Before the

UNITED STATES NUCLEAR REGULATORY COMMISSION

Docket No. 50-466

Allens Creek Nuclear Generating Station Unit 1

Amendment 50 to the

PSAR

Houston Lighting & Power Company, applicant in the above captioned proceeding, hereby files Amendment 50 to the Preliminary Safety Analysis Report filed in connection with its application.

Amendment 50 consists of additional PSAR information related to issues identified in telephone conversations between Houston Lighting & Power Company and the Nuclear Regulatory Commission.

Respectfully submitted

HOUSTON LIGHTING & POWER COMPANY

E. a. Turner

E. A. Turner
Vice President
Power Plant Construction
& Technical Services

7901170261

STATE OF TEXAS

COUNTY OF HARRIS

E. A. TURNER, being first duly sworn, deposes and says: That he is Vice President of HOUSTON LIGHTING & POWER COMPANY, an Applicant herein; that the foregoing amendment to the application has been prepared under his supervision and direction; that he knows the contents thereof; and that to the best of his knowledge and belief said documents and the facts contained therein are true and correct.

DATED: This 9th day of Jan, 1979.

Signed: E. A. Turner
E. A. Turner

Subscribed and sworn to before me
this 9th day of Jan, 1979.

Rita P. Villanueva
Notary Public in and for the
County of Harris, State of Texas

My commission expires
4-30-79.

HOUSTON LIGHTING & POWER COMPANY
ALLENS CREEK NUCLEAR GENERATING STATION - UNIT NO. 1
PRELIMINARY SAFETY ANALYSIS REPORT
AMENDMENT NO. 50
INSTRUCTION SHEET

This amendment contains additional information which is submitted to clarify the applicants position with regard to several of the safety review issues identified by the NRC as well as updated information. Each revised page bears the notation Am. No. 50, 1/15/79 at the bottom of the page. Vertical bars with the number 50 representing Amendment No. 50 have been used in the margin of the revised pages to indicate the location of the revision on the page. The revised pages have the question number (eg Q361.4) next to the appropriate information which responds to the question.

The following page removals and insertions should be made to incorporate Amendment No. 50 into the PSAR.

CHAPTER 2

Remove (Existing Pages)	Insert (Amendment No. 50 Pages)
14*	14*
15*	15*
25*	25*
36*	36*
48*	48*
2.5-55	2.5-55
2.5-56	2.5-56
2.5-73	2.5-73
F2.5.5-8	F2.5.5-8
F2.5.5-9	F2.5.5-9
F2.5.5-10	F2.5.5-10
F2.5.5-12	F2.5.5-12
F2.5.5-13	F2.5.5-13
-	Appendix 2.5-M1 through
-	2.5-M13
-	FM1 & FM2
-	FM3a, FM3b, FM4
-	FM5a, FM5b, FM6
-	FM7a, FM7b, FM8
-	FM9a, FM9b, FM9c
-	FM10 through FM36

*Effective Pages/Figures Listings

CHAPTER 3

Remove (Existing Pages)	Insert (Amendment No. 50 Pages)
1*	1*
8*	8*
17*	17*
3.7A-1	3.7A-1
F3.7.A-21 through 26	F3.7.A-21 through 26

CHAPTER 14

1*	1*
14.1-1a	14.1-1a
14.1-1b	14.1-1b
14.1-2	14.1-2

APPENDIX M

M361.4-1	M361.4-1

APPENDIX N

N211.3-16	N211.3-16
N361.5-1	N361.5-1

*Effective Pages/Figures Listings

EFFECTIVE PAGES LISTING (Cont'd)
CHAPTER 2
SITE CHARACTERISTICS

Page No.	Amendment No.
2.5-46	20
2.5-46a	20
2.5-46b	48
2.5-47	-
2.5-48	20
2.5-48a	-20
2.5-48b	20
2.5-48c	20
2.5-48d	20
2.5-48e	20
2.5-48f	20
2.5-48g	20
2.5-48h	20
2.5-49	36
2.5-50	--
2.5-51	38
2.5-51a	36
2.5-52	38
2.5-52a	36
2.5-53	4
2.5-54	41
2.5-54a	41
2.5-55	50
2.5-55a	42
2.5-56	50
2.5-57	36
2.5-58	36
2.5-59	20
2.5-60	20
2.5-61	20
2.5-62	20
2.5-62a	20
2.5-63	20
2.5-63A	20
2.5-64	20
2.5-65	20
2.5-66	42
2.5-66a	20
2.5-67	20
2.5-67a	20
2.5-68	38
2.5-68a	3
2.5-68b	4

EFFECTIVE PAGES LISTING (Cont'd)
CHAPTER 2
SITE CHARACTERISTICS

Page No.	Amendment No.
2.5-69	4
2.5-69a	41
2.5-70	41
2.5-70a	36
2.5-71	4
2.5-72	38
2.5-73	50
2.5-73a	42
2.5-74	42
2.5-75	44
2.5-75a	42
2.5-76	3
2.5-77	4
2.5-78	38
2.5-78a	4
2.5-79	38
2.5-79a	36
2.5-80	--
2.5-81	--
2.5-82	38
2.5-83	--
2.5-84	--
2.5-85	--
2.5-86	--
2.5-87	--
2.5-88	4
2.5-89	4
2.5-90	4
2.5-90a	4
2.5-90b	4
2.5-91	4
2.5-92	38
2.5-93	4
2.5-94	--
2.5-95	--
2.5-96	--
2.5-97	--
2.5-98	--
2.5-99	38
2.5-100	38
2.5-100a	38
2.5-101	38
2.5-102	--
2.5-103	38
2.5-103a	38
2.5-103b	38

EFFECTIVE PAGES LISTING (Cont'd)
CHAPTER 2
SITE CHARACTERISTICS

Page No.	Amendment No.
i (Appendix L, Section 2.5)	32
ii	32
iii	32
iv	32
v	32
vi	32
vii	32
1-1	32
1-2	32
2-1	32
3-1	32
3-2	32
4-1	32
4-2	32
5-1	32
5-2	32
5-3	32
5-4	32
6-1	32
7-1	32
7-2	32
7-3	32
7-4	32
7-5	32
7-6	32
2.5-M1 (Appendix M, Section 2.5)	50
2.5-M2	50
2.5-M3	50
2.5-M4	50
2.5-M5	50
2.5-M6	50
2.5-M7	50
2.5-M8	50
2.5-M9	50
2.5-M10	50
2.5-M11	50
2.5-M12	50
2.5-M13	50

EFFECTIVE FIGURES LISTING (Cont'd)
CHAPTER 2
SITE CHARACTERISTICS

Figure No.	Amendment No.
2.5.5-3	4
2.5.5-4	4
2.5.5-5	4
2.5.5-6	4
2.5.5-7	4
2.5.5-8	50
2.5.5-9	50
2.5.5-10	50
2.5.5-11	4
2.5.5-12	50
2.5.5-13	50
2.5.5-14	4
2.5.6-1	--
2.5.6-2A	20
2.5.6-2A (Cont.)	20
2.5.6-2B	20
2.5.6-2B (Cont.)	20
2.5.6-2C	20
2.5.6-2C (Cont.)	20
2.5.6-2D	20
2.5.6-2D (Cont.)	20
2.5.6-2E	20
2.5.6-2F	20
2.5.6-2F (Cont.)	20
2.5.6-2G	20
2.5.6-2G (Cont.)	20
2.5.6-2H	20
2.5.6-2H (Cont.)	20
2.5.6-2I	20
2.5.6-2I (Cont.)	20
2.5.6-2J	20
2.5.6-2J (Cont.)	20
2.5.6-2K	20
2.5.6-2K (Cont.)	20
2.5.6-2L	20
2.5.6-2L (Cont.)	20
2.5.6-2M	20
2.5.6-2M (Cont.)	20
2.5.6-2N	20
2.5.6-2N (Cont.)	20
2.5.6-2O	20
2.5.6-2O (Cont.)	20
2.5.6-2P	20
2.5.6-2P (Cont.)	20
2.5.6-2Q	20
2.5.6-2Q (Cont.)	20
2.5.6-2R	20
2.5.6-2R (Cont.)	20

EFFECTIVE FIGURES LISTING (Cont'd)
CHAPTER 2
SITE CHARACTERISTICS

Figure No.	Amendment No.
18	32
19	32
20	32
21	32
22	32
23	32
24	32
M1 (Appendix M, Section 2.5)	50
M2	50
M3a	50
M3b	50
M4	50
M5a	50
M5b	50
M6	50
M7a	50
M7b	50
M8	50
M9a	50
M9b	50
M9c	50
M10	50
M11	50
M12	50
M13	50
M14	50
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M20	50
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M22	50
M23	50
M24	50
M25	50
M26	50
M27	50
M28	50
M29	50
M30	50
M31	50
M32	50
M33	50
M34	50
M35 & 36	50

2.5.4.5.3 Gradation Limitations and Compaction Requirements for Engineering Fill

a) Class I-a Fill - (to be used for seismic Category I structures within the Nuclear Plant Island) - Well-graded sand and gravel having a maximum size of six inches and containing a maximum of 15 percent passing the No. 200 sieve. The source of this soil will be the silty sands, sands, and gravels of the Montgomery formation obtained from the various plant area excavations. Class I-a fill shall be compacted to 95 percent of the maximum density obtained from the Shake Table Test (ASTM D2049) or 95 percent of the maximum obtained from the Modified Proctor Test ASTM D1557 (Method D) whichever yields the higher value. Based upon a continuing statistical study, the 95 percent value will be revised upwards or downwards to yield the required design in-place relative density of 80 percent. Maximum, minimum density tests, modified proctor tests, static strength tests and dynamic strength tests have been performed on bulk samples of the actual material to be used as Class I-a fill, at the specified densities. An evaluation of this data indicates that adequate strength is provided at these densities. The data and a discussion of the results are presented in Appendix L to Section 2.5. The tolerances and minimum density acceptance criteria will be specified in the backfill specifications. Refer to Section 2.5.4.5.4. The compaction requirement is to yield a minimum design relative density of 80 percent. 42(C) 36(U) 50

b) Class I-b Fill - (to be used for construction of the Ultimate Heat Sink Diversion Dike and Causeway) - Clay material having a plasticity index greater than 30 and with at least 70 percent passing the No. 200 sieve. The source of this soil will be the clays of the recent flood plain deposits, obtained from a borrow area within the cooling lake. Class I-b fill shall be compacted to 95 percent of the maximum density obtained from the Standard Proctor Test (ASTM D 698). Laboratory tests have been performed on samples of the actual material to be used as Class I-b fill, at the specified densities. An evaluation of this data indicates that adequate strength is provided at these densities. Refer to Section 2.5.6.6 for a discussion of laboratory testing. 42 Q 361.4

c) Class II Fill - (to be used for non-seismic structures) - Granular soil capable of practical compaction using standard equipment and techniques. The source of this material will be the silty sands, sands, and gravels of the Montgomery formation which do not meet the gradation requirements for Class I fill and obtained from the 42(C)

tions for Class III fill will be prepared later; as these clay soils are difficult to work, the specification will be based upon a proposed full-scale compaction test section, further discussed in Section 2.5.5.4. | 50(U)
Class II requirements on density will be that 10 percent will be allowed to fall below the specified density, with the absolute minimum 5 percent below the specified requirements. |42(U)

All compaction operations shall be closely monitored with field tests in accordance with the provisions of the Quality Assurance Manual established for this project. Refer to Chapter 17 for additional information concerning the Quality Assurance Manual.

2.5.4.6 Groundwater Conditions

a) Existing Groundwater Conditions

In the site area, groundwater is found unconfined in the Montgomery formation and in the Recent alluvial clays in the Brazos River floodplain. Groundwater levels have been recorded periodically at the site since September, 1972, by means of observation the excavation for the western portion of the heat sinks will encounter the thin portion of the sand deposits which pinch out toward the east. If this deposit is encountered, a clay liner may be required over the sand layer to prevent seepage losses from the heat sink excavations. After more thorough investigations of the heat sink area, a construction program to reduce or prevent seepage will be proposed.

2.5.5 SLOPE STABILITY

2.5.5.1 Slope Characteristics

Four main types of cross-sections will be analyzed for slope stability. These sections are: 36(U)

a) the natural bluff of the cooling lake (particularly in front of the main plant area),

b) the constructed cooling lake dam,

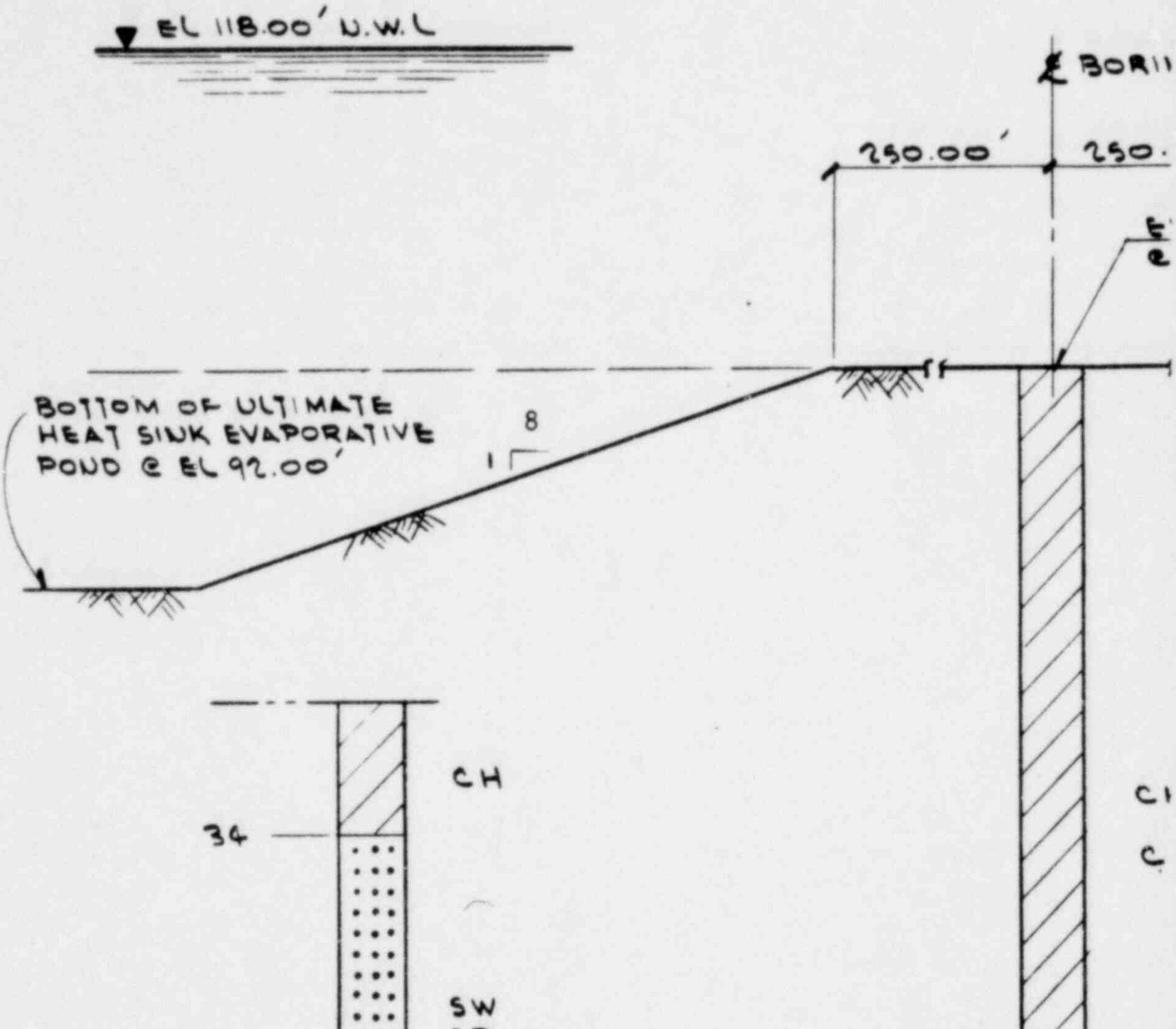
c) the constructed cooling lake diversion dike, and

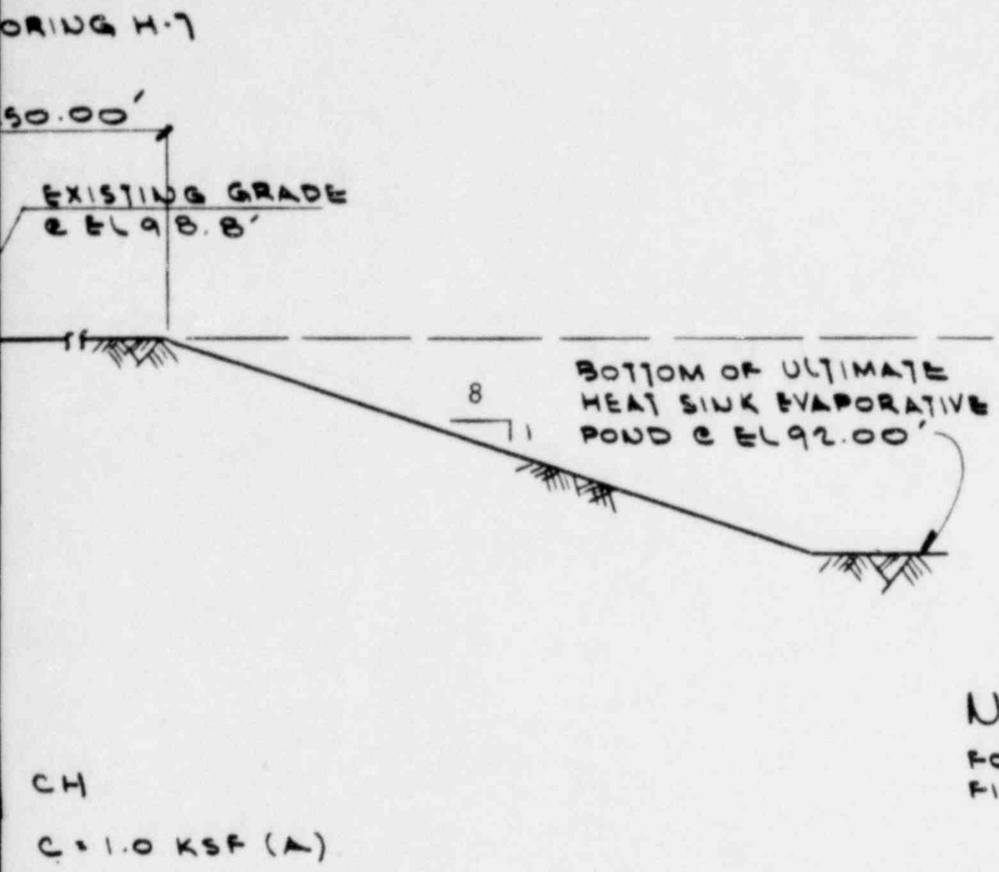
d) the constructed diversion dikes and slopes of the ultimate heat sinks.

Detailed cross-sections of slopes a), b) and d) including conservatively assumed water levels and conservative soil properties are provided in Figures 2.5.5-1 through 2.5.5-14. Table 2.5.5-1 presents the calculated safety factors. Revised cross-sections of slopes a), b) and d) and cross-section c) in addition to a revised Table 2.5.5-1 will be presented by amendment to update these analyses with the new locations (1977) of cooling lake facilities. 0,3 Q2.73 4 36(U)

The geologic conditions at the site have been discussed in detail in Section 2.5.1.2. As presented in Section 2.5.4.3, the soil deposits at the site consist of dense to very dense sands and stiff to hard clays. Laboratory tests have been performed on remolded samples of borrow material. Refer to Sections 2.5.6.4, 2.5.6.5, and 2.5.6.6 for results. The high shear strengths and low compressibility as evidenced by the laboratory test results on recompacted samples indicate that adequate factors of safety against slope failure should be obtained when detailed calculations are performed. The minimum safety factors for both static and dynamic conditions are given below in Sections 2.5.5.2.1 and 2.5.5.2.2.

A summary of the properties of embankments and foundation soils underlying all the slopes are presented with the detailed cross-sections. All man-made slopes excluding temporary constructed slopes will have a geometrical configuration of 1 vertical to 6 horizontal. This slope is conservative for the types of soils encountered at the site. The soil properties are substantiated by the laboratory data presented 50


EL 118.00' N.W.L
℄ BORI
250.00'
250.
E
@
BOTTOM OF ULTIMATE HEAT SINK EVAPORATIVE POND @ EL 92.00'
8
1
CH
34
SW
SP
Ø = 38° (R)
C
C



NOTES

FOR GENERAL NOTE SEE
FIGURE 2.5.5-8

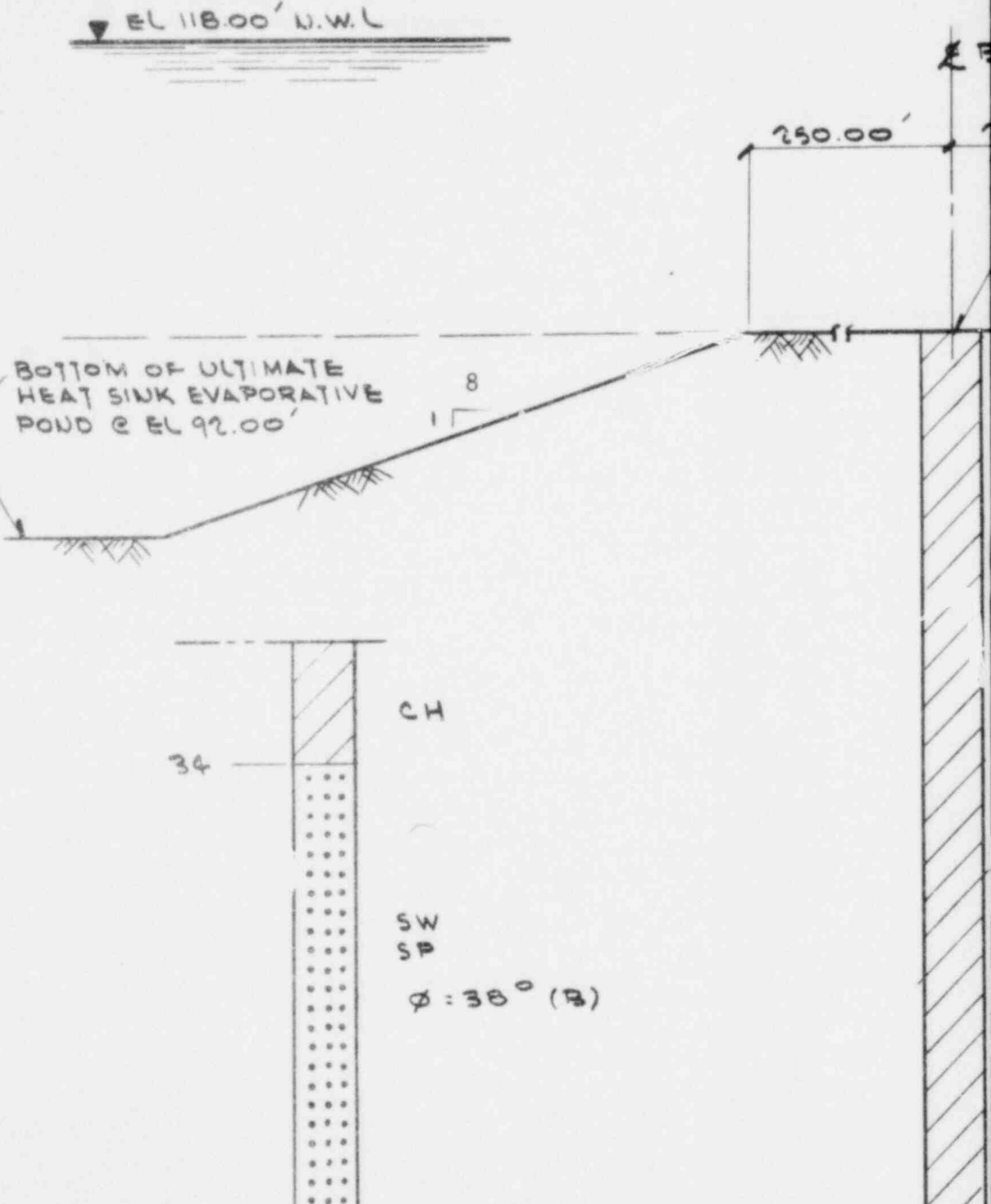
SCALE :- 1" = 5'-0"

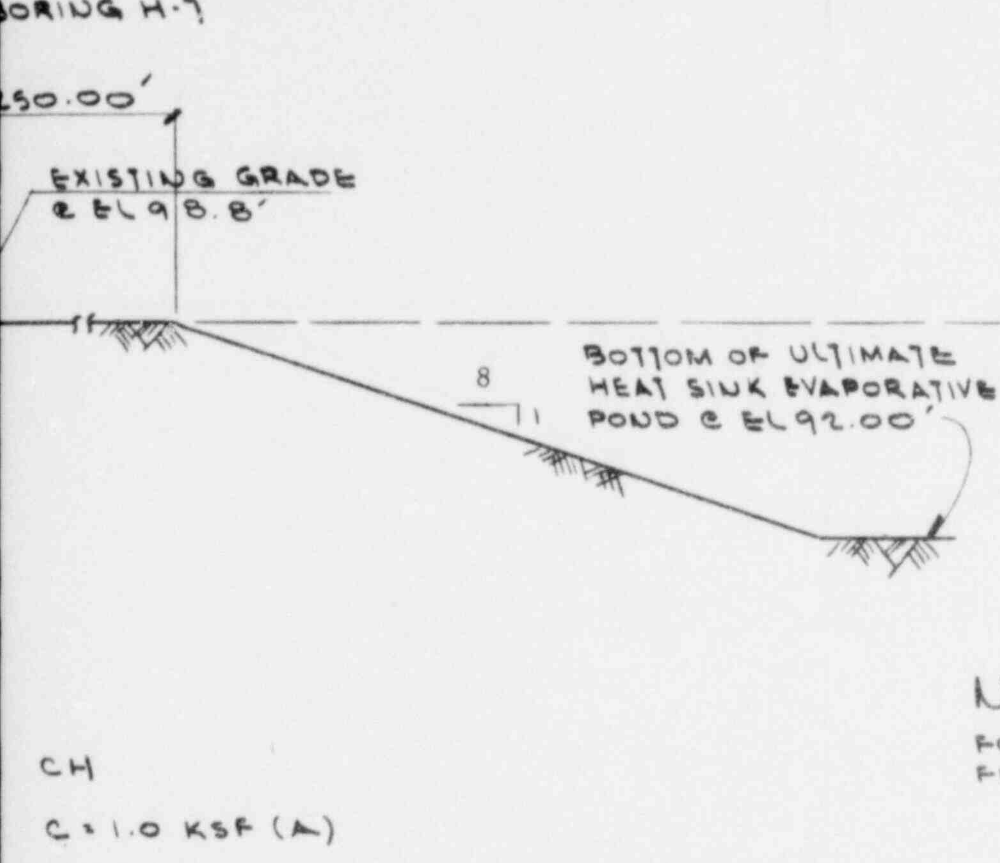
Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Units 1 & 2

SLOPE CROSS SECTIONS
(U.H.S. – H-7)

FIGURE 2.5.5-9


EL 118.00' N.W.L
250.00'
BOTTOM OF ULTIMATE HEAT SINK EVAPORATIVE POND @ EL 92.00'
8
1
CH
34
SW
SP
Ø : 38° (B)



NOTES

FOR GENERAL NOTE SEE FIGURE 2.5.5-8

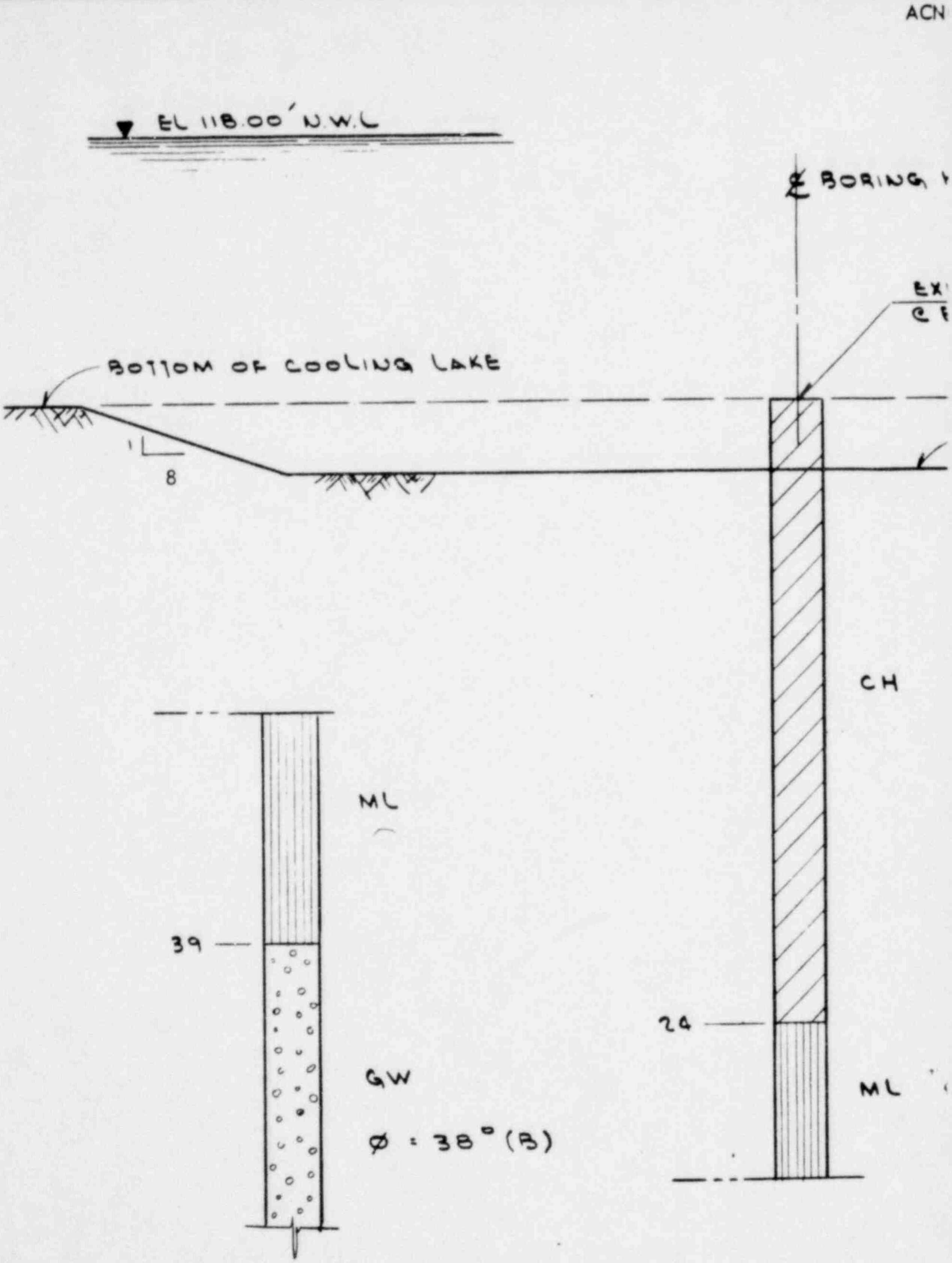
SCALE :- 1" = 5'-0"

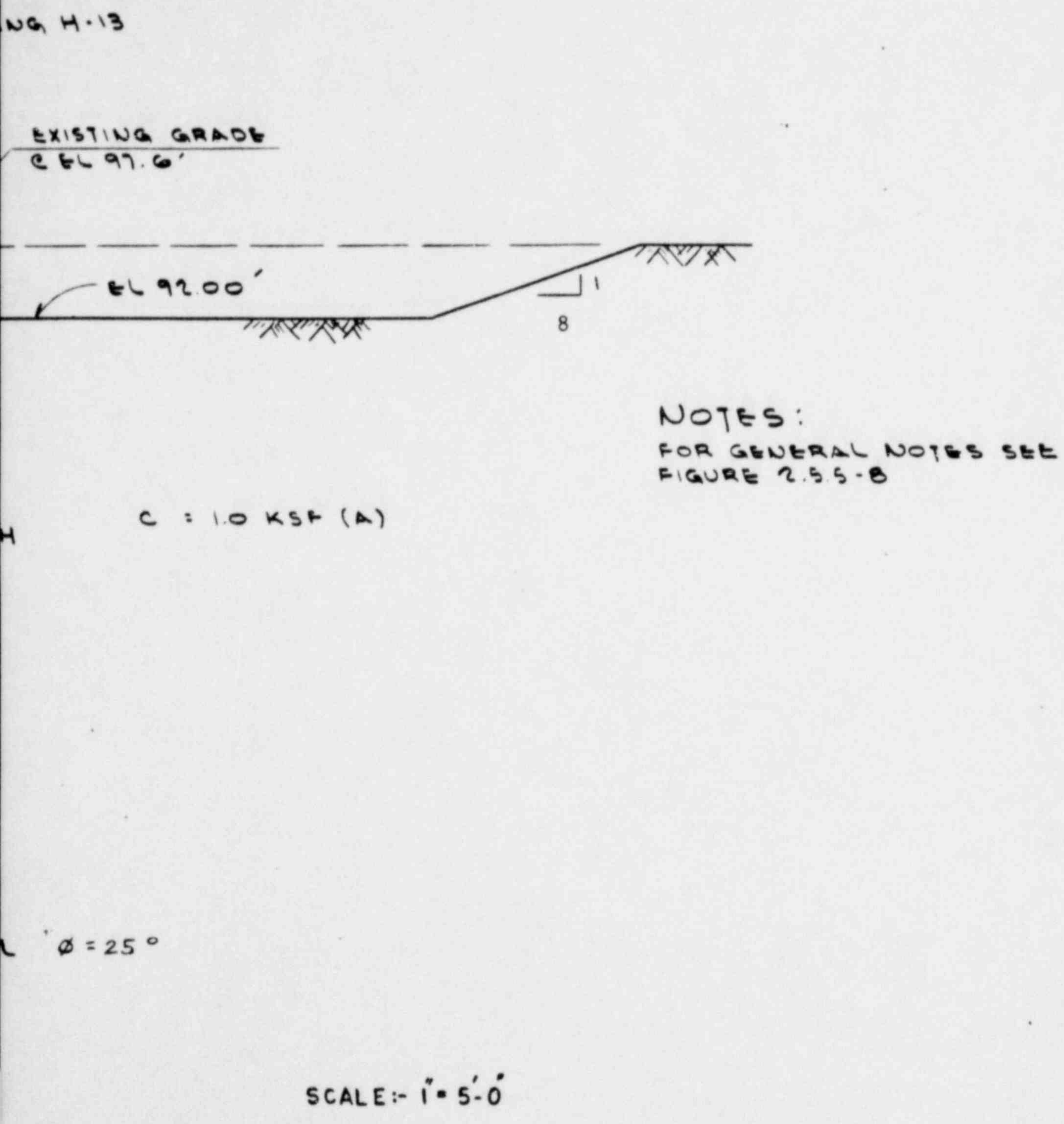
Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Units 1 & 2

SLOPE CROSS SECTIONS
(U.H.S. – H-7)

FIGURE 2.5.5-9


ACN
EL 118.00' N.W.L
℄ BORING
EX
C F
BOTTOM OF COOLING LAKE
1
8
CH
ML
39
GW
Ø = 38° (B)
24
ML



NOTES:
FOR GENERAL NOTES SEE
FIGURE 2.5.5-8

SCALE:- 1" = 5'-0"

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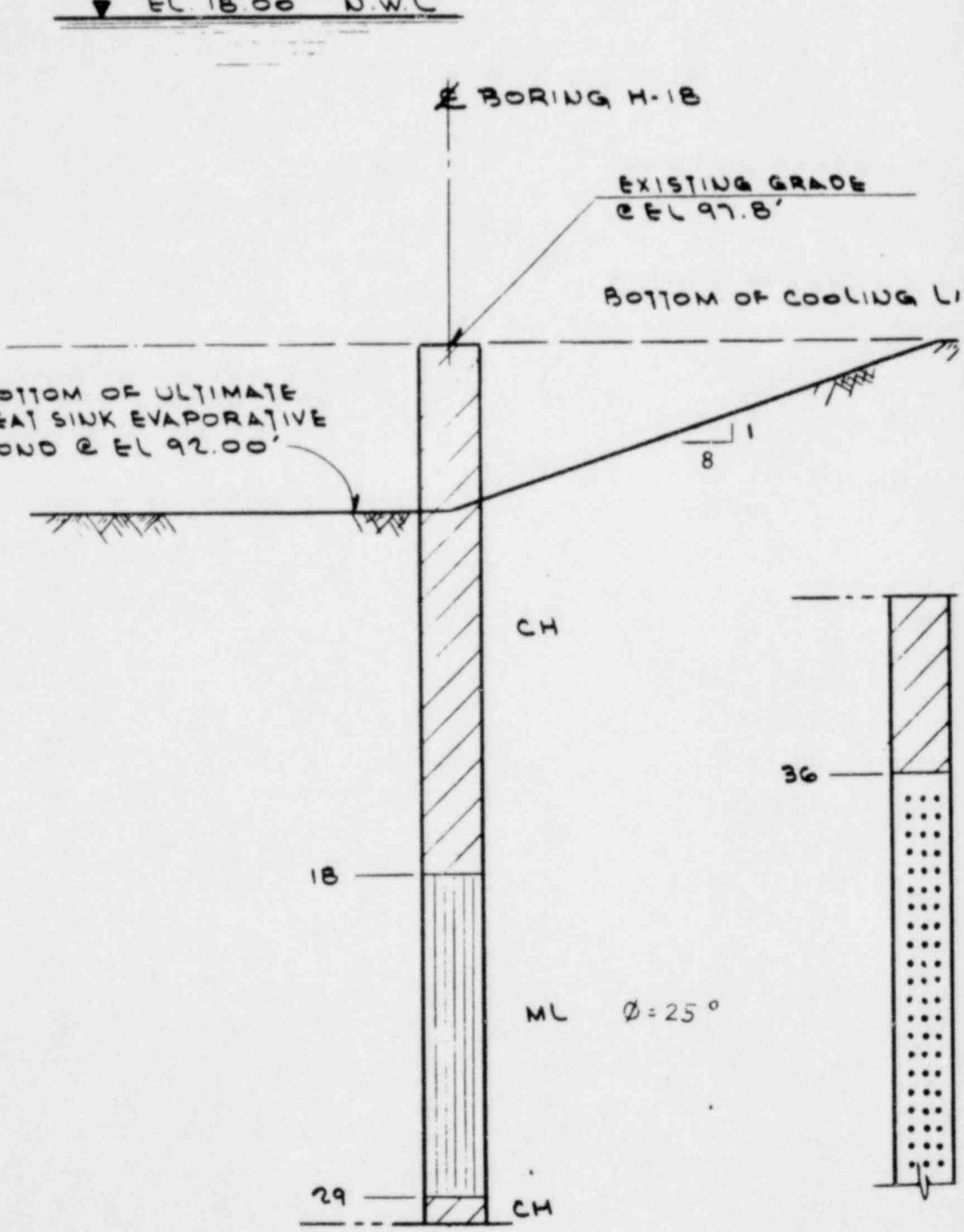
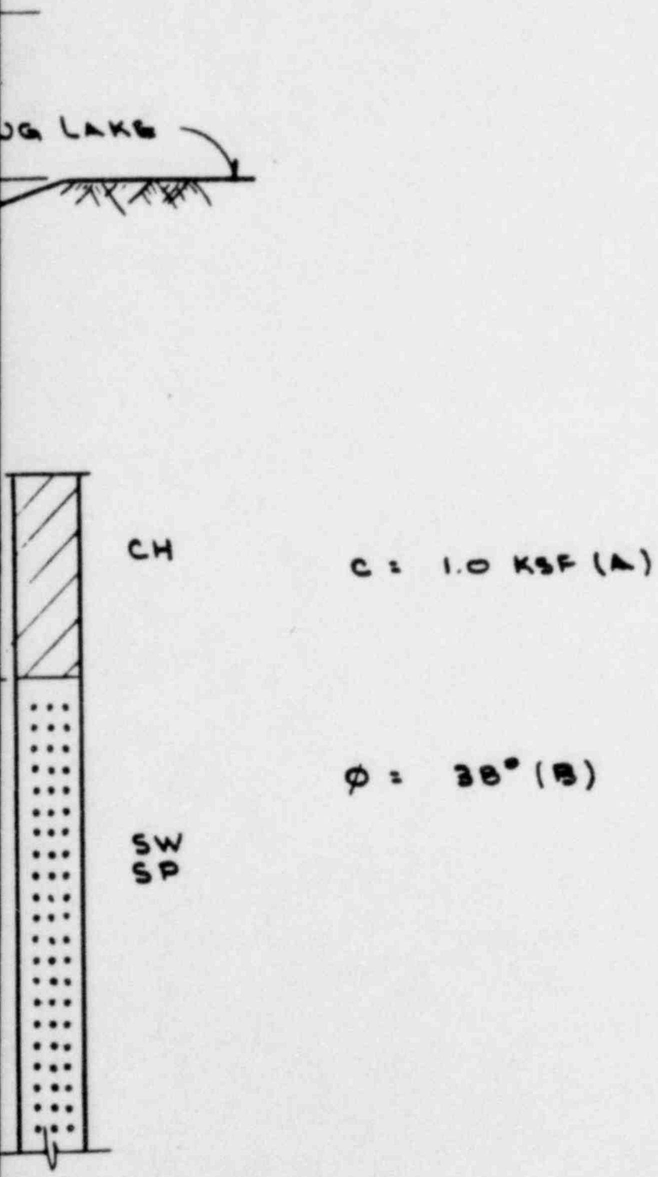
Allens Creek Nuclear Generating Station
Units 1 & 2

