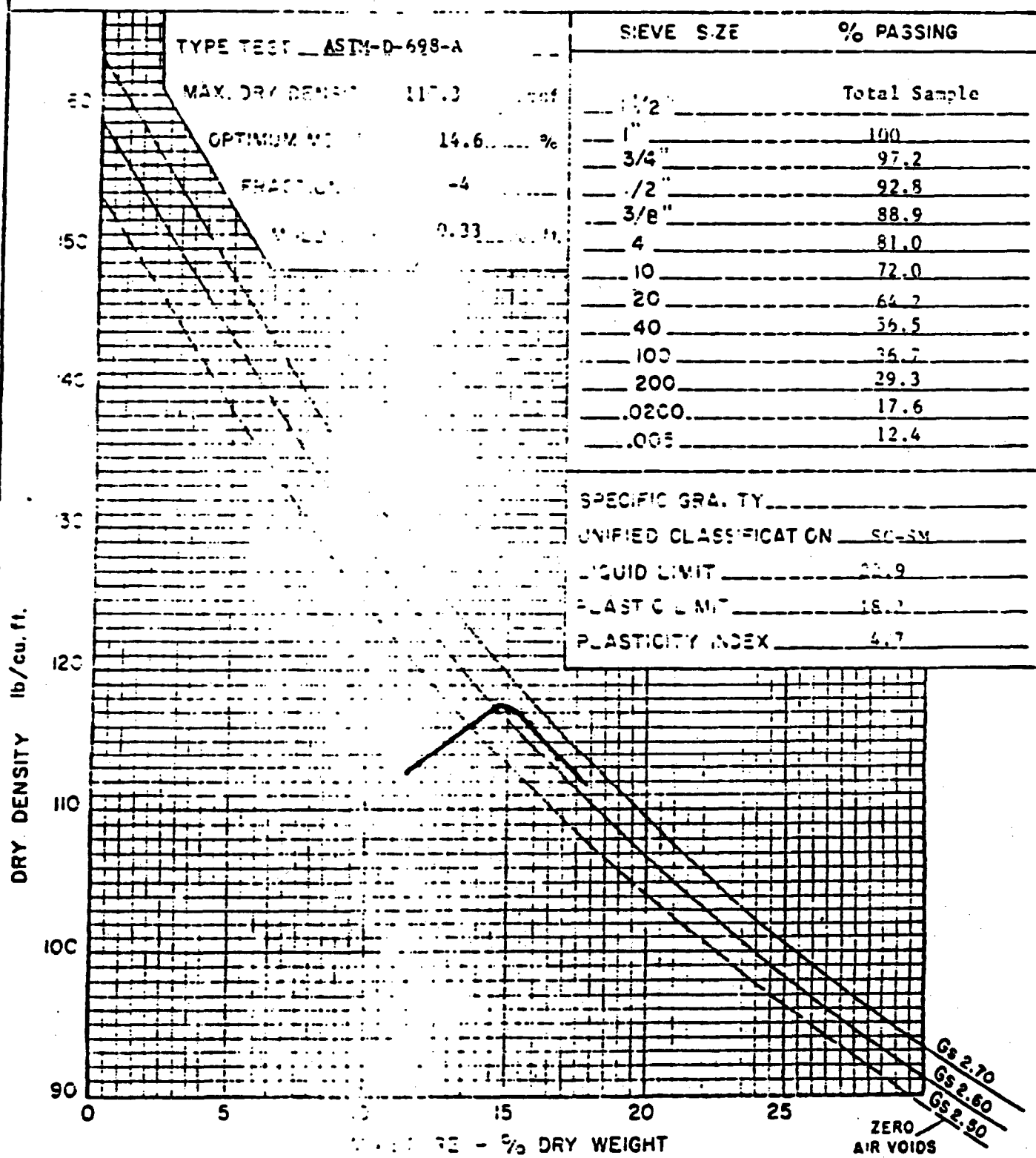
SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SC-SM

LIQUID LIMIT 22.9

PLASTIC LIMIT 18.2

PLASTICITY INDEX 4.7



MOISTURE — DENSITY RELATION

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir - Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #1

SOIL TYPE Sample scalped on 3/4" ASTM-D-698 5.3

TEST No. 71177-J #6

DATE 12-1-89

TEST BY DPW

TYPE TEST ASTM-D-698-C

MAX. DRY DENSITY 120.8

OPTIMUM WATER 12.5 %

FRACTION TEST -3/4

MOLD VOL. .075 cu. ft.

SIEVE SIZE	% PASSING
1 1/2"	Total Sample
1"	100
3/4"	97.5
1/2"	92.4
3/8"	89.0
4	81.4
10	71.4
20	62.6
40	59.4
100	48.8
200	36.0
.0200	19.3
.005	12.2

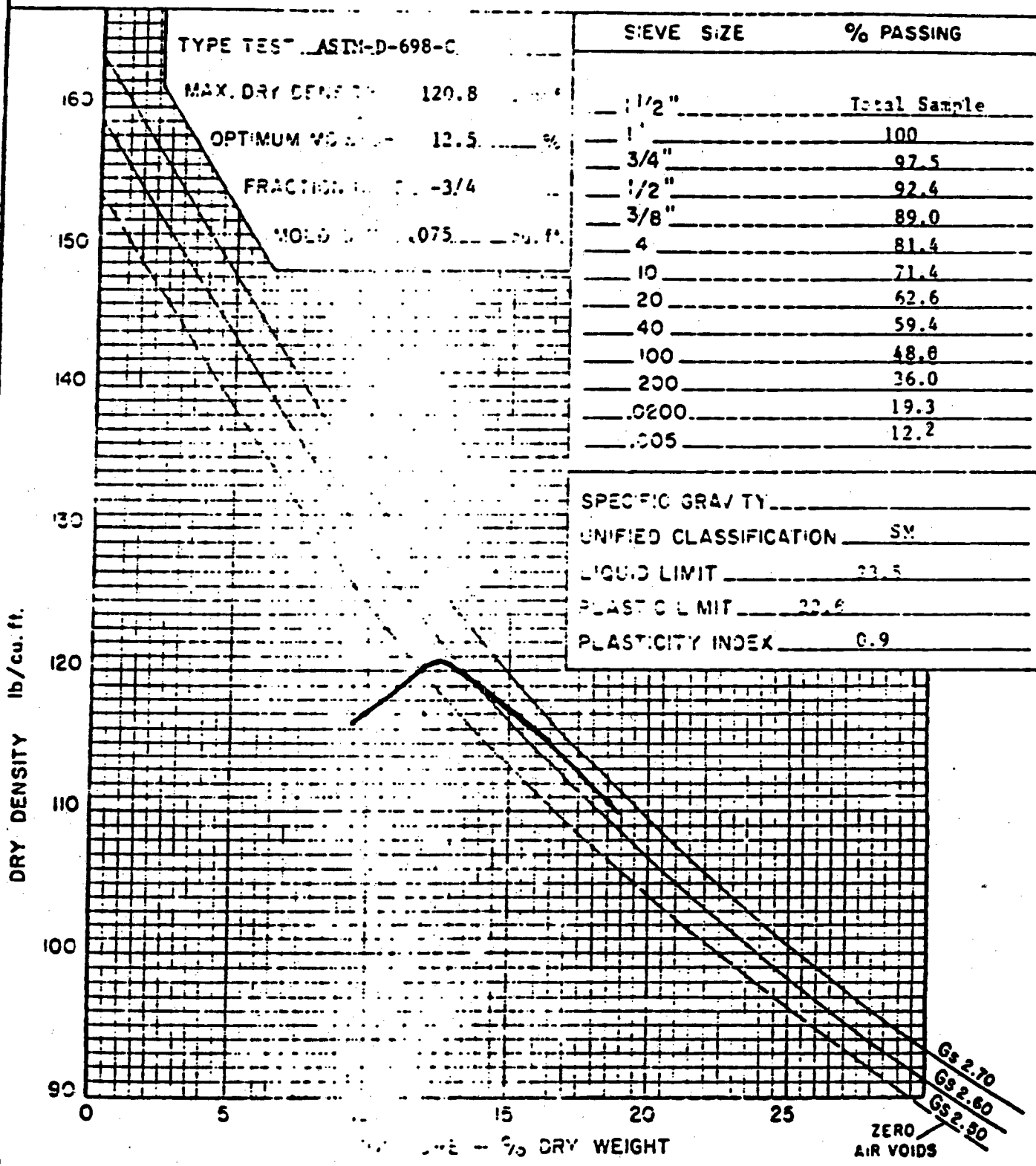
SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SM

LIQUID LIMIT 23.5

PLASTIC LIMIT 22.6

PLASTICITY INDEX 0.9



MOISTURE — DENSITY RELATION

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir - Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #2

SOIL TYPE Sample scalped on #4 sieve

TEST NO. 71177-1 #7

DATE 12-1-89

TEST BY DPW

TYPE TEST ASTM-D-689-C

MAX. DRY DENSITY 124.6

OPTIMUM MOISTURE 11.3 %

GRADE -3/4

[illegible] .075

SIEVE SIZE	% PASSING
1-1/2"	Total sample
1"	100
3/4"	99.2
1/2"	95.7
3/8"	92.5
4	85.4
10	73.6
20	65.8
40	59.5
100	43.8
200	38.3
.0200	22.8
.005	14.8

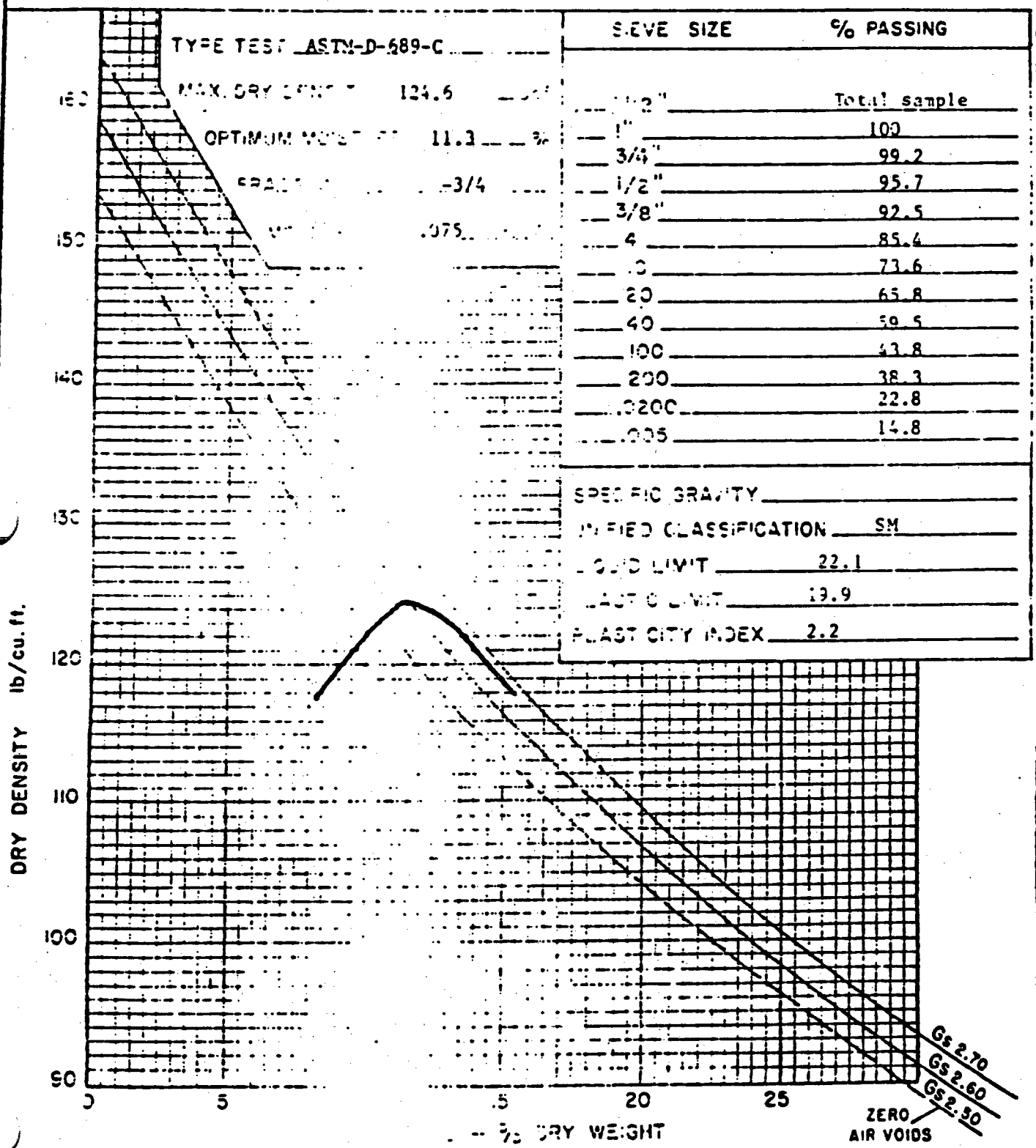
SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SM

LIQUID LIMIT 22.1

PLASTIC LIMIT 19.9

PLASTICITY INDEX 2.2



MOISTURE — DENSITY [illegible]

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir - Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #3

SOIL TYPE Sample scalped on #4 sieve

TEST NO. 71177-J #8

DATE 12-1-89

TEST BY CMB

TYPE TEST ASTM-D-698-C

MAX. DRY DENSITY 119.2

OPTIMUM MC 14.2 %

-3/4

.075

SIEVE SIZE	% PASSING
1 1/2"	Total sample
1"	100
3/4"	97.4
1/2"	93.7
3/8"	90.4
4	84.2
10	77.2
20	70.6
40	53.7
100	42.6
200	41.2
.0200	25.8
.005	16.4

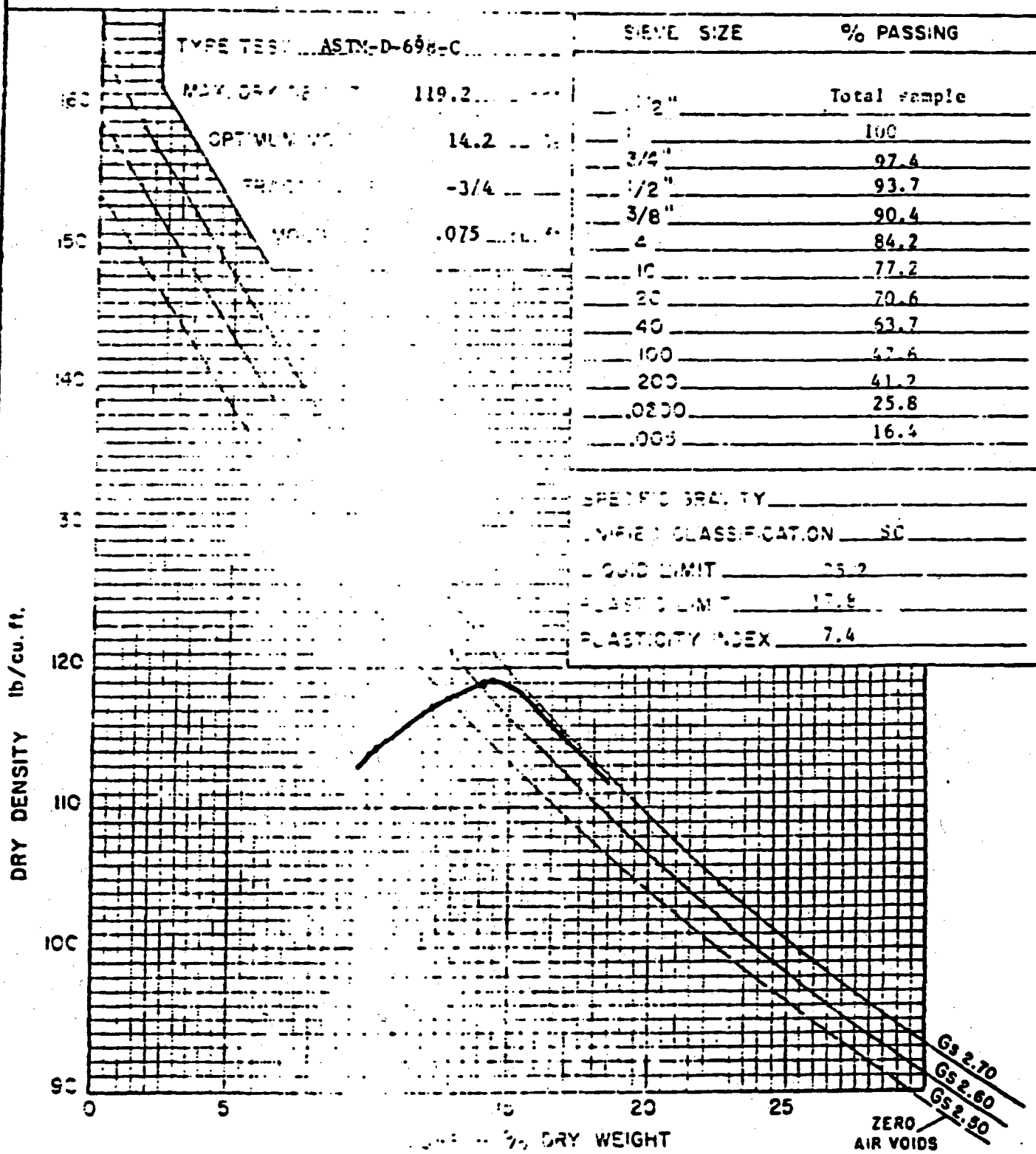
SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SC

LIQUID LIMIT 25.2

PLASTIC LIMIT 17.8

PLASTICITY INDEX 7.4



MOISTURE — DENSITY RELATIONS

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir - Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #4

SOIL TYPE Sample scalped on #4 sieve

TEST No. 71177-1 #9

DATE 12-1-89

TEST BY CMB

TYPE TEST ASTM-D-698-C

MAX. DRY DENSITY 123.4

OPTIMUM MOISTURE 11.4

FRACTION -3/4

[illegible] .075

SIEVE SIZE	% PASSING
1 1/2"	Total sample
1"	100
3/4"	99.1
1/2"	94.3
3/8"	89.5
4	77.7
10	64.9
20	55.6
40	49.7
100	34.1
200	24.9
.0200	14.2
.005	9.5

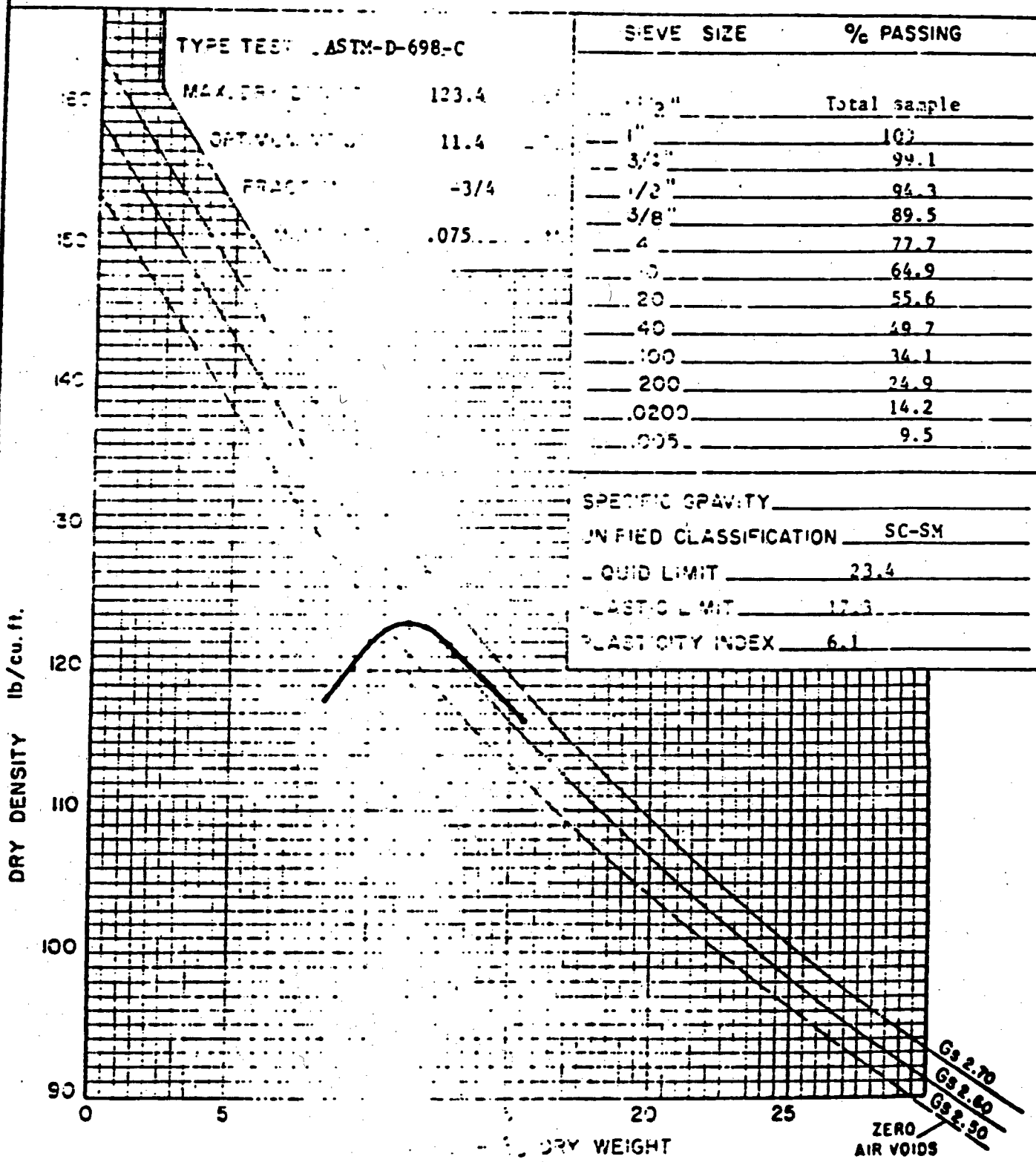
SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SC-SM

LIQUID LIMIT 23.4

PLASTIC LIMIT 17.3

PLASTICITY INDEX 6.1



MOISTURE — DENSITY [illegible]

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

PROJECT Cheney Reservoir - Grand Junction

CLIENT M.K. Ferguson

SAMPLE LOCATION Test Pile #5

SOIL TYPE Sample scalped on #4 sieve

TEST No. 71177-1 #10

DATE 12-1-89

TEST BY CM

TYPE TEST ASTM-D-698-C

MAX. DRY DENSITY 122.9 pcf

OPTIMUM MOISTURE 12.3 %

FRACTION -3/4

MOLD .075 cu. ft.

SIEVE SIZE	% PASSING
	Total Sample
1 1/2"	
1	100
3/4"	97.2
1/2"	92.8
3/8"	88.9
4	81.0
10	72.0
20	64.2
40	56.5
100	36.7
200	29.3
.0200	17.6
.005	12.4

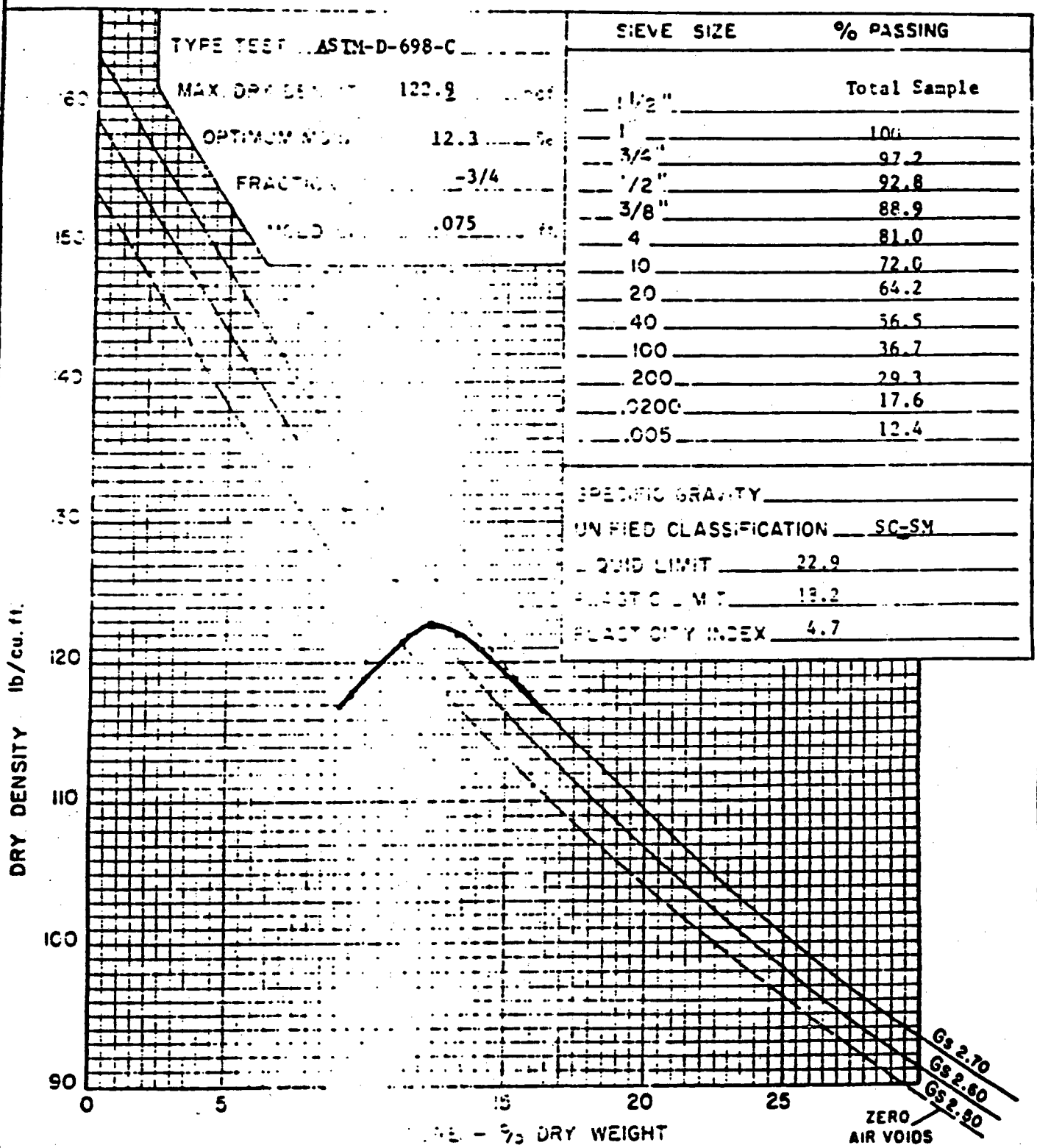
SPECIFIC GRAVITY

UNIFIED CLASSIFICATION SC-SM

LIQUID LIMIT 22.9

PLASTIC LIMIT 18.2

PLASTICITY INDEX 4.7



MOISTURE — DENSITY

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, GRAND JUNCTION, PUEBLO, GLENWOOD SPRINGS
WYOMING: EVANSTON

5025-16
MATERIAL PROP
GRAVELLY LAYER
- 24
HL

MK-FERGUSON COMPANY
A MORRISON KNUDSEN COMPANY

MKE DOC. 5025-GRJ-I-02-04230-00

INTER-OFFICE CORRESPONDENCE

DATE: April 19, 1990

TO: Rob Cooney

FROM: John Pepin

LOCATION: Albuquerque, NM

LOCATION: Grand Junction, CO

SUBJECT: CHENEY DISPOSAL SITE
GRAVEL STOCKPILE

Enclosed for your use and review are 20 gradation results from the subject stockpile.

Please have them analyzed for determination of required stockpile quantity using the most current design.

Should you have any questions, feel free to contact Thom Myer.

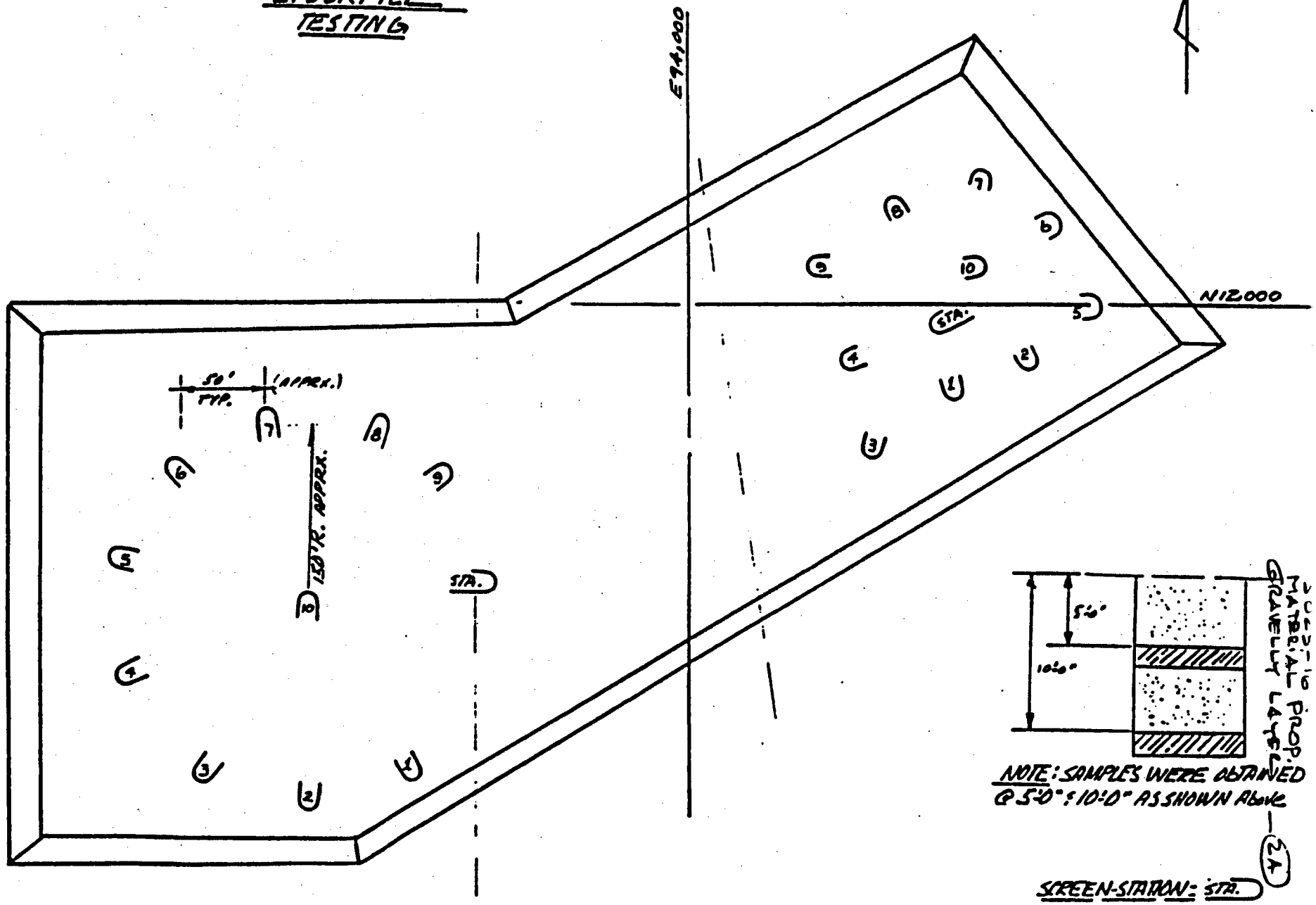
cc: w/att.
F. Feliz
T. Myer
GRJ90188

Attachment: MKE DOC. 5025-GRJ-R-02-04233-00

RECEIVED · MKE
APR 27 1990
UMTRA-S.E.

STA001

EROSION PROTECTION STOCKPILE TESTING



NOTE: SAMPLES WERE OBTAINED @ 5'-0" & 10'-0" AS SHOWN ABOVE

SCREEN-STATION: STA.

TEST LOCATION

TESTED BY: [illegible] 4-13-90

[illegible] 4-13-90

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A
MKE DOC. 5025-GRJ-R-02-04233-00

146 (1)

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/12/90 TEST NO.: EP-6-01 (o/s)

SAMPLE LOCATION: EROSION PROTECTION STOCKPILE

MATERIAL DESCRIPTION: MINUS 14" COBBLE MATERIAL W/SANDS & SILTS

SPECIFICATION: N/A SCALE ID NO.: CRANE 159276, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			76.60		2.4	2.4	97.6	
5			395.23		12.2	14.6	85.45	
3			283.56		8.7	23.3	76.7	N/A
	N/A		(SPLIT)	N/A				
1			42.38		19.1	42.4	57.6	
-1			127.68		57.6	100		

TOTAL SAMPLE NET WEIGHT 3,247 lbs

REMARKS:

TESTED BY: [illegible] DATE: 4/12/90

REVIEWED BY: [illegible] DATE: 4/13/90

0068MIS.DOC

147 2

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

SIEVE ANALYSIS

PROJECT: GRS-3050 DATE: 4/12/90 TEST NO.: ED-G-02 (D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand & silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, GRS-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			42.55		1.3	1.3	98.7	
5			188.72		5.8	7.2	92.8	
3			257.39		8.0	15.1	84.9	
	N/A			N/A	17.5			N/A
1			284.9		~~17.5~~ 32	32.6	67.4	
-1			109.91		67.4	100		

TOTAL SAMPLE NET WEIGHT 3,232 lbs

REMARKS:

TESTED BY: Carole Roesch DATE: 4/12/90

REVIEWED BY: [illegible] DATE: 4/14/90

0068MIS.DOC

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

148 3

SIEVE ANALYSIS

PROJECT: GRJ-8050 DATE: 4/12/90 TEST NO.: EP-G-031 D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: minus 14" Cobble Material w/sand & silts

SPECIFICATION: N/A SCALE ID NO.: CENE 15976, GES-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			140.16		4.5	4.5	95.5	
5			154.53		4.9	9.4	90.6	
3			243.82		7.8	17.2	82.8	
	N/A			N/A				N/A
1			28.09		15.2	32.4	67.6	
-1			125.22		~~67.6~~ JR	100		
					67.6			

TOTAL SAMPLE NET WEIGHT 3,130 lbs

REMARKS:

TESTED BY: Jade Reesed DATE: 4/12/90

REVIEWED BY: [signature] DATE: 4/14/90

0068MIS.DOC

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A)

149 4

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/12/90 TEST NO.: EP-G-04 (DIS)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: minus 14" cobble material w/sand & silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, 655-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			313.14		8.4	8.4	91.6	
5			239.03		6.4	14.8	85.2	
3			288.89		7.7	22.5	77.5	
	N/A			N/A				N/A
1			23.61		12.5	35.0	65.0	
-1			123.35		65.0	100		

TOTAL SAMPLE NET WEIGHT 3,733 lbs

REMARKS:

TESTED BY: Jack Rosenl DATE: 4/12/90

REVIEWED BY: [illegible] DATE: 4/14/90

0068MIS.DOC

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

150 5

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/12/90 TEST NO.: ED-G-05 (D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: minus 14" Cobble Material w/sand & silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			220.07		6.6	6.6	93.4	
5			224.85		6.7	13.3	86.7	
3			294.20		8.9	22.2	77.89	
	N/A			N/A				
1			271.01		13.7	35.8	64.2	
-1			126.80		64.2	100		

TOTAL SAMPLE NET WEIGHT 3,350 lbs

REMARKS:

TESTED BY: Gade Research DATE: 4/12/90

REVIEWED BY: [illegible] DATE: 4/14/90

0068MIS.DOC

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

UMTRA - GRJ 151
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - EA

SIEVE ANALYSIS

PROJECT: GRJ 3050 DATE: 4/12/90 TEST NO.: ED-G-06-(D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" Cobble material w/sand & silts

SPECIFICATION: N/A SCALE ID NO.: CENE 15976 GRJ-OX-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			291.0		8.3	8.3	91.7	
5			161.08		4.6	12.9	87.1	
3			238.33		6.8	19.8	80.2 SDM 4/11/90	
	N/A			N/A				
1			22.4		12.43 SDM 4/11/90	32.1	67.9	
-1			123.14		67.9	100		

TOTAL SAMPLE NET WEIGHT 3,495 lbs

REMARKS:

TESTED BY: Jack Rosch DATE: 4/12/90

REVIEWED BY: [illegible] DATE: 4/14/90

0068MIS.DOC

UMTRA - SRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

152 7

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/12/90 TEST NO.: ED-G-07 & D(S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand & silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15926, GRS-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			343.47		9.6	9.6	90.4	
5			188.38		5.2	14.8	85.2	
3			211.06		5.9	20.7	79.3	
	N/A			N/A				
1			23.19		13.4	34.1	65.9	
-1			113.75		65.9	100		

TOTAL SAMPLE NET WEIGHT 3,592 lbs

REMARKS:

TESTED BY: [illegible] DATE: 4/12/90

REVIEWED BY: [illegible] DATE: 4/14/90

006RMIS.DOC

UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

153 8

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ.3050 DATE: 4/12/90 TEST NO.: EP-G-08 (D/S)

SAMPLE LOCATION: Erosion protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand & silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			176.50		4.8	4.8	95.2	
5			139.12		3.8	8.6	91.4	
3			225.04		6.1	14.8	85.2	
	N/A			N/A				
1			2216		13.5	28.3	71.7	
-1			117.94		71.87	100		

TOTAL SAMPLE NET WEIGHT 3663 lbs

REMARKS:

TESTED BY: Jack Rousch DATE: 4/12/90

REVIEWED BY: [signature] DATE: 4/14/90

0068MIS.DOC

UMTRA-GRJ 5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

154

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/12/90 TEST NO.: ED 6-09 (-D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sands & silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			55.70		1.5	1.5	98.5	
5			202.62		5.5	7.0	93.0	
3			178.49		4.8	11.8	88.2	
	N/A			N/A				
1			19.22		12.3	24.1	75.9	
-1			118.80		75.9	100		

TOTAL SAMPLE NET WEIGHT 3,690 lbs

REMARKS:

TESTED BY: Jade Research DATE: 4/12/90

REVIEWED BY: [illegible] DATE: 4/14/90

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - (2A)

155 10.

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/12/90 TEST NO.: EP-G-10(-0/5)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: minus 14" cobble material w/sand & silts

SPECIFICATION: N/A SCALE ID NO.: CENE. 15976, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			64.57		1.9	1.9	98.1	
5			201.52		5.8	7.7	92.3	
3			279.03		8.0	15.7	84.3	
	N/A			N/A				
1			23.39		14.5	30.2	69.8	
-1			112.51		69.8	100		

TOTAL SAMPLE NET WEIGHT 3,475 lbs

REMARKS:

TESTED BY: Jack Roesch DATE: 4/12/90

REVIEWED BY: [illegible] DATE: 4/14/90

0068MIS.DOC

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

15 11

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/12/90 TEST NO.: ED-GR-11 (D/S)

SAMPLE LOCATION: Erosion protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand + silts

SPECIFICATION: N/A SCALE ID NO.: CEMNE 15976, GRJ-GR-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			486.94		13.5	13.5	86.5	
5			354.46		9.8	23.3	76.76	
3			331.10		9.2	32.86	67.84	
	N/A			N/A				
1			24.97		12.0	44.5	55.84	
-1			115.46		55.84	100		

TOTAL SAMPLE NET WEIGHT 3602 lbs

REMARKS:

TESTED BY: Jack Rousch DATE: 4-13-90

REVIEWED BY: William G. Moore DATE: 4/14/90

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

157 12

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/13/90 TEST NO.: EP-G-12 (D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand + silts

SPECIFICATION: _____ SCALE ID NO.: CRANE 1 5976, GRS-OC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			209.38		4.6	4.6	95.4	
5			228.97		5.0	9.6	90.4	
3			254.78		5.6	15.2 ~~15.5~~	84.87	
	N/A			N/A				
1			30.64		17.7	33.8 ~~32.9~~	67.0	
-1			116.18		67.0	100		

TOTAL SAMPLE NET WEIGHT 4544 lbs

REMARKS: _____

TESTED BY: Jack Rouch DATE: 4-13-90

REVIEWED BY: Ellen Gilbert DATE: 4/14/90

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UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

158 13

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/13/90 TEST NO.: ED-G-13 (D15)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand + silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15076, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			140.82		3.5	3.5	96.5	
5			366.37		9.1	12.6	87.4	
3			337.34		8.4	21.0	79	
	N/A			N/A				
1			33.61		17.3	38.3	61.7	
-1			119.45		61.7	100		

TOTAL SAMPLE NET WEIGHT 4013 lbs

REMARKS:

TESTED BY: Opal Rosel DATE: 4-13-90

REVIEWED BY: [illegible] DATE: 4/16/90

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159 1c

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A MORRISON KNUDSEN COMPANY

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/13/90 TEST NO.: ED-G-14 (D15)

SAMPLE LOCATION: Erosion Protection Stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand+silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 159726, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			74.60		2.3	2.3	97.87	
5			194.31		6.0	8.2	91.8	
3			229.92		7.1	15.3	84.7	
1			25.82		16.2	31.5	68.5	
-1			108.94		68.5	100		

TOTAL SAMPLE NET WEIGHT 3,260 lbs

REMARKS:

TESTED BY: Jack Roach DATE: 4-13-90

REVIEWED BY: [illegible] DATE: 4/14/90

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A MORRISON KNUDSEN COMPANY

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 24

160 1E

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/13/90 TEST NO.: EP-G-15(D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand + silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			76.74		2.2	2.2	97.8	
5			192.80		5.5	7.7	92.3	
3			202.22		5.8	13.5	86.5	
	N/A			N/A				
1			24.54		14.8	28.3	71.7	
-1			118.83		71.7	100		

TOTAL SAMPLE NET WEIGHT 3,490 lbs

REMARKS:

TESTED BY: Jade Roach DATE: 4-13-90

REVIEWED BY: Steve Martz DATE: 4/14/90

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UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
GRAVELLY LAYER - 2A

161 LL

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/13/90 TEST NO.: EP-G-16 (DIS)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble material w/sand+silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15-176, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			55.06		1.7	1.7	98.3	
5			118.66		3.7	5.4	94.6	
3			242.57		7.5	12.8	87.2	
	N/A			N/A				
1			24.64		14.7	~~27.45~~ Sam 4/14/90	~~72.45~~ Sam 4/14/90	
-1			121.05		~~72.45~~ Sam 4/14/90	100		

TOTAL SAMPLE NET WEIGHT 3,240 lbs

REMARKS:

TESTED BY: Jade Roesch DATE: 4-13-90

REVIEWED BY: [signature] DATE: 4/14/90

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UMTRA-GRJ
5025-16
MATERIAL PROP.
GRAVELLY LAYER - 2A

162

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/13/90 TEST NO.: EP-G-17 (D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: minus 14" cobble material w/sand + silt

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, GRJ-GRCH02

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			109.24		3.2	3.2	96.8	
5			235.71		6.9	10.1	89.9	
3			278.98		8.2	18.3	81.7	
	N/A			N/A				
1			30.54		17.3	35.6	64.4	
-1			113.82		64.4	100		

TOTAL SAMPLE NET WEIGHT 3,410 lbs

REMARKS:

TESTED BY: Jade Roesch DATE: 4-13-90

REVIEWED BY: [signature] DATE: 4/14/90

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UMTRA-GRJ
5025-16
MATERIAL PROP.
GRAVELLY LAYER - 2A

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MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ-3050 DATE: 4/13/90 TEST NO.: EP-G-18 (D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble materials w/sand + silts

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, GRJ-QC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			377.00		10.4	10.4	89.6	
5			186.18		5.1	15.5	84.5	
3			336.48		9.3	24.87 SDM 4/14/90	75.23 SDM 4/14/90	
	N/A			N/A				
1			28.19		14.45 SDM 4/14/90	39.2	60.8	
-1			118.68		60.8	100		

TOTAL SAMPLE NET WEIGHT 3635 lbs

REMARKS:

TESTED BY: Jack Rossi DATE: 4-13-90

REVIEWED BY: William C. Martin DATE: 4/14/90

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UMTRA-GRJ
5025-16
MATERIAL PROP.
GRAVELLY LAYER - 2A

1-4 19

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: GRJ-80510 DATE: 4/13/90 TEST NO.: ED-G-19 (D18)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble materials w/sand+silts

SPECIFICATION: N/A SCALE ID NO.: CEME 15976, GRJ-WC-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			73.61		2.2	2.2	97.8	
5			267.83		8.1	10.3	89.7	
3			237.89		7.2	17.5 ~~20.0~~ 4/14/90	82.5	
	N/A			N/A				
1			23.32		14.4	32.0	~~67.9~~ ~~68.0~~ 4/14/90	
-1			108.47		~~67.9~~ ~~68.0~~	100		

TOTAL SAMPLE NET WEIGHT 3323 lbs

REMARKS:

TESTED BY: Jack Roesch DATE: 4-13-90

REVIEWED BY: Steven Martz DATE: 4/14/90

UMTRA-GRJ 5025-16
MATERIAL PROP.
GRAVELLY LAYER - 2A

165 2

MK-FERGUSON
A MORRISON KNUDSEN COMPANY

SIEVE ANALYSIS

PROJECT: 625-3050 DATE: 4/13/90 TEST NO.: EP-G1-20 (D/S)

SAMPLE LOCATION: Erosion Protection stockpile

MATERIAL DESCRIPTION: Minus 14" cobble materials w/sandstits

SPECIFICATION: N/A SCALE ID NO.: CRANE 15976, 625-CR-102

SIEVE SIZE	RETAINED WT. + TARE	TARE	WEIGHT RETAINED	ORIGINAL WT. - TARE	PERCENT RETAINED	CUMULATIVE % RET.	CUMULATIVE % PASS.	REQUIRED
14			0		0	0	100	
8			172.77		4.8	4.8	95.2	
5			175.70		4.9	9.8	Sign 4/14/90 ~~90.2~~	
3			306.03		8.6	18.34	81.76	
	N/A			N/A		Sign 4/14/90	Sign 4/14/90	
1			22.77		12.6	Sign 4/14/90 ~~30.9~~ 31.0	69.0	
-1			124.88		69.0	100		

TOTAL SAMPLE NET WEIGHT 3,565 lbs

REMARKS:

TESTED BY: Opal Rogers DATE: 4-13-90

REVIEWED BY: [illegible] DATE: 4/14/90

REVISION 3

LAYER 3 CLAYEY LAYER - In-situ

The material excavated from this layer will be used as a radon barrier.

For Geotechnical Design Purposes, the soil parameters are summarized below:

Dry Density, γ_d = 107.4 psf

In-situ Moisture Content = 9.2 %

In-situ Density = 117.3 pcf

Saturated Density = 130 pcf

Specific Gravity = 2.7

SHEAR STRENGTH PARAMETERS

End of Construction : c = 1000 psf, $\phi = 0^\circ$

Long-Term Seismic : c = 0°, $\phi = 18.4^\circ \approx 19^\circ$

Long-Term Static : c = 0°, $\phi = 28.8^\circ \approx 29^\circ$

CONSOLIDATION PARAMETERS

P_p = 6.0 ksf $C_v = 2 \times 10^{-2}$ cm²/sec.

$C_{c\varepsilon} = 0.116$

$C_{v\varepsilon} = 0.015$

Density, Moisture Content and Specific Gravity

(a) Dry Density $= \frac{101.3 + 101.7 + 117.9 + 108.7}{4} = 107.4$ pcf

(b) In-situ Moisture Content $= \frac{8.4 + 4.8 + 5.6 + 6.7 + 7.9 + 11.9 + 11.9 + 11.2 + 17.2 + 10.4 + 5}{11}$

$= 9.2$ (%)

(c) Average Moist Density $= 107.4 \times 1.092 = 117.3$ pcf

(d) Specific Gravity : Use 2.7, which is a good number for clay soils

(e) Saturated Density :

TP-2 (18-18.7') $- 101.3 \times 1.189 = 120.5$

TP4 (19-20') $- 101.7 \times 1.233 = 125.4$

TP6 (17-17.7') $- 117.9 \times 1.226 = 144.5$

Average $= 130.1$ pcf

Average w/c $= \frac{18.9 + 23.3 + 22.6}{3} = 21.6$ pct

Checking : Void ratio, $e_{ave} = \frac{62.4 \times 2.7}{107.4} - 1 = 0.568$.

$w = (0.568 \times 62.4) / 2.7 \times 62.4 = 21.0$ ∴ o.k.

REV. 3 — LAYER 3 CLAYEY LAYER

STRENGTHS BASED ON LIQUIDITY INDICES

LIQUID LIMIT AND PLASTICITY INDEX MEASUREMENT

(DATA FROM SUMMARY OF SOIL TEST ON SH. —)

$$LL_{AVE.} = \frac{23+23+33+40+40+41+43+37+44+33+41+29+28+38}{14}$$

$$= 35.2\%$$

$$PI_{AVE.} = \frac{10+7+18+24+25+19+25+22+27+17+24+15+13+24}{14}$$

$$= 19.3\%$$

USE $W_{AVE.} = 21\%$ → FROM PREVIOUS PAGE

$$PL_{AVE.} = LL_{AVE.} - PI_{AVE.}$$

$$= (35.2 - 19.3)\% = 15.9\%$$

$$LI_{AVE.} = \frac{W_{AVE.} - PL_{AVE.}}{LL_{AVE.} - PL_{AVE.}}$$

$$= \frac{21 - 15.9}{35.2 - 15.9} = 0.26$$

REV. 3: Due to the similarity of the site and soil properties, it is reasonable to use in REV. 3 the previous shear strength and consolidation test results. H.L. 6/18/

Shear Strength

The undrained shear strengths from UU and CU tests as follows:

Sample	Strength	Test Type
TP-4, 19-20'	c= 272.3 psf; ϕ=0	UU
TP-6, 17-17.7'	c= 763.9 psf; ϕ=0	UU
		CU

These shear strengths were extremely low: soft to barely medium stiff. This is quite in contradiction to:

(i) The high blow counts and hard excavation efforts experienced

(ii) The fact that the saturated moisture contents before shearing were close to the plastic limits, which would indicate high preconsolidation pressures and consequently high undrained shear strengths.

The low shear strength measurements are attributed primarily to the poor quality of the samples: the samples were probably extremely disturbed, and sometimes cracked. This contention is supported by (a) the shape of the consolidation curves (Sheet ~~144~~ 180), (b) the sketches of the samples at failure, showing the presence of the vertical tensile cracks, and (c) conversations with personnel from Western Engrs. & Chem & Assoc. After all, you do not obtained good, undisturbed samples through driving.

The alternative chosen therefore, was to investigate the shear strength value based on indirect methods, as presented on the next sheet.

REVISION 3

(Rewrite of Previous Calculation + changes)

For clays of moderate to low sensitivity, e.g. sensitivity of 2 to 4 for LI = 0.26, P_p = 2 tsf to 4 tsf ($P_{p\,ave.} = \frac{2+4}{2} = 3$ tsf = 6 ksf)
This is higher than the 3 ksf P_p determined from the consolidation curve.

$$(p_o')_{AVE.\,20'} \cong 95 \times 2' + 130 \times 18' = 2530 \text{ psf}$$

(Existing Overburden)

$$\therefore OCR = \frac{6000}{2530} = 2.37$$

$$A = \left(\frac{S_u}{p_o'}\right)_{NC} = 0.11 + 0.0037(PI) = 0.11 + 0.0037 \times 19.3$$
$$= 0.181 \text{ (Ref. 16, page 142)}$$

$$B = \left(\frac{S_u}{p_o'}\right)_{OCR} \quad \text{(LADD 1977, REF. 14, p. 57)}$$

$$\frac{B}{A} = OCR^{0.8} \text{ for } OCR \leq 7$$

$$\therefore S_{u\,OCR} = 0.181 \times 2530 \times (2.37)^{0.8} = 913 \text{ psf} \cong 1{,}000 \text{ psf}$$

Therefore, if layer 3 got saturated without time for further consolidation, it will have a strength of at least 1000 psf. However, based on the rainfall pattern, the low permeability of layer 3 as well as its low degree of saturation in-situ, it is very likely that little change in moisture content will take place by the end of construction (short term). The lowest blow count is 49 blows per foot, which corresponds to a hard clay. Conservatively we assume a stiff clay, S_u = 1,500 psf.

For long-term seismic stability, it is logical to assume that the clay layer has essentially consolidated 100 percent under the overburden pressure before being subjected to the seismic force. Therefore, the consolidated undrained (CU) tests will be applicable.

Cont'd to next sheet

REVISION 3

Continuation from previous Sheet.

The CU test Mohr circles are presented on Sheet 179. The total stress Mohr circle for the highest confining pressure is above the inferred envelope. We believe, this is due to the fact that the higher confining pressure closes off the cracks and mitigates the disturbance effects as evident from the SHANSEP concepts from M.I.T. Therefore, this circle may be representative of the field conditions.

We recommend,

For long-term seismic, $\phi = 18.4^\circ$, $C = 0$

For long-term static, $\phi = 28.8^\circ$, $C = 0$

MAT. FOR RADON BARRIER

UMTRA-GRJ
~~5025-16~~
~~MATERIAL PROPERTIES~~
CLAYEY LAYER – (3)

H.L. 6/23/9
SHB 6-22-90

SUMMARY OF SOIL TEST RESULTS

Job No 5025-16 CLAYEY LAYER – (3) Project Name UMTRA – GRAND JUNCTION Feature CHENEY DISPOSAL SITE Date 6/9/90

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Sand	Fines	Atterberg Limits LL	PI	Specific Gravity G	Natural W %	Natural γd	Compaction Test	Optimum W	Optimum γd	Shear Strength Test	Initial W	Initial γd	C	φ	Permeability γd	95% & 100%	Consolidation Cc	Cr	R	Notes
114	5	0	1	SC/SM	4	24	72	23	10																	CLAY CONTENT (HYDROMETER) = 19%
122	9	7	12	SC	6	54	40	23	7				SP	12.5	120.7											CLAY CONTENT = 18%
123	10	14	23	CL	17	34	49	33	18	2.79			SP	15.5	114.3						105.5 / —	4.7×10⁻⁷ / —				CLAY CONTENT = 19%
124	11	18.5	23	CL	3	9	88	40	24				SP	16.1	112.2											CLAY CONTENT = 43%
125	12	18	21	CL	4	8	88	40	25	2.77			SP	17.1	109.7						97.6 / 102.7	3.9E-7 / 1.6E-8				CLAY CONTENT = 42%
126	13	1	4	CL	38	15	47	41	19																	CLAY CONTENT = 31%
126	14	14	21	CL	0	6	94	43	25				SP	16.5	110.2											CLAY CONTENT = 42%
127	15	15	25.5	CL	3	30	67	37	22	2.77			SP	14	117.6						105 / —	4.7E-8 / —				CLAY CONTENT = 32%
128	16	16	25	CL	0	5	95	44	27				SP	12.9	109.2						97 /	2.9E-8 /				CLAY CONTENT = 47%
130	17	10.5	14.5	CL	0	36	64	33	17	2.77			SP	16	110.6						100.5 / 106.3	4E-7 / 2E-7				CLAY CONTENT = 25%
131	18	0	1.5	CL	4	23	73	41	24																	CLAY CONTENT = 42%
131	19	17	25	CL	2	32	66	29	15	2.76			SP	14.2	114.9						104.9 /	1.2E-7 /				CLAY CONTENT = 26%
133	20	11	15	CL	4	44	52	28	13				SP	13.9	116.2											CLAY CONTENT = 16%
136	21	10	17	CL	1	11	88	38	24	2.79			SP	15.7	114.9						102.4 / 109.3	2.4E-7 / 1.4E-8				CLAY CONTENT = 41%
				AVERAGE	6	24	70	35.2	19.3	2.78				14.55	113.7						102.7 / 106.1	2.18E-7 / 8.18E-8				

ADDITIONAL TESTS ON PERMEABILITY

TP #128 (100% Comp) γd (pcf)	k (cm/sec)
97	2.9E-8

TP #130 (100% Comp) γd (pcf)	k (cm/sec)
106.7	1.5E-7

NOTE: SEE SH. 173 FOR HOLE OR TRENCH NUMBER

FOR PERMEABILITY TESTS RESULTS:

For 100% Compaction (Geometric Average)

$$k_{AVG} = \sqrt[6]{1.6E-8 \times 2.9E-8 \times 2E-7 \times 1.4E-8 \times 8.18E-8 \times 1.5E-7}$$
$$= 5.02 \times 10^{-8} \text{ cm/sec}$$

* Visual Classification

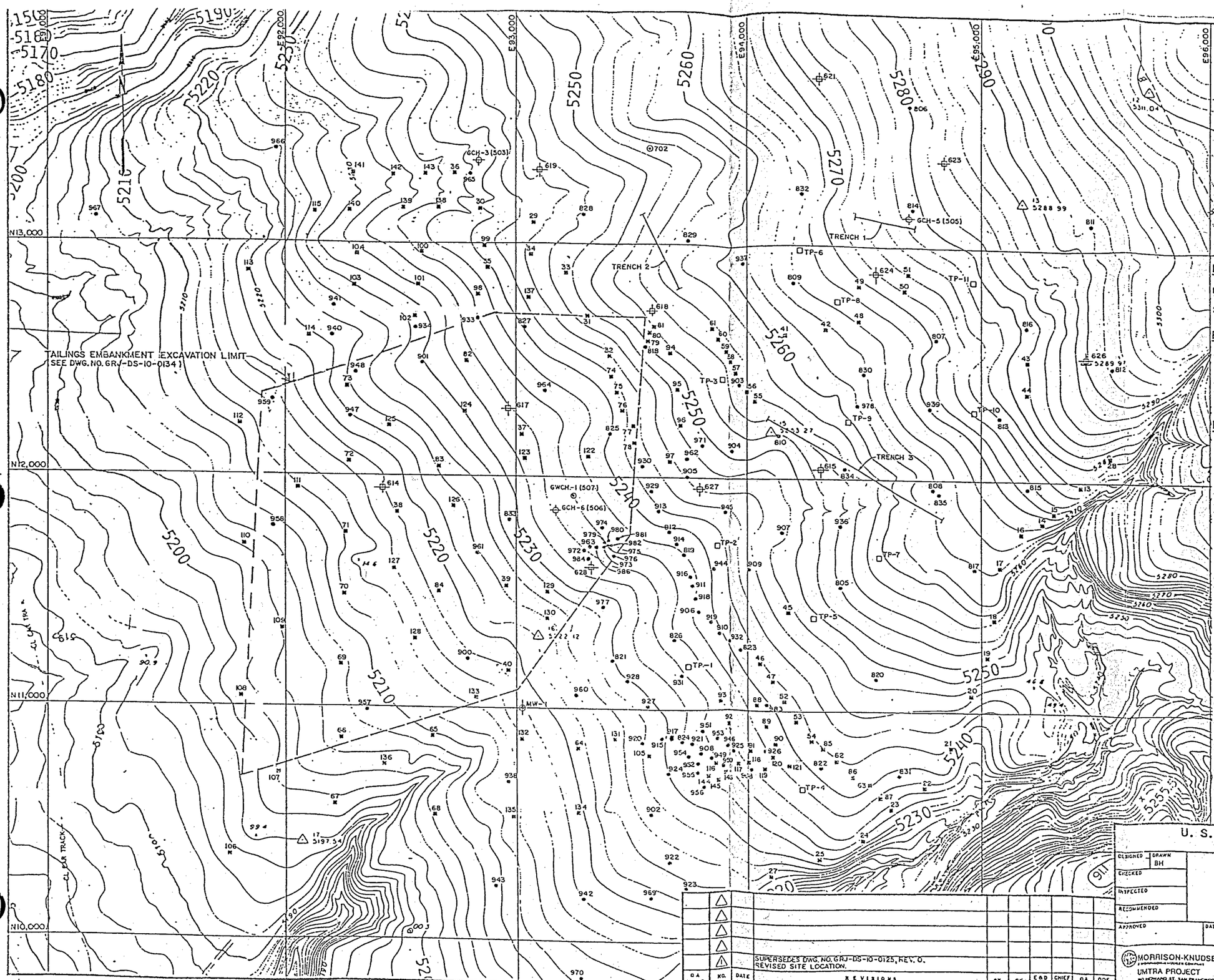
SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

(1) SOIL COMPACTED TO ≅ 95% (STANDARD PROCTOR
(2) SOIL COMPACTED TO ≅ 100% (STANDARD PROCTOR

173



BOREHOLES & TEST PITS	COORDINATES :	
GWCH-2 (508)	N14,170	E96,180
GWCH-3 (509)	N14,335	E94,140
GCH-1 (501)	N15,240	E95,360
GCH-2 (502)	N14,280	E94,160
GCH-4 (504)	N14,140	E96,240
613	N15,270	E95,100
701	N14,460	E95,020
26	N10,170	E93,985
935	NOT DRILLED	
985	NOT DRILLED	

NOTE:

1. GROUND WATER DATA AND SUBSURFACE EXPLORATION LOGS ARE AVAILABLE FROM THE CONTRACTOR.
2. BOREHOLES AND TEST PITS NOT SHOWN ON DRAWING ARE LISTED ON TABLE ABOVE.

REFERENCE DRAWINGS:

GRJ - DS - 10 - 0125 CONSTRUCTION FACILITIES AND SITE DRAINAGE

GRJ - DS - 10 - 0134 TAILINGS EMBANKMENT AREA EXCAVATION PLAN

LEGEND:

5240 — EXISTING SITE FEATURES AND CONTOURS
N12,000 — CONSTRUCTION GRID COORDINATE
GCH-6 — BORINGS DRILLED BY NUS IN 1982
GWCH-3 — MONITOR WELLS INSTALLED IN 1982
701 — MONITOR WELLS INSTALLED BY TAC IN 1985
616 — TEST PITS DUG BY TAC IN 1984
617 — TEST PITS DUG BY LINCOLN DEVORE/MKE IN 1985
TP-5 — TEST PITS DUG BY WESTERN ENG./MKE IN 1987
MW-1 — EXISTING WATER SUPPLY WELL BY MKE IN 1987
41 — TEST PIT BY TAC IN 1989
930 — WELL-BOREHOLE BY TAC IN 1989

ALL RADON BARRIER MATERIAL WILL BE OBTAINED FROM THE EXCAVATION FOR THE TAILINGS EMBANKMENT
H.L. 6/23/90
SW 6-22-90

SCALE 200 0 200 400 FEET

QA MK-F	NO.	DATE	REVISIONS	BY	CK	E&D MGR.	CHIEF ENG.	QA MGR.	DOE APP.
	1		SUPERSEDES DWG. NO. GRJ-DS-10-0125, REV. 0. REVISED SITE LOCATION.						

U. S. DEPARTMENT OF ENERGY
ALBUQUERQUE, NEW MEXICO

DESIGNED / DRAWN BH
CHECKED
INSPECTED
RECOMMENDED
APPROVED — DATE — DOE PROJECT ENGINEER — DATE

CHENEY RESERVOIR DISPOSAL SITE
GRAND JUNCTION, COLORADO
PHASE II CONSTRUCTION

BORINGS AND TEST PITS LOCATION PLAN

MORRISON-KNUDSEN ENGINEERS, INC.
UMTRA PROJECT
180 HOWARD ST. SAN FRANCISCO, CA 94105

PROJECT NO. DE-AC04-83AL18796
DRAWING NO. GRJ-DS-10-0223 REV. 1

Consolidation Tests

2 consolidation tests were run on this layer: one on transported clay (TP-2, 18.0-18.5') and one on a decomposed shale (TP-9, 20.0-20.7'). The shale sample was very disturbed and crumbly, and therefore its results are discarded. The other test was replotted on a newer scale, and the relevant parameters were computed (See Sheet ~~144~~ 180)

They are:

$P_p = 3.0$ ksf

$C_{ca} = 0.116$

$C_{ra} = 0.015$

$C_v = 2 \times 10^{-2}$ cm²/sec (DM-7 Fig 3-5) (Sheet 141)
(corresponding to LL=38)

As discussed earlier, the samples were disturbed, and therefore a low value of P_p was obtained. We recommend $P_p = 6.0$ ksf as determined by LI analysis on Sheet ~~141~~. 177

Note: Presented in Sheet ~~138A~~ 175 is a set of time rate of settlement curves for the transported clay sample. The 90 pct consolidation occurs around 0.5 min = 30 secs.

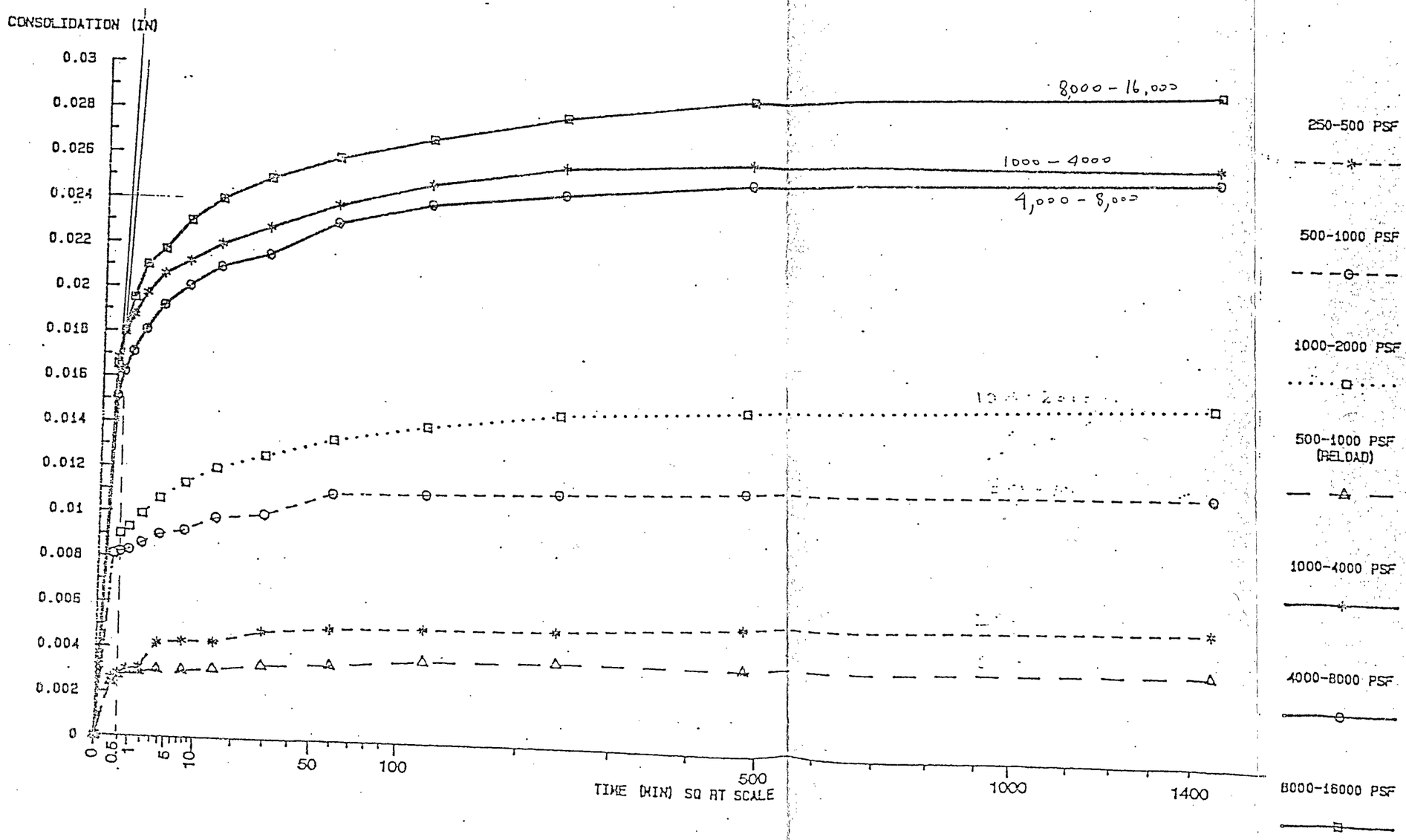
For a 1" thickness, and double drainage, $C_v = \frac{(1.0 \times 2.54)^2 \times 0.848}{4 \times 30} = 0.046$

This is somewhat higher than the C_v recommended. However, as seen on Sheet ~~141~~ 184, the permeability of this sample is the highest. Therefore, the recommended value is chosen to represent a more typical sample.

