TTI-2-5-74-10-1

TEXAS
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COOPERATIVE
RESEARCH

CORRELATION OF THE TEXAS HIGHWAY DEPARTMENT CONE PENETROMETER TEST WITH UNCONSOLIDATED-UNDRAINED SHEAR STRENGTH OF COHESIVE SOILS

in cooperation with the
Department of Transportation
Federal Highway Administration

RESEARCH REPORT 10-1
STUDY 2-5-74-10
THD CONE PENETROMETER TEST

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle
CORRELATION OF THE TEXAS HIGHWAY DEPARTMENT CONE PENETROMETER TEST WITH UNCONSOLIDATED-UNDRAINED SHEAR STRENGTH OF COHESIVE SOILS

5. Report Date
August, 1974

6. Performing Organization Code

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8. Performing Organization Report No.
Research Report 10-1

9. Performing Organization Name and Address
Texas Transportation Institute
Texas A&M University
College Station, Texas 77843

10. Work Unit No.

11. Contract or Grant No.
Research Study 2-5-74-10

12. Sponsoring Agency Name and Address
Texas Highway Department
11th and Brazos
Austin, Texas 78701

13. Type of Report and Period Covered
Interim - September 1973 - August 1974

14. Sponsoring Agency Code

15. Supplementary Notes
Research performed in cooperation with DOT, FHWA
Research Study Title: "Correlation of the THD Cone Penetrometer Test N-Value with Shear Strength of the Soil Tested."

16. Abstract
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A reasonably good correlation was established between the unconsolidated-undrained shear strength C_u-values and penetration resistance N-values, particularly for homogeneous CH and silty Cl soils. Constants of proportionality between C_u and N, based on a linear relationship, were obtained for these soil groups. A correlation was also established for CH soils with secondary structure and for sandy CL soils, but there was more scatter in the data for these groups. Equations were developed which relate the unconsolidated-undrained shear strength, $C_{u_{ST}}$, to the Standard Penetration Test resistance value, N_{SPT}, for homogeneous CH, silty CL, and sandy CL soils.

17. Key Words
THD Cone Penetrometer, Unconsolidated-Undrained Shear Strength, Texas Triaxial Test, ASTM Triaxial Test, Cohesive Soils.

18. Distribution Statement

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	135	

Form DOT F 1700.7 (8-69)

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Correlation of the THD Cone Penetrometer Test
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Sponsored by
The Texas Highway Department
in Cooperation with the
U.S. Department of Transporation
Federal Highway Administration

August 1974

TEXAS TRANSPORTATION INSTITUTE
Texas A&M University
College Station, Texas

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ABSTRACT

Correlations were established between the Texas Highway Department Cone Penetrometer Test and the unconsolidated-undrained shear strength for cohesive soils. Both field and laboratory investigations were conducted to obtain the data necessary to establish the correlations. The field investigations included seven borings taken at four different sites where the Cone Penetrometer Test was conducted and undisturbed soil samples were obtained. The Texas Triaxial Test and the ASTM Triaxial Test were used in the laboratory investigation to obtain soil shear strength. Soils were classified and grouped by the Unified Soil Classification System.

A reasonably good correlation was established between the unconsolidated-undrained shear strength C_u-values and penetration resistance N-values, particularly for homogeneous CH and silty Cl soils. Constants of proportionality between C_u and N, based on a linear relationship, were obtained for these soil groups. A correlation was also established for CH soils with secondary structure and for sandy CL soils, but there was more scatter in the data for these groups. Equations were developed which relate the unconsolidated-undrained shear strength, $C_{u_{ST}}$, to the Standard Penetration Test resistance value, N_{SPT}, for homogeneous CH, silty CL, and sandy CL soils.

KEY WORDS: THD Cone Penetrometer, Unconsolidated-Undrained Shear Strength, Texas Triaxial Test, ASTM Triaxial Test, Cohesive Soils.

SUMMARY

The information presented in this report was developed during the first year of a three-year study on the determination of in-situ soil shear strength by means of dynamic sub-surface sounding tests. The objective of the research is to develop an improved correlation between the THD Cone Penetrometer Test N-value and the shear strength of different soil types including sand, silt, clay, and various combinations thereof.

A brief historical background of penetrometer tests is presented together with a description of the THD Cone Penetrometer Test method and equipment. The geologic histories of test locations are summarized and underlying soil formations are described.

Field operational methods and procedures used to obtain THD Cone Penetrometer Test data and undisturbed soil samples are described. The laboratory test methods used to classify the soil and determine its unconsolidated-undrained shear strength are discussed. The Texas Triaxial test was the primary method used for determining shear strength. ASTM Triaxial and THD Transmatic Triaxial tests were conducted to determine the shear strength of selected samples.

The various factors affecting the magnitude of the THD Cone Penetrometer Test N-value and the soil shear strength are analyzed. The soil shear strength is correlated with the N-value for four categories of fat and lean clay. Equations are developed which relate the unconsolidated-undrained shear strength, Cu_{ST}, to the Standard Penetration Test resistance value, N_{STP}, for homogeneous CH, silty CL, and sandy CL soil.

IMPLEMENTATION STATEMENT

Correlations between the THD cone penetrometer test N-value and the unconsolidated-undrained soil shear strength are used to predict the bearing capacity of drilled shaft and pile foundations which support highway bridge superstructures. The data presented in this report were obtained from testing soils commonly found along the upper Texas gulf coast region. Correlations were obtained for four soil types which are based primarily upon the Unified System of soil classification, the secondary division being made according to soil structure or grain size content. The information required to classify the soil is obtained from standard laboratory tests. The shear strength is determined by Texas Triaxial Test Method Tex-118-E. Use of the correlations given herein will aid the design engineer in selecting the most economical pile or shaft diameter and depth of embedment.

It is recommended that implementation of the results be limited to similar soils from the same regions and having similar geologic origins and histories. Also, it should be borne in mind that data of the nature being obtained in this study are subject to statistical variation, and the reliability of correlations achieved are directly dependent on the amount of data obtained. Consequently, any study findings which are implemented prior to completion of the study should be considered tentative and subject to change at any time.

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INTRODUCTION

Historical Background of Penetration Tests.--The process of obtaining the variation in penetration resistance of a soil along vertical lines is known as sub-surface sounding. The tool used to make the sounding is commonly known as a penetrometer. The use of the penetrometer evolved from the need of acquiring data on sub-surface soils that were not obtainable by any other means.

For several generations engineers have made crude attempts to determine the strength of subsurface soils by driving or pushing rods or pipes into the ground and recording the resistance to penetration. Today, the resistance to penetration is measured both statically and dynamically.

The static penetration test is widely accepted and used in Scandinavian and European countries. The method was originally developed in France in 1846 (20)*. A Vicat-type needle one millimeter (0.039 in.) in diameter, weighing one kg (2.20 lb) was used to estimate the cohesion of different types of clay soils at different consistencies.

The dynamic penetration test has been in use as a sounding test for the past half century or more in most countries. Around

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*Numbers in parentheses refer to the references listed in Appendix I.

1920 dynamic penetration tests were initiated in the United States. One of the most widely used procedures for measuring resistance to penetration is the Standard Penetration Test (SPT). According to Desai (6), the penetrometer used with the SPT is the split-spoon that was developed by Raymond Concrete Pile Company. Several other types of penetrometers have been more or less standardized. Most of these use a steel cone drive point.

The Texas Highway Department is currently using a Cone Penetration Test. Resistance to penetration is expressed as the number of blows per foot of penetration caused by a free falling hammer. The purpose of the test is to obtain an estimate of the in situ properties of the soil. The cone penetrometer shown in Fig. 1 is used to perform the test in accordance with Texas Highway Department specifications (5). The drilling rod used is a three thread "N" rod with a wall thickness of 0.281 in. (7.137mm). The hammer weighs 170 pounds (77 kg) and falls freely a distance of 2 ft (0.61 m).

The Texas Highway Department Cone Penetrometer Test is normally performed in each identifiable soil layer or every 5 ft (1.27m), whichever is smaller. The cone is lowered to the bottom of a boring and the tip seated into the undisturbed soil. If the cone penetrates into the undisturbed soil under its own weight without driving, the penetration is measured and a zero resistance is recorded. Otherwise, the cone is driven 12 blows in order to properly seat it in the soil. Then the number of blows of the hammer, which causes the cone to penetrate an additional 12 in. (304.8mm) into the soil,

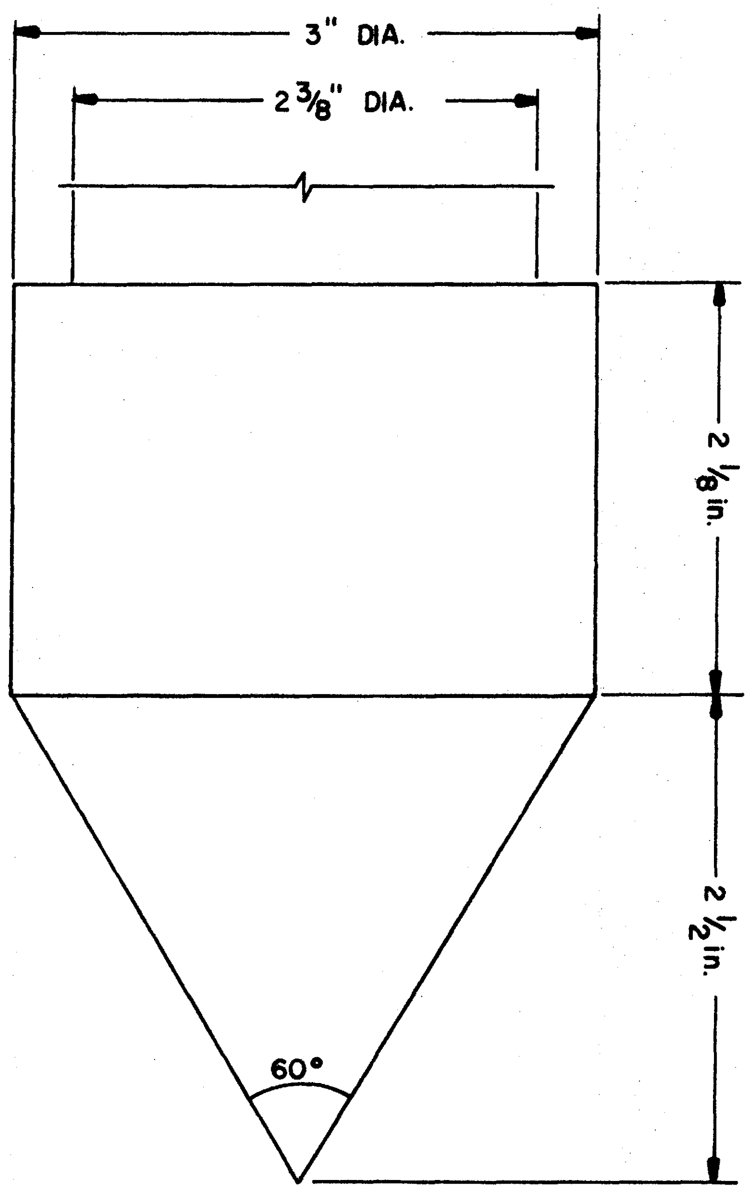


FIG-1 DETAILS OF THD CONE PENETROMETER
AFTER VIJAYVERGIYA, HUDSON AND REESE (32)
(1.0 in = 25.4 mm)

is recorded and is called the N-value.

Present Status of the Problem.--Dynamic penetrometers were originally designed to obtain qualitative data on the resistance to penetration of a soil and in particular to determine the compactness of cohesionless soils which are usually difficult to sample. Today, their use has been extended to aid in the determination of required depth of embedment of foundations into a soil bearing strata. Penetrometer data are used to determine the shear strength parameters of the soil. These parameters are then used in bearing capacity equations to determine the depth at which the soil will carry the required foundation load.

The "quick" or unconsolidated-undrained shear strength of a cohesive soil is the most commonly used shear strength parameter for evaluating the bearing capacity in cohesive soils (23). Various researchers have developed relationships between the dynamic penetration resistance, N, from the Standard Penetration Test and the quick shear strength for cohesive soils (9, 24, 27). The quick shear strength is measured in the laboratory by the unconfined compression test or in the field by the in situ vane.

The Foundation Manual (5) presently used by the Texas Highway Department includes a correlation between the N-value obtained from the Texas Highway Department Cone Penetrometer Test and the soil shear strength. However, this correlation was established for many soil types and is known to be conservative for some soil types.

During recent years research has been conducted at Texas A&M University, Texas Transportation Institute (TTI), on driven piling and at the University of Texas, Center for Highway Research (CHR), on drilled shafts. As part of these research studies, soil shear strength was obtained in the laboratory from unconfined compression tests by TTI and from triaxial quick tests by CHR on undisturbed samples obtained from load test sites (1, 8). Also, the N-values from the Cone Penetrometer Test were obtained. In addition, similar data were recently collected randomly from Texas Highway Department district laboratories. The shear strength collected from the district laboratories was obtained by either the Texas Triaxial Test (TAT) or the Texas Transmatic Triaxial Test. Comparison between these data and the Texas Highway Department correlation as shown in Fig. 2 substantiates that the correlation is conservative.

According to the Texas Highway Department Foundation Manual (5), the Cone Penetrometer Test is a standard test used to determine the consistency and load carrying capacity of foundation materials encountered in bridge foundation work. Furthermore, the Manual (5) states that:

> The load carrying properties of a material are: 1) its shear strength, 2) its bearing strength. These properties are determined by one or more of the following tests:
>
> a- Triaxial Test
> b- Unconfined Compression Test
> c- THD Cone Penetrometer Test
> d- In-place Vane Shear Test
> e- Miniature Vane Shear Test

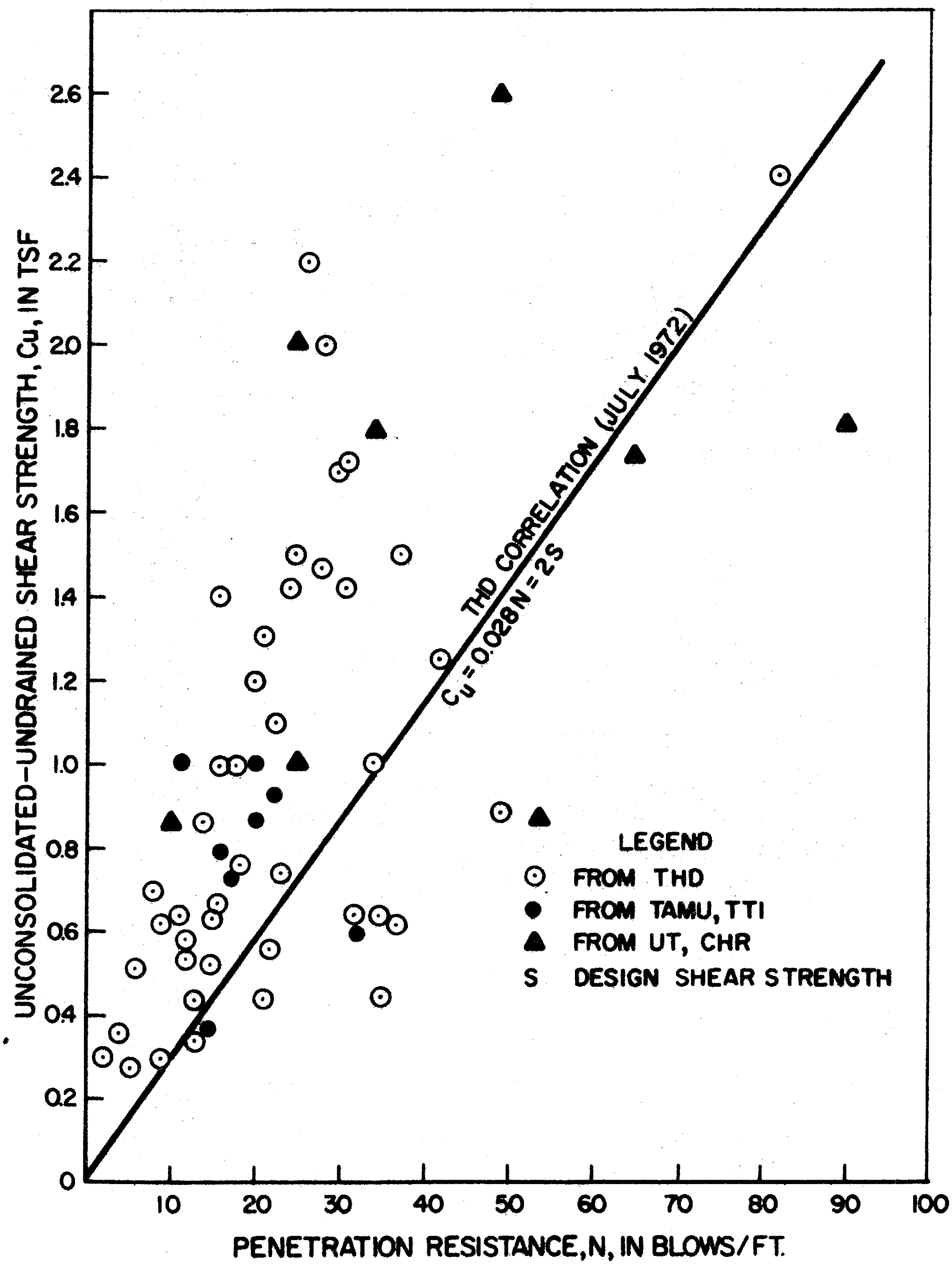


FIG. 2 — RELATIONSHIP BETWEEN UNCONSOLIDATED-UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION OF THD CONE PENETROMETER (1.0 ft. = .305 m., 1.0 tsf = 9.58×10^4 N/m^2)

The laboratory tests (items a, b, and e above) for determining soil shear strength are often omitted in routine subsurface investigations because of the additional expense involved. Consequently, the THD Cone Penetrometer Test is the primary means of determining soil shear strength at bridge sites. Therefore, a better correlation between the N-value and soil shear strength could result in significant financial savings in the design and construction of bridges.

Objective.--The objective of this study is to develop an improved correlation between the N-value obtained from the Texas Highway Department Cone Penetrometer Test and the unconsolidated-undrained shear strength of three different groups of cohesive soils. According to the Unified Soil Classification System, these soils are CH, CL, and SC which are defined as follows:

CH - Inorganic clays of high plasticity, fat clays,

CL - Inorganic clays of low plasticity, sandy clays, silty clays, lean clays,

SC - Clayey sands, sand-clay mixtures.

TEST SITES

A preliminary site location survey was conducted in order to locate a variety of cohesive soils to include CH, CL, and SC. An effort was made to locate sites where a test load on driven piling or drilled shafts had been conducted. Four locations yielding a reasonable variety of cohesive soils were located. These sites are designated as sites A, B, C, and D, respectively. At three of the sites (A, B, and C), a test load on drilled shafts had been conducted. Figure 3 shows the general location and the geological formations of the test sites.

Test Site Locations.---Test site A is located at a new bridge that crosses the Little Brazos River on State Highway 21, approximately 10 miles (16.1 km) southwest of Bryan, Texas. Test sites B, C, and D are located within the city limits of Houston, Texas. Site B is located at Interstate Highway 610 - HB&T Railroad overpass. Site C is located at the proposed overpass of State Highway 288 and Brays Bayou. Site D is located at Interstate Highway 45 and Nettleton Street.

Test Site Geology.---According to the United States Department of Agriculture (16), test site A is located in the flood plain of the Brazos River. Flood plain deposits are likely to have a fairly regular structure (27). However, at any point or line of continuity, these deposits can be broken by bodies of other sediments occupying troughs or abandoned river channels (14). These flood plain deposits

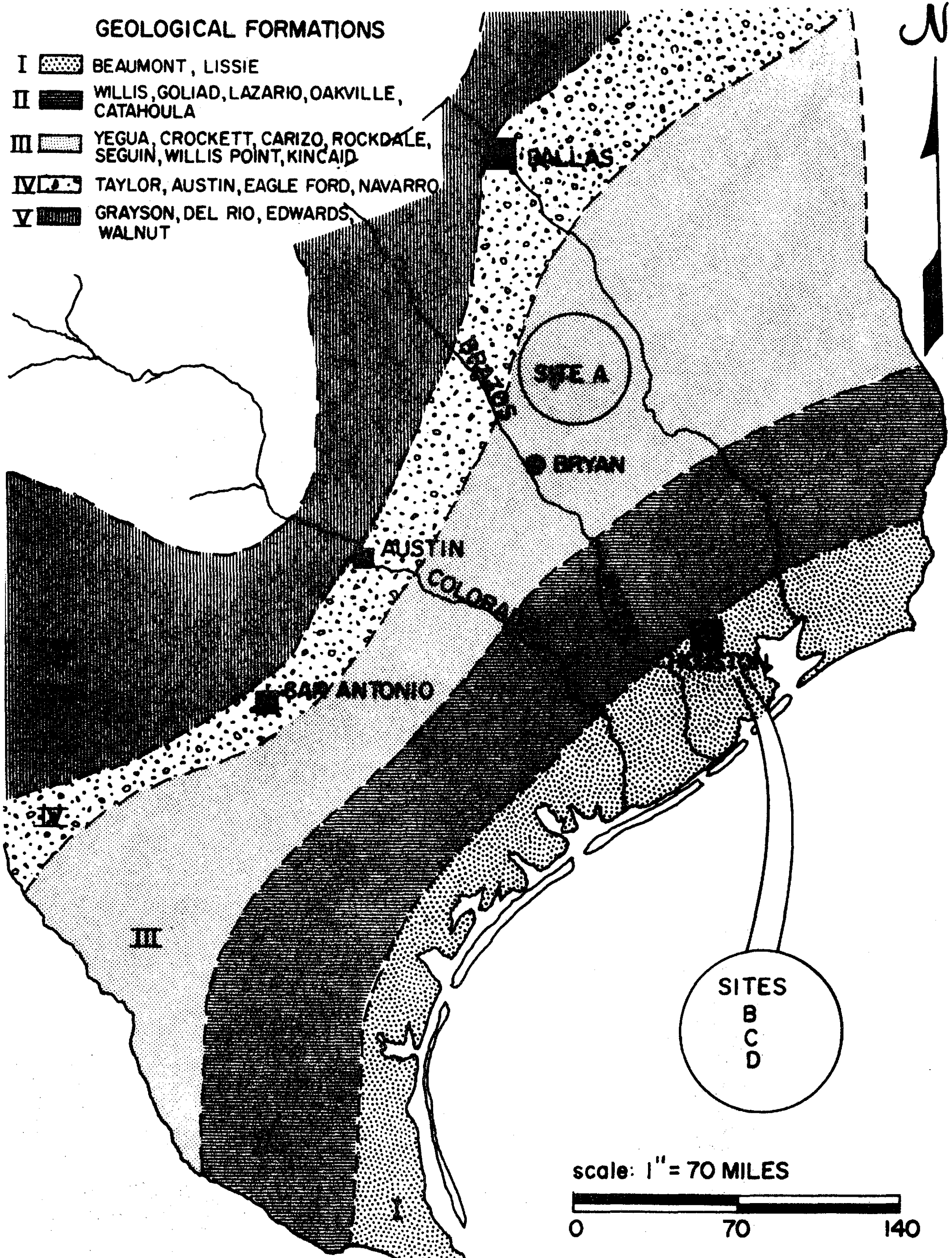


FIG. 3 GENERAL LOCATION OF TEST SITES

are underlain by the Crockett Shale formation (2). This formation is primarily medium gray, fossiliferous shale of normal marine origin. However, it is not known whether this formation was deposited in shallow or deep water. The Crockett Shale is often referred to as the "Cook Mountain Shale" because of its previous inclusion in the Cook Mountain formation.

Test sites B, C, and D are located within the outcrop of the Beaumont clay formation. This formation, which was laid down during the early Wisconsin glacial stage in the form of coalescing alluvial and deltaic plains, is the youngest of a series of Pleistocene terraces forming the Gulf Coastal Plain. The formation consists of poorly bedded plastic clay interbedded with silt and sand lentils and some more-or-less continuous sand layers (26). As a result of exposure to weathering during the late Wisconsin glacial stage, when the sea was more than 400 ft (122 m) below its present level, the clays are overconsolidated by desiccation. These oxidized and leached clays are typically light gray, tan, and red in color with inclusions of calcareous and ferrous nodules. Structurally, the clay is jointed and frequently contains slickensides created by nonuniform shrinkage and expansion. The predominant clay mineral is calcium montmorillonite, and the nonclay minerals are quartz and feldspar (17).

Just to the north of site B is the Lissie sand formation. Between the Lissie sand and Beaumont clay is a secondary formation, locally termed the second terrace. The divisions of different formations in this area are almost indistinguishable, and the formations tend to blend together (1).

SOIL INVESTIGATIONS

Both field and laboratory investigations were required to obtain the information necessary to achieve the objective of this study. The information required includes the resistance to penetration, the corresponding unconsolidated-undrained shear strength, and the properties needed to classify the soils according to the Unified Soil Classification System.

Field Investigation.---The purpose of the field investigation was to obtain the resistance to penetration using the THD Cone Penetrometer Test. At the same time, soil profiles were established and undisturbed soil samples were taken for use in the laboratory investigation. Location, boring number, and depth of penetration of the seven soil borings taken are given in Table 1.

TABLE 1.--Summary of Soil Borings

Location	Boring number	Site designation	Depth of penetration, ft
Brazos County	1a* 1b	A A	26 46
Harris County	2 3	B B	70 70
Harris County	4 5	C C	30 30
Harris County	6 7	D D	42 43.5

1.0 ft = 0.305 m
*Boring 1a was terminated at 26 ft because it was too close to a previously drilled boring.

These borings were made using a truck-mounted Failing-1500 rotary drilling rig.

The THD Cone Penetrometer Test was conducted in borings 1, 3, 4, and 7 respectively. With the exception of boring 1, the Cone Test was conducted at 2.5-ft (0.7625-m) intervals. In boring 1 it was conducted at 5-ft (1.525-m) intervals.

The procedure used to obtain the penetration resistance is described in detail in the Texas Highway Department Manual (5). Although the specifications require the penetrometer to "be driven twelve blows in order to seat it in the soil or rock," this seating process was determined by the driller. Driving then proceeds in increments of six inches at a rate of 18 to 24 blows per minute. The reported blow count is the number of blows required to drive the cone a distance of one foot. When the number of blows required for one foot of penetration exceeds 100, driving stops and the penetrated distance is recorded.

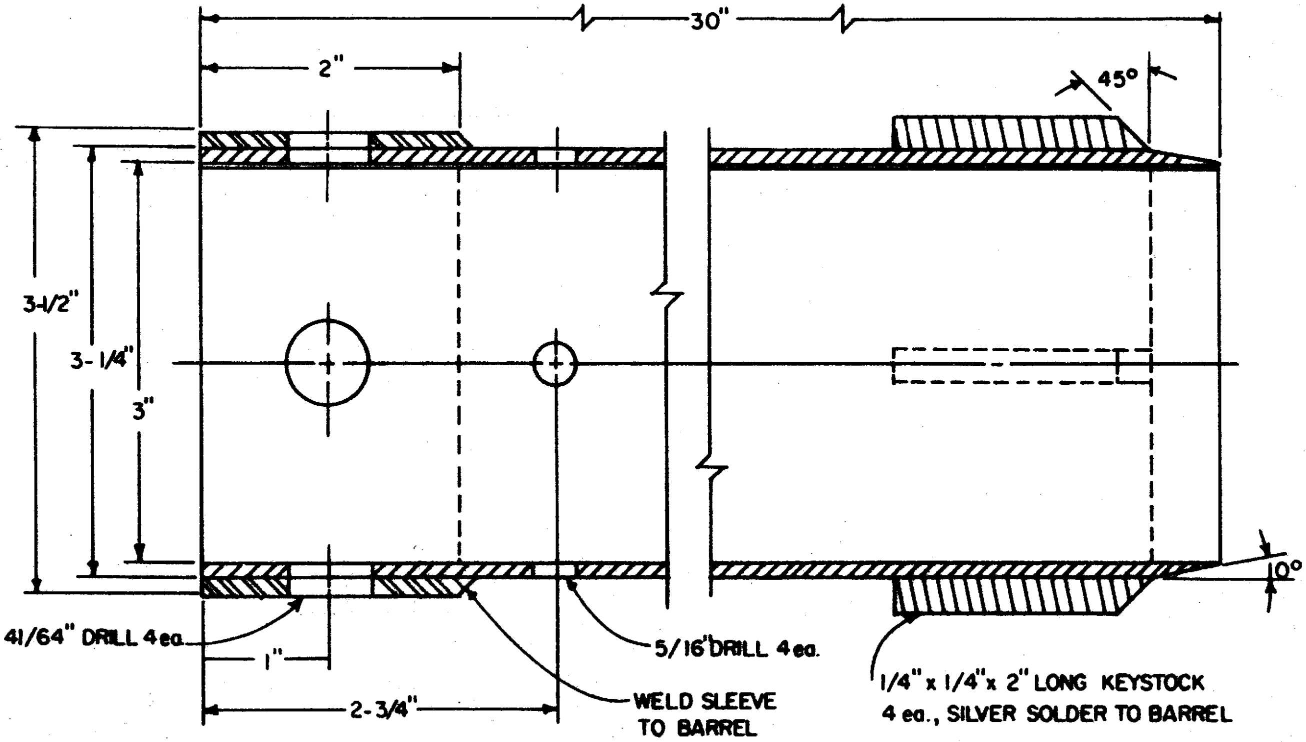
Careful consideration was given to the cleaning of the bottom of the bore hole after completion of each cone penetrometer test. This was accomplished by coring the soil at the bottom of the hole with a push barrel sampler and slowly extracting the drill pipe. Although the sample in the barrel was disturbed because of the penetrometer action, it was extruded and used for visual classification.

In borings 2, 5, and 6 undisturbed samples were taken continuously. In boring 1 samples were taken above and below the depth at which the THD Cone Penetrometer Test was conducted. Soils were sampled with the push barrel sampler shown in Fig. 4. Each core was examined in the field by personnel from both the Texas Highway Department and TTI. Representative portions of each core were sealed and packaged for transportation to the TTI Soils Laboratory.

The depth to groundwater in the open bore holes was measured at various times ranging from a few hours to 72 hours following completion of the boring. Only very slight changes in the depth to water occurred after the 24-hour reading. The depth to groundwater was recorded on the right corner of the appropriate boring log.

Laboratory Investigation.--The purpose of the laboratory investigation was to obtain the unconsolidated-undrained strength and to classify the soils according to the Unified Classification System. Soil shear strength was determined by using the types of tests listed in Table 2. Also shown in Table 2 are the numbers of each type of test conducted for each site.

Since the Texas Highway Department uses both the Texas Triaxial Test and the Transmatic Triaxial Test, it was necessary to use these two tests as the means of obtaining soil shear strength. However, the Transmatic Triaxial is limited to a maximum confining pressure of 25 psi (172.5 kN/m^2) and can only be used with soils that are firm in consistency. Therefore, the Texas Triaxial Test (TAT)



BARREL — 3-1/4" O.D. x 3" I.D. SEAMLESS STL. TUBING

SLEEVE — 3-1/2 O.D. x 3-1/4 I.D. SEAMLESS STL. TUBING

FIG. 4 PLATED 3" PUSH BARREL
stk no. 150770

COURTESY OF THE TEXAS HIGHWAY DEPARTMENT
1 in. = 25.4 mm

was the primary test used in this study. Selected samples were tested in accordance with the procedures specified in ASTM Test 2850-7. The ASTM tests were compared to the TAT test results in a relative manner similar to the method used by Lumb (15).

TABLE 2.--Type and Number of Triaxial Tests

Type of test	Number of tests			
	Site A	Site B	Site C	Site D
Texas Triaxial, unconsolidated-undrained,				
single-stage	16	20	12	9
multi-stage	2	5	2	1
Transmatic Triaxial Compression, unconsolidated-undrained, multi-stage	1	1	1	1
ASTM Triaxial unconsolidated-undrained,				
single-stage	3	4	5	4
multi-stage	1	5	2	1

Since the loading rate that produces failure of foundations may occur before any appreciable drainage can take place, it was considered appropriate to conduct unconsolidated-undrained or quick strength tests. The quick shear strength of each sample tested in the single-stage type test was determined by using a confining pressure approximately equal to the effective overburden pressure. The effective overburden pressure was determined from the unit dry weights and moisture contents of the soils above the depth at which the sample was obtained, and from the location of the groundwater table. The plot of effective overburden pressure versus depth for each test site is shown in Fig. 5. For example, from Fig. 5a, a soil sample that was recovered from 23 ft (7.015-m) was tested using a confining pressure equal to 20 psi (138 kN/m^2).

When conducting the quick test, the deviator stress, that is the vertical stress minus the confining pressure, at failure is independent of the magnitude of the confining pressure for saturated soils (3). This is also known as the $\phi = 0$ condition. In order to investigate whether the $\phi = 0$ condition existed for the soils tested in this study multi-stage triaxial tests were conducted on selected samples from each soil strata. If the $\phi = 0$ condition did not exist, the soil was either partially saturated or it was a fissured soil which was tested at a confining pressure less than the overburden pressure (4). In order to determine the degree of saturation it was necesary to conduct a specific gravity test on the samples that were tested in multi-stage tests.