SLOPE CROSS SECTIONS
(U.H.S. - H-13)

FIGURE 2.5.5-10


EL 18.00' N.W.L
℄ BORING H-18
EXISTING GRADE @ EL 97.8'
BOTTOM OF COOLING LI
BOTTOM OF ULTIMATE HEAT SINK EVAPORATIVE POND @ EL 92.00'
1
8
CH
18
ML
Ø = 25°
29
CH
36



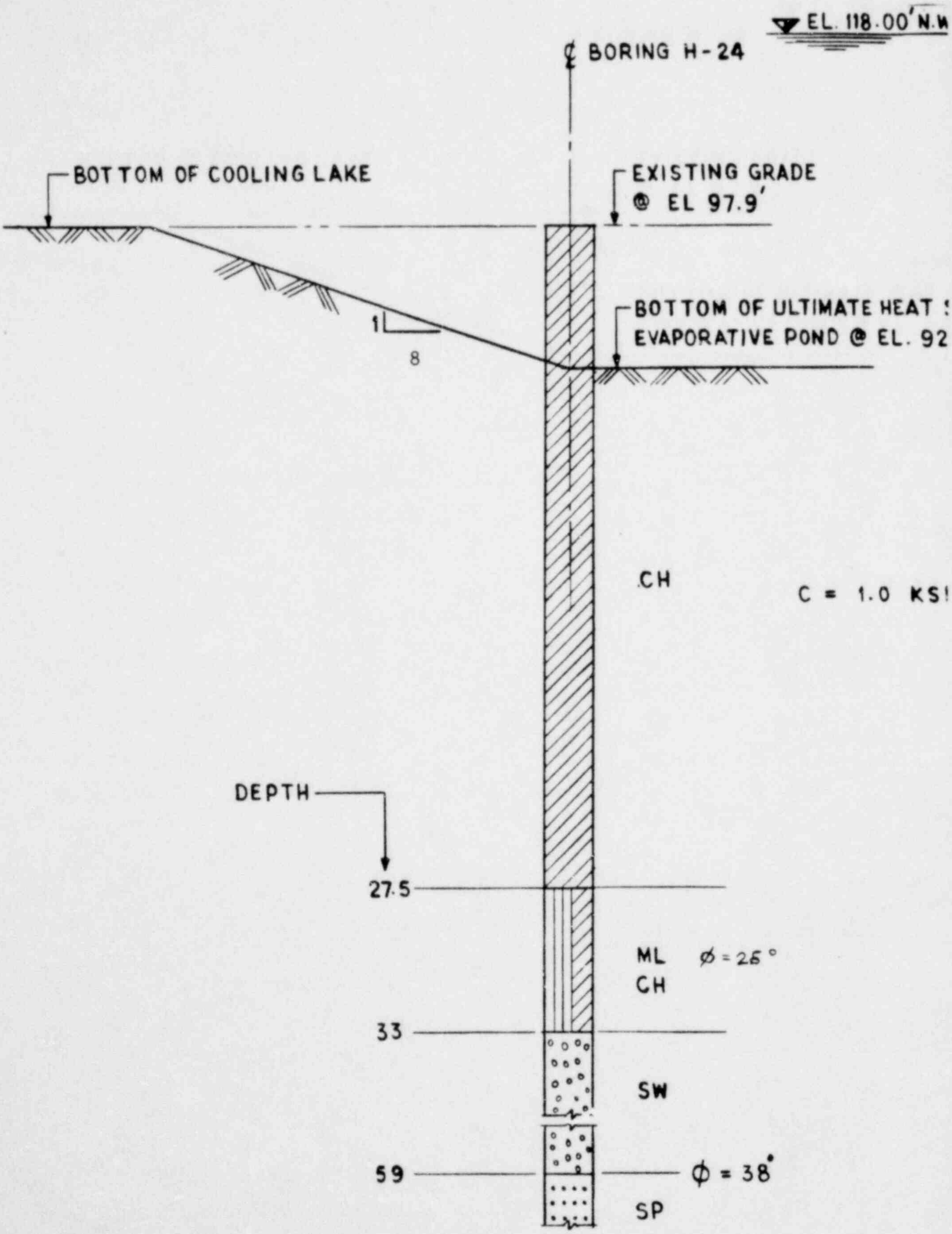
NOTES:
FOR GENERAL NOTES SEE
FIGURE 2.5.5-B

Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Units 1 & 2

SLOPE CROSS SECTIONS
(U.H.S. – H-18)

FIGURE 2.5.5-12


EL. 118.00' N.W
℄ BORING H-24
BOTTOM OF COOLING LAKE
EXISTING GRADE
@ EL 97.9'
1
8
BOTTOM OF ULTIMATE HEAT
EVAPORATIVE POND @ EL. 92
CH
C = 1.0 KS
DEPTH
27.5
ML ϕ = 25°
CH
33
SW
59
ϕ = 38°
SP

00' N.W.L

HEAT SINK
EL. 92.00'

D KSF (A)

NOTES:-

FOR GENERAL NOTES SEE FIGURE 2.5.5-8

SCALE :- 1" = 5'-0

Am. No. 50, 1/15/79

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Allens Creek Nuclear Generating Station
Units 1 & 2

SLOPE CROSS SECTIONS
(U.H.S. - H-24)

FIGURE 2.5.5-13

APPENDIX M TO
SECTION 2.5 ACNGS-PSAR
DESIGN OF ULTIMATE HEAT SINK SLOPES
USING SPECIAL RESIDUAL CLAY STRENGTHS

M1 INTRODUCTION

In order to obtain additional data and confirm the location and design of the UHS a subsurface investigation was performed in April and May, 1978. Additionally, special laboratory shear tests were performed in order to completely respond to the NRC question concerning slickensided clays.

M2 FIELD INVESTIGATION

Subsurface soil conditions at the site were investigated by 7 borings drilled to depths ranging from 8 to 150 ft. at locations illustrated on Fig. No. M1. A cross section through the borings is shown in Fig. No. M2. Detailed descriptions of the soils encountered are given on the boring logs presented on Figures M3 through Figures M9. A key to the symbols and terms appearing on the logs is included on Figure M10. 50 361.4

Borings were drilled with truck-mounted drilling equipment. In the ultimate heat sink area, samples were obtained continuously to 20 ft or completion depth, whichever was the lesser, 5-ft intervals to 100 ft and at 10-ft intervals below 100 ft. Samples of cohesive soils were generally obtained by alternating a 3-in. thin-walled tube and a 2-in. split-barrel. Most granular samples were obtained with a 2-in. split-barrel. Driving resistances for the split-barrel sampler are recorded in the "Blows Per Foot" column on the boring logs. Each of these samples was removed from the sampler in the field and examined and classified by a soil technician. Representative portions of each sample were sealed and packaged for transportation to the laboratory.

A Hvorslev-type stationary piston sampler, with a 3-in. thin-walled tube was used to obtain undisturbed granular samples from Borings H-42A, H-43A and H-44A. The tubes and soil were weighed immediately after sampling to determine the undrained density of the soil. The samples were retained in the tube by using porous caps (to allow drainage) and transported to the laboratory for further testing. Density results obtained from the piston samples are presented on Table M1.

The depth to water in most boreholes was measured at least 24 hours after completion. The depths to water and the dates of observations are recorded in the lower-right corner of the individual boring logs. In addition, four piezometers were installed to monitor groundwater level; two were installed in Boring H-44 to 10 and 25-ft depth and a similar installation was done in Boring H-48.

Test Pits

A test pit was excavated near each of the ultimate heat sink borings for the purpose of visually examining the surface clays and in-place density testing and bulk sampling of the near surface sands. In-place density tests were performed at several depths with a rubber balloon-densometer in accordance with ASTM Procedure D 2167-66. Results of these tests are presented on Table M2. Bulk samples were sealed for transportation to the laboratory.

M3 LABORATORY INVESTIGATION

The laboratory program was directed towards evaluation of strength, compressibility and classification properties of the foundation soils, primarily of the slickensided clays.

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Strength Tests

In order to estimate the undrained residual shear strength parameters of the foundation soils, several repeated direct shear tests were performed on two typical samples of the clay. These tests were conducted as consolidated-undrained multiple-specimen type tests at incremental normal stresses. The samples were strained forward and moved back manually in the shear box several times until the minimum shear stress (residual strength) was obtained for each load. Results are presented as Mohr's diagram. Stress-strain curves are presented for the respective tests. Figures No. M11 through M16 present the results.

Consolidated drained repeated direct shear tests were performed in accordance with Appendix IXA of EM 1110-2-190 Engineering and Design, Laboratory Soils Testing, Drained Repeated Direct Shear Test. This procedure includes presplitting samples and the repeated straining of them to simulate the drained strength along slickensided surfaces. Results are presented as Mohr's diagrams. Stress-strain curves are presented for the respective tests on Figure No. M17 through M19 present the results.

The shear strength properties of the near surface sands were estimated by performing consolidated-drained triaxial tests. These tests were conducted on undisturbed sand samples obtained from a Hvorslev piston-type sampler. The results of these tests are presented as a Mohr's diagram on Figure No. M20.

Density Tests

Modified Proctor (ASTM D 1557-70) and Maximum-Minimum Density (ASTM D 2049-69) tests were performed on each bulk sample of granular material. Maximum-Minimum density tests were performed by the dry method. Results of these tests are presented on Figure Nos. M21 through M25, and Table M3.

Consolidation and Classification Tests

The compressibility characteristics of the foundation materials were investigated by consolidation tests conducted on undisturbed cohesive samples. Results are presented on Figure Nos. M26 through M30.

Atterberg limit tests were performed for several samples to evaluate soil plasticity and aid in soil identification. Grain-size analyses were performed on all Hvorslev and bulk samples and on several other selected granular samples to aid in soil identification.

Laboratory Classification Test Results

The results of the soil classification tests performed for this study are plotted or tabulated on the boring logs presented on Fig. Nos. M3 through M9 or on the following figures and tables.

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Grain size analyses are presented on Figure Nos. M31 through M33. Table No. M4 presents additional classification tests on the samples tested in accordance with the WES procedure for drained repeated direct shear tests shown on Figure Nos. M17 through M19.

M4 GROUNDWATER LEVEL

Observations in open boreholes and all piezometers indicated that the groundwater level in the ultimate heat sink area was about EL +94 at the time of the investigation during the month of May 1978. Measurements in the piezometers on July 24, 1978 indicate that the groundwater level was also about EL +94. Groundwater levels can be expected to fluctuate with seasonal and climatic conditions.

M5 ADDITIONAL INVESTIGATIONS

Section 2.5.6.7 of the PSAR presents all of the field and laboratory test results performed in the Ultimate Heat Sink area. Borings H-37 through H-41 drilled in May 1977 provide additional data in the area of the causeway, intake and basin area of the UHS. Figure No. 2.5.4-5C indicates the location of all the borings in the UHS with the exception of the most recent borings.

M6 DESIGN PARAMETERS - SHEAR STRENGTH

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Several shear strength values are needed to completely define the strength of the clay in the UHS under drained and undrained conditions. The purpose of this section is to discuss the different types of strength and when each value is applicable.

Figure 2.5.6-26 of the PSAR indicates the undrained shear strength of the recent flood plain clays with depth. The undrained shear strength ranges from 0.5 ksf to 3.0 ksf with a lower bound average of 1.0 ksf for all depths.

Figure 2.5.6-27AA of the PSAR presents the Mohr circle results of triaxial unconsolidated undrained tests on near surface samples of clay in the area of the UHS. The undisturbed shear strength varies from 0.8 ksf to 1.9 ksf with a lower bound average of approximately 1.0 ksf.

Figure 2.5.6-27AA also presents the Mohr circle results of the remolded unconsolidated undrained shear strength. The remolded strength was obtained from samples kneaded, reshaped and retested. The values range from 0.4 ksf to 3.8 ksf with an average value of 1.0 ksf. Based on this result the undrained strength of the clay could be assigned a value of 1.0 ksf. This includes in some way the effect of slickensided surfaces since the samples were remolded.

Figures 2.5.6-27G, 27I and 27J of the PSAR present the Mohr circle results of triaxial consolidated undrained triaxial tests with pore pressure measurements on undisturbed and remolded samples from the area of the UHS. The total strength or undrained results from undisturbed samples shown on Figure 2.5.6-27G varies from 0.7 ksf to 1.2 ksf with an average of approximately 0.8 ksf. The remolded undrained strength shown on Figure 2.5.6-27G varies from 0.4 ksf to 2.4 ksf with an average of 1.0 ksf.

The effective strength or drained results from undisturbed sample shown on Figure 2.5.6-27I varies from $\phi = 28^\circ$ to $\phi = 21^\circ$ and $C = 0$ using the maximum deviator stress as the peak and varies from $\phi = 33^\circ$ to $\phi = 25^\circ$ and $C = 0.3$ ksf using the maximum effective stress ratio as the peak. From this data a conservative effective or drained strength would be $\phi = 21^\circ$ and $C = 0.3$ ksf. The samples presented on Figure 2.5.6-27I were recompacted to 90 pcf which is approximately 95% of the maximum density obtained using ASTM D 698 as the base standard. The drained strength as shown on Figure 2.5.6-27J varies from $\phi = 30^\circ$ to $\phi = 43^\circ$ using the maximum deviator stress as the peak and varies from $\phi = 30^\circ$ to $\phi = 47^\circ$ using the maximum effective stress ratio as the peak. Both of these drained strengths are considerably greater than those shown on the undisturbed samples in Figure 2.5.6-27I suggesting that the samples in the undisturbed state failed along some weak plane, which can be assumed to be along the slickensides. Therefore it would not be unreasonable to use the drained strength from Figure 2.5.6-27I as the drained residual shear strength of the UHS clays.

Figure Nos. M34 through M36 presents the results of unconsolidated undrained triaxial tests on samples recently obtained in Boring H44. The stress strain curves were carried out to 25% strain to develop the ordinary residual undrained shear strength of the clays. The undisturbed strength of the peak is approximately 1.5 ksf similar to that shown on Figure 2.5.6-26, 2.5.6-27A and 2.5.6-27G. The residual shear strength is 0.5 to 0.8 ksf shown on the lower portion of Figure M34. This compares favorable with the values from 2.5.6-27AA (remolded).

The lower bound average of shear strength for all the undrained undisturbed shear strength samples is therefore 1.0 ksf for undisturbed samples and C = 0.5 ksf for a remolded sample. The lower bound of shear strength for all the drained shear strength samples is therefore $\phi = 21^{o}$, C = 0.3 ksf for undisturbed samples and $\phi = 30^{o}$ for a remolded sample. 50 361.4

At the NRC's consultants (WES) request tests were performed on presplit and repeated direct shear samples. This dated is summarized on Figures M11 & M12 for undrained condition and Figure M17 for drained conditions. The lowest undrained residual strength is $\phi = 8.5^{o}$, C = 0.1 ksf shown on Figure M12. The lowest drained residual strength is $\phi = 9^{o}$. As is the normal case for this type of shear test there is practically no difference in strength in drained or undrained conditions suggesting that both tests measure drained parameters. Therefore the absolutely lowest drained shear strength is $\phi = 9^{o}$ using the most critical test procedures, of Appendix IX A, EM 110-2-1906 of the Corps of Engineers. This value is extremely conservative for use at Allens Creek since the clays at the site are slickensided as a result of drying and shrinkage, not large scale movements. There are no large scale slickensided surfaces in the Allens Creek clays, slickensides are approximately 1/4" in size, irregular, nonplanar and are randomly distributed within the clays. Only large scale movements could result in the gross reduction to residual shear strength values as obtained from the WES test procedure. At Allens Creek, as discussed in the following section, large scale slope movements will not occur.

Two articles presented in the ASCE publication, Research Conference on Shear Strength of Cohesive Soils, University of Colorado June, 1960 discuss the use of residual strength of saturated clay, Article 1. The Physical Components of the Shear Strength of Saturated Clays by M Juul Hvorslev indicates that the residual strength of some clays is attained only after very large deformations and that the decrease in shear strength after failure is primarily caused by a transient increase in pore water pressure and a thixotropic loss in strength, which is regained in time upon cessation of the deformations. This artical supports the statements previously noted and indicates that the strength can be regained. Article 2, The Relevance of the Triaxial Test to the Solution of the Stability Problems by Alan W. Bishop and Lauritus Bjerrum states that the presence of fissures is reflected in the factors of safety obtained using the effective stress analysis. Article 2 recommends that a factor of safety of at least 1.0 be ensured. Table M5 attached presents the recommended safety factors from the Corps of Engineers publication EM1110-2-1902, April 1, 1970. Discussions with WES indicated that they would like to see a safety factor of 1.25 for Class I slopes using the residual strength. It should be noted that the design shear strength of $\phi = 9^{\circ}$ and a safety factor of 1.25 for effective stress conditions is very conservative and unrealistic. As discussed, the conservative properties will be used in the appropriate places in the analyses, only because it is insignificant to the UHS slopes since they are such slight excavations and minor cuts.

M7 STABILITY ANALYSES

Two representative cross-sections covering the various soil strata were analyzed to determine the slope stability characteristics under different conditions. Figure M2 indicates the cross-sections, designated E-E, UHS Causeway and F-F, UHS Basin. Section D-D on Figure M2 indicates the different soil strata, standard penetration test results and field descriptions. 50 361.4

The range of soil parameters used in the analyses are indicated in tables for each cross-section. The parameters considered for the various cases are consistent with the recommendations of Table M5 and developed as the result of laboratory tests as noted in Section M3. Drained and undrained parameters are used for static conditions including the consolidated drained repeated direct shear test results from Figure No. M17. Undrained parameters are used for rapid drawdown and dynamic analyses. At Allens Creek rapid drawdown can only occur from El.118 to 100 as a result of loss of the Main Dam. Below El.100 the water is contained within the UHS basin and is recirculated. A drained state of soil properties would be characteristic of a long term static condition in which any buildup of pore pressures in the soil due to construction is considered to be dissipated. The laboratory tests yielding drained soil strength properties were therefore established to simulate this field condition of normal water level pore pressures. An undrained soil condition is one whereby the pore pressure in the soil has been built up as a result of a quick load application as characterized by the water level rapid drawdown or design seismic event.

Two methods of analysis, the simplified Bishop slip circle method and the U.S. Army Corps of Engineers sliding wedge method were used to investigate the stability of all the slopes.

In performing the slip circle method of stability analysis the Ebasco computer program was used. The method employed by the program, the simplified Bishop approach, is one in which a circular failure surface is assumed to form about its center of rotation. The circle through the slope is then divided into vertical slices and the tangential resisting and driving forces along the circular surface are computed for each slice. The factor of safety against sliding is computed as the ratio of the sum of the resisting moments taken about the center of rotation to the sum of the driving moments about the same center of rotation.

To use the program the slope geometry must be fully defined on a coordinate grid system along with changes in soil layers. The soil encountered on the slope being analyzed must be fully defined with respect to its saturated unit weight and shear strength. Water levels along the slope must also be defined, whether it be in the form of freestanding water, groundwater, or pore pressure built up within the soil. Finally, if applicable, the horizontal (0.1g) and vertical (0.067g) components of the design basis earthquake are input.

To find the worst possible radius and center of rotation yielding the circle with the lowest factor of safety, a search routine is built into the program by which a trial center of rotation is selected. The program will investigate different radii from that center of rotation computing and recording the safety factor for each radius. It then moves the center of rotation at a prescribed increment to a different trial location and the above process is repeated until the lowest safety factor is reached.

The simplified Bishop solution yields results that are conservative in that shear resistance between slices, which would tend to raise the factor of safety against sliding, is neglected. When the simplified Bishop solution is used to compute a factor of safety under dynamic loading additional conservatism is built into the program in that the computed safety factor is calculated assuming the components of the design earthquake acceleration act only in one direction, neglecting any back and forth motion, and the magnitude of the acceleration of the design earthquake is taken to be a constant over the entire slope for an infinite length of time. 50 361.4

In performing the sliding wedge method the Ebasco computer program was also used. The sliding wedge method consists of an active wedge being mobilized against a neutral horizontal block and a passive resisting wedge. The factor of safety is calculated as the ratio of the sum of the resisting forces in the horizontal direction to the sum of the driving forces in the horizontal direction. In applying the sliding wedge method to the two cross-sections the input data and search routine is similar to that of the slip circle analysis previously discussed. This method also includes a seismic loading in the analyses. This was done by including the product of the weights of the wedges and the neutral block with the horizontal acceleration factor of 0.1g. This force was then considered to act in the direction of the postulated slide as a driving force. The vertical component of the seismic loading is also incorporated into the solution tending to reduce frictional resistance between the sliding wedges. This vertical seismic force is computed as the product of the weights of the neutral block and the wedges with the vertical acceleration factor of 0.067g.

The results of each of these analyses are presented on the tables on Figure M2. In all cases the actual safety factor exceeds the recommended minimum safety factor from Table M5, indicating that the slopes are safe.

M8 SUMMARY AND CONCLUSIONS

The above described detailed investigation has accurately established the soil conditions in the area of the ultimate heat sink at Allens Creek. The continuous sampling in the upper soils and careful undisturbed sampling of clays and sand establishes a sound basis for the selection of lower bound strength samples. Selection of design strength parameters incorporated the use of lower bound strength parameters from the test results, using very conservative test procedures. Results of the analyses indicated satisfactory safety factors. Reflected in the analyses are the changes required to obtain the required safety factors. In order to maintain the 1 vertical to 3 horizontal slope of the causeway it was necessary to excavate the surface clays from beneath the causeway. Additionally the slopes of the ultimate heat sink basin have been flattened to 1 vertical to 8 horizontal from the original 1 vertical to 3 horizontal. These changes are the result of using the $\phi = 9^\circ$ from the consolidated drained repeated direct shear tests. 50 361.4

TABLE M1

UNIT WEIGHTS OF SAND SAMPLED WITH HVORSLEV PISTON SAMPLER

Boring No	Penetration Feet	Wet Unit Weight, pcf Undrained	Wet Unit Weight, pcf Drained	Dry Unit Weight, pcf
H-42A	7-9	101	101	96
	10-12	110	none	none
	13-15	116	109	98
H-43A	4-6	114	113	97
H-44A	4-6	111	109	86
	6-8	117	116	96

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Note: Drained wet unit weights were determined by allowing the tubes to drain through porous caps for 48 hours, inverting the tube at the end of 24 hours.

TABLE M2

SUMMARY OF IN-PLACE DENSITIES
Balloon Densometer
Allens Creek

Test Pit No.	Penetration Feet	Material	Wet Density pcf	Moisutre Content, %	Dry Density pcf
6	5.5	Fine sand	95.5	5.1	90.9
6	5.5	Fine sand	104.9	4.8	100.1
6	10.5	Fine sand with clay pockets	85.0	11.3	76.4
6	10.6	Fine sand	97.7	11.8	87.4
7	4.5	Fine sand	119.8	15.3	103.9
7	4.6	Fine sand	118.5	18.5	100.0
8	4.0	Fine sand	121.0	21.7	99.4
8	4.0	Fine sand	120.9	21.7	99.3
9	4.0	Fine sand	122.8	22.1	100.6
9	3.9	Fine sand	120.8	25.2	96.5

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TABLE M3

RELATIVE DENSITY TEST RESULTS

Test Pit No.	Penetration Feet	Material	Densities, pcf Minimum	Maximum
6	5	Fine sand	86	107
	10	Fine sand	89	107
7	5	Fine sand	85	104
8	5	Fine sand	83	100
9	5	Silty fine sand	79	96

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Note: Relative densities determined by dry method

TABLE M4

CLASSIFICATION AND DENSITY TEST RESULTS

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Boring	Sample Depth (ft)	W (%)	LL	PL	G	γ (pcf)
H44	14	-	82	29	2.73	
H44	16	35	-	-	-	87
H44	17.5	36	85	27	-	-
H44	20	39	-	-	-	81

TABLE M5

Minimum Factors of Safety (Reproduced from EM 1110-2-1902 April 1, 1970)

Case No.	Design Condition	Minimum Factor of Safety	Shear Strength	Remarks
I	End of construction	1.3††	Q or S‡	Upstream and downstream slopes
II	Sudden drawdown from maximum pool	1.0‡‡	R, S	Upstream slope only. Use composite envelope. See Fig. 4
III	Sudden drawdown from spillway crest or top of gates	1.2‡‡	R, S	Upstream slope only. Use composite envelope. See Fig. 4
IV	Partial pool with steady seepage	1.5	$\frac{R+S}{2}$ for $R < S$ S for $R > S$	Upstream slope only. Use intermediate envelope. See Fig. 5
V	Steady seepage with maximum storage pool	1.5	$\frac{R+S}{2}$ for $R < S$,	Downstream slope only. Use intermediate envelope. See Fig. 5
VI	Steady seepate with surcharge pool	1.4	S for $R > S$	
VII	Earthquake (Cases I, IV and with seismic loading)	1.0 1.15*	§	Upstream and downstream slopes

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† Not applicable to embankments on clay shale foundations.

†† For embankments over 50 ft high on relatively weak foundations use minimum factor of safety of 1.4.

‡ In zones where no excess pore water pressures are anticipated, use S strength.

‡‡ The safety factor should not be less than 1.5 when drawdown rate and pore water pressures developed from flow nets (Appendix III) are used in stability analyses.

§ Use shear strength for case analyzed without earthquake except that it is not necessery to analyze sudden drawdown for earthquake effects

* The minimum safety factor of 1.15 is suggested in NAVFAC DM.7, since this is more conservative it will be utilized in design.

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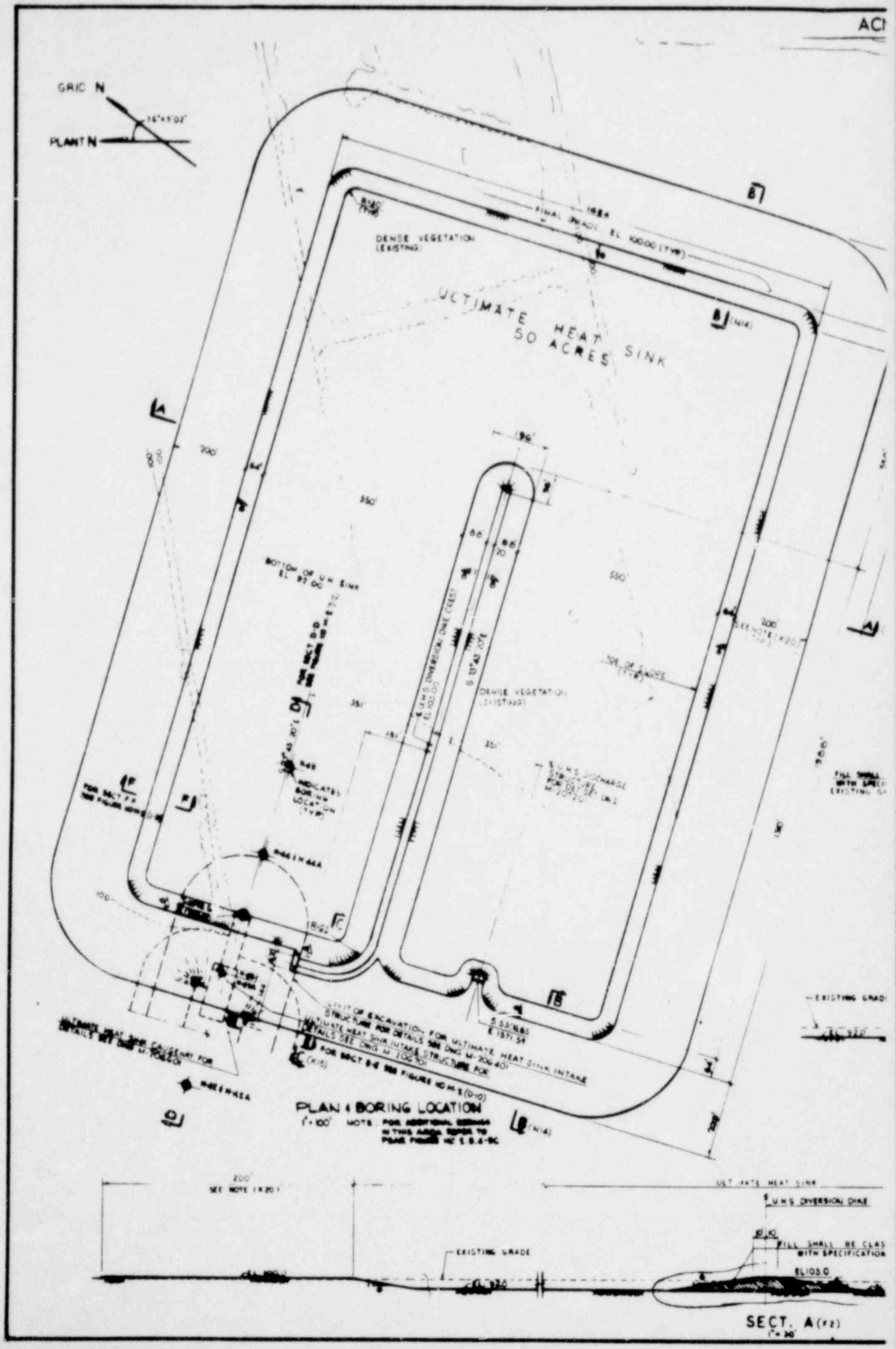
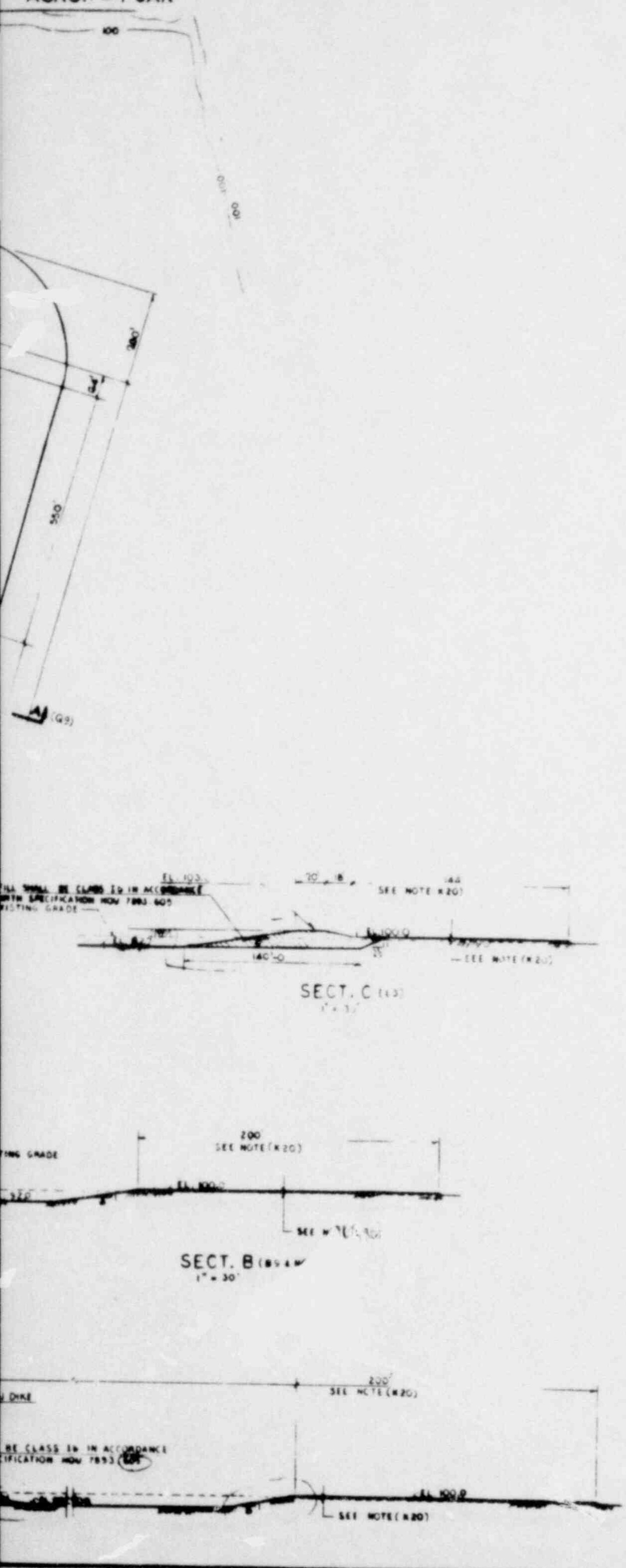

ACI
GRID N
16°45'02"
PLANT N
DENSE VEGETATION (EXISTING)
FINAL GRADE EL 100.00 (TYP)
1884
ULTIMATE HEAT SINK
50 ACRES
B (N14)
200'
550'
196'
66'
BOTTOM OF U.H. SINK EL 92.00
U.H.S. DIVERSION DIKE CREST EL 102.00
DENSE VEGETATION (EXISTING)
TOE OF SLOPE (TYP)
SEE NOTE (K20)
351'
181'
U.H.S. DISCHARGE STRUCTURE FOR DETAILS SEE DWG M-207201
INDICATES BORING LOCATION (TYP)
FOR SECT F-F SEE FIGURE 10 M-3
LIMIT OF EXCAVATION FOR ULTIMATE HEAT SINK INTAKE STRUCTURE FOR DETAILS SEE DWG M-206401
ULTIMATE HEAT SINK INTAKE STRUCTURE FOR DETAILS SEE DWG M-206701
ULTIMATE HEAT SINK CAUSEWAY FOR DETAILS SEE DWG M-706401
C (K15)
FOR SECT E-E SEE FIGURE 10 M-3 (D10)
S 53786.85
E 1571.59
B (N14)
FILL SHALL ... WITH SPECIF... EXISTING G...
EXISTING GRADE
PLAN & BORING LOCATION
1" = 100'
NOTE: FOR ADDITIONAL BORINGS IN THIS AREA REFER TO PSAR FIGURE NO. 2.5.4-9C
200'
SEE NOTE (K20)
ULTIMATE HEAT SINK
U.H.S. DIVERSION DIKE
FILL SHALL BE CLAS... WITH SPECIFICATION
EXISTING GRADE
EL 100.0
EL 92.0
EL 103.0
SECT. A (F2)
1" = 30'



QUANTITIES (NET BY FIELD UNLESS NOTED)

EXCAVATION	644,500 CU YDS
CLASS Ib FILL	131,300 CU YDS

LEGEND

[illegible]	EXISTING CONTOUR LINE
[illegible]	FINISHED CONTOUR LINE
[illegible]	TREES & VEGETATION

NOTES:

THIS DRAWING IS FOR THE CONSTRUCTION OF THE ULTIMATE HEAT SINK ONLY. THE ULTIMATE HEAT SINK DISCHARGE STRUCTURE, CAUSEWAY, INTAKE STRUCTURE AND THE ASSOCIATED EXCAVATION AND BACKFILL ARE SHOWN HERE FOR INFORMATION ONLY.

ALL COORDINATES ARE REFERRED TO THE NORTH-SOUTH & EAST-WEST BASELINES OF THE PLANT.

ALL ELEVATIONS ARE REFERRED TO MEAN SEA LEVEL DATUM, 1943 ADJUSTMENT.

CONSTRUCTION OF THE ULTIMATE HEAT SINK SHALL BE IN ACCORDANCE WITH SPECIFICATION HOU 7893.615

EXCAVATION OF THE ULTIMATE HEAT SINK SHALL BE IN ACCORDANCE WITH SPECIFICATION HOU 7893.604

DESIGNATED FILL REQUIRED SHALL BE IN ACCORDANCE WITH SPECIFICATION HOU 7893.605

ANY TEMPORARY BERMS (SUCH AS THOSE USED FOR EROSION CONTROL DURING CONSTRUCTION) WHICH ARE LOCATED WITHIN 200 FT OF THE ULTIMATE HEAT SINK SHALL BE COMPLETELY REMOVED PRIOR TO LAKE FILLING OPERATION.

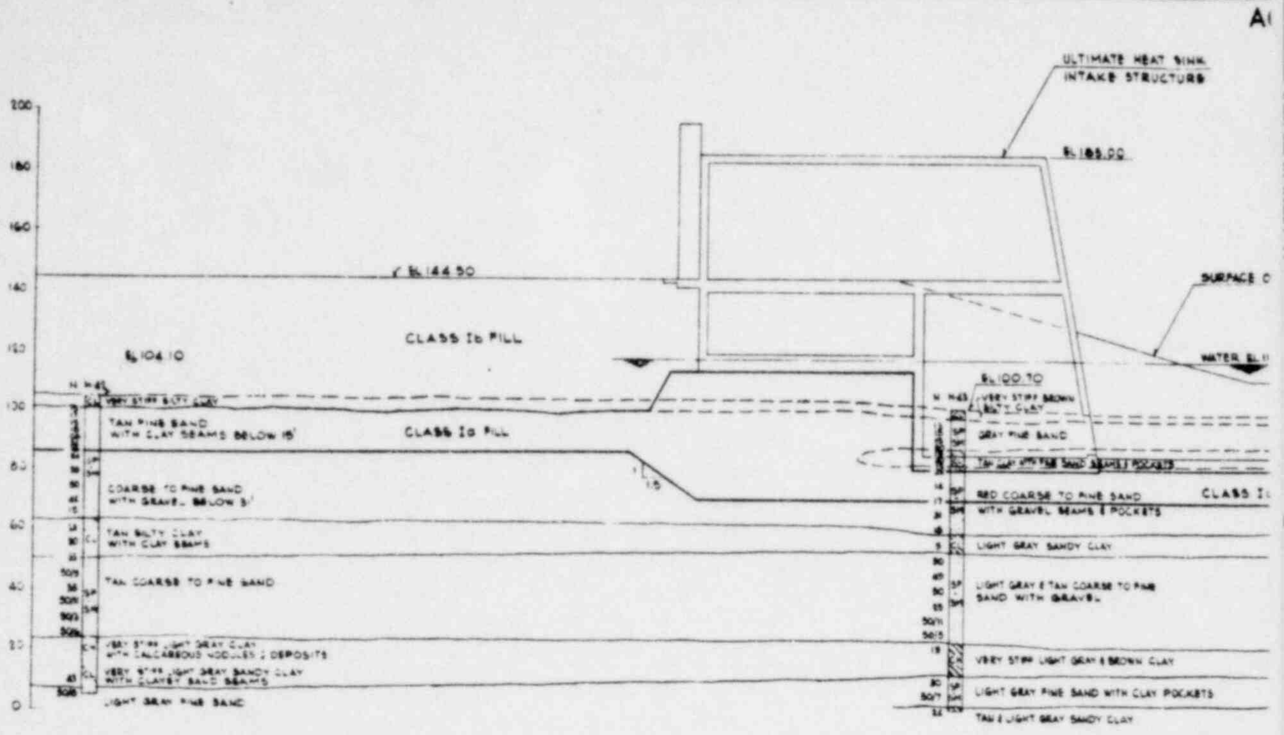
ALL AREAS WITHIN 200 FT ABOUT THE ULTIMATE HEAT SINK SHALL BE CLEARED, GRUBBED, PROOF-ROLLED AND LEVELED TO ELEV. 100 FT. IN ACCORDANCE WITH SPECIFICATION HOU 7893.604. FILL REQUIRED IN THIS AREA SHALL BE CLASS Ib IN ACCORDANCE WITH SPECIFICATION HOU 7893.615.

