TENSION PILE STUDY

VOLUME I

SITE INVESTIGATION AND SOIL CHARACTERIZATION STUDY AT BLOCK 58 WEST DELTA AREA GULF OF MEXICO

Report Number 82-200-1

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Report
to
CONOCO NORWAY, INC.

through
DET NORSKE VERITAS
Oslo, Norway

* * *

by
ERTEC, INC.
Houston, Texas

April, 1982

Ertec

Ertec
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April 30, 1982
Project No. 82-200

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Attention: Mr. Tore J. Kvalstad

TENSION PILE STUDY
CNRD 13-2

Volume I
Site Investigation and Soil Characterization
Study at Block 58A, West Delta Area

Gentlemen:

In accordance with the contract between Ertec, Inc. and Det Norske Veritas submitted herein is the first of a series of reports concerning the Tension Pile Study, CNRD 13-2, currently in progress. This report presents complete documentation of the proposed Gulf of Mexico field test site. The information and analyses reported herein were derived from results of Task 2, Site Investigation and Laboratory Testing, activities.

Also included, as Appendix A, are field and laboratory results from tests performed by McClelland Engineers, Inc. concurrent with Ertec's program. The combination of in situ and laboratory data provides stratigraphic, physical property, and soil strength information required to interpret results of small and large diameter pile load tests to be performed at the offshore site.

This report constitutes a milestone in our scheduled program to improve the understanding of pile-soil interaction resulting from static and cyclic tensile loading. If there are any questions regarding the contents of this report, please contact us.

Sincerely,

G. Leon Holloway, P.E.
Staff Engineer

Thomas K. Hamilton, P.E.
Project Engineer

GLH/TKH:sac

Distribution: (8) Det Norske Veritas
(2) Conoco, Inc.
Attention: Mr. J. H. C. Chan
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TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

TABLES

ILLUSTRATIONS

ILLUSTRATIONS (continued)

INTRODUCTION

General

This report presents descriptions of the site specific information regarding the general subsurface conditions at a proposed offshore pile test site. The site is located at an decommissioned CAGC platform in Block 58 of the West Delta area, Gulf of Mexico. This program is part of a larger study with the overall objective of improving the understanding of pile-soil interaction during cyclic tensile loading, such as the loading which is expected to be produced by a deepwater Tension Leg Platform (TLP).

The site investigation was conducted to verify the suitability of soils at the location for use as a test medium for studying tension pile foundations in soft clay. The results of the detailed site investigation and laboratory testing program, and the discussion of the site stratigraphy and soil properties serve to fully document the site for future analysis of pile test data.

The overall project objectives and a brief background of the events leading to the recommendations for a load testing program are discussed below.

Project Objectives

The principal objectives of this project were defined during ~~of~~ several meetings held between the parties listed below:

- Conoco Norway, Inc.
- Conoco Research and Development
- Conoco Production Engineering Services
- Det Norske Veritas
- Ertec, Inc.

During these meetings, preliminary information was reviewed concerning the characteristics of deepwater soils with respect to the potential influence on pile behavior. It was concluded that the present foundation design technology does not adequately address the special problems of Tension Leg Platforms where piles are loaded cyclically in tension. Since it was decided that an economical level of confidence could not be achieved using conventional site investigation, laboratory testing, and design procedures, a pile load test should be performed. The following objectives were identified:

- Evaluate pile "set-up" characteristics,
- Determine the potential loss of frictional capacity due to degradation of soil strength during cyclic loading,
- Improve the basis for axial capacity estimation and safety factor selection, and
- Provide data for calibration of improved analytical models.

Background

Discussions were held with representatives of Conoco during January and February, 1981, regarding the need for an improved understanding of TLP foundations. A field testing program and companion analytical study was proposed by Ertec, Inc. to investigate the problems affecting TLP foundations. The initial task, presented in Ertec Proposal No. P81-332 dated February, 1981, was to conduct a preliminary planning study for a subsequent engineering program.

Later meetings with Conoco, Ertec and Det Norske Veritas, who had previously and independently proposed a laboratory study for TLP foundations, evolved into a joint proposal for a comprehensive Tension Pile Study. The principal investigators were to be Det Norske Veritas (laboratory) and Ertec, Inc. (field).

Authorization to begin the planning study was received from Conoco in June, 1981. The results of this study are given in the following documents:

1. Tension Pile Planning Study
   Subproject CNRD 13-1
   Final Report
   by Det Norske Veritas
   Report No. 80-0587; 23 August 1981

2. Final Technical Report
   Subproject CNRD 13-1
   by Ertec, Inc.
   Report No. 81-204; 28 August 1981

These reports specified the tasks to be performed to complete the engineering study for TLP foundations. The study included four primary parts as follows:

1. Laboratory study of model instrumented piles,

2. Field study and in situ tests on small-diameter instrumented pile segments,

3. Field study using a large-diameter instrumented test pile, and

4. Analytical development of an improved soil-pile model with calibration to be performed from the results of the previous laboratory and field tests.

The report presented herein is the first volume of a series of reports describing the results of project activities.

TEST SITE SELECTION

General

A search for a suitable pile load test site at onshore and offshore locations in southern Louisiana and the Gulf of Mexico revealed five potential locations. The general requirements on which the test site selection was based were as follows:

1. Test stratum homogeneity,
2. Soil type and stress history,
3. Stratum thickness, and
4. Operational considerations.

After an extensive review of the published data for each potential site, three were eliminated from the list. Of the two remaining, one was an offshore site and the other an onshore site. Further evaluations of these two candidate sites and a trade off study revealed that the offshore site was the more desirable. The advantages of this site are considerable compared to the other prospective sites and are itemized below:

1. Soil conditions appear to be very similar to the proposed offshore TLP sites.
2. The continuous, homogeneous stratigraphy would allow installation of a long test pile with a relative stiffness comparable to that of prototype piles.
3. Representative in situ total pressures and pore pressures normal to the pile wall can be measured due to the capability of using long test pile.
4. The existing platform would supply the reaction needed for testing. Therefore, the free-field excess pore pressures which would have been created (and would have remained partially undissipated) from the driving of reaction piles at an onshore site would not be a problem.
5. The availability of the offshore platform and Conoco's deck lifting frame would simplify the requirements for the loading system.

The site selected to perform the large and small scale testing is located in Block 58, West Delta area of the Gulf of Mexico. A location map showing the general area of the offshore test site is presented on Plate 1. The water depth at this location is approximately 53 feet. A generally homogeneous clay stratum extends from the seafloor to 253 feet. The strength of the cohesive material at the site varies linearly from a very soft to a stiff greenish gray clay. These deltaic clays are highly plastic with accumulations of methane gas and are categorized as Recent Mississippi River deltaic deposits. A more detailed interpretation of the geologic sequence for the delta region is presented in the following section.

Geology of the Mississippi Delta Region

The Mississippi, the largest river system in North America drains an area of 3.3 million km^2. The annual sediment discharge has been estimated at 6.2×10^8 metric tons with the suspended load characterized by 65 percent clays and 35 percent silt and very fine sand. The coarse material is deposited at or near the distributary mouths because of rapid effluent deceleration and salt water entrainment as it leaves the mouth of the distributary. The fine-grained sediment is kept in suspension and spreads laterally far beyond the immediate mouth of the channel. The wide lateral dissemination of fine grained sediment has built a platform fronting the delta that consists of clays which were rapidly deposited. These clays have an extremely high water content and because of the abundant fine grained organics, which are rapidly degraded by bacteria, large accumulations of sedimentary gas are present.

In the early stages, the delta was initiated at the head of the present alluvial valley near Cairo, Illinois. At the lower end of the plain is the post glacial deltaic complex of numerous distributaries consisting of old abandoned mouths and the present modern day bird-foot delta. In the past 7000 years, the Mississippi River has constructed a broad deltaic plain composed of several large, small and often overlapping depositional lobes (Handley, 1980 and Kolb et al, 1966). Within the last 5000 years there have been at least seven of these deltas built by the Mississippi River as the river has alternated its depositional forces while prograding further southward in the Gulf. An illustration of this southward migration of delta lobes is shown on Plate 2.

The modern bird-foot, or Balize, delta is the longest lobe of the Mississippi River deltaic formation. Radiocarbon dating indicates that the delta lobe has formed within the past 600-800 years (Fisk, et al, 1954). The area of the subaerial bird foot delta is 1900 km^2, compared to an average aerial extent of 6200 km^2 of the older delta lobes (see Plate 2). The confinement of the modern delta to a small area has been compensated for by the expansion of its vertical thickness. The past 3000 years has seen the mouth of the river progress from west of the proposed test site to the present location, which is almost due east of the West Delta block. This eastward migration and associated depositional process has influenced the stratigraphy at the site as will be noted in subsequent sections.

The delta front configuration indicates the presence of several topograhphic zones based upon slope and roughness. The uppermost zone is the narrow shallow platform of very low slope (less than 1 percent slope). This low slope zone is the region where the Block 58A structure stands and where the tests will be conducted. From 15-62 m (50-200 ft) water depth, is a zone of rough irregular topography made up of closely spaced ridges and gullies. The overall slope within this zone is a 1-3 percent slope, but locally it can be greater. The lower zone of the delta front has a smoother topography of broad valleys and higher terraces and is marked by an area of surface scarps carved by numerous deep-seated faults. This lower zone delimits the delta or continental shelf, from the continental slope at approximately 200 m (656 ft).

A study performed by Trabant in 1978 correlated air-gun seismic data with engineering borehole data. This has permitted siesmic-stratigraphic analysis of portions of the Mississippi Delta Front. From this study three prominent reflectors have been mapped throughout most of the survey area by this seismic stratigraphic technique (Payton, 1977).

A lithologic and geotechnic transect across the Delta area is shown in Plate 3. The lower most reflector, "A", has been correlated with a gray silty fine sand. This reflector is believed to represent a middle or late Wisconsin transgressive phase (rise in sea level). Reflector "B" has been correlated with a sandy clay/clayey sand unit containing shell fragments. This reflector is believed to represent the initial Holocene rise in sea-level. The clay strata between seismic reflectors "A" and "B" have been described as firm gray clays having shear strengths in the range of 40 to 100 KPa (0.84 to 2.1 KSF). These values indicate normally to slightly underconsolidated sediments for this unit.

Reflector "C" has been correlated with the top of a "shell hash" unit. This shelly clay unit has not been well defined on the basis of borehole lithologic descriptions. It does, however, offer a conspicuous seismic reflector throughout most of the delta front. Bea and Bernard (1973) have interpreted this unit as the base of the modern delta clays, and as a "glide plane" upon which the recent deltaic deposits ride downslope. Reflector "C" generally appears to correspond to a change in geotechnical properties where shear strengths increase from values of less than 20 KPa to over 40 KPa (0.42 to 0.84 KSF) and sediments exhibit a slightly underconsolidated character (Shepard et al, 1978). This slightly underconsolidated sediment is present at the test location site.

In terms of time, the stratigraphic units may be dated by either eustatic changes in sea level (Curray 1969) or in terms of absolute geochronologic measurements. Radiocarbon dates indicate an age of approximately 17,000 years before present (YBP) for stratigraphic unit C (shell hash). Ages of 30,000 YBP and older have been obtained from carbonate shells within unit B (Fisk 1971; Bea and Bernard 1973). An Isopach map (Coleman and Suhayda, 1979) shown in Plate 4 represents the recent deposits overlying the strand plain sands of pleistocene age. In regions adjacent to the modern delta lobe, the thickness of recent deposits averages 50 meters, but in the immediate vicinity of the modern delta lobe, sediments thicken considerably. No direct geochronologic ages have been determined for the transgressive phase which produced seismic stratigraphic unit A. The absolute age for this unit on the basis of available paleo-climatic data and eustatic changes in sea level is not available due to conflicting data (Sidner et al, 1977).

DETAILED SITE INVESTIGATION

General

A detailed site investigation program was performed adjacent to structure A in Block 58 of the West Delta area, Gulf of Mexico. A plan of the general boring location is presented in Plate 5. This site is located west of the present Mississippi River Delta in a water depth of 16.2 m (53 ft). The program was initiated to determine the site specific soil parameters for comparison and evaluation in order to fully understand the results of subsequent field tests on both large and small diameter test piles. These results would then be utilized in arriving at a set of design criteria for tension leg platform foundations.

The geotechnical site investigation was planned by Ertec, Inc. with the field work contracted to McClelland Engineers. The onsite work was performed from November 4 to November 12, 1981. The investigation involved three separate borings to fully characterize the soil at the test site. Each boring consisted of a different mode of testing or sampling. However, due to some bad weather on several occasions, two of the borings were aborted before being completed to the required termination depth. These borings were successfully completed later in the nine-day period after weather conditions improved.

McClelland Engineers had previously performed three borings in Block 58 of the West Delta area. Therefore, to maintain consistency with the nomenclature used for those earlier borings, the borings conducted for this study were designated as 4, 5 and 6.

The site investigation program was performed as follows:

1. Boring 4/1* - Continuous cone penetrometer testing from 3.7 - 69.5 m (12-228 ft).

   Boring 4/2 - Push sampling using 76.2 mm (3 in) tubes at 0.91 m (3 ft) intervals from 69.5-73.2 m (228-240 ft).

2. Boring 5/1 - Push sampling using 76.2 mm (3 in) tubes at 1.52 m (5 ft) intervals from 0-17.4 m (0-57 ft).

Boring 5/2 - Push sampling using 76.2 mm (3 in) tubes at 1.52 m (5 ft) intervals from 10.7-69.8 m (35-229 ft).

Boring 5/2 - Continuous cone penetrometer testing from 69.8-77.1 m (229-253 ft).

3. Boring 6/1 - Alternate push sampling using 76.2 mm (3 in) tubes with remote vane at 3.05 m (10 ft) intervals from 6.1-31.1 m (20-102 ft).

Boring 6/2 - Alternate push sampling using 76.2 mm (3 in) tubes with remote vane from 24.4-74.4 m (80-244 ft).

* The boring label indicates first the boring designation followed after the slash by the consecutive number of set-ups.

In a few instances, an insufficient quantity of material was collected from the push samples. Therefore, additional samples were taken at 1.52 m (5 ft) intervals in Boring 6. Also, as a check for continuity between the borings, samples were taken from the areas where overlap occurred in Borings 5 and 6.

Operations

The M/V "R.L. Perkins" was utilized in this investigation. It was positioned about the platform by setting the two bow anchors and tying soft line from the stern to the existing structure. Once a boring was completed, the lines were loosened and the anchor winches used to "crab" the ship onto a new location without resetting the existing anchors or soft lines.

The most desirable side of the platform to conduct the site investigation was littered on the sea floor with remains of drilling and production equipment. This was discovered by divers who performed a bottom survey prior to the site investigation. During this survey the divers also marked the existing pipelines in the immediate area so that the position of the pipelines would be known during anchoring operations.

Water depth was determined at the beginning of Boring 4 (first location for this study) on 5 November 1981 using an electronic seafloor sensor lowered through the drill string. It was recorded as 16.2 m (53 ft) and rechecked before drilling each new boring. At no time did the reported water depth vary more than 0.15 m (0.5 ft). Corrections for tidal variations during drilling and sampling were not performed since tides in the Gulf of Mexico generally vary less than 0.3 m (1 ft).

Drilling and Sampling

Drilling and sampling was performed using a skid mounted Failing 2000 rotary rig operating through a center well in the deck of the M/V "R.L. Perkins". The borings were drilled with 114.75 mm (4-1/2 in) IF drill pipe through which 64 mm (2.5 in) OD liner samples and 76 mm (3 in) OD push samples were taken. A heave compensation system was used to control the vertical motion of the drill string during sampling and in situ testing operations.

Liner samples were taken to a depth of 11.3 m (37 ft) at which time the strength of the soil was sufficient so that normal push samples could be obtained from this depth forward. The push samples were taken with a latch-in sampler. The sampler operates on the technique of pushing a 76 mm (3 in) OD thin-walled tube into the soil by latching the sampling tube into the drill bit and using the weight of the drill pipe to advance the sample tube into the soil. This procedure operated trouble free and very high-quality samples were obtained.

After recovering each soil specimen, drilling fluid and cuttings were cleaned from the top of the sample tube. The soil was then classified at both the bottom and top of the sample tube. Miniature vane and residual miniature vane tests were performed as well as Torvane tests. The majority of the samples were sealed in the sample tubes for shipment to the various laboratories. In some cases, the samples were extruded to assure sample quality was being maintained.

In Situ Testing

General. - In addition to the soil sampling program, two types of in situ tests were performed. The tests were performed using McClelland's Swordfish (cone penetrometer) and remote vane systems. These were required to verify the homogeneity of the test stratum and to provide additional soil shear strength data. These tests are discussed in greater detail in the following paragraphs.

Cone Penetration Tests. - Continous Cone Penetrometer Tests (CPT) were performed to verify the continuity and homogeneity of the test stratum. The Swordfish system uses a hydraulic ram to push a standard cone (60°apex, 10 cm^2 base, and 150 cm^2 friction sleeve) into the soil below the drilled depth of the boring. The tests were performed in accordance with procedures outlined in ASTM D-3441-75, using a penetration rate of 2 cm/sec. During penetration, the cone resistance and sleeve friction are plotted in analog form and also recorded directly into a combined amplifier/digitizer/memory unit.

The soft nature of the soil at the test site required the boring to be advanced to 3.7 m (12 ft) before sufficient lateral stability was provided to operate the tool safely. The edited results of the cone penetrometer test data are presented in Appendix A. The editing of the cone log consisted of subtracting the hydrostatic head developed at the bottom of the borehole and removing the "shoulders" caused by drilling disturbance at the start of each stroke.

Interpretation of the cone log indicates 1) there were no unusual formations or thin layers of dissimilar soil present in the stratigraphy and 2) the soil generally exhibited a smooth linear increase in resistance versus penetration. This is significant in selecting future locations for instruments for the pile load test. If zones of material had been found which were not representative of the stratigraphy, instrumentation at these levels would have been avoided. A detailed discussion of the CPT results is presented in the section on general site conditions.

Remove Vane Test. - Twenty-two Remote Vane tests were performed as part of the geotechnical site investigation. These tests were performed not only to provide additional soil shear strength information but also to provide an indication of the degree of disturbance suffered by the recovered samples due to volumetric expansion from total stress relief and dissolved gases coming out of solution.

In situ vane tests were made in Boring 6 from 7.32 m (24 ft) to 74.39 m (244 ft) below the seafloor and at approximately 3.05 m (10 ft) intervals. The wire line operated vane is first pushed 0.91 m (3 ft) to 1.52 m (5 ft) into the soil below the bottom of the borehole. The four-bladed vane is then rotated at 18°/min until soil failure occurs. During rotation the torque required to shear a cyclindrical surface of soil is measured and recorded on a strip chart recorder. This torque is subsequently converted to shear strength of the soil.

The results of the remote vane test are summarized in Appendix A and are presented in a later section of this report. The test data obtained from these tests may prove to be some of the most reliable indications of in situ soil shear strength. The strengths given from the remote vane tests were generally 30 to 50 percent higher than those measured using the miniature vane in the laboratory.

LABORATORY TESTING PROGRAM

General

A geotechnical laboratory testing program was undertaken by Ertec to determine the subsurface conditions at the test site 1) for use in the interpretation of the pile load test results, and 2) to aid in the development of parameters for the extrapolation of the results to TLP design. The laboratory tests were performed on soil samples obtained from the test site during the detailed site investigation described in the previous chapter. Laboratory tests were performed at the following laboratories:

1. Ertec Western, Inc., Long Beach, California
2. McClelland Engineers, Inc., Houston, Texas
3. Det Norske Veritas, Oslo, Norway
4. Norwegian Institute of Technology, Trondheim, Norway
5. Norwegian Geotechnical Institute, Oslo, Norway

The results of the test performed at Ertec and McClelland laboratories are summarized and integrated in this report to provide a clear characterization of the test site. Particular emphasis was given to characterizing the soil over the instrumented section of the future test pile. Therefore, the majority of the laboratory work was concentrated above 67 m (220 ft) with some additional testing extending beyond this limit. The laboratory program consisted of three categories of testing as follows:

1. Classification Tests,
2. Physical Property Tests, and
3. Strength Tests.

These categories are further broken down into the individual tests performed on specimens and will be discussed in more detail in the following sections of this report. A brief description of the tests procedures are presented in Appendix B with a tabulation of the tests performed presented in Appendix C and on the boring log, Plate 6. A key to soil classification and symbols is presented on Plate 7.

Sample Selection and Preparation

Selection of undisturbed samples for strength and physical property testing was made from both radiographic and visual examinations of the soil in each sample tube. Upon arrival in our Long Beach laboratory, X-rays of each sample were taken to check for disturbance within the tube. The radiographs of each sample tube were mapped and areas along the tube which indicated questionable sample quality were noted on description sheets. These sheets were then used to identify areas of least disturbance and marked as candidate areas for use as test specimens.

Each candidate specimen was extruded and visually examined with comments regarding consistency, color, and texture being noted on the sample description sheet. Any visual disturbance was also noted and compared to areas which were previously noted on X-ray logs as being questionable. If the quality of the test specimen was found to be good, and the soil type met the requirements for a specified test, the test specimen was prepared for testing. This consisted of trimming the specimen to an appropriate diameter and length before being weighed. Moisture content tests were performed on the trimmings while the remaining material was retained in a water tight container for possible future use. To obtain as much comparative information as possible regarding the soil at a particular depth, it was sometimes necessary to use specimens from consecutive tubes in the same boring or specimens at the same depth from adjacent borings in order to fulfill test requirements.

Classification Tests

A comprehensive program of classification tests (summarized and tabulated in Appendix C) was carried out to evaluate the basic characteristics of the soil at the test site. The following type of tests were included in this category:

1. Natural moisture content,
2. Unit weight,
3. Specific gravity,
4. Atterberg limits, and
5. Hydrometer tests.

Natural moisture content and submerged unit weight determinations were made in conjunction with each strength and consolidation test performed. A plot of the moisture content versus penetration is shown on the boring log (Plate 6). The submerged unit weight of the soil versus penetration is shown on Plate 8. Specific gravity determinations were made for use in calculations of void ratio and degree of saturation. Since most of the soil was classified as clay, Atterberg limit tests were also conducted. Plastic and liquid limit tests were performed on the cohesive samples for use in determining the Plasticity Index (Plate 9). These results are shown on the plasticity chart on Plate 10. The Liquidity Index was also calculated and presented versus penetration below the seafloor on Plate 11.

Physical Property Tests

A series of physical property tests were performed to further characterize the soils at the test site. A limited study of soil compressibility was conducted using one-dimensional oedometer and K_o triaxial consolidation tests. One dimentional consolidation test results are used to determine the stress history of the soil and to obtain the vertical coefficient of consolidation. This coefficient, c_v, will be used for estimating the reconsolidation of the soil mass after installation of the test pile. The K_o consolidation tests were run in conjunction with both the consolidated undrained triaxial compression tests and the consolidated undrained direct simple shear tests. The K_o condition is fundamental to the reconsolidation of specimens according to an anisotopic stress path resembling that which occurs in situ. For practical problems dealing with clays, and where deformations are a concern, laboratory consolidation to in situ K_o stresses is a first requirement for obtaining meaningful stress-strain data.

One-Dimensional Consolidation Tests. - Four consolidation test were performed on representative undisturbed samples obtained from various depths below the seafloor. These tests were conducted in conjunction with strength tests to provide complete characterization of the soil.

data and plotted on a Liquidity Index versus vertical stress curve, Plate 12B. Correlations based on nine soil borings from the Gulf of Mexico (Audibert, et al, 1982) are also shown for comparison.

One implication of this stress history interpretation is that in situ shear strengths are probably lower than would be estimated from commonly used indirect shear strength calculations based on strength ratios for normally consolidated clay and hydrostatic vertical stresses. Further evaluation of the consolidation test results and methods for determining in situ shear strength are given in subsequent sections of this report.

K_o Triaxial Consolidation Tests. - The consolidation characterisitcs of the soil were also evaluated under K_o (anisotropic) conditions. Five tests were carried out using a procedure similar to that described in the literature (Bishop and Henkel, 1957 and Abdelhamid and Krizek, 1976).

In this procedure, a K_o consolidation condition is maintained by imposing values of σ_1 and σ_3 such that no lateral deformation in the sample occurs during consolidation. The ratio, σ_3/σ_1, which produces this condition is considered equal to K_o for the imposed stress conditions. During the tests, several stages of increasing consolidation pressures were used to develop the K_o condition. A plot of K_o extracted from $\overline{CK_oU}C$ and $\overline{CK_oU}DSS$ tests is presented versus penetration on Plate 13.

Estimates of K_o based on plasticity (Ladd, et al, 1977) are also shown on Plate 13. The trend of these empirical correlations compare favorably with laboratory results.

Strength Tests

General. - A series of strength tests were conducted to evaluate the stress-strain, strength and volume change characteristics of the soil for use in the interpretation of the future small and large-diameter load tests results. This program included the following tests:

1. Miniature Vane Shear (MV) Tests,

2. Unconfined Compression (UC) tests,
3. Unconsolidated Undrained Triaxial Compression (UU) Tests,
4. Isotropically Consolidated Undrained Triaxial Compression ($\overline{\text{CIU}}$C) Tests,
5. K_o- Consolidated Undrained Triaxial Compression ($\overline{\text{CK}_o\text{U}}$C) Tests, and
6. K_o- Consolidated Undrained Direct Simple Shear (C$\overline{\text{K}_o\text{U}}$DSS) Tests.

Results of these tests are tabulated in Appendix C (Plate C-2).

Miniature Vane Shear Tests. - A total of 81 miniature vane shear tests (plus 12 residual vane tests) were performed on undisturbed samples before the soil was extruded from the Shelby tubes. The undrained shear strength from these tests are summarized in Appendix A and on Plate C-2 of Appendix C. Values are also plotted on Plate 14 versus penetration below the seafloor. The measured strengths ranged from 1.2 KPa (0.025 KSF) to 93.4 KPa (1.95 KSF).

The measured values of in situ shear strength obtained from McClelland's remote vane were generally higher than from tests performed using the laboratory miniature vane. Since the miniature vane test is performed while the sample is still in the tube, the existing degree of confinement is expected to vary from sample to sample depending on the degree of disturbance and the amount of free gas present. Furthermore, since the vane is embedded only a short distance into the sample, the confining stresses would be limited to the capillary stresses (or residual stresses) existing in the sample. These limitations, together with the well known limitations associated with the mode of shear and strain rate effects (Ladd, 1973 and Schmermann, 1975) often detract from the value of this test.

On the other hand, these tests are inexpensive, can be easily performed in the field prior to suffering distrubance from shipping and handling, and (because of the number performed) can be used to readily identify regions of soil strength variability. Also, API specifies miniature vane shear test results as a method for obtaining parameters for pile capacity calculation.

An interesting point is that the miniature vane tests performed in the field gave higher values of shear strength than the same tests performed later in the laboratory. Two possible explanations for this reduction in shear strength are:

1. Sample disturbance occurring in the soils during transportation to the laboratory, expecially in the softer material, and

2. The long term stress relief occurring in the samples, thus allowing gases to further come out of solution.

Although miniature vane test should certainly be a part of any laboratory program on cohesive soils, the inherent limitations noted show cause for also including more sophisticated laboratory testing techniques. We believe that the Stress History and Normalized Soil Engineering Properties (SHANSEP) technique allows a better definition of the in situ strength of the soil. For this reason, normalized triaxial and simple shear testing was performed on representative samples as part of the comprehensive test program.

Unconfined Compression, (UC), and Unconsolidated Undrained Triaxial Compression (UU) Tests - Six unconfined and twenty unconsolidated undrained triaxial compression tests were performed on test specimens from all three borings at the site. Five remolded UU tests were also included in the laboratory testing program. The results of these tests are summarized in Tables 2 and 3 and in Appendix A. In addition to the strength data, failure strains and other pertinent index properties are included. The stress-strain curves from both the UC and UU tests are presented in Appendix C.

Test results for both types of tests are plotted versus penetration on Plate 15. Although the samples used for UC and UU tests had water contents in the same range, the strengths determined from the UU tests averaged 35 percent greater than the strengths determined from UC tests. Since both types of test specimens were exposed to the same conditions, disturbance cannot be considered to play a major role in the variance of strength between one test and another. Probably the most important contributing factor between differences in strength was the confinement provided by the cell pressure in the UU tests.

Due to the fact that these samples were highly charged with gas, the confinement of the specimens to its original stress before failure led to the higher undrained shear strengths. Thus, it is concluded that the strengths obtained from the UU tests provided a more realistic estimate of the in situ compressive strength of the soils existing at the West Delta test site.

Isotropically Consolidated Undrained Triaxial Compression ($C\overline{IU}C$) Tests. - Seven static $C\overline{IU}C$ tests were performed on undisturbed samples of the West Delta site clay. The test specimens were consolidated in the laboratory to obtain normally consolidated and overconsolidated specimens with known overconsolidation ratios (OCR). Specimens were tested with OCRs of 1 and 2. The procedure for consolidation followed the guidelines of the SHANSEP approach presented by Ladd and Foott (1974). The use of this procedure requires that the soil be amenable to normalized testing methods. As a minimum requirement for establishing this prerequisite, a number of normally consolidated specimens must be tested at different consolidation stress levels. The soil may be assumed to follow normalized behavior if the normalized shear strength, S_u/ σ'_{3c}, is independent of the consolidation stress level. Once it has been established that the soil exhibits normalized behavior, the SHANSEP approach may be used to evaluate profiles of in situ undrained shear strength.

Another requirement for the successful application of the SHANSEP approach in design is to have an accurate assessment of the in situ maximum past pressure, σ'_{vm}. This is necessary to determine the maximum pressures required for consolidation of test specimens in the laboratory to minimize sample disturbance. Usually, σ'_{3c} is set at 1.5 to 2.0 times σ'_{vm}. Otherwise, the maximum consolidation stress used in the laboratory may be too low, and test results may still be affected by sample disturbance.

The maximum past pressure of the West Delta site was estimated from consolidation tests performed on samples obtained by pushing a thin-wall tube into the ground. Even though this technique has been proven to give higher quality samples than driven samples, some disturbance naturally occurs from stress relief and during the transportation of the tubes. Individual specimens were selected after the tubes were X-rayed and the least disturbed portion of the sample identified. This procedure has previously been discussed in the section on sample selection and preparation. The in situ maximum past pressures, estimated on the basis of consolidation tests and other empirical procedures, was reported to be between 34.5 KPa (0.72 KSF) and 167.7 KPa (3.5 KSF) for the four depths tested in the Ertec laboratory test program (Plate 12A). To insure that the samples were consolidated into their virgin range, a value of the maximum consolidation stress was selected to be at least three times σ'_{vm}.

The results of the $\overline{CIUC}$ tests are summarized in Table 4 together with index properties, consolidation stresses, OCRs, shear strength ratios and failure strains. Normalized stress-strain curves and normalized excess pore pressures versus axial strain curves are presented in Plates 16 and 17. The normalized effective stress paths are presented on Plate 18.

Four tests (as shown in Table 4) were performed on normally consolidated specimens and three tests performed on specimens with overconsolidation ratios of 2.0. Three of the four normally consolidated $\overline{CIUC}$ tests produced shear strength ratios ($S_u/\ \sigma'_{3c}$) ranging from 0.26 to 0.29. These specimens were all from segments of the stratigraphy which were classified as CH material. The $\overline{CIUC}$ test performed on the specimen classified as CL with and OCR of 1.0 yielded a shear strength ratio of 0.34. The tests performed at OCR = 2 resulted in values of 0.35 and 0.36 for the clay with high plasticity and 0.59 for the lower plasticity material.

The effective stress paths for seven $\overline{CIUC}$ tests are shown on Plate 18. It is interesting to note the difference in behavior between the tests performed on CH specimens and the CL material. For the normally consolidated CL sample (Sample 61), the mean effective stress decreases to a minimum value and then increases slightly. The normally consolidated CH specimens all showed a continuously decreasing mean effective stress. The overconsolidated (OCR = 2) sample of CL material (Sample 76) failed at a mean effective stress greater than its initial condition, whereas the CH samples increased above their initial mean effective stress but finally failed at a value less than or equal to σ'_{3c}.

The failure envelope drawn on Plate 18 indicates an effective friction angle, ϕ' of 25°. The stress-strain and pore pressure-strain curves are plotted and presented in Appendix C on Plates C-15 through C-22.

The normalized undrained shear strengths versus the logarithm of the overconsolidation ratio from the seven tests performed on "undisturbed" samples are presented on Plate 19. Normalized strengths from $\overline{CIUC}$ tests on soft clays published in the literature (MIT 1969, Koutsoftas and Fischer 1976) are also shown for comparison. It can be seen that the normalized undrained strengths of the West Delta test site are approximately within the range of values reported in the literature.

<u>K_o - Consolidated Undrained Triaxial Compression ($\overline{CK_oUC}$) Tests</u>. - Three $\overline{CK_oUC}$ tests were performed as part of Ertec's laboratory test program. The tests were performed over a representative range of the boring at depths of 12.4, 24.8 and 54.9 meters (40.5, 81.3 and 180.1 feet). Two additional $\overline{CK_oUC}$ tests were performed by McClelland engineers. As part of the requirements for this test, the sample was initially consolidated to a K_o-condition. This was achieved by applying consolidation stresses in small increments. The axial and lateral stresses were monitored and adjusted as necessary to maintain equal axial and volumetric strains, thus achieving a condition of zero lateral strain during consolidation. This condition is, by definition, the requirement for K_o-consolidation.

Normalized stress and normalized pore pressures versus strain plots are presented in Plates 20 and 21. Normalized effective stress paths are presented on Plate 22. Table 4 gives a summary of the Index properties, consolidation stresses, OCR's, shear strength ratios and failure strains for the $\overline{CK_oUC}$ tests. The stress-strain and pore pressure strain results are presented in Appendix C on Plates C-23 thru C-25.

Normalized strengths are plotted versus the logarithm of the overconsolidation ratio in Plate 19 together with normalized shear strengths obtained from $\overline{CIUC}$ tests. The shear strength ratios obtained from the $\overline{CK_oUC}$ tests are very close to the values obtained from $\overline{CIUC}$ tests at corresponding OCRs. It is important to note that in general, soft clays are known to exhibit normalized undrained shear strength ratios S_u/σ'_{vc}, for both $C\overline{K_0U}C$ and $C\overline{IU}C$ tests (Ladd, 1965, Donaghe and Townsend, 1978). From Plate 19, it can be deduced that the behavior of the West Delta site clay is apparently consistent with the behavior of soft clays for which the SHANSEP approach is known to have been used successfully in design.

K_0 - Consolidated Undrained Direct Simple Shear, ($C\overline{K_0U}DSS$) Tests. - Six CK_0UDSS tests were performed on specimens to obtain a measurement of the soil shear strength and stress-deformation characteristics. These tests were reconsolidated in the laboratory to obtain normally consolidated and overconsolidated specimens with known OCRs using the procedures described by Ladd and Foott (1974). Detailed test procedures are presented in Appendix B with the test results illustrated in Appendix C.

Table 5 summarizes the results of the monotonic simple shear tests at both $(\tau_h)_{max}$ and $(\tau_h/\sigma'_{vc})_{max}$. The normalized shear stress (τ_h/σ'_{vc}) versus strain and the normalized excess pore pressure $(\Delta u/\sigma'_{vc})$ versus strain curves are presented in Plates 23 and 24. The normalized effective stress paths are presented in Plate 25. A range of values for ϕ' was calculated to be from 21 to 24 degrees.

The strains at failure (maximum shear stress) for these tests are quite large ranging between 10.5 and 19.0 percent with the average value approximately 16 percent. Positive excess pore pressures developed during shearing for soils consolidated at an OCR equal to 1.0. Negative excess pore pressures initially developed for soil with OCR's of 2.0. However for both stress conditions the excess pore water was increasing when the test was stopped. The shear strength ratio, Su/σ'_{vc} (at $Su = (\tau_h)_{max}$) averaged 0.23 for the normally consolidated tests with a variation of 0.03.

Plate 26 presents the normalized undrained strengths $(\tau_h)_{max}/\sigma'_{vc}$ versus the logarithm of the overconsolidation ratio, where $(\tau_h)_{max}$ is the maximum horizontal shear stress applied to the test specimen and σ'_{vc} is the effective vertical consolidation stress. Ranges of $(\tau_h)_{max}/\sigma'_{vc}$ published in the literature for soft clays are also shown for comparison. The test results fall within the range of the published data.

Comparison of Test Results. - A comparison of shear strengths obtained from triaxial tests ($\overline{CIU}C$ and $\overline{CK_0U}C$) with strengths obtained from direct simple shear tests ($\overline{CK_0U}DSS$) is presented on Plate 27 as plots of S_u from $\overline{CIU}C$ and $\overline{CK_0U}C$ tests and $(\tau_h)_{max}$ from direct simple shear tests versus the consolidation stress. As expected, the undrained shear strength obtained from the triaxial tests are slightly (~ 20%) higher than the direct simple shear test results at the same consolidation stress level. This agrees with the relationship reported by Mayne (1982) that the ratio of Su/σ'_{vo} (NC) from $\overline{CK_0U}DSS$ to that from $\overline{CIU}C$ and $\overline{CK_0U}C$ tests generally varied between 0.6 and 0.8 for normally consolidated specimens.

Another correlation which can be made with Su/σ'_{vo} is one suggested by Ladd (1977). The correlation between the in situ shear strength and the effective overburden pressure is given in the following approximate relationship:

$$Su/\sigma'_{vo} = (0.23 \pm 0.05)\ (OCR)^{0.8}$$

A plot of this relationship is presented along with the shear strengths reported by the different tests in Plate 27, for normally consolidated (OCR = 1) specimens. A generally good correlation exists between the laboratory results from the direct simple shear tests and the relationship proposed by Ladd.

Plate 28 represents a comparison of results from laboratory strength tests determined 1) from standard laboratory tests performed on undisturbed samples and 2) from normalized tests using SHANSEP procedures. The solid line labeled "Interpreted Shear Strength Profile" is based on corrected miniature vane, in situ vane, and unconsolidated undrained triaxial test results (Plate 30). The various symbols shown were determined from the normalized test results summarized on Table 6 and the estimated maximum past consolidation pressure presented on Plate 12A. The shaded area is the relationship previously discussed (Plate 27) using past $\overline{CK_0U}DSS$ test results and $\sigma'_{vo} = \sigma'_{vm}$ from Plate 12A.

From this plate, the results of tests on undisturbed samples indicate a higher shear strength than that derived from the normalized tests. However, it should be noted that the shear strength value determined for a particular depth using SHANSEP test resuls is extremely sensitive to the calculated consolidation pressure. For example, had a hydrostatic effective vertical pressure been assumed, the resulting shear strength profile would have been much higher than the profile determined from the tests on undisturbed samples.

Summary - The results of the laboratory testing described in this chapter fulfilled three very important objectives. First, design variables were obtained which can be applied to any of the pile capacity prediction methods currently used in practice. Second, basic properties of the soil at the West Delta tests site were determined for input into 1) constituative models to be developed using advanced analytical methods and 2) interpretive analyses of model and large scale test currently planned. Finally, a thorough documentation of the site is provided to enable the site to be referenced in the development of future, not yet envisioned, pile capacity methodology.

SITE CHARACTERIZATION

General Site Conditions

The subsurface conditions at the test site in Block 58, West Delta Area can be characterized based on the drilling and sampling program, classification of soils in the laboratory, and cone penetrometer soundings as follows:

Stratum	Depth, m (Ft) From - To	Soil Description
I	0 - 24.4 (0 - 80)	Very soft to soft olive gray clay with silt pockets and partings.
II	24.4 - 48.8 (80 - 160)	Soft to stiff gray clay.
III	48.8 - 77.1 (160 - 253)	Stiff to very stiff gray clay with shell fragments.
IV	77.1+ (253+)	Gray fine sand.

In general, the stratigraphy can be described as a very soft to very stiff olive gray clay (Strata I, II, and III) overlying a gray fine sand (Stratum IV). The stratigraphy is given in more detail on the boring log (Plate 6).

Soil Properties

The soils which were considered to be similar to the proposed site for the TLP and make up the test stratum for the pile load test were identified as Strata I through III. These strata are predominately composed of homogeneous clays.

Each of the three idealized strata (I, II, and III) can be characterized in terms of the physical soil properties based on laboratory and in situ tests as shown in Table 7, Plate 29, and as described in the following sections.

Stratum I. - Soils in Stratum I are generally very soft to soft clays with silt pockets and partings. The expansive behavior observed after sampling was caused primarily by dissolved gas in the pore water and bubble phase gas in the interstitial voids. Natural moisture contents average about 75 percent in the upper 12 m (40 ft) and approximately 60 percent in the lower portion of the stratum. Indications from the plasticity chart (Plate 9) are that there is a lower plasticity clay in the upper half of Stratum I. The average plastic limit in this zone is 19 percent with the average liquid limit at 50 percent. The lower portion of Stratum I has an average plastic limit of 27 and a liquid limit of 72 percent. The wet unit weights measured averaged 15.09 KN/m^3 (96 PCF) in the upper 12 m (40 ft) and gradually increased the remaining portion of the stratum to 17.2 KN/m^3 (110 PCF) at 24 m (79 ft).

Cone penetrometer test sounding records (Appendix A) demonstrate the homogeneity and continuity of the stratigraphy. The tip resistance increased linearly from a value of 190 KPa (4.0 KSF) at 4 m (13 ft) to 326 KPa (6.8 KSF) at 24 m (79 ft). The cone sleeve friction gradually increased to a depth of 18 m (60 ft) from a value of 1.90 KPa (0.04 KSF) to 17.20 KPa (0.36 KSF). At this depth it became constant for the remaining portion of the stratum.

Stratum II. - The properties of the soils found in Stratum II are much more consistent than those found in Stratum I. Stratum II is essentially the same in appearance as Stratum I except that soils of Stratum II are noticeably lower in plasticity. The plastic limits average 24 percent with the corresponding liquid limit values averaging 50 percent. The natural moisture contents averaged approximately 40 percent throughout the stratum. The wet unit weights were somewhat higher than those found in the upper 24 m (79 ft) with an average value of 17.2 KN/m^3 (110 PCF). The soils in Stratum II still exhibited an expansive behavior upon sampling and were slightly underconsolidated.

The records from the cone penetrometer soundings do not indicate any zones or layers of noncohesive material. Both the tip resistance and cone sleeve friction gradually increase with depth at approximately the same rate. The sleeve friction linearly increases from 17.20 KPa (0.36 KSF) at 24 m (79 ft) to 25.4 KPa (0.53 KSF) at 49 m (163 ft). The point resistance is somewhat more variable with three different slopes defining its increase over the 24 m (80 ft) stratum. Values for the tip resistance range between 326 KPa (6.8 KSF) and 670 KPa (14.0 KSF).

Stratum III. - The soils encountered in Stratum III are somewhat similar in plasticity to the soils found in the lower portion of Stratum I. The plastic limits averaged approximately 30 percent with the liquid limit averaging 85 percent. The natural moisture content averaged 57 percent over the stratum with wet unit weights averaging 16.7 KN/m^3 (106 PCF). Shell fragments were recovered in numerous samples throughout this stratum. One major physical difference between Stratum III and the other two strata was that the soil exhibited a platy structure. This structure was first noticed in the sample recovered at 55 m (180 ft) and possibly can explain why there was a noticeable increase in shear strength beginning at approximately this depth.

The sleeve friction shows a sharp increase between 49 m (163 ft) and 52 m (172 ft). The values measured ranged between 25.4 KPa (0.53 KSF) and 33.5 KPa (0.70 KSF). The sleeve friction recorded for the remaining portion of this stratum linearly increased to 36.4 KPa (0.76 KSF). More variance was recorded in the tip resistance in Stratum III especially below 58.5 m (192 ft). Shell fragments were recovered in samples beginning at approximately this depth which could possibly explain why the tip resistance displayed "spikes" as it passed through the soil in this layer. The maximum tip resistance measured in the clay stratum was 1,200 KPa (25.1 KSF) just prior to entering the sand stratum at 77.1 m (253 ft).

Stress History

The state of consolidation of the soil at the test site remains somewhat unclear at present. Although some degree of underconsolidation was expected due to the geologic history of the Mississippi River Delta, it is believed that the soil at the test site is not as severely underconsolidated as the results of the consolidation tests and subsequent maximum past pressure calculations would tend to indicate.

Samples recovered from this site experienced disturbance from dissolved gases coming out of solution following the total stress relief associated with sampling. Much of this disturbance is irreparable and results in calculations of maximum past pressures which are lower than probable for the in situ state. Even the different methods for testing, constant stress versus constant rate of strain, produced a wide variation in stress history calculations.

During the small diameter segment test phase, pore pressure transducers on the model pile will allow measurement of ambient pore pressure after equilibrium is attained. From this data, an in situ pore pressure distribution will be plotted and applied to total stress conditions, thus allowing a better evaluation of stress history and in situ effective pressure and shear strength.

CONVENTIONAL AXIAL PILE DESIGN ANALYSIS

General

Existing technology relevant to the design of pile foundations for tension leg platforms is based on studies of the frictional component of pile capacity. Several methods of analysis have been used in research and practice to estimate the frictional capacity of piles with varying (or unknown) degrees of accuracy. The complicated mechanism of interaction between soil and pile, the uncertainty of in situ soil properties, and the limited number of field load tests on instrumented piles combine to make conventional prediction of frictional capacity of piles a matter of empiricism and educated guesswork. Even less information is available on the behavior of piles under cyclic tension loading. Degradation of shear resistance during cyclic loading may be accounted for through conservative design, but is not usually recognized in current practice. Table 8 presents the current offshore practices predicting skin friction for piles in clay. Several of these methods will be discussed and applied in an analysis of the pull-out capacity of the test pile at the site. The methods to be considered are:

1. API RP 2A (1981) method,
2. Lambda method (Vijayvergiya and Focht, 1972),
3. Effective Stress Method (Burland, 1973), and
4. Simplified General Effective Stress Method (Esrig and Kirby, 1979).

API RP 2A (1981) Method. - In this total stress approach method, the unit skin friction, f, along a pile embedded in clay is correlated to the undrained shear strength, S_u, by a dimensionless multiplier, α, viz:

$$f = \alpha S_u$$

The value of α varies between 0.5 and 1.0 for Gulf of Mexico clay deposits. The skin friction is equal to S_u for underconsolidated and normally consolidated clays. For overconsolidated clays, the skin friction should not exceed 48 KPa (1 KSF) for shallow penetration or the undrained shear strength of a normally consolidated clay for deeper penetrations, whichever is greater.

Lambda Method (Vijayvergiya and Focht, 1972). - The Lambda (λ) method is, in fact, a hybrid incorporating effective stress and total stress principles. It is based on the assumption that pile driving displaces soil sufficiently to develop passive soil pressure. This method yields the following relationship for the average frictional resistance over the full length of the pile:

$$f = \lambda \ (\sigma'_m + 2C_m)$$

where σ_m' = average vertical effective stress, and
C_m = average undrained shear strength over the embedded depth.

Values of the λ factor vary from 0.49 at the mudline to about 0.12 for a pile embedded 61 m (200 ft) or more. These factors were empirically determined from interpretation of the results of 47 pile load tests, many of these being the same tests from which the API criteria were developed. It should be noted that the Lambda-method does not describe the variation in unit friction along the pile length, but is primarily a method by which the total pile-head capacity may be calculated. It is therefore difficult (or inappropriate) to apply in cases of layered stratigraphies.

Effective Stress Method (Burland, 1973). - The effective stress method, or β method, is expressed by the following relationships:

$$f = \sigma'_h \text{ Tan } \delta$$

where σ'_h is the lateral effective stress and δ is the friction angle between soil and pile.

The relationship further assumes that:

$$f = K_o \ \sigma'_{vo} \ \text{Tan} \ \delta$$

where K_o is the coefficient of earth pressure at rest and σ'_{vo} is the vertical effective stress prior to the pile installation. Thus, it is assumed that the state of stress in the soil is not changed by pile installation, subsequent setup consolidation and loading. If better K_o values cannot be determined, it is further assumed that for normally consolidated clay,

$$K_o = 1 - \text{Sin} \ \phi'$$

where ϕ' = effective friction angle of the soil and
δ = ϕ' for soft clay.

Thus, the equation becomes

$$f = (1 - \sin \ \phi') \ \sigma'_{vo} \ \text{Tan} \ \phi' = \beta \ \sigma'_{vo}$$

where $\beta = (1 - \text{Sin} \ \phi') \tan \ \phi'$

For pile penetrations greater than 23 m (75 ft), β values may range from 0.1 to 0.2 when correlated with full-scale pile load tests (Meyerhof, 1976).

Simplified General Effective Stress Method (Esrig and Kirby, 1979). - Recently, efforts have been made to develop a general effective stress method based on the cavity expansion theory (Wroth et al, 1979) and the critical state soil model (Esrig et al, 1977). This method is categorized as the "General Effective Stress Method" (GESM) and postulates that the effective lateral stress depends on four items:

- The initial state of stress prior to pile installaton,
- The stress change due to pile installation,
- The stress change due to pile setup (reconsolidation of the pile driving induced excess pore pressures), and

- The stress change due to pile loading to yield.

The following assumptions were made by the proponents of the GESM:

1. Changes of lateral stress due to pile driving can be estimated by plane-strain cylindrical cavity expansion theory.

2. Stress changes due to soil reconsolidation can be estimated using the concept of critical state soil mechanics. The critical state condition is defined as that in which the soil has been sheared to such large strains that further strains result in no additional change in volume.

3. Changes in mean normal total stress can be modeled by finite element analyses based on the following assumptions:

   (a) Linear elastic behavior of the pile and soil.
   (b) No slippage between soil and pile.

For preliminary design purposes, Esrig et al (1979) suggested that:

$$f = \beta \sigma'_{vo},$$

where β = a coefficient from Figure 10 or 11 of the referenced paper.

Calculated values are given in the referenced paper. Although this method is a significant step toward a rational prediction of ultimate skin friction, it has not yet proven adequate and may yield unconservative results. This is primarily due to the failure of the soil model to reflect the special conditions of large displacements concentrated along the critical surface of slip located at, or near, the pile-soil interface. Also, no effort was made to account for the degradation of frictional resistance resulting from the cyclic motion of the upper portion of the pile.

Interpretation of Soil Properties and Design Parameters

The shear strength and unit weight profiles shown on Plate 29 represent the interpretation of the assembled laboratory and field test results. In developing the shear strength profile for the cohesive soils at this site, the assembled soil test and in situ test results were combined to produce a curve through the data considered to best represent the actual shear strength of the soil. The tests results from which the interpreted shear strength profile was developed were the remote vane, miniature vane, torvane, and unconsolidated undrained triaxial tests. The laboratory test values were corrected to compensate for disturbance in the samples caused by the abundance of gas. The remote vane results were corrected to bring the shear strength values down to numbers more in line with the type tests on which current empirical methods for computing pile capacity are based. The correction factors selected are as follows:

Test Type	Correction Factor
Remote Vane	0.75
Miniature Vane	1.10
Torvane Vane	1.10
Unconsolidated Undrained Triaxial Compression	1.10

The corrected shear strength profile considered to best represent the shear strength at the site is shown on Plate 30. The unit skin friction distribution curves shown on Plate 31 was developed 1) from the interpreted shear strength profile based on results of tests on undisturbed samples (API, λ) and 2) results of SHANSEP test results with σ'_{vm} from Plate 12A applied as the effective overburden pressure (Burland, GESM). This pressure distribution is shown on Plate 32.

Results of Ultimate Pile Capacity Predictions

Using the unit skin friction curves and the maximum past pressure as determined by consolidation tests, conventional pile capacity curves were computed by API, Lambda, GESM, and Burland methods. The results are presented on Plate 33 for a 76.2 cm (30 in) diameter driven pipe pile, which is the diameter planned for the large scale pile at West Delta 58.

The following tabulation presents the computed ultimate tensile pile capacities for pipe piles driven to a depth of 67.1 m (220 ft) below the seafloor:

Ultimate Tensile Pile Capacity KN (KIPS)

Pile Diameter cm (inches)	Lambda	Burland	API	GESM
76.2 (30)	3114 (700)	4680 (1052)	5160 (1160)	4115 (925)

Calculations using Burland, API, and General Effective Stress methods produced very close to the same capacity, with the API method being the highest. The Lambda method predicted an ultimate capacity considerably below the other three methods. After ambient pore pressures are measured in the field, a better estimation of in situ effective stress can be made. Revisions to the Burland and GESM methods may be required at that time.

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TABLES

BORING NUMBER	SAMPLE NUMBER	DEPTH M (FT)	CLAY CONTENT <2 μ (%)	G_s	C_c MEASURED	C_c CALCULATED	σ'_{Vo} KPa(KSF)	σ'_{Vmax} KPa(KSF)	$\frac{\sigma_{Vmax}}{(\sigma'_{Vo})\,HYD.}$
6	10	18.6 (61.0)	38	2.72	0.59	0.357	97.7(2.04)	47.9 (1.0)	0.49
6	26	31.6 (103.5)	34	2.73	0.36	0.272	185.9(3.88)	34.5 (0.72)	0.19
5	92	48.8 (160.0)	54	2.75	0.53	0.385	310.4(6.48)	119.7 (2.5)	0.39
5	123	64.0 (210.5)	64	2.82	0.65	0.539	412.0(8.60)	167.7 (3.5)	0.41

TABLE I SUMMARY OF CONSOLIDATION TEST RESULTS

BORING NUMBER	SAMPLE NUMBER	DEPTH M (FT.)	γ d Mg/m³ (PCF)	MOISTURE CONTENT (%)	AT FAILURE Su KPa (KSF)	ξa(%)
5	13	6.55 (21.5)	1.55 (96.6)	65.1	9.24 (0.19)	12.57
5	37	18.7 (61.4)	1.56 (97.1)	70.1	14.8 (0.31)	8.33
5	48	24.6 (80.7)	1.78 (110.9)	39.6	15.7 (0.33)	13.33
5	92	48.9 (160.4)	1.72 (107.2)	43.5	43.7 (0.91)	10.00
5	106	55.1 (180.7)	1.61 (100.4)	49.8	21.5 (0.45)	15.00
6	70	73.4 (240.6)	1.72 (107.5)	49.0	74.51 (1.56)	3.33

TABLE 2 SUMMARY OF UNCONFINED TRIAXIAL COMPRESSION TEST RESULTS

BORING NUMBER	SAMPLE NUMBER	DEPTH M (FT)	LL (%)	PI (%)	MOISTURE CONTENT (%)	AT FAILURE ξa(%)	AT FAILURE Su KPa (KSF)	ξ_{50}(%)
5	1	0.15 (0.5)	45	28	77.2	22.2	2.43 (0.05)	3.5
4	3	1.68 (5.5)	57	35	46.7	21.3	2.63 (0.06)	2.9
5	5	3.05 (10.0)	55	35	75.6	21.7	3.55 (0.07)	0.5
5	7	4.57 (15.0)	47	30	79.4	16.7	4.71 (0.10)	2.2
5	11	6.25 (20.5)	86	54	74.9	7.7	11.51 (0.24)	0.05
5	15	7.92 (26.0)	74	48	75.1	20.5	10.44 (0.22)	1.1
6	5	12.3 (40.5)	79	52	56.5	15.0	15.8 (0.33)	1.1
6	10	18.83 (60.0)	61	38	51.8	15.0	23.2 (0.48)	1.6
5	48	24.8 (81.4)	49	30	41.0	20.0	23.5 (0.49)	2.4
6	26	31.4 (103.0)	49	29	40.9	20.0	27.7 (0.58)	2.5
5	76	39.5 (129.5)	45	24	35.9	20.0	29.0 (0.61)	3.1
5	92	49.2 (161.4)	65	43	52.7	13.3	53.9 (1.13)	1.1
5	106	55.3 (181.4)	76	48	51.4	20.0	34.3 (0.72)	2.1
5	123	64.0 (210.0)	87	62	54.9	5.0	65.1 (1.36)	1.0
6	70	73.6 (241.5)	91	63	53.6	5.33	69.8 (1.46)	0.8

TABLE 3 SUMMARY OF UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

BORING NUMBER	SAMPLE NUMBER	DEPTH METERS (FEET)	WET UNIT WEIGHT Mg/m^3 (PCF)	WATER CONTENT (%)		LABORATORY STRESS HISTORY			$(\sigma_1-\sigma_3)_{MAX}$ KPa (KSF)	q/σ'_{3C}	p'/σ'_{3C}	AXIAL STRAIN ξ_a
				INITIAL	FINAL	σ'_{3C} KPa(KSF)	$\sigma'_{1C}/\sigma'_{3C}$	OCR				
$CI\overline{U}C$ TESTS												
B-5	37	18.69-18.81 (61.3-61.7)	1.52 (95.8)	69.6	50.6	193.1 (4.03)	1.0	1.0	103.3 (2.16)	0.27	0.74	9.5
B-5	61	32.32-32.47 (106.0-106.5)	1.70 (105.8)	45.8	33.4	103.4 (2.16)	1.0	1.0	69.6 (1.45)	0.34	0.85	16.6
B-6	45	49.09-49.27 (161.0-161.6)	1.70 (106.0)	52.9	41.4	358.6 (7.49)	1.0	1.0	210.3 (4.39)	0.29	0.71	7.5
B-6	62	64.21-64.39 (210.6-211.2)	1.70 (106.0)	49.3	43.4	503.3 (10.51)	1.0	1.0	266.1 (5.56)	0.26	0.70	10.0
B-5	20	12.53-12.71 (41.1-41.7)	1.60 (100.1)	67.2	51.0	103.4 (2.16)	1.0	2.0	72.5 (1.51)	0.35	0.98	10.4
B-5	76	39.36-39.51 (129.1-129.6)	1.76 (109.7)	41.3	25.7	273.7 (5.76)	1.0	2.0	327.6 (6.84)	0.59	1.22	12.6
B-6	62	64.02-64.18 (210.0-210.5)	1.71 (106.9)	54.2	39.2	503.3 (10.51)	1.0	2.0	365.5 (7.63)	0.36	1.12	10.2
$CK_0\overline{U}C$ TESTS										q/σ'_{3C}		
B-5	20	12.35-12.5 (40.5-41.0)	1.54 (96.3)	73.0	55.6	75.8 (1.58)	1.27	1.0	63.6 (1.33)	0.33	0.67	12.8
B-6	16	24.94-25.0 (81.8-82.0)	1.77 (110.5)	39.2	32.0	196.5 (4.10)	1.44	1.0	176.1 (3.68)	0.31	0.60	7.2
B-6	53	54.91-55.06 (180.1-180.6)	1.63 (101.5)	56.4	43.0	361.3 (7.54)	1.33	1.0	253.9 (5.30)	0.26	0.65	6.2

TABLE 4 SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

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BORING NUMBER	SAMPLE NUMBER	DEPTH METERS (FEET)	σ'_{vc} KPa (KSF)	OCR	AT τ_H = MAXIMUM				AT τ_H / σ'_v = MAXIMUM				STRAIN RATE IN/HR	B-VALUE
					γ (%)	τ_H/σ'_{vc}	σ'_v/σ'_{vc}	τ_H/σ'_v	γ (%)	τ_H/σ'_{vc}	σ'_v/σ'_{vc}	τ_H/σ'_v		
5	20	12.71-12.80 (41.7-42.0)	103.5 (2.16)	1.0	18.6	0.266	0.620	0.415	25.0	0.263	0.593	0.447	0.059	0.97
5	20	12.71-12.80 (41.7-42.0)	103.5 (2.16)	2.0	18.1	0.461	1.040	0.444	25.0	0.430	1.000	0.464	0.053	0.96
5	76	39.02-39.12 (128.0-128.3)	275.9 (5.76)	1.0	10.56	0.210	0.608	0.347	27.4	0.185	0.503	0.419	0.089	0.95
6	26	31.52-31.59 (103.4-103.6)	275.9 (5.76)	2.0	12.0	0.380	1.045	0.364	25.0	0.353	0.922	0.412	0.055	0.96
6	62	64.24-64.27 (210.7-210.8)	503.5 (10.51)	1.0	15.6	0.226	0.727	0.312	18.2	0.226	0.725	0.313	0.057	0.97
6	70	73.17-73.23 (240.0-240.2)	440.0 (9.19)	2.0	19.0	0.411	1.102	0.373	21.9	0.403	1.094	0.376	0.052	0.95

TABLE 5 SUMMARY OF MONOTONIC SIMPLE SHEAR TEST RESULTS

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The Earth Technology Corporation

APPROX. DEPTH, METERS (FEET)	SOIL TYPE	OCR = 1			OCR = 2	
		$\overline{CIU}$	$\overline{CK_oU}$	$\overline{CK_oUDSS}$	$\overline{CIU}$	$\overline{CK_oUDSS}$
		S_u/σ'_{3c}	S_u/σ'_{vc}	$\tau_{H(max)}/\sigma'_{vc}$	S_u/σ'_{3c}	$\tau_{H(max)}/\sigma'_{vc}$
12.5 (40.0)	CH		0.33	0.27	0.35	0.46
18.7 (61.5)	CH	0.27				
21.0 (69.0) *	CH			0.32		
25.0 (82.0)	CL		0.31			
32.0 (105.5)	CL	0.34				0.38
34.9 (114.5) *	CH		0.27			
39.3 (129.0)	CL			0.21	0.59	
45.5 (149.5) *	CH			0.28		
49.1 (161.0)	CH	0.29				
55.0 (181.5)	CH		0.26			
57.7 (189.2 *	CH		0.23			
64.0 (210.0	CH	0.26		0.23	0.36	
69.7 (228.7 *	CH			0.23		
73.2 (240.1)	CH					0.41

*TESTS PERFORMED BY McCLELLAND

TABLE 6 SUMMARY OF NORMALIZED TEST RESULTS

SOIL PROPERTY	STRATUM I				STRATUM II				STRATUM III			
	LOW	HIGH	AVE.	STD. DEV.	LOW	HIGH	AVE.	STD. DEV.	LOW	HIGH	AVE.	STD. DEV.
NATURAL MOISTURE CONTENT, %	39.6	79.4	61.9	14.56	35.9	52.7	41.92	5.55	49.3	56.4	52.9	2.61
SPECIFIC GRAVITY	-	-	2.72	-	-	-	2.73	-	2.75	2.82	2.79	0.05
PLASTICITY INDEX, %	17	32	23.7	4.81	19	27	22.6	2.62	28	36	31	3.32
LIQUID LIMIT, %	45	86	63.7	14.9	45	76	56.6	10.6	76	91	83	7.04
LIQUIDITY INDEX	0.52	2.15	1.07	0.59	0.43	0.72	0.54	0.12	0.34	0.71	0.47	0.12
UNIT WET WEIGHT, PCF	90	111	101	6.39	105	115	108.5	4.72	100.4	107.5	105	2.41

TABLE 7 COMPARISON OF INDEX PROPERTIES

METHODS	PROCEDURES	CURRENT PRACTICE	REFERENCECES
TOTAL STRESS α	$fs = \alpha Su$	(1) $\alpha = 1.0$ FOR $Su \leq 500$ PSF (2) α DECREASES LINEARLY FROM 1.0 TO 0.5 FOR 500 PSF $< Su <$ 1500 PSF (3) $\alpha = 0.5$ FOR $Su \geq 1500$ PSF	API (1981)
		α DEPENDS ON Su, L/D, AND CLAY INDEX PROPERTIES	FLAATE(1968) TOMLINSON (1970) API (1980)
EFFECTIVE STRESS β	$fs = \beta \sigma'_{vo}$	$\beta = (1-\sin\phi')\tan\phi'$	CHANDLER (1968), BURLAND (1973)
		$\beta = 1.5\,(1-\sin\phi')\tan\phi'\sqrt{OCR}$	MEYERHOF (1976)
		$\beta = \dfrac{\sin\phi'\cos\phi'}{1+\sin\phi'}$	PARRY AND SWAIN (1977)
		$\beta = \left(\dfrac{d+20}{2d+20}\right)0.4\sqrt{OCR}$	FLAATE AND SELNES (1978)
COMBINED TOTAL & EFFECTIVE STRESS λ	$fs = \lambda(\sigma'_{vo}+2Su)$	λ VARIES WITH DEPTH OF PENETRATION FROM ABOUT 0.49 AT THE GROUND SURFACE TO ABOUT 0.12 AT 200 FEET	VIJAYVERGIYA AND FOCHT (1972)
CPT CORRELATION	$fs = fc$ $fs = \alpha' q_c$	α' DEPENDS ON THE SOIL TYPE	BEGEMANN (1965) DE RUITER AND BERINGER (1979)
SIMPLIFIED GENERAL EFFECTIVE STRESS	$fs = \beta\sigma'_{vo}$	β DEPENDS ON OCR AND PLASTICITY INDEX	ESRIG AND KIRBY (1979)

fs = Ultimate Skin Friction

d = Depth Below Ground Surface At Any Point Along Pile, Meters

σ'_{vo} = Initial Vertical Effective Stress (Prior to Pile Installation)

