2E SOIL & FOUNDATIONS INVESTIGATION REPORT

APPENDIX 2E

SOIL AND FOUNDATIONS
INVESTIGATION REPORT

1.0 INTRODUCTION

The purpose of this soil and foundation investigation program was to establish the suitability of the site and to provide the basic criteria for design of a nuclear power plant for the Sacramento Municipal Utility District. This report describes the drilling and sampling, laboratory testing and analysis performed in the evaluation of the engineering properties of the soil and foundation at the Rancho Seco site. The data developed from the program also provided the basic information necessary to satisfy the requirements of the Preliminary Safety Analysis Report (PSAR).

The drilling and sampling program began June 28, 1967 and was concluded August 25, 1967. Preceding the drilling and sampling program, a geologic reconnaissance and mapping program was performed by Bechtel geologists in consultation with Roger Rhoades, consultant geologist to Bechtel Corp.

Geophysical logging techniques were employed in the one deep geologic hole at the site. These techniques provided useful information not only for obtaining a continuous geophysical log of materials with depth between sampling intervals, but also as an indication of changes of materials, density and firmness with depth. Refraction seismograph traverses were also run in the general area of the proposed site with a portable seismic device. An interpretation has been made between the velocities obtained and the densities or changes in interpreted properties of materials with depth. The locations of the seismic profile lines and the details and interpretation of the seismic survey as described in Appendix 2C, Geology and Seismology.

The entire investigation program was under the supervision of Bechtel Corporation and the drilling was carried out under a subcontract with Boyles Bros., Auburn, California, and Myren Drilling, Sacramento, California. Selected soil samples were tested by Soil Mechanics and Foundation Engineers, Inc., Palo Alto, California soils laboratory and supplemented by classification and other testing performed by SMUD'S soils laboratory facility near Placerville, California.

2.0 SUMMARY AND CONCLUSIONS

This report summarizes the soil and foundation investigation program and laboratory analysis performed to provide the basic data required to establish the suitability of the Rancho Seco Nuclear Generating Plant site.

The site investigation was performed concurrently with the geologic and geophysical investigations. The drilling and sampling was carried out under subcontract with Boyles Bros. Drilling Company, Auburn, California and Myren Drilling Company, Sacramento, California under the supervision

347

of Bechtel Corporation. The laboratory testing was performed by Soil Mechanics and Foundation Engineering Inc., Palo Alto, California supplemented by testing at SMUD'S facilities near Placerville, California. All laboratory testing was performed under the supervision and direction of Bechtel Corporation.

The results of the drilling and sampling and laboratory testing provided the basic technical data from which the foundation and engineering properties of the soil were analyzed.

The following conclusions were developed from the results of the soil and foundation investigation program:

- The soils are sufficiently strong to safely support the nuclear containment structure, turbine and appurtenent facilities.

4 - The soils at the site can be categorized as dense-to-very-dense sandy silts, sandy clays and very dense sand with some gravel.

- Ground water was found at approximately 143 feet below the existing ground surface which is sufficiently deep to not adversely affect the soil or foundation at the site.

- The reactor containment structure, on a circular mat approximately
4 35 feet below the finished grade of the plant, has an allowable soil pressure of 9,000 pounds per square foot based on settlement criteria. Total settlements are anticipated as approximately 4 to 5 inches with differential settlement of less than 1.5 inches on the bottom of the mat. All settlements will be elastic and comprised of rebound and recompression.

- The turbine foundation will be a mat located approximately 10 feet below the plant finished grade. The allowable soil pressure beneath the turbine is 5,000 pounds per square foot based on an estimated total settlement of approximately 1.5 to 2 inches and an estimated differential settlement of less than 1 inch on the foundation mat.

- The remainder of the turbine and miscellaneous structures can be supported on spread footings. All footings should be at least 4 feet wide and should be at least 5 feet below finished grade. Post construction settlements will be minimal.

- The auxiliary building can be supported on a mat at its present
4 location of approximately 20 feet below finished grade. Total settlements are anticipated to be 1 to 1.5 inches with differential settlements of less than 0.5 inch on the mat.

4 - The spent fuel storage structure will be supported on a mat approximately 6 feet below finished grade. Post construction settlement will be minimal.

348

- Roads and paving can be designed for conventional flexible pavements. Based on an estimated California bearing ratio (CBR) value of 12 or less, a combined thickness of pavement and base of 10 inches is recommended. The upper few feet of soil will be removed and the subgrade soils should be appropriately prepared and compacted. The surface courses should be designed to be compatible with the anticipated design wheel loads.

- The use of conventional construction equipment is anticipated during the site excavation and preparation. However, materials encountered at the site have been classified as hard-to-very-hard and some use of non-conventional equipment may be necessary.

- Erosion control measures should be incorporated in the site development. Appropriate slope protection and site grading should be considered to inhibit erosion.

3.0 SITE DESCRIPTION

The proposed plant site is located in Section 29 of Township 6 North, Range 8 East, approximately 25 miles southeast of Sacramento, California. The tract of land purchased by the Sacramento Utility District for this project comprises approximately 2,000 acres.

The terrain is open, rolling, grass-covered hills with the maximum difference in elevation of approximately 200 feet. The main drainage pattern trends southwest.

Groundwater was not encountered in any of the soil borings, however it was located in the deep geologic hole, hole number 23, at 143 feet below ground surface.

4.0 DESCRIPTION OF BORINGS

Based on the boring logs, the foundation soils at Rancho Seco are predominantly dense-to-very-dense sandy silts and sandy clays. The upper 3 to 10 feet of each boring consisted of sandy gravel with some silt. Below this zone, a consistent increase in firmness, hardness, and density with depth is evident from the boring logs. An indication of the relative increase in strength is shown in the tabulated unconfined compression testing results 4
in Table 2 of Appendix 2C. No loose soils were encountered in any of the borings. A light unit weight material, encountered in the upper portions of some of the borings, has been identified as a porous silt, composed of some glassy volcanic fine sand intermixed with wind-blown materials, all alluvially transported.

Drawing C-119-E shows the locations of the soil borings, the geologic test trenches and bucket auger holes. The logs of materials encountered in the soil borings have been summarized on drawings C-120-E, 121-E, and 122-E.

4 The test boring closest to the final location of the major plant structures is boring 104. In this boring, silty gravel was encountered to a depth of approximately 10 feet, underlain by poorly graded sand to a depth of approximately 20 feet. Below 20 feet, silts and fine sandy silts were encountered to a depth of approximately 105 feet. At 105 feet, poorly graded fine sand was encountered to termination of the boring at 115 feet.

5.0 INVESTIGATION PROGRAM

The drilling and sampling was carried out with the following equipment.

a. Joy model 22 drill mounted on a Kenworth model 6-282 truck

b. Chicago pneumatic model CP-8 drill mounted on a Ford model 600 truck

c. Portadrill mounted on an International R-190 series truck

d. Bucket auger model 36 and 45 earth drill

e. Backhoe

The soil sampling was carried out with a pitcher sampler, bucket auger and standard penetrometer split spoon sampler. Drilling mud was not used in the soil sampling program.

The pitcher sampler is 3-1/2 inches in diameter, 36 inches in length, and is capable of retaining a sample 2-7/8 inches in diameter in an inner brass or steel Shelby tube. The inner tube of the sampler advances in front of the drilling bit affixed to the outer tube, and the sampler is pushed rather than rotated into the soil. The relatively "undisturbed" samples obtained in the Shelby tubes were sealed top and bottom and transmitted to the soils laboratory for testing.

The bucket auger simply excavates the soil in a rotary motion and provides "disturbed" samples. Samples obtained from bucket auger holes were placed in sample bags and transmitted to the soils laboratory for testing. The bucket auger provided holes of a large diameter in which field geologists descended and obtained a visual log sequence of materials encountered.

The standard penetrometer is 2 inches in outside diameter, 36 inches long, and consists of two semi-circular longitudinal sections secured by the penetrating bit on the lower end and the threaded drill sub on the upper end. Samples obtained in the standard penetrometer spoon were not tested in the laboratory but retained for visual classification.

350

The penetrometer is driven 18 inches into the soil by a 140-pound free-falling hammer dropping 30-inches on a collar on the upper end of the drill rods. The resistance to penetration of the standard sampler spoon was recorded for each 6-inch interval the sampler was driven. The number of blows required to drive the last 12-inches is known as the standard penetration (n). Standard penetration exceeding 50 blows is considered to be refusal for this investigation program.

Numerous backhoe trenches were excavated throughout the site. Field geologists logged these backhoe trenches in the same manner as the large diameter bucket auger holes in addition to obtaining disturbed bag samples for laboratory testing and evaluation.

6.0 LABORATORY TESTING

Laboratory testing for the Rancho Seco Nuclear Power Generating Station was performed by Soil Mechanics & Foundation Engineers, Inc., Palo Alto, California and was supplemented by classification and identification testing performed at the owner's Pollock Pines facilities near Placerville, California. All laboratory testing was under the supervision of Bechtel Corporation, in consultation with the soil testing firm. All undisturbed samples obtained from the drilling and sampling program were transmitted to the laboratory and carefully handled to minimize sample disturbance. Upon receipt at the laboratory, soil samples were appropriately stored to minimize moisture and chemical changes within the samples.

Testing was performed on samples representative of the soils encountered in the site investigation program. The testing program consisted of classification and identification, and static strength testing. Standardized testing procedures were utilized throughout the program. The results of these tests provided a quantitative definition of the engineering properties of the soil and foundation characteristics at the Rancho Seco site as well as the basic data required for the design of the facility.

Details of the specific test procedures, techniques and tabulation of test
results are found in reports by Soil Mechanics and Foundation Engineers, 4
Inc. dated Sept. 15, 1967 and Nov. 17, 1967 which are appended to this
report.

7.0 BASIS OF ANALYSIS AND EVALUATION

This evaluation of the soil and foundation of the Rancho Seco site is
based on the combined results of the field investigation, laboratory testing, and a semi-empirical approach which has been evolved from the observed
performance of structures correlated with standard penetration testing. 351

The standard penetration blow counts were used in the conventional manner described in "Soil Mechanics in Engineering Practice," by Terzghi and Peck. The standard penetration is an approximate measure of the relative density of sands and the compressive strength of clays, both of which are related to bearing capacity.

Also, settlement computations were performed analyzing the foundation deflection over a uniform area with a uniform applied pressure. The loads resulting in the maximum tolerable settlements were considered to be the allowable soil pressures for that particular structure.

8.0 DISCUSSION AND RECOMMENDATIONS

- 4 The current plant configuration has established the plant finished grade at elevation 165 requiring the excavation of 10 to 20 feet of material at the site. The foundation for the reactor building has tentatively been established at approximately 35 feet below the plant finished grade.
- The turbine generator foundation will be located on a mat at approximately 10 feet below finished grade of the plant.
- Miscellaneous buildings such as the shop warehouse and administration building can be carried on spread footings just below the finished grade of the plant.
- The relatively lightly-loaded cooling towers can probably be supported on spread footings and common excavation is anticipated for the canal.
- Roads and parking areas will be designed in a conventional manner. A 10-inch minimum thickness of base and paving will be required. Prior to placing the base course, the foundation soils should be appropriately prepared to obtain the required compaction.
- Site grading can probably be performed with conventional heavy construction equipment.

8.1 REACTOR FOUNDATION

4 The foundation for the reactor structure has been established at approximately 35 feet below the grade surface. The soil encountered at this depth in boring 104 is classified as a dense silt. Below this depth in boring 23, the deep geologic hole, the material becomes more dense and indurated and is geologically classified as a siltstone.

4 The ultimate strength of the dense silt beneath the relatively wide foundation of the containment structure is quite high, based on laboratory shear strength. In cases such as this, the load which will result in the maximum tolerable settlement is considered to be the allowable soil bearing pressure.

There will be immediate elastic settlements comprised of rebound upon excavation and recompression upon application of the total loads. A circular mat is proposed for the nuclear containment structure. Based on settlement analysis of the structure, an allowable bearing pressure of 9000 pounds per square foot is recommended.

352

The settlement of the structure is dependent upon the soils immediately below the foundation. The dense silt on which the containment structure is located is continuous to approximately 103 feet, below which a dense 4 fine sand was encountered. Total settlement should be approximately 4 to 5 inches with a differential settlement of less than 1.5 inches based on elastic methods of analysis.

8.2 TURBINE FOUNDATION

The turbine foundation configuration is a mat located approximately 10 feet below the finished grade of the plant. The soil encountered at this depth, in boring number 104, is classified as a dense sandy silt and extends 4 some 85 feet below this foundation. All settlements are anticipated to be elastic and comprised of rebound and recompression and post construction settlements should be minimal. The allowable soil pressure on the soils beneath the turbine foundation will be on the order of 5000 pounds per square foot based on an estimated total settlement of 1.5 to 2 inches and an estimated differential settlement of less than 1 inch based on elastic methods of analysis.

8.3 TURBINE STRUCTURE

Turbine structure foundations other than the turbine pedestal can be spread footings approximately 10 feet below the plant finished grade. The soil on which this foundation will be supported is a dense sandy silt. 4 Based on the present configuration and settlement criteria, the allowable soil bearing pressure is 10,000 pounds per square foot on a minimum footing of 16 square feet.

8.4 SPENT FUEL STORAGE AND AUXILIARY BUILDING

The spent fuel storage structure will be supported on a mat approximately 22 feet below the finished plant grade. The foundation will be supported on a dense sandy silt. Based on an estimated total settlement of 1 to 1.5 4 inches, with differential settlements of less than 0.5 inch, the allowable soil pressure will be on the order of 5,000 pounds per square foot. All settlements are anticipated to be elastic and comprised of rebound and recompression. Post construction settlements should be minimal.

8.5 COOLING TOWERS AND INTAKE STRUCTURE

The upper soils at the site have characteristics that indicate swelling and consolidation properties. Accordingly, preliminary recommendations are that approximately the upper 10 feet of original material should be removed and replaced with mechanically-compacted select material when the cooling tower foundations are less than 10 feet below original grade.

The circulating water intake structure is located approximately 21 feet below the proposed plant finished grade. The intake structure is founded on a hard silty clay. Based on maximum tolerable settlements, the allowable soil pressure for this structure is on the order of 5,000 pounds per square foot.

8.6 ROADS AND PAVING

The roads and paved areas at the Rancho Seco site will be flexible pavements. Flexible pavements consist of well compacted subgrade, high quality granular base and sub-base materials, and bituminous paving or suitable surface courses of selected soils and aggregate.

The surface soils at the site are light unit weight silts and clays with gravels and are generally not suited as sub-base or base course material. Based on an estimated CBR (California Bearing Ratio) value of 12 or less for these soils and an assumed maximum wheel load of 8,000 lbs, a combined thickness of pavement and base of 10 inches is recommended. Assuming all road subgrade will be in cuts, the top 0.5 feet of subgrade should be scarified and recompacted to a minimum of 95 percent maximum density. From 0.5 feet to 1 foot a minimum of 90 percent density should be obtained. The base course should be a select, free-draining granular material and should be uniformly-compacted to the required density. Properly prepared subgrade drainage installations should be located in areas of potential inundation by storm runoff water.

The surface courses for other paved areas should be designed so as to be compatible with the anticipated design wheel loads.

8.7 GRADING, SITE PREPARATION AND EROSION CONTROL

The purpose of the grading and site preparation recommendations is to assure that the site construction procedures are consistent with the engineering design conclusions based on the results of the soil and foundation investigation and analysis of the site.

Unsuitable material such as trees, organic material, debris, major root systems, and soft or compressible soils should be located and removed. All surface ground water should be carried through the site by permanent or temporary drainage systems and be diverted during construction in an appropriate manner. Excavated surfaces should be properly graded to prevent ponding of water during construction.

All grading and site preparation should be accomplished in accordance with the following guide specifications and under continual guidance and inspection of a soil engineer. Cut slopes excavated should not be graded steeper than the following.

Height of Slope (Feet)	Recommended Overall Slope Inclination (Horizontal to Vertical)
0 to 10	1-1/2 to 1
10 to 20	1-3/4 to 1
20 plus	2 to 1

In selected areas, temporary construction slopes can be excavated at steeper slopes. The soils at the site have a moderately high potential for erosion. It is therefore recommended that all cut slopes more than 15 feet in height have a 6-foot wide horizontal berm to intercept storm water runoff. The berms should be constructed at mid height of the slopes or at the maximum vertical spacing of 15 feet. All newly-graded fill slopes and cut slopes should be planted to further inhibit slope erosion.

Erosion control is an important consideration in soils that exist at the Rancho Seco site. Vegetation or other appropriate cover should be provided on slopes and areas subject to erosion. The surface soils should be selected and prepared where landscaping is proposed for erosion control. The planting should obviously be consistent with the soil and climatic conditions at the site in order to assure permanent or long-term growth.

9.0 GUIDE SPECIFICATIONS - ENGINEERED FILL

9.1 GENERAL CONDITIONS

9.1.1 Definition of Terms

a. Fill: A soil or soil-rock material placed to raise the existing grade or to backfill excavations.

b. On-site Material: Material obtained from the site.

c. Import Material: Material hauled in from off-site borrow areas.

d. Engineered Fill: A fill that has been constructed in accordance with the specification requirements.

e. Degree of Compaction: A ratio of dry densities expressed as a percentage of the constructed fill to that of the same material determined by ASTM D-1557 for fine grained-material. For granular material the degree of compaction should be obtained by relative density method ASTM D-2049.

355

9.1.2 Duties of Soil Engineer

A soil engineer shall be the owner's representative whose duty it shall be to observe the grading operations during preparation of the site and the construction of any engineered fill. The soil engineer shall make a sufficient number of field observations and tests to enable him to form an opinion of the fill material, and the degree of compaction of the fill. Any fill that does not meet the specification requirements shall be reconstructed until the requirements are satisfied.

9.1.3 Soil Conditions

A soil investigation has been performed for this site. The contractor shall familiarize himself with the soil conditions at the site, whether covered in that report or not, and shall thoroughly understand all recommendations associated with grading.

9.2 SITE PREPARATION

9.2.1 Excavation

All excavations shall be carefully made true to the grades and elevations shown on the plans. The excavated surfaces shall be properly graded to prevent ponding of water during construction. After stripping and excavating, the exposed surfaces to be filled shall be scarified to a depth of 6 inches and compacted to the requirements of engineered fill. Scarifying of hard rock surfaces, as determined by the soil engineer, will not be required.

Subsurface drains will be required where indicated on the plans and where seepages are observed during grading. The soil engineer will determine the final locations of the subdrains, and the construction of subdrains shall be in accordance with the subdrain specifications for this project.

Before placing fill, the contractor shall obtain the soil engineer's approval of the site preparation in the area to be filled. The requirements of this section may be omitted only when approved in writing by the soil engineer.

9.2.2 Material used for Fill

All fill material must be approved by the soil engineer. The material shall be a soil or soil-rock mixture which is free from organic matter or other deleterious substances. Fill material placed within 2 feet of finished grades or in fill slopes on steep hillsides shall not contain rocks over 6 inches in greatest dimension. Larger rock may be incorporated into the lower portion of fills provided the rock is well mixed with soil and the method of placing is approved by the soil engineer. Pockets of

356

large rock with little or no soil will not be permitted. The soils and rock from the site, but below the stripped layer containing organic material, are suitable for use in fills.

9.2.3 Placing and Compacting Fill Material

All fill material shall be compacted as specified below or by other methods if approved by the soil engineer, so as to produce a minimum degree of compaction of 90 percent where the fill is 30 feet or less in depth. Where deeper fills are placed, the portion of the fill below a depth of 30 feet is recommended to be compacted to a minimum compaction of 95 percent of modified procter maximum density (ASTM D-1557) for fine-grained material and 80 percent relative density (ASTM D-2049) for granular material. Fill material shall be spread in uniform lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill shall be brought to a water contact that will permit proper compaction by either: (a) aerating the material if it is too wet, or (b) spraying the material with water if it is too dry. Each lift shall be thoroughly mixed before compaction to ensure a uniform distribution of water content.

9.2.4 Preparation for Filling

All vegetation including grasses, trees, major root systems, and other organic material shall be stripped to a minimum depth of 4 inches or to such greater depth as the soil engineer in the field may consider as being advisable to remove material which, in his opinion, is unsatisfactory. The stripped material shall either be removed from the site or stockpiled for later use as topsoil; none of the stripped material shall be used as engineered fill.

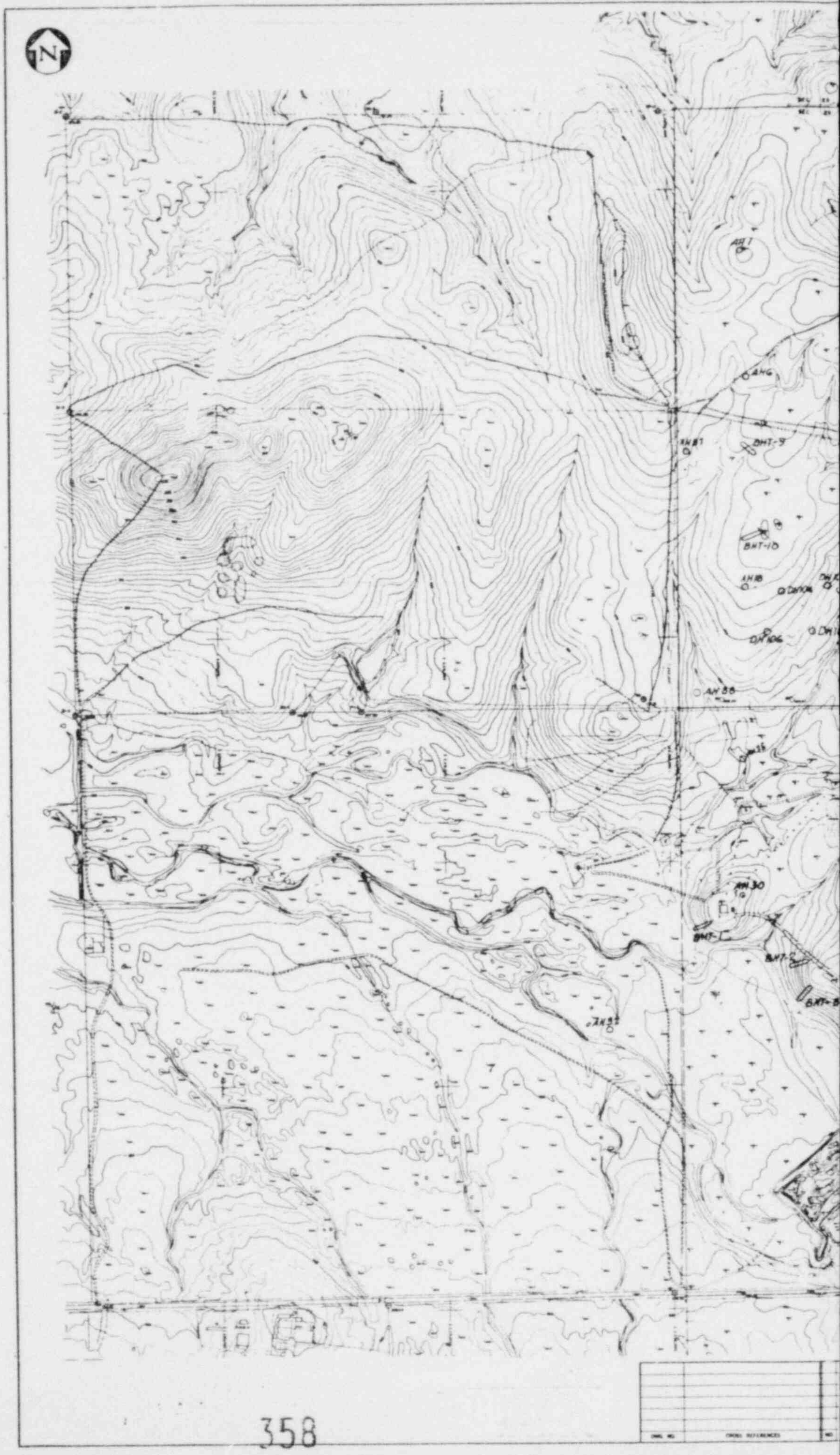
After stripping, deeper excavating shall be performed where necessary to remove any soft and loose soils in areas to be filled. The bottom of the excavations shall extend well into stiff and dense soils; if such soils are not encountered, the excavations shall extend to a satisfactory depth as determined by the engineer. Excavations shall be equipment width in order to permit compaction with full-sized grading equipment.

In the cut-fill transitions, the surface soils shall be excavated to a sufficient depth to remove all soft and loose soils and to provide the specified thickness of engineered fill beneath footings.

9.2.5 Treatment After Completion of Grading

After grading is completed and the soil engineer has finished his observations of the work, no further excavation or filling shall be done, except with the approval of, and under the observation of the soil engineer.

357



358

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SACRAMENTO MUNICIPAL UTILITY DISTRICT
RANCHO SECO NUCLEAR STATION - UNIT 1

FOUNDATION INVESTIGATION MAP

S M U D

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DESIGNED	JOB CHECK	CHIEF CHECK	PROJ. ENGR.	
T.C.	WAB	KHW		

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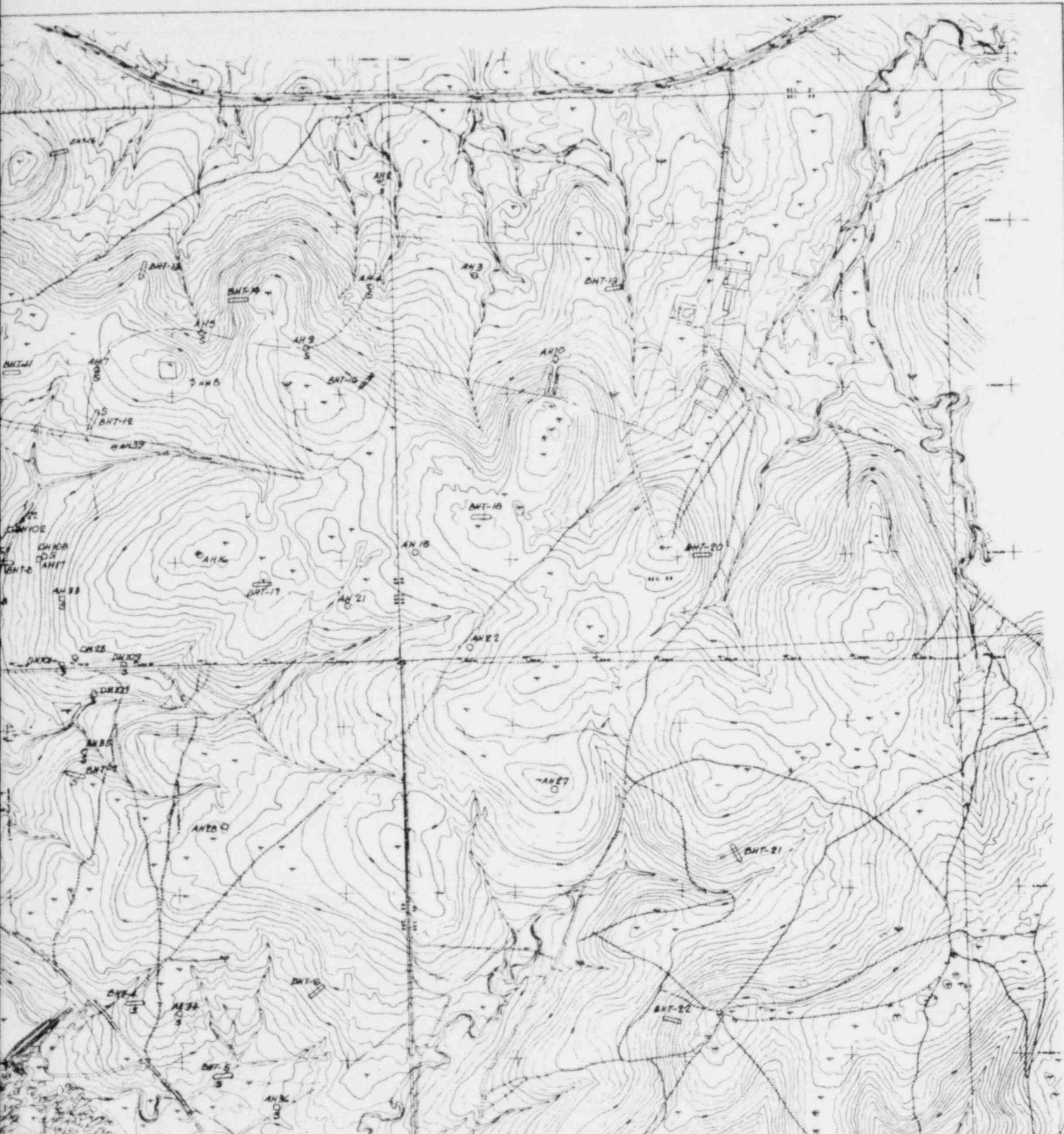
360