~~138~~ A
175

PS 2/9/88
WYL 2/5/88

CHENEY RESERVOIR
TP 2 @ 18-18.5
CONSOLIDATION DATA



1/45 176

PS 2/4/88
WYL 2/5/88

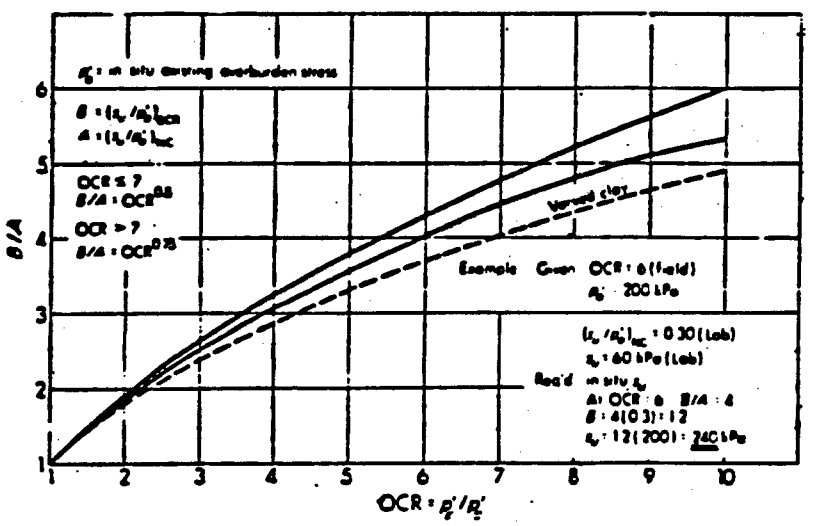


Figure 2-18 Normalized plot to obtain the ratio of undrained shear strength at some preconsolidation pressure to a normally consolidated value for several OCR. [*After Ladd et al.* (1977).] Note that above plot is based on direct simple shear (DSS) tests.

obtain this curve all had high w_N and w_L, but with five clays involved a trend seems to be established so that the curve might be used for clays with lesser w_N and w_L (less than 50 percent). In order to use this plot it is necessary to obtain an undisturbed sample for consolidation testing. From the consolidation test obtain p'_c. Remold a sample and K_o consolidate it to the present in situ p'_o state. Obtain s_u from this sample. Now with s_u, p'_o, and p'_c one can compute the OCR, enter Fig. 2-18, and obtain the ratio B/A. Since A is known, we can compute B and using B compute s_u at the present OCR as

$$s_{u,\mathrm{OCR}} = B(p'_o)$$

FROM: REF. 14, Page 57

Foundation Analysis And Design / Bowles

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project UMTRA-GRJ
Feature EMBANKMENT DESIGN
Item MATERIAL PROPERTIES

Contract No. 5025-16
Designed H.L.
Checked SKB

Sheet HT 177
File No.
Date 6/18/90
Date 6-22-90

REV. 3 6/18/90
H.L.

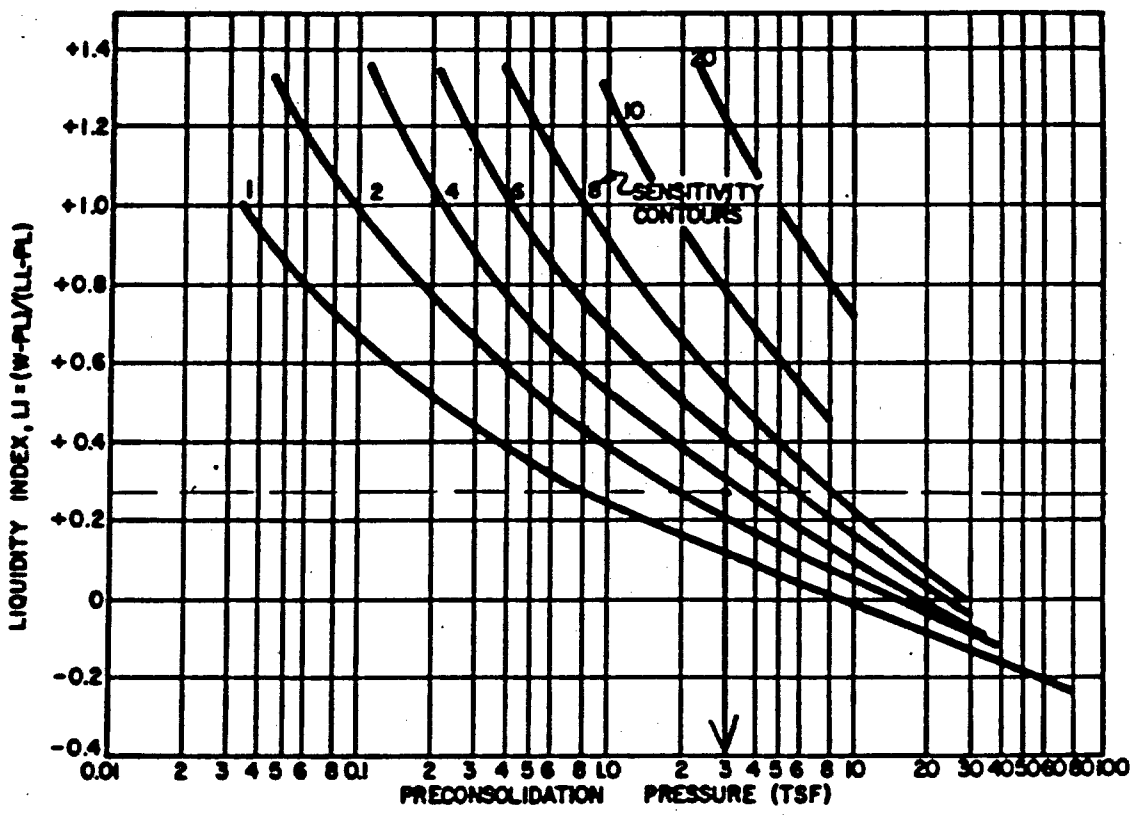


FIGURE 3
Preconsolidation Pressure vs. Liquidity Index

FROM: REF. 16, page 7.1-142

FROM
REF: 16, page 7.1-144

PS 2/4/88
WYL 2/5/88

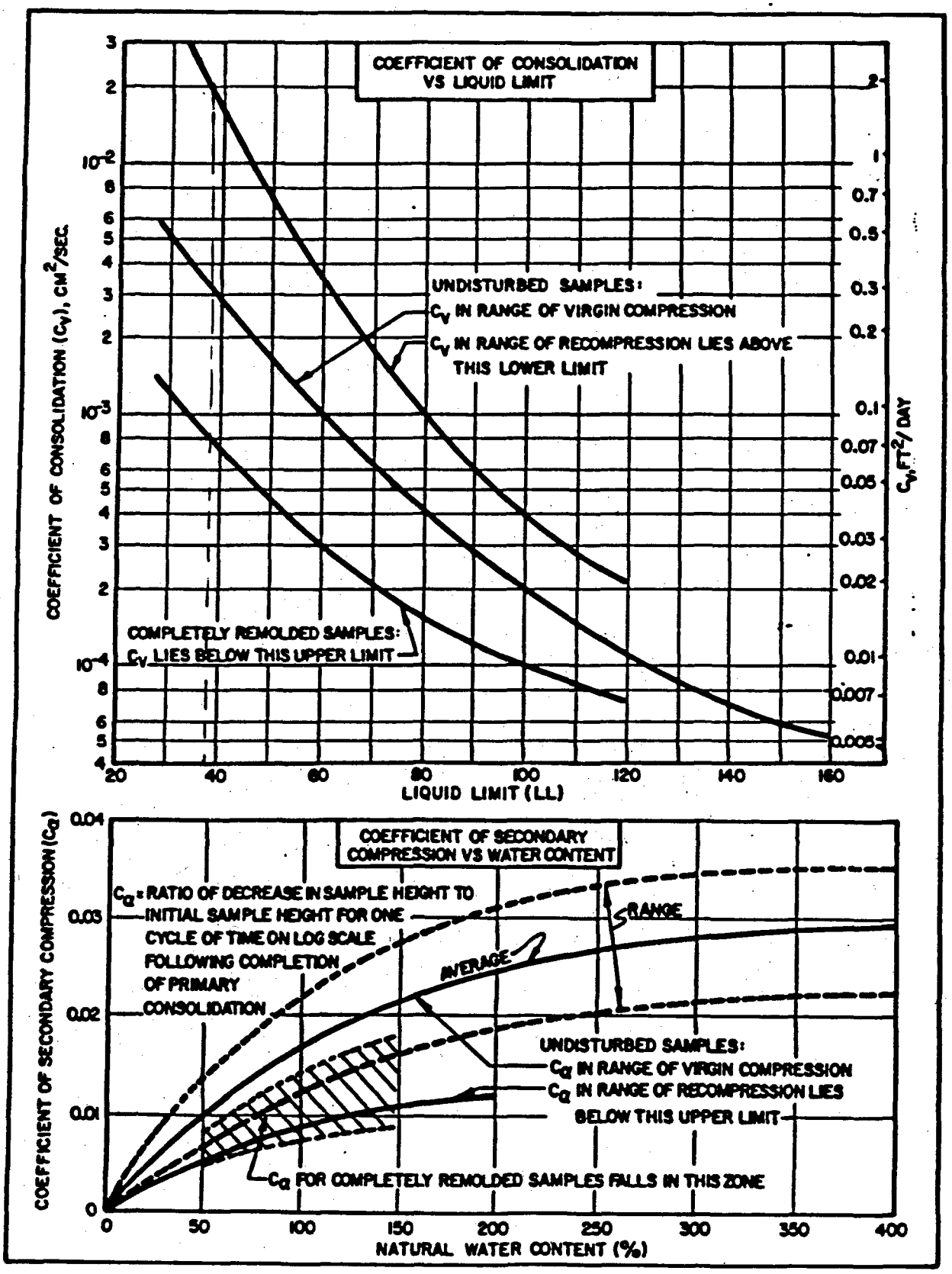
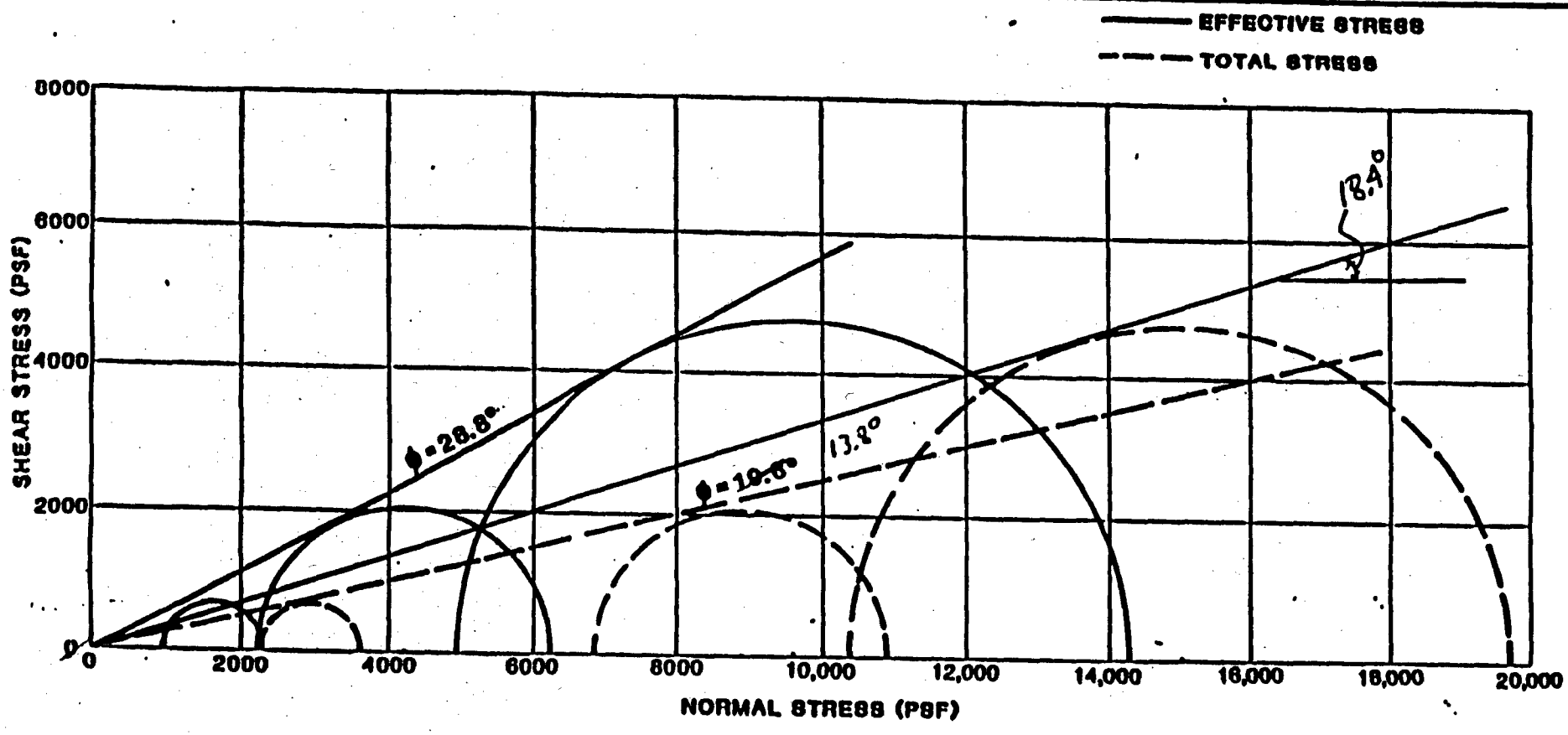


FIGURE 4
Approximate Correlations for Consolidation Characteristics

PS 2/9/88

MKE DOCUMENT NO. 5025-GRJ-C-09-01601-00



CHENEY RESERVOIR

TP 2 AT 18-18.7

MOHR ENVELOPE

TRIAXIAL TEST DATA

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WYL 2/5/88

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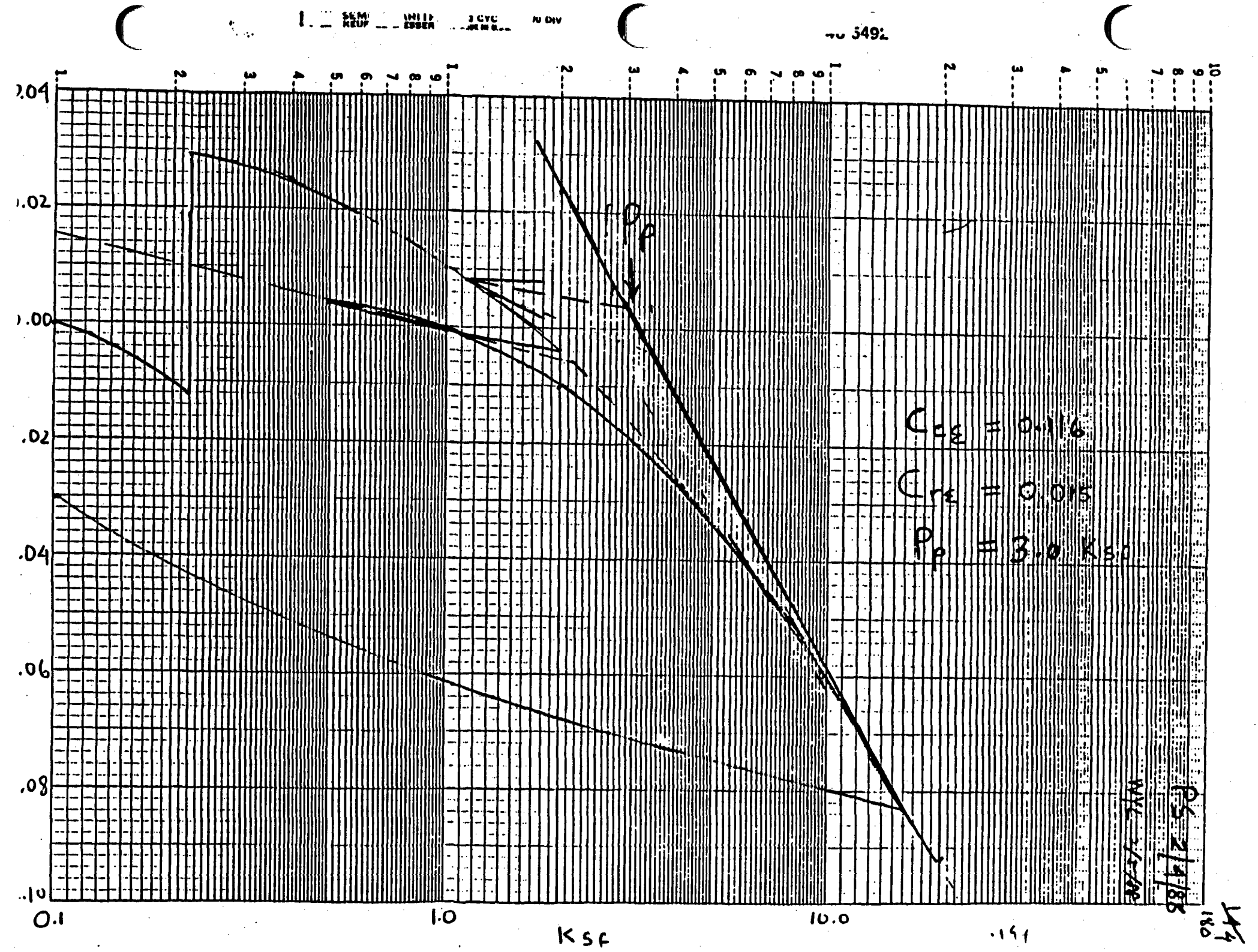
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3 CYC
N DIV
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.04
.02
.00
.02
.04
.06
.08
.10
0.1
1.0
10.0
KSF
P_p
C_{uu} = 0.116
C_{ru} = 0.015
P_p = 3.0 KSF

REV. 3

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES — — — SWS 6-22-9
LAYER 3 — CLAYEY LAYER

181
H.L. 6/18

SUMMARY OF SOIL TEST RESULTS

Job No 5025-16 Project Name UMTRA - GRAND JUNCTION Feature CHENEY DISPOSAL SITE Date 6/18/90

Hole or Trench Number	Sample Number	Depth (FEET) From	Depth (FEET) To	Laboratory Classification	Mechanical Analysis – Gravel	Mechanical Analysis – Sand	Mechanical Analysis – Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γ_d	Compaction Test	Compaction Optimum w	Compaction Optimum γ_d	In-Situ Shear Strength Test	Initial w	Initial γ_d	c / ϕ	c' / ϕ'	Permeability γ_d	k	Consolidation c	C_c / C_r	$\bar{p}$ (ksf)	Notes
SHEAR STRENGTHS AND CONSOLIDATION PARAMETERS (TAKEN FROM PREVIOUS TESTS)																										
TP-2	TUBE	18	18.5	CL												CU	18.9		0 / 18.4	0 / 28.8				0.116 / 0.015	3.0	
TP-4	"	19	20	CL												UU	23.3		272.3 / 0				0.046			
TP-6	"	17	17.7	CL												UU	22.6		763.9 / 0							
NATURAL MOISTURE CONTENT AND DRY DENSITY (TAKEN FROM PREVIOUS TESTS)																										
615		7	8	CL							17.2															
TP-1		19.5	21.5	CL							8.4															
TP-2	TUBE	18	18.5	CL							4.8	101.3														
TP-3		15.5	20	CL							5.6															
TP-4		19	20	CL							6.7	101.7														
TP-6		17	17.7	CL							7.9	117.9														
TP-9		18.5	20	CL							11.9															
TP-9		20	20.7	CL							11.9	108.7														
TP-11		18.5	20.0	CL							11.2															
GCH-4		35	44	CL/GC							10.4															
GCH-6		21	29	CL/GC							5															
						AVERAGE					9.2	107.4														

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

NOTE: Material from this layer will be compacted and used as a random barrier.

LAS 182

PS 2/4/88

WYL 2/5/88

REV. 3

Use this for natural moisture content

H.L. 6/15/9

TEST RESULTS

CHENEY RESERVOIR DISPOSAL SITE

Item 4: Atterberg Limits and Natural Moisture Contents

Test Pit No.	Depth (ft)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Natural Moisture Content
TP-1	19.5 - 21.5	41.6	24.1	8.4%
TP-3	15.5 - 20	27.1	9.6	5.6%
TP-9	18.5 - 20	42.4	19.8	11.9%
TP-11	18.5 - 20	45.8	26.2	11.2%

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CONSULTING ENGINEERS / LAND SURVEYORS
2150 Hwy 6 & 50, Grand Junction, CO 81505 • 303 242 5202

PS 2/4/88

WYL 2/5/88

January 15, 1988

Mr. Partha Sircar
Morrison-Knudsen Engineers, Inc.
180 Howard Street
San Francisco, CA 94105

RE: Subcontract GRJ-87-02, SCN.04 & SCN.06

Gentlemen:

The following tests have been completed under SCN.04 Item 6 and SCN.06 Item 002:

Sample I.D.	Test
TP 2 @ 18-18.7	Multiple Stage saturated CU triaxial test with pore pressure measurements.
TP 4 @ 19-20	Saturated UU triaxial test with pore pressure measurements.
TP 6 @ 17-17.7	Saturated UU triaxial test with pore pressure measurements.
TP 2 @ 18-18.5	Saturated, timed swell/consolidation test with rebound.
TP 4 @ 20-20.7	Saturated, timed swell/consolidation test with rebound.
TP 4 @ 0-2.5	Saturated permeability tests.
Large scale Sieve No. 1 1" minus fraction with 5% sodium Bentonite:	Saturated permeability test.

The following data is enclosed: MKE DOCUMENT NO. 5025-GRJ-C-09-01577-00

A. Triaxial Tests:
   1. Test data sheets
   2. Strain/pore pressure curves
   3. Strain/deviator stress curves
   4. Mohr curves

B. Consolidation Tests:
   1. Test data sheets
   2. Consolidation/load curves
   3. Time - consolidation log plots
   4. Time - consolidation square root plots
   5. Time - rebound log plots
   6. Time - rebound square root plots

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Additional test data is as follows:

A. Triaxial Tests

Sample	Initial Moisture (%)	Final Moisture (%)	Initial Dry Unit Weight (pcf)	Saturated Pore Pressure Factor (B)	Permeability (cm/sec)
TP 2 @ 18-18.7	4.8	18.9	101.3	0.97	1×10^{-6}
TP 4 @ 19-20	6.7	23.3	101.7	1.00	2×10^{-7}
TP 6 @ 17-17.7	7.9	22.6	117.9	0.98	1.8×10^{-8}

B. Permeability Tests:

Sample	Initial Moisture (%)	Final Moisture (%)	Head (Ft.)	Dia. (in)	Load (pcf)	(1) Load (%)	Initial Dry Unit Weight (pcf)	Permeability (cm/sec)
LSS#1/ w/ Bentonite	13.2	17.3	3.5	20	300	0.3	112.3	9.4×10^{-7}
TP 4 @ 0-2.5	15.5	-	5.0	8	300	-0.2 (swell)	105.8	2.05×10^{-8}

Additional notes on the test procedures and results are presented below:

A. Triaxial Tests:

1. A pore pressure factor B of 97% or greater was considered adequate to indicate saturation.

2. The shale samples were too disturbed and crumbly to obtain a triaxial sample that would give meaningful results.

3. The 2 UU tests were performed with confining pressures of 7 KSF rather than 5 KSF as originally requested. This was necessary to provide a higher degree of saturation.

4. It was necessary to maintain a small consolidation pressure of about 300 psf on sample TP 4 @ 19-20 in order to keep the membrane sealed to the caps.

5. In all but one case, the failure defined by the maximum deviator stress was practically the same as that using the maximum principal stress ratio. The stress strain curves are based on maximum deviator stress. However, for sample TP 6 @ 17-17.7, there was a big difference between these two failure criteria. This difference is shown on the respective stress/strain curve.

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WYL 2/5/88

6. The effective stress Mohr circles for the multiple stage test on sample TP 2 @ 18-18.7 were fairly consistent. However, the total stress circles were not making it difficult to determine the total stress envelope. We do not have an explanation for this occurrence.

B. Consolidation Tests:

1. In addition to the load increments requested, we added a seating load of 100 psf. The sample was seated, loaded to 250 psf and then saturated. This provided an indication of how the soil responds to moisture influence. It was seen that both samples swelled a significant amount when wetted.

2. The sample TP 9 @ 20-20.7 was highly disturbed and crumbly shale. We do not know how representative the results are of field conditions.

C. Permeability Test:

1. In order to achieve full saturation of permeability samples, the procedure was as follows:

a. The equipment used was that described in U.S. Bureau of Reclamation Standards E-13 and E-14. The larger mold (20 inch dia.) was used for the large scale sieve sample. The constant head tanks were used in order to maintain a slightly subatmospheric pressure in the head tank to help keep the water de-aired. The size of tanks used were so that temperature variation effects were minimized. In addition to use of de-aired water, the head tank was heated to keep the water at a slightly higher temperature than the soil in the permeameter, as recommended in ASTM D-2434.

b. In order to assure full saturation, a partial vacuum (about 20 inches of mercury) was applied to the top of the specimen while water was running into the bottom of the sample. This was maintained until the flow stabilized and at least one full exchange of pore water had occurred. Saturation time was 2 to 3 weeks.

c. After saturation, a confining load and a hydraulic gradient was applied to the specimen.

If you have any questions, please call.

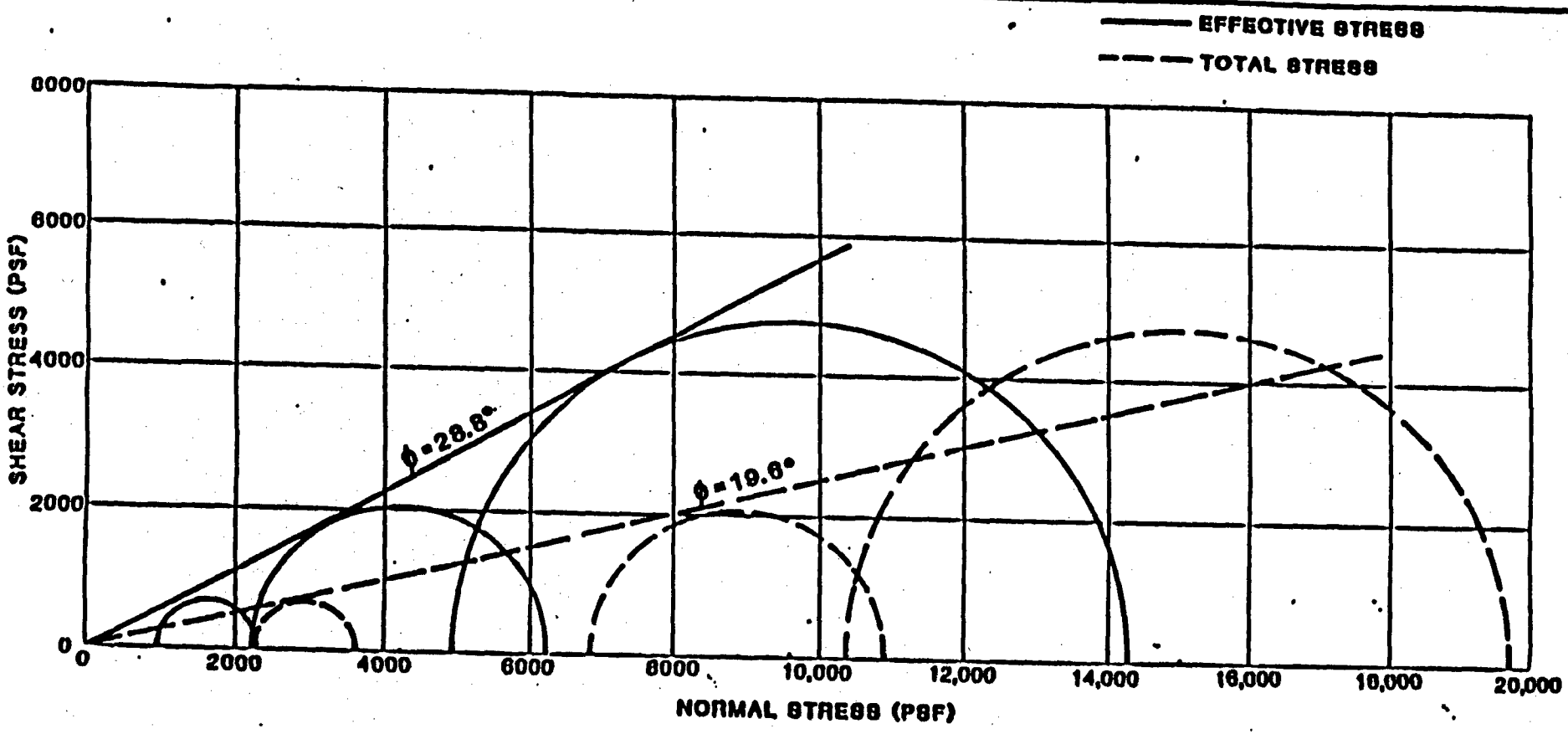
Very truly yours,

WESTERN ENGINEERS, INC.

Bruce D. Marvin, P.E.
Vice President

cc: R.M. Harrison
MK-Ferguson
P.O. Box 9136
Albuquerque, NM 87119

MKE DOCUMENT NO. 5025-GRJ-C-09-01601-00



CHENEY RESERVOIR

TP 2 AT 18-18.7

MOHR ENVELOPE

TRIAXIAL TEST DATA

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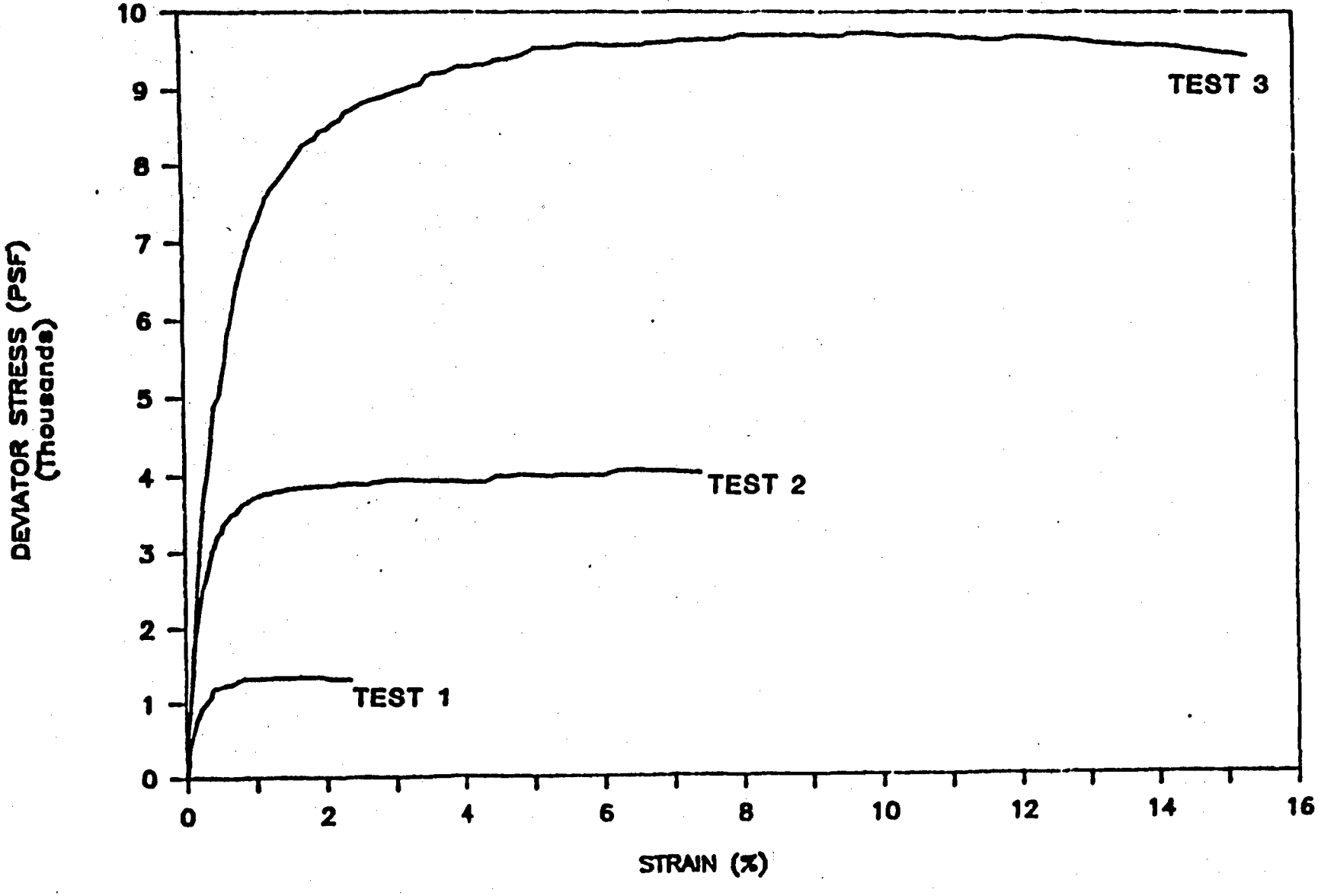
CHENEY RESERVOIR

TP 2@18–18.7



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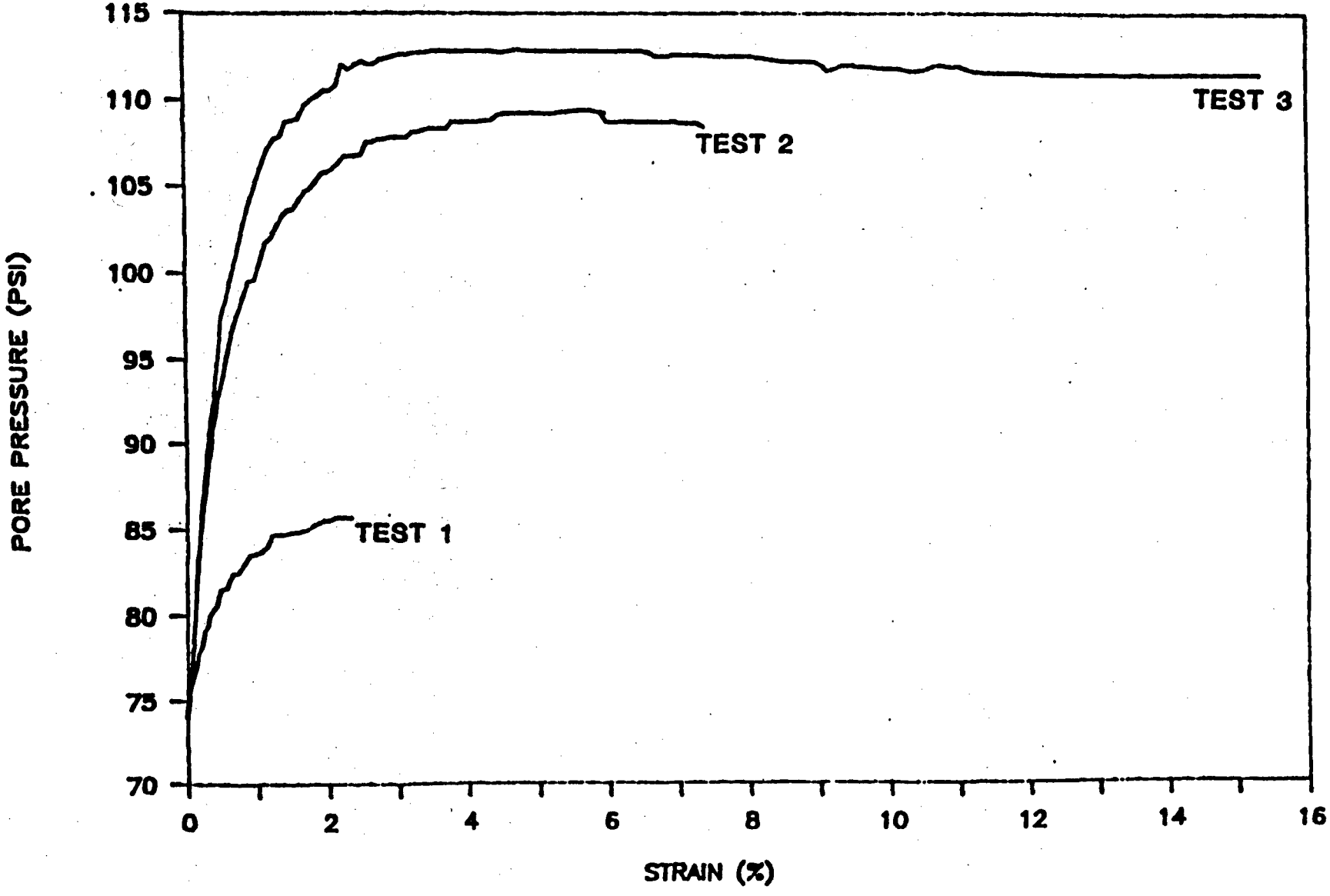
CHENEY RESERVOIR

TP 2@18–18.7



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TP 28 10-10.7	BY:	DATE:	

PS 2/9/88
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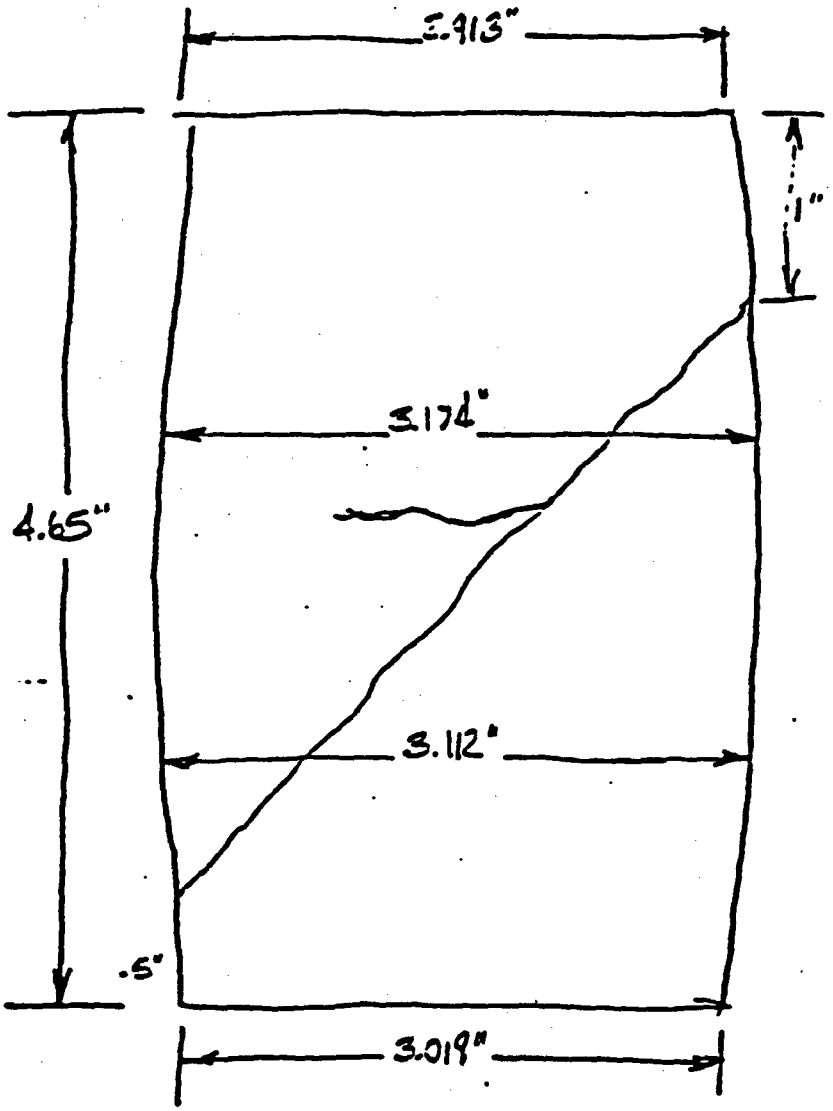

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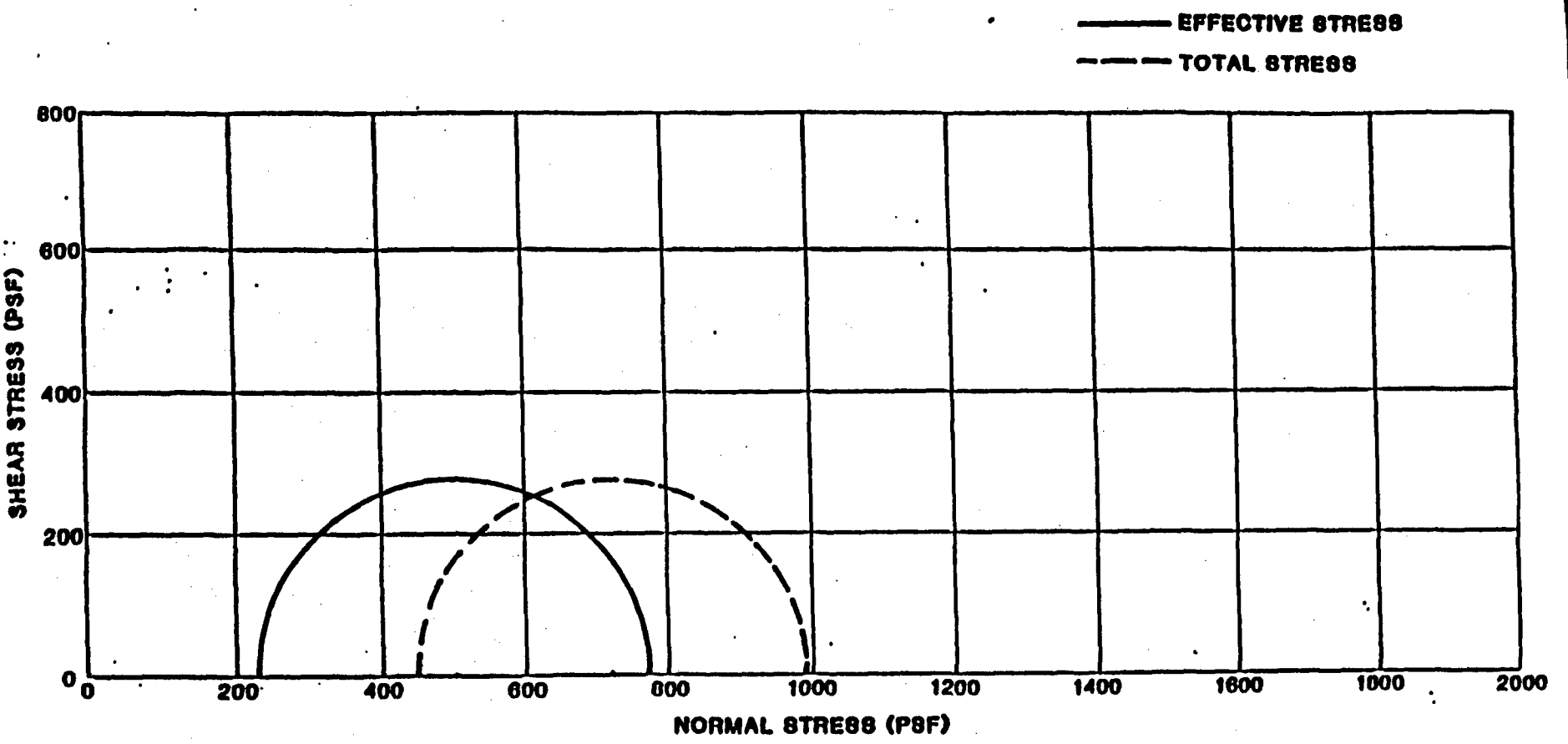

CHENEY RESERVOIR

TP 4 AT 19-20

MOHR CIRCLES

TRIAXIAL TEST DATA

WYL 2/5/88

PS 2/4/88

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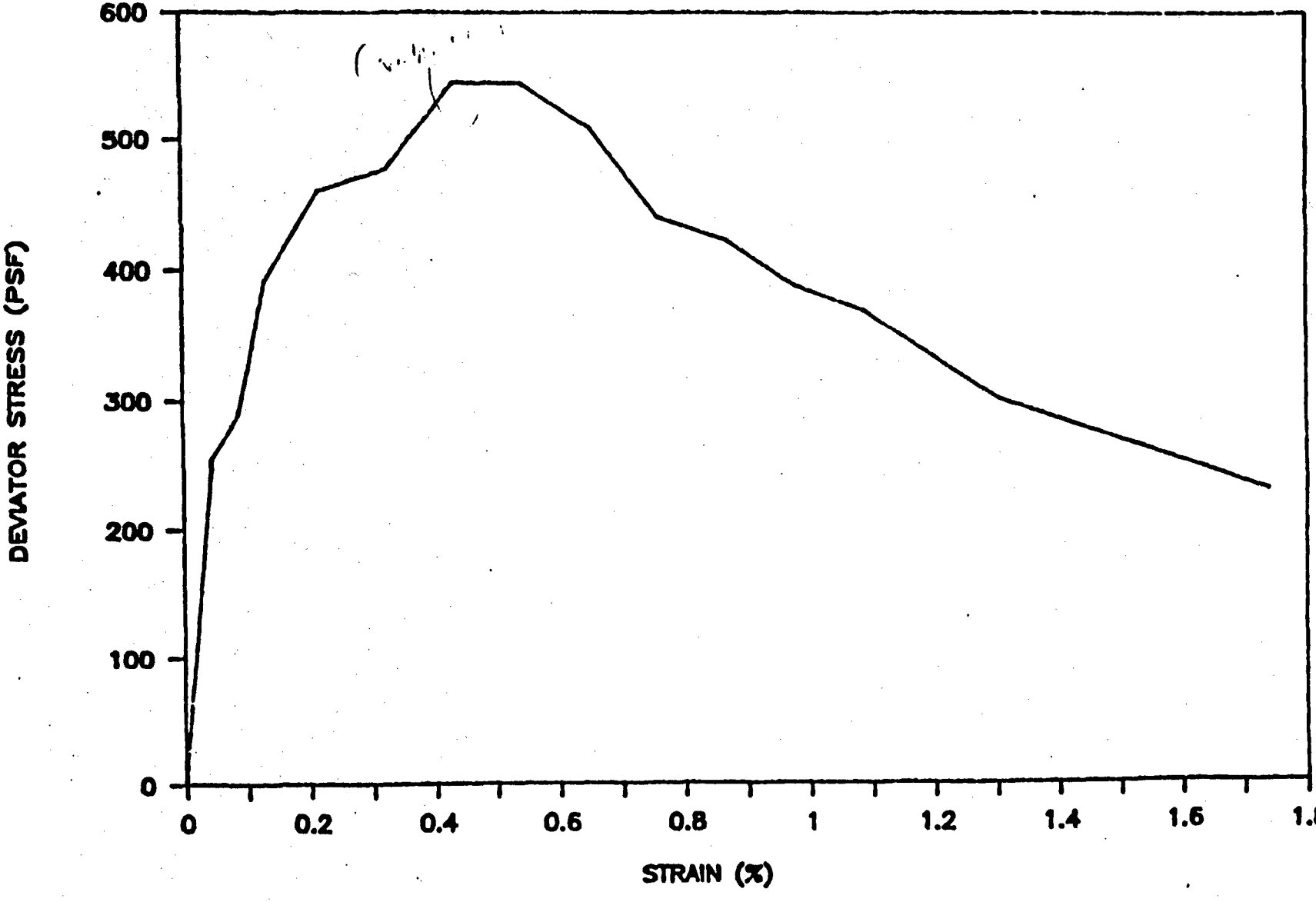
CHENEY RESERVOIR

TP 4 ● 19–20



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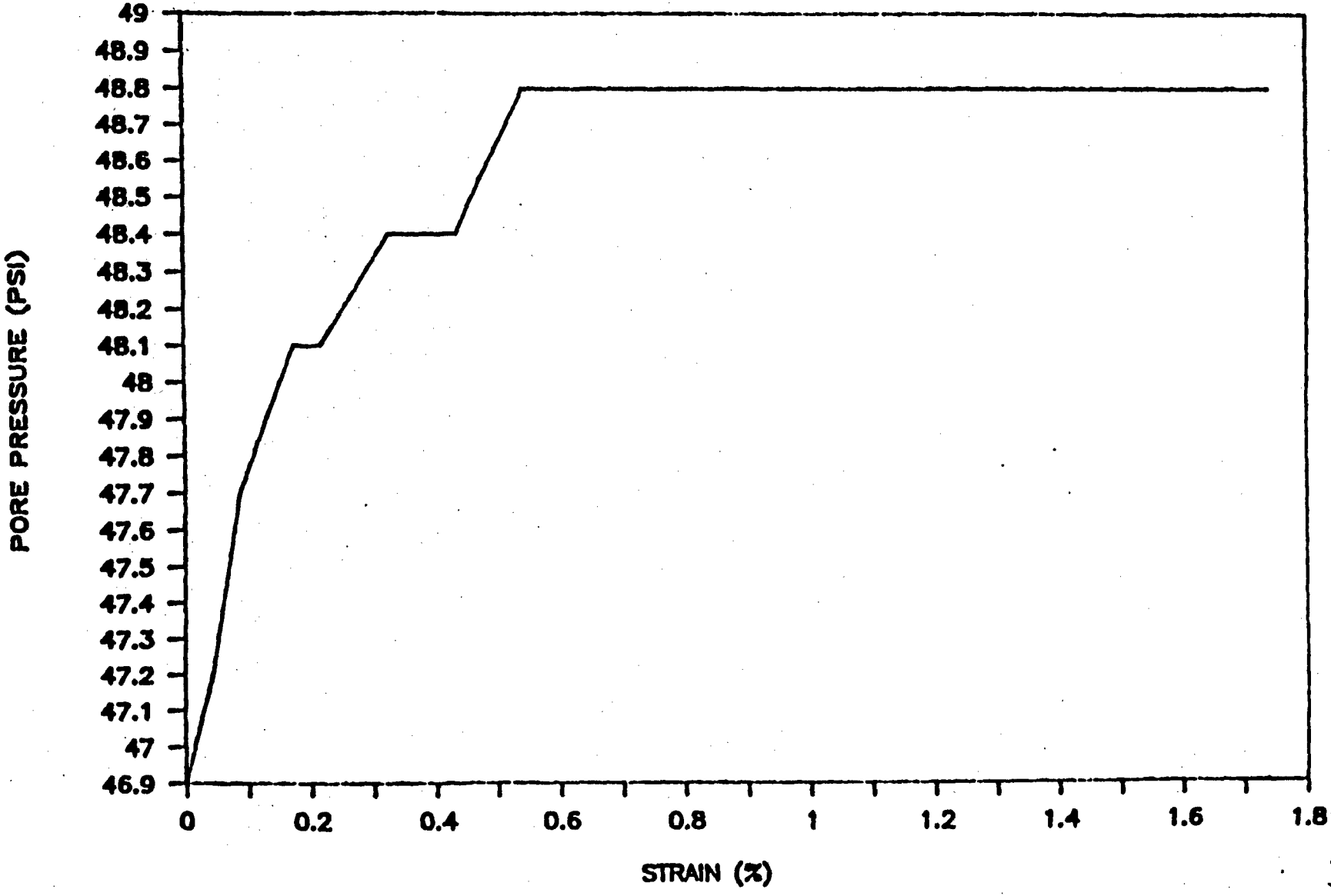
1 of 4
Fig.

CHENEY RESERVOIR

TP 4 @ 19–20



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TP 4 @ 19-20	BY:	DATE:	

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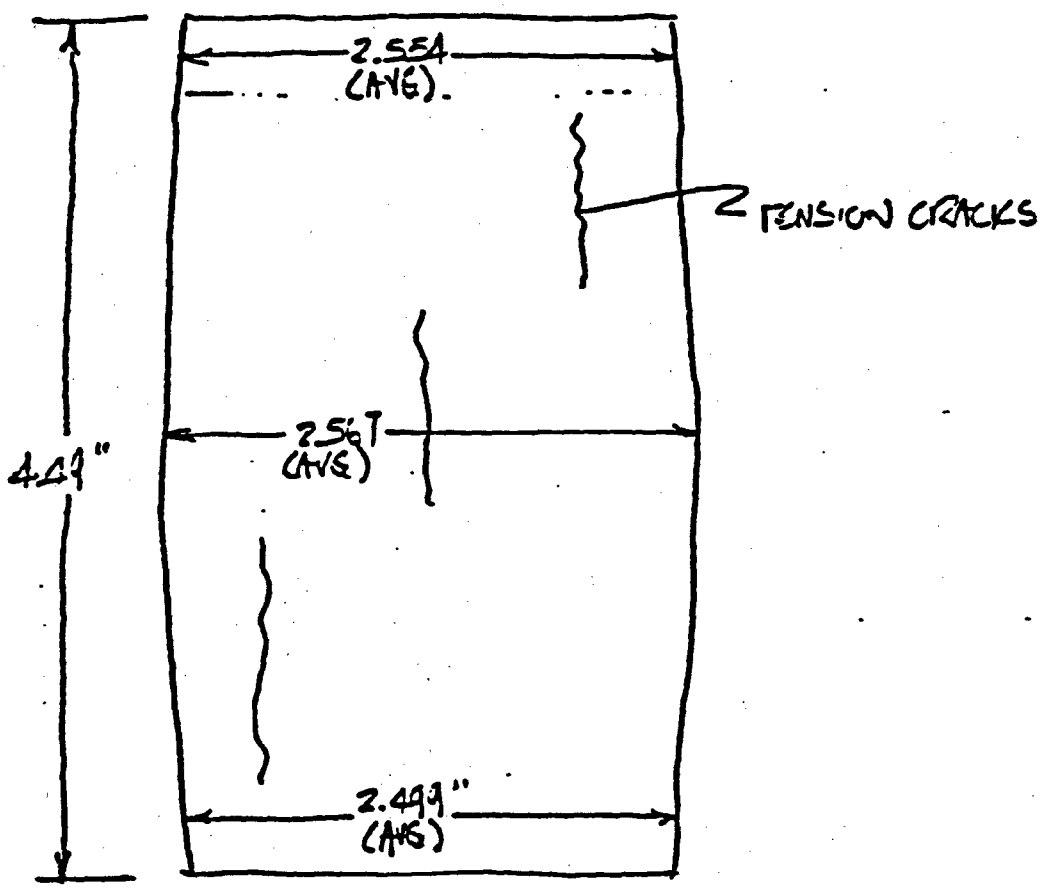

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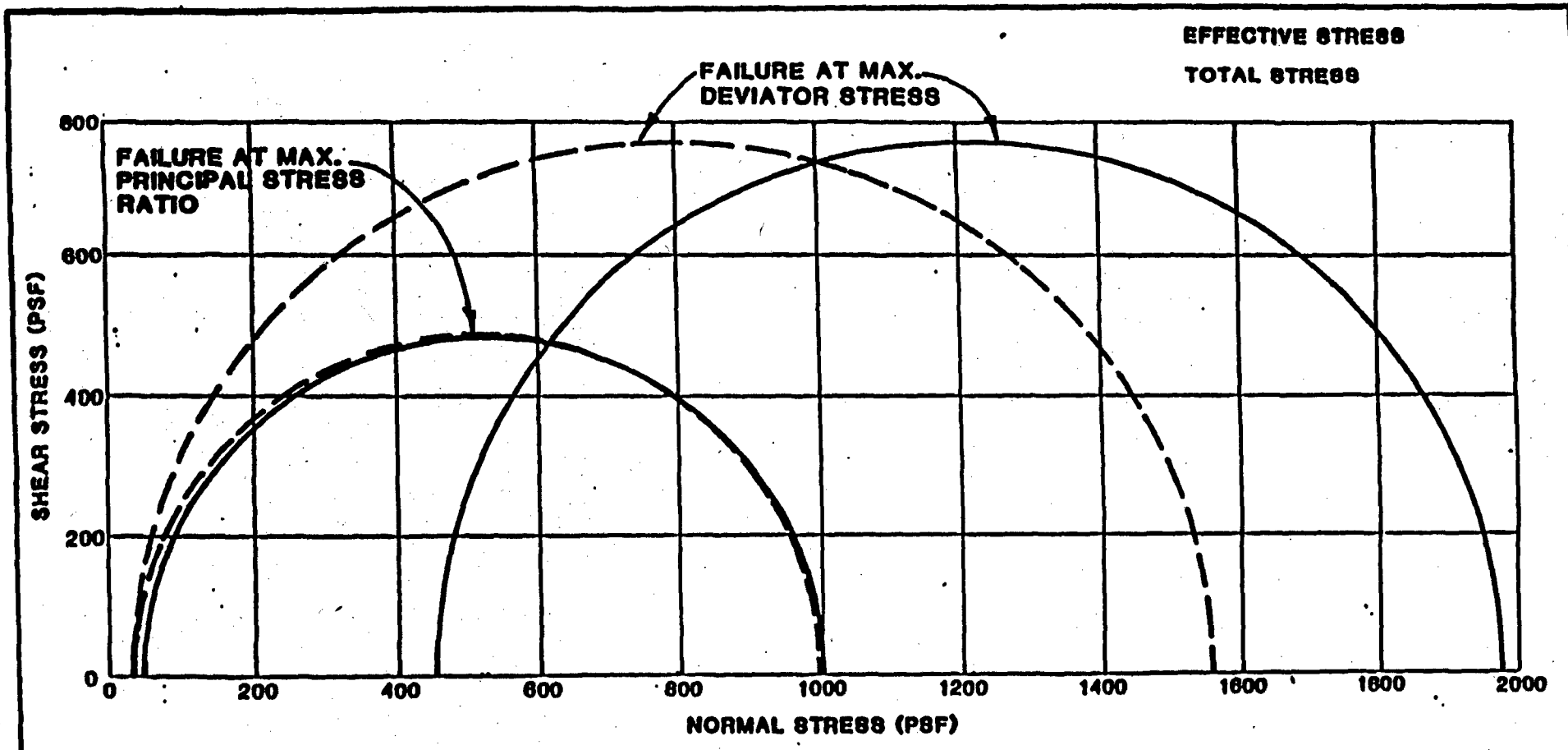

CHENEY RESERVOIR

TP 6 AT 17-17.7

MOHR CIRCLES

TRIAXIAL TEST DATA

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WJL 2/5/88

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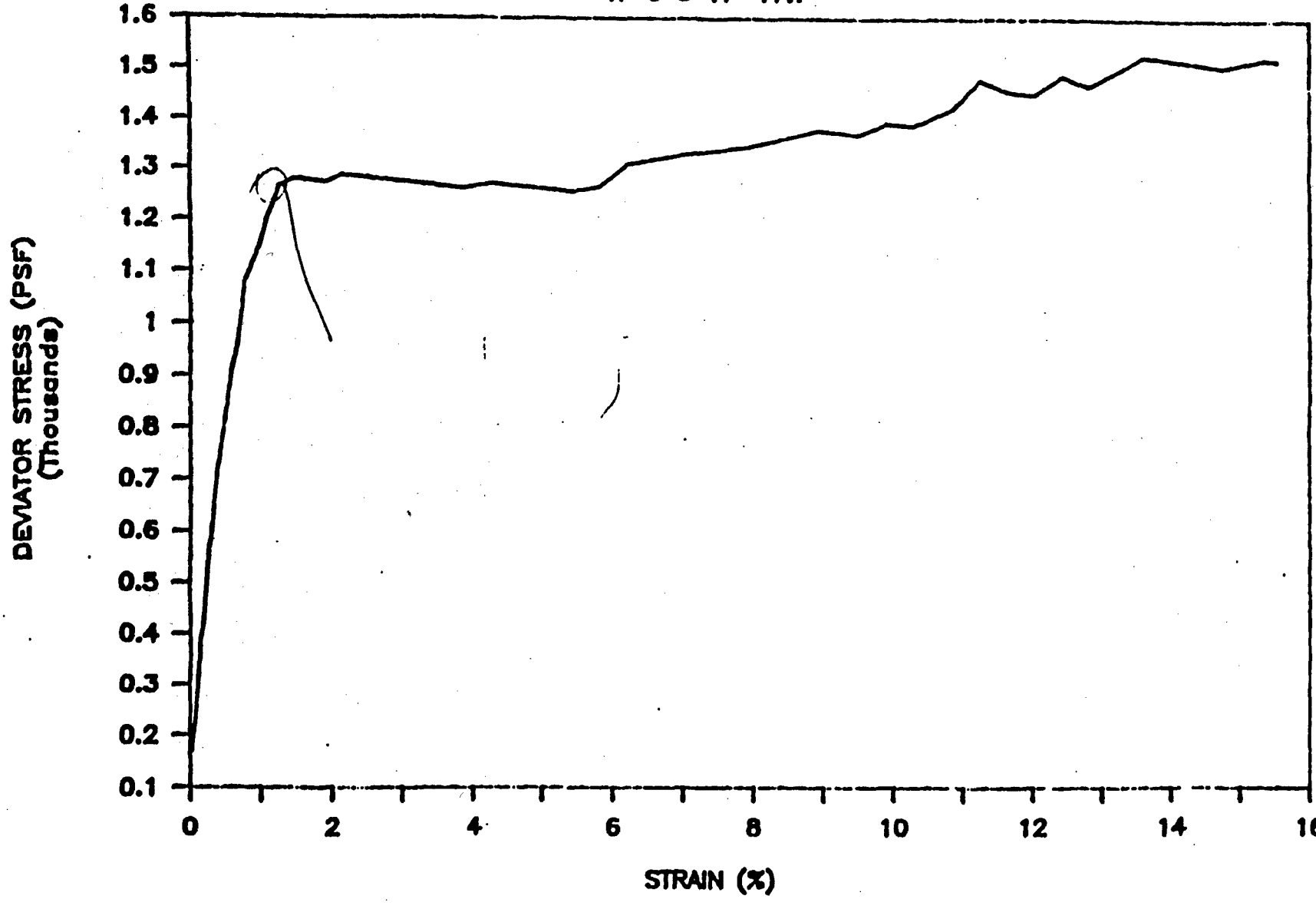
CHENEY RESERVOIR

TP 6 @ 17–17.7



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WYL 2/5/88

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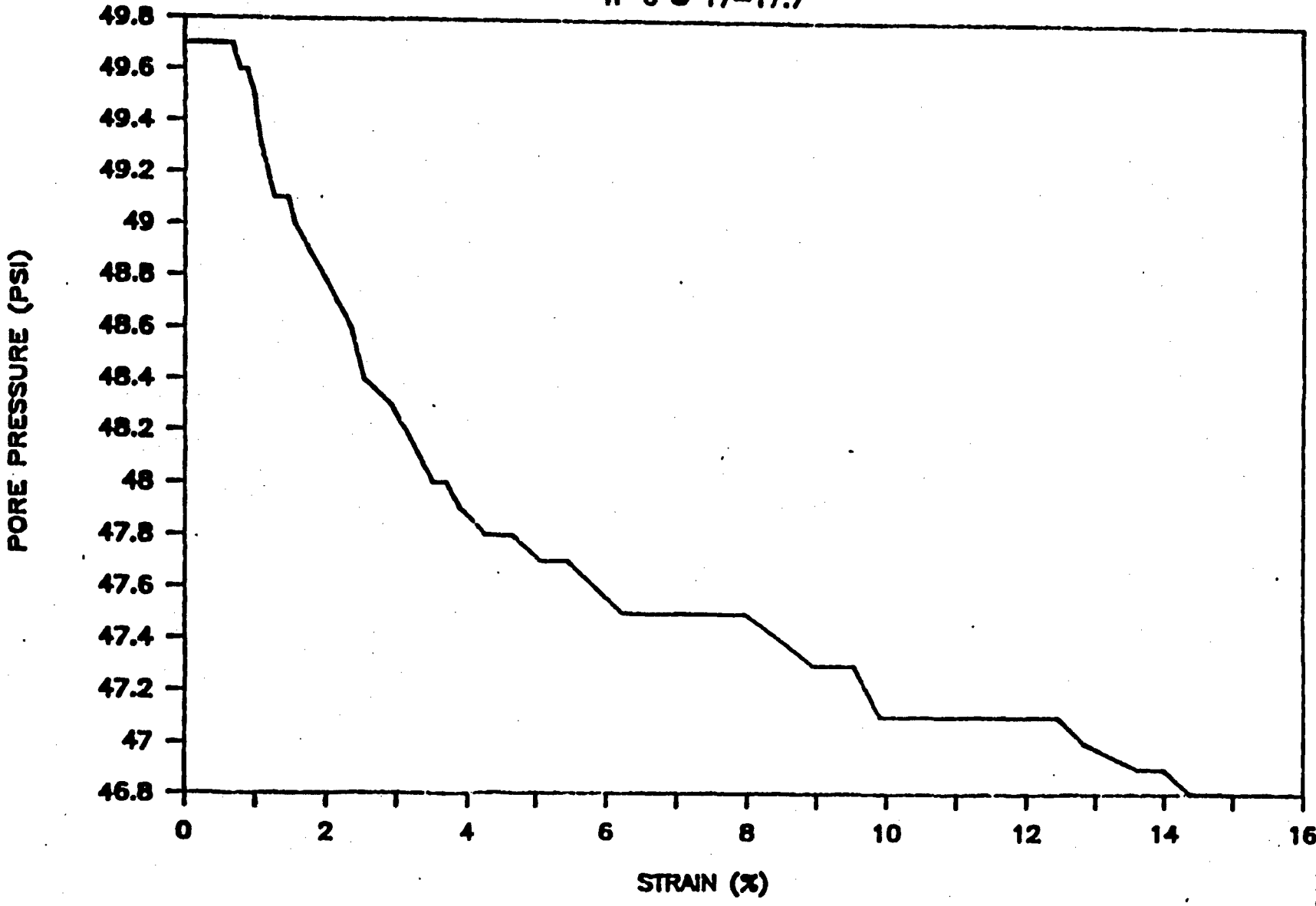
CHENEY RESERVOIR

