THE STRENGTH DEFORMATION PROPERTIES OF

SMITH BAY ARCTIC SILTS

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by

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STRENGTH-DEFORMATION BEHAVIOR OF SMITH BAY ARCTIC SILTS

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ABSTRACT

This research evaluates the consolidation and undrained strength-deformation behavior of Arctic silts at two Smith Bay sites in the Beaufort Sea, Alaska. Both sites consist of about 15 to 20 feet of silty clay overlying relict permafrost. Site T was subjected to extensive reworking by ice gouging, whereas Site W is nongouged. The laboratory testing program included: radiography, index properties, consolidation tests and consolidated-undrained triaxial and direct simple shear tests.

Estimates of preconsolidation pressures from the consolidation test program were used to develop stress history profiles at both sites. Site W is highly precompressed, with preconsolidation pressures increasing only slightly with depth. Preconsolidation pressures within the upper portion of Site T are much lower, but increase significantly at depths greater than 8 ft and eventually approach the magnitude of those at Site W. The progressive reduction of preconsolidation pressure near the mudline is attributed to the effects of ice gouging.

A fairly extensive program of Ko consolidated- undrained direct simple shear (CKoUDSS) tests run according to SHANSEP technique developed normalized stress-strain-strength properties for highly gouged and nongouged material as a function of overconsolidation ratio. SHANSEP triaxial compression tests were also run at Site T to evaluate anisotropy. Comparison of these data to results on samples reconsolidated to in situ stresses gave conflicting conclusions.

The results from the consolidation and SHANSEP DSS programs were used to develop undrained strength profiles for resistance to base sliding of a mobile gravity structure. At Site T, very low strengths at shallow depths correspond to ice gouging and the strength increases with depth as gouging effects diminish. Undrained strengths at Site W are markedly higher near the mudline and continue to increase with depth. Compared to these profiles, data from field vane and laboratory UUC

and miniature vane tests overestimate strengths appropriate for design by about 60% for the highly gouged zone at Site T and by factors of 2 to 5 within the other more heavily consolidated material.

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Chapter 1

INTRODUCTION

1.1 BACKGROUND

The Beaufort Sea (Figure 1-1) is thought to contain tremendous petroleum reserves. However, development is complicated by remoteness, cold temperatures, and difficult ice conditions. Drilling ships are limited to two months of operation in the Beaufort Sea due to ice buildup during the rest of the year. Even with ice-free conditions, drilling is hampered by permafrost, gas hydrates, ice scouring and weak soil deposits. All of these factors increase the cost of Arctic exploration.

Gravel islands, which are able to withstand the large lateral ice loads, have allowed year-round drilling to take place in shallow Arctic waters since 1973 (Watt, 1982). However, as exploration has moved to deeper water, gravel islands are no longer feasible. Instead, mobile gravity structures have been developed that are able to withstand the large ice loads. The ability to move the drilling structure to other locations also reduces costs.

More detailed examination of foundation conditions has accompanied the design of mobile gravity structures. The subsea soil profile generally consists of permafrost overlain by "Arctic silts" (the deposits are designated as "silts" based on grain-size analysis, rather than Casagrande's plasticity chart). The in situ behavior of Arctic silts is not well understood, especially since these soils behave differently from soil deposits in other offshore areas where oil and gas have been found. There is also little experience with designing structures for which the horizontal forces constitute the dominant design load (Wang et al., 1982; Vivitrat and Watt, 1983)

The design of Arctic gravity structures requires consideration of the following (Ladd, 1984):

1. The short term (undrained) behavior of the soil under gravity loads from the structure;

2. The possibility of severe lateral squeezing and large radial deformations resulting from the large width of the structure in relation to the thickness of the weak soil layer;

3. The magnitude and rate of strength gain due to consolidation of the foundation soils under the weight of the structure, possibly accelerated by prior installation of vertical drains;

4. A reliable estimate of the foundation resistance available during ice loading wherein the applied horizontal force may produce large rotations and possible reversals in the direction of the shear stresses acting within the foundation soils;

5. A foundation design for a mobile exploratory drilling structure that must have the capability to break away from the sea floor for relocation and also contend with a range of seafloor conditions.

Past geotechnical programs in the Arctic have relied upon the exploration and testing methods used for empirical designs of pile supported platforms in the Gulf of Mexico (Ladd, 1984). Such procedures cannot provide the detailed strength-deformation soil properties needed to design gravity structures subjected to complex loading conditions. New approaches are required to reliably evaluate various mobile gravity structures currently being developed to provide more economical exploratory drilling platforms.

1.2 MIT CENTER FOR SCIENTIFIC EXCELLENCE IN OFFSHORE ENGINEERING

In September of 1983 the Center for Scientific Excellence in Offshore Engineering was established at MIT under a $2 million grant from The Standard

Oil Company. The primary purpose of the Center is to conduct research on technical problems related to hydrocarbon development in the Beaufort Sea. The grant, which is spread over 5 years, was one of five awarded by The Standard Oil Co. to universities as a result of a nationwide competition for programs which would involve university based collaborative research on problems of national significance.

The Center at MIT supports research activities in the Departments of Civil and Ocean Engineering related to ice mechanics, geotechnical, structural, hydrodynamic, and risk and reliability aspects of offshore Arctic development. In addition to directly supporting research activity, the Center also promotes scientific interchange on the topic of Arctic engineering through the sponsorship of symposia, seminars and short courses. All the activities of the Center are closely linked with industry through involvement of the Standard Oil Production Company (Technology Center in Dallas, Texas) personnel at both the administrative and technical levels.

The Center's program in geotechnical engineering sponsors research in two areas: (1) engineering properties of Arctic silts, and (2) theoretical procedures for assessing the foundation stability of Arctic gravity structures. Research on the first topic started at the time of the Center's inception, and studies in the second topic commenced in September of 1984.

Funding from the MIT Sea Grant Program and from industry sponsors was initiated in July of 1984 to further support the research on geotechnical properties of Arctic silts. The industry sponsors include: Bedford Institute of Oceanography, Nova Scotia; Brian Watt Associates, Houston (first year); Fugro International, Inc., Houston (second year); Golder Associates, Canada (second year); McClelland- EBA, Inc., Alaska; Norwegian Geotechnical Institute, Oslo; Stone and Webster Engineering Corp., Boston; and The Earth Technology Corporation, Long Beach (first year). Bi-annual meetings have been held with SOPC and the industry sponsors both to present results from the cooperative research and to take advantage of the practical expertise these organizations have in dealing with geotechnical exploration and design of offshore structures.

1.3 RESEARCH OBJECTIVES

Ultimately the aim of the Center's research in experimental geotechnical engineering is to develop recommended procedures for measuring the engineering properties of Arctic silts which are necessary for the safe and economical foundation design of offshore structures. Specifically the program addresses the following issues (Sauls et al., 1984):

1. Why Arctic silts exhibit unique behavior compared to other offshore sediments, which negates reliance on past empirical correlations.

2. What types of in situ and laboratory test programs should be used to develop reliable estimates of the initial strength-deformation properties needed to predict the performance of gravity structures during and after setdown.

3. What types of laboratory shear tests should be used to select design strengths in order to evaluate foundation stability against massive horizontal forces due to ice loading.

Evaluation of these experimental results (together with available field performance) and input from the industry co-sponsors will provide the basis for developing guidelines for recommended practice. The research program developed to achieve these objectives was divided into three phases of investigation:

1. Geology and composition of the deposits;

2. Basic strength-deformation properties as a function of temperature, stress history, and failure mode;

3. Foundation stability against ice loading.

1.4 PRIOR RESEARCH ACTIVITIES

The Standard Oil Production Company (SOPC), through the initiative of Dr. Weaver, furnished MIT in January 1984 with 15 undisturbed silt samples taken from one boring beneath Mukluk Island and from three others several miles to the north. These were all located within or near the so-called "soft zone area" of Harrison Bay (Figure 2-1). The experimental program conducted on these samples included: radiography; preliminary compositional analyses; consolidation tests to determine the influence of test temperature on the estimated preconsolidation pressure; and a preliminary series of consolidated-undrained triaxial compression and extension and direct simple shear tests to investigate stress-strain-strength anisotropy and behavior under ice loading and whether or not normally consolidated silt exhibits reasonable "normalized" behavior. These results, plus a summary of research by MIT into the geology of Harrison Bay, are contained in the Center's first Research Report, entitled "Strength-Deformation Properties of Harrison Bay Arctic Silts" by D.P. Sauls, J.T. Germaine and C.C. Ladd (1984). A paper (Ladd et al., 1985) co-authored by Dr. Weaver and the same three MIT staff, presented at the ARCTIC '85 ASCE Specialty Conference, highlights the principal findings.

Experimental research during the second year involved the new supply of Arctic silt obtained in April 1984 from the special program of undisturbed sampling and in situ testing conducted off the ice near the edge of Mukluk Island. SOPC funded the program, which was executed under the supervision of Dr. Germaine acting as Standard Oil's field representative and in coordination with Dr. Weaver. Besides providing the project with a vital supply of 20 high quality samples, it also demonstrated the definite advantage of using fixed piston rather than the conventional (for the Arctic) push sampling technique; the amount of suitable soil was increased by a factor of two to three with little increase in sampling cost.

Extensive consolidation tests run on these samples, plus the radiography, show that this 25 ft thick deposit actually has two distinct layers. The Upper Layer is characterized by: highly stratified macrofabric; low natural water content; small clay size fraction; and a fairly high and relatively uniform preconsolidation pressure. In contrast, the Lower Layer is characterized by: generally uniform macrofabric; significantly higher water content and clay size fraction; and much lower preconsolidation pressures. Although the geological mechanisms responsible for this unusual condition are still unresolved, results of carbon dating, pollen analysis and review of historic sea levels suggest that the deposit is less than 5,000 years old.

Strength testing on these samples focused on undrained stress-strain-strength anisotropy via K_0 consolidated- undrained triaxial compression-extension and direct simple shear modes of failure. Most of these tests were run on normally consolidated samples in order to investigate the relative influence of macrofabric, water content, clay content and consolidation stress level on undrained shear behavior. Sufficient Recompression and SHANSEP[1] type data are also available to reasonably predict the initial in situ response of this overconsolidated Arctic silt deposit. These results are contained in MIT 1985 Master's theses, "Undrained Triaxial Strength-Deformation Behavior of Harrison Bay Arctic Silts" and "Consolidation and Direct Simple Shear Behavior of Harrison Bay Arctic Silts", by K.D. Ayan and E. Y-P. Yin, respectively.

[1]An acronym for Stress History And Normalized Soil Engineering Properties; Ladd and Foott (1974)

1.5 THESIS SCOPE AND OBJECTIVES

SOPC sponsored geotechnical exploration programs at four sites in Smith Bay in early 1985 that were executed by The Earth Technology Corporation (TETC). The field testing program at each site included several borings with undisturbed sampling and Piezocone penetration and one with field vane and pressuremeter tests. Laboratory testing by TETC at each site included classification and index properties, UU triaxial tests, a series of oedometer tests and several CU direct simple shear and triaxial tests. The Center worked with SOPC to evaluate the results of the geotechnical program and assist SOPC and its design consultant (EBA Engineering Consultants Ltd., Canada) in development of a site specific design. MIT also performed some additional special laboratory testing. The Center obtained access to extensive data from other offshore sites for comparison with results obtained at Harrison Bay in order to determine differences in basic behavioral trends. This collaboration also allowed MIT's research results to be applied to a real life design problem, with benefits to both groups.

Samples from Smith Bay were sent to MIT for a special test program at one site to supplement data obtained by TETC, plus additional testing at a second site to further MIT's research objectives. The MIT test program included consolidation tests to resolve discrepancies and fill in gaps in data for the development of stress history profiles. CU triaxial tests were run at the request of EBA to provide data for calibration of its soil model to predict foundation deformation. Also, a comprehensive series of CU direct simple shear tests was run in order to better assess the resistance of the mobile platform to horizontal sliding.

One objective of this program was to compare the basic strength-deformation behavior of Smith Bay and Harrison Bay Arctic silts. The Smith Bay deposits consist of silty clay, in contrast to the clayey silts tested at Harrison Bay. Also, the effects of ice gouging have significantly affected the stress history and strength profiles at one site in Smith Bay. The test programs have allowed SHANSEP strength profiles to be compared with each other, and with results from conventional strength testing. Assessment of the effects of ice gouging and comparison of strength profiles provide help in the development of guidelines for practice since representative types of Arctic silt should be studied, both compositionally and in terms of geologic history.

This research was supervised by Prof. C.C. Ladd and Dr. J.T. Germaine. Drs. J.S. Weaver and S. Law, SOPC's Technical Representatives for geotechnical engineering, have been very helpful in contributing to the development of the research. The cooperation of Mr. David Rodger, Standard Alaska Production Company, was also essential in providing a new supply of undisturbed samples from Smith Bay and in obtaining permission to use the TETC test data.

1.6 ORGANIZATION

Chapter 2 describes the Beaufort Sea environment and the general properties of Smith Bay Arctic Silts. The shortcomings of geotechnical site characterization methods previously used in the Arctic are also discussed.

Chapter 3 contains information concerning the field testing and sampling program and covers the basic characteristics of the samples. Results from index tests are presented, and the overall test program is described.

The results from engineering tests are presented in Chapters 4, 5, and 6, regarding consolidation, CU triaxial and CU direct simple shear test programs, respectively. Detailed information from engineering tests are located in appendices.

The overall results are analyzed in Chapter 7. Chapter 8 provides a summary, conclusions and recommendations based on research completed to date.

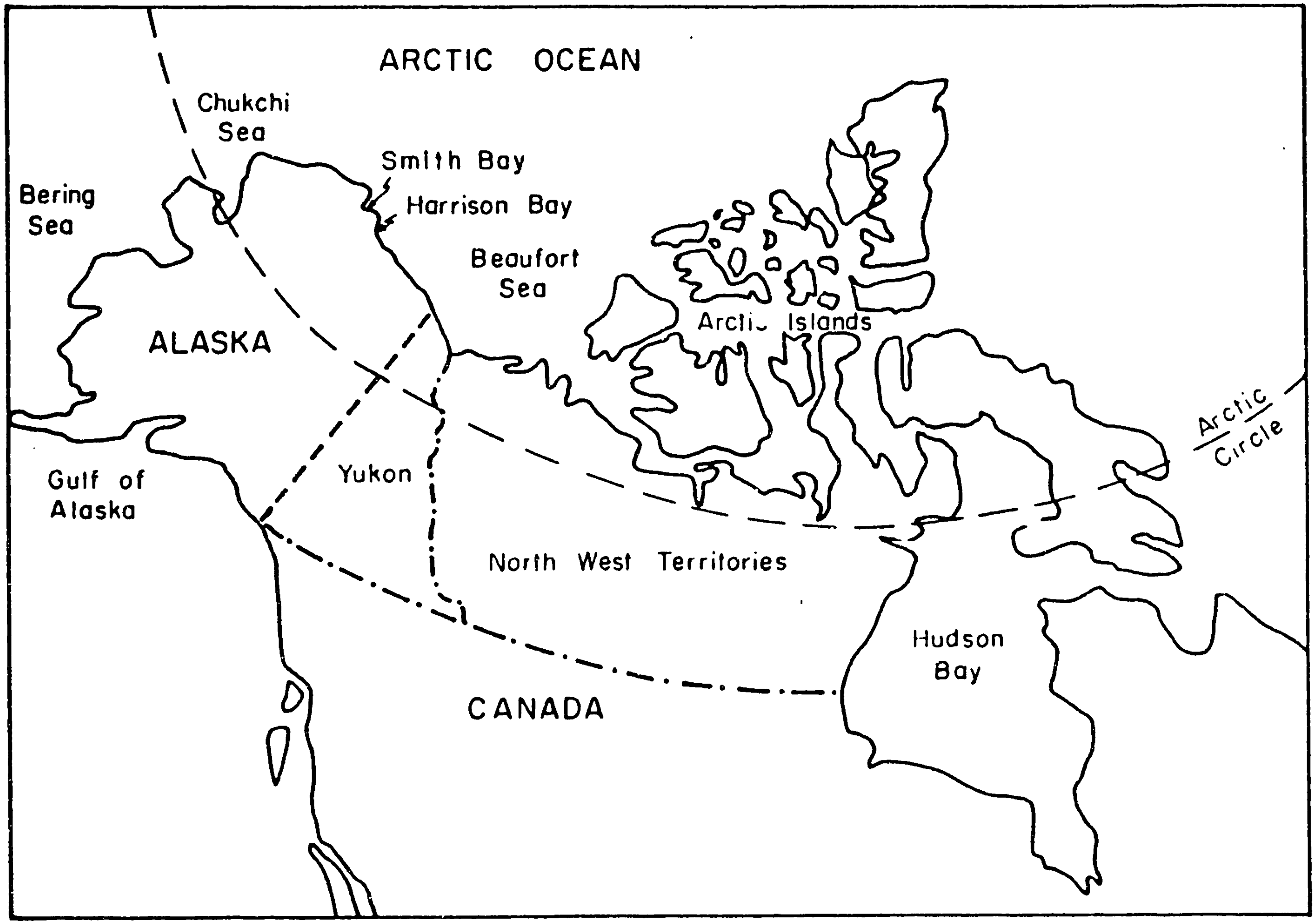


Figure 1-1: Areas of Interest to Offshore Arctic Exploration, (after Noble, 1983)

Chapter 2

BACKGROUND

2.1 REGIONAL SETTING OF SMITH BAY ARCTIC SILTS

The location of Smith Bay in the Alaskan Beaufort Sea is shown in Figure 2-1. The general soil conditions at the two locations investigated for this thesis consist of about 15 to 20 feet of silty clay and clayey silt overlying relict permafrost. The site nearer land (Site W) has a very thin layer of soft Holocene material over much stronger Pleistocene cohesive soil. The stratigraphy of the other site (Site T) is much more complex due to extensive ice gouging, a process discussed in Section 2.1.2.

2.1.1 Geology

The coastal plain of Northern Alaska is an area of low relief underlain by alluvial and glacial fans extending northward (Wolf et al., 1985). The tundra surface is pitted with thousands of thaw lakes along the coastline. The coastal belt east of Point Barrow consists of barrier islands, tundra bluffs, river deltas, bays and estuaries.

The western part of the Beaufort Shelf (Point Barrow to 145 °W longitude) is dominated by the Barrow Arch. The metavolcanic basement rock was deformed during a major episode of faulting and uplift in the early Cretaceous. The axis of the arch trends East-West, resulting in a southward dipping platform to the South and a northward sloping continental margin to the North (Sharma, 1979). The eastern part of the Beaufort Shelf (from 145 °W to the Canadian border) is characterized by multiple anticlines and synclines.

The Quaternary deposits in the coastal area consist of the Gubik Formation and Holocene deposits. The Gubik Formation is an "unconsolidated" mix of silt and fine-grained sand with clay and gravel of glacial marine origin. Evidence of lacustrine, fluvial and lagoonal deposits is found in the upper layer. Holocene marine deposits were deposited during the most recent transgression. The Holocene deposits form a wedge which thickens offshore to a maximum thickness of more than 100 feet near the shelf break. The wedge is thinner on the western half of the shelf than on the eastern half (Grantz and Dinter, 1980). The Holocene sediment thickness ranges from 15 to 30 feet, with older sediment cropping out in areas of ice scour and non-deposition (Barnes and Reimnitz, 1974). Holocene sediment is thin or absent along the coast. Offshore Holocene deposits frequently overlie relict permafrost (permafrost formed in the past during a colder climate, and not in thermal equilibrium with the present ground temperature).

About 20,000 years ago, glaciation was at a maximum and sea level reached a minimum depth of 280 feet below present (Wang et al., 1982). As a result of sea level fluctuations, the continental shelf within the 100 foot isobath was exposed to the Wisconsin cold climate for more than 15,000 years, forming permafrost (now termed "relict", as described above). Sediment embedded in ice was released when the ice was grounded, a process called ice rafting. As the glacial ice retreated, sea level has risen continuously to the present, although at varying rates and with minor fluctuations (Hopkins, 1973). The rise in sea level resulted in an influx of sediment from coastal erosion. Presently, the shelf is characterized by ice scour and low deposition.

2.1.2 Ice Regime

The ice regime in the Beaufort Sea can be divided into three zones: landfast ice, shear (or stamukhi), and polar pack ice (Figure 2-2; Barnes and Reimnitz, 1983). Landfast ice forms in September or October, growing outward from the shoreline. It extends to about the 60 ft contour line, reaching a thickness of about 6 ft. Shorefast ice near rivers is inundated by flood waters each spring, and open water is present again by July (Ayan, 1985).

The polar pack ice lies seaward of about the 130 ft contour and is composed mostly of multi-year ice. The polar pack is in motion much of the year, rotating East to West under the influence of the Arctic gyre. The penetration of the polar pack ice onto the shelf is controlled by seasonal weather patterns and ice drift. Shifting winds may bring the polar pack ice in contact with the shoreline.

Between the landfast and polar pack ice lies the shear, or stamukhi, zone. Shear ridges form as a result of shear action between ice features and consist of seasonal and multi-year ice. They are present throughout the winter and sometimes into summer. The width of the stamukhi zone varies from year to year and seasonally, depending on the geographic location and position of the polar pack, and ranges from 30 to 90 miles (Ayan, 1985). Ice gouging of the seafloor by grounded ice ridges is dominant in this zone. Seasonal gouging may occur at depths less than 60 ft; however, the resulting gouges can be smoothed over by the waves and currents of a single summer (Reimnitz and Barnes, 1974). Rates of gouging inshore of the stamukhi zone are 1-2 % of the seafloor per year (Grantz et al., 1980).

Gouges are an average of 3 ft deep and 6 to 9 ft wide, although they have been measured as deep as 18 ft (Barnes and Reimnitz, 1974). Gouge density can be as high as 160 per mile. The highest density is found on steep seaward facing slopes and topographic highs, with the lowest densites landward of such highs and landward of islands and areas adjacent to river deltas (Reimnitz and Barnes, 1974). Estimated recurrence of ice gouging at a given site, on average, is 50 to 100 years (Reimnitz et al., 1980).

2.1.3 Depositional environments

Sedimentary processes and features are related to the various ice zones (Barnes and Reimnitz, 1983). Delta facies form in the nearshore zone, adjacent to the coast, as the result of the influx of sediment from rivers, ice breakup and coastal erosion. Marine reworking by currents and waves is seasonally interrupted by ice flooding and vertical drainage through the ice canopy (strudel) creating scour depressions. Bedding may be discontinuous due to scouring and filling in of depressions.

The landfast ice zone deposits consist of sedimentary features formed by waves and currents, with sufficient ice gouging taking place so that surficial sediment is completely reworked in less than 200 years. The result is a bedded sequence of sand and finer material with contorted fine-grained beds. Laterally discontinuous beds are composed of ice gouge infill, resulting in interfingered deposits.

Deposits in the stamukhi zone grade seaward from the landfast ice zone bedded deposits through a zone of contorted bedding to a completely heterogeneous sequence of gravelly silts and clays in waters 100 to 130 ft deep. Angular, striated clasts are of glacial origin, probably carried to their present location by ice rafting. The extensive ice gouging in this zone reworks the sediment, mixing the relict gravels with more recent deposits. The low rate of deposition also prevents the burial of relict gravel.

In waters more than 160 ft deep, few ice ridges are large enough to interact with the seafloor, so hydraulic processes dominate. The result is a bedded sequence of laterally continuous silts and clays, sometimes containing sand, with only traces of Holocene sediment.

2.2 PRECONSOLIDATION OF BEAUFORT SEA DEPOSITS

Preconsolidated sediment is commonly observed on the Beaufort Shelf. The pattern of overconsolidation is highly variable, and seems to transgress geologic boundaries (Reimnitz et al., 1980). Erosion of overburden would result in a constant difference between the preconsolidation pressure and the in situ vertical effective stress; this is not what is observed. There is also nothing in the geologic record to indicate that substantial overburden was removed in the Holocene to explain the high overconsolidation ratios observed in surficial sediments.

A number of mechanisms have been proposed for preconsolidation of such deposits (Sauls et al., 1984):

1. Ice loading and gouging could cause a highly variable pattern of overconsolidation.

2. Desiccation has been found to be significant in many offshore deposits.

3. Freezing and thawing cycles have been suggested as a possible cause of overconsolidation in the Beaufort Sea (Chamberlain et al., 1978).

4. Wave action induces repeated shear stresses in ocean sediments (Madsen, 1978) which could result in preconsolidation.

5. Natural cementation between soil particles results in an increase in the measured preconsolidation pressure.

6. Secondary compression (aging) has been shown to cause preconsolidation (Leonards and Altschaefl, 1964; Bjerrum, 1967) although not of the magnitude exhibited by Arctic silts.

Freeze-thaw is a frequently cited mechanism for overconsolidation at a number of locations in the Beaufort Sea. Since it represents a rather unique mechanism compared to most offshore deposits, it is discussed below in more detail.

Laboratory experiments have documented the overconsolidation of sediments by freeze-thaw cycling (Chamberlain, 1981; Chamberlain and Gow, 1979). In these tests the overconsolidation is attributed to negative pore pressures developing during freezing. This causes water flow from the unfrozen zone to feed the ice lenses, therefore increasing the effective stress on adjacent material. As a result, soil particles are reoriented and consolidated into a more compact structure. Water content decreases in the area adjacent to the zone of ice segregation to a critical value at which the unfrozen zone can no longer supply water at the rate requires by suction forces. The desiccated zone is quickly frozen, with a new zone of ice segregation established. This process continues as long as the net heat flow outward is positive and water is available for freezing.

Freeze-thaw during periods of lowered sea level could clearly produce overconsolidated sediment; however, "subtle" or seasonal seafloor freezing can also have similar effects. Seasonal freezing of the seabed can be demonstrated by calculating the freezing point over the soil profile and comparing this to the measured temperature profile. The depth of influence of seasonal freeze-thaw ranges from a few inches to 6 ft (Lee et al., 1985). Pore water does not freeze completely because as freezing progresses, salt concentration in the remaining water increases and reduces the freezing temperature of the remaining unfrozen water. Therefore, ice bonding may not be strong enough to form visible ice.

Recent work has attempted to distinguish the effects on engineering properties of these different types of freezing. Freeze-thaw during sea level lowering would have the maximum effect on consolidation properties; high levels of overconsolidation and water contents near the plastic limit are indicative of sediment subaerially exposed and severely frozen. Sediment of Pleisocene age could easily have experienced this level of freeze-thaw.

Shallow freezing influences sediment in one of two ways:

1. Subtle freeze-thaw taking place after the development of a Holocene profile would result in a crust of overconsolidated sediment over normally consolidated sediment (Ladd et al., 1985; Ayan, 1985; Yin, 1985).

2. Seasonal freeze-thaw may also occur continuously as the sediment column is deposited, resulting in uniform overconsolidation versus depth.

For both cases, freeze-thaw is not related to subaerial exposure and the water content would not decrease to the plastic limit. However, a mechanism for contact between the seabottom and cold Arctic air is needed.

In water less than 6 ft deep, the sea floor sediment is coupled to cold temperatures by the downward growth of the winter ice sheet. In deeper water, coupling may occur by either grounding of ice keels, or transgression of barrier islands. But ice grounding is unlikely to exist for extended time, therefore significant freezing would not occur. It is possible that repeated gouging could have a cumulative effect, although this has not been documented. Overconsolidated sediment has been found seaward of barrier islands, providing support for the second mechanism (Chamberlain, 1981).

2.3 GENERAL SOIL PROFILES OF SMITH BAY SITES

Proprietary reports made available to MIT prior to the start of testing summarized results of the geotechnical program carried out by TETC. Each of the two sites in Smith Bay covers an area of 500 ft by 500 ft. A preliminary assessment of the geologic history of each site was made by TETC, based on a literature search and correlation with onshore units of known age and origin. Generally the offshore sediments consist of silty clay and clayey silt extending to a depth of 25 ft, underlain by frozen sand.

Quaternary sediments record a series of transgressive marine sequences deposited during high stands of sea level. Low stands of sea level correspond to glacial intervals during which sediment was subaerially exposed and subjected to cold Arctic temperatures and erosion.

Site W is located in the landfast ice zone; ice gouging is therefore limited, but may cause soft sediment deformation in the uppermost sediments. The upper 3 ft of the deposit consist of a transgressive sequence of silt overlying silty sand of Holocene age. The presence of occasional clasts and deformed layers in this upper layer may indicate slight reworking of deposits by ice. Below this is a fining upward (transgressive) sequence ranging from silt to silty clay of Pleistocene age. It is separated from the upper unit by an erosional unconformity. Clasts present in this unit are presumably ice rafted. Frozen material appears 20 ft below the mudline.

Site T is located farther offshore, near the stamukhi zone; therefore, ice gouging is a dominant process. Extensive micorelief of the seabottom is the result of ice action. The uppermost transgressive unit of silt overlying silty sand has a thickness of 10 to 20 ft. Although Holocene material is present in this unit, it is impossible to define a lower boundary due to ice gouging effects. Ice has reworked the deposits and incorporated the underlying Pleistocene material, as evidenced by the presence of ice rafted pebbles and contorted beds throughout the unit. Beneath the gouged zone is a transgressive sequence grading from silt to silty clay, also with scattered pebbles. Permafrost is present at depths greater than about 15 ft below the mudline.

2.4 PROBLEM DEFINITION (abstracted and modified from Ayan, 1985)

The procedures used in pre-1984 geotechnical studies in Harrison Bay closely paralleled those developed for the empirical design of pile supported platforms in the Gulf of Mexico. In particular, undrained strengths were generally estimated from results of the following types of tests (Sauls et al., 1984; TETC, 1985):

1. In situ tests such as the field vane and the Dutch cone penetrometer;
2. Strength index tests (Torvane, Pilcon vane, miniature lab vane, pocket penetrometer) performed on "undisturbed" push samples;
3. Other laboratory shear tests on tube samples such as unconsoldated-undrained triaxial compression (UUC) and isotropically consolidated-undrained triaxial compression (CIUC) tests.

All three methods are suspect when applied to Arctic silt deposits.

In situ tests require empirical correlations to obtain strength values appropriate for design. The correction factor versus plasticity index recommended by Bjerrum (1972) is commonly used to adjust measured field vane strengths. However, the case histories from which the recommended correction curve was developed did not include Arctic silt type soils. The validity of the field vane test as an undrained shearing process within Arctic silts may be questionable. The permeability of some Arctic silts may be so high as to allow partially drained conditions to develop during the test. Measurement of strength using the Dutch cone poses similar problems; the empirically developed data base for the cone factor does not include Arctic silt type material, and there can be uncertainty as to the drainage conditions (Sauls et al., 1984).

In situ tests are usually the most reliable and cost effective tool for measuring spatial variability of deposits, but only for well defined drainage conditions, i.e. either undrained or fully drained. Laboratory strength index tests are generally less efficient for this purpose due to the high cost of obtaining samples and are often unreliable due to problems caused by varying degrees of sample disturbance.

The use of UUC tests to obtain design strengths depends on uncontrollable compensating errors: the strength increase due to rapid shearing and neglect of the effect of anisotropy offsetting the effect of sample disturbance (e.g. Koutsoftas and Ladd, 1985). CIUC tests, although more sophisticated than UUC tests, are also inadequate. In CIUC tests the sample is isotropically consolidated and then sheared with the major principal stress acting vertically. This type of failure mode greatly overpredicts design strengths for horizontal shearing under ice loads.

A further measure of uncertainty associated with the measurements of undrained strength in Arctic silts is illustrated by the following examples. At Harrison Bay, Ladd et al. (1985) reported $c_u(UUC)/\sigma'_p = 0.185 \pm 0.065$ SD. The field vane strength was usually several times higher. In contrast, as will be shown, UUC strengths equalled field vane values at Smith Bay. But both values are considered much too high for design.

Issues relevant to the behavior of Arctic silts are explicitly ignored by the above strength test procedures. Specifically, the effects of anisotropy, changes in stress history, sample disturbance, strain rate, and environmental factors are not considered. These factors are discussed below.

Strength-deformation properties of sedimentary soils vary with the direction (δ angle) of the applied major principal stress relative to the vertical direction of deposition (Ladd et al., 1977). This property, anisotropy, usually causes a substantial decrease in undrained strength and increase in strain at failure as δ varies from 0^o to 90^o. Thus triaxial compression type tests, when considered alone, overestimate design strengths.

In considering foundation stability, anisotropy is coupled with the phenomenon of progressive failure - all elements within the zone of shearing will not reach their peak strength simultaneously. Soils exhibiting strain softening will create a situation in which come soil elements lose resistance before the strength in other elements are fully mobilized (assuming a constant value of shear strain along the potential rupture surface). The net result of averaging shear strength values at different strain levels is a decrease in the average strength. An approach which considers this "strain compatibility" is important when selecting design strengths (Koutsoftas and Ladd, 1985).

For offshore structures in the Arctic, ice loading will probably make anisotropy and progressive failure effects even more important because of the potential reversal in the direction of the major principal stress during shear. As illustrated in Figure 2-3, the foundation soils will first be subjected to significant radial shear stresses (possibly accompanied by large radial shear deformations due to lateral squeezing), followed by consolidation and strengthening. During this complex process the direction (given by the angle δ) of the major principal consolidation stress, σ'_{1c}, will undergo rotations, the magnitude of which will depend on the depth of the soil element in the silt layer and the distance from the centerline of the structure. Application of an ice load further complicates the analyses by producing large rotations in the direction of the major principal stress within most of the foundation soils. An assessment of the limiting equilibrium condition against horizontal sliding requires knowledge of the available strength for σ'_{1f} acting at $\delta = 45 + \phi'/2$ degrees to the vertical direction (Sauls et al.; 1984). The inadequacy of using CIUC tests to replicate the behavior of soil under this complex loading condition is obvious.

Other considerations in evaluating design strengths from laboratory tests are strain rate effects, test temperature, and procedures to minimize the adverse effects of sample disurbance. UUC tests usually shear specimens at strains of 60% per hour. Such rapid rates will often increase the measured strength by 20 $\pm$ 10% over values from tests conducted at strain rates of 1% per hour (Ladd et al., 1977). Standard practice is also to test specimens at room temperature whereas field conditions in the Arctic involve in situ temperatures around 0 °C, a difference which could also affect the measured strength. It is generally accepted that samples must be reconsolidated in the laboratory, both to obtain proper initial state of stress and to minimize sample disturbance effects.

Consideration of the above issues suggests the existence of considerable uncertainty in foundation designs developed based on past test procedures. The Center's objective is to provide practical ways to take these factors into account. Two design techniques, SHANSEP and Recompression, specifically address some of the difficulties associated with stress-strain-strength measurements in soils. This study of the behavior of Arctic silts was developed based on the test procedures advocated by these two approaches. The philosophy and techniques underlying both methods are described in the succeeding section.

2.5 EXPERIMENTAL APPROACH FOR STRENGTH TESTING

The SHANSEP procedure is a design methodology for evaluating the in situ stress-strain-strength properties of cohesive soil (Ladd and Foott, 1974). SHANSEP is an acronym for Stress History And Normalized Soil Engineering Properties. The basic steps used in applying this technique are outlined in Table 2-1. For overconsolidated deposits, soil specimens are K_0 consolidated into the virgin compression range and then unloaded prior to shear to obtain data as a function of overconsolidation ratio (OCR). The procedure assumes that mechanical overconsolidation produced in the laboratory will simulate in situ behavior even though the deposit may be overconsolidated due to other mechanisms. This laboratory reconsolidation technique is specifically aimed at minimizing the adverse effects of sample disturbance. (Note that the method is not applicable to deposits of cemented and highly structured clays.) By conducting different types of tests (triaxial extension, triaxial compression, and direct simple shear), this method can also be used to provide measurements of soil anisotropy. Fundamental to SHANSEP is the assumption that the soil exhibits reasonable normalized behavior. This requires that, for a particular value of OCR, identical stress-strain-strength characteristics result when normalized with respect to consolidation stress.

Figure 2-4 shows results obtained from applying the SHANSEP procedure to direct simple shear test data on six soils. The resulting relationship can be approximated by the expression:

$$c_u/\sigma'_{vc} = S\,(OCR)^m$$

where:

$S = c_u/\sigma'_{vc}$ for normally consolidated soil

OCR = overconsolidation ratio

m = 0.8 $\pm$ 0.05

c_u = undrained shear strength

σ'_{vc} = effective vertical consolidation stress

Once this relationship is established for a deposit, the in situ c_u profile can be computed based on knowledge of the in situ overburden stress and preconsolidation pressure. The normalized parameters thus provide a powerful design tool. SHANSEP has been successfully applied in other offshore areas (e.g. Ladd and Azzouz, 1983). At the start of this phase of research, available data were insufficient to determine if the requirement of normalized behavior would be satisfied for the case of Smith Bay Arctic silts.

Similar in philosophy to SHANSEP is the Recompression technique (Bjerrum, 1973). This method also recognizes the problems associated with sample disturbance and attempts to mitigate these effects by K_0 reconsolidating specimens to the in situ effective overburden pressure. It is not known to what degree the resulting volume decrease may affect the measured strength, particularly for low overconsolidated clays. The method is better suited for testing naturally cemented soils and for highly structured clays with high liquidity index and sensitivity (Jamiolkowski et al., 1985). For these types of deposits consolidating into the normally consolidated range, as would be done in the SHANSEP technique, would destroy the soil structure and seriously alter the normalized soil properties. The Recompression technique is also recommended for highly overconsolidated stiff clays and within weathered crusts where sample disturbance is likely to be less of a problem and where it may be difficult to perform SHANSEP tests at very high OCR values.

Initial efforts of the Center's experimental program were aimed at further developing detailed stress history profiles at the two Smith Bay sites. This information provides basic consolidation parameters, gives perspective on the magnitude of reasonable undrained strength values, and is one of the essential steps in applying the SHANSEP technique. Further tests at the Center have concentrated on evaluating the stress-strain-strength behavior of normally consolidated Smith Bay silt. The laboratory consolidation of specimens into the virgin compression range was done to allow comparison of the behavior of this deposit with that of other Arctic silts and also clays in general. The tests were run at different values of consolidation stress to check if the soil exhibited normalized behavior. Some SHANSEP type tests were run on mechanically overconsolidated specimens to measure the effect of OCR. Recompression tests had already been performed by TETC to provide a basis for comparison with the SHANSEP results. The combined testing program included three types of c_u strength tests (triaxial compression, triaxial extension and direct simple shear) to measure anisotropy.

Direct Simple Shear (DSS) tests composed a major portion of the MIT testing program. This type of test offers several advantages (Sauls et al., 1984):

1. The horizontal failure mode of shearing is especially relevant to the ice loading condition.

2. The test is easier and quicker to perform than K_0 consolidated undrained triaxial tests.

3. The test requires less soil than triaxial tests, a very important consideration due to the limited supply of undisturbed material available for testing at MIT.

4. The measured undrained shear strength generally gives reliable to somewhat conservative estimates of the in situ undrained strength appropriate for undrained stability and bearing capacity analysis.

The SHANSEP technique has been successfully applied to major projects both on land and offshore. The method is relatively expensive and time consuming. The benefit is that the Normalized Soil Parameters, once determined, can be reused in analyzing different types of stability problems, and at other sites having similar Arctic silt deposits. Little reliable geotechnical data are available from earlier test programs in the Arctic. In such a situation, development of a comprehensive method for evaluating soil strength as achieved through a SHANSEP type program should prove particularly useful and cost effective, if the technique is found to apply in a reasonable fashion.

Table 2-1: Basic Steps in Application of the SHANSEP Design Procedure for Estimating the Initial In Situ Undrained Strength Profile

1. Subdivide the soil deposit into representative layers based on boring logs, in situ testing, index properties etc.

2.Develop the "best estimate" and range in the stress history profile using a combination of lab consolidation tests to measure σ'_p results from the in situ tests (e.g. field vane and/or piezo-cone penetrometer) and knowledge of the local geology.

3. Decide what types of laboratory CK_0 shear tests best model the field stress conditions and the range of OCR values for which normalized soil properties are required. (Note: MIT includes direct simple shear tests in all offshore programs since they require the least amount of soil and yield average strengths appropriate for stability analysis.

4. Perform the CK_0U test program, first checking that normalized behavior applies by varying the ratio σ'_{vc}/σ'_p and then determining the influence of overconsolidation ratio.

5. Compare the results of step 4 to data for other deposits of similar geological composition and then select the best estimate and range in the NSP versus OCR relationship.

6. Apply the NSP relationships to the in situ stress history. For example at any given depth:

$$c_u = \sigma'_{vo} \cdot c_u/\sigma'_{vc}$$

corresponding to the OCR = σ'_p/σ'_{vo} at that depth.

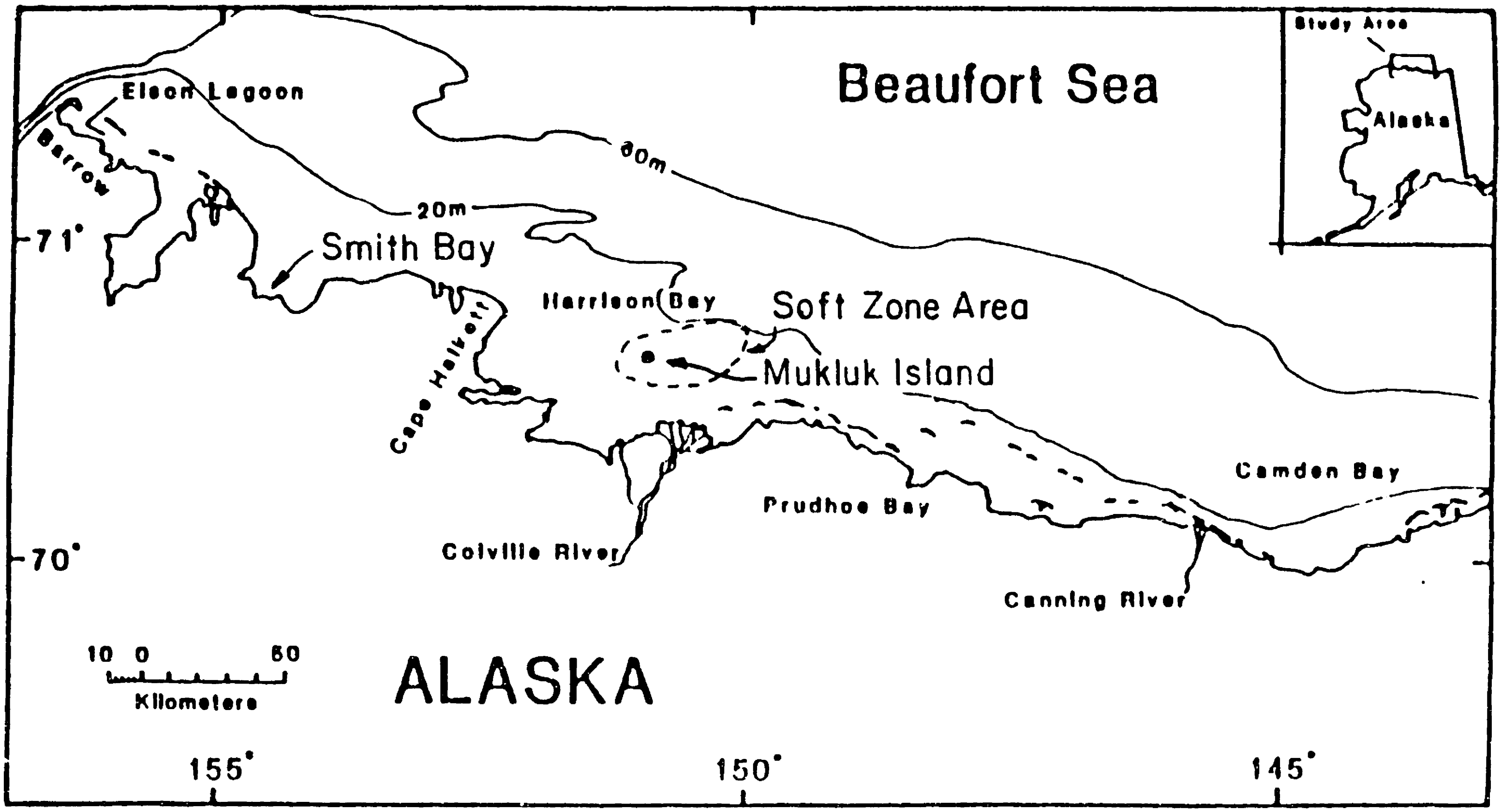


Figure 2-1: Location of Smith Bay in the Beaufort Sea

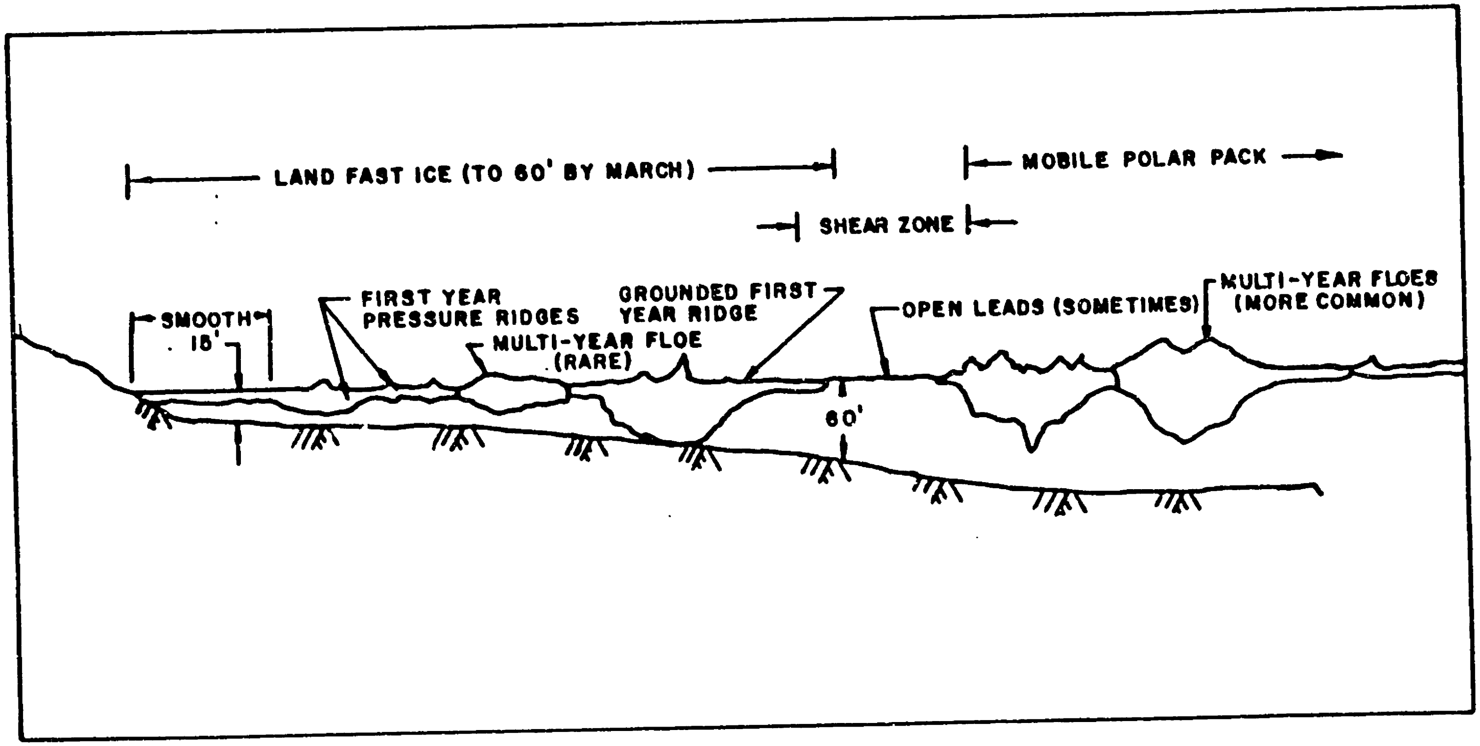


Figure 2-2: Typical Ice Features in the Beaufort Sea
(after Croasdale and Marcellus, 1978)

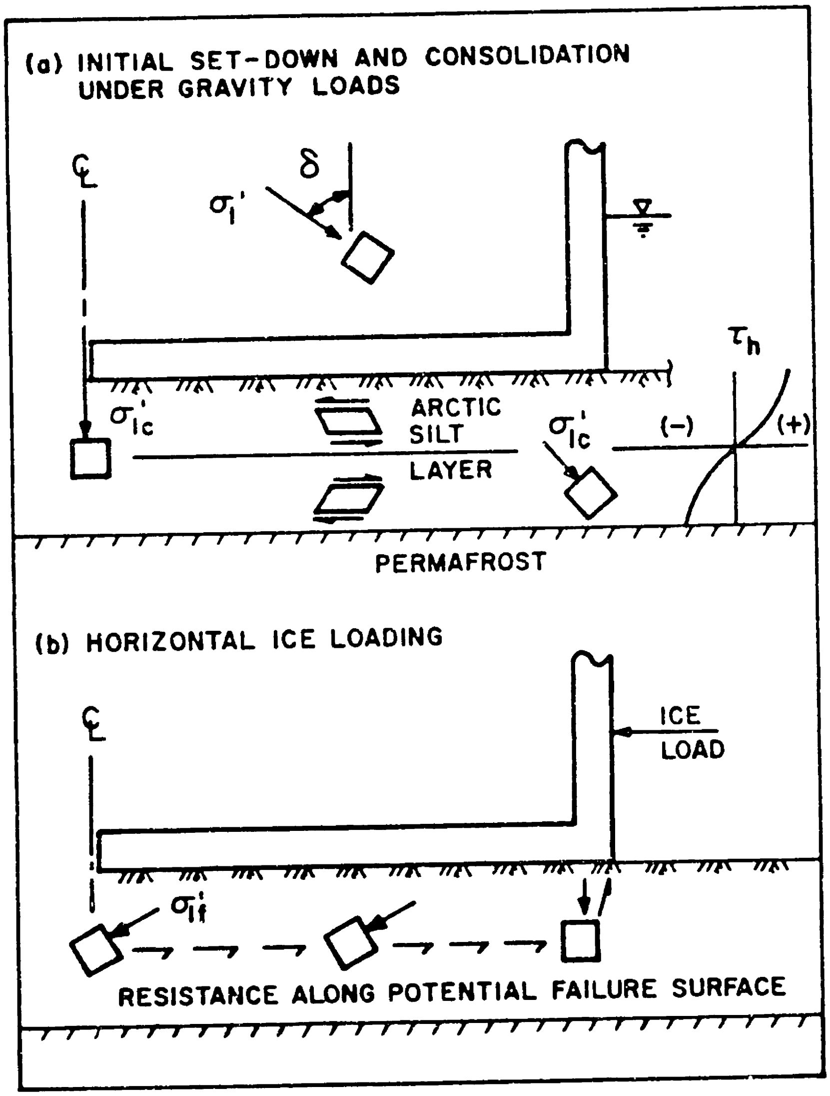


Figure 2-3: Simplified Illustration of Complex Stress Conditions within Foundation Soils for Offshore Gravity Structures (after Ladd, 1984)

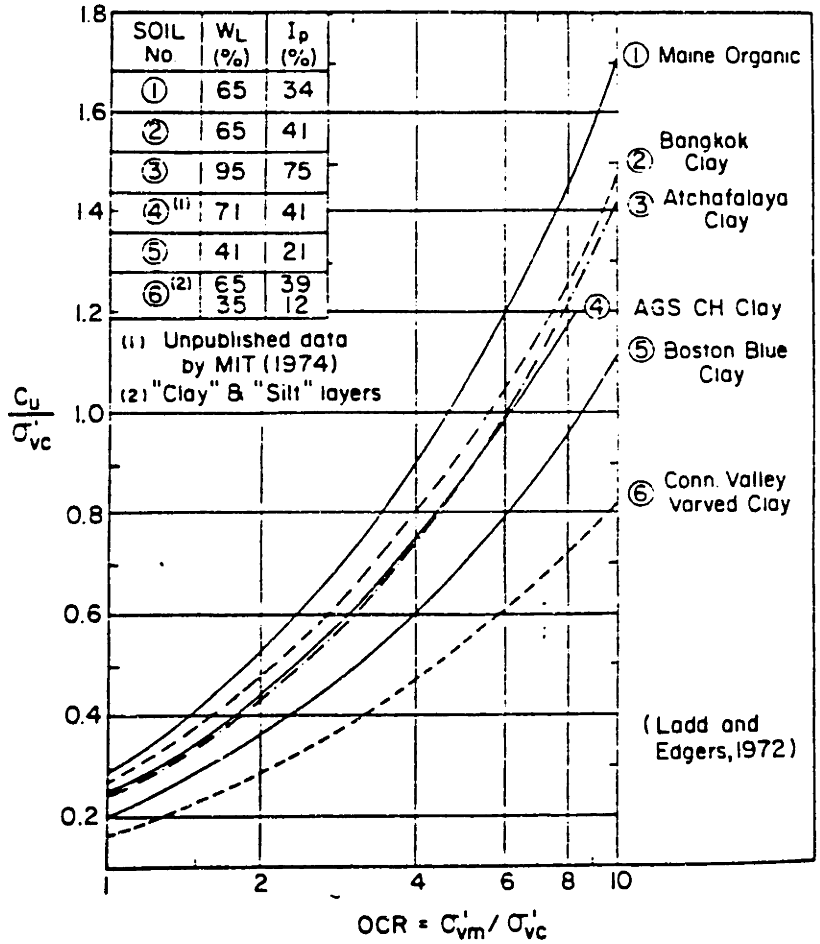


Figure 2-4: Undrained Strength Ratios versus OCR from CK_0UDSS
Tests on Six Clays (Ladd et al., 1977)

Chapter 3

SAMPLES AND SCOPE OF EXPERIMENTAL WORK

3.1 FIELD TESTING AND SAMPLING PROGRAM

3.1.1 General

The field testing and sampling programs were performed by The Earth Technology Corporation (TETC) in early 1985 under the general supervision of Drs. Jeff Weaver and Steve Law of the Dallas, Texas, SOPC Arctic Technology Division. A total of 34 samples (50 linear feet) were recovered from three boreholes at Site W and 37 samples (49 linear feet)were recovered from four boreholes at Site T. Details of the exploration program are taken from proprietary reports provided to MIT. The field program at the two sites, shown in Figure 3-1, consisted of (TETC ,1985):

1. One central deep soil boring drilled and sampled to 150 ft. At site T, and additional boring was drilled adjacent to this one (5B1).

2. Two peripheral shallow soil borings drilled and sampled to 50 ft (NW and SE corners of the site)

3. Five thermal piezocone penetration tests (at center and at each corner of the site)

4. In-situ vane shear tests adjacent to the center boring

5. Pressuremeter testing (Site T)

6. Installation of one thermistor string

7. Lead line bathymetric survey for seafloor topography

The Earth Technology Corporation (TETC) supplied personnel and communication equipment. Foundex Alaska of Anchorage was the drilling sub-contractor and provided drilling personnel, a drilling supervisor, a custom built HT-700 rotary wash drilling rig and the sampling equipment. Oceaneering International Inc. was contracted by TETC to provide positioning services in order to locate and mark the sites for the boreholes and bathymetric surveys. The operation was supported by a B-212 helicopter contracted from Air Logistics Inc.; the helicopter was used for crew transport between the sites and the NARL base camp at Barrow, Alaska. On site polar bear monitoring was provided by Polar Bear Monitoring Services of Fairbanks, Alaska.

The test program was conducted off the ice at Site W; all drilling, sampling and in-situ testing were done using a Rolligon (balloon-tired tractor). However, difficult ice conditions required that operations be converted to a heliportable mode for Site T.

3.1.2 In Situ Testing

The piezocone penetrometer tests (PCPT) consist of simultaneous and continuous recordings of the tip resistance, skin friction and pore water pressure. The piezocone penetration soundings were performed using a 60° conical tip with a 15 sq cm area followed by a 200 sq cm friction sleeve. Pore pressures were measured at the mid-height of the cone. Temperatures were also recorded during penetration by adding additional sensors to the instrument.

Reaction force for penetration was provided by the drill rig weight and by the Rolligon weight. Lateral support to prevent buckling of the drill string was provided by a specially designed lightweight 6-inch winged casing. The probe was penetrated at a rate of 2 cm/sec to the impenetrable interface. A break of 15 seconds was required every 3 ft for addition of a new drill rod. Prior to each sounding, the pore pressure sensing system was saturated with a glycol mixture using Earth Technology's standard procedure.

In situ vane tests were performed in unfrozen fine-grained surface soil at both sites using Acker Vane Shear Test equipment, deployed through the drill casing. A series of torque rods, with the vane mounted at the tip, were lowered to the bottom of the casing, and the blades were then twisted by applying torque at the surface. A surface torque head allowed the measurement of torque versus angular displacement during the vane test.

The pressuremeter used in this study was of the self-boring type. The instrument consists of a thick-walled hollow tube which is pushed into the soil, below the tip of the drill casing using the BQ drill rods. The soil displaced by the instrument is removed by pumping high-pressure mud through jets located on the central rod just inside the instrument cutting shoe. When the instrument is at the desired test depth, the penetration is stopped and the mud pump turned off. The test is then performed within minutes of reaching the desired depth level. Upon completion of the test, the pressuremeter string may be tripped out of the hole, and then the pressuremeter is lowered again. In soft soil, multiple tests can be performed during one push, depending only on how far the probe can be pushed. The tests were performed by applying gas pressure at a constant rate to the inside of the flexible membrane on the probe. The radial displacement at three central locations, the total pressure inside the probe and the pore pressure on the surface of the membrane were measured electrically and sent to the surface for display and recording. Results of pressuremeter tests will not be discussed further.

3.1.3 Sampling and Related Testing

High quality 3-inch diameter Acker type fixed piston samples were obtained at three foot intervals to a depth of 14 ft and at 10 ft intervals thereafter. A thin walled push tube sampler was used in stiff soil or poorly bonded permafrost. Where dense coarse grained soil or well bonded permafrost was encountered, a thick walled drive sampler was used to obtain disturbed samples.

Drilling and sampling involved advancing the borehole to the sampling interval and washing cuttings out using seawater. The insert center bit was then tripped out by wire line to permit sampling through the end of the drill string. In unfrozen soils, the fixed piston sampler, fitted with a 3-inch Shelby tube, was lowered down the drill string. When the sample tube reached the bottom of the borehole, it was hydraulically pushed into the soil while holding the piston stationary from the suface by means of center rods.

All recovered samples were immediately examined in the field laboratory by a geotechnical engineer or engineering geologist, who made strength index measurements using pocket penetrometer, Torvane and miniature vane tests. Sample temperatures were measured using a hand-held thermistor probe. The end portions of the samples were visually classified according to the Unified Soil Classification system (ASTM D2487) prior to packaging for shipment. Samples from Boring 1B (all depths at Site W; RE > 7.5 ft at Site T) were extruded in the field and visually classified. Cores were split with a chisel, photographed and observed at 0 °C. These samples were not tested further.

Procedures used in packaging and shipping of frozen and unfrozen soil samples were designed to minimize mechanical and thermal disturbance. The samples were sealed using mechanical O-ring packers, then wrapped in bubble-pack for mechanical cushioning and foam rubber for thermal insulation. Wrapped, unfrozen samples were placed in a large ice chest packed with warm snow at a temperature slightly above 0 °C. Frozen samples were stored in a separate ice chest maintained at a sub-freezing temperature. The ice chests were shipped by priority air freight to The Earth Technology Corporation (Long Beach, California). Each chest was appropriately labelled as to temperature control required during transit and upon delivery. During subsequent shipment to MIT, the remaining tubes were wrapped with bubble pack and foam rubber and were packed with a number of blue gelatin ice packs to keep the temperature just above 0 °C. At MIT the samples were kept in a freezer near 0 °C until ready to test.

3.2 SCOPE OF LABORATORY TESTING PROGRAM

Laboratory testing was performed by both TETC and MIT; the location and type of tests performed at Sites T and W are summarized in Tables 3-1 and 3-2, respectively. A brief soil description is also included. The locations of samples are listed in terms of Relative Elevation (RE) in order to preserve the proprietary nature of the results presented. Relative Elevation is defined for each site, with RE = 0 for the boring having the highest elevation. The RE of the mudline for each boring at Site T is: 3 ft (Boring 1B); 0.5 ft (Boring 3B); 1.0 ft (Boring 5B); and 0.0 ft (Boring 5B1). At Site W, mudline elevations are at RE of 0.9 ft (Boring 1B), 0.5 ft (Boring 3B) and 0.0 ft (Boring 5B).

TETC radiographed all samples in steel sample tubes. However, lack of shielding of the sides of the tubes caused "burnout" (overexposure to x-rays), resulting in poor quality radiographs. Classification and index tests performed by TETC included: water contents, Atterberg limits, total unit weights and salt concentrations. Index strength tests included 21 Unconsolidated-Undrained triaxial tests (UUC) (10 at Site T, 11 at Site W). A series of oedometer tests (12 at Site T, 11 at Site W) were performed at 1 °C in order to determine a stress history profile for each site. Strength tests of the CK_0U-Recompression type included: six triaxial compression (two Site T, four Site W); two triaxial extension (one at each site); and five direct simple shear (two at Site T, three at Site W). The locations of all the above tests are included in Tables 3-1 and 3-2.

Following completion of TETC's testing program, all remaining soil was shipped to MIT (12/85-1/86). MIT received undisturbed soil in 3-inch diameter steel tubes; glass jars and plastic bags contained disturbed soil. All soil was stored near 0 °C to minimize thermal disturbance. Upon receipt at MIT, each sample was radiographed to assess sample quality. Comments on the radiographs are included in Tables 3-1 and 3-2.

Little undisturbed soil was available to MIT from the upper 20 ft of soil at the two sites. Soil from Site T consisted of eleven tube samples of variable length from 3 borings: one from Boring 1B, six from Boring 3B and four from Boring 5B1. Of the 7.9 linear feet of soil available to MIT, about 4.5 feet was considered suitable for testing (with some reservations due to the occurrence of gravel in most of the samples). Only eight samples from two borings were available for Site W, over a range of depths from 2 to 17 feet. Of the 6.6 linear feet supplied to MIT, about 3.6 feet could be considered suitable for testing.

MIT performed grain size analyses and Atterberg limits on material adjacent to that used in the consolidation and strength tests to supplement the index and classification tests run by TETC.

The location of all consolidation and strength tests performed by MIT on samples from Sites T and W is shown in Tables 3-1 and 3-2. MIT's test program included oedometer, CK_0U direct simple shear and CU triaxial compression tests. Conventional incremental oedometer tests were conducted at room temperature in order to supplement TETC's stress history and compressibility data (seven at Site T, one at Site W). Most tests were run on Site T material because of the complexity of that site as a result of ice gouging. Also, Site T was the location of a design for a large mobile drilling structure. A temperature controlled oedometer was run at Site T in order to measure the sensitivity of the preconsolidation pressure and compressibility to changes in ambient temperature.

A total of ten CU triaxial compression tests were performed by MIT on soil from Site T to evaluate stress-strain-strength properties and anisotropy of the deposit. Six were Recompression tests to supplement tests performed by TETC. Three isotropically consolidated undrained compression (CIUC) tests were performed at the request of EBA (the design consultant) at OCR = 1, 2 and 5. One K_0-consolidated, undrained compression test was run at OCR = 1 on the gouged portion of Site T for comparison with the CIUC data. Twenty CK_0U direct simple shear tests were performed on normally consolidated and overconsolidated samples (11 at Site T, 9 at Site W). These tests were run on both gouged and nongouged material to determine normalized stress-strain-strength properties as part of the SHANSEP technique for estimating design parameters, especially regarding resistance to base sliding under ice loading conditions.

3.3 SAMPLE CHARACTERISTICS

3.3.1 Radiography

Samples sent to MIT were radiographed using the equipment and procedures described in detail by Sauls et al. (1984). Good quality radiographs were obtained using the following procedure:

Exposure Time = 5 minutes
Input Voltage = 160 r. V
Input Current = 3.8 mA
Distance from x-ray = 6 ft
Developing time = 7 minutes

Approximately 35 radiographs were taken of the 16 tubes. Each tube was x-rayed 10 in. at a time then returned to the freezer. In cases where MIT received partial tubes, the x-rays were compared with those provided by TETC in order to help determine the exact depth of the sub-sample within the boring sample.

Radiographs of Smith Bay samples revealed the following: uniform versus layered macrofabric; disturbance due to sample disturbance and/or ice gouging; and the presence of gravel. Most samples at Site T were layered and contained scattered pebbles (Figure 3-2). Occasionally shells were also present. There was also some evidence of inclined bedding planes, the result of ice gouging. The soil at Site W exhibited slight layering, but was otherwise uniform. Less gravel was observed compared to Site T (Figure 3-3).

3.3.2 Index Properties

Classification and index properties of samples from Sites T and W are presented in Tables 3-3 and 3-4, respectively. Also indicated on the tables is the source of the data. Figure 3-4 presents total unit weight (γ_t) versus depth for Sites T and W. Unit weights increase with depth at Site T, from about 110 pcf to 130 pcf at a depth of 14 ft. Values are more uniform at Site W, equalling 120.9 pcf $\pm$4.21 SD.

Figures 3-5 and 3-6 present water content, Atterberg limit and liquidity index data versus depth for Sites T and W, respectively. Both TETC and MIT data are included in these plots. The water content at Site T decreases from 0.4 to 0.8 at the surface to a value of 20% at a RE of 14 ft. The wide range in water content at RE of 7.5 to 8 ft might be the result of ice gouging. Site W exhibits a relatively uniform water content of 29.8% $\pm$2.7 SD below a depth of 2 to 3 ft. Two borings had a very thin surface layer of soft Holocene clay with a slightly higher water content of 35-42%.

At Site T, the liquidity index is scattered over the upper eight feet of the deposit, ranging from 40-80% and decreasing with depth. At greater depths, the liquidity index decreases with depth and equals 0.24 $\pm$0.17 SD. At Site W, the liquidity index changes less rapidly with depth and is equal to 0.22 ± 0.12 SD (excluding the Holocene layer). The combined plot of liquidity index versus depth (Figure 3-7) shows convergence of values for the lower portion of Site T (less gouged) with those for Site W (nongouged).

All Atterberg limits from Sites T and W plot on or above the A-line on the plasticity chart (Figure 3-8). The Site T and W deposits are therefore CL-CH silty clays rather than the ML-MH clayey silts mainly found within the Soft Zone Area of Harrison Bay [Note: essentially all of the soil tested by the Center from the Mukluk Proximal sampling program plotted below the A-line (Ayan, 1985; Yin, 1985)]. The plasticity index for both sites is plotted versus RE in Figure 3-9. The plasticity index at Site T decreases with depth, and equals 24.95% ± 5.75 SD for RE less than nine feet, and 18.7% ± 2.1 SD for RE greater than nine feet. Site W is more uniform, with a plasticity index of 23.1% ± 2.8.

MIT performed hydrometer analyses on 16 samples (11 from Site T, 5 from Site W). The resulting grain size distribution curves are summarized in Figure 3-10. All Site T samples fall within the indicated range, with 33-49% clay size. A range for Site W is also indicated, with clay contents of 49-57%. One Site W test (indicated WDSS2 in Figure 3-10) clearly plots in the middle of the Site T range. This was a Holocene sample from shallow depth (RE = 3.4 ft) and is therefore not representative of Site W nongouged material.

Figure 3-11 plots plasticity index versus clay fraction. The small range of values of plasticity index at Smith Bay precludes a definitive correlation. The linear regression line through Harrison Bay data is drawn for comparison, along with individual points. The Smith Bay values fall well below the Harrison Bay line established for ML-MH Arctic silts, which spanned a much larger range of plasticity index.

Figure 3-12 shows the clay fraction and activity versus depth for both sites. Clay fraction at Site T is essentially constant and equal to 42% $\pm$4.34 SD. Site W is also very uniform, except for an upper Holocene sample which is more silty. The remaining values are very consistent and equal to 54.0% $\pm$3.6 SD. Activity decreases slightly with depth at Site W and is equal to 0.44$\pm$0.06 SD, excluding the surface sample. Activity at Site T is equal to 0.52$\pm$0.08.

Salt concentration and temperature for both sites are plotted versus depth in Figure 3-13. Salt concentration is almost constant at both sites (except for one outlier at Site W). Concentration (in g/l) at Site T equals 31.3$\pm$3.34, and at Site W is 30.06$\pm$1.60. Temperature at Site T is fairly constant, with an average value of 30.2 °F $\pm$1.1 SD. Temperatures at Site W are generally higher except for the surface layer, which agrees very well with Site T values. The average temperature of the deposit is 31.66°F $\pm$1.25 SD.

3.3.3 Strength Index Tests

Tabulated data for field vane and UUC tests are presented in Tables 3-5 and 3-6, respectively. UUC strengths (obtained at 1°C and $\dot{\epsilon}$ = 1%/min) were generally taken at the end of the test and therefore may not be peak values. On the following plots, values representing a "minimum" estimate of strength are indicated by arrows. Results of other strength index tests (miniature vane, Torvane, and pocket penetrometer) are presented in Tables 3-7 and 3-8.

Results of all strength index tests are plotted versus Relative Elevation for Site W in Figure 3-14. Very low strengths measured in the upper layer of Holocene material (RE less than 2-3 ft) will not be considered in the following discussion. Pocket penetrometer and Torvane strengths show large scatter, from less than one to over four ksf for all RE. The field vane, miniature vane and UUC tests are much more consistent and equal 2.58 ksf $\pm$0.58 SD (excluding Holocene values). Linear regression of the field vane, UUC and minivane strengths produces the line shown.The Torvane values consistently underestimate the linear regression strength, and the pocket penetrometer generally overestimates this strength.

The results of strength index tests for Site T are shown in Figure 3-15. The soil at RE less than 8 ft is much weaker than the remainder of the deposit, presumably due to ice gouging. The field vane, minivane and UUC tests appear to give the most consistent results. Linear regression on these strength results for RE less than 9 ft with strength less than 1.5 ksf gives the line shown. The dashed line represents the linear regression for values with RE greater than 7 ft with strengths greater than 1.5 ksf. Comparison of the two linear regression lines shows significant difference in the rate of strength increase with depth for the upper gouged layer versus the lower, less gouged material.

Table 3-1: SAMPLE CHARACTERISTICS AND SCOPE OF ENGINEERING TESTS: SITE T

E = Earth Technology M = MIT

Boring	Sample No.	Relative Elev. (Ft.)	Description: Field	Description: Radiography (1)	Length (In.) Recovered / Sent to MIT	STD (2) Class.	UUC	Consol. (3) 1° C	Consol. (3) RT	Consol. (3) ΔT	Recompr. CU TC	Recompr. CU TE	Recompr. CU DSS	SHANSEP CK_oUDSS (4) No. of Tests at Given OCR	Other CU TC Tests Type of Test & Nominal OCR
1B	O1	3-5.1	SILTY CLAY (CL) gray, laminated, soft		22?/0	E	E	E							
	O2	6-7.5	trace shells, gravel	L, P	22?/8.5	E,M	E	E		M				3 @ OCR=1	
	P3	9-10.5	SILTY CLAY (CL) gray, stiff		17/0	E									
	P4	12-13			12/0	E									
	S5	15-16	trace sand, shells		12/0	E									
3B	P1	1.5-3.5		L, P	24/6	E	E	E							
	P2	4.5-6.5		L, P	24/2.75,7.5	E,M	E	E	M				E	1 @ OCR=1 1 @ OCR=8.5	
	P3 (T)	8.2-8.8	SILTY CLAY (CL) gray, laminated, soft	L, P	24/5	E,M	E	E					M	2 @ OCR=1 1 @ OCR=8	
	P3 (B)	8.9-9.4		P		E	E								
	P4	10.5-12.1		P	17/10	E,M		E	M						
	S5	13.5-14.8		P	15/3.75	E		E							

(1) L = Layered, P = Pebbles, S = Shells, U = Uniform
(2) w_N, γ_t, Atterberg Limits, salt concentration by Earth Technology
w_N, Atterberg Limits, grain size and/or G_s by MIT
(3) 1°C = Nominal temperature by Earth Technology
RT = At room temperature by MIT
ΔT - Variable temperature by MIT
(4) Run by MIT

Table 3-1 (cont.): SAMPLE CHARACTERISTICS AND SCOPE OF ENGINEERING TESTS: SITE T

E = Earth Technology   M = MIT

Boring	Sample No.	Relative Elev. (Ft.)	Description: Field	Description: Radiography (1)	Length (In.) Recovered / Sent to MIT	STD(2) Class.	UUC	Consol.(3) 1° C	Consol.(3) RT	Consol.(3) ΔT	Recompr. CU TC	Recompr. CU TE	Recompr. CU DSS	SHANSEP CK_oUDSS (4) No. of Tests at Given OCR	Other CU TC Tests Type of Test & Nominal OCR
5B	P1	1-2.5		L, P	18/0	E	E	E							
	P2	4-6	SILTY CLAY (CL) gray w/ shell fragments, bedded, soft	L, P	24/0	E	E	E			E				
	P3 (T)	7-7.5		L	20/0	E	E	E					E		
	P3 (B)	7.8-8.4		P		E	E	E							
	S4	10-12	SILTY CLAY (CL) gray, horiz. bedding, trace coarse sand, very stiff		5/0	E									
	S5	13-14.5		P, L	18/0	E	E	E							
5B1	P1	1-3	SILTY CLAY (CL) gray, soft to stiff w/ silt layers	L, P, S	22/18	M					M				CIUC @ OCR=1,2 CK_oUC @ OCR=1
	P2	4-5.9		L, P	22/12	E						E			CIUC @ OCR=5
	P3	7-8.7		L, P	20/2.5,9	E,M			M(2)		E,M			1 @ OCR=16 2 @ OCR=1	
	S4	10.4-11.3	trace gravel becomes hard	L, P	13/10	M			M(2)		M(2)				

(1) L = Layered, P = Pebbles, S = Shells, U = Uniform
(2) W_N, γ_t, Atterberg Limits, salt concentration by Earth Technology
W_N, Atterberg Limits, grain size and/or G_s by MIT
(3) 1°C = Nominal temperature by Earth Technology
RT = At room temperature by MIT
ΔT - Variable temperature by MIT
(4) Run by MIT

Table 3-2: SAMPLE CHARACTERISTICS AND SCOPE OF ENGINEERING TESTS: SITE W

E = Earth Technology M = MIT

Boring	Sample No.	Relative Elev. (Ft.)	Description: Field	Description: Radiography (1)	Length (In.) Recovered / Sent to MIT	STD(2) Class.	UUC	Consol.(3) 1° C	Consol.(3) RT	Consol.(3) ΔT	Recompr. CU TC	Recompr. CU TE	Recompr. CU DSS	SHANSEP CK_0UDSS (4) No. of Tests at Given OCR
1B	S1	1.4-1.9	SILTY CLAY (CL) gray firm to hard w/shell		14/0	E								
	S2	4.9-5.4	fragments		20/0	E								
	S3	7.9-8.4	occas. trace gravel		20/0	E								
	S4	9.9-11.5			18/0	E								
	S5	12.9-14			15/0	E								
	S7	17.9-19.1			14/0	E								
3B	P1	0.9-2.8	SILT(ML) gray loose w/clayey silt		20/0	E	E	E			E		E,M	
	P2	2.9-4.1	SILTY CLAY (CL) gray, stiff		15/0	E	E	E						
	P3	6.9-8.8	pebbles		24/4	E	E	E			E		E,M	
	S4	9.9-11.4		U, P	18/6	E	E	E						
	O6	12.9-13.8			11/0	E								
	S6	15.9-17.7	(trace chert pebbles)	U, P	15/9	E	E	E						

(1) L = Layered, P = Pebbles, S = Shells, U = Uniform

(2) W_N, γ_t, Atterberg Limits, salt concentration by Earth Technology
W_N. Atterberg Limits, grain size and/or G_s by MIT

(3) 1°C = Nominal temperature by Earth Technology
RT = At room temperature by MIT
ΔT = Variable temperature by MIT

(4) Run by MIT

Table 3-2 (cont.): SAMPLE CHARACTERISTICS AND SCOPE OF ENGINEERING TESTS: SITE W

E = Earth Technology M = MIT

Bor-ing	Sample No.	Relative Elev. (Ft.)	Description		Length (In.)	STD(2) Class.	UUC	Consol.(3)			Recompr. CU			SHANSEP CK_0UDSS (4)
			Field	Radiography (1)	Recovered / Sent to MIT			1° C	RT	ΔT	TC	TE	DSS	No. of Tests at Given OCR
5B	P1	1-2.9	SILTY CLAY (CL) black, firm	L	23/21.5	E		E			E	E		
	PV-S1	3-4.7		L	23/21.5	M								1 @ OCR=1
	P2	3-5	SILTY CLAY (CL-CH) gray, firm-hard	U	13/0	E	E	E						
	P3	7-9		U	24/3.5	E,M	E	E			E		E	1 @ OCR=1
	P4	9-10.9		L, P	23/10.5	E,M	E	E						3 @ OCR=1 1 each @ OCR=5,10,20
	P5	13-14.9		L, P	22/10	E,M	E	E						1 @ OCR=1
	S6	16-18.2			26/15	E	E	E	M					

(1) L = Layered, P = Pebbles, S = Shells, U = Uniform
(2) w_N, γ_t, Atterberg Limits, salt concentration by Earth Technology
w_N. Atterberg Limits, grain size and/or G_s by MIT
(3) 1°C = Nominal temperature by Earth Technology
RT = At room temperature by MIT
ΔT = Variable temperature by MIT
(4) Run by MIT

Table 3-3: CLASSIFICATION AND INDEX PROPERTIES OF SITE T SAMPLES

Boring No.	Sample No.	RE (ft.)	w_N (%)	γ_t (pcf)	w_L w_p (%)	I_p I_L (%)	Clay Fraction (%)	Salt Conc (g/l)	In Situ Temp (°F)	Tested By
1B	O1	3-5.1	45.1	108.0	58 24	31 68	-	28	30.1	E
	O2	6-7.5	36.3	119.0	51 25	26 44	-	29	30.1	E
	O2	7.0	41.5	-	52.7 27.6	25.1 55.4	43	-	-	M
	P3	9-10.5	23.1	-	38 20	18 17	-	33	33.9	E
	P4	12-13	23.4	-	39 20	19 18	-	34	30.5	E
	S5	15-16	17.8	-	33 17	16 5	-	34	-	E
3B	P1	1.5-3.5	42.9	118.0	48 23	25 79.5	-	28	30	E
	P2	4.5-6.5	35.0	118.7	46 24	22 50	-	30	29.9	E
	P2	4.65	36.3	-	50.2 26.2	24 42	41.5	-	-	M
	P3(T)	8.2-8.8	51.1	105.5	- -	- -	-	-	29.6	E
	P3(T)	7.6	39.0	-	35.3 23.2	12.1 130(?)	33	-	-	M
	P3(T)	7.9	54.4	-	65.8 32.7	33.1 65.5	49	-	-	M
	P3(B)	8.9-9.4	27.4	128.5	40 20	20 37	-	33	-	E
	P4	10.5-12.1	24.9	126.8	41 19	22 27	-	33	29.6	E
	P4	11.0	22.0	-	40.6 21	19.6 5	42.5	-	-	M
	S5	13.5-14.8	20.02	130.7	35 20	15 0		33	30.5	E
5B	P1	1-2.5	42.7	113.4	49 24	25 75	-	29	29.5	E
	P2	4-6	38.5	116.2	63 28	35 30	-	26	29.9	E
	P3(T)	7-7.5	50.9	104.3	57 27	30 80	-	28	29.7	E
	P3(B)	7.8-8.4	31.4	-	40 20	20 57	-	30	-	E
	S4	10-12	24.4	115.9	39 20	19 23	-	34	30.4	E
	S5	13-14.5	19.4	133.1	36 18	18 8	-	39	29.7	E
5B1	P1	1.9	40.3	-	49.5 25.6	23.9 61.5	48	-	(29.6) E	M
	P2	4-5.9	30.1	123.5	- -	- -	-	-	-	E
	P3	7-8.7	34.9	118.8	- -	- -	-	-	-	E
	P3	7.05	37.4	-	47.3 26.8	20.5 51.5	40	-	-	M
	P3	8.2	33.8	-	45.2 23.7	21.5 47	44	-	-	M
	P3	8.4	32.9	-	46.3 25.2	21.1 36.5	39	-	-	M
	S4	10.5-11	22.3	-	39.5 19.0	20.5 16.1	42	-	-	M

Table 3-4: CLASSIFICATION AND INDEX PROPERTIES
OF SITE W SAMPLES

Boring No.	Sample No.	RE (ft.)	w_N (%)	γ_t (pcf)	w_L w_P (%)	I_P I_L (%)	Clay Fraction (%)	Salt Conc (g/l)	In Situ Temp (°F)	Tested By
1B	S1	1.4-1.9	35.3	111.6	47 25	22 46.8	-	32	30.1	E
	S2	4.9-5.4	28.6	121.7	55 28	27 2.2	-	32	30.8	E
	S3	7.9-8.4	29.1	119.5	46 23	23 26.5	-	32	31.6	E
	S4	9.9-11.5	29.2	123.6	48 25	23 18.3	-	29	31.8	E
	S5	12.9-14	27.3	-	48 25	23 10	-	28	30.5	E
	S7	17.9-19.1	29.4	-	45 20	25 37.6	-	27	32.2	E
3B	P1	0.9-2.8	24.3	126.8	-	-	11	31	29.6	E
	P2	2.9-4.1	30.9	120.3	50 24	26 26.5	-	30	31.8	E
	P3	6.9-8.8	30.8	122.4	48 27	21 18.1	-	31	-	E
	S4	9.9-11.4	29.0	122.7	47 24	23 21.7	-	32	30.0	E
	O5	12.9-13.8	30.2	-	53 25	28 18.6	-	30	31.2	E
	S6	15.9-17.7	27.6	123.1	48 23	25 18.4	-	28	33.9	E
5B	P1	1-2.9	41.5	113.1	-	-	-	48	31.8	E
	PV-S1*	3.4	41.7	-	48.3 24.3	24 72.5	34	-	-	M
	P2	3-5	33.2	120.4	50 25	25 32.8	-	30	32.0	E
	P3	7-9	30.7	122.3	45 27	18 20.6	-	31	-	E
	P3	7	36.6	-	53.5 26.4	27.1 37.6	57	-	-	M
	P4	9-10.9	29.3	124.4	44 24	20 26.5	-	30	33.2	E
	P4	9.3	29.4	-	-	-	56	-	-	M
	P5	13-14.9	27.9	124.6	46 27	19 4.7	-	30	32.7	E
	P5	14.2	29.2	-	51.6 25.2	26.4 14.0	54	-	-	M
	S6	16-18.2	33.2	116.8	48 26	22 32.7	-	28	33.3	E
	S6	16.3	29.5	-	-	-	49	-	-	M

* Sample not listed by TETC; may be adjacent boring.

Table 3-5: FIELD VANE TESTS BY TETC

Site	Boring	RE(ft.)	c_u(ksf)
W	5B	1.25	0.006
		8.25	2.04
		12.5	3.00
T	5B	2.5	0.18
		4.5	0.52
		6.5	1.12
		8.5	2.10
		10.5	2.70

Table 3-6: UUC TEST DATA AT SITES W AND T
(From TETC, 1985)

Site	Boring	Sample	RE (ft.)	w_N (%)	σ_c (ksf)	ε_f (%)	q_f (ksf)
W	3B	P1	2.8	24.3	2.6	17.0	2.35
		P2	4.1	30.9	2.7	17.0	2.35
		P3	6.45	30.7	3.0	17.5	2.0
		S4	11.1	29.0	3.6	17.0	3.15
		S6	17.35	27.6	4.3	14.5	2.9
	5B	P2	4.5	33.2	2.9	17.0	1.9
		P3	8.6	30.7	3.3	17.5	1.9
		P4	10.4	29.3	3.6	17.1	2.95
		P5	14.35	27.9	4.0	17.0	4.15
		S6	17.55	33.2	4.5	17.3	2.15
T	1B	O1	4.7	45.1	4.2	$\geq$17.0	0.26
		O2	6.35	36.3	4.5	>17.0	>0.97
	3B	P1	3.0	42.9	4.3	15.0	0.40
		P2	5.5	35.0	4.7	>17.0	>1.05
		P3(T)	8.5	51.1	5.0	12.5	0.61
		P3(B)	9.1	27.4	5.0	>17.0	>1.51
	5B	P1	2.1	42.7	4.2	$\geq$17.0	$\geq$0.32
		P2	5.5	38.5	4.5	15.0	0.62
		P3(T)	7.25	50.9	4.8	12.0	0.41
		P3(B)	8.1	31.4	4.8	$\geq$17.0	$\geq$0.87
		S5	13.45	19.4	5.6	$\geq$17.0	$\geq$5.01

Table 3-7: STRENGTH INDEX TEST DATA
AT SITE W (From TETC, 1985)

Site	Boring	Sample	RE (ft.)	Shear Strength (ksf) Min Vane	Torvane	PP
W	1B	S1	1.4	-	1.2	0.7
		S2	3.6	-	0.6	3.0
		S3	7.4	-	3.8	4.3
		S4	10.9	-	1.4	2.4
		S5	14.4	-	0.8	1.9
		S7	18.9	-	1.8	2.8
	3B	P1	1.7	-	1.8	1.8
		P2	4.2	-	1.2	2.1
		P3	7.7	-	2.6	2.8
		S4	11.7	-	2.1	-
		D5	13.4	-	-	2.4
		S6	17.2	-	2.3	2.5
	5B	P1	1.95	0.92	-	-
		P1	2.0	-	1.1	-
		P2	3.0	-	2.0	2.3
		P2	4.55	2.72	-	-
		P3	8.0	2.52	-	-
		P3	8.5	-	2.1	4.0
		P4	9.95	2.48	-	-
		P4	10.5	-	2.2	-
		P4	11.0	-	-	4.0
		P5	13.5	-	1.9	3.0
		P5	13.95	3.24	-	-
		S6	17.0	-	1.1	2.0
		S6	17.1	2.34	-	-
		S7	19.65	2.26	-	-
		S7	20.0	-	1.5	2.0

Table 3-8: STRENGTH INDEX TEST DATA
AT SITE T (From TETC, 1985)

Site	Boring	Sample	RE (ft.)	Shear Strength (ksf) Min Vane	Torvane	PP
T	1B	O1	4.05	0.2	-	-
		O1	4.2	-	0.9	0.6
		O2	7.0	-	-	1.9
		P3	9.5	-	-	2.8
		P3	10.0	-	3.0	-
	3B	P1	2.5	0.42	-	-
		P1	3.5	-	0.6	0.4
		P2	5.5	1.0	-	-
		P2	6.5	-	0.8	0.9
		P3	8.5	2.0	1.8	2.0
		P4	11.3	4.2	-	-
		P4	12.0	-	1.4	-
		S5	14.0	-	-	3.6
	5B	P1	1.75	0.30	-	-
		P1	2.5	-	0.40	-
		P1	3.7	-	-	0.2
		P2	5.0	0.42	-	-
		P2	6.0	-	-	0.4
		P2	6.5	-	-	0.4
		P3	7.75	2.2	2.0	2.4
		S5	13.75	>4.2	-	-
		S5	14.0	-	2.4	2.6
	5B1	P1	2.0	<0.42	-	-
		P1	2.25	-	-	0.4
		P2	4.95	2.74	-	-
		P2	5.75	-	0.8	1.2
		P3	7.85	1.78	-	-
		S4	9.0	-	1.4	1.8
		S4	10.5	-	2.8	3.4

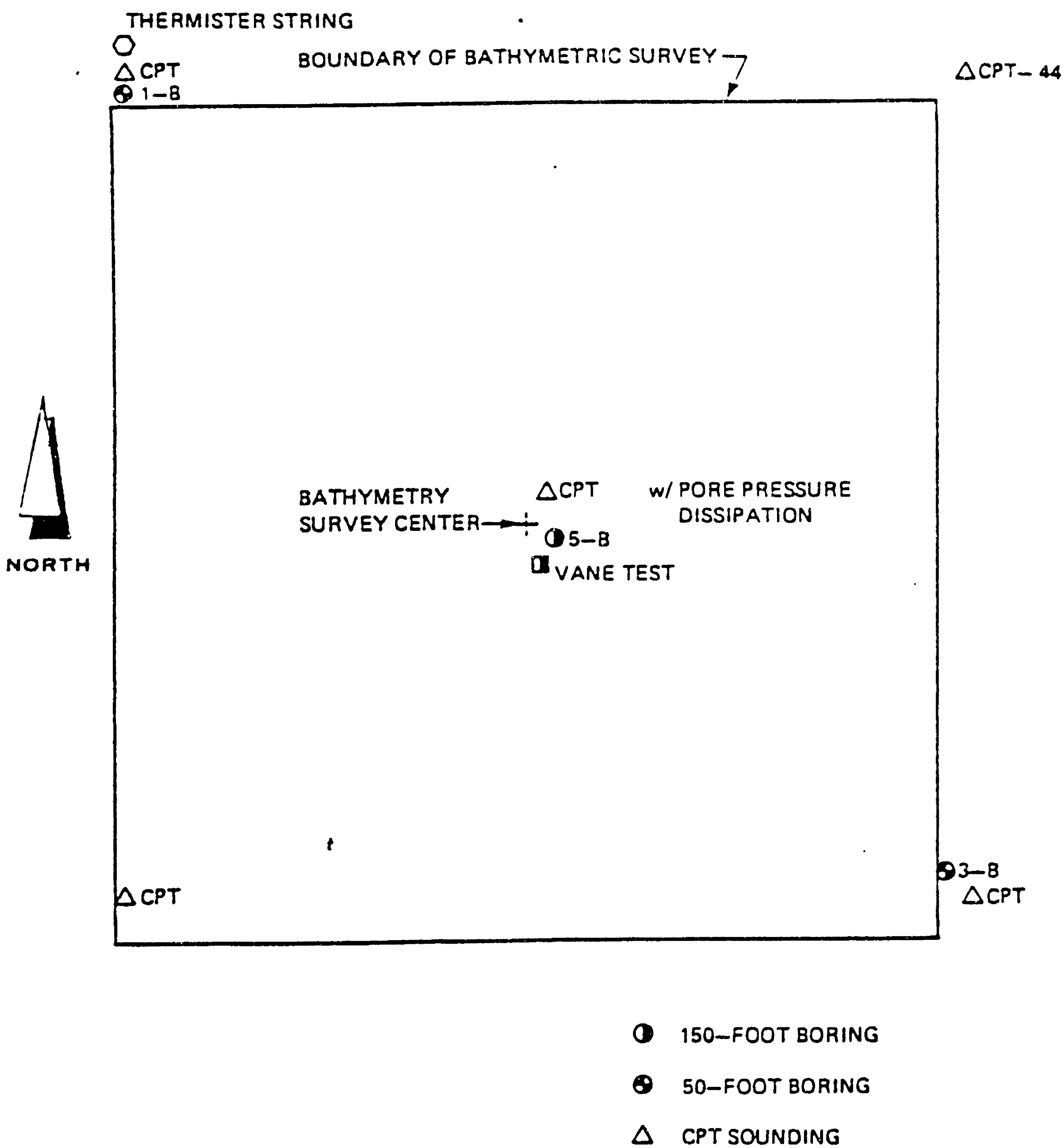
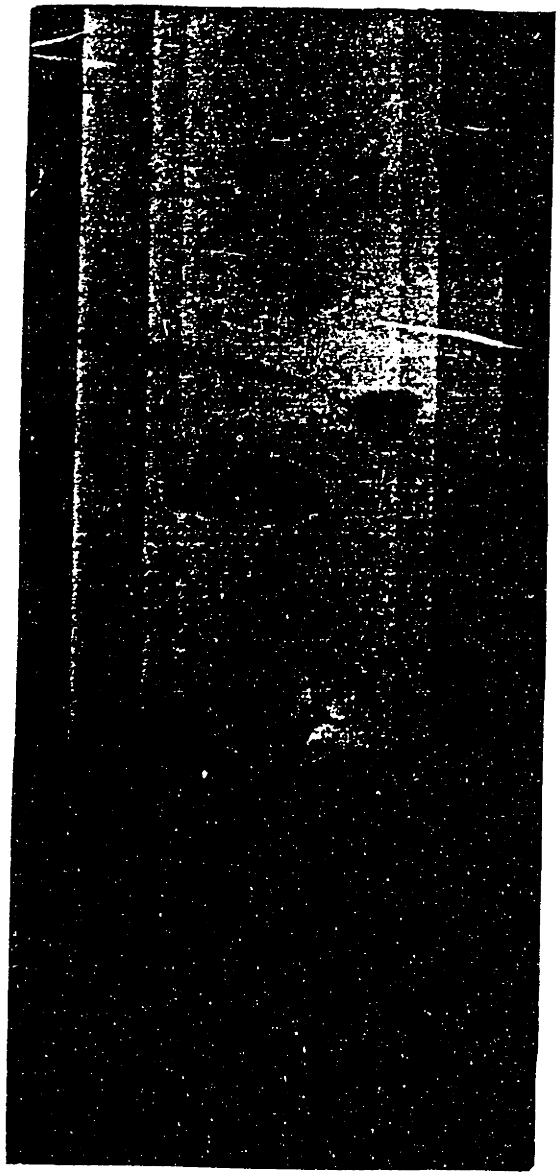
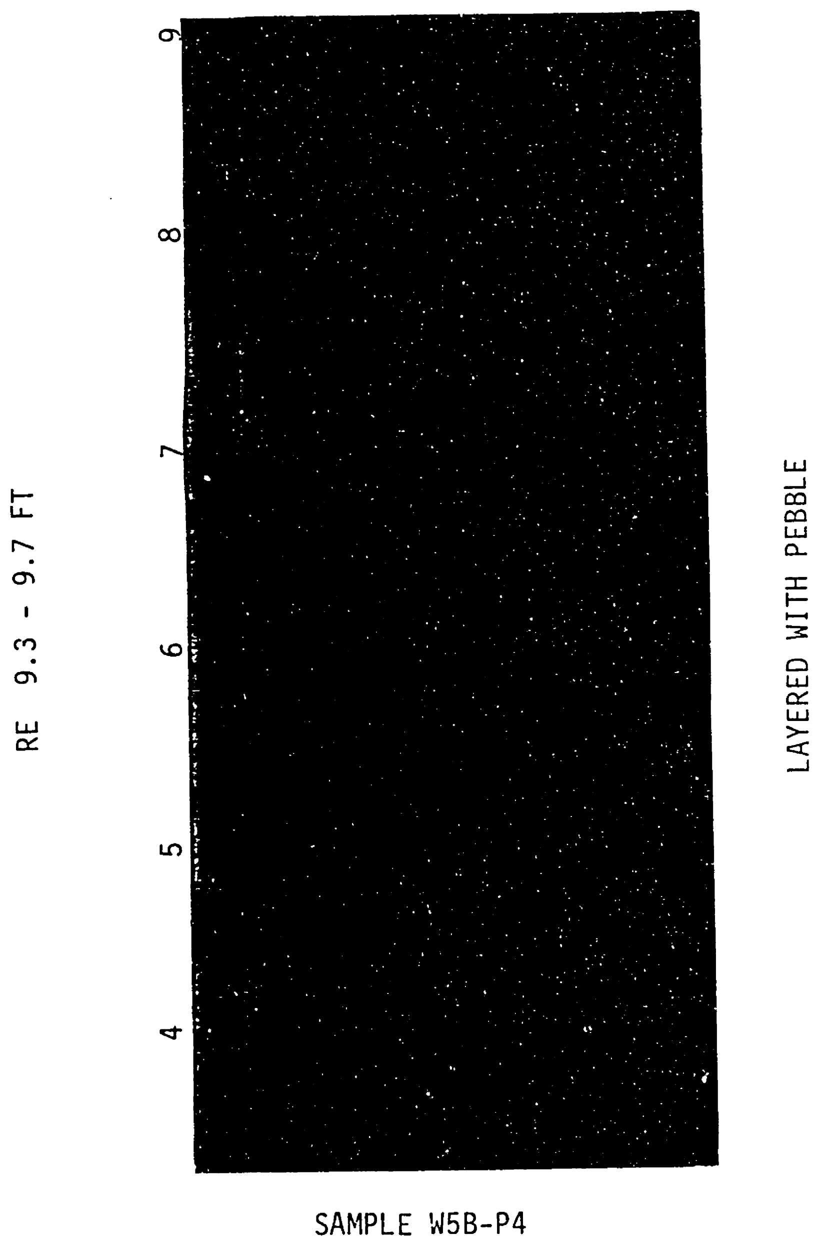


Figure 3-1: Site Investigation Plan



SAMPLE T5B1-P1

Figure 3-2: Radiograph Print: Site T



SAMPLE W5B-P4

Figure 3-3: Radiograph Print: Site W

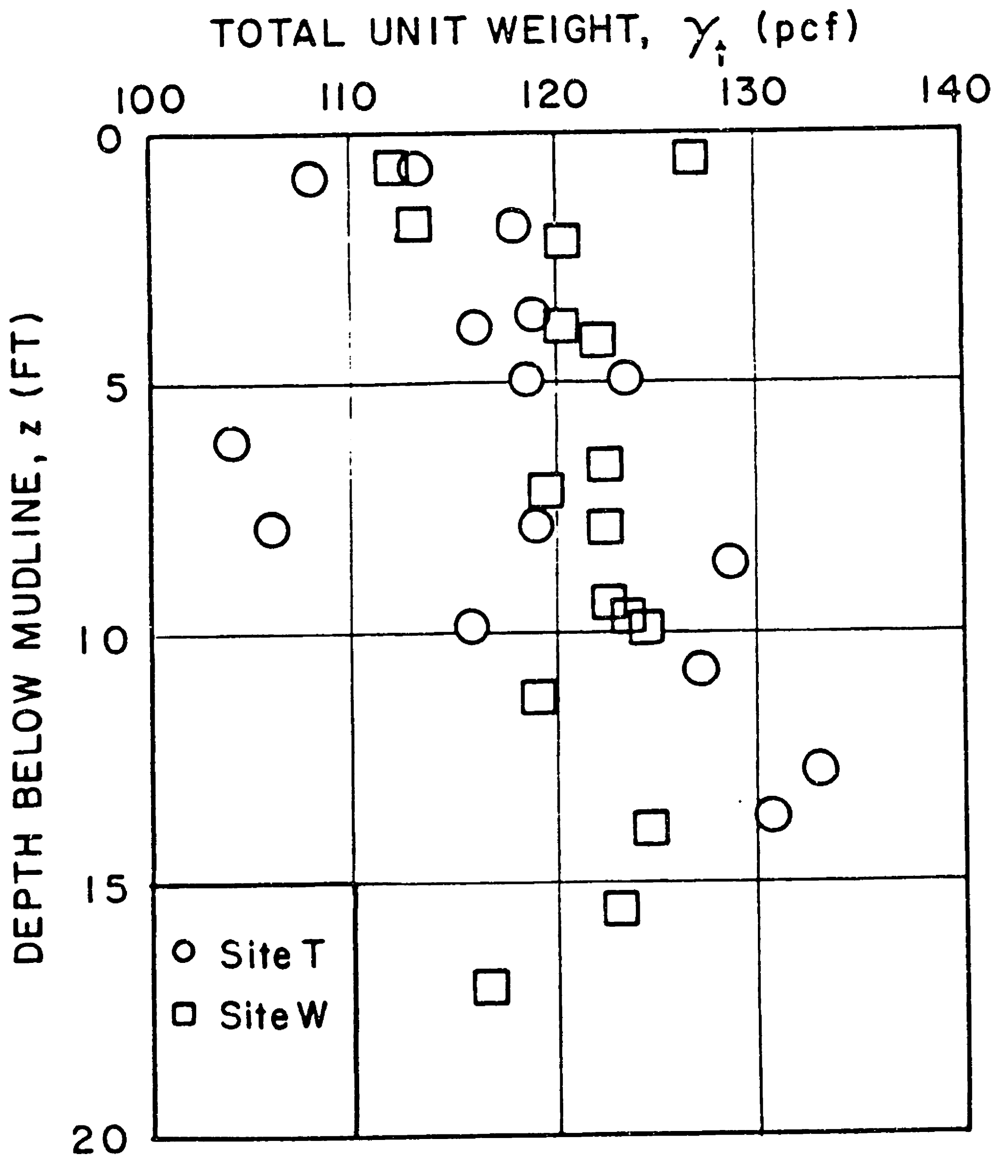


Figure 3-4: Profile of Unit Weights for Sites T and W

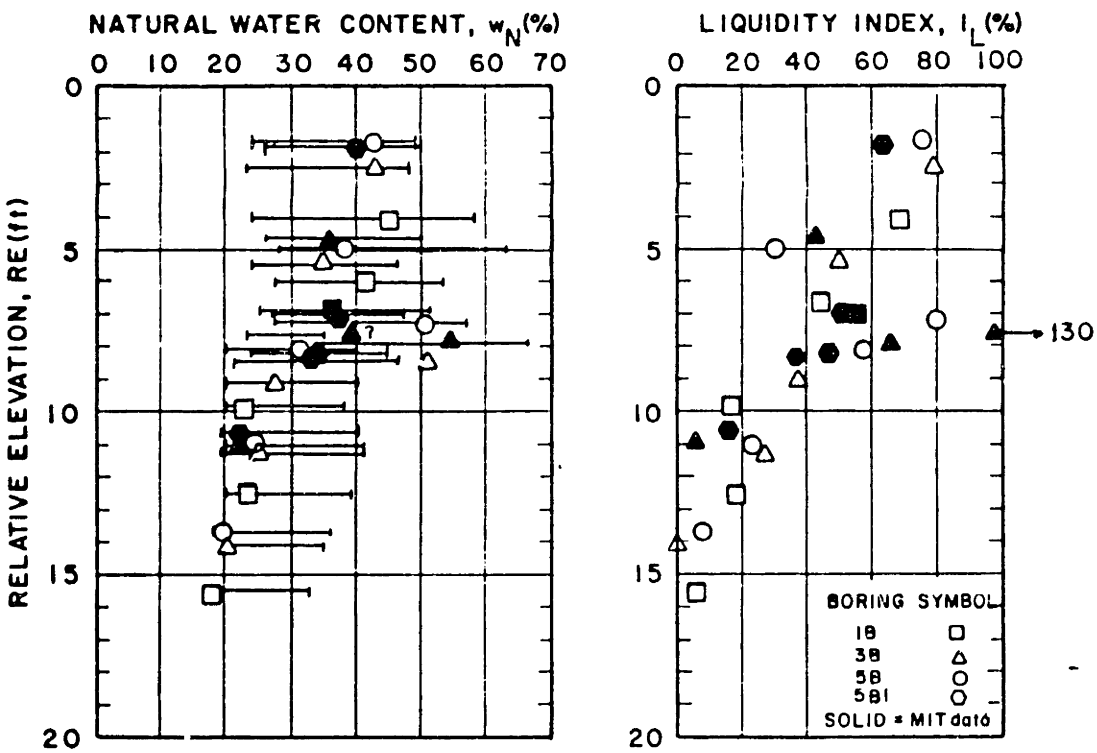


Figure 3-5: Natural Water Content and Liquidity Index for Site T

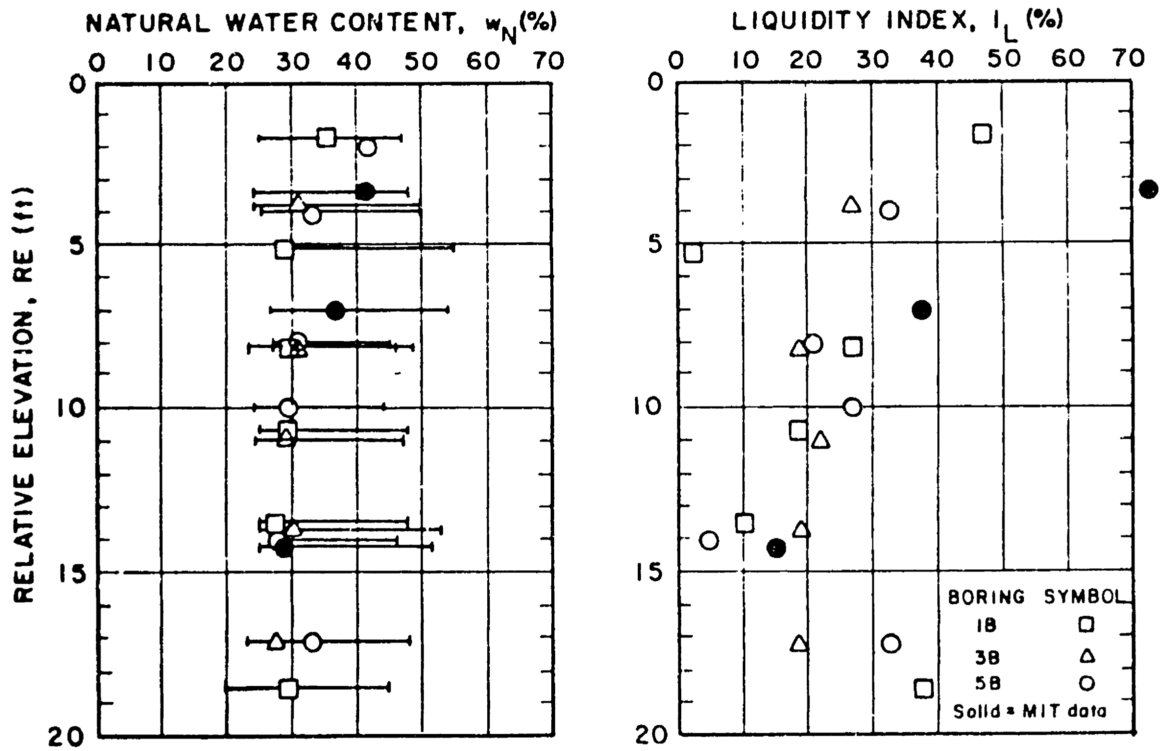


Figure 3-6: Natural Water Content and Liquidity Index for Site W

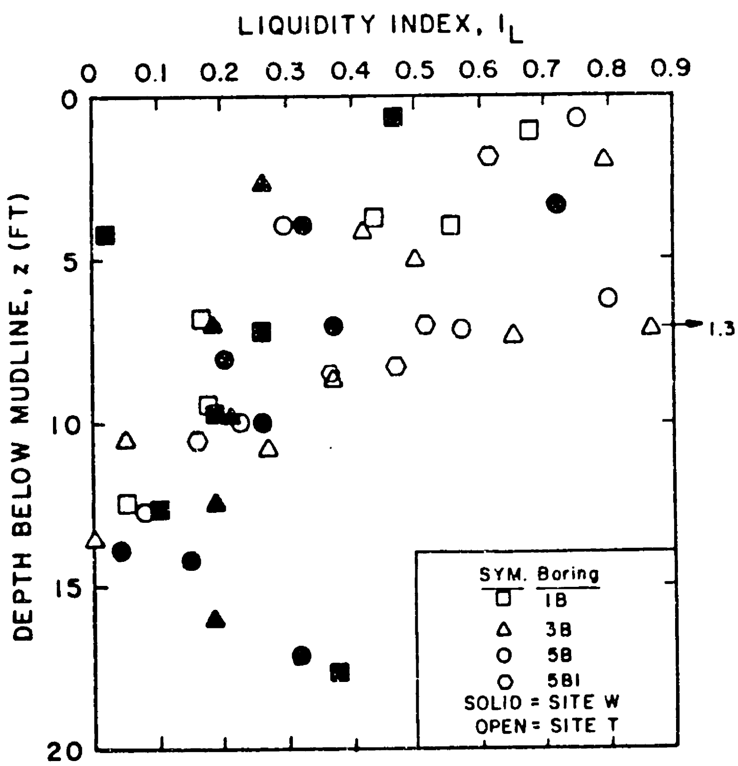


Figure 3-7: Profile of Liquidity Index for Smith Bay

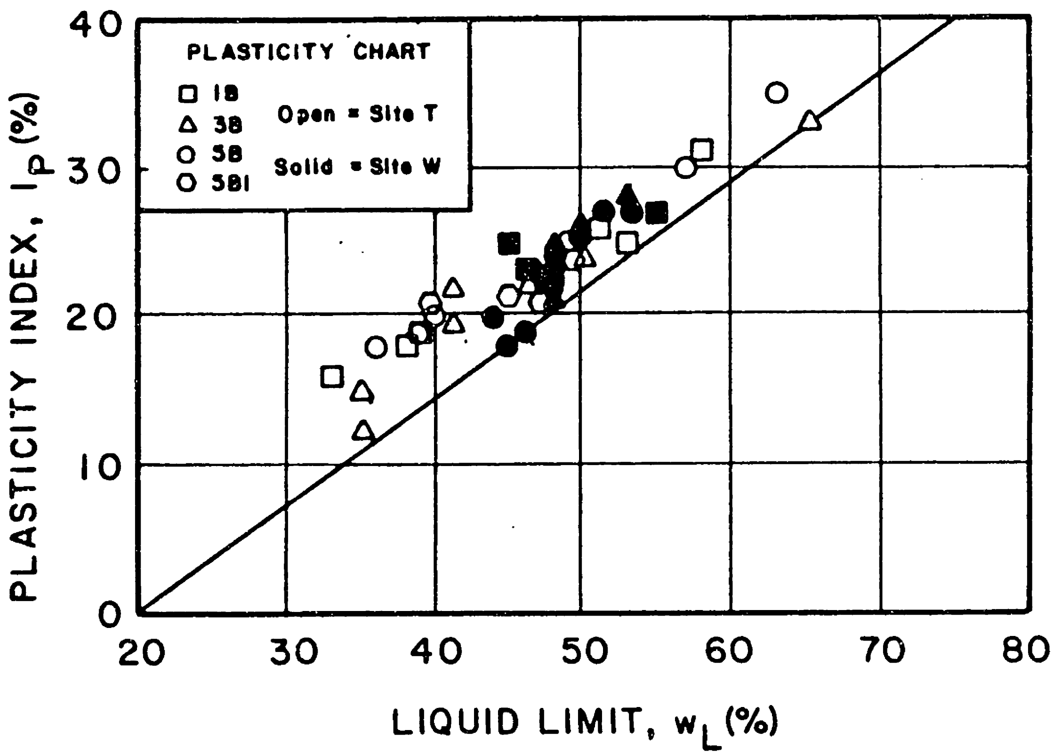


Figure 3-8: Plasticity Chart for Smith Bay Samples

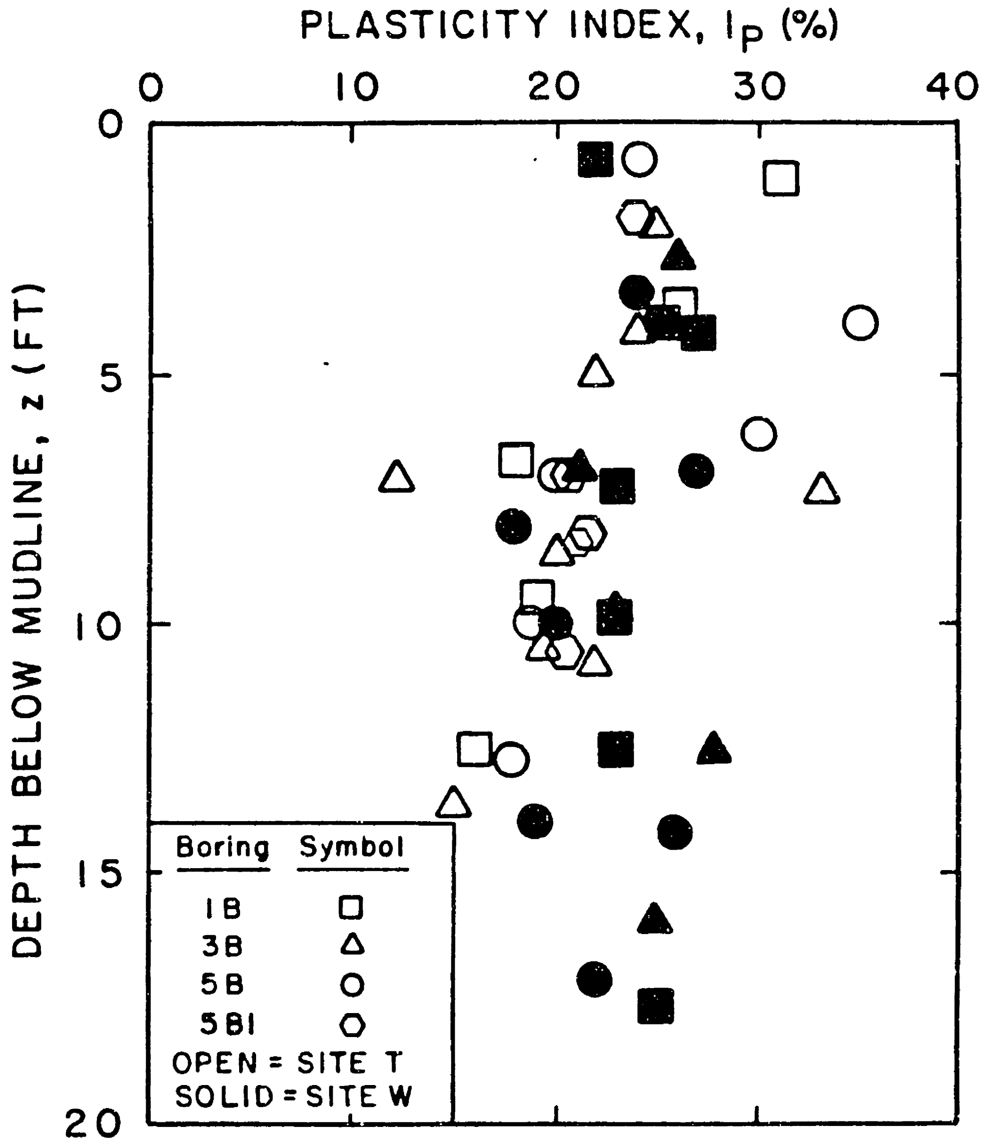


Figure 3-9: Plasticity Index versus Depth

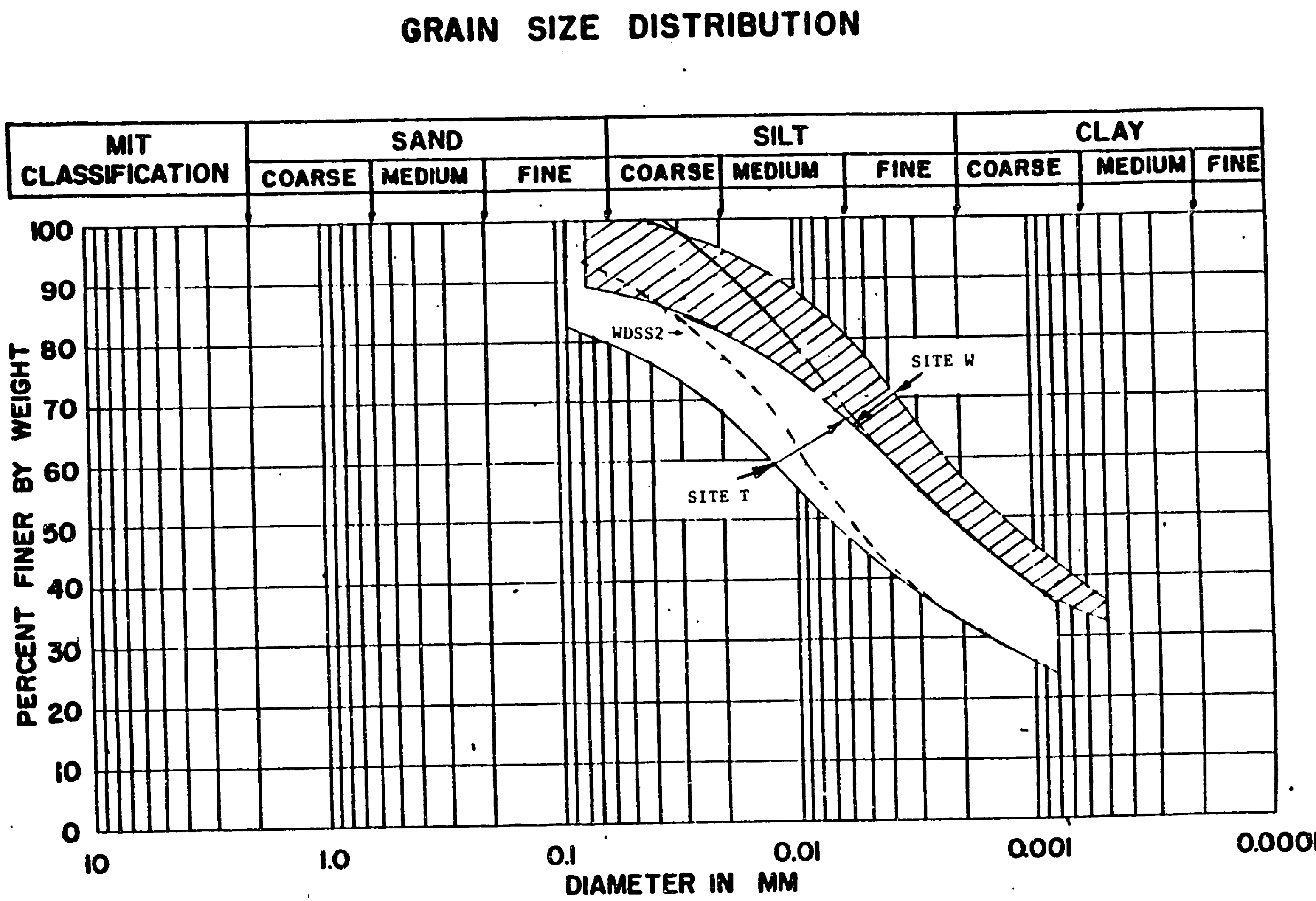


Figure 3-10: Grain Size Curve for Smith Bay Samples

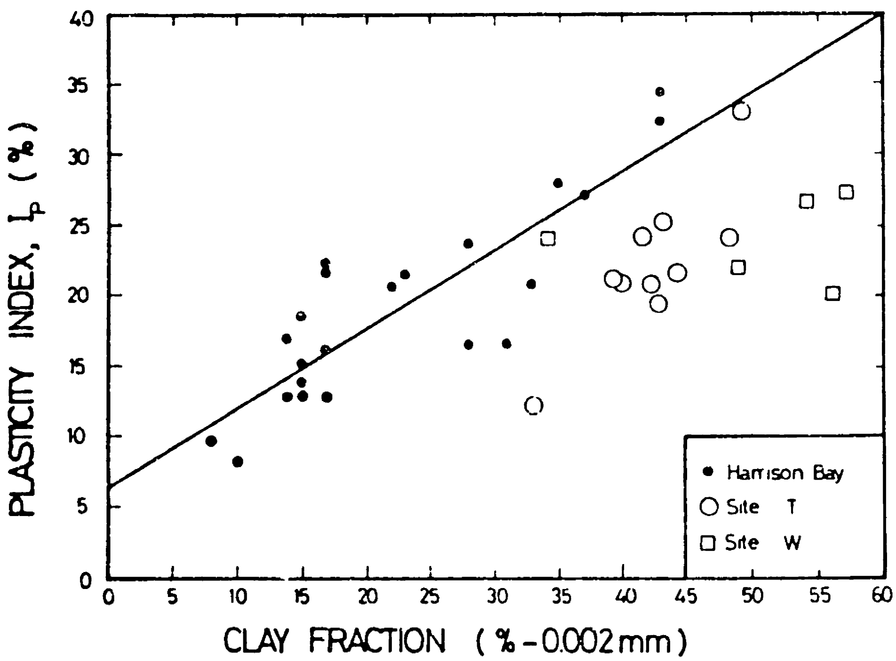


Figure 3-11: Plasticity Index versus Clay Fraction for Smith Bay Samples

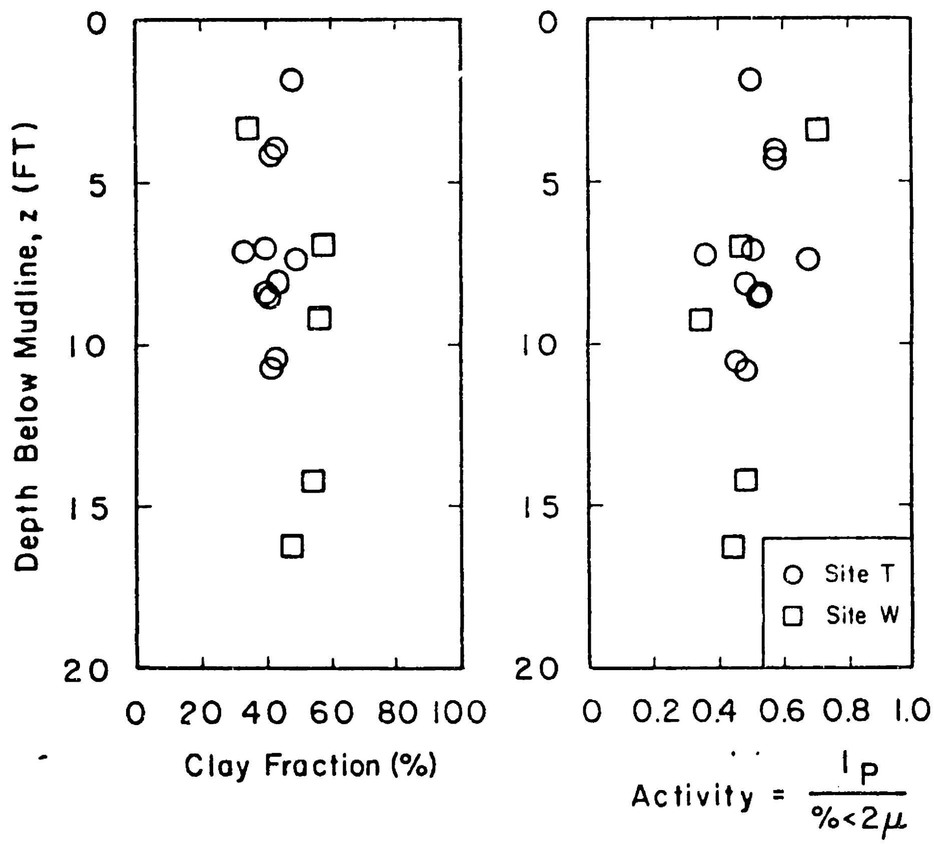


Figure 3-12: Profiles of Clay Fraction and Activity for Smith Bay

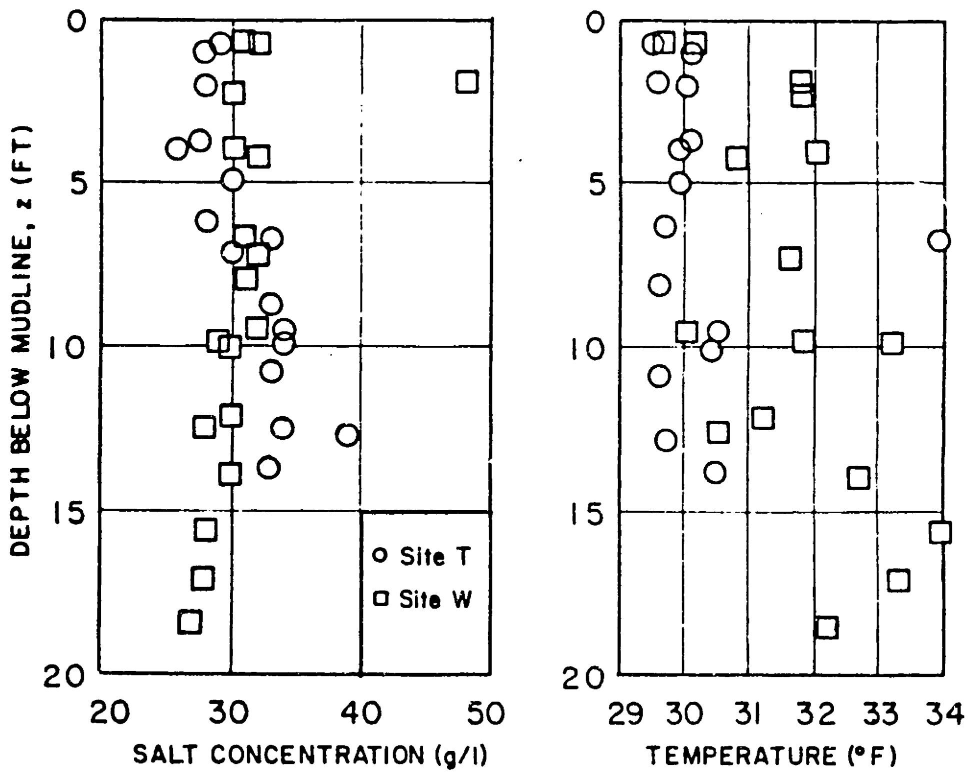


Figure 3-13: Profiles of Salt Concentration and Temperature for Smith Bay

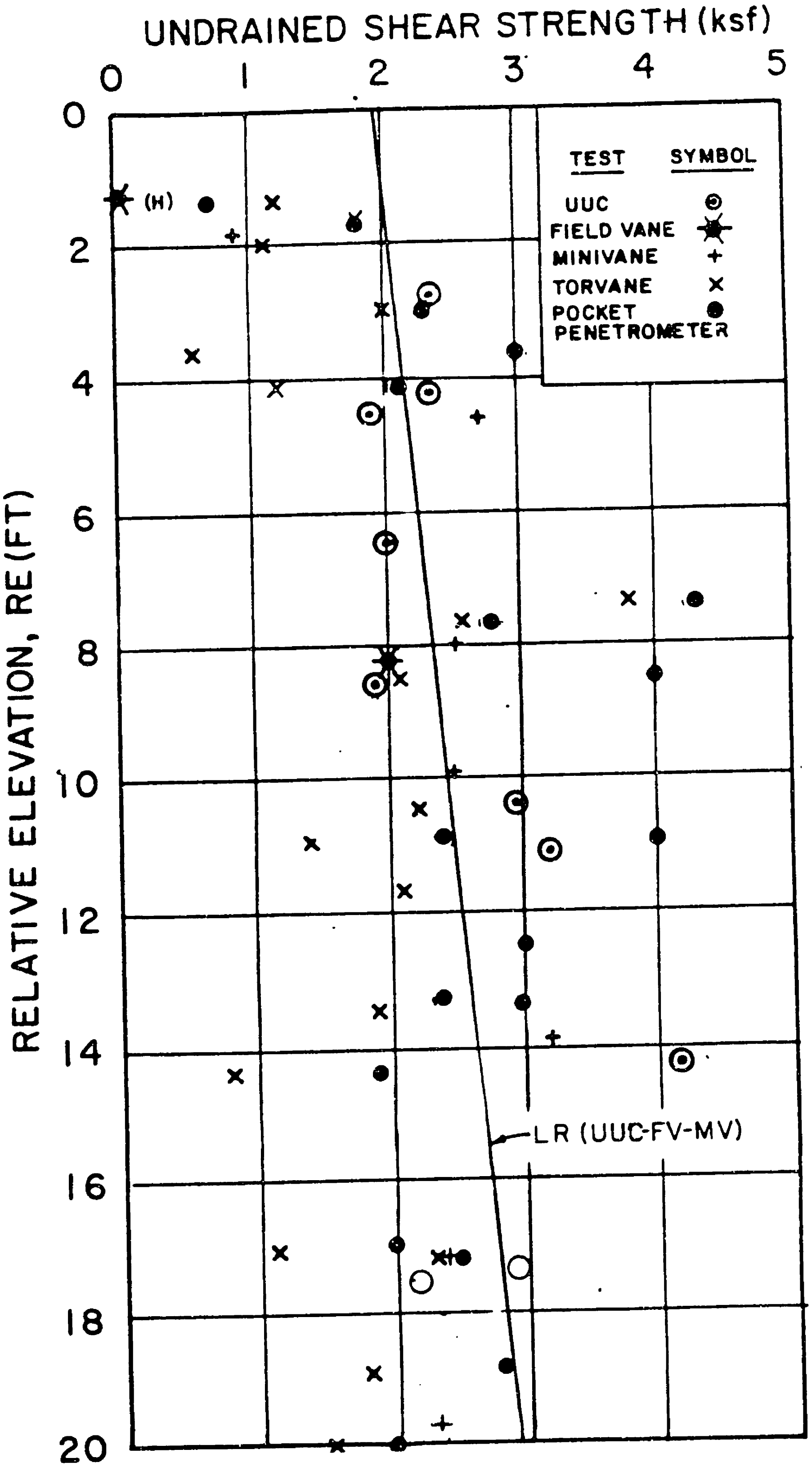


Figure 3-14: Strength Index Test Results for Site W

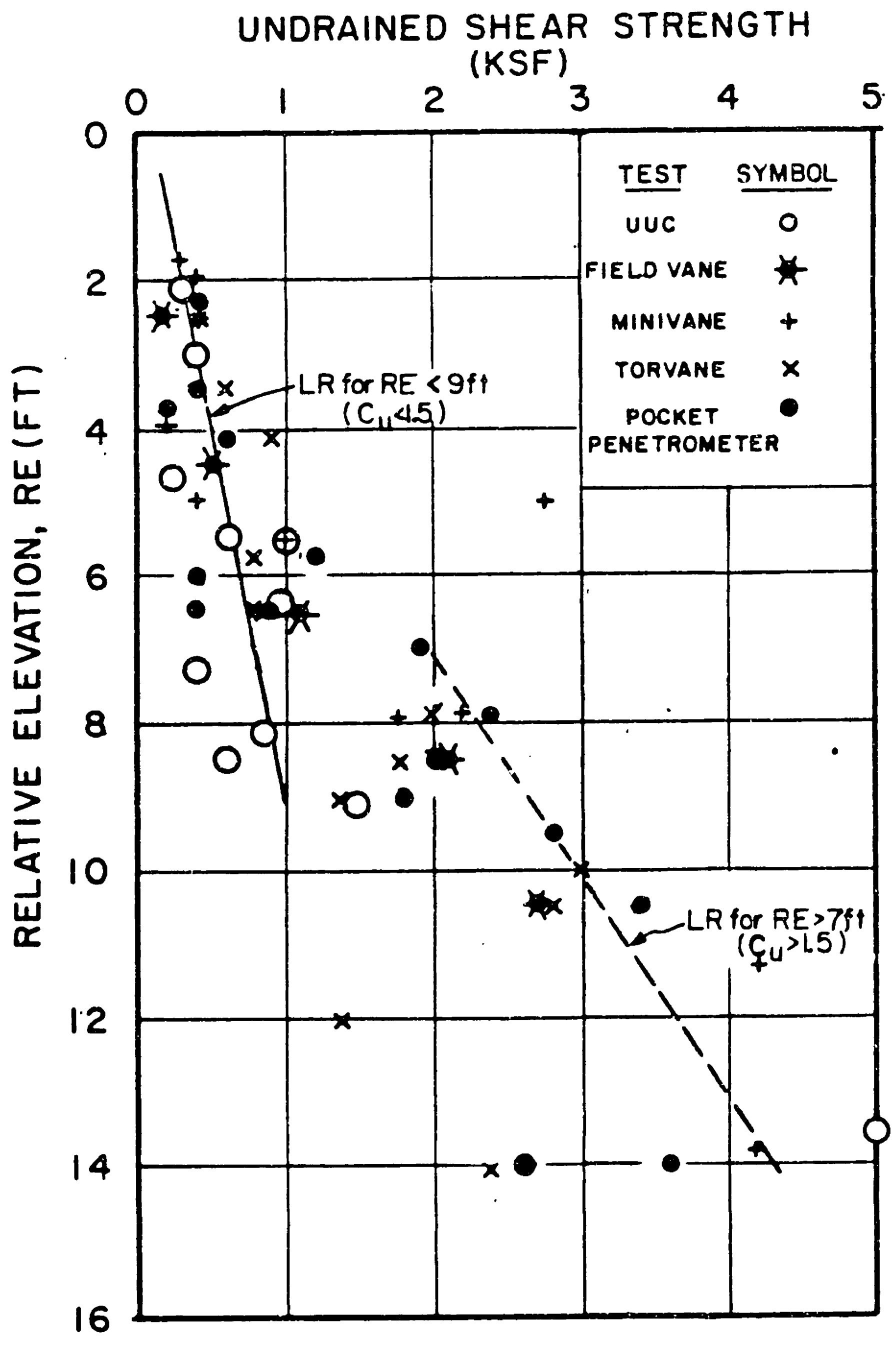


Figure 3-15: Strength Index Test Results for Site T

Chapter 4

CONSOLIDATION TEST PROGRAM

4.1 INTRODUCTION

Consolidation tests were used to estimate the stress history and to measure the compressibility - consolidation characteristics of Smith Bay Arctic silts. Tables 3-1 and 3-2 show the location of the consolidation tests run by TETC and MIT. TETC performed a total of 23 consolidation tests (12 on Site T and 11 on Site W samples). All TETC tests were run at 1 °C in a standard dead-load type consolidometer loaded to a maximum stress of 64 ksf. Details of TETC testing procedures are summarized in Appendix B. MIT performed 8 additional tests of the following types:

1. Five standard incremental oedometer tests on Site T samples run at room temperature with a maximum vertical stress equal to 58 ksf;

2. Two "high stress" oedometer tests (one each Sites T and W), consisting of a standard oedometer cell loaded in a high capacity load frame to 294 ksf;

3. One temperature controlled oedometer test (Site T), performed in order to assess the effect of temperature on compressibility-consolidation characteristics.

These tests are described in detail in the following section.

4.2 MIT TEST EQUIPMENT AND PROCEDURES

4.2.1 Standard Oedometer Tests

The one-dimensional incremental oedometer tests were run at room temperature on seven samples from Site T and one sample from Site W. Tests were performed between December, 1985, and June, 1986. Test locations were selected based on two criteria: 1) to fill in gaps in the TETC data or to supplement scattered results; and 2) where radiography indicated samples of reasonable to high quality. If necessary, sample tubes were cut in order to facilitate extrusion. MIT's conventional incremental equipment uses a 6.35 cm diameter by 2.2 ± 0.1 cm high sample. Following extrusion, samples were trimmed and tested according to procedures described by Yin (1985). A load increment ratio (LIR) of one was used for all standard tests, in order to duplicate the loading schedule used by TETC. Six of the room temperature tests included an unload-reload cycle to OCR = 16; the other test (T13) was unloaded to OCR = 8. The temperature controlled test (T17) had two unload-reload cycles to OCR = 8 (described in Section 4.2.3). The preconsolidation pressure was estimated using the Casagrande construction procedure on compression curves based on strains corresponding to the end of primary consolidation, as recommended by Ladd (1973). The end of primary strains were determined from displacement versus log time data. The coefficient of consolidation (c_v) for normally consolidated increments was estimated using the average of the the Casagrande Log-Time Fitting Method and Taylor's Square Root of Time Method. These procedures are described by Lambe (1951).

The oedometers were equipped with Direct Current Displacement Transducers (DCDT) to measure vertical displacement. A central data acquisition system recorded voltages and plotted results versus time. Measured displacements were corrected for apparatus compressibility.

4.2.2 High Stress Oedometer Tests

Two of the room temperature incremental oedometer tests (one each from Sites T and W) were reloaded in a high stress loading frame following completion of a standard test. After the standard test had been unloaded to the seating load (σ'_{vc} = 0.06 ksf) for at least 24 hours, the sample height was measured and the sample placed in the high stress load frame. Further increments were applied to a maximum stress of 294 ksf. Displacement was measured by a DCDT, and visual readings were used to construct displacement versus time plots. Reduction and analysis of data followed the same methods as for the standard oedometer tests.

4.2.3 Temperature Controlled Oedometer Test

One temperature controlled oedometer test was run on a sample from Site T. Techniques for extrusion and trimming of the sample were identical to those used for the standard incremental oedometer test. Procedures for temperature control were developed by Sauls et al. (1984). A standard oedometer cell was immersed in a constant temperature bath attached to a circulator which pumped a 50% ethylene glycol-50% distilled water solution. The first cycle of loading was run at 1.4 °C to "duplicate" the in situ temperature condition. The sample was then unloaded at 1.4 °C. The second cycle of loading was run at 20 °C to investigate the effect that testing at room temperature had on the behavior of a soil subjected to a known preconsolidation pressure (i.e. from the first cycle) at 1.4 °C. Following unloading at 20 °C, a third loading cycle was run at 1.4 °C to serve as a check on reversibility and repeatability.

4.2.4 Consolidation Phase of Direct Simple Shear Tests

Eighteen consolidated-undrained Geonor Direct Simple Shear (DSS) tests were performed using the equipment and procedures described in Chapter 6. As part of the SHANSEP testing program, the first phase of these tests consisted of consolidating samples into the normally consolidated range.

Although the original intent was to use the consolidation portion of the DSS test as part of the stress history data, most of these tests could not be used. In general, the samples were not consolidated to sufficiently high vertical effective stresses to establish the virgin compression line with confidence. Tests consolidated to "high" stresses (16 ksf for Site T; 24 ksf for Site W) yielded better defined virgin compression lines; therefore, estimates of preconsolidation pressure from these tests were used to help establish the stress history profile.

4.3 RESULTS OF CONSOLIDATION TESTS

Tables 4-1 and 4-2 summarize the results of the incremental consolidation tests. In addition to classification data, the tables present:

1. The measured vertical strain at the computed overburden stress, σ'_{vo} , and at the estimated preconsolidation pressure σ'_p.
2. Best estimates of preconsolidation pressure, σ'_p , determined via the Casagrande construction procedure (1936). The values presented are averages of at least three estimates.
3. The virgin Compression Ratio (CR), defined as the slope of the strain vs $\log\sigma'_{vc}$ in the normally consolidated region;
4. The Swelling Ratio (SR), defined as the average slope over one cycle of rebound from the maximum stress at the end of the test to OCR = 8 (not available for TETC tests);
5. The Recompression Ratio (RR) defined as the average slope of an unload-reload cycle to OCR = 16 (except for tests T13 and T17, with Δ OCR = 8);

All strains for TETC tests were calculated from the e-logσ'_{vc} plots contained in the TETC reports or from tabulated data provided to MIT. Detailed tabulated data and compression curves for MIT's incremental consolidation tests are presented in Appendix C. Table 4-3 presents data from the consolidation phase of five DSS tests. Compression curves for these DSS tests are presented and further discussed in Chapter 6.

4.3.1 Standard Oedometer Tests

Figure 4-1 presents compression curves for two samples at Site T. The low water content sample produced a flat, rounded compression curve, with compressibility increasing at the higher stresses. Estimates of preconsolidation pressure are difficult to make from this type of curve. Low water content material is characteristic of the lower, intact portion of Site T. The second curve, for a high water content sample, has a more well defined virgin compression ratio, and the Casagrande construction to estimate preconsolidation pressure is easier to perform. High water content samples are characteristic of the shallow, gouged portion of Site T.