BECHTEL CORPORATION
LOS ANGELES

SACRAMENTO MUNICIPAL UTILITY DISTRICT
RANCHO SECO NUCLEAR STATION - UNIT 1

FOUNDATION INVESTIGATION MAP

DRAWN B. D.	JOB NO. 6292	DRAWING NO. C-119 -E
		REV. A

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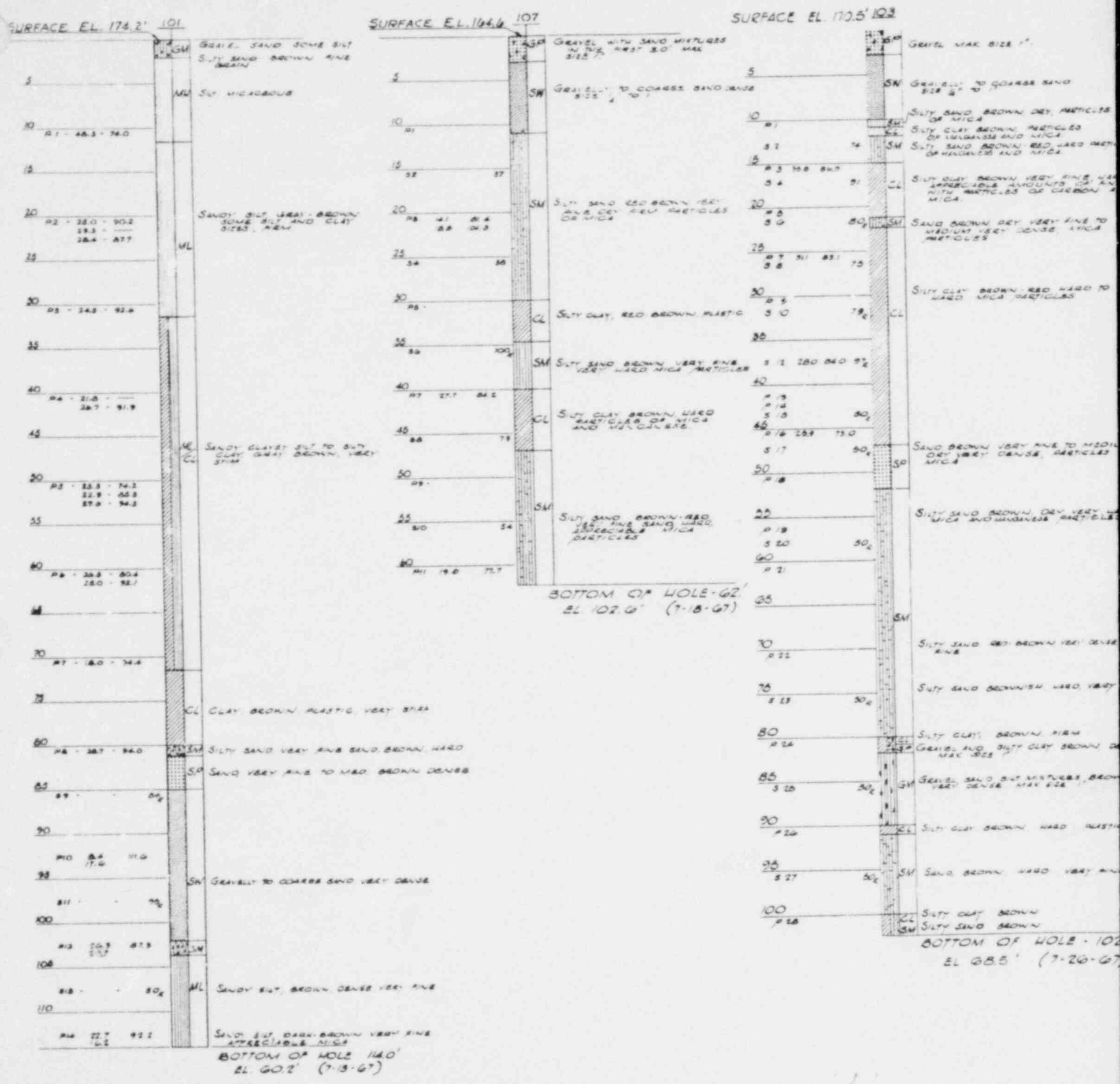
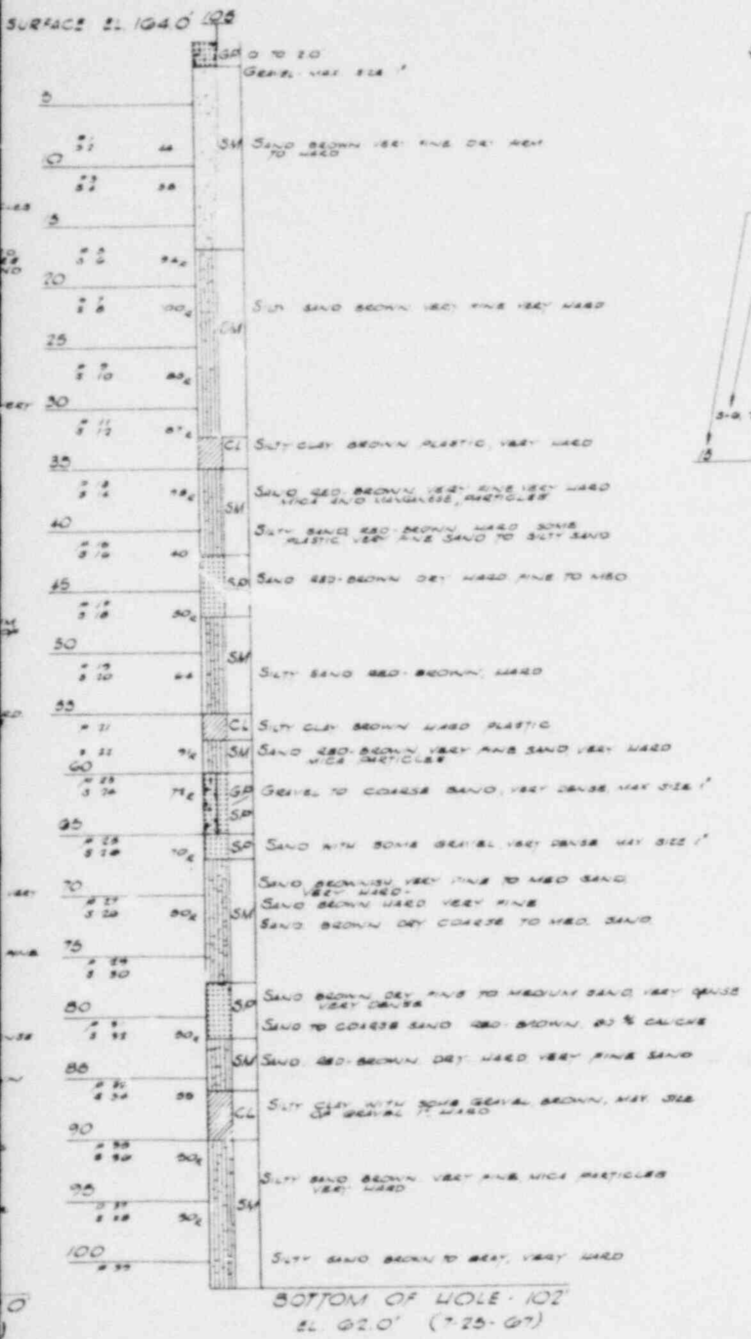
361



SURFACE EL. 174.2' 101
GM Gravel, sand, some silt
Silty sand brown fine grain
ML Silt micaceous
ML Sandy silt gray-brown some silt and clay sizes, firm
ML-CL Sandy clayey silt to silty clay gray brown, very stiff
CL Clay, brown plastic, very stiff
SM Silty sand very fine sand, brown, hard
SP Sand very fine to med. brown dense
SW Gravelly to coarse sand very dense
SM
ML Sandy silt, brown dense very fine
Sandy silt, dark-brown very fine appreciable mica
BOTTOM OF HOLE 114.0'
EL 60.2' (7-13-67)
SURFACE E.L. 164.6 107
GM Gravel with sand mixtures in the first 5.0' max size 1"
SW Gravelly to coarse sand dense size 1/4" to 1"
SM Silty sand red brown very fine dry, few particles of mica
CL Silty clay, red brown, plastic
SM Silty sand brown very fine very hard mica particles
CL Silty clay brown, hard particles of mica and manganese
SM Silty sand brown-red very fine sand, hard, appreciable mica particles
BOTTOM OF HOLE-62'
EL 102.6' (7-18-67)
SURFACE EL. 170.5' 103
GP Gravel max size 1"
SW Gravelly to coarse sand size 1/4" to 1"
SM Silty sand brown dry, particles of mica
CL Silty clay brown, particles of manganese and mica
SM Silty sand brown, red, hard particles of manganese and mica
CL Silty clay brown very fine, with appreciable amounts of ... with particles of carbon ... mica
SM Sand brown dry very fine to medium very dense, mica particles
CL Silty clay brown-red hard to hard mica particles
SP Sand brown very fine to medium dry very dense, particles mica
SM Silty sand brown, dry very ... mica and manganese particles
Silty sand red-brown very dense fine
Silty sand brownish, hard, very
Silty clay, brown, firm
Gravel and silty clay brown max size 1"
GM Gravel sand silt mixtures, brown very dense max size 1"
CL Silty clay brown, hard, plastic
SW Sand, brown, hard, very fine
CL Silty clay brown
SM Silty sand brown
BOTTOM OF HOLE - 102'
EL 68.5' (7-26-67)

362



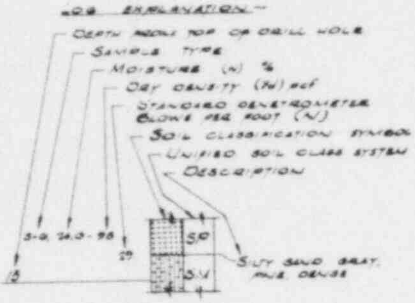
LEGEND

SAMPLE TYPES–

S = STANDARD PENETRATION TEST FOR BLOW COUNT (N) = NO. OF BLOWS PER FOOT FOR 2" O.D. SAMPLER, DRIVEN WITH 140# WEIGHT DROPPING 30".

D = PITCHER SAMPLER

LOG EXPLANATION–



NOTE

50_R INDICATES MORE THAN 50 BLOWS FOR PENETRATION LESS THAN 0.5'.

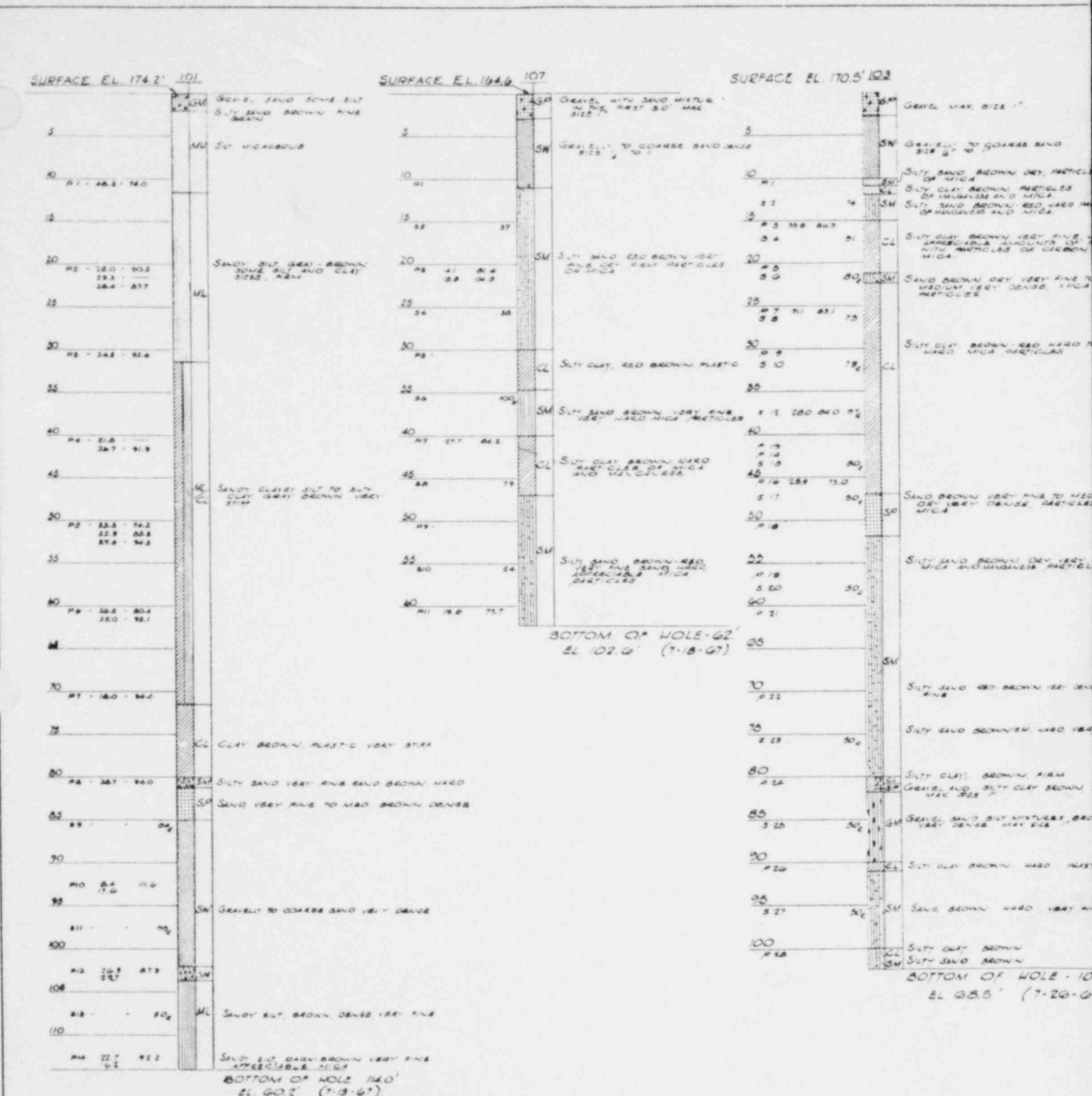
BECHTEL CORPORATION LOS ANGELES	
SACRAMENTO MUNICIPAL UTILITY DISTRICT RANCHO SECO NUCLEAR STATION - UNIT 1	
SOIL AND FOUNDATION INVESTIGATION LOGS OF BORINGS SHEET NO. 1	
NAB · 6783	C-120 E · REV A

363

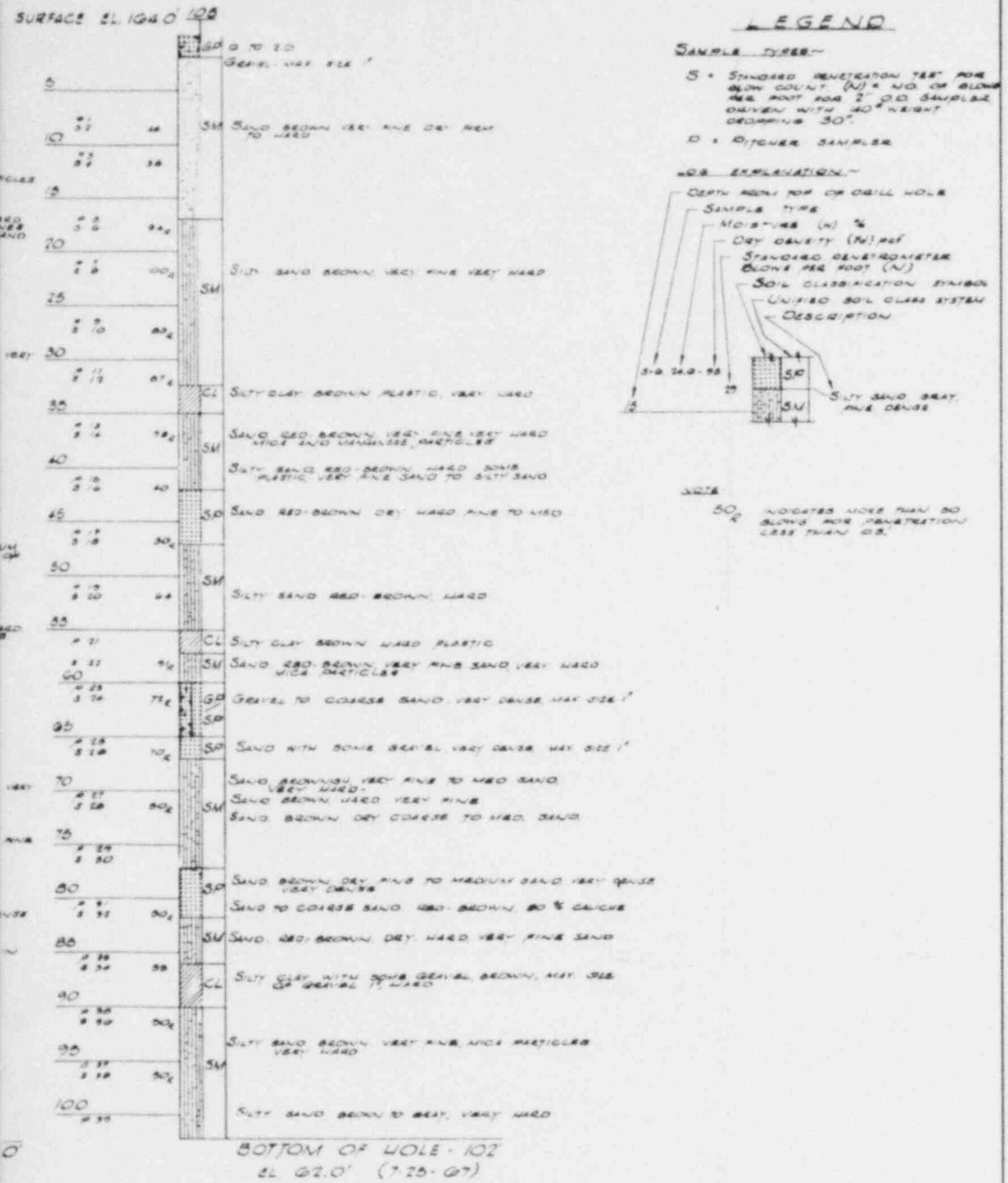
IMAGE EVALUATION
TEST TARGET (MT-3)

6"

MICROCOPY RESOLUTION TEST CHART



SURFACE EL. 174.2' 101
SURFACE EL. 164.6 107
SURFACE EL. 170.5' 103
Gravel, sand some silt
Clay brown, plastic, very stiff
Silty sand very fine sand brown hard
Sand very fine to med. brown dense
Gravelly to coarse sand very dense
Sandy silt, brown, dense very fine
BOTTOM OF HOLE 114.0'
EL. 60.2' (7-13-67)
Silty clay, red brown plastic
BOTTOM OF HOLE-62'
EL. 102.6' (7-18-67)
Gravel max. size 1"
Silty clay, brown, firm
Silty clay brown
Silty sand brown
EL. 68.5' (7-26-6
001



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BECHTEL CORPORATION
LOS ANGELES

SACRAMENTO MUNICIPAL UTILITY DISTRICT
RANCHO SECO NUCLEAR STATION - UNIT 1

SOIL AND FOUNDATION INVESTIGATION
LOGS OF BORINGS
SHEET NO. 1

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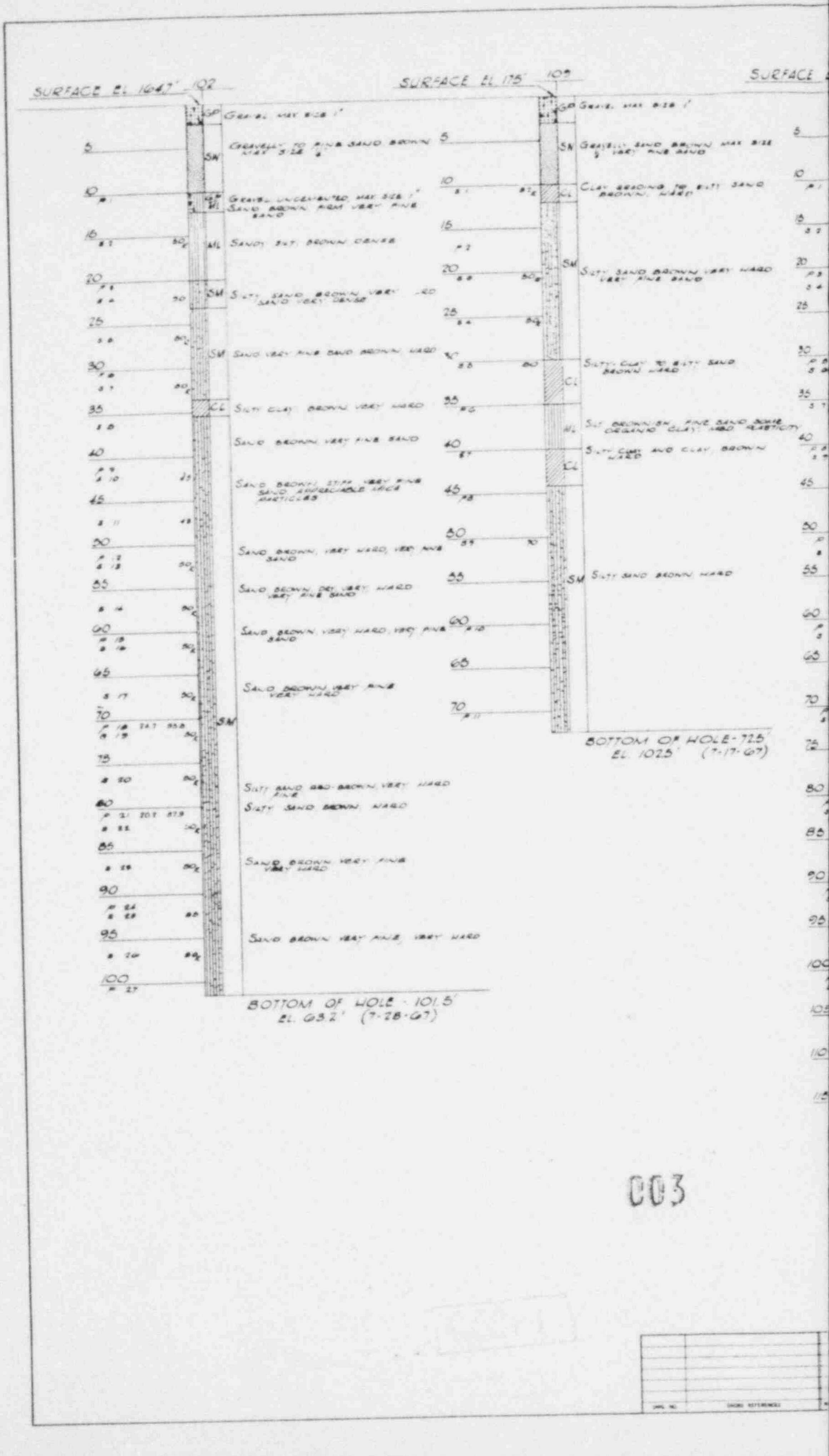
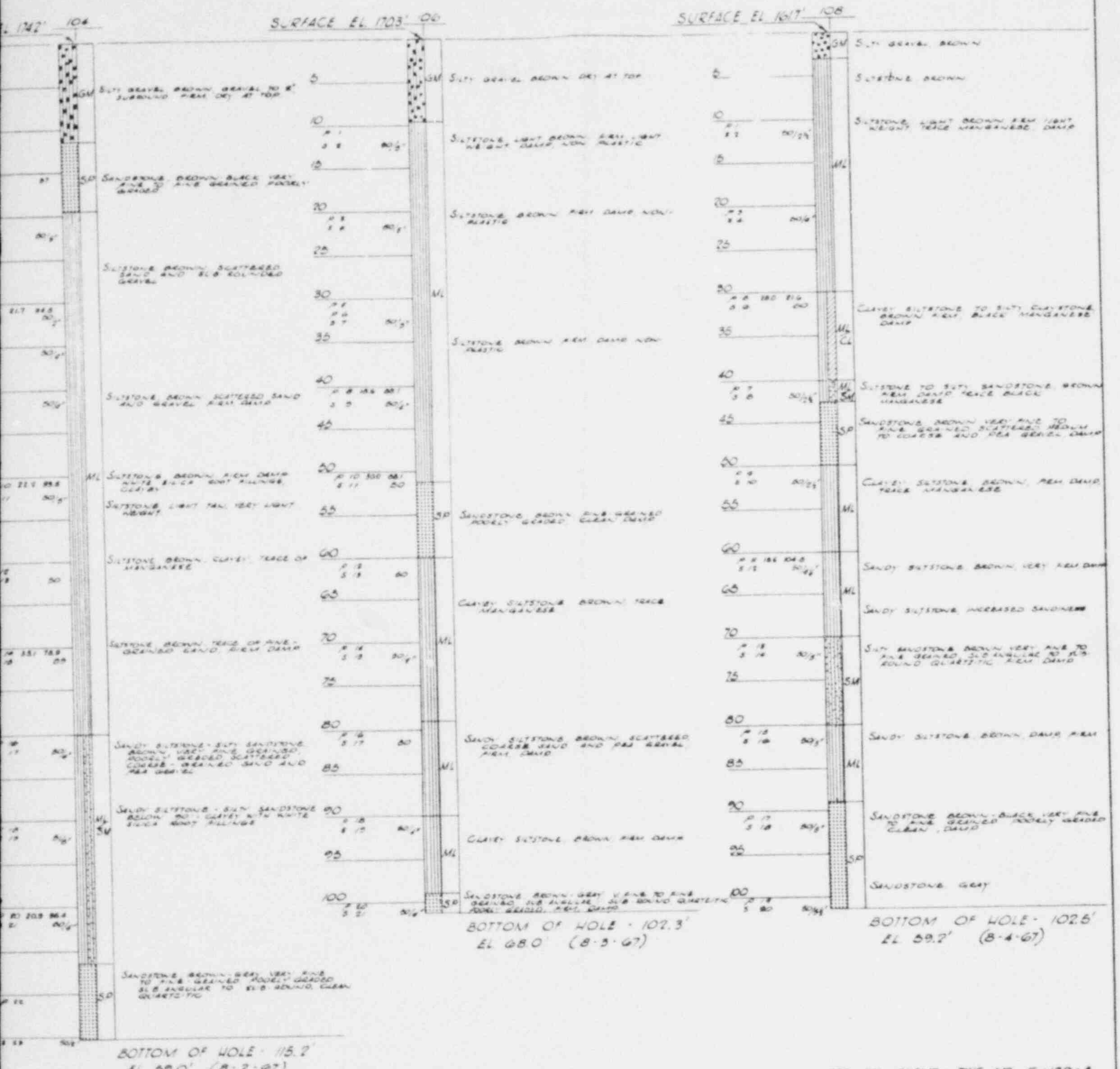

SURFACE EL 164.7' 102
SURFACE EL 175' 109
SURFACE
Gravel max size 1"
Silty clay brown very hard
Sand brown very fine sand
Silty sand brown hard
Sand brown very fine, very hard
BOTTOM OF HOLE - 101.5'
EL. 63.2' (7-28-67)
BOTTOM OF HOLE - 72.5'
EL. 102.5' (7-17-67)

003



NOTE: SEE LEGEND - DWG. NO. C-120-E

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BECHTEL CORPORATION
LOS ANGELES

SACRAMENTO MUNICIPAL UTILITY DISTRICT
RANCHO SECO NUCLEAR STATION - UNIT 1

SOIL AND FOUNDATION INVESTIGATION
LOGS OF BORINGS
SHEET NO. 2

A ISSUED FOR PRELIMINARY SAFETY ANALYSIS REPORT

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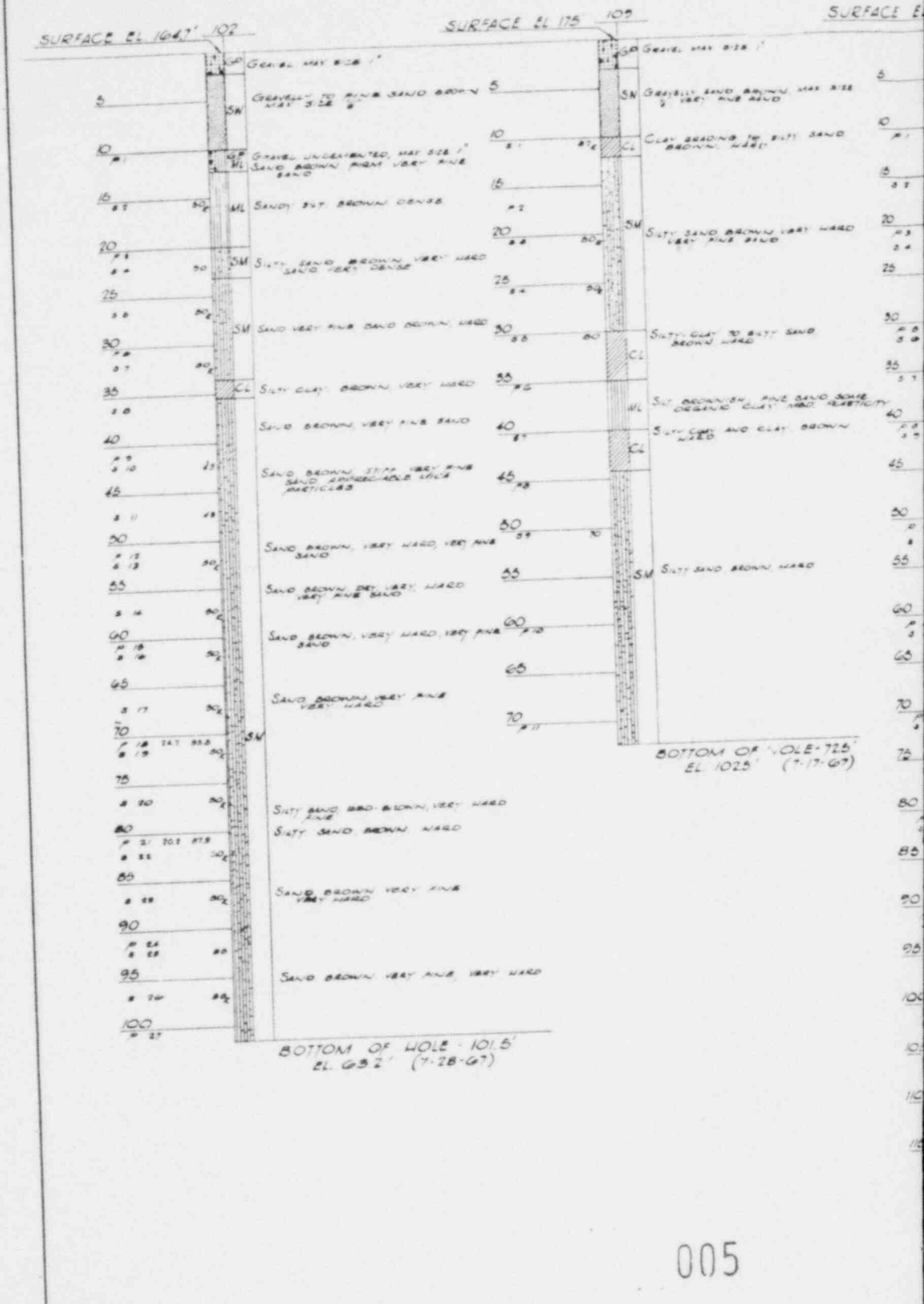
C-121-E

A


SURFACE EL 164.7' 102
GP Gravel max size 1"
SW Gravelly to fine sand brown max size
GP Gravel uncemented, max size 1"
ML Sand brown firm very fine sand
ML Sandy silt brown dense
SM Silty sand brown very hard sand very dense
SM Sand very fine sand brown hard
CL Silty clay brown very hard
Sand brown very fine sand
Sand brown stiff very fine sand appreciable mica particles
Sand brown very hard very fine sand
Sand brown dry very hard very fine sand
Sand brown, very hard, very fine sand
Sand brown very fine very hard
SW
Silty sand red-brown, very hard fine
Silty sand brown hard
Sand brown very fine very hard
Sand brown very fine, very hard
BOTTOM OF HOLE - 101.5'
EL 63.2' (7-28-67)
SURFACE EL 175 103
GP Gravel max size 1"
SW Gravelly sand brown max size 1" very fine sand
CL Clay grading to silty sand brown hard
SM Silty sand brown very hard very fine sand
CL Silty clay to silty sand brown hard
ML Silt brownish fine sand some organic clay med. plasticity
CL Silty clay and clay brown hard
SM Silty sand brown hard
BOTTOM OF HOLE - 72.5'
EL 102.5' (7-17-67)
SURFACE E