TP 6 @ 17–17.7



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WNL 2/5/88

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TP 6 @ 17-17.7	BY:	DATE:	

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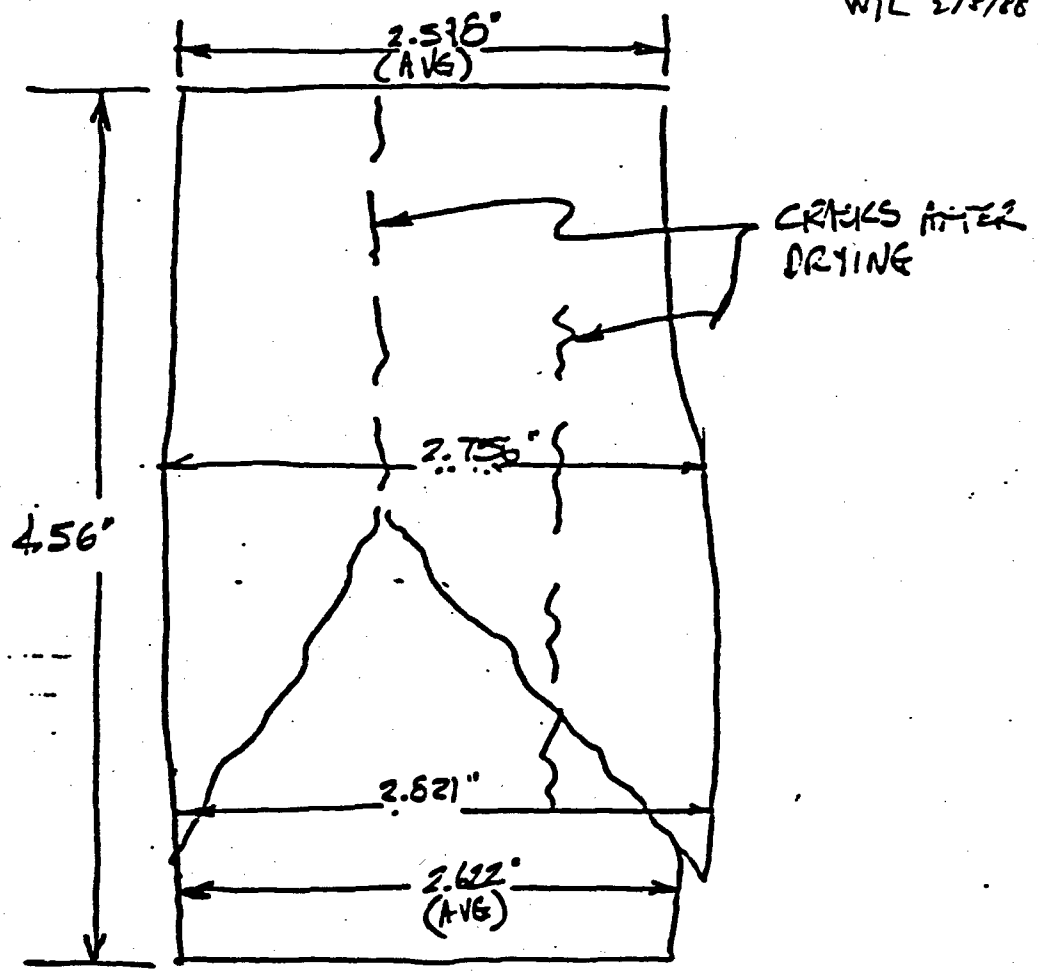
WYL 2/5/88



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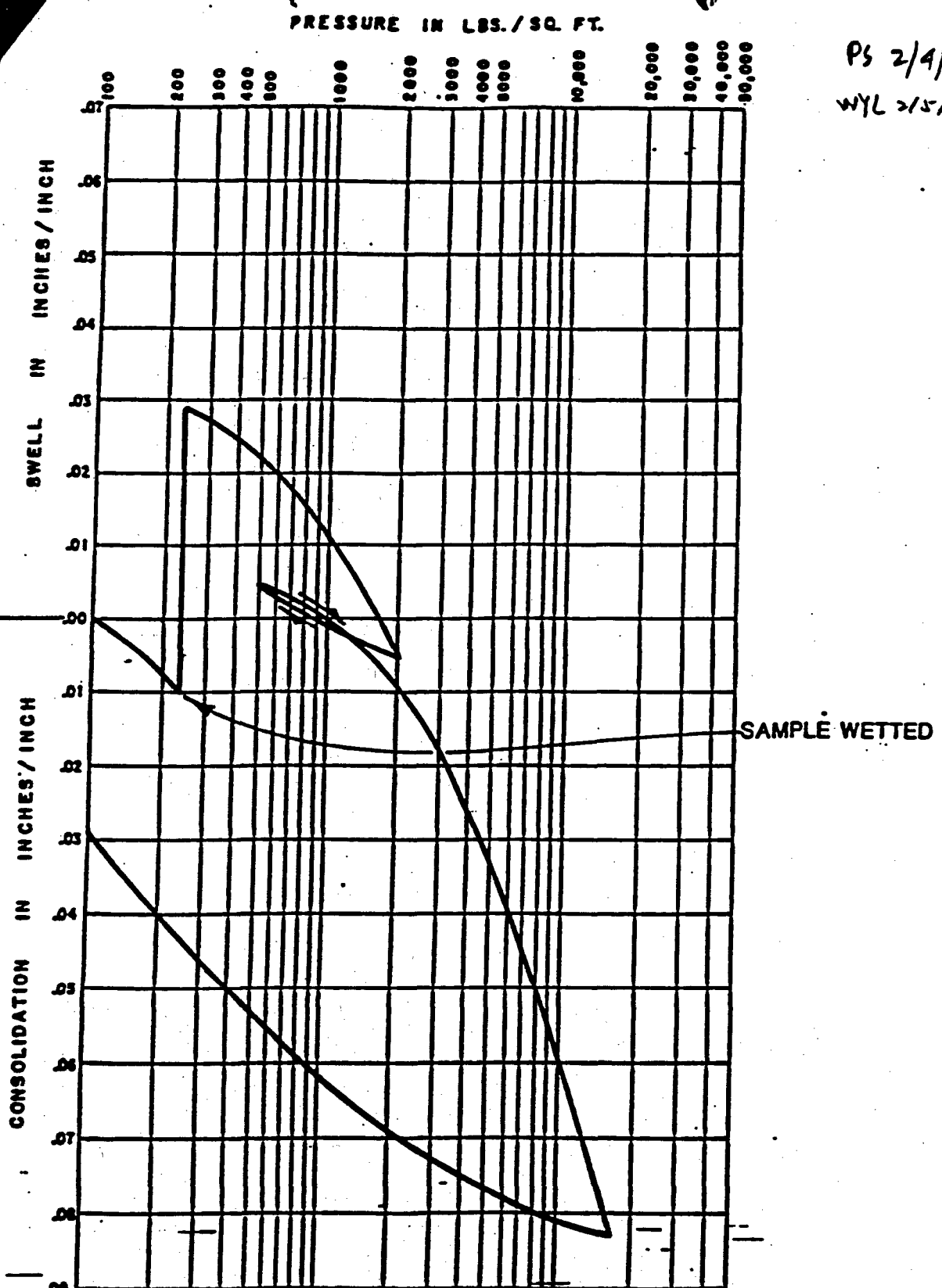

BORING NO. AND DEPTH	SOIL TYPE	MOISTURE CONTENT BEFORE	MOISTURE CONTENT AFTER	DRY DENSITY IN LBS/CU. FT.
TP 2 at 18-18.5	CLAY-SILT	4.7	19.9	107.9

CHENEY RESERVOIR

CONSOLIDATION DATA

PLATE

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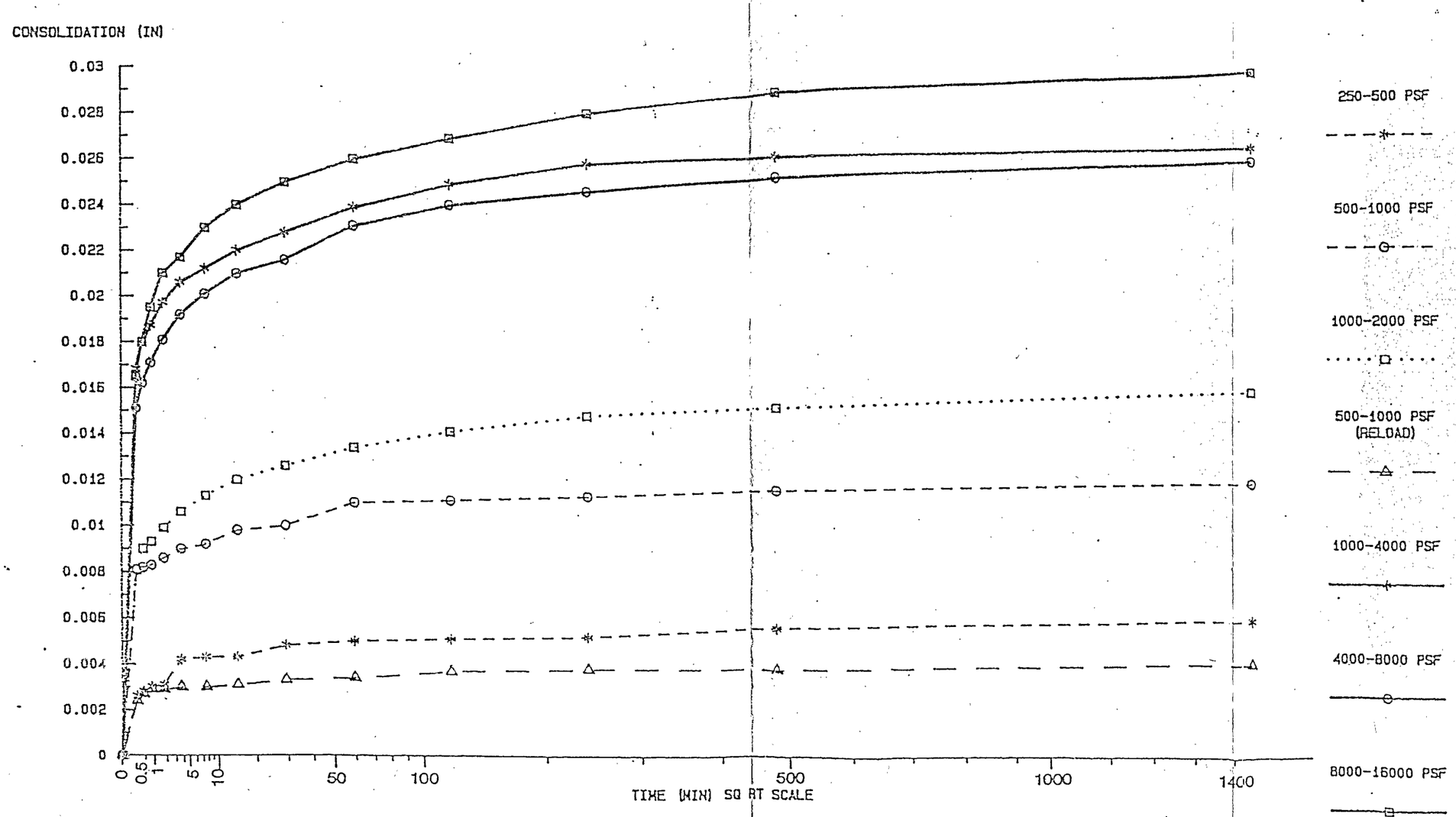
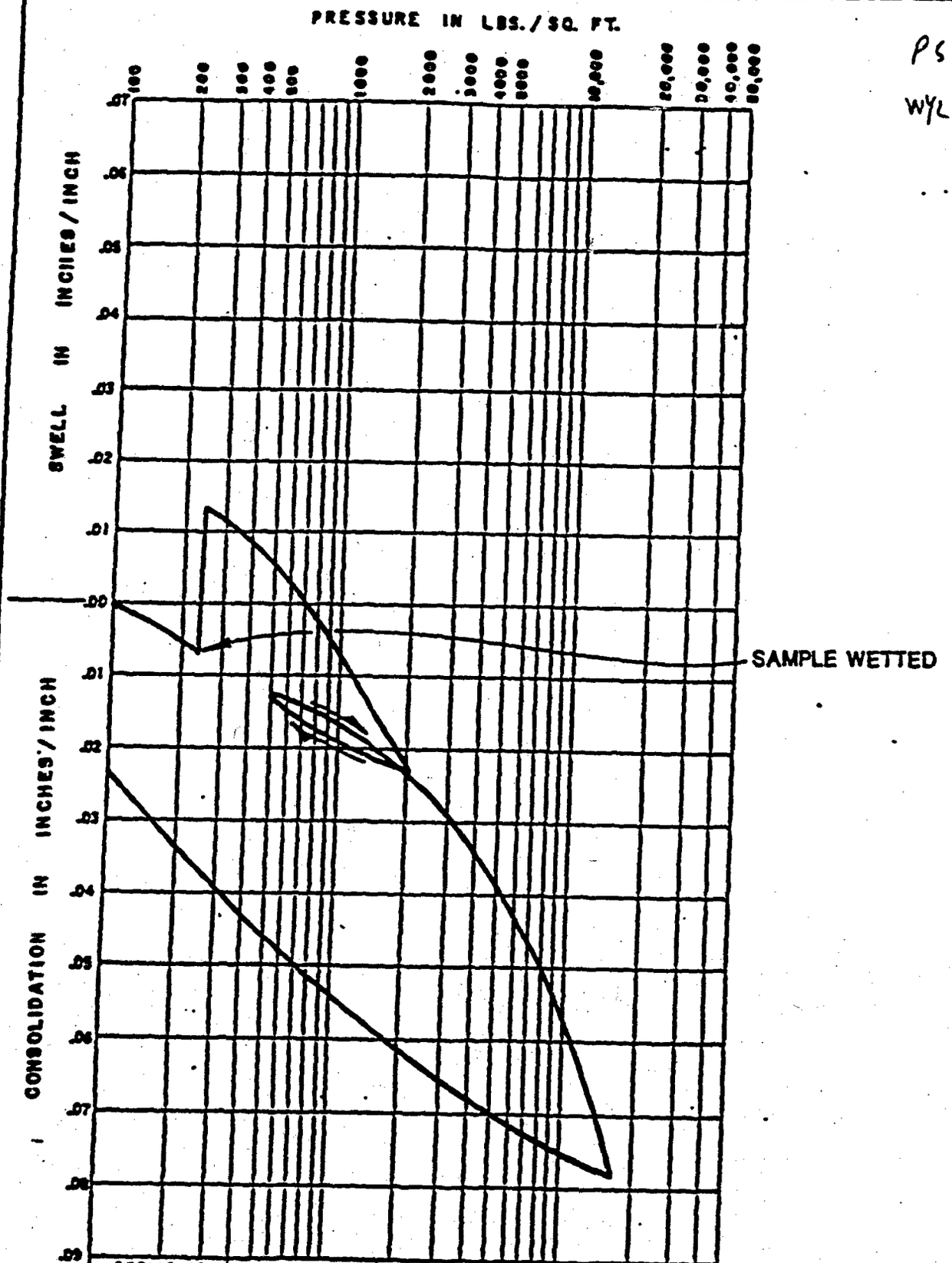
PS 2/4/88
WYL 2/5/88

CHENEY RESERVOIR
TP 2 @ 18-18.5
CONSOLIDATION DATA



PS 2/4/88
WYL 2/5/88



BORING NO. AND DEPTH	SOIL TYPE	MOISTURE CONTENT BEFORE	MOISTURE CONTENT AFTER	DRY DENSITY IN LBS/CU. FT.
TP 9 at 20-20.7	SHALE	11.9	20.7	106.7

CHENEY RESERVOIR
CONSOLIDATION DATA

PLATE

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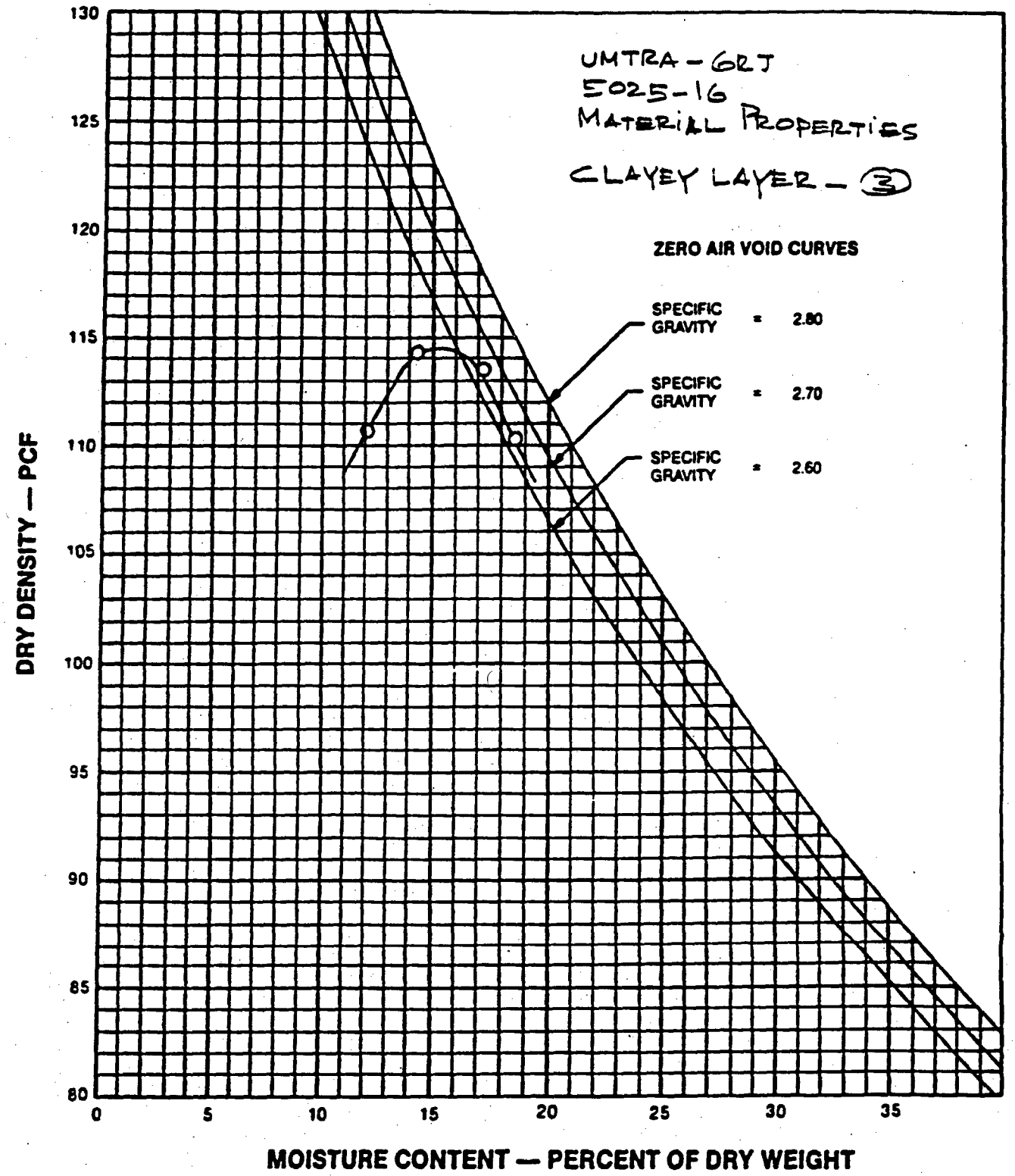
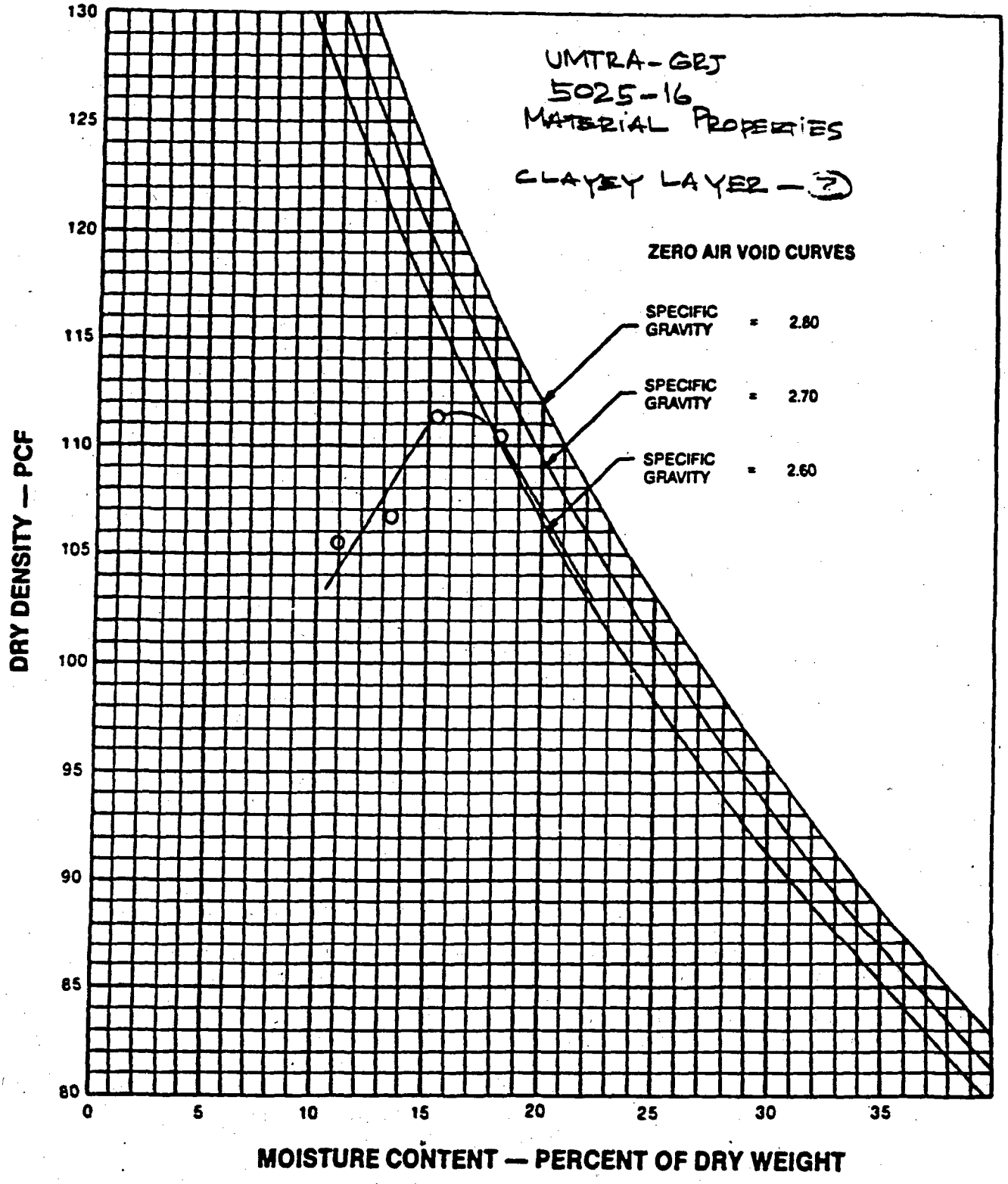
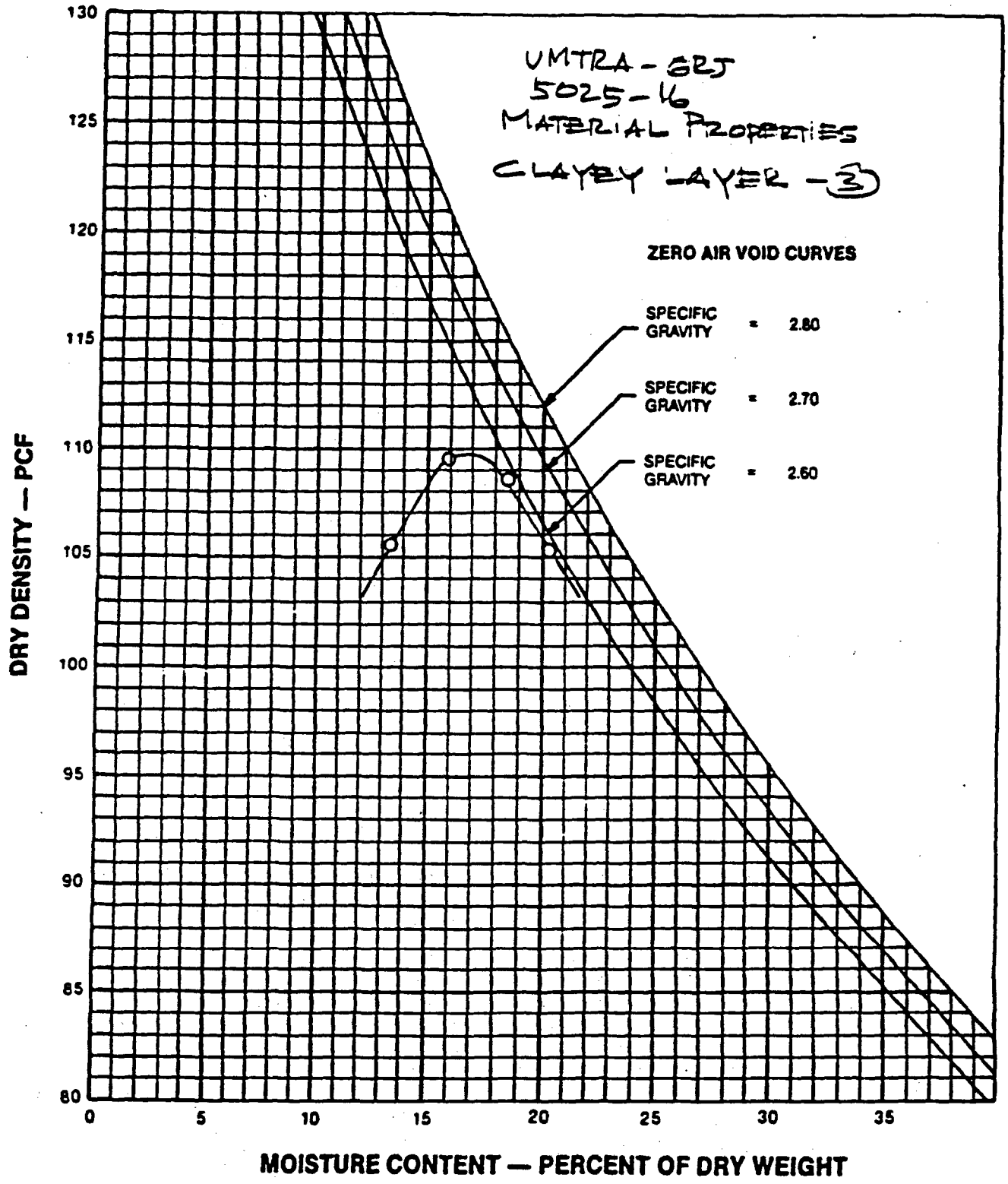


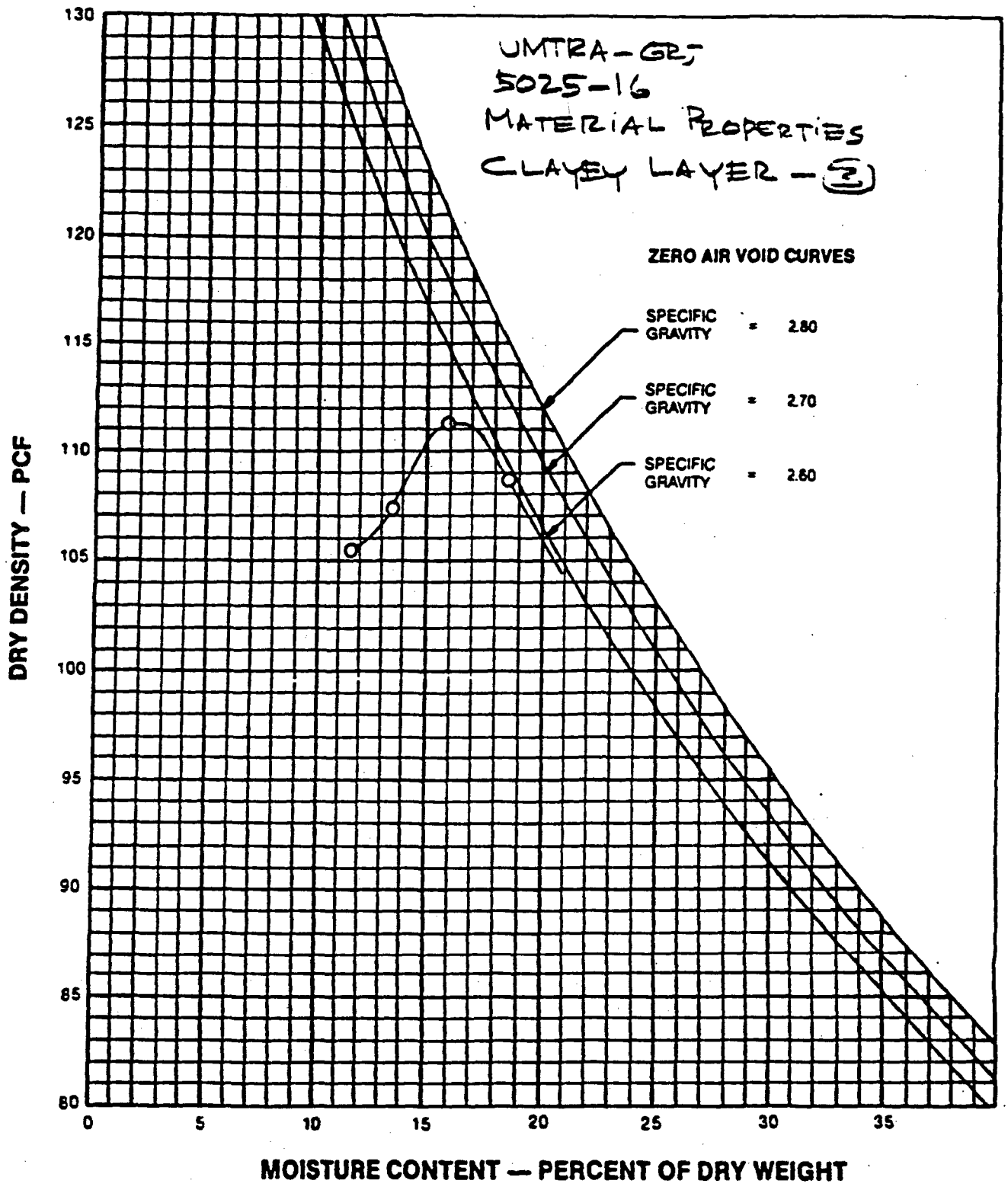
LOCATION:	MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: TP 123 DEPTH: 14-23 SAMPLE NO.: 10	
SOIL DESCRIPTION: SC; Clayey sand with gravel	*Chen-Northern, Inc.*
MAX. DRY DENSITY: 114.3 PCF OPT. MOIST. CONTENT: 15.5 %	PROCEDURE: ASTM D698-78, Meth. "A"
LIQUID LIMIT: 33 PLASTICITY INDEX: 18	JOB NO.: 1 126 90 — FIG. NO.
GRAVEL: 17 % SAND: 34 % SILT AND CLAY (-200): 49 %	DATE: OCTOBER 31, 1989 — 9



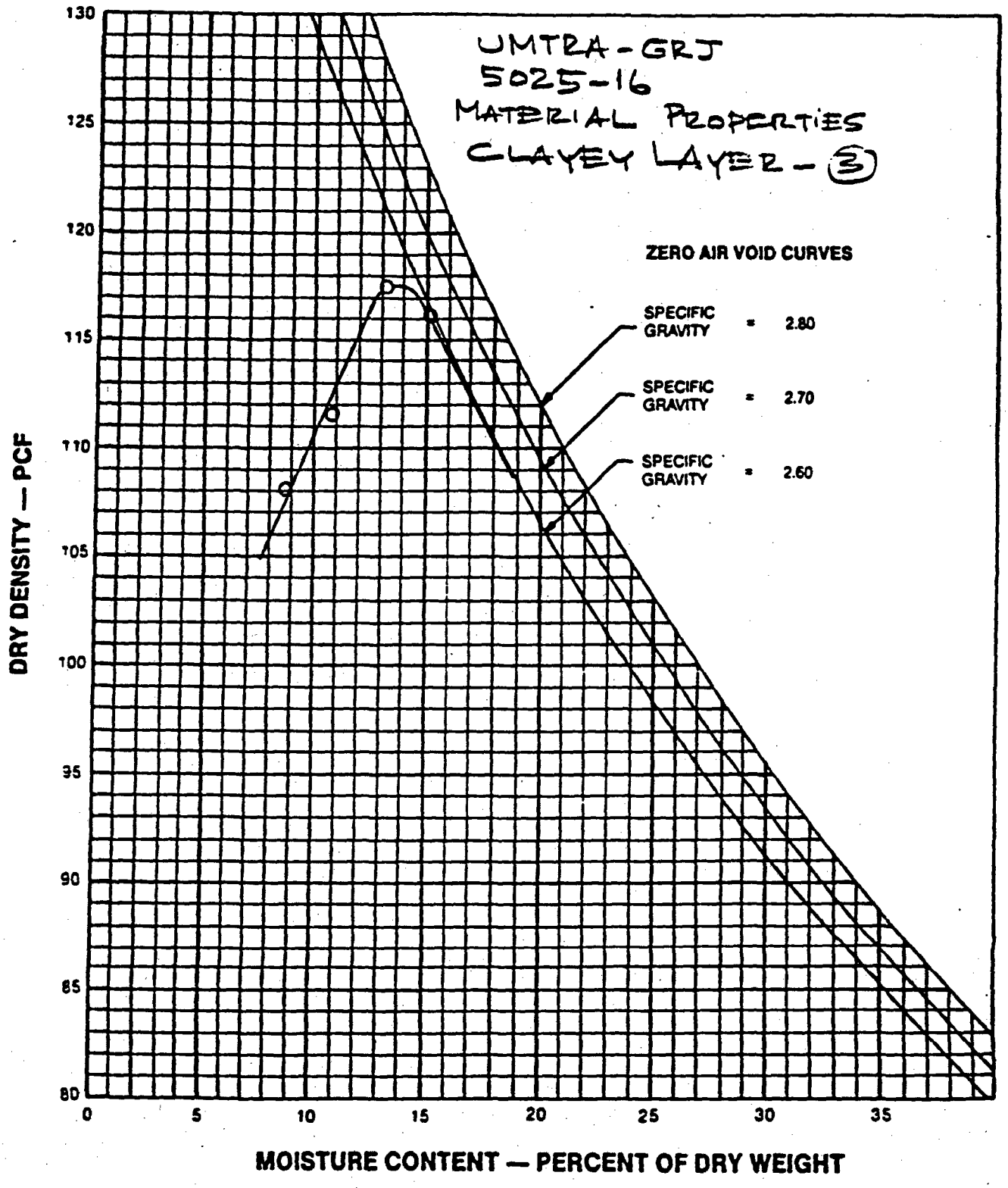
LOCATION:	MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: TP 124 DEPTH: 18.5-23 SAMPLE NO.: 11	
SOIL DESCRIPTION: CL; Lean clay	Chen-Northern, Inc.
MAX. DRY DENSITY: 112.2 PCF OPT. MOIST. CONTENT: 16.1 %	PROCEDURE: ASTM D698-78, Meth."A"
LIQUID LIMIT: 40 PLASTICITY INDEX: 24	JOB NO.: 1 126 90 — FIG. NO.
GRAVEL: 3 % SAND: 9 % SILT AND CLAY (-200): 88 %	DATE: OCTOBER 31, 1989 — 10



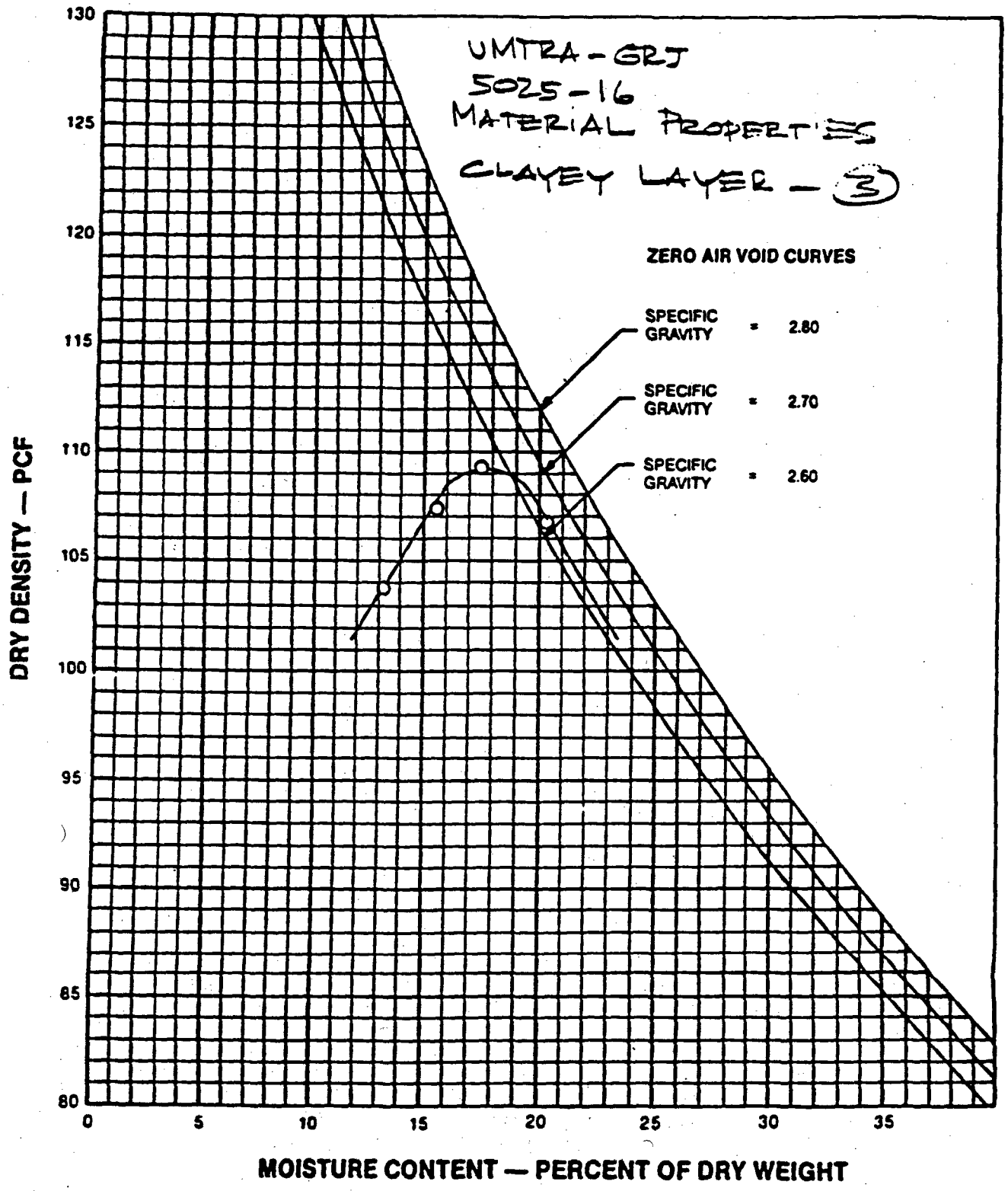
LOCATION:		MOISTURE—DENSITY RELATIONSHIPS	
HOLE NO: TP 125 DEPTH: 18-21 SAMPLE NO.: 12			
SOIL DESCRIPTION: CL; Lean clay		Chen-Northern, Inc.	
MAX. DRY DENSITY: 109.7 PCF	OPT. MOIST. CONTENT: 17.1 %	PROCEDURE: ASTM D698-78, Meth."A"	
LIQUID LIMIT: 40	PLASTICITY INDEX: 25	JOB NO.: 1 126 90	FIG. NO.
GRAVEL: 4 % SAND: 8 %	SILT AND CLAY (-200): 88 %	DATE: NOVEMBER 3, 1989	11



LOCATION:		MOISTURE—DENSITY RELATIONSHIPS	
HOLE NO: TP 126 DEPTH: 14-21 SAMPLE NO.: 14			
OIL DESCRIPTION: CL; lean clay		Chen-Northern, Inc.	
MAX. DRY DENSITY: 110.2 PCF	OPT. MOIST. CONTENT: 16.5 %	PROCEDURE: ASTM D698-78, Meth."A"	
LIQUID LIMIT: 43	PLASTICITY INDEX: 25	JOB NO.: 1 126 90	FIG. NO.
GRAVEL: 0 % SAND: 6 %	SILT AND CLAY (-200): 94 %	DATE: OCTOBER 31, 1989	12

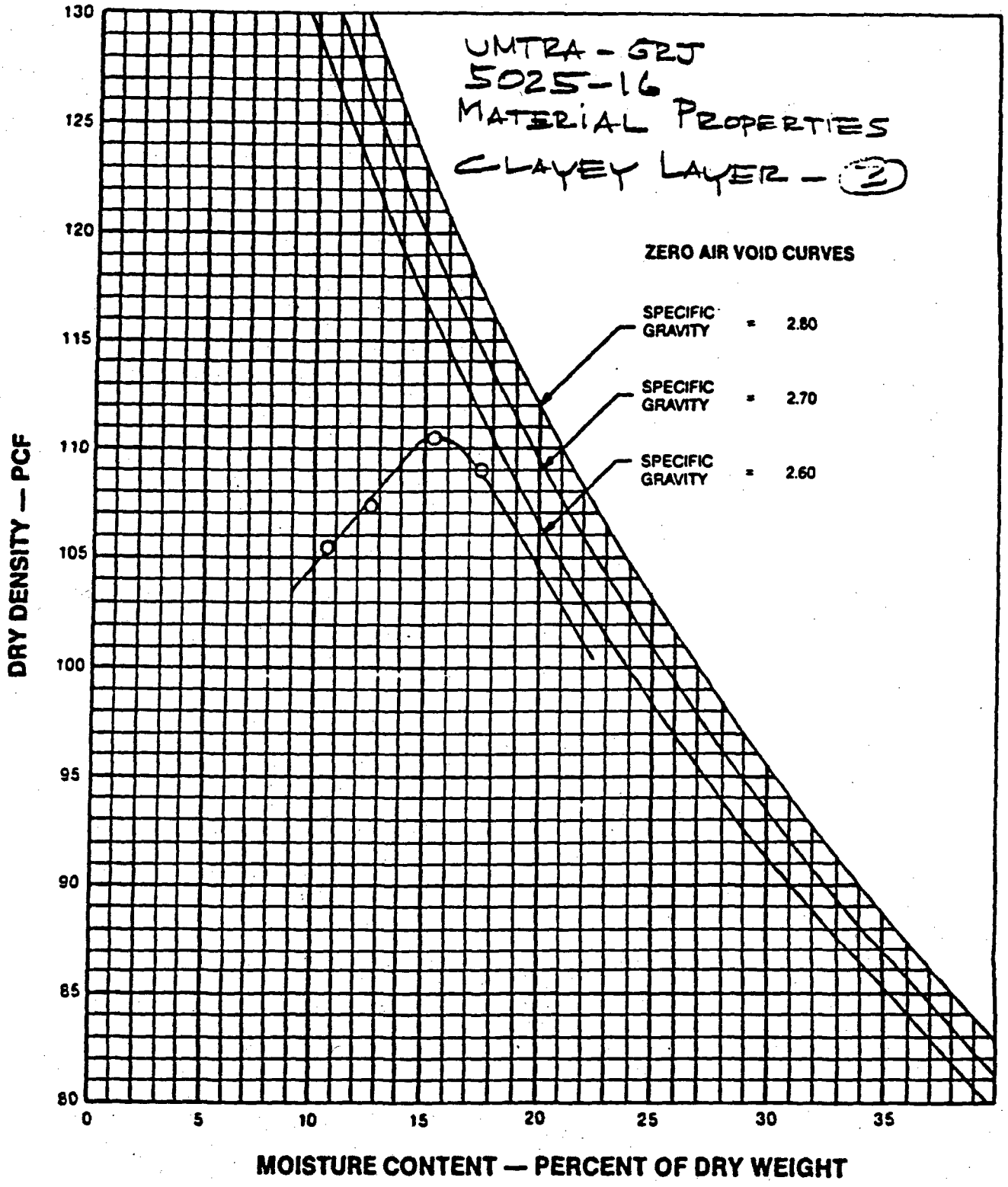


LOCATION:		MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: TP 127 DEPTH: 15-25.5 SAMPLE NO.: 15		
SOIL DESCRIPTION: CL; Sandy lean clay		Chen-Northern, Inc.
MAX. DRY DENSITY: 117.6 PCF OPT. MOIST. CONTENT: 14.0 %		PROCEDURE: ASTM D698-78, Meth."A"
LIQUID LIMIT: 37 PLASTICITY INDEX: 22		JOB NO.: 1 126 90 / FIG. NO.
GRAVEL: 3 % SAND: 30 % SILT AND CLAY (-200): 67 %		DATE: OCTOBER 31, 1989 / 13

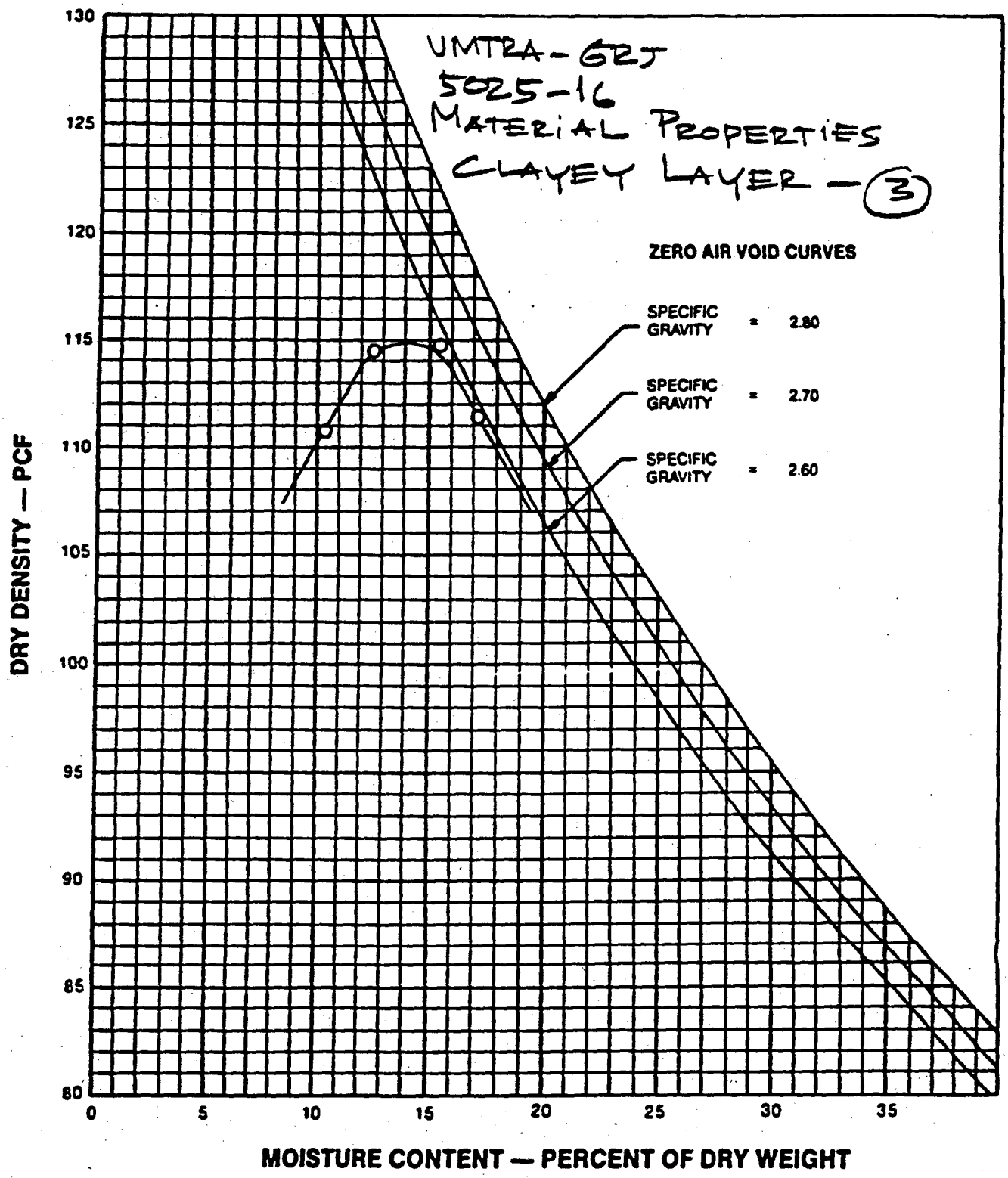


LOCATION:			MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: TP 128	DEPTH: 16-25	SAMPLE NO.: 16	
SOIL DESCRIPTION: CL; Lean clay			Chen-Northern, Inc.
MAX. DRY DENSITY: 109.2 PCF	OPT. MOIST. CONTENT: 12.9 %		PROCEDURE: ASTM D698-78, Meth. "A"
LIQUID LIMIT: 44	PLASTICITY INDEX: 27		JOB NO.: 1 126 90 — FIG. NO.
GRAVEL: 0 %	SAND: 5 %	SILT AND CLAY (-200): 95 %	DATE: OCTOBER 31, 1989 — 14

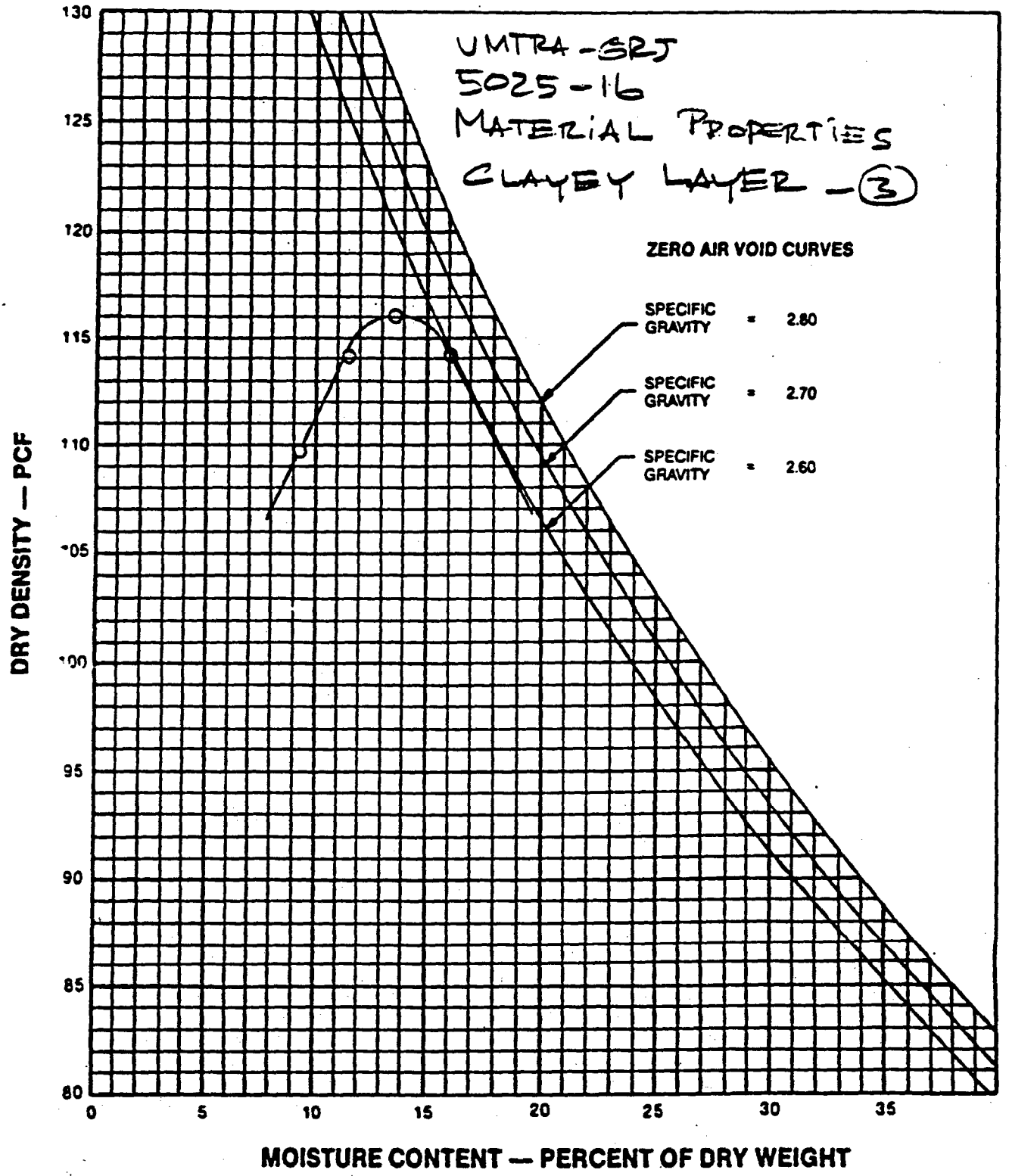
WRT



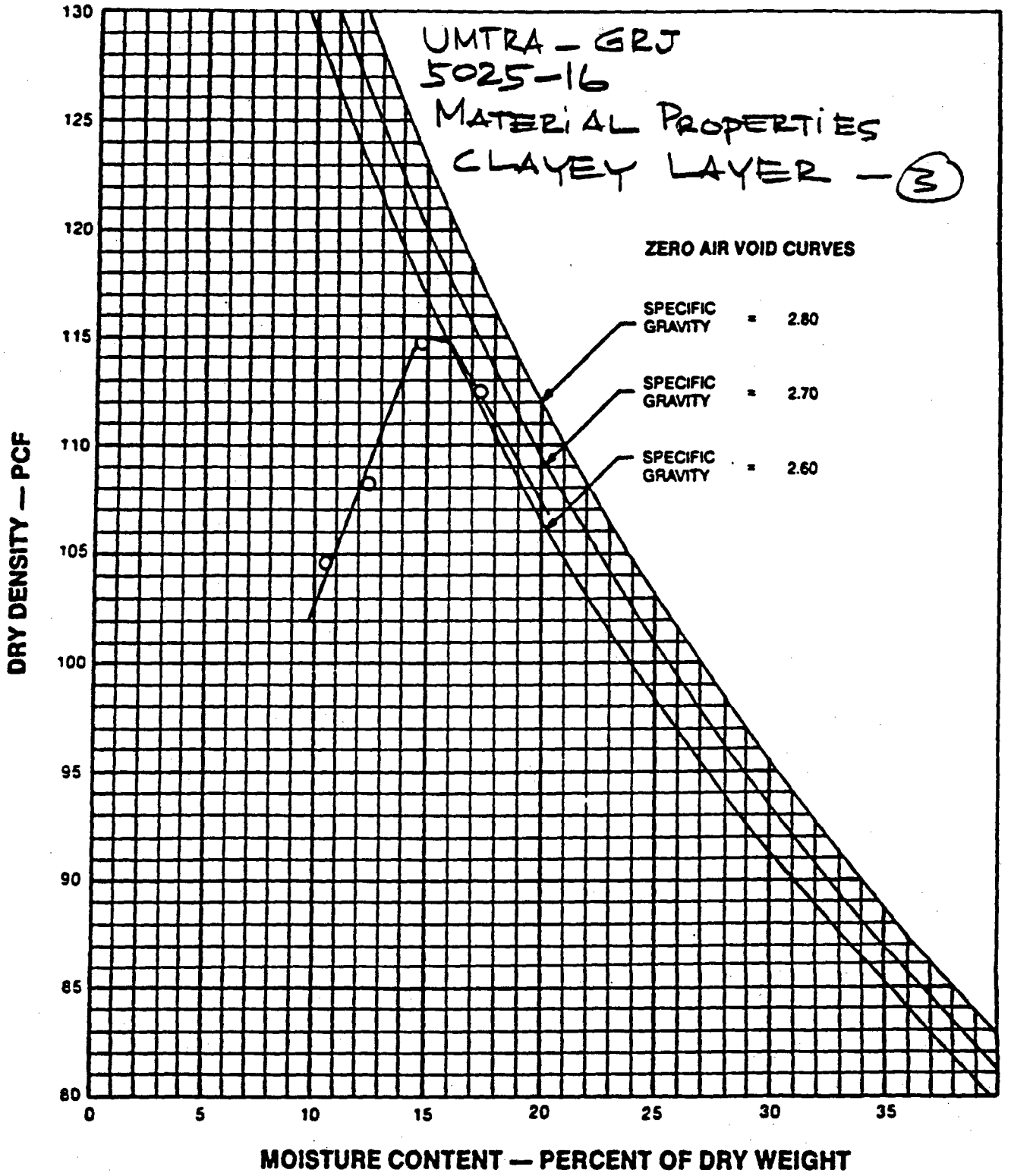
LOCATION:	MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: TP 130 DEPTH: 10.5-14.5 SAMPLE NO.: 17	
SOIL DESCRIPTION: CL; Sandy lean clay	Chen-Northern, Inc.
MAX. DRY DENSITY: 110.6 PCF OPT. MOIST. CONTENT: 16.0 %	PROCEDURE: ASTM D698-78, Meth."A"
LIQUID LIMIT: 33 PLASTICITY INDEX: 17	JOB NO.: 1 126 90 — FIG. NO.
GRAVEL: 0 % SAND: 36 % SILT AND CLAY (-200): 64 %	DATE: OCTOBER 31, 1989 — 15



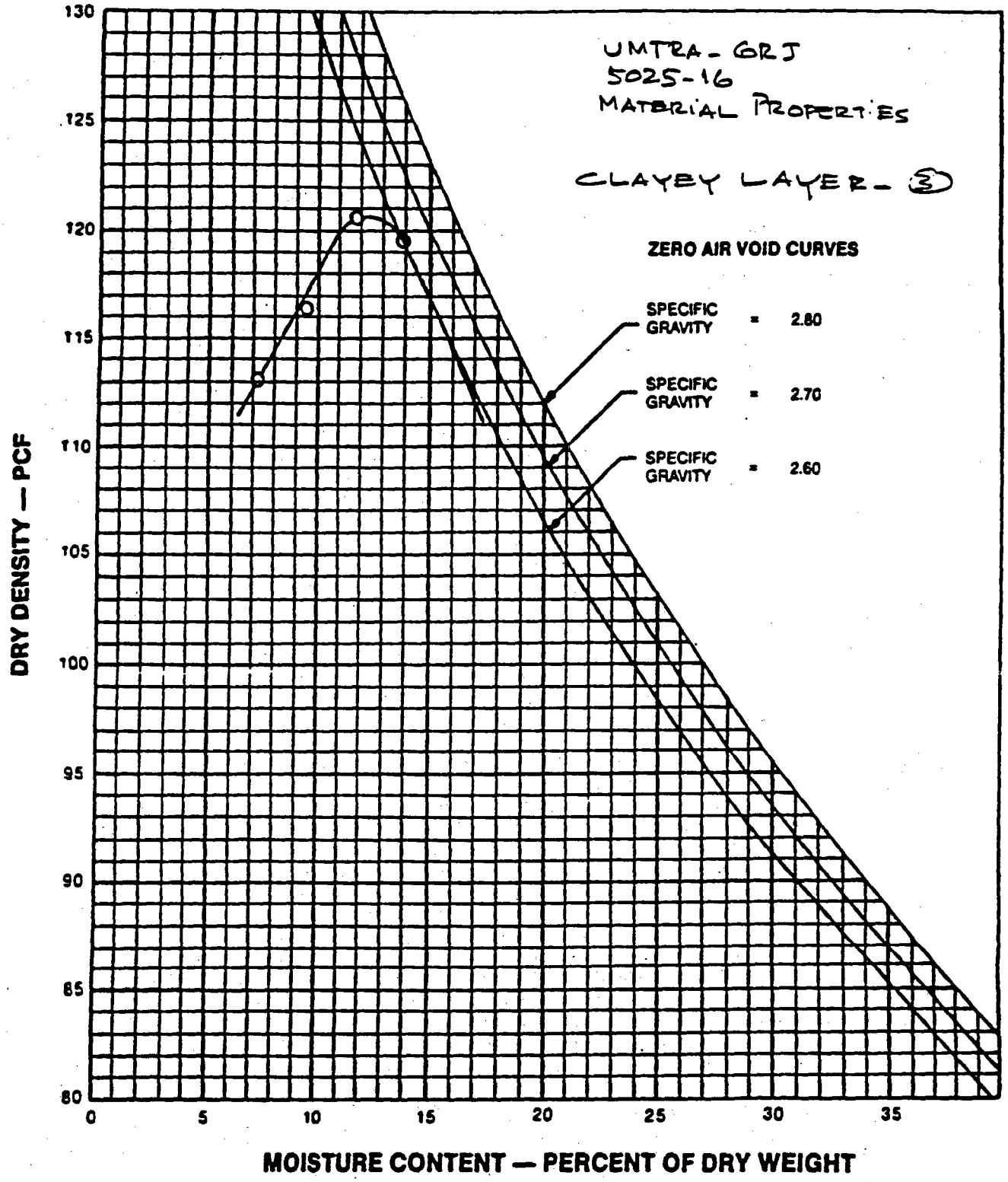
LOCATION:		MOISTURE—DENSITY RELATIONSHIPS	
HOLE NO: TP 131	DEPTH: 17-25	SAMPLE NO.: 19	
SOIL DESCRIPTION: CL; Sandy lean clay		Chen-Northern, Inc.	
MAX. DRY DENSITY: 114.9 PCF	OPT. MOIST. CONTENT: 14.2 %	PROCEDURE: ASTM D698-78, Meth."A"	
LIQUID LIMIT: 29	PLASTICITY INDEX: 15	JOB NO.: 1 126 90	FIG. NO.
GRAVEL: 2 %	SAND: 32 %	SILT AND CLAY (-200): 66 %	DATE: NOVEMBER 3, 1989	16



LOCATION:		MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: TP 133 DEPTH: 11-15 SAMPLE NO.: 20		
SOIL DESCRIPTION: CL; Sandy lean clay		Chen-Northern, Inc.
MAX. DRY DENSITY: 116.2 PCF	OPT. MOIST. CONTENT: 13.9 %	PROCEDURE: ASTM D698-78, Meth."A"
LIQUID LIMIT: 28	PLASTICITY INDEX: 13	JOB NO.: 1 126 90 / FIG. NO.
GRAVEL: 4 % SAND: 44 %	SILT AND CLAY (-200): 52 %	DATE: OCTOBER 31, 1989 / 17



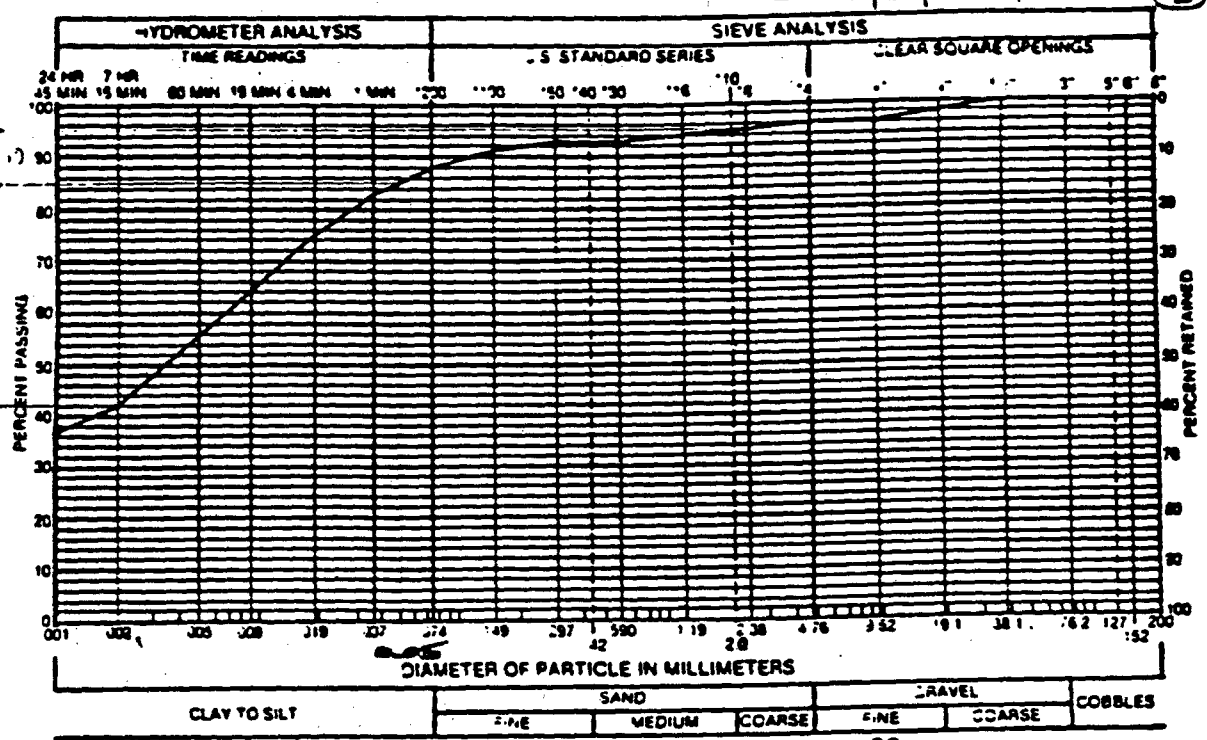
LOCATION:			MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: TP 136	DEPTH: 10-17	SAMPLE NO.: 21	
OIL DESCRIPTION: CL; Lean clay			Chen-Northern, Inc.
MAX. DRY DENSITY: 114.9 PCF	OPT. MOIST. CONTENT: 15.7 %		PROCEDURE: ASTM D698-78, Meth."A"
LIQUID LIMIT: 38	PLASTICITY INDEX: 24		JOB NO.: 1 126 90 — FIG. NO.
GRAVEL: 1 %	SAND: 11 %	SILT AND CLAY (-200): 88 %	DATE: NOVEMBER 3, 1989 — 18