A typical compression curve from a Site W sample is presented in Figure 4-2. These samples had low water contents, producing flat, rounded curves. These curves resemble those of the lower, intact Site T material.

4.3.2 High Stress Oedometer Tests

Two high stress incremental oedometer tests were performed in order to assess the shape of the compression curve at high stress, due to difficulties in defining the normally consolidated region from the standard oedometer curves. Results of the high stress oedometer tests are shown in Figures 4-3 and 4-4 for Sites

T and W, respectively. Estimates of the preconsolidation pressure for the high stress curves are higher than for the standard curves. The Compression Ratio, CR, for the last two points of the standard test (Figure 4-3) is 0.12, compared to 0.13 including the high stress data. Estimates of preconsolidation pressure increase from 5.3 to 6.15 ksf. At Site W, CR increases from 0.12 to 0.14 when the high stress values are considered, with an increase in estimated σ'_p from 4.3 to 6.9 ksf.

4.3.3 Temperature Controlled Oedometer

Figure 4-5 presents the compression curve for the temperature controlled consolidation test performed on a Site T sample. The compression curve resembles those of other tests having similar high natural water contents. A preconsolidation pressure of 3.9 ksf $\pm$0.6 SD was estimated using the Casagrande construction.

To assess the influence of test temperature on σ'_p , the virgin compression lines for 1.4 °C and 20 °C were compared. Consolidation at 1.4 °C results in a 10% increase in effective stress at the same strain. This is less than the 20-60% change predicted for some clays based on limited data from Gray (1936), Ladd and Luscher (1965) and Habibagahi (1973). However, the 10% change is close to the 13% difference noted by Yin (1985) for Harrison Bay Arctic silt. Temperature also affected compressibility during unload-reload cycles. The average recompression ratio over the OCR = 8 was less for reloading at 1.4 °C than at 20 °C (RR = 0.017 versus 0.029, respectively). Therefore, the in situ soil may be stiffer during recompression than measured in tests run at room temperature. This agrees with results of temperature control tests on Harrison Bay Arctic silts (Yin, 1985).

4.3.4 Comparison of Compression Curves

Figure 4-6 plots e-logσ'_{vc} data from the loading portion of Smith Bay oedometer tests and shows the range of consolidation behavior observed at Smith Bay. Site T curves show a wide variation in e_0 corresponding to the transition from the upper, highly gouged material to lower, less gouged material. Site T curves show a consistent trend of progressively lower e-logσ'_{vc} curves with decreasing e_0. In contrast, the Site W tests are located within a narrow band (e_0 = 0.7-0.9). The Site W curves cross the Site T curves, showing that the Site W soil is less compressible at high vertical consolidation stresses than Site T samples of comparable e_0.

Figure 4-7 shows representative strain data from selected oedometers at Site T. At high stresses, there is a very large difference in the virgin Compression Ratio, CR, and maximum strains for samples of different water contents. Higher water contents associated with ice gouging decrease as gouging effects diminish, with a corresponding decrease in maximum strain.

Figure 4-8 shows e-logσ'_{vc} plots with consolidation curves from four CK_0UDSS tests superposed on oedometer results. Several DSS tests exhibited high compressibility, unlike that observed for oedometer tests (e.g. TDSS4). The low maximum consolidation stress used for most of the DSS tests make it difficult to determine whether the sample is on the virgin compression line (e.g. test TDSS6). Estimates of preconsolidation pressure from these curves were consistently less than estimates from adjacent oedometers, and therefore may not be representative. DSS tests consolidated to higher stresses permit more reliable determination of the virgin compression line (e.g. TDSS10 and WDSS4). Estimates of preconsolidation pressure from such curves agree very well with those from oedometers and are used to supplement oedometer data in development of stress history profiles, as shown in Table 4-3.

4.4 STRESS HISTORY

Figure 4-9 presents the stress history developed for Site T. The in situ vertical effective stress, σ'_{vo} , was calculated using the average buoyant unit weight of γ_b = 0.055 kcf determined by TETC (1985). The plot includes data from 12 TETC consolidation tests, eight MIT consolidation tests and two DSS tests. Preconsolidation pressures estimated via the Casagrande technique for the MIT room temperature oedometer and DSS tests were increased by 10% in accordance with the temperature effects observed in the temperature-controlled tests (Section 4.3.3). The MIT and TETC values of preconsolidation pressure are generally consistent.

The upper 7 ft of the deposit has low preconsolidation pressures of less than 3 ksf, with a modest increase with depth. Below 7 ft, the preconsolidation pressure increases rapidly as the effects of ice gouging diminish. There is no significant difference among results from the four separate borings.

Figure 4-10 shows good correlation of liquidity index to estimated preconsolidation pressure for both sites. The Site T gouged material has a large range of liquidity index values, in contrast to the narrow range of values for Site W. There is a progressive reduction of preconsolidation pressure as the liquidity index increases for samples nearer to the mudline (see also Figure 3-7). The increased effects of ice gouging are responsible for large decreases in preconsolidation pressure at shallow depths.

The selected stress history profile for the upper layer consists of a linear regression line to RE = 7 ft (σ'_p = 0.73 + 0.273RE with r^2 = 0.42). Linear interpolation between statistical means taken of the groups of data clustered about RE = 8, 11 and 13.8 ft was used to define a stress history profile over the remainder of the deposit.

The Site W stress history profile presented in Figure 4-11 shows a nearly constant preconsolidation pressure with depth. The profile includes ten TETC oedometers, one MIT oedometer, and three MIT DSS tests. Estimates of preconsolidation pressure were increased by 10% for the MIT room temperature tests. The data are less scattered than at Site T, with TETC and MIT tests showing good agreement. One oedometer in the upper layer of Holocene material (RE = 1.4 ft) gave a very low preconsolidation pressure; this value was ignored in development of a stress history profile. Linear regression was used to obtain the stress history profile shown ($\sigma'_p = 7.2 + 0.0605$ RE with $r^2 = 0.081$).

Figure 4-12 compares estimates of preconsolidation pressure for both sites. Extrapolation of the linear regression of the Site W σ'_p profile agrees well with the Site T results at depth, suggesting that the two sites had a common initial stress history which was later altered by ice gouging at Site T. The soil at Site W is thought to be of Pleistocene age, except for the uppermost 0-3 ft of Holocene material (seen only in Boring 5B). Comparison with the Site T data suggests that the lower part of Site T is also Pleistocene; however, the ice gouging has reworked the deposit sufficiently so that the Pleistocene/Holocene boundary can not be defined. Subaerial exposure during the Pleistocene provided opportunities for erosion and severe freezing of the soil. Thawing of material then occurred during the subsequent sea level rise. The occurrence of freeze-thaw would result in a decrease of water content to near the plastic limit for pre-Holocene sediment (see Section 2.2).

Figure 3-7 shows a very large decrease in Liquidity Index with depth for Site T with values at Site T below RE = 10 ft comparable to those at Site W. This transition is associated with the highly gouged-less gouged boundary. I_L values at Site W are high only in the upper few feet of Holocene material, then decrease to a nearly constant value. The low values of Liquidity Index correspond to a water content approaching the plastic limit, supporting the occurrence of at least one freeze-thaw episode during the Pleistocene (Lee et al., 1985). Freeze-thaw, possibly coupled with removal of overburden, could explain the observed high values of preconsolidation pressure at Site W and within the lower portion of Site T. Reduction of preconsolidation pressure in the upper part of Site T occurred as a result of mechanical reworking by ice to produce the present profile.

4.5 CONSOLIDATION PROPERTIES

The compressibility parameters CR, RR and SR from the oedometer tests at Sites T and W are summarized in Tables 4-1 and 4-2, respectively, and plotted versus depth in Figures 4-13 and 4-14. Values from the TETC tests were computed by MIT. Final unloading data were not provided by TETC, hence no values of SR are shown for these tests. MIT and TETC results are consistent for Site T; the one test run by MIT at Site W also fits the TETC data.

At Site T, the virgin Compression Ratio, CR, is scattered but constant with depth and equal to 0.154 ± 0.03 SD for RE less than 9 ft. CR decreases to 0.093 ± 0.015 SD for RE greater than 10 ft. The Recompression Ratio, RR, was measured for OCR = 8 and is nearly constant throughout the deposit, with a value of 0.027 ± 0.005 SD. Site W has a uniform CR (0.124 ± 0.02 SD) and RR (0.028 ± 0.006 SD) with depth, except for one high CR value measured for the Holocene material at RE = 1.4 ft.

According to Ladd (1973), the Recompression Ratio, RR, is typically 0.1 to 0.2 times the virgin Compression Ratio, CR, for mechanically overconsolidated cohesive soils. The compressibility ratio values (RR/CR) for Site T (Figure 4-13) fall around the upper end of this range (RR/CR = 0.206 ± 0.032 SD). RR/CR is slightly greater from 10 to 14 ft, since CR decreases. At Site W, RR/CR = 0.229 ± 0.05 SD, corresponding to values in the lower (less gouged) portion of Site T.

Figure 4-15 compares the virgin compressibility parameters, C_c and CR, versus three empirical correlations reported by Nishida (1956), Terzaghi and Peck (1967) and Lambe and Whitman (1969). The Nishida correlation overestimates the relationship between C_c and void ratio for higher values of e_0 (>1). The Lambe and Whitman relationship underestimates CR, especially at w_N less than 40%. The Terzaghi and Peck relationship is surprising since: the highly gouged upper Site T material fits the correlation proposed for undisturbed clay; whereas the Site W and lower Site T soil fall below the correlation for remolded clay.

Table 4-4 presents the average ± 1 SD of the coefficient of secondary compression (C_α), defined as the change in vertical strain per log cycle of time after the end of primary, in the virgin range. Data are available for four TETC tests and all MIT tests. A profile of normally consolidated C_α versus depth is presented for Site T in Figure 4-16. Measured values of C_α decrease with depth, becoming constant for tests at RE greater than 10 ft. The ratio, C_α/CR, equals 0.029 ± 0.007 SD, and falls at the lower end of the range of the 0.04 ± 0.01 SD quoted by Mesri and Choi (1985) for inorganic clays.

Table 4-4 also contains the average ± 1 SD of the normally consolidated coefficient of consolidation (c_v) for Sites T and W. The average value over the normally consolidated range and the number of increments in each average are shown in Table 4-4 for the MIT tests. Values of c_v obtained for individual increments are included in the tabulated data from oedometer tests (Appendix C). The MIT values are an average of the Log-Time and Square Root of Time methods. Estimates of c_v for the four TETC tests were made by MIT based on limited data provided by TETC. MIT assumed an initial sample height of 1 inch (as per TETC procedures, Appendix B), and calculated the height at the end of primary consolidation using tabulated strain data provided by TETC. Displacement versus time curves were available for only a few increments of four tests at Site T; no TETC time data were available for Site W. Estimates of c_v were made using the Log Time method for the TETC tests.

Based on Sauls et al. (1984), the in situ c_v at 0 °C should be about 0.6 times that measured at room temperature due to the change in viscosity of the pore fluid. Figure 4-17 plots the mean $c_v \pm 1$ SD versus RE. Room temperature tests run by MIT were adjusted to the in situ temperature by using the above 0.6 correction factor. Excluding the high value at RE = 5 ft, there is relatively little scatter and $c_v = 0.05 \pm 0.019$ SD ft^2/day. The single Site W test included in the plot agrees very well with the lower Site T values.

The coefficient of consolidation is also plotted versus liquid limit, using the NAVFAC DM-7.1 (1982) empirical correlation (Figure 4-18). All c_v values are corrected to 20 °C in this plot (TETC values times 1.6). Most results plot near the lower limit for remolded samples, including data from moderate-high ice gouged zones. Data from Harrison Bay Arctic Silts plot above the average line (Yin, 1985).

4.6 SUMMARY

Site W, located in relatively shallow water, consists of Pleistocene material which is not affected by ice gouging over a 20 ft depth. Site T is located in deeper water, and contains heavily reworked Pleistocene and Holocene material, i.e. ice gouging significantly affected the top half of the 14 ft thickness.

TETC ran 12 and 11 standard oedometers at 1 °C at Sites T and W, respectively. MIT ran one variable temperature oedometer and 7 oedometers at room temperature at Site T. The consolidation phase of two DSS tests were also used to supplement the stress history profile. MIT performed one oedometer at room temperature at Site W to supplement the results of the TETC tests. Three DSS tests from Site W were also used in determination of the stress history profile.

The MIT temperature controlled oedometer test on typical upper layer (gouged) Site T material indicated that tests at room temperature lead to preconsolidation pressures about 10% lower than at the in situ temperature. This agrees with results on Harrison Bay Arctic silts (Yin, 1985). Hence all values of σ'_p from room temperature tests were increases by 10% when developing the in situ stress history profile.

Figures 4-9 and 4-11 present the stress history profiles for Sites T and W, respectively. The Site T plot shows a modest increase in preconsolidation pressure to RE = 7 ft, below which the preconsolidation pressure increases dramatically, corresponding to the transition from highly gouged to less gouged material. The stress history profile was estimated using linear regression to RE = 7 ft, then linear interpolation between statistical means at RE = 8, 11 and 13.8 ft. The Site W deposit has less scatter and shows nearly constant preconsolidation pressure with depth; linear regression was used to select the stress history profile. The two sites are presumed to have shared a common initial stress history, as values of preconsolidation pressure are compatible for Site W and the lower part of Site T (Figure 4-12). The high preconsolidation pressures are thought to be a result of removal of overburden and/or freeze-thaw effects resulting from subaerial exposure during the Pleistocene. The Site T stress history profile was subsequently altered by ice gouging, resulting in the present profile.

Compressibility of Sites T and W versus depth are shown in Figures 4-13 and

4-14. The virgin Compression Ratio, CR, at Site T is higher for gouged material. A decrease in CR with depth corresponds to the transition to less gouged material. Values of CR are nearly constant at Site W, and comparable to values in the lower part of Site T. The virgin compressibility of the Smith Bay deposits does not agree with empirical correlations with liquid limit, initial void ratio or natural water content.

Normally consolidated coefficient of consolidation (c_v) data (adjusted to the in situ temperature) are plotted versus depth in Figure 4-17. Values of c_v (now at room temperature) are less than predicted via the DM-7 correlation (Figure 4-18), and are one tenth of those measured for Harrison Bay Arctic silt (Yin, 1985).

Table 4-1: SUMMARY OF CONSOLIDATION TEST DATA ON SITE T SAMPLES

All Stresses in ksf

No.	Sample	RE (ft.)(1) σ'_{vo}	Tested By (Temp.°)	w_N Test AL(2) (%)	w_L w_P (%)	I_P I_L(3) (%)	ε_v(%) at σ'_{vo} σ'_p	Est(4) σ'_p	CR (e_o)	RR(5) SR(6)	RR/CR	Test(7) Quality
T1	5BP2	5.15 0.283	TETC (1°C)	58.4 38.5	63 28	35 30	0.5 6.7	2.46 ±0.05	0.22 (1.53)	0.033 -	0.15	G
T2	5BP1	1.75 0.096	TETC (1°C)	46.3 42.7	49 24	25 75	0.8 9.1	1.40 ±0.07	0.16 (1.20)	0.030 -	0.19	P
T3	5BP3B	7.75 0.426	TETC (1°C)	25.4 31.4	40 20	20 57	0.1 3.9	6.08 ±0.6	0.11 (0.69)	0.023 -	0.21	P
T4	5BP3T	7.55 0.415	TETC (1°C)	48.2 50.9	57 27	30 80	1.4 5.9	1.71 ±0.16	0.17 (1.31)	0.030 -	0.175	G
T5	5BS5	13.45 0.740	TETC (1°C)	19.4 19.4	36 18	18 8	0.7 5.0	11.3 ±3.6	0.08 (0.59)	0.018 -	0.225	VP
T6	1BO1	4.35 0.074	TETC (1°C)	45.1 45.1	58 24	31 68	0.5 7.0	1.30 ±0.2	0.155 (1.31)	0.028 -	0.181	P
T7	1BO2	6.75 0.206	TETC (1°C)	38.4 36.3	51 25	26 44	0.2 5.3	2.12 ±0.3	0.14 (1.08)	0.032 -	0.229	P
T8	3BP1	2.6 0.116	TETC (1°C)	45.1 42.9	48 23	25 79.5	0.5 5.6	1.07 ±0.1	0.145 (1.19)	0.030 -	0.207	P
T9	3BP2	5.1 0.253	TETC (1°C)	35.1 35.0	46 24	22 50	0.6 4.9	3.04 ±0.28	0.135 (0.93)	0.033 -	0.244	E
T10	3BP3T	8.15 0.421	TETC (1°C)	51.7 51.1	- -	- -	0.9 5.5	2.87 ±0.17	0.19 (1.43)	0.037 -	0.195	F
T11	3BP4	11.7 0.616	TETC (1°C)	24.9 24.9	41 19	22 27	0.3 2.6	5.75 ±0.3	0.11 (0.66)	0.023 -	0.209	G
T12	3BS5	14.15 0.778	TETC (1°C)	20.2 20.0	35 20	15 0	0.3 3.2	11.9 ±4.7	0.08 (0.55)	0.019 -	0.238	F
T13	3BP3	7.6 0.391	MIT (20°C)	51.7 54.4	65.8 32.7	33.1 65.5	1.5 5.8	1.7 ±0.12	0.18 (1.57)	0.025 0.027	0.139	E
T14	3BP4	10.9 0.572	MIT (20°C)	22.2 22.0	40.6 21.0	19.6 5	0.5 4.6	7.6 ±1.7	0.081 (0.66)	0.021 0.020	0.259	F
T15	5B1P3	8 0.440	MIT (20°C)	41.2 33.8	45.2 23.7	21.5 47	0.5 5.5	3.5 ±0.28	0.158 (1.109)	0.030 0.025	0.190	G
T16	5B1S4	10.4 0.572	MIT (20°C)	23.1 22.3	39.5 19	20.5 16.1	0.4 4.4	6.2 ±1.3	0.093 (0.659)	0.018 0.020	0.194	F
T17	1B-02	7.3 0.237	MIT (ΔT)	37.6 41.5	52.7 27.6	25.1 55.4	2.5 8	3.9 ±0.6	0.115 (0.061)	0.025 0.020	0.217	F-G
T18	5B1S4A	11 0.605	MIT (20°C)	23.8 22.3	39.5 19	20.5 16.1	0.6 4.8	7.5 ±1.3	0.111 (0.705)	0.028 0.020	0.252	F
T19	5B1P3	8.5 0.468	MIT (20°C)	27.9 32.9	46.3 25.2	21.1 36.5	0.75 6.0	6.15 ±1.01	0.126 (0.756)	0.027 0.020	0.214	G

(1) Mean RE of sample tested by TETC

(2) Water content of specimen used to determine Atterberg Limits

(3) Based on W_N from Atterberg Limits

Table 4-2: SUMMARY OF CONSOLIDATION TEST DATA
ON SITE W SAMPLES

All Stresses in ksf

No.	Sample	RE(1) (ft.) $\sigma_{vo'}$	Tested By (Temp.)	W_N Test AL(2) (%)	W_L W_P (%)	I_P I_L(3) (%)	ϵ_v(%) at σ'_{vo} σ'_p	Est(4) σ'_p	CR (e_o)	RR(5) SR(6)	RR/CR	Test(7) Quality
W1	3BP1	2.25 0.058	TETC (1°C)	25.2 24.3	- -	- -	0 3.5	7.5 ±0.5	0.094 (0.717)	0.0129 -	0.137	F
W2	3BP2	3.35 0.118	TETC (1°C)	30.2 30.9	50 24	26 26.5	0 2.5	5.8 ±0.26	0.116 (0.816)	0.032 -	0.272	F-G
W3	3BP3	6.8 0.308	TETC (1°C)	33.5 30.8	48 27	21 18.1	0.2 3.8	9.0 ±0.5	0.126 (0.842)	0.0271 -	0.215	G
W4	3BS4	11.45 0.564	TETC (1°C)	31.7 29.0	47 24	23 21.7	0.9 4.1	7.8 ±0.35	0.120 (0.90)	0.035 -	0.292	P
W5	3BS6	17.65 0.905	TETC (1°C)	27.7 27.6	48 23	25 18.4	0.3 -	6.9 ±0.12	0.11 (0.774)	0.028 -	0.255	G-E
W6	5BP1	1.4 0.077	TETC (1°C)	75.7 41.5	- -	- -	0.2 4.7	0.4(8)	0.175 (2.026)	0.029 -	0.166	VP
W7	5BP2	4.85 0.267	TETC (1°C)	33.2 33.2	50 25	25 22.8	0.2 3.6	6.6 ±0.23	0.137 (0.872)	0.039 -	0.285	E
W8	5BP3	8.1 0.446	TETC (1°C)	31.9 30.7	45 27	18 20.6	0.3 3.4	8.4 ±0.6	0.128 (0.827)	0.027 -	0.211	G
W9	5BP4	10.65 0.586	TETC (1°C)	29.2 29.3	44 24	20 26.5	0.4 3.5	8.3 ±1.0	0.124 (0.756)	0.025 -	0.202	P
W10	5BP5	14.65 0.806	TETC (1°C)	28.9 27.9	46 27	19 4.7	0.4 3.2	10.3 ±1.2	0.111 (0.765)	0.030 -	0.270	P
W11	5BS6	17.9 0.985	TETC (1°C)	32.1 33.2	48 26	22 32.7	0.4 3.4	7.7 ±0.76	0.112 (0.861)	0.027 -	0.241	F-P
W12	5BS6	16.3 0.897	MIT (20°C)	29.5 33.2	48 26	22 32.7	0.9 6.3	6.9 ±1.3	0.135 (0.817)	0.027 0.02	0.20	G-E

(1) Mean RE of sample tested by TETC

(2) W_N of specimen used to determine Atterberg Limits

(3) Based on Atterberg Limits W_N

(4) From Casagrande based on 3 or more estimates (except W12)

(5) Ave. slope of unload/reload cycle for ΔOCR= 16

(6) Ave. slope of final unload cycle for ΔOCR=8

(7) Excellent/Good/Fair/Poor

(8) Holocene (?) sample

Table 4-3: CK_0UDSS TESTS USED IN STRESS HISTORY PROFILES

All Stresses in ksf

Site	No.	Sample	RE (ft.) σ_{vo}'	Tested By (Temp °C)	w_N Test AL(1) (%)	w_L w_P (%)	I_P I_L(2) (%)	ε_v(%) at σ_{vo}' σ_p'	Est(3) σ_p'	CR (e_o)	σ'_v(max)
T	TDSS9	1B-02	7.2 0.23	M (20°C)	45.9 41.5	52.7 27.6	25.1 55.4	1.0 10.0	2.41 ±0.25	0.19 (1.32)	16
	TDSS10	5B1-P3	8.4 0.462	M (20°C)	38.4 32.9	46.3 25.2	21.1 36.5	1.2 5.6	3.29 ±0.25	0.14 (1.00)	16
W	WDSS4	5B-P4	9.2 0.506	M (20°C)	28.9 29.3	44 24	20 26.5	0.8 7.0	6.7 ±0.99	0.123 (0.83)	24.6
	WDSS6	5B-P4	9.4 0.512	M (1°C)	30.0 29.3	44 24	20 26.5	1.1 6.9	8.0 ±0	0.118 (0.92)	24.6
	WDSS7	5B-P4	9.5 0.517	M (20°C)	29.4 29.3	44 24	20 26.5	0.4 6.0	7.2 ±0.28	0.105 (0.850)	24.6

(1) Water content of sample used to determine Atterberg Limits

(2) Based on Atterberg Limits w_N

(3) From Casagrande based on $\geq$ 2 estimates

Table 4-4: COEFFICIENT OF CONSOLIDATION AND RATE OF SECONDARY COMPRESSION IN THE NORMALLY CONSOLIDATED RANGE

Site	No.	RE (ft.)	Tested By	w_N w_L (%)	σ_{vc}' (ksf)	C_v (ft.^{2}/day)			$C\alpha$ (%)		$C\alpha/CR$
						$\sqrt{t}$	log t	Mean ± SD	Value	Mean ± SD	
T	T1	5.15	TETC (1° C)	58.4 63	4-8 8-16 16-32 32-64	- - - -	0.202 0.093 0.051 0.050	0.099 ±0.07	0.20 0.80 0.70 0.60	0.575 ±0.26	0.026
	T5	13.45	TETC (1° C)	19.4 36	8-16 16-32 32-64	- - -	0.073 0.070 0.049	0.064 ±0.013	0.22 0.3 0.3	0.273 ±0.05	0.034
	T7	6.75	TETC (1° C)	38.4 51	2-4 4-8 8-16 16-32 32-64	- - - - -	0.039 0.039 0.055 0.045 0.066	0.048 ±0.012	0.7 0.59 - 0.4 0.4	0.523 ±0.15	0.037
	T10	8.15	TETC (1° C)	51.7 -	4-8 8-16 16-32 32-64	- - - -	0.057 0.058 0.052 0.072	0.060 ±0.01	0.8 0.85 0.4 0.3	0.059 ±0.28	0.031
	T13	7.6	MIT (20° C)	51.7 65.8	2-53 (6 inc.)	0.049 ±0.005	0.041 ±0.01	0.045 ±0.01	-	0.628 ±0.16	0.035
	T14	10.9	MIT (20° C)	22.2 40.6	8-57 (4 inc.)	0.087 ±0.02	0.075 ±0.014	0.081 ±0.02	-	0.271 ±0.09	0.033
	T15	8	MIT (20° C)	41.2 45.2	4-58 (5 inc.)	0.051 ±0.01	0.056 ±0.01	0.054 ±0.01	-	0.267 ±0.04	0.017
	T16	10.4	MIT (20° C)	23.1 39.5	8-57 (4 inc.)	0.078 ±0.02	0.072 ±0.02	0.075 ±0.02	-	0.28 ±0.07	0.030
	T17	7.3	MIT (ΔT)	37.6 52.7	16-33 (20° C) 8-43 (3 inc.) @ 1.4° C	0.064 0.043 ±0.01	0.053 0.031 ±0.001	0.059 ±0.01 0.037 ±0.004	0.473	 0.393 ±0.16 (4 inc.)	0.04 0.034
	T18	11	MIT (20° C)	23.8 39.5	8-57 (4 inc.)	0.099 ±0.01	0.108 ±0.03	0.103 ±0.02	-	0.263 ±0.06	0.024
	T19	8.5	MIT (20° C)	27.9 45.2	8-294 (7 inc.)	0.093 ±0.05	0.086 ±0.16	0.091 ±0.03	-	0.294 ±0.06	0.023
W	W12	16.3	MIT (20° C)	28.8 48	8-294 (7 inc.)	0.094 ±0.03	0.081 ±0.01	0.088 ±0.02	-	0.258 ±0.09	0.019

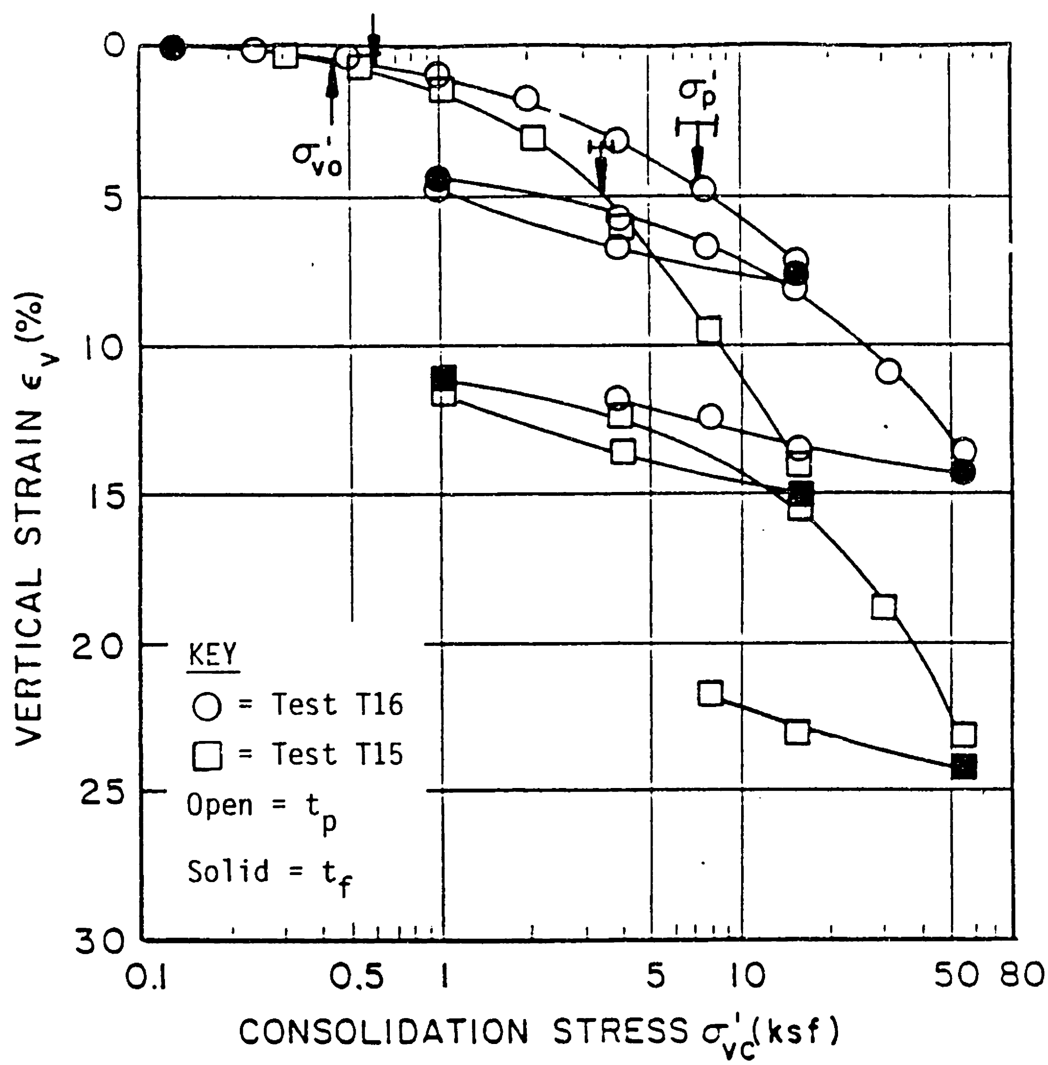


Figure 4-1: Compression Curves Comparing Upper and Lower Soil Samples from Site T, Smith Bay

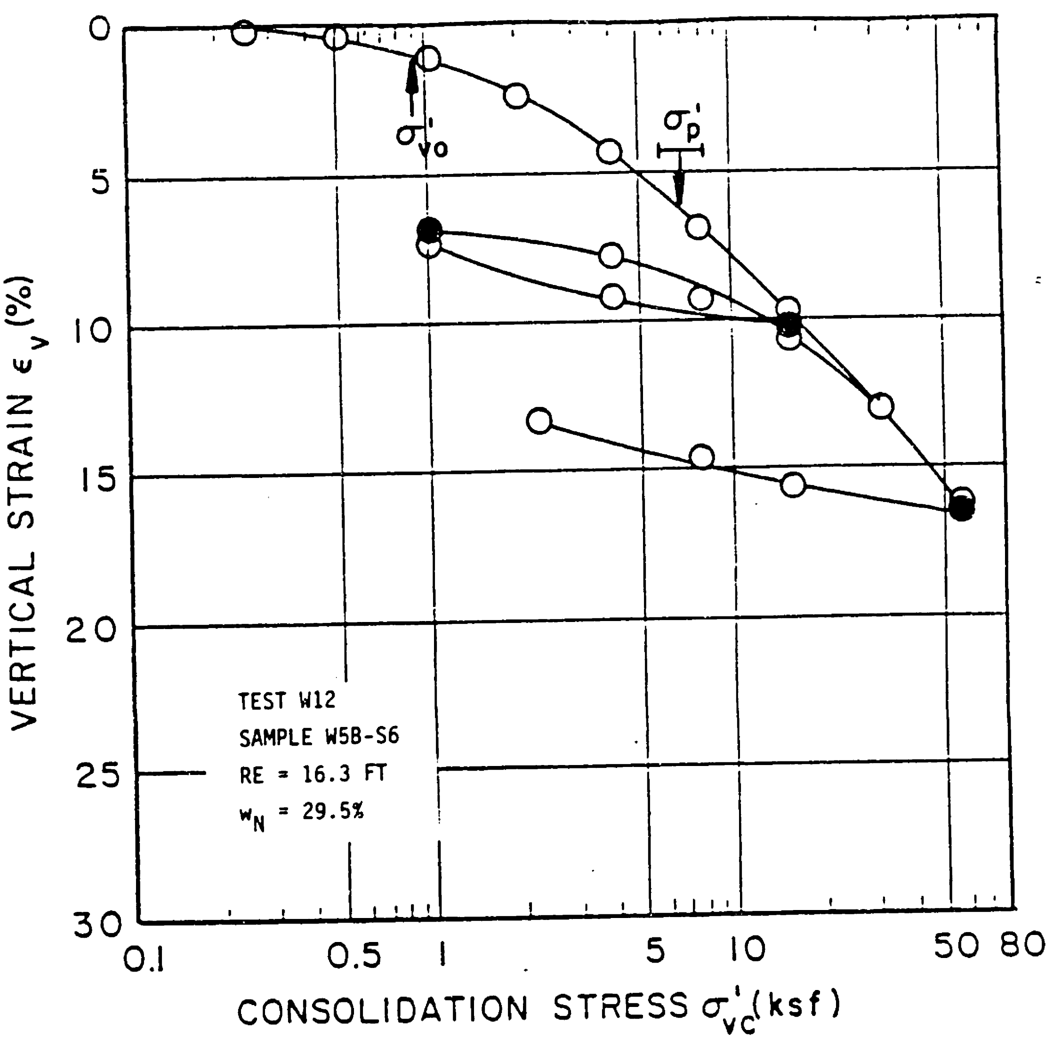


Figure 4-2: Compression Curve from Site W, Smith Bay

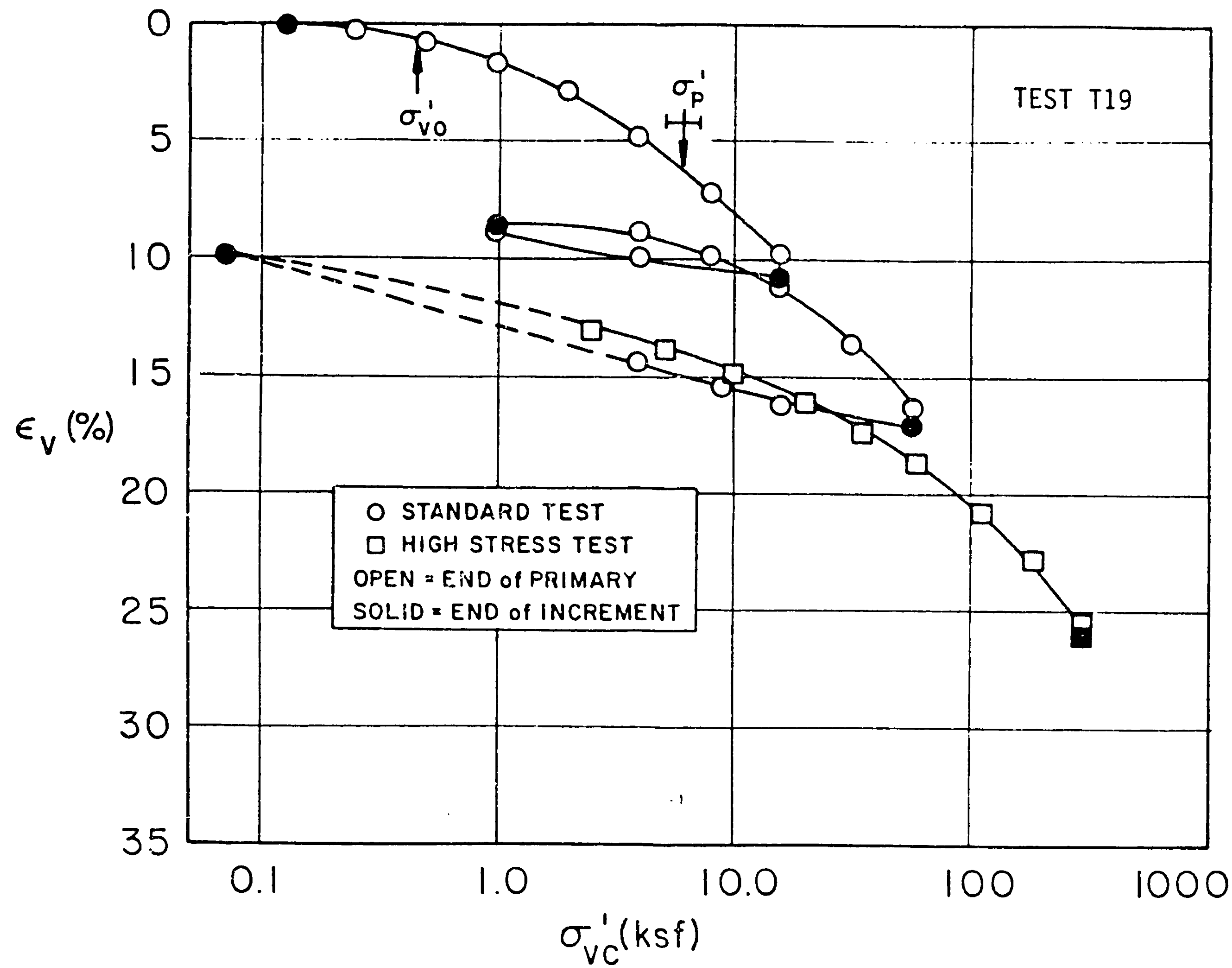


Figure 4-3: Compression Curve from High Stress Oedometer Test T19, Site T, Smith Bay

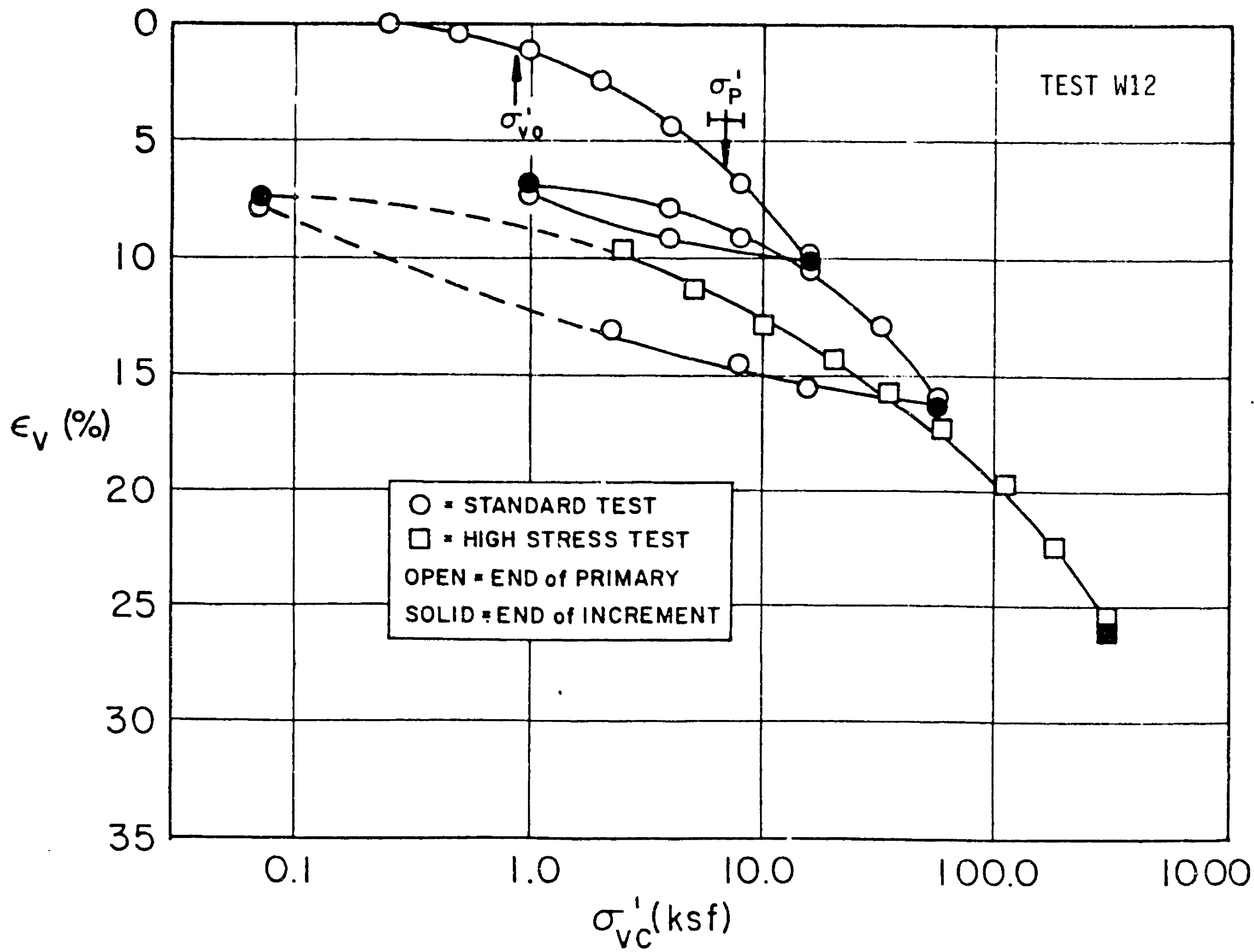


Figure 4-4: Compression Curve from High Stress Oedometer Test W12, Site W, Smith Bay

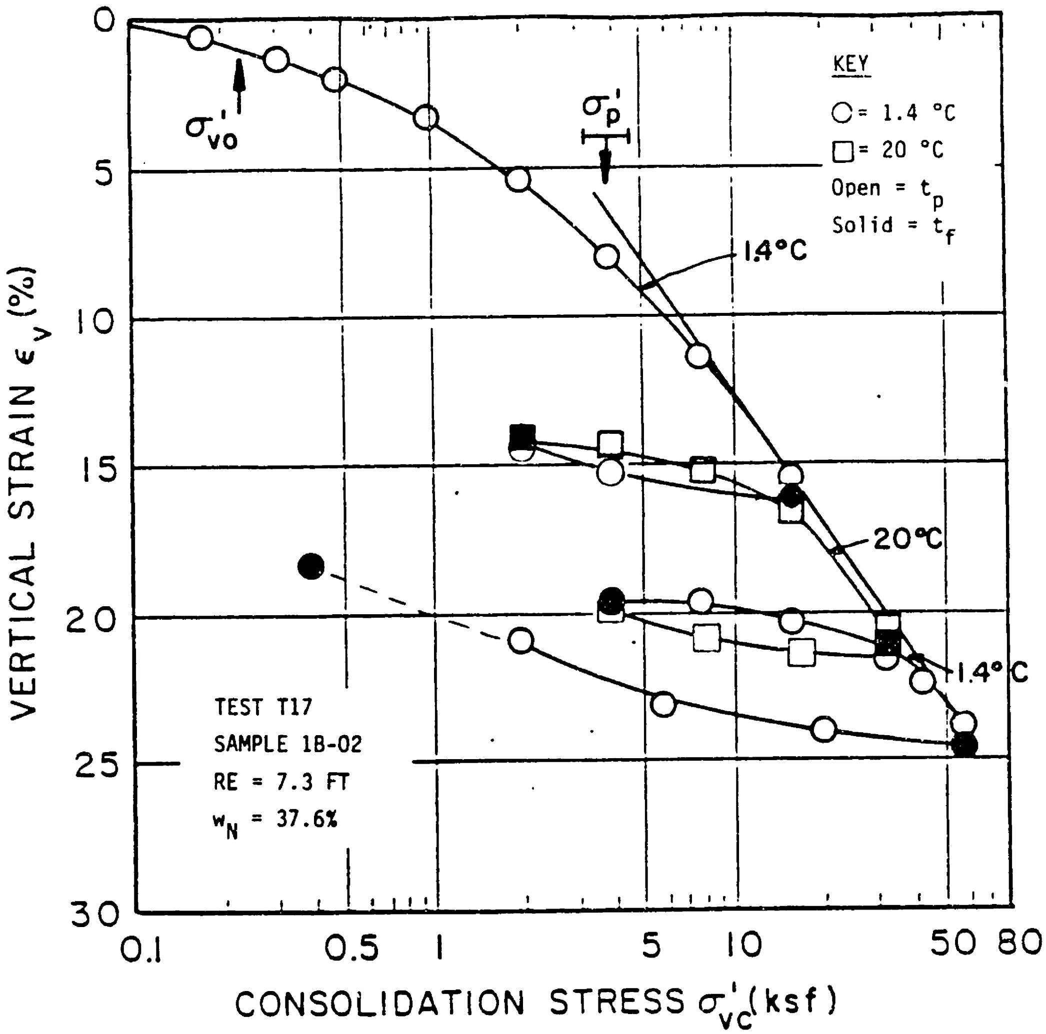


Figure 4-5: Compression Curve for Temperature Controlled Oedometer Test, Site T, Smith Bay

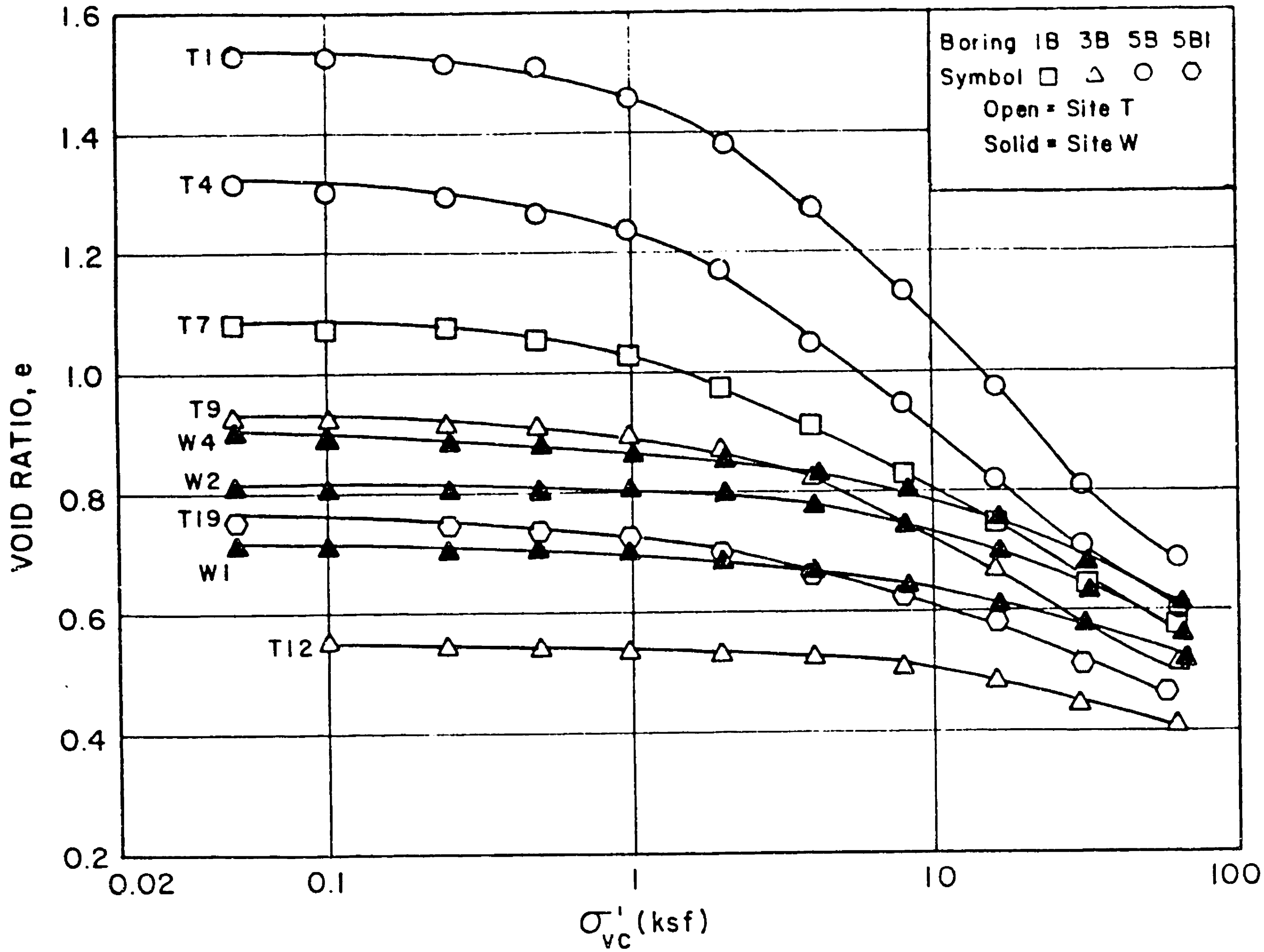


Figure 4-6: e - logσ'_{vc} Compression Curves from Selected Smith Bay Samples

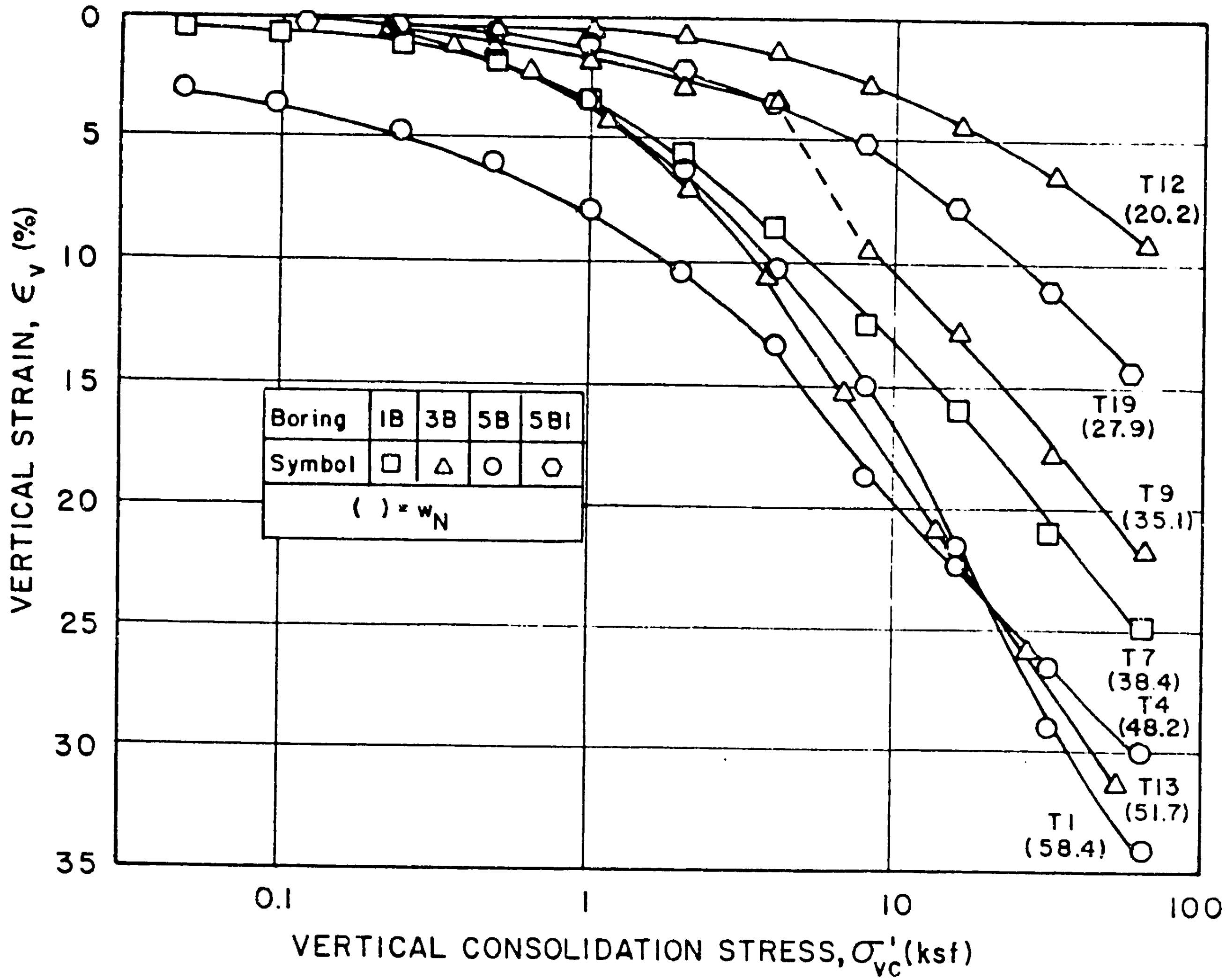


Figure 4-7: ϵ - logσ'_{vc} Compression Curves from Selected Smith Bay Samples

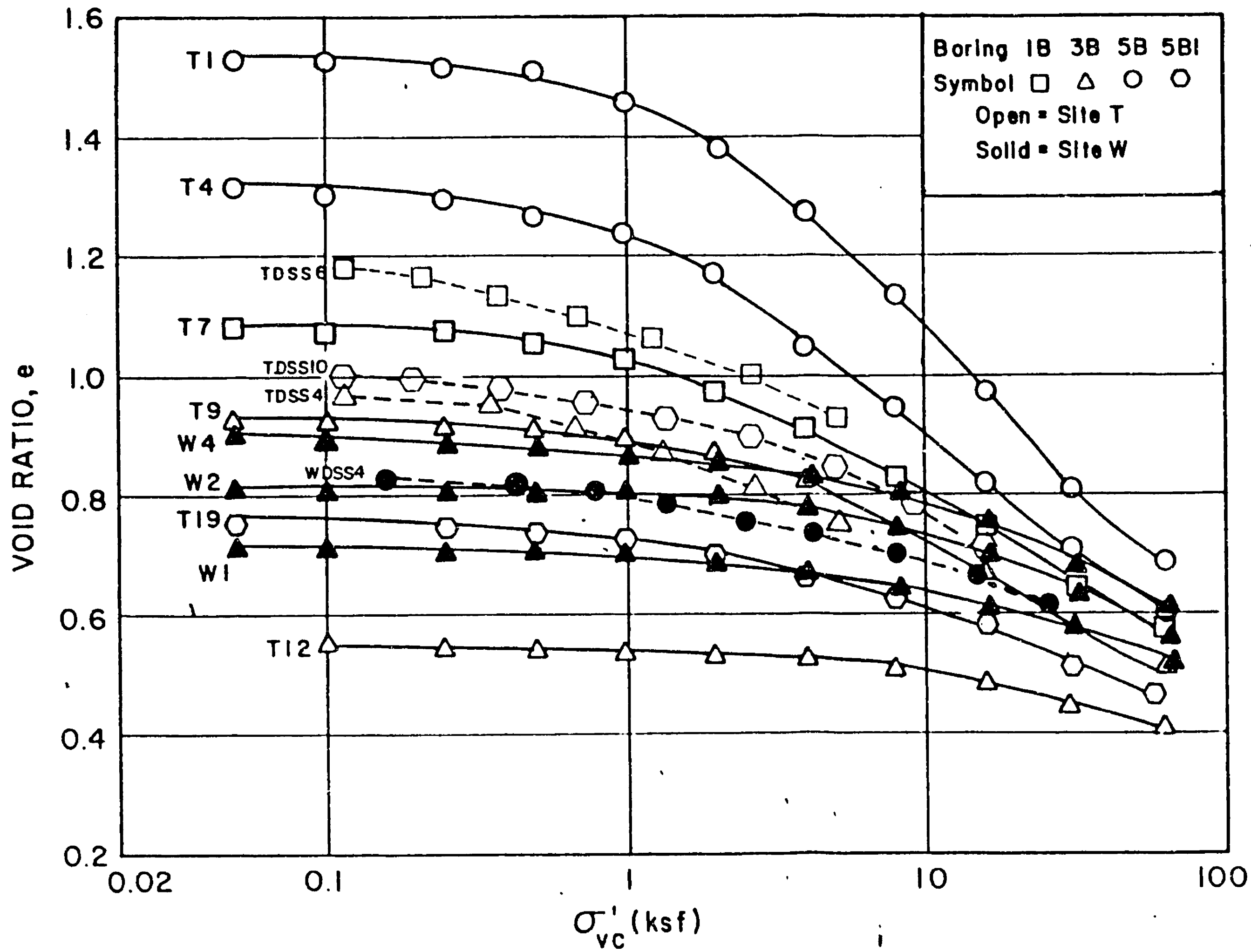


Figure 4-8: Compression Curves from DSS Tests Compared to Oedometer Results

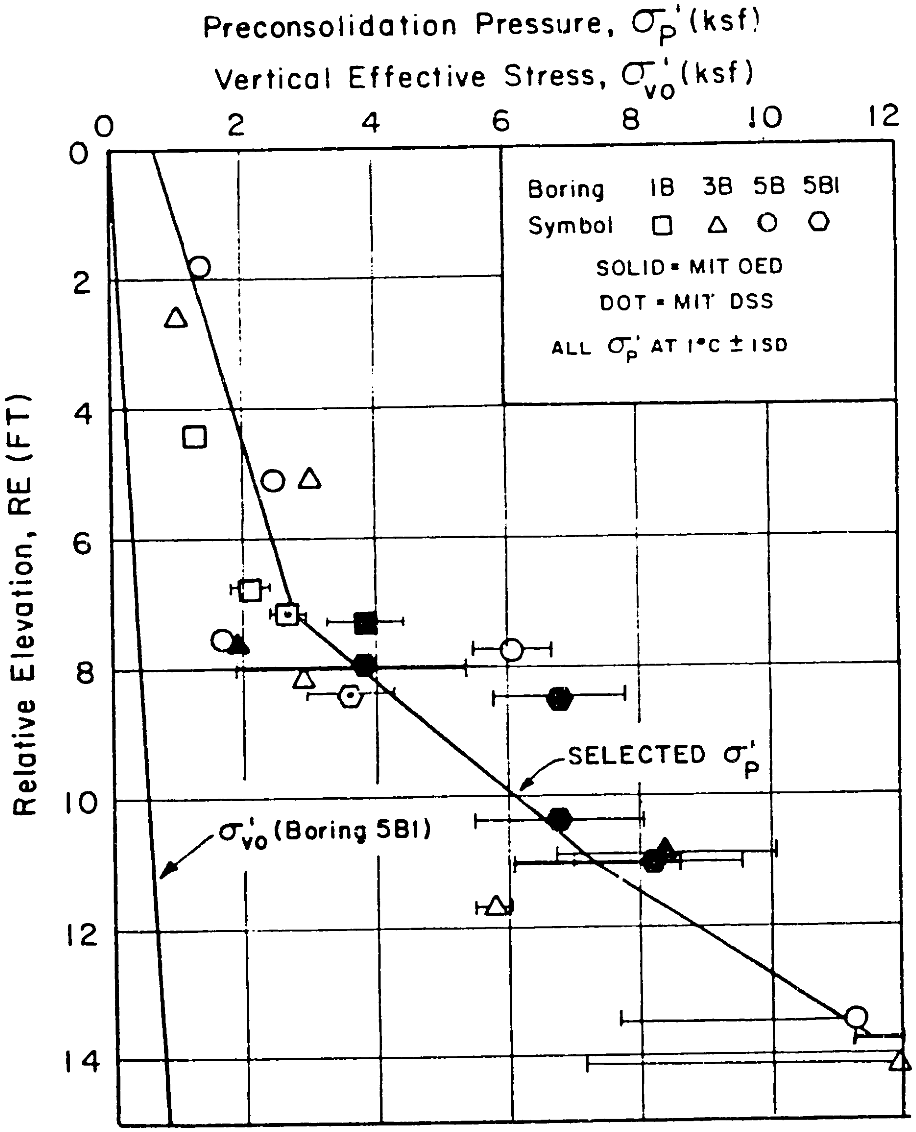


Figure 4-9: Stress History Profile for Site T

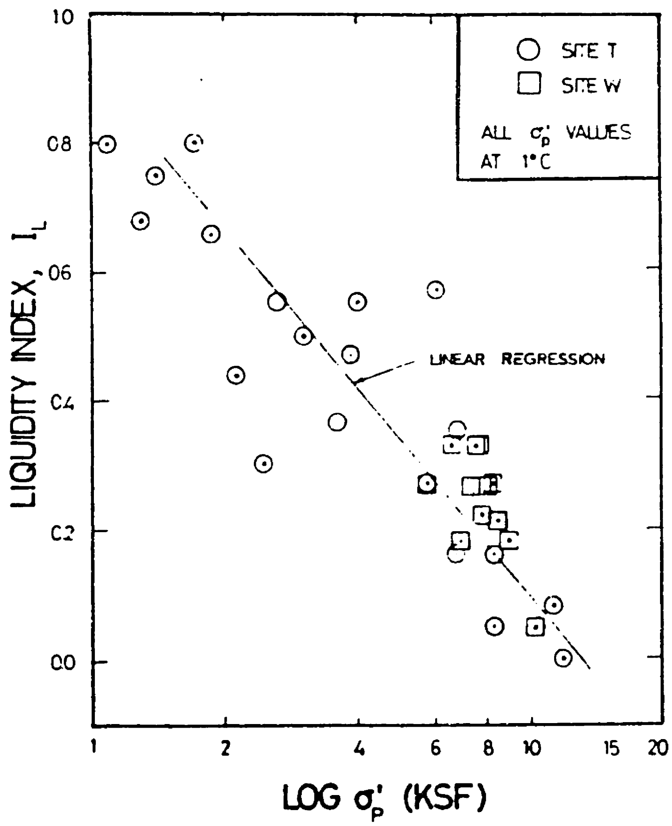


Figure 4-10: Liquidity Index versus Preconsolidation Pressure for Sites T and W

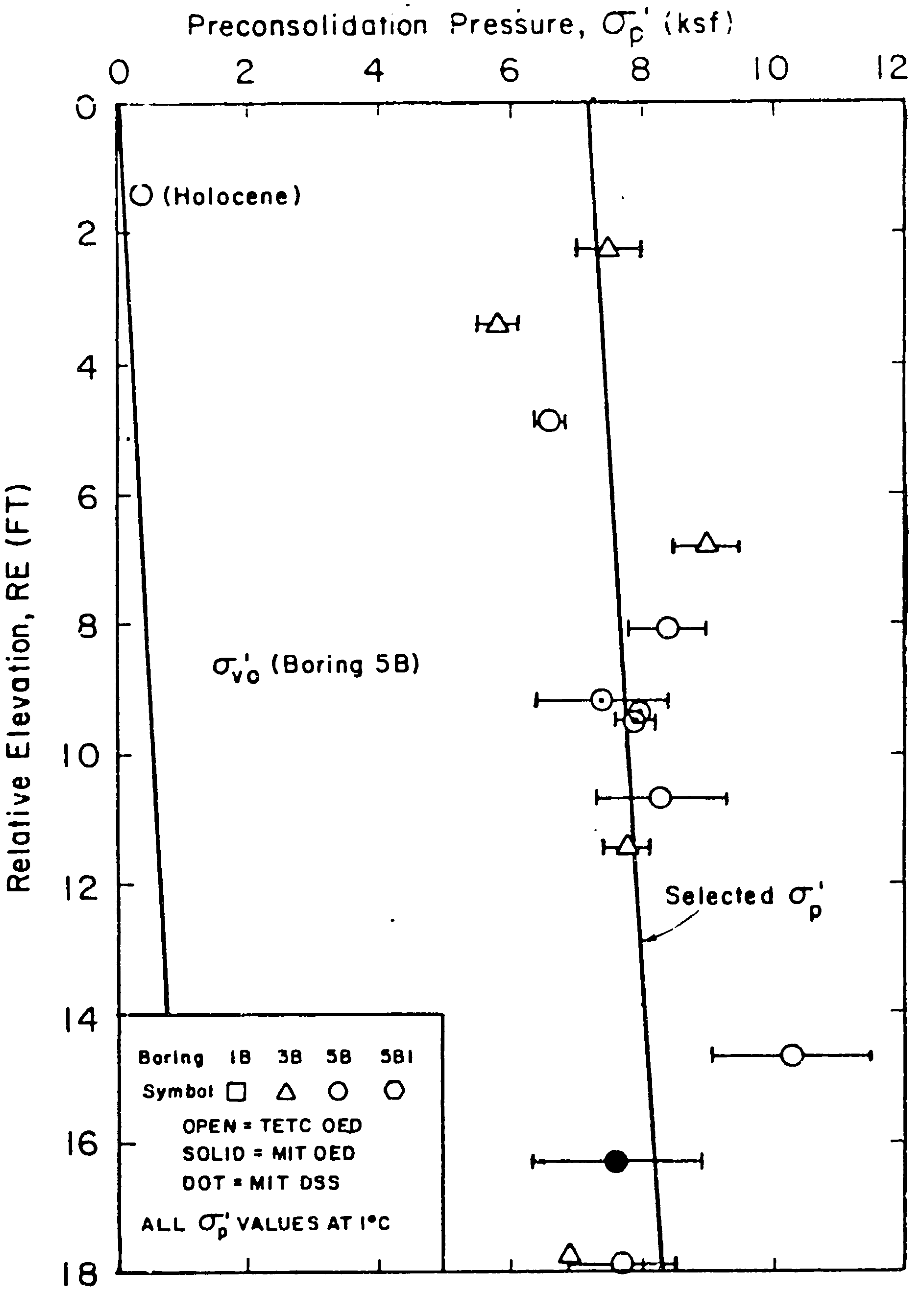


Figure 4-11: Stress History for Site W

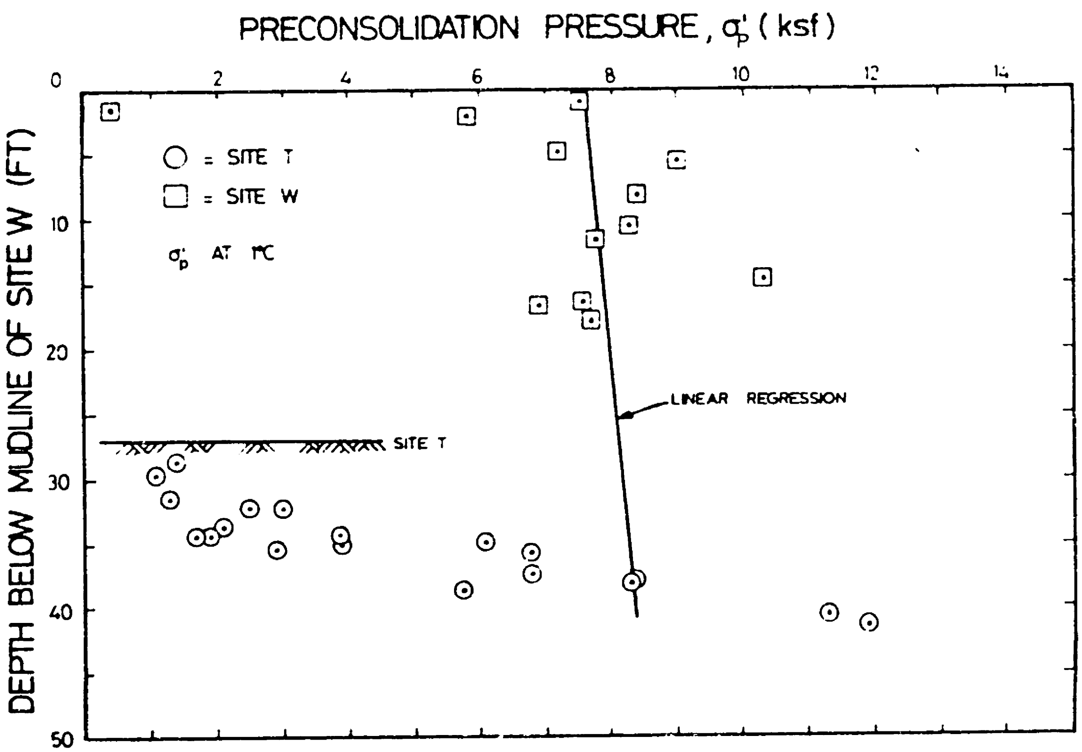


Figure 4-12: Comparison of Stress Histories at Sites T and W

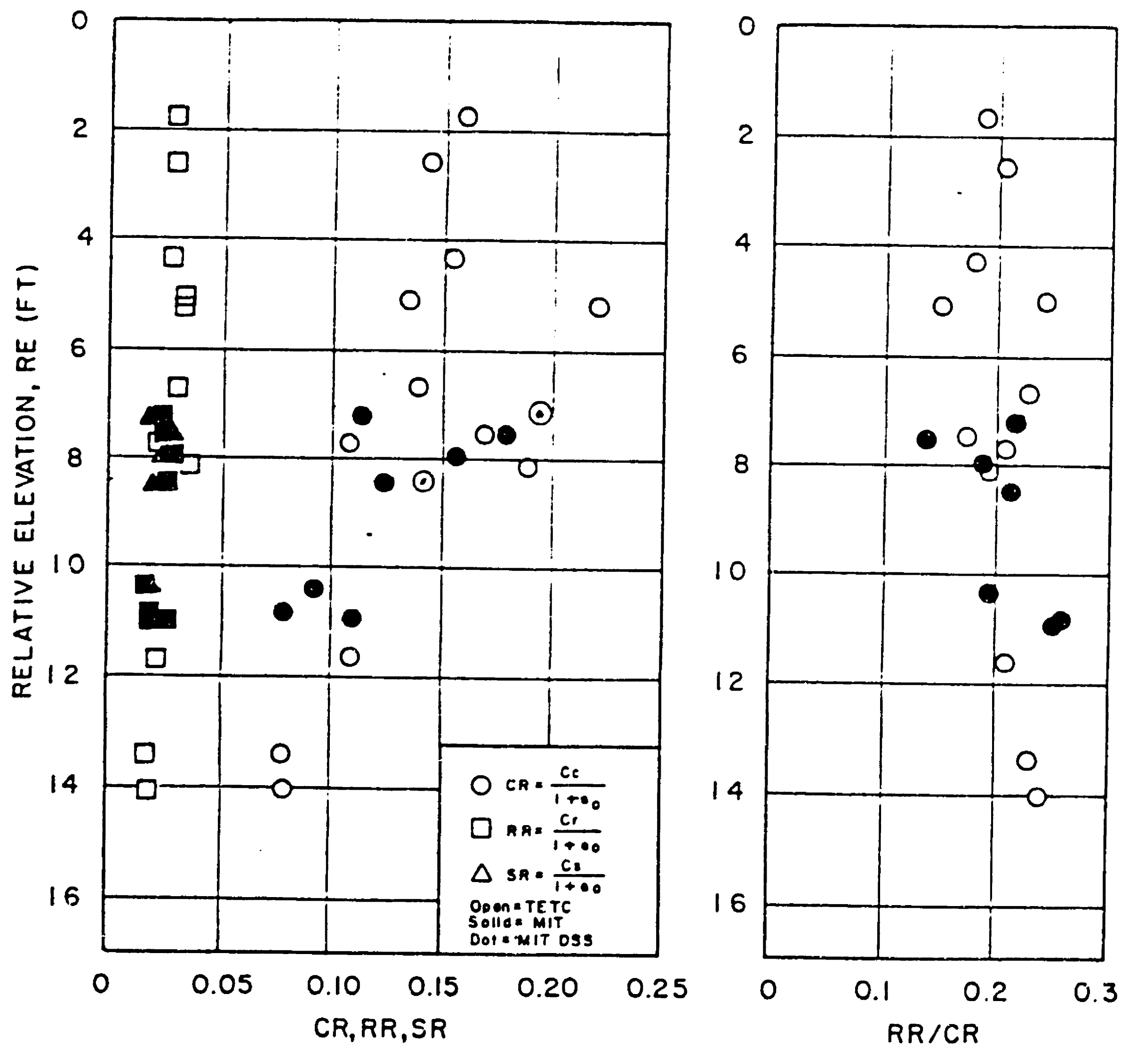


Figure 4-13: Compressibility Parameters for Site T

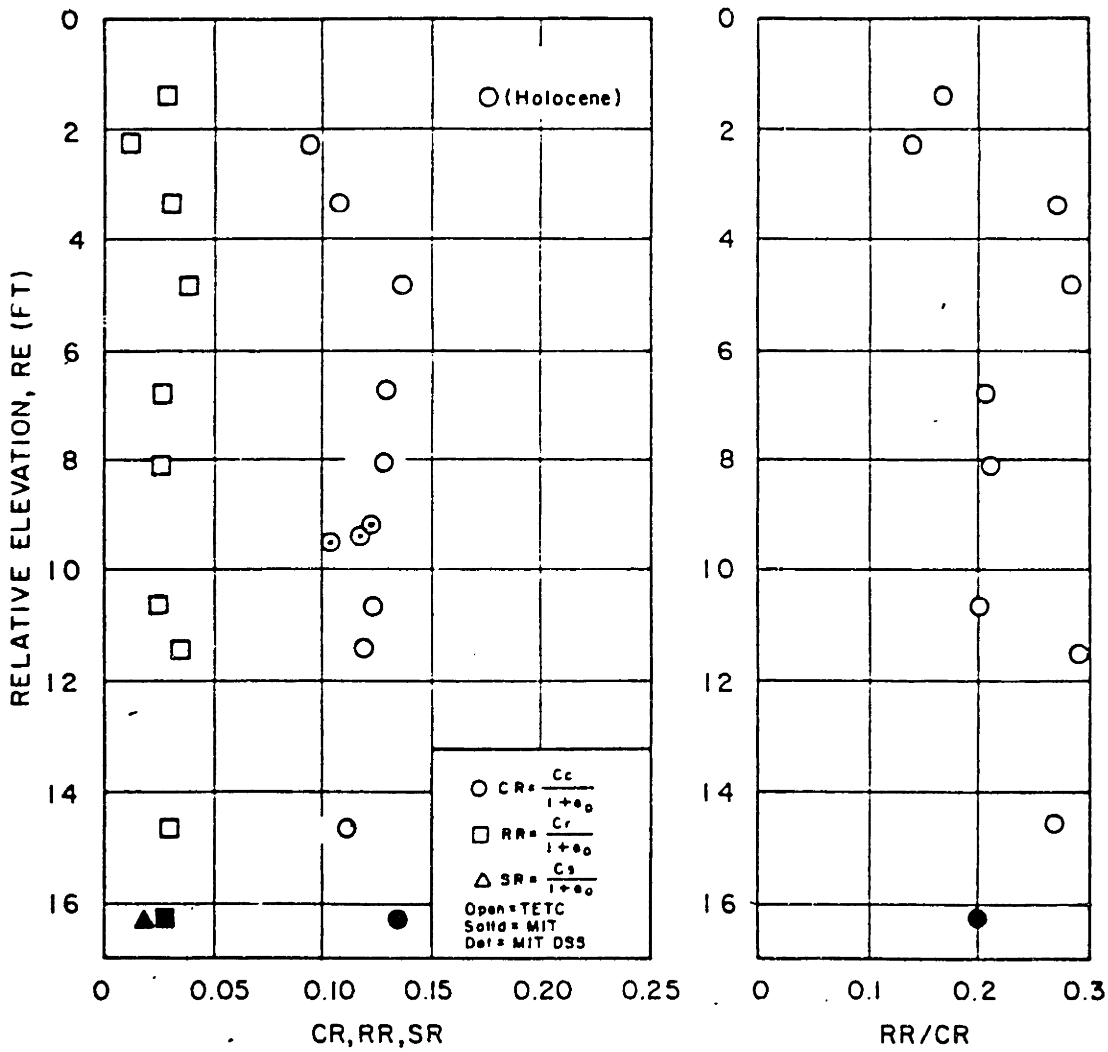


Figure 4-14: Compressibility Parameters for Site W

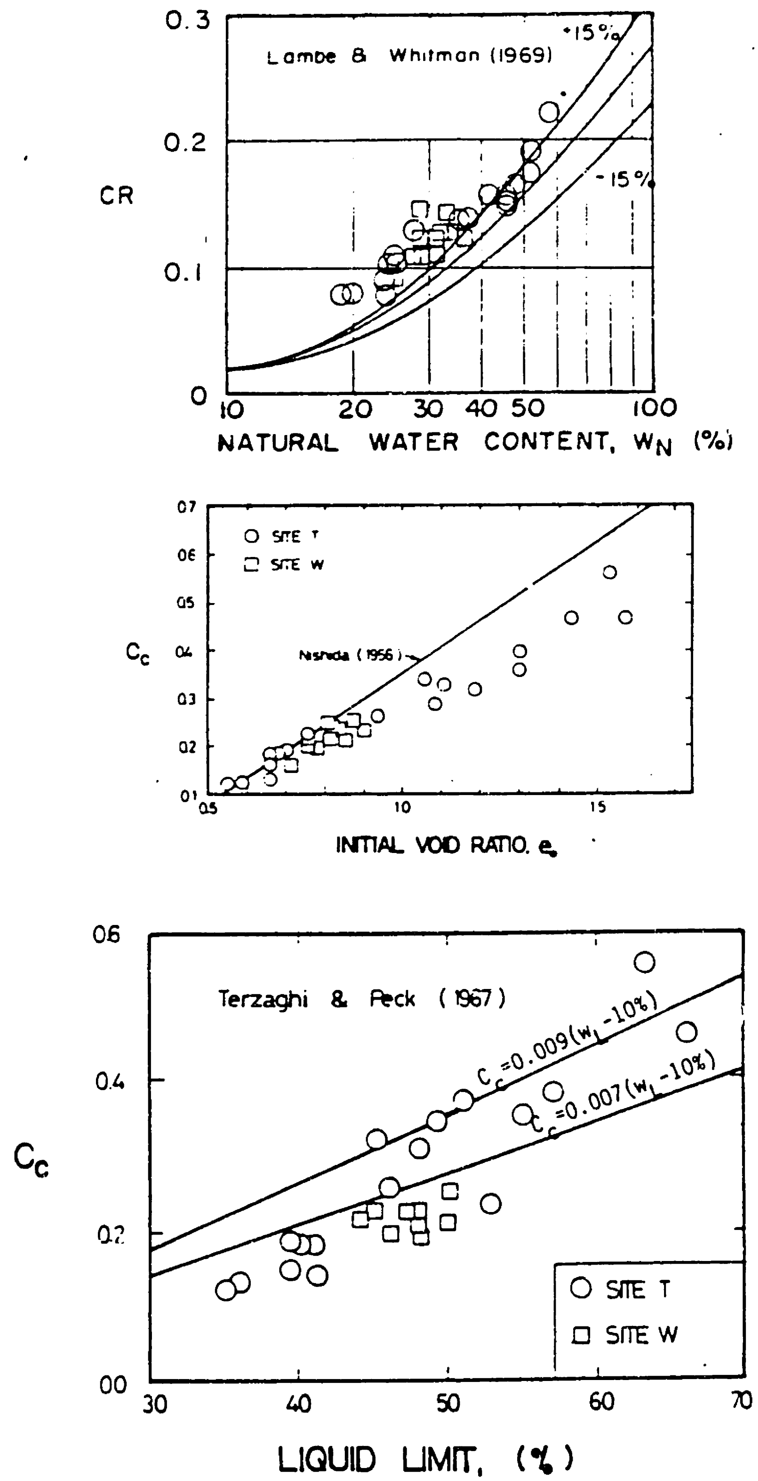


Figure 4-15: Empirical Correlation of Compressibilty Parameters for Smith Bay

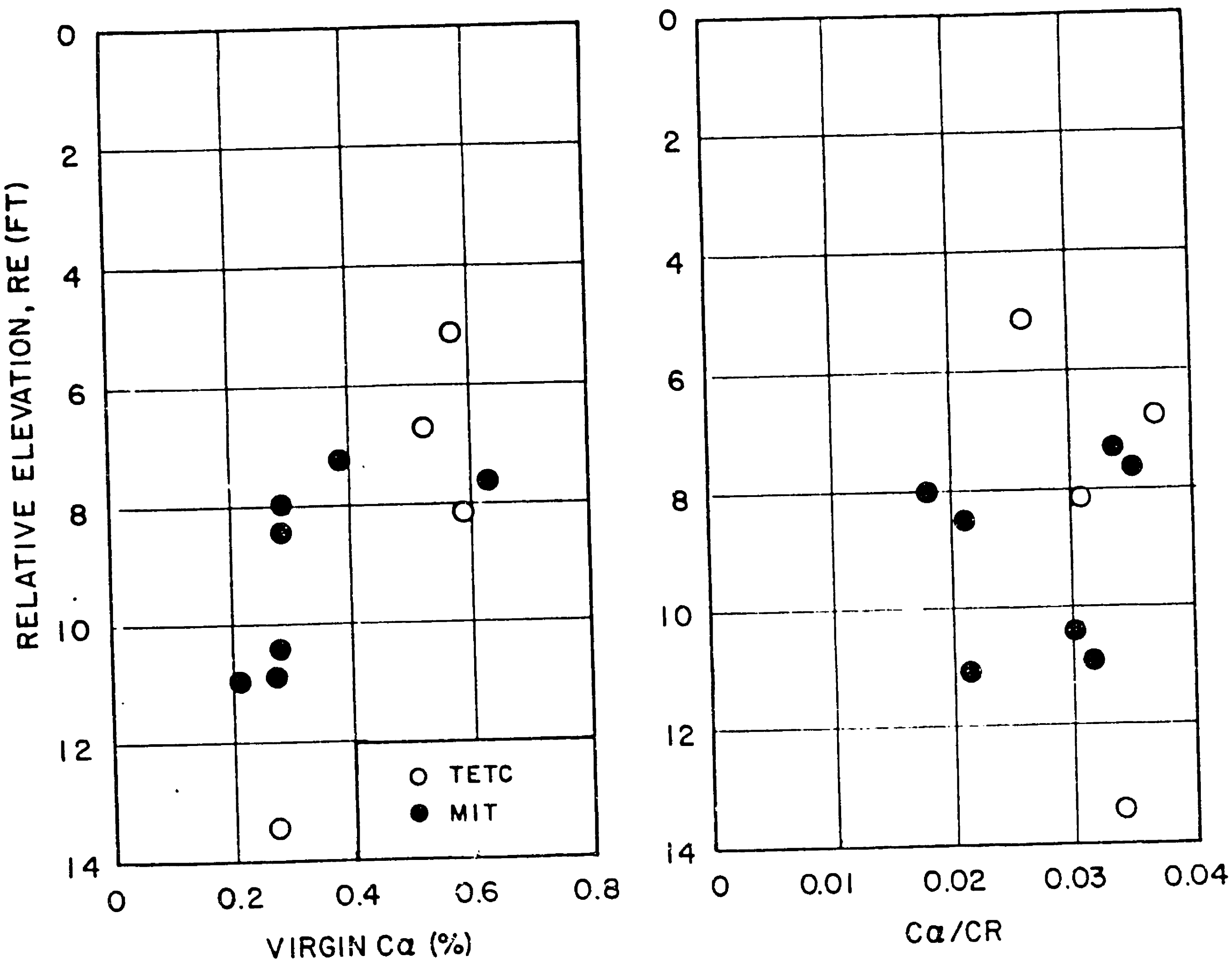


Figure 4-16: Profile of Coefficient of Secondary Compression for Smith Bay

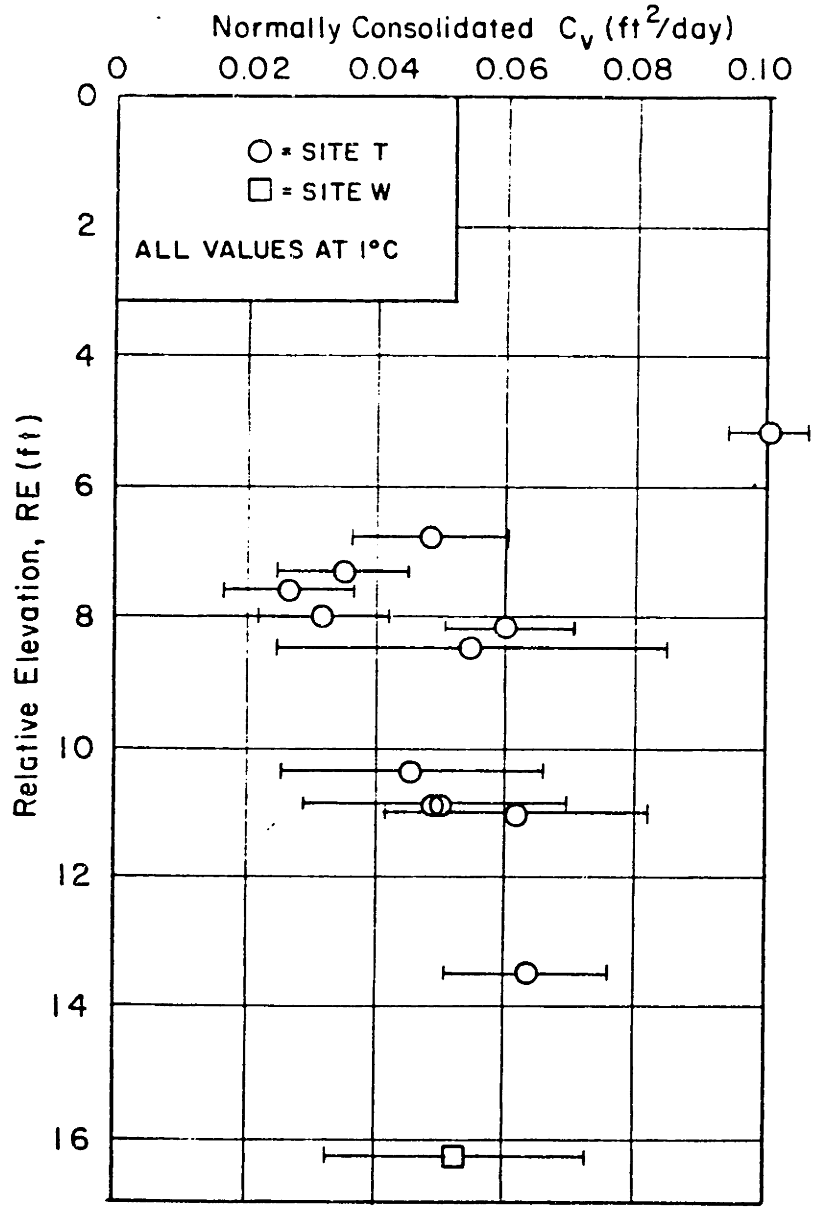


Figure 4-17: Profile of Average Normally Consolidated Coefficient of Consolidation for Smith Bay

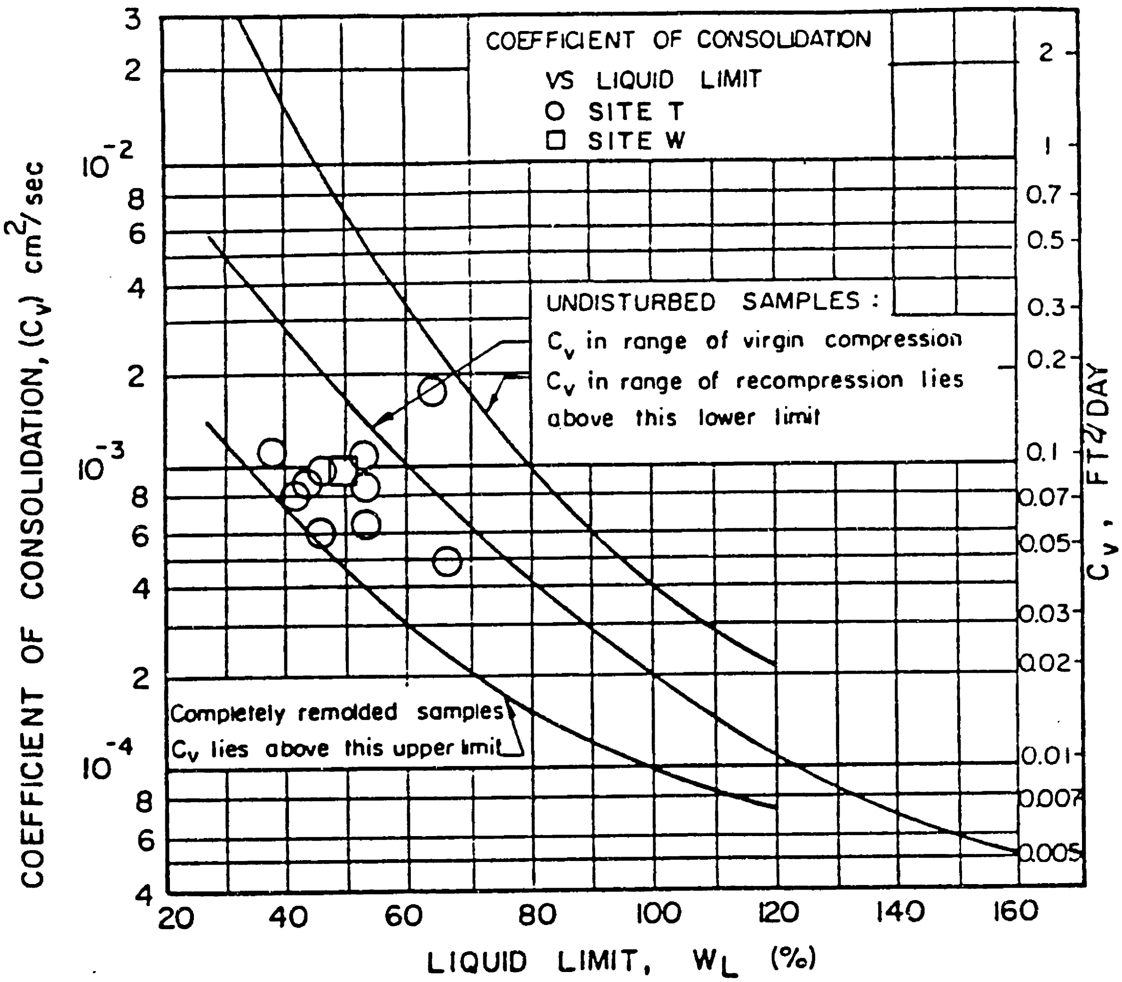


Figure 4-18: Correlation of Coefficient of Consolidation and Liquid Limit for Mukluk SZA (from NAVFAC DM7-1, 1982)

Chapter 5

CONSOLIDATED UNDRAINED TRIAXIAL TEST PROGRAM

5.1 SCOPE AND OBJECTIVES

Fourteen consolidated-undrained triaxial compression tests were conducted on samples from Smith Bay (10 at Site T, 4 at Site W). TETC performed a series of Recompression tests at both sites. MIT performed additional Recompression tests at Site T to supplement the TETC results and conducted a CUC test program on Site T material at the request of EBA (the design consultant for SOPC). EBA requested this information for use in calibrating its soil model that will be employed to predict foundation behavior during and after installation and under ice loadings. A description of the types of tests performed follows.

<u>SHANSEP Normally Consolidated CIUC and CK_0UC</u>: One isotropically consolidated-undrained triaxial compression (CIUC) test was performed on ice gouged material at Site T using a consolidation stress 10 times greater than the in situ preconsolidation pressure. A companion sample was K_0 consolidated to evaluate the effects of isotropic versus K_0 consolidation.

<u>Overconsolidated CIUC</u>: Two isotropically consolidated-undrained triaxial compression (CIUC) tests were performed on ice gouged material at Site T on samples consolidated into the normally consolidated range and then rebounded to overconsolidation ratios, OCR, of 2 and 5.

<u>Recompression</u>: Ten Recompression consolidated-undrained tests involved consolidation of samples to approximately the in situ vertical effective stress followed by shearing in triaxial compression. Six tests were performed at Site T (2

by TETC, 4 by MIT) and four tests at Site W. Results of these tests will be compared with strength predictions using the SHANSEP method. In addition, TETC ran one Recompression triaxial extension test at each site. Due to questions about test quality, these results were not included in the comparisons.

In addition to the tests described above, two CIUC tests were run by MIT on nongouged soil at Site T. Although intended to be OCR = 1 tests, the specimens were not consolidated to high enough stresses to achieve normally consolidated behavior. The text will not discuss these tests further. Tabulated data for all MIT tests are presented in Appendix D. Details of TETC's test procedures for the triaxial testing program are included in Appendix B.

Tables 5-1 and 5-2 provide a summary of the data obtained from the triaxial testing program. This chapter presents and discusses the results of these tests.

5.2 MIT TEST EQUIPMENT AND PROCEDURES

The MIT triaxial equipment and test procedures are described in detail by Ayan (1985). The tests were performed at room temperature in Wykeham Farrance triaxial cells modified by MIT to better suit research needs. Vertical displacements were measured using a direct current displacement transducer (DCDT). Cell pressure and back pressure were monitored by a common pressure transducer. This minimizes the error in determination of effective stresses that involve a small difference between two large pressures. A double burette system of water and silicon oil was used to measure volume changes. Back pressure and cell pressure were applied using mercury pot systems. The vertical consolidation stress was controlled by loading weights onto a hanger connected to the piston. A load cell was used to monitor forces during shear under a constant strain rate.

Locations within each tube selected for triaxial testing were made after careful study of radiographs. Each sample was trimmed in a humid room using a wire saw and mitre box to final dimensions of 1.4-inch diameter and 3-inch height. Filter strips were used to increase the rate of consolidation for the CIUC tests. In the Recompression tests, however, filter strips were not used as consolidation was minor. Two prophylactics were used as thin membrane coverings on each sample and sealed in place with rubber o-rings. Cell water was added and a cell pressure of 0.4 ksf was immediately applied with the piston locked in place. Drainage lines were kept closed and the pore pressure was monitored. If negative pore pressures developed, the cell pressure was increased with the piston locked in place. The sample was left in this state overnight, after which backpressure was applied equivalent to the pore pressure which had developed.

Testing proceeded in three stages: backpressuring, consolidation, and shearing. Samples were backpressured in increments of 0.4 ksf or less to a final value of 4 ksf; during this process the effective stress was maintained at a constant level equal to that developed by the sample prior to backpressuring. B values ($B = \Delta u/\Delta\sigma_c$) were measured after backpressuring to check the degree of saturation.

Consolidation procedure for the <u>isotropic</u> tests consisted of increasing the cell pressure and adding weights to the piston hanger in equivalent increments to maintain an isotropic state of stress on the sample. After applying each increment, the top and bottom drainage lines were opened to allow consolidation to proceed.

The <u>K_0 (one-dimensional)</u> consolidation procedures were more complex and time consuming (see Ayan, 1985). The K_0 condition is defined as no lateral strain ($\epsilon_2=\epsilon_3=0$), and is satisfied if the calculated average cross-sectional area of the sample remained constant. The vertical effective stress was increased incrementally by about 20% of the previous vertical effective stress value. For each increment, an isotropic stress equal to $\Delta\sigma_3$ was first applied. Pore pressures were allowed to dissipate for about 30 seconds before application of the deviatoric component (= $\Delta\sigma_1$ - $\Delta\sigma_3$). Inadvertent failure of the sample was avoided by applying the isotropic component first and using small increments. This procedure is described in more detail by Fayad (1986).

The Recompression tests used consolidation methods similar to those used for isotropic consolidation. The vertical effective consolidation stress was set as near as possible to the estimated in situ effective vertical stress. As this stress was generally small, most tests required only one consolidation increment.

To standardize the amount of secondary compression for each test, the last consolidation increment was left on for 24 hours prior to shearing. Lab procedures followed after consolidation of samples were similar for all test types. Shear was displacement controlled at about 0.5% axial strain per hour, a rate estimated to be low enough to allow pore pressure equilibration. Shearing was continued until the formation of a failure surface or until the piston reached its limit of travel.

A central data acquisition system recorded voltages corresponding to axial load, cell pressure, pore pressure, vertical displacement and input voltage. Reduction and plotting of the results was done using a companion HP87. The parabolic area correction was used in the calculations. Corrections for filter strips and membrane restraint were also made where applicable.

Following shear, most samples were radiographed to determine post-shear shape and to locate any internal anomalies (pebbles, layering).

5.3 TESTS ON NORMALLY CONSOLIDATED SAMPLES

The index properties and results from the two normally consolidated, CUC shear tests performed by MIT are summarized in Table 5-1 and plotted in Figures 5-1 through 5-3. Both specimens were from the highly gouged portion of Site T (RE < 1.5 ft) with similar water contents. Estimates of in situ preconsolidation pressure were made from the linear regression line of oedometer and DSS compression results (Figure 4-9). Consolidation stresses were 7-9 times the estimated σ'_p. Compression curves for both tests are included in Appendix C.

The peak strength for the CIUC test was selected at the end of the test, with ϵ_a = 11.2%. The CK_0UC test reached its peak value at a much lower strain, as shown in Figure 5-1. Stress paths (Figure 5-2) show that the CIUC test exhibits some tendency for dilatancy whereas the CK_0UC test shows none.

Experiments on NC soft clays have shown that K_0-consolidation, when compared to isotropic consolidation, changes the stress-strain-strength behavior in the following manner (Ladd, 1965):

1. q_f/σ'_{vc} changes by $\pm$15%
2. ϕ' and A_f generally decrease
3. ϵ_f decreases substantially and strain softening is more pronounced

The strength of the K_0-consolidated sample is about equal to that measured in the CIUC test. The similar strengths measured for the two tests suggest that isotropically consolidated tests should provide reasonable estimates of strengths in the gouged zone.

The strain to peak is also less for the CK_0UC test. However, the stress-strain curve for the K_0-consolidated test is very flat, so the peak is not well-defined. This results in poorly defined values of ϕ' and A_f. The value of ϕ' for the CIUC test appears too low. The K_0 consolidated sample had a much stiffer response than the CIUC test, as shown in Figure 5-3.

5.4 OVERCONSOLIDATED CIUC TRIAXIAL TESTS

Two samples from Site T (gouged) were consolidated to a consolidation stress 6 to 7 times greater than the estimated preconsolidation pressure, then rebounded to OCR = 2 and 5. Compression curves for these tests are included in Appendix C. The results, summarized in Table 5-1, were used to determine the m value for use in the relationship:

$$q_f/\sigma'_{vc} = S\ (OCR)^m$$

where $S = q_f/\sigma'_{vc}$ for NC soil. (Note: Section 7.2 presents the basis for using this relationship)

Figures 5-4 through 5-6 compare results of CIUC tests at OCR = 1,2 and 5. The stress-strain curves have a consistent increase in strength with OCR. The stress paths in Figure 5-5 show increased dilatancy with increasing OCR as would be expected. Normalized modulus values are plotted in Figure 5-6 for CIUC tests at OCR = 1, 2 and 5, with the NC CK_0UC test included for comparison.

Figure 5-7 plots log normalized strength versus log OCR. Linear regression on CIUC and CK_0UC data gives S = 0.32 and m = 0.78. For S = 0.32, the test at OCR = 2 gives an m value of 0.86, which seems too high. The backcalculated m from the OCR = 5 test is equal to 0.76, a more reasonable value. Based on judgment, the selected m = 0.76 and S = 0.32. These values will be used in conjunction with the stress history profile (Chapter 4) to develop strength profiles in Chapter 7 for triaxial compression.

5.5 RECOMPRESSION TRIAXIAL COMPRESSION TESTS

TETC performed two Recompression tests on gouged material at Site T. MIT performed two additional tests on gouged material and two tests on the lower, nongouged portion of Site T. Four Recompression tests were performed by TETC at Site W. A summary of the Recompression test results is contained in Table 5-2.

Consolidation stresses for the MIT tests were chosen based on estimates of overburden pressure. In some cases, the vertical consolidation stress had to be increased in order to balance negative pore pressure of the sample (e.g. Test T5B1T3 had σ'_{vc} of 1.58, compared to σ'_{vo} of 0.590). TETC used σ'_{vc} = 0.43 ksf for five of its tests and 1.24 for the other one. These values were often significantly greater than the estimated σ'_{vo}.

The MIT Recompression tests were performed at room temperature, and sheared in compression at a rate of 0.5% per hour to about 20% strain. The TETC tests were performed at 1 °C, using a rate of 4.8% per hour to a maximum of 20% strain. Details of test procedures followed by TETC are included in Appendix B.

Stress-strain and stress paths for the four Site T, MIT Recompression tests, performed over a range of depths, are shown in Figure 5-8. Strength increases continuously until the end of the test. Strengths from Recompression tests will be compared with SHANSEP predictions in Chapter 7.

5.6 DISCUSSION OF TRIAXIAL COMPRESSION RESULTS

Figure 5-9 shows the peak q-p' values for the MIT and TETC tests. The data fit together very well, with a' generally between 25° and 30° (ϕ' = 28° - 35°). It is rather surprising that all MIT Recompression and SHANSEP tests fall near the same envelope, considering that tests include both NC and heavily OC samples from the highly gouged and deeper, less gouged zones. The dashed line represents the linear regression through the MIT values (r^2 = 0.995). TETC data for Site T lie on or significantly above the linear regression line. Site W results are plotted for comparison. Although less consistent, these data agree quite well with the trends established for Site T.

SHANSEP parameters for triaxial compression of S = 0.32 and m = 0.77 were used for the non-dilatant (lower) material at Harrison Bay. The value of S selected for Smith Bay is identical, and the selected m is also very close. The value of S selected at Harrison Bay was used with m = 0.76 $\pm$ 0.01 to compare SHANSEP predictions versus measured Recompression q_f.

Table 5-1: SUMMARY OF CIU & CK_oU TRIAXIAL COMPRESSION TESTS

All Stresses in ksf

Test No. (type)	RE (ft.) w_N(%)	I_p (%) I_L	In Situ σ_{vo}' σ_p'	Initial e S(%)	Preshear e w(%)	Test σ_{vc}' K_c	Test OCR σ_{vm}'	ε_a ε_{vol} (%)	At Maximum q ε_a (%)	At Maximum q q/σ_{vc}'	At Maximum q p'/σ_{vc}'	At Maximum q ϕ'	At Maximum q A_f	At Maximum Obliquity ε_a	At Maximum Obliquity q/σ_{vc}'	At Maximum Obliquity p'/σ_{vc}'	At Maximum Obliquity ϕ'	$\frac{E_u(50)}{qf}$	Gi/σ_{vc}' Rf
T5B1T4 (CIUC)	1.45 45.0	23.9 81.2	0.08 1.1(1)	1.215 99.0	0.762 27.7	10.0 1.00	1.0 10.0	11.0 20.4	11.2	0.317	0.701	25.9	0.98	11.2	0.317	0.701	25.9	310	44.6 0.97
T5B1T5 (CK_oUC)	1.12 43.9	23.9 76.6	0.06 1.00	1.174 97.4	0.823 29.9	6.55 0.50	1.0 6.55	16.3 16.1	5.6	0.311	0.648	28.6	1.39	5.8	0.310	0.648	28.6	900	141 1.00
T5B1T6 (CIUC)	2.3 42.8	23.9 72.0	0.127 1.4	1.134 99.7	0.732 26.6	5.0 1.0	2.00 10.00	8.3 18.9	14.2	0.582	1.18	-	0.34	10.1	0.577	1.16	-	370	84.0 0.98
T5B1T7 (CIUC)	4.3 36.7	23.9 46.4	0.237 1.9	0.933 99.0	0.675 28.1	2.43 1.00	4.94 12.0	5.62 13.3	12.4	1.08	2.27	-	-.09	7.0	0.984	2.02	-	90	68.5 0.93

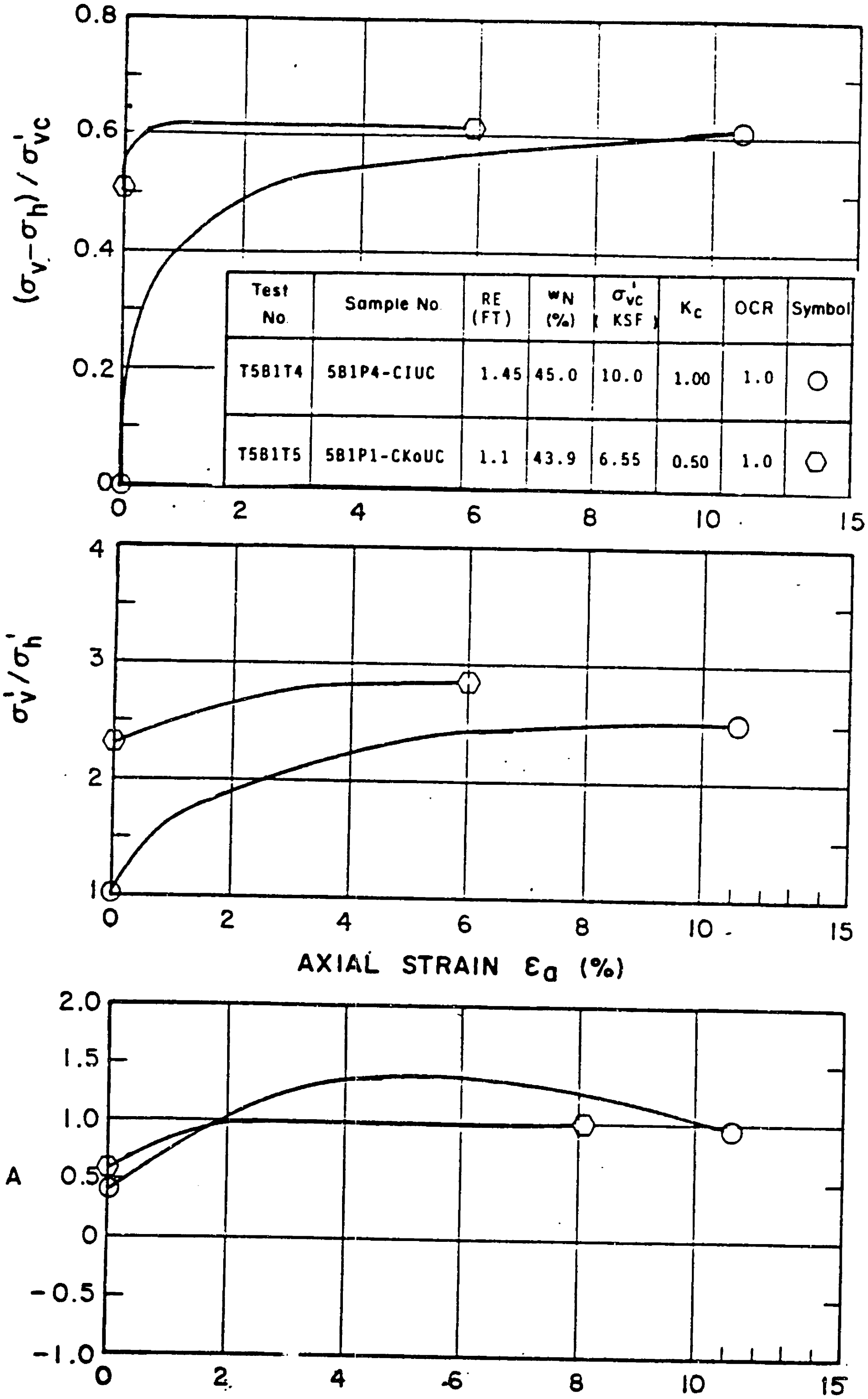
(1) From Linear Regression

Table 5-2: SUMMARY OF RECOMPRESSION TRIAXIAL COMPRESSION TESTS

All Stresses in ksf

Site	Test No.	Boring	RE (ft.)	W_N (%)	Run By	I_p I_L (%)	σ_{vo}' σ_p'	σ_{vc}' K_c	Test OCR	At maximum q			
										ε_a (%)	q/σ_{vc}'	p'/σ_{vc}'	A_f
T	T5B1T1	5B1	1.85	40.3	M	23.9 61.5	0.102 1.40(a)	0.107 0.96	13.7	20.7	1.54	3.29	-0.301
T	T5B1T2	5B1	8.24	33.8	M	21.5 47.0	0.453 3.50(b)	0.688 1.01	5.1	17.7	1.54	2.94	-0.12
T	T5B1T3	5B1	10.7	21.6	M	20.5 12.7	0.590 7.50(c)	1.58 0.82	4.75	19.8	1.82	3.80	-0.34
T	T5B1T8	5B1	10.8	22.1	M	20.5 15.1	0.594 7.50(c)	0.717 1.00	21.4	18.1	2.66	5.51	-0.35
T	T5BP2	5B	4.3	36.4	E	35 24	0.18 1.9(d)	1.24 0.59	1.5	12.2	0.67	1.16	-
T	T5B1P3	5B1	8.5	34.9	E	21.5 52.1	0.47 3.5(b)	0.432 0.53	8.1	11.5	1.85	2.0	-
W	W3BP1	3B	1.9	35.6	E	-	0.039 7.5(e)	0.432 0.53	17.4	11.5	1.52	2.6	-
W	W3BP3	3B	5.9	31.8	E	21.0 22.8	0.259 9.0(f)	0.432 0.67	20.8	16.5	2.42	5.8	-
W	W5BP1	5B	1.65	41.5	E	-	0.091 7.3(d)	0.432 0.67	16.9	16.5	1.83	2.5	-
W	W5BP3	5B	7.6	32.2	E	18 28.9	0.418 8.4(g)	0.432 0.67	19.4	19.5	3.66	6.42	-

(a) From T2, RE=5.2 ft.
(b) From T15, RE=8 ft.
(c) From T18, RE=11 ft.
(d) From Linear Regression
(e) From W1, RE=2.25 ft.
(f) From W3, RE=6.8 ft.
(g) From W8, RE=8.1 ft.



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Symbol
T5B1T4	5B1P4-CIUC	1.45	45.0	10.0	1.00	1.0	○
T5B1T5	5B1P1-CKoUC	1.1	43.9	6.55	0.50	1.0	⬡

Figure 5-1: Normalized Shear Stress, Obliquity and A Parameter vs. Strain for Normally Consolidated CIUC and CK_0UC Tests

Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Symbol
T5B1T4	5B1P4-CIUC	1.45	45.0	10.0	1.00	1.0	○
T5B1T5	5B1P1-CKoUC	1.1	43.9	6.55	0.50	1.0	⬡

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$q = 0.5(\sigma_v - \sigma_h)$

$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$

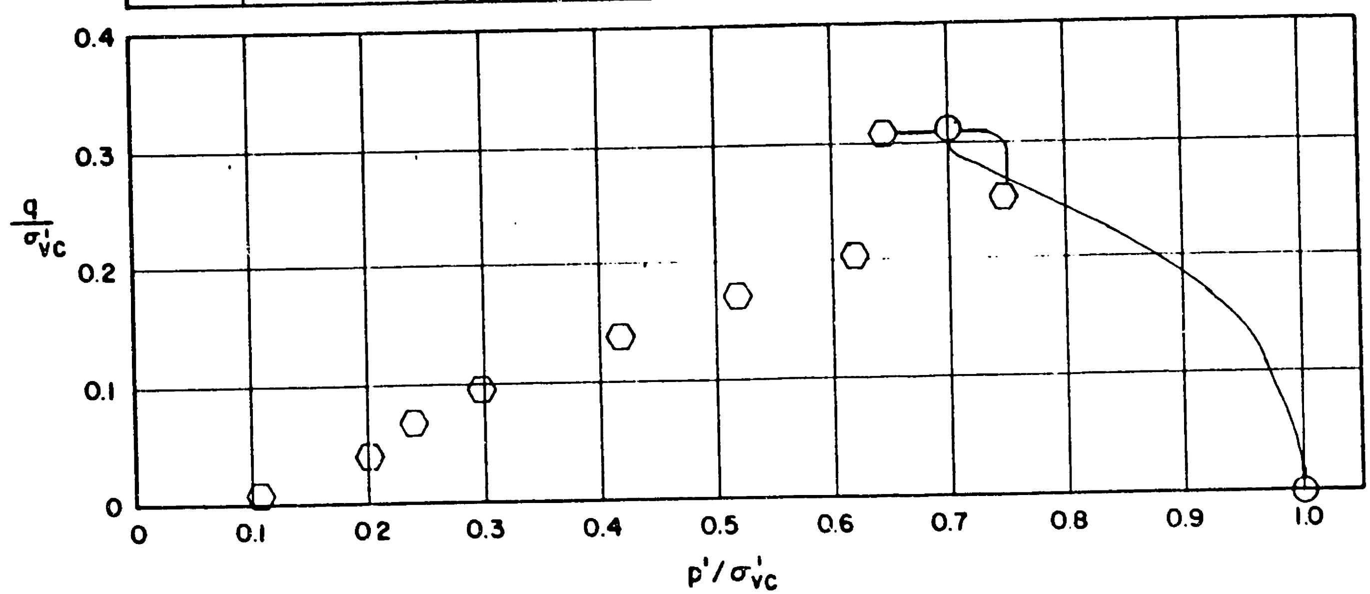
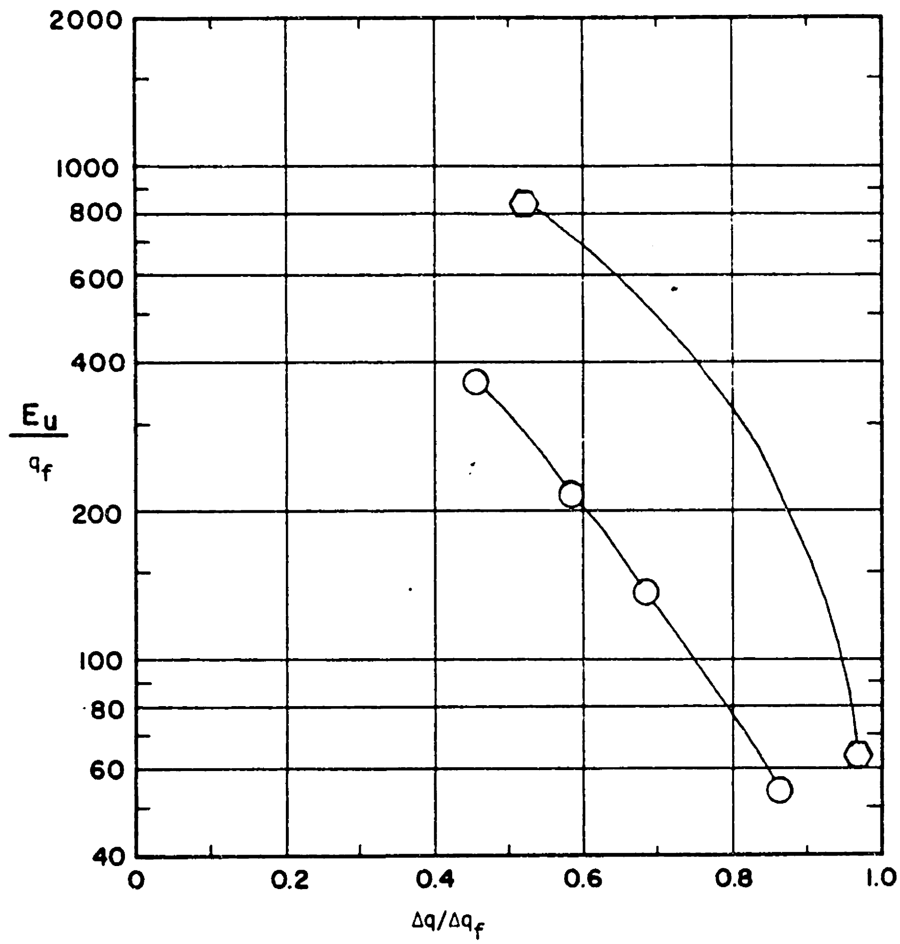


Figure 5-2: Normalized Effective Stress Paths for Normally Consolidated CIUC and CK_0UC Tests



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
T5B1T4	5B1P4	1.45	45.0	10.0	1.0	○
T5B1T5	5B1P1	1.1	43.9	6.55	1.0	⬡

Figure 5-3: Normalized Modulus for Normally Consolidated CIUC and CK_0UC Tests

Test No	Sample No	RE (FT)	w_N (%)	σ'_{vc} KSF	K_c	OCR	Symbol
T581T4	T581P1	1.5	45.0	10.0	1.00	1.0	○
T581T6	T581P1	2.3	42.8	5.0	1.00	2.0	△
T581T7	T581P2	4.3	36.7	2.43	1.00	4.94	□

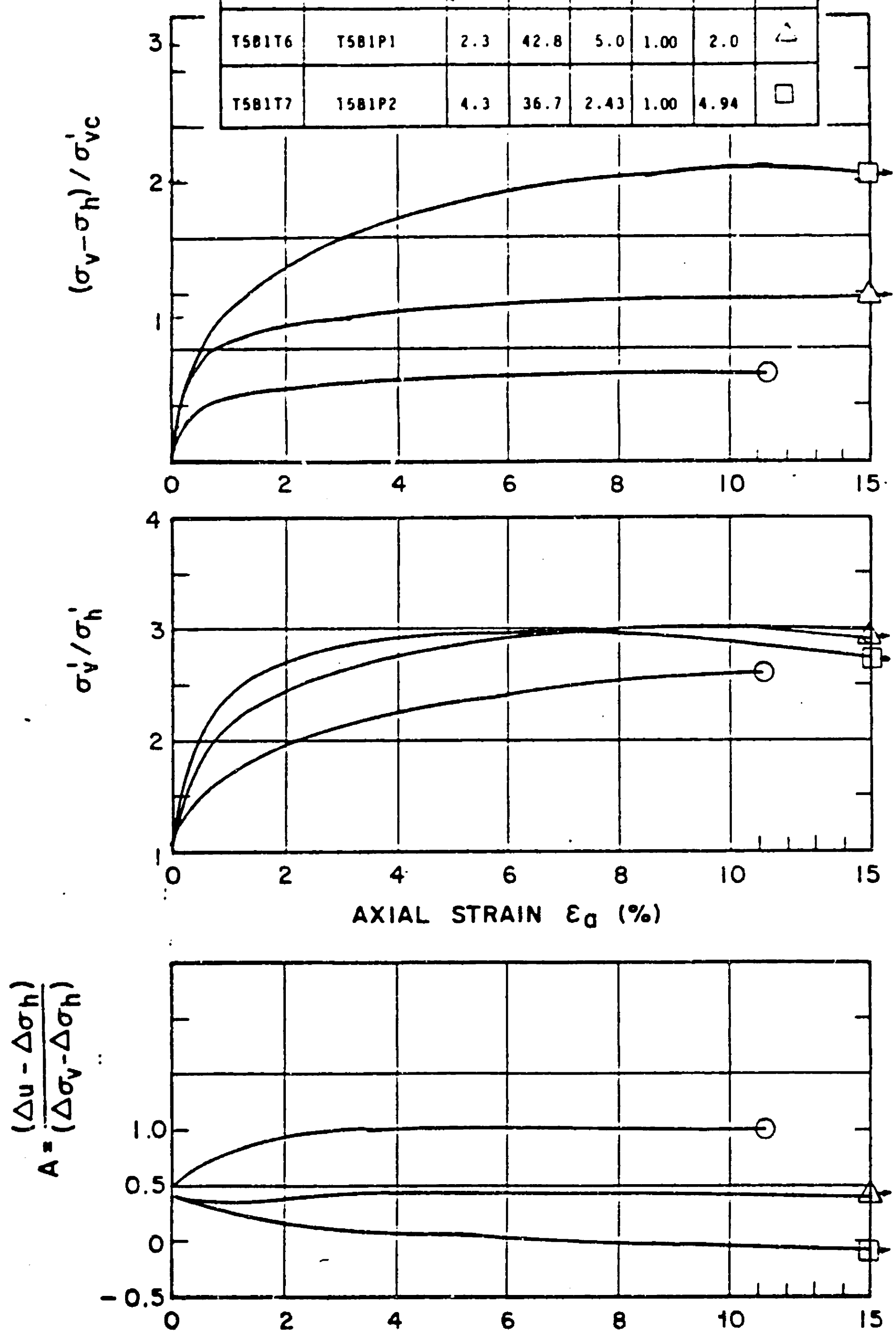


Figure 5-4: Normalized Shear Stress, Obliquity and A Parameter vs. Strain for CIUC Tests at OCR's = 1, 2 and 5

Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Symbol
T5B1T4	T5B1P1	1.5	45.0	10.0	1.00	1.0	○
T5B1T6	T5B1P1	2.3	42.8	5.0	1.00	2.0	△
T5B1T7	T5B1P2	4.3	36.7	2.43	1.00	4.94	□

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$q = 0.5(\sigma_v - \sigma_h)$

$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$

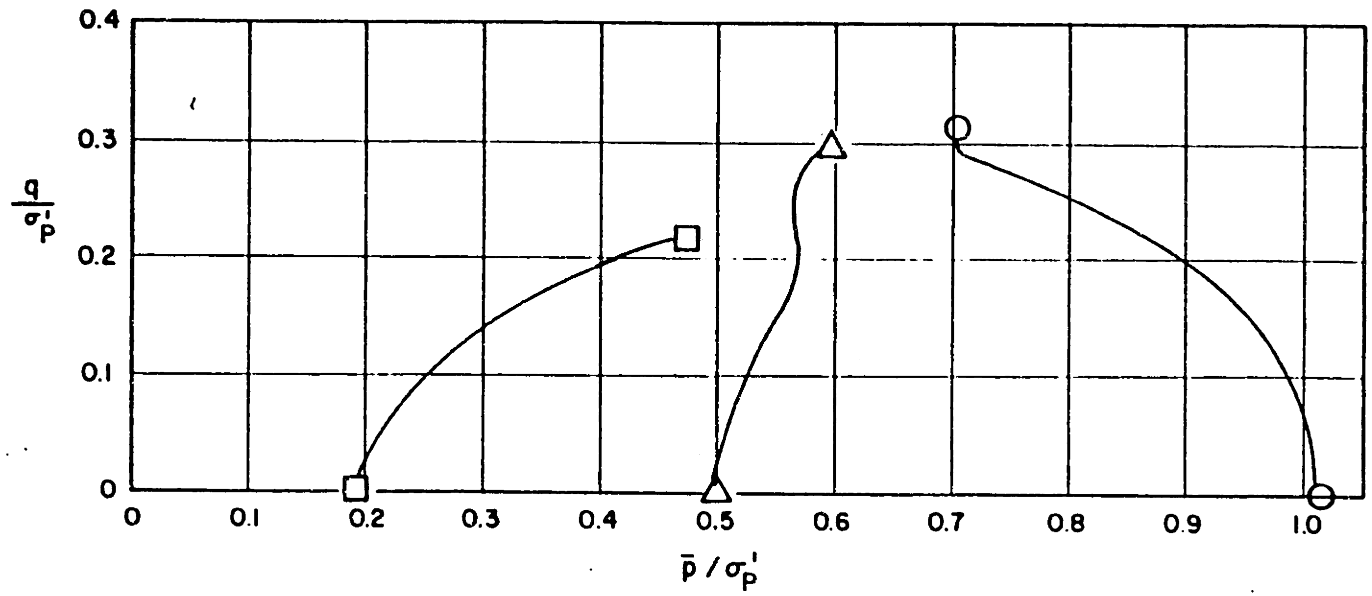
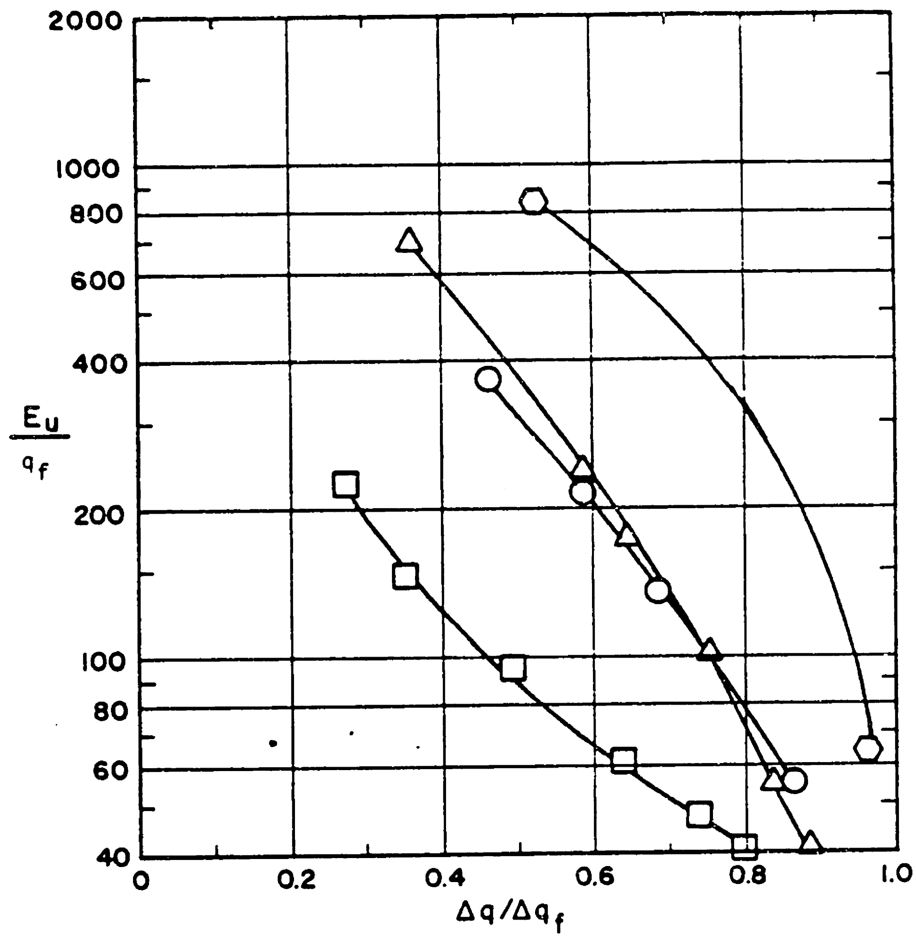


Figure 5-5: Normalized Effective Stress Paths for CIUC Tests at OCR's = 1, 2 and 5



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
T5B1T4	5B1P4	1.45	45.0	10.0	1.0	○
T5B1T6	5B1P1	2.3	42.9	5.0	2.0	△
T5B1T7	5B1P2	4.3	36.7	2.43	4.9	□
T5B1T5	5B1P1	1.1	43.9	6.55	1.0	⬡

Figure 5-6: Normalized Modulus for CIUC Tests at OCR's = 1, 2 and 5

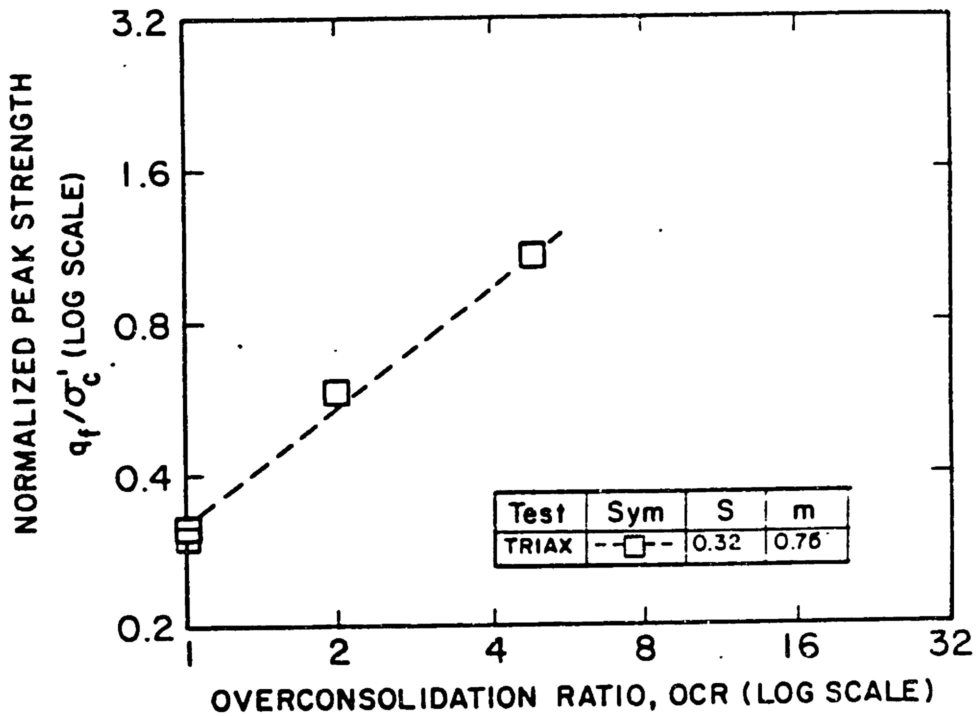
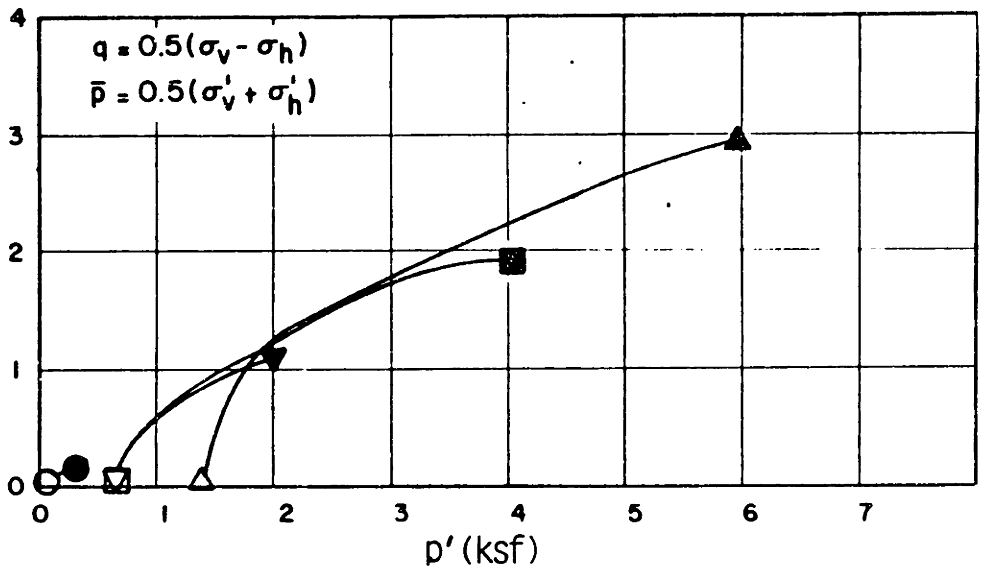


Figure 5-7: Normalized Strength vs OCR (log scales)

Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Symbol
T5B1T1	T5B1P1	1.85	40.3	0.11	0.96	13.7	○
T5B1T2	T5B1P3	8.24	33.8	0.69	1.01	7.8	▽
T5B1T3	T5B1S4	10.7	21.6	1.58	0.82	12.7	△
T5B1T8	T5B1S4	10.8	22.1	0.716	1.00	12.6	□



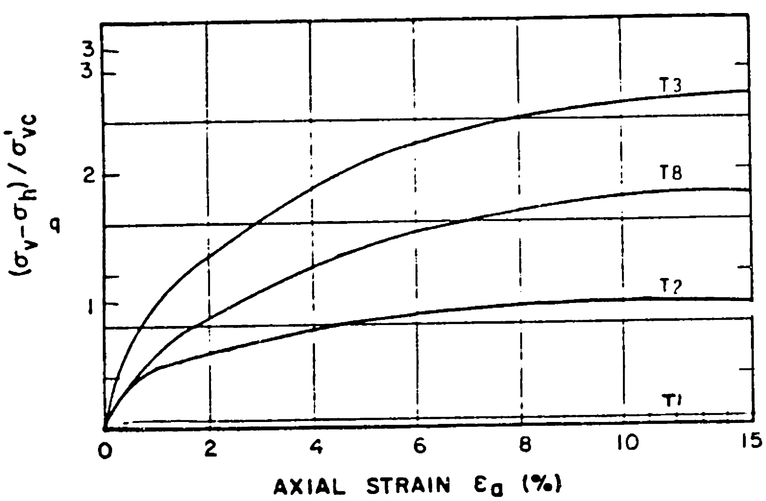


Figure 5-8: Stress-Strain and Effective Stress Paths for Recompression Tests

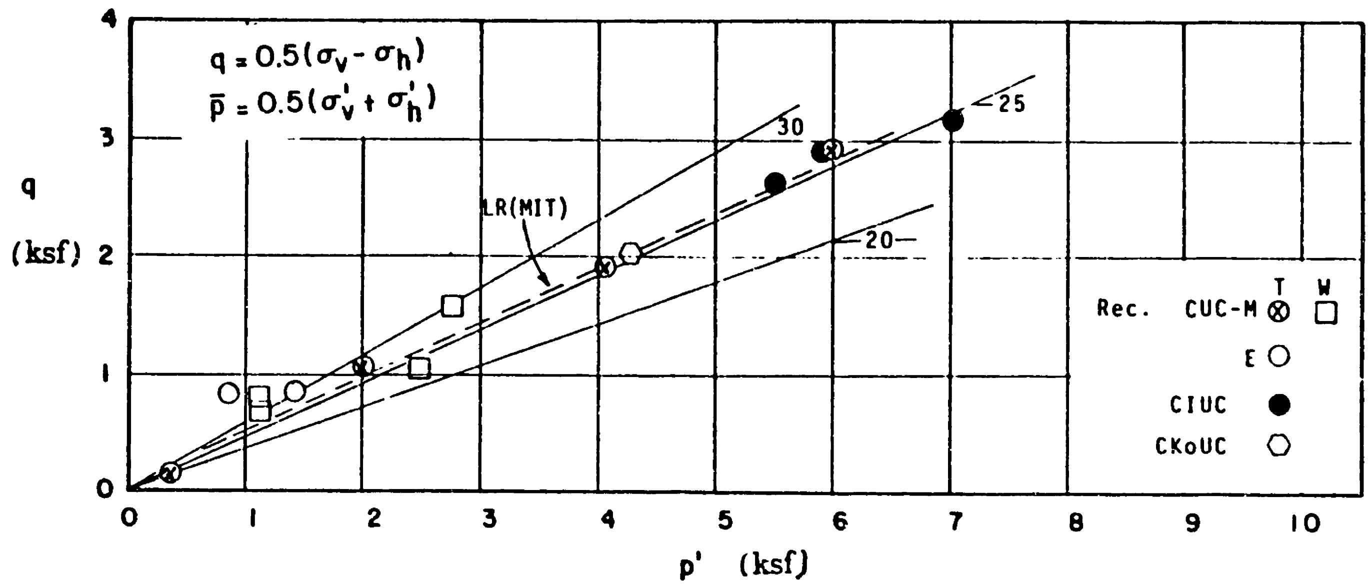


Figure 5-9: Peak Strength Values for Triaxial Compression Tests

Chapter 6

CK_0U DIRECT SIMPLE SHEAR TEST PROGRAM

6.1 SCOPE AND OBJECTIVES

Twenty CK_0UDSS tests were performed using the SHANSEP technique on samples from Sites T and W. This program of normally consolidated and overconsolidated tests had the following objectives: 1) to evaluate the normalized behavior of soil consolidated beyond the in situ preconsolidation pressure; 2) to characterize gouged versus nongouged deposits; and 3) to develop SHANSEP estimates of the initial in situ undrained stress-strain-strength properties at two Smith Bay locations. A description of the different types of SHANSEP CK_0UDSS tests follows.

1. Normally Consolidated Tests : Tests on samples reconsolidated to stresses greater than the preconsolidation pressure were performed on eight specimens from Site T and six specimens from Site W. The consolidation stress level was varied in order to assess its effect on undrained behavior. Comparison of results from Sites T and W is also used to assess the effects of ice gouging on normalized stress-strain-strength parameters.

2. Overconsolidated Tests Overconsolidated samples were reconsolidated to a vertical stress greater than the in situ preconsolidation pressure, then "mechanically" unloaded to various overconsolidation ratios, OCR. Tests with OCR = 7.9, 8.5 and 16 were performed on samples from Site T and tests at OCR = 5, 10 and 20 were performed on Site W samples.

3. Temperature Controlled Tests Two of the normally consolidated DSS tests (one each from Sites T and W) were run at a temperature of 1 °C in order to assess the effect of temperature on shear behavior.

The DSS program also included six tests wherein the specimens were reconsolidated to vertical stresses approximately equal to the in situ values, three each at Sites T and W. TETC performed five of these tests at a temperature of 1 °C; the other was run by MIT at room temperature. Tables 6-1 through 6-6 summarize the results obtained from the DSS test program at Smith Bay. Detailed tabulated data and curves for the MIT tests are presented in Appendix E.

6.2 MIT TEST EQUIPMENT AND PROCEDURES

The Geonor Direct Simple Shear apparatus used at MIT is described by Bjerrum and Landva (1966). The test procedures are essentially the same as those presented in Appendix B of Ladd and Edgers (1972), but using a cylindrical sample having an area of 35 sq cm and a height of 2 to 2.4 cm.

The locations within each tube selected for DSS testing were made after careful study of radiographs. Once extruded, each specimen was trimmed using a cutting shoe, then enclosed in a wire reinforced rubber membrane which is stiff in radial deformation but very flexible in simple shear. Five of the tests at Site W used a high capacity membrane which permitted the application of a vertical effective stress of 24.6 ksf (compared to 16 ksf for the standard membrane). Three of the tests on Site T samples used special top and bottom porous stones with pins to penetrate the soil. While the pins improve the shear stress transfer, they also make rigorous interpretation of the strain data impossible and may cause excessive disturbance of the top and bottom portions of the specimen. Pins are spaced 0.2 inches apart on the porous stones, with a height of 0.245 inches for the Site T tests. The pins were shortened to 0.15 inches for use on the Site W test at OCR = 20. Consolidation strains were calculated the same way as for tests without pins (vertical displacement divided by height of the specimen). However, shear strains for tests with pins were based on the height of the specimen minus the height of the pins.

The soil was K_0-consolidated incrementally to a vertical effective stress greater than the in situ preconsolidation pressure, and hence into the normally consolidated region, for tests run in accordance with the SHANSEP technique. The overconsolidated tests were then unloaded to varying OCR. The maximum consolidation stress (and the final vertical effective stress for overconsolidated samples) was applied for 24 hours to allow at least one cycle of secondary compression prior to undrained shear. A horizontal shear stress was applied at a shear strain of about 5% per hour (as per Ladd and Edgers, 1972; Yin, 1985) while varying the vertical effective stress (σ'_v) to maintain a constant height, hence constant volume. The automated height control used for tests on Arctic silt compensates for apparatus compressibilty. A correction for membrane resistance was also applied, with a change in shear stress of 0.044 ksf at 30% shear strain. The DSS is equipped with Direct Current Displacement Transducers (DCDT) to measure horizontal and vertical loads and displacements. A central data acquisition system records voltages; reduction of the data is carried out by a companion HP87.

Since the stress conditions in the DSS are unknown, the peak undrained strength is arbitrarily taken as equal to τ_{hmax}, which probably lies between $q_f = 0.5(\sigma_1-\sigma_3)_f$ and $\tau_{ff} = q\cos\phi'$ in typical DSS tests on clays (Ladd and Edgers, 1972). It is even more difficult to estimate the direction of σ_1 (i.e. the δ angle) corresponding to the τ_{hmax} condition.