Su = Undrained Shear Strength

fc = Local Sleeve Friction

OCR = Overconsolidation Ratio

L = Length of Pile Penetration

D = Pile Diameter

q_c = Cone Resistance

TABLE 8 COMPARISON OF CURRENT OFFSHORE PRACTICES IN PREDICTING SKIN FRICTION FOR PILES IN CLAY

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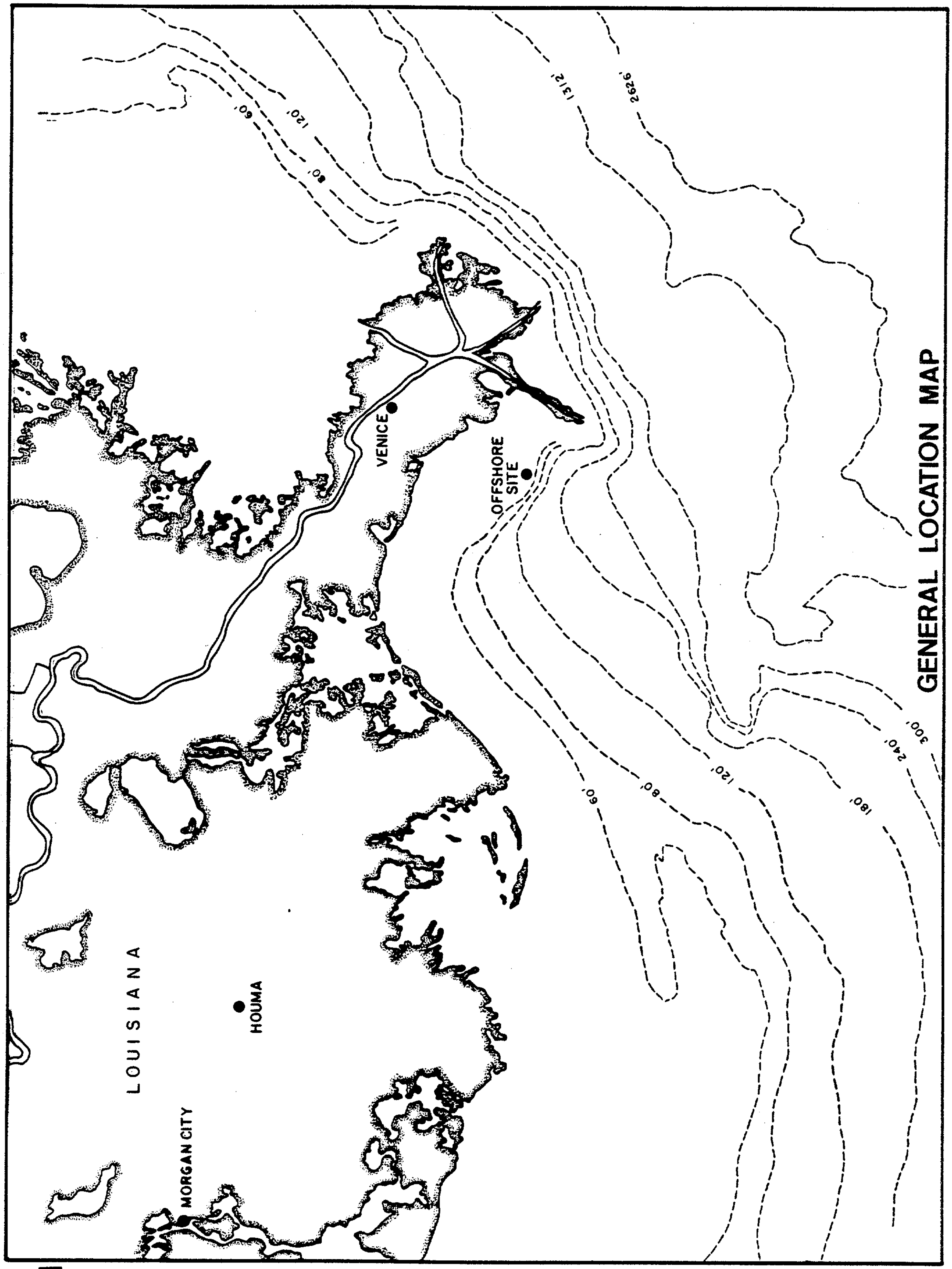
ILLUSTRATIONS



GENERAL LOCATION MAP

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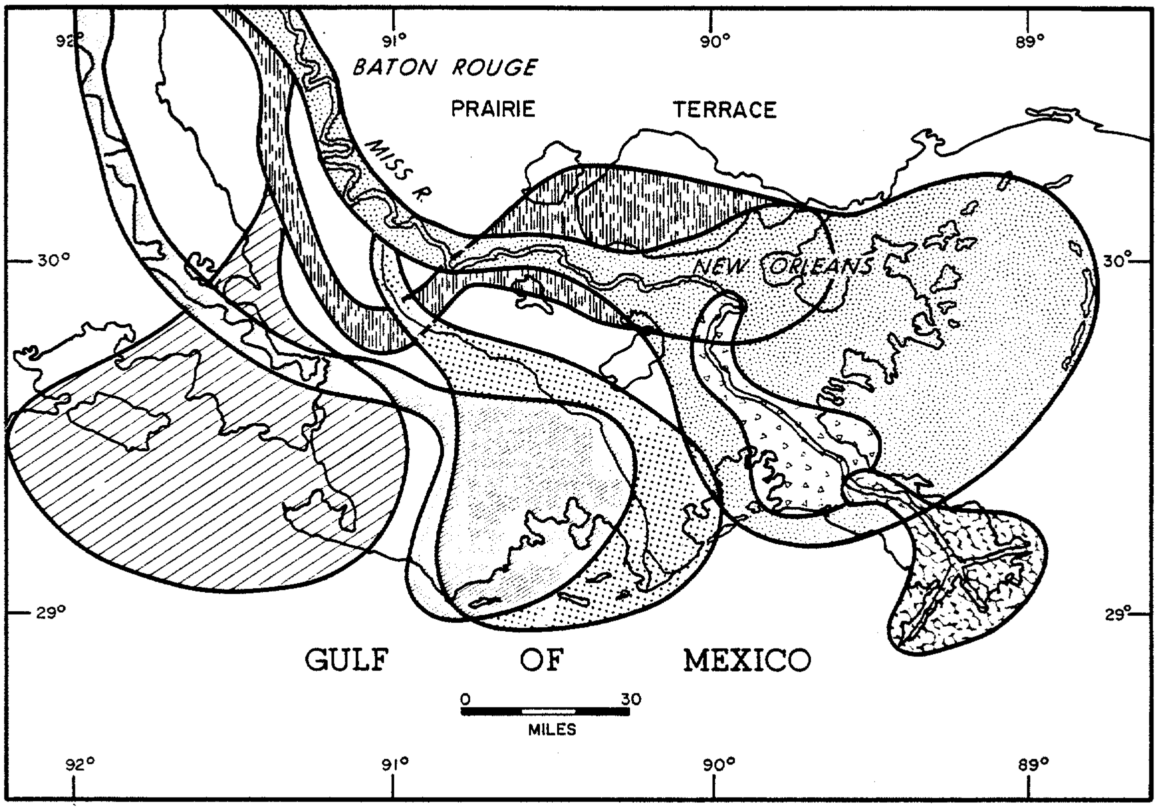
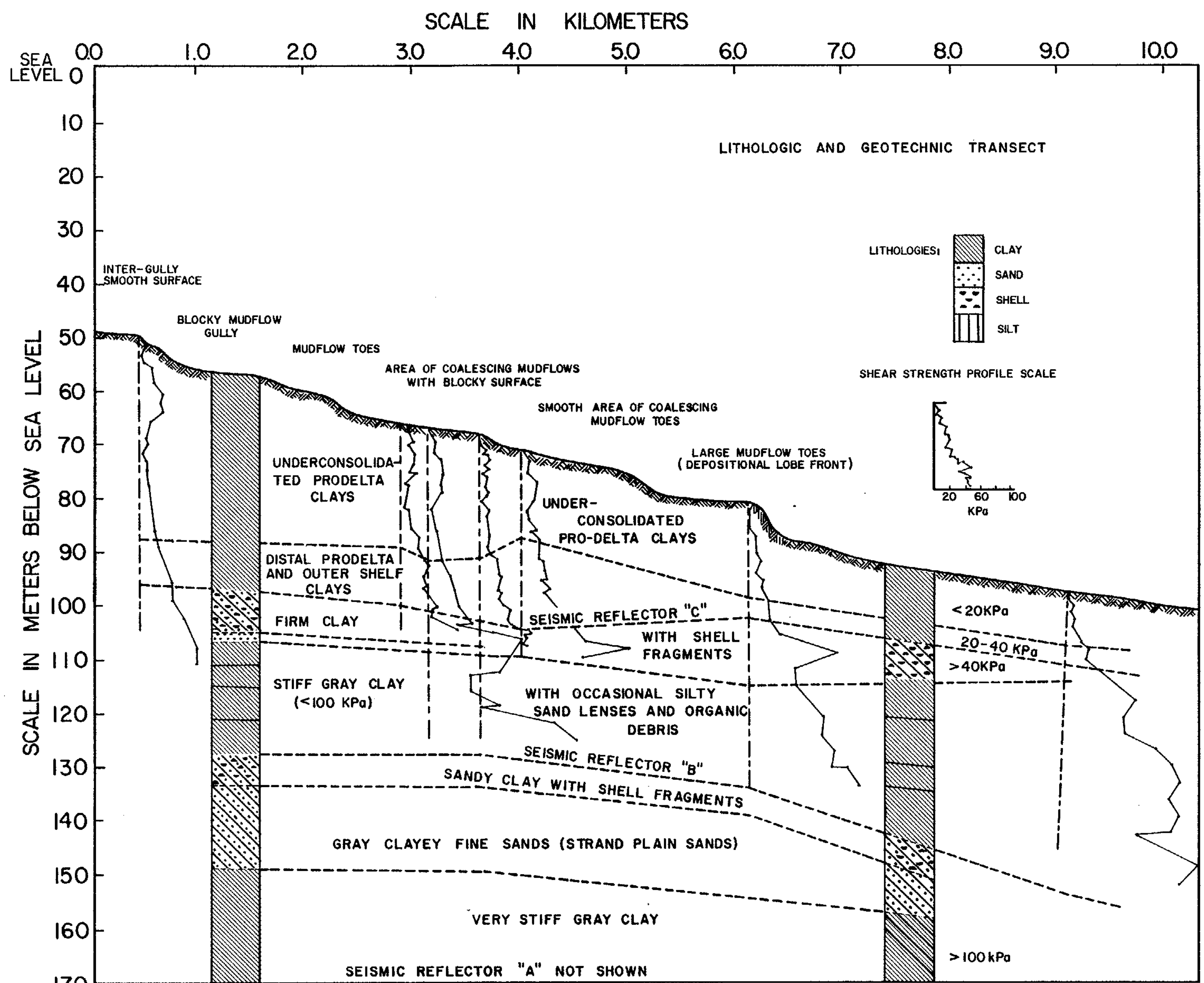
PLATE 1



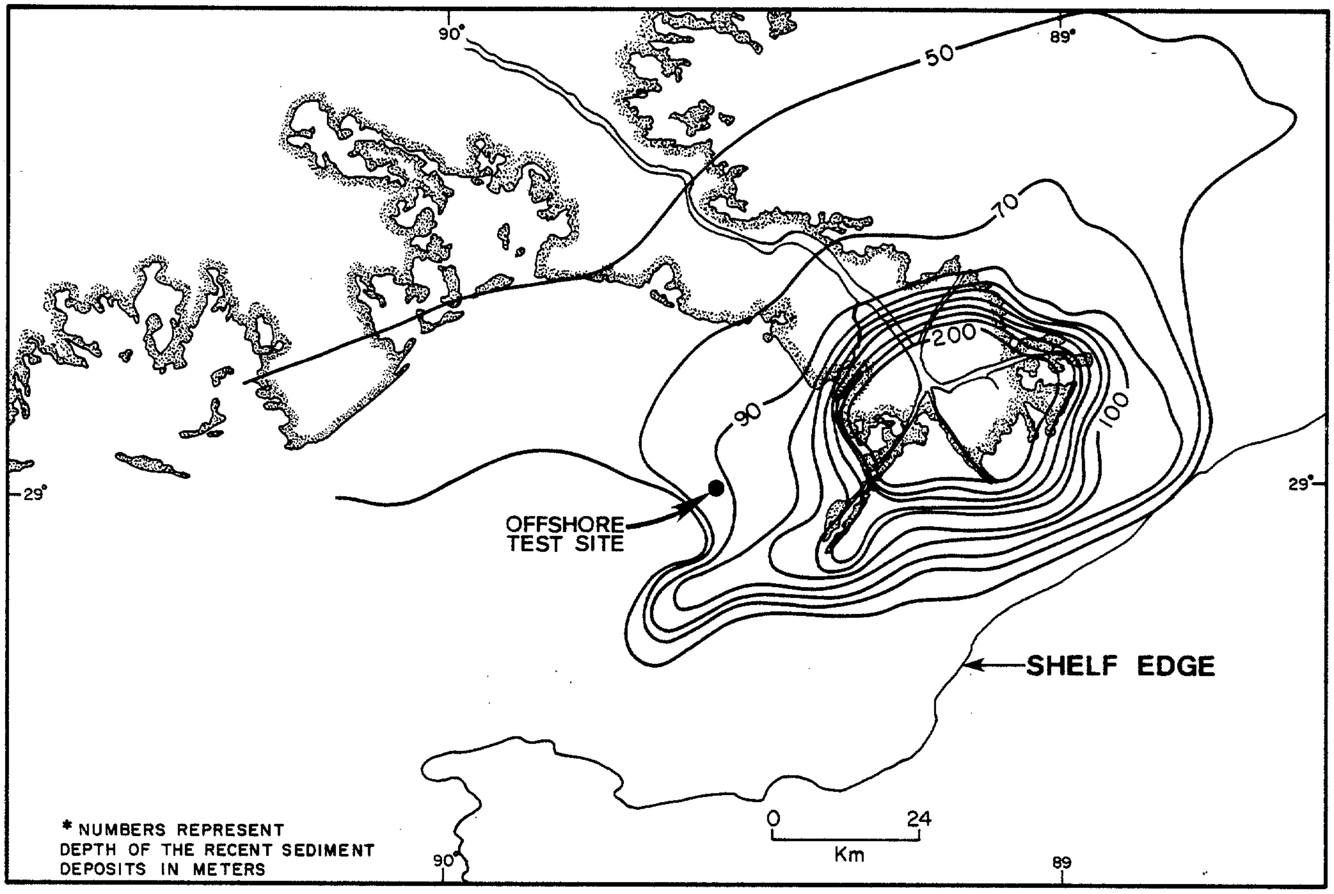
LEGEND OF CHRONOLOGICAL DELTAS

SALE CYPREMORT		> 4600 YRS. B.P.
COCODRIE	CA	4600–3500 YRS. B.P.
TECHE	CA	3500–2800 YRS. B.P.
ST. BERNARD	CA	2800–1000 YRS. B.P.
LAFOURCHE	CA	1000–300 YRS. B.P.
PLAQUEMINE	CA	750–500 YRS. B.P.
BALIZE		< 550 YRS. B.P.

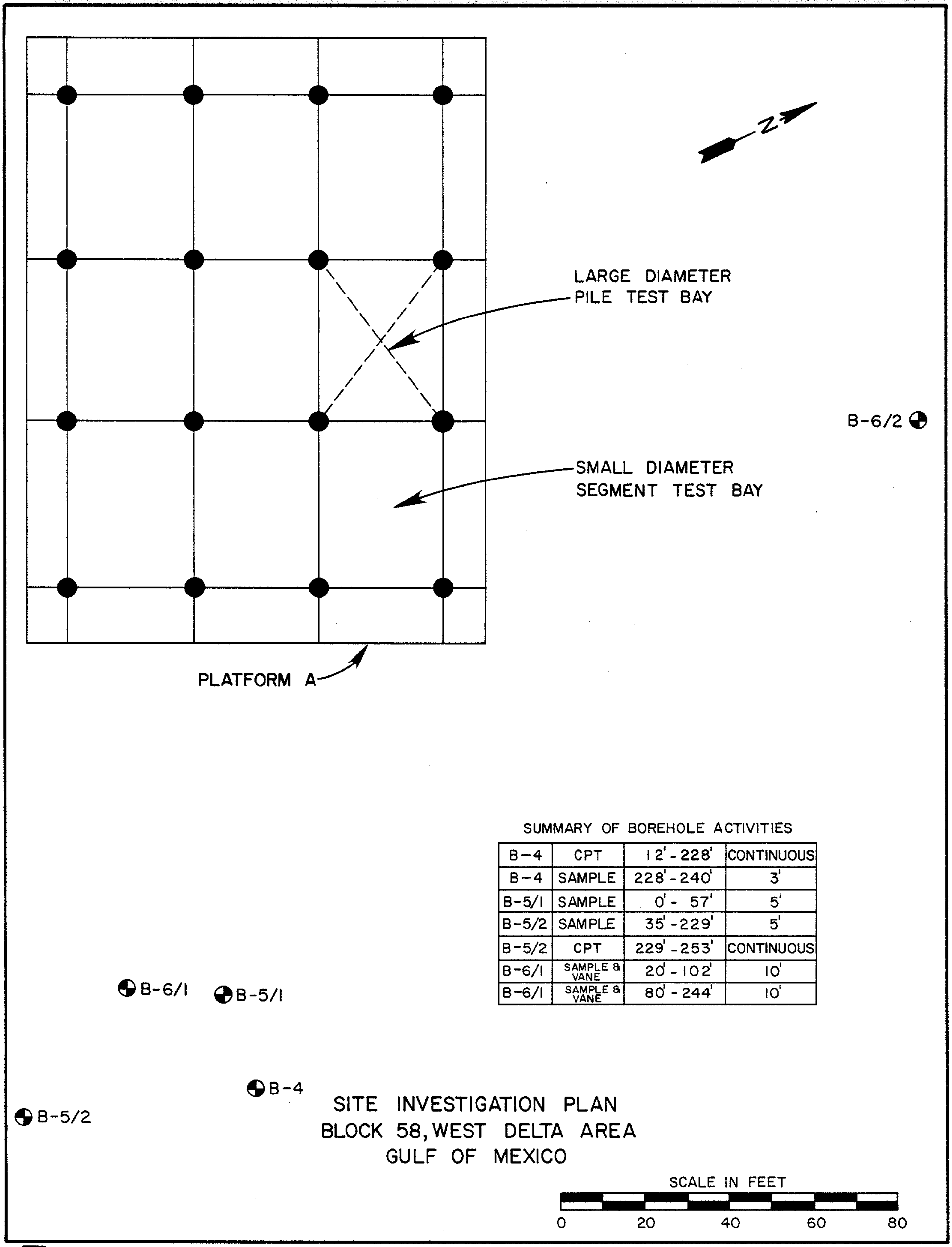
SUCCESSION OF LOBES OF THE MISSISSIPPI DELTA
(AFTER KOLB AND VAN LOPIK, 1958)



LITHOLOGIC AND GEOTECHNIC TRANSECT ACROSS DELTA AREA
(AFTER TRABANT, 1978)



ISOPACH MAP OF THE RECENT SEDIMENTS
IN THE MISSISIPPI RIVER DELTA
(AFTER COLEMAN AND SUHAYDA, 1979)



SUMMARY OF BOREHOLE ACTIVITIES

B-4	CPT	12'-228'	CONTINUOUS
B-4	SAMPLE	228'-240'	3'
B-5/1	SAMPLE	0'- 57'	5'
B-5/2	SAMPLE	35'-229'	5'
B-5/2	CPT	229'-253'	CONTINUOUS
B-6/1	SAMPLE & VANE	20'-102'	10'
B-6/1	SAMPLE & VANE	80'-244'	10'

SITE INVESTIGATION PLAN
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO

Ertec
The Earth Technology Corporation

SYMBOL	SAMPLE NO.	BLOWS/FT.	SOIL DESCRIPTION	Ø, DEGREES	% PASSING #200 SIEVE
	5-1, 4-3, 5-5, 5-7, 5-11, 5-13, 5-16, 5-20, 6-5, 5-37, 6-10	0, 0, 0, 0, 0, 0	†Seafloor at Elevation - 53.0 ft.		

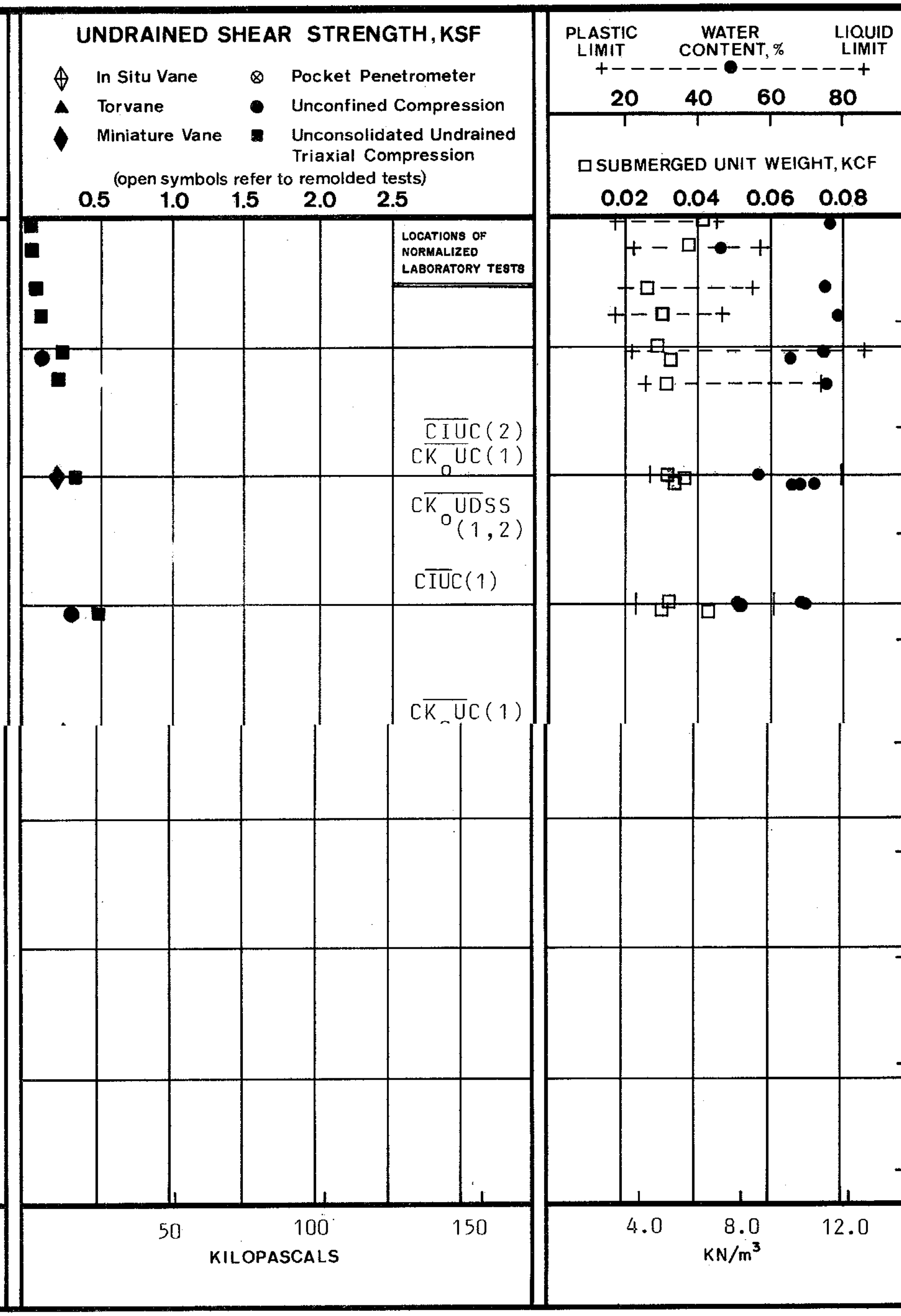
Depth scale (ft): 0, 20, 40, 60 … 340, 360, 380, 400

Very soft to stiff olive gray clay
- -with silt pockets at 6' and 26.5'
- -banded with brown, 15' to 30'
- -porous, 15' to 196.5'
- -with silt partings to 21'
- -expansive, 25' to 199'
- -gray below 30'

with clayey silt pockets at 35'

Note 1: Tests results are from laboratory work performed by Ertec. See appendix A for other test results.

Note 2: All $\overline{CIUC}$, $\overline{CK_oUC}$, and $\overline{CK_oUDSS}$ (tests were performed at OCR shown in parentheses.)



Job No : 82-200
Final Penetration : 253'
†Water Depth Measurement: 0800 hours on November 5, 1981
Date Completed : November 11, 1981
Field Engineer : G. L. Holloway

LOG OF BORING AND TEST RESULTS
BORINGS 4,5 AND 6, BLOCK 58
WEST DELTA AREA

Ertec

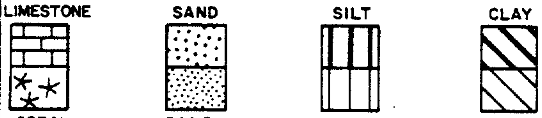
PLATE 6

KEY TO SOIL CLASSIFICATION AND SYMBOLS

SOIL TYPE

(Shown in Symbol Column)

LIMESTONE SAND SILT CLAY



CORAL SANDY SILTY CLAYEY

PEDOMINANT TYPE SHOWN HEAVY

SAMPLE TYPE

(Shown in Samples Column)

UNDISTURBED ROCK CORE SPLIT SPOON NO RECOVERY

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS (Major Portion Retained on No. 200 Sieve)

Includes (1) clean gravels & sand described as fine, medium or coarse, depending on distribution of grain sizes & (2) silty or clayey gravels & sands (3) fine grained low plasticity soils (Pl 10) such as sandy silts. Condition is rated according to relative density, as determined by lab tests or estimated from resistance to sampler penetration.

Descriptive Term	Penetration Resistance	Relative Density
Loose	0 -10	0 to 40 %
Medium Dense	10 -30	40 to 70 %
Dense	30 -50	70 to 90 %
Very Dense	over 50	90 to 100 %

** Blows /Ft., 140 hammer, 30" drop*

FINE GRAINED SOILS (Major Portion Passing No. 200 Sieve)

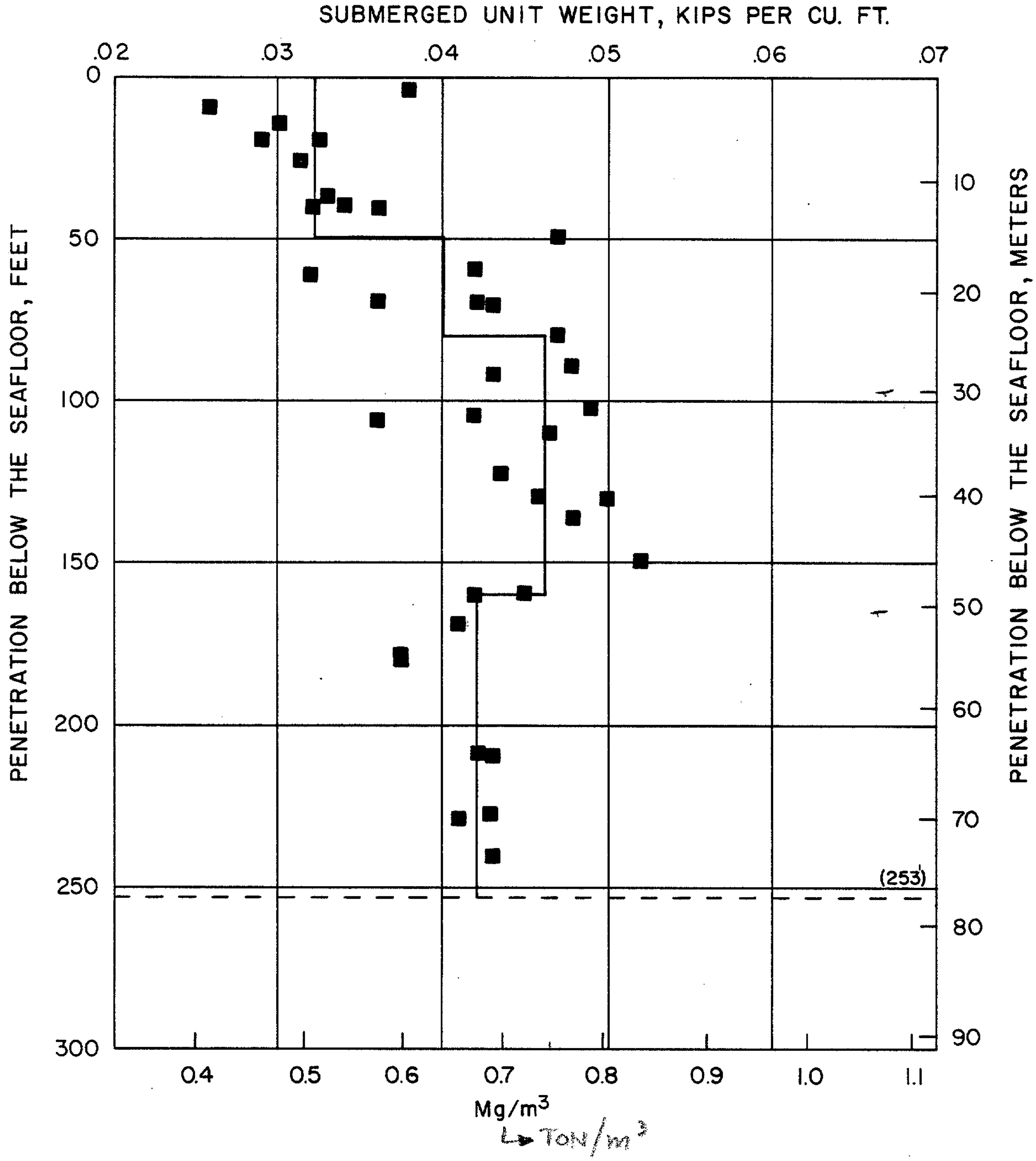
Includes (1) inorganic & organic silts & clays,(2) sandy, gravelly or silty clays, & (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings or by unconfined compression tests for soils with Pl 10.

Descriptive Term	Cohesive Shear Strength kips / sq. ft.	Cohesive Shear Strength kilopascals
Very Soft	Less than 0.25	Less than 10
Soft	0.25 to 0.50	10 to 25
Firm	0.50 to 1.00	25 to 50
Stiff	1.00 to 2.00	50 to 100
Very Stiff	2.00 to 4.00	100 to 200
Hard	4.00 and Higher	200 and Higher

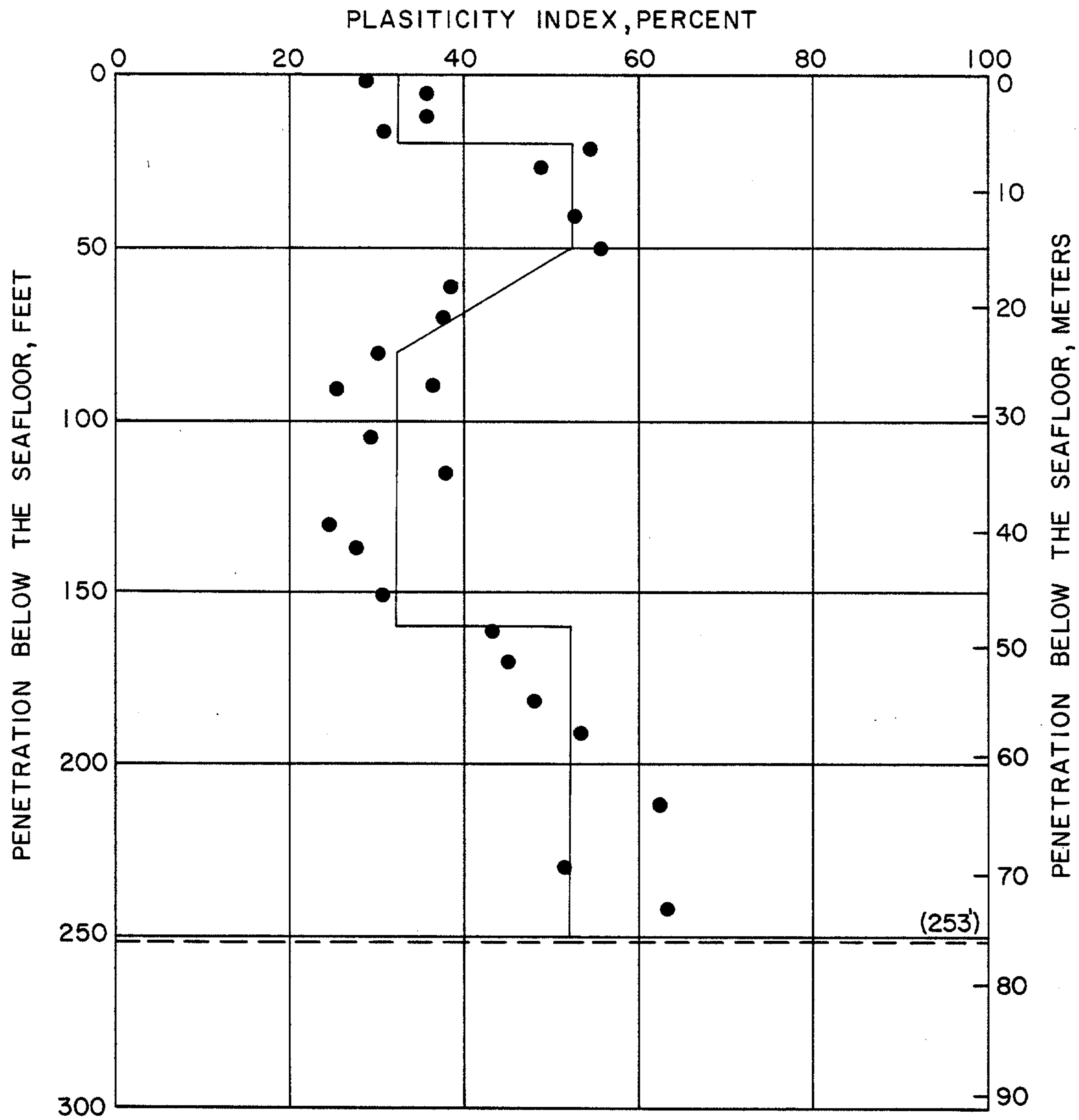
NOTE: SLICKENSIDED AND FISSURED CLAY MAY HAVE LOWER UNCONFINED COMPRESSIVE STRENGTHS THAN SHOWN ABOVE, BECAUSE OF PLANES OF WEAKNESS OR SHRINKAGE CRACKS; CONSISTENCY RATINGS OF SUCH SOILS ARE BASED ON HAND PENETROMETER READINGS.

TERMS CHARACTERIZING SOIL STRUCTURE

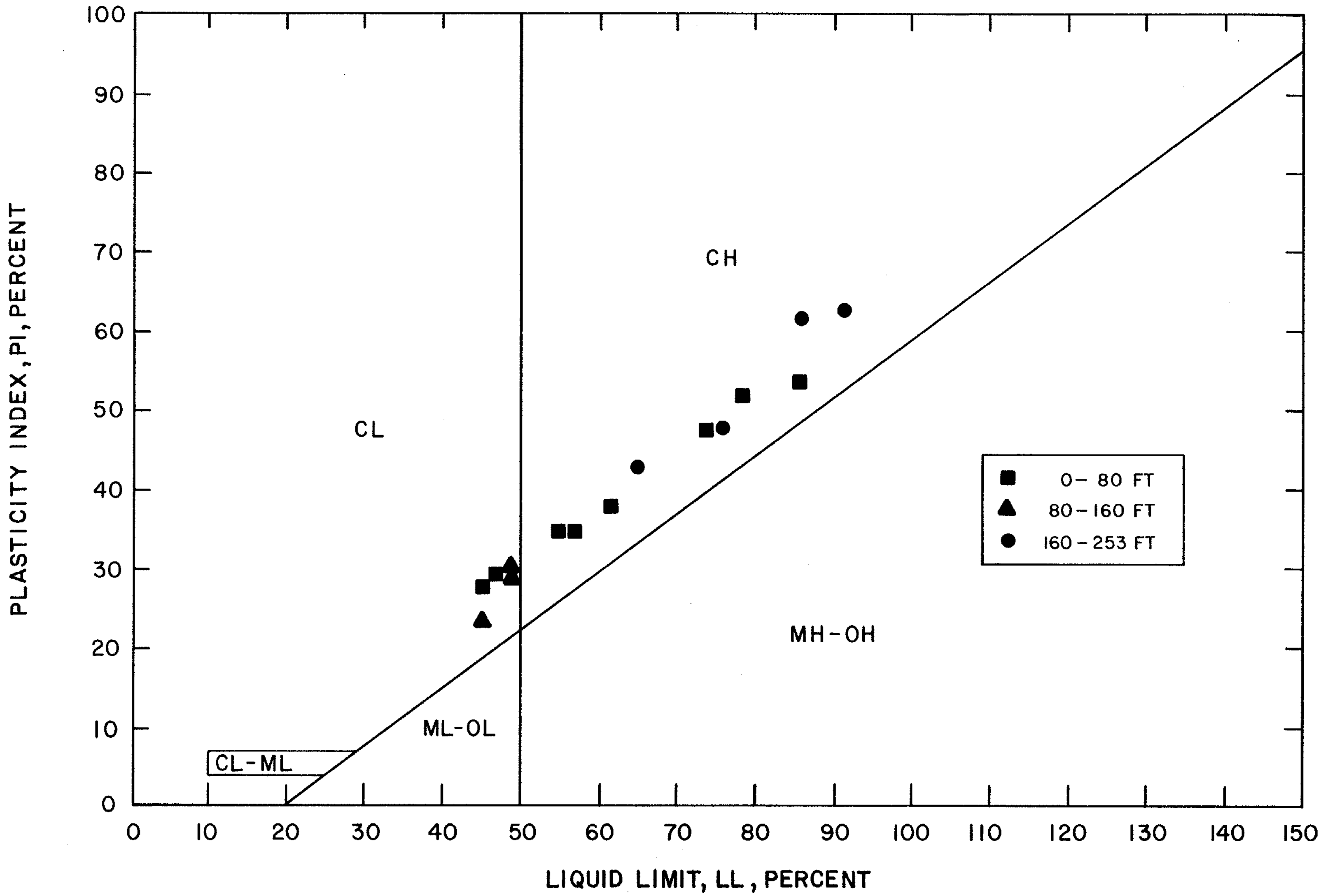
Parting:	paper thin in size
Seam:	1/8"-3" thick
Layer:	greater than 3"
Fissured:	containing shrinkage cracks, frequently filled with fine sand or silt ; usually more or less vertical
Sensitive:	pertaining to cohesive soils that are subject to appreciable loss of strength when remolded
Interbedded:	composed of alternate layers of different soil types
Laminated:	composed of thin layers of different soil types
Calcareous:	containing appreciable quantities of calcium carbonate
Well Graded:	having wide range in grain sizes and substantial amounts of all intermediate particle sizes
Poorly Graded:	predominately of one grain size, or having a range of sizes with some intermediate size missing
Flocculated:	pertaining to cohesive soils that exhibit a loose knit or flakey structure
Slickensided:	having inclined planes of weakness that are slick and glossy in appearance
Slightly Slickensided:	slickensides present at intervals of 1'-2'; soil does not easily break along these planes
Moderately Slickensided:	slickensides spaced at intervals of 1'-2'; soil breaks easily along these planes
Extremely Slickensided:	continuous and interconnected slickensides spaced at intervals of 4"-12"; soil breaks along the slickensides into pieces 3"-6" in size
Intensely Slickensided:	slickensides spaced at intervals of less than 4", continuous in all directions; soil breaks down along planes into nodules 1/4"-2" in size



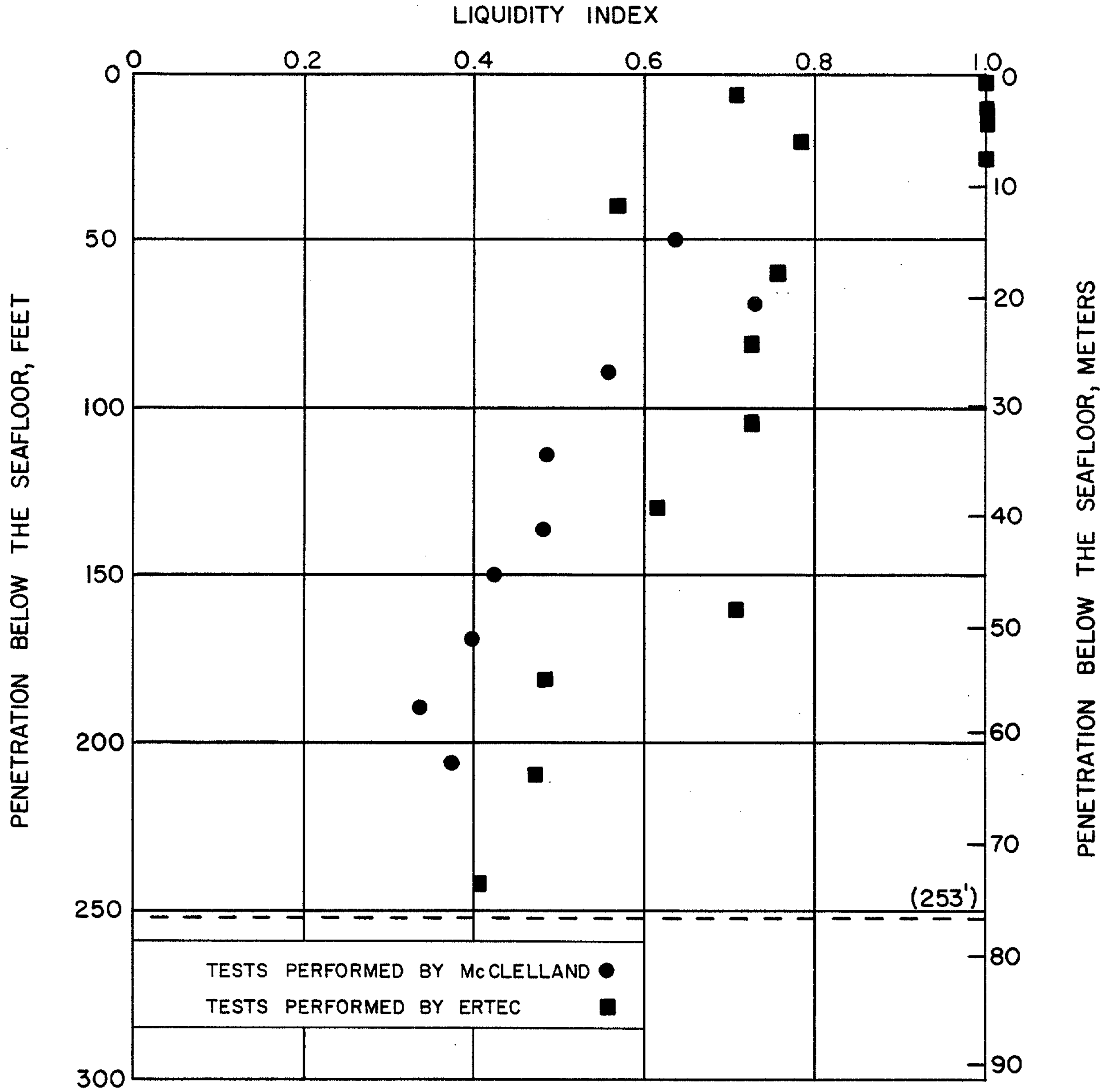
SUBMERGED UNIT WEIGHT PROFILE
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO



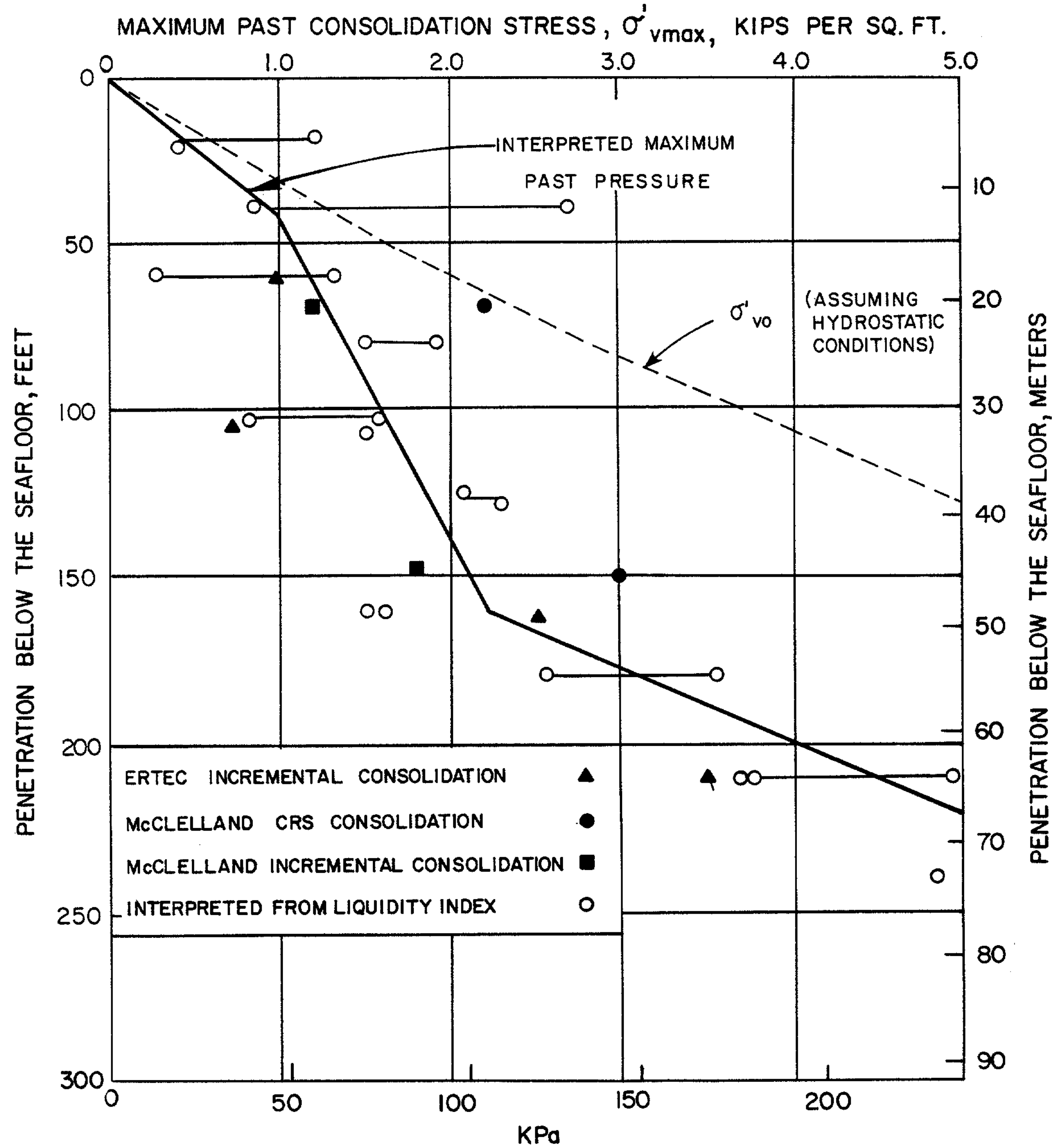
PLASITICITY INDEX VERSUS PENETRATION



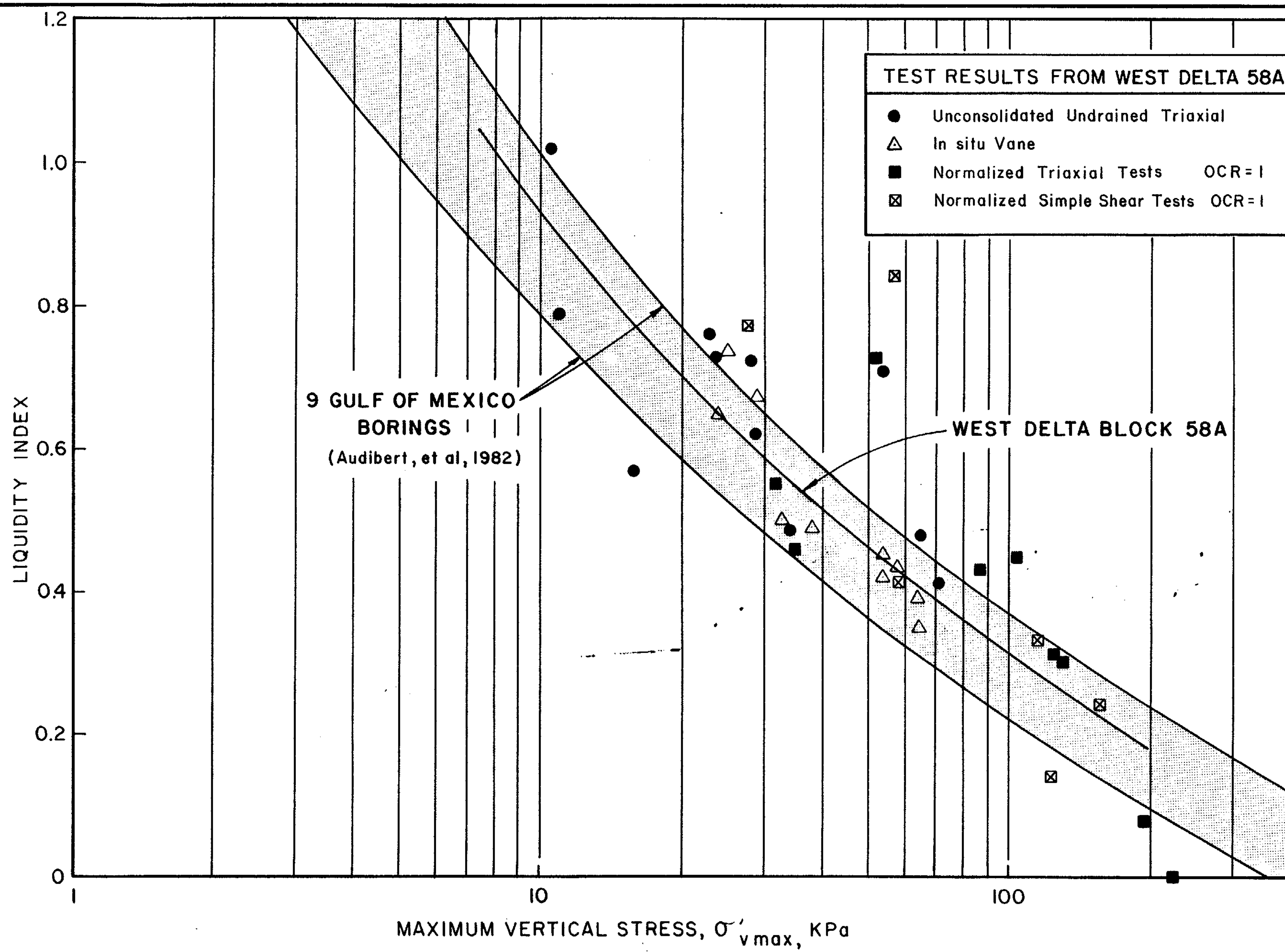
PLASTICITY CHART
BLOCK 58, WEST DELTA AREA



LIQUIDITY INDEX VERSUS PENETRATION
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO



MAXIMUM PAST PRESSURE VERSUS PENETRATION
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO



MAXIMUM PAST PRESSURE AS A FUNCTION OF LIQUIDITY INDEX

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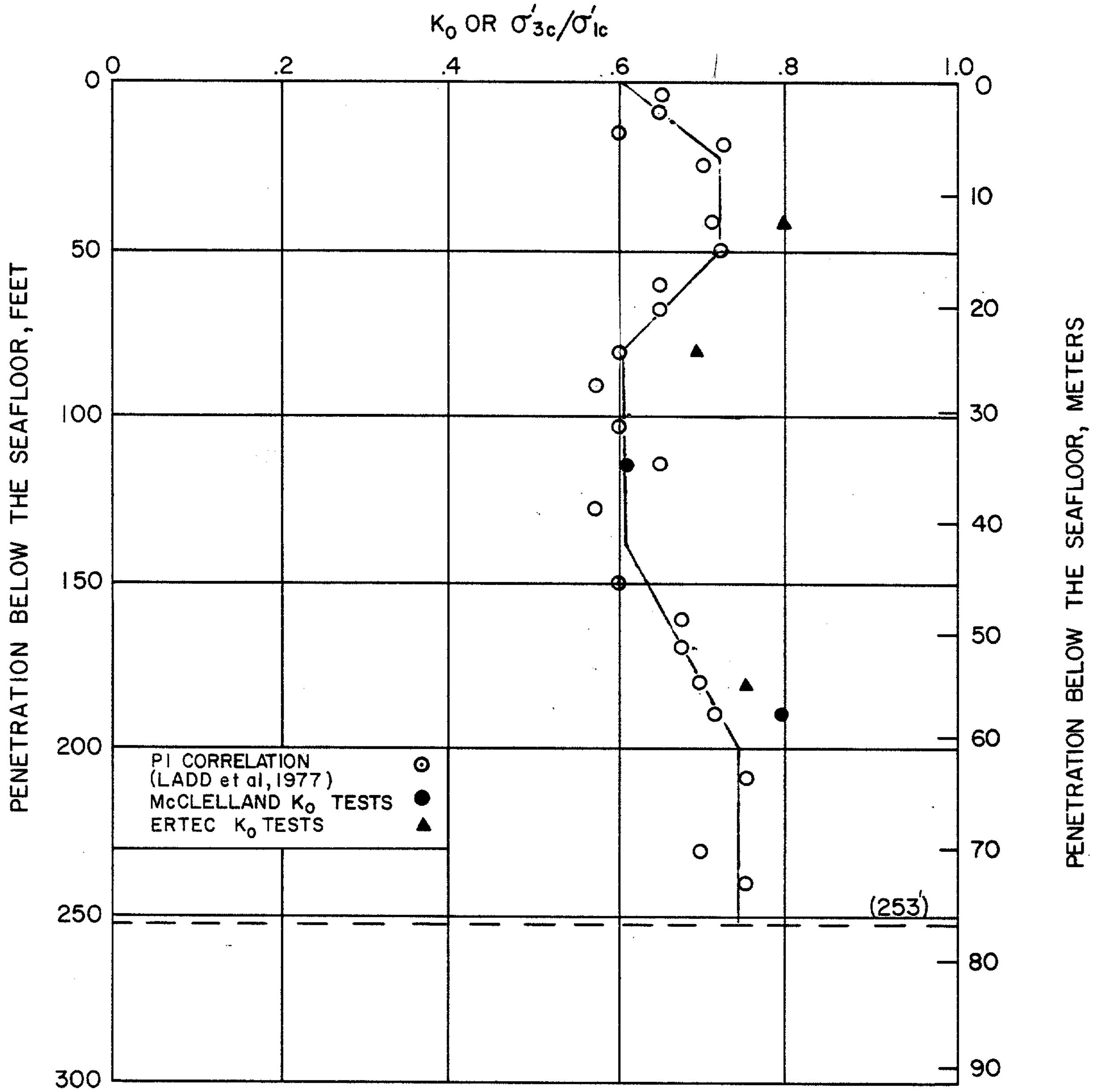
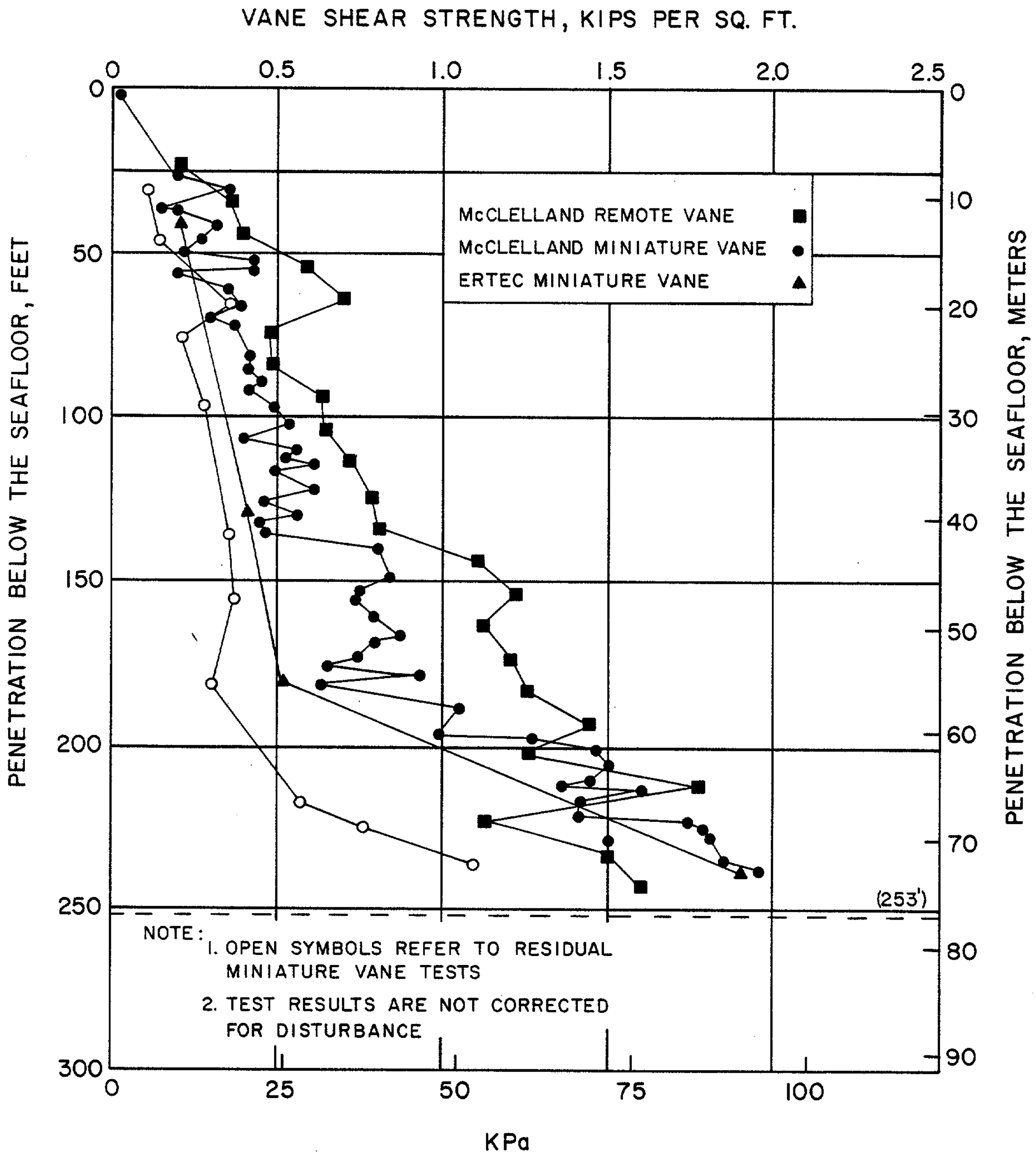
The Earth Technology Corporation