LOCATION:	MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: TP122 DEPTH: 7-12 SAMPLE NO.: 9	
SOIL DESCRIPTION: SC-SM; Silty, clayey sand	Chen-Northern, Inc.
MAX. DRY DENSITY: 120.7 PCF OPT. MOIST. CONTENT: 12.5 %	PROCEDURE: ASTM D698-78, Meth "A"
LIQUID LIMIT: 23 PLASTICITY INDEX: 7	JOB NO.: 1 126 90 — FIG. NO.
GRAVEL: 6 % SAND: 54 % SILT AND CLAY (-200): 40 %	DATE: NOVEMBER 3, 1989 — 8

5025-16

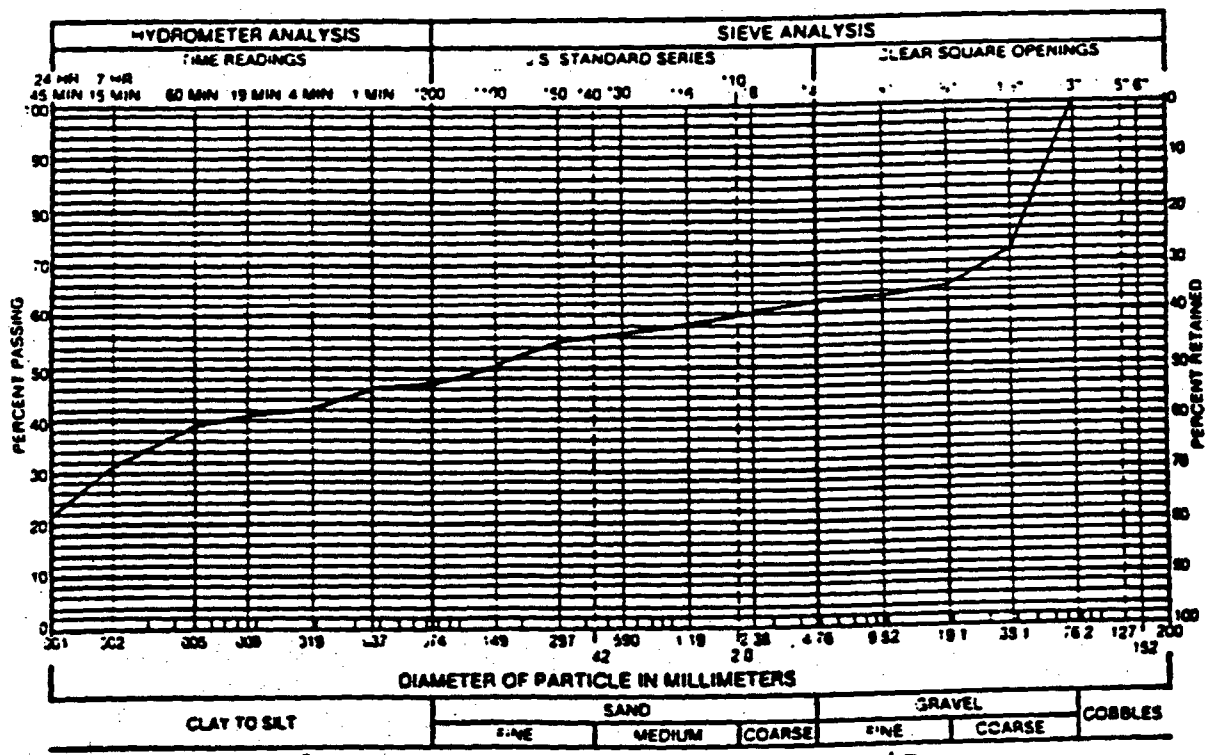
MATERIAL PROPERTIES
CLAYEY LAYER - 3



GRAVEL 4 % SAND 8 % SILT AND CLAY 88 %

LIQUID LIMIT 40 % PLASTICITY INDEX 25 %

SAMPLE OF Lean clay FROM TP 125 at depth 18.0-21.0, Sample No. 12

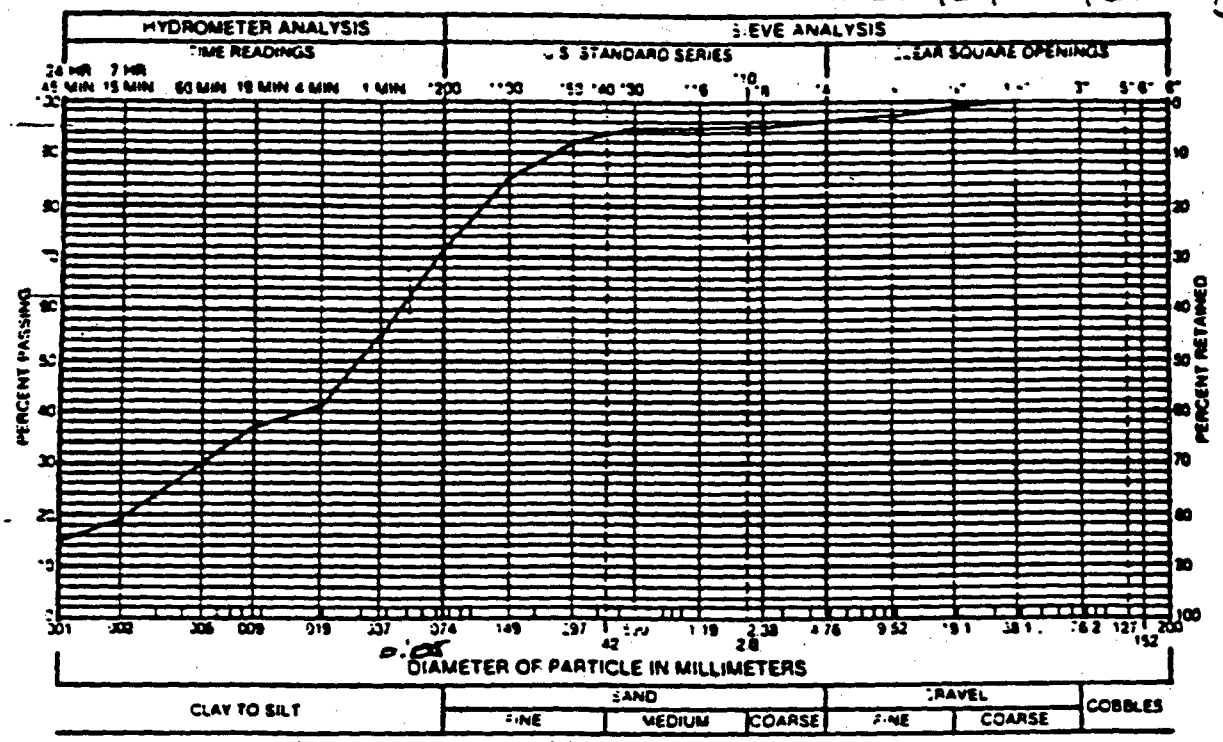


GRAVEL 38 % SAND 15 % SILT AND CLAY 47 %

LIQUID LIMIT 41 % PLASTICITY INDEX 19 %

SAMPLE OF Clayey gravel with sand FROM TP 126 at depth 1.0-4.0, Sample No. 13

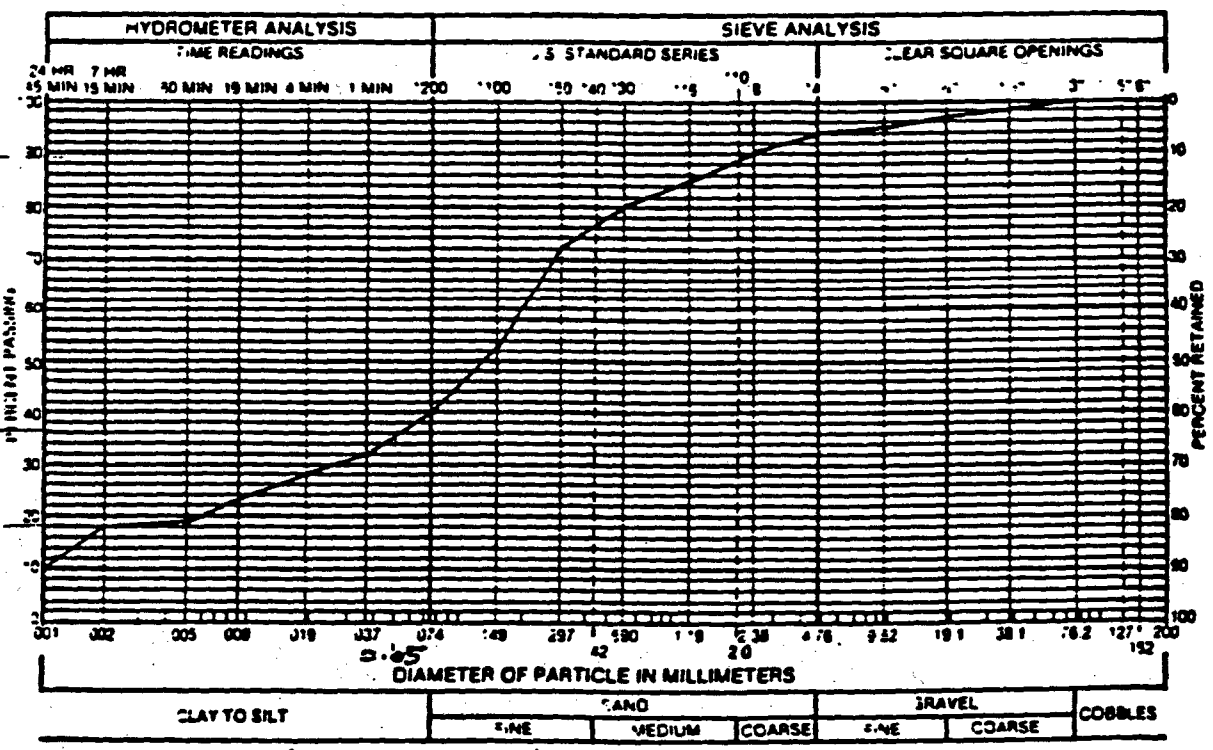
1 126 90	Chen Northern, Inc.	GRADATION TEST RESULTS	Fig. 3



GRAVEL 4 % SAND 24 % SILT AND CLAY 72 %

LIQUID LIMIT 23 % PLASTICITY INDEX 10 %

SAMPLE OF Lean clay with sand FROM TP 114 at depth 0-1.0, Sample No. 5



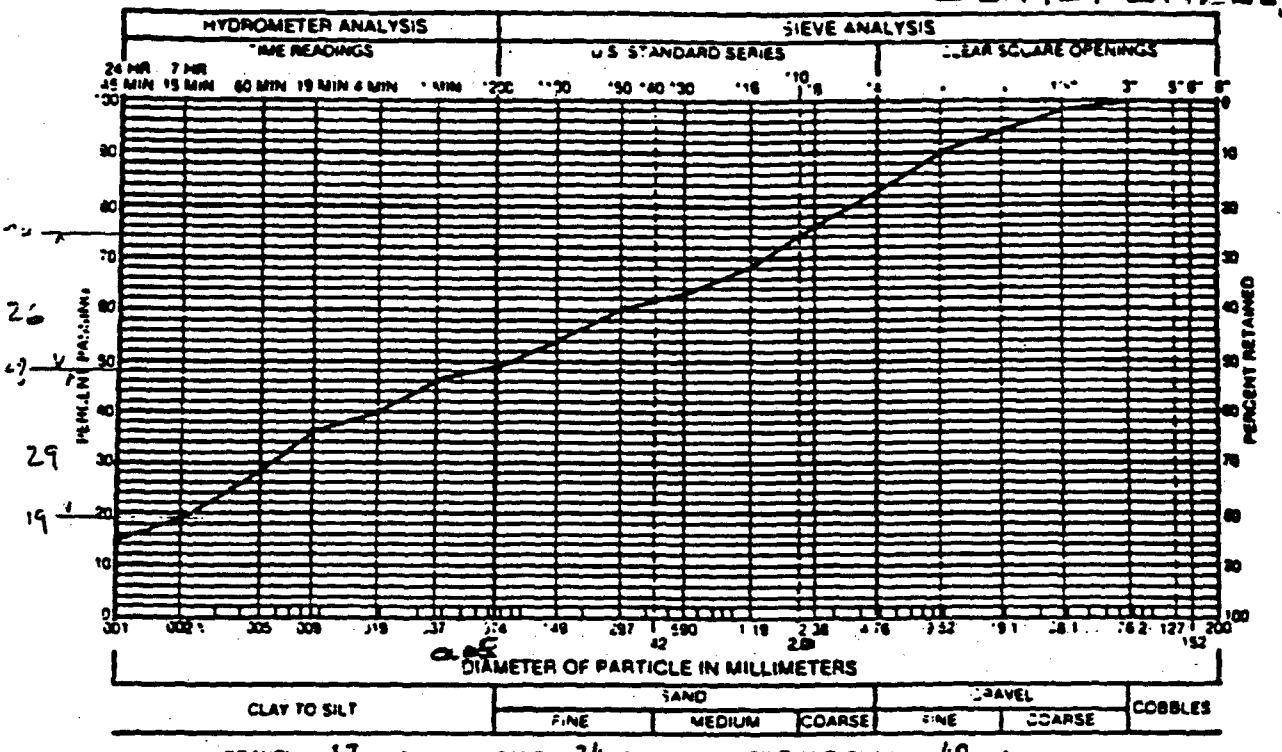
GRAVEL 6 % SAND 54 % SILT AND CLAY 40 %

LIQUID LIMIT 23 % PLASTICITY INDEX 7 %

SAMPLE OF Silty, clayey sand FROM TP 122 at depth 7.0-12.0, Sample No. 9

1 126 90	Chen Northern, Inc.	GRADATION TEST RESULTS	Fig. 1

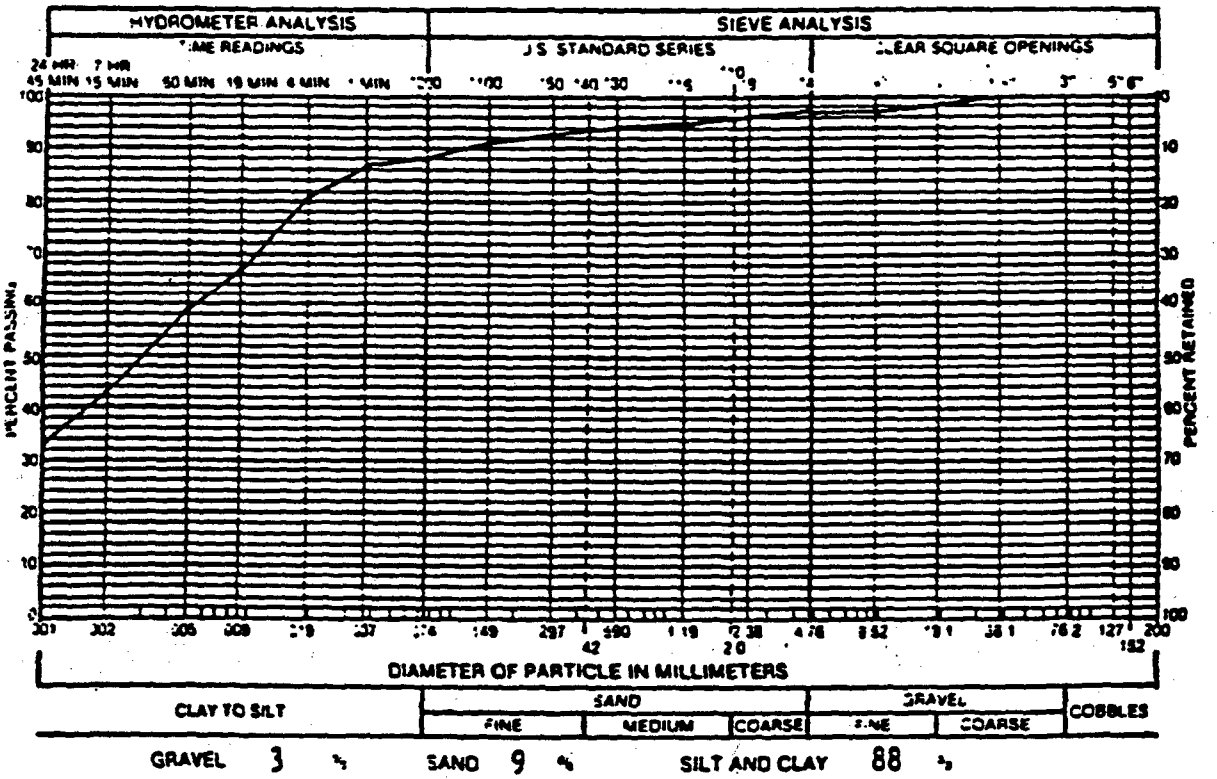
UMTRA-GRJ 214
5025-1
MATERIAL PROPERTIES
CLAYEY LAYER-3



GRAVEL 17 % SAND 34 % SILT AND CLAY 49 %

LIQUID LIMIT 33 % PLASTICITY INDEX 18 %

SAMPLE OF Clayey sand with gravel FROM TP 123 at depth 14.0-23.0, Sample No. 10



GRAVEL 3 % SAND 9 % SILT AND CLAY 88 %

LIQUID LIMIT 40 % PLASTICITY INDEX 24 %

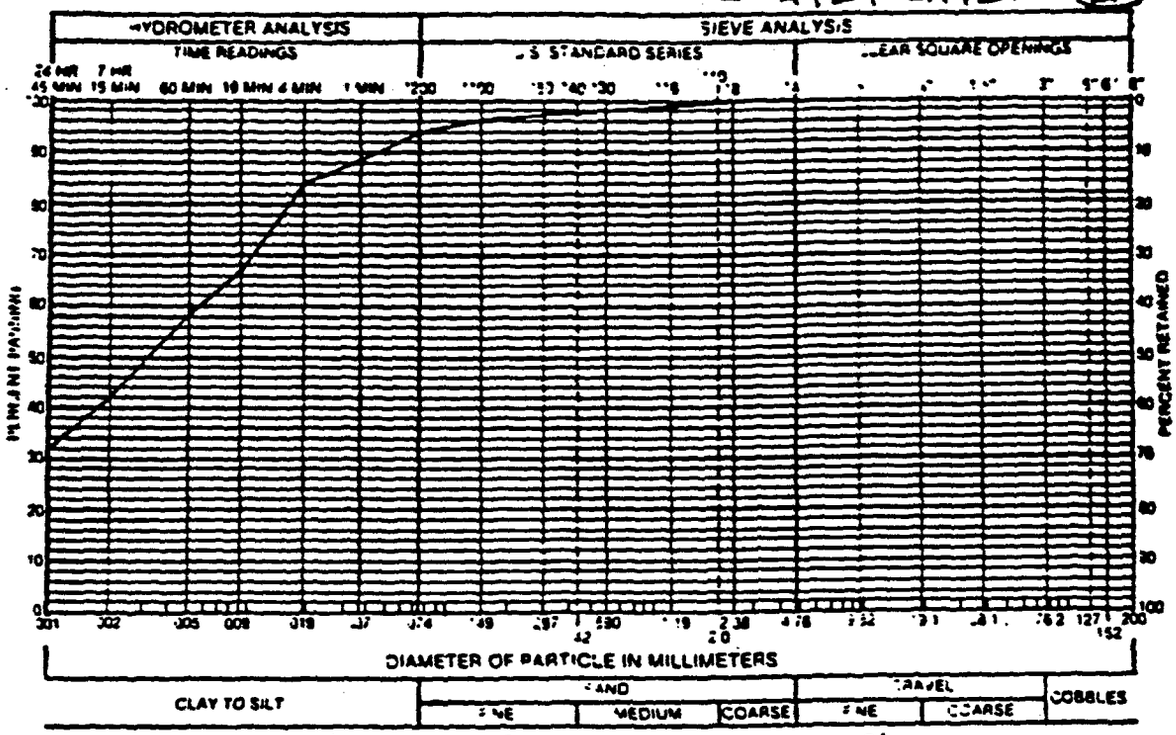
SAMPLE OF Lean clay FROM TP 124 at depth 18.5-23.0, Sample No. 11

1 126 90	Chen Northern, Inc.	GRADATION TEST RESULTS	Fig. 2

5025-16

215

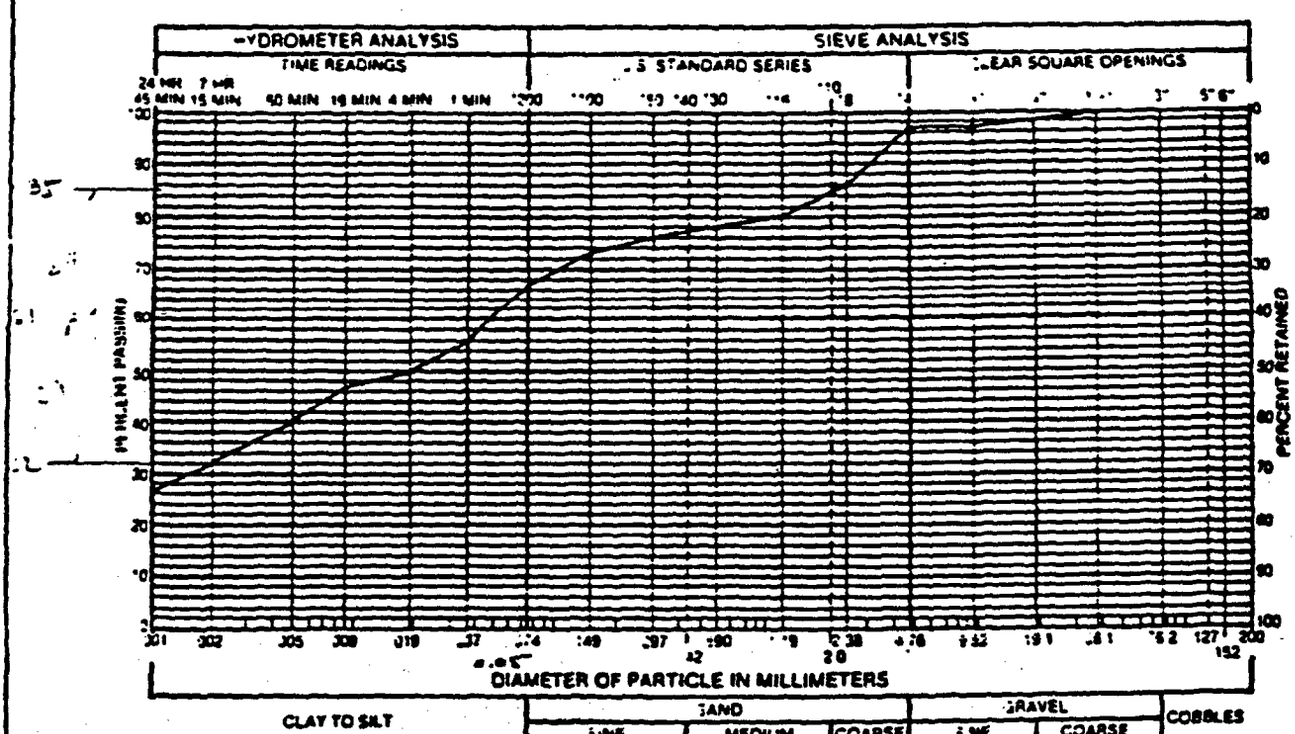
MATERIAL PROPERTIES
CLAYEY LAYER - 3



GRAVEL 0 % SAND 6 % SILT AND CLAY 94 %

LIQUID LIMIT 43 % PLASTICITY INDEX 25 %

SAMPLE OF Lean clay FROM TP 126 at depth 14.0-21.0, Sample No. 14



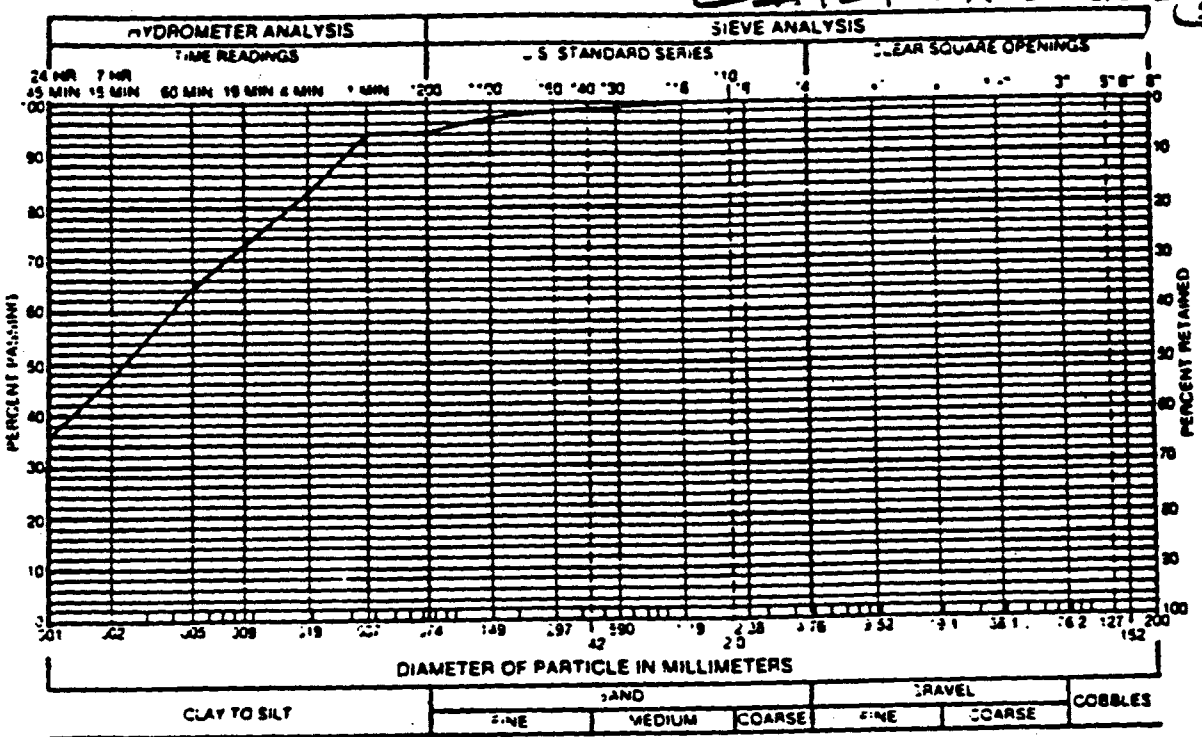
GRAVEL 3 % SAND 30 % SILT AND CLAY 67 %

LIQUID LIMIT 37 % PLASTICITY INDEX 22 %

SAMPLE OF Sandy lean clay FROM TP 127 at depth 15.0-25.5, Sample No. 15

1 126 90	Chen Northern, Inc.	GRADATION TEST RESULTS	Fig. 4

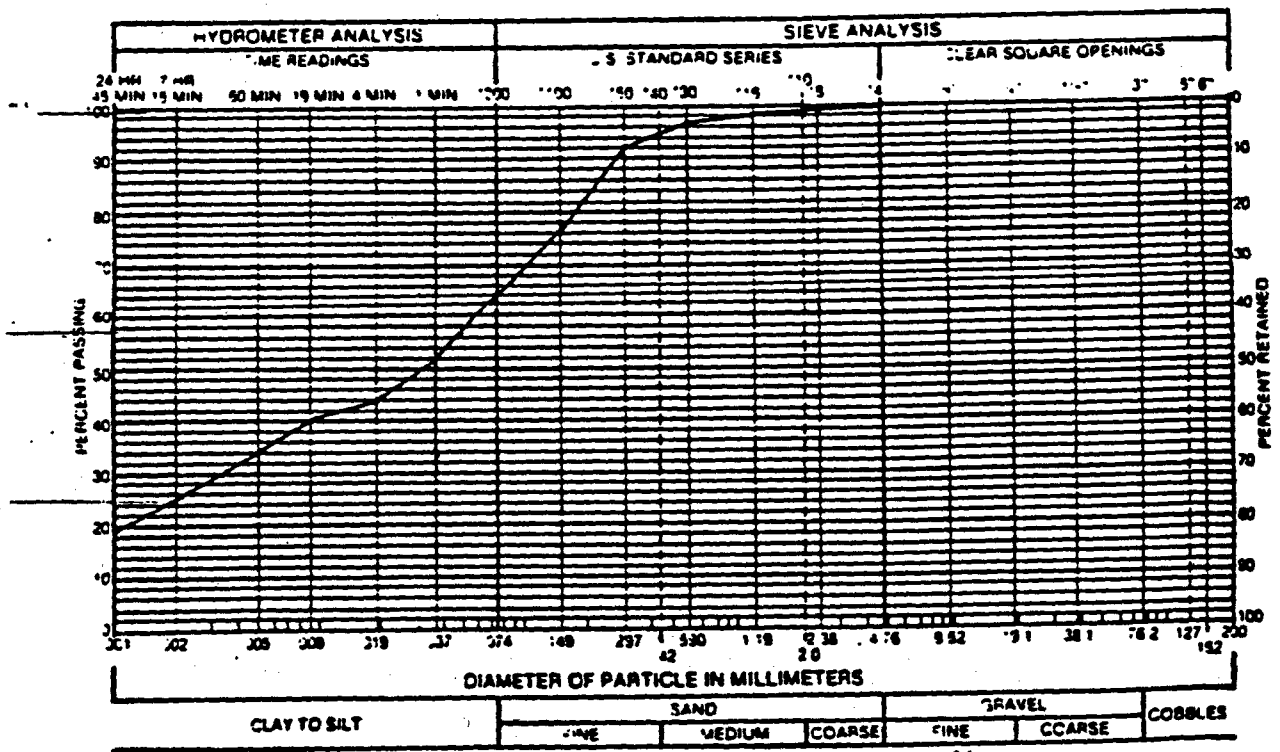
UMTRA-GRJ 216
5025-16
MATERIAL PROPERTIES
CLAYEY MATERIAL-3



GRAVEL 0 % SAND 5 % SILT AND CLAY 95 %

LIQUID LIMIT 44 % PLASTICITY INDEX 27 %

SAMPLE OF Lean clay FROM TP 128 at depth 16.0-25.0, Sample No. 16



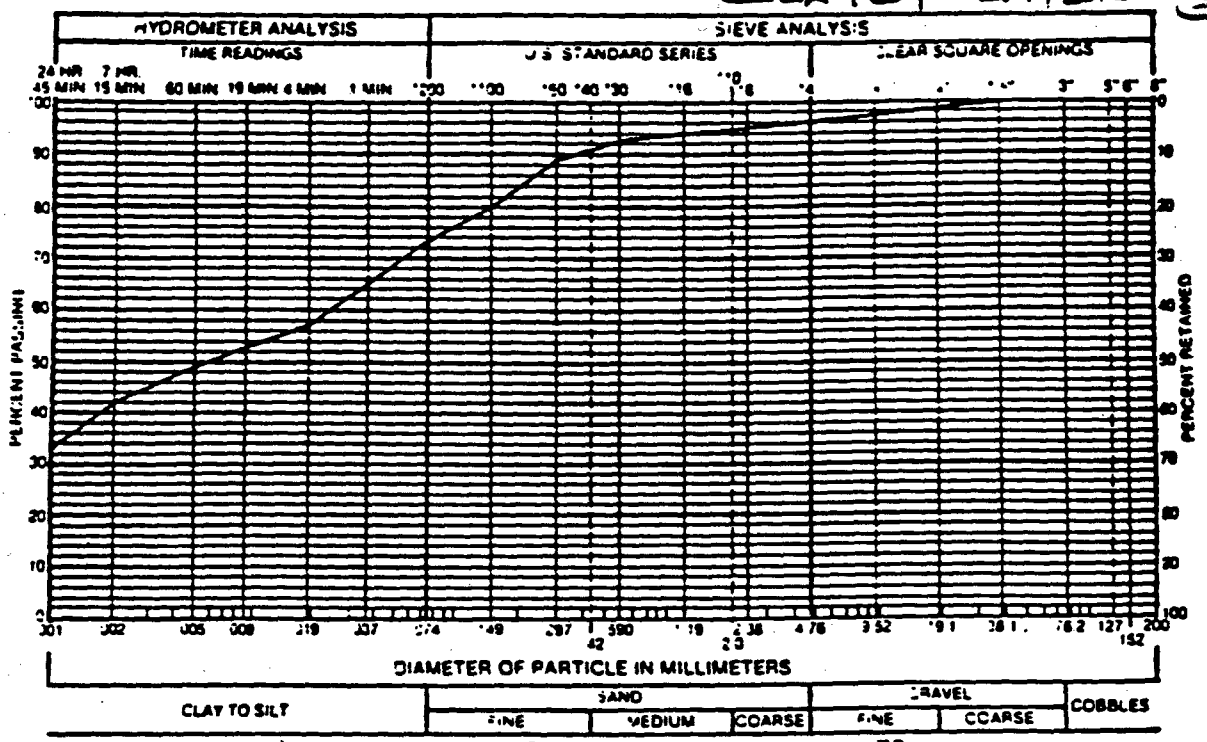
GRAVEL 0 % SAND 36 % SILT AND CLAY 64 %

LIQUID LIMIT 33 % PLASTICITY INDEX 17 %

SAMPLE OF Sandy lean clay FROM TP 130 at depth 10.5-14.5, Sample No. 17

1 126 90	Chen Northern, Inc.	GRADATION TEST RESULTS	Fig. 5

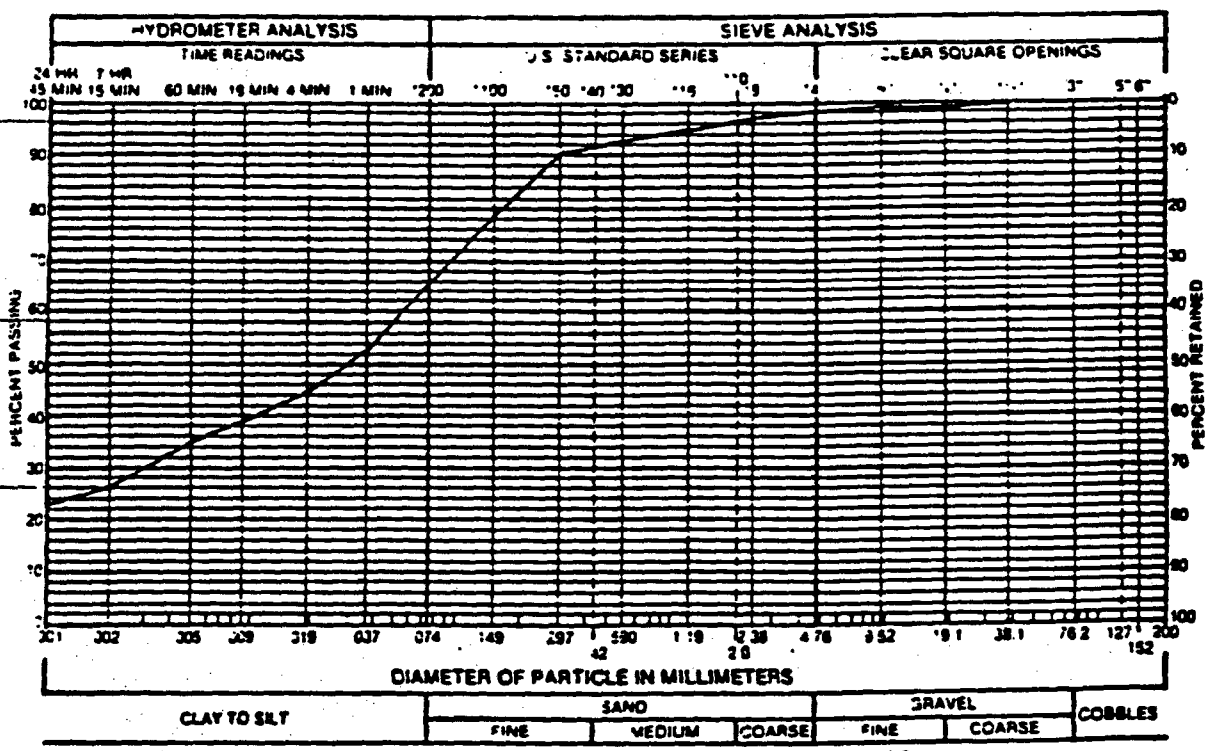
UMTRA-GRJ 217
5025-16
MATERIAL PROPERTIES
CLAYEY LAYER - 3



GRAVEL 4 % SAND 23 % SILT AND CLAY 73 %

LIQUID LIMIT 41 % PLASTICITY INDEX 24 %

SAMPLE OF Lean clay with sand FROM TP 131 at depth 0-1.5, Sample No. 18



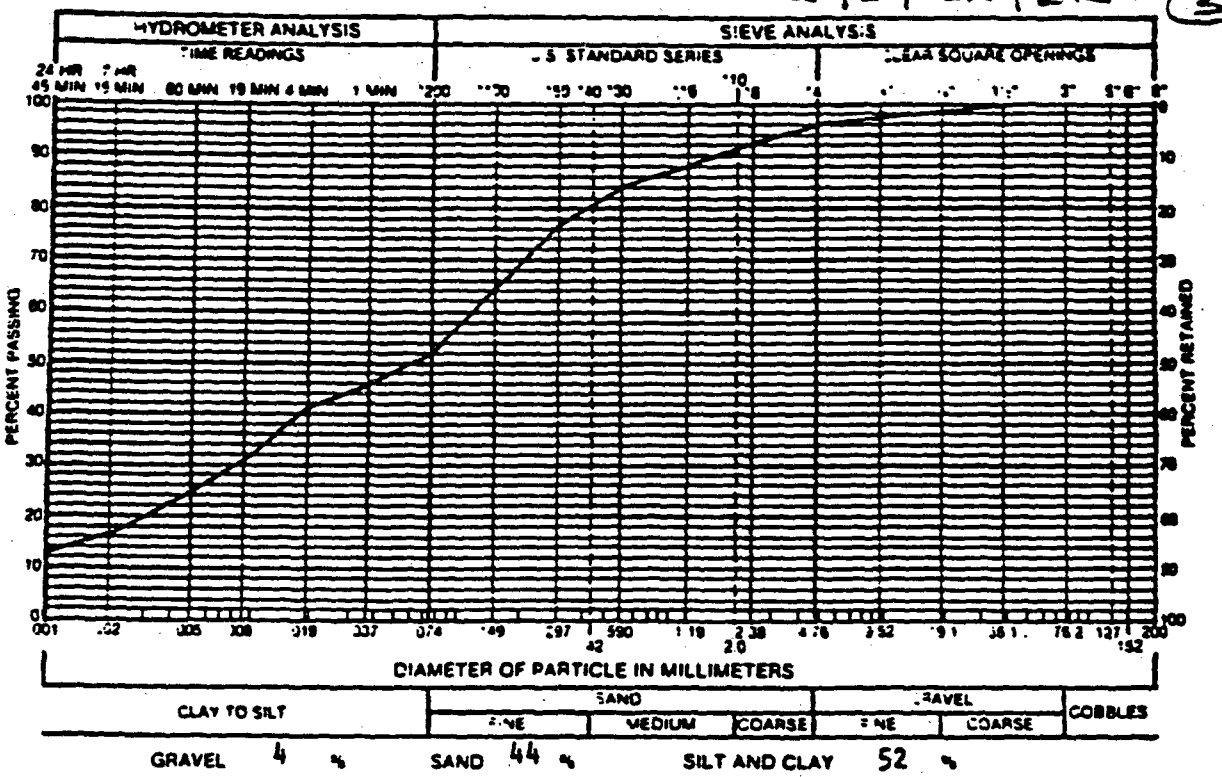
GRAVEL 2 % SAND 32 % SILT AND CLAY 66 %

LIQUID LIMIT 29 % PLASTICITY INDEX 15 %

SAMPLE OF Sandy lean clay FROM TP 131 at depth 17.0-25.0, Sample No. 19

1 126 90	Chen Northern, Inc.	GRADATION TEST RESULTS	Fig. 6

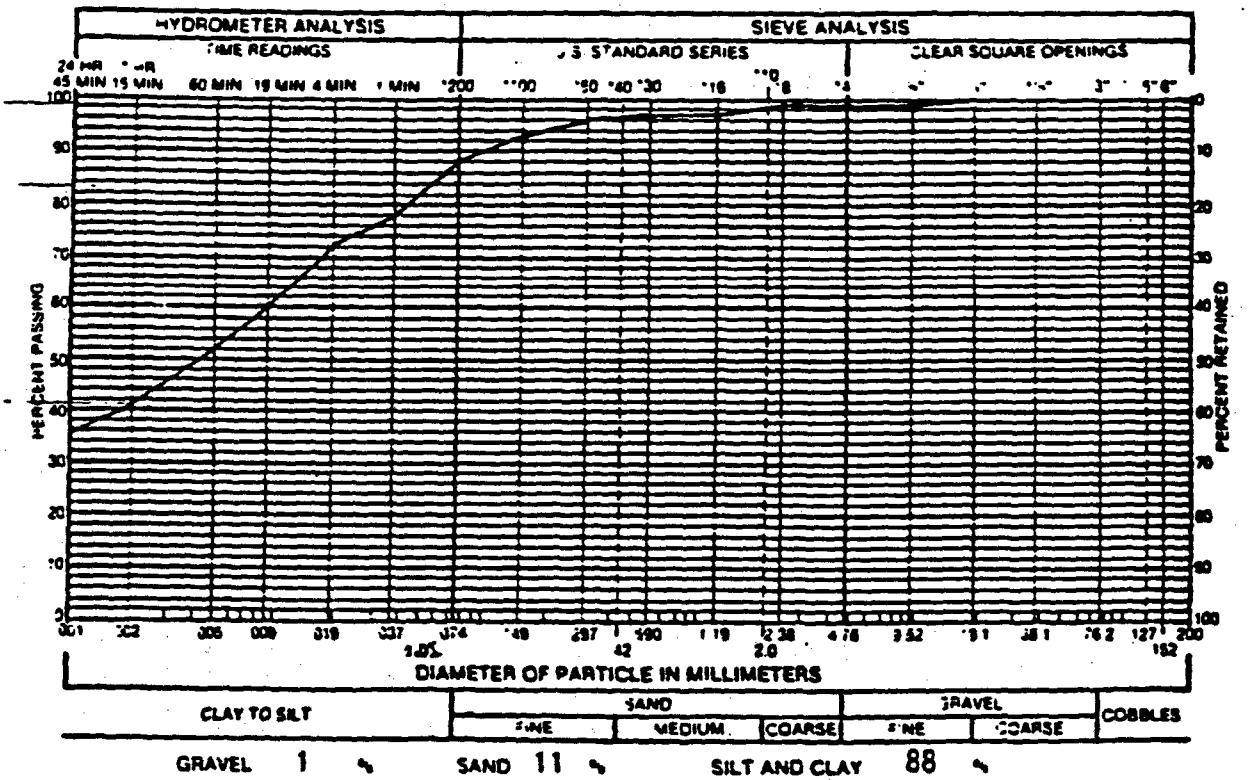
UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
CLAYEY LAYER - (3)



GRAVEL 4 % SAND 44 % SILT AND CLAY 52 %

LIQUID LIMIT 28 % PLASTICITY INDEX 13 %

SAMPLE OF Sandy lean clay FROM TP 133 at depth 11.0-15.0, Sample No. 20



GRAVEL 1 % SAND 11 % SILT AND CLAY 88 %

LIQUID LIMIT 38 % PLASTICITY INDEX 24 %

SAMPLE OF Lean clay FROM TP 136 at depth 10.0-17.0, Sample No. 21

1 126 90	Chen Northern, Inc.	GRADATION TEST RESULTS	Fig. 7

Chen Northern, Inc.
Consulting Engineers and Scientists
96 South Zuni Street
Denver Colorado 80223
303 744-7105
303 744-0210 Facsimile

UMTRA-GRJ
5025-16
MATERIAL PROP.
CLAYEY LAYER-(5)

Received: 1-24-90

MKE DOC. 5025-GRJ-L-09-3888-00

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JAN 24 1990
UMTRA-S.F.

MK-FERGUSON CO.
ALBUQUERQUE
JAN 19 1990
RECEIVED

January 17, 1990

MK - Ferguson Company
P.O. Box 9136
Alburquerque, New Mexico 87119

Attention: Mr. Jerry Hymes

Subject: Laboratory Testing, P.O. No. 3050-511-10034

Job No. 1 126 90

Dear Mr. Hymes:

As requested, we have completed the triaxial permeability tests on soil samples from the UMTRAP Cheney site with the exception of the tests on the Mancos shale samples. The results are presented in Table III. Also enclosed are copies of all worksheets.

If you have any questions regarding this submittal, please call.

Sincerely,

Sally K. Miller

Sally K. Miller, A.E.T.
Soils Laboratory Supervisor

SKM/sej
Rev. By: NFL
Enclosures

Attachment: MKE DOC. 5025-GRJ-R-09-03889-00

UMTRA-GRJ
5025-16
MATERIAL Prop. MKE DOC. 5025-GRJ_R-09-03889-00
CLAYEY LAYER - (3) January 16, 1990
Job No. 1 126 90

CHEM-NORTHERN, INC.

TABLE II

SUMMARY OF PERMEABILITY TEST RESULTS

Test Pit No.	Depth, Feet	Initial Moisture Content, %	Initial Dry Density, pcf	Initial Compaction, %	Final Moisture Content, %	Final Dry Density, pcf	Final Saturation, %	Coefficient of Permeability, cm/sec
123	14 - 23	14.3	110.1	96.3	21.9	105.5	93.9	4.7×10^{-7}
125	18 - 21	17.1	104.0	94.8	25.1	97.6	90.0	3.9×10^{-7}
125	18 - 21	16.8	109.6	99.9	22.9	102.7	92.7	1.6×10^{-8}
127	15 - 25.5	12.4	113.2	96.3	21.0	105.0	89.9	4.7×10^{-8}
130	10.5 - 14.5	16.1	104.8	94.7	23.5	100.5	90.6	4.0×10^{-7}
130	10.5 - 14.5	15.3	111.6	100.9	20.9	106.3	92.6	2.0×10^{-7}
131	17 - 25	13.6	109.8	95.5	21.6	104.9	92.5	1.2×10^{-7}
136	10 - 17	13.9	110.9	96.5	23.2	102.4	93.8	2.4×10^{-7}
136	10 - 17	15.0	115.4	100.4	20.1	109.3	96.3	1.4×10^{-8}

UMTRA-GRJ REV. 3 H.L. 6/18/90
5025-16
MATERIAL PROPERTIES
CLAYEY LAYER

m.220

Chen Northern, Inc.
Consulting Engineers and Scientists
96 South Zuni Street
Denver, Colorado 80223
303 744-7105
303 744-0210 Facsimile

MKE DOC. GRJ-L-09-04330-00 PS 221

UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
CLAYEY LAYER - 3

May 29, 1990

MK - Ferguson Company
P.O. Box 9136
Albuquerque, New Mexico 87119

Attention: Mr. Jerry Hymas

Subject: Laboratory Testing, P.O. Box 3050-511-10034, Change Order No. 5

Job No. 1 126 90

Dear Mr. Hymas

As requested by Mr. Partha Sircar, we have completed two additional triaxial permeability tests on soil samples from the UMTRAP Cheney site. A minimum value of 0.98 was obtained for the B-parameter during saturation before permeability measurements were initiated. The results are presented in Table I. Also enclosed are copies of all worksheets.

If you have questions regarding this submittal, please call.

Sincerely,

RECEIVED-MKE
JUN 0 1990
UMTRA-S.F.

CHEN-NORTHERN, INC.

Jane D. Maeder for Skp

Sally K. Miller, A.E.T.
Soils Laboratory Supervisor

Rev. By: JTJ
SKM/sej
Enclosures
cc: Mr. Partha Sircar
MK - Environmental

Mr. Thom Myer
MK - Ferguson Company

Attachment: left to document

Chen Northern, Inc.

CHEN-NORTHERN, INC.

May 29, 1990
Job No. 1 126 90

TABLE I
SUMMARY OF PERMEABILITY TEST RESULTS

Test Pit No.	Depth, Feet	Initial Moisture Content, %	Initial Dry Density, pcf	Initial Compaction, %	B-Parameter After Saturation	Final Moisture Content, %	Final Dry Density, pcf	Final Saturation, %	Coefficient of Permeability, cm/sec
128	16 - 25	11.6	110.2	100.9	0.99	26.8	97.0	98.1	2.9×10^{-8}
130	10.5-14.5	15.9	111.0	100.4	0.98	20.7	106.7	96/6	1.5×10^{-7}

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
CLAYEY LAYER - (3)

222

Consulting Engineers and Scientists

~~Layer 5 — Weathered Shale~~ Layer 4 – Mancos Shale Layer

REV. 3 H.L. 6/18/90

Introduction

The weathered shale (Layer ~~5~~ 4) is the bottom most layer pertinent to our analysis and design. The available test results on this material are summarized on the next sheet. As evident, there are no density or strength measurements, which are therefore inferred as below.

Density

Since water table exists above the top of the shale, the shale is assumed to be saturated. Average water content is 10.3 from next sheet. Assuming a $G_s = 2.75$,

$$e = wG_s = 2.75 \times 0.103 = 0.283$$

$$\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.75 \times 62.4}{1+0.283} = 133.7 \text{ pcf.}$$

$$\gamma = 133.7 \times 1.103 = 147.5 \text{ pcf. Use } \underline{\gamma = 145 \text{ pcf}}$$

Strength

The Mancos Shale is very highly weathered at the top. So, use a relatively low shear strength — $\underline{S_u = 5000 \text{ psf}}$ (barely a hard clay. Alternatively, due to the presence of fractures, etc., a value of $\phi = 35°$, $c = 0$ could be used, considering it as a gravel. This is less conservative for the present case. Further, it assumes that the fractures will be ~~fo~~ fully drained, which may not be true for short term loading, e.g. seismic loading. Thus, use S_u for short term, and ϕ for long term loading.

Recommended Design Parameters

Density (In-Situ and Saturated) = 145 pcf ; Moisture Content = 10.3 pct

Shear Strength: Short Term (Undrained): $S_u = 5000$ psf; Long Term Static — $c = 0$, $\phi = 35°$

SUMMARY OF SOIL TEST RESULTS

Job No 5025 Project Name UMTRA - Grand Junction Feature Cheney Reservoir Site Date 2/5/89 WYL

PS 2/9/89

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γ_d	Compaction Test	Compaction Optimum w	Compaction Optimum γ_d	Shear Strength Test	Shear Strength Initial w	Shear Strength Initial γ_d	Shear Strength C	Shear Strength ϕ	Permeability γ_d	Permeability k cm/sec	Consolidation C_c	Consolidation C_r	Consolidation P_c	Notes
LAYER	No. 5																									
(501) GCH-1		31.0	40.0	Weathered shale				32	17		10.3											$<1\times10^{-7}$				
"		40.0	50.0	"																		$<1\times10^{-7}$				
(502) GCH-2		36.0	46.0	"																		$<1\times10^{-7}$				
"		41.0	51.0	"																		$<5\times10^{-5}$				
(503) GCH-3		30.0	40.0	"																		$<1\times10^{-7}$				
"		40.0	50.0	"																		1×10^{-6}				
(504) GCH-4		44.0	56.0	"				34	16		10.4											$<5\times10^{-8}$				
(505) GCH-5		38.0	51.0	"																		1×10^{-7}				
(506) GCH-6		29.0	40.0	"																		1×10^{-4} to 1×10^{-5}	(Very weathered shale)			
"		40.0	51.0	Weathered shale																		5×10^{-4} to 1×10^{-5}	(Very weathered shale)			
Average								33	17		10.3											Approx. 1×10^{-7}				

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

REVISION 3

LAYER 5 – GENERAL FILL

The general fill material consists of clay and sand excavated from the tailing embankment. It is expected that the general fill material will be used to restore processing and disposal site. The following test results are available

- Grain size including Hydrometer
- Compaction Tests ASTM D-698
- Permeability Tests

DENSITY, MOISTURE CONTENT, AND SPECIFIC GRAVITY

NOTE: Data from Summary of Test Results on Th. 22

(a) Dry density, $\gamma_{dry\ ave.} = \frac{108.5 + 103.8 + 108.5 + 106.1 + 104.1}{5}$

$= 107.2$ pcf

(b) Optimum Moisture Content, $W_{AVE.} = \frac{16.5 + 22.6 + 17.5 + 18.2 + 17.5}{5}$

$= 18.5\ \%$

(c) Assume Specific Gravity $= 2.7$

(d) Average Moist Density, $\gamma = 107.2 \times 1.185$

$= 127$ pcf

(e) Average Saturated density

void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 62.4}{107.2} - 1 = 0.572$

REVISION 3

LAYER 5 – GENERAL FILL

$$\gamma_{AVE.} = \frac{G_s \gamma_w + e \gamma_w}{1+e} = \frac{2.7 \times 62.4 + 0.572 \times 62.4}{1+0.572}$$

$$= 129.8 \text{ pcf}$$

$$W_{sat.} = \frac{S e}{G} = \frac{100\% \times 0.572}{2.7} = 0.21$$

$$W_{sat} = 21\%$$

STRENGTH

There is no shear strength test available when this report is made, however, for design purposes, we can use data from USBR (From sh 248) For short-term and long-term loading, use $c = 280$ psf and $\phi = 28°$

RECOMMENDED DESIGN PARAMETERS

Density (Moist Density) = 120.7 pcf
Moisture Content = 18.5%
Shear Strength:

Short-Term : $c = 280$ psf, $\phi = 28°$
Long-Term : $c = 280$ psf, $\phi = 28°$

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
GENERAL FILL — (5)
CLAYEY SOILS FOR
RESERVE RADON BARRIER

SJG 6-22-90

DATA REFERENCE:
MKE DOC. 5025-GRJ-L-09-04141-00

SUMMARY OF SOIL TEST RESULTS

Job No 5025-16 Project Name UMTRA - GRAND JUNCTION Feature CHENEY DISPOSAL SITE Date 6/12/90

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural W %	Natural γ_d	Compaction Test	Compaction Optimum W	Compaction Optimum γ_d	Shear Strength Test	Shear Strength Initial W	Shear Strength Initial γ_d	Shear Strength C	Shear Strength ϕ	Permeability (cm/sec) γ_d	Permeability (cm/sec) 95% COMPACTION	Consolidation C_c	Consolidation C_r	Consolidation P_c	Notes
	1				0	11	89						SP	16.5	108.5											
	2				0	11	89						SP	22.6	103.8											
	3				0	13	87						SP	17.5	108.5											
	4				0	37	63						SP	18.2	106.1							1.4×10^{-6}				
	5				0	54	46						SP	17.5	109.1							5.6×10^{-7}				
AVERAGE =						25	75							18.5	107.2											

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Spiral - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

P5

3-28-90 228

MKE DOC. 5025-GRJ-L-09-04141-00

WESTERN ENGINEERS, INC.
CONSULTING ENGINEERS / LAND SURVEYORS
2150 Hwy. 6 & 50, Grand Junction, CO 81505-9422 • (303) 242-5207 • FAX (303) 242-1672

March 21, 1990

These are clayey soils from the Cheney Excavation which are being stockpiled as 'reserve' radon barrier material — Pa, Hm

Mr. Jose Cercone
MK Environmental Services
180 Howard Street
Sand Francisco, CA 84105

RE: Cheney Disposal Site Soil Tests

Jose:

Enclosed are the results of Hydrometer Tests and Standard Proctor Curves for samples submitted on March 13, 1990, for the above project. In addition, Permeability Tests were performed on samples 4 and 5 at a density of 95% Standard Proctor with a confining load of 300 psf. Sample 4 exhibited a permeability of 1.4×10^{-6} cm/sec and sample 5 a permeability of 5.6×10^{-7} cm/sec.

White soluble material was visible in all of these samples. Sample 4 appeared to contain a significantly higher percentage. During the test, the permeability of sample 4 increased from 9×10^{-7} cm/sec to 1.4×10^{-6} cm/sec.

If you have any questions or if we can be of any further assistance, please do not hesitate to call me.

Submitted by:

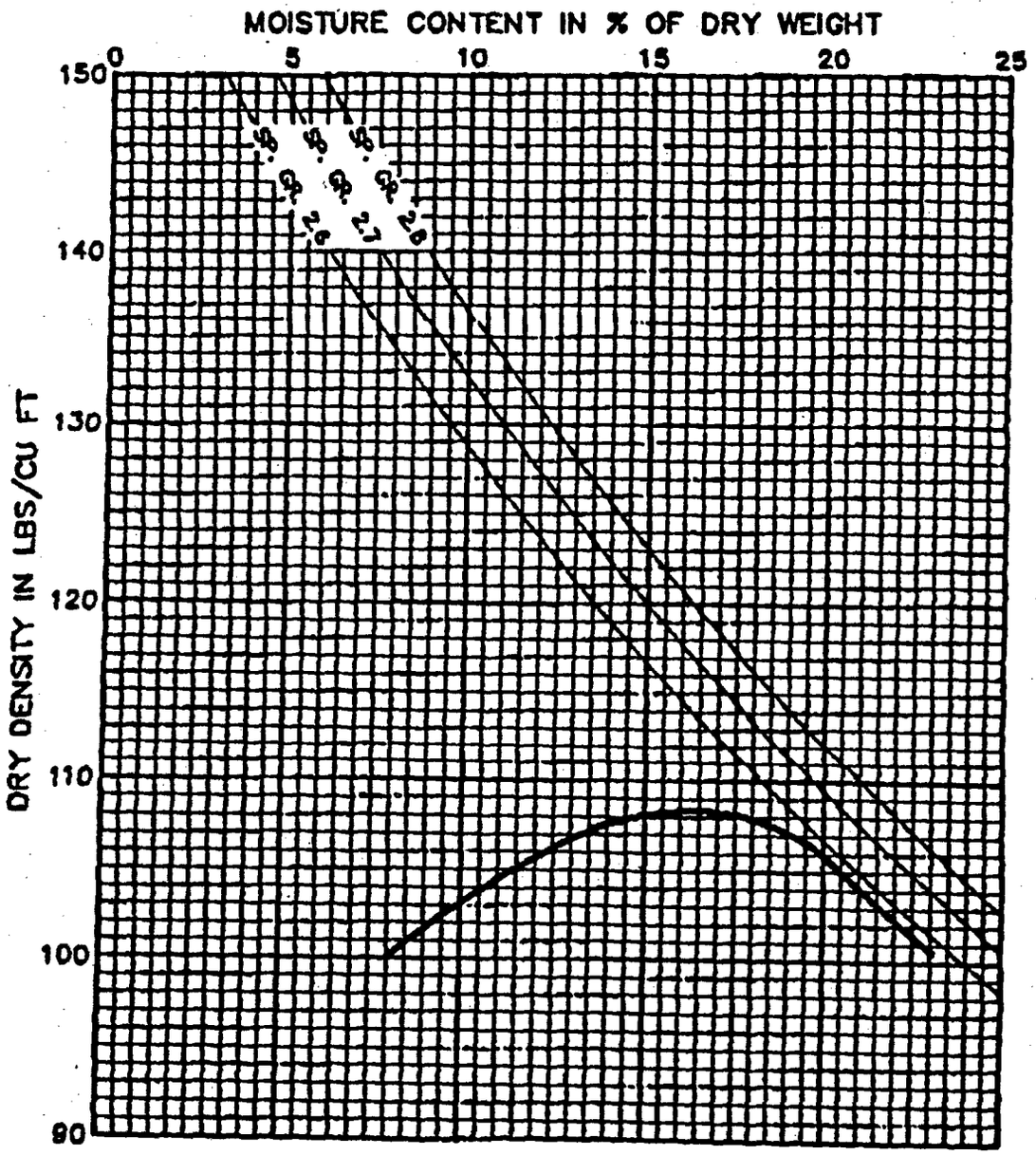
WESTERN ENGINEERS, INC.

Bruce D. Marvin, P.E.
President

[W.O. # 2633]

Attachment: Left to document.

SAMPLE NO. 1 DEPTH _____ ELEVATION _____
SOIL _____
LOCATION CHENEY DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 16.5%
MAXIMUM DRY DENSITY 108.5 PCF
METHOD OF COMPACTION ASTM D-698, METHOD A

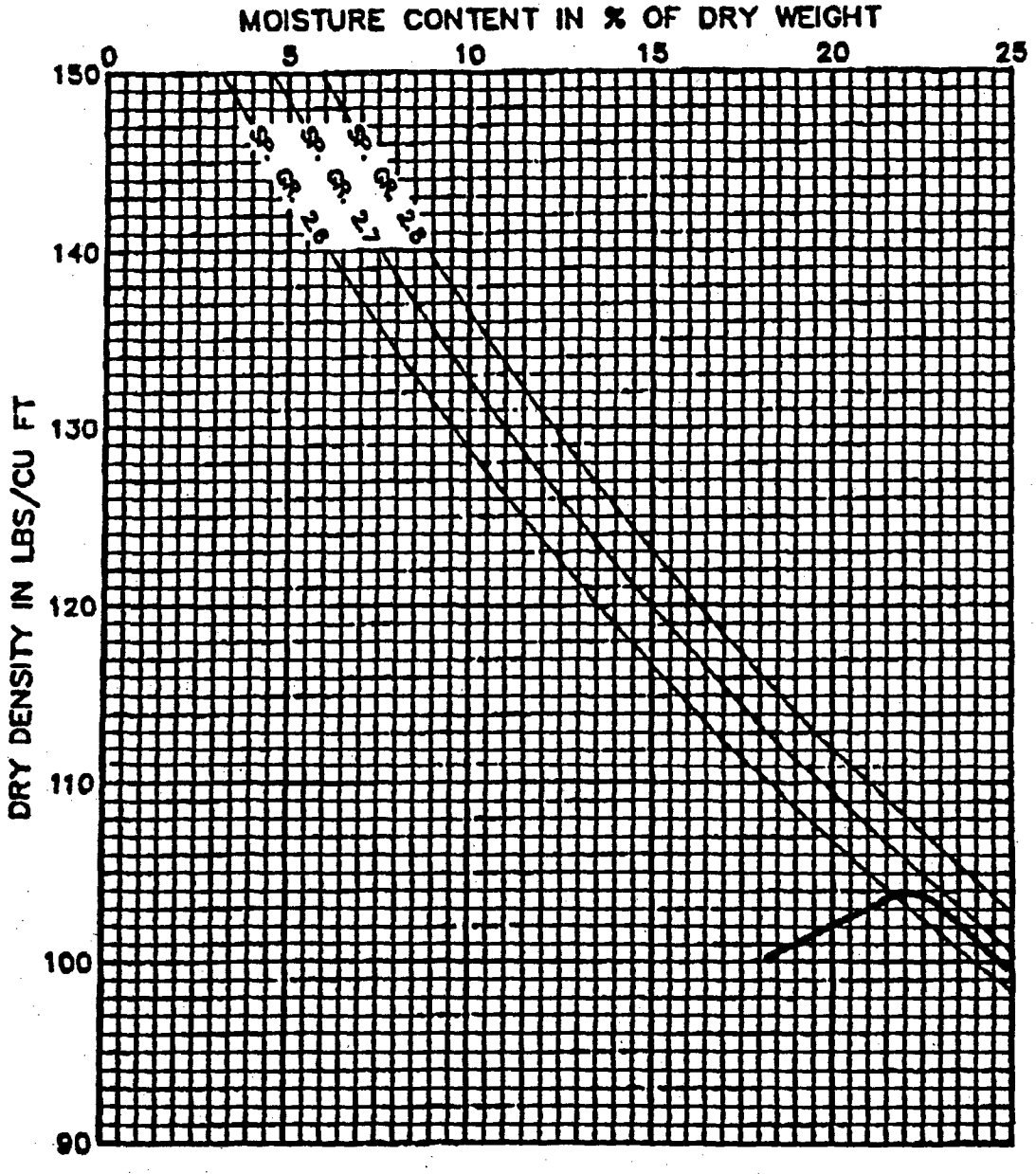


M.K. FERGUSON
COMPACTION TEST DATA

PLATE

WESTERN

SAMPLE NO. 2 DEPTH _____ ELEVATION _____
SOIL _____
LOCATION CHENEY DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 22.6%
MAXIMUM DRY DENSITY 103.8 PCF
METHOD OF COMPACTION ASTM D-698, METHOD A

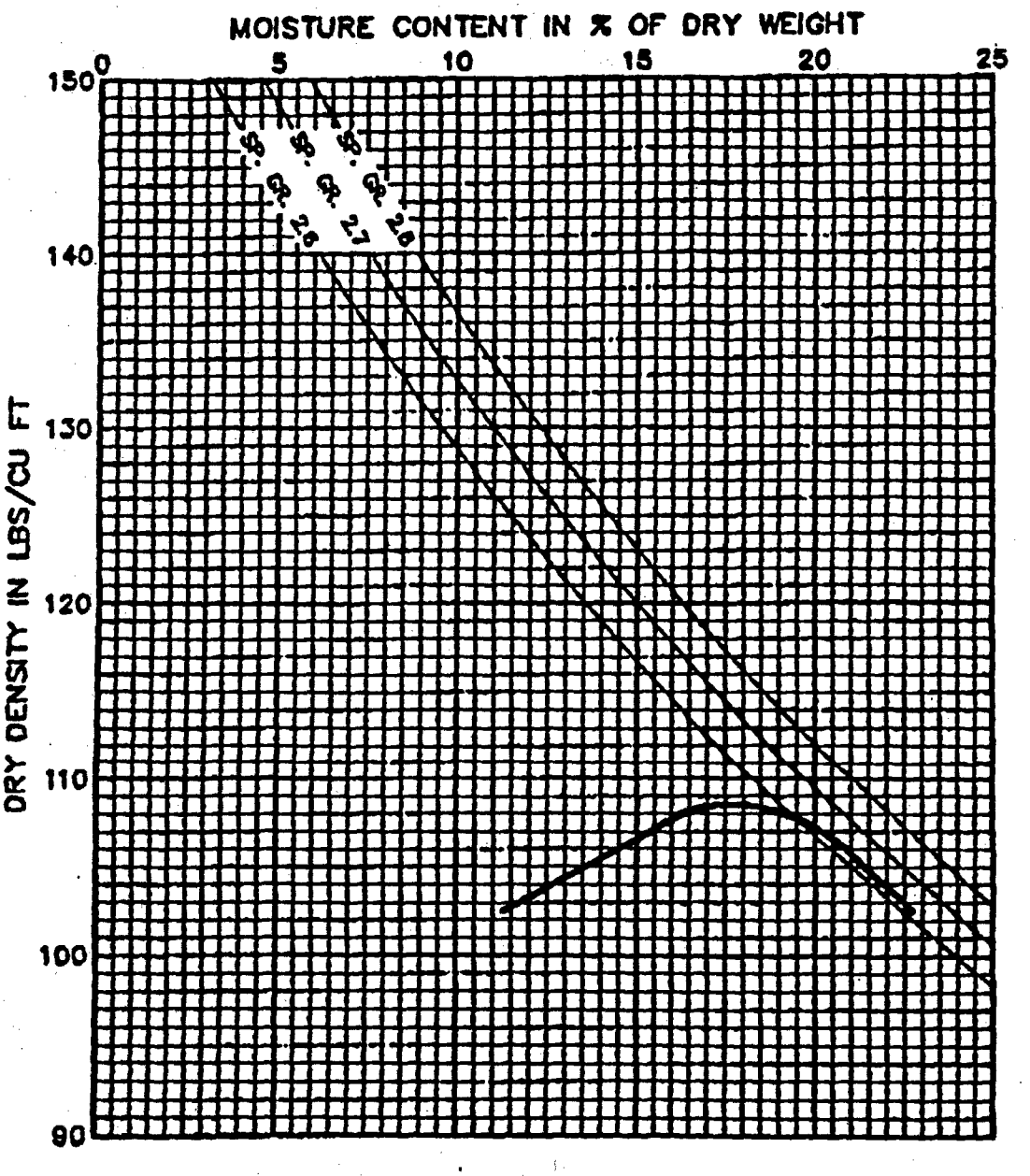


M.K. FERGUSON
COMPACTION TEST DATA

PLATE

3/14/90 WESTERN ENGINEERS, INC.