Diagrams of the triaxial test apparatus used in this study are shown in Figs. 6,7, and 8. The Texas Triaxial Test apparatus that is shown diagrammatically in Fig. 6 includes a rubber membrane 0.051 in. (1.3mm) thick that is fitted to a lightweight stainless steel cylinder. The ASTM Triaxial Test apparatus shown diagrammatically in Fig. 7 includes a 0.012 in. (0.30mm) thick rubber membrane which completely seals the sample. The sealed sample is enclosed in a cell where it can be subjected to either fluid or air pressure. The Transmatic Triaxial cell is similar to an ASTM

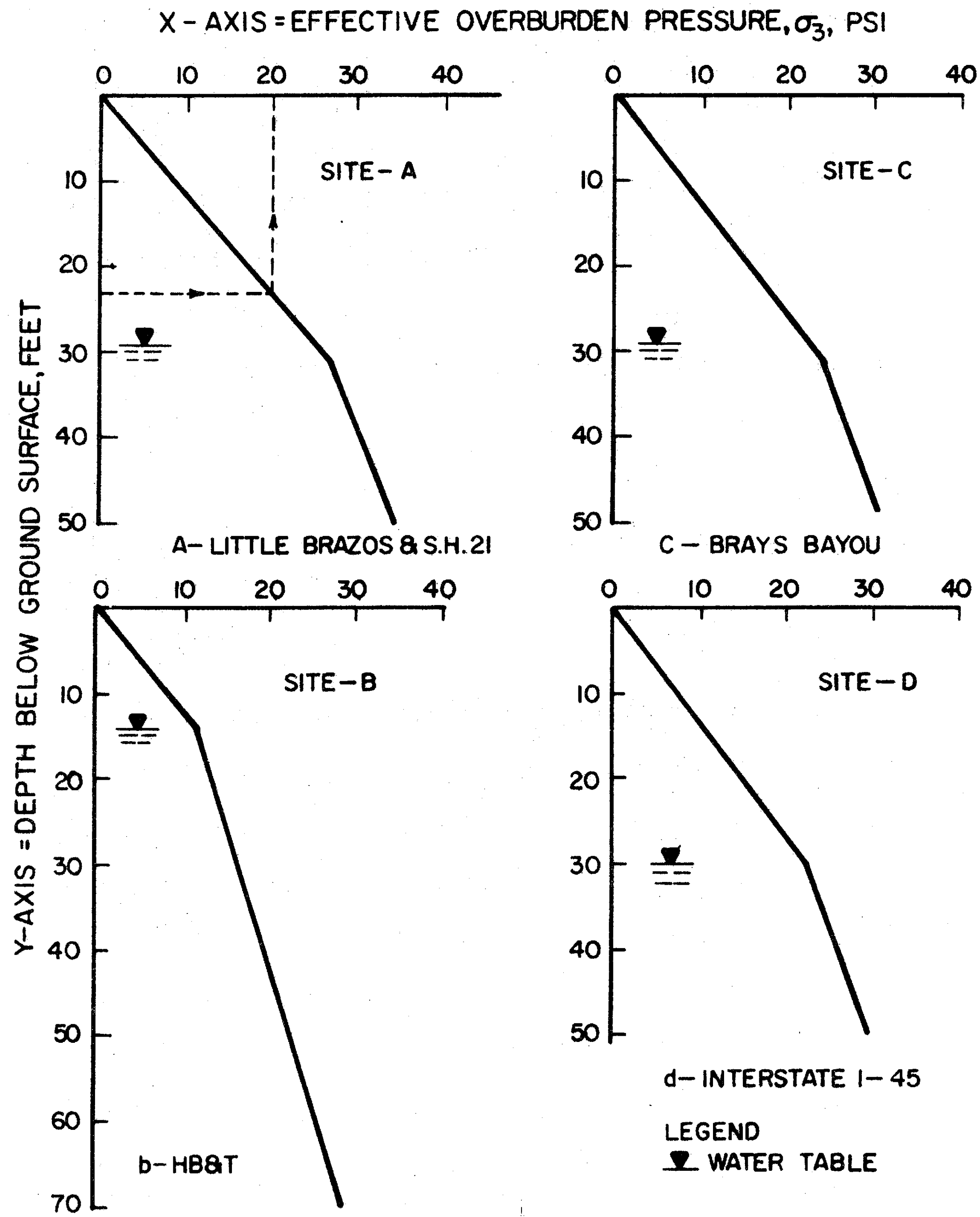


FIG. 5 EFFECTIVE OVER BURDEN PRESSURE VS. DEPTH

(1 FT. 0.305 m, 1 PSI = 6.9 KN/m^2)

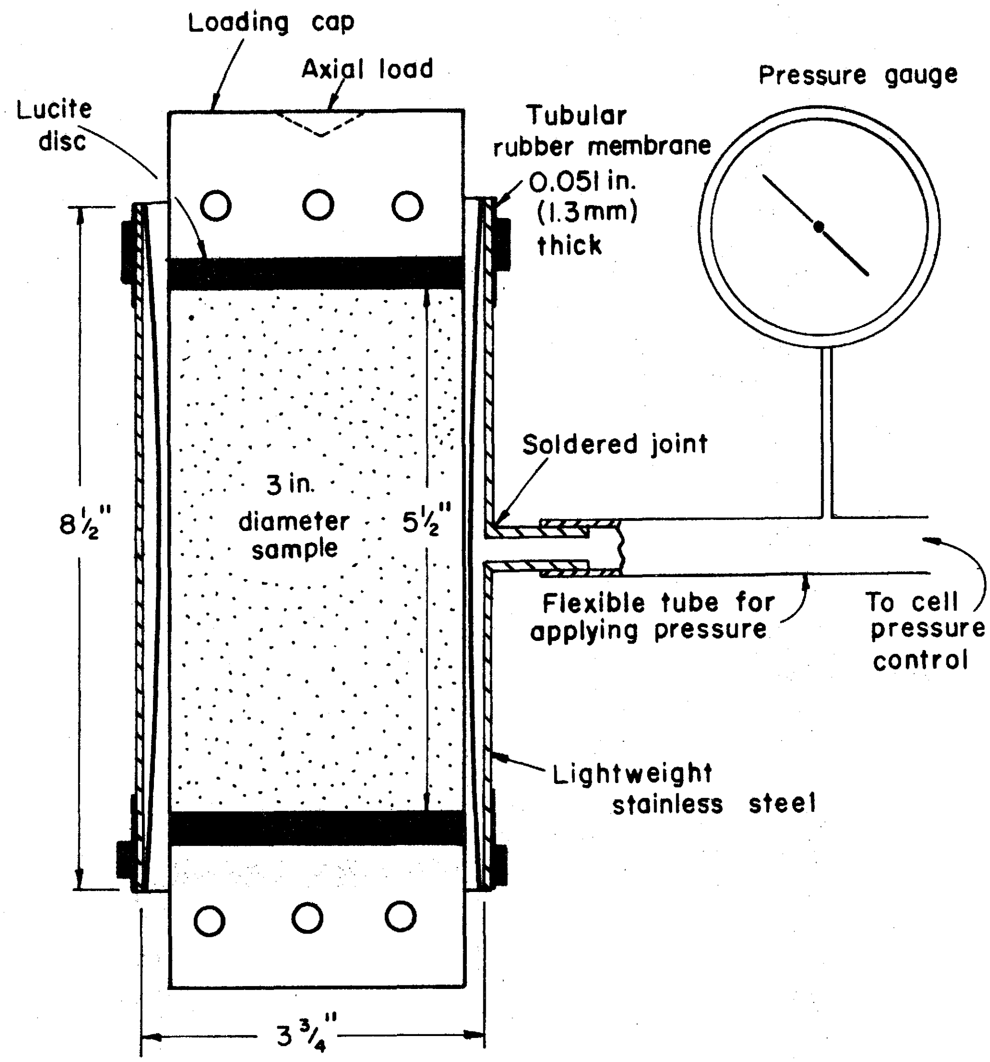


FIG. 6— DIAGRAMMATIC LAYOUT OF THE TEXAS TRIAXIAL TEST (1.0 in. = 25.4 mm)

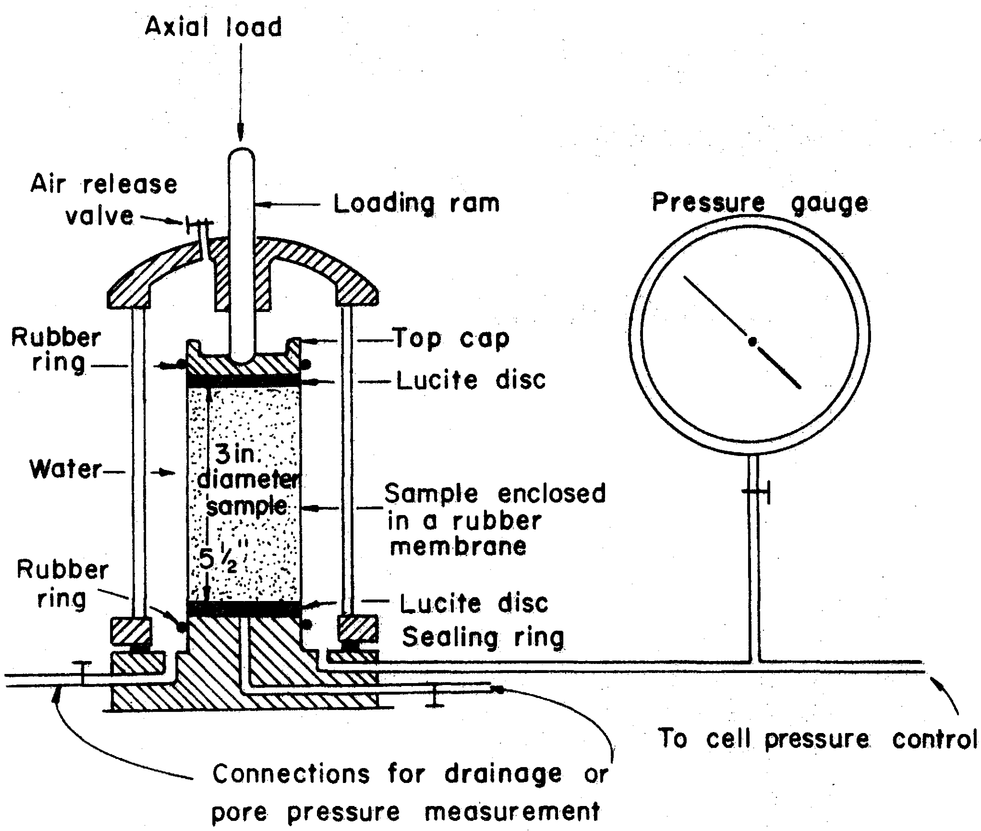


FIG. 7— DIAGRAMMATIC LAYOUT OF THE ASTM TRIAXIAL TEST (1.0in.= 25.4mm)

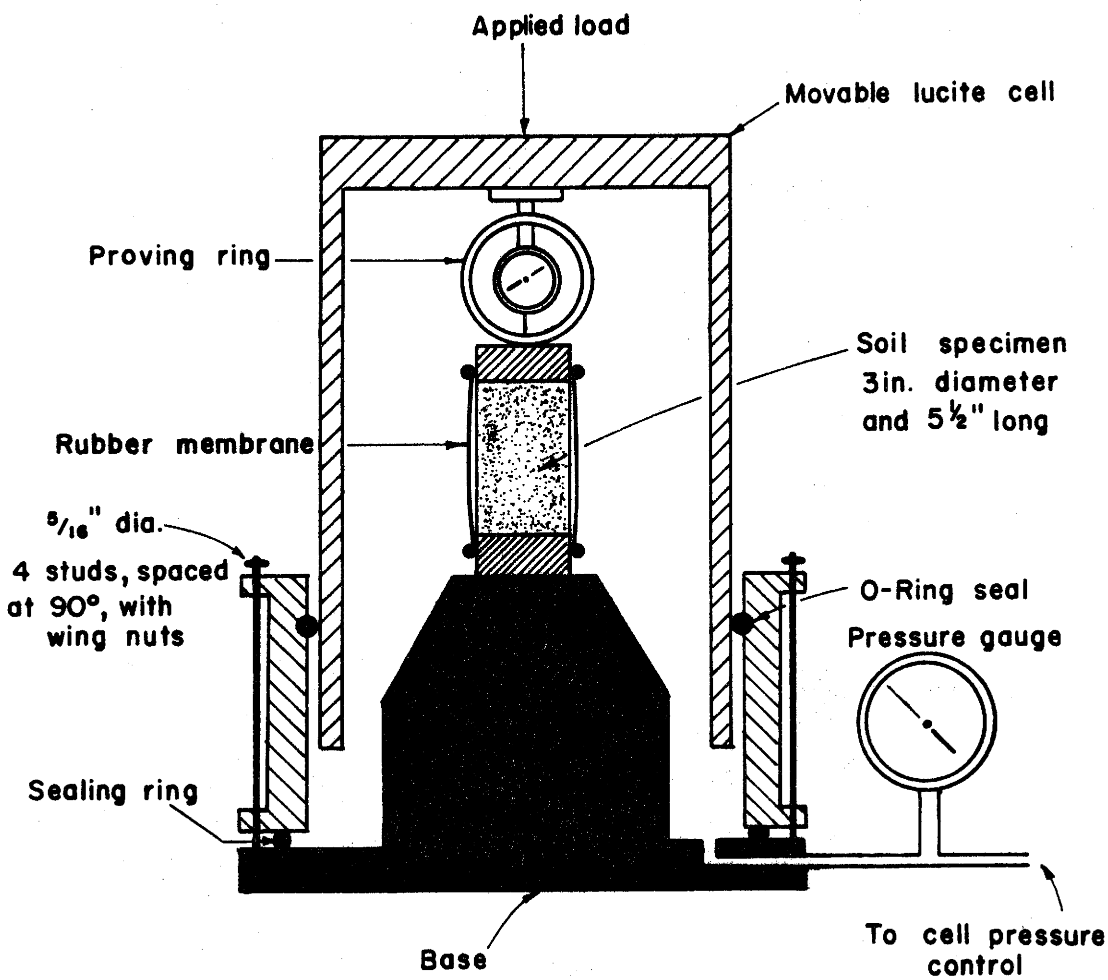


FIG. 8— DIAGRAMMATIC LAYOUT OF THE TRANSMATIC TRIAXIAL COMPRESSION TEST (1.0in. = 25.4mm)

Triaxial cell, except that the proving ring is placed directly on top of the soil sample inside the cell as shown in Fig. 8, and air pressure is used as the confining pressure. The apparatus used to conduct the classification tests are the conventional ones that are normally found in every soil mechanics laboratory.

Water content and unit dry weight determinations were made for all samples tested. Atterburg Limits and percent passing No. 200 sieve were also determined.

Tests were conducted in accordance with the procedures outlined in the Texas Highway Department Manual of Testing (28). Table 3 shows the type of test and the corresponding procedure used. The ASTM Triaxial and the Transmatic Triaxial Test procedures are not given in the THD Testing Manual (28). However, the procedure used was essentially the same as that used for the Texas Triaxial Test. As noted previously, the membrane used to seal the sample in the ASTM and Transmatic Tests was considerably thinner than the one used for the TAT Test. Each sample was tested in compression using the same motorized press assembly geared to travel at a rate of 0.135 in. (3.429 mm) per minute. Simultaneous readings of load and deformation were taken at intervals of 0.01 in. (.254 mm) deformation until the sample failed.

The procedure used to conduct the multi-stage triaxial test on a soil sample was to confine it initially at a pressure somewhat less than the effective overburden pressure. The sample was then loaded in compression at a constant rate of 0.135 in. (3.429 mm) per minute until the load was only increasing about 2 lb (.906 kg) per 0.01 in. (.254 mm) of deformation. The deviator stress at this stage was taken as the failure stress under the applied confining pressure. The confining pressure was then increased and this process was repeated for two additional stages. The two additional confining pressures, one equal to and the other greater than the in situ effective overburden pressure, were used.

TABLE 3.--Type of Test and Procedure

Type of test	Test method used
Texas Triaxial Test (1)	Tex-118-E
Moisture content	Tex-103-E
Liquid Limit	Tex-104-E
Plastic limit	Tex-105-E
Minus No. 200 sieve	Tex-111-E
Specific gravity (2)	Tex-108-E

(1) The sample dimensions were 3 in. (76.2 mm) in diameter and 5.5 in. (139.7 mm) in length. Confining pressure used equaled the effective overburden pressure.

(2) A partial vacuum was used. The weight of dry sample was determined after test was performed.

SUMMARY OF TEST RESULTS

The laboratory test results, except soil shear strength, were summarized in the form of a boring log for each test site. The unconsolidated-undrained shear strength and N-values were summarized on a separate boring log to facilitate the development of a correlation between these two parameters. Pertinent soil properties were used to describe the soil conditions. The results from Atterberg Limits and the particle size tests were used to classify the soils.

Soil Conditions and Classifications.--As shown in Fig. 9, the underlying soils at site A are primarily clays of high to moderate plasticity with some interlayered silt. A relatively pervious layer of silt, about 3 ft (0.915 m) thick, was encountered at 27.5 ft (8.39 m) below ground surface. Beneath this pervious layer, the soils are primarily fissured clays of moderate plasticity having broken skeletal remains of marine organisms.

All soils encountered at this site were naturally deposited. The upper strata to a depth of about 30 ft (9.15 m) is mostly clay with a combination of red and brown color. This clay has relatively high shear strengths and corresponding low moisture contents, characteristics which may be produced by oxidation and desiccation. The water contents generally range between 20% and 30%. The liquid limits range from about 42 for the silty clays to about 70 for the homogeneous clays. The plastic limits range between 18 and 25.

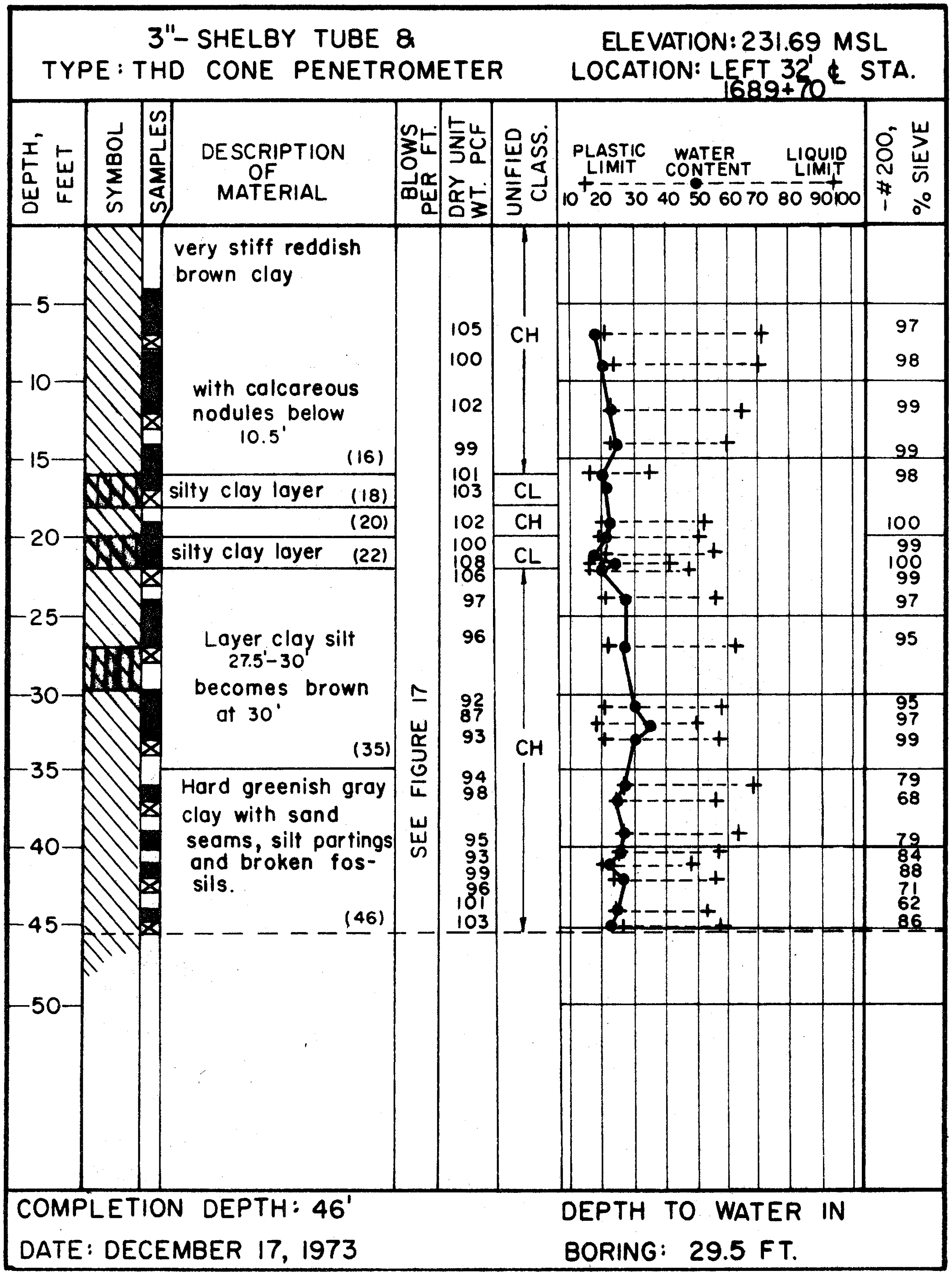


FIG. 9 – LOG OF BORING I, SITE A, SH 21 AND LITTLE BRAZOS RIVER, BRAZOS COUNTY

Natural water contents are generally near the plastic limit indicating low compressibility. Nearly 100% of the particles pass the No. 200 sieve, and the degree of saturation ranges between 88% and 100%. The pervious layer with the reddish brown color contained some sand and exhibited very little resistance to cone penetration. The clay below the pervious layer is gray and green in color, and highly saturated. The natural water content is nearly equal to the plastic limit which ranges from 25 to 30. The liquid limit varies from about 51 to 60. The Unified Classification for the soils from site A are shown in Fig. 10 which reveals that the clays are primarily CH materials. Detailed test data for site A are given in Table III-1, Appendix III.

The significant characteristics of the underlying soils at site B are given in Fig. 11. This boring log reveals an erratic variation in the natural soil deposits. A 3-ft (.92-m) layer of sandy clay and shell fill material was encountered at the surface of this site. Beneath this fill light gray and tan clayey sand exists to a 12-ft (3.66-m) depth. The percent of materials that pass the No. 200 sieve range between 38% to 48%. The natural water content which is generally near plastic limit ranges between 17% and 18%. The liquid limit ranges between 25 and 35. This clayey sand is highly saturated. This layer is underlain by a layer of silty fine sand which could not be sampled to a depth of 21 ft (6.4 m) below ground surface. However, the resistance to cone penetration indicates that this sand is medium dense. Beneath the

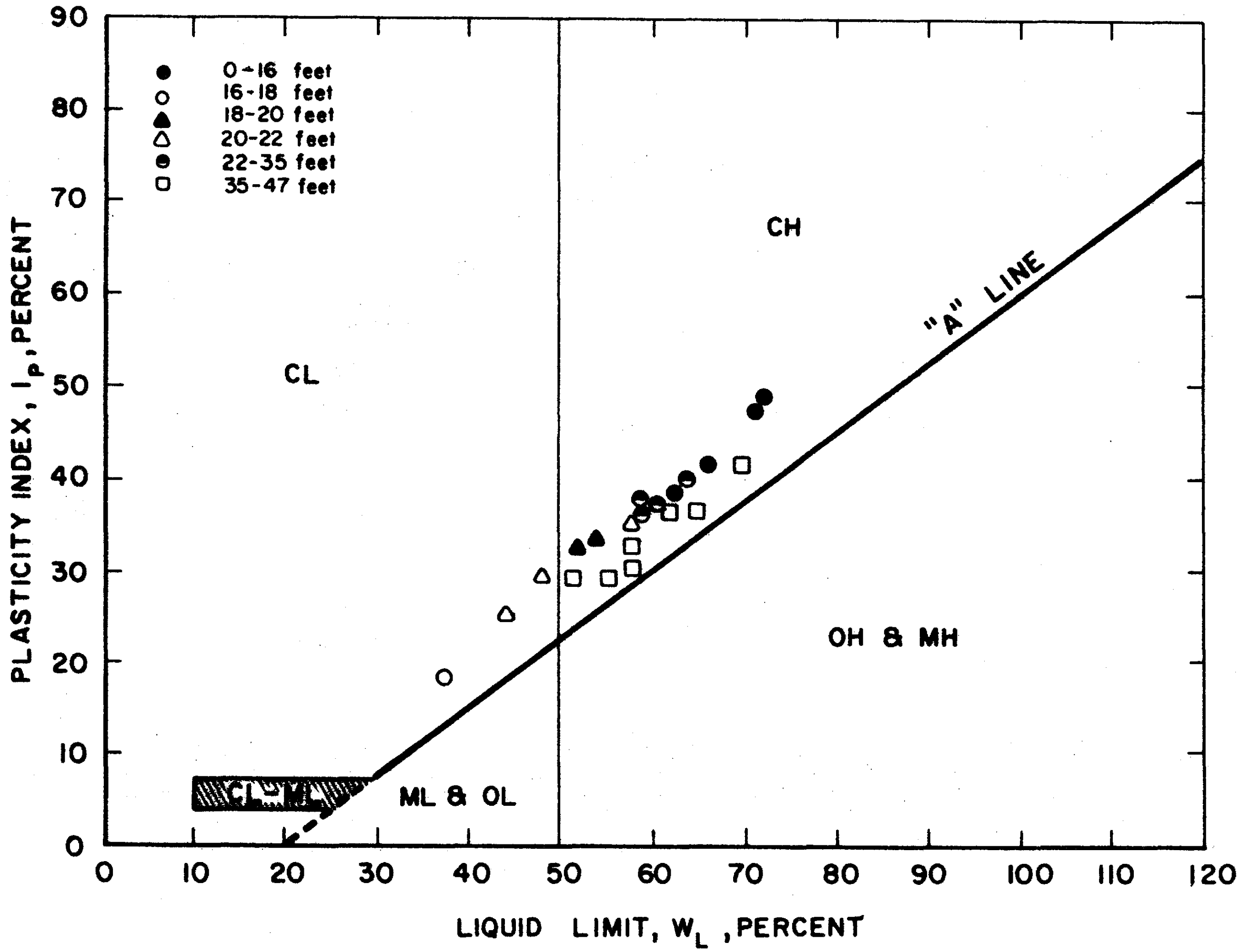


FIG. 10– UNIFIED CLASSIFICATION OF SITE-A

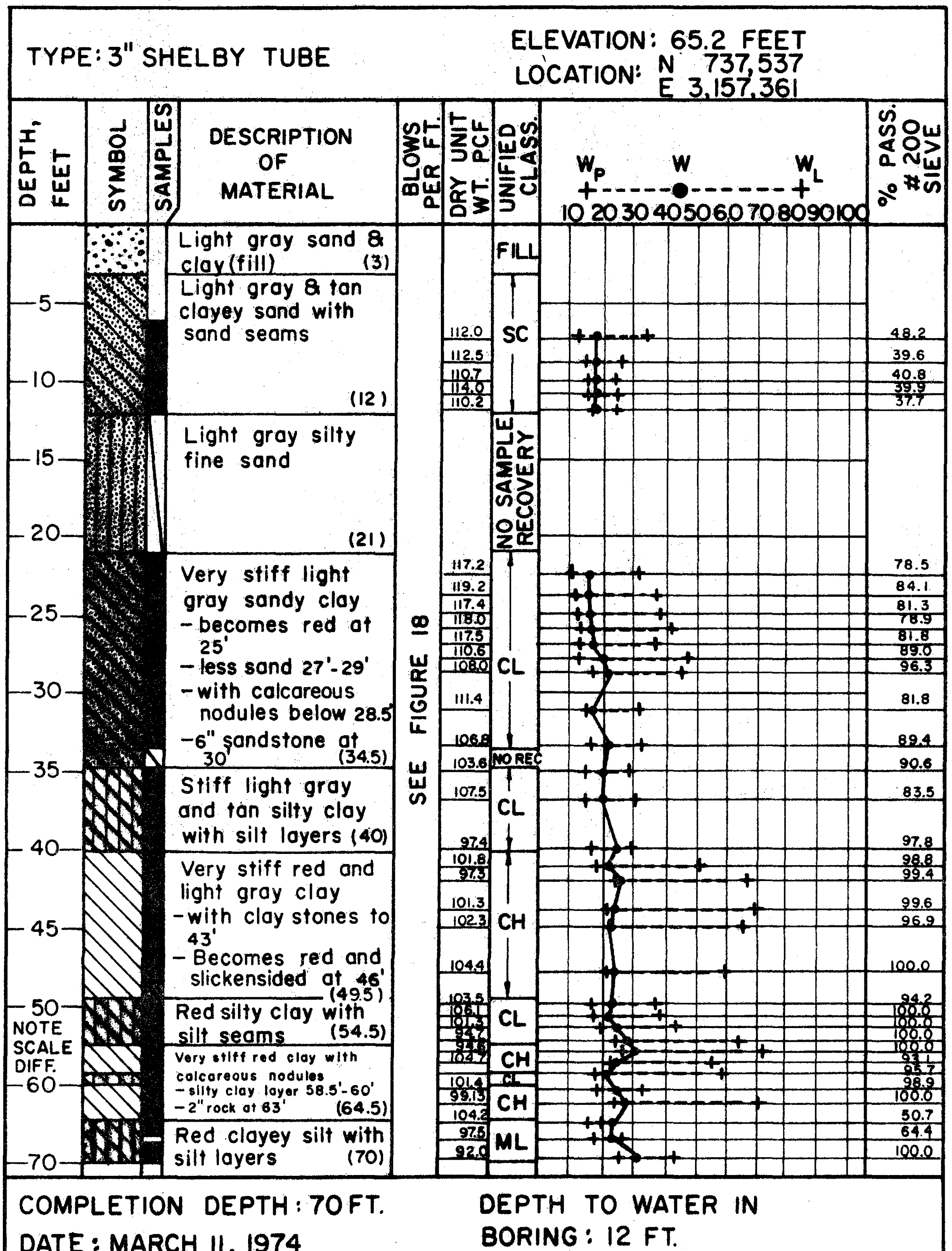


FIG. 11 — LOG OF BORING 2, SITE B, INTERSTATE HIGHWAY 610 AND HB&T RAILROAD, HOUSTON, TEXAS

silty sand the soils are primarily clays to a depth of 64.5 ft (19.67 m). The thickness of these clays vary, and they contain layers and seams of cemented soils at various depths. The liquid limits range from about 31 for the sandy and silty clays to about 72 for the clays that exhibited a slickensided structure. The plastic limits vary between 15 and 25 and are generally near the natural water contents. The percent of materials that pass the No. 200 sieve vary between 51 for the sandy clays to 100 for the other clays. These clays are underlain by clayey silt to the termination depth of 70 ft (21.4 m). The Unified Classification for the soils from site B are shown in Fig. 12 which reveals that the clays are both CL and CH materials. All test data for site B are given in Table III-2 of Appendix III.

As shown in Fig. 13 the stratification at site C consists of the following: a surface fill of brown and tan sandy clay to a depth of about 5 ft (1.53 m); 3 ft (0.92 m) of naturally deposited, light gray and tan, sandy clay containing calcareous nodules; and 4 ft (1.22 m) of light brown and light gray clay with calcareous nodules. Below the depth of 12 ft (3.66 m) lean clays exist that are silty to a depth of 25 ft (7.63 m) and then become sandy to a depth of 28 ft (8.54 m). At the termination depth of 30 ft (9.15 m) silty fine sand was encountered. This sand is overlain by a transitional 1-ft (.305-m) layer of clayey sand.

The water contents of the natural soils at site C generally

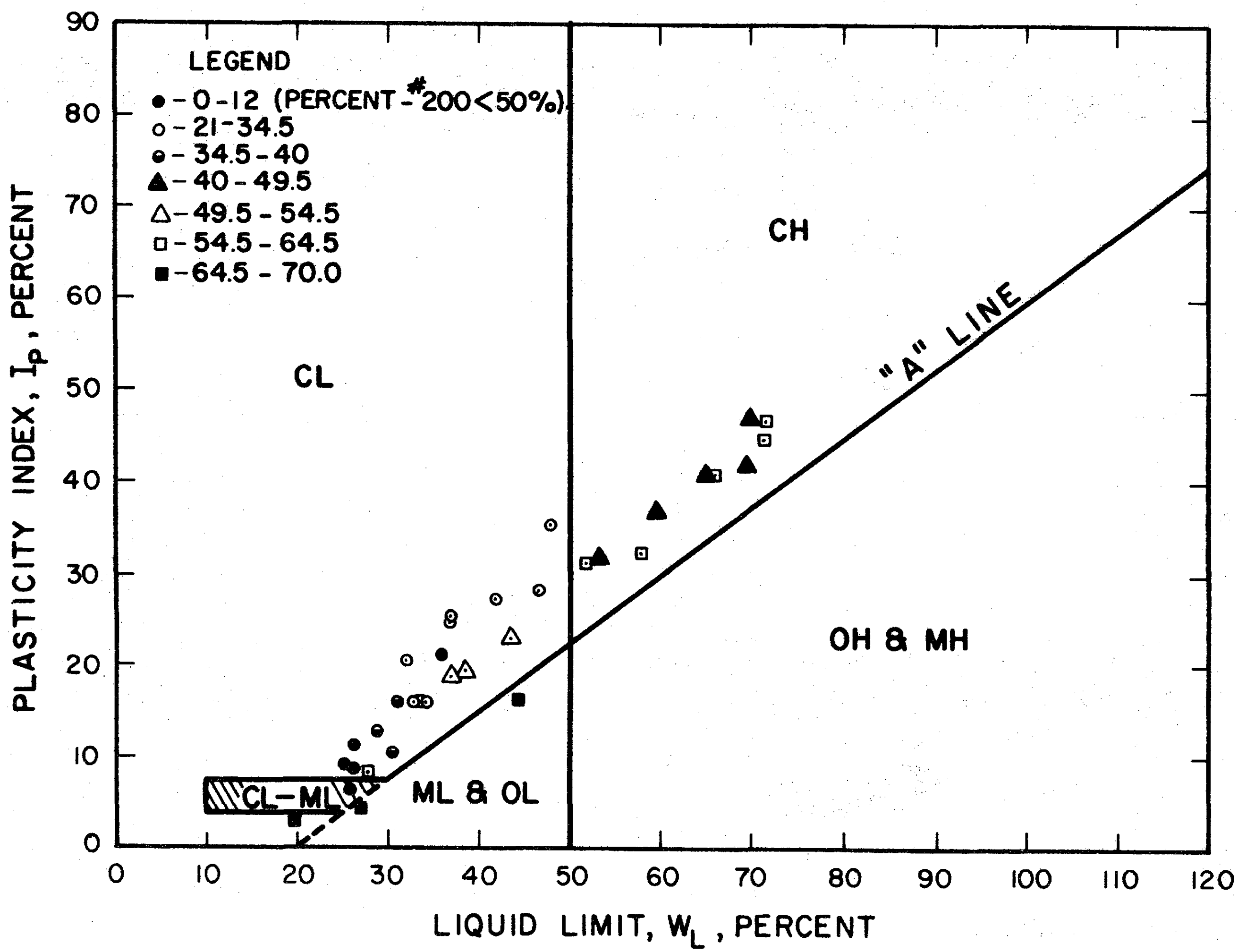


FIG. 12 — UNIFIED CLASSIFICATION OF SITE B

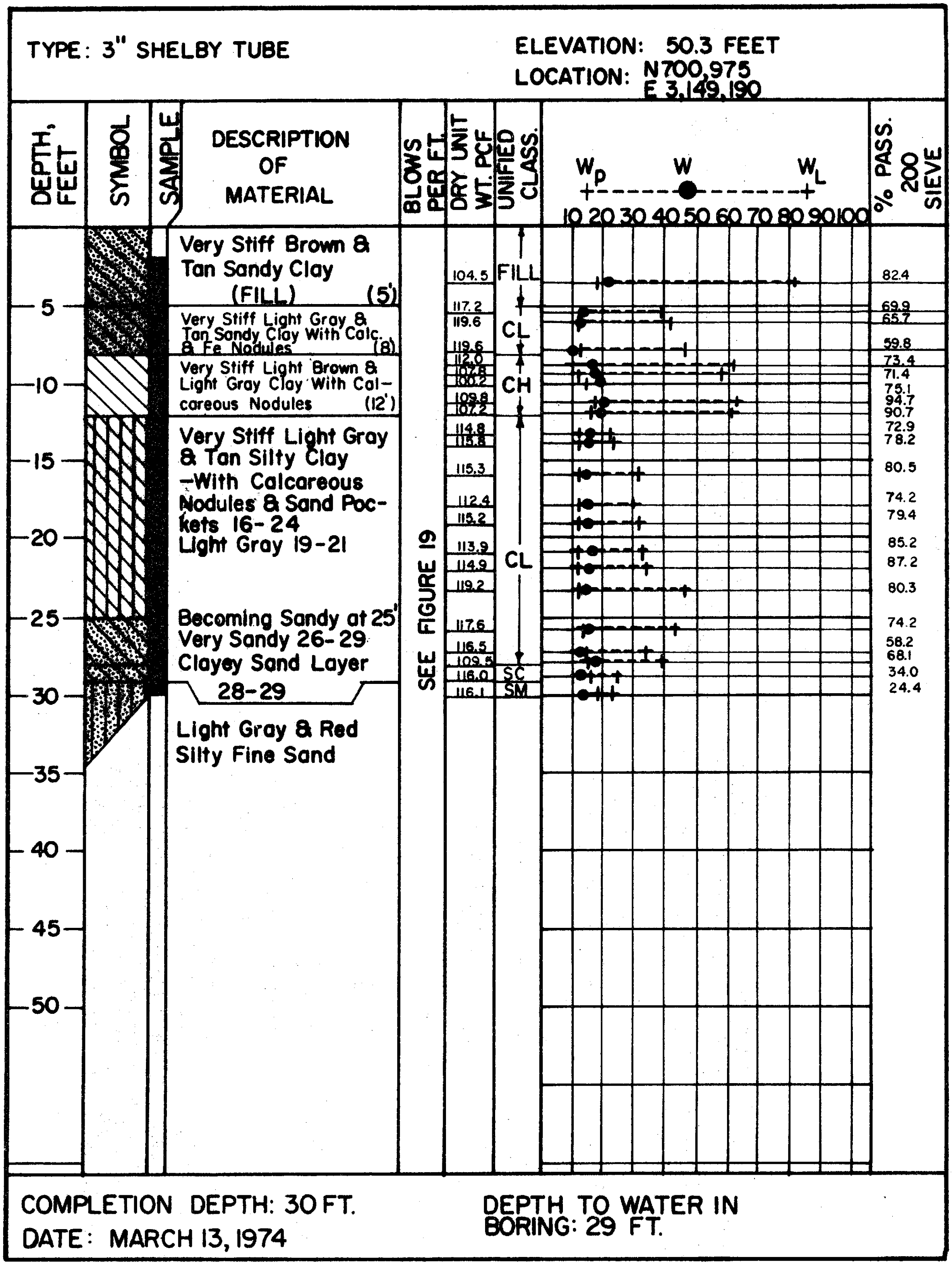


FIG. 13–LOG OF BORING 5, SITE C, BRAYS BAYOU AT STATE HIGHWAY 288, HOUSTON, TEXAS

vary between 10% and 20%. The plastic limits of the clays vary in the narrow range of 11 to 15. The clayey sand plastic limit is 17. The liquid limits of the clays vary from 20 to 35 for the silty clays and from 39 to 46 for the sandy clays. The homogeneous clay liquid limit is around 60. The amount of material passing the No. 200 sieve for the clays varies between 59% for the sandy clays and 95% for the other clays. The degree of saturation varies between 75% and 100%. The Unified Classification for the site C soils is given in Fig. 14. Site C is primarily CL materials. The data of all tests for site C are given in Table III-3 of Appendix III.