PLATE 12B

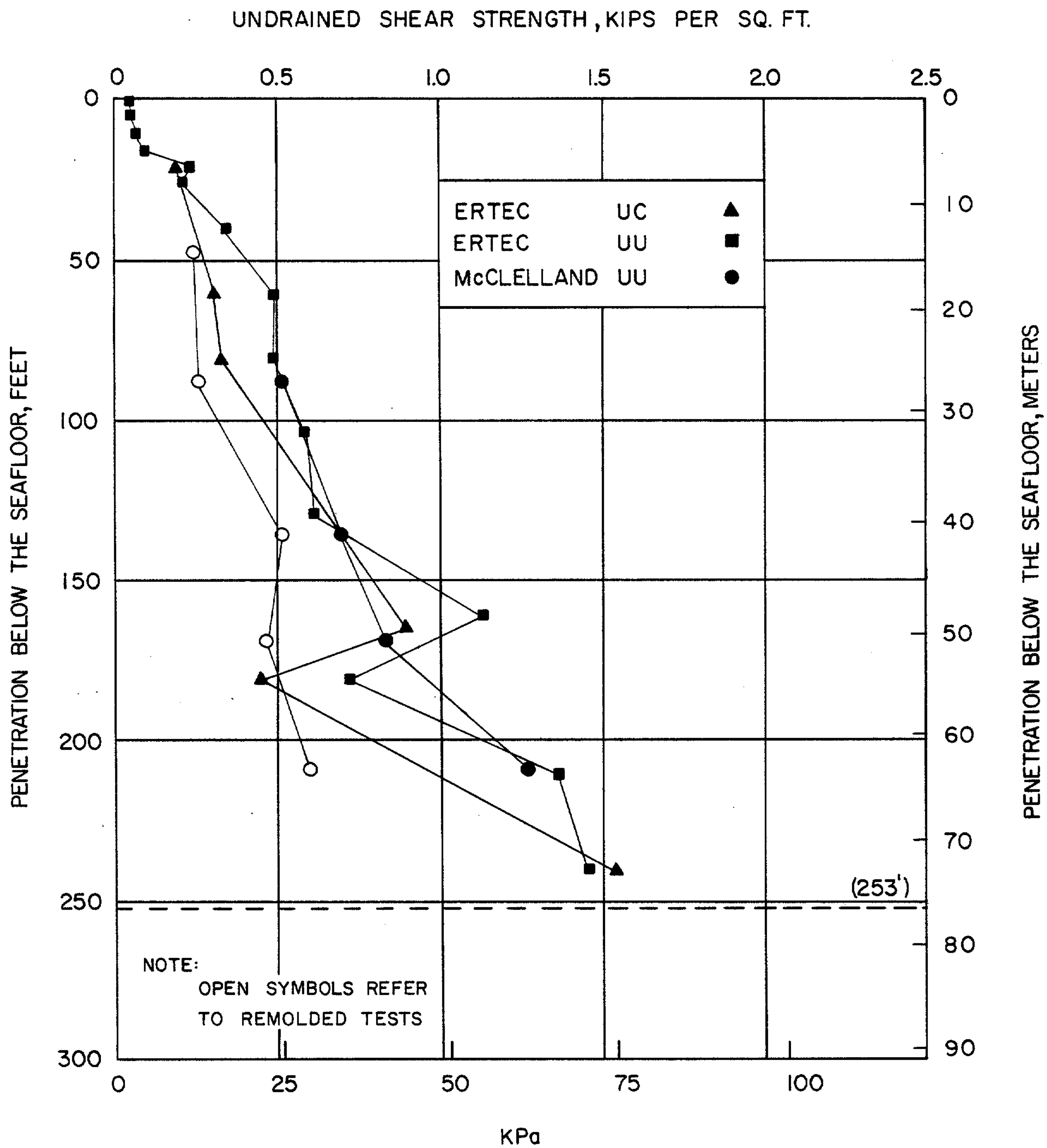


K_0 VERSUS PENETRATION
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO

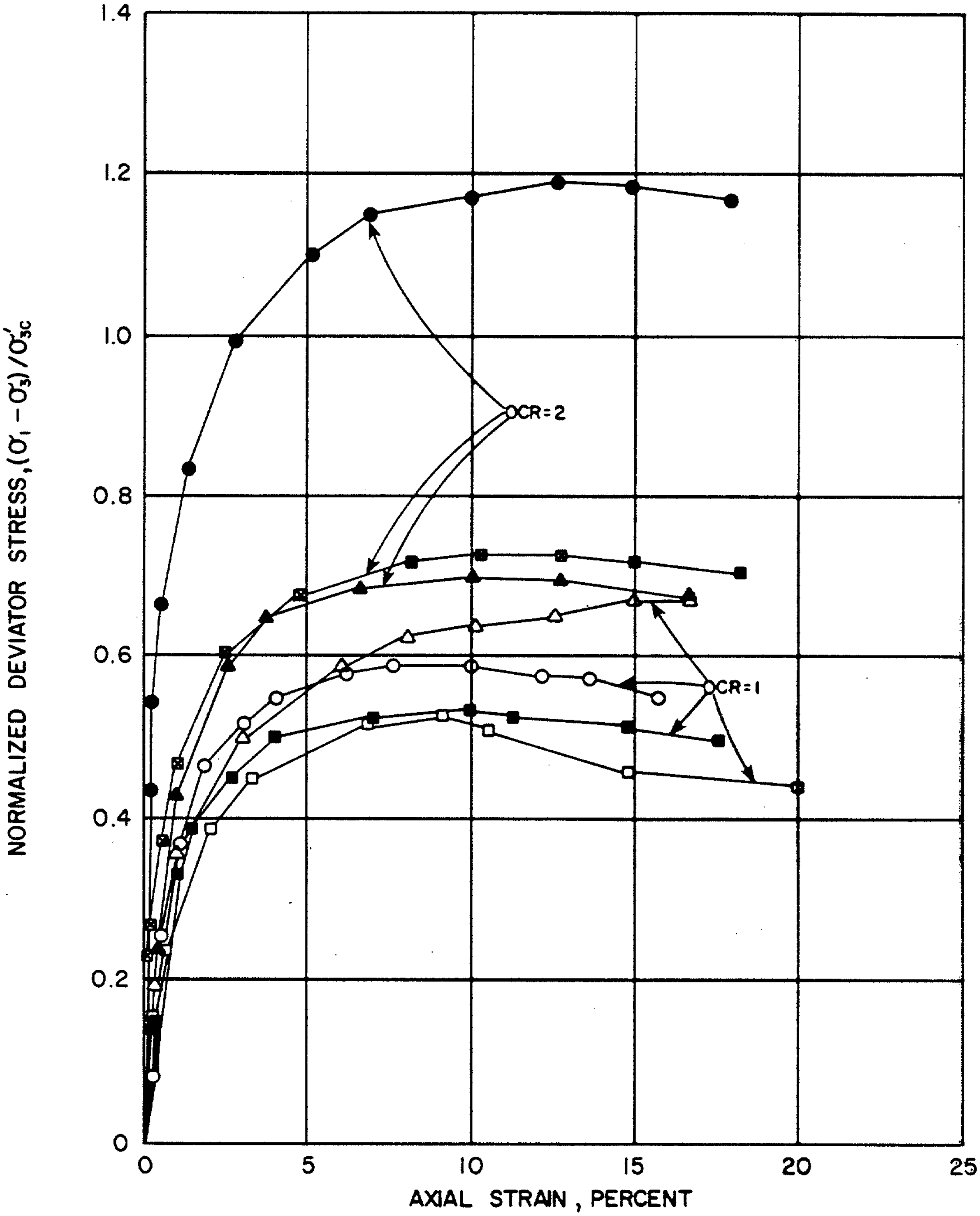
Ertec
The Earth Technology Corporation



MINIATURE VANE AND REMOTE VANE
SHEAR STRENGTH VERSUS PENETRATION

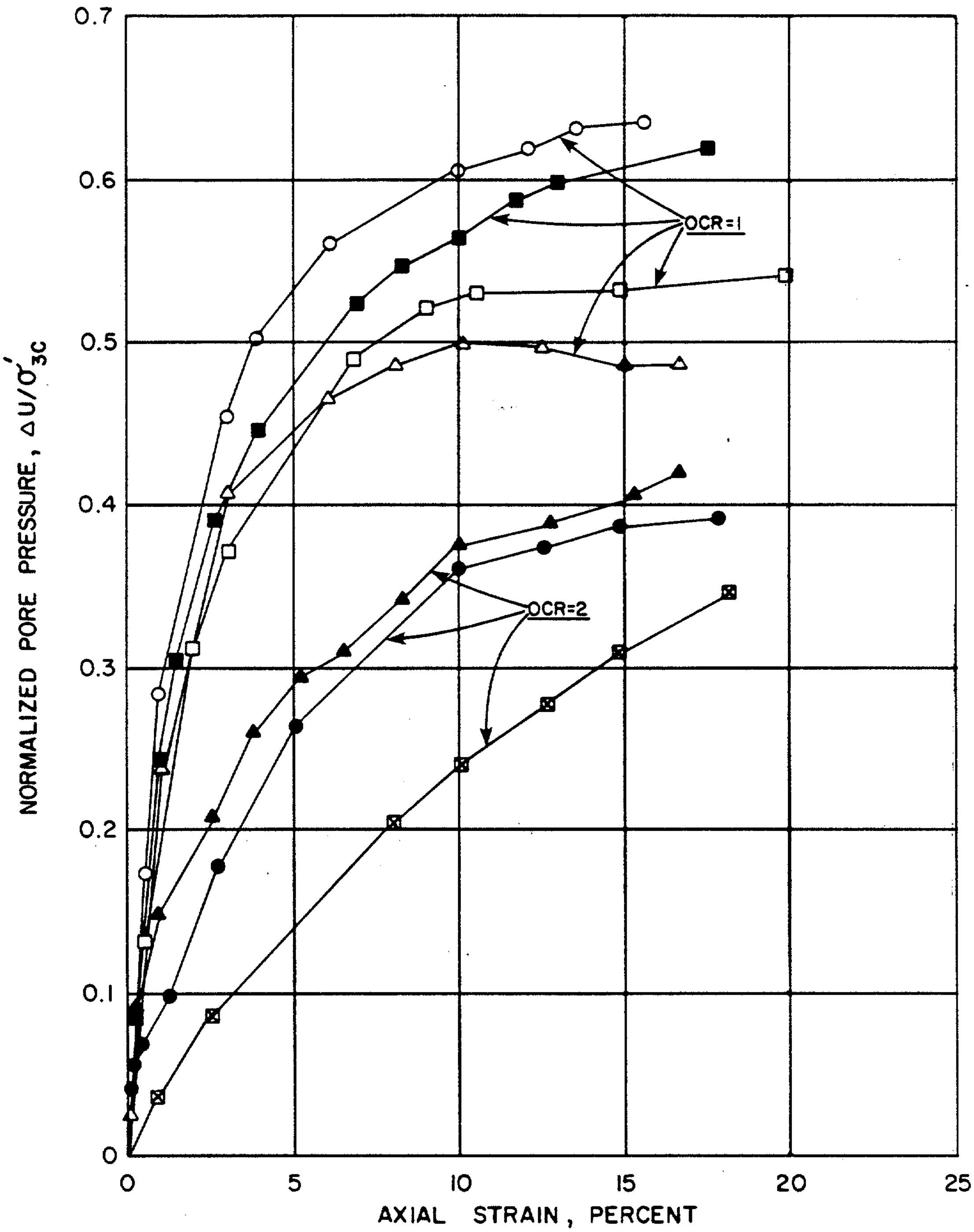


UNDRAINED SHEAR STRENGTH VERSUS PENETRATION
UU AND UC TESTS



SYMBOL	SAMPLE NUMBER	DEPTH M (FT)	σ'_{3c}, KPA (KSF)	OCR
□	37	18.69 – 18.81 (61.3 – 61.7)	193.1 (4.03)	1
△	61	32.32 – 32.47 (106.0 – 106.5)	103.4 (2.16)	1
○	45	49.09 – 49.27 (161.0 – 161.6)	358.5 (7.48)	1
■	62	64.21 – 64.39 (210.6 – 211.2)	503.3 (10.51)	1
▲	20	12.53 – 12.71 (41.1 – 41.7)	103.4 (2.16)	2
●	76	39.36 – 39.51 (129.1 – 129.6)	273.7 (5.76)	2
⊠	62	64.02 – 64.18 (210.0 – 210.5)	503.3 (10.51)	2

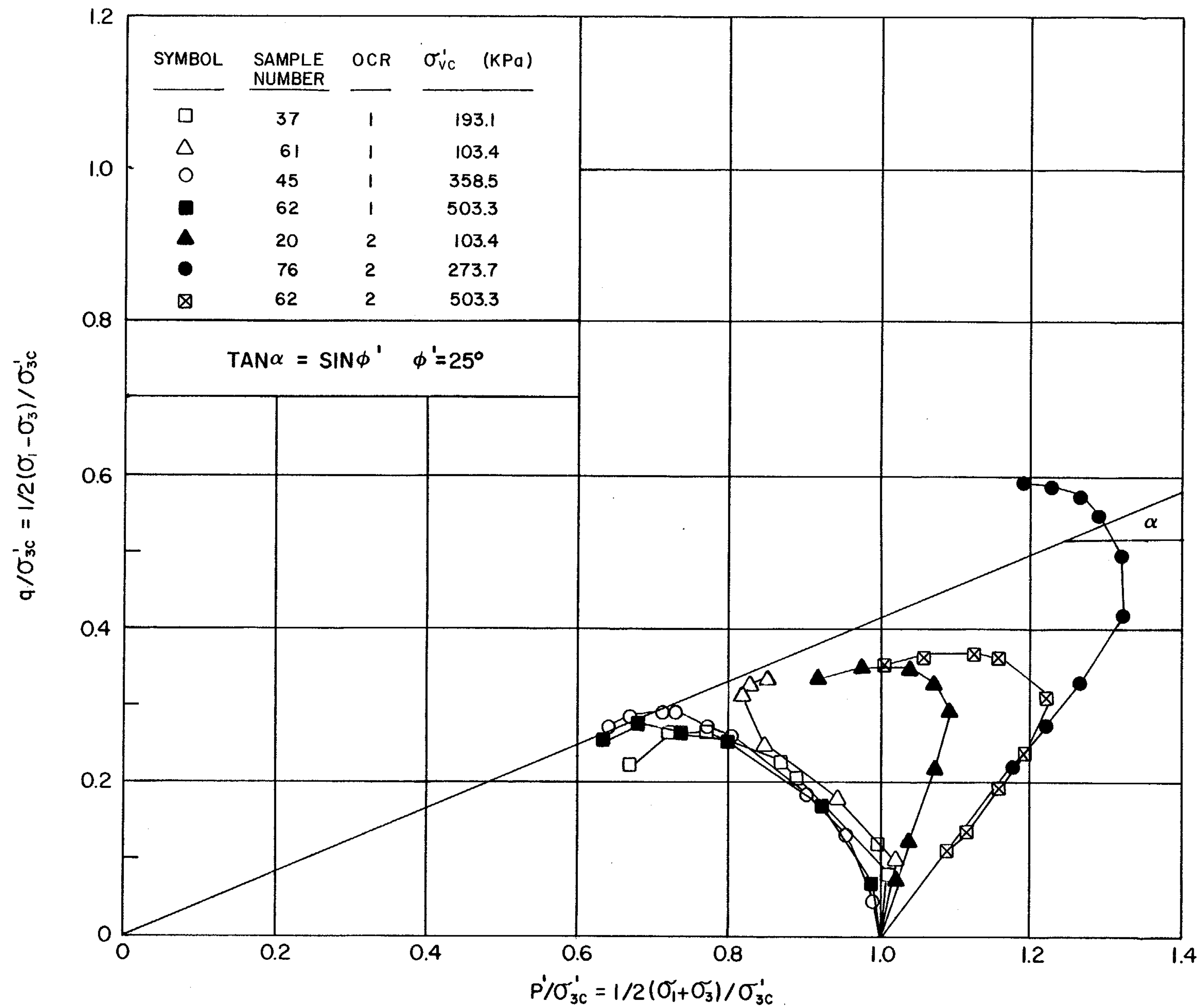
NORMALIZED STRESS-STRAIN CURVES - $CI\overline{U}C$ TESTS



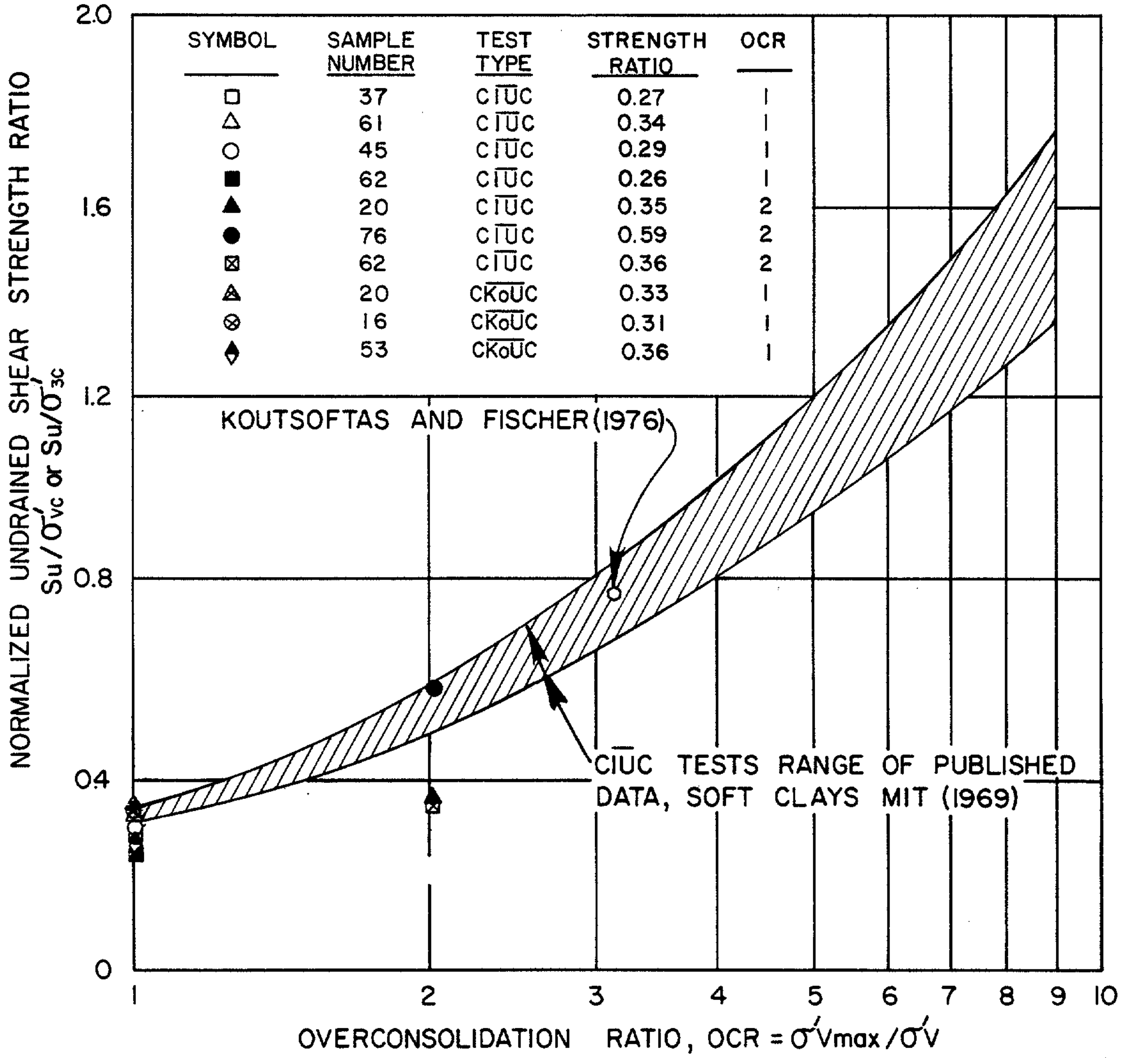
SYMBOL	SAMPLE NO.	DEPTH M(FT)	σ'_{3c} KPa (KSF)	OCR
□	37	18.69 - 18.81 (61.3 - 61.7)	193.1 (4.03)	1
△	61	32.32 - 32.47 (106.0 - 106.5)	103.4 (2.16)	1
○	45	49.09 - 49.27 (161.0 - 161.6)	358.5 (7.48)	1
■	62	64.21 - 64.39 (210.6 - 211.2)	503.5 (10.51)	1
▲	20	2.53 - 12.7 (41.1 - 41.7)	103.4 (2.16)	2
●	76	39.36 - 39.51 (129.1 - 129.6)	273.7 (5.76)	2
⊠	62	64.04 - 64.18 (210.0 - 210.5)	503.3 (10.51)	2

NORMALIZED PORE PRESSURE — STRAIN CURVES — $\overline{\text{CIUC}}$ TESTS

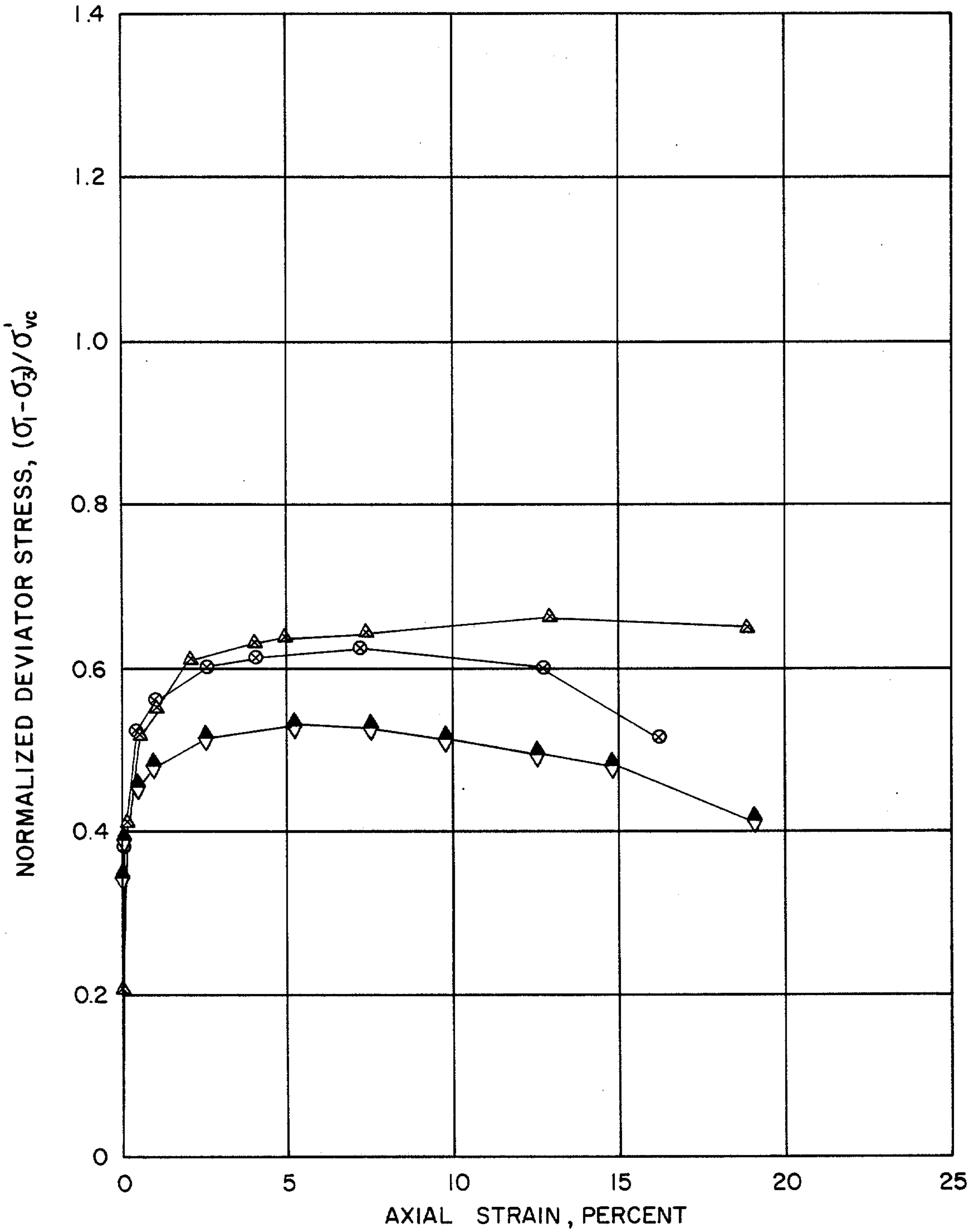
Ertec
The Earth Technology Corporation



NORMALIZED EFFECTIVE STRESS PATHS - $C\overline{IU}C$ TESTS



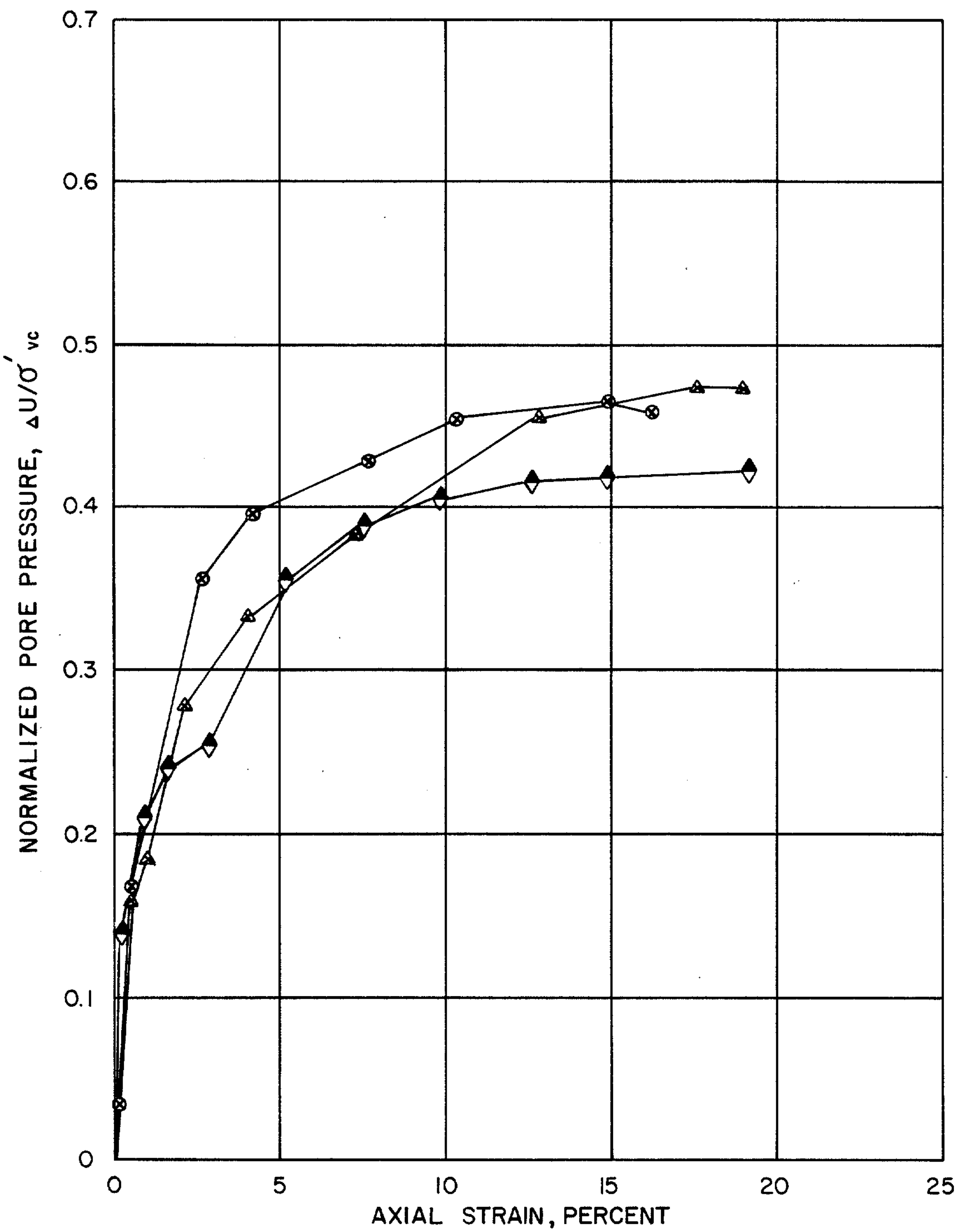
NORMALIZED UNDRAINED SHEAR STRENGTH
VERSUS
OVERCONSOLIDATION RATIO
CIUC AND CKoUC TESTS



SYMBOL	SAMPLE NUMBER	DEPTH M (FT.)	σ'_{vc} KPa(KSF)	OCR
△	20	12.35 - 12.50(40.5 - 41.0)	75.8 (1.58)	1
⊗	16	24.94 - 25.00(81.3 - 82.0)	196.5 (4.10)	1
◆	53	54.91 - 55.06(180.1 - 181.6)	361.3 (7.54)	1

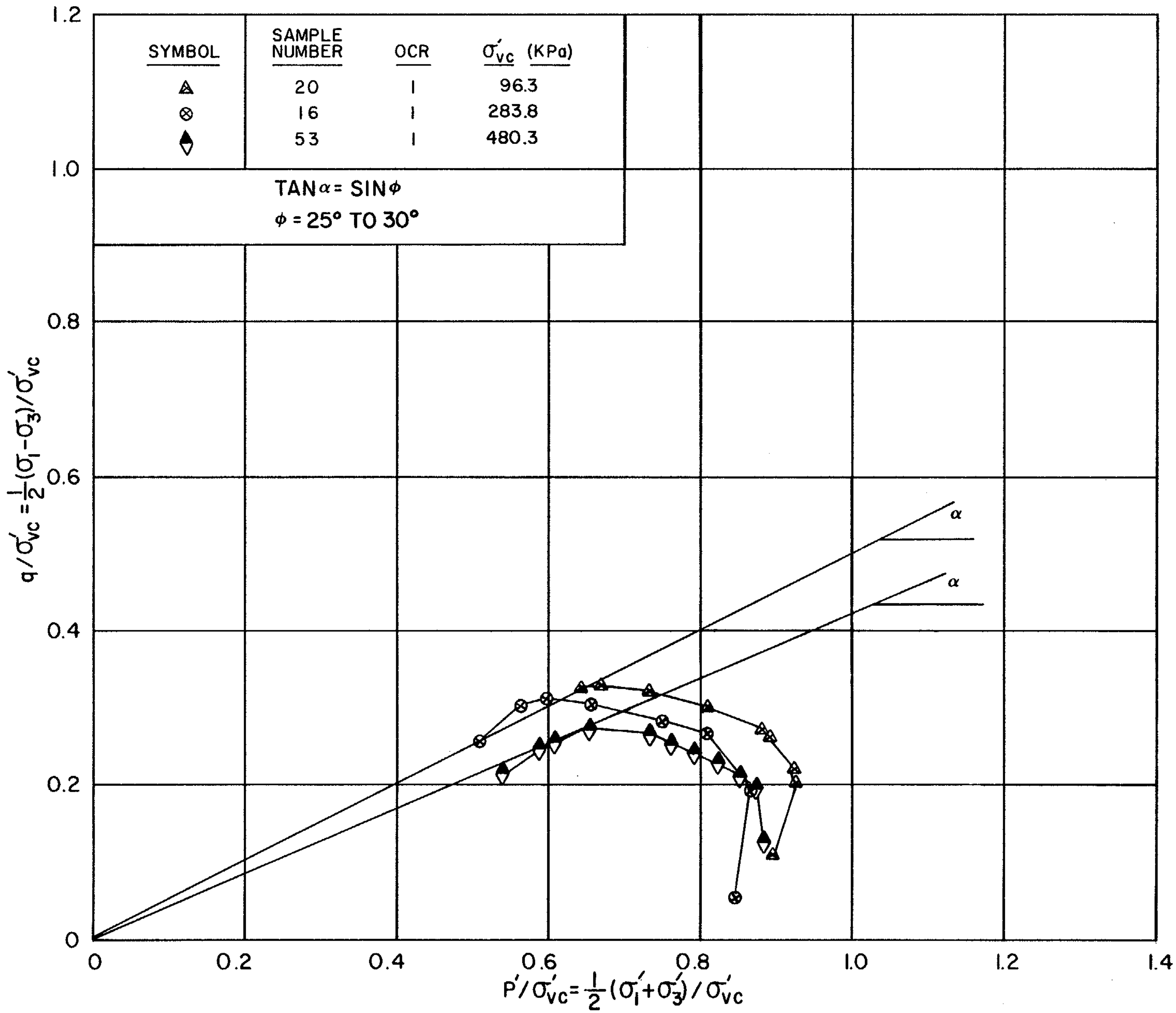
NORMALIZED STRESS-STRAIN CURVES-CK̄oŪC TESTS

Ertec
The Earth Technology Corporation

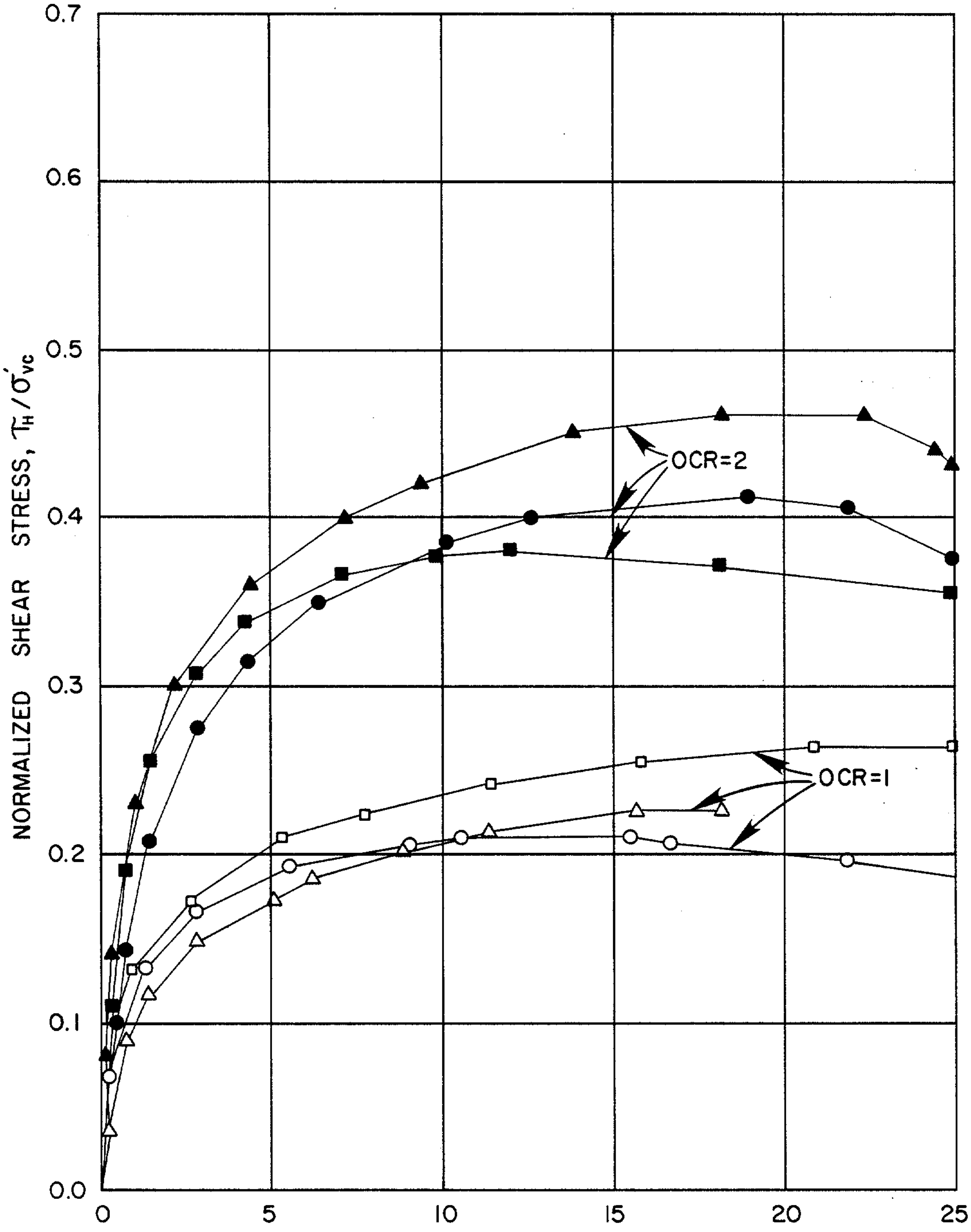


SYMBOL	SAMPLE NUMBER	DEPTH M (FT.)	σ'_{vc} KPa (KSF)	OCR
△	20	12.35 - 12.50 (40.5 - 41.0)	75.8 (1.58)	1
⊗	16	24.94 - 25.00 (81.8 - 82.0)	196.5 (4.10)	1
◆	53	54.91 - 55.06 (180.0 - 180.6)	361.3 (7.54)	1

NORMALIZED PORE PRESSURE - STRAIN CURVES $CK_o\overline{UC}$ TESTS



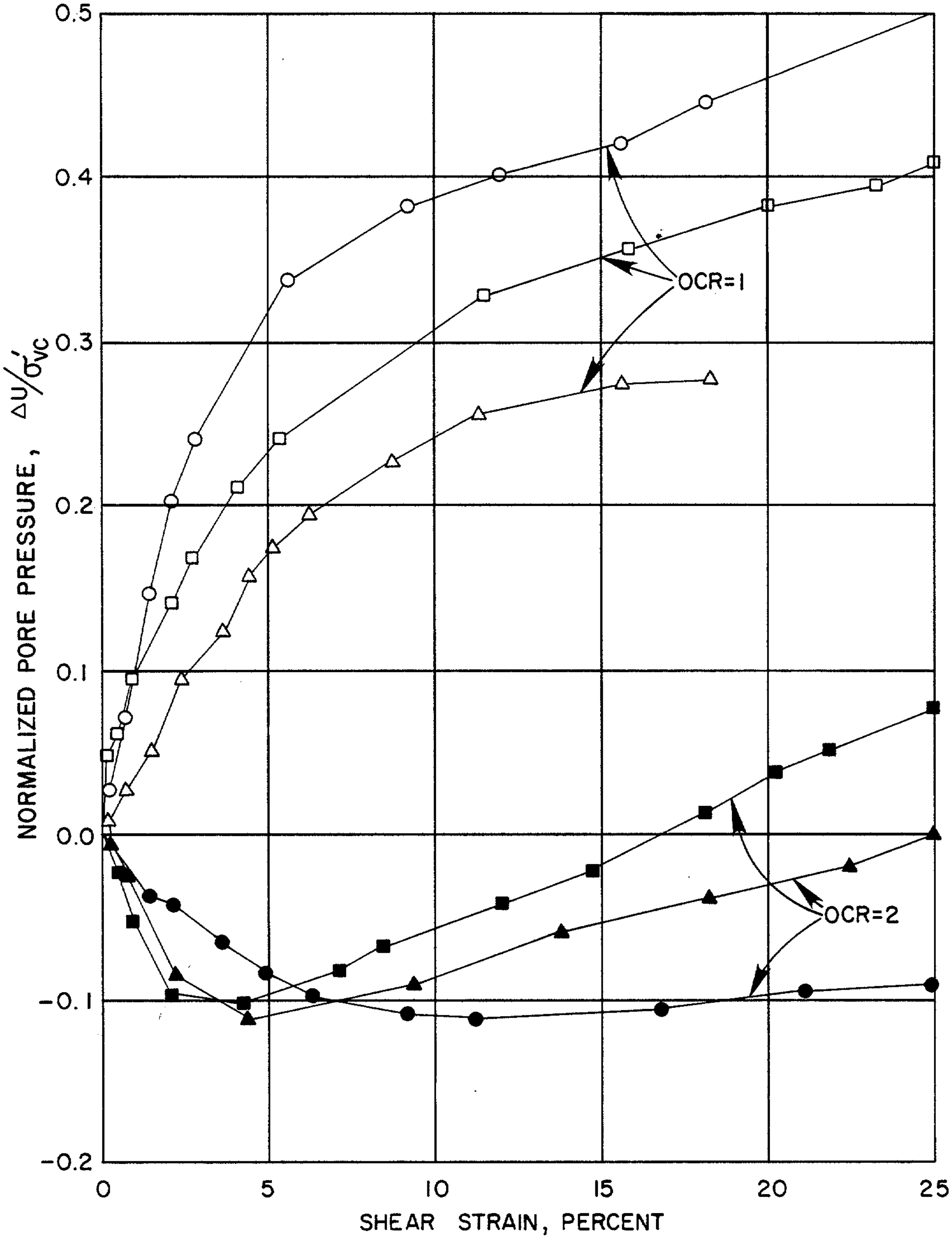
NORMALIZED EFFECTIVE STRESS PATHS — $\overline{\text{CKoUC}}$ TESTS



SYMBOL	SAMPLE NUMBER	DEPTH M (FT.)	σ'_{vc} KPa (KSF)	OCR
□	20	12.71 – 12.80 (41.7 – 42.0)	103.5 (2.16)	1
△	20	12.71 – 12.80 (41.7 – 42.0)	103.5 (2.16)	2
○	76	39.02–39.12 (128.0 – 128.3)	275.9 (5.76)	1
■	26	31.5 –31.59 (103.4 – 103.6)	275.9 (5.76)	2
▲	62	64.24–64.27 (210.7 – 210.8)	503.5 (10.51)	1
●	70	73.17–73.23 (240.0 – 240.2)	440.0 (9.19)	2

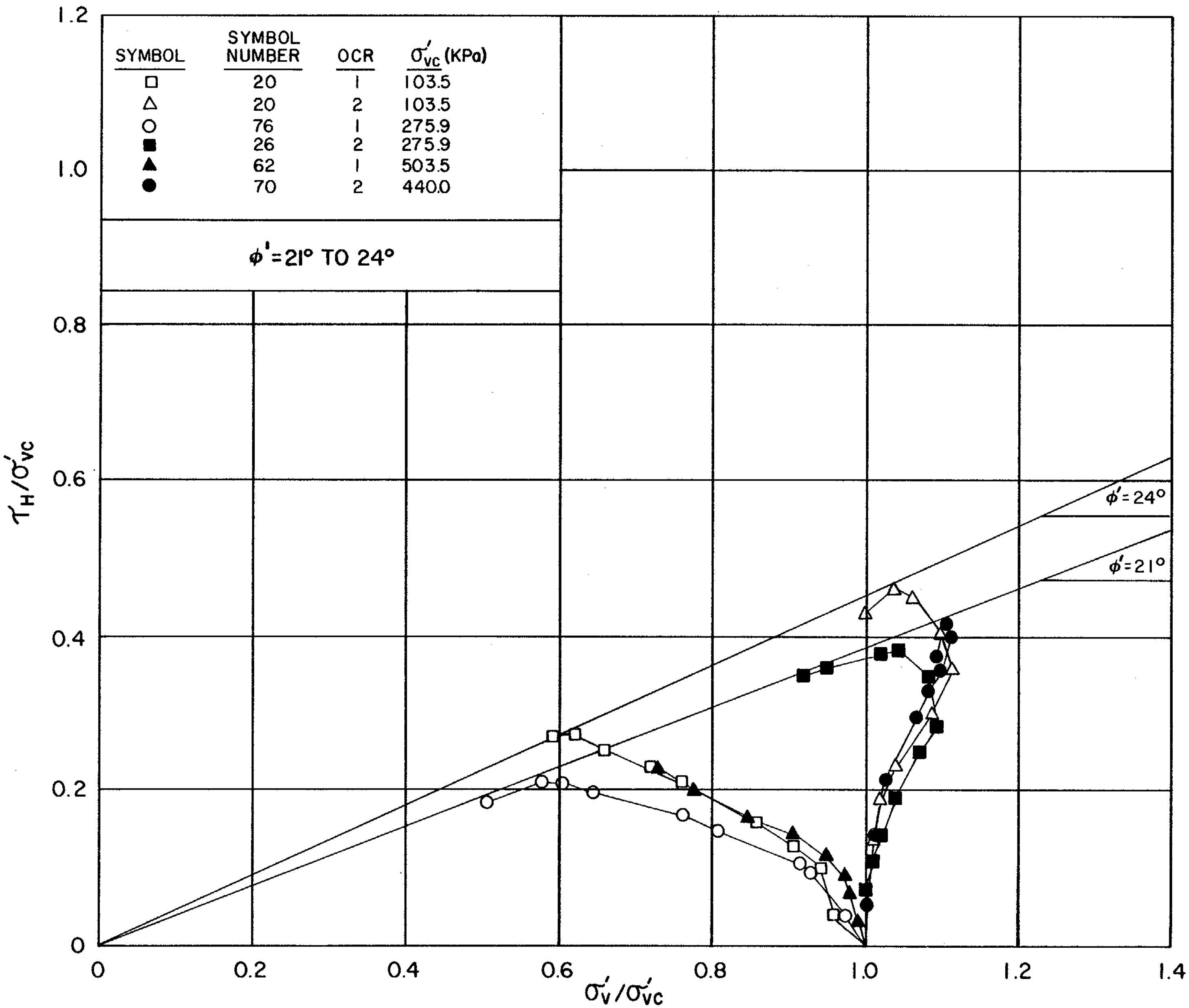
NORMALIZED STRESS STRAIN CURVES $CK_o\overline{UD}SS$ TESTS

Ertec
The Earth Technology Corporation



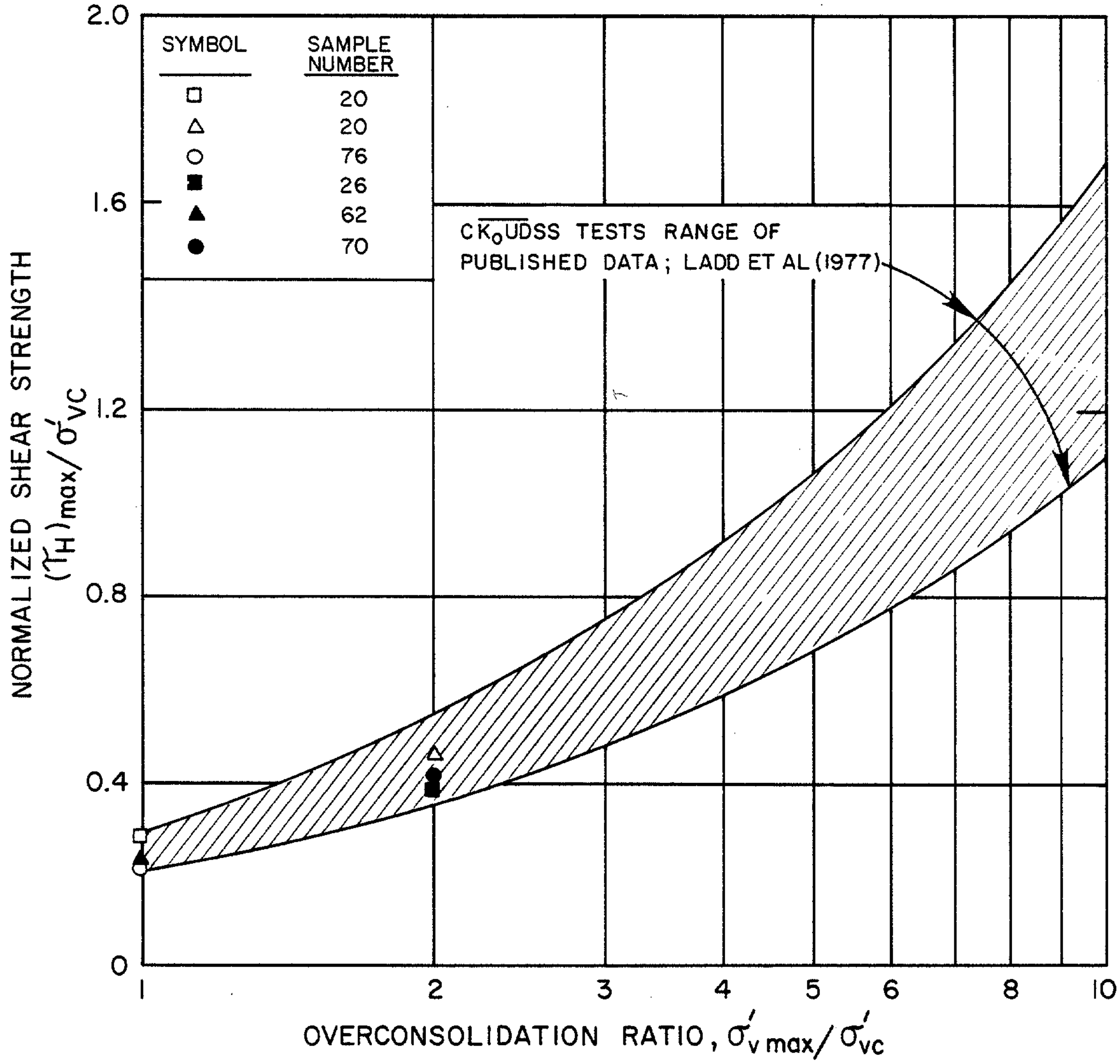
SYMBOL	SAMPLE NUMBER	DEPTH M (FT)	σ'_{vc} KPa (KSF)	OCR
□	20	12.71 - 12.80 (41.7 - 42.0)	103.5 (2.16)	1
△	20	12.71 - 12.80 (41.7 - 42.0)	103.5 (2.16)	2
○	76	39.02-39.12 (128.0 - 128.3)	275.9 (5.76)	1
■	26	31.52-31.59 (103.4 - 103.6)	275.9 (5.76)	2
▲	62	64.24-64.27 (210.7 - 210.8)	503.5 (10.51)	1
●	70	73.17-73.23 (240.0 - 240.2)	440.0 (9.19)	2

NORMALIZED PORE PRESSURE-STRAIN CURVES-CKoUDSS TESTS

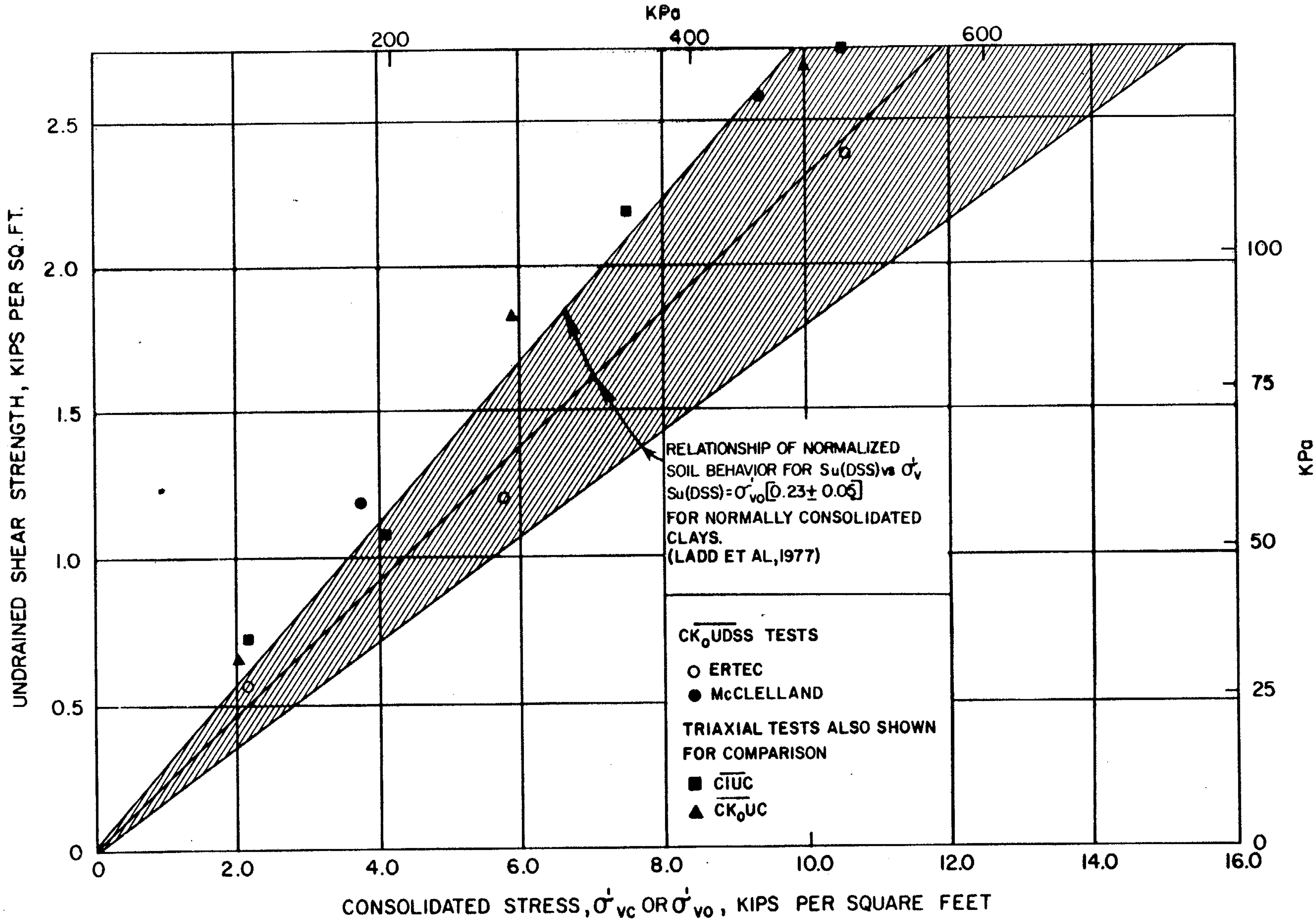


NORMALIZED EFFECTIVE STRESS PATHS – $\overline{\text{CKoUDSS}}$ TESTS

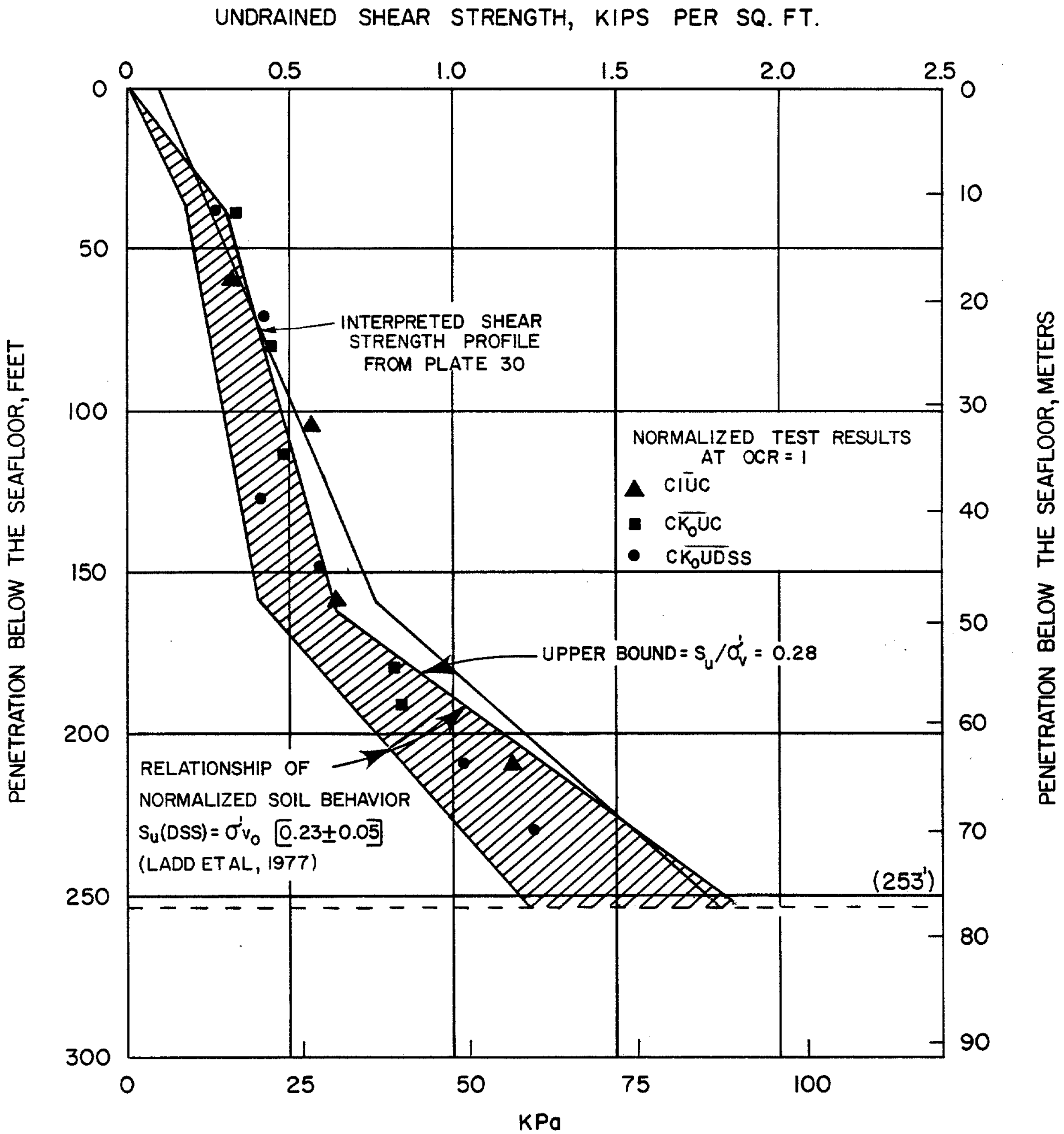
Ertec
The Earth Technology Corporation



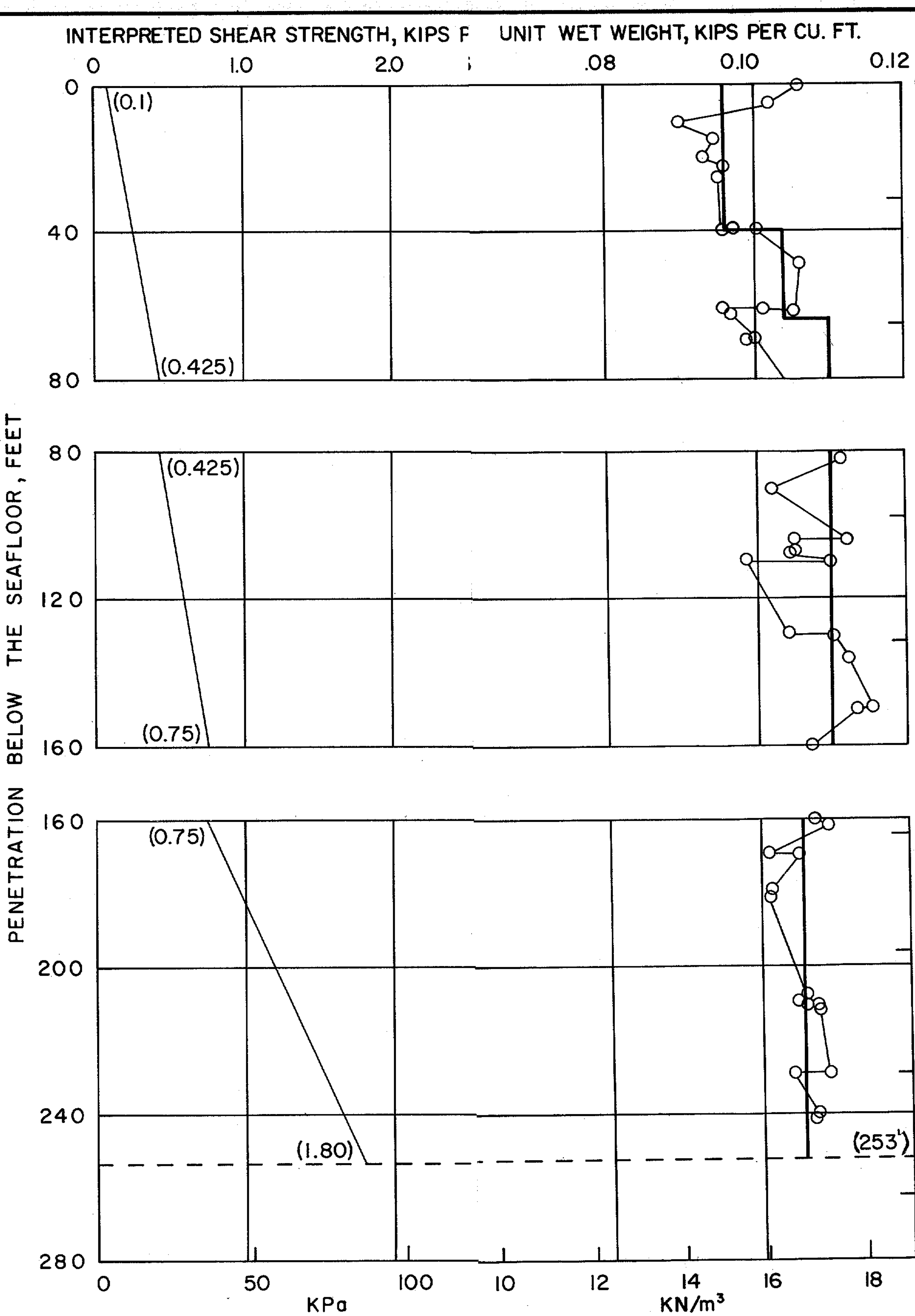
NORMALIZED SHEAR STRENGTHS
VERSUS
OVERCONSOLIDATION RATIO
C $\overline{K_0 U}$DSS TESTS



EFFECTIVE STRESS HISTORY VERSUS UNDRAINED SHEAR STRENGTH
$\mathrm{CI\overline{U}C}$, $\mathrm{C\overline{K_0 U}C}$ AND $\mathrm{C\overline{K_0 U}DSS}$ TESTS



COMPARISON OF UNDRAINED SHEAR STRENGTH PROFILES

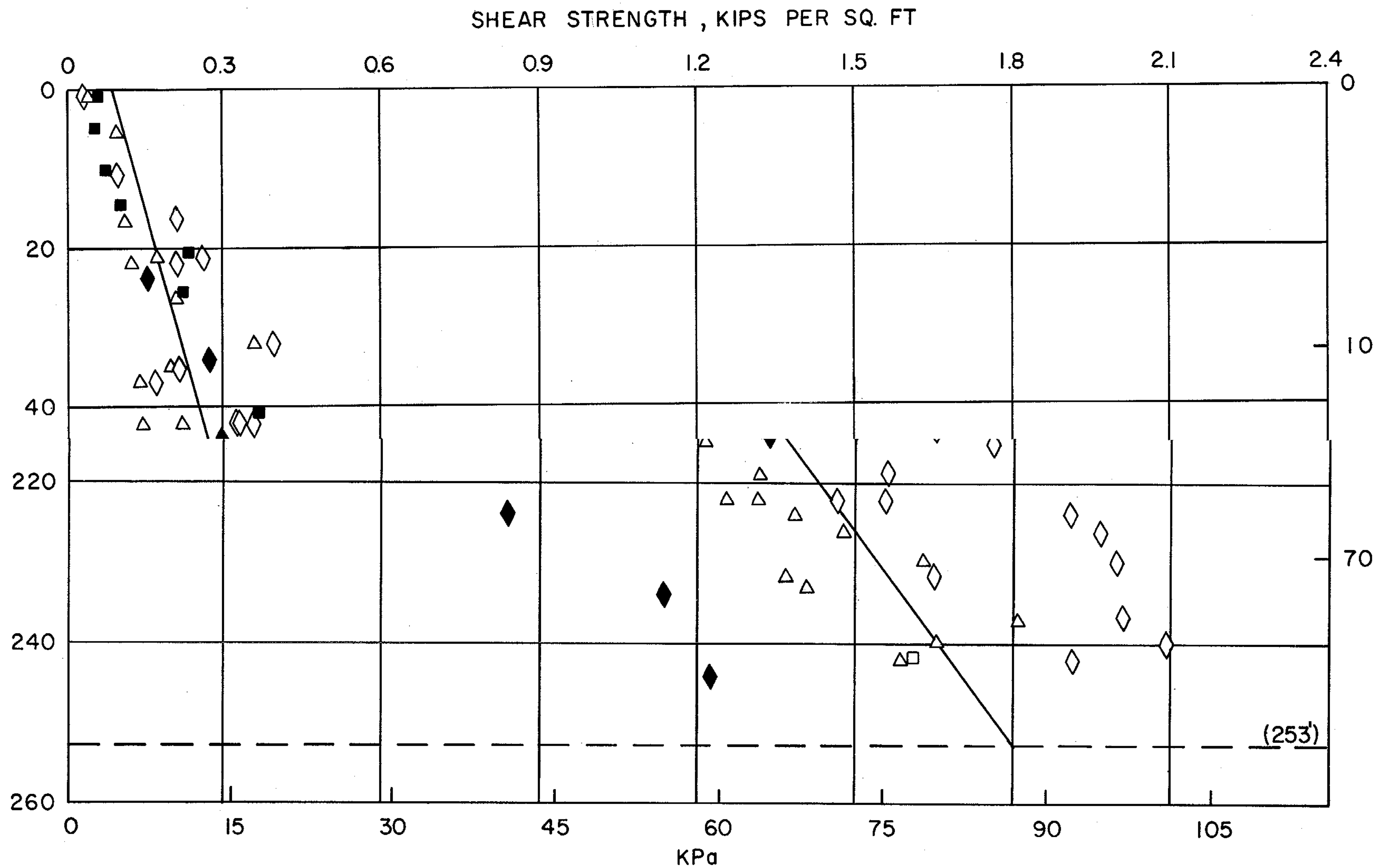


NOTE:

NUMBERS WITHIN PARENTHESIS
HAND MARGINS INDICATE PENE
NUMBER ON THE CURVES INDI
INTERPRETED SHEAR STRENG

IES FOR IDEALIZED STRATIGRAPHY
K 58, WEST DELTA AREA
GULF OF MEXICO

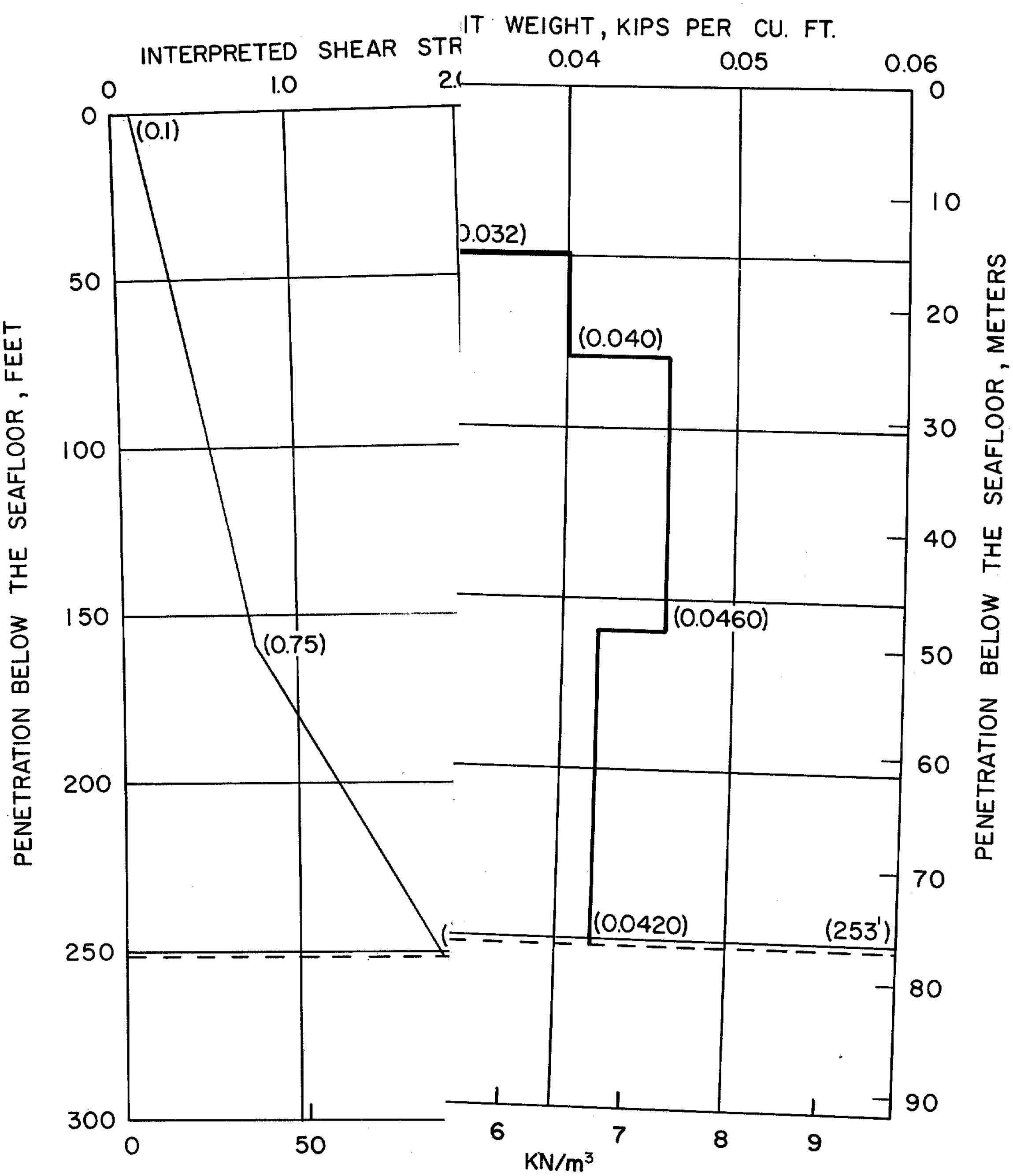
Ertec
The Earth Technology Corporation



DESIGN SHEAR STRENGTH PROFILE
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO

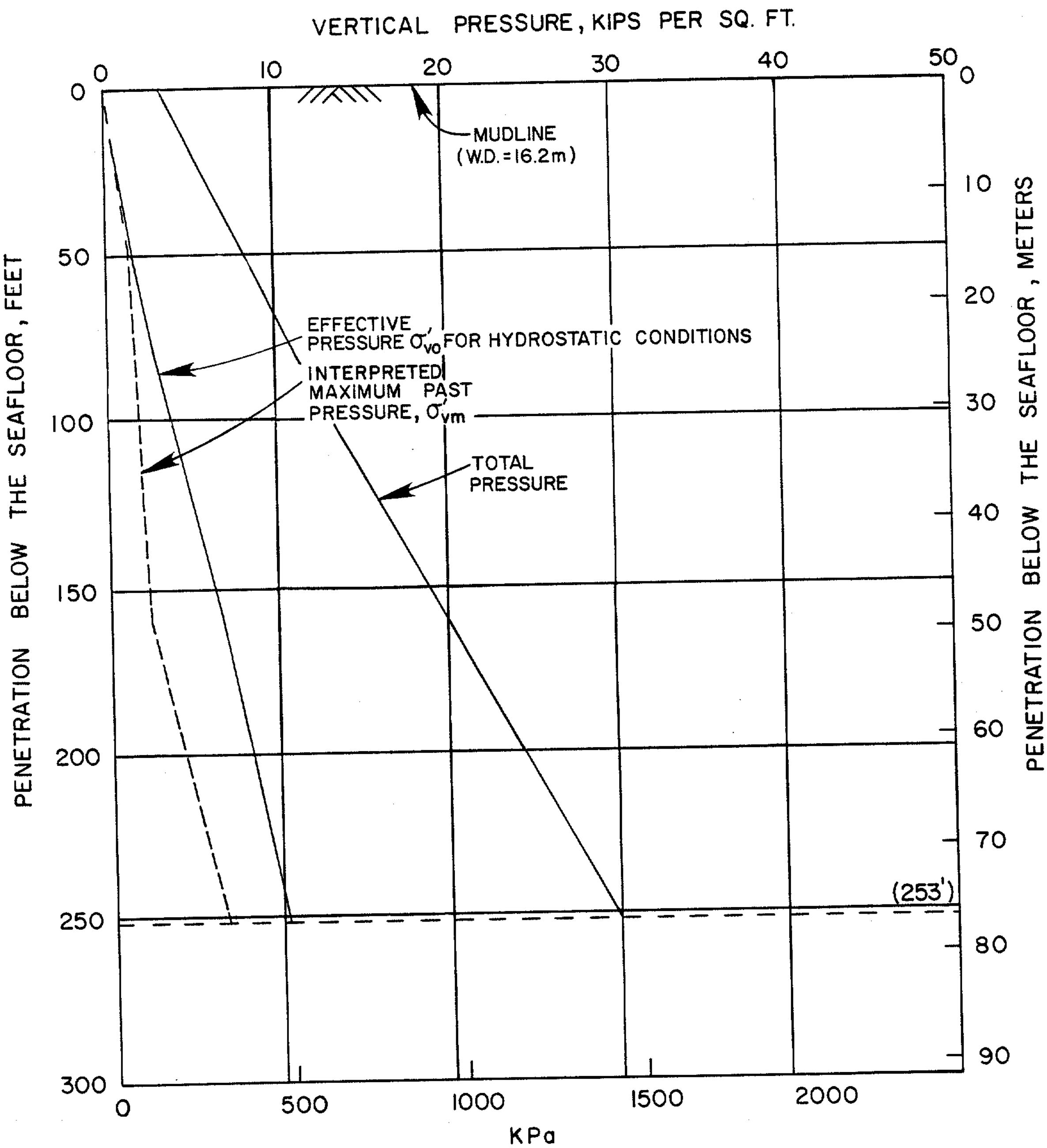
Ertec
The Earth Technology Corporation

PLATE 30

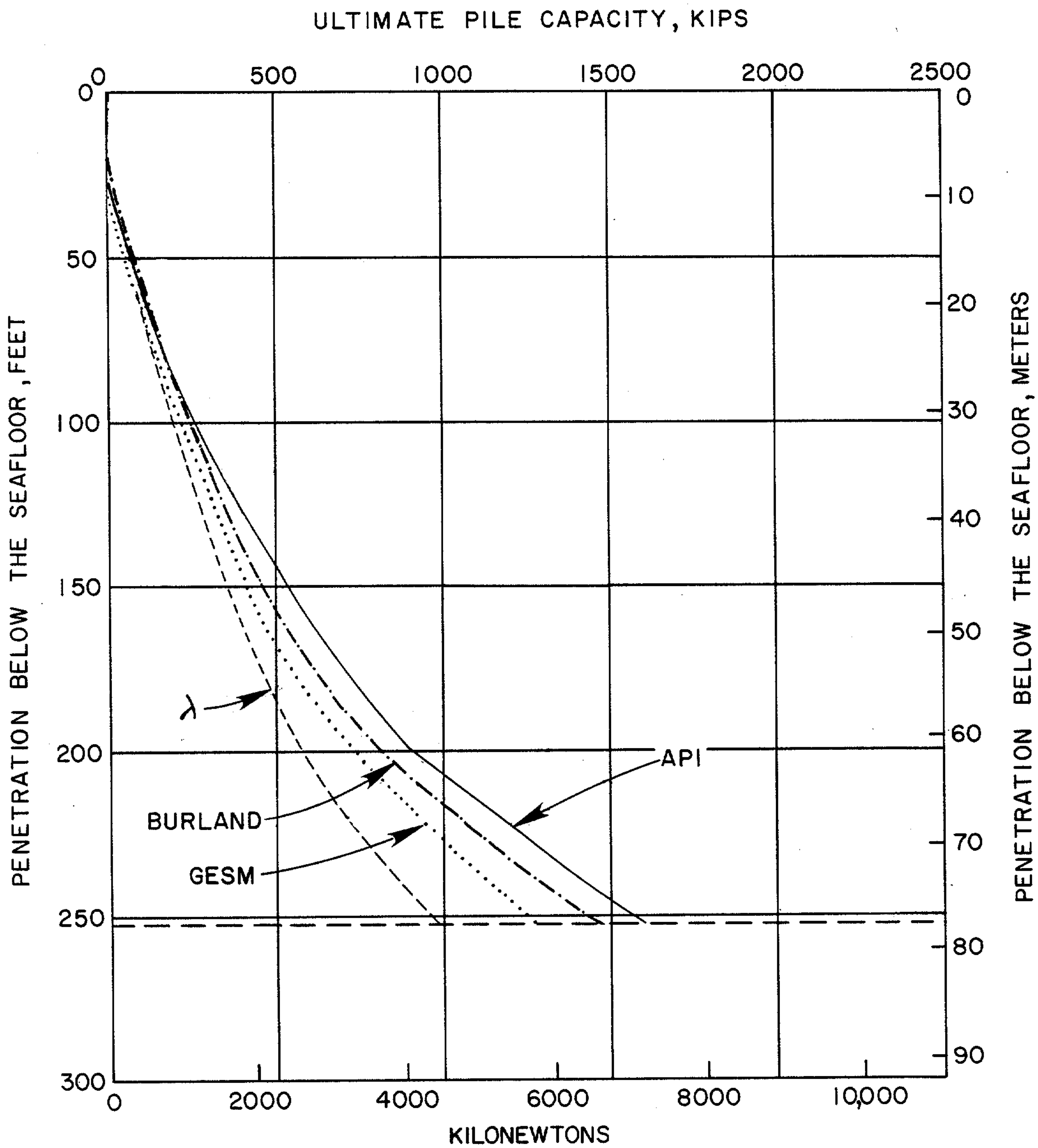


PILE DESIGN DATA
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO

Ertec
The Earth Technology Corporation



VERTICAL PRESSURE DISTRIBUTION



ULTIMATE PILE CAPACITY CURVES
30-IN DIAMETER PIPE PILE
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO

APPENDIX A

FIELD INVESTIGATION REPORT

Geotechnical Investigation
Borings 4, 5, & 6, Block 58
West Delta Area
Gulf of Mexico

Report to
Conoco Inc.
Houston, Texas

by
McClelland Engineers, Inc.
February 1982

Ertec

GEOTECHNICAL INVESTIGATION
BORINGS 4, 5,& 6, BLOCK 58
WEST DELTA AREA
GULF OF MEXICO

* * *

R e p o r t

t o

CONOCO INC.
Houston, Texas

* * *

B y

M c C L E L L A N D E N G I N E E R S, I N C.
Geotechnical Consultants
Houston, Texas

February 1982

McClelland engineers, inc. / geotechnical consultants
6100 HILLCROFT / HOUSTON, TEXAS 77081
TEL. 713 / 772-3701 / TELEX 762-447

Report No. 0181-0217
February 19, 1982

Conoco Inc.
c/o Mr. Jack Chan
P. O. Box 2197
Houston, Texas 77001

Attention: Mr. Tore J. Kvalstad

Geotechnical Investigation
Borings 4, 5 & 6, Block 58
West Delta Area
Gulf of Mexico

This report presents the results of our geotechnical investigation to explore soil and foundation conditions at the West Delta, Block 58 site. This study was authorized by Mr. Jack Chan in a telex dated October 29, 1981.