Two of the normally consolidated tests were run at 1 °C in order to assess the effect of temperature. These were performed as for tests at room temperature, except that the water bath was cooled to 1 °C during consolidation and shear. Plexiglass plates were used to insulate the sample base from the metal DSS frame. For the test on Site T material, a 50% ethylene glycol - 50% distilled water mixture was pumped into the bath surrounding the sample and maintained at 1 °C. Due to concerns regarding osmotic pressures across the DSS membrane, the technique was modified for use on the Site W sample. For this test, the glycol-water mixture was pumped through a 0.64 cm diameter copper coil surrounding the sample in the water bath, and the bath filled with saltwater at the same concentration as seawater (30 g/l). The temperature during the test was monitored by a thermistor in the water bath; the temperature was usually 1 °C at the top of the bath (farthest from the uppermost coil), but ranged up to 3 °C.

6.3 NORMALLY CONSOLIDATED CK_0U DIRECT SIMPLE SHEAR RESULTS

Test results are summarized in Tables 6-1 and 6-2 for Sites T and W, respectively. The tables present the following information:

1. Classification data from Atterberg limit tests and grain size analysis.
2. The computed in situ effective overburden stress (σ'_{vo}) based on hydrostatic pore pressures and the estimated in situ preconsolidation pressure using the Casagrande technique;
3. The test preshear vertical consolidation stress (σ'_{vc}) and its corresponding overconsolidation ratio, OCR = 1.00;
4. The vertical strain (ϵ_v) at the maximum vertical consolidation stress;
5. Values at the peak horizontal shear stress (τ_{hmax}) of the shear strain (γ), normalized stresses (τ_h/σ'_{vc}, σ'_v/σ'_{vc}, τ_h/σ'_v) and the angle ψ(= arctan τ_h/σ'_v);
6. Similar data at the end of shearing (γ = ca. 30%);
7. Values of the secant undrained Young's modulus (E_u) normalized with respect to the DSS undrained shear strength (c_u) at 50% of the failure stress. Also, the normalized stress-strain parameters (G_i/σ'_{vc}) and R_f based on a hyperbolic stress-strain relationship.

8. Remarks pertaining to the test quality or special test conditions.

6.3.1 Temperature Controlled Tests

Two tests were performed at 1 °C to assess the effect of temperature on the measured shear behavior of Arctic Silt. Figures 6-1 through 6-4 compare 1 °C versus room temperature tests for Site T gouged material and Site W intact material.

Both Site T samples were taken from the same tube (within 0.4 ft of each other) and have only slight differences in water content. The room temperature curve had higher strains and appears more disturbed than the 1 °C curve (Figure 6-1). Stress-strain curves and stress paths agree very well, with the 1 °C test having a slightly higher strength (0.288 versus 0.279) at a higher shear strain (15% versus 11.8%). This may be the result of differences in the consolidation behavior of the two samples, where the 1 °C sample was slightly stiffer. Normalized modulus data were virtually the same.

At Site W, tests were run at 1 °C and 20 °C on two samples 0.2 ft apart (tests WDSS4 and WDSS6). Water contents are very close (28.9% and 30%) and the compression curves are almost identical. The peak shear strengths were 0.248 at 20 °C and 0.242 at 1 °C reached at about the same shear strain and following very similar effective stress paths. Modulus data are again almost identical.

For both sites the difference in measured shear strength was only $\pm$ 2-3%. Therefore, correction of room temperature results to the in situ temperature is not necessary and the tests at 1 °C are not differentiated from room temperature tests in subsequent discussions.

6.3.2 Site T Results

The eight CK_0UDSS tests performed at Site T had Relative Elevations, RE, less than 8.5 ft. Therefore, the results presented in Figures 6-5 through 6-8 are characteristic of ice gouged material only. Excluding the low plasticity TDSS1B and the high water content TDSS3, the test specimens had quite similar index properties as follows: water content, w = 39.8% $\pm$ 3.9 SD; I_p = 23.5% $\pm$ 2.1; and clay content = 41.6% $\pm$ 1.6 SD.

Five of the samples were consolidated to 5 ksf. Four tests (TDSS 4,6,7 and 11) show good agreement amongst the consolidation curves. Test TDSS1B had unusually low strains; this test is probably not representative as it contained a sand layer in the middle of the sample. In addition, this test was consolidated into the normally consolidated range following a Recompression - type shear test at 0.41 ksf. Test TDSS3 had high strains corresponding to its unusully high water content. The compression curves (Figure 6-5) have poorly defined Compression Ratios, CR, less than those measured in oedometer tests. Estimates of preconsolidation pressure from these curves were also consistently less than estimates made from oedometer tests.

Two additional tests were consolidated to 16 ksf. Test TDSS9 was run on material adjacent to test TDSS6 with similar water content and the consolidation curves show good agreement. However, compressibilty continued to increase with increasing consolidation stress and TDSS9 gave a better defined virgin compression line with a higher (and more representative) value of preconsolidation pressure. Test TDSS10 had low strains compared to other samples tested; it was also the deepest sample and was located adjacent to oedometer TP3, which had a water content of 28%. Hence, test TDSS10 appears to be near the bottom of the highly gouged zone.

Comparison of the DSS compression curves with compressibility data in Chapter 4 shows that several of the curves did not reach the steepest portion of the virgin compression line. These tests therefore can not give reliable values of preconsolidation pressure and may not be representative of truly normally consolidated behavior.

The average peak undrained strength ratio (c_u/σ'_{vc}) (where c_u is defined as the maximum value of the horizontal shear stress) equals 0.279 ± 0.007 SD for the six tests consolidated to 5 ksf and 0.237 ± 0.01 SD for the two tests consolidated to 16 ksf. Consideration of all test yields a strength of 0.268 ± 0.020 SD.

The stress paths in Figure 6-6 for the six tests consolidated to 5 ksf show extremely good agreement, with peak points located at σ'_v/σ'_{vc} of 0.587 ± 0.021. Test TDSS1B is at the low end, possibly because it was a second shear. The stress paths have a "peaked" shape, with fairly steep slopes on either side of the peak strength. The two tests consolidated to 16 ksf agree with each other. They peak at about the same σ'_v/σ'_{vc} as tests consolidated to 5 ksf, but have a significantly lower strength. The stress paths also tend to have flatter slopes near the peak strength.

Normalized shear stress versus shear strain plots in Figure 6-7 fall into two groups of high and low strength. There is good agreement for tests consolidated to 5 ksf, with peak strengths occurring over a range of 12 to 18% shear strain. It should be noted that tests TDSS1B and TDSS7 used stones with pins, so the higher strains at failure may not be accurate. Those strains were calculated based on a net height equal to the height of the sample minus the height of the pins. The two tests consolidated to 16 ksf agree with each other, and have shear strains at peak in the same range as tests consolidated to 5 ksf. Considering all tests without pins, the shear strain at peak = $13.1\% \pm 1.2$ SD.

Pore pressures are the same for all tests up to peak. Higher pore pressures after peak are related to increased strain softening. All tests exhibit significant strain softening except those with pins, perhaps because the strains were adjusted for the height of the pins. At large strains (γ = 30%), the undrained resistance reduces to 0.156$\pm$0.040 SD (all tests without pins).

The ψ angle at peak strength (=arctan τ_h/σ'_v) equals 24.6°$\pm$1.9 SD for all tests. At shear strain equal to 30% it increases to 28.3°$\pm$2.8 SD (for tests without pins).

Normalized modulus values are presented in Figure 6-8. Tests TDSS1B and TDSS7 used stones with pins and the modulus values based on corrected strains plot somewhat below the other results. The remaining tests fit together very well, with no distinction between tests consolidated to 5 ksf and 16 ksf. $E_u(50)/c_u$ ranges from 250 to 322, with an average value of 280$\pm$34 SD for tests without pins; the average for all tests is 255$\pm$53 SD. The hyperbolic parameters were also calculated for Site T results, in an effort to represent undrained deformation behavior. Normalized initial shear modulus (G_i/σ'_{vc}) and R_f parameters were obtained from CK_0UDSS results using the following relationship:

$$\frac{\tau_h}{\sigma'_{vc}} = \frac{\gamma}{(a + b\gamma)}$$

where $$a = \frac{1}{G_i/\sigma'_{vc}}$$

$$b = \frac{R_f}{c_u/\sigma'_{vc}}$$

By plotting $\gamma/(\tau_h/\sigma'_{vc})$ versus γ , the intercept of the line is a, and the slope is b. Average values of G_i/σ'_{vc} and R_f for Site T are 40.7$\pm$7.4 SD and 0.95$\pm$0.01 SD, respectively.

The test program included samples having variable σ'_{vc}/σ'_p (in situ), as shown in Table 6-4. Tests consolidated to 5 ksf gave consistent strengths of 0.279$\pm$0.007 SD for σ'_{vc}/σ'_p ranging from 1.3 to 2.9. However, tests consolidated to 16 ksf with σ'_{vc}/σ'_p of 4.7 to 6.5 had much lower shear strengths of 0.237$\pm$0.01 SD, showing dependence of strength on the stress level. Hence these samples of Arctic silt do not exhibit normalized behavior. The average strength for all tests is 0.268$\pm$0.020 SD. Although the coefficient of variation (COV) of 0.02/0.268 = 7.5% is quite small, 0.237/0.279 = 0.85 is a significant change in the measured strength associated with changing consolidation stress. Section 6.3.4 will further investigate the possible reasons for differences in strength in terms of σ'_{vc}/σ'_p and other parameters after presenting the results for Site W.

6.3.3 Site W Results

A total of six normally consolidated DSS tests were performed on Site W material. As shown in Table 6-2, one test was run on Holocene material at a shallow depth (RE = 3.4 ft), with a water content = 41.7% and 34% clay content; another at RE = 7.0 ft had a water content of 36.6% and 57% clay. The remaining four tests had water contents of 29.4%$\pm$0.5 SD. Samples were consolidated to 9.0, 16.4 or 24.6 ksf, the latter stress obtained by using a special high capacity membrane for the DSS. Results are shown in Figures 6-9 through 6-12.

Compression curves (Figure 6-9) show good agreement among the four low water content samples. The two higher water content samples had higher strains at the same consolidation stress, especially WDSS2 on the Holocene clay. Due to the rounded shape of the curves and the low maximum consolidation stress applied, estimates of preconsolidation pressure are difficult to make. The samples consolidated to 24.6 ksf have a better defined virgin compression line and estimates of preconsolidation pressures from two of these tests also show good agreement with oedometer values at the same depth (see Figure 4-11).

The peak undrained strength ratio c_u/σ'_{vc} equals 0.242$\pm$0.005 SD excluding tests WDSS1 and WDSS5. The tests included in this average were two high water content specimens consolidated to 9 or 16 ksf and two low water content (30%) specimens consolidated to 24.6 ksf. Figure 6-10 shows the stress paths for all tests; four tests agree very well. The two tests with water contents, w = 30% and σ'_{vc} = 16.4 ksf (WDSS1 and WDSS5) have much higher strengths (0.30 and 0.275, respectively) and peak at a higher value of σ'_v/σ'_{vc} (0.7-0.73 compared to 0.6-0.65 for the others). These two curves also have a pronounced peaked shape, which is less apparent for tests giving lower strengths. The peak strength for all tests = 0.257$\pm$0.025 SD. Normalized τ_h/σ'_{vc} versus shear strain curves show good agreement for four tests. The peak strength is reached at = 10.9%$\pm$1.9 SD (excluding tests WDSS1 and WDSS5). Tests WDSS1 and WDSS5 reach their higher strengths at lower strains (8.8%, 9.6%) than the average. The change in pore pressure is approximately the same for all tests. Tests having the highest peak strength show the greatest decrease in strength. At large strains (γ = 30%), the undrained resistance reduces to 0.132 $\pm$0.019 SD (all tests). The ψ angle at peak strength (= arctan τ_h/σ'_v) equals 21.3°$\pm$0.7 SD for all tests. At a shear strain of 30%, ψ increases to 23.6°$\pm$1.5 SD.

The normalized secant undrained Young's modulus at 50% peak strength, $E_u(50)/c_u$, ranges from 172 to 343, with an average value of 242$\pm$76 SD. The results normalize very well, with no distinction between tests consolidated to different consolidation stresses. Hyperbolic parameters were calculated as for Site T (Section6.3.2). Site W results gave G_i/σ'_{vc} = 33.9$\pm$8.4 SD and R_f = 0.94$\pm$0.01 SD.

Values of σ'_{vc}/σ'_p ranged from 1.6 to 3.7 for normally consolidated tests at Site W (excluding WDSS2). Tests giving high strengths were low water content samples consolidated to σ'_{vc}/σ'_p = 1.6 and 2.0. The remaining tests gave very consistent normalized strengths, with σ'_{vc}/σ'_p equal to 2 to 3.7 and consolidation stresses of 16 or 24.6 ksf. Comparable strength was also obtained for the Holocene sample, with σ'_{vc}/σ'_p = 22.7 and σ'_{vc} = 9 ksf.

6.3.4 Discussion of Normally Consolidated Results

Normally consolidated material at Sites T and W develops positive excess pore pressures (i.e. decrease in vertical effective stress) throughout shear. In other words, the material is always "contractive" rather than developing "dilatant" behavior with continued shearing, as observed in many prior CU triaxial compression tests on Arctic silts (Wang et al., 1982). Stress paths for samples tested at "low" consolidation stresses have a peaked shape; when consolidated to "high" consolidation stresses the curves are smoother and more typical of those for soft sedimentary clays (Ladd and Edgers, 1972).

Results of the DSS test program for Sites T and W are summarized in Table 6-3. The normalized shear strength at Site T varied with consolidation stress. At Site W, the low water content samples showed dependence on stress level. Other specimens with higher water contents and low water content samples consolidated to high stresses gave very similar results. The lowest measured strengths at Sites T and W are very close. The strain to peak is high for both sites (13.1% for T, 10.9% for W). Strain softening was observed at both sites, but was more severe at Site W. The angle ψ is higher at Site T than at W; modulus values $E_u(50)/c_u$ and hyperbolic parameters are also somewhat higher at site T.

This section will now examine the normally consolidated results in order to evaluate possible reasons why constant undrained strength ratios were not obtained. The relationship of strength to σ'_{vc}/σ'_p vertical strain and compression ratio, CR, will be presented.

Consolidation data from DSS tests are summarized in Table 6-4. Figure 6-13 plots the undrained strength ratio, S ($= c_u/\sigma'_{vc}$) versus the stress ratio, σ'_{vc}/σ'_p , for both sites T and W. Preconsolidation pressures are based on the best available estimate from adjacent oedometer tests or from DSS curves, if considered reliable. The plot shows a general trend of decreasing strength with increasing σ'_{vc}/σ'_p considering all data.

Site T results are generally consistent, showing a modest decrease in S for σ'_{vc}/σ'_p up to three, then a more rapid decrease at higher stress ratios. It is not clear whether even larger values of the stress ratio would lead to lower values of the undrained strength ratio. The plot suggests this, but uncertainty in the determination of the σ'_{vc}/σ'_p ratio for tests TDSS9 and TDSS10 precludes a definite conclusion. In any case, Site T samples do not exhibit normalized behavior at σ'_{vc}/σ'_p greater than 1.5 to 2, as recommended by Ladd and Foott (1974) for SHANSEP testing programs.

Four Site W samples with water contents of about 30% show a consistent and dramatic decrease in the undrained strength ratio as σ'_{vc}/σ'_p increases from 1.6 to greater than 3. Given the above and the two tests at higher water contents, a value of S = 0.24 is reasonable for truly normally consolidated Site W material.

As previously noted, Site T DSS tests were run on soil that had been moderately to highly remolded by ice gouging. One would expect remolded material to have a higher undrained strength ratio than "undisturbed" soil since it should exist in situ at a lower I_L versus log σ'_p relationship. Such remolding may also explain the substantial decrease in the undrained strength ratio at high values of σ'_{vc}/σ'_p compared to more usual sedimentary clays.

Figure 6-14 shows the undrained strength ratio versus ϵ_v, the vertical strain under the maximum consolidation stress. A consistent trend of decreasing strength with increasing strain is evident for Site W results. A constant S ($=c_u/\sigma'_{vc}$) for vertical strain greater than 12% is found when the two samples with higher water content are included. The Site T values have a more scattered trend. One would expect to see a strength decrease as the vertical strain increases until reaching the virgin compression line; then, at higher strains, the normalized strength should be unaffected.

Figure 6-15 shows c_u/σ'_{vc} versus Compression Ratio, CR, for all NC CK_0UDSS tests. It was expected that if the consolidation stress is not on the virgin compression line, then the values of CR (last increment) would be too low, resulting in a higher S. Values of CR_{DSS} are compared to those from associated oedometers on the same sample. The oedometer values mostly fall between 0.1-0.15. A consistent pattern of increasing strength with lower CR is apparent for Site W results, although the strength is very sensitive to slight changes in CR. DSS tests with CR_{DSS} equal to CR_{OED} should lie on the virgin compression line and hence give consistent estimates of the undrained strength ratio. The two higher strength Site W tests have CR much less than CR_{OED}. The Site T data show little trend; tests consolidated to 5 ksf had values of CR close to those measured in the oedometer, yet still gave high strengths. Possibly ice gouging has caused sufficient remolding that CR is not a reliable indicator of whether or not the test specimen is on the virgin compression line.

Figure 6-16 plots NC strength versus (w_N + ϵ_v). Site W shows a very consistent trend, leading to S = 0.24 at the larger values of this empirical parameter. Although Site T is not consistent, the plot does show a distinct difference between T and W except for the two Site T tests consolidated to high consolidation stresses.

A plot of the peak c_u/σ'_{vc} versus ψ angle (= arctan τ_h/σ'_v) (Figure 6-17) compares Smith Bay results to an empirical correlation for 12 clays of varying plasticity and sensitivity, including data for the Harrison Bay Arctic silts. The Smith Bay data plot in three groups. One cluster agrees with the empirical correlation for soils having S = 0.24. This group includes all DSS tests consolidated to "high" consolidation stresses A second group comprises samples from Site T consolidated to 5 ksf, that plot around the empirical correlation at S = 0.28. It is interesting to see the Site T material move down the line as consolidation stress increases. Two additional points consist of the two Site W tests with low water content (30%) consolidated to 16 ksf, which lie well below the linear regression line. This is surprising since the two tests from Harrison Bay having S = 0.30 fall on the linear regression line.

The peak points of the NC effective stress paths (τ_h/σ'_{vc} versus σ'_v/σ'_{vc}), and also those at maximum obliquity, are plotted in Figures 6-18 and 6-19 for sites T and W, respectively. The Site T peaks plot around ψ = 25° , with ψ increasing at 30% strain. The Site W ψ_{peak} values lie between 20° and 25°, increasing with strain. The Site T samples fail at a significantly lower stress than those from Site W, σ'_v/σ'_{vc} = 0.59 $\pm$ 0.02 SD versus 0.63$\pm$ 0.025 SD (excluding WDSS1 and WDSS5).

$E_u(50)/c_u$ versus G_i/σ'_{vc} is plotted in Figure 6-20. The results show a fairly consistent trend of increasing initial shear modulus with an increase in $E_u(50)/c_u$. Using the hyperbolic relationship:

$$\frac{E_u(50)}{c_u} = \frac{G_i/\sigma'_{vc} \times 3(1 - 0.5R_f)}{(c_u/\sigma'_{vc}) = S}$$

Lines were drawn for values of S = 0.24 and 0.28 for R_f = 0.945 (mean for all tests). Note that the two Site T tests with pins fall below the other data.

6.4 OVERCONSOLIDATED DIRECT SIMPLE SHEAR RESULTS

Table 6-5 summarizes results for tests on overconsolidated (OC) Arctic Silt; it provides the same information as stated in Section 6.3. Six tests (3 Site T, 3 Site W) were performed on samples which were first consolidated to a stress greater than the in situ preconsolidation pressure and then rebounded to various overconsolidation ratios. In all cases, samples were taken adjacent to material used for normally consolidated testing.

Ladd et al. (1977) presents a relationship that assumes the overconsolidated strength is related to the normally consolidated strength and OCR by:

$$c_u/\sigma'_{vc} = S\,(OCR)^m$$

where $S = c_u/\sigma'_{vc}$ at OCR = 1. For each site, this relationship was used to calculate a value of m based on the results of the OCR = 1 and the overconsolidated tests.

6.4.1 Site T Results

Results of the three overconsolidated tests at Site T are presented in Figures 6-21 through 6-23. Compression curves are presented in Appendix E. Figure 6-21 presents the normalized stress paths from the three tests on overconsolidated material compared to results for the average of all normally consolidated tests consolidated to 5 ksf. If the normally consolidated effective stress path represents the State Boundary Surface for the DSS mode of shearing, then all overconsolidated tests would be expected to hit this boundary and follow it with continued straining. The stress paths indeed fit together very well.

Normalized shear stress versus shear strain plots are presented in Figure 6-22. The plot shows a consistent increase in normalized strength and decrease in $\Delta u/\sigma'_{vc}$ with increasing OCR. The increased shear strain at peak for the OCR = 16 test is probably an artifact of the correction for pins, since the NC tests with pins had a shear strain at peak greater than that for the test without pins. The stress-strain curves generally are fairly flat, with a peak strength occurring over a range of strain values. If the degree of strain softening is defined as $\tau(\gamma = 30\%)/\tau_{peak}$, strain softening decreases with increasing OCR. The correction to γ due to pins accounts for the lack of strain softening observed for the OCR = 16 test. The normalized modulus versus stress ratio is plotted in Figure 6-23 with the range for normally consolidated tests. The data are consistent with modulus decreasing with increasing OCR.

Figure 6-24 plots the log of normalized strengths from CK_0UDSS tests versus log OCR. A backcalculated m value of 0.73 was obtained for S = 0.28 by performing linear regression on the average value of normally consolidated tests and the three overconsolidated tests with maximum consolidation stress of 5 ksf (r^2 = 0.986).

6.4.2 Site W Results

Compression curves for overconsolidated tests on Site W material are presented in Appendix E. Vertical strains at the maximum consolidation stress varied by about 3% for the three tests. The normalized stress paths (Figure 6-25) compare OC data to results for an average NC test of the same water content consolidated to 24.6 ksf. The overconsolidated tests fit in very well with an extrapolation of the NC curve. Normalized shear stress versus shear strain plots are presented in Figure 6-26. The plots show increase in the normalized strength with increasing OCR, at about constant strain to peak. Less strain softening occurred at higher OCR except at OCR = 10, with slightly higher strain softening. Low strain softening at OCR = 20 is probably due to the correction of γ due to the use of stones with pins. Normalized modulus versus stress ratio is plotted in Figure 6-27 with the range for NC tests; modulus decreases with increased OCR.

Figure 6-28 plots normalized strengths from CK_0UDSS tests versus OCR. A backcalculated m value of 0.71 was obtained for S = 0.24 by performing linear regression on all values from tests with a maximum consolidation stress of 24.6 ksf (r^2 = 0.990). This m value is in the lower part of the range of 0.8 ± 0.1 found for typical clays, possibly because of the higher C_s/C_c ratio of this deposit. Estimates of m are further discussed in Section 6.4.3.

6.4.3 Discussion of Overconsolidated CK_0UDSS Results

Tests on Smith Bay samples at both sites yielded progressively higher strengths at increasing OCR, accompanied by a decrease in strain softening. Tests without pins had about the same values of γ_{peak} and decreased modulus with increasing OCR. The results of tests on overconsolidated material were used to calculate the m values needed to estimate SHANSEP strength profiles. Both sites gave low m values: m = 0.73 at Site T and 0.71 at Site W.

Critical State Soil Mechanics (i.e. the Modified Cam-Clay) predicts:
$m = 1 - C_s/C_c$, where C_s and C_c refer to the slopes of the swelling and virgin compression lines, respectively. Figure 6-29 plots measured m values versus $(1-C_s/C_c)$. The measured m values were calculated for each of the six

overconsolidated tests run at Smith Bay using S from an adjacent NC test in the relationship:

$$\tau_h/\sigma'_{vc} = S\,(OCR)^m$$

Analysis of triaxial compression and DSS data for five clays (Ladd, 1986) gave the following linear regression line shown in Figure 6-29 (r = 0.98):

$$m_{measured} = 1.15(1 - C_s/C_c) - 0.26$$

While this relationship shows that the MCC overpredicts m, the m parameter does increase in a consistent fashion as C_s/C_c decreases. All points from the Smith Bay tests lie above this line, and hence closer to the "Critical State" estimate. Four of the DSS tests give a very consistent trend; two tests are outliers (TDSS2 at OCR = 8 and WDSS9 at OCR = 20) for reasons that are not clear.

The plot also contains two points (solid symbols) based on oedometer results. An average value of C_s/C_c was obtained from RR/CR (to OCR = 8) data for the oedometers and plotted versus the "measured" m calculated from linear regression of the DSS results. These values plot near most of the data calculated from individual tests.

The measured m values are in the low end of the range of 0.8 ± 0.1 obtained for typical DSS testing (Ladd et al., 1977). C_s/C_c for Smith Bay soils seems to be higher than that for "typical" clays. The dashed line on the plot represents linear regression through all Smith Bay points; it is nearly parallel to the other linear regression line.

In conclusion, the m values of 0.73 and 0.71 selected for use at Site T and Site W, respectively, appear to be reasonable. There are problems for both sites due to the dependence of S on the consolidation stress. Calculations have used S values associated with consolidation stresses equal to the maximum consolidation stress for the OC tests: 0.28 for Site T and 0.24 for Site W.

6.5 RECOMPRESSION TESTS

TETC performed two Recompression tests at Site T and three at Site W. All tests were run at 1°C, using a Geotest simple shear device. Details of TETC test procedures are included in Appendix B. MIT performed one additional Recompression test at Site T. All results are summarized in Table 6-6.

The compression curve and normalized modulus results for the MIT Recompression test are included in Appendix E. Figures 6-30 and 6-31 present the stress path and stress-strain curve, respectively. This test used stones with pins, so shear strains and modulus values are corrected for the height of the pins. The sample had low strains during compression, possibly due to the presence of a sand layer. The shear stress-shear strain curve has a peaked shape at peak strength and the stress path fits well with the data in Figure 6-21.

TETC obtained the maximum shear stress for Recompression tests at Site T at a shear strain of 20%. However, the stress-strain curves show no pronounced peak, in contrast to the MIT test. Three TETC tests were performed at Site W. The stress strain curves rise quickly then are relatively flat. Tests on two samples (W3B-P1 and W5B-P3) do not look like they have reached a distinct peak. No Recompression tests run at Site W by MIT are available for comparison.

Recompression test results will be compared with strengths estimated using the SHANSEP technique in Chapter 7.

6.6 DISCUSSION OF DIRECT SIMPLE SHEAR RESULTS

DSS strength parameters for 12 OCR = 1 tests on Harrison Bay Arctic silts are summarized below:

γ_{peak}	$= 9.6\% \pm 2.1$ SD
τ_h/σ'_{vc} at peak	$= 0.24 \pm 0.01$ SD
τ_h/σ'_{vc} ($\gamma = 30\%$)	$= 0.154 \pm 0.01$ SD
ψ_{peak}	$= 22.6^o \pm 1.5$ SD
$\psi_{(\gamma = 30\%)}$	$= 29.8^o \pm 5.0$ SD
$E_u(50)/c_u$	$= 385 \pm 100$ SD
G_i/σ'_{vc}	$= 47.7 \pm 13.1$ SD
R_f	$= 0.949 \pm 0.016$ SD

"Low" strengths at Smith Bay are comparable to the strength measured at Harrison Bay (excluding the surface material with w_N of about 35%), with differences in the strain at peak of only 1-3%. This is interesting, considering that the results are for very different materials: ML-MH silts at Harrison Bay versus CL clays at Smith Bay. Strain softening was observed for both Smith Bay and Harrison Bay results, with a value of τ_h/σ'_{vc} at a shear strain of 30% from 50-64% of the peak strength. The mean values of undrained modulus at Smith Bay were in the lower end of Yin's (1985) range. The initial shear modulus at Smith Bay was also in the lower end of the range of 47.4 ± 13.1 SD found at Harrison Bay.

All Harrison Bay DSS test results plotted along the linear regression line in Figure 6-17, the plot of ψ angle versus τ_h/σ'_{vc}. Even tests with strengths equal to 0.30 plotted on this line , whereas Site W tests with strengths of 0.275 and 0.30 lie well below the linear regression line

Figure 6-32 presents the peak values of normally consolidated effective stress paths for Harrison Bay CK_0UDSS tests. These data show significant scatter in the peak σ'_v/σ'_{vc} for tests having c_u/σ'_{vc} = 0.24 $\pm$ 0.01 SD, whereas the Smith Bay results in Figures 6-18 and 6-19 encompass a smaller σ'_v/σ'_{vc} range considering Sites T and W separately. However, the two Site W tests having a high c_u/σ'_{vc} also have a high σ'_v/σ'_{vc}, in contrast to the two Mukluk Island surface samples with strength ratios of 0.30. These two Site W tests are not "truly" normally consolidated, as previously noted when discussing Figure 6-17.

Again, soil conditions at Harrison Bay and Smith Bay are very different. Harrison Bay deposits consist of silty Holocene material (ML-MH) located in the Soft Zone Area. Results at that location gave consistent values of normally consolidated τ_h/σ'_{vc} (S), unlike results at Smith Bay (particularly Site T) where the S value is dependent on the consolidation stress. This difference could be the result of differences in geologic history. Material tested at Site T is part of a highly gouged zone and subject to intense reworking by ice. Site W gave more consistent results, with high measured strengths clearly the results of insufficient consolidation stress, whereas Site T material was consolidated to σ'_{vc}/σ'_p greater than four before "low" strengths comparable to those at other sites were measured.

OCR = 1 undrained strength ratios from Harrison Bay and Smith Bay DSS tests are plotted versus the Cumulative Index Parameter (CIP) in Figure 6-33. CIP is defined as the sum of the natural water content (w_N), plasticity index (I_P) and clay fraction. The Smith Bay DSS strengths plot higher than those for Harrison Bay. This CIP plot, although able to describe the range of behavior for Harrison Bay, does not provide a useful framework for the Smith Bay sites. Values of strength measured at Smith Bay Sites extend over a wide range independent of the CIP.

Linear regression of normally consolidated and overconsolidated DSS tests on soil at Harrison Bay yielded an m value = 0.77, compared to 0.73 at Site T and 0.71 at Site W. Data from Yin (1985) shows RR/CR = 0.095 $\pm$ 0.025 for the upper layer (best defined), giving a $(1 - C_s/C_c)$ of 0.915 $\pm$ 0.025 SD. This is much higher than the $(1 - C_s/C_c)$ of 0.81 for Site T and 0.76 for Site W (using RR/CR from oedometer tests). The measured m of 0.77 at Harrison Bay agrees well with the value of 0.79 predicted by linear regression for five clays (Ladd, 1986) shown in Figure 6-29.

Table 6-1: SUMMARY OF NORMALLY CONSOLIDATED CK_oUDSS TESTS - SITE T

All Stresses in ksf

Test RE (ft.)	Boring	I_P / I_L; w_N (%); w_L / -2μ	IN SITU σ_{vo}' / σ_p'	TEST OCR / σ_{vc}'	max ϵ_v (%)	AT τ_h MAX γ (%)	AT τ_h MAX $\frac{\tau_h}{\sigma_{vc}'}$	AT τ_h MAX $\frac{\sigma_v'}{\sigma_{vc}'}$	AT τ_h MAX τ_h/σ'_v	AT τ_h MAX $\phi°$	AT γ = 30% γ (%)	AT γ = 30% $\frac{\tau_h}{\sigma_{vc}'}$	AT γ = 30% τ_h/σ_v' / $\sigma_v/_{vc}'$	AT γ = 30% $\phi°$	E_{50}/c_u / G_i/σ'_{vc} / R_f	Remarks
TDSS1B 7.5	3B	12.1 / 118.2; 37.5; 35.3 / 33	0.385 1.7(a)	1.00 4.71	7.7	15.8*	0.275	0.549	0.501	26.6	30.1*	0.242	0.571 0.424	29.7	180* 48.5 0.954	- sand layer - stones w/ pins - 2nd shear
TDSS3 7.9	3B	33.1 / 65.6; 54.4; 65.8 / 49	0.407 1.7(a)	1.00 5.04	15.6	13.2	0.270	0.588	0.459	24.7	29.9	0.191	0.522 0.366	27.6	320 50.5 0.962	
TDSS4 4.7	3B	24.0 / 37.9; 35.3; 50.2 / 41.5	0.231 3.04(b)	1.00 4.92	11.7	13.8	0.284	0.595	0.477	25.5	30.5	0.194	0.533 0.364	28.1	280 42.8 0.955	- "disturbed" (extrusion)
TDSS6 7.0	1B	25.1 / 55.3; 41.5; 52.7 / 43	0.220 3.9(c)	1.00 4.94	12.0	11.8	0.279	0.598	0.467	25.0	30.8	0.179	0.554 0.323	29.0	250 38.9 0.947	- 1-2 mm sandy layer near base of sample
TDSS7 7.0	5B1	20.5 / 46.8; 36.4; 47.3 / 40	0.385 3.5(d)	1.00 4.94	13.7	18.4*	0.276	0.611	0.452	24.3	29.9*	0.258	0.498 0.518	26.5	186* 35.7 0.951	- stones w/ pins
TDSS9 7.2	1B	25.1 / 72.9; 45.9; 52.7 / 43	0.231 3.9(c)	1.00 15.56	22.6	12.8	0.230	0.605	0.380	20.8	31.1	0.094	0.456 0.206	24.5	256.6 34.2 0.95	
TDSS10 8.4	5B1	21.1 / 62.6; 38.4; 46.3 / 39	0.462 3.5(d) 6.15(e)	1.00 15.56	15.5	11.75	0.244	0.571	0.427	23.2	31.3	0.122	0.521 0.234	27.6	321.7 45.6 0.953	
TDSS11 7.4	1B	25.1 / 63.3; 43.5; 52.7 / 43	0.242 3.9(c)	1.00 5.00	10.2	15.0	0.288	0.579	0.497	26.4	30.0	0.157	0.654 0.240	33.2	248 29.3 0.939	Run at 1° C

(a) T13, RE=7.6 ft.
(b) T9, RE=5.1 ft.
(c) T17, RE=7.3 ft.
(d) T15, RE=8 ft.
(e) T19, RE=8.5 ft.

* Corrected for height of pins

Table 6-2: SUMMARY OF NORMALLY CONSOLIDATED CK_oUDSS TESTS - SITE W

All Stresses in ksf

Test RE (ft.)	Bor-ing	I_p W_N W_L I_L (%) -2μ	IN SITU σ_{vo}' σ_p'	TEST OCR σ_{vc}'	max ε_v (%)	AT τ_h MAX					AT $\gamma \cong 30\%$				E_{50}/c_u G_i/σ'_{vc} R_f	Remarks
						γ (%)	$\frac{\tau_h}{\sigma_{vc}'}$	$\frac{\sigma_v'}{\sigma_{vc}'}$	τ_h/σ'_v	$\psi°$	γ (%)	$\frac{\tau_h}{\sigma_{vc}'}$	τ_h/σ_v' σ_v/σ_{vc}'	$\psi°$		
WDSS1 14.2	5B	26.4 29.2 51.6 15.2 54	0.781 10.3(a)	1? 16.3	9.9	8.8	0.301	0.735	0.410	22.2	30.5	0.114	0.422 0.270	22.9	222.7 37.9 0.935	
WDSS2 3.4	5B	24.0 41.7 48.3 72.5 34	0.187 -	1.00 9.09	19.6	11.2	0.241	0.600	0.402	21.9	34.1	0.155	0.492 0.315	26.2	331.0 44.9 0.955	Holocene
WDSS3 7.0	5B	27.1 36.6 53.5 37.6 57	0.385 8.4(b)	1.00 16.4	14.8	9.5	0.236	0.639	0.369	20.3	30.5	0.145	0.453 0.320	24.3	313.3 40.1 0.948	
WDSS4 9.2	5B	20 28.9 44 24.5 56	0.506 6.7(d)	1.00 24.6	12.4	13.3	0.248	0.656	0.378	20.7	29.9	0.149	0.438 0.340	23.7	171.6 24.4 0.930	
WDSS5 9.3	5B	20 29.4 44 27.0 56	0.512 8.3(c)	1? 16.4	10.9	9.65	0.275	0.700	0.367	21.5	31.5	0.111	0.416 0.267	22.5	203.21 31.4 0.931	
WDSS6 9.4	5B	20 30.0 44 30.0 56	0.517 8.0(d)	1.00 24.6	12.4	12.9	0.242	0.628	0.385	21.1	27.3	0.120	0.404 0.297	22.0	183.0 24.8 0.935	Run at 1° C

(a) Test W10, RE=14.65 ft.

(b) Test W8, RE=8.1 ft.

(c) Test W9, RE=10.65 ft.

(d) DSS curve

Table 6-3: COMPARISON OF RESULTS OF NORMALLY CONSOLIDATED CK_oUDSS TESTS, SITES T AND W

	SITE T		SITE W	
	Mean ± 1 SD	Remarks	Mean ± 1 SD	Remarks
Peak τ_h/σ_{vc}'	0.28 ± 0.01 0.24 ± 0.01 0.27 ± 0.02	σ_{vc}'=5 ksf σ_{vc}'=16.4 ksf all tests	0.29 ± 0.02 0.24 ± 0.01 0.26 ± 0.03	tests 1,5 only exclude 1,5 all tests
Peak σ_v'/σ_{vc}'	0.587 ± 0.021 0.587 ± 0.020	σ_{vc}' = 5 all tests	0.631 ± 0.024	exclude 1,5
γ peak (%)	13.1 ± 1.2	tests without pins	10.9 ± 1.9	exclude 1,5
ψ peak	26.2 ± 3.4°	all tests	21.0 ± 0.7°	exclude 1,5
τ_h/σ_{vc}' at γ=30%	0.16 ± 0.04	tests without pins	0.13 ± 0.02	all tests
ψ at γ=30%	28.3 ± 2.8°	tests without pins	23.6 ± 1.5	all tests
$E_u(50)/C_u$	279.4 ± 34.1	tests without pins	242.5 ± 75.5	all tests
	255.3 ± 53.1	all tests		
G_i/σ_{vc}'	40.7 ± 7.4 40.20± 7.7	all tests tests without pins	33.9 ± 8.4	all tests
R_f	0.95 ± 0.01 0.951± 0.01	all tests tests without pins	0.939 ± 0.01	all tests

Table 6-4: CONSOLIDATION PROPERTIES OF DSS TESTS

Test	CR	σ_{vm}' (ksf)	σ_p'(ksf) Mean ± 1SD	$\frac{\sigma_{vm}'}{\sigma_p'}$	Test OCR	SR
TDSS1B	-	4.71	1.7 ± 0.12	2.77	1	
TDSS2	0.125	5.00	1.7 ± 0.12	2.94	8	0.026
TDSS3	0.204	5.04	1.7 ± 0.12	2.96	1	
TDSS4	0.120	4.92	3.0 ± 0.21	1.64	1	
TDSS5	0.133	5.00	3.0 ± 0.21	1.67	8.5	0.032
TDSS6	0.126	4.94	3.9 ± 0.6	1.27	1	-
TDSS7	0.113	4.94	3.5 ± 0.28	1.41	1	-
TDSS8	0.11	5.00	3.5 ± 0.28	1.43	15.75	0.0225
TDSS9	0.19	15.56	*2.41 ± 0.25	6.46	1	-
TDSS10	0.138	15.56	*3.29 ± 0.69	4.73	1	-
TDSS11 (1°C)	0.128	5.00	3.9 ± 0.6	1.28	1	-
WDSS1	0.08	16.3	10.3± 1.2	1.58	1	-
WDSS2	0.120	9.09	0.4(?)	22.73	1	-
WDSS3	0.127	16.4	8.4 ± 0.6	1.95	1	-
WDSS4	0.123	24.6	*6.7± 0.99	3.67	1	-
WDSS5	0.085	16.4	8.3 ± 1.0	1.38	1	-
WDSS6 (1°C)	0.118	24.6	*8.0	3.08	1	-
WDSS7	0.12	24.6	*7.2 ± 0.28	3.42	5	0.027
WDSS8	0.10	24.6	8.3 ± 1.0	2.96	10	0.029
WDSS9	0.11	24.6	8.3 ± 1.0	2.96	20	0.03

*Estimates from DSS curves
(All others from adjacent oedometers)

Table 6-5: SUMMARY OF OVERCONSOLIDATED CK_0UDSS TESTS

All Stresses in ksf

Site	TEST RE (ft.)	Bor-ing	I_p W_N W_L I_L (%) -2μ	IN SITU σ_{vo}' σ_p'	TEST OCR σ_{vc}'	max ϵ_v (%)	AT τ_h MAX γ (%)	$\frac{\tau_h}{\sigma_{vc}'}$	$\frac{\sigma_v'}{\sigma_{vc}'}$	τ_h/σ_p' σ_v/σ_p'	ϕ°	AT γ = 30% γ (%)	$\frac{\tau_h}{\sigma_{vc}'}$	τ_h/σ_p' σ_v'/σ_p'	ϕ°	E_{50}/c_u G_i/σ_{vc}' Rf	Remarks
T	TUSS2 7.7	3B	12.1 35.3 40.2 1.4(?) 33	0.396 1.7(a)	7.93 0.631	12.1	10.2	1.482	2.035	0.187 0.332	29.3	30.5	0.946	0.119 0.194	31.5	110 109.80 0.84	- sand layer
T	TDSS5 4.6	3B	24.0 50.2 37.2 45.8 41.5	0.226 3.04(b)	8.5 0.588	12.9	12.0	1.222	2.392	0.144 0.281	27.1	30.5	0.930	0.109 0.191	29.7	90 64.8 0.80	
T	TDSS8 7.1	5B1	20.5 47.3 38.3 56.1 40	0.391 3.5(c)	15.75 0.307	15.7	23.4*	2.053	3.607	0.130 0.229	29.7*	29.3	2.021	0.128 0.220	30.2	50* 115.4 0.87	- stones w/ pins - end of tube
W	WDSS7 9.5	5B	20.0 44.0 29.4 27.0 56.0	0.523 7.2 (d)	5.06 4.86	9.6	8.71	0.742	1.805	0.148 0.361	22.3	30.2	0.418	0.084 0.236	19.5	116.4 53.0 0.86	- pebble removed during trimming
W	WDSS8 9.6	5B	20.0 44.0 29.0 25.0 56.0	0.528 8.3(e)	10.09 2.44	10.3	9.26	1.036	2.205	0.103 0.219	26.2	30.0	0.443	0.044 0.110	21.8	86.4 84.9 0.81	
W	WDSS9 9.7	5B	20.0 44.0 28.9 24.5 56.0	0.534 8.3(e)	20.16 1.22	10.0	10.8*	2.321	4.756	0.115 0.236	26.0	29.9*	1.702	0.084 0.172	26.0	49.2* 68.7 0.66	- stones w/ pins

(a) From T13, RE=7.6 ft.
(b) From T9, RE=5.1 ft.
(c) From T15, RE=8 ft.
(d) From DSS curve
(e) From W9, RE=10.65 ft.

* Corrected for height pins

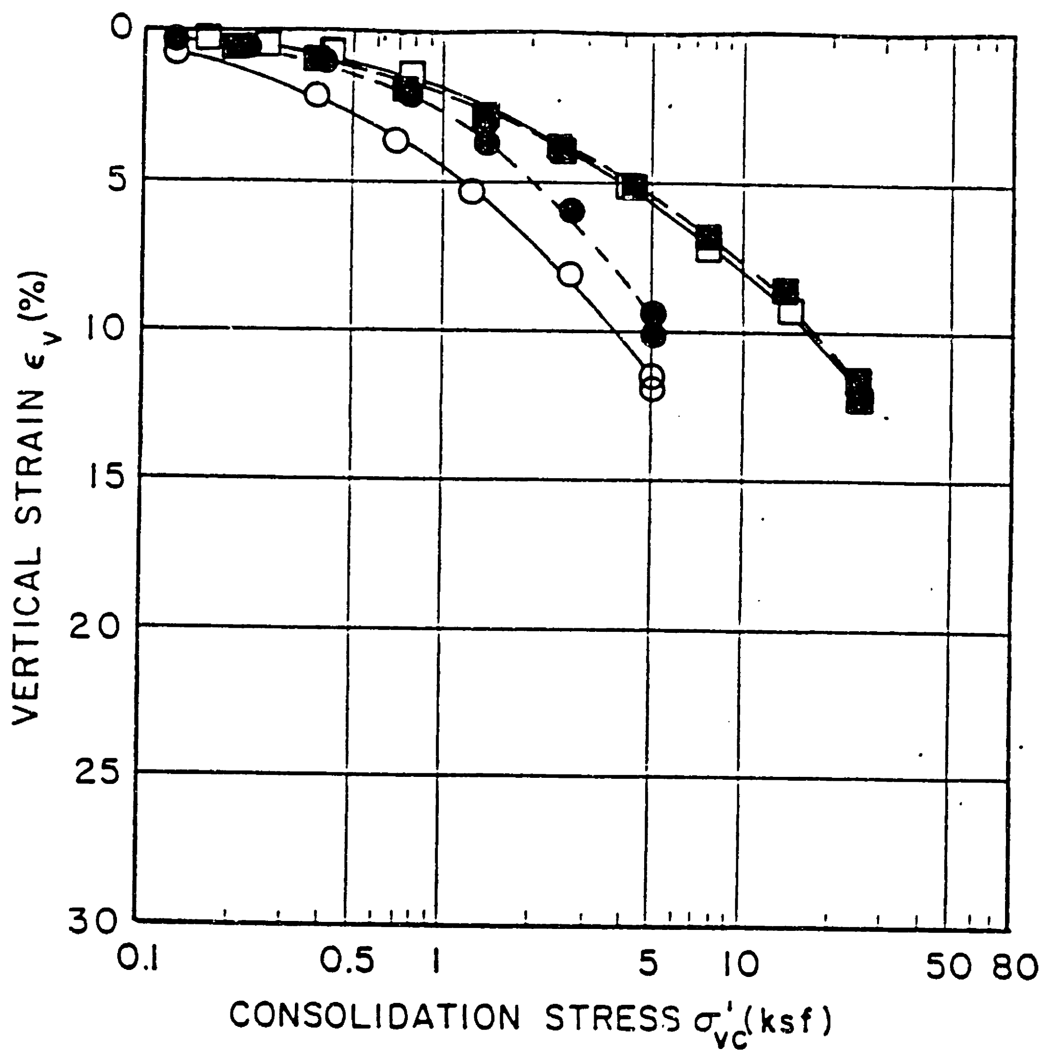
Table 6-6: SUMMARY OF CK_0UDSS RECOMPRESSION TESTS

All Stresses in ksf

Site	Test RE (ft.)	Bor- ing	I_P w_N w_L I_L (%) -2μ	IN SITU σ_{vo}' σ_p'	TEST OCR σ_{vc}'	AT τ_h MAX γ (%)	$\frac{\tau_h}{\sigma_{vc}'}$	$\frac{\sigma_v'}{\sigma_{vc}'}$	τ_h/σ_v' σ_v'/σ_p'	$\psi°$	Remarks
T	TDSS1 7.5	3B	12.1 35.3 37.5 118.2 33	0.385 1.7(a)	4.4 0.387	11.9*	0.644	1.146	0.179 0.318	29.3	MIT - used stones w/ pins
T	T3B-P2 5.1	3B	24.0 50.2 35.2 11.5 37.5	0.253 3.0(b)	11.9 0.288	20	1.95	1.75			TETC
T	T5B-P3 7.1	5B	12.1 35.3 23.8 33 5.0	0.336 1.7(c)	5.06 0.374	20	1.96	2.12			TETC
W	W3B-P1 2.25	3B	- - 23.8 - -	0.058 7.5(d)	129.3 0.072	20	2.78	2.40			TETC
W	W3B-P3 6.8	3B	21 48 29.5 11.9 -	0.308 9.9(c)	29.2 0.317	24	1.55	2.04			TETC
W	W5B-P3 8.15	5B	27.1 53.5 29.8 12.5 57	0.448 8.4(f)	18.75 1.44	19	0.78	1.06			TETC

(a) From T13, RE=7.6 ft.
(b) From T9, RE=5.1 ft.
(c) From T4, RE=7.6 ft.
(d) From W1, RE=2.25 ft.
(e) From W3, RE=6.8 ft.
(f) From W8, RE=8.1 ft.

* corrected for height of pins



Test No.	TEMPERATURE	RE (ft)	w_N (%)	σ'_{vc} (ksf)	OCR	Symbol
TDSS6	20° C	7.0	41.5	4.9	1	—○—
TDSS11	1° C	7.4	43.5	5.0	1	--●--
WDSS4	20° C	9.2	28.9	24.6	1	—□—
WDSS6	1° C	9.4	30.0	24.6	1	--■--

Figure 6-1: Compression Curves from Normally Consolidated CK_0UDSS Tests at 0 °C and 20 °C

Test No.	TEMPERATURE	RE (ft)	w_N (%)	σ'_{vc} (ksf)	OCR	Symbol
TDSS6	20° C	7.0	41.5	4.9	1	—○—
TDSS11	1° C	7.4	43.5	5.0	1	- - ● - -
WDSS4	20° C	9.2	28.9	24.6	1	—□—
WDSS6	1° C	9.4	30.0	24.6	1	- - ■ - -

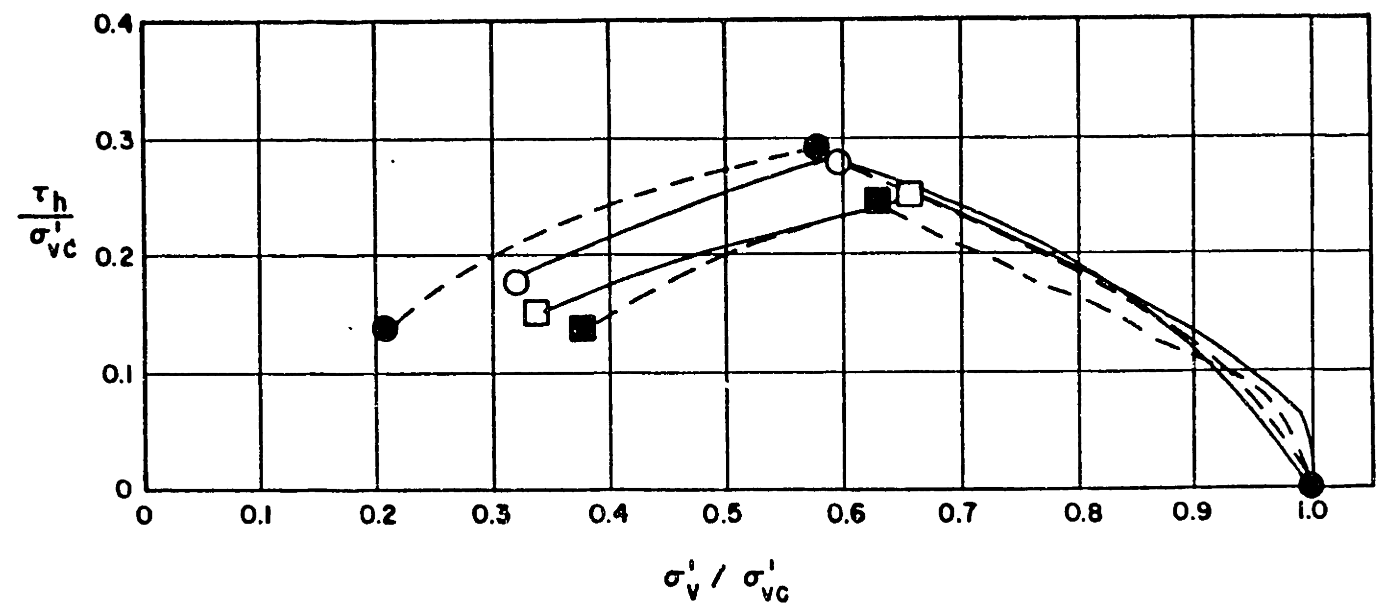
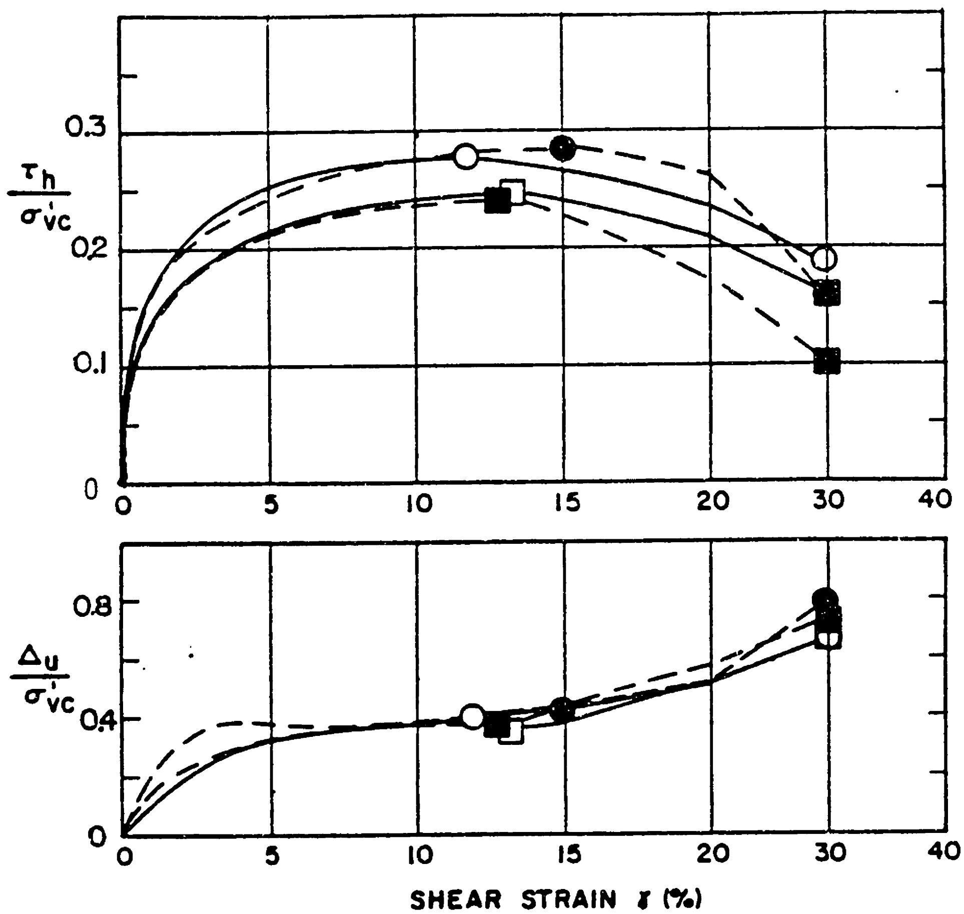
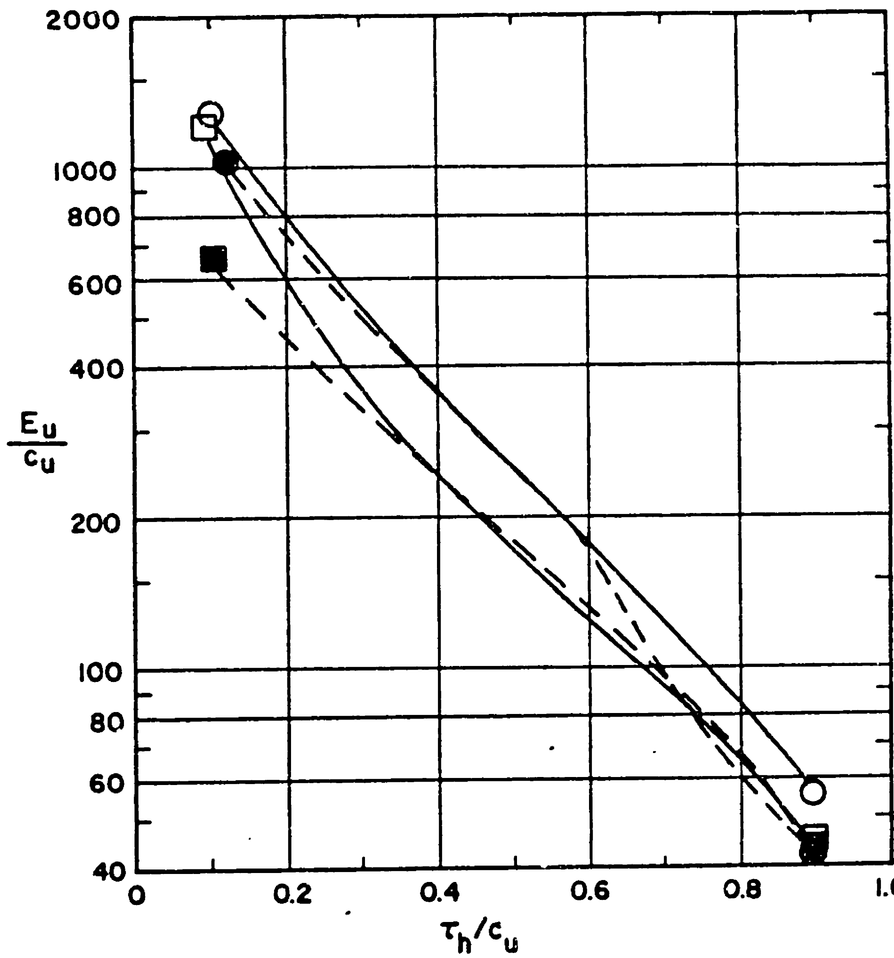


Figure 6-2: Normalized Stress Paths from Normally Consolidated CK_0UDSS Tests at 0 °C and 20 °C



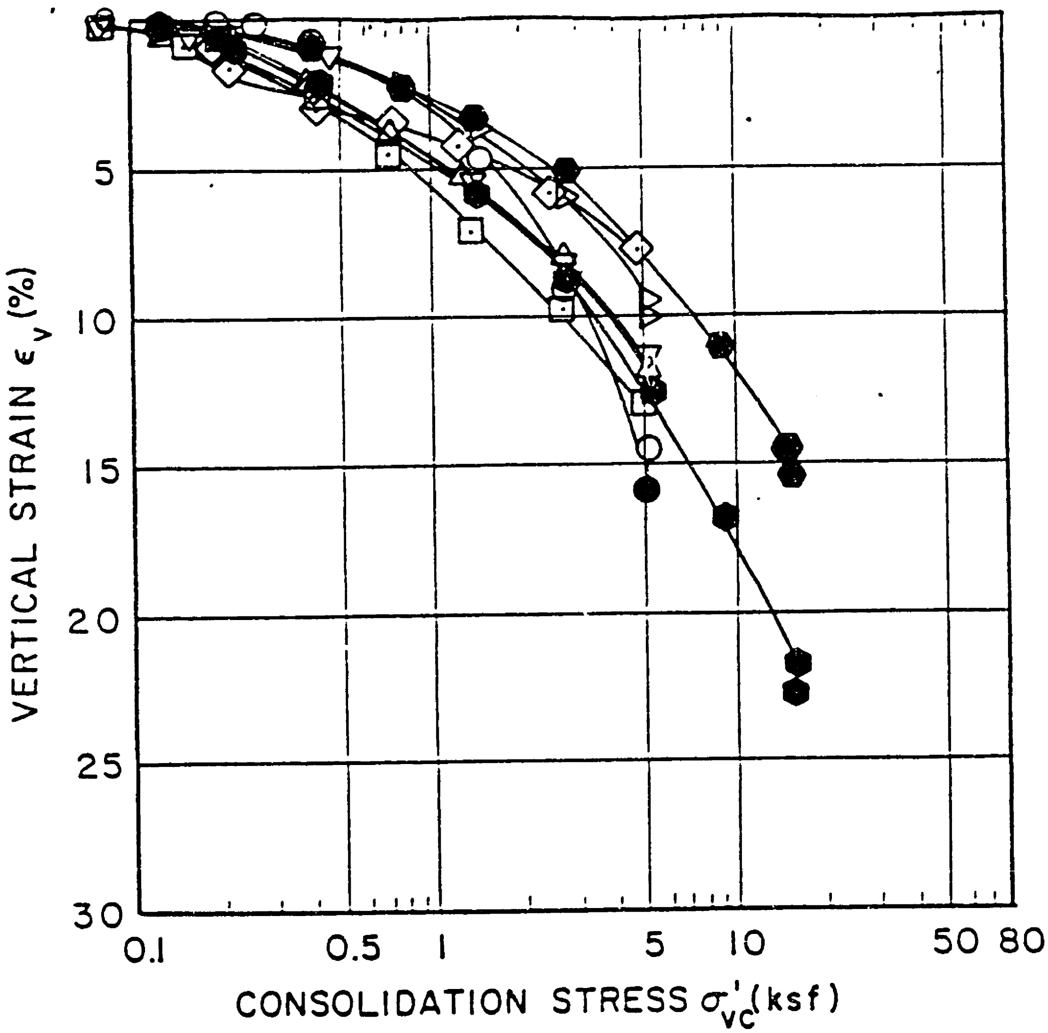
Test No.	TEMPERATURE	RE (ft)	w_N (%)	σ'_{vc} (ksf)	OCR	Symbol
TDSS6	20° C	7.0	41.5	4.9	1	—○—
TDSS11	1° C	7.4	43.5	5.0	1	--●--
WDSS4	20° C	9.2	28.9	24.6	1	—□—
WDSS6	1° C	9.4	30.0	24.6	1	--■--

Figure 6-3: Normalized Stress vs. Strain from Normally Consolidated CK_0UDSS Tests at 0 °C and 20 °C



Test No.	TEMPERATURE	RE (ft)	w_N (%)	σ'_{vc} (ksf)	OCR	Symbol
TDSS6	20° C	7.0	41.5	4.9	1	—○—
TDSS11	1° C	7.4	43.5	5.0	1	--●--
WDSS4	20° C	9.2	28.9	24.6	1	—□—
WDSS6	1° C	9.4	30.0	24.6	1	--■--

Figure 6-4: Normalized Modulus from Normally Consolidated CK_0UDSS Tests at 0 °C and 20 °C



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
TDSS1B	3B-P3 (pins)	7.5	37.5	4.7	1.0	◇
TDSS3	3B-P3	7.9	54.4	5.0	1.0	○
TDSS4	3B-P2	4.7	35.3	4.9	1.0	▽
TDSS6	1B-02	7.0	41.5	4.9	1.0	△
TDSS7	5B1-P3 (pins)	7.0	36.4	4.9	1.0	⊡
TDSS9	1B-02	7.2	45.9	15.6	1.0	⬢
TDSS10	5B1-P3	8.4	38.4	15.6	1.0	●
TDSS11	1B-02 (1°C)	7.4	43.5	5.0	1.0	▷

Figure 6-5: Compression Curves from Normally Consolidated CK_0UDSS Tests at Site T

Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
TDSS1B	3B-P3 (pins)	7.5	37.5	4.7	1.0	◈
TDSS3	3B-P3	7.9	54.4	5.0	1.0	○
TDSS4	3B-P2	4.7	35.3	4.9	1.0	▽
TDSS6	1B-02	7.0	41.5	4.9	1.0	△
TDSS7	5B1-P3 (pins)	7.0	36.4	4.9	1.0	⊡
TDSS9	1B-02	7.2	45.9	15.6	1.0	⬢
TDSS10	5B1-P3	8.4	38.4	15.6	1.0	⬢
TDSS11	1B-02 (1°C)	7.4	43.5	5.0	1.0	▷

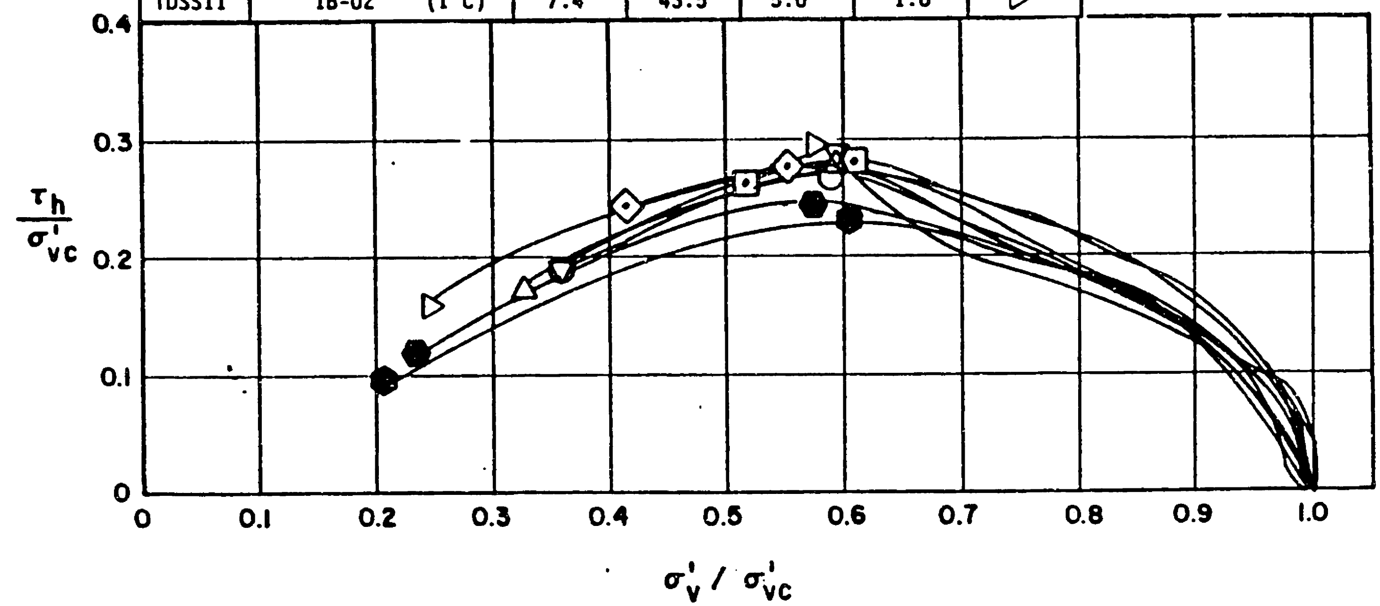
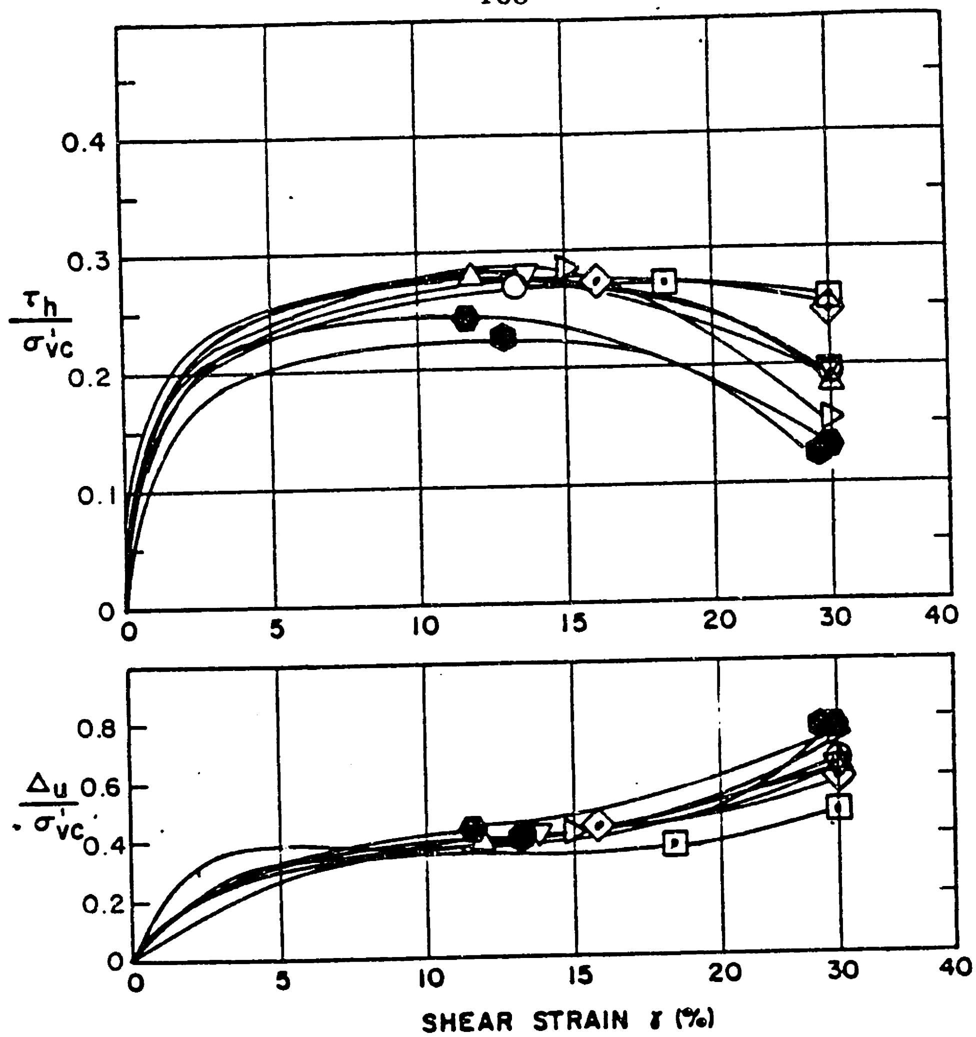
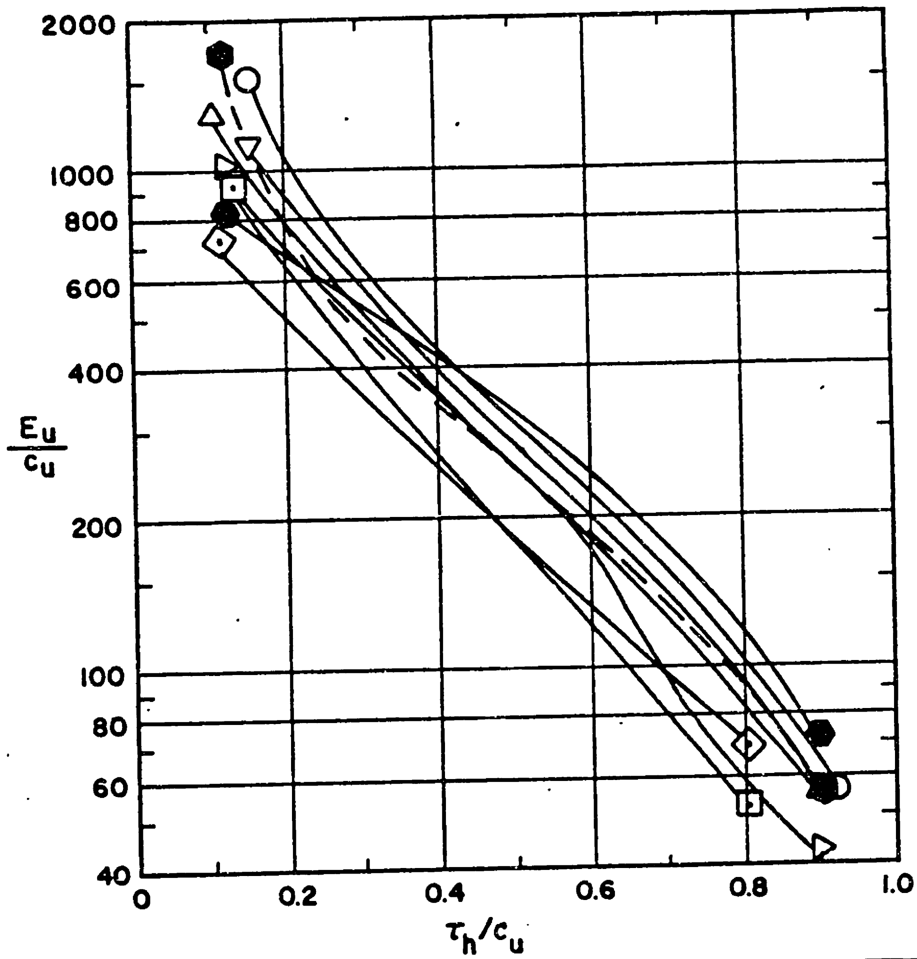


Figure 6-6: Normalized Stress Paths from Normally Consolidated CK_0UDSS Tests at Site T



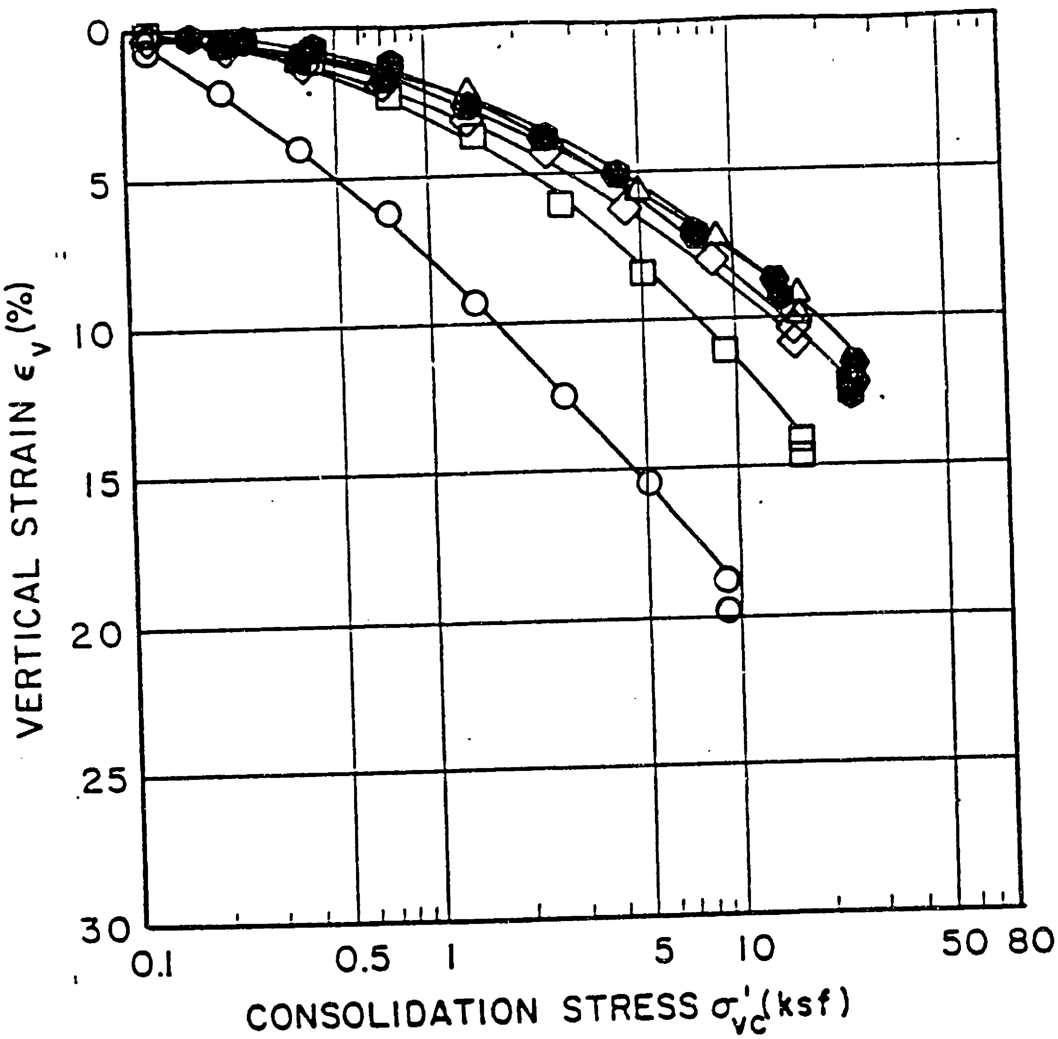
Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
TDSS1B	3B-P3 (pins)	7.5	37.5	4.7	1.0	◇
TDSS3	3B-P3	7.9	54.4	5.0	1.0	○
TDSS4	3B-P2	4.7	35.3	4.9	1.0	▽
TDSS6	1B-02	7.0	41.5	4.9	1.0	△
TDSS7	5B1-P3 (pins)	7.0	36.4	4.9	1.0	⊡
TDSS9	1B-02	7.2	45.9	15.6	1.0	⬢
TDSS10	5B1-P3	8.4	38.4	15.6	1.0	⬢
TDSS11	1B-02 (1°C)	7.4	43.5	5.0	1.0	▷

Figure 6-7: Normalized Stress vs. Strain from Normally Consolidated CK_0UDSS Tests at Site T



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
TDSS18	3B-P3 (pins)	7.5	37.5	4.7	1.0	◇ (with dot)
TDSS3	3B-P3	7.9	54.4	5.0	1.0	○
TDSS4	3B-P2	4.7	35.3	4.9	1.0	▽
TDSS6	1B-02	7.0	41.5	4.9	1.0	△
TDSS7	5B1-P3 (pins)	7.0	36.4	4.9	1.0	⊡
TDSS9	1B-02	7.2	45.9	15.6	1.0	⬢
TDSS10	5B1-P3	8.4	38.4	15.6	1.0	⬢
TDSS11	1B-02 (1°C)	7.4	43.5	5.0	1.0	▷

Figure 6-8: Normalized Modulus from Normally Consolidated CK_0UDSS Tests at Site T



Test No.	Sample No.	RE (ft)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS1	5B-P5	14.2	29.2	16.3	1.0	△
WDSS2	5PV-S1	3.4	41.7	9.1	1.0	○
WDSS3	5B-P3	7.0	36.6	16.4	1.0	□
WDSS4	5B-P4	9.2	28.9	24.6	1.0	⬢
WDSS5	5B-P4	9.3	29.4	16.4	1.0	◇
WDSS6	5B-P4	9.4	30.0	24.6	1.0	●

Figure 6-9: Compression Curves from Normally Consolidated CK_0UDSS Tests at Site W

Test No.	Sample No.	RE (ft)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS1	5B-P5	14.2	29.2	16.3	1.0	△
WDSS2	5PV-S1	3.4	41.7	9.1	1.0	○
WDSS3	5B-P3	7.0	36.6	16.4	1.0	□
WDSS4	5B-P4	9.2	28.9	24.6	1.0	⬢
WDSS5	5B-P4	9.3	29.4	16.4	1.0	◇
WDSS6	5B-P4	9.4	30.0	24.6	1.0	⬢

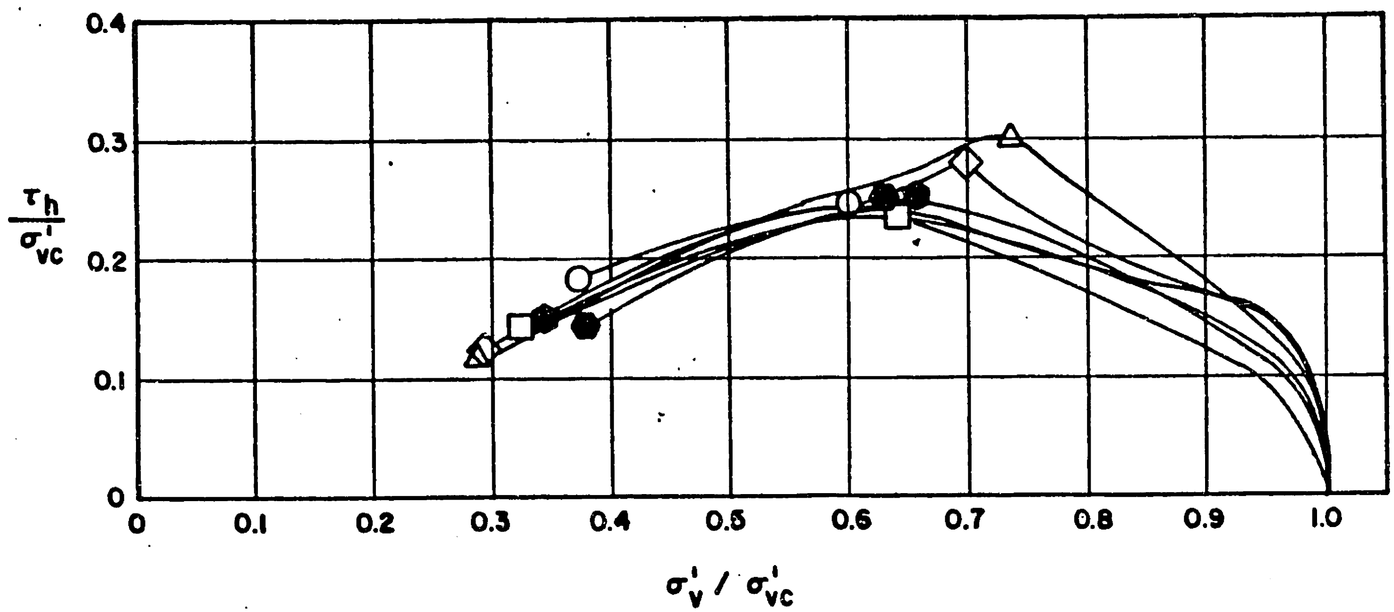
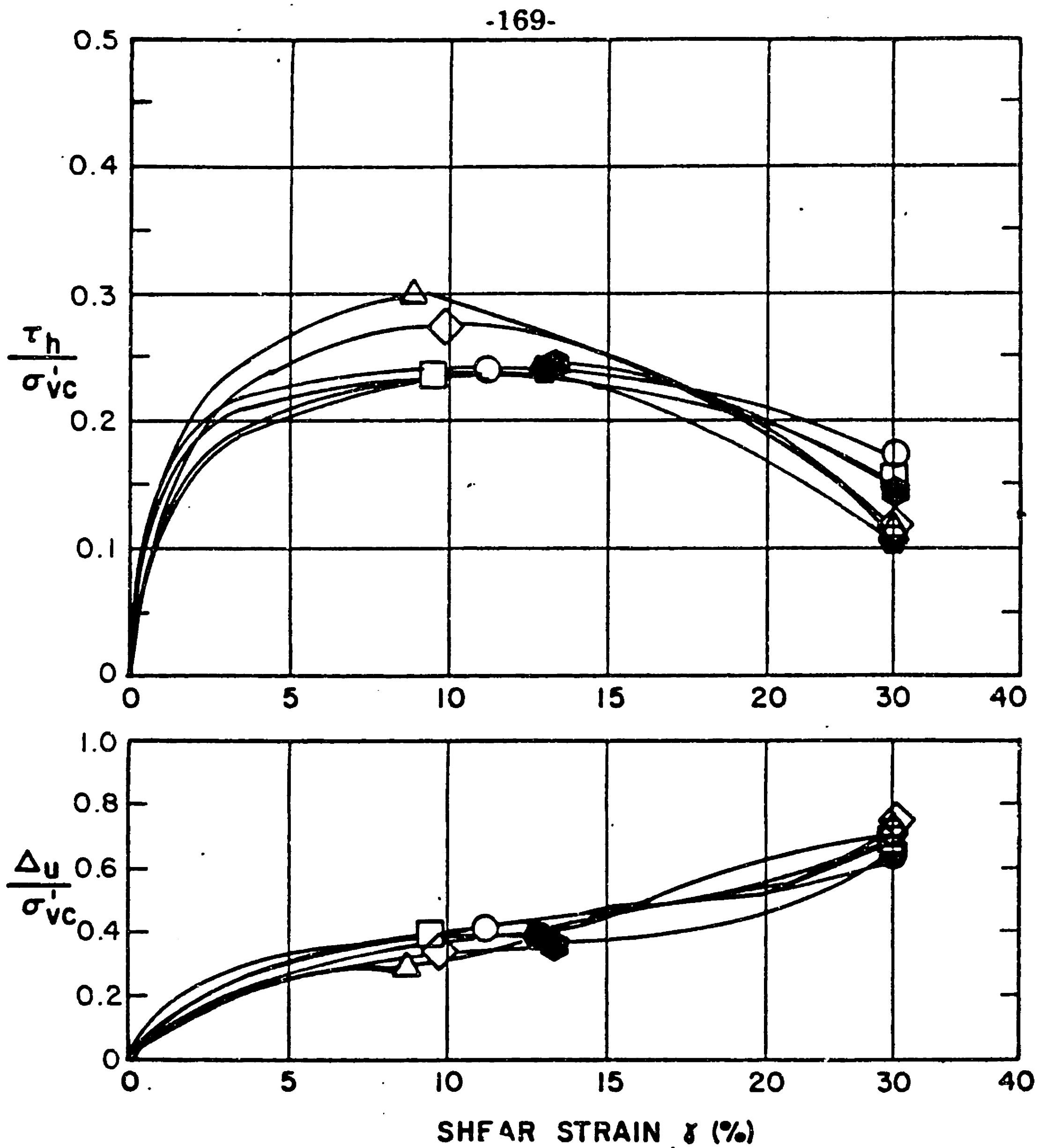
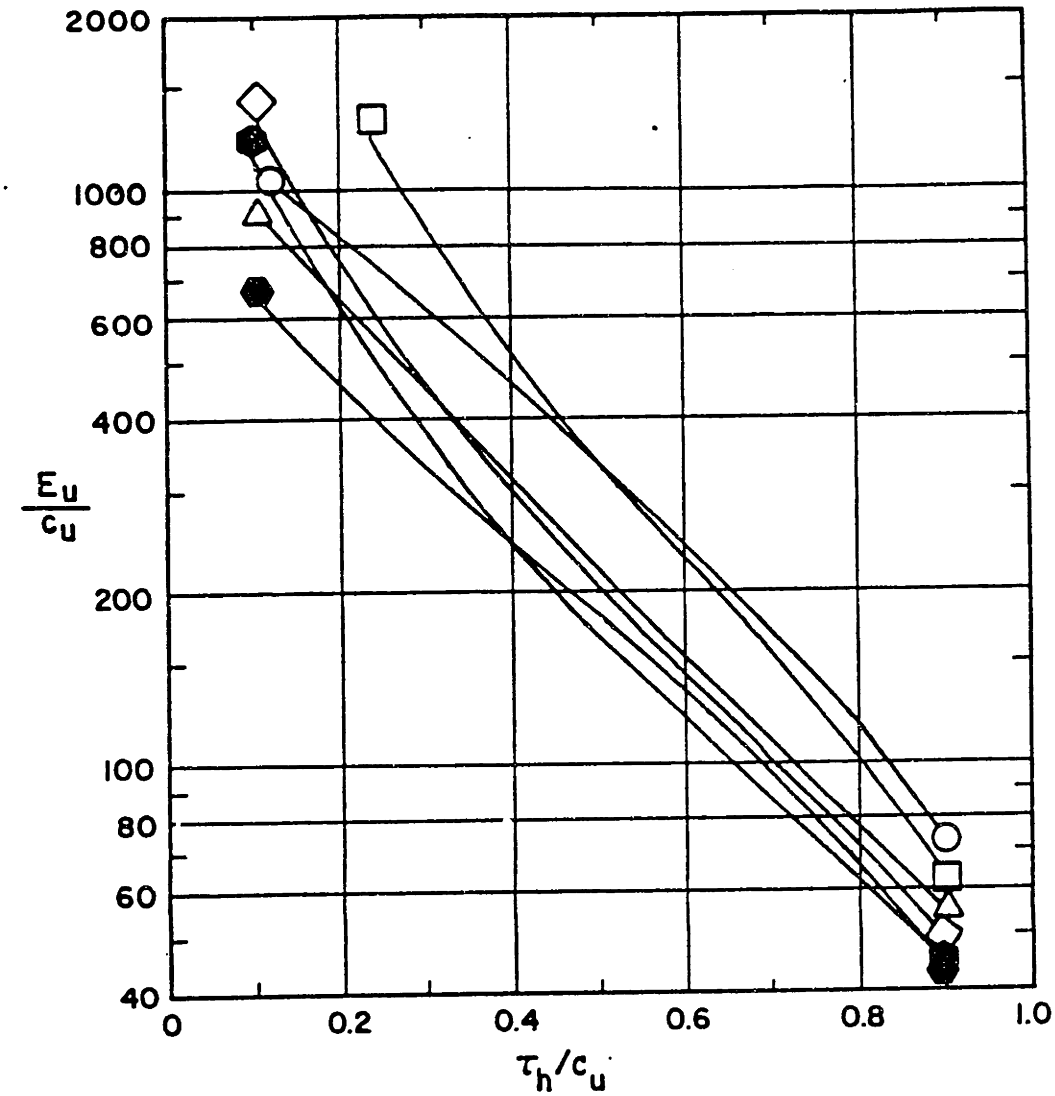


Figure 6-10: Normalized Stress Paths from Normally Consolidated CK_0UDSS Tests at Site W



Test No.	Sample No.	RE (ft)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS1	5B-P5	14.2	29.2	16.3	1.0	△
WDSS2	5PV-S1	3.4	41.7	9.1	1.0	○
WDSS3	5B-P3	7.0	36.6	16.4	1.0	□
WDSS4	5B-P4	9.2	28.9	24.6	1.0	●
WDSS5	5B-P4	9.3	29.4	16.4	1.0	◇
WDSS6	5B-P4	9.4	30.0	24.6	1.0	●

Figure 6-11: Normalized Stress vs. Strain from Normally Consolidated CK_0UDSS Tests at Site W



Test No.	Sample No.	RE (ft)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS1	5B-P5	14.2	29.2	16.3	1.0	△
WDSS2	5PV-S1	3.4	41.7	9.1	1.0	○
WDSS3	5B-P3	7.0	36.6	16.4	1.0	□
WDSS4	5B-P4	9.2	28.9	24.6	1.0	⬢
WDSS5	5B-P4	9.3	29.4	16.4	1.0	◇
WDSS6	5B-P4	9.4	30.0	24.6	1.0	⬢

Figure 6-12: Normalized Modulus from Normally Consolidated CK_0UDSS Tests at Site W

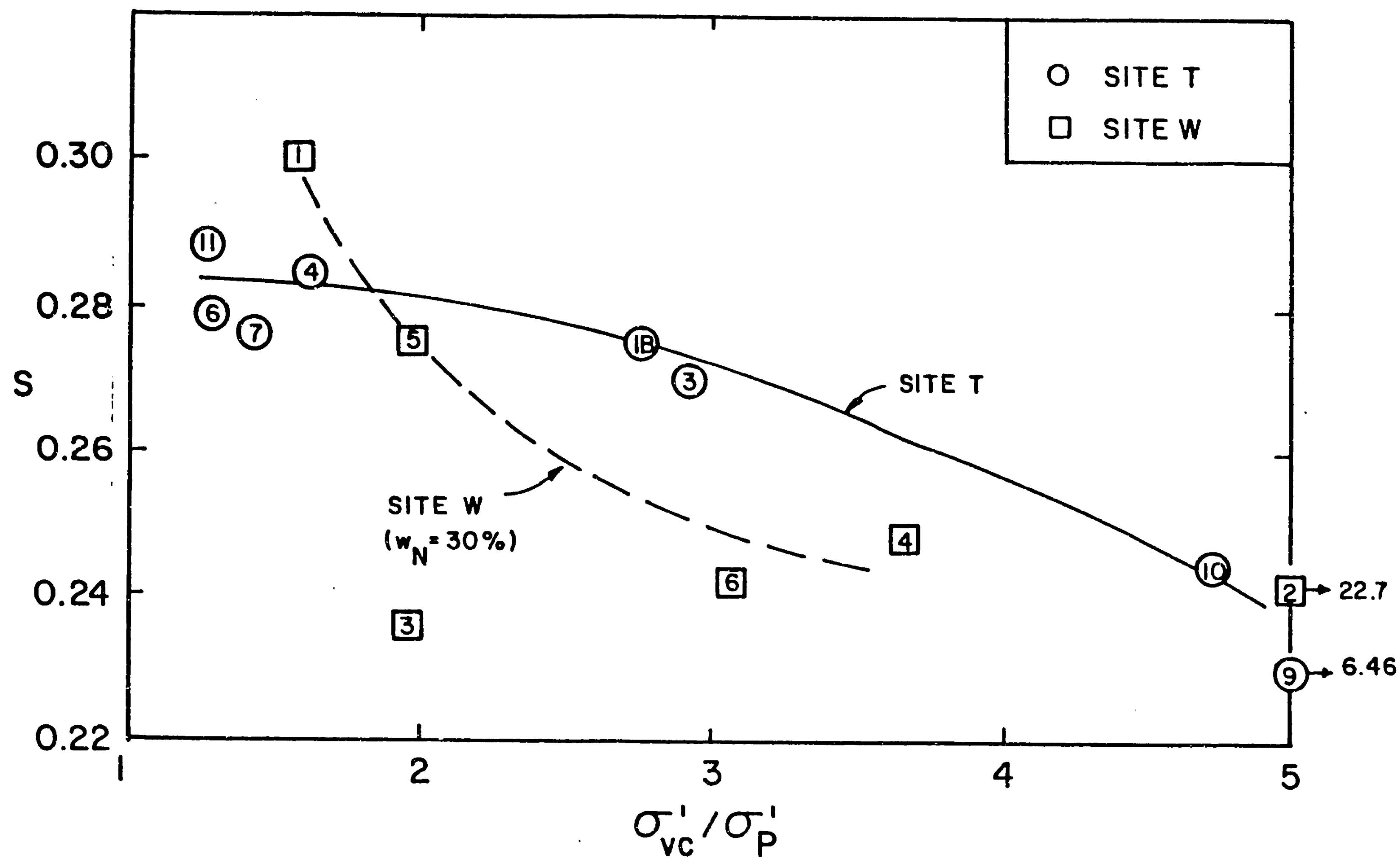


Figure 6-13: Normalized Shear Strength versus Stress Ratio, σ'_{vc}/σ'_p

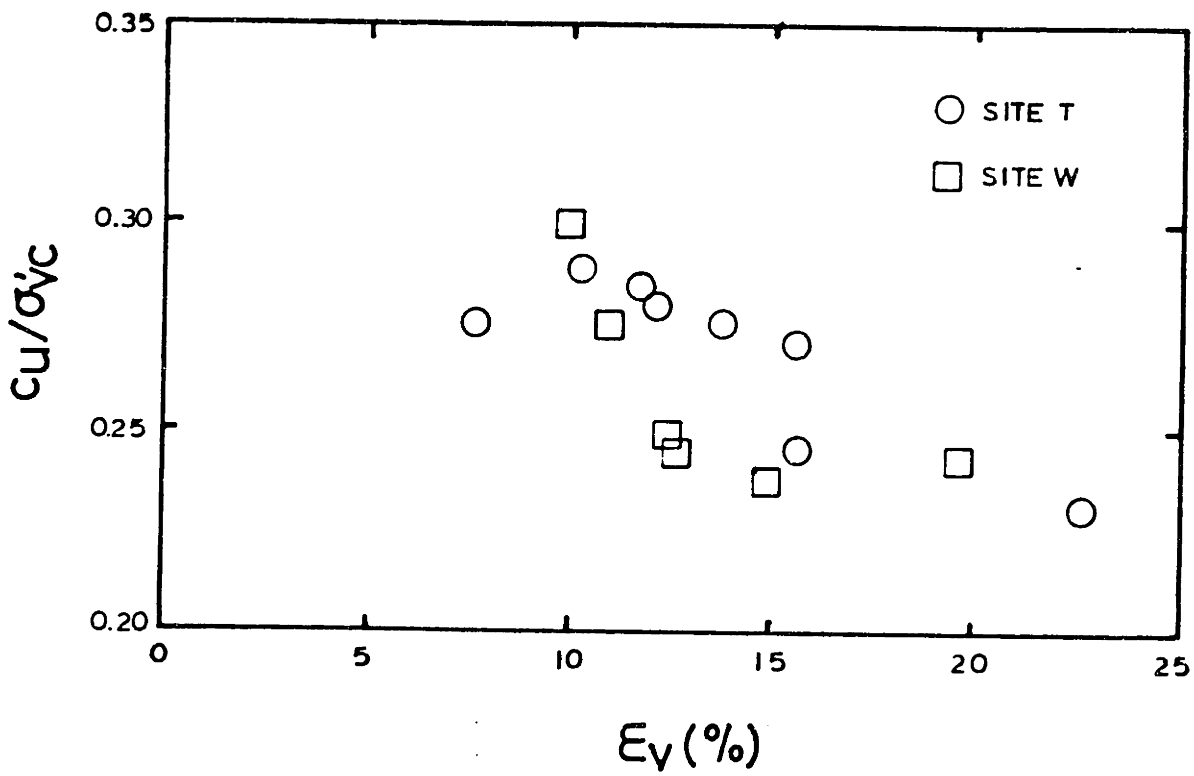


Figure 6-14: Normalized Shear Strength versus Vertical Consolidation Strain

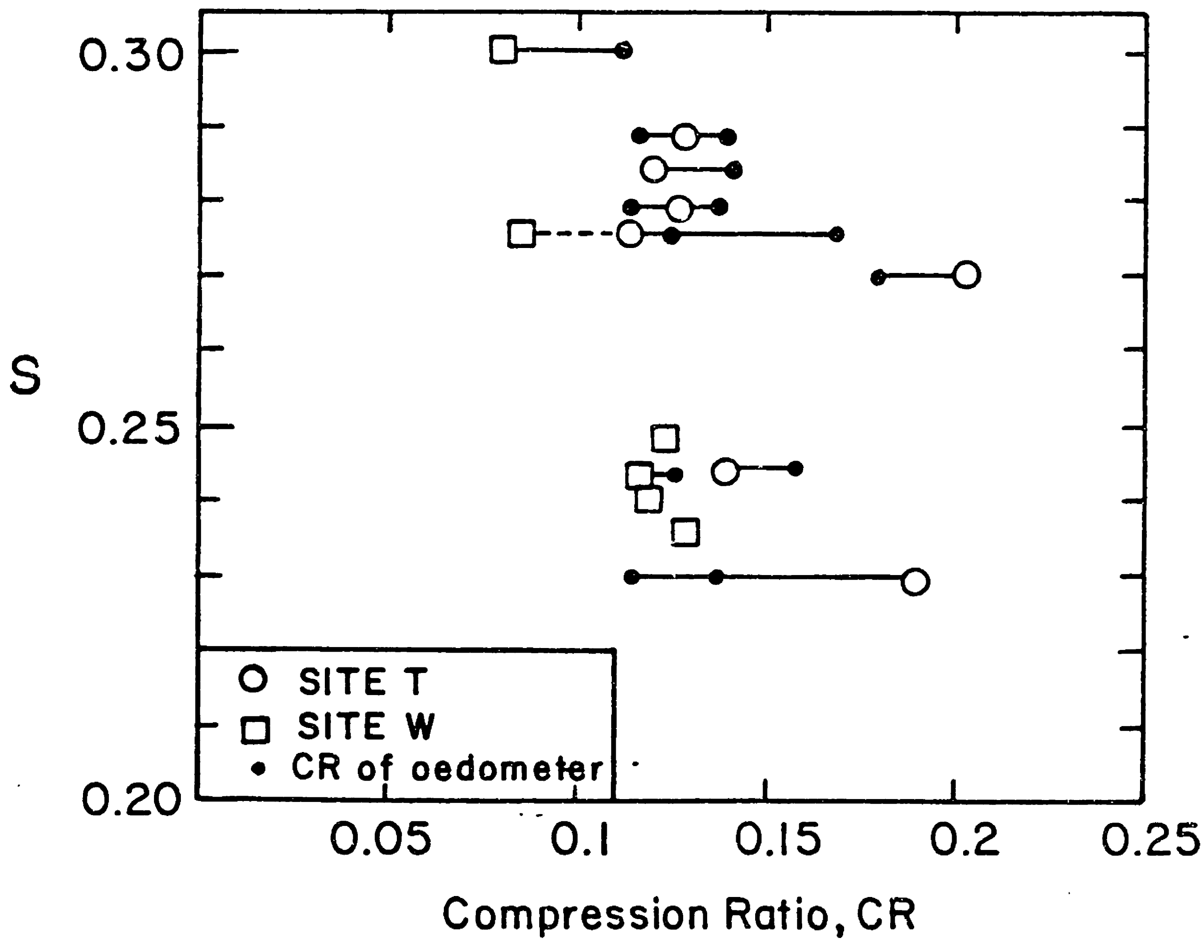


Figure 6-15: Normalized Shear Strength versus Compression Ratio

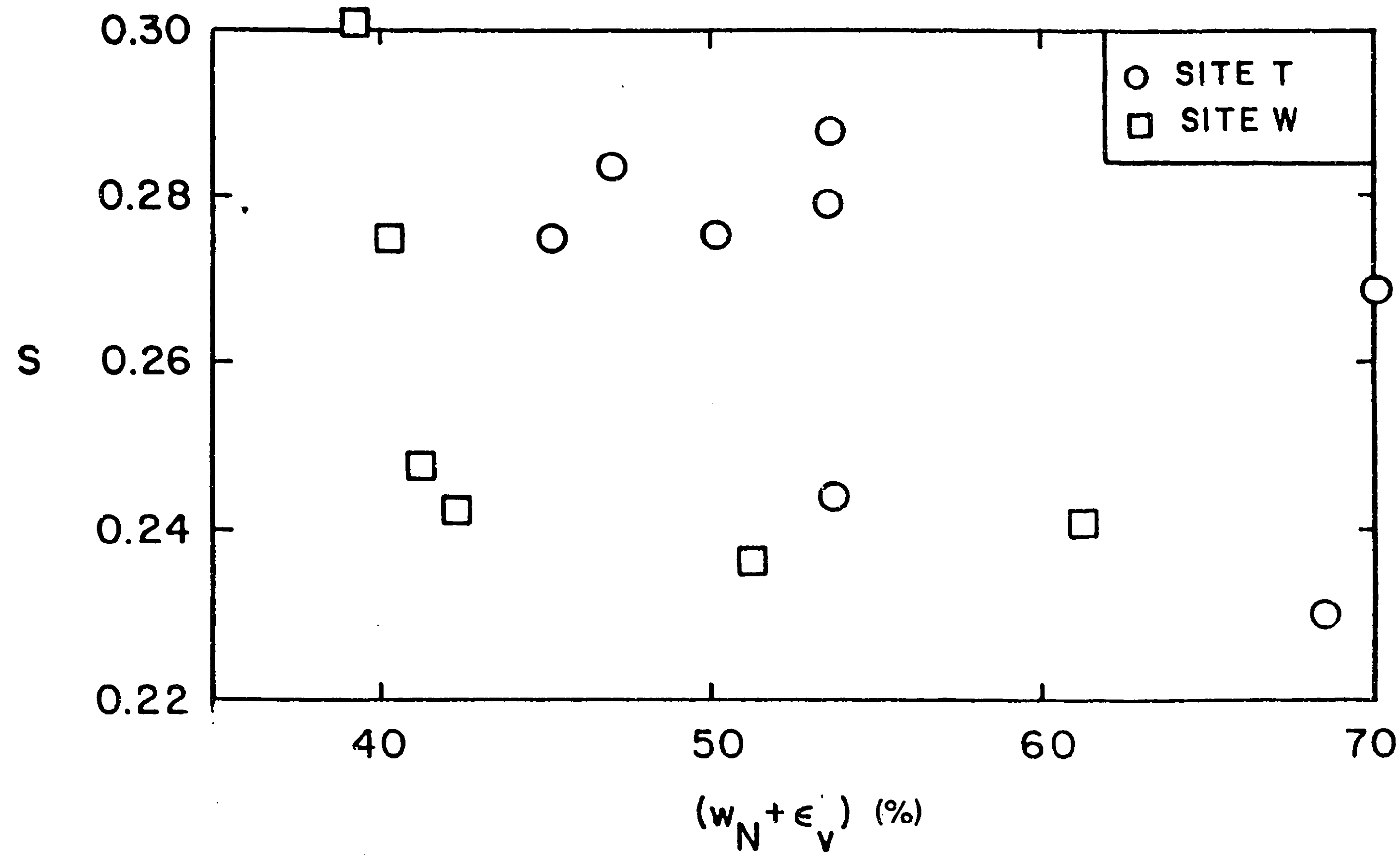


Figure 6-16: Normalized Shear Strength versus $(w_N + \epsilon_v)$

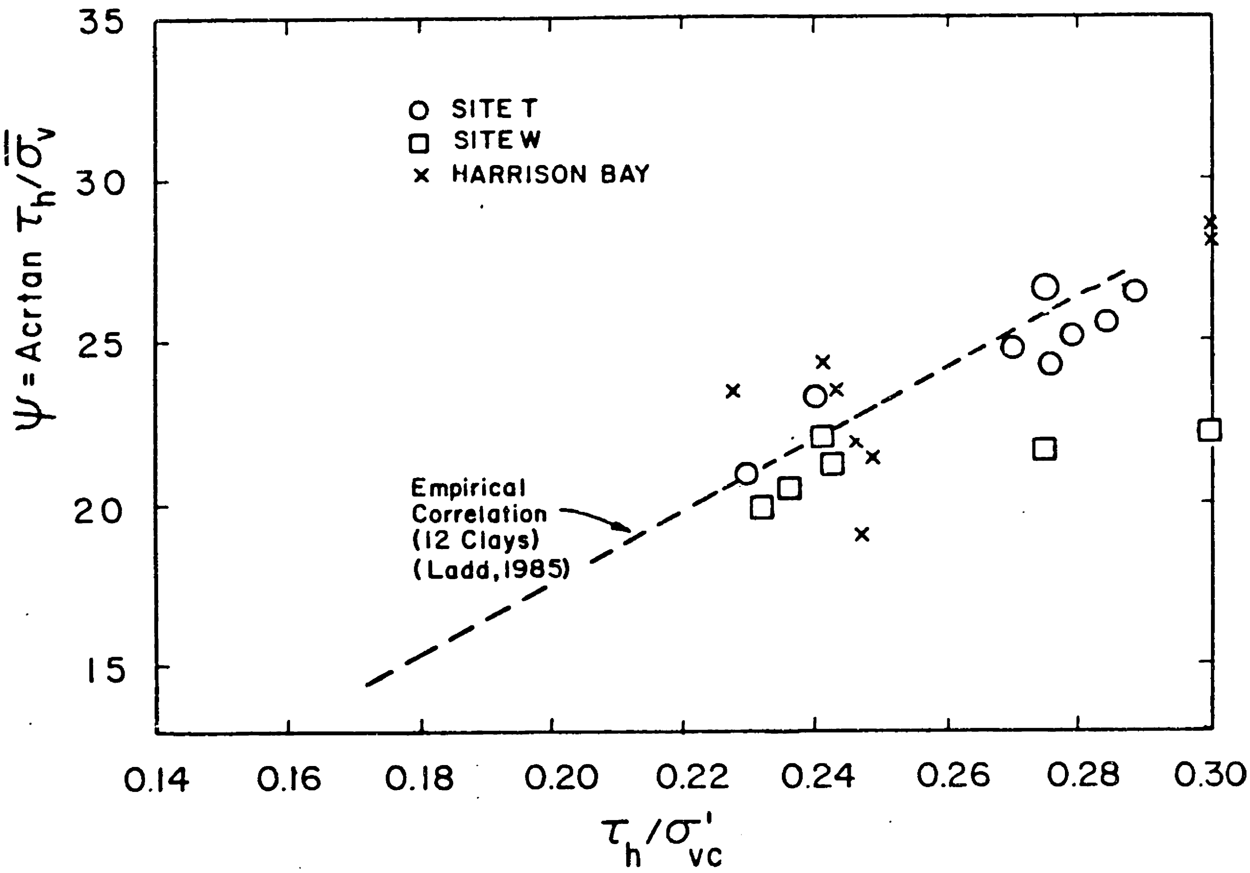


Figure 6-17: Normalized CK_0UDSS Strength versus ψ Angle for Normally Consolidated Arctic Silt

Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
TDSS18	3B-P3 (pins)	7.5	37.5	4.7	1.0	◇
TDSS3	3B-P3	7.9	54.4	5.0	1.0	○
TDSS4	3B-P2	4.7	35.3	4.9	1.0	▽
TDSS6	1B-02	7.0	41.5	4.9	1.0	△
TDSS7	5B1-P3 (pins)	7.0	36.4	4.9	1.0	⊡
TDSS9	1B-02	7.2	45.9	15.6	1.0	⬢
TDSS10	5B1-P3	8.4	38.4	15.6	1.0	⬢
TDSS11	1B-02 (1°C)	7.4	43.5	5.0	1.0	▷

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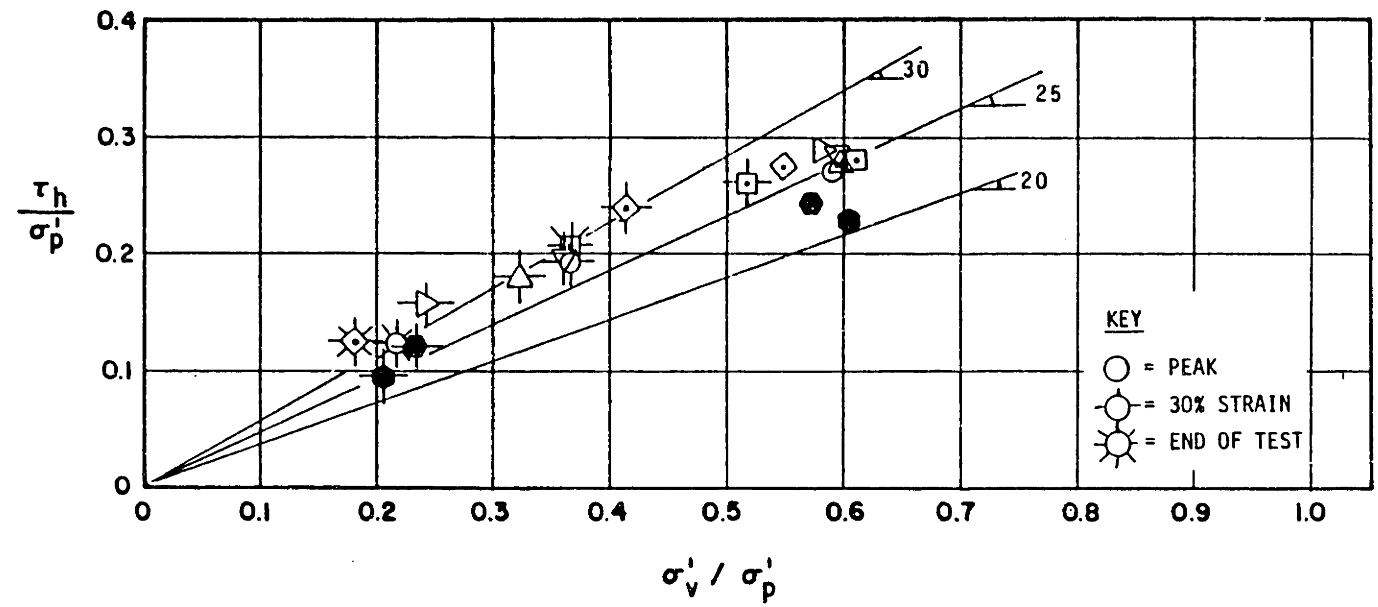

Figure 6-18: Peak and Maximum Obliquity of Effective Stress Paths for Normally Consolidated CK_0UDSS Tests at Site T

Test No.	Sample No.	RE (ft)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS1	5B-P5	14.2	29.2	16.3	1.0	△
WDSS2	5PV-S1	3.4	41.7	9.1	1.0	○
WDSS3	5B-P3	7.0	36.6	16.4	1.0	□
WDSS4	5B-P4	9.2	28.9	24.6	1.0	⬢
WDSS5	5B-P4	9.3	29.4	16.4	1.0	◇
WDSS6	5B-P4	9.4	30.0	24.6	1.0	⬢

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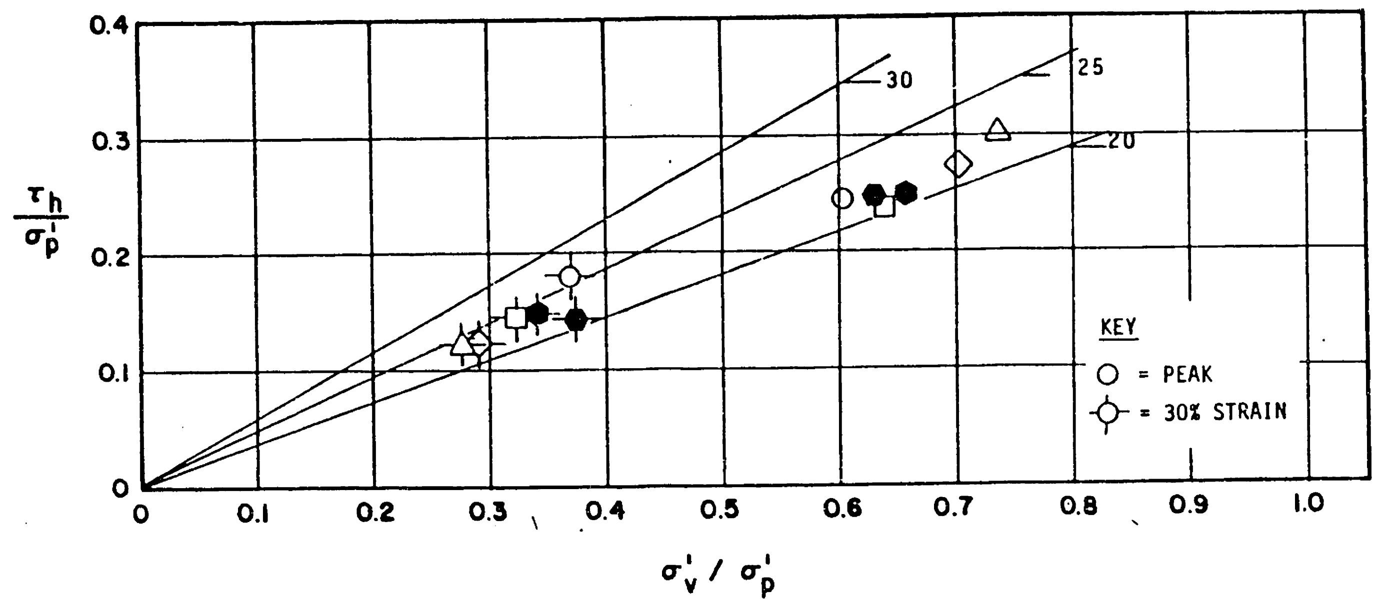


Figure 6-19: Peak and Maximum Obliquity of Effective Stress Paths for Normally Consolidated CK_0UDSS Tests at Site W

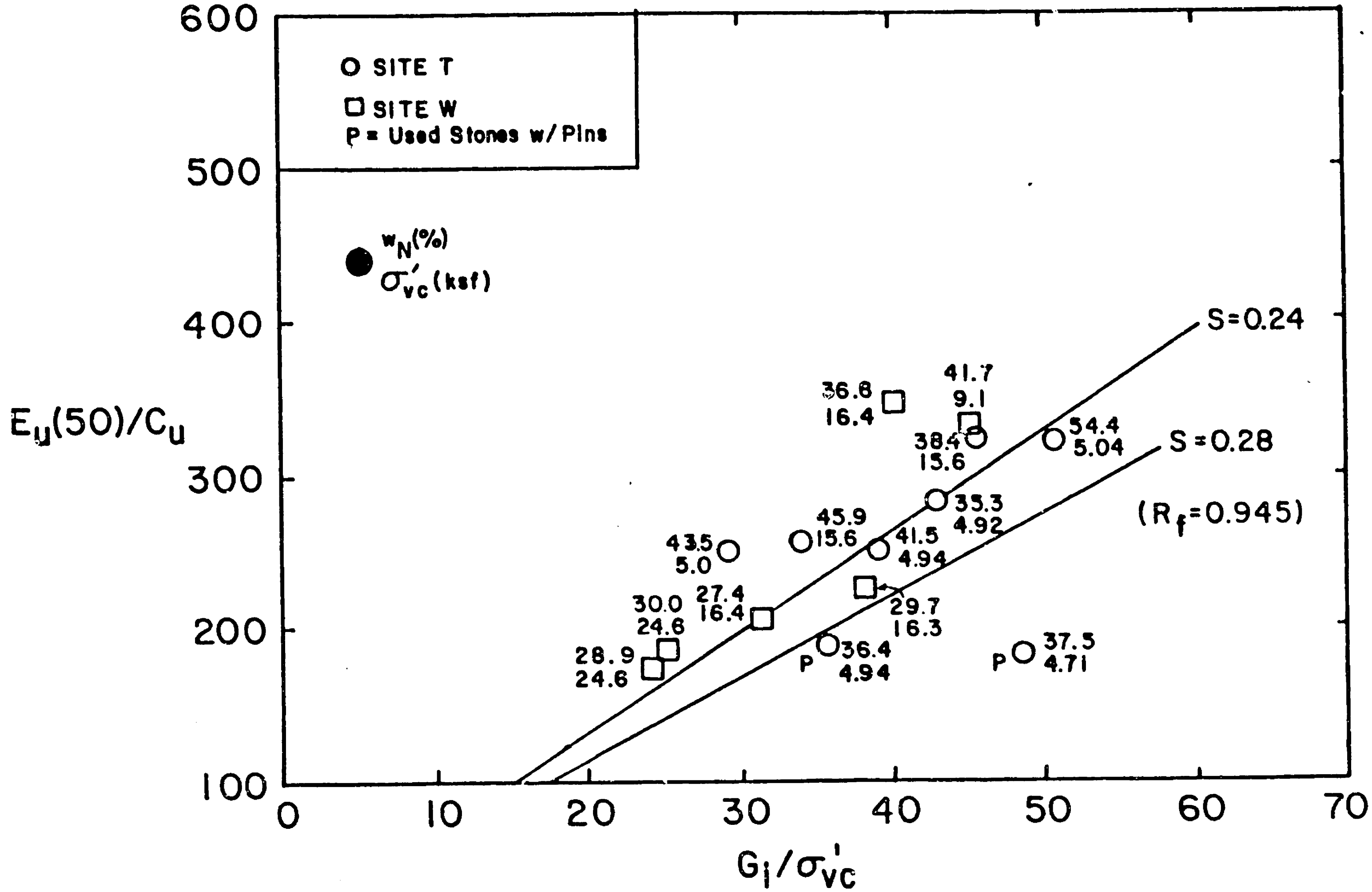


Figure 6-20: Normalized Undrained Modulus versus Initial Shear Modulus for Normally Consolidated CK_0DSS Tests at Smith Bay

Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (ksf)	σ'_p (ksf)	OCR	Symbol
TDSS2	3B-P3	1.7	40.2	0.631	5.0	7.9	○
TDSS5	3B-P2	4.6	37.2	0.588	5.0	8.5	▽
TDSS8	5B1-P3 (pins)	7.1	38.3	0.313	4.93	15.75	⊡

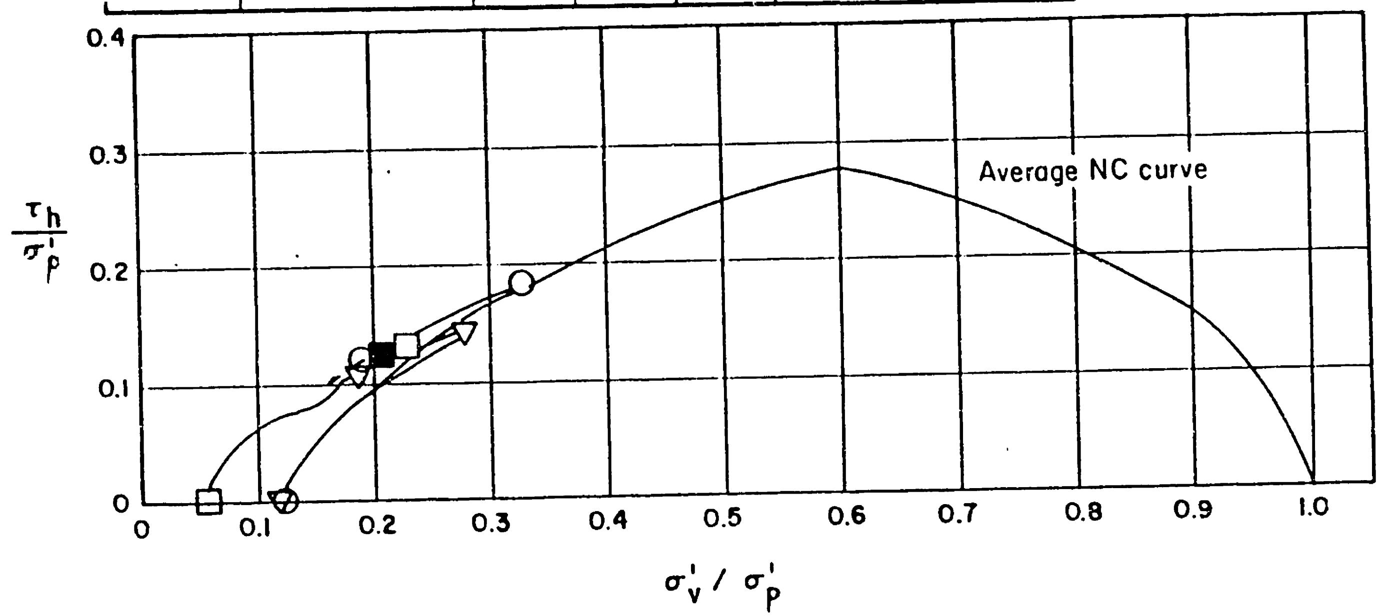
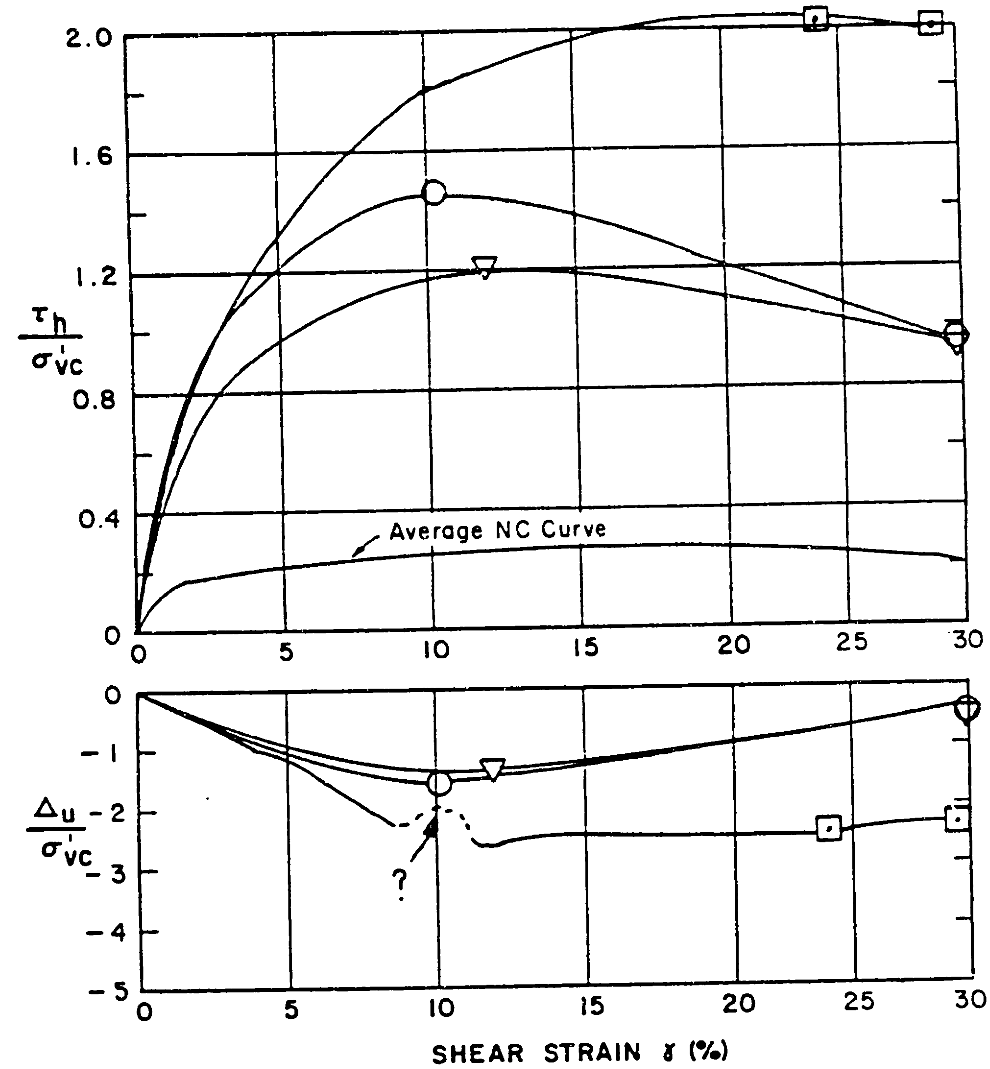
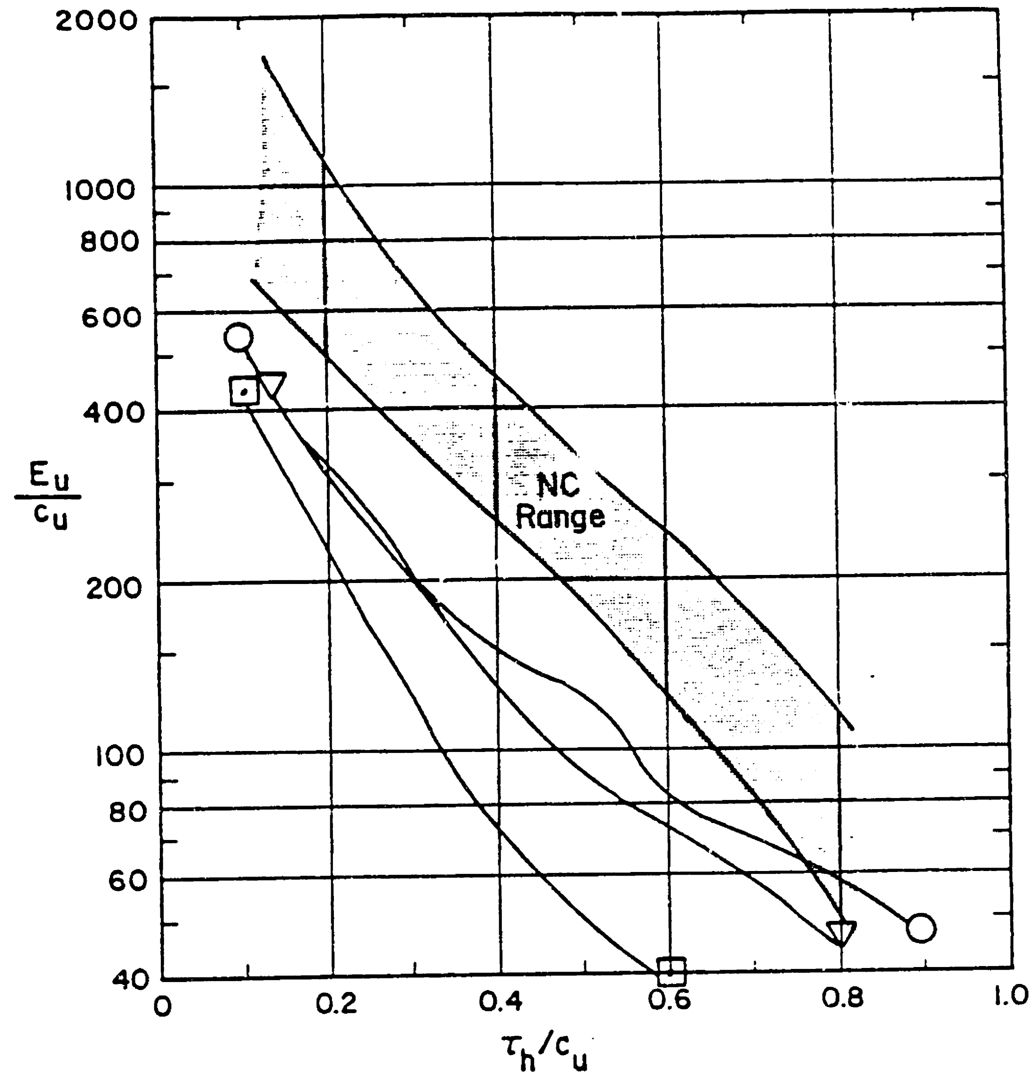


Figure 6-21: Normalized Stress Paths for Overconsolidated CK_0UDSS Tests at Site T



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (ksf)	σ'_p (ksf)	OCR	Symbol
TDSS2	3B-P3	1.7	40.2	0.631	5.0	7.9	○
TDSS5	3B-P2	4.6	37.2	0.588	5.0	8.5	▽
TDSS8	5B1-P3 (pins)	7.1	38.3	0.313	4.93	15.75	⊡

Figure 6-22: Normalized Stress vs. Strain for Overconsolidated CK_0UDSS Tests at Site T



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (ksf)	σ'_p (ksf)	OCR	Symbol
TDSS2	3B-P3	1.7	40.2	0.631	5.0	7.9	○
TDSS5	3B-P2	4.6	37.2	0.588	5.0	8.5	▽
TDSS8	5B1-P3 (pins)	7.1	38.3	0.313	4.93	15.75	⊡

Figure 6-23: Normalized Modulus for Overconsolidated CK_0UDSS Tests at Site T

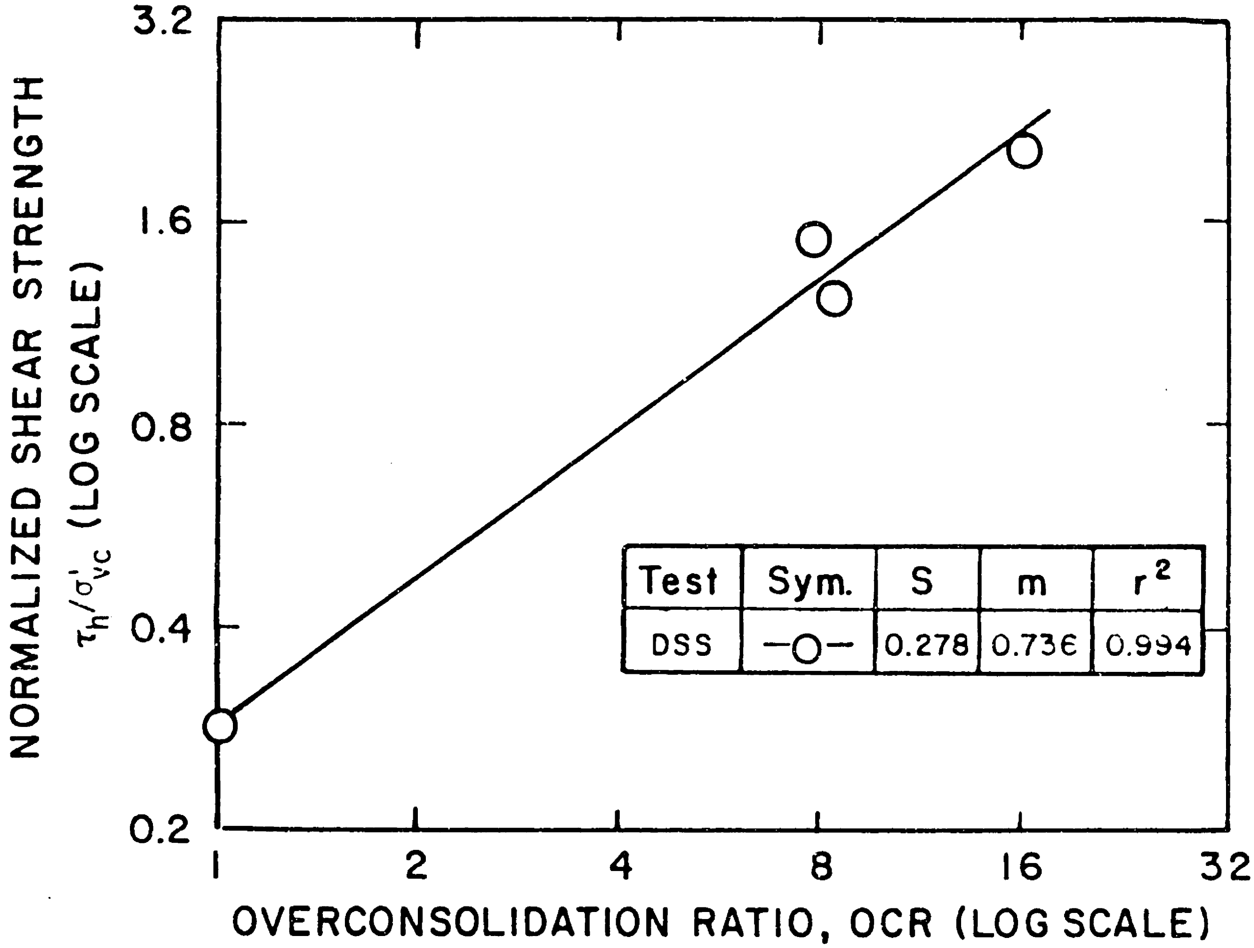


Figure 6-24: Normalized Shear Strength vs. OCR from CK_0UDSS Tests at Site T

Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	σ'_p (KSF)	OCR	Symbol
WDSS7	W5B-P4	9.5	29.4	4.92	24.6	5.0	○
WDSS8	W5B-P4	9.6	29.0	2.44	24.6	10.1	□
WDSS9	W5B-P4 (pins)	9.7	28.9	1.22	24.6	20.2	△

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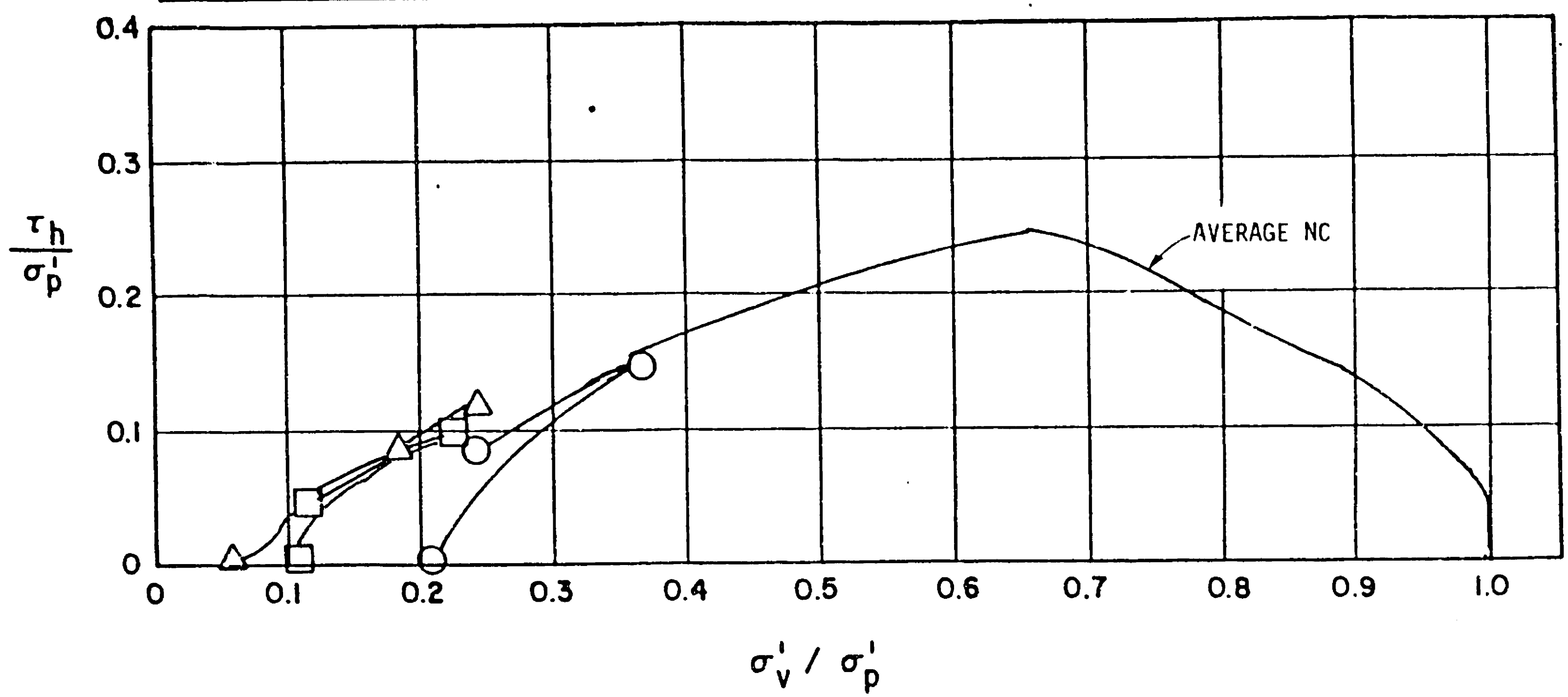
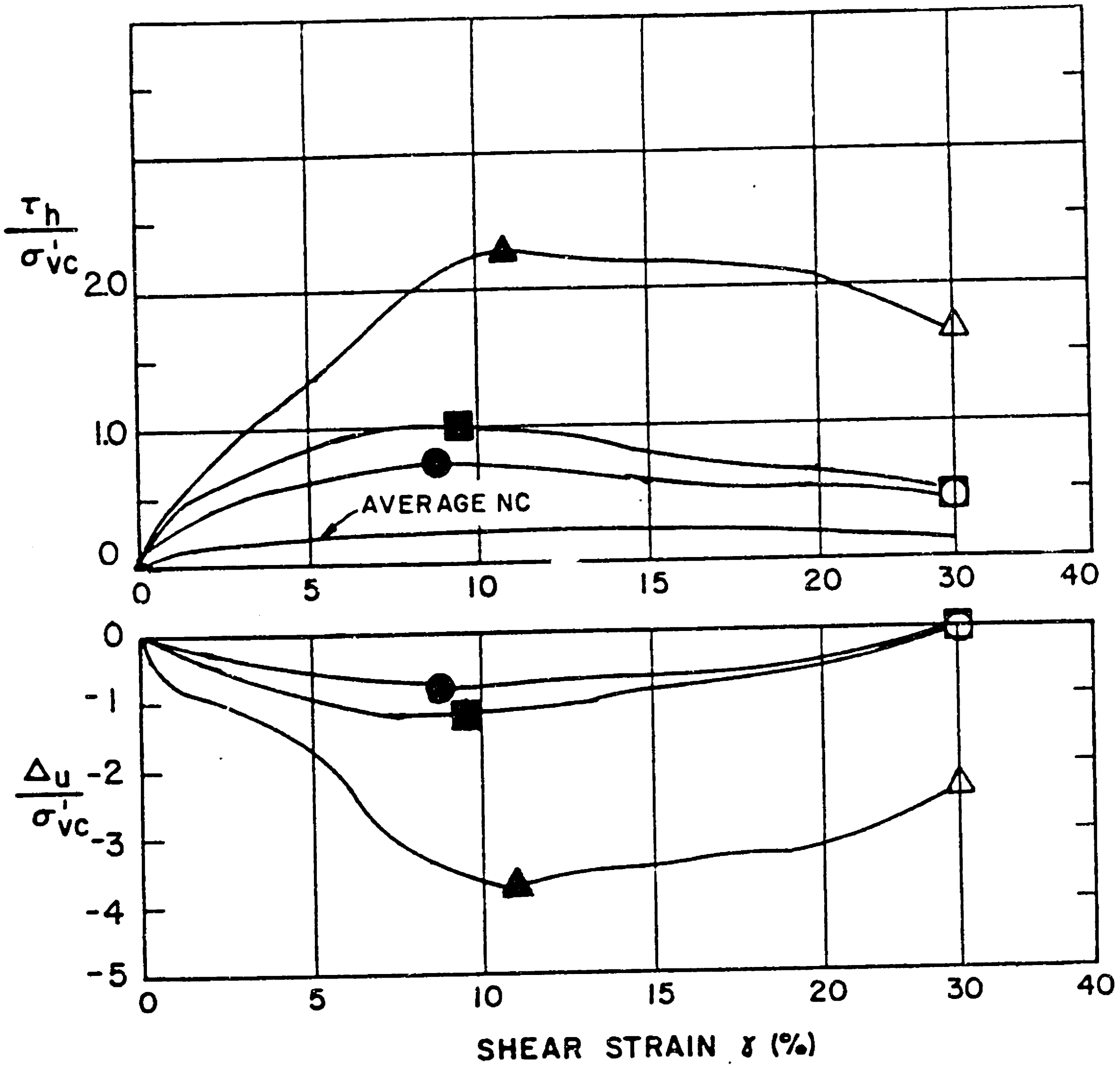
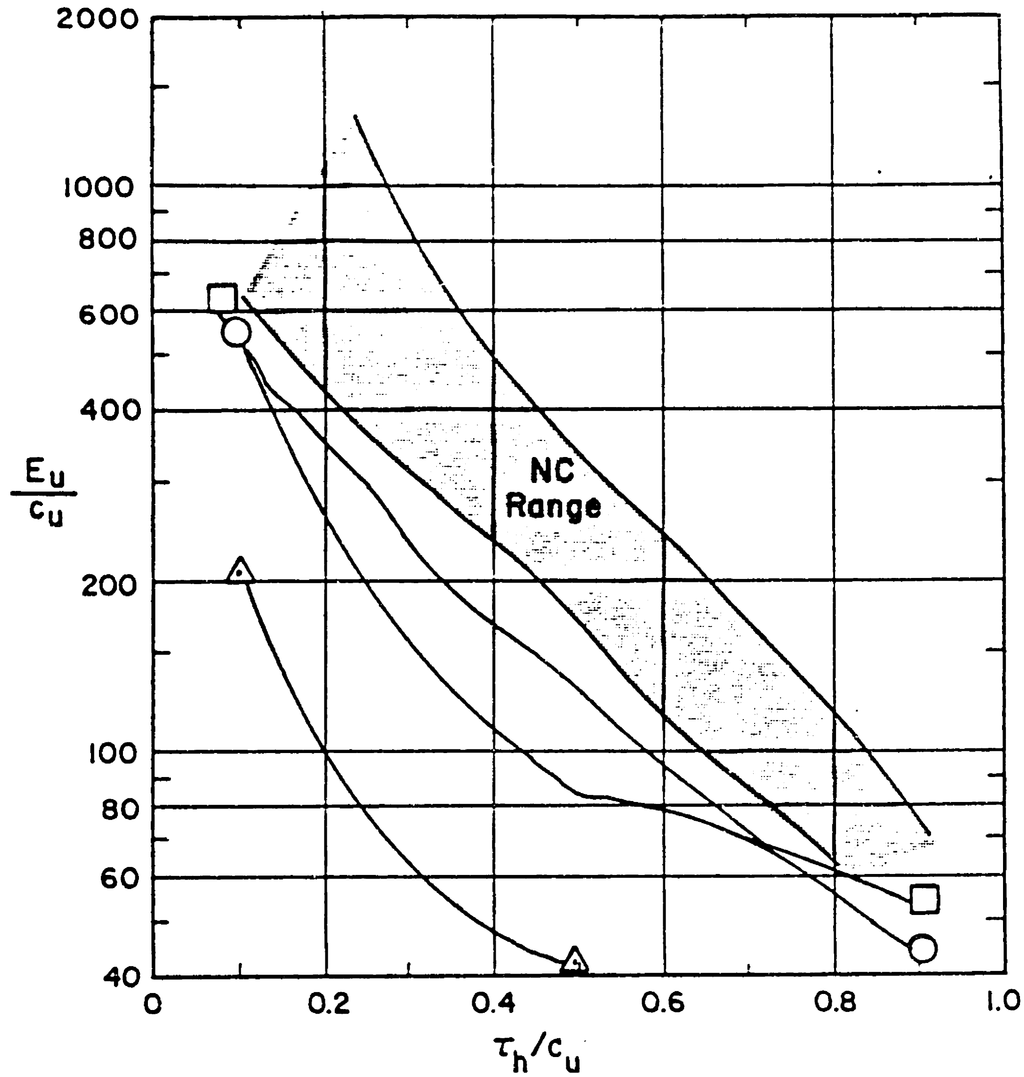