The boring log of the underlying soils at site D is presented in Fig. 15. These soils are of the Beaumont Clay formation. A highly saturated tan and gray clay was encountered to a depth of 24 ft (7.32 m) below ground surface. This clay is slightly silty and contains calcareous and ferrous nodules. A 6-in. (152.4-mm) layer of silt that is underlain by a 12-in. (304.8-mm) layer of clayey sand was sandwiched in the middle of this clay strata. The liquid limits vary from 58 to 70. The plastic limits range between 15 and 18. Natural water contents are generally near plastic limits, indicating low compressibility. Beneath this plastic clay there is a silty clay that becomes sandy clay below the 26-ft (7.93-m) depth to about the 40-ft (12.2-m) depth. These lean clays contain layers of sand and silt at various depths. The liquid limits are between 25 and 40. The natural water contents

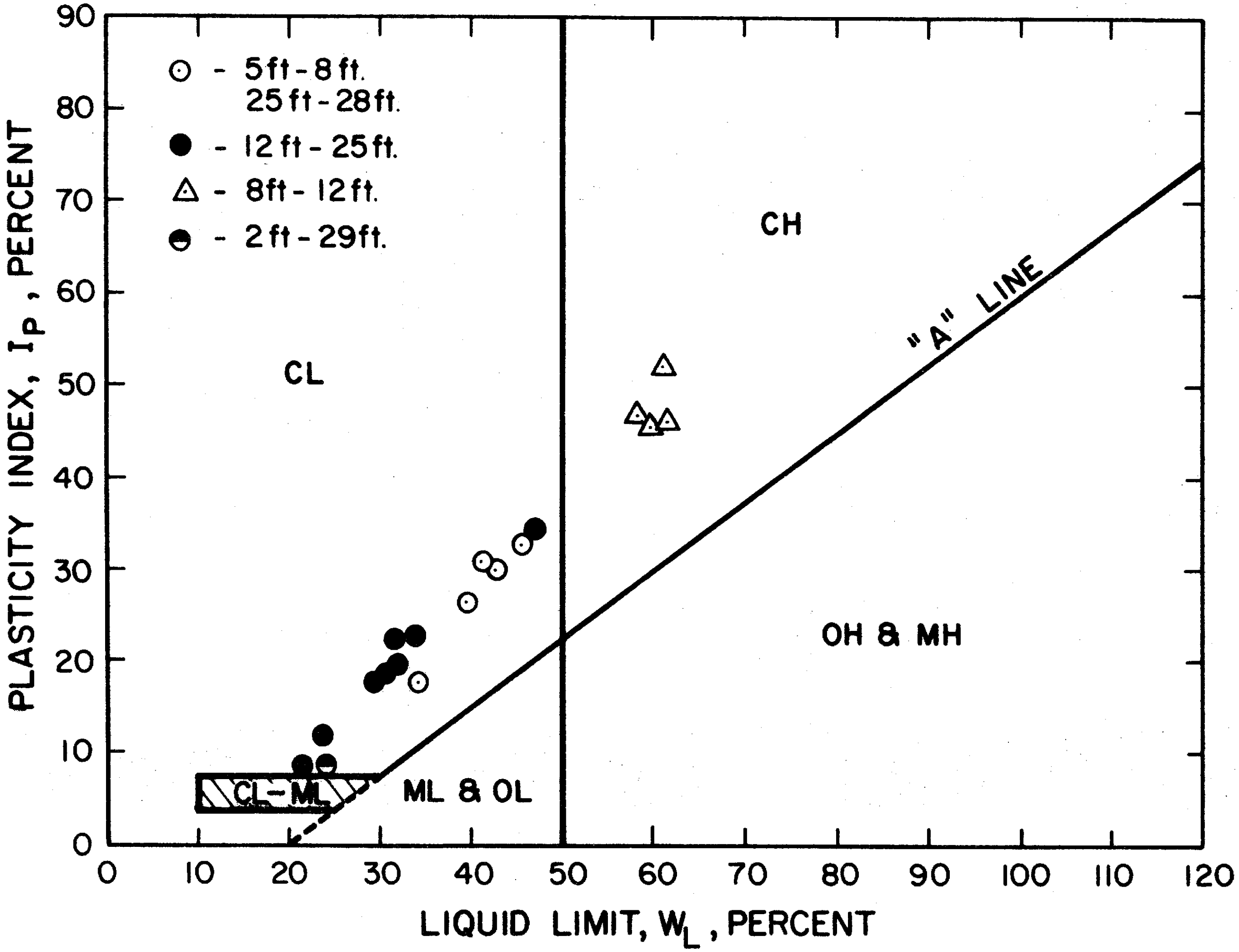


FIG. 14—UNIFIED CLASSIFICATION OF SITE C

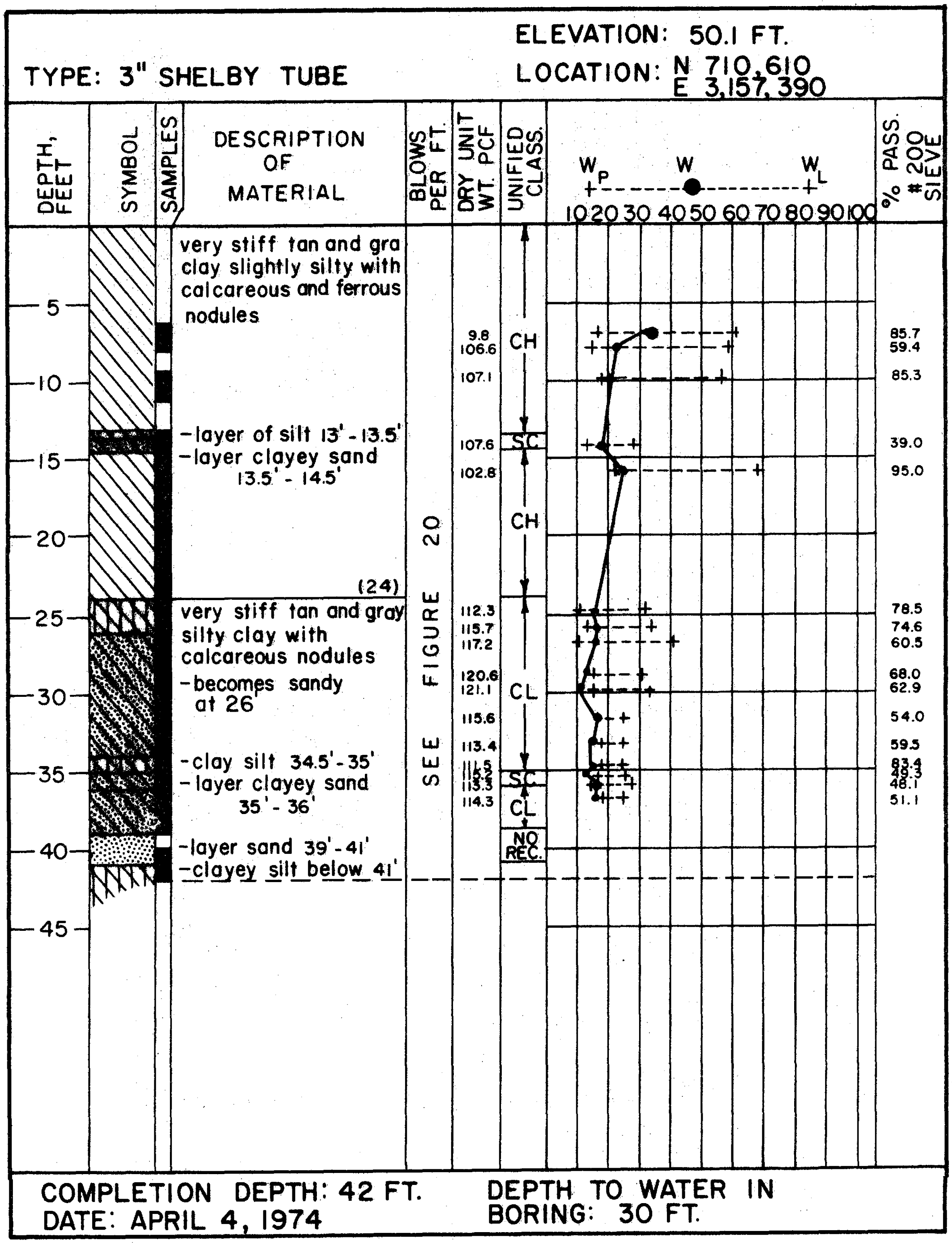


FIG. 15—LOG OF BORING 6, SITE D, INTERSTATE HIGHWAY 45 AT NETTLETON STREET, HOUSTON, TEXAS

of these highly saturated clays are near the plastic limits which vary between 10 and 20. The percent of materials that pass the No. 200 sieve varies between 51% and 80%. These clays are underlain by a tan and gray clayey silt to the termination depth of 42 ft (12.81 m). The Unified Classifications for the site D soils are given in Fig. 16. This figure indicates that the site D soils are both CL and CH materials. All test data for site D are presented in Table III-4 of Appendix III.

Groundwater Observations.--Observations made in the open bore holes at site A, two days after drilling was completed, showed that the borings had collapsed at 22 ft (6.71 m) and were dry. Further observations were made approximately two months later in an open hole during construction of drilled shafts at the same site. These observations indicated that the depth to water at site A was 29.5 ft (9.0 m) below ground surface. This depth was also verified by previous explorations that were made by the Texas Highway Department.

Observations made in the open bore holes at site B, 18, 24, and 36 hours after drilling was completed, indicated that groundwater level at this site was at a depth of 12 ft (3.7 m) below the ground surface. The observed groundwater level was the same after the 24-hour and 36-hour readings.

The water level reported on the boring logs at site C was determined from previously drilled borings at the same site. The reported level was 29 ft (8.85 m). It was impossible to observe

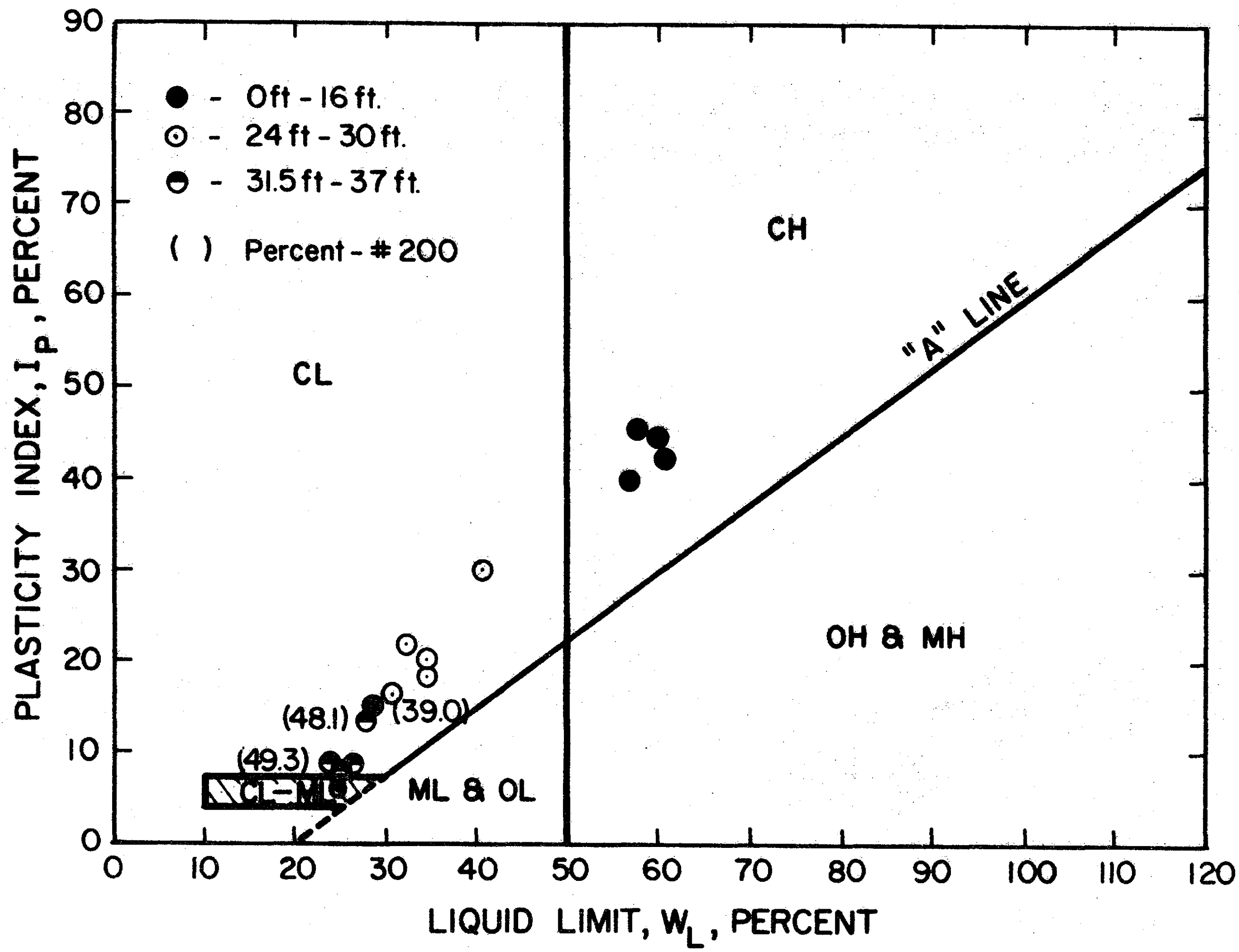


FIG. 16 – UNIFIED CLASSIFICATION OF SITE D

the groundwater level at this site at the time since the borings were located on a pedestrian route, and it was necessary to seal them immediately after the boring operation was completed.

At site D several observations were made to determine the groundwater level during the same day the drilling was accomplished. The 2-hour reading indicated that the water level was at about 1.4 ft (0.43 m) below ground surface. This level was not used because the water in the bore hole did not have enough time to stabilize with the actual groundwater level. However, two previously drilled borings by the Texas Highway Department showed that the groundwater level was at 10 ft (3.05 m) and 30 ft (9.15 m). These two borings are in the same vicinity as site D. The water table level of 10 ft (3.05 m) was probably a perched water. Therefore, the 30-ft (9.15-m) water level was used.

Cone Penetrometer N-values and Soil Shear Strength.--Since the objective of this study is to develop a correlation between the THD Cone Test N-value and the corresponding soil shear strength, it is appropriate to summarize these data together on a boring log of each test site. Figures 17 through 20 are the profiles of the resistance to penetration and the unconsolidated-undrained shear strength. The shear strength was obtained by the Texas Triaxial Test. The data presented in these figures will be used as the basis for the correlation of these two parameters. The N-values that are lower than 100 blows per foot are the actual blow counts

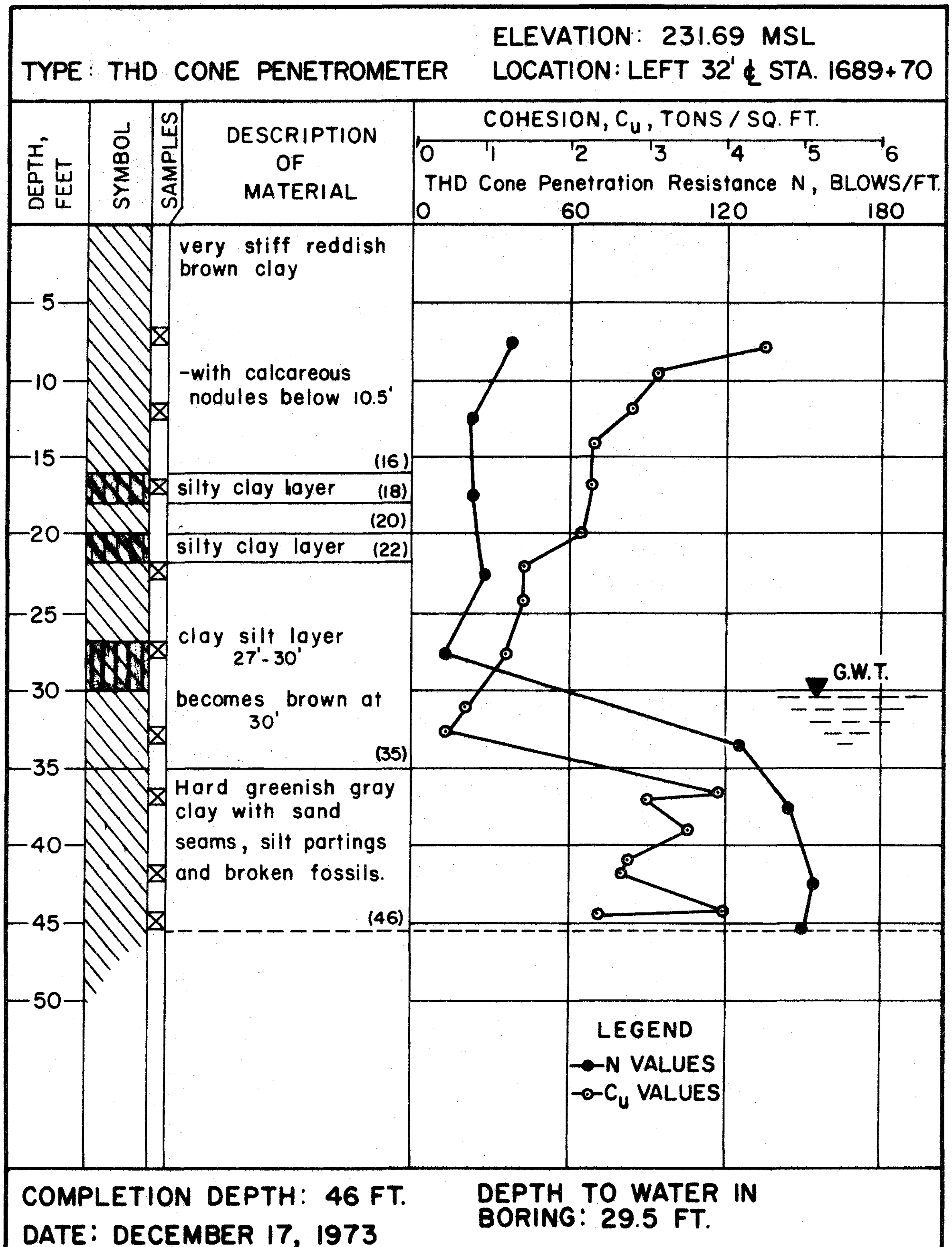


FIG. 17—BORING I, SITE A, VARIATIONS OF RESISTANCE TO PENETRATION, N, AND TAT SHEAR STRENGTH WITH DEPTH

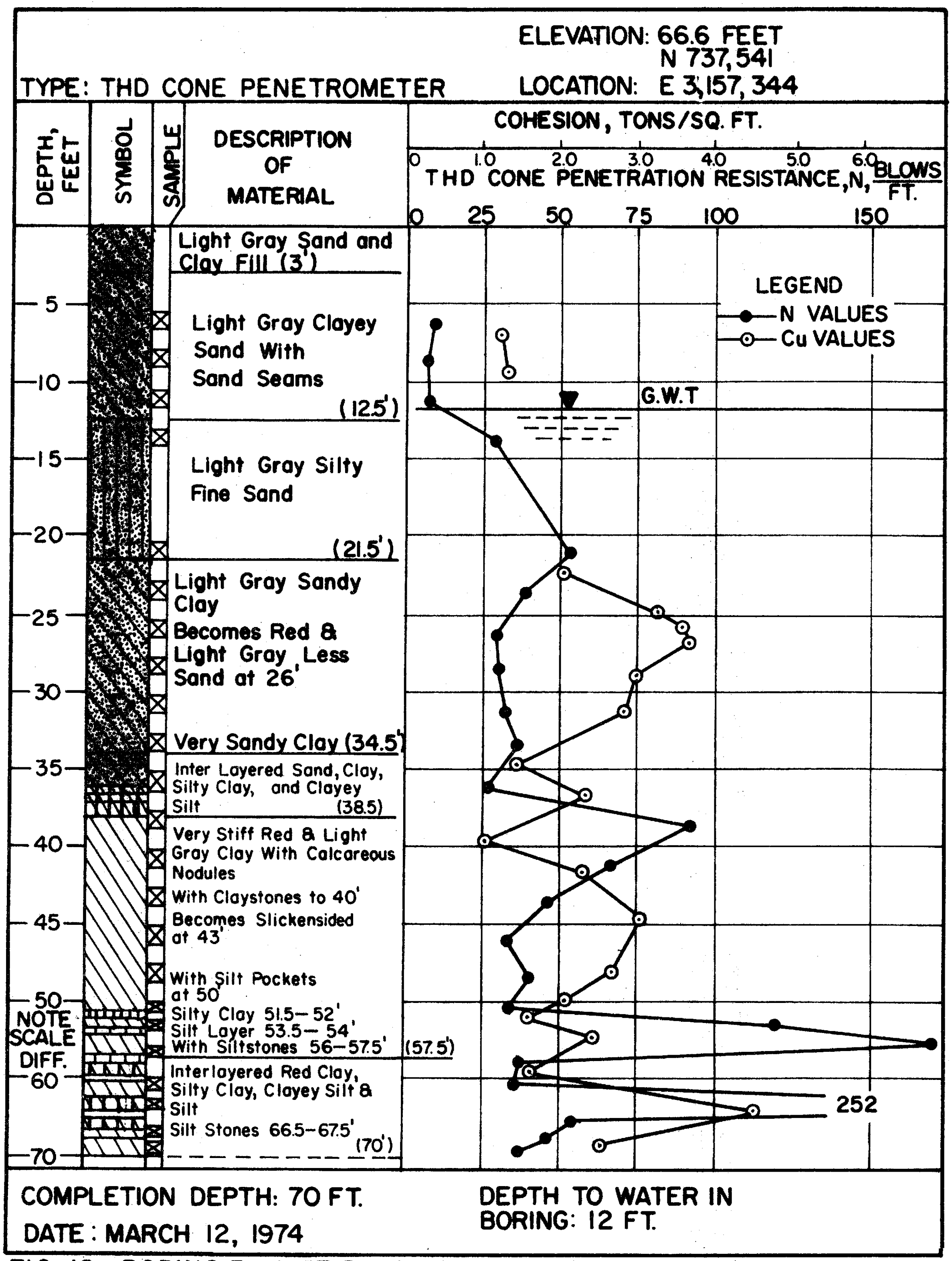


FIG. 18—BORING 3, SITE B, VARIATIONS OF RESISTANCE TO PENETRATION, N, AND TAT SHEAR STRENGTH WITH DEPTH

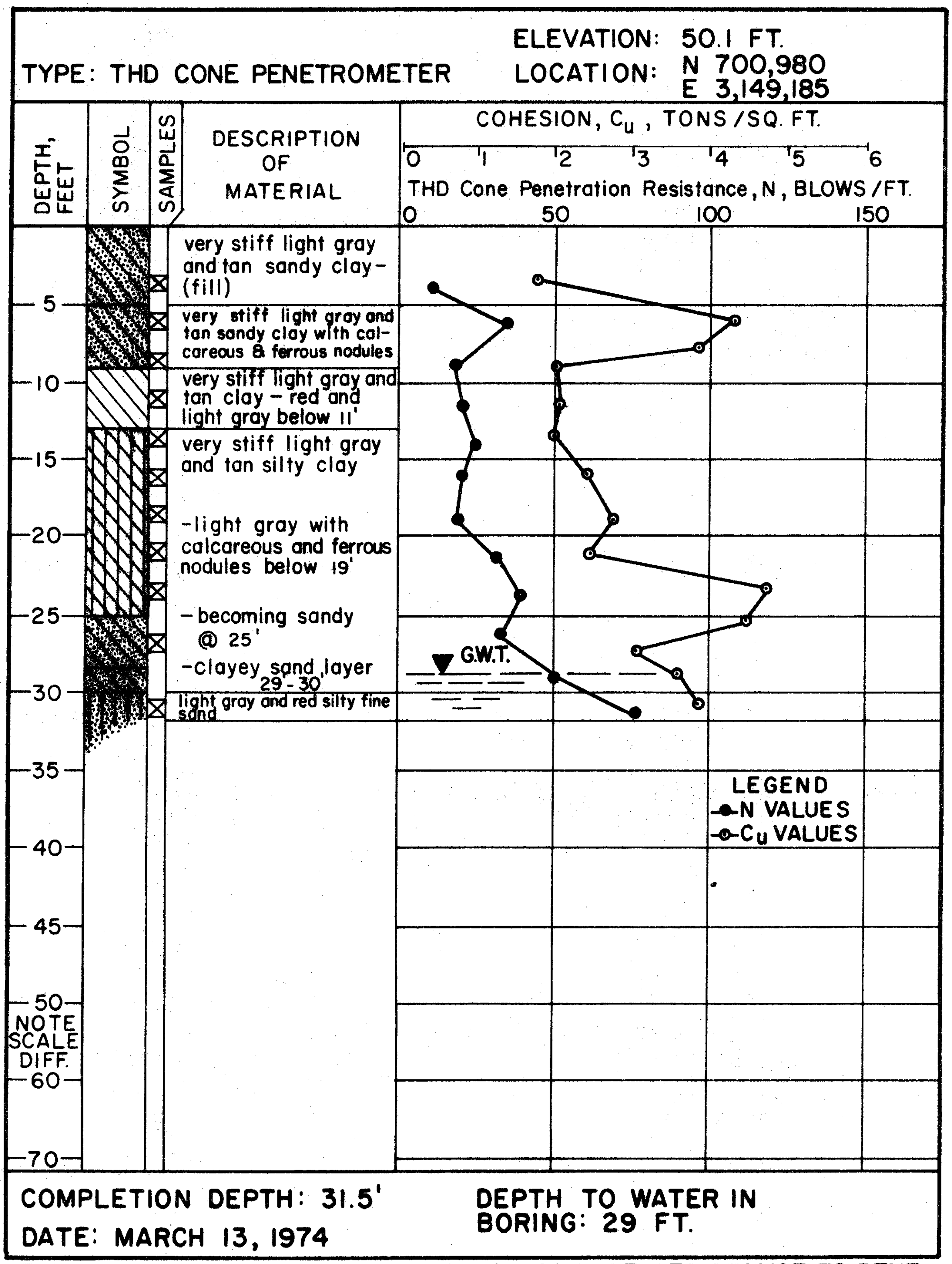


FIG. 19—BORING 4, SITE C, VARIATIONS OF RESISTANCE TO PENETRATION, N, AND TAT SHEAR STRENGTH WITH DEPTH

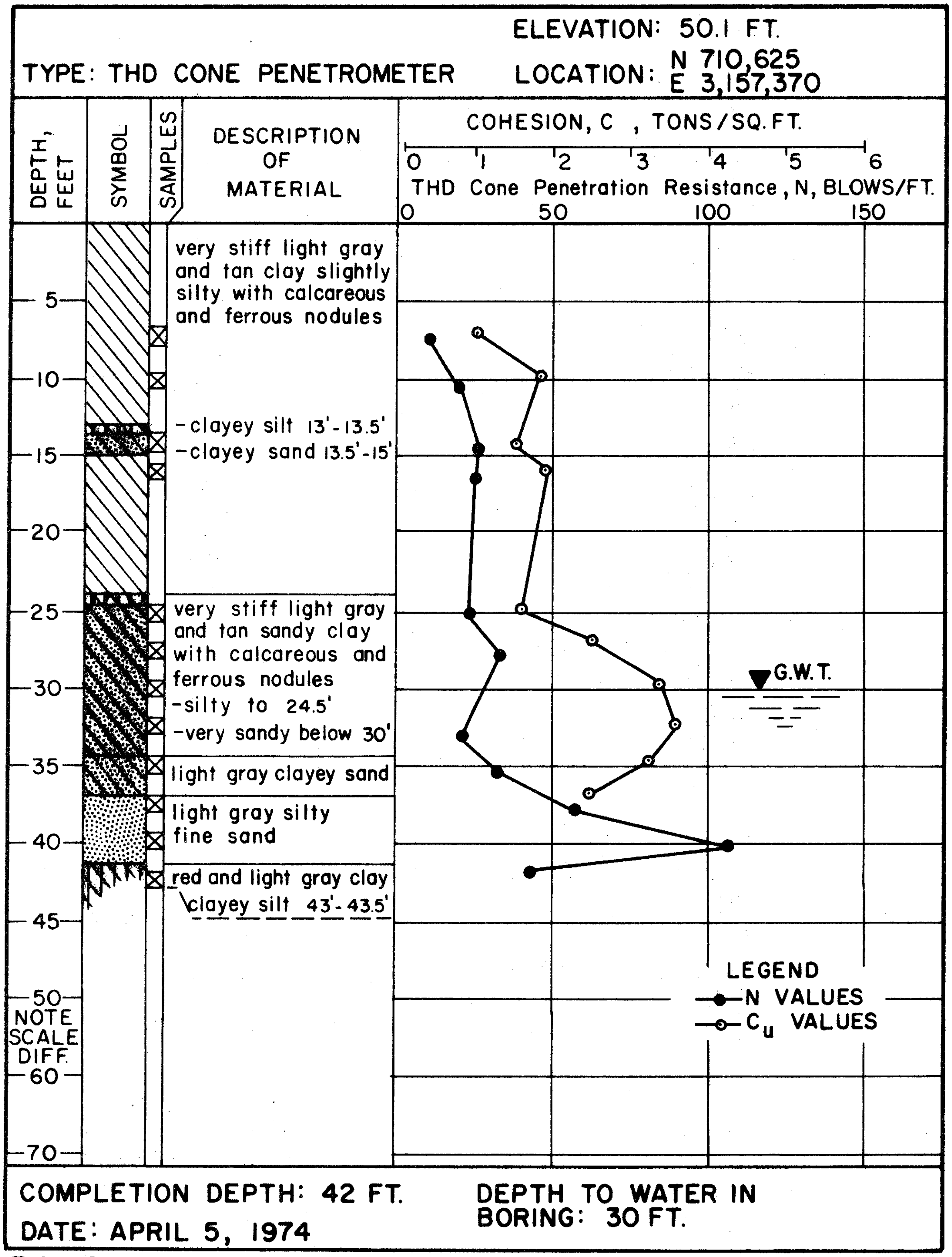


FIG. 20—BORING 7, SITE D, VARIATIONS OF RESISTANCE TO PENETRATION, N, AND TAT SHEAR STRENGTH WITH DEPTH

obtained in the field. However, when N reached 50 blows without achieving six inches penetration, the distance penetrated by the 50 blows was recorded. Since a numerical value for the full twelve inches was necessary for the purpose of the correlation, the following equation was used to obtain these values:

$$N = \frac{50}{D} \times 12 \text{ in.} \qquad (1)$$

where N = Penetration resistance, in blows per foot; and D = distance penetrated by 50 blows, in inches.

The values of the unconsolidated-undrained shear strength which is normally called cohesion, C_u, were computed for the Texas Triaxial Test as follows:

$$C_u = \left[\left(\frac{P_m}{A_c}\right) - \sigma_c\right] \times 0.5 \qquad (2)$$

where P_m = the maximum observed load, i.e., the sum of the vertical load induced by the confining pressure and the applied vertical load in tons; A_c = the corrected area in square feet; σ_c = the confining pressure in tons per square foot; and C_u = the cohesion in tons per square foot.

The results that are tabulated in Appendix III for the other two types of triaxial tests were obtained using the following equation:

$$C_u = \left(\frac{P_v}{A_c}\right) \times 0.5 \qquad (3)$$

where P_v = the applied vertical load in tons; A_c = the corrected area in square feet; and C_u = the cohesion in tons per square foot.

The difference between equation 2 and 3 is due to the initial state of stress upon confinement. In the case of the Texas Triaxial Test the initial state of stress is anisotropic. On the other hand, the initial state of stress for the other two triaxial tests is isotropic.

The results of the multi-stage type triaxial tests are presented in Figs. IV-1 through IV-20 in Appendix IV. These figures show the failure envelope for the unconsolidated-undrained tests.

ANALYSIS OF TEST RESULTS

Comparisons of resistance to penetration, N, and the unconsolidated-undrained shear strength, C_u, have been presented in Figs. 17 through 20. Since the relationship between these two parameters is not always constant, it is necessary to discuss the factors influencing each parameter before a correlation is attempted.

