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SOIL, FOUNDATION & GEOLOGICAL INVESTIGATIONS

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GEOTECHNICAL INVESTIGATION FOR THE RANCHERS WESTERN COLORADO PROJECT IN THE EAST HALF OF SECTION 34, TOWNSHIP 46 NORTH, RANGE 16 WEST, NEW MEXICO PRINCIPAL MERIDIAN, SOUTHERN MONTROSE COUNTY, COLORADO

Prepared For

Ortloff Mineral Services Corporation

Job No. 1331-2955
March 29, 1977

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SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

1. As currently planned, the site is suitable for the proposed leaching operation. Soils on the site will be suitable for construction of liner and embankments when properly compacted. Bedrock materials should be adequately broken down for liner use, or eliminated from the proposed construction.

2. Special precautions will be necessary to divert existing drainages around the proposed leach tanks.

3. Other design and construction details are presented below.

INTRODUCTION

This report presents the results of a geotechnical investigation and recommendations to be considered in the design, construction, operation and subsequent reclamation of a parcel of land to be used for a uranium leaching operation site. The site is located in the east half of Section 34, Township 46 North, Range 16 West of the New Mexico Principal Meridian, southwestern Montrose County, Colorado.

PROPOSED CONSTRUCTION

Currently proposed construction will include several earth fill embankment structures and tailing handling facilities. Refer to Test Hole Location Plan, Figure 2.

The earth fill embankment structures will include 3 uranium leaching tanks, 11 evaporation ponds (5 of which will be constructed only as required by plant operations), 2 drainage diversion structures and 5 runoff retention structures.

SCOPE OF WORK

The scope of this investigation includes proposed embankment slope stability, liner, embankment and in-situ soil permeability and design and construction recommendations for leach tanks and evaporation ponds. In addition, leach tank and evaporation pond liner permeabilities are in the process of being investigated with the addition of sulfuric acid to the permeameter water. These analyses will be completed after issuance of this report.

Process plant construction recommendations were not included in the scope of this investigation.

SITE CONDITIONS AND GENERAL GEOLOGY

The project area is presently vacant and slopes topographically from south to north with a pronounced topographic high located approximately in the center of the site. The site is traversed by three intermittent, south to north trending drainages. The existing drainages show evidence of changing depositional energy patterns in the form of coarse sand, gravel and cobbles in the various reaches of the drainage ways (refer to Figure 2 for drainage locations).

Geologically, the site is located approximately on the northwest, southeast axis of the Coke Oven sinclinal basin, which is approximately 4 miles long and 2 miles wide. The bedrock encountered in the test holes and noted at outcrop areas on the site is of the Cretaceous Mancos Formation.

Existing site vegetation is sparse and consists primarily of sage brush. Much of the soil mantle is exposed to the weather and present mechanical erosional rates appear to be moderately high.

The site is located within a Zone 1 seismic risk area as mapped by S. T. Algermissen of the United States Geological Survey. Zone 1 areas are considered low risk areas, and seismic stability analyses are not generally conducted for construction in areas mapped as less than Zone 3. A seismic analysis was not included in the scope of this investigation.

FIELD INVESTIGATION

The field investigation was initiated by an overall site reconnaissance.

Test hole locations were then chosen by our office and located in the field by personnel of Ortloff Mineral Services Corporation.

The drilling program consisted of 22 exploration holes on-site (9 in the evaporation pond areas, 4 for the surface structures and 9 in the leach tank areas) and 10 exploration holes on the proposed access road. Also included in the drilling program were 32 percolation test holes, 14 of which were located in the evaporation ponds, and 18 in the leach tanks. The exploration holes were drilled with a 4 inch diameter continuous flight power auger. The percolation holes were drilled with a 6 inch diameter continuous flight power auger. The materials encountered were logged by personnel of this office on the site during the drilling operations. The various soil strata encountered during the drilling is shown on the Logs of Test Holes, Appendix A, Figures A-1 through A-4. The soils were sampled with a California Sampler, from which Standard Penetration Tests, as well as undisturbed samples for laboratory analysis, were obtained. Disturbed bulk soil samples were retrieved from the drilled percolation test holes and the exploration holes for the access road.

Percolation tests to determine estimated in-situ soil permeability were conducted simultaneously with the drilling operation in the leach tank and evaporation pond areas. Results of these tests are presented on Table V.

The test holes generally encountered 0 to more than 20 feet of slightly moist, stiff to very stiff, silty, sandy clay and sandy, gravelly clay over mudstone and shale of the Mancos Formation. Generally, the soil and bedrock encountered during the test hole drilling has a low in-situ permeability ranging from 7.6×10^{-6} to 5.6×10^{-5} cm/sec, as estimated from field percolation rates. The bedrock encountered was highly fractured near the bedrock-soil interface. These shallow fractures cause bedrock permeabilities to be as high or higher than the soil permeabilities.

LABORATORY INVESTIGATION

Undisturbed and disturbed soil samples were inspected and classified from the exploration and percolation holes for determination of applicable laboratory testing.

The undisturbed soil samples were tested to determine their engineering characteristics for support of surface structures and embankments. Testing included swell-consolidation tests, unconfined compressive strength tests and natural moisture and dry density tests. A summary of the undisturbed soil laboratory testing is shown on Table IV and in Appendix A.

The bulk soil samples, which were representative of the material to be used in embankment construction, were divided into 5 separate groups possessing similar engineering characteristics (see Table I). Laboratory testing relative to design and construction of embankments was then initiated for each of the five groups on remolded samples. The testing included mechanical analyses, Atterberg limits, Proctor analyses, unconsolidated undrained triaxial compression tests, unconfined compression tests, one dimensional consolidation and permeability tests. Laboratory data is summarized on Table VI and is presented in Appendix A.

Conventional procedures for measuring permeability of compacted embankment soils were initiated early in the lab testing program; however, after several days of sample saturation time under various heads to 20 psi, it was found that the samples would not permeate water and the tests could not be conducted by conventional permeability methods within the time frame of this report. For this reason, consolidation tests were run on remolded embankment soils and soil permeabilities were calculated from these test results. Calculated permeabilities for the compacted soils range from 8.6

x 10^{-8} cm/sec to 2.6 x 10^{-9} cm/sec (refer to Table III).

Refer to Appendix C for laboratory test procedures utilized in the laboratory testing.

CONSTRUCTION RECOMMENDATIONS

Construction recommendations for the individual structures are presented below:

Leach Tanks

Three leach tanks will be constructed in the south and southwestern portions of the site. It is our understanding that excavated material from the interior sections of the tanks (i.e., percolation holes 18 through 32) will be used to construct the leach tank embankments. Soils included in the proposed excavation areas include soil Groups II through V.

The maximum height of the embankment fills will be approximately 20 feet on the north side of the tanks. The crest width of the embankments will be approximately 20 feet and will be constructed to allow truck traffic for tailings deposition. A drain collection system will be provided in each of the 3 leach tanks, with individual underflow lines exiting through the northern embankments 25 feet on center to a main collection system. When the drain system is operating, total hydraulic head on the bottom of the leach tanks is expected to be on the order of 1 foot or less. Proposed tank embankments will be constructed with 1:1 grades on interior slopes and 2:1 grades on exterior slopes.

A 1 foot thick compacted soil liner will be constructed in the bottom of the 3 tanks by utilizing the natural soils on the site. Laboratory testing indicates that the natural soils and/or adequately broken down bedrock material will serve as a nearly impervious liner and embankment material, when properly compacted. It should be understood that liner and embankment saturation is not expected during the 6 month expected operation life of the leach tanks. For purposes of construction and stability analysis, it is assumed that the entire compacted embankment will serve as liner. However, where cut faces in natural soils or bedrock are exposed, special requirements will be necessary to create an impervious liner as discussed below.

Construction of the liner may be accomplished by scarifying the bottom tank surfaces after final excavation to a depth of 8 inches, where bedrock is not encountered, with subsequent compaction to 95% of ASTM D-1557. If bedrock is encountered, an additional 8 inches should be subcut and the bedrock broken down and recompacted in place. Suitability of the bedrock to serve as soil liner in the bottom of the tanks and/or evaporation ponds should be verified during construction. It may be more economical to specify that where bedrock is encountered at the bottom of the tanks, it will be removed and replaced with compacted natural soils for liner. Additional liner material should be placed in maximum 8 inch lifts and compacted to obtain the specified liner thickness. The top of each lift should be scarified prior to additional fill placement to assure proper bond between layers. This is especially important for prevention of piping and hydraulic fracturing in the lower portions of the fill. Embankment construction should proceed with fill placement as above. Liner construction on cut faces in natural soils can be accomplished by utilizing fill placement on existing slopes. However, it may be more advantageous to over-excavate the sides of the tank in cut sections to allow placement of a compacted embankment around the entire perimeter of the tank by conventional means. The specifications for fill placement have been forwarded, under separate cover, and should be referred to. A competent soils engineer should be retained for quality control and supervision of the fill placement.

Piping along the underflow lines extending through the embankments should also be considered during and after construction. Seepage collars should be provided near the interior embankment slope face. The collars may be constructed of cast-in-place concrete or welded steel plate, as detailed on Figure 3. It is recommended that the underflow lines be installed through the embankment after placement of the bottom soil liner, and a minimum of 18 inches of compacted embankment. Trenches should be cut through the embankment material with subsequent pipe installation. Concrete seep collars should be constructed without forms against the exposed bottom and sides of the trench. It is advisable to provide "key" forms at least 8 inches deep in the bottom and sides of the trench for collar installation. We are available to discuss this with you. Backfill placed against the seep collars in the trenches should be moistened and compacted, as discussed above. Due to limited working space, the use of small mechanical compactors is recommended in the trenches.

A stability evaluation of the leach tank embankments required that a mathmatical model be established to accurately simulate the conditions on the site. For this analysis, a simplified Bishop's slope stability method was employed (i.e., a common slice and slip circle method). This method utilizes the principles of static analysis, where certain conditions are assumed so that an idealized system can be created for model simulation. The primary assumption of the analysis is that the material composing the embankment is isotropic and homogeneous in each unitized stratum.

The cross-sections analyzed were taken from the proposed grading plans furnished by Ortloff Mineral Services Corporation. Because of the low permeability characteristics of the embankments soils, full consolidation and drainage of pore pressures created during construction in the soils should not take place over the useful life of the structure. For these reasons, the unconsolidated undrained strengths of embankment materials were utilized in the analysis. Partial saturation of embankment soils will be limited to a small section near the interior toe of the embankment during the 6 month operation of individual leach tanks due to low permeability. Thus, a steady state phreatic surface was not incorporated into the analysis.

The strength criteria used in the design was obtained from triaxial shear data as discussed above, with remolded samples of proposed embankment material compacted to 95% of maximum density (ASTM D-1557) at 2% over optimum moisture content. Group III soils were found to have the lowest strength characteristics and were used in evaluation as a conservative analysis.

Based on the computer analysis, the proposed 2:1 slope will have a minimum factor of safety of 8.1 against major slope failure with complete filling of tailings on the tank interior. The interior 1:1 slope will have a minimum factor of safety of 9.1 when the tank is empty. However, this does not preclude minor failures (due to sloughing, erosion or truck traffic) near the top of the slope prior to complete tailings placement.

Drainage Diversion

In order to construct the leach tanks at the proposed locations, it will be necessary to divert two existing drainages to the east sides of tanks LT202 and LT203. On site inspection indicates that the drainages are filled with high energy deposits of sand and gravel with some cobbles and boulders. Care should be taken during construction to properly divert these drainages to prevent historic channels from carrying ground water beneath the tanks or diversion embankments. This can be accomplished by removing all coarse grained soils (sand and gravel) from the existing channels beneath the area to be filled by diversion embankment. The extent of this material could not be ascertained because of the limited test hole spacing. For this reason, the total amount of material to be removed should be determined by a representative of this office during construction. It may be advisable to excavate test pits to determine total depth of these deposits prior to construction. Diversion embankments should consist of well compacted fill, as discussed above.

Evaporation Ponds

The evaporation ponds will be constructed in the northern portion of the site for the purpose of disposing liquid waste. Six evaporation ponds are proposed for immediate construction, with an area to the northwest set aside for six future evaporation ponds, should plant operation make them necessary. The evaporation ponds will be constructed of earth fill embankments similar to the leach tanks. All borrow areas for the evaporation pond embankment materials is assumed to be from interior portions of the ponds. Soils found in these borrow areas are generally composed of Group I soils, which are suitable for both liner and embankment construction.

The size of the evaporation pond embankments will range from 5 to 10 feet in height and approximately 12 feet in crest width. Embankment grades will be 2:1 on both interior and exterior slopes. Maximum fluid depths for the majority of the ponds will be on the order of 2 feet. Pond embankments will be leveled after the plant is dismantled and upon complete evaporation of liquid wastes. Wave action on the embankments is not assumed to be critical because of the short fetch length. One to two feet of freeboard is planned.

Using the proposed embankment geometry for the ponds, slope stability analyses were conducted with the same assumptions as in the leach tank analyses, with the exception of embankment soil strengths where Group I soil strengths were utilized. Information supplied by Ortloff Mineral Services Corporation indicates that total evaporation of all liquids will take place in approximately 6 months after the 14 month useful plant life. This condition, coupled with the low permeabilities of the compacted Group I embankment materials, makes the creation of a steady state phreatic flow through the embankments doubtful. A phreatic surface was not simulated in the analysis; however, the hydrostatic pressure created by fluids behind the dams was utilized.

Based upon these assumptions, the computer analysis indicates a minimum factor of safety greater than 10 exists against a major slope failure in the evaporation pond embankments.

Liners and embankments for the ponds should be constructed as detailed in Leach Tanks. Cut slopes should be lined and/or constructed as discussed above.

A minimum 1 foot thick compacted soil liner should be constructed in the bottom of the ponds to minimize pond leakage. To prevent piping along pond overflow pipes, seepage collars should be provided as discussed above.

Retention Structures

A surface hydrology study was not included in the scope of this project; however, maximum 100 year storm runoff rates have been provided by Ortloff Mineral Services Corporation. Retention dam structures will range in size from 5 to 11 feet in height and will be constructed on 2:1 upstream and downstream slopes. Total hydraulic head on the upstream sides of these embankments is expected to be on the order of 5 feet maximum.

It is assumed that borrow material for these structures will be derived on site; however, the exact location is not known at this time. It is recommended that fill placed for these retention dams be compacted to 95% of ASTM D-1557. A stability analysis was not conducted on any of these structures;

however, the stability analysis for the evaporation pond embankments is assumed to be applicable, due to the similar embankment geometry and soil conditions involved.

SITE MAINTENANCE AND RECLAMATION

After completion of embankment fill in the leach tanks, evaporation ponds and runoff retention areas, it is very important that mechanical surface erosion rates due to wind and water be controlled. It is recommended that all exposed embankment slopes be revegetated as soon as practicable to prevent this type of erosion. Possible types of revegetation were not covered in the scope of this investigation; however, useful revegetation alternatives may be obtained from the United States Soil Conservation Service or the United States Forest Service.

It is also recommended that after completion of embankment fills, drilled piezometers be installed in the leach tank embankments to monitor the trend of any phreatic buildup. It is recommended that piezometer holes be placed a minimum of 300 feet on center at the top of the exterior portion of the embankment. The piezometers should be checked weekly during operation to insure that embankment saturation is not occurring. The trend of any buildups should be reported to the writers for subsequent evaluation.

In addition to periodic piezometer readings, surface embankment condition should also be inspected. Particular care should be taken to note longitudinal or transverse cracks on the crest or slopes of the embankments. These inspections should be carried out at least once monthly and after every heavy rainstorm.

It is our understanding that the estimated operation time of the leaching operation is approximately 6 months per tank. At the end of the proposed life, the surface structures will be removed, and the leach tanks flushed to drain pollutants and covered with soil. The evaporation ponds will be covered with 2 feet of soil and the embankments leveled. It is very important that all the disturbed areas be revegetated to curb erosion and the final grades sloped in such a manner as to facilitate adequate surface drainage. The leach tanks should be completely drained prior to covering, with provisions for continued drainage by leaving existing drainage pipes open to the atmosphere. This will help insure that leach tank embankments do not become saturated after the project has been terminated.

DISCUSSION

The construction recommendations presented in this report are based upon the uranium leaching project as currently planned. If design modifications are necessary, either prior to construction or during construction, it is very important that our office be consulted as to the safety of those modifications.

In any subsoil investigation it is necessary to assume that soil engineering characteristics do not vary from those encountered during the test hole drilling or laboratory investigation. Our experience has shown that many times these variations do exist. For this reason, a qualified engineering technician or soils engineer should be present during construction to insure that if soil variations are encountered, new soil characteristics can be determined and analyzed for construction purposes, and to insure that our recommendations are properly interpreted and followed. We are available to discuss this with you at your convenience. Thank you for the opportunity of working with you on this project.

F. M. FOX & ASSOCIATES, INC.

Donald R. Clark, P.E.
Project Engineer
DRC/cae
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Reviewed by:

Frederick M. Fox, Jr., P.E.
President

FREDERICK M. FOX, JR. REGISTERED 4310 PROFESSIONAL ENGINEER STATE OF COLORADO

BORROW AREA SOIL GROUPINGS

BAG SAMPLE FROM PERCOLATION TEST NUMBER	SOIL GROUP
P-1	Group I
P-2	Group I
P-3	Group I
P-4	Group I
P-5	Group I
P-6	Group I
P-7	Group I
P-10	Group I
P-11	Group I
P-14	Group I
P-27	Group II
P-28	Group II
P-29	Group II
P-30	Group II
P-31	Group II
P-32	Group II
P-6	Group III
P-19	Group III
P-20	Group III
P-21	Group III
P-24	Group III
P-15	Group IV*
P-16	Group IV*
P-8	Group V*
P-9	Group V*
P-12	Group V*
P-13	Group V*
P-32 (below 13 feet)	Group V*

* Mudstone bedrock, broken down in the laboratory for lab testing.

TABLE I

BORROW AREA SOIL PROPERTIES

Soil Group	I (CL)	II (CL)	III (SC)	IV (CL)*	V (CL)*
% Sand	34	28	52	37	34
Liquid Limit/ Plasticity Index	25/12	33/18	32/18	46/24	46/27
Shear Strength at 95% ASTM D-1557 (lb/ft^2)	5,760	5,616	5,040	5,472	6,336
Ø angle at 95% ASTM D-1557	0°	12°	13°	4°	10°
Shear Strength Optimum of ASTM D-1557 (lb/ft^2)	9,650	19,152	5,375	9,749	9,216
Maximum Dry Density/ Optimum Moisture Content ASTM D-1557 (psf/%)	124.5/12.0	123.5/12.5	126/11.0	118/16.0	122/14.0

* Mudstone Bedrock groups that were broken down to minus number 4 sieve size for laboratory testing.

Ø Apparent angle of Internal Friction.