Figure 6-25: Normalized Stress Paths for Overconsolidated CK_0UDSS Tests at Site W



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	σ'_p (KSF)	OCR	Symbol
WDSS7	W5B-P4	9.5	29.4	4.92	24.6	5.0	○
WDSS8	W5B-P4	9.6	29.0	2.44	24.6	10.1	□
WDSS9	W5B-P4 (pins)	9.7	28.9	1.22	24.6	20.2	△

Figure 6-26: Normalized Stress vs. Strain for Overconsolidated CK_0UDSS Tests at Site W



Test No.	Sample No.	RE (FT)	w_N (%)	σ'_{vc} (KSF)	σ'_p (KSF)	OCR	Symbol
WDSS7	W5B-P4	9.5	29.4	4.92	24.6	5.0	○
WDSS8	W5B-P4	9.6	29.0	2.44	24.6	10.1	□
WDSS9	W5B-P4 (pins)	9.7	28.9	1.22	24.6	20.2	△

Figure 6-27: Normalized Modulus for Overconsolidated CK_0UDSS Tests at Site W

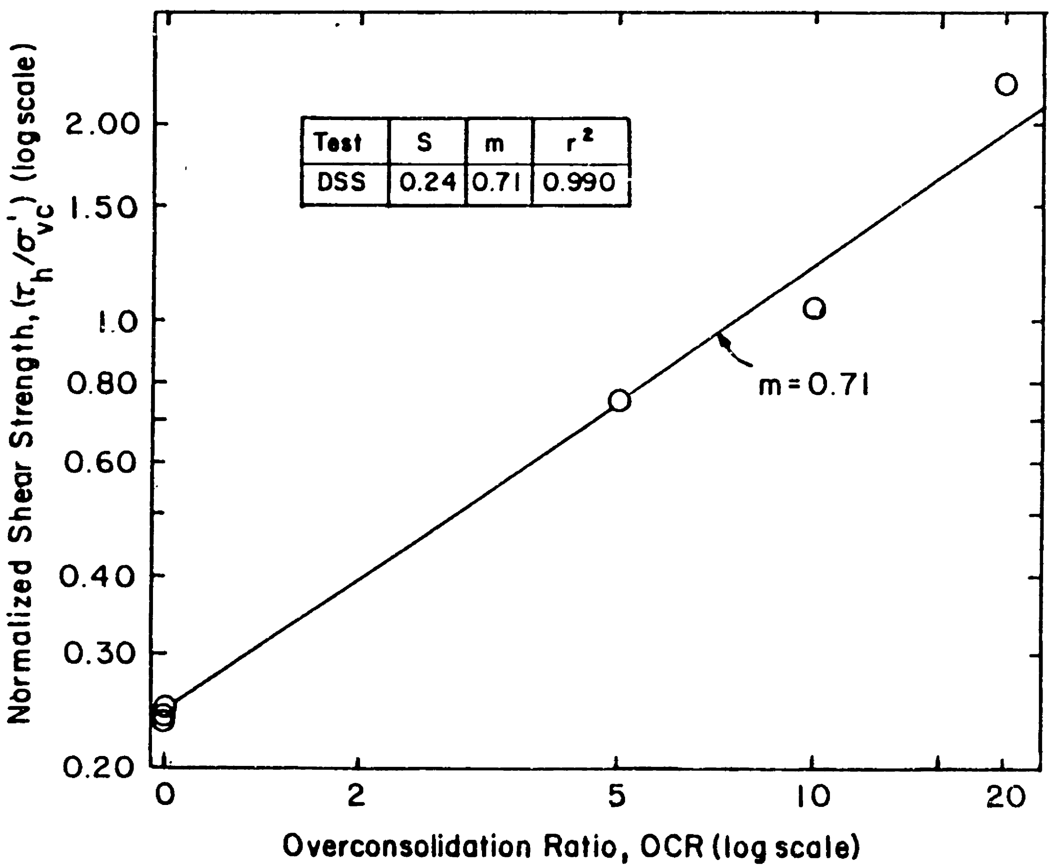


Figure 6-28: Normalized Shear Strength vs. OCR from CK_0UDSS Tests at Site W

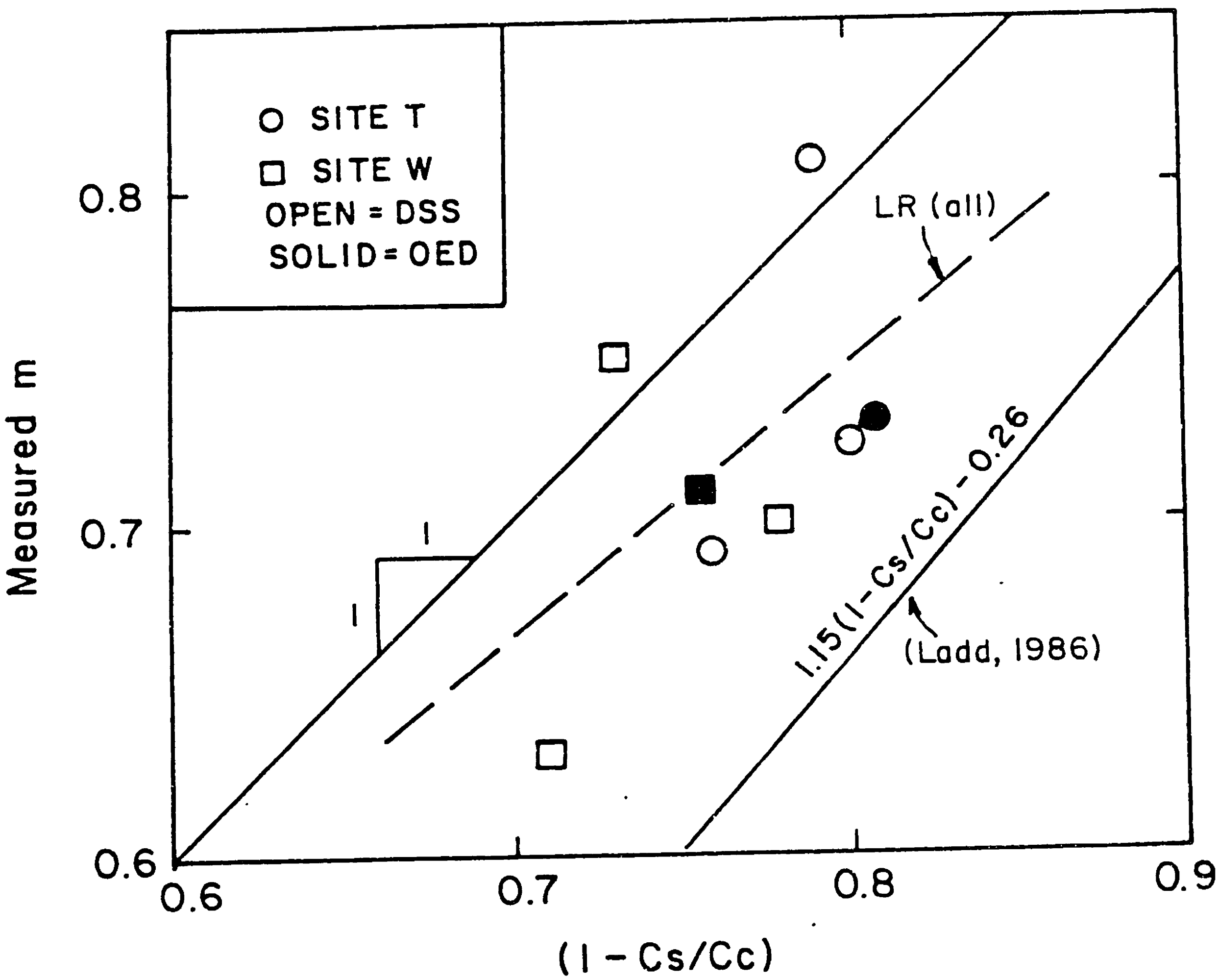


Figure 6-29: $m_{measured}$ versus $(1-C_s/C_c)$

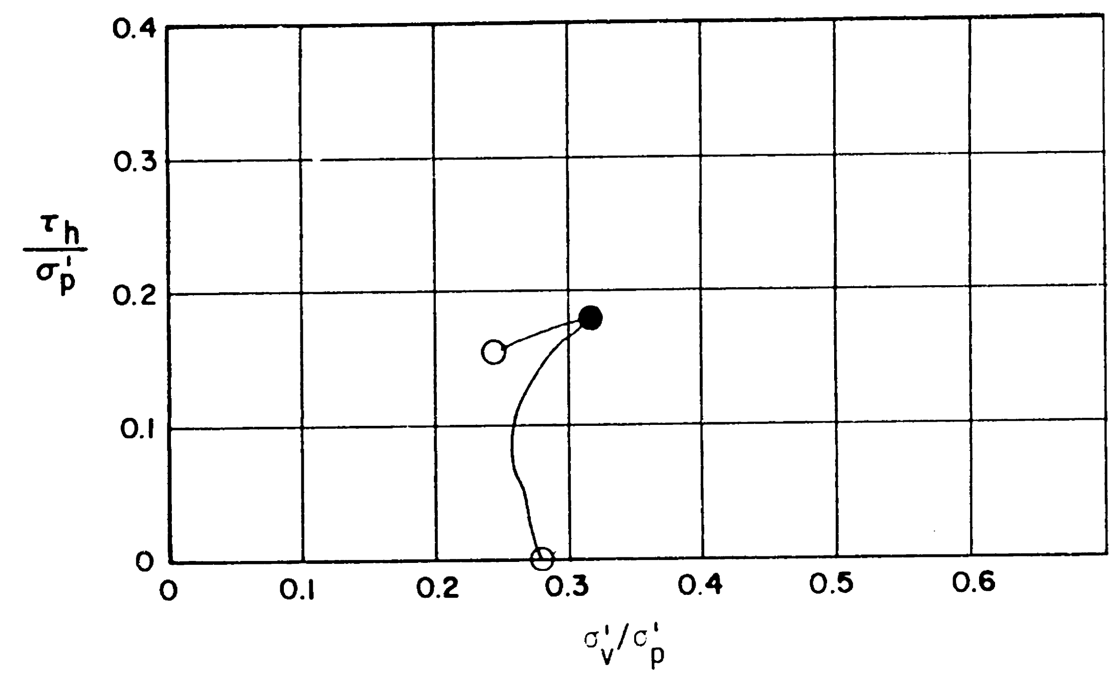


Figure 6-30: Normalized Stress Path for Recompression Test at Site T

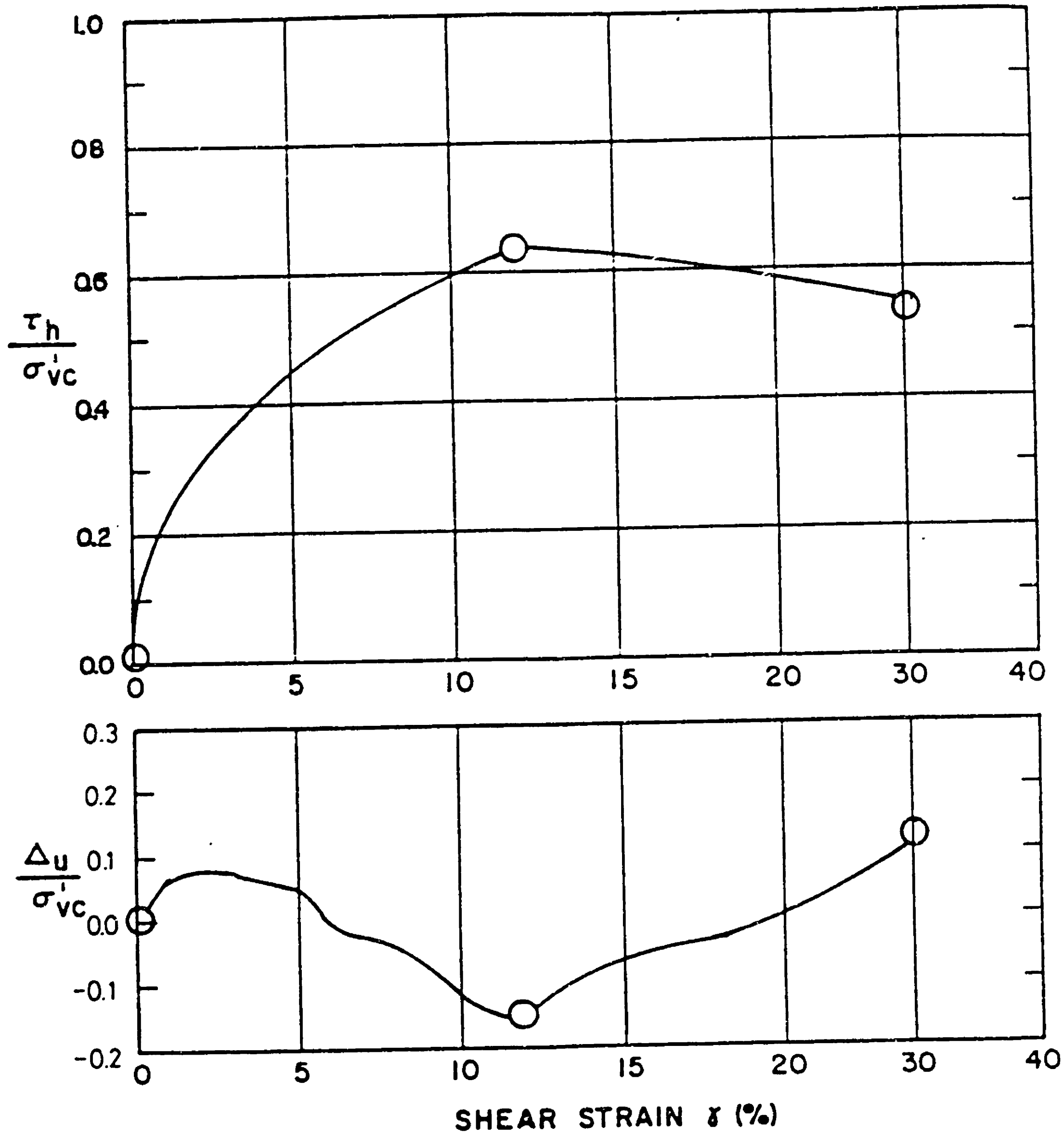


Figure 6-31: Normalized Stress vs. Strain for Recompression Test at Site T

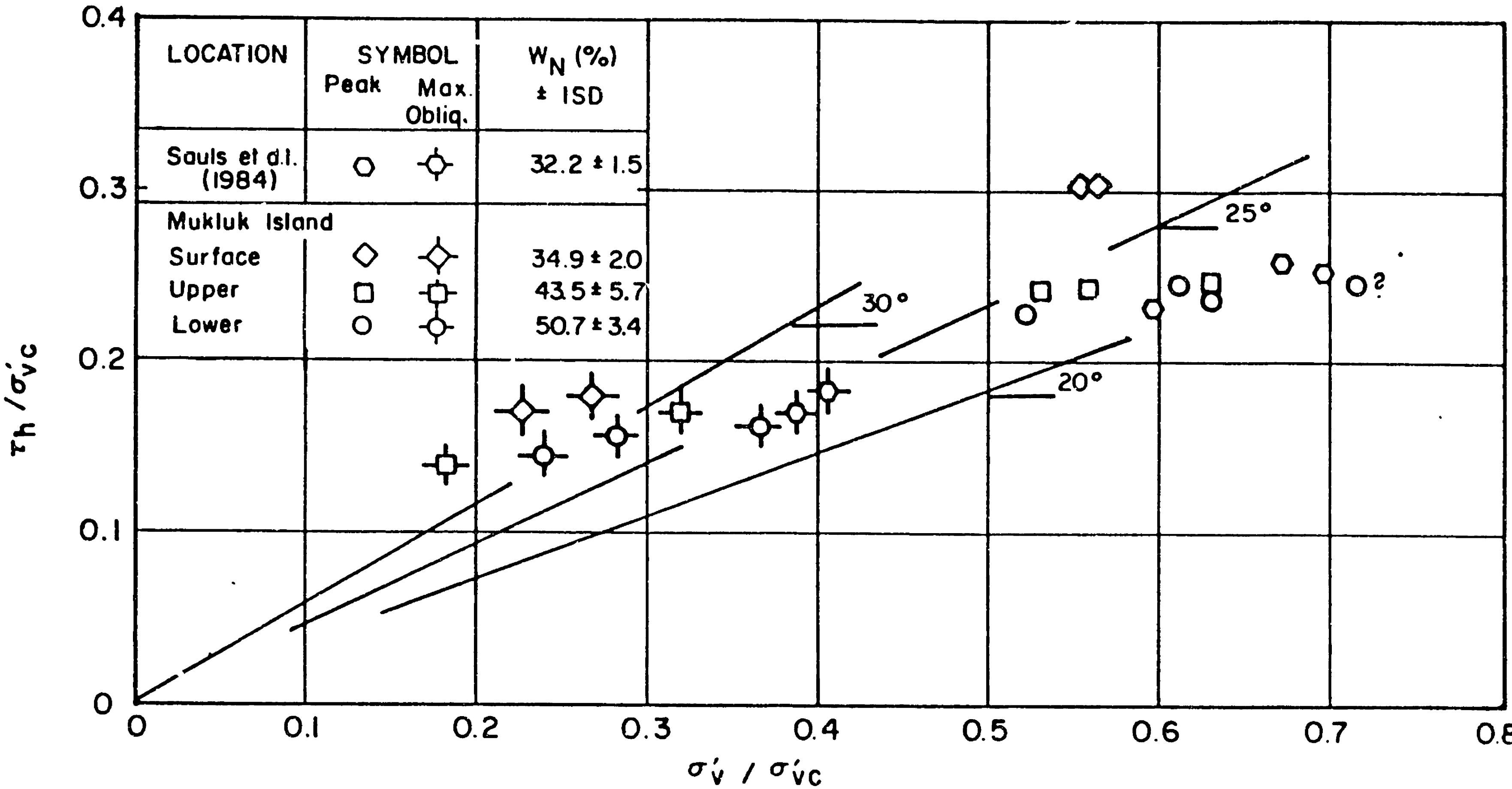


Figure 6-32: Peak and Maximum Obliquity of Effective Stress Paths from Normally Consolidated CK_0UDSS Tests on Arctic Silts from Mukluk Proximal, Harrison Bay (Yin,1985)

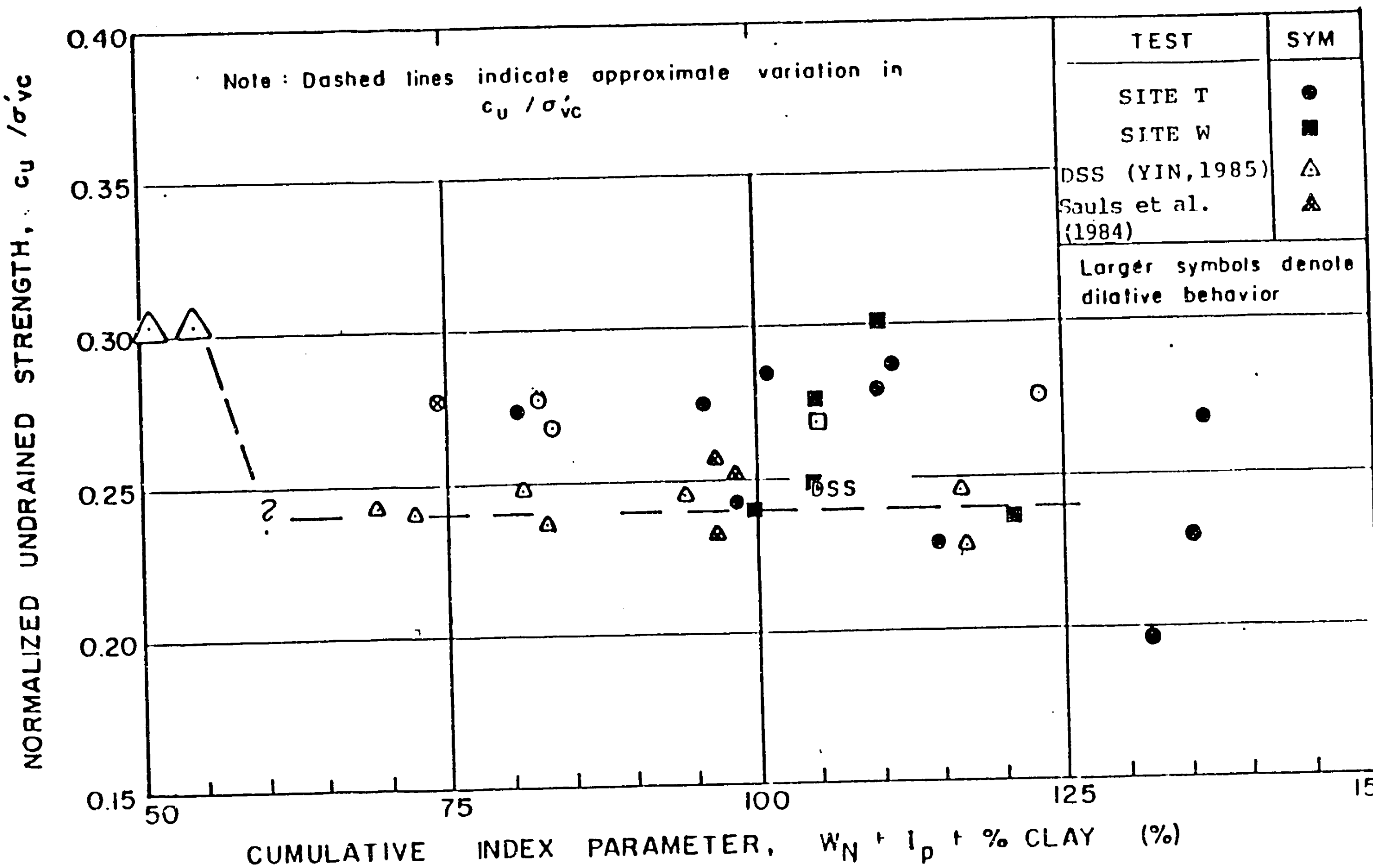


Figure 6-33: Normalized Shear Strength versus Cumulative Index Parameter (CIP)

Chapter 7

EVALUATION OF UNDRAINED STRENGTH PROFILES

7.1 INTRODUCTION

All available data support differentiation of the two Smith Bay sites, described in detail in Chapter 3. Site W is nongouged, with a thin layer of Holocene material overlying stiff, overconsolidated silty clay. Site T, however, has experienced intense ice gouging, resulting in reworking and mixing of Holocene sediment with the underlying Pleistocene deposits, so that no clear boundary between the two can be defined. The highly gouged zone at Site T extends to a depth of about eight to ten feet. The lower soil at Site T resembles the Pleistocene material of Site W, with comparable index properties, preconsolidation pressure, and strength index values.

The preceding chapters have analyzed results from the oedometer and triaxial compression and DSS strength testing programs. The primary purpose of this chapter is to synthesize these data to obtain a better overall understanding of the undrained SHANSEP behavior of Smith Bay deposits. The results of the strength testing program conducted by MIT will also be compared with results from conventional strength tests (e.g. field vane and laboratory UUC) and also to the Recompression tests.

7.2 SELECTION OF SHANSEP UNDRAINED STRENGTH PARAMETERS

The SHANSEP strength profiles for Sites T and W are developed based on strength parameters obtained from the triaxial compression and DSS test programs conducted by MIT (Chapters 5 and 6, respectively). The selected values of S and m were used in the relationship:

$$c_u/\sigma'_{vc} = S\,(OCR)^m \qquad \text{Eq. 7.1}$$

where $c_u = \tau_h max$ for DSS tests and q_f for triaxial tests. OCR values used in Equation 7.1 are based on the selected stress history profiles developed in Chapter 4 (Figures 4-9 and 4-11).

Table 7-1 summarizes the selected values of SHANSEP undrained strength parameters for Sites T and W. Values of S and m for the DSS mode of shear were obtained from the testing programs performed at both sites. DSS tests run on gouged material at Site T gave values of S = 0.28 and m = 0.73. These values are based on DSS tests consolidated to a maximum stress of 5 ksf. A minimum value of S = 0.24 is also presented, recognizing that the value of S decreases with increasing consolidation stress. These values are judged appropriate for use in the upper eight feet of the deposit. No SHANSEP type tests were run on the lower, nongouged material at Site T. Therefore, estimates of S and m for RE greater than 8 ft are based on results from Site W DSS tests, giving S = 0.24 and m = 0.71. Values of S = 0.28 and m=0.73 were selected as maximum values based on the results of DSS tests in the upper material at Site T.

DSS tests were performed over a range of depths at Site W. Linear regression of the results gave m = 0.71 for S = 0.24. These parameters correspond to low (30%) water content specimens consolidated to a stress of 24.6 ksf and high water content samples consolidated to a lower stress. These values are considered appropriate for the upper 20 feet of Site W material.

The DSS values of m lie at the lower end of the 0.8 $\pm$ 0.1 range quoted by Ladd et al. (1977) as being typical for CK_0UDSS tests on sedimentary clays. However, values of m at Smith Bay are consistent with each other, as shown in Figure 6-29. A consistent relationship of decreased measured m with increasing C_s/C_c was also found for these tests, in agreement with a similar trend found for five low plasticity clays. Since C_s/C_c is higher than usual for the Smith Bay deposits, the lower m is considered both reasonable and reliable.

The triaxial compression values of S = 0.32 and m = 0.76 were selected based on the four $CIUC$-CK_0UC tests perormed at Site T (see Section 5.4). No SHANSEP type triaxial tests were available on Site W material; however, results of the Site T testing program were judged reasonable for use at Site W since S for triaxial compression tests appears to be fairly constant independent of soil type (Jamiolkowski et al., 1985).

7.3 OVERVIEW OF CONVENTIONAL UNDRAINED STRENGTH INDEX DATA

Figures 7-1 and 7-2 plot undrained strength versus relative elevation for Sites T and W, respectively. The data are from Field Vane, UUC, and miniature vane and Recompression triaxial compression and DSS tests. As discussed in Section 3.3.3, the field vane, UUC and miniature vane strengths were fairly consistent and hence are selected as being representative of the "conventional" test program conducted at Smith Bay. In cases where Recompression test specimens were consolidated to stresses greater than the in situ vertical effective stresses, strengths were corrected to the in situ stress using the relationship:

$$\frac{\text{corrected } c_u}{\text{measured } c_u} = \left(\frac{\sigma'_{vo}}{\sigma'_{vc}}\right)^{1-m}$$

which is derived from Eq. 7.1. This correction used the SHANSEP values of m presented in Table 7-1.

The conventional test strengths (UUC, FV, MV) at Site T are very low in the upper 8 to 9 feet of the deposit, with a large increase in strength at greater depths. The scatter in the data is likely the result of variable ice gouging. Conventional tests at Site W show consistently high strengths throughout the deposit, except for the upper few feet of Holocene material. Linear regression lines for UUC-MV-FV values are plotted for RE $\leq$ 9 ft and $c_u \leq$ 1.5 ksf at Site T and ignoring the Holocene clay at Site W.

The TETC UUC tests at both sites were run on tubes having an oedometer test. Analysis of these data give: $c_u(\mathrm{UUC})/\sigma'_p$ = 0.28 $\pm$ 0.092 SD at Site T for RE $\leq$ 9 ft and 0.332 $\pm$ 0.076 SD at Site W. These ratios are very high for such heavily overconsolidated deposits and also much larger than c_u/σ'_p backcalculated from case histories of loading failures on sedimentary clays.

7.4 EVALUATION AND COMPARISON OF UNDRAINED STRENGTHS FROM TRIAXIAL TESTING

7.4.1 Site T

UUC and Recompression triaxial compression tests agree fairly well with depth (Figure 7-1). Figure 7-3 plots natural (preshear) water content versus strength (q_f) for the UUC and TC (corrected) tests. All UUC tests were performed by TETC, with water contents taken from tabulated data provided to MIT. Of the 6 Recompression TC tests, 4 were performed by MIT and 2 by TETC. Linear regression on these data (but excluding the low Recompression strength at w_N = 40.3%) gave the line shown in Figure 7-3 with r^2 = 0.78. Both the Recompression and UUC tests fit this line quite well. Hence, the increase in measured strength resulting from the high strain rate used in UUC testing must therefore balance the decrease in strength due to sample disturbance.

Table 7-2 compares the Recompression strengths with predictions based on the SHANSEP parameters. For five of the six tests SHANSEP predicts significantly lower strengths of 0.71 $\pm$ 0.10 times the Recompression values. These differences could be due to the following:

(1) The much lower strain rate used for the SHANSEP tests since strain rate effects are known to be especially important in triaxial testing of highly overconsolidated samples (e.g. Lacasse, 1979).

(2) The values of S and/or m used for the SHANSEP predictions are too low. But agreement requires S = 0.32/0.7 = 0.46, or m = 0.9, both of which appear very high.

(3) The values of σ'_p used for the SHANSEP predictions are too low. Although there were problems estimating σ'_p, there is no reason to expect the values to be systematically low.

The writer does not know which of the above explanations is most reasonable. Moreover, the difference may be caused by more than one factor.

On the other hand, the Chapter 5 SHANSEP CIUC and CK_0UC tests give water content versus log strength values which plot to the right of the Site T linear regression line in Figure 7-3 (w_f = 28.1% $\pm$ 1.4 and q_f = 2.7 ksf $\pm$ 0.5). In other words, SHANSEP predicts higher strengths at the same water content. This finding contradicts the above comparison based on stress history and cannot be explained.

7.4.2 Site W

Figure 7-3 shows the Site W water content versus log strength data from UUC testing and the resulting linear regression line. Its location compared to the Site T relationship can be reasonably explained by the fact that remolding (ice gouging) produces a decreases in undrained strength at constant water content.

The Recompression TC strengths are consistently less than the UUC values at a given RE (Figure 7-2). The plot of water content versus log c_u (Figure 7-3) also shows that three of the four tests give much lower strengths than UUC tests at the same water content. These low strengths are probably the result of poor testing. Effective stress path data for the Recompression tests show very low failure values of $p' = (\sigma_1' + \sigma_3')/2$), much less than expected for a highly overconsolidated deposit. Three of the four Site W tests can thus be logically discounted due to poor quality test procedures.

The Recompression triaxial compression results are compared with SHANSEP predictions in Table 7-3. Three of the four measured strengths are low compared to SHANSEP predictions due to the poor test quality. The ratio of predicted to measured strengths for all tests equals 1.4 ± 0.4 SD.

The SHANSEP predicted TC c_u values in Table 7-3 are much less than the UUC results. The UUC strengths are believed too high due mainly to the strain rate effect, which is especially important at very high OCR.

7.5 EVALUATION AND COMPARISON OF UNDRAINED STRENGTH FOR DSS TESTS

Figure 7-4 plots water content versus log undrained strength (= τ_h) for all DSS tests performed at Smith Bay. Water contents for the Recompression tests are the natural water contents of the samples. For the SHANSEP type tests, a preshear water content was calculated from the final water content, taken after the specimen was allowed to swell for at least 24 hours after shearing. The preshear water content was backcalculated from this value by assuming that the change in water content was directly related to the change in height of the sample. Preshear water contents calculated from the initial water contents of trimmings and consolidation strains were generally higher, probably because samples were not 100% saturated initially. Values calculated using both methods are included in Appendix E.

DSS tests show a general trend of increased strength (τ_h) with decreasing water content. The range of values is the result of the wide variation in the initial water contents of samples tested, including both gouged and nongouged material. Site T results are especially scattered, but seem to converge with Site W values at low water contents. This is reasonable, considering that the sites consist of the same material, but subjected to gouging at Site T. Results are further discussed by site.

7.5.1 Site T

The data in Figure 7-4 for Site T corresponds to a wide range in initial w_N (= 35 to 54% from Chapter 6) as a result of ice gouging. This results in a large amount of scatter of Site T strengths as well. Test TDSS3 (w = 42%) had an unusually high initial water content of 54.4%, and plots well above the other tests.

Test TDSS1B was a second shear, possibly accounting for its high backcalculated water content of 40% which is greater than the water content of trimmings. But even excluding these two tests, the scatter is still too large to develop a meaningful water content versus log strength relationship.

Table 7-2 compares Recompression DSS strengths with SHANSEP predictions based on parameters derived from the test program at Site T. The MIT Recompression test strength is very low, perhaps due to a marginal quality sample and added disturbance associated with the use of stones with pins. The two TETC tests show predicted to measured ratios of 0.95 $\pm$ 0.2 SD. Although based on very limited data, the comparison does tend to support the validity of the stress history profile developed in Chapter 4 and the values of S and m selected in Chapter 6.

7.5.2 Site W

Most Site W tests had a narrow range of initial water contents. Hence, preshear water content versus log c_u data in Figure 7-4 are more consistent. The linear regression line on the SHANSEP data is flat, i.e. a large change in strength for small changes in water content.

Recompression tests at Site W are compared with SHANSEP predictions in Table 7-3. SHANSEP predictions are about 1.7 $\pm$ 0.6 SD times the Recompression values, with the ratio of predicted to measured strength decreasing from 2.34 to 1.08 over RE from 2 to 8 ft, with a corresponding decrease in OCR from 192 to 19. Two of the three Recompression tests agreee with the trend established by the SHANSEP tests in Figure 7-4. One test gave an unusually low strength for a low water content sample (w = 24%). High quality Recompression tests may be especially difficult to perform at shallow depths, where the OCR values rise to over 100 for the tests.

7.6 DESIGN STRENGTH PROFILES FOR BASE SLIDING

7.6.1 Methodology based on SHANSEP

The critical foundation design condition for Arctic gravity structures occurs from ice loadings. Resistance to base sliding is best modeled via the DSS mode of failure. A representative strength profile is constructed using the SHANSEP parameters for the DSS described in Section 7.2. Stress history profiles develped in Chapter 4 were used to estimate σ'_p for use in calculating strengths. Although Site T material did not exhibit normalized behavior, SHANSEP did predict reasonable strengths compared to limited Recompression DSS data. Site W material had normalized behavior; lack of agreement between SHANSEP and c_u(DSS) values is attributed to problems with the latter test program. Results are discussed in detail by site.

7.6.2 Undrained Strength Profile at Site T

Figure 7-5 presents the average SHANSEP c_u(DSS) profile calculated from Equation 7.1 using the S and m parameters shown in Table 7-1. Calculations at representative depths are shown in Table 7-4. Values of σ'_{vo} were obtained from Figure 4-9, which also shows the available σ'_p data. For the upper seven feet of Site T, σ'_p was taken from a linear regression of results from six oedometer compression curves. At depths of 8, 11, and 13.8 feet σ'_p was estimated based on the statistical mean of values clustered at those depths, with linear interpolation between them. Room temperature σ'_p values were adjusted to 1 °C. The SHANSEP strength was calculated at each of those depths and linear interpolation was performed between those values to complete the strength profile.

The UUC, FV and MV data from Figure 7-1 and its linear regression line are plotted for comparison with the selected SHANSEP c_u(DSS) profile. Also plotted are the results of Recompression TC and DSS tests. Conventional strength test results are about 1.6 times greater than SHANSEP c_u(DSS) values in the upper highly gouged zone. Less conventional test data are available for comparison below RE = 9 ft, but values are on the order of 2.2 to 3.4 times greater than the SHANSEP predicted strengths.

The SHANSEP method gave strengths significantly less than measured in conventional tests for a number of reasons discussed in Section 7.6.4. These include: questions regarding applicability of empirical correlations; strain rate effects; and anisotropy effects.

7.6.3 Site W

A profile of the average SHANSEP c_u(DSS) for Site W is shown in Figure 7-6. This profile was obtained using similar means as for Site T. Values of σ'_{vo} and σ'_p were obtained from Figure 4-11. For the upper 20 ft of the deposit, σ'_p was taken from linear regression of results of 11 oedometer and three DSS compression curves. The MIT σ'_p value was adjusted to 1 °C. SHANSEP parameters were used to calculate c_u(DSS) at several depths with interpolation between plotted points (Table 7-4). Strengths are plotted $\pm$1 SD due to uncertainty in σ'_p. The use of linear σ'_{vo} and σ'_p curves yields a smooth strength curve, unlike that developed for Site T.

The Recompression TC/DSS tests are also plotted versus RE, but are not considered representative due to poor testing procedures. Linear regression of conventional test results is shown for comparison. The ratio of c_u(DSS)/c_u(UUC-FV-MV) is equal to 0.2 at RE of 1 ft, and increases to 0.4 at RE of 20 ft. Hence conventional tests overestimate strengths appropriate for design by 2.5 to 5 times. SHANSEP predictions are greater than Recompression strengths due to poor testing of the latter.

7.6.4 Conventional Tests versus SHANSEP Predictions

In situ vane tests require empirical correlations to obtain strength values appropriate for design which may not be applicable for Arctic silt (see Section 2.4). Field vane tests also may not be truly undrained. UUC values rely heavily on compensating errors (see also Section 2.4). Sample disturbance leads to a decrease in the measured c_u. However, the high strain rate at which the test is performed results in overestimates of strength. The effects of strain rate are especially important at high OCR. For UUC tests on Arctic silt at 1 °C, effects may be even larger, i.e more "structural viscosity". UUC tests also ignore the effects of anisotropy, which further increases the measured strength. Data from CK_0U TC, DSS and TE tests at OCR = 1 show significant anisotropy for cohesive soils of low to moderate plasticity (Jamiolkowski et al., 1985). Three clays of moderate-high OCR and with $I_p = 20 \pm 5\%$ yielded measured $q_f(TC)/\tau_h(DSS)$ of about 2 ± 0.5 (Ladd, 1985). Hence Smith Bay deposits would be expected to exhibit significant anisotropy at their in situ OCR. Measured UUC strengths certainly should not be used to determine the resistance to horizontal sliding at these sites.

7.7 SUMMARY OF SMITH BAY RESULTS

Ice gouging can have a significant effect on the in situ strength of Arctic Silts. The gouged Site T experienced progressive reduction in preconsolidation pressure approaching the mudline; the transition from gouged to nongouged is also marked by changes in index properties. The effects on the strength profile are very clear; low strengths are associated with the gouged, reworked material, with the intact material possessing higher strengths. Site W has uniform soil conditions, a well-defined stress history showing the deposit to be heavily overconsolidated and much higher strengths at shallow depths.

Results of conventional tests at Site W (UUC, lab vane and field vane) yield strengths larger than reasonable for design by several times. Site T, although exhibiting effects of ice gouging, also gives UUC, lab vane and field vane strengths greater than those determined by SHANSEP. The ratio of measured conventional strengths to predicted SHANSEP c_u(DSS) appears to increase with increasing OCR based on the collective results. The overprediction by UUC tests at both sites can be explained by the effects of strain rate and anisotropy.

Results of Recompression TC and DSS test programs are reliable only if the tests are of high quality. Poor quality Recompression tests at Site W resulted in low measured strengths compared to SHANSEP predictions. At Site T, however, SHANSEP predictions are less than or equal to the values from Recompression tests. Although there is some uncertainty in the estimates of S and m values, the resultant error is probably small compared to the scatter in σ'_p data at Site T.

7.8 COMPARISON WITH RESULTS FROM HARRISON BAY

Figure 7-7 shows the c_u(DSS) profile developed for Harrison Bay Arctic silts at Mukluk Island, calculated using the SHANSEP parameters shown. Values of preconsolidation pressure were taken from the corresponding profiles of σ'_p and σ'_{vo} versus depth. Details of how this strength profile was obtained are found in Ayan (1985) and Yin (1985).

The stress history and the change in S at z = 6 ft results in the four zones plotted. Strength values were obtained by interpolation where data were lacking to produce a "best estimate" of the complete c_u profile. The interpolation was based on knowledge of the soil properties.

The overall picture given by this profile for Arctic silt next to Mukluk Island is of a highly overconsolidated upper zone, with several sublayers, overlying much weaker material. It is this bottom soft cohesive soil from which the Soft Zone Area draws its name.

The strength profile developed for Mukluk Island is very different from either of those developed for Smith Bay. These differences are the result of the variable geologic environment in the Beaufort Sea. Mukluk Island results characterize a soft zone of Holocene material deposited very rapidly. The mechanism causing the overconsolidated crust is not fully explained, but may be the result of freeze-thaw mechanisms following deposition.

Comparison of conventional c_u data versus c_u(DSS) at Harrison Bay yielded a far different relationship than that observed at Smith Bay. Mean UUC and SHANSEP profiles agreed quite well, but with very high scatter compared to Site W or even Site T. Field vane measurements at Harrison Bay were two times greater than the UUC results. This contrasts the situation at Smith Bay, where UUC and field vane results were in close agreement, but 1.6 to 5 times greater than the SHANSEP predicted c_u(DSS).

Table 7-1: SELECTED SHANSEP UNDRAINED STRENGTH PARAMETERS FOR SITES T AND W

Site	RE (ft.)	Mode of Shear	S	m	Remarks
T	2-14	TC q_f	0.32	0.76	From CIUC @ OCR=1, 2, 5
	<8 (gouged)	DSS τ_h	0.28	0.73	From DSS @ OCR=1, 8, 16
			0.24	0.73	Minimum values
	>8 (nongouged)	DSS τ_h	0.24	0.71	Best estimate
			0.28	0.73	Max. value
W	1-20	TC q_f	0.32	0.76	From Site T test program
		DSS τ_h	0.24	0.71	From DSS @ OCR=1, 5, 10, 20

Table 7-2: COMPARISON OF SHANSEP vs. RECOMPRESSION FOR SITE T

All Stresses in ksf

Test Type	Test No. (Run By)	Boring RE (ft.)	w_N (%)	σ'_{vc} / σ'_{vo}	C_u*	Comp. Test	RE (ft.)	w_N (%)	σ'_p	OCR	Pred.(1) C_u	$\frac{\text{Pred.}}{\text{Meas.}}\ \frac{C_u}{C_u}$
Triaxial Comp.	T5B1T1 (M)	5B1 1.85	40.3	0.11 0.102	0.162	T2	1.75	46.3	1.40 ±0.07	13.7	0.24	1.44
	T5B1T2 (M)	5B1 8.24	33.8	0.69 0.45	0.96	T15	8.0	39.6	3.5 ±0.28	7.77	0.69	0.72
	T5B1T3 (M)	5B1 10.7	21.6	1.58 0.59	2.27	T18	11.0	23.3	7.5 ±1.3	12.7	1.30	0.58
	T5B1T6 (M)	5B1 10.8	22.1	0.717 0.59	1.82	T18	11.0	23.3	7.5 ±1.3	12.6	1.30	0.72
	T5B-P2 (E)	5B 4.3	36.4	1.24 0.18	0.52	Estimate from Linear Reg.			1.9	9.9	0.35	0.67
	T5B1-P3 (E)	5B1 8.5	34.9	0.43 0.47	0.85	T15	8.0	39.6	3.5 ±0.28	7.5	0.69	0.84
DSS	T5B-P3 (E)	3B 5.1	35.2	0.29 0.25	0.54	T9	5.1	35.3	3.0 ±0.21	12.0	0.43	0.79
	T5B-P3 (E)	5B 7.1	23.8	0.37 0.34	0.71	T3	7.7	25.4	6.1 ±0.6	17.9	0.78	1.10
	TB3DSS1 (pins) (M)	B3 7.5	37.5 45.1 (no sand)	0.39 0.39	0.25	T13	7.6	51.7	1.7 ±0.12	4.4	0.32	1.29

* Recompression strengths corrected to σ'_{vo}
$C_u = q_f$ for triaxial tests
$C_u = \tau_h$ for DSS tests

(1) For triaxial tests: S=0.32, m=0.76
(2) For DSS tests: S=0.28, m-0.73

Table 7-3: COMPARISON OF SHANSEP vs. RECOMPRESSION FOR SITE W

All Stresses in ksf

Test Type	Test No.	Boring RE (ft.)	w_N (%)	'vc 'vo	C_u*	Comp. Test	RE (ft.)	w_N (%)	'p	OCR	Pred.(1) C_u	Pred. C_u / Meas. C_u
Triaxial Comp.	W3B-P1	3B 1.9	35.6	0.432 0.039	0.368	W1	2.25	25.2	7.5 ·0.5	192.3	0.679	1.85
	W3B-P3	3B 5.9	31.8	0.432 0.259	0.924	W3	6.8	33.5	9.0 +0.5	34.75	1.23	1.33
	W5B-P1	5B 1.65	41.5	0.432 0.091	0.544	From Linear Regression			7.3	80.2	0.816	1.50
	W5B-P3	5B 7.6	32.2	0.432 0.418	1.569	W8	8.1	31.9	8.4 ±0.6	20.1	1.308	0.834
DSS	W3B-P1	3B 2.25	23.6	0.072 0.058	0.188	W1	2.25	25.2	7.5 ±0.5	129.3	0.439	2.34
	W3B-P3	3B 6.8	29.5	0.317 0.308	0.487	W3	6.8	33.5	9.0 ±0.5	29.2	0.812	1.67
	W5B-P3	3B 8.15	29.8	1.44 0.448	0.798	W8	8.1	31.9	8.4 ±0.6	18.75	0.862	1.08

* Recompression strengths corrected to vo'
$C_u = q_f$ for triaxial tests
$C_u = {}_h$ for DSS tests

(1) For triaxial tests: S=0.32, m=0.76
(2) For DSS tests: S=0.24, m-0.71

Table 7-4: CALCULATIONS FOR SHANSEP STRENGTH PROFILE

All Streses in ksf

Site	RE	σ_{vo}'	σ_p'	OCR	C_u (DSS)	Parameters
W	1	0.055	7.25	131.8	0.42	S=0.24, m=0.71
	5	0.275	7.49	27.2	0.69	
	10	0.550	7.79	14.2	0.87	
	15	0.825	8.09	9.8	1.00	
	20	1.10	8.39	7.6	1.11	
T	1	0.055	1.00	18.2	0.13	S=0.28, m=0.73
	5	0.275	2.10	7.6	0.34	
	8	0.440	3.70	8.4	0.58	
	11	0.605	7.30	12.1	0.85	S=0.24, m=0.71
	13.8	0.759	11.60	15.3	1.26	

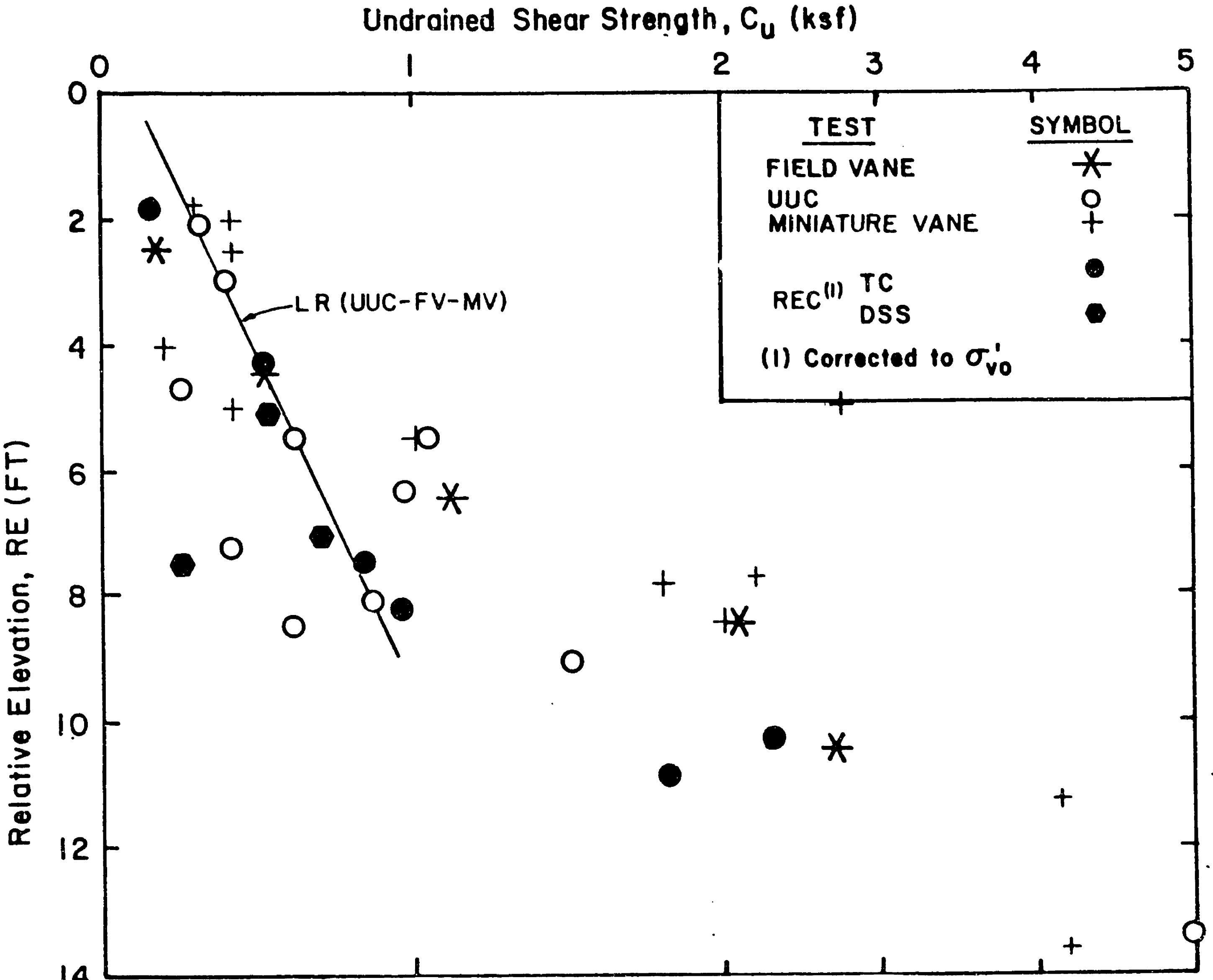


Figure 7-1: Profile of Undrained Strengths from Conventional and Recompression Tests, Site T

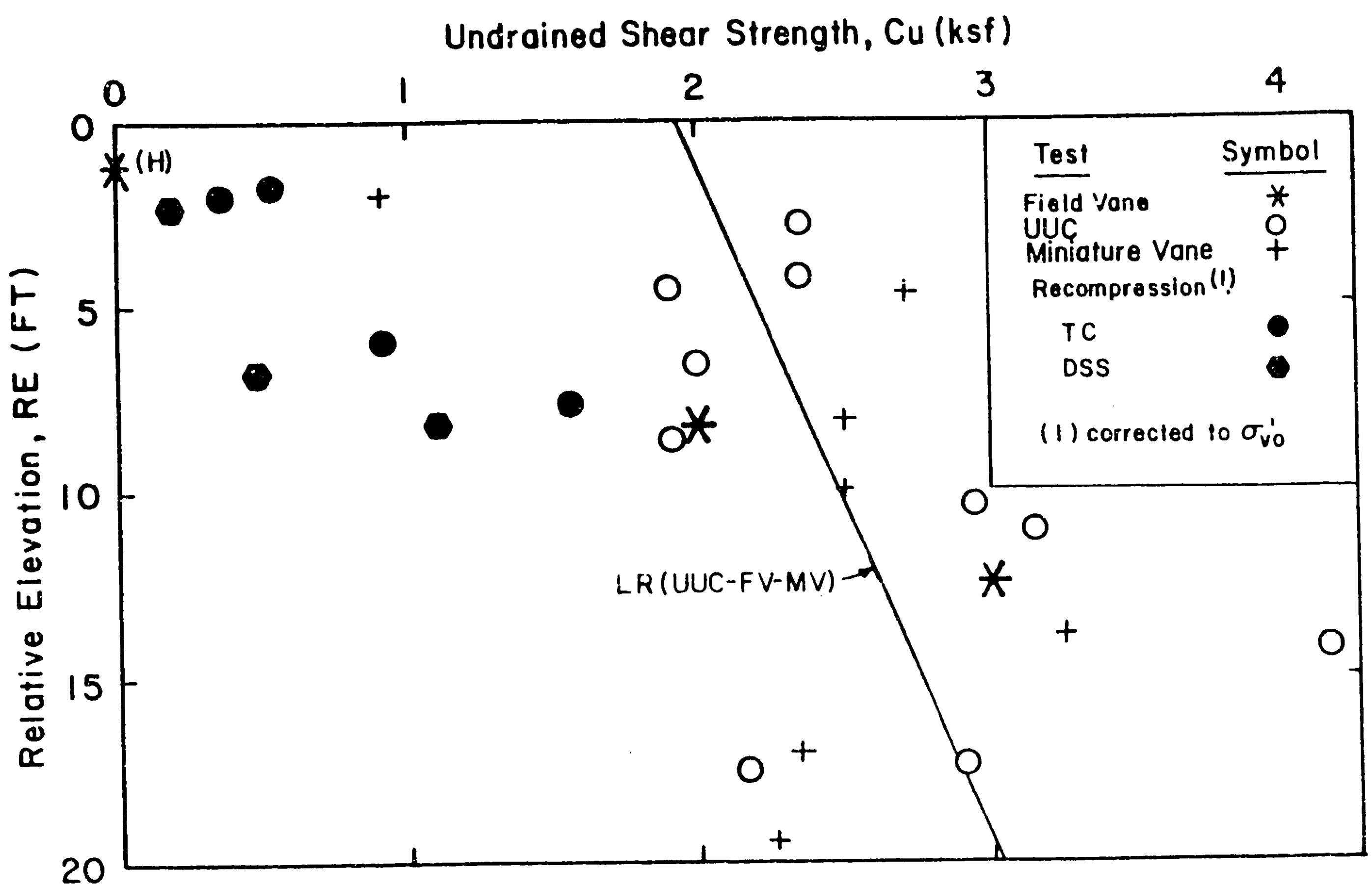


Figure 7-2: Profile of Undrained Strengths from Conventional and Recompression Tests, Site W

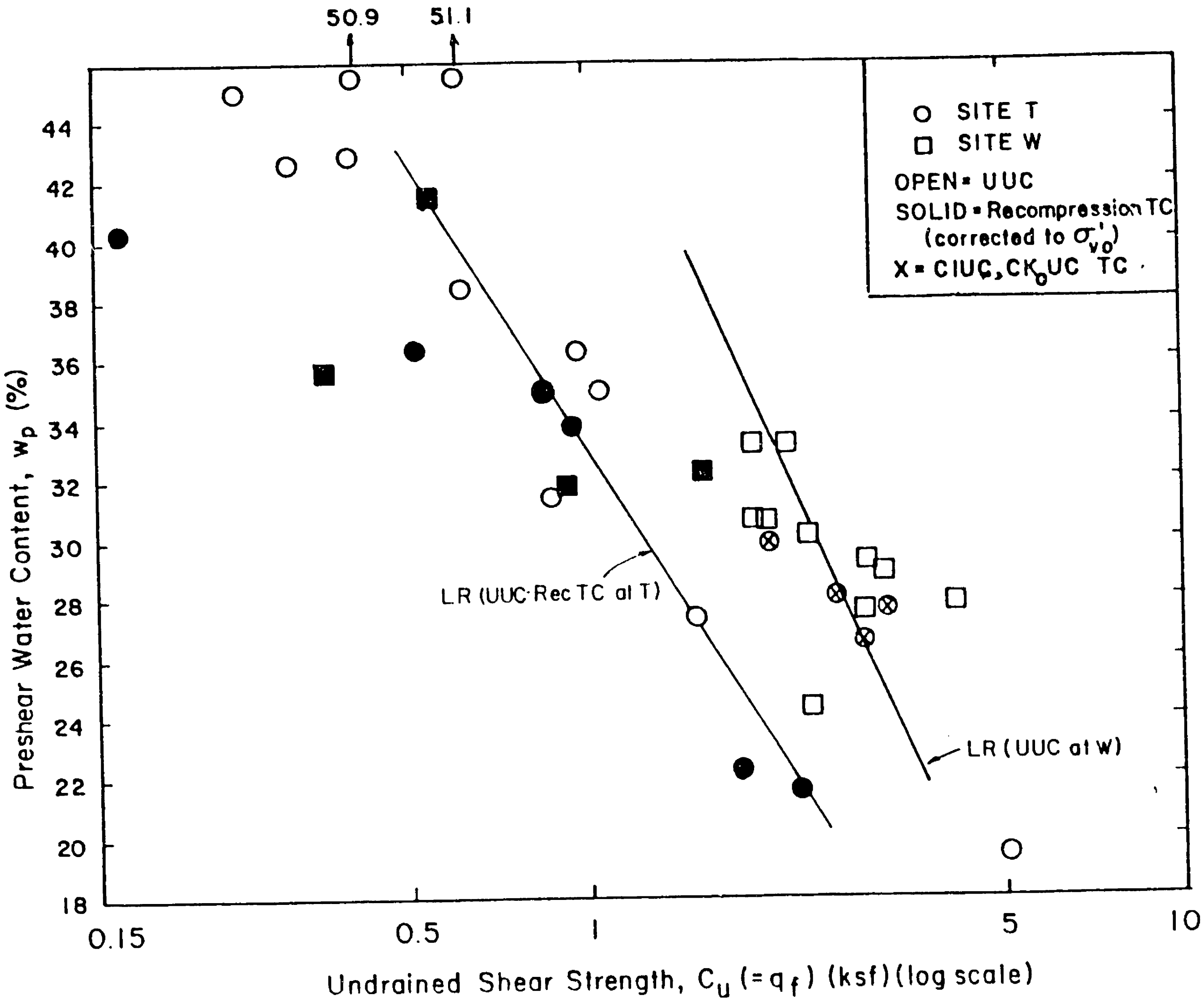


Figure 7-3: Preshear Water Content versus Undrained TC Strength,(q_f)

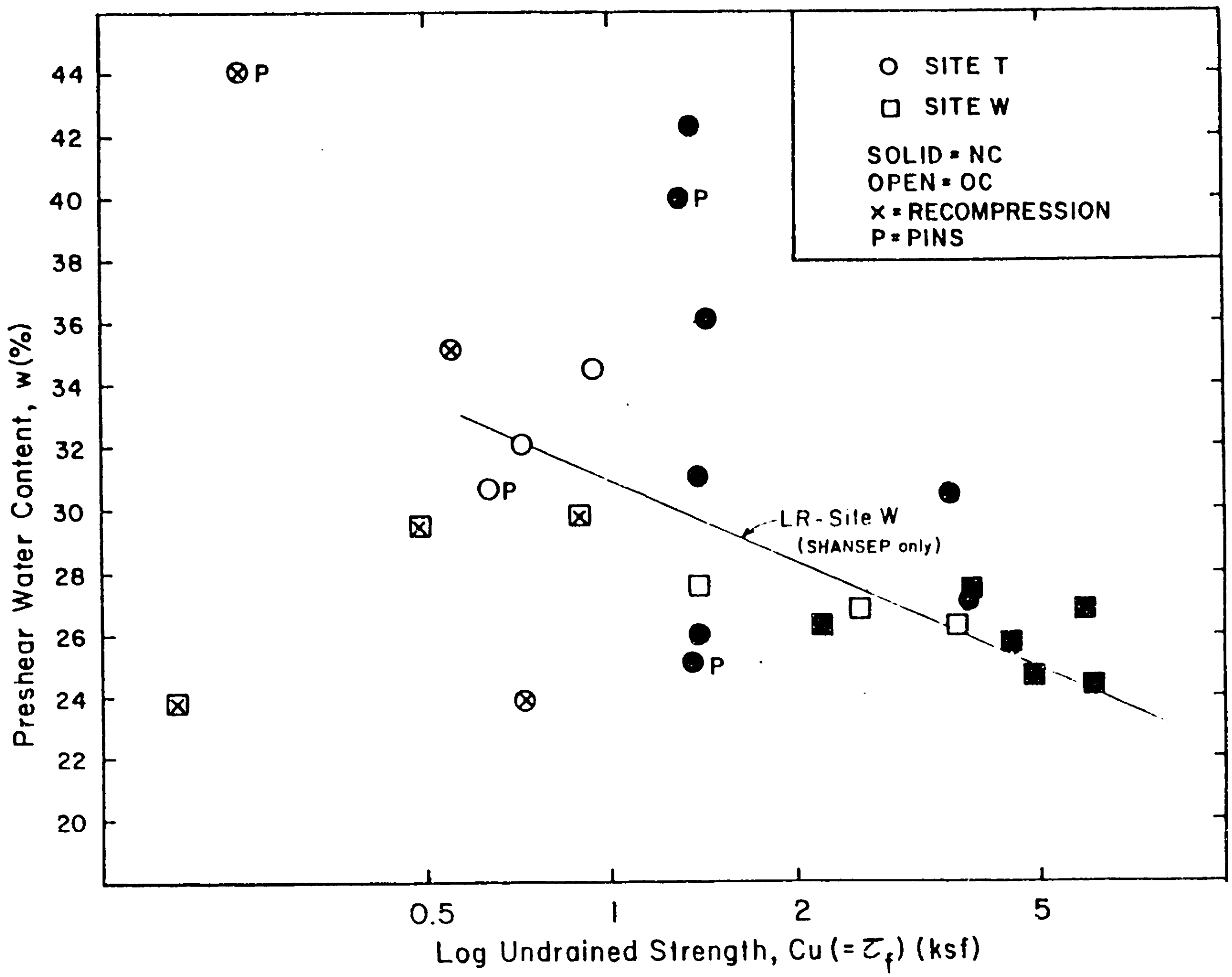


Figure 7-4: Water Content versus Undrained Strength. (τ_f)

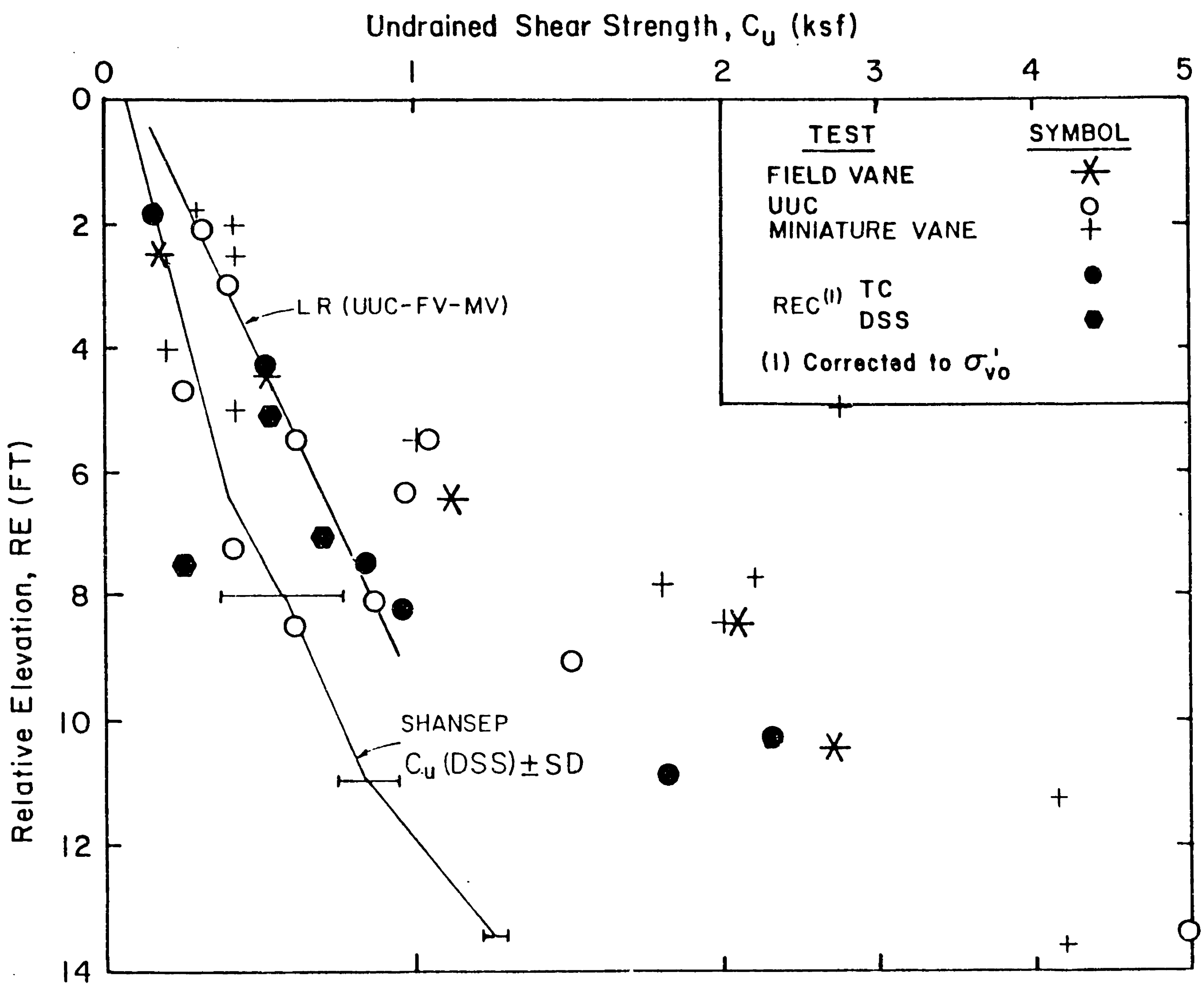


Figure 7-5: Undrained Shear Strength Profile, Site T

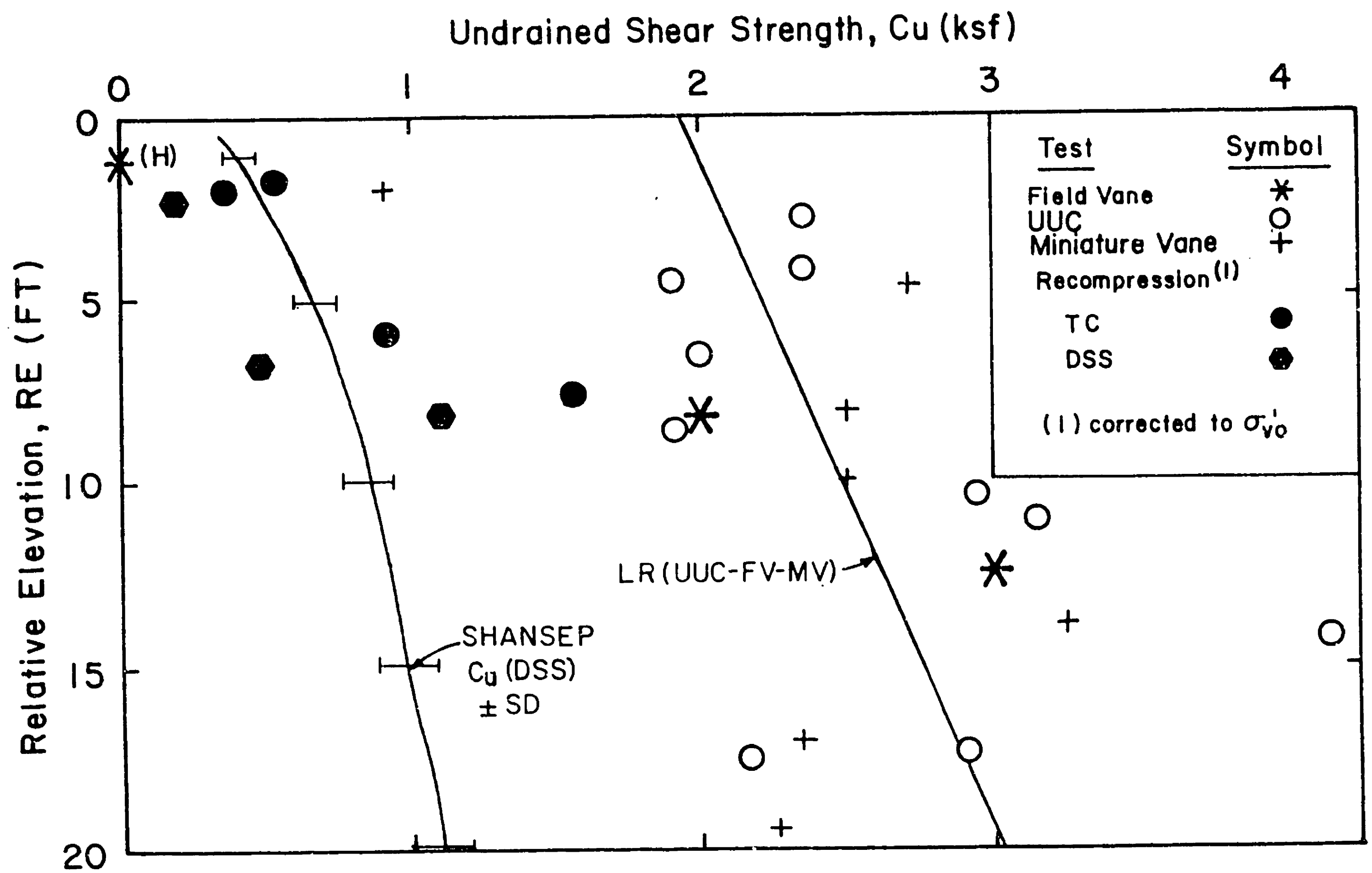


Figure 7-6: Undrained Shear Strength Profile, Site W

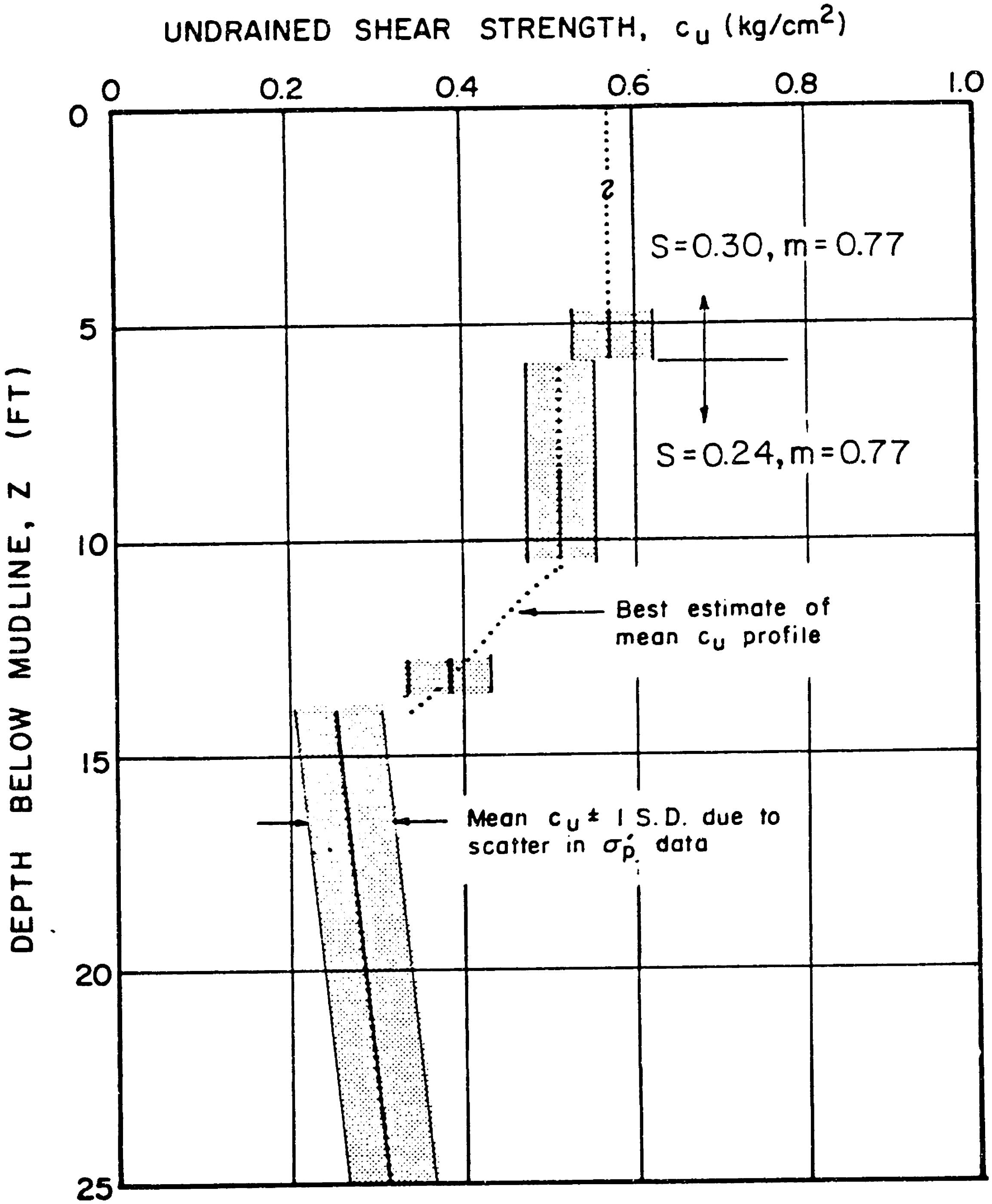


Figure 7-7: Undrained Shear Strength Profile, Mukluk Island, Harrison Bay

Chapter 8

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 BACKGROUND

In September, 1983, the Center for Scientific Excellence in Offshore Engineering was established at MIT under a five year, $2 million grant from The Standard Oil Company. The primary purpose of the Center is to conduct research on technical problems related to hydrocarbon development in the Beaufort Sea. The Center at MIT supports research activities in the Departments of Civil and Ocean Engineering related to ice mechanics, geotechnical, structural, hydrodynamic and risk and reliability aspects of offshore Arctic engineering.

The Center's program in geotechnical engineering addresses research in two areas: (1) an experimental evaluation of the engineering properties of Arctic silts; and (2) theoretical procedures for assessing the foundation stability of Arctic gravity structures. Research on the first topic was initiated in September 1983 and studies on the second topic commenced in September 1984.

8.1.1 Objectives

The ultimate aim of the geotechnical experimental program is to develop specific guidelines regarding recommended in situ and laboratory equipment and procedures to reliably measure those engineering properties of Arctic silts needed to execute safe, economical foundation designs of offshore structures. Specifically, the program seeks to address the following issues (Sauls et al., 1984):

1. Why Arctic silts exhibit unique behavior compared to other offshore sediments, which negates reliance on past empirical correlations.

2. What types of in situ and laboratory test programs should be used to develop reliable estimates of the initial strength-deformation properties needed to predict the performance of gravity structures during and after setdown.

3. What types of laboratory shear tests should be used to obtain design strengths in order to evaluate foundation stability against massive horizontal forces due to ice loadings.

8.1.2 Prior Research Activities

Experimental testing at MIT on the engineering properties of Arctic silts started in January 1984 using 15 tube samples supplied by The Standard Oil Production Company (SOPC) from the "soft zone area" of Harrison Bay. The location of Harrison Bay is shown in Figure 2-1. The results obtained on these samples have been published by the Center in Research Report No. 1, "Strength-Deformation Properties of Harrison Bay Arctic Silts", by Sauls, Germaine and Ladd (1984).

In April,1984, SOPC sponsored a special program of undisturbed sampling and in situ testing conducted next to Mukluk Island. Samples from this 25 foot thick deposit range from a highly overconsolidated low plasticity silt to a soft uniform clayey silt. Results of research conducted on these samples was presented in the MIT 1985 Master's theses, "Undrained Triaxial Strength-Deformation Behavior of Harrison Bay Arctic Silts" and "Consolidation and Direct Simple Shear Behavior of Harrison Bay Arctic Silts", by K.D. Ayan and E.Y-P. Yin, respectively.

8.2 OVERVIEW OF SMITH BAY RESEARCH PROGRAM

SOPC sponsored geotechnical exploration programs at four sites in Smith Bay in early 1985. Figure 2-1 shows the location of Smith Bay in the Beaufort Sea. The Center worked with SOPC to evaluate the results of the geotechnical program and assist SOPC and its design consultant (EBA Engineering Consultants Ltd., Canada) in development of a site specific design. MIT also performed additional special laboratory testing.

8.2.1 Site Characteristics

In December, 1985, MIT received undisturbed soil from the Standard Oil Company from two sites in Smith Bay. Both sites consisted of about 15-20 ft of silty clay/clayey silt overlying relict permafrost. The site nearer land (Site W) has a very thin layer of soft Holocene material over much stronger Pleistocene cohesive soils. The stratigraphy of the other site (Site T) is much more complex due to extensive ice gouging. Strengths are fairly constant at the nongouged site, while the gouged site has very low strengths in the upper layer which increase progressively with depth as ice gouging effects diminish. This thesis presents the results of research conducted on samples obtained from the two Smith Bay sites.

8.2.2 Field Program

The field testing and sampling programs were performed by The Earth Technology Corporation (TETC) in early 1985 under the general supervision of the Dallas, Texas, SOPC Arctic Technology Division. A total of 34 samples (50 linear feet) were recovered from three boreholes at Site W and 37 samples (49 linear feet)were recovered from four boreholes at Site T. Details of the exploration program are taken from proprietary reports provided to MIT. The field program at the two sites consisted of (TETC ,1985):

1. One central deep soil boring drilled and sampled to 150 ft. At site T, and additional boring was drilled adjacent to this one (5B1).
2. Two peripheral shallow soil borings drilled and sampled to 50 ft (NW and SE corners of the site)
3. Five thermal piezocone penetration tests (at center and at each corner of the site)
4. In-situ vane shear tests adjacent to the center boring
5. Pressuremeter testing (Site T)
6. Installation of one thermistor string
7. Lead line bathymetric survey for seafloor topography

Details of the field testing program are presented in Section 3.1.

8.2.3 Background of Smith Bay Arctic Silts

Chapter 2 describes the Beaufort Sea environment and the general properties of Smith Bay Arctic silts. Sections 2.1 and 2.2 summarize the geologic environment of Smith Bay, examining some of the mechanisms which could be responsible for the observed properties of the deposits, including freeze-thaw and ice gouging. Section 2.3 describes the general soil profiles at both sites. Section 2.4 describes and discusses possible shortcomings regarding the geotechnical investigations conducted previously in the Beaufort Sea. These programs generally used the same "conventional" techniques of in situ and laboratory testing procedures developed for the empirical design of pile supported platforms in the Gulf of Mexico. Section 2.5 describes the SHANSEP methodology (Ladd and Foott, 1974) and the Recompression testing technique (Bjerrum, 1973) that are investigated in this research as potentially better methods for evaluating strength parameters compared to previous procedures.

8.2.4 Scope of Laboratory Test Program

Tables 3-1 and 3-2 summarize, by site, the tests performed on samples by both TETC and MIT. TETC radiographed samples and performed classification and index tests as well as an extensive consolidation and strength testing program.

Remaining samples were sent to MIT, where radiography, index tests and consolidation and strength tests were performed to supplement the TETC data.

Nineteen conventional incremental oedometers were performed on material at Site T, and 12 at Site W. These tests were conducted to determine the stress history and consolidation properties of the deposits.

Two consolidated-undrained triaxial compression tests (CIUC, CK_0UC) were run by MIT on samples consolidated into the normally consolidated range to evaluate anisotropy of gouged material at Site T. Two additional CIUC tests were performed by MIT at OCR = 2 and 5 to determine SHANSEP strength parameters for triaxial compression in the gouged zone. Seven triaxial compression and three direct simple shear Recompression tests were performed by MIT and TETC at both sites.

MIT conducted twenty SHANSEP type CK_0UDSS tests on normally consolidated and overconsolidated material in order to determine if Smith Bay Arctic silt exhibited normalized behavior, and to provide information on resistance to horizontal sliding for the strength profile. Three Recompression DSS tests were conducted on material at each site (five by TETC and one by MIT).

Finally, the results from the consolidation, triaxial and DSS programs are collectively evaluated in order to present recommended normalized undrained strength parameters for design. A SHANSEP strength profile was calculated for both sites, and compared with results from the conventional and Recompression testing programs.

8.3 INDEX AND CLASSIFICATION PROPERTIES

The sample locations and scope of the MIT and TETC laboratory testing programs are described in Chapter 3. TETC classification and index tests included measurements of salinity, water content, Atterberg Limits and total unit weight. All samples sent to MIT were radiographed in order to detect zones of disturbance, presence of gas pockets and soil macrofabric. Atterberg limits and grain size analyses were performed by MIT to supplement TETC results.

The results of classification and tests are plotted in Figures 3-4 to 3-15. Index-classification testing on Smith Bay Arctic silts yielded the following results.

(1) The Atterberg limits for both sites plotted above the A-line, hence the soil is classified as a CL-CH silty clay.

(2) Water contents at Site T decrease from values of 40-45% at the surface to 20% at a depth of 14 ft. Water contents at Site W are fairly constant below a depth of 2-3 feet (29.8% $\pm$2.7 SD).

(3) Plasticity Index at Site T decreases from 25.0% $\pm$5.8 in the upper nine feet of the deposit to 18.7% $\pm$2.1 SD at greater depths. At Site W, Ip is more uniform and equal to 23.1% $\pm$2.8 SD.

(4) Site T has a nearly constant grain size distribution with depth (about 42% clay). The clay fraction at Site W is higher (about 54%) and constant with depth, except for a more silty upper layer of Holocene material.

8.4 CONSOLIDATION TEST PROGRAM

The consolidation test data obtained on samples from Smith Bay are presented in Chapter 4. This test program consisted of: (1) 23 standard incremental oedometer tests performed at 1 °C by TETC (12 at Site T, 11 at Site W); (2) additional oedometers performed by MIT at room temperature (seven at Site T and one at Site W); and (3) one temperature-controlled oedometer test at Site T to measure the sensitivity of the preconsolidation pressure and compressibility to ambient temperatures.

The temperature controlled test on gouged material at Site T indicate that tests at room temperature lead to estimates of preconsolidation pressure about 10% lower than at 1 °C. Estimates of preconsolidation pressure from oedometer tests and the consolidation phase of DSS tests were used to construct stress history profiles for both sites. At Site T, preconsolidation pressure increases gradually over the upper 8 ft of the deposit, with a large increase at greater depths (Figure 4-9). The progressive reduction in and scattered nature of the preconsolidation pressure approaching the mudline resulted from ice gouging effects. Temperature corrected values of the preconsolidation pressure data are represented by linear regression through the upper 7 ft. The selected stress history profile at greater depths was estimated by linear interpolation between statistical means calculated at RE = 8, 11 and 13.8 feet. The stress history profile at Site W is more uniform with a nearly constant preconsolidation pressure with depth (except for the upper Holocene layer) (Figure 4-11). The selected temperature corrected profile is described by linear regression through the oedometer and DSS data. The two sites are presumed to have shared a common initial stress history, as values of preconsolidation pressure are compatible for Site W and the lower part of Site T (Figure 4-12). Moreover, they have a unique liquidity index versus preconsolidation pressure relationship (Figure 4-10).

8.5 CONSOLIDATED-UNDRAINED STRENGTH TESTING

The program of consolidated-undrained shear tests had the following scope and principal objectives concerning the undrained stress-strain strength characteristics of Smith Bay Arctic silts.

(1) Normally consolidated CK_0UDSS tests were conducted at both sites to determine whether or not these soils exhibit normalized behavior and to compare gouged versus nongouged material.

(2) Normally consolidated CK_0UC and CIUC tests were performed to evaluate the anisotropy of gouged material at Site T and to define the normally consolidated strength for vertical loading.

(3) Overconsolidated CK_0UDSS tests and CIUC triaxial compression tests were performed to obtain overconsolidated undrained strength ratios necessary for a SHANSEP analysis.

(4) Recompression triaxial and DSS tests were conducted to measure the in situ overconsolidated undrained shear strengths for comparison with those predicted via SHANSEP.

8.5.1 SHANSEP Predicted Undrained Strength

The SHANSEP strength profiles for Sites T and W are developed based on strength parameters obtained from the triaxial compression and DSS test programs conducted by MIT (Chapters 5 and 6, respectively). The selected values of S and m were used in the relationship:

$$c_u/\sigma'_{vc} = S(OCR)^m$$

where $c_u = \tau_h$max for DSS tests and q_f for triaxial tests. OCR values are based on the selected stress history profiles developed in Chapter 4 (Figures 4-9 and 4-11).

8.5.2 Triaxial Program

The principal results and conclusions for the triaxial program presented in Chapter 5 are as follows:

(1) Two CIUC and CK_0UC tests on normally consolidated ice gouged material at Site T indicated that isotropically consolidated tests should provide reasonable estimates of OCR=1 undrained strength in triaxial compression.

(2) Establishment of SHANSEP strength parameters for triaxial compression based on CIUC and CK_0UC tests on gouged material at Site T. Values of S = 0.32 and m = 0.76 were selected for use based on linear regression through all data (Figure 5-7).

8.5.3 DSS Program

The principal results and conclusions from CK_0UDSS test program presented in Chapter 6 are as follows.

(1) Soil at Site T did not exhibit normalized behavior. However, reasonable SHANSEP parameters were established for use in estimating resistance to base sliding agree with estimates from DSS Recompression tests. Selected values were S = 0.28 and m = 0.73 for the upper highly gouged material.

(2) Soil at Site W gave more consistent estimates of normally consolidated strength, with a selected S of 0.24. Much higher values of S (up to 0.30) were measured for samples consolidated to insufficient stress to reach the virgin compression line.

(3) Results of tests on samples overconsolidated by the SHANSEP technique showed a consistent increase in c_u/σ'_{vc} for both sites. Linear regression through all test results gave m values of 0.73 for Site T (Figure 6-24) and 0.71 for Site W

(Figure 6-28). These values are in the lower end of the 0.8 ± 0.1 range quoted by Ladd et al. (1977), probably due to the relatively high ratio of Swelling Index (C_s) to Virgin Compression Index (C_c) measured for these samples.

8.5.4 Collective Evaluation

Chapter 7 presents a collective evaluation of the undrained strength data at both sites. Using results from the stress history (Chapter 4) and the strength testing programs (Chapters 5 and and 6), predicted triaxial compression (TC) and direct simple shear (DSS) strengths according to the SHANSEP procedure (Section 8.5.1) were compared with the results of Recompression and conventional tests.

Recompression Test Results

Undrained strengths measured in Recompression triaxial compression tests at Site T overestimate those predicted by SHANSEP by about 1.4 times (Table 7-2) for unexplained reasons. In contrast, Recompression test strengths at Site W were much less than those predicted by SHANSEP, probably due to poor testing procedures of these high OCR materials (Table 7-3).

Recompression triaxial compression tests agree with measured UUC strengths at Site T. This implies that the strength increase due to the high strain rate of the UUC tests is balanced by disturbance effects. Recompression triaxial compression tests at Site W are less than the UUC values due to poor testing procedures of the former.

Recompression direct simple shear tests had strengths comparable to those predicted by SHANSEP at Site T. However, Recompression tests at Site W were of very low quality and therefore were significantly less than the SHANSEP strengths.

Conventional Test Results

Profiles for resistance to base sliding were developed for both sites using

SHANSEP parameters from the DSS program. Values of preconsolidation pressure were taken from Figures 4-9 and 4-11 for Sites T and W, respectively. The resultant error is probably small compared to the scatter in σ'_p data at Site T.

The UUC, FV and MV data are compared to the SHANSEP c_u(DSS) profile at Site T in Figure 7-5. Conventional strength test results are about 1.6 times greater than the SHANSEP c_u(DSS) values in the upper highly gouged zone. Fewer conventional tests are available for comparison below RE = 9 ft, but values are on the order of 2.2 to 3.4 times greater than the SHANSEP predicted strengths.

The UUC, FV and MV data are compared to the SHANSEP c_u(DSS) profile at Site W in Figure 7-6. Conventional strength test results are about 5 times greater than the SHANSEP c_u(DSS) values at RE of 1 ft, and two times greater at RE of 20 ft.

The discrepancy between conventional test strengths and SHANSEP predictions appears to be greater at higher OCR, possibly due to the increased importance of strain rate effects for highly overconsolidated cohesive soils.

The results for Smith Bay contrast Harrison Bay findings. In the "soft zone area" mean UUC values (although scattered) agreed with the SHANSEP design profile, while field vane strengths were two times greater.

8.6 FUTURE RESEARCH

The amount of available undisturbed soil is extremely limited for Smith Bay sites; however, several tests would still be worthwhile to perform. The following tests are recommended for Smith Bay:

(1) Normally consolidated CK_0UDSS tests at high consolidation stresses (= 24 ksf) at Site T to determine whether the normally consolidated strength (S) decreases with further increases in consolidation stress, or remains stable.

(2) Overconsolidated tests on Site T soil consolidated to a maximum stress of 16 or 24 ksf to find the m corresponding to S = 0.24, for comparison with the m corresponding to S = 0.28.

(3) Overconsolidated tests at very high OCR. This will necessitate the development of an alternative to the stones with pins presently used to improve the shear stress transfer. These potentially create more disturbance, plus complicate test setup and analysis of strain and modulus results.

(4) Recompression TC and DSS tests at Site W (since TETC results were no good) for comparison with SHANSEP predictions for nongouged material. Again, an alternative to stones with pins is necessary to ensure reliable results.

In situ tests, such as the piezocone penetrometer (PCPT), might provide more reliable and potentially more economical means of assessing the variability of in situ undrained strength (and maybe preconsolidation pressure) than laboratory testing. Although the PCPT pore pressure data at Smith Bay are suspect, q_c data should be thoroughly analyzed in order to attempt correlation to c_u and possibly σ_p.

The ultimate objective of the Center is to develop specific guidelines regarding recommended in situ and laboratory equipment and procedures to reliably measure those engineering properties of Arctic silts needed to execute safe, economical foundation designs for offshore Arctic gravity platforms. Since comparison of conventional test results to SHANSEP predicted design strengths at Smith Bay and Harrison Bay Soft Zone Area yielded completely different results, accomplishment of this objective may require detailed data from sites having a variety of geologic environments.

Chapter 9

REFERENCES

Note: ASCE = American Society of Civil Engineers
ASTM = American Society for Testing and Materials
ICSMFE = International Conference on Soil Mechanics and Foundation Engineering
JGED = Journal of Geotechnical Engineering
JSMFD = Journal of the Soil Mechanics and Foundations Division
STP = Special Technical Publication
USGS = United States Geological Survey

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Appendix A
LIST OF SYMBOLS

Prefix Δ indicates a change

Suffix f indicates a final or failure condition

A superscript prime on a stress indicates an effective stress

A superscript prime on a property indicates value in terms of effective stress

GENERAL

NC	Normally consolidated
OC	Overconsolidated
SD	Standard deviation
SZA	Soft Zone Area
z	Depth below mudline
RE	Relative Elevation

INDEX AND CLASSIFICATION PROPERTIES

e	Void ratio
e_o	Initial void ratio
G_s	Specific gravity
I_L	Liquidity index
I_P	Plasticity Index
S	Degree of saturation
w	Water content

w_L	Liquid limit
w_N	Natural water content
w_P	Plastic limit
γ_b	Buoyant unit weight
γ_d	Dry unit weight
γ_t	Total unit weight
γ_w	Unit weight of water

STRESSES, STRAINS, MODULI,AND STRENGTH PARAMETERS

c_u	Undrained shear strength
c_u(DSS)	c_u from direct simple shear test
c_u(TC)	c_u from triaxial compression test
c_u(TE)	c_u from triaxial extension test
E,E'	Young's modulus
E_u	Undrained secant E
E_u(50)	E_u half way to failure
G_i	Initial undrained shear modulus
K_c	$\sigma'_{hc}/\sigma'_{vc}$
K_0	Coefficient of earth pressure at rest
K_s	Anisotropic strength ratio = $q_f(E)/q_f(C)$
m,n	OCR exponents
OCR	Overconsolidation ratio = σ'_p/σ'_{vo}, σ'_p/σ'_{vc}
R_f	Hyperbolic stress-strain parameter

S	Normally consolidated undrained strength ratio
t_c	Consolidation time under last increment
t_f	Time to failure
u	Pore water pressure
ϵ_a	Axial strain
ϵ_{vol}	Volumetric strain
$\epsilon_1, \epsilon_2, \epsilon_3$	Principal strains
δ	Angle between σ_{1f} and vertical direction
γ, γ_f	Shear strain
σ, σ'	Normal total stress, normal effective stress
$\sigma_1, \sigma_2, \sigma_3$	Principal stresses
$\sigma_{ff}, \sigma'_{ff}$	Normal stress on failure plane at failure
σ_h	Horizontal normal stress
σ_v	Vertical normal stress
τ	Shear stress
τ_{ff}	τ on failure plane at failure
τ_h	τ on horizontal plane (direct simple shear test)
ϕ, ϕ'	Slope of Mohr-Coulomb failure envelope
ψ	"Friction angle" in DSS tests = Arctan τ_h/σ_v

CONSOLIDATION PARAMETERS

c_v	Coefficient of consolidation for vertical flow
C_c	Virgin compression index

C_r	Recompression index
C_s	Swelling index
C_α	Rate of secondary compression $= \Delta\epsilon_v/\Delta \log t$
CR	Virgin compression ratio $= \Delta\epsilon_v/\Delta \log\sigma_{vc}$
k	Permeability
m_v	Coefficient of volume change $= \Delta\epsilon_v/\Delta\sigma_v$
RR	Recompression Ratio
SR	Swelling Ratio
T	Temperature
t	time
t_p, t_{100}	t required for primary consolidation
ϵ_v	Vertical strain
σ'_{vc}	Vertical consolidation stress
σ'_{vo}	Initial vertical effective stress
σ'_p	Preconsolidation pressure

CONSOLIDATION AND STRENGTH TESTS

CIUC	Isotropically consolidated-undrained triaxial compression test
CK_0U	K_0 consolidated -undrained shear test
CK_0UC	CK_0U triaxial compression test
CK_0UDSS	CK_0U direct simple shear test
CK_0UE	CK_0U triaxial extension test
DSS	Direct simple shear

LV	Laboratory vane test
MV	Miniature vane test
PP	Pocket penetrometer test
TC	Triaxial compression
TE	Triaxial extension
TV	Torvane test
UU	Unconsolidated-undrained shear test
UUC	UU triaxial compression test

Appendix B
TETC LABORATORY TESTING PROCEDURES

Details were taken from TETC (1985).

B.1 Salinity Tests

Salinity tests were performed on the same samples as used for moisture content determinations. After oven-drying, the samples were blended thoroughly with 100 ml of distilled water. After a soaking period of 24 hours, the fluid was extracted out of the samples and the salinity test was performed using a conductivity probe. The test results were corrected for the amount of water added.

B.2 Consolidation Testing

One-Dimensional Oedometer Consolidation Tests were performed at 1 °C in a cold room using a standard dead-load type consolidometer. After the trimmed sample (1.0 inch high by 2.5 inch diameter) is placed in the load frame, the first load is applied and axial deformations are recorded at doubling-time increments (0.25 min, 0.5 min, 1.0 min, 2.0 min, etc.). After the sample has been allowed to consolidate under that load until the end of primary consolidation, the next load is added; the procedure is repeated until the entire specified loading and unloading history has been completed. Since most samples were taken from shallow depths, it was desirable that consolidation pressures started from 0.05 ksf. By calculating the deflection representing 100 percent primary consolidation, plots can be developed showing the calculated void ratio and coefficient of consolidation as a function of logarithm of applied vertical stress.

B.3 Strength Testing

Laboratory strength tests were performed on both pushed and driven samples. All tests were performed under controlled temperature at 1 °C.

Consolidated-Undrained CU Static Simple Shear Tests were performed using a Geotest simple shear device. The undisturbed sample was enclosed in a wire reinforced membrane (zero radial strain) and was placed in the test chamber. The sample was then saturated using backpressure. A B-value of 0.95 was obtained indicating good sample saturation. A strain rate of 0.0006 inch per minute was used during shearing. Shear load, pore pressure and deformation were monitored during the test.

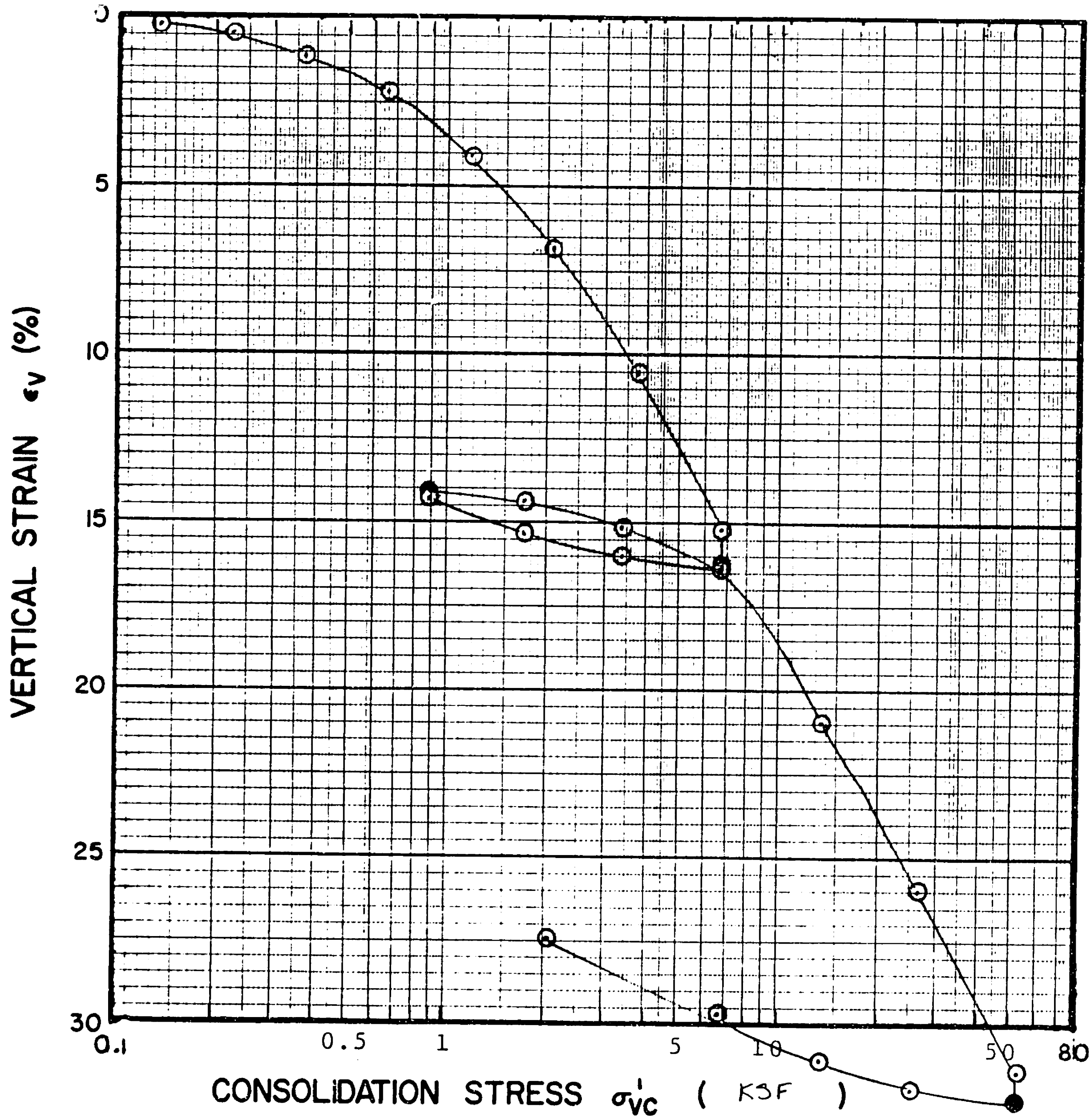
Unconsolidated Undrained (UU) Triaxial Tests were performed on selected samples in accordance with ASTM D-2850. The basic procedure is to place the trimmed sample into a membrane, place it in the triaxial cell, apply a total confining pressure equal to the estimated in situ total vertical stress and, with the drainage lines still closed, axially strain the sample at a rate of about one percent per minute. During the test, axial load and deformation are recorded.

K_0-Consolidated-Undrained Triaxial Tests were performed by consolidating samples under effective vertical pressures near the estimated in situ vertical stresses with no radial strain. Saturation of the samples was achieved utilizing back pressure. B-values of 0.95 or better indicate good saturation. The lowest possible consolidation pressure (3.0 psi) was applied onto the samples. The consolidation pressure could be higher than the in-situ pressures for the samples from shallow depths. The strain rate of loading was 0.08 percent per minute, adequate to allow 95 percent pore pressure equalization as based upon the consolidation phase of the tests.

Appendix C
CONSOLIDATION TESTS

Appendix C summarizes the tabulated results and figures from each of the consolidation tests performed by MIT on the Smith Bay Arctic silts from the 1985 test program. The test series included seven room temperature incremental oedometers and one temperature-controlled test. The test equipment and procedures are described in Chapter 4.

The compression curves and corresponding tabulated data are presented in the following order: Site T tests T13 through T19, Site W test W6.



Sample No. TB3-P3

Depth (RE) 7.6'

Soil Type ARCTIC SILT SILTY CLAY (CL)

w_N (%) 51.7

w_L (%) 65.8

w_P (%) 32.7

I_P (%) 33.1

Estimated

σ'_{vo} 0.391 σ'_p 1.7 ± 0.12

CR 0.18 RR 0.025

G_s 2.75 e_0 1.571 S(%) 90.5

o At t_p or hr

• At (t_f) hr

Remarks Corrected for apparatus compressibility.

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COMPRESSION CURVE

TEST NO. T13 (TB3CON1)

FIGURE

CONSOLIDATION TEST

Project Sohio Type of Test Standard No. T13 (TB3 CON1) Tested by CDG Date 12/19/86

Soil Type Arctic Silt Location Smith Bay Site 7 Sample Height 1.704 CM

Silty Clay (CL) Boring TB3-P3, RE = 7.6 FT. Sample Diameter 6.362 CM

Initial w(%) 51.7 G_s 2.78 w_N(%) 54.4 w_L(%) 65.8 Corrections Apparatus Compressibility

Void Ratio e 1.571 S(%) 91.5 w_p(%) 32.7 P.I.(%) 33.1 Units: $\bar{\sigma}_{vc}$ KSC c_v $\times 10^{-3}$ cm²/sec

$\bar{\sigma}_{vc}$	Primary			Total			$C_{\alpha e}$(%)	Coef. of Consol.		Remarks
	t (hr)	ε_v(%)	e	t (hr)	ε_v(%)	e		$\sqrt{t}$	log t	Avg k ($\times 10^{-8}$ cm/s)
0.066	—	—	—	0.27	0.206	1.566				
0.109	0.37	0.380	1.561	0.78	0.571	1.556	0.283	1.63	0.636	5.15
0.181	0.27	1.09	1.543	11.22	1.30	1.537	0.506	0.927	0.977	6.29
0.323	0.37	2.19	1.514	2.58	2.42	1.509	0.562	0.938	0.679	6.51
0.581	0.38	4.04	1.467	3.25	4.37	1.459	1.18	0.784	0.643	5.23
1.02	0.8	6.87	1.394	6.30	7.25	1.385	1.12	0.571	0.378	3.25
1.84	2.0	10.5	1.301	12.7	11.5	1.276	1.69	0.421	0.378	1.95
3.31	0.78	15.2	1.180	24.8	16.3	1.151	1.91	0.508	0.319	1.53
1.66	0.12	16.0	1.160	11.1	15.9	1.163				
0.835	0.3	15.3	1.179	14.4	15.0	1.185				
0.437	0.68	14.3	1.203	8.0	13.1	1.235				
0.835	0.25	14.3	1.204	21.2	14.4	1.202	0.152	1.212	1.214	13.5
1.66	0.7	15.1	1.184	42.9	15.3	1.178	0.225	1.197	0.976	1.19
3.31	0.25	16.3	1.152	10	17.0	1.135	0.826	1.822	1.283	1.39
6.64	0.5	21.0	1.031	13	21.9	1.007	1.74	0.549	0.432	8.52
13.0	0.3	26.0	0.902	12.8	27.2	0.871	2.11	0.529	0.501	5.34
26.0	0.28	31.5	0.760	26	32.4	0.739	1.12	0.569	0.623	3.56
12.8	0.06	32.0	0.749	10	31.8	0.752				

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Remarks KSC × 2.04618 = KSF

CONSOLIDATION TEST

Project Sohio Type of Test Standard No. T13 (TB3-CONT) Tested by CLG Date 12/17/85

Soil Type Arctic Silt - Silty clay (CL) Location Smith Bay - Site T Boring TB3-P3, Elev [illegible] Sample Height 1.704 cm

Sample Diameter 6.362 cm

Initial w(%) 51.7 G_s [illegible] w_N(%) 54.4 w_L(%) [illegible] Corrections Apparatus Compression

Void Ratio e 1.571 S(%) [illegible] w_p(%) 32.7 I_p(%) [illegible] Units: σ'_{vc} - KSC c_v - $\times 10^{-3}$ cm²/sec

σ'_{vc}	Primary			Total			C_α(%)	Coef. of Consol.		Remarks
	t (hr)	ϵ_v(%)	e	t (hr)	ϵ_v(%)	e		$\sqrt{t}$	log t	
6.64	0.15	31.1	0.771	10	30.8	0.778				
3.32	0.67	29.7	0.806	12.5	29.5	0.812				
1.02	1.3	27.4	0.867	6	26.7	0.885				

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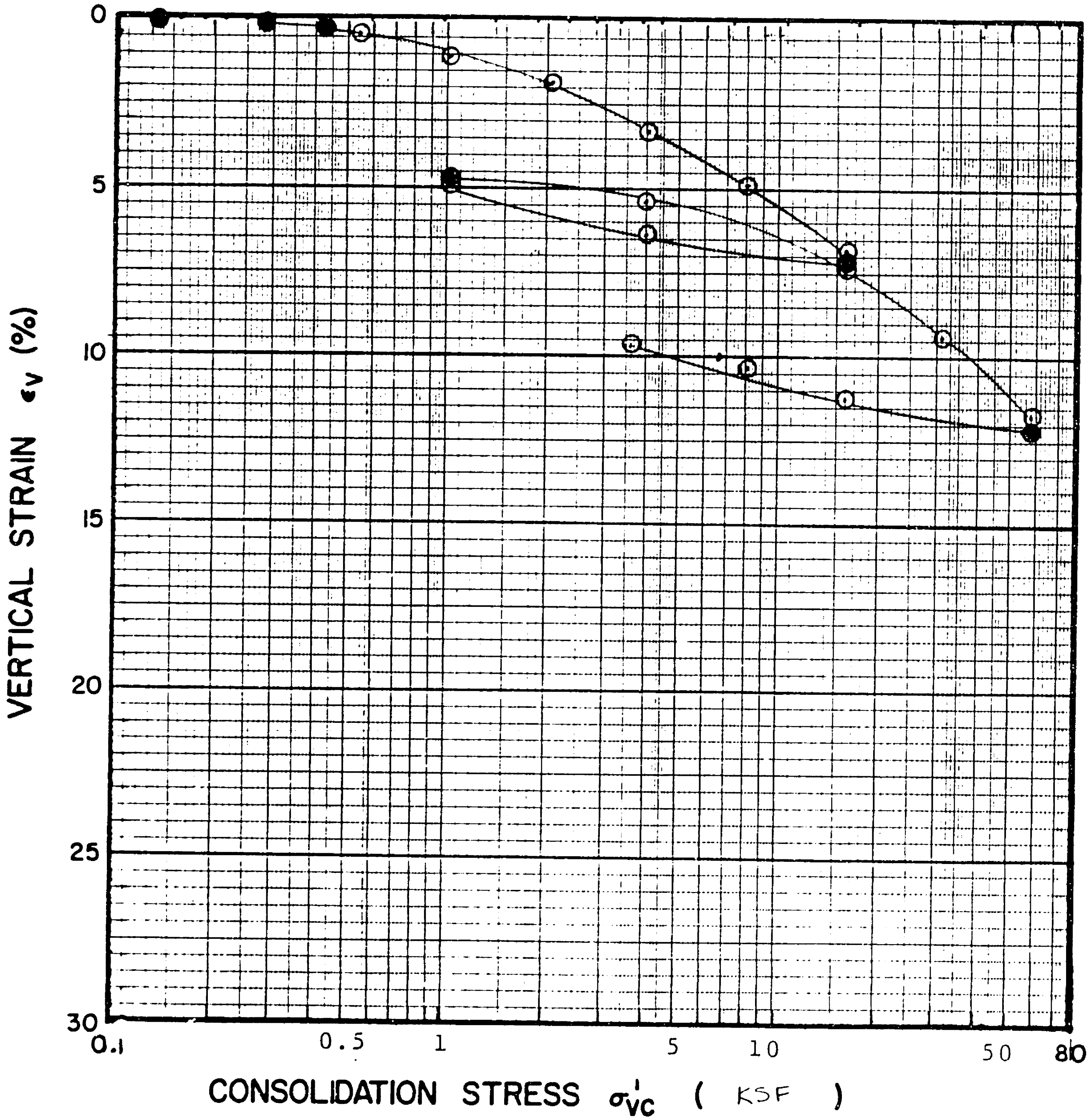
Remarks KSC × 2.048 = [illegible] F



Sample No. TB3-P4

Depth (KE) 10.0 FT

Soil Type ARCTIC SILT SILTY CLAY (CL)

w_N (%) 22.2

w_L (%) 40.6

w_P (%) 21.0

I_P (%) 19.6

Estimated

σ'_{vo} 0.572 σ'_p 7.6 ± 1.7

CR 0.081 RR 0.021

G_s 2.78 e_0 0.66 S(%) 90.2

○ At t_p or hr

● At (t_f) hr

Remarks Corrected for apparatus compression 7/.

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COMPRESSION CURVE

TEST NO. T14 (TB3CON2) 2/30

FIGURE

CONSOLIDATION TEST

Project Somio Type of Test Standard No. (T14) Tested by CLG Date [illegible]

Soil Type Arctic Silt Location [illegible] Bay - Site 7 Sample Height 2.019 CM

SILTY CLAY (CL) Boring E3-F4, RE - 10.9 FT. Sample Diameter 6.35 CM

Initial w(%) 22.2 G_s 2.78 w_N(%) 22.8 w_L(%) 40.6 Corrections Apparatus Compressibility

Void Ratio e 0.660 S(%) 93.5 w_P(%) 21.0 I_P(%) 19.6 Units: σ'_{vc} - KSC c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}	Primary t (hr)	Primary ϵ_v(%)	Primary e	Total t (hr)	Total ϵ_v(%)	Total e	$C_{\alpha e}$(%)	Coef. of Consol. $\sqrt{t}$	Coef. of Consol. log t	Remarks
0.066	—	—	—	0.2	0.05	[illegible]				Avg. k ($\times 10^{-9}$ cm/s)
0.095	—	—	—	0.4	0.13	0.658				
0.138	—	—	—	10.4	[illegible]	0.656				
0.209	—	—	—	3.3	0.33	0.654				
0.266	1.7	0.46	0.652	2.2	0.46	0.652				
0.509	0.67	1.04	0.643	1.5	1.08	0.642	0.221	0.887	0.569	16.4
0.99	0.67	1.97	0.627	2.8	2.05	0.622	0.147	0.874	0.560	14.6
1.98	0.6	3.26	0.606	14.1	3.44	0.603	0.179	0.638	0.661	8.55
3.94	0.55	4.88	0.579	4.0	5.02	0.576	[illegible]	0.715	0.627	5.87
7.85	0.43	6.83	0.546	27.4	7.22	0.542	0.506	0.885	0.722	4.32
1.98	0.3	6.37	0.554	2.4	6.27	0.556				
1.51	1.4	4.81	0.580	14.7	4.67	0.582				
1.98	0.3	5.42	0.570	1.92	5.50	0.568		0.640	0.629	2.72
7.85	0.2	7.42	0.537	18.9	7.63	0.533	0.138	1.259	1.170	4.33
15.68	0.3	9.37	0.504	2.7	9.64	0.500	0.502	1.106	0.966	2.93
29.[illegible]	0.3	11.70	0.466	27.5	12.17	0.458	0.413	1.054	0.890	2.01

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Remarks KSC × 2.04816 = KSF

CONSOLIDATION TEST

Project Sohio Type of Test Standard No. T14 (TB33-0N2) Tested by CLG Date 1/16/86

Soil Type Arctic Silt Location Smith Bay - Site F Sample Height 2.09 CM

Silty Clay (CL) Boring 33-R4, EL = 10.9' Sample Diameter 6.35 CM

Initial w(%) 22.2 G_s 2.78 w_N(%) 22.8 w_L(%) 40.6 Corrections Apparatus Compressibility

Void Ratio e 0.660 S(%) 93.5 w_P(%) 21.0 I_P(%) 19.6 Units: σ'_{vc} KSC c_v $\times 10^{-3}$ CM²/SEC

σ'_{vc}	Primary			Total			C_α(%)	Coef. of Consol.		Remarks
	t (hr)	ϵ_v(%)	e	t (hr)	ϵ_v(%)	e		$\sqrt{t}$	log t	
7.85	0.2	11.22	0.473	2.2	11.19	0.474				
3.94	0.6	10.33	0.488	13.7	10.29	0.480				
1.78	0.9	9.71	0.499	7.9	9.61	0.500				

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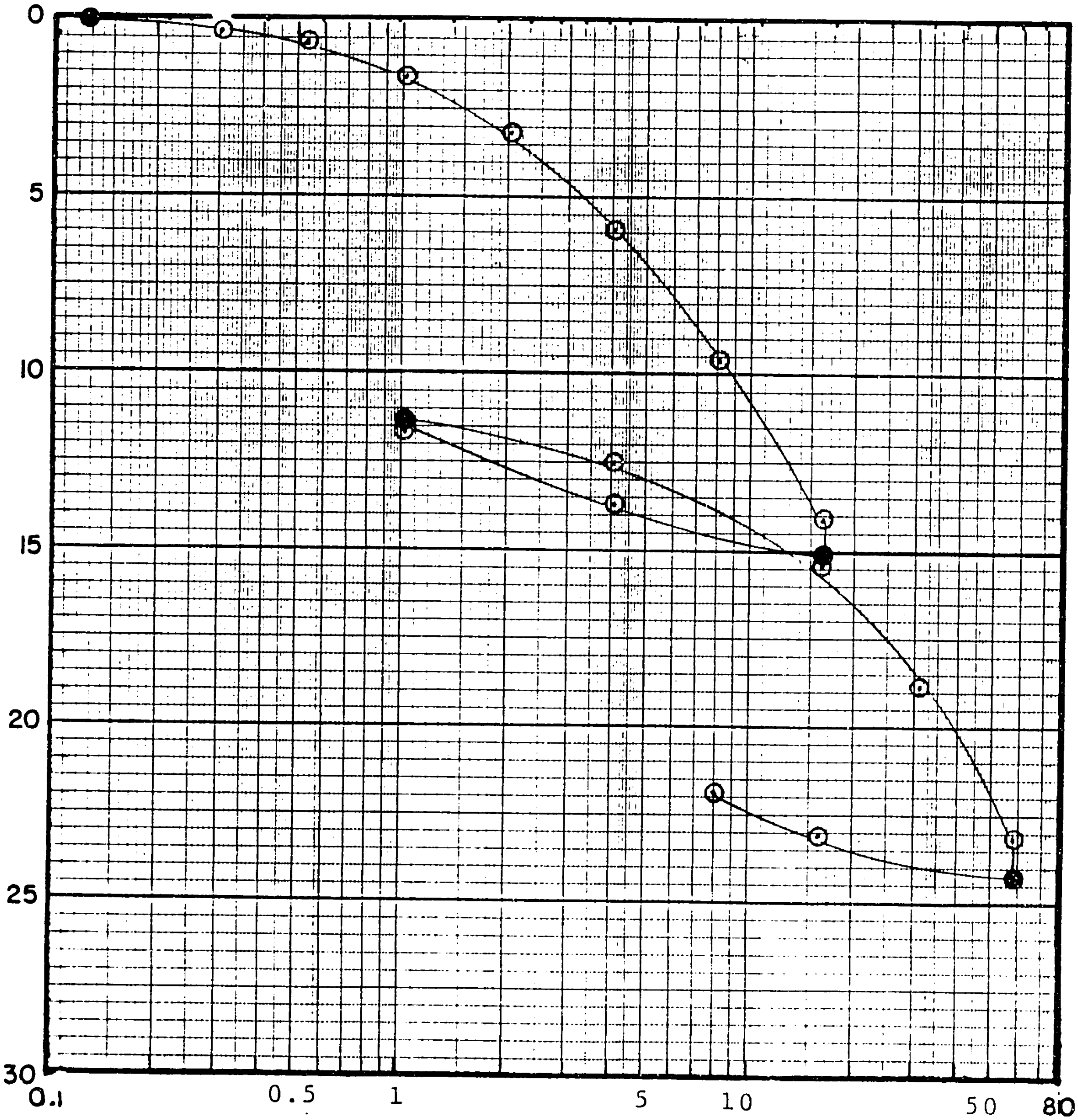
Remarks KSC × 2.04816 = KSF



Sample No. T5B1-P3

Depth (RE) 8'

Soil Type Arctic Silt SILTY CLAY (CL)

w_N (%) 41.2

w_L (%) 45.2

w_P (%) 23.7

I_P (%) 21.5

Estimated

σ'_{vo} 0.440 σ'_p 3.5 ± 0.28

CR 0.158 RR 0.030

G_s 2.78 e_o 1.109 S(%) 99.2

○ At t_p or hr

● At (t_f) hr

Remarks Corrected for apparatus compressibility

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COMPRESSION CURVE

TEST NO. T15 (aka T5B1P3)

FIGURE

CONSOLIDATION TEST

Project Sohio   Type of Test Standard   No. (T15 15B1P3)   Tested by CLG   Date 1/17/86

Soil Type Arctic Silt   Location Smith Bay - Site T   Sample Height 2.319 cm

Silty Clay (CL)   Boring SB1-P3, RE = 8'   Sample Diameter 6.35 cm

Initial w(%) 41.2 G_s 2.78 w_N(%) 33.3 w_L(%) 45.2   Corrections Apparatus Compressibility

Void Ratio e 1.109   S(%) 102 w_p(%) 23.7 I_p(%) 21.5   Units: σ'_{vc} KSC c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}	Primary			Total			$C_{\alpha e}$(%)	Coef. of Consol.		Remarks
	t (hr)	ϵ_v(%)	e	t (hr)	ϵ_v(%)	e		$\sqrt{t}$	log t	
0.062	—	—	—	0.6	0.06	1.108				Avg. k ($\times 10^{-8}$ cm/s)
0.148	1.0	0.34	1.102	1.3	0.35	1.102				
[illegible]	0.9	0.61	1.096	2.7	0.67	1.095				
0.505	1.0	1.61	1.075	14.5	1.76	1.072		0.584	0.468	2.19
0.991	1.0	3.18	1.042	10.0	3.53	1.035	0.287	0.546	0.442	1.63
1.978	1.1	5.96	0.983	14.2	6.55	0.971	0.444	0.465	0.535	1.47
3.938	1.3	9.62	0.906	8.4	10.32	0.892	0.604	0.495	0.562	1.07
7.842	0.8	14.2	0.810	38.4	15.06	0.792	0.544	0.513	0.568	0.73
1.978	0.4	13.7	0.819	3.1	13.67	0.821				
0.505	0.4	11.6	0.865	4.2	11.3	0.872				
1.978	1.0	12.5	0.846	16.0	12.6	0.844				
7.842	0.3	15.4	0.785	6.5	15.5	0.777	0.238	1.074	1.167	0.618
15.680	0.4	18.93	0.710	2.2	19.4	0.700	0.695	0.670	0.755	0.393
28.42	0.3	23.13	0.621	37.7	24.3	0.597	0.529	0.623	0.608	0.260
7.819	0.3	23.1	0.622	1.9	23.0	0.625				
3.909	0.9	21.93	0.647	2.1	21.8	0.650				
1.711										

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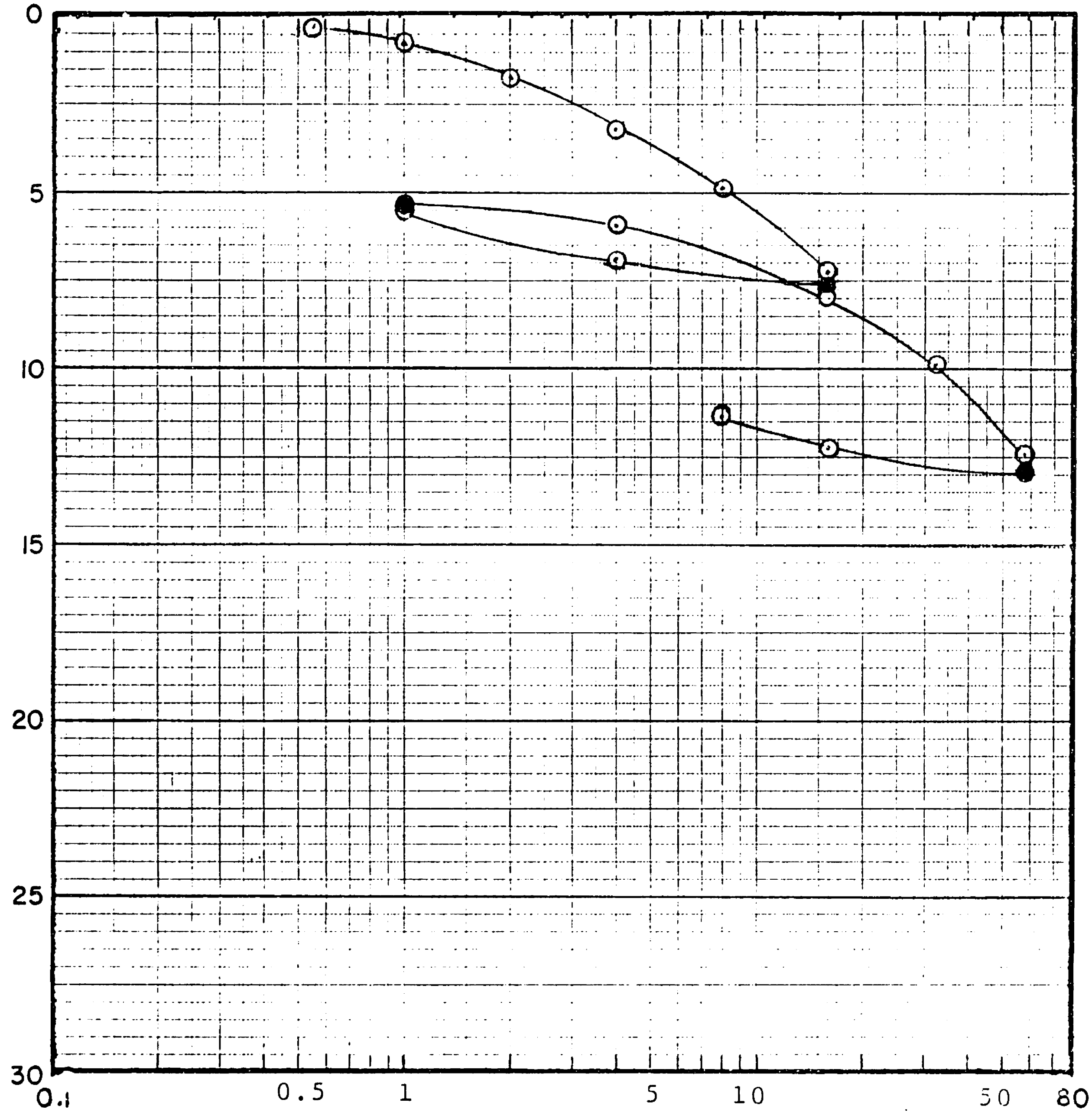
Remarks KSC × 2.04816 = KSF



CONSOLIDATION STRESS σ'_{vc} (ksf)

Sample No. T5B1-54 | w_N (%) 22.9 | Estimated

Depth (RE) 10.4 FT | w_L (%) 39.5 | σ'_{vo} 0.572 σ'_p 6.2

Soil Type Arctic Silt | w_P (%) 19.0 | CR 0.093 RR 0.018

I_P (%) 20.5 | G_s 2.78 e_0 0.659 S(%) 96.5

○ At t_p or hr

● At (t_f) hr

Remarks Corrected for apparatus Compressibility

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
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COMPRESSION CURVE

TEST NO. T16 (T5B154)

FIGURE

CONSOLIDATION TEST

Project Sohio   Type of Test Standard   No. T/6 75B154   Tested by CDG   Date 4/19/86

Soil Type Arctic Silt   Location Smith Bay Site T   Sample Height 1.991 cm

Silty Clay (CL)   Boring 5B1-54, RE = 10.4 ft.   Sample Diameter 6.35 cm

Initial w(%) 23.1 G_s 2.78 w_N(%) 22.9 w_L(%) 39.5   Corrections Apparatus Compressibility

Void Ratio e 0.659   S(%) 97.4 w_p(%) 19.0 I_p(%) 20.5   Units: σ'_{vc} KSC c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}	Primary			Total			$C_{\alpha e}$(%)	Coef. of Consol.		Remarks
	t (hr)	ϵ_v(%)	e	t (hr)	ϵ_v(%)	e		$\sqrt{t}$	log t	Avg k ($\times 10^{-9}$ cm/s.)
0.057	—	—	—	0.1	0.01	0.659				
0.086	—	—	—	0.3	0.02	0.659				
0.143	—	—	—	0.8	0.05	0.658				
0.271	1.2	0.29	0.654	13.7	0.31	0.654		0.990	0.470	0.138
0.514	1.3	0.71	0.647	5.9	0.76	0.640	0.07	0.867	0.339	0.106
1.0	1.0	1.66	0.631	4.3	1.82	0.629	0.31	0.605	0.418	0.103
1.98	0.9	3.19	0.606	13.9	3.38	0.603	0.21	0.533	0.449	7.72
3.94	0.5	4.92	0.577	3.6	5.11	0.574	0.34	0.650	0.548	5.82
7.85	0.5	7.18	0.540	43.1	7.67	0.532	0.40	0.735	0.652	4.24
1.98	0.3	6.92	0.544	2.9	6.87	0.545				
0.514	1.0	5.44	0.568	4.1	5.35	0.570				
1.98	0.4	5.96	0.560	16.3	6.03	0.559		0.885	0.725	2.81
7.85	0.25	7.45	0.527	5.8	8.08	0.525	0.16	1.346	1.082	4.45
15.68	0.3	9.92	0.494	1.9	10.23	0.489	0.56	1.001	0.849	2.64
28.27	0.3	12.30	0.455	38.4	12.95	0.446	0.56	0.951	0.949	2.01
7.85	0.2	12.18	0.457	1.6	12.11	0.458				
3.44	0.5	11.3	0.471	1.8	11.28	0.472				

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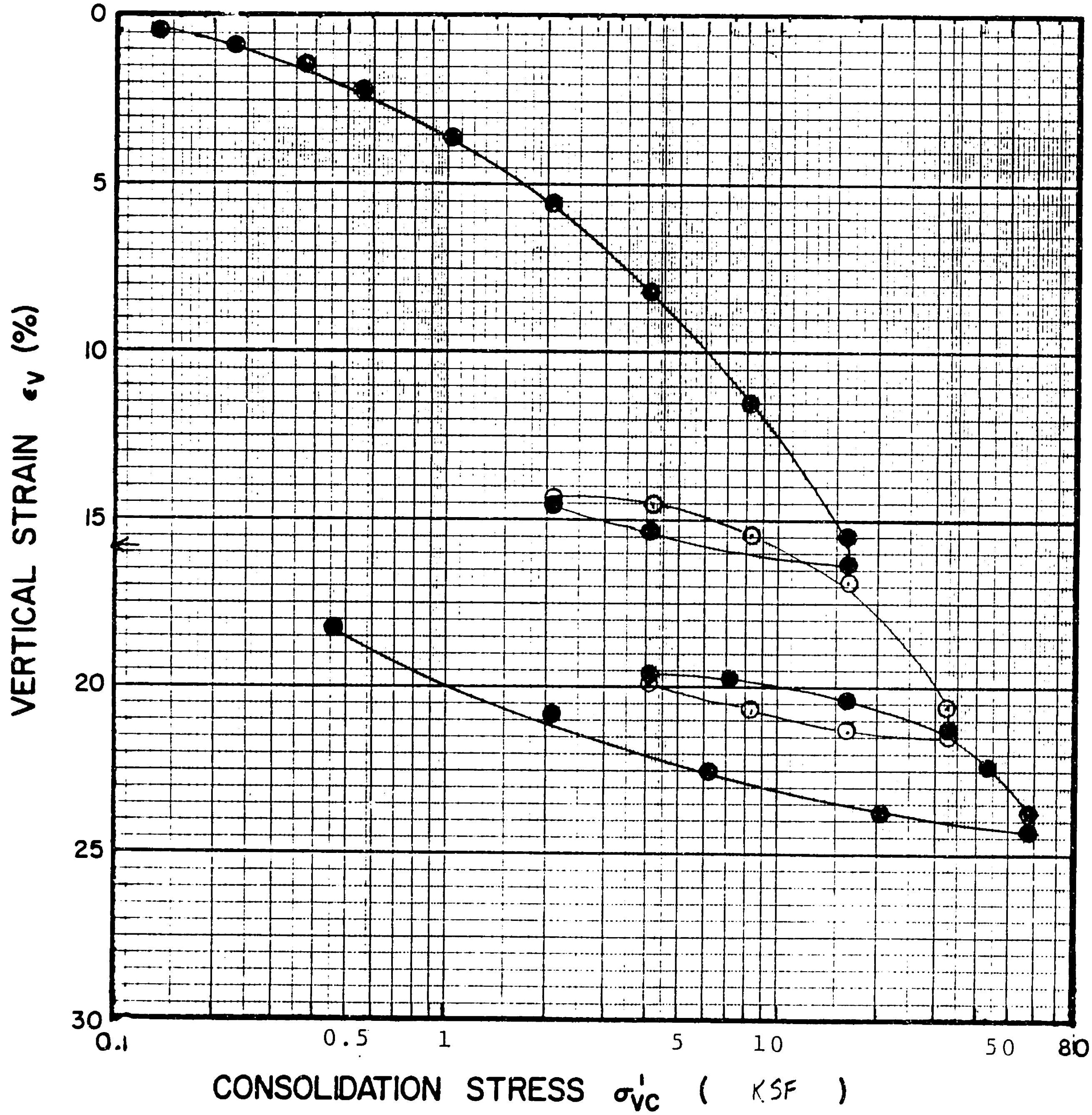
M.I.T.