Preliminary information was sent to you on November 20, 1981. This information included a field boring log, a summary of field operations, a summary of Remote Vane data, and a plot of field and interpreted cone penetrometer data. This report includes all field and laboratory data in final form.

We appreciate the opportunity to work with you on this investigation. Please call us when we can be of further assistance.

Very truly yours,

McCLELLAND ENGINEERS, INC.

Alan G Young

Alan G Young, P.E.
Engineer Manager

DEH/GWQ/AGY/ps
Copies Submitted:
Mr. Horace F. House, Conoco Inc., Houston (1)
Mr. Jack Chan, Conoco Inc., Houston (6)

CONTENTS

ILLUSTRATIONS

APPENDICES

SUMMARY

McClelland Engineers performed a geotechnical investigation in West Delta, Block 58 in the Gulf of Mexico to explore soil and foundation conditions at a pile load test site. To meet these objectives, we drilled and sampled a boring to 242 ft (73.8 m) below the seafloor. We obtained soil samples by pushing a 3.0-in.-diameter (76-mm) thin-wall tube. In-situ shear strengths of the soils at the site were measured using the Remote Vane. In addition, we performed cone penetrometer tests using our Swordfish system to obtain continuous information on soil conditions. Conventional and advanced laboratory tests were performed on recovered soil specimens to evaluate the pertinent physical and strength properties of the foundation soils.

Results of our investigation show that soils at the study site consist of moderately to highly plastic clays from the seafloor to the final sample penetration of 242 ft (73.8 m). The consistency of the clays ranges from very soft at the seafloor to stiff at about 242 ft (73.8 m). We measured the water depth to be 53 ft (16.2 m) at 0910 hours on November 11, 1981.

This report presents a composite log of soil description based on Borings 4, 5, and 6. The log also shows a graphical representation of the results of the standard testing performed for these borings. Strength data from the various borings has been color coded. Data from Boring 4 is printed in blue; Boring 5 is in black; Boring 6 is in red. The results of the Remote Vane tests have been plotted in black on the boring log's graph and are tabulated in Appendix B. The cone penetrometer log has been edited and is presented on a separate plate. The text of the report presents a general description of field and laboratory work performed. The text also includes a brief discussion of stress history, sensitivity, SHANSEP design method, cone penetrometer data, and possible variations in clay type as they apply to this site. A detailed summary of the standard testing has been placed in Appendix B. Special testing for this job consisted of K_o consolidated-undrained triaxial compression, static simple shear, constant-rate-of-strain consolidation and incremental consolidation tests. Tabulated and graphic results of the special testing are available in Appendix B.

INTRODUCTION

Project Description

McClelland Engineers, Inc., conducted a geotechnical investigation to develop information on soil and foundation conditions at your site in Block 58 of West Delta Area in the Gulf of Mexico. The study area is located on the Mississippi Delta where the water depth is 53 ft (16.2 m). Conoco plans to conduct a tension pile load test at this site.

Purpose and Scope

The main purpose of our geotechnical study was to obtain information on soil and foundation conditions at the proposed load test location. To meet this objective, we drilled three borings and quantified soil properties, using three techniques: (1) 3-in.-diameter pushed samples, (2) in-situ undrained shear measurements with our Remote Vane, and (3) cone penetrometer tests using our Swordfish system. Standard and special laboratory tests on samples were used to characterize the soil conditions at the test site.

Report Format

This report begins with a brief description of the field and laboratory phases of the investigation. These sections are followed by a brief discussion of soil conditions at the West Delta Block 58 site. Appendix B presents the standard and special laboratory test results.

FIELD INVESTIGATION

McClelland Engineers' field crews explored soil conditions at the West Delta Block 58, Platform A site from November 4 to November 12, 1981 by drilling, sampling, and testing three borings. Previously, three borings had been drilled by McClelland Engineers in this block. To remain consistent with the nomenclature used for those earlier borings, we numbered consecutively, the borings performed for this study as 4, 5, and 6. The borings were performed adjacent to structure "A" in the subject block. A sketch of the relative location of the borings is presented on Plate 1. Excessive boat motion due to inclement weather on several occasions required the suspension of drilling operations. The borings are labeled to indicate the consecutive number of the set-ups and appear as the number after the slash on the boring designation. A water depth of 53 ft (16.2 m) was measured at 0910 hours on November 11, 1981, using an electronic seafloor sensor through the drill pipe. The water depth was checked before drilling each boring. It was found not to vary more than 0.5 feet. We did not correct for tidal variations during drilling and sampling since tides in the Gulf of Mexico generally vary less than 1.0 ft.

The borings were drilled with 4-1/2-in. IF drill pipe by a skid-mounted Failing 2000 rotary rig operating through a centerwell in the deck of the M/V "R.L. Perkins." A 2.5-in.-OD (64-mm-OD), 1.125-in.-ID (29-mm-ID) liner sampler was used to obtain samples to 37-ft (11.3-m) penetration. All other samples were taken using a latch-in push sampler developed by McClelland Engineers, Inc. The technique involves pushing a 3.0-in.-OD (76-mm-OD), 2.25-in.-ID (57-mm-ID), thin-wall tube sampler into the soil by latching the sampling tube into the drill bit and using the weight of drill pipe to advance the sampler into the soil. Boring 4 was used to perform cone penetrometer tests from 12- (3.7-) to 228-ft (69.5-m) penetration. Samples were taken at three-foot (one-meter) intervals in this boring from 228- (69.5-) to 240-ft (73.2-m) penetration. Boring 5 was drilled to provide samples at closely spaced intervals from the seafloor to 227.5-ft (69.3-m) penetration. Cone penetrometer tests were conducted from 229.5- (70.0-) to 254-ft (77.4-m) penetration in this boring. Boring 6 was used to provide Remote Vane shear strength data at 10-ft (3.0-m) intervals from 24- (7.3-) to 244-ft (74.4-m) penetration. Samples were also taken in this boring at selected intervals.

After recovering the soil specimen, our field engineer or soil technician cleaned the drilling fluid and cuttings from the top of the sample tubes, classified the soil in the bottom of the tube, performed miniature vane and Torvane tests, and then either sealed the tube or extruded the sample, examined it, and then sealed representative portions in containers. Samples were returned to Houston for either testing by McClelland Engineers or shipment to Ertec, Inc.

The boring log shown on Plate 2 represents a composite of information gathered in all three borings. The samples taken using the liner sampler are indicated by a blow count of zero. Because of the nearly continuous sampling at this site, the description under "Blow Count" is PUSH for all samples below 37-ft (11.3-m) penetration. We have color coded the strength test results in order to distinguish between the three borings. Shear strength test results from from Boring 4, 5, and 6 have been plotted in blue, black, and red, respectively.

In addition to the sampling program, McClelland Engineers' crews also made in-situ shear strength measurements using the Remote Vane and conducted cone penetrometer tests using the Swordfish system. The Remote Vane is pushed 3 (.9) to 5 ft (1.5 m) into the soil below the bottom of the borehole, and the four-bladed vane is rotated by an electric motor. The undrained shear strength is measured from the torque-rotation data recorded during the test. All Remote Vane data are presented on the boring log and in the summary of test results in Appendix B. Corrections were not made for plasticity.

The Swordfish system uses a hydraulic ram to push a standard cone (60-degree apex angle, 10-square centimeter base area, 150-square centimeter friction sleeve) into the soil below the drilled depth of the boring. The tests were performed in accordance with procedures outlined in ASTM D-3441-75, using a penetration rate of 2 cm per second. During penetration, cone resistance and sleeve friction are recorded in analog form and fed directly into a combined amplifier/digitizer/memory unit. A plot of edited cone penetrometer data is presented on Plate 3. Editing consisted primarily of subtracting hydrostatic head developed at the bottom of the borehole and removing the "shoulders" caused by drilling disturbance at the start of the stroke and by the inability to penetrate further at the completion of the stroke.

Appendix A provides a brief chronological summary of the field operations at this site.

FIELD AND LABORATORY TESTS

We planned our field and laboratory test programs mainly to evaluate pertinent physical and strength properties of the foundation materials. The types and numbers of tests performed are presented in this section along with some general comments. For a more detailed discussion on the specific test procedures and results, refer to Appendix B.

Classification Tests

We performed soil classification tests in the laboratory to confirm our field classifications and to supplement strength test data. The following number of classification and soil properties tests were performed:

Type of Test	Number of Tests
Plastic and Liquid Limits	9
Specific Gravity	2

The results of most of these tests are presented on the plate entitled, Summary of Test Results in Appendix B.

Strength and Compressibility Tests

Engineering properties of the soils such as shear strength and compressibility were obtained by the following tests:

Type of Test	Number of Tests
Miniature Vane	
Undisturbed	76
Torvane	73
Unconsolidated-Undrained Triaxial Compression	
Undisturbed	5
Remolded	5
K_o Consolidated-Undrained Triaxial Compression with Pore Pressure Measurement	2
Consolidated-Undrained Static Simple Shear	3
Constant-Rate-of-Strain Consolidation	3
Incremental Consolidation	3

We performed some strength tests in the field concurrently with drilling operations. Undrained shear strengths of the cohesive samples were determined by miniature vane tests. We also made estimates of the shear strength of the cohesive soils using a Torvane. The results of miniature vane and Torvane tests performed on samples not retained by McClelland Engineers are tabulated separately in Appendix B. All other tests were conducted in the laboratory and reported on the Summary of Test Results in Appendix B.

DISCUSSION

The scope of this report does not allow for a detailed discussion of the test results. However, limited analysis has been performed using techniques similar to those applied to data from deepwater borings. Also, a preliminary assessement of the cone penetrometer data has been made. Finally, a short discussion of possible changes in soil condition is included.

In-Situ Vertical Effective Stress

Consolidation tests were performed to help develop the stress history of the site. To provide an independent check of results, both incremental and constant-rate-of-strain (CRS) consolidation tests were performed at the same interval. Examination of the curves resulting from the consolidation tests indicate more disturbed samples than those taken in deep water with the same latch-in sampling technique. The curves from the CRS consolidation tests resulted in higher preconsolidation pressures than those determined using the incremental consolidation tests. Plate 4 presents the preconsolidation pressures for the two types of tests, along with the effective overburden pressure profile for a normally consolidated clay. The preconsolidation pressures, when compared with the computed effective overburden stress profile, indicate that the soils at this site are underconsolidated.

Comparison of Undrained Strength Measurements

To compare shear strengths from this boring and those from deep water borings, we plotted remolded, laboratory, and Remote Vane shear strengths vs. Liquidity Index (LI). Several studies of soil properties have indicated that LI vs. log in-situ shear strength data is a straight line in the strength range under consideration. Also, this line is nearly parallel to the LI vs. log remolded shear strength line. We plotted the remolded strengths on Plate 5 to determine a relationship. This relationship, line AA', was then transcribed to plots of LI vs. Remote Vane shear strengths and laboratory shear strengths, Plate 6 and 7, respectively. Lines with sensitivities of two and three have been added to provide a reference. The shear strengths from the Remote Vane indicate a sensitivity of approximately two. The plot of laboratory shear strength includes points from a limited number of miniature vane tests, unconsolidated-undrained (UU) triaxial compression tests, K_o consolidated-undrained (K_oCU) triaxial compression tests, and static simple shear tests. Examination of miniature vane and UU triaxial compression test results on Plate 7 shows a sensitivity of less than two. The sensitivity of the clays from deep water borings was approximately 50% greater than those determined from miniature vane and UU triaxial compression tests for this boring. A slightly higher value of sensitivity is suggested by the limited data from K_oCU triaxial compression and static simple shear tests. These tests provide a more reasonable estimate of sensitivity perhaps because the consolidation phase of the test helps reduce the effects of disturbance. We believe the apparent low sensitivity of the miniature vane and UU triaxial tests is a function of the disturbance caused by the gassy nature of this deposit.

To further evaluate the undrained shear strength at the West Delta Block 58 site, we applied the SHANSEP(2) design method to the results obtained from the static simple shear, K_oCU triaxial compression, and CRS consolidation tests. The vertical consolidation pressure, $\bar{\sigma}_v$, vs the shear strength for the static simple shear and selected K_oCU triaxial compression tests have been plotted on Plate 8. The ratio, $S_u/\bar{\sigma}_v$, was determined by selecting a line of best fit. The value of $S_u/\bar{\sigma}_v$ was found to be 0.26. When this value was applied to the preconsolidation pressures from the CRS consolidation tests, the resulting strengths are within the range of the measured shear strengths.

Evaluation of Cone Penetrometer Data

In addition to the edited cone penetrometer log, we also plotted friction ratio vs. log cone resistance and Remote Vane shear strength vs. cone resistance less the existing overburden pressure. The literature presents several criteria for determining material type based on the relationship of friction ratio and log cone resistance. The trace of friction ratio vs. log cone resistance, presented on Plate 9, also includes the material descriptions suggested by Douglas and Olsen[1]. The trace indicates a sensitive clay grading to a sensitive mixture of clay and silt. Quantitative measures of sensitivity are not available from the literature. However, we expect them to exceed the measured value of less than two. Sample disturbance caused by the gassy nature of this deposit could account for the apparent discrepancy in sensitivity between shear strength measurements and cone data. Another plot, typically produced from cone data for clays, is cone resistance vs. shear strength. Plate 10 presents this plot. We selected for this correlation, the shear strength measured by the Remote Vane. The cone resistances which have been corrected for hydrostatic pressure, shown on Plate 3, have been further modified by subtracting the effective overburden pressure at the test depth. The resulting correction to the raw cone resistances is to remove the total overburden pressure at the test point. A correlation line, forced through zero, has a slope, N_k, equal to 6.2. The literature typically reports N_k values greater than 6.2. The correlations presented in the literature are generally for overconsolidated and normally consolidated clays and are based on shear strengths measured from laboratory tests rather than in-situ vane tests. The reduction in cone resistance resulting from the total overburden term was approximately 50 percent of the cone resistance. In addition, the shear strengths from the Remote Vane are generally the upper bound of possible shear strength profiles. These two factors may have combined to reduce N_k.

Soil Conditions

The clay soils at this site could be separated into at least two subdivisions. Descriptions of the soils indicate that differences in soil structure start to occur at about 180-ft penetration. This change was also evident in the results of the laboratory index and strength tests and cone penetrometer tests. First, the Liquidity Index, LI, at 190-ft penetration changes from a gradually decreasing number and begins to increase. This can be seen on Plate 11, a plot of LI vs. penetration. Secondly, the submerged unit weights on Plate 2 show a shift at approximately 170-ft penetration.

Thirdly, the friction ratio begins to decrease at approximately 192-ft penetration. Lastly, the slope of shear strength, with respect to depth, increases significantly below 190-ft penetration. The scope of McClelland Engineers' testing program did not allow for further investigation of the differences in the two subdivisions.

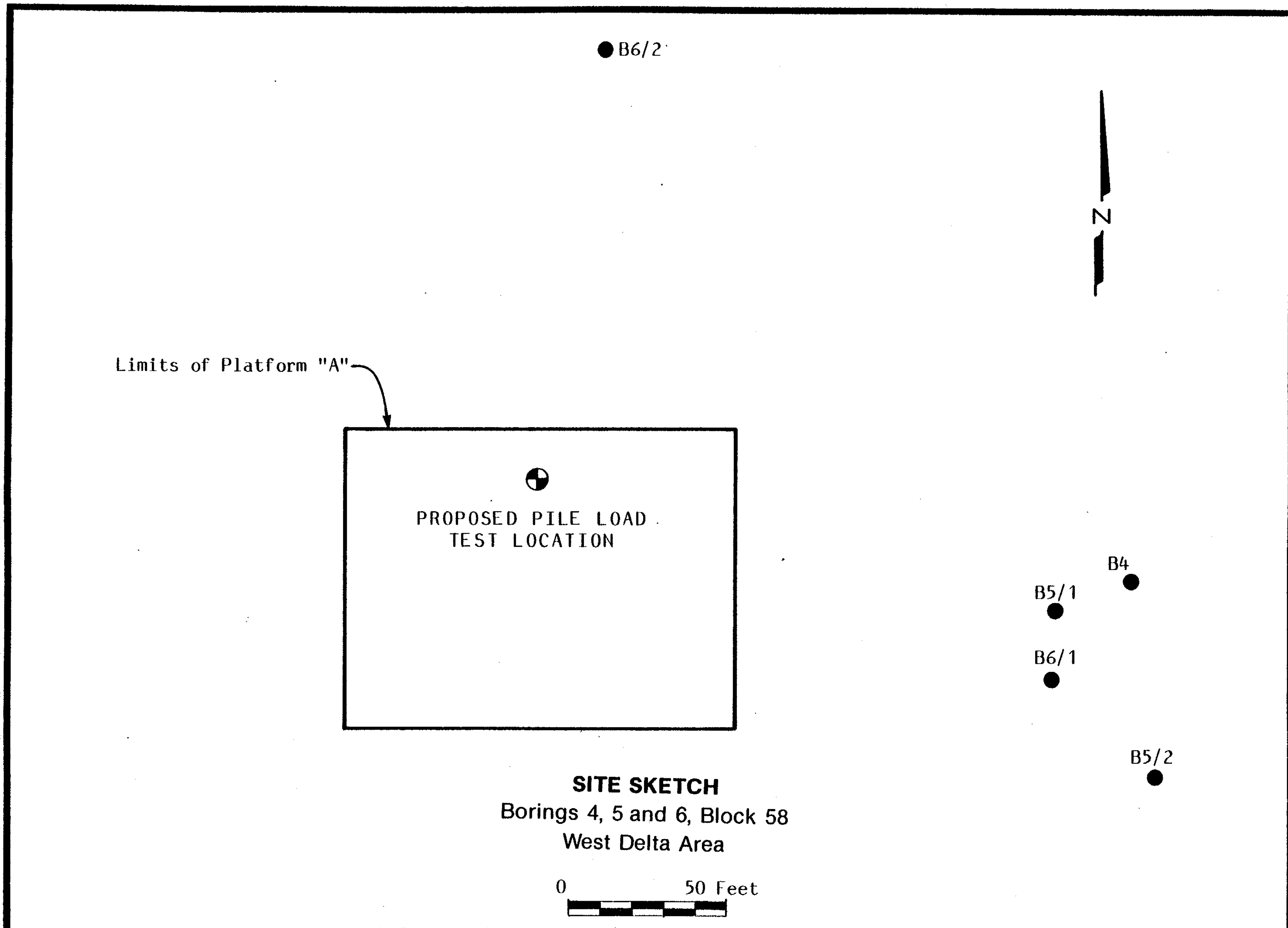
REFERENCES

(1) Douglas, B.J. and Olsen, R. S. (1981), "Soil Classification Using Electric Cone Penetrometer," Proceedings, Session sponsored by the Geotechnical Engineering Division at the ASCE National Convention, St. Louis, Missouri, October 26-30, 1981, pp. 209-227.

(2) Ladd, C.C. and Foott, R. (1974), "New Design Procedures for Stability of Soft Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT7, pp. 763-786.

ILLUSTRATIONS

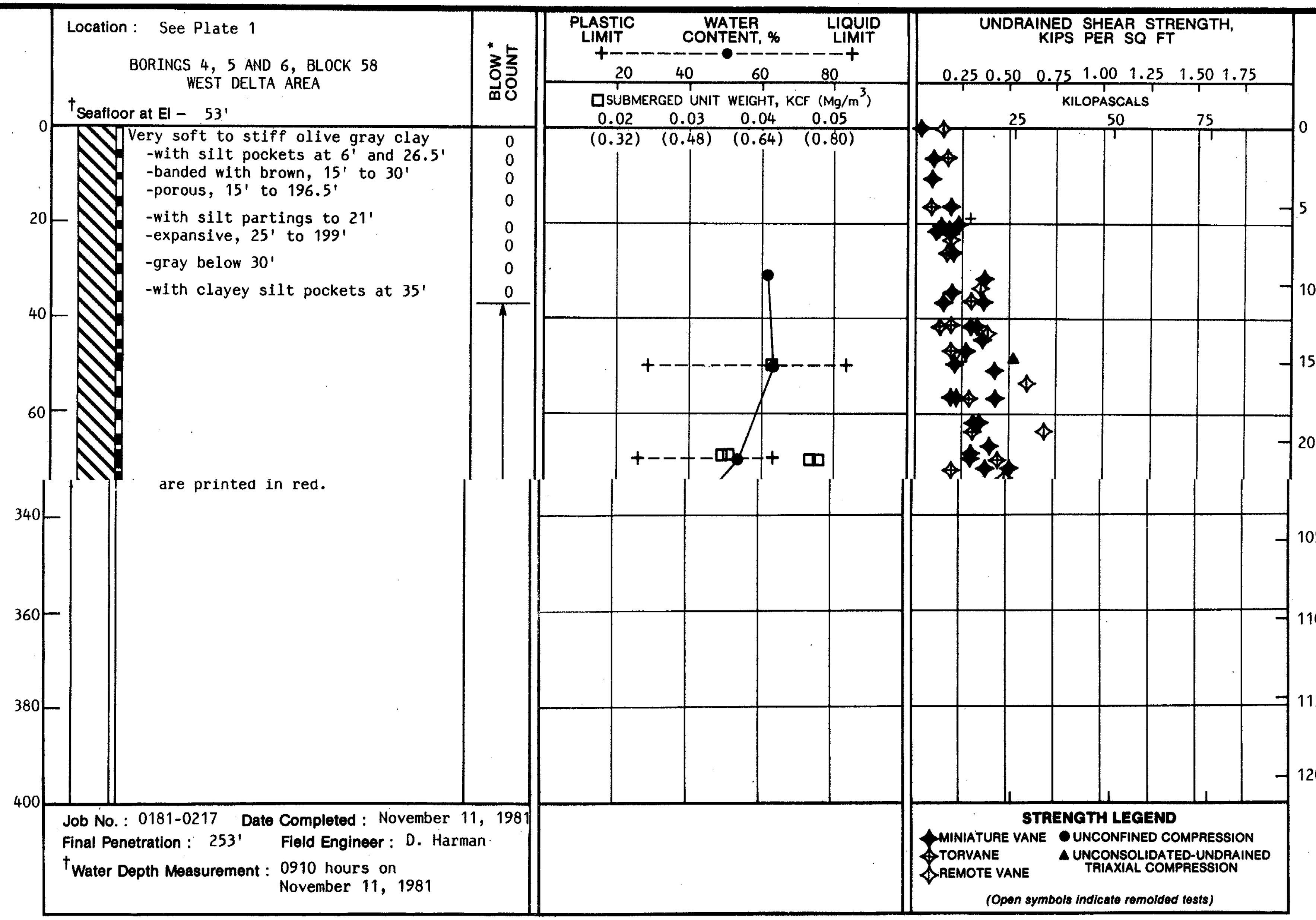
Drafted: Checked: Date: Approved: Date: Form DFT-1.00 (12/81) Job No.:



SITE SKETCH
Borings 4, 5 and 6, Block 58
West Delta Area

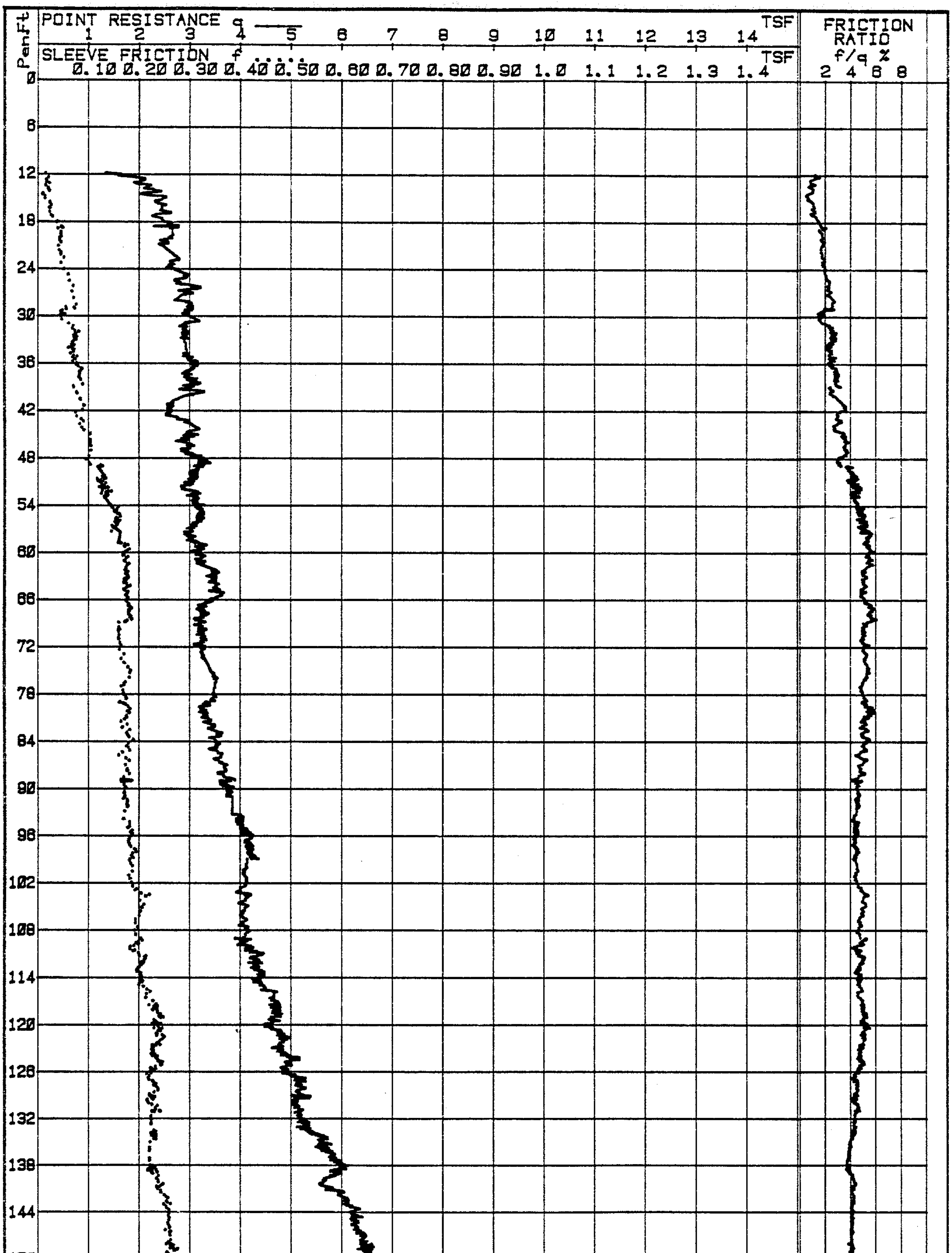
McCLELLAND ENGINEERS

Drawn: ______ Date: ______ Drafted: [signature] Date: 2-8-82 Checked: ______ Date: ______ Approved: ______ Date: ______ Form 108-11.00 (80)



LOG AND TEST RESULTS
BORINGS 4, 5 AND 6, BLOCK 58
WEST DELTA AREA

McCLELLAND ENGINEERS



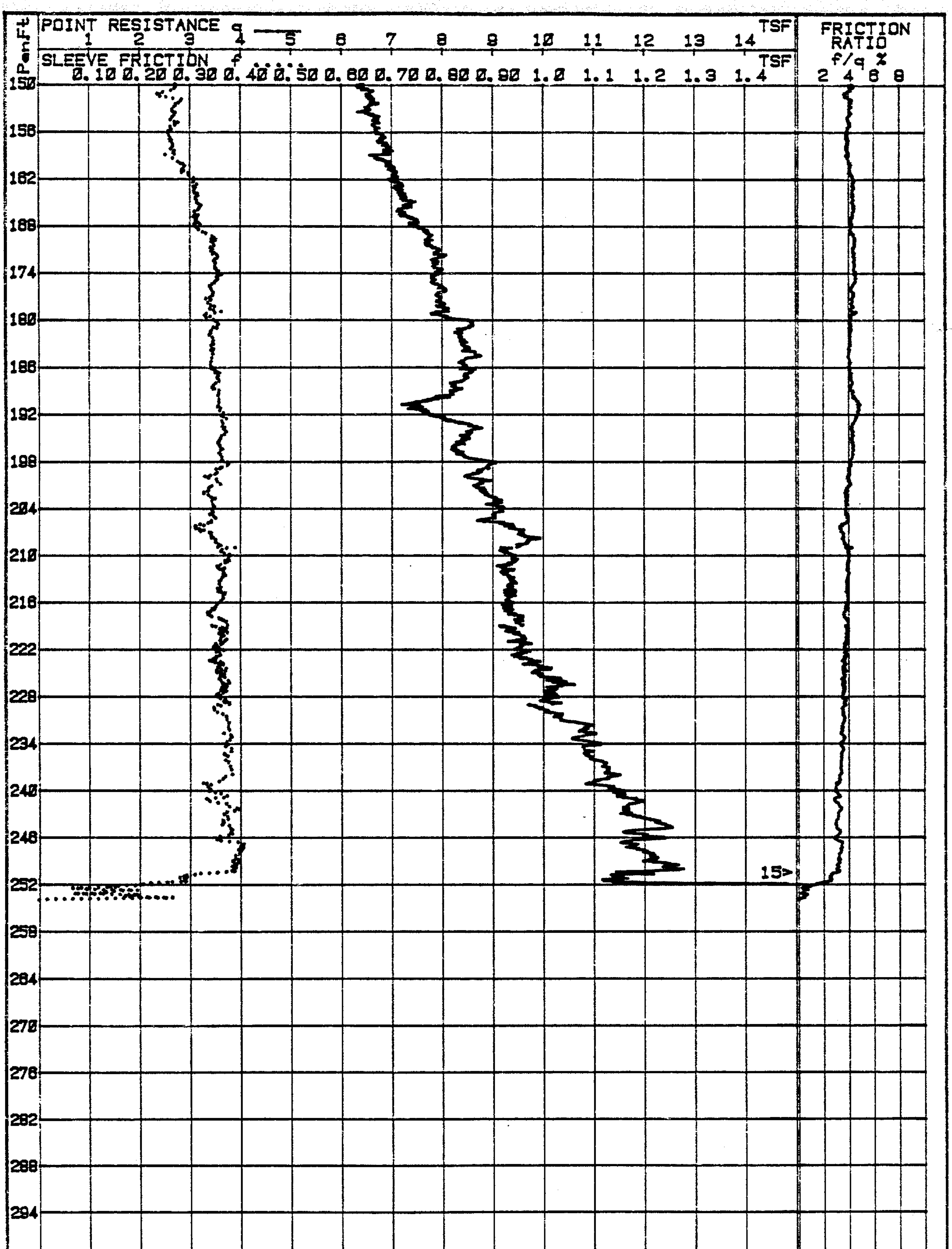
TEST PERFORMED: NOVEMBER 1981

RESULTS OF CONE PENETRATION TEST WD 58

LOCATION: WEST DELTA 58 WATER DEPTH: 53.0 Ft CONOCO 0181-0217

McCLELLAND ENGINEERS

PLATE 3a



TEST PERFORMED: NOVEMBER 1981

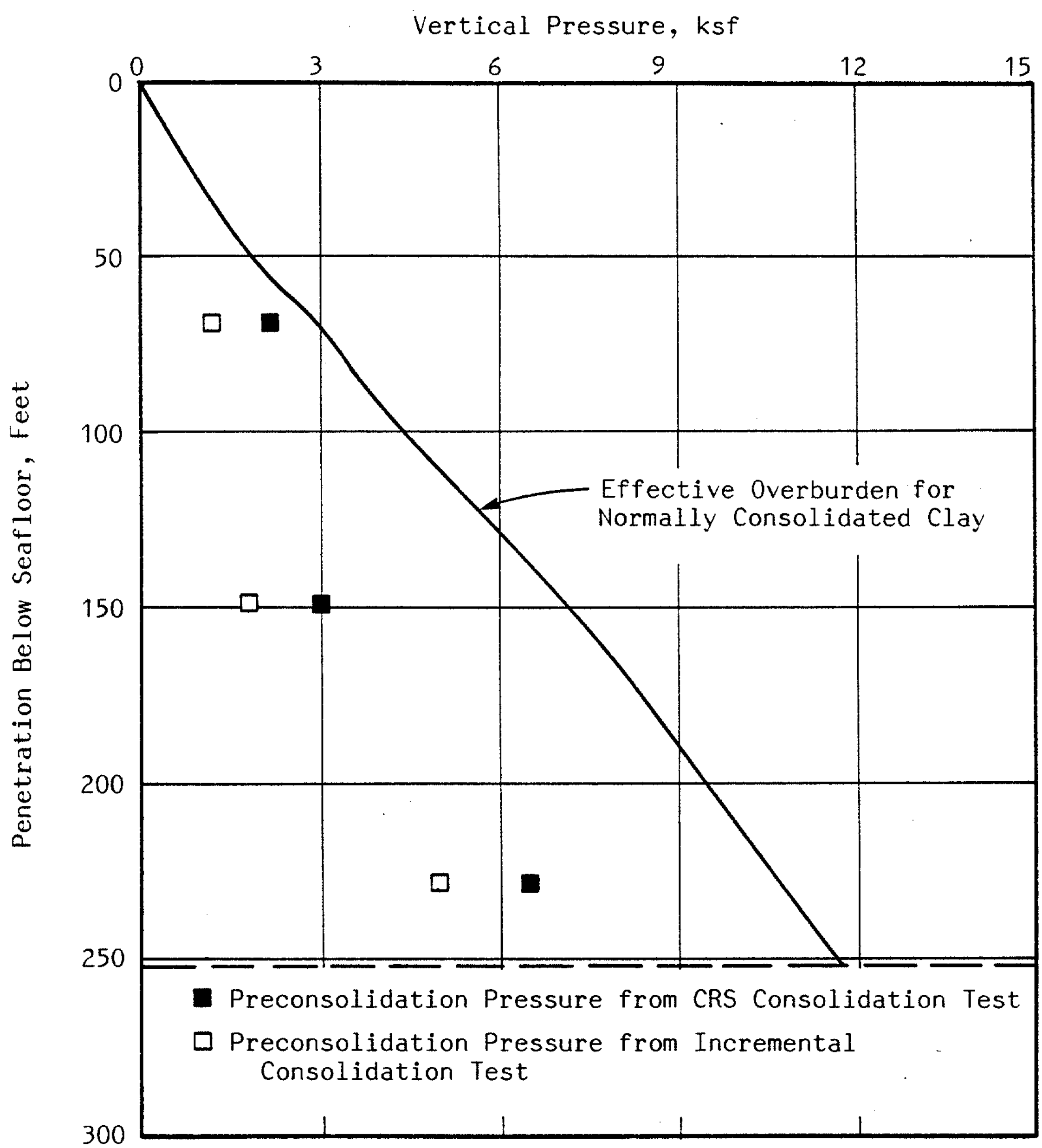
RESULTS OF CONE PENETRATION TEST WD 58

LOCATION: WEST DELTA 58 WATER DEPTH: 53.0 Ft

CONOCO 0181-0217

McCLELLAND ENGINEERS

PLATE 3b

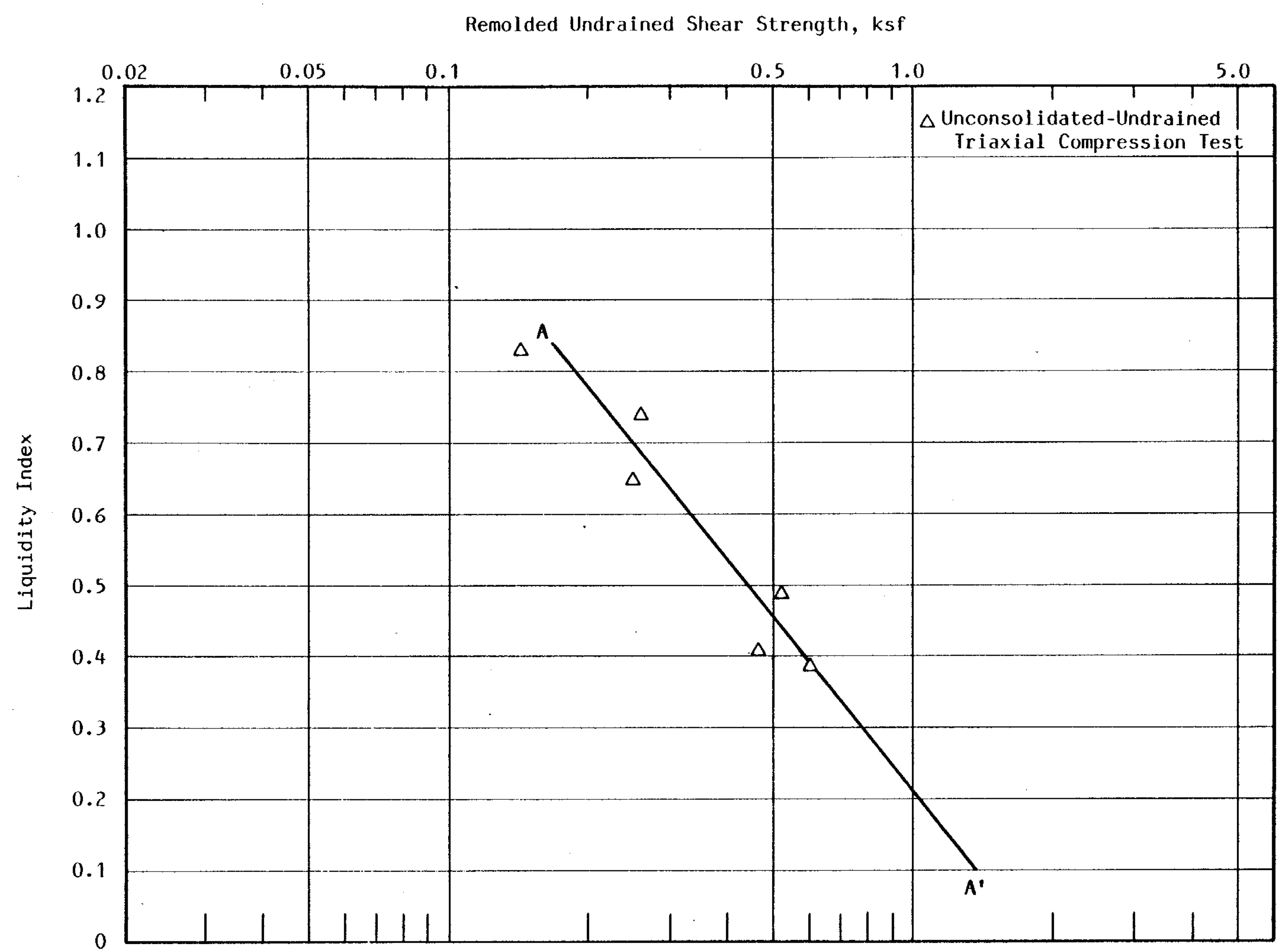


VERTICAL PRESSURE vs PENETRATION

Borings 4 and 5, Block 58

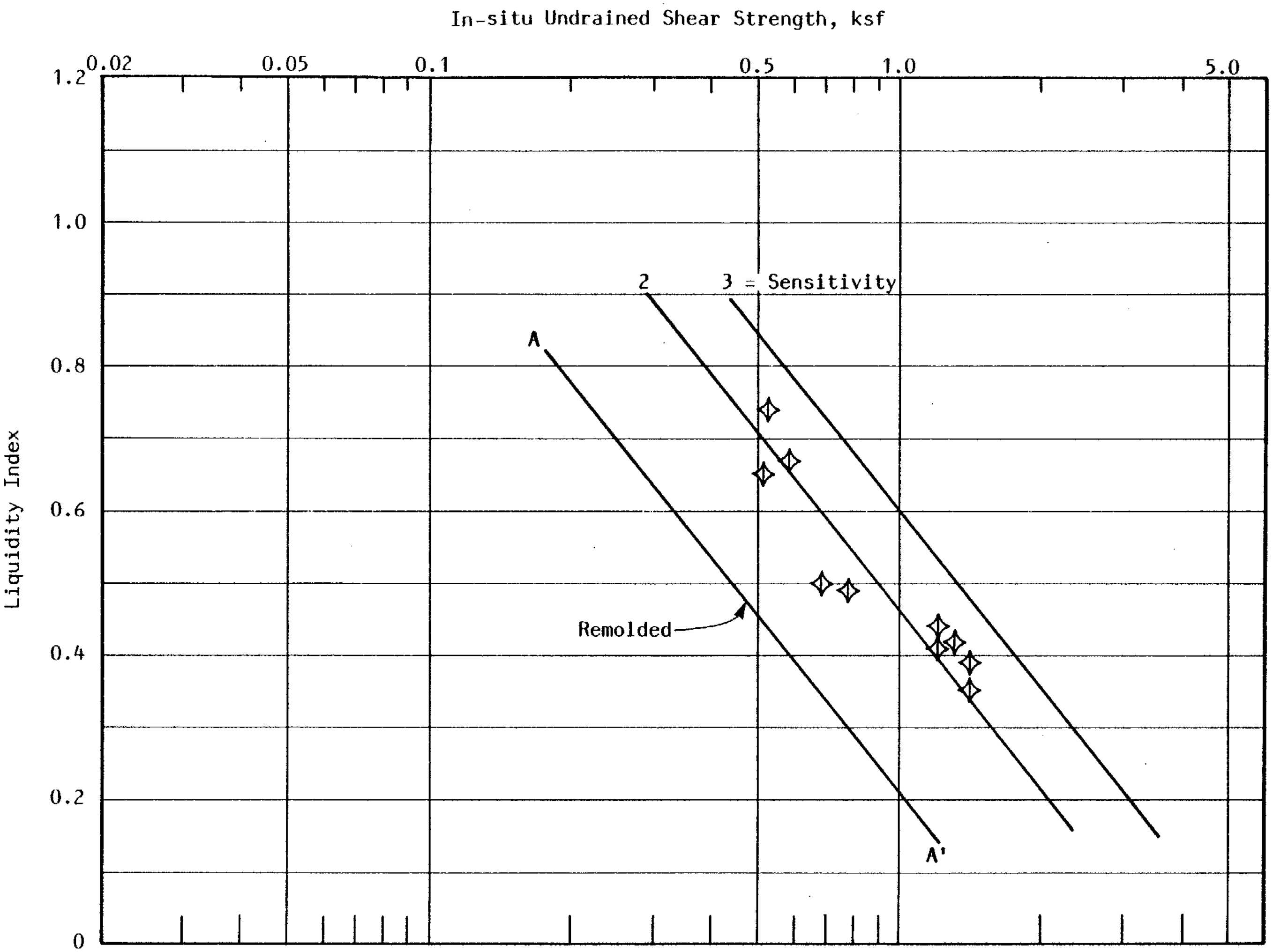
West Delta Area

Job No.: 0181-0215 Form DFT-1.00 (12/81) Date: Approved: Date: Checked: Date: 2-4-82 Drafted: J. Ulrich



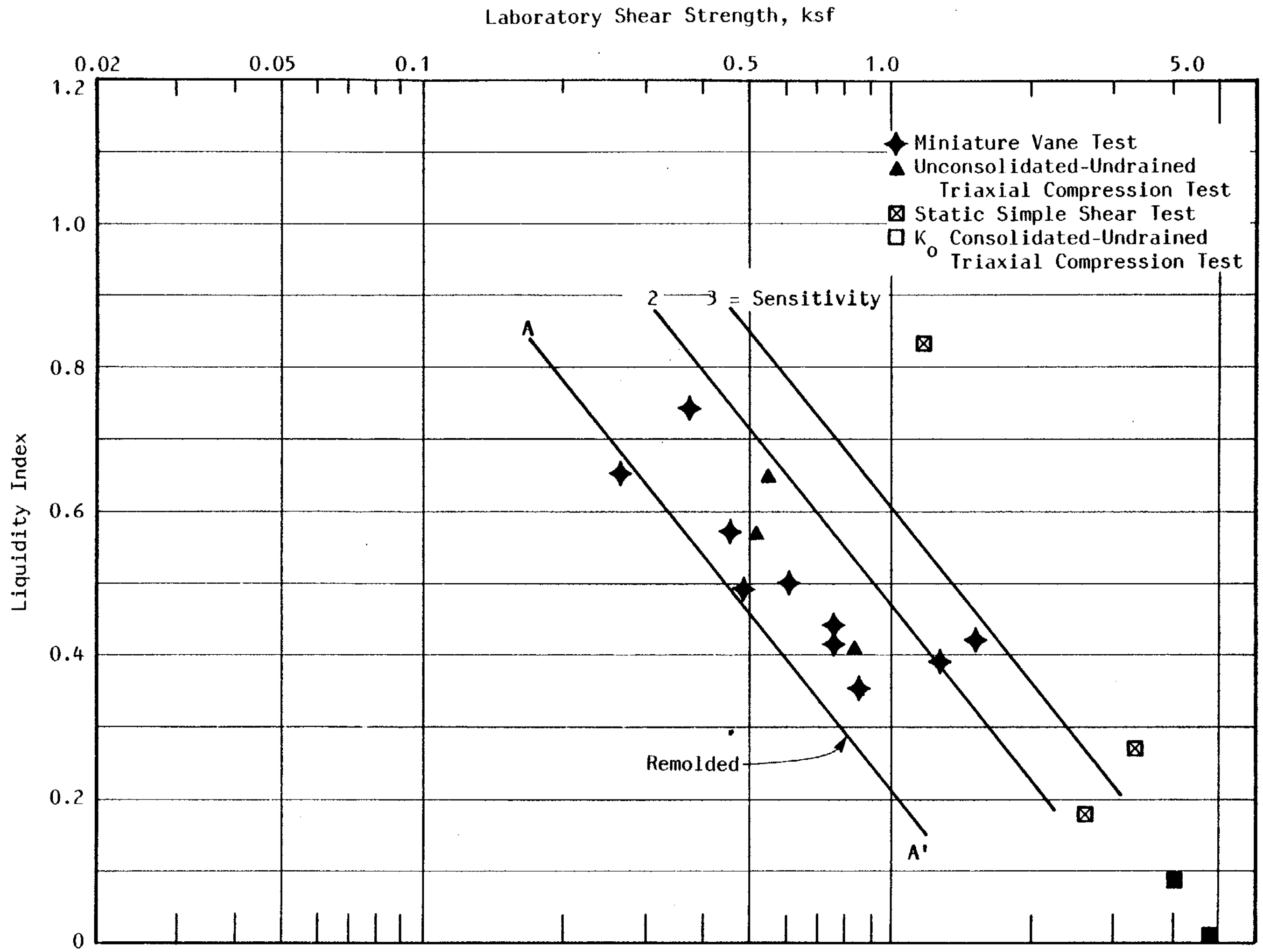
LIQUIDITY INDEX vs REMOLDED SHEAR STRENGTH
Borings 4, 5 and 6, Block 58
West Delta Area

Job No. 018-0247



LIQUIDITY INDEX vs IN-SITU SHEAR STRENGTH
Boring 6, Block 58
West Delta Area

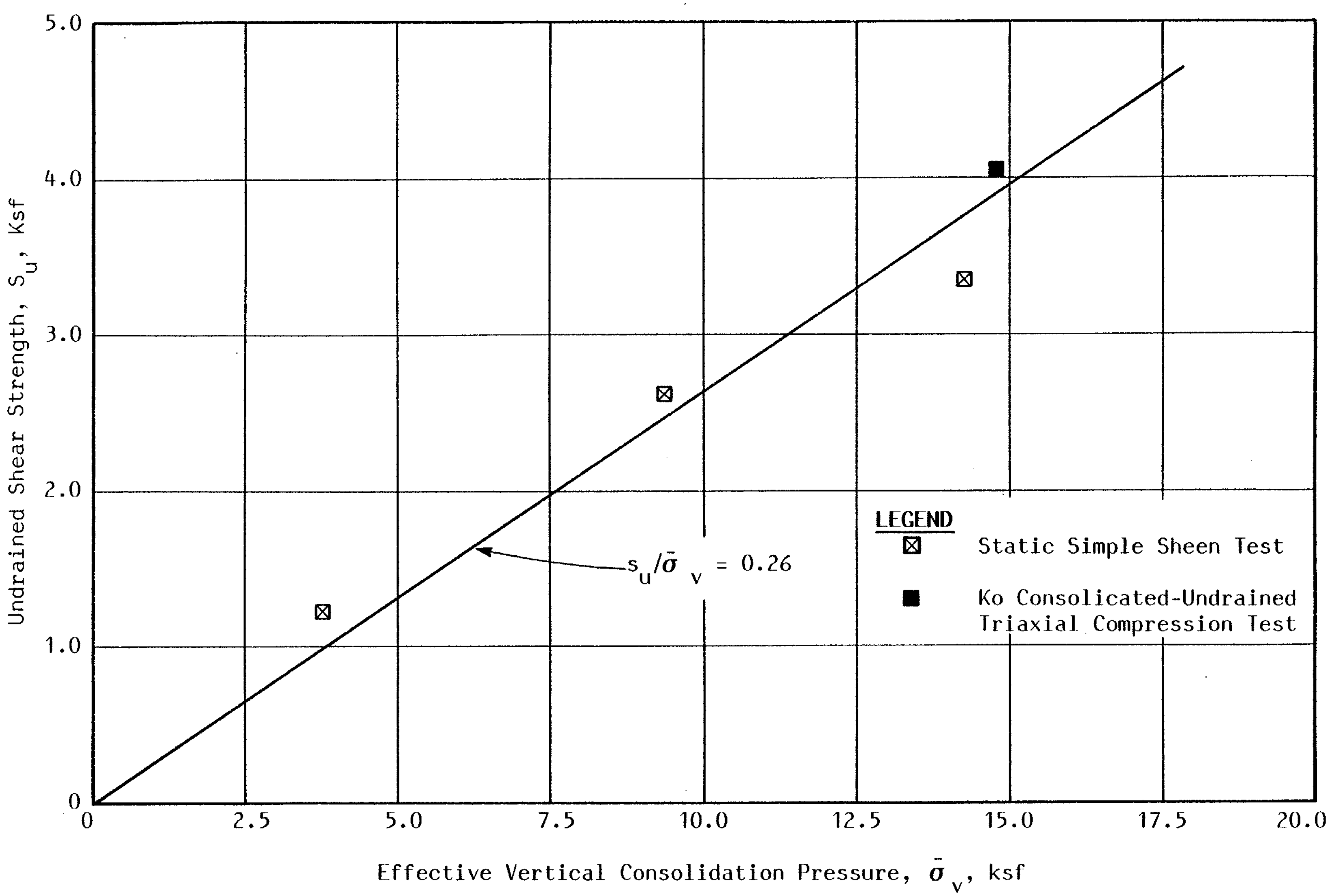
Job No. 018-0217



LIQUIDITY INDEX vs LABORATORY SHEAR STRENGTH

Borings 4, 5 and 6, Block 58

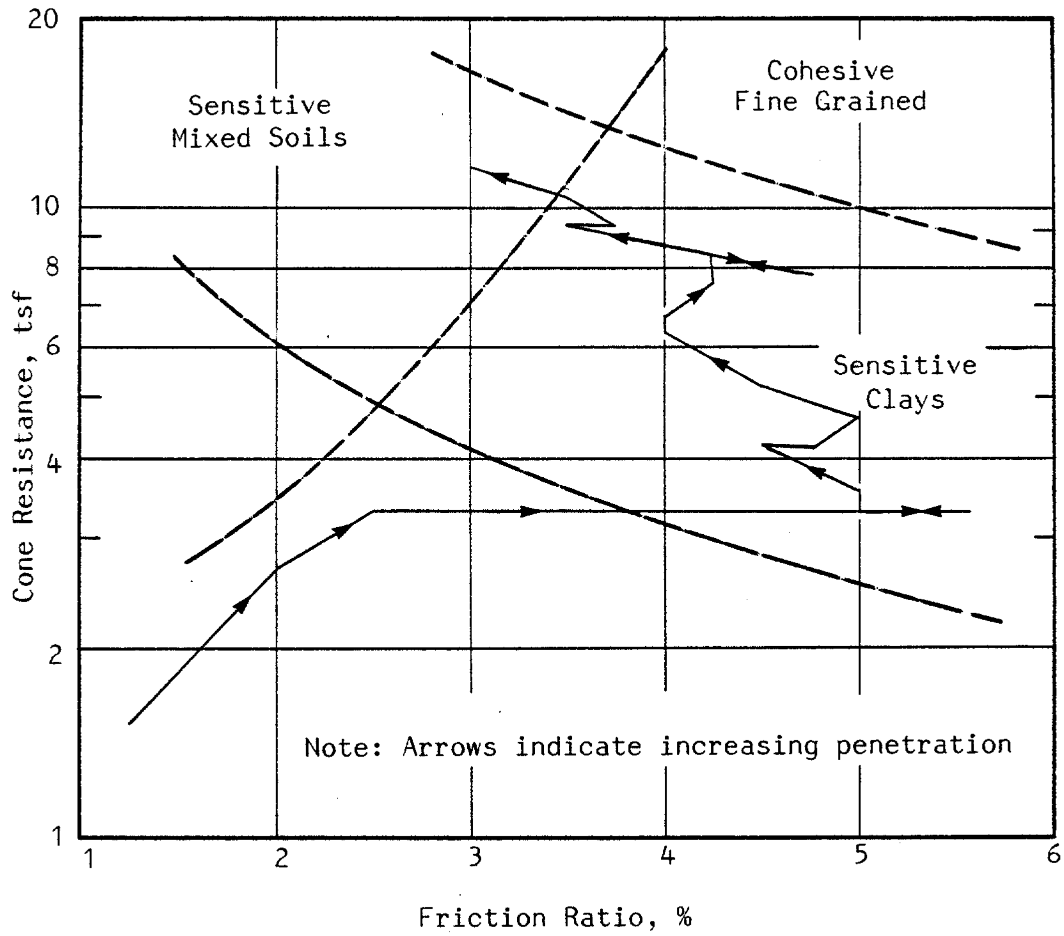
West Delta Area



EFFECTIVE VERTICAL CONSOLIDATION PRESSURE vs SHEAR STRENGTH

Boring 5, Block 58

West Delta Area



FRICTION RATIO vs CONE RESISTANCE

Boring 4, Block 58

West Delta Area

Job No. : 0201-0521

Form DFT-1.00 (12/81)

Date :

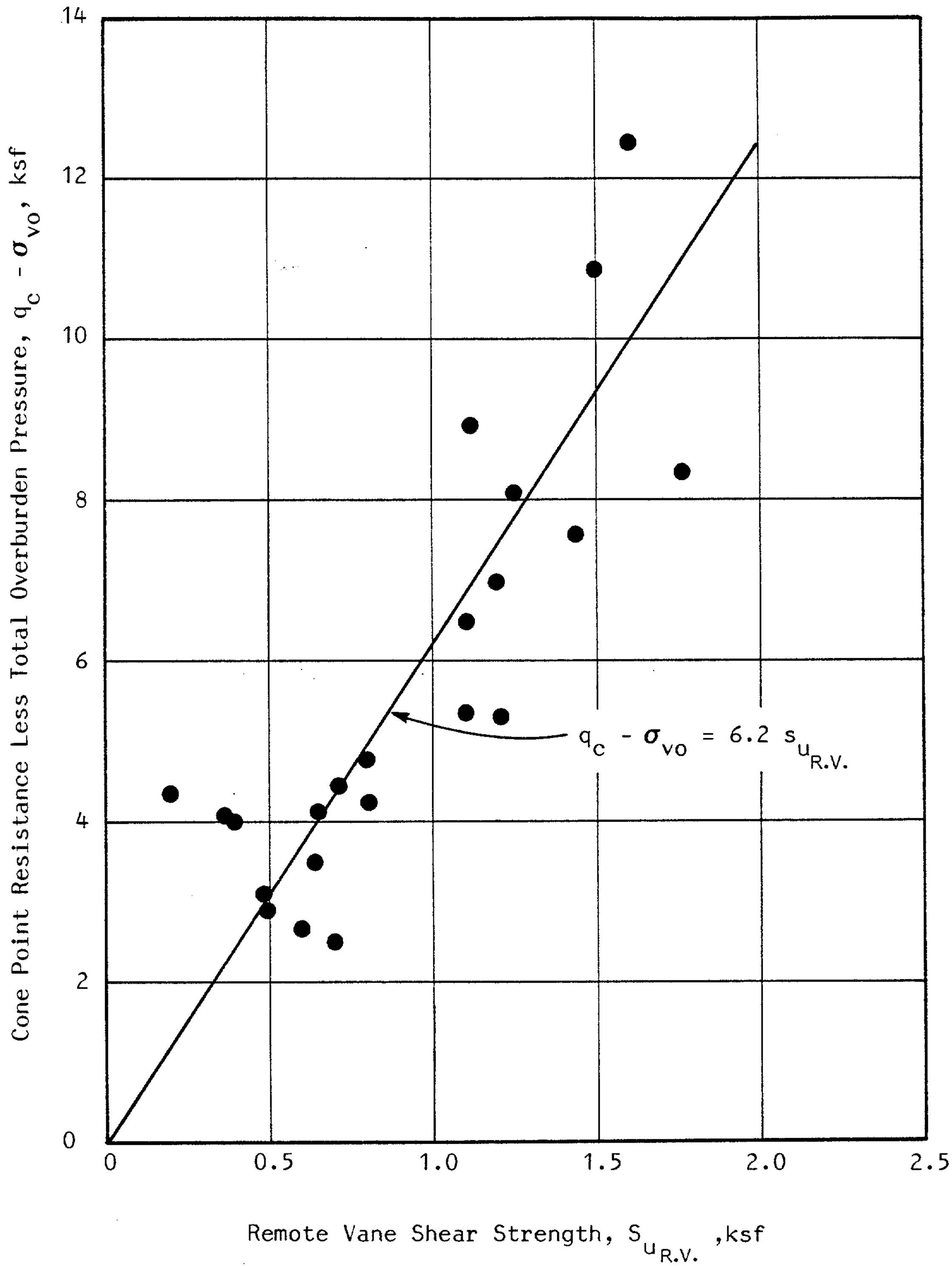
Approved :

Date :

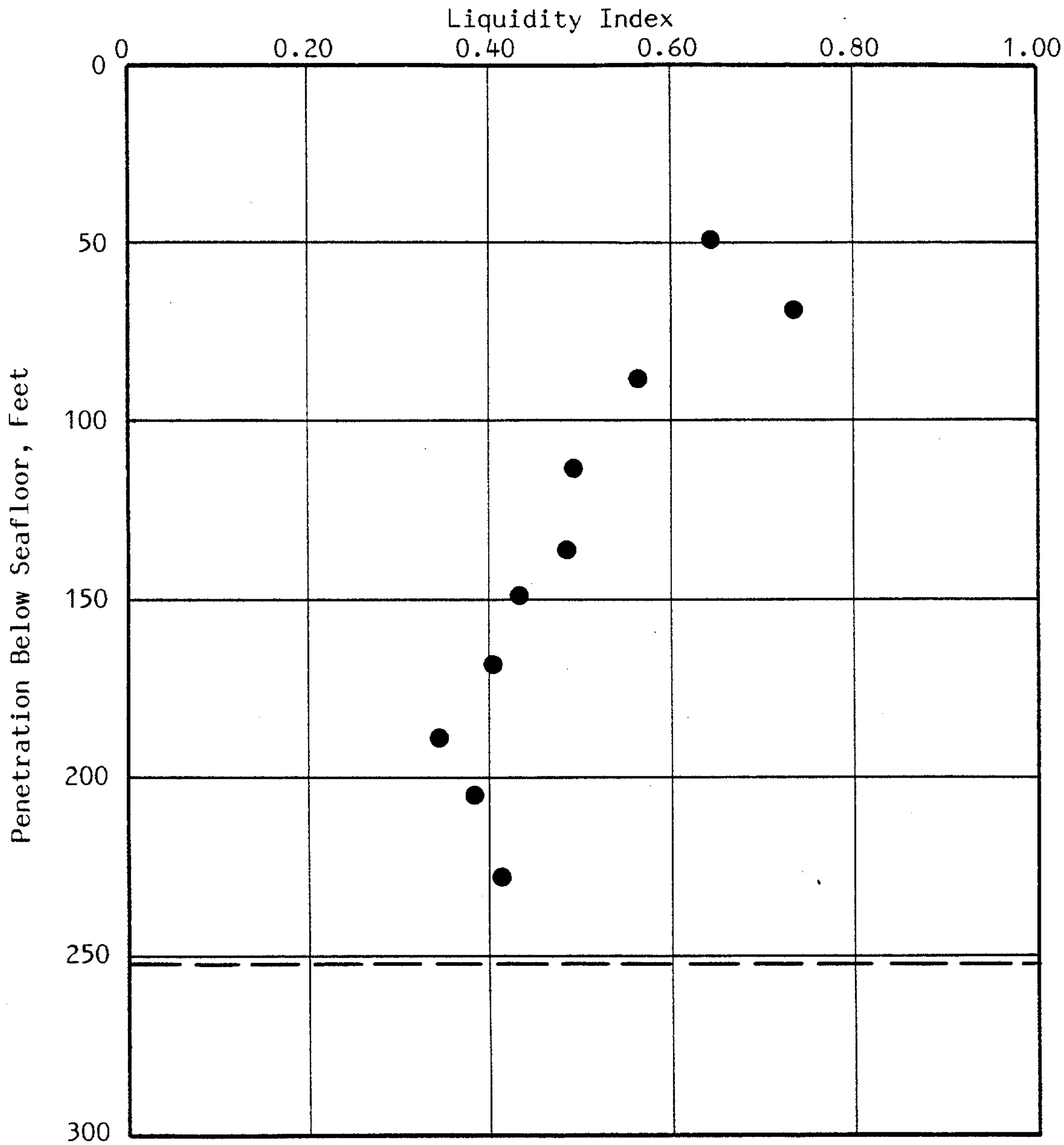
Checked :

Date : 2-2-

Drafted : J. Ulmer



MODIFIED CONE RESISTANCE vs REMOTE VANE SHEAR STRENGTH
Boring 4 and 6, Block 58
West Delta Area



LIQUIDITY INDEX vs PENETRATION
Boring 5 and 6, Block 58
West Delta Area

APPENDIX A

SUMMARY OF FIELD OPRATIONS

Date	Time From	Time To	Description of Activity
November 4, 1981	----	1130	M/V "R.L. Perkins" arrives at dock in Grand Isle, Louisiana
	1130	2400	Loading equipment and mud
November 5, 1981	0000	0100	Loading mud
	0100	0500	Traveling to Block 58, West Delta Area
	0500	0800	Setting anchors
	0800	2400	Cone testing Boring 4
November 6, 1981	0000	1615	Cone testing and grouting Boring 4
	1615	1700	Relocating on anchor spread
	1700	2330	Drilling and sampling Boring 5
	2330	2400	Waiting for improved sea conditions
November 7, 1981	0000	1215	Waiting for improved sea conditions
	1215	1315	Relocating on anchor spread
	1315	2400	Drilling and sampling Boring 5
November 8, 1981	0000	1415	Drilling and sampling Boring 5
	1415	1530	Grouting Boring 5
	1530	1900	Relocating on anchor spread
	1900	2400	Drilling, sampling, and remote vane testing Boring 6
November 9, 1981	0000	0430	Drilling and sampling Boring 6

(Continued on Plate A1b)

SUMMARY OF FIELD OPERATIONS
Borings 4, 5 and 6, Block 58
West Delta Area

Date	Time From	Time To	Description of Activity
	(Continued from A1a)		
November 9, 1981	0430	1600	Waiting for improved sea conditions (reduced rate)
	1600	1700	Pulling anchors (reduced rate)
	1700	2400	Waiting for improved sea conditions (reduced rate)
November 10, 1981	0000	1600	Waiting for improved sea conditions (reduced rate)
	1600	1730	Setting anchors (reduced rate)
	1730	2400	Waiting for improved sea conditions (reduced rate)
November 11, 1981	0000	0400	Waiting for improved sea conditions (reduced rate)
	0400	2345	Drilling, sampling, and remote vane testing Boring 6, used a total of 900 bags of weight material, 400 bags of saltwater gel material, and 90 bags of cement
	2345	2400	Pulling anchors
November 12, 1981	0000	0045	Pulling anchors
	0045	0400	Traveling to Grand Isle, Louisiana to demobilize equipment
	0400	0900	Waiting for crane
	0900	1030	Offloading equipment
	1030	----	M/V "R.L. Perkins" departs for next client's location

SUMMARY OF FIELD OPERATIONS

Borings 4, 5 and 6, Block 58

West Delta Area

APPENDIX B

LABORATORY SOIL TEST RESULTS

CONTENTS

ILLUSTRATIONS

APPENDIX B

LABORATORY SOIL TEST RESULTS

Static Strength Tests

Several procedures were used in the field and in the laboratory to determine the strengths of foundation soils under various conditions. The different test procedures used are described in the following paragraphs. Reference is made to the manner in which results are presented.