005

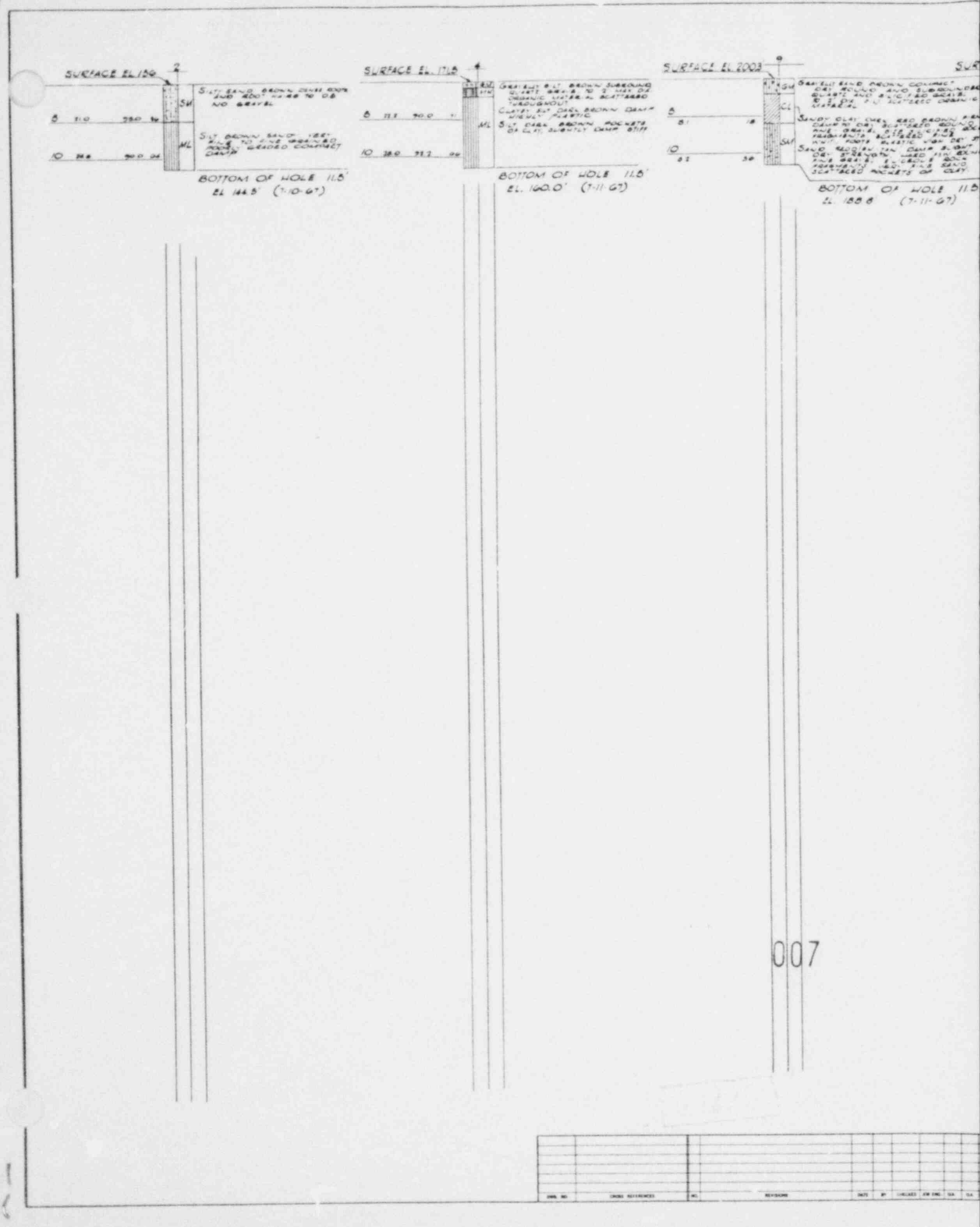
NOTE: SEE LEGEND - DWG. NO. C-120-E

BECHTEL CORPORATION
LOS ANGELES

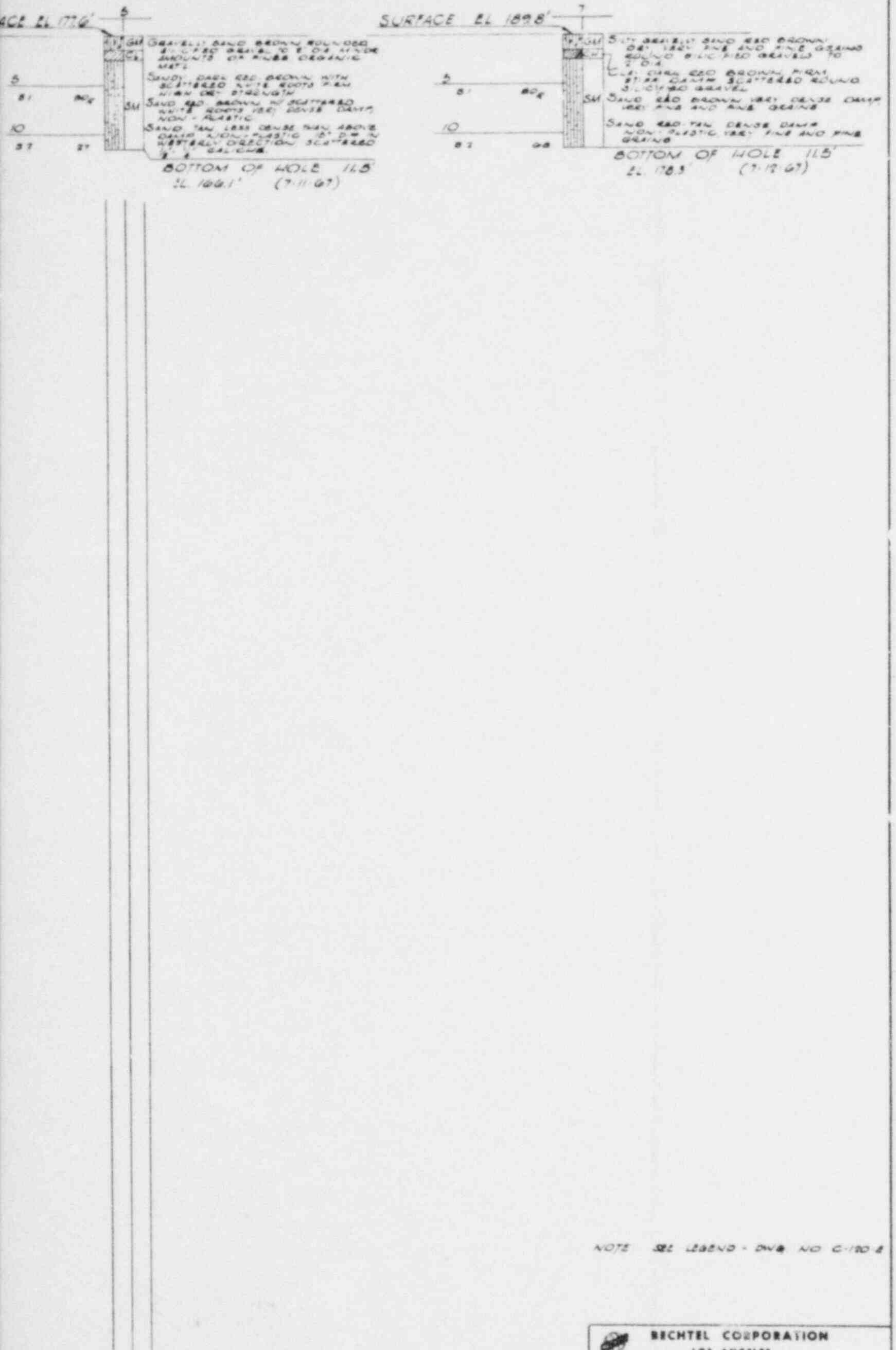
SACRAMENTO MUNICIPAL UTILITY DISTRICT
RANCHO SECO NUCLEAR STATION - UNIT 1

SOIL AND FOUNDATION INVESTIGATION
LOGS OF BORINGS
SHEET NO. 2

C-121-E A



SURFACE EL 156
2
SW
ML
BOTTOM OF HOLE 11.5'
EL 144.5' (7-10-67)
SURFACE EL. 171.5
4
ML
BOTTOM OF HOLE 11.5'
EL. 160.0' (7-11-67)
SURFACE EL 200.3
9
GM
CL
SM
BOTTOM OF HOLE 11.5
EL. 188.8 (7-11-67)
SUR
007
REVISIONS
CROSS REFERENCES



NOTE: SEE LEGEND - DWG NO C-120-E

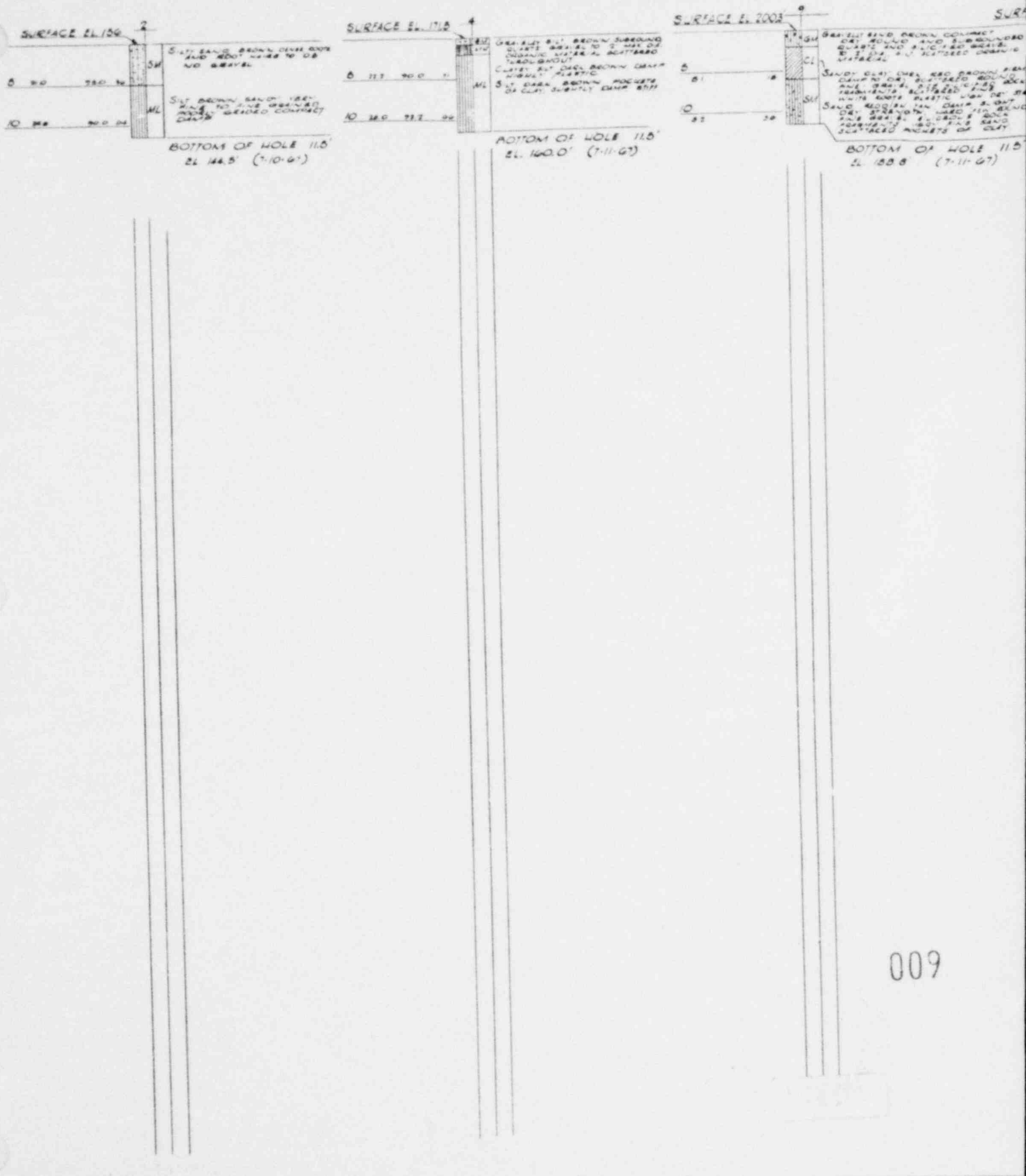
BECHTEL CORPORATION
LOS ANGELES

SACRAMENTO MUNICIPAL UTILITY DISTRICT
RANCHO SECO NUCLEAR STATION - UNIT 1

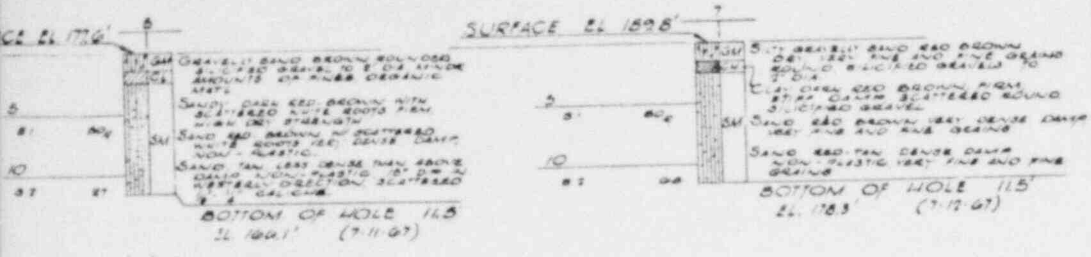
SOIL AND FOUNDATION INVESTIGATION
LOGS OF BORINGS
SHEET NO 3

C-122-E A

008


SURFACE EL.156
2
SW
ML
BOTTOM OF HOLE 11.5'
EL 144.5' (7-10-67)
SURFACE EL. 171.5
4
ML
BOTTOM OF HOLE 11.5'
EL. 160.0' (7-11-67)
SURFACE EL. 200.3
GM
CL
SM
BOTTOM OF HOLE 11.5'
EL 188.8' (7-11-67)

009



NOTE SEE LEGEND - DWG NO C-120-E

BECHTEL CORPORATION
LOS ANGELES

SACRAMENTO MUNICIPAL UTILITY DISTRICT
RANCHO SECO NUCLEAR STATION - UNIT 1

SOIL AND FOUNDATION INVESTIGATION
LOGS OF BORINGS
SHEET NO. 3

C-122-E A

010

SOIL MECHANICS
and FOUNDATION
ENGINEERS Inc.

November 17, 1967
Project 2111

Bechtel Corporation
P. O. Box 58587
Los Angeles, California 90050

Attention: Mr. Edward Rose

Subject: Supplementary Report of Laboratory Testing
Rancho Seco Nuclear Generating Station, Unit No. 1
Sacramento Municipal Utility District

Gentlemen:

The results of our laboratory investigation on the Rancho Seco project for the appropriate engineering properties of the soil samples recovered from Drill Holes 104, 105, and 106 by your field forces are appended. The testing program, carried out according to our agreement of July 10, was similar in scope and test methods to the first phase of the laboratory testing, the results of which were forwarded to you September 15, 1967.

LABORATORY TEST PROGRAM

The samples were contained in Pitcher Tubes of 2.875 inch diameter and were picked up by us at the Pollock Pines Office on September 15, 1967 and transported to our Palo Alto laboratory for sorting and logging.

After receipt of samples, the test schedules were set up through discussion with your firm and the writer and formalized in your letter dated September 20, 1967. Some deviations from this proposed schedule were necessary due to soil variation encountered when the actual testing program commenced. A summary of the laboratory tests performed is shown on Table A.

LABORATORY TESTING PROCEDURES

All laboratory testing procedures, except for the one-dimensional consolidation test, were identical to those discussed in our first report to you dated September 15, 1967. The only change in procedure for the consolidation test was the performance of a rebound stress loop piror to the most probable preconsolidation pressure so as to better define the recompression characteristics of the samples tested. Hence, the specimens were loaded to 4.0 ksf, rebounded to 0.5 ksf, and then reloaded to 64.0 ksf. The standard load-increment ratio of 1 was used for all loadings except that of the final rebound which was accomplished with the usual load-decrement ratio of 4.

DISCUSSION OF TEST RESULTS

The soils tested ranged from silts of moderately high to low plasticity to well graded medium sands. Material similar to the elastic silt discussed in the first phase of the laboratory investigation was encountered only in Hole 104 at 60 to 62 feet depth during the present investigation.

The consolidation tests indicate that the soils tested are relatively incompressible for loads below the most probably preconsolidation stress, since recompression indices of between 0.003 to 0.027 were obtained for the five consolidation tests. In general, the materials tested do not remember their stress or geologic history very well; however, it is our opinion, after interpreting the available field and laboratory data, that a stress of approximately 10 ksf above the existing overburden pressure has occurred at the project site.

The triaxial shear tests (consolidated undrained with pore pressure measurements) resulted in an effective angle of shearing resistance of 37.5 degrees and a cohesion intercept of 5.5 psi for the column of material tested in DH 104 between 15 to 40 foot depth. The second envelope, which was an attempt to determine the strength parameters for the deeper soils, was not completely successful due to the substantial variation of plasticity indices of the tested soils.

Very truly yours,

SOIL MECHANICS and
FOUNDATION ENGINEERS, Inc.

J V Lowney

John V. Lowney
RE15774

JVL:vs

Enc.

the soils in their natural condition, drying them in an oven maintained at a temperature of 105°C. for about 24 hours or until a constant weight was attained. The oven-dried samples were then weighed and the moisture content calculated as the percentage of the weight of contained water to the weight of the oven-dried soil. The results of the water content determinations are presented in Table 1.

Dry Density -Dry density determinations were obtained by accurately measuring the volume of the specimen, weighing the specimen after it has been oven dried and calculating the unit weight. In general, though, the dry density was not calculated upon the measurement of the total volume of oven-dried material but upon a measurment of the total weight of original undisturbed material extruded from a container of known volume from which a small representative portion was removed for a moisture content determination. The dry density of the total sample section was thereby computed. The results of the dry density determinations are presented in Table 1.

Gradation Analysis - The grain-size distribution was determined in accordance with ASTM Test Designation D422. Soil samples were soaked in water with a dispersing agent for at least 16 hours and then washed through a No. 200 sieve. Mechanical sieve analyses were performed on the sample retained on the No. 200 sieve and hydrometer analyses were performed on the material passing the No. 200 sieve. Results of the gradation analyses are presented on Figure 1, Sheets 1 and 2.

Atterberg Limits - Atterberg Limits determinations on some of the finer grained soils were performed in accordance with ASTM Test Designation D423 (Liquid Limit) and ASTM Test Designation D424 (Plasticity Index). The results of the Atterberg Limits are shown plotted on the Plasticity Chart, Sheets 1 and 2 of Figure 2.

Specific Gravity - Specific gravity determinations were performed upon specified soil samples in accordance with ASTM Test Designation D854 to aid in the evaluation of the low-density surface soils. Results of the specific gravity tests are presented in Table 2.

LABORATORY TESTING PROCEDURES - MECHANICAL PROPERTIES

Specific samples were selected for mechanical property determinations such as consolidation, shear strength, and permeability both on the basis of the results of the physical property determinations and because of their location with respect to a proposed structure.

Consolidation Tests - Undisturbed specimens extruded from the tube samples were carefully trimmed and tested in the consolidometer under one-dimensional consolidation to determine the consolidation characteristics of the soils. Specimens were trimmed into a 2.5 inch diameter, 1 inch high, consolidometer ring. Vertical loads were applied instantaneously in increments such as to produce a load-increment ratio equal to one (applied load equal to existing load on consolidometer). Prior to setting the strain dials and commencing the test, a slight seating load was applied so as to minimize the effects of any surface disturbance resulting from trimming.

The specimens were saturated with water at applied loads generally between 1000 to 2000 pounds per square foot, and any concomitant consolidation or swelling was noted. Specimens were loaded up to a maximum pressure of generally 64,000 pounds per square foot so as to produce a relatively straight-line virgin compression curve; rebound was in decrements such that each remaining load was equivalent to one-fourth that of the previous applied load. (i.e., three-quarters of the load was removed for each decrement).

Loads were usually applied for periods of 24 hours. The consolidation test results in the form of void ratio-logarithm of pressure curves together with the corresponding time-compression curves are shown on Figures 3 through 8.

Triaxial Shear Tests- Consolidated undrained triaxial shear tests with pore pressure measurements were performed on certain undisturbed samples of 2.875 inch diameter specimens. The height to diameter ratio of all specimens was equal to or greater than 2.0. The specimens extruded from the tubes were cut to the desired length, weighed, measured, placed in the triaxial cell and encased in a thin rubber membrane.

Before the specimen was completely sealed, all connections were made and all lines were filled with water. The chamber was then filled with water. A controlled internal back pressure of 70 psi on the pore water-air phase and a slightly greater chamber pressure was applied in increments. A slight gradient of water was introduced between the ends of the sample to achieve saturation which was accomplished generally in one to three days. The use of a back pressure caused a general dissolution of air entrapped in the water thereby permitting a high degree of saturation to be obtained.

Preselected chamber pressures were then applied. The difference between the chamber pressure and the internal back pressure was maintained equal to the desired effective consolidation pressure; a condition of equilibrium was reached generally under three days.

The sample drainage lines were then closed and the pore pressure measurement device (transducer) was attached. The sample was then loaded to failure using a constant rate of strain of 0.0025 inches per minute (5 to 7 percent per hour) resulting in shearing periods of from two to three hours.

Because the samples were occasionally of insufficient length to obtain many triaxial specimens, one specimen was retested in a second stage at a higher effective consolidation pressure. The stage testing was accomplished by first consolidating the specimen under the procedures outlined above, then loading the specimen unil failure was achieved. The specimen drainage lines were then reopened, the effective consolidation pressure was adjusted to the second stage pressure by increasing the chamber pressure, and the sample was allowed to consolidate until internal pressure equilibrium was again established under the higher chamber pressure. The drainage lines were then closed, a transducer was attached to measure the pore pressures, and the specimen was again loaded to failure.

The maximum effective principal stress ratio was used as the failure criterion which would best define the shear strength. Mohr circles, based on the above failure criterion, are shown plotted with the total and effective stresses acting on the specimens at failure.

As requested, besides the Mohr circles for total and effective stresses, the obliquity or principal stress ratio-strain curves, the stress-strain curves, the pore pressure-strain curves, and initial and final water contents are shown on Figure 9.

Permeability

Constant-head permeability determinations were performed on undisturbed specimens in the triaxial cells prior to consolidation and shear but with the application of an internal back pressure. Hence, the permeability values are expected to be realistic and relatively unaffected by the presence of air, since most of this would have been forced into solution. The results of the permeability tests are shown on the summary sheet for the triaxial shear tests, Figure 9.

DISCUSSION OF TEST RESULTS

The soils tested range from clayey silts (ML) to silty clays (CL) of low to moderate plasticity. One sample at ten foot depth in DH 101 was found to be an elastic silt (MH), perhaps loessial in nature, and swelled slightly under a consolidation load of 2000 pounds per square foot. Also, this sample was found to be of low unit weight, composed of particles mainly of silt size but having a relatively normal specific gravity (2.78). Hence, a porous soil structure is indicated. Further observation using a stereo binocular microscope also indicated an open structure, lacking in any preferred orientation or layering. However, a thin section examination did not reveal a porous structure. With the evidence at hand, we do not believe that the material is loess but suggest further study if greater quantities of this soil are encountered.

The consolidation tests indicate that the tested soils are relatively incompressible for applied loads below the preconsolidation stress. Hence, the movement to be expected below these stresses will be due to recompression. Virgin compression would be expected above these values. The preconsolidation is probably due to dessication and erosion of overburden. Also, the soils were found to be fairly elastic: (a) the time-compression curves were found to be rather linear in shape and (b) the stress-strain curve data obtained in the triaxial shear tests were straight over substantial portions of the curves.

The permeability values are more or less in line with what would be expected for these relatively impervious materials.

The triaxial shear tests, consolidated undrained with pore pressure measurement, resulted in an effective angle of shearing resistance of 32 degrees and a cohesion intercept of 15 psi for the column of materials tested in Drill Hole 101 between 10 and 70 foot depth. An unusual uniformity in strength results was achieved, as can be seen on the Mohr circle plots by the tangency of the envelope to the circles. In keeping with their presumed elasticity, the soils are believed to be of a brittle nature as judged by the low strains at failure (about 2 to 3 percent).

No unusual properties were found during the testing program other than the light weight, elastic silts, as noted above.

Very truly yours,

SOIL MECHANICS and
FOUNDATION ENGINEERS, Inc.

John V. Lowney

John V. Lowney
RE15774

JVL:jw

Enc.

Table 1

RESULTS OF NATURAL WATER CONTENT AND DRY DENSITY TESTS

Sample Hole No.	Sample Depth (ft.)	Natural Water Content (%)	Natural Dry Density (pcf)
101	10-11.6	48.3	74.0
	20-22.0	25.0	90.2
		29.3	
		28.4	87.7
	30-31.4	24.5	92.6
	40-42.2	21.8	
		26.7	91.6
	50-51.8	23.3	74.2
		22.9	85.3
		27.6	94.3
	60-62.0	26.5	80.4
		25.0	92.1
	70-71.5	18.0	94.4
	80-81.3	28.7	94.0
	92-94.5	17.6	
		5.4	111.6
	102-103.8	29.7	
		26.3	87.3
	112-114	16.2	
		22.7	92.2
103	15-17	35.8	86.5
	25-26	31.1	83.1
	37-38.4	28.0	84.0
	45-47	25.9	73.0
107	20-22	14.1	81.4
		18.8	104.3
	40-42	27.7	84.2
	60-62	19.6	77.7

020

TABLE A
SUMMARY OF LABORATORY TESTS PERFORMED

NOTE: The letter and/or number presented below each test type refers to the table or figure number of Appendix B on which the test data is presented.

HOLE NO.	SAMPLE DEPTH (FEET)	LABORATORY TESTS PERFORMED										
		WATER CONTENT & DRY DENSITY	FIELD DENSITY	GRADATION ANALYSIS	HYDROMETER ANALYSIS	ATTERBERG ANALYSIS	SPECIFIC GRAVITY	COMPACTION	CONSOLIDATION	TRIAXIAL	PERMEABILITY	DYNAMIC TESTS
104	30-32	T-1		F-1	F-1	F-2				F-8		
	40-42	T-1		F-1	F-1	F-2				F-8		
	60-62	T-1		F-1	F-1	F-2	T-2		F-3	F-9		
	80-82	T-1				F-2	T-2		F-4			
	90-92	T-1		F-1	F-1	F-2				F-9		
	110-112	T-1		F-1		F-2						
105	15-17	T-1		F-1	F-1	F-2				F-8		
	30-32	T-1		F-1	F-1	F-2	T-2		F-5			
	55-57	T-1		F-1	F-1	F-2	T-2		F-6			
	70-72	T-1		F-1	F-1	F-2	T-2		F-7	F-9		
	90-92	T-1										
106	10-12	T-1				F-2						
	20-22	T-1				F-2						
	32-34	T-1		F-1	F-1	F-2						
	60-62	T-1				F-2						
	70-72	T-1				F-2						
	80-82	T-1				F-2						
	90-92	T-1				F-2						
	100-102	T-1				F-2						

SOIL MECHANICS and FOUNDATION ENGINEERS INC.