TABLE II

CONSOLIDATION TEST DATA

	LOAD INCREMENT (psf)	C_v (10^{-4} cm/sec)	A_v (10^{-6} cm^2/gm)	e	K (10^{-8} cm/sec)
GROUP I	4,320	7.8	9.4	.34	.54
	8,640	6.6	9.4	.32	.46
	17,280	12.8	4.7	.30	.45
	34,360	24.0	2.3	.28	.42
GROUP II	4,320	9.8	19.0	.33	1.36
	8,640	6.8	9.5	.31	.48
	17,280	7.9	7.1	.28	.43
	34,560	9.1	3.6	.25	.26
GROUP III	500	8.7	41.0	.40	2.5
	1,000	12.7	41.0	.39	3.7
	2,000	14.1	20.5	.38	2.1
	4,000	8.3	20.5	.36	1.2
	8,000	10.1	10.2	.34	.76
GROUP IV	500	22.2	61.0	.55	8.6
	1,000	13.7	53.0	.54	4.7
	2,000	11.9	26.6	.53	2.1
	4,000	9.1	19.5	.51	1.2
	8,000	5.4	16.9	.47	0.6
GROUP V	500	22.5	41.0	.47	6.2
	1,000	14.8	41.0	.46	4.1
	2,000	10.3	41.0	.44	2.9
	4,000	7.1	31.0	.41	1.5
	8,000	3.0	15.0	.38	0.3

DEFINITIONS:

C_v - Coefficient of Consolidation
A_v - Coefficient of Compressibility
e - Void Ratio
K - Coefficient of Permeability

TABLE III

SUMMARY OF UNDISTURBED SAMPLE LABORATORY TESTING

XPLORATION HOLE NO.	DEPTH OF SAMPLE (ft)	NATURAL DRY DENSITY (pcf)	NATURAL MOISTURE CONTENT (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	TYPE OF SOIL	REMARKS
1	9	106	10.4	--	CLAY	See Figure A-5
3	4	106	6.9	--	CLAY	See Figure A-5
5	9	114	7.3	--	CLAY	See Figure A-6
7	4	108	6.3	--	CLAY	See Figure A-6
8	4	106	3.3	--	SAND, clayey	See Figure A-7
10	4	--	10.3	--	SHALE	could not run unconfined test due due to sample fractures
10	9	--	10.3	--	SHALE	"
11	4	115.8	13.1	5,850	SHALE	Failed on Fractur
11	14	120.4	11.4	9,750	MUDSTONE	--
12	4	105	11.6	--	CLAY	See Figure A-7
12	9	122	9.2	18,800	CLAY	--
13	4	118	6.2	--	SAND, clayey	See Figure A-8
13	9	132	5.5	--	SAND, clayey	See Figure A-8

TABLE IV

SUMMARY OF UNDISTURBED SAMPLE LABORATORY TESTING

XPLORATION HOLE NO.	DEPTH OF SAMPLE (ft)	NATURAL DRY DENSITY (pcf)	NATURAL MOISTURE CONTENT (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	TYPE OF SOIL	REMARKS
15	9	126	10.7	--	MUDSTONE	See Figure A-9
16	4	108	4.5	--	SAND	See Figure A-9
17	9	127	9.9	--	MUDSTONE	See Figure A-10
19	4	106	7.2	--	CLAY	See Figure A-10
20	4	108	6.2	--	SAND, clayey	See Figure A-11
21	9	114	15.0	--	CLAY	See Figure A-11
22	4	114	7.9	--	CLAY	See Figure A-12

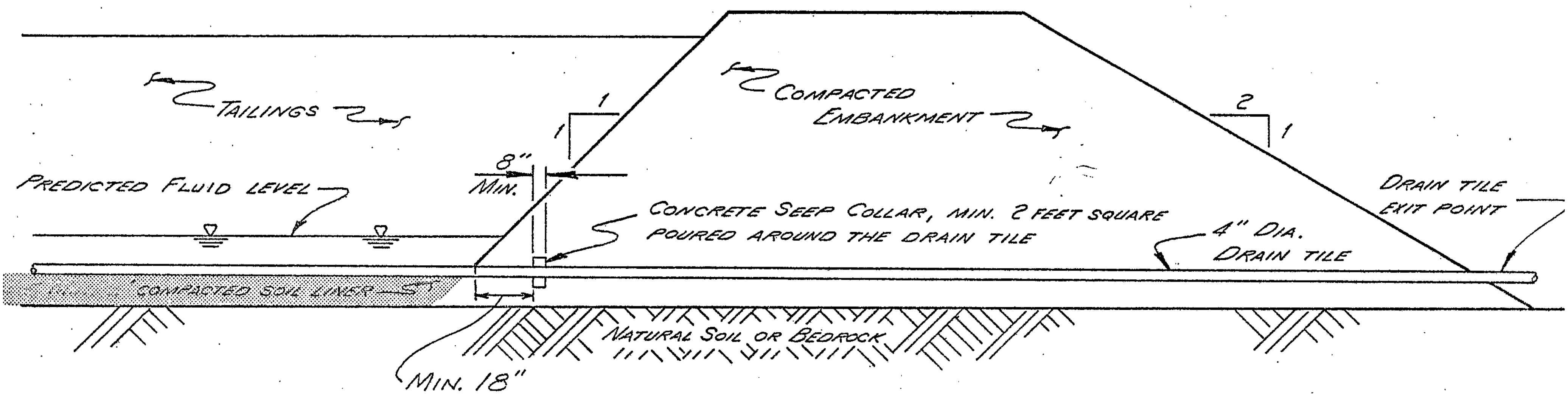
TABLE IV, continued

PERCOLATION RATES AND CALCULATED
IN-SITU PERMEABILITIES

HOLE NUMBER	DEPTH (feet)	SOIL TYPE (lower 3 feet)	PERCOLATION RATE (inches/minute)	CALCULATED PERMEABILITY cm/sec x 10^{-5}
1	4	CLAY	0.0625	4.3
2	3	CLAY	0.0357	2.5
3	3	CLAY	0.0370	2.6
4	4	CLAY	0.0555	3.9
5	4	CLAY	0.0455	3.2
6	3	CLAY	0.0400	2.8
7	4	CLAY	0.0323	2.7
8	3	MUDSTONE	0.0167	1.2
9	4	MUDSTONE	0.0110	0.76
10	4	SAND, clayey with GRAVEL	0.0357	2.5
11	3	CLAY	0.0225	1.6
12	3	MUDSTONE	0.0048	0.34
13	3	MUDSTONE	0.0200	1.4
14	3	CLAY	0.0263	1.8
15	5	MUDSTONE	0.0227	1.6
16	10	MUDSTONE	0.0164	1.1
17	10	MUDSTONE	0.0140	0.98
18	12	SAND, clayey with GRAVEL	0.0400	2.8
19	15	MUDSTONE	0.0555	3.9
20	15	MUDSTONE	0.0250	1.8
21	5	MUDSTONE	0.0212	1.5
22	4	CLAY	0.0250	1.8
23	4	CLAY	0.0323	2.3
24	15	MUDSTONE	0.0455	3.2
25	15	MUDSTONE	0.0227	1.6
26	5	CLAY	0.0417	2.9
27	3	CLAY	0.0476	3.4
28	7	CLAY	0.0426	3.0
29	10	CLAY	0.0667	4.7
30	7	CLAY	0.0769	5.4
31	15	CLAY	0.0800	5.6
32	17	MUDSTONE	0.0800	5.6

NOTE: Percolation tests were conducted in 6 inch diameter drilled holes.

TABLE V



Note: Backfill around the drain tile and seep collar should be compacted to 95% of ASTM D-1557.

SEEP COLLAR DETAIL
FOR TYPICAL LEACH TANK EMBANKMENT

SCALE: 1" = 10'

FIGURE 3

APPENDIX A

LABORATORY TEST DATA

APPENDIX A

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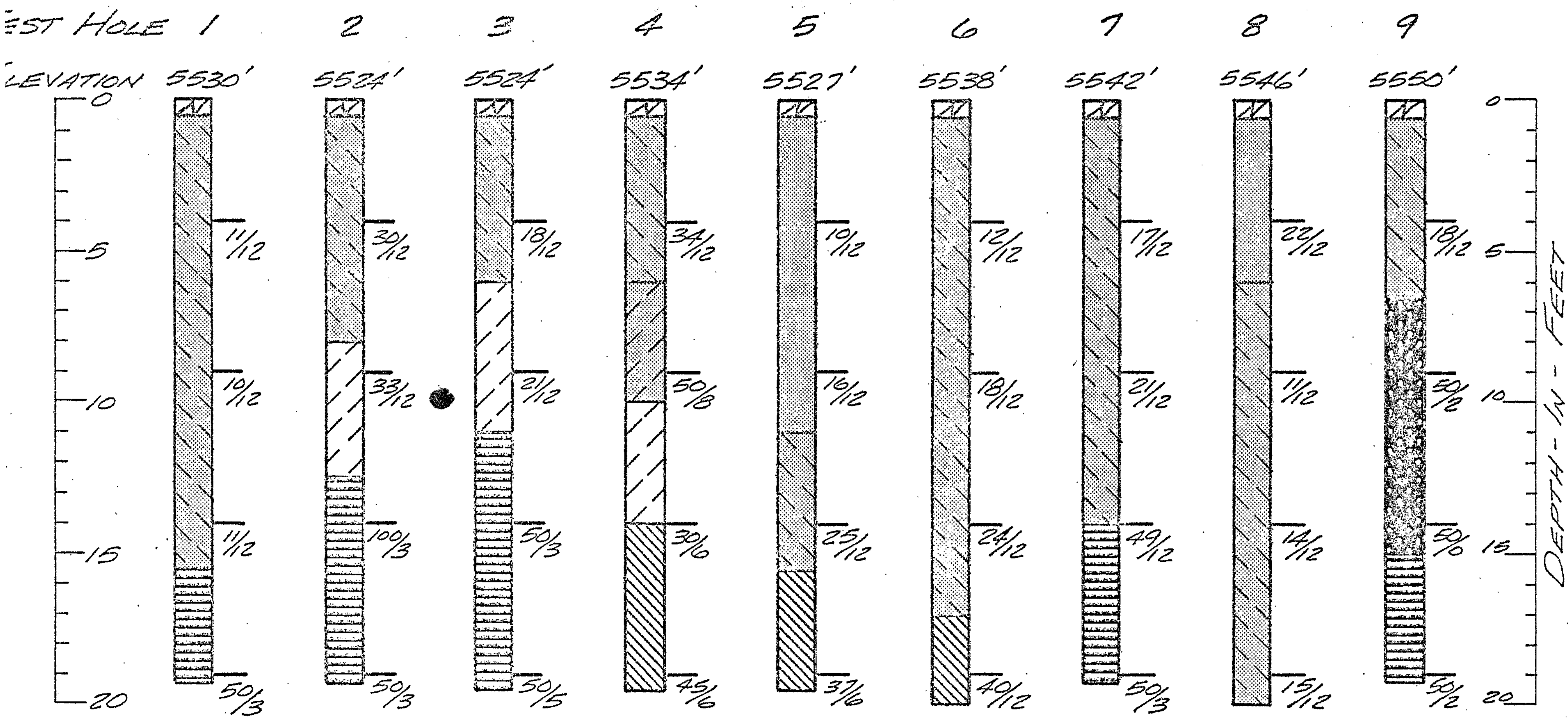


Refer to Figure A-4 for legend and notes.

FIGURE A-1

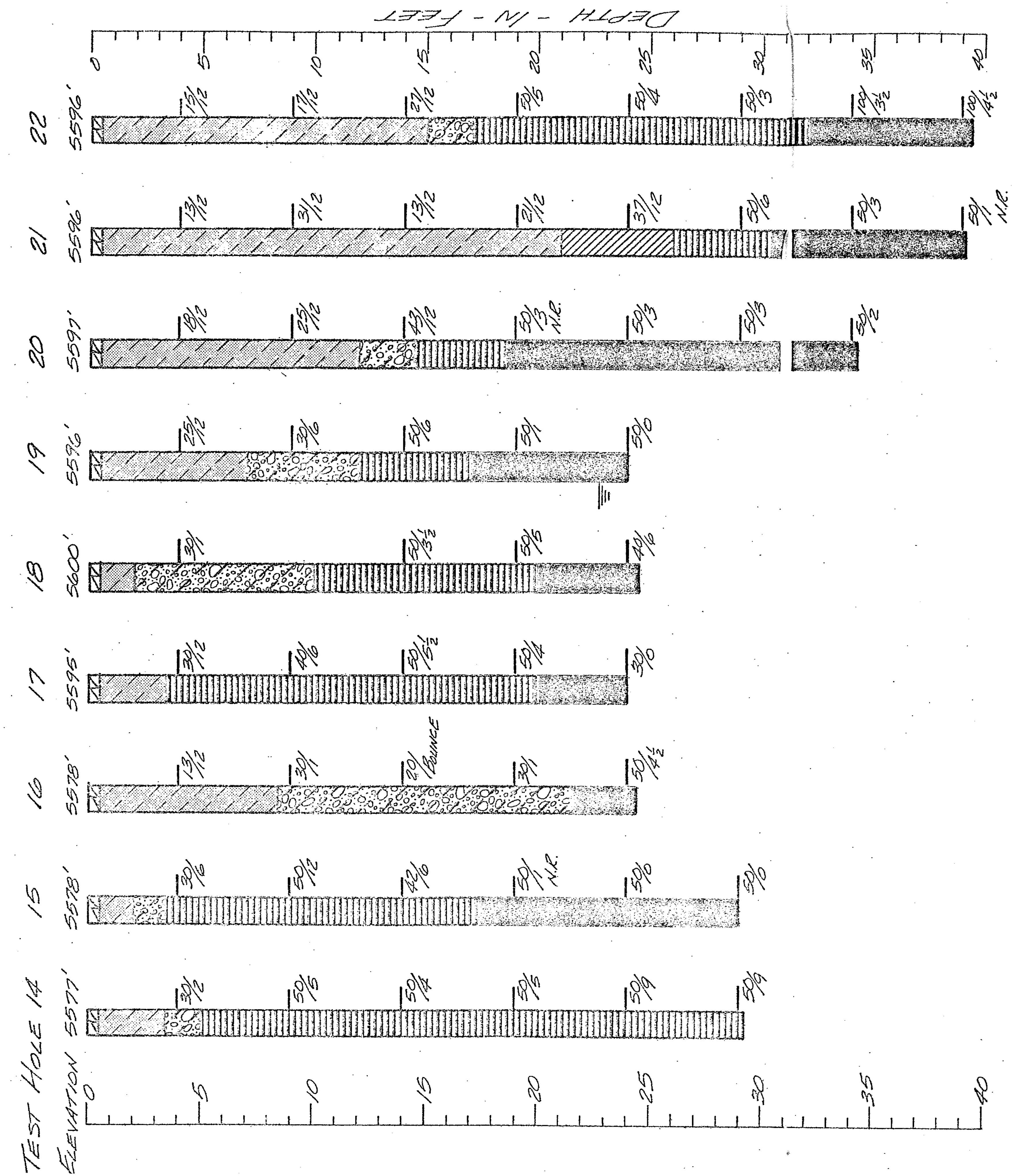


FIGURE A-3

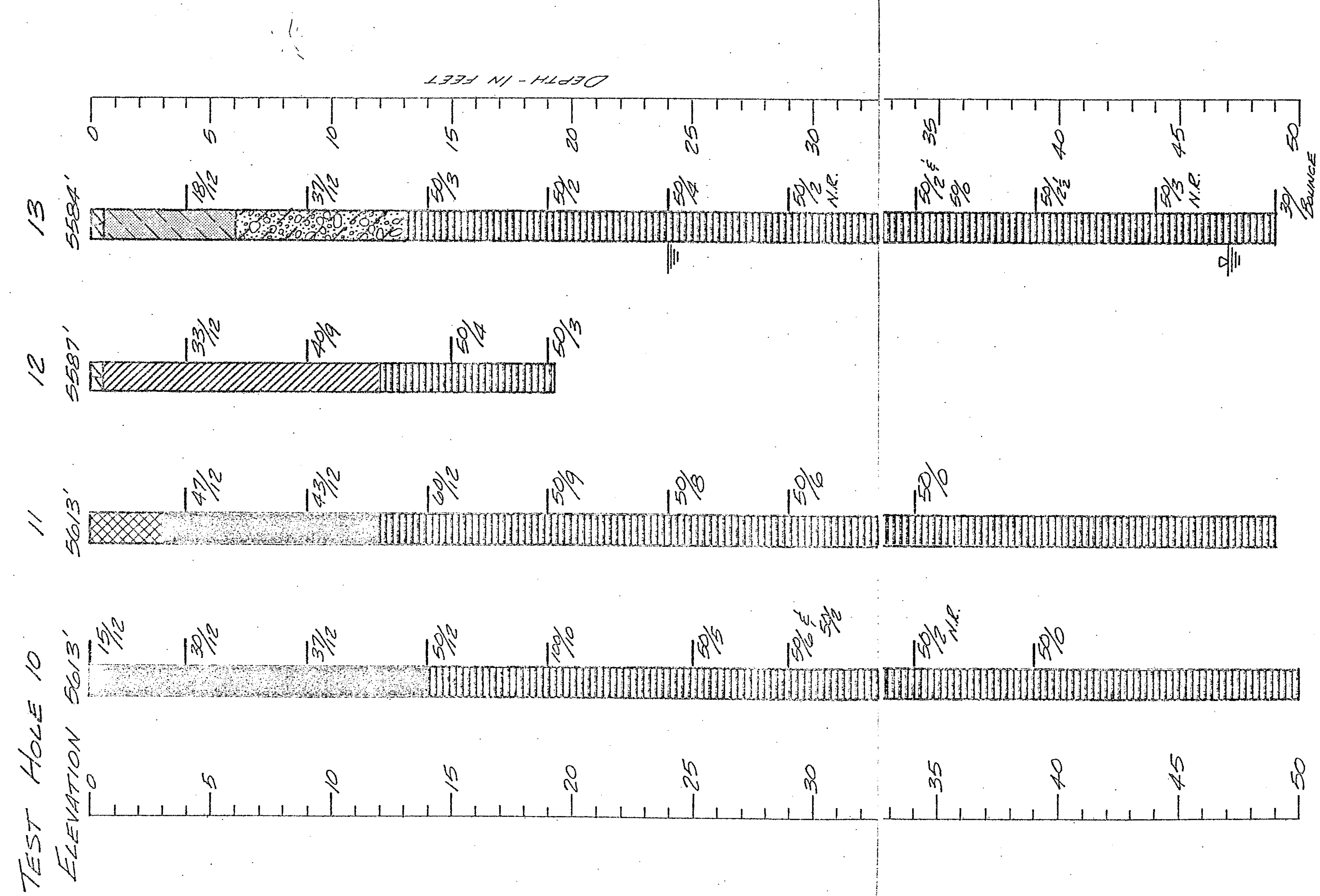

TEST HOLE
ELEVATION
DEPTH - IN FEET
10
5613'
11
5613'
12
5587'
13
5584'
0
5
10
15
20
25
30
35
40
45
50

FIGURE A-2

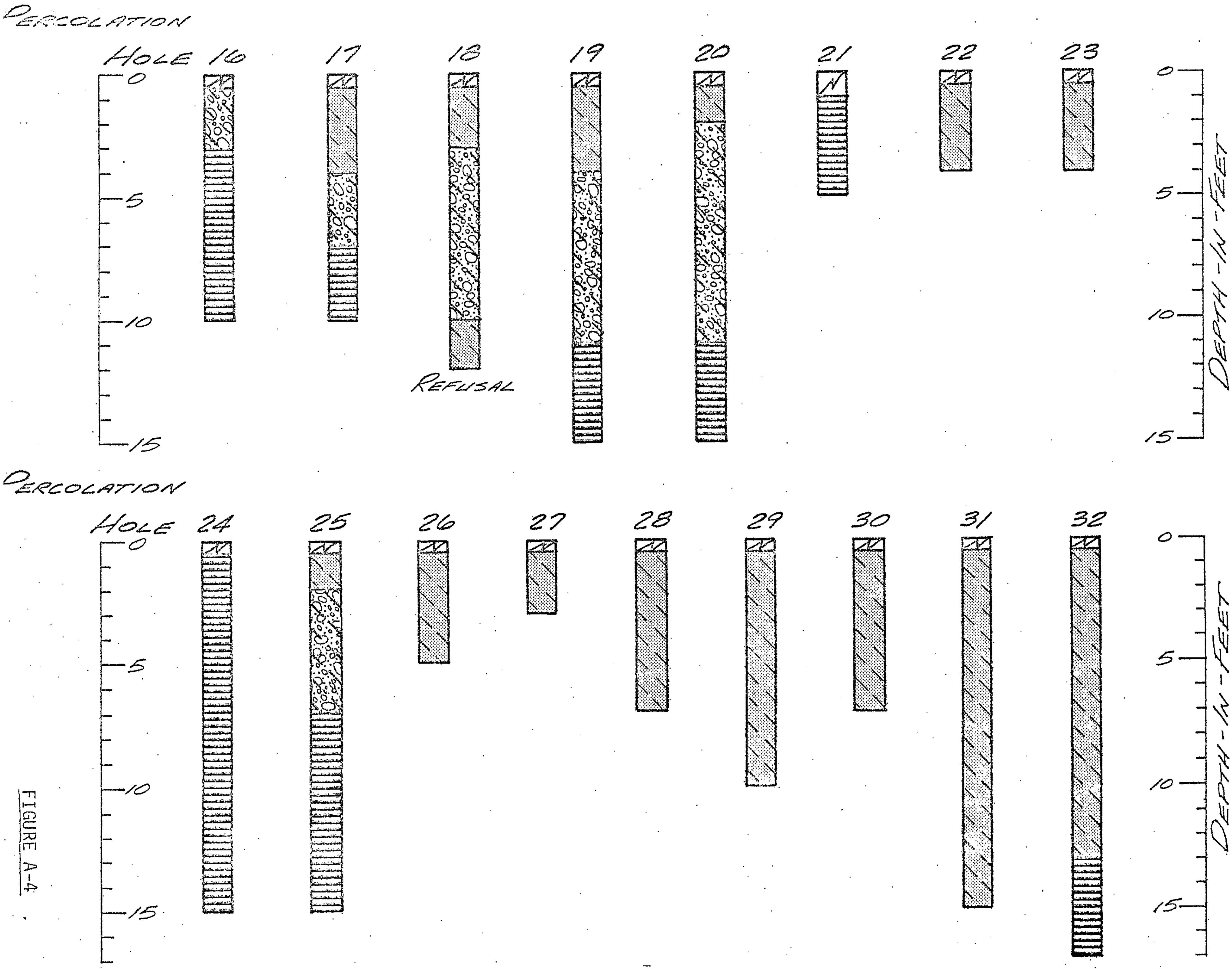


FIGURE A-4

LEGEND

MAN-MADE FILL

TOPSOIL

CLAY, silty, sandy, stiff to very stiff, slightly moist to medium moist, red brown (CL-SM)

CLAY, silty to SILT, some clayey, sandy, medium dense, slightly moist, light gray (CL-SM)

SILT, sandy in part, medium dense, slightly moist, light gray (ML)

CLAY, sandy in part, silty in part, very stiff to hard, moist, medium to high plasticity, dark green to red brown (CL-CH)

SAND, fine grained, silty, loose to medium dense, slightly moist, light red brown (SP)

SAND & GRAVEL, poorly graded, some cobbles, silty, clayey, slightly moist, medium dense to dense, brown to red brown (SM-GM & SC-GC)

SANDSTONE BEDROCK, very hard, firm to medium grained, well cemented, slightly clayey, medium moist to moist, yellow brown to light gray

MUDSTONE, some very sandy, hard to very hard, moist, fractured near top, gray

SHALE, hard to very hard, moist, fractured near top, dark gray to black

N.R. Indicates no sample recovered.