Miniature Vane and Torvane Tests. Two types of strength tests were performed on the soil samples in the field as they were recovered. Undisturbed shear strengths of cohesive samples were determined with a motorized miniature vane device while the samples were still in the sampling tubes. Estimates of shear strength were also made using a Torvane device. The results of these tests are tabulated on Plate B-1 and are plotted on Plate 2.

Unconsolidated-Undrained Triaxial Compression Tests. In this type of strength test the soil specimen is enclosed in a thin rubber membrane and subjected to a selected confining pressure. The specimen is not allowed to consolidate under the influence of this confining pressure. The specimen is then loaded axially to failure at a constant rate of strain without any drainage from the specimen. For this investigation, the confining pressure was selected to be about equal to the computed soil buoyant overburden pressure.

Shear strengths of undisturbed cohesive samples determined in the laboratory by this type of test are included in the graphic plots on Plate 2. All these test data are tabulated on Plate B-1 together with the confining pressure, percent strain at failure, and type of failure.

K_o Consolidated-Undrained Triaxial Tests

The physical set-up of this test is similar to the unconsolidated-undrained triaxial compression test described above. The major difference is that the specimen is allowed to drain under a particular cell confining pressure. To fully saturate this sample, back-pressure is applied. Increments of vertical and horizontal stress are then added in such a manner as to make all changes in specimen water content a function of sample height change. Final confining pressures were selected so as to assure the sample would be consolidated in excess of the estimated maximum past-vertical consolidation pressure.

Upon completion of consolidation, the specimen is sheared with the drainage lines closed. Shear is induced by increasing the axial load at a constant rate of strain. Parameters measured during shear include axial load, axial deformation, and excess pore pressure.

Plates B-4 through B-7 present the stress-strain curves and p'-q diagrams determined in the laboratory for 2 samples by this type of test. Summaries of the test results are tabulated on Plate B-3. The summary on Plate B-3 includes initial and final moisture contents, initial unit dry weight, confining pressure, failure strain, and other information for each test.

Static Simple Shear Tests. Simple shear specimens were trimmed to 0.75-in.-height and 1.875-in.-diameter to fit into a wire-reinforced rubber membrane. The membrane restricts lateral deformation during consolidation. Increments of normal (vertical) load were applied to consolidate the sample. After consolidation, the specimen was sheared to failure at constant volume by applying a horizontal shear load. Summaries of the test results are tabulated on Plate B-8. Consolidation pressure and stress-strain curves for these specimens are included on Plates B-9 through B-14.

Consolidation Tests

Two types of consolidation tests were performed for this project: (1) incremental and (2) controlled-rate-of strain (CRS). For an incremental consolidation test, the total load on the specimen remains constant and deflection is measured. During the CRS consolidation test, load is applied to the specimen by introducing an increasing strain into it.

In the incremental-load oedometer test, the soil specimen is placed in a 1.765-in.-ID ring and immersed in water. Then, loads are added to prevent swelling. When the swell pressure is established, vertical load is added in increments that are usually doubled, yielding a load increment ratio of one. Each load increment is held for 24 hours with primary consolidation determined by the logarithm of time method. The data readings are used To compute vertical strain, vertical pressure, and coefficient of consolidation. The results of this type test are presented on Plate B-15 through Plate B-17.

The CRS consolidation testing equipment is similar to that used for a consolidated-undrained triaxial test. The base of a conventional triaxial cell is fitted with a 1.875-in.-ID stainless steel ring. Back pressure, used in saturating the soil specimen, can be provided through porous stones fitted at each end of the specimen. The rate of strain is selected to produce a minimum excess pore pressure of 1 psi and limit the ratio of maximum excess pore pressure to applied vertical pressure to 30 percent. Vertical loading deflection and pore pressure response of the specimen are all monitored continuously using electronic instrumentation. The test results for this type of test are presented on Plates B-18 through B-20 as curves of percent change in height (vertical strain) versus applied vertical effective pressure.

Classification Tests

Plastic and liquid limits, collectively termed the Atterberg limits, were determined for the cohesive samples to provide classification information. Natural water content tests were also performed on selected specimens. The results of these water content tests, together with the Atterberg limit test results, are plotted on Plate 2. Natural water content and density determinations were made for each compression test specimen. To complete the water content profile shown on the above plates, results of water content tests made in conjunction with the compression tests; additional water content tests are also plotted. All of the above data are tabulated on Plate B-1.

ILLUSTRATIONS

APPENDIX B

SUMMARY OF TEST RESULTS

SAMPLE NUMBER	PENETRATION, FEET	CLASSIFICATION TESTS: LIQUID LIMIT	PLASTIC LIMIT	WATER CONTENT, %	UNIT WET WEIGHT, LB/CU FT	PERCENT PASSING NO. 200 SIEVE	TORVANE: SHEAR STRENGTH, KSF/(kPa)	MINIATURE VANE: TYPE OF TEST	SHEAR STRENGTH KSF/(kPa)	COMPRESSION TESTS: TYPE OF TEST	WATER CONTENT, % Initial	WATER CONTENT, % Final	UNIT DRY WEIGHT, PCF/(Mg/m3)	SHEAR STRENGTH, KSF/(kPa)	ϵ_{50} STRAIN, %	LATERAL PRESSURE, KSF/(kPa)	FAILURE STRAIN, %	TYPE OF FAILURE
								BORING 5										
25	49.5				106					2-U 2-R	49 47		71 67	0.54 0.25		1.73 1.73	7.5 13.5	C A,C
	15.09m				(1.70)					2-U 2-R	49 47		(1.14) (1.07)	(25.84) (11.96)		(82.78) (82.78)	7.5 13.5	C A,C
26	50.0	84	29	64			0.22	U	0.22									
	15.24m	84	29	64			(10.53)	U	(10.53)									
42	69.0									CRS and Incremental Consolidation Tests (See Plates B-18 and B-15) Static Simple Shear Test (See Plate B-9 and B-10)								
	21.03m									CRS and Incremental Consolidation Tests (See Plates B-16 and B-15) Static Simple Shear Test (See Plates B-9 and B-10)								
43	69.5	64	27	54			0.20	U	0.30									
	21.19m	64	27	54			(9.57)	U	(14.35)									
53	89.5				112					2-U 2-R	38 39		80 76	0.51 0.26		3.17 3.17	15.8 9.9	A A
	27.29m				(1.79)					2-U 2-R	38 39		(1.28) (1.22)	(24.40) (12.44)		(151.67) (151.67)	15.8 9.9	A A
54	90.0	49	24	38			0.34	U	0.45									
	27.44m	49	24	38			(16.27)	U	(21.53)									
64	109.5			37			0.32	U	0.56									
	33.38m			37			(15.31)	U	(26.79)									
69	114.5									3-U	42	30	70	4.03		9.06	4.4	A
	34.91m									3-U	42	30	(1.12)	(192.82)		(433.50)	4.4	A
70	115.0	64	27	45			0.40	U	0.61									
	35.06m	64	27	45			(19.14)	U	(29.19)									
75	126.0						0.40	U	0.46									
	38.41m						(19.14)	U	(22.00)									
85	149.5									CRS and Incremental Consolidation Tests (See Plates B-19 and B-16) Static Simple Shear Test (See Plate B-11 and B-12)								
	45.58m									CRS and Incremental Consolidation Tests (See Plates B-19 and B-16) Static Simple Shear (See Plates B-11 and B-12)								
86	150.0	54	24	37			0.44	U	0.84									
	45.73m	54	24	37			(21.05)	U	(40.19)									
89	152.5			48			0.48											
	46.49m			48			(22.97)											
98	169.5				105					2-U 2-R	45 48		71 68	0.83 0.47		7.06 7.06	10.2 15.9	A,B A
	51.69m				(1.68)					2-U 2-R	48 48		(1.14) (1.09)	(39.71) (22.49)		(337.80) (337.80)	10.2 15.9	A,B A
99	170.0	76	31	49			0.62	U	0.79									
	51.83m	76	31	49			(29.67)	U	(37.80)									
110	189.2									3-U	54	36	66	4.74		15.98	5.5	A,B
	57.67m									3-U	54	36	(1.05)	(226.79)		(764.60)	5.5	A,B

(Continued on Plate B-1b)

LEGEND AND NOTES

TYPE OF TEST
1 UNCONFINED COMPRESSION
2 UNCONSOLIDATED-UNDRAINED TRIAXIAL
3 Ko CONSOLIDATED-UNDRAINED TRIAXIAL
U = UNDISTURBED R = REMOLDED

TYPE OF FAILURE
A = BULGE
B = SINGLE SHEAR PLANE
C = MULTIPLE SHEAR PLANE
D = VERTICAL FRACTURE

(a) GRAIN-SIZE DISTRIBUTION CURVE PRESENTED SEPARATELY
(b) STRESS-STRAIN CURVE PRESENTED SEPARATELY

BORING 5, BLOCK 56
WEST DELTA AREA

Seafloor at El - 53'

McCLELLAND ENGINEERS

SUMMARY OF TEST RESULTS

SAMPLE NUMBER	PENETRATION, FEET	CLASSIFICATION TESTS: LIQUID LIMIT	CLASSIFICATION TESTS: PLASTIC LIMIT	CLASSIFICATION TESTS: WATER CONTENT, %	CLASSIFICATION TESTS: UNIT WET WEIGHT, PCF/(Mg/m3)	CLASSIFICATION TESTS: PERCENT PASSING NO. 200 SIEVE	TORVANE: SHEAR STRENGTH, KSF/(kPa)	MINIATURE VANE: TYPE OF TEST	MINIATURE VANE: SHEAR STRENGTH KSF/(kPa)	COMPRESSION TESTS: TYPE OF TEST	COMPRESSION TESTS: WATER CONTENT, % Initial	COMPRESSION TESTS: WATER CONTENT, % Final	COMPRESSION TESTS: UNIT DRY WEIGHT, PCF/(Mg/m3)	COMPRESSION TESTS: SHEAR STRENGTH, KSF/(kPa)	COMPRESSION TESTS: ϵ_{50} STRAIN, %	COMPRESSION TESTS: LATERAL PRESSURE, KSF/(kPa)	COMPRESSION TESTS: FAILURE STRAIN, %	COMPRESSION TESTS: TYPE OF FAILURE
							(Continued from Plate B-1a)											
112	190.0	89	36	54			0.70	U	1.05									
	57.93m	89	36	54			(33.49)	U	(50.23)									
116	200.0			54			0.86	U	1.27									
	60.96m			54			(41.15)	U	(60.77)									
122	210.0				106					2-U 2-R	55 52		68 66	1.27 0.60		8.78 8.78	7.0 14.5	C A
	64.02m				(1.70)					2-U 2-R	55 52		(1.09) (1.06)	(60.77) (28.71)		(420.10) (420.10)	7.0 14.5	C A
138	228.7									CRS and Incremental Consolidation Test (See Plates B-20 and B-17) Static Simple Shear Test (See Plate B-13 and B-14)								
	69.73m									CRS and Incremental Consolidation Test (See Plates B-20 and B-17) Static Simple Shear Test (See Plates B-13 and B-14)								
139	229.0	83	32	53			1.48	U	1.80									
	69.82m	83	32	53			(70.81)	U	(86.12)									
								BORING 4										
6	233.0			57			1.28	U	1.25									
	71.04m			57			(61.24)	U	(59.81)									
								BORING 6										
23	81.0			41				U	0.21									
	24.70m			41				U	(10.05)									
25	101.5			37			0.44	U	0.53									
	30.95m			37			(21.05)	U	(25.36)									
31	121.0			37				U	0.59									
	36.89m			37				U	(28.23)									
36B	136.0				112					2-U 2-R	36 34		82 85	0.70 0.52		5.62 5.62	12.9 12.9	A A
	41.46m				(1.79)					2-U 2-R	36 34		(1.31) (1.36)	(33.49) (24.88)		(268.90) (268.90)	12.9 12.9	A A
37	136.5	51	24	37			0.38	U	0.52									
	41.62m	51	24	37			(18.18)	U	(24.88)									
52	176.3			51			0.52	U	0.94									
	53.75m			51			(24.88)	U	(44.98)									
58	200.8						0.70											
	61.22m						(33.49)											

LEGEND AND NOTES

TYPE OF TEST
1 UNCONFINED COMPRESSION
2 UNCONSOLIDATED-UNDRAINED TRIAXIAL
3 Ko CONSOLIDATED-UNDRAINED TRIAXIAL
U = UNDISTURBED R = REMOLDED

TYPE OF FAILURE
A = BULGE
B = SINGLE SHEAR PLANE
C = MULTIPLE SHEAR PLANE
D = VERTICAL FRACTURE

(a) GRAIN-SIZE DISTRIBUTION CURVE PRESENTED SEPARATELY
(b) STRESS-STRAIN CURVE PRESENTED SEPARATELY

BORINGS 4, 5 AND 6, BLOCK 58
WEST DELTA AREA

Seafloor at El - 53'

McCLELLAND ENGINEERS

SUMMARY OF TEST RESULTS

SAMPLE NUMBER	PENETRATION, FEET	CLASSIFICATION TESTS: LIQUID LIMIT	PLASTIC LIMIT	WATER CONTENT, %	UNIT WET WEIGHT, PCF/(Mg/m3)	PERCENT PASSING NO. 200 SIEVE	TORVANE: SHEAR STRENGTH, KSF/(kPa)	MINIATURE VANE: TYPE OF TEST	SHEAR STRENGTH KSF/(kPa)	COMPRESSION TESTS: TYPE OF TEST	WATER CONTENT, % Initial	WATER CONTENT, % Final	UNIT DRY WEIGHT, PCF/Mg/m3)	SHEAR STRENGTH, KSF/(kPa)	ϵ_{50} STRAIN, %	LATERAL PRESSURE, KSF/(kPa)	FAILURE STRAIN, %	TYPE OF FAILURE
							REMOTE VANE											
							BORING 6											
	24.0								0.20									
	7.32m								(9.57)									
	34.0								0.36									
	10.37m								(17.22)									
	44.0								0.39									
	13.41m								(18.66)									
	54.0								0.60									
	16.46m								(28.71)									
	64.0								0.70									
	19.51m								(33.49)									
	74.0								0.48									
	22.56m								(22.97)									
	84.0								0.49									
	25.61m								(23.44)									
	94.0								0.64									
	28.66m								(30.62)									
	104.0								0.65									
	31.71m								(31.10)									
	114.0								0.72									
	34.76m								(34.45)									
	125.0								0.79									
	38.11m								(37.80)									
	134.0								0.81									
	40.85m								(38.76)									
	144.0								1.10									
	43.90m								(52.63)									
	154.0								1.22									
	46.95m								(58.37)									
	164.0								1.12									
	50.00m								(53.59)									

(Continued on Plate B-1d)

LEGEND AND NOTES

TYPE OF TEST
1 UNCONFINED COMPRESSION
2 UNCONSOLIDATED-UNDRAINED TRIAXIAL
3 Kc CONSOLIDATED-UNDRAINED TRIAXIAL
U = UNDISTURBED R = REMOLDED

TYPE OF FAILURE
A = BULGE
B = SINGLE SHEAR PLANE
C = MULTIPLE SHEAR PLANE
D = VERTICAL FRACTURE

(a) GRAIN-SIZE DISTRIBUTION CURVE PRESENTED SEPARATELY
(b) STRESS-STRAIN CURVE PRESENTED SEPARATELY

BORING 6, BLOCK 58
WEST DELTA AREA

Seafloor at El - 53'

SUMMARY OF TEST RESULTS

SAMPLE NUMBER	PENETRATION, FEET	CLASSIFICATION TESTS: LIQUID LIMIT	PLASTIC LIMIT	WATER CONTENT, %	UNIT WET WEIGHT, PCF/(Mg/m3)	PERCENT PASSING NO. 200 SIEVE	TORVANE: SHEAR STRENGTH, KSF/(kPa)	MINIATURE VANE: TYPE OF TEST	SHEAR STRENGTH KSF/(kPa)	COMPRESSION TESTS: TYPE OF TEST	WATER CONTENT, % Initial	WATER CONTENT, % Final	UNIT DRY WEIGHT, PCF/(Mg/m3)	SHEAR STRENGTH, KSF/(kPa)	ϵ_{50} STRAIN, %	LATERAL PRESSURE, KSF/(kPa)	FAILURE STRAIN, %	TYPE OF FAILURE
							(Continued from Plate											
							REMOTE VANE											
							BORING 6											
	174.0								1.20									
	53.05m								(57.42)									
	184.0								1.25									
	56.08m								(59.81)									
	194.0								1.44									
	59.15m								(68.90)									
	214.0								1.77									
	65.24m								(84.69)									
	224.0								1.12									
	68.29m								(53.59)									
	234.0								1.50									
	71.34m								(71.77)									
	244.0								1.60									
	74.39m								(76.56)									

LEGEND AND NOTES

TYPE OF TEST
1 UNCONFINED COMPRESSION
2 UNCONSOLIDATED-UNDRAINED TRIAXIAL
3 Ko CONSOLIDATED-UNDRAINED TRIAXIAL
U = UNDISTURBED R = REMOLDED

TYPE OF FAILURE
A = BULGE
B = SINGLE SHEAR PLANE
C = MULTIPLE SHEAR PLANE
D = VERTICAL FRACTURE

(a) GRAIN-SIZE DISTRIBUTION CURVE PRESENTED SEPARATELY
(b) STRESS-STRAIN CURVE PRESENTED SEPARATELY

BORING 6, BLOCK 58
WEST DELTA AREA

Seafloor at El - 53'

McCLELLAND ENGINEERS

Boring No.	Penetration, Ft (m)	Torvane ksf (kPa)	Miniature Vane ksf (kPa)
5	0.5 (1.64)	0.032 (1.53)	0.025 (1.20)
	6.0 (19.68)	0.088 (4.21)	0.087 (4.16)
	11.0 (36.08)	-	0.087 (4.16)
	17.0 (5.18)	0.100 (4.78)	0.190 (9.10)
	21.5 (6.55)	0.148 (7.09)	0.230 (11.01)
	26.5 (8.08)	0.184 (8.81)	0.200 (9.57)
	32.0 (11.28)	0.320 (15.32)	0.360 (17.23)
	32.0 (11.28) R	-	0.102 (4.88)
	35.5 (10.82)	0.180 (8.62)	0.190 (9.10)
	37.0 (11.28)	0.120 (5.74)	0.150 (7.18)
	42.0 (12.80)	0.200 (9.57)	0.320 (15.32)
	47.0 (14.33)	0.200 (9.57)	0.270 (12.93)
	47.0 (14.33) R	-	0.145 (6.96)
	52.0 (15.85)	0.300 (14.36)	0.420 (20.11)
	52.0 (15.85)	-	0.033 (1.57)
	57.0 (17.38)	0.300 (14.36)	0.420 (20.11)
	57.0 (17.38)	0.216 (10.34)	0.200 (9.57)
	62.0 (18.90)	0.224 (10.72)	0.350 (16.76)
	67.0 (20.43)	0.300 (14.36)	0.390 (18.67)
	67.0 (20.43) R	-	0.371 (17.73)
	72.0 (21.95)	0.200 (9.57)	0.370 (17.71)
	77.0 (23.47)	0.180 (8.61)	0.361 (17.28)
	77.0 (23.47) R	-	0.216 (10.32)
	82.0 (25.00)	0.220 (10.53)	0.420 (20.11)
	87.0 (26.52)	0.200 (9.57)	0.408 (19.52)
	92.0 (28.05)	0.280 (13.40)	0.450 (21.54)
	97.0 (29.57)	-	0.488 (23.34)
	97.0 (29.57) R	-	0.286 (13.69)
	102.0 (31.09)	-	0.544 (26.03)
	107.0 (32.61)	-	0.399 (19.07)
	112.0 (34.14)	-	0.530 (25.36)
	117.0 (35.67)	0.340 (16.28)	0.500 (23.94)
	117.0 (35.67)	-	0.236 (11.22)
	122.0 (37.20)	0.460 (22.02)	0.610 (29.20)
	130.0 (39.63)	0.380 (18.19)	0.560 (26.81)
	132.0 (40.24)	-	0.440 (21.06)
	137.0 (41.77)	0.360 (17.23)	0.470 (22.50)
	137.0 (41.77) R	-	0.349 (16.71)
	142.0 (43.29)	0.540 (25.85)	0.810 (38.78)
	152.0 (46.33)	0.520 (24.89)	0.750 (35.90)
	157.0 (47.87)	0.560 (26.81)	0.730 (34.95)
	157.0 (47.87) R	-	0.370 (17.69)

(Continued on Plate B-2b)

Note: R Denotes residual miniature vane test

SUMMARY OF TORVANE AND MINIATURE VANE TEST RESULTS FOR SAMPLES NOT RETAINED BY McCLELLAND ENGINEERS, INC.

Borings 4, 5, and 6, Block 58

West Delta Area

(Continued from Plate B-2a)

Boring No.	Penetration, Ft (m)	Torvane ksf (kPa)	Miniature Vane ksf (kPa)
	162.0 (49.39)	-	0.790 (37.82)
	167.0 (50.91)	0.640 (30.64)	0.830 (39.73)
	174.0 (53.04)	0.520 (24.89)	0.740 (35.43)
	177.0 (53.96)	0.540 (25.85)	0.650 (31.11)
	179.0 (54.57)	0.700 (33.51)	0.920 (44.04)
	182.0 (55.49)	0.700 (33.51)	0.637 (30.46)
	182.0 (55.49) R	-	0.308 (14.74)
	187.0 (57.00)	0.700 (33.51)	0.976 (46.68)
	197.0 (60.06)	0.840 (40.21)	0.990 (47.39)
	202.0 (61.58)	0.980 (46.91)	1.470 (70.37)
	207.0 (63.11)	1.260 (60.32)	1.500 (71.81)
	212.0 (64.63)	1.320 (63.19)	1.440 (68.94)
	213.0 (64.94)	1.500 (71.81)	1.360 (65.11)
	215.0 (65.55)	1.100 (52.66)	1.600 (76.60)
	219.0 (66.77)	1.200 (57.45)	1.410 (67.50)
	219.0 (66.77) R	-	0.575 (27.52)
	224.0 (68.29)	1.260 (60.32)	1.730 (82.82)
	226.0 (68.90)	1.340 (64.15)	1.790 (85.69)
	226.0 (68.90) R	-	0.760 (36.36)
4	231.0 (70.42)	1.240 (59.36)	1.500 (71.81)
	237.0 (72.26)	1.640 (78.51)	1.850 (88.56)
	237.0 (72.26) R	-	1.087 (52.03)
	240.0 (73.17)	1.500 (71.83)	1.900 (90.96)
6	22.0 (6.71)	1.108 (5.17)	0.190 (9.10)
	42.0 (12.80)	0.130 (6.22)	0.290 (13.88)
	62.0 (18.90)	0.200 (9.57)	0.300 (14.36)
	82.0 (25.00)	0.200 (9.57)	0.340 (16.28)
	101.0 (30.79)	0.360 (17.23)	0.380 (18.19)
	105.0 (32.01)	0.200 (9.57)	0.460 (22.02)
	128.0 (39.02)	-	0.840 (40.21)
	142.0 (43.29)	0.460 (22.02)	0.590 (28.24)
	146.6 (44.70)	0.480 (22.98)	0.680 (32.55)
	162.0 (49.30)	0.540 (22.85)	0.770 (36.86)
	182.0 (55.49)	0.620 (29.68)	0.780 (37.34)
	212.0 (64.63)	1.040 (49.79)	1.190 (56.97)
	222.0 (67.68)	1.200 (57.45)	1.330 (63.67)
	242.0 (73.78)	1.440 (68.94)	1.730 (82.82)

Note: R denotes residual miniature vane test

SUMMARY OF TORVANE AND MINIATURE VANE TEST RESULTS FOR SAMPLES NOT RETAINED BY McCLELLAND ENGINEERS, INC.

Borings 4, 5 and 6, Block 58

West Delta Area

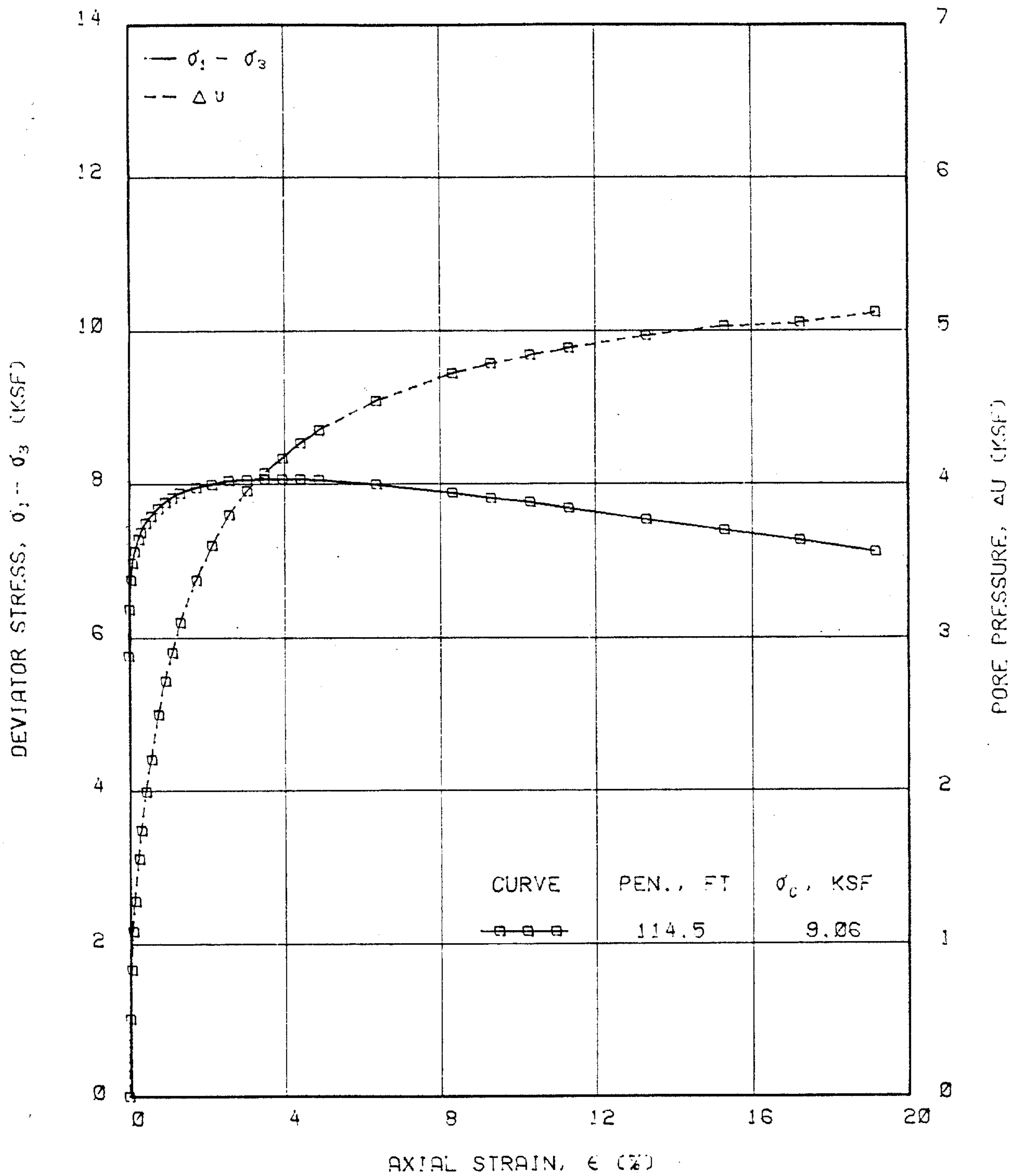
Boring No.	Sample No.	Penetration, Ft (m)	Liquid Limit, %	Plastic Limit, %	Water Content, %		Liquidity Index Final	Unit Dry Weight Final, pcf (Mg/m^3)	Sample Dimensions, in., (mm)		Anisotropic Consolidation Data		Failure Pressure and Stresses, ksf (kPa)				Undrained Shear Strength, s_u, ksf (kPa)	$s_u/\bar{\sigma}_v$	Strain at Failure, %
					Initial	Final			Initial Diameter	Initial Height	Confining Pressure, ksf (kPa)	K_o	σ_1	Δu	$\bar{\sigma}_1$	$\bar{\sigma}_3$			
5	69	114.5 (34.9)	64	27	42	30	0.08	92.8 (1.490)	2.15 (54.61)	3.90 (99.06)	9.06 (434)	0.61	17.13 (820)	4.26 (204)	12.87 (616)	4.80 (23.0)	4.03 (192.9)	0.27	4.43
5	110	189.2 (57.7)	89	36	54	36	0.00	100.7 (1.613)	2.20 (52.88)	3.82 (47.02)	15.98 (765)	0.79	25.44 (1218)	6.77 (324)	18.68 (894)	9.21 (441)	4.73 (226.5)	0.23	5.46

SUMMARY OF Ko CONSOLIDATED-UNDRAINED TRIAXIAL TEST RESULTS

Boring 5, Block 58

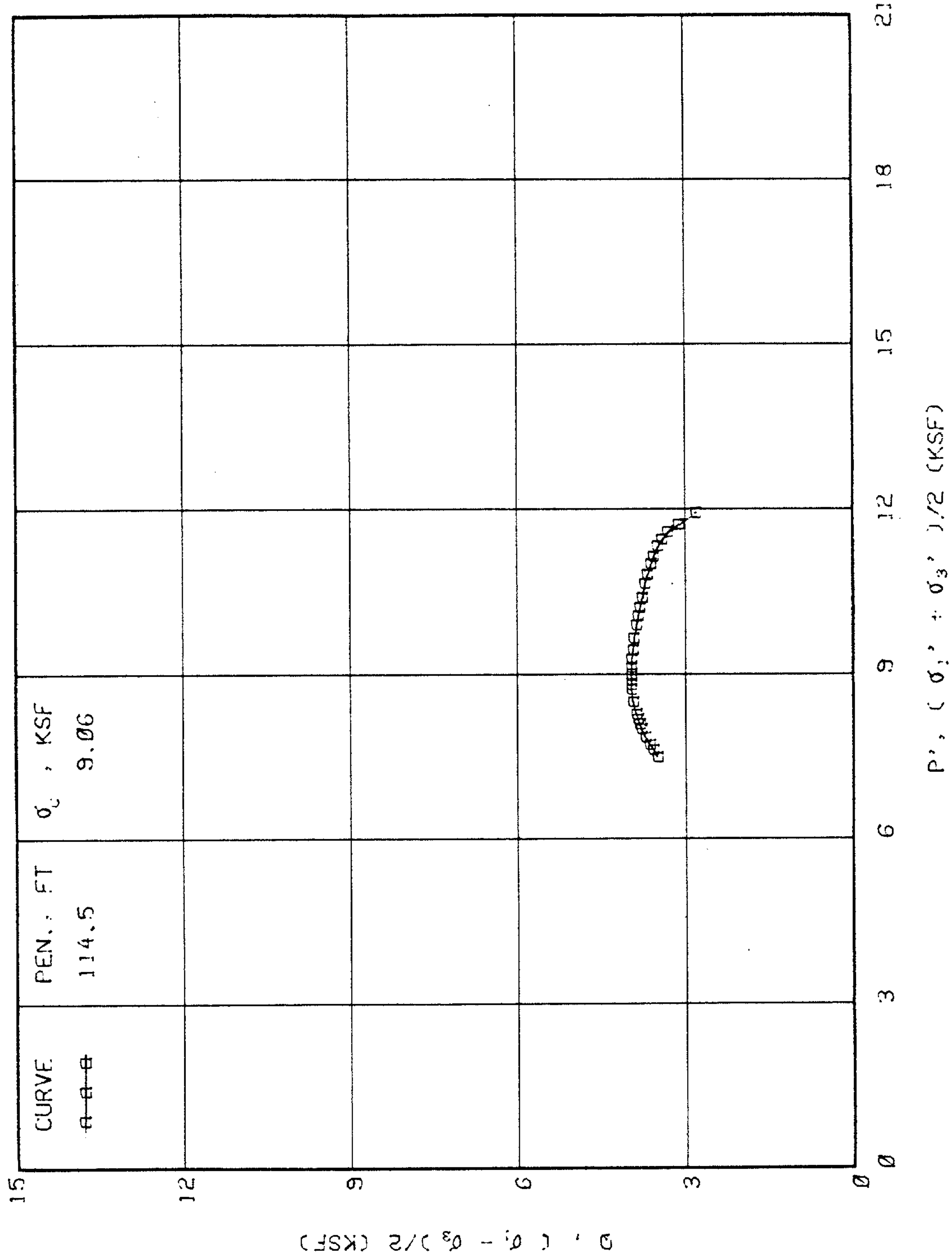
West Delta Area

Ø181-Ø217



STRESS-STRAIN CURVES
Ko Consolidated-Undrained Triaxial Test
Boring 5

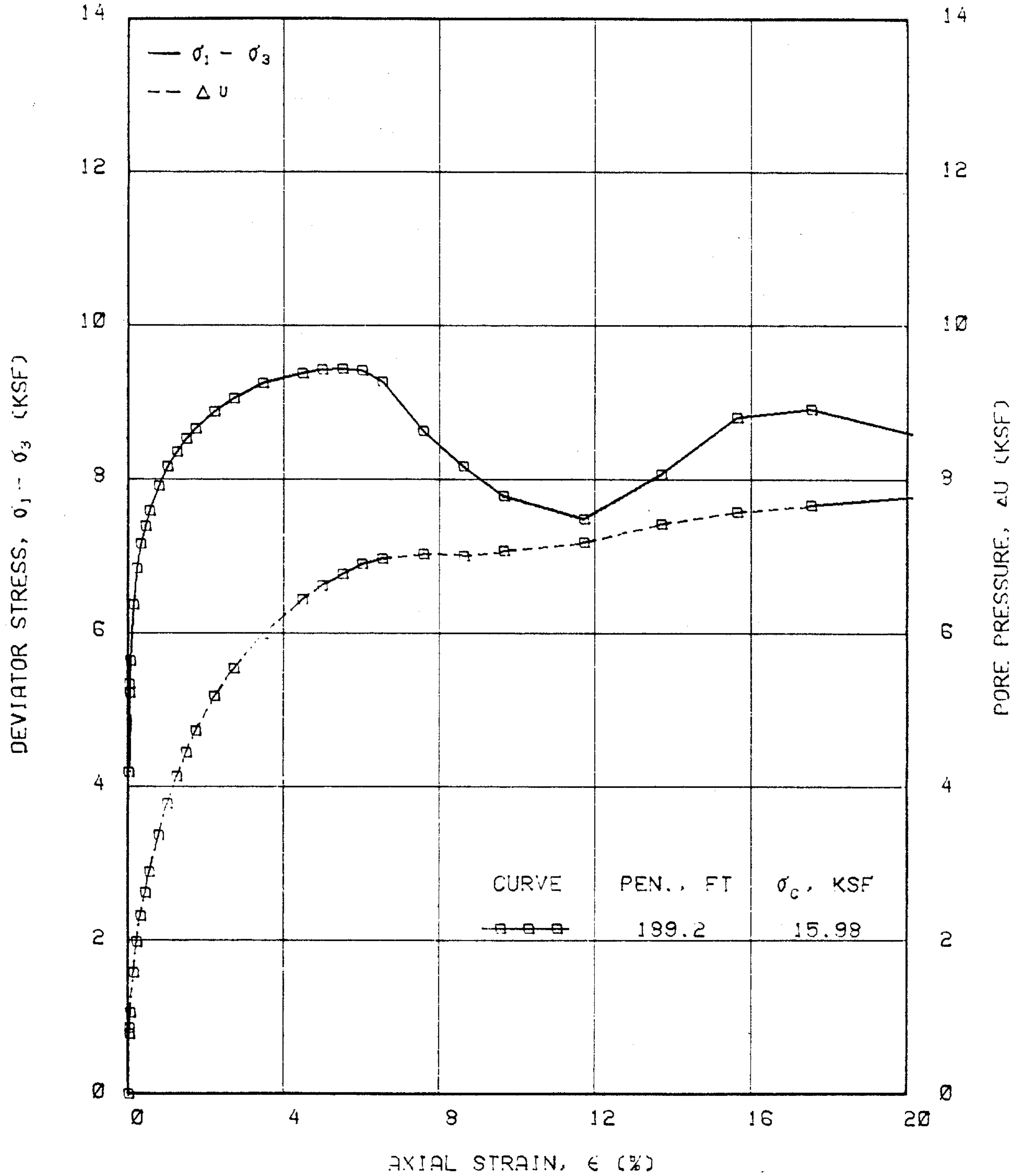
0191-0217



P' - Q DIAGRAM

Ko Consolidated-Undrained Triaxial Test

Boring 5

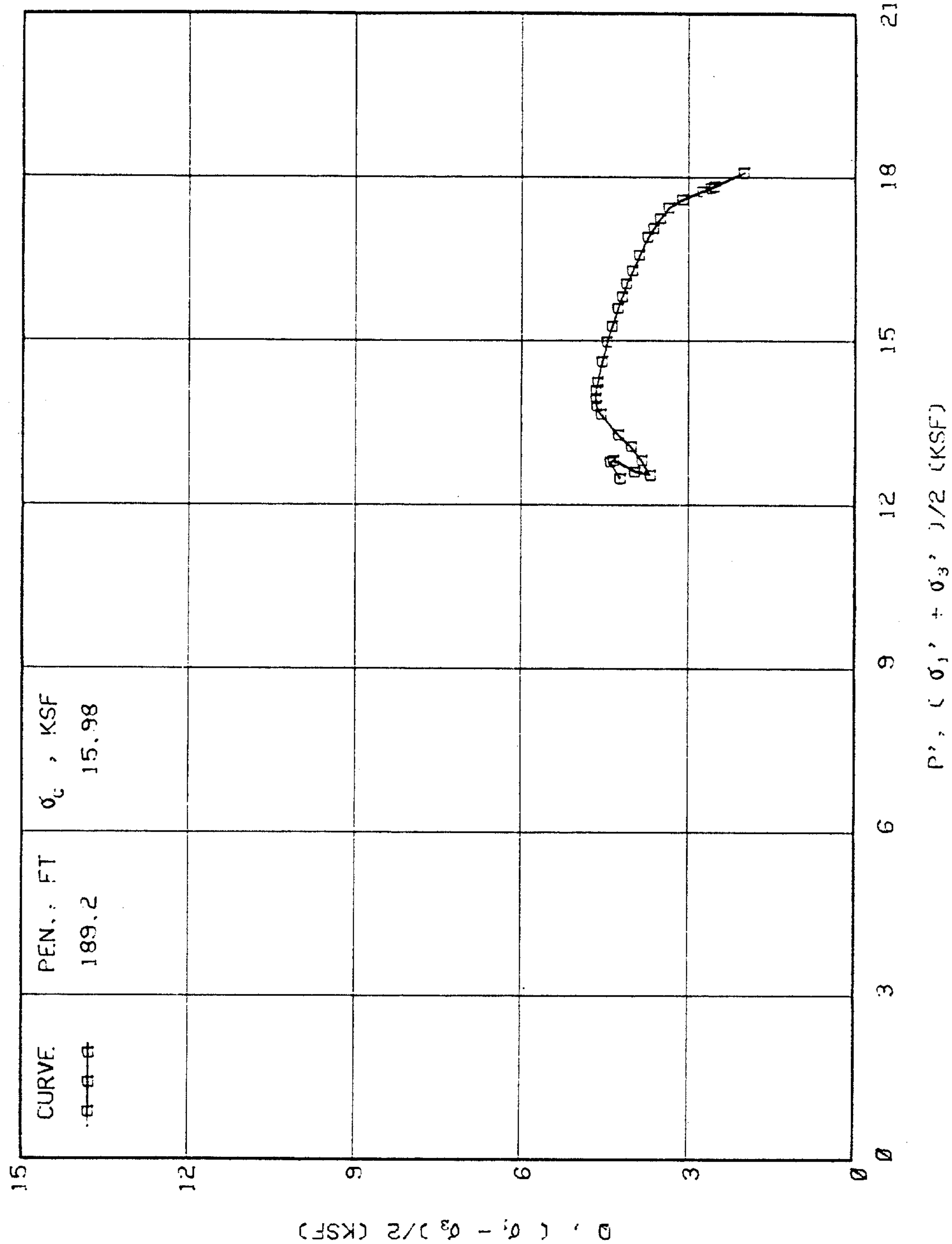


STRESS-STRAIN CURVES

Ko Consolidated-Undrained Triaxial Test

Boring 5

Ø181-Ø217



P' - Q DIAGRAM

Ko Consolidated-Undrained Triaxial Test

Boring 5

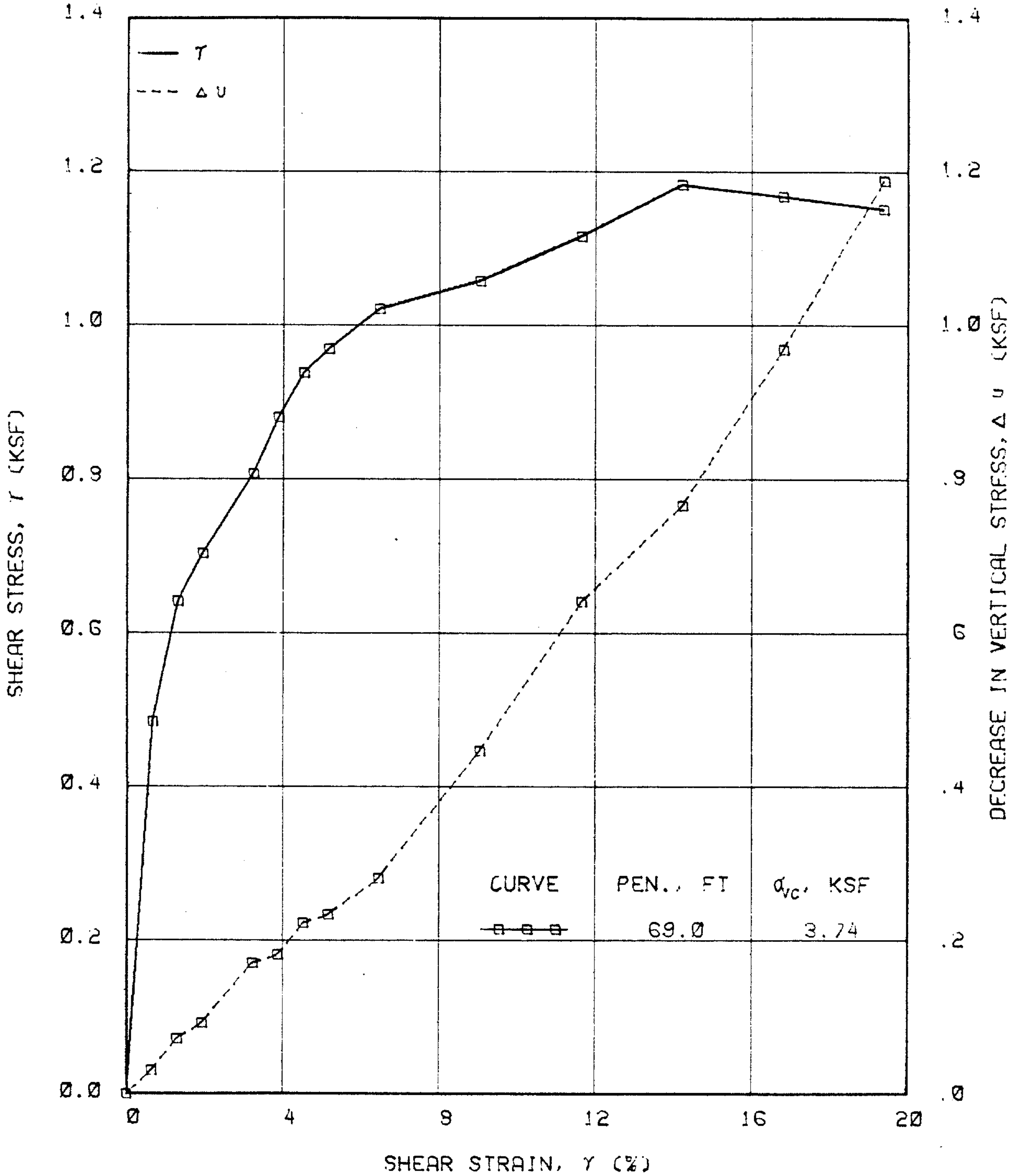
Boring No.	Sample No.	Penetration, Ft (m)	Liquid Limit, %	Plastic Limit, %	Water Content, %		Liquidity Index Final	Sample Height, in. (mm)		Effective Vertical Consolidation Pressure, $\bar{\sigma}_{vc}$, ksf (kPa)	OCR	Undrained Shear Strength, s_u, ksf (kPa)	$s_u/\bar{\sigma}_{vc}$	Shear Strain at Failure, %
					Initial	Final		Initial	Final					
5	42	69.0 (21.0)	64	27	73	58	0.84	0.75 (19.05)	0.61 (15.50)	3.74 (179)	1	1.18 (56.7)	0.32	14.1
5	85	149.5 (45.5)	54	24	36	29	0.17	0.75 (19.05)	0.63 (16.00)	9.36 (448)	1	2.58 (12.3)	0.28	13.8
5	138	228.7 (69.7)	83	32	63	44	0.24	0.75 (19.05)	0.62 (15.75)	14.26 (683)	1	3.30 (158)	0.23	19.2

SUMMARY OF STATIC SIMPLE SHEAR TEST RESULTS

Boring 5, Block 58

West Delta Area

Ø181-Ø217

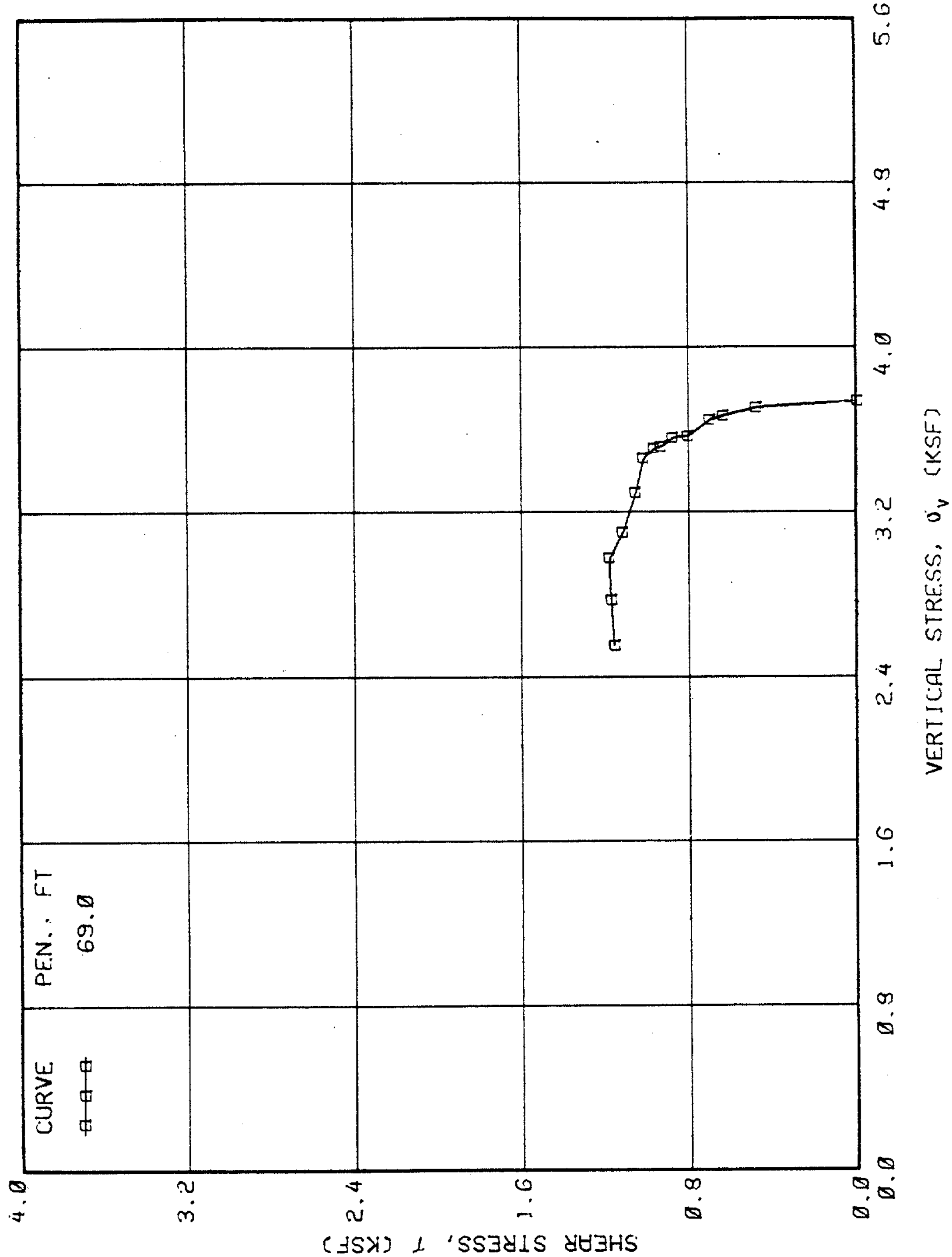


STRESS-STRAIN CURVES

DEFORMATION-CONTROLLED SIMPLE SHEAR TEST

BORING 5

J181-Ø217

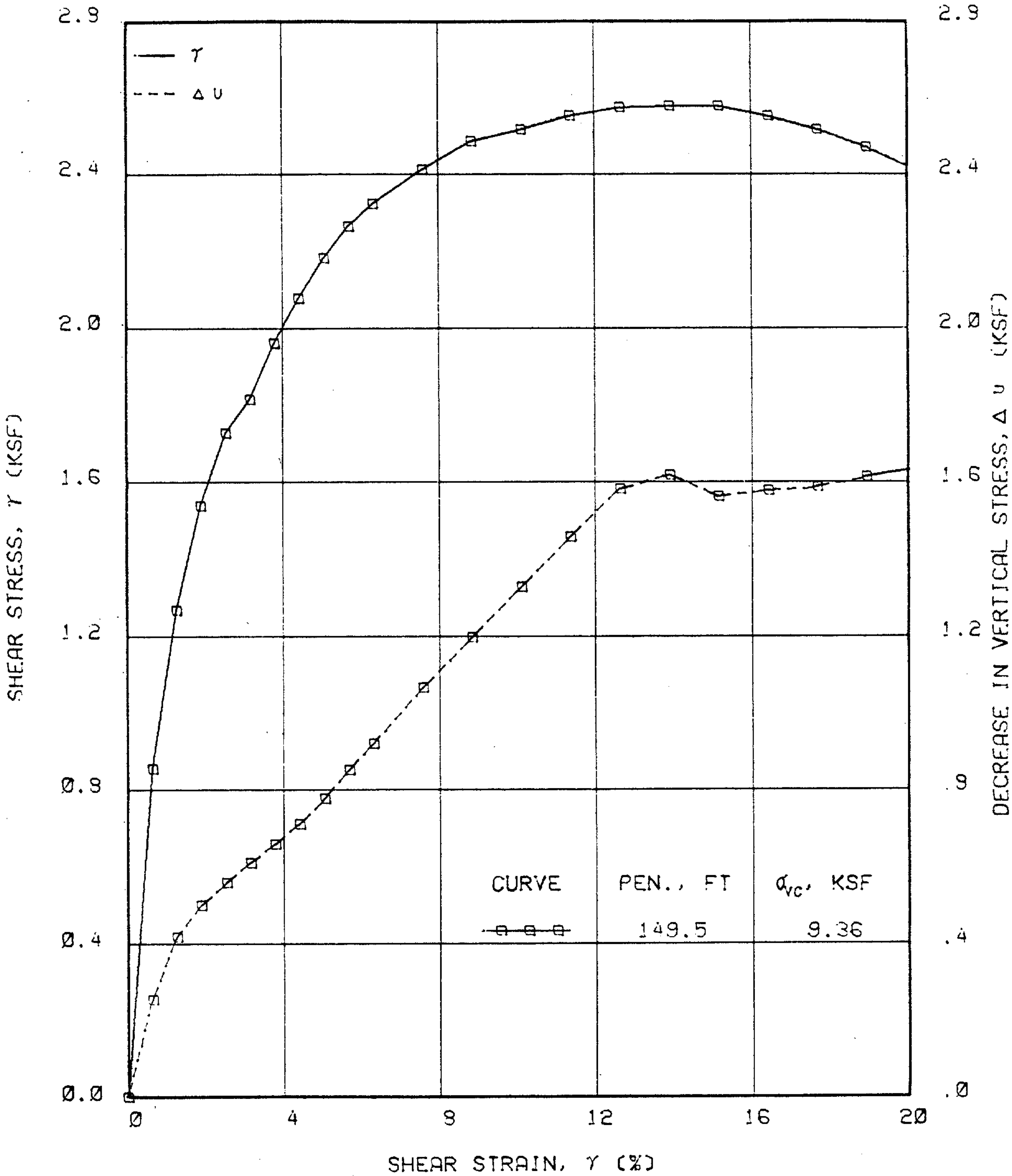


STRESS PATH

DEFORMATION-CONTROLLED SIMPLE SHEAR TEST

BORING 5

0181-0217

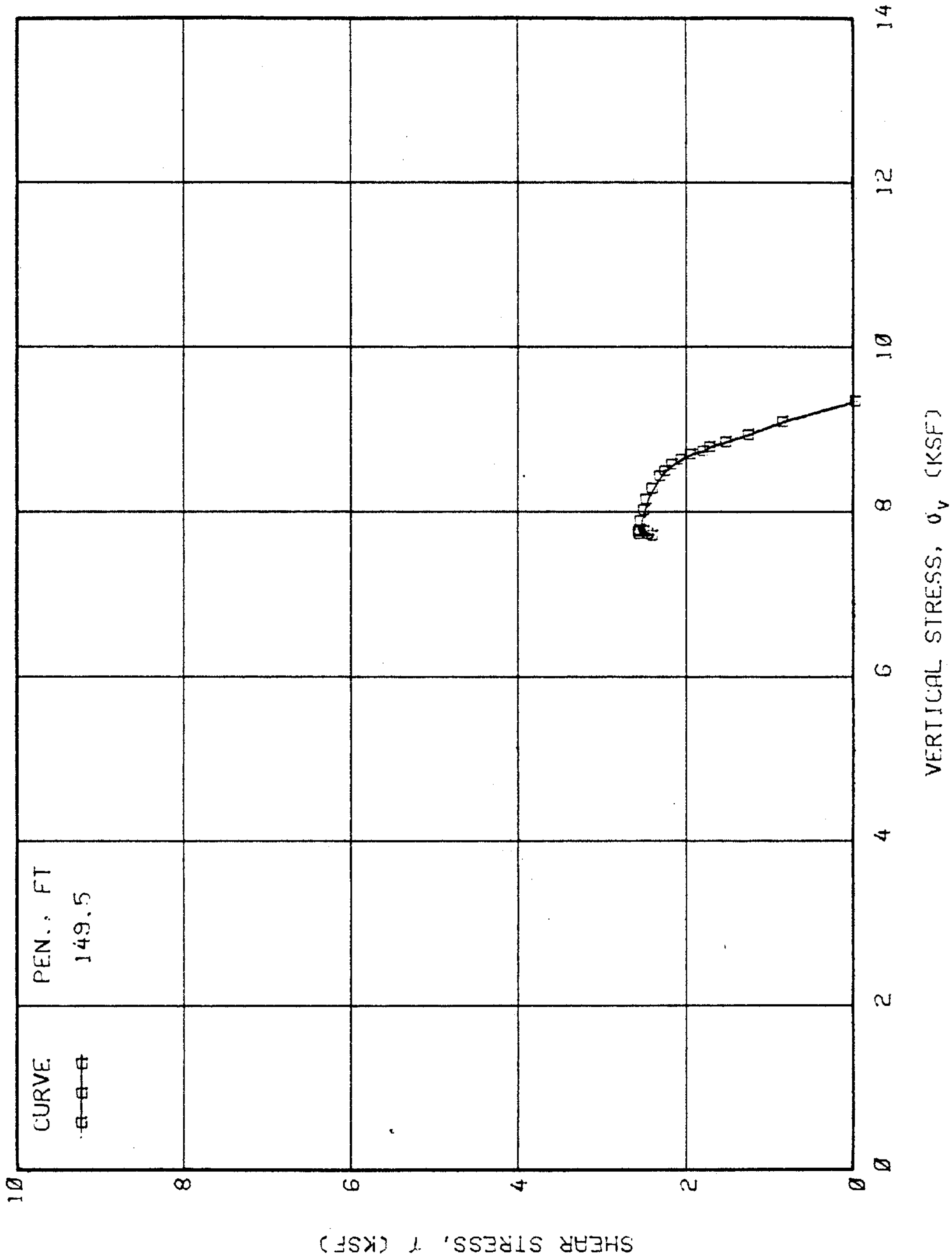


STRESS-STRAIN CURVES

DEFORMATION-CONTROLLED SIMPLE SHEAR TEST

BORING 5

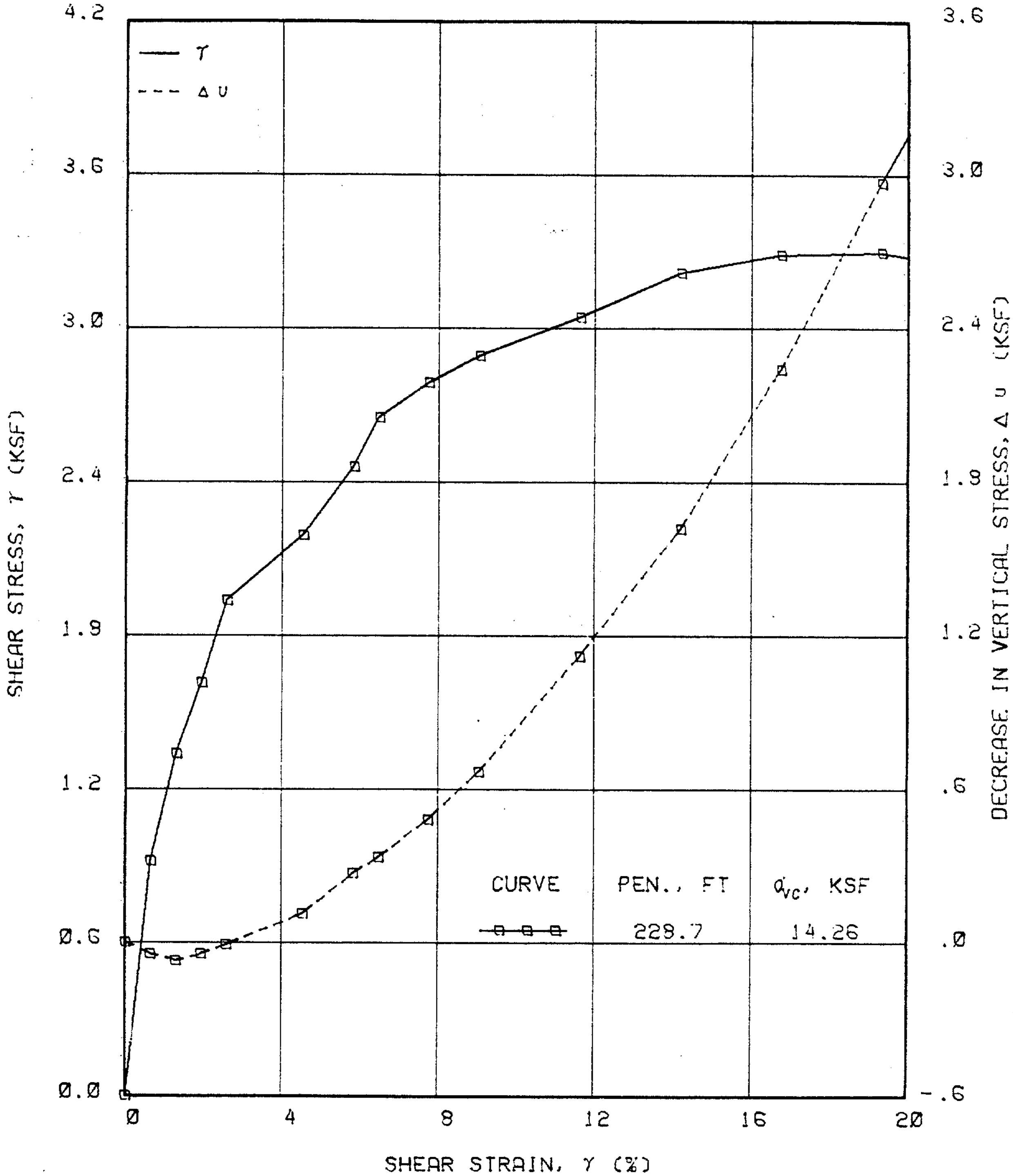
McCLELLAND ENGINEERS



STRESS PATH

DEFORMATION-CONTROLLED SIMPLE SHEAR TEST

BORING 5

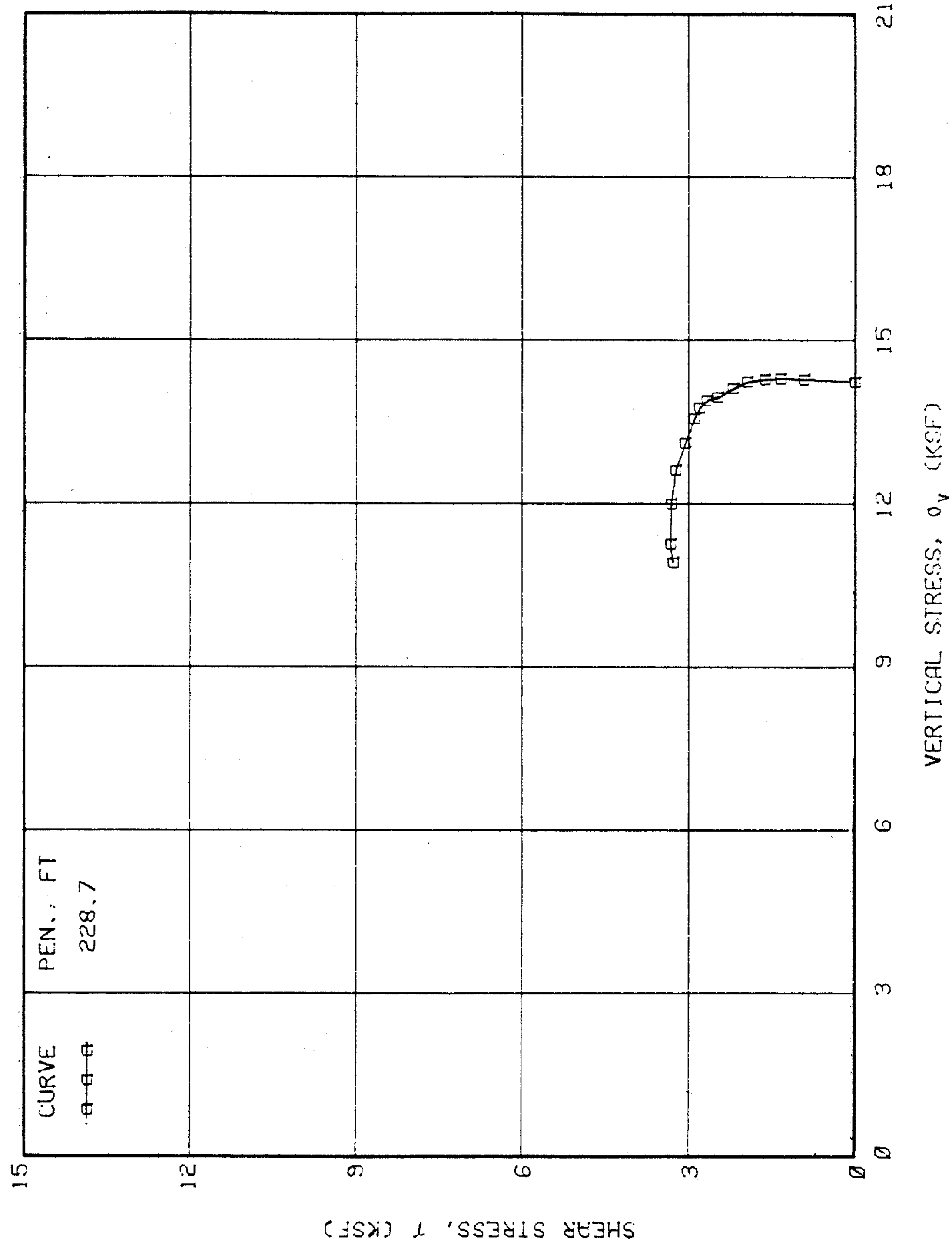


STRESS-STRAIN CURVES

DEFORMATION-CONTROLLED SIMPLE SHEAR TEST

BORING 5

Ø181-Ø217



STRESS PATH

DEFORMATION-CONTROLLED SIMPLE SHEAR TEST

BORING 5

181-Ø217

BORING: 5 PENETRATION: 69.Ø FT.