AM. NO. 50, 1.15/79

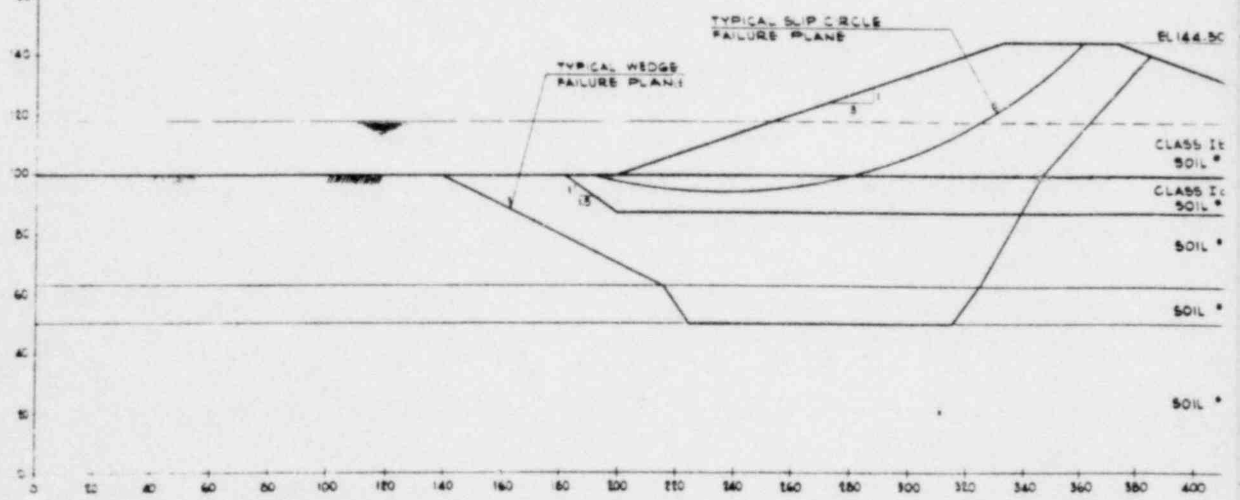
HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Unit 1

ULTIMATE HEAT SINK
PLAN AND SECTIONS

FIGURE NO. M-1



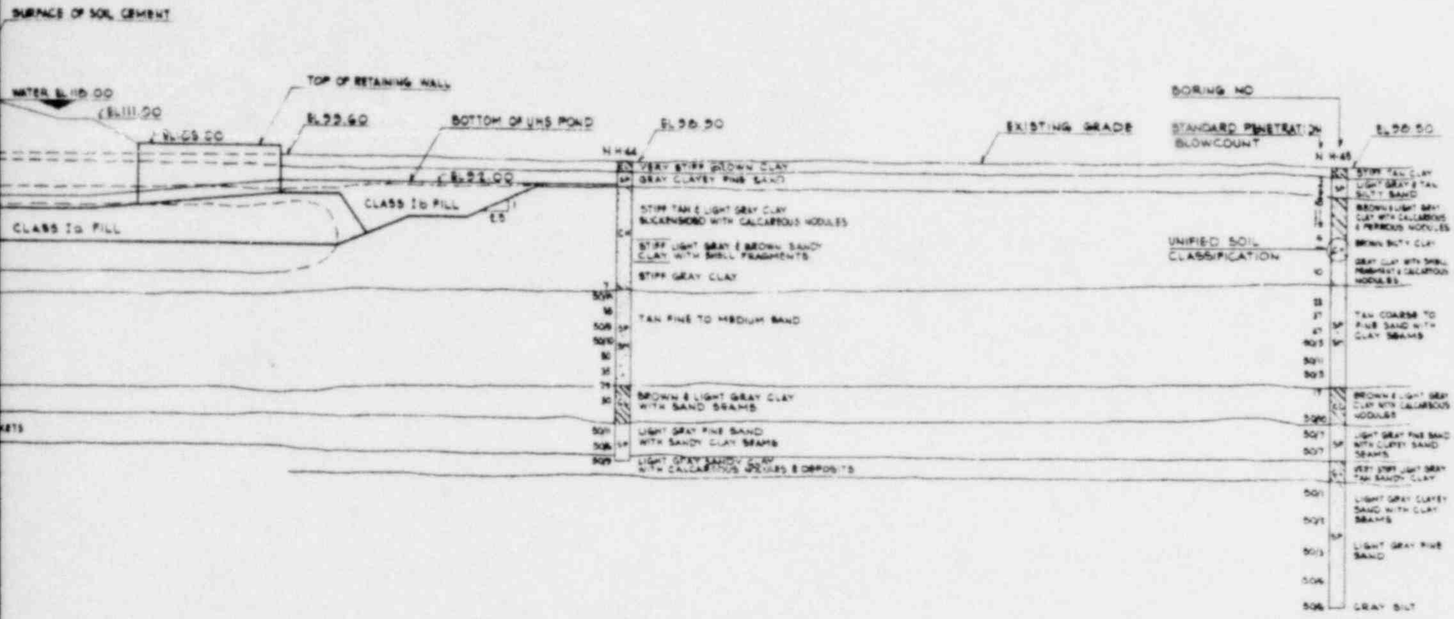
CASE NO	STRUCTURE	ANALYSIS TYPE	FAILURE PLANE	GROUND WATER LEVEL	SOIL PROPERTIES	SUGGESTED MINIMUM SAFETY FACTOR	ACTUAL MINIMUM SAFETY FACTOR
1	UHS CAUSEWAY	STATIC	CIRCLE	NORMAL (EL 116.00)	*1 C=1000 PSF, *2 φ=38°, *3 C=1000 PSF, *4 φ=38°	1.5	1.96
2	" "	"	WEDGE	" "	" " " "	"	1.78
3	" "	"	CIRCLE	" "	*1 φ=21°, C=300 PSF, *2 φ=38°, *3 φ=21° C=300 PSF, *4 φ=38°	1.3	1.92
4	" "	"	WEDGE	" "	" " " "	"	2.24
5	" "	"	CIRCLE	" "	*1 φ=21°, C=300 PSF, *2 φ=38°, *3 φ=9, *4 φ=36°	1.25	1.92
6	" "	"	WEDGE	" "	" " " "	"	1.62
7	" "	RAPID DRAWDOWN	CIRCLE	NORMAL TO EL 111.00	*1 C=1000 PSF, *2 φ=38°, *3 C=1000 PSF, *4 φ=38°	1.2	2.10
8	" "	" "	WEDGE	" "	" " " "	"	1.56
9	" "	EARTHQUAKE	CIRCLE	NORMAL	*1 C=1000 PSF, *2 φ=38°, *3 C=1000 PSF, *4 φ=36°	1.15	1.35
10	" "	"	WEDGE	"	" " " "	"	1.15



U.H.S. CAUSEWAY

SECT E-E SEE FIGURE NC

1:20.0



SECT D-D SEE FIGURE NO M-1 (I-3)
1"=20.0

CASE NO	STRUCTURE	ANALYSIS TYPE	FAILURE PLANE	GROUND WATER LEVEL	SOIL PROPERTIES	SUGGESTED MINIMUM SAFETY FACTOR	ACTUAL MINIMUM SAFETY FACTOR
11	UHS BASIN SLOPE	STATIC	CIRCLE	NORMAL (EL116.00)	*1 C=1000 PSF, *2 φ=38°	1.5	13.29
12	″ ″ ″	″	WEDGE	″ ″	″ ″	″	12.37
13	″ ″ ″	″	CIRCLE	″ ″	*1 φ=21°, C=300PSF, *2 φ=38°	1.5	3.35
14	″ ″ ″	″	WEDGE	″ ″	″ ″	″	9.26
15	″ ″ ″	″	CIRCLE	″ ″	*1 φ=9°, *2 φ=38°	1.25	1.25
16	″ ″ ″	″	WEDGE	″ ″	″ ″	″	2.29
17	″ ″ ″	RAPID DRAWDOWN	CIRCLE	NORMAL TO EL100.00	*1 C=1000 PSF, *2 φ=38°	1.2	13.28
18	″ ″ ″	″	WEDGE	″ ″	″ ″	″	12.37
19	″ ″ ″	EARTHQUAKE	CIRCLE	NORMAL	*1 C=1000 PSF, *2 φ=38°	1.15	3.32
20	″ ″ ″	″	WEDGE	″	″ ″	″	4.25

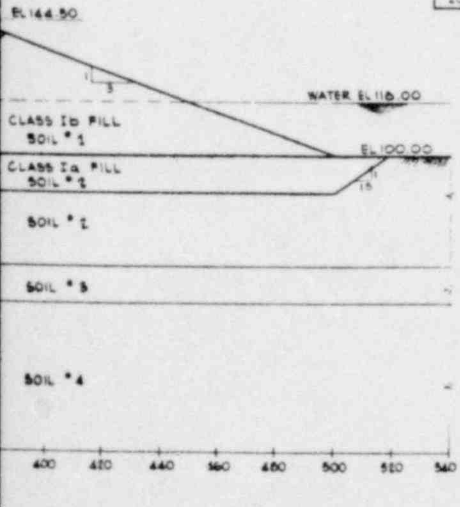
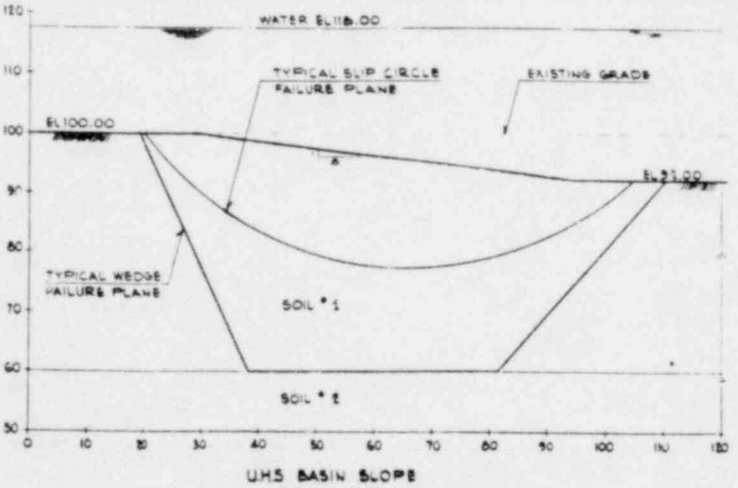


FIGURE NO M-1 (N-4)



UHS BASIN SLOPE
SECT F-F SEE FIGURE NO M-1 (J-1)
1"=10.0

AM. NO. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Unit 1

U.H.S. CROSS-SECTION AND
STABILITY ANALYSES

FIGURE NO. M-2

LOG OF BORING NO. H-42
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS

TYPE: 3" thin-walled tube & 2" split-barrel LOCATION:

DEPTH, FT	DESCRIPTION OF MATERIAL SURF. EL: 104.1'	BLOWS PER FT	UNIT DRY WT LB/CU FT	-#200,%
	Very stiff dark gray & brown silty clay -with roots to 1' -with very stiff gray sandy clay layer below 3.5'			
5		15		
	Tan fine sand	17		
10		15		
		15		12
		28		
15	-with clay seams below 15'	22		5
	-with clay layers below 17'	18		
20		24		
25		50		
30		50		
	-coarse to fine with gravel below 31'			
35		42		13
40		15		8
	Tan silty clay with clay seams -with calcareous nodules to 48'			
45		23		
	-with sandy clay seams & coarse sand pockets below 49.5'			
50		30		

(Continued on Plate 2b)

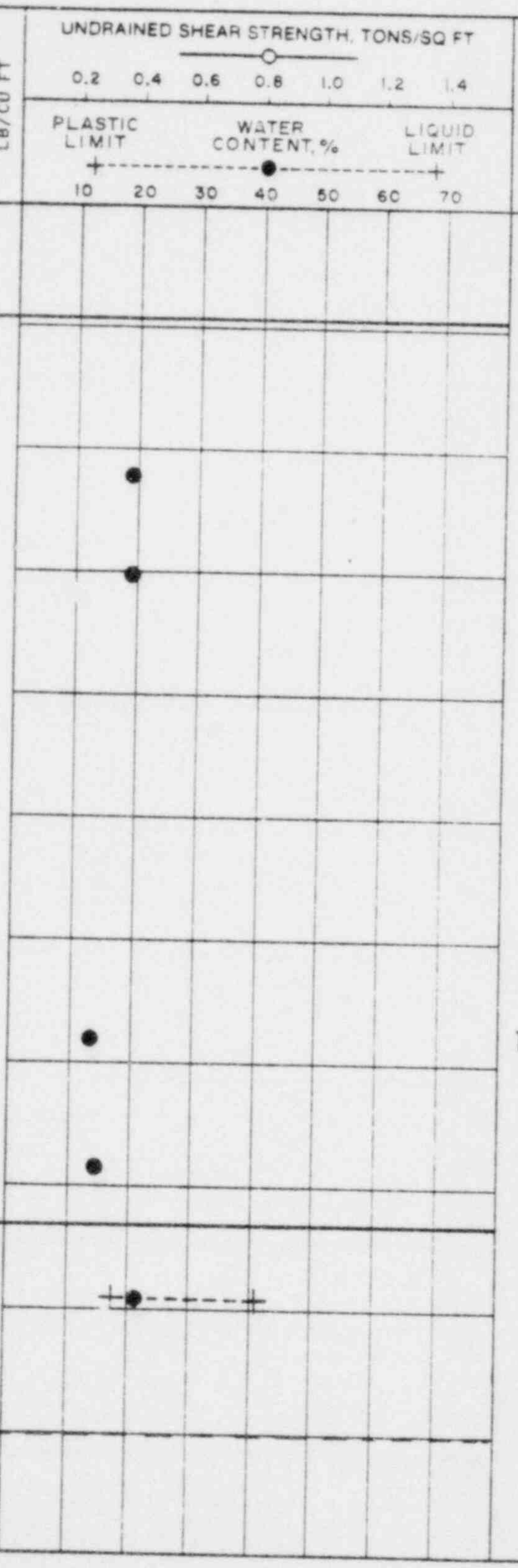
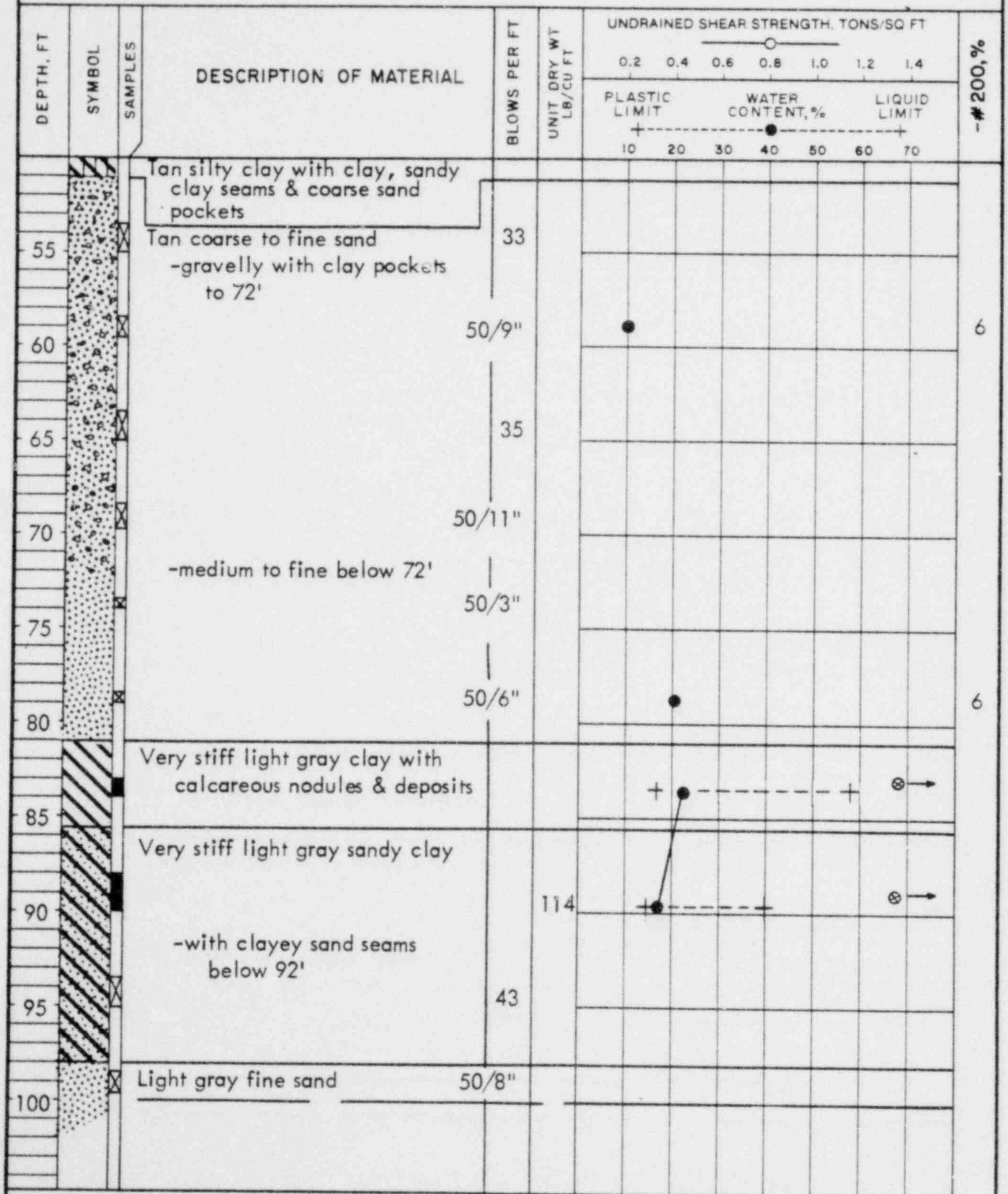


FIG NO. M3a

LOG OF BORING NO. H-42 (Cont'd)
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS



COMPLETION DEPTH: 99.5'
DATE: April 29, 1978

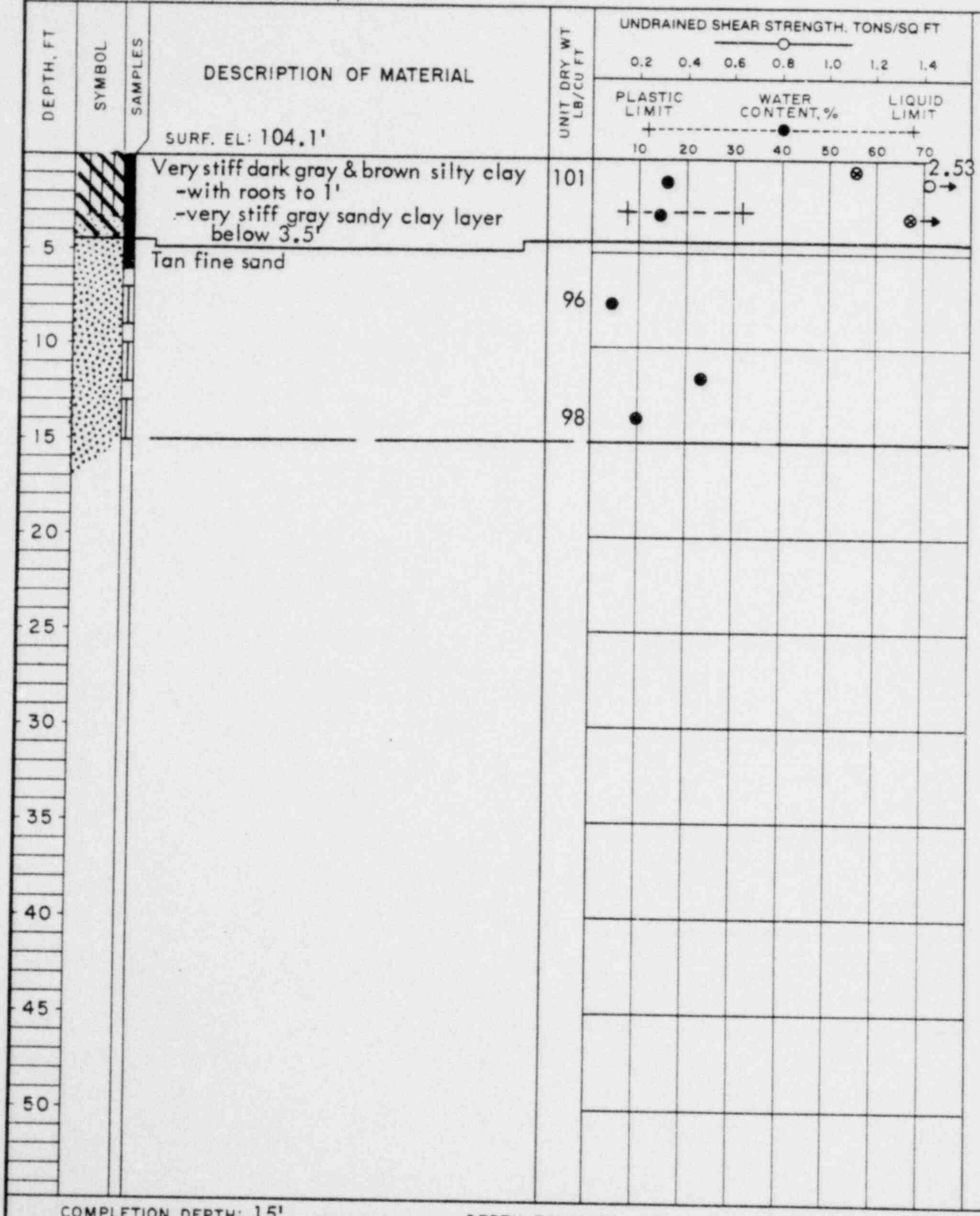
DEPTH TO WATER IN BORING: 10.5' Caved at 10.7'

DATE: May 1, 1978

LOG OF BORING NO. H-42A
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS

TYPE: 3" thin-walled tube & Hvorslev Piston Sampler

LOCATION:



SURF. EL: 104.1'

Very stiff dark gray & brown silty clay
-with roots to 1'
-very stiff gray sandy clay layer below 3.5'

Tan fine sand

COMPLETION DEPTH: 15'
DATE: April 26, 1978

DEPTH TO WATER IN BORING: grouted

DATE: April 26, 1978

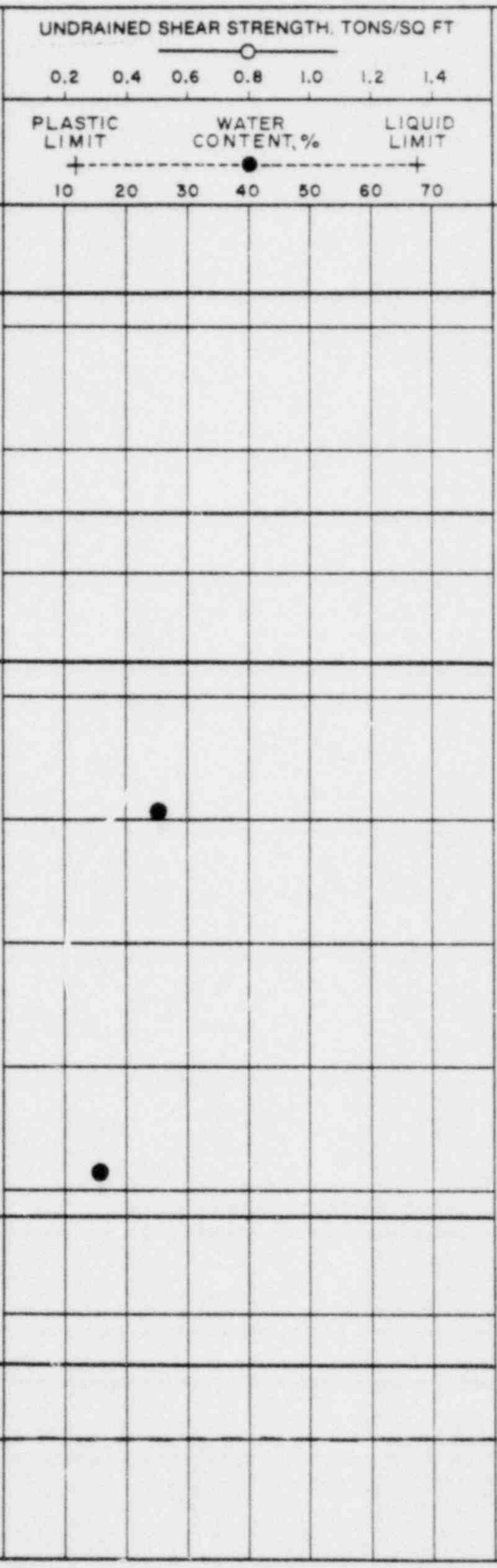
LOG OF BORING NO. H-43

ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS

TYPE: 3" thin-walled tube & 2" split-barrel LOCATION:

SURF. EL: 100.7'

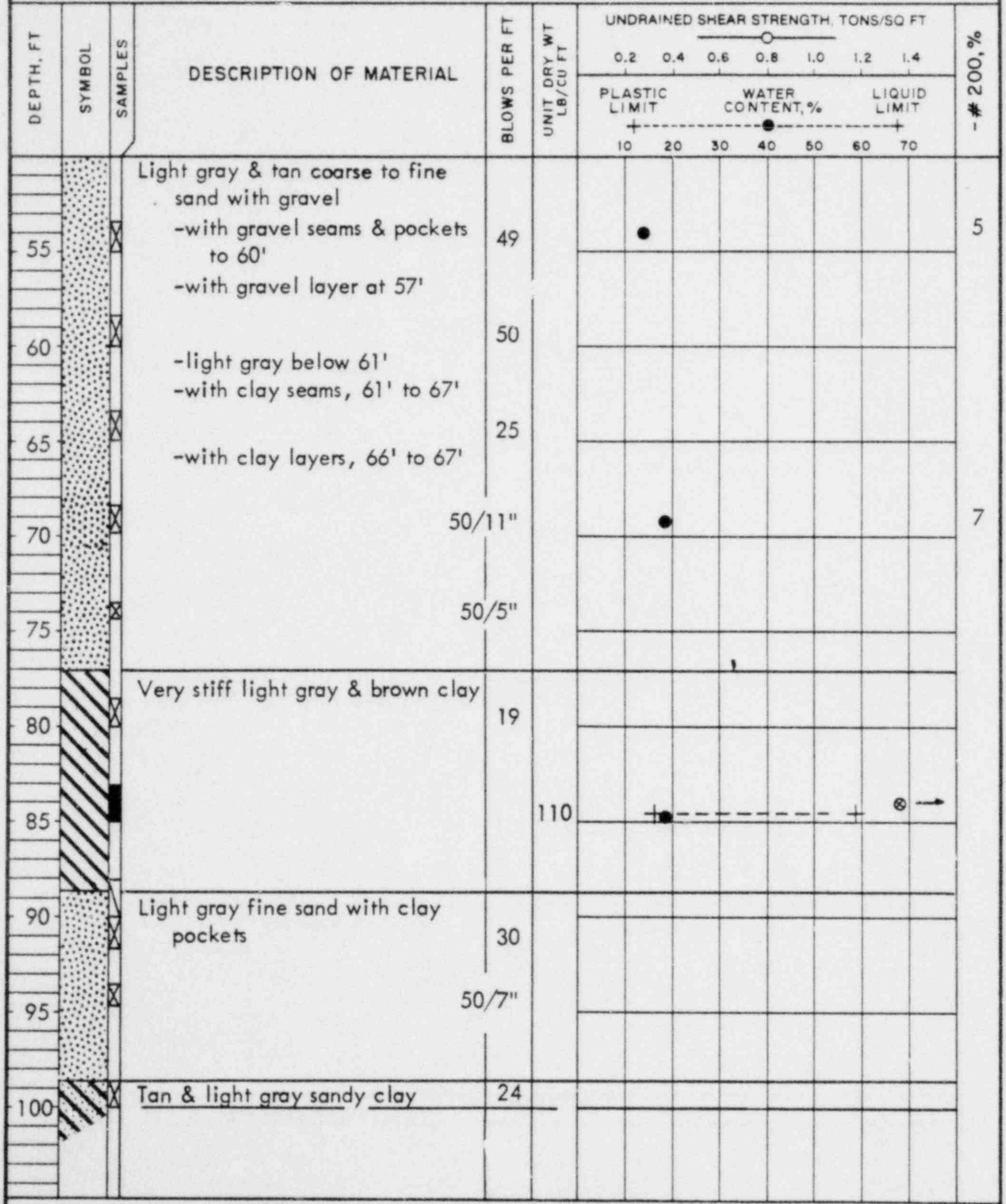
DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	-#200,%
	Very stiff brown silty clay -with roots to 1.5' -with clay seams below 1.5' -dark gray below 3'			
5	Gray fine sand	13		
	-with gray sandy clay layers, 6.5' to 7.5'	15		
10	-with clay pockets below 7.5' -coarse to fine below 8.5'	15		
	-tan below 12'	20		
	Tan clay with fine sand seams & pockets	15		
15	-with gravel pockets, 14' to 18'	20		
		12		
20	Red coarse to fine sand with gravel seams & pockets	22		
25	-with clay layers, 24' to 25'	14		60
	-light gray with clay pockets below 26'			
30		17		
35		31		
40		18		6
	Light gray sandy clay			
45		9		
50	Light gray & tan coarse to fine sand with gravel	50		



(Continued on Plate 4b)

0110-400

LOG OF BORING NO. H-43 (Cont'd)
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	-#200, %
Light gray & tan coarse to fine sand with gravel			
-with gravel seams & pockets to 60'	49		5
-with gravel layer at 57'			
	50		
-light gray below 61'			
-with clay seams, 61' to 67'			
	25		
-with clay layers, 66' to 67'			
	50/11"		7
	50/5"		
Very stiff light gray & brown clay	19		
		110	
Light gray fine sand with clay pockets	30		
	50/7"		
Tan & light gray sandy clay	24		

COMPLETION DEPTH: 100'
DATE: April , 1978

DEPTH TO WATER IN BORING: 7.2' Caved at 40'
DATE: May 1, 1978

Am. No. 50, 1/15/79

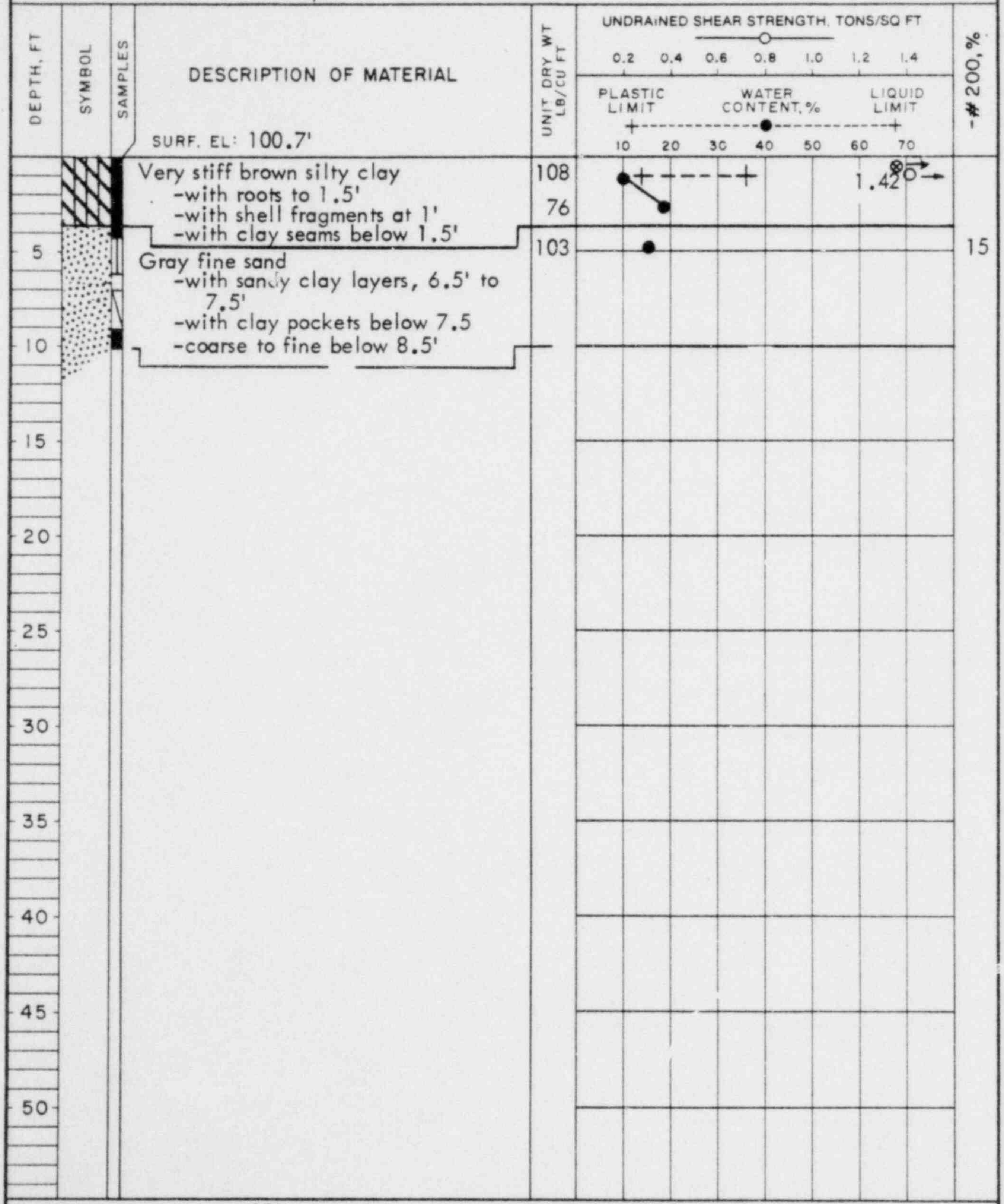
FIG NO. M5b

0178-088

LOG OF BORING NO. A-43A
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS

TYPE: 3" thin-walled tube & Hvorslev Piston Sampler

LOCATION



DESCRIPTION OF MATERIAL

SURF. EL: 100.7'

Very stiff brown silty clay
-with roots to 1.5'
-with shell fragments at 1'
-with clay seams below 1.5'

Gray fine sand
-with sandy clay layers, 6.5' to 7.5'
-with clay pockets below 7.5
-coarse to fine below 8.5'

COMPLETION DEPTH: 10'
DATE: April 26, 1978

DEPTH TO WATER IN BORING: grouted

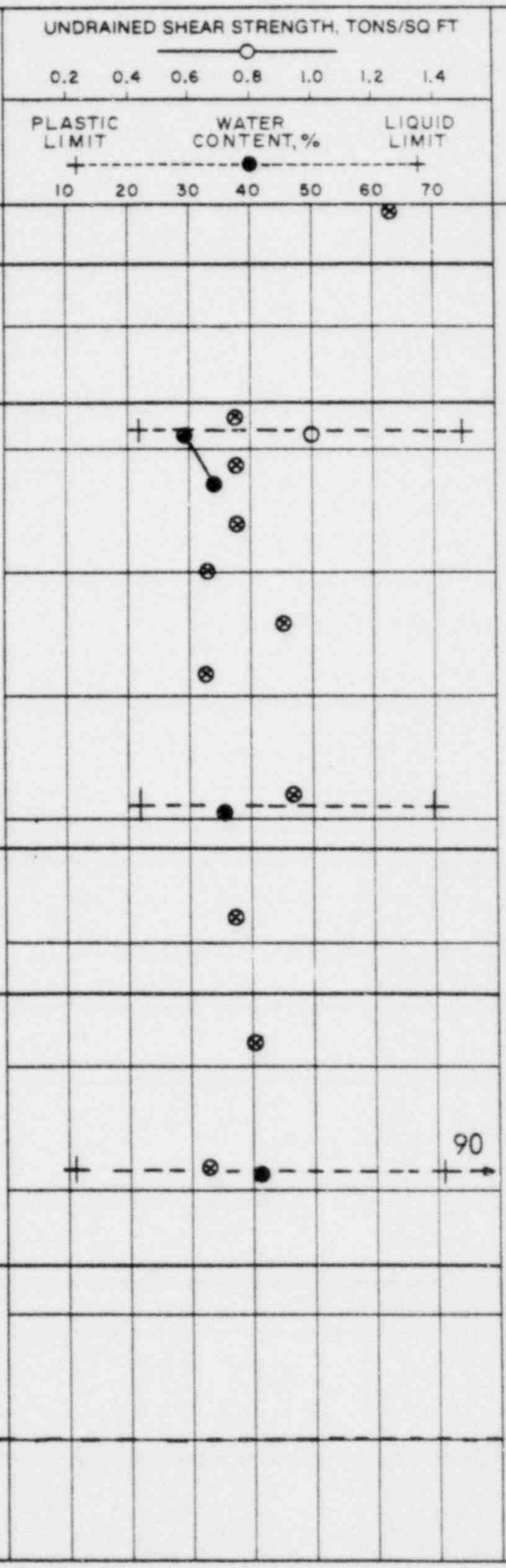
DATE: April 26, 1978

LOG OF BORING NO. H-44
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS

TYPE: 3" thin-walled tube & 2" split-barrel LOCATION

SURF. EL: 98.9'

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT
Very stiff brown clay -with roots to 2'		
Gray fine sand -with silty clay seams to 6' -with clayey silt seams below 7'		
Stiff tan & light gray clay, slickensided with calcareous nodules -light gray & brown below 16'		98 92 87
Stiff light gray & brown sandy clay with shell fragments		
Stiff gray clay -slickensided to 36' -with organic matter & shell fragments, 36' to 41.5' -with fine sand seams below 39' -with sandy clay layers below 42'	7	
Tan fine to medium sand -with gravel seams, 44' to 56' -with sandstone layer, 44.5' to 47'	50/8" 38	

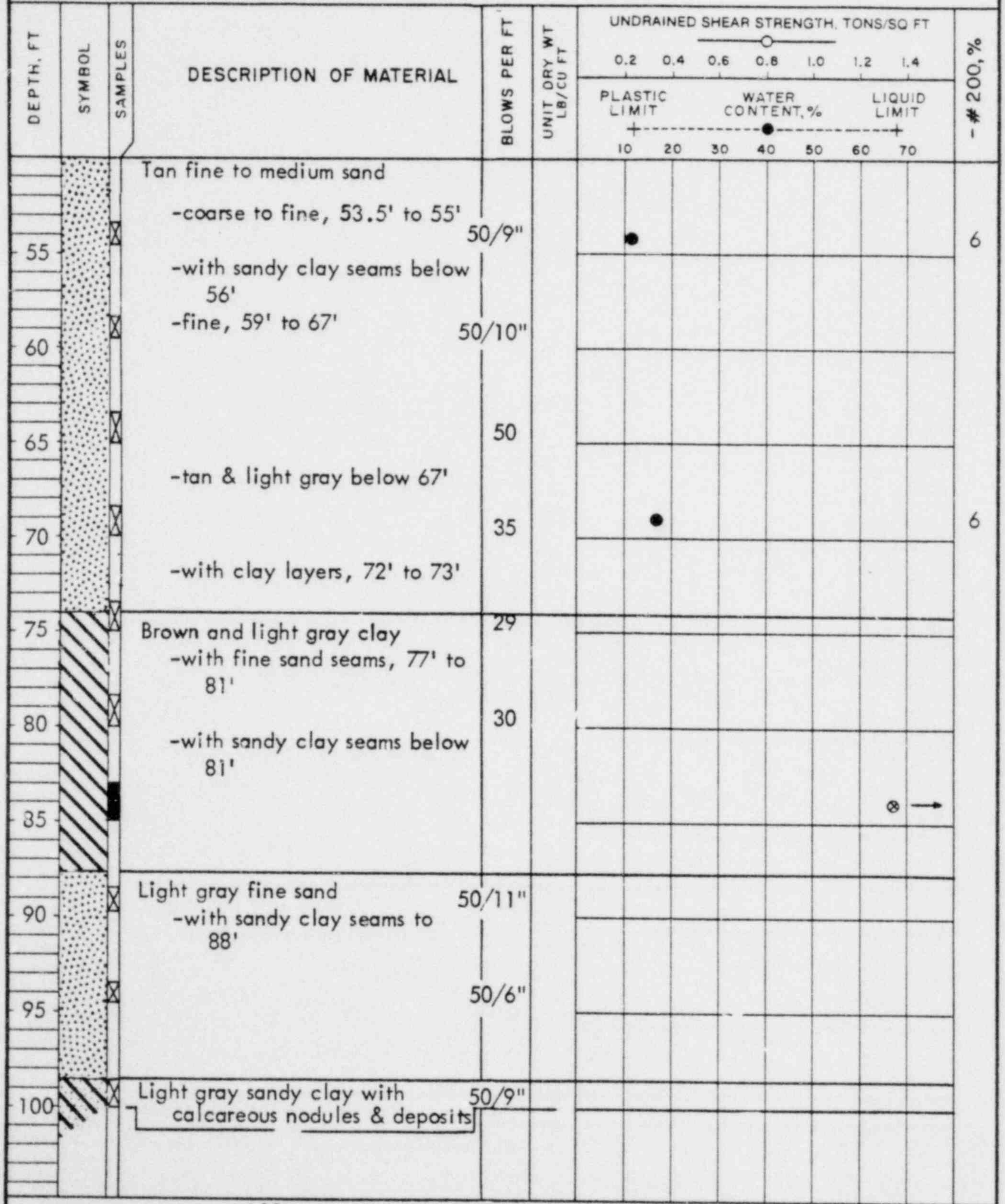


DEPTH, FT: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

(Continued on Plate 6b)

LOG OF BORING NO. H-44 (Cont'd)

ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS



Tan fine to medium sand
-coarse to fine, 53.5' to 55'
-with sandy clay seams below 56'
-fine, 59' to 67'
-tan & light gray below 67'
-with clay layers, 72' to 73'

Brown and light gray clay
-with fine sand seams, 77' to 81'
-with sandy clay seams below 81'

Light gray fine sand
-with sandy clay seams to 88'