Factors Affecting Resistance to Penetration, N.--The magnitude of the N-value reflects the ease with which the THD cone penetrates the subsoil. As previously stated, the action of the cone was examined by recovering the subsoil with a push barrel. The examination of the soils recovered at sites B, C, and D, respectively, showed that the moving cone created a cavity. This action probably caused the soil to have both a lateral and an upward movement. The amount of these movements is probably dependent upon the soil type, degree of compactness, the overburden pressure, and the degree of saturation.

Desai (6) reported that the upward displacement of the subsoil will occur until a certain depth or surcharge pressure is reached which will no longer permit such displacement. Furthermore, at depths where the upward displacement becomes small, the lateral displacement will form an important part of the total displacement. Desai (6) therefore concluded that density, structure, depth and location of the groundwater table will have an effect on the resistance to penetration. The available data from this study have been analyzed to investigate the effect of the overburden pressure, dry unit weight, and the degree of saturation on the magnitude of the resistance to penetration N-value. In addition, it was observed while studying the data that the amount of sand in the soil influenced the magnitude of the N-value. This was especially true at site C below the 25-ft (7.625-m) depth, at site D below the 30-ft (9.15-m) depth, and at site B between the 21.5-ft (6.56-m) depth and the 34.5-ft (10.52-m) depth. A definite conclusion concerning the effect of any one of these properties (overburden pressure, dry unit weight, degree of saturation, or per cent sand content) is not possible because of the scatter of the results.

At site A high values of N, that is, greater than 100 blows per foot, were obtained below the 30-ft (9.15-m) depth. Although the clay below this depth has relatively high dry unit weights and contains 20% to 30% sand, these two factors alone may not explain the high N-values for the following reasons:

1. The sand is primarily calcium carbonate which is relatively soft compared to quartz sand; and
2. High dry unit weight is also found at the 7-ft (2.14-m) depth at the same site, and the N-value is only 34 blows per foot even though the degree of saturation is about the same.

Therefore, the contribution of these two factors to the high N-value is small. However, the cone penetration test was conducted below the groundwater table when the 30-ft (9.15-m) depth was reached, and this may explain the high N-values. Generally, at all sites an increase in the N-value was observed when the THD Cone Penetrometer Test was conducted below the groundwater table. A large portion of the driving energy is probably transmitted to the pore water and causes the N-value to increase. According to Sanglerat (20), in impervious, saturated cohesive soils below water table the resistance to penetration is mostly due to skin friction and the resistance of the pore water under sudden impact. Other researchers (6, 22) reported that friction was appreciable in saturated loose sands and all types of clay soils as well as in stratified deposits. However, the diameter of the cone used in these studies (6, 22) was either equal to or smaller than the drill pipe which was attached to the cone, whereas, the THD cone penetrometer has a larger diameter than the drill pipe to which it is attached. Also the side contact area is relatively small and the side friction is likely to be small compared to point resistance (see Fig. 1). Therefore, it would seem appropriate to correct the N-values for the influence of the water table. However, it was not possible to establish a correction for the N-value in this study because of the limited amount of data available.

Another factor that may cause high N-values is the nonhomogeneity or stratification of the soil. For example, at site B very high N-values, that is, equal to or greater than 100 blows per foot, were obtained at various depths. A possible reason for these high N-values is stratification as indicated in Table 4.

TABLE 4.--Effect of Soils Stratification on N-value

Depth below ground surface feet	N-value blows/foot	Description of material tested
38-39	89	Very stiff red and light gray clay with calcareous and ferrous nodules and claystone seams.
53-54	120	Very stiff red and light gray clay with nodules, silt layer and siltstone seams.
55.5-56.5	183	Very stiff red and light gray clay with nodules, silt layer and siltstone seams.
63-64	253	Interlayered red clayey silt, sandy silt, clay and sandstones.

It is apparent at this point that there are many factors which could affect the N-value. Jonson and Kavannagh (13) have summarized their findings by stating that the resistance to penetration is a function of the shearing resistance of the soil. Since the shearing resistance of a soil is a function of the physical properties of the soil, the findings of other researchers (6, 13, 22) and the findings presented from this study are consistent. The factors which affect the N-value are obviously inter-related, and it is difficult, if not impossible, to isolate a single, most important factor.

Factors Affecting Soil Shear Strength.--The quick test results that were obtained in this study indicate that most soils encountered at the four sites are primarily stiff to very stiff clays. Some of these clays are either fissured, as in the case of the clay at site A below the 30-ft (9.15-m) depth, or slickensided as in the case at site B, at various depths. The results of the laboratory tests used to determine the strength of these soils may not represent the actual strengths of the soils in situ. Apart from the test method used, the most important factor that influences shear strength is the soil structure. According to Hvorslev (11), the average strength may be subjected to considerable although slowly progressing change when the stress condition is altered. Laboratory tests will give low strengths when planes of failure in the test specimen follow joints or slickensides, and high shear strengths when planes of failure and joints intersect each other.

The results that were obtained by the Texas Triaxial Test will be compared in a relative way with the results obtained by the ASTM Triaxial Test which is currently used in most soil mechanics laboratories. When conducting the ASTM Triaxial Test, the soil is failed by increasing the vertical pressure while holding the confining pressure constant. The confining pressure causes all surfaces of the soil sample to be stressed equally. However, when conducting the Texas Triaxial Test, the confining pressure does not cause all surfaces of the soil sample to be stressed equally. In fact, it causes the soil sample to be extended because the vertical pressure is initially less than the confining pressure. It was observed during testing that the vertical pressure varied linearly with the confining pressure for a specific type of soil. However, when the same confining pressure was applied to a different type of soil, the magnitude of the vertical pressure changed.

During any quick triaxial testing the vertical pressure is increased rapidly enough that there is not sufficient time for water movement to occur as the soil sample is deformed. This is especially true for low permeable soils such as the clay soils used in this study. This type of loading was used to obtain the shear strength data in this study regardless of the type of triaxial test used. As mentioned previously, this test is called the unconsolidated-undrained triaxial or quick test. During a quick test the natural water content of a clay soil should not change. However, it was noted that for 18 of the soil samples tested in this study, the moisture content after the Texas Triaxial Test was completed was less than the initial value, indicating a loss of water during testing. A typical example showing change of water content can be found in Appendix III, site B, sample number 38. It was also observed, especially during testing of the silty clays, that the fines mixed with water tended to be squeezed out when the confining pressure was applied and when the vertical pressure was increased.

The undrained condition did not occur during the testing of these 18 specimens. Therefore, the Texas Triaxial Test does not always provide unconsolidated-undrained conditions.

Normal procedure for reducing the data from a multi-stage triaxial test is to plot the Mohr's circles representing the state of stress of failure for each confining pressure and then draw the failure envelope tangent to the Mohr's circles. A horizontal failure envelope indicates the existence of the $\phi = 0$ condition. This was demonstrated when the multi-stage Transmatic Test was performed on a highly saturated clay sample as shown in Fig. IV-3 in Appendix IV. Furthermore, the samples tested in the ASTM triaxial device, as shown in Figs. IV-10, IV-11, IV-13 and IV-16 of Appendix IV, respectively, show that the $\phi = 0$ condition existed. However, for all samples tested in the Texas Triaxial device as a multi-stage test, the $\phi = 0$ condition did not exist.

In general, the $\phi = 0$ condition does not occur for a partially saturated soil. For example, Fig. IV-15 in Appendix IV shows the results for a sample with 85% saturation tested in the Transmatic Triaxial device. These results demonstrate the expected Mohr failure envelope for a partially saturated soil. The deviator stress at failure is found to increase with increasing confining pressure. However, this increase becomes progressively smaller as the air in the voids is compressed and passes into solution and ceases when the stresses are large enough to cause full saturation (3).

It is evident at this point that the TAT allows partial drainage and does not duplicate the undrained condition. The magnitude of the shear strength obtained by this test is high when compared with the ASTM Triaxial results. This comparison is shown in Table III-5 of Appendix III. The reasons for the higher values of shear strength obtained by the TAT test are probably due to a combination of the following:

1. The Texas Triaxial device has a membrane which is four times as thick as the membrane used with the ASTM Triaxial device. The thicker membrane induces extra compressive strength which is a function of the stiffness of the membrane used.
2. The Texas Triaxial device allows some soils to lose water during testing. This is particularly true for the CL-Sa and CL-Si soils. The loss of water will cause a decrease in water content and a corresponding increase in strength.
3. The friction which occurs between the upper cap and the membrane causes the observed proving ring reading which is a function of the confining pressure to be higher. This induced strength is not a part of the soil shear strength.

A separate correction for each of the above mentioned factors is beyond the scope of this study, but a cumulative correction based on the results of the ASTM Triaxial Test can be made. During this study, a limited number of paired samples were tested by the TAT and the ASTM methods. Specifically, 3 pairs of CH-H, 5 pairs of

CL-Si, and 6 pairs of CL-Sa samples were tested. Based upon this limited number of tests, a tentative relationship between the shear strengths obtained by the TAT and ASTM test methods was developed and is shown in Fig. 21. The data are also tabulated in Table III-5, Appendix III. This relationship was obtained using a least square fit of the data and may be expressed in equation form as:

$$C_{u_{ST}} = 0.60\, C_{u_{TAT}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

where $C_{u_{ST}}$ = shear strength obtained by the ASTM Triaxial Test;
and $C_{u_{TAT}}$ = shear strength obtained by the Texas Triaxial Test.
It is important to note that the data were obtained by testing soil samples taken from the same boring and essentially having the same physical properties.

In summary, it has been shown that the magnitude of C_u is affected by many factors. One very important factor is the secondary structure which exists within a soil sample. The effect of the physical properties of the soils tested on the magnitude of the shear strength obtained has not been presented or discussed because it is well documented in the literature. The magnitude of the unconsolidated-undrained shear strength that was obtained in this study was definitely a function of the type of triaxial test. The Texas Triaxial Test gave higher shear strength values than those obtained by the ASTM Triaxial Test. The relationship between these two values is presented in Eq. 4.

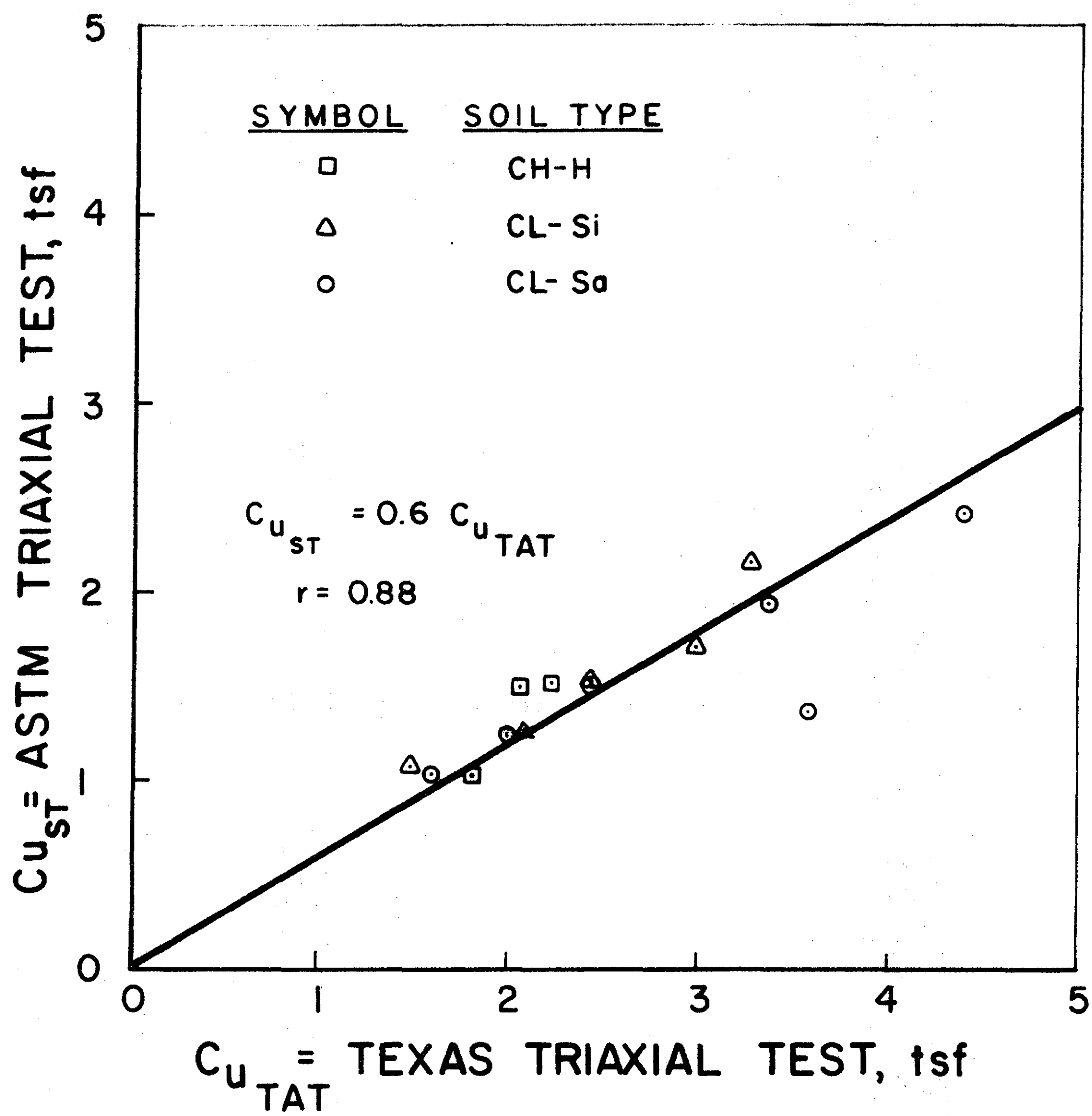


FIG. 21 — RELATIONSHIP BETWEEN TEXAS TRIAXIAL AND ASTM TRIAXIAL SHEAR STRENGTH

($1 tsf = 9.58 \times 10^4\ N/m^2$)

Correlation of Resistance to Penetration, N, with Soil Shear Strength, C_u.---As previously indicated in Figs. 17 through 20, when both N and C_u are plotted versus depth, a relatively linear relationship exists between these two parameters. This relationship is not as evident in Fig. 18 for the case of the erratic soil at site B.

If a linear relationship does exist between N and C_u, the best way to correlate these two parameters is to evaluate the constant of proportionality as given in the following equation:

$$C_u = KN \qquad (5)$$

where K = constant of proportionality; N = THD cone resistance to penetration in blows per foot; and C_u = unconsolidated-undrained shear strength in tons per square foot.

A statistical procedure was used to evaluate the constant of proportionality. This was accomplished as follows:

1. The soils were placed into groups of similar properties.
2. Using all available data, plots of C_u versus N were made for each group.
3. A best fit linear curve was established using the least square method.

The first step in the correlation was to place the soils into groups of similar properties. The Unified Soil Classification System was employed to group the soils initially, as stated in the objective. However, analysis of the data revealed that all CH materials could not be placed into one group. The penetration resistance for CH materials that contain secondary structure such as joints, fissures, or slickensides in this study were generally in excess of 100 blows per foot. Also, as mentioned previously, the shear strength values obtained in the laboratory for these soils do not necessarily represent the strength of the soil in situ. On the other hand, the CH materials that did not contain secondary structure (herein called homogeneous CH) had N-values that did not in general exceed 50 blows per foot. In addition the shear strength values determined in the laboratory are considered an acceptable representation of the in situ soil strength. Therefore, the CH materials were divided into two subgroups. These subgroups are the homogeneous CH soils and the CH soils with secondary structure. As far as the CL materials group was concerned, high N-values were associated with either stratified CL soils or with the amount of sand present in the sample. Following the practice of grouping the soils according to similar properties it was considered appropriate to divide the CL materials into three subgroups. These are the silty CL soils, the sandy CL soils, and the stratified CL soils. The SC soils were not divided into subgroups since only a limited amount of data were obtained for SC soils in this study.

The second step in the correlation was to plot C_u versus N for each subgroup. C_u values obtained in the laboratory from both the Texas Triaxial Test (TAT) and the ASTM Triaxial Test (ST) were used. The N-values were determined by doubling the number of blows for the 6 inches (152.4 mm) of penetration that occurred in close proximity of the soil sample used to obtain the C_u value. The reasons for determining the N-values in this manner are as follows:

1. Experience has shown that, in normally consolidated clay, the number of blows required for the first and second 6 inches (152.4 mm) are generally the same (5).
2. The sample used to determine the C_u value represents approximately six inches (152.4 mm) of the soil tested by the THD Cone Penetrometer Test.
3. The N-value obtained for a 6-in. (152.4 mm) penetration can be realistically compared with the C_u value obtained for a soil sample taken from the same depth in a soil boring.

All data for the homogeneous CH soils are summarized in Table 5 and plotted in Fig. 22. Also, in Fig. 22 the curves representing the least square fit for both the Texas Triaxial Test and the ASTM Triaxial Test data are shown. The scatter in the data is probably due to a combination of the factors affecting both N and C_u as discussed previously. The slope of the curves represent the constant of proportionality, K, as presented in Eq. 5. The equation for the TAT data may be written as follows:

$$C_{u_{TAT}} = 0.11\ N \qquad (6)$$

Equation 6 may be used to predict the shear strength based on the Texas Triaxial Test if the resistance to penetration, N-value, is available, and provided that the soil tested is a homogeneous CH soil. In order to predict the shear strength based on the ASTM

TABLE 5.--Homogeneous CH Soils			
Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
*	32	4.50	--
*	32	--	3.43
A-3	36	4.54	--
*	32	4.28	--
*	32	--	2.00
A-4	32	3.17	--
*	32	1.14	--
*	30	--	2.43
*	22	1.81	--
A-8	22	2.82	--
*	22	4.71	--
*	22	--	1.58
*	22	2.27	--
A-9	18	2.32	--
*	18	3.01	--
*	20	--	1.75
*	20	1.92	--
*	24	--	1.78
*	20	1.95	--
*	18	--	1.19

(Continued)

TABLE 5-CONTINUED.--Homogeneous CH Soils			
Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
A-13	12	1.47	--
A-15	18	--	1.51
A-16	18	2.21	--
*	28	1.50	--
*	28	--	0.93
A-19	14	1.45	--
*	14	--	0.82
*	14	1.94	--
A-22	12	1.25	--
*	12	--	0.38
*	12	1.27	--
*	12	1.24	--
*	20	1.50	--
*	18	--	0.90
A-23	12	0.74	--
B-30	28	2.68	--
B-39	32	--	1.33
B-40	32	2.47	--
B-43	30	--	1.62
C-1	10	1.78	--
C-6	16	1.99	--

(Continued)

TABLE 5-CONTINUED.--Homogeneous CH Soils

Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
C-8	20	--	1.63
C-9	18	2.05	--
C-10	18	--	1.50
D-1	10	1.03	--
D-2	22	--	1.03
D-3	18	1.80	--
D-7	24	1.92	--

*From previous THD research studies

Note: 1 ft - .305 m; 1 TSF = 9.58×10^4 N/m^2

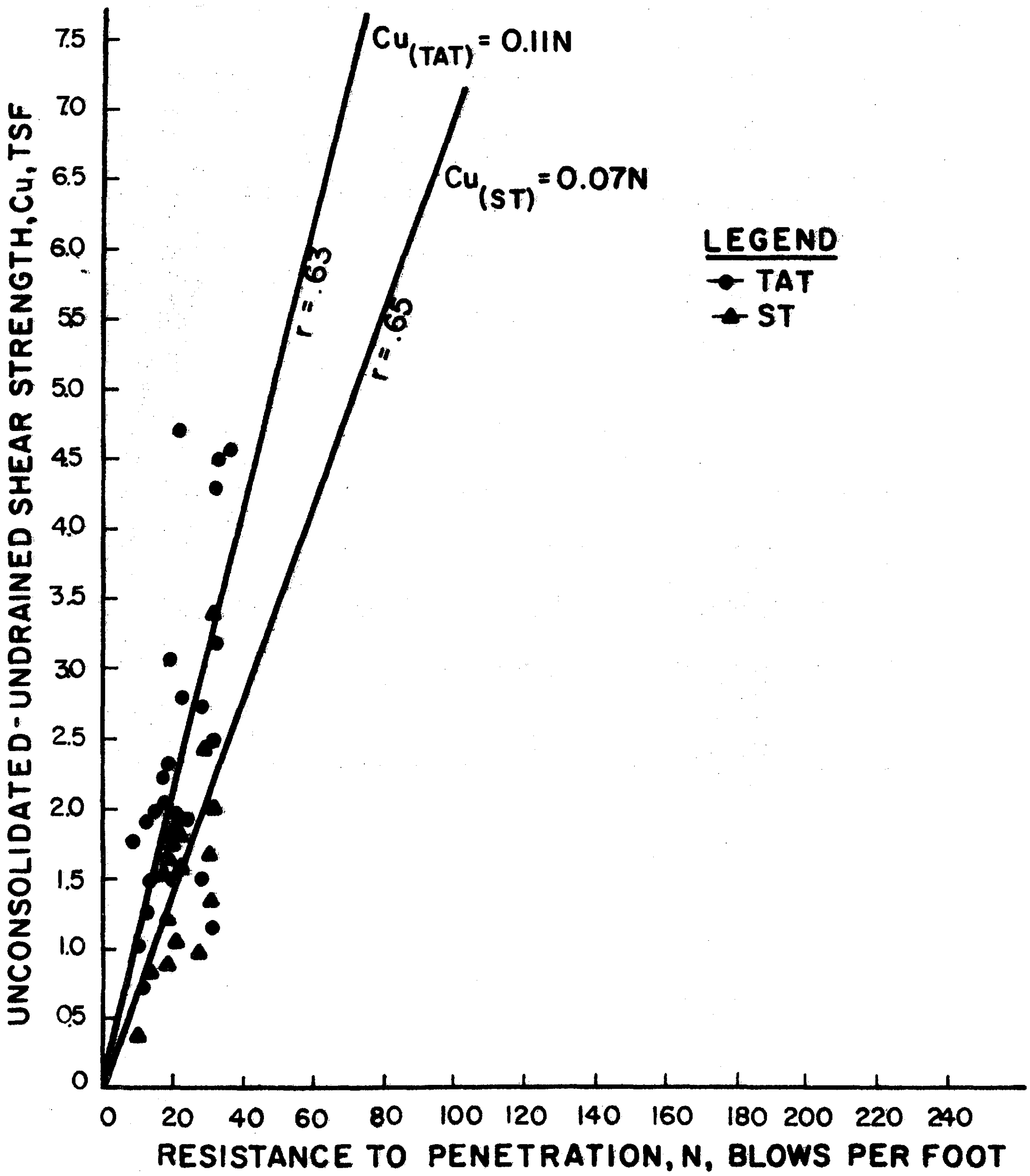


FIG. 22 CORRELATION BETWEEN UNCONSOLIDATED UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR HOMOGENEOUS CH SOILS ($1 TSF = 9.58 \times 10^4 N/m^2$, 1ft. = .305m.)

Triaxial Test a constant of proportionality of 0.07 should be used, and eq. 5 becomes:

$$C_{u_{ST}} = 0.07\ N \quad \ldots\ldots (7)$$

All of the data for the CH soils with secondary structure are presented in Table 6 and plotted in Fig. 23. There is considerably more scatter of the data in this case when compared with the homogeneous CH soils. This is due to the difficulties associated with determining the shear strength for clays that have a secondary structure. Also, as noted previously, the majority of the N-values were obtained below water table. From a practical point of view, it may not be proper to fit a curve to these data. However, if a best fit curve is used for the Texas Triaxial Test data the resulting equation is:

$$C_{u_{TAT}} = 0.02\ N \quad \ldots\ldots (8)$$

The best fit curve for the ASTM Triaxial Test data is represented by the following equation:

$$C_{u_{ST}} = 0.018\ N \quad \ldots\ldots (9)$$

These curves and corresponding equations were also obtained using the least square method.

The data for the silty CL soils are given in Table 7 and plotted in Fig. 24. The silty CL soils are those clays which contain less than 20% of material retained on the No. 200 sieve and do not contain sand or silt seams. The choice of less than 20%

TABLE 6.--CH Soils With Secondary Structure			
Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
A-24	104	--	0.55
A-25	104	0.44	--
*	104	--	2.09
A-26	134	3.99	--
A-27	172	3.06	--
*	120	--	1.70
A-28	150	3.52	--
A-29	150	--	2.35
*	200	--	3.52
*	200	3.55	--
*	184	--	2.95
A-31	184	2.71	--
*	155	--	2.96
*	126	4.52	--
*	126	--	4.50
A-30	184	2.93	--
A-32	240	4.04	--
A-33	184	2.42	--
*	184	--	3.45
*	212	--	2.80

(Continued)

TABLE 6-CONTINUED.--CH Soils With Secondary Structure			
Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
*	200	3.63	--
*	153	4.34	--
*	134	--	1.66
*	134	4.24	--
*	134	--	4.40
*	134	--	3.25
*	121	4.35	--
*	108	--	3.70
B-22	118	2.06	--
B-23	70	2.31	--
B-25	46	--	1.67
B-27	40	3.06	--
B-37	120	2.44	--
B-38	184	2.27	--

*From previous THD research studies

Note: 1 ft - .305 m; 1 TSF = 9.58 x 10^4 N/m^2

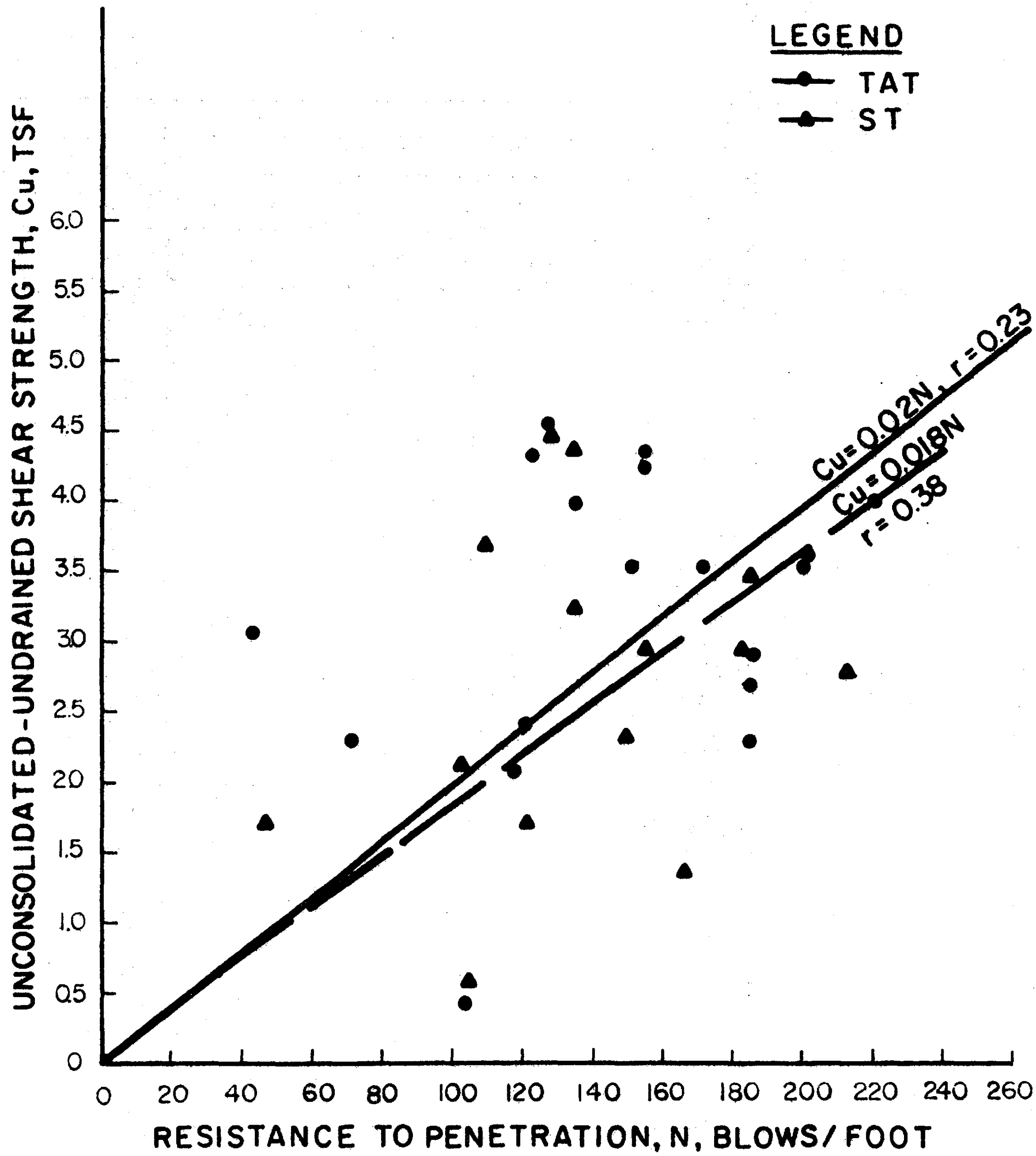


FIG. 23 CORRELATION BETWEEN UNCONSOLIDATED UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR CH MATERIAL WITH SECONDARY STRUCTURE
(1TSF = 9.58×10^4 N/m^2, 1ft. = .305m.)

TABLE 7.--Silty CL Soils			
Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
A-12	24	2.31	--
A-14	28	--	0.98
B-8	28	--	2.17
B-9	32	3.27	--
B-11	28	3.67	--
B-12	32	--	1.71
B-13	28	2.99	--
B-15	26	2.82	--
B-16	24	--	1.08
B-19	18	2.36	--
B-33	28	2.09	--
C-16	22	2.41	--
C-22	30	2.42	--
C 24	32	--	1.53
C-24	38	4.76	--
*	18	--	0.75
*	12	--	0.45

*From previous THD research studies

Note: 1 ft = .305 m; 1 TSF = 9.58 x 10^4 N/m^2

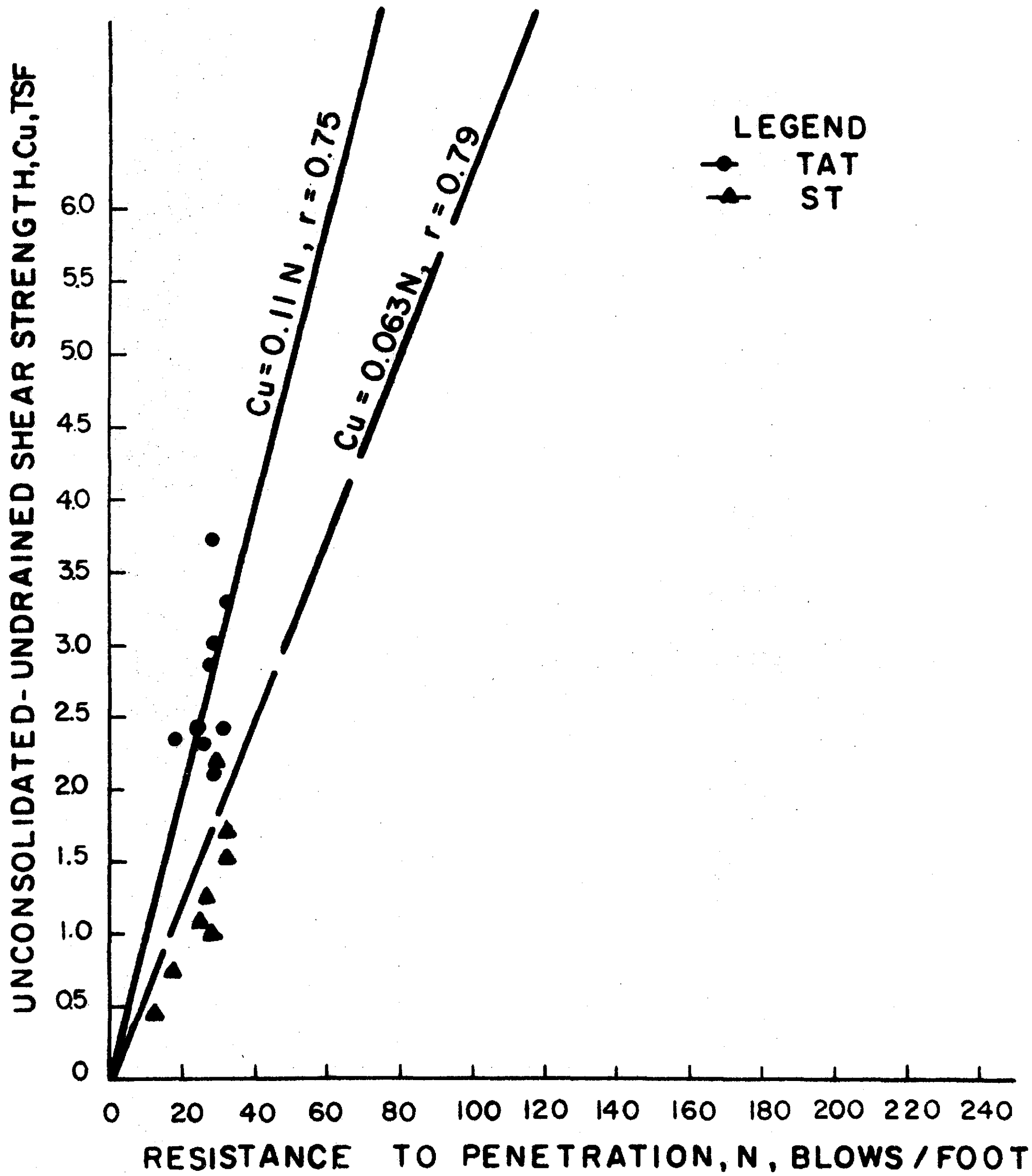


FIG. 24 CORRELATION BETWEEN UNCONSOLIDATED UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR CL (SILTY) SOILS (1 TSF = 9.58 x 10^4 N/ m^2; 1 ft. = .305 m.)

retained on the No. 200 sieve is based on the data presently available from this study. This percentage may change when more data are available. As shown in Fig. 24 a fairly good linear relationship exists between C_u and N for the silty CL clays. The linear relationship is better for the ASTM Triaxial Test data. The appropriate equations obtained using the least square method for the silty CL soils are as follows:

$$C_{u_{TAT}} = 0.11\ N \qquad (10)$$

$$C_{u_{ST}} = 0.063\ N \qquad (11)$$

The data for the sandy CL soils are given in Table 8. Fig. 25 presents the results of the correlation for the sandy CL soils. The sandy CL soils are those clays that contain more than 20% of material retained on the No. 200 sieve and do not contain sand or silt seams. The appropriate equations for the data available in this subgroup are as follows:

$$C_{u_{TAT}} = 0.095\ N \qquad (12)$$

$$C_{u_{ST}} = 0.053\ N \qquad (13)$$

It should be noted that the correlation for the sandy CL soils is not as good as the correlation for the silty CL soils; that is, there is more scatter in the data for the sandy CL soils. Also, for practical purposes, the constant of proportionality is essentially the same in both cases. However, the correlation for both the silty and the sandy CL soils has been presented because the

TABLE 8.--Sandy CL Soils			
Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
B-6	26	2.03	--
B-10	30	3.60	--
C-2	40	--	2.43
C-3	40	4.38	--
C-5	34	3.86	--
C-12	24	1.98	--
C-13	24	--	1.24
C-18	22	--	1.50
C-19	18	2.72	--
C-30	30	4.48	--
C-32	44	3.04	--
C-33	44	--	2.19
D-9	22	1.59	--
D-10	22	--	1.05
D-11	32	2.50	--
D-13	32	--	1.95
D-14	26	3.36	--
D-17	22	3.56	--
D-19	28	--	1.39
D-24	46	2.47	--

Note: 1 ft = .305 m; 1 TSF = 9.58×10^4 N/m^2

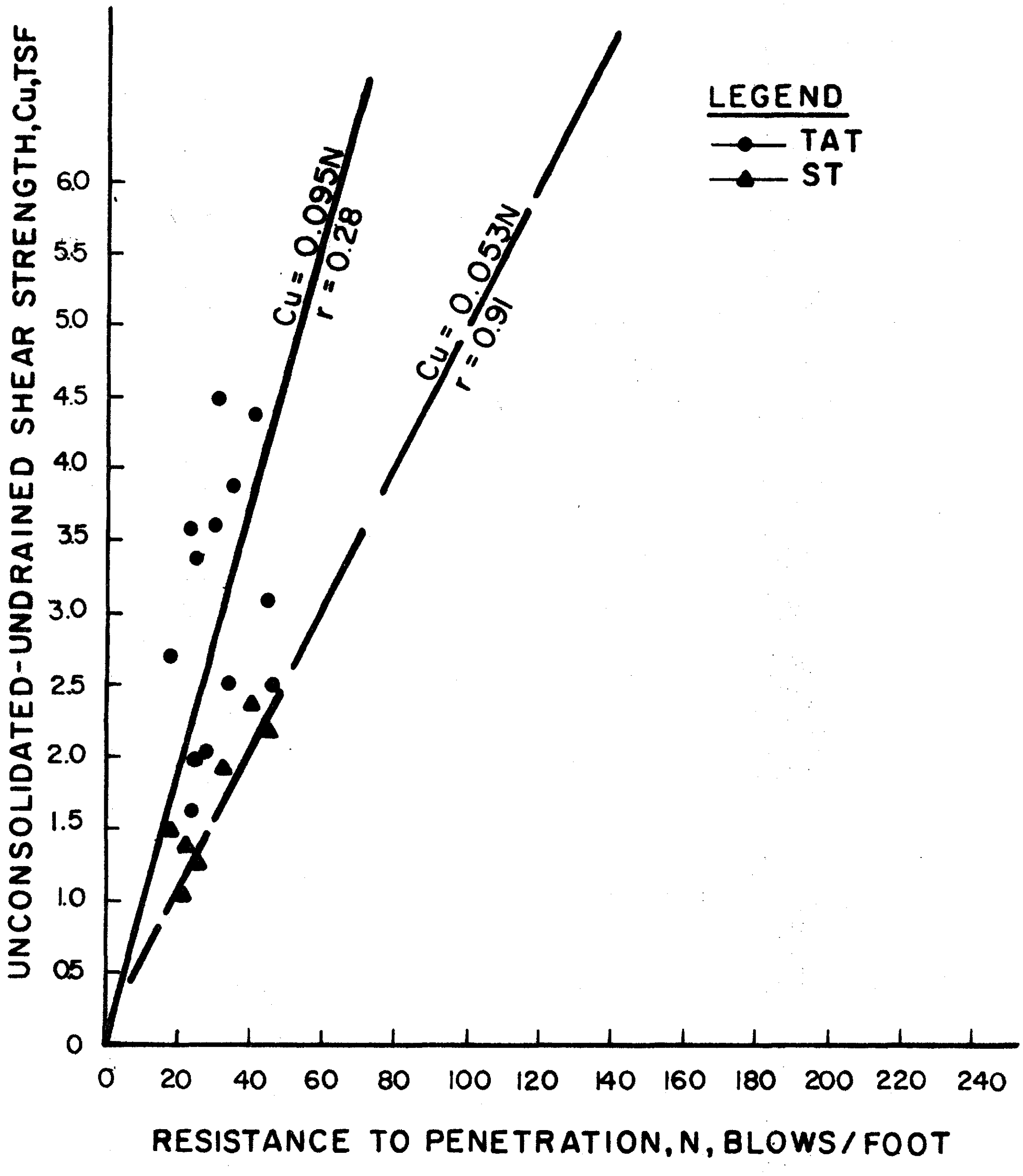


FIG-25 CORRELATION BETWEEN UNCONSOLIDATED UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR CL (SANDY) SOILS ($TSF = 9.58 \times 10^4\ N/m^2$, 1 ft. = .305 m.)

correlation for the silty CL soils is better, and when additional data are made available for the sandy CL soils, the constant of proportionality may change.