Remarks KSC × 2.04818 = KSF



Sample No. TB1-02

Depth 6.8 Ft

Soil Type ARCTIC SILT - SILTY CLAY (CL)

w_N (%) 37.6

w_L (%) 52.7

w_P (%) 27.6

I_P (%) 25.1

Estimated

σ'_{vo} 0.237 σ'_p 3.9 ± 0.6

CR 0.115 RR 0.025

G_s 2.78 e_o 1.661 S(%) 100

○ At ~~t_p or ——~~ hr 20°C

● At ~~(—) hr~~ 1.4°C

Remarks • Corrected for apparatus compressibility

• Temperature Controlled

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COMPRESSION CURVE

TEST NO. T17 (aka. TBICON1)

FIGURE

CONSOLIDATION TEST

Project Sohio Type of Test TEMP CONTROLLED No. T17 (TBICON) Tested by CDG Date 1/24/86

Soil Type Arctic Silt - Silty Clay-(CL) Location Smith Bay - Site T, Boring IB-02, RE = 7.3' Sample Height 1.979 CM

Sample Diameter 6.35 CM

Initial w(%) 37.6 G_s 2.78 w_N(%) 41.5 w_L(%) 52.7 Corrections Apparatus Compressibility

Void Ratio e 1.061 S(%) 97.4 w_P(%) 27.6 I_P(%) 25.1 Units: σ'_{vc} KSC c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}	Primary t(hr)	Primary ϵ_v(%)	Primary e	Total t(hr)	Total ϵ_v(%)	Total e	$C_{\alpha e}$(%)	Coef. of Consol. $\sqrt{t}$	Coef. of Consol. log t	Remarks
										TEMP. = 1.4°C
0.066	—	—	—	1.1	0.45	1.052		—	—	
0.110	—	—	—	0.5	0.79	1.044		—	—	AVG. K
0.180	[illegible]	1.49	1.030	14.1	1.70	1.026		0.297	0.228	5.61 × 10^{-8} cm/s
0.266	1.5	2.18	1.016	[illegible]	2.43	1.010		0.236	0.186	1.72 "
0.509	[illegible]	3.51	0.989	21	3.95	0.980		0.248	0.215	1.29 "
1.01	1.0	5.62	0.945	8.8	6.02	0.937	0.682	0.379	0.278	1.48 "
1.98	0.9	8.16	0.893	14	8.75	0.880	1.091	0.361	0.319	9.41 × 10^{-9} cm/s
3.93	1.4	11.5	0.824	[illegible]	12.0	0.813	1.169	0.377	0.342	6.79 "
7.86	1.0	15.5	0.742	[illegible]	16.3	0.726	0.974	0.429	0.347	4.59 "
1.98	0.5	15.3	0.745	10.1	15.2	0.748				
1.01	1.2	[illegible]	[illegible]	2.5	14.3	0.765				INCREASE TEMP TO 20°C
		↓	↓		↓	↓				↓
1.01	—	14.5	0.762	20	14.3	0.765				
2.02	[illegible]	14.5	0.762	1.2	14.6	0.761	0.132	0.983	[illegible]	4.75 × 10^{-9} cm/s
4.03	0.3	15.4	0.745	9	15.6	0.740	0.273	1.249	0.984	4.97 "
7.86	0.4	16.8	0.715	15.5	17.3	0.705	0.655	0.684	0.571	3.64 "
16.04	0.6	20.6	0.636	[illegible]	21.5	0.619	0.974			

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR
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Remarks KSC × 0.024618 - KSF

CONSOLIDATION TEST

Project Sohio   Type of Test TEMP CONTROLLED   No. T17 (TBICON1)   Tested by CDG   Date 1/24/86

Soil Type ARCTIC SILT   Location SMITH BAY - SITE 7   Sample Height 1.979 CM

SILT CLAY (CL)   BORING IB-02, RE = 7.3'   Sample Diameter 6.35 CM

Initial w(%) 37.6 G_s 2.78 w_N(%) 41.5 w_L(%) 52.[illegible]   Corrections Apparatus Compressibility

Void Ratio e 1.061   S(%) 97.4 w_P(%) 27.6 I_P(%) 25.1   Units: σ'_{vc} KSC c_v $\times 10^{-3}$ cm^2/sec

σ'_{vc}	Primary t (hr)	Primary ϵ_v(%)	Primary e	Total t (hr)	Total ϵ_v(%)	Total e	$C_{\alpha e}$(%)	Coef. of Consol. $\sqrt{t}$	Coef. of Consol. log t	Remarks
8.03	0.1	21.3	0.623	2.5	21.2	0.623				
4.03	0.3	20.7	0.634	16.5	20.5	0.637				DECREASE TEMP TO 1.4°C
2.02	0.8	19.9	0.651	2.4	19.6	0.650				
		↓			↓					
2.02	—	19.9	0.651	10	19.6	[illegible]				
4.03	0.2	19.7	0.656	2.8	19.7	0.654	0.078	1.149	0.991	1.74 × 10^{-9} cm/s
8.03	0.3	20.3	0.643	3	20.4	0.640	0.156	0.905	0.824	1.73 "
16.0	0.2	21.3	0.622	2	21.5	0.618	0.320	1.033	0.960	1.59 "
21.0	1.3	22.4	0.600	15.4	22.7	0.592	0.417	0.570	0.327	12.3 "
28.0	1.3	23.8	0.570	25.0	24.3	0.559	0.680			
10.04	0.2	23.8	0.570	2.6	23.8	0.570				
3.02	0.4	22.5	0.597	3.2	22.3	0.602				
1.02	1.8	20.8	0.633	14.5	20.4	0.640				
0.223	6.7	18.3	0.684	26	17.8	0.694				
0.047	—	—	—	21	15.9	0.734				

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR
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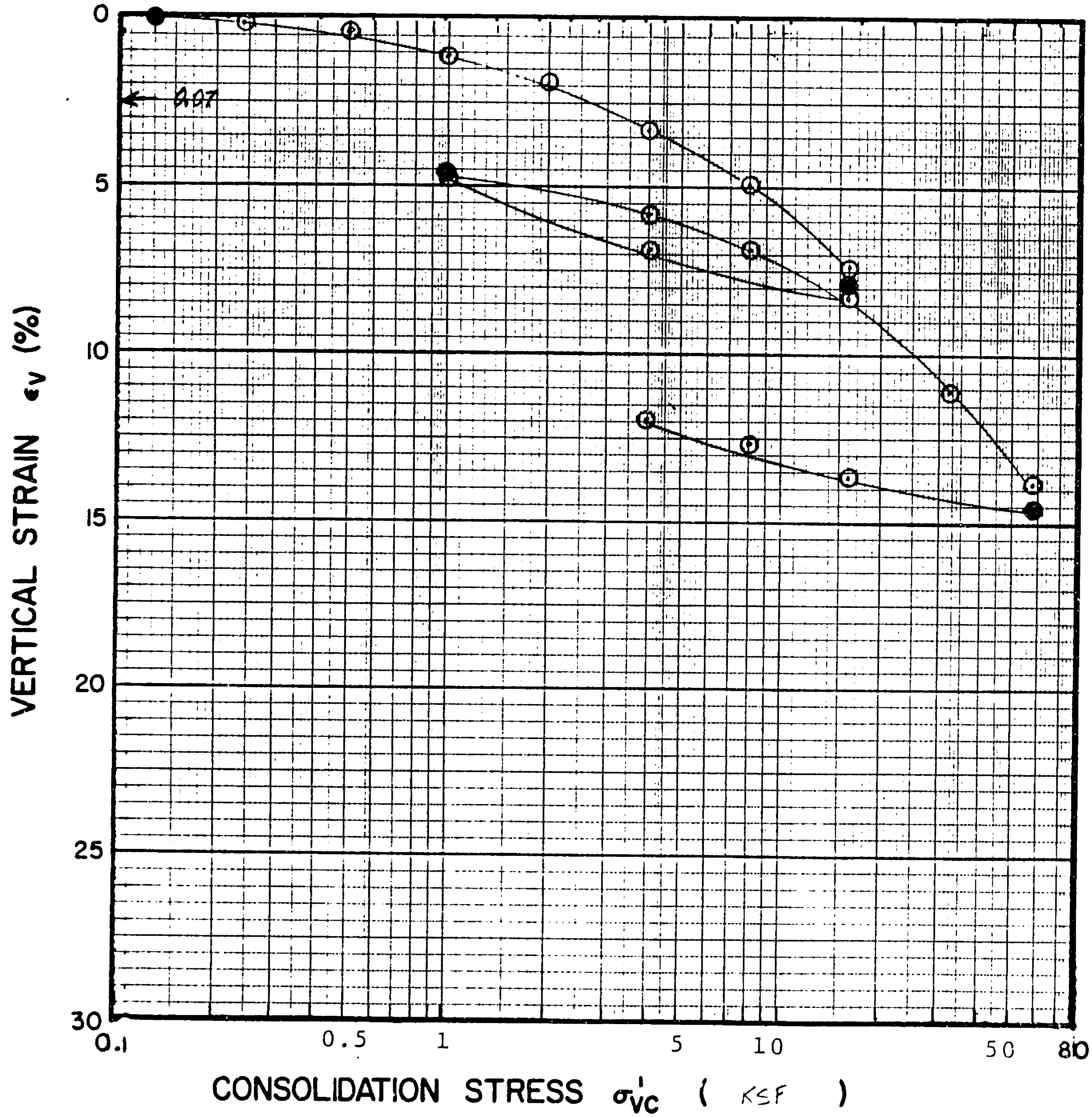
Remarks KSC × 0.524[illegible] = KSF



Sample No. 5B1-S4

Depth (RE) 11'

Soil Type ARCTIC SILT - SILTY CLAY (CL)

w_N (%) 23.3

w_L (%) 39.5

w_P (%) 19.0

I_P (%) 20.5

Estimated

σ'_{vo} 0.605 σ'_p 7.5 ± 1.3

CR 0.111 RR 0.028

G_s 2.78 e_0 0.705 S(%) 92.0

○ At t_p or hr

● At (t_f) hr

Remarks Corrected for apparatus Compression %.

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COMPRESSION CURVE

TEST NO. T18 (aka T5B1S4A)

FIGURE

CONSOLIDATION TEST

Project Sohio Type of Test Standard No. T18 (15B1C4A) Tested by GY Date 3/15/86

Soil Type Arctic Silt Location Smith Bay Site T Sample Height 2.057 cm

Silty Clay (CL) [illegible] 5B1-54, RE = 11' Sample Diameter 6.35 cm

Initial w(%) 23.8 G_s 2.78 $w_N(\%)$ 22.3 $w_L(\%)$ 39.5 Corrections Apparatus Compressibility

Void Ratio e 0.705 S(%) 93.8 $w_P(\%)$ 19.0 $I_P(\%)$ 20.5 Units: σ'_{vc} KSC/KSF c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}		Primary			Total			$C_\alpha(\%)$	Coef. of Consol.		Remarks
		t (hr)	$\epsilon_v(\%)$	e	t (hr)	$\epsilon_v(\%)$	e		$\sqrt{t}$	log t	
KSC	KSF										Avg K ($\times 10^{-9}$ cm/sec)
0.06	0.13	—	—	—	0.08	0.01	0.705				
0.12	0.24	0.075	0.18	0.702	0.15	0.18	0.702				
0.24	0.5	0.03	0.41	0.697	[illegible]	0.48	0.697	0.03			
0.49	1.0	0.32	1.07	0.687	2.6	1.10	0.687	—	1.94	1.14	36.5
0.98	2.0	0.4	1.76	0.673	1.5	2.05	0.670	0.20	1.85	1.04	26.7
1.96	4.0	0.55	3.33	0.648	22.3	3.51	0.645	0.12	1.57	0.88	18.5
3.91	8.0	0.57	4.95	0.621	1.97	5.06	0.619	0.185	1.00	0.94	8.3
7.82	16.0	0.375	7.36	0.580	24.5	7.89	0.571	0.509	1.07	0.96	6.3
1.96	4.0	0.13	6.83	0.589	2.80	6.75	[illegible]		1.90	1.66	1.73
0.49	1.0	0.67	4.73	0.625	13.0	4.53	[illegible]		0.349	0.343	5.43
1.96	4.0	0.38	5.81	0.606	3.0	5.86	0.605	0.099	0.762	0.946	6.87
3.91	8.0	0.25	6.87	0.588	2.3	6.93	0.587	0.074	1.80	1.09	8.34
7.82	16.0	0.27	8.23	0.565	1.9	8.34	[illegible]	0.153	1.17	1.60	5.19
15.64	32.0	0.43	11.06	0.517	1.97	11.18	[illegible]	0.247	1.19	1.67	5.67
28.37	58.1	0.45	13.76	0.471	58.7	14.54	[illegible]	0.309	1.00	1.07	2.49

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Remarks KSC × 2.04816 = KSF

CONSOLIDATION TEST

Project Sohio Type of Test Standard No. T18 (T8 315-A) Tested by GY Date 3/5/36

Soil Type Arctic Silt Location Smith Bay - Site T Sample Height 2.057 CM

Silty Clay (CL) Boring SBI-54, RE = 11' Sample Diameter 6.35 CM

Initial w(%) 23.8 G_s 2.78 w_N(%) 22.3 w_L(%) 39.5 Corrections Apparatus Compressibility

Void Ratio e 0.705 S(%) 93.8 w_p(%) 19.0 I_p(%) 20.5 Units: σ'_{vc} KSC/KSF c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}		Primary			Total			C_α(%)	Coef. of Consol.		Remarks
		t (hr)	ϵ_v(%)	e	t (hr)	ϵ_v(%)	e		$\sqrt{t}$	log t	
KSC	KSF										
7.82	16.0	0.22	13.57	0.414	4.9	13.49	0.475		2.87	2.21	0.26
3.91	8.0	0.75	12.59	0.4[illegible]1	24.7	12.43	0.493		0.584	0.584	1.69
1.77	4.0	1.5	11.46	0.510	26.3	11.25	0.51[illegible]		0.302	0.299	2.20
.03	0.07	—	—	—	21.3	2.52	0.662		—	—	

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DEPT. OF CIVIL ENGR
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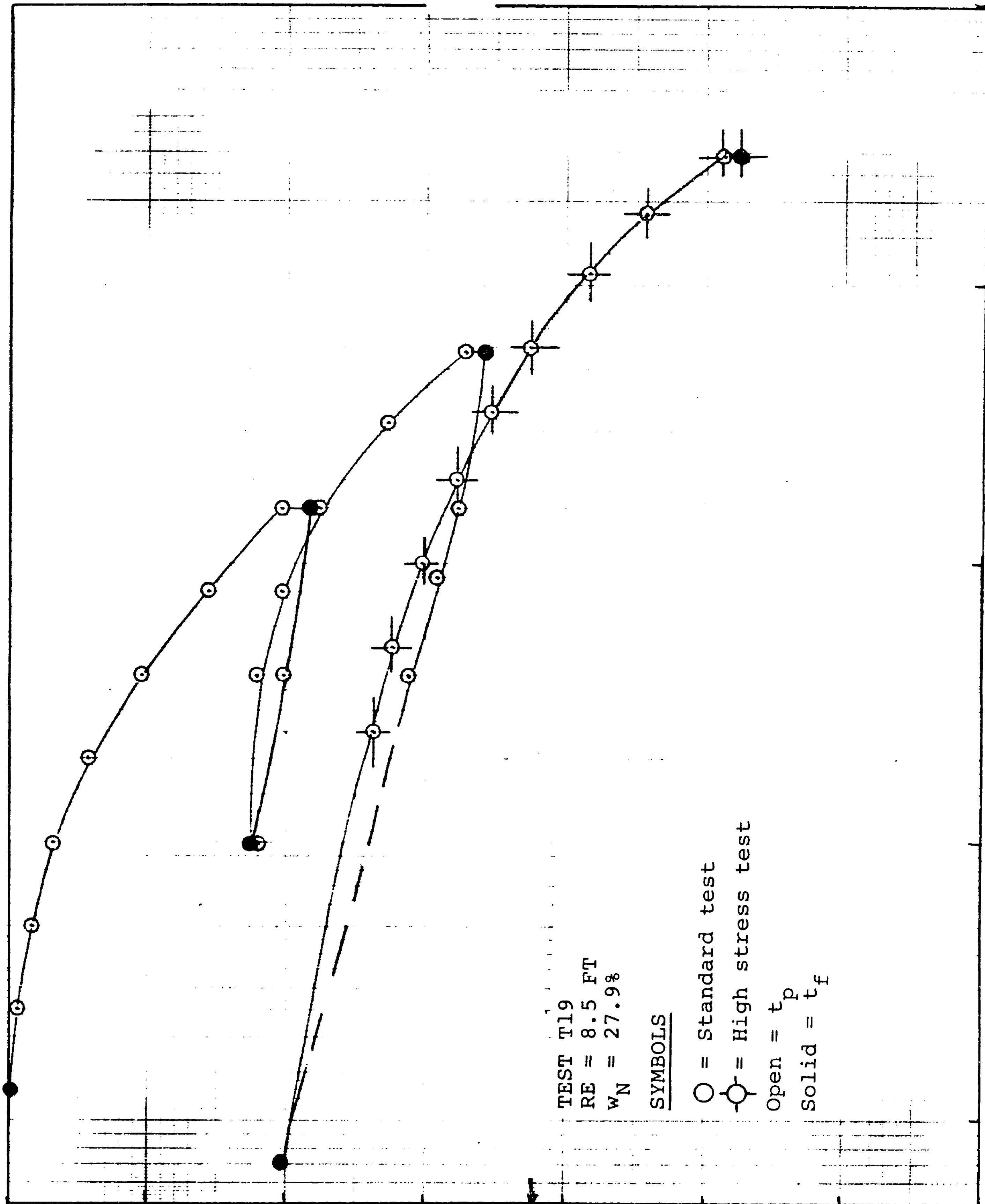
Remarks KSC × 2.04816 = KSF


TEST T19
RE = 8.5 FT
w_N = 27.9%
SYMBOLS
○ = Standard test
-○- = High stress test
Open = t_p
Solid = t_f
0
5
10
15
20
25
30
35
ε_v(%)

CONSOLIDATION TEST

Project SOLID Type of Test [illegible] No. T19 (TP3) Tested by GY Date 9/10/86

Soil Type ARCTIC SILT Location [illegible] Sample Height 1.998 CM

SILTY CLAY (CL) [illegible] -P3, EL. 8.5' Sample Diameter 6.35 CM.

Initial w(%) 27.4 G_s 2.75 w_N(%) [illegible] w_L(%) 45.2 Corrections Apparatus Compressibility

Void Ratio e 0.7559 S(%) 100 w_P(%) [illegible] I_P(%) 21.5 Units: σ'_{vc} KSC c_v x 10^{-3} cm²/sec

σ'_{vc}	Primary t (hr)	Primary ϵ_v(%)	Primary e	Total t (hr)	Total ϵ_v(%)	Total e	C_α(%)	Coef. of Consol. $\sqrt{t}$	Coef. of Consol. log t	Remarks
										Avg. K ($\times 10^{-9}$ cm/sec)
0.06	—	—	—	0.15	0.07	0.755				
0.12	0.08	0.33	0.751	0.12	0.33	0.750				
0.24	0.5	0.79	0.742	1.5	0.80	0.742				
0.49	0.6	1.58	0.729	1.3	1.66	0.727				
0.98	0.6	2.90	0.705	2.0	2.99	0.704		0.539	0.695	16.9
1.96	0.6	4.80	0.672	17.2	5.14	0.666	0.32	0.774	0.605	13.9
3.91	0.6	7.22	0.629	3.3	7.51	0.624	0.36	0.614	0.679	7.73
7.81	0.4	9.91	0.582	49	10.84	0.566	0.18	0.735	0.911	6.21
1.96	0.3	9.82	0.584	3.0	9.75	0.585				
0.49	1.3	8.02	0.615	16.9	7.80	0.619				
1.96	0.7	8.93	0.600	6.2	8.93	0.599	0.03	0.612	0.688	4.44
3.91	0.3	9.84	0.584	23.0	9.91	0.581	0.06	1.065	1.196	5.83
7.81	0.3	11.16	0.560	18.0	11.41	0.556	0.18	1.254	1.049	4.33
15.64	0.3	13.56	0.518	31.2	14.18	0.507	0.31	0.993	0.966	3.44
28.36	0.3	16.35	0.469	72	17.08	0.456	0.36	1.129	1.124	2.91

Remarks KSC x 2.04816 = KSF

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR
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CONSOLIDATION TEST

Project Sohio Type of Test Standard No. T19 (TP3) Tested by SY Date 4/10/86

Soil Type Arctic Silt - Silty Clay (CL) Location Smith Bay - Site 7 Boring 581-E3, RF: 8.5' Sample Height 1.998 cm

Sample Diameter 6.35 cm

Initial w(%) 27.9 G_s 2.75 w_N(%) 33.8 w_L(%) 45.2 Corrections Apparatus Compressibility

Void Ratio e 0.7559 S(%) 100 w_p(%) 23.7 I_p(%) 21.5 Units: σ'_{vc} KSC c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}	Primary t (hr)	Primary ϵ_v(%)	Primary e	Total t (hr)	Total ϵ_v(%)	Total e	C_α(%)	Coef. of Consol. $\sqrt{t}$	Coef. of Consol. log t	Remarks
7.81	0.22	16.19	0.472	96	16.05	0.474				AVG. k ($\times 10^{-9}$ cm/s)
3.91	0.5	15.37	0.486	72	15.17	0.490				
1.96	1.2	14.37	0.504	38	14.13	0.508				
0.04	13	—	—	26	7.06	0.632				
1.21	0.8	13.17	0.525	2.5	13.18	0.525				
2.45	0.5	13.84	0.513	1.[illegible]	13.89	0.512		0.862	0.701	4.94
4.89	0.3	14.92	0.494	4.0	15.71	0.480	0.09	0.586	0.565	2.99
9.16	0.4	16.21	0.472	14.9	17.22	0.454	0.05	0.820	0.687	6.10
17.09	0.3	17.29	0.453	2.0	18.53	0.431	0.06	0.841	0.452	1.13
29.3	0.3	18.70	0.428	2.6	18.83	0.426	0.14	1.053	0.800	1.29
53.7	0.2	20.69	0.393	18.9	21.14	0.385	0.30	1.159	1.055	1.10
87.9	0.2	22.87	0.355	18.2	23.45	0.344	0.28	2.144	0.996	1.50
143.3	0.3	25.64	0.306	24.8	26.16	0.297	0.2[illegible]	0.480	0.743	0.41
0.04	—	—	—	25	18.95	0.423				

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR
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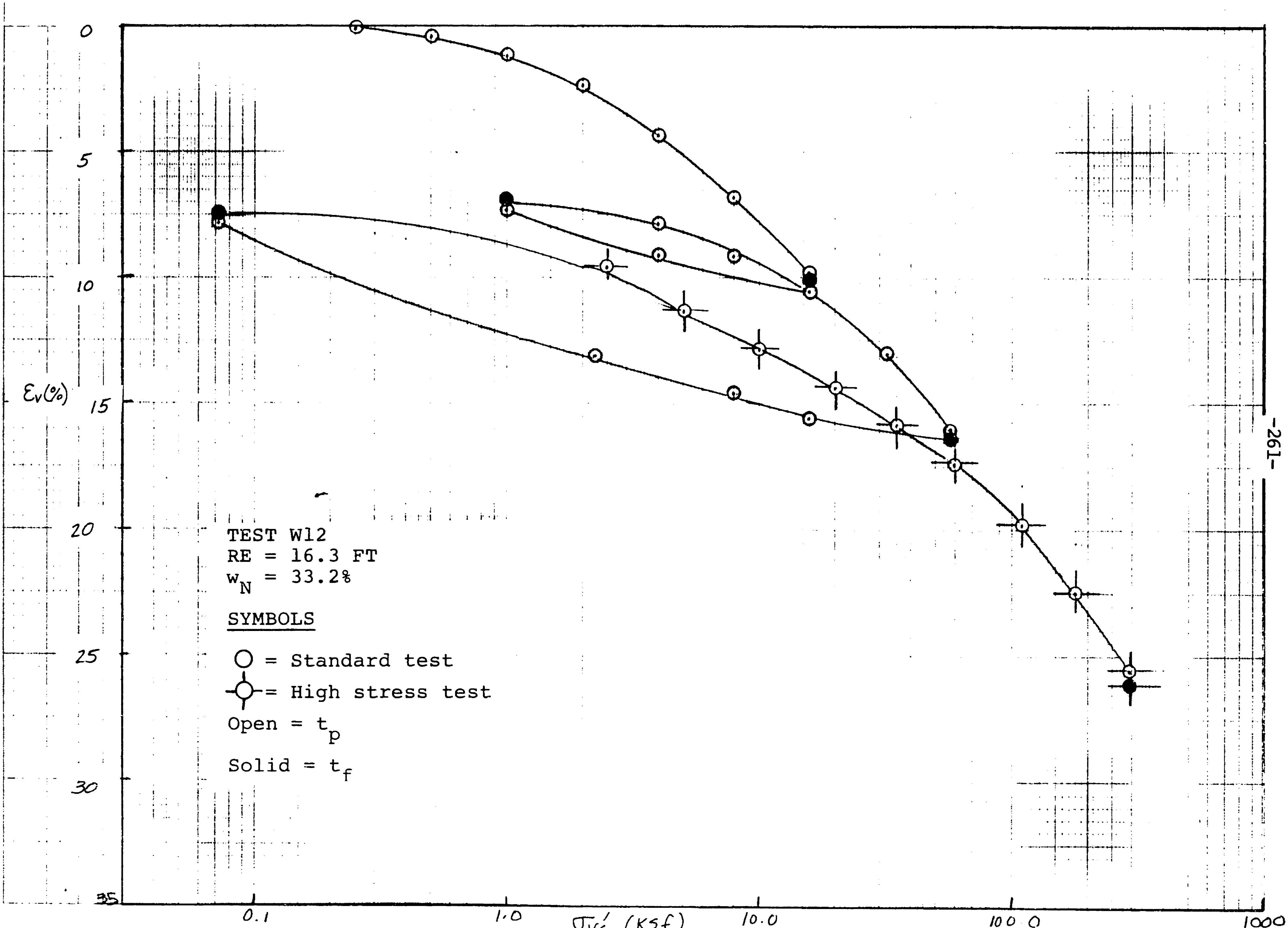
Remarks KSC × 2.0481 = KSF


TEST W12
RE = 16.3 FT
w_N = 33.2%
SYMBOLS
= Standard test
= High stress test
Open = t_p
Solid = t_f
ε_v(%)
0
5
10
15
20
25
30
35
0.1
1.0
10.0
100.0
1000

CONSOLIDATION TEST

Project Solid   Type of Test STD INC CELL   No. W12 (WS6)   Tested by CCG/SI   Date 5/24 86

Soil Type Arctic Silt   Location Smith Bay - Site W   Sample Height 2.021 CM.

Silty Clay (CL)   Boring 55-86, RE = 10.3 FT.   Sample Diameter 6.35 CM

Initial w(%) 28.8 G_s 2.75 w_N(%) 33.2 w_L(%) 48   Corrections Apparatus Compressibility

Void Ratio e 0.817   S(%) 96.9 w_P(%) 26 I_P(%) 22   Units: σ'_{vc} KSC c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}	Primary t(hr)	Primary ϵ_v(%)	Primary e	Total t(hr)	Total ϵ_v(%)	Total e	C_α(%)	Coef. of Consol. $\sqrt{t}$	Coef. of Consol. log t	Remarks
0.06		—	—		—	—				Average k
0.12		0.09	0.815		0.09	0.815				
0.24		0.38	0.810		0.41	0.808				
0.49		1.17	0.796		1.17	0.796				
0.98		2.42	0.773		2.53	0.771				($\times 10^{-9}$ cm/s)
1.96		4.42	0.736		4.62	0.733	0.338	0.538	0.500	5.62
3.91		6.80	0.693		7.12	0.687	0.362	0.569	0.702	3.71
7.81		9.76	0.639		10.2	0.632	0.202	0.799	0.941	3.66
1.96		9.10	0.651		9.03	0.653				
0.49		7.30	0.684		6.96	0.690				
1.96		7.87	0.674		7.96	0.672				
3.91		9.16	0.650		9.19	0.650				
7.81		10.5	0.626		10.71	0.622	0.213	1.15	1.09	2.05
15.6		13.0	0.580		13.4	[illegible]	0.151	1.43	0.869	2.15
28.4		16.1	0.524		16.4	0.521	0.201	1.03	0.813	1.19

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Remarks KSC × [illegible] = TSF

CONSOLIDATION TEST

Project Sohio Type of Test Standard No. W12 (WSb) Tested by [illegible] Date [illegible]

Soil Type Arctic Silt Location Smith Bay - Site W Sample Height 2.021 cm

Silty Clay (CL) Boring WEB-56, [illegible] FT Sample Diameter 6.35 cm

Initial w(%) 28.8 G_s 2.75 w_N(%) 33.2 w_L(%) 48 Corrections Apparatus Compressibility

Void Ratio e 0.817 S(%) 96.9 w_P(%) 26 I_P(%) 22 Units: σ'_{vc} KSC c_v $\times 10^{-3}$ cm²/sec

σ'_{vc}	Primary			Total			C_α(%)	Coef. of Consol.		Remarks
	t (hr)	ϵ_v(%)	e	t (hr)	ϵ_v(%)	e		$\sqrt{t}$	log t	
7.81		15.6	0.533		15.5	0.535				Avg k ($\times 10^{-8}$ cm/sec)
3.91		14.6	0.551		14.4	0.555				
1.96		13.2	0.578		12.3	0.594				
0.04		7.78	0.675		7.38	0.671				
1.21		9.61	0.642		9.77	0.639	0.318	0.570	0.350	10.0
2.45		11.3	0.611		11.5	0.607	0.141	0.411	0.327	2.96
4.89		12.8	0.585		12.9	0.582	0.283	0.714	0.575	2.02
9.76		14.4	0.555		14.7	0.550	0.141	0.708	0.667	3.62
17.1		15.8	0.529		15.9	0.528	0.106	1.32	1.10	1.18
29.3		17.4	0.501		17.6	0.497	0.144	1.24	1.33	0.85
53.7		19.9	0.454		20.3	0.447	0.194	1.10	1.14	0.68
87.9		22.5	0.385		23.1	0.397	0.389	0.987	0.898	1.75
143.3		25.6	0.352		26.2	0.341	0.318	1.17	0.764	0.16

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Remarks KSC × 2.04816 = KSF

Appendix D
TRIAXIAL TESTS

The tabulated results and figures for each of the triaxial tests performed on the Smith Bay Arctic Silts from the 1985 test program are presented in the succeeding pages. These data are presented in the following order:

(1) CK_0UC test;

(2) CIUC tests;

(3) Recompression tests.

For each test four pages of tabulated data are provided which include a summary of index parameters, test conditions (where recorded), and consolidation and shearing results. Plots include stress, A-parameter, obliquity and pore pressure changes versus axial strain for each test. Stress paths and modulus plots are contained in Chapter 5.

CK_0UC TEST

T5B1T5

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO
SOIL TYPE ARCTIC SILT
LOCATION SITE T
BORING NO. TB1 SAMPLE NO. TS
DEPTH 1.12'

TEST NO. TSB1TS
TYPE OF TEST CK₀UC
APPARATUS NO. WFE 8
TESTED BY CDG
DATE 02/09/86

WATER CONTENT

INITIAL, BASED ON TRIMMINGS 43.9 %
INITIAL, BASED ON SAMPLE – %
FINAL, BASED ON SAMPLE 30.2 %

ATTERBERG LIMITS

W_p 25.6 %
W_L 49.5 %
I_p 23.9 % I_L

PHASE RELATIONSHIPS

γWET 1.778 g/cc. γDRY 1.236 g/cc.
e_i 1.174 e_f 0.823
S_i 97.4 % $S_{precons}$ 100 %
G_s 2.687

MISCELLANEOUS

B 100 %
SATURATION ΔV 1.12 c.c.
CONSOLIDATION ΔV 12.69 cc. (16.11 %)
MEMBRANES – THICK .2 THIN
CORRECTION FACTOR 1.962 ×ε
FILTER STRIPS 8 X 1 1/4"
CONFIGURATION VERTICAL
CORRECTION FACTOR 40.64 × ε
AREA CORRECTION PARABOLIC

GRAIN SIZE

% -#200
% -2μ 48
C_u C_c

SAMPLE APPEARANCE

FINAL AREA
( )

TOP

MIDDLE

BOTTOM

AVERAGE

INITIAL, FRONT FINAL, FRONT FINAL, SIDE FINAL, X-RAY

COMMENTS

GEOTECHNICAL LABORATORY
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- M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CK0UC TEST NO. TSBITS

STRESS HISTORY

IN SITU CONDITIONS		TEST CONDITIONS	
σ'_{vo}	0.030 ksc	σ'_{vc}	3.199 ksc.
σ'_p	–	σ'_p	3.199 ksc.
OCR	–	OCR	1
		K_c	0.50
		U_b	1.999 ksc.
TORVANE STRENGTH	–	STRAIN RATE	0.5 %/HOUR
TORVANE W_c	– %	FINAL ε_a	(SHEAR) 6.06 %

STRENGTH DATA

AT MAXIMUM q		AT MAXIMUM OBLIQUITY	
ε_a	5.58 %	ε_a	5.77 %
q/σ'_{vc}	0.311	q/σ'_{vc}	0.310
p'/σ'_{vc}	0.648	p'/σ'_{vc}	0.648
$\Delta u - \Delta\sigma_h/\sigma'_{vc}$	0.163	$\Delta u - \Delta\sigma_h/\sigma'_{vc}$	0.163
ϕ'	28.6	ϕ'	28.6
A_f	2.69		
TIME TO q_{max}	11.2 HR.	$E_{u(50\%)}/q_f$	7.0

TIME RECORD		HYPERBOLIC STRESS-STRAIN PARAMETERS	
SET UP	2 HR.	G_i/σ'_{vc}	141
START OF CONSOLIDATION	02/16/86 20:30	R_f	1.00
START OF SHEAR	02/26/86 14:40	r^2	0.9999
END OF SHEAR	02/27/86 02:50		
REMOVAL		RADIOGRAPHY	
TOTAL TIME IN APPARATUS		kV	160
		mA	3.8
CONSOLIDATION-SHEAR Δt	30'	EXPOSURE TIME	45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CK0UC TEST NO. 75B1T5

SAMPLE DIMENSIONS

	L (cm)	A (cm²)	V (cm³)	ε_a (b) (%)	ε_{vol} (b) (%)	W (g)
INITIAL	8.024	9.820	78.79	-	—	140.09
PRECONSOLIDATION	8.024	9.820	78.79	—	—	141.12
PRESHEAR	6.715	9.844	66.10	16.31	16.11	128.17
POST SHEAR	6.308	10.479	66.10	6.06	—	128.17
FINAL	6.308	10.479	66.10	21.39	16.11	128.17
FINAL MEASURED	6.339	—	—	21.00	—	126.21

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN KSC.

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}	0.113	0.188	0.384	0.803	0.946	1.268	1.483	1.810	2.217	2.680
σ'_{hc}	0.099	0.202	0.204	0.533	0.536	0.634	0.733	0.912	1.101	1.336
t_c (HRS)	13.8	50.0	27.7	17.5	6.3	20.4	23.1	23.8	3.6	20.0
ε_a (%)	0.36	1.10	5.01	7.33	7.95	10.66	11.49	12.50	13.29	15.09
ε_{vol} (%)	0.98	3.50	5.72	9.09	9.57	11.35	12.04	13.01	13.68	15.05
K_c	0.88	1.07	0.53	0.66	0.57	0.50	0.49	0.50	0.50	0.50

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}	3.224									
σ'_{hc}	1.602									
t_c (HRS)	27.0									
ε_a (%)	16.31									
ε_{vol} (%)	16.11									
K_c	0.50									

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

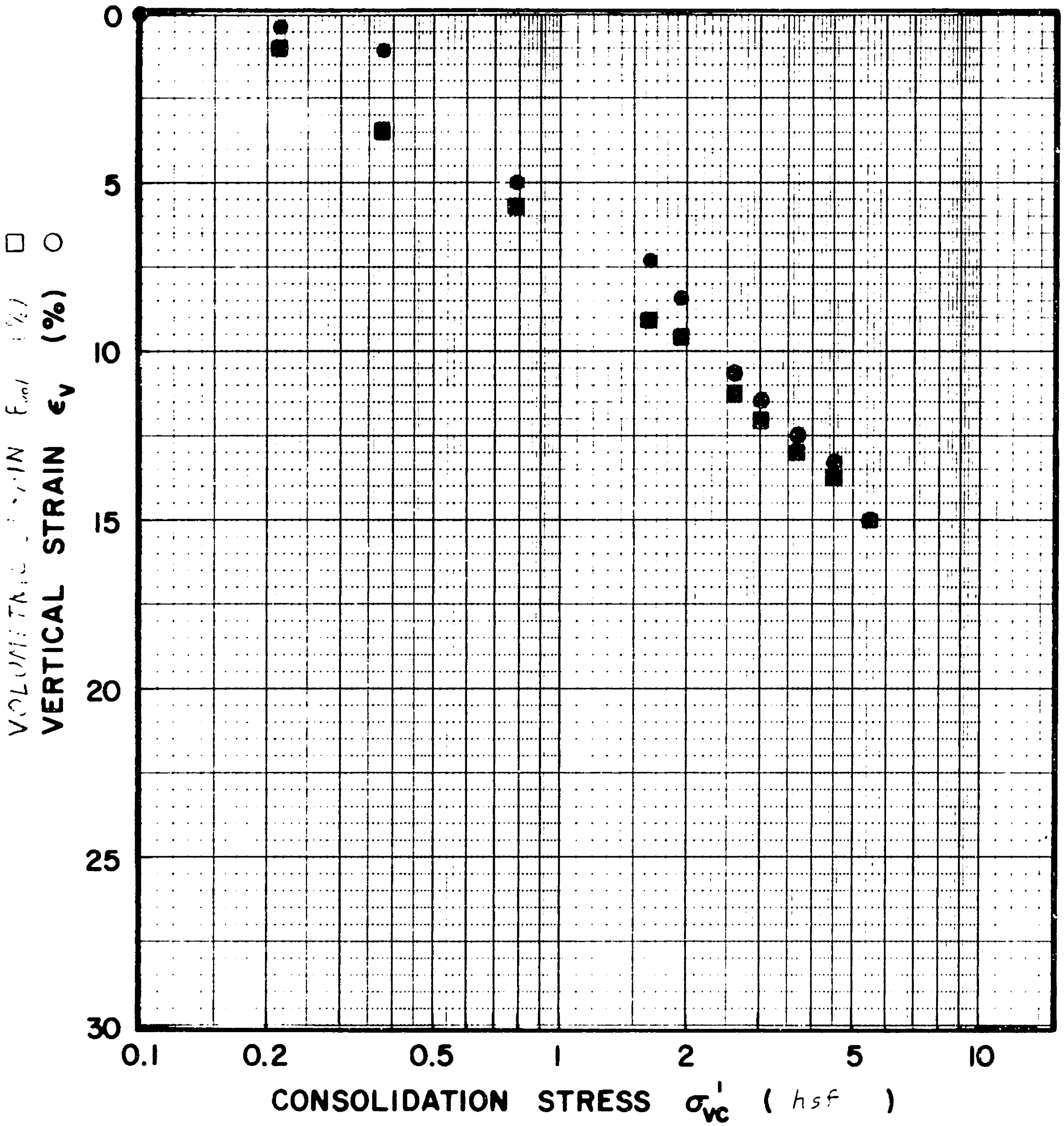
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED—UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CK_0UC No. TSB1TS

Time (hr)	ε_a (%)	$\frac{(\sigma_v-\sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u-\Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{q_f}$
	0.000	–	–	–	–	–	–	–	–
	0.025	0.563	0.032	2.201	0.491	0.282	0.751	817.2	0.526
	0.058	0.586	0.040	2.271	0.456	0.293	0.754	487.4	0.714
	0.107	0.599	0.046	2.316	0.456	0.300	0.755	300.7	0.819
	0.158	0.605	0.050	2.342	0.470	0.303	0.754	216.4	0.868
	0.202	0.609	0.053	2.360	0.485	0.305	0.752	174.7	0.898
	0.309	0.613	0.060	2.392	0.528	0.307	0.747	119.2	0.936
	0.408	0.615	0.067	2.417	0.575	0.308	0.742	91.6	0.951
	0.599	0.618	0.078	2.460	0.658	0.309	0.732	63.6	0.969
	0.803	0.618	0.088	2.497	0.739	0.309	0.722	47.7	0.975
	0.994	0.619	0.096	2.530	0.804	0.310	0.714	38.9	0.982
	1.441	0.618	0.112	2.590	0.944	0.309	0.698	26.5	0.973
	2.124	0.618	0.129	2.662	1.086	0.309	0.681	18.1	0.975
	2.746	0.619	0.140	2.717	1.181	0.310	0.670	14.0	0.978
	2.992	0.618	0.144	2.731	1.213	0.309	0.666	12.8	0.975
	3.245	0.618	0.147	2.747	1.238	0.309	0.663	11.8	0.976
	3.796	0.620	0.153	2.781	1.272	0.310	0.658	10.3	0.989
	4.042	0.619	0.154	2.787	1.300	0.310	0.656	9.5	0.979
	4.658	0.621	0.159	2.816	1.317	0.311	0.652	8.4	0.995
	4.902	0.620	0.160	2.820	1.342	0.310	0.650	7.9	0.986
	5.333	0.620	0.162	2.833	1.348	0.310	0.649	7.3	0.993
	5.582	0.621	0.163	2.840	1.347	0.311	0.648	6.9	1.000
	5.827	0.620	0.163	2.840	1.359	0.310	0.647	6.7	0.993
	6.076	0.619	0.164	2.837	1.384	0.310	0.646	6.3	0.980

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



Sample No. T5B1-P1(A) w_N(%) 43.9 Estimated

Depth(RE) 1.12 Ft w_L(%) ____ σ'_{vo} 0.061 σ'_p ____

Soil Type Smith Bay w_P(%) ____ CR ____ RR ____

Arctic Silt I_P(%) 23.9 G_s 2.687 e_o 1.174 S(%) 100

○ At t_p or ____ hr Remarks end of increment

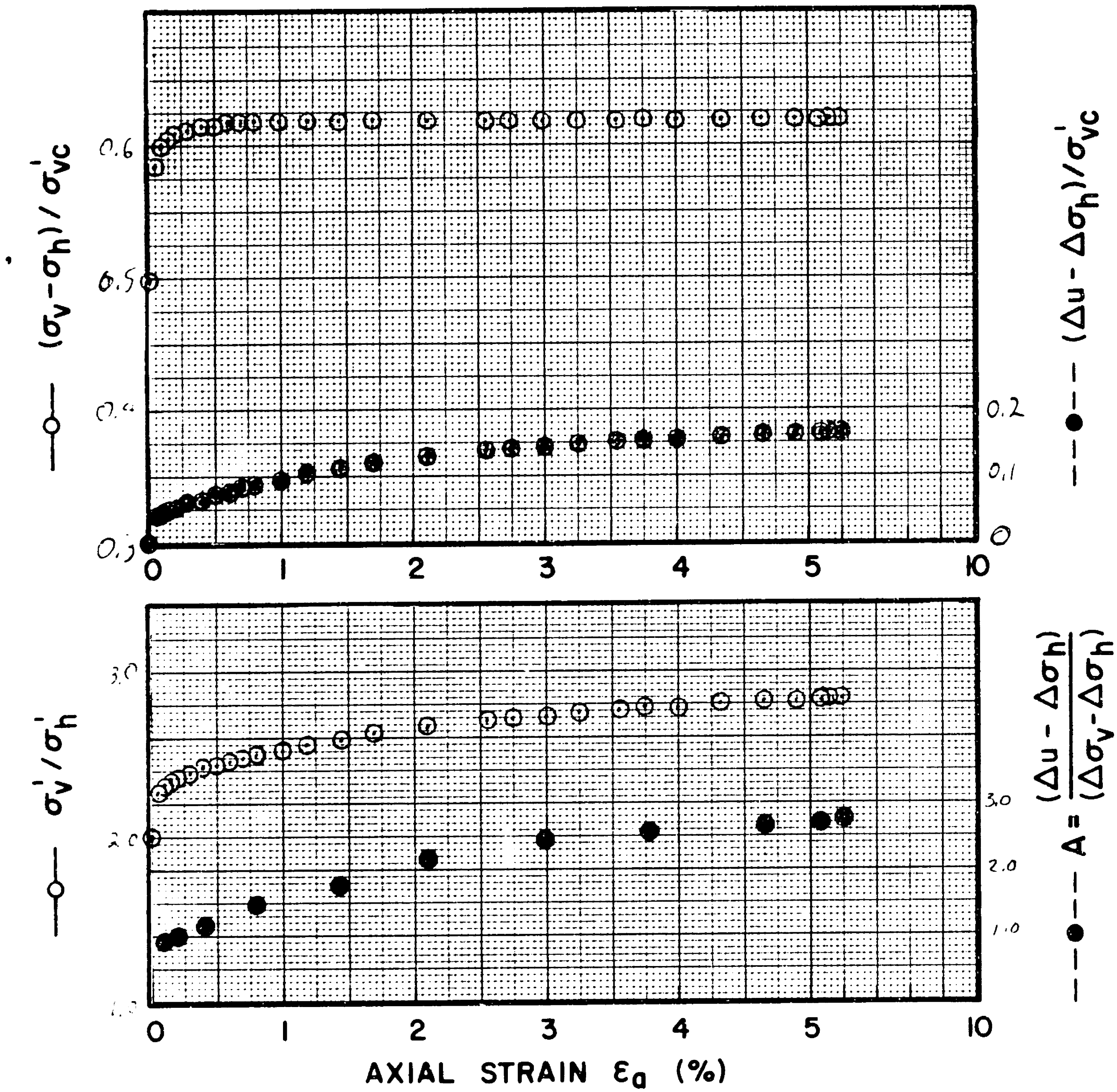
● At (t_f) hr

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. T5B1T5

FIGURE



Sample No. T5B1-P1(A) w_N(%) 43.9 σ'_{vc}(ksf) 6.4 K_c 0.50

Depth (RE) 1.12' w_L(%) ____ OCR 1.0 t_c(Days) ____

Soil Type Smith Bay Arctic Silt w_P(%) ____ Estimated σ'_{vo}(KSF) 0.061

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CK_oU TEST No. TEBIT5

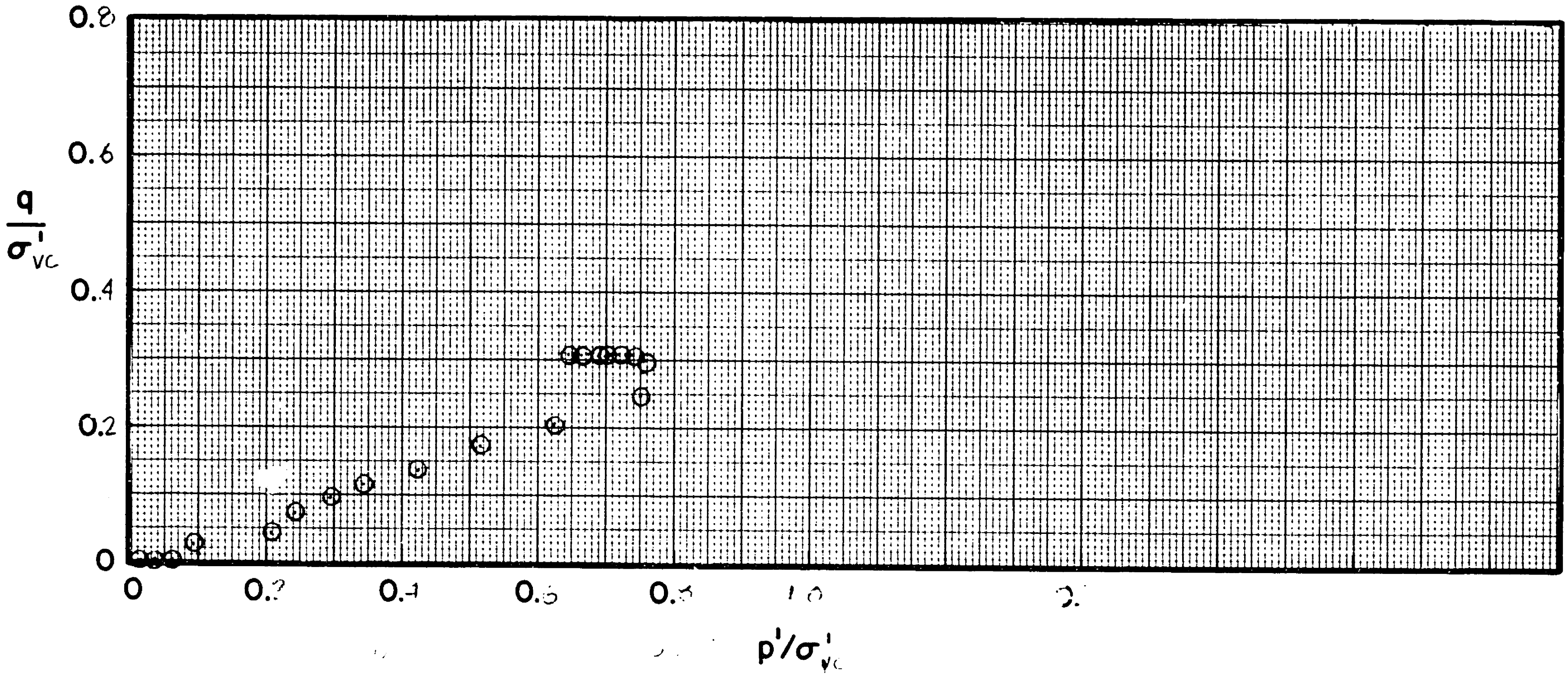
FIGURE

Test No.	Sample No.	Depth RE	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Sym.
T5B175 (K₁)	T5E1-P1 (A)	1.12'	43.9	6.4	0.50	1.0	⊙

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M. I. T.

$$q = 0.5(\sigma_v - \sigma_h)$$

$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



NORMALIZED STRESS PATHS FROM CK₀UC TESTS

BORING 5E1 SOIL TYPE Smith Bay-Arctic Silt

FIGURE

CIUC TESTS

T3BT1 T3BT2	Note: Not consolidated into Normally Consolidated Range
T5B1T4	OCR = 1.0
T5B1T6	OCR = 2.0
T5B1T7	OCR = 5.0

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO
SOIL TYPE ARCTIC SILT
LOCATION SITE T
BORING NO. 3B SAMPLE NO. T1
DEPTH 11.1'

TEST NO. T3BT1
TYPE OF TEST CIUC
APPARATUS NO. WFE4
TESTED BY CAQ
DATE 02/15/86

WATER CONTENT

INITIAL, BASED ON TRIMMINGS 21.2 %
INITIAL, BASED ON SAMPLE – %
FINAL, BASED ON SAMPLE 19.8 %

ATTERBERG LIMITS

W_P 21 %
W_L 40.6 %
I_P 19.6 % I_L

PHASE RELATIONSHIPS

γWET 2.078 g/cc. γDRY 1.715 g/cc.
e_i 0.560 e_f 0.480
S_i 98.2 % $S_{precons}$ 100 %
G_s 2.675

MISCELLANEOUS

B 95% %
SATURATION ΔV 0.52 cc.
CONSOLIDATION ΔV 4.21 cc. (5.11 %)
MEMBRANES – THICK 2 THIN
CORRECTION FACTOR 1942 E.
FILTER STRIPS 8 X 1/4"
CONFIGURATION VERTICAL
CORRECTION FACTOR 40.64 E
AREA CORRECTION PARABOLIC.

GRAIN SIZE

% -#200
% -2μ 42.5
C_u C_c

SAMPLE APPEARANCE

FINAL AREA ( )

INITIAL, FRONT | FINAL, FRONT | FINAL, SIDE | FINAL, X-RAY

TOP
MIDDLE
BOTTOM
AVERAGE

COMMENTS * NOT LOADED INTO NC RANGE *

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
- M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC TEST NO. T3BT1

STRESS HISTORY

IN SITU CONDITIONS		TEST CONDITIONS	
σ'_{vo}	0.285 KSC	σ'_{vc}	4.895 KSC.
σ'_p	–	σ'_p	4.895 KSC
OCR	–	OCR	1
		K_c	1.00
		U_b	2.016 KSC.
TORVANE STRENGTH	–	STRAIN RATE	0.5 %/HOUR
TORVANE W_c	– %	FINAL ε_a (SHEAR)	17.79 %

STRENGTH DATA

AT MAXIMUM q		AT MAXIMUM OBLIQUITY	
ε_a	15.7 %	ε_a	5.93 %
q/σ'_{vc}	0.680	q/σ'_{vc}	0.576
p'/σ'_{vc}	1.51	p'/σ'_{vc}	1.16
$\Delta u - \Delta\sigma_h/\sigma'_{vc}$	0.168	$\Delta u - \Delta\sigma_h/\sigma'_{vc}$	0.412
ϕ'	26.7	ϕ'	29.7
A_f	0.245		
TIME TO q_{max}	31.4 HR.	$E_{u(50\%)}/q_f$	285

TIME RECORD		HYPERBOLIC STRESS-STRAIN PARAMETERS	
SET UP		G_i/σ'_{vc}	22.6
START OF CONSOLIDATION	–	R_f	0.910
START OF SHEAR		r^2	0.997
END OF SHEAR			
REMOVAL		RADIOGRAPHY	
TOTAL TIME IN APPARATUS		kV 160	mA 38
CONSOLIDATION-SHEAR Δt		EXPOSURE TIME	45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO **TYPE OF TEST** CIUC **TEST NO.** T3BT1

SAMPLE DIMENSIONS

	L (cm)	A (cm^2)	V (cm^3)	ε_a (b) (%)	ε_{vol} (b) (%)	W (g)
INITIAL	8.066	10.204	82.31	—	—	171.06
PRECONSOLIDATION	8.066	10.204	82.31	—	—	171.58
PRESHEAR	7.896	9.891	78.10	2.11	5.11	167.26
POST SHEAR	6.491	12.032	78.10	17.79	—	167.26
FINAL	6.491	12.032	78.10	19.53	5.11	167.26
FINAL MEASURED	6.524	—	—	19.12	—	169.40

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN Ksc.

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}	0.511	0.999	1.993	3.441	4.895					
σ'_{hc}	0.503	0.999	1.999	3.441	4.897					
t_c (HRS)	30.2	18.8	40	18.5	24.8					
ε_a (%)	0.20	0.62	1.14	1.71	2.11					
ε_{vol} (%)	0.70	1.74	2.98	4.23	5.11					
K_c	0.98	1.00	1.00	1.00	1.00					

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

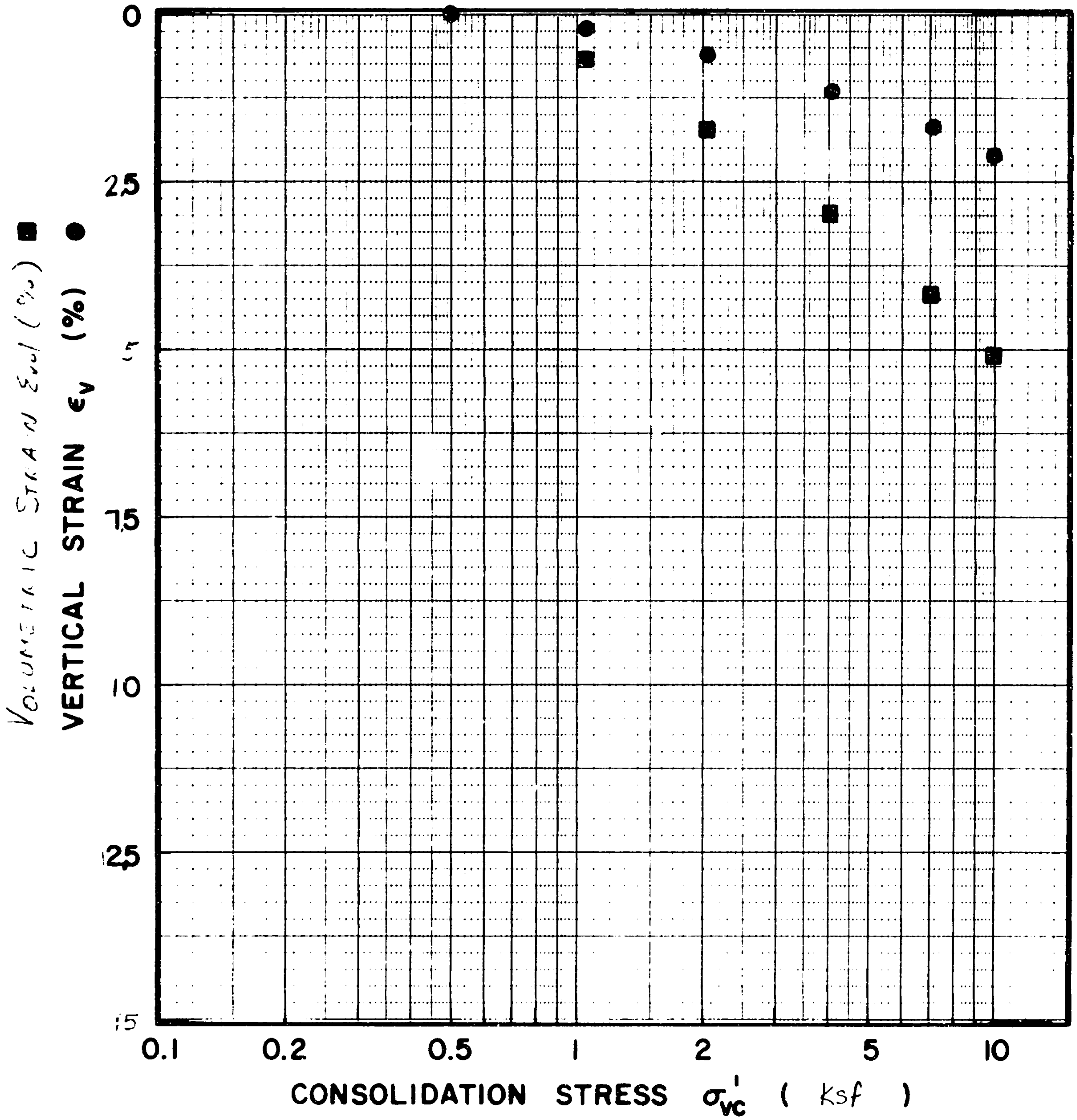
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED — UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CIUC No. T3BT1

Time (hr)	ε_a (%)	$\frac{(\sigma_v - \sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u - \Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{\Delta q_f}$
	0.000	–	–	–	–	–	1.000	–	–
	0.253	0.381	0.245	1.503	0.643	0.191	0.946	221.1	0.280
	0.497	0.494	0.329	1.735	0.665	0.247	0.919	146.3	0.363
	0.748	0.573	0.381	1.925	0.663	0.287	0.907	112.9	0.422
	0.992	0.633	0.414	2.078	0.653	0.317	0.903	93.8	0.465
	1.508	0.733	0.456	2.345	0.621	0.367	0.912	71.5	0.539
	2.000	0.808	0.474	2.534	0.586	0.404	0.930	59.5	0.594
	2.989	0.924	0.482	2.782	0.521	0.462	0.980	45.5	0.679
	4.027	1.020	0.467	2.911	0.457	0.510	1.043	37.2	0.749
	5.015	1.093	0.442	2.956	0.403	0.547	1.106	32.1	0.804
	5.992	1.153	0.414	2.965	0.358	0.577	1.164	28.3	0.848
	6.976	1.204	0.384	2.952	0.318	0.602	1.219	25.4	0.885
	8.024	1.248	0.350	2.918	0.280	0.624	1.275	23.0	0.917
	9.003	1.284	0.319	2.884	0.248	0.642	1.323	21.0	0.943
	10.047	1.314	0.288	2.843	0.219	0.657	1.370	19.1	0.965
	11.022	1.334	0.264	2.811	0.197	0.667	1.404	17.8	0.980
	12.024	1.346	0.241	2.771	0.178	0.673	1.433	16.5	0.989
	13.016	1.354	0.220	2.734	0.161	0.677	1.458	15.3	0.995
	14.008	1.360	0.201	2.700	0.146	0.680	1.480	14.3	0.999
	14.999	1.359	0.181	2.657	0.132	0.680	1.500	13.3	0.999
	16.050	1.360	0.162	2.622	0.118	0.680	1.519	12.5	0.999
	16.987	1.358	0.148	2.592	0.106	0.679	1.533	11.8	0.998
	17.803	1.355	0.136	2.566	0.098	0.678	1.543	11.2	0.996

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



Sample No. 3B-P4	w_N(%) 21.2	Estimated
Depth (RE) 11.1 ft.	w_L(%) ____	σ'_{vo} 0.583 σ'_p ____
Soil Type Smith Bay	w_P(%) ____	CR ____ RR ____
Arctic Silt	I_P(%) ____	G_s 2.675 e_o 0.560 S(%) 100

○ At t_p or ____ hr Remarks End of increment

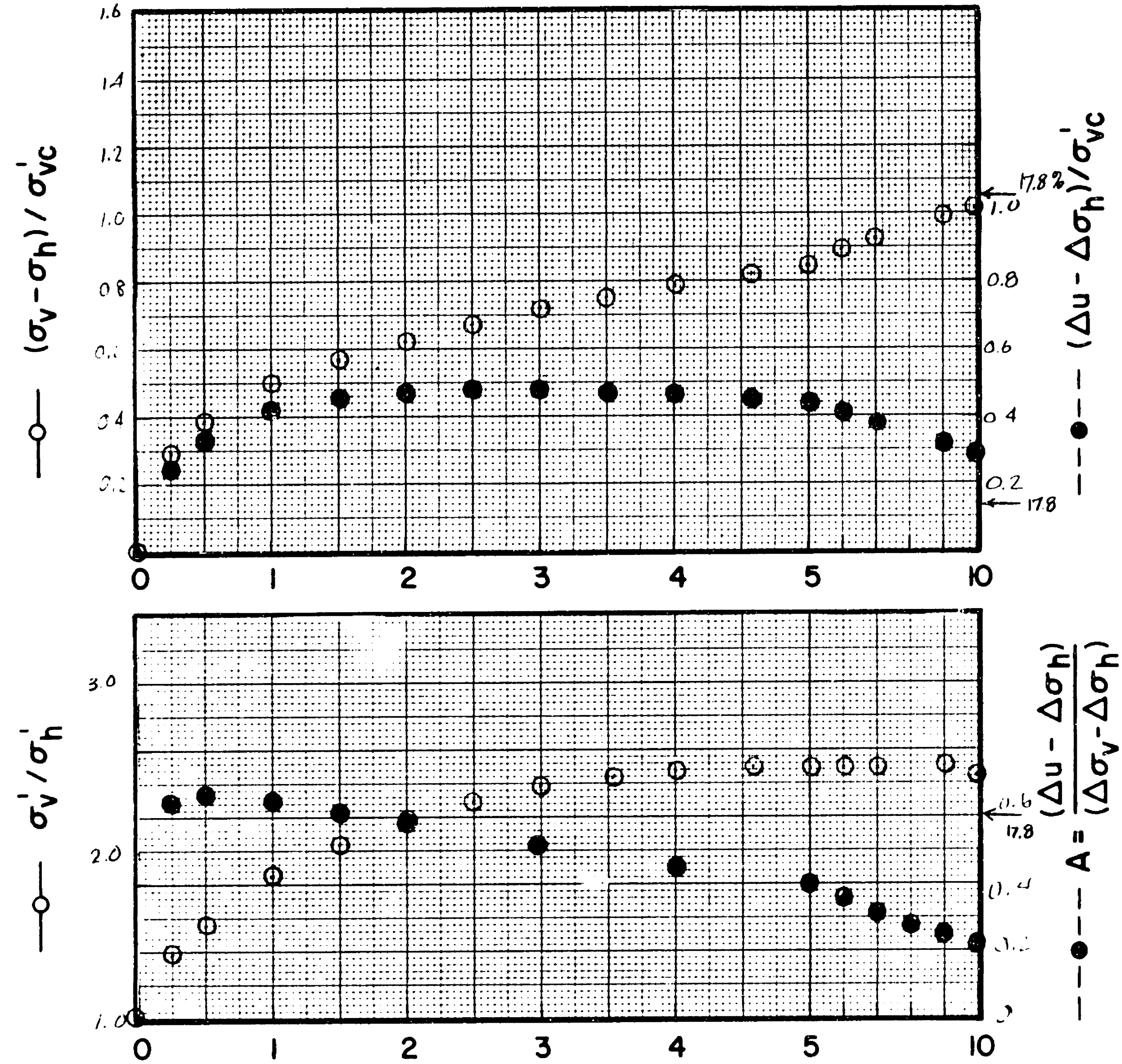
● At (t_f) hr

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. T3BT1

FIGURE



Sample No. 3B-P4 w_N(%) 21.2 σ'_{vc}(KSF) 10 K_c 1.00

Depth (RE) 11.1' w_L(%) ____ OCR 10 t_c(Days) ____

Soil Type South Bay Arctic Silt w_P(%) ____ Estimated σ'_{vo}(KSF) 0.583

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

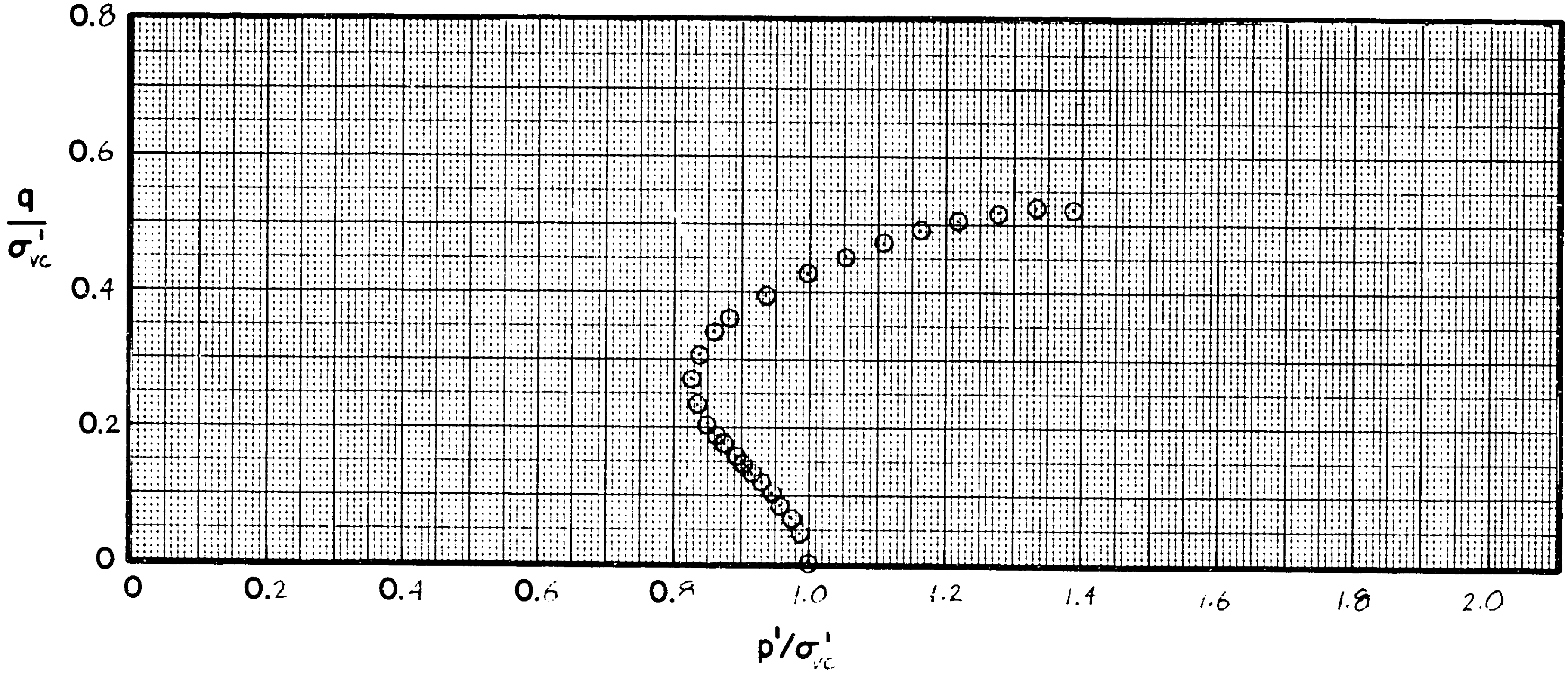
NORMALIZED STRESS VS STRAIN
CK_oU TEST No. T3BT1

FIGURE

Test No.	Sample No.	Depth	w_N (%)	σ'_{vc}	K_c	OCR	Sym.
T3BT1	T3B - P4	11.1	21.2	1.0	1.0	1.0	o

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M. I. T.

$$q = 0.5(\sigma_v - \sigma_h)$$
$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



NORMALIZED STRESS PATHS FROM CK₀UC TESTS

BORING 3B SOIL TYPE Smith Bay - Arctic Silt

FIGURE

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT Sohio

SOIL TYPE Arctic Silt

LOCATION Site T

BORING NO. 3B SAMPLE NO. T2

DEPTH 11.4'

TEST NO. TJBT2

TYPE OF TEST CIUC

APPARATUS NO.

TESTED BY CDG.

DATE 02/21/86

WATER CONTENT

INITIAL, BASED ON TRIMMINGS 21.4 %

INITIAL, BASED ON SAMPLE — %

FINAL, BASED ON SAMPLE 20.3 %

ATTERBERG LIMITS

W_p 21 %

W_L 40.6 %

I_p 19.6 % I_L

PHASE RELATIONSHIPS

γWET 2.060 g/cc. γDRY 1.697 g/cc.

e_i 0.547 e_f 0.481

S_i 99.6 % $S_{precons}$ 100 %

G_s 2.626

MISCELLANEOUS

B 112 %

SATURATION ΔV 0.12 cc.

CONSOLIDATION ΔV 3.52cc (4.30 %)

MEMBRANES — THICK 2 THIN

CORRECTION FACTOR 1.942 E

FILTER STRIPS 8 X 1/4"

CONFIGURATION Vertical

CORRECTION FACTOR 40.64E

AREA CORRECTION Parabolic.

GRAIN SIZE

% -#200

% -2μ 42.5

C_u — C_c —

SAMPLE APPEARANCE

—

FINAL AREA ( )

TOP

MIDDLE

BOTTOM

AVERAGE

INITIAL, FRONT

FINAL, FRONT

FINAL, SIDE

FINAL, X-RAY

COMMENTS -

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC TEST NO. 73BT2

STRESS HISTORY

IN SITU CONDITIONS		TEST CONDITIONS	
σ'_{vo}	0.293 KSC	σ'_{vc}	4.889 KSC.
σ'_p	–	σ'_p	4.889 KSC.
OCR	–	OCR	1
		K_c	1.00
		U_b	2.005 KSC.
TORVANE STRENGTH	–	STRAIN RATE	0.5 %/HOUR
TORVANE W_c	– %	FINAL ε_a (SHEAR)	16.56 %

STRENGTH DATA

AT MAXIMUM q		AT MAXIMUM OBLIQUITY	
ε_a	16.4 %	ε_a	5.01 %
q/σ'_{vc}	0.675	q/σ'_{vc}	0.555
p'/σ'_{vc}	1.65	p'/σ'_{vc}	1.11
$\Delta u-\Delta\sigma_h/\sigma'_{vc}$	0.227	$\Delta u-\Delta\sigma_h/\sigma'_{vc}$	0.441
ϕ'	27.8	ϕ'	29.9
A_f	0.318		
TIME TO q_{max}	32.3 HR.	$E_{u(50\%)}/q_f$	327

TIME RECORD		HYPERBOLIC STRESS-STRAIN PARAMETERS	
SET UP		G_i/σ'_{vc}	38.61
START OF CONSOLIDATION		R_f	0.934
START OF SHEAR		r^2	0.9960
END OF SHEAR			
REMOVAL		RADIOGRAPHY	
TOTAL TIME IN APPARATUS		kV 160	mA 3.8
CONSOLIDATION-SHEAR Δt		EXPOSURE TIME	45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC TEST NO. T3BT2

SAMPLE DIMENSIONS

	L (cm)	A (cm^2)	V (cm^3)	ε_a (b) (%)	ε_{vol} (b) (%)	W (g)
INITIAL	8.051	10.175	81.92	—	—	168.77
PRECONSOLIDATION	8.051	10.175	81.92	—	—	168.89
PRESHEAR	7.885	9.943	78.40	2.06	4.30	165.27
POST SHEAR	6.579	11.917	78.40	16.56	—	165.27
FINAL	6.579	11.917	78.40	18.28	4.30	165.27
FINAL MEASURED	6.534	—	—	18.84	—	167.87

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN ksc.

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}	0.991	2.002	3.448	4.885						
σ'_{hc}	0.999	1.989	3.438	4.885						
t_c (HRS)	21.3	20.5	7.1	23.2						
ε_a (%)	0.60	1.18	1.66	2.06						
ε_{vol} (%)	0.88	2.17	3.31	4.30						
K_c	1.01	0.99	1.00	1.00						

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

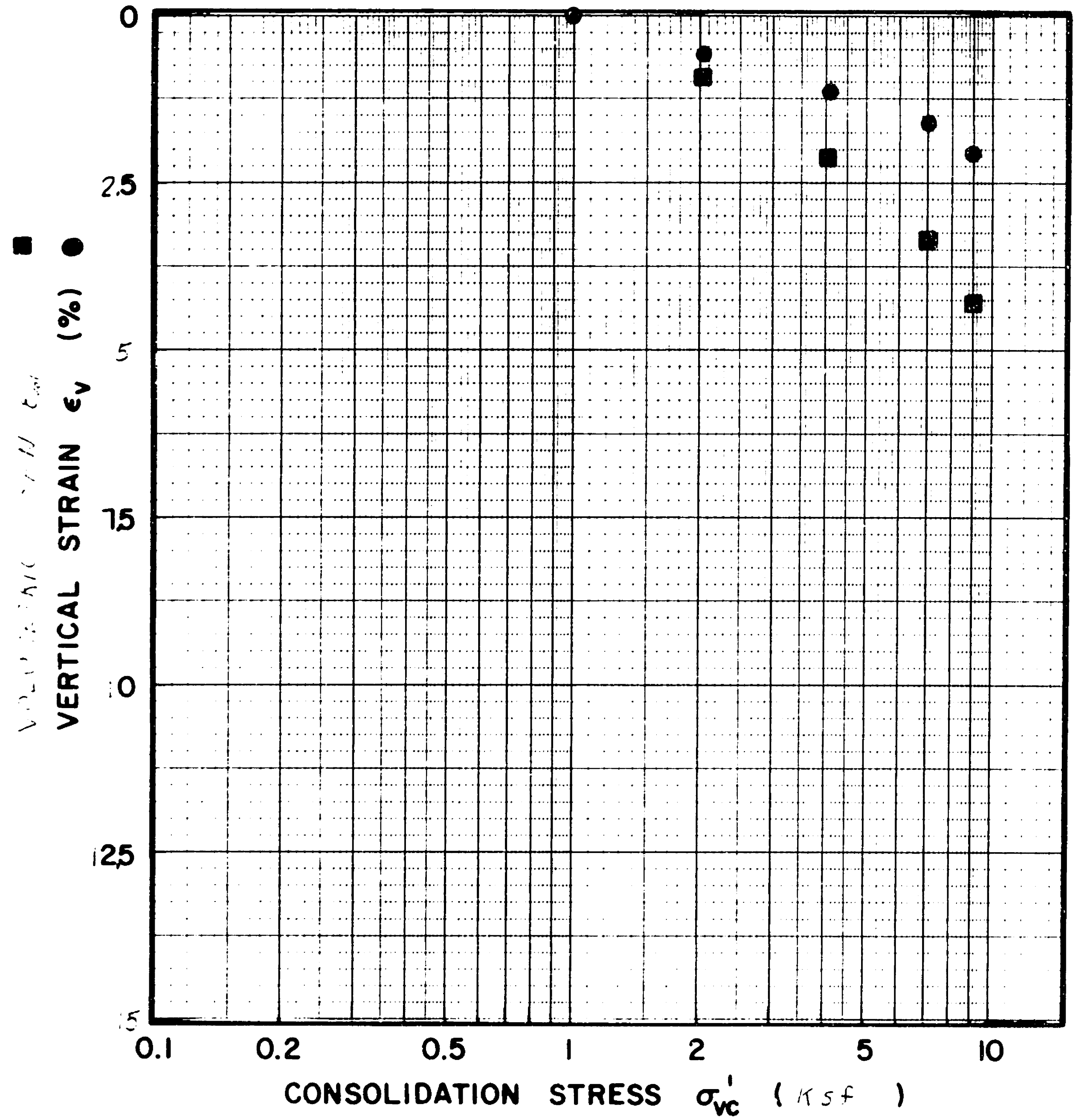
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED—UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CIUC No. T3BT2

Time (hr)	ε_a (%)	$\frac{(\sigma_v-\sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u-\Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{\Delta q_f}$
	0.000	—	—	—	—	—	—	—	—
	0.248	0.398	0.266	1.543	0.670	0.199	0.932	237.1	0.294
	0.495	0.505	0.347	1.774	0.688	0.253	0.905	151.6	0.374
	0.670	0.558	0.383	1.905	0.687	0.279	0.895	123.1	0.413
	1.019	0.642	0.430	2.128	0.670	0.321	0.891	93.2	0.475
	1.502	0.734	0.465	2.374	0.636	0.367	0.901	72.3	0.543
	2.018	0.813	0.483	2.575	0.598	0.407	0.922	59.5	0.601
	2.971	0.930	0.489	2.822	0.528	0.465	0.976	46.3	0.689
	4.011	1.037	0.470	2.959	0.456	0.519	1.048	38.2	0.767
	5.011	1.109	0.441	2.989	0.401	0.555	1.112	32.7	0.821
	6.058	1.168	0.409	2.978	0.353	0.584	1.175	28.5	0.865
	7.039	1.221	0.376	2.958	0.311	0.611	1.234	25.8	0.904
	7.991	1.258	0.345	2.924	0.277	0.629	1.283	23.3	0.931
	9.011	1.282	0.317	2.879	0.250	0.641	1.323	21.0	0.949
	10.032	1.300	0.291	2.837	0.227	0.650	1.358	19.2	0.963
	11.101	1.321	0.266	2.802	0.203	0.661	1.394	17.5	0.978
	12.067	1.333	0.244	2.765	0.185	0.667	1.422	16.3	0.987
	13.100	1.337	0.222	2.720	0.169	0.669	1.446	15.1	0.990
	13.930	1.340	0.205	2.687	0.155	0.670	1.465	14.2	0.992
	15.068	1.338	0.187	2.647	0.142	0.669	1.481	13.0	0.990
	16.058	1.336	0.173	2.618	0.132	0.668	1.494	12.3	0.989
	16.431	1.351	0.227	2.748	0.159	0.676	1.448	12.1	1.000

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



Sample No. T3B-P4 w_N(%) 21.4 Estimated

Depth (RE) 11.4 ft. w_L(%) ______ σ'_{vo} 0.60 σ'_p ______

Soil Type Arctic Silt w_P(%) ______ CR ______ RR ______

Smith Bay I_P(%) ______ G_s 2.626 e_o 0.547 S(%) 100

o At t_p or ___ hr Remarks At end of increment

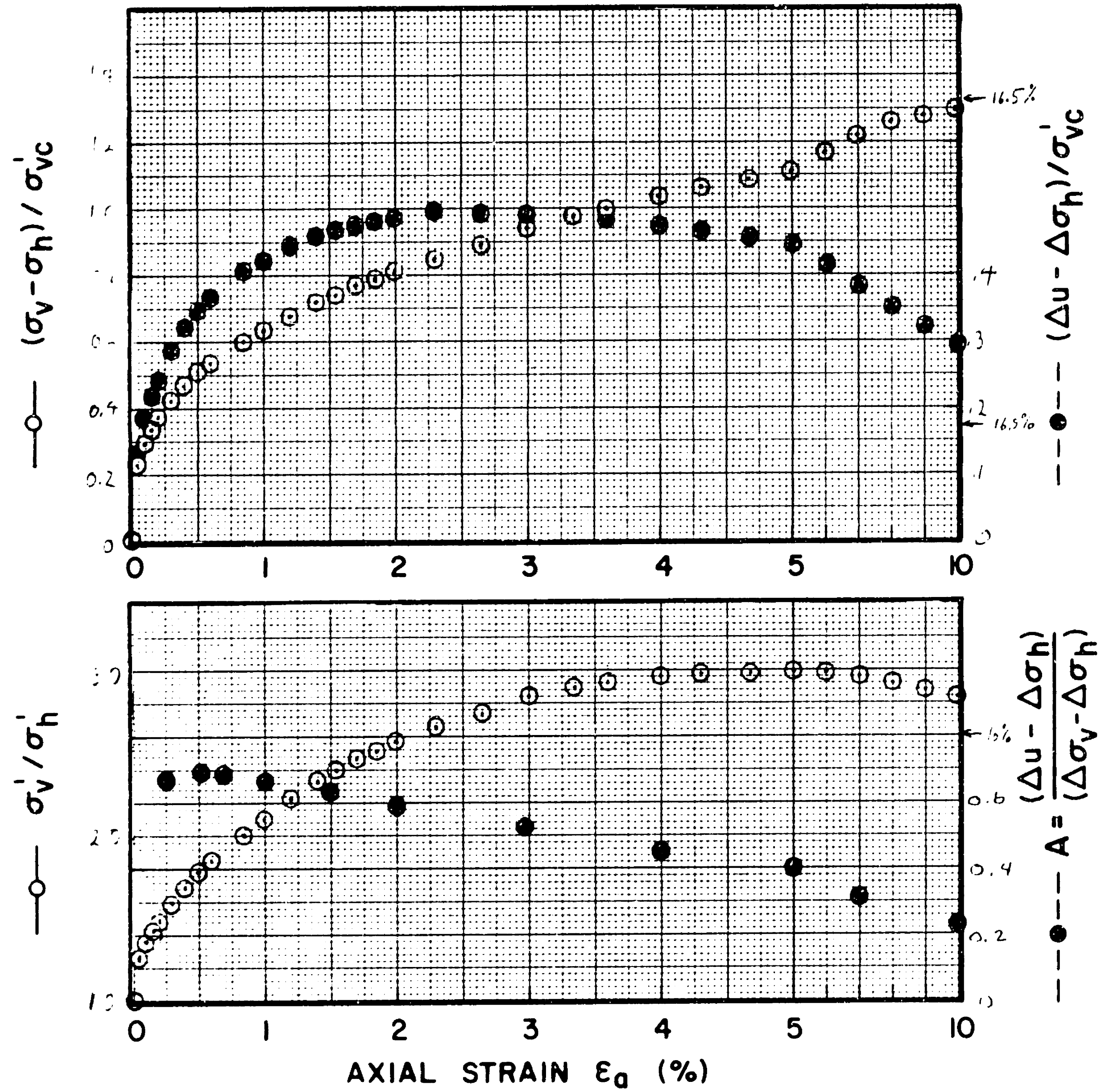
● At (t_f) hr

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. T3BT2

FIGURE



Sample No. 3B-P4 w_N(%) 21.4 σ'_{vc}(KSF) 10 K_c 1

Depth (RE) 11.4' w_L(%) ____ OCR 1 t_c(Days) ____

Soil Type Arctic Silt Smith Bay w_P(%) ____ Estimated σ'_{vo}(KSF) 0.60

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CIUC TEST No. T3B T2

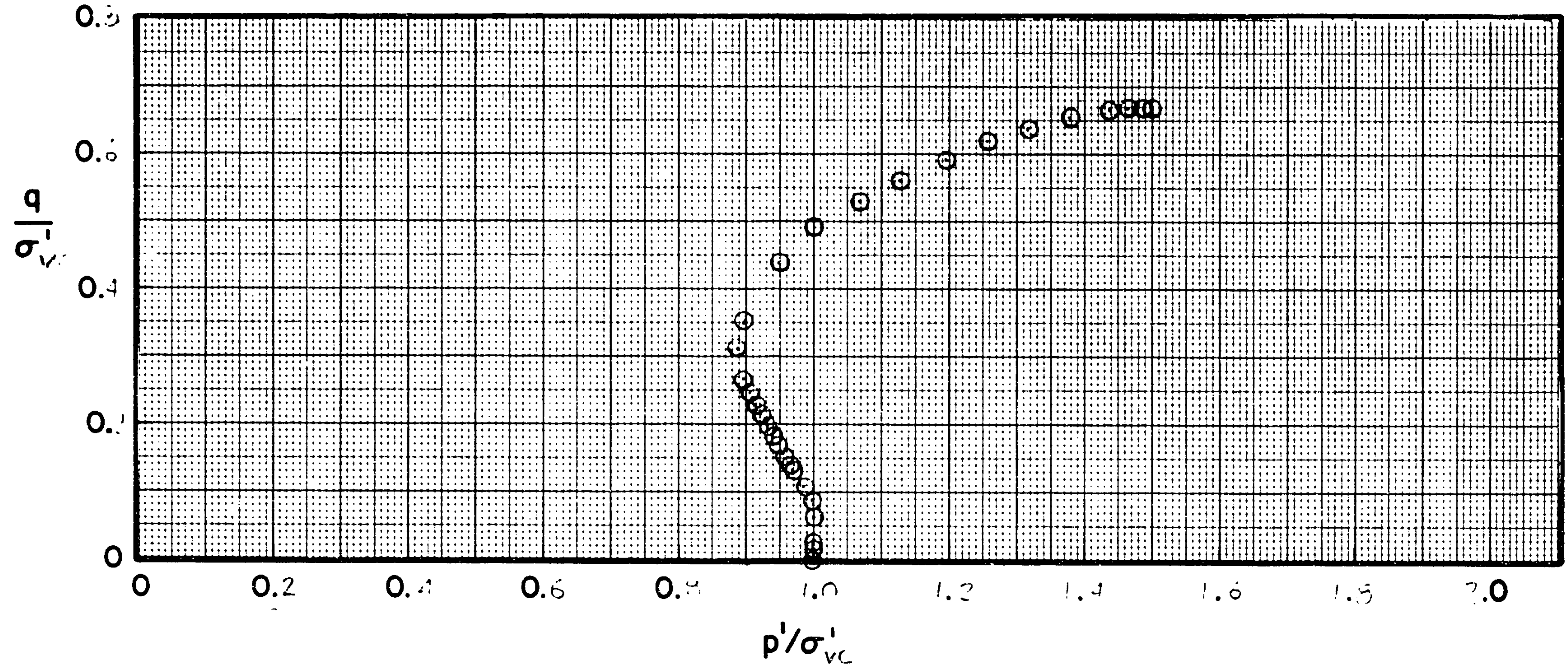
FIGURE

Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc}	K_c	OCR	Sym.
TB3T2	TB3-1-4	11.4'	21.4	10	1	1	⊙

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M.I.T.

$$q = 0.5(\sigma_v - \sigma_h)$$

$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



NORMALIZED STRESS PATHS FROM CK₀UC TESTS

BORING B3 SOIL TYPE Arctic Silt [illegible]

FIGURE

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO
SOIL TYPE ARCTIC SILT
LOCATION SITE T
BORING NO. TJB1 SAMPLE NO. T4
DEPTH 1.45'

TEST NO. TJB1T4
TYPE OF TEST CIUC
APPARATUS NO. WFE 7
TESTED BY CDG.
DATE 02/06/86

WATER CONTENT

INITIAL, BASED ON TRIMMINGS 45.0 %
INITIAL, BASED ON SAMPLE — %
FINAL, BASED ON SAMPLE 29.1 %

ATTERBERG LIMITS

W_P 25.6 %
W_L 49.5 %
I_P 23.9 % I_L

PHASE RELATIONSHIPS

γWET 1.802 g/cc. γDRY 1.242
e_i 1.215 e_f 0.762
S_i 99.0 % $S_{precons}$ 100 %
G_s 2.752

MISCELLANEOUS

B 103 %
SATURATION ΔV 0.44 cc.
CONSOLIDATION ΔV 15.70 cc. (20.43 %)
MEMBRANES — THICK 2 THIN
CORRECTION FACTOR 1.942 × ε
FILTER STRIPS 8 X 1/4"
CONFIGURATION VERTICAL
CORRECTION FACTOR 40.64 × ε
AREA CORRECTION PARABOLIC

GRAIN SIZE

% -#200
% -2μ 48
C_u C_c

SAMPLE APPEARANCE

FINAL AREA (—)

INITIAL, FRONT | FINAL, FRONT | FINAL, SIDE | FINAL, X-RAY

TOP
MIDDLE
BOTTOM
AVERAGE

COMMENTS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
- M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC TEST NO. TSB1T4

STRESS HISTORY

IN SITU CONDITIONS

σ'_{vo} 0.039 KSC.

σ'_p –

OCR –

TORVANE STRENGTH –

TORVANE W_c – %

TEST CONDITIONS

σ'_{vc} 4.886 KSC

σ'_p 4.886 KSC.

OCR 1

K_c 1.00 U_b 2.033 KSC.

STRAIN RATE 0.5 %/HOUR

FINAL ε_a (SHEAR) 11.179 %

STRENGTH DATA

AT MAXIMUM q

ε_a 11.2 %

q/σ'_{vc} 0.317

p'/σ'_{vc} 0.701

$\Delta u-\Delta\sigma_h/\sigma'_{vc}$ 0.605

ϕ' 25.9

A_f 1.97

TIME TO q_{max} 22.4 HR

AT MAXIMUM OBLIQUITY

ε_a 11.2 %

q/σ'_{vc} 0.317

p'/σ'_{vc} 0.701

$\Delta u-\Delta\sigma_h/\sigma'_{vc}$ 0.605

ϕ' 25.9

$E_{u(50\%)}/q_f$ 17.9

TIME RECORD

SET UP 2 HR.

START OF CONSOLIDATION 02/11/86 18:50

START OF SHEAR 02/17/86 12:45

END OF SHEAR 02/18/86 09:35

REMOVAL 02/18/86 12:15

TOTAL TIME IN APPARATUS 264 HR.

CONSOLIDATION-SHEAR Δt 30'

HYPERBOLIC STRESS-STRAIN PARAMETERS

G_i/σ'_{vc} 44.6

R_f 0.971

r^2 0.9990

RADIOGRAPHY

kV 160 mA 3.8

EXPOSURE TIME 45''

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC TEST NO. TSBIT4

SAMPLE DIMENSIONS

	L (cm)	A (cm²)	V (cm³)	ε_a (%) (b)	ε_{vol} (%) (b)	W (g)
INITIAL	8.007	9.595	76.83	–	–	138.44
PRECONSOLIDATION	8.007	9.595	76.83	–	–	138.88
PRESHEAR	7.127	8.577	61.13	10.99	20.43	122.71
POST SHEAR	6.330	9.657	61.13	11.18	–	122.71
FINAL	6.330	9.657	61.13	20.94	20.43	122.71
FINAL MEASURED	6.282	–	–	21.54	–	120.13

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN KSC.

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}	0.090	0.182	0.497	0.638	0.992	1.997	3.494	4.383		
σ'_{hc}	0.122	0.244	0.497	0.512	0.994	1.999	3.496	4.382		
t_c (HRS)	17.5	25.3	27.7	14.0	6.5	17.7	8.3	15.4		
ε_a (%)	0.70	2.18	4.56	5.97	6.49	8.51	9.97	10.99		
ε_{vol} (%)	1.74	4.67	8.34	9.71	11.36	15.32	18.34	20.43		
K_c	1.36	1.34	1.00	0.74	1.00	1.00	1.00	1.00		

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

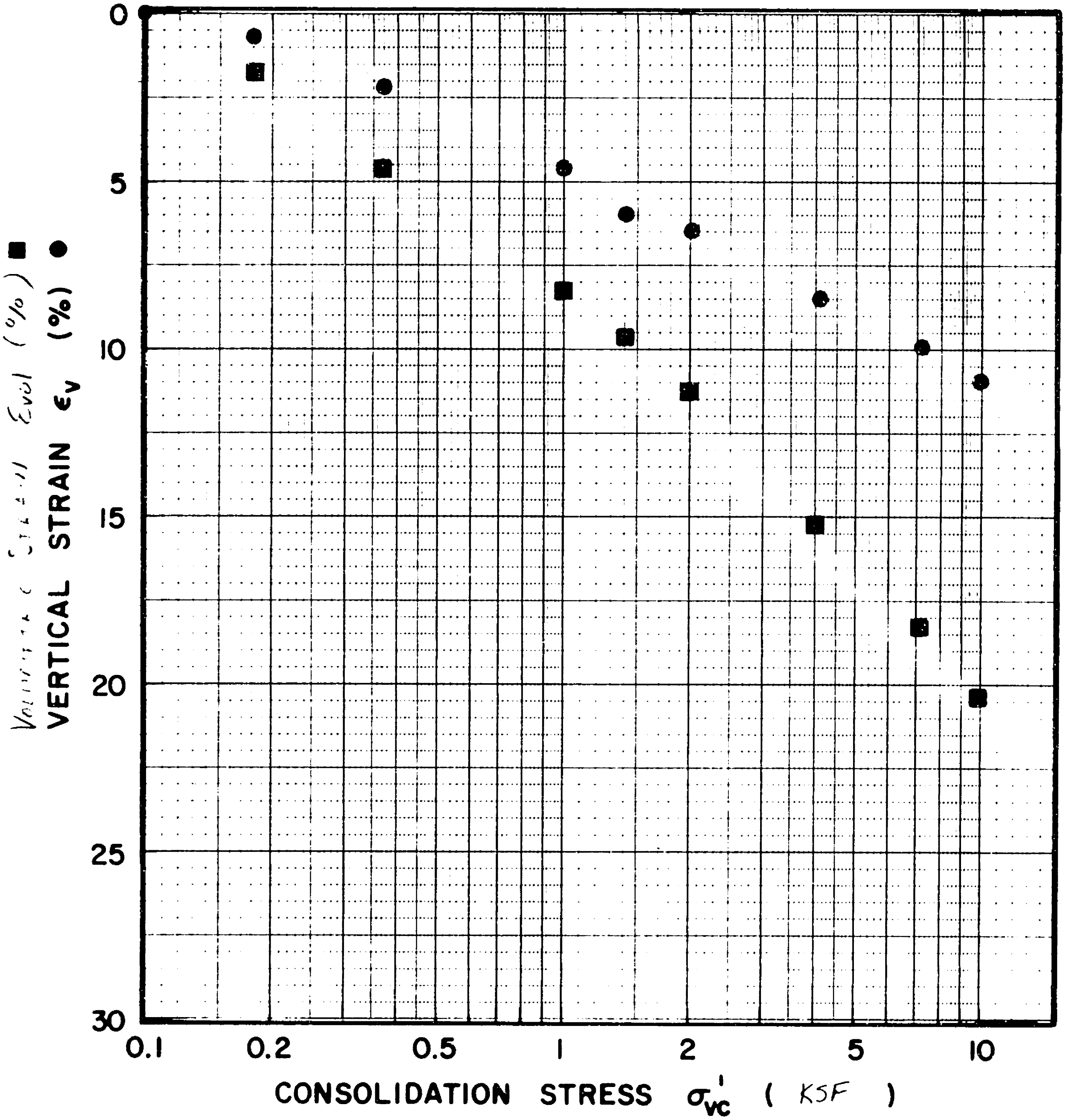
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED — UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CIUC No. TSB1T4

Time (hr)	ε_a (%)	$\frac{(\sigma_v - \sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u - \Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{q_f}$
	0.000	—	—	—	—	—	—	—	—
	0.250	0.293	0.180	1.345	0.659	0.142	0.960	367.2	0.460
	0.533	0.358	0.264	1.487	0.740	0.179	0.914	218.5	0.583
	0.766	0.394	0.313	1.574	0.795	0.197	0.884	167.1	0.642
	1.016	0.423	0.353	1.655	0.838	0.212	0.857	135.3	0.689
	2.027	0.488	0.456	1.898	0.936	0.244	0.787	78.3	0.795
	3.043	0.529	0.515	2.091	0.975	0.265	0.749	56.5	0.861
	4.008	0.551	0.548	2.221	0.995	0.276	0.727	44.7	0.898
	5.042	0.560	0.571	2.306	1.020	0.280	0.708	36.1	0.912
	6.034	0.570	0.584	2.374	1.025	0.285	0.700	30.8	0.929
	7.013	0.586	0.593	2.444	1.013	0.293	0.699	27.2	0.956
	8.087	0.599	0.599	2.497	1.002	0.300	0.700	24.1	0.976
	9.087	0.602	0.602	2.516	1.002	0.301	0.698	21.6	0.981
	10.073	0.605	0.603	2.527	0.998	0.303	0.699	19.6	0.986
	11.179	0.614	0.605	2.556	0.987	0.701	0.307	17.9	1.000

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



Sample No. T-5B1-P1 w_N(%) 45.0 Estimated

Depth (RE) .45 ft w_L(%) ______ σ'_{vo} 0.080 σ'_p ______

Soil Type Smith Bay w_P(%) ______ CR ______ RR ______

Arctic Silt I_P(%) ______ G_s 2.752 e_0 1.215 S(%) 100

○ At t_p or ____ hr Remarks Plotted for ...

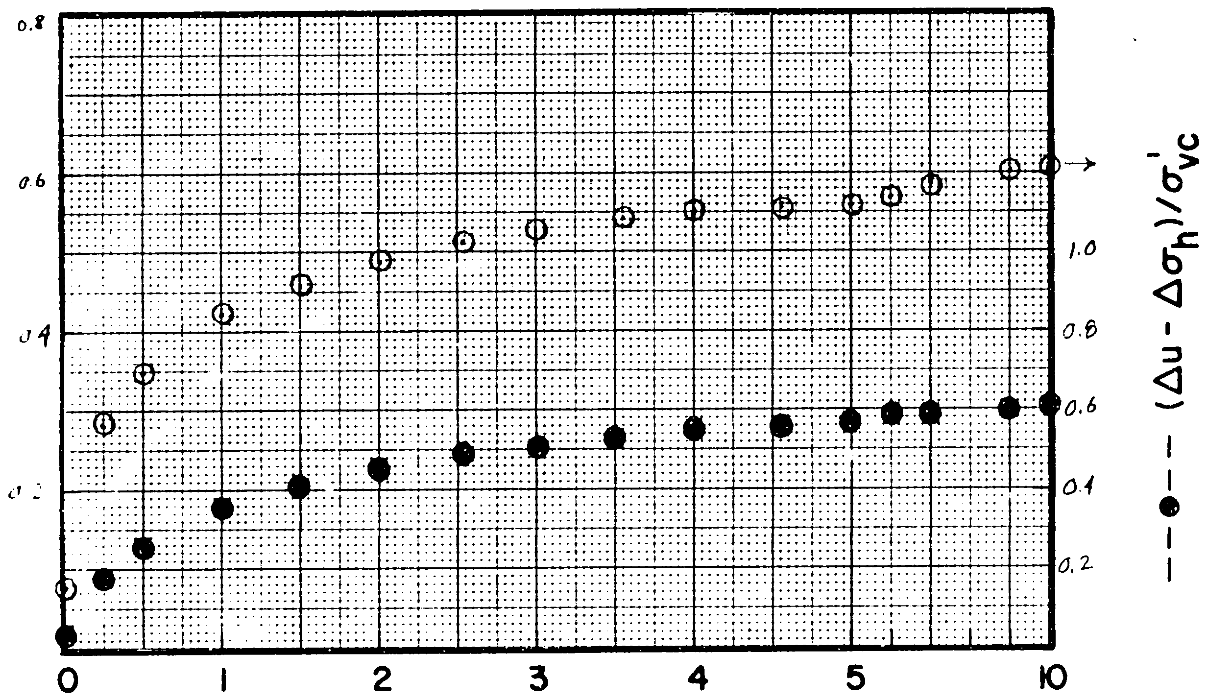
● At (t_f) hr

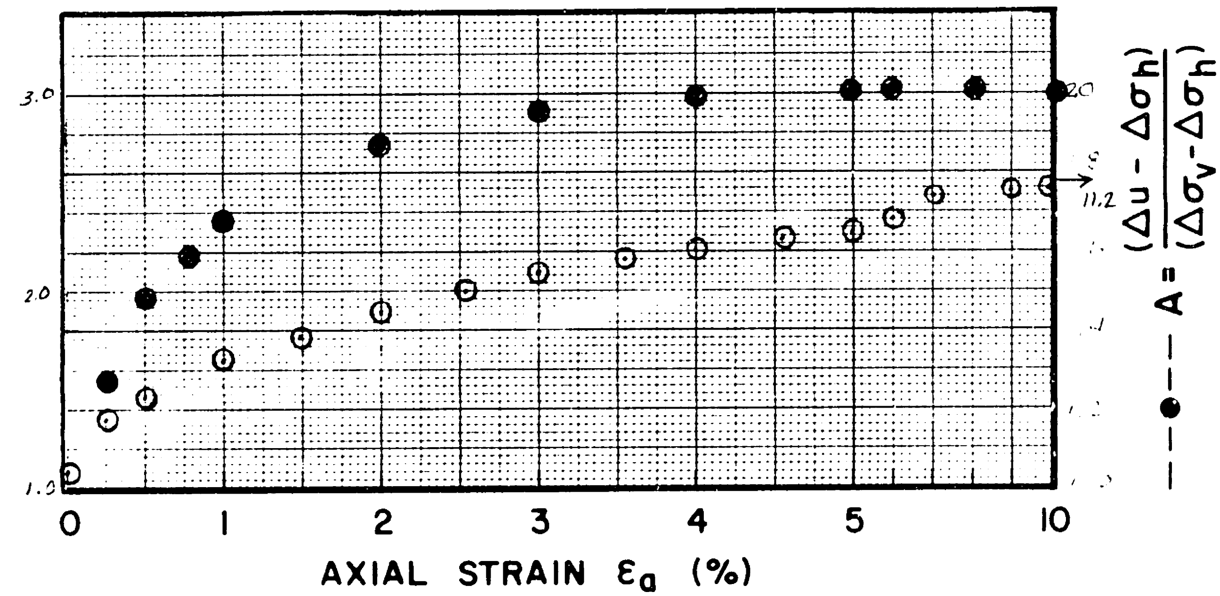
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. T5B1T4

FIGURE





Sample No. T5B1-P1 w_N(%) 45.0 σ'_{vc}(Ksf) 10 K_c 1.00

Depth (RE) 1.45 ft w_L(%) ____ OCR 1 t_c(Days) ____

Soil Type Smith Bay Arctic Silt w_P(%) ____ Estimated σ'_{vo}(KSF) 0.030

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

TEST No. T5B1T4

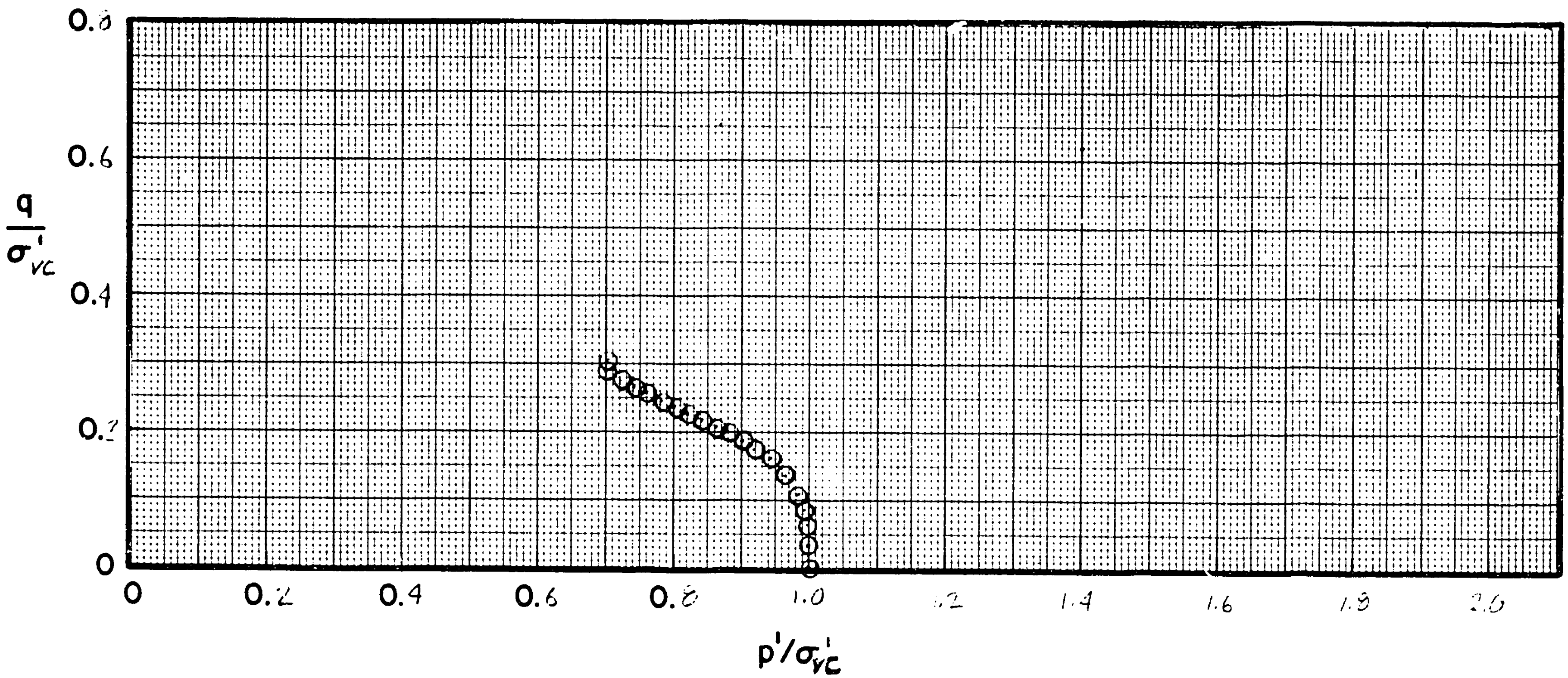
FIGURE

Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Sym.
T5B1T4	5B1-P1	1.45'	45.0	10	1.00	1	⊙

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M. I. T.

$$q = 0.5(\sigma_v - \sigma_h)$$

$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



NORMALIZED STRESS PATHS FROM C I UC TESTS

BORING 5B1 SOIL TYPE Smith Bay-Arctic Silt

FIGURE

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO
TEST NO. TSBITL
SOIL TYPE ARCTIC SILT
TYPE OF TEST CIUC (αR=2)
LOCATION SITE T
APPARATUS NO. WFE7
BORING NO. SBI SAMPLE NO. T6
TESTED BY CDQ
DEPTH 2.3'
DATE 03/08/86

WATER CONTENT

INITIAL, BASED ON TRIMMINGS 42.8 %
INITIAL, BASED ON SAMPLE — %
FINAL, BASED ON SAMPLE 28.3 %

ATTERBERG LIMITS

W_p 25.6 %
W_L 49.5 %
I_p 23.9 % I_L

PHASE RELATIONSHIPS

γWET 1.823 g/cc. γDRY 1.277 g/cc.
e_i 1.134 e_f 0.732
S_i 99.7 % $S_{precons}$ 100 %
G_s 2.726

MISCELLANEOUS

B 100 %
SATURATION ΔV 0.14 cc.
CONSOLIDATION ΔV 14.62 cc. (18.85 %)
MEMBRANES — THICK 2 THIN
CORRECTION FACTOR 1.942 C
FILTER STRIPS 8 X 1/4"
CONFIGURATION vertical
CORRECTION FACTOR 40.64 C
AREA CORRECTION PARABOLIC.

GRAIN SIZE

% -#200
% -2μ 48
C_u C_c

SAMPLE APPEARANCE

FINAL AREA ( )

TOP

MIDDLE

BOTTOM

AVERAGE

INITIAL, FRONT FINAL, FRONT FINAL, SIDE FINAL, X-RAY

COMMENTS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
- M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT 80440 TYPE OF TEST CIUC TEST NO. TXB1T6

STRESS HISTORY

IN SITU CONDITIONS

σ'_{vo} 0.062 KSC

σ'_p –

OCR –

TORVANE STRENGTH –

TORVANE W_c – %

TEST CONDITIONS

σ'_{vc} 2.439 KSC.

σ'_p 4.384 KSC.

OCR 2

K_c 1.00 U_b 2.007 KSC.

STRAIN RATE 0.5 %/HOUR

FINAL ε_a (SHEAR) 18.25 %

STRENGTH DATA

AT MAXIMUM q

ε_a 14.2 %

q/σ'_{vc} 0.582

p'/σ'_{vc} 1.18

$\Delta u - \Delta\sigma_h/\sigma'_{vc}$ 0.403

ϕ' 29.6

A_f 0.688

TIME TO q_{max} 23.4 HR.

AT MAXIMUM OBLIQUITY

ε_a 10.1 %

q/σ'_{vc} 0.577

p'/σ'_{vc} 1.16

$\Delta u - \Delta\sigma_h/\sigma'_{vc}$ 0.420

ϕ' 29.9

$E_{u(50\%)}/q_f$ 19.5

TIME RECORD

SET UP

START OF CONSOLIDATION

START OF SHEAR

END OF SHEAR

REMOVAL

TOTAL TIME IN APPARATUS

CONSOLIDATION-SHEAR Δt

HYPERBOLIC STRESS-STRAIN PARAMETERS

G_i/σ'_{vc} 84.0

R_f 0.977

r^2 0.9994

RADIOGRAPHY

kV 160 mA 3.8

EXPOSURE TIME 45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC TEST NO. TSB1T6

SAMPLE DIMENSIONS

	L cm	A cm^2	V cm^3	ε_a (b) (%)	ε_{vol} (b) (%)	W (g)
INITIAL	8.037	9.651	77.56	—	—	141.44
PRECONSOLIDATION	8.037	9.651	77.56	—	—	141.58
PRESHEAR	7.371	8.539	62.94	8.29	18.85	126.50
POST SHEAR	6.026	10.445	62.94	13.25	—	126.50
FINAL	6.026	10.445	62.94	25.02	18.85	126.50
FINAL MEASURED	5.988	—	—	25.49	—	127.42

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN KSC.

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}	0.084	0.210	0.499	0.997	1.996	3.500	4.884	2.439		
σ'_{hc}	0.098	0.242	0.499	0.999	1.996	3.500	4.884	2.439		
t_c (HRS)	3.8	13.8	4.4	18.2	5.3	8.5	25.7	21.7		
ε_a (%)	0.37	1.23	2.80	4.43	6.02	7.50	8.64	8.29		
ε_{vol} (%)	1.11	3.80	6.59	10.35	13.87	17.12	19.56	18.85		
K_c	1.17	1.15	1.00	1.00	1.00	1.00	1.00	1.00		

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

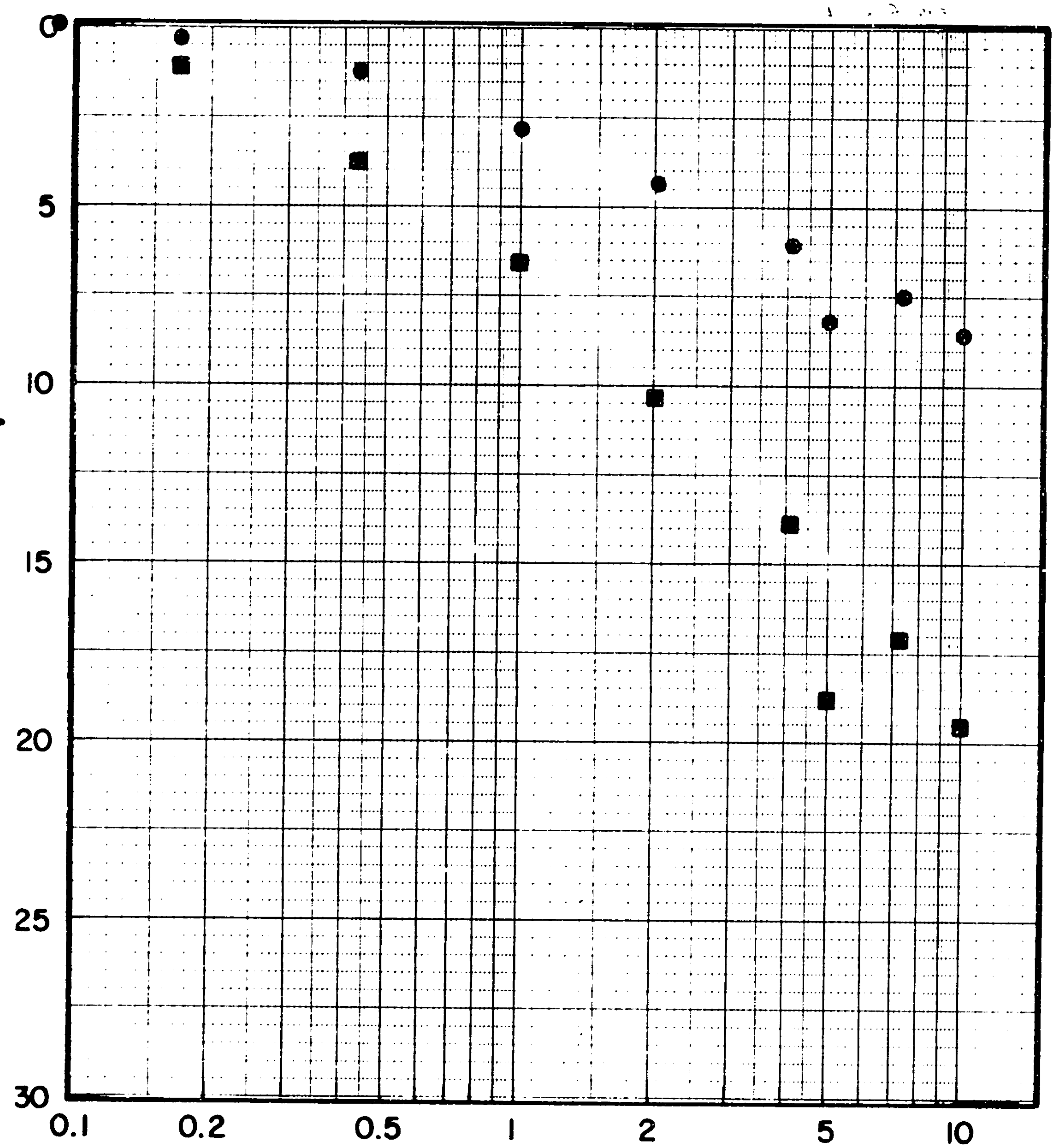
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED — UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CIUC No. TSB1T6

Time (hr)	ε_a (%)	$\frac{(\sigma_v - \sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u - \Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{q_f}$
	0.000	—	—	—	—	—	—	—	—
	0.103	0.419	0.151	1.494	0.362	0.210	1.057	699.3	0.360
	0.513	0.690	0.219	1.884	0.318	0.345	1.126	231.1	0.593
	0.763	0.761	0.244	2.008	0.322	0.381	1.135	171.2	0.653
	1.013	0.811	0.267	2.107	0.330	0.406	1.138	137.3	0.696
	1.449	0.872	0.299	2.245	0.343	0.436	1.137	103.3	0.749
	2.030	0.930	0.332	2.393	0.357	0.465	1.132	78.6	0.798
	3.034	0.982	0.368	2.556	0.376	0.491	1.122	55.6	0.843
	4.251	1.035	0.396	2.715	0.383	0.518	1.121	41.8	0.889
	5.000	1.063	0.406	2.792	0.383	0.532	1.125	36.5	0.913
	6.017	1.107	0.413	2.887	0.375	0.554	1.140	31.6	0.950
	7.040	1.110	0.419	2.911	0.378	0.555	1.135	27.1	0.953
	7.945	1.119	0.420	2.933	0.376	0.560	1.139	24.2	0.961
	9.081	1.131	0.421	2.957	0.372	0.566	1.143	21.4	0.971
	10.136	1.154	0.420	2.994	0.362	0.577	1.156	19.5	0.991
	11.161	1.139	0.417	2.959	0.365	0.570	1.151	17.5	0.979
	12.156	1.141	0.412	2.945	0.360	0.571	1.157	16.2	0.980
	13.015	1.142	0.408	2.931	0.356	0.571	1.162	15.1	0.981
	14.212	1.164	0.403	2.954	0.344	0.582	1.178	14.1	1.000
	15.040	1.140	0.401	2.903	0.349	0.570	1.169	13.0	0.979
	16.063	1.134	0.396	2.880	0.347	0.567	1.171	12.1	0.974
	17.038	1.132	0.391	2.860	0.344	0.566	1.175	11.4	0.972
	18.063	1.147	0.387	2.872	0.336	0.574	1.186	10.8	0.985
	18.243	1.152	0.386	2.879	0.334	0.576	1.189	10.8	0.989

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



CONSOLIDATION STRESS σ'_{vc} (KSF)

Sample No. T5B1-P1	w_N(%) 42.8	Estimated
Depth (RE) 2.3 FT	w_L(%) ______	σ'_{vo} 0.127 σ'_p ______
Soil Type SMITH BAY	w_P(%) ______	CR ______ RR ______
ARCTIC SILT	I_P(%) ______	G_s 2.726 e_o 1.136 S(%) 100

o At t_p or ____ hr

● At (t_f) hr

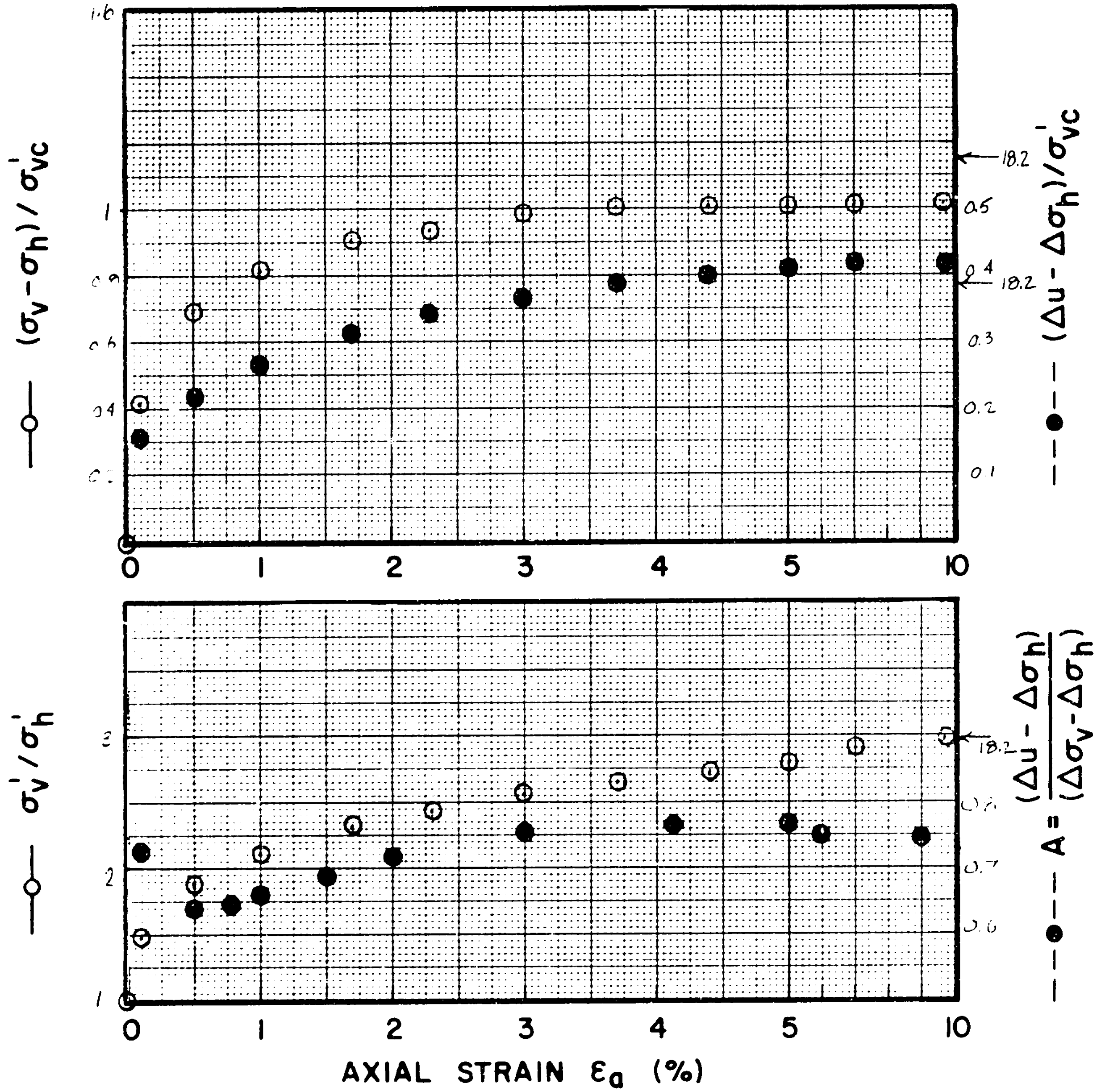
Remarks ______________________

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. T5B1T6

FIGURE



Sample No. TEB1-P1 w_N(%) 42.8 σ'_{vc}(KSF) 5 K_c 1.00

Depth (RE) 2.3 FT. w_L(%) ______ OCR 2 t_c(Days) ______

Soil Type Arctic SILT SMITH BAY w_P(%) ______ Estimated σ'_{vo}(KSF) 0.127

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CK_oU TEST No. TSB1T6

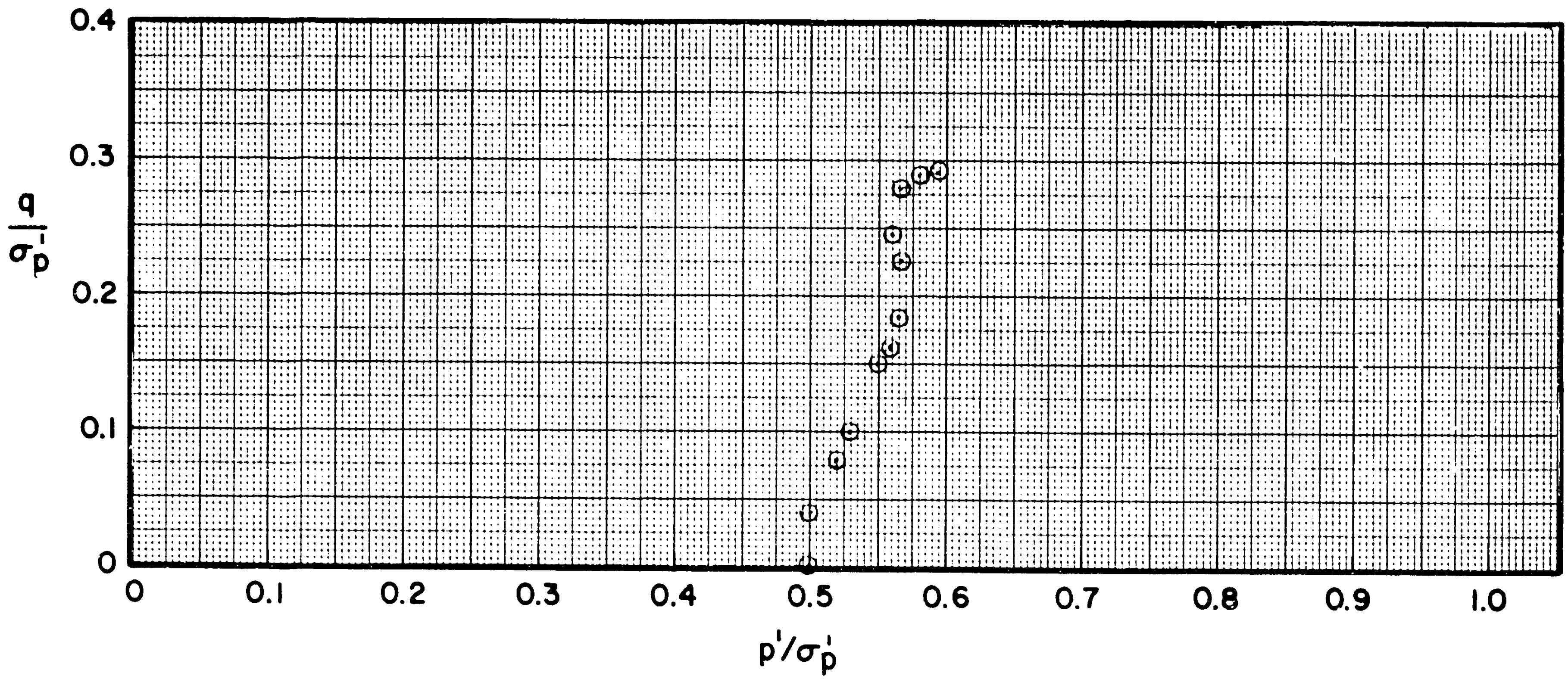
FIGURE

Test No.	Sample No.	Depth (ft)	w_N (%)	σ'_{vc} (TSF)	K_c	OCR	Sym.
75B1T6	15B1 - F1 (C)	[illegible]	42.9	5.0	1.00	2	⊙

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M. I. T.

$q = 0.5(\sigma_v - \sigma_h)$

$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$



NORMALIZED STRESS PATHS FROM CK_oUC TESTS

BORING 15B1 SOIL TYPE Arctic Silt - Simpson Bay

FIGURE

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO

SOIL TYPE ARCTIC SILT

LOCATION SITE T

BORING NO. TBI SAMPLE NO. T7

DEPTH 43'

TEST NO. TBIT7

TYPE OF TEST CIUC (OCR= 4.9)

APPARATUS NO. WFE6

TESTED BY CAG.

DATE 03/08/86

WATER CONTENT

INITIAL, BASED ON TRIMMINGS 36.7 %

INITIAL, BASED ON SAMPLE – %

FINAL, BASED ON SAMPLE 28.1 %

ATTERBERG LIMITS

W_p 25.6 %

W_L 49.5 %

I_p 23.9 % I_L

PHASE RELATIONSHIPS

γWET 1.836 g/cc. γDRY 1.343 g/cc

e_i 0.933 e_f 0.675

S_i 99.0 % $S_{precons}$ 100 %

G_s 2.593

GRAIN SIZE

% -#200

% -2μ 48

C_u C_c

MISCELLANEOUS

B 96.0 %

SATURATION ΔV 0.37 cc.

CONSOLIDATION ΔV 10.70cc (13.31 %)

MEMBRANES – THICK 2 THIN

CORRECTION FACTOR 1.942 E

FILTER STRIPS 8 X 1/4"

CONFIGURATION Vertical

CORRECTION FACTOR 10.64 E

AREA CORRECTION PARABOLIC.

SAMPLE APPEARANCE

FINAL AREA ( )

TOP

MIDDLE

BOTTOM

AVERAGE

INITIAL, FRONT FINAL, FRONT FINAL, SIDE FINAL, X-RAY

COMMENTS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
- M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT Sohio TYPE OF TEST CIUC TEST NO. TSB1T7

STRESS HISTORY

IN SITU CONDITIONS

σ'_{vo} 0.116 ksc

σ'_p –

OCR –

TORVANE STRENGTH –

TORVANE W_c – %

TEST CONDITIONS

σ'_{vc} 1.188 ksc.

σ'_p 5.868 ksc

OCR 4.9

K_c 1.00 U_b 1.977 ksc.

STRAIN RATE 0.5 %/HOUR

FINAL ε_a (SHEAR) 17.91 %

STRENGTH DATA

AT MAXIMUM q

ε_a 12.4 %

q/σ'_{vc} 1.08

p'/σ'_{vc} 2.27

$\Delta u - \Delta\sigma_h/\sigma'_{vc}$ -0.193

ϕ' 28.4

A_f -0.182

TIME TO q_{max} 24.8 HR.

AT MAXIMUM OBLIQUITY

ε_a 7.03 %

q/σ'_{vc} 0.984

p'/σ'_{vc} 2.02

$\Delta u - \Delta\sigma_h/\sigma'_{vc}$ -0.037

ϕ' 29.2

$E_{u(50\%)}/q_f$ 26.0

TIME RECORD

SET UP

START OF CONSOLIDATION –

START OF SHEAR

END OF SHEAR

REMOVAL

TOTAL TIME IN APPARATUS

CONSOLIDATION-SHEAR Δt

HYPERBOLIC STRESS-STRAIN PARAMETERS

G_i/σ'_{vc} 68.5

R_f 0.929

r^2 0.9964

RADIOGRAPHY

kV 160 mA 3.3

EXPOSURE TIME 45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT Sohio TYPE OF TEST CIUC TEST NO. TV3177

SAMPLE DIMENSIONS

	L (cm)	A (cm^2)	V (cm^3)	ε_a (b) (%)	ε_{vol} (b) (%)	W (g)
INITIAL	8.003	10.047	80.40	–	–	147.59
PRECONSOLIDATION	8.003	10.047	80.40		–	147.96
PRESHEAR	7.553	9.228	69.70	5.62	13.31	136.94
POST SHEAR	6.200	11.242	69.70	17.91	–	136.94
FINAL	6.200	11.242	69.70	22.53	13.31	136.94
FINAL MEASURED	6.233	–	–	22.12	–	137.97

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN KSC.

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}	0.102	0.265	0.492	1.002	1.998	3.499	4.886	5.873	3.520	1.188
σ'_{hc}	0.101	0.249	0.502	1.005	1.997	3.498	4.885	5.709	3.518	1.186
t_c (HRS)	15.6	3.8	17.8	5.4	4.8	15.2	6.3	26.6	18.2	24.3
ε_a (%)	-0.04	0.43	1.23	2.49	3.72	5.25	6.06	6.81	6.54	5.62
ε_{vol} (%)	0.19	1.70	4.04	6.58	9.04	12.25	13.88	15.37	14.94	13.31
K_c	0.99	0.94	1.02	1.00	1.00	1.00	1.00	0.97	1.00	1.00

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

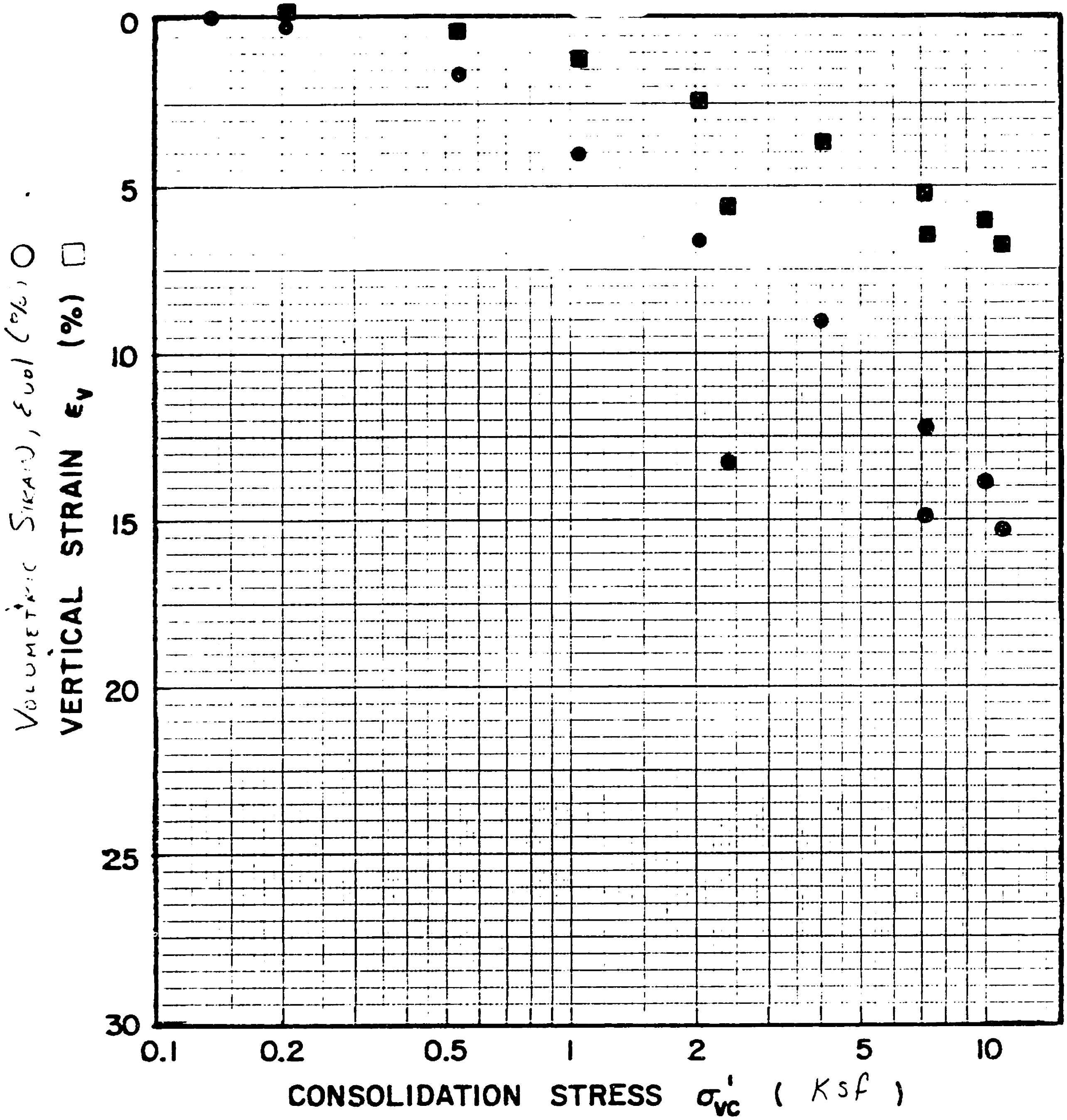
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED — UNDRAINED TRIAXIAL TEST (continued)

Project Sohio Soil ARCTIC SILT Type of Test CIUC No. TSBIT7

Time (hr)	ε_a (%)	$\frac{(\sigma_v - \sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u - \Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{q_f}$
	0.000	—	—	—	—	—	—	—	
	0.002	0 023	0.846	1.023	0.200	0.012	1.005	1313.6	0.010
	0.240	0.577	0.218	1.739	0.378	0.289	1.069	222.4	0.267
	0.485	0.768	0238	2.010	0.310	0.384	1.145	146.7	0.356
	0 749	0925	0.240	2.219	0.260	0.463	1.221	1143	0.429
	1.015	1.052	0.233	2.374	0.222	0.526	1.219	96.0	0.488
	1.529	1.238	0.209	2.568	0.169	0 619	1.408	75.0	0.574
	2.089	1.376	0.178	2.675	0 129	0.688	1.508	61.0	0.638
	3.105	1.587	0.125	2.815	0.078	0.794	1.668	47.4	0 736
	4.024	1.721	0.083	2.882	0.047	0.861	1.775	39.5	0.798
	5.068	1 814	0.040	2.893	0.020	0.907	1.864	33.2	0 841
	6.201	1.906	-0.005	2.898	-0.006	0.953	1.956	28.5	0.884
	7.027	1.968	-0.037	2.901	-0.022	0.984	2.019	26.0	0913
	8.015	2.026	-0.074	2888	-0.040	1.013	2.085	23.4	0 939
	9.000	2.076	-0.109	2.873	-0.056	1.038	2.146	21.4	0.963
	10.072	2.108	-0.143	2.846	-0.070	1 054	2.194	19.4	0 978
	11.001	2.121	-0.162	2.826	-0.079	1.061	2.221	18.0	0.984
	12.092	2.148	-0.186	2.812	-0.089	1.074	2.258	16.5	0 996
	12.373	2.156	-0.193	2.810	-0.091	1.078	2.269	16.3	1.000
	13.032	2.144	-0.205	2.781	-0.097	1 072	2.275	15.3	0.994
	14.081	2.125	-0.225	2.737	-0.107	1 063	2.285	14.0	0.986
	15.479	2.131	-0.251	2.706	-0.118	1.066	2.315	12.8	0.988
	16.025	2.134	-0.260	2.695	-0.122	1.067	2.326	12.3	0.990
	17.028	2.135	-0.275	2.676	-0.129	1.068	2.340	11.6	0.990
	17.910	2.125	-0.285	2.654	-0.135	1.063	2.346	11.0	0.985

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



Sample No. 15B1 w_N(%) 36.7   Estimated

Depth (RE) 4.3' w_L(%) ____ σ'_{vo} 0.237 σ'_p ____

Soil Type Arctic Silt w_P(%) ____   CR ____ RR ____

Smith Bay I_P(%) ____ G_s 2.83 e_0 0.933   S(%) 100

○ At t_p or ____ hr   Remarks ____

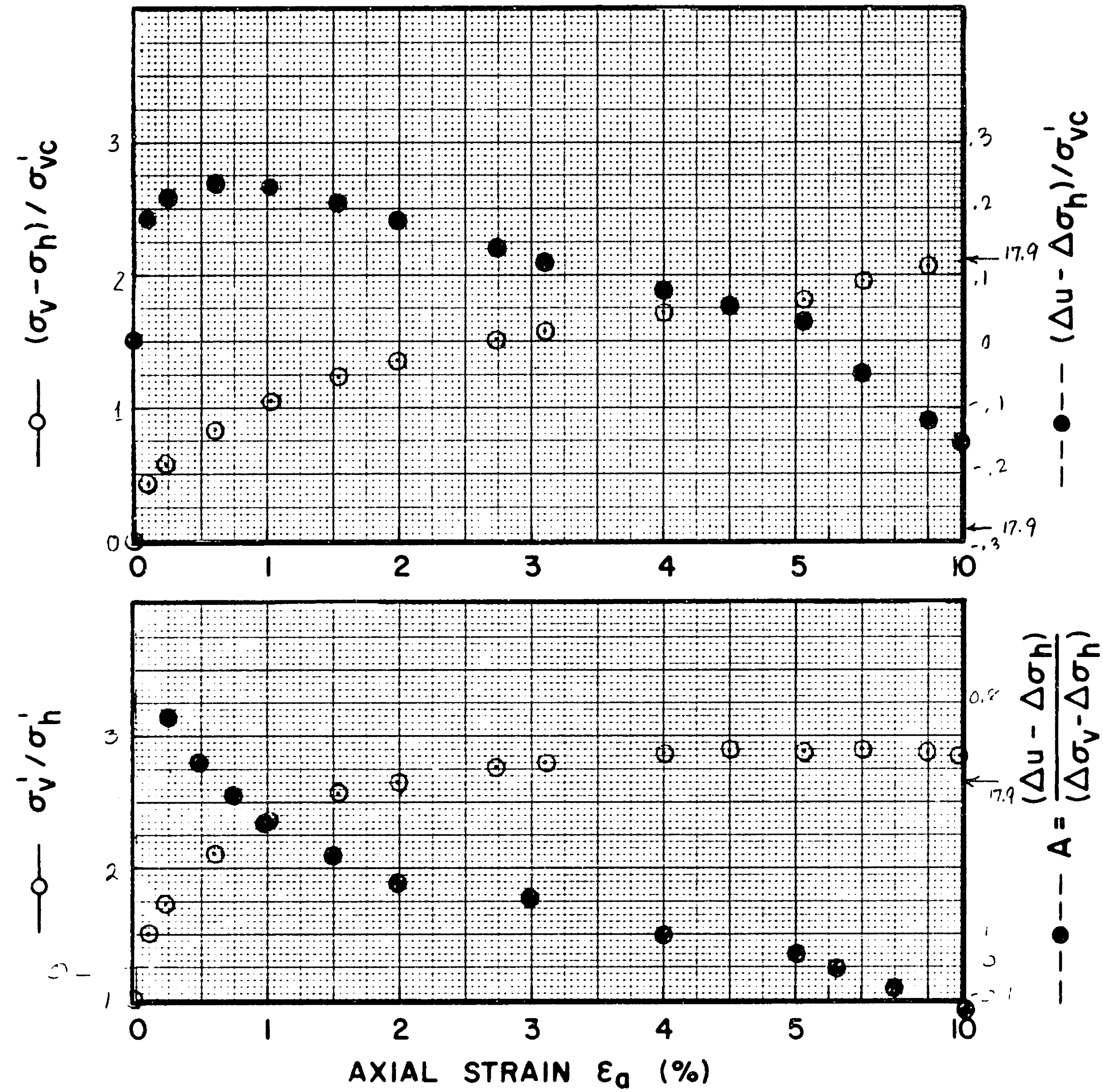
● At ( ) hr

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. 15B1T7

FIGURE



Sample No. T5B1-P2 w_N(%) 36.7 σ'_{vc}(ksf) 2.43 K_c ____

Depth (BE) 4.3 ft w_L(%) ____ OCR 5 t_c(Days) ____

Soil Type Arctic Silt Smith Bay w_P(%) ____ Estimated σ'_{vo}(ksf) 0.237

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CK_0U TEST No. T5B1T7

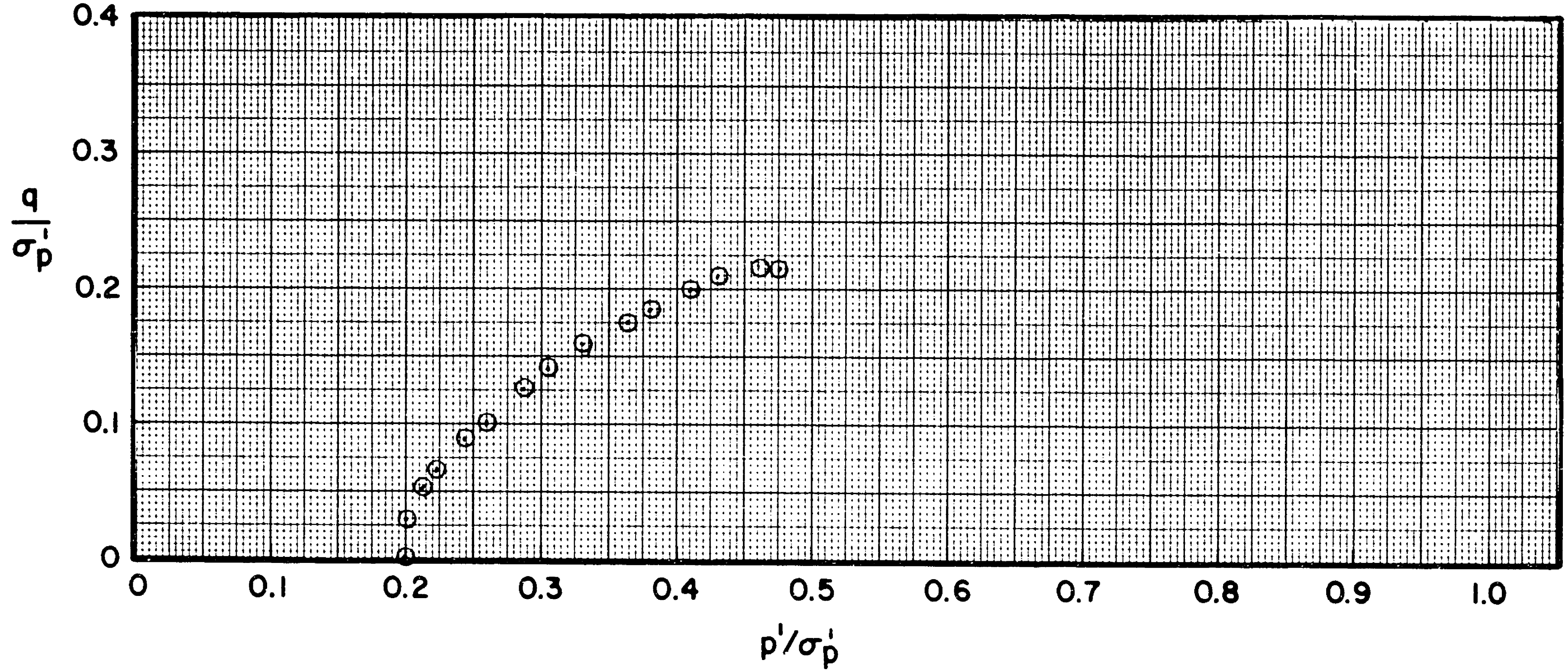
FIGURE

Test No.	Sample No.	Depth (RE) ft.	w_N (%)	σ'_{vc} ksf	K_c	OCR	Sym.
T5B117	T5B1- P2	4.25	36.7	2.43	1.00	5	⊙

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M. I. T.

$$q = 0.5(\sigma_v - \sigma_h)$$

$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



NORMALIZED STRESS PATHS FROM CK_oUC TESTS

BORING T5B1 SOIL TYPE Arctic Silt - Smith Bay

FIGURE

RECOMPRESSION TESTS

T5B1T1

T5B1T2

T5B1T3

T5B1T8

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO
SOIL TYPE ARCTIC SILT
LOCATION SITE T
BORING NO. SB1 SAMPLE NO. T1
DEPTH 1.85'

TEST NO. SB1T1
TYPE OF TEST CIUC (DECOMP)
APPARATUS NO. WFE 7
TESTED BY CAB
DATE 01/31/86

WATER CONTENT
INITIAL, BASED ON TRIMMINGS 40.3 %
INITIAL, BASED ON SAMPLE – %
FINAL, BASED ON SAMPLE 40.6 %

ATTERBERG LIMITS
W_P 25.6 %
W_L 49.5 %
I_P 23.9 % I_L

PHASE RELATIONSHIPS
γWET 1.786 g/cc γDRY 1.273 g/cc
e_i 0.998 e_f 0.998
S_i 99.8 % $S_{precons}$ 100 %
G_s 2.542

MISCELLANEOUS
B 98 %
SATURATION ΔV 0.16 cc.
CONSOLIDATION ΔV –
MEMBRANES – THICK 2 THIN
CORRECTION FACTOR 1.942 E
FILTER STRIPS 8 X 1/4"
CONFIGURATION VERTICAL
CORRECTION FACTOR 40.64 E
AREA CORRECTION PARABOLIC.