Indicates water table at time of drilling.

Indicates free water level 1 day after drilling.

REFUSAL Indicates refusal encountered while drilling with a 4 inch diameter power auger.

NOTES

1. Test holes were drilled on February 20 through 24, 1977 with a 4 inch diameter continuous flight auger.

2. Percolation holes were drilled with a 6 inch diameter power auger.

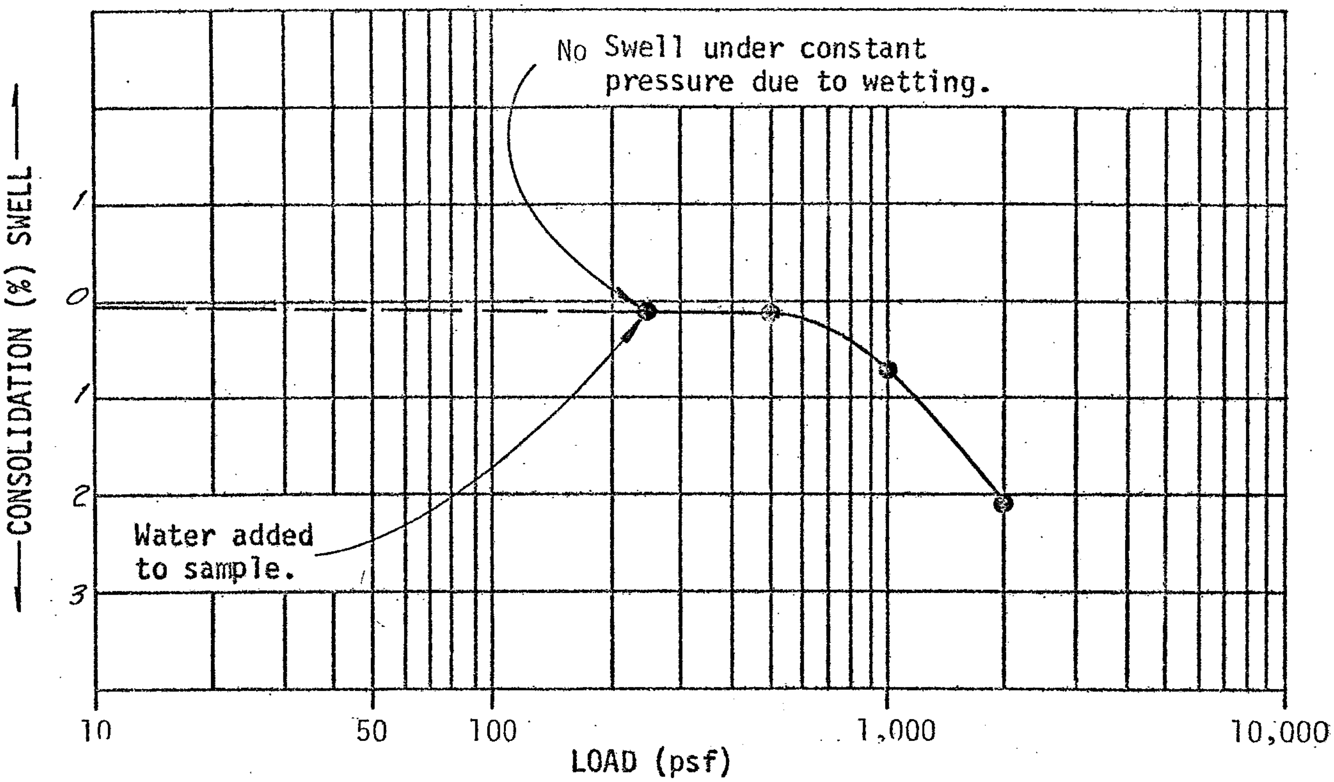
3. (11/12) location of Standard Penetration Test; indicates that 11 blows with a 140 pound hammer, falling 30 inches, were required to drive a 2 inch diameter sampler 12 inches.

4. Test hole elevations were supplied by the Barnes Engineering Company.

LOGS OF TEST HOLES

SWELL-CONSOLIDATION TESTS

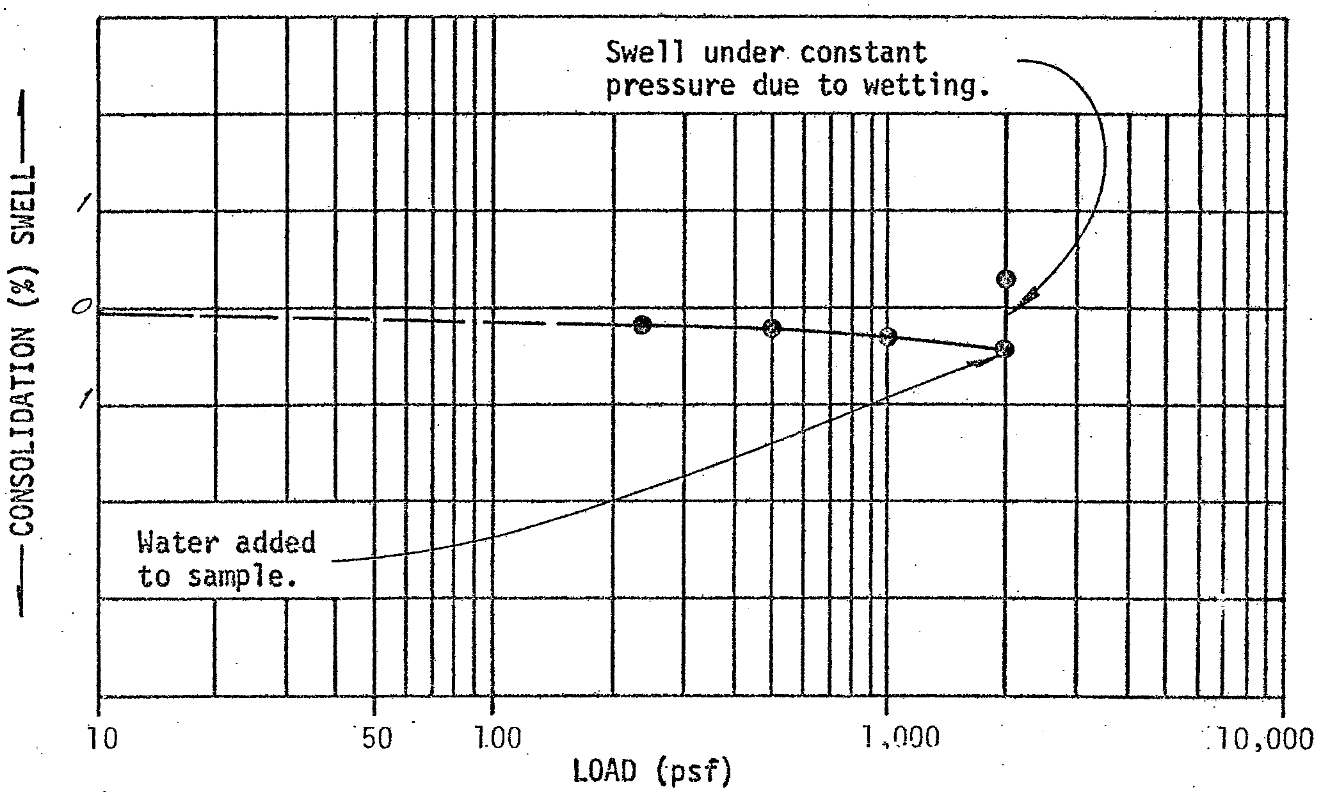
Hole 1 @ 7



Sample of CLAY from test hole 1 from depth 9 feet.

Natural Moisture Content 10.4% Natural Dry Density 106 pcf

Job No. 1-2955 Hole 3 @ 4



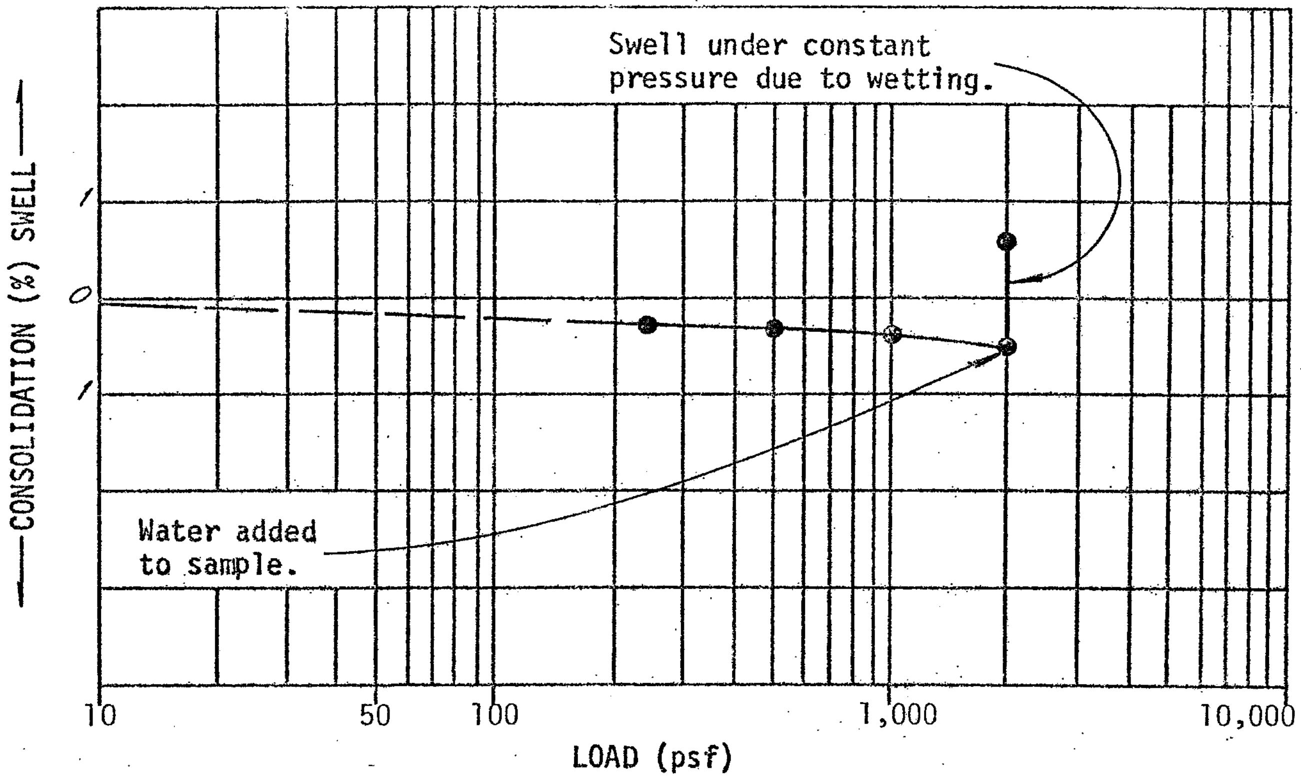
Sample of CLAY from test hole 3 from depth 4 feet.

Natural Moisture Content 6.9% Natural Dry Density 106 pcf

Form 1-109

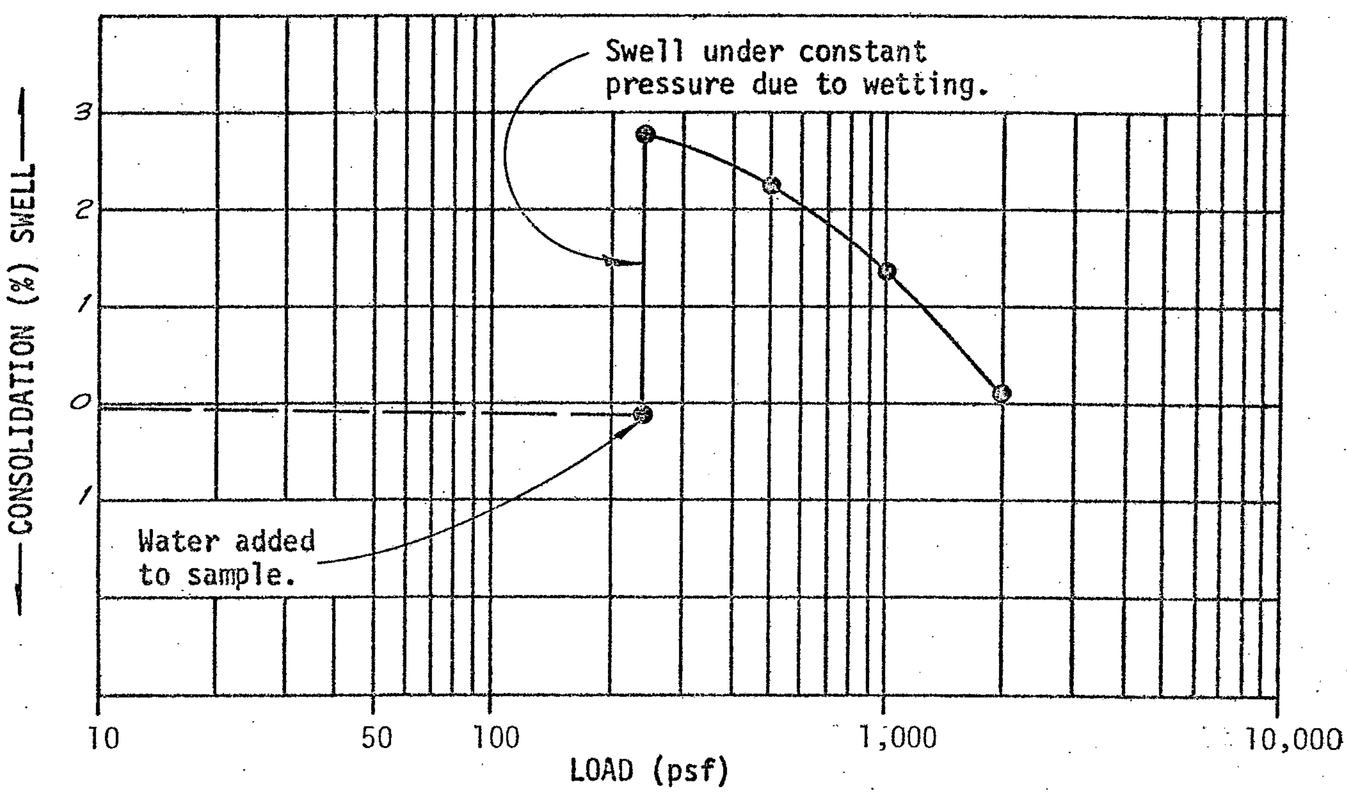
FIGURE A-5

SWELL-CONSOLIDATION TESTS



Sample of CLAY from test hole 5 from depth 9 feet.

Natural Moisture Content 7.3% Natural Dry Density 114 pcf



Sample of CLAY from test hole 7 from depth 4 feet.

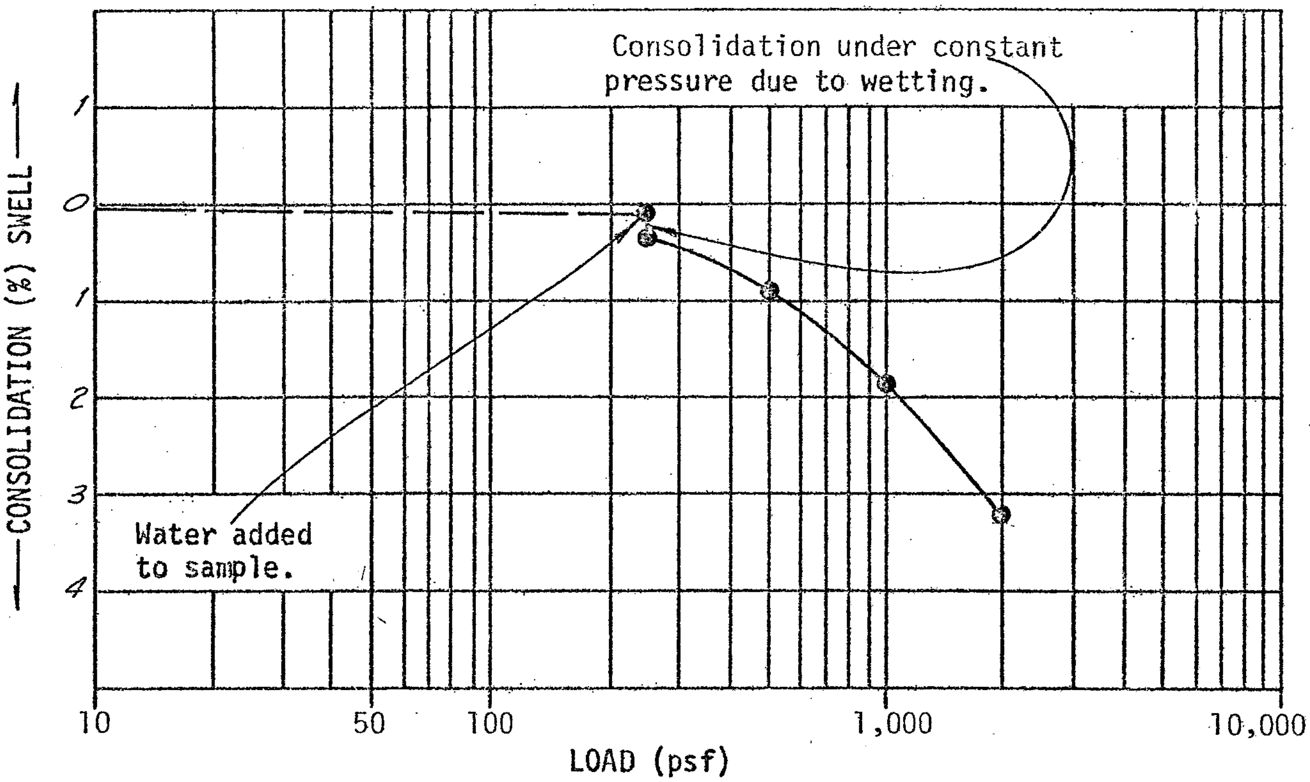
Natural Moisture Content 6.3% Natural Dry Density 108 pcf

Hole

Job No. Hole

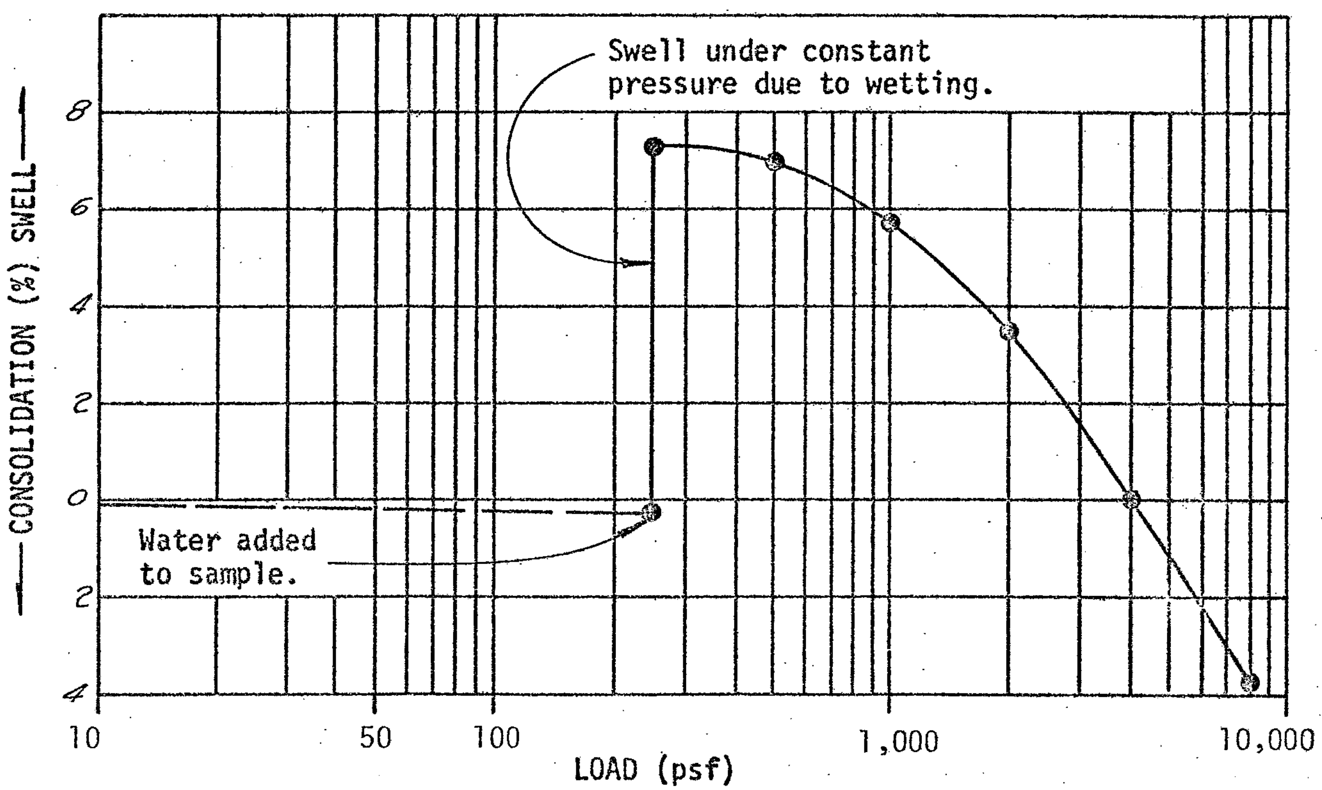
Form 1-109

SWELL-CONSOLIDATION TESTS



Sample of SAND, clayey from test hole 8 from depth 4 feet.