The data for the stratified CL soils are given in Table 9. The stratified CL soils are those clays that contain seams and/or layers of silt or sand.

TABLE 9.--Stratified CL Soils

Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
B-18	24	1.48	--
B-21	64	1.04	--
B-32	40	--	1.25
B-37	40	1.58	--
B-41	30	1.25	--
B-42	42	1.62	--

Note: 1 ft = .305 m; 1 TSF = 9.58 x 10^4 N/m^2

No attempt was made to correlate these data because of the limited amount of data available. If the data in Table 9 are plotted, the scatter is significant.

The data for the SC soils are given in Table 10. The same situation exists for the SC soils as for the stratified CL soils. Therefore, a correlation of these data was not attempted.

TABLE 10.--SC Soils			
Site and sample number	N-value blows per foot	C_u values, TSF	
		TAT	ST
B-1	12	1.23	--
B-2	8	1.31	--
B-3	8	1.09	--
B-4	8	--	0.98
C-34	54	3.55	--
D-6	24	1.51	--
D-23	34	--	2.03
Note: 1 ft = .305 m; 1 TSF = 9.58×10^4 N/m^2			

Equations 7, 11, and 13 are applicable with N-values obtained by the THD Cone Penetrometer Test. However, Touma and Reese (30) have developed a relationship between the N-values obtained by the THD Cone Penetrometer Test and N-values obtained by the Standard Penetration Test (SPT). For clay soils the relationship is:

$$N_{SPT} = 0.7\ N_{THD} \tag{14}$$

By combining Eqs. 7, 11, and 13 with Eq. 14 the following equations are obtained:

$$C_{u_{ST}} = 0.1\ N_{SPT} \quad \text{(homogeneous CH soils)} \tag{15}$$

$$C_{u_{ST}} = 0.09\ N_{SPT} \quad \text{(silty CL soils)} \tag{16}$$

$$C_{u_{ST}} = 0.076\ N_{SPT} \quad \text{(sandy CL soils)} \tag{17}$$

Equations 15, 16, and 17 are shown graphically in Fig. 26.

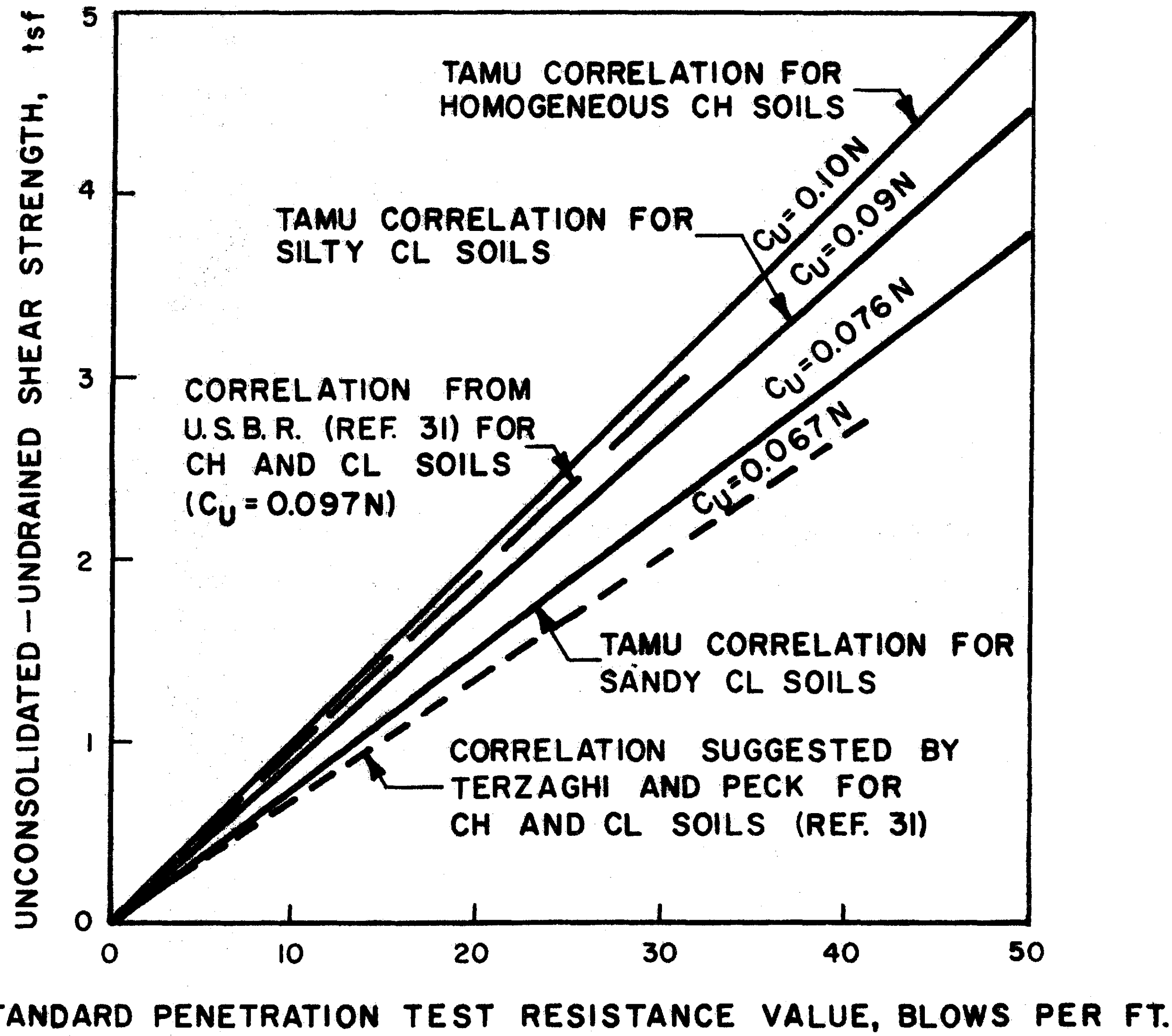


FIG. 26—RELATIONSHIP BETWEEN UNCONSOLIDATED—UNDRAINED SHEAR STRENGTH AND THE STANDARD PENETRATION TEST RESISTANCE VALUE (1 psi = 6.9 KN/M^2)

Also shown in Fig. 26 are a correlation curve developed by U.S.B.R. (31) and another curve given by Terzaghi and Peck, both curves being for CH and CL soils. The lines representing Eqs. 15, 16, and 17 are in good agreement with the correlations of the U.S.B.R. and Terzaghi and Peck. Equations 15, 16, and 17 were developed so that the results of this study can be used with N-values obtained by the Standard Penetration Test. However, it is not recommended that these results be used in geographical areas where comparative studies have not been made.

In summary, the soils investigated in this study were divided into groups according to similar behavior. It has been shown that a linear relationship exists between THD cone penetrometer N-value and C_u values obtained by either the Texas Triaxial Test or the ASTM Triaxial Test for several of the soil groups. Using the least square method K values were determined for use in Eq. 5. In addition, equations were developed which relate the unconsolidated-undrained shear strength, $C_{u_{ST}}$, to the Standard Penetration Test resistance value, N_{SPT}, for homogeneous CH, silty CL, and sandy CL soils. The quick shear strength for these three soil groups can be predicted using the N-value obtained by either the THD Cone Penetrometer or the Standard Penetration Test.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions--Improved correlations have been developed between the Texas Highway Department Cone Penetrometer Test N-values and unconsolidated-undrained shear strength for a group of cohesive soils. The soil shear strength used in the correlation was determined by both the Texas Triaxial Test and the ASTM Triaxial Test. It was necessary to group the soils tested into six subgroups based on similar behavior. The CH soils were subgrouped into homogeneous CH soils and CH soils with secondary structure. The CL soils were subgrouped into silty CL soils, sandy CL soils and stratified CL soils. The SC soils were not subgrouped. It was not possible to develop a correlation for either the SC soils or the stratified CL soils because of lack of sufficient data. Based on the data available for this study, the following conclusions are made:

1. The shear strengths determined by the Texas Triaxial Test were higher than the shear strengths determined by the ASTM Triaxial Test on identical samples. A linear relationship exists between these shear strengths as follows:

$$c_{u_{ST}} = 0.60\, c_{u_{TAT}}$$

2. (a) The correlations of the Texas Triaxial Test (TAT) shear strength with the Texas Highway Department

cone penetrometer N-value show that the following equations can be used to predict this shear strength:

$C_{u_{TAT}} = 0.11\ N$ - Homogeneous CH soils

$C_{u_{TAT}} = 0.02\ N$ - CH soils with secondary structure

$C_{u_{TAT}} = 0.10\ N$ - Silty CL soils

$C_{u_{TAT}} = 0.095\ N$ - Sandy CL soils

(b) The ASTM Triaxial Test (ST) shear strength can also be predicted from the THD cone penetrometer N-value. The equations that can be used to predict this shear strength are as follows:

$C_{u_{ST}} = 0.07\ N$ - Homogeneous CH soils

$C_{u_{ST}} = 0.018\ N$ - CH soils with secondary structure

$C_{u_{ST}} = 0.063\ N$ - Silty CL soils

$C_{u_{ST}} = 0.053\ N$ - Sandy CL soils

(c) A reasonably good correlation exists for the homogeneous CH soils and the silty CL soils based on the smaller amount of scatter observed in the results for these two types of soils.

3. Equations were developed which can be used to predict the unconsolidated-undrained shear strength from the Standard Penetration Test. The results obtained by Touma and Reese (30) were used to develop the following equations:

$C_{u_{ST}} = 0.1\ N_{SPT}$ (homogeneous CH soils)

$C_{u_{ST}} = 0.09\ N_{SPT}$ (silty Cl soils)

$C_{u_{ST}} = 0.076\ N_{SPT}$ (sandy CL soils)

Recommendations.---In view of the limited amount of data available for use in this study, the results should not be indiscriminately applied for all soil types investigated but can be applied for soils that have similar physical and engineering properties. Additional research is recommended on a wide variety of cohesive soils particularly from different geological formations. The following specific recommendations are made for future research:

1. To further ascertain the validity of the correlations developed in this study between the N-value and the quick shear strength for CH soils and CL soils, additional tests should be made on a larger number of soil samples.
2. To qualitatively use the THD Cone Penetrometer Test, which is a quick and simple test, it is necesaary to conduct additional study concerning the factors that affect the magnitude of the resistance to penetrations. Additional study of the effect of the groundwater table on the magnitude of the N-values is particularly important.
3. Bridges are commonly constructed over a river channel or over a flood plain. The natural soil deposit of a river channel according to Terzaghi (27) is likely to be distinguished by important and erratic variations, such as

stratified soil. This type of soil is a "problem" soil. In flood plains, hair cracks, joints or slickenside commonly occur. A soil that contains secondary structure is also a "problem" soil. Further study concerning these two types of soils is also needed.

4. Correlation between resistance to penetration and the quick shear strength of both the SC soils and the sandy CL soils is needed in order to establish an accurate mathematical model that can be used for these two soils.
5. Additional study is needed to determine what modifications are required to obtain shear strengths from the Texas Triaxial Test which are in closer agreement with the strengths obtained from the ASTM Triaxial Test. The study should include the effects of sample disturbance, the effects of the confining membrane, and the effects of friction between the membrane and the upper cap.

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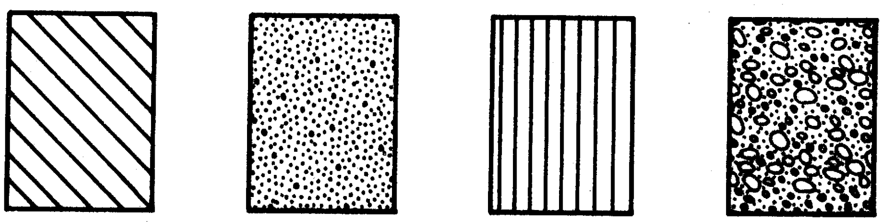
APPENDIX II.--DEFINITIONS AND NOTATIONS

The symbols and terms used on boring logs are:

SOIL TYPE

(shown in symbol column)

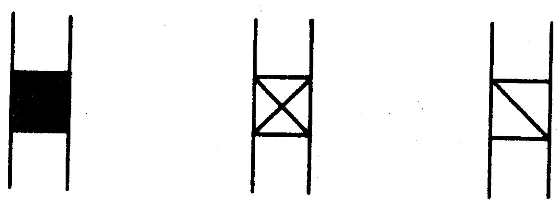


CLAY SAND SILT FILL

Predominant type shown heavy

SAMPLER TYPES

(shown in samples column)



PUSH BARREL

THD CONE PENETROMETER

NO RECOVERY

Consistency is rated according to shearing strength, as indicated by the Standard Triaxial Test:

Description Term	Cohesion, ton/sq ft
Firm	0.25 to 0.50
Stiff	0.50 to 1.00
Very Stiff	1.00 to 2.00
Hard	2.00 and higher

Seam - 1/8 in. (3.18 mm) to 3 in. (76.2 mm) thick

Layer - greater than 3 in. (76.2 mm) thick

Fissured - containing shrinkage cracks, frequently filled with fine sand or silt

Calcareous - containing appreciable quantities of calcium carbonate

Slickensided - having inclined planes of weakness that are slick and glossy in appearance

Interlayered - composed of alternate layers of different soil types; also called stratified

N - the number of blows required to drive the THD cone penetrometer one foot; also noted as N_{THD}

N^* - the number of blows required to drive the THD cone penetrometer six inches.

N_{SPT} - the number of blows required to drive the standard split spoon one foot

ϕ - the angle of shearing resistance

D - the distance, in inches, of the THD cone penetrated by 50 blows.

P_m - the sum of the vertical load induced by the confining pressure and the applied vertical load during the Texas Triaxial Test in tons

A_c - the corrected area in square feet

σ_c - the confining pressure in tons per square foot

C_u - the cohesion in tons per square foot

P_v - the applied vertical load in tons

$C_{u_{TAT}}$ - the Standard Triaxial shear strength during a "quick" test, in tons per square foot

$C_{u_{TAT}}$ - the Texas Triaxial shear strength during a "quick" test, in tons per square foot

K - constant of proportionality = C_u/N

Ai - regression coefficient

LL - liquid limit in percent; also noted as W_L

PI - plasticity index in percent; also noted as I_P

-200 - percent of the materials that pass the No. 200 sieve

WC - percent water content

UDW - unit dry weight in pounds per cubic foot

S - percent saturation

P_o - effective overburden pressure

APPENDIX III

SUMMARY OF TEST DATA

The following notations were used to identify soil subgroups:

- H - Homogeneous CH soils
- W - CH soils with secondary structure
- Si - Silty CL soils
- Sa - Sandy CL soils
- S - Stratified CL soils

TABLE III-1.--Summary of Tests Results

			3	4	8	9	12	13	14
SAMPLE NUMBER			3	4	8	9	12	13	14
PENETRATION, FT			6.5-7	8.5-9	11.5-12	14-14.5	16-16.5	21-21.5	21.5-22
PENETRATION RESISTANCE, N*			36	32	22	18	24	12	28
CLASSIFICATION TESTS	Liquid Limit, %		71.7	70.6	66.2	62.4	36.8	57.9	48.4
	Plastic Limit, %		23.5	24.1	23.9	23.4	18.2	23.1	19.1
	Plasticity Index, %		48.2	46.5	42.3	39.0	18.6	34.8	29.3
	Percent Passing No. 200 Sieve		97.4	97.6	99.0	98.9	98.0	99.6	99.1
	Unified Classification		CH	CH	CH	CH	CL	CH	CL
	Subgroup		H	H	H	H	Si	H	Si
TRIAXIAL COMPRESSION	Type of Test		1	1	1	1	1	1	3
	WATER CONTENT	Initial	22.5	23.3	24.0	25.5	22.0	25.4	19.9
		Final	22.5	23.3	23.9	23.9	22.0	27.0	26.5
	Unit Dry Wt. lb/ft^3		105.2	99.9	101.7	99.1	104.0	100.5	103.2
	Cohesion, ton/ft^2		4.54	3.17	2.82	2.32	2.31	1.47	0.98
	Lateral Pressure, PSI		6.0	8.0	10.0	13.0	14.0	19.0	19.0
OTHER SOIL PROPERTIES	Specific Gravity		2.72	2.72	2.72	2.72	2.75	2.75	2.75
	Percent Saturation		100.	91.0	98.0	96.0	95.0	100.	96.0

<u>Legend and Notes</u>

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 1a, ab

SITE A-Little Brazos at State Highway 21
Brazos County

TABLE III-1-CONTINUED.--Summary of Tests Results

			15	2nd stage	3rd stage	16	2nd stage	3rd stage	
SAMPLE NUMBER			15	2nd stage	3rd stage	16	2nd stage	3rd stage	
PENETRATION, FT			19-19.5			19.5-20			
PENETRATION RESISTANCE, N*			18			18			
CLASSIFICATION TESTS	Liquid Limit, %		53.7			51.7			
	Plastic Limit, %		21.3			20.2			
	Plasticity Index, %		32.4			31.4			
	Percent Passing No. 200 Sieve		99.7			98.9			
	Unified Classification		CH			CH			
	Subgroup			H			H		
TRIAXIAL COMPRESSION	Type of Test		3a[b]	3a[b]	3a[b]	1a[b]	1a[b]	1a[b]	
	WATER CONTENT	Initial		22.6			23.6		
		Final		21.0			23.6		
	Unit Dry Wt. lb/ft^3			101.7			101.8		
	Cohesion, ton/ft^2		1.31	1.37	1.51	2.00	2.21	2.98	
	Lateral Pressure, PSI		5.0	17.0	30.0	5.0	17	30	
OTHER SOIL PROPERTIES	Specific Gravity			2.73			2.73		
	Percent Saturation			88.0			96.0		

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 1a, 1b

SITE A-Little Brazos at State Highway 21
Brazos County

TABLE III-1-CONTINUED.--Summary of Tests Results

			17	2nd stage	3rd stage	19	22	23	24
SAMPLE NUMBER			17	2nd stage	3rd stage	19	22	23	24
PENETRATION, FT			21-21.5			24-24.5	26.5-27	30.5-31	32-32.5
PENETRATION RESISTANCE, N*						14	12	12	104
CLASSIFICATION TESTS	Liquid Limit, %		43.9			59.0	63.9	59.6	51.8
	Plastic Limit, %		18.8			23.5	24.3	22.8	19.7
	Plasticity Index, %		25.1			35.5	39.5	36.8	32.1
	Percent Passing No. 200 Sieve		99.1			97.4	94.7	94.9	96.8
	Unified Classification		CL			CH	CH	CH	CH
	Subgroup			Si		H	H	H	W
TRIAXIAL COMPRESSION	Type of Test		2a[b]	2a[b]	2a[b]	1	1	1	3
	WATER CONTENT	Initial		19.6		28.9	27.8	31.2	37.2
		Final		19.5		26.1	25.9	30.3	32.0
	Unit Dry Wt. lb/ft^3			108.1		96.9	95.8	91.8	86.9
	Cohesion, ton/ft^2		2.57	2.50	2.52	1.45	1.25	0.74	0.55
	Lateral Pressure, PSI		5.0	18.0	30.0	21.0	24.0	26.0	27.0
OTHER SOIL PROPERTIES	Specific Gravity			2.75		2.75	2.75	2.75	2.71
	Percent Saturation			98.0		98.0	93.0	97.0	99.0

<u>Legend and Notes</u>

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 1a, 1b

SITE A-Little Brazos at State Highway 21 Brazos County

TABLE III-1-CONTINUED.--Summary of Tests Results

			25	26	27	28	29	31	32
SAMPLE NUMBER			25	26	27	28	29	31	32
PENETRATION, FT			32.5-33	36-36.5	36.5-37	39-39.5	39.5-40	41.5-42	44-44.5
PENETRATION RESISTANCE, N*			104	134	172	150	150	184	240
CLASSIFICATION TESTS	Liquid Limit, %		59.2	70.0	57.8	65.1	62.0	57.5	55.0
	Plastic Limit, %		20.8	29.0	25.5	28.6	26.1	26.0	25.8
	Plasticity Index, %		38.3	40.9	32.3	36.5	35.8	31.4	29.3
	Percent Passing No. 200 Sieve		98.6	79.2	67.8	78.6	88.0	70.6	61.6
	Unified Classification		CH	CH	CH	CH	CH	CH	CH
	Subgroup		W	W	W	W	W	W	W
TRIAXIAL COMPRESSION	Type of Test		1	1	1	1	3	1	1
	WATER CONTENT	Initial	31.7	29.1	27.4	29.1	28.6	28.3	25.9
		Final	36.3	30.3	28.3	28.0	28.0	26.7	24.9
	Unit Dry Wt. lb/ft^3		88.1	94.3	97.7	94.9	93.1	96.3	100.7
	Cohesion, ton/ft^2		0.44	3.99	3.06	3.52	2.35	2.71	4.04
	Lateral Pressure, PSI		27.0	29.0	29.0	30.0	30.0	31.0	32.0
OTHER SOIL PROPERTIES	Specific Gravity		2.71	2.69	2.69	2.69	2.69	2.69	2.69
	Percent Saturation		100.	100.	100.	100.	95.0	100.	100.

<u>Legend and Notes</u>

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 1a, 1b

SITE A-Little Brazos at State Highway 21 Brazos County

TABLE III-1-CONTINUED.--Summary of Tests Results

			30	2nd stage	3rd stage	33			
SAMPLE NUMBER			30	2nd stage	3rd stage	33			
PENETRATION, FT			41-41.5			44.5-45			
PENETRATION RESISTANCE, N*			184			184			
CLASSIFICATION TESTS	Liquid Limit, %		50.9			57.9			
	Plastic Limit, %		21.9			28.4			
	Plasticity Index, %		29.0			29.5			
	Percent Passing No. 200 Sieve		84.0			85.5			
	Unified Classification		CH			CH			
	Subgroup			W		W			
TRIAXIAL COMPRESSION	Type of Test		1a^b	1a^b	1a^b	1			
	WATER CONTENT	Initial		24.3		24.0			
		Final		26.9		26.4			
	Unit Dry Wt. lb/ft^3			98.6		100.2			
	Cohesion, ton/ft^2		2.53	2.83	2.93	2.42			
	Lateral Pressure, PSI		10	20	31	32			
OTHER SOIL PROPERTIES	Specific Gravity			2.69		2.69			
	Percent Saturation			98.0		100.			
Legend and Notes 1 = Unconsolidated-undrained Texas Triaxial 2 = Unconsolidated-undrained Transmatic 3 = Unconsolidated-undrained Triaxial a = Multi-stages b = See Appendix IV N* = Blow count for twelve inches penetration						BORINGS 1a, 1b SITE A-Little Brazos at State Highway 21 Brazos County			

TABLE III-2.--Summary of Tests Results

			1	2	3	2nd stage	3rd stage	6	8
SAMPLE NUMBER			1	2	3	2nd stage	3rd stage	6	8
PENETRATION, FT			6.5-7	9-9.5	9.5-10			22-22.5	24-24.5
PENETRATION RESISTANCE, N*			12	8	8			26	28
CLASSIFICATION TESTS	Liquid Limit, %		35.1	25.7	24.8			31.3	37.1
	Plastic Limit, %		13.8	15.2	15.7			11.1	12.0
	Plasticity Index, %		21.3	10.5	9.1			20.2	25.1
	Percent Passing No. 200 Sieve		48.2	39.6	40.8			78.5	84.1
	Unified Classification		SC	SC	SC			CL	CL
	Subgroup		-	-	-	-	-	Sa	Si
TRIAXIAL COMPRESSION	Type of Test		1	1	1a[b]	1a[b]	1a[b]	1	3
	WATER CONTENT	Initial	16.4	16.5		16.9		13.5	14.3
		Final	18.0	17.2		17.5		14.1	14.8
	Unit Dry Wt. lb/ft^3		112.0	112.5		110.7		117.2	119.2
	Cohesion, ton/ft^2		1.23	1.31	0.81	1.09	1.50	2.03	2.17
	Lateral Pressure, PSI		5.5	8.0	2.5	7.5	17.5	13.5	14.0
OTHER SOIL PROPERTIES	Specific Gravity		2.70	2.70		2.70		2.73	2.73
	Percent Saturation		92	91		89		83	93

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2,3

SITE B-Interstate 610 at HB&T Railroad, Houston, Texas

TABLE III-2-CONTINUED.--Summary of Tests Results

			4	2nd stage	3rd stage	5	2nd stage	3rd stage	9
SAMPLE NUMBER			4	2nd stage	3rd stage	5	2nd stage	3rd stage	9
PENETRATION, FT			11-11.5			11.5-12			24.5-25
PENETRATION RESISTANCE, N*			8						32
CLASSIFICATION TESTS	Liquid Limit, %		24.9			25.2			37.8
	Plastic Limit, %		16.0			17.7			13.5
	Plasticity Index, %		8.9			7.5			24.3
	Percent Passing No. 200 Sieve		39.9			37.7			81.3
	Unified Classification		SC			SC			CL
	Subgroup		-	-	-	-	-	-	Si
TRIAXIAL COMPRESSION	Type of Test		3a[b]	3a[b]	3a[b]	2a	2a	2a	1
	WATER CONTENT	Initial		16.7			17.6		14.2
		Final		17.0			18.0		15.4
	Unit Dry Wt. lb/ft^3			114.0			110.2		117.4
	Cohesion, ton/ft^2		0.84	0.98	1.08	0.41	0.41	0.41	3.27
	Lateral Pressure, PSI		4	9	19	4	9	19	14.5
OTHER SOIL PROPERTIES	Specific Gravity			2.70			2.70		2.73
	Percent Saturation			95			91		90

Legend and Notes

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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2, 3

SITE B-Interstate 610 at HB&T Railroad, Houston, Texas

TABLE III-2-CONTINUED.--Summary of Tests Results

		10	2nd stage	3rd stage	11	12	2nd stage	3rd stage
SAMPLE NUMBER		10	2nd stage	3rd stage	11	12	2nd stage	3rd stage
PENETRATION, FT		25.5-26			26.5-27	27.5-28		
PENETRATION RESISTANCE, N*		30			28	32		
CLASSIFICATION TESTS	Liquid Limit, %	42.0			36.9	48.0		
	Plastic Limit, %	13.2			12.7	12.4		
	Plasticity Index, %	28.8			24.2	35.6		
	Percent Passing No. 200 Sieve	78.9			81.8	89.0		
	Unified Classification	CL			CL	CL		
	Subgroup		Sa		Si		Si	
TRIAXIAL COMPRESSION	Type of Test	$1a^b$	$1a^b$	$1a^b$	1	$3a^b$	$3a^b$	$3a^b$
	WATER CONTENT Initial		14.5		14.5		17.7	
	WATER CONTENT Final		15.3		15.9		18.9	
	Unit Dry Wt. lb/ft^3		118.0		117.5		110.6	
	Cohesion, ton/ft^2	3.44	3.60	3.72	3.67	1.45	1.71	1.93
	Lateral Pressure, PSI	5	15	25	15	5	15	25
OTHER SOIL PROPERTIES	Specific Gravity		2.73		2.73		2.67	
	Percent Saturation		92		92		97	

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2, 3

SITE B-Interstate 610 at HB&T Railroad, Houston, Texas

TABLE III-2-CONTINUED.--Summary of Tests Results

			13	15	16	18	19	21	23
SAMPLE NUMBER			13	15	16	18	19	21	23
PENETRATION, FT			28.5-29	31-31.5	33-33.5	34.5-35	36.5-37	39.5-40	41.5-42
PENETRATION RESISTANCE, N*			28	26	24	24	18	64	70
CLASSIFICATION TESTS	Liquid Limit, %		46.1	32.5	33.4	28.9	31.1	30.0	68.6
	Plastic Limit, %		17.6	16.4	17.0	15.1	15.5	19.1	26.4
	Plasticity Index, %		28.5	16.1	16.4	13.8	15.6	10.9	42.2
	Percent Passing No. 200 Sieve		96.3	81.8	89.4	90.6	83.5	97.8	99.4
	Unified Classification		CL	CL	CL	CL	CL	CL	CH
	Subgroup		Si	Si	Si	Si	Si	Si	W
TRIAXIAL COMPRESSION	Type of Test		1	1	3	1	1	1	1
	WATER CONTENT	Initial	19.0	17.1	19.4	19.5	19.1	24.4	26.3
		Final	21.5	17.8	20.8	23.4	17.8	25.6	25.4
	Unit Dry Wt. lb/ft^3		108.0	111.4	106.9	103.6	107.5	97.4	97.3
	Cohesion, ton/ft^2		2.99	2.82	1.08	1.48	2.36	1.04	2.31
	Lateral Pressure, PSI		16.0	17.0	17.0	17.5	18.5	19.0	20.0
OTHER SOIL PROPERTIES	Specific Gravity		2.67	2.73	2.67	2.67	2.73	2.67	2.75
	Percent Saturation		100.	90	96	94	86	94	93

Legend and Notes

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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2, 3

SITE B-Interstate 610 at HB&T Railroad, Houston, Texas

TABLE III-2-CONTINUED.--Summary of Tests Results

SAMPLE NUMBER			22	2nd stage	3rd stage	25	2nd stage	3rd stage	27
PENETRATION, FT			40.5-41			43.5-44			44.5-45
PENETRATION RESISTANCE, N*			118			70			40
CLASSIFICATION TESTS	Liquid Limit, %		53.3			69.7			65.2
	Plastic Limit, %		20.3			22.9			24.2
	Plasticity Index, %		33.0			46.8			41.0
	Percent Passing No. 200 Sieve		98.8			99.6			96.9
	Unified Classification		CH			CH			CH
	Subgroup			W			W		W
TRIAXIAL COMPRESSION	Type of Test		1a[b]	1a[b]	1a[b]	3a[b]	3a[b]	3a[b]	1
	WATER CONTENT	Initial		21.7			24.9		24.5
		Final		23.8			25.6		23.2
	Unit Dry Wt. lb/ft^3			101.8			101.3		102.3
	Cohesion, ton/ft^2		1.86	2.06	2.22	1.62	1.67	1.81	3.06
	Lateral Pressure, PSI		9.5	19.5	29.5	10.5	20.5	30.5	21.0
OTHER SOIL PROPERTIES	Specific Gravity			2.75			2.75		2.75
	Percent Saturation			91			100		97

Legend and Notes

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a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2, 3