GRAIN SIZE
% -#200
% -2μ 48
C_u – C_c –

SAMPLE APPEARANCE

FINAL AREA ( )

TOP
MIDDLE
BOTTOM
AVERAGE

INITIAL, FRONT FINAL, FRONT FINAL, SIDE FINAL, X-RAY

COMMENTS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
- M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC (RECONP.) TEST NO. NBIT1

STRESS HISTORY

IN SITU CONDITIONS

σ'_{vo} 0.050 KSC.
σ'_p 0.684 KSC
OCR 13.7

TORVANE STRENGTH –
TORVANE W_c – %

TEST CONDITIONS

σ'_{vc} 0.052 KSC
σ'_p 0.052 KSC
OCR 1
K_c 0.96 U_b 1.953 KSC.
STRAIN RATE 0.5 %/HOUR
FINAL ε_a (SHEAR) 21.56 %

STRENGTH DATA

AT MAXIMUM q

ε_a 20.7 %
q/σ'_{vc} 1.54
p'/σ'_{vc} 3.29
$\Delta u - \Delta\sigma_h/\sigma'_{vc}$ -0.788
ϕ' 28.0
A_f -0.614
TIME TO q_{max} 41.4 HR.

AT MAXIMUM OBLIQUITY

ε_a 13.1 %
q/σ'_{vc} 1.35
p'/σ'_{vc} 2.54
$\Delta u - \Delta\sigma_h/\sigma'_{vc}$ -0.231
ϕ' 32.1

$E_{u(50\%)}/q_f$ 13.1

TIME RECORD

SET UP
START OF CONSOLIDATION –
START OF SHEAR
END OF SHEAR
REMOVAL
TOTAL TIME IN APPARATUS
CONSOLIDATION-SHEAR Δt

HYPERBOLIC STRESS-STRAIN PARAMETERS

G_i/σ'_{vc} 455
R_f 0.917
r^2 0.9969

RADIOGRAPHY

kV 160 mA 3.8
EXPOSURE TIME 45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC (RECOMP) TEST NO. TSB/T1

SAMPLE DIMENSIONS

	L (cm)	A (cm²)	V (cm³)	ε_a (b) (%)	ε_{vol} (b) (%)	W (g)
INITIAL	8.016	9.919	79.51	–	–	141.97
PRECONSOLIDATION	8.016	9.919	79.51	–	–	142.13
PRESHEAR	8.016	9.919	79.51	–	–	142.13
POST SHEAR	6.321	12.579	79.51	21.15	–	142.13
FINAL	6.321	12.579	79.51	21.15	–	142.13
FINAL MEASURED	6.275	–	–	21.72	–	141.12

(a) Measured
(b) Based on initial dimensions.

CONSOLIDATION DATA

STRESSES IN

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

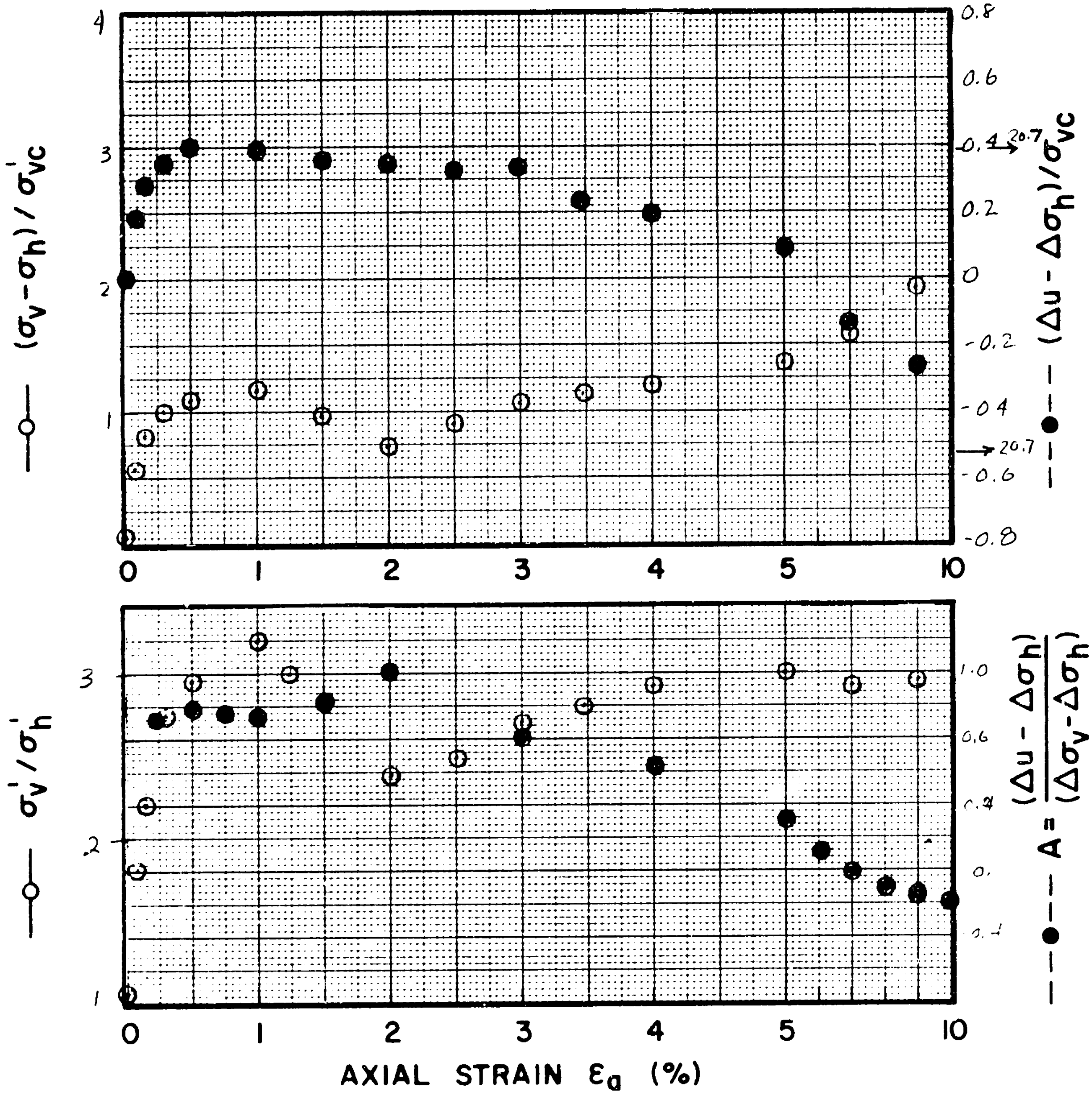
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED—UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CIUC No. TS8IT1

Time (hr)	ε_a (%)	$\frac{(\sigma_v - \sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u - \Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{q_f}$
	0.000	–	–	–	–	–	–	–	–
	0.244	0.958	0.346	2.523	0.359	0.479	1.096	243.9	0.300
	0.495	1.084	0.404	2.957	0.387	0.542	1.096	136.5	0.341
	0.757	1.164	0.442	3.190	0.373	0.582	1.115	96.1	0.367
	1.004	1.155	0.442	3.202	0.370	0.578	1.096	71.8	0.364
	1.511	0.986	0.423	2.818	0.414	0.493	1.019	40.4	0.309
	2.029	0.755	0.385	2.320	0.512	0.378	0.942	229	0.234
	3.024	1.074	0.346	2.714	0.300	0.537	1.154	22.1	0.339
	4.065	1.219	0.327	2.904	0.218	0.610	1.250	18.8	0.385
	5.025	1.366	0.288	3.020	0.144	0.683	1.346	17.0	0.433
	6.030	1.604	–	3.088	0.062	0.802	–	16.7	0.510
	7.013	1.591	0.115	2.864	-0.008	0.796	1.635	14.3	0.506
	8.075	1.639	0.058	2.799	-0.044	0.820	1.712	12.8	0.522
	9.078	1.936	-0.019	2.964	-0.075	0.968	1.942	13.5	0.618
	10.044	2.307	-0.077	3.221	-0.096	1.154	2.173	14.6	0.738
	11.058	2.316	-0.115	3.126	-0.130	1.158	2.231	13.3	0.741
	12.025	2.386	-0.154	3.116	-0.151	1.193	2.308	12.6	0.764
	13.101	2.702	-0.231	3.262	-0.157	1.351	2.538	13.1	0.867
	14.038	2.823	-0.308	3.199	-0.173	1.412	2.673	12.8	0.906
	15.077	2.733	-0.404	2.991	-0.201	1.367	2.731	11.6	0.877
	16.029	2.764	-0.423	2.974	-0.212	1.382	2.769	10.9	0.887
	17.004	3.004	-0.481	3.065	-0.209	1.502	2.942	11.2	0.964
	18.020	3.000	-0.577	2.938	-0.224	1.500	3.019	10.7	0.963
	19.076	2.914	-0.635	2.814	-0.240	1.457	3.038	9.7	0.935
	20.708	3.113	-0.788	2.774	-0.307	1.557	3.288	9.6	1.000

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



Sample No. 5B1-P1 w_N(%) 40.3 σ'_{vc}(ksf) 0.11 K_c 0.96

Depth (PE) 1.25' w_L(%) ____ OCR 1 t_c(Days) ____

Soil Type Smith Bay Arctic Silt w_P(%) ____ Estimated σ'_{vo}(KSF) 0.102

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

TEST No. T5B1T1

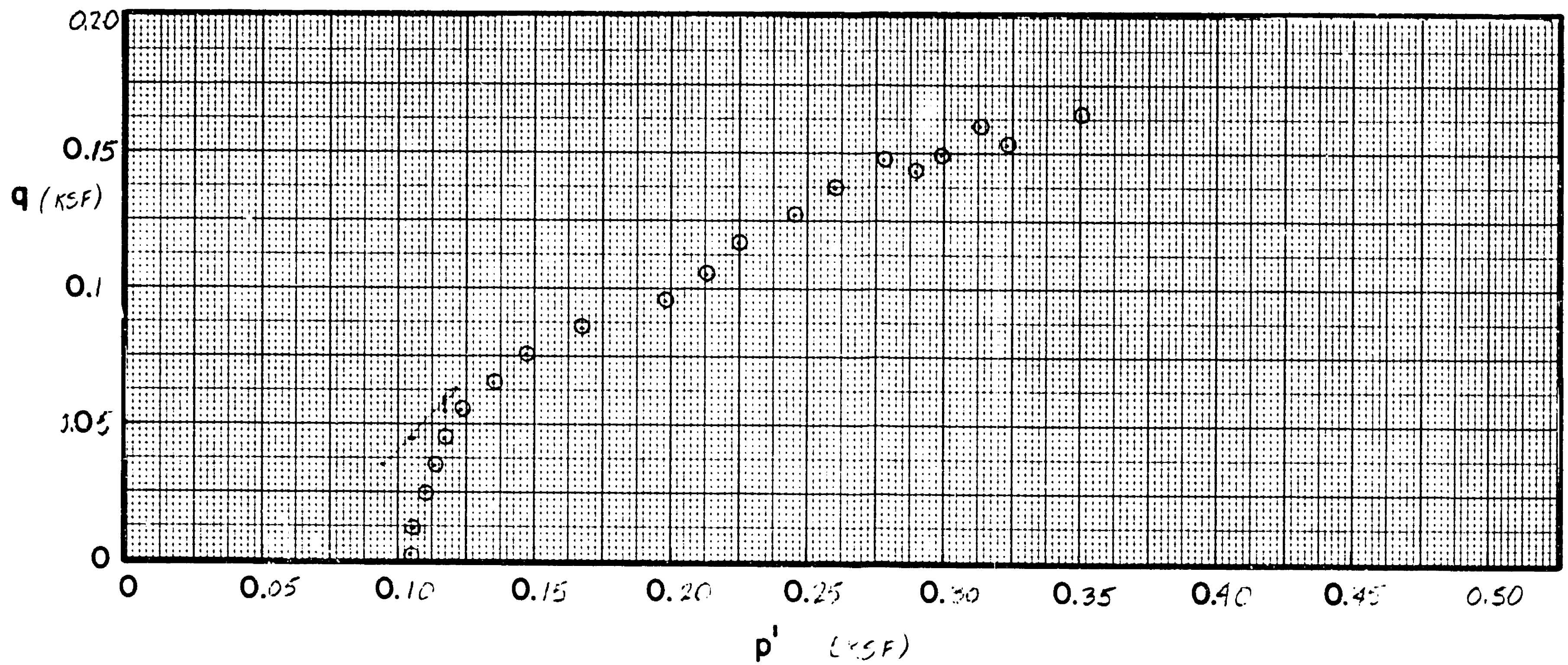
FIGURE

Test No.	Sample No.	Depth (FE)	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Sym.
TSB1T1	TSB1 - P1 (B)	1.85'	40.3	0.11	0.96		⊙

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M.I.T.

$$q = 0.5(\sigma_v - \sigma_h)$$

$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



STRESS PATHS FROM C I UC TESTS (Recompression)

BORING TSB1 SOIL TYPE ARCTIC SILT - SMITH BAY

FIGURE

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT: SOHIO

SOIL TYPE: ARCTIC SILT

LOCATION: SITE T

BORING NO. VB1 SAMPLE NO. T2

DEPTH: 8.24'

TEST NO. VB1T2

TYPE OF TEST: CIUC (RECONP.)

APPARATUS NO. WFE6

TESTED BY: ADQ.

DATE: 07/01/86

WATER CONTENT

INITIAL, BASED ON TRIMMINGS 33.8 %

INITIAL, BASED ON SAMPLE – %

FINAL, BASED ON SAMPLE 34.4 %

ATTERBERG LIMITS

W_p 23.7 %

W_L 45.2 %

I_p 21.5 % I_L

PHASE RELATIONSHIPS

γWET 1.920 g/cc. γDRY 1.435 g/cc.

e_i 0.891 e_f 0.891

S_i 99.8 % $S_{precons}$ 100 %

G_s 2.712

MISCELLANEOUS

B 95 %

SATURATION ΔV 0.06 cc.

CONSOLIDATION ΔV –

MEMBRANES – THICK 2 THIN

CORRECTION FACTOR 1.942 E

FILTER STRIPS 8 X 1/4"

CONFIGURATION VERTICAL

CORRECTION FACTOR 40.64 E

AREA CORRECTION PARABOLIC

GRAIN SIZE

% -#200

% -2μ 44

C_u C_c

SAMPLE APPEARANCE

INITIAL, FRONT

FINAL, FRONT

FINAL, SIDE

FINAL, X-RAY

FINAL AREA ( )

TOP

MIDDLE

BOTTOM

AVERAGE

COMMENTS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
- M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO **TYPE OF TEST** CIUC RECON? **TEST NO.** TSB1T2

STRESS HISTORY

IN SITU CONDITIONS

- σ'_{vo} 0.221 KSC.
- σ'_{p} 1.710 KSC
- OCR 7.7
- TORVANE STRENGTH –
- TORVANE W_c – %

TEST CONDITIONS

- σ'_{vc} 0.336
- σ'_{p} 0.336
- OCR 1
- K_c 1.01 U_b 1.879 KSC.
- STRAIN RATE 0.5 %/HOUR
- FINAL ε_a (SHEAR) 21.16 %

STRENGTH DATA

AT MAXIMUM q

- ε_a 17.7 %
- q/σ'_{vc} 1.54
- p'/σ'_{vc} 2.94
- $\Delta u - \Delta\sigma_h/\sigma'_{vc}$ -0.378
- ϕ' 31.7
- A_f -0.241
- TIME TO q_{max} 35.4 HR.

AT MAXIMUM OBLIQUITY

- ε_a 2.53 %
- q/σ'_{vc} 0.982
- p'/σ'_{vc} 1.63
- $\Delta u - \Delta\sigma_h/\sigma'_{vc}$ 0.363
- ϕ' 37.0
- $E_{u(50\%)}/q_f$ 50.7

TIME RECORD

- SET UP
- START OF CONSOLIDATION –
- START OF SHEAR
- END OF SHEAR
- REMOVAL
- TOTAL TIME IN APPARATUS
- CONSOLIDATION-SHEAR Δt

HYPERBOLIC STRESS-STRAIN PARAMETERS

- G_i/σ'_{vc} 81.3
- R_f 0.936
- r^2 0.9962

RADIOGRAPHY

- kV 160 mA 3.8
- EXPOSURE TIME 45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT Sohio TYPE OF TEST CIUC (REC.) TEST NO. TSBIT2

SAMPLE DIMENSIONS

	L (cm)	A (cm²)	V (cm³)	ε_a [b] (%)	ε_{vol} [b] (%)	W (g)
INITIAL	8.054	9.707	78.18	–	–	150.07
PRECONSOLIDATION	8.054	9.707	78.18	–	–	150.13
PRESHEAR	8.254	9.707	78.18	–	-	150.13
POST. SHEAR	6.350	12.312	78.18	21.16	—	150.13
FINAL	6.350	12.312	78.18	21.16	–	150.13
FINAL MEASURED	6.364	–	—	20.98	–	151.53

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN ____________

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

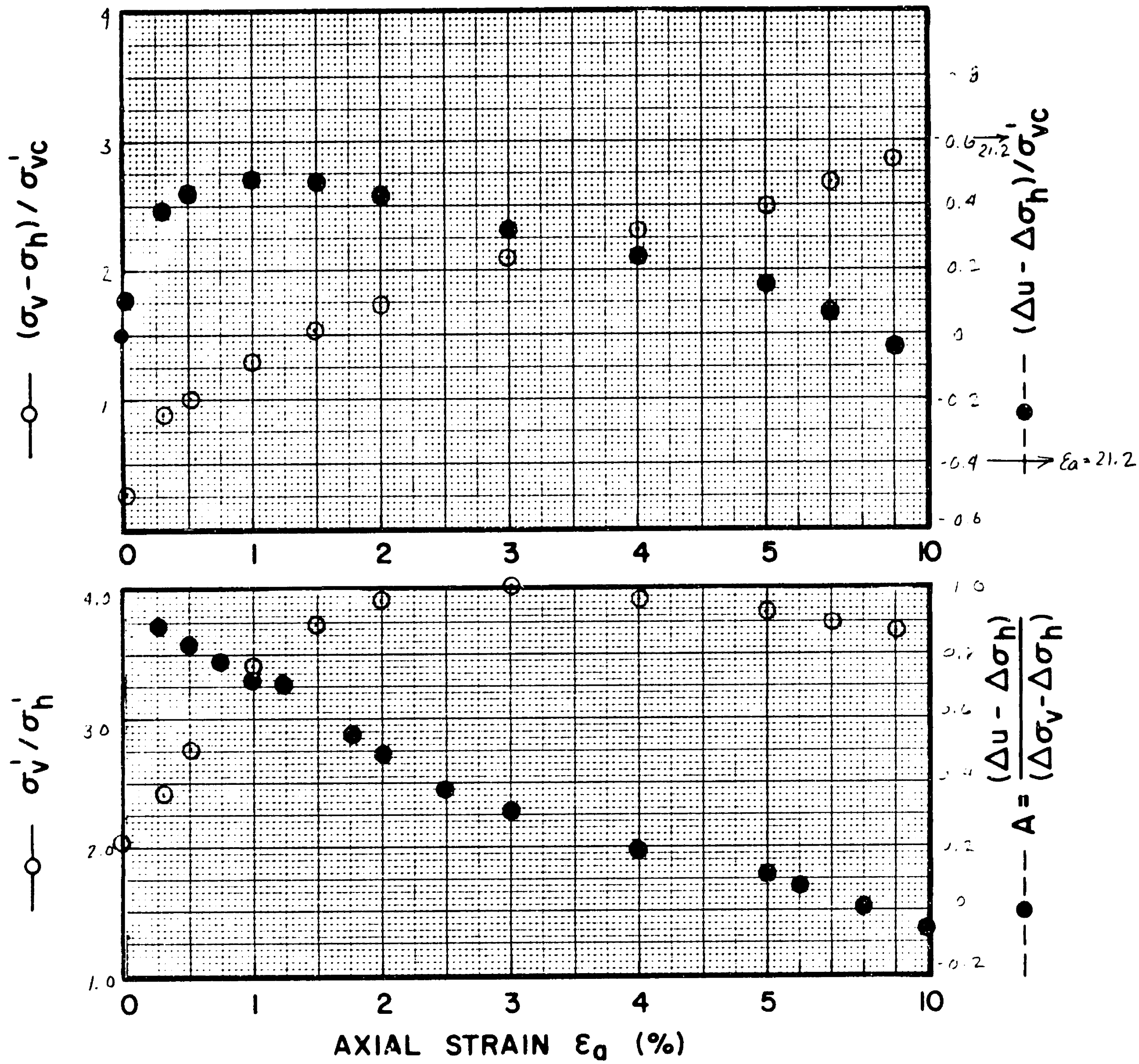
STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED—UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CIUC No. TSBIT2

Time (hr)	ε_a (%)	$\frac{(\sigma_v-\sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u-\Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{q_f}$
	0.000	—	—	—	—	—	—	—	—
	0.243	0.816	0.366	2.259	0.444	0.408	1.057	221.4	0.268
	0.513	1.030	0.435	2.778	0.419	0.515	1.095	131.8	0.337
	0.752	1.179	0.461	3.133	0.390	0.590	1.140	102.8	0.385
	1.023	1.314	0.473	3.424	0.359	0.657	1.199	84.1	0.428
	1.260	1.262	0.452	3.239	0.358	0.631	1.193	66.7	0.412
	1.758	1.536	0.423	3.586	0.277	0.768	1.360	57.1	0.500
	2.002	1.748	0.420	3.930	0.243	0.874	1.470	57.0	0.568
	2.530	1.968	0.363	4.021	0.189	0.984	1.634	50.6	0.639
	3.047	2.109	0.315	4.009	0.153	1.055	1.756	45.1	0.685
	4.086	2.337	0.217	3.934	0.097	1.169	1.964	37.2	0.758
	5.039	2.495	0.137	3.843	0.059	1.248	2.125	44.1	0.809
	6.084	2.587	0.095	3.812	0.040	1.294	2.211	27.7	0.839
	7.024	2.683	0.048	3.776	0.022	1.342	2.307	24.9	0.870
	8.040	2.780	0.003	3.745	0.003	1.390	2.402	22.4	0.901
	9.057	2.851	-0.048	3.681	-0.016	1.426	2.488	20.4	0.924
	10.029	2.914	-0.086	3.647	-0.030	1.457	2.557	18.9	0.944
	11.066	2.949	-0.131	3.570	-0.045	1.475	2.619	17.4	0.956
	12.015	2.970	-0.173	3.500	-0.058	1.485	2.673	16.1	0.962
	13.069	3.005	-0.211	3.447	-0.071	1.503	2.729	15.0	0.974
	14.035	2.996	-0.244	3.376	-0.082	1.498	2.759	13.9	0.971
	15.161	3.039	-0.289	3.329	-0.094	1.520	2.821	13.0	0.984
	16.022	3.057	-0.315	3.295	-0.103	1.529	2.860	12.5	0.990
	17.063	3.078	-0.357	3.241	-0.115	1.539	2.911	11.6	0.997
	17.684	3.082	-0.378	3.211	-0.121	1.541	2.935	11.3	0.999
	18.039	3.070	-0.375	3.208	-0.121	1.535	2.923	11.0	0.995
	19.001	3.047	-0.372	3.194	-0.122	1.524	2.911	10.5	0.987
	20.128	3.039	-0.390	3.162	-0.128	1.520	2.923	9.8	0.984
	21.078	3.035	-0.402	3.140	-0.133	1.518	2.935	9.4	0.983



Sample No. T5B1-P3 w_N(%) 33.8 σ'_{vc}(ksf) ____ K_c 1.01

Depth (RE) 8.24' w_L(%) ____ OCR 1 t_c(Days) ____

Soil Type Smith Bay Arctic Silt w_P(%) ____ Estimated σ'_{vo}(KSF) 0.452

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

TEST No. T5B1T2

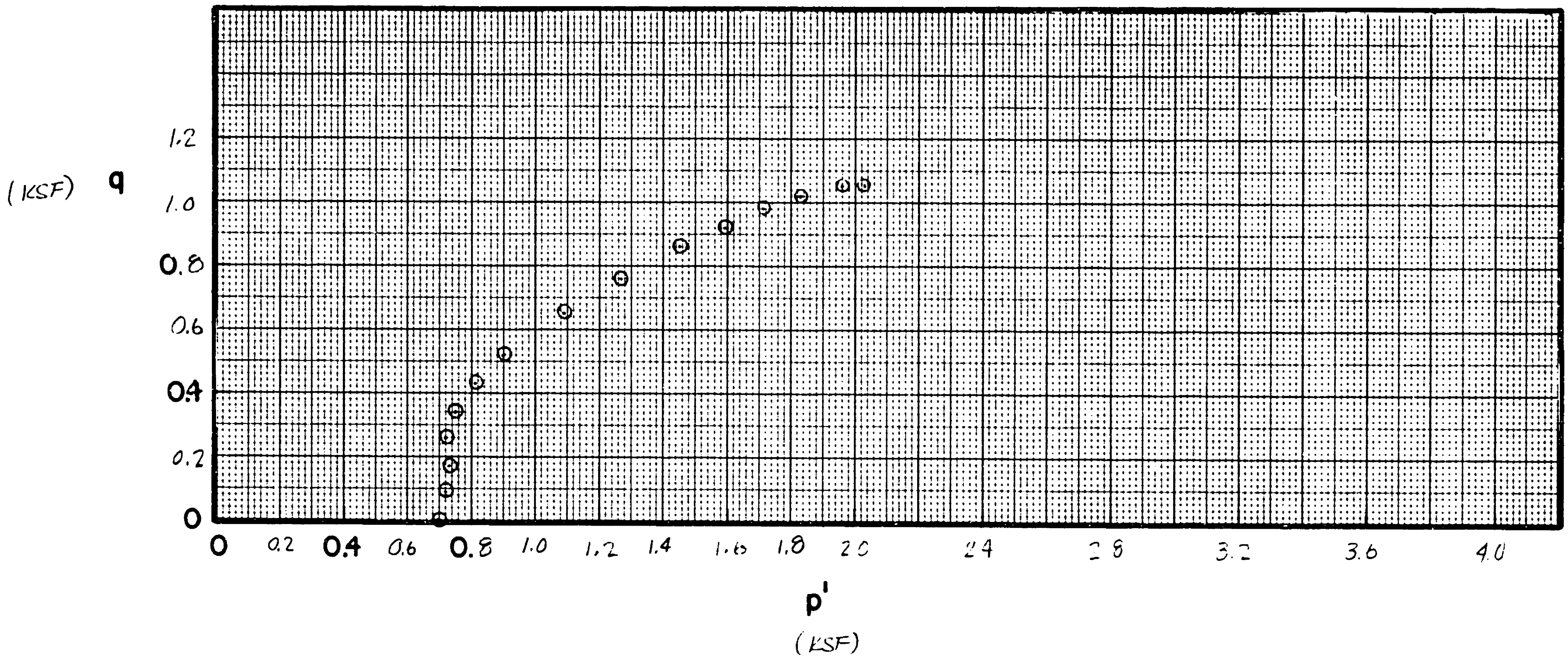
FIGURE

Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Sym.
TSB1T2	TSB1-P3	8.24'	33.8	0.69	1.01		⊙

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M. I. T.

$$q = 0.5(\sigma_v - \sigma_h)$$

$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



STRESS PATHS FROM CIUC TESTS (Recompression)

BORING TSB1 SOIL TYPE ARCTIC SILT

FIGURE

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT: SOHIO
SOIL TYPE: ARCTIC SILT
LOCATION: SITE T
BORING NO.: SB1 SAMPLE NO.: T3
DEPTH: 10.7'

TEST NO.: TSB1T3
TYPE OF TEST: CIUC RECOMP
APPARATUS NO.: WFE 4
TESTED BY: CDG
DATE: 07/01/86

WATER CONTENT
INITIAL, BASED ON TRIMMINGS: 21.6 %
INITIAL, BASED ON SAMPLE: – %
FINAL, BASED ON SAMPLE: 23.0 %

ATTERBERG LIMITS
W_P: 19.0 %
W_L: 39.5 %
I_P: 20.5 % I_L: ______

PHASE RELATIONSHIPS
γ_{WET}: 2.084 g/cc. γ_{DRY}: 1.714 g/cc.
e_i: 0.583 e_f: 0.583
S_i: 97.5 % $S_{precons}$: 100 %
G_s: 2.713

MISCELLANEOUS
B: 105 %
SATURATION ΔV: 0.71 cc.
CONSOLIDATION ΔV: –
MEMBRANES: – THICK 2 THIN
CORRECTION FACTOR: 1.942 ε
FILTER STRIPS: 8 X 1/4"
CONFIGURATION: VERTICAL
CORRECTION FACTOR: 40.64 ε
AREA CORRECTION: PARABOLIC

GRAIN SIZE
% -#200: ______
% -2μ: 42
C_u ______ C_c ______

SAMPLE APPEARANCE

–

FINAL AREA
( )

INITIAL, FRONT FINAL, FRONT FINAL, SIDE FINAL, X-RAY

TOP
MIDDLE
BOTTOM
AVERAGE

COMMENTS ______

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
- M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST CIUC (RECOMP) TEST NO. TSB1T3

STRESS HISTORY

IN SITU CONDITIONS

σ'_{vo} 0.288 ksc.

σ'_p 3.665 ksc.

OCR 12.7

TORVANE STRENGTH —

TORVANE W_c — %

TEST CONDITIONS

σ'_{vc} 0.769

σ'_p 0.769

OCR 1

K_c 0.82 U_b 1.880 ksc.

STRAIN RATE 0.5 %/HOUR

FINAL ε_a (SHEAR) %

STRENGTH DATA

AT MAXIMUM q

ε_a 19.8 %

q/σ'_{vc} 1.82

p/σ'_{vc} 3.80

$\Delta u-\Delta\sigma_h/\sigma'_{vc}$ -1.16

ϕ' 28.6

A_f -0.669

TIME TO q_{max} 39.6 HR.

AT MAXIMUM OBLIQUITY

ε_a 2.61 %

q/σ'_{vc} 1.01

p/σ'_{vc} 1.69

$\Delta u-\Delta\sigma_h/\sigma'_{vc}$ 0.142

ϕ' 36.6

$E_{u(50\%)}/q_f$ 38.8

TIME RECORD

SET UP

START OF CONSOLIDATION

START OF SHEAR

END OF SHEAR

REMOVAL

TOTAL TIME IN APPARATUS

CONSOLIDATION-SHEAR Δt

HYPERBOLIC STRESS-STRAIN PARAMETERS

G_i/σ'_{vc} 70.4

R_f 0.919

r^2 0.9959

RADIOGRAPHY

kV 160 mA 3.8

EXPOSURE TIME 45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT Sohio TYPE OF TEST CIUC (Recomp.) TEST NO. TSBIT3

SAMPLE DIMENSIONS

	L (cm)	A (cm²)	V (cm³)	ε_a [b] (%)	ε_{vol} [b] (%)	W (g)
INITIAL	8.054	9.456	76.16	–	–	158.72
PRECONSOLIDATION	8.054	9.456	76.16	–	–	159.43
PRESHEAR	8.054	9.456	76.16	–	–	159.43
POST SHEAR	6.417	11.868	76.16	20.33	–	159.43
FINAL	6.417	11.868	76.16	20.33	–	159.43
FINAL MEASURED	6.375	–	–	20.85	–	159.97

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN ____________

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

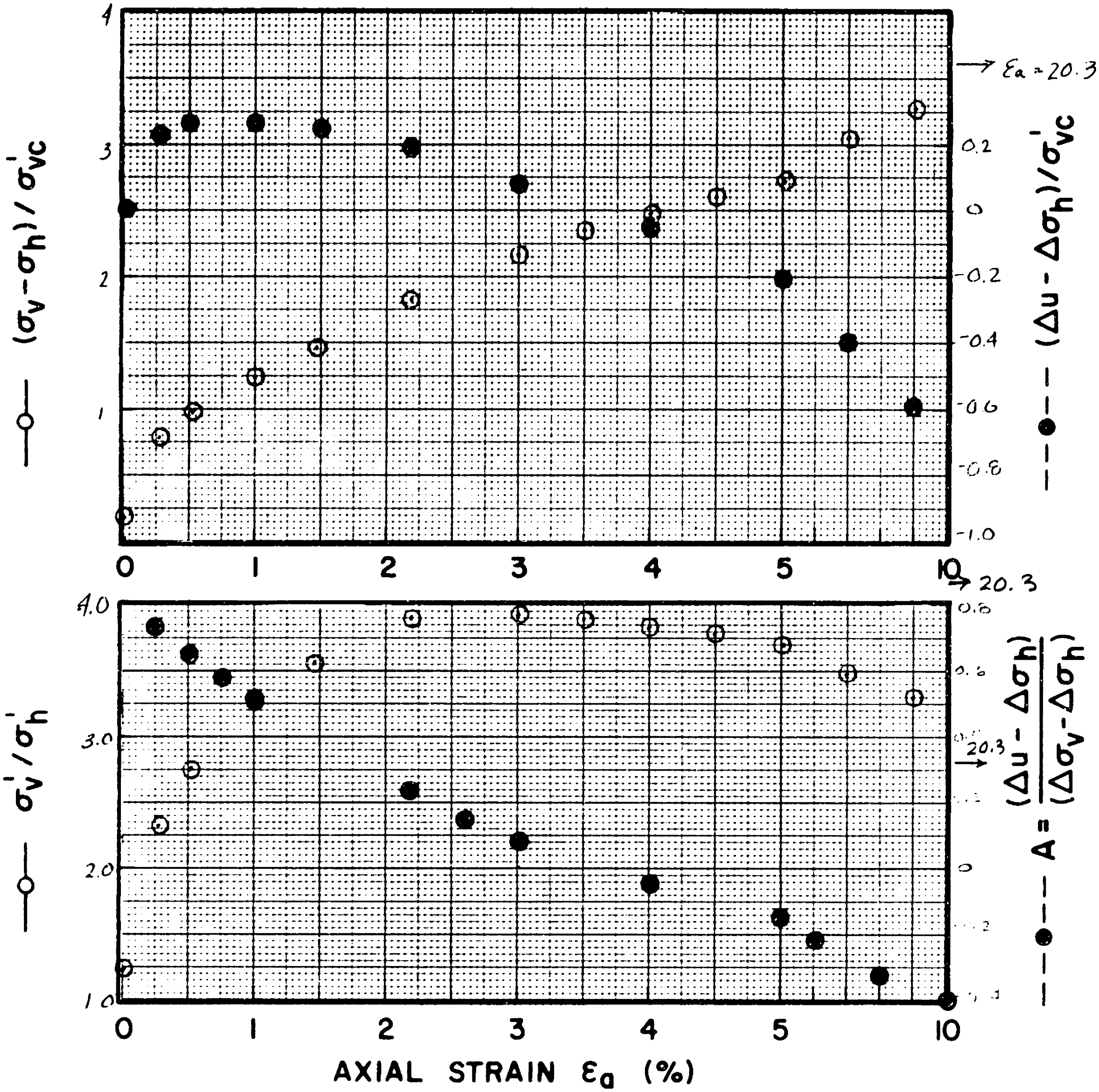
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED—UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CIUC No. TSBIT3

Time (hr)	ε_a (%)	$\frac{(\sigma_v-\sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u-\Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{q_f}$
	0.000	–	–	–	–	–	–	–	–
	0.257	0.795	0.230	2.340	0.372	0 398	0.992	132.6	0.179
	0.524	0.984	0.264	2.756	0.325	0 492	1.053	84.6	0.233
	0.753	1 117	0 273	3.027	0.289	0.559	1 111	68.6	0.272
	1 024	1.248	0.272	3.261	0.253	0.624	1.177	57.6	0.310
	2.183	1.824	0.196	3.904	0.118	0.912	1.541	41.5	0.476
	2.608	2.017	0 142	3.956	0 076	1.009	1.692	38.8	0.532
	3.033	2.182	0.082	3.938	0 041	1.091	1.835	36.4	0.579
	4.011	2.494	-0.056	3.837	-0.024	1.247	2.127	31.8	0.669
	5.048	2.735	-0.187	3.705	-0.076	1.368	2.380	27.9	0.739
	6.072	2.923	-0.307	3.586	-0.112	1.462	2.593	24.9	0.793
	7 035	3.055	-0.411	3.475	-0 118	1.528	2.763	22.6	0.831
	8.004	3.165	-0.510	3.375	-0.120	1.583	2.917	20.5	0.863
	9.040	3.256	-0.586	3.309	-0.190	1.628	3.040	18.7	0.889
	10.011	3.313	-0.650	3.247	-0.207	1.657	3.133	17.2	0.906
	11.012	3.370	-0 711	3.195	-0.223	1.685	3.222	15.9	0.922
	12.067	3.418	-0.765	3.152	-0.237	1.709	3.300	14.8	0.936
	13.042	3.450	-0.813	3.109	-0.249	1.725	3 363	13.8	0.945
	14.021	3.490	-0.865	3.068	-0.262	1.745	3.436	13.0	0.957
	15.073	3.534	-0.928	3.017	-0.277	1.767	3.520	12.2	0.970
	16.064	3.564	-0.980	2.976	-0.290	1.782	3.588	11.6	0.978
	17.047	3.588	-1.030	2.937	-0.302	1.794	3.649	11.0	0.985
	18.038	3.611	-1.078	2.899	-0.315	1.806	3.709	10.5	0.992
	19.033	3.627	-1.125	2.862	-0.327	1.814	3.765	10.0	0.997
	19.793	3.638	-1.157	2.837	-0.335	1.819	3.801	10.9	1.000
	20.323	3.632	-1.169	2.823	-0.339	1.816	3.810	9.3	0.998

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



Sample No. 5B1-P1 w_N(%) 21.6 σ'_{vc}(ksf) 1.57 K_c 0.92

Depth (RE) 10.7' w_L(%) ____ OCR 1.0 t_c(Days) ____

Soil Type Smith Bay Arctic silt w_p(%) ____ Estimated σ'_{vo}(ksf) 0.589

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

TEST No. T5B1T3

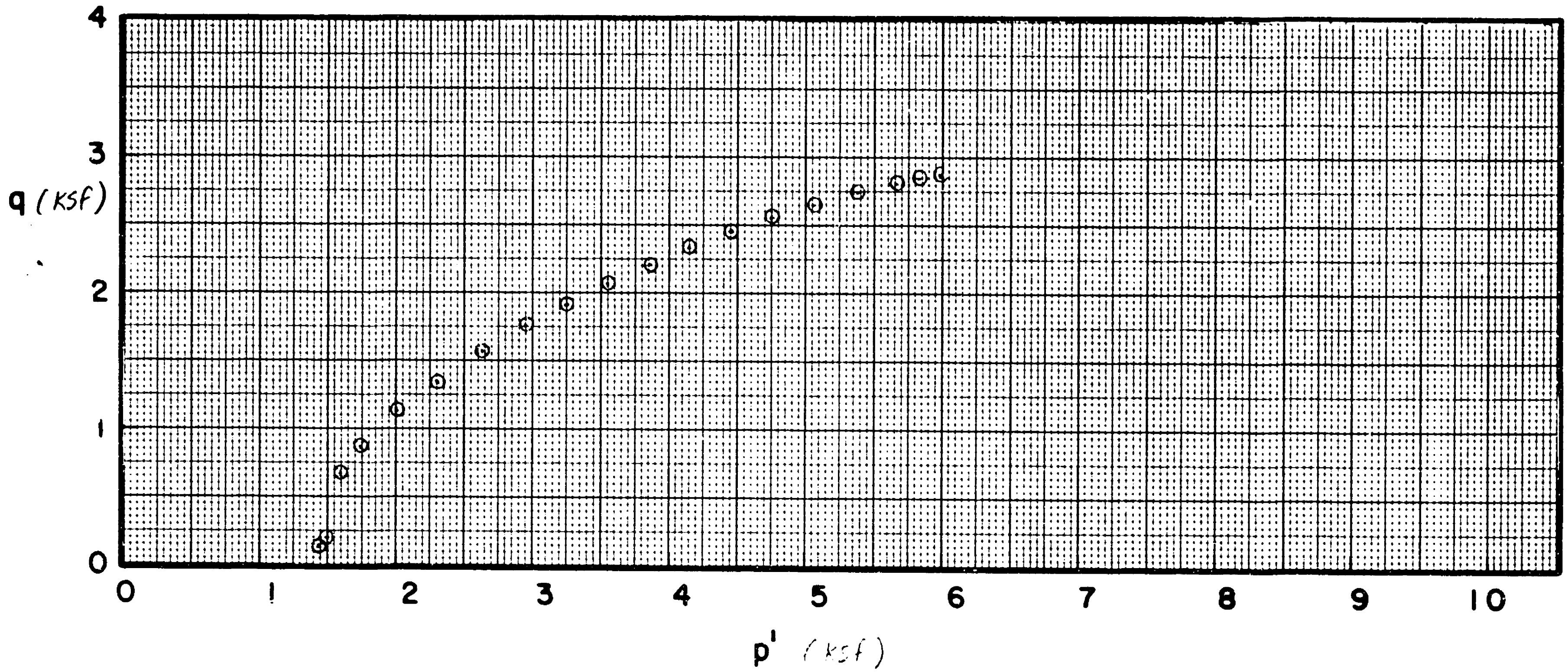
FIGURE

Test No.	Sample No.	Depth RE	w_N (%)	σ'_{vc}	K_c	OCR	Sym.
TSB1T3	TSB1-:4	107'	21.6	1.57	0.82		⊙

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M.I.T.

$$q = 0.5(\sigma_v - \sigma_h)$$

$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



STRESS PATHS FROM CIUC TESTS (Recompression)

BORING TSB1 SOIL TYPE ARCTIC SILT

FIGURE

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT: 20440
SOIL TYPE: ARCTIC SILT
LOCATION: SITE T
BORING NO.: SB1 SAMPLE NO.: TB
DEPTH: 10.8'

TEST NO.: TB1TB
TYPE OF TEST: RECOMP.
APPARATUS NO.: WFE 4
TESTED BY: CAB.
DATE: 03/03/86

WATER CONTENT

INITIAL, BASED ON TRIMMINGS 22.1 %
INITIAL, BASED ON SAMPLE - %
FINAL, BASED ON SAMPLE 24.3 %

ATTERBERG LIMITS

W_p 19.0 %
W_L 39.5 %
I_p 20.5 % I_L

PHASE RELATIONSHIPS

γWET 2.075 g/cc. γDRY 1.699 g/cc.
e_i 0.632 e_f 0.632
S_i 96.1 % $S_{precons}$ 100% %
G_s 2.773

MISCELLANEOUS

B 92 %
SATURATION ΔV 135 cc
CONSOLIDATION ΔV —
MEMBRANES - THICK 2 THIN
CORRECTION FACTOR 1.942E
FILTER STRIPS 8 X 1/4"
CONFIGURATION VERTICAL.
CORRECTION FACTOR 40.64·E
AREA CORRECTION PARABOLIC.

GRAIN SIZE

% -#200
% -2μ 42
C_u C_c

SAMPLE APPEARANCE

FINAL AREA ( )

TOP

MIDDLE

BOTTOM

AVERAGE

INITIAL, FRONT FINAL, FRONT FINAL, SIDE FINAL, X-RAY

COMMENTS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT Sohio TYPE OF TEST CIUC (Recon?) TEST NO. TJ3IT8

STRESS HISTORY

IN SITU CONDITIONS

σ'_{vo} 0.290 KSC.
σ'_p 3.665 KSC
OCR 12.6

TORVANE STRENGTH —
TORVANE W_c — %

TEST CONDITIONS

σ'_{vc} 0.350 KSC
σ'_p 0.350 KSC
OCR 1
K_c 1.00 U_b 1.998 KSC.
STRAIN RATE 0.5 %/HOUR
FINAL ε_a (SHEAR) 21.90 %

STRENGTH DATA

AT MAXIMUM q

ε_a 18.1 %
q/σ'_{vc} 2.66
p'/σ'_{vc} 5.51
$\Delta u - \Delta\sigma_h/\sigma'_{vc}$ -1.85
ϕ' 28.9
A_f -0.69
TIME TO q_{max} 36.2 HR.

AT MAXIMUM OBLIQUITY

ε_a 2.46 %
q/σ'_{vc} 1.45
p'/σ'_{vc} 2.39
$\Delta u - \Delta\sigma_h/\sigma'_{vc}$ 0.063
ϕ' 37.4

$E_{u(50\%)}/q_f$ 44.2

TIME RECORD

SET UP
START OF CONSOLIDATION
START OF SHEAR
END OF SHEAR
REMOVAL
TOTAL TIME IN APPARATUS
CONSOLIDATION-SHEAR Δt

HYPERBOLIC STRESS-STRAIN PARAMETERS

G_i/σ'_{vc} 108
R_f 0.917
r^2 0.9965

RADIOGRAPHY

kV 160 mA 3.3
EXPOSURE TIME 45"

REMARKS

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST

PROJECT SOHIO TYPE OF TEST DECOMP. TEST NO. TV-B1TB

SAMPLE DIMENSIONS

	L (cm)	A (cm²)	V (cm³)	ε_L (b) (%)	ε_{vol} (b) (%)	W (g)
INITIAL	8.054	10.075	81.14	—	—	168.35
PRECONSOLIDATION	8.054	10.075	81.14	—	—	170.20
PRESHEAR	8.054	10.075	81.14	—	—	170.20
POST SHEAR	6.290	12.900	81.14	21.90	—	170.20
FINAL	6.290	12.900	81.14	21.90	—	170.20
FINAL MEASURED	6.387	—	—	20.70	—	171.21

(a) Measured
(b) Based on initial dimensions

CONSOLIDATION DATA

STRESSES IN ____________

STEP	1	2	3	4	5	6	7	8	9	10
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	11	12	13	14	15	16	17	18	19	20
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

STEP	21	22	23	24	25	26	27	28	29	30
σ'_{vc}										
σ'_{hc}										
t_c (HRS)										
ε_a (%)										
ε_{vol} (%)										
K_c										

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

CONSOLIDATED—UNDRAINED TRIAXIAL TEST (continued)

Project SOHIO Soil ARCTIC SILT Type of Test CIUC No. TSB1TB

Time (hr)	ε_a (%)	$\frac{(\sigma_v-\sigma_h)}{\sigma'_{vc}}$	$\frac{(\Delta u-\Delta\sigma_3)}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_h}$	A	$\frac{q}{\sigma'_{vc}}$	$\frac{p'}{\sigma'_{vc}}$	$\frac{E_u}{q_f}$	$\frac{\Delta q}{q_f}$
	0.000	—	—	—	—	—	—	—	—
	0.006	0.231	0.083	1.252	0.356	0.116	1.034	1541.3	0.043
	0.102	0.719	0.254	1.965	0.352	0.360	1.106	265.9	0.135
	0.246	1.018	0.306	2.468	0.300	0.509	1.203	155.5	0.191
	0.516	1.356	0.323	3.003	0.238	0.678	1.357	98.9	0.255
	0.739	1.567	0.314	3.290	0.201	0.784	1.469	79.8	0.295
	1.012	1.805	0.300	3.576	0.165	0.903	1.606	67.1	0.340
	1.592	2.256	0.229	3.925	0.100	1.128	1.900	53.3	0.425
	2.080	2.617	0.143	4.058	0.054	1.309	2.166	47.3	0.493
	2.464	2.897	0.063	4.096	0.020	1.449	2.389	44.2	0.545
	3.150	3.337	-0.094	4.050	-0.032	1.669	2.766	39.9	0.628
	4.351	3.918	-0.394	3.811	-0.105	1.959	3.357	33.9	0.737
	5.409	4.299	-0.629	3.644	-0.150	2.150	3.780	29.9	0.809
	6.017	4.484	-0.749	3.566	-0.172	2.242	3.994	28.0	0.844
	7.006	4.706	-0.920	3.453	-0.200	2.353	4.277	25.4	0.886
	8.182	4.800	-1.054	3.338	-0.226	2.400	4.457	22.2	0.903
	9.024	4.861	-1.140	3.275	-0.240	2.431	4.574	20.3	0.915
	10.218	4.961	-1.234	3.223	-0.254	2.481	4.717	18.3	0.934
	11.119	4.993	-1.300	3.174	-0.264	2.497	4.797	16.9	0.940
	12.012	5.030	-1.383	3.113	-0.277	2.515	4.900	15.8	0.947
	13.010	5.091	-1.466	3.068	-0.289	2.546	5.014	14.7	0.958
	14.060	5.156	-1.546	3.027	-0.300	2.578	5.179	13.8	0.970
	15.047	5.185	-1.637	2.969	-0.315	2.593	5.231	13.0	0.976
	16.271	5.228	-1.740	2.911	-0.330	2.614	5.357	12.1	0.984
	17.482	5.274	-1.823	2.870	-0.343	2.637	5.463	11.4	0.993
	18.103	5.313	-1.849	2.868	-0.345	2.657	5.509	11.0	1.000
	19.026	5.296	-1.886	2.838	-0.353	2.648	5.534	10.5	0.997
	20.098	5.304	-1.920	2.819	-0.360	2.652	5.574	9.9	0.998
	21.018	5.284	-1.943	2.797	-0.366	2.642	5.589	9.5	0.995
	21.859	5.289	-1.980	2.777	-0.372	2.645	5.626	9.1	0.996

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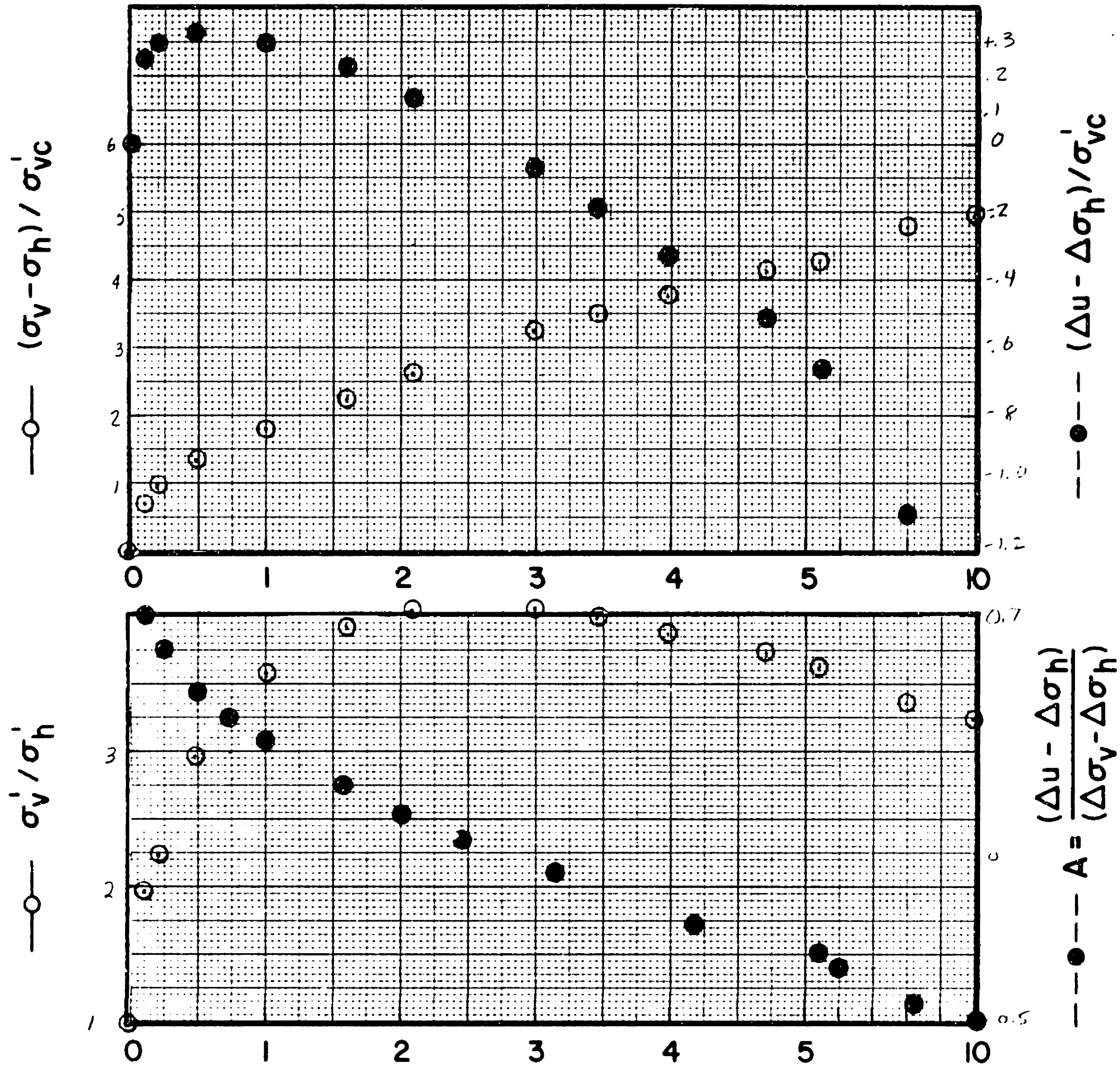

Sample No. 75B1- w_N(%) 22.1 σ'_{vc}(KSF) 0.716 K_c 1.00

Depth (BE) 10.8' w_L(%) ______ OCR 1.0 t_c(Days) ______

Soil Type Arctic Silt Smith Bay w_P(%) ______ Estimated σ'_{vo}(KSF) 0.593

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M.I.T.

NORMALIZED STRESS VS STRAIN
CK_oU TEST No. 75B1T8

FIGURE

Test No.	Sample No.	Depth	w_N (%)	σ'_{vc} (KSF)	K_c	OCR	Sym.
T5B1T8	T5B1	10.8'	22.1	0.716	1.00		⊙

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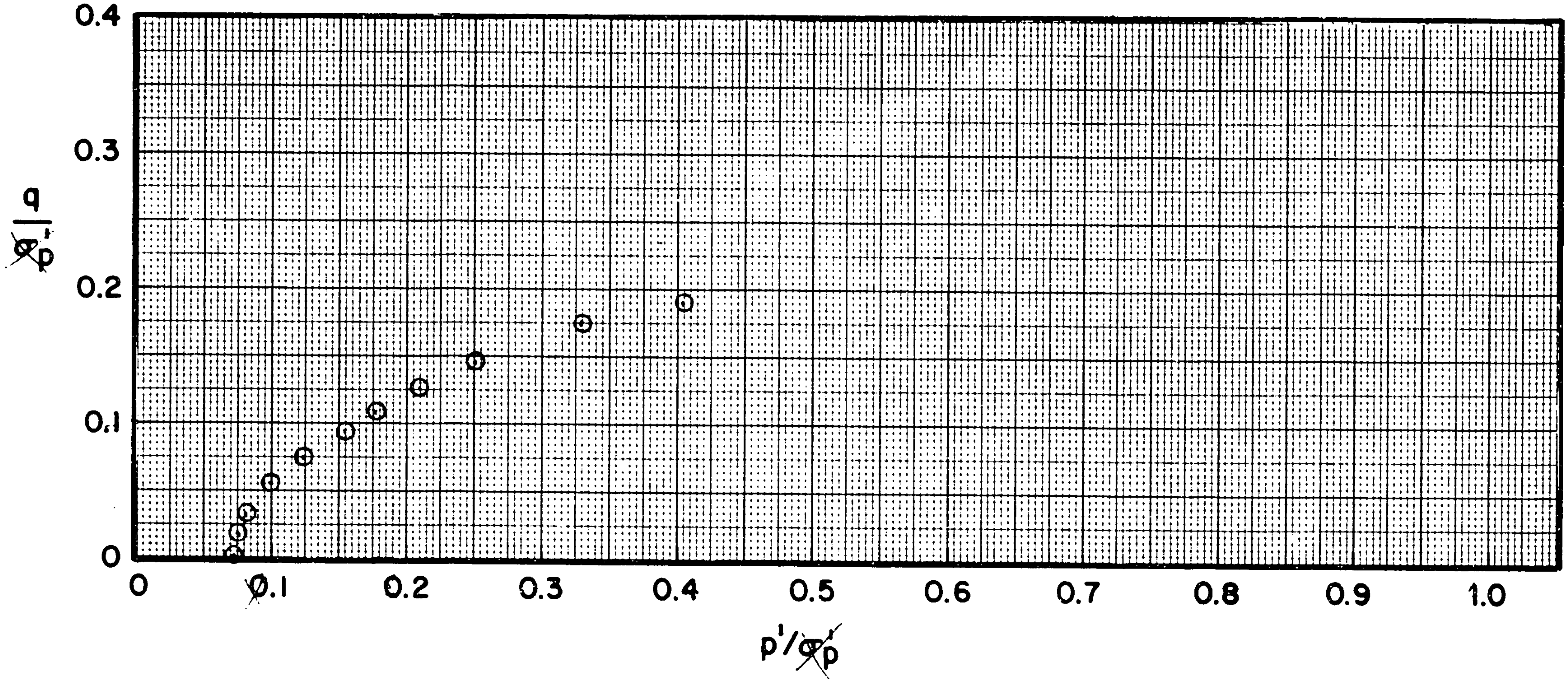
DEPT. OF CIVIL ENGR., M. I. T.

$$q = 0.5(\sigma_v - \sigma_h)$$

$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



NORMALIZED STRESS PATHS FROM CI UC TESTS (RECOMPRESSION)

BORING T5B1- SOIL TYPE Arctic Silt- Smith Bay

FIGURE

Appendix E
K_0-CONSOLIDATED DSS TESTS

Appendix E summarizes the laboratory esults and figures for each of the CK_0UDSS tests performed by MIT on Smith Bay Arctic silt form the 1985 test program. The test series included: eight normally consolidated and three overconsolidated tests at Site T; and six normally consolidated and three overconsolidated tests at Site W. The Geonor DSS apparatus used at MIT is described by Bjerrum and Landva (1966). The test procedure for all DSS tests are essentially the same as those presented in Appendix B of Ladd and Edgers (1972), but using cylindrical samples having an area of 35 cm^2 and a height of 2 to 2.5 cm. Also, tests on Arctic silt were automated to maintain constant volume. Section 6.2 describes the test equipment and procedures in detail. The tables and figures are presented in the following order:

(1) Consolidation curves;

(2) Tabulated shear results;

(3) Normalized stress paths;

(4) Normalized shear stress versus shear strain curves;

(5) Normalized modulus versus normalized shear stress curves.

Site T results are presented first (TDSS1 through TDSS11), followed by Site W results (WDSS1 through WDSS9).

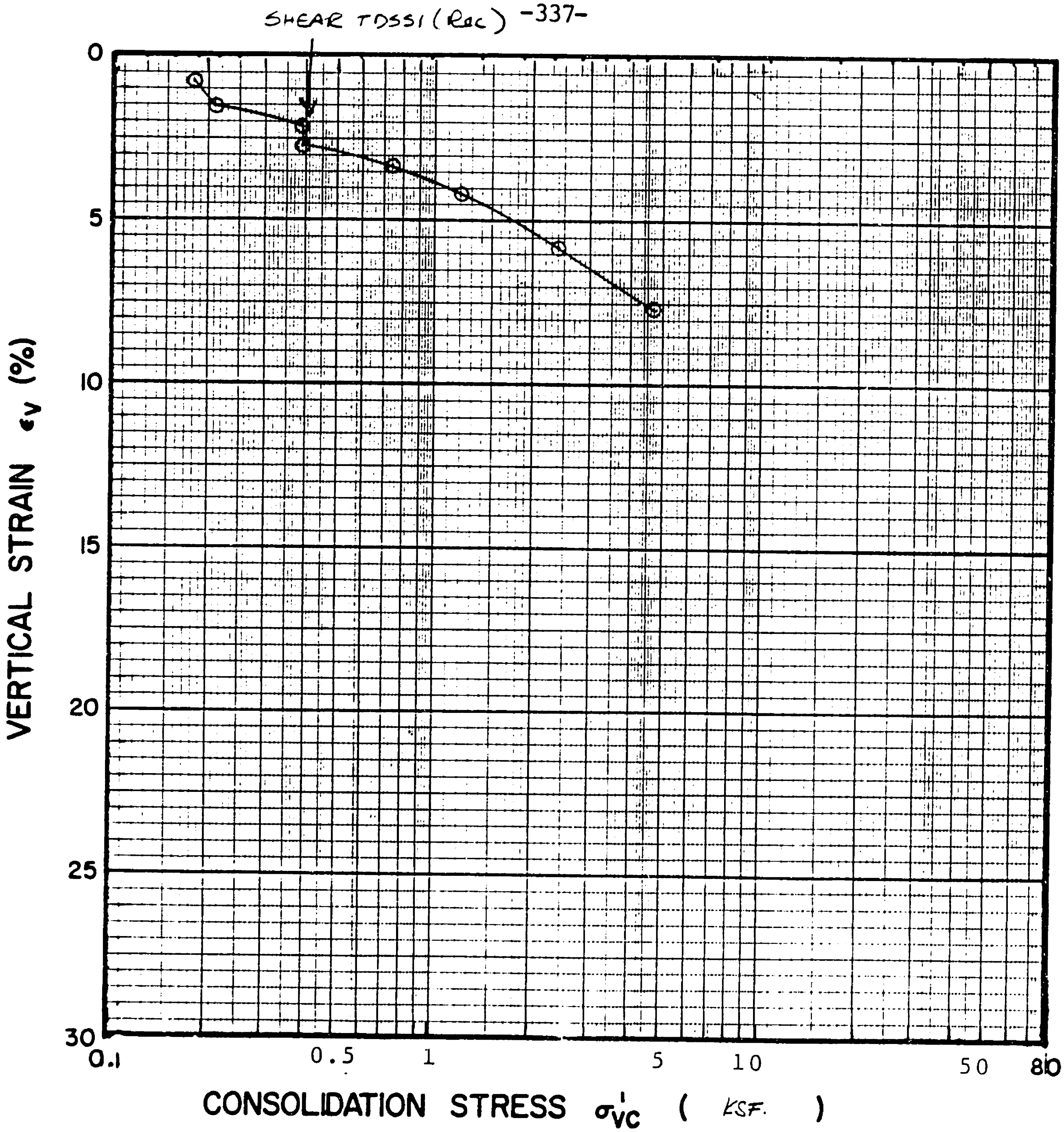
TABLE E-1: CALCULATION OF PRESHEAR WATER CONTENTS

SITE	TEST	W_t (1)	W_f (2)	W_p (3)	W_{init} (4)	W_{pre} (5)
T	DSS1*	37.5	36.9	44.1	51.4	36.0
	DSS1B*	37.5	36.9	40.01	45.7	31.8
	DSS2	40.2	36.3	34.6	42.5	32.6
	DSS3	54.4	48.4	42.4	57.0	40.2
	DSS4	35.3	29.5	26.0	35.6	26.9
	DSS5	37.6	33.9	32.1	39.8	27.8
	DSS6	41.6	34.8	31.1	40.5	32.2
	DSS7*	36.4	27.5	25.1	35.0	26.4
	DSS8*	38.3	32.5	30.7	40.5	34.1
	DSS9	45.9	38.9	30.6	49.5	27.3
	DSS10	38.4	31.9	27.1	38.3	26.8
	DSS11	42.9	39.4	36.2	39.8	35.4
W	DSS1	29.2	29.0	24.6	31.0	22.7
	DSS2	41.7	31.0	26.3	41.1	26.4
	DSS3	36.6	32.5	27.5	38.2	25.8
	DSS4	28.9	29.0	24.4	32.6	20.9
	DSS5	29.4	30.0	25.8	33.0	22.3
	DSS6	30.0	30.5	26.8	35.4	21.8
	DSS7	29.4	30.1	26.3	32.7	26.6
	DSS8	29.0	30.7	26.8	31.7	26.8
	DSS9*	28.9	29.8	27.6	31.6	27.1

* Used stones with pins
(1) Water content of trimmings
(2) Final water content (24 hours after shear)
(3) Preshear water content, backcalculated from (2)
(4) Initial water content, calculated from (2) and (3)
(5) Preshear water content, calculated from (1) and consolidation strains, in:

$$w_p = w_t - \varepsilon_c\left(\frac{1}{G} + w_t\right) \qquad \text{assuming } S = 100\%$$

SITE T TESTS



Sample No. T5B-P3 w_N (%) 37.5 Estimated

Depth (RE. 7.5 FT w_L (%) 35.3 σ'_{vo} 0.385 σ'_p

Soil Type ARCTIC SILT w_p (%) 23.2 CR — RR —

I_p (%) 12.1 G_s 2.75 e_o 1.05 S(%)

○ At t_p or hr Remarks Corrected for apparatus compressibility

● At ( ) hr

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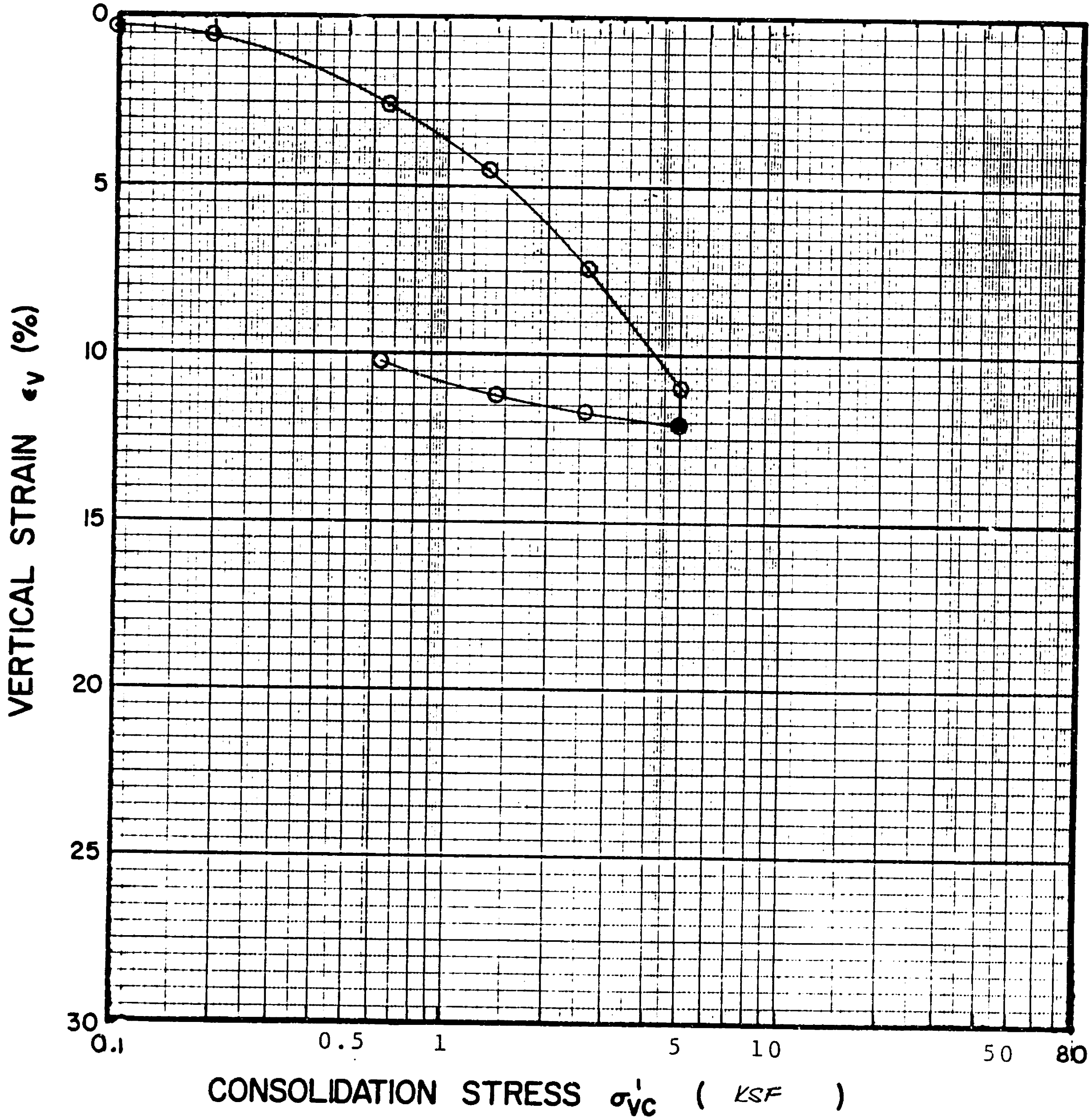
COMPRESSION CURVE

TEST NO. TDSS1B



Sample No. TBB-P3 w_N (%) 40.2 Estimated

Depth (RE) 7.7 FT w_L (%) 35.3 σ'_{vo} 0.385 σ'_p

Soil Type ARCTIC SILT w_P (%) 23.2 CR 0.125 RR —

I_P (%) 12.1 G_s 2.75 e_0 1.16 S(%) 95.3

○ At t_p or hr Remarks Corrected for apparatus

● At ( ) hr Compressibility

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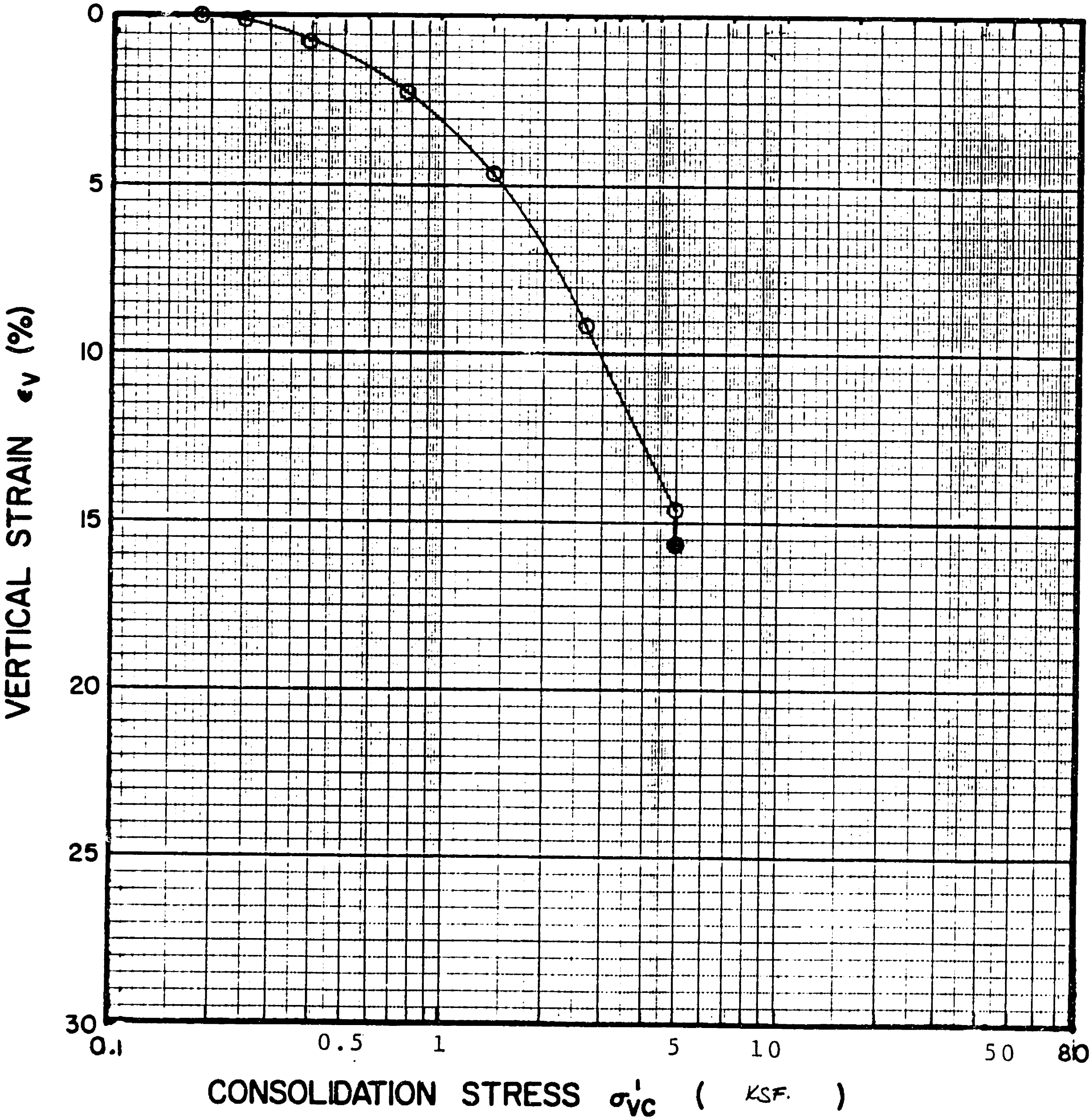
COMPRESSION CURVE

TEST NO. TDSS2 (TB3DSS2)



Sample No. T3B-P3 w_N (%) 54.4 **Estimated**

Depth (RE) 7.9 FT w_L (%) 65.8 σ'_{vo} 0.407 σ'_p

Soil Type ARCTIC SILT w_P (%) 32.7 CR 0.204 RR —

I_P (%) 33.1 G_s 2.75 e_o 1.609 S(%) 92.98

○ At t_p or hr

● At (t_f) hr

Remarks Corrected for apparatus Compressibility

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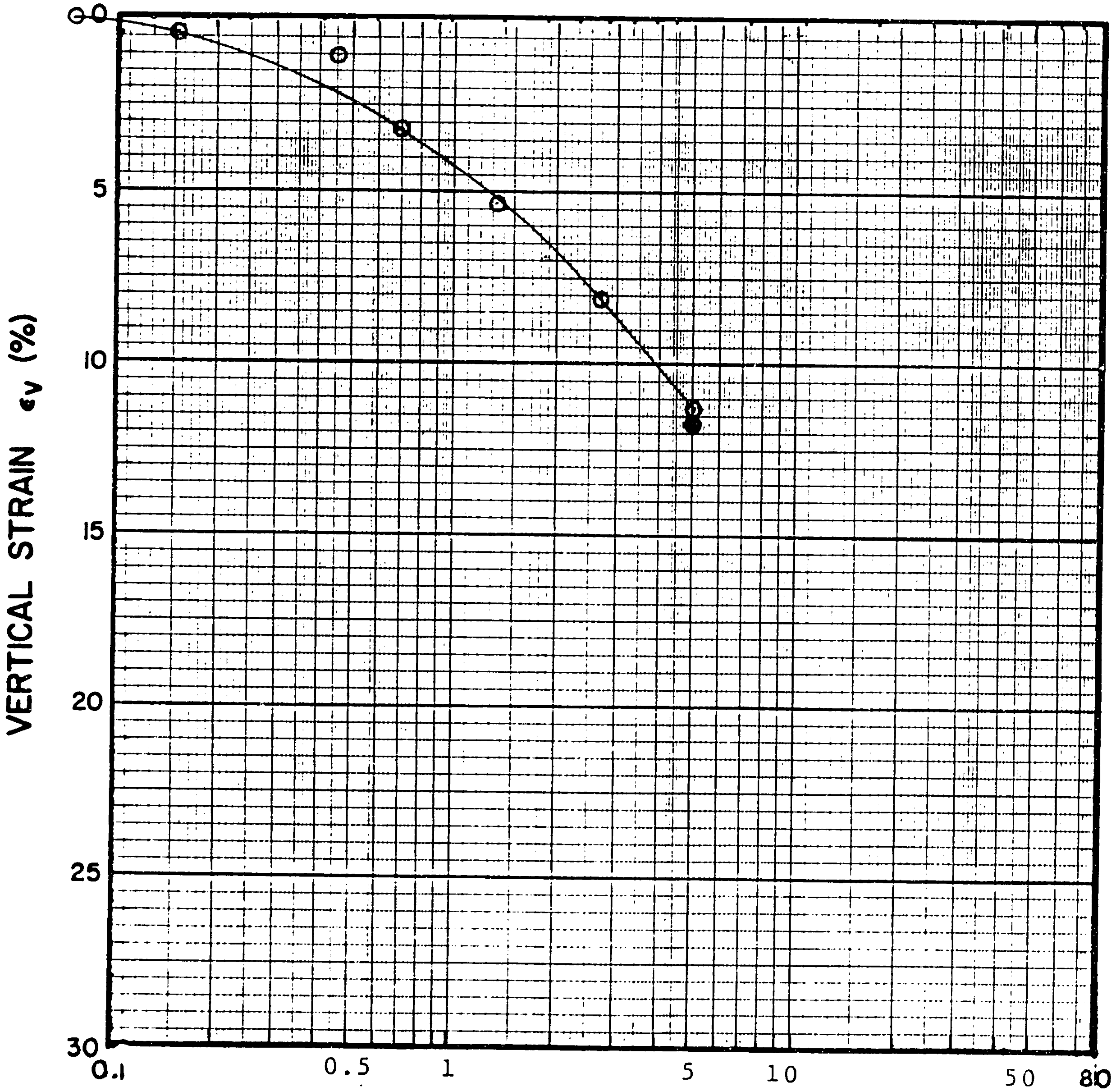
COMPRESSION CURVE

TEST NO. TDSS3 (TB3DSS3)

FIGURE



CONSOLIDATION STRESS σ'_{vc} (KSF)

Sample No. T3B-P2 w_N (%) 35.3 Estimated

Depth (RE) 4.7 FT w_L (%) 50.2 σ'_{vo} 0.23 σ'_p —

Soil Type ARCTIC SILT w_P (%) 26.2 CR 0.120 RR —

I_P (%) 24.0 G_s e_0 0.979 S(%)

○ At t_p or hr Remarks Corrected for apparatus compressibility

● At ( ) hr

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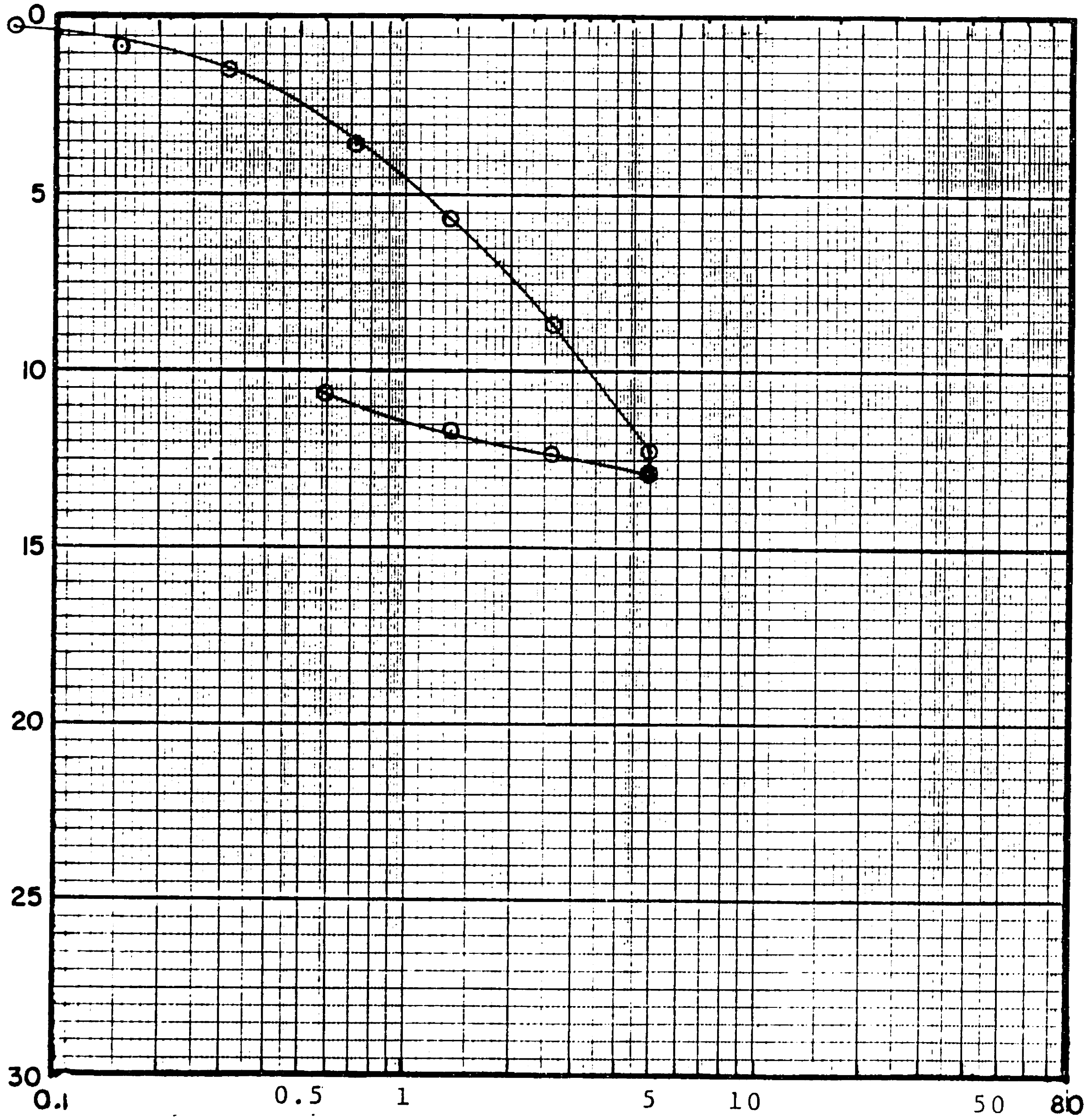
COMPRESSION CURVE

TEST NO. TDSS4 (TB3-DSS4)

FIGURE



Sample No. T3B-P2 | w_N (%) 37.2 | Estimated

Depth (RE) 4.6' | w_L (%) 50.2 | σ'_{vo} 0.226 σ'_p —

Soil Type ARCTIC SILT | w_P (%) 26.2 | CR 0.133 RR —

I_P (%) 24.0 | G_s 2.75 e_0 1.042 S(%)

○ At t_p or hr

● At (t_f) hr

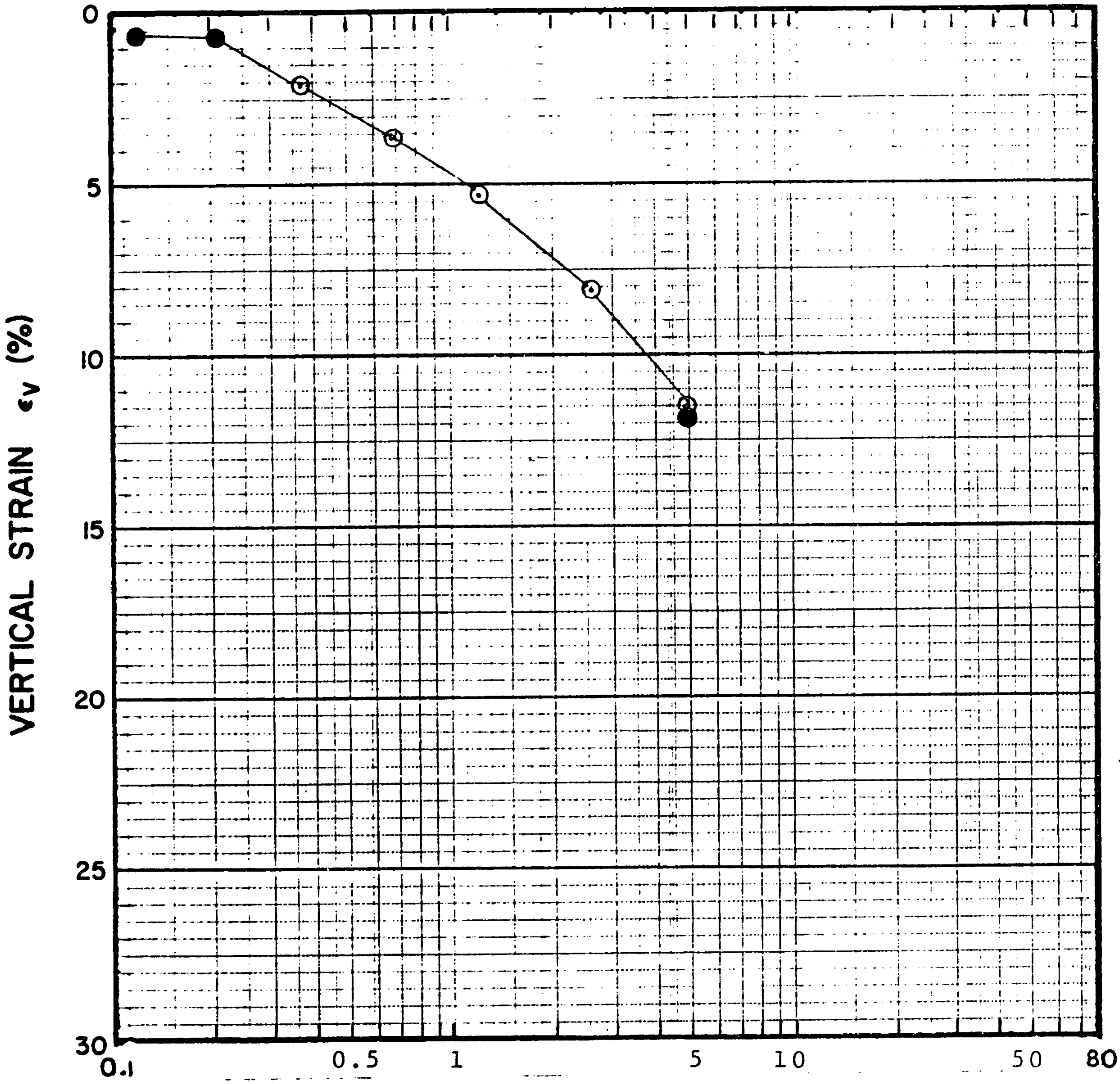
Remarks CORRECTED FOR APPARATUS COMPRESSIBILITY

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. TD555

FIGURE



CONSOLIDATION STRESS σ'_{vc} (KSF)

Sample No. TIB-02 | w_N (%) 41.5 | Estimated

Depth (RE) 7' | w_L (%) 52.7 | σ'_{vo} 0.220 σ'_p —

Soil Type ARCTIC SILT | w_P (%) 27.6 | CR 0.126 RR —

I_P (%) 25.1 G_s e_o 1.18 S(%)

o At t_p or hr

● At (t_f) hr

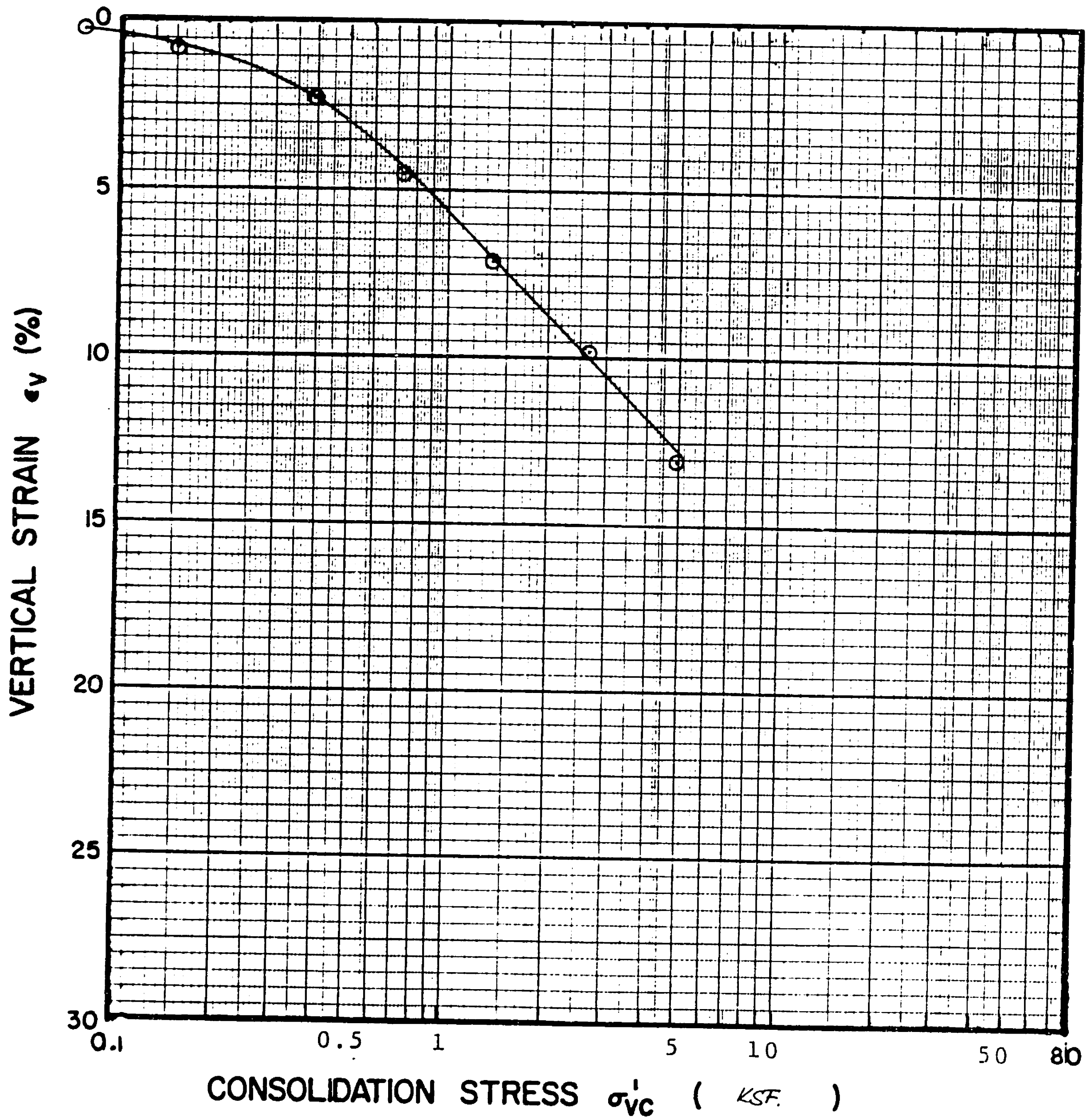
Remarks Corrected for apparatus compressibility

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. TD336

FIGURE



Sample No. 5B1-P3 w_N (%) 36.4 Estimated

Depth (RE) 7.0 FT w_L (%) 47.3 σ'_{vo} 0.385 σ'_p

Soil Type ARCTIC SILT w_p (%) 26.8 CR 0.113 RR

I_p (%) 20.5 G_s 2.78 e_o 1.012 S(%)

o At t_p or hr Remarks Corrected for apparatus

● At (t_f) hr compression.

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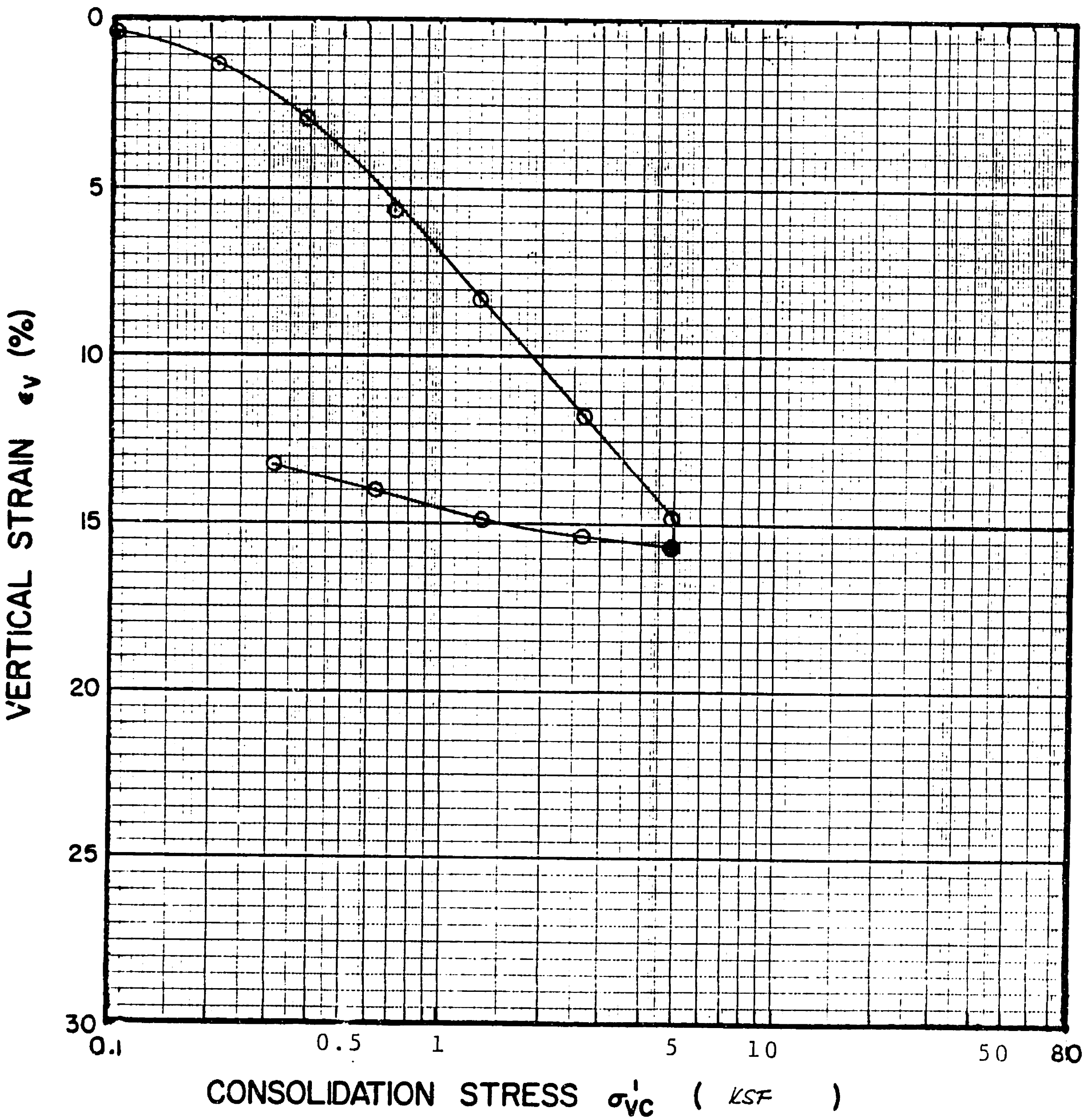
COMPRESSION CURVE

TEST NO. TSB1 DSS7

FIGURE



Sample No. T5B1-P3

Depth (RE) 7.1 FT

Soil Type ARCTIC SILT

w_N (%) 38.3

w_L (%) 47.3

w_P (%) 26.8

I_P (%) 20.5

G_s

Estimated

σ'_{vo} 0.391 σ'_p —

CR 0.11 RR —

e_o 1.07 S(%)

○ At t_p or hr

● At (t_f) hr

Remarks Corrected for apparatus compressibility

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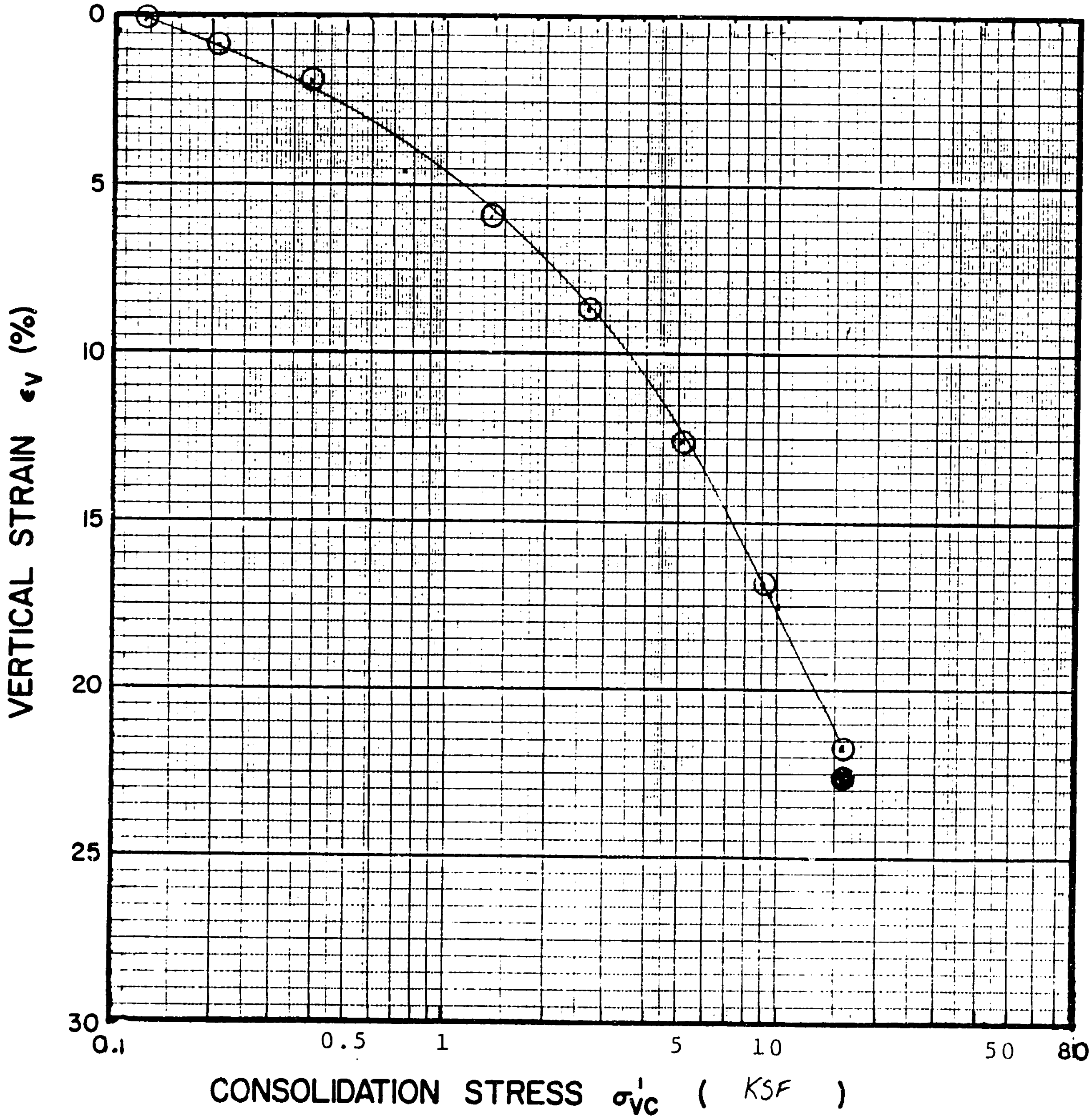
M.I.T.

COMPRESSION CURVE

TEST NO. TDSS8

FIGURE



Sample No. TB1-02
Depth (RE) 7.2'
Soil Type Arctic Silt

w_N (%) 45.9
w_L (%) 52.7
w_P (%) 27.6
I_P (%) 25.1

Estimated
σ'_{vo} 0.231 σ'_p
CR 0.19 RR
G_s 2.75 e_o 1.32 S(%) 95.6

o At t_p or hr
• At ( ) hr

Remarks Corrected for apparatus Compression

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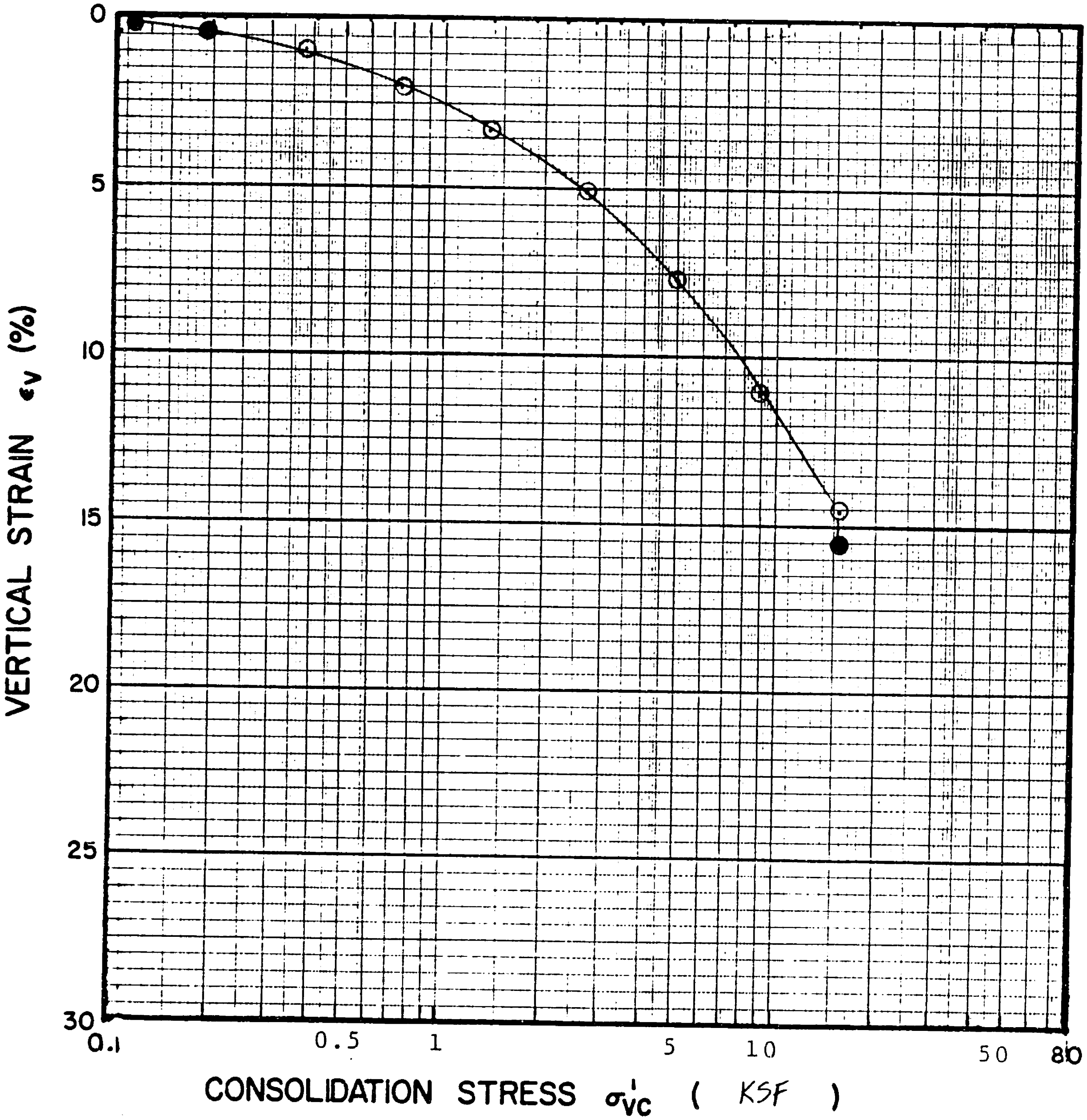
M.I.T.

COMPRESSION CURVE

TEST NO. TB1DSS9

FIGURE



Sample No. T5B1 -P3 w_N (%) 38.4 Estimated

Depth (RE) 8.4 ' w_L (%) 46.3 σ'_{vo} 0.462 σ'_p

Soil Type Arch. silt w_P (%) 25.2 CR 0.138 RR

I_P (%) 21.1 G_s e_o 1.0 S(%)

○ At t_p or hr Remarks Corrected for apparatus compressibility

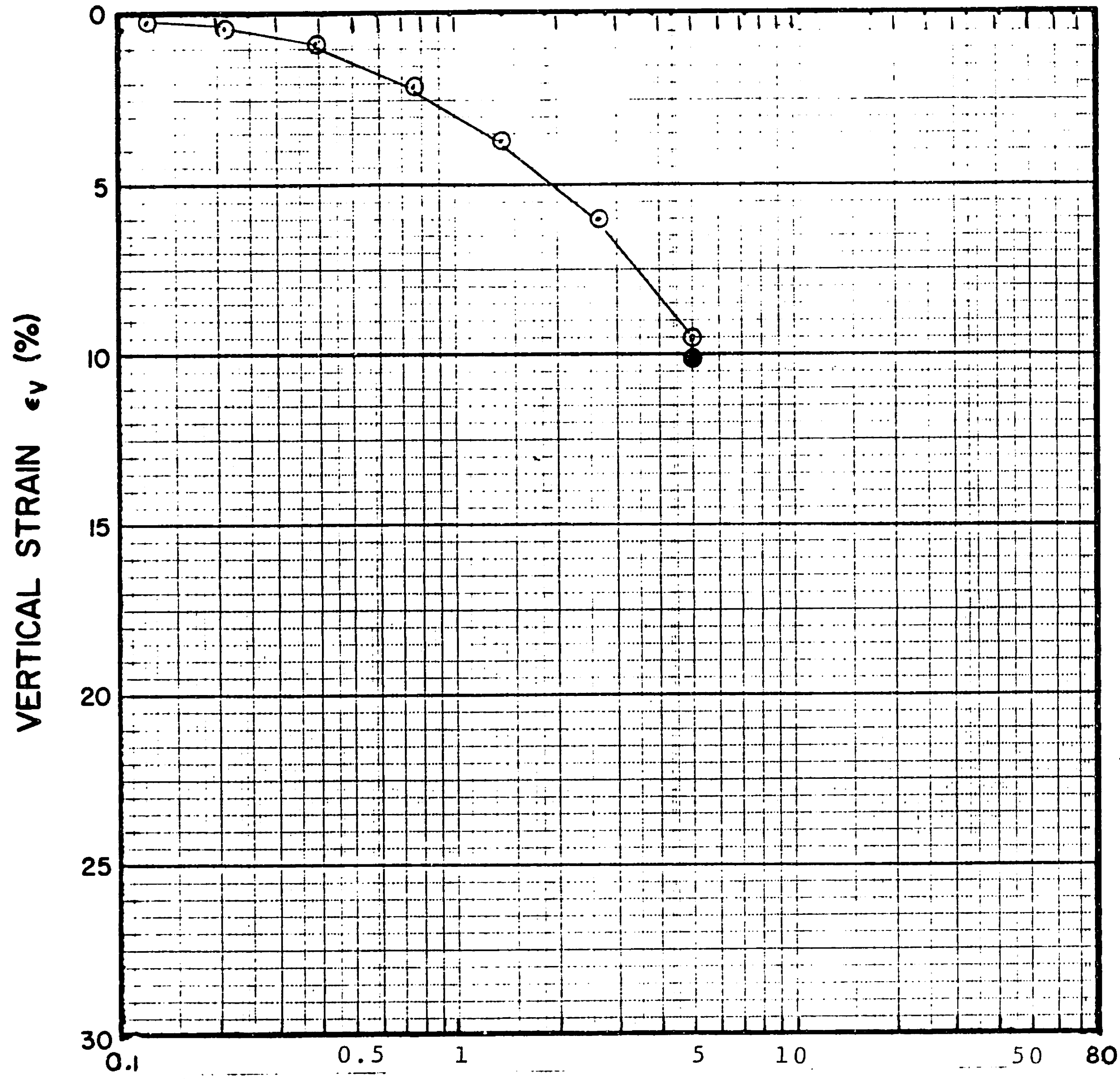
● At (t_f) hr

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. TDSS10

FIGURE



CONSOLIDATION STRESS σ'_{vc} (   )

Sample No. T16-02 (S11) w_N (%) 43.5 Estimated

Depth (RE) 7.4' w_L (%) 52.7 σ'_{vo} 0.242 σ'_p

Soil Type ARCTIC SILT w_P (%) 27.6 CR 0.128 RR

I_P (%) 25.1 G_s e_o 1.25 S(%)

o At t_p or hr

• At (t_f) hr

Remarks corrected for apparatus compressibility

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M.I.T.

COMPRESSION CURVE

TEST NO. TDSS11

FIGURE

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK_0UDSS **NO.** TB3DSS1 **OCR** 4.4

SOIL TYPE Arctic Silt **TESTED BY** JTG **DEVICE** Geonor **DATE** 12/23/85

LOCATION Smith Bay
Site T - Boring 3B
Sample P3 - RE - 7.5'

CONSOLIDATION (Stresses in ksc**)**

σ'_{vc} 0.189 τ_{hc} — σ'_p ____

t_c(Day) 1.0 ε_v(%) ____ γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	37.5	1.05		1.968
Preshear	44.1	1.01		1.927
Final	36.9			1.734

DURING SHEAR
Controlled Strain ✓ Stress ____
Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN* (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	0.2778	
	0.038	0.0300	0.0130	0.9870	0.0466	0.0083	0.2742	363.03
	0.158	0.1027	0.0132	0.9868	0.1595	0.0285	0.2741	302.24
	0.292	0.1588	0.0142	0.9858	0.2467	0.0441	0.2738	252.74
	0.592	0.2138	0.0434	0.9566	0.3322	0.0594	0.2657	168.39
	1.063	0.2583	0.0859	0.9141	0.4013	0.0717	0.2539	113.16
	1.538	0.2988	0.0765	0.9235	0.4642	0.0830	0.2565	90.55
	2.226	0.3438	0.0722	0.9278	0.5341	0.0955	0.2577	71.98
	3.019	0.3808	0.0625	0.9375	0.5916	0.1058	0.2604	58.79
	3.824	0.4177	0.0207	0.9793	0.6489	0.1160	0.2720	50.91
	4.602	0.4516	0.0194	0.9806	0.7017	0.1255	0.2724	45.74
	5.839	0.4914	0.0202	0.9798	0.7635	0.1365	0.2722	39.23
	6.905	0.5268	-0.0273	1.0273	0.8184	0.1463	0.2854	35.56
	8.209	0.5611	-0.0479	1.0479	0.8717	0.1559	0.2911	31.86
	9.733	0.6002	-0.0967	1.0967	0.9325	0.1667	0.3047	28.74
	11.897	0.6436	-0.1463	1.1463	1.0000	0.1788	0.3184	25.21
	15.114	0.6290	-0.0698	1.0698	0.9773	0.1747	0.2972	
	17.383	0.6218	-0.0258	1.0258	0.9661	0.1727	0.2849	
	19.647	0.6193	-0.0193	1.0193	0.9622	0.1720	0.2831	
	21.938	0.6134	-0.0117	1.0117	0.9530	0.1704	0.2810	
	23.743	0.6023	0.0070	0.9930	0.9358	0.1673	0.2758	
	26.460	0.5864	0.0449	0.9551	0.9110	0.1629	0.2653	
	29.673	0.5623	0.1257	0.8743	0.8737	0.1562	0.2429	
	35.003	0.5393	0.1513	0.8487	0.8379	0.1498	0.2357	
	40.851	0.4981	0.2191	0.7809	0.7739	0.1384	0.2169	

SOIL MECHANICS LABORATORY
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REMARKS: Corrected for [illegible] and membrane deflections

• Used [illegible] w/ pins

* based on $H_{sample} - H_{pins}$

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK_0UDSS **NO.** TB3DSS1B **OCR** 1

SOIL TYPE Arctic Silt **TESTED BY** JTG/CDG **DEVICE** Geonor **DATE** 1/12/86

LOCATION Smith Bay
Site I: Boring 3B
Sample P3, ZE = 7.5

CONSOLIDATION (Stresses in KSC**)**

σ'_{vc} 2.301 τ_{hc} — σ'_p ______

t_c(Day) 1.0 ε_v(%) ______ γ_c(%) — t_c(Day) —

	W, %	e	S, %	H (cm)
Initial	37.5	1.05		1.968
Preshear	40.0			1.817
Final	36.9			1.734

DURING SHEAR

Controlled Strain ✓ Stress ______

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)*	τ_h/σ'_{vc}	$\Delta u/\sigma'_{vc}$	σ'_v/σ'_{vc}	τ_h/S_u	τ_h/σ'_p	σ'_v/σ'_p	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.012	0.0070	0.0087	0.9913	0.0254	0.0070	0.9913	629.32
	0.053	0.0420	0.0290	0.9710	0.1532	0.0420	0.9710	868.88
	0.106	0.0622	0.0383	0.9617	0.2264	0.0622	0.9617	643.62
	0.235	0.0957	0.0642	0.9358	0.3486	0.0957	0.9358	444.86
	0.514	0.1358	0.1175	08825	0.4969	0.1358	0.8825	288.62
	1.029	0.1731	0.1793	0.8207	0.6304	0.1731	0.8207	193.83
	1.489	0.1936	0.2201	0.7799	0.7052	0.1936	0.7799	142.08
	2.020	0.2103	0.2565	0.7435	0.7663	0.2103	0.7435	113.81
	2.497	0.2215	0.2815	0.7185	0.8069	0.2215	0.7185	96.94
	3.002	0.2305	0.3077	0.6923	0.8398	0.2305	0.6923	83.93
	3.520	0.2385	0.3218	0.6782	0.8690	0.2385	0.6782	74.07
	4.051	0.2454	0.3417	0.6583	0.8939	0.2454	0.6583	66.20
	4.609	0.2522	0.3524	0.6476	0.9187	0.2522	0.6476	59.80
	5.071	0.2551	0.3579	0.6421	0.9293	0.2551	0.6421	54.97
	6.069	0.2640	0.3794	0.6206	0.9618	0.2640	0.6206	47.54
	7.016	0.2693	0.3937	0.6063	0.9812	0.2693	0.6063	41.96
	7.695	0.2696	0.4133	0.5867	0.9820	0.2696	0.5867	38.29
	8.500	0.2704	0.4251	0.5749	0.9852	0.2704	0.5749	34.77
	9.419	0.2743	0.4297	0.5703	0.9991	0.2743	0.5703	31.82
	10.414	0.2745	0.4507	0.5493	1.0000	0.2745	0.5493	28.81
	12.068	0.2722	0.4709	0.5291	0.9915	0.2722	0.5291	
	14.203	0.2696	0.4964	0.5036	0.9822	0.2696	0.5036	
	16.105	0.2620	0.5159	0.4841	0.9546	0.2620	0.4841	
	18.015	0.2526	0.5490	0.4510	0.9201	0.2526	0.4510	

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* Based on Hsample - Hpins

REMARKS:

- Corrected for stone and membrane deflections
- Second shear
- Used stones w/ pins

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK_0UDSS NO. TB3DSS1B

TIME (Hr.)	STRAIN (%) *	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	20.067	0.2428	0.5782	0.4218	0.8846	0.2428	0.4218	
	21.968	0.2315	0.6084	0.3916	0.8454	0.2315	0.3916	
	23.960	0.2176	0.6368	0.3632	0.7927	0.2176	0.3632	
	26.045	0.2032	0.6666	0.3334	0.7403	0.2032	0.3334	
	28.248	0.1871	0.6921	0.3079	0.6815	0.1871	0.3079	
	30.899	0.1680	0.7276	0.2724	0.6121	0.1680	0.2724	
	33.514	0.1514	0.7594	0.2406	0.5517	0.1514	0.2406	
	36.531	0.1333	0.7918	0.2082	0.4856	0.1333	0.2082	
	38.600	0.1216	0.8164	0.1836	0.4429	0.1216	0.1836	

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* BASED ON Hsample - Hpins

REMARKS: CORRECTED FOR STONE AND MEMBRANE DEFLECTIONS

• SECOND SHEAR

• USED STONES W/ PINS

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CK_0UDSS NO. TB3DSS2 OCR 7.93

SOIL TYPE Arctic Silt TESTED BY JTG/CDG DEVICE GEONOR DATE 1/4/86

LOCATION Smith Bay - Site T, TB3-P3, RE = 7.7'

CONSOLIDATION (Stresses in KSC)

$\bar{\sigma}_{vc}$ 0.308 τ_{hc} — $\bar{\sigma}_{vm}$ 2.44

t_c(Day) 1.0 ε_v(%) 10.1 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	40.2	1.155		2.075
Preshear	34.6	0.938		1.8656
Final	36.3			1.909

DURING SHEAR

Controlled Strain ✓ Stress ____

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\tau_h / \bar{\sigma}_{vc}$	$\Delta u / \bar{\sigma}_{vc}$	$\bar{\sigma}_v / \bar{\sigma}_{vc}$	τ_h / S_u	$\tau_h / \bar{\sigma}_{vm}$	$\bar{\sigma}_v / \bar{\sigma}_{vm}$	E_u / S_u
	0.00	0.00	0.00	1.00	0.00	0.00	0.1261	
	0.057	0.1532	-0.0210	1.0210	0.1033	0.0193	0.1288	548.61
	0.104	0.2269	-0.0692	1.0692	0.1530	0.0286	0.1348	442.21
	0.208	0.3158	-0.0922	1.0922	0.2130	0.0398	0.1377	307.45
	0.306	0.3762	-0.1642	1.1642	0.2538	0.0474	0.1468	248.40
	0.406	0.4294	-0.2478	1.2478	0.2897	0.0542	0.1573	213.93
	0.548	0.4857	-0.2341	1.2341	0.3276	0.0612	0.1556	179.20
	0.796	0.5794	-0.4048	1.4048	0.3908	0.0731	0.1772	147.21
	1.444	0.7516	-0.5161	1.5161	0.5070	0.0948	0.1912	105.30
	2.068	0.8813	-0.6917	1.6917	0.5210	0.1111	0.2133	86.22
	2.966	1.0406	-0.9317	1.9317	0.7019	0.1312	0.2436	71.00
	3.910	1.1750	-1.1305	2.1305	0.7926	0.1482	0.2687	60.81
	5.187	1.2948	-1.3354	2.3354	0.8734	0.1633	0.2945	50.51
	6.283	1.3701	-1.4648	2.4648	0.9241	0.1728	0.3108	44.13
	7.375	1.4231	-1.5397	2.5397	0.9599	0.1795	0.3203	39.05
	8.524	1.4591	-1.5729	2.5729	0.9842	0.1840	0.3245	34.64
	9.695	1.4825	-1.6348	2.6346	1.000	0.1869	0.3322	30.94
	11.510	1.4693	-1.5767	2.5767	0.9911	0.1853	0.3249	
	13.081	1.4307	-1.4961	2.4961	0.9650	0.1804	0.3148	
	14.053	1.4058	-1.4111	2.4111	0.9482	0.1773	0.3048	
	16.284	1.3470	-1.2733	2.2733	0.9086	0.1699	0.2867	
	18.944	1.2552	-1.0996	2.0996	0.8467	0.1583	0.2648	
	22.079	1.1540	-0.8969	1.8969	0.7784	0.1455	0.2392	
	25.260	1.0657	-0.7370	1.7370	0.7189	0.1344	0.2190	
	30.270	0.9015	-0.4462	1.4462	0.6081	0.1137	0.1824	

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REMARKS: CORRECTED FOR STONE AND MEMBRANE DEFLECTIONS

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CK₀UDSS NO. 7B3DSS 3 OCR 1

SOIL TYPE Arctic Silt TESTED BY GY/JTG DEVICE Geonor DATE 1/8/86

LOCATION Smith Bay - Site T Boring 3B-P3 (BE=7.9')

CONSOLIDATION (Stresses in ksc)

σ'_{vc} 2.45 τ_{hc} — σ'_p 2.45

t_c(Day) 1.0 ε_v(%) 15.6 γ_c(%) ___ t_c(Day) ___

	W,%	e	S,%	H (cm)
Initial	54.4	1.609		2.172
Preshear	42.4	1.201		1.834
Final	48.4			1.973

DURING SHEAR

Controlled Strain ✓ Stress ___

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	τ_h/σ'_{vc}	$\Delta u/\sigma'_{vc}$	σ'_v/σ'_{vc}	τ_h/S_u	τ_h/σ'_p	σ'_v/σ'_p	E_u/S_u
	0.00	0.00	0.00	1.00	0.00	0.00	1.00	
	0.132	0.0781	0.0249	0.9751	0.2895	0.0781	0.9751	656.27
	0.264	0.1073	0.0501	0.9499	0.3975	0.1073	0.9499	451.42
	0.349	0.1210	0.0608	0.9392	0.4482	0.1210	0.9392	384.86
	0.475	0.1361	0.0754	0.9246	0.5044	0.1361	0.9246	318.59
	0.578	0.1446	0.0876	0.9124	0.5357	0.1446	0.9124	278.17
	0.667	0.1530	0.1005	0.8995	0.5668	0.1530	0.8995	254.81
	0.820	0.1629	0.1128	0.8872	0.6036	0.1629	0.8872	220.71
	0.933	0.1697	0.1205	0.8795	0.6289	0.1697	0.8795	202.25
	1.050	0.1760	0.1292	0.8708	0.6520	0.1760	0.8708	186.20
	1.726	0.2022	0.1709	0.8291	0.7493	0.2022	0.8291	130.24
	2.393	0.2193	0.1999	0.8001	0.8126	0.2193	0.8001	101.89
	3.161	0.2314	0.2498	0.7502	0.8575	0.2314	0.7502	81.39
	4.108	0.2421	0.2817	0.7183	0.8970	0.2421	0.7183	65.51
	5.077	0.2503	0.3054	0.6946	0.9274	0.2503	0.6946	54.80
	6.106	0.2560	0.3259	0.6741	0.9468	0.2560	0.6741	46.61
	7.178	0.2621	0.3339	0.6661	0.9711	0.2621	0.6661	40.59
	8.454	0.2646	0.3641	0.6359	0.9805	0.2646	0.6359	34.80
	9.069	0.2654	0.3700	0.6300	0.9834	0.2654	0.6300	32.53
	10.303	0.2678	0.3845	0.6155	0.9922	0.2678	0.6155	28.89
	11.588	0.2693	0.3942	0.6058	0.9978	0.2693	0.6058	25.83
	13.171	0.2699	0.4122	0.5878	1.000	0.2699	0.5878	22.78
	15.114	0.2695	0.4243	0.5757	0.9984	0.2695	0.5757	
	17.332	0.2675	0.4487	0.5513	0.9912	0.2675	0.5513	
	19.250	0.2643	0.4627	0.5373	0.9794	0.2643	0.5373	

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REMARKS: CORRECTED FOR MEMBRANE AND STONE DEFLECTIONS

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio **SOIL** Arctic Silt **TYPE OF TEST** CK_oUDSS **NO.** TB3DSS3

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\bar{\sigma}_p}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_p}$	REMARKS
	22.186	0.2472	0.4949	0.5051	0.9159	0.2472	0.5051	
	24.408	0.2248	0.5423	0.4577	0.8330	0.2248	0.4577	
	27.187	0.2070	0.5908	0.4092	0.7669	0.2070	0.4092	
	30.636	0.1970	0.6414	0.3586	0.6921	0.1870	0.3586	
	35.451	0.1631	0.7001	0.2999	0.6044	0.1631	0.2999	
	43.905	0.1195	0.7836	0.2164	0.4426	0.1195	0.2164	

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REMARKS: CORRECTED FOR MEMBRANE AND STONE DEFLECTIONS

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK_oUDSS **NO.** TB3 DSS4 **OCR** 1

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 1/19/86

LOCATION Smith Bay Site T: Boring 3B-P2 RE: 4.7'

CONSOLIDATION (Stresses in KSC)

σ'_{vc} 2.40 τ_{hc} — σ'_p

t_c(Day) 1.0 ε_v(%) γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	35.3	0.981		2.113
Preshear	26.0	0.750		1.867
Final	29.51			1.938

DURING SHEAR

Controlled Strain ✓ Stress

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/S_u
	0.00	0.00	0.00	1.00	0.00	0.00	1.00	
	0.054	0.0512	0.0123	0.9877	0.1804	0.0512	0.9877	1010.65
	0.122	0.0737	0.0193	0.9807	0.2598	0.0737	0.9807	641.36
	0.246	0.1033	0.0343	0.9657	0.3641	0.1033	0.9657	443.54
	0.379	0.1247	0.0527	0.9473	0.4394	0.1247	0.9473	347.86
	0.529	0.1424	0.0720	0.9280	0.5022	0.1424	0.9280	284.53
	0.798	0.1651	0.0970	0.9030	0.5819	0.1651	0.9030	218.69
	1.205	0.1880	0.1429	0.8571	0.6627	0.1880	0.8571	165.00
	1.905	0.2116	0.1995	0.8005	0.7460	0.2116	0.8005	117.50
	2.855	0.2315	0.2400	0.7600	0.8160	0.2315	0.7600	85.73
	4.299	0.2491	0.2970	0.7030	0.8782	0.2491	0.7030	61.29
	5.977	0.2630	0.3345	0.6655	0.9272	0.2630	0.6655	46.54
	6.861	0.2675	0.3471	0.6529	0.9430	0.2675	0.6529	41.24
	8.044	0.2733	0.3639	0.6361	0.9635	0.2733	0.6361	35.94
	9.135	0.2758	0.3709	0.6291	0.9721	0.2758	0.6291	31.03
	10.023	0.2790	0.3735	0.6265	0.9834	0.2790	0.6265	29.44
	11.092	0.2817	0.3782	0.6218	0.9930	0.2817	0.6218	26.86
	12.057	0.2820	0.3944	0.6056	0.9941	0.2820	0.6056	24.73
	13.007	0.2834	0.4009	0.5991	0.9990	0.2834	0.5991	23.04
	13.797	0.2837	0.4048	0.5952	1.000	0.2837	0.5952	21.76
	14.585	0.2831	0.4124	0.5876	0.9980	0.2831	0.5876	
	15.572	0.2808	0.4192	0.5808	0.9900	0.2808	0.5808	
	17.231	0.2688	0.4595	0.5405	0.9476	0.2688	0.5405	
	19.164	0.2579	0.4793	0.5207	0.9091	0.2579	0.5207	
	21.085	0.2469	0.5036	0.4964	0.8705	0.2469	0.4964	

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REMARKS: Corrected for membrane and stone deflections

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio **SOIL** Arctic Silt **TYPE OF TEST** CK₀UDSS **NO.** TB3 D-S4

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{Su}$	$\frac{\tau_h}{\bar{\sigma}_p}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_p}$	REMARKS
	23.277	0.2359	0.5415	0.4585	0.8315	0.2359	0.4585	
	25.354	0.2247	0.5706	0.4294	0.7921	0.2247	0.4294	
	27.485	0.2134	0.5919	0.4081	0.7522	0.2134	0.4081	
	29.609	0.1997	0.6177	0.3823	0.7041	0.1997	0.3823	
	30.500	0.1942	0.6356	0.3644	0.6845	0.1942	0.3644	

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REMARKS: Corrected for membrane and stone deflections

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK_0UDSS **NO.** TB3DSS5 **OCR** 8.5

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 1/23/86

LOCATION Smith Bay
Site T: Boring B3
Sample P2, RE = 4.65'

CONSOLIDATION (Stresses in KSC**)**

σ'_{vc} 0.287 τ_{hc} — σ'_p 2.44

t_c(Day) 1.0 ε_v(%) 10.47 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	37.6	1.04		1.9558
Preshear	32.1	0.829		1.7510
Final	33.9			1.7986

DURING SHEAR
Controlled Strain ✓ Stress ____
Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{Su}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/Su
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	0.1176	
	0.013	0.0235	0.0028	0.9972	0.0192	0.0028	0.1173	453.19
	0.020	0.0627	0.0069	0.9931	0.0514	0.0074	0.1168	775.27
	0.125	0.2096	-0.0417	1.0417	0.1715	0.0247	0.1226	410.65
	0.507	0.3880	-0.1591	1.1591	0.3175	0.0456	0.1364	187.77
	1.056	0.5120	-0.3122	1.3122	0.4191	0.0602	0.1544	114.11
	2.028	0.6699	-0.5053	1.5053	0.5478	0.0787	0.1771	81.05
	2.548	0.7378	-0.6315	1.6315	0.6038	0.0868	0.1919	71.86
	3.092	0.8020	-0.7018	1.7018	0.6564	0.0944	0.2002	63.69
	3.516	0.8436	-0.7660	1.7660	0.6905	0.0922	0.2078	58.91
	4.076	0.8957	-0.8731	1.8731	0.7331	0.1054	0.2204	53.95
	4.497	0.9327	-0.9497	1.9497	0.7633	0.1097	0.2294	50.92
	5.078	0.9731	-1.0213	2.0213	0.7965	0.1145	0.2378	47.06
	5.525	1.0025	-1.0359	2.0359	0.8205	0.1179	0.2395	44.55
	6.117	1.0372	-1.0805	2.0805	0.8489	0.1220	0.2448	41.64
	6.559	1.0665	-1.1949	2.1949	0.8729	0.1255	0.2582	39.93
	7.001	1.0896	-1.1748	2.1748	0.8918	0.1282	0.2559	38.22
	7.596	1.1169	-1.2575	2.2575	0.9142	0.1314	0.2656	36.10
	8.053	1.1390	-1.2942	2.2942	0.9322	0.1340	0.2699	34.73
	8.530	1.1558	-1.3355	2.3355	0.9459	0.1360	0.2748	33.27
	9.140	1.1737	-1.3299	2.3299	0.9606	0.1381	0.2741	31.53
	9.707	1.1976	-1.3759	2.3759	0.9801	0.1409	0.2795	29.13
	10.572	1.2088	-1.3832	2.3832	0.9894	0.1422	0.2804	28.08
	11.231	1.2155	-1.3681	2.3681	0.9948	0.1430	0.2786	26.57
	11.556	1.2183	-1.4387	2.4387	0.9971	0.1433	0.2869	25.88

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REMARKS:
Corrected for membrane and stone deflection

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK$_0$UDSS NO. TB3DSS5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	E_u/S_u
	12.042	1.2218	-1.3919	2.3919	1.0000	0.1437	0.2814	24.91
	13.122	1.2117	-1.3624	2.3624	0.9917	0.1426	0.2779	
	15.018	1.1933	-1.3186	2.3186	0.9767	0.1404	0.2728	
	17.033	1.1793	-1.2402	2.2402	0.9652	0.1387	0.2636	
	19.137	1.1613	-1.1284	2.1284	0.9505	0.1366	0.2504	
	21.276	1.1384	-1.1036	2.1036	0.9317	0.1339	0.2475	
	23.014	1.1177	-1.0201	2.0201	0.9147	0.1315	0.2377	
	25.222	1.0661	-0.9195	1.9195	0.8726	0.1254	0.2258	
	27.404	1.0110	-0.8011	1.8011	0.8275	0.1189	0.2119	
	29.161	0.9637	-0.6979	1.6979	0.7887	0.1134	0.1998	
	30.472	0.9295	-0.6250	1.6250	0.7608	0.1094	0.1912	

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REMARKS: CORRECTED FOR MEMBRANE AND STONE DEFLECTION

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CKoUDSS **NO.** TB1DSS6 **OCR** 1

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 1/26/86

LOCATION Smith Bay
Site T - Boring 1B
Sample O-2, RE=7'

CONSOLIDATION (Stresses in KSC**)**

σ'_{vc} 2.413 τ_{hc} — σ'_p

t_c(Day) 1.0 ε_v(%) 12.0 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	41.6	1.184		2.1668
Preshear	31.1	0.921		1.907
Final	34.8			2.010

DURING SHEAR

Controlled Strain ✓ Stress

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	τ_h/σ'_{vc}	$\Delta u/\sigma'_{vc}$	σ'_v/σ'_{vc}	τ_h/S_u	τ_h/σ'_p	σ'_v/σ'_p	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.000	0.0000	1.0000	
	0.024	0.0298	0.0612	0.9388	0.1068	0.0298	0.9388	1324.66
	0.116	0.0668	0.0477	0.9523	0.2394	0.0668	0.9523	617.81
	0.248	0.0957	0.0551	0.9449	0.3432	0.0957	0.9449	41505
	0.540	0.1348	0.1071	0.8929	0.4834	0.1348	0.8929	268.50
	1.041	0.1707	0.1480	0.8520	0.6120	0.1707	0.8520	176.43
	2.030	0.2054	0.2114	0.7886	0.7363	0.2054	0.7886	108.83
	3.033	0.2274	0.2634	0.7366	0.8150	0.2274	0.7366	80.10
	4.041	0.2420	0.2988	0.7012	0.8675	0.2420	0.7012	64.39
	5.096	0.2532	0.3171	0.6829	0.9076	0.2532	0.6829	53.43
	6.044	0.2612	0.3309	0.6691	0.9362	0.2612	0.6691	46.47
	7.027	0.2666	0.3581	0.6419	0.9555	0.2666	0.6419	40.79
	8.066	0.2704	0.3722	0.6278	0.9692	0.2704	0.6278	36.05
	9.107	0.2727	0.3829	0.6171	0.9776	0.2727	0.6171	32.20
	10.017	0.2744	0.3911	0.6089	0.9836	0.2744	0.6089	29.46
	11.051	0.2773	0.3958	0.6042	0.9941	0.2773	0.6042	26.99
	11.815	0.2790	0.4023	0.5977	1.0000	0.2790	0.5977	25.39
	13.068	0.2781	0.4170	0.5830	0.9968	0.2781	0.5830	
	15.068	0.2705	0.4474	0.5526	0.9696	0.2705	0.5526	
	17.130	0.2537	0.4954	0.5046	0.9095	0.2537	0.5046	
	17.986	0.2502	0.5034	0.4966	0.8970	0.2502	0.4966	
	19.105	0.2468	0.5132	0.4868	0.8846	0.2468	0.4868	
	21.114	0.2360	0.5473	0.4527	0.8460	0.2360	0.4527	
	23.098	0.2268	0.5706	0.4294	0.8131	0.2268	0.4294	
	25.250	0.2132	0.6020	0.3980	0.7643	0.2132	0.3980	

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REMARKS:
CORRECTED FOR APPARATUS AND STONE DEFLECTIONS

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK_0UDSS NO. TBIDSS6

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\bar{\sigma}_p}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_p}$	REMARKS
	27.330	0.2003	0.6281	0.3719	0.7179	0.2003	0.3719	
	29.015	0.1891	0.6564	0.3436	0.6780	0.1891	0.3436	
	30.752	0.1793	0.6771	0.3229	0.6427	0.1793	0.3229	

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REMARKS: Corrected for apparatus and stone deflections

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CK_0UDSS NO. T5B1DSS7 OCR 1

SOIL TYPE Arctic Silt TESTED BY GY DEVICE Geonor DATE 1/29/86

LOCATION Smith Bay - Site T, Boring SB1-P3, RE-7'

CONSOLIDATION (Stresses in KSC)

$\bar{\sigma}_{vc}$ 2.409 τ_{hc} — $\bar{\sigma}_{vm}$

t_c(Day) — ε_v(%) 13.7 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	36.3	1.012		2.238
Preshear	25.1	0.736		1.931
Final	27.5			2.006

DURING SHEAR

Controlled Strain ✓ Stress

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%) *	$\tau_h/\bar{\sigma}_{vc}$	$\Delta u/\bar{\sigma}_{vc}$	$\bar{\sigma}_v/\bar{\sigma}_{vc}$	τ_h/S_u	$\tau_h/\bar{\sigma}_{vm}$	$\bar{\sigma}_v/\bar{\sigma}_{vm}$	E_u/S_u
	0.0000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.0398	0.0349	0.0278	0.9722	0.1266	0.0349	0.9722	942.7
	0.1003	0.0576	0.0299	0.9701	0.2089	0.0576	0.9701	627.3
	0.2597	0.0880	0.0461	0.9539	0.3191	0.0880	0.9539	369.0
	0.4840	0.1151	0.0671	0.9329	0.4173	0.1151	0.9329	258.4
	0.8101	0.1389	0.0985	0.9015	0.5036	0.1389	0.9015	186.5
	1.4775	0.1675	0.1558	0.8442	0.6072	0.1675	0.8442	122.6
	2.660	0.1946	0.2155	0.7845	0.7053	0.1946	0.7845	79.6
	4.7378	0.2219	0.2770	0.7230	0.8045	0.2219	0.7230	50.9
	6.2664	0.2338	0.3062	0.6938	0.8475	0.2338	0.6938	40.6
	8.8102	0.2494	0.3302	0.6698	0.9039	0.2494	0.6698	30.8
	10.4170	0.2566	0.3488	0.6512	0.9301	0.2566	0.6512	26.8
	11.8925	0.2627	0.3572	0.6428	0.9523	0.2627	0.6428	24.01
	13.3916	0.2666	0.3647	0.6353	0.9664	0.2666	0.6353	21.64
	14.8789	0.2709	0.3637	0.6363	0.9818	0.2709	0.6363	19.79
	16.3781	0.2742	0.3765	0.6235	0.9937	0.2742	0.6235	18.20
	18.4083	0.2759	0.3894	0.6106	1.0000	0.2759	0.6106	16.29
	19.9827	0.2753	0.3982	0.6018	0.9980	0.2753	0.6018	
	21.3874	0.2745	0.4119	0.5881	0.9951	0.2745	0.5881	
	22.7817	0.2724	0.4188	0.5812	0.9875	0.2724	0.5812	
	25.1470	0.2689	0.4393	0.5607	0.9748	0.2689	0.5607	
	28.1127	0.2617	0.4772	0.5228	0.9486	0.2617	0.5228	
	29.926	0.2577	0.4816	0.5184	0.9340	0.2577	0.5184	

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REMARKS: CORRECTED FOR MEMBRANE AND STONE DEFLECTIONS

• USED POROUS STONES W/ PINS

* BASED ON $H_{SAMPLE} - H_{PINS}$

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK_0UDSS **NO.** T5B1DSS8 **OCR** 15.75

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 2/4/86

LOCATION Smith Bay
Site T - Boring SB1
Sample P3 - RE = 7.1'

CONSOLIDATION (Stresses in KSC**)**

σ'_{vc} 0.153 τ_{hc} — σ'_p 2.41

t_c(Day) 1.0 ε_v(%) 13.1 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	38.8	1.067		2.034
Preshear	30.7	0.796		1.766
Final	32.5			1.815

DURING SHEAR
Controlled Strain ✓ Stress ____
Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN* (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{Su}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	Eu/Su
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	0.0635	
	0.085	0.2384	-0.0423	1.0423	0.1161	0.0151	0.0662	411.5
	0.151	0.3197	-0.0633	1.0633	0.1557	0.0203	0.0675	307.32
	0.244	0.3924	-0.0685	1.0685	0.1911	0.0249	0.0678	235.7
	0.506	0.5141	-0.1647	1.1647	0.2504	0.0326	0.0739	148.4
	0.814	0.6150	-0.2342	1.2342	0.2995	0.0390	0.0784	110.47
	1.141	0.6956	-0.3761	1.3761	0.3388	0.0442	0.0874	89.05
	1.657	0.8077	-0.5069	1.5069	0.3933	0.0513	0.0957	71.2[illegible]
	2.047	0.8785	-0.5703	1.5703	0.4278	0.0558	0.0997	62.7
	2.430	0.9399	-0.6728	1.6728	0.4577	0.0597	0.1062	56.51
	3.223	1.0640	-0.8757	1.8757	0.5182	0.0676	0.1191	48.2
	4.050	1.1894	-1.0634	2.0634	0.5792	0.0755	0.1310	42.9
	5.065	1.3369	-1.1985	2.1985	0.6511	0.0849	0.1396	38.56
	5.701	1.4358	-1.5528	2.5528	0.6992	0.0912	0.1621	36.8
	6.349	1.5009	-1.7131	2.7131	0.7309	0.0953	0.1723	34.5
	7.897	1.6574	-2.1116	3.1116	0.8072	0.1032	0.1976	30.7
	8.8014	1.7376	-2.3335	3.3335	0.8462	0.1103	0.2116	28.8
	11.141	1.8448	-2.6617	3.6617	0.8984	0.1171	0.2325	24.2
	12.532	1.9338	-2.6683	3.6683	0.9418	0.1228	0.2329	22.5
	14.005	1.9644	-2.4928	3.4928	0.9567	0.1247	0.2218	20.5
	15.520	1.9729	-2.5464	3.5464	0.9608	0.1253	0.2252	18.6
	17.240	2.0038	-2.5397	3.5397	0.9758	0.1272	0.2247	17.0
	18.748	2.0211	-2.6447	3.6447	0.9842	0.1283	0.2314	15.7
	20.229	2.0457	-2.6122	3.6122	0.9962	0.1299	0.2293	14.8
	21.770	2.0466	-2.5832	3.5832	0.9967	0.1299	0.2275	13.7

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REMARKS: Corrected for membrane and stone deflections

* based on hsample - hpins

• Used stones w/ pins

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK_0UDSS NO. TSB1DSS8

TIME (Hr.)	STRAIN (%)*	$\tau_h / \bar{\sigma}_{vc}$	$\Delta u / \bar{\sigma}_{vc}$	$\bar{\sigma}_v / \bar{\sigma}_{vc}$	τ_h / S_u	$\tau_h / \bar{\sigma}_p$	$\bar{\sigma}_v / \bar{\sigma}_p$	E_u / S_u
	23.381	2.0534	-2.6065	3.6065	1.0000	0.1304	0.2290	12.8
	25.462	2.0386	-2.5122	3.5122	0.9928	0.1294	0.2230	
	27.806	2.0321	-2.4874	3.4874	0.9896	0.1290	0.2214	
	29.344	2.0212	-2.4575	3.4575	0.9843	0.1283	0.2195	

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REMARKS: CORRECTED FOR MEMBRANE AND STONE DEFLECTIONS

• USED STONES W/ PINS

* based on Hsample - Hpins

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CKoUDSS NO. TB1DSS9 OCR 1

SOIL TYPE Arctic Silt TESTED BY GY DEVICE Geonor DATE 3/28/86

LOCATION Smith Bay
Site T: Boring B1
Sample 0.2, RE = 7.2'

CONSOLIDATION (Stresses in KSC)

σ'_{vc} 7.6 τ_{hc} — σ'_p

t_c(Day) 1.0 ε_v(%) 22.6 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	45.9	1.324		2.129
Preshear	30.6	0.80		1.647
Final	38.9			1.858

DURING SHEAR
Controlled Strain ✓ Stress
Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	τ_h / σ'_{vc}	$\Delta u / \sigma'_{vc}$	σ'_v / σ'_{vc}	τ_h / S_u	τ_h / σ'_p	σ'_v / σ'_p	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.104	0.0514	0.0008	0.9992	0.2240	0.0514	0.9992	647.05
	0.252	0.0752	0.0049	0.9951	0.3277	0.0752	0.9951	390.11
	1.029	0.1423	0.0986	0.9014	0.6199	0.1423	0.9014	180.69
	1.592	0.1627	0.1607	0.8393	0.7085	0.1627	0.8393	133.55
	2.116	0.1748	0.1657	0.8343	0.7612	0.1748	0.8343	107.95
	2.388	0.1800	0.1690	0.8310	0.7838	0.1800	0.8310	98.47
	3.534	0.1959	0.2314	0.7686	0.8530	0.1959	0.7686	72.41
	4.121	0.2029	0.2462	0.7538	0.8838	0.2029	0.7538	64.35
	4.599	0.2056	0.2762	0.7238	0.8956	0.2056	0.7238	55.42
	5.081	0.2096	0.2784	0.7216	0.9127	0.2096	0.7216	53.89
	5.561	0.2131	0.2888	0.7112	0.9281	0.2131	0.7112	50.07
	6.045	0.2153	0.3096	0.6904	0.9375	0.2153	0.6904	46.52
	6.390	0.2167	0.3170	0.6830	0.9436	0.2167	0.6830	44.30
	7.034	0.2194	0.3280	0.6720	0.9555	0.2194	0.6720	40.75
	8.038	0.2231	0.3375	0.6625	0.9716	0.2231	0.6625	36.26
	8.558	0.2252	0.3436	0.6564	0.9808	0.2252	0.6564	34.38
	9.438	0.2271	0.3542	0.6458	0.9892	0.2271	0.6458	31.44
	10.485	0.2286	0.3680	0.6320	0.9950	0.2286	0.6320	28.48
	11.191	0.2291	0.3683	0.6221	0.9979	0.2291	0.6221	26.75
	12.093	0.2293	0.3881	0.6119	0.9987	0.2293	0.6119	24.78
	12.808	0.2296	0.3949	0.6051	1.0000	0.2296	0.6051	23.42
	14.274	0.2288	0.4125	0.5875	0.9967	0.2288	0.5875	
	16.105	0.2246	0.4373	0.5627	0.9781	0.2246	0.5627	
	18.213	0.2153	0.4667	0.5333	0.9376	0.2153	0.5333	

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REMARKS: Corrected for stone and membrane deflections

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK₀UDSS NO. TBIDSS9

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	20.042	0.2065	0.4958	0.5042	0.8995	0.2065	0.5042	
	22.526	0.1893	0.5607	0.4393	0.8246	0.1893	0.4393	
	26.367	0.1408	0.4497	0.3130	0.6131	0.1408	0.3130	
	28.505	0.1178	0.7432	0.2568	0.5130	0.1178	0.2568	
	32.152	0.0869	0.8063	0.1937	0.3785	0.0869	0.1937	

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REMARKS: Corrected for Stone and Membrane Deflections

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK₀UDSS **NO.** TDSS10 **OCR** 1

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 4/11/86

LOCATION Smith Bay
Site T - Boring 5B1
Sample P.3, RE = 8.4'

CONSOLIDATION (Stresses in Ksc**)**

σ'_{vc} 7.6 τ_{hc} — σ'_p

t_c(Day) 1.0 ε_v(%) 15.5 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	32.9	1.0		2.108
Preshear	27.1	0.69		1.781
Final	31.94			1.922

DURING SHEAR
Controlled Strain ✓ Stress

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	τ_h / σ'_{vc}	$\Delta u / \sigma'_{vc}$	σ'_v / σ'_{vc}	τ_h / S_u	τ_h / σ'_p	σ'_v / σ'_p	E_u / S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.010	0.0046	0.0140	0.9860	0.0188	0.0046	0.9860	584.52
	0.054	0.0363	0.0233	0.9767	0.1486	0.0363	0.9767	830.36
	0.121	0.0614	0.0271	0.9729	0.2514	0.0614	0.9729	623.96
	0.416	0.1164	0.0698	0.9302	0.4766	0.1164	0.9302	343.61
	1.006	0.1628	0.1494	0.8506	0.6665	0.1628	0.8506	198.74
	1.505	0.1829	0.1917	0.8083	0.7488	0.1829	0.8083	149.25
	2.102	0.1974	0.2362	0.7638	0.8082	0.1974	0.7638	115.34
	2.487	0.2043	0.2556	0.7444	0.8364	0.2043	0.7444	100.91
	3.014	0.2119	0.2798	0.7202	0.8674	0.2119	0.7202	86.33
	3.577	0.2183	0.3010	0.6990	0.8936	0.2183	0.6990	74.94
	4.014	0.2218	0.3142	0.6858	0.9079	0.2218	0.6858	67.85
	4.609	0.2262	0.3312	0.6688	0.9260	0.2262	0.6688	60.28
	5.061	0.2289	0.3417	0.6583	0.9373	0.2289	0.6583	55.56
	5.520	0.2313	0.3510	0.6490	0.9469	0.2313	0.6490	51.46
	6.139	0.2339	0.3621	0.6379	0.9577	0.2339	0.6379	46.80
	7.075	0.2377	0.3733	0.6267	0.9730	0.2377	0.6267	41.26
	8.018	0.2414	0.3876	0.6124	0.9843	0.2414	0.6124	36.83
	9.153	0.2417	0.4071	0.5929	0.9897	0.2417	0.5929	32.44
	10.114	0.2434	0.4130	0.5870	0.9966	0.2434	0.5870	29.56
	10.926	0.2441	0.4246	0.5754	0.9993	0.2441	0.5754	27.44
	11.750	0.2443	0.4294	0.5706	1.0000	0.2443	0.5706	25.53
	13.101	0.2426	0.4456	0.5544	0.9930	0.2426	0.5544	
	15.023	0.2344	0.4858	0.5142	0.9596	0.2344	0.5142	
	17.194	0.2223	0.5325	0.4675	0.9101	0.2223	0.5325	

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REMARKS:
Corrected for stone and membrane deflections

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio **SOIL** Arctic Silt **TYPE OF TEST** CK₀UDSS **NO.** TDSS10

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	19.433	0.2094	0.5702	0.4298	0.8573	0.2094	0.4298	
	21.214	0.1995	0.5992	0.4008	0.8166	0.1995	0.4008	
	23.077	0.1837	0.6369	0.3631	0.7520	0.1837	0.3631	
	25.441	0.1653	0.6786	0.3214	0.6765	0.1653	0.3214	
	27.350	0.1509	0.7090	0.2910	0.6176	0.1509	0.2910	
	29.988	0.1306	0.7512	0.2488	0.5347	0.1306	0.2488	
	31.344	0.1222	0.7662	0.2338	0.5001	0.1222	0.2338	

SOIL MECHANICS LABORATORY
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REMARKS: CORRECTED FOR STONE AND MEMBRANE DEFLECTIONS.