Project No. 2111

November 1967

TABLE 1

SUMMARY OF LABORATORY TEST DATA

HOLE NO	DEPTH (feet)	SAMPLE DESCRIPTION	UNIFIED SOIL CLASSIFICATION SYMBOL	NATURAL WATER CONTENT (%)	NATURAL DRY DENSITY (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	UNCONFINED SHEAR STRENGTH (psf)	DIRECT SHEAR TEST RESULTS	
									NORMAL STRESS (psf)	SHEAR STRENGTH (psf)
104	30-32	SILT, Medium brown slightly sandy	ML	24.0	91.7	38	8			
	40-42	SILT, Light brown	ML	23.0	91.0	34	4			
	60-62	SILT, Light brown with trace of Mg	ML	32.8	74.9	47	6			
	80-82	SILT, Light brown fine sandy	ML(NP)	19.5	86.6	NP	NP			
	90-91	SAND, Mottled brown medium	SW	17.0	--	NP	NP			
	91-92	SILT, Medium brown fine sandy	ML	23.7	98.0	38	3			
	110-112	SAND, Gray well sorted fine	SW	13.0	79.2	NP	NP			
105	15-17	SILT, Light brown slightly sandy	ML	32.8	82.3	37	5			
	30-32	SILT, Light brown slightly sandy	ML	32.6	82.9	41	7			
	55-57	SAND, Light brown silty	SM(NP)	14.9	89.8	NP	NP			
	70-72	SILT, Light brown fine sandy	ML(NP)	27.4	82.6	NP	NP			
106	10-12	SILT, Medium brown slightly sandy	ML	27.8	73.2	47	3			
	20-22	SILT, Light brown slightly sandy	ML	28.3	85.2	43	4			
	32-34	SILT, Medium with trace of Mg	ML	23.8	96.5	36	3			
	60-62	SILT, Medium with trace of Mg	ML	30.1	89.3	42	6			
	70-72	SILT, Medium dark brown sandy	ML	20.3	95.5	33	5			
	80-82	SILT, Light brown slightly sandy	ML	29.2	84.4	40	3			
	90-92	SAND, Gray-brown silty	SM(NP)	20.3	97.4	NP	NP			
	100-102	SAND, Gray poorly sorted silty fine	SP	12.4	92.1	NP	NP			

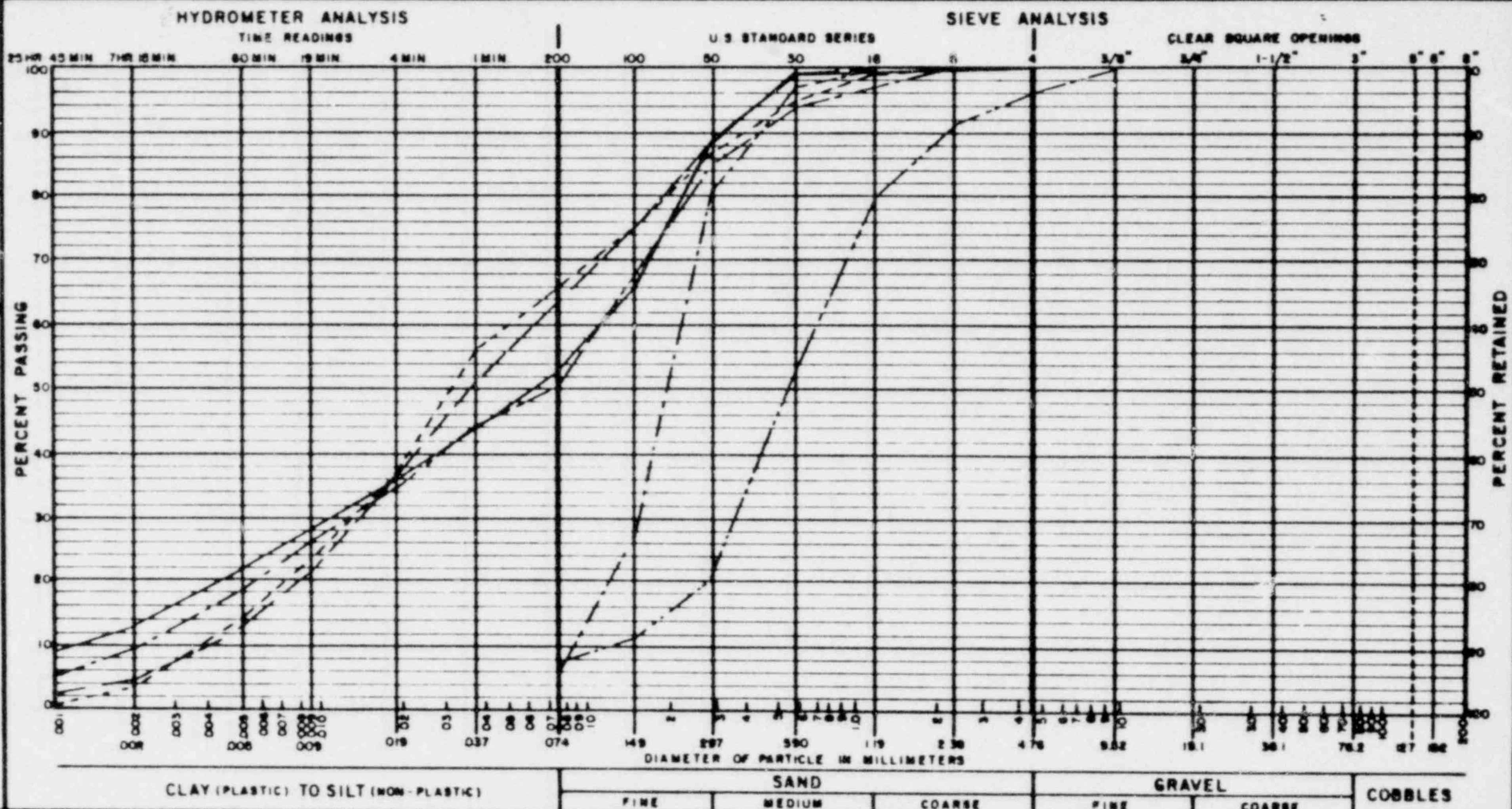
TABLE 2

SPECIFIC GRAVITY RESULTS

Hole No.	Depth (ft.)	Specific Gravity
104	60-62	2.69
104	80-82	2.68
105	30-32	2.77
105	55-57	2.70
105	70-72	2.67

023

LL	38	34	47	38					
PL	30	30	41	35					
PI	8	4	6	3	NP	NP			
NAT. W/C	24.0	23.0	55.2	23.7	17.0	13.0			
CLASSIF. SYMB.	ML	ML	ML	ML	SW	SW			
SAMPLE NO.	2	3	5	8	8	9			
DEPTH FT	30-32	40-42	60-62	91-92	90-91	110-112			
HOLE NO.	104	104	104	104	104	104			



Tested By RLV | Appr By | Date 11/67

Hole No. 104 | Area

GRADATION TEST RESULTS

Job No. 2111 | File No | Fig No. 1

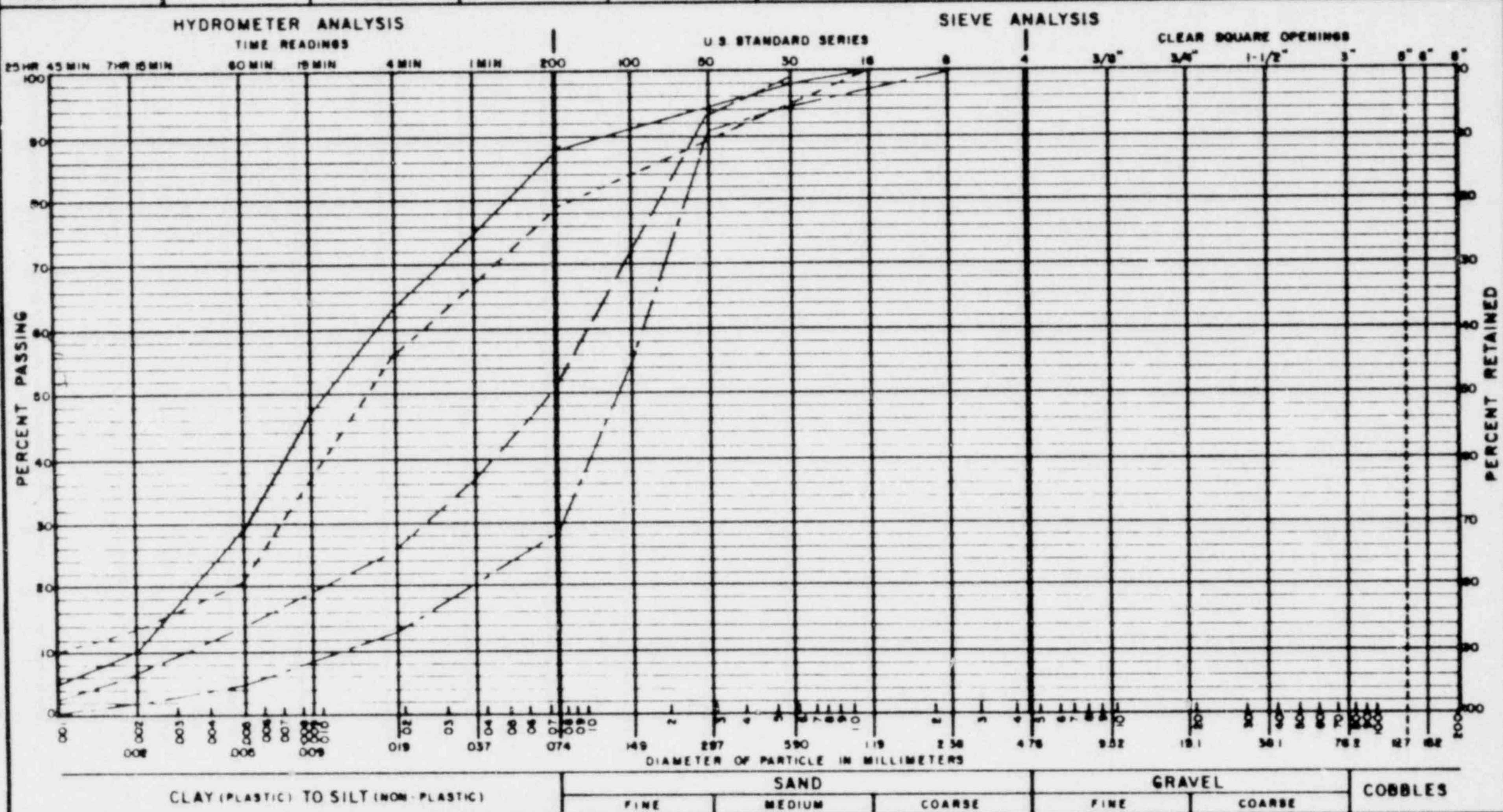
Sheet 1

SOIL MECHANICS & FOUNDATION ENGINEERS INC

24

785

LL	37	41	NP	32					
PL	32	34	NP	NP					
PI	5	7	NP	NP					
NAT. W/C	32.8	32.6	14.9	22.7					
CLASSIF. SYMB.	ML	ML	SM(NP)	ML(NP)					
SAMPLE NO.	5	11	21	27					
DEPTH FT	15-16.8	30-32	55-57	70-72					
HOLE NO.	105	105	105	105					



Tested By RLV	Appr By	Date 11/67	GRADATION TEST RESULTS		
Hole No. 105	Area		Job No. 2111	File No.	Fig No. 1

Sheet 2

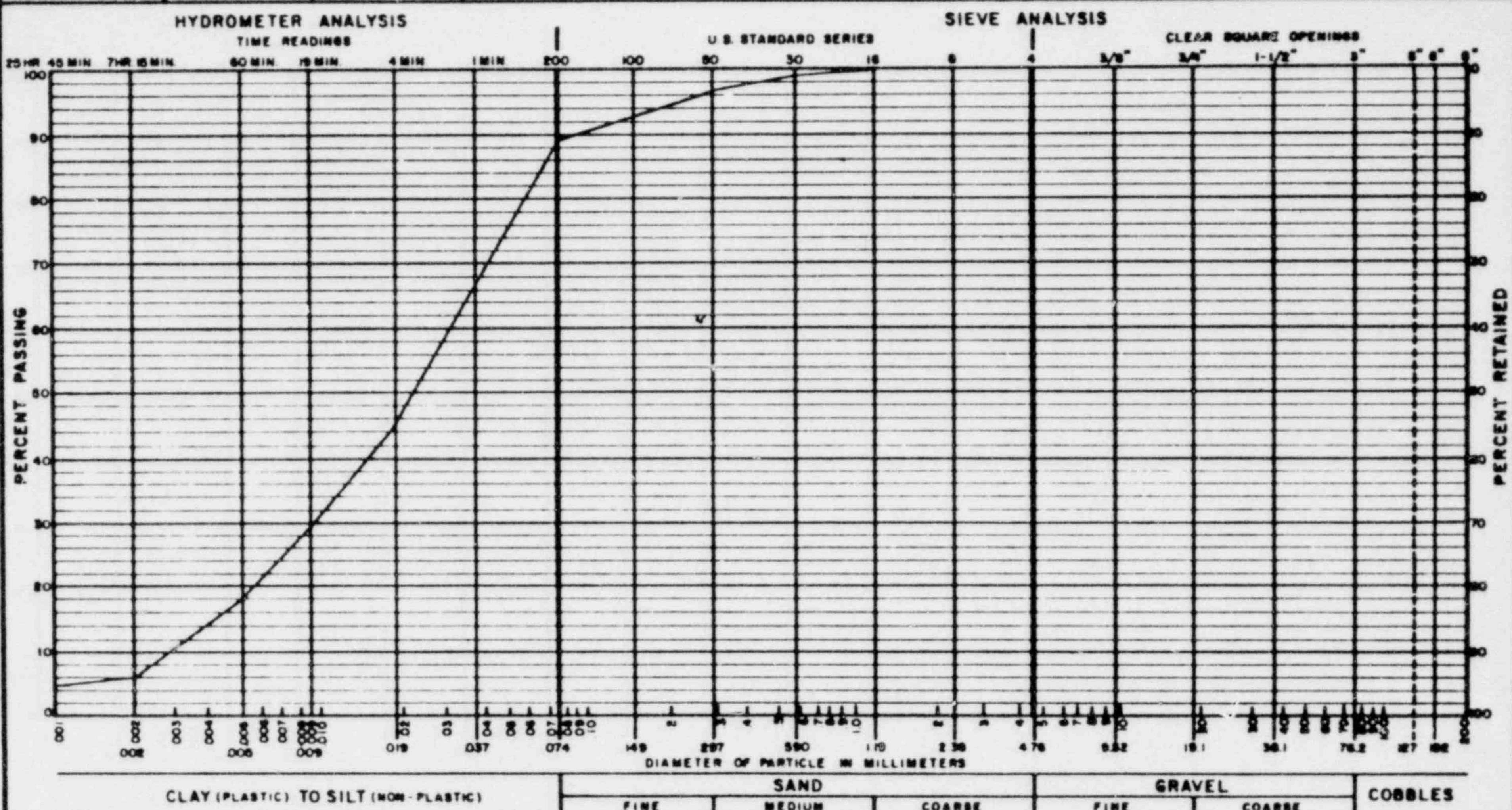
025

SOIL MECHANICS & FOUNDATION ENGINEERS, INC.

785

KEY

LL	36
PL	33
PI	3
NAT. W/C	23.8
CLASSIF. SYMB.	ML
SAMPLE NO.	--
DEPTH FT	32-34
HOLE NO.	106



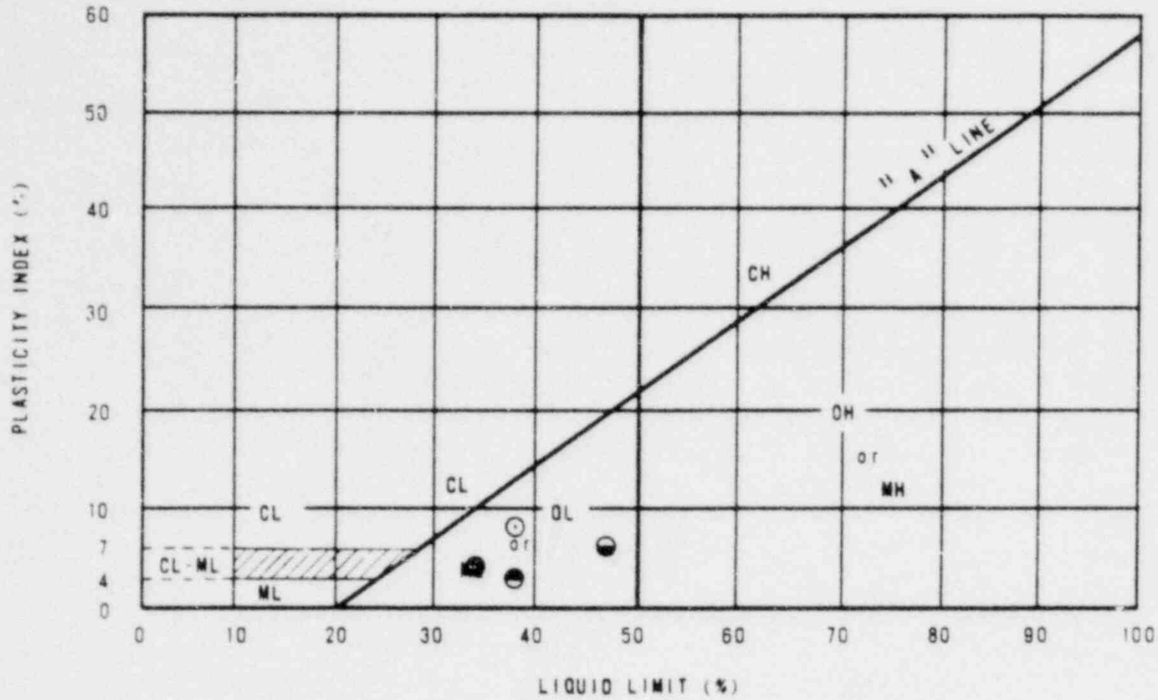
Tested By RLV	Appr. By	Date 11/67	GRADATION TEST RESULTS
Hole No. 106	Area	Job No. 2111	File No. / Fig No. 1

SOIL MECHANICS & FOUNDATION ENGINEERS INC

Sheet 3

026

PLASTICITY CHART



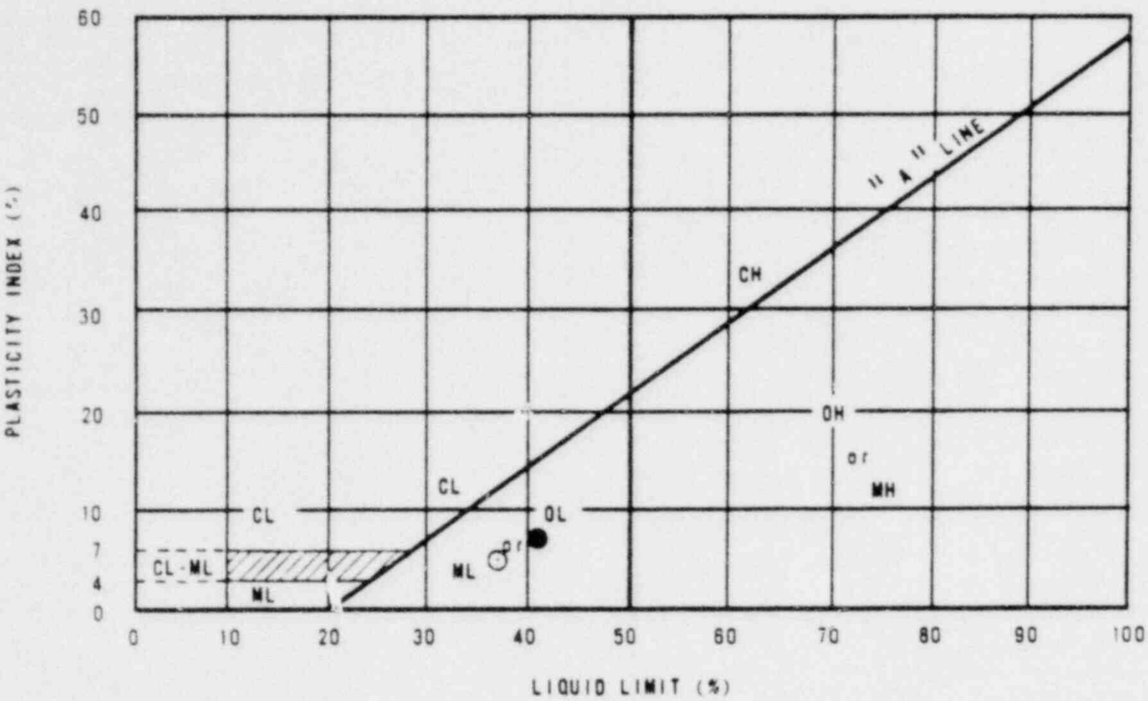
PLASTICITY DATA

KEY SYMBOL	HOLE NO	DEPTH (feet)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	UNIFIED SOIL CLASSIFICATION SYMBOL
○	104	30-32	38	8	ML
●	104	40-42	34	4	ML
◒	104	60-62	47	6	ML
	104	80-82	NP	NP	SM(NP)
◓	104	90-92	38	3	ML
	104	110-112	NP	NP	SW

Project No. 2111

November, 1967

PLASTICITY CHART



PLASTICITY DATA

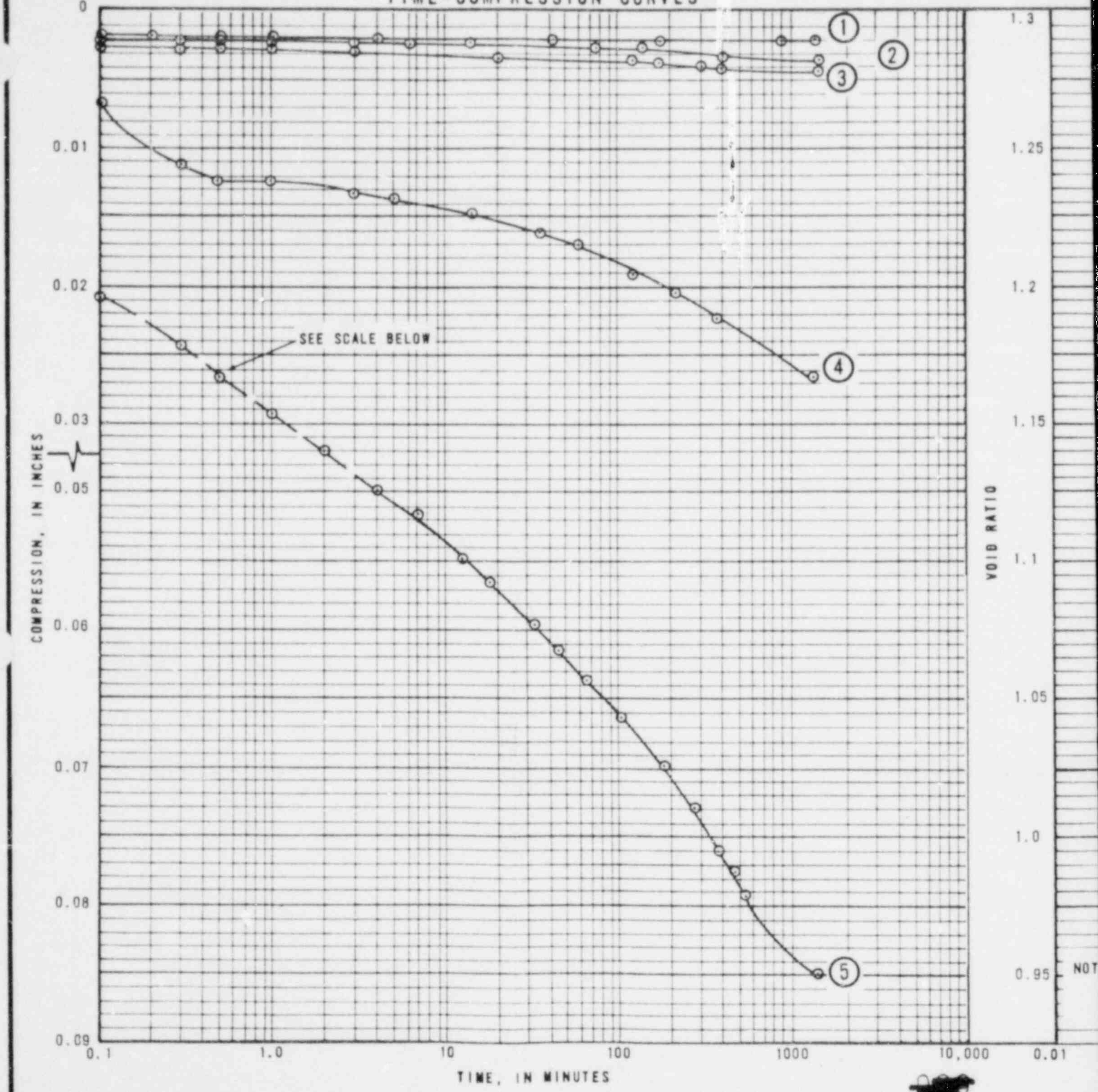
KEY SYMBOL	HOLE NO	DEPTH (feet)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	UNIFIED SOIL CLASSIFICATION SYMBOL
○	105	15-16.8	37	5	ML
●	105	30-32	41	7	ML
	105	55-57	NP	NP	SM(NP)
	105	70-72	32	NP	ML(NP)

028

Project No. 2111 November 1967

740 2

TIME-COMPRESSION CURVES



PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES

CURVE NO.	PRESSURE INCREMENT FROM (KSF)	TO (KSF)
1	2.0	4.0
2	4.0	8.0
3	8.0	16.0
4	16.0	32.0
5	32.0	64.0

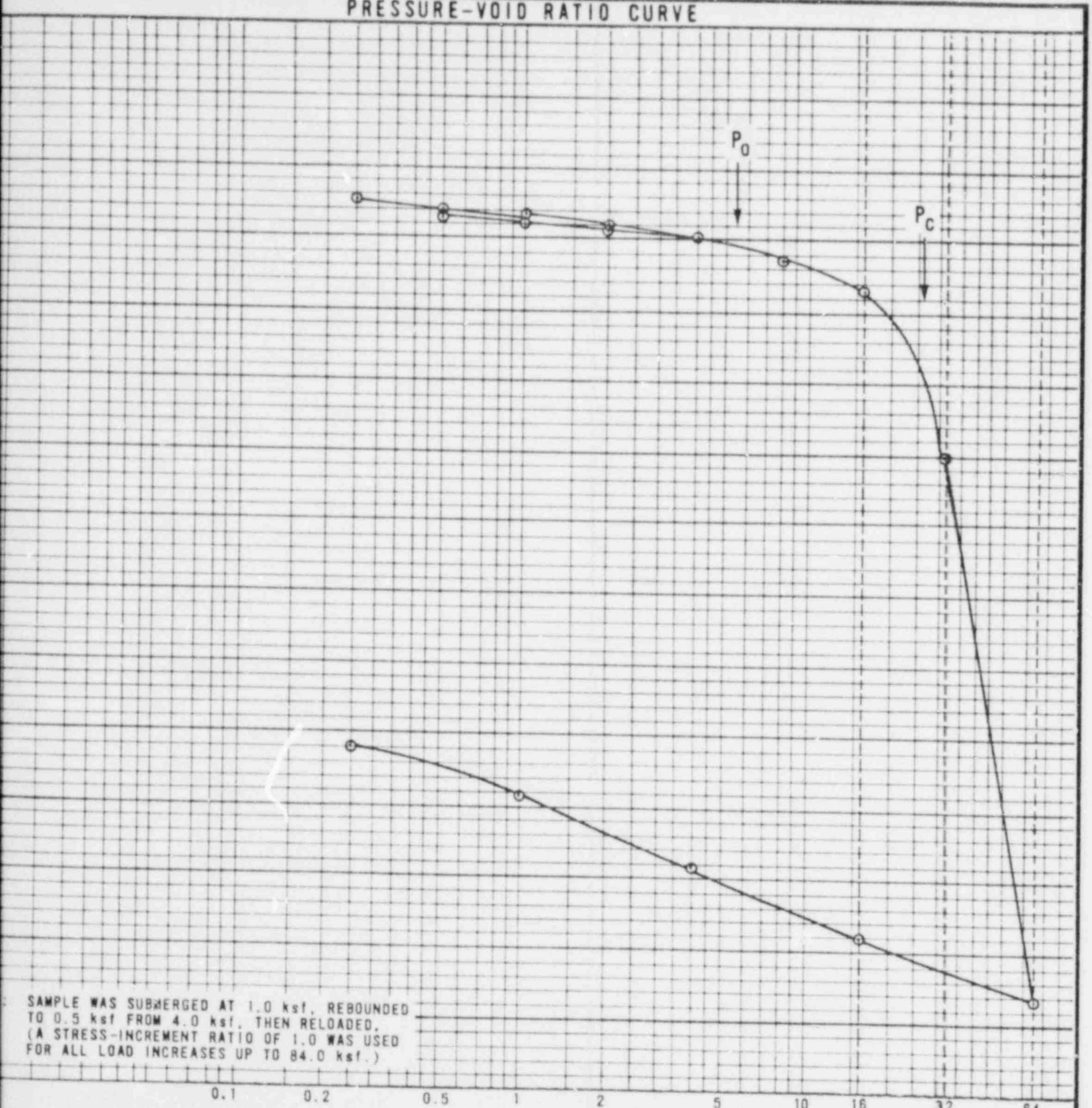
DESCRIPTION OF SPECIMEN: Tan SILT with trace of manganese (ML-MH)

ATTERBERG LIMIT SPECIMEN

STRATUM DESIGNATION	
LIQUID LIMIT (LL)	47
PLASTIC LIMIT (PL)	41
PLASTICITY INDEX (PI)	6
NATURAL WATER CONTENT % (w)	32.8
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	− 0.53
SPECIFIC GRAVITY (G)	2.89

CONSOLIDATION SPECIMEN

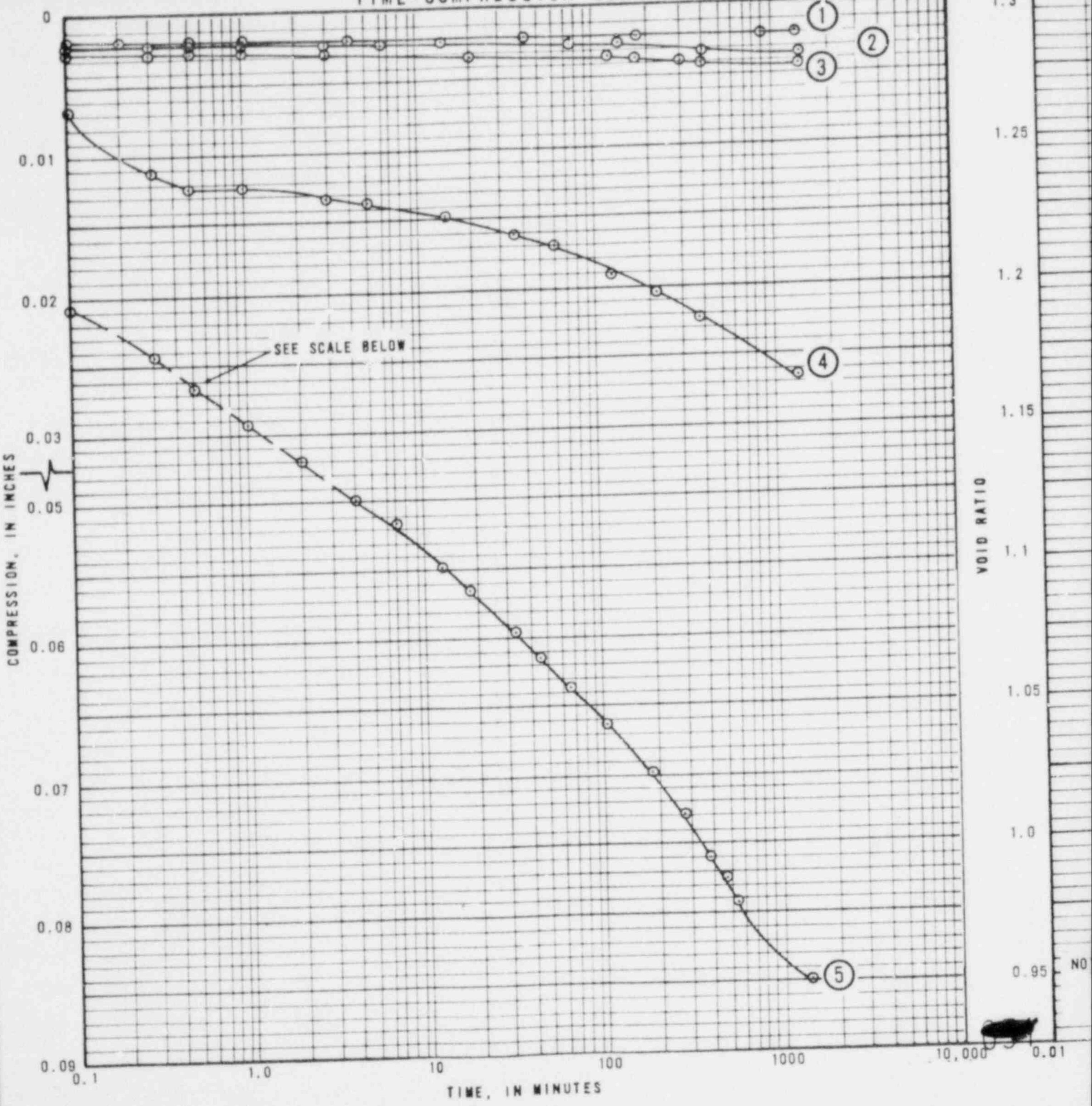
INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inches)	
INITIAL THICK. OF SPEC. (inches)	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION %	
FINAL DEGREE SATURATION %	



030

			HOLE NO. 104	DEPTH (ft) 60-62
74.9	INITIAL VOID RATIO (e_o) ————	1.238	COMPRESSION INDEX (C_c)	0.63
80.8	FINAL VOID RATIO (e_f) - - - - - -	1.046	SWELLING INDEX (C_s), REBOUND FROM e = 1.004	0.035
2.50	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL $P_c \downarrow$	26	COEFFICIENT OF CONSOL. (C_v) $cm^2/sec.$	
1.000	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL $P_o \downarrow$	5.3	RECOMPRESSION INDEX (C_r)	0.008
32.8				
36.2	SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATION TEST	SAMPLE NO.
71			PROJECT NO: 2111 / DATE: NOV. 1967 / DRAWING NO: FIGURE 3	5
92.9	PALO ALTO • WHITTIER • CALIF.			