SITE B-Interstate 610 at HB&T Railroad, Houston, Texas

TABLE III-2-CONTINUED.--Summary of Tests Results

			30	32	2nd stage	3rd stage	33	36	37
SAMPLE NUMBER			30	32	2nd stage	3rd stage	33	36	37
PENETRATION, FT			48-48.5	49.5-50			50-50.5	52-52.5	54.5-55
PENETRATION RESISTANCE, N*			28	40				40	120
CLASSIFICATION TESTS	Liquid Limit, %		59.2	36.1			38.6	43.0	65.6
	Plastic Limit, %		22.0	17.6			19.4	19.8	24.6
	Plasticity Index, %		37.2	18.5			19.2	23.2	41.0
	Percent Passing No. 200 Sieve		100	94.2			100	100	100
	Unified Classification		CH	CL			CL	CL	CH
	Subgroup		H		Si		Si		W
TRIAXIAL COMPRESSION	Type of Test		1	3a[b]	3a[b]	3a[b]	1	1	1
	WATER CONTENT	Initial	22.9		21.9		19.0	23.8	29.1
		Final	23.0		21.6		22.6	24.0	26.6
	Unit Dry Wt. lb/ft^3		104.4		103.5		106.1	101.3	94.7
	Cohesion, ton/ft^2		2.68	1.12	1.25	1.34	2.09	1.58	2.44
	Lateral Pressure, PSI		22	12.5	22.5	32.5	22.5	23.0	24.0
OTHER SOIL PROPERTIES	Specific Gravity		2.75		2.67		2.67	2.67	2.75
	Percent Saturation		98		95		97	99	94

<u>Legend and Notes</u>

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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2, 3

SITE B-Interstate 610 at HB&T Railroad, Houston, Texas

TABLE III-2-CONTINUED.--Summary of Tests Results

SAMPLE NUMBER			38	2nd stage	3rd stage	39	40	42	46
PENETRATION, FT			55-55.5			56.5-57	57-57.5	59.5-60	64.5-65
PENETRATION RESISTANCE, N*			184			32	32	42	
CLASSIFICATION TESTS	Liquid Limit, %		71.6		20% DEFORMATION EXCEEDED	52.2	57.2	33.0	19.8
	Plastic Limit, %		26.4			20.7	24.0	17.0	17.2
	Plasticity Index, %		45.2			31.5	33.2	16.0	2.6
	Percent Passing No. 200 Sieve		100			93.1	95.2	98.9	50.7
	Unified Classification		CH			CH	CH	CL	ML
	Subgroup		W	-	-	H	H	Si	
TRIAXIAL COMPRESSION	Type of Test		1a	1a	1a	3	1	1	1
	WATER CONTENT	Initial		32.3		19.0	24.9	20.8	24.0
		Final		27.2		20.6	23.4	23.5	21.7
	Unit Dry Wt. lb/ft^3			94.6		104.7	101.4	105.4	104.2
	Cohesion, ton/ft^2		2.27	2.14		1.33	2.47	1.62	4.51
	Lateral Pressure, PSI		14	24	34	0	24.5	25.5	26.0
OTHER SOIL PROPERTIES	Specific Gravity			2.75		2.75	2.75	2.67	2.70
	Percent Saturation			100		85	96	100	100

<u>Legend and Notes</u>

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2, 3

SITE B-Interstate 610 and HB&T Railroad, Houston, Texas

TABLE III-2-CONTINUED.--Summary of Tests Results

			41	2nd stage	3rd stage	43	2nd stage	3rd stage	47
SAMPLE NUMBER			41	2nd stage	3rd stage	43	2nd stage	3rd stage	47
PENETRATION, FT			58.5-59			61-61.5			66.5-67
PENETRATION RESISTANCE, N*			30			30			
CLASSIFICATION TESTS	Liquid Limit, %		27.5			71.0			25.2
	Plastic Limit, %		18.6			24.4			19.7
	Plasticity Index, %		8.9			46.6			5.5
	Percent Passing No. 200 Sieve		95.7			100			64.4
	Unified Classification		CL			CH			CL-ML
	Subgroup			Si			H		
TRIAXIAL COMPRESSION	Type of Test		1a[b]	1a[b]	1a[b]	3a[b]	3a[b]	3a[b]	3
	WATER CONTENT	Initial		23.8			25.9		24.6
		Final		22.5			27.0		24.5
	Unit Dry Wt. lb/ft^3			101.4			99.1		97.5
	Cohesion, ton/ft^2		0.94	1.25	1.49	1.55	1.62	1.61	0.49
	Lateral Pressure, PSI		15	25	35	16	26	36	27.5
OTHER SOIL PROPERTIES	Specific Gravity			2.67			2.75		2.7
	Percent Saturation			96			99		100

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2, 3

SITE B-Interstate 610 and HB&T Railroad, Houston, Texas

TABLE III-2-CONTINUED.--Summary of Tests Results

SAMPLE NUMBER			48						
PENETRATION, FT			69-69.5						
PENETRATION RESISTANCE, N*									
CLASSIFICATION TESTS	Liquid Limit, %		43.6						
	Plastic Limit, %		26.0						
	Plasticity Index, %		17.6						
	Percent Passing No. 200 Sieve		100						
	Unified Classification		ML						
	Subgroup								
TRIAXIAL COMPRESSION	Type of Test		1						
	WATER CONTENT	Initial	31.0						
		Final	31.3						
	Unit Dry Wt. lb/ft^3		92.0						
	Cohesion, ton/ft^2		2.57						
	Lateral Pressure, PSI		28.5						
OTHER SOIL PROPERTIES	Specific Gravity		2.7						
	Percent Saturation		100						

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 2, 3

SITE B-Interstate 610 and HB&T Railroad, Houston, Texas

TABLE III-3.--Summary of Tests Results

			1	2	3	5	6	2nd stage	3rd stage
SAMPLE NUMBER			1	2	3	5	6	2nd stage	3rd stage
PENETRATION, FT			3-3.5	5-5.5	5.5-6	7.5-8	8.5-9		
PENETRATION RESISTANCE, N*			10	40	40	34	16		
CLASSIFICATION TESTS	Liquid Limit, %		81.3	39.5	42.8	45.6	61.9		
	Plastic Limit, %		18.7	13.1	11.5	12.7	10.3		
	Plasticity Index, %		62.6	26.4	31.3	32.9	51.6		
	Percent Passing No. 200 Sieve		82.4	69.9	65.7	59.8	73.4		
	Unified Classification		CH	CL	CL	CL	CH		
	Subgroup		H	Sa	Sa	Sa		H	
TRIAXIAL COMPRESSION	Type of Test		1	3	1	1	$1a^b$	$1a^b$	$1a^b$
	WATER CONTENT	Initial	20.8	12.9	12.3	11.6		16.2	
		Final	24.7	13.8	12.1	11.9		17.5	
	Unit Dry Wt. lb/ft^3		104.5	117.2	119.6	119.6		112.0	
	Cohesion, ton/ft^2		1.78	2.43	4.38	3.86	1.65	1.99	2.73
	Lateral Pressure, PSI		3.5	5	5	6.5	2	7	17
OTHER SOIL PROPERTIES	Specific Gravity		2.74	2.75	2.75	2.75		2.74	
	Percent Saturation		98	79	77	75		88	

<u>Legend and Notes</u>

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 4,5

SITE C
Brays Bayou at State Highway 288, Houston Texas

TABLE III-3-CONTINUED.--Summary of Tests Results

		7	2nd stage	3rd stage	8	2nd stage	3rd stage	9
SAMPLE NUMBER		7	2nd stage	3rd stage	8	2nd stage	3rd stage	9
PENETRATION, FT		9-9.5			9.5-10			11-11.5
PENETRATION RESISTANCE, N*					20			18
CLASSIFICATION TESTS	Liquid Limit, %	58.6			59.8			62.8
	Plastic Limit, %	11.7			14.2			17.2
	Plasticity Index, %	46.9			45.6			45.6
	Percent Passing No. 200 Sieve	71.4			75.1			94.7
	Unified Classification	CH			CH			CH
	Subgroup					H		H
TRIAXIAL COMPRESSION	Type of Test	$2a^b$	$2a^b$	$2a^b$	$3a^b$	$3a^b$	$3a^b$	1
	WATER CONTENT: Initial		18.2			18.3		18.1
	WATER CONTENT: Final		18.0			18.3		20.1
	Unit Dry Wt. lb/ft^3		107.8			110.3		109.8
	Cohesion, ton/ft^2	1.91	2.06	2.15	1.63	1.63	1.66	2.05
	Lateral Pressure, PSI	2.5	7.5	17.5	3	8	18	9.5
OTHER SOIL PROPERTIES	Specific Gravity		2.74			2.74		2.74
	Percent Saturation		85			94		94

Legend and Notes

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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 4, 5

SITE C
Brays Bayou at State Highway 288, Houston Texas

TABLE III-3-CONTINUED.--Summary of Tests Results

SAMPLE NUMBER			10	12	13	16	18	19	22
PENETRATION, FT			11.5-12	13-13.5	13.5-14	15.5-16	17.5-18	18.5-19	20.5-21
PENETRATION RESISTANCE, N*			18	24	24	22	22	18	30
CLASSIFICATION TESTS	Liquid Limit, %		60.7	22.2	23.7	31.5	29.7	31.4	33.1
	Plastic Limit, %		14.7	12.5	11.7	11.7	11.9	12.3	11.2
	Plasticity Index, %		46.0	9.7	12.0	19.8	17.8	19.1	21.9
	Percent Passing No. 200 Sieve		90.7	72.9	78.2	80.5	74.2	79.4	85.2
	Unified Classification		CH	CL	CL	CL	CL	CL	CL
	Subgroup		H	Sa	Sa	Si	Sa	Sa	Si
TRIAXIAL COMPRESSION	Type of Test		3	1	3	1	3	1	1
	WATER CONTENT	Initial	19.6	13.8	13.5	13.5	15.9	15.0	16.1
		Final	20.0	13.8	14.4	14.3	13.5	14.4	15.9
	Unit Dry Wt. lb/ft^3		107.2	114.8	115.8	115.3	112.4	115.2	113.9
	Cohesion, ton/ft^2		1.50	1.98	1.24	2.41	1.50	2.72	2.42
	Lateral Pressure, PSI		9.5	11.0	11.5	13.0	14.5	15.5	17.0
OTHER SOIL PROPERTIES	Specific Gravity		2.74	2.75	2.75	2.75	2.75	2.75	2.75
	Percent Saturation		91	77	80	78	77	83	87

<u>Legend and Notes</u>

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a = Multi-stages
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N* = Blow count for twelve inches penetration

BORINGS 4, 5

SITE C
Brays Bayou at State Highway 288, Houston Texas

TABLE III-3-CONTINUED.--Summary of Tests Results

		24	26	30	32	2nd stage	3rd stage	34
SAMPLE NUMBER		24	26	30	32	2nd stage	3rd stage	34
PENETRATION, FT		21.5-22	23-23.5	25.5-26	27-27.5			28.5-29
PENETRATION RESISTANCE, N*		32	38	30	44			54
CLASSIFICATION TESTS	Liquid Limit, %	34.3	46.2	43.3	33.7			25.4
	Plastic Limit, %	11.2	12.1	13.3	15.3			16.5
	Plasticity Index, %	23.1	34.1	30.0	18.4			8.9
	Percent Passing No. 200 Sieve	87.2	80.3	74.2	58.9			34.0
	Unified Classification	CL	CL	CL	CL			SC
	Subgroup	Si	Si	Sa	-	Sa	-	-
TRIAXIAL COMPRESSION	Type of Test	3	1	1	1a[b]	1a[b]	1a[b]	1
	WATER CONTENT: Initial	15.4	14.3	14.6		13.7		14.3
	WATER CONTENT: Final	16.7	13.8	14.9		16.7		13.9
	Unit Dry Wt. lb/ft^3	114.9	119.2	117.6		116.5		116.0
	Cohesion, ton/ft^2	1.53	4.76	4.48	2.72	3.04	3.48	3.55
	Lateral Pressure, PSI	17.5	19.0	21.0	12	22	32	22.0
OTHER SOIL PROPERTIES	Specific Gravity	2.75	2.75	2.75		2.75		2.65
	Percent Saturation	89	88	88		88		88

Legend and Notes

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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 4, 5

SITE C
Brays Bayou at State Highway 288, Houston Texas

TABLE III-3-CONTINUED.--Summary of Tests Results

			33	2nd stage	3rd stage	35			
SAMPLE NUMBER			33	2nd stage	3rd stage	35			
PENETRATION, FT			27.5-28			29.5-30			
PENETRATION RESISTANCE, N*			44						
CLASSIFICATION TESTS	Liquid Limit, %		39.0			19.4			
	Plastic Limit, %		14.5			17.6			
	Plasticity Index, %		24.5			1.8			
	Percent Passing No. 200 Sieve		68.1			24.4			
	Unified Classification		CL			SM			
	Subgroup			Sa					
TRIAXIAL COMPRESSION	Type of Test		$3a^b$	$3a^b$	$3a^b$	1			
	WATER CONTENT	Initial		16.6		14.0			
		Final		17.0		13.3			
	Unit Dry Wt. lb/ft^3			109.5		116.1			
	Cohesion, ton/ft^2		2.05	2.19	2.19	3.87			
	Lateral Pressure, PSI		12.5	22.5	32.5	24			
OTHER SOIL PROPERTIES	Specific Gravity			2.75		2.65			
	Percent Saturation			81		87			

Legend and Notes

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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 4, 5

SITE C
Brays Bayou at State Highway 288, Houston Texas

TABLE III-4.--Summary of Tests Results

			1	2	3	6	7	9	10
SAMPLE NUMBER			1	2	3	6	7	9	10
PENETRATION, FT			6.5-7	7.5-8	9.5-10	14-14.5	15-16	24-25	25-26
PENETRATION RESISTANCE, N*			10	22	18	24	24	22	22
CLASSIFICATION TESTS	Liquid Limit, %		60.1	59.0	56.4	28.2	67.4	32.5	33.6
	Plastic Limit, %		17.4	14.5	16.8	13.4	22.1	11.0	12.9
	Plasticity Index, %		42.7	44.5	39.6	14.8	45.3	21.5	20.7
	Percent Passing No. 200 Sieve		85.7	59.4	85.3	39.0	95.0	78.5	74.6
	Unified Classification		CH	CH	CH	SC	CH	CL	CL
	Subgroup		H	H	H		H	Sa	Sa
TRIAXIAL COMPRESSION	Type of Test		1	3	1	1	1	1	3
	WATER CONTENT	Initial	23.3	22.2	20.5	18.4	24.1	15.7	15.7
		Final	23.3	16.8	21.0	18.0	22.7	16.2	15.4
	Unit Dry Wt. lb/ft^3		98.7	106.6	107.1	107.6	102.8	112.3	115.7
	Cohesion, ton/ft^2		1.03	1.03	1.80	1.51	1.92	1.59	1.05
	Lateral Pressure, PSI		5.0	6.0	7.5	11.0	11.5	18.5	19.5
OTHER SOIL PROPERTIES	Specific Gravity		2.75	2.75	2.75	2.70	2.75	2.73	2.73
	Percent Saturation		86	81	96	86	94	84	90

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 6, 7

SITE D
Interstate Highway
45 at Nettleton St.,
Houston, Texas

TABLE III-4-CONTINUED.--Summary of Tests Results

			11	13	14	17	19	21	2nd stage
SAMPLE NUMBER			11	13	14	17	19	21	2nd stage
PENETRATION, FT			26-27	28-29	29-30	31.5-32	33-33.5	34.5-35	
PENETRATION RESISTANCE, N*			32	32	26	22	28		
CLASSIFICATION TESTS	Liquid Limit, %		40.7	30.6	33.1	25.0	24.8	23.9	
	Plastic Limit, %		10.9	16.0	14.8	16.5	17.3	18.1	
	Plasticity Index, %		9.8	14.6	18.3	8.5	7.5	5.8	
	Percent Passing No. 200 Sieve		60.5	68.0	62.9	54.0	59.5	83.4	
	Unified Classification		CL	CL	CL	CL	CL	CL-ML	
	Subgroup		Sa	Sa	Sa	Sa	Sa		
TRIAXIAL COMPRESSION	Type of Test		1	3	1	1	3	1a[b]	1a[b]
	WATER CONTENT	Initial	15.0	13.5	12.1	15.8	15.6		15.9
		Final	13.2	14.0	12.6	16.3	16.5		16.6
	Unit Dry Wt. lb/ft^3		117.2	120.6	121.1	115.6	113.4		111.5
	Cohesion, ton/ft^2		2.50	1.95	3.36	3.56	1.39	2.35	3.13
	Lateral Pressure, PSI		20.0	21.5	23.0	23.0	23.5	14	24
OTHER SOIL PROPERTIES	Specific Gravity		2.73	2.73	2.73	2.73	2.73		2.70
	Percent Saturation		84	90	83	92	87		84

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 6, 7

SITE D
Interstate Highway 45
at Nettleton Street,
Houston, Texas

TABLE III-4-CONTINUED.--Summary of Tests Results

SAMPLE NUMBER			21-3d stage	22	2nd stage	3rd stage	23	2nd stage	3rd stage
PENETRATION, FT				35-35.5			35.5-36		
PENETRATION RESISTANCE, N *				34			34		
CLASSIFICATION TESTS	Liquid Limit, %			24.5			27.3		
	Plastic Limit, %			16.9			14.5		
	Plasticity Index, %			7.6			12.8		
	Percent Passing No. 200 Sieve			49.3			48.1		
	Unified Classification			SC			SC		
	Subgroup								
TRIAXIAL COMPRESSION	Type of Test		1a[b]	2a	2a	2a	3a[b]	3a[b]	3a[b]
	WATER CONTENT	Initial		14.2				15.5	
		Final		14.5				15.6	
	Unit Dry Wt. lb/ft^3			115.1				113.3	
	Cohesion, ton/ft^2		3.88	0.62	0.62	0.62	1.65	1.89	2.03
	Lateral Pressure, PSI		34	4.5	14.5	24.5	14.5	24.5	34.5
OTHER SOIL PROPERTIES	Specific Gravity				2.70			2.70	
	Percent Saturation				81			86	

Legend and Notes

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a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 6, 7

SITE D
Interstate Highway 45
at Nettleton Street,
Houston, Texas

TABLE III-4-CONTINUED.--Summary of Tests Results

SAMPLE NUMBER			24							
PENETRATION, FT			36.5-37							
PENETRATION RESISTANCE, N*			46							
CLASSIFICATION TESTS	Liquid Limit, %		24.8							
	Plastic Limit, %		17.5							
	Plasticity Index, %		7.3							
	Percent Passing No. 200 Sieve		51.1							
	Unified Classification		CL							
	Subgroup		Sa							
TRIAXIAL COMPRESSION	Type of Test		1							
	WATER CONTENT	Initial	16.9							
		Final	16.6							
	Unit Dry Wt. lb/ft^3		114.3							
	Cohesion, ton/ft^2		2.47							
	Lateral Pressure, PSI		25.0							
OTHER SOIL PROPERTIES	Specific Gravity		2.73							
	Percent Saturation		92							

<u>Legend and Notes</u>

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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

BORINGS 6, 7

SITE D
Interstate Highway 45
at Nettleton Street,
Houston, Texas

TABLE III-5.--Summary of Tests Results

SAMPLE NUMBER AND SITE		15-A	16-A	28-A	29-A	3-B	4-B	8-B
PENETRATION, FT		19-19.5	19.5-20	39-39.5	39.5-40	9.5-10	11-11.5	24-24.5
PENETRATION RESISTANCE, N		36	36	300	300	16	16	56
CLASSIFICATION TESTS	Liquid Limit, %	53.7	51.7	65.1	62.0	24.8	24.9	37.1
	Plastic Limit, %	21.3	20.2	28.6	26.1	15.7	16.0	12.0
	Plasticity Index, %	32.4	31.4	36.5	35.8	9.1	8.9	25.1
	Percent Passing No. 200 Sieve	99.7	98.9	78.6	88.0	40.8	39.9	84.1
	Unified Classification	CH	CH	CH	CH	SC	SC	CL
	Subgroup	H	H	W	W	-	-	Si
TRIAXIAL COMPRESSION	Type of Test	3	1	1	3	1	3	3
	WATER CONTENT Initial	22.6	23.6	29.1	28.6	16.9	16.7	14.3
	WATER CONTENT Final	21.0	23.6	28.0	28.0	17.5	17.0	14.8
	Unit Dry Wt. lb/ft^3	101.7	101.8	94.9	93.1	110.7	114.0	119.2
	Cohesion, ton/ft^2	1.51	2.21	3.52	2.35	1.50	1.08	2.17
	Lateral Pressure, PSI	17.0	17.0	30.0	30.0	17.5	19	14.0
OTHER SOIL PROPERTIES	Specific Gravity	2.73	2.73	2.69	2.69	2.7	2.70	2.73
	Percent Saturation	88	96	100	95	89	90	93

Legend and Notes

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2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

ALL BORINGS

Comparison of shear strength results

TABLE III-5-CONTINUED.--Summary of Tests Results

			9-B	12-B	13-B	16-B	18-B	25-B	27-B
SAMPLE NUMBER AND SITE			9-B	12-B	13-B	16-B	18-B	25-B	27-B
PENETRATION, FT			24.5-25	27.5-28	28.5-29	33-33.5	34.5-35	43.5-44	44.5-45
PENETRATION RESISTANCE, N			64	64	56	48	48	92	80
CLASSIFICATION TESTS	Liquid Limit, %		37.8	48.0	46.1	33.4	28.9	69.7	65.2
	Plastic Limit, %		13.5	12.4	17.6	17.0	15.1	22.9	24.2
	Plasticity Index, %		24.3	35.6	28.5	16.4	13.8	46.8	41.0
	Percent Passing No. 200 Sieve		81.3	89.0	96.3	89.4	90.6	99.6	96.9
	Unified Classification		CL	CL	CL	CL	CL	CH	CH
	Subgroup		Si	Si	Si	Si	Si	W	W
TRIAXIAL COMPRESSION	Type of Test		1	3	1	3	1	3	1
	WATER CONTENT	Initial	14.2	17.7	19.0	19.4	19.5	24.9	24.5
		Final	15.4	18.9	21.5	20.8	23.4	25.6	23.2
	Unit Dry Wt. lb/ft^3		117.4	110.6	108.0	106.9	103.6	101.3	102.3
	Cohesion, ton/ft^2		3.27	1.71	2.99	1.08	1.48	1.67	3.06
	Lateral Pressure, PSI		14.5	15	16.0	17.0	17.5	20.5	21.0
OTHER SOIL PROPERTIES	Specific Gravity		2.73	2.67	2.67	2.67	2.67	2.75	2.75
	Percent Saturation		90	97	100	96	94	100	97

<u>Legend and Notes</u>

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3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

ALL BORINGS

Comparison of shear strength results

TABLE III-5-CONTINUED.--Summary of Tests Results

SAMPLE NUMBER AND SITE			32-B	33-B	2-C	3-C	9-C	10-C	12-C
PENETRATION, FT			49.5-50	50-50.5	5-5.5	5.5-6	11-11.5	11.5-12	13-13.5
PENETRATION RESISTANCE, N			80	80	80	80	36	36	48
CLASSIFICATION TESTS	Liquid Limit, %		36.1	38.6	39.5	42.8	62.8	60.7	22.2
	Plastic Limit, %		17.6	19.4	13.1	11.5	17.2	14.7	12.5
	Plasticity Index, %		18.5	19.2	26.4	31.3	45.6	46.0	9.7
	Percent Passing No. 200 Sieve		94.2	100	69.9	65.7	94.7	90.7	72.9
	Unified Classification		CL	CL	CL	CL	CH	CH	CL
	Subgroup		Si	Si	Sa	Sa	H	H	Sa
TRIAXIAL COMPRESSION	Type of Test		3	1	3	1	1	3	1
	WATER CONTENT	Initial	21.9	19.0	12.9	12.3	18.1	19.6	13.8
		Final	21.6	22.6	13.8	12.1	20.1	20.0	13.8
	Unit Dry Wt. lb/ft^3		103.5	106.1	117.2	119.6	109.8	107.2	114.8
	Cohesion, ton/ft^2		1.25	2.09	2.43	4.38	2.05	1.50	1.98
	Lateral Pressure, PSI		22.5	22.5	5	5	9.5	9.5	11.0
OTHER SOIL PROPERTIES	Specific Gravity		2.67	2.67	2.75	2.75	2.74	2.74	2.75
	Percent Saturation		95	97	79	77	94	91	77

Legend and Notes	ALL BORINGS
1 = Unconsolidated-undrained Texas Triaxial 2 = Unconsolidated-undrained Transmatic 3 = Unconsolidated-undrained Triaxial a = Multi-stages b = See Appendix IV N* = Blow count for twelve inches penetration	Comparison of shear strength results

TABLE III-5-CONTINUED.--Summary of Tests Results

			13-C	16-C	18-C	22-C	24-C	2-D	3-D
SAMPLE NUMBER AND SITE			13-C	16-C	18-C	22-C	24-C	2-D	3-D
PENETRATION, FT			13.5-14	15.5-16	17.5-18	20.5-21	21.5-22	7.5-8	9.5-10
PENETRATION RESISTANCE, N			48	44	44	60	64	44	36
CLASSIFICATION TESTS	Liquid Limit, %		23.7	31.5	29.7	33.1	34.3	59.0	56.4
	Plastic Limit, %		11.7	11.7	11.9	11.2	11.2	14.5	16.8
	Plasticity Index, %		12.0	19.8	17.8	21.9	23.1	44.5	39.6
	Percent Passing No. 200 Sieve		78.2	80.5	74.2	85.2	87.2	59.4	85.3
	Unified Classification		CL	CL	CL	CL	CL	CH	CH
	Subgroup		Sa	Sa	Sa	Si	Si	H	H
TRIAXIAL COMPRESSION	Type of Test		3	1	3	1	3	3	1
	WATER CONTENT	Initial	13.5	13.5	15.9	16.1	15.4	22.2	20.5
		Final	14.4	14.3	13.5	15.9	16.7	16.8	21.0
	Unit Dry Wt. lb/ft^3		115.8	115.3	112.4	113.9	114.9	106.6	107.1
	Cohesion, ton/ft^2		1.24	2.41	1.50	2.42	1.53	1.03	1.80
	Lateral Pressure, PSI		11.5	13.0	14.5	17.0	17.5	6.0	7.5
OTHER SOIL PROPERTIES	Specific Gravity		2.75	2.75	2.75	2.75	2.75	2.75	2.75
	Percent Saturation		80	78	77	87	89	81	96

Legend and Notes

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2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

ALL BORINGS

Comparison of shear strength results

TABLE III-5-CONTINUED.--Summary of Tests Results

SAMPLE NUMBER AND SITE			9-D	10-D	13-D	14-D	17-D	19-D	
PENETRATION, FT			24-25	25-26	28-29	29-30	31.5-32	33-33.5	
PENETRATION RESISTANCE, N			44	44	64	52	44	56	
CLASSIFICATION TESTS	Liquid Limit, %		32.5	33.6	30.6	33.1	25.0	24.8	
	Plastic Limit, %		11.0	12.9	16.0	14.8	16.5	17.3	
	Plasticity Index, %		21.5	20.7	14.6	18.3	8.5	7.5	
	Percent Passing No. 200 Sieve		78.5	74.6	68.0	62.9	54.0	59.5	
	Unified Classification		CL	CL	CL	CL	CL	CL	
	Subgroup		Sa	Sa	Sa	Sa	Sa	Sa	
TRIAXIAL COMPRESSION	Type of Test		1	3	3	1	1	3	
	WATER CONTENT	Initial	15.7	15.7	13.5	12.1	15.8	15.6	
		Final	16.2	15.4	14.0	12.6	16.3	16.5	
	Unit Dry Wt. lb/ft^3		112.3	115.7	120.6	121.1	115.6	113.4	
	Cohesion, ton/ft^2		1.59	1.05	1.95	3.36	3.56	1.39	
	Lateral Pressure, PSI		18.5	19.5	21.5	23.0	23.0	23.5	
OTHER SOIL PROPERTIES	Specific Gravity		2.73	2.73	2.73	2.73	2.73	2.73	
	Percent Saturation		84	90	90	83	92	87	

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained Transmatic
3 = Unconsolidated-undrained Triaxial
a = Multi-stages
b = See Appendix IV
N* = Blow count for twelve inches penetration

ALL BORINGS

Comparison of shear strength results

APPENDIX IV

MOHR'S DIAGRAMS

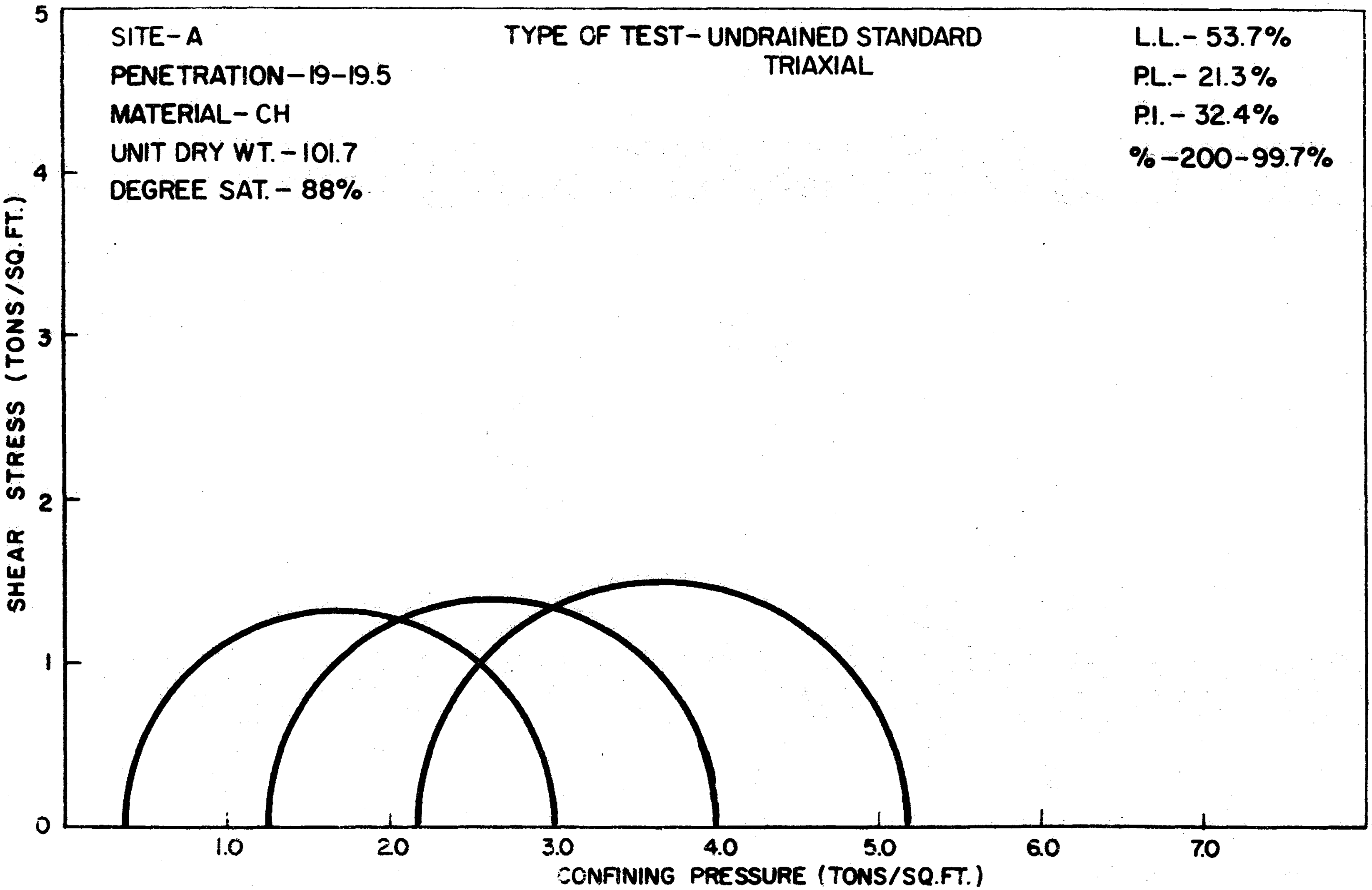

FIG.-IV-1 TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

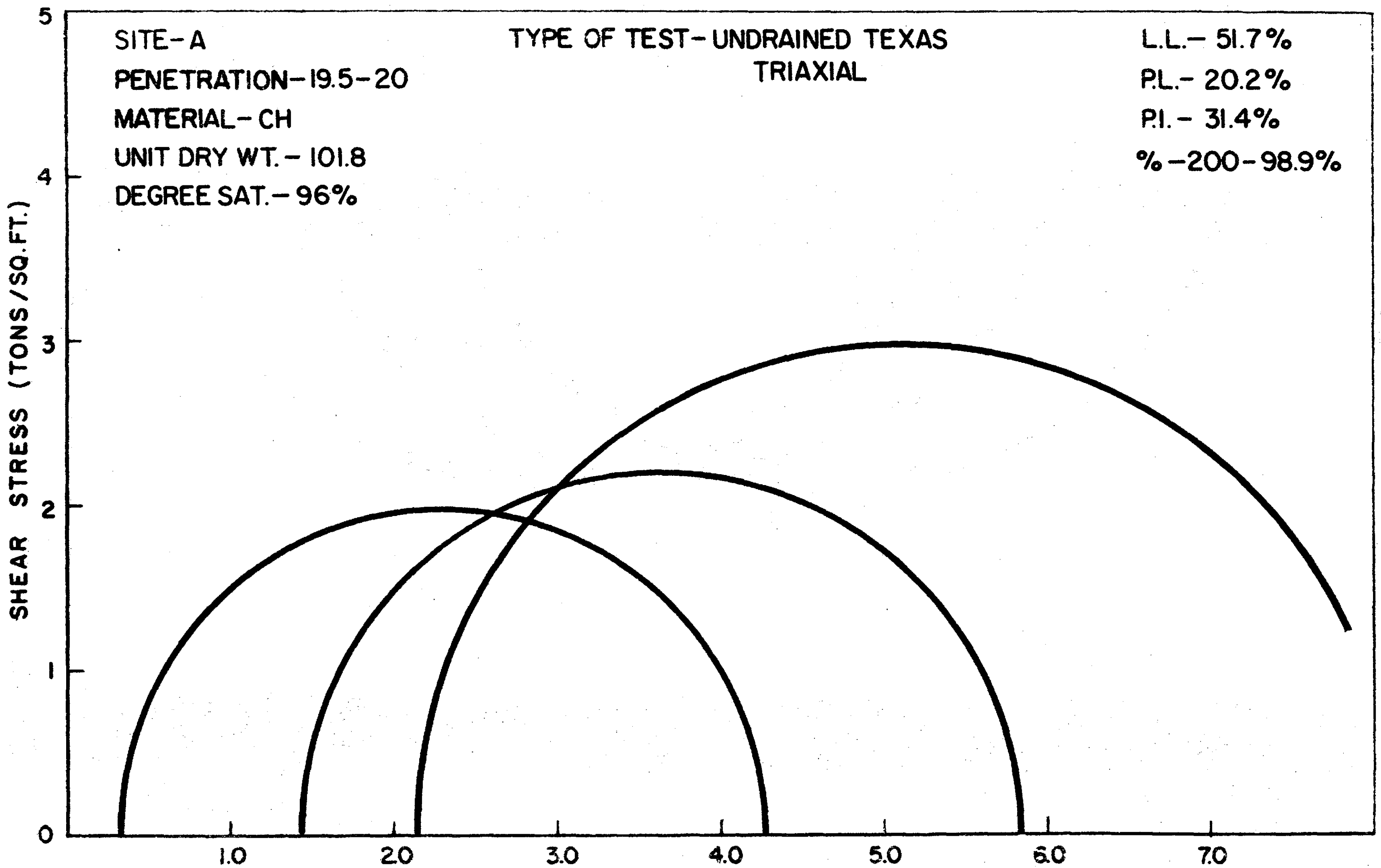

FIG.-IV-2 TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

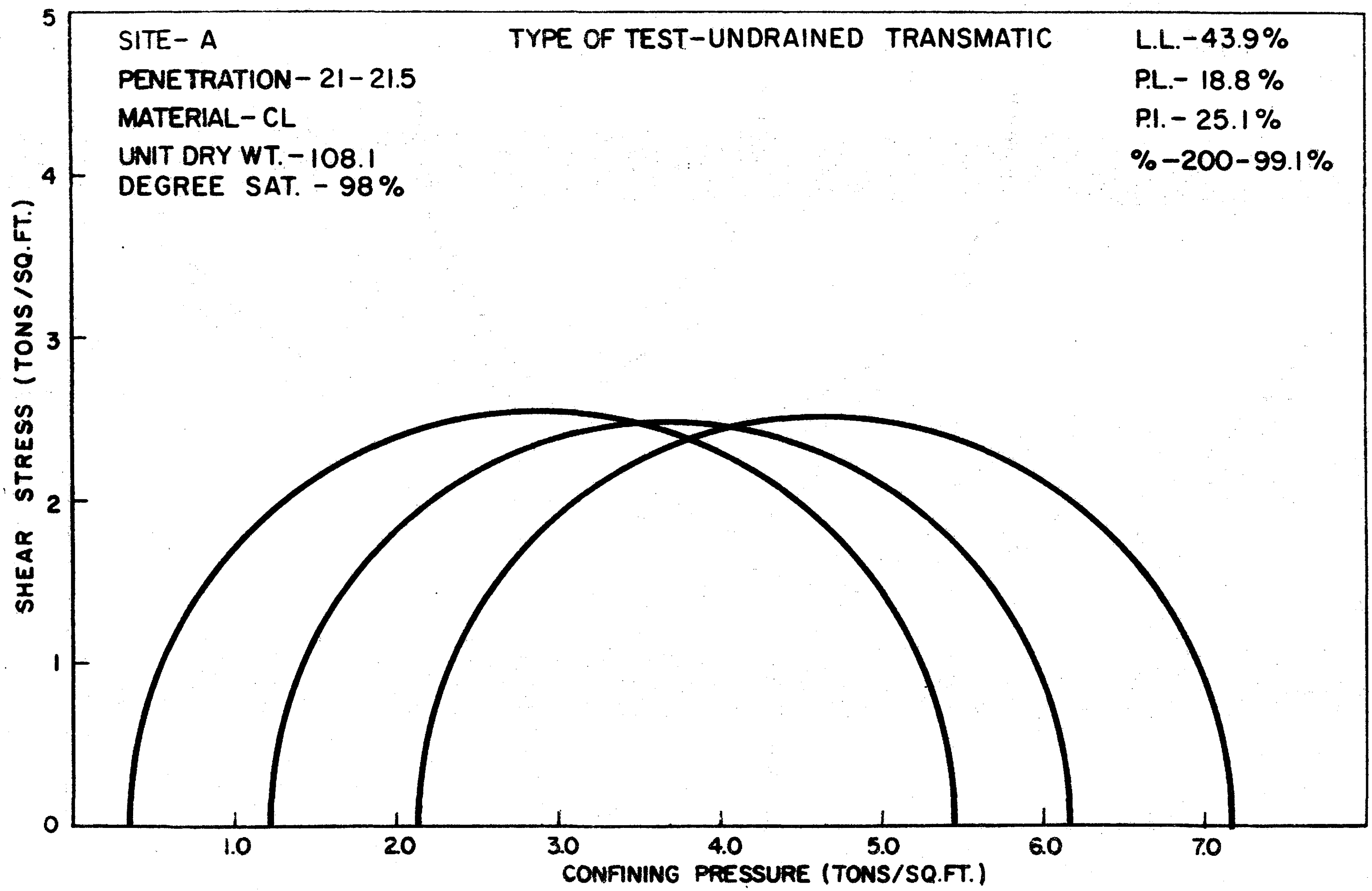

FIG.-IV-3-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-
UNDRAINED MULTIPLE STAGE