SAMPLE NO. 3 DEPTH _____ ELEVATION _____
SOIL _____
LOCATION CHENEY DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 17.5%
MAXIMUM DRY DENSITY 108.5 PCF
METHOD OF COMPACTION ASTM D-698, METHOD A

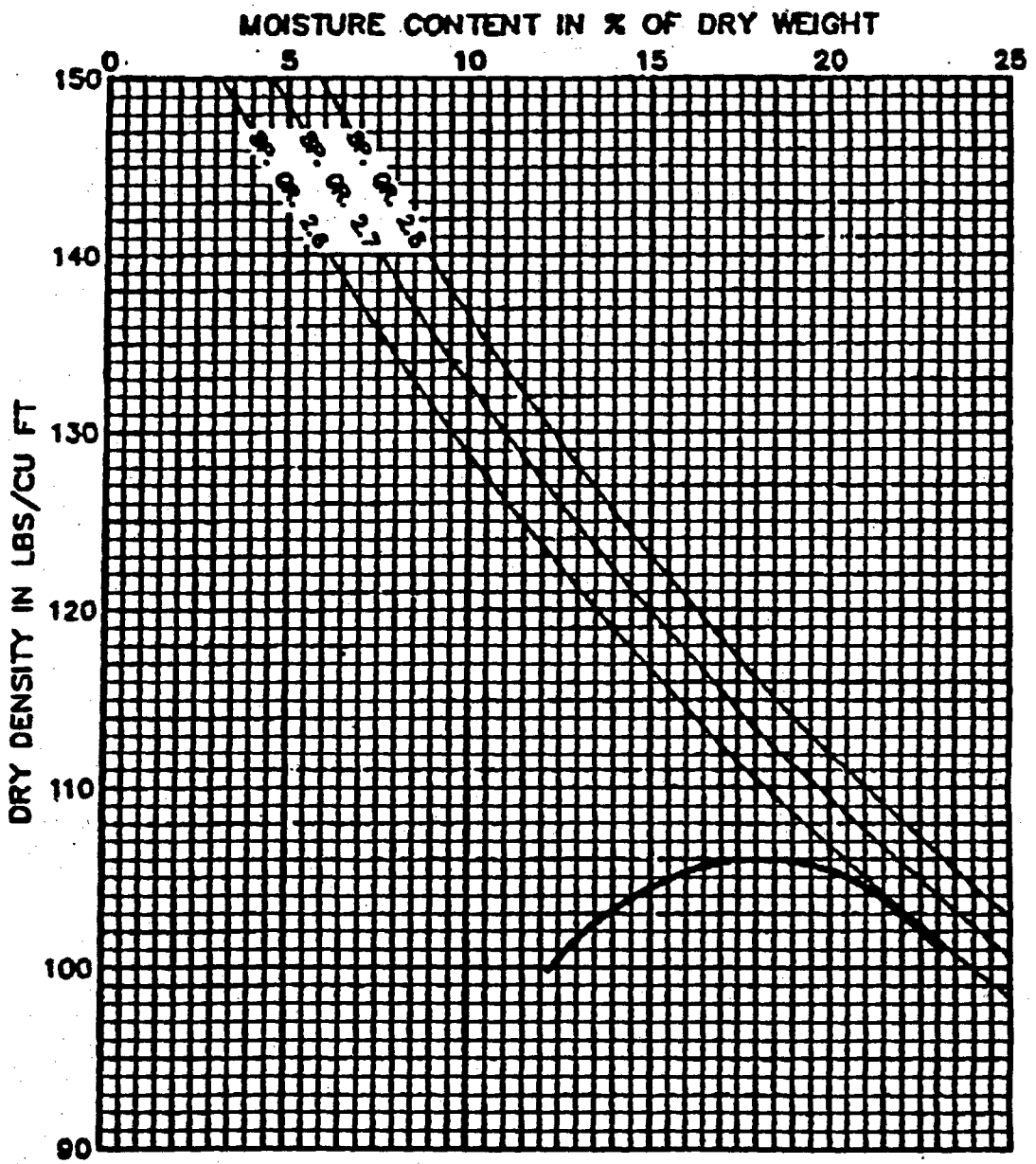


M.K. FERGUSON
COMPACTION TEST DATA

PLATE

3/14/90 WESTERN ENGINEERS, INC.

SAMPLE NO. 4 DEPTH _____ ELEVATION _____
SOIL _____
LOCATION CHENEY DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 18.2%
MAXIMUM DRY DENSITY 106.1 PCF
METHOD OF COMPACTION ASTM D-698, METHOD A

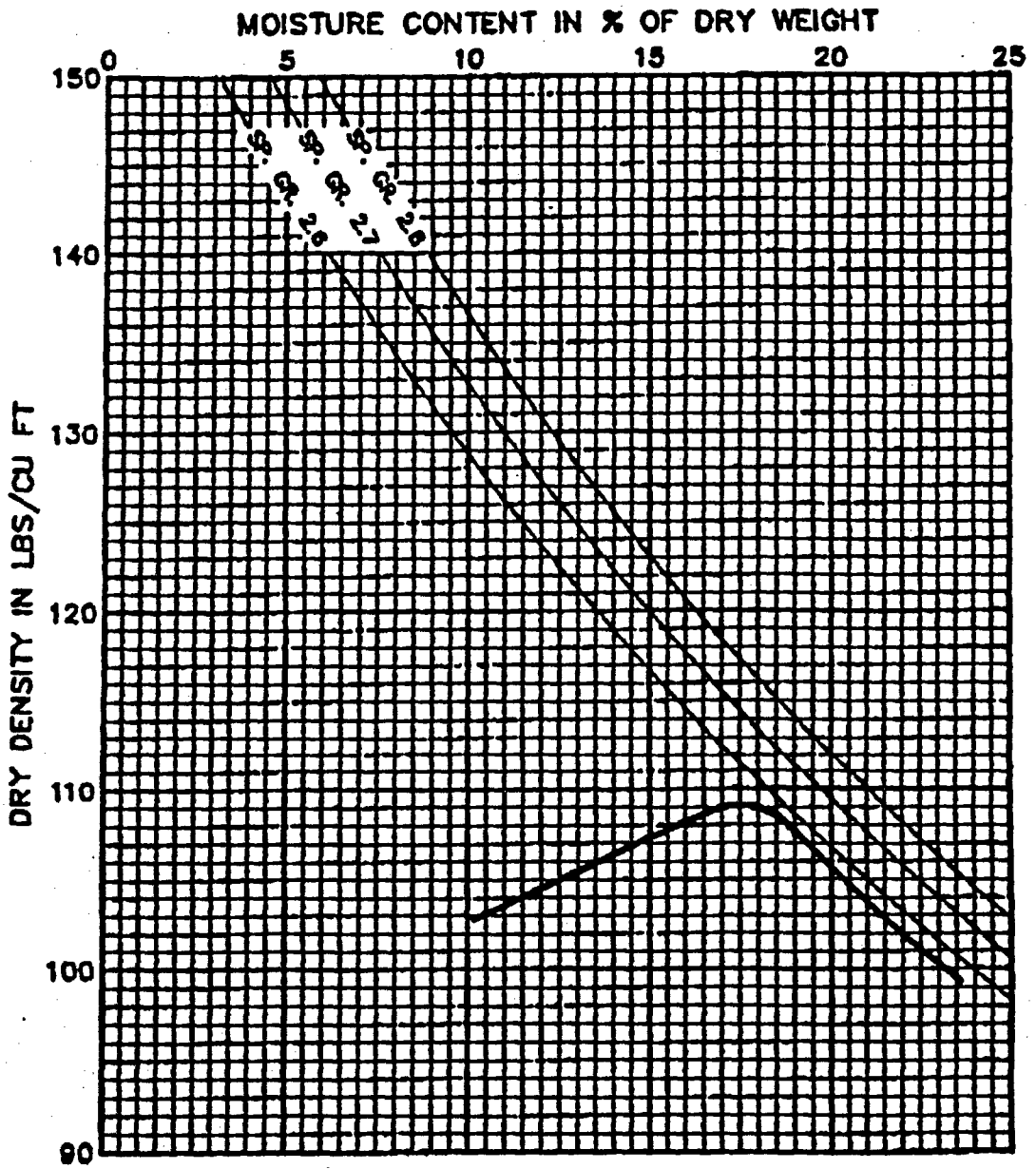


M.K. FERGUSON
COMPACTION TEST DATA

PLATE

3/14/90 WESTERN ENGINEERS, INC.

SAMPLE NO. 5 DEPTH _____ ELEVATION _____
SOIL _____
LOCATION CHENEY DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 17.5%
MAXIMUM DRY DENSITY 109.1 PCF
METHOD OF COMPACTION ASTM D-698, METHOD A

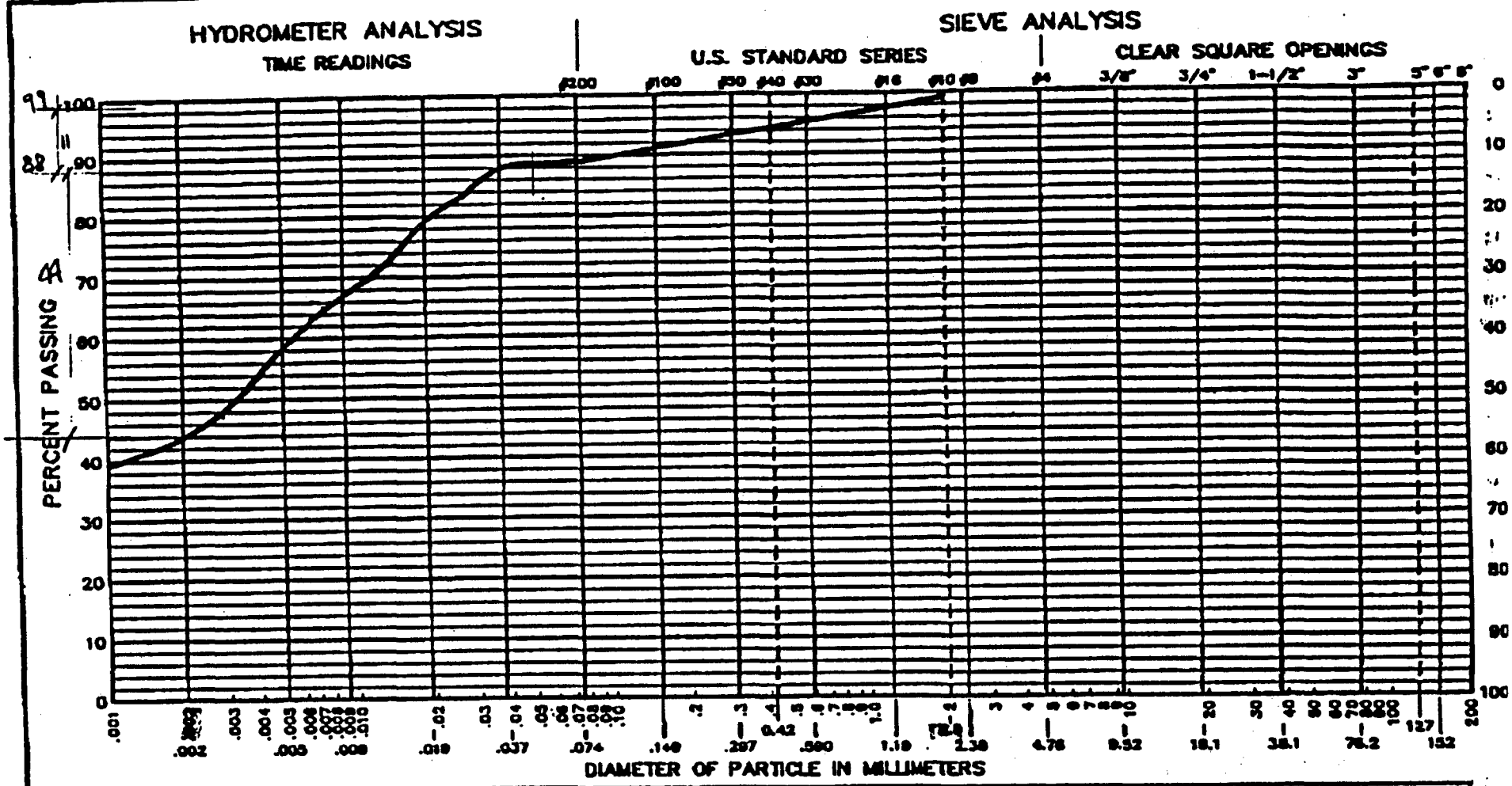


M.K. FERGUSON
COMPACTION TEST DATA

PLATE

9/14/90 WESTERN ENGINEERS, INC.

WESTERN ENGRS. FAX 303-242-1672



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES:

PERCENT PASSING #10 - 99.5

M.K. FERGUSON
CHENEY DISPOSAL SITE
GRADATION TEST

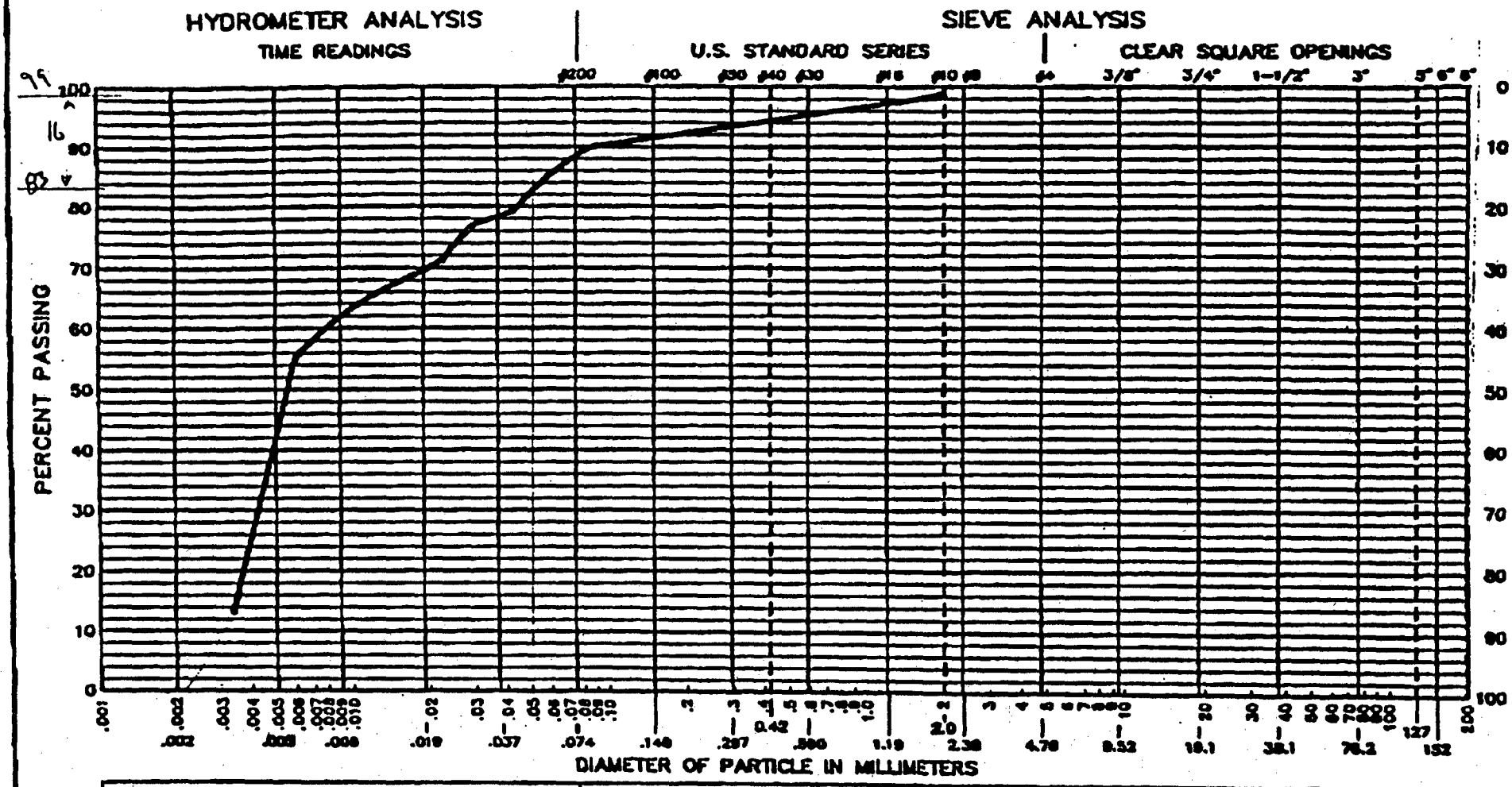
PLATE

WESTERN ENGINEERS, INC.

DESIGNATION SAMPLE #1 EXCAVATION NO. 1 DEPTH (FT.)

3/14

WESTERN ENGRS. FAX 303-242-1672



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES:

PERCENT PASSING #10 - 98.8

M.K. FERGUSON
CHENEY DISPOSAL SITE
GRADATION TEST

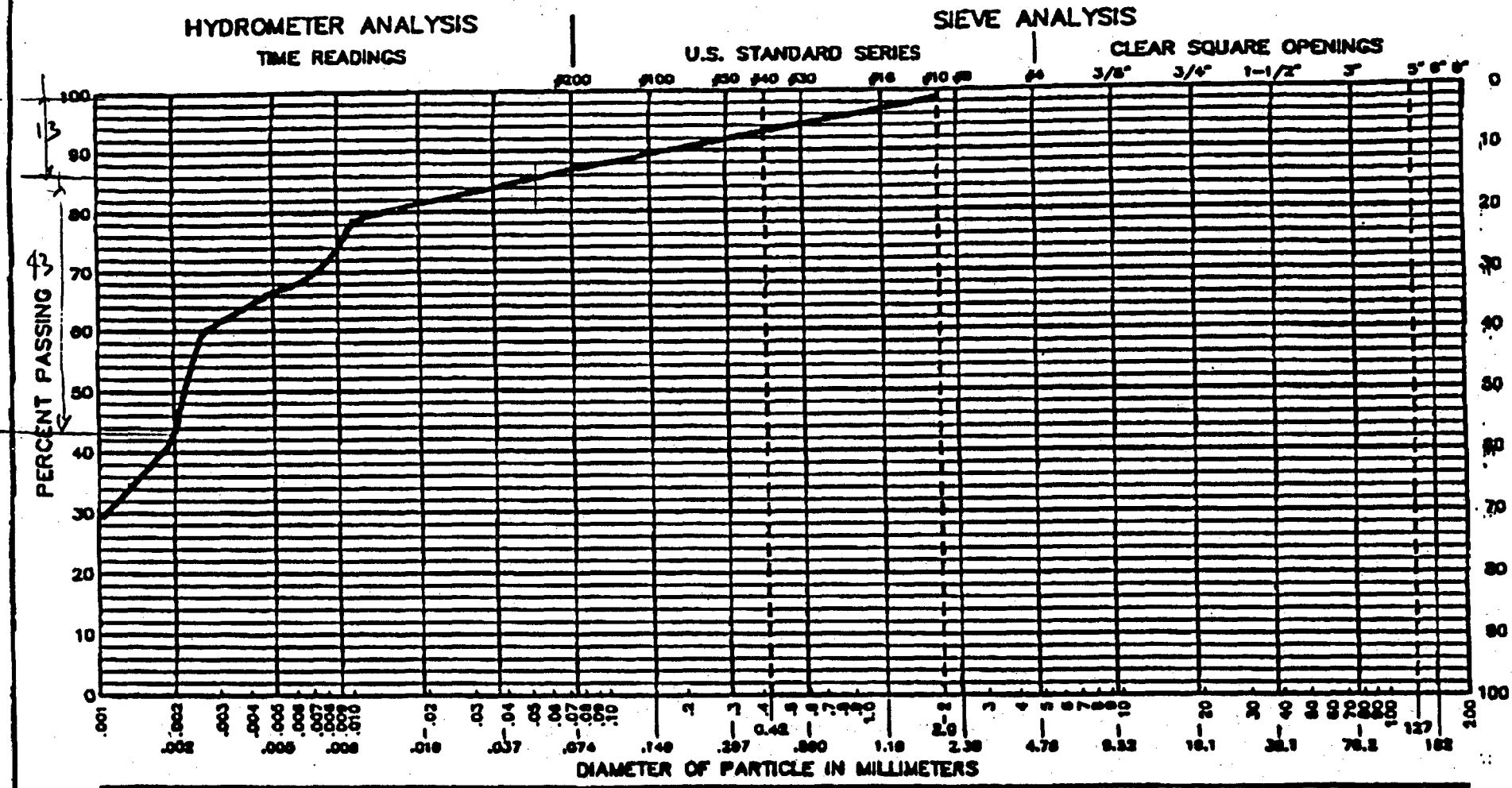
PLATE

WESTERN ENGINEERS, INC.

DESIGNATION SAMPLE #2 EXCAVATION NO. 2 DEPTH (FT.)

3/14

WESTERN ENGRS. FAX 303-242-1672



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES:

PERCENT PASSING #10 - 99.0

M.K. FERGUSON
CHENEY DISPOSAL SITE
GRADATION TEST

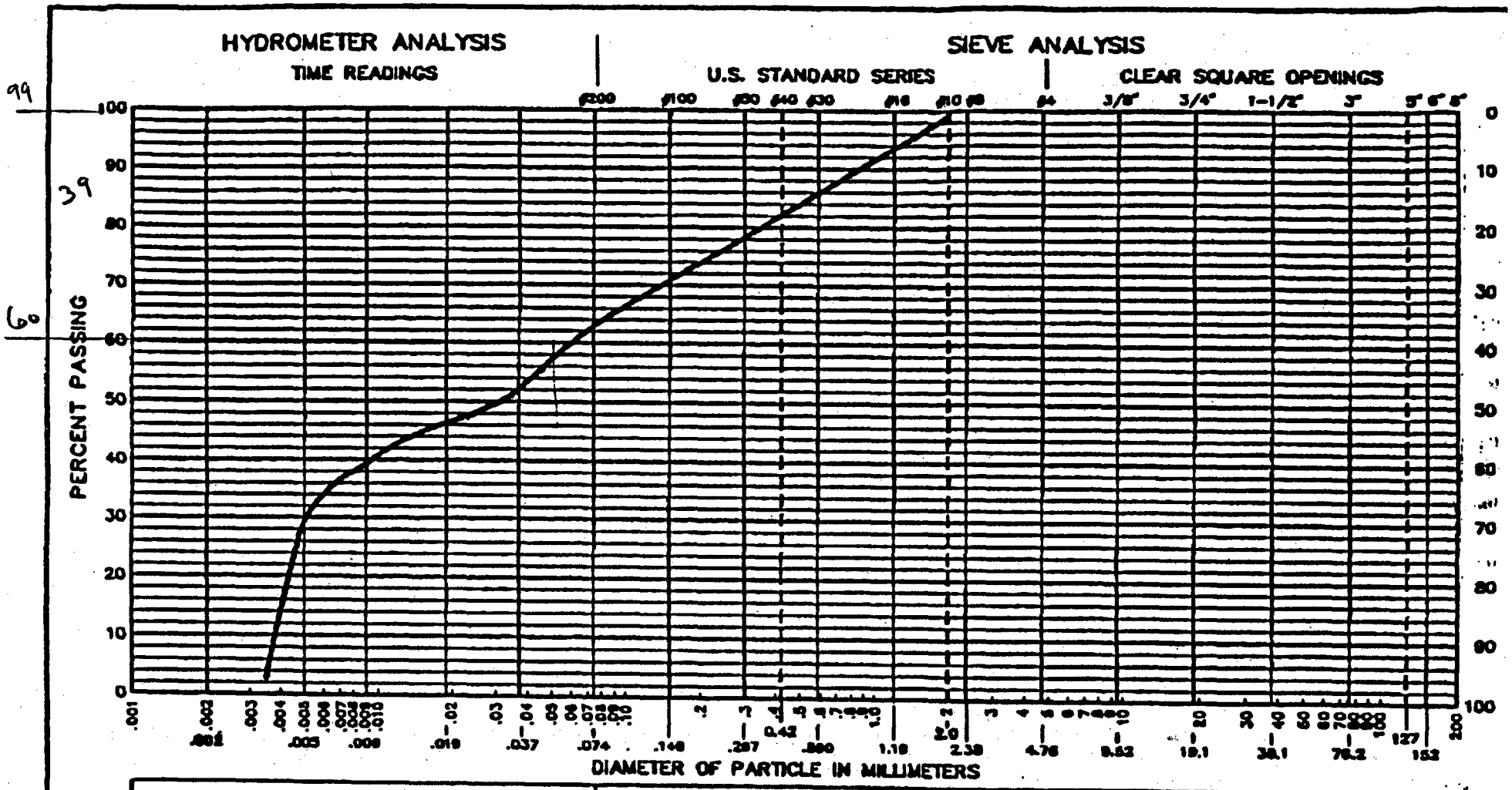
PLATE

WESTERN ENGINEERS, INC.

DESIGNATION SAMPLE #3 EXCAVATION NO. 3 DEPTH (FT.)

3/14/

WESTERN ENGRS. FAX 303-242-1672



NOTES:

PERCENT PASSING #10 - 99.1

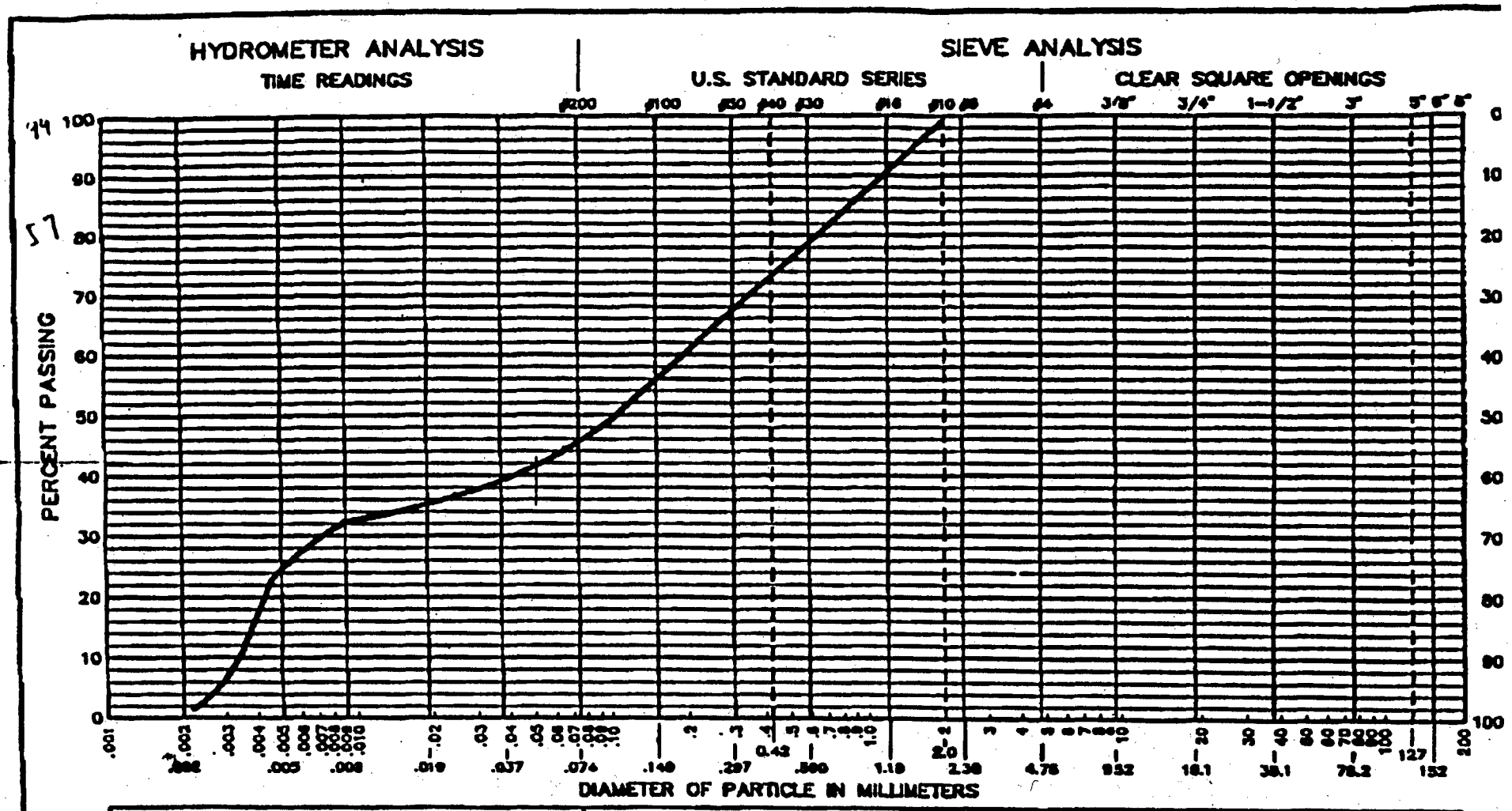
M.K. FERGUSON
CHENEY DISPOSAL SITE
GRADATION TEST

PLATE

WESTERN ENGINEERS, INC.

DESIGNATION SAMPLE #4 EXCAVATION NO. 4 DEPTH (FT.)

3/14/



NOTES:

PERCENT PASSING #10 - 99.3

M.K. FERGUSON
CHENEY DISPOSAL SITE
GRADATION TEST

PLATE

WESTERN ENGINEERS, INC.

DESIGNATION SAMPLE #5 EXCAVATION NO. 5 DEPTH (FT.)

3/14/

UMTRA-GRJ

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MESSAGE :

THANK YOU!

REVISION 3

RADON BARRIER MATERIAL PROPERTIES

The thickness of radon barrier will be 2 feet on the top slope and 3.5 feet on the side slope (Ref. 7). The radon barrier will be constructed from

- Layer 3 - Clayey Material
- Layer 2 - 1" material (see Sh. 294)

All radon barrier materials will be compacted to 100% Standard Proctor (ASTM D-698).

The available data from Layer 3 are:

- Grain Size
- Compaction Tests ASTM D-698
- Atterberg Limit
- Specific Gravity
- Natural Moisture Content (From Previous Tests)
- Permeability

Density, Moisture Content, and Specific gravity Data From Summary of Soil Tests on Sh. 172

(a) $\gamma_{d\ AVE.} = \frac{120.7+114.3+112.2+109.7+110.2+117.6+109.2+110.6+114.9+116.2+111}{11}$

$= 113.7$ pcf

(b) $W_{OPT.\ AV.} = \frac{12.5+15.5+16.1+17.1+16.5+14+12.9+16+14.2+13.9+15.7}{11}$

$= 14.9\%$

(c) Average Moist Density $= 113.7 \times 1.149 = 130.6$ pcf
Radon Barrier will be compacted 100%

(d) Specific gravity $= 2.78$

REVISION 3

(e) Saturated Density

$$\text{void ratio, } e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.78 \times 62.4}{113.7} - 1 = 0.526$$

$$\gamma_{ave.} = \frac{G_s \gamma_w + e \gamma_w}{1+e} = \frac{2.78 \times 62.4 + 0.526 \times 62.4}{1 + 0.526} = 135.2 \text{ pc}$$

$$w_{sat.} = \frac{Se}{G} = \frac{100\% \times 0.526}{2.78} = 0.189 \rightarrow w = 18.9\%$$

STRENGTH

The shear strengths are computed by indirect method. The proposed overburden will be from:

- 24" Rooting soil ($\gamma = 95$ pcf, Sh. 261)
- 15" Rock and 12" Drain Layer (use $\gamma = 125$, Sh. 272)
- 24" Radon Barrier ($\gamma = 130$ pcf, Sh. 240)

The overburden pressure is:

$$p_o' = 95(2) + 125(2.25) + 130(2) = 731 \text{ psf}$$

$LI = 0.26$ (from Sh. 168)

$p_{p_{AVE.}} = 6$ ksf (from Sh. 166)

$$OCR = \frac{6000}{731} = 8.21$$

$$A = \left(\frac{S_u}{p_o'}\right)_{NC} = 0.11 + 0.0037(PI) = 0.11 + 0.0037(19.3) = 0.181$$

(from Ref. 16, page 142)

$$B = \left(\frac{S_u}{p_o'}\right)_{OCR} \quad \text{(Ref. 14, p. 57)}$$

$$\frac{B}{A} = OCR^{0.8} \quad \text{for } OCR \leq 7$$

$$S_{u_{OCR}} = 0.181 \times 731 \times (8.21)^{0.8} = 713 \text{ psf} \simeq 750 \text{ psf}$$

REVISION 3

For short-term static and long term seismic : $S_u = 750$ psf

For Long-term static : $c = 280$ psf
$\phi = 28°$

(From Ref. 17 on Sheet 248)

Recommended Design Parameters

In-place density = 130.6 pcf ; $\gamma_{saturated} = 135.2$ pcf
Moisture Content = 14.9%
Shear Strength :
Short-term : $S_u = 750$ psf
Long-term : Static $c = 280$ psf
$\phi = 28$
Seismic $S_u = 750$ psf

FOR DESIGN ARE GIVEN ON SHEETS ~~136~~ 241 AND ~~137~~ 242

CONSOLIDATION TESTS - RADON BARRIER MATERIAL

NO CONSOLIDATION TESTS WERE PERFORMED ON COMPACTED LAYER 3 MATERIAL. However C_c and C_v values can be estimated from soil index properties. C_r will be negligable for small load above 95% compacted clay soil:

$C_c = 0.178$

C_r = Negligable for compacted clay

$P_p = 2.2$ KSF OCR = 3.7

$C_v = .026$ cm²/s FOR LL = 35.4

USE $P_p = 7.2$ KSF (FROM LI/sensitivity calc.)

NOTE: PRESENTED IN SHEET 138 A IS A SET OF TIME RATE OF SETTLEMENT CURVES FOR THE TRANSPORTED CLAY SAMPLE. THE 90 PCF CONSOLIDATION OCCURS AROUND 0.5 MIN = 30 S.

FOR A 1" THICKNESS AND DOUBLE DRAINAGE: $C_v = \frac{(1 \times 2.54)^2 \times .848}{4 \times 30} = .046$

THIS IS HIGHER THAN THE C_v RECOMMENDED, HOWEVER, THE PERMEABILITY (SHT 147) OF THIS SAMPLE IS THE HIGHEST, THEREFORE THE RECOMMENDED VALUE IS CHOSEN TO REPRESENT A MORE TYPICAL SAMPLE.

FROM Terzaghi & Peck, 1967: $C_c = .007(LL-10)$ for remolded soil

$C_c = .007(35.4-10) = .178$

~~139~~ ~~184~~ 24

SUMMARY OF SOIL TEST RESULTS

Job No 5025 Project Name UMTRA - GRAND JUNCTION Feature CHENEY RESERVOIR SITE Date ~~SAB~~ 1-12-89

RMB 1-18-89

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γ_d	Compaction Test	Compaction Optimum w	Compaction Optimum γ_d	In-Situ Shear Strength Test	Initial w	Initial γ_d	C / ϕ	C' / ϕ'	Permeability γ_d	Permeability k* cm/SEC	Consolidation C	Cc / Cr	K (KSF)	Notes
LAYER	NO. 3	& LAYER		NO 4																						
LAYER	NO 3																									
GCH-1		12.0	22.0	ML/CL																						
GCH-1		21.0	31.0	ML/CL																		4×10^{-6}				
GCH-2		20.0	31.0	CL																		1×10^{-6}				#501
GCH-3		14.0	28.0	ML																		3×10^{-6}				#502
GCH-4		22.0	34.0	ML																		1×10^{-7}				#503
GCH-5		18.0	28.0	ML																		5×10^{-6}				#504
701		15.0	16.0	CL	0	15	85															9×10^{-6}				#505
702		20.0	21.0	CL	3	12	85															9×10^{-6}				
615		7.0	8.0	CL			77	46	25		17.2															
ADDITIONAL		DATA	FROM	TEST PITS																						
TP-1		19.5	21.5	CL				42	24		8.4															
2		16.0	20.0	CL	5	23	72	28	14	2.71			SP	12.6	117.8						111.9	3.8×10^{-8}				
2	TUBE	18.0	18.5								4.8	101.3				CU	18.9		0 / 18.4	0 / 28.2						
3				CL				27	10		5.6												0.046	0.115 / 0.015	3.0	
3		15.5	20.0		5	37	58			2.71			SP	12.2	117.9						112.0	8.2×10^{-8}				
4	TUBE	19.0	20.0								6.7	101.7				UU	23.3		27.2 / 0							
6		15.5	17.0	CL	4	6	90	42	27	2.74			SP	15.8	113.0						107.4	8.5×10^{-9}				
6	TUBE	17.0	17.7								7.9	117.9				UU	22.6		76.3 / 0							
8		15.5	20.0		6	35	59																			
9		18.5	20.0	CL	0	11	89	42	20		11.9															
9	TUBE	20.0	20.7								11.9	108.7									107.7	1.2×10^{-8}				
10		13.5	20.5	CL	32	25	43	34	19	2.76			SP	15.1	115.9						110.1	3.3×10^{-8}				
11		18.5	20.0	CL	12	10	78	46	26	2.72	11.2		SP	15.5	111.9						106.3	2.7×10^{-8}				
11		19.0	19.7																		118.4	3.6×10^{-9}				
AVERAGE	LAYER 3			CL	7	19	74	38	21	2.73	9.5	107.4		14.2	115.3							2.2×10^{-6}				
LAYER	NO 4																									
GCH-4		35.0	44.0	CL/GC				34	16		10.4															
GCH-5		23.0	35.0	ML/GC																		6×10^{-5}				#504
GCH-6		21.0	29.0	CL/GC				23	5		5.0											1×10^{-5}				#505
701		23.0	24.0	CL/GC	30	11	59															1×10^{-5}				#506
702		25.0	26.0	GC	55	31	14																			
AVERAGE	LAYERS 3 & 4				14	20	67	36	19	2.73	9.2	107.4		14.2	115.3							1.5×10^{-6}				

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

Note: Material from this Layer will be compacted and used as radon barrier.

* SOIL COMPACTED TO ≈ 95% $\gamma_{d\,MAX}$ STANDARD PROCTOR

SHT ~~190~~ 245

RADON BARRIER MATERIALS

SUMMARY OF SOIL TEST RESULTS

Job No 5025 Project Name UMTRA - GRAND JUNCTION Feature CHENEY RESERVOIR SITE Date SLB 1-12-89

RMB 1-19-89

Hole or Trench Number	Sample Number	Depth From	Depth To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γ_d	Compaction Test	Compaction Optimum w	Compaction Optimum γ_d	Shear Strength Test	Initial w	Initial γ_d	C	ϕ	Permeability γ_d	Permeability k CM/SEC	Consolidation C_c	Consolidation C_r	Consolidation P_c	Notes
-1" FRACTION FROM LARGE SCALE SIEVE SAMPLES FROM LAYER 2																										
			--	--																						
	LSS-1				29	46	25	25.9	5.5	2.78			SP	(1) 12.0	121.2						119.7	5.83×10^{-6}				8 PCT. CLAY
													SP	(2) 10.5	125.5						107.0	8.65×10^{-5}				
	LSS-2				24	32	44	28.5	11.1	2.83			SP	14.0	117.7											11 PCT. CLAY
													SP	12.2	120.5											
	LSS-3				22	39	39	26.8	9.3	2.81			SP	13.3	115.6						120.2	8.91×10^{-8}				12 PCT. CLAY
													SP	10.0	126.5						108.2	2.67×10^{-5}				
	LSS-X	(3)											SP	14.0	113.8						112.6	9.4×10^{-7}				
													SP	13.2	118.2											

*Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

(1) - #4 FRACTION
(2) - FULL SAMPLE
(3) LSS-1 MIXED WITH 5 PERCENT BENTONITE

~~197~~ 246

PS 2/4/88

WYL 2/5/88

From: Ref 14, page 57

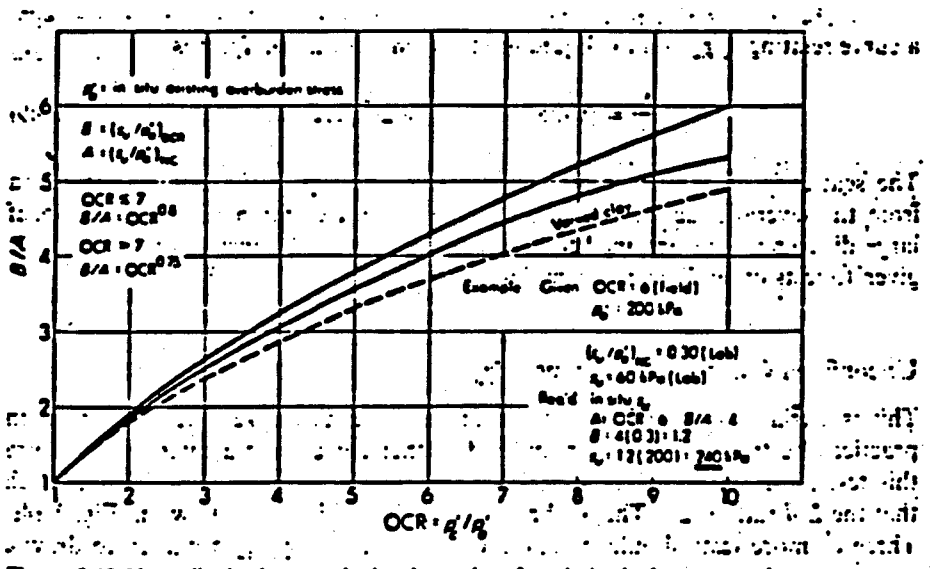


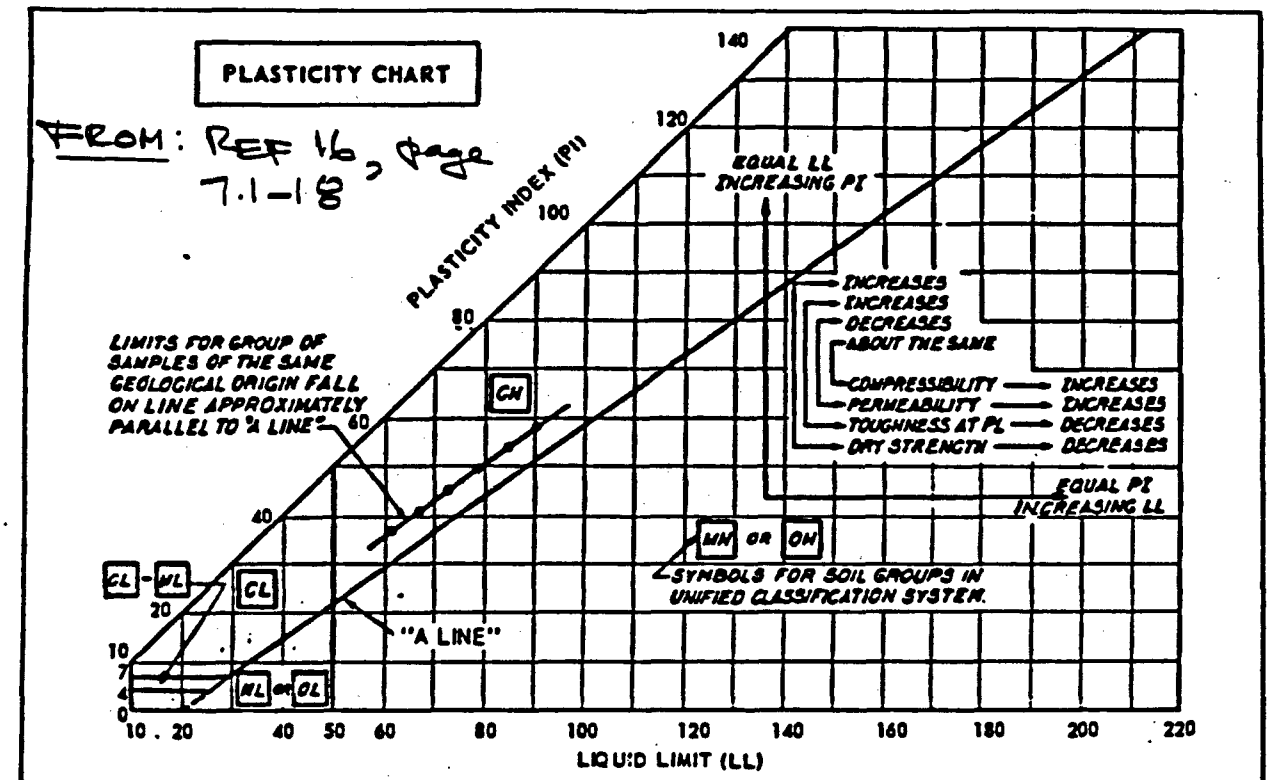
Figure 2-18 Normalized plot to obtain the ratio of undrained shear strength at some preconsolidation pressure to a normally consolidated value for several OCR. [*After Ladd et al.* (1977).] Note that above plot is based on direct simple shear (DSS) tests.

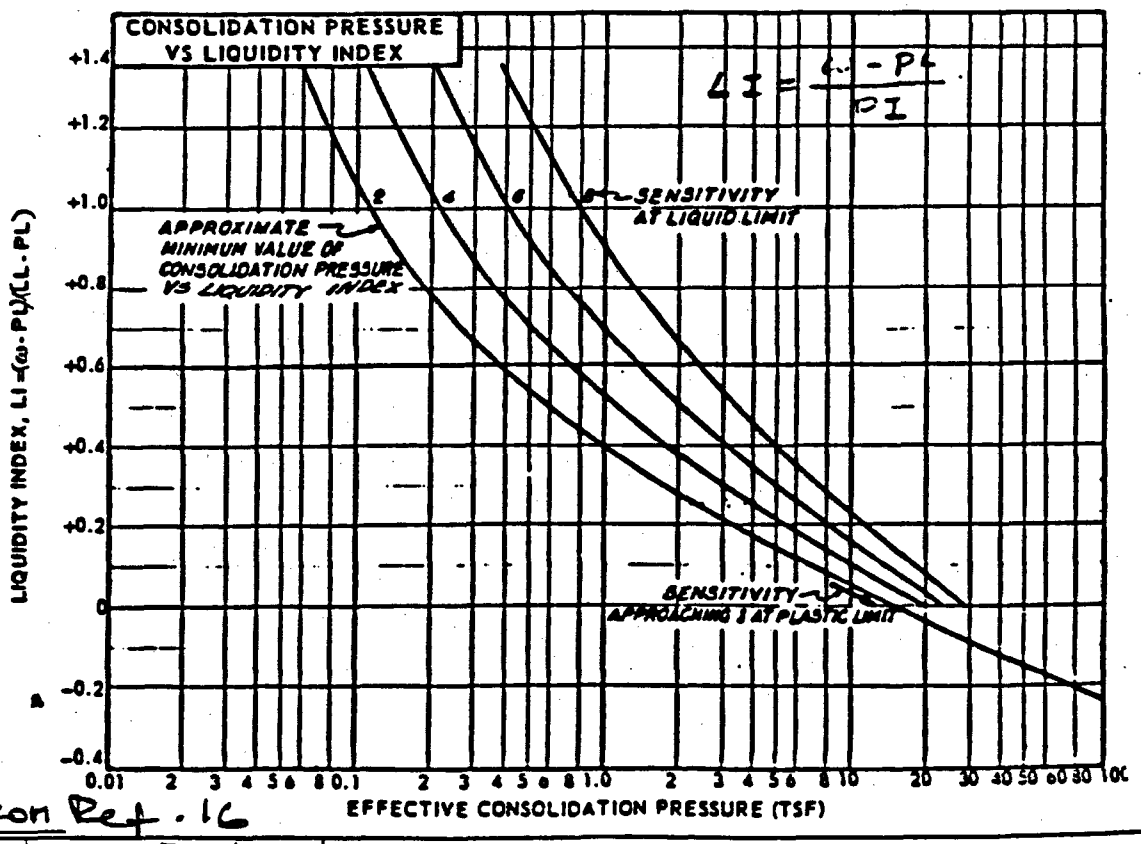
obtain this curve all had high w_N and w_L, but with five clays involved a trend seems to be established so that the curve might be used for clays with lesser w_N and w_L (less than 50 percent). In order to use this plot it is necessary to obtain an undisturbed sample for consolidation testing. From the consolidation test obtain p'_c. Remold a sample and K_o consolidate it to the present in situ p'_o state. Obtain s_u from this sample. Now with s_u, p'_o, and p'_c one can compute the OCR, enter Fig. 2-18, and obtain the ratio B/A. Since A is known, we can compute B and using B compute s_u at the present OCR as

$$s_{u,OCR} = B(p'_o)$$

Foundation Analysis And Design / Bowles

~~192~~ 247
PS 2/4/88
WYL 2/5/88





From Ref. 16
page 7.1-142

UMTRA-GRJ
5025-16
MATERIAL PROP. REV. 3 H.L. 6/19/90
6-2590

Sh. 248

SHEAR STRENGTH OF MATERIALS - Continued

Table 2. - Average effective shear strength parameters of compacted soils

Unified classification	Soil type	Standard Proctor Compaction† Maximum dry density lb/ft³	Optimum moisture content %	As compacted cohesion C_o' t/ft²	Saturated cohesion C_{sat}' t/ft²	Friction angle ϕ deg
GW	Well-graded clean gravels, gravel-sand mixture	>119	<13.3	*	*	>38
GP	Poorly graded clean gravels, gravel sand mixture	>110	<12.4	*	*	>37
GM	Silty gravels, poorly graded gravel-sand-silt	>114	<14.5	*	*	>34
GC	Clayey gravels, poorly graded gravel-sand-clay	>115	<14.7	*	*	>31
SW	Well-graded clean sands, gravelly sands	119±5	13.3±2.5	0.41±0.04	*	38±1
SP	Poorly graded clean sands, sand-gravel mixture	110±2	12.4±1.0	0.24±0.06	*	37±1
SM	Silty sands, poorly graded sand-silt mixture	114±1	14.5±0.4	0.53±0.06	0.21±0.07	34±1
SM-SC	Sand-silt-clay with slightly plastic fines	119±1	12.8±0.5	0.21±0.07	0.15±0.06	33±3
SC	Clayey sands, poorly graded sand-clay mixture	115±1	14.7±0.4	0.78±0.16	0.12±0.06	31±3
ML	Inorganic silts and clayed silts	103±1	19.2±0.7	0.70±0.10	0.09±*	32±2
ML-CL	Mixtures of inorganic silts and clays	109±2	16.8±0.7	0.66±0.18	0.23±*	32±2
CL	Inorganic clays of low to medium plasticity	108±1	17.3±3	0.91±0.11	0.14±0.02	28±2
OL	Organic silts and silty clays of low plasticity	*	*	*	*	*
MH	Inorganic clayey silts, elastic silts	82±4	36.3±3.2	0.76±0.31	0.21±0.09	25±3
CH	Inorganic clays of high plasticity	94±2	25.5±1.2	1.07±0.35	0.12±0.06	19±5
OH	Organic clays and silty clays	*	*	*	*	*

* Denotes insufficient data, > is greater than, < is less than (1 lb/ft³ = 16.02 kg/m³, 1 t/ft² = 95.8 kPa).
† Standard Proctor refers to ASTM D-698 method of test.

Table 2, Par. 4.11

From: Ref 17: Design standards Embankment Dams No. 13, Chapter 4 Static Stability Analysis, USBR, Engineering and Research Center, Denver, Colorado, August 1987

From Ref. 16, page 7.1-144 PS 2/9/88 249
WYL 2/5/88

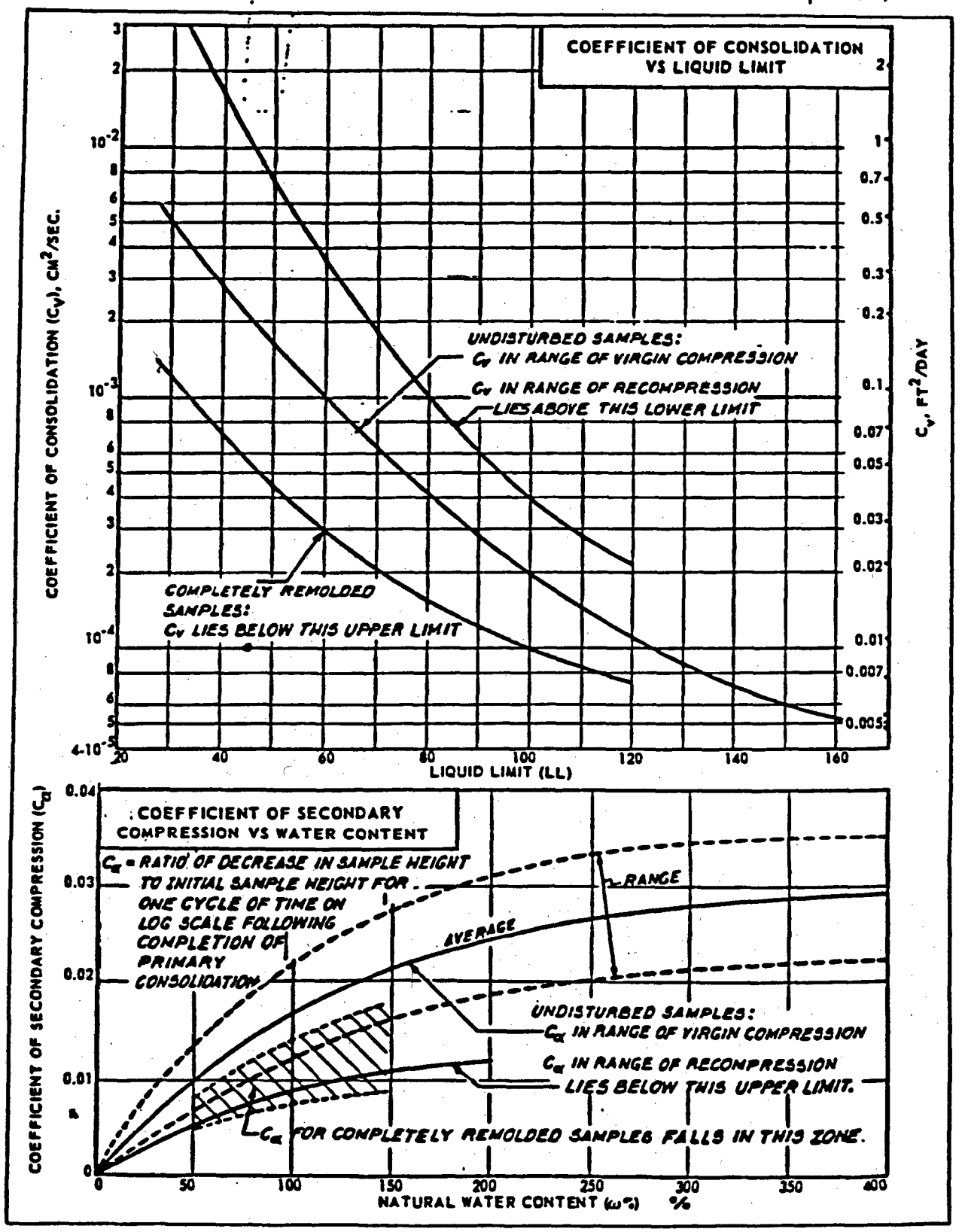


Figure 4

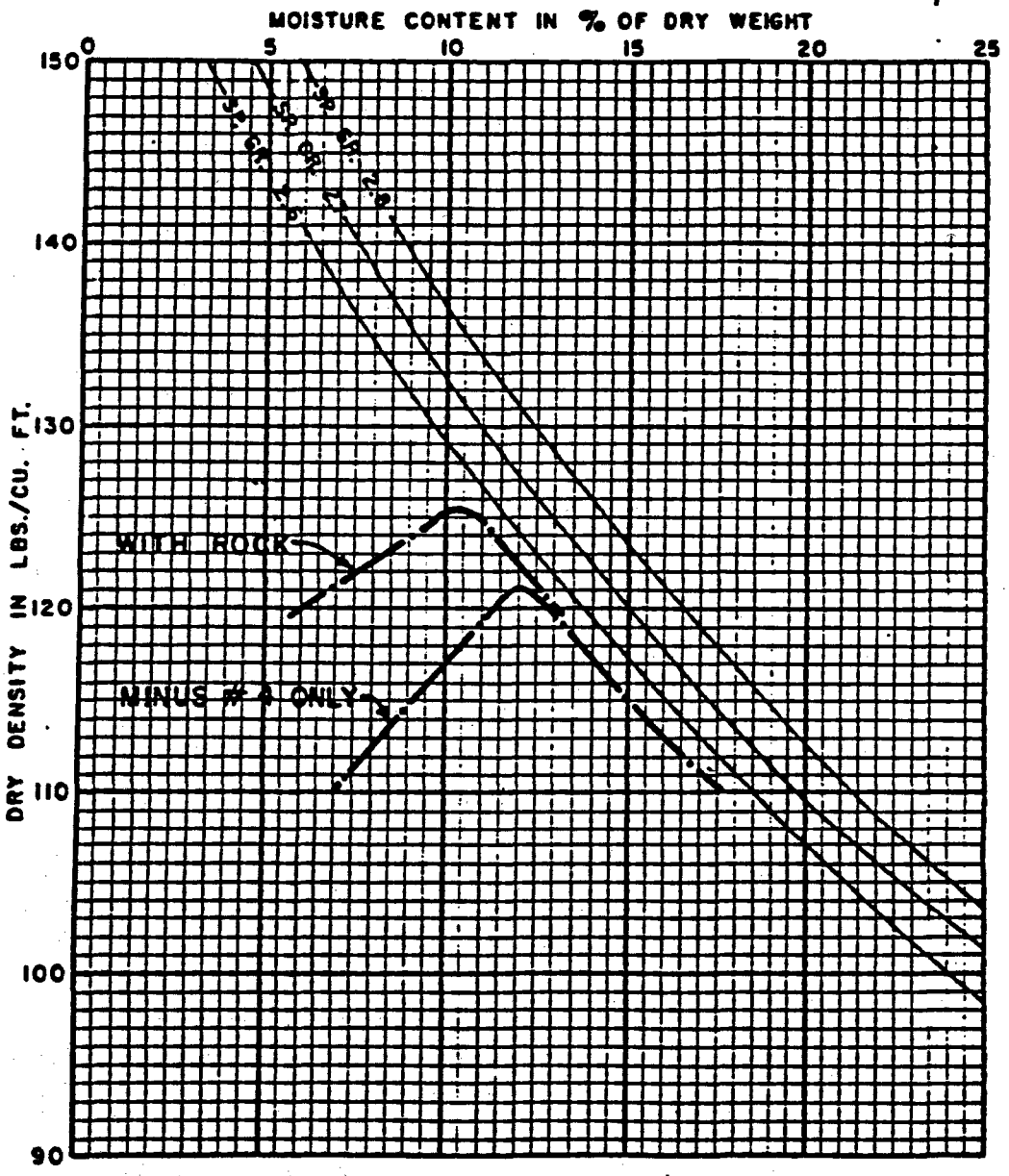
Approximate Correlations for Consolidation Characteristics of Silts and Clays

7.1-144

178 145 250

LARGE SCALE
SAMPLE NO. 1 DEPTH — ELEVATION —
SOIL MINUS 1" FRACTION
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 12.0% (-#4) 10.5% (ROCK)
MAXIMUM DRY DENSITY 121.2 pcf (-#4) 125.5 pcf (ROCK)
METHOD OF COMPACTION ASTM D698 METHODS A & D

PS 2/4/88
WYL 2/5/88



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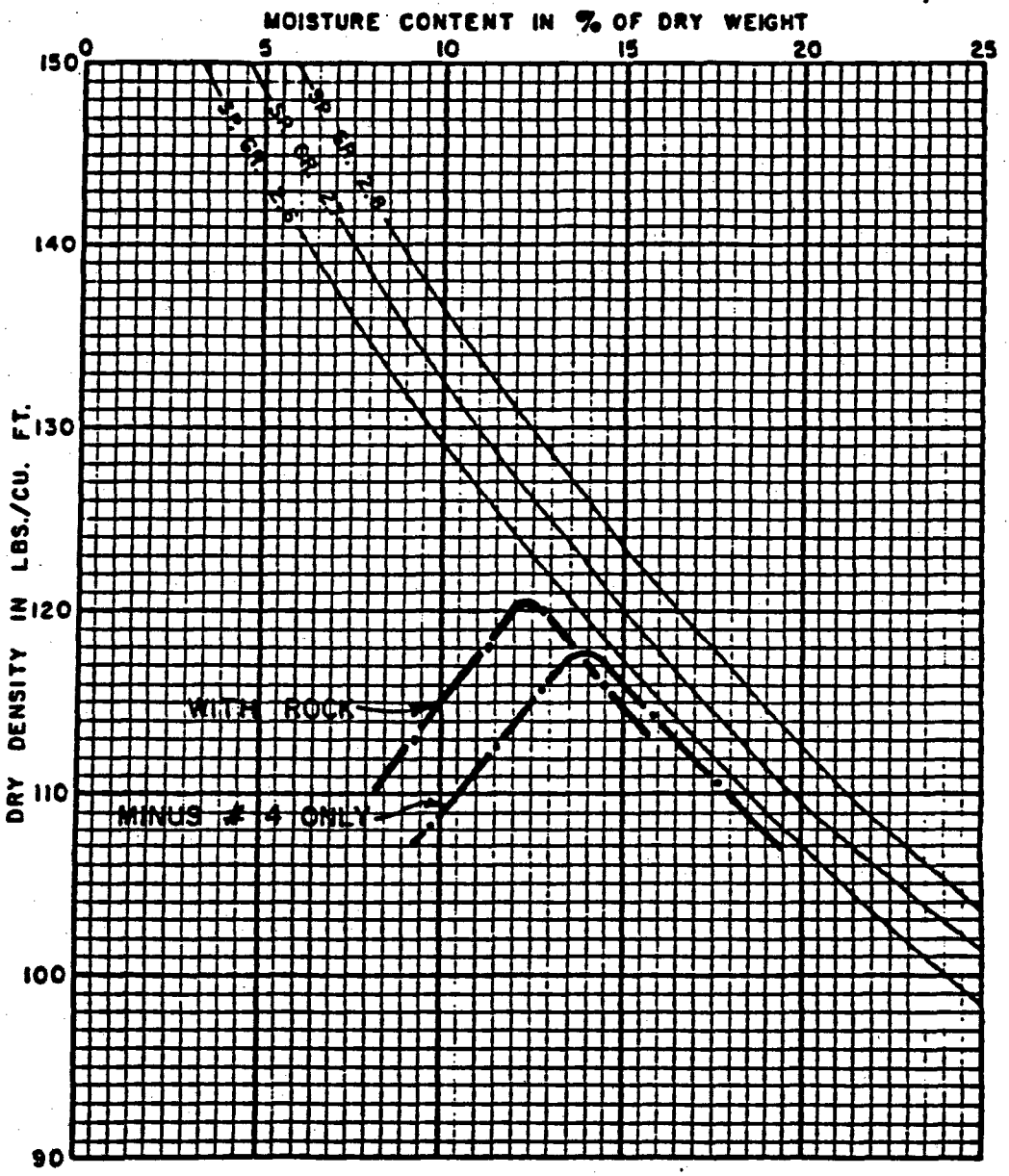
COMPACTION TEST DATA

W/O 1851

PLATE 1

LARGE SCALE
SAMPLE NO. 2 DEPTH — ELEVATION —
SOIL MINUS 1" FRACTION
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 14.0% (-#4) 12.2% (ROCK)
MAXIMUM DRY DENSITY 117.7 pcf (-#4) 120.5 pcf (ROCK)
METHOD OF COMPACTION ASTM D698 METHOD A

PS 2/9/88
WYL 2/5/88

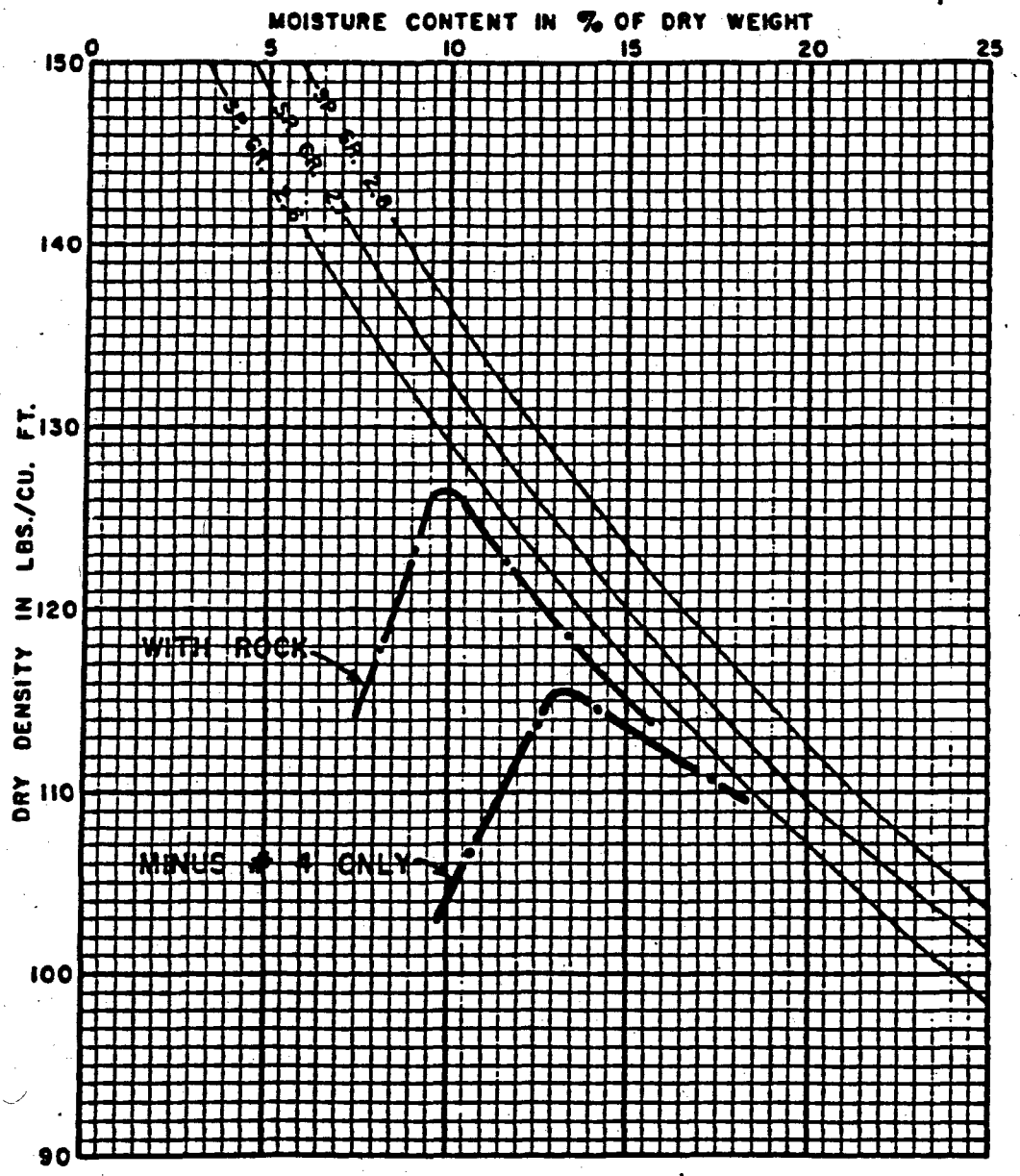


COMPACTION TEST DATA

~~180~~
197
252

LARGE SCALE
SAMPLE NO. 3 DEPTH — ELEVATION —
SOIL MINUS 1" FRACTION
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 13.3% (-#4) 10.0% (ROCK)
MAXIMUM DRY DENSITY 115.6 pcf (-#4) 126.5 pcf (ROCK)
METHOD OF COMPACTION ASTM D698 METHOD C

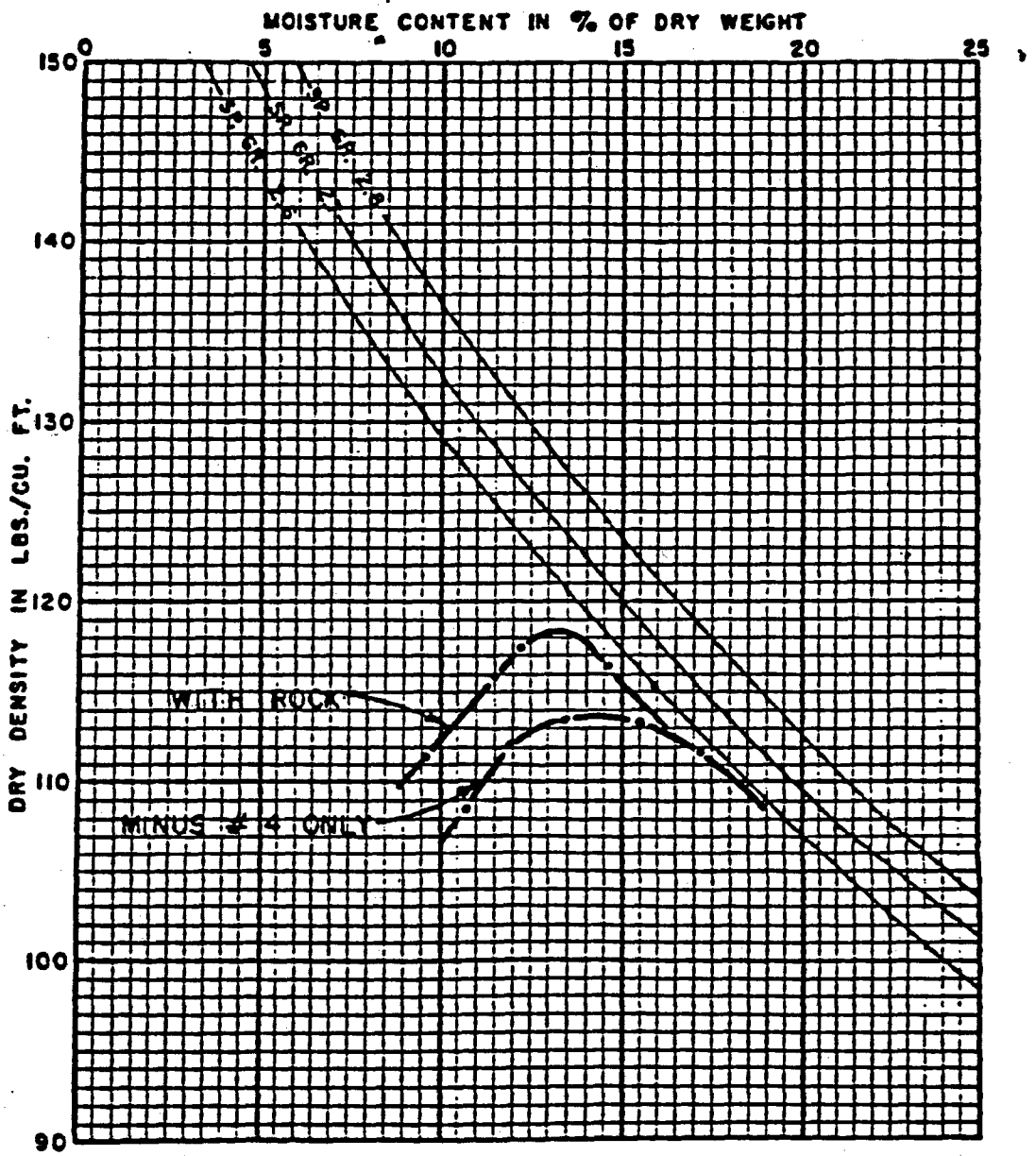
PS 2/9/88
WYL 2/5/88



COMPACTION TEST DATA

253 ~~198~~ ~~185A~~
PS 2/5/88

LARGE SCALE
SAMPLE NO. 1 DEPTH _____ ELEVATION _____
SOIL MINUS 1" FRACTION WITH 5% BENTONITE WYL 2/5/88
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 14.0%(-#4) 13.2%(ROCK)
MAXIMUM DRY DENSITY 113.8pcf(-#4) 118.2pcf (ROCK)
METHOD OF COMPACTION ASTM D 698 METHOD D



Note: The percentage of bentonite was calculated, based on the weight of the entire sample with rock

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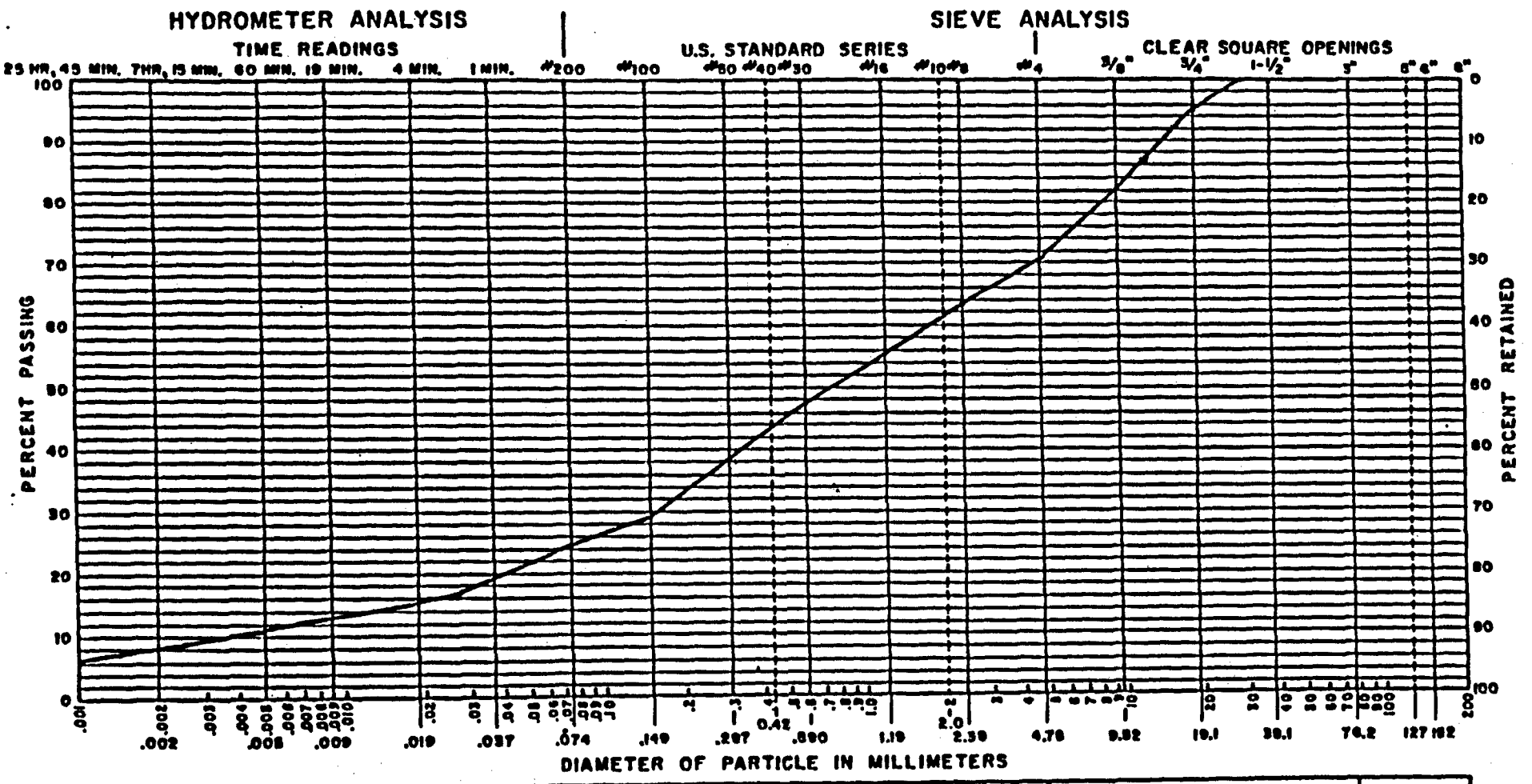
UMTRA-S.E.