Natural Moisture Content 3.3% Natural Dry Density 106 pcf



Sample of CLAY from test hole 12 from depth 4 feet.

Natural Moisture Content 11.6% Natural Dry Density 105 pcf

Hole 8 @ 4

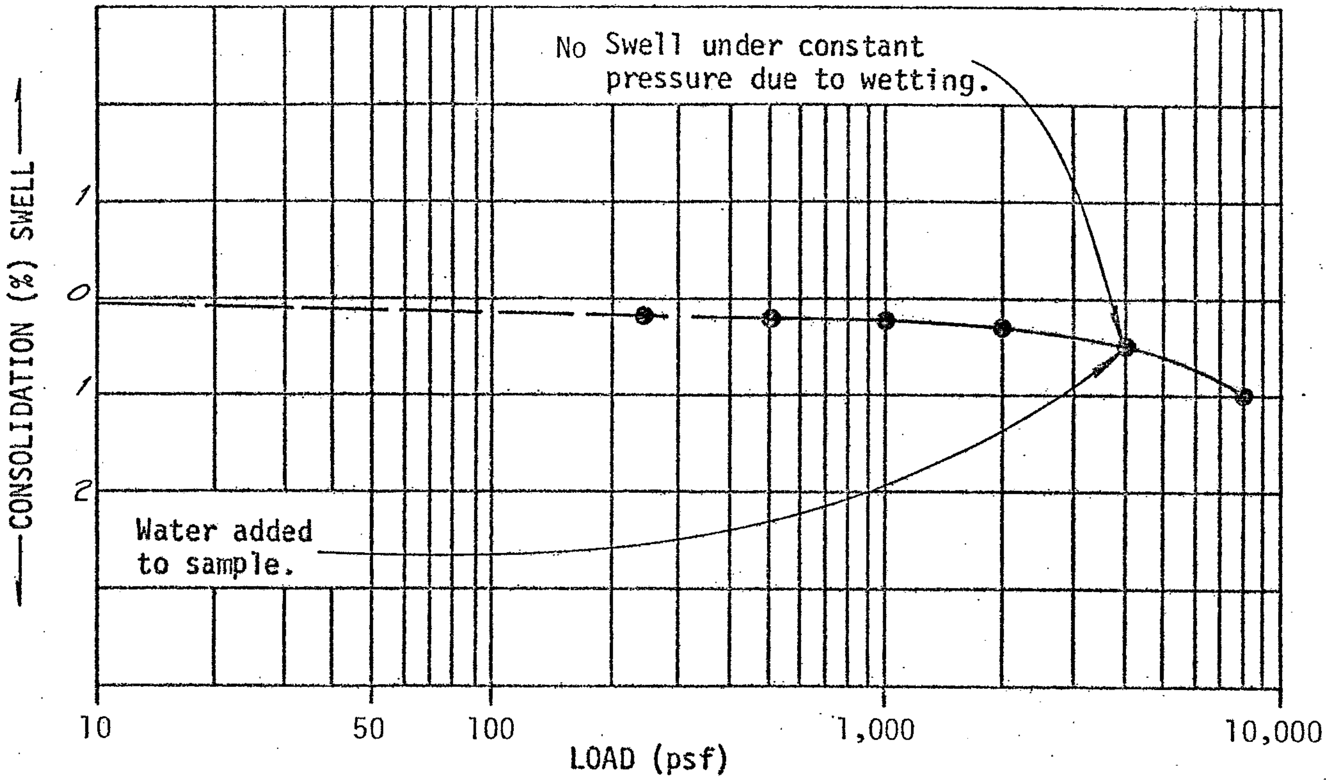
Hole 12 @ 4

JOB NO 12295

Form 1-109

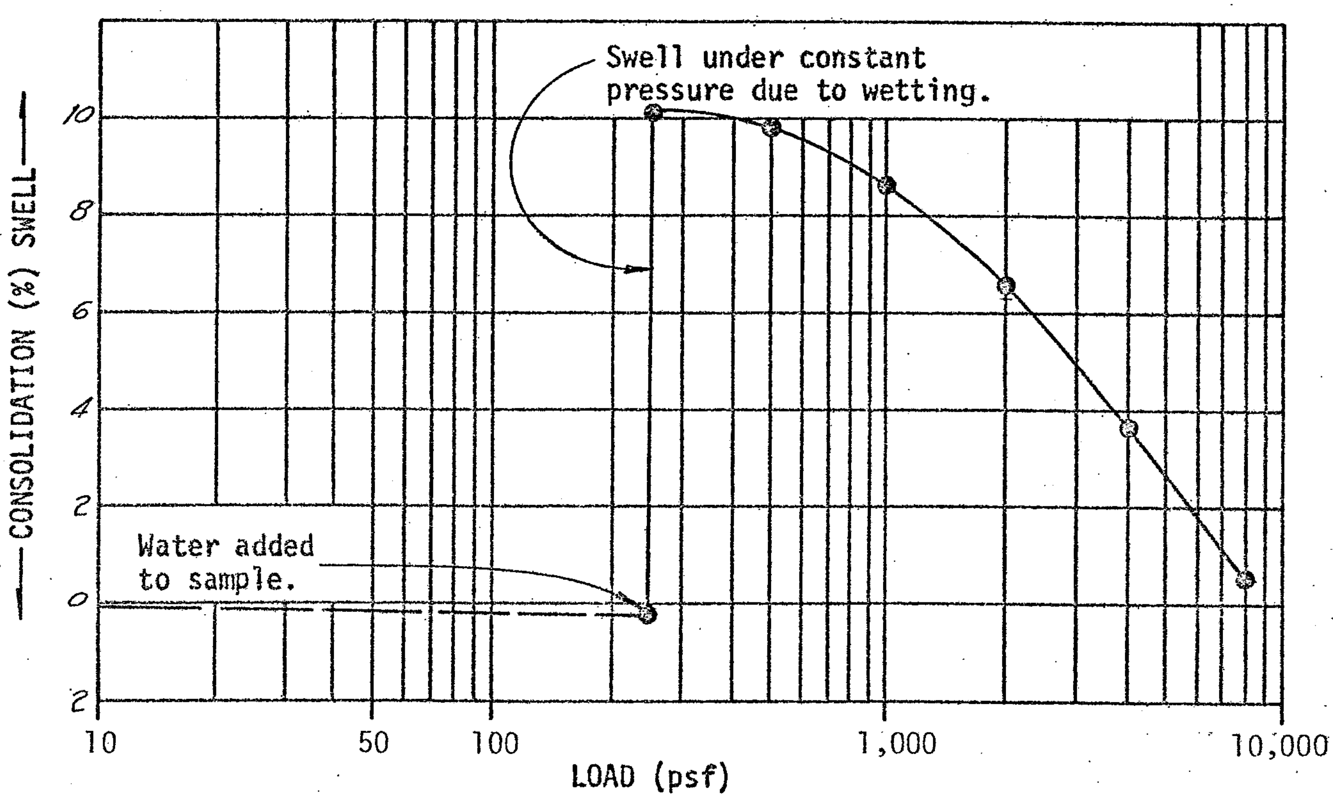
FIGURE A-7

SWELL-CONSOLIDATION TESTS



Sample of SAND, clayey from test hole 13 from depth 4 feet.

Natural Moisture Content 6.2% Natural Dry Density 118 pcf



Sample of SAND, clayey from test hole 13 from depth 9 feet.

Natural Moisture Content 5.5% Natural Dry Density 132 pcf

Hole 13 @ 4

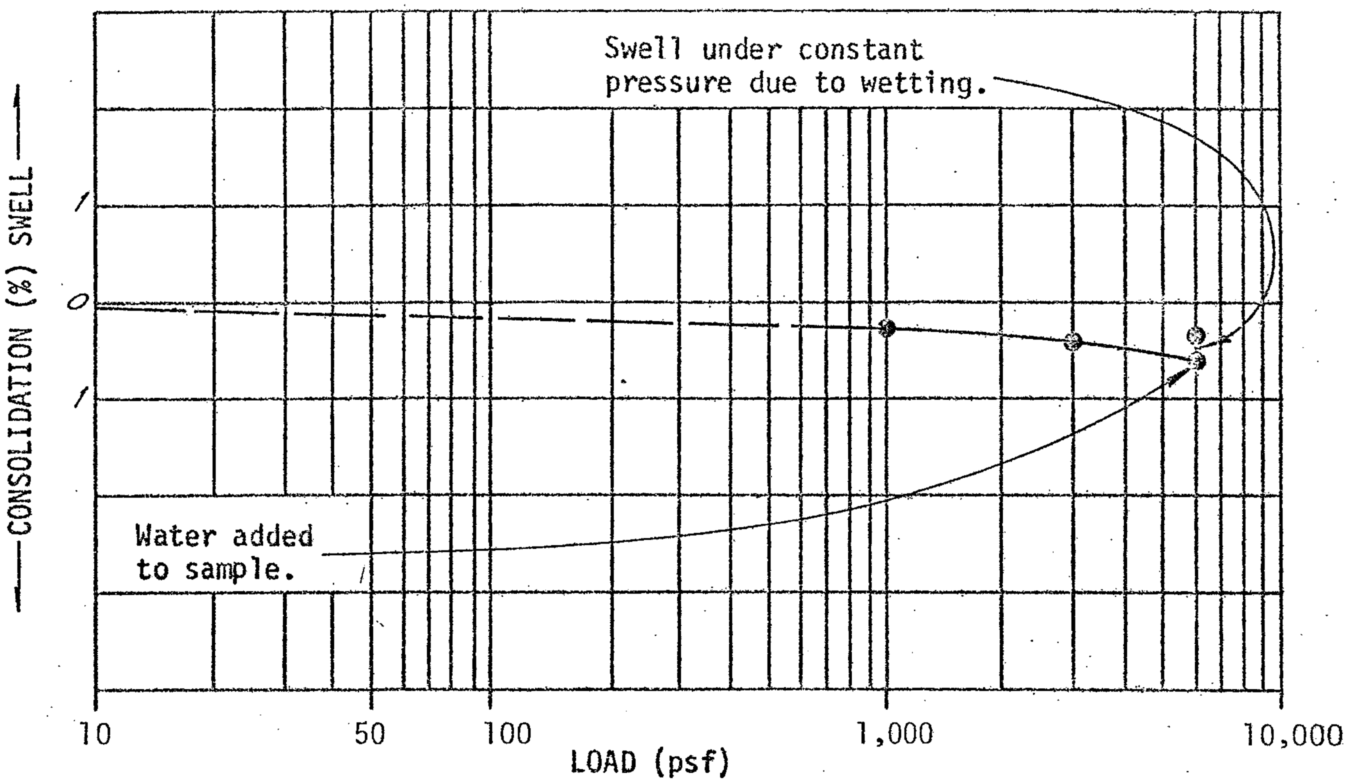
Job No. 1-2455 Hole 13 @ 9

Form 1-109

FIGURE A-8

SWELL-CONSOLIDATION TESTS

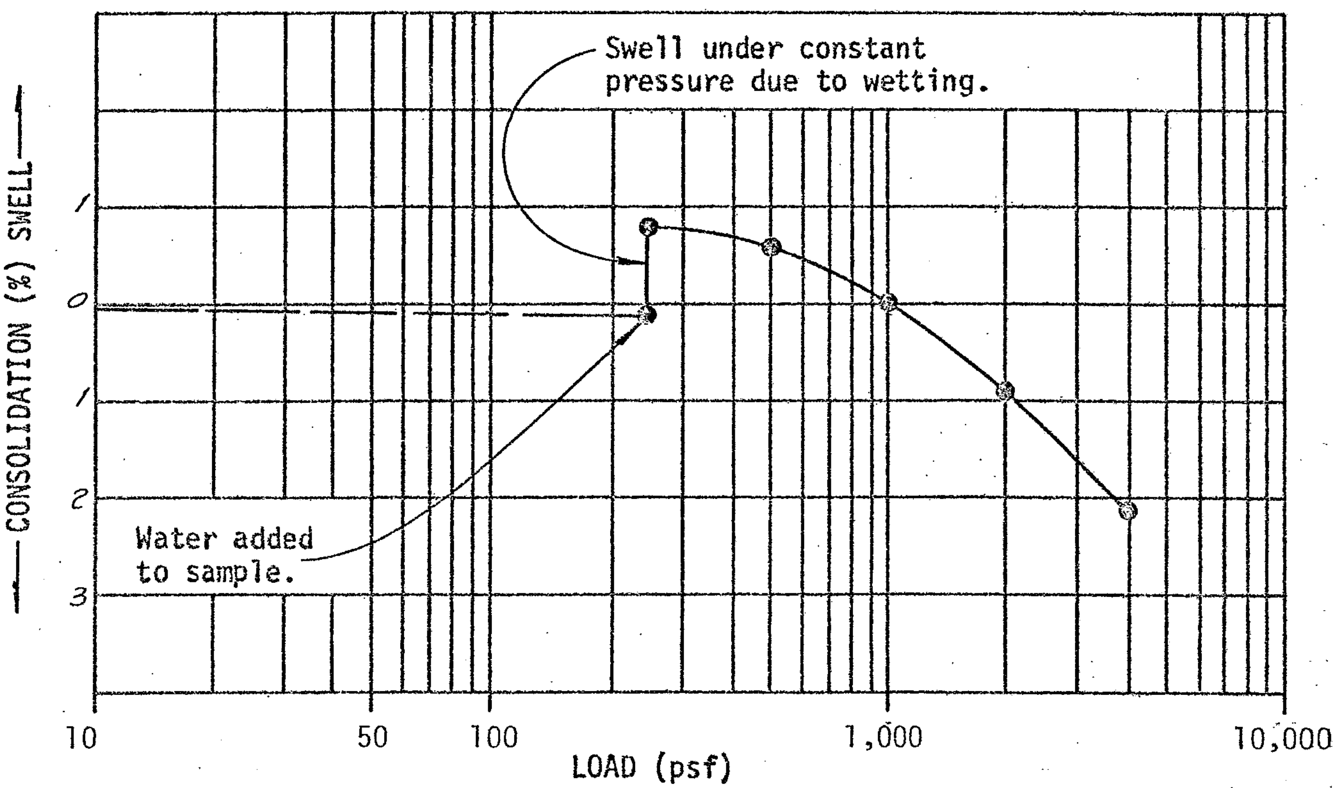
Hole 15 @ 9



Sample of MUDSTONE from test hole 15 from depth 9 feet.

Natural Moisture Content 10.7% Natural Dry Density 126 pcf

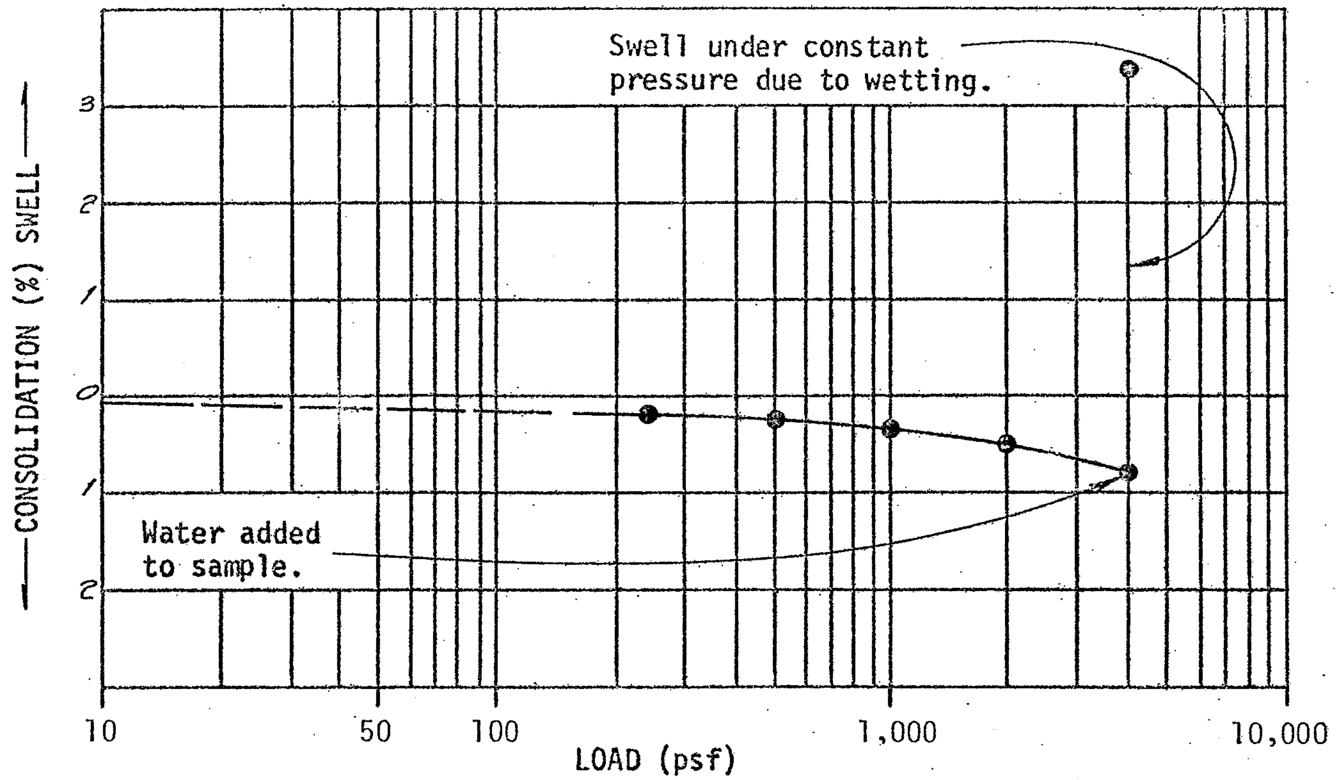
Job No. 1-2355 Hole 16 @ 4



Sample of SAND from test hole 16 from depth 4 feet.