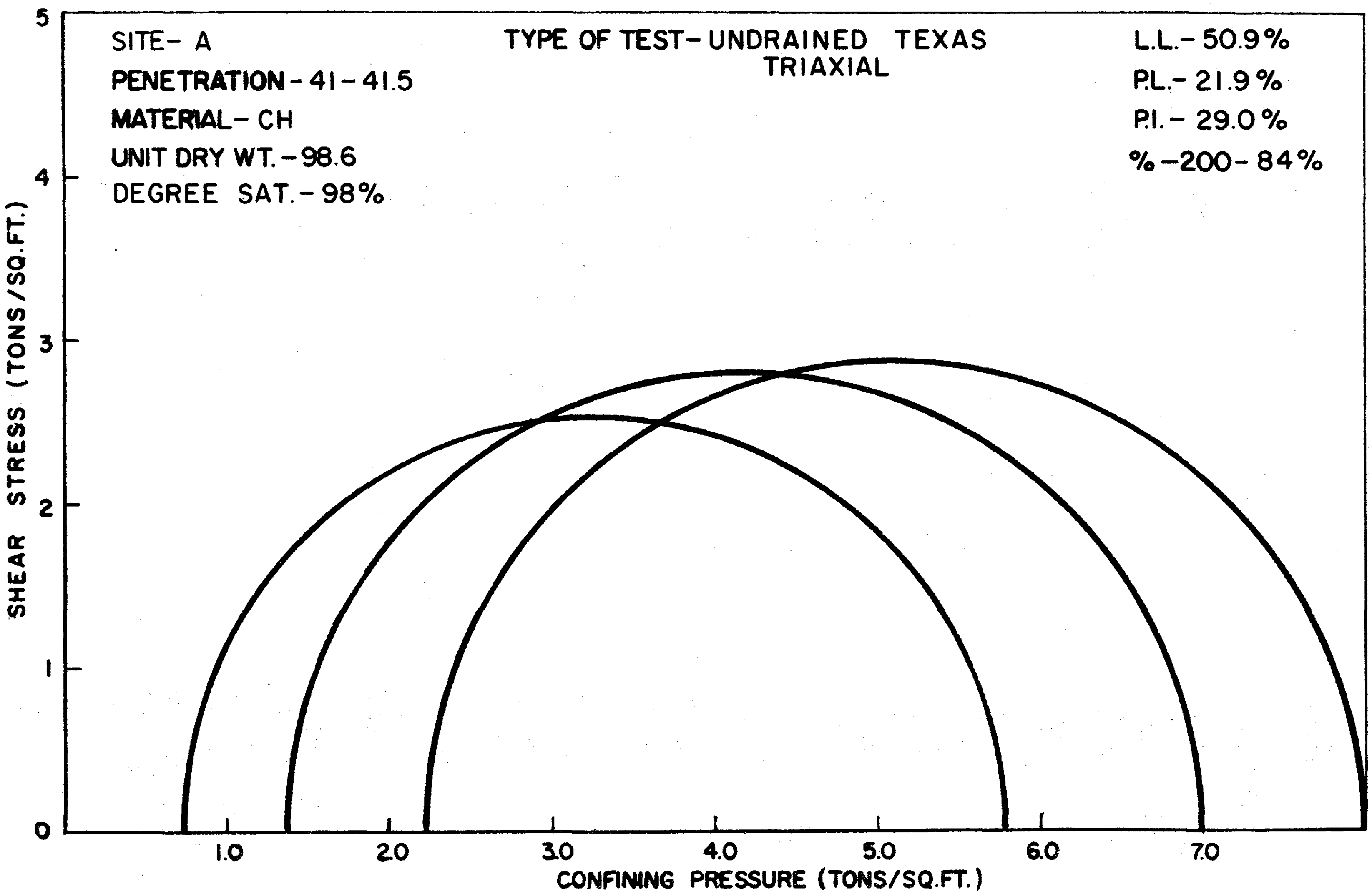

FIG.-IV-4-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

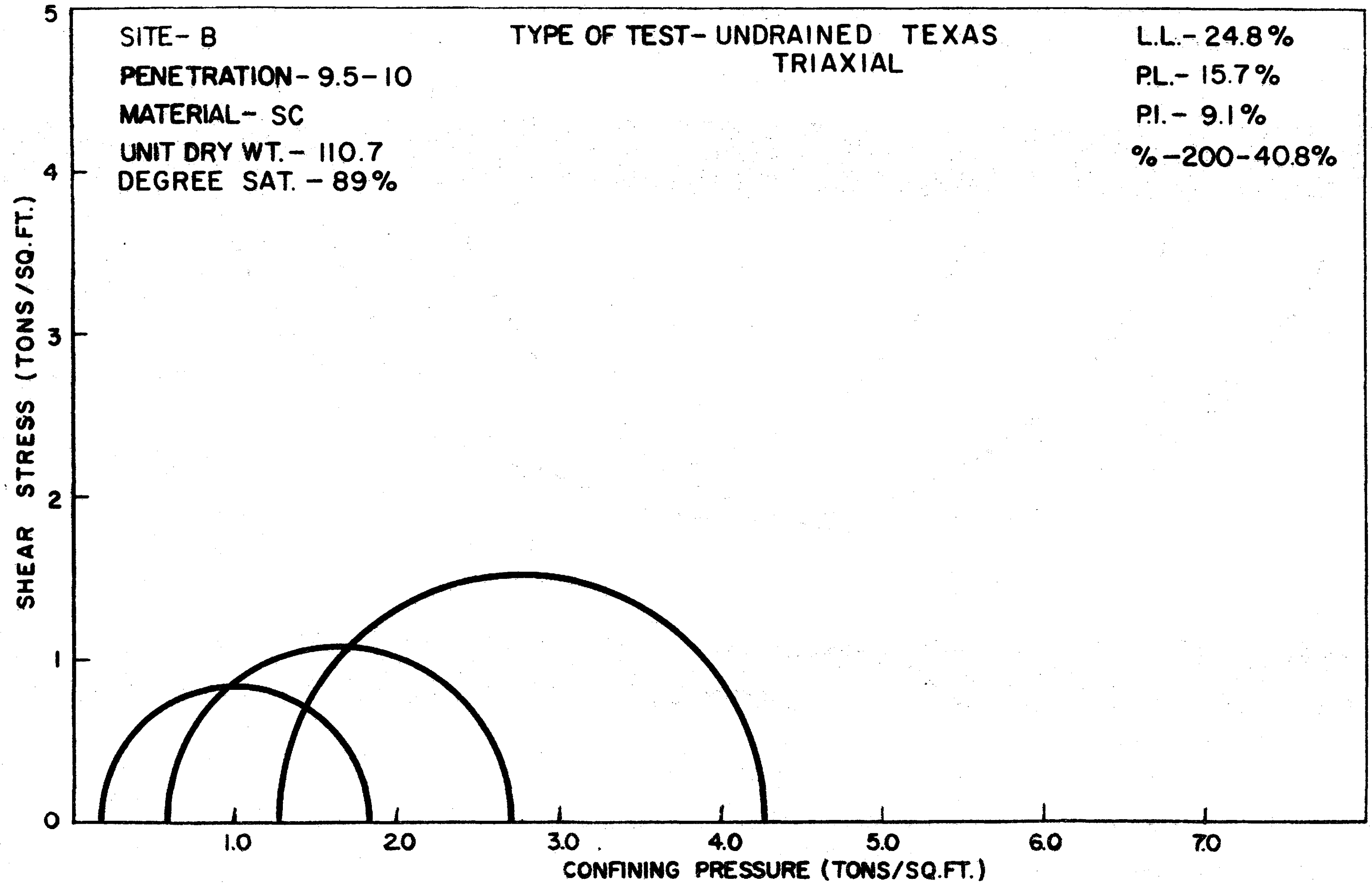

FIG.-IV-5-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

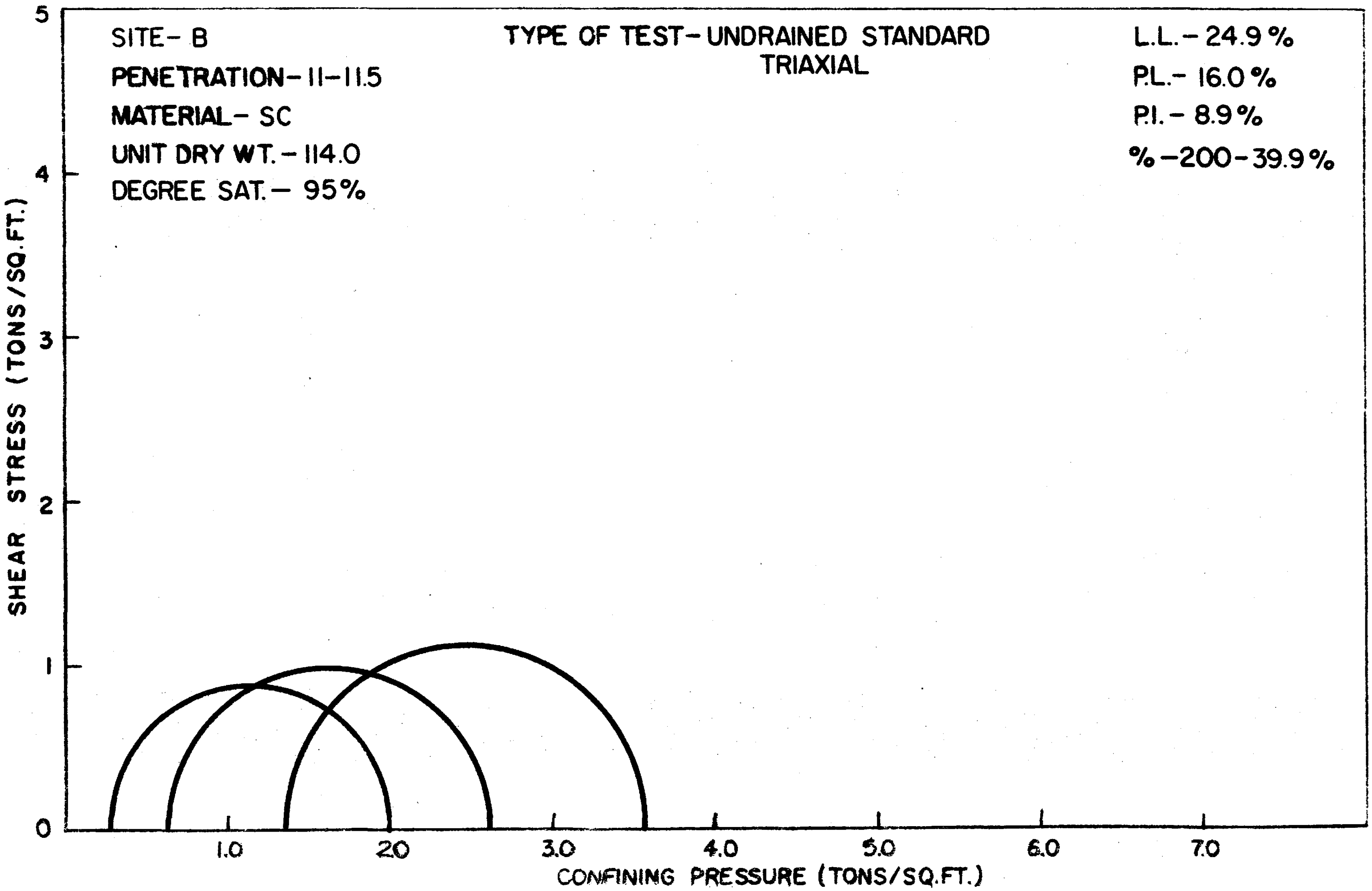

FIG.-IV-6 TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

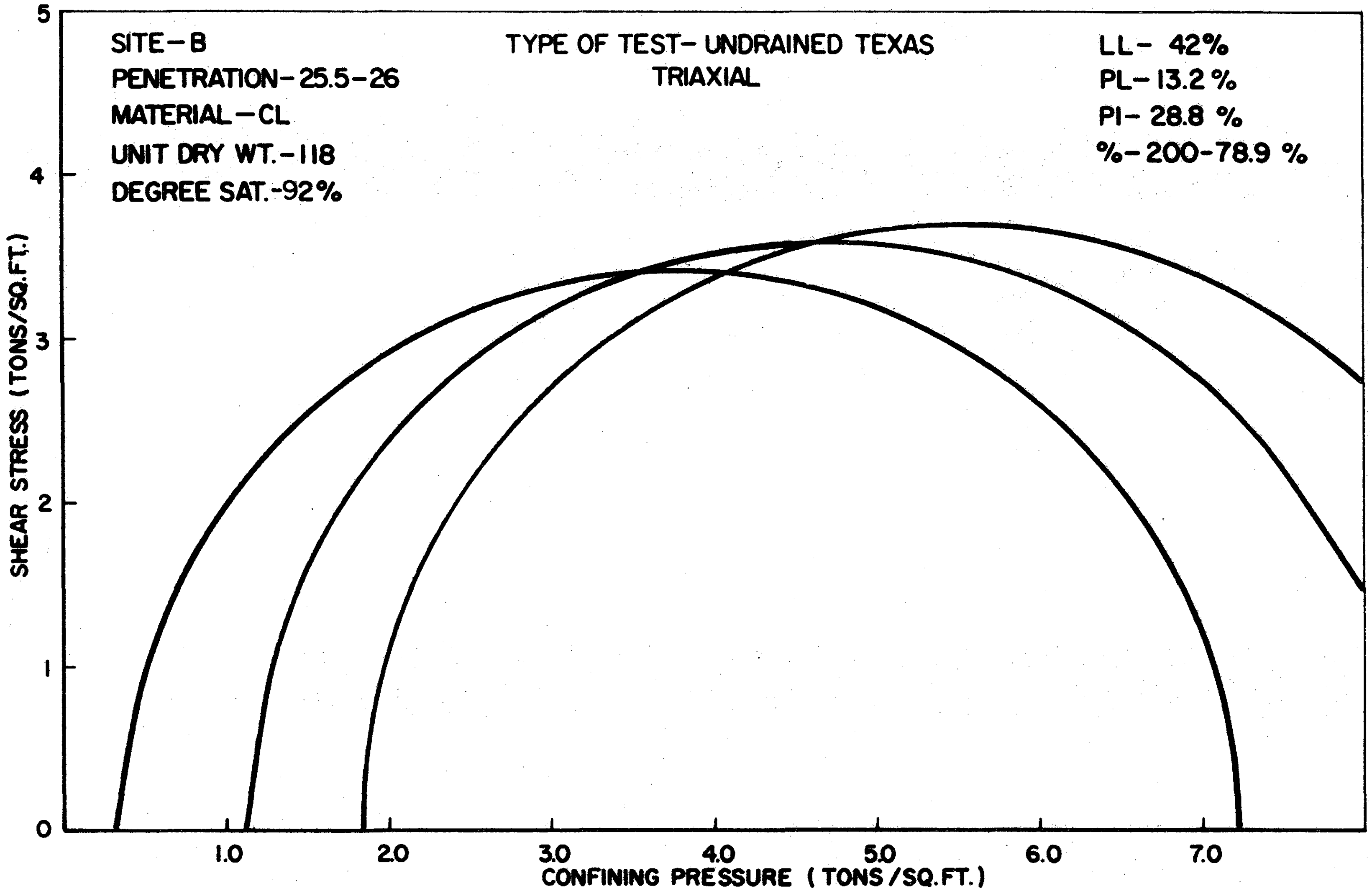

FIG.- IV-7 TRIAXIAL COMPRESSION TEST RESULTS
UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

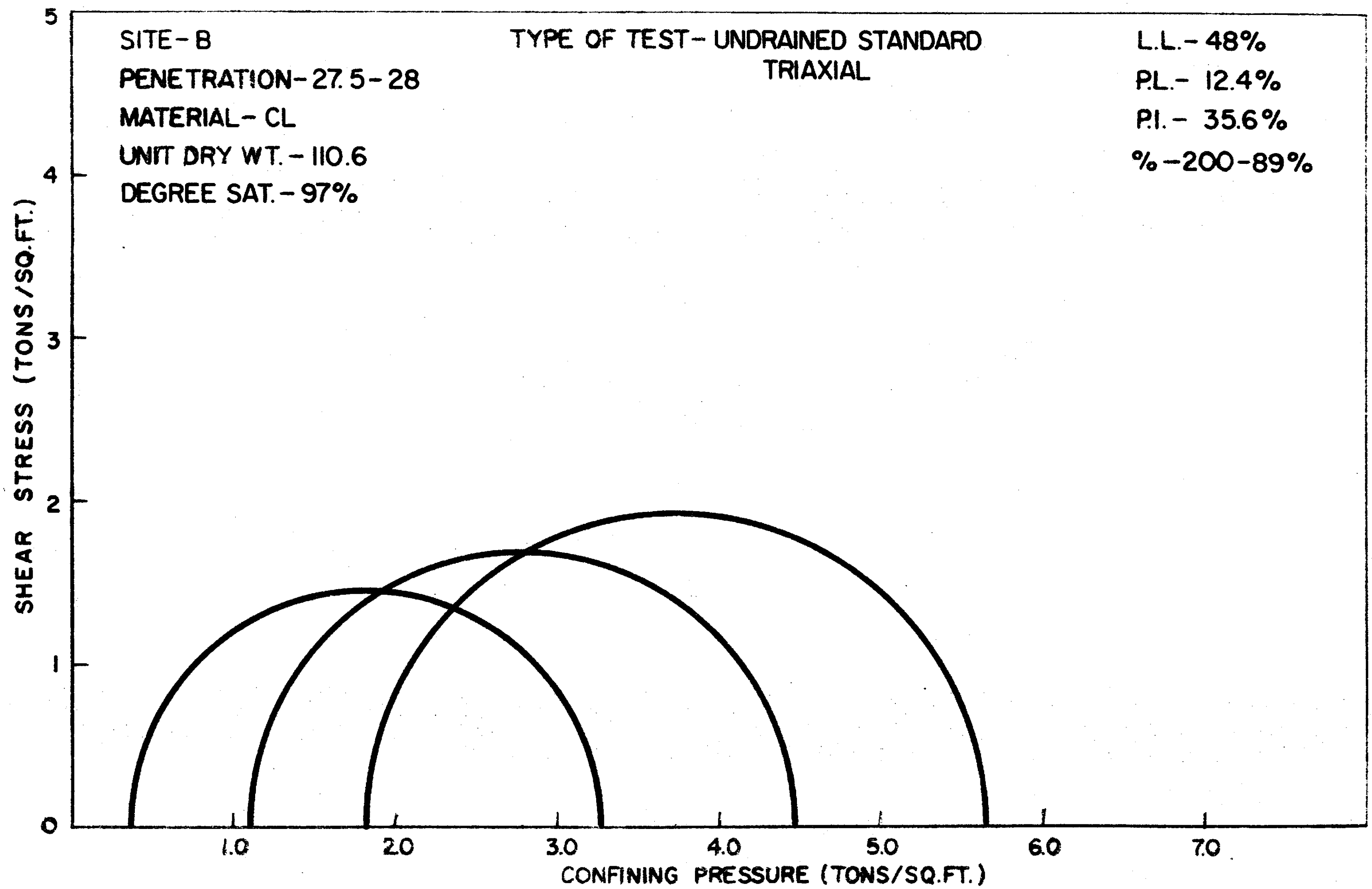

FIG.-IV-8 TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-
UNDRAINED MULTIPLE STAGE