COMPACTION TEST DATA

PLATE



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES: Liquid limit: 25.9
Plasticity index: 5.5

CHENEY SITE

GRADATION TEST

PLATE

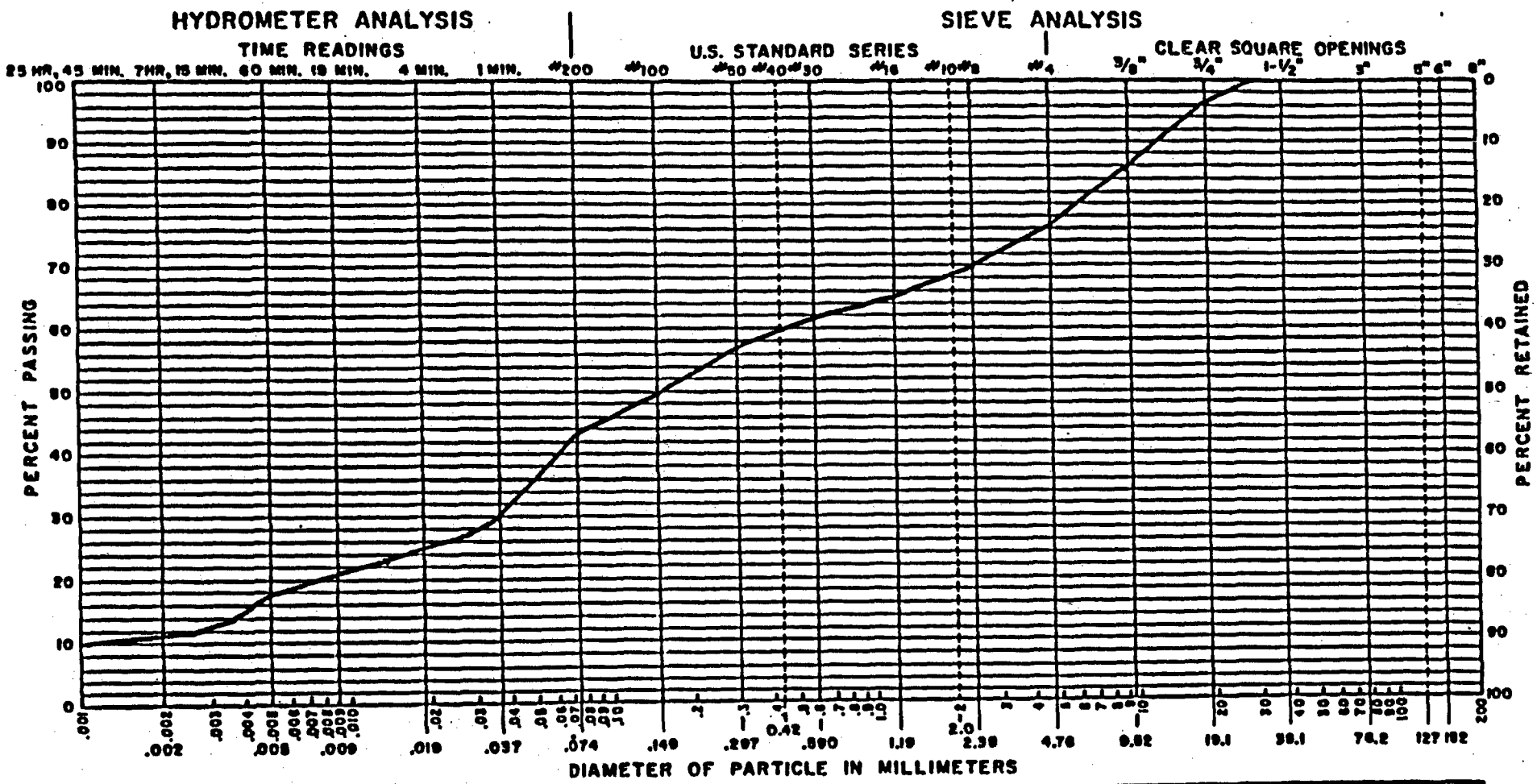
PS 2/4/88
WYL 2/5/88

Large Scale Sieve Sample No. 1 , 1" minus fraction

DESIGNATION _____ EXCAVTION NO. _____ DEPTH (FT.) _____

WESTERN ENGINEERS INC.

MKE DOCUMENT NO. 5025-GRJ-C-09-01271-00 254



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES: Liquid limit: 28.5
plasticity index: 11.1

CHENEY SITE

GRADATION TEST

PLATE

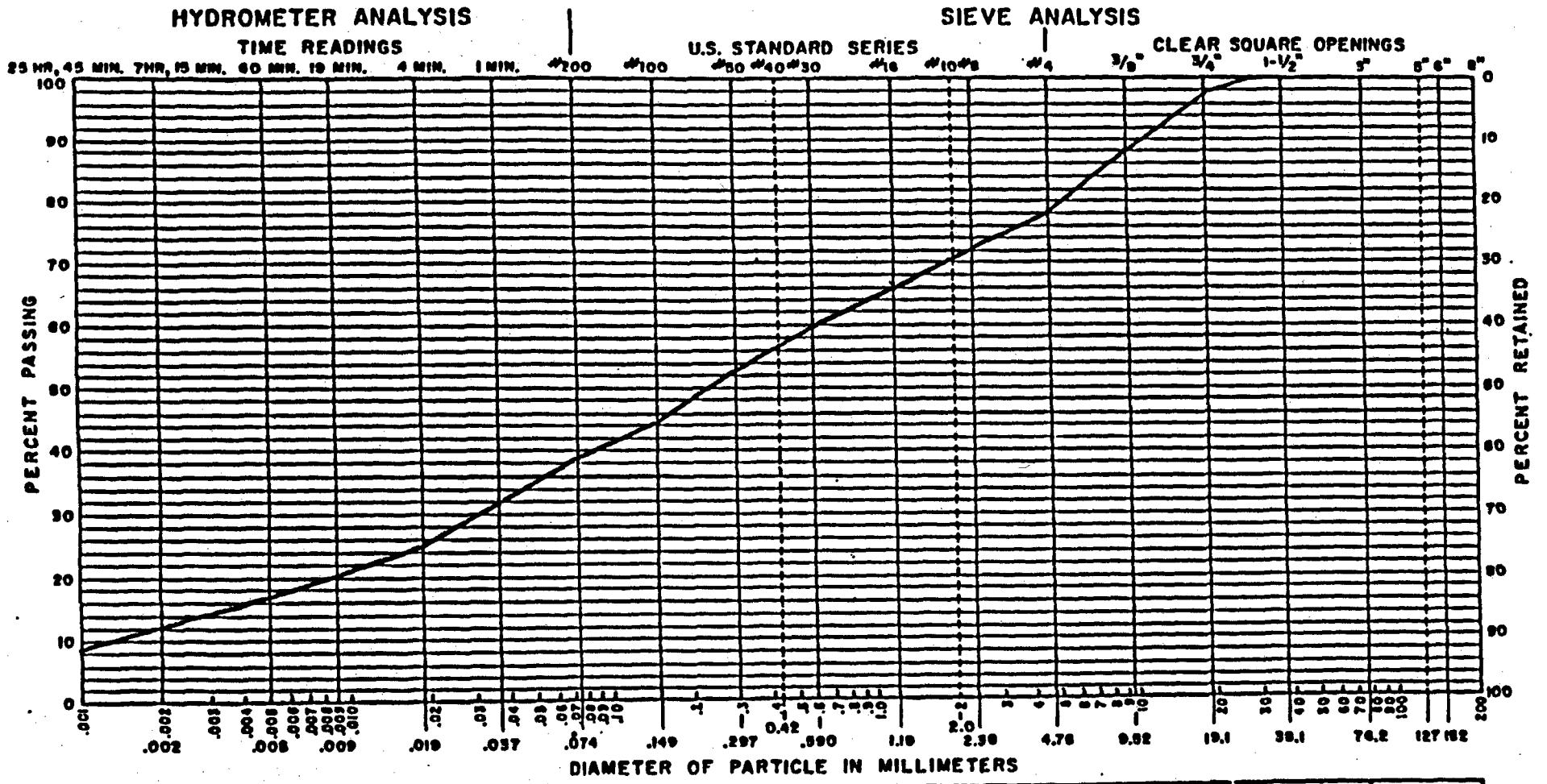
WESTERN ENGINEERS, INC.

Large Scale Sieve Sample No. 2, 1" minus fraction

DESIGNATION _____ EXCAVTION NO. _____ DEPTH (FT.) _____

WL 2/5/88
PS 2/4/88

255



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES: liquid limit: 26.8
plasticity index: 9.3

CHENEY SITE

GRADATION TEST

PLATE

WYL 2/4/88

Large Scale Sieve Sample No. 3, 1" minus fraction

DESIGNATION _____ EXCAVTION NO. _____ DEPTH (FT.) _____

WESTERN ENGINEERS INC.

252

MKE DOCUMENT ...5025-GRJ-C-09-01593-00

PS 2/6/88

WYL 2/6/88

195A
302
257

January 20, 1988
Cheney Reservoir
Disposal Site,
UMTRA, Grand Junction,
Colorado
Job No. 1 1017 87

CHEN AND ASSOCIATES

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Sample No.	Test Pit	Depth, Feet	Calcium Carbonate Eqiv., %	Organic Content, %	Shrinkage Limit, %	Specific Gravity of "-#4" Material
1	TP-3	0-4.5	27.4	0.57	9.4	2.75
2	TP-4	0-2.5	24.4	0.54	—	2.72
3	TP-6	0-3	15.3	—	—	2.78
4	TP-10	0-2.5	14.8	—	10.0	2.73
5	#1	—	—	0.18	13.1	2.78*
6	#2	—	—	—	—	2.83*
7	#3	—	—	0.21	14.0	2.81*

* These samples contained appreciable amounts of "+#4" material therefore this specific gravity may not be representative of the total sample.

SUMMARY OF PERMEABILITY TEST RESULTS

Cheney Reservoir Site, SCN.06, Item 002 & SCN.09, Item 001

Test No.	Sample	Initial Moisture (%)	Final (Saturated) Moisture (%)	Initial Dry Density (pcf)	Final Dry Density (pcf)	Sample Diameter (in)	Load (psf)	Consolidation Under Load (%)	Head (ft)	Permeability (cm/sec)
1	TP-4, 0-25	15.5	23.7	105.8	105.6	8	300	-0.2 (swell)	5.0	2.05×10^{-8}
2	LSS No. 1 minus 1"	10.5	22.5	106.7* (85%)	107.0	20	300	0.3	2.0	8.65×10^{-5}
3	LSS No. 1 minus 1" w/5% Bentonite	13.2	20.8	112.3	112.6	20	300	0.3	3.5	9.4×10^{-7}
4	LSS No. 3 minus 1"	10.0	20.0	108.1* (85.5%)	108.2	20	300	0.1	3.0	2.67×10^{-5}
5	LSS No. 1 minus 1" (Re-test)	13.3	15.1	119.2	119.7	20	300	0.4	3.0	5.83×10^{-6}
6	LSS No. 3 minus 1" (Re-test)	10.0	16.0	120.2	120.2	20	300	-0.1 (Swell)	4.0	8.91×10^{-8}
7	TP No.3 @ 0 - 4.5 ft.	20.5	26.5	100.7	101.8	2.5	300	1.1	1.0	1.19×10^{-7}
8	TP No.6 @ 0 - 4.5 ft.	19.4	24.2	101.6	103.8	2.5	300	2.1	1.0	1.56×10^{-7}
9	TP No.10 @ 0 - 2.5 ft.	17.1	25.0	103.5	106.9	2.5	300	3.2	1.0	9.71×10^{-8}

* Samples were not compacted to 95% of maximum dry density, due to a mistaken computation by the lab technician.

CHAMBERLIN ARCH.
MKE DOCUMENT NO. 0025-GRJ-C-09-01976-00
Zoning
Bldg 2/17/
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JUN 27 1989

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project UMTRA - GRJ	Contract No. 5025	Sheet 195 of 201	
Feature EMBANKMENT DESIGN	Designed SJM	File No.	Date 1-13-89
Item SOIL PROPERTIES	Checked RMB		Date 1-19-89

PERMEABILITY OF FOUNDATION SOILS

IN-SITU PERMEABILITY MEASUREMENTS REPORTED IN THE DRAFT RAP ARE PRESENTED ON THE NEXT SHEET. ADDITIONAL PERMEABILITY DATA IS INCLUDED IN THE LAYER 3 SUMMARY SHEET. THERE IS NO AVAILABLE DATA FOR LAYER 1, HOWEVER THIS MAY NOT BE SIGNIFICANT SINCE LAYER 1 WILL BE FULLY EXCAVATED IN THE EMBANKMENT FOOTPRINT AREA.

260
~~L952~~
PS 2/4/88
WYL 2/5/88

Table E.7.2 In-situ permeability tests at Cheney Reservoir

Location[a]	Depth interval (ft)	Permeability (cm/sec)
501	12-22	4×10^{-6} Layers 2 & 3
	21-31	1×10^{-6} Highly weathered Shale
	31-40	$<1 \times 10^{-7}$ Highly Weathered/Weathered Sh.
	40-50	$<1 \times 10^{-7}$ Weathered Shale
502	10-14	2×10^{-4} Layer 2
	20-31	3×10^{-6} Layer 3/4
	36-46	$<1 \times 10^{-7}$ Highly weathered
	41-51	$<1 \times 10^{-7}$ Mod. weathered
503	8-14	5×10^{-5} Layer 2
	14-28	1×10^{-7} Layer 3 (?)
	30-40	$<1 \times 10^{-7}$ Layer 5
	40-50	1×10^{-6} Weathered Shale (Weathered ?)
504	10-18	3×10^{-5} Layer 2
	22-34	5×10^{-6} Layer 3
	35-44	6×10^{-5} Layer 3
	44-56	$<5 \times 10^{-8}$ Highly weathered shale
505	8-18	1×10^{-5} Layer 2
	18-28	9×10^{-6} Layer 3/4
	23-35	1×10^{-5} Layer 3/4/5
	38-51	1×10^{-7} Very weathered shale
506	10-19	6×10^{-5} Layer 2
	21-29	1×10^{-4} Layer 3/4
	29-40	6×10^{-5} Very weathered shale
		to 1×10^{-4} with gypsum fractures
	40-51	1×10^{-5}
		to 5×10^{-5} "

[a]Boring locations are shown in Figure E.7.1.

Ref. NUS, 1983.

Inferred Design Values: Layer 1 — Not Available
2 — 7.0×10^{-5} cm/sec
3 — ~~4.4×10^{-6}~~ cm/sec ← 2.2×10^{-6}
4 — 5.7×10^{-5} cm/sec
Bedrock — 1×10^{-7} cm/sec

ROOTING SOIL

Introduction

The rooting soil will consist of soil from Layer 1, top 1-ft. The soil will be placed as evenly as possible, and every effort be made to avoid undue compaction. Gradation, and compaction data for this layer is presented in Sheets 262 through 271.

The following are assumed for our calculations

$\gamma_d = 84.8$ pcf $\qquad \gamma_{in\text{-}place} = 95.0$ pcf

$S_u = 250$ psf $\qquad w = 12°$

$\phi' = 20°$

The densities correspond to 80 pcf of ASTM D698, and the moisture content is at placement. The strength is for soft to very soft clays

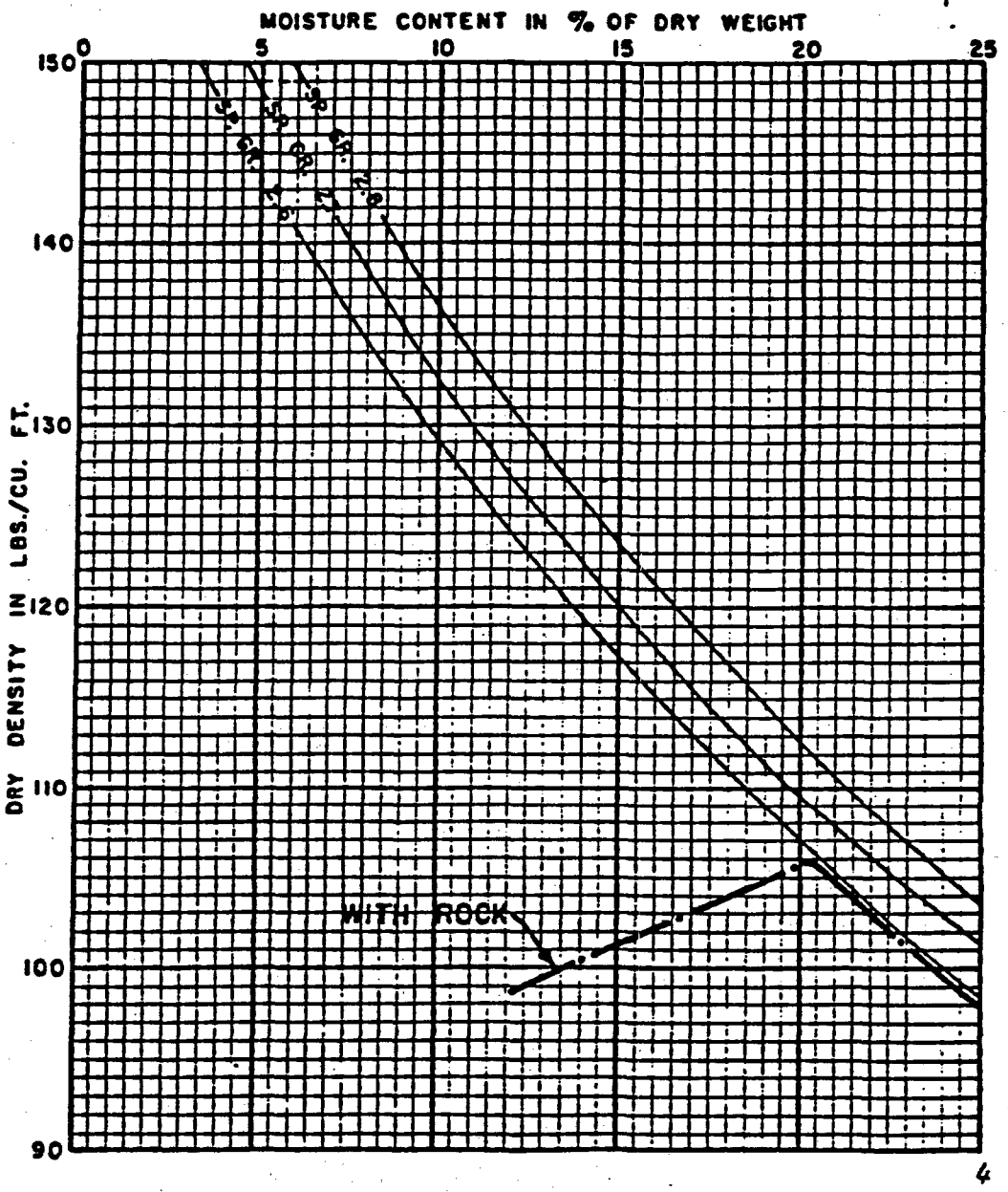
Revision 3

Additional gradation tests were run on the soils from the top-soil stockpiles at the site. These are presented in Sheets thru .

187
207
262

SAMPLE NO. TP-3 DEPTH 0-4.5' ELEVATION —
SOIL SILTY CLAY, A LITTLE ROCK
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 20.5 %
MAXIMUM DRY DENSITY 106.0 pcf
METHOD OF COMPACTION ASTM D698 METHOD C

PS 2/4/88
WYL 2/5/88

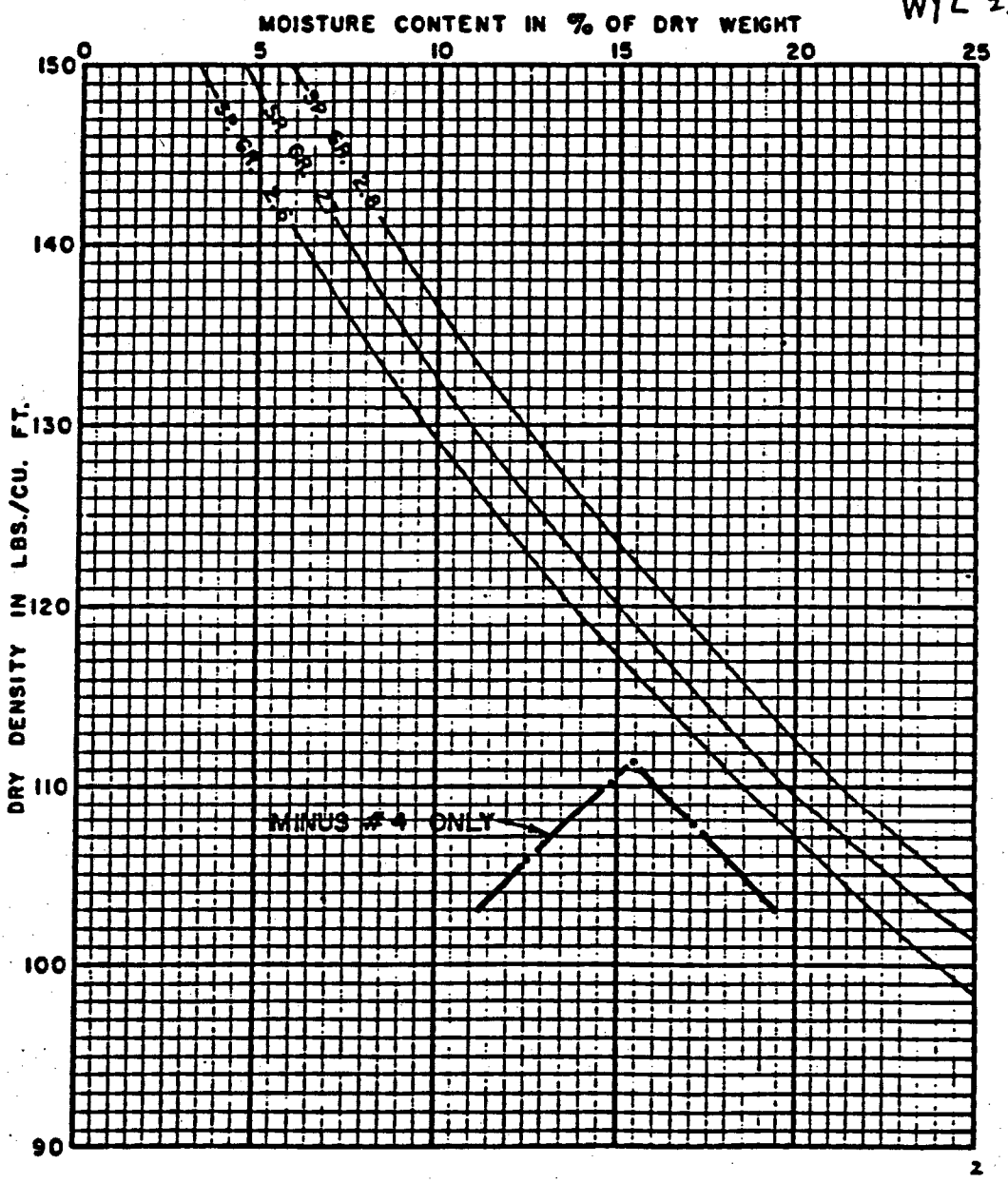


COMPACTION TEST DATA

~~182~~
~~208~~
263

SAMPLE NO. TP-4 DEPTH 0-2.5' ELEVATION —
SOIL SILTY CLAY, A LITTLE ROCK
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 15.5 %
MAXIMUM DRY DENSITY 111.4 pcf
METHOD OF COMPACTION ASTM D698 METHOD A

PS 2/4/88
WYL 2/5/88

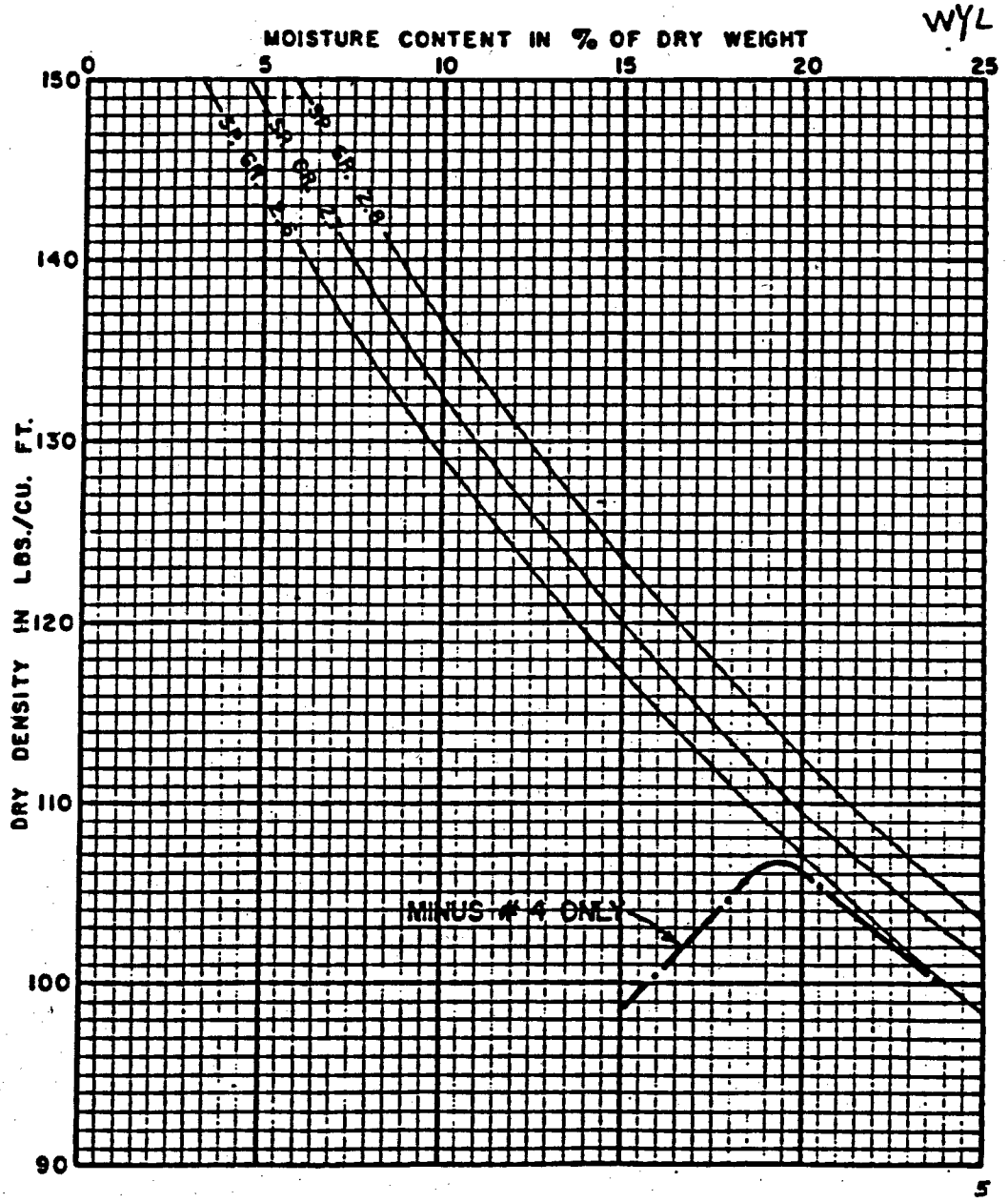


COMPACTION TEST DATA

183
~~209~~
264

SAMPLE NO. TP-6 DEPTH 0-3.0' ELEVATION —
SOIL SILTY CLAY, A LITTLE ROCK
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 19.4%
MAXIMUM DRY DENSITY 106.9 pcf
METHOD OF COMPACTION ASTM D698 METHOD A

PS 2/4/88
WYL 2/5/88

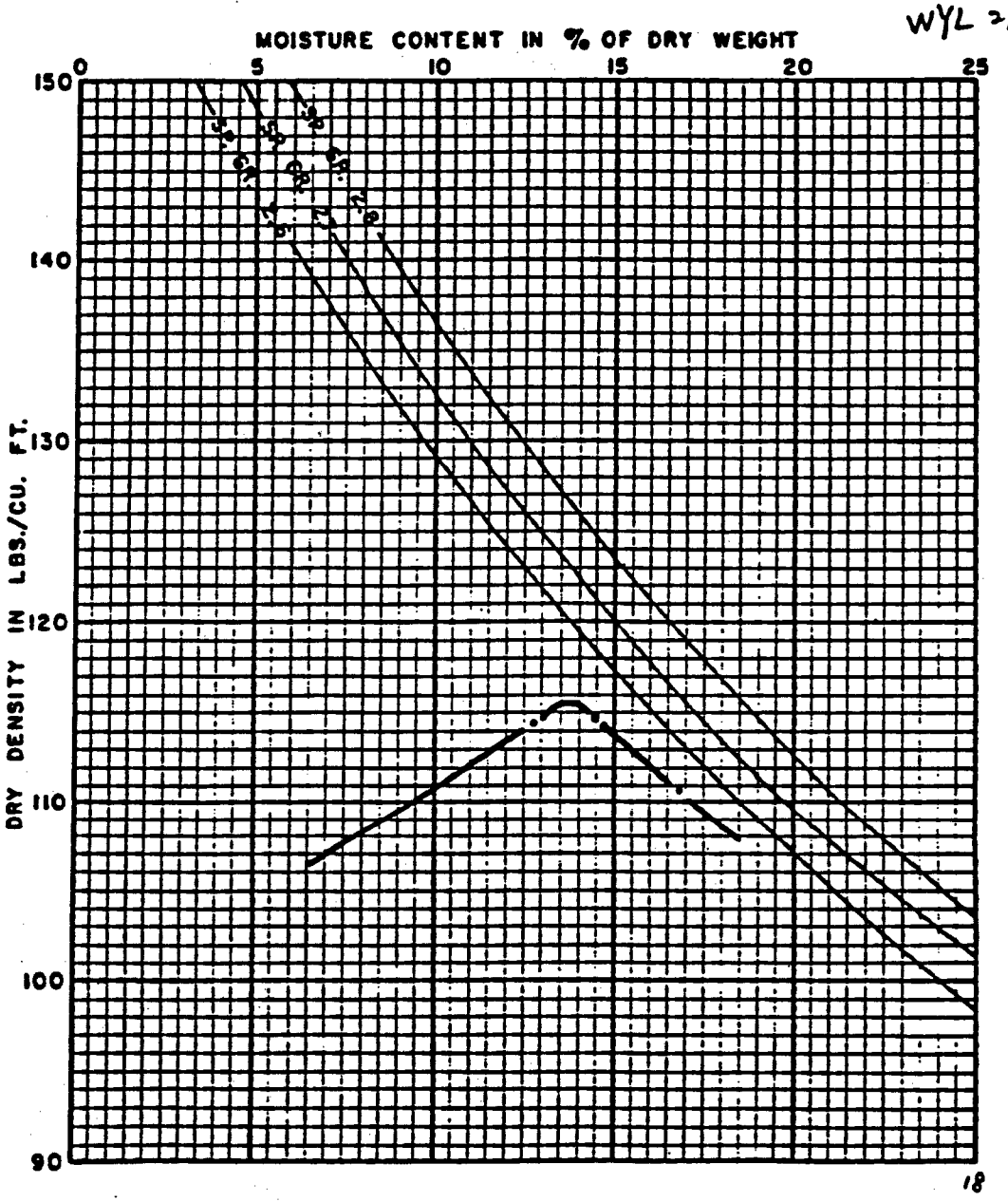


COMPACTION TEST DATA

~~18~~
210
265

SAMPLE NO. TP-8 DEPTH 0-3.0' ELEVATION —
SOIL SILTY CLAY SOME ROCK
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 13.8 %
MAXIMUM DRY DENSITY 115.7 pcf
METHOD OF COMPACTION ASTM D698 METHOD C

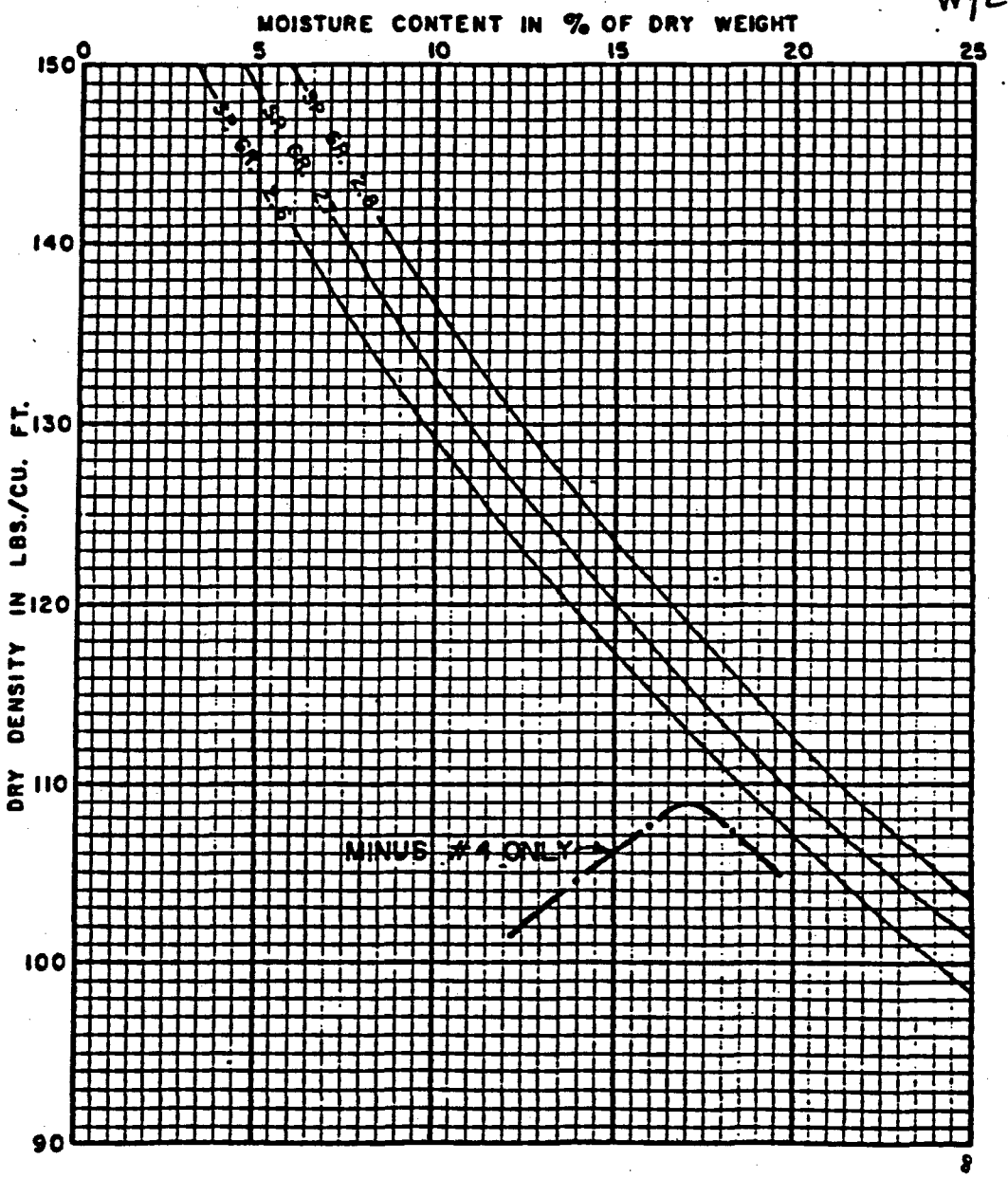
PS 2/9/88
WYL 2/5/88



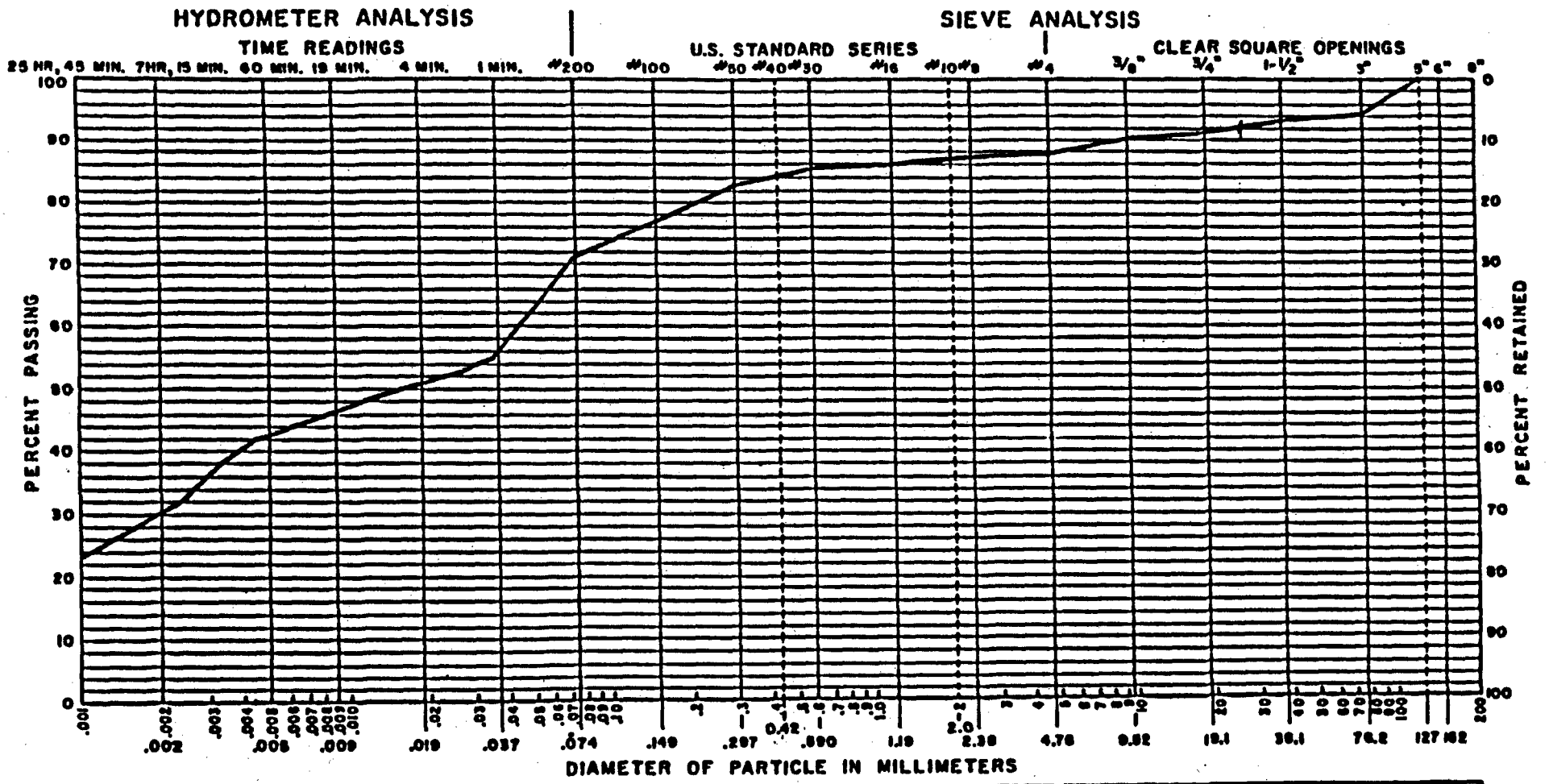
COMPACTION TEST DATA

SAMPLE NO. TP-10 DEPTH 0-2.5' ELEVATION —
SOIL SILTY CLAY, A LITTLE ROCK
LOCATION CHENEY RESERVOIR DISPOSAL SITE
OPTIMUM MOISTURE CONTENT 17.1 %
MAXIMUM DRY DENSITY 109.0 pcf
METHOD OF COMPACTION ASTM D698 METHOD A

PS 2/4/88
WYL 2/5/88



COMPACTION TEST DATA



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES: liquid limit: 44.8
plasticity index: 25.4
natural moisture content: 9.9%

CHENEY SITE

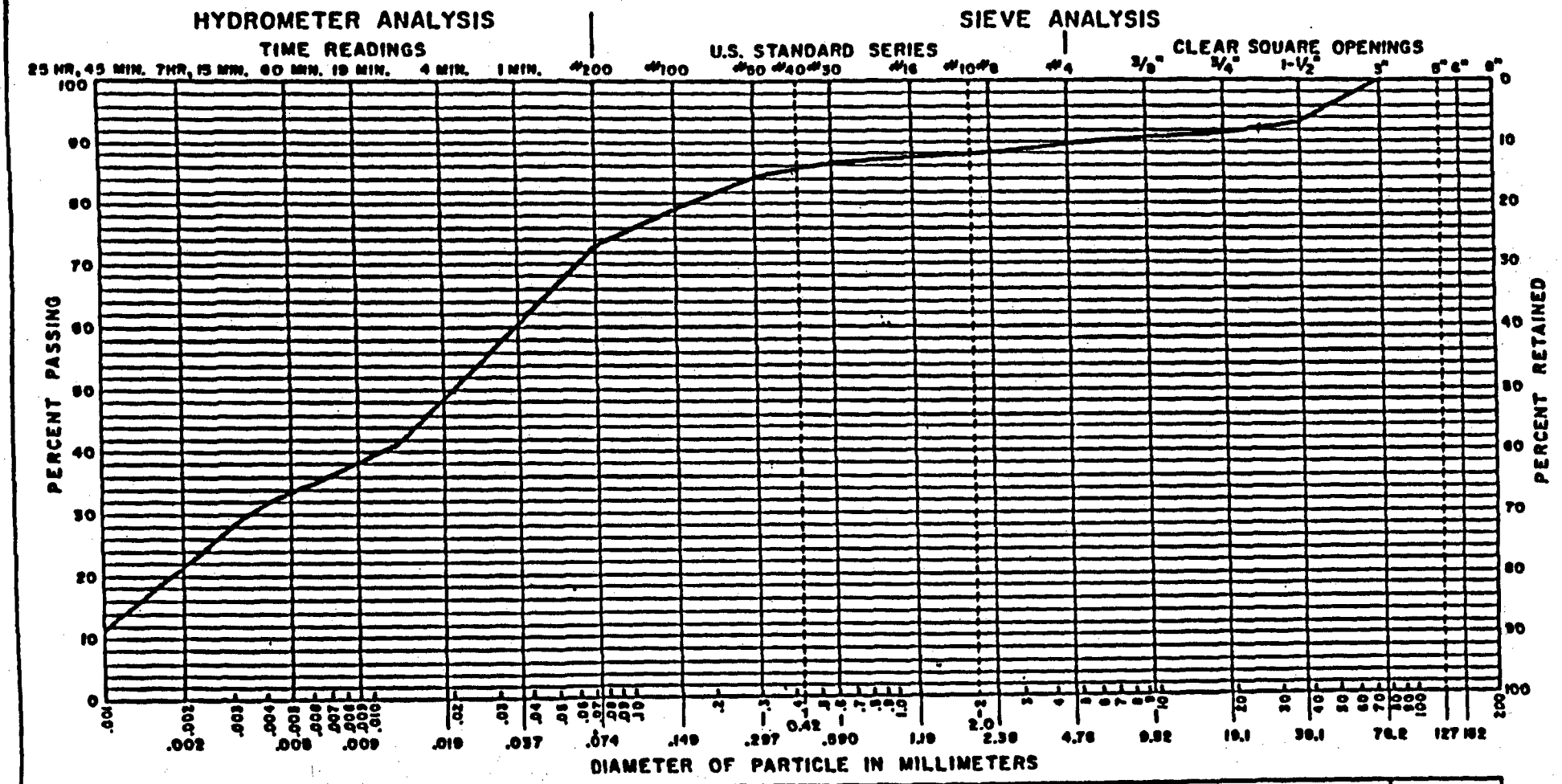
GRADATION TEST

PLATE

WESTERN ENGINEERS INC.

DESIGNATION _____ EXCAVTION NO. TP-3 DEPTH (FT.) 0-4.5'

267



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES: liquid limit: 34.9
plasticity index: 15.9
natural moisture content: 10.1%

CHENEY SITE

GRADATION TEST

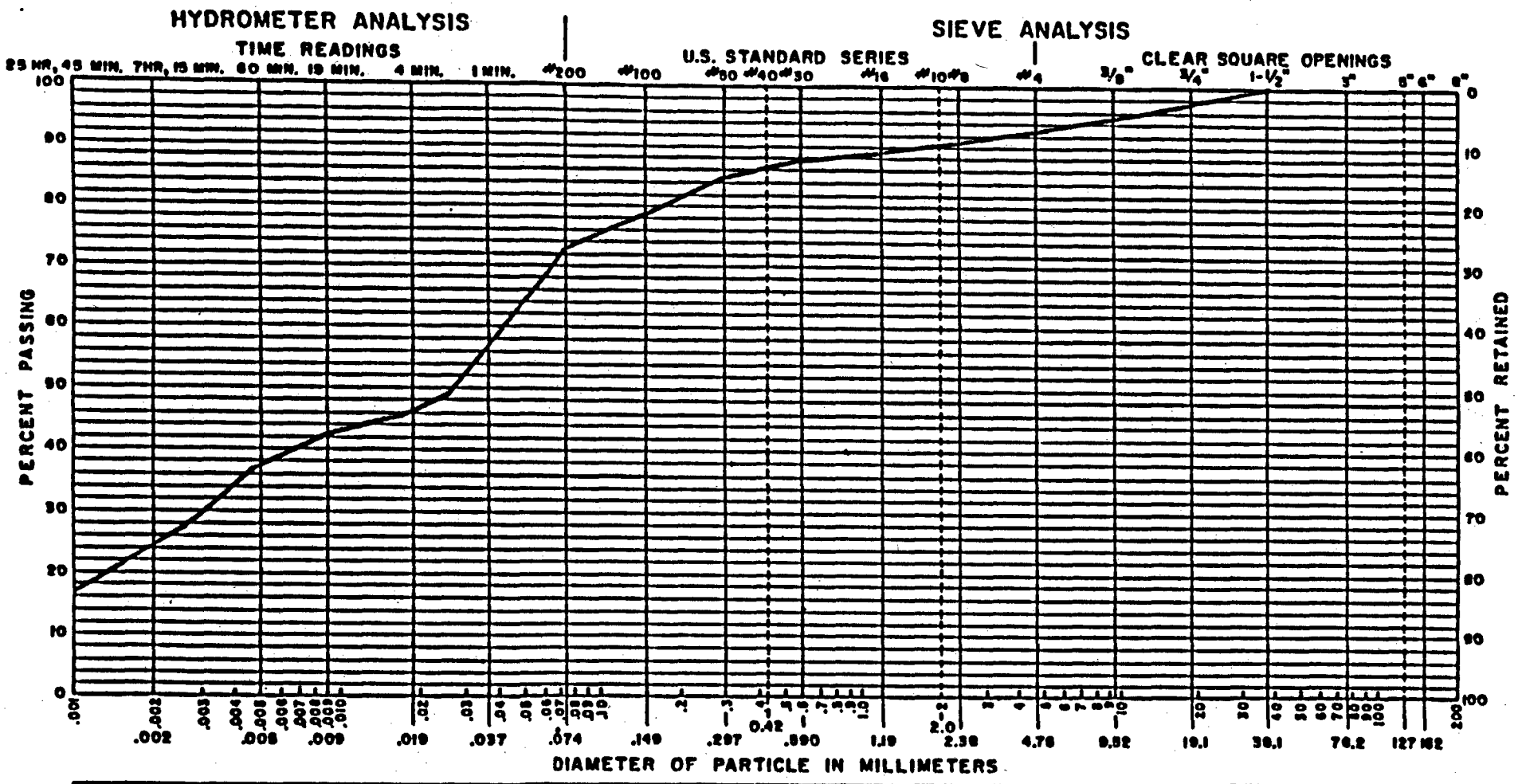
PLATE

WESTERN ENGINEERS INC.

DS 2/4/88
WVL 2/17/88

DESIGNATION _____ EXCAVTION NO. TP-4 DEPTH (FT.) 0-2.5'

268



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES: liquid limit: 41.2
plasticity index: 21.8
natural moisture content: 14.9%

CHENEY SITE

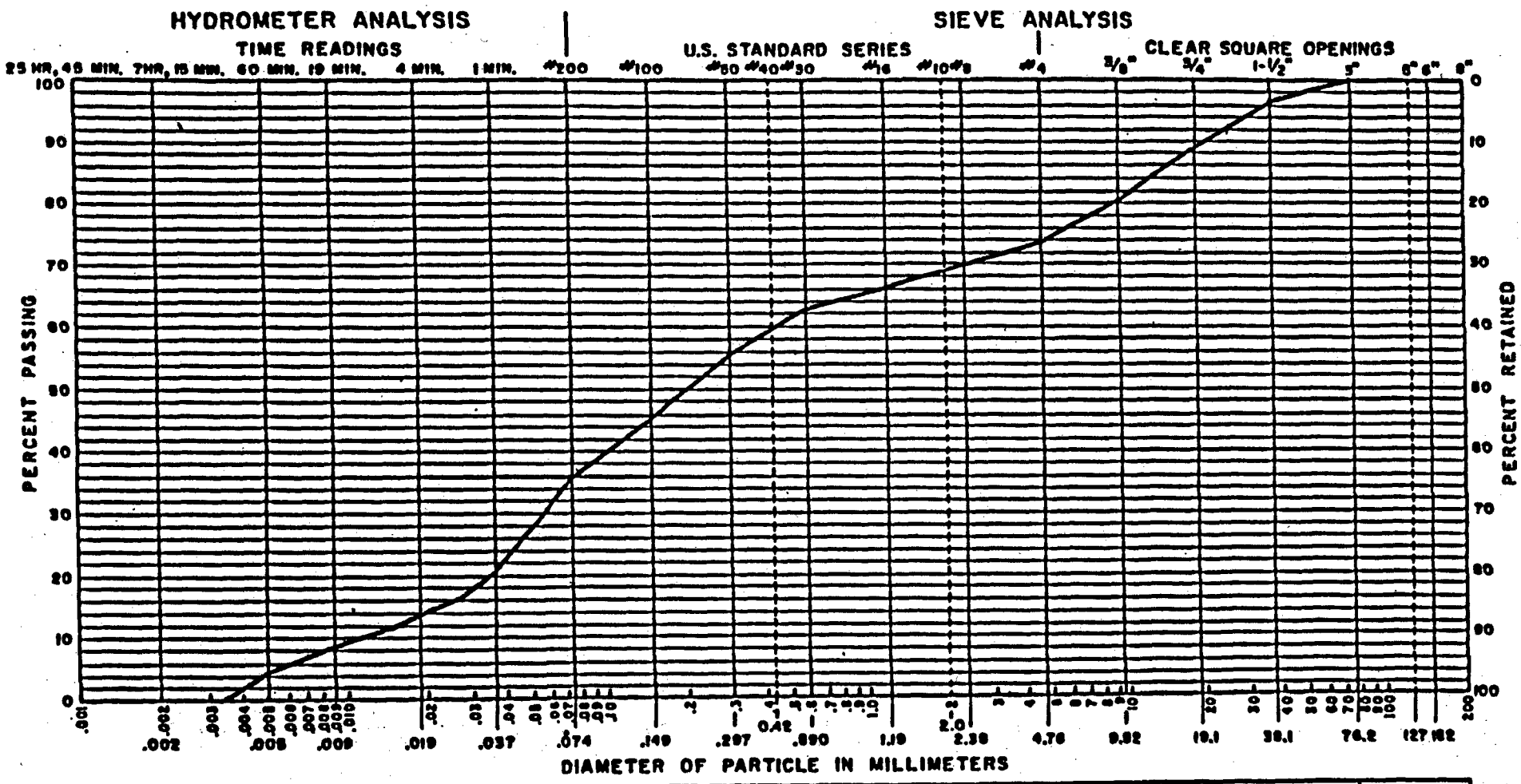
GRADATION TEST

PLATE

WESTERN ENGINEERS INC.

DESIGNATION _____ EXCAVTION NO. TP-6 DEPTH (FT.) 0-3'

PS 2/4/88
WYL 2/5/88
2nd 4/7
269



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES: liquid limit: 32.9
plasticity index: 8.6
natural moisture content: 6.5%

CHENEY SITE

GRADATION TEST

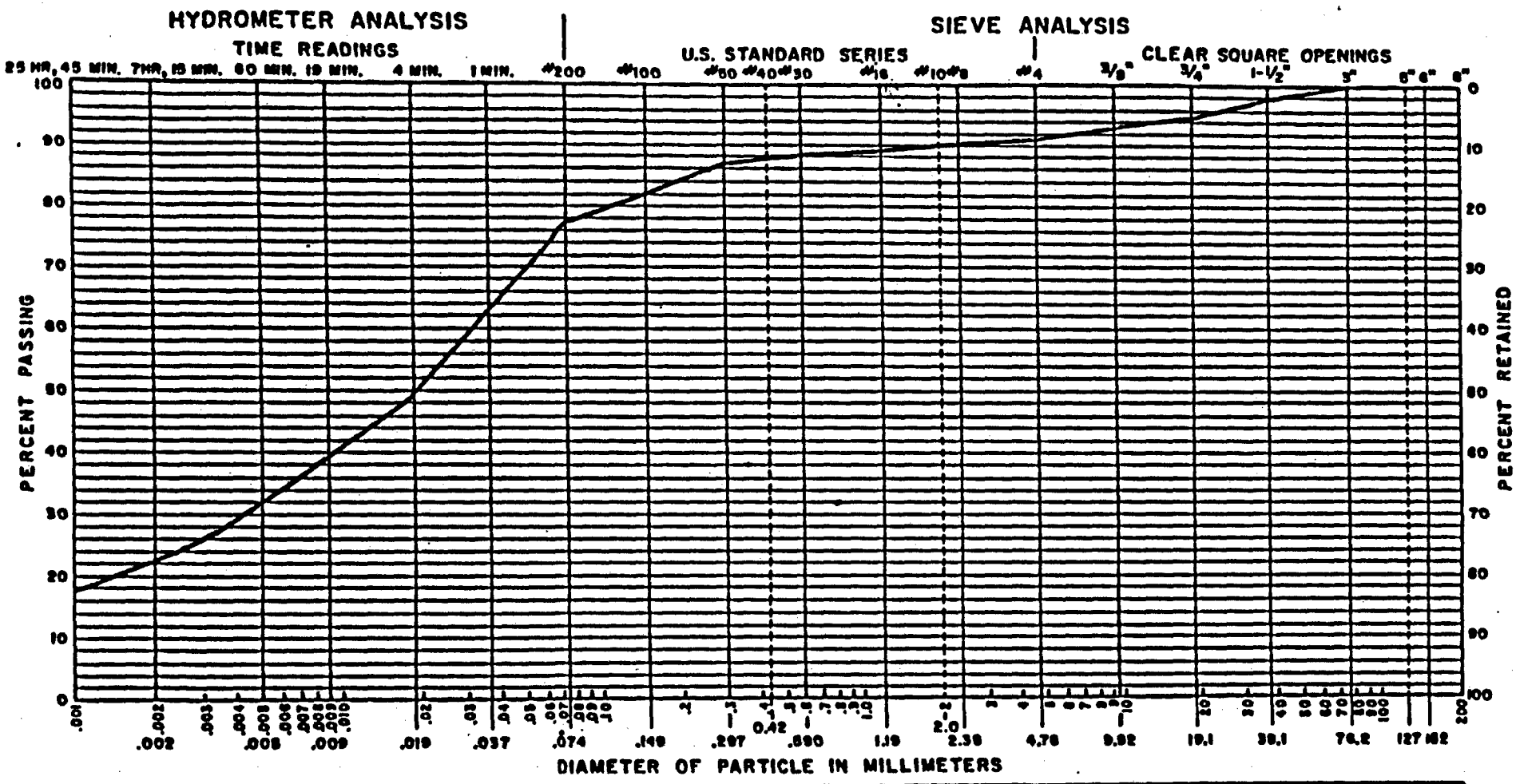
PLATE

WESTERN ENGINEERS, INC.

DESIGNATION _____ EXCAVTION NO. TP-8 DEPTH (FT.) 0-3'

PS 2/4/83
WYL 2/5/83

270



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

NOTES: liquid limit: 43.0
plasticity index: 26.4
natural moisture content: 12.7%

CHENEY SITE

GRADATION TEST

PLATE

WWL 2/4/88

DESIGNATION _____ EXCAVTION NO. TP-10 DEPTH (FT.) 0-2.5'

WESTERN ENGINEERS, INC.

RIPRAP, DRAIN AND BIO-INTRUSION LAYERS

The rip-rap and bio-intrusion layers will be from processed materials from the layer 2 gravelly soils. The rock properties are discussed in detail in Reference 18 HL 6/19/90 SKB 6-25-90

The drain material characteristics are discussed in Reference 19. H.L. 6/19/90 SKB 6-25-90

The material properties as required for stability and settlement analyses are as follows

$$\gamma = 125 \text{ pcf}$$

$$\phi = \phi' = 38^\circ$$

$$c = 0$$

The ϕ' of 38° is conservative for the basaltic rocks, as may be justified from the values on the next sheet, particularly since they are subjected to very low confining pressures, being near the surface.