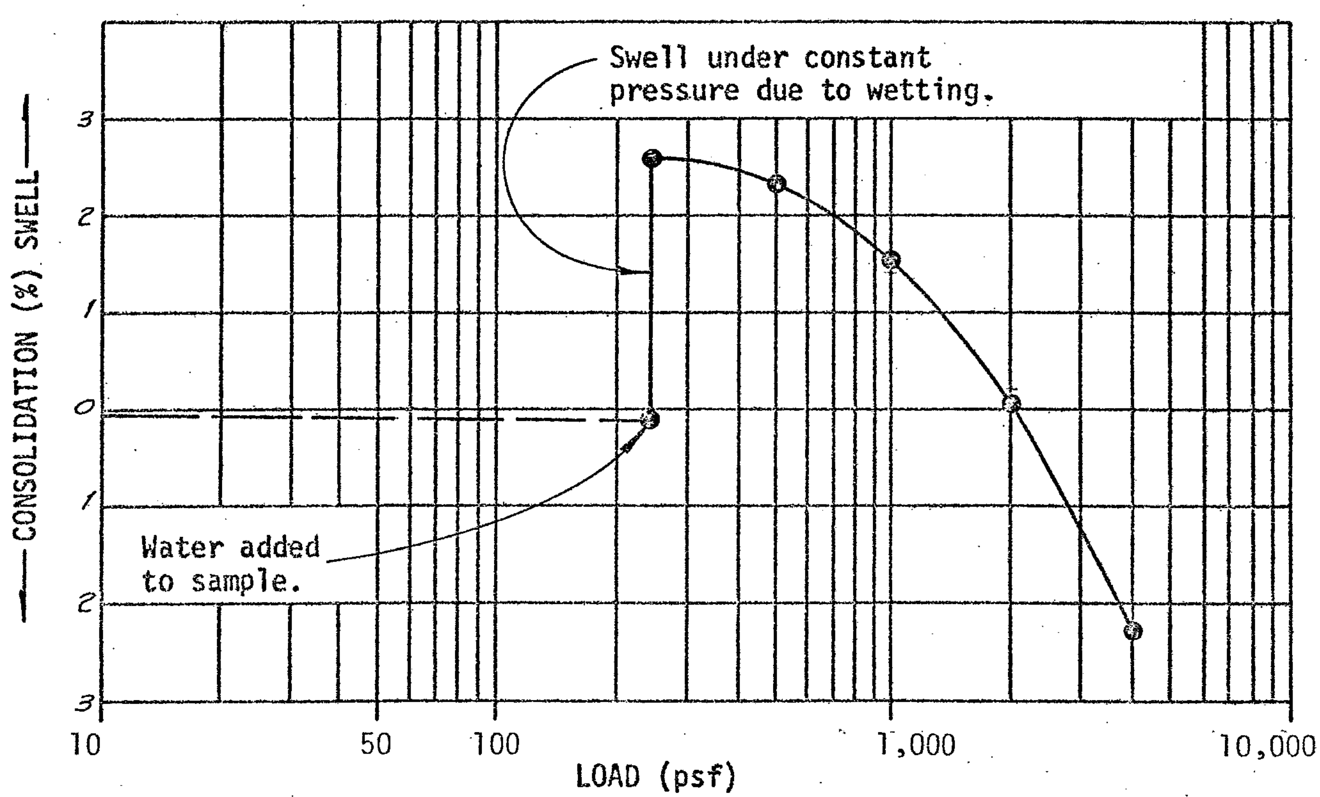
Natural Moisture Content 4.5% Natural Dry Density 108 pcf

SWELL-CONSOLIDATION TESTS



Sample of MUDSTONE from test hole 17 from depth 9 feet.

Natural Moisture Content 9.9% Natural Dry Density 127 pcf



Sample of CLAY from test hole 19 from depth 4 feet.

Natural Moisture Content 7.2% Natural Dry Density 106 pcf

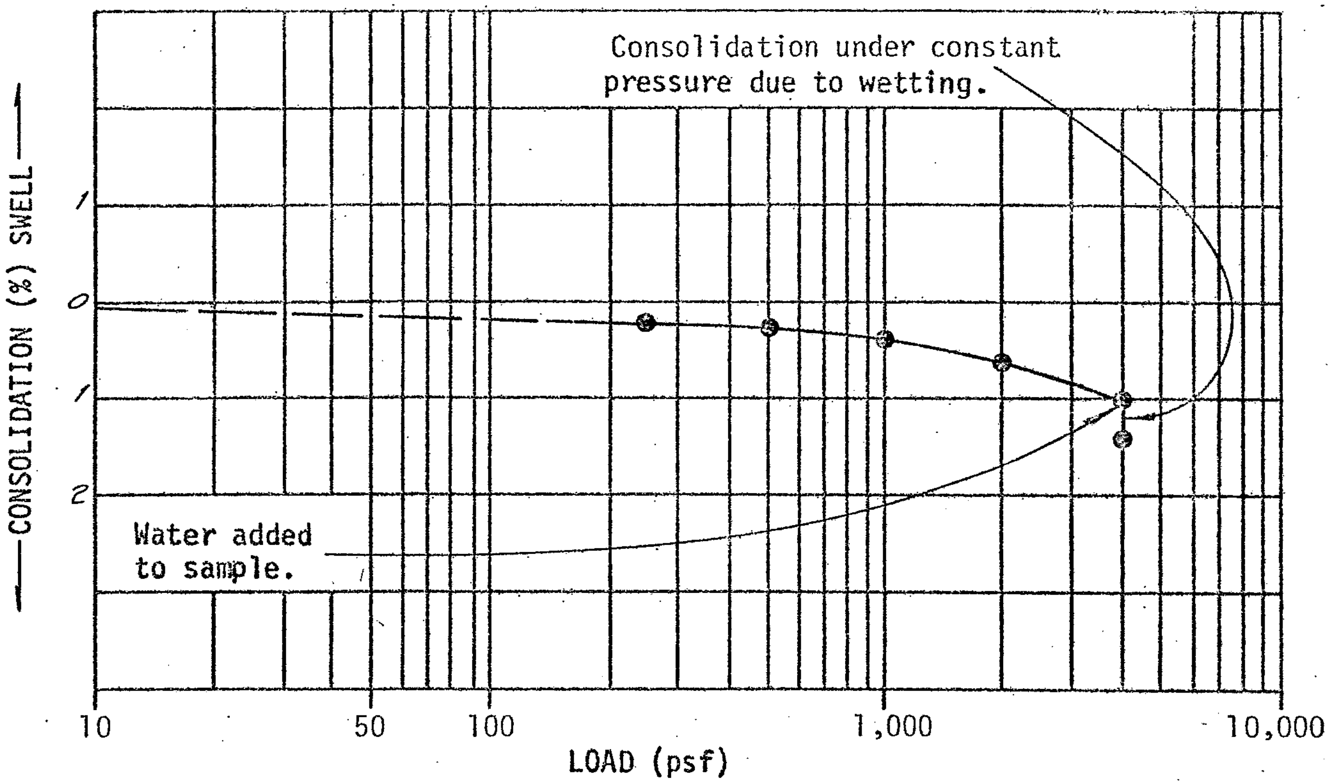
Hole 17 @ 9

Job No. 1-2455 Hole 19 @ 4

Form 1-109

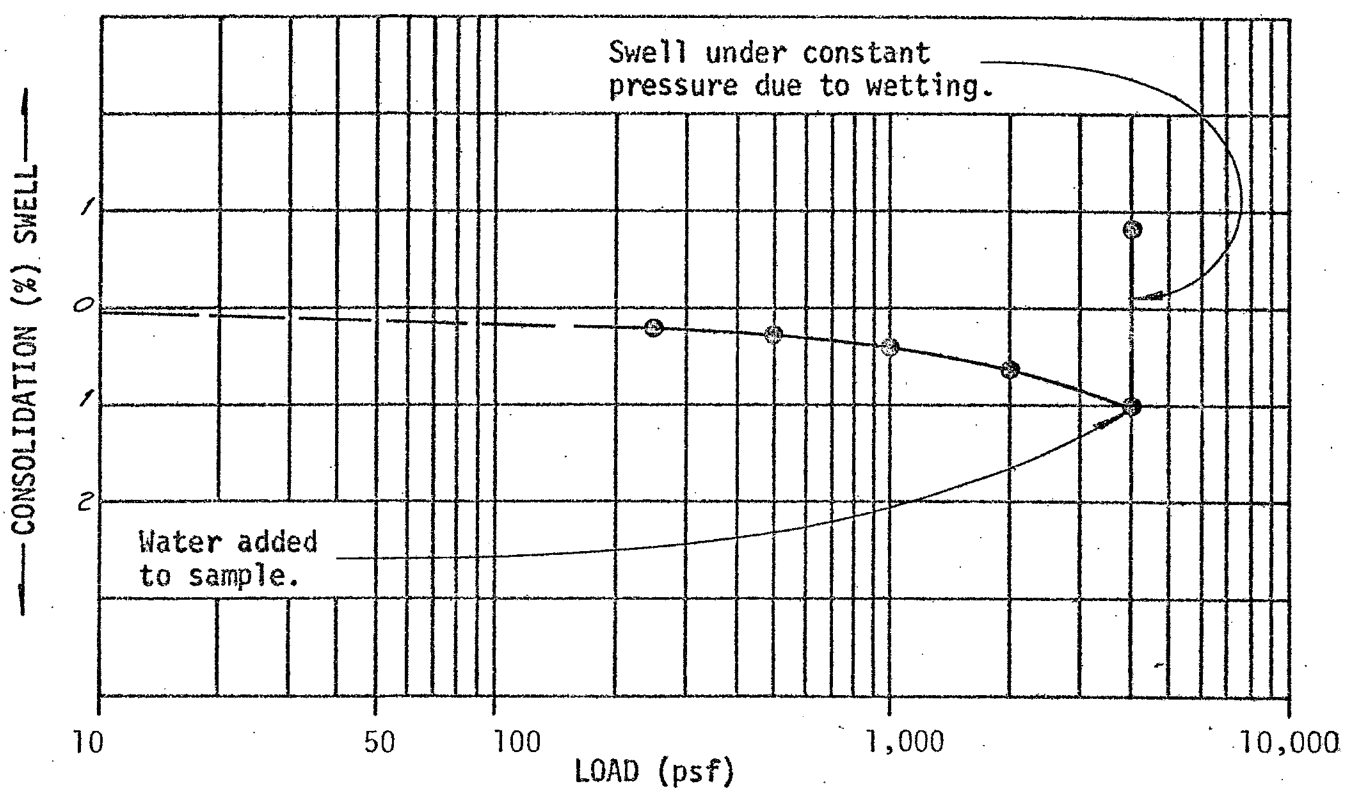
FIGURE A-10

SWELL-CONSOLIDATION TESTS



Sample of SAND, clayey from test hole 20 from depth 4 feet.

Natural Moisture Content 6.2% Natural Dry Density 108 pcf

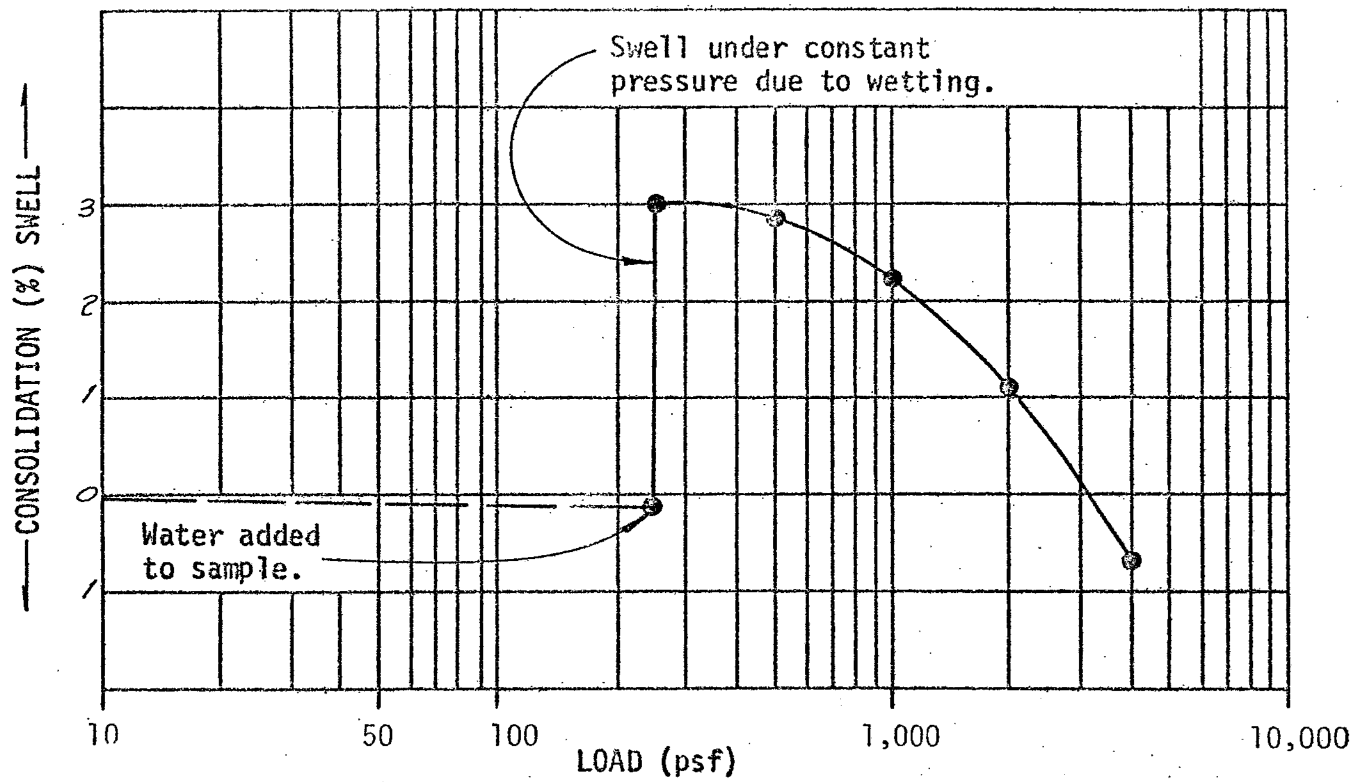


Sample of CLAY from test hole 21 from depth 9 feet.

Natural Moisture Content 15.0% Natural Dry Density 114 pcf

Hole 20 @ [illegible] — Hole 21 @ [illegible] — Job No 1-2955

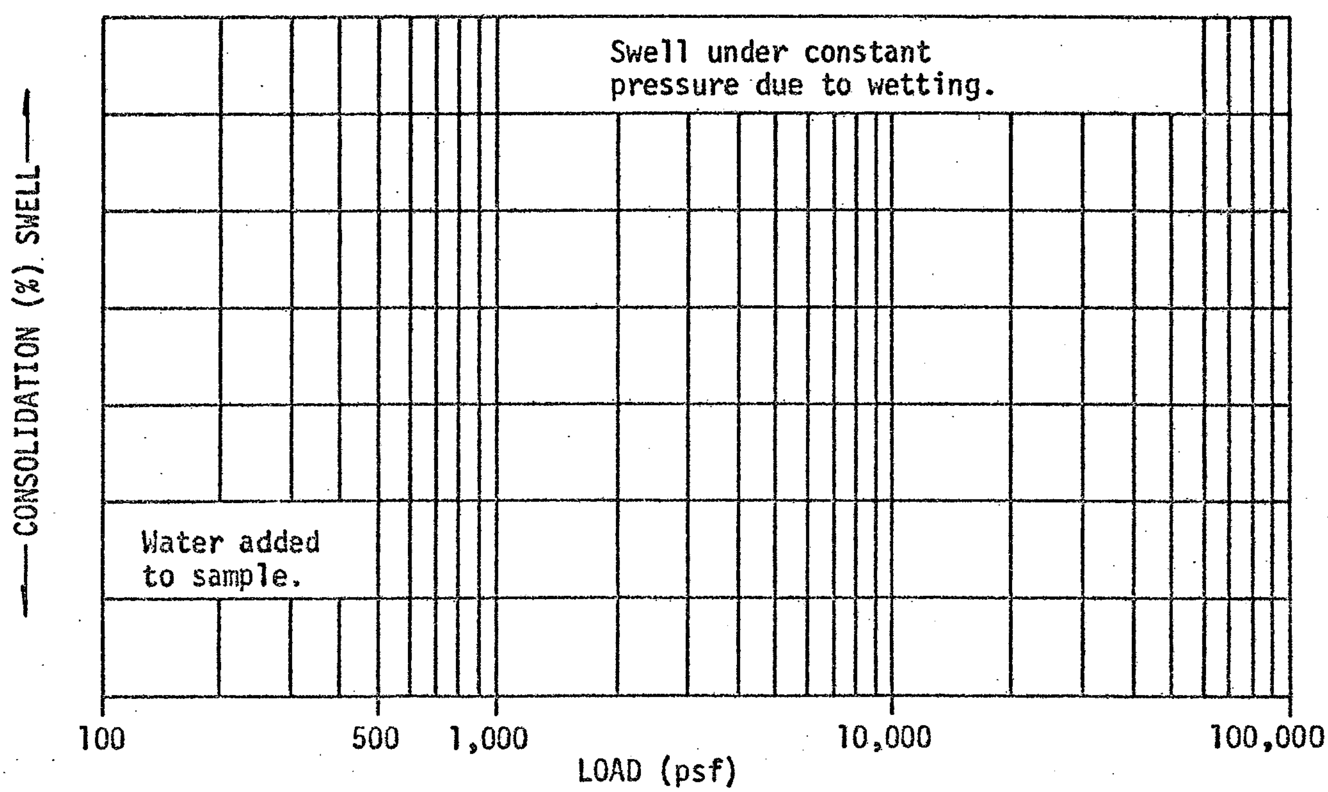
SWELL-CONSOLIDATION TESTS



Sample of CLAY from test hole 22 from depth 4 feet.

Natural Moisture Content 7.9% Natural Dry Density 114 pcf

Hole 22 @ 4

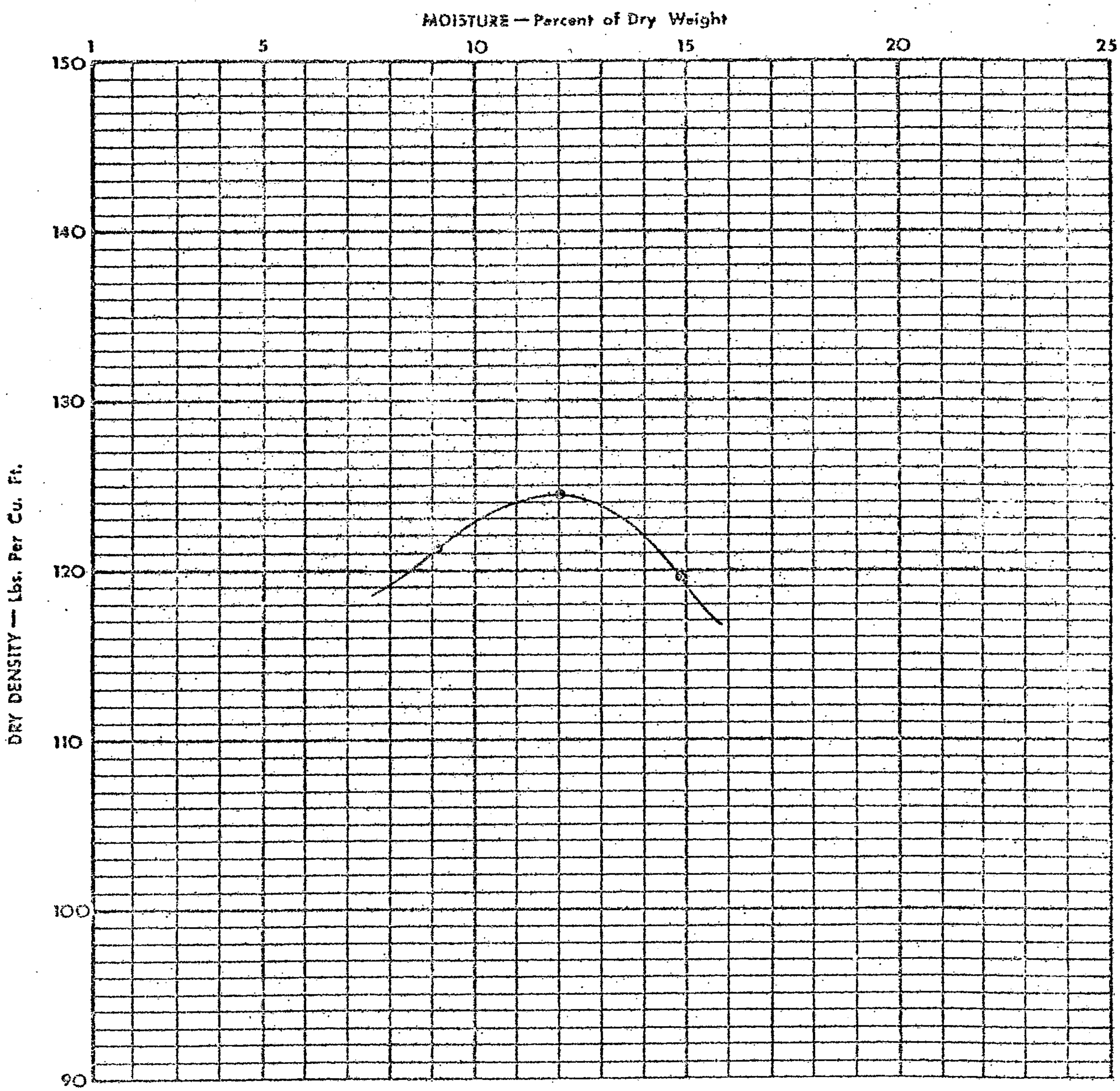


Sample of ____________ from test hole ____ from depth ____ feet.