TIME-COMPRESSION CURVES



PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES

CURVE NO.	PRESSURE INCREMENT FROM (KSF)	TO (KSF)
①	2.0	4.0
②	4.0	8.0
③	8.0	16.0
④	16.0	32.0
⑤	32.0	64.0

DESCRIPTION OF SPECIMEN: Tan SILT with trace of manganese (ML-MH)

ATTERBERG LIMIT SPECIMEN

STRATUM DESIGNATION	
LIQUID LIMIT (LL)	47
PLASTIC LIMIT (PL)	41
PLASTICITY INDEX (PI)	6
NATURAL WATER CONTENT % (w)	32.8
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	- 0.53
SPECIFIC GRAVITY (G)	2.69

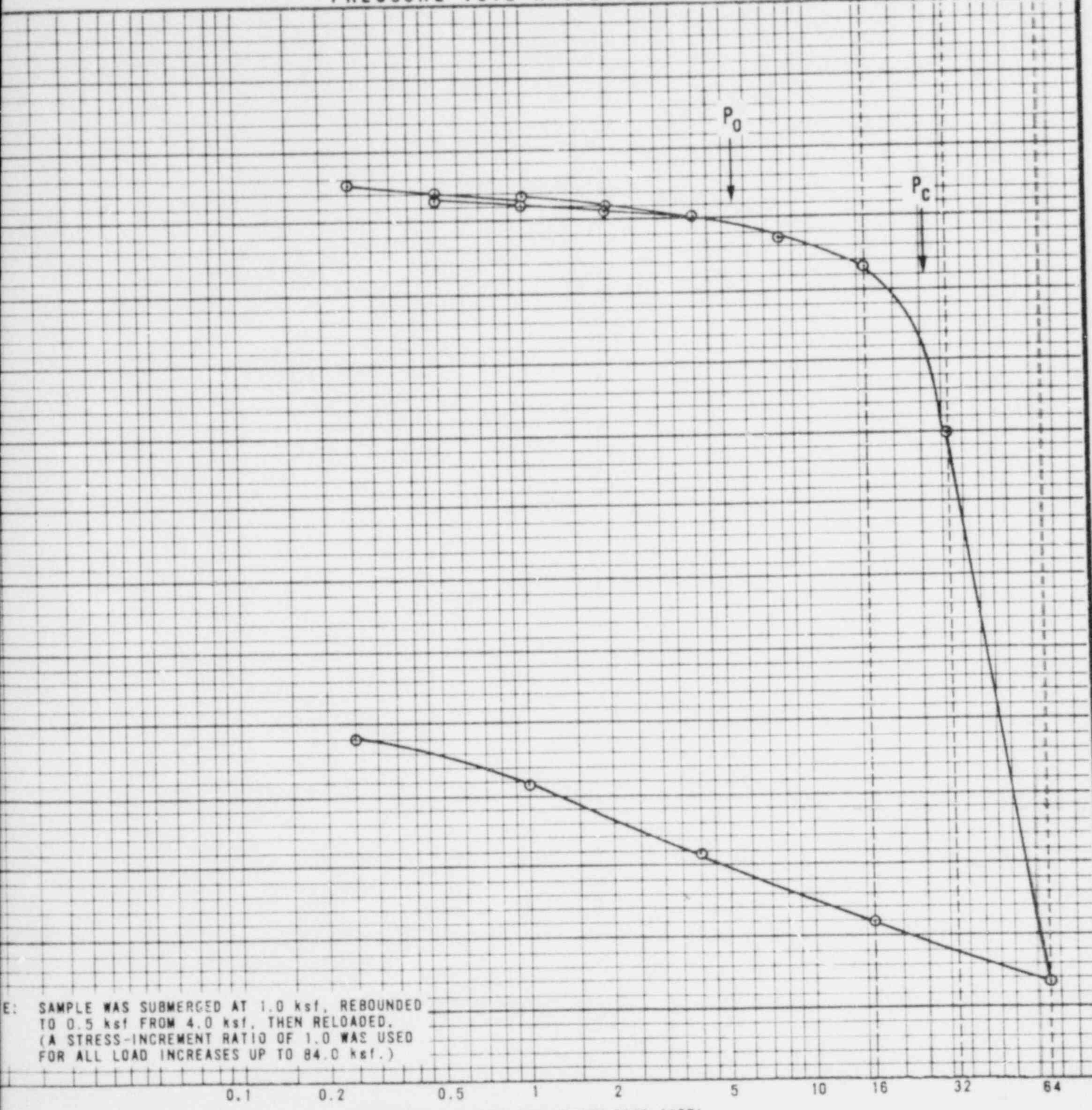
CONSOLIDATION SPECIMEN

INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inches)	
INITIAL THICK. OF SPEC. (inches)	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION %	
FINAL DEGREE SATURATION %	

853A

31

PRESSURE-VOID RATIO CURVE

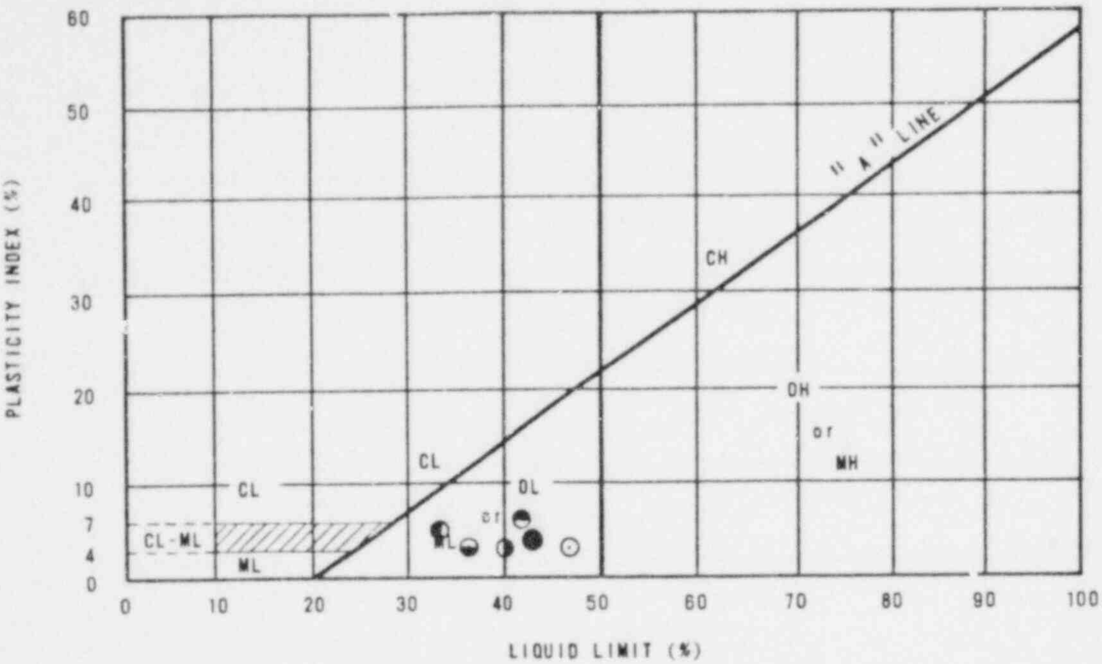


			HOLE NO. 104	DEPTH (ft) 60-62
74.9	INITIAL VOID RATIO (e_o)	1.238	COMPRESSION INDEX (C_c)	0.63
80.8	FINAL VOID RATIO (e_f)	1.046	SWELLING INDEX (C_s), REBOUND FROM $e = 1.004$	0.035
2.50	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c ↓	26	COEFFICIENT OF CONSOL. (C_v) cm^2/sec.	
1.000	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o ↓	5.3	RECOMPRESSION INDEX (C_r)	0.008

32.8	SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATION TEST		SAMPLE NO. 5
36.2			PROJECT NO.	DATE	DRAWING NO.
71			2111	NOV. 1967	FIGURE 3
92.9		PALO ALTO • WHITTIER • CALIF.			

32

PLASTICITY CHART



PLASTICITY DATA

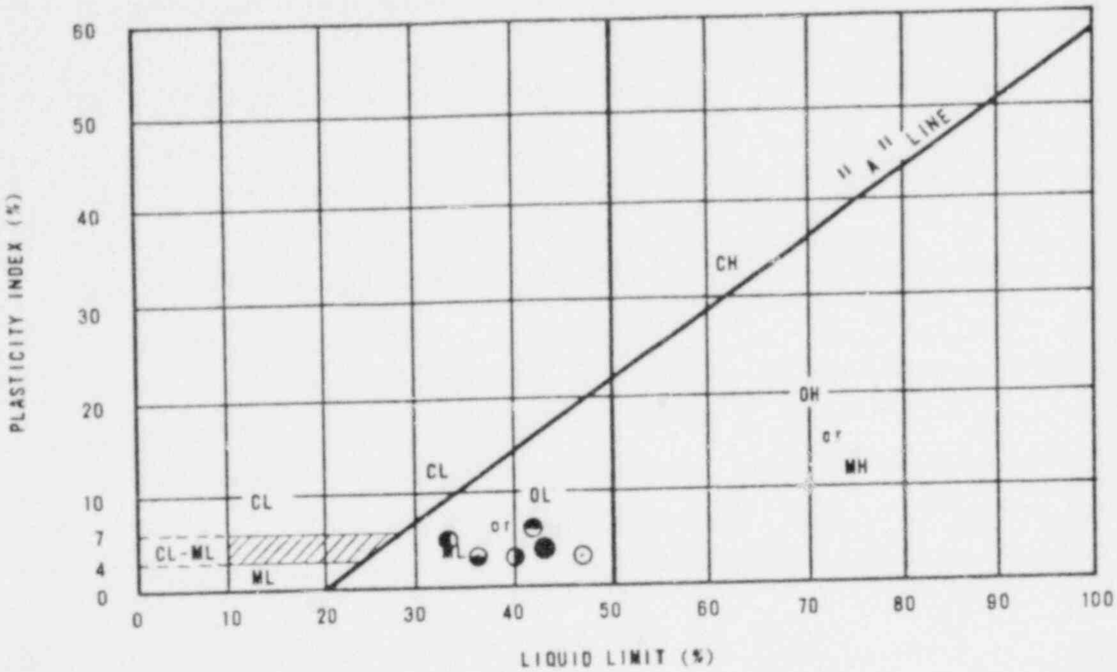
KEY SYMBOL	HOLE NO	DEPTH (feet)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	UNIFIED SOIL CLASSIFICATION SYMBOL
○	106	10-12	47	3	ML
●	106	20-22	43	4	ML
◒	106	30-32	36	3	ML
◓	106	60-62	42	6	ML
◐	106	70-72	33	5	ML
◑	106	80-82	40	3	ML
	106	90-92	NP	NP	SM(NP)
	106	100-102	NP	NP	SP

033

Project No. 2111 November 1967

740-2

PLASTICITY CHART



PLASTICITY DATA

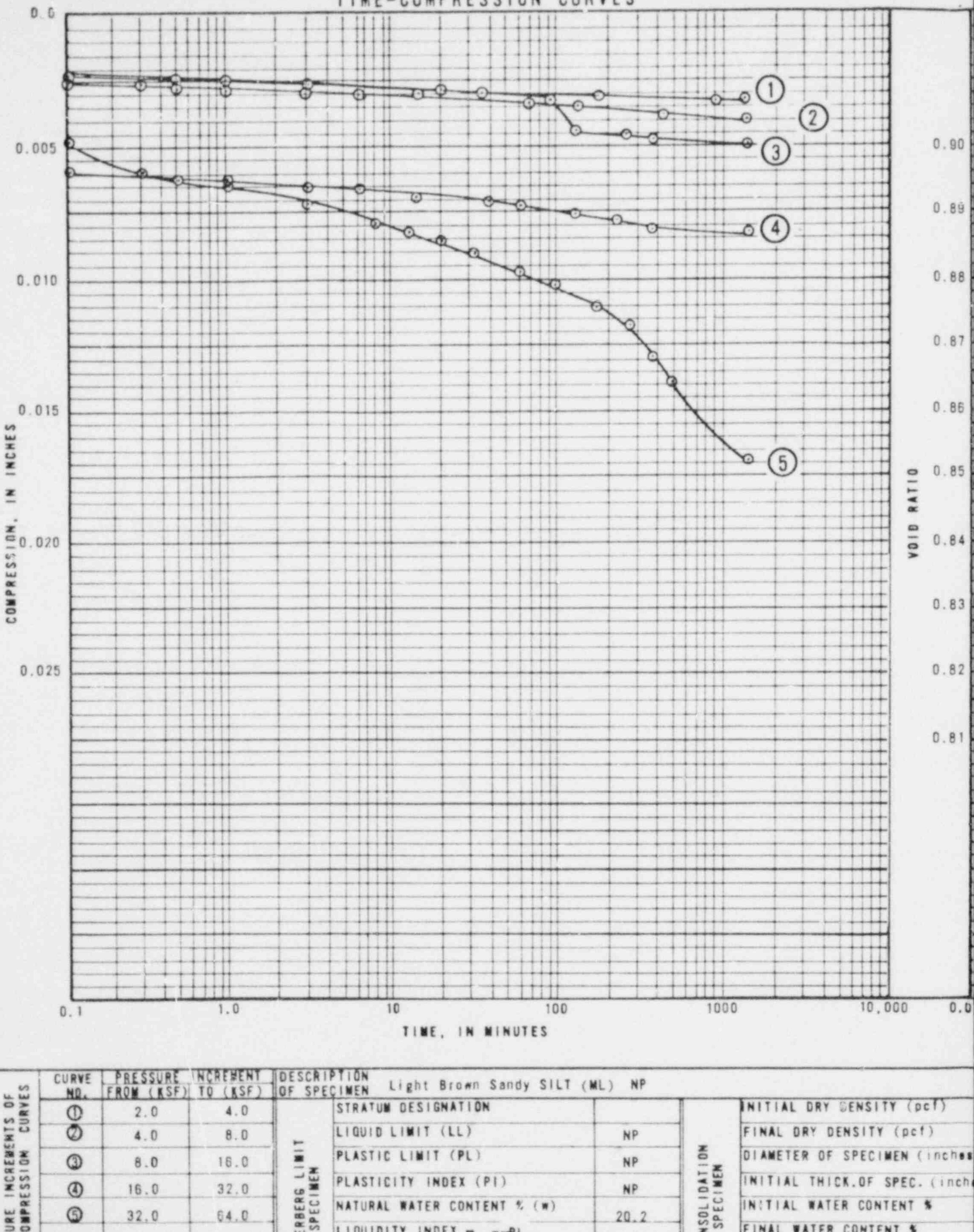
KEY SYMBOL	HOLE NO	DEPTH (feet)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	UNIFIED SOIL CLASSI-FICATION SYMBOL
○	106	10-12	47	3	ML
●	106	20-22	43	4	ML
◒	106	30-32	36	3	ML
◓	106	60-62	42	6	ML
◐	106	70-72	33	5	ML
◑	106	80-82	40	3	ML
	106	90-92	NP	NP	SM(NP)
	106	100-102	NP	NP	SP

034

Project No. 2111 November 1967

740-2

TIME-COMPRESSION CURVES



PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES

CURVE NO.	PRESSURE FROM (KSF)	INCREMENT TO (KSF)
①	2.0	4.0
②	4.0	8.0
③	8.0	16.0
④	16.0	32.0
⑤	32.0	64.0

DESCRIPTION OF SPECIMEN Light Brown Sandy SILT (ML) NP

ATTERBERG LIMIT SPECIMEN

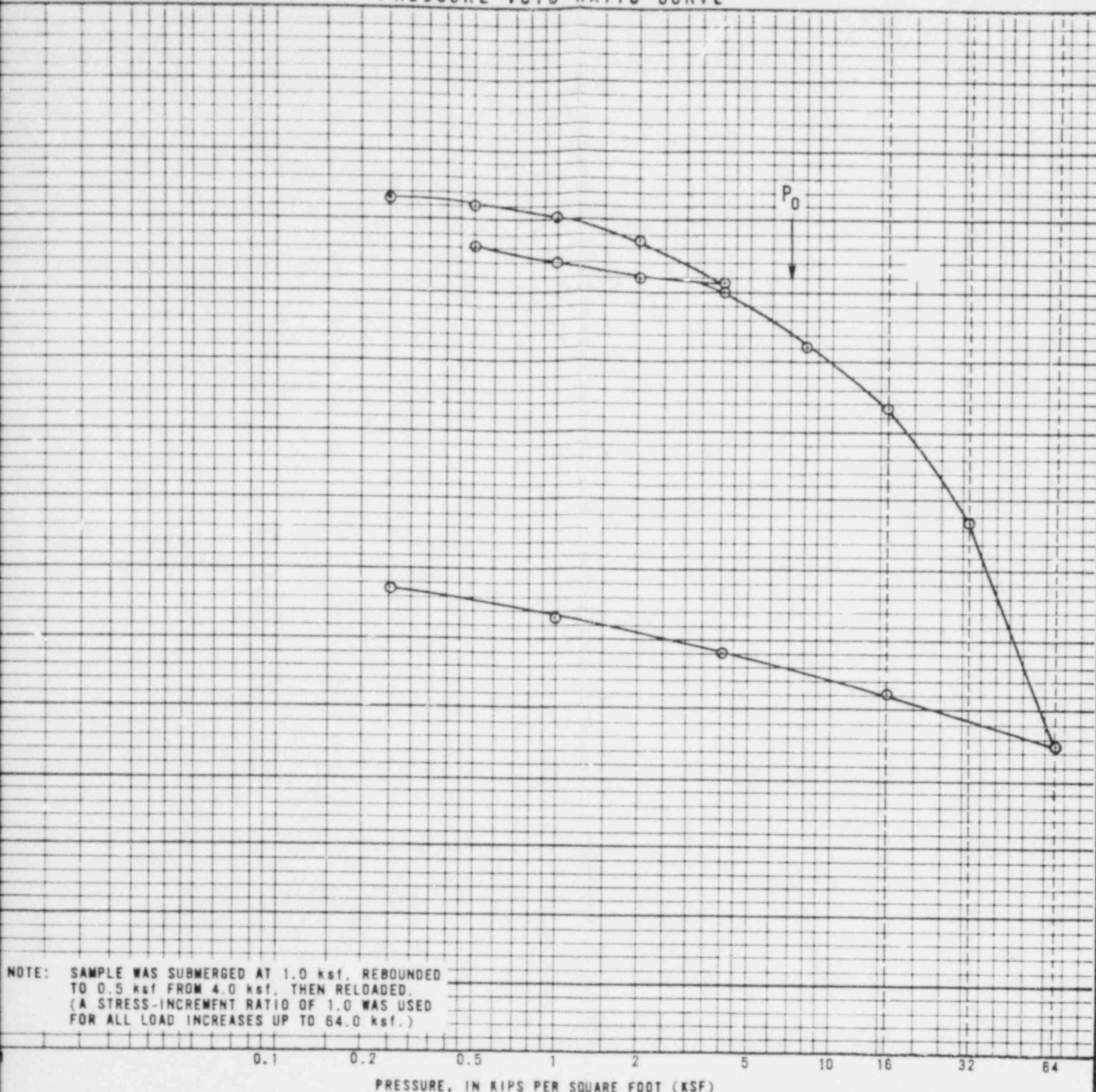
STRATUM DESIGNATION	
LIQUID LIMIT (LL)	NP
PLASTIC LIMIT (PL)	NP
PLASTICITY INDEX (PI)	NP
NATURAL WATER CONTENT % (w)	20.2
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	----
SPECIFIC GRAVITY (G)	2.68

CONSOLIDATION SPECIMEN

INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inches	
INITIAL THICK.OF SPEC. (inch	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION %	
FINAL DEGREE SATURATION %	

853A

PRESSURE-VOID RATIO CURVE

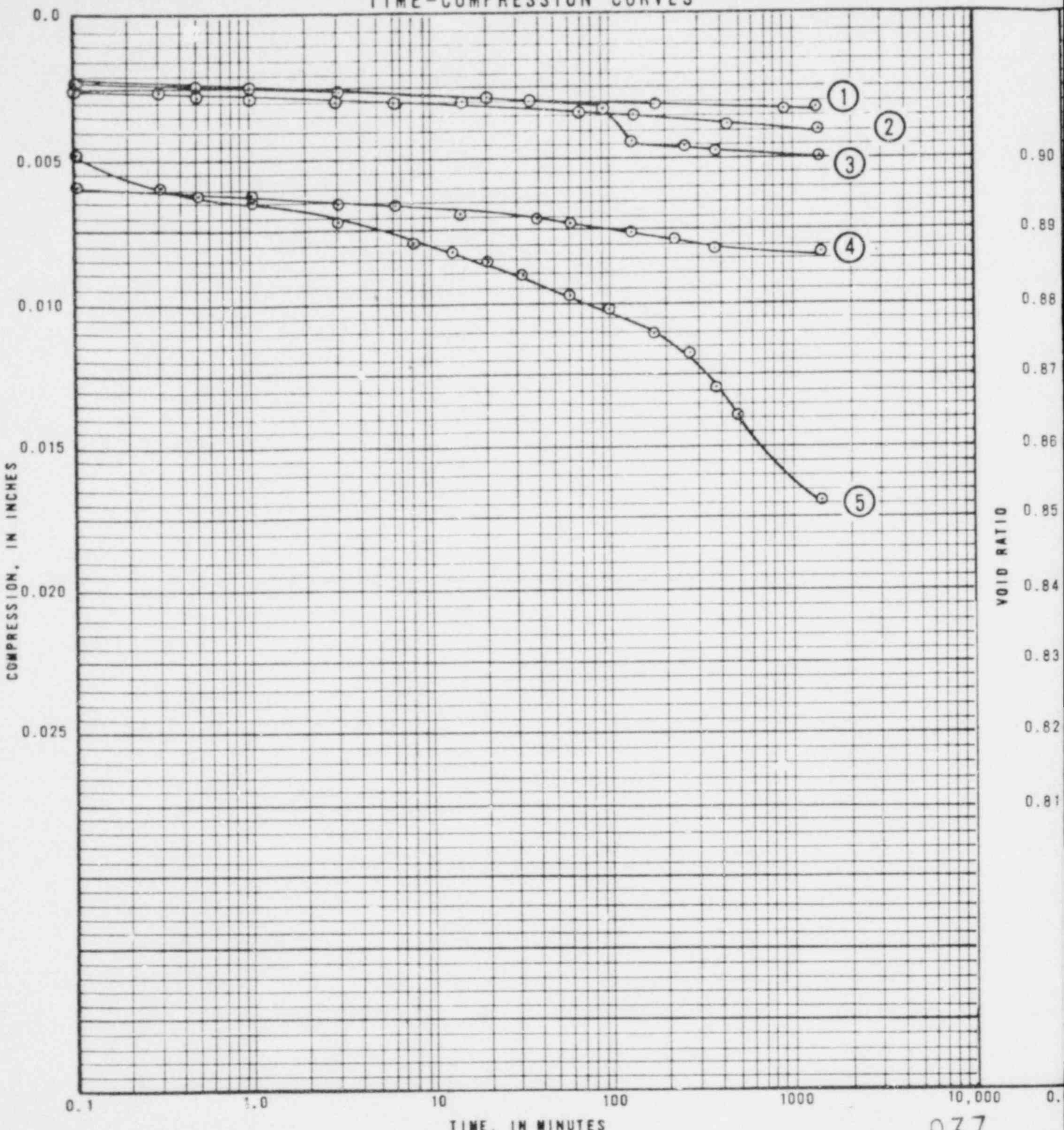


				HOLE NO. 104	DEPTH (ft) 80-82
88.3	INITIAL VOID RATIO (e_o)	0.894	COMPRESSION INDEX (C_c)		0.11
90.1	FINAL VOID RATIO (e_f)	0.837	SWELLING INDEX (C_s), REBOUND FROM e = 0.828		0.008
2.50	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c↓		COEFFICIENT OF CONSOL. (C_v) cm^2/sec.		
s) 1.00	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o↓	7.0	RECOMPRESSION INDEX (C_r)		0.007
20.2					

27.7	SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATION TEST		SAMPLE NO.
60.5			PROJECT NO.	DATE	DRAWING NO.
88.5	PALO ALTO • WHITTIER • CALIF.		2111	NOV. 1967	FIGURE 4
					7

36

TIME-COMPRESSION CURVES



037

PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES

CURVE NO.	PRESSURE INCREMENT FROM (KSF)	TO (KSF)
1	2.0	4.0
2	4.0	8.0
3	8.0	16.0
4	16.0	32.0
5	32.0	64.0

DESCRIPTION OF SPECIMEN Light Brown Sandy SILT (ML) NP

ATTERBERG LIMIT SPECIMEN

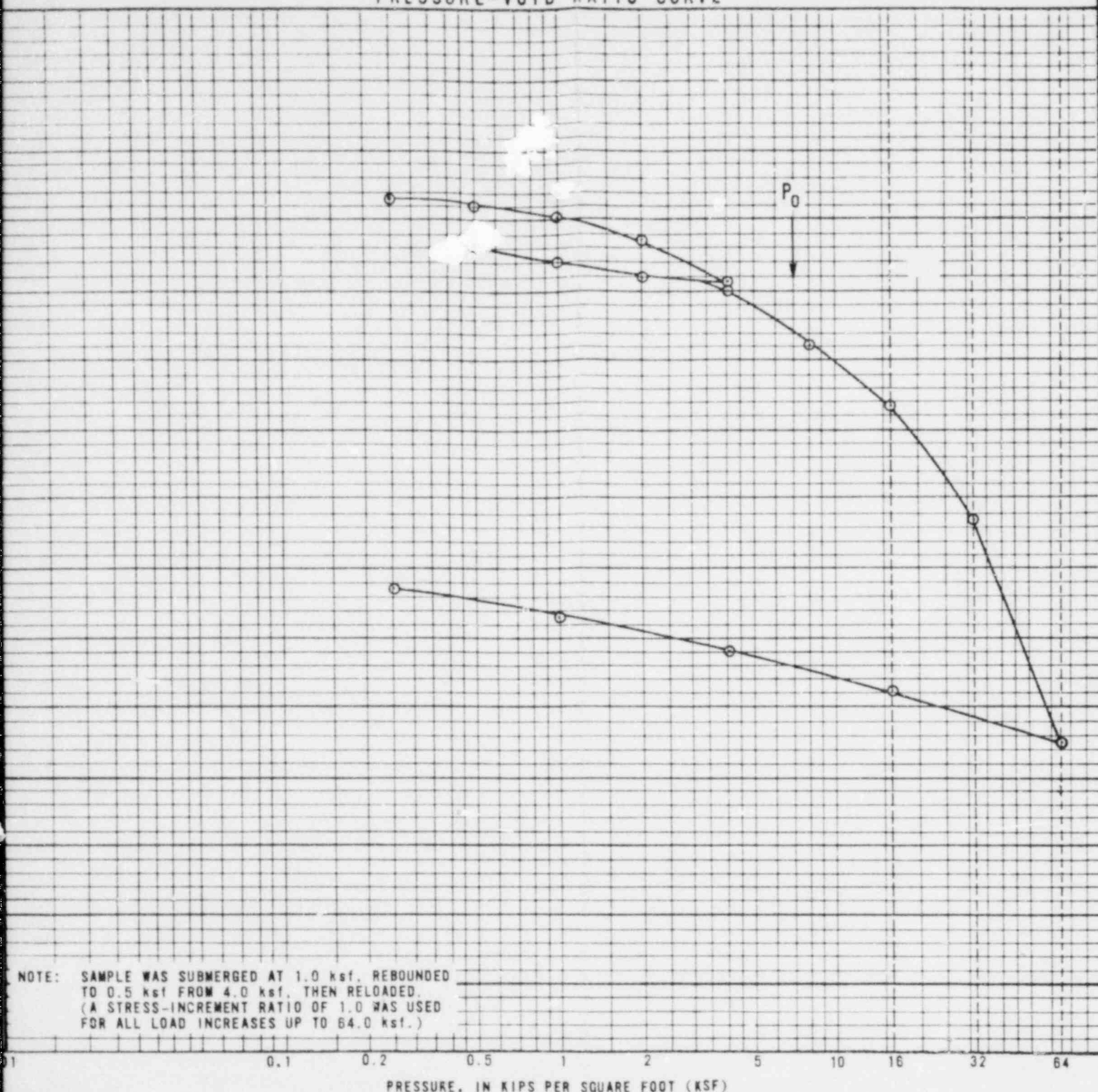
STRATUM DESIGNATION	
LIQUID LIMIT (LL)	NP
PLASTIC LIMIT (PL)	NP
PLASTICITY INDEX (PI)	NP
NATURAL WATER CONTENT % (w)	20.2
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	----
SPECIFIC GRAVITY (G)	2.68

CONSOLIDATION SPECIMEN

INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inche	
INITIAL THICK. OF SPEC. (inc	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION %	
FINAL DEGREE SATURATION %	