Light gray sandy clay with calcareous nodules & deposits

COMPLETION DEPTH: 100'
DATE: April 26, 1978

DEPTH TO WATER IN BORING: 4.2' Caved at 24.1'
DATE: May 1, 1978

Am. No. 50, 1/15/79

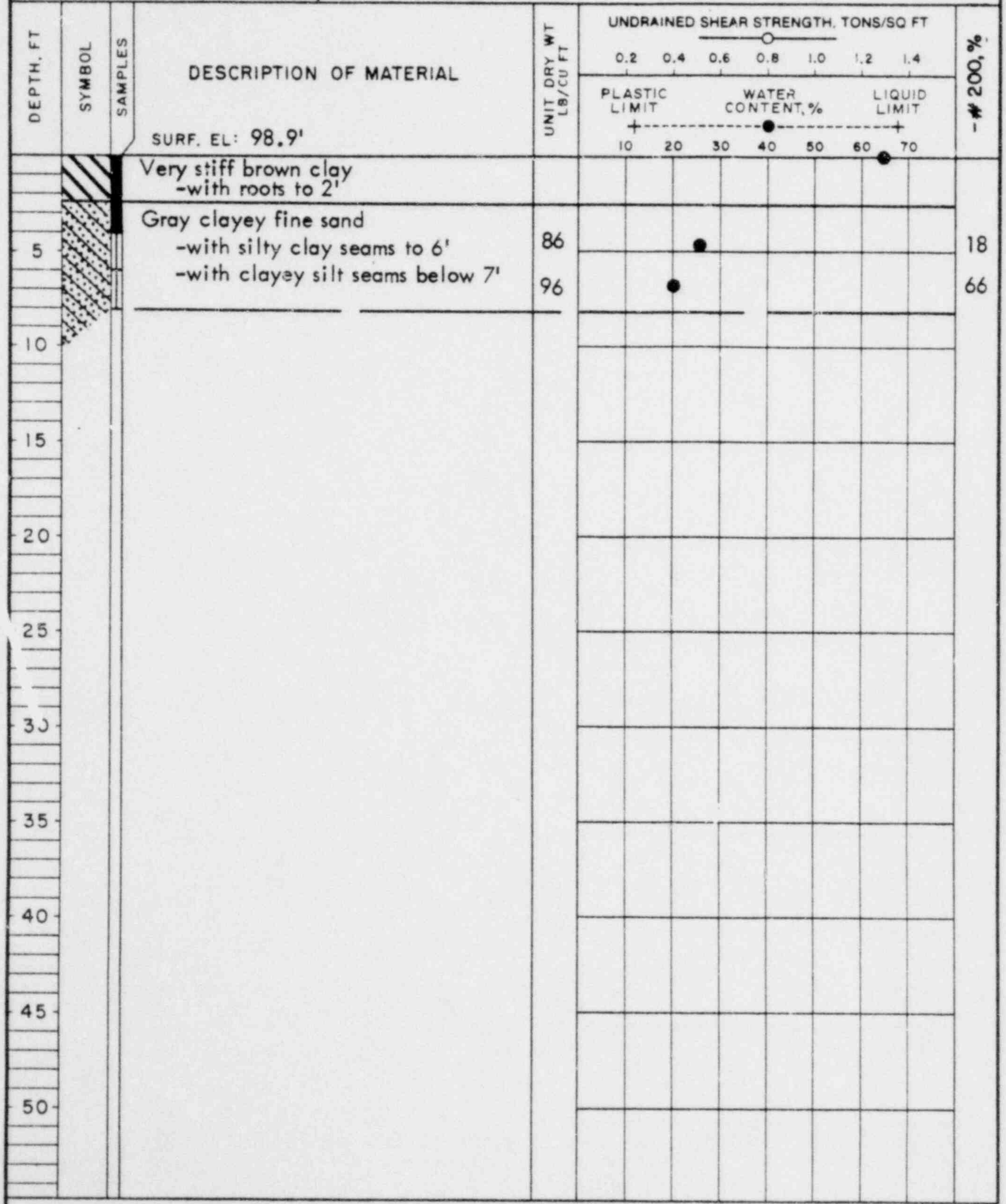
FIG NO. M7b

LOG OF BORING NO. H-44A
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS

TYPE: 3" thin-walled tube & Hvorslev Piston Sampler

LOCATION

SURF. EL: 98.9'



DESCRIPTION OF MATERIAL	UNIT DRY WT LB/CU FT	-# 200, %
Very stiff brown clay -with roots to 2'		
Gray clayey fine sand -with silty clay seams to 6'	86	18
-with clayey silt seams below 7'	96	66

COMPLETION DEPTH: 8'
DATE: April 26, 1978

DEPTH TO WATER IN BORING: grouted

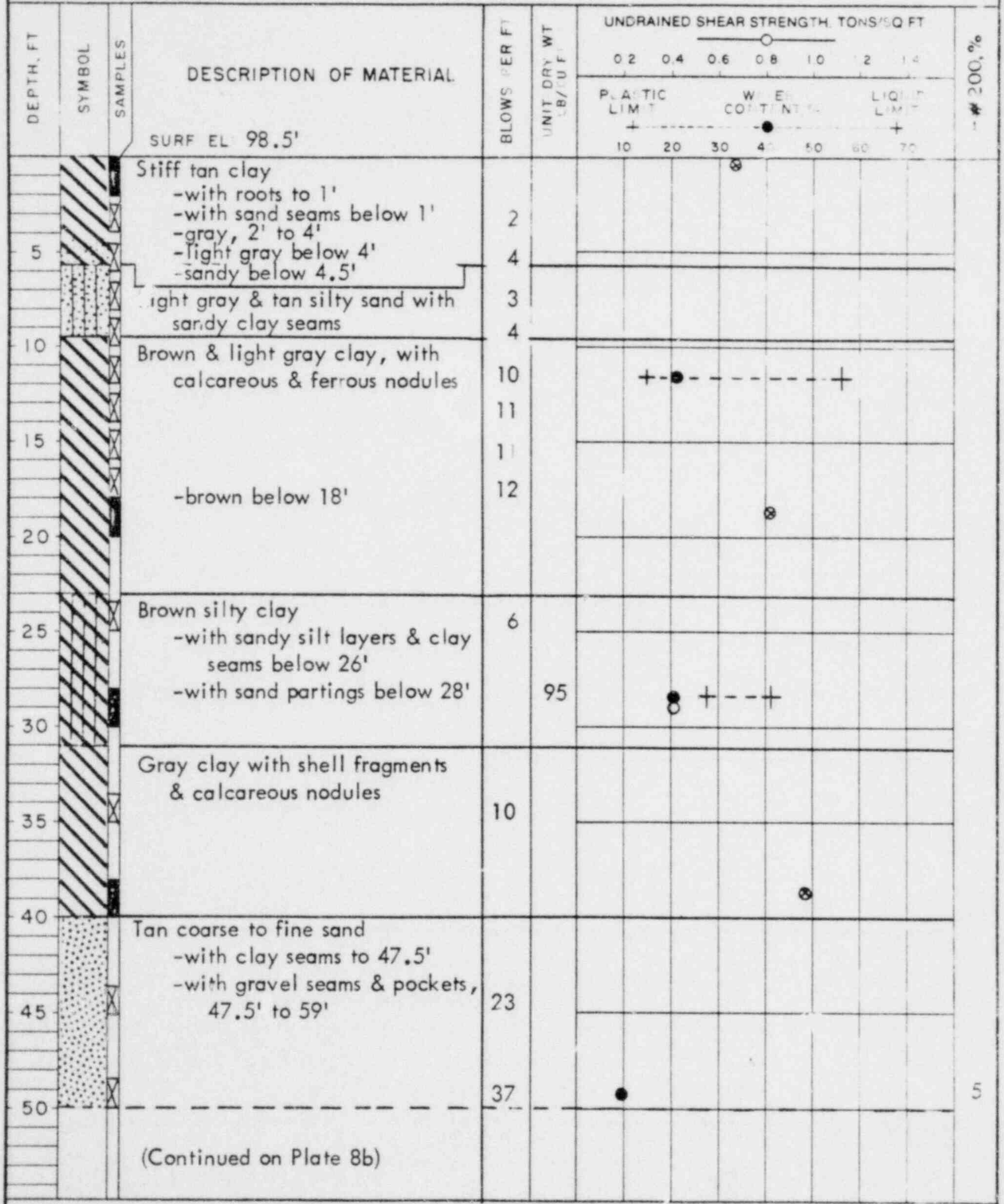
DATE: April 26, 1978

Am. No. 50, 1/15/79

FIG NO. M8

LOG OF BORING NO. H-45
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS

TYPE: 3" thin-walled tube & 2" split-barrel LOCATION:



Stiff tan clay
-with roots to 1'
-with sand seams below 1'
-gray, 2' to 4'
-light gray below 4'
-sandy below 4.5'

Light gray & tan silty sand with sandy clay seams

Brown & light gray clay, with calcareous & ferrous nodules

-brown below 18'

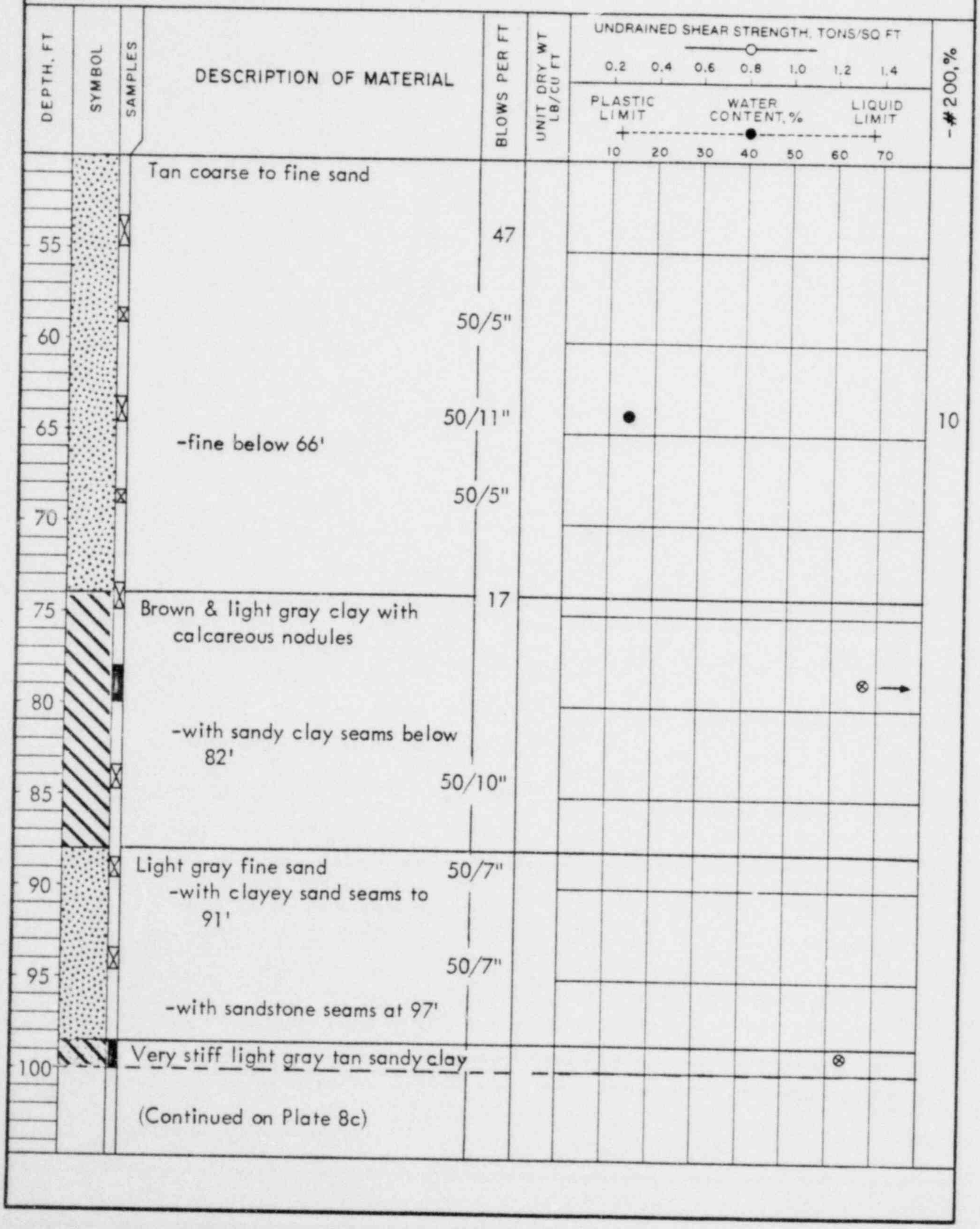
Brown silty clay
-with sandy silt layers & clay seams below 26'
-with sand partings below 28'

Gray clay with shell fragments & calcareous nodules

Tan coarse to fine sand
-with clay seams to 47.5'
-with gravel seams & pockets, 47.5' to 59'

(Continued on Plate 8b)

LOG OF BORING NO. H-45 (Cont'd)
ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS

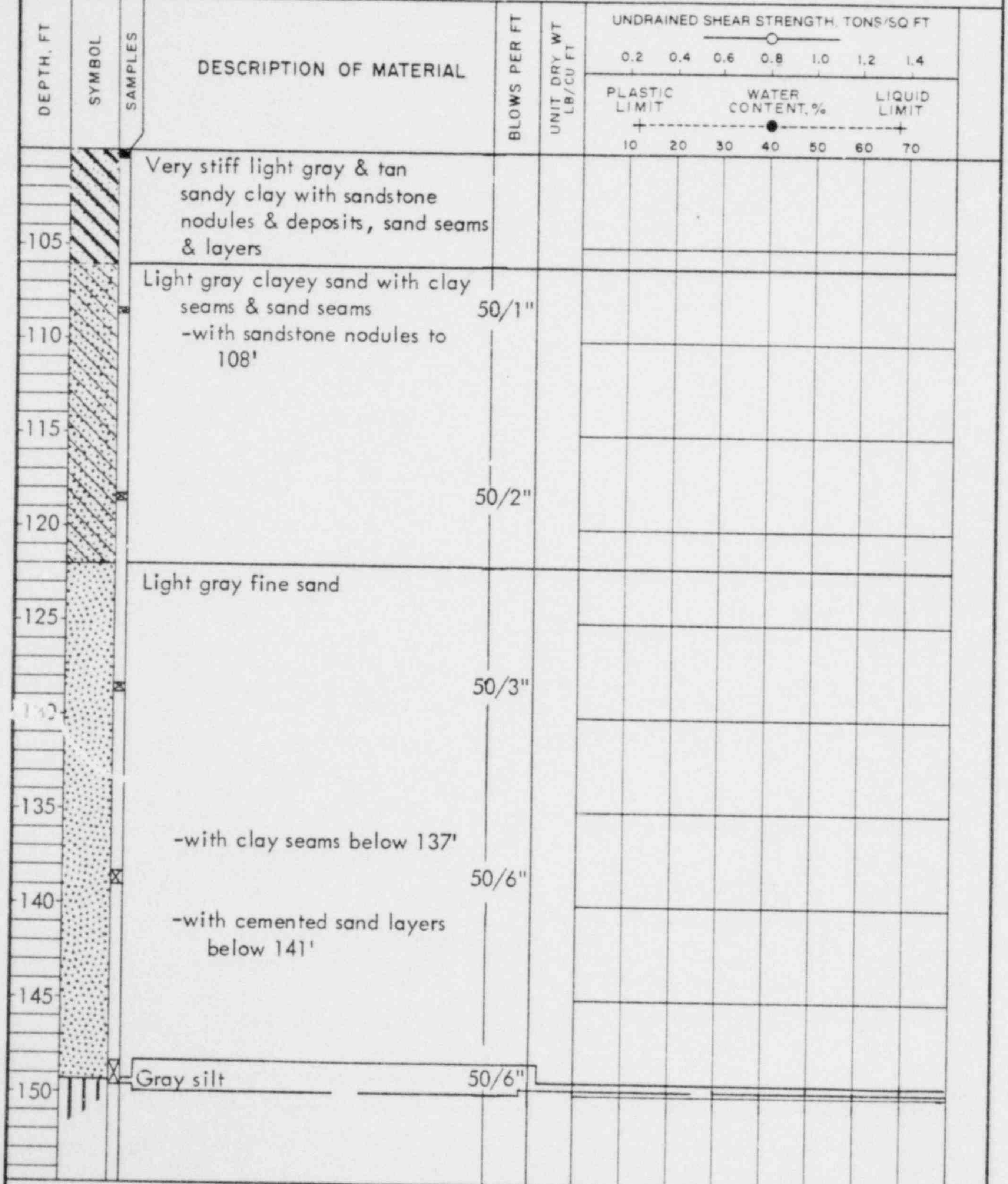


DESCRIPTION OF MATERIAL	BLOWS PER FT	-#200,%
Tan coarse to fine sand	47	
	50/5"	
	50/11"	10
-fine below 66'	50/5"	
Brown & light gray clay with calcareous nodules	17	
-with sandy clay seams below 82'	50/10"	
Light gray fine sand -with clayey sand seams to 91'	50/7"	
	50/7"	
-with sandstone seams at 97'		
Very stiff light gray tan sandy clay		

(Continued on Plate 8c)

LOG OF BORING NO. H-45 (Cont'd)

ALLENS CREEK NUCLEAR GENERATING STATION
ULTIMATE HEAT SINK
WALLIS, TEXAS



Very stiff light gray & tan sandy clay with sandstone nodules & deposits, sand seams & layers

Light gray clayey sand with clay seams & sand seams
-with sandstone nodules to 108'

50/1"

50/2"

Light gray fine sand

50/3"

-with clay seams below 137'

50/6"

-with cemented sand layers below 141'

Gray silt

50/6"

COMPLETION DEPTH: 149.5'
DATE: May 2, 1978

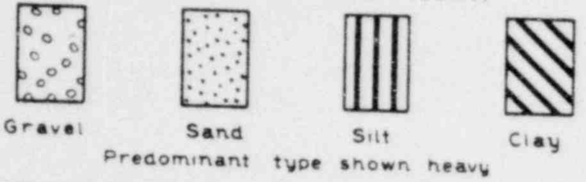
DEPTH TO WATER IN BORING: grouted

DATE: May 2, 1978

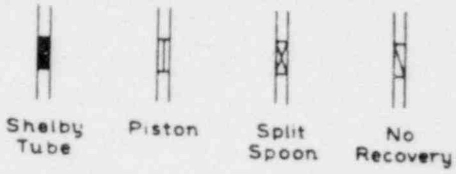
FIG NO. M9c

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES
(SHOWN IN SYMBOL COLUMN)



SAMPLER TYPES
(SHOWN IN SAMPLES COLUMN)



TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS (major portion retained on No. 200 sieve): includes (1) clean gravels and sands, and (2) silty or clayey gravels and sands. Condition is rated according to relative density, as determined by laboratory tests.

DESCRIPTIVE TERM	RELATIVE DENSITY
Loose	0 to 40%
Medium dense	40 to 70%
Dense	70 to 100%

FINE GRAINED SOILS (major portion passing No. 200 sieve): includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings or by unconfined compression tests.

DESCRIPTIVE TERM	UNCONFINED COMPRESSIVE STRENGTH TON/SQ FT
Very soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very stiff	2.00 to 4.00
Hard	4.00 and higher

Note: Slickensided and fissured clays may have lower unconfined compressive strengths than shown above, because of planes of weakness or cracks in the soil. The consistency ratings of such soils are based on penetrometer readings.

TERMS CHARACTERIZING SOIL STRUCTURE

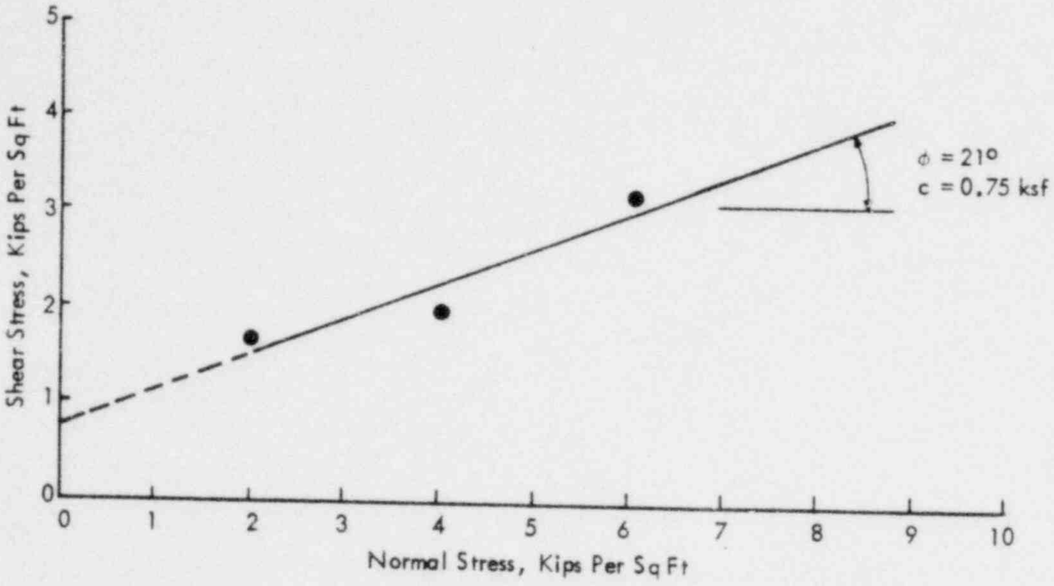
Slickensided	– having inclined planes of weakness that are slick and glossy in appearance.
Fissured	– containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
Laminated	– composed of thin layers of varying color and texture.
Interbedded	– composed of alternate layers of different soil types.
Calcareous	– containing appreciable quantities of calcium carbonate.
Well graded	– having wide range in grain sizes and substantial amounts of all intermediate particle sizes.
Poorly graded	– predominantly of one grain size, or having a range of sizes with some intermediate size missing.

Terms used in this report for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in Technical Memorandum No 3-357, Waterways Experiment Station, March 1953

FIG NO. M10

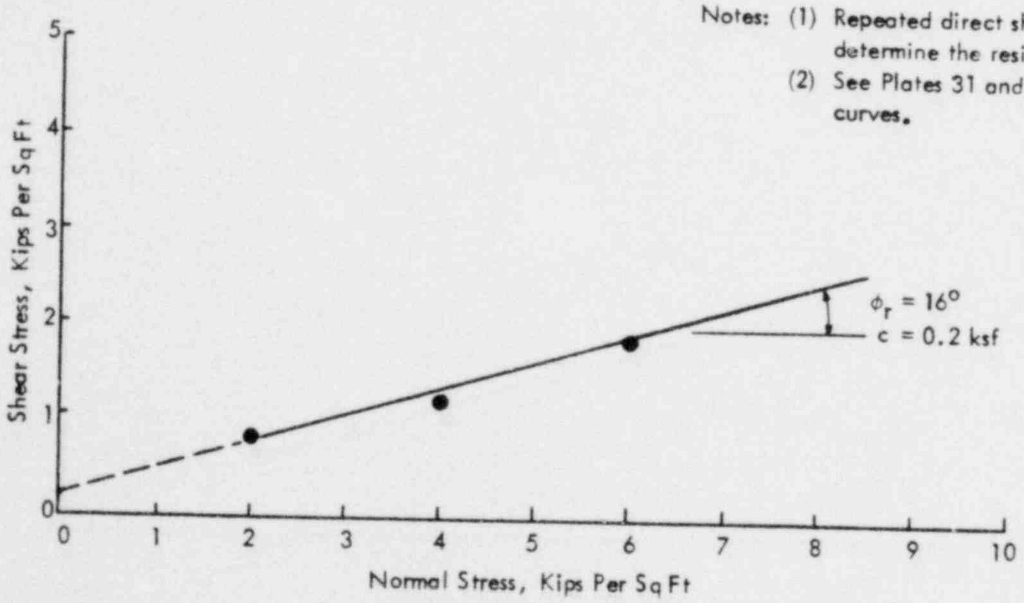
Boring: H-43A Depth: 3'
Material: Stiff dark gray sandy clay with clay pockets and calcareous deposits

γ_d = 102 psf
w_i = 18
LL = 36
PL = 14



UNDISTURBED

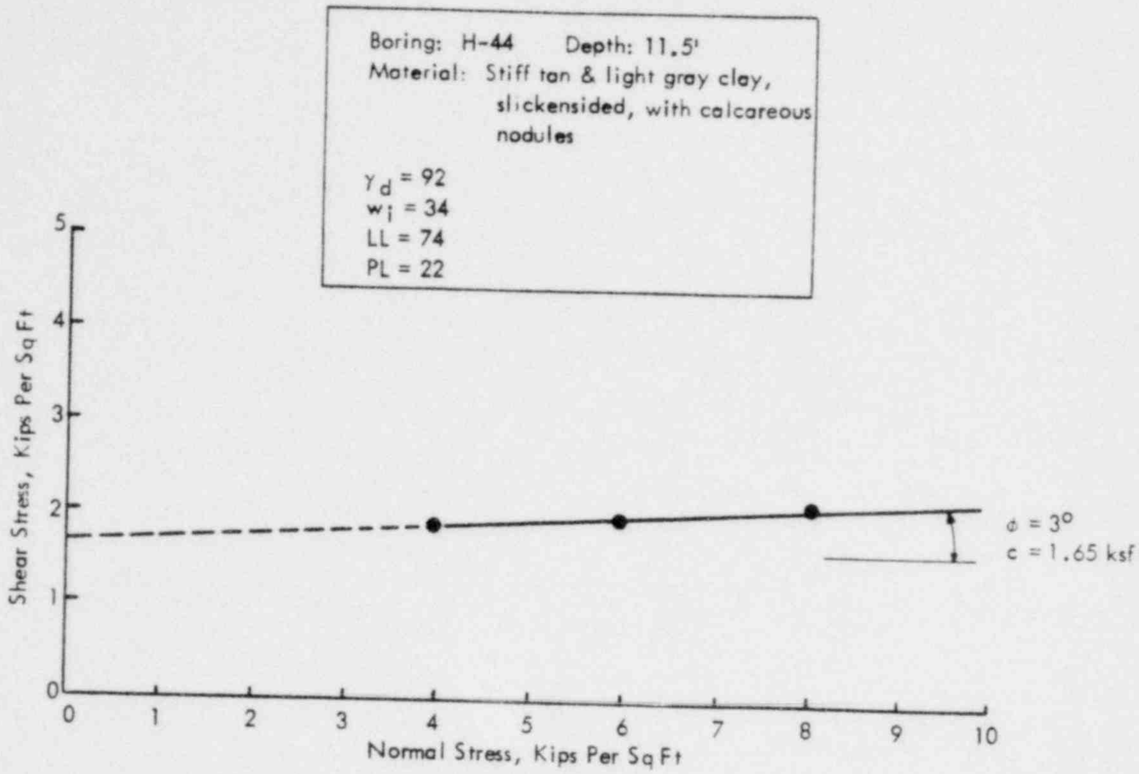
Notes: (1) Repeated direct shear test used to determine the residual shear strength.
(2) See Plates 31 and 32 for stress-strain curves.



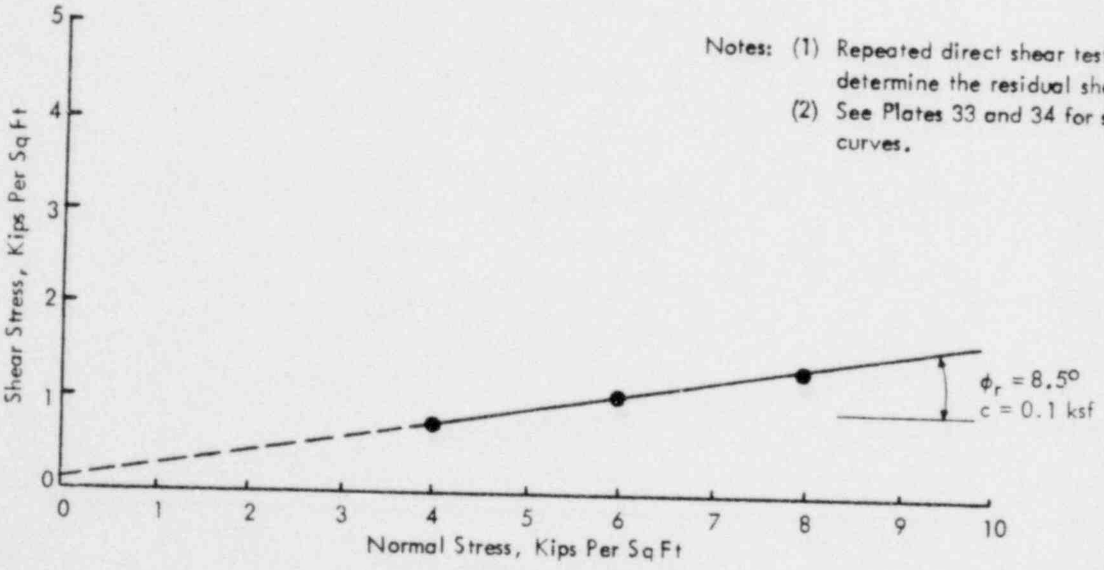
RESIDUAL

DIRECT SHEAR TEST RESULTS

Consolidated-Undrained
Multiple-Stage Type



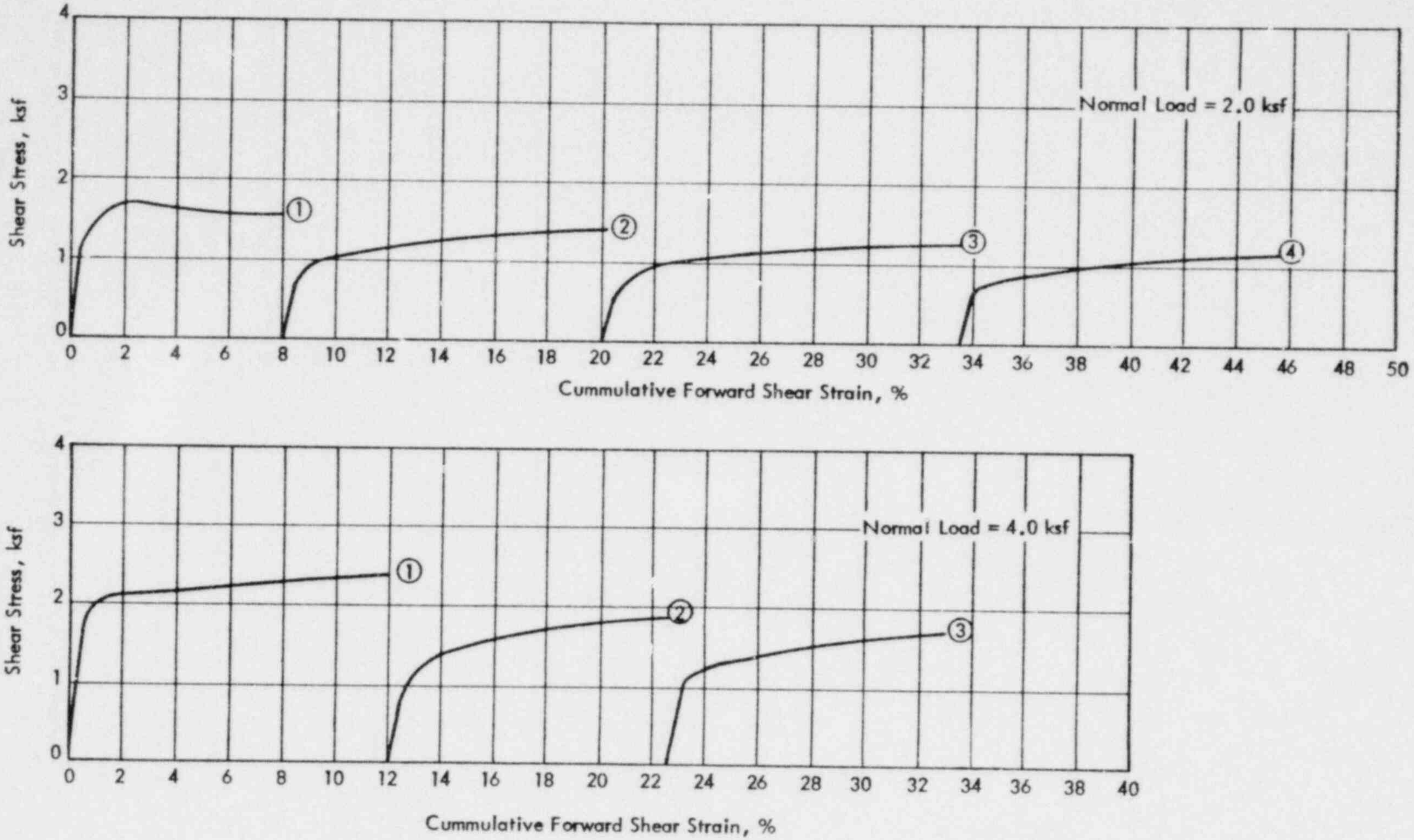
UNDISTURBED



RESIDUAL

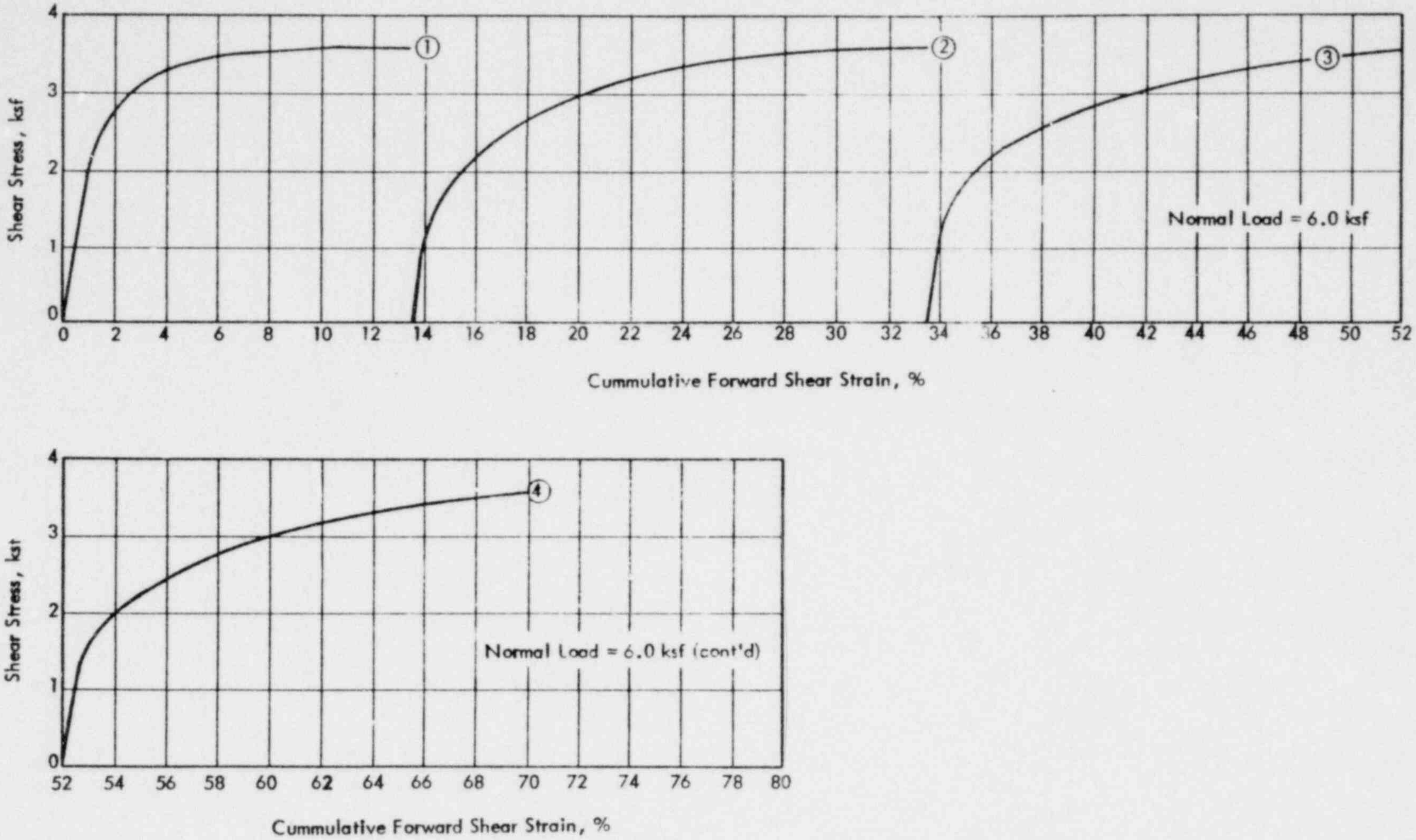
Notes: (1) Repeated direct shear tests used to determine the residual shear strength.
(2) See Plates 33 and 34 for stress-strain curves.