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project: UMTRA-GRJ
Feature: Embankment Design
Item: Material Properties

Contract No. 5625
Designed: PS
Checked: SHB

Sheet 273
File No.
Date 2/17/86
Date 6-25-90

FROM REF. 15, PAGE 264

TABLE 7.4. STRENGTH AND DEFORMATION CHARACTERISTICS OF LARGE ROCKFILL SPECIMENS. Confining Pressure = 350 psi.

Dam or Place	Material	Gradation	Particle Shape	$\phi°$	ϵ_{lf}, percent	ϵ_{vf}, percent	$\sigma_{3\,crit}$, psi	Reference
Oroville	Dredger tailings	Well-graded 6" fines	Rounded	43	6.5	1.5	120	Marachi et al., 1969
Pinzandaran	Sand and gravel (dry)	Well-graded 8"-fines	Rounded	39	8	4.7	60	Marsal, 1967b
San Francisco	Basalt	Well-graded 8"-1/4"	Angular	39	15	6	60	Marsal, 1967a
San Francisco	Basalt	Well-graded 3"-1/4"	Angular	38				Marsal, 1967b
San Francisco	Basalt	Poorly-graded 6"-fines	Angular	37	20	6.5	40	Marachi et al., 1969
Malpaso	Conglomerate (dry)	Well-graded 8"-fines	Angular	37	13	4.5	20	Marsal, 1967b
El Infiernillo	Silicified conglomerate (dry)	Poorly-graded 8"-fines	Angular	36.5	14	5.5	30	Marsal, 1965a
Pyramid	Argillite	Poorly-graded 6"-fines	Angular	36.5	20	5.5	25	Marachi et al., 1969
El Infiernillo	Diorite (dry)	Poorly-graded 8"-fines	Angular	35	15	10	25	Marsal, 1967b
El Granero	Shale*	Well-graded 8"-1/4"	Angular	35	> 14	> 10	10	Marsal, 1967b
El Granero	Shale*	Poorly-graded 8"-3/4"	Angular	33	> 14	> 10	5	Marsal, 1967b
Mica	Granitic gneiss*	Well-graded 8"-fines	Angular	32	> 14	6	20	Marsal, 1967a
Mica	Granitic gneiss*	Poorly-graded 8"-1 1/2"	Angular	25	14	10	5	Marsal, 1967a

TABLE 7.5. STRENGTH AND DEFORMATION CHARACTERISTICS OF INTERMEDIATE-SIZE ROCKFILL SPECIMENS.

Dam or Place	Material	Gradation	Particle Shape	σ_3, psi	$\phi°$	ϵ_{lf}, percent	ϵ_{vf}, percent	$\sigma_{3\,crit}$, psi	Reference
Furnas Dam, Trans'n.	Quartzite	Well-graded 1"-fines		510	42	7	2	80	Casagrande, 1965
Oroville	Dredger tailings	Well-graded 2"-fines	Rounded	500	40	8	1.5	110	Marachi et al., 1969
El Infiernillo	Silicified conglomerate (dry)	Well-graded 1-1/2"-1/4"	Angular	350	40.8	14.5	5.6	50	Marsal, 1965a
Mica	Sandy gravel (D_r = 87 percent)	Well-graded 3"-fine sand	Rounded	450	39.5	7	2	140	Casagrande, 1965
Furnas Dam, Trans'n.	Quartzite	Well-graded 3/8"-fines		510	39	7	2	80	Casagrande, 1965
Furnas	Hard quartzite	Well-graded 3/8"-fines		510	39	12	3	40	Casagrande, 1965
Mica	Granitic gneiss	Well-graded 1-1/2"-1/4"	Angular	350	39.3	14.5	5.3	20	Marsal, 1965a
San Francisco	Basalt	Poorly-graded 2"-fines	Angular	500	38	19	8	30	Marachi et al., 1969
Pyramid	Argillite	Poorly-graded 2"-fines	Angular	500	37	19	8	25	Marachi et al., 1969
Mica	Sandy gravel (D_r = 50 percent)	Well-graded 3"-fines	Rounded	450	37	18	5	25	Casagrande, 1965
Furnas	Hard quartzite	Uniformly-graded 1"-3/8"	Angular	510	34	> 20	13	25	Casagrande, 1965

REVISION 3

CLEAN-FILL DIKE

Two different kinds of clean-fill dikes will be constructed on the side slope of the tailing embankment.
They are:

1. Below Ground: Compacted Mancos shale (95% Compaction)
2. Above Ground: Compacted -1" material (95% Compaction)

The pertinent design properties are presented in the following sheets.

REVISION 3 COMPACTED MANCOS SHALE

COMPACTED MANCOS SHALE

Introduction

In general, the upper few feet of the Mancos Shale bedrock at the site is very highly weathered. It is almost a hard clay. It is felt that this very highly weathered shale is easily ripped. In addition, it appears that this material is easily compactable to a low permeability barrier. Thus, this material may be used as back-up material for radon barrier (the lower clays are the primary radon barrier material) and, if needed, for low permeability liners. It is with this in mind that compacted Mancos Shale was tested for its properties.

It should be noted that all the compacted Mancos Shale tested was from Trench #3, which is outside the present excavation area. Also, it is a general observation that the thickness of very highly weathered Mancos Shale is considerably ~~is considerably~~ less. Finally, it is to be expected that as the degree of weathering decreases, the excavated Mancos Shale will become less suitable for use as a low permeability barrier

Tests Performed

A summary of the soil test results is presented in Sheet 278.

REVISION 3

COMPACTED MANCOS SHALE

DENSITY, MOISTURE CONTENT, AND SPECIFIC GRAVITY

NOTE: DATA FROM SUMMARY OF TEST RESULTS ON Sh. 278

(a) $\gamma_{DRY\ AVE.} = \left\{\frac{109.1 + 110 + 108.2 + 106.8 + 110.5}{5}\right\} 0.95 = 103.5\,pc$

↳ Dry Density, compacted to 95%

(b) $W_{OPT.\ AVE.} = \frac{18.2 + 18.2 + 19 + 19.1 + 18}{5} = 18.5\ \%$

↳ Optimum Moisture Content

(c) Assume Specific Gravity = 2.75

(d) Average Moist Density, $\gamma = 103.5 \times 1.185 = 122.6\,pc$

(e) Average Saturated Density

void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.75 \times 62.4}{103.5} - 1 = 0.658$

$\gamma_{AVE.} = \frac{G_s \gamma_w + e \gamma_w}{1 + e} = \frac{2.75 \times 62.4 + 0.658 \times 62.4}{1 + 0.658} = 128.3\,pcf$

$W_{sat} = \frac{Se}{G} = \frac{100\% \times 0.658}{2.75} = 0.239$

$W_{sat} = 23.9\%$

REVISION 3

COMPACTED MANCOS SHALE

STRENGTH

Present plan calls for placement of compacted shale on the side slope below the existing ground. It is expected to have 95% compaction on the mancos. For short-term, use the undrained shear strength from the lab. Test results. From Test Results (Sh. 278)

$$\text{Cohesion, } c_{ave.} = \frac{0.150 + 0.165 + 0.175 + 0.185 + 0.20}{5}$$

$$= 0.175 \text{ ksf}$$
$$= 175 \text{ psf}$$

For long-term seismic, use the CU (Consolidated Undrained) test results. From Test Results (Sh. 278)

$$\phi \text{ (Total Stress)} = 20^\circ$$

RECOMMENDED DESIGN PARAMETERS

Density (Moist Density) = 122.6 pcf
Moisture Content = 18.5%
Shear Strength:
Short-Term (Undrained): $c = 175$ psf, $\phi = 0$
Long-Term Seismic (CU, total stress): $c = 0$; $\phi = 20$
Long-Term Static (CU, effective stress): $c = 0$, $\phi = 28^\circ$

DATA FROM: MKE Doc. 5025-GRJ-L-09-04072-00

H.L. 6/25/90 Lab 6-2540 278

SUMMARY OF SOIL TEST RESULTS (E) - COMPACTED MANCOS SHALE

Job No 5025-16 Project Name UMTRA-GRJ Feature CHENEY DISPOSAL SITE Date 6/8/90

Trench Number	Sample Number	Depth (Feet) From	Depth (Feet) To	Laboratory Classification	Mechanical Analysis Gravel	Mechanical Analysis Sand	Mechanical Analysis Fines	Atterberg Limits LL	Atterberg Limits PI	Specific Gravity G	Natural w %	Natural γd	Compaction Test	Compaction Optimum w	Compaction Optimum γd	Shear Strength Test	Shear Strength Initial w	Shear Strength Initial γd	Shear Strength C ksf	Shear Strength φ	Permeability γd	Permeability 95% Comp.	Consolidation Cc	Consolidation Cc'	Consolidation Pc	Notes
1																										
2																										
3	1	3	8	Weathered									SP	18.2	109.1	UU 90% COMP	17.6	98.5	.045		98.3	6.5×10^{-7}	11.6			CU w/ PORE PRESSURE MEASUREMENTS ON SAMPLE 1
	"	—	—	Shale												UU 96.2 COMP	16.4	105.1	.150		105.4	1.4×10^{-7}	22.1			TOTAL STRESS: $c = 0.06$ ksf, $\phi = 20.2°$
	"	—	—	"												UU 100 COMP	17.7	109.5	.305		110.7	4.6×10^{-8}				EFF. STRESS: $\bar{c} = 0.90$ ksf, $\bar{\phi} = 28.2°$
	2A	8	13	"									SP	18.2	110.0	UU	17.6	105.2	0.165		105.6	4.3×10^{-7}				
	2B	13	18	"									SP	19	108.2	UU	18.4	103	0.175		103.7	9.6×10^{-7}				
	3A	18	23	"									SP	19.1	106.8	UU	18.4	102.1	0.185							
	3B	23	28	"									SP	18	110.5	UU	17.6	105.3	0.20							
							AVERAGE =							18.5	108.9						104.7	4.5×10^{-7}				

* Visual Classification

SP = Standard Proctor
MP = Modified Proctor
S = Special - See Text

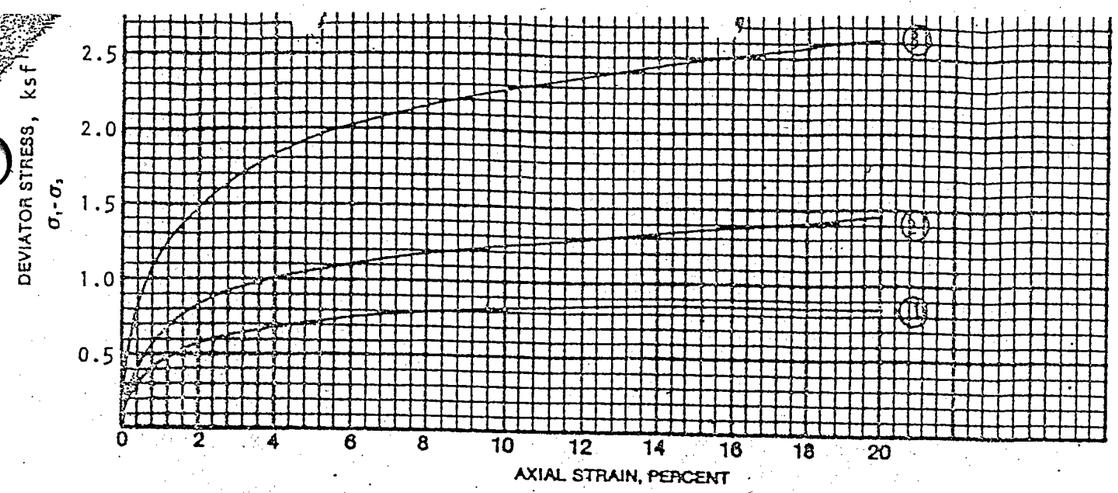
TC = Triaxial Compression
UC = Unconfined Compression
DS = Direct Shear

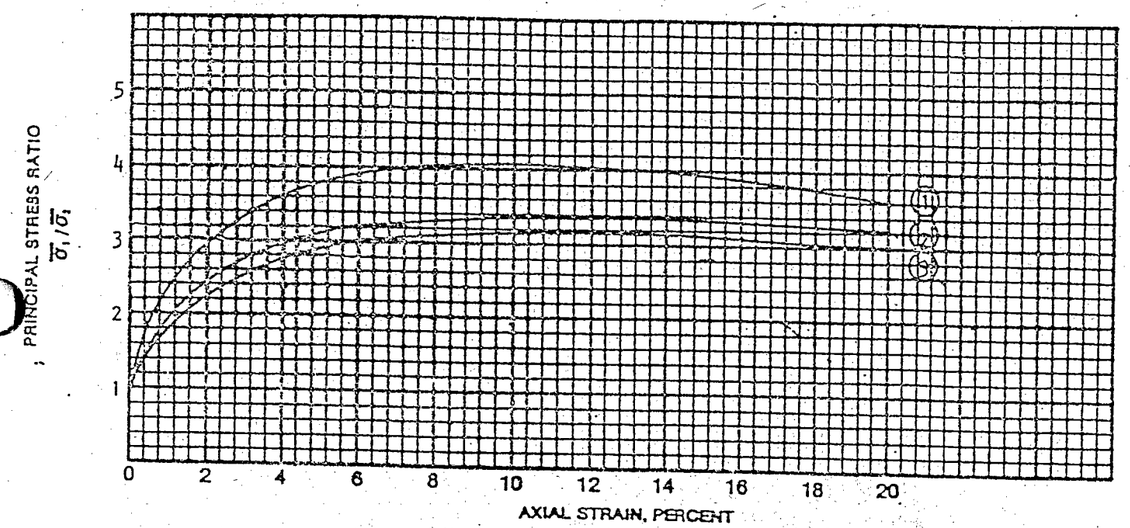
UU = Unconsolidated Undrained
CU = Consolidated Undrained
CD = Consolidated Drained

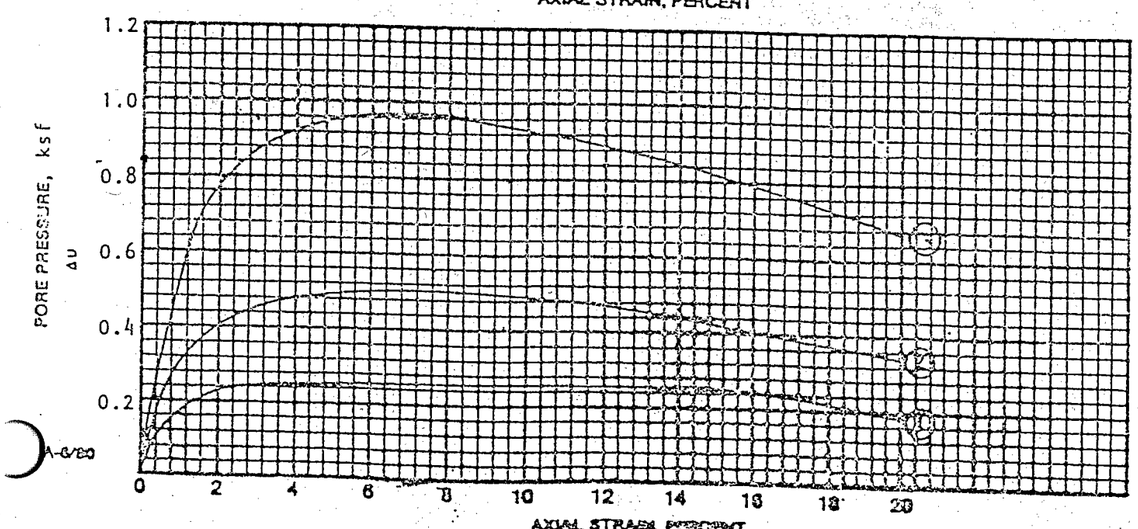
SHEAR STRENGTH OF SOIL IN TRIAXIAL COMPRESSION

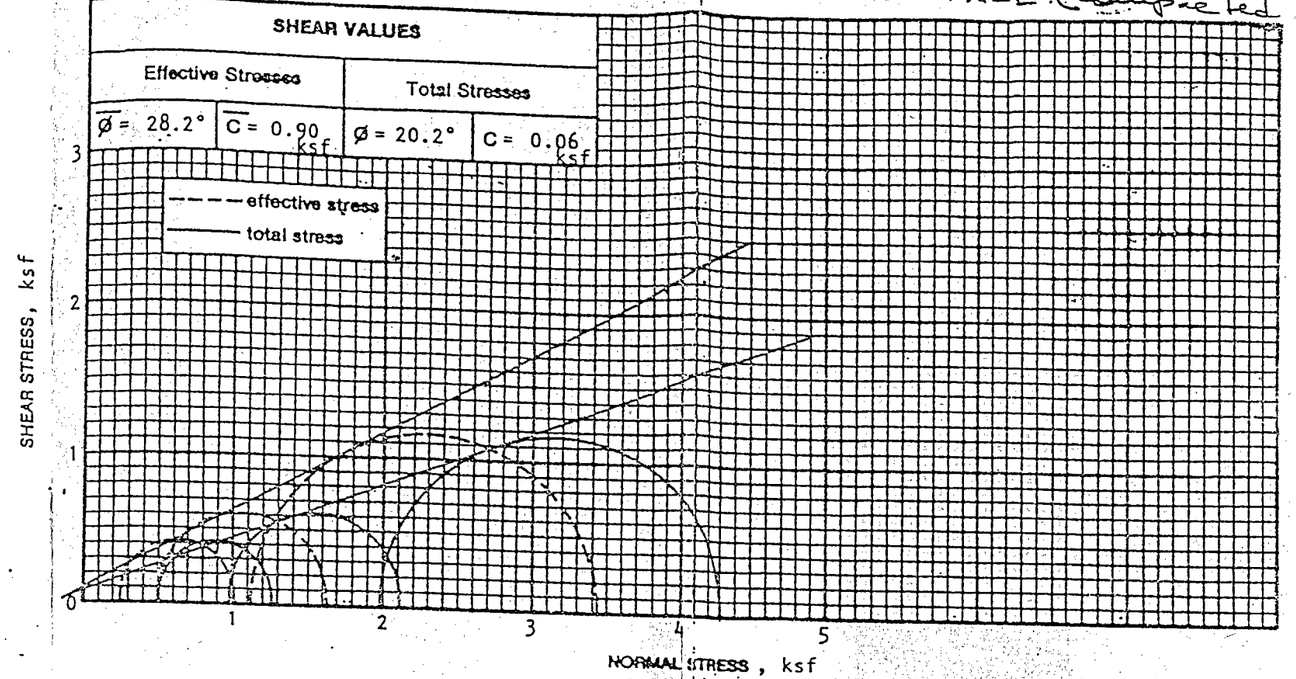
UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Compacted)

Job No. 1 126 90
Date February 7, 1990
Type of Test Saturated, Consolidated, Undrained, with pore pressure measurements









Stage Number	Specimen Location		Initial Specimen Data					Soil Description
	Boring Number	Depth (Ft)	Sample Type	Length (in)	Diameter (in)	Dry Density (P.C.F.)	Moisture Content (%)	
1	1	5-8	Remolded	3.914	1.882	104.7	16.9	Mancos Shale
2	1	5-8	Remolded	3.957	1.882	104.1	17.2	Mancos Shale
3	1	5-8	Remolded	3.959	1.882	104.0	17.5	Mancos Shale

Stage Number	"B" Parameter	Test Values at Maximum Principal Stress Ratio							Remarks
		Total Confining Stress σ_3	Total Axial Stress σ_1	Deviator Stress $\sigma_1-\sigma_3$	Effective Lateral Stress $\bar{\sigma}_3$	Effective Axial Stress $\bar{\sigma}_1$	Pore Pressure μ	Percent Strain ϵ%	
1	0.98	0.50	1.27	0.77	0.24	1.02	0.26	10.0	Units in ksf
2	0.97	0.99	2.14	1.15	0.50	1.65	0.49	8.0	Units in ksf
3	0.98	2.00	4.32	2.32	1.12	3.45	0.88	12.0	Units in ksf

Remarks:

Fig. 1

UMTRA - GRJ
5025 -16
MATERIAL PROPERTIES
MANCOS SHALE (Compacted)

280

MK-FERGUSON COMPANY
A MORRISON KNUDSEN COMPANY

MKE DOC. 5025-GRJ-I-02-04071-00

INTER-OFFICE CORRESPONDENCE

DATE: March 6, 1990

TO: F. J. Feliz

FROM: R. E. Cooney

LOCATION: San Francisco, CA - MKES

LOCATION: Albuquerque, NM - 3050

SUBJECT: Grand Junction, Phase II
Triaxial Compression Tests, P.O. 3050-511-10034,
C.O. #3

Enclosed is a letter from Chen-Northern, dated February 27, 1990, providing the triaxial compression results per your IOC dated January 17, 1990 (#03852-00). Please advise Tom Jennings whether these results are acceptable for payment purposes.

If you have any questions, please advise.

REC:TPJ:kja

Enclosure

cc w/o enc.: R. E. Cooney
T. P. Jennings
J. G. Pepin
File

Attachment: MKE DOC. 5025-GRJ-L-09-04072-00

RECEIVED-MKE
MAR 07 1990
UMTRA-S.F.

4030K

STA001

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Continued)
MKE DOC. 5025-GRJ-L-09-04072-00

281

Chen Northern, Inc.

Engineers and Scientists
36 South 20th Street
Denver, Colorado 80223
303 744-7135
303 744-0215 Facsimile

MK-FERGUSON CO.
ALBUQUERQUE

MAR 02 1990

RECEIVED

February 27, 1990

MK - Ferguson Company
P.O. Box 9136
Albuquerque, New Mexico 87119

Attention: Mr. Jerry Hymes

Subject: Laboratory Testing, P.O. No. 3050-511-10034, Change Order No. 3

Job No. 1 126 90

Dear Mr. Hymes:

As requested, we have completed the triaxial shear tests on the Mancos shale samples from the UMTRAP Cheney site. The results are presented on Figs. 1 through 6. Also enclosed are copies of all worksheets.

If you have any questions regarding this submittal, please call.

Sincerely,

CHEN-NORTHERN, INC.

Sally K. Miller

Sally K. Miller, A.E.T.
Soils Laboratory Supervisor

SKM/sej
Rev. By: JB
Enclosures

Attachment: Left to document.

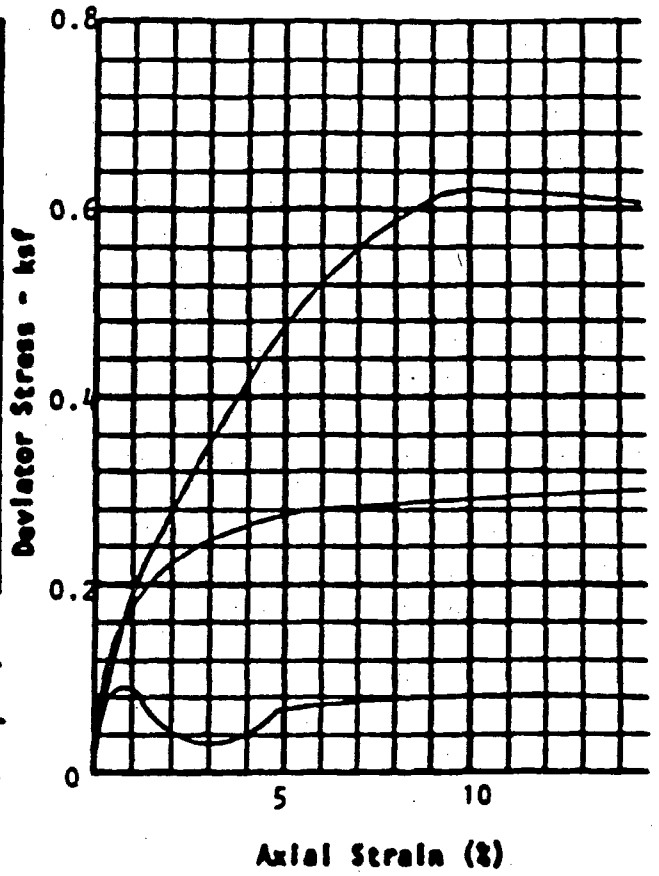
RECEIVED-MKE
MAR 7 1990

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Compacted)

Chen Northern, Inc.

Consulting Soil and Foundation Engineers

TEST NUMBER	1	2	3	4
LOCATION	1 3'-8'	1 3'-8'	1 3'-8'	
HEIGHT - INCH	3.953	3.953	3.953	
DIAMETER - INCH	1.882	1.882	1.882	
WATER CONTENT - %	17.6	16.4	17.7	
DRY DENSITY - pcf	98.5	105.1	109.5	
$\sigma_1 - \sigma_3$ - ksf	0.09	0.30	0.61	
σ_3 - ksf	0.99	0.99	0.99	
σ_1 - ksf	1.08	1.29	1.60	



TYPE OF SPECIMEN: Remolded

SOIL DESCRIPTION: Mancos Shale

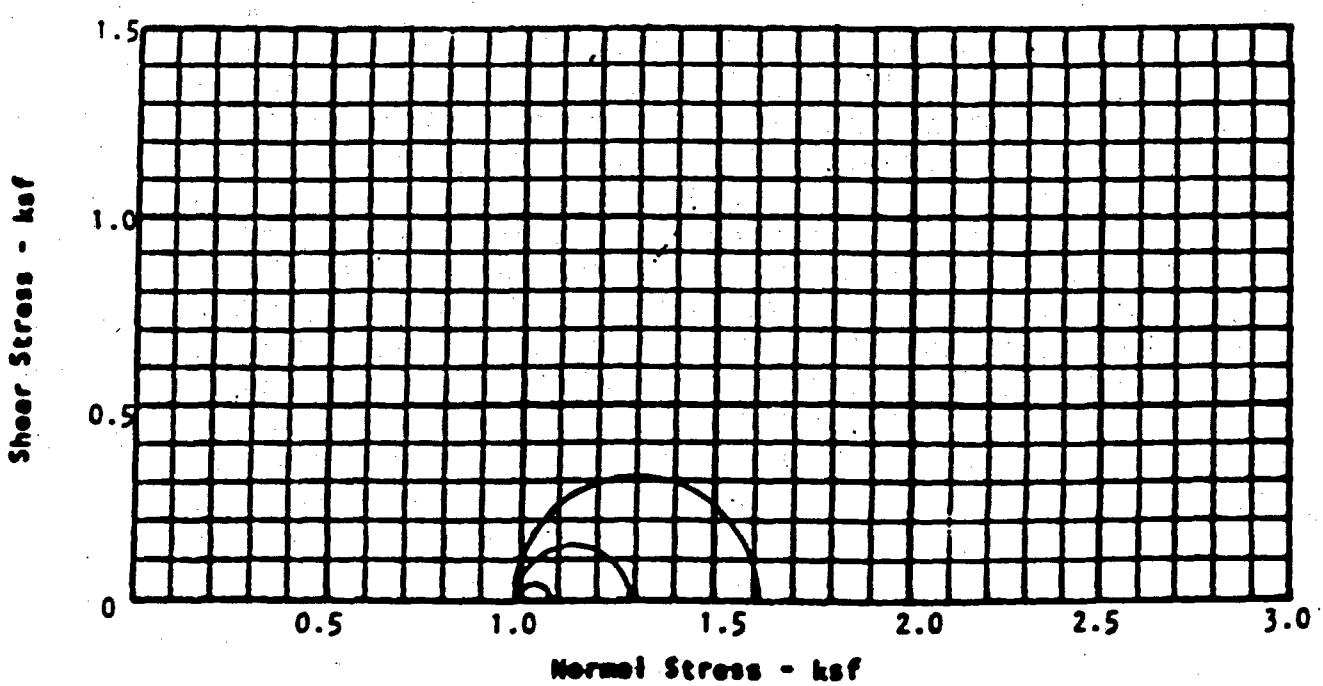
TYPE OF TEST: Saturated, Unconsolidated, Undrained

% Compaction = 90.3, 96.3, 100.4

TAN ϕ: N/A

ϕ: N/A

COHESION - ksf: N/A



TRIAXIAL SHEAR TEST RESULTS

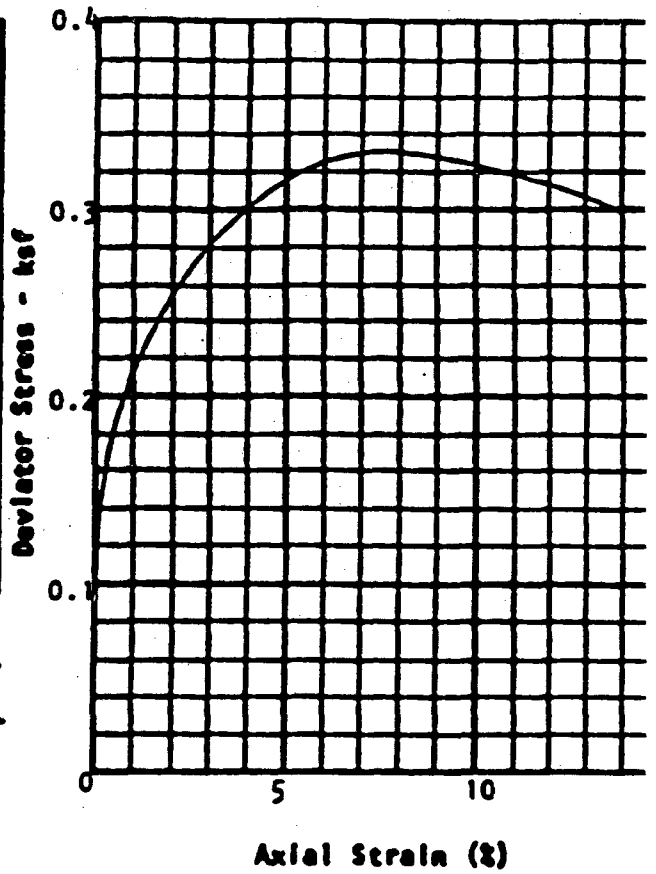
5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Compacted)

283

Chen Northern, Inc.

Consulting Soil and Foundation Engineers

TEST NUMBER	1	2	3	4
LOCATION	2A 8'-13'			
HEIGHT - INCH	3.953			
DIAMETER - INCH	1.882			
WATER CONTENT - %	17.6			
DRY DENSITY - pcf	105.2			
$\sigma_1 - \sigma_3$ - ksf	0.33			
σ_3 - ksf	0.99			
σ_1 - ksf	1.32			



TYPE OF SPECIMEN: Remolded

SOIL DESCRIPTION: Mancos Shale

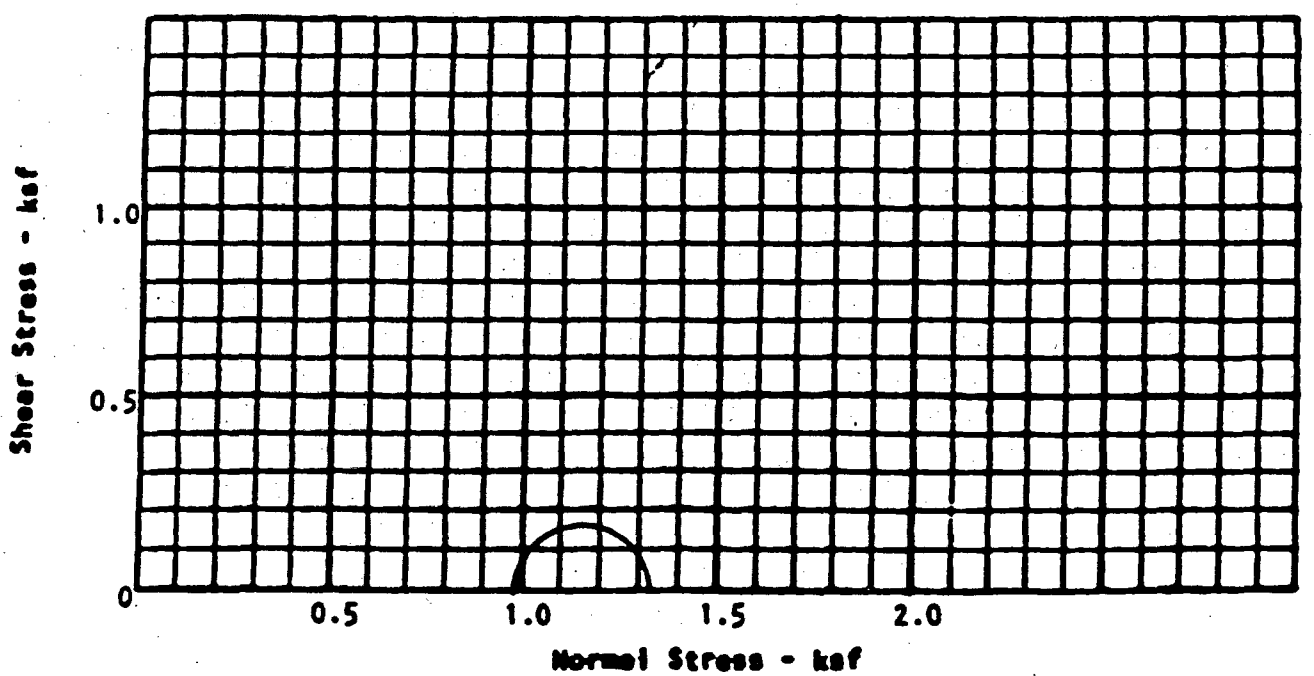
TYPE OF TEST: Saturated, Unconsolidated, Undrained

Compaction = 95.6%

TAN ϕ: N/A

ϕ: N/A

COHESION - ksf: N/A



1 126 90

TRIAXIAL SHEAR TEST RESULTS

Fig. 3

Chen & Northern, Inc.

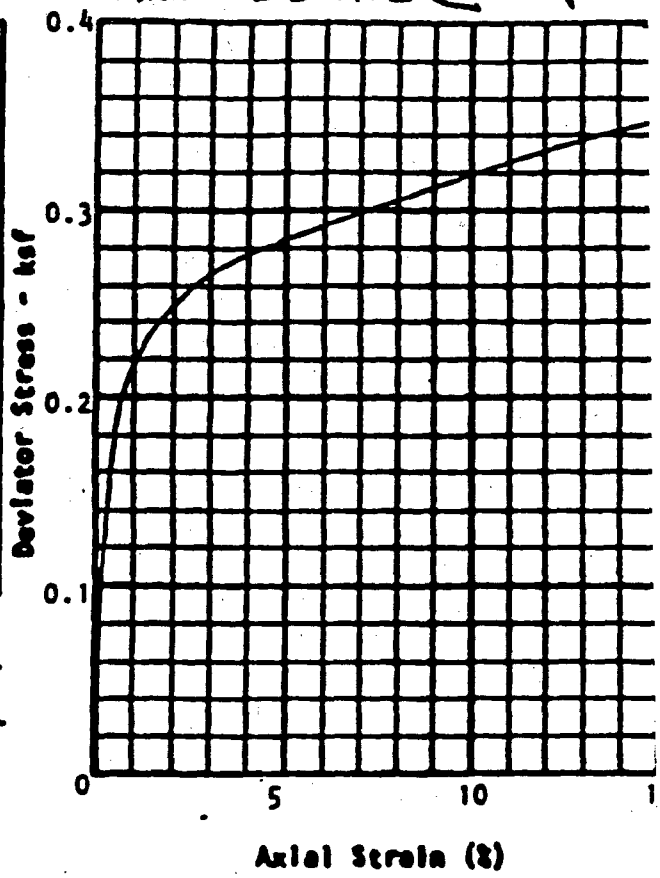
UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Compact)

284

Consulting Engineers and Scientists

Consulting Soil and Foundation Engineers

TEST NUMBER	1	2	3	4
LOCATION	2B 13'-18'			
HEIGHT - INCH	3.961			
DIAMETER - INCH	1.882			
WATER CONTENT - %	18.4			
DRY DENSITY - pcf	103.0			
$\sigma_1 - \sigma_3$ - ksf	0.35			
σ_3 - ksf	0.99			
σ_1 - ksf	1.34			



TYPE OF SPECIMEN: Remolded

SOIL DESCRIPTION: Mancos Shale

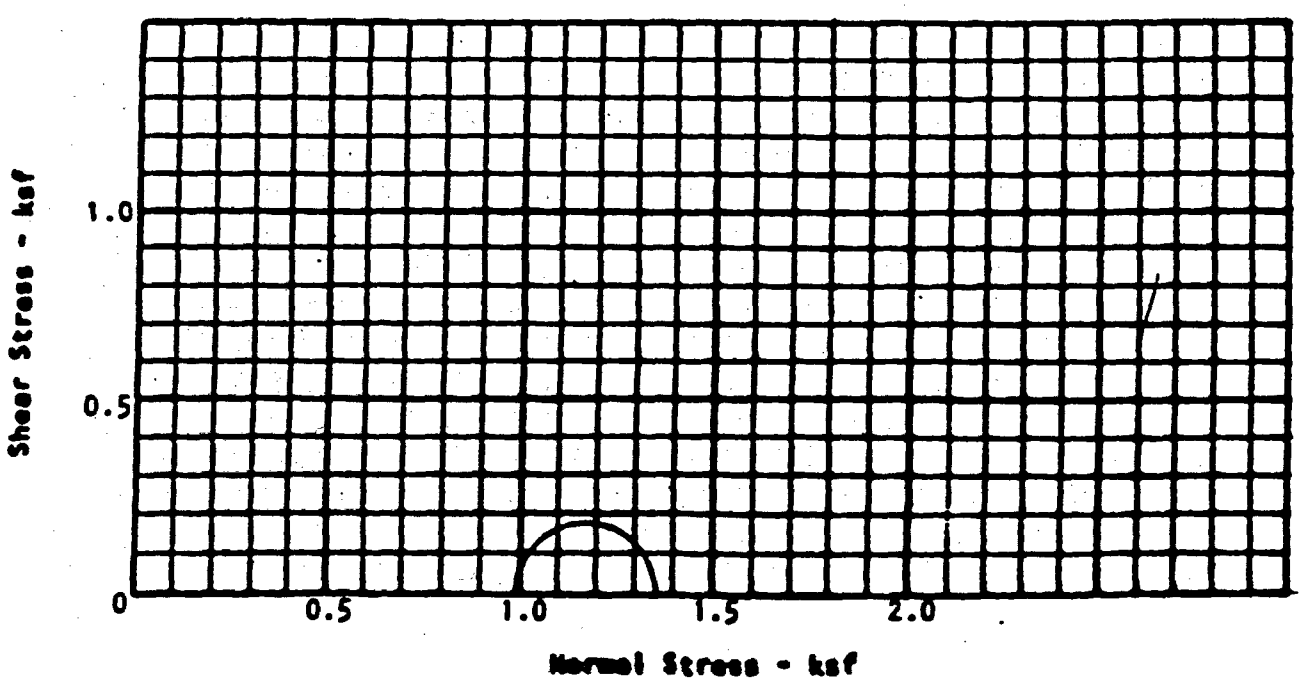
TYPE OF TEST: Saturated, Unconsolidated, Undrained

Compaction = 95.2%

TAN ϕ: N/A

ϕ: N/A

COHESION - ksf: N/A



1 126 90

TRIAXIAL SHEAR TEST RESULTS

Fig. 4

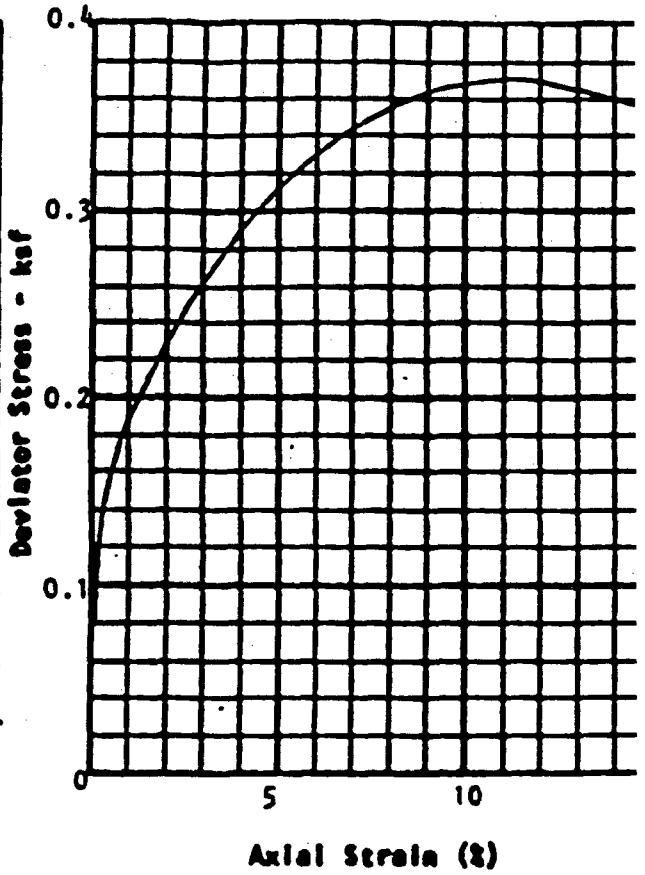
Chen Northern, Inc.

UMTRA-GRJ
5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Compacted)

285

Consulting Soil and Foundation Engineers

TEST NUMBER	1	2	3	4
LOCATION	3A 18'-23'			
HEIGHT - INCH	3.953			
DIAMETER - INCH	1.882			
WATER CONTENT - %	18.4			
DRY DENSITY - pcf	102.1			
$\sigma_1 - \sigma_3$ - ksf	0.37			
σ_3 - ksf	0.99			
σ_1 - ksf	1.36			



TYPE OF SPECIMEN: Remolded

SOIL DESCRIPTION: Mancos Shale

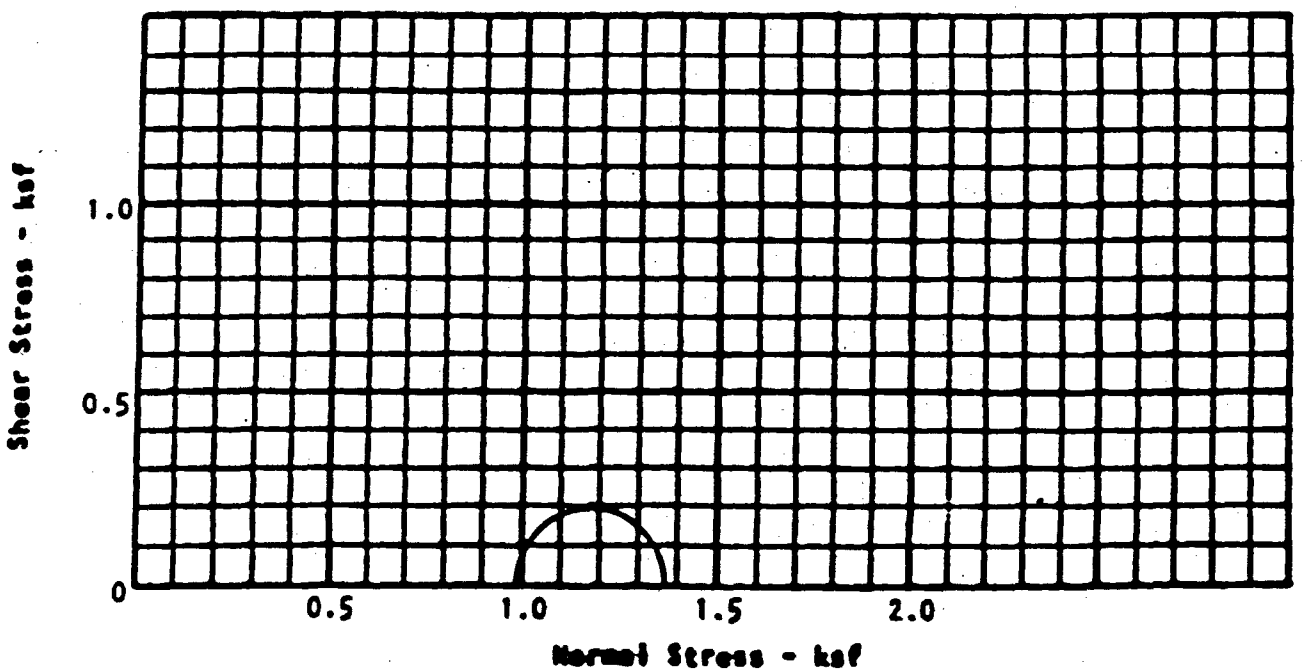
TYPE OF TEST: Saturated, Unconsolidated, Undrained

Compaction = 95.6%

TAN ϕ: N/A

ϕ: N/A

COHESION - ksf: N/A



1 126 90

TRIAXIAL SHEAR TEST RESULTS

Fig. 5

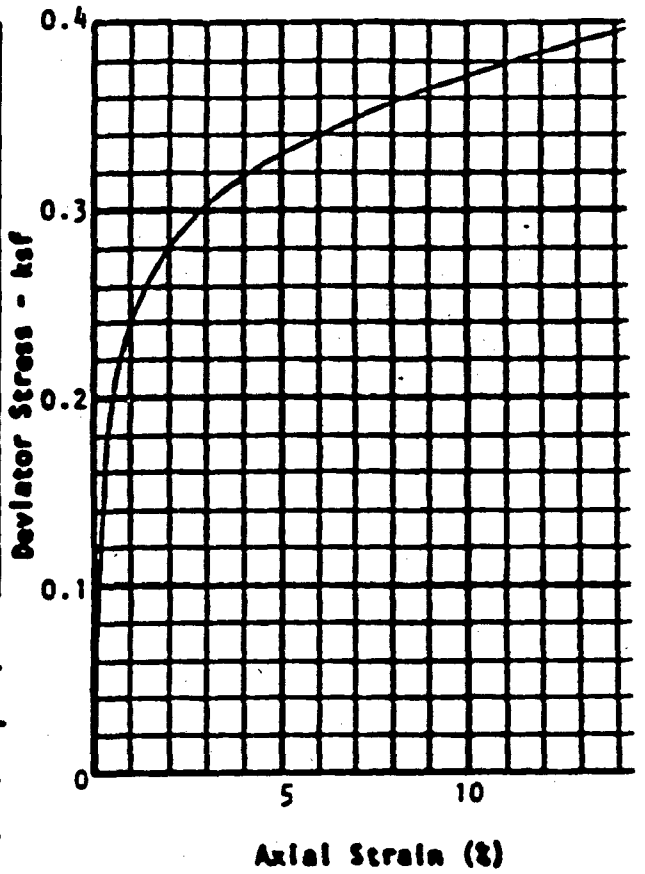
Chen Northern, Inc.

5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Compacted)

286

Consulting Soil and Foundation Engineers

TEST NUMBER	1	2	3	4
LOCATION	3B 23'-28'			
HEIGHT - INCH	3.955			
DIAMETER - INCH	1.882			
WATER CONTENT - %	17.6			
DRY DENSITY - pcf	105.3			
$\sigma_1 - \sigma_3$ - ksf	0.40			
σ_3 - ksf	0.99			
σ_1 - ksf	1.39			



TYPE OF SPECIMEN: Remolded

SOIL DESCRIPTION: Mancos Shale

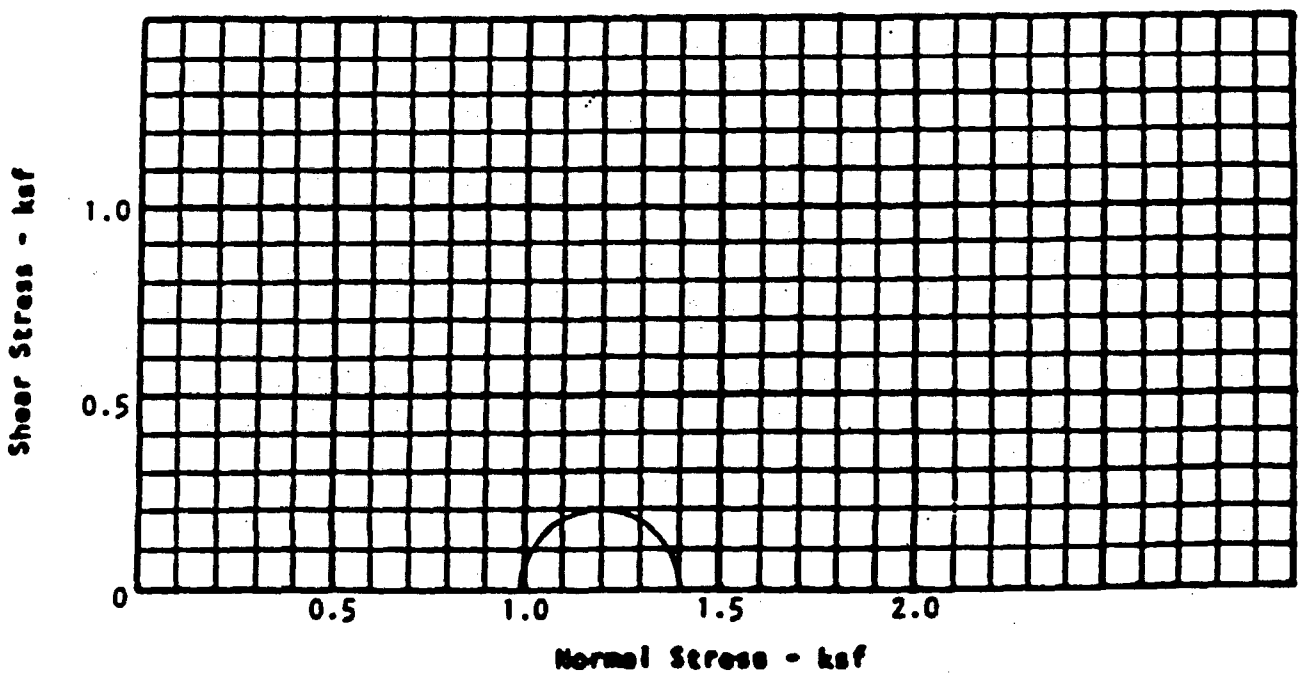
TYPE OF TEST: Saturated, Unconsolidated, Undrained

Compaction = 95.3%

TAN ϕ: N/A

ϕ: N/A

COHESION - ksf: N/A



1 126 90

TRIAXIAL SHEAR TEST RESULTS

Fig. 6

R. 2-15-90 PS 287

Chen Northern, Inc.

MKE DOC. No. 5025-GRJ-L-09-03986-00

UMTRA - GRJ
5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Compacted)

February 2, 1990

MK-FERGUSON CO.
ALBUQUERQUE
FEB 07 1990
RECEIVED

MK - Ferguson Company
P.O. Box 9136
Alburquerque, New Mexico 87119

Attention: Mr. Jerry Hymes

Subject: Laboratory Testing, P.O. No. 3050-511-10034, Change Order No. 1

Job No. 1 126 90

Dear Mr. Hymes:

As requested, we have completed the triaxial permeability tests on the Mancos shale samples from the UMTRAP Cheney site. The results are presented in Table IV and Figs. 1 through 5. Also enclosed are copies of all worksheets.

If you have any questions regarding this submittal, please call.

Sincerely,

CHEN-NORTHERN, INC.

Sally K. Miller

Sally K. Miller, A.E.T.
Soils Laboratory Supervisor

SKM/sej
Rev. By: JB
Enclosures

February 2, 1990
Job No. 1 126 90

CHEN-NORTHERN, INC.

TABLE IV

SUMMARY OF PERMEABILITY TEST RESULTS

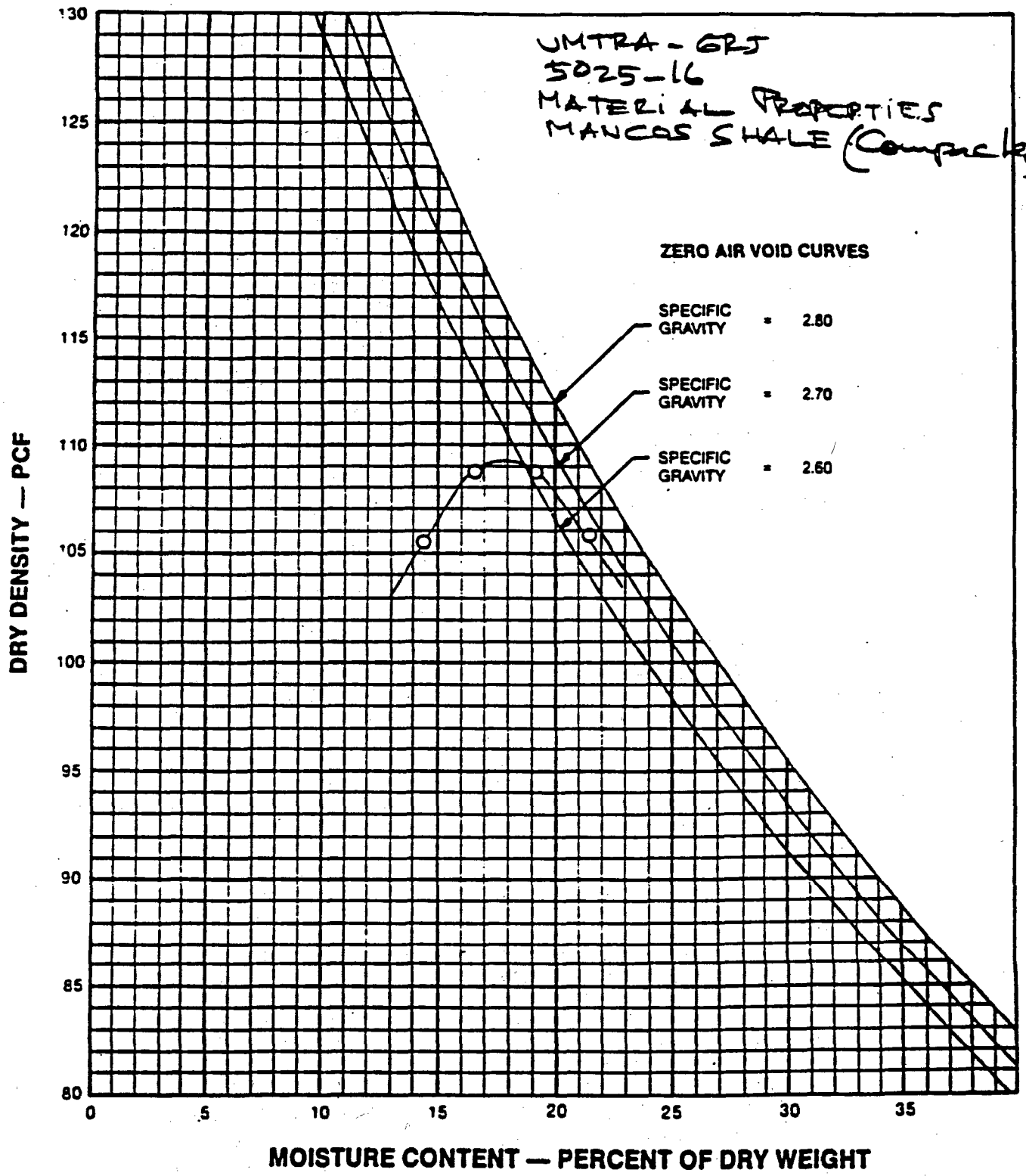
Sample No.	Depth, Feet	Initial Moisture Content, %	Initial Dry Density, pcf	Initial Compaction, %	Final Moisture Content, %	Final Dry Density, pcf	Final Saturation, %	Coefficient of Permeability, cm/sec
1	5 - 8	17.8	98.3	90.1	31.6	90.9	99.7	6.5×10^{-7}
1	5 - 8	16.8	105.4	96.6	30.2	95.3	100+	1.4×10^{-7}
1	5 - 8	16.2	110.7	101.5	27.9	96.4	100+	4.6×10^{-8}
2A	8 - 13	17.6	105.6	96.0	28.4	97.4	100+	4.3×10^{-7}
2B	13 - 18	17.8	103.7	95.8	29.2	94.2	99.8	9.6×10^{-7}
	AVERAGE	17.2	104.7	96			100	4.45×10^{-7}

USE AVERAGE COMPACTION = 95%

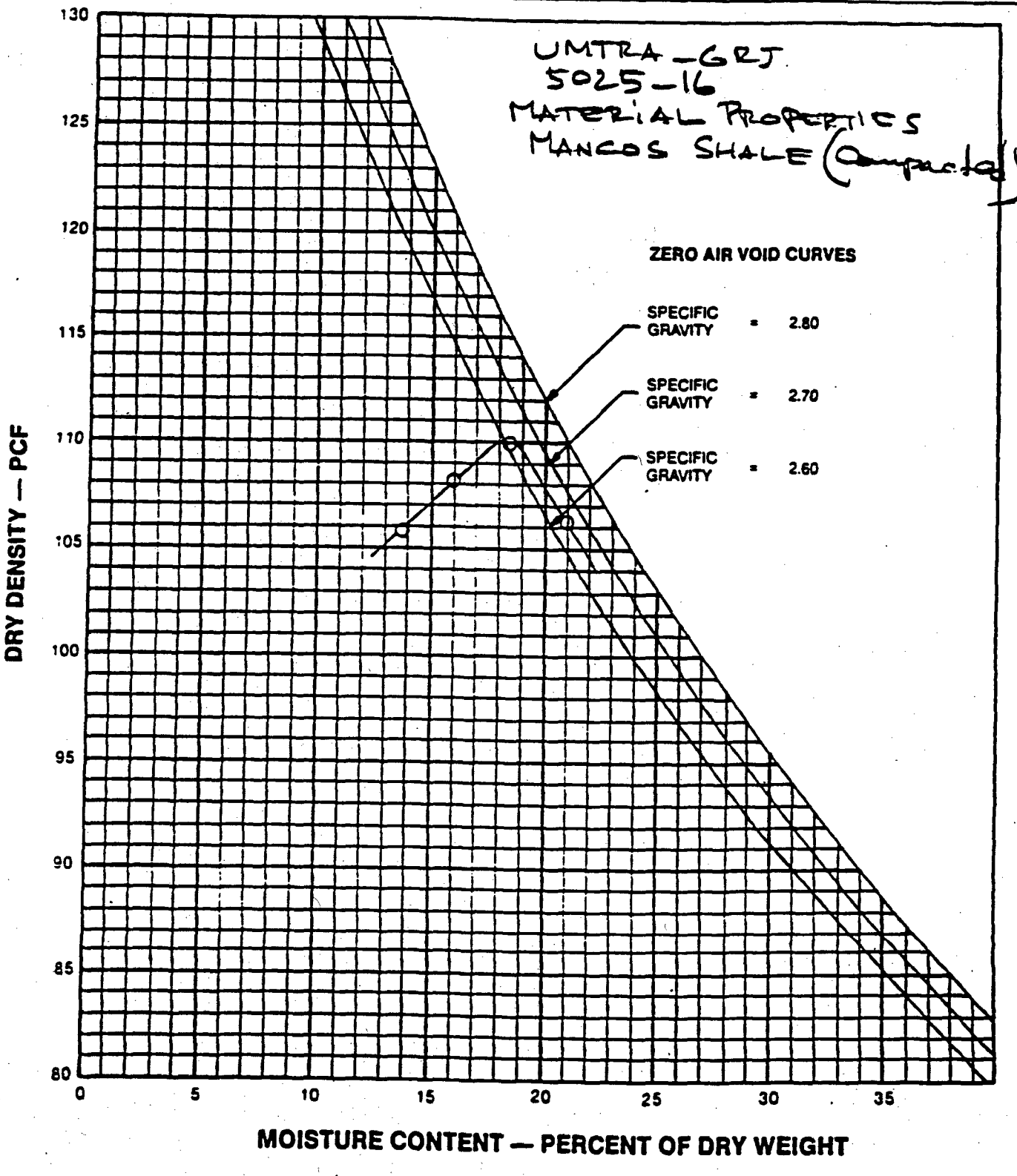
UMTRA-GJ
5025-16
MATERIAL PROPERTIES
MANCOS SHALE (Compacted)

288

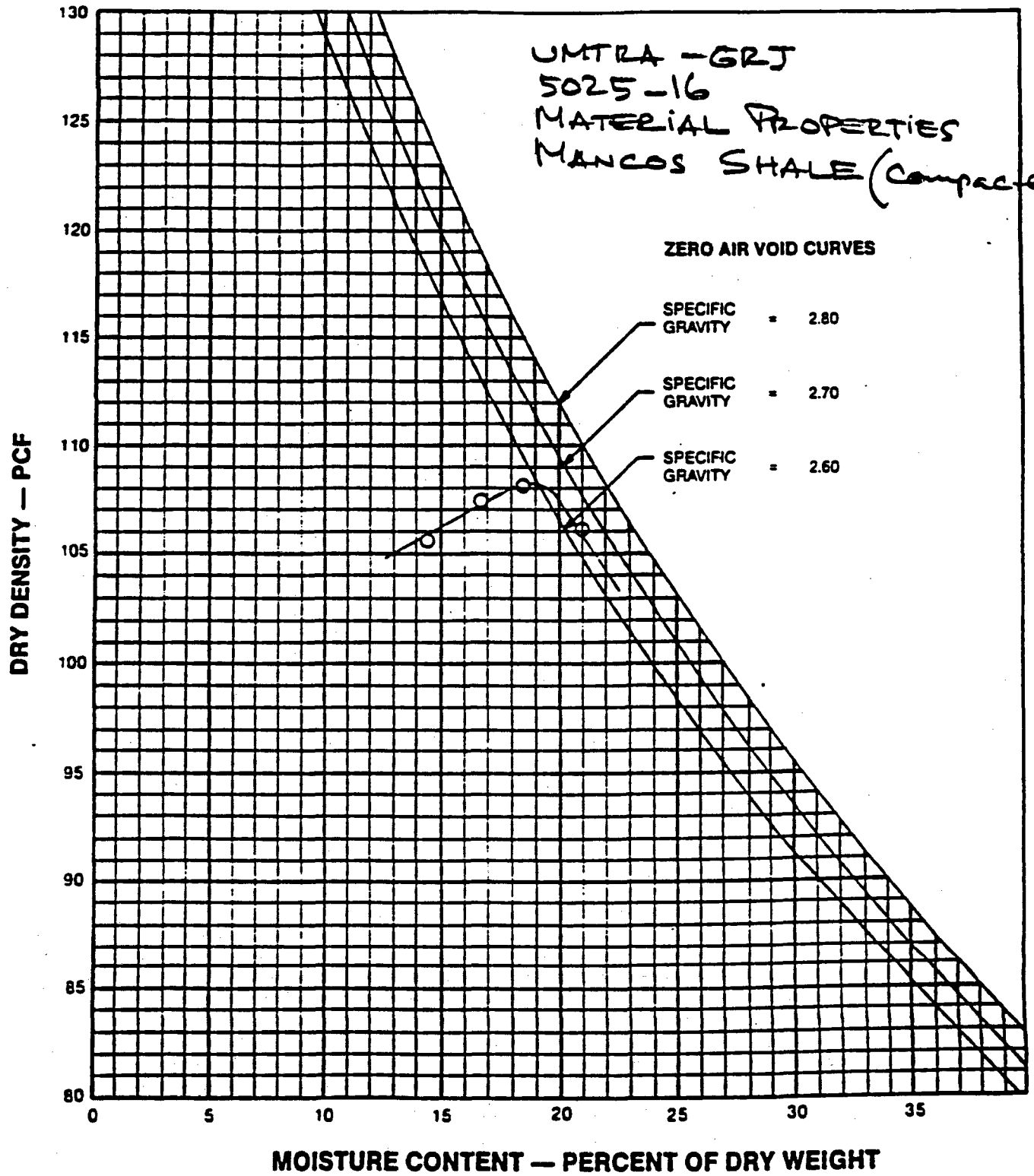
MKE DOC. No. 5025-GRJ-R-09-03987-00



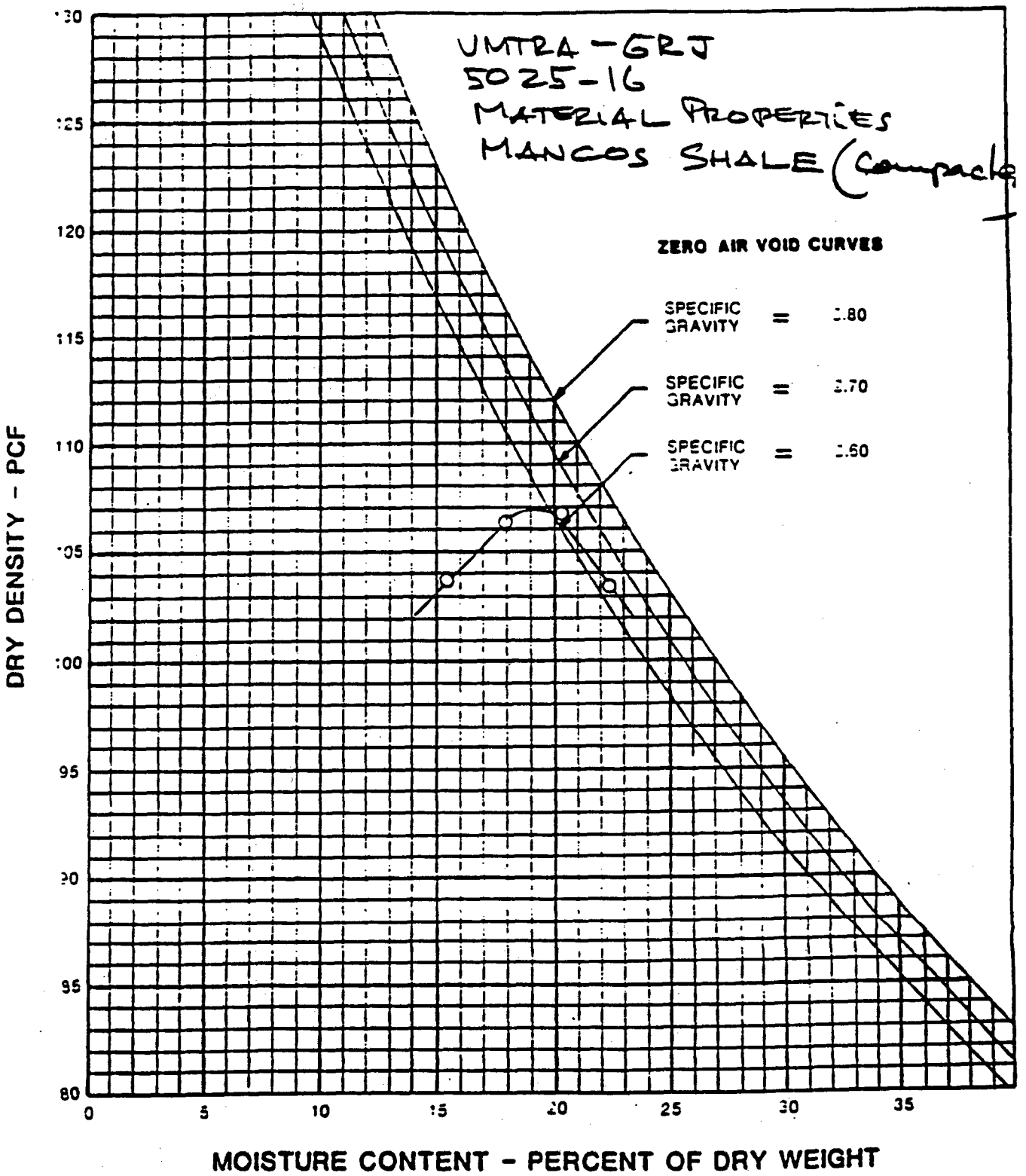
LOCATION:		MOISTURE—DENSITY RELATIONSHIPS
HOLE NO:	DEPTH: 3'-8' SAMPLE NO.: 1	
SOIL DESCRIPTION: Mancos Shale		Chen-Northern, Inc.
MAX. DRY DENSITY: 109.1 PCF	OPT. MOIST. CONTENT: 18.2 %	PROCEDURE: ASTM D698-78, Meth. "A"
LIQUID LIMIT: _____	PLASTICITY INDEX: _____	JOB NO.: 1 126 90 — FIG. NO.
GRAVEL: ____% SAND: ____%	SILT AND CLAY (-200): ____%	DATE: December 7, 1989 — 1



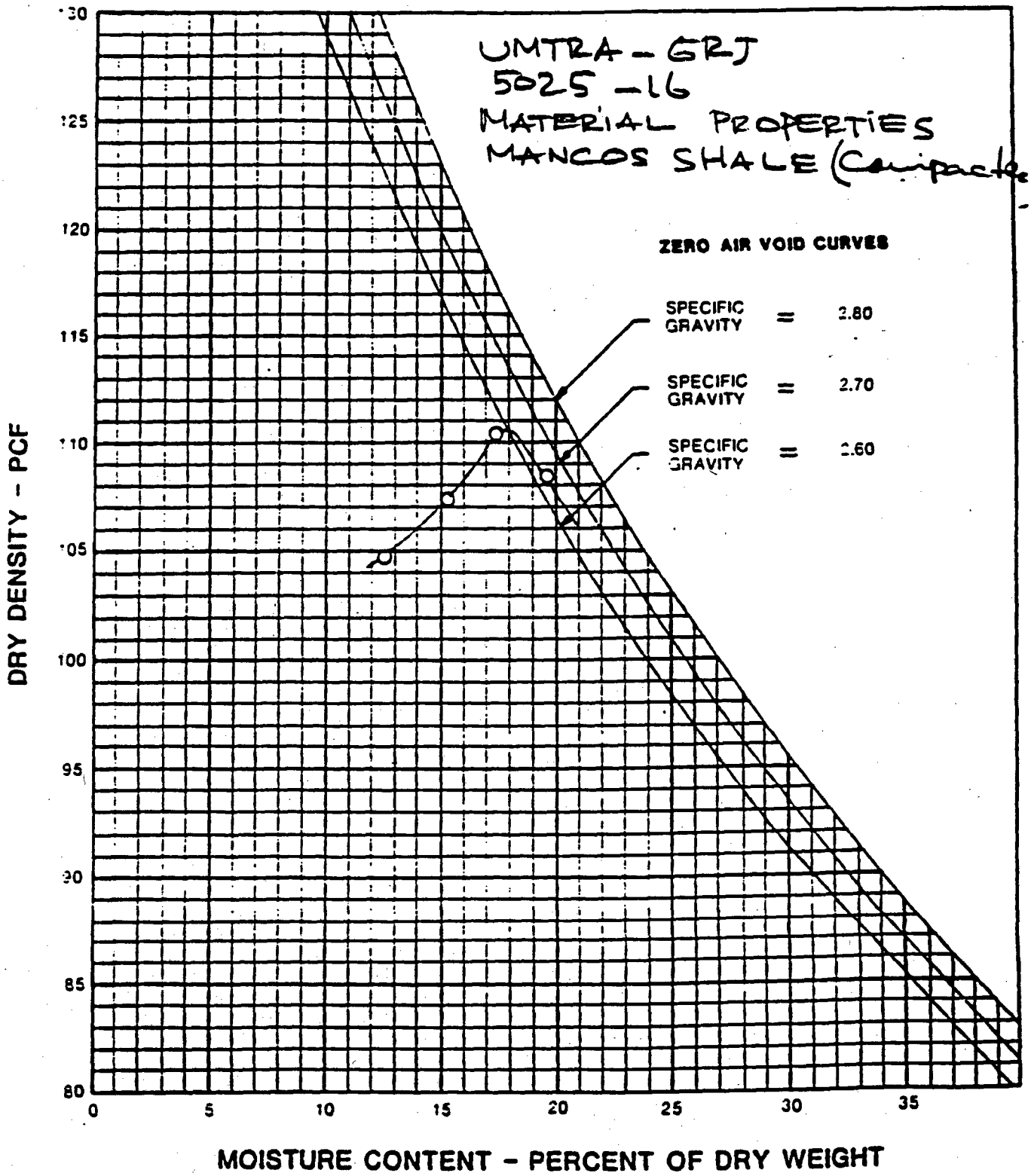
LOCATION:		MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: DEPTH: 8'-13' SAMPLE NO.: 2A		
SOIL DESCRIPTION: Mancos Shale		Chen-Northern, Inc.
MAX. DRY DENSITY: 110.0 PCF OPT. MOIST. CONTENT: 18.2 %		PROCEDURE: ASTM D698-78, Meth. "A"
LIQUID LIMIT: _____ PLASTICITY INDEX: _____		JOB NO.: 1 126 90
GRAVEL: ____% SAND: ____% SILT AND CLAY (-200): ____%		DATE: December 7, 1989
		FIG. NO. 2



LOCATION:		MOISTURE—DENSITY RELATIONSHIPS
HOLE NO: DEPTH: 13'-18' SAMPLE NO.: 2B		
SOIL DESCRIPTION: Mancos Shale		Chen-Northern, Inc.
MAX. DRY DENSITY: 108.2 PCF	OPT. MOIST. CONTENT: 19.0 %	PROCEDURE: ASTM D698-78, Meth. "A"
LIQUID LIMIT: _____	PLASTICITY INDEX: _____	JOB NO.: 1 126 90 — FIG. NO.
GRAVEL: ____% SAND: ____%	SILT AND CLAY (-200): ____%	DATE: December 7, 1989 — 3



LOCATION :		MOISTURE-DENSITY RELATIONSHIPS
HOLE NO. :	DEPTH : 18'-23' SAMPLE NO. : 3A	
SOIL DESCRIPTION : Mancos Shale		Chen-Northern, Inc.
MAX. DRY DENSITY : 106.8 PCF	OPT. MOIST. CONTENT : 19.1 %	PROCEDURE : ASTM D698-78, Meth. "A"
LIQUID LIMIT :	PLASTICITY INDEX :	JOB NO. : 1 126 90 — FIG. NO.
GRAVEL : %	SAND : % SILT AND CLAY (-200) : %	DATE : January 11, 1990 — 4



LOCATION :	MOISTURE-DENSITY RELATIONSHIPS
HOLE NO. : DEPTH : 23'-28' SAMPLE NO. : 3B	
SOIL DESCRIPTION : Mancos Shale	Chen-Northern, Inc.
MAX. DRY DENSITY : 110.5 PCF OPT. MOIST. CONTENT : 18.0 %	PROCEDURE : ASTM D698-78, Meth. "A"
LIQUID LIMIT : _____ PLASTICITY INDEX : _____	JOB NO. . 1 126 90 — FIG. NO.
GRAVEL : % SAND : % SILT AND CLAY (-200) : %	DATE : January 11, 1990 — 5

REVISION 3

-1" Material

The -1" material will be obtained from layer 2 (Gravelly Layer). It is expected to use this material as clean-fill dike on the side slope above the existing ground. The proposed compaction will be 95% based on ASTM D 698, method C

Density, Moisture Content, and Specific Gravity

Data From Sheet. 123

(a) $\gamma_{dry\ AVE.} = \left\{\frac{120.8 + 124.6 + 119.2 + 123.4 + 122.9}{5}\right\} 0.95 = 116.1$

(b) $W_{OPT.\ AVE.} = \frac{12.5 + 11.3 + 14.2 + 11.4 + 12.3}{5} = 12.3\%$

(c) Assume Specific Gravity = 2.8 (similar to sheet 245)

(d) Average Moist Density, $\gamma = 116.1 \times 1.123 = 130.3$ pcf.