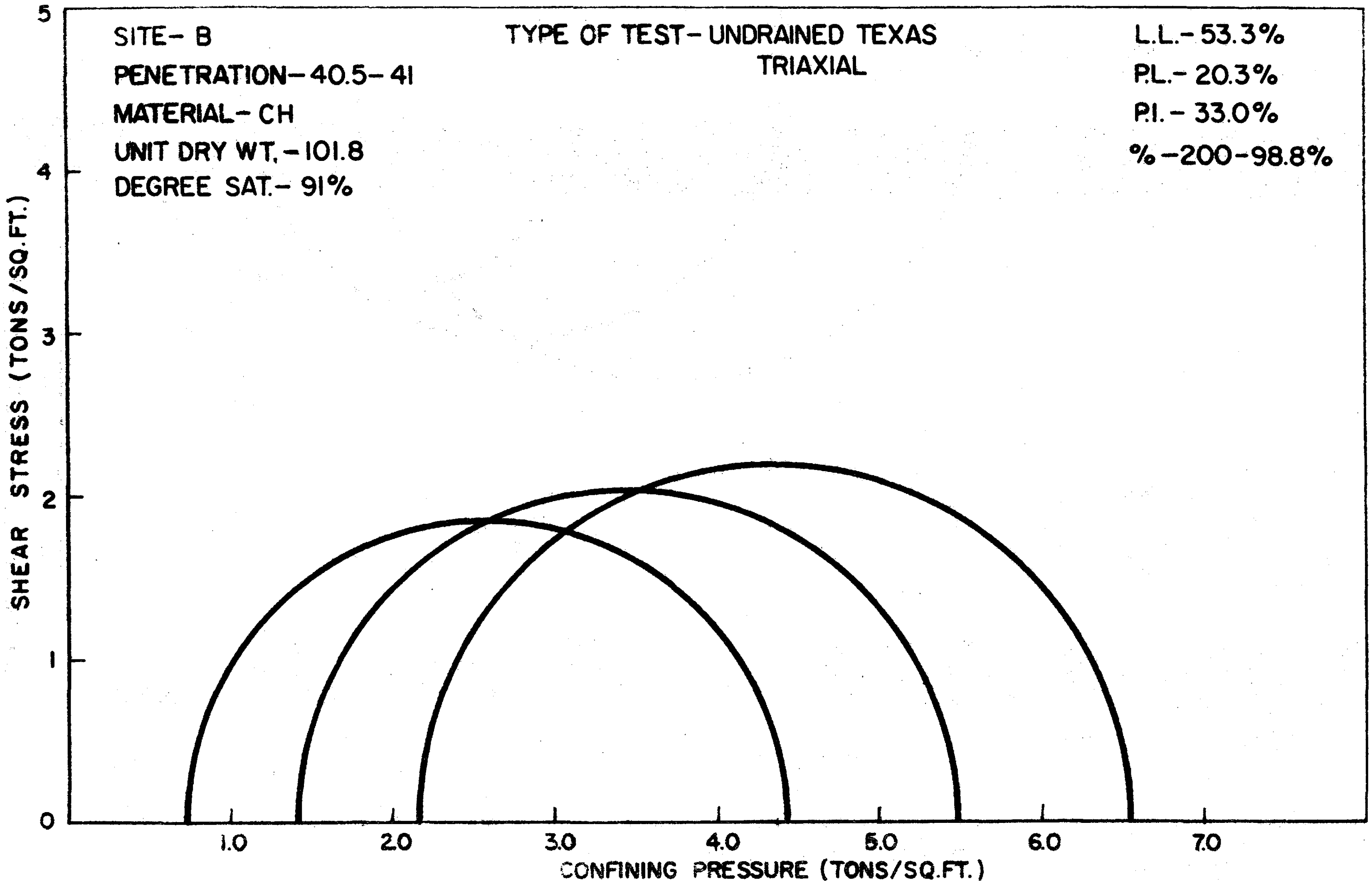

FIG.-IV-9 TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

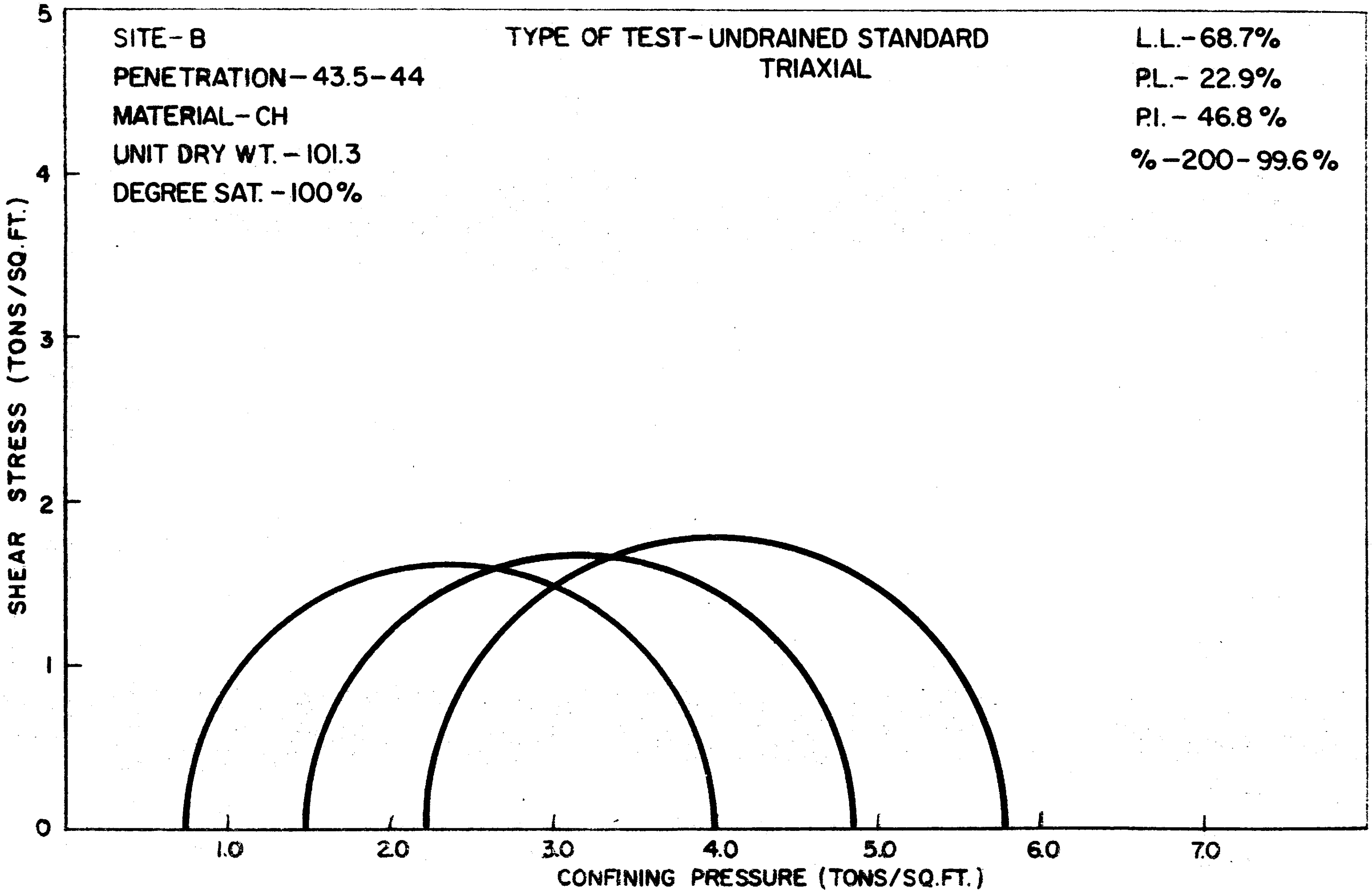

FIG.-IV-10 TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

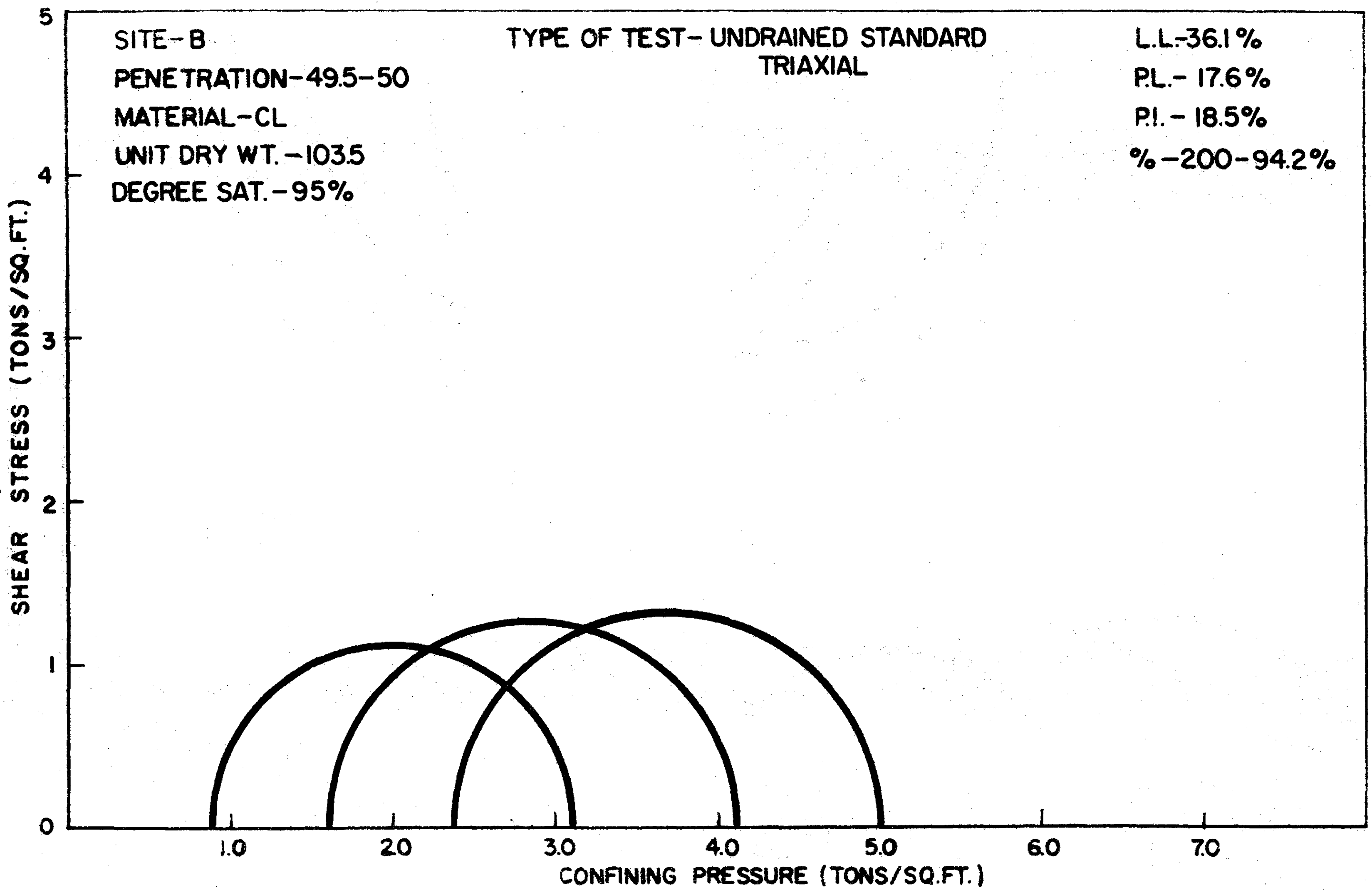

FIG.-IV-11 TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

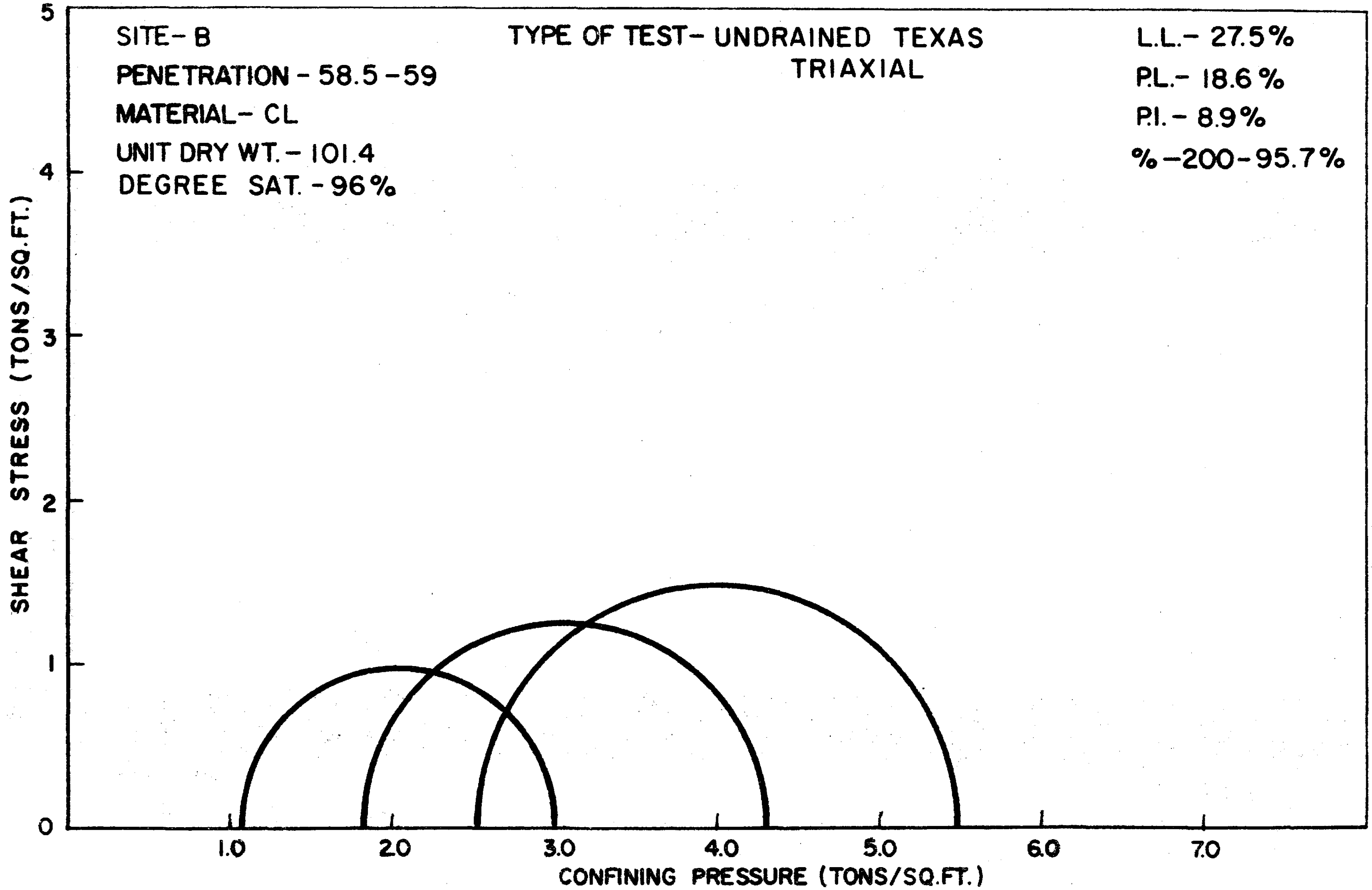

FIG.-IV-12-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

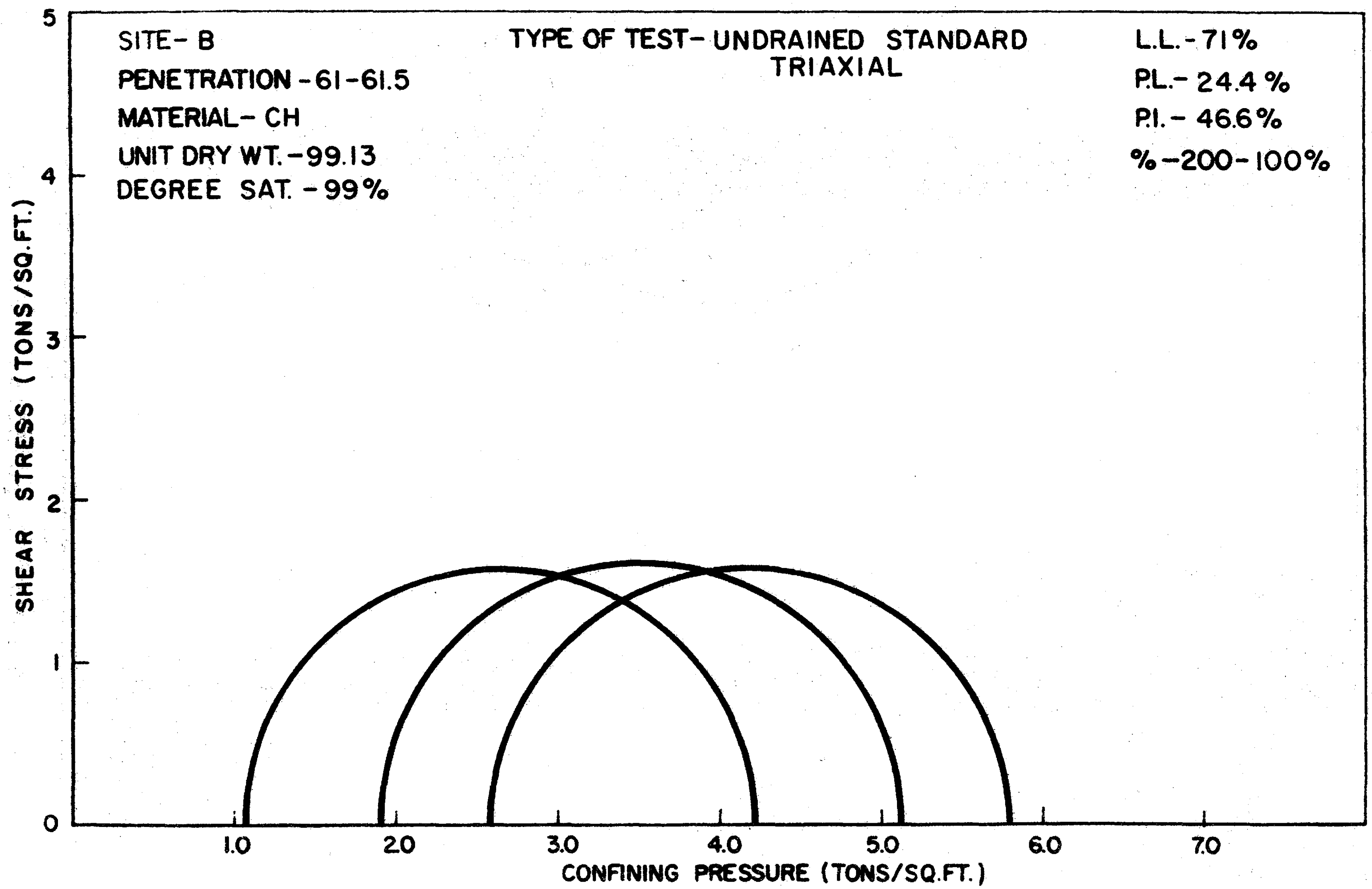

FIG.-IV-13-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

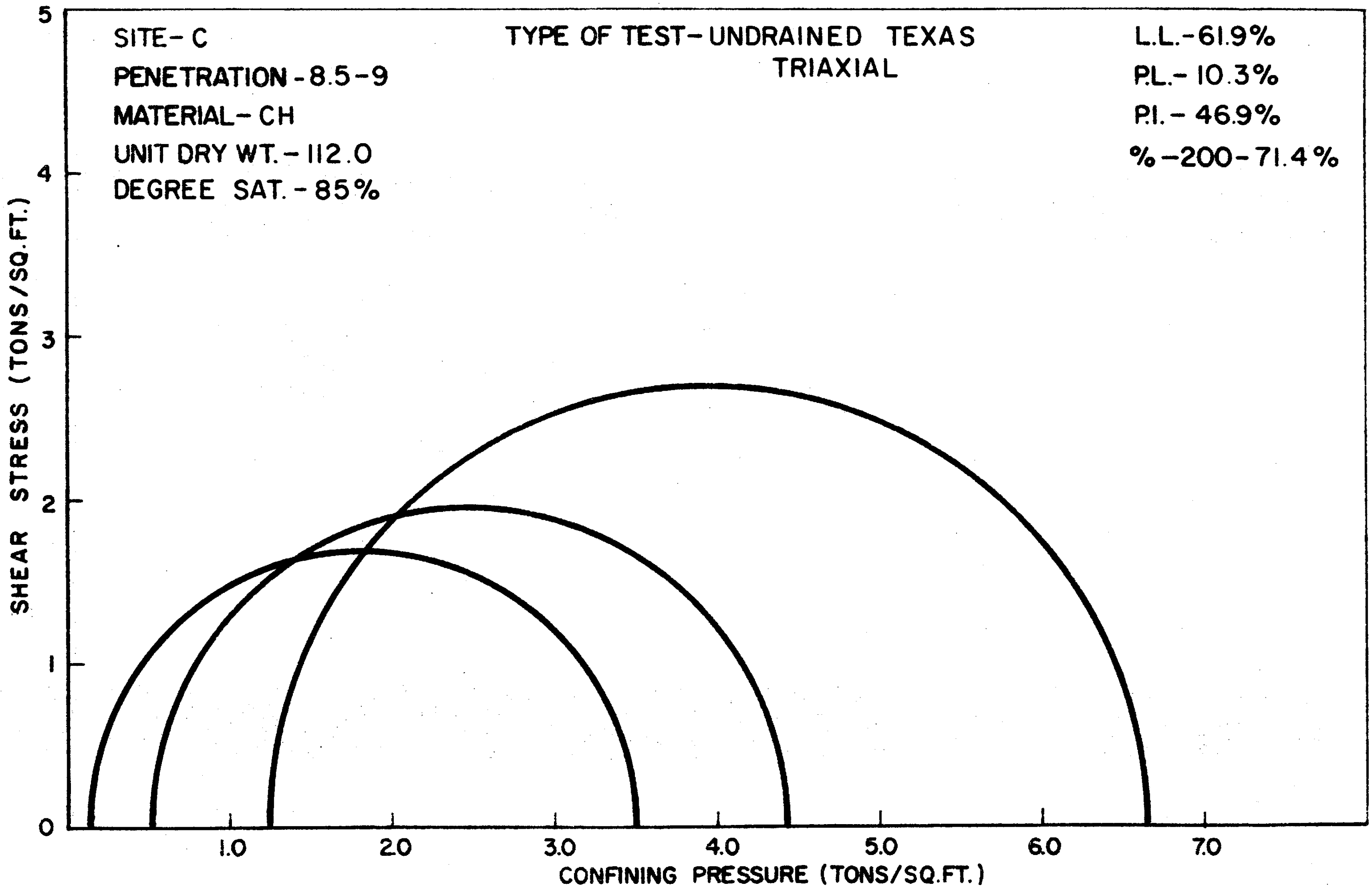

FIG.-IV-14-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

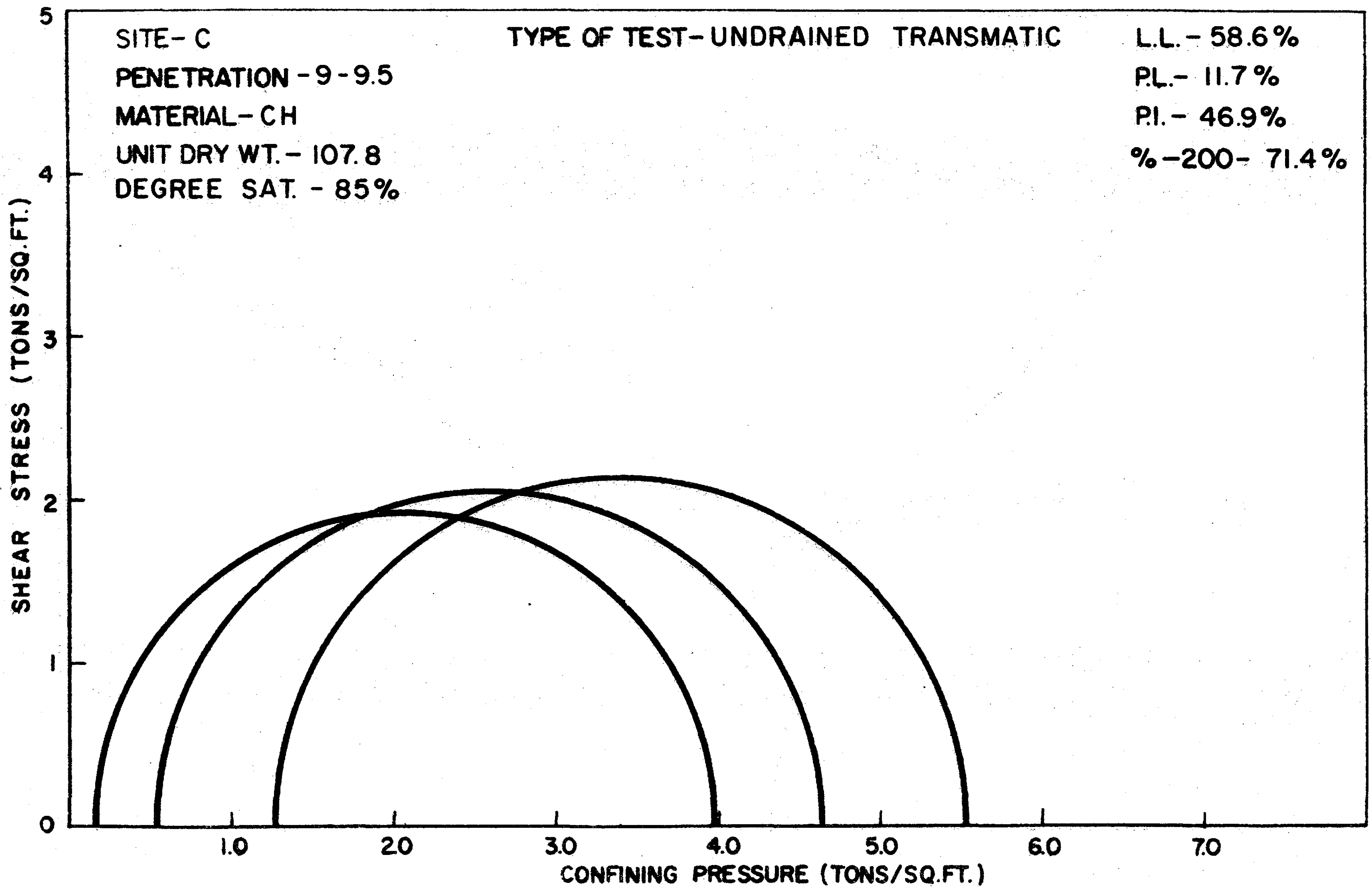

FIG.-IV-15-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

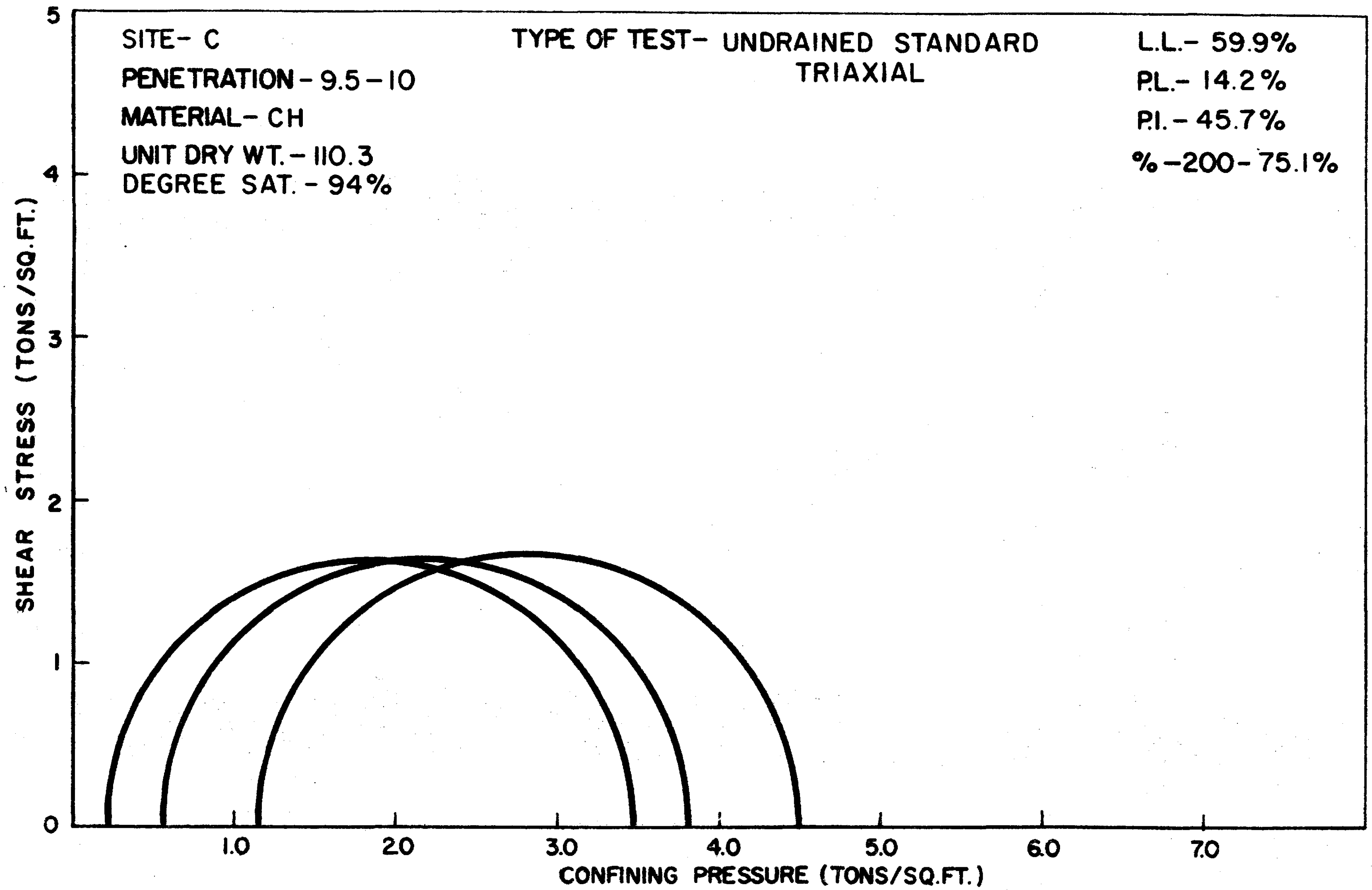

FIG.-IV- 16-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-
UNDRAINED MULTIPLE STAGE

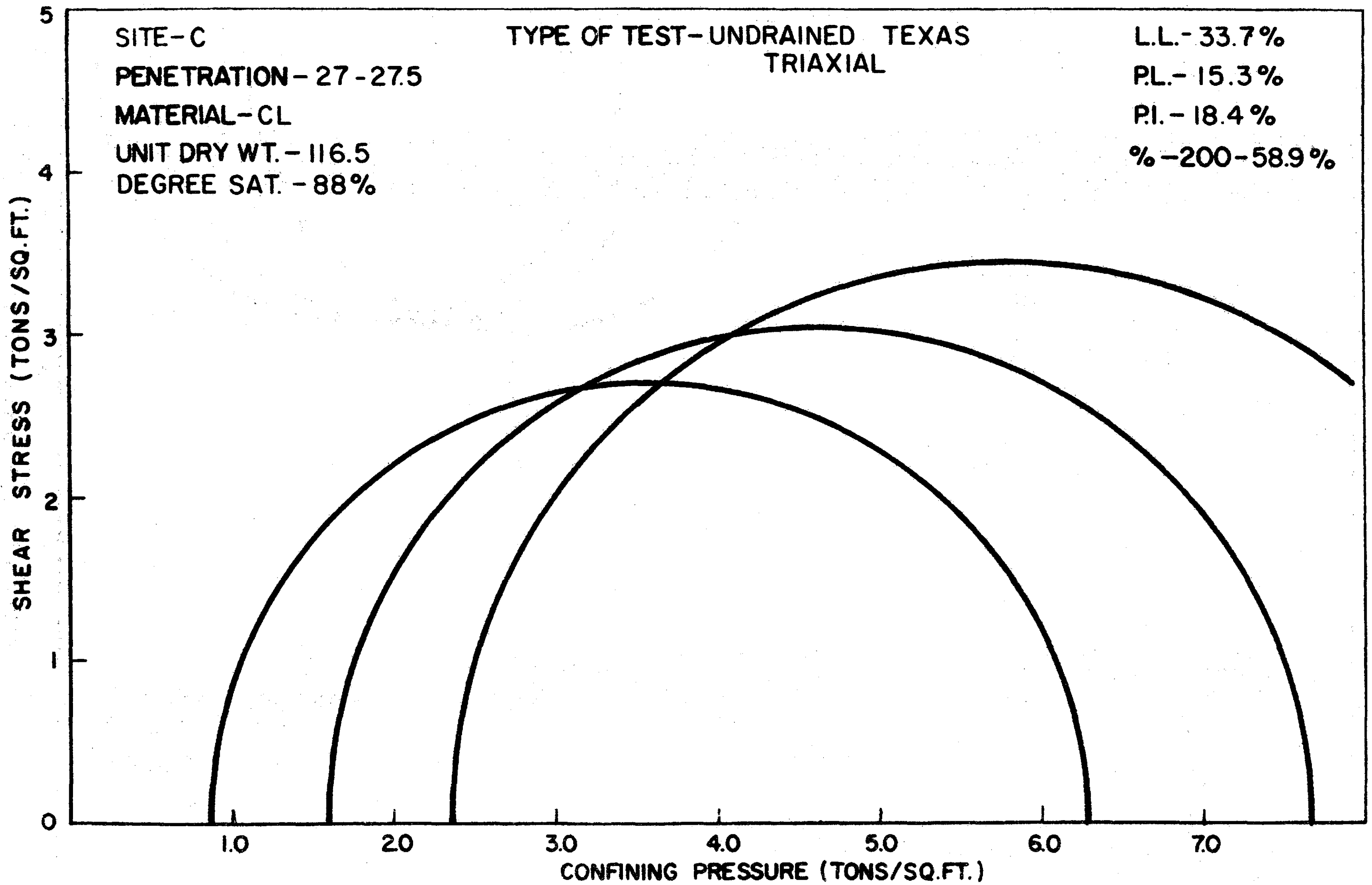

FIG.-IV-17-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

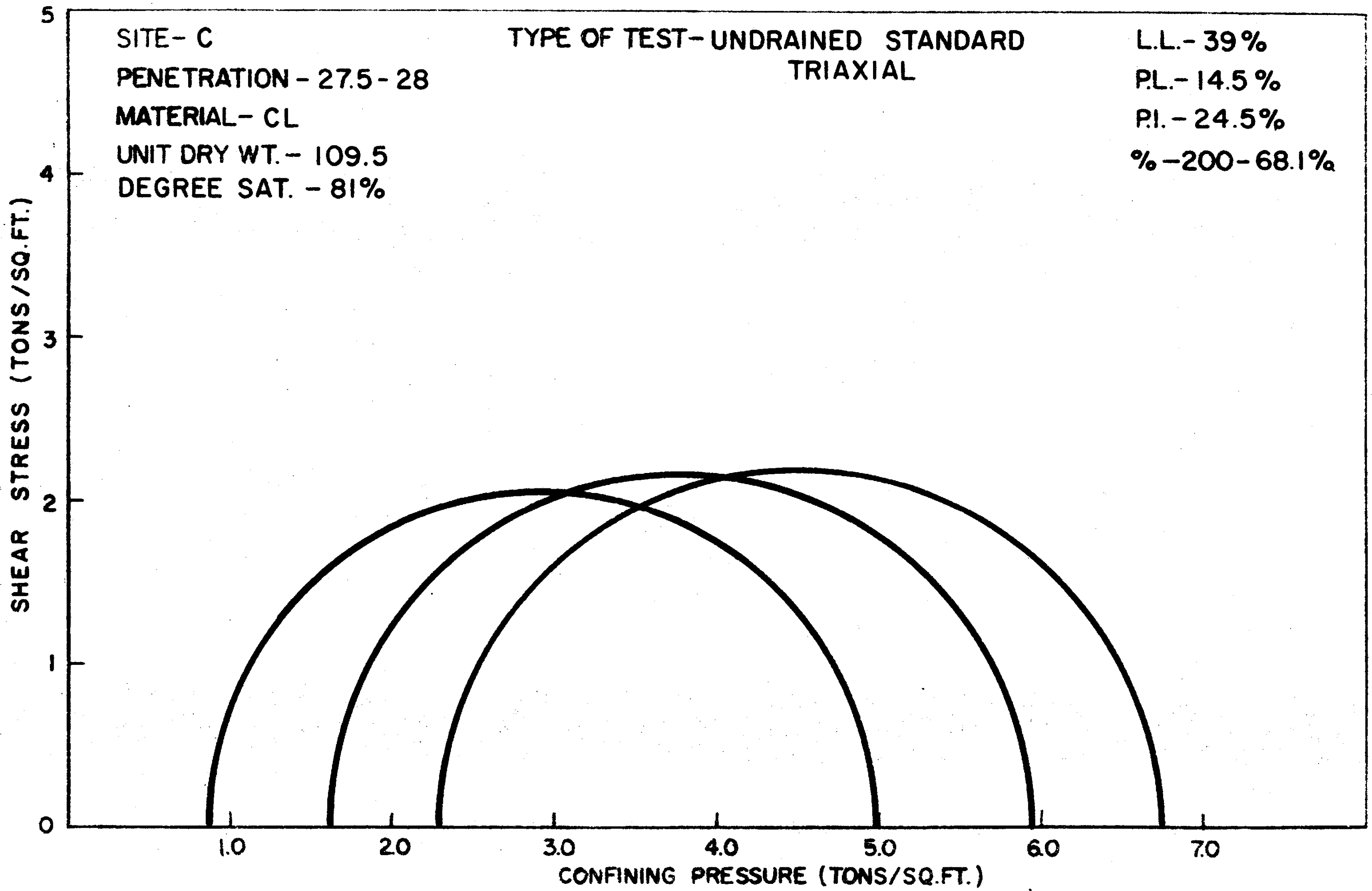

FIG.-IV-18-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

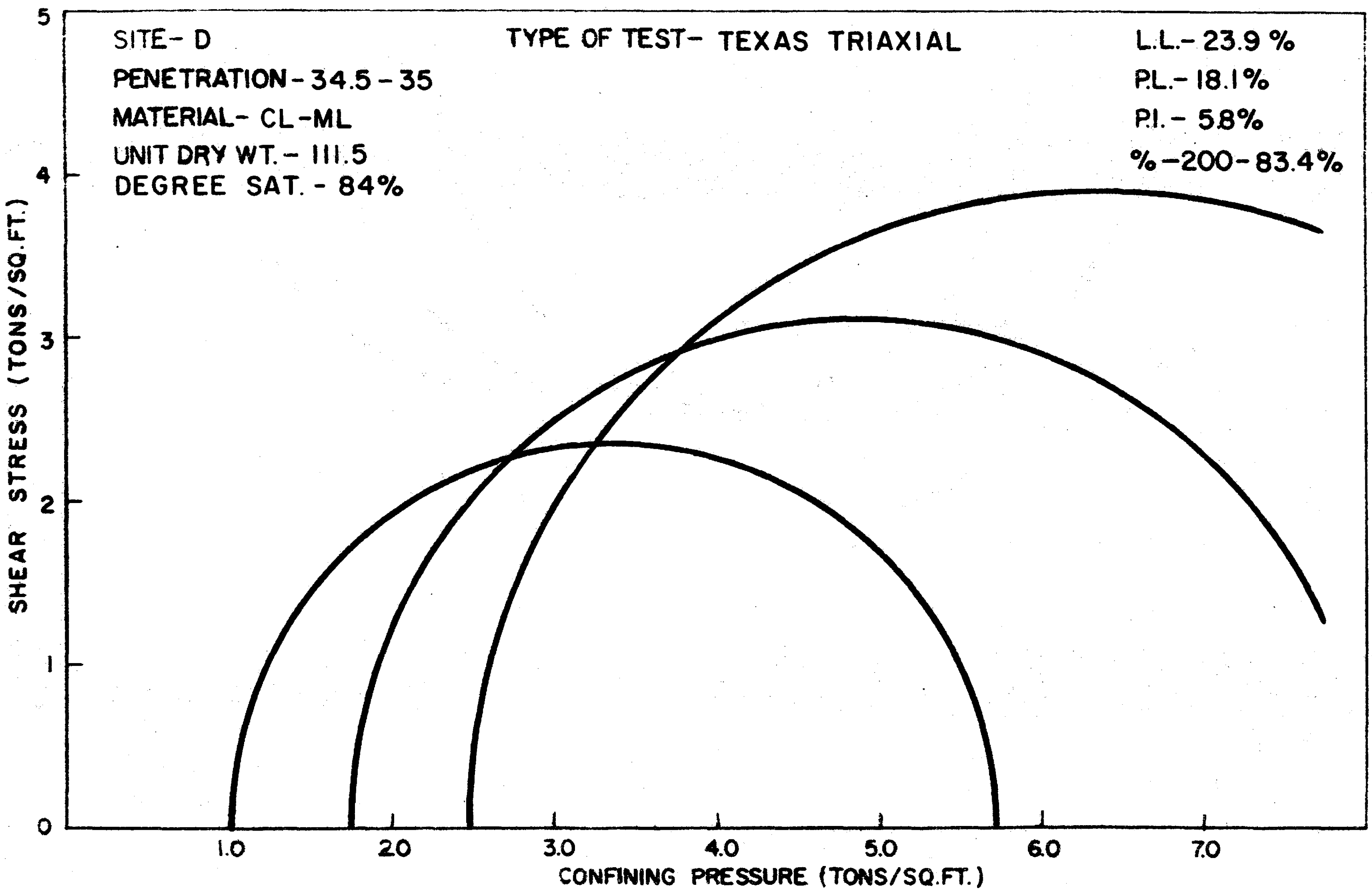

FIG.-IV-19-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

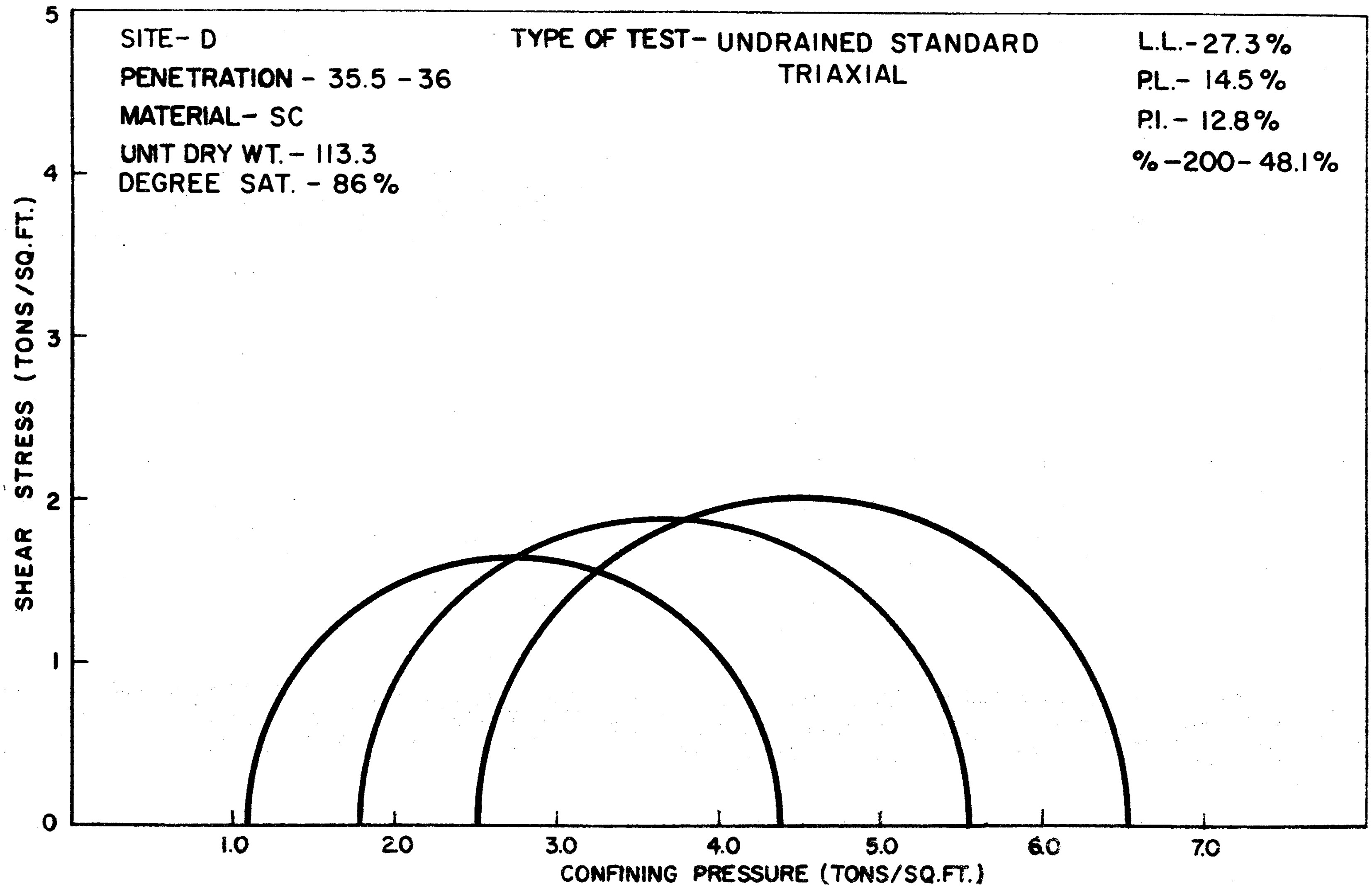

FIG.-IV-20-TRIAXIAL COMPRESSION TEST RESULTS UNCONSOLIDATED-UNDRAINED MULTIPLE STAGE

END OF REPORT