DIRECT SHEAR TEST RESULTS
Consolidated-Undrained
Multiple-Specimen Type



REPEATED DIRECT SHEAR
Stress-Strain Curves
Boring H-43A, 3-ft Depth

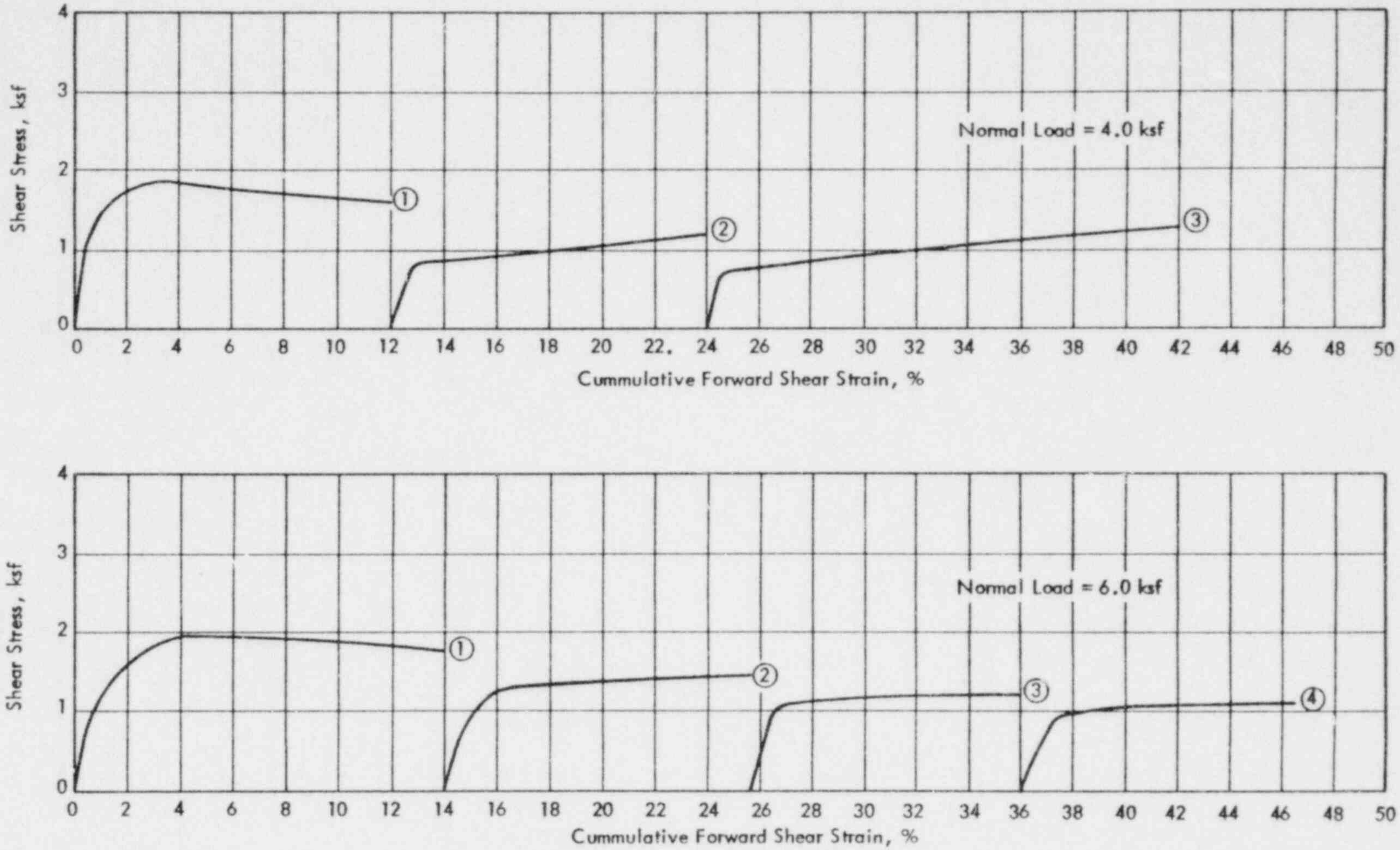
FIG NO. M13



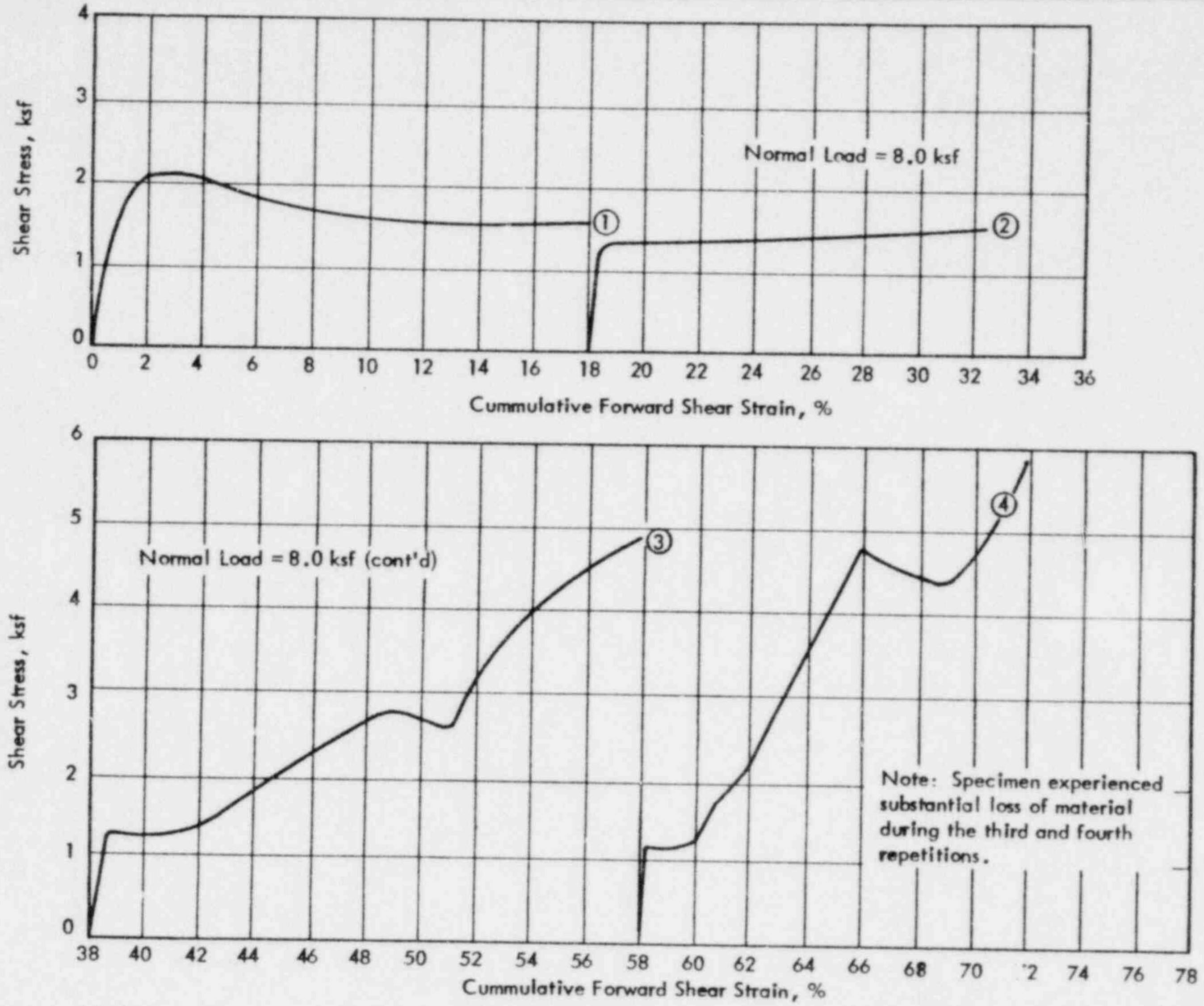
REPEATED DIRECT SHEAR

Stress-Strain Curves

Boring H-43A, 3-ft Depth



REPEATED DIRECT SHEAR
Stress-Strain Curves
Boring H-44, 11.5-ft Depth



Normal Load = 8.0 ksf

Normal Load = 8.0 ksf (cont'd)

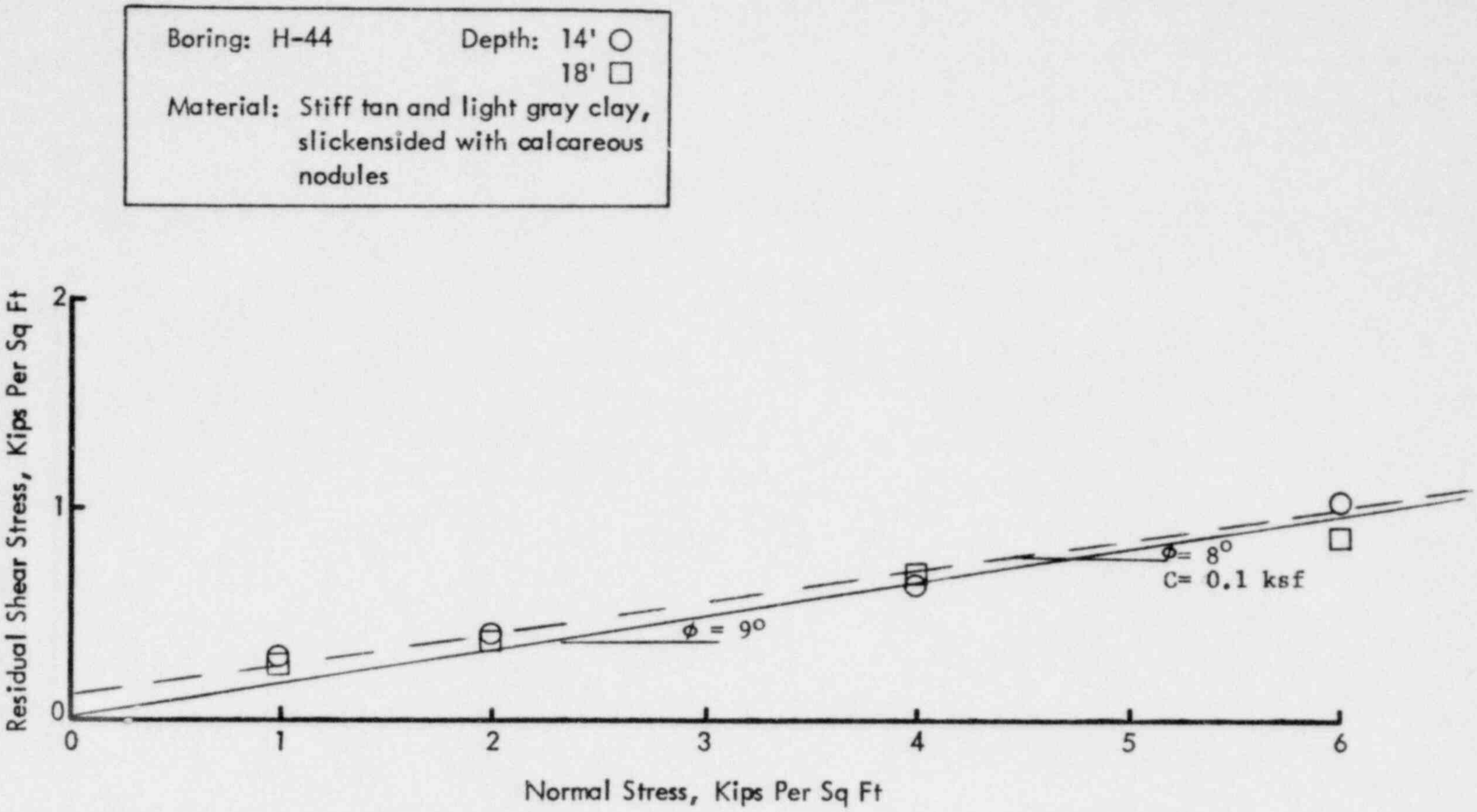
Note: Specimen experienced substantial loss of material during the third and fourth repetitions.

REPEATED DIRECT SHEAR

Stress-Strain Curves

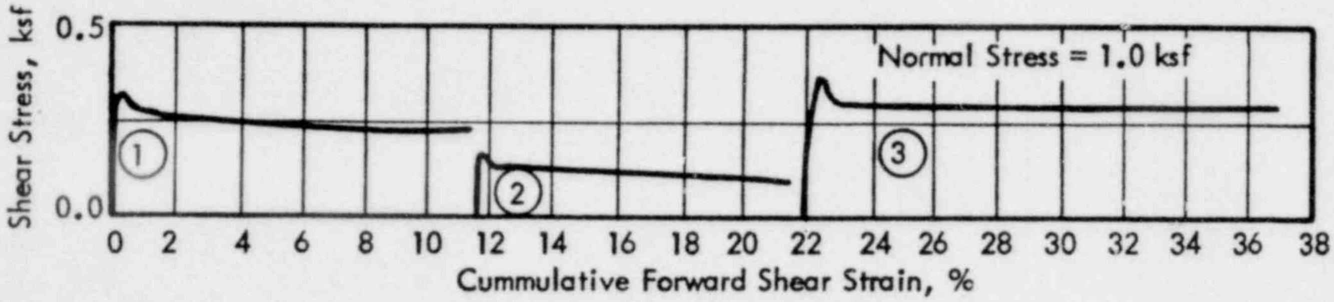
Boring H-44, 11.5-ft Depth

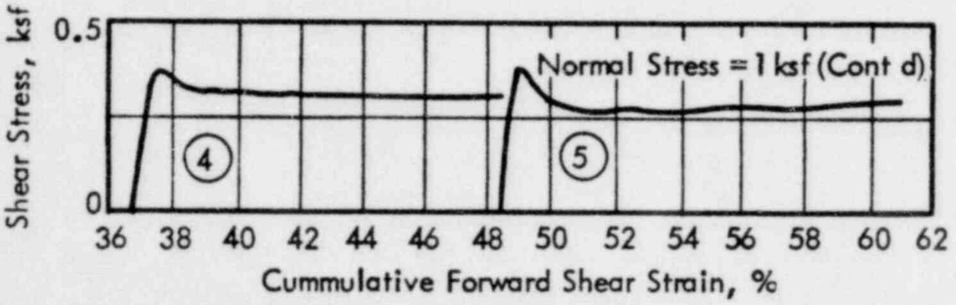
FIG NO. M16

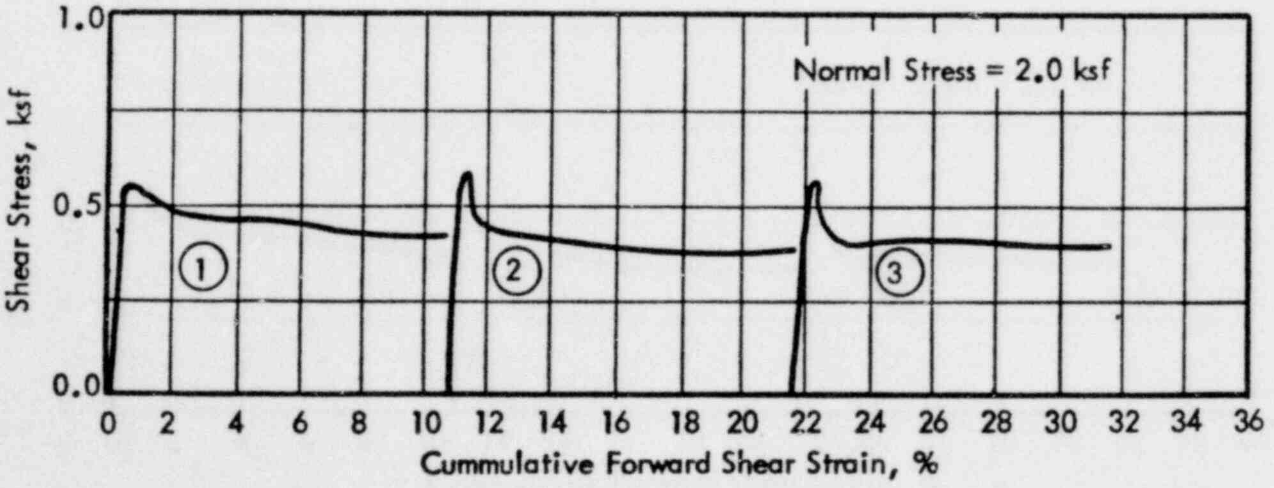


CONSOLIDATED-DRAINED REPEATED DIRECT SHEAR TEST RESULTS

Boring: H-44 Depth: 14'
Material: Stiff tan and light gray clay, slickensided with calcareous nodules

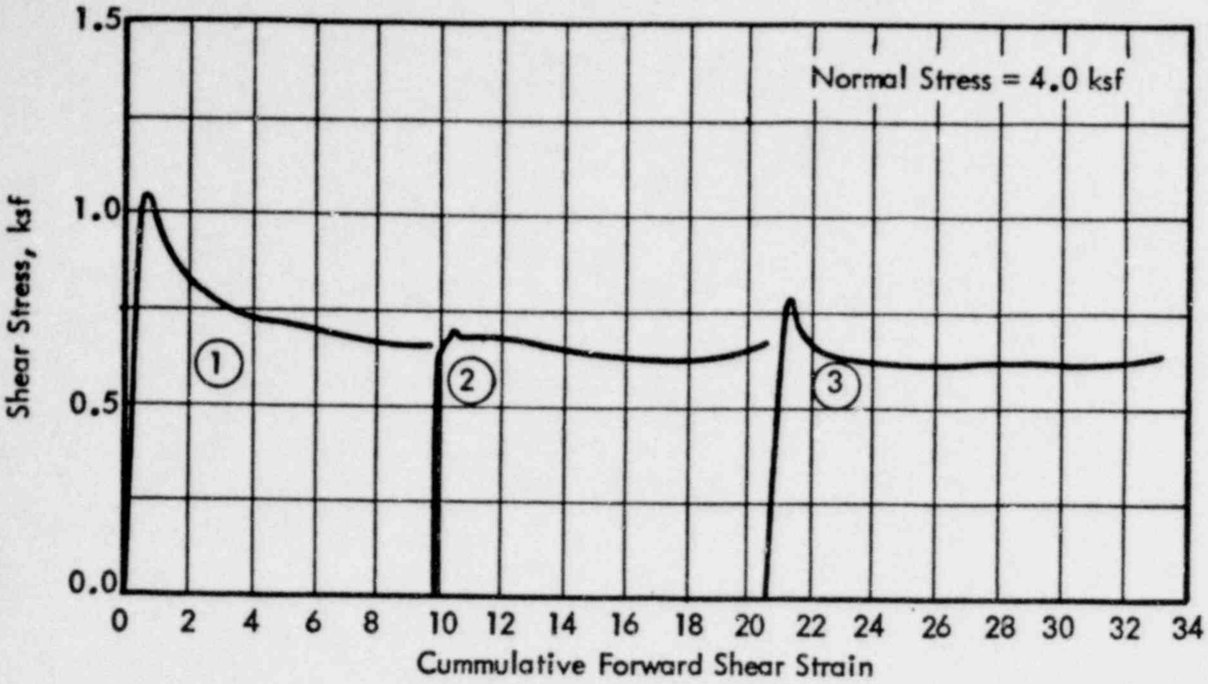


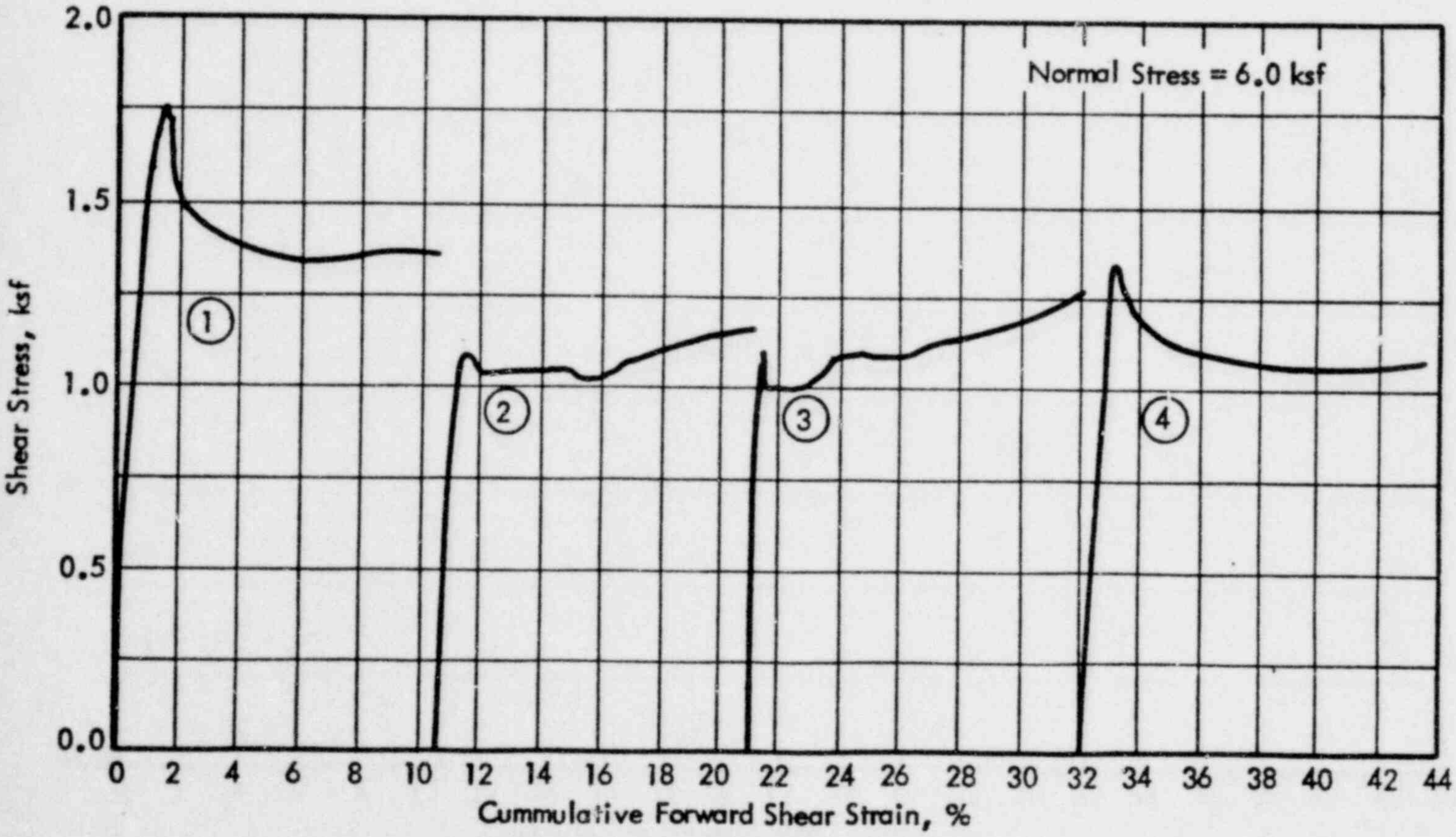




Shear Stress, ksf

Shear Stress, ksf

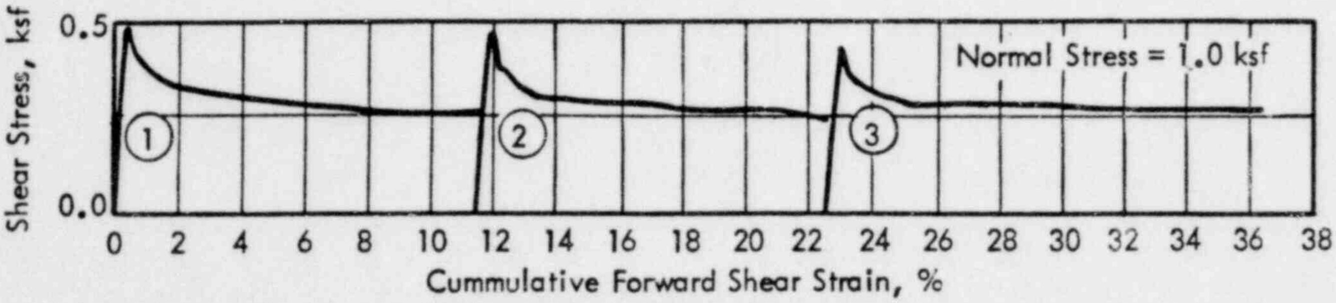


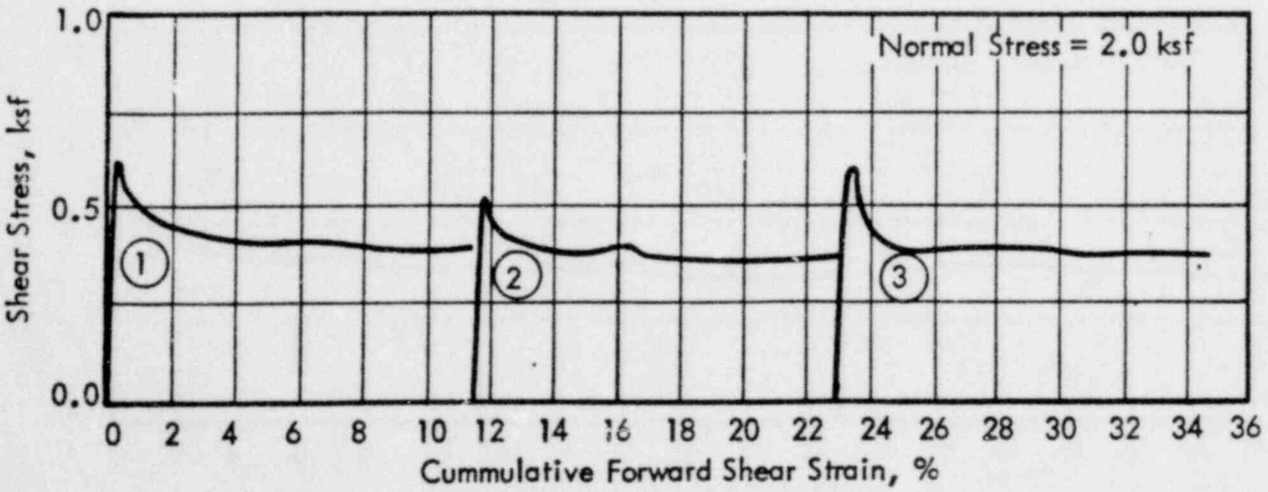


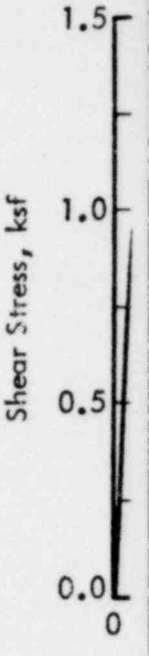
CONSOLIDATED-DRAINED REPEATED DIRECT SHEAR
Stress-Strain Curves

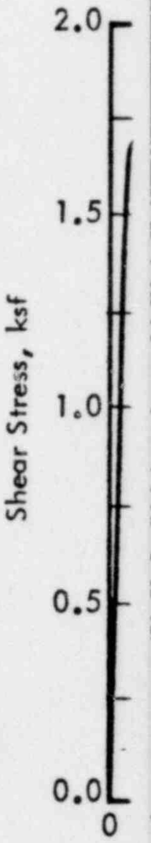
Boring: H-44 Depth: 18'
Material: Stiff tan and light gray clay, slickensided with calcareous nodules

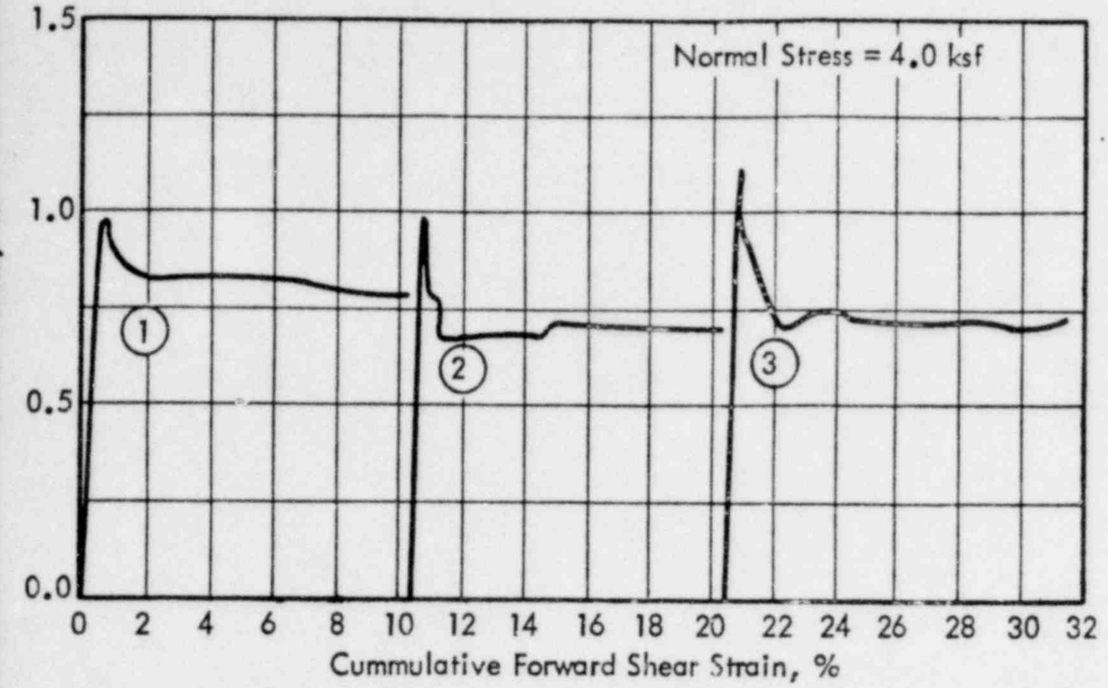
Notes: (1) Tests were mutiple-specimen type
(2) Samples were allowed to consolidate to equilibrium prior to each shear cycle.

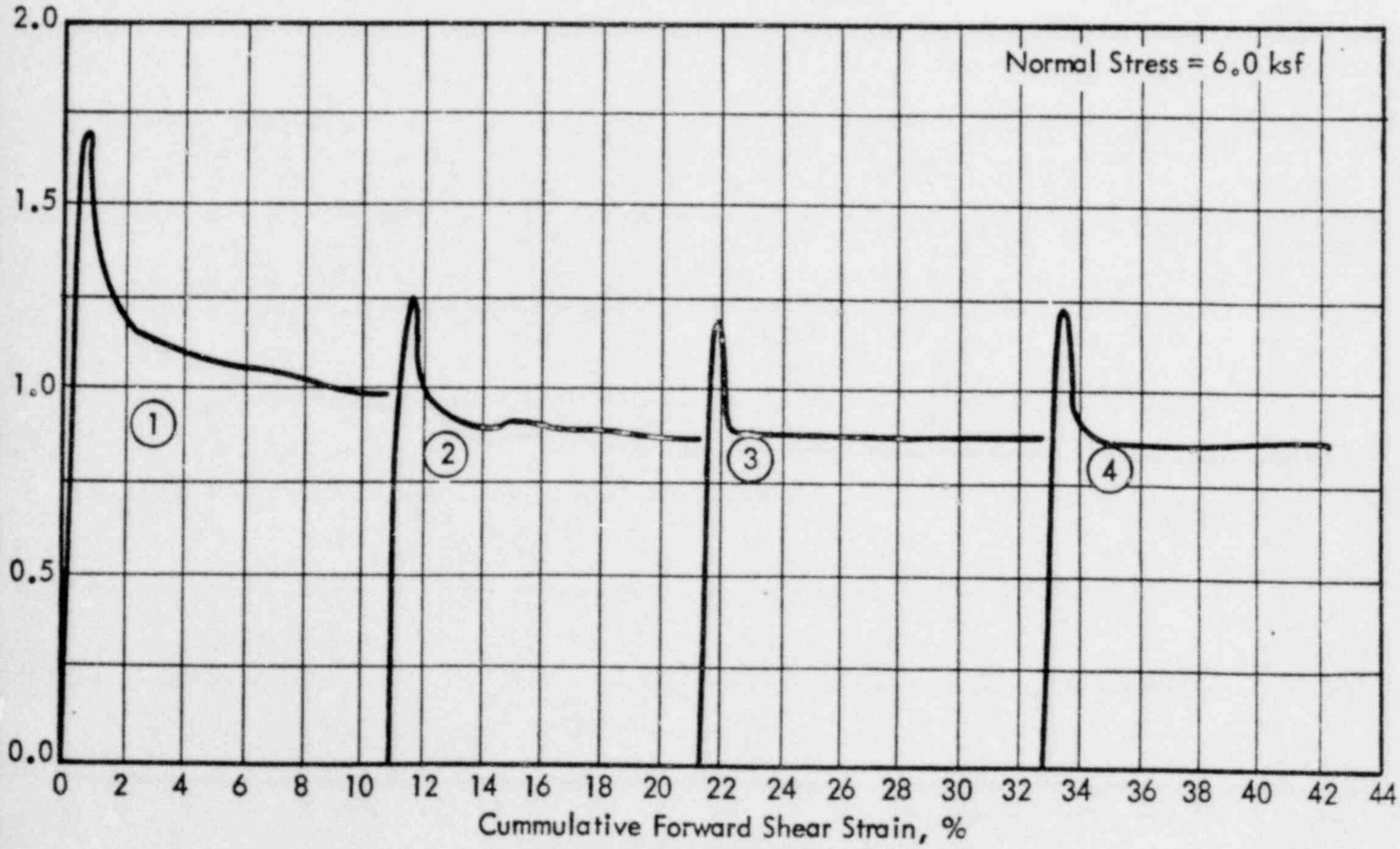






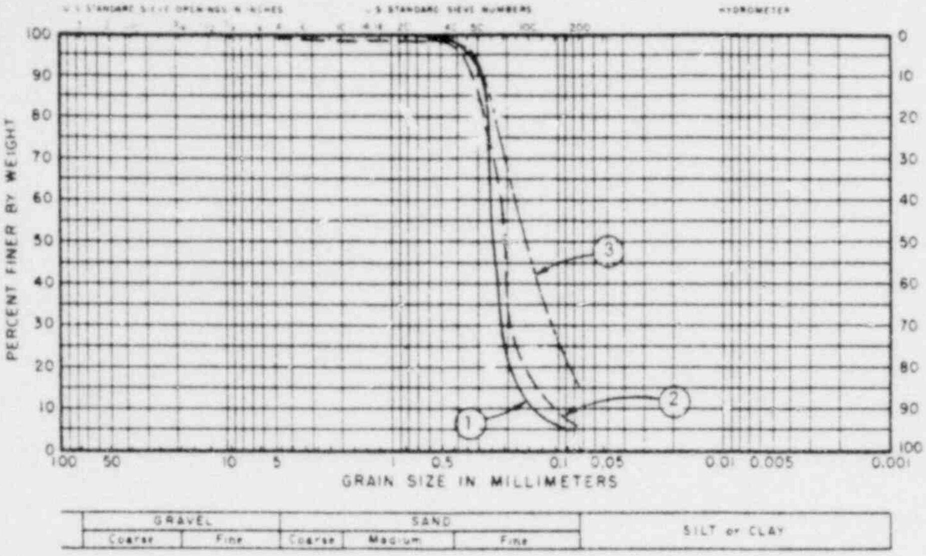






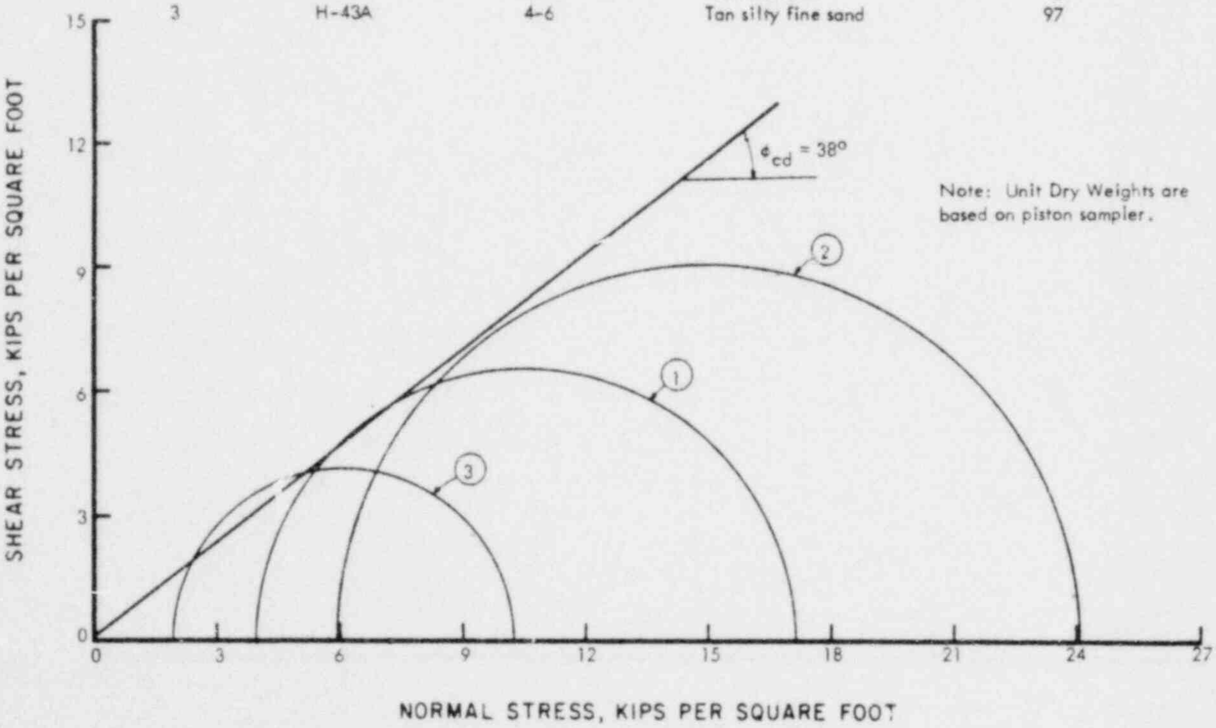
CONSOLIDATED-DRAINED REPEATED DIRECT SHEAR

Stress-Strain Curves



GRADATION OF TEST SPECIMEN

Curve	Boring	Penetration	Material	Unit Dry Weight, pcf
1	H-42A	7-9	Tan fine sand	96
2	H-42A	13-15	Tan fine sand	98
3	H-43A	4-6	Tan silty fine sand	97



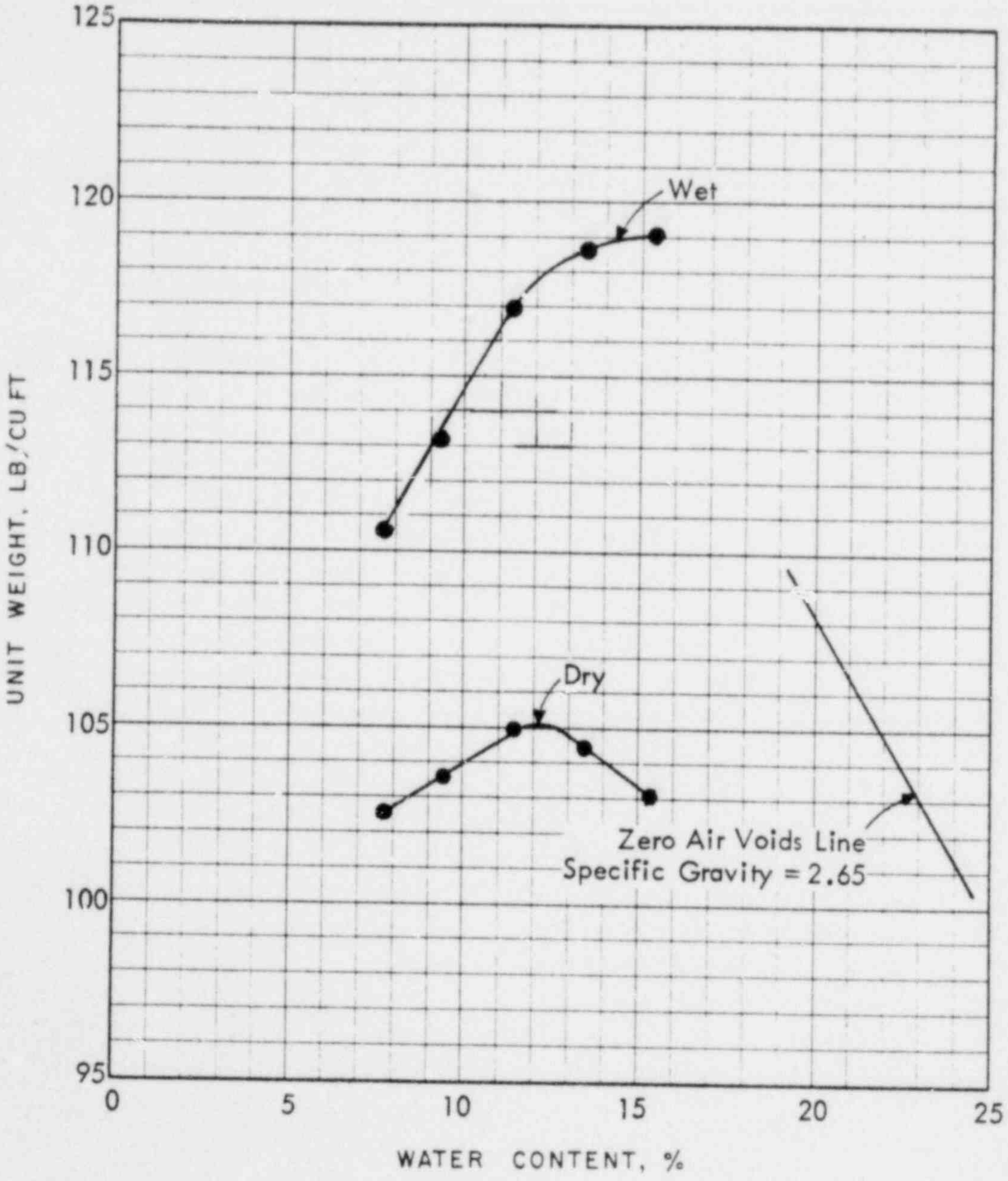
Note: Unit Dry Weights are based on piston sampler.