Natural Moisture Content ____________ Natural Dry Density ____________

Job No. 1-2,55 Hole ____ @ ____

COMPACTION TEST RESULTS



Maximum Dry Density (P.C.F) 124.5

Optimum Moisture Content (%) 12.0

Sample Description Group I, CLAY, silty, sandy

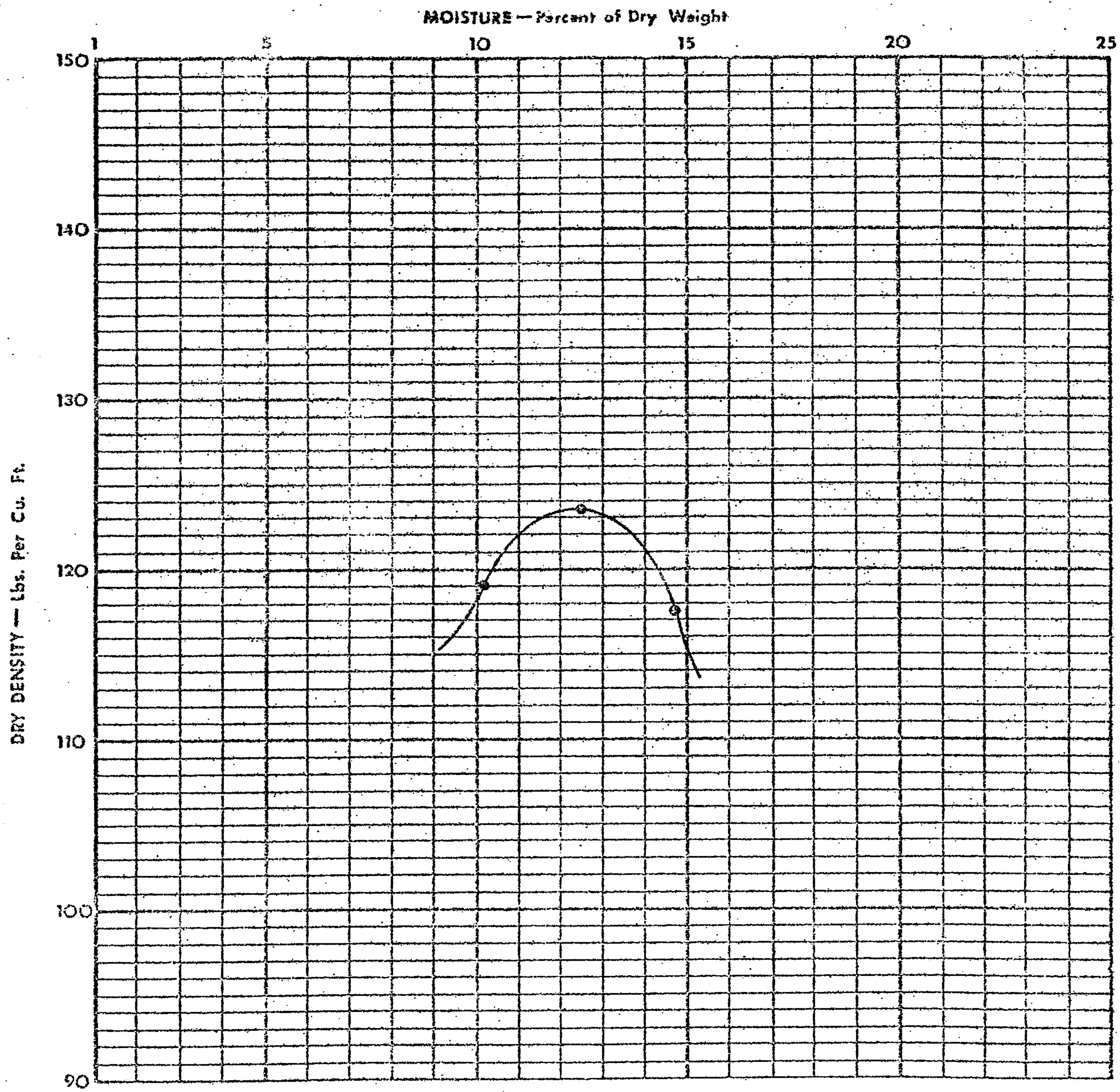
From Proposed uranium leaching operation site

Compaction Test Procedure ASTM D-1557

Form 4-1

F. M. Fox & Assoc. Inc.

COMPACTION TEST RESULTS



Maximum Dry Density (P.C.F) 123.5

Optimum Moisture Content (%) 12.5

Sample Description Group II, CLAY, silty

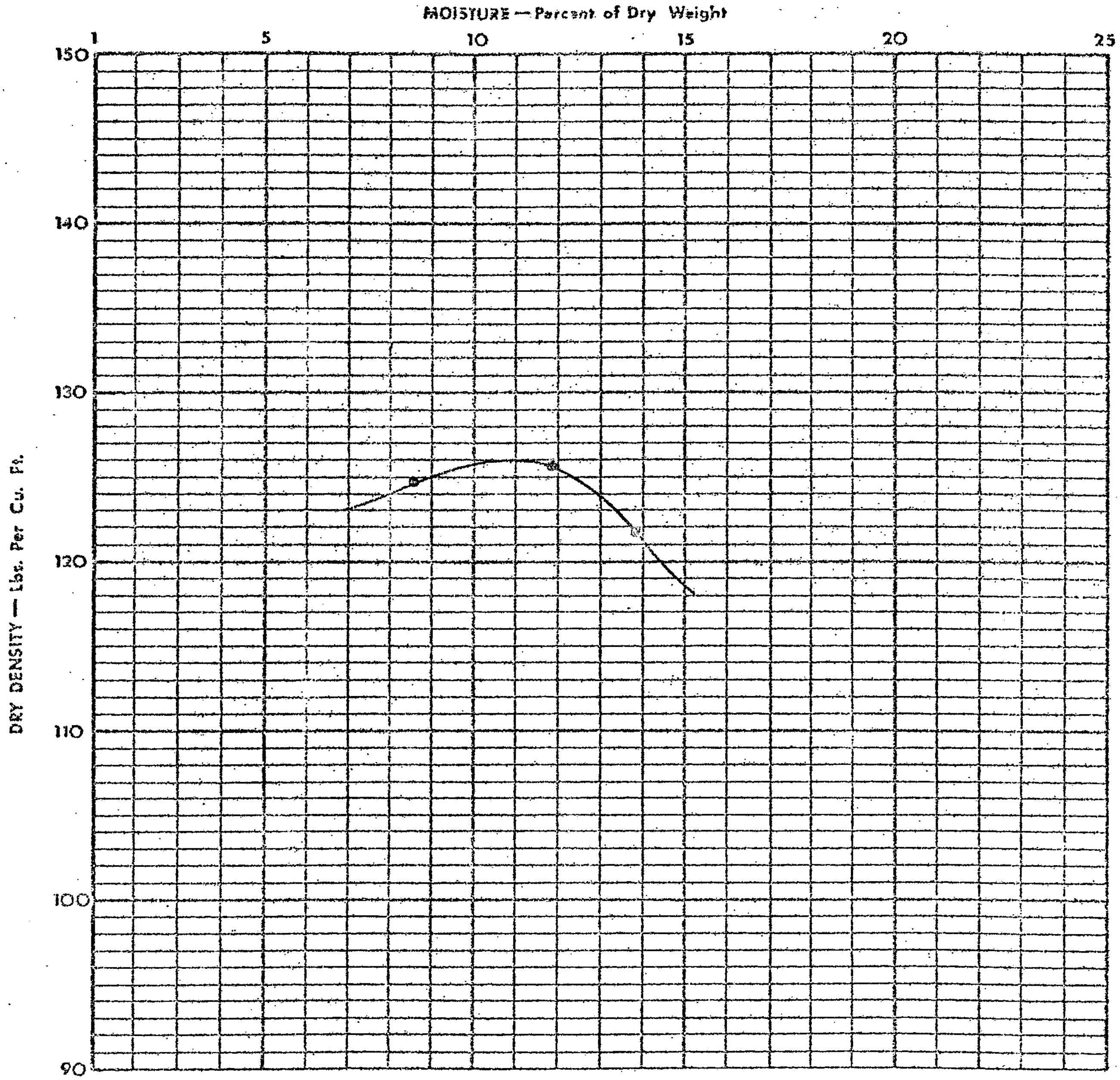
From Proposed uranium leaching operation site

Compaction Test Procedure ASTM D-1557

Form 4-1

F. M. Fox & Assoc. Inc.

COMPACTION TEST RESULTS



Maximum Dry Density (P.C.F) 126.0

Optimum Moisture Content (%) 11.0

Sample Description Group III, SAND, clayey

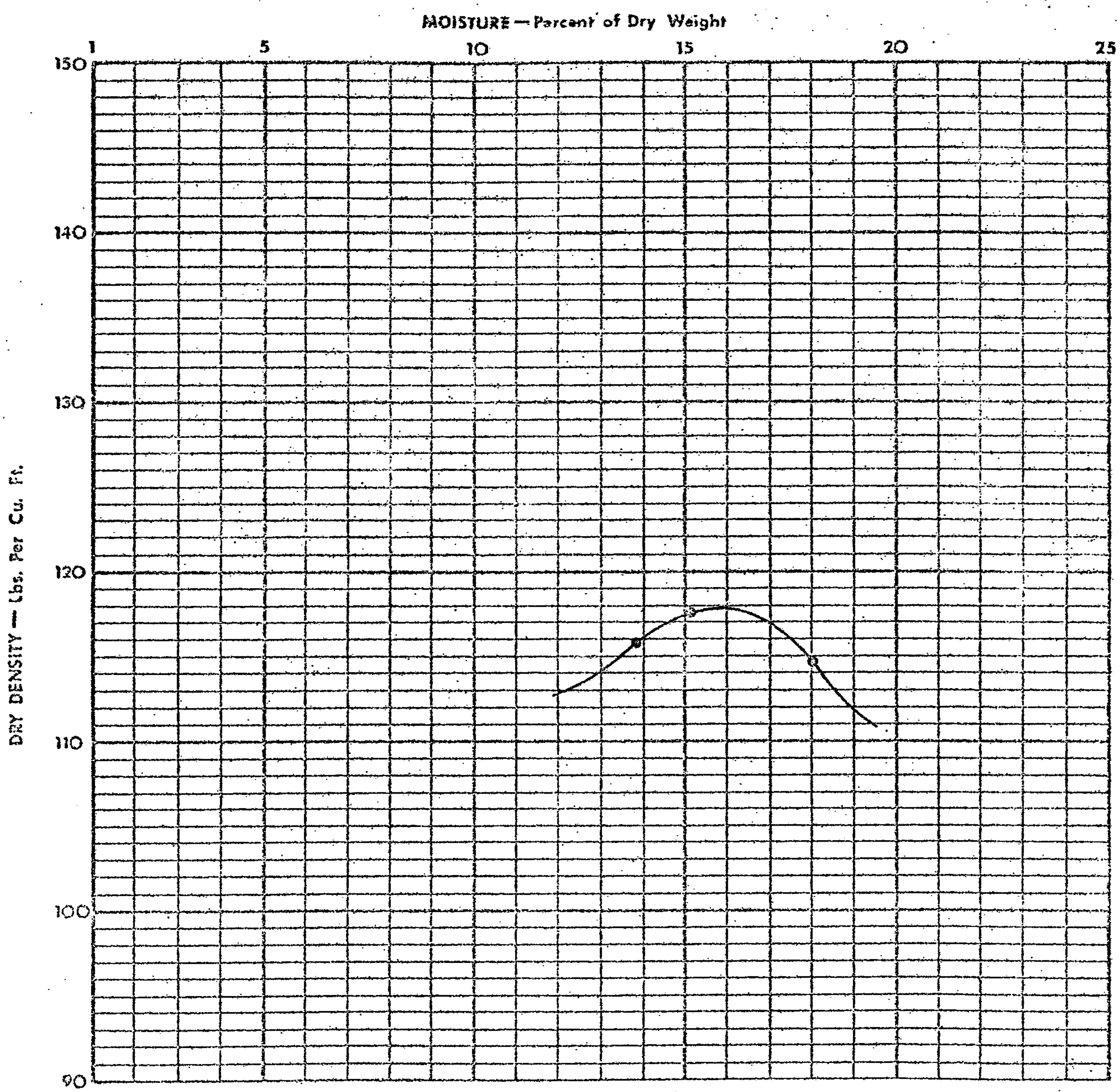
From Proposed uranium leaching operation site

Compaction Test Procedure ASTM D-1557

Form 4-1

F. M. Fox & Assoc. Inc.

COMPACTION TEST RESULTS



Maximum Dry Density (P.C.F) 118.0

Optimum Moisture Content (%) 16.0

Sample Description Group IV, CLAY, sandy

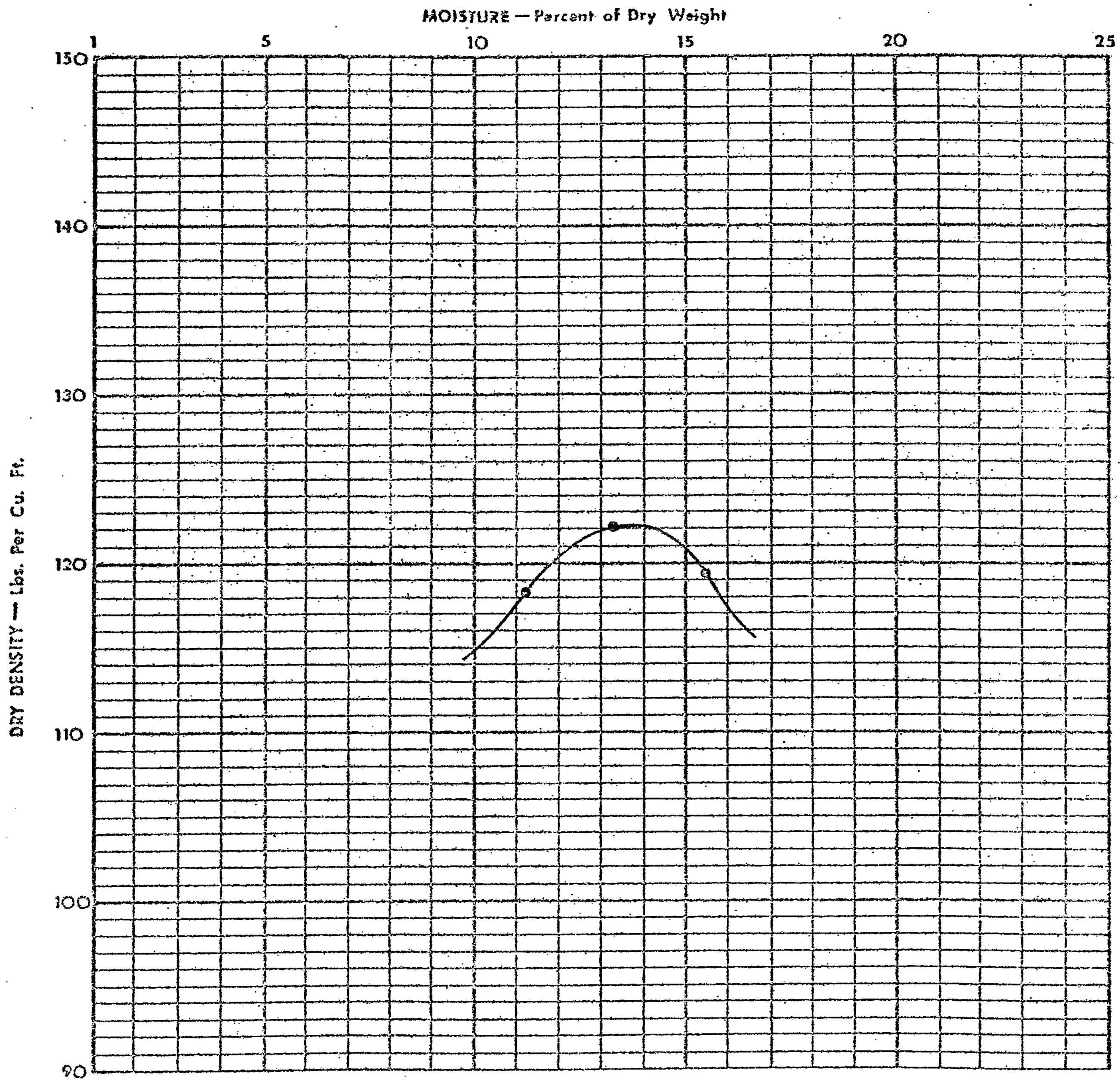
From Proposed uranium leaching operation site