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CKoUDSS NO ~~(TBIDSS7)~~ TDSS11 OCR 1

SOIL TYPE Arctic Silt TESTED BY GY DEVICE Geonor DATE 3/20/86

LOCATION Smith Bay
Site T - Boring 1B
Sample 02, RE=

CONSOLIDATION (Stresses in KSC)

σ'_{vc} 2.449 τ_{hc} — σ'_p 1.9

t_c(Day) 1.0 ε_v(%) 10.2 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	42.9	1.25		2.078
Preshear	36.2	1.02		1.870
Final	39.4			1.952

DURING SHEAR

Controlled Strain ✓ Stress

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.068	0.0521	0.0218	0.9782	0.1811	0.0521	0.9782	797.73
	0.249	0.1005	0.0654	0.9346	0.3494	0.1005	0.9346	421.65
	0.616	0.1466	0.1384	0.8616	0.5096	0.1466	0.8616	248.38
	1.083	0.1767	0.2169	0.7831	0.6143	0.1767	0.7831	178.12
	1.608	0.1951	0.2878	0.7122	0.6783	0.1951	0.7122	126.57
	2.207	0.1979	0.3647	0.6353	0.6878	0.1979	0.6353	93.50
	3.061	0.2143	0.3860	0.6140	0.7450	0.2143	0.6140	73.02
	3.499	0.2224	0.3830	0.6170	0.7733	0.2224	0.6170	66.31
	4.082	0.2306	0.3835	0.6165	0.8018	0.2306	0.6165	58.92
	4.524	0.2378	0.3506	0.6494	0.8266	0.2378	0.6494	54.82
	5.112	0.2459	0.3634	0.6366	0.8547	0.2459	0.6366	50.16
	6.007	0.2543	0.3708	0.6292	0.8840	0.2543	0.6292	44.15
	6.756	0.2608	0.3763	0.6237	0.9068	0.2608	0.6237	40.27
	7.816	0.2680	0.3830	0.6170	0.9317	0.2680	0.6170	35.76
	9.058	0.2738	0.3897	0.6103	0.9520	0.2738	0.6103	31.53
	10.123	0.2784	0.3947	0.6053	0.9679	0.2784	0.6053	28.68
	11.083	0.2816	0.3992	0.6008	0.9791	0.2816	0.6008	26.50
	12.046	0.2836	0.4043	0.5957	0.9860	0.2836	0.5957	24.56
	13.132	0.2861	0.4102	0.5898	0.9946	0.2861	0.5898	22.72
	14.068	0.2867	0.4153	0.5847	0.9966	0.2867	0.5847	21.25
	15.023	0.2876	0.4208	0.5792	1.0000	0.2876	0.5792	19.97
	17.102	0.2822	0.4392	0.5608	0.9813	0.2822	0.5608	
	19.057	0.2674	0.4809	0.5191	0.9296	0.2674	0.5191	
	21.146	0.2542	0.5329	0.4671	0.8839	0.2542	0.4671	

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REMARKS:
Corrected for stone and membrane deflections

Run at 0°C

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio **SOIL** Arctic Silt **TYPE OF TEST** CK_0UDSS **NO.** TBIDSS2

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	
	22.109	0.2503	0.5423	0.4577	0.8702	0.2503	0.5423	
	24.021	0.2145	0.6764	0.3236	0.7459	0.2145	0.3236	
	26.020	0.1764	0.7390	0.2610	0.6132	0.1764	0.2610	
	28.072	0.1655	0.7497	0.2503	0.5755	0.1655	0.2503	
	29.987	0.1571	0.7604	0.2396	0.5460	0.1571	0.2396	
	32.214	0.1468	0.7761	0.2239	0.5103	0.1468	0.2239	
	33.336	0.1355	0.7900	0.2100	0.4710	0.1355	0.2100	

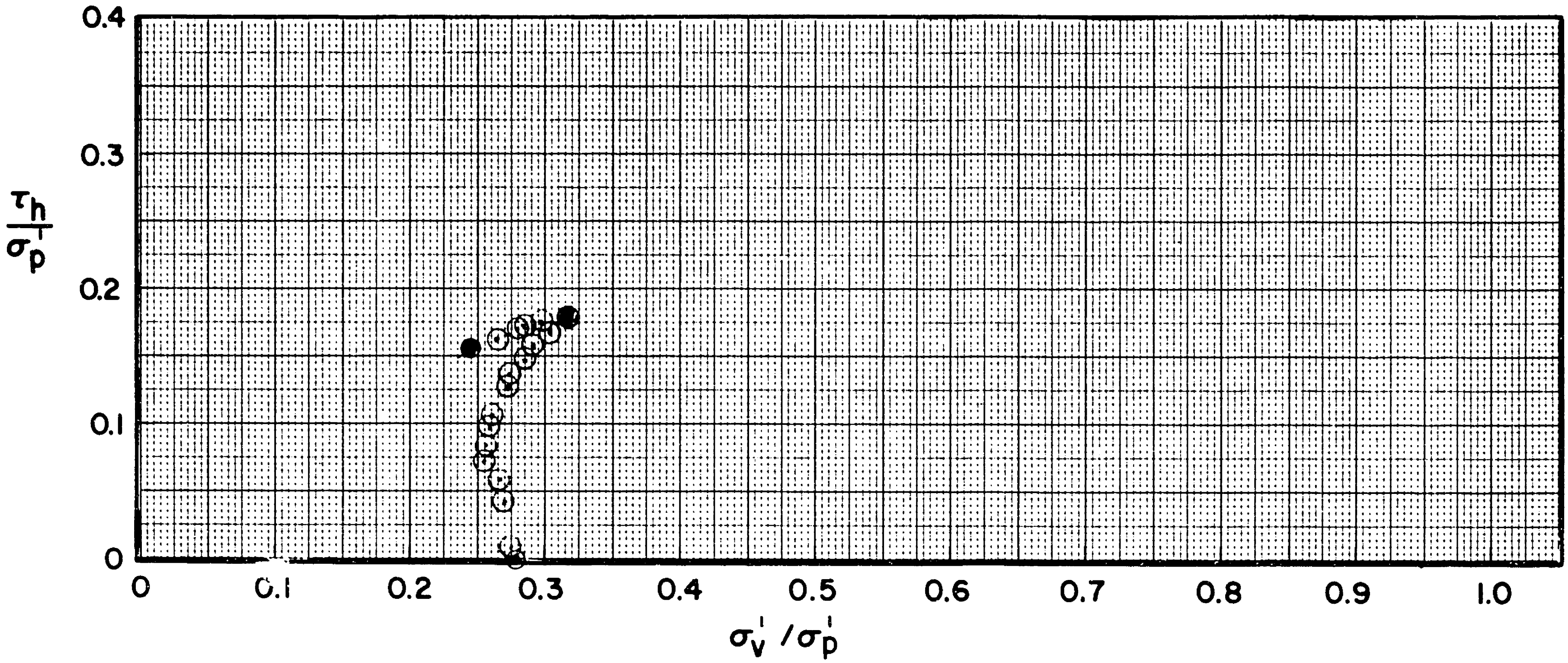
SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS: Corrected for stone and membrane deflections

• Run at 0°C

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

Test No.	Sample No.	Depth (ft)	w_N (%)	σ'_{vc} (KSC)	σ'_p (KSC)	OCR	Symbol
TDSS1	TB3-1S (with pins)	7.5'	37.5 (all) 45.1 (no sam)	0.189	0.83	4.4	



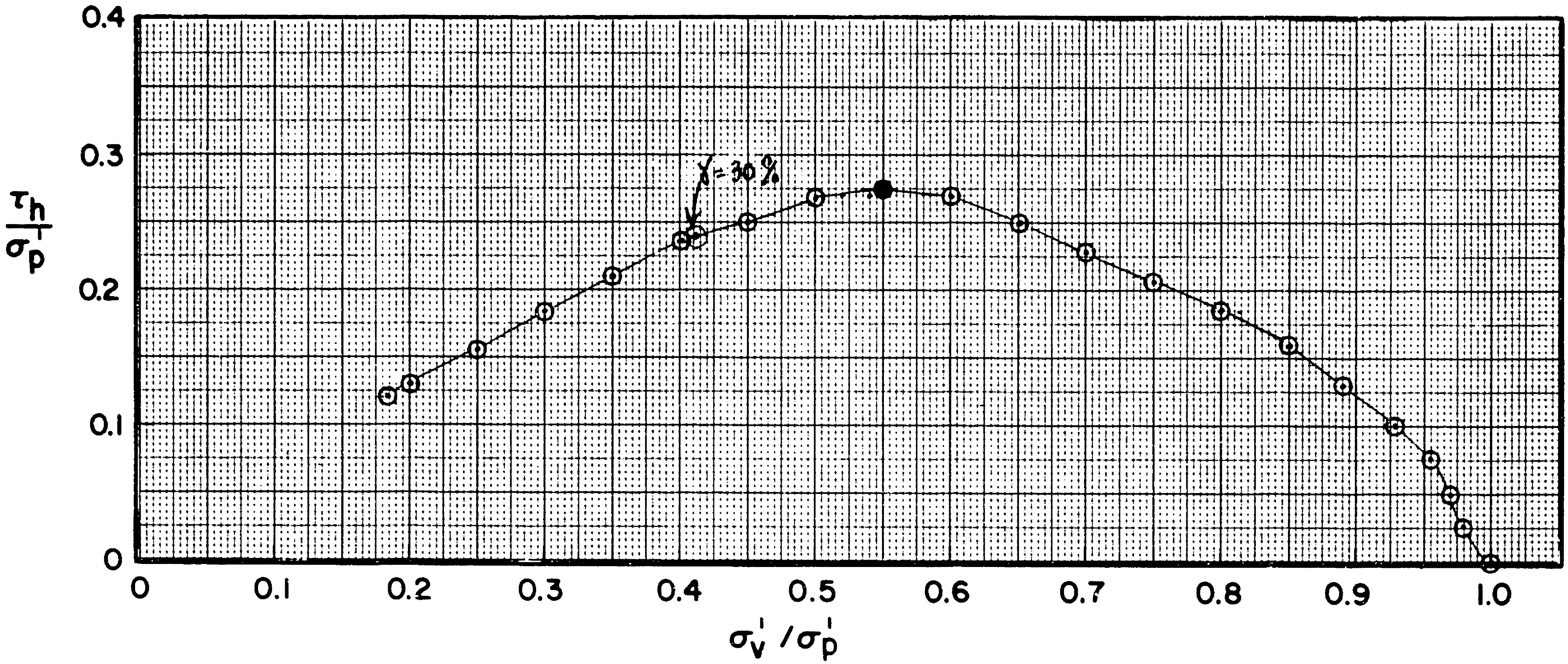
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring B3 Soil Type ARCTIC SILT

FIGURE DSS1

Test No.	Sample No.	Depth	w_N (%)	σ'_{vc} (Ksc)	σ'_p (Ksc)	OCR	Symbol
DSS1B	7B3-P3 (with pins)	7.5'	37.5 (all) 45.1 (w/o sand)	2.301	0.83	1	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

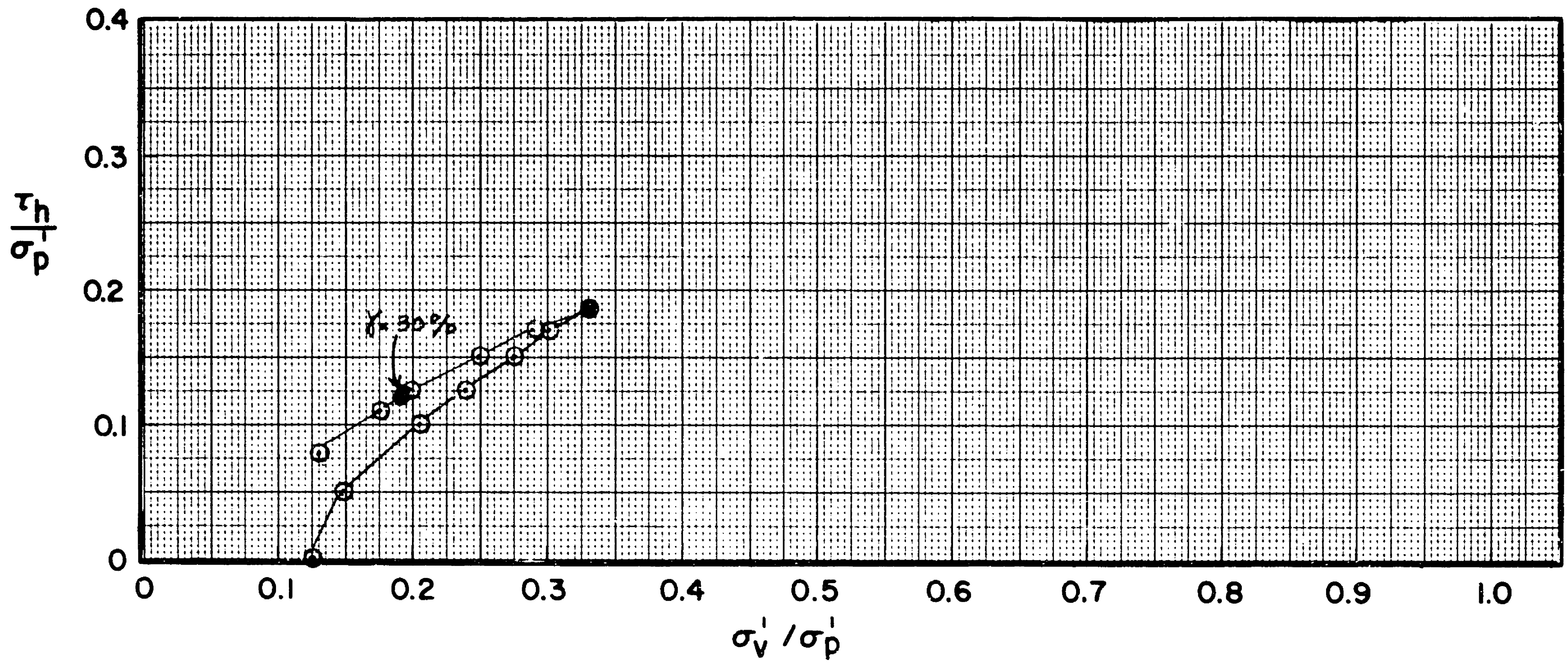
Boring ________ Soil Type TB3DSS1 SB (OCR=1) Arctic Silt (Smith Bay)

FIGURE

CO9 12/27/85

Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (ksc)	σ'_p (ksc)	OCR	Symbol
TB3DSS2	B3-P3	7.7'	40.2	0.308	2.44	7.93	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



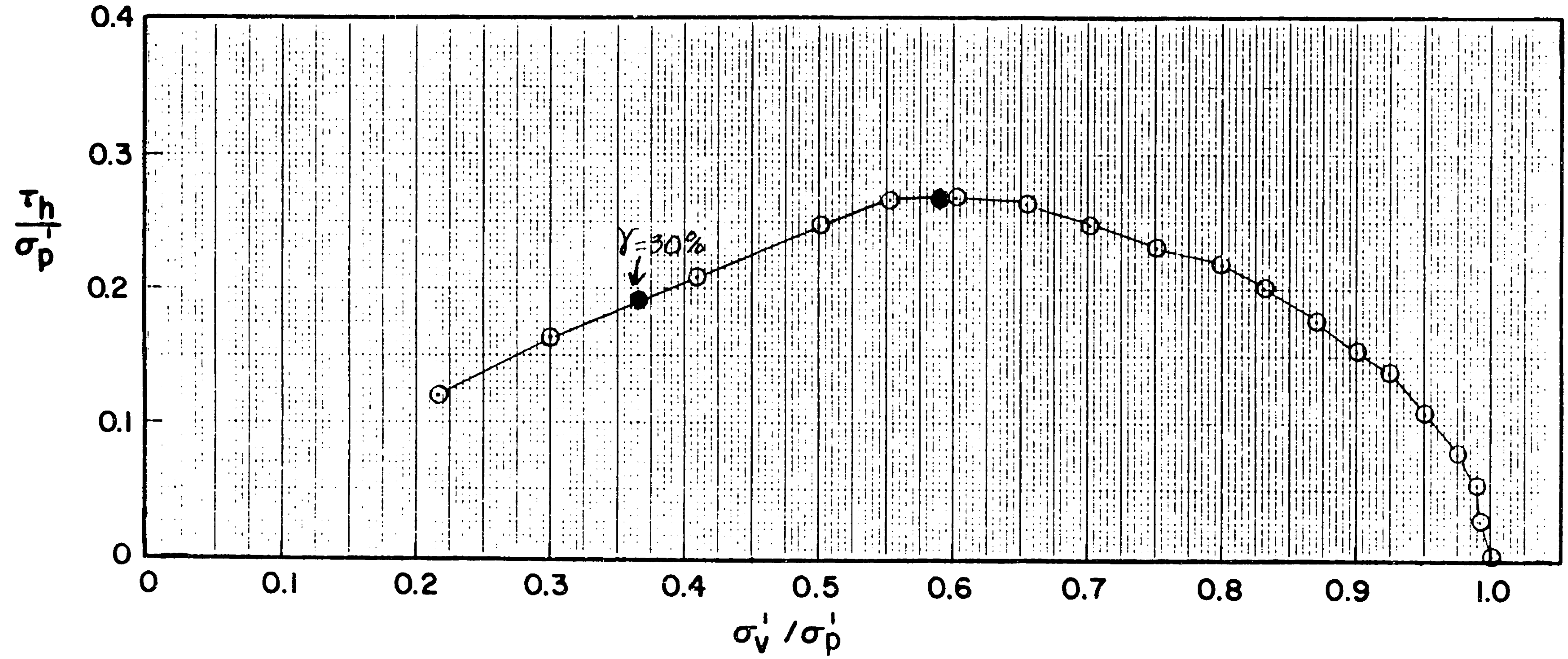
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring TB3P3 Soil Type Arctic Silt

FIGURE DSS-2(c)

Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (Ksc)	σ'_p ( )	OCR	Symbol
DSS3	TB3-P3	7.9'	54.4	2.46		1	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



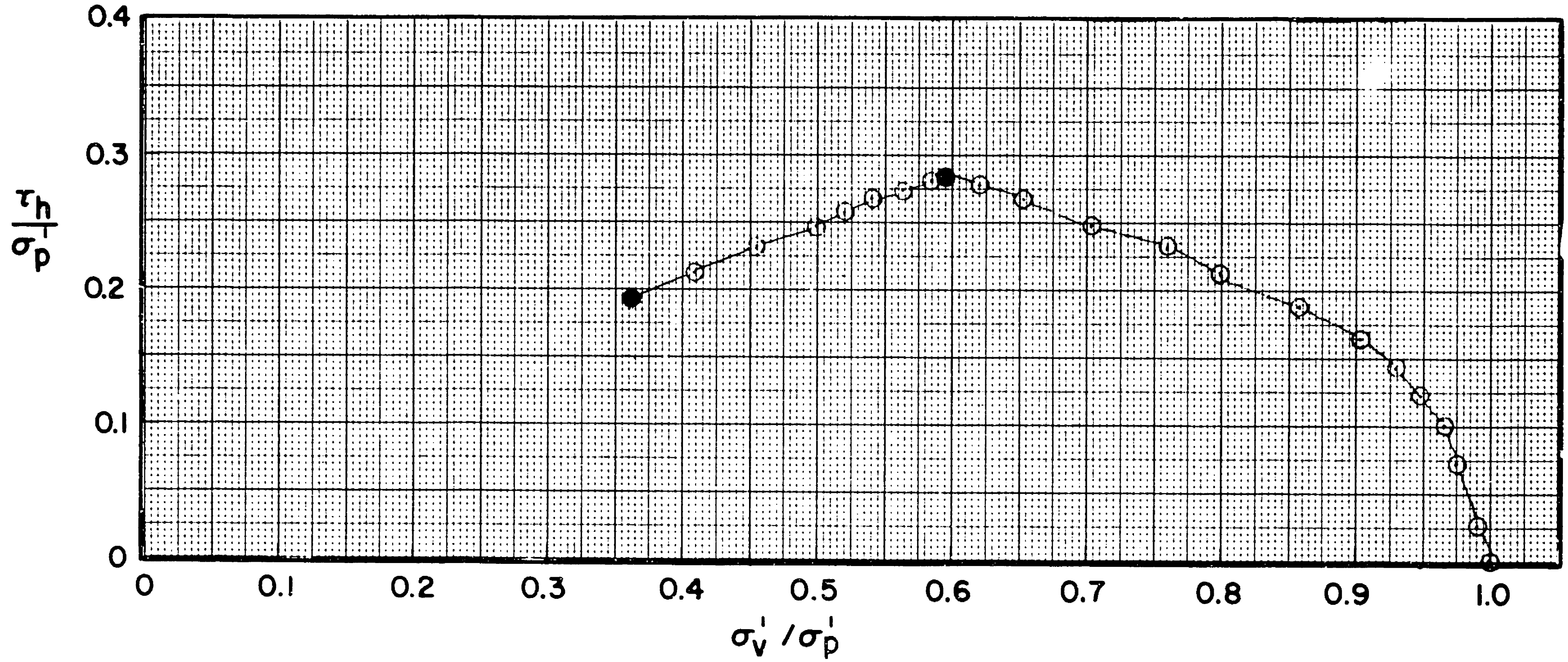
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring TB3-P3 Soil Type Arctic Silt

FIGURE DSS-3 (c)

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

Test No.	Sample No.	Depth ([illegible])	w_N (%)	σ'_{vc} ([illegible])	σ'_p ( )	OCR	Symbol
[illegible]54	[illegible]	4.7	35.3	2.4		1	



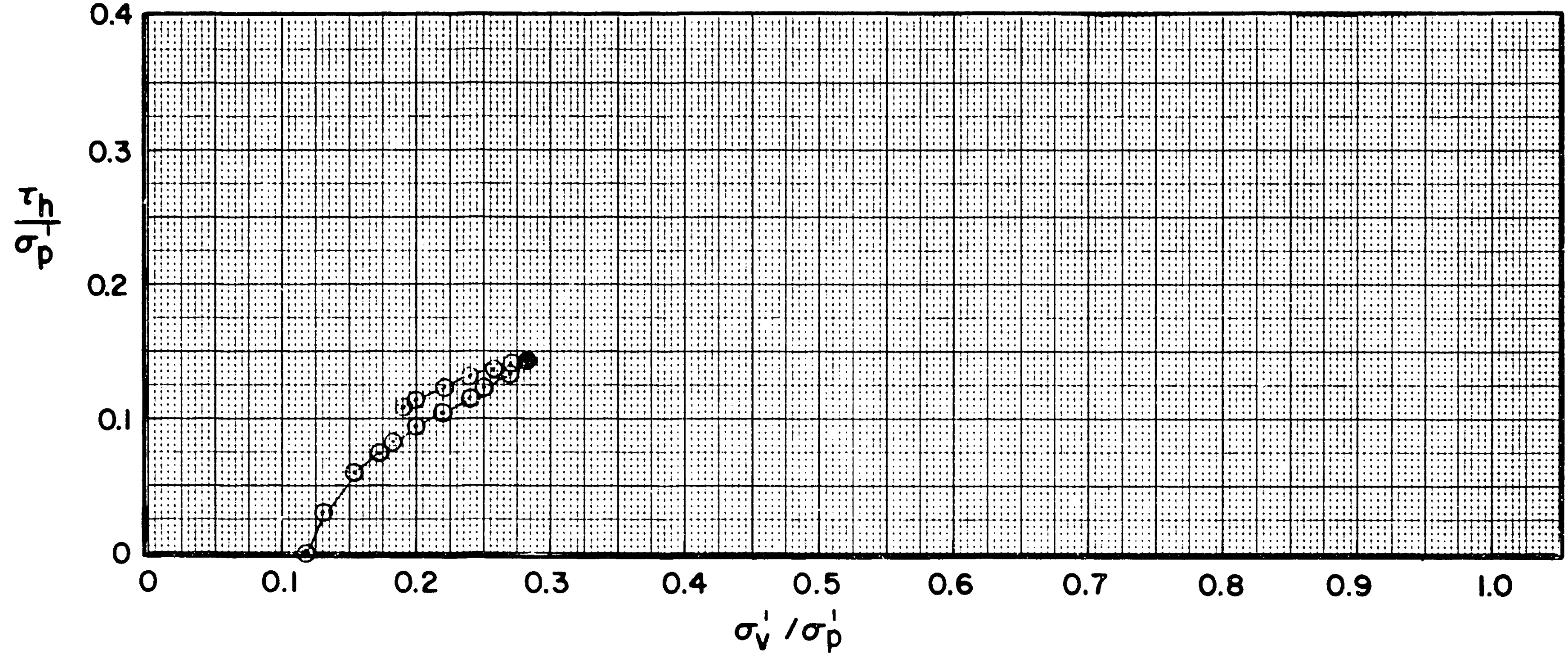
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring TB3 Soil Type Arctic Silt

FIGURE DSS-4

Test No.	Sample No.	Depth ([illegible])	w_N (%)	σ'_{vc} ([illegible])	σ'_p ( )	OCR	Symbol
TB3DSS5	TB3-P2	4.6'	37.2	[illegible]		8.5	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring TB3 Soil Type [illegible]

FIGURE DSS-5(c)

Test No.	Sample No.	Depth	w_N (%)	σ'_{vc} (ksc)	σ'_p ( )	OCR	Symbol
DSS 6	T1B-02	7'	41.5			1	

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DEPT. OF CIVIL ENGR.
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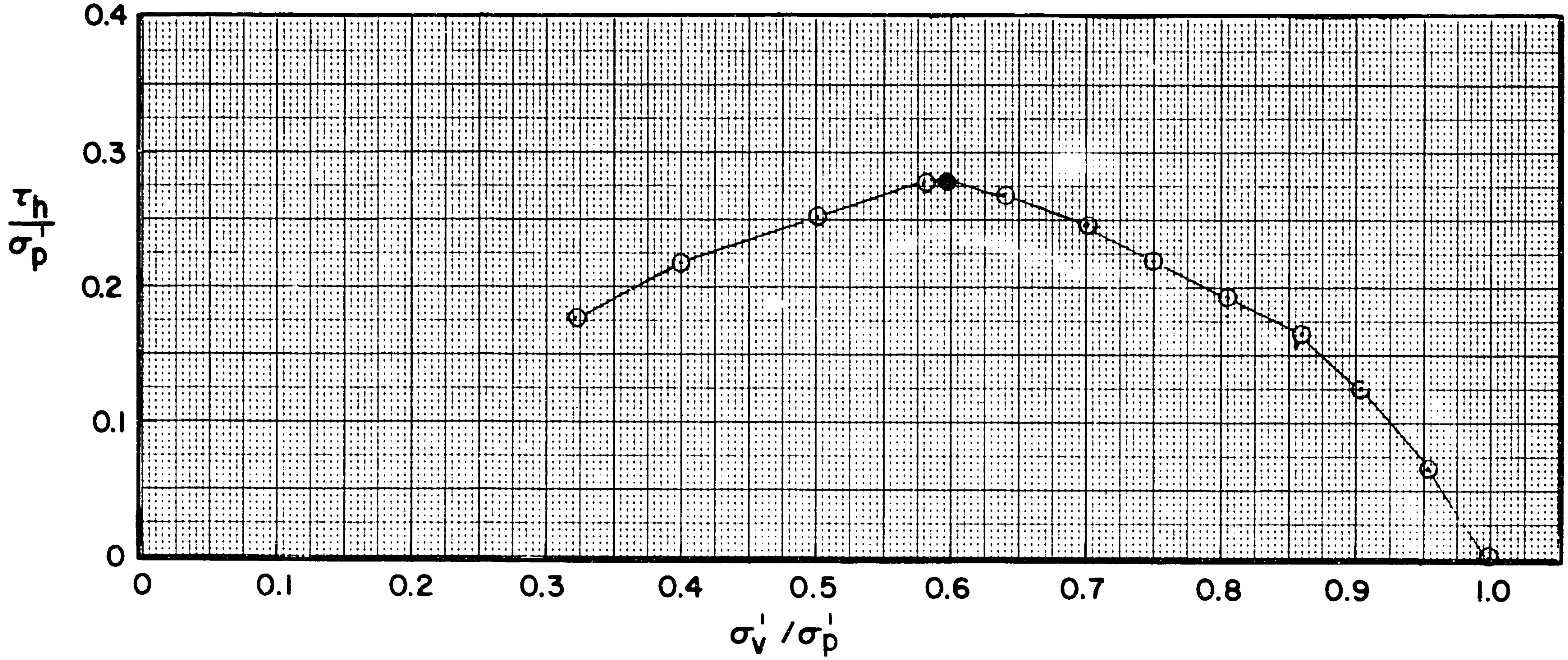


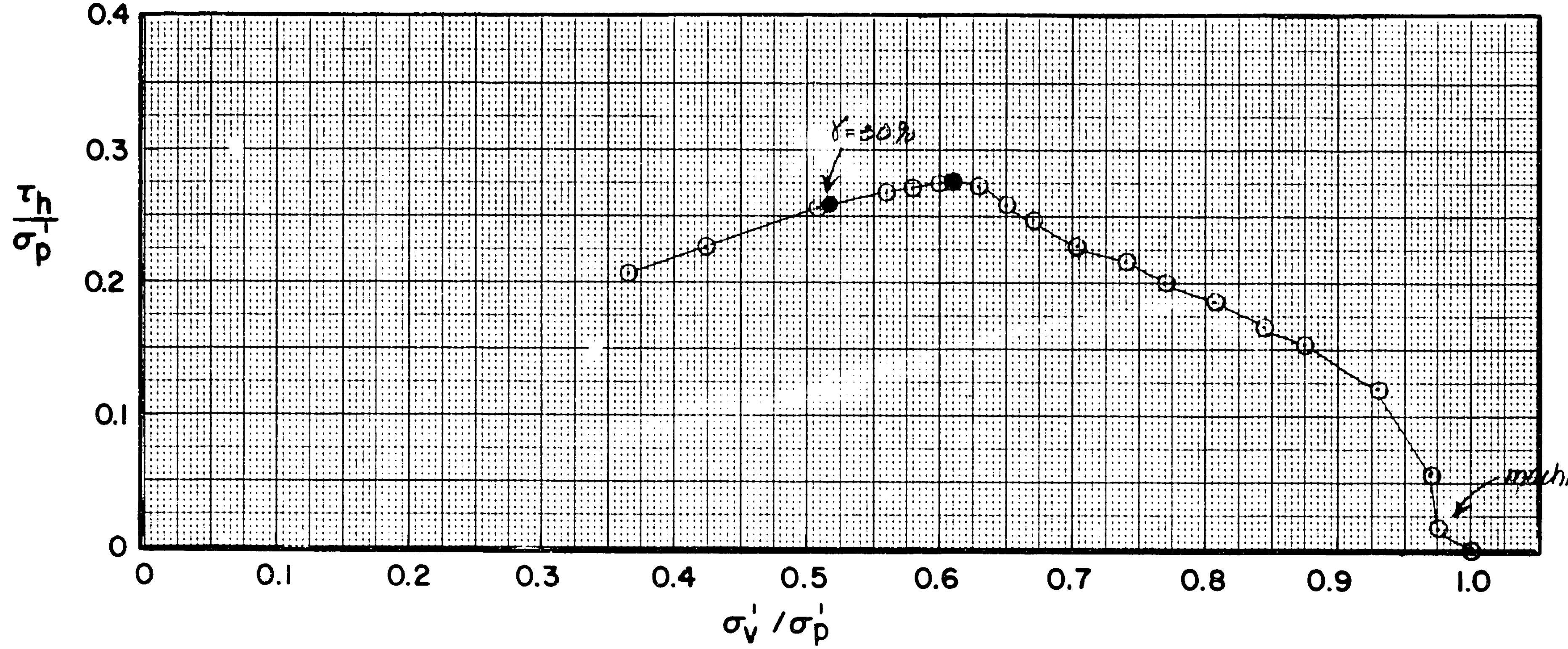
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring 1B-D-2 Soil Type Arctic Silt

FIGURE DSS-6 (c)

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

Test No.	Sample No.	Depth (ft)	w_N (%)	σ'_{vc} (ksc)	σ'_p (ksc)	OCR	Symbol
T5B1 DSS7	T5B1-P3 (small piece)	7'	36.4	2.41	2.41	1 w/ pins	



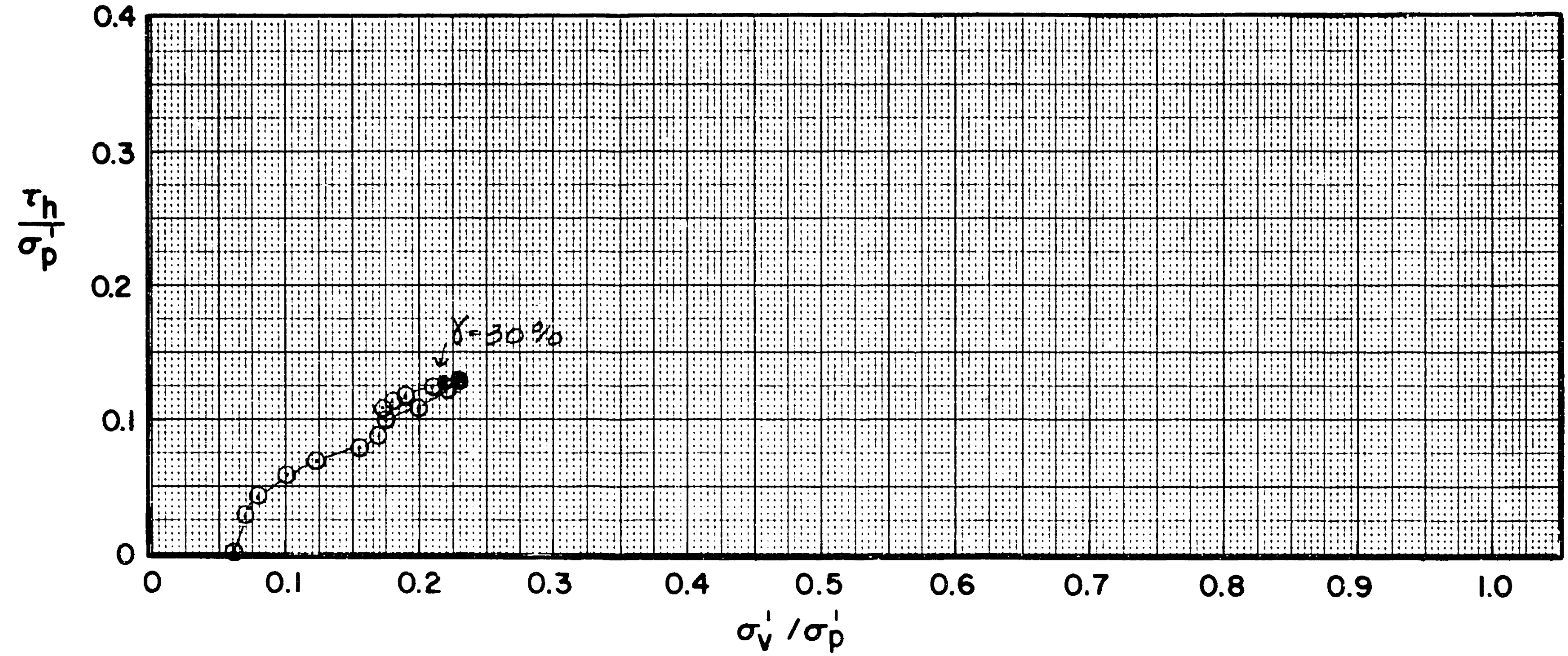
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring T5B1 Soil Type Arctic Silt

FIGURE DSS-7 (c)

Test No.	Sample No.	Depth (ft)	w_N (%)	σ'_{vc} (ksc)	σ'_p (ksc)	OCR	Symbol
T5B1 DSS8	T5B1-F5 (with pins)	7.1'	38.3	0.153	2.41	15.75	

GEOTECHNICAL LABORATOR
DEPT. OF CIVIL ENGR.
M.I.T.



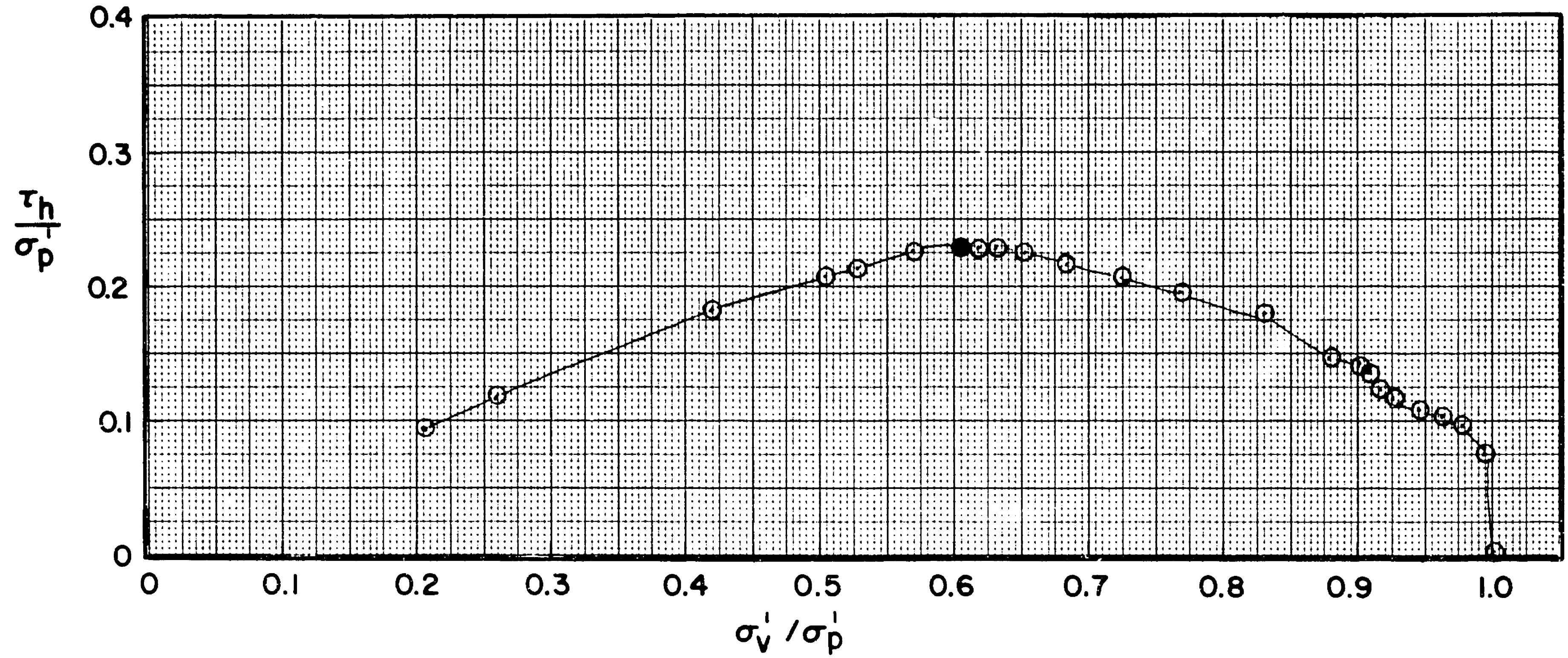
NORMALIZED STRESS PATHS FROM CK_oUDSS TESTS

Boring T5B1-F5 Soil Type ARCTIC SILT

FIGURE DSS-8(c)

Test No.	Sample No.	Depth KE (Ft)	w_N (%)	σ'_{vc} (KSF)	σ'_p ( )	OCR	Symbol
7B1DS09	71B-02	7.2	45.9	15.56		1	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



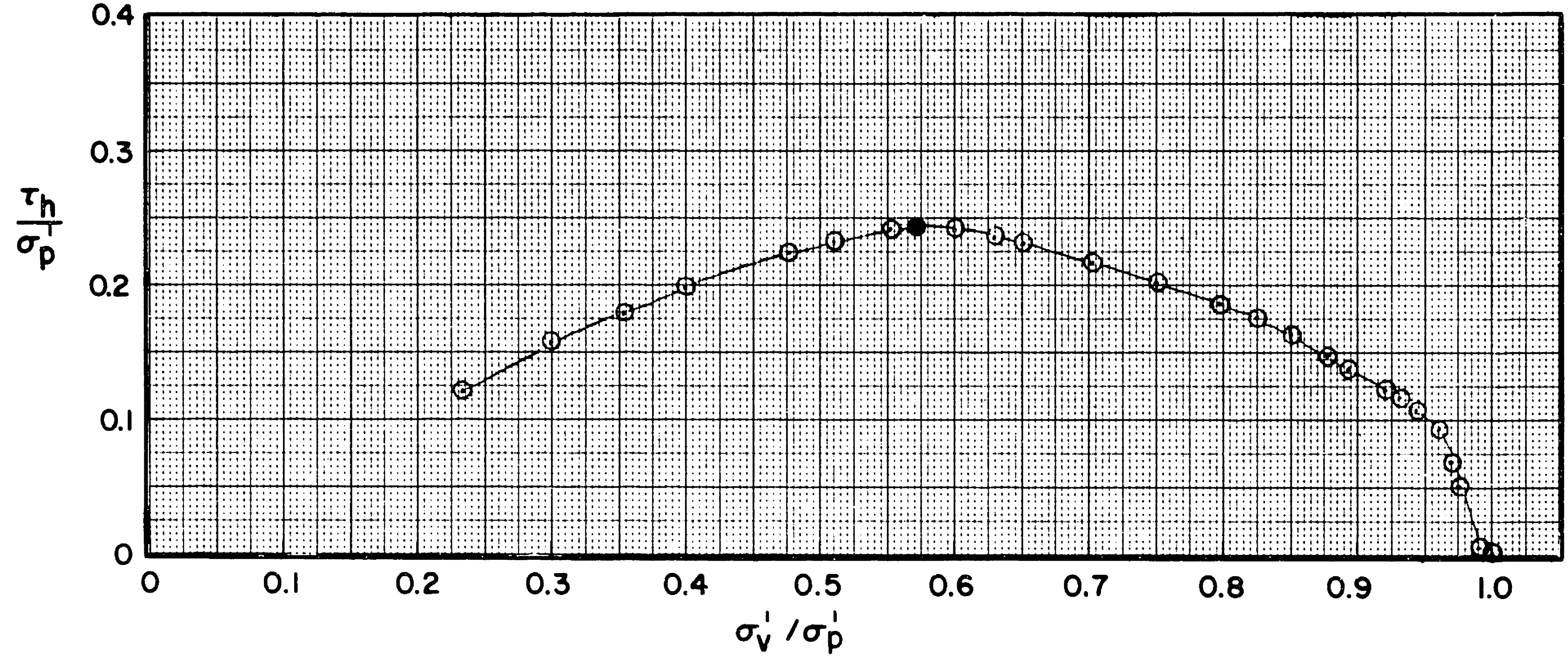
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring ________ Soil Type ______________________

FIGURE

Test No.	Sample No.	Depth (PE)	w_N (%)	σ'_{vc} (KSF)	σ'_p ( )	OCR	Symbol
TD510	75B1-P3	8.4'	38.4	15.6		1	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring 75B1 Soil Type Arctic Silt - Smith Bay, Site T

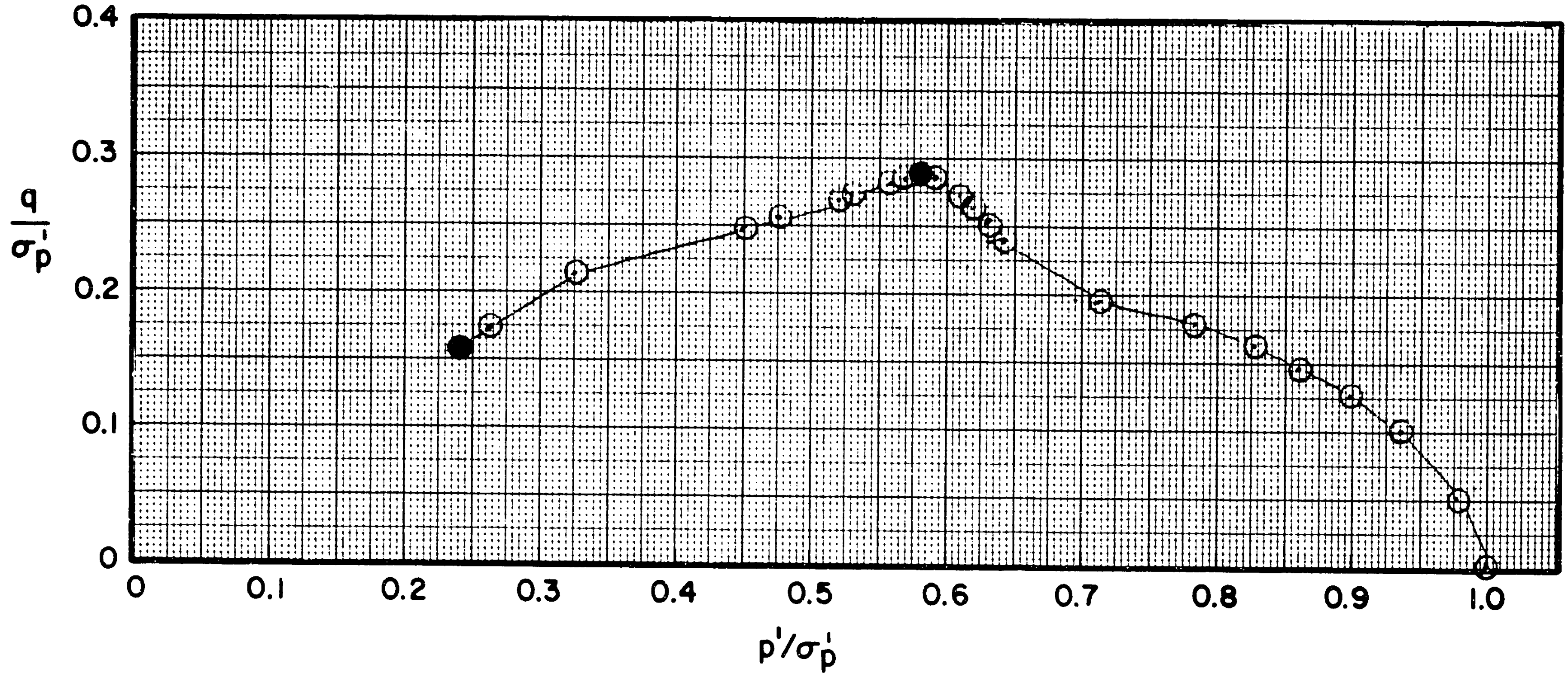
FIGURE

Test No.	Sample No.	Depth ft	w_N (%)	σ'_{vc} (ksf)	K_c	OCR	Sym.
T5B1(CS511)	T1B-02(S1)	7.4'	43.5	2.44		1	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR., M. I. T.

$$q = 0.5(\sigma_v - \sigma_h)$$

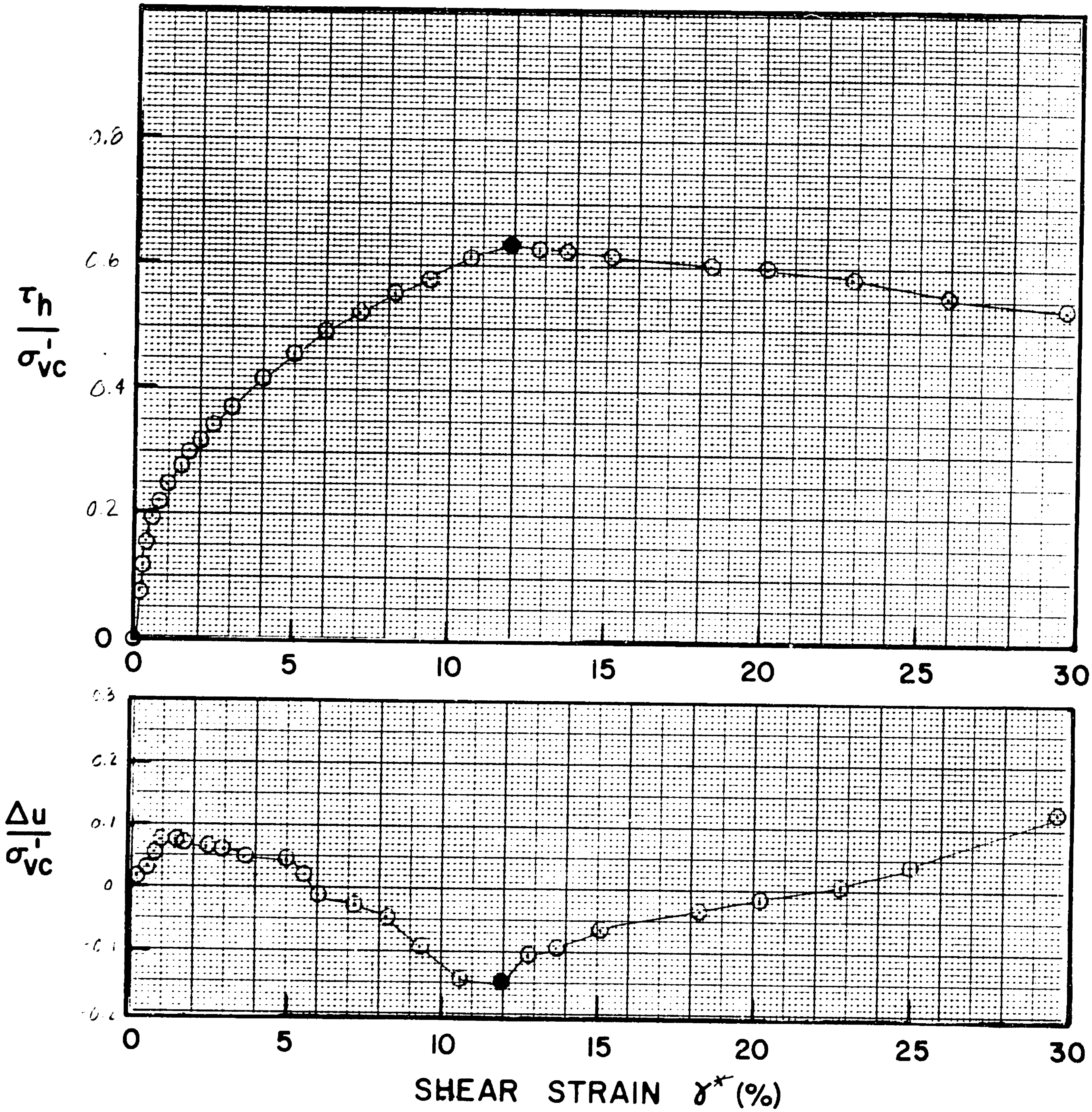
$$\bar{p} = 0.5(\sigma'_v + \sigma'_h)$$



NORMALIZED STRESS PATHS FROM CK_oUC TESTS

BORING ____________ SOIL TYPE ______________

FIGURE



Sample No. TB3 P3 w_N(%) 37.5 (all) 45.1 (w/o sand) σ'_{vc}(KSC) 0.189 t_c(Days) ____

Depth (RE) 7.5' w_L(%) 35.3 $\sigma'^{\dagger}_p$ (KSC) 0.83 OCR 4.4

Soil Type Arctic Silt (Smith Bay) w_P(%) 23.2 Estimated σ'_{vo}(KSC) 0.188

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

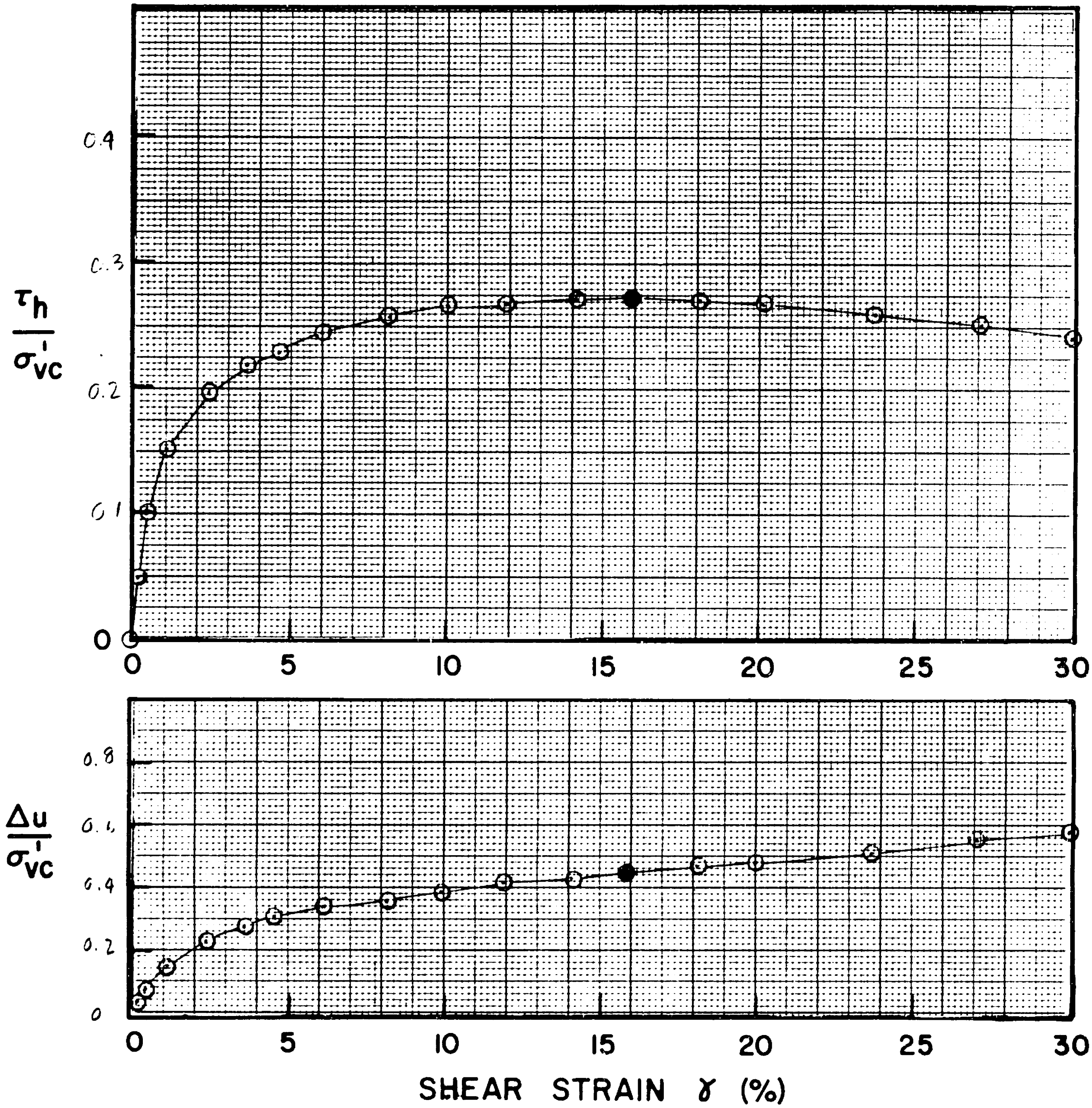
CK_oUDSS TEST NO. DSS1 (Recompression)

* for $h_{sample-hpins}$

FIGURE DSS1

† from adjacent oedometer

GY 1/2/86



Sample No. T B3-P3 w_N(%) 37.5 (all) 45.1 (w/o Sand) σ'_{vc}(KSC) 2.301 t_c(Days) ____

Depth 75' w_L(%) 35.3 σ'_p † (KSC) 0.83 OCR 1

Soil Type Arctic Silt (Smith Bay) w_p(%) 23.2 Estimated σ'_{vo}(KSC) 0.188

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

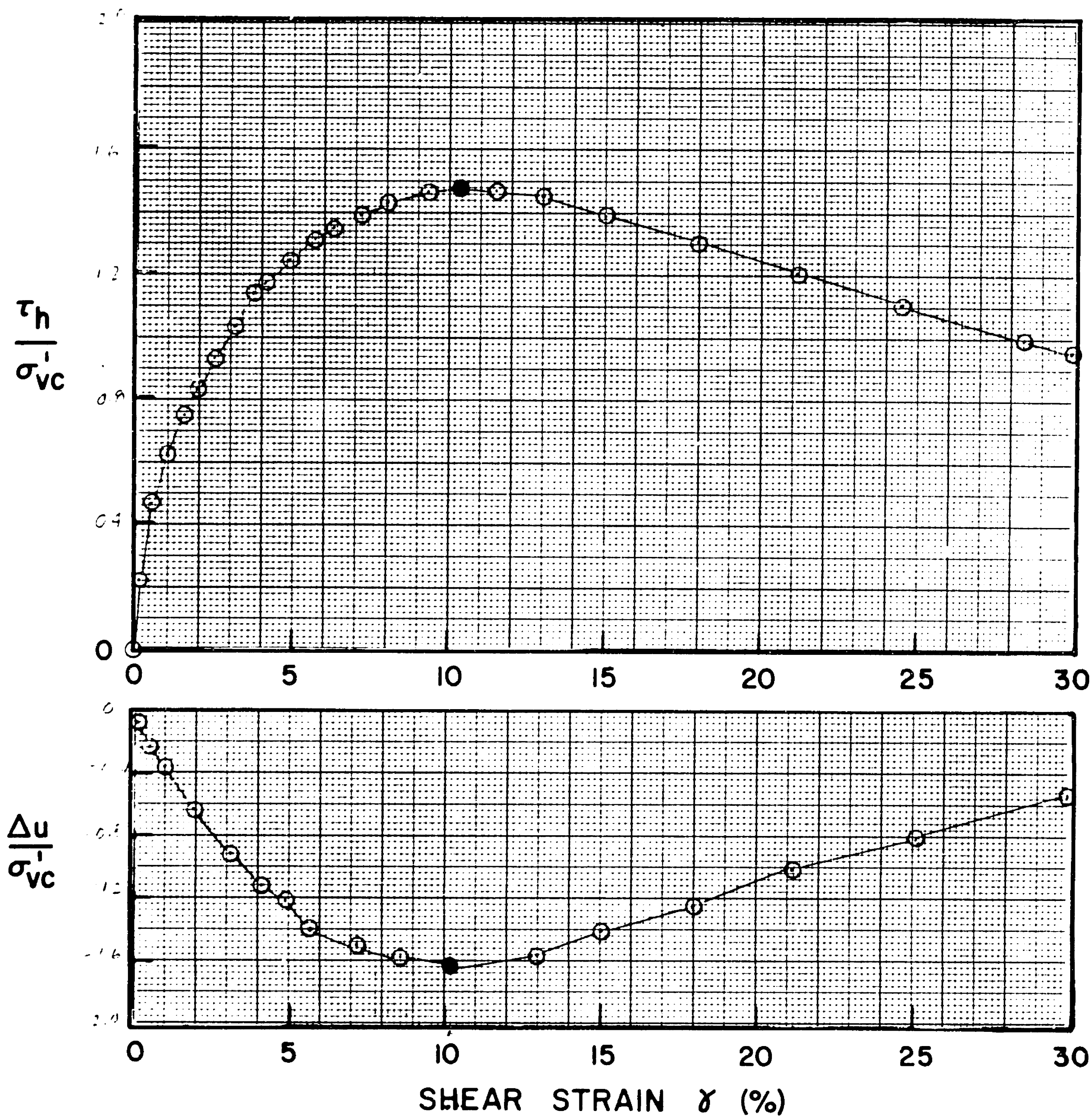
NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. DSS-1B (Second Shear)

* regrt - h fins

FIGURE DSS1B

† from adjacent oedometer



Sample No. TB3-P3 w_N(%) 40.2 σ'_{vc}(ksc) 0.368 t_c(Days) ____

Depth (RE) 7.7' w_L(%) 35.3 σ'_p (ksc) 2.44 OCR 7.93

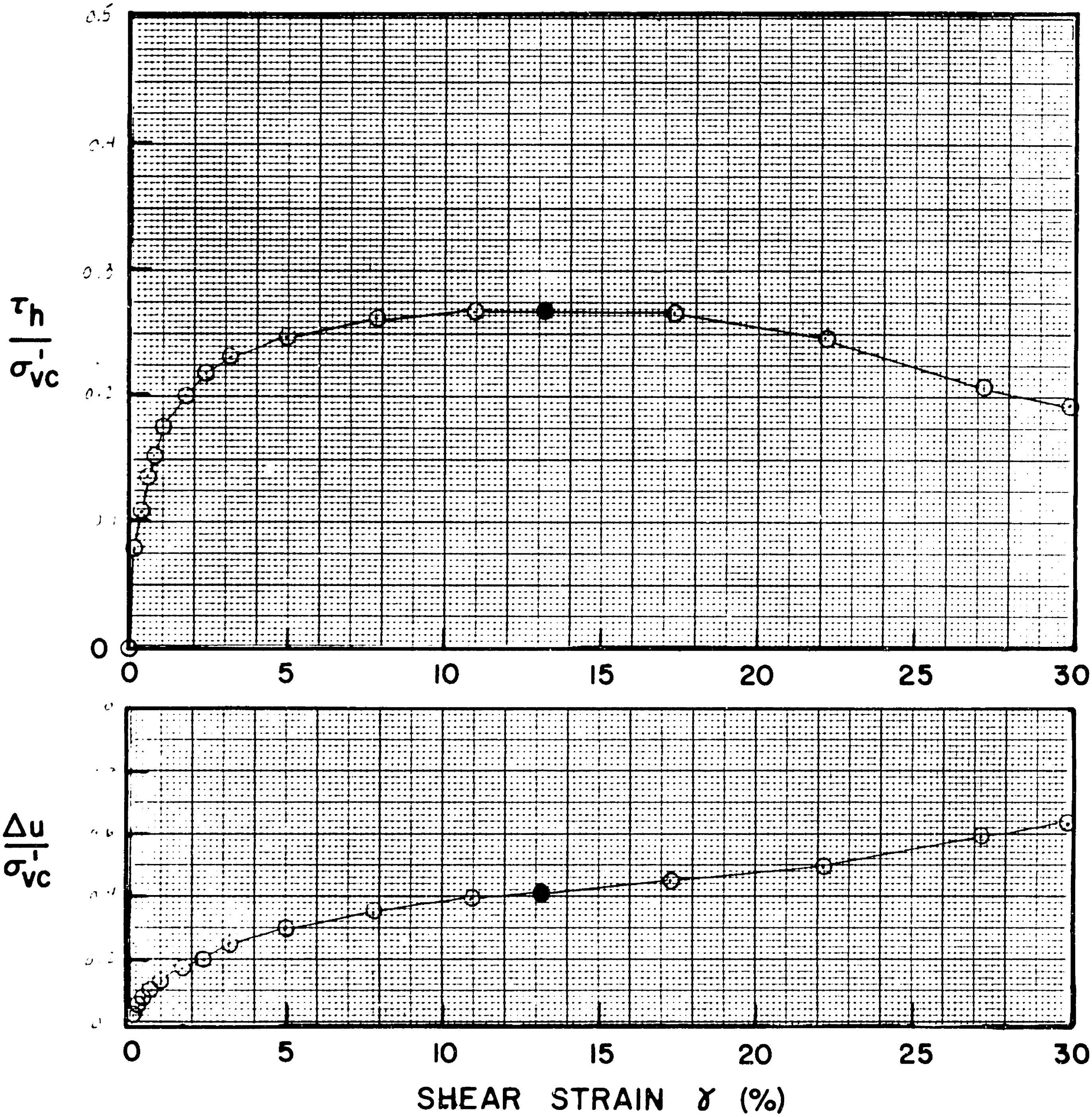
Soil Type Arctic Silt w_P(%) 23.2 Estimated σ'_{vo}(ksc) 0.193

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CK_oUDSS TEST NO. TB3DSS2

FIGURE DSS-2

GX 2/12/86



Sample No. IB3-P3 w_N(%) 54.4 σ'_{vc}(KSC) 2.45 t_c(Days) ___

Depth (FE) 79' w_L(%) 65.3 σ'_p ( ) ___ OCR 1

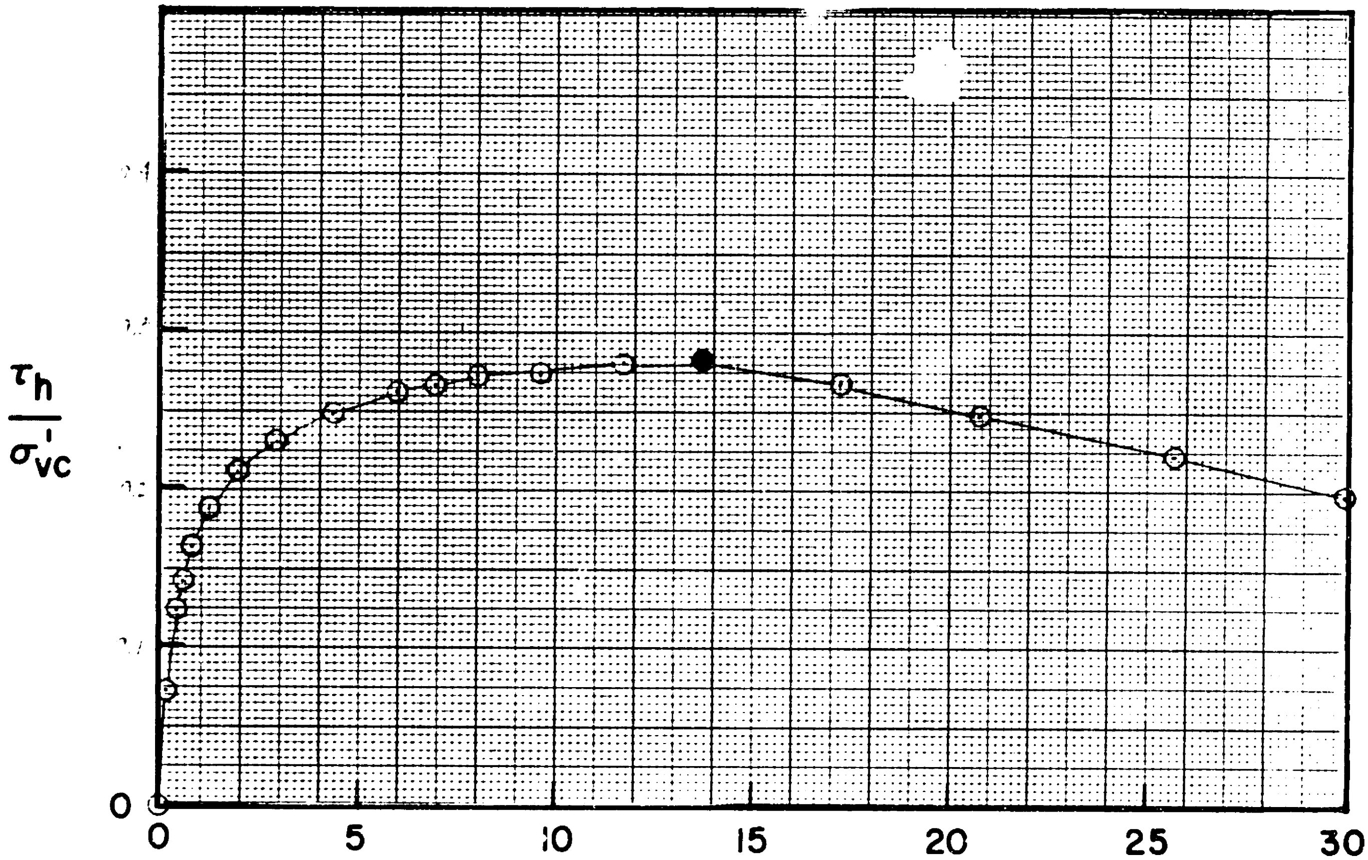
Soil Type Arctic Silt w_P(%) 32.7 Estimated σ'_{vo}(KSC) 0.199

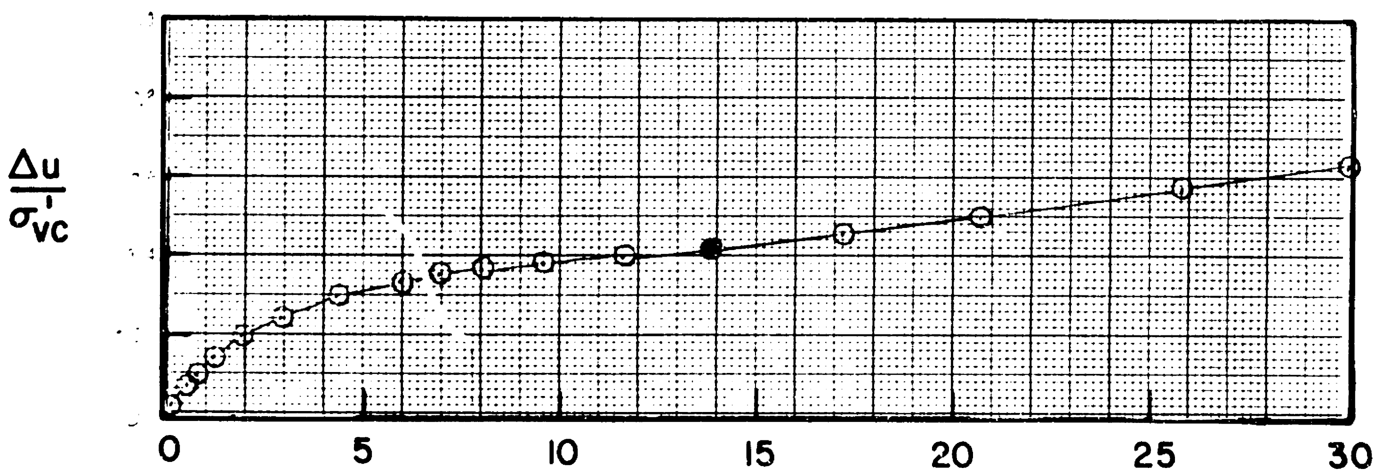
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. DSS-3

FIGURE DSS-3





Sample No. TB2 1 w_N(%) 35.3 σ'_{vc}(KSC) 2.4 t_c(Days) ___

Depth (ME) 4.7 ft w_L(%) 50.2 σ'_p ( ) ___ OC'. 1

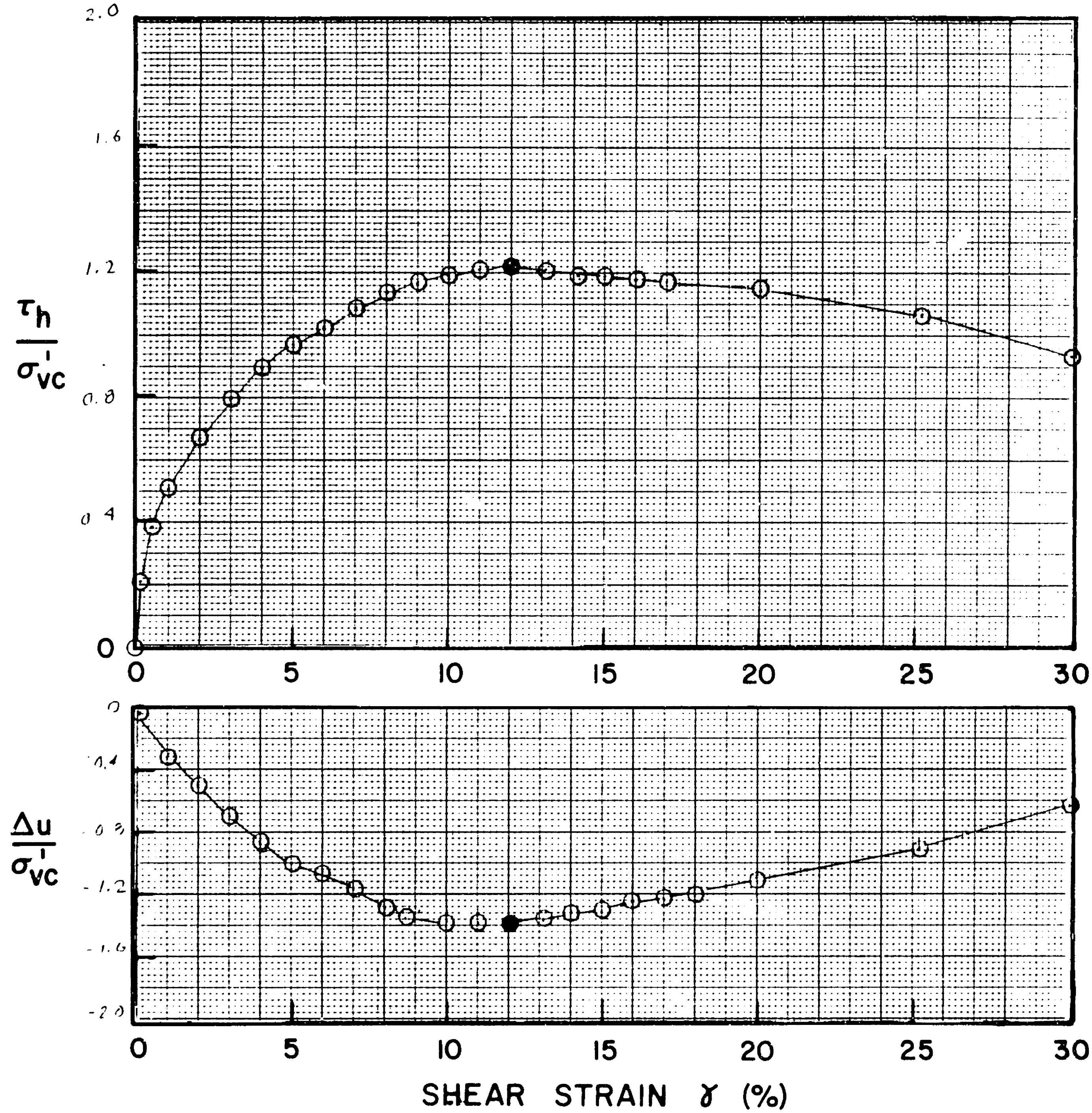
Soil Type Arctic SP w_P(%) 26.2 Estimated σ'_{vo}(KSC) 0.113

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. TB2

FIGURE DSS-4



Sample No. TB3-P2 w_N(%) 37.2 σ'_{vc}(ksc) .237 t_c(Days) ____

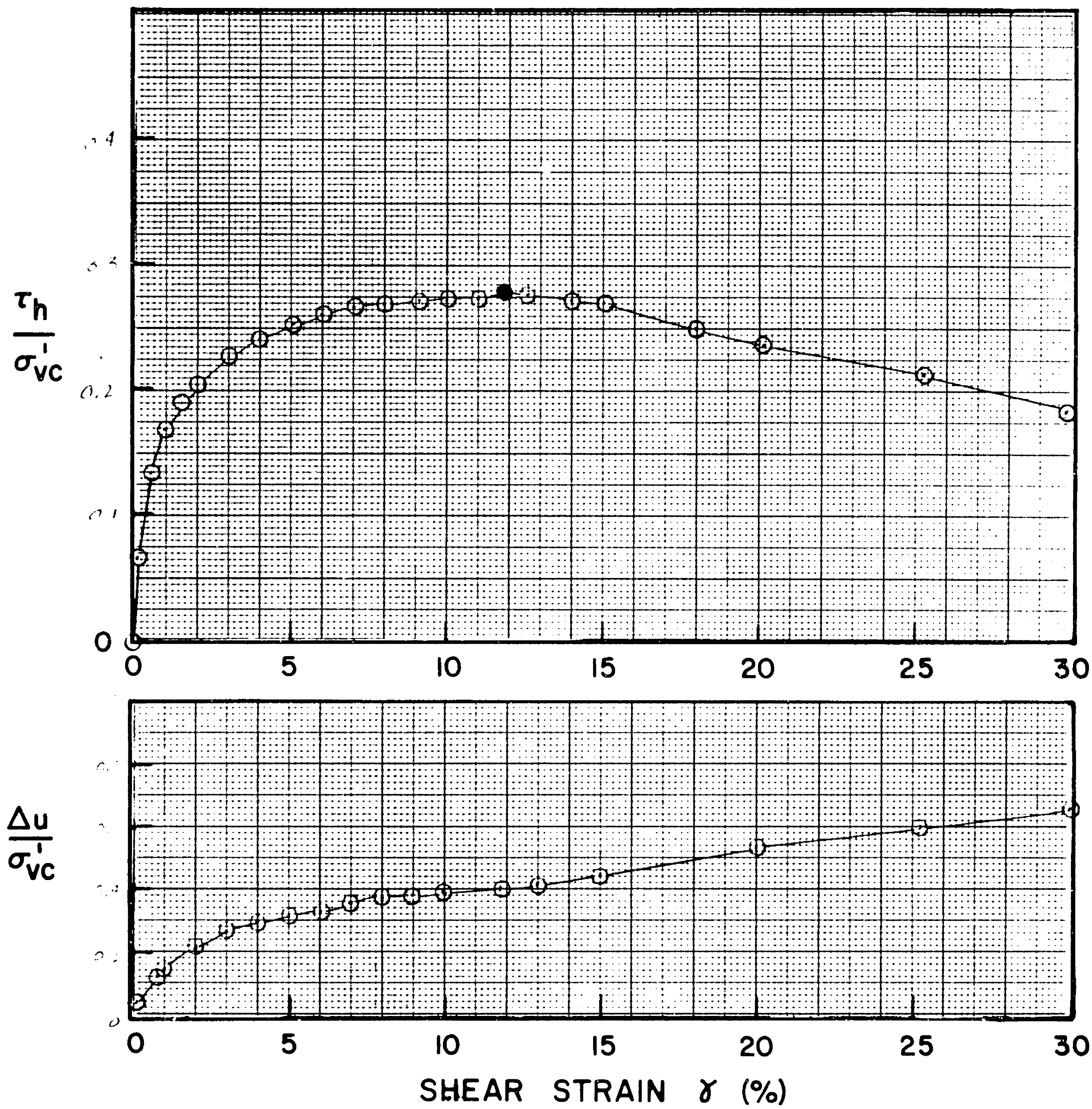
Depth (ft) 4.6' w_L(%) 50.2 σ'_p ( ) ____ OCR 8.5

Soil Type Arctic Silt w_P(%) 26.2 Estimated σ'_{vo}(ksc) 0.110

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CK_oUDSS TEST NO. TB3-DSS5

FIGURE DSS-5



Sample No. T1B-02 w_N(%) 41.5 σ'_{vc}(KSC) 2.41 t_c(Days) ____

Depth 7' w_L(%) 52.7 σ'_p ( ) ____ OCR 1

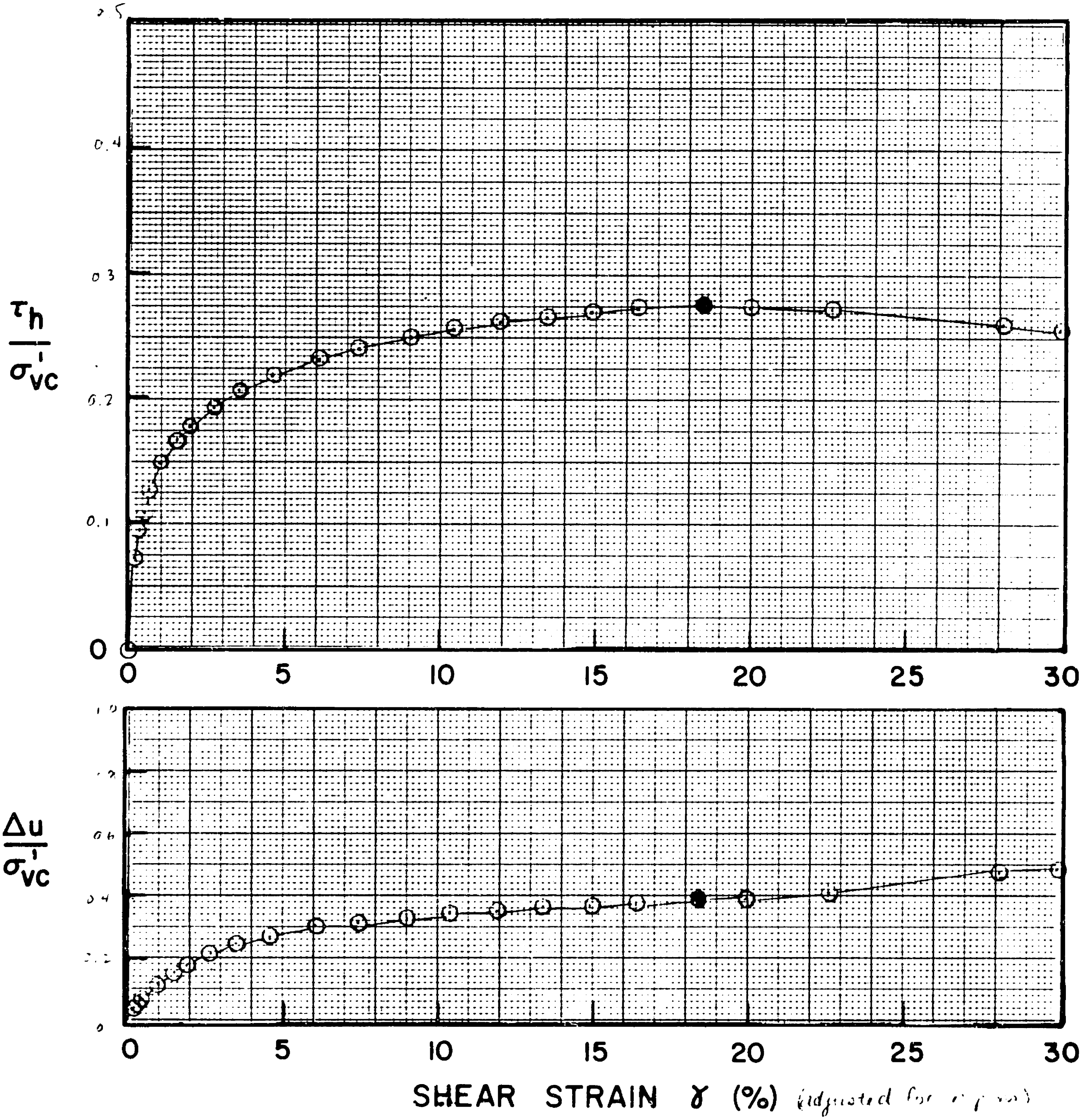
Soil Type Arctic Silt w_P(%) 27.6 Estimated σ'_{vo}(KSC) 0.107

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. TB1-DSS 6

FIGURE DSS-6



Sample No. I5B1P3 w_N(%) 36.4 σ'_{vc}(ksc) 2.41 t_c(Days) ___

Depth (RE) 7' w_L(%) 47.3 σ'_p ( ) ___ OCR 1

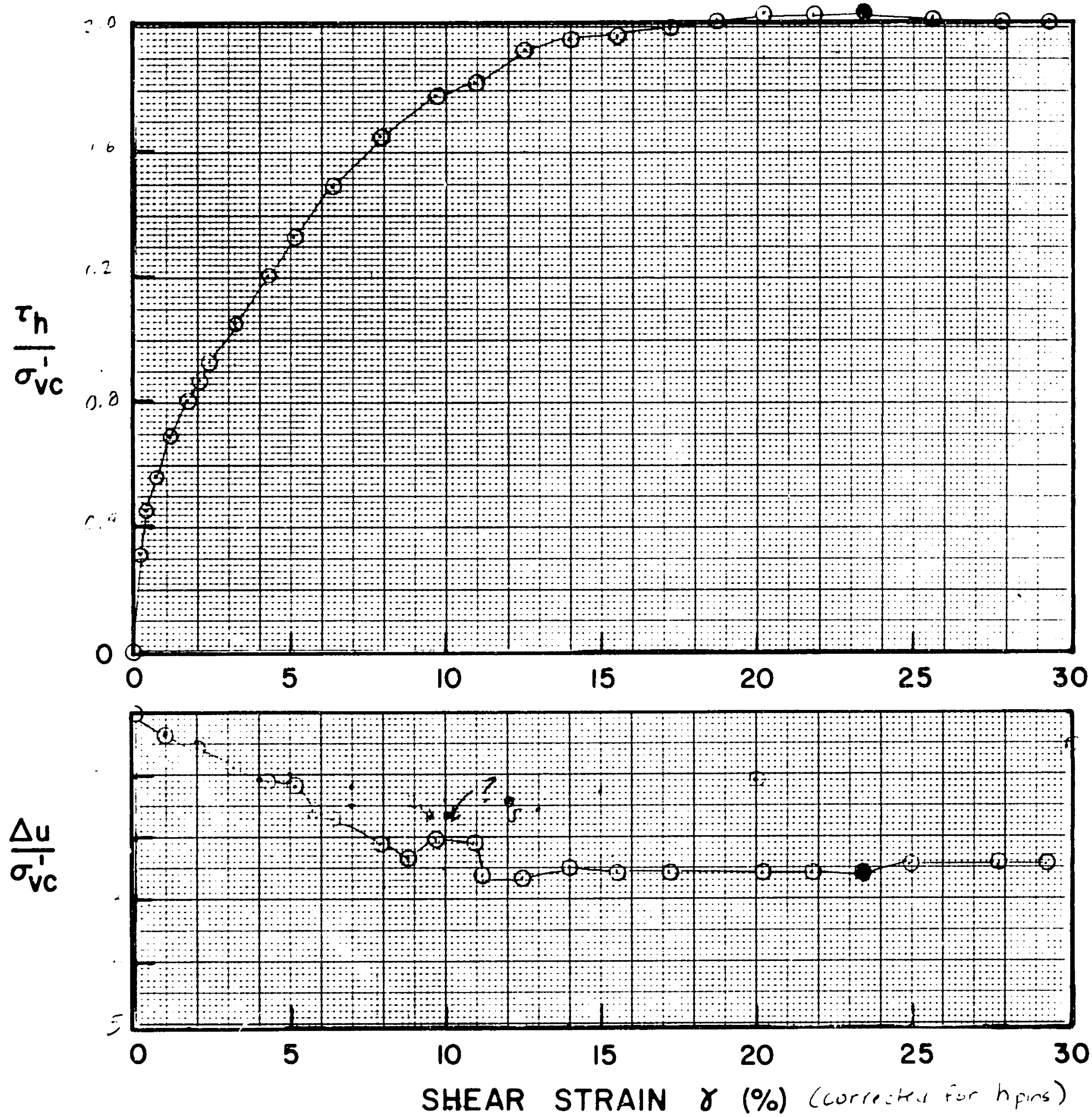
Soil Type Arctic Silt w_P(%) 26.8 Estimated σ'_{vo}(ksc) 0.188

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. T5B1DSS7 w/ pins

FIGURE DSS-7 (b)



Sample No. BB1-P3 w_N(%) 33.3 σ'_{vc}(KSC) 3.03 t_c(Days) ____

Depth (FT.) 7.1' w_L(%) 47.3 σ'_p (KSC) 3.41 OCR 15.75

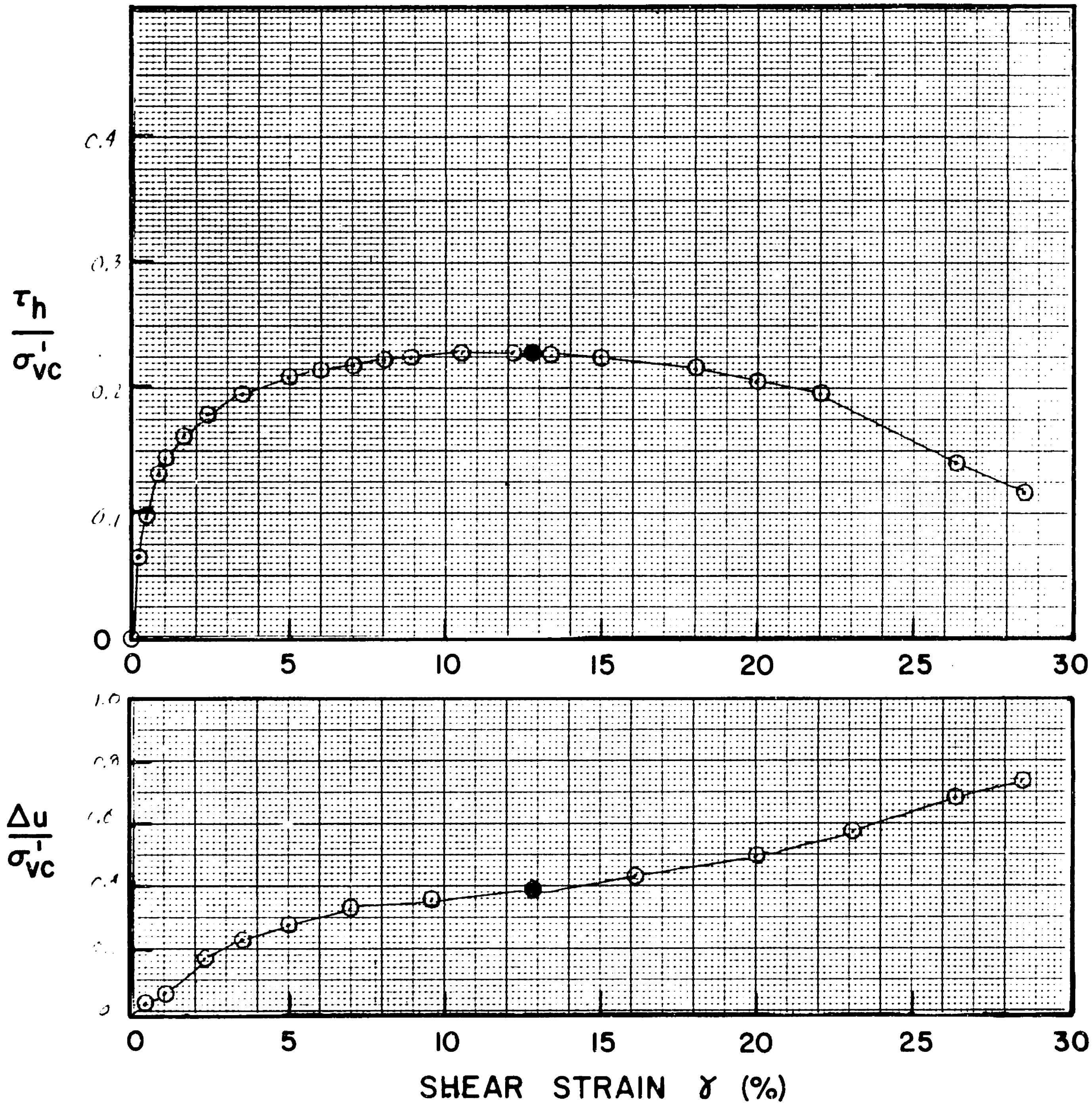
Soil Type Arctic Silt w_P(%) 26.8 Estimated σ'_{vo}(KSC) 0.191

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. BB1-DS8

FIGURE DSS-8 (b)



Sample No. JIB-02 w_N(%) 45.9 σ'_{vc}(KSF) 15.56 t_c(Days) ____

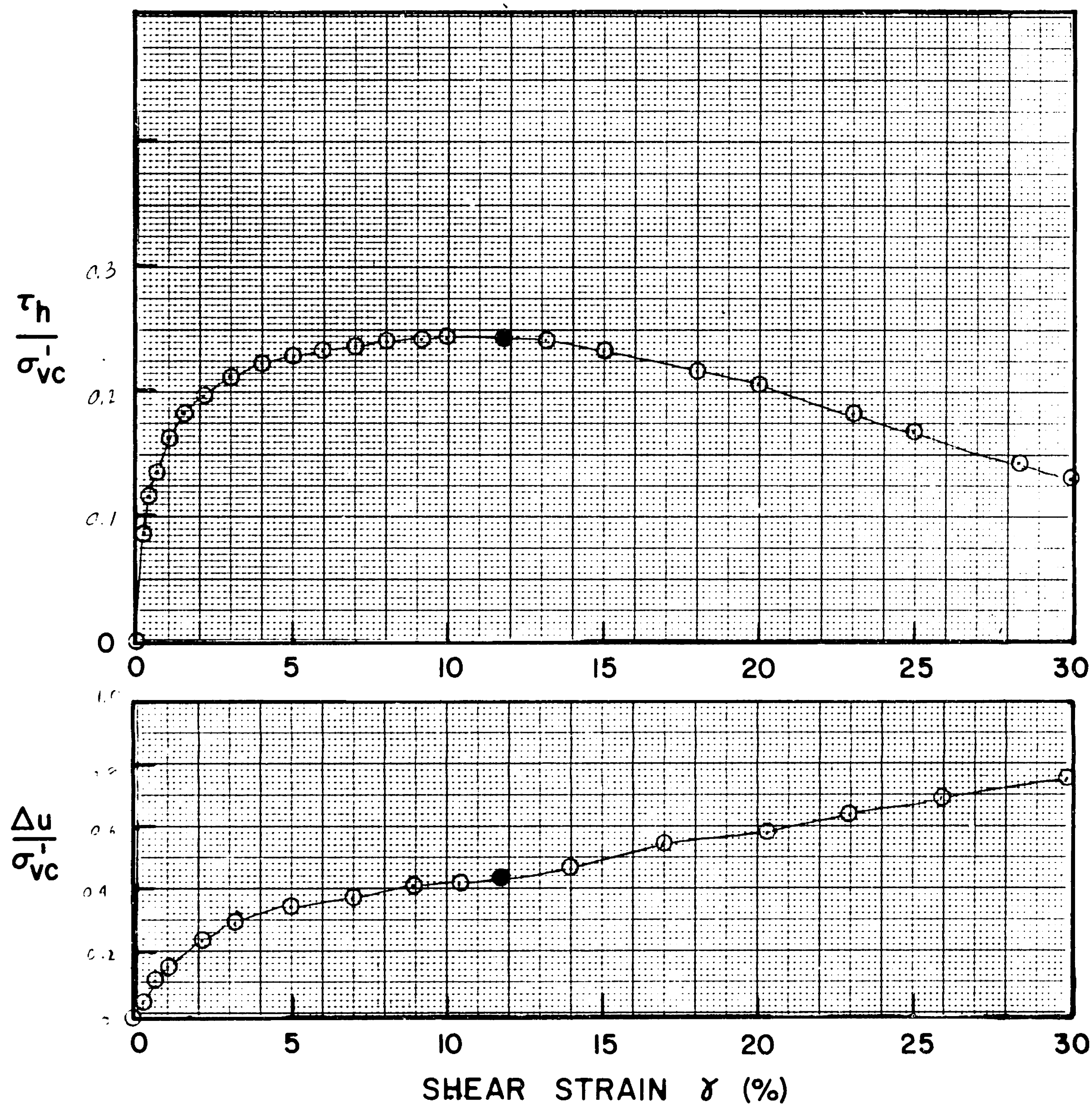
Depth (ft) 7.2' w_L(%) 52.7 σ'_p ( ) ____ OCR 1

Soil Type Arctic Silt w_P(%) 27.6 Estimated σ'_{vo}(KSC) 0.113

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CK_oUDSS TEST NO. JB1DSS9

FIGURE



Sample No. T5B1-P3 w_N(%) 33.4 σ'_{vc}(KSF) 15.6 t_c(Days) 1.0

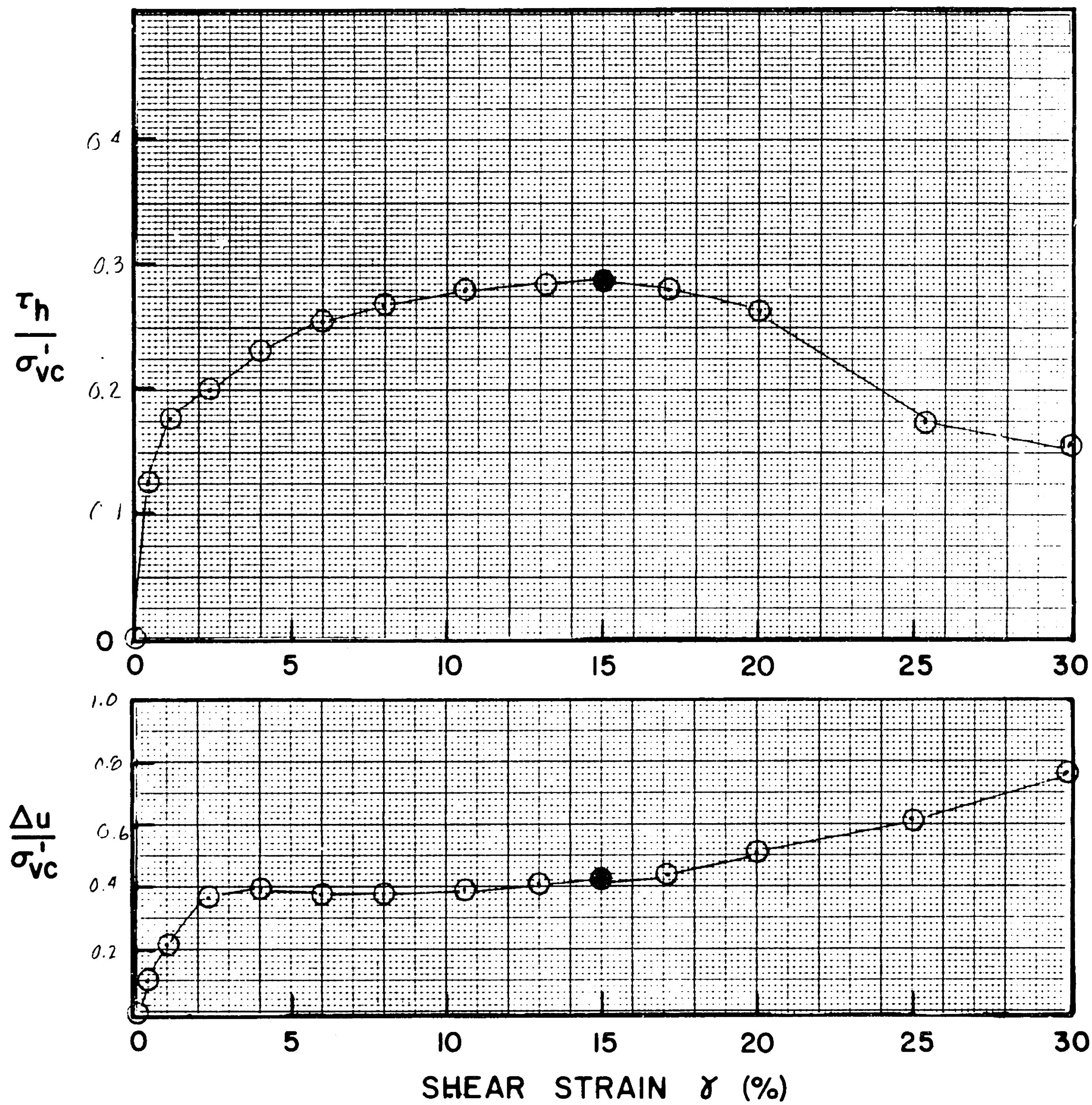
Depth (RE) 8.4' w_L(%) 46.3 σ'_p ( ) ___ OCR 1

Soil Type Arctic SILT w_P(%) 25.2 Estimated σ'_{vo}(KSC) 0.226

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CK_oUDSS TEST NO. TDSS10

FIGURE



Sample No. <u>T18-02</u> w_N(%) <u>43.5</u> σ'_{vc}(KSF) <u>5.0</u> t_c(Days) ____

Depth(RE) <u>7.4 FT</u> w_L(%) <u>52.7</u> σ'_p ( ) ____ OCR <u>1.0</u>

Soil Type <u>Arctic Silt SMITH BAY</u> w_P(%) <u>[illegible]</u> Estimated σ'_{vo}(KSC) <u>0.118</u>

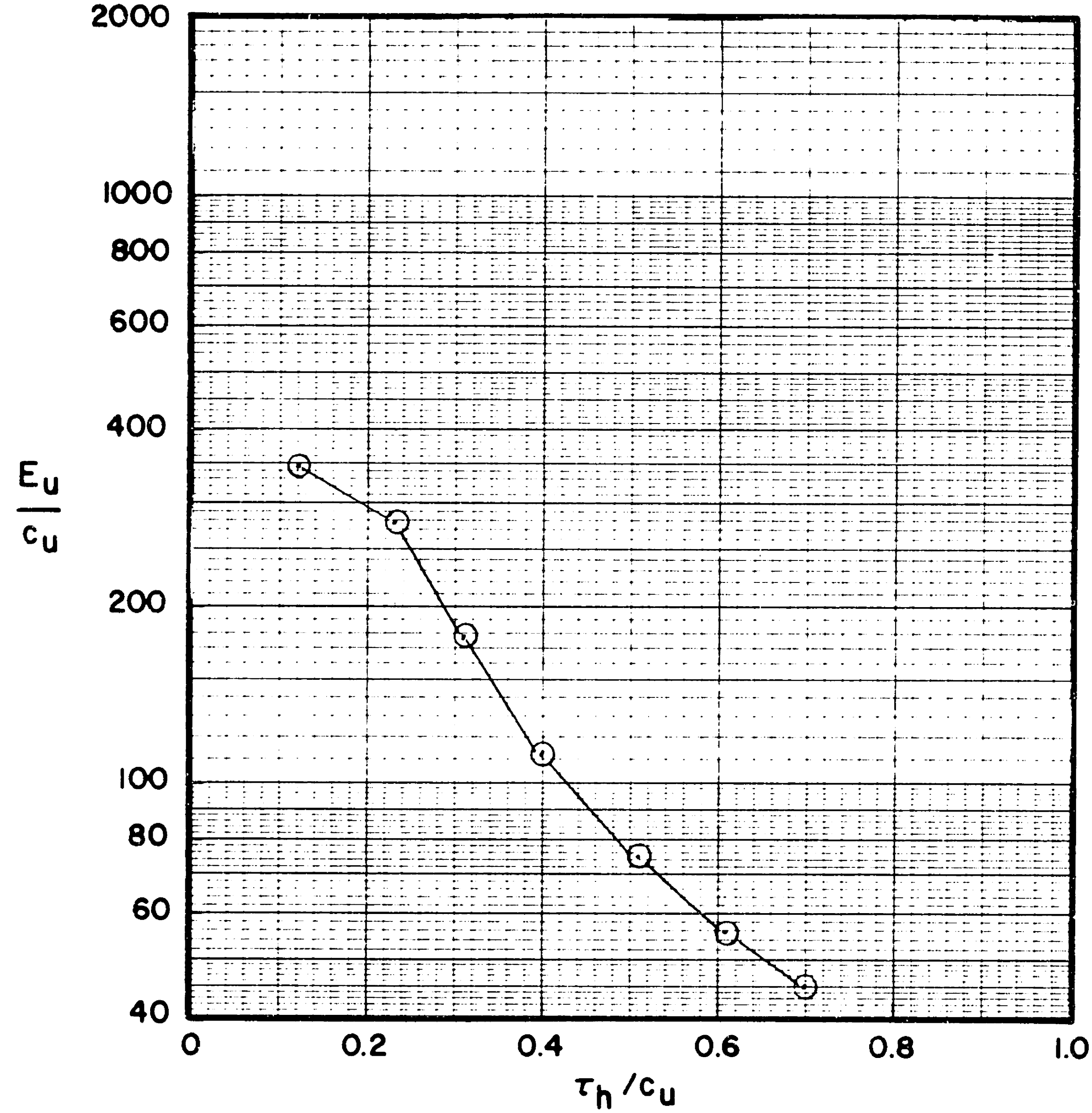
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. <u>TDSS11</u>
(TBIDSS2)

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

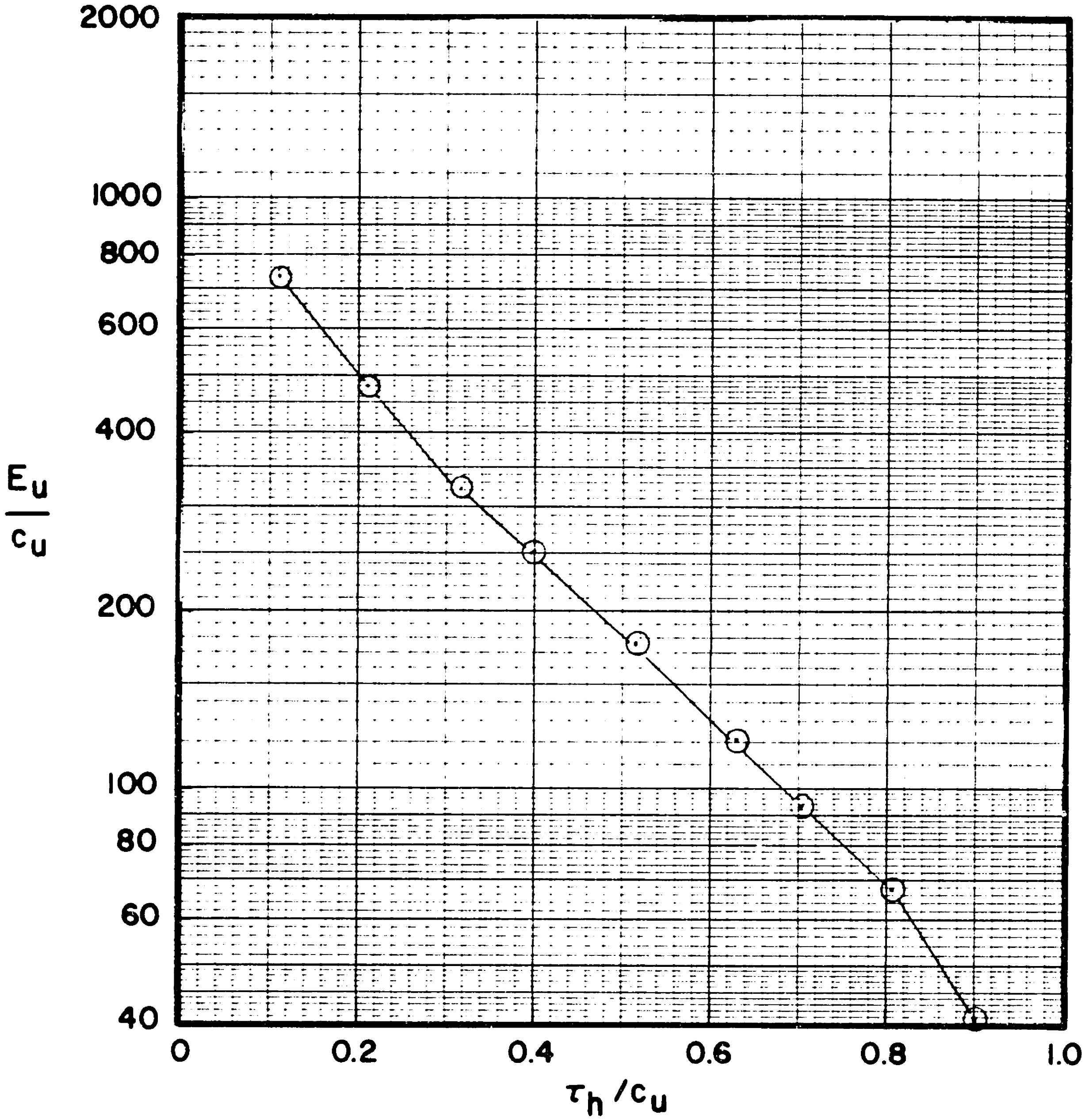


Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (ksc)	OCR	Symbol
DSS-1	TB3-P3	7'9"	37.5 (all) 45.1 (w/o sand)	0.189	4.4	
(Recomp)	(with pins)					

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING TB3-P3 **SOIL TYPE** Arctic Silt

FIGURE



Test No.	Sample No.	Depth	w_N (%)	σ'_{vc} (ksc)	OCR	Symbol
DSS1	TB3P3 (with pins)	7.5'	37.5 (all) 45.1 (w/o sand)	2.301	1	

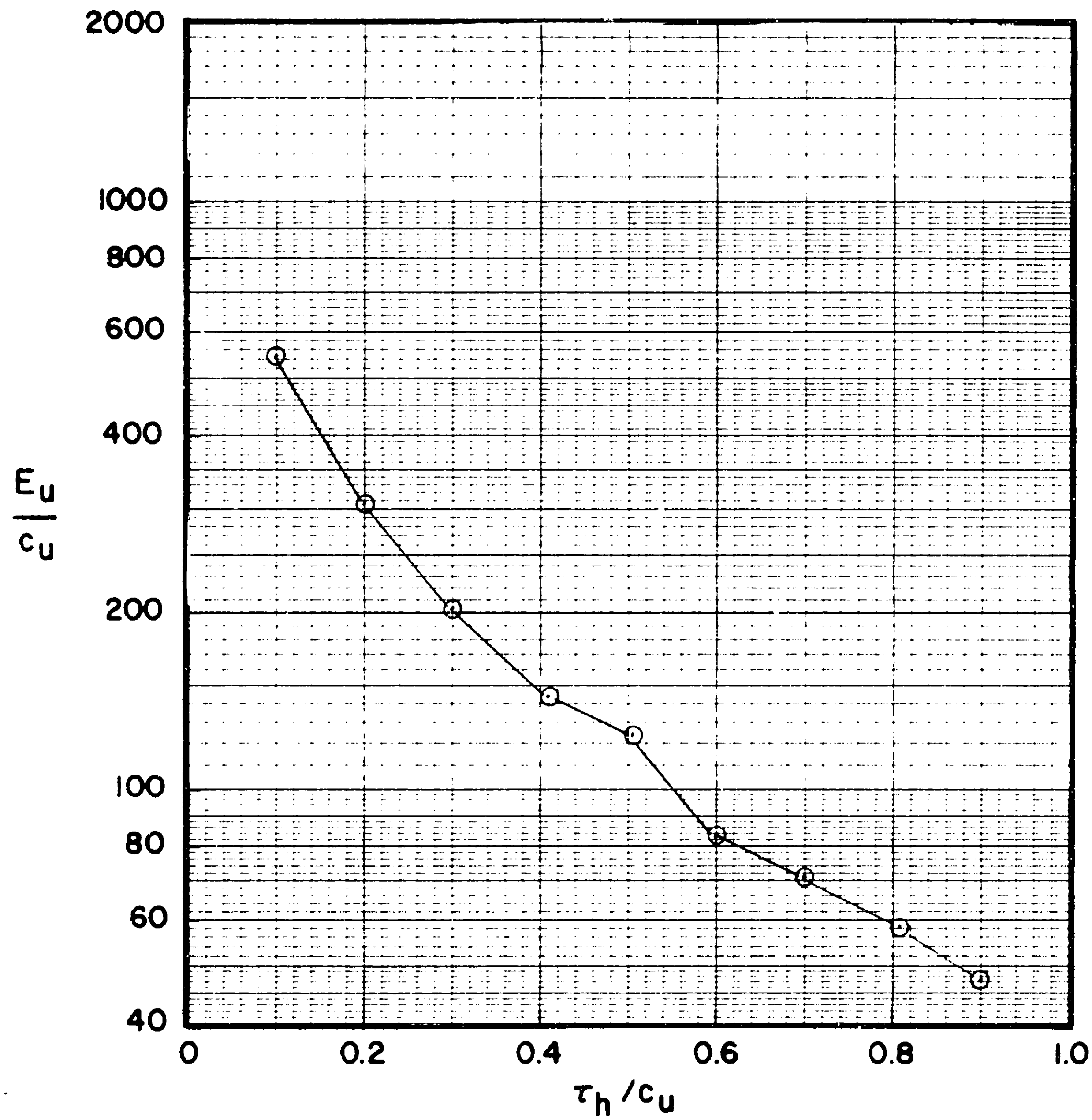
NORMALIZED MODULUS FROM CK$_0$UDSS TESTS

BORING TB3-P3 **SOIL TYPE** Arctic Silt

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

FIGURE

GY 1/10/'96



Test No.	Sample No.	RE Depth	w_N (%)	σ'_{vc} (KSC)	OCR	Symbol
DSS2	TB3-P3	7.7'	40.2	0.308	7.93	

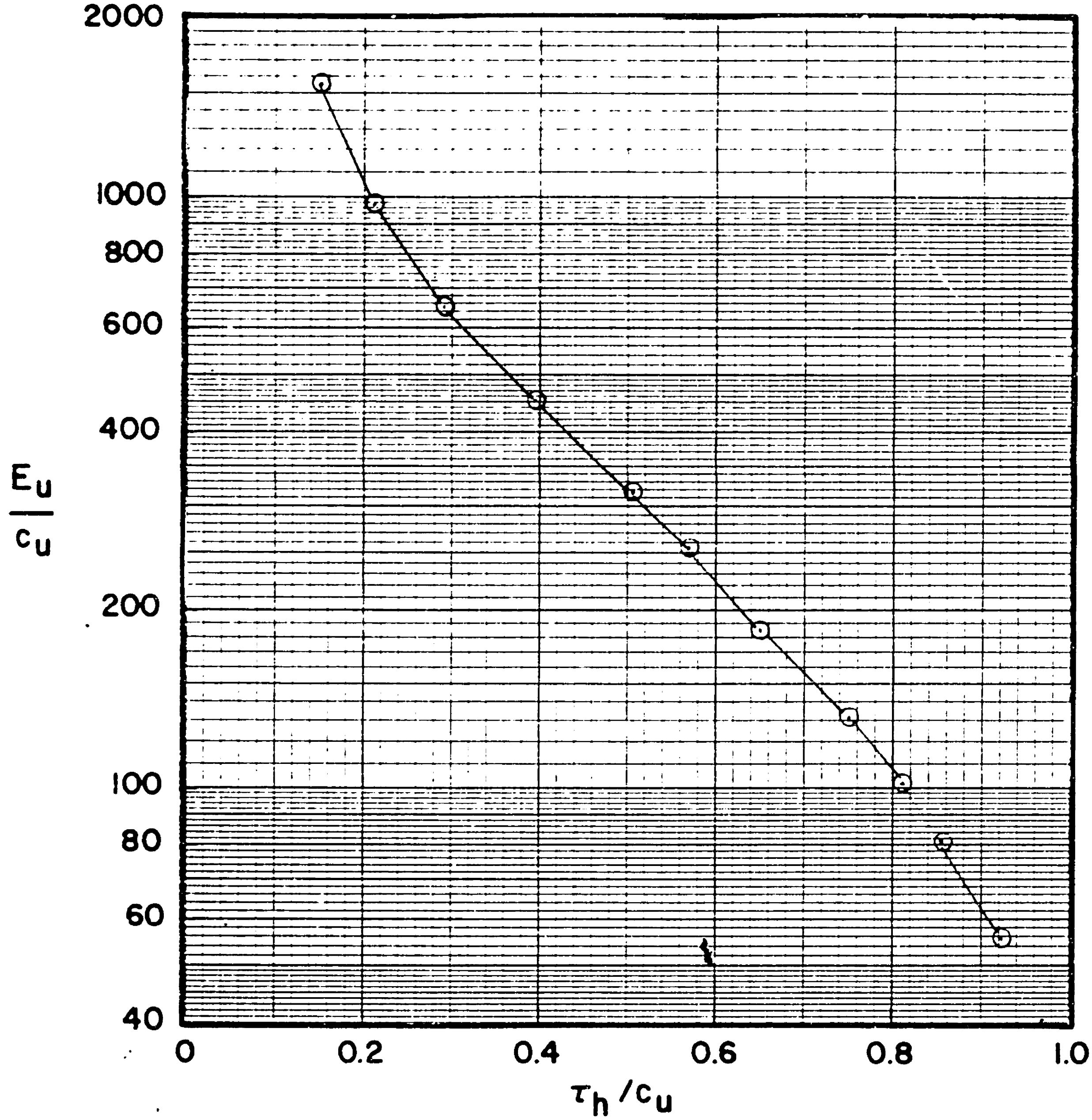
NORMALIZED MODULUS FROM CK₀UDSS TESTS

BORING TB3-P3 SOIL TYPE Arctic Silt

FIGURE

GY 1/7/86

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



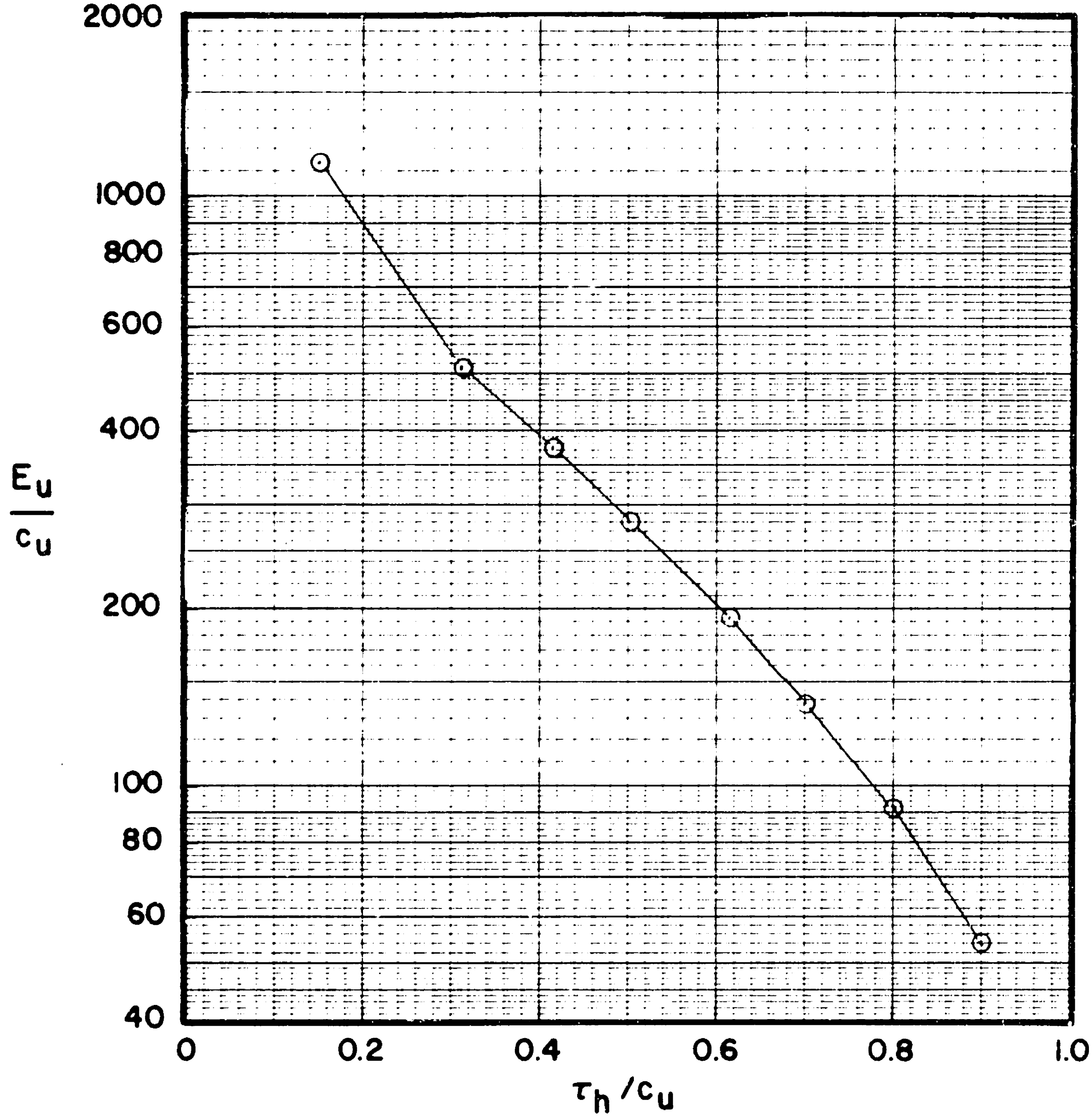
Test No.	Sample No.	Depth RE	w_N (%)	σ'_{vc} (KSC)	OCR	Symbol
DSS3	TB3 P3	7.9'	54.4	2.45	1	

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

NORMALIZED MODULUS FROM CK_0UDSS TESTS

BORING TB3-P3 **SOIL TYPE** Arctic Silt

FIGURE



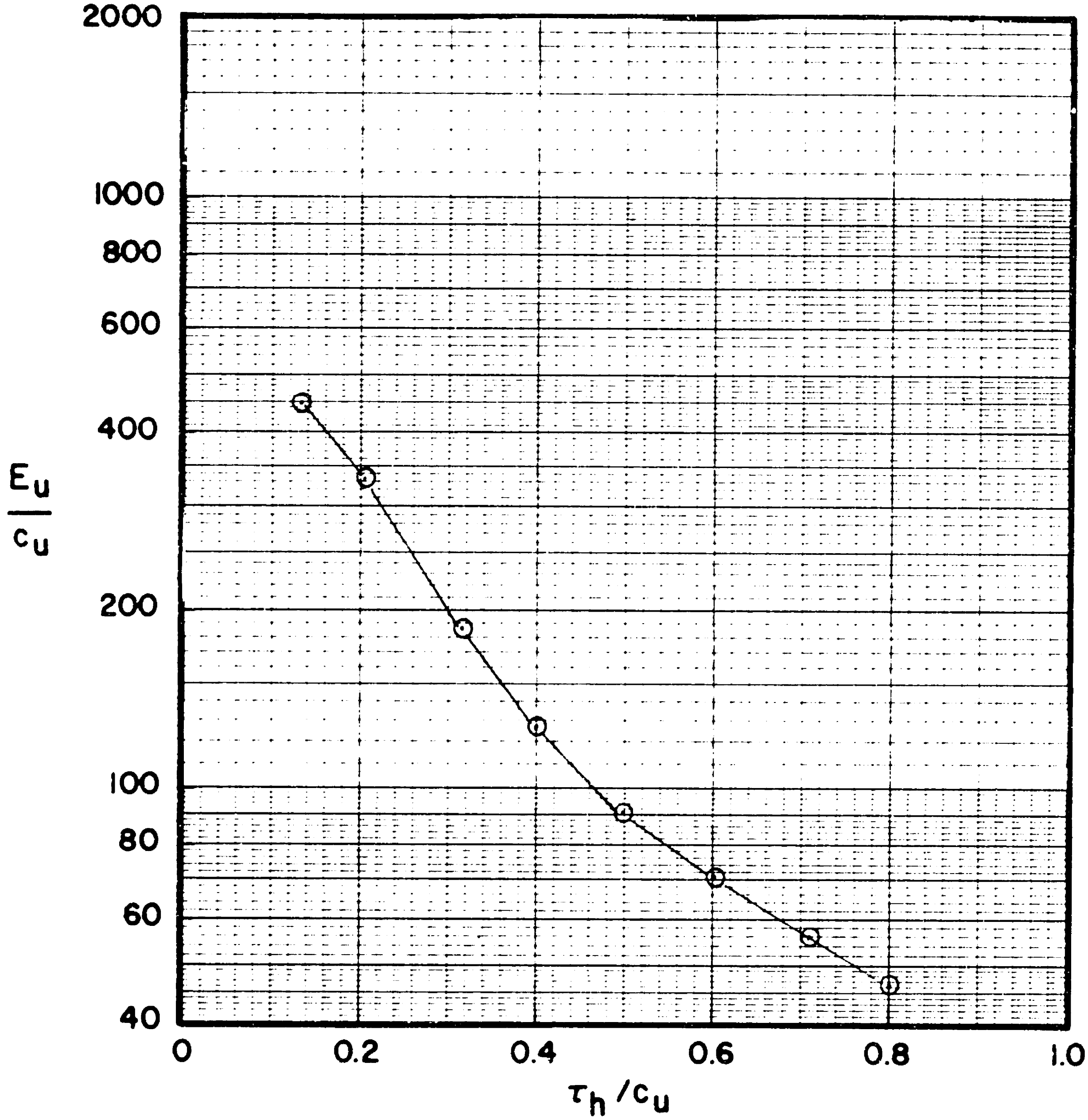
Test No.	Sample No.	Depth (FE)	w_N (%)	σ'_{vc} (KSC)	OCR	Symbol
DSS4	TB3-P2	4.7'	35.3	2.4	1	

NORMALIZED MODULUS FROM CK_0UDSS TESTS

BORING TB3-P2 **SOIL TYPE** Arctic Silt

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



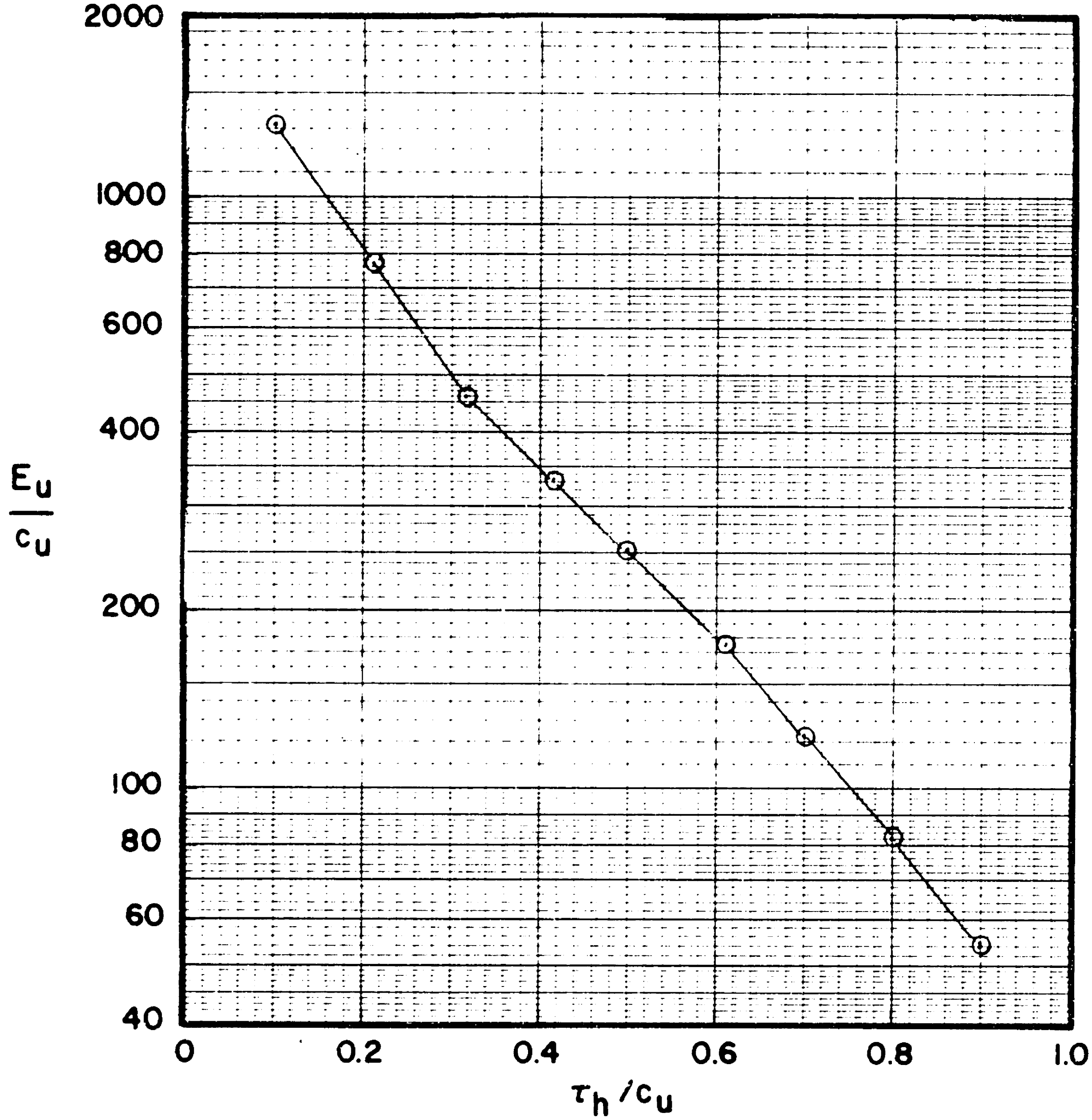
Test No.	Sample No.	Depth (PE)	w_N (%)	σ'_{vc} (KSC)	OCR	Symbol
DSS5	TB3 - P2	4.6'	37.2	0.287	3.5	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING TB3-P2 **SOIL TYPE** [illegible]