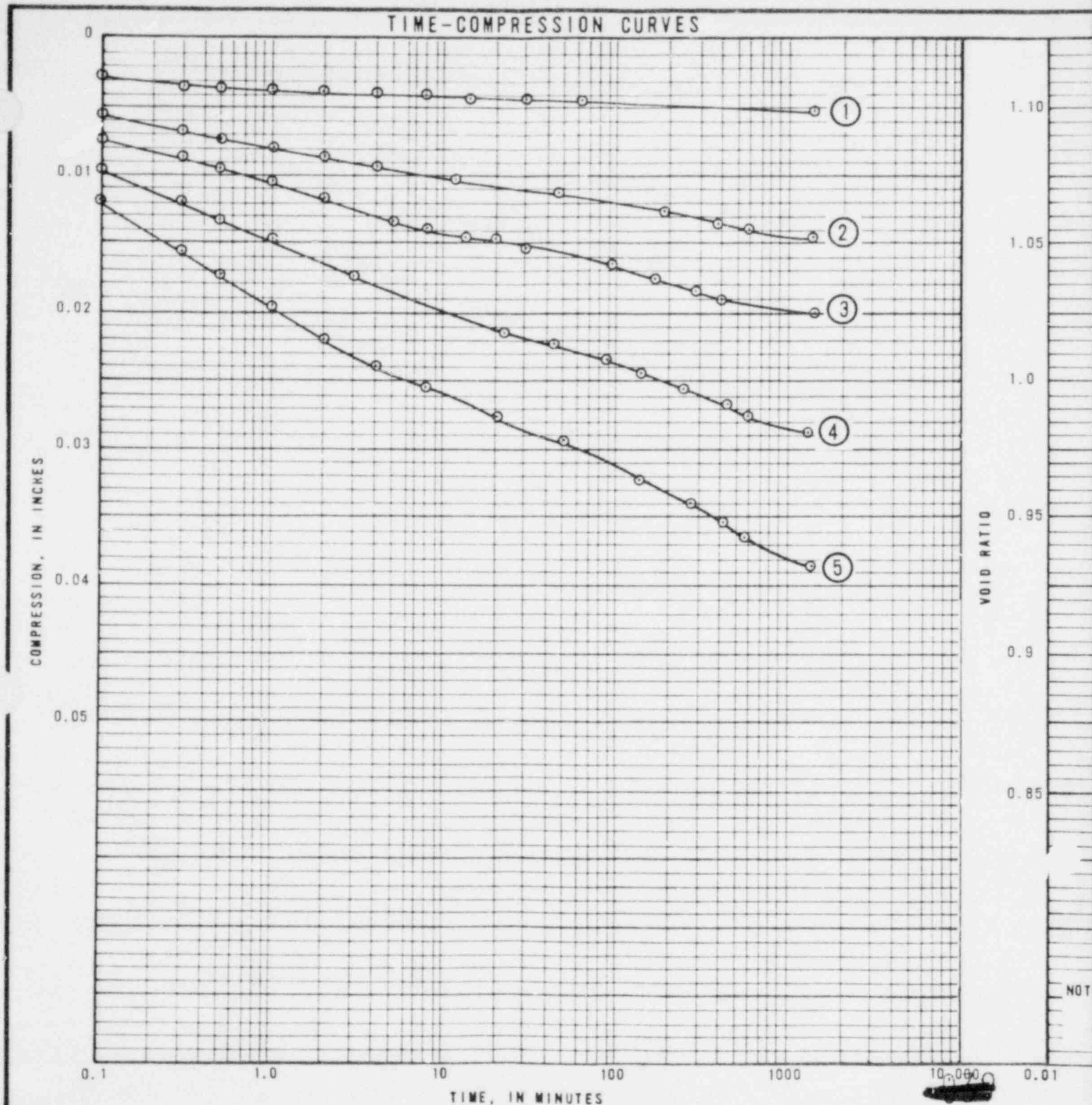
853A

PRESSURE–VOID RATIO CURVE



			HOLE NO. 104	DEPTH (ft) 80-82
88.3	INITIAL VOID RATIO (e_o)	0.894	COMPRESSION INDEX (C_c)	0.11
90.1	FINAL VOID RATIO (e_f)	0.837	SWELLING INDEX (C_s), REBOUND FROM e = 0.828	0.008
s) 2.50	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c↓		COEFFICIENT OF CONSOL. (C_v) cm^2/sec.	
hes) 1.00	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o↓	7.0	RECOMPRESSION INDEX (C_r)	0.007

20.2	SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATION TEST		SAMPLE NO.
27.7					
60.5			PROJECT NO.	DATE	DRAWING NO.
88.5		PALO ALTO • WHITTIER • CALIF.	2111	NOV. 1967	FIGURE 4	7



PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES

CURVE NO.	PRESSURE INCREMENT FROM (KSF)	PRESSURE INCREMENT TO (KSF)
1	2.0	4.0
2	4.0	8.0
3	8.0	16.0
4	16.0	32.0
5	32.0	64.0

DESCRIPTION OF SPECIMEN Tan, very fine, sandy SILT (ML)

ATTERBERG LIMIT SPECIMEN

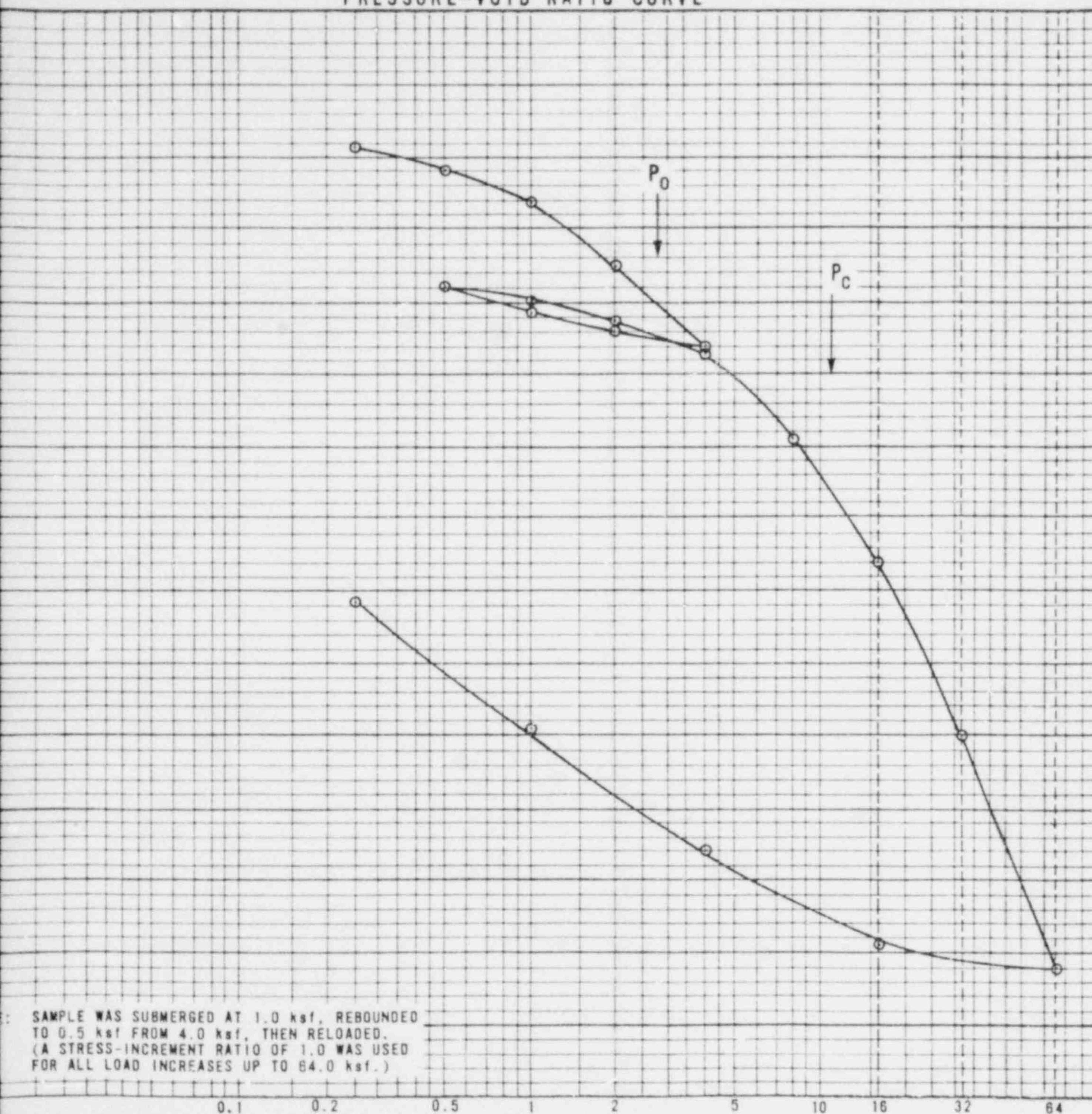
STRATUM DESIGNATION	
LIQUID LIMIT (LL)	41
PLASTIC LIMIT (PL)	34
PLASTICITY INDEX (PI)	7
NATURAL WATER CONTENT % (w)	32.6
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	− 0.2
SPECIFIC GRAVITY (G)	2.77

CONSOLIDATION SPECIMEN

INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inches)	
INITIAL THICK. OF SPEC. (inches)	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION %	
FINAL DEGREE SATURATION %	

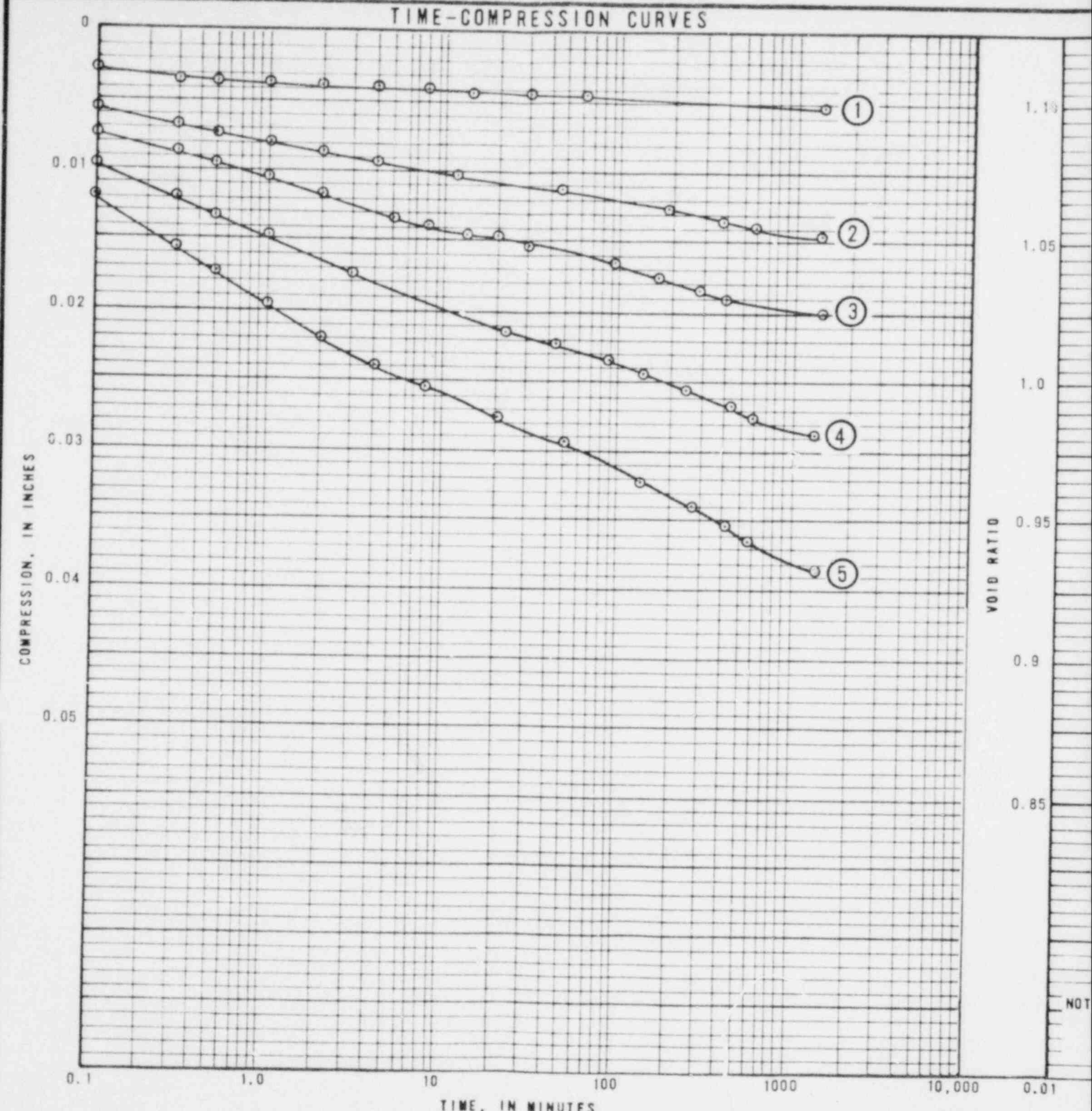
853A

PRESSURE-VOID RATIO CURVE



			HOLE NO. 105	DEPTH (ft) 30-32	
62.9	INITIAL VOID RATIO (e_o)	1.081	COMPRESSION INDEX (C_c)		0.27
90.2	FINAL VOID RATIO (e_f)	0.922	SWELLING INDEX (C_s), REBOUND FROM e = 0.835		0.072
2.50	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c	11.0	COEFFICIENT OF CONSOL. (C_v) cm^2/sec.		
1.00	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o	2.8	RECOMPRESSION INDEX (C_r)		0.027
32.6					

32.4	SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATION TEST		SAMPLE NO.
81			PROJECT NO.	DATE	DRAWING NO.
97	PALO ALTO • WHITTIER • CALIF.		2111	NOV. 1967	FIGURE 5
					11



[redacted]

PRESS. INCREMENTS OF TIME-COMPRESSION CURVES

CURVE NO.	PRESSURE INCREMENT FROM (KSF)	PRESSURE INCREMENT TO (KSF)
1	2.0	4.0
2	4.0	8.0
3	8.0	16.0
4	16.0	32.0
5	32.0	64.0

DESCRIPTION OF SPECIMEN Tan, very fine, sandy SILT (ML)

ATTERBERG LIMIT SPECIMEN

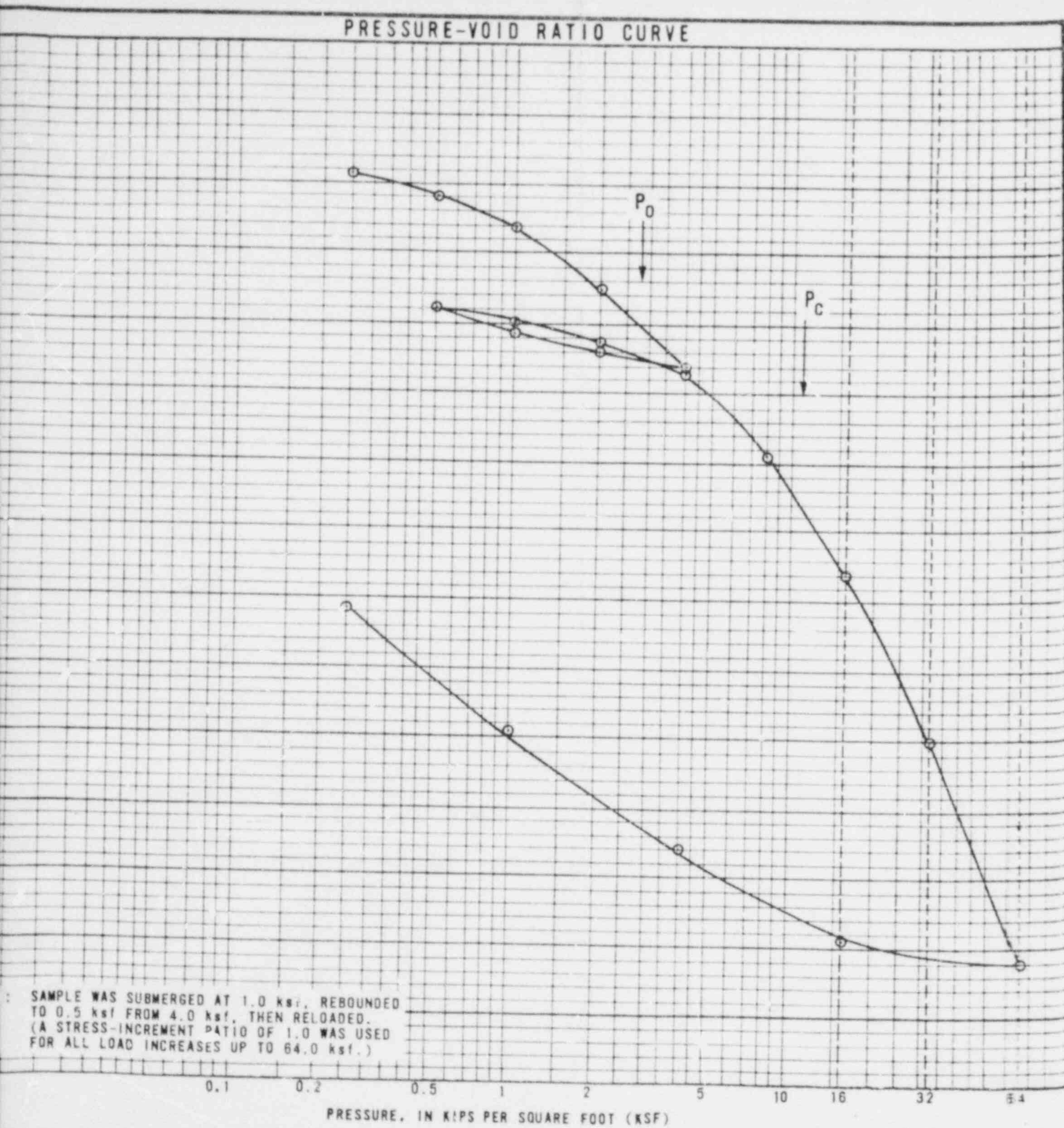
STRATUM DESIGNATION	
LIQUID LIMIT (LL)	41
PLASTIC LIMIT (PL)	34
PLASTICITY INDEX (PI)	7
NATURAL WATER CONTENT % (w)	32.6
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	− 0.2
SPECIFIC GRAVITY (G)	2.77

CONSOLIDATION SPECIMEN

INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inches)	
INITIAL THICK. OF SPEC. (inches)	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION %	
FINAL DEGREE SATURATION %	

853A

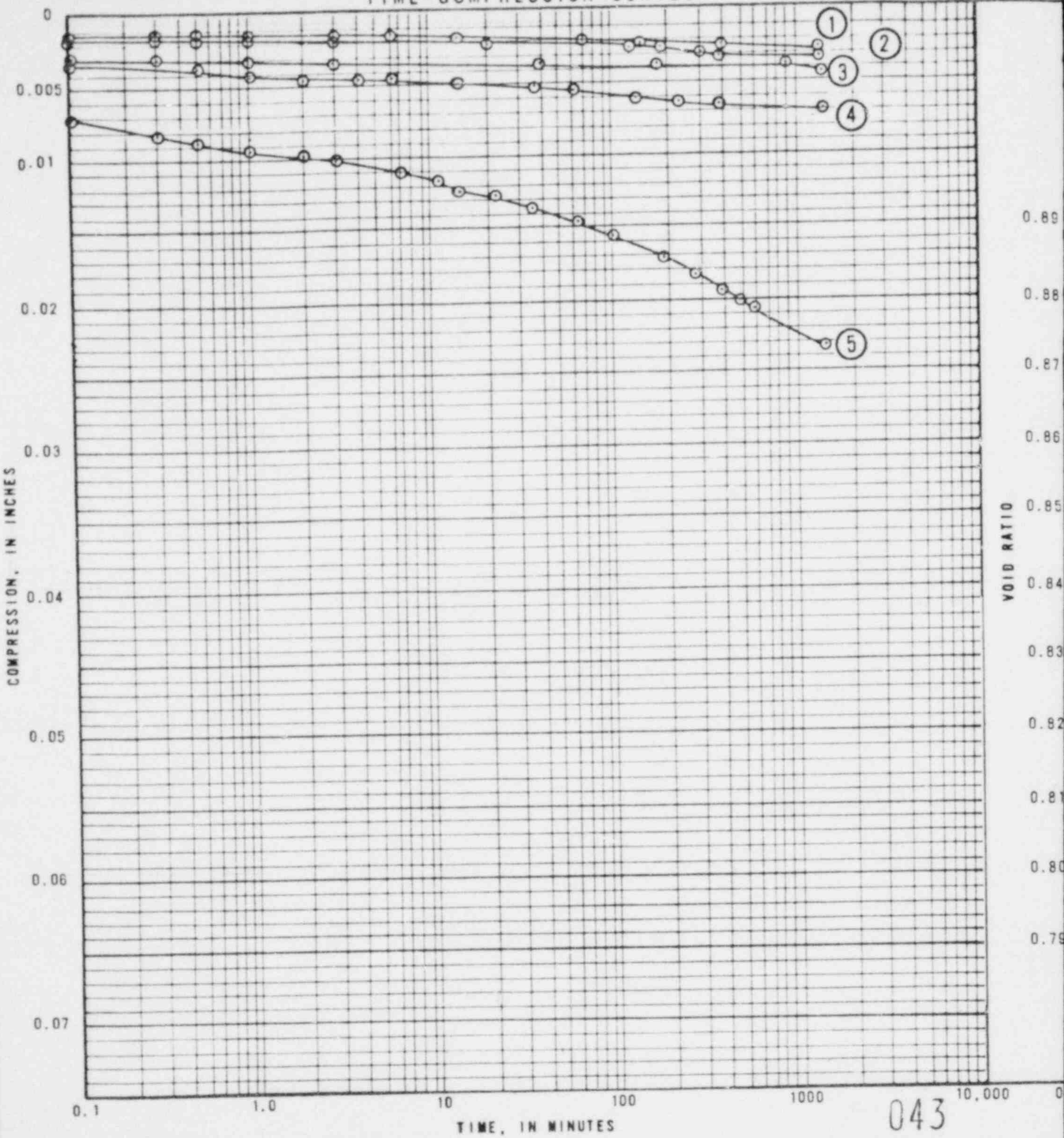
41



			HOLE NO. 105	DEPTH (ft) 30-32	
82.9	INITIAL VOID RATIO (e_o)	1.081	COMPRESSION INDEX (C_c)		0.27
90.2	FINAL VOID RATIO (e_f)	0.922	SWELLING INDEX (C_s), REBOUND FROM e = 0.835		0.072
2.50	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c	11.0	COEFFICIENT OF CONSOL. (C_v) cm^2/sec.		
1.00	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o	2.8	RECOMPRESSION INDEX (C_r)		0.027
32.6					
32.4	SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATION TEST		SAMPLE NO.
81			PROJECT NO. 2111	DATE NOV. 1967	DRAWING NO FIGURE 5
97	PALO ALTO • WHITTIER • CALIF.				11

42

TIME-COMPRESSION CURVES



043

PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES

CURVE NO.	PRESSURE INCREMENT FROM (KSF)	TO (KSF)
①	2.0	4.0
②	4.0	8.0
③	8.0	16.0
④	16.0	32.0
⑤	32.0	64.0

DESCRIPTION OF SPECIMEN Light Brown, silty, fine SAND (SM)

ATTERBERG LIMIT SPECIMEN

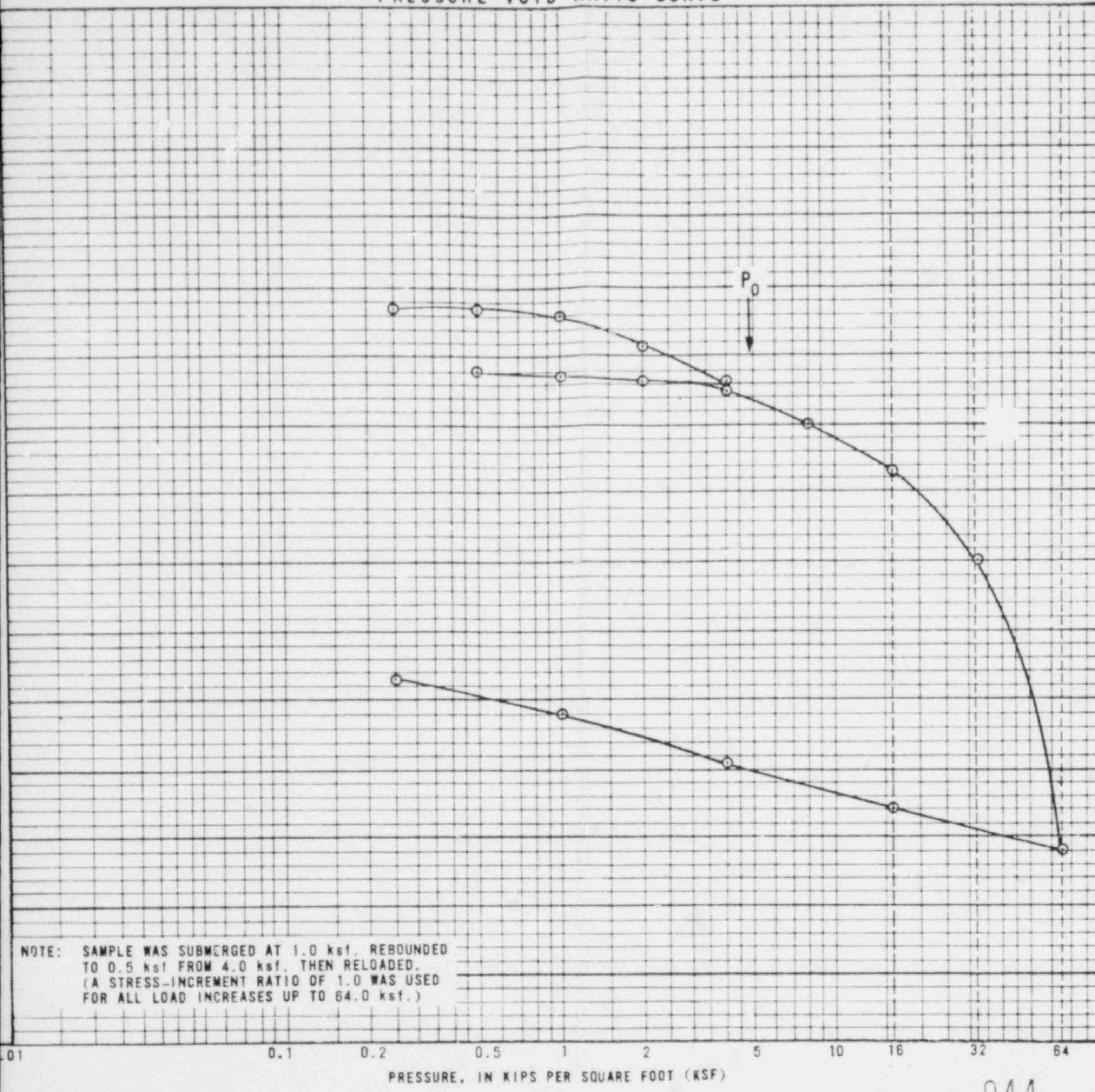
STRATUM DESIGNATION	
LIQUID LIMIT (LL)	NP
PLASTIC LIMIT (PL)	NP
PLASTICITY INDEX (PI)	NP
NATURAL WATER CONTENT % (w)	14.9
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	----
SPECIFIC GRAVITY (G)	2.70

CONSOLIDATION SPECIMEN

INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inch	
INITIAL THICK. OF SPEC. (in	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION	
FINAL DEGREE SATURATION %	

853A

PRESSURE-VOID RATIO CURVE



044

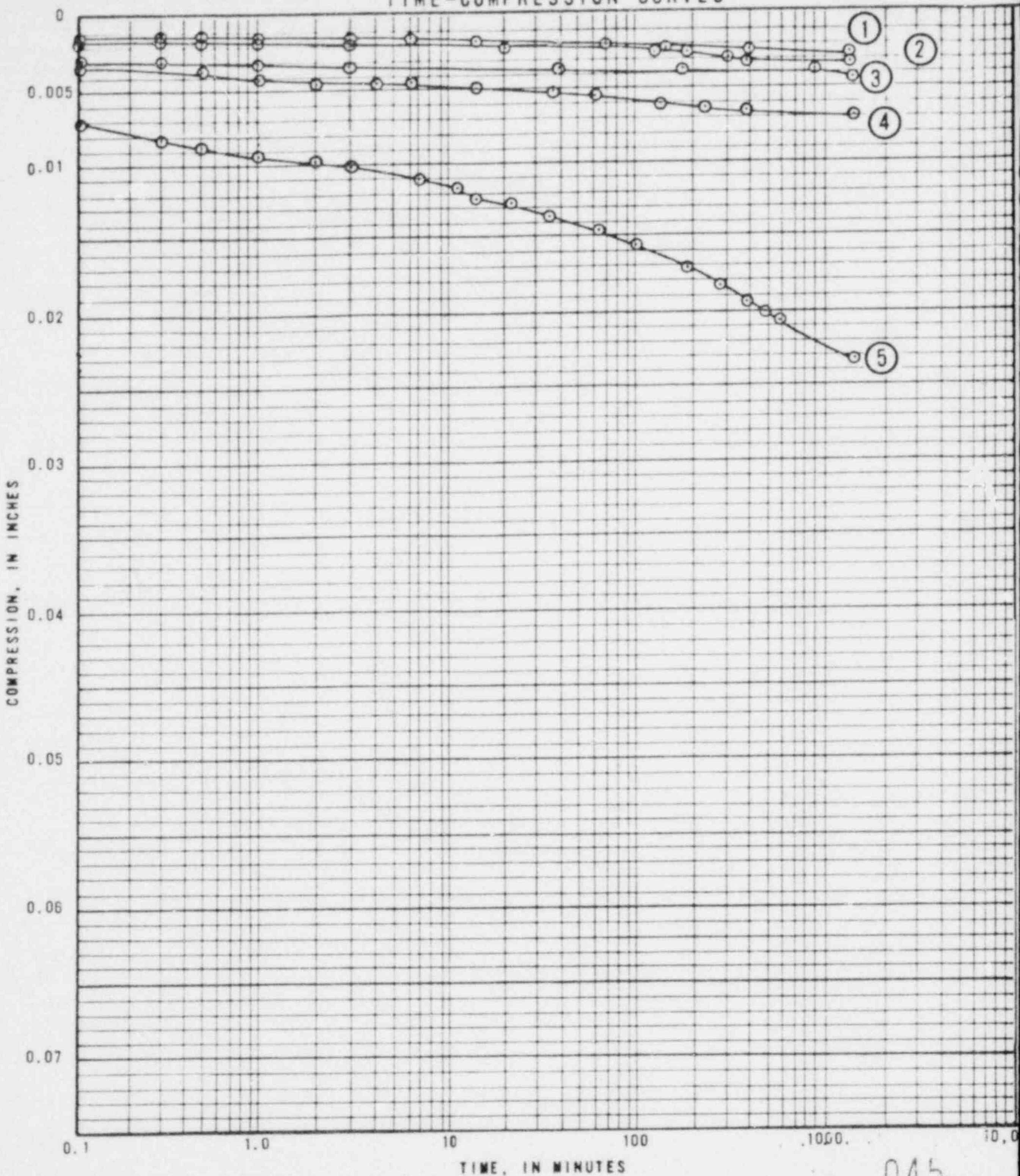
				HOLE NO. 105	DEPTH (ft) 55-57
	89.8	INITIAL VOID RATIO (e_o)	0.878	COMPRESSION INDEX (C_c)	0.14
	90.9	FINAL VOID RATIO (e_f)	0.823	SWELLING INDEX (C_s), REBOUND FROM e = 0.811	0.012
es)	2.5	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c↓		COEFFICIENT OF CONSOL. (C_v) cm^2/sec.	
ches)	1.00	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o↓	4.9	RECOMPRESSION INDEX (C_r)	0.003
	14.9				
	24.4				
s	46				
	80				

SOIL MECHANICS and FOUNDATION ENGINEERS Inc. PALO ALTO • WHITTIER • CALIF.

RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1

CONSOLIDATION TEST			SAMPLE NO.
PROJECT NO.	DATE	DRAWING NO.	
2111	NOV. 1967	FIGURE 6	21

TIME-COMPRESSION CURVES



045

PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES

CURVE NO.	PRESSURE INCREMENT FROM (KSF)	TO (KSF)
1	2.0	4.0
2	4.0	8.0
3	8.0	16.0
4	16.0	32.0
5	32.0	64.0

DESCRIPTION OF SPECIMEN Light Brown, silty, fine SAND (SM)

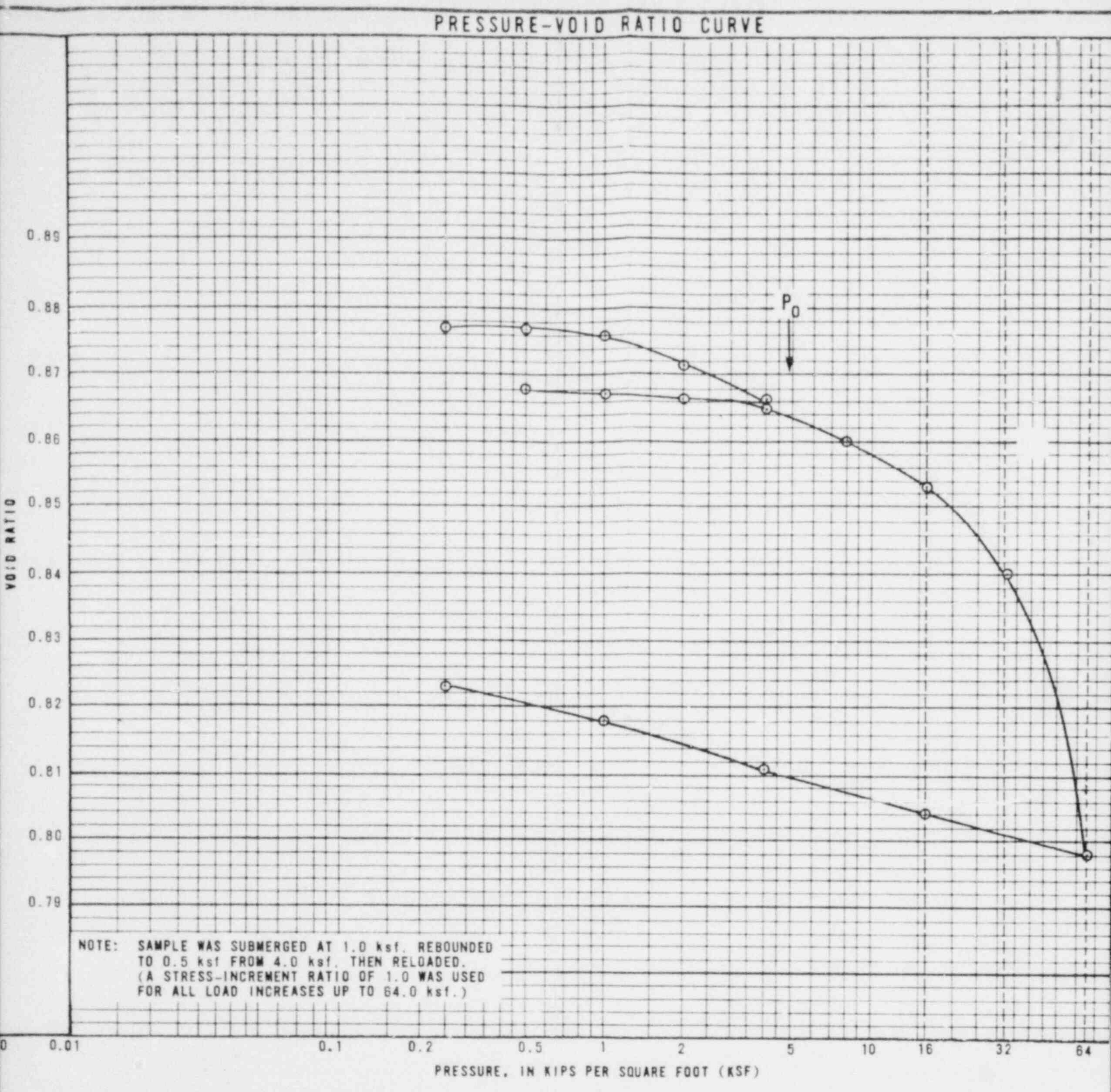
ATTERBERG LIMIT SPECIMEN

STRATUM DESIGNATION	
LIQUID LIMIT (LL)	NP
PLASTIC LIMIT (PL)	NP
PLASTICITY INDEX (PI)	NP
NATURAL WATER CONTENT % (w)	14.9
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	----
SPECIFIC GRAVITY (G)	2.70

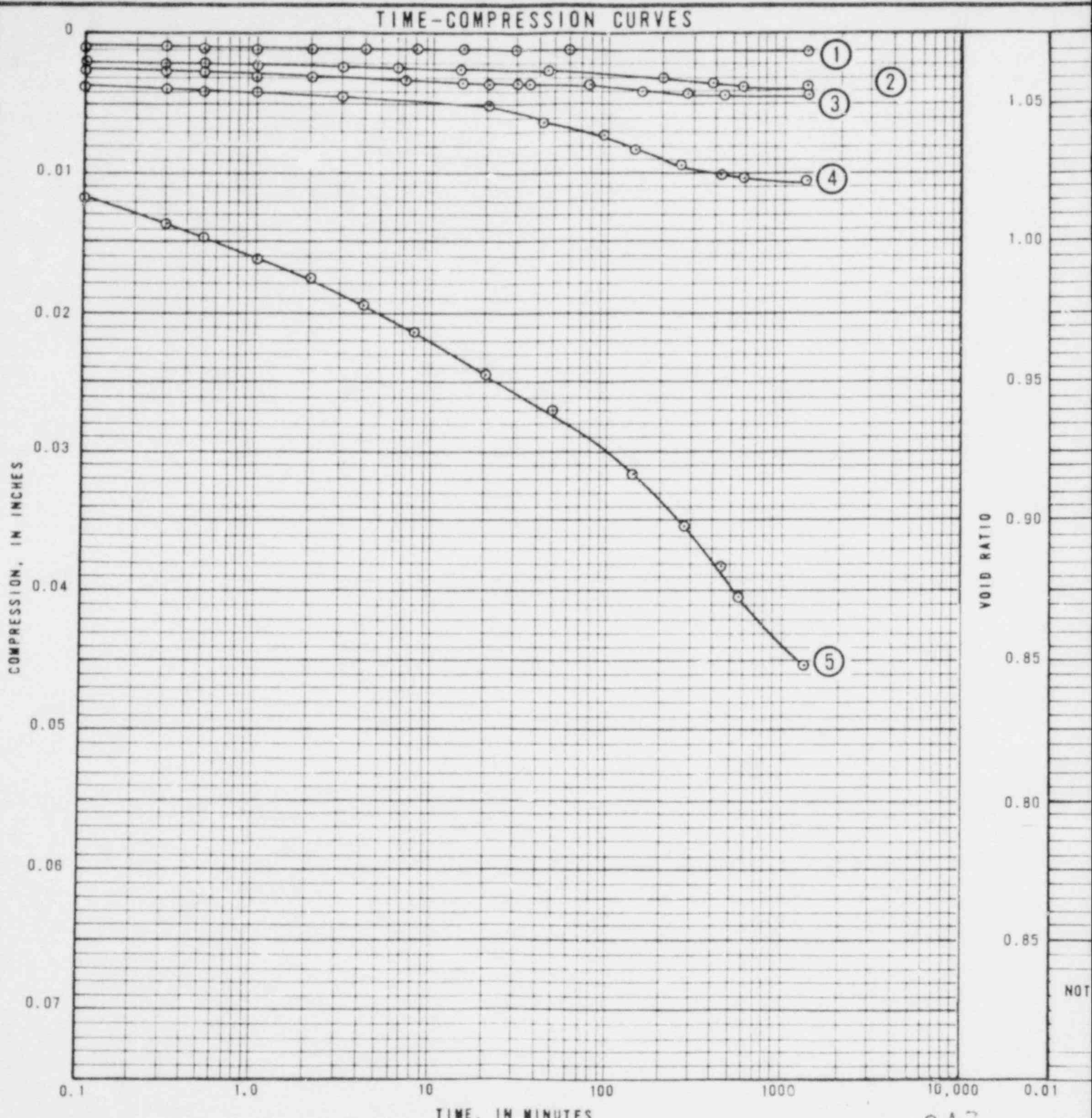
CONSOLIDATION SPECIMEN

INITIAL DRY DENSIT	
FINAL DRY DENSITY	
DIAMETER OF SPECIM	
INITIAL THICK.OF S	
INITIAL WATER CONT	
FINAL WATER CONTEN	
INITIAL DEGREE SAT	
FINAL DEGREE SATUR	

853A



				HOLE NO. 105	DEPTH (ft) 55-57	
(pcf)	89.8	INITIAL VOID RATIO (e_o)	0.878	COMPRESSION INDEX (C_c)		0.14
pcf)	90.9	FINAL VOID RATIO (e_f)	0.823	SWELLING INDEX (C_s), REBOUND FROM e = 0.811		0.012
N (inches)	2.5	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c		COEFFICIENT OF CONSOL. (C_v) cm^2/sec.		
EC. (inches)	1.00	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o	4.9	RECOMPRESSION INDEX (C_r)		0.003
NT %	14.9					
%	24.4	SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATION TEST		SAMPLE NO. 21
RATION %	46			PROJECT NO. 2111	DATE NOV. 1967	DRAWING NO. FIGURE 6
TION %	80		PALO ALTO • WHITTIER • CALIF.			



047

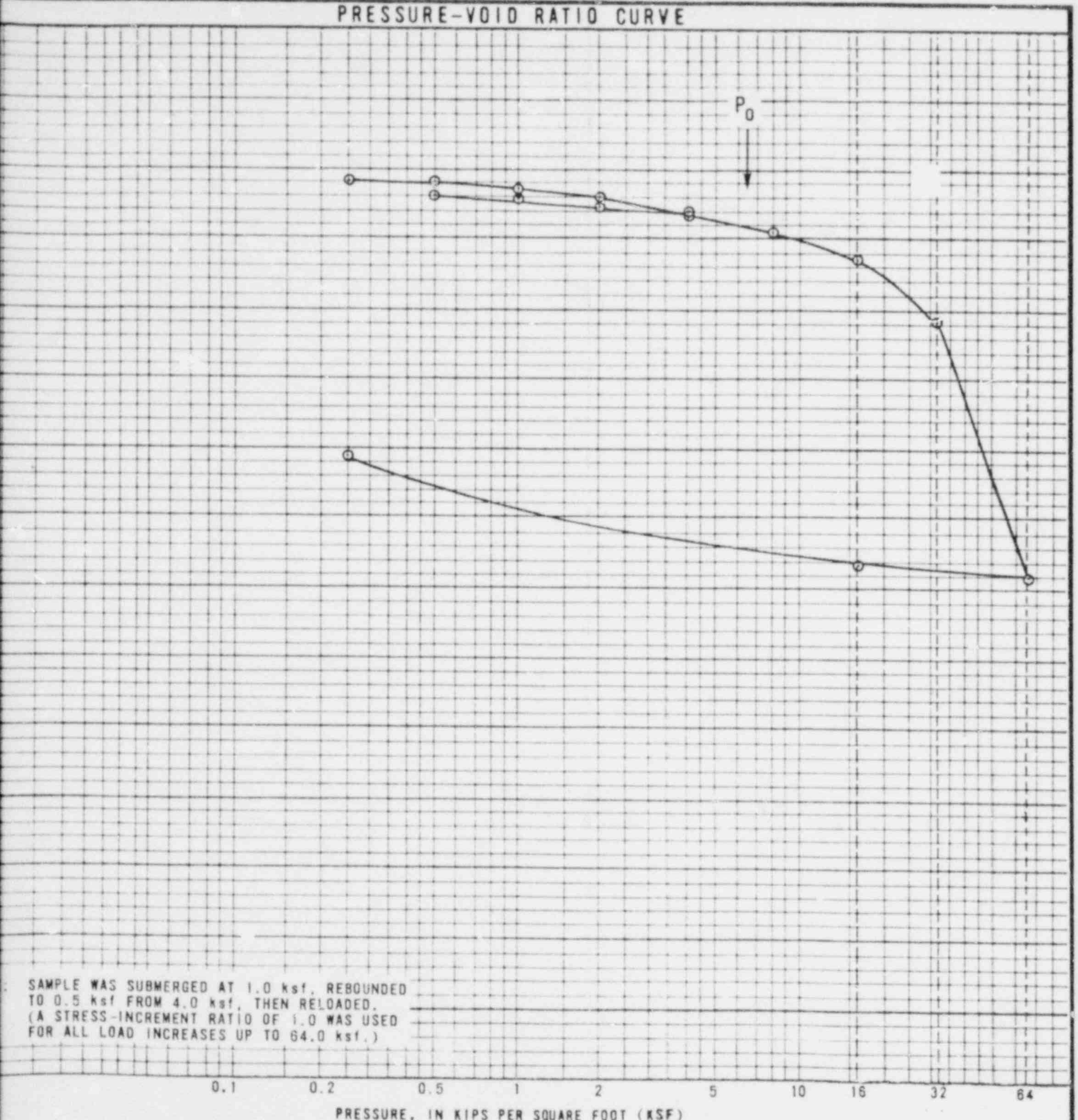
PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES		
CURVE NO.	PRESSURE INCREMENT FROM (KSF)	TO (KSF)
1	2.0	4.0
2	4.0	8.0
3	8.0	16.0
4	16.0	32.0
5	32.0	64.0

DESCRIPTION OF SPECIMEN Medium brown, fine, sandy SILT (ML) NP

ATTERBERG LIMIT SPECIMEN	
STRATUM DESIGNATION	
LIQUID LIMIT (LL)	32
PLASTIC LIMIT (PL)	NP
PLASTICITY INDEX (PI)	NP
NATURAL WATER CONTENT % (w)	27.4
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	----
SPECIFIC GRAVITY (G)	2.67

CONSOLIDATION SPECIMEN	
INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inches)	
INITIAL THICK. OF SPEC. (inches)	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION %	
FINAL DEGREE SATURATION %	

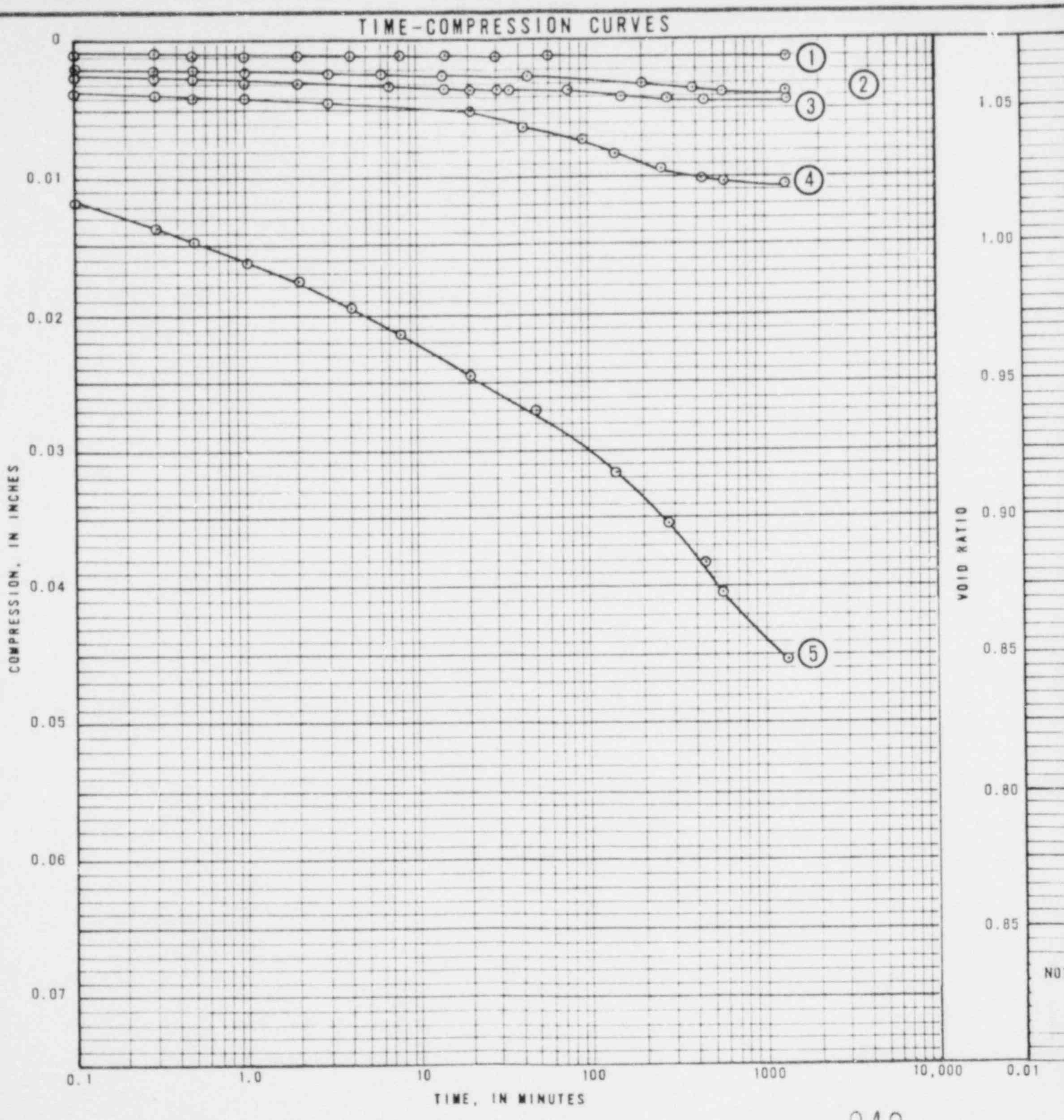
853A



048

		HOLE NO. 105	DEPTH (ft) 70–72	
82.6	INITIAL VOID RATIO (e_o)	1.022	COMPRESSION INDEX (C_c)	0.31
86.5	FINAL VOID RATIO (e_f)	0.923	SWELLING INDEX (C_s), REBOUND FROM e = 0.884	0.019
2.50	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c ↓		COEFFICIENT OF CONSOL. (C_v) cm²/sec.	
1.00	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o ↓	6.3	RECOMPRESSION INDEX (C_r)	0.006
27.4				
30.6	SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATION TEST	SAMPLE NO.
72			PROJECT NO. 2111 — DATE NOV. 1967 — DRAWING NO. FIGURE 7	27
89	PALO ALTO • WHITTIER • CALIF.			

TIME-COMPRESSION CURVES



049

PRESSURE INCREMENTS OF TIME-COMPRESSION CURVES		
CURVE NO.	PRESSURE INCREMENT FROM (KSF)	TO (KSF)
①	2.0	4.0
②	4.0	8.0
③	8.0	16.0
④	16.0	32.0
⑤	32.0	64.0

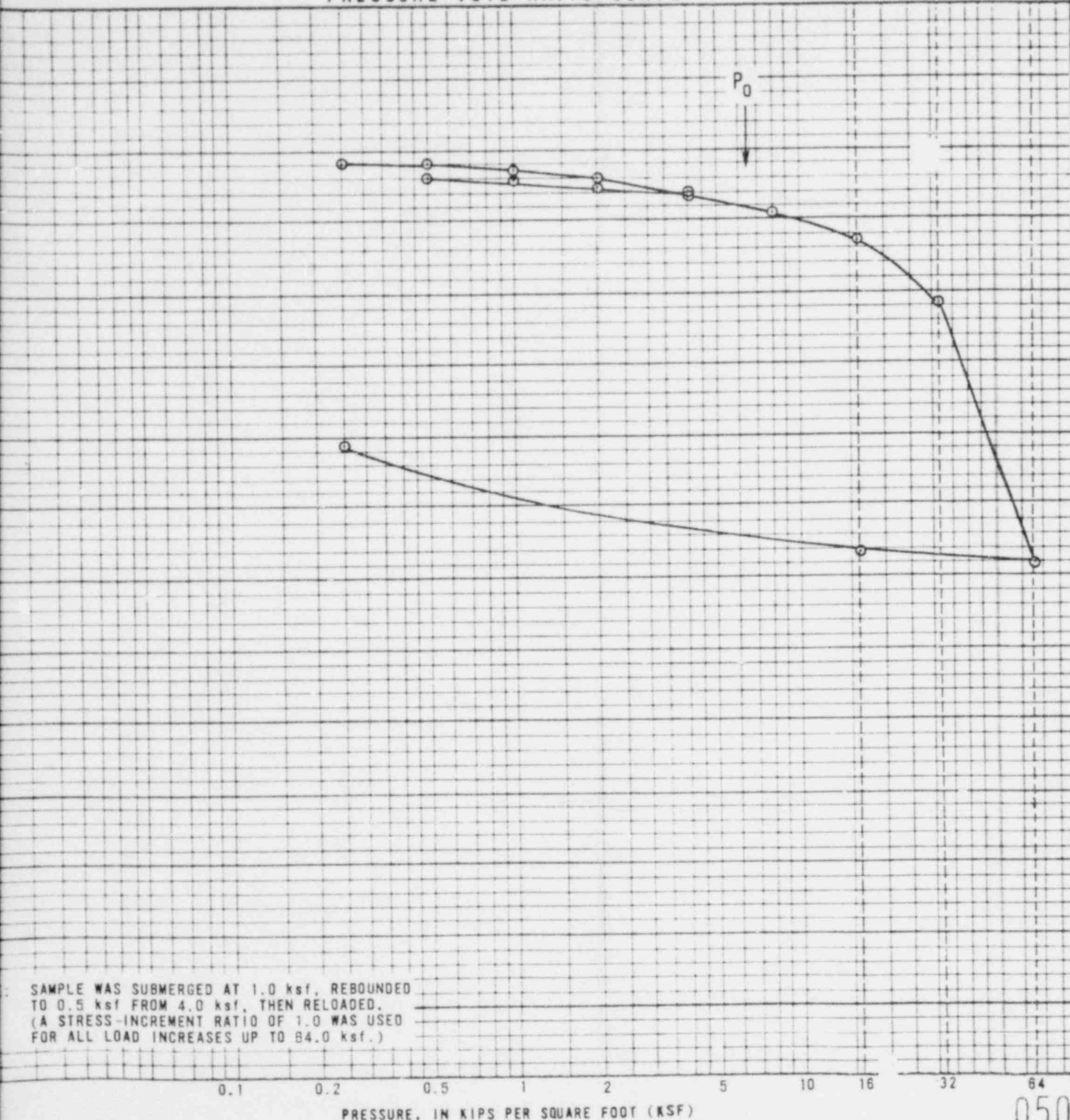
DESCRIPTION OF SPECIMEN Medium brown, fine, sandy SILT (ML) NP

ATTERBERG LIMIT SPECIMEN	
STRATUM DESIGNATION	
LIQUID LIMIT (LL)	32
PLASTIC LIMIT (PL)	NP
PLASTICITY INDEX (PI)	NP
NATURAL WATER CONTENT % (w)	27.4
LIQUIDITY INDEX = $\frac{w-PL}{LL-PL}$	----
SPECIFIC GRAVITY (G)	2.67

CONSOLIDATION SPECIMEN	
INITIAL DRY DENSITY (pcf)	
FINAL DRY DENSITY (pcf)	
DIAMETER OF SPECIMEN (inches)	
INITIAL THICK. OF SPEC. (inches)	
INITIAL WATER CONTENT %	
FINAL WATER CONTENT %	
INITIAL DEGREE SATURATION %	
FINAL DEGREE SATURATION %	

853A

PRESSURE-VOID RATIO CURVE



050

			HOLE NO. 105	DEPTH (ft) 70–72
82.6	INITIAL VOID RATIO (e_o)	1.022	COMPRESSION INDEX (C_c)	0.31
86.5	FINAL VOID RATIO (e_f)	0.923	SWELLING INDEX (C_s), REBOUND FROM $e = 0.884$	0.019
2.50	MOST PROBABLE PRECONSOL. STRESS (Ksf) SYMBOL P_c		COEFFICIENT OF CONSOL. (C_v) cm^2/sec.	
1.00	EXISTING OVERBURDEN STRESS (Ksf) SYMBOL P_o	6.3	RECOMPRESSION INDEX (C_r)	0.006

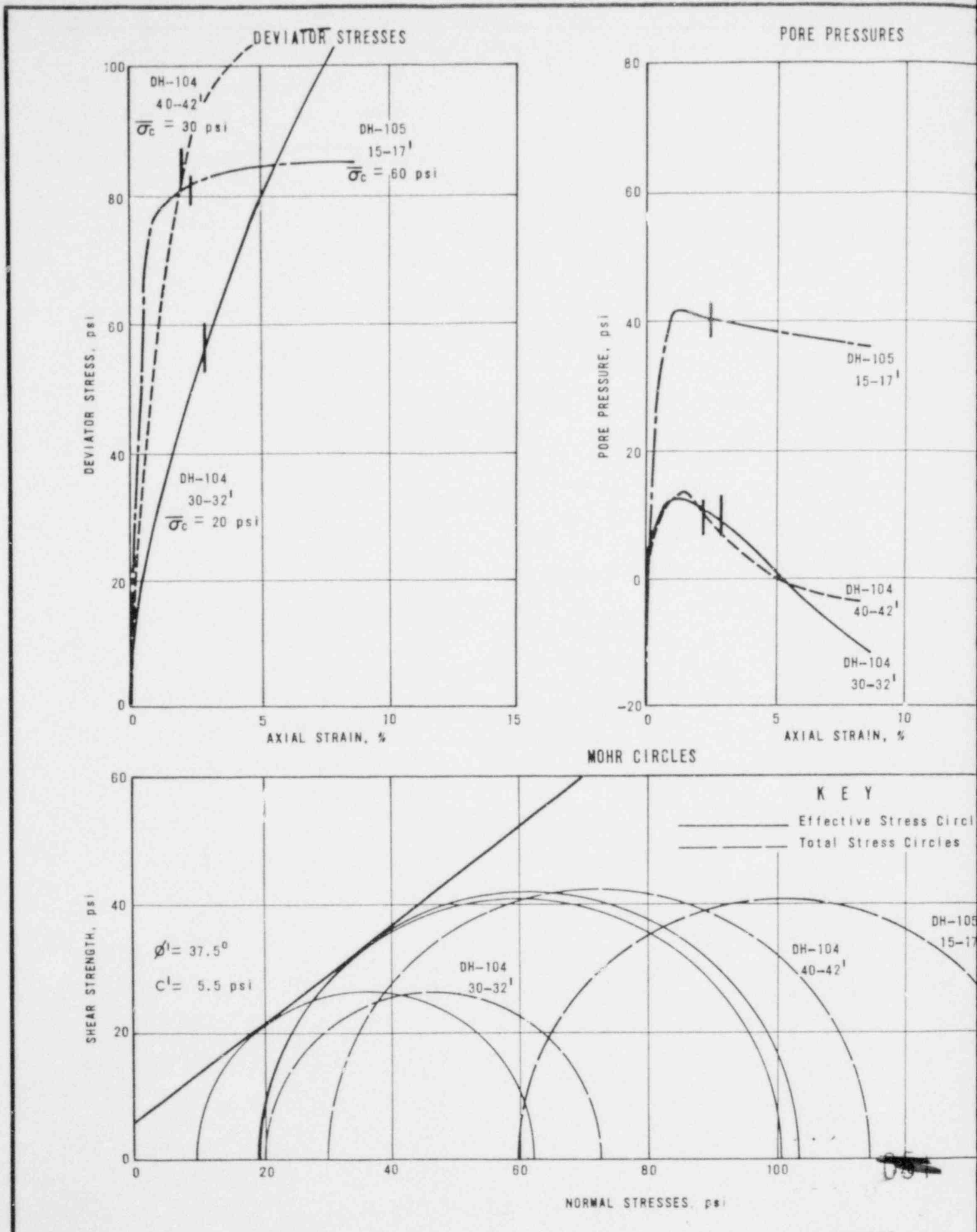
27.4
30.6
72
89

SOIL MECHANICS and FOUNDATION ENGINEERS Inc.
PALO ALTO • WHITTIER • CALIF.

RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1

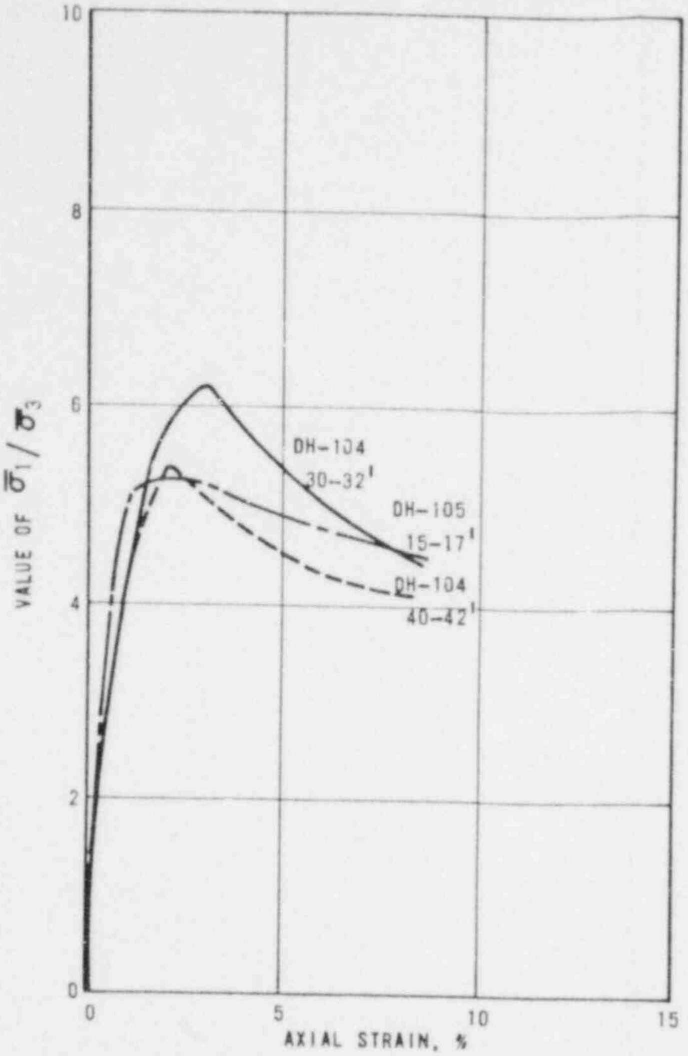
CONSOLIDATION TEST

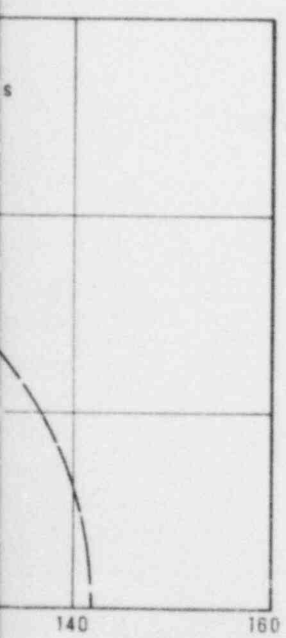
PROJECT NO.	DATE	DRAWING NO	SAMPLE NO.
2111	NOV. 1967	FIGURE 7	27


DEVIATOR STRESSES
DH-104
40-42'
$\overline{\sigma_c}$ = 30 psi
DH-105
15-17'
$\overline{\sigma_c}$ = 60 psi
DH-104
30-32'
$\overline{\sigma_c}$ = 20 psi
DEVIATOR STRESS, psi
AXIAL STRAIN, %
PORE PRESSURES
DH-105
15-17'
DH-104
40-42'
DH-104
30-32'
PORE PRESSURE, psi
AXIAL STRAIN, %
MOHR CIRCLES
K E Y
Effective Stress Circl
Total Stress Circles
ϕ' = 37.5°
C' = 5.5 psi
DH-104
30-32'
DH-104
40-42'
DH-105
15-17
SHEAR STRENGTH, psi
NORMAL STRESSES, psi

51

EFFECTIVE STRESS RATIOS



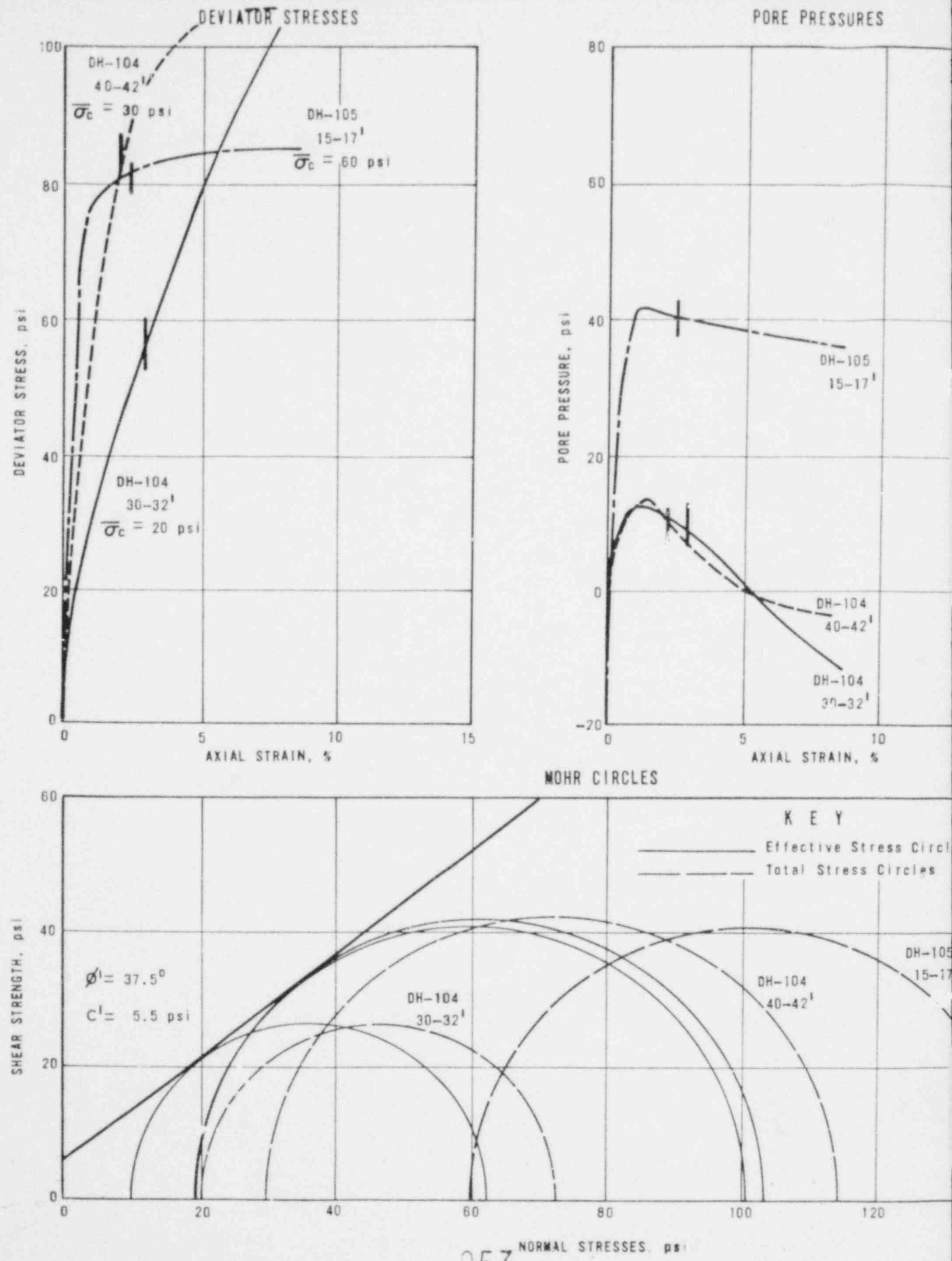


SPECIMEN DATA

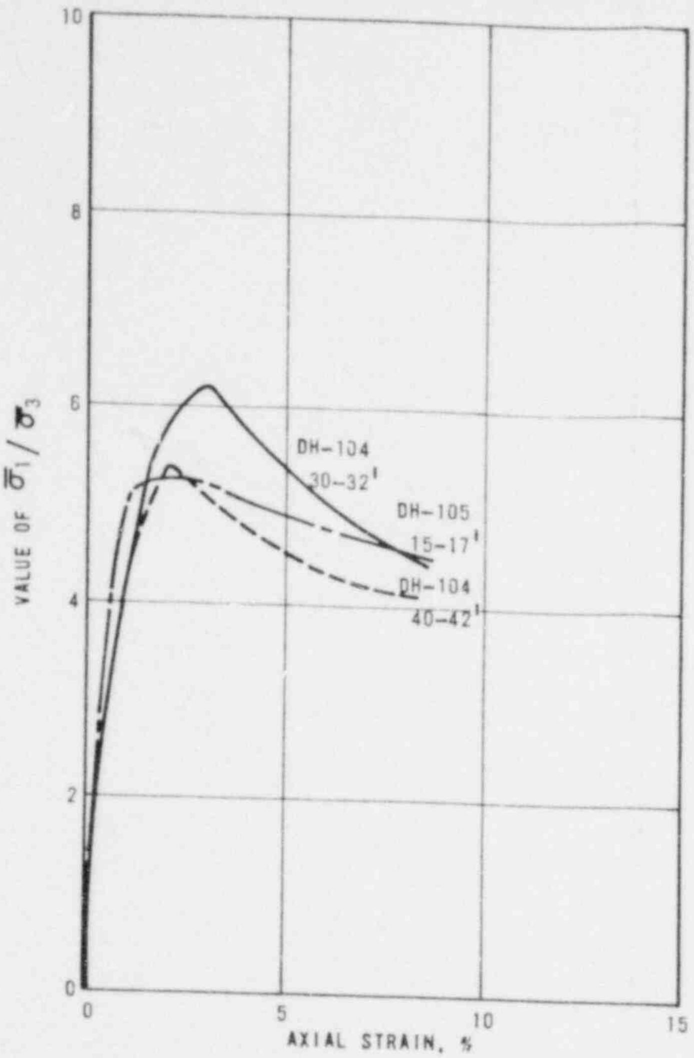
HOLE NO.	DEPTH (feet)	INITIAL DRY DENSITY (pcf)	INITIAL WATER CONTENT (%)	FINAL WATER CONTENT (%)	PLASTICITY		PASSING NO.200 SIEVE (%)
					LL (%)	PI (%)	
104	30-32	91.7	24.0	32.2	38	8	53
104	40-42	91.0	23.0	32.7	34	4	66
105	15-17	82.3	32.8	40.9	37	5	88

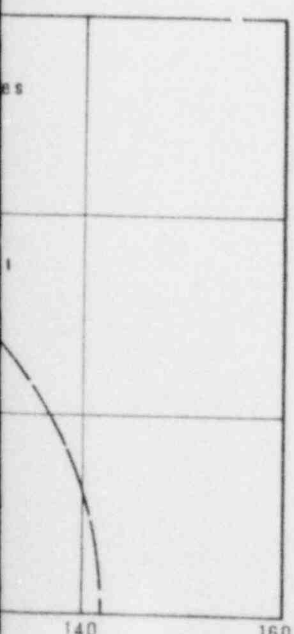
052

SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA		
	PALO ALTO · WHITTIER · CALIF.	PROJECT NO. 2111	DATE NOV. 1967	DRAWING NO. FIGURE 8


DEVIATOR STRESSES
PORE PRESSURES
DH-104
40-42'
$\overline{\sigma_c}$ = 30 psi
DH-105
15-17'
$\overline{\sigma_c}$ = 60 psi
DH-104
30-32'
$\overline{\sigma_c}$ = 20 psi
DEVIATOR STRESS, psi
AXIAL STRAIN, %
PORE PRESSURE, psi
DH-105
15-17'
DH-104
40-42'
DH-104
30-32'
AXIAL STRAIN, %
MOHR CIRCLES
K E Y
Effective Stress Circl
Total Stress Circles
SHEAR STRENGTH, psi
Ø' = 37.5°
C' = 5.5 psi
DH-104
30-32'
DH-104
40-42'
DH-105
15-17
NORMAL STRESSES, psi

EFFECTIVE STRESS RATIOS



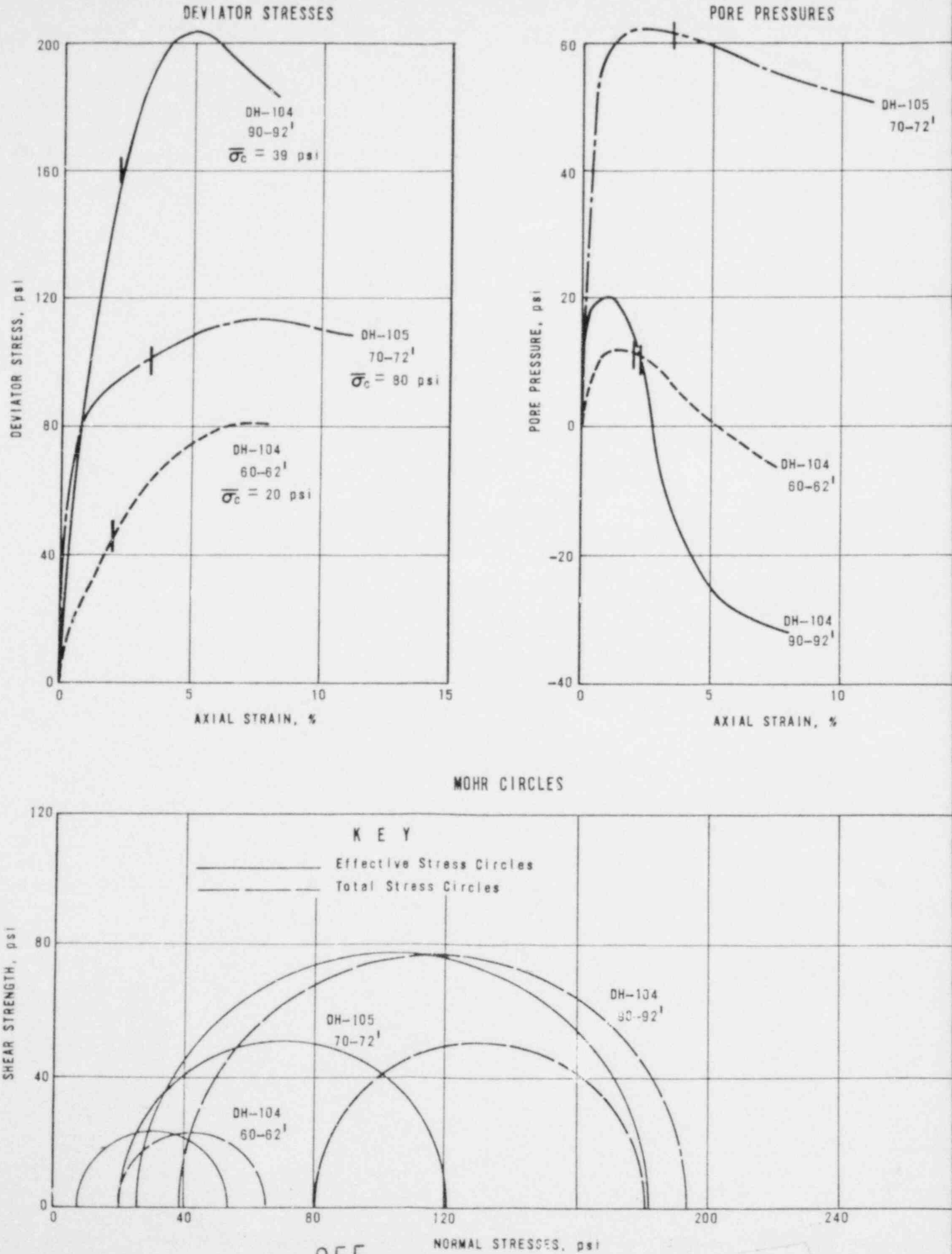


SPECIMEN DATA

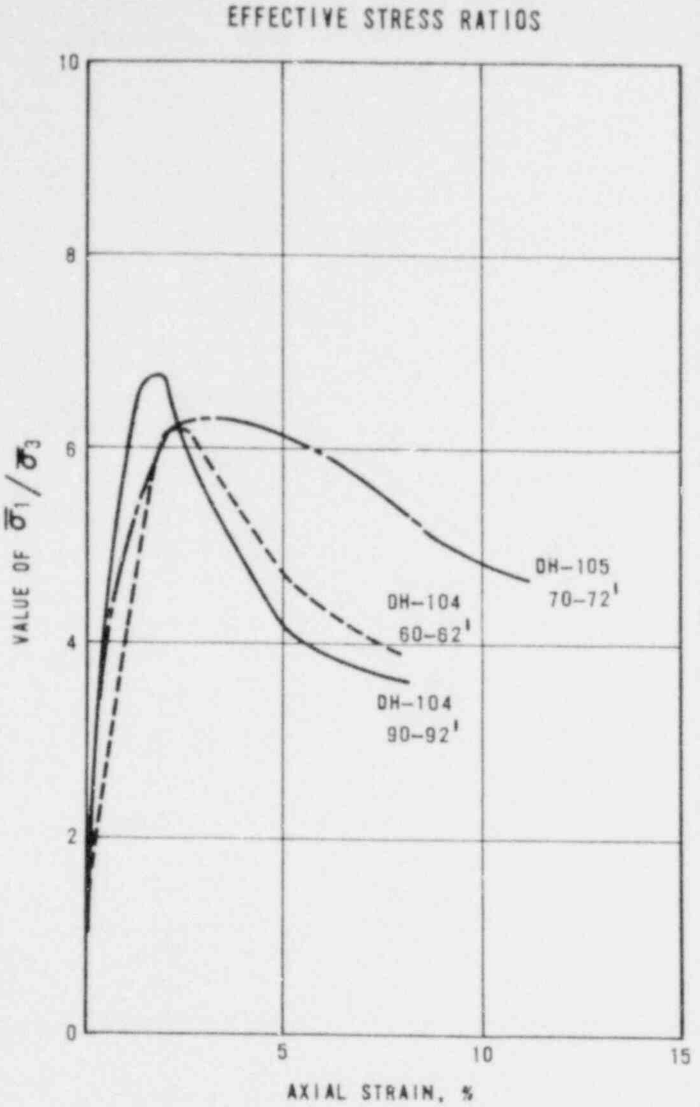
HOLE NO.	DEPTH (feet)	INITIAL DRY DENSITY (pcf)	INITIAL WATER CONTENT (%)	FINAL WATER CONTENT (%)	PLASTICITY		PASSING NO. 200 SIEVE (%)
					LL (%)	PI (%)	
104	30-32	91.7	24.0	32.2	38	8	53
104	40-42	91.0	23.0	32.7	34	4	66
105	15-17	82.3	32.8	40.9	37	5	88

54

SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1	CONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA		
	PALO ALTO · WHITTIER · CALIF.	PROJECT NO. 2111	DATE NOV. 1967	DRAWING NO. FIGURE 8



DEVIATOR STRESSES
PORE PRESSURES
DEVIATOR STRESS, psi
AXIAL STRAIN, %
DH-104 90-92'
σ̄c = 39 psi
DH-105 70-72'
σ̄c = 80 psi
DH-104 60-62'
σ̄c = 20 psi
PORE PRESSURE, psi
MOHR CIRCLES
KEY
Effective Stress Circles
Total Stress Circles
SHEAR STRENGTH, psi
NORMAL STRESSES, psi



HOLE NO.	DEPTH (feet)	INITIAL DRY DENSITY (pcf)	INITIAL WATER CONTENT (%)	FINAL WATER CONTENT (%)	PLASTICITY		PASSING NO. 200 SIEVE (%)
					LL (%)	PI (%)	
104	60-62	68.5	32.8	35.0	47	6	52
105	70-72	90.9	22.7	31.0	NP	NP	51
104	90-92	98.0	23.7	29.2	38	3	64

~~054~~

056

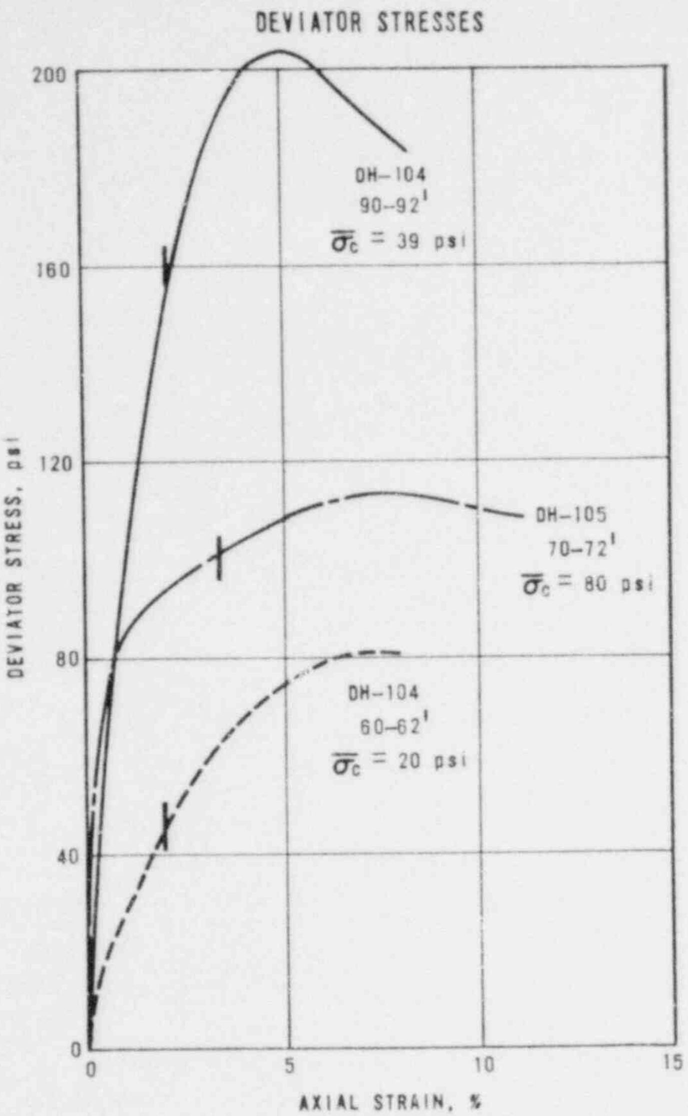
SOIL MECHANICS and FOUNDATION ENGINEERS Inc.

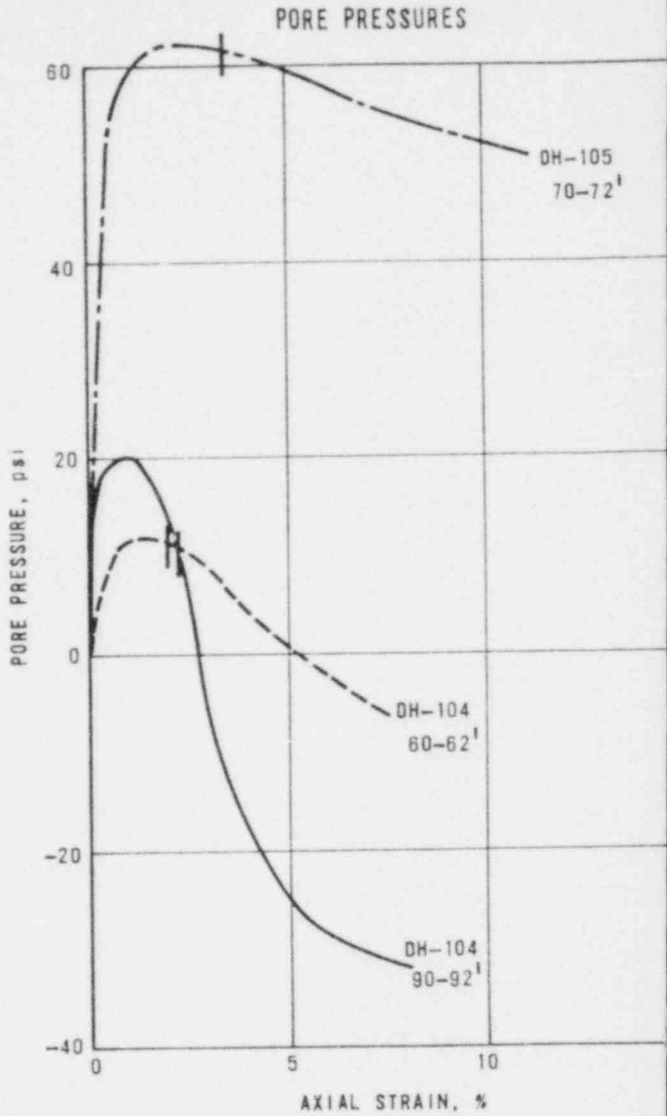
PALO ALTO • WHITTIER • CALIF.

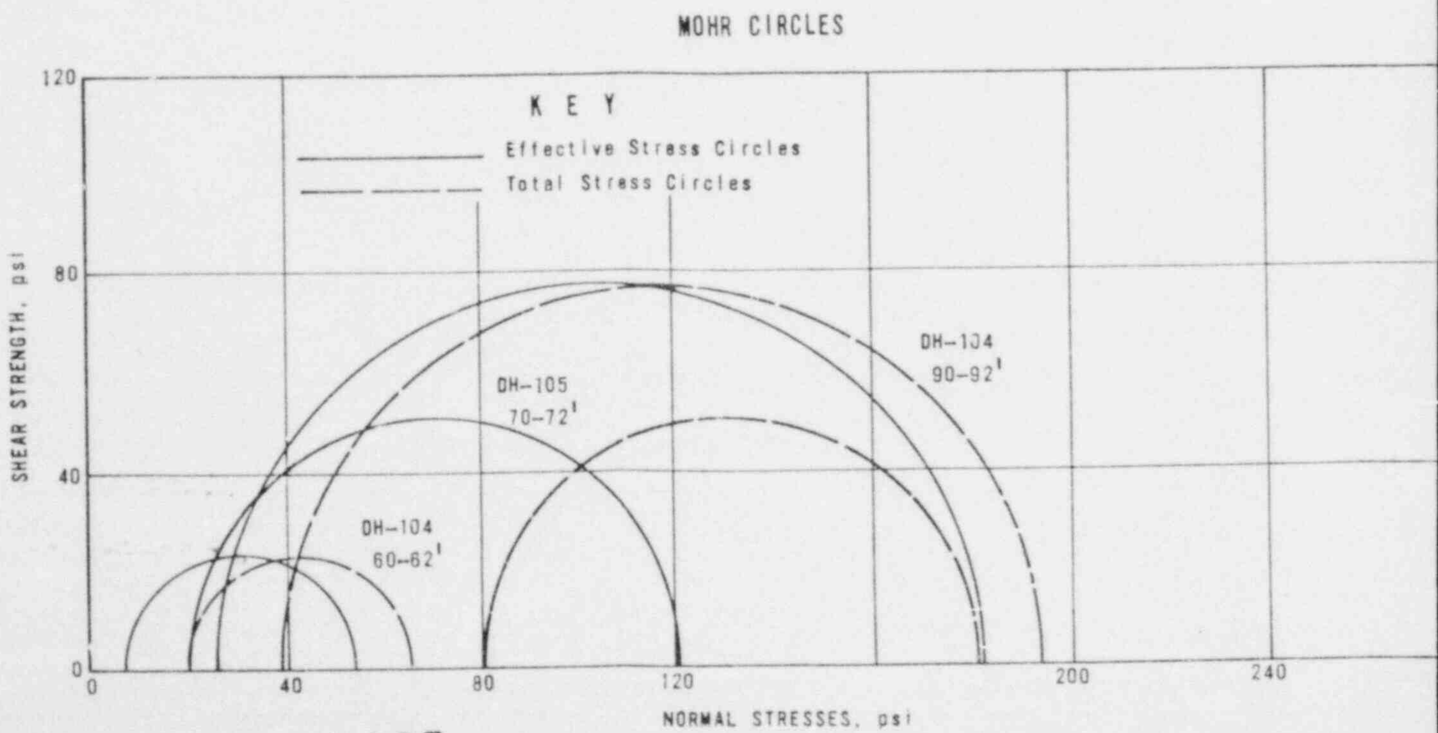
RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1

CONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA

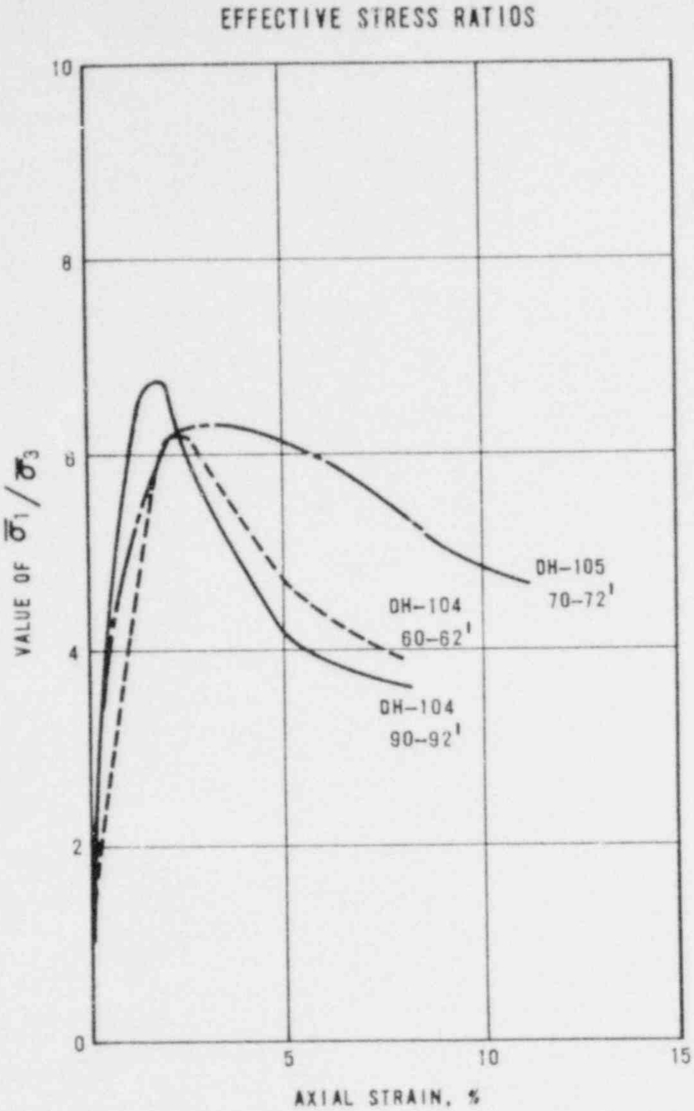
PROJECT NO.	DATE	DRAWING NO.
2111	NOV. 1967	FIGURE 9


DEVIATOR STRESSES
DEVIATOR STRESS, psi
AXIAL STRAIN, %
0
40
80
120
160
200
0
5
10
15
DH-104
90-92'
$\overline{\sigma}_c$ = 39 psi
DH-105
70-72'
$\overline{\sigma}_c$ = 80 psi
DH-104
60-62'
$\overline{\sigma}_c$ = 20 psi


PORE PRESSURES
PORE PRESSURE, psi
AXIAL STRAIN, %
-40
-20
0
20
40
60
0
5
10
DH-105
70-72'
DH-104
60-62'
DH-104
90-92'


MOHR CIRCLES
KEY
Effective Stress Circles
Total Stress Circles
SHEAR STRENGTH, psi
NORMAL STRESSES, psi
0
40
80
120
0
40
80
120
200
240
DH-104
90-92'
DH-105
70-72'
DH-104
60-62'

057



HOLE NO.	DEPTH (feet)	INITIAL DRY DENSITY (pcf)	INITIAL WATER CONTENT (%)	FINAL WATER CONTENT (%)	PLASTICITY		PASSING NO. 200 SIEVE (%)
					LL (%)	PI (%)	
104	60-62	68.5	32.8	35.0	47	6	52
105	70-72	90.9	22.7	31.0	NP	NP	51
104	90-92	98.0	23.7	29.2	38	3	64

058

SOIL MECHANICS and FOUNDATION ENGINEERS Inc.	RANCHO SECO NUCLEAR GENERATING STATION, UNIT 1 PALO ALTO • WHITTIER • CALIF.	CONSOLIDATED-UNDRAINED TRIAXIAL TEST DATA		
		PROJECT NO.	DATE	DRAWING NO.
		2111	NOV. 1967	FIGURE 9