Compaction Test Procedure ASTM D-1557

Form 4-1

F. M. Fox & Assoc. Inc.

COMPACTION TEST RESULTS



Maximum Dry Density (P.C.F) 122.0

Optimum Moisture Content (%) 14.0

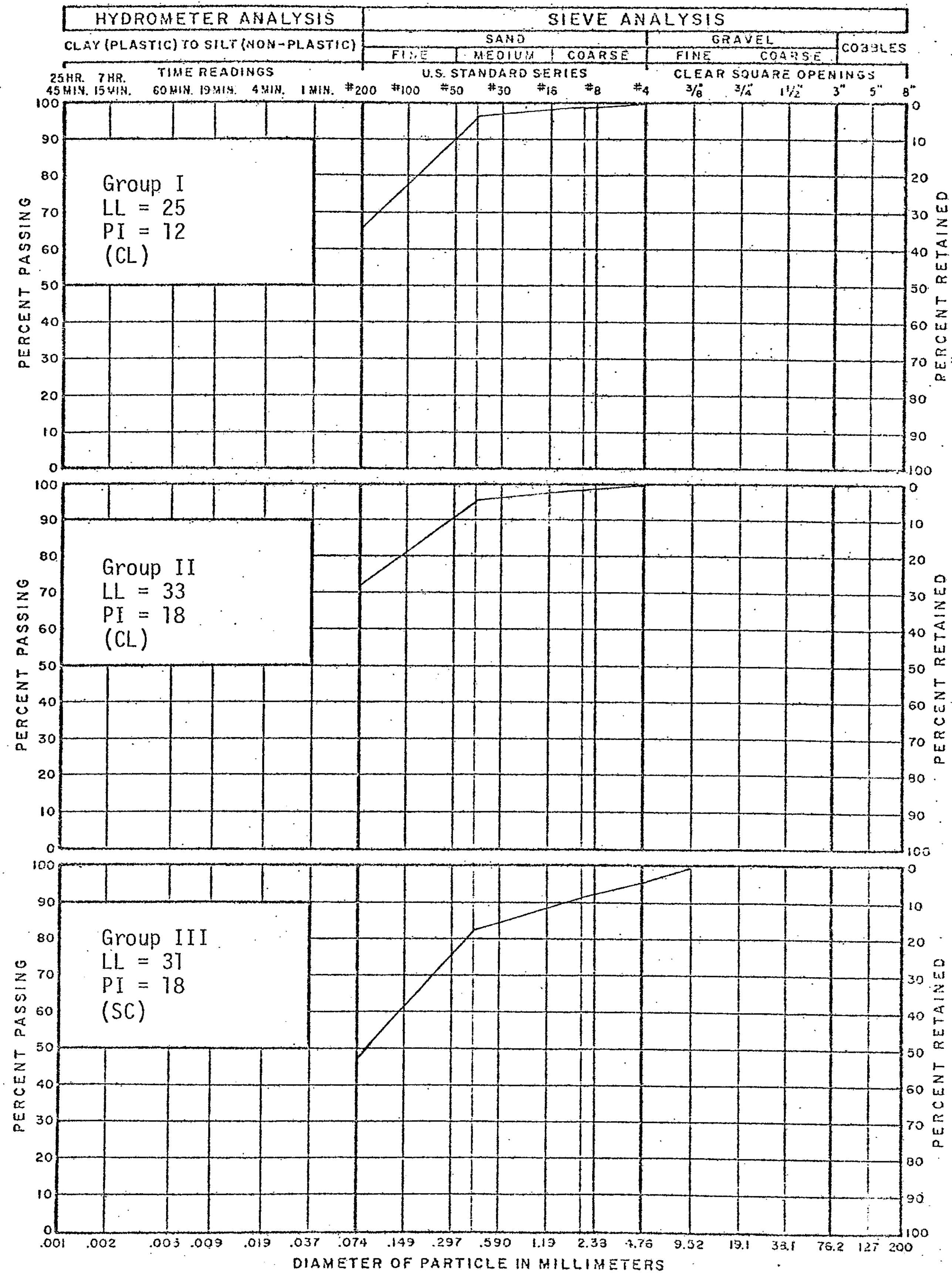
Sample Description Group V, CLAY, sandy

From Proposed uranium leaching operation site

Compaction Test Procedure ASTM D-1557

Form 4-1

GRADATION ANALYSIS



F. M. Fox & Assoc. Inc.

GRADATION ANALYSIS

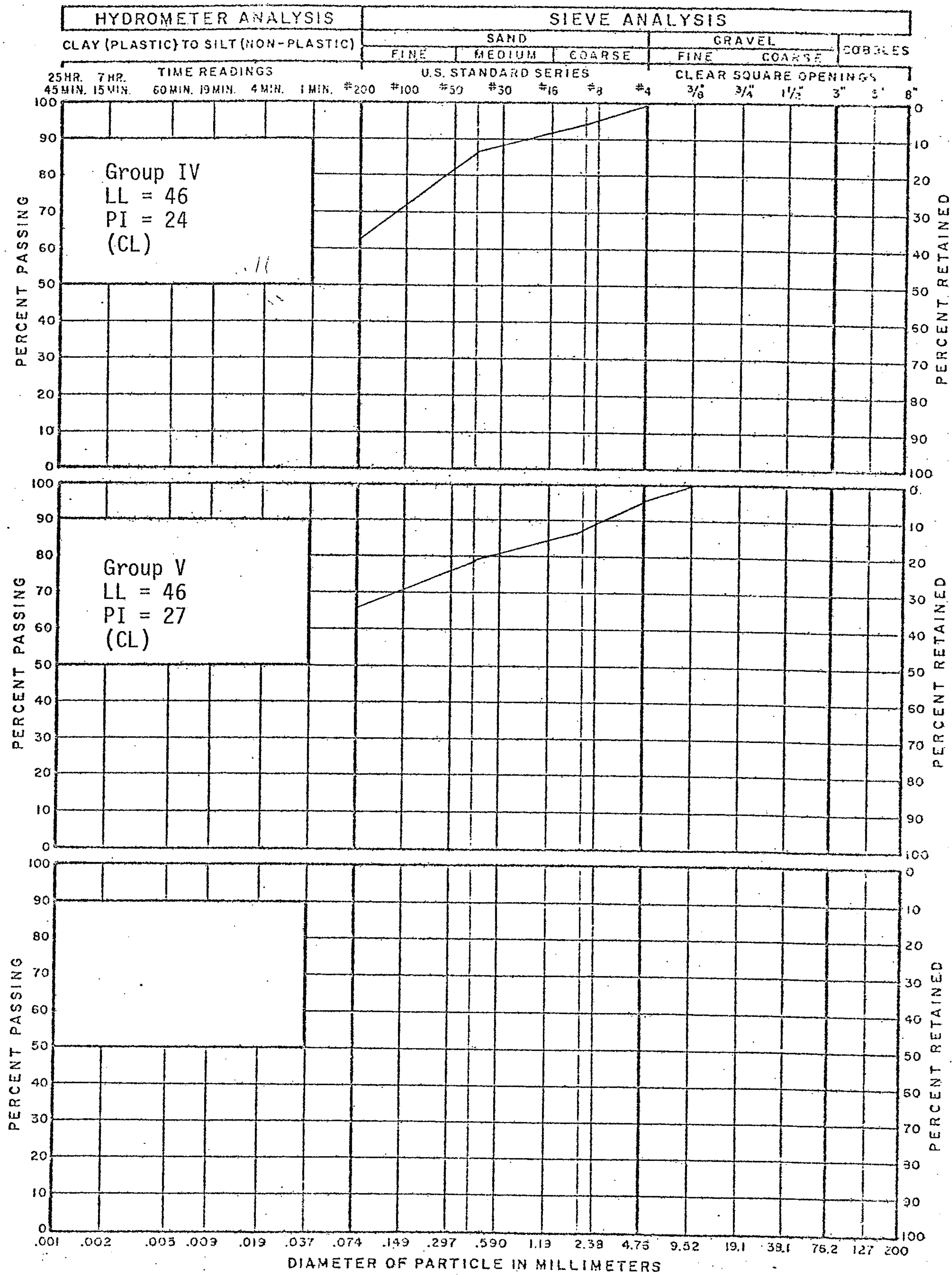
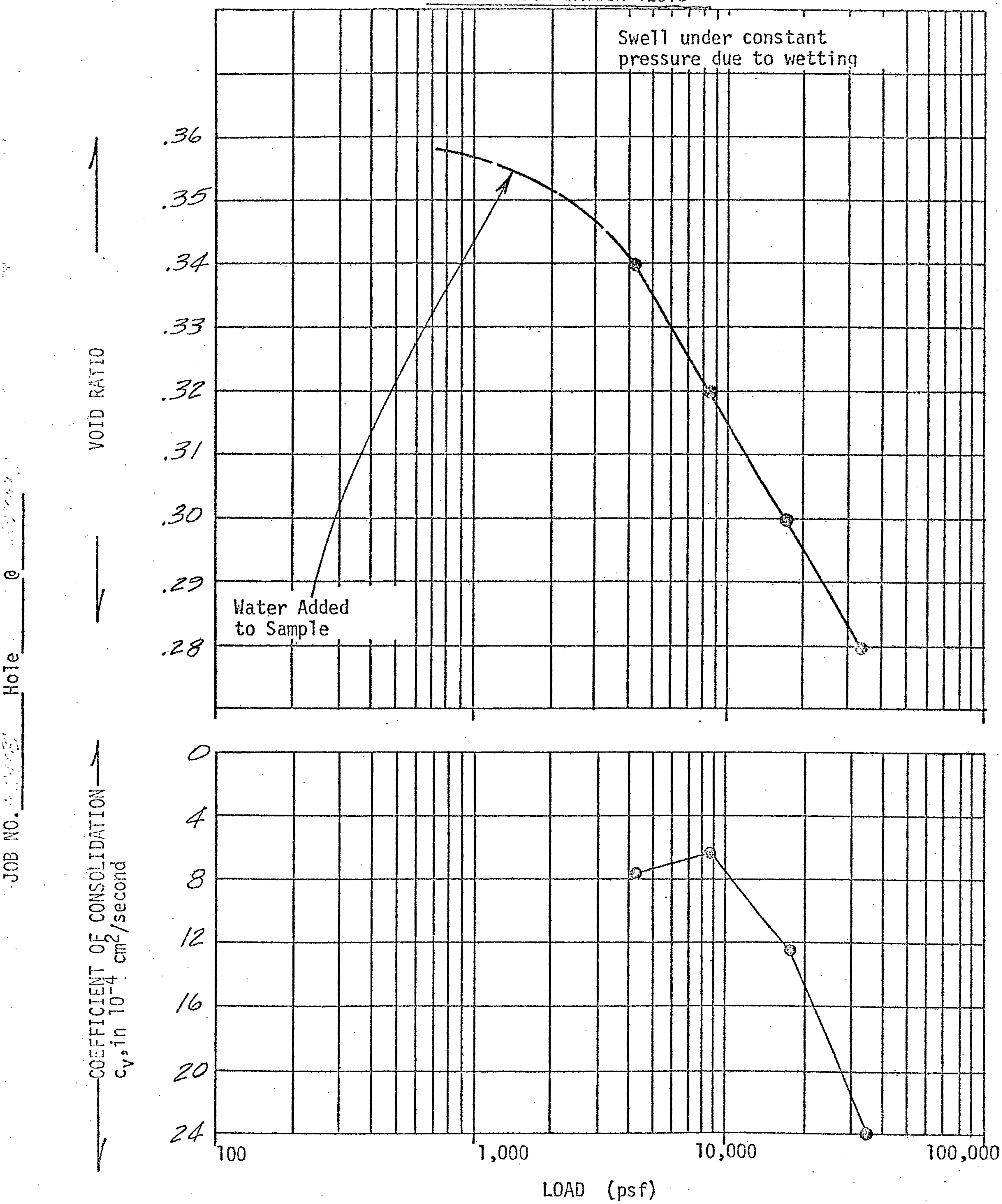