MATERIAL:

DRY MASS DENSITY: 57 PCF

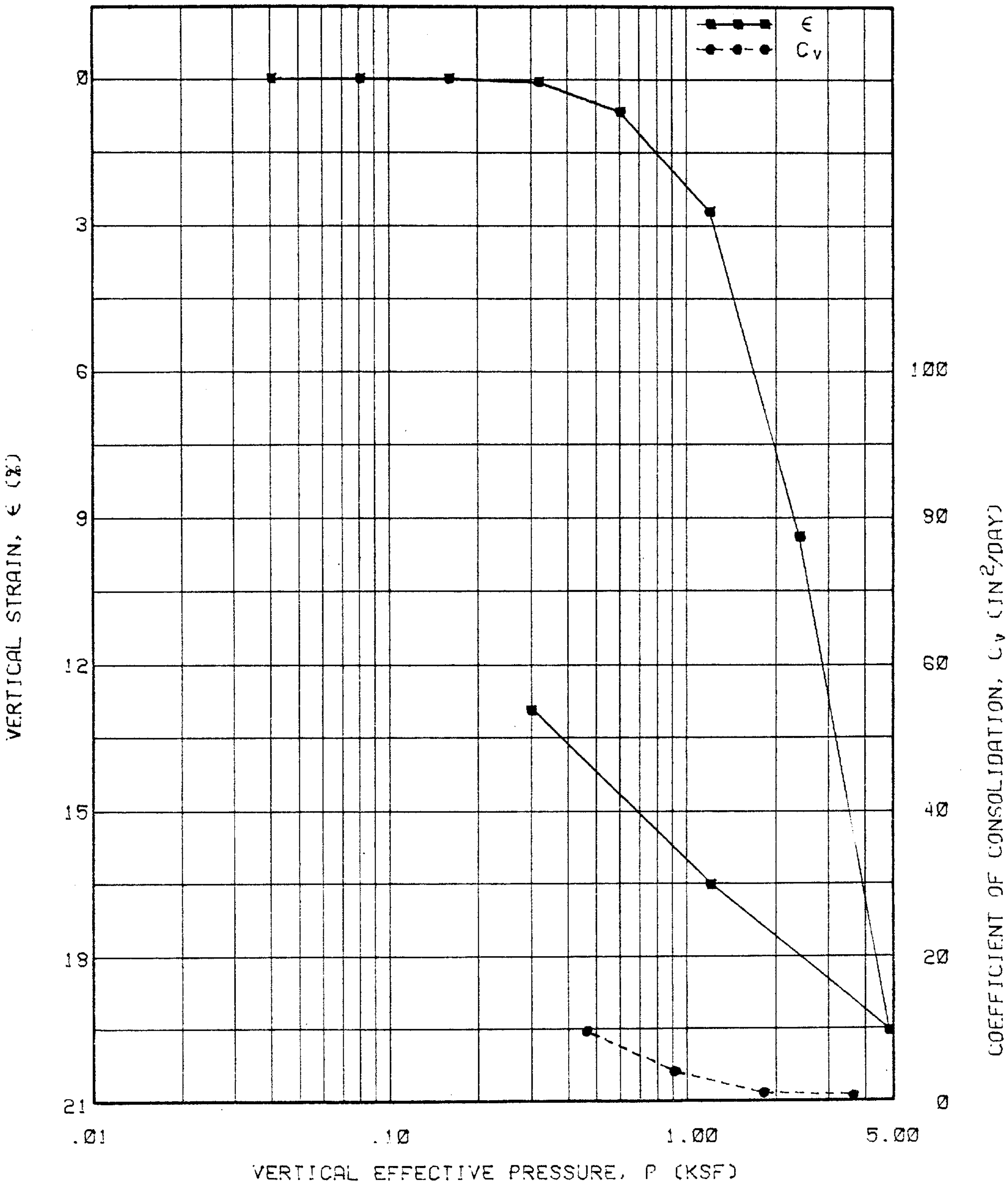
WATER CONTENT: 74 %

LIQUID LIMIT:

PLASTIC LIMIT:

SPECIFIC GRAVITY: 2.75 (ASSUMED)

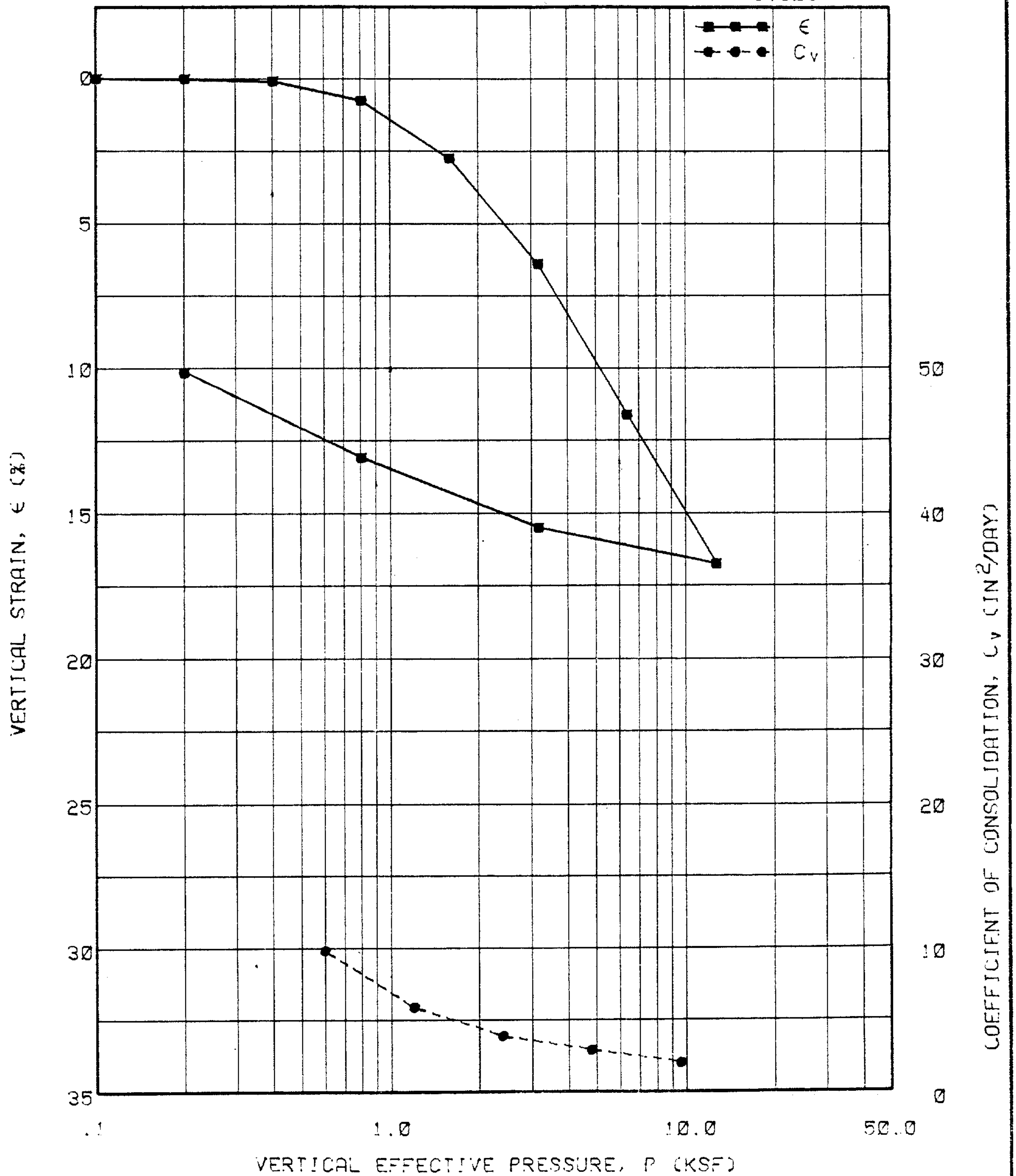
INITIAL VOID RATIO: 2.Ø24



INCREMENTAL CONSOLIDATION TEST RESULTS

181-Ø217

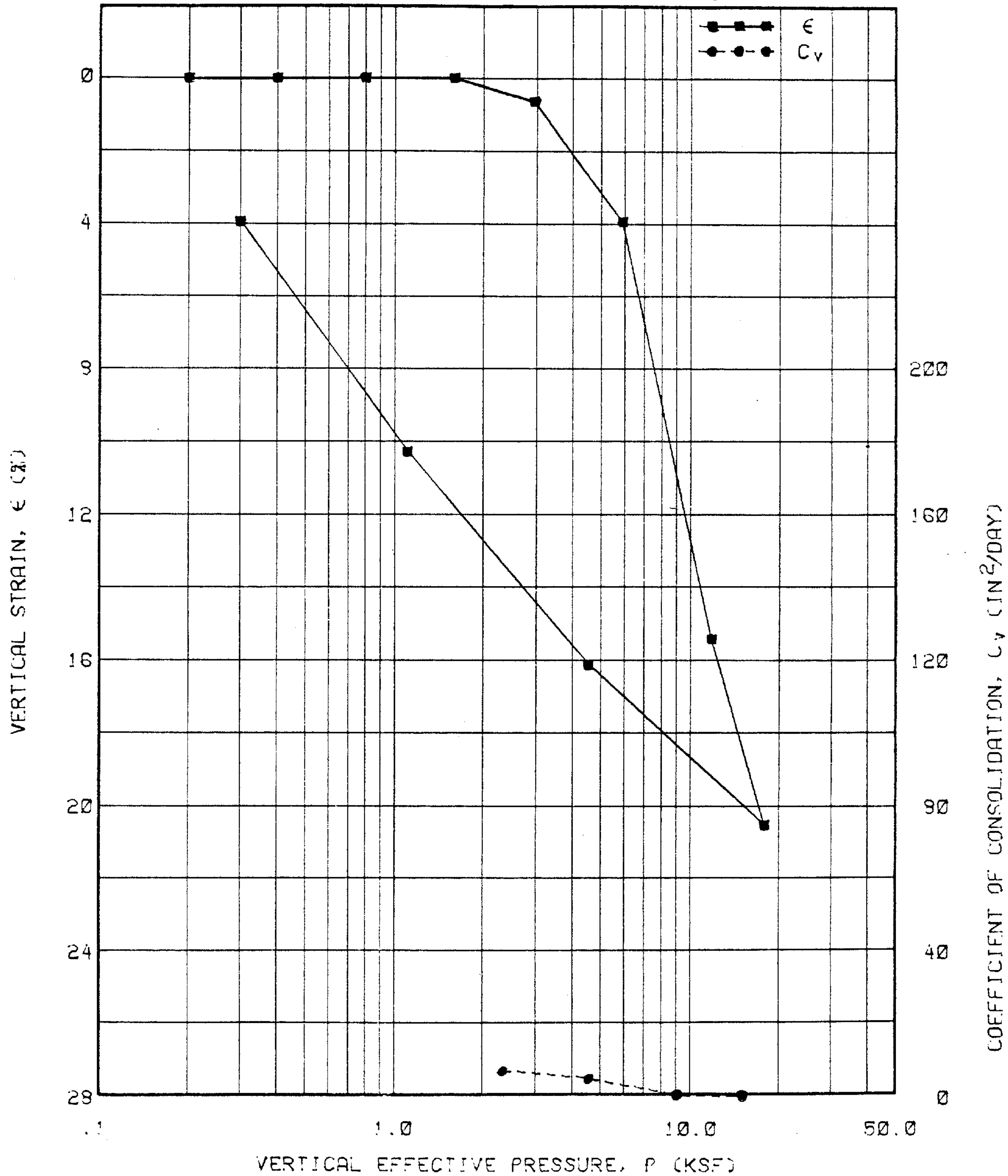
BORING:	5	DRY MASS DENSITY:	91 PCF
PENETRATION:	149.5 FT.	WATER CONTENT:	39 %
MATERIAL:		LIQUID LIMIT:	
		PLASTIC LIMIT:	
		SPECIFIC GRAVITY:	2.75 (ASSUMED)
		INITIAL VOID RATIO:	1.1Ø9



INCREMENTAL CONSOLIDATION TEST RESULTS

BORING: 5 PENETRATION: 228.7 FT.
MATERIAL:

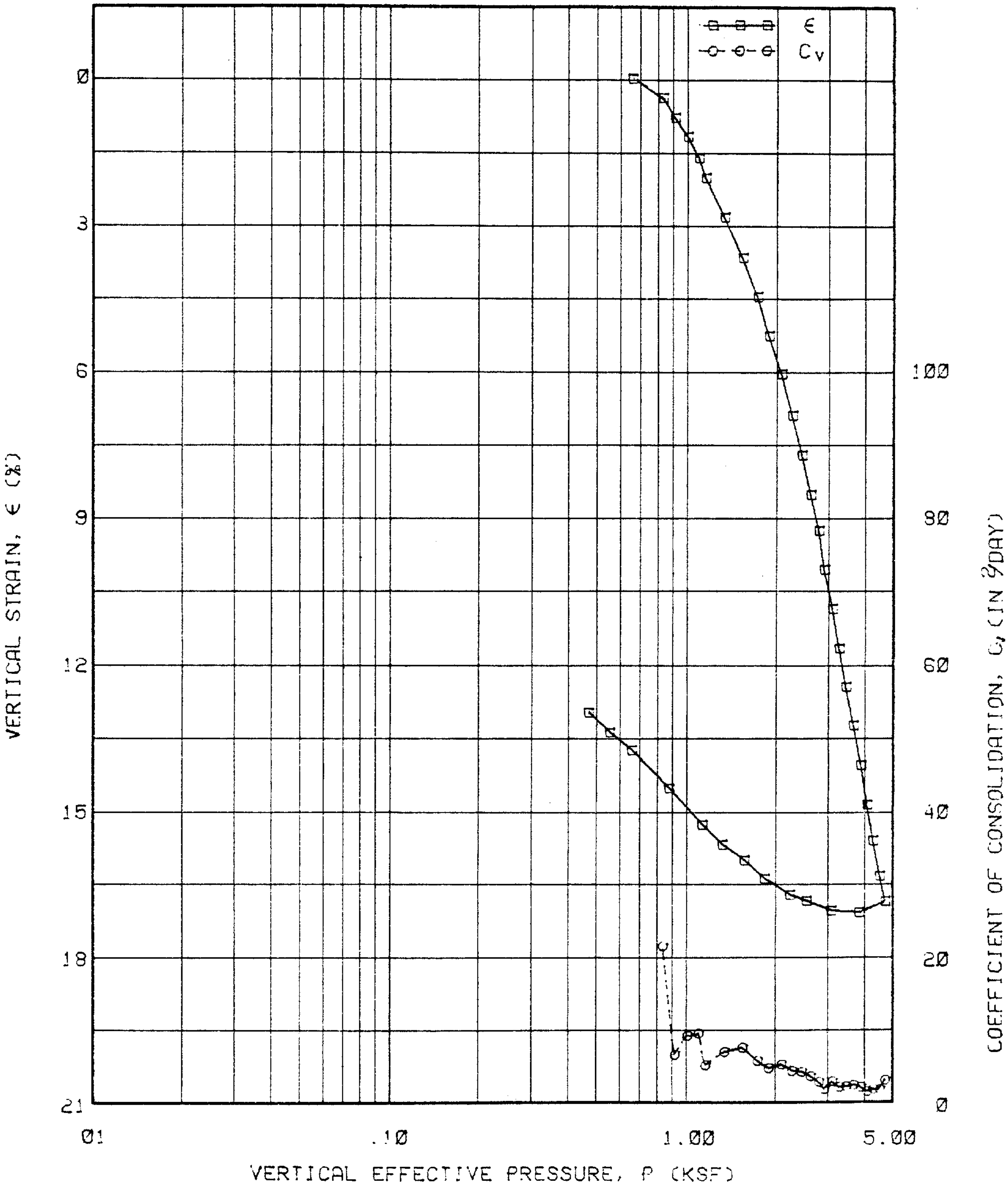
DRY MASS DENSITY: 64 PCF
WATER CONTENT: 62 %
LIQUID LIMIT:
PLASTIC LIMIT:
SPECIFIC GRAVITY: 2.90
INITIAL VOID RATIO: 1.726



INCREMENTAL CONSOLIDATION TEST RESULTS

Ø181-Ø217

BORING: 5　　PENETRATION: 69.Ø FT.　　DRY MASS DENSITY: 57 PCF
MATERIAL:　　WATER CONTENT: 76 %
LIQUID LIMIT:
PLASTIC LIMIT:
SPECIFIC GRAVITY: 2.9Ø
INITIAL VOID RATIO: 2.Ø94



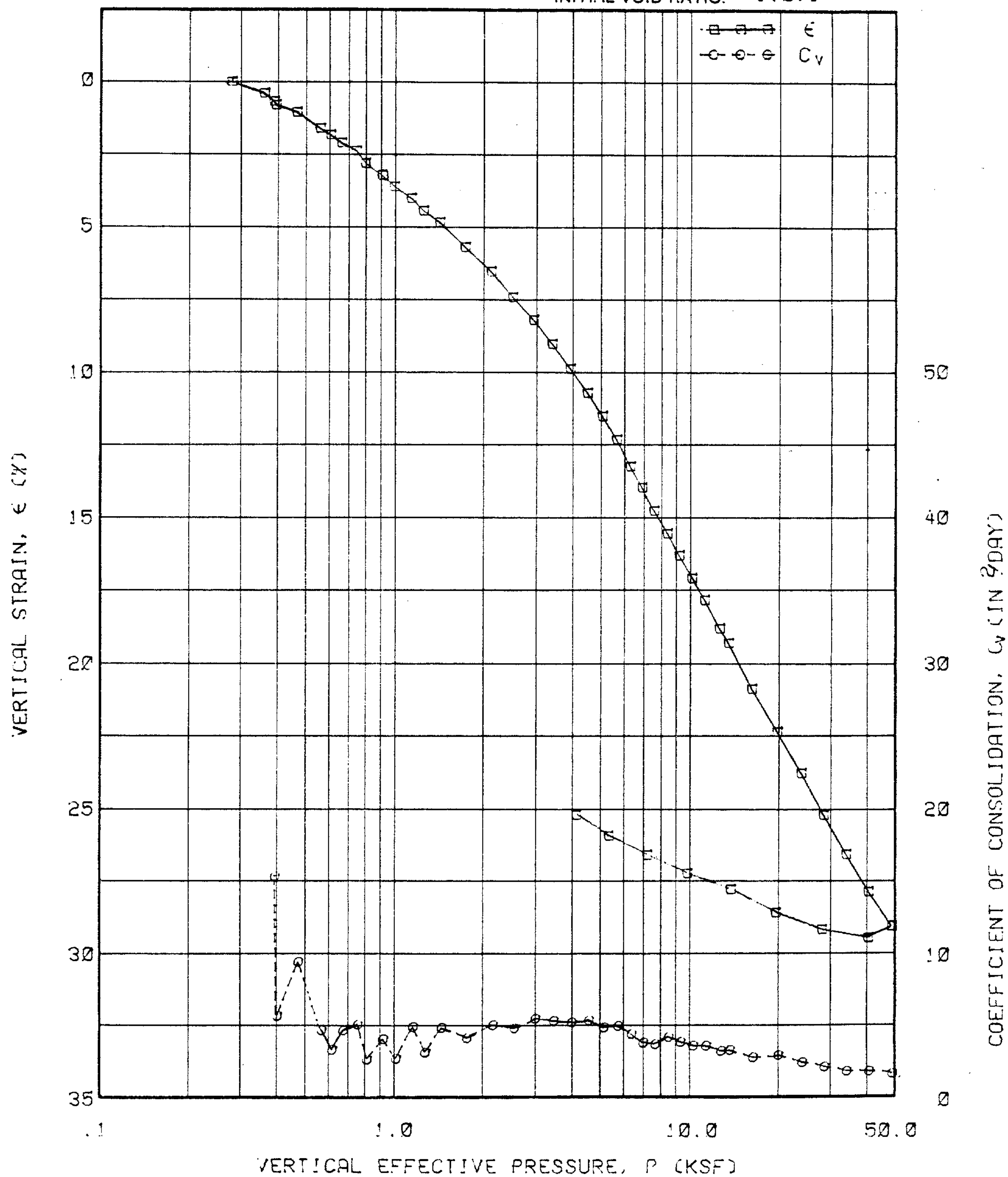
CRS CONSOLIDATION TEST RESULTS

0181-0217

BORING: 5
MATERIAL:

PENETRATION: 149.5 FT.

DRY MASS DENSITY: 83 PCF
WATER CONTENT: 39 %
LIQUID LIMIT:
PLASTIC LIMIT:
SPECIFIC GRAVITY: 2.75 (ASSUMED)
INITIAL VOID RATIO: 1.071

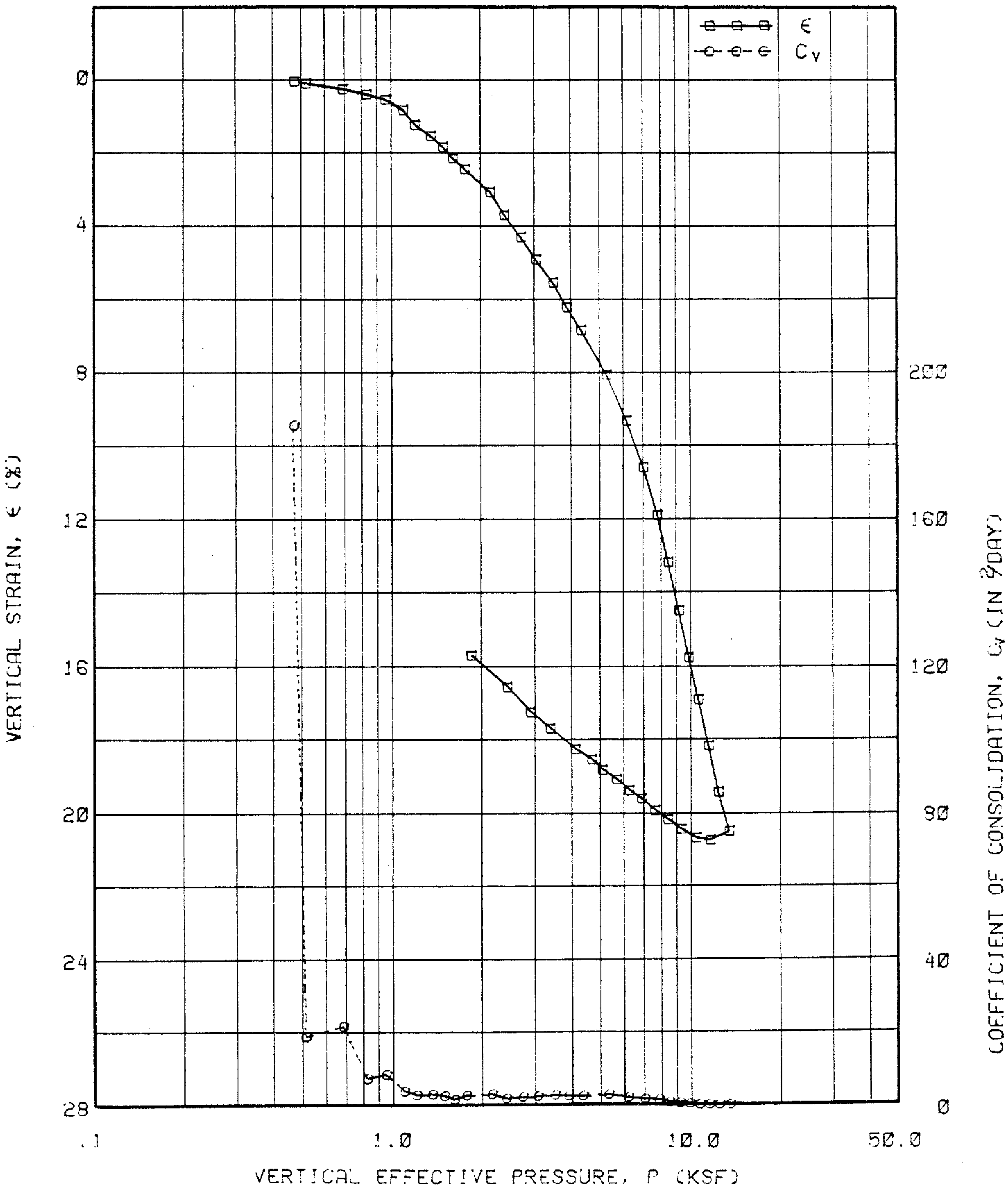


CRS CONSOLIDATION TEST RESULTS

McCLELLAND ENGINEERS

0181-0217

BORING: 5 PENETRATION: 228.7 FT. DRY MASS DENSITY: 67 PCF
MATERIAL: WATER CONTENT: 62 %
LIQUID LIMIT:
PLASTIC LIMIT:
SPECIFIC GRAVITY: 2.90
INITIAL VOID RATIO: 1.600



CRS CONSOLIDATION TEST RESULTS

APPENDIX AA

ERTEC COMMENTARY ON INTERPRETATION
OF CPT DATA

APPENDIX AA

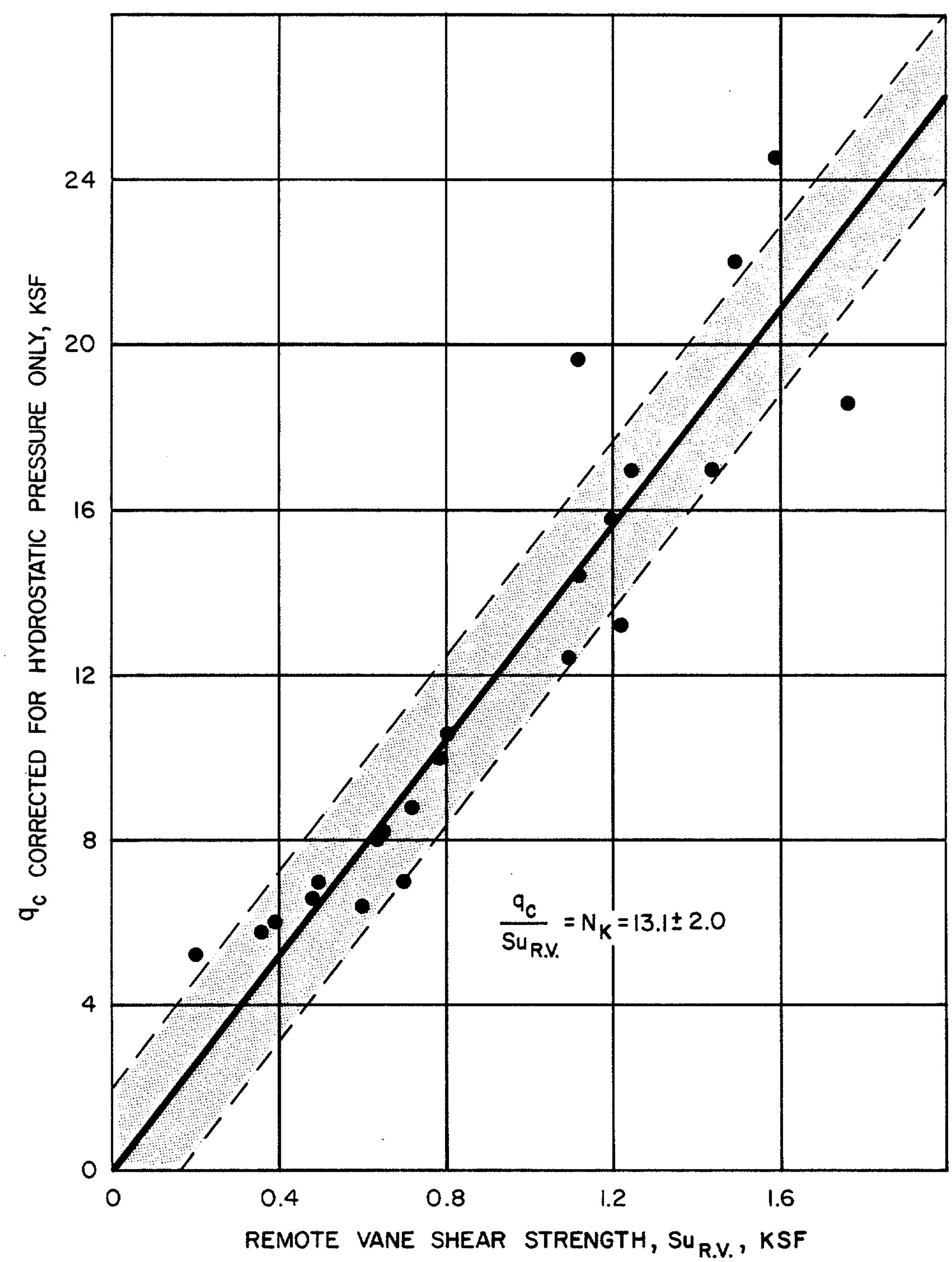
ERTEC COMMENTARY ON INTERPRETATION OF CPT DATA

The cone penetrometer log presented on Plate 3a and 3b of Appendix A (report by McClelland Engineers, Inc.) shows corrected cone resistance, q_c, as the actual pressure acting on the tip minus the hydrostatic pressure at the bottom of the borehole. Plate 10 of Appendix A further corrects q_c for the total overburden pressure. For this case, effective overburden pressure was subtracted since the water pressure had already been subtracted. The resulting value of $N_k = 6.2$ was derived from in situ vane tests and the q_c value corrected for total overburden, including water pressure.

In offshore work, a more conventional procedure is to correct q_c for the hydrostatic pressure existing at the bottom of the borehole only. To estimate N_k from q_c and a known S_u value the following equation is used:

$$N_k = \frac{q_c}{S_u}$$

where no overburden pressure is subtracted from q_c. Using this method and the in situ vane shear strengths obtained, the resulting N_k values would be 13.1 ± 2.0 as shown on Plate AA-1. This latter value for N_k is more in line with previous offshore cone penetrometer interpretations and experience.



CONE RESISTANCE VS REMOTE VANE SHEAR STRENGTH

APPENDIX B

LABORATORY TESTING PROCEDURES

INDEX PROPERTY TESTS

Grain Size Analysis

Grain size analyses were performed in accordance with ASTM D 422. Test specimens were prepared using the dry preparation method as described in the ASTM D 421.

Atterberg Limits

Atterberg limit tests were performed in accordance with ASTM D 423 and ASTM D 424. Test specimens were prepared using the wet preparation method as described in the Procurement B of ASTM D 2217-66.

Specific Gravity

Specific gravity tests were performed in accordance with ASTM D 854. The test specimens were prepared in acordance with ASTM D 421.

PHYSICAL PROPERTY TESTS

Consolidation Test

Consolidation tests were performed in accordance with ASTM D 2435 using standard dead-load type consolidometers. For each pressure increment, the sample was allowed to consolidate for 24 hours, and the deformation versus time readings were recorded. Void ratio and coefficient of consolidation were determined and plotted against consolidation pressure.

STRENGTH TESTS

Miniature Vane Shear Test

Miniature vane shear tests were performed using a Wykeham Farrance testing device. The test procedure used in this test was a modified version of ASTM D 2573. The tube samples were cut into 100 mm (4 in) sections and secured to the frame. The vane was then inserted to a tip depth of 3 to 5 cm. The vane was rotated at a rate of 10 degrees per minute until the maximum reading of rotation was obtained.

Unconfined Compression Test

Unconfined compression tests were performed in accordance with ASTM D 2166-66. The test specimens were extruded, trimmed, and placed in the Wykeham Farrance loading machine. The strain rate used in this test was approximately one percent per minute.

Unconsolidated-Undrained Triaxial Test

Unconsolidated-undrained triaxial tests were performed in accordance with ASTM D 2850. Samples were extruded from the tubes and trimmed to a 64 mm (2.5 in) diameter and a 152 mm (6.0 in) length. Trimmings were then used for determining moisture content prior to the test. During the loading phase, all samples were sheared at a strain rate of one percent per minute.

Isotropically Consolidated-Undrained Triaxial Test

Isotropically consolidated-undrained triaxial tests were performed on representative samples in accordance with Corps of Engineers procedures outlined in Engineer Manual, EM 1110-2-1906. Samples were extruded and trimmed to a 64 mm (2.5 in) diameter by 152 mm (6.0 in) length. Four samples were isotropically consolidated with pressures about three times higher than the estimated in situ vertical effective stresses. The other three samples were isotropically consolidated to about six times higher than the estimated in situ vertical effective stresses, and then rebounded in order to induce an overconsolidation ratio of 2. After the end of the primary consolidation, back pressures were increased in increments until a B value of 0.95 was attained. The applied strain rates used in the tests were computed from the measured consolidation behavior. Typically, the time to failure (or 20 percent axial strain) was on the order of 24 hours. This testing period was selected to ensure at least 95 percent pore water pressure equalization at failure following the method presented by Blight (1963). Test data such as load, deflection, and pore water pressure were automatically recorded with a data logger and computer processed to produce final test results.

K_o Consolidated-Undrained Triaxial Test

The shear strength characteristics of the soil were also evaluated in a triaxial cell under K_o conditions. These tests were carried out with a similar test procedure as described in the section titled "Isotropically Consolidated-Undrained Triaxial Test" except the samples were anisotropically consolidated to K_o condition before failure.

In this procedure, a K_o consolidation condition is maintained by imposing values of σ_1 and σ_3 in the triaxial cell such that no lateral deformation in the sample occurs during consolidation. The ratio, σ_3/σ_1, which produces this condition is considered equal to K_o for the imposed stress conditions. For most soils, the K_o value will generally decrease as consolidation pressure increases until a normally consolidated condition is reached. As consolidation pressures increase past this point, the value of K_o remains constant.

Monotonic Simple Shear Test

Consolidated-undrained monotonic simple shear tests were performed using a Geotechnical Equipment Corporation Model SS 104 simple shear device modified to allow slow strain-controlled loading. The 64 mm (2.5 in) diameter by 20 mm (0.8 in) high trimmed samples were confined in wire reinforced rubber membrane. Trimmings were used for calculation of initial moisture content.

Tests were performed on samples with overconsolidation ratios of 1.0 and 2.0. The samples were saturated utilizing back pressure to attain a B value of at least 0.95. Due to the zero lateral deflection of wire-reinforced rubber membrance, K_o consolidation can be achieved.

Applied strain rates used for each test were calculated as previously described in the section titled "Consolidated-Undrained Triaxial Test." Typically, these rates resulted in a time to failure (or 25 percent shear strain) of about 3.5 hours. The rate is higher than used for triaxial tests because the pore fluid equalization path is much shorter, being only 10 mm (0.4 in) as compared with about 76 mm (3.0 in) for the triaxial tests.

APPENDIX C

LABORATORY TEST RESULTS

APPENDIX ILLUSTRATIONS

Boring Number	Sample Number	Penetration Meter (Feet)	Classification Tests: Liquid Limit	Plastic Limit	Water Content %	Unit Wet Weight Mg/m3 (lb/cu ft)	Specific Gravity	Clay Content <2µ %	Consolidation Tests: σ_{vm} kPa (kips/sq ft)	C_c	Miniature Vane: Type of Test	Shear Strength kPa (kips/sq ft)	Compression Tests: Type of Test	Water Content % Initial	Water Content % Final	Unit Wet Weight Mg/m3 (lb/cu ft)	Shear Strength kPa (kips/sq ft)	Axial Strain at Failure %	Type of Failure	Normalized Strength Tests: Type of Test	Water Content % Initial	Water Content % Final	OCR	Shear Strength kPa (kips/sq ft)	Strain at Failure %	Type of Failure
											BORING 4															
4	S-3	1.68	57	22									2-U	46.7		1.63	2.63	21.3								
		(5.5)	57	22									2-U	46.7		(102)	(0.06)	21.3								
											BORING 5															
5	S-1	0.15	45	17									2-U	77.2		1.70	2.43	22.2								
		(0.5)	45	17									2-U	77.2		(106)	(0.05)	22.2								
5	S-5	3.20	55	20									2-U	75.5		1.44	3.55	21.7								
		(10.5)	55	20									2-U	75.5		(90.0)	(0.07)	21.7								
5	S-7	4.73	47	17									2-U	79.4		1.51	4.71	22.2								
		(15.5)	47	17									2-U	79.4		(94.5)	(0.10)	22.2								
5	S-11	6.25	86	32									2-U	74.9		1.49	11.51	20.1								
		(20.5)	86	32									2-U	74.9		(93.0)	(0.24)	20.1								
5	S-13	6.55											1-U	65.1		1.55	8.11	12.6	A							
		(21.5)											1-U	65.1		(96.5)	(0.17)	12.6	A							
5	S-15	7.93	74	26									2-U	75.1		1.53	10.44	20.5								
		(26.0)	74	26									2-U	75.1		(95.0)	(0.22)	20.5								
5	S-20	12.35				1.54														4-U	73.0	55.6	1.0	31.80	12.8	
		(40.5)				(96.0)														4-U	73.0	55.6	1.0	(0.66)	12.8	
5	S-20	12.71				1.54														5-U	69.6	67.2	1.0	27.45	18.6	
		(41.7)				(96.0)														5-U	69.6	67.2	1.0	(0.57)	18.6	
	S-20	12.71				1.60														5-U	72.1	60.1	2.0	47.90	18.1	
		(41.7)				(100)														5-U	72.1	60.1	2.0	(1.0)	18.1	
	S-20	12.71				1.60														3-U	67.2	51.0	2.0	36.30	10.4	
		(41.7)				(100)														3-U	67.2	51.0	2.0	(0.76)	10.4	
5	S-37	18.29				1.54							1-U	70.1		1.56	14.61	8.30	C	3-U	69.6	50.6	1.0	51.65	9.5	
		(60.0)				(96.0)							1-U	70.1		(97.0)	(0.31)	8.30	C	3-U	69.6	50.6	1.0	(1.08)	9.5	
5	S-48	24.39	49	19							U	12.45	1-U	39.6		1.78	15.69	13.3	A							
		(80.0)	49	19							U	(0.26)	1-U	39.6		(111)	(0.33)	13.3	A							
5	S-48	24.39											2-U	41.0		1.78	23.45	20.0								
		(80.0)											2-U	41.0		(111)	(0.49)	20.0								
5	S-61	32.01				1.70														3-U	45.8	33.4	1.0	35.0	16.6	
		(105.0)				(106)														3-U	45.8	33.4	1.0	(0.73)	16.6	
5	S-76	39.02	45	21		1.76					U	19.64								3-U	41.3	25.7	2.0	163.60	12.6	
		(128.0)	45	21		(110)					U	(0.41)								3-U	41.3	25.7	2.0	(3.42)	12.6	
5	S-76	39.02				1.84							2-U	35.9		1.82	29.0	20.0		5-U	36.5	30.8	1.0	58.15	10.6	
		(128.0)				(115)							2-U	35.9		(114)	(0.61)	20.0		5-U	36.5	30.8	1.0	(1.21)	10.6	
5	S-92	48.72	65	22	48.8	1.73	2.75	0.0016	119.8	0.53			1-U	43.5		1.72	43.73	10.0	B							
		(160.0)	65	22	48.8	(108)	2.75	0.0016	(2.50)	0.53			1-U	43.5		(107)	(0.91)	10.0	B							
5	S-92	48.72											2-U	52.7		1.75	53.91	13.3								
		(160.0)											2-U	52.7		(109)	(1.13)	13.3								
5	S-106	54.88	76	28							U	24.43	1-U	49.8		1.61	21.51	15.0	C							
		(180.0)	76	28							U	(0.51)	1-U	49.8		(100)	(0.45)	15.0	C							
5	S-106	54.88											2-U	51.4		1.63	34.34	20.0								
		(180.0)											2-U	51.4		(102)	(0.72)	20.0								
5	S-123	64.02	87	25	54.3	1.71	2.82	0.001	167.7	0.65			2-U	54.9		1.72	65.08	5.0								
		(210.0)	87	25	54.3	(107)	2.82	0.001	(3.5)	0.65			2-U	54.9		(107.5)	(1.36)	5.0								
											BORING 6															
6	S-5	12.2	79	27							U	10.1	2-U	56.5		1.57	15.44	20.0								
		(40.0)	79	27							U	(0.21)	2-U	56.5		(98.0)	(0.32)	20.0								
6	S-10	18.29	61	23	51.8	1.70	2.72	0.0045	47.9	0.59			2-U	51.8		1.69	23.17	15.0								
		(60.0)	61	23	51.8	(106)	2.72	0.0045	(1.00)	0.59			2-U	51.8		(106)	(0.48)	15.0								
6	S-26	24.39				1.77														4-U	39.2	32.0	1.0	88.05	7.2	
		(80.0)				(110.5)														4-U	39.2	32.0	1.0	(1.84)	7.2	
6	S-26	31.40	49	20	42.1	1.68 1.83	2.73	0.009	34.5	0.36			2-U	40.9		1.80	27.68	20.0		5-U	39.0	26.8	2.0	104.84	12.0	
		(103.0)	49	20	42.1	(105) (114)	2.73	0.009	(0.72)	0.36			2-U	40.9		(113)	(0.58)	20.0		5-U	39.0	26.8	2.0	(2.19)	12.0	

Continued on Plate C-1b

LEGEND AND NOTES

TYPE OF TEST

1 UNCONFINED COMPRESSION
2 UNCONSOLIDATED UNDRAINED TRIAXIAL
3 ISTROPICALLY CONSOLIDATED UNDRAINED
4 KO-CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION
5 KO-CONSOLIDATED UNDRAINED DIRECT SIMPLE SHEAR

U = UNDISTURBED
R = REMOLDED
S = STRESS-STRAIN CURVE PRESENTED SEPARATELY
T = RESULTS PRESENTED SEPARATELY

TYPE OF FAILURE

A = BULGE
B = SINGLE SHEAR PLANE
C = MULTIPLE SHEAR PLANE
D = VERTICAL FRACTURE

BORINGS 4, 5 AND 6, BLOCK 58
WEST DELTA AREA

SEAFLOOR AT ELEVATION -53 FEET

SUMMARY OF TEST RESULTS

Ertec
The Earth Technology Corporation

BORING NUMBER	SAMPLE NUMBER	PENETRATION METER (FEET)	CLASSIFICATION TESTS: LIQUID LIMIT	PLASTIC LIMIT	WATER CONTENT %	UNIT WET WEIGHT Mg/M3 (LB/CU FT)	SPECIFIC GRAVITY	CLAY CONTENT -2M %	CONSOLIDATION TESTS: KPa (KIPS/SQ FT)	C_c	MINIATURE VANE: TYPE OF TEST	SHEAR STRENGTH KPa (KIPS/SQ FT)	COMPRESSION TESTS: TYPE OF TEST	WATER CONTENT % INITIAL	WATER CONTENT % FINAL	UNIT WET WEIGHT Mg/M3 (LB/CU FT)	SHEAR STRENGTH KPa (KIPS/SQ FT)	AXIAL STRAIN AT FAILURE %	TYPE OF FAILURE	NORMALIZED STRENGTH TESTS: TYPE OF TEST	WATER CONTENT % INITIAL	WATER CONTENT % FINAL	OCR	SHEAR STRENGTH KPa (KIPS/SQ FT)	STRAIN AT FAILURE %	TYPE OF FAILURE
									Continued from Plate B-1A																	
6	S-45	48.78				1.67														3-U	52.9	41.4	1.0	105.1[illegible]	7.5	
		(160.0)				(106)														3-U	52.9	41.4	1.0	(2.20)	7.5	
6	S-53	54.88				1.63														4-U	56.4	43.0	1.0	128.00	6.2	
		(180.0)				(101.5														4-U	56.4	43.0	1.0	(2.65)	6.2	
6	S-62	64.02				1.70														3-U	49.3	43.4	1.0	133.05	10.0	
		(210)				(106)														3-U	49.3	43.4	1.0	(2.78)	10.0	
6	S-62	64.02				1.71														3-U	54.2	39.2	2.0	182.75	10.2	
		(210)				(107)														3-U	54.2	39.2	2.0	(3.82)	10.2	
6	S-62	64.02				1.70														5-U	54.9	45.3	1.0	114.16	15.6	
		(210)				(106)														5-U	54.9	45.3	1.0	(2.38)	15.6	
6	S-70	73.2	91	28		1.68					U	91.49	1-U	49.0		1.72	74.49	3.3	B	5-U	54.1	41.9	2.0	181.00	19.0	
		(240.0)	91	28		(105)					U	(1.91)	1-U	49.0		(107.5)	(1.55)	3.3	B	5-U	54.1	41.9	2.0	(3.78)	19.0	
6	S-70	73.2											2-U	53.6		1.71	69.83	7.5								
		(240.0)											2-U	53.6		(107)	(1.46)	7.5								

LEGEND AND NOTES

TYPE OF TEST

1. UNCONFINED COMPRESSION
2. UNCONSOLIDATED-UNDRAINED TRIAXIAL
3. ISTROPICALLY CONSOLIDATED UNDRAINED
4. KO-CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION
5. KO-CONSOLIDATED UNDRAINED DIRECT SIMPLE SHEAR

U = UNDISTURBED
R = REMOLDED
S = STRESS-STRAIN CURVE PRESENTED SEPARATELY
T = RESULTS PRESENTED SEPARATELY

TYPE OF FAILURE

A = BULGE
B = SINGLE SHEAR PLANE
C = MULTIPLE SHEAR PLANE
D = VERTICAL FRACTURE

BORINGS 4, 5 AND 6, BLOCK 58

WEST DELTA AREA

SEAFLOOR AT ELEVATION -53 FEET

SUMMARY OF TEST RESULTS

Ertec

The Earth Technology Corporation

PLATE C-1B

TABULATION OF TESTS PERFORMED BY ERTEC

CLASSIFICATION TESTS

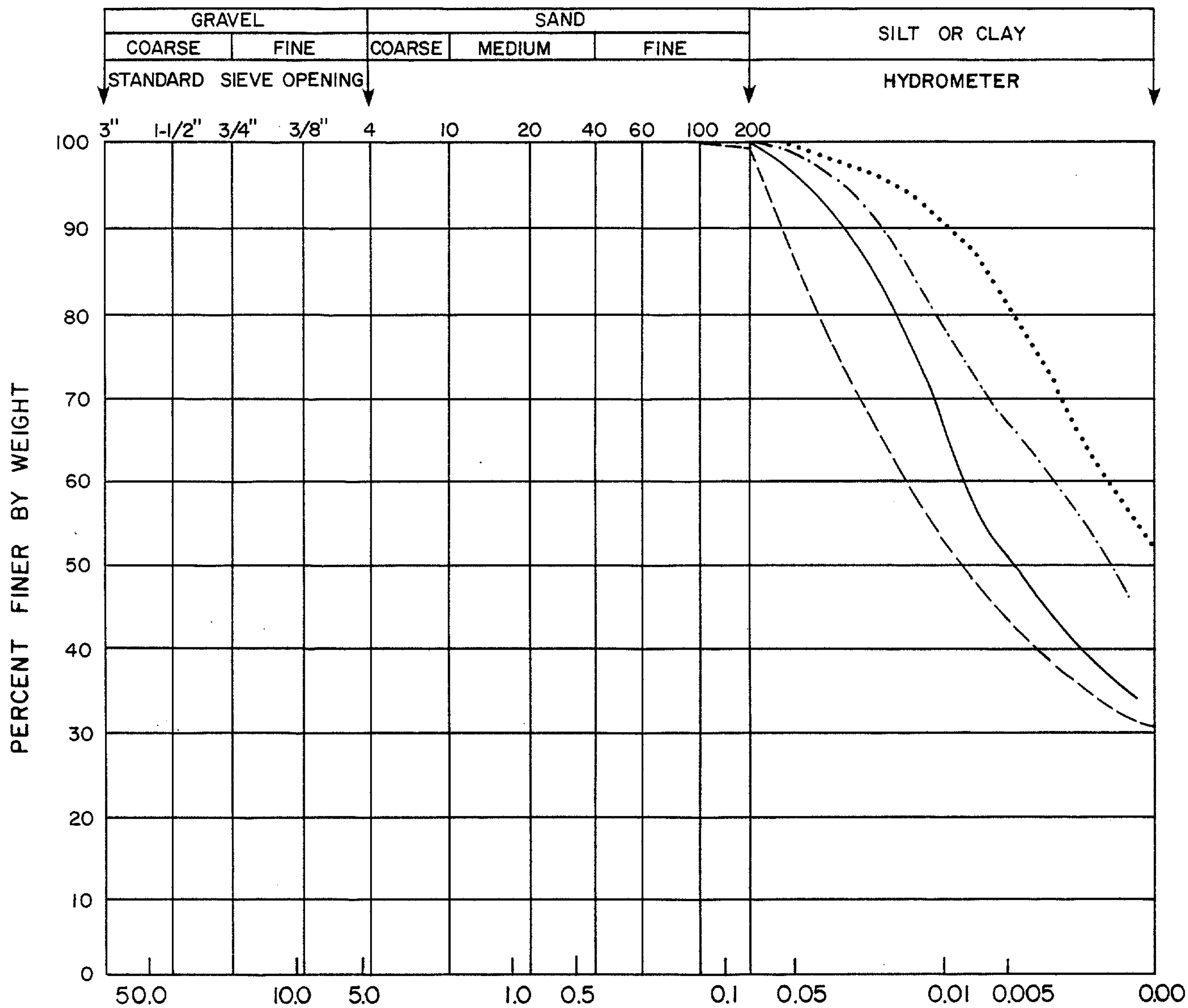
Type of Test	No. of Test
Liquid and Plastic Limits	15
Specific Gravity	4
Natural Moisture Content	21
Unit Weight	21
Hydrometer	4

PHYSICAL PROPERTY TESTS

Type of Test	No. of Test
One Dimensional Consolidation	4
K_o Consolidation	3

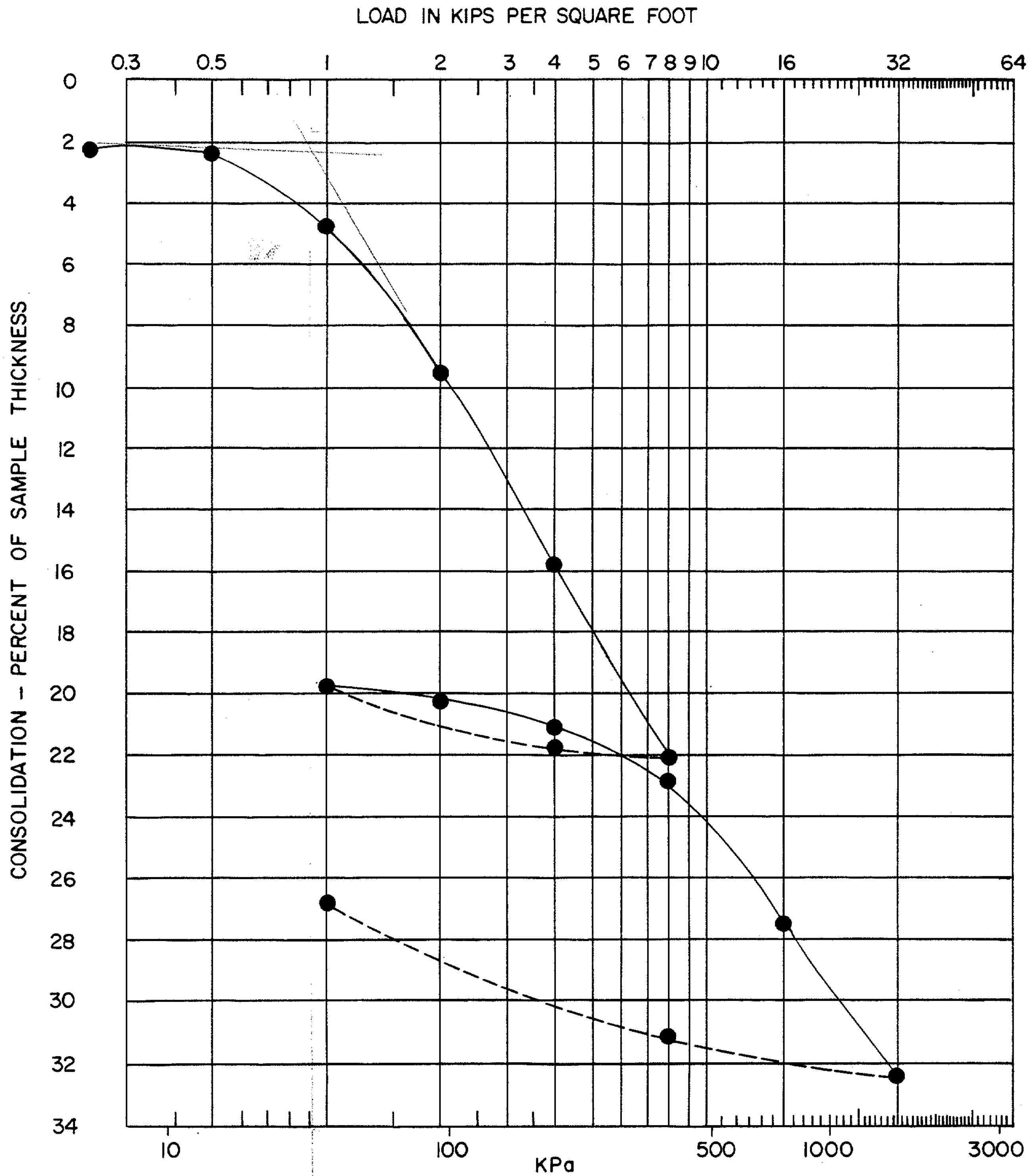
STRENGTH TESTS

Type of Test	No. of Test
Miniature Vane Shear Tests	5
Unconfined Compression, UC	6
Unconsolidated Undrained Triaxial, UU	15
Isotropically Consolidated Undrained Triaxial Compression, $\overline{\text{CIUC}}$	7
K_o-Consolidated Undrained Triaxial Compression $\overline{\text{CK}_o\text{UC}}$	3
K_o-Consolidated Undrained Direct Simple Shear, $\overline{\text{CK}_o\text{U}}$DSS	6



SYMBOL	BORING NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	SOIL TYPE
————	6	10	60.8 – 61.1	CH
— — — —	6	26	103.6 – 104.0	CL
—·—·—	5	92	161.0 – 163.3	CH
·········	5	123	210.2 – 210.5	CH

GRAIN SIZE DISTIRIBUTION CURVES



BORING NO.: B-6

SAMPLE NO.: 10

DEPTH: 18.5-18.63 (60.8-61.1 FT.)

e_o = 1.518

γ_d = 1.085 Mg/m^3 (67.7 PCF)

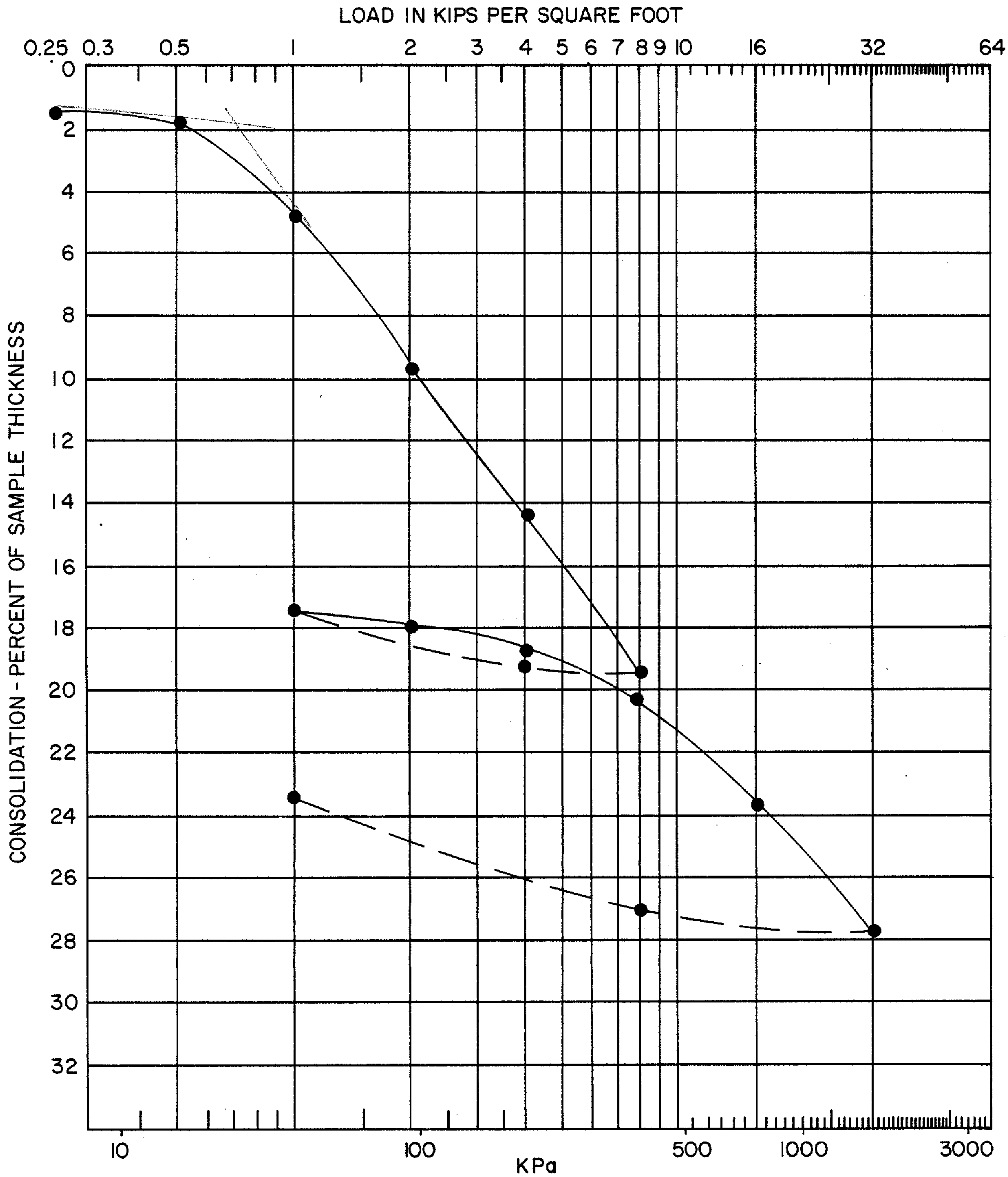
W/C = 50.2 % (INITIAL)

Pc = 47.9 KPa (1.0 KSF)

INCREMENTAL CONSOLIDATION TEST RESULTS

Ertec
The Earth Technology Corporation

PLATE C-4



BORING NO : B-6

SAMPLE NO : 26

DEPTH : 31.6 – 31.8 M (103.6 – 104.4 FT.)