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



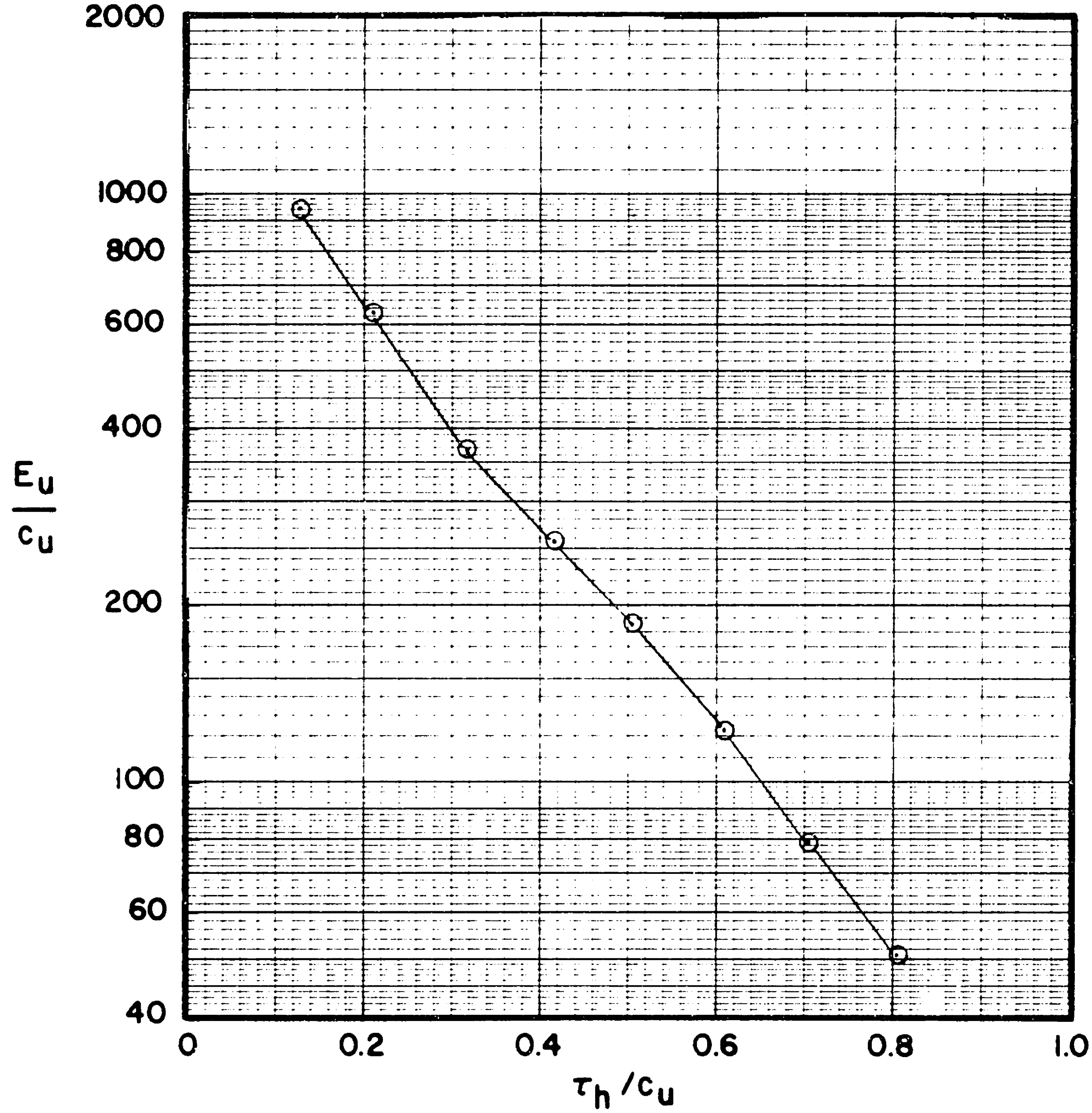
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (ksc)	OCR	Symbol
TB1DSS 6	TB1 02	7'	41.5	2.41	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING TB1 02 SOIL TYPE Arctic Silt

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



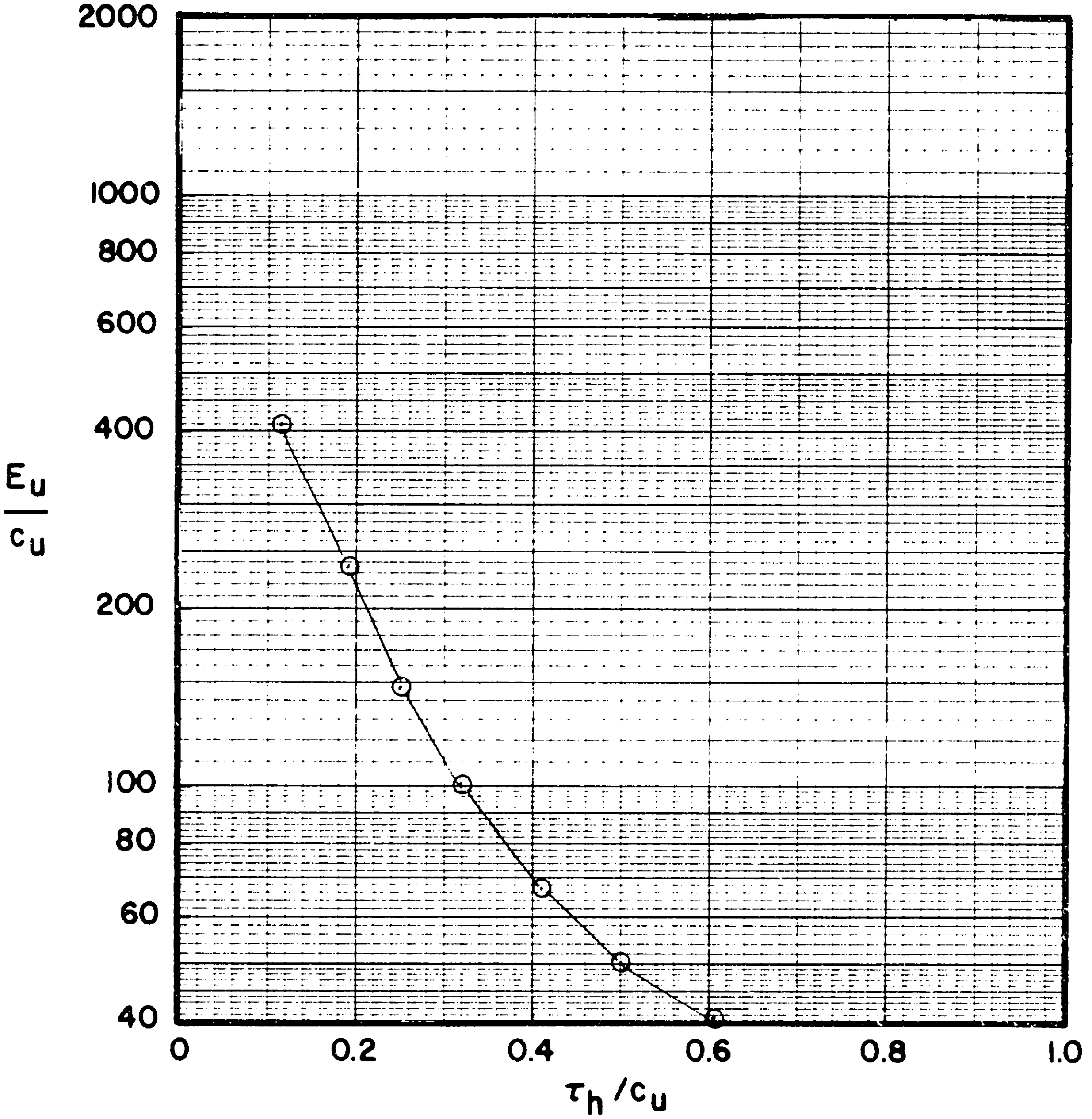
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (Ksc)	OCR	Symbol
T5B1DSS7 w/ pins	T5 B1P3	7'	36.4	2.41	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING T5B1-P3 SOIL TYPE Arctic Silt

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

FIGURE



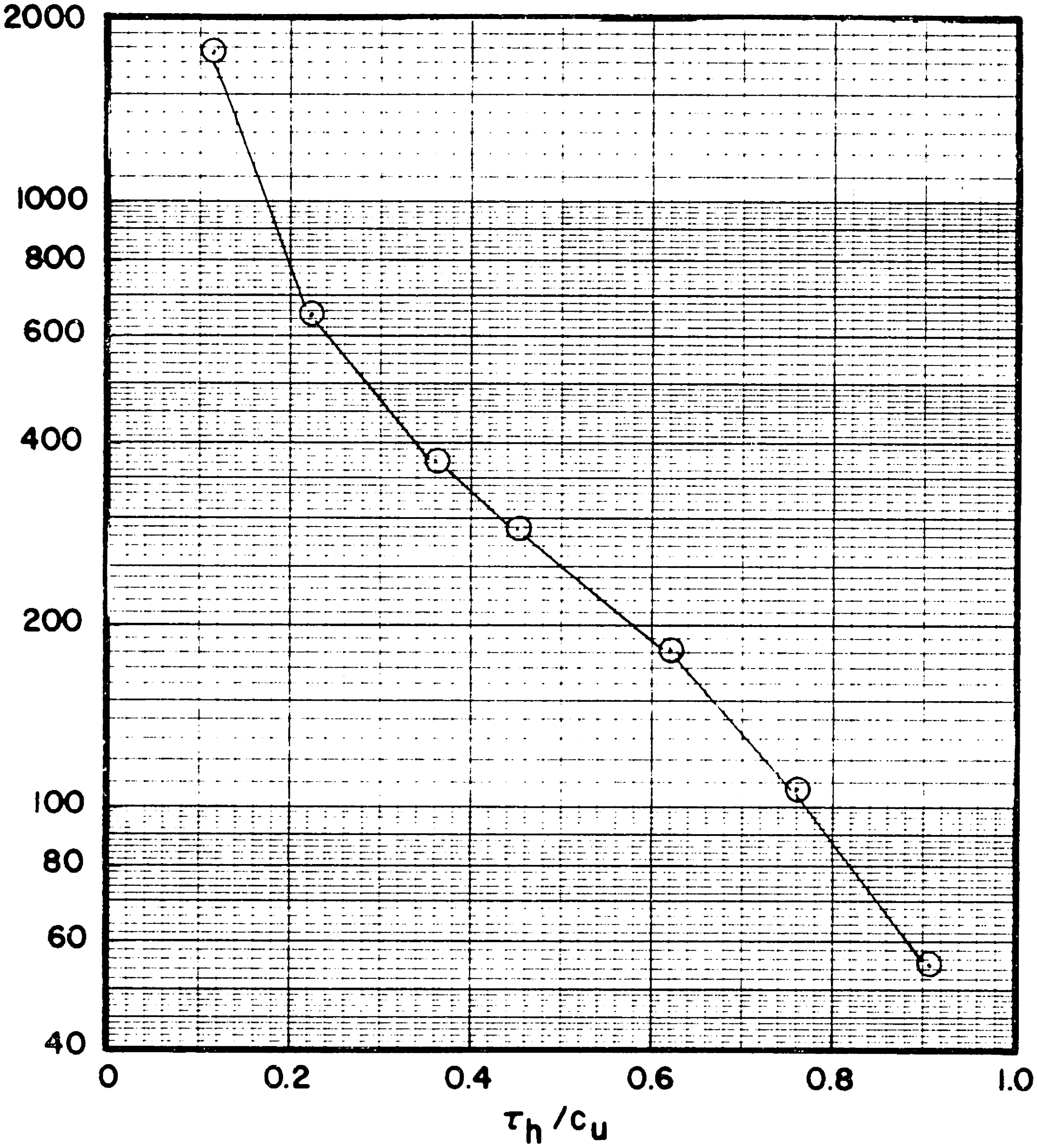
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSC)	OCR	Symbol
T5B1 DSS 8	T5B1-P3	7.1'	38.3	0.153	15.75	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING T5B1-P3 **SOIL TYPE** ARCTIC SILT

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

FIGURE



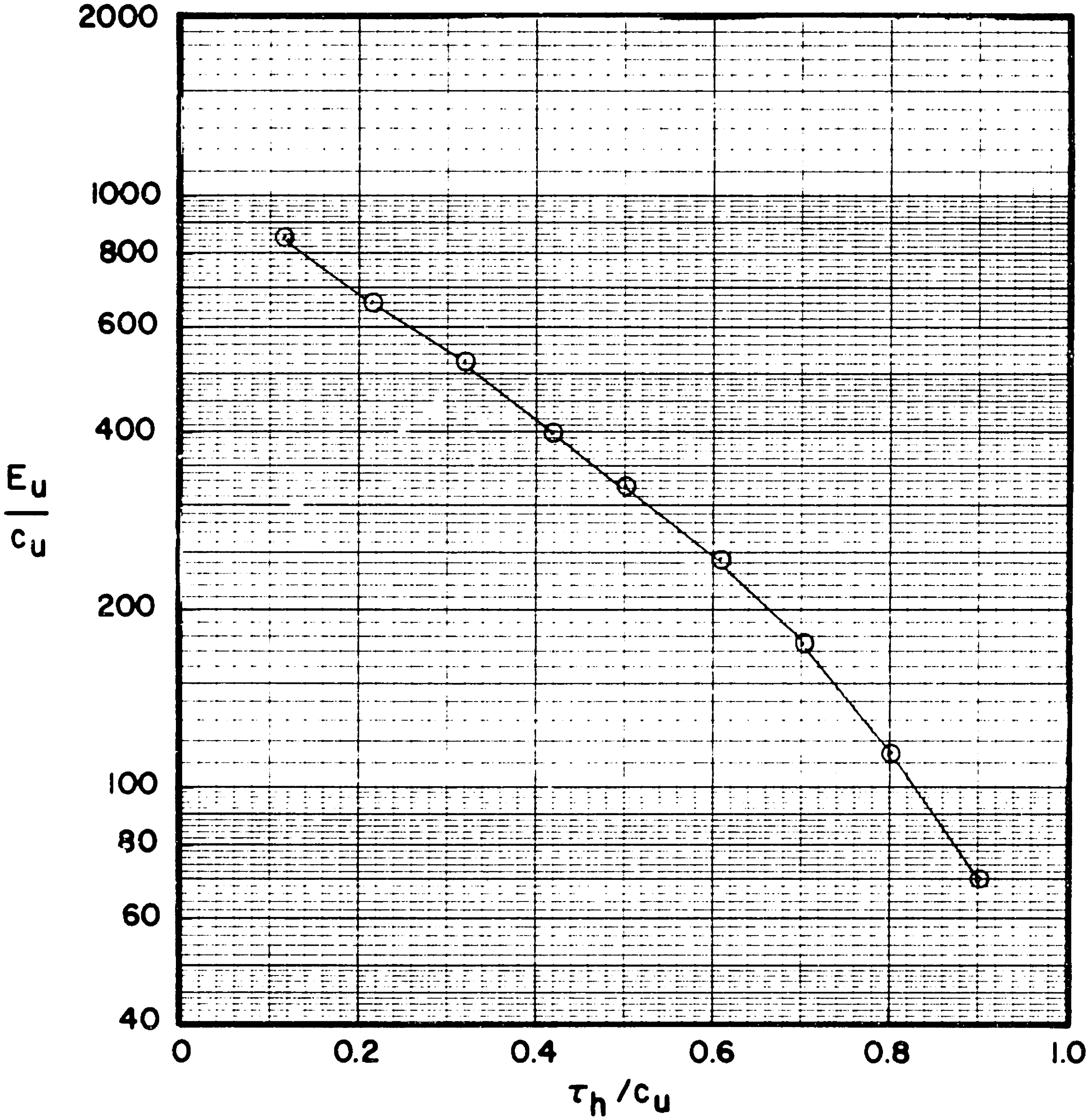
Test No.	Sample No.	Depth (FE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
DSS9	TIB-02 (S11)	7.2'	45.9	5.6	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING TIB-02 SOIL TYPE ARCTIC SILT

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



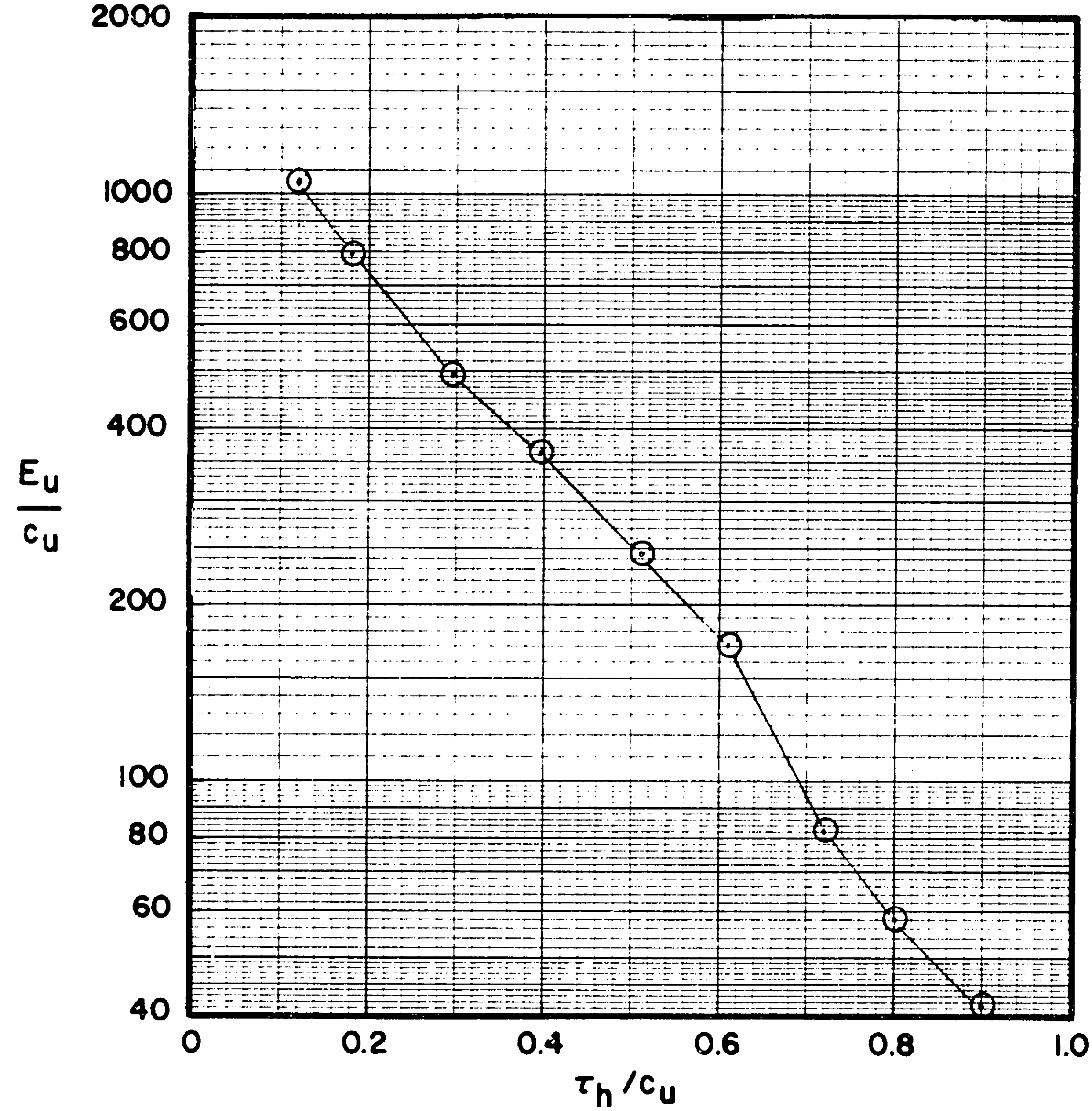
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
TDSS10	T5B1-P3	8.4'	38.4	15.6	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING T5B1 **SOIL TYPE** Arctic Silt

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

FIGURE



Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSC)	OCR	Symbol
TDSS11	T1B-02 (S11)	7.4'	43.5	2.44	1.0	
(0°C)						

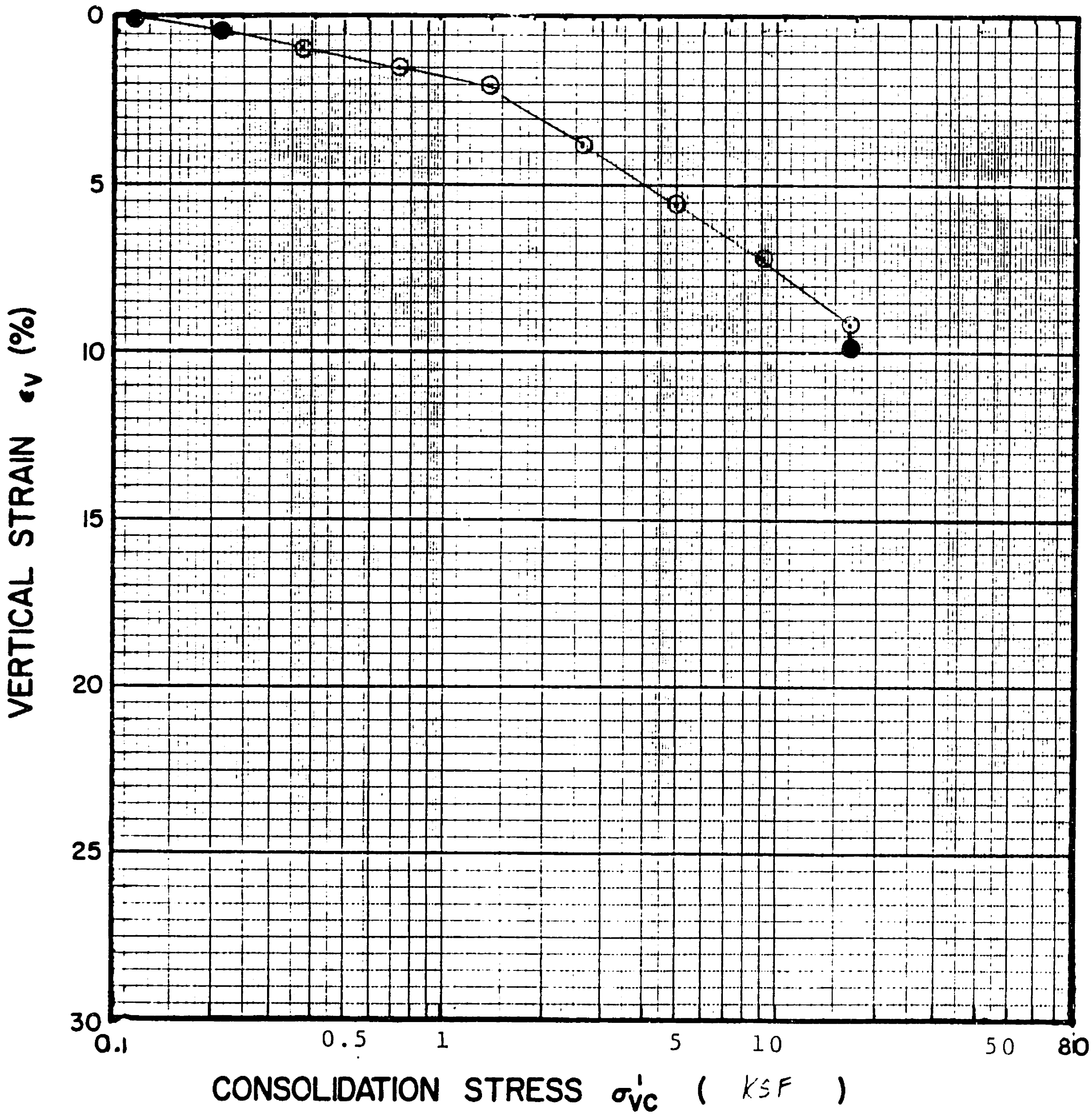
NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING 1B SOIL TYPE ARCTIC SILT

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

FIGURE

SITE W TESTS



Sample No. 5B-r5

Depth (RE) 14.2'

Soil Type Arctic Silt

w_N (%) 29.2

w_L (%) 51.6

w_P (%) 25.2

I_P (%) 26.4

Estimated

σ'_{vo} 0.781 σ'_p 9.3

CR 0.08 RR —

G_s 2.75 e_o 0.817 S(%) 98.3

○ At t_p or hr

● At (t_f) hr

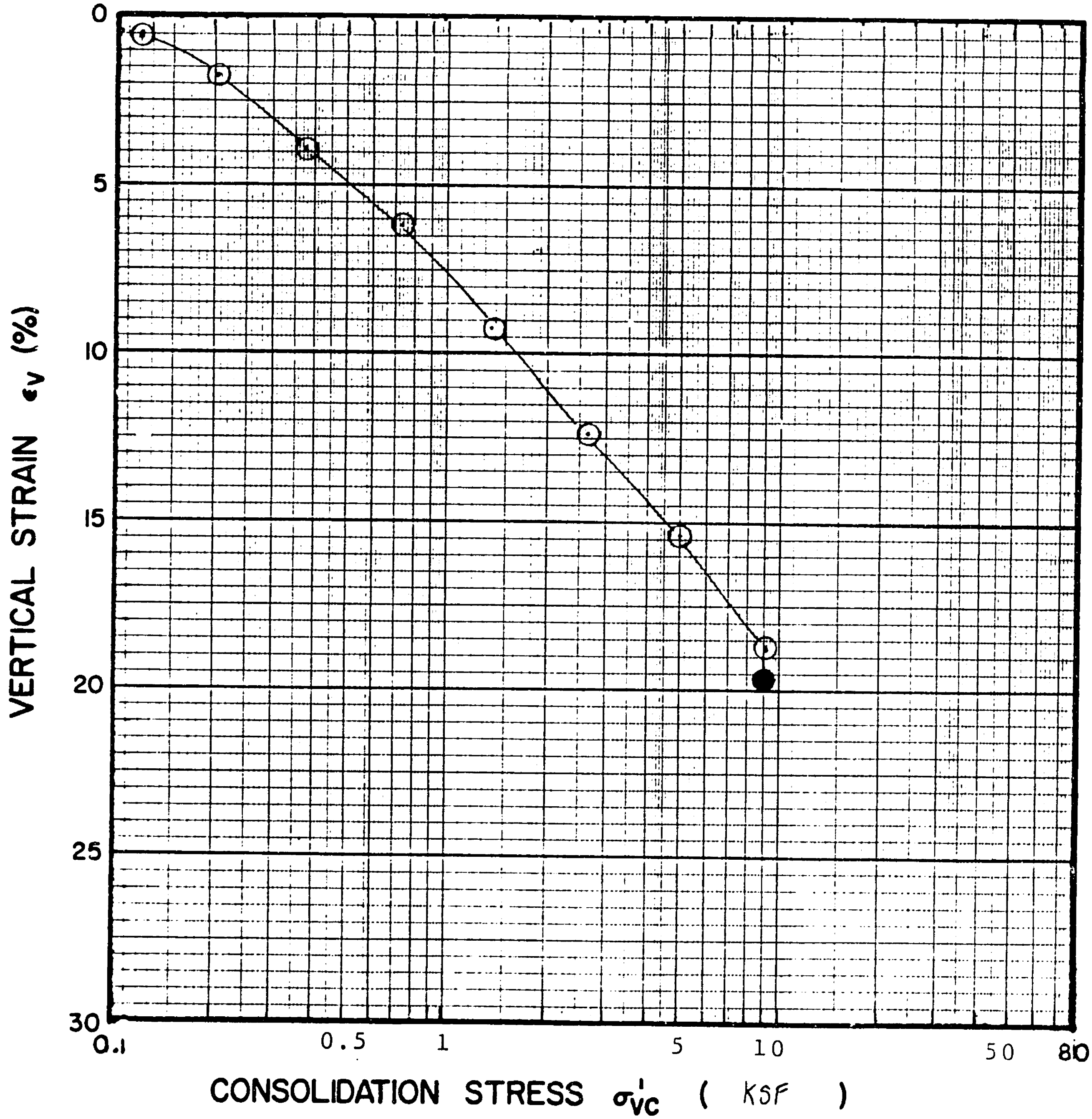
Remarks Corrected for Apparatus Compressibility

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WDSS 1

FIGURE



Sample No. W5PV-51

Depth (f.f.) 3.4'

Soil Type Arctic Silt

w_N (%) 41.7

w_L (%) 48.3

w_P (%) 24.3

I_P (%) 24.0

Estimated

σ'_{vo} 0.187 σ'_p —

CR 0.12 RR —

G_s 2.75 e_0 1.103 S(%) 100

○ At t_p or hr

● At (t_f) hr

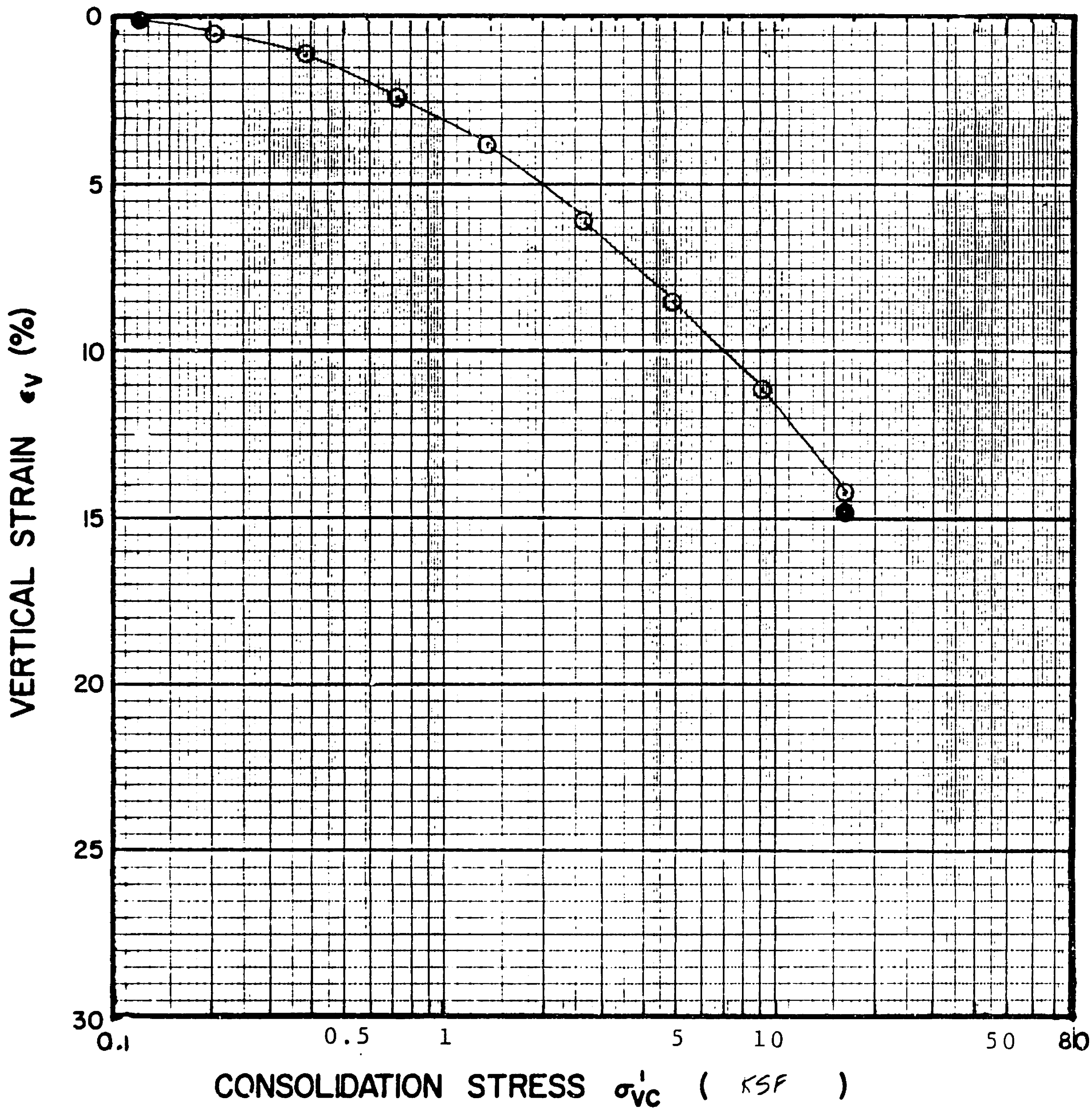
Remarks Corrected for apparatus compressibility.

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WD552

FIGURE



Sample No. W5B-P3

Depth (RE) 7'

Soil Type ARCTIC SILT

w_N (%) 36.6

w_L (%) 53.5

w_P (%) 26.4

I_P (%) 27.1

Estimated

σ'_{vo} 0.385 σ'_p 7.8

CR 0.127 RR –

G_s 2.75 e_0 0.796 S(%) 100

o At t_p or hr

• At (tf) hr

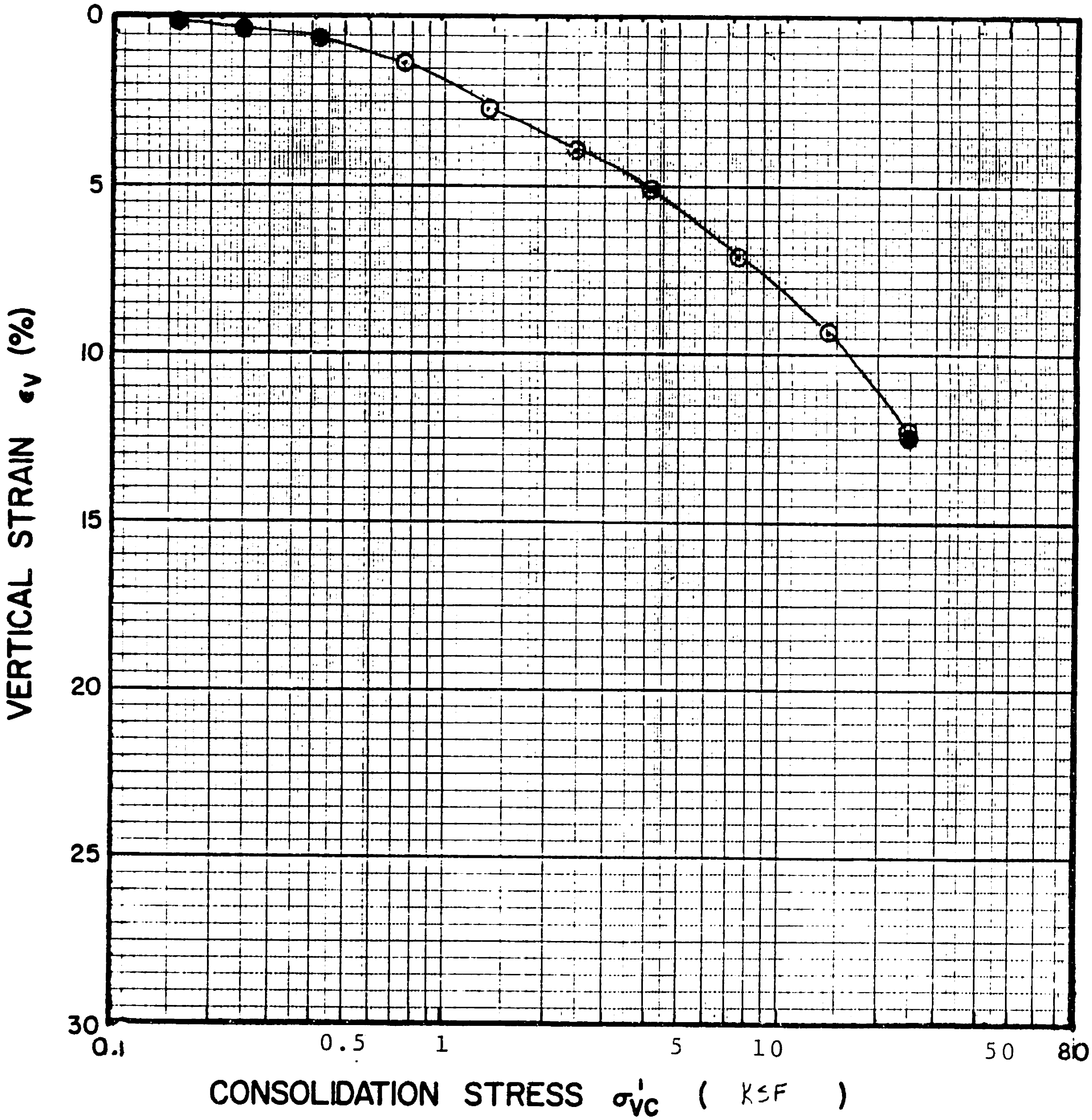
Remarks Corrected for Apparatus Compressibility

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WD553

FIGURE



Sample No. W5B-P4 | w_N (%) 28.9 | Estimated
Depth (PE) 9.3' | w_L (%) 44 | σ'_{vo} 0.506 σ'_p
Soil Type Arctic Silt | w_p (%) 24 | CR .123 RR
I_p (%) 20 | G_s 2.75 | e_0 0.833 S(%) 95.4

○ At t_p or hr
● At ( ) hr

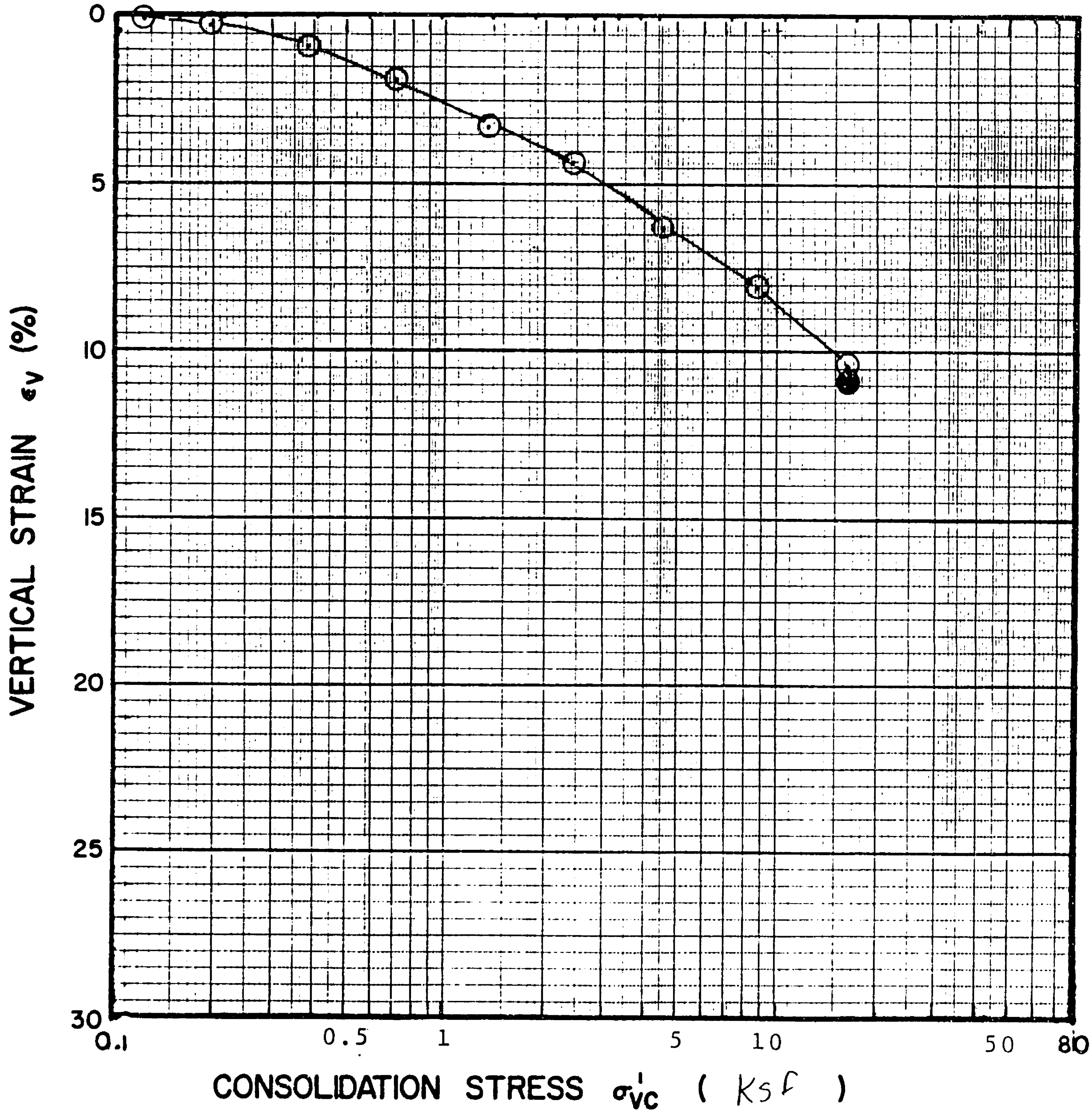
Remarks Corrected for apparatus compression

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WDSS4

FIGURE



Sample No. W5B-P4	w_N (%) [illegible]7.4	Estimated
Depth [illegible] 9.3'	w_L (%) 44	σ'_{vo} 0.512 σ'_p 7.5
Soil Type Arctic Silt	w_P (%) 24	CR 0.085 RR –
	I_P (%) 20	G_s 2.75 e_0 0.83 S(%)

○ At t_p or hr

● At (t_f) hr

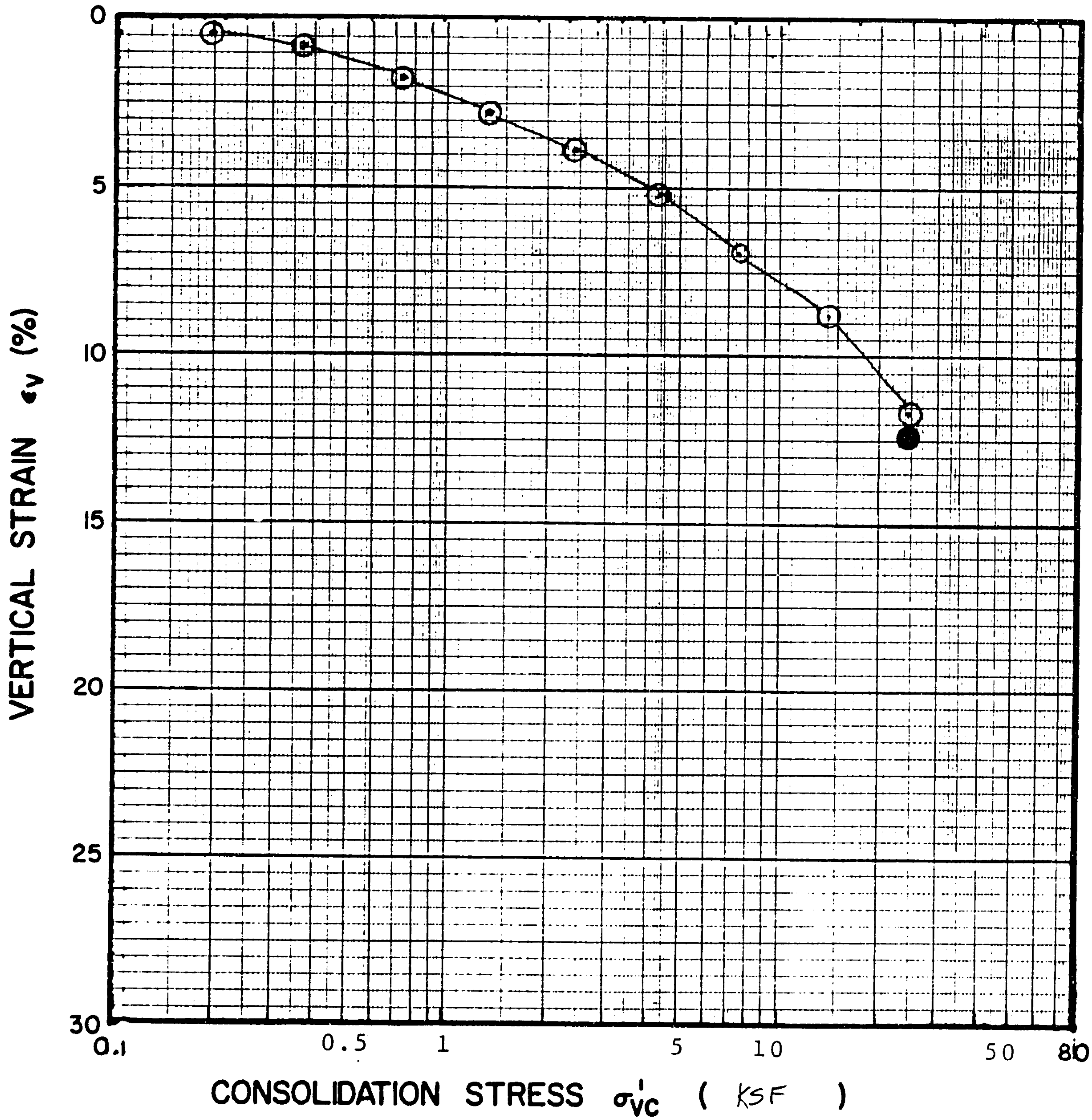
Remarks Corrected for [illegible]

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WD555

FIGURE



Sample No. W5B-P4

Depth (FT) ?.4'

Soil Type ARCTIC SILT

w_N (%) 30.0

w_L (%) 44

w_P (%) 24

I_P (%) 20

Estimated

σ'_{vo} 0.517 ksf σ'_p

CR 0.118 RR

G_s 2.75 e_o 0.92 S(%)

○ At t_p or hr

● At (t_f) hr

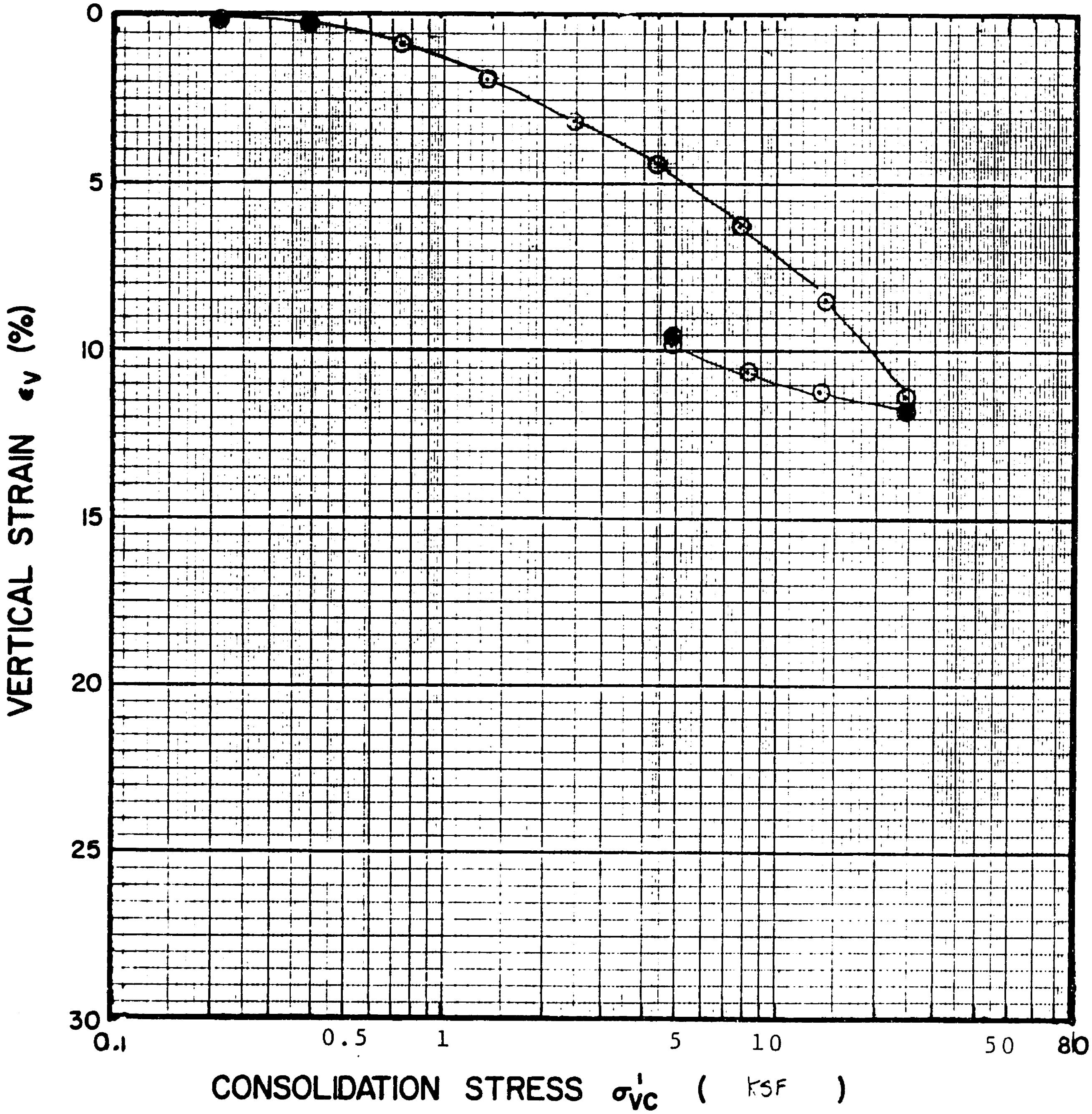
Remarks Corrected for Apparatus Compressibility for at 1°C

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WDSS6

FIGURE



Sample No. W5B-P4 w_N (%) 25.4 **Estimated**

Depth (RE) 9.5' w_L (%) 44.0 σ'_{vo} 0.523 σ'_p

Soil Type Arctic Silt w_p (%) 24.0 CR 0.12 RR —

I_p (%) 20.0 G_s 2.75 e_o S(%)

○ At t_p or hr Remarks Corrected for apparatus compression

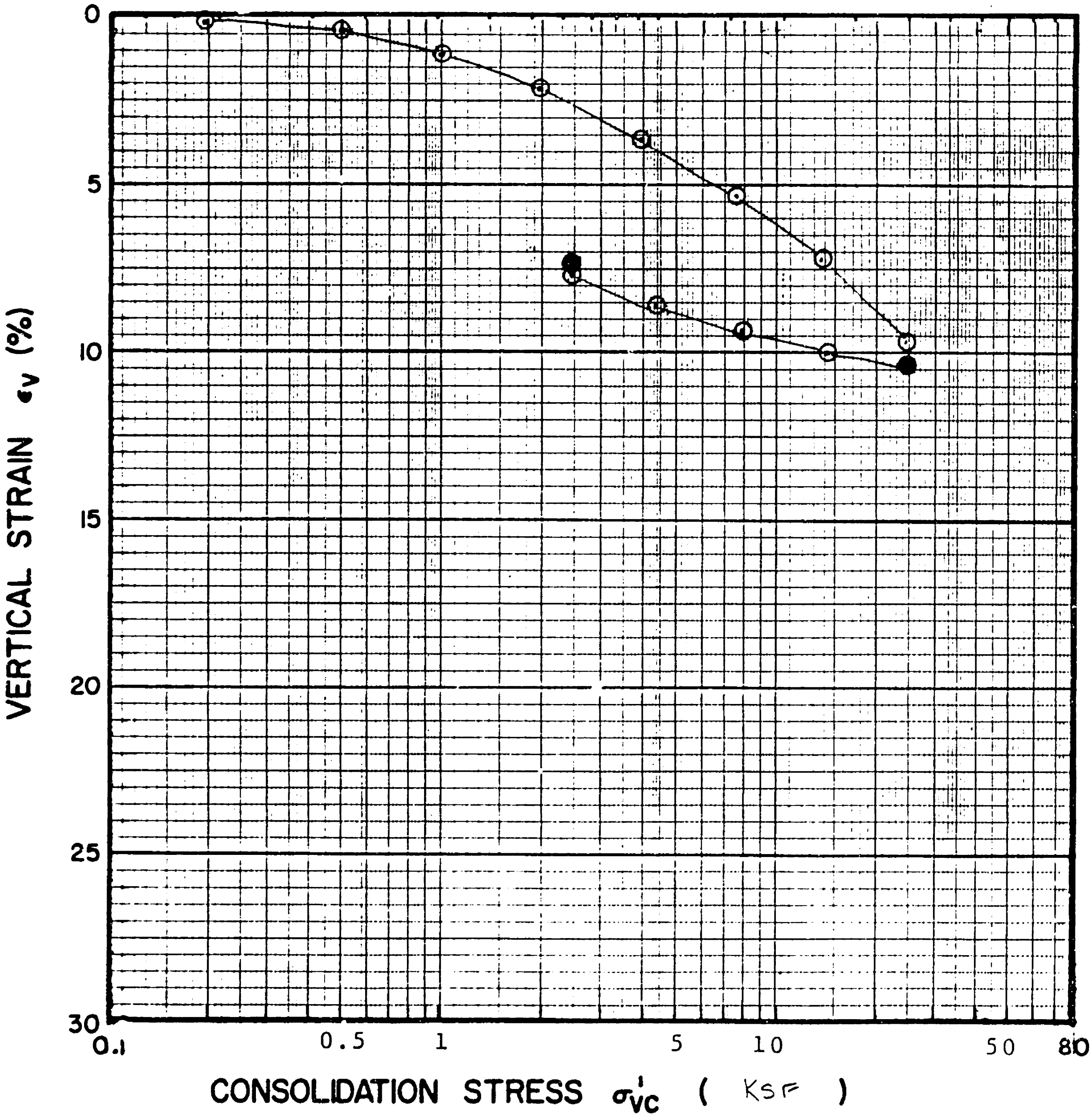
● At (t_f) hr

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WD557

FIGURE



Sample No. WSB-P4

Depth(RE) 9.6'

Soil Type Arctic silt

w_N (%) 29.0

w_L (%) 44

w_P (%) 24

I_P (%) 20

Estimated

σ'_{vo} 0.528 σ'_p –

CR 0.10 RR –

G_s 2.75 e_0 0.81 S(%)

o At t_p or hr

• At (t_f) hr

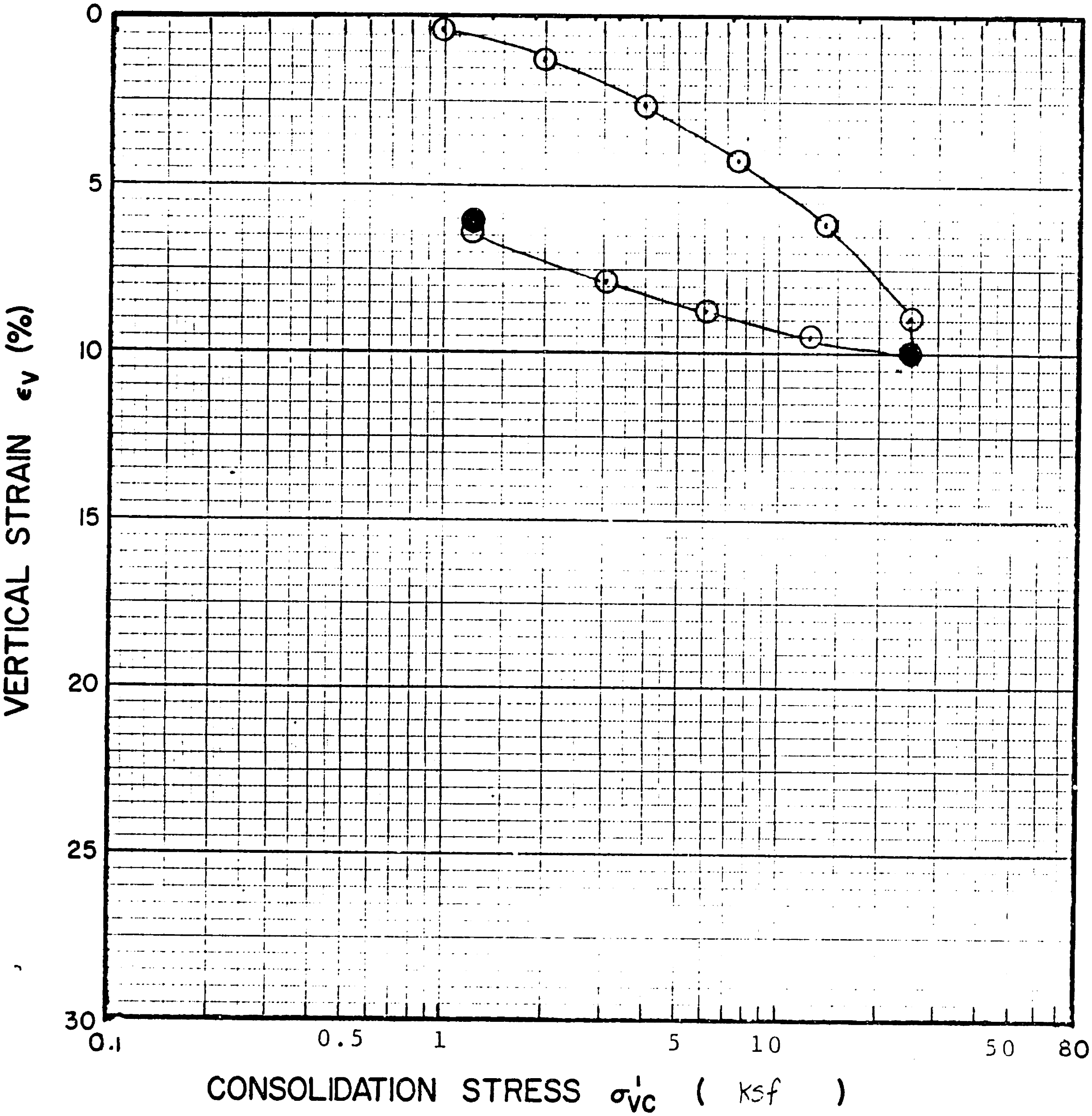
Remarks Corrected for apparatus compressibility

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WDSS 3

FIGURE



Sample No. WSB-P4

Depth (RE) 9.7'

Soil Type ARCTIC SILT

w_N (%) 28.9

w_L (%) 44

w_P (%) 24

I_P (%) 20

Estimated

σ'_{vo} 0.534 σ'_p –

CR 0.11 S~~W~~R .03

G_s 2.75 e_0 0.80 S(%)

○ At t_p or hr

● At (t_f) hr

Remarks Corrected for apparent compressibility

Used stones ...

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

COMPRESSION CURVE

TEST NO. WDSS9

FIGURE

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CK_0UDSS NO. WDSS1 OCR 1

SOIL TYPE Arctic Silt TESTED BY GY DEVICE Geonor DATE 4/4/86

LOCATION Smith Bay Site W

CONSOLIDATION (Stresses in KSC)

σ'_{vc} 7.981 τ_{hc} – σ'_p ______

t_c(Day) 1.0 ε_v(%) 9.9 γ_c(%) ____ t_c(Day) ______

	W, %	e	S, %	H (cm.)
Initial	29.2	0.817		2.266
Preshear	24.6	1.10		2.042
Final	29.0			2.195

DURING SHEAR

Controlled Strain ✓ Stress ______

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.054	0.0411	-0.0008	1.0008	0.1369	0.0411	1.0008	761.60
	0.118	0.0669	-0.0002	1.0002	0.2226	0.0669	1.0002	566.0
	0.208	0.0907	0.0081	0.9919	0.3019	0.0907	0.0081	434.61
	0.518	0.1354	0.0524	0.9476	0.4506	0.1354	0.9476	260.99
	1.042	0.1756	0.0920	0.9080	0.5841	0.1756	0.9080	168.09
	1.513	0.1987	0.1291	0.8709	0.6611	0.1987	0.8709	131.05
	2.037	0.2137	0.1509	0.8491	0.7109	0.2137	0.8491	104.91
	2.541	0.2293	0.1657	0.8343	0.7629	0.2293	0.8343	90.07
	3.000	0.2381	0.1809	0.8191	0.7921	0.2381	0.8191	79.21
	3.514	0.2478	0.1972	0.8028	0.8245	0.2478	0.8028	70.39
	4.038	0.2575	0.2012	0.7988	0.8567	0.2575	0.7988	63.65
	4.510	0.2645	0.2121	0.7879	0.8799	0.2645	0.7879	58.33
	5.053	0.2722	0.2192	0.7808	0.9055	0.2722	0.7808	53.76
	5.541	0.2783	0.2281	0.7719	0.9206	0.2783	0.7719	50.14
	6.040	0.2833	0.2354	0.7646	0.9426	0.2833	0.7646	46.82
	6.558	0.2887	0.2379	0.7621	0.9606	0.2887	0.7621	43.94
	7.070	0.2927	0.2438	0.7562	0.9738	0.2927	0.7562	41.32
	7.463	0.2952	0.2511	0.7489	0.9822	0.2952	0.7489	39.48
	8.005	0.2977	0.2583	0.7417	0.9903	0.2977	0.7417	37.11
	8.827	0.3006	0.2650	0.7350	1.0000	0.3006	0.7350	33.99
	9.699	0.3001	0.2812	0.7188	0.9983	0.3001	0.7188	
	11.151	0.2877	0.3253	0.6747	0.9571	0.2877	0.6747	
	12.477	0.2724	0.3719	0.6281	0.9062	0.2724	0.6281	
	14.116	0.2549	0.4219	0.5785	0.8480	0.2549	0.5785	

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:
CORRECTED FOR STONE AND MEMBRANE DEFLECTIONS

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK₀UDSS NO. WDSS1

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	15.087	0.2462	0.4454	0.5546	0.8191	0.2462	0.5546	
	17.058	0.2286	0.4863	0.5137	0.7606	0.2286	0.4863	
	19.090	0.2107	0.5299	0.4701	0.7009	0.2107	0.4701	
	21.507	0.1894	0.5749	0.4251	0.6302	0.1894	0.4251	
	23.977	0.1664	0.6227	0.3773	0.5536	0.1664	0.3773	
	26.414	0.1463	0.6622	0.3378	0.4867	0.1463	0.3378	
	28.078	0.1321	0.6910	0.3090	0.4395	0.1321	0.3090	
	30.485	0.1143	0.7297	0.2703	0.3802	0.1143	0.2703	

SOIL MECHANICS LABORATORY
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REMARKS: CORRECTED FOR STONE AND MEMBRANE DEFLECTIONS

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK₀UDSS **NO.** WDSS2 **OCR** 1

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 4/16/86

LOCATION Smith Bay
Site W. Boring SB
Sample 5PV-31, RE = 3.4'

CONSOLIDATION (Stresses in KSC**)**

σ'_{vc} 4.43 τ_{hc} — σ'_p ______

t_c(Day) 1.0 ε_v(%) 19.6 γ_c(%) ___ t_c(Day) ______

	W,%	e	S,%	H (cm)
Initial	41.7	1.10		2.146
Preshear	26.3	0.689		1.725
Final	30.98			1.859

DURING SHEAR

Controlled Strain ✓ Stress ______

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	τ_h / σ'_{vc}	$\Delta u / \sigma'_{vc}$	σ'_v / σ'_{vc}	τ_h / S_u	τ_h / σ'_p	σ'_v / σ'_p	E_v/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	[illegible]	
	0.024	0.0225	0.0011	0.9989	0.0933	0.0225	0.9989	1149.10
	0.103	0.0605	0.0021	0.9979	0.2509	0.0605	0.9979	732.40
	0.204	0.0849	0.0045	0.9955	0.3519	0.0849	0.9955	518.21
	0.331	0.1064	0.0081	0.9919	0.4410	0.1064	0.9919	399.57
	0.499	0.1246	0.0186	0.9814	0.5163	0.1246	0.9814	310.17
	1.041	0.1630	0.1013	0.8987	0.6754	0.1630	0.8987	194.70
	1.588	0.1834	0.1704	0.8296	0.7603	0.1834	0.8296	143.66
	2.063	0.1941	0.2054	0.7946	0.8046	0.1941	0.7946	117.03
	2.495	0.2019	0.2265	0.7735	0.8368	0.2019	0.7735	100.61
	3.018	0.2094	0.2459	0.7541	0.8680	0.2094	0.7541	86.[illegible]
	3.410	0.2144	0.2626	0.7374	0.8888	0.2144	0.7374	76.34
	4.015	0.2197	0.2790	0.7210	0.9105	0.2197	0.7210	68.02
	4.485	0.2227	0.2956	0.7044	0.9228	0.2227	0.7044	61.73
	5.052	0.2261	0.3130	0.6870	0.9369	0.2261	0.6870	55.64
	5.604	0.2292	0.3209	0.6791	0.9498	0.2292	0.6791	50.84
	6.021	0.2303	0.3396	0.6604	0.9546	0.2303	0.6604	47.56
	6.763	0.2325	0.3545	0.6455	0.9636	0.2325	0.6455	42.75
	7.506	0.2346	0.3627	0.6373	0.9722	0.2346	0.6373	38.86
	8.415	0.2372	0.3741	0.6259	0.9830	0.2372	0.6259	35.05
	9.405	0.2395	0.3817	0.6183	0.9926	0.2395	0.6183	31.66
	10.413	0.2404	0.3935	0.6065	0.9963	0.2404	0.6065	28.70
	11.173	0.2413	0.3997	0.6003	1.0000	0.2413	0.6003	26.85
	12.530	0.2405	0.4182	0.5818	0.9969	0.2405	0.5818	
	13.302	0.2403	0.4293	0.5707	0.9960	0.2403	0.5707	

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REMARKS: Corrected for stone and membrane deflections

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio **SOIL** Arctic Silt **TYPE OF TEST** CK₀UDSS **NO.** WDSS2

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	14.337	0.2388	0.4413	0.5587	0.9897	0.2388	0.5587	
	15.326	0.2364	0.4557	0.5443	0.9796	0.2364	0.5443	
	16.046	0.2345	0.4627	0.5373	0.9721	0.2345	0.5373	
	17.615	0.2306	0.4780	0.5220	0.9558	0.2306	0.5220	
	19.708	0.2251	0.5056	0.4944	0.9329	0.2251	0.4944	
	20.864	0.2218	0.5134	0.4866	0.9195	0.2218	0.4866	
	22.270	0.2158	0.5341	0.4659	0.8946	0.2158	0.4659	
	24.772	0.2053	0.5632	0.4368	0.8511	0.2053	0.4368	
	26.452	0.1978	0.5847	0.4153	0.8197	0.1978	0.4153	
	28.380	0.1879	0.6109	0.3891	0.7788	0.1879	0.3891	
	29.782	0.1800	0.6315	0.3685	0.7463	0.1800	0.3685	
	32.448	0.1662	0.6602	0.3398	0.6890	0.1662	0.3398	
	34.105	0.1554	0.6847	0.3153	0.6442	0.1554	0.3153	

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REMARKS: Corrected for Stone and Membrane Deflections

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CK₀UDSS NO. WDSS3 OCR 1

SOIL TYPE Arctic Silt TESTED BY GY DEVICE Geonor DATE 4/21/86

LOCATION Smith Bay
Site W - Boring 5B
Sample P3 · RE = 7 FT.

CONSOLIDATION (Stresses in KSC)

σ'_{vc} 8.0 τ_{hc} – σ'_p

t_c(Day) 1.0 ε_v(%) 14.8 γ_c(%) t_c(Day)

	W,%	e	S,%	H (cm)
Initial	36.6	1.00		2.244
Preshear	27.5	0.70		1.911
Final	32.5			2.069

DURING SHEAR

Controlled Strain ✓ Stress

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.052	0.0554	0.0056	0.9944	0.2347	0.0554	0.9944	1344.07
	0.105	0.0711	0.0109	0.9891	0.3015	0.0711	0.9891	863.39
	0.220	0.0927	0.0218	0.9782	0.3927	0.0927	0.9782	535.80
	0.505	0.1233	0.0582	0.9418	0.5225	0.1233	0.9418	310.45
	1.029	0.1531	0.1130	0.8870	0.6488	0.1531	0.8870	189.21
	1.517	0.1710	0.1542	0.8458	0.7249	0.1710	0.8458	143.38
	2.038	0.1838	0.1820	0.8180	0.7791	0.1838	0.8180	114.65
	2.537	0.1934	0.2035	0.7965	0.8194	0.1934	0.7965	96.82
	3.161	0.2030	0.2272	0.7728	0.8604	0.2030	0.7728	81.65
	3.564	0.2080	0.2403	0.7597	0.8813	0.2080	0.7597	74.19
	4.418	0.2088	0.2637	0.7363	0.8849	0.2088	0.7363	60.08
	5.057	0.2188	0.2753	0.7247	0.9274	0.2188	0.7247	55.01
	5.435	0.2217	0.2811	0.7189	0.9395	0.2217	0.7189	51.86
	6.145	0.2266	0.2953	0.7047	0.9604	0.2266	0.7047	46.89
	6.631	0.2292	0.3036	0.6964	0.9712	0.2292	0.6964	43.94
	7.083	0.2311	0.3111	0.6889	0.9796	0.2311	0.6889	41.49
	7.611	0.2332	0.3224	0.6776	0.9883	0.2332	0.6776	38.95
	8.751	0.2353	0.3443	0.6557	0.9974	0.2353	0.6557	34.19
	9.486	0.2360	0.3613	0.6387	1.0000	0.2360	0.6387	31.62
	10.712	0.2359	0.3855	0.6145	0.9997	0.2359	0.6145	
	11.896	0.2341	0.4048	0.5952	0.9923	0.2341	0.5952	
	13.658	0.2305	0.4339	0.5661	0.9767	0.2305	0.5661	
	15.704	0.2258	0.4598	0.5402	0.9570	0.2258	0.5402	
	16.649	0.2231	0.4716	0.5284	0.9456	0.2231	0.5284	

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REMARKS: Corrected for stone and membrane [illegible]

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CKoUDSS NO. WDSS3

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	17.263	0.2209	0.4814	0.5186	0.9361	0.2209	0.5186	
	18.556	0.2166	0.4941	0.5059	0.9181	0.2166	0.5059	
	20.678	0.2061	0.5277	0.4723	0.8735	0.2061	0.4723	
	21.210	0.2028	0.5373	0.4627	0.8596	0.2028	0.4627	
	23.186	0.1897	0.5707	0.4293	0.8041	0.1897	0.4293	
	24.424	0.1821	0.5902	0.4098	0.7717	0.1821	0.4098	
	25.511	0.1753	0.6081	0.3919	0.7428	0.1753	0.3919	
	26.632	0.1691	0.6246	0.3754	0.7166	0.1691	0.3754	
	27.552	0.1629	0.6383	0.3617	0.6903	0.1629	0.3617	
	28.610	0.1571	0.6505	0.3495	0.6660	0.1571	0.3495	
	30.539	0.1448	0.6796	0.3204	0.6138	0.1448	0.3204	

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REMARKS: Corrected for stone and membrane deflections

DIRECT - SIMPLE SHEAR TEST

PROJECT Soho **TYPE OF TEST** CK0UDSS **NO.** NDSS4 **OCR** 1

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 5/9/86

LOCATION Smith Bay
Site W - Boring SB-P4
RE = 9.2'

CONSOLIDATION (Stresses in KSC)

σ'_{vc} 12.0 τ_{hc} — σ'_p

t_c(Day) 1.0 ε_v(%) 12.4 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	28.9	0.8331	95.4	2.223
Preshear	24.4	0.6047		1.946
Final	29.0	0.7330	100	2.102

DURING SHEAR
Controlled Strain ✓ Stress

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	τ_h / σ'_{vc}	$\Delta u / \sigma'_{vc}$	σ'_v / σ'_{vc}	τ_h / S_u	τ_h / σ'_p	σ'_v / σ'_p	E_u / S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.127	0.0533	-0.0001	1.0001	0.2151	0.0533	1.0001	508.39
	0.215	0.0681	0.0069	0.9931	0.2748	0.0681	0.9931	333.56
	0.500	0.1002	0.0418	0.9582	0.4040	0.1002	0.9582	242.45
	1.031	0.1342	0.0903	0.9097	0.5414	0.1342	0.9097	153.10
	1.549	0.1528	0.1224	0.8776	0.6161	0.1528	0.8776	119.30
	2.090	0.1678	0.1562	0.8438	0.6766	0.1678	0.8438	97.11
	2.555	0.1780	0.1760	0.8240	0.7180	0.1780	0.8240	84.31
	3.034	0.1872	0.1948	0.8052	0.7549	0.1872	0.8052	74.65
	4.038	0.2019	0.2230	0.7770	0.8141	0.2019	0.7770	60.49
	5.055	0.2139	0.2436	0.7564	0.8629	0.2139	0.7564	51.20
	5.989	0.2219	0.2662	0.7338	0.8947	0.2219	0.7338	44.82
	6.950	0.2279	0.2797	0.7203	0.9192	0.2279	0.7203	39.68
	8.065	0.2338	0.2936	0.7064	0.9430	0.2338	0.7064	35.08
	9.059	0.2380	0.3055	0.6945	0.9598	0.2380	0.6945	31.78
	10.049	0.2419	0.3154	0.6846	0.9758	0.2419	0.6846	29.13
	11.060	0.2452	0.3221	0.6779	0.9887	0.2452	0.6779	26.82
	12.084	0.2468	0.3331	0.6669	0.9952	0.2468	0.6669	24.71
	13.277	0.2479	0.3441	0.6559	1.0000	0.2479	0.6559	22.59
	14.031	0.2471	0.3521	0.6479	0.9965	0.2471	0.6479	
	15.133	0.2427	0.3740	0.6260	0.9788	0.2427	0.6260	
	16.569	0.2343	0.4085	0.5915	0.9450	0.2343	0.5915	
	17.794	0.2266	0.4374	0.5626	0.9138	0.2266	0.5626	
	19.420	0.2152	0.4759	0.5241	0.8680	0.2152	0.5241	
	21.050	0.2042	0.5126	0.4874	0.8234	0.2042	0.4874	

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REMARKS: Corrected for stone and membrane deflections

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK₀UDSS NO. WDSS4

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	22.248	0.1969	0.5344	0.4656	0.7943	0.1969	0.4656	
	23.045	0.1925	0.5485	0.4515	0.7763	0.1925	0.4515	
	24.262	0.1851	0.5714	0.4286	0.7465	0.1851	0.4286	
	25.068	0.1803	0.5839	0.4161	0.7273	0.1803	0.4161	
	26.261	0.1737	0.6033	0.3967	0.7007	0.1737	0.3967	
	27.062	0.1690	0.6151	0.3850	0.6814	0.1690	0.3850	
	28.284	0.1614	0.6352	0.3648	0.6508	0.1614	0.3648	
	29.922	0.1493	0.6602	0.3398	0.6021	0.1493	0.3398	

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REMARKS: CORRECTED FOR STONE AND MEMBRANE DEFLECTIONS

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK_0UDSS **NO.** WCSS2 **OCR** 1

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 5/22/86

LOCATION Smith Bay
Site W - Boring SB-P4
RE - 9.3'

CONSOLIDATION (Stresses in KSC**)**

σ'_{vc} 8.0 τ_{hc} — σ'_p ____

t_c(Day) 1.0 ε_v(%) 10.8 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	29.4			2.350
Preshear	25.76			2.096
Final	30.0			2.245

DURING SHEAR

Controlled Strain ✓ Stress ____

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.044	0.0395	-0.0009	1.0009	0.1436	0.0395	1.0009	972.87
	0.113	0.0622	0.0020	0.9980	0.2261	0.0622	0.9980	602.16
	0.242	0.0894	0.0081	0.9919	0.3252	0.0894	0.9919	402.67
	0.429	0.1196	0.0179	0.9821	0.4349	0.1196	0.9821	266.63
	1.002	0.1535	0.0743	0.9257	0.5584	0.1535	0.9257	167.24
	1.468	0.1763	0.1237	0.8763	0.6414	0.1763	0.8763	131.09
	2.003	0.1938	0.1472	0.8528	0.7050	0.1938	0.8528	105.58
	2.450	0.2048	0.1837	0.8163	0.7450	0.2048	0.8163	91.22
	3.046	0.2168	0.2017	0.7983	0.7886	0.2017	0.7983	77.68
	3.514	0.2251	0.2148	0.7852	0.8187	0.2251	0.7852	69.89
	4.017	0.2321	0.2344	0.7656	0.8440	0.2321	0.7656	63.03
	4.543	0.2338	0.2381	0.7619	0.8502	0.2338	0.7619	56.14
	5.034	0.2424	0.2469	0.7531	0.8816	0.2424	0.7531	52.53
	5.555	0.2471	0.2616	0.7384	0.8987	0.2471	0.7384	48.54
	6.063	0.2523	0.2672	0.7328	0.9176	0.2523	0.7328	45.40
	6.596	0.2564	0.2717	0.7283	0.9325	0.2564	0.7283	42.41
	7.103	0.2610	0.2721	0.7279	0.9492	0.2610	0.7279	40.09
	8.031	0.2661	0.2986	0.7014	0.9679	0.2661	0.7014	36.16
	9.078	0.2741	0.2985	0.7015	0.9970	0.2741	0.7015	32.95
	9.645	0.2750	0.3002	0.6998	1.0000	0.2750	0.6998	31.11
	10.624	0.2739	0.3192	0.6808	0.9961	0.2739	0.6808	
	11.475	0.2723	0.3239	0.6761	0.9903	0.2723	0.6761	
	12.498	0.2679	0.3408	0.6592	0.9744	0.2679	0.6592	
	14.124	0.2569	0.3713	0.6287	0.9344	0.2569	0.6287	

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REMARKS: Corrected for stone and membrane deflections

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK_0UDSS NO. WDSS5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	15.015	0.2506	0.3885	0.6115	0.9116	0.2506	0.6115	
	16.960	0.2266	0.4592	0.5408	0.8241	0.2266	0.5408	
	18.856	0.2074	0.5015	0.4985	0.7544	0.2074	0.4985	
	20.731	0.1920	0.5389	0.4611	0.6984	0.1920	0.4611	
	22.603	0.1769	0.5807	0.4193	0.6433	0.1769	0.4193	
	24.460	0.1622	0.6127	0.3873	0.5899	0.1622	0.3873	
	26.710	0.1456	0.6535	0.3465	0.5294	0.1456	0.3465	
	28.209	0.1335	0.6804	0.3196	0.4857	0.1335	0.3196	
	30.039	0.1206	0.7098	0.2902	0.4387	0.1206	0.2902	
	31.925	0.1106	0.7335	0.2665	0.4024	0.1106	0.2665	

REMARKS: CORRECTED FOR STONE AND MEMBRANE DEFLECTIONS

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DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CK₀UDSS NO. WDSS6 OCR 1

SOIL TYPE Arctic OI TESTED BY GI DEVICE Geonor DATE 6/5/86

LOCATION Smith Bay
Site 11 - Boring SB-P4
G.E = 9.4'

CONSOLIDATION (Stresses in KSC)

σ'_{vc} 12.0 τ_{hc} — σ'_p

t_c(Day) 1.0 ε_v(%) 12.4 γ_c(%) — t_c(Day) —

	W, %	e	S, %	H (cm)
Initial	30.0	0.920		2.341
Preshear	26.8	0.683		2.0518
Final	30.5			2.177

DURING SHEAR

Controlled Strain ✓ Stress

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.011	0.0110	-0.0008	1.0008	0.0455	0.0110	1.0008	1247.75
	0.069	0.0214	0.0073	0.9927	0.1293	0.0214	0.9927	566.25
	0.128	0.0469	0.0144	0.9856	0.1936	0.0469	0.9856	452.94
	0.246	0.0681	0.0265	0.9735	0.2810	0.0681	0.9735	342.27
	0.513	0.0985	0.0588	0.9412	0.4064	0.0985	0.9412	237.60
	1.031	0.1331	0.1444	0.8556	0.5491	0.1331	0.8556	159.30
	1.489	0.1513	0.1662	0.8338	0.6240	0.1513	0.8338	125.72
	2.021	0.1661	0.1810	0.8190	0.6849	0.1661	0.8190	101.68
	2.563	0.1767	0.2509	0.7491	0.7288	0.1767	0.7491	85.29
	3.042	0.1847	0.2620	0.7380	0.7618	0.1847	0.7380	75.12
	3.757	0.1957	0.2669	0.7331	0.8074	0.1957	0.7331	64.48
	4.492	0.2049	0.2831	0.7169	0.8450	0.2049	0.7169	56.44
	5.503	0.2141	0.3030	0.6970	0.8827	0.2141	0.6970	48.13
	6.692	0.2204	0.3314	0.6686	0.9091	0.2204	0.6686	40.75
	7.486	0.2253	0.3365	0.6635	0.9292	0.2253	0.6635	37.24
	8.573	0.2301	0.3453	0.6547	0.9489	0.2301	0.6547	33.21
	9.535	0.2341	0.3529	0.6471	0.9655	0.2341	0.6471	30.32
	10.486	0.2374	0.3554	0.6446	0.9792	0.2374	0.6446	28.02
	11.467	0.2406	0.3603	0.6397	0.9922	0.2406	0.6397	25.96
	12.874	0.2424	0.3717	0.6283	1.0000	0.2424	0.6283	23.30
	14.020	0.2397	0.3845	0.6155	0.9887	0.2397	0.6155	21.16
	15.246	0.2279	0.4278	0.5722	0.9400	0.2279	0.5722	
	16.501	0.2127	0.4723	0.5277	0.8772	0.2127	0.5277	
	17.726	0.1993	0.5148	0.4852	0.8222	0.1993	0.4852	

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REMARKS: Corrected for stone and membrane deflections

— Run at 0°C

DIRECT - SIMPLE SHEAR (continued)

PROJECT SOHIO SOIL ARCTIC SILT TYPE OF TEST [illegible] NO. [illegible]

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	18.534	0.1911	0.5425	0.4575	0.7881	0.1911	0.4575	
	19.368	0.1829	0.5583	0.4417	0.7546	0.1829	0.4417	
	20.520	0.1751	0.5679	0.4321	0.7221	0.1751	0.4321	
	21.698	0.1661	0.5809	0.4191	0.6849	0.1661	0.4191	
	22.460	0.1609	0.5879	0.4121	0.6636	0.1609	0.4121	
	23.642	0.1531	0.5978	0.4022	0.6316	0.1531	0.4022	
	24.768	0.1468	0.6064	0.3936	0.6053	0.1468	0.3936	
	25.549	0.1425	0.6148	0.3852	0.5879	0.1425	0.3852	
	27.269	0.1203	0.7027	0.2973	[illegible]	0.1203	[illegible]	

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REMARKS: CORRECTED FOR STONE AND MEMBRANE DEFLECTIONS

- RUN AT 0°C

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK_oUDSS **NO.** WDSS7 **OCR** 5

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 6/12/86

LOCATION Smith Bay
[illegible]
[illegible]

CONSOLIDATION (Stresses in KSC**)**

σ'_{vc} 2.373 τ_{hc} — σ'_p 12

t_c(Day) 1.0 ε_v(%) 9.6 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	29.4	0.85		2.263
Preshear	26.3	0.67		2.0456
Final	30.1			2.177

DURING SHEAR

Controlled Strain ✓ Stress ____

Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	τ_h/σ'_{vc}	$\Delta u/\sigma'_{vc}$	σ'_v/σ'_{vc}	τ_h/S_u	τ_h/σ'_p	σ'_v/σ'_p	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	1.0000	
	0.052	0.0707	-0.0940	1.0940	0.0953	0.0141	0.2168	550.47
	0.123	0.1301	-0.0904	1.0904	0.1753	0.0260	0.2181	427.70
	0.244	0.1820	-0.1045	1.1045	0.2452	0.0364	0.2209	301.87
	0.486	0.2434	-0.1434	1.1434	0.3281	0.0487	0.2287	[illegible]
	1.069	0.3410	-0.2453	1.2453	0.4596	0.0682	0.2491	128.92
	1.495	0.3946	-0.3076	1.3076	0.5318	0.0789	0.2615	106.70
	2.053	0.4533	-0.3761	1.3761	0.6109	0.0907	0.2752	89.26
	2.517	0.4933	-0.4241	1.4241	0.6648	0.0987	0.2848	79.25
	3.120	0.5356	-0.4669	1.4669	0.7218	0.1071	0.2934	69.41
	3.489	0.5576	-0.5016	1.5016	0.7514	0.1115	0.3003	64.[illegible]
	4.005	0.5810	-0.5502	1.5502	0.7830	0.1162	0.3100	58.65
	4.499	0.6108	-0.5927	1.5927	0.8231	0.1222	0.3185	54.89
	5.006	0.6346	-0.6311	1.6311	0.8552	0.1269	0.3262	51.25
	5.517	0.6558	-0.6575	1.6575	0.8838	0.1312	0.3315	48.06
	6.033	0.6758	-0.6914	1.6914	0.9107	0.1352	0.3383	45.29
	6.557	0.6941	-0.7277	1.7277	0.9354	0.1388	0.3455	42.80
	7.084	0.7109	-0.7603	1.7603	0.9580	0.1422	0.3521	40.57
	7.479	0.7215	-0.7780	1.7780	0.9723	0.1443	0.3556	39.00
	8.017	0.7343	-0.8041	1.8041	0.9895	0.1469	0.3608	37.03
	8.572	0.7411	-0.8065	1.8065	0.9988	0.1482	0.3613	34.95
	8.712	0.7420	-0.8053	1.8053	1.0000	0.1484	0.3611	34.44
	9.418	0.7413	-0.8167	1.8167	0.9991	0.1483	0.3633	
	10.141	0.7348	-0.8043	1.8043	0.8903	0.1470	0.3609	
	10.594	0.7329	-0.7826	1.7826	0.9756	0.1448	0.3565	

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REMARKS: Corrected for stone and membrane deflections

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK_0UDSS NO. WDS37

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	11.474	0.7112	-0.7666	1.7666	0.9584	0.1422	0.3533	
	12.526	0.6975	-0.7261	1.7261	0.9400	0.1395	0.3452	
	13.438	0.6872	-0.7058	1.7058	0.9262	0.1374	0.3412	
	14.532	0.6775	-0.6775	1.6775	0.9130	0.1355	0.3355	
	15.642	0.6636	-0.6491	1.6491	0.8944	0.1327	0.3298	
	16.752	0.6520	-0.6230	1.6230	0.8787	0.1304	0.3246	
	17.490	0.6418	-0.5968	1.5968	0.8650	0.1284	0.3194	
	18.600	0.6233	-0.5632	1.5632	0.8399	0.1247	0.3126	
	19.724	0.6037	-0.5266	1.5266	0.8136	0.1207	0.3053	
	20.845	0.5839	-0.4905	1.4905	0.7870	0.1168	0.2981	
	21.591	0.5704	-0.4668	1.4668	0.7687	0.1141	0.2934	
	22.706	0.5503	-0.4313	1.4313	0.7416	0.1101	0.2863	
	23.841	0.5291	-0.3947	1.3947	0.7130	0.1058	0.2789	
	24.586	0.5148	-0.3698	1.3698	0.6937	0.1030	0.2740	
	25.706	0.4946	-0.3323	1.3323	0.6666	0.0989	0.2665	
	26.449	0.4809	-0.3069	1.3069	0.6480	0.0952	0.2614	
	27.569	0.4615	-0.2691	1.2691	0.6219	0.0923	0.2538	
	28.689	0.4430	-0.2307	1.2307	0.5970	0.0886	0.2461	
	29.800	0.4243	-0.1911	1.1911	0.5716	0.0849	0.2382	
	30.182	0.4179	-0.1794	1.1794	0.5631	0.0836	0.2359	

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REMARKS: Corrected for stone and membrane deflections

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio **TYPE OF TEST** CK_0UDSS **NO.** WDSS8 **OCR** 10

SOIL TYPE Arctic Silt **TESTED BY** GY **DEVICE** Geonor **DATE** 7/8/86

LOCATION Smith Bay
Site W- Boring SB-P4
RE = 9.6'

CONSOLIDATION (Stresses in KSC**)**

σ'_{vc} 1.189 τ_{hc} — σ'_p 12.0

t_c(Day) 1.0 ε_v(%) ___ γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	29.0	0.810		2.407
Preshear	26.8	0.677		2.228
Final	30.7			2.366

DURING SHEAR
Controlled Strain ✓ Stress ___
Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{\sigma'_v}$	$\frac{\tau_h}{S_u}$	$\frac{\sigma'_v}{\sigma'_p}$	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	0.0991	8115.40
	0.134	0.1706	-0.0201	1.0201	0.1673	0.1647	0.1011	367.75
	0.283	0.2364	-0.0535	1.0535	0.2243	0.2282	0.1044	242.08
	0.554	0.3123	-0.1045	1.1045	0.2828	0.3015	0.1095	163.38
	0.759	0.3607	-0.1606	1.1606	0.3108	0.3482	0.1150	137.67
	1.075	0.4188	-0.2289	1.2289	0.3408	0.4043	0.1218	112.81
	1.503	0.4867	-0.2433	1.2433	0.3915	0.4699	0.1232	93.80
	2.031	0.5850	-0.5479	1.5479	0.3779	0.5647	0.1534	83.39
	2.583	0.6777	-0.6344	1.6344	0.4147	0.6542	0.1620	75.98
	3.031	0.7400	-0.7157	1.7157	0.4313	0.7143	0.1700	70.71
	3.599	0.8108	-0.8140	1.8140	0.4470	0.7827	0.1798	65.24
	4.057	0.8602	-0.9102	1.9102	0.4503	0.8304	0.1893	61.40
	5.019	0.9448	-1.0806	2.0806	0.4541	0.9120	0.2062	54.51
	6.004	0.9988	-1.2206	2.2206	0.4498	0.9642	0.2201	48.18
	7.016	1.0278	-1.2262	2.2262	0.4617	0.9922	0.2206	42.42
	8.063	1.0326	-1.2056	2.2056	0.4682	0.9968	0.2186	37.09
	8.599	1.0315	-1.2020	2.2020	0.4685	0.9958	0.2182	34.74
	9.131	1.0352	-1.2081	2.2081	0.4688	0.9993	0.2188	32.83
	9.264	1.0359	-1.2050	2.2050	0.4698	1.0000	0.2185	32.38
	9.793	1.0330	-1.1978	2.1978	0.4700	0.9972	0.2178	
	10.055	1.0319	-1.1927	2.1927	0.4706	0.9961	0.2173	
	10.649	1.0263	-1.1752	2.1752	0.4718	0.9907	0.2156	
	11.003	1.0188	-1.1669	2.1669	0.4702	0.9835	0.2148	
	12.024	1.0020	-1.1438	2.1438	0.4674	0.9673	0.2125	
	13.041	0.9805	-1.1103	2.1103	0.4646	0.9465	0.2091	

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REMARKS: Corrected for Stone and Membrane Deflections

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK_oUDSS NO. WDSS8

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	14.064	0.9539	-1.0666	2.0666	0.4616	0.9209	0.2048	
	15.148	0.8566	-0.8530	1.8530	0.4623	0.8269	0.1836	
	16.187	0.8245	-0.8370	1.8370	0.4488	0.7959	0.1821	
	17.232	0.7738	-0.7593	1.7593	0.4398	0.7469	0.1744	
	18.247	0.7477	-0.7294	1.7294	0.4324	0.7218	0.1714	
	19.294	0.7033	-0.6403	1.6403	0.4287	0.6789	0.1626	
	20.350	0.6367	-0.4787	1.4787	0.4306	0.6146	0.1466	
	21.352	0.6216	-0.4691	1.4691	0.4231	0.6000	0.1456	
	22.012	0.6100	-0.4521	1.4521	0.4200	0.5888	0.1439	
	23.020	0.5865	-0.4082	1.4082	0.4165	0.5662	0.1396	
	24.029	0.5605	-0.3526	1.3526	0.4144	0.5411	0.1341	
	25.045	0.5258	-0.2482	1.2482	0.4213	0.5076	0.1237	
	26.427	0.4802	-0.1596	1.1596	0.4141	0.4635	0.1149	
	27.084	0.4744	-0.1536	1.1536	0.4113	0.4580	0.1143	
	28.070	0.4647	-0.1411	1.1411	0.4072	0.4486	0.1131	
	29.062	0.4554	-0.1274	1.1274	0.4039	0.4396	0.1117	
	30.041	0.4428	-0.1073	1.1073	0.3999	0.4274	0.1097	
	31.372	0.4267	-0.1042	1.1042	0.3864	0.4119	0.1094	
	32.035	0.4089	-0.0794	1.0794	0.3788	0.3947	0.1070	
	33.072	0.3874	-0.0372	1.0372	0.3735	0.3739	0.1028	

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REMARKS: Corrected for [illegible]

DIRECT - SIMPLE SHEAR TEST

PROJECT Sohio TYPE OF TEST CK₀UDSS NO. WDSS9 OCR 20

SOIL TYPE Arctic Silt TESTED BY GY DEVICE Geonor DATE

LOCATION Smith Bay
STE W ; RE = 9.7'

CONSOLIDATION (Stresses in KSC)

σ'_{vc} 0.593 τ_{hc} — σ'_p

t_c(Day) 1.0 ε_v(%) 6.04 γ_c(%) — t_c(Day) —

	W,%	e	S,%	H (cm)
Initial	28.9	0.80		2.388
Preshear	27.6	0.69		2.244
Final	29.8			2.322

DURING SHEAR
Controlled Strain ✓ Stress
Rate (% / Hr.) 5.0

TIME (Hr.)	STRAIN (%) *	τ_h / σ'_{vc}	$\Delta u / \sigma'_{vc}$	σ'_v / σ'_{vc}	τ_h / S_u	τ_h / σ'_p	σ'_v / σ'_p	E_u/S_u
	0.000	0.0000	0.0000	1.0000	0.0000	0.0000	0.0496	
	0.037	0.1159	-0.6226	1.6226	0.0499	0.0057	0.0835	403.11
	0.155	0.2551	-0.6527	1.6527	0.1099	0.0127	0.0820	212.45
	0.240	0.3133	-0.6777	1.6777	0.1350	0.0155	0.0932	168.45
	0.567	0.4518	-0.7230	1.7230	0.2047	0.0224	0.0855	102.91
	1.07	0.5920	-0.8220	1.8220	0.2564	0.0295	0.0903	72.66
	1.57	0.7122	-0.9778	1.9778	0.3069	0.0353	0.0942	58.67
	2.07	0.8329	-1.0710	2.0710	0.3559	0.0413	0.1027	52.03
	2.47	0.9229	-1.1585	2.1585	0.3977	0.0458	0.1071	48.41
	3.11	1.0679	-1.3367	2.3367	0.4602	0.0530	0.1161	44.37
	3.65	1.1726	-1.4694	2.4694	0.5053	0.0582	0.1225	41.55
	4.06	1.2374	-1.4972	2.4972	[illegible]	0.0614	0.1239	39.38
	4.49	1.2997	-1.5600	2.5600	0.5601	0.0645	0.1270	37.43
	5.03	1.4167	-1.9193	2.9193	0.6105	0.0703	0.1448	36.39
	5.60	1.5408	-2.2775	3.2775	0.6640	0.0764	0.1626	35.85
	5.96	1.6430	-2.5020	3.5020	0.7080	0.0815	[illegible]	35.64
	6.50	1.7717	-2.8950	3.8950	0.7635	0.0879	0.1932	35.20
	7.17	1.9258	-3.1297	4.1297	0.8299	0.0955	0.2048	34.70
	8.0	2.0655	-3.3367	4.3367	0.8901	0.1025	0.2151	33.36
	8.44	2.1280	-3.4437	4.4437	0.9170	0.1056	0.2204	32.59
	9.42	2.2534	-3.7073	4.7073	0.9711	0.1118	0.2335	30.92
	10.0	2.2972	-3.7495	4.7495	0.9900	0.1140	0.2356	29.65
	10.8	[illegible]	-3.7559	4.7559	1.0000	0.1151	0.2359	27.84
	12.0	2.2699	-3.6231	4.6231	0.9782	0.1126	0.2293	
	14.4	[illegible]	[illegible]	4.5261	0.9728	[illegible]	[illegible]	

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* based on tsample-hairs

REMARKS:
- Corrected for membrane and stone deflections
- Used [illegible] pins

DIRECT - SIMPLE SHEAR (continued)

PROJECT Sohio SOIL Arctic Silt TYPE OF TEST CK_0UDSS NO. UDSS9

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma'_{vc}}$	$\frac{\Delta u}{\sigma'_{vc}}$	$\frac{\sigma'_v}{\sigma'_{vc}}$	$\frac{\tau_h}{S_u}$	$\frac{\tau_h}{\sigma'_p}$	$\frac{\sigma'_v}{\sigma'_p}$	REMARKS
	16.75	2.2265	-3.3778	4.3778	0.9505	0.1104	0.2172	
	18.73	2.1902	-3.2656	4.2656	0.9438	0.1086	0.2[illegible]6	
	20.70	2.1432	-3.1791	4.1791	0.9[illegible]36	0.1063	0.2073	
	21.90	2.0885	-3.1133	4.1133	0.9000	0.1036	0.2040	
	24.30	1.9699	-2.9362	3.9362	0.8499	0.0977	0.1952	
	27.50	1.8218	-2.6973	3.6973	0.7851	0.0904	0.1834	
	29.90	1.7021	[illegible]	3.4757	[illegible]	[illegible]	[illegible]	

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REMARKS: Corrected for membrane and stone deflections
• used stones w/ pins

Test No.	Sample No.	Depth RE (ft)	w_N (%)	σ'_{vc} (KSF)	σ'_p (KSF)	OCR	Symbol
WDSS1	5B-P5	14.2	29.2	16.4		1	

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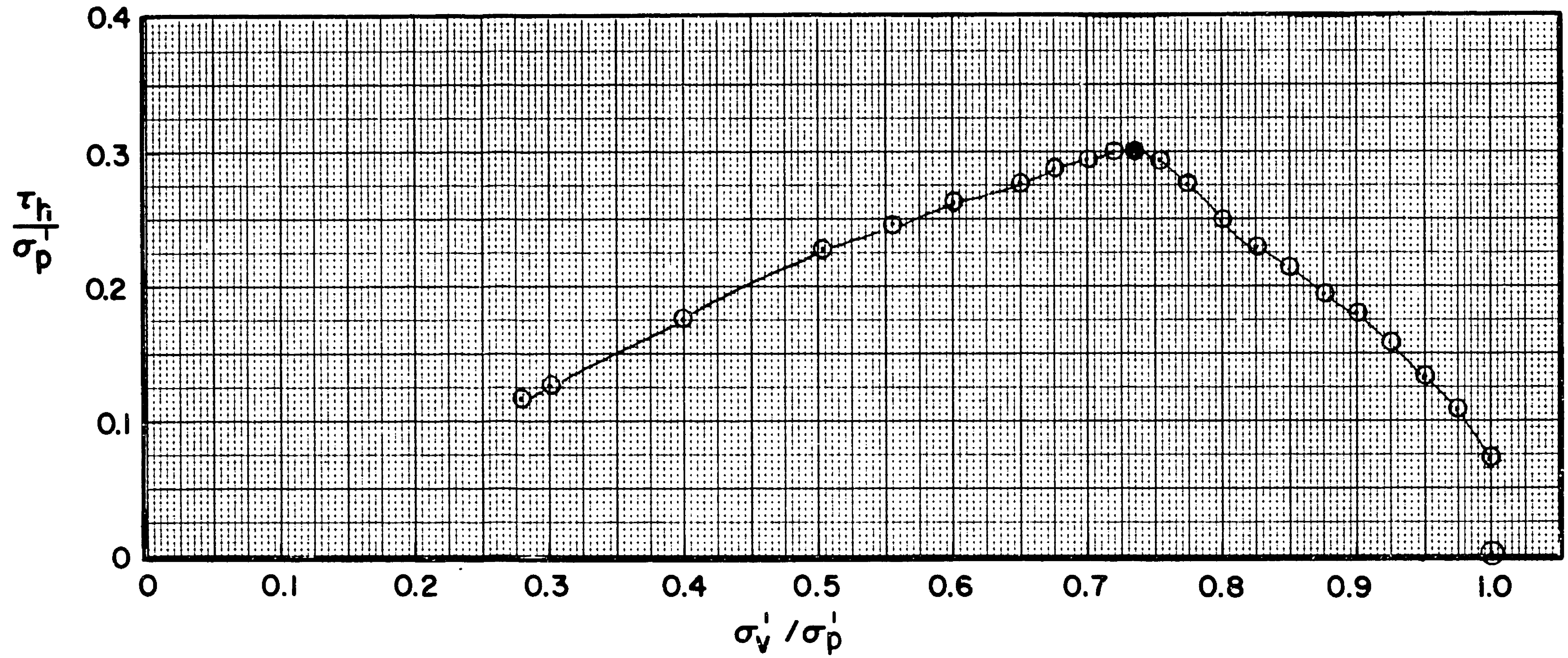
M.I.T.



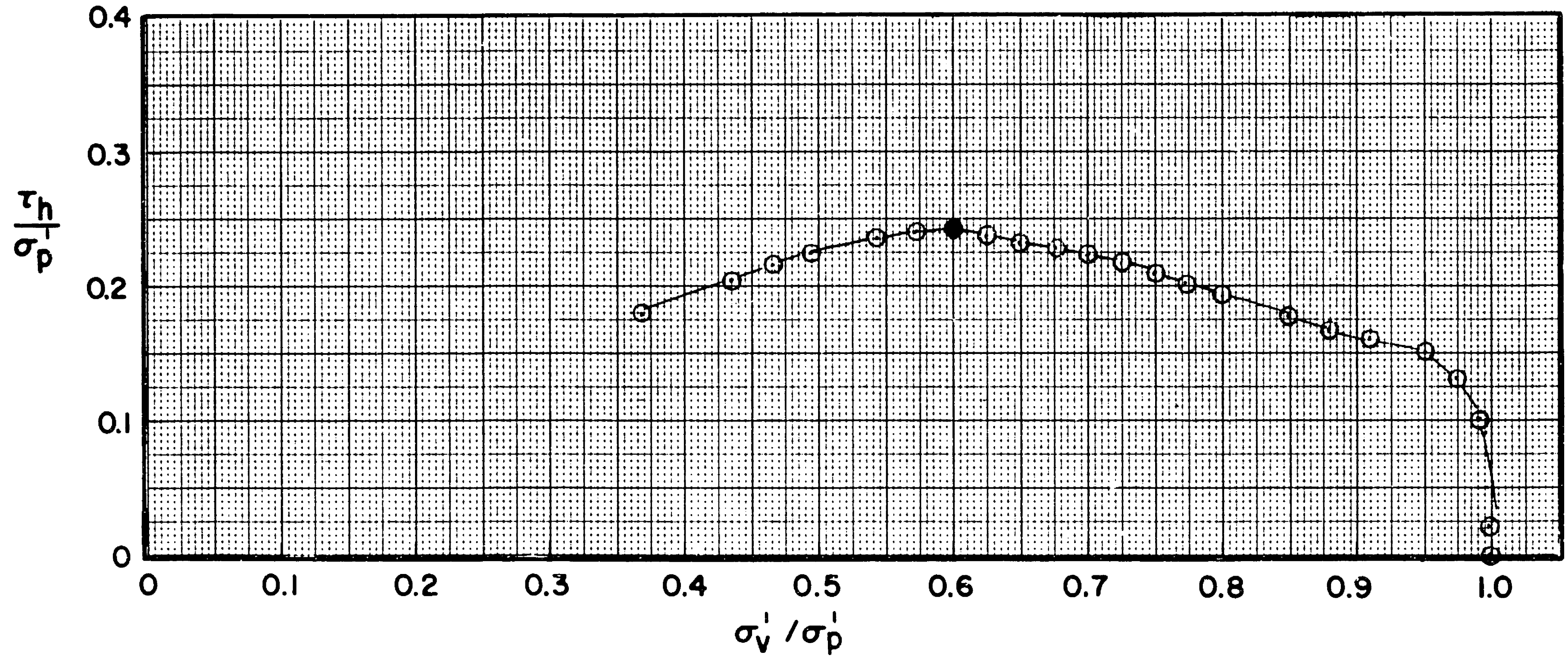
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring W56 Soil Type ARCTIC SILT

FIGURE

Test No.	Sample No.	Depth (FE)	w_N (%)	σ'_{vc} (KSF)	σ'_p ( )	OCR	Symbol
WD552	W5PV-51	3.4'	41.7	9.09		1	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



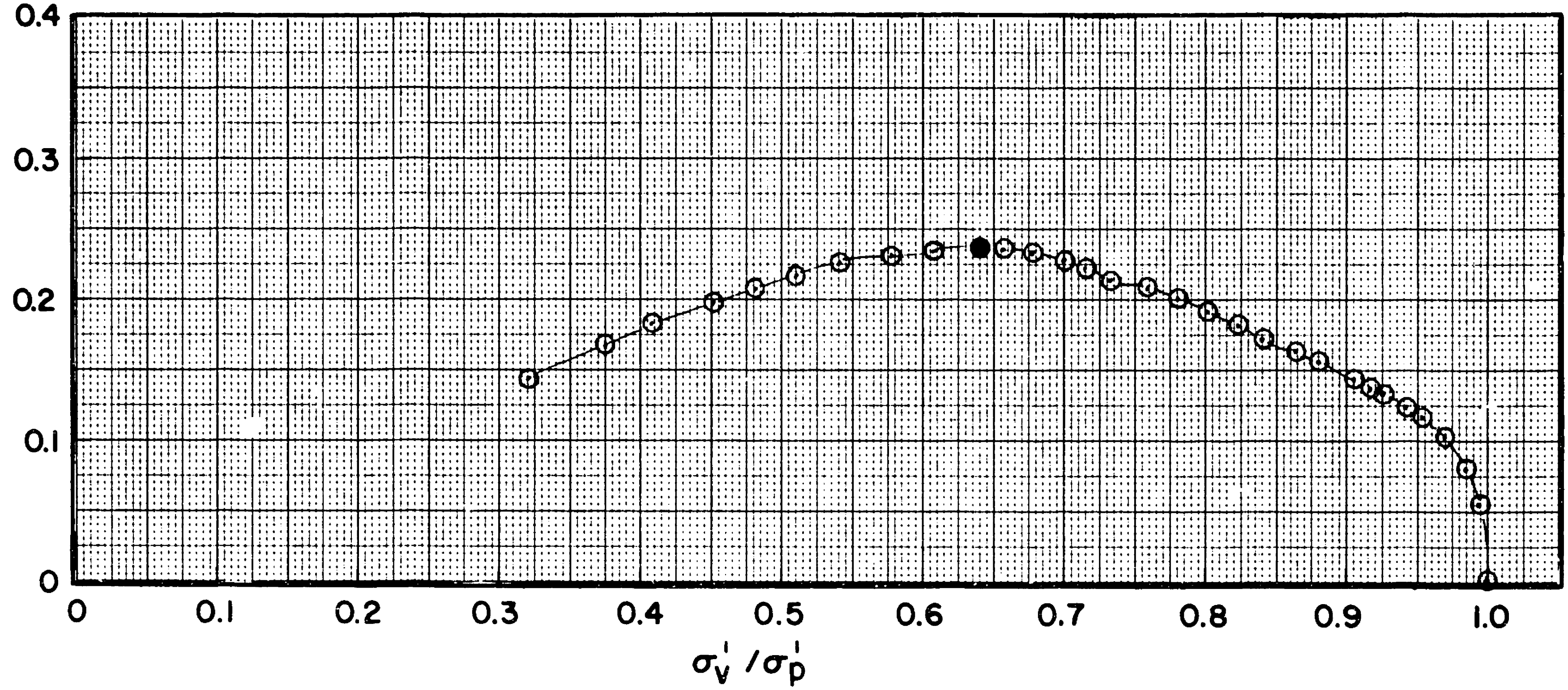
NORMALIZED STRESS PATHS FROM CK_oUDSS TESTS

Boring W5B Soil Type Arctic Silt

FIGURE

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	σ'_p ( )	OCR	Symbol
WDSS3	WSB-P3	7'	36.6	10.4		1	



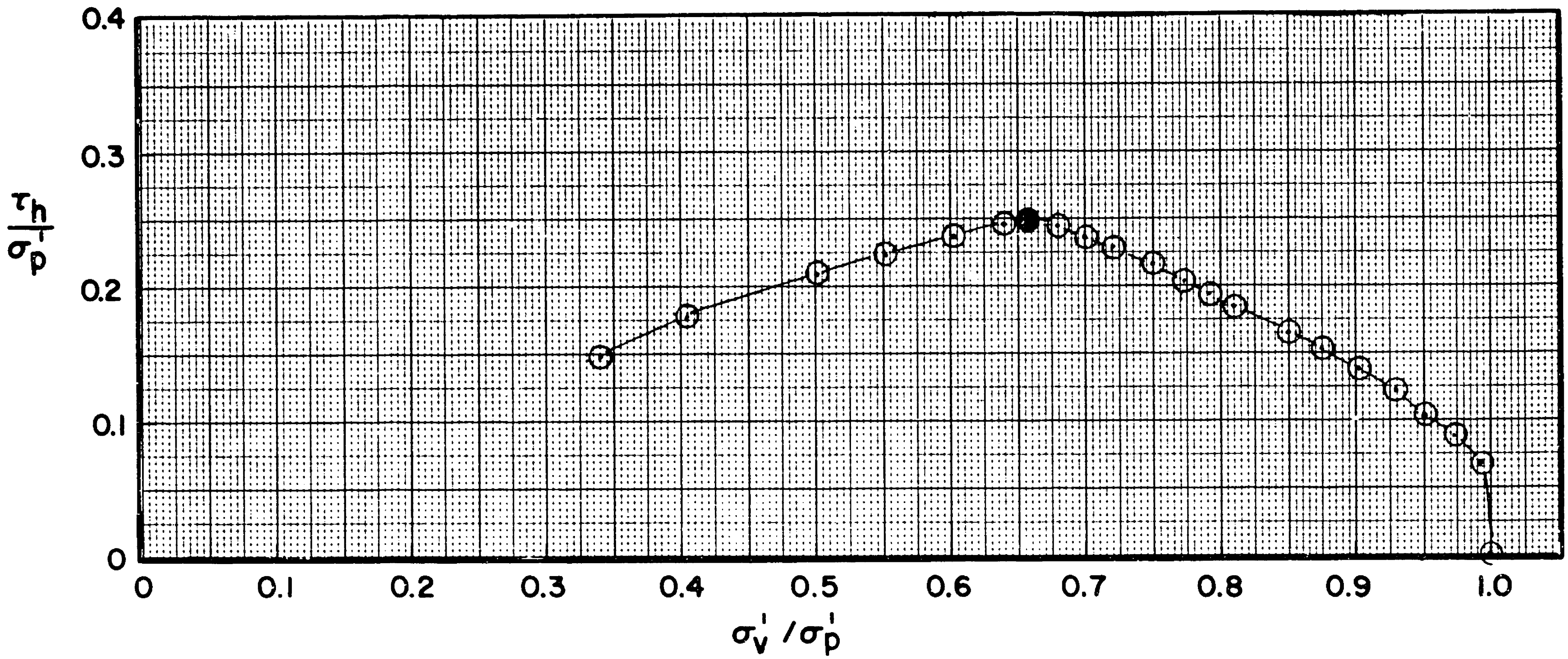
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring WSB Soil Type ARCTIC SILT

FIGURE

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

Test No.	Sample No.	Depth ([illegible])	w_N (%)	σ'_{vc} (KSF)	σ'_p ( )	OCR	Symbol
WDSS4	T5B-14	[illegible]	28.9	24.6		1	



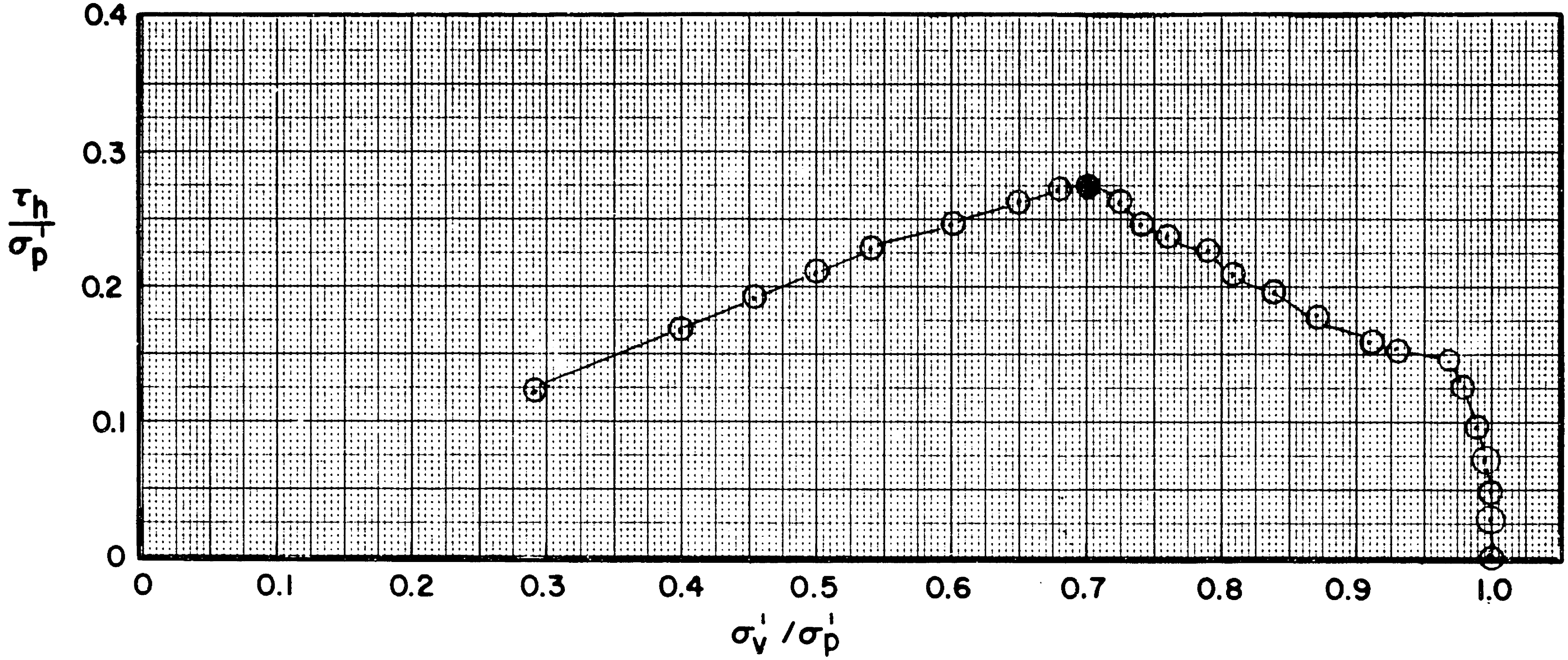
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring W5B-P4 Soil Type ARCTIC SILT

FIGURE

Test No.	Sample No.	Depth (FT)	w_N (%)	σ'_{vc} (KSF)	σ'_p ( )	OCR	Symbol
WDSS5	W5B-F4	9.3'	29.4	16.4		1	

GEOTECHNICAL LABORATORY
DEPT OF CIVIL ENGR.
M.I.T.



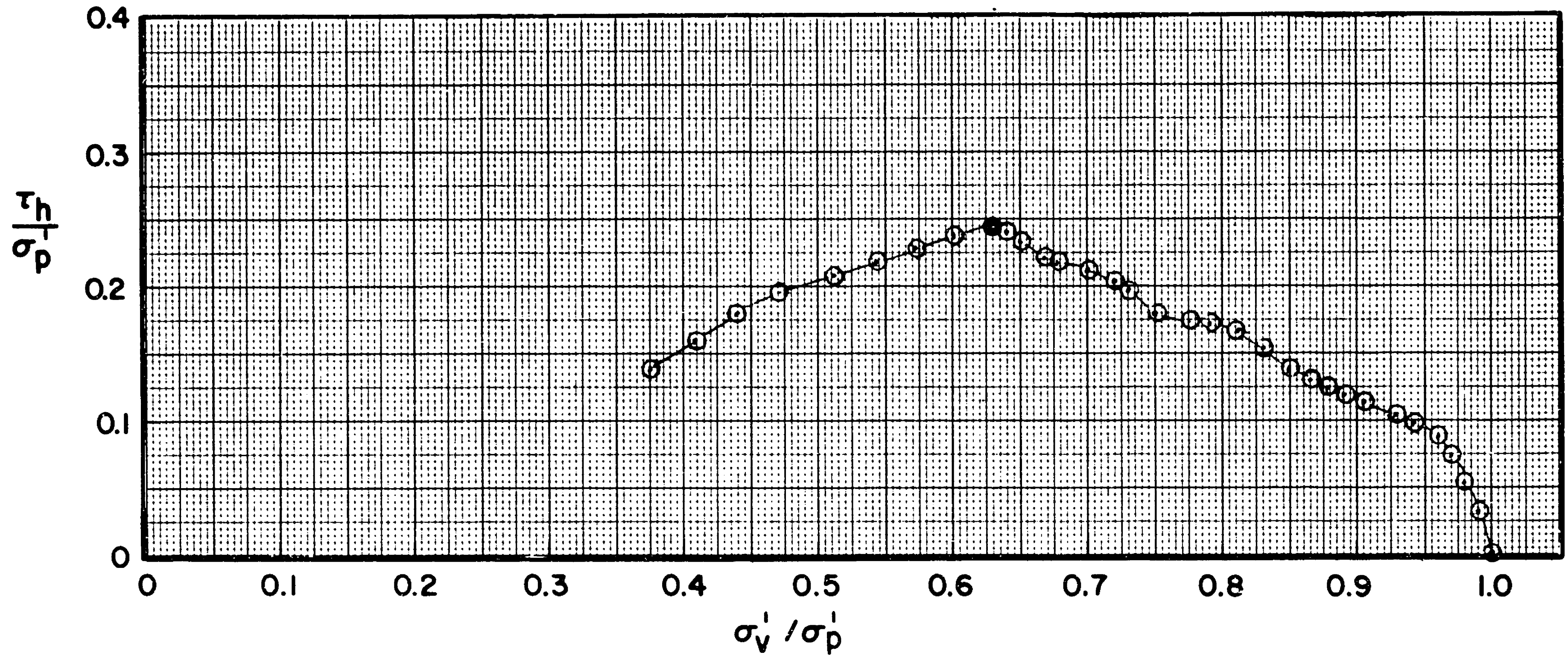
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring W5B-F4 Soil Type ARCTIC SILT

FIGURE

Test No.	Sample No.	Depth (FE)	w_N (%)	σ'_{vc} (KSF)	σ'_p ( )	OCR	Symbol
WD556	W5B-P4	9.4'	30.0	14.5		1	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



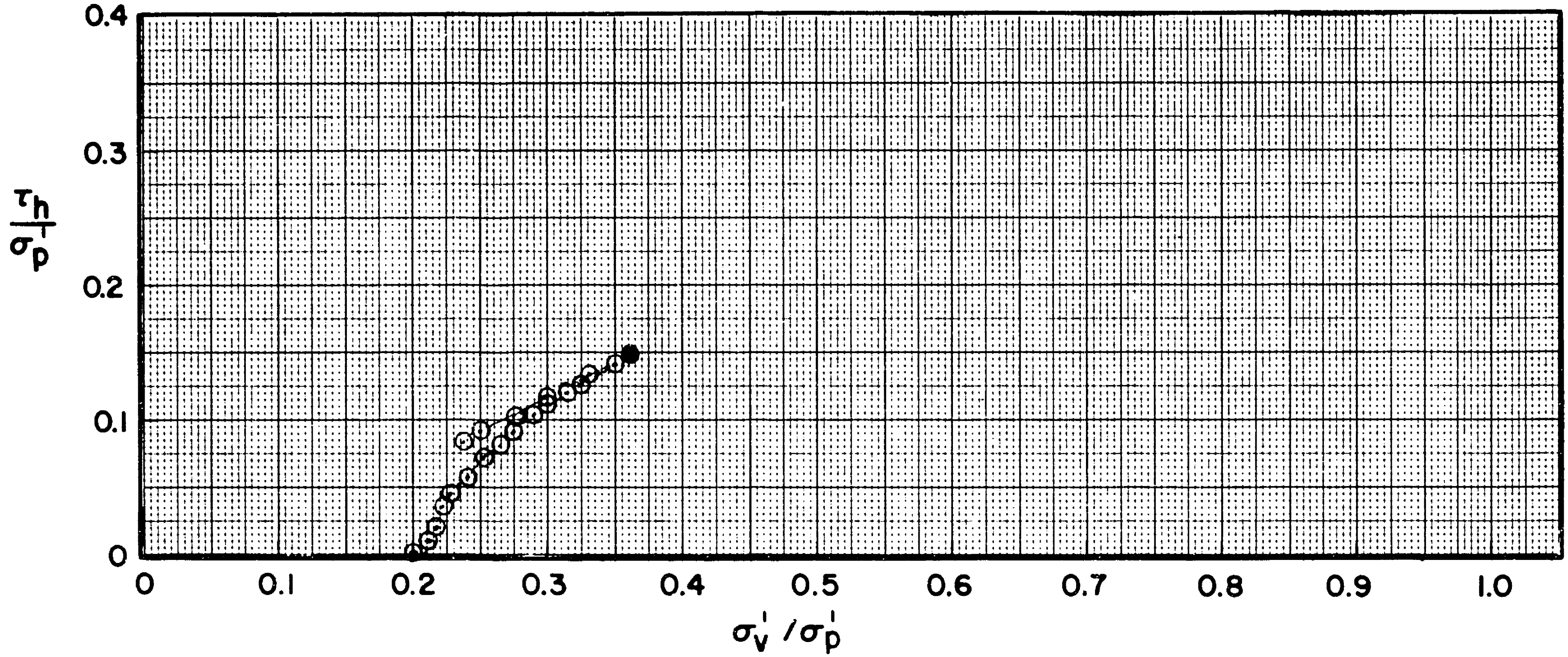
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring W5B Soil Type ARCTIC SILT

FIGURE

Test No.	Sample No.	Depth (FT)	w_N (%)	σ'_{vc} (KSF)	σ'_p (KSF)	OCR	Symbol
WDSS7	W5B-P4	9.5'	29.4	4.92	24.6	5	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



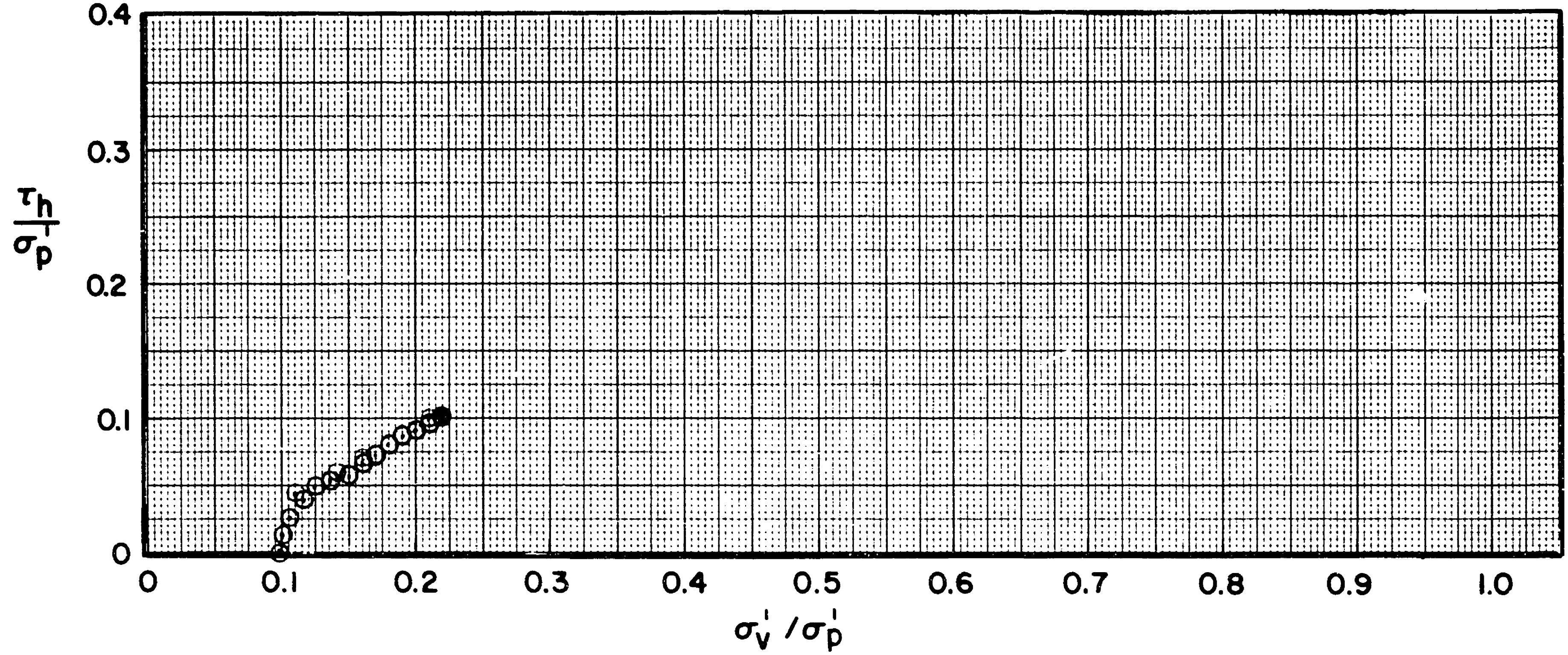
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring W5B-P4 Soil Type ARCTIC SILT

FIGURE

Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	σ'_p (KSF)	OCR	Symbol
WDSS8	W5B-P4	9.6'	29.0	2.44	24.6	10.04	

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



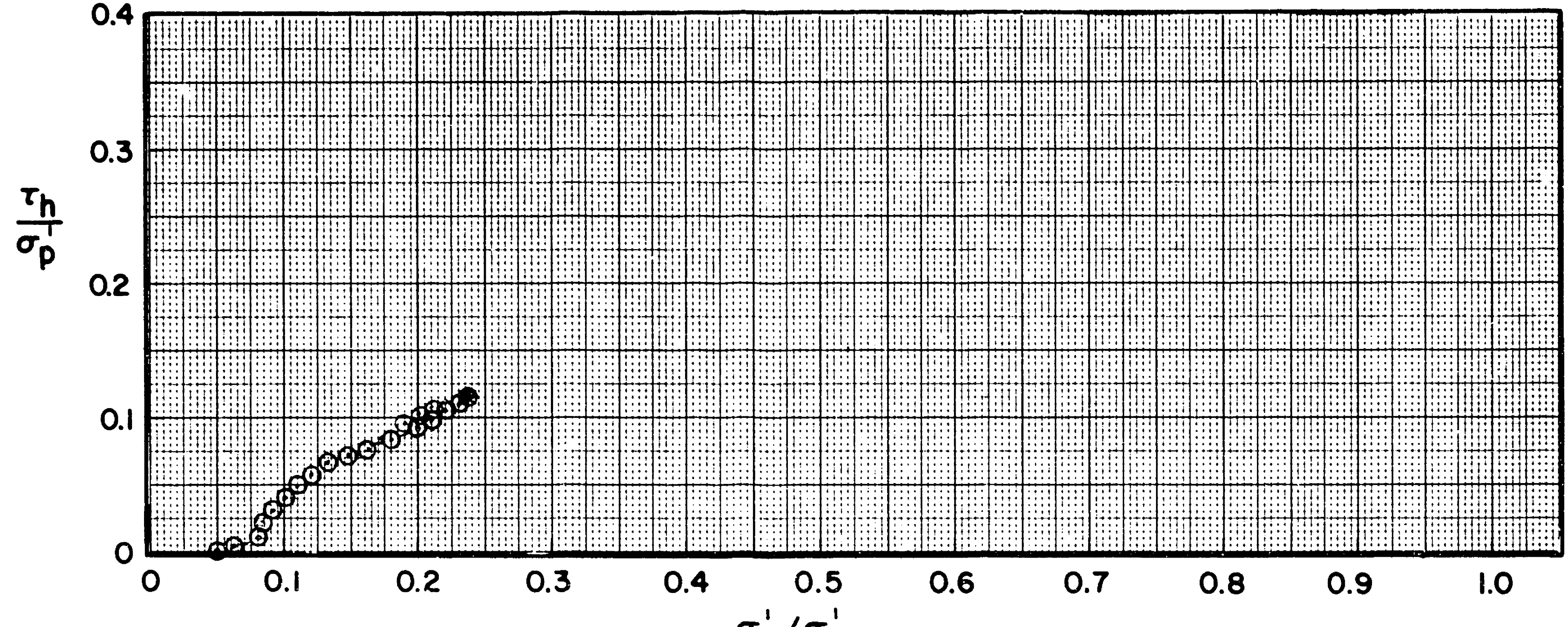
NORMALIZED STRESS PATHS FROM CK_0UDSS TESTS

Boring 5B-P4 Soil Type Arctic Silt

FIGURE

Test No.	Sample No.	Depth (FE)	w_N (%)	σ'_{vc} (ksf)	σ'_p (ksf)	OCR	Symbol
WDS39	W5B-P4	9.7'	28.9	1.22	24.6	20.2	

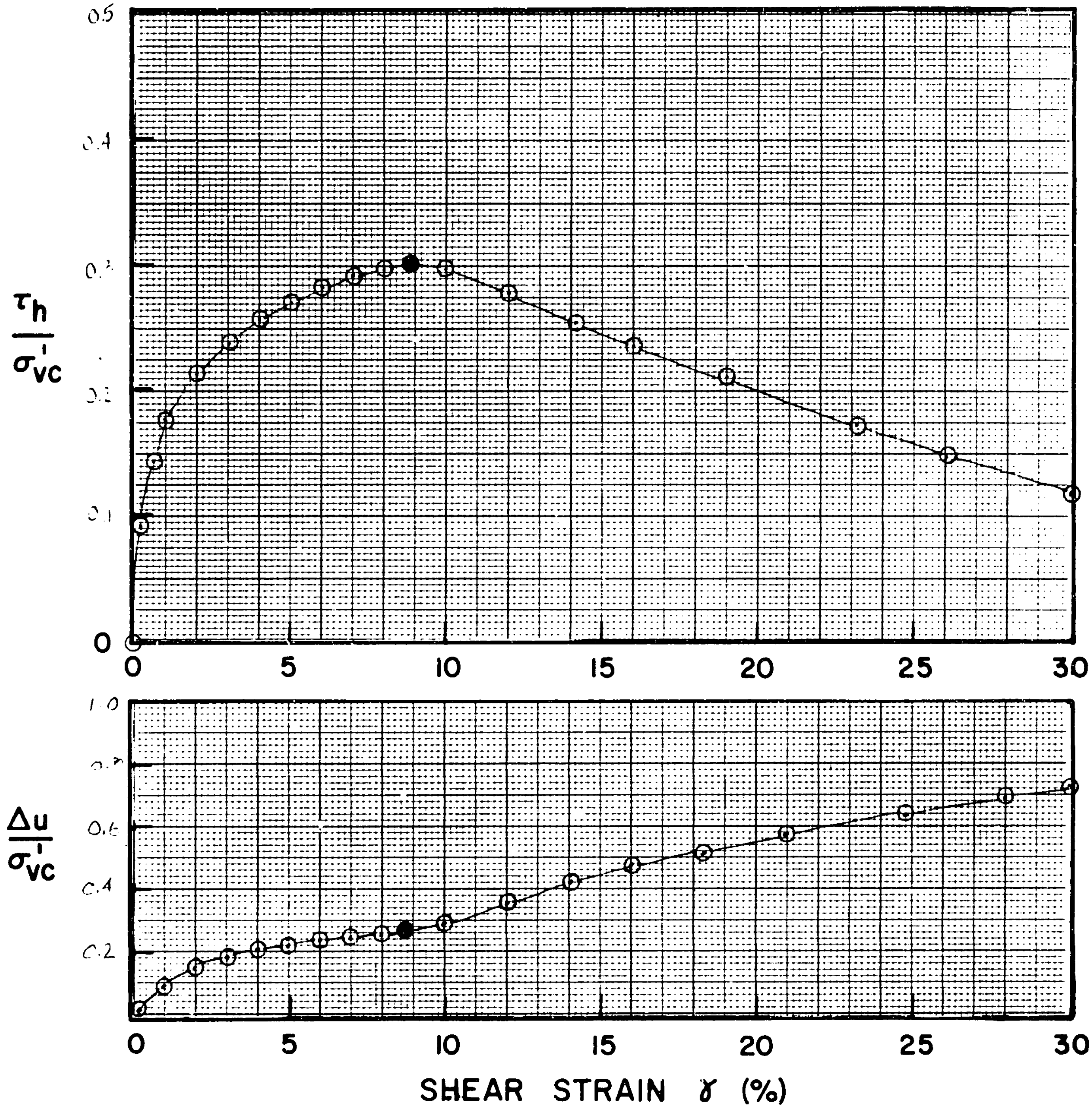
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.



NORMALIZED STRESS PATHS FROM CK_oUDSS TESTS

Boring ________ Soil Type ________________

FIGURE



Sample No. W5B-P5 w_N(%) 29.2 σ'_{vc}(KSF) 10.4 t_c(Days) ___

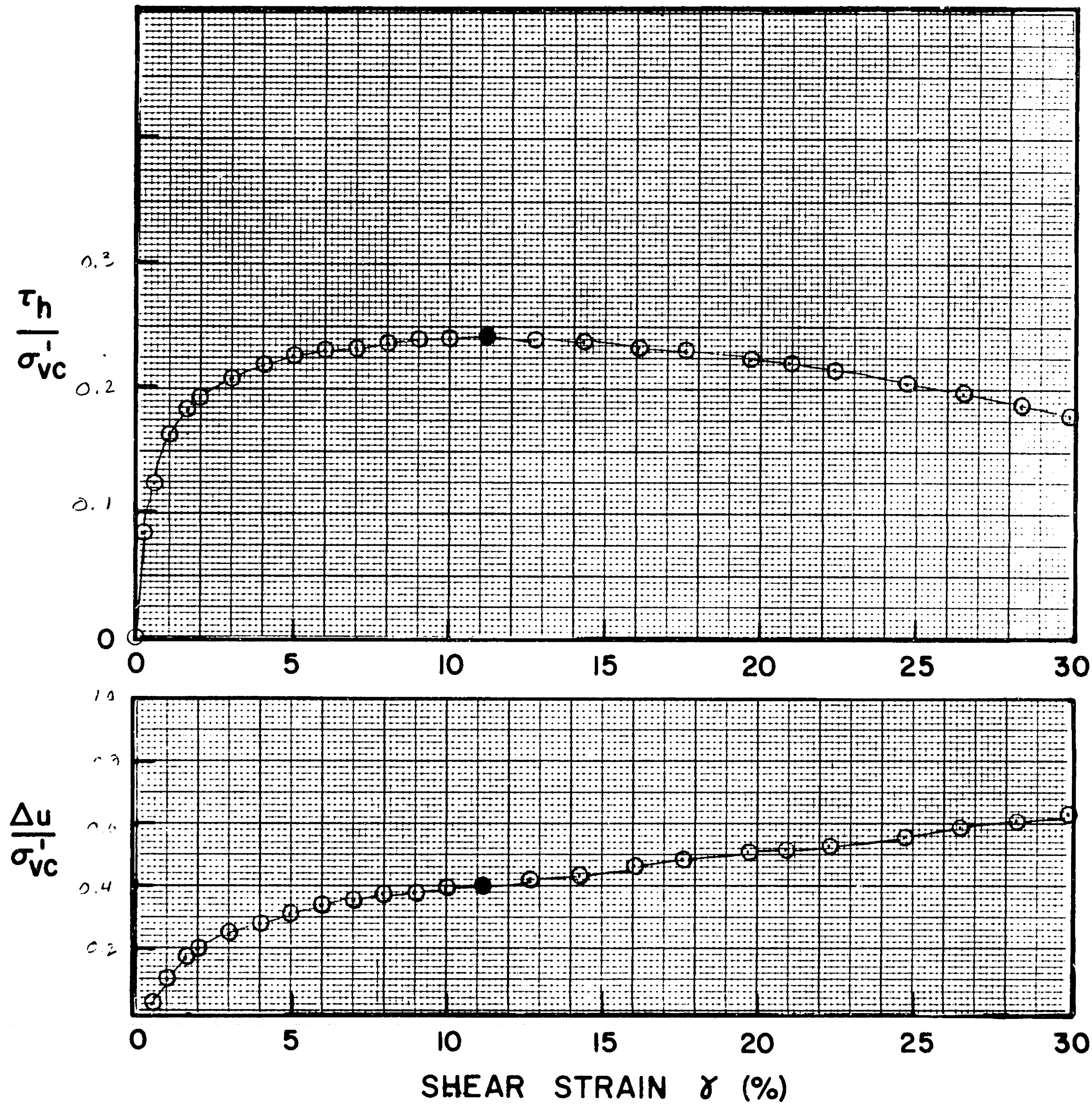
Depth (RE) 14.2' w_L(%) 51.6 σ'_p (KSF) 9.3 OCR 1

Soil Type Archc Silt w_P(%) 25.2 Estimated σ'_{vo}(ksf) 0.781

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN
CK_oUDSS TEST NO. WDSS1

FIGURE



Sample No. W5PV-S1 w_N(%) 41.7 σ'_{vc}(KSF) 9.09 t_c(Days) ____

Depth (RE) 3.4' w_L(%) 48.3 σ'_p ( ) ____ OCR 1