FIG A-19

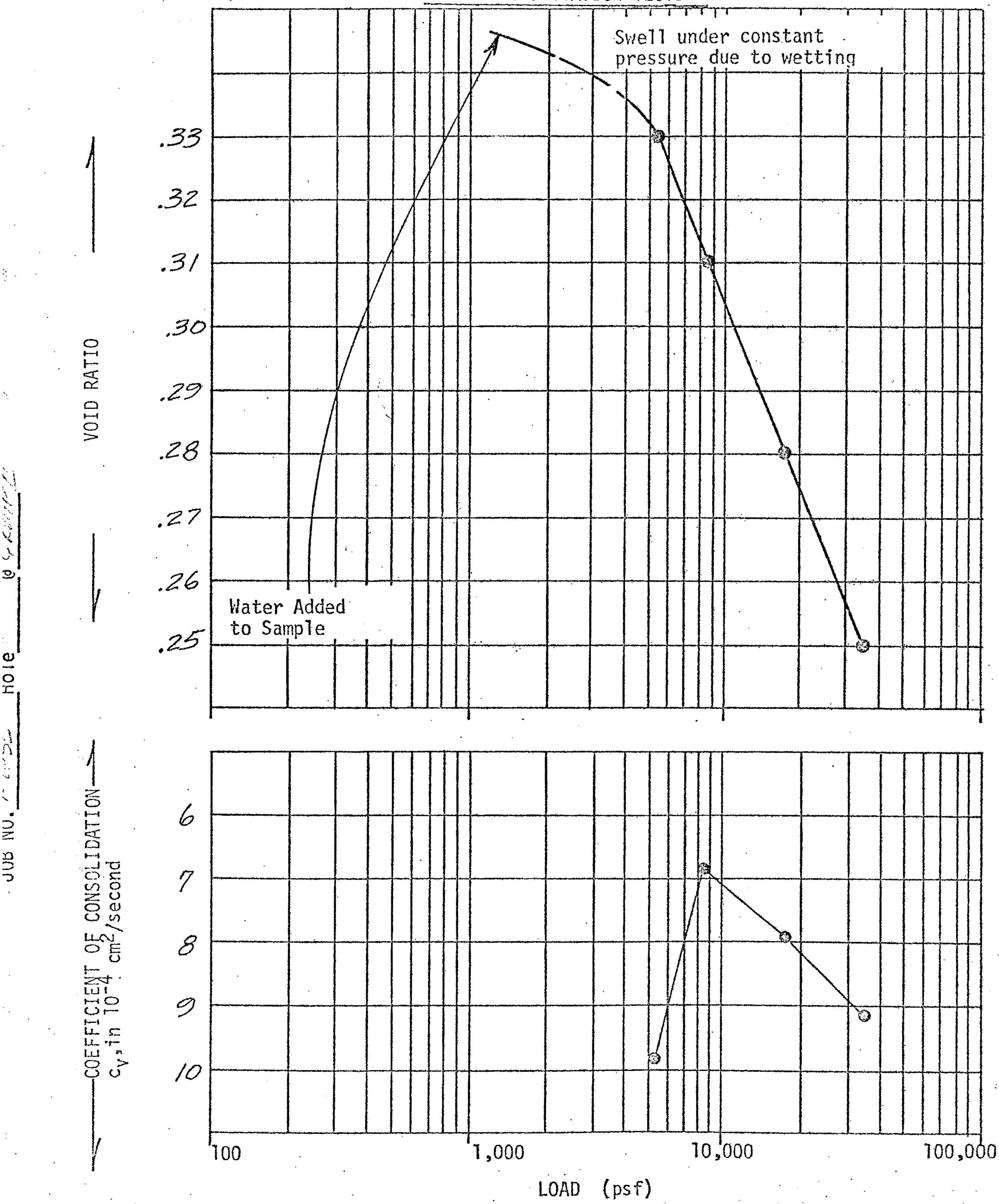
SWELL CONSOLIDATION TESTS



Sample of GROUP I SOILS
Final Moisture Content 13.1%

FIGURE A-20

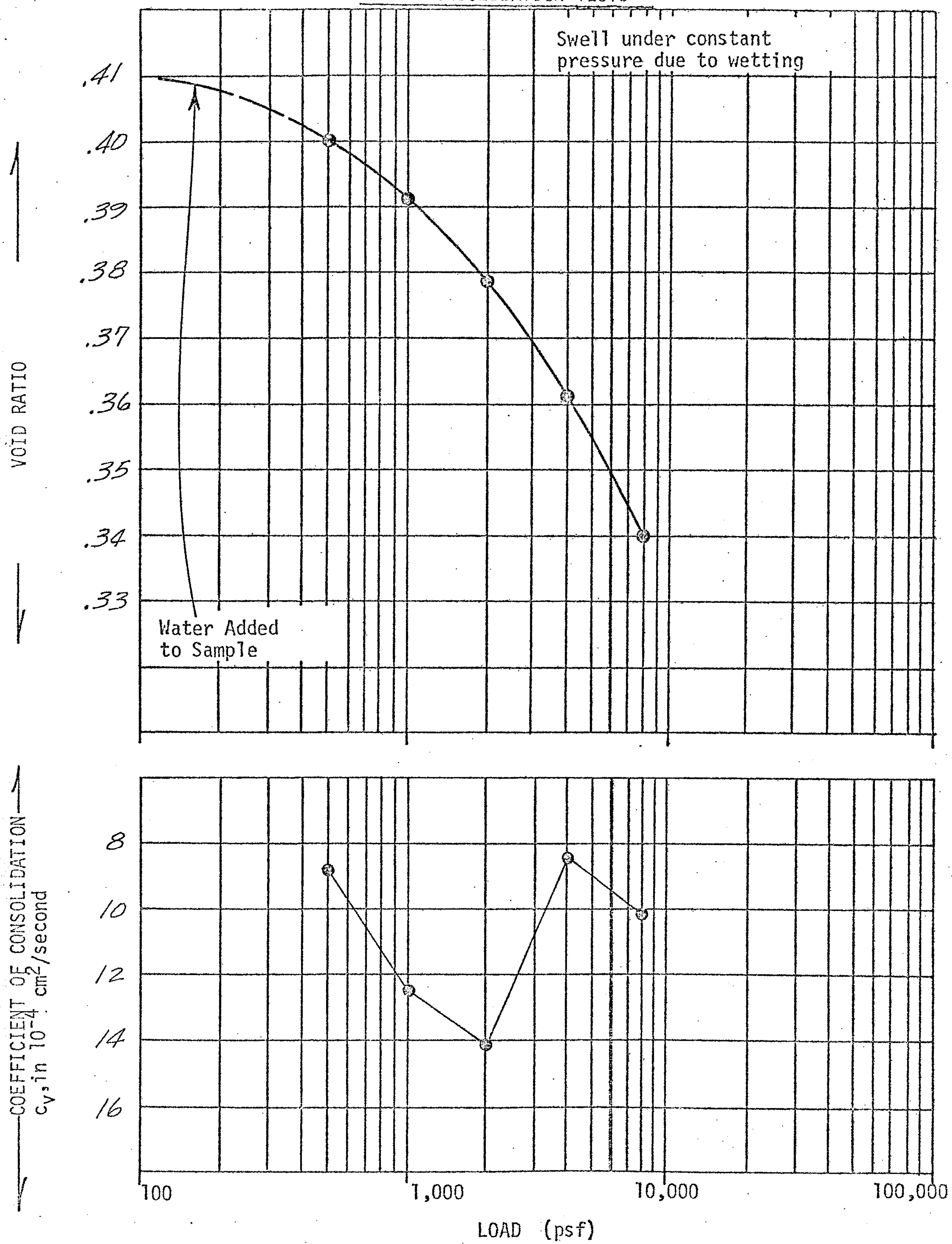
SWELL CONSOLIDATION TESTS



Sample of GROUP II SOILS

FIGURE A-21

SWELL CONSOLIDATION TESTS

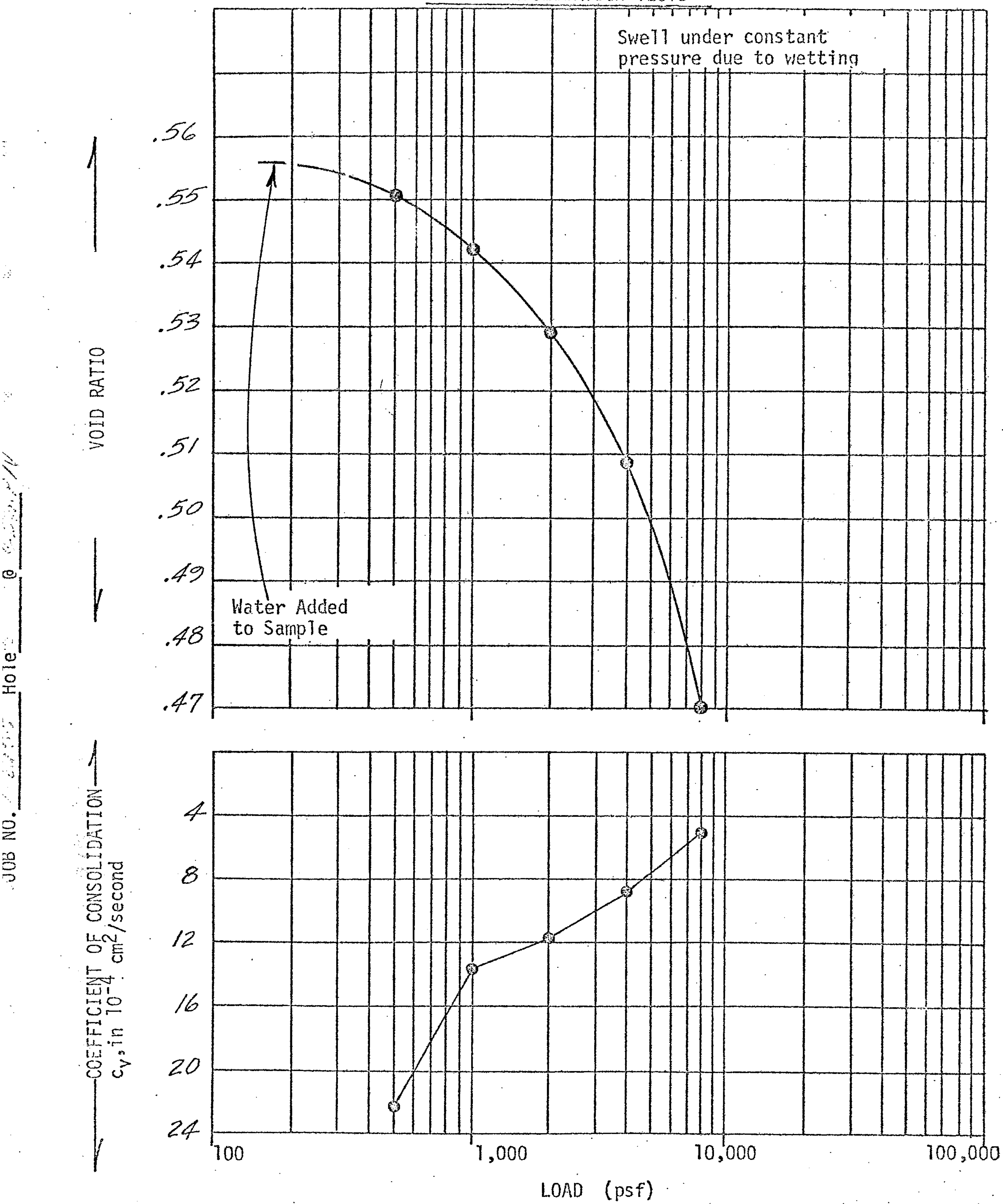


JOB NO. ______ Hole ______ @ ______

Sample of GROUP III SOILS

FIGURE A-22

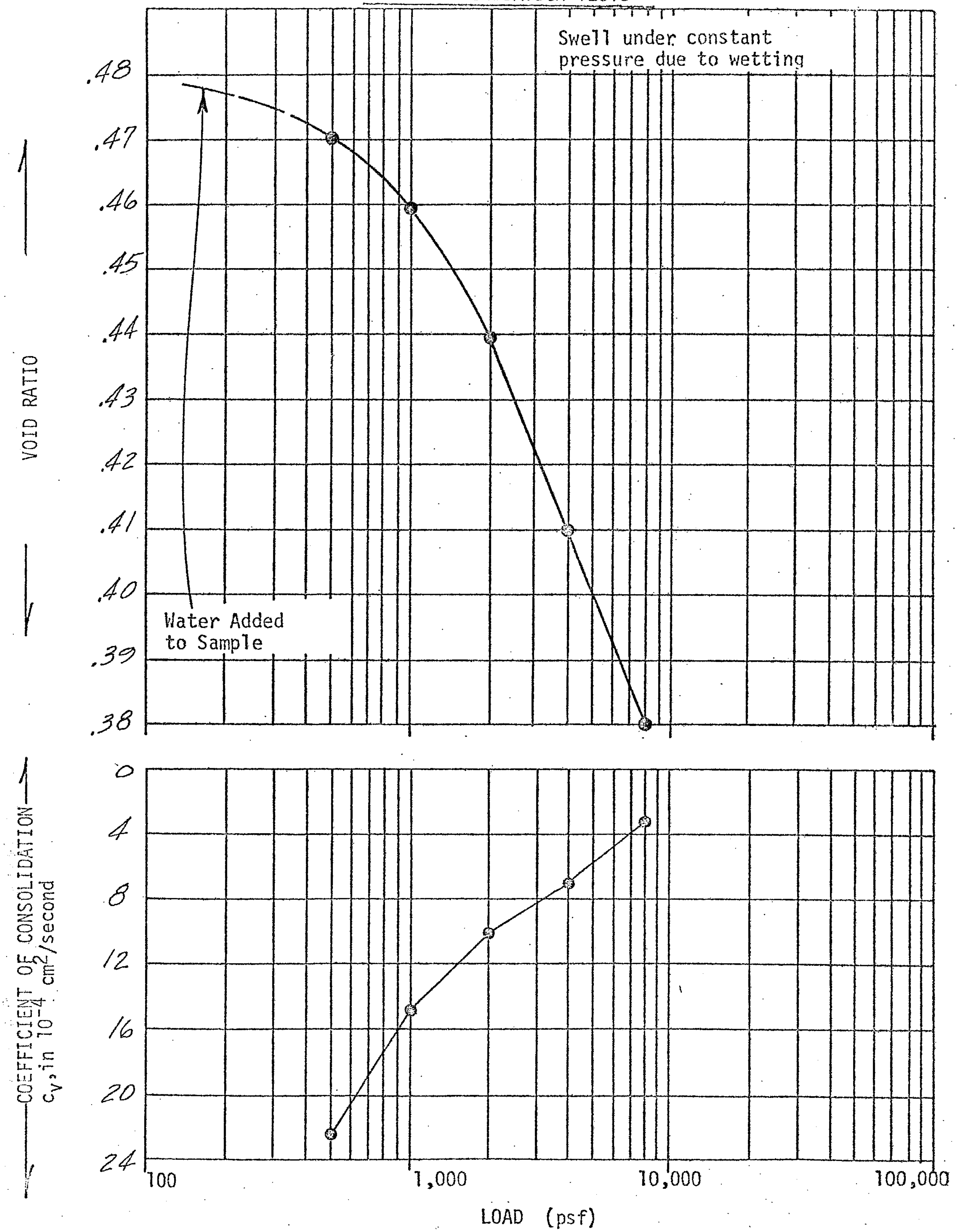
SWELL CONSOLIDATION TESTS



Sample of GROUP IV SOILS

FIGURE A-23

SWELL CONSOLIDATION TESTS



JOB NO. ____ Hole ____ @ ____

Sample of GROUP V SOILS

FIGURE A-24

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Job No. 1-1331-2955

Description Group I at 95% ASTM D-1557
2% over optimum moisture content

c = 40 psi

Ø = 0°

w = 14%

γ_d = 120

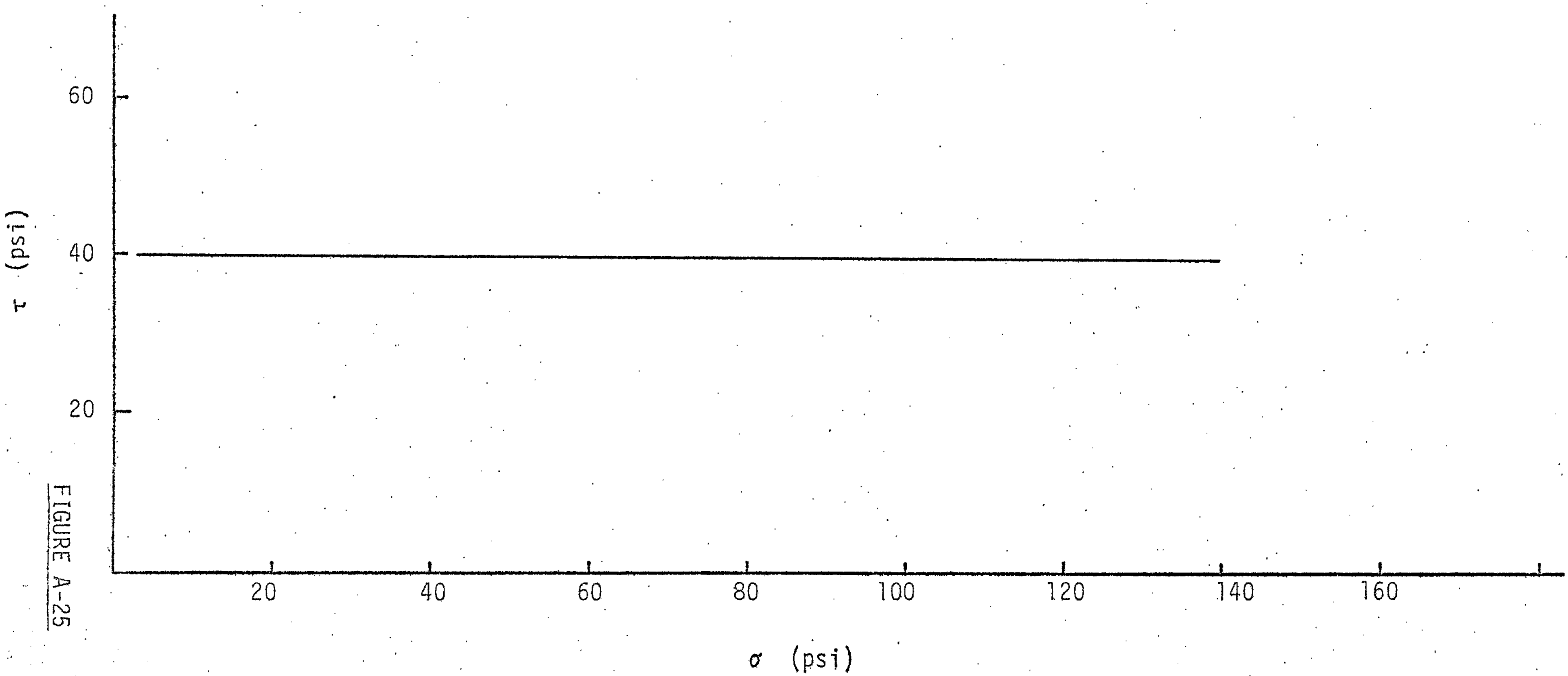


FIGURE A-25

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Job No. 1-1331-2955

Description Group II at 95% ASTM D-1557
2% over optimum moisture content

c = 39 psi

Ø = 12°

w = 13.9

γd = 120

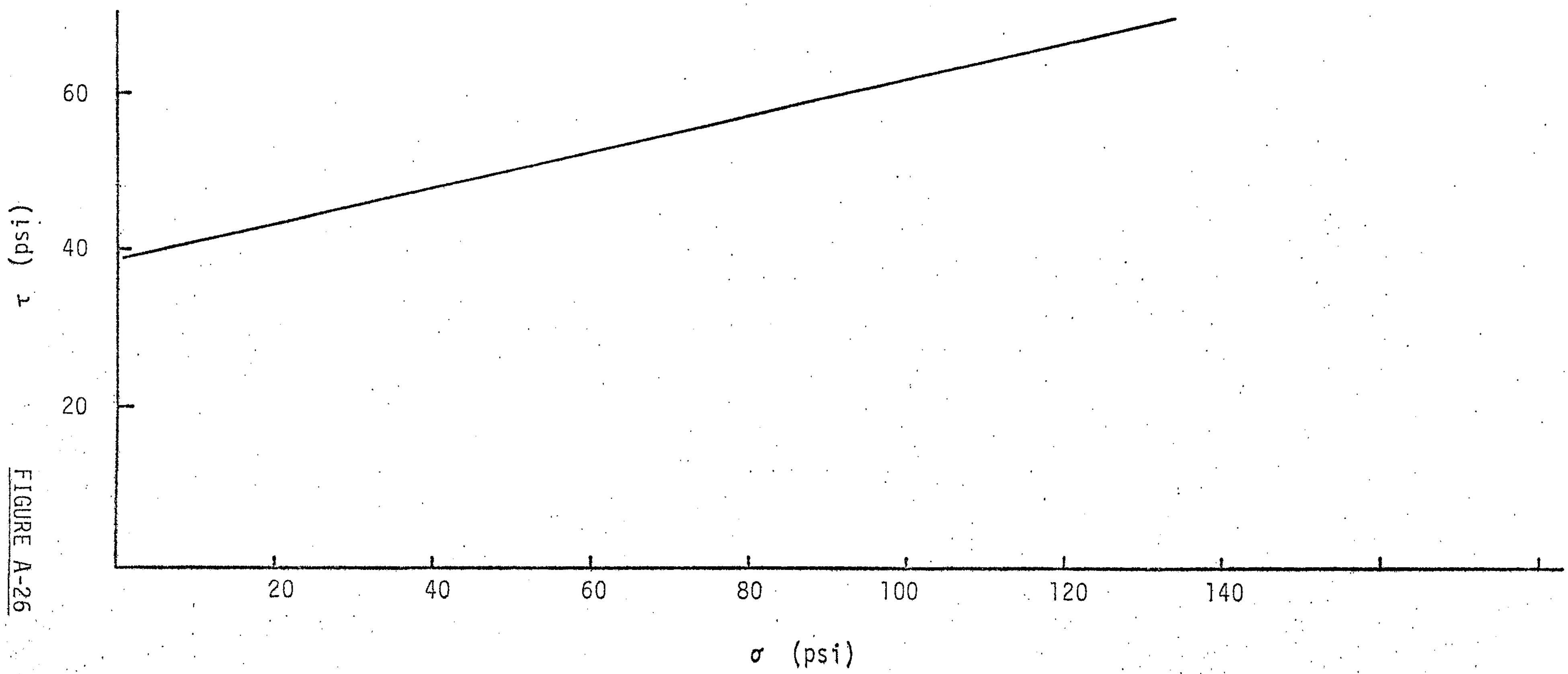


FIGURE A-26

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Job No. 1-1331-2955

Description Group III at 95% ASTM D-1557
2% over optimum moisture content

c = 35 psi

Ø = 13°

w = 13.0

γd = 120

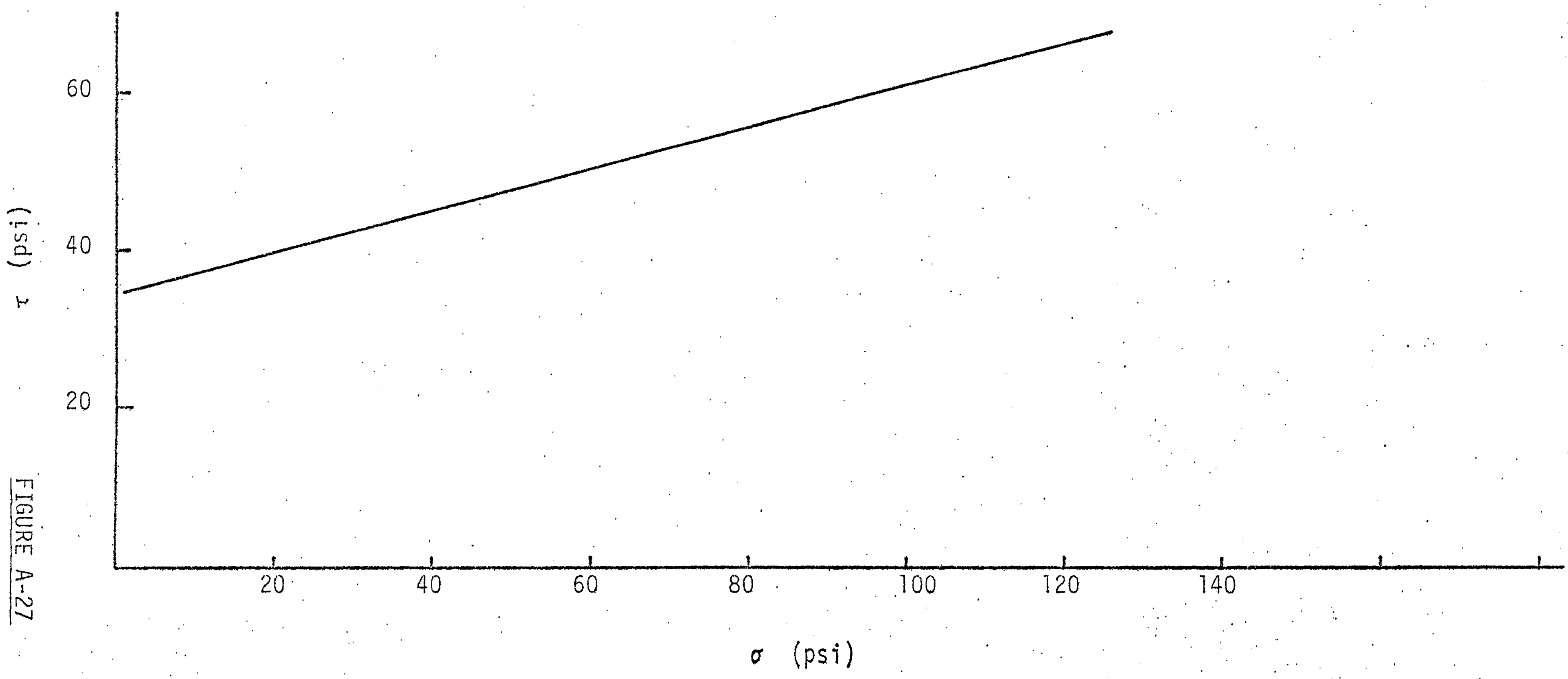


FIGURE A-27

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Job No. 1-1331-2955

Description Group IV, at 95% ASTM D-1557
2% over optimum moisture content

c = 38 psi

Ø = 4°

w = 17.5

γd = 115

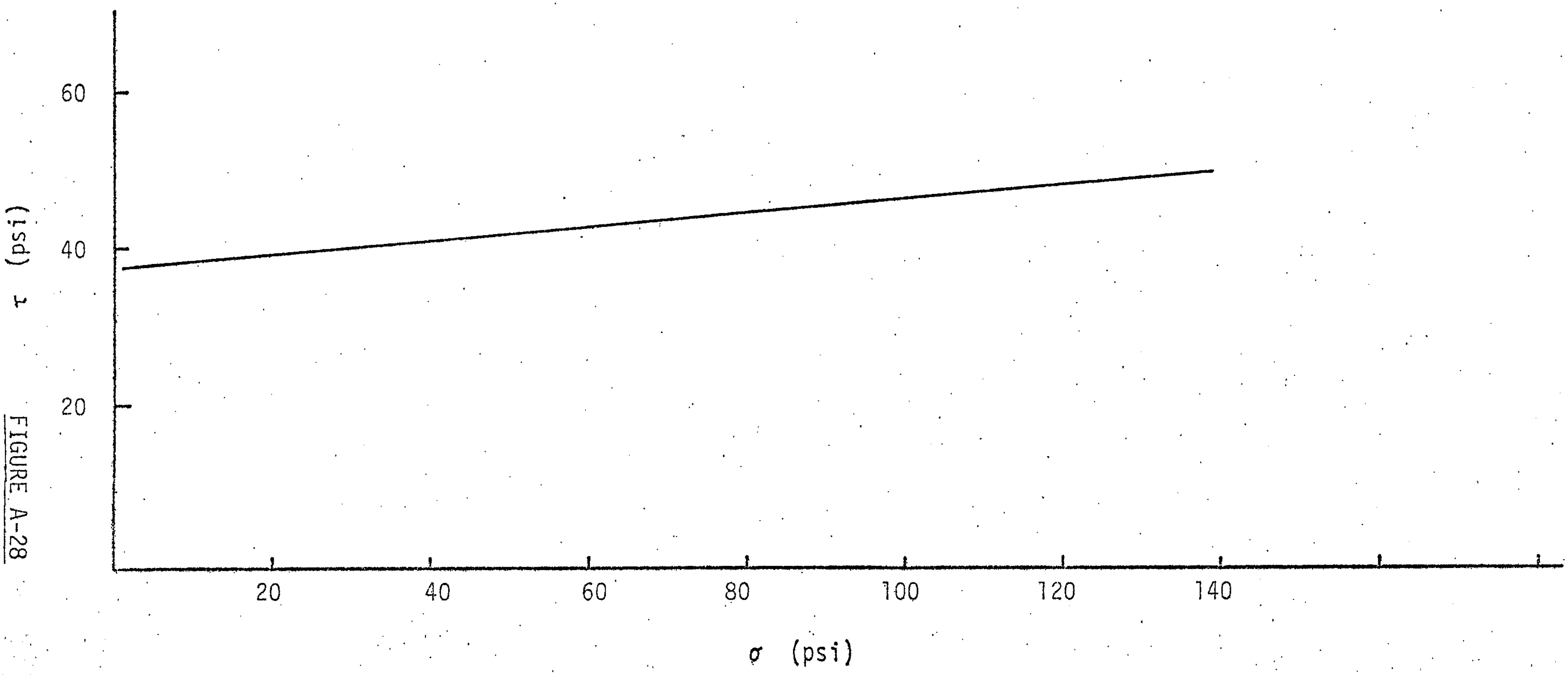


FIGURE A-28

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Job No. 1-1331-2955

Description Group V at 95% ASTM D-1557
2% over optimum moisture content

c = 44 psi

ϕ = 10°

w = 15.7

γd = 116

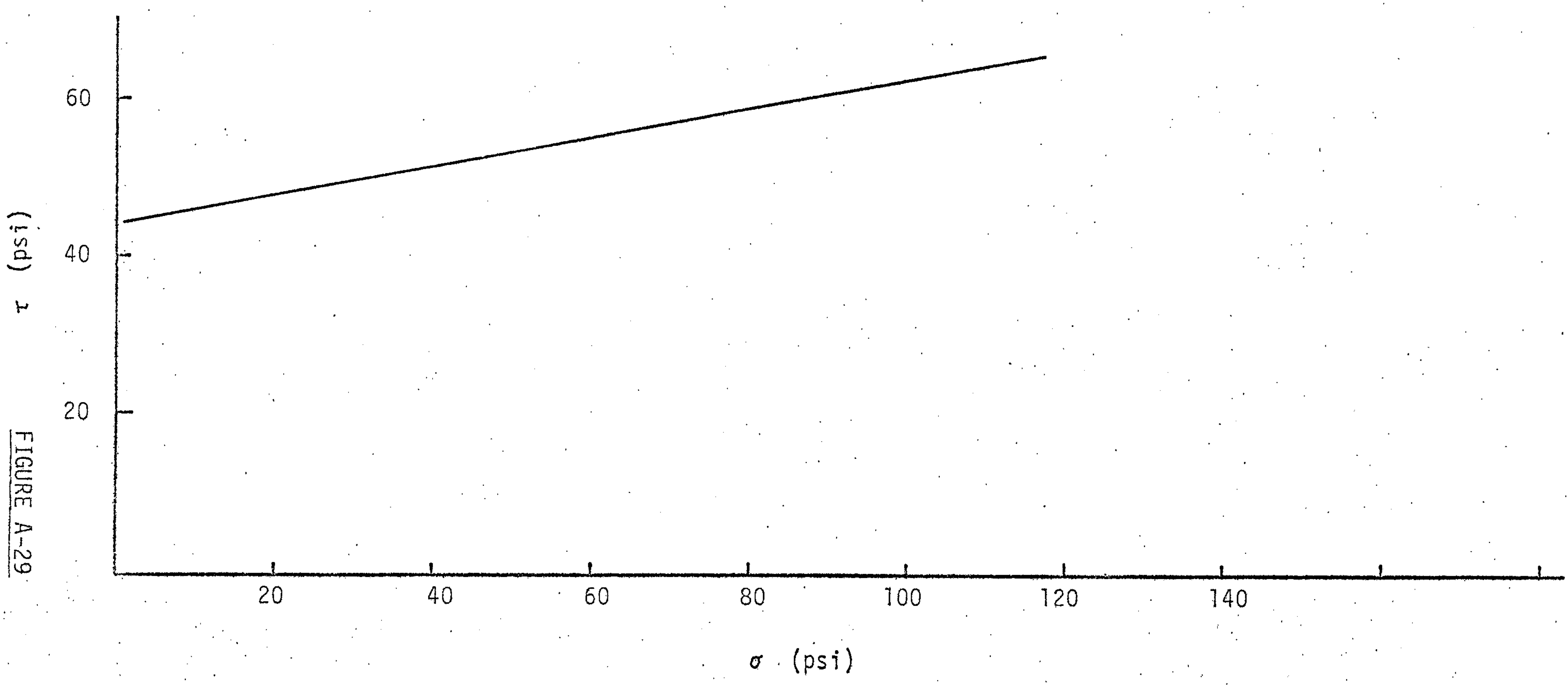


FIGURE A-29

APPENDIX B

LABORATORY AND FIELD TESTING PROCEDURES

Laboratory and Field Testing Procedures

The laboratory and field testing program included the following testing procedures conducted in accordance with the referenced designation of the American Society for Testing and Materials:

1. Dry Preparation of Disturbed Soil Samples (ASTM D 421-58)
2. Mechanical Analysis (ASTM D 422)
3. Test for Liquid Limit of Soils (ASTM D 423-63) (1972)
4. Test for Plastic Limit and Plasticity Index of Soils (ASTM D 424-59)(1971)
5. Maximum Density and Optimum Moisture (ASTM D 1557)
6. Penetration Tests & Split Barrel Sampling of Soils (ASTM D 1586-67)
7. Unconfined Compressive Strength of Cohesive Soils (ASTM D 2166-66)
8. Bulk Density and Moisture Content (ASTM D 2216-63T)
9. Test for One Dimensional Consolidation Properties of Soils (ASTM D 2435-68)
10. Test for Unconsolidated, Undrained Strength of Cohesive Soils in Triaxial Compression (ASTM D 2850-71)

FIGURE B-1