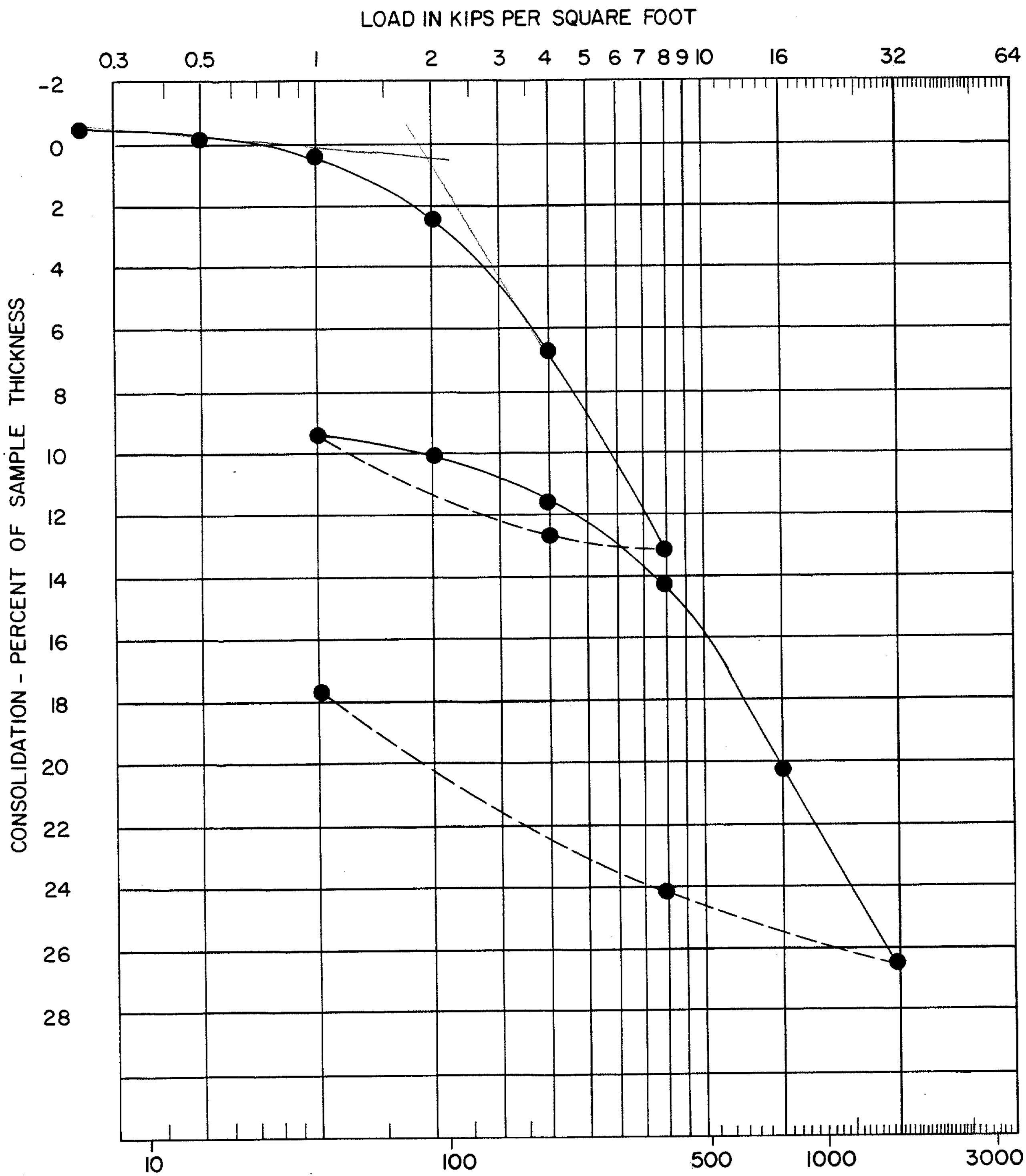
e_0 = 1.336

γ_d = 1.18 Mg/m³ (73.7 PCF)

W/C = 42.1 % (INITIAL)

Pc = 34.5 KPa (0.72 KSF)

INCREMENTAL CONSOLIDATION TEST RESULTS



BORING NO.: B-5

SAMPLE NO.: 92

DEPTH: (49.1 – 49.2 M) 161.0 – 161.3 FT.

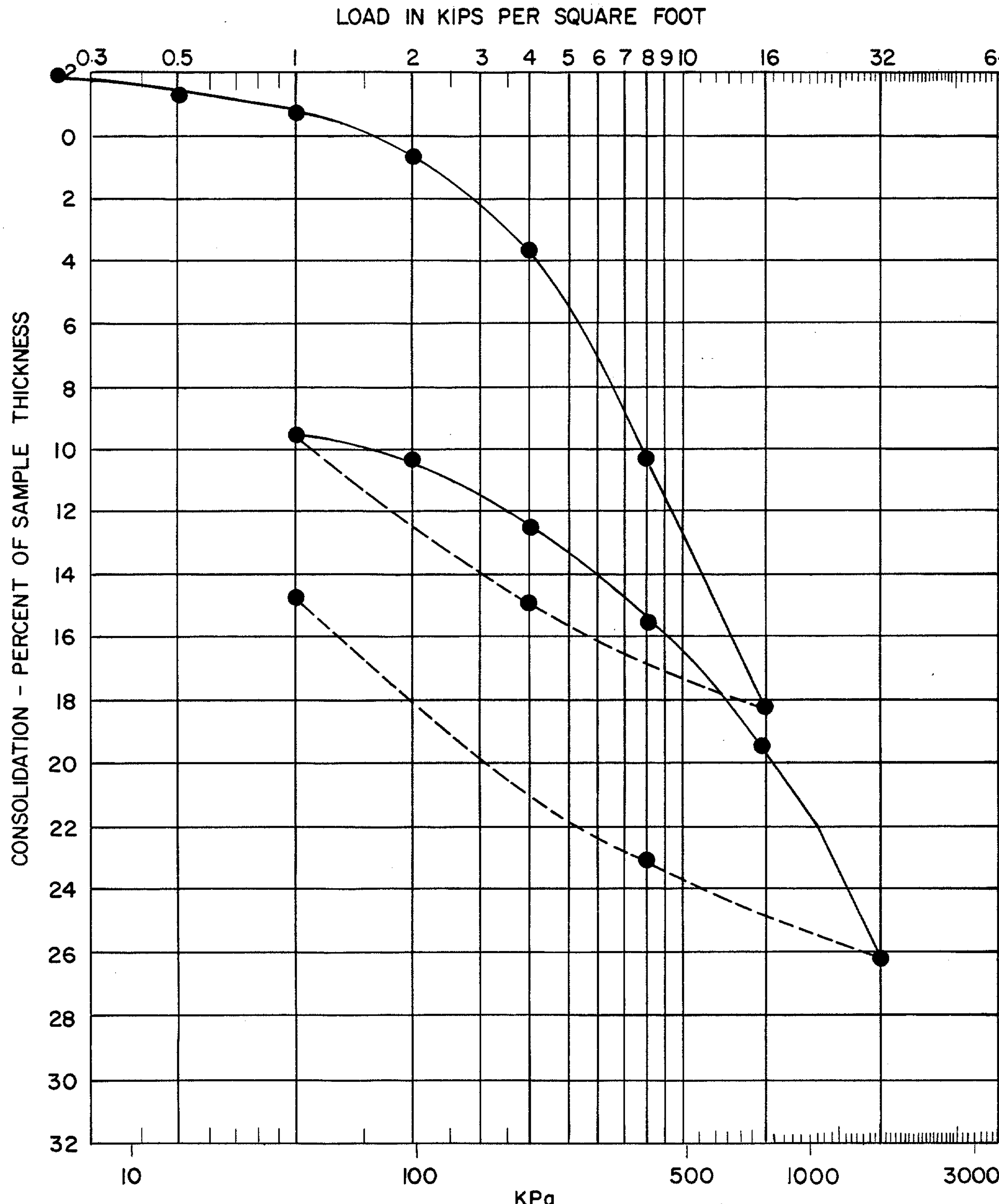
e_o = 1.447

γ_d = 1.16 Mg/m³ (72.1 PCF)

W/C = 48.8 % (INITIAL)

Pc = (119.8 KPa) 2.5 KSF

INCREMENTAL CONSOLIDATION TEST RESULTS



BORING NO : B-5

SAMPLE NO : 123

DEPTH : 64.1-64.2 M (210.2-210.5 FT.)

e_o = 1.540

γ_d = 1.11 Mg/m³ (69.5 PCF)

W/C = 54.3 % (INITIAL)

Pc = 167.7 KPa (3.5 KSF)

INCREMENTAL CONSOLIDATION TEST RESULTS

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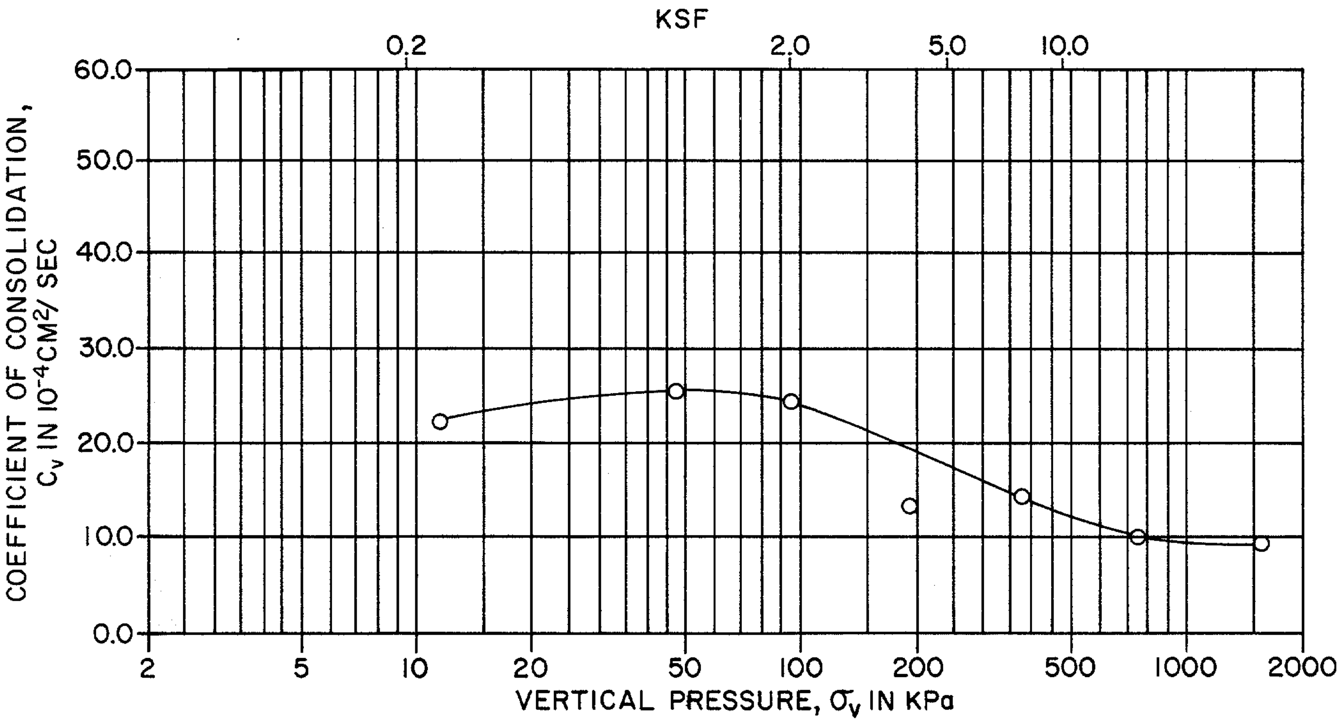
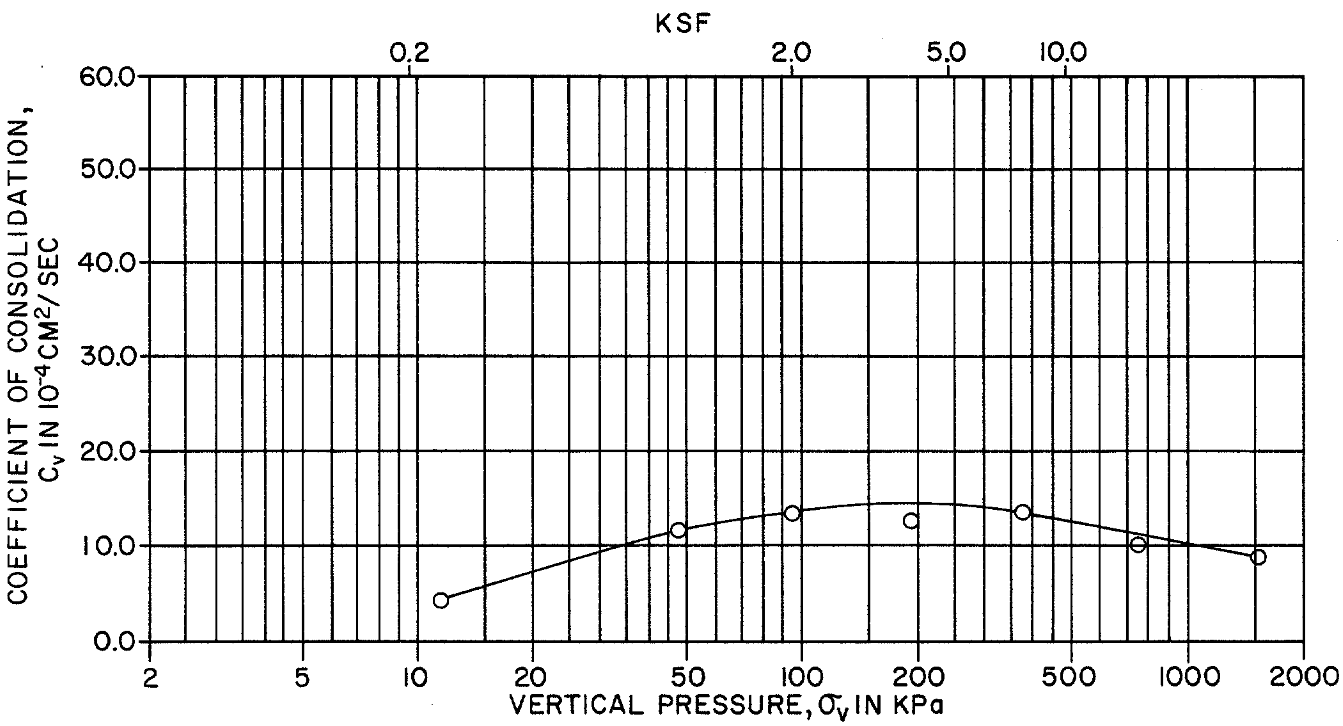


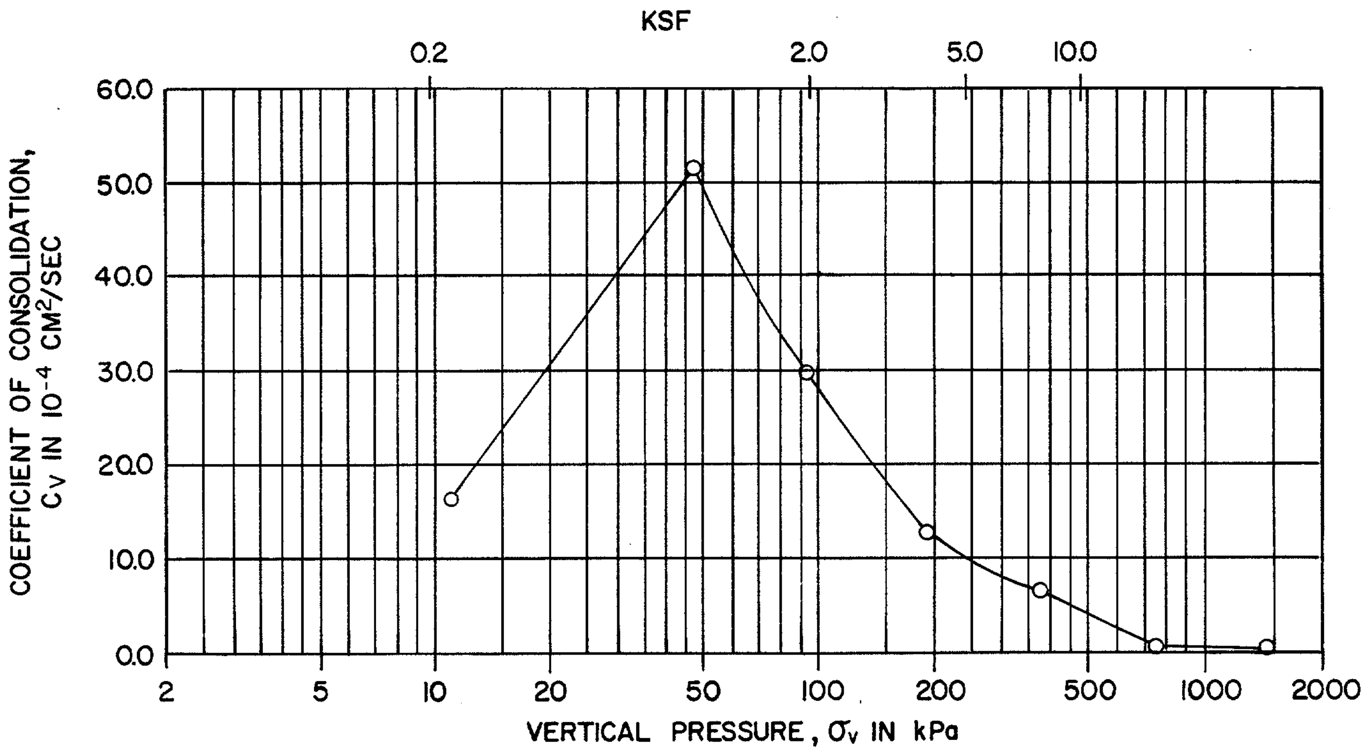
BORING : B-6 SAMPLE NO : 10 DEPTH : 18.5-18.6 M (60.8-61.1 FT) Pc = 47.9 KPa (1.0 KSF)



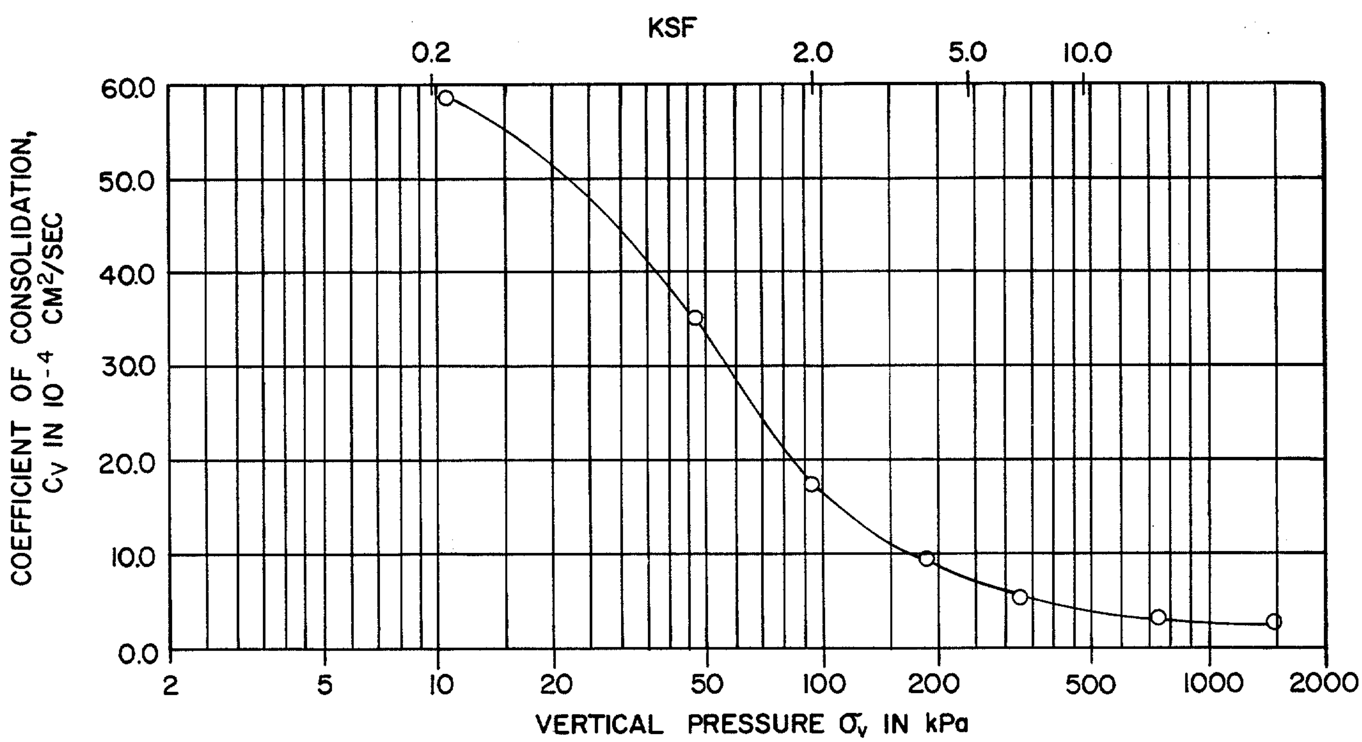
BORING : B-6 SAMPLE NO : 26 DEPTH : 31.6-31.8 M (103.6-104.4 FT) Pc = 34.5 KPa (0.72 KSF)

COEFFICIENT OF CONSOLIDATION
VERSUS
VERTICAL PRESSURE

Ertec
The Earth Technology Corporation



BORING NO.: B-5 SAMPLE NO.: 92 DEPTH: 161.0 - 161.3 FT. 49.1 - 49.2 M Pc = 119.8 KPa (2.5 KSF)

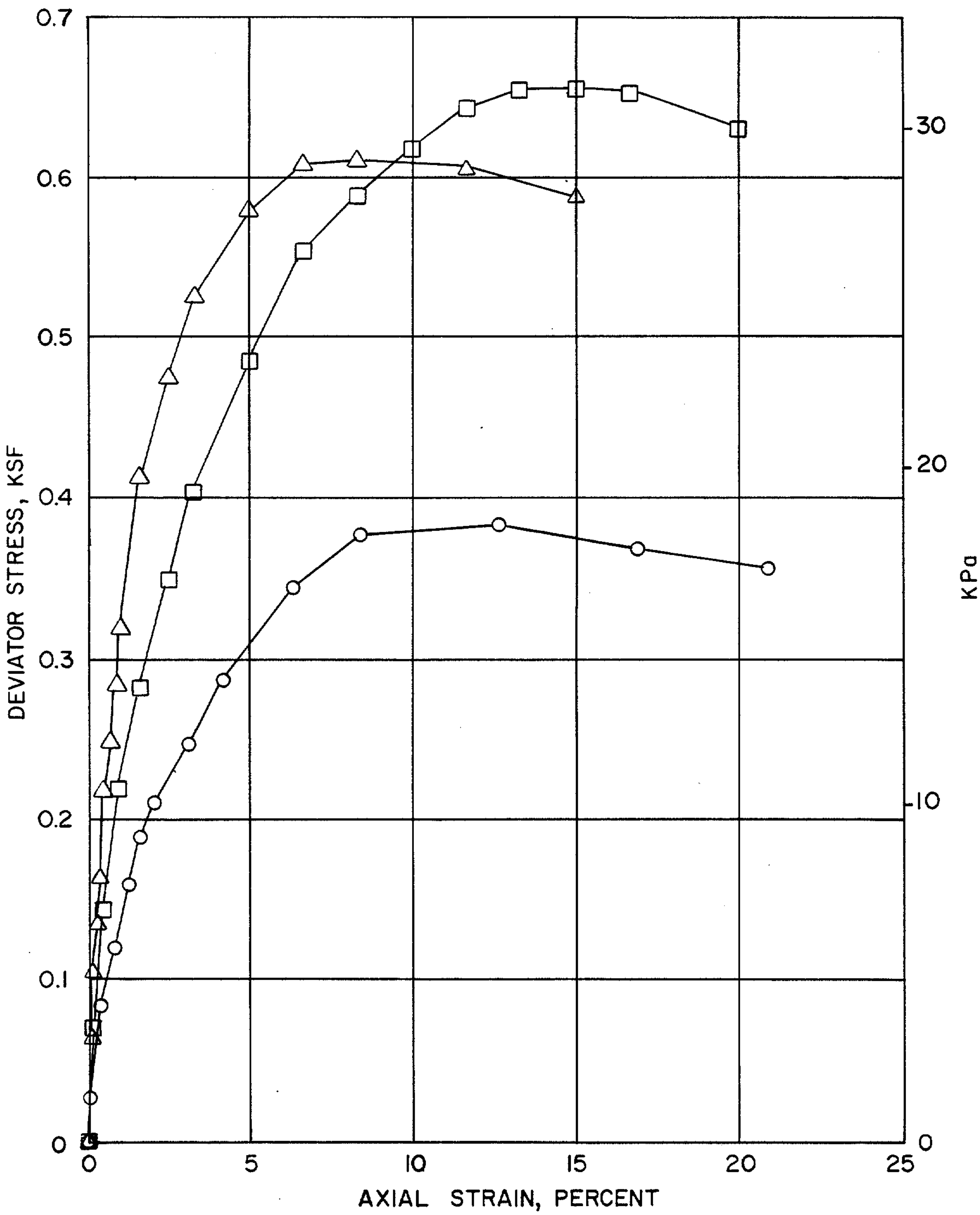


BORING NO.: B-5 SAMPLE NO.: 123 DEPTH: 210.2 - 210.5 FT 64.0 - 64.2 M Pc = 167.7 KPa (3.5 KSF)

COEFFICIENT OF CONSOLIDATION
VERSUS
VERTICAL PRESSURE

Ertec
The Earth Technology Corporation

PLATE C-9

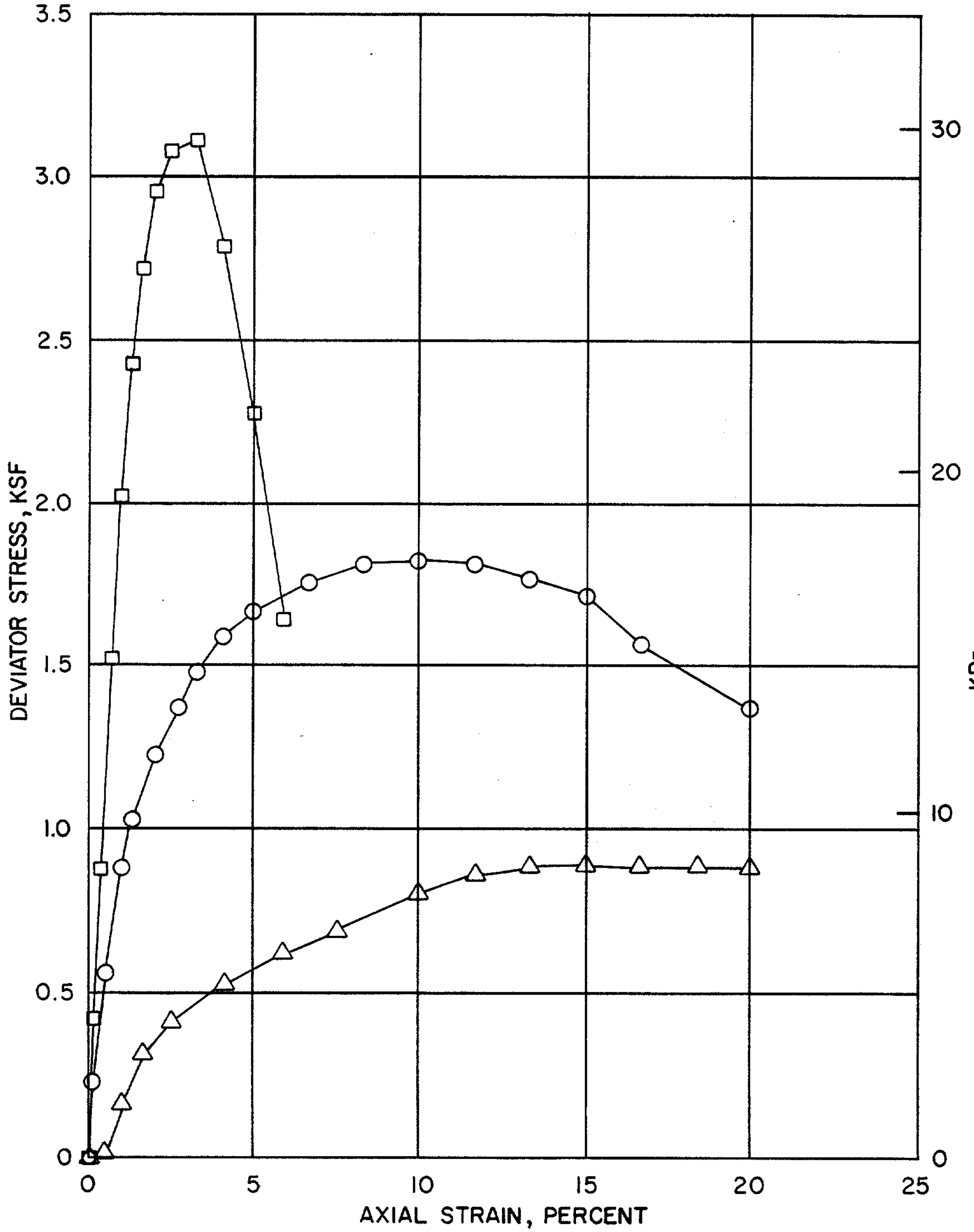


BORING No.	SAMPLE No.	DEPTH M(FT)	SYMBOL
5	13	6.55 (21.5)	○
5	37	18.72 (61.4)	△
5	48	24.6 (80.7)	□

STRESS - STRAIN CURVES
UNCONFINED COMPRESSION TESTS

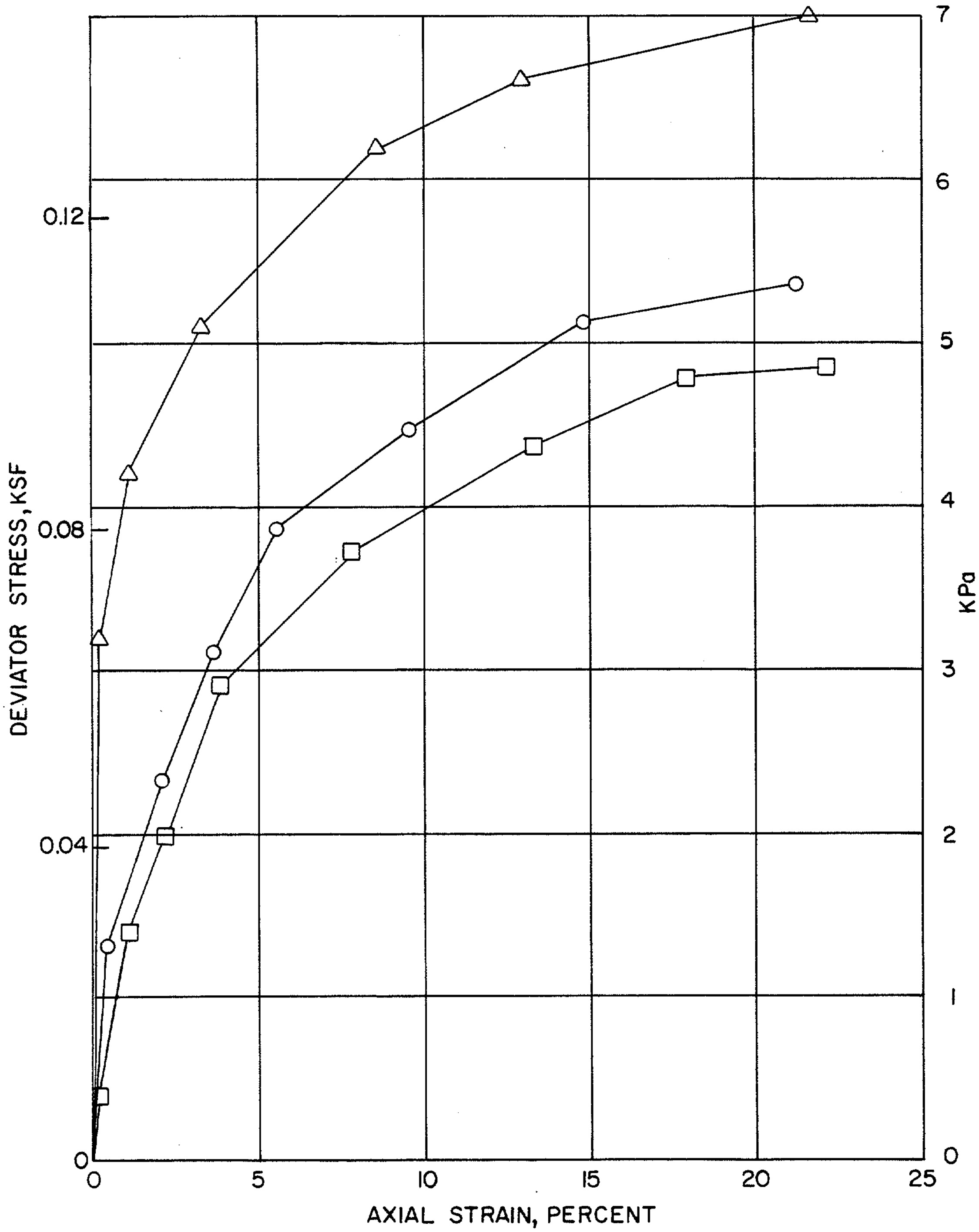
Ertec
The Earth Technology Corporation

PLATE C-10



BORING NO.	SAMPLE NO.	DEPTH M(FT)	SYMBOL
5	92	48.9 (160.4)	○
5	106	55.1 (180.7)	△
6	70	73.4 (240.6)	□

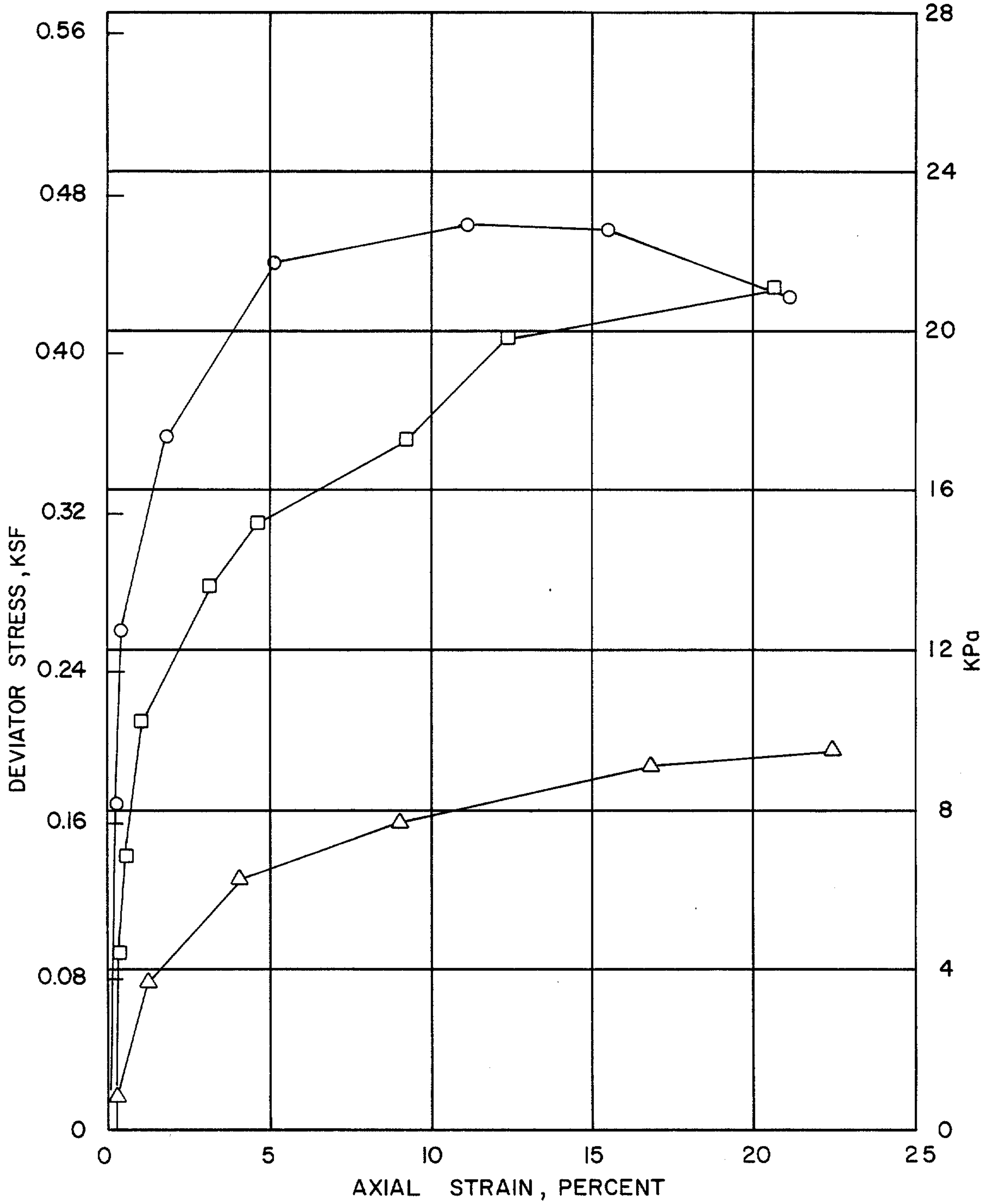
STRESS – STRAIN CURVES
UNCONFINED COMPRESSION TEST



BORING No.	SAMPLE No.	DEPTH M(FT)	CONFINING PRESSURE KPa	(KSF)	SYMBOL
5	5	3.05 (10.0)	24.10	(0.50)	△
4	3	1.68 (5.5)	13.90	(0.29)	○
5	1	0.15 (0.5)	3.40	(0.07)	□

STRESS-STRAIN CURVES
UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS

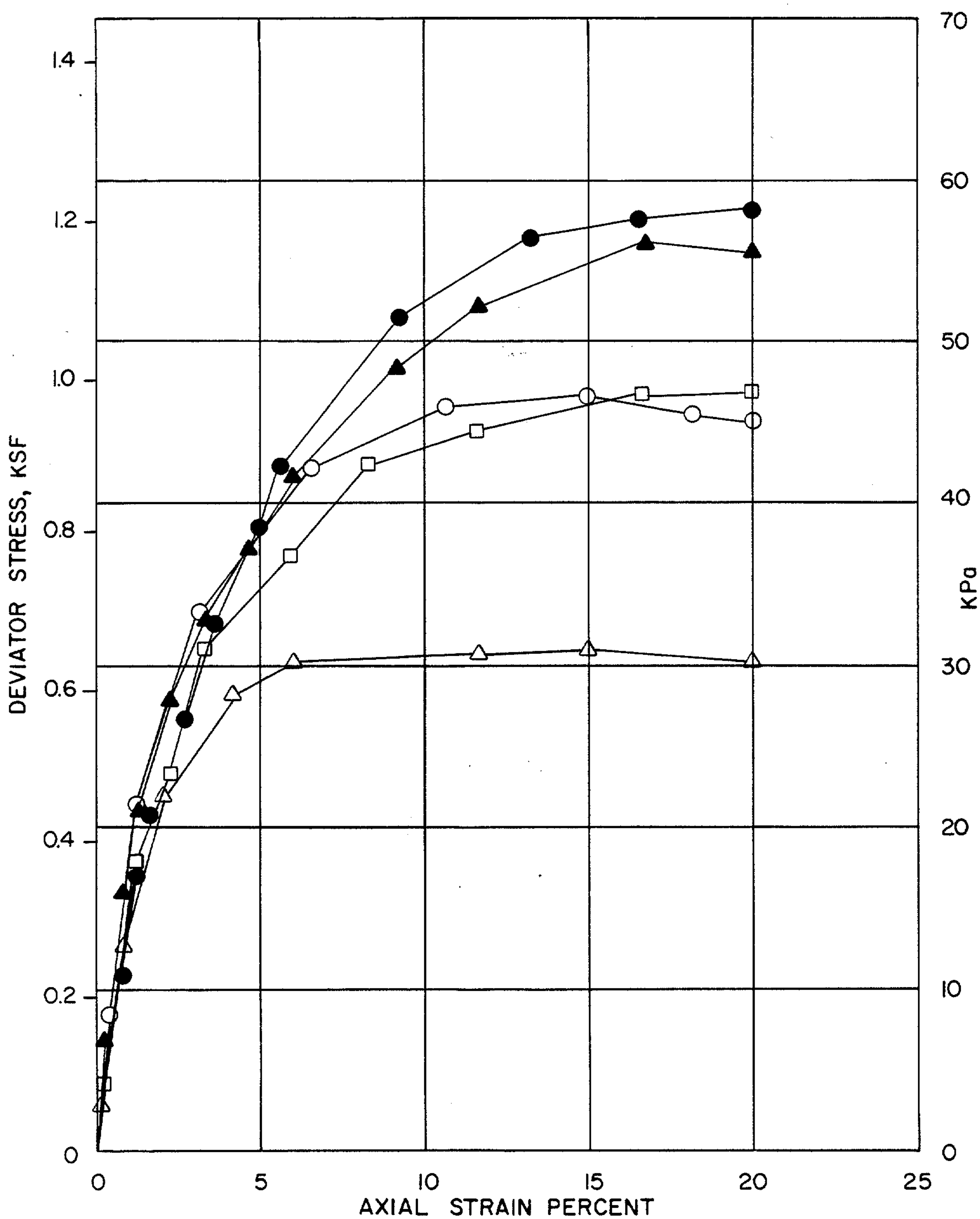
Ertec
The Earth Technology Corporation



BORING NO.	SAMPLE NO.	DEPTH M (FT)		CONFINING PRESSURE, KPa (KSF)		SYMBOL
5	7	4.57	(15.0)	34.5	(0.72)	△
5	11	6.25	(20.5)	44.8	(0.94)	○
5	15	7.93	(26.0)	55.2	(1.15)	□

STRESS-STRAIN CURVES

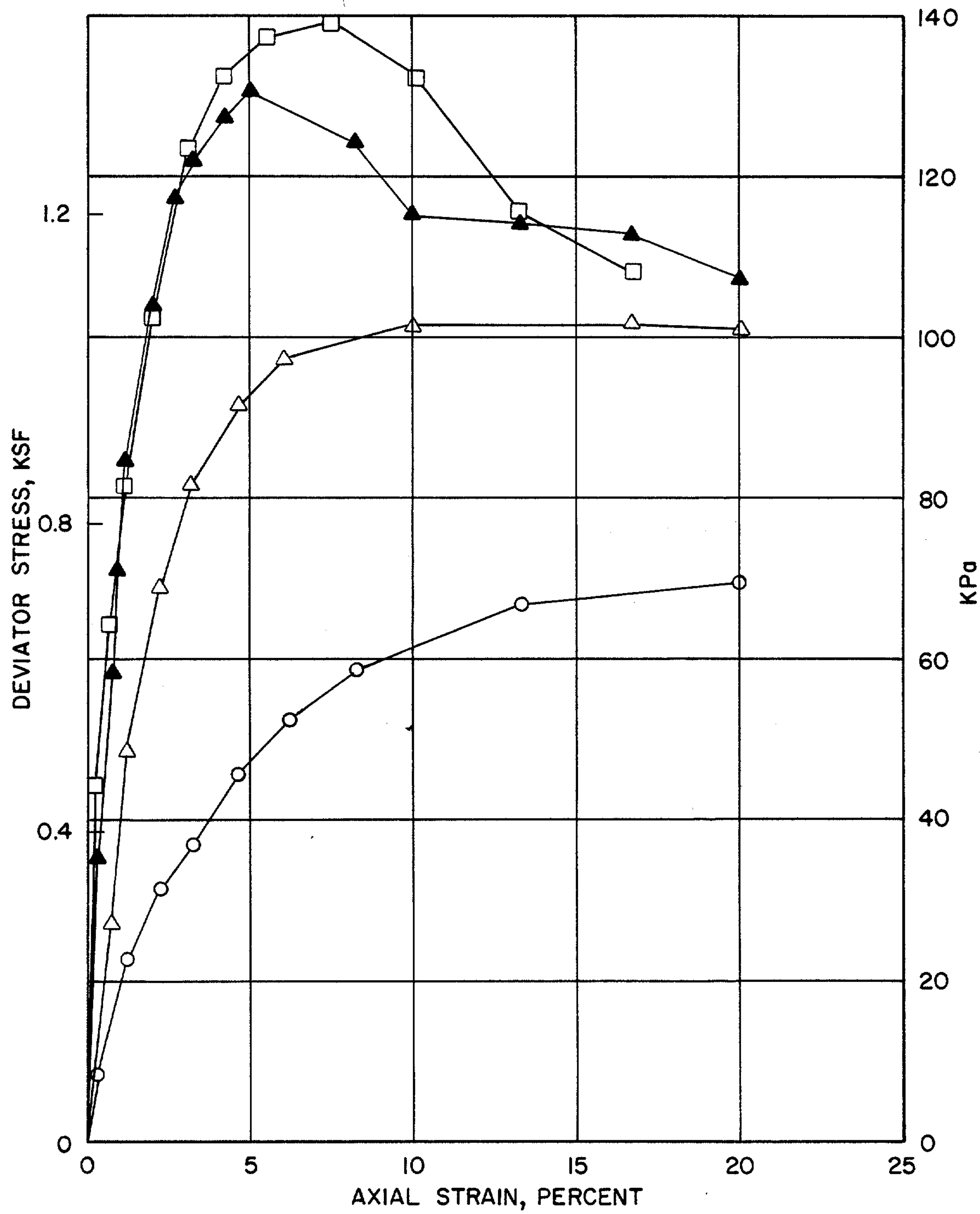
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS



BORING NO.	SAMPLE NO.	DEPTH M(FT)	CONFINING PRESSURE KPa	(KSF)	SYMBOL
6	5	12.35 (40.5)	89.6	1.87	△
6	10	18.29 (60.0)	137.9	2.88	○
5	48	24.82 (81.4)	179.3	54.66	□
6	26	31.40 (103.0)	241.3	5.04	▲
5	76	39.48 (129.5)	275.8	5.76	●

STRESS-STRAIN CURVES
UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS

Ertec
The Earth Technology Corporation

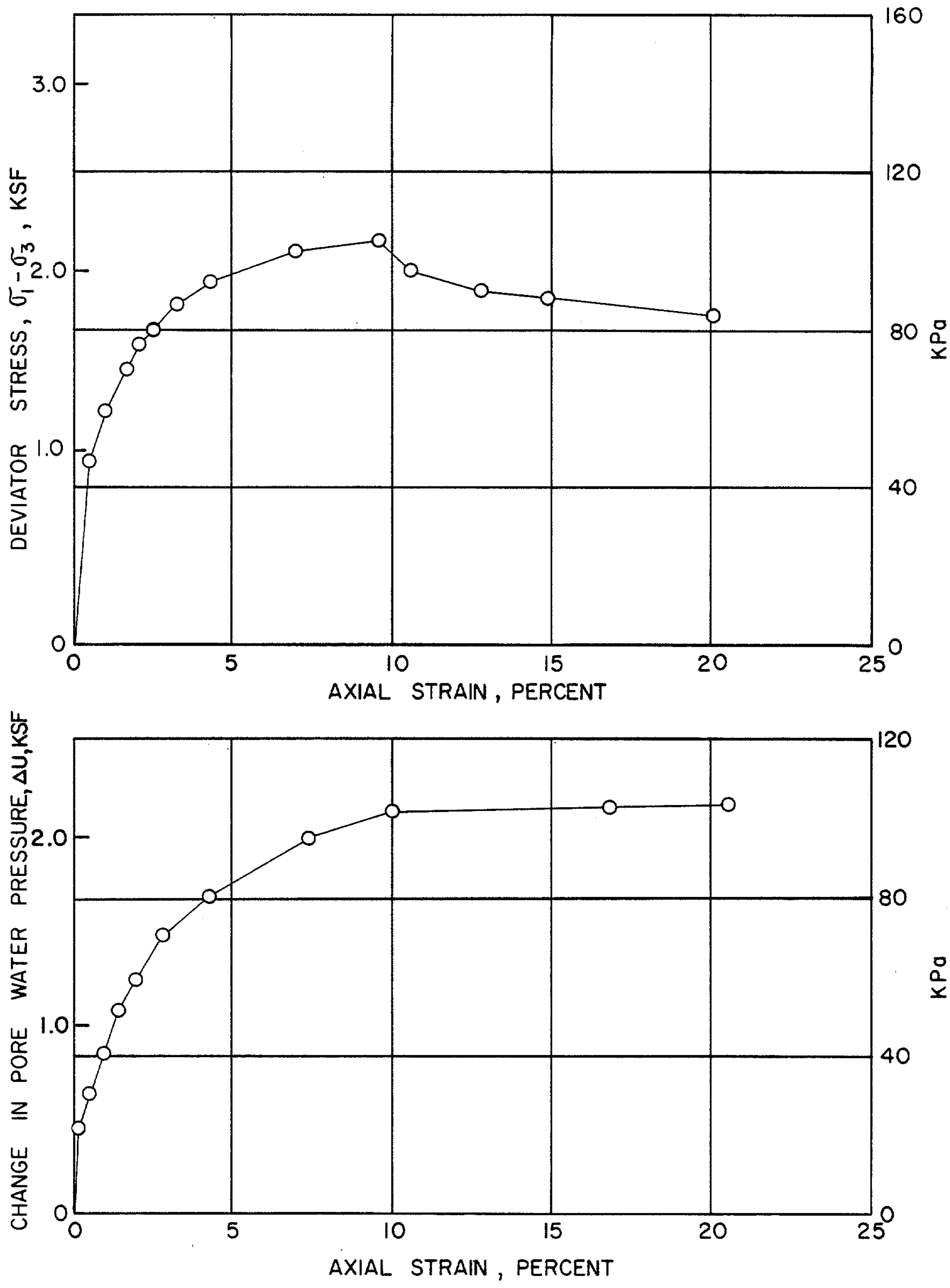


BORING NO.	SAMPLE NO.	DEPTH M(FT)	CONFINING PRESSURE KPa	(KSF)	SYMBOL
5	92	49.2 (161.4)	344.7	(7.20)	△
5	106	55.3 (181.4)	393.0	(8.20)	○
5	123	64.0 (210.0)	448.2	(9.36)	▲
6	70	73.6 (241.5)	524.0	(10.94)	□

STRESS - STRAIN CURVES

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS

Ertec
The Earth Technology Corporation



BORING NO: 5

SAMPLE NO: 37

DEPTH: 18.69-18.81 m (61.3-61.7 FT)

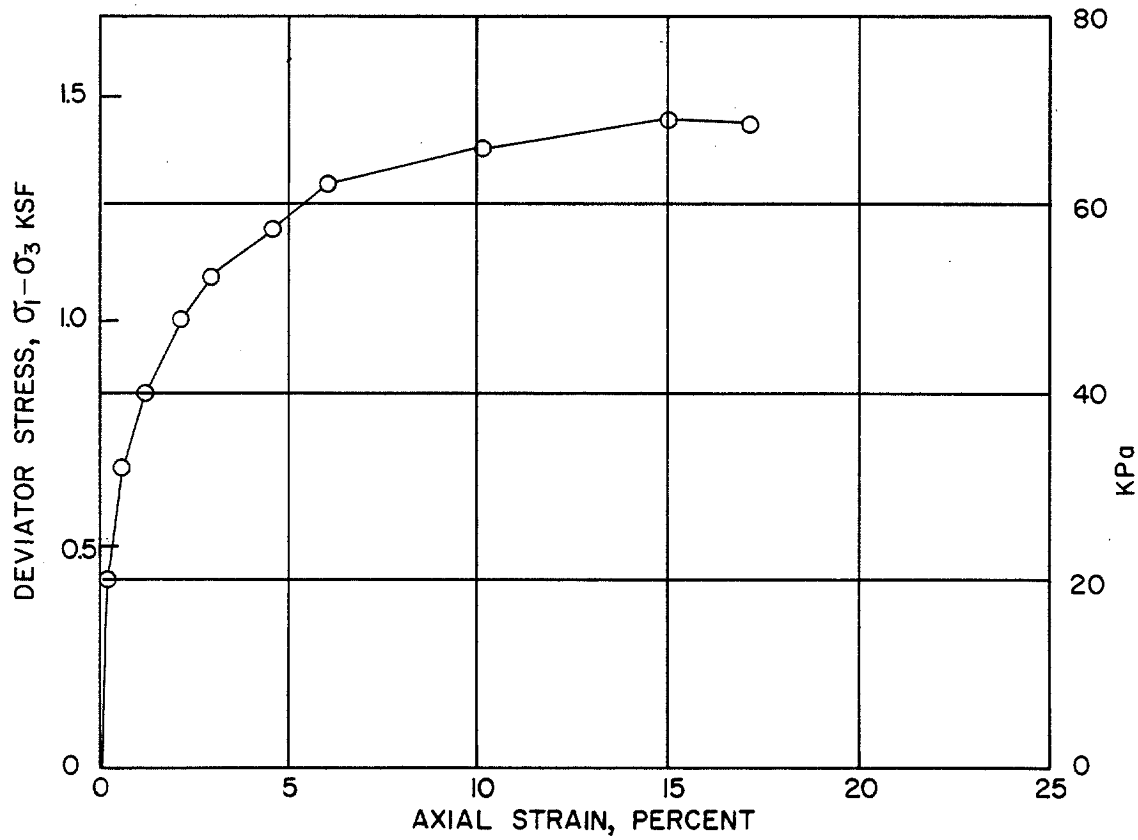
σ'_{3c} : 193.1 KPa (4.03 KSF)

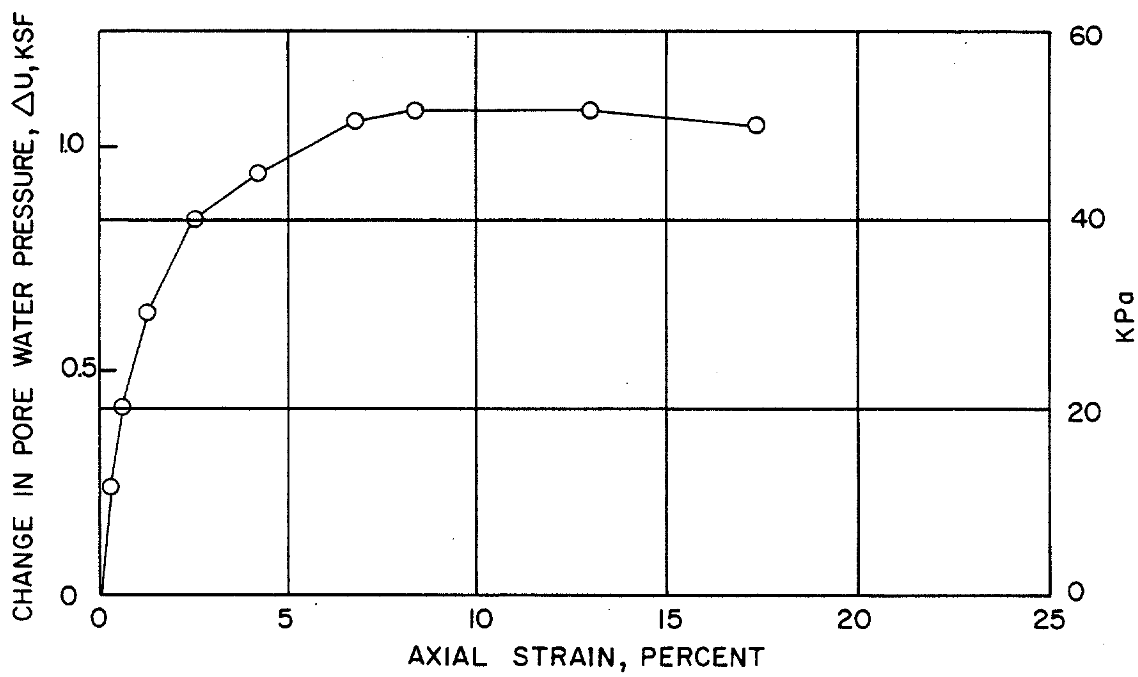
OCR : 1.0

DEVIATOR STRESS AND PORE WATER PRESSURE VERSUS AXIAL STRAIN

CI$\overline{U}$C TEST

Ertec
The Earth Technology Corporation

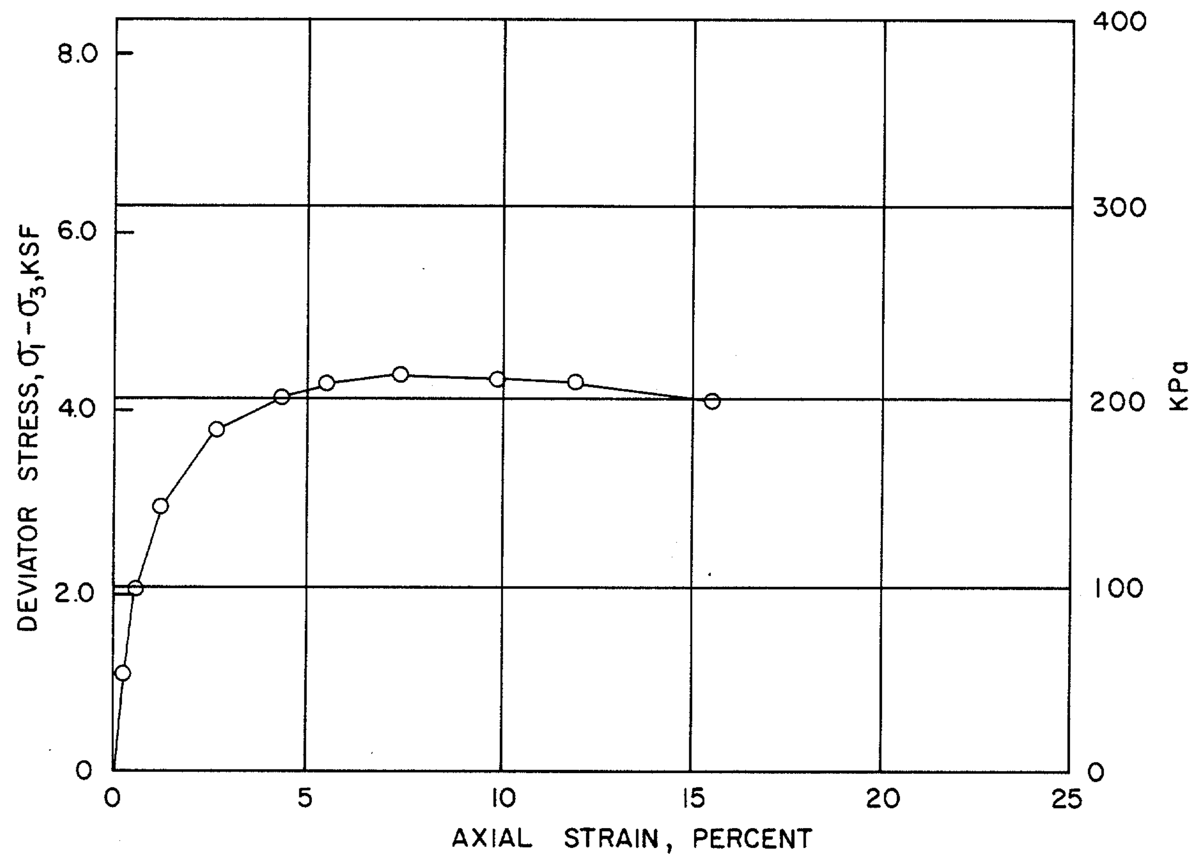


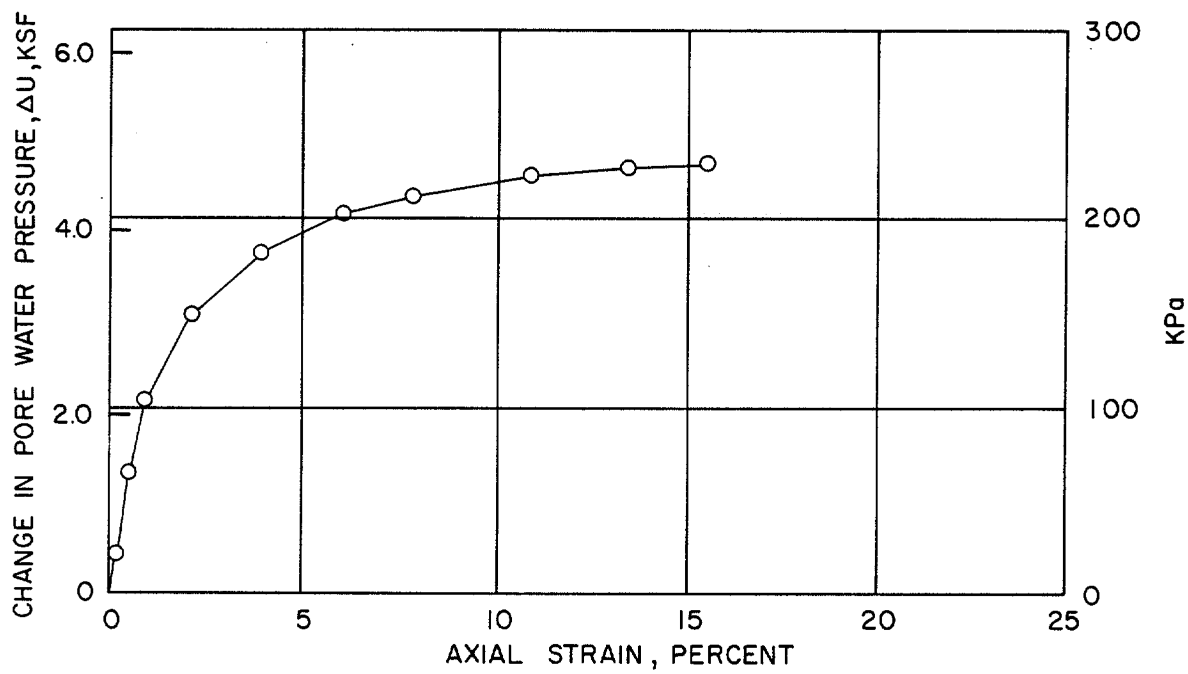


BORING NO:	5
SAMPLE NO:	61
DEPTH:	32.32-32.47 M (106.0-106.5 FT)
σ'_{3c}:	103.4 KPa (2.16 KSF)
OCR:	1.0

DEVIATOR STRESS AND PORE WATER PRESSURE
VERSUS AXIAL STRAIN
$\mathrm{C\overline{IU}C}$ TEST

Ertec
The Earth Technology Corporation





BORING NO: 6
SAMPLE NO: 45
DEPTH: 49.09-49.27 M (161.0-161.6 FT)
σ'_{3c}: 358.6 KPa (7.49 KSF)
OCR: 1.0

DEVIATOR STRESS AND PORE WATER PRESSURE
VERSUS AXIAL STRAIN
$C\overline{IU}C$ TESTS

Ertec
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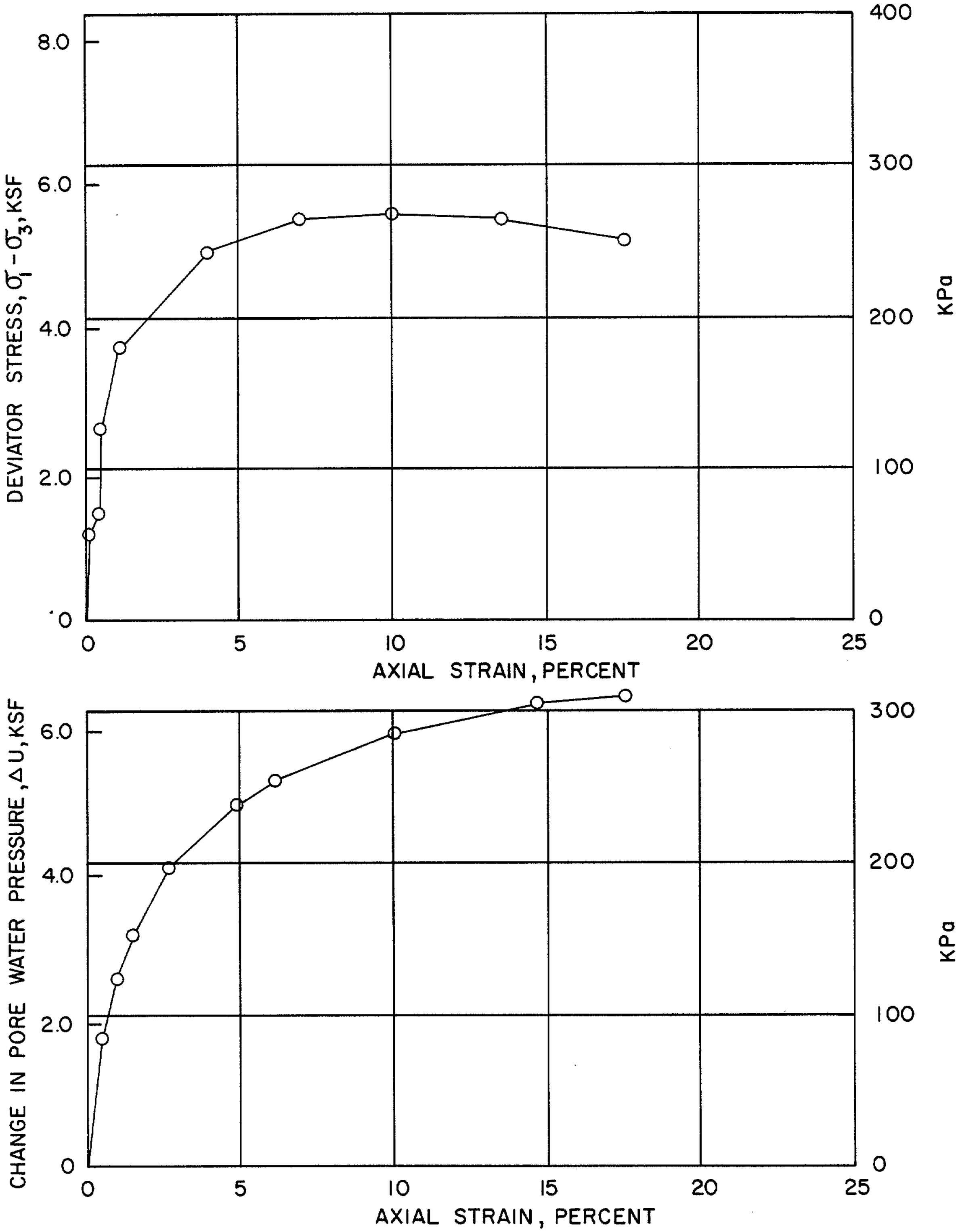
PLATE C-18