(e) Average Saturated Density

void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.8 \times 62.4}{116.1} - 1 = 0.505$

$\gamma_{AVE.} = \frac{G_s \gamma_w + e \gamma_w}{1+e} = \frac{2.8 \times 62.4 + 0.505 \times 62.4}{1 + 0.505} = 137.0$ pcf

$W_{sat} = \frac{Se}{G} = \frac{100\% \times 0.505}{2.8} = 0.180$

$W_{sat.} = 18.0\%$

REV. 3

-1" MATERIAL

STRENGTH

There is no strength tests performed on "-1" material, however, we can use USBR recommendation for GC soil (clayey gravels, poorly graded gravel-sand-clay) (From Sh. 248)

For Long-Term Static: use $\phi' = 31^\circ$
$C = 0$

For Short-Term | Long-term seismic: $\phi' = 25^\circ$
$C = 0$

MKES Doc. No. 5025-GRJ-C-01-02694-02

Calculation Cover Sheet

Calc. No. 05-670-06-~~0102~~

Contract No. 5025-16 Discipline Environmental No. of Sheets 21

Project

UMTRA – GRAND JUNCTION

Feature

EMBANKMENT DESIGN

Item

SETTLEMENT AND COVER CRACKING

Sources of Data

See Sheet i

Sources of Formulae & References

See Sheet i

Preliminary Calc. ☐ Final Calc. ☒ Supersedes Calc. No. 05-670-06-00

Rev. No.	Revision	Calculation By	Date	Checked By	Date	Approved By	Date
02	RESPONDED TO TAC COMMENT	LUBIS, H	1/8/91	S.E. Bottfield	1.9.91	P. Simon	1-9-91
01	———	LUBIS, H	6/30/90	S.E. Bottfield	7/10/90	S.E. Bottfield	7/10/90

Form 42-110

UMTRA DOCUMENT REVIEW FORM

SECTION 1

Site: Grand Junction Date: 11 December 1990

Document: Calculations - Final Design

Commentor: Greg Smith

Comment: Calculation No. 05-670-06-01, Settlement and Cover Cracking

The note presented in the latest calculation set states that the extra 6 feet additional excavated depth of the Cheney disposal cell will not affect cover cracking should be backed up with some numbers. The additional excavation was made to accommodate an increased volume of VP materials. Calculate the predicted settlement of the section at maximum depth with the same tailings depth used in the original calc, but add 6 more feet of VP material. Compare the new prediction with the original settlement to show that cover cracking will not be affected.

SECTION 2

Responses: Page 3 By: H. Lubis Date: 01-10-91

Agreed. Based on the revised calculation (Rev. 2), the cover cracking will not be affected by the additional 6 feet of excavation.

Plans for Implementation:

SECTION 3

Confirmation of Implementation:

Checked by: _____ Date: _____

Approved by: _____ Date: _____

REFERENCES

1. "Embankment Design - Material Properties", MKE CALC. NO. 05-670-05-01

2. "Embankment Design - Settlements And Cover Cracking", MKE CALC. NO. 05-670-03-01

3. Lee and Shen, "Horizontal Movements Related to Subsidence", Journal Of Soil Mechanics And Foundations Division, Proceedings Of ASCE, Jan. 1969

4. UMTRA - Design Procedure, Rev. 7, Jan. 1989, MKE DOC. No. 4005-GEN-Q-01-06596-06

TABLE OF CONTENTS

	Sheet
I. Purpose	1
II. Analysis	1
III. Radon Barrier Cracking Potential	2
IV. Conclusions	2
Appendix A: Excerpts Of Computations From Reference 2	A1-A3
Appendix B	B1-B7

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project UMTRA - GRJ
Feature
Item

Contract No. 3885.34
Designed SKB
Checked PS

Sheet iii
File No.
Date 12.29.90
Date 12-21-90

EMBANKMENT DESIGN - SETTLEMENT AND COVER CRACKING

CALCULATION # 05-670-06-01

NOTE: THE SETTLEMENT AND COVER CRACKING CALCULATION WILL NOT BE AFFECTED BY 6 FEET ADDITIONAL DEPTH OF TAILINGS PLACED BELOW GRADE IN THE MANCOS SHALE AT THE BOTTOM OF THE EXCAVATION.

REV. 2
HL 1/15/91
SKB 1.15.91

The additional 6 feet of excavation was designed to accomodate the additional volume of main-pile tailings. The settlement and cover cracking calculation was based on an additional 6 feet thick of main-pile tailings at the bottom of excavation

EMBANKMENT DESIGN - SETTLEMENT AND COVER CRACKING

I. Purpose

The purpose of this calculation is to evaluate the settlement in the radon barrier. The settlement may be caused by tailings and foundation soils that underlie the radon barrier as well as placement of cover material. Excessive settlement can cause cracking of the radon barrier and ponding in localized areas.

In the previous calculation (See Appendix A), the assessments indicate that all settlements due to placement of the main-pile tailings, off-pile tailings and radon barrier will have taken place by the time the radon barrier construction is completed. Therefore, the primary concern in this calculation is the settlement due to the placement of the erosion protection material.

II. Analysis

The excavation and final grading plans are shown on Sheet 3 and 4. The idealized section for settlement analysis is shown on Sheet 5. This section is chosen because it has the highest elevation difference between the top and side slopes. The material properties used are presented on Sheet 6. The settlement is evaluated in the following conditions:

1. Settlement due to the placement of the main-pile tailings, the off-pile tailings, and the radon barrier (See Appendix A).

2. Settlement due to the placement of the riprap (The settlement that occured subsequent to the placement of the radon barrier. See Appendix B).

In addition, the radon barrier cracking potential is also evaluated here. It will be caused primarily by the differential settlement after the placement of riprap. The swell (shrinkage) potential will not be evaluated here since there is a 4-foot thick of soil and rock cover installed on the side slopes.

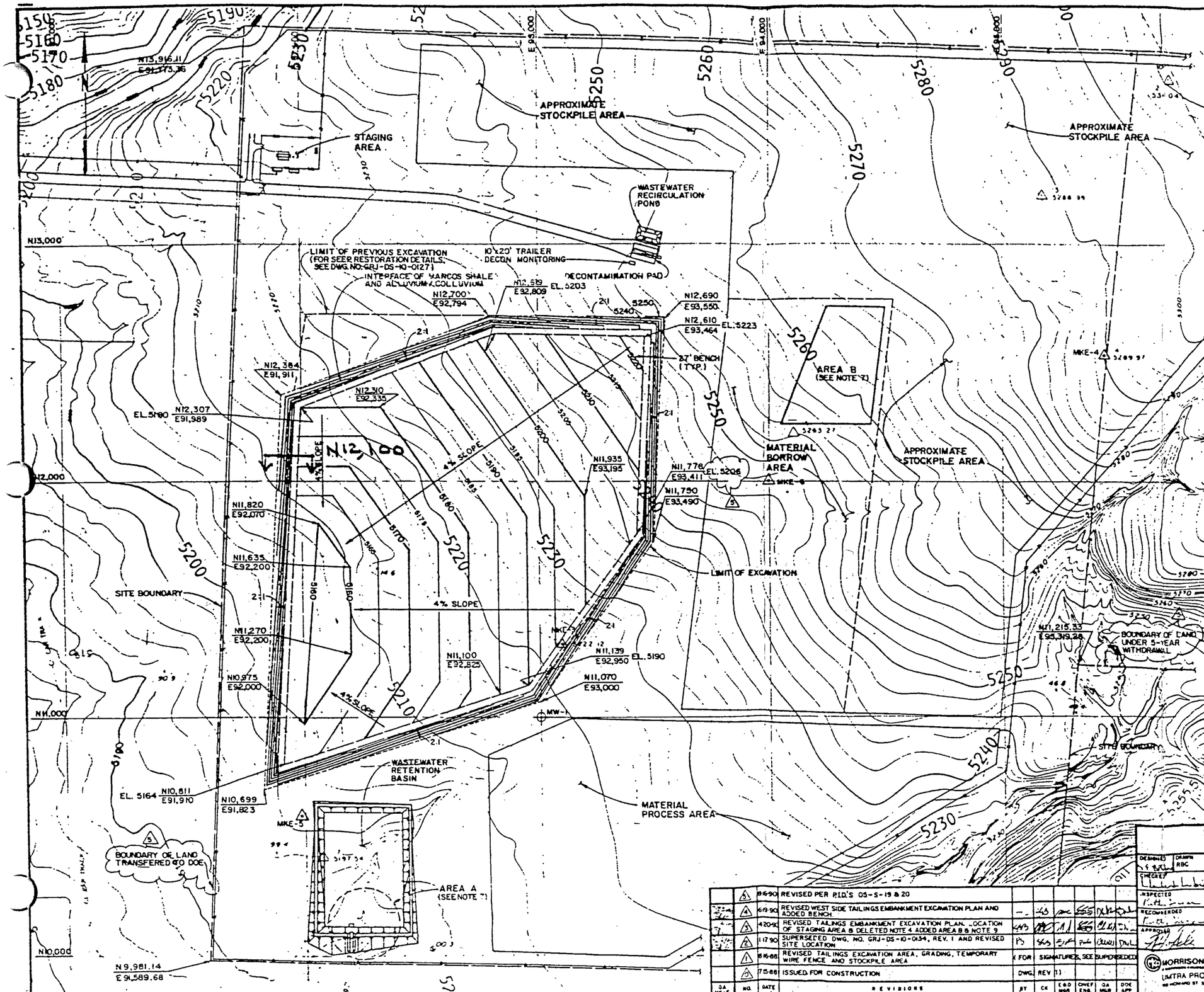
III. Radon Barrier Cracking Potential

The primary cause of radon barrier cracking is differential settlement. In this calculation, cracking due to differential settlement will be evaluated based on the Lee and Shen Method (Ref. 3). The calculations are presented in Appendix B.
Based on the computation results in Appendix B, the radon barrier cracking potential is minimal.

IV. Conclusions

The tailings embankment cover is judged safe from settlement and cracking.

REV. 2 SHT 3



NOTES:

1. ALL UNCONTAMINATED MATERIAL EXCAVATED FROM THE SITE DURING CONSTRUCTION SHALL BE PLACED TEMPORARILY WITHIN THE AREAS INDICATED WITH STABLE SLOPES NO STEEPER THAN 2(H) TO 1(V), MAXIMUM HEIGHT OF THE PILE SHALL BE 30 FEET.
2. THE SUBCONTRACTOR SHALL PROVIDE SUMP AND PUMP TO DRAIN THE PONDED NON-CONTAMINATED WATER FROM RAINSTORM AND SNOW MELT FROM THE TAILINGS EMBANKMENT AREA INTO THE NATURAL DRAIN.
3. THE SUBCONTRACTOR SHALL NOT EXCAVATE INTO THE MANCOS SHALE PRIOR TO THE ISSUANCE OF SUCH INSTRUCTIONS BY THE CONTRACTOR.
4. (DELETED)
5. THE LOCATION OF THE INTERFACE OF MANCOS SHALE WITH ALLUVIUM/COLLUVIUM IS APPROXIMATE ONLY AND IS BASED ON SOIL BORING LOGS. ACTUAL LOCATION MAY VARY IN THE FIELD.
6. THE DESIGNATED MATERIAL BORROW AREA SHALL NOT BE USED AS STOCKPILE AREA WITHOUT THE CONTRACTOR'S APPROVAL.
7. THE SUBCONTRACTOR MAY SPOIL OVERSIZE ROCK FROM THE DISPOSAL CELL EXCAVATION INTO THE DRAINAGE GULLY UNDER THE PROPOSED WASTEWATER RETENTION BASIN (AREA A) IF, AND AS AVAILABLE. IF THIS AREA IS NOT AVAILABLE, THE SUBCONTRACTOR SHALL TEMPORARILY STOCKPILE THE OVERSIZE MATERIAL IN THE DESIGNATED AREA B. HE SHALL DISPOSE OF THIS MATERIAL AS REQUIRED BY THE CONTRACTOR.
8. LOCATIONS OF WASTEWATER RECIRCULATION POND AND DECONTAMINATION PAD ARE APPROXIMATE.
9. A WORKING BENCH APPROXIMATELY 27' WIDE SHALL BE PLACED BELOW THE TRANSITION BETWEEN MANCOS SHALE AND ALLUVIUM. THE SIDESLOPE IN THE MANCOS MAY VARY AS APPROVED BY THE CONTRACTOR.

REFERENCE DRAWINGS:

GRJ-DS-10-0125, CONSTRUCTION FACILITIES AND SITE DRAINAGE
GRJ-DS-10-0223, BORINGS AND TEST PITS LOCATION PLAN

LEGEND:

- EXISTING SITE FEATURES AND CONTOURS
- N 12,000 — CONSTRUCTION GRID COORDINATE
- 5250 — CONTOURS OF EXCAVATION
- TEMPORARY CHAIN LINK FENCE AND GATE
- TEMPORARY WIRE FENCE
- ACCESS ROAD
- EXCAVATION
- EMBANKMENT
- MKE-3 — EXISTING PERMANENT SURVEY MONUMENT
- MW-1 — EXISTING WATER SUPPLY WELL

EDWIN S. SMITH, REGISTERED PROFESSIONAL ENGINEER, STATE OF COLORADO

2/15/90

200 0 200 400
SCALE FEET

NO.	DATE	REVISIONS	BY	CK	E&D MGR	CHIEF ENG.	QA MGR	DOE APP
5	8-6-90	REVISED PER RID'S OS-S-19 & 20						
4	6-9-90	REVISED WEST SIDE TAILINGS EMBANKMENT EXCAVATION PLAN AND ADDED BENCH						
3	4-20-90	REVISED TAILINGS EMBANKMENT EXCAVATION PLAN. LOCATION OF STAGING AREA B DELETED NOTE 4 ADDED AREA B & NOTE 9						
2	1-17-90	SUPERSEDED DWG. NO. GRJ-DS-10-0134, REV. 1 AND REVISED SITE LOCATION						
1	6-16-88	REVISED TAILINGS EXCAVATION AREA, GRADING, TEMPORARY WIRE FENCE AND STOCKPILE AREA						
0	7-5-88	ISSUED FOR CONSTRUCTION						

QA MGR

U. S. DEPARTMENT OF ENERGY
ALBUQUERQUE, NEW MEXICO

CHENEY RESERVOIR DISPOSAL SITE
GRAND JUNCTION, COLORADO
PHASE II CONSTRUCTION

TAILINGS EMBANKMENT AREA EXCAVATION PLAN

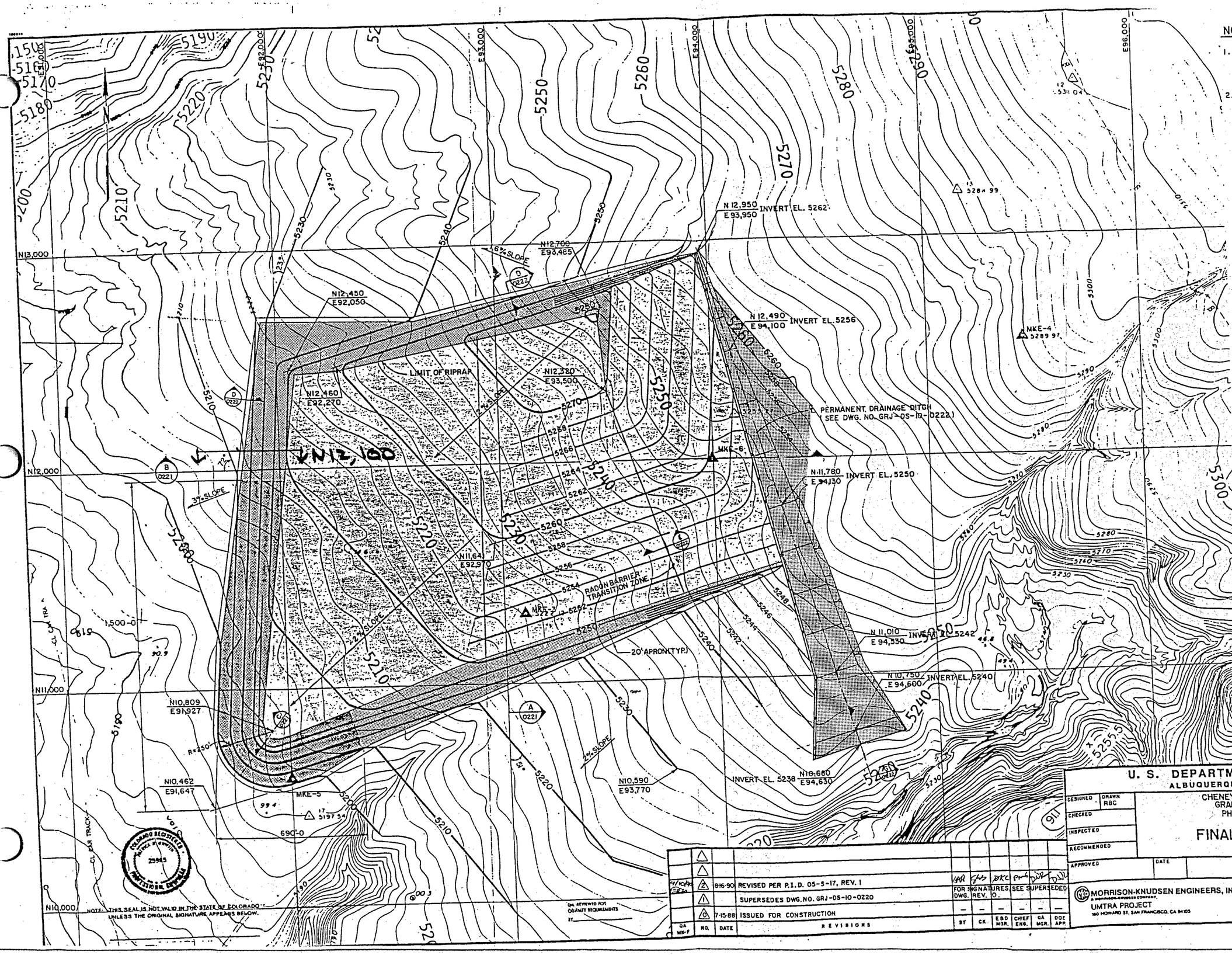
DESIGNED | DRAWN RBC | CHECKED | INSPECTED | RECOMMENDED | APPROVED | DATE 1-17-90 | E.S. Smith | DOE PROJECT ENGINEER | DATE 2-2-90

MORRISON-KNUDSEN ENGINEERS, INC.
UMTRA PROJECT
180 HOWARD ST SAN FRANCISCO CA 94105

PROJECT NO. DE-AC04-83AL18796

DRAWING NO. GRJ-DS-10-0134 REV 5

SHT. 4



NOTES:

1. WHERE CORNER CONTOURS ON THE TAILINGS EMBANKMENT ARE TOO SHARP TO BE CONSTRUCTED, CONTOURS WILL BE ROUNDED WITH A 50 FOOT RADIUS (APPROXIMATELY) IN THE PLAN VIEW.
2. THE CONTOURS OF THE TOP SLOPE ARE APPROXIMATE AND ARE DEPENDENT ON ACTUAL VOLUME OF CONTAMINATED MATERIALS RELOCATED.

REV. 2

UMTRA GRJ
3685-34
Settlement
HL 1/8/91
SKB 1.8.91

REFERENCE DRAWINGS:

GRJ-DS-10-0124	SITE PLAN
GRJ-DS-10-0125	CONSTRUCTION FACILITIES AND SITE DRAINAGE
GRJ-DS-10-0134	TAILINGS EMBANKMENT AREA & EXCAVATION PLAN
GRJ-DS-10-0221	TAILINGS EMBANKMENT-SECTIONS AND DETAILS
GRJ-DS-10-0222	DITCHES AND MISCELLANEOUS-SECTIONS AND DETAILS
GRJ-DS-10-0223	BORINGS AND TEST PITS LOCATION PLAN

LEGEND:

- 5280 — EXISTING SITE FEATURES AND CONTOURS
- PERMANENT DRAINAGE DITCH
- 5270 — FINISHED GRADE CONTOURS
- N10,000 — CONSTRUCTION GRID COORDINATES
- EXISTING PERMANENT SURVEY MONUMENT
- RIPRAP TYPE B
- RIPRAP TYPE C
- RIPRAP TYPE A
- EXCAVATION

200 0 200 400
SCALE FEET

NOTE: THIS SEAL IS NOT VALID IN THE STATE OF COLORADO UNLESS THE ORIGINAL SIGNATURE APPEARS BELOW.

COLORADO REGISTERED PROFESSIONAL ENGINEER 25945

NO.	DATE	REVISIONS	BY	CK	E&D MGR.	CHIEF ENG.	QA MGR.	DOE APP.
3								
2	8-16-90	REVISED PER P.I.D. 05-5-17, REV. 1	FOR SIGNATURES SEE SUPERSEDED DWG. REV. 0.					
1		SUPERSEDES DWG. NO. GRJ-DS-10-0220	—	—	—	—	—	—
0	7-15-88	ISSUED FOR CONSTRUCTION						

QA REVIEWED FOR QUALITY REQUIREMENTS BY ____

U. S. DEPARTMENT OF ENERGY
ALBUQUERQUE, NEW MEXICO

CHENEY RESERVOIR DISPOSAL SITE
GRAND JUNCTION, COLORADO
PHASE II CONSTRUCTION

FINAL GRADING PLAN

DESIGNED | DRAWN RBC | CHECKED | INSPECTED | RECOMMENDED | APPROVED | DATE | DOE PROJECT ENGINEER | DATE

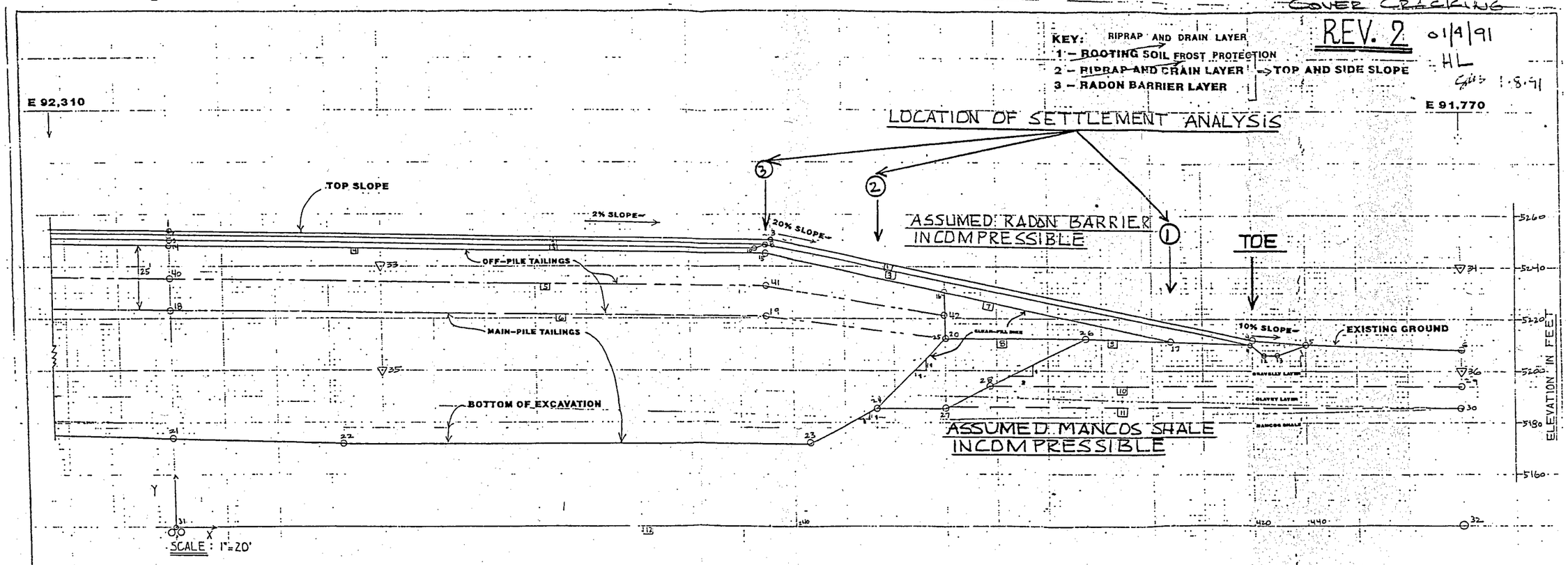
MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY
UMTRA PROJECT
180 HOWARD ST. SAN FRANCISCO, CA 94105

PROJECT NO. DE-AC04-83AL18796
DRAWING NO. GRJ-DS-10-0220

Sh 5

UMTRA-GRJ
3885-34
SETTLEMENT AND
COVER CRACKING

REV. 2 01/4/91
HL
1.8.91



CROSS SECTION FOR SLOPE STABILITY ANALYSIS

CROSS SECTION N 12,100 – 12,100

PROJECT: UM
CONTRAC

1

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project UMTRA – GRAND JUNCTION
Feature EMBANKMENT DESIGN
Item MATERIAL PROPERTIES

Contract No. 5025-16
Designed H. L.
Checked P. S.

Sheet 3
File No.
Date 6/20/90
Date 6/27/90

UMTRA-GRJ
5025-16
Settlement and Cover Grading

Sh. 6

~~REV. 3~~

SUMMARY OF DESIGN PARAMETERS

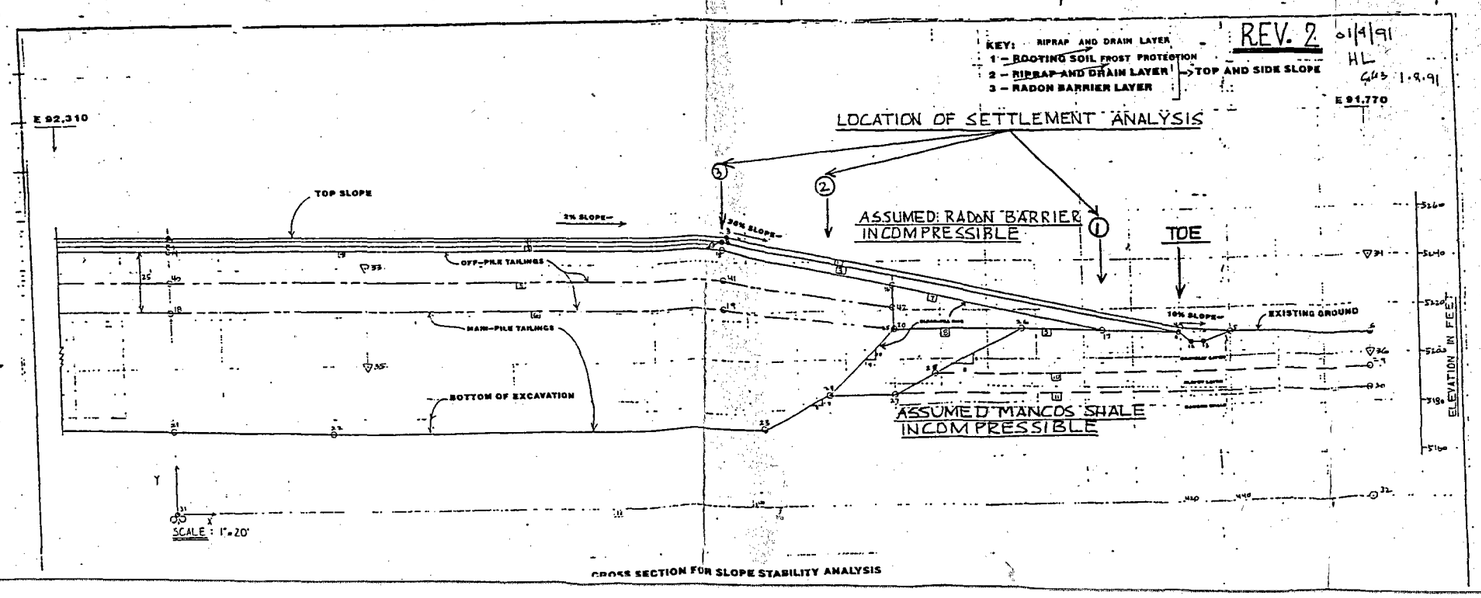
(FROM: REF. 1, Sh. 3)

H.L. 6/30/90
GWS 7-1-90

REV. 2
HL 1/8/91
GWS 1.8.91

LAYER No	DESCRIPTION	IN-PLACE DRY DENSITY γ_d, PCF	IN-PLACE DENSITY γ_t, PCF	IN-PLACE MOIST. W, %	SATURATED DENSITY γ_{SAT}	SATURATED MOIST W_{SAT}	SHORT TERM STATIC C, PSF	SHORT TERM SEISMIC Ø, DEGREE	LONG TERM STATIC C, PSF	LONG TERM STATIC Ø, DEGREE	LONG TERM SEISMIC C, PSF	LONG TERM SEISMIC Ø, DEGREE	CONSOLIDATION P_p, KSF	C_v, $\frac{FT^2}{YR}$	C_c	C_R	e	PERMEAB. k, $\frac{CM}{SEC}$
~~1~~	~~ROOTING SOIL~~	~~94.3~~	~~95.0~~	~~12.0~~	—	—	~~Su = 290 psf~~		~~0~~	~~20°~~	~~Su = 350 psf~~		—	—	—	—	—	—
2	RIPRAP | DRAIN LAYER	125.0	125.0	—	125.0	0	0	38°	0	38°	0	38°	—	—	—	—	—	—
3	RADON BARRIER 100% COMPACTION	113.7	130.6	14.9	135.2	18.9	Su = 750 psf		280	28°	Su = 750 psf		2.2	0.026	0.178	—	0.526	5.02×10^{-8}
4	OFF-PILE TAILINGS 95% COMPACT.	111.0	124.8	12.4	132.1	19.0	Su = 4000 psf * 0	18.5°	0 * 0	30° 30°	0 0	18.5° 18.5°	2.2	2.3×10^{-5}	0.23	0.007	0.437	2.0×10^{-6}
5	MAIN-PILE TAILINGS 90% COMPACT.	96.5	105.5	22	117.2	35.5	0	25°	0	35°	0	25°	1.0	1.0×10^{-3}	0.2	0.02	0.97	7.0×10^{-5}
6	CLEAN-FILL DIKE ABOVE GRND	116.1	130.3	12.3	137.0	18.0	0	25	0	31°	0	25°	—	—	—	—	—	—
7	CLEAN-FILL DIKE BELOW GRND	103.5	122.6	18.5	128.3	23.9	175	0	0	20°	0	20°	—	—	—	—	0.328	
8	LAYER 1	93.4	95	12	115.4	39.0	Su = 500 psf		0	30°	Su = 500		—	—	—	—	1.250	—
9	TOP SOIL LAYER 2	130	130	7.0	142	9.5	0	35	0	35°	0	35°	—	—	—	—	—	7.0×10^{-5}
10	GRAVELLY LAYER LAYER 3	107.4	117.3	9.2	130	21	Su = 1500 psf		0	29°	0	19°	6.0	2.3×10^{-5}	0.116	0.015	0.568	5.7×10^{-5} TO 4.4×10^{-6}
11	CLAYEY LAYER LAYER 4	133.7	145.0	10.3	145.0	10.3	Su = 5000 psf		0	35°	Su = 5000 psf						0.293	Approx. 1×10^{-7}
12	MANCOS SHALE LAYER 5 GENERAL FILL	107.2	120.7	18.5	129.3	21	280	28°	280	28°	280	28°	—	—	—	—	0.572	1.4×10^{-6} TO 5.6×10^{-7}

* For off-pile tailings greater that 12.5. thickness on the top layer, use C=0, Ø=18.5° for short term static/seismic. For settlement calculation, use saturated density.



CROSS SECTION FOR SLOPE STABILITY ANALYSIS

APPENDIX A

EXCERPTS OF COMPUTATIONS FROM REFERENCE 2

Additional Settlement due to Self-weight after placement is complete

First, let us analyze if there is any additional settlement yet to occur due to self-weight of the relocated tailings. We will use Figure 7-3 of the UMTRA Design Manual, which is presented on Sheet 7. For a 40-ft thick layer to be constructed in 2 years $m = 20$ ft/yr

$$\therefore m^2t/C_v = \frac{(20)^2 \times 2}{0.001\ ft^2/sec \times 60 \times 60 \times 24 \times 365} = 0.025$$

This is much smaller than the lowest value in the curve, e.g. 0.25. Therefore it may be inferred that there is no additional settlement left to occur due to self weight, at the time, the top of the pile is reached.

A similar computation is made for the ponds area material. For a 20-ft thick layer to be constructed in 1 year

$$m^2t/C_v = \frac{(20)^2 \times 1}{5.5 \times 10^{-5} \times 60 \times 60 \times 24 \times 365} = 0.23$$

This is time for single drainage. However, for the ponds area material it is actually a double drainage case, since drainage will take place both through the top, and into the main pile tailings. Therefore the value of m^2t/C_v will be considerably low for a compressible double drainage layer of constant thickness. That value is 0.25 not that of a single layer. Additionally, this value of C_v is conservative. Thus, no additional settlement is expected in this layer.

PS 2/15/88
WYL 2/16/88

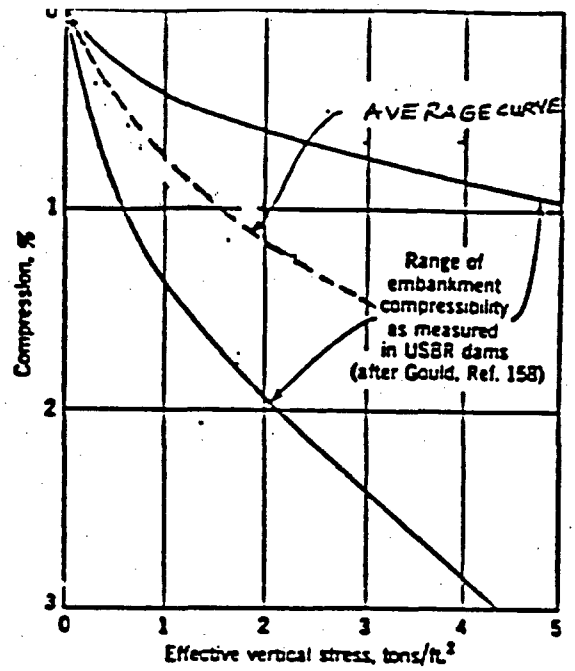


FIGURE 7-2 -- COMPRESSION VERSUS EFFECTIVE VERTICAL STRESS FOR USBR DAMS (after Refs. 7-6 and 7-7)

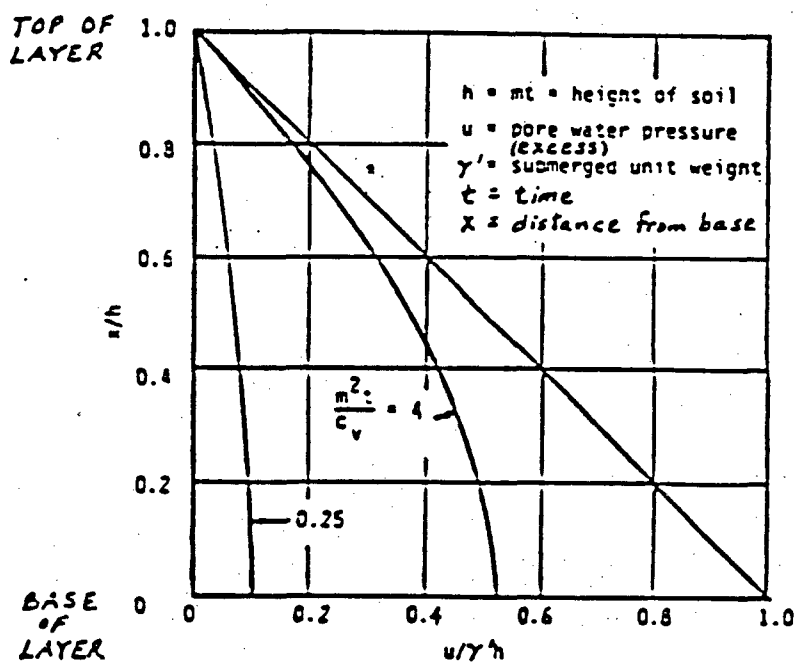


FIGURE 7-3 -- CONSOLIDATION IN SOIL LAYER INCREASING IN HEIGHT WITH TIME (IMPERMEABLE BASE) (after Refs. 7-4 and 7-8)

Total Settlement of the Hard Clays at End of Radon Barrier Placen

Calculated for Section (5) in Sheet 5.

$$p_o = 116.1 \times (15.5)/2 \approx 900 \text{ psf}$$

$$\Delta p = (2.0 \times 121.5) + (21.0 \times 131.3) + (38.0 \times 105.5)$$

$$= 243.0 + 2{,}757.3 + 4{,}009.0 = 7{,}009.3$$

$$\therefore p_o + \Delta p = 7009.3 + 900 = 7909 \text{ psf}$$

$$\therefore \text{Total settlement} = \frac{1}{1+e_o}\left[C_r \log \frac{6000}{900} + C_c \log \frac{7009}{6000}\right] 15.5$$

$$= \frac{1}{1.568}\left[0.0124 + 0.0078\right] 15.5$$

$$= 0.4917 \text{ ft} = \underline{5.9 \text{ inches}}$$

90 percent consolidation takes place at t_{90}, where

$$\frac{C_v t_{90}}{H^2} = 0.848.$$

Assuming one-way drainage due to the very low permeability of the Mancos Shale below, $H = 15.5'$; $C_v = 2.3 \times 10^{-5}$ ft²/sec.

$$\therefore t_{90} = \frac{0.848 \times (15.5)^2}{2.3 \times 10^{-5}} = 8{,}858{,}000 \text{ seconds} = 0.28 \text{ years}$$

Therefore it is reasonable to assume that virtually all settlement will have taken place by the end of construction of the radon barrier.

APPENDIX B

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project: [illegible]
Feature: EMBANKMENT DESIGN
Item: SETTLEMENT

Contract No. [illegible] Sheet 51
Designed H.L. File No.
Checked [illegible] Date [illegible]
Date 7-1-90

LOCATION - 1 — 32 FEET FROM TOE

$\Delta P = 2 * 125 = 250$ PSF

$\Delta H = ((H*C)/(1+eo))*LOG\ (Po + \Delta P)/Po$

LAYER	THICKNESS (FEET)	γ SAT. (PCF)	Po (**) (PSF)	Po+ΔP (PSF)	Pp (PSF)	Cc OR Cr (*)	LOG (Po+ΔP)/Po	eo	ΔH (FEET)	REMARKS
RADON BARRIER	3.5	135.2								ASSUMED: R. BARRIER INCOMPRESSIBLE
GRAVELLY LAYER	17.0	142.0								
CLAYEY LAYER	8.0	130.0	3407.2	3657.2	6000	0.015	0.030751	0.568	0.002353	
MANCOS SHALE								---------	---------	ASSUMED: MANCOS SHALE INCOMPRESSIBLE
								TOTAL	0.002353	
									0.028236 INCHES	

NOTES:

(*) - OFF-PILE TAILINGS: Cc = 0.23, Cr = 0.007 (FROM REF. 1, SH. 3)

- MAIN-PILE TAILINGS: Cc = 0.2, Cr = 0.02 (FROM REF. 1, SH. 3)

USE Cc IF Po+ΔP > Pp

Cr IF Po +ΔP < Pp

(**) - Po CALCULATED AT THE CENTER OF LAYER

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project: UMTRA - GRR
Feature: EMBANKMENT DESIGN
Item: SETTLEMENT

Contract No. 5025-16
Designed: HL
Checked: SKB

Sheet: 5 of
File No.: 61301 96
Date: 6/30/96
Date: 7.1.90

LOCATION - 2 — 146 FEET FROM TOE

$\Delta P = 2 * 125 = 250$ PSF

$\Delta H = ((H*C)/(1+eo))*LOG\ (Po + \Delta P)/Po$

LAYER	THICKNESS (FEET)	γ SAT. (PCF)	Po (**) (PSF)	Po+ΔP (PSF)	Pp (PSF)	Cc OR Cr (*)	LOG (Po+ΔP)/Po	eo	Δ H (FEET)	REMARKS
RADON BARRIER	3.5	135.2	236.6							ASSUMED: R. BARRIER INCOMPRESSIBLE
OFF-PILE TAILINGS	5.0	132.1	803.45	1053.45	2200	0.007	0.117655	0.437	0.002865	
	5.0	132.1	1463.95	1713.95	2200	0.007	0.068471	0.437	0.001667	
	5.0	132.1	2124.45	2374.45	2200	0.23	0.048316	0.437	0.038666	
	5.0	132.1	2784.95	3034.95	2200	0.23	0.037334	0.437	0.029877	
MAIN-PILE TAILINGS	10.0	117.2	4361.7	4611.7	1000	0.2	0.024205	0.97	0.024573	
	10.0	117.2	5533.7	5783.7	1000	0.2	0.019190	0.97	0.019482	
	10.0	117.2	6705.7	6955.7	1000	0.2	0.015896	0.97	0.016138	
MANCOS SHALE								---------	---------	ASSUMED: MANCOS SHALE INCOMPRESSIBLE
								TOTAL	0.133272	
									1.599273 INCHES	

NOTES:

(*) - OFF-PILE TAILINGS: Cc = 0.23, Cr = 0.007 (FROM REF. 1, SH. 3)

- MAIN-PILE TAILINGS: Cc = 0.2, Cr = 0.02 (FROM REF. 1, SH. 3)

USE Cc IF Po+ΔP > Pp

Cr IF Po +ΔP < Pp

(**) - Po CALCULATED AT THE CENTER OF LAYER

MK-ENVIRONMENTAL SERVICES
A DIVISION OF MK-FERGUSON

Project: UMTRA - GJPO
Feature: EMBANKMENT DESIGN
Item: SETTLEMENT + COVER CRACKING
Contract No.: 3385-34
Designed: H. LUBIS
Checked: SKG
File No.: 8
Date: 1/8/91
Date: 1/9/91
Sheet: B3

REV. 2

REVISION 2

LOCATION - 3 -- 189 FEET FROM TOE
Δp = 1.5*125 = 187.5 PSF

$\Delta H = ((H*C)/(1+e_o))*LOG(P_o + \Delta P)/P_o$

LAYER	THICKNESS (FEET)	γSAT (PCF)	po (**) (PSF)	po+Δp (PSF)	Pp (PSF)	Cc OR Cr (*)	LOG (po+Δp)/po	eo	ΔH	REMARKS
RADON BARRIER	3.5	135.2	236.60							ASSUMED: R. BARRIER INCOMPRESSIBLE
OFF-PILE TAILINGS	5.0	132.1	803.45	990.95	2200	0.007	0.091093	0.437	0.002219	
	5.0	132.1	1463.95	1651.45	2200	0.007	0.052339	0.437	0.001275	
	5.0	132.1	2124.45	2311.95	2200	0.230	0.036732	0.437	0.029396	
	5.0	132.1	2784.95	2972.45	2200	0.230	0.028297	0.437	0.022645	
	5.0	132.1	3445.45	3632.95	2200	0.230	0.023013	0.437	0.018557	
MAIN-PILE TAILINGS	10.0	117.2	4361.70	4549.2	1000	0.200	0.018279	0.970	0.018557	
	10.0	117.2	5533.70	5721.2	1000	0.200	0.014472	0.970	0.014692	
	10.0	117.2	6705.70	6893.2	1000	0.200	0.011976	0.970	0.012158	
	10.0	117.2	7877.70	8065.2	1000	0.200	0.010216	0.970	0.010372	
	8.0	117.2	8932.5	9120.0	1000	0.200	0.009022	0.970	0.007328	
MANCOS SHALE								======= TOTAL	========== 0.137059	ASSUMED: MANCOS SHALE INCOMPRESSIBLE
									1.644708"	

NOTES:

(*) - OFF-PILE TAILINGS: Cc = 0.23, Cr = 0.007 (FROM REF. 1, SH. 3)
- MAIN-PILE TAILINGS: Cc = 0.20, Cr = 0.02 (FROM REF. 1, SH. 3)
USE Cc IF po+Δp > Pp
Cr IF po+Δp < Pp

(**) - po CALCULATED AT THE CENTER OF LAYER

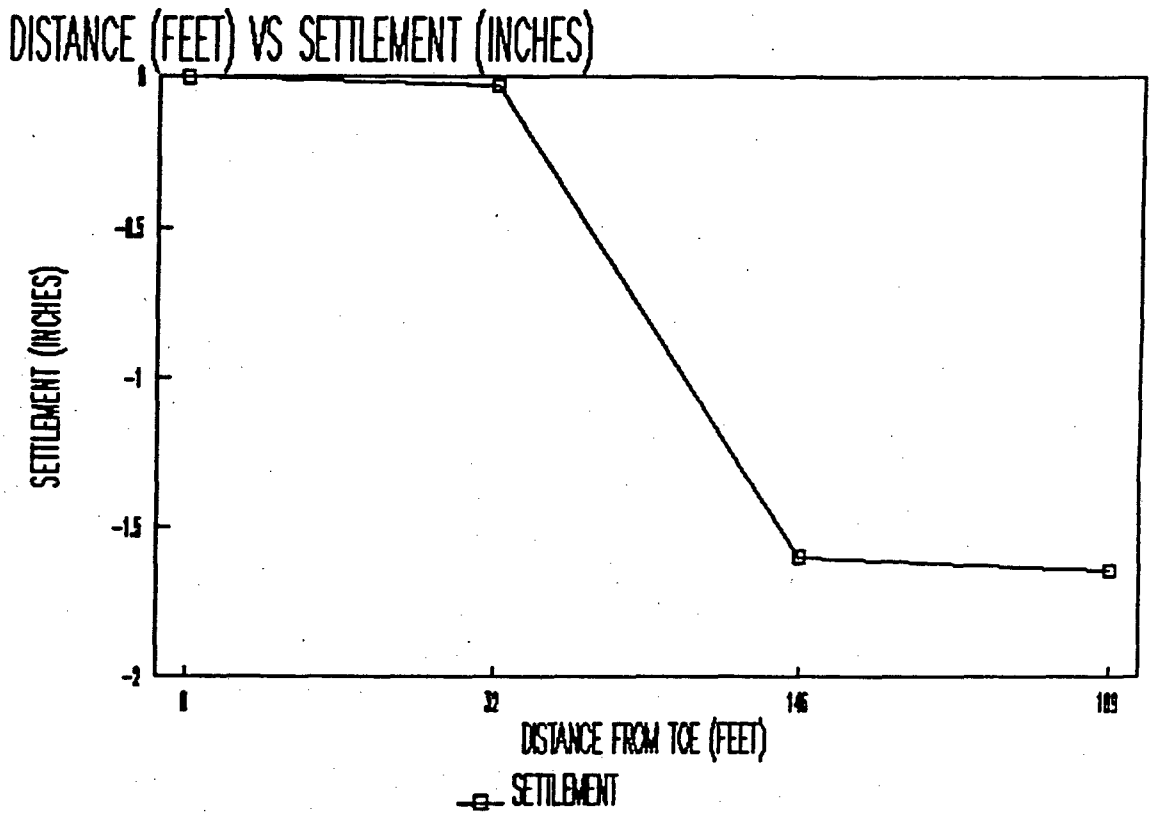
MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

Project UMTRA - GJ
Feature EMBANKMENT DESIGN
Item SETTLEMENT + COVER CRACKING

Contract No. 3885-34
Designed HL
Checked SWS

Sheet B4
File No.
Date 1/8/91
Date 1.8.91

REV. 2



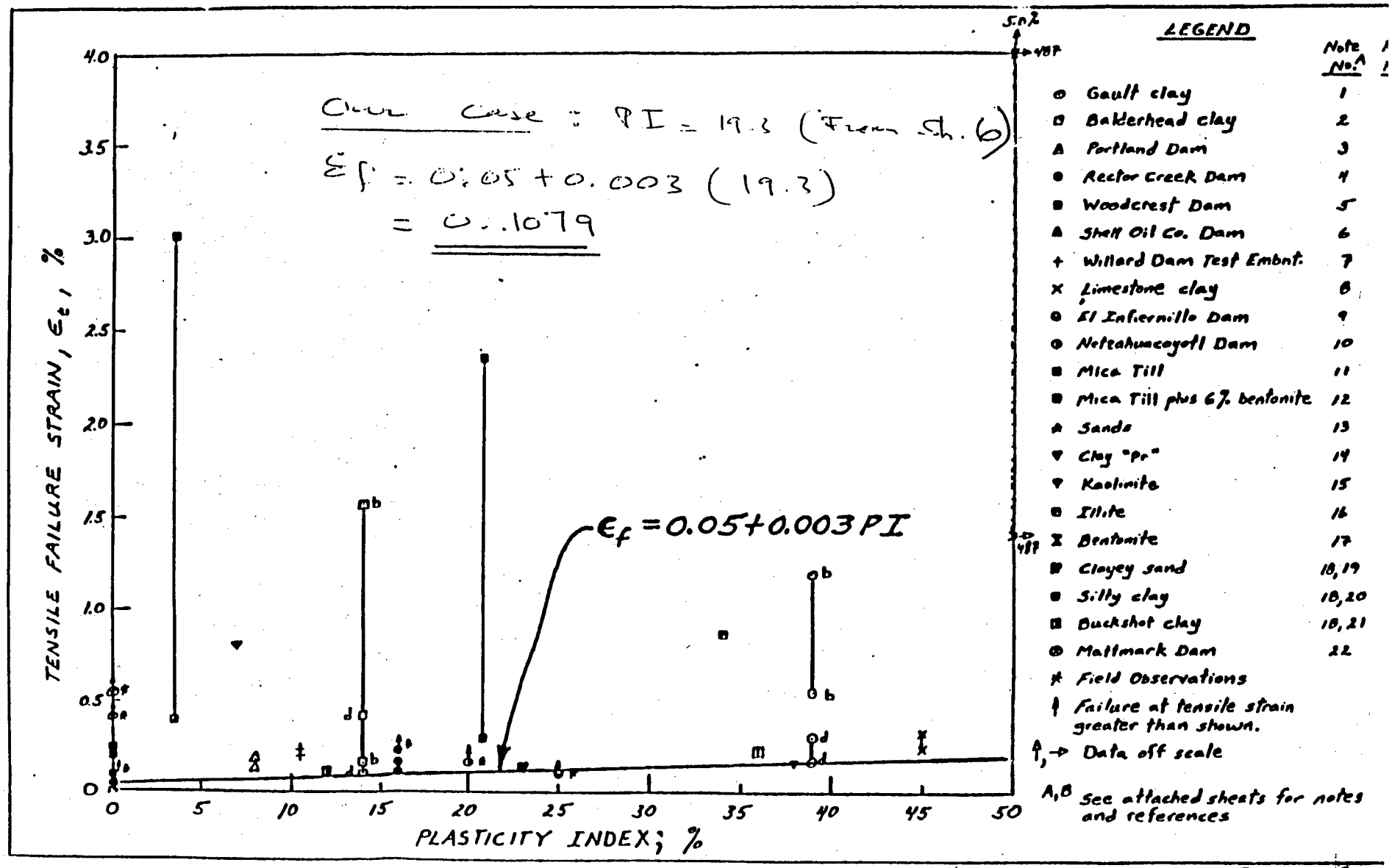


Figure 14-1: Tensile Failure Strain vs. Plasticity Index for Various Soils (Sheet 1 of 5) FROM: REF. 4

GEOTEK-GRC
SD 25-16
Sellinger
H.L. 6/30/90
Sly 7.1.90

that shown in Fig. 1, and, therefore, it was felt that the average movements from the two tests were sufficiently accurate for comparison with predicted values. The measured values of horizontal movements are compared with the movements predicted from Eq. 1 for the case of $h = H$ shown by the dashed line in Fig. 4. This case corresponds to no slippage and therefore a neutral axis at the bottom of the beam, which might logically be expected to apply for these tests because of friction between the soil and the wooden bottom of the box. However, it is clear that Eq. 1 with $h = H$ predicts excessively large horizontal movements, even compared to the case with moving sides where arching would lead to excessively large horizontal surface movements.

The average of these test data is reproduced in Fig. 5 along with the average horizontal strains determined from this data. The nature of the measured data is similar to that shown in Fig. 1, which represents the form of movements observed from many field cases.

An attempt was made to fit Eq. 2 with the average values of observed horizontal movement. As shown in Fig. 5 it was found that the observed movements were very closely represented by Eq. 2 using a value of $k = 2/3$, i.e.,

$$m = \frac{2}{3} H\alpha \qquad (3)$$

In this calculation:

$m = \frac{2}{3} H \times S$

where
m = horizontal movement
H = thickness of rubber barrier
S = local slope, determined from graph on Sh. B4

THEORETIC) (FEM)

The finite-el l method of extreme genera ems in soil mechanics. Clou ormulation for use with higl e of its use for soil mechani odward (4).

The values of ere determined by a serie e soil was compacted to 1 to which it was compacted ire ranging from 5 psi to 2 the initial tangent to the s e confining pressure by

$$E = 2000\sigma \qquad (4)$$

in which E = th = the value of the confining p ip is similar to those determined for other granular soils and it was assumed that a relationship of this same form could also be used to represent the modulus of the styrofoam. The analytical results indicated that the computed horizontal movements were not particularly sensitive to modulus. Therefore, Eq. 4 was used to calculate the modulus for various depths below the surface of the model beam. For these calculations the confining pressure was taken as one-half of the vertical overburden pressure. Thus, the value of E used in the analysis increased in direct proportion to the square root of the depth. For most of the studies the value of Poisson's ratio was assumed equal to 0.3, which agrees with measured values from other granular soils.

The boundary conditions used in the FEM studies were as follows: (1) All points on the bottom surface moved vertically down a prescribed amount to

conform to the imposed subsidence profile of the test; (2) all points on each end were prevented from any horizontal movements, but were free to move vertically; and (3) all points on the surface and in the interior were given no restrictions. No external loads or body forces were applied. The only source of deformations was the induced subsidence at the bottom of the beam.

For most of the analyses the beam was divided into 248 elements each 3 in. × 3 in. square. However, analyses performed using twice as many elements gave results which were the same, indicating that the computed values of stress, strain, and displacement were not influenced by the finite-element representation chosen for the analyses.

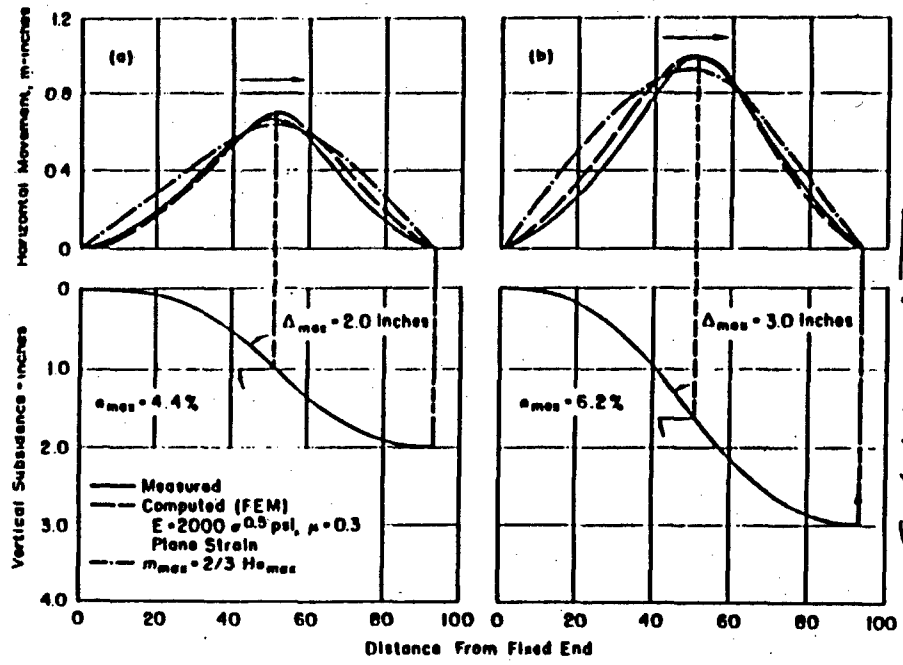


FIG. 6.—MEASURED AND COMPUTED SURFACE HORIZONTAL MOVEMENTS

The horizontal surface movements and horizontal strains computed by the FEM for a maximum 1-in. vertical deflection are shown in Fig. 5. It may be noted that these movements are remarkably similar to the measured movements, and to those computed by Eq. 3. Similar comparisons for the data from the tests for 2-in. and 3-in. maximum deflections are shown in Fig. 6. Again there is good agreement between the measured values and the two methods of computation.

EFFECT OF DIFFERENT ELASTIC PROPERTIES

The good agreement between the FEM and the experimental data suggested that the FEM would be useful in investigating other more complicated situa-

From: Ref. 3

CONTRA-GRT 5025-16 Settlement H.L. Gilbert 6/30/96 Sh. 7.

MK-ENVIRONMENTAL SERVICES
A DIVISION OF MK-FERGUSON

Project UMTRA - GRJ
Feature EMBANKMENT DESIGN
Item SETTLEMENT

Contract No. 3885-34
Designed L.L
Checked SWB

Sheet B7
File No.
Date 1/8/91
Date 1.8.91

REV. 2

Calculation Of Strains

HORIZONTAL MOVEMENT AT LOCATION 1 - 32 FEET FROM TOE

ALPHA = (0.0282-0.0)/(32*12) = 0.000073

m = (2*H*12*ALPHA)/(3) = (2*3.5*12*0.000073)/(3) = 0.0020"

HORIZONTAL MOVEMENT AT LOCATION 2 - 146 FEET FROM TOE

ALPHA = (1.5993-0.0282)/((146-32)*12) = 0.0011

m = (2*H*12*ALPHA)/(3) = (2*3.5*12*0.0011)/(3) = 0.0308"

HORIZONTAL MOVEMENT AT LOCATION 3 - 189 FEET FROM TOE

ALPHA = (1.6447-1.5993)/((189-146)*12) = 0.000088

m = (2*H*12*ALPHA)/(3) = (2*3.5*12*0.000088)/(3) = 0.0025"

HORIZONTAL STRAIN FROM LOCATION 1 - LOCATION 2

HORIZONTAL STRAIN = (0.0308-0.0020)/(114*12) = 0.000021 -------- 0.0021%

HORIZONTAL STRAIN FROM LOCATION 2 - LOCATION 3

HORIZONTAL STRAIN = (0.0308-0.0025)/(43*12) = 0.00005 -------- 0.005%

THE LARGEST STRAIN = 0.005% -------THIS IS LESS THAN 0.1079% (SHEET B5)

* RADON POTENTIAL CRACKING WILL NOT OCCUR