TRIAXIAL COMPRESSION TEST RESULTS

Consolidated-Drained, Multiple-Specimen Type

Test Pit: 6
Depth: 5'
TEST METHOD: ASTM 1557
MATERIAL: Tan fine sand

OPTIMUM WATER CONTENT: 12 %
MAX UNIT DRY WEIGHT: 105 LB/CU FT



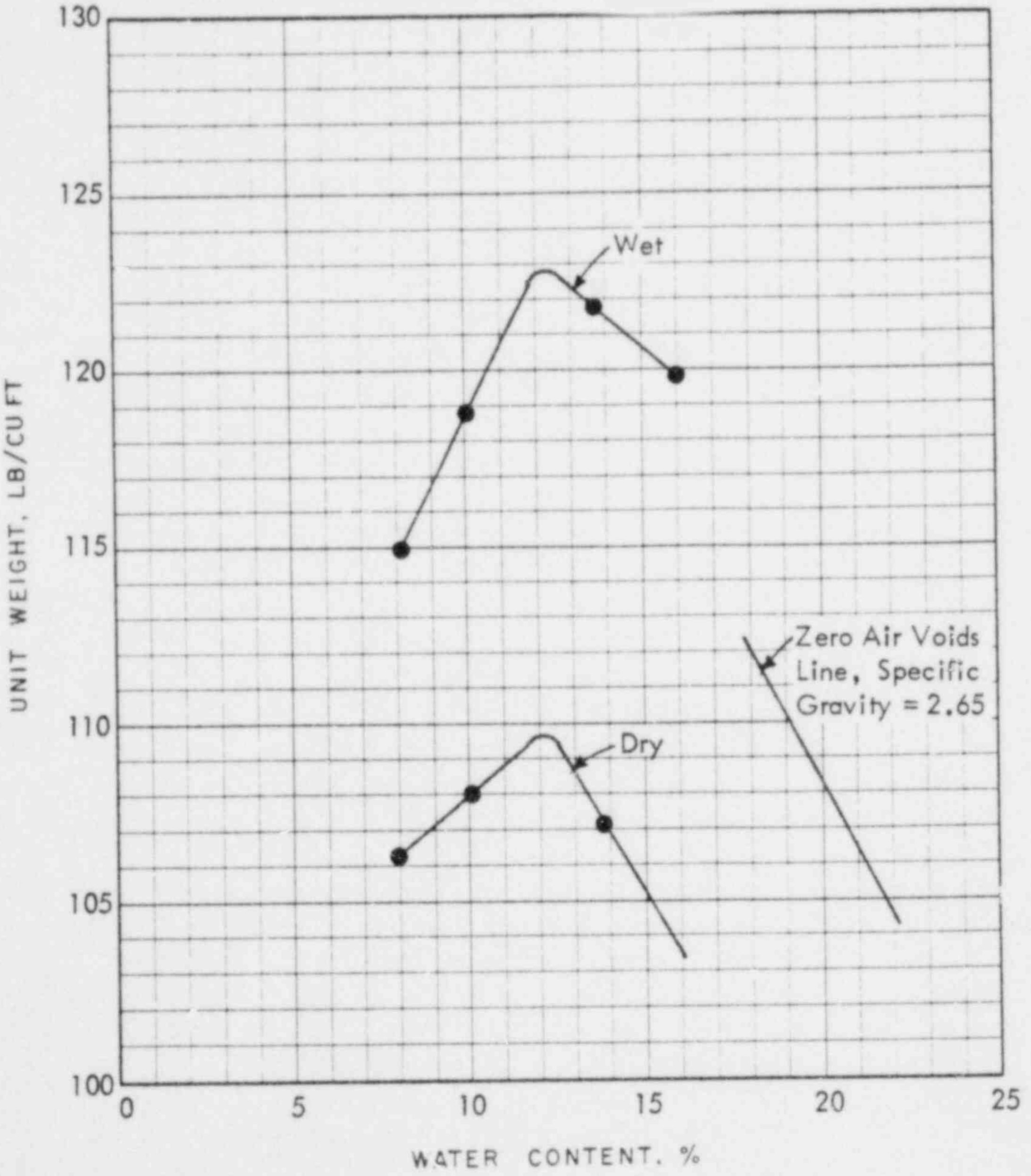
COMPACTION TEST RESULTS

Am. No. 50, 1/15/79

FIG NO. M21

Test Pit: 6
Depth: 10'
TEST METHOD: ASTM 1557
MATERIAL: Tan fine sand

OPTIMUM WATER CONTENT: 12 %
MAX UNIT DRY WEIGHT: 110 LB/CU FT



COMPACTION TEST RESULTS

Test Pit: 7

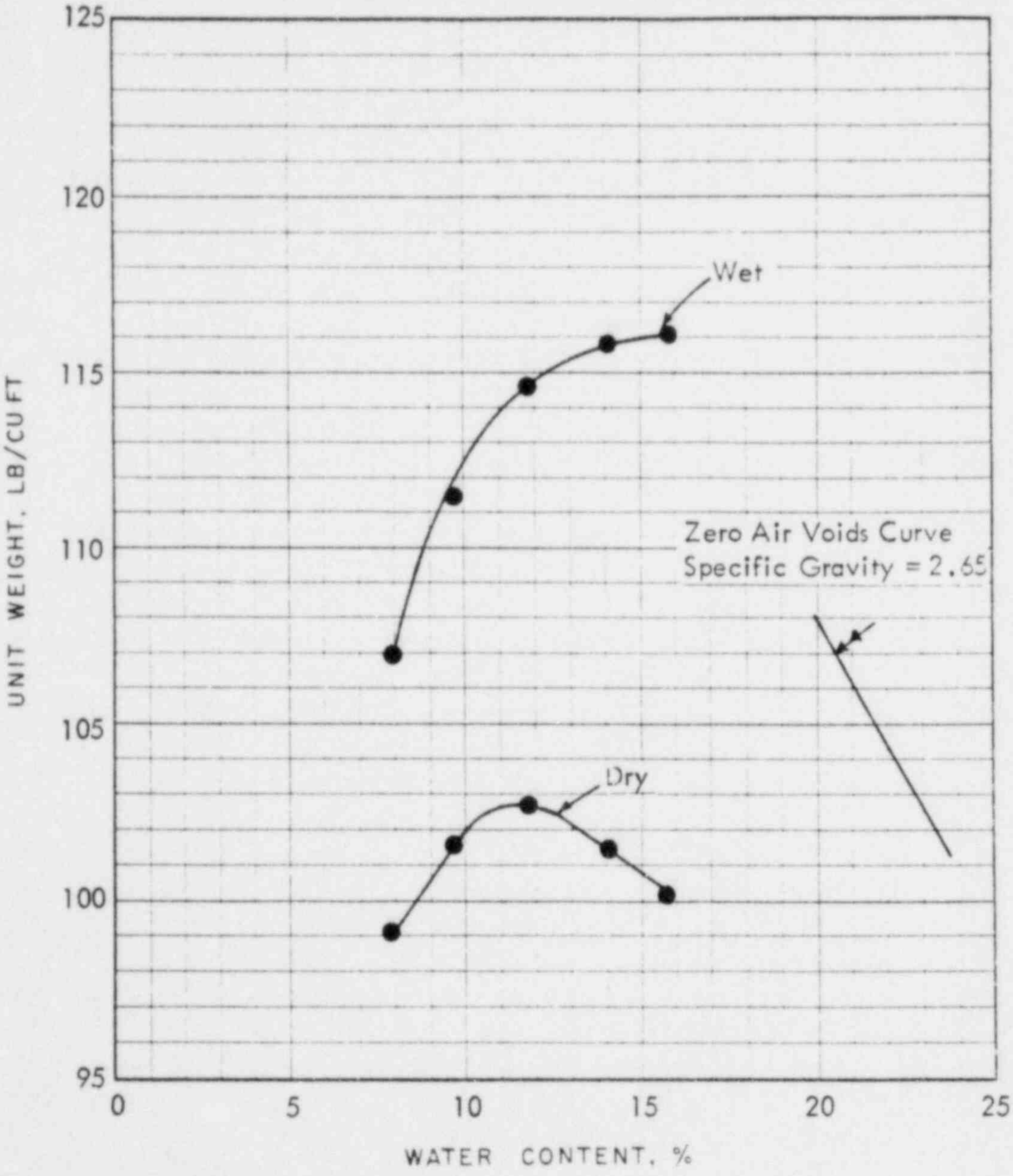
Depth: 5'

TEST METHOD: ASTM 1557

MATERIAL: Tan fine sand

OPTIMUM WATER CONTENT: 12 %

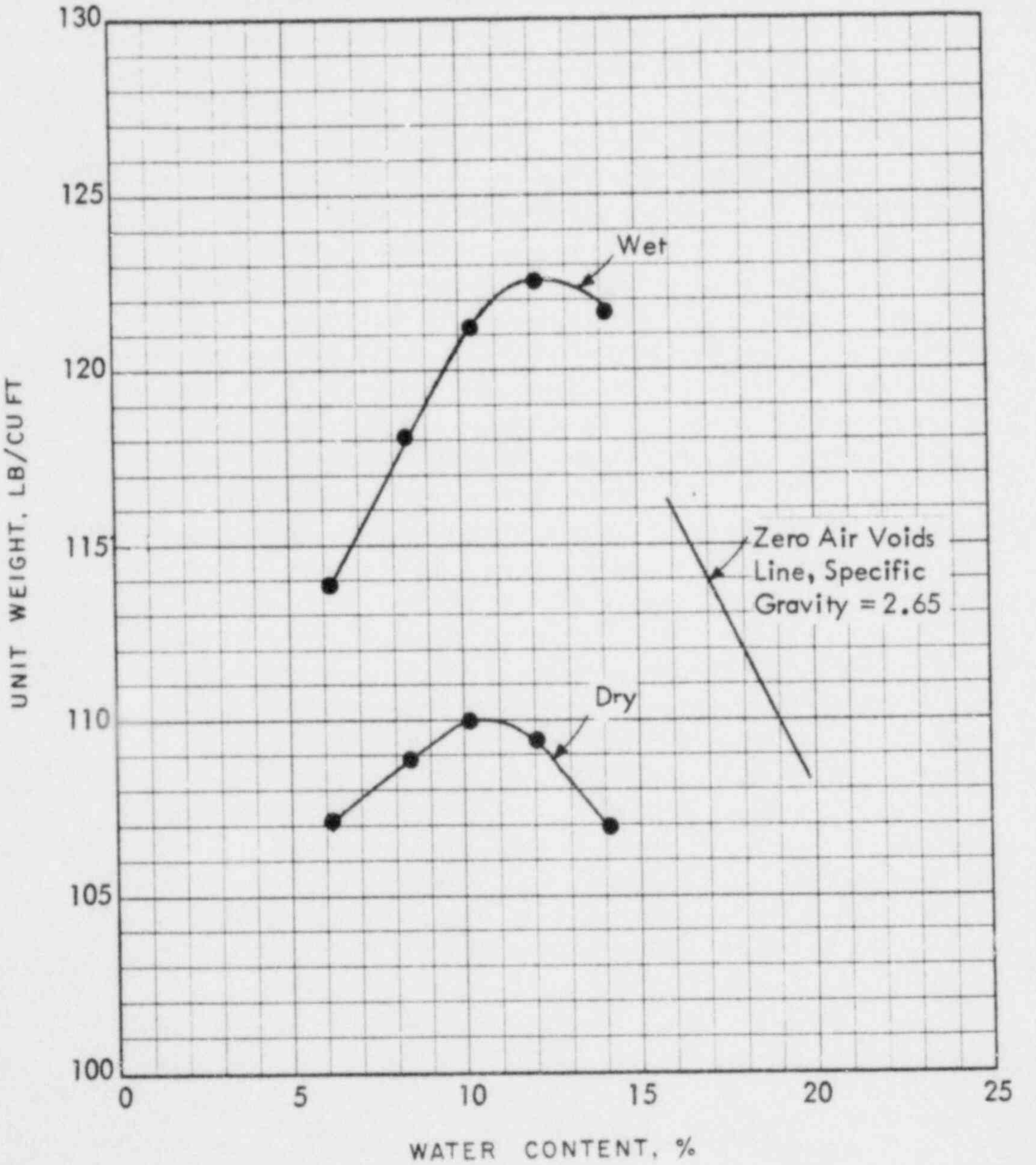
MAX UNIT DRY WEIGHT: 103 LB/CU FT



COMPACTION TEST RESULTS

Test Pit: 8
Depth: 5'
TEST METHOD: ASTM 1557
MATERIAL: Tan fine sand

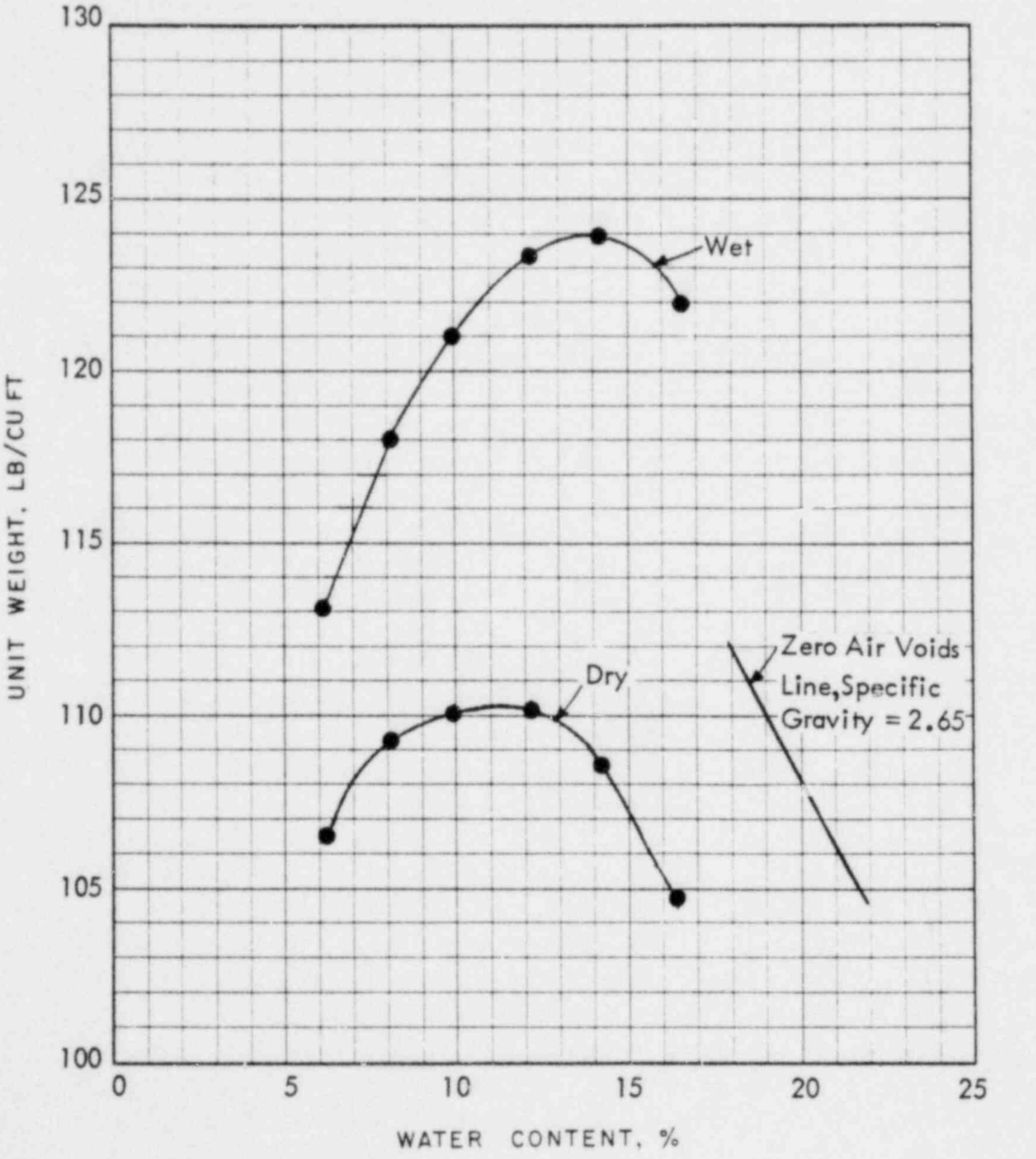
OPTIMUM WATER CONTENT: 11 %
MAX UNIT DRY WEIGHT: 110 LB/CU FT



COMPACTION TEST RESULTS

Test Pit: 9
Depth: 5'
TEST METHOD: ASTM-1557
MATERIAL: Tan silty fine sand

OPTIMUM WATER CONTENT: 11 %
MAX UNIT DRY WEIGHT: 110 LB/CU FT



COMPACTION TEST RESULTS

BORING: H-42 **DEPTH:** 90'

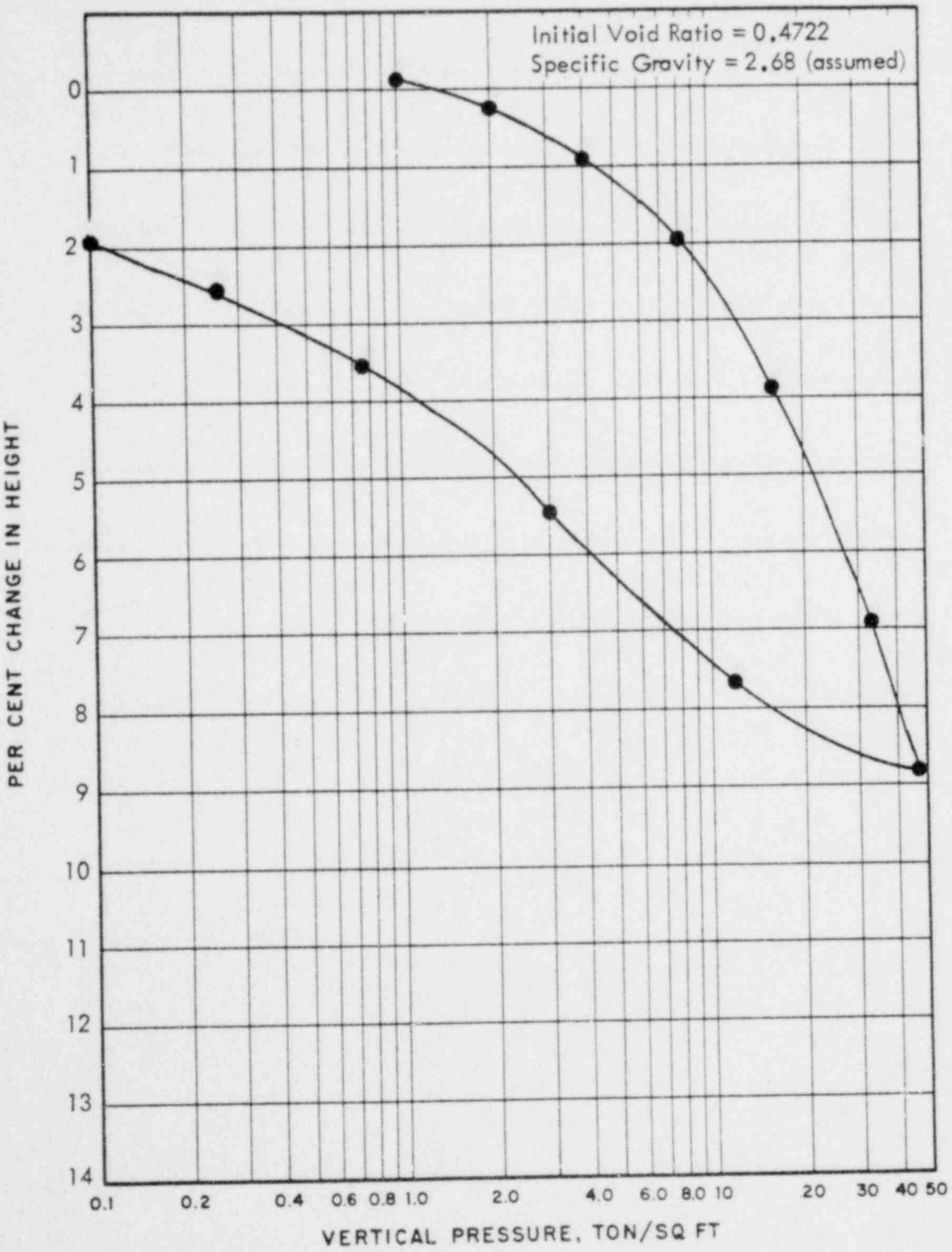
MATERIAL: Very stiff light gray sandy clay

UNIT DRY WEIGHT: 114 LB/CU FT

WATER CONTENT: 17 %

LIQUID LIMIT: 39

PLASTIC LIMIT: 14



Initial Void Ratio = 0.4722

Specific Gravity = 2.68 (assumed)

CONSOLIDATION TEST RESULTS

FIG NO. M26

BORING: H-42A **DEPTH:** 3'

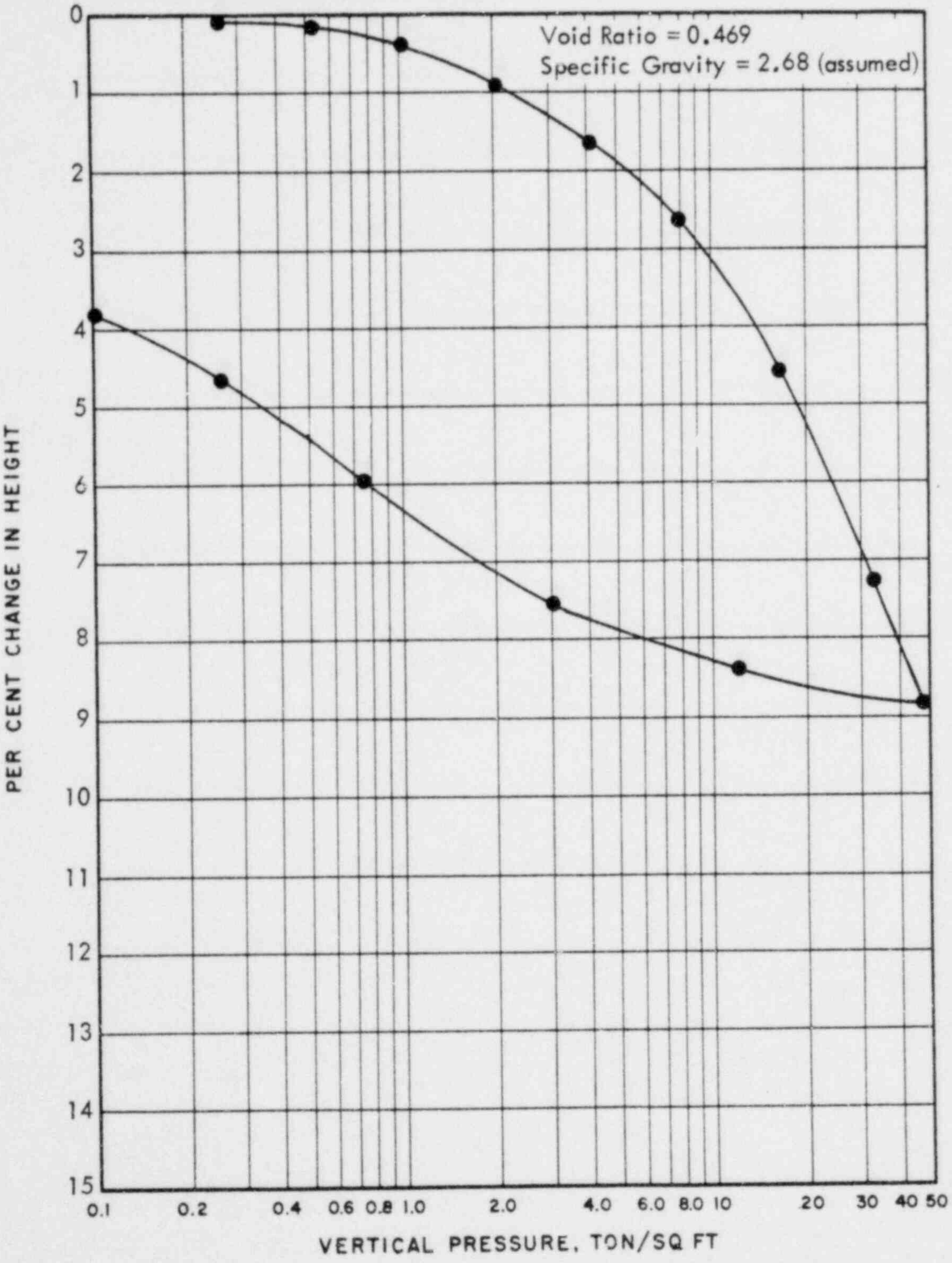
MATERIAL: Very stiff dark gray & brown silty clay

UNIT DRY WEIGHT: 114 LB/CU FT

WATER CONTENT: 16 %

LIQUID LIMIT: 32

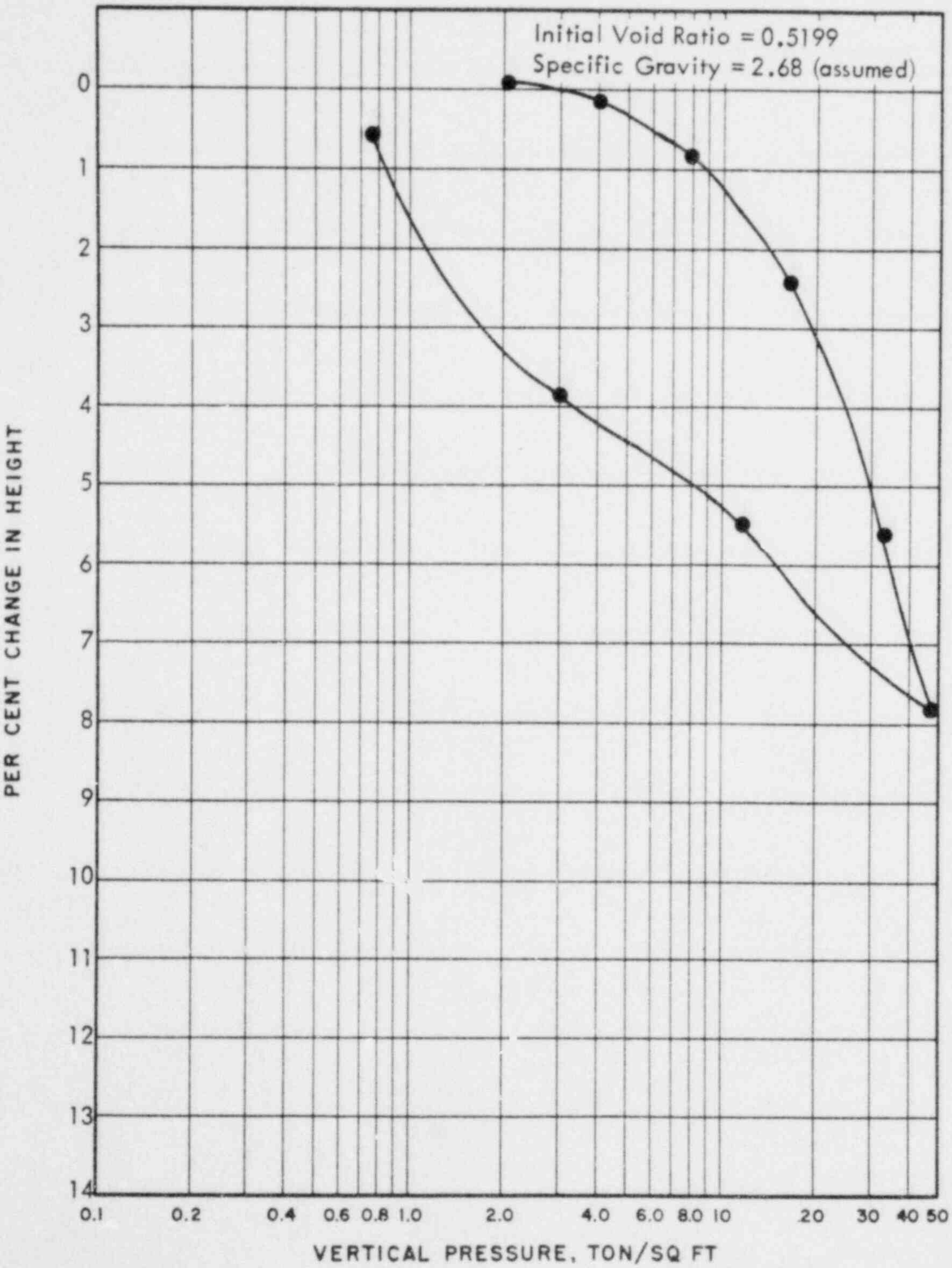
PLASTIC LIMIT: 13



CONSOLIDATION TEST RESULTS

BORING: H-43 DEPTH: 85'
MATERIAL: Very stiff light gray sandy clay

UNIT DRY WEIGHT: 110 LB/CU FT
WATER CONTENT: 18 %
LIQUID LIMIT: 59
PLASTIC LIMIT: 17



CONSOLIDATION TEST RESULTS

BORING: H-43A **DEPTH:** 1.5'

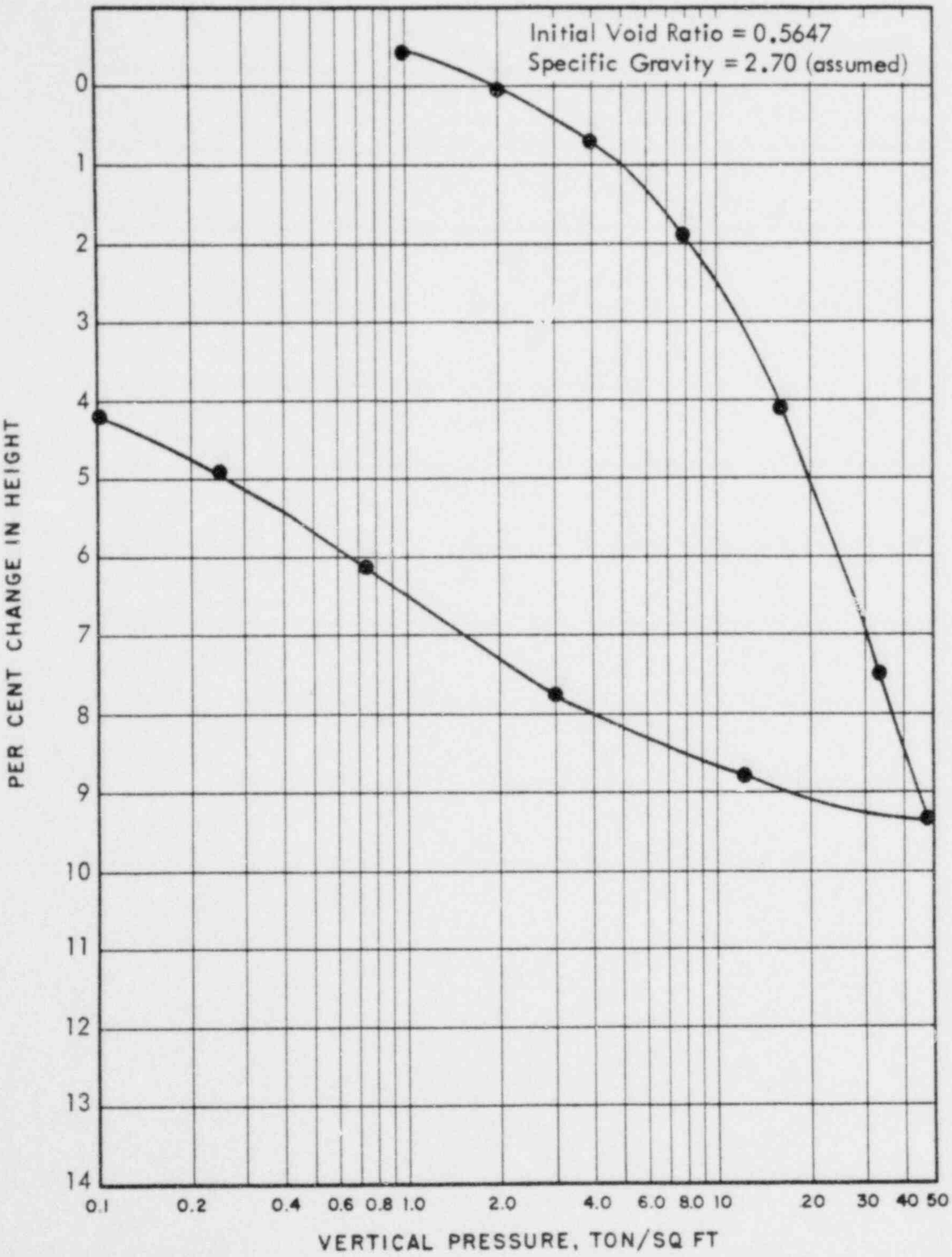
MATERIAL: Very stiff brown silty clay

UNIT DRY WEIGHT: 108 LB/CU FT

WATER CONTENT: 13 %

LIQUID LIMIT: 36

PLASTIC LIMIT: 14



CONSOLIDATION TEST RESULTS

Am. No. 50, 1/15/79

FIG NO. M29

BORING: H-44 **DEPTH:** 25'

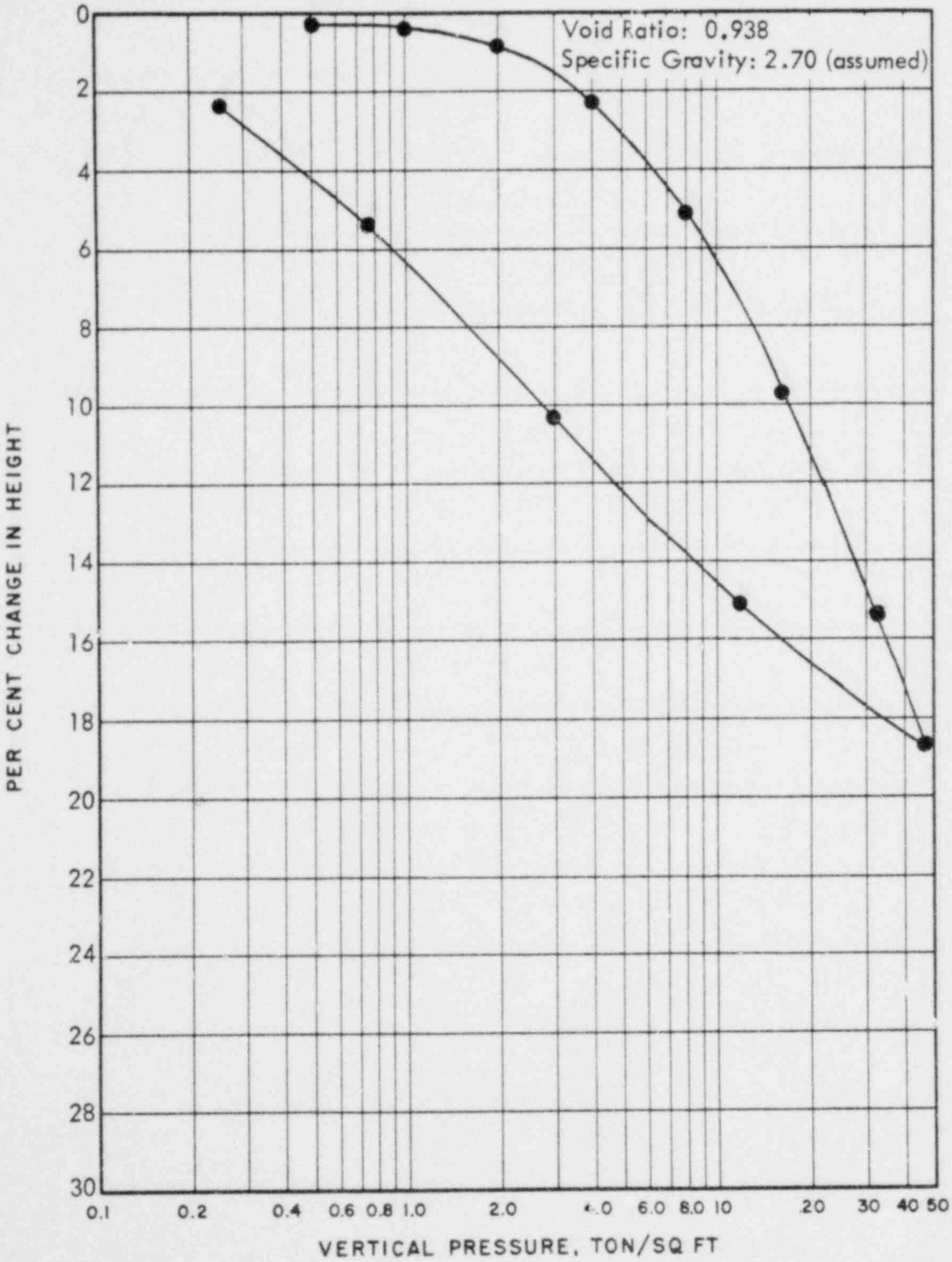
MATERIAL: Very stiff light gray & brown clay with calcareous nodules

UNIT DRY WEIGHT: 87 LB/CU FT

WATER CONTENT: 36 %

LIQUID LIMIT: 70

PLASTIC LIMIT: 22



Void Ratio: 0.938

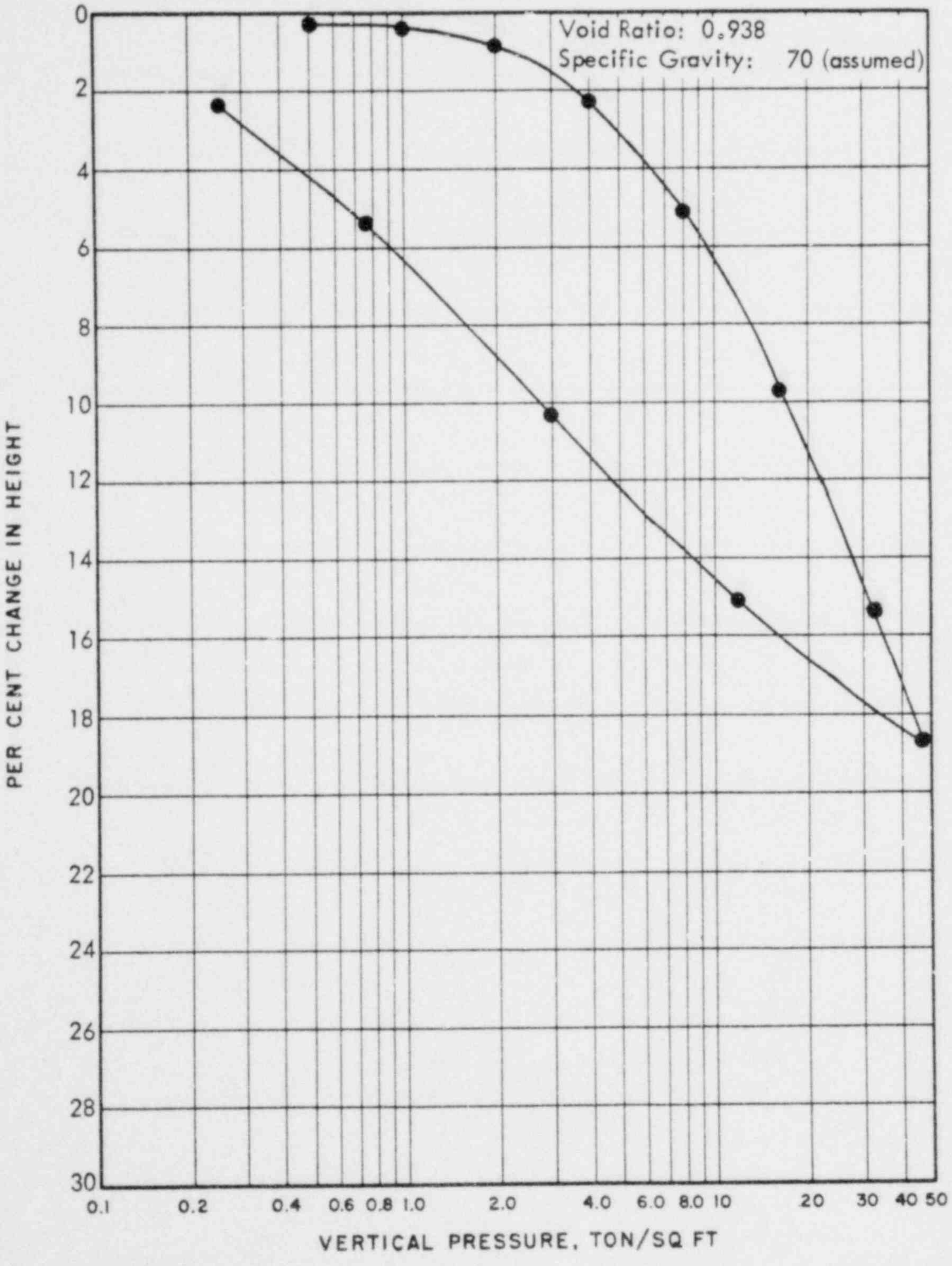
Specific Gravity: 2.70 (assumed)

CONSOLIDATION TEST RESULTS

FIG NO. M30

BORING: H-44 **DEPTH:** 25'
MATERIAL: Very stiff light gray & brown clay with calcareous nodules

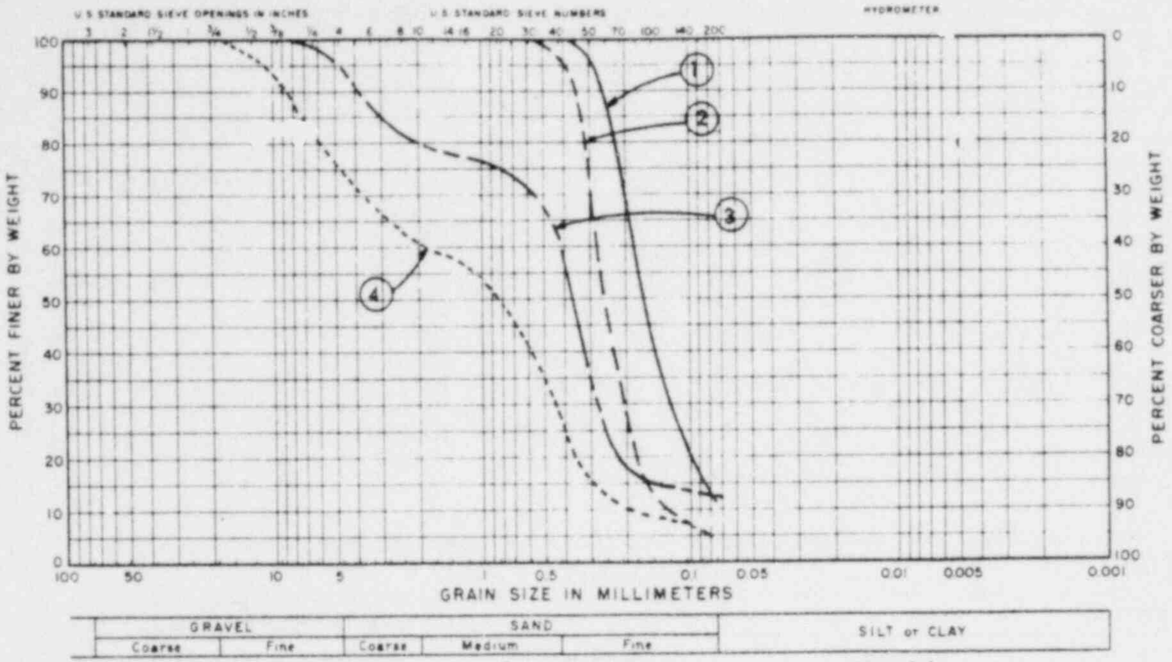
UNIT DRY WEIGHT: 87 LB/CU FT
WATER CONTENT: 36 %
LIQUID LIMIT: 70
PLASTIC LIMIT: 22



Void Ratio: 0.938
Specific Gravity: 70 (assumed)

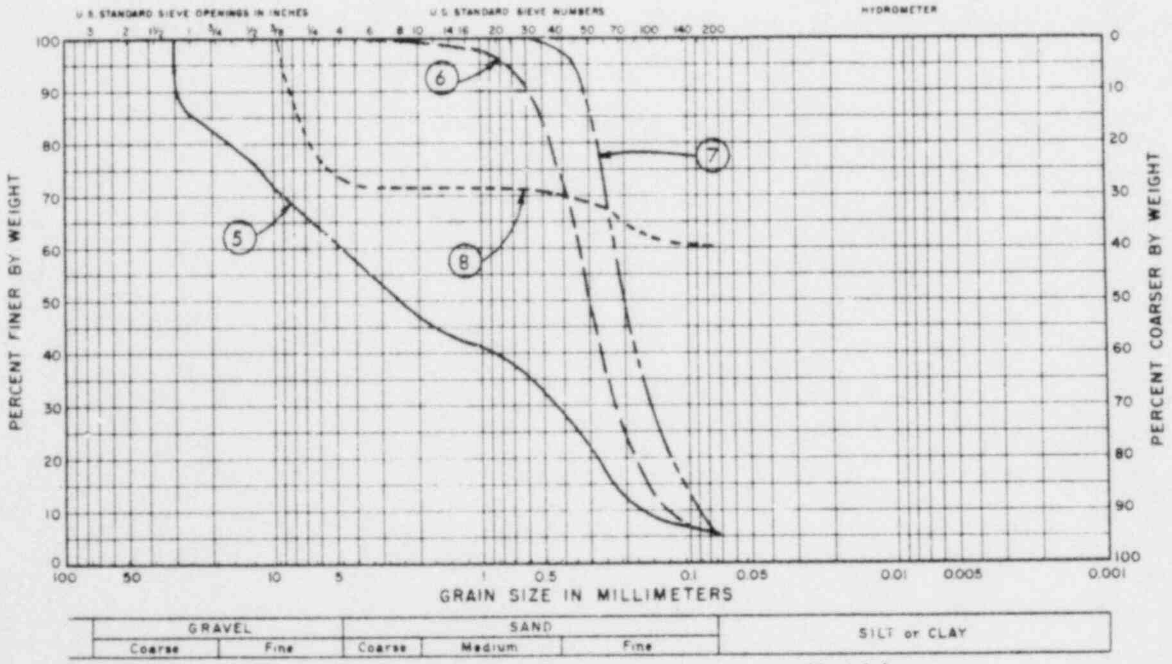
CONSOLIDATION TEST RESULTS

GRAIN SIZE CURVES



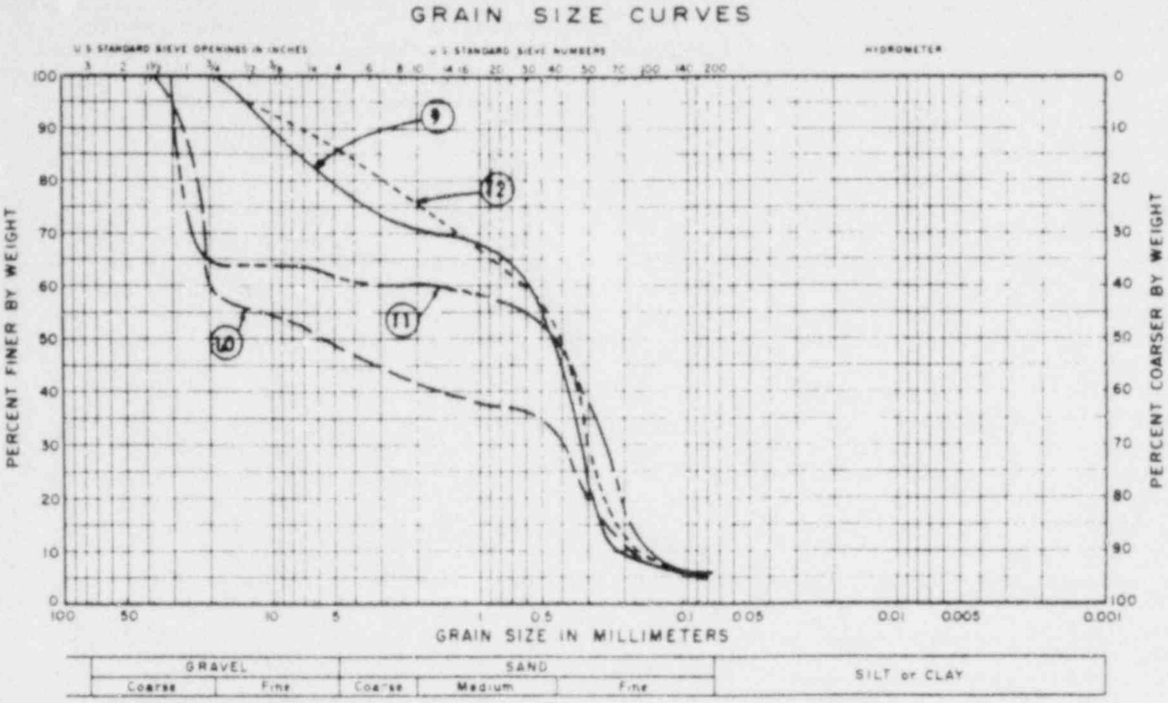
Curve No.	Boring No.	Penetration, Ft	Material
1	H-42	11.5	Tan fine sand
2	H-42	15.5	Tan fine sand
3	H-42	34.5	Tan coarse to fine sand with gravel
4	H-42	39.5	Tan coarse to fine sand with gravel

GRAIN SIZE CURVES

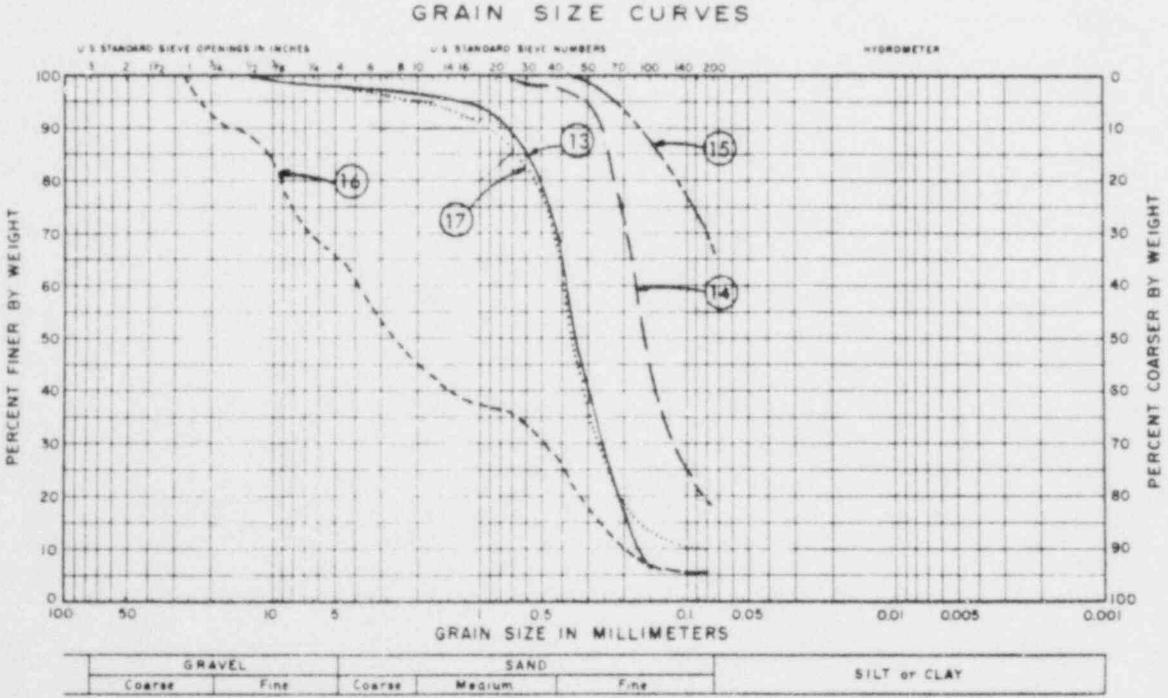


Curve No.	Boring No.	Penetration, Ft	Material
5	H-42	59.0	Tan gravelly coarse to fine sand
6	H-42	79.0	Tan medium to fine sand
7	H-42A	12.0	Tan fine sand
8	H-43	25.0	Red coarse to fine sand with gravel & clay layers

FIG NO. M31



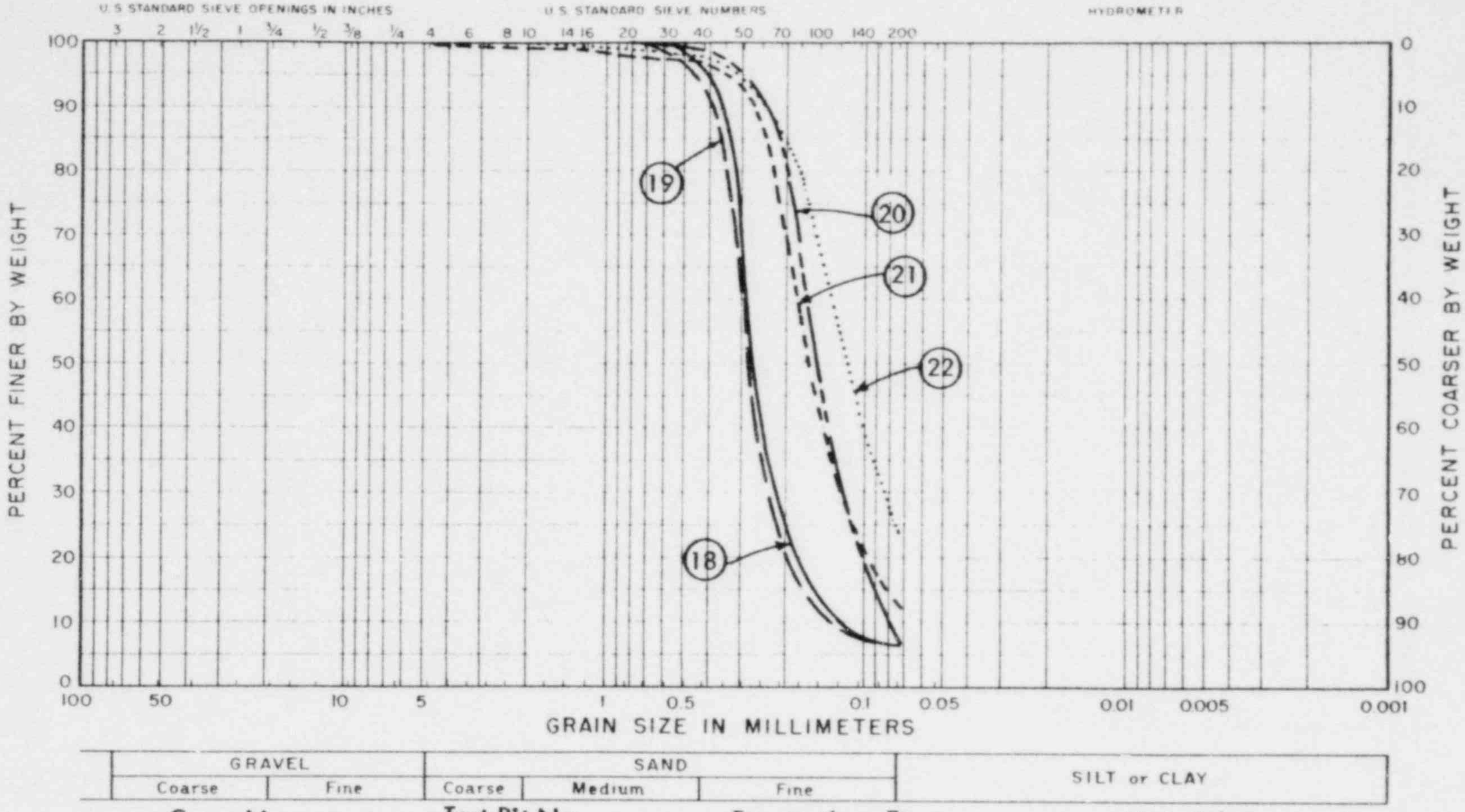
Curve No.	Boring No.	Penetration, Ft	Material
9	H-43	[illegible]	Light gray coarse to fine sand with gravel
10	H-43	[illegible].0	Light gray & tan coarse to fine sand with gravel
11	H-43	69.0	Light gray coarse to fine sand with gravel
12	H-44	54.5	Tan coarse to fine sand with gravel



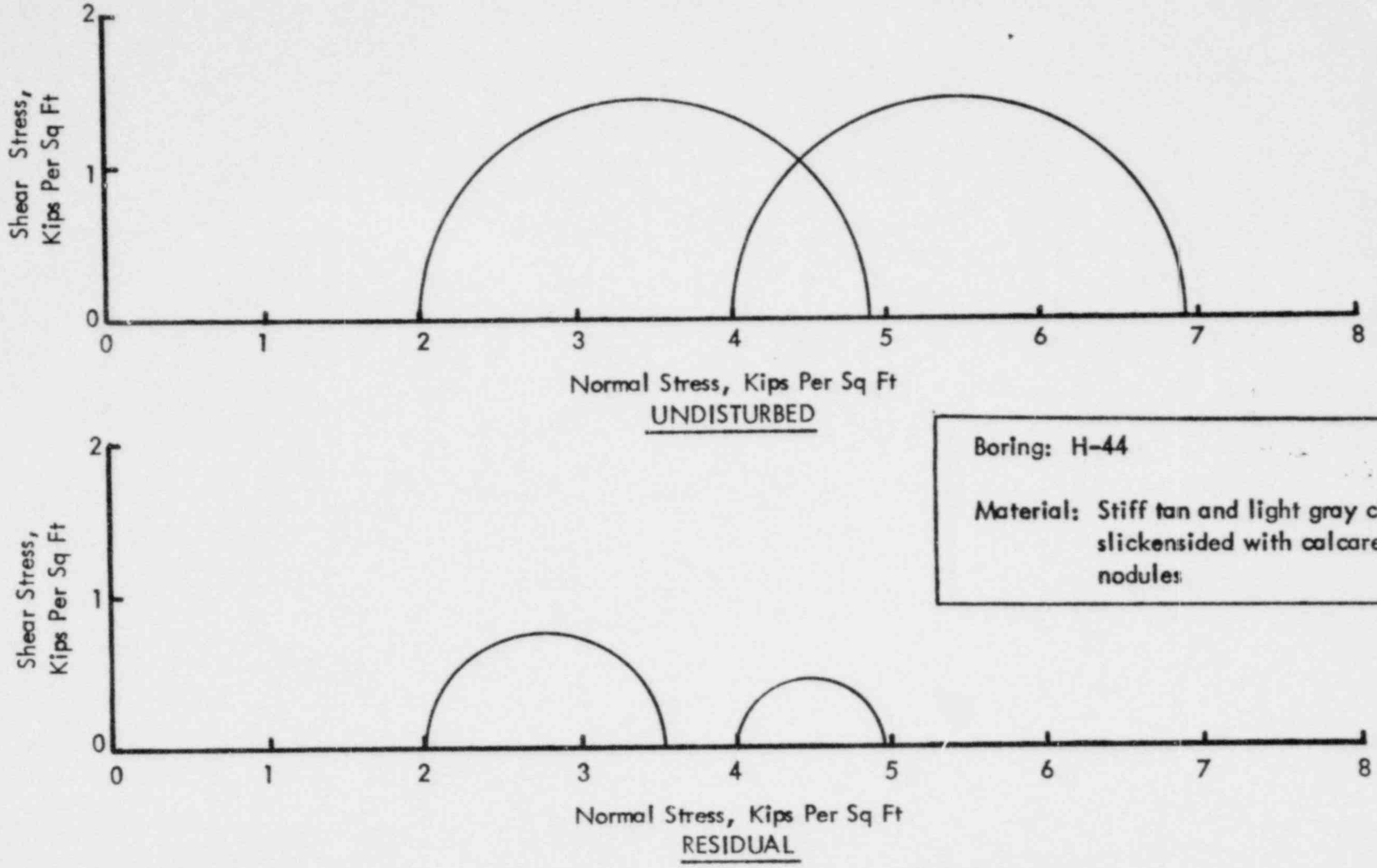
Curve No.	Boring No.	Penetration, Ft	Material
13	H-44	69.0	Tan & light gray medium to fine sand
14	H-44A	4-6	Gray clayey fine sand
15	H-44A	6-8	Light brown clayey sand
16	H-45	49.5	Tan coarse to fine sand with gravel
17	H-45	64.5	Brown coarse to fine sand

FIG NO. M32

GRAIN SIZE CURVES



Curve No.	Test Pit No.	Penetration, Ft	Material
18	6	5	Brown fine sand
19	6	10	Brown fine sand
20	7	5	Brown fine sand
21	8	5	Light brown fine sand
22	9	5	Brown silty fine sand



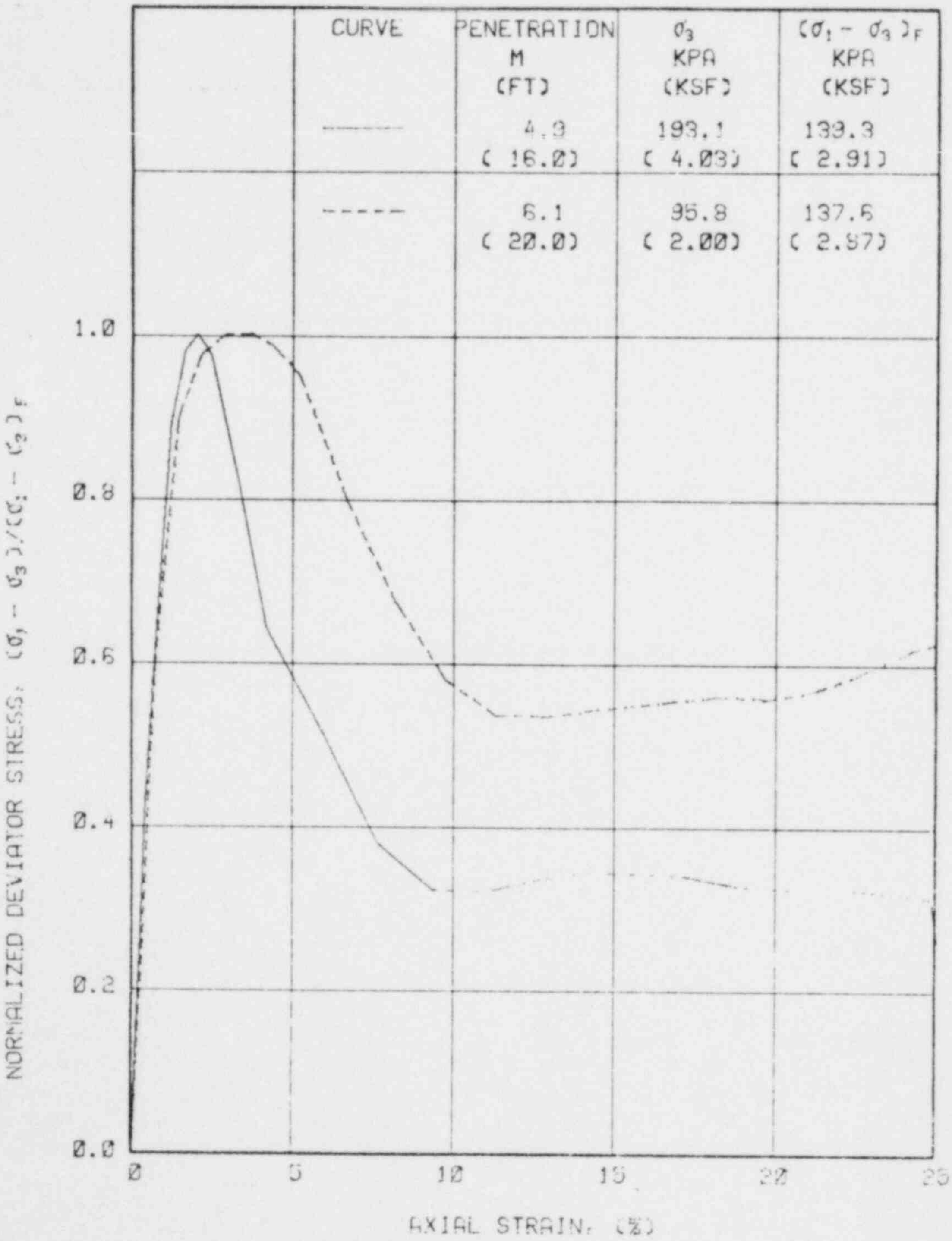
Boring: H-44

Material: Stiff tan and light gray clay, slickensided with calcareous nodules

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

FIG NO. M34

0178-388



UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Stress-Strain Curves

EFFECTIVE PAGES LISTING
CHAPTER 3

DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT AND SYSTEMS

PAGE	AMENDMENT
1*	50
1a*	48
2*	39
3*	39
4*	47
5*	49
6*	48
7*	49
8*	50
9*	46
10*	45
11*	49
12*	48
13*	42
14*	47
15*	48
16*	44
16a*	39
17*	50
18*	48
i	35
ii	35
iii	35
iv	35
v	35
vi	35
vii	35
viii	35
ix	35
x	35
xi	35
xii	35
xiii	37
xiv	35
xv	35
xvi	44
xvii	44
xviii	44
xix	48
xx	35
xxi	44
xxii	35
xxiii	35
xxiv	35
xxv	35

* Effective Pages/Figures Listing

EFFECTIVE PAGES LISTING
CHAPTER 3
DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT AND SYSTEMS

Page	Amendment
3.7-28c	42
3.7-28d	42
3.7-29	35
3.7-30	44
3.7-31	35
3.7-32	35
3.7-32a	35
3.7-32b	35
3.7-32c	44
3.7-32d	44
3.7-33 (deleted)	37
3.7-34 (deleted)	37
3.7-34a (deleted)	37
3.7-35 (deleted)	37
3.7.A-1	50
3.7.A-2	49
3.7.A-3	49
3.7.A-4	48
3.7.A-5	
3.7.A-6	48
3.7.A-7	48
3.7.A-8	48
3.7.A-9	48
3.7.A-10	48
3.7.A-11	48
3.7.A-12	48
3.7.A-13	48
3.7.A-14	48
3.7.A-15	48
3.7.A-16	48
3.7.A-17	48
3.7.A-18	48
3.8-1	35
3.8-2	35
3.8-3	41
3.8-4	35
3.8-4a	35
3.8-4b	35
3.8-4c	35
3.8-4d	35
3.8-4e	35
3.8-4f	35
3.8-4g	39
3.8-4h	35
3.8-4i	35
3.8-4j	35
3.8-5	35

EFFECTIVE FIGURE LIST*
CHAPTER 3
DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT AND SYSTEMS

Figure No.	Amendment No.
3.7-30	35
3.7-31	35
3.7-32	35
3.7A-1	48
3.7A-2	48
3.7A-3	48
3.7A-4	48
3.7A-5	48
3.7A-6	48
3.7A-7	48
3.7A-8	48
3.7A-9	48
3.7A-10	48
3.7A-11	48
3.7A-12	48
3.7A-13	48
3.7A-14	48
3.7A-15	48
3.7A-16	48
3.7A-17	48
3.7A-18	48
3.7A-19	48
3.7A-20	48
3.7A-21	50
3.7A-22	50
3.7A-23	50
3.7A-24	50
3.7A-25	50
3.7A-26	50
3.7A-27	48
3.7A-28	48
3.7A-29	48
3.7A-30	48
3.7A-31	48
3.7A-32	48
3.7A-33	48
3.7A-34	48
3.8-1	5
3.8-2	-
3.8-3	-
3.8-4	26
3.8-4a	30
3.8-4b	30
3.8-4c	30
3.8-4d	30

* All Figures whether labelled "Unit 1" or "Units 1 & 2" are to be considered applicable to Unit No. 1.

APPENDIX 3.7.A
SEISMIC DESIGN CONSIDERATIONS

1.0 INTRODUCTION

This appendix presents an in-depth discussion of the soil structure interaction analysis methodology employed to design Allens Creek NGS - Unit No. 1 for earthquakes. It demonstrates that the analytical methods presented herein result in a conservative treatment of the seismic design of those structures, systems and components important to safety. 48 N130.6

Table 3.7.A-1 provides a summary listing of the various analyses performed.

2.0 INPUT MOTION

2.1 This section has been deleted. 50

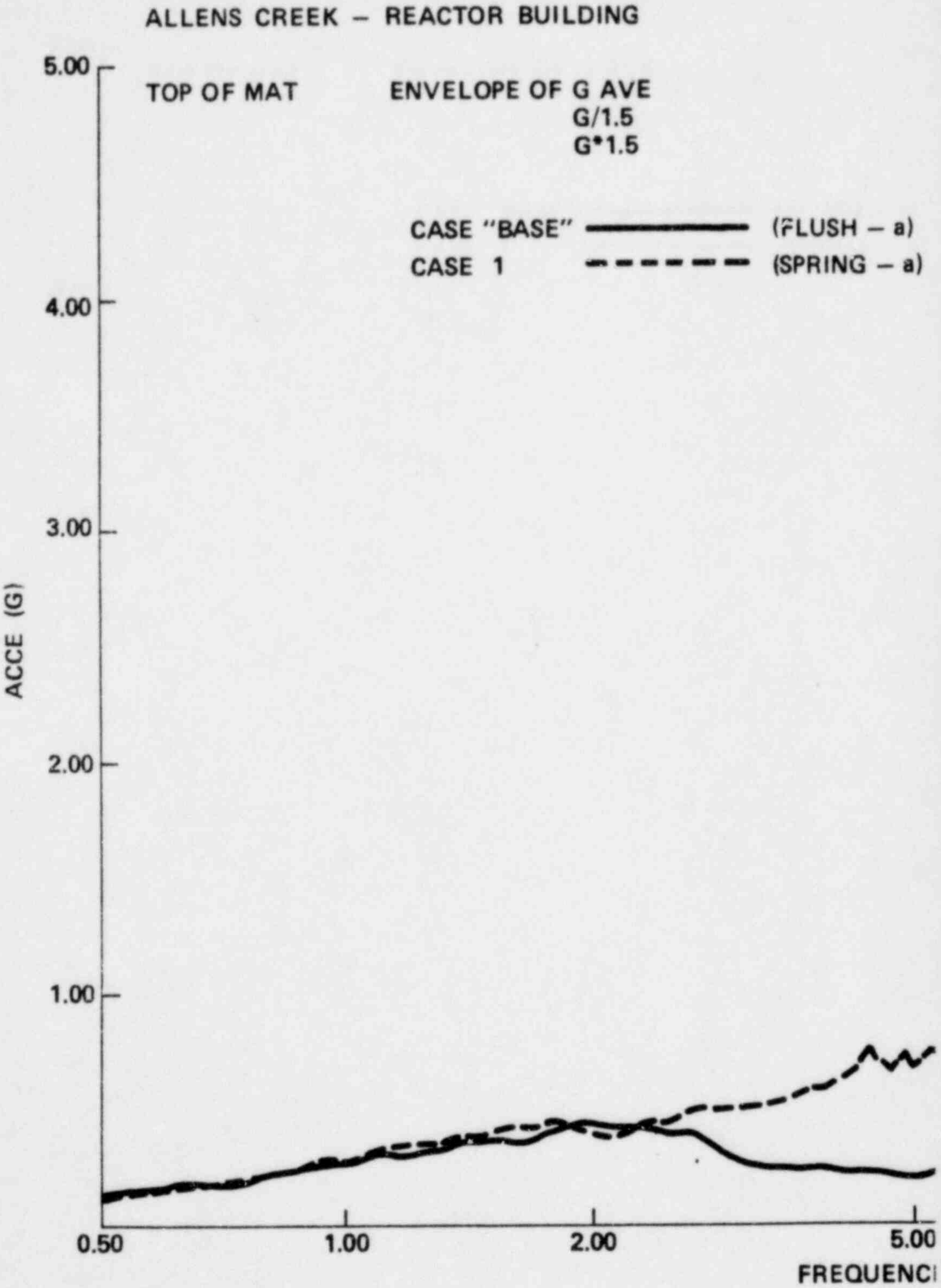
2.2 DESIGN RESPONSE SPECTRA

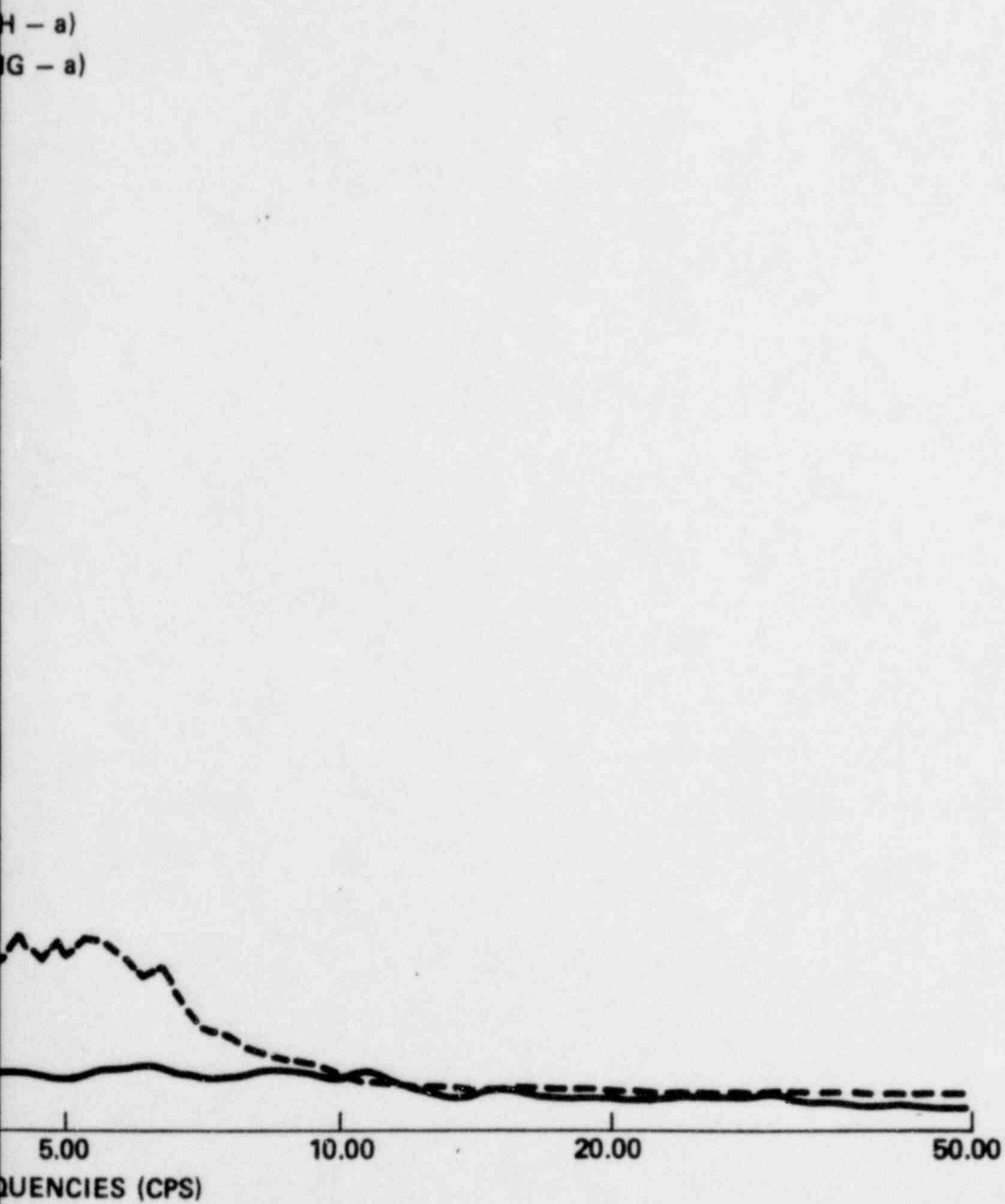
Design response spectra were obtained in accordance with guidelines provided in Regulatory Guide 1.60. The Regulatory Guide 1.60 spectral shape is considered conservative over certain frequency ranges when applied to the deep alluvium deposit Allens Creek site. As such, the use of the Regulatory Guide 1.60 response spectra rather than site-specific response spectra provides additional conservatism for the ACNGS seismic soil-structure interaction analyses. 49

2.3 CONTROL MOTION ELEVATION

For Allens Creek, the design time histories (consistent with Regulatory Guide 1.60 response spectra) will be applied at the foundation level of each Category I structure. 49

During discussions with the NRC, analyses were performed comparing the effect of the location of the control motion with respect to the Reactor Building. A comparison of accelerations obtained at various points (refer to Figure 3.7A-2) indicates a relatively close agreement between results obtained with the control motion defined at the bottom of the Reactor Building mat (FLUSH-b) vs. at the ground surface (FLUSH-a). Maximum accelerations at various points from the above two cases are presented in Table 3.7.A-3 and 48 N130.6

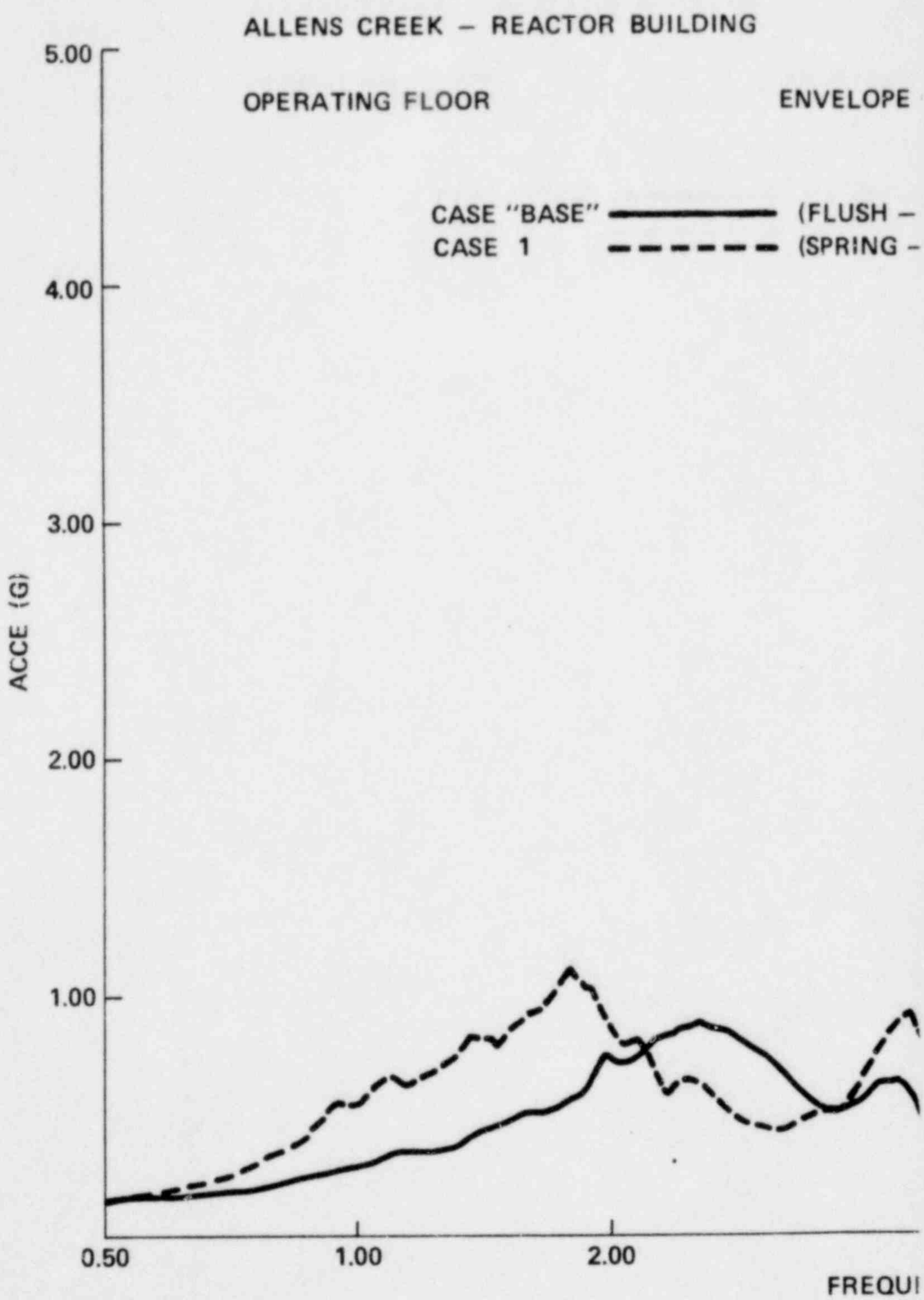

ALLENS CREEK - REACTOR BUILDING
TOP OF MAT
ENVELOPE OF G AVE
G/1.5
G*1.5
CASE "BASE"
(FLUSH - a)
CASE 1
(SPRING - a)
ACCE (G)
5.00
4.00
3.00
2.00
1.00
0.50
1.00
2.00
5.00
FREQUENC

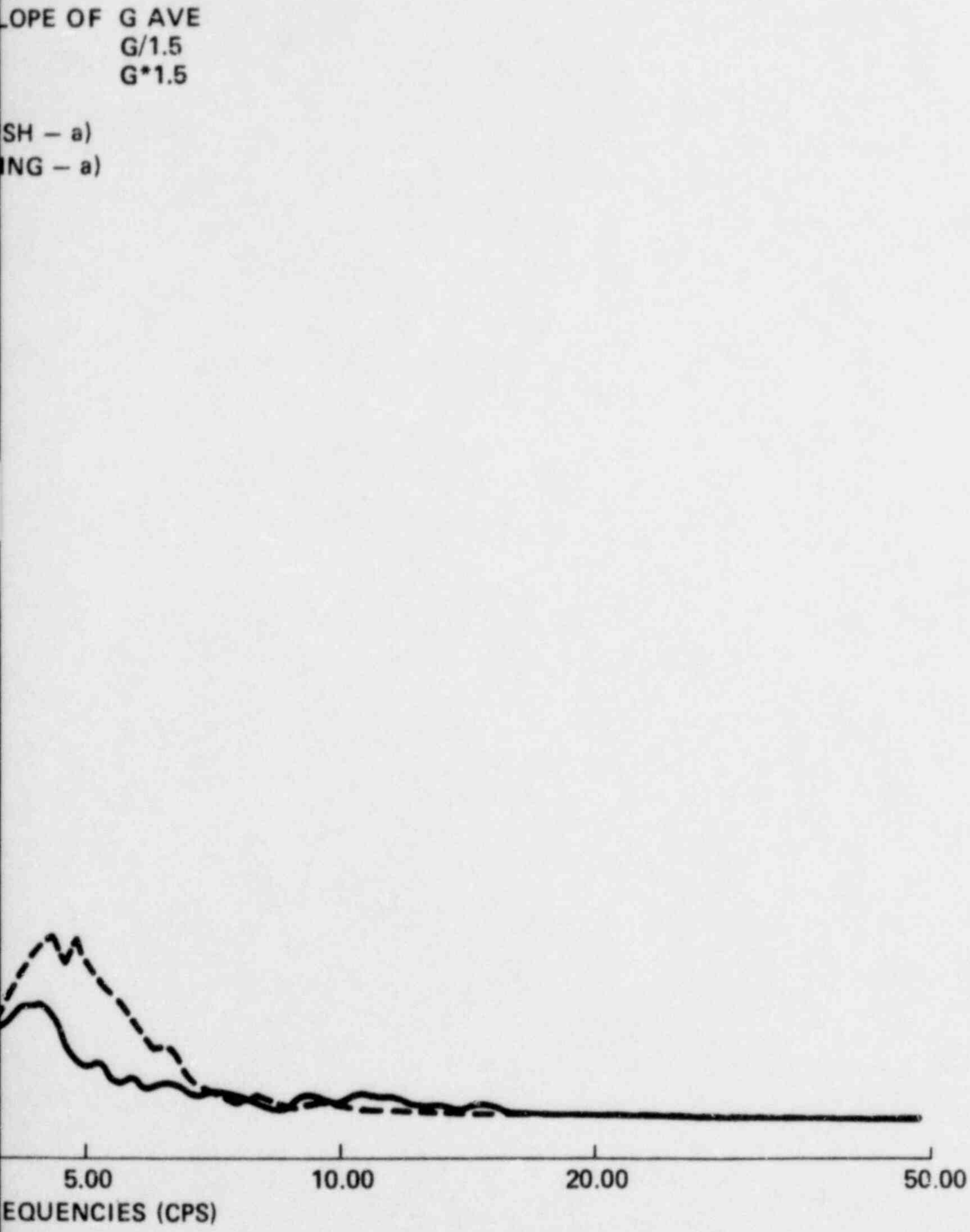


Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Unit 1

SPECTRA COMPARISON
FLUSH-A VS SPRING-A
FIGURE 3.7.A–21


ALLENS CREEK – REACTOR BUILDING
OPERATING FLOOR
ENVELOPE
CASE "BASE" (FLUSH –
CASE 1 (SPRING –
ACCE (G)
5.00
4.00
3.00
2.00
1.00
0.50
1.00
2.00
FREQU

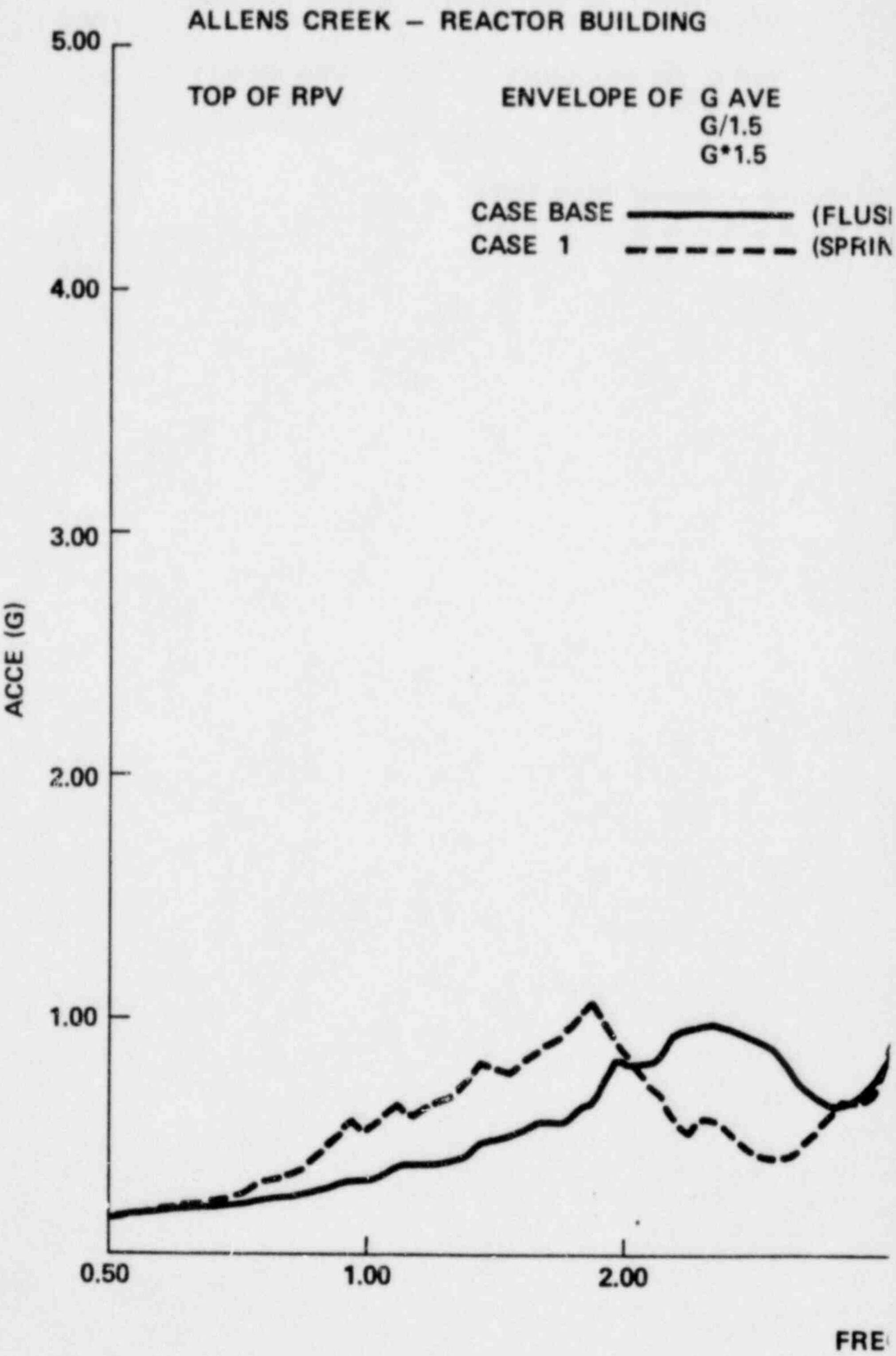


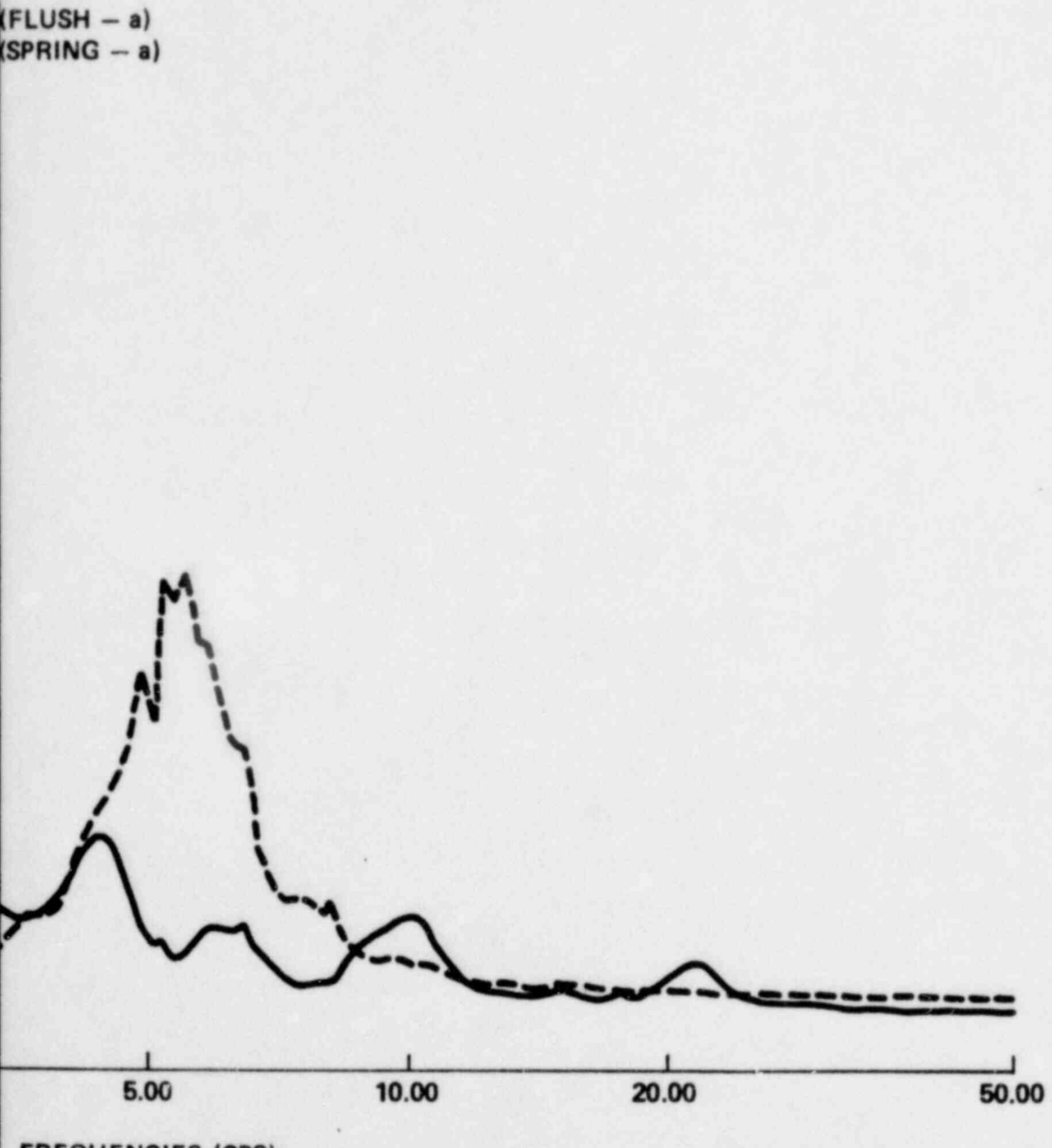
Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Unit 1

SPECTRA COMPARISON
FLUSH-A VS SPRING-A
FIGURE 3.7.A–22

AC


ALLENS CREEK – REACTOR BUILDING
TOP OF RPV
ENVELOPE OF G AVE
G/1.5
G*1.5
CASE BASE
(FLUS
CASE 1
(SPRIN
ACCE (G)
5.00
4.00
3.00
2.00
1.00
0.50
1.00
2.00
FRE

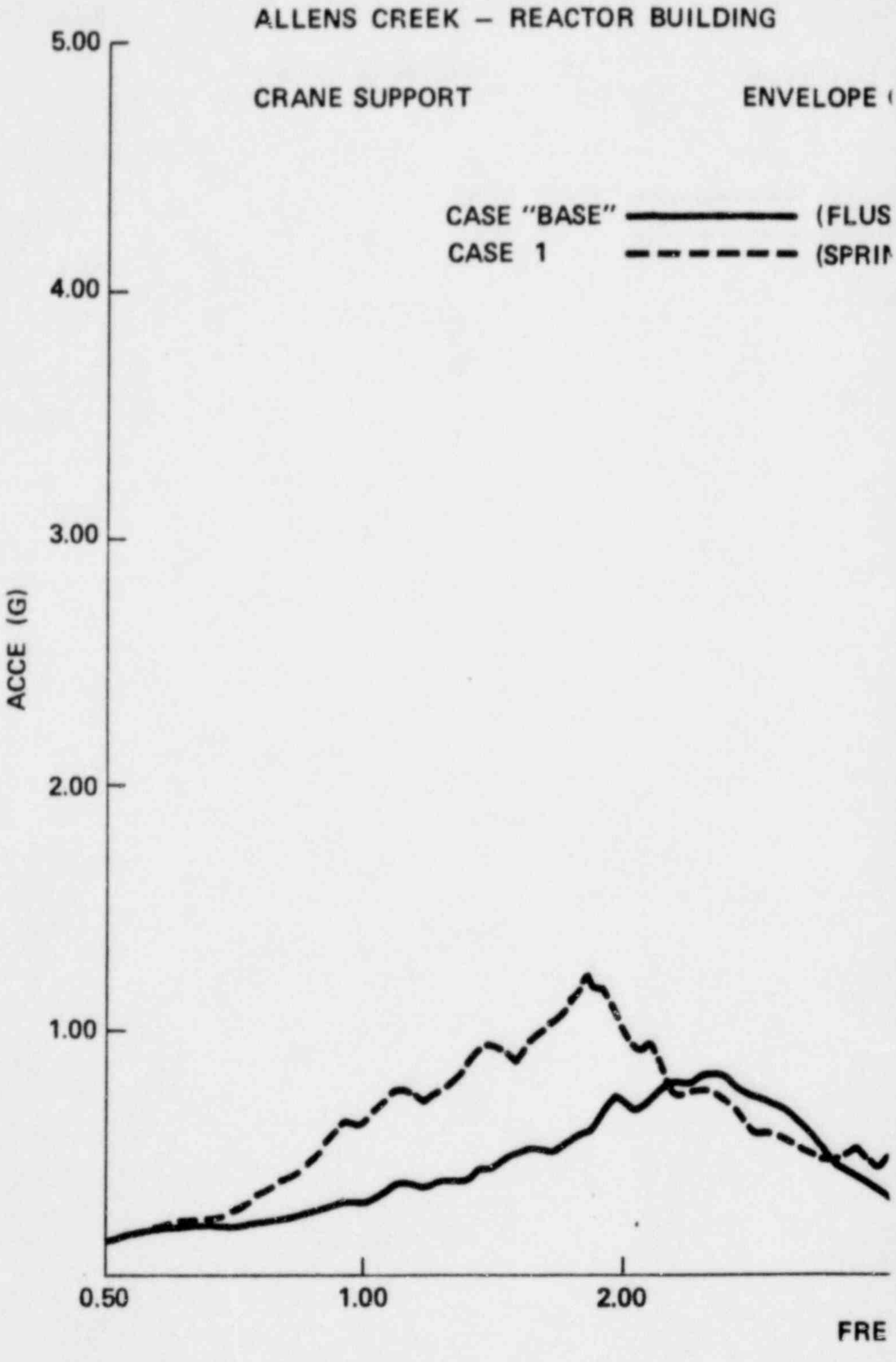


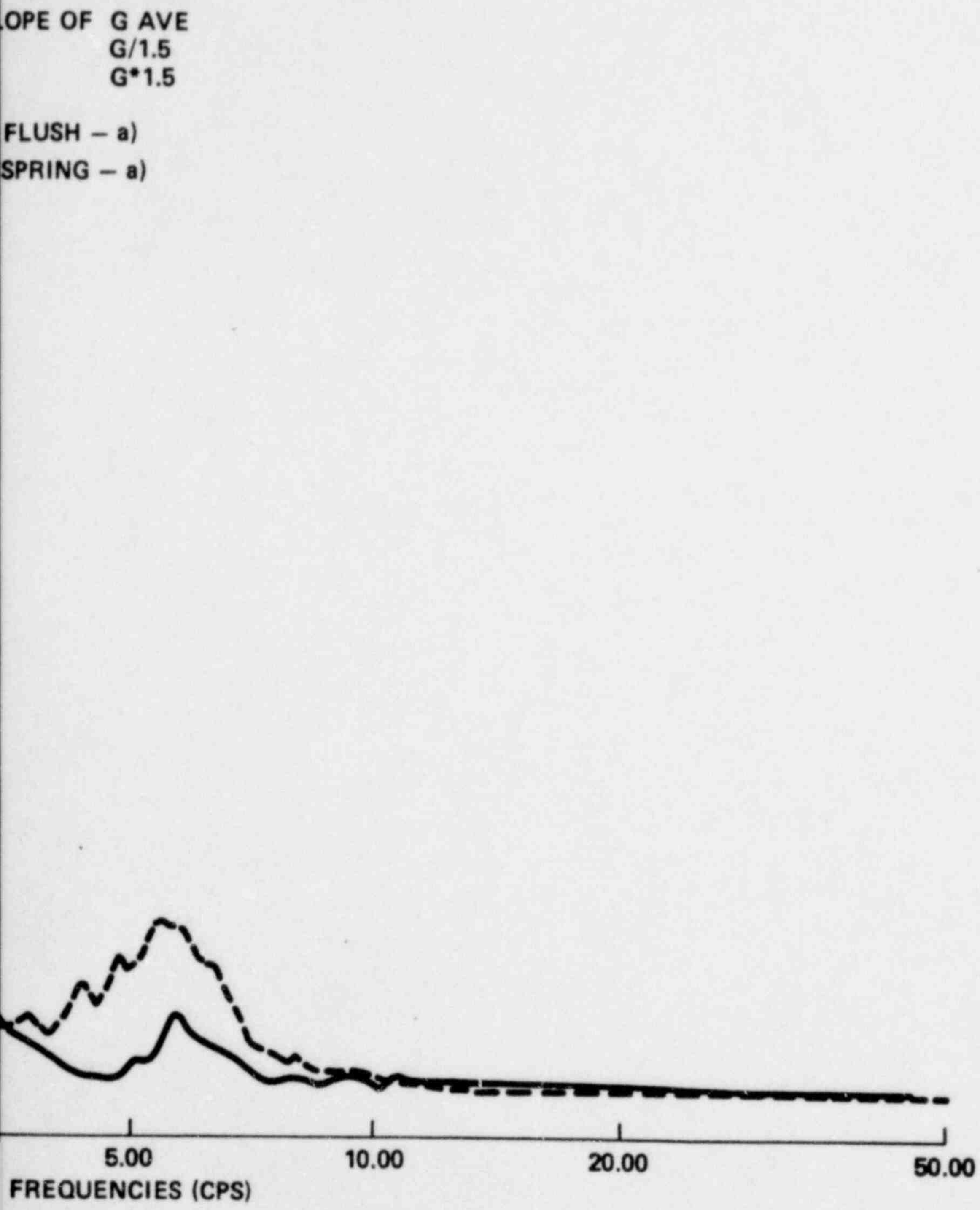
Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Unit 1

SPECTRA COMPARISON
FLUSH-A VS SPRING-A
FIGURE 3.7.A–23

AC


ALLENS CREEK – REACTOR BUILDING
CRANE SUPPORT
ENVELOPE
CASE "BASE" (FLUS
CASE 1 (SPRI
ACCE (G)
5.00
4.00
3.00
2.00
1.00
0.50
1.00
2.00
FRE

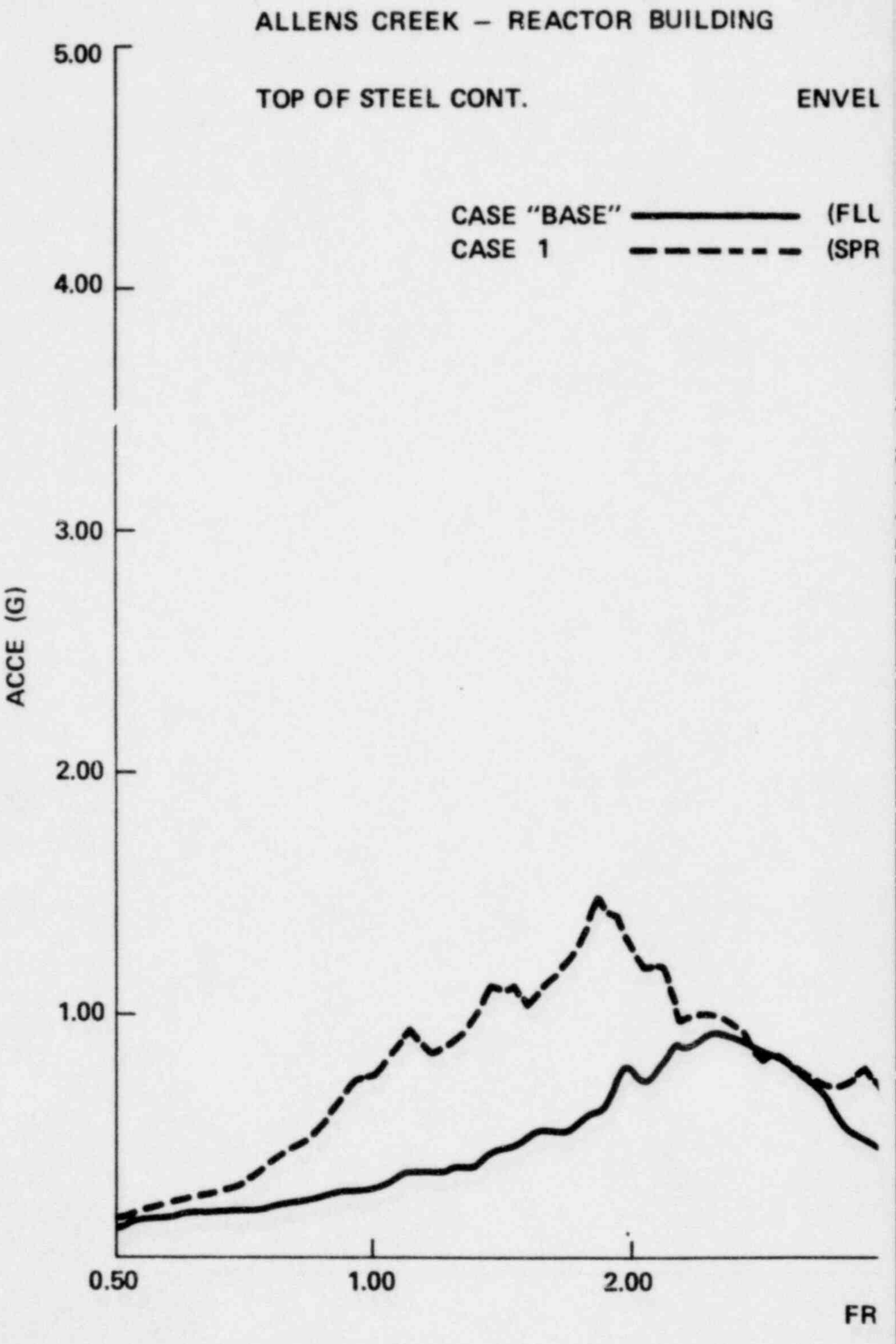


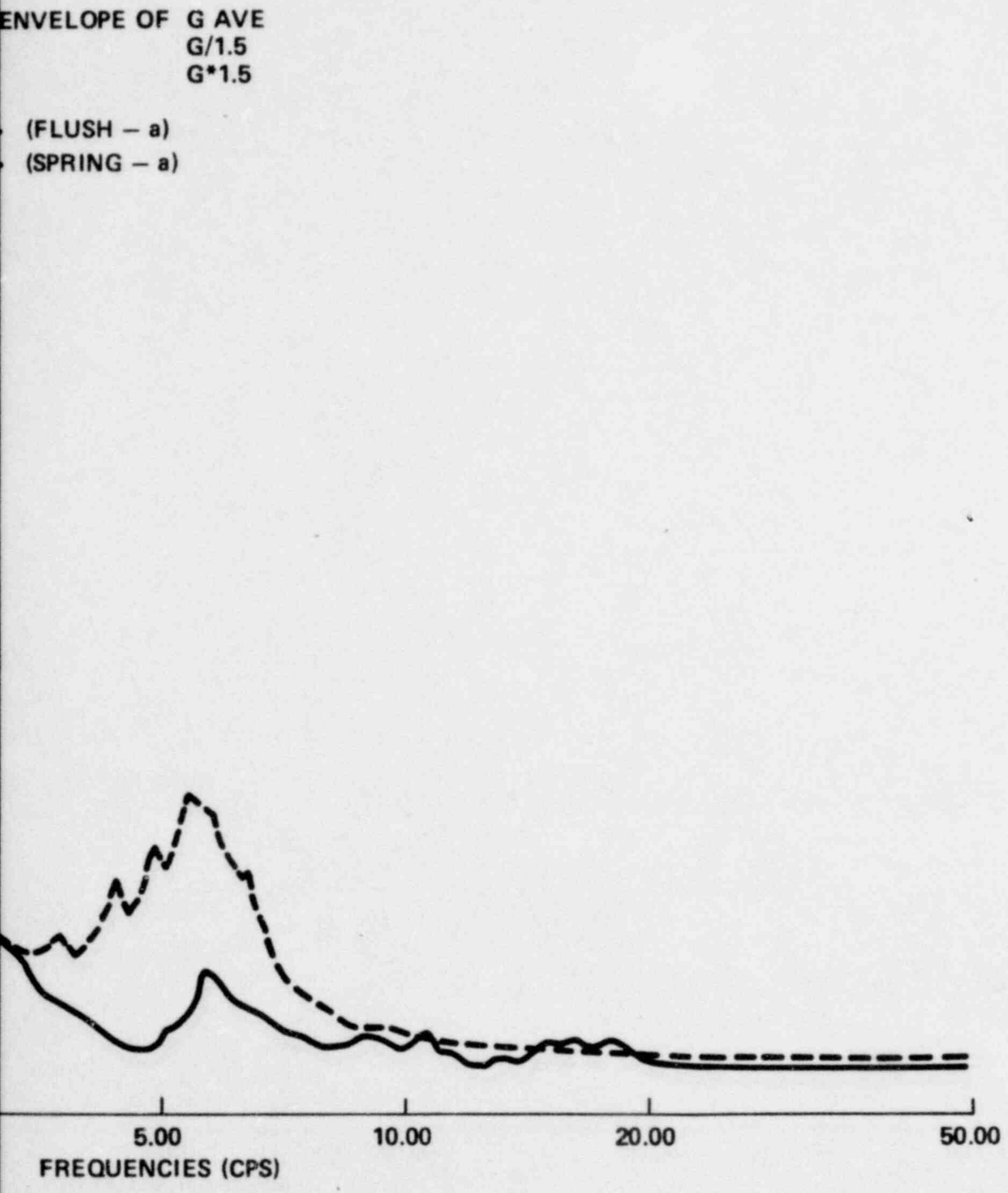
Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Unit 1

SPECTRA COMPARISON
FLUSH-A VS SPRING-A
FIGURE 3.7.A–24

ACN


ALLENS CREEK – REACTOR BUILDING
TOP OF STEEL CONT.
ENVEL
CASE "BASE"
(FLU
CASE 1
(SPR
ACCE (G)
5.00
4.00
3.00
2.00
1.00
0.50
1.00
2.00
FR

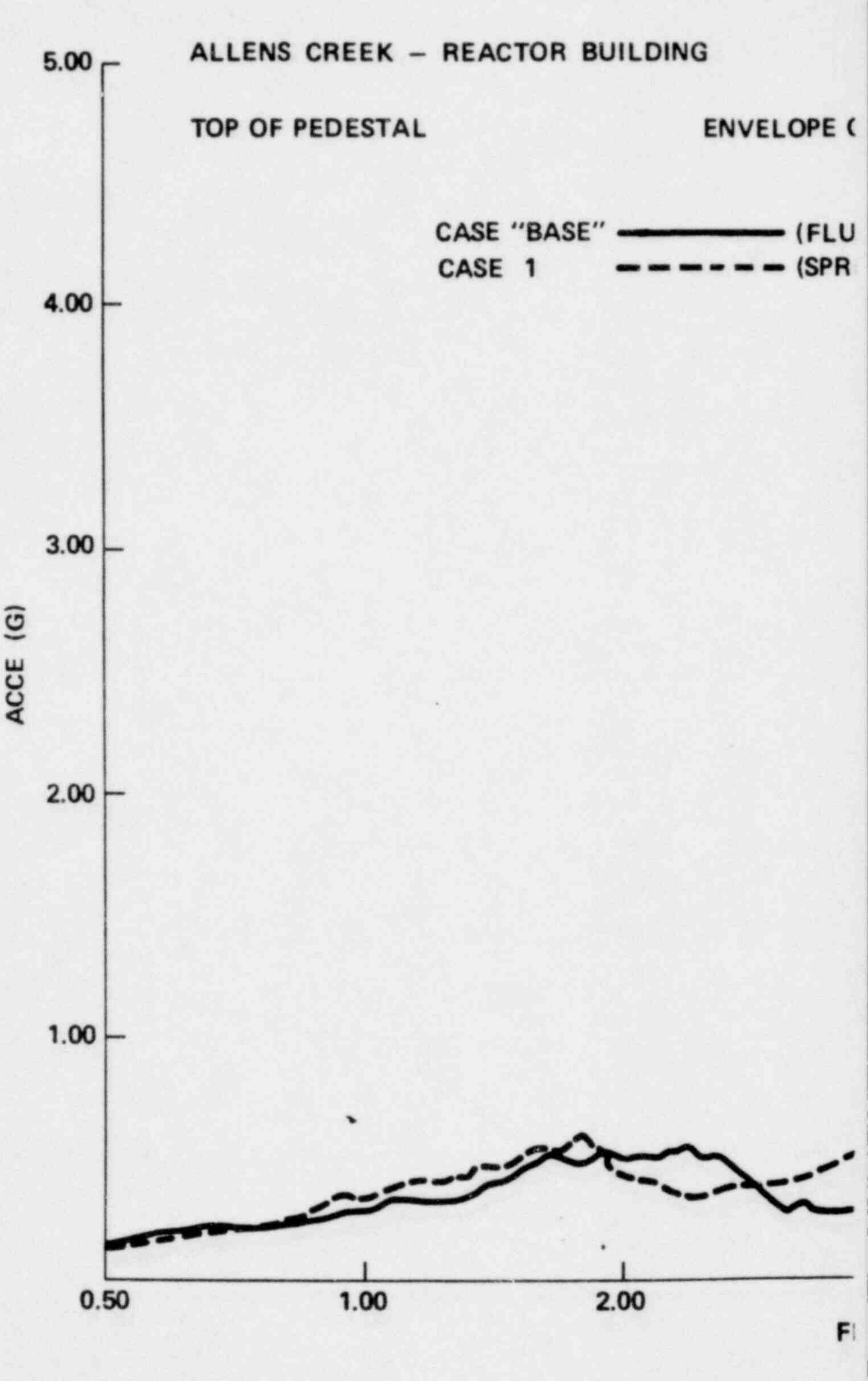


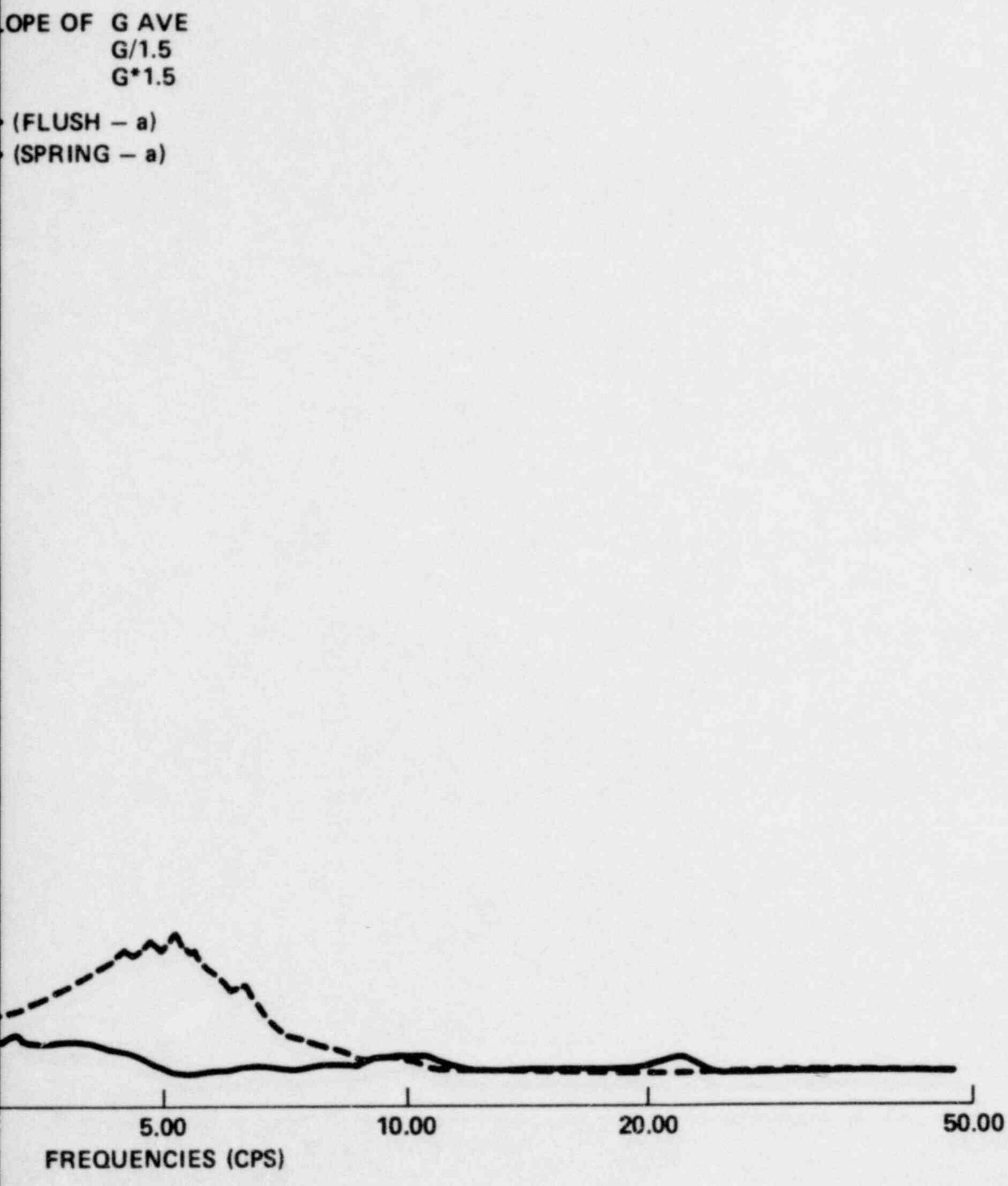
Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Unit 1

SPECTRA COMPARISON
FLUSH-A VS SPRING-A
FIGURE 3.7.A–25

ACI


ALLENS CREEK – REACTOR BUILDING
TOP OF PEDESTAL
ENVELOPE (
CASE "BASE" (FLU
CASE 1 (SPR
ACCE (G)
5.00
4.00
3.00
2.00
1.00
0.50
1.00
2.00
F



Am. No. 50, 1/15/79

HOUSTON LIGHTING & POWER COMPANY
Allens Creek Nuclear Generating Station
Unit 1

SPECTRA COMPARISON
FLUSH-A VS SPRING-A
FIGURE 3.7.A–26

EFFECTIVE PAGES LISTING
CHAPTER 14
INITIAL TESTS AND OPERATION

Page	Amendment
1*	50
2*	33
i	33
ii	33
iii	33
14.1-1	41
14.1-1a	50
14.1-1b	50
14.1-2	50
14.1-3	33
14.1-4	33
14.1-5	33
14-1-6	41
14.1-6a	41
14.1-7	33
14.1-8	33
14.1-9	11
14.1-10	-
14.1-11	33
14.1-12	-
14.1-13	33
14.1-14	33
14.1-15	33
14.1-16	33
14.1-17	33
14.1-18	-
14.1-19	-
14.2-1	-

* Effective Pages/Figures Listings

c) To provide baseline data upon which future normal and safe operations of the plant may be based and to assist in the evaluation of subsequent periodic tests.

In general the initial test program will be developed in accordance with the guidance contained in NRC Regulatory Guide (RG) 1.68, "Preoperational and Initial Startup Test Programs for Water-Cooled Power Reactors." Additional NRC Division I RGs applicable to the development of the initial test program will be used based on the Applicant's position stated in Appendix C of the PSAR. 41 Q 423.1

It is recognized that some insight to certain problem areas may be gained through review of abnormal occurrence reports from operating reactors. This could lead to detection and correction of these problem areas during the initial test program. The abnormal occurrence reports will be screened, categorized and filed by Nuclear Engineering Department personnel, with additional review by Energy Production Department personnel. 41 Q 423.2 Appicable abnormal occurrence reports will be identified to the individuals responsible for writing test procedures so that the reports may be used as input for the initial test program. 50

14.1.1 ADMINISTRATIVE PROCEDURES (TESTING)

HL&P personnel will have overall responsibility for the initial test program. This includes the review and approval of test procedures, the review and approval of system performance, and the documentation of results. 41 Q 423.3

The Plant Superintendent and/or Assistant Superintendent, assisted by the plant supervisory and professional-technical staff and representatives of GE and EBASCO, shall be responsible for the preparation of preoperational and initial startup test procedures. These procedures shall be reviewed by the Plant Nuclear Safety Review Committee (PNSRC), described in Section 16.6.5.1, prior to approval. 11 These test procedures shall be approved and signed by the Plant Superintendent, or his designated alternate, before being implemented. Test results shall be reviewed and certified by the PNSRC. Predesigned forms shall be utilized for test review, data logging, review and approval of test results. These forms shall be retained as part of the permanent plant records. Personnel qualifications of those involved in the above administration procedures shall be furnished by amendment when the organization is fully developed.

Plant operating and emergency procedures will be prepared by plant operating and technical personnel with the assistance of others. The procedures will be tested and revised as required during the onsite training of plant personnel. The procedures will be trial-use-tested to the fullest extent practicable during the initial test program. 41 Q 423.3

A description of the methods that will be used during the preoperational testing and initial operation period to demonstrate the adequacy and feasibility of the normal and emergency operating procedures shall be included in the FSAR. Draft versions of the normal and emergency operating procedures will be available during the initial test program. This will give the operating staff an opportunity to familiarize themselves with the procedures, compare them to the test procedures, and modify them if necessary to assure their completeness. Proper training for the safe and dependable normal and emergency operation of the various plant systems and subsystems is described in Section 13.2.1, "Plant Staff Training Program." 41 Q 423 .3

The qualifications of individuals performing key functions in preoperational and startup testing programs will be as follows:

Minimum qualifications of individuals that direct or supervise the conduct of individual Preoperational Tests. (At the time that the individual is assigned to the task).

a) A bachelor's degree in engineering or the physical sciences or the equivalent and one year of applicable power plant experience. Included in the one year of experience should be at least three months of indoctrination/training in nuclear power plant systems and component operation of a nuclear power plant that is substantially similar in design to the type at which the individual will perform the function or

b) A high school diploma or the equivalent and four years of power plant experience. Credit for up to two years of this four year experience may be given for related technical training on a one-for-one time basis. Included in the four years of experience should be at least three months of indoctrination/training in nuclear power plant systems and component operation of a nuclear power plant that is substantially similar in design to the type at which the individual will be employed.

Minimum qualifications of individuals that direct or supervise the conduct of individual startup tests. (At the time of assignment to the task).

a) A bachelor's degree in engineering or the physical sciences or the equivalent and two years of applicable power plant experience of which at least one year shall be applicable nuclear power plant experience or,

b) A high school diploma or the equivalent and five years of applicable power plant experience of which at least two years shall be applicable nuclear power plant experience. Credit for up to two years of non-nuclear experience may be given for related technical training on a one-for-one time basis. 50

Minimum qualifications of individuals assigned to groups responsible for review and approval of Preoperational and Startup Test Procedures and/or review and approval of test results. (At the time the activity is being performed).

a) Eight years of applicable nuclear power plant experience with a minimum of two years of applicable nuclear power plant experience. A maximum of four years of the non-nuclear experience may be fulfilled by satisfactory completion of academic training at the college level.

An appropriate number of qualified engineers (approximately twenty) will be on hand to carry out the test program consistent with the test schedule and the requirements for personnel for each test.

50

14.1.2 ADMINISTRATIVE PROCEDURES (MODIFICATIONS)

The Plant Superintendent will be immediately informed of any proposed system modification and/or changes in procedures resulting from test results. Modifications following plant operations shall maintain the same level of quality assurance as it would have received if installed originally. All of the applicable quality requirements, modified to the extent necessary to suit the modification, shall be utilized. 11

The following sections describe the manner in which a modification shall be implemented.

14.1.2.1 Identification and Notification of Required Modifications

Modifications to the as-built characteristics of components, systems and structures may be required for several reasons:

a) Operational performance of the item or system does not satisfy required conditions or criteria, e.g., insufficient pump head, improper valve closure time, etc.

b) Changes to federal regulations or industrial codes and standards require backfitting or upgrading of equipment, e.g., issuance of Regulatory Guides, addenda to codes, etc.

c) Recommendations from the Allens Creek Architect-Engineer, NSSS Supplier or equipment vendors based on additional testing, inspections, analyses, or operating data

d) Modification required in the switchyard due to additional generating capacity at the station, electrical auxiliary systems or additional transmission lines from the station or changes in the arrangement of the switchyard to improve the reliability of the switchyard

The Plant Superintendent shall be notified immediately when such possible modifications are identified. The primary sources of notifications will be the plant operating staff and the HL&P home office.

14.1.2.2 Administration of Modification Activities

Three distinct phases will exist for the administration of modification activities. Specifically they will consist of:

a) Determination of necessity for the modification

b) Development of the procedures to be employed to perform the modification

Open Item No.	
361.4	Following its review of responses to Item 361.4 that you provided in Amendments 42 and 44, the Corps of Engineers in its letter of June 23, 1978 (copy attached) provided additional comments about compaction requirements for Class I-a Fill and the effect of slickensided surfaces on the design shear strength. Provide the clarification outlined by the Corps of Engineers in its letter of June 23, 1978.

RESPONSE

Class 1-a Fill material will be compacted to a minimum relative density of 80 percent. The field control will be determined using the Modified Proctor test. 48 Q 361.4

The referenced Corps of Engineers' letter comments that Class I slope stability analyses be performed using residual shear strength parameters. The Applicant has performed an effective stress stability analysis on Class I slopes using residual strengths for natural clays, remolded strengths for compacted clays and a safety factor of 1.25. The results of this analysis as discussed in Appendix 2.5M in Chapter 2. 50 Q 361.4

ASSUMPTIONS

1) A maximum of two LPCI pumps (specifically LPCI "A" and LPCI "B") can be fully diverted at ten minutes to the containment spray mode. (NOTE: LPCI "A" shares an emergency diesel generator with the LPCS; LPCI "B" and "C" share an emergency diesel generator. The pump associated with LPCI "C" cannot be diverted to containment sprays.)

2) The standard SAR assumption of one automatic depressurization system (ADS) valve failure combined with the worst additional single failure was retained because this assumption is built into the present model. In addition, failure to account for this ADS valve failure would result in limitations on the operation of Allens Creek plant which could affect plant availability. This bounding assumption yields conservatively higher calculated peak cladding temperatures (PCTs) by approximately 100°F. | 50

3) Approved Appendix K analysis models were used, except that some LPCI flow to the reactor vessel was stopped ten minutes after the accident.

RESPONSE TO SPECIFIC NRC CONCERNS

Only those accident cases which are not reflooded to the hot node before ten minutes are affected by the assumed LPCI diversion. Once the core has been reflooded, only one ECCS pump is necessary to keep the core covered. Thus, the breaks affected include small breaks less than approximately .02 ft^2 (depending on the break location) and outside steam line breaks (OSLB). The effect of the assumed LPCI diversion on the OSLB is small and is discussed in a later section of this report.

The following break locations were considered: A) core spray line, B) recirculation line, C) feedwater line, D) the steam line, and E) LPCI line. A brief summary of each analysis is provided below.

Open Item No.

361.5 In Section 9.2.5.3.2 of the PSAR you state "In the event that the rate of sediment accumulation is such that it appears that the allowable level of accumulation will be exceeded during the life of the plant, the sediment will be removed before that allowable level is reached." In addition to level of sediment accumulation, limits on slope of the surface of the accumulated sediment should be considered to assure that unacceptable consequences will not result from flow into pump intake during design basis events. State the allowable configurations for accumulated sediments within the cooling lake and provide a preliminary description of the technical specifications that will be used to assure maintenance of acceptable sediment configurations. Include criteria, procedures, and technical specifications for maintaining sediment configurations.

RESPONSE

The applicant will periodically inspect the UHS to determine if unacceptable sediment buildup is occurring. Both depth of sedimentation and slope will be measured to determine buildup. The allowable limits and the method chosen for monitoring slope and depth of sediment will be presented to the NRC after issuance of the Construction Permit but prior to the initiation of Construction of the Ultimate Heat Sink. 50