BORING NO: 6
SAMPLE NO: 62
DEPTH: 64.21-64.39 M (210.6-211.2 FT)
σ'_{3c}: 503.3 KPa (10.51 KSF)
OCR: 1.0

DEVIATOR STRESS AND PORE WATER PRESSURE VERSUS AXIAL STRAIN
$\overline{\text{CIUC}}$ TESTS

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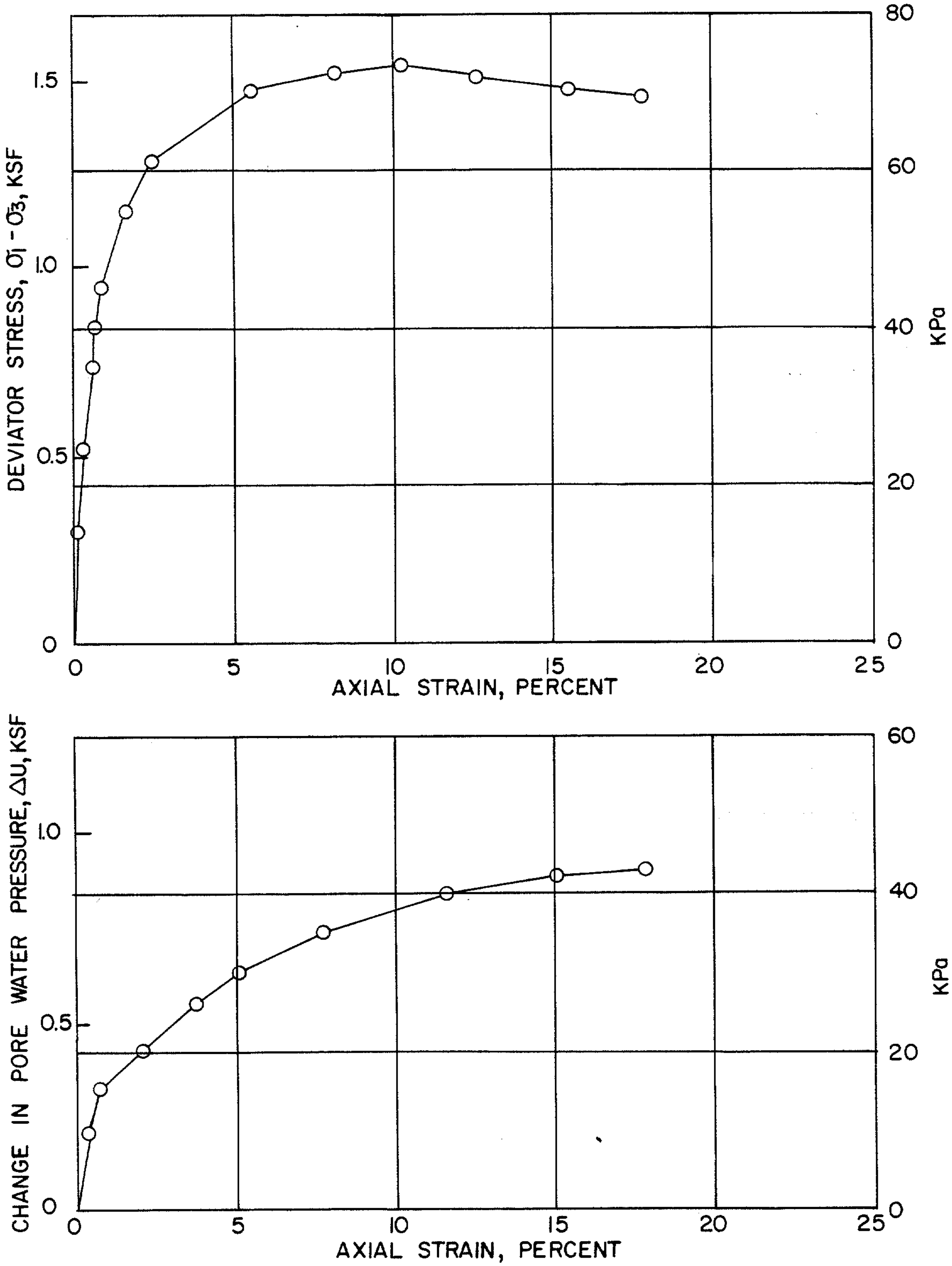

BORING NO:	5
SAMPLE NO:	20
DEPTH:	12.53-12.71 M (41.1-41.7 FT)
σ'_{3c}:	1034 KPa (2.16 KSF)
OCR:	2.0

DEVIATOR STRESS AND PORE WATER PRESSURE
VERSUS AXIAL STRAIN
$C\overline{IU}C$ TESTS

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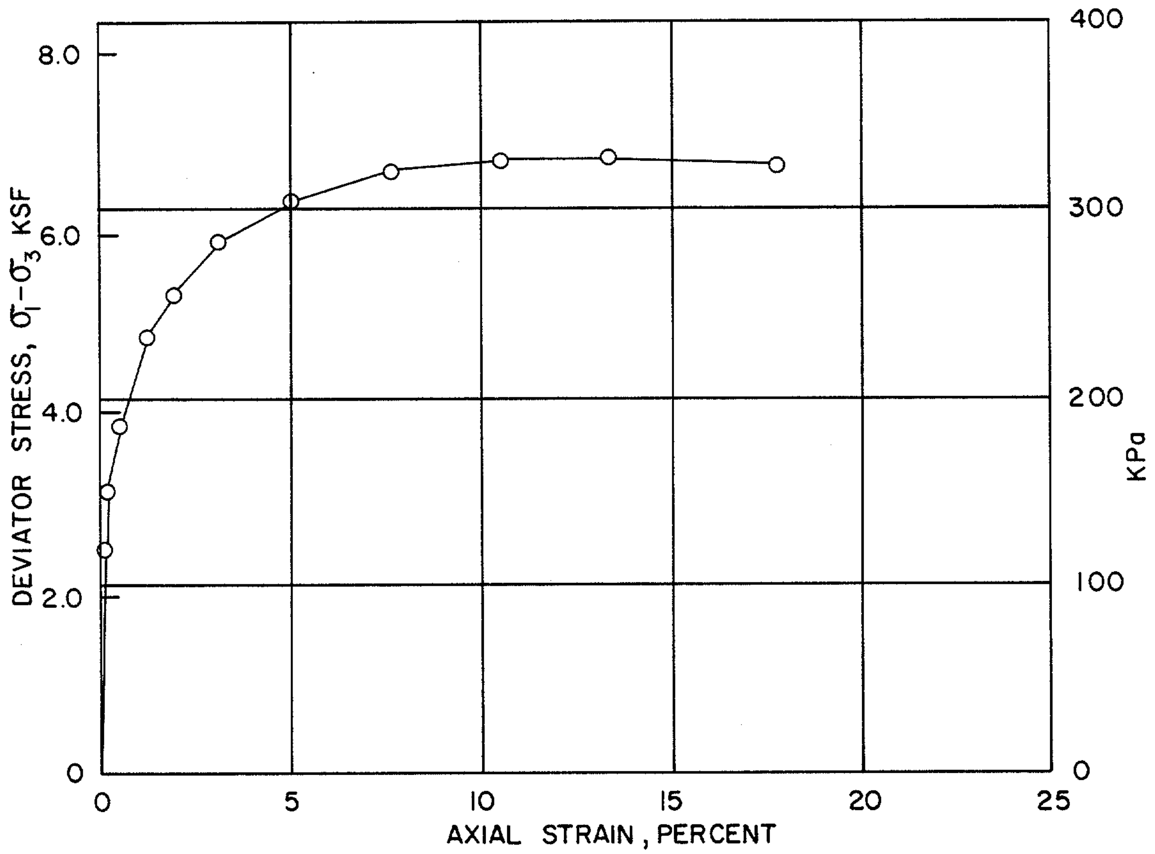

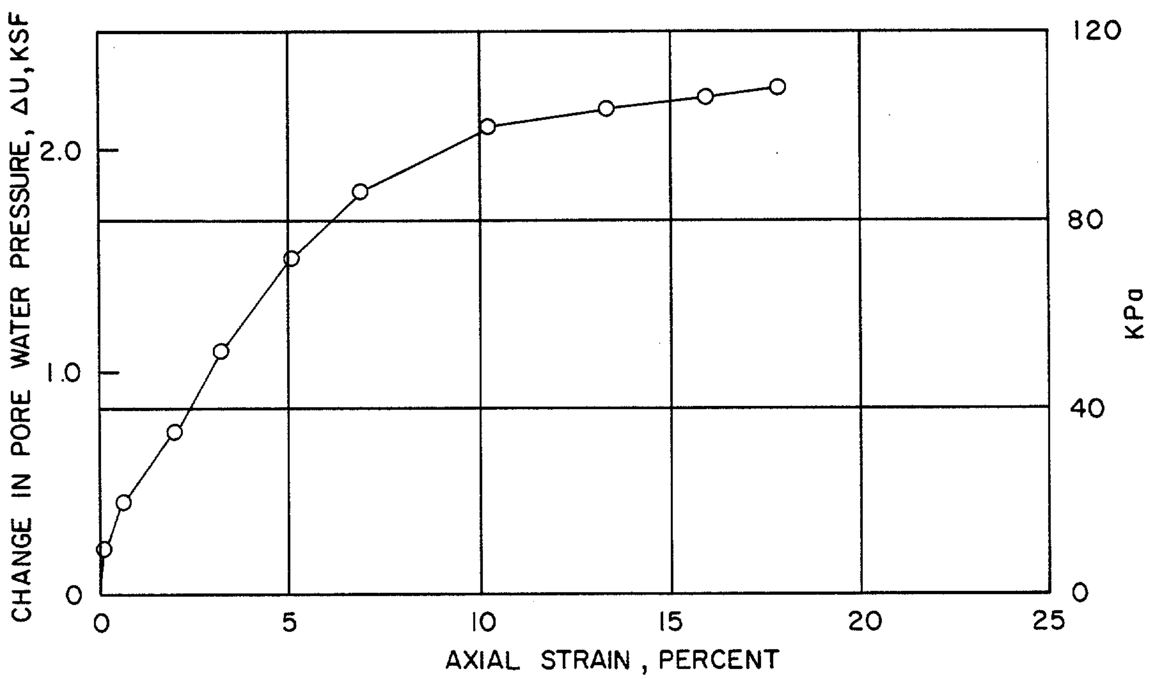

BORING NO: 5
SAMPLE NO: 76
DEPTH: 39.36-39.51M (129.1-129.6FT)
σ'_{3c}: 275.8 KPa (5.76 KSF)
OCR: 2.0

DEVIATOR STRESS AND PORE WATER PRESSURE
VERSUS AXIAL STRAIN
$CI\overline{U}C$ TESTS

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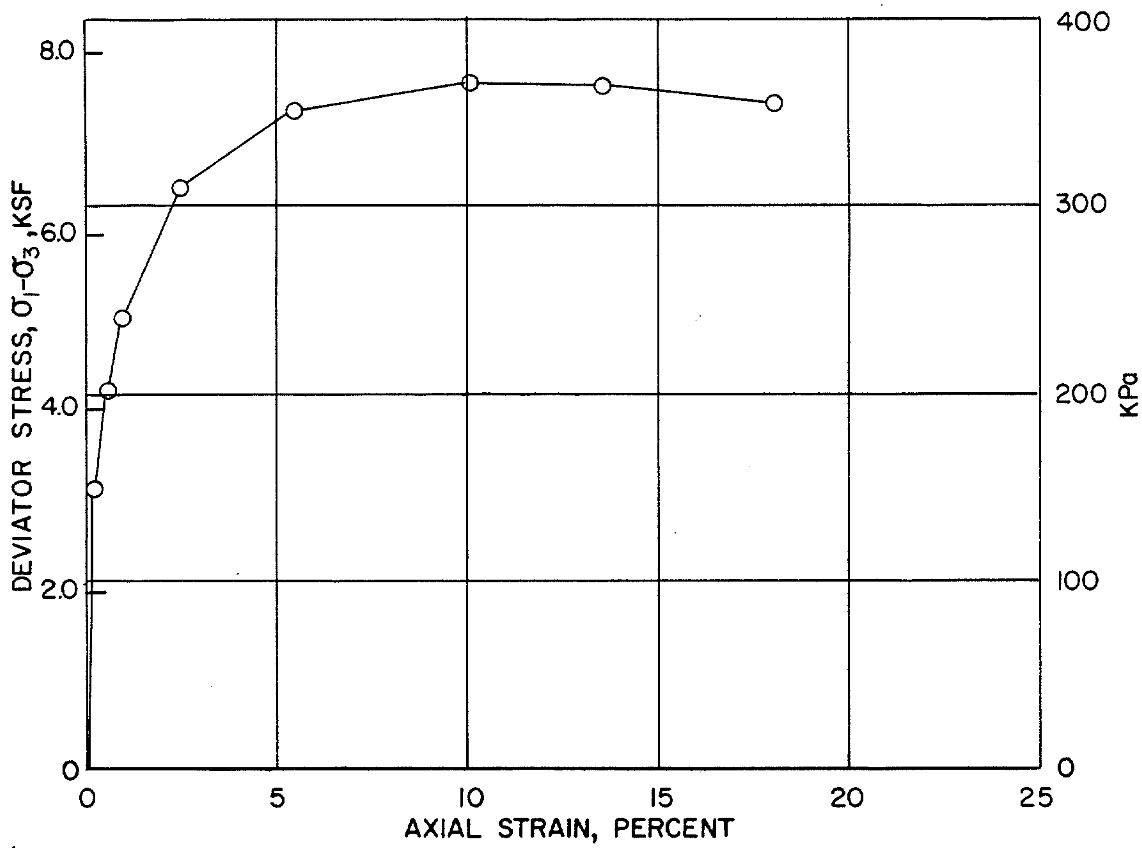

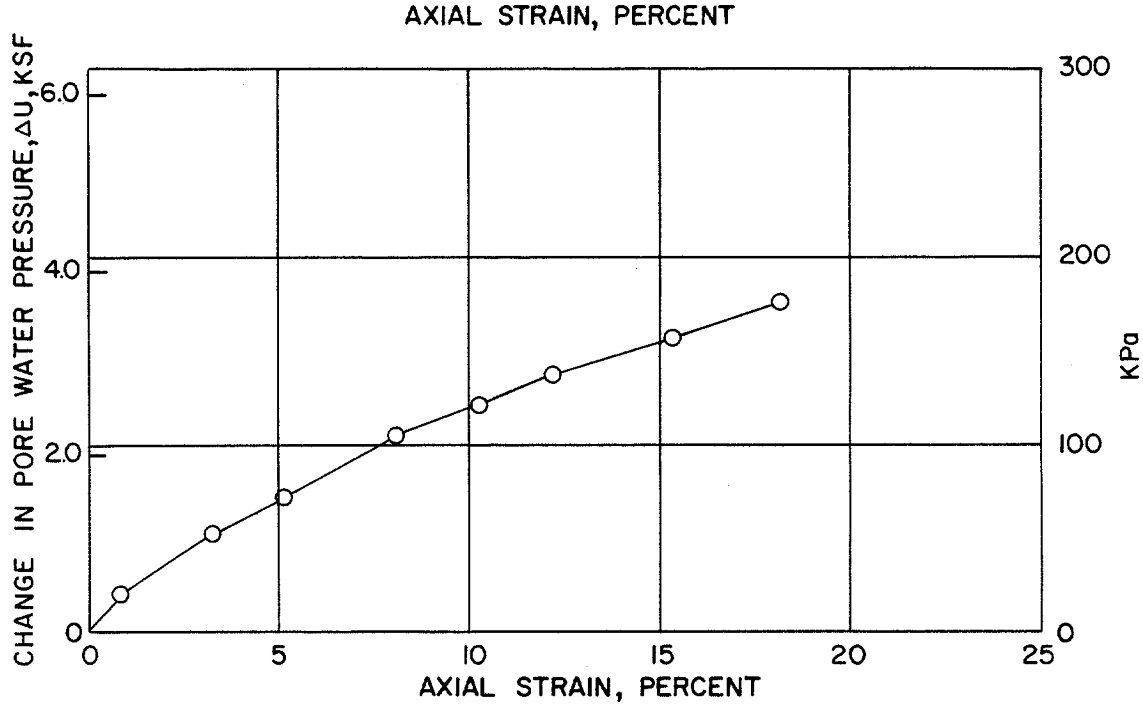

BORING NO: 6

SAMPLE NO: 62

DEPTH: 64.02-64.18 M (210.0-210.5 FT)

σ_{3c}': 503.3 KPa (10.5 KSF)

OCR: 2.0

DEVIATOR STRESS AND PORE WATER PRESSURE VERSUS AXIAL STRAIN $C\overline{IU}C$ TESTS

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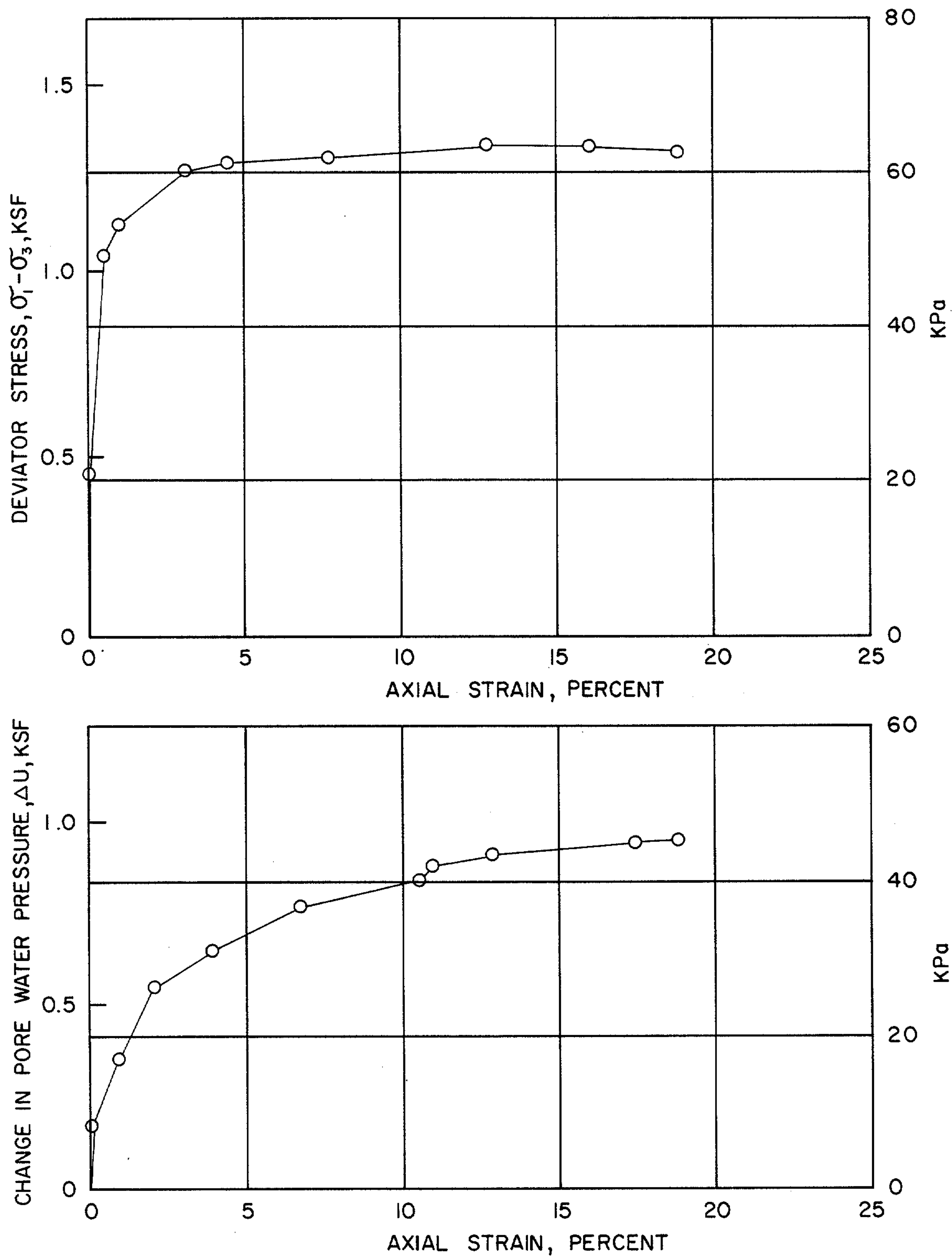
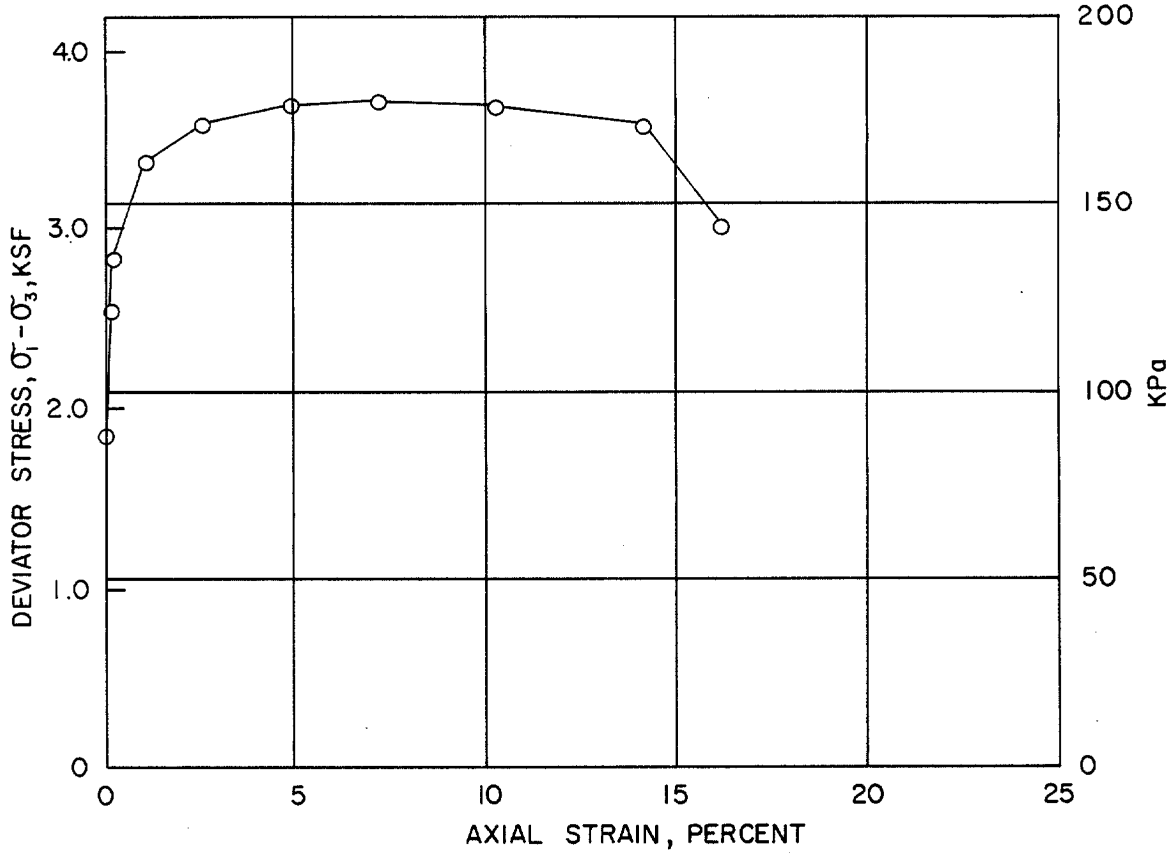
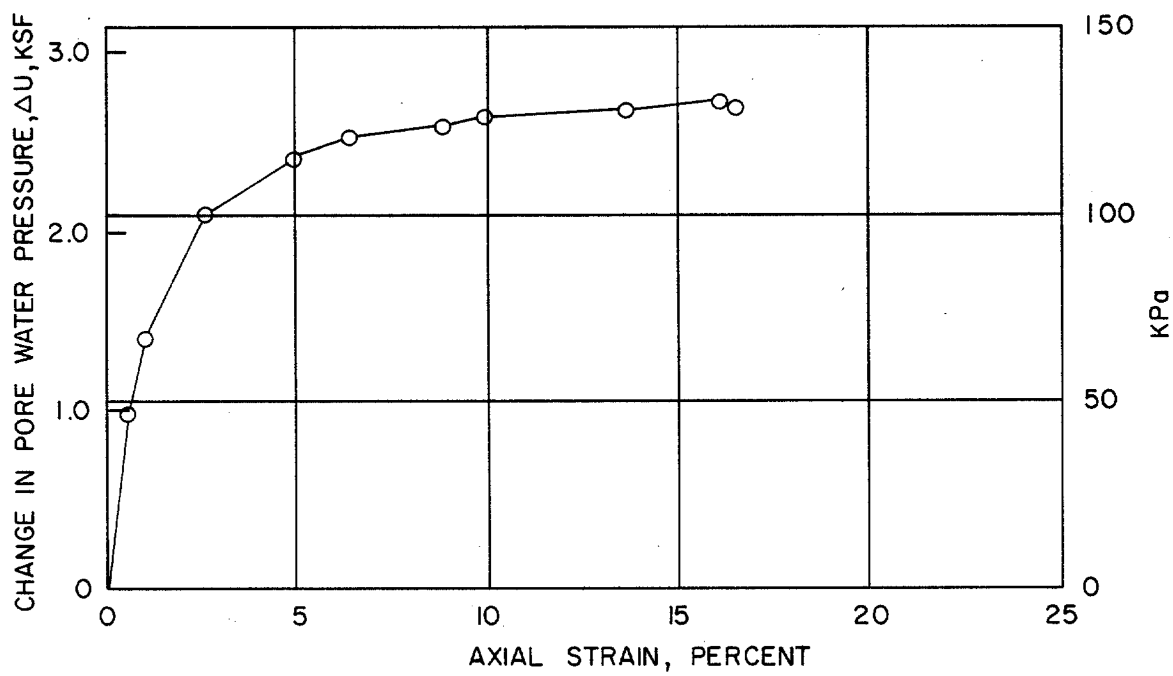
The Earth Technology Corporation



BORING NO: 5
SAMPLE NO: 20
DEPTH: 12.35-12.50 (40.5-41.0 FT)
σ'_{3c}: 75.8 KPa (1.58 KSF)
OCR: 1.0

DEVIATOR STRESS AND PORE WATER PRESSURE
VERSUS AXIAL STRAIN
$C\overline{K_oU}C$ TEST





BORING NO: 6
SAMPLE NO: 16
DEPTH: 24.94-25.0 M (81.8-82.0 FT)
σ'_{3c}: 196.5 KPa (4.10 KSF)
OCR: 1.0

DEVIATOR STRESS AND PORE WATER PRESSURE
VERSUS AXIAL STRAIN
$C\overline{K_oU}C$ TEST

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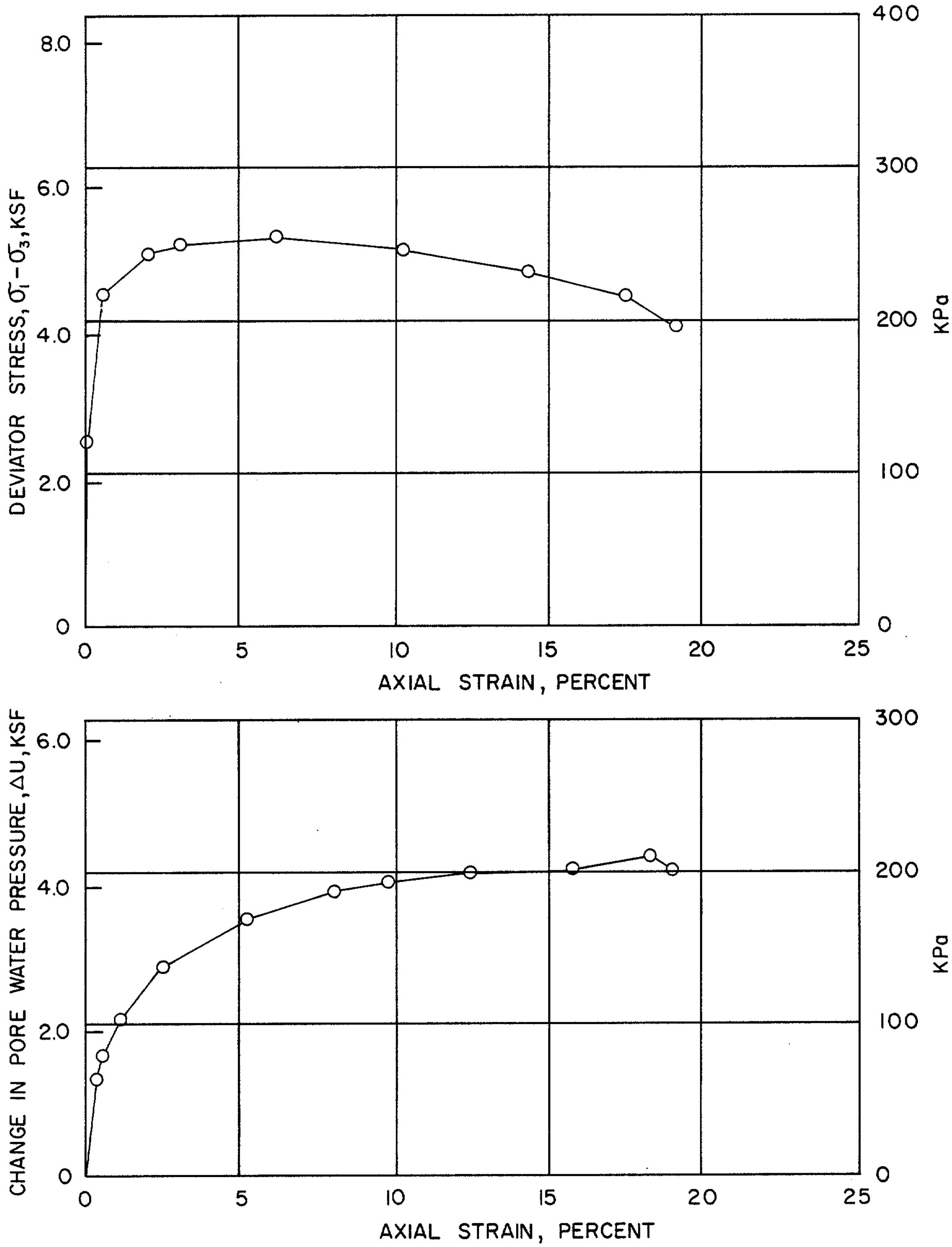
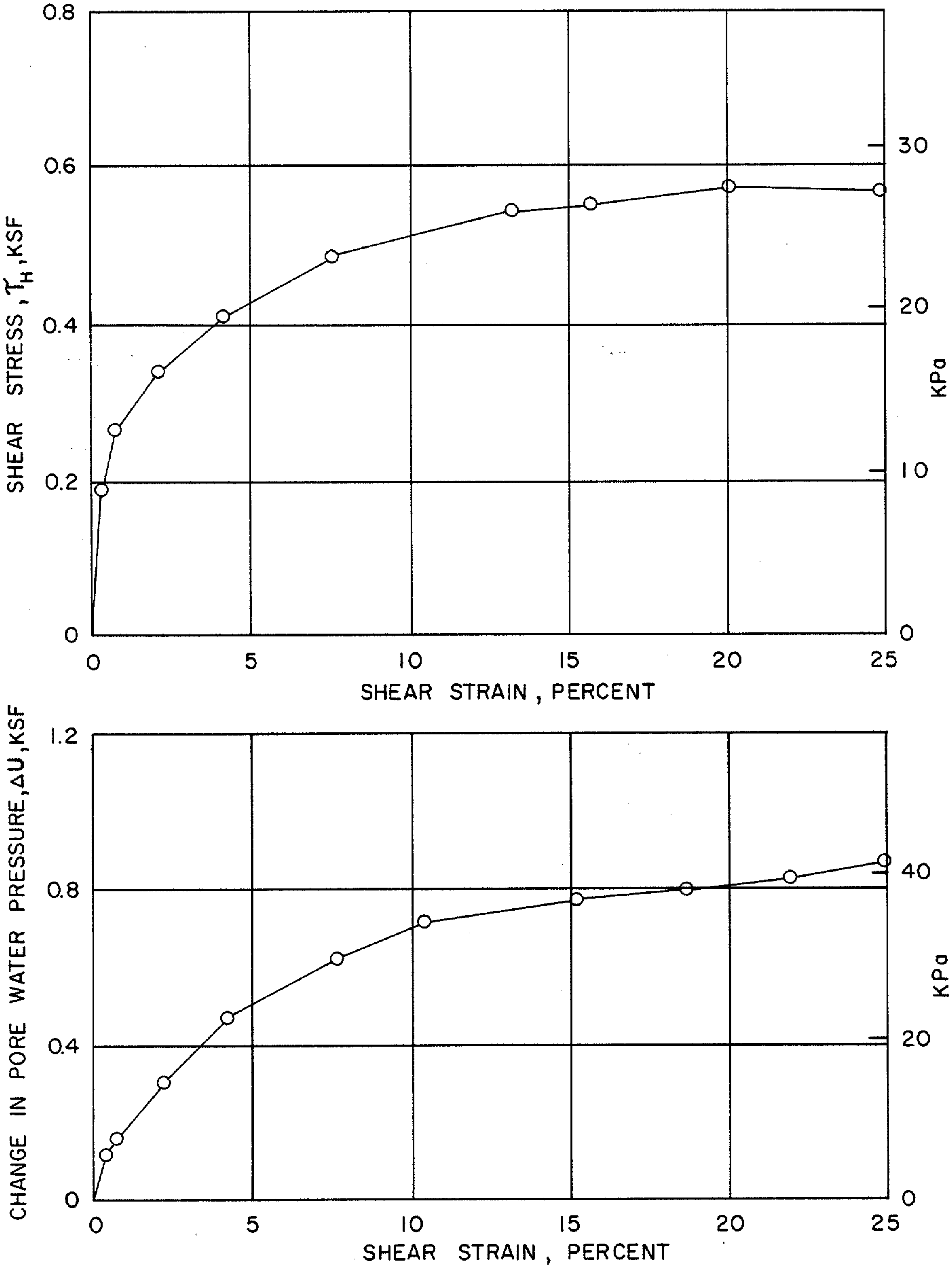


BORING NO: 6
SAMPLE NO: 53
DEPTH: 54.9-55.1 M (180.1-180.6 FT)
$\bar{\sigma}_{3c}$: 361.3 KPa (7.54 KSF)
OCR: 1.0

DEVIATOR STRESS AND PORE WATER PRESSURE
VERSUS AXIAL STRAIN
$CK_o\overline{U}C$ TEST

BORING NUMBER	SAMPLE NUMBER	DEPTH METERS (FT)	INITIAL DRY UNIT WEIGHT Mg/m_3 (PCF)	WATER CONTENT %		σ'_{VC} KPa(KSF)	OCR	AT FAILURE		
				INITIAL	FINIAL			τ_H KPa(KSF)	γ, %	ΔU KPa(KSF)
5	20	12.71-12.81 (41.7-42.0)	0.905 (56.5)	69.6	67.2	103.5 (2.16)	1.0	27.5 (0.57)	18.6	41.4 (0.86)
5	20	12.71-12.81 (41.7-42.0)	0.921 (57.5)	72.1	60.1	103.5 (2.16)	2.0	47.7 (0.99)	18.1	-4.14 (-0.09)
5	76	39.0-39.12 (128.0-128.3)	1.352 (84.4)	36.5	30.8	275.9 (5.76)	1.0	58.1 (1.21)	10.6	108.3 (2.26)
6	26	31.5-31.6 (103.4-103.6)	1.317 (82.2)	40.0	26.8	275.9 (5.76)	2.0	104.8 (2.19)	12.0	-12.4 (-0.26)
6	62	64.2-64.3 (210.7-210.8)	1.096 (6.84)	54.9	45.3	503.5 (10.51)	1.0	114.2 (2.38)	15.6	138.0 (2.88)
6	70	73.17-73.21 (240.0-240.2)	1.089 (68.0)	54.1	41.9	440.0 (9.19)	2.0	181.0 (3.78)	19.0	-44.8 (-0.94)

SUMMARY OF MONOTONIC SIMPLE SHEAR TEST RESULTS



BORING NO.: 5
SAMPLE NO.: 20
DEPTH: 12.71–12.81 M (41.7–42.0 FT)
σ'_{vc}: 103.5 KPa (2.16 KSF)
OCR: 1.0

SHEAR STRESS AND PORE WATER PRESSURE
VERSUS SHEAR STRAIN
$CK_o\overline{U}DSS$ TESTS

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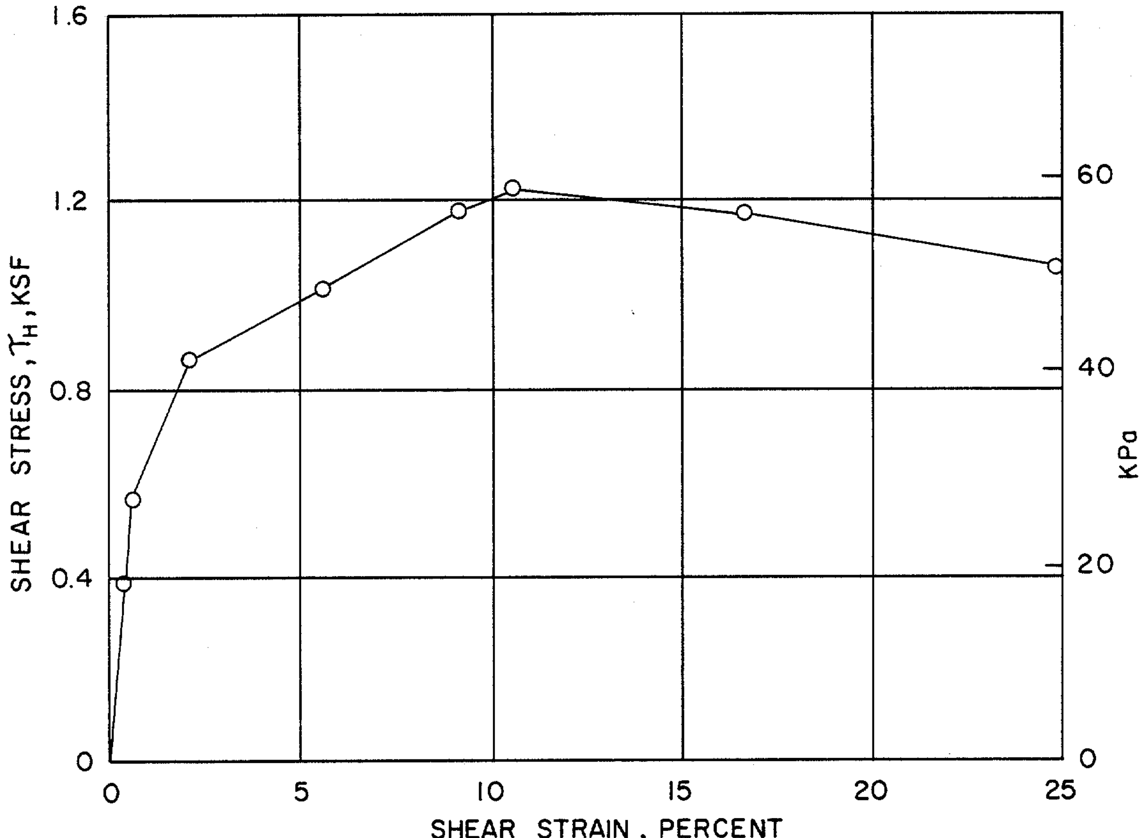
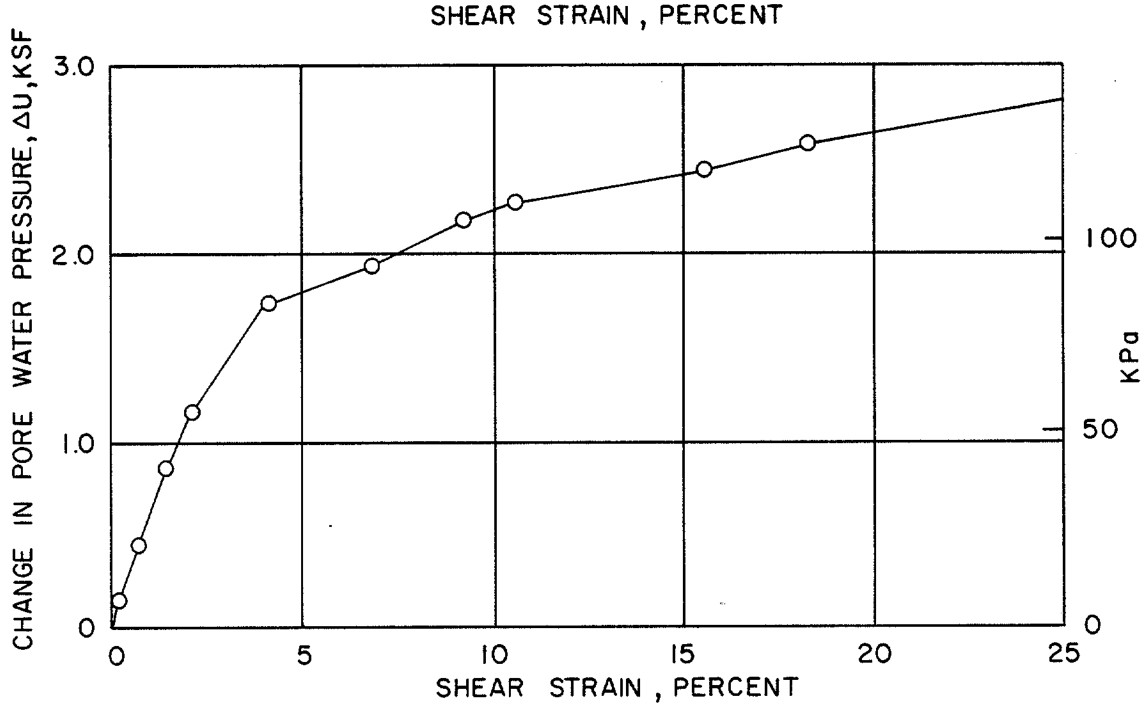


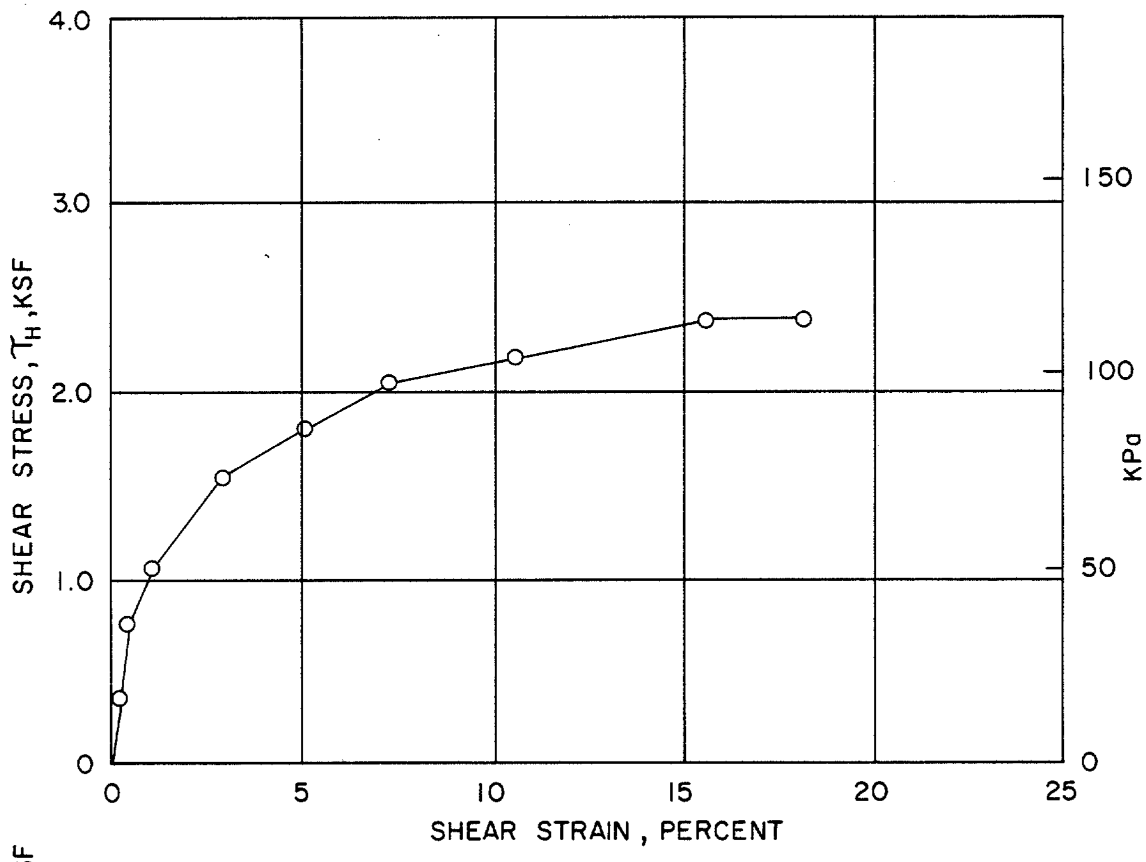


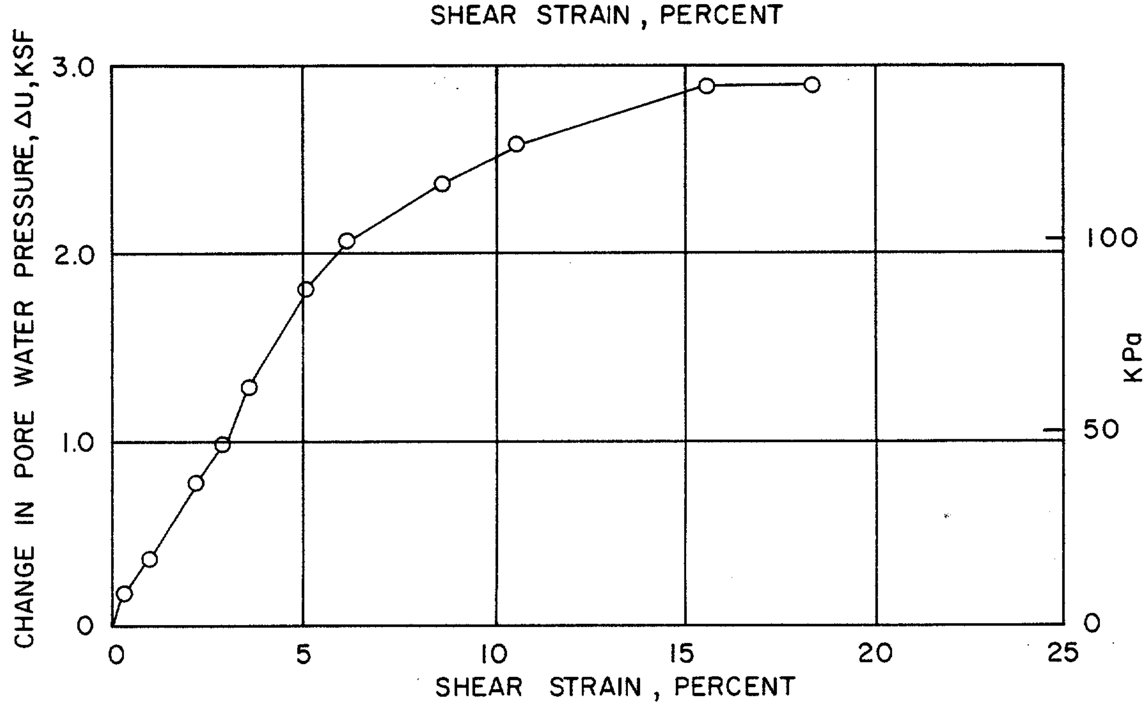
BORING NO. 5
SAMPLE NO. 76
DEPTH: 39.02-39.12 M (128.0-128.3 FT.)
σ'_{vc}: 275.9 KPa (5.76 KSF)
OCR: 1.0

SHEAR STRESS AND PORE WATER PRESSURE
VERSUS SHEAR STRAIN
$C\overline{K_oU}DSS$ TESTS

Ertec
The Earth Technology Corporation

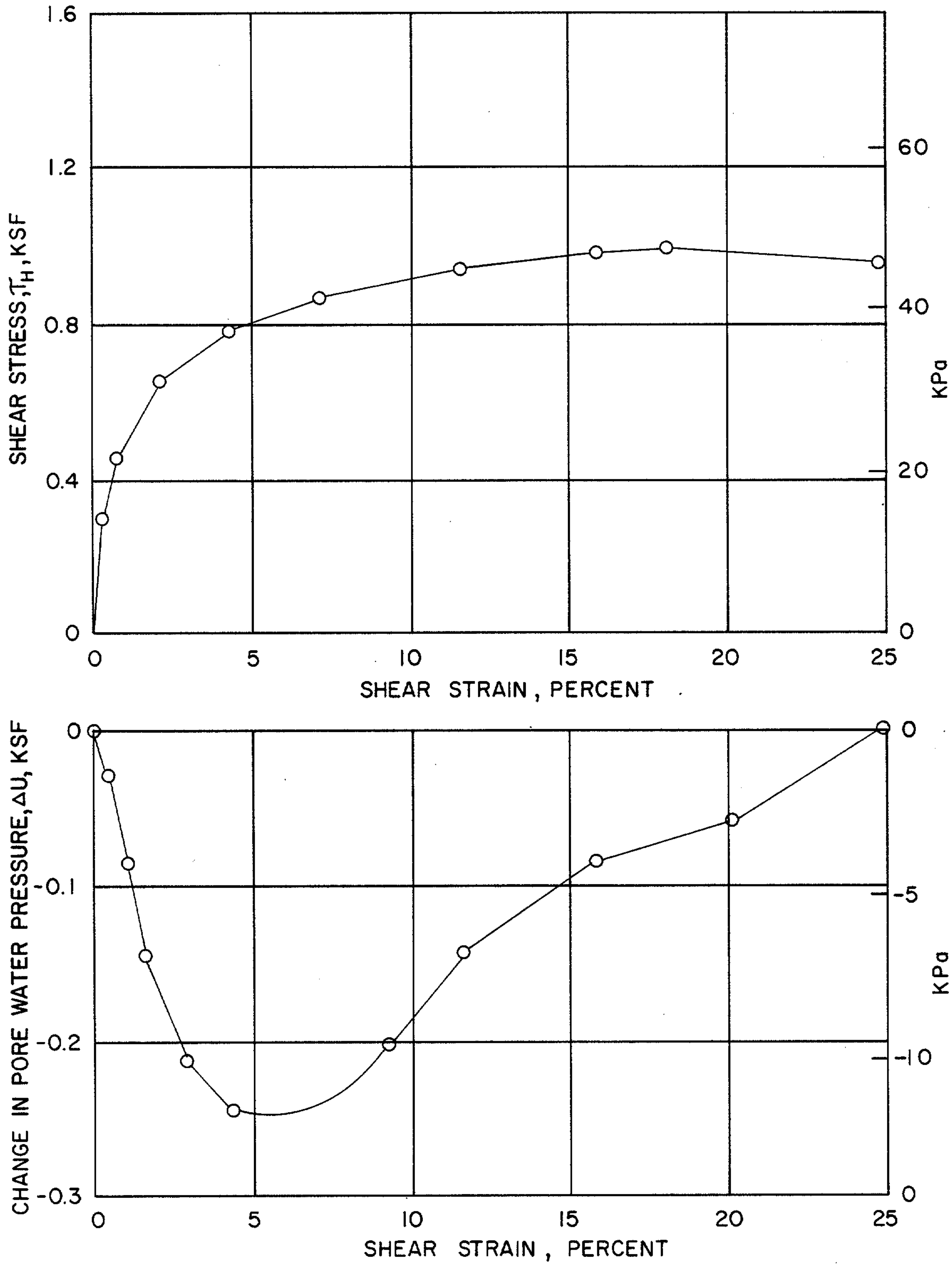
PLATE C-28





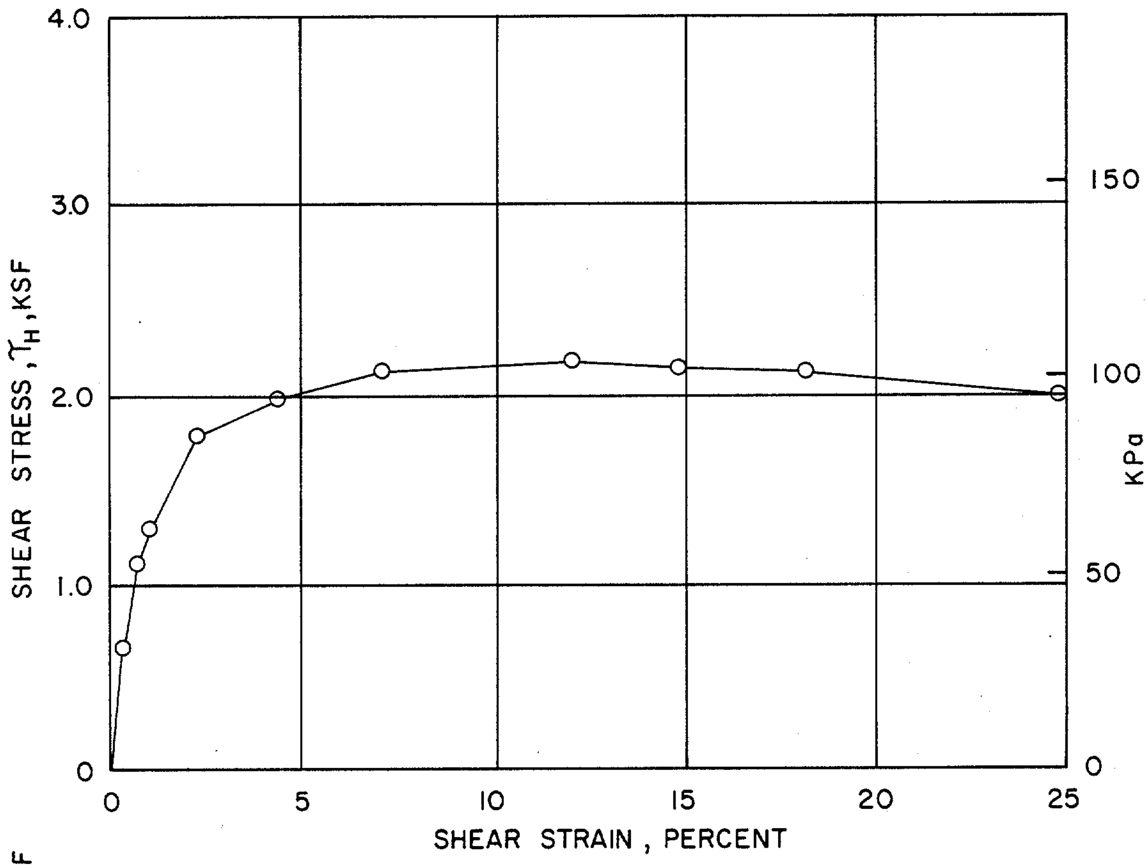
BORING NO.: 6
SAMPLE NO.: 62
DEPTH: 64.24-64.27M (210.7-210.8 FT)
σ'_{vc}: 503.5 KPa (10.51 KSF)
OCR: 1.0

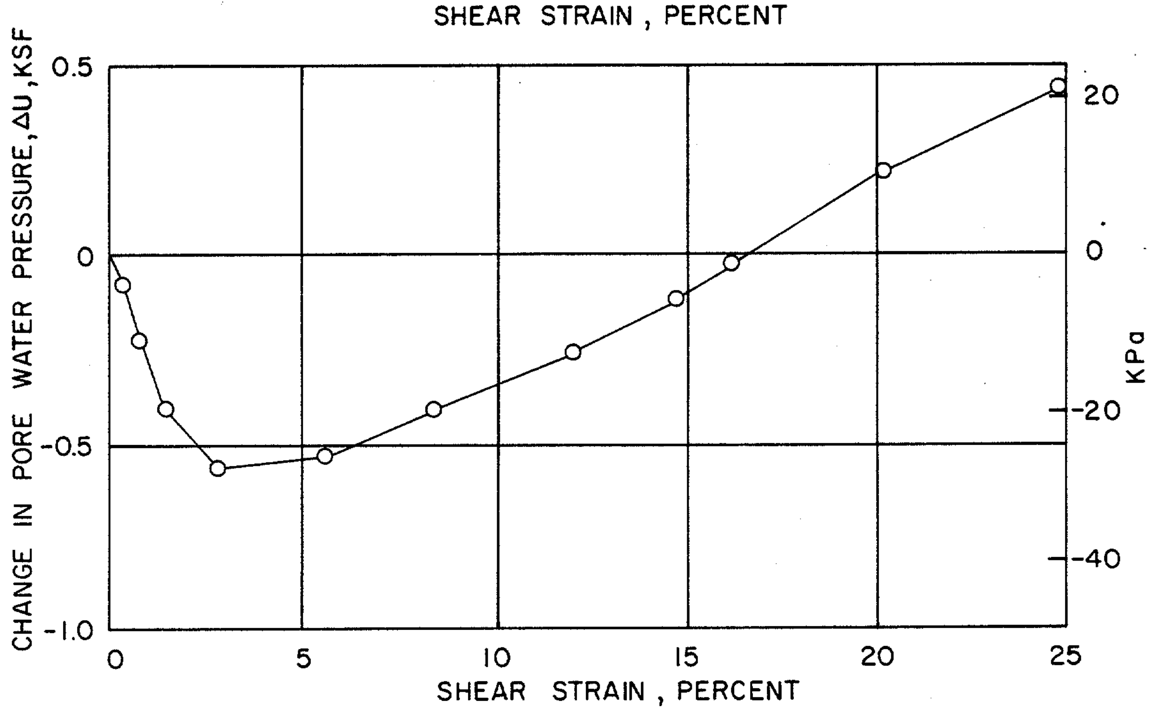
SHEAR STRESS AND PORE WATER PRESSURE
VERSUS SHEAR STRAIN
$C\overline{K_oU}DSS$ TESTS



BORING NO. 5
SAMPLE NO. 20
DEPTH: 12.71 – 12.81 M (41.7 - 42.0 FT)
σ'_{vc}: 103.5 KPa (2.16 KSF)
OCR: 2.0

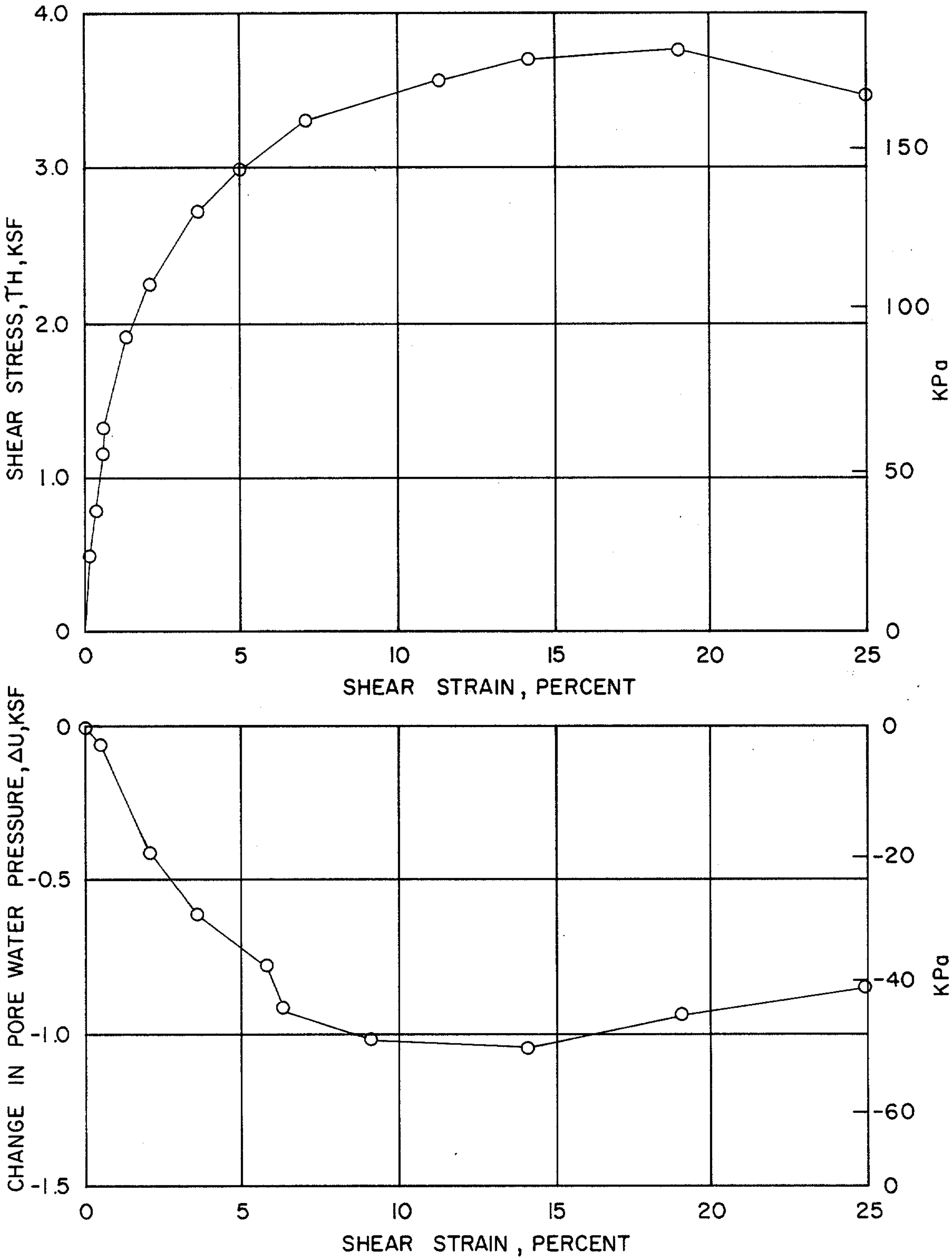
SHEAR STRESS AND PORE WATER PRESSURE
VERSUS SHEAR STRAIN
$C\overline{K_oU}DSS$ TESTS





BORING NO.: 6
SAMPLE NO.: 26
DEPTH: 31.52–31.59 M (103.4–103.6 FT.)
σ'_{vc}: 275.9 KPa (5.76 KSF)
OCR: 2.0

SHEAR STRESS AND PORE WATER PRESSURE
VERSUS SHEAR STRAIN
$\overline{CK_o}$UDSS TESTS



BORING NO: 6
SAMPLE NO: 70
DEPTH: 73.17-73.21 M(240.0-240.2 FT)
$\bar{\sigma}_{vc}$: 440.0 KPa (9.19 KSF)
OCR: 2.0

SHEAR STRESS AND PORE WATER PRESSURE
VERSUS SHEAR STRAIN
$C\bar{K}_oUDSS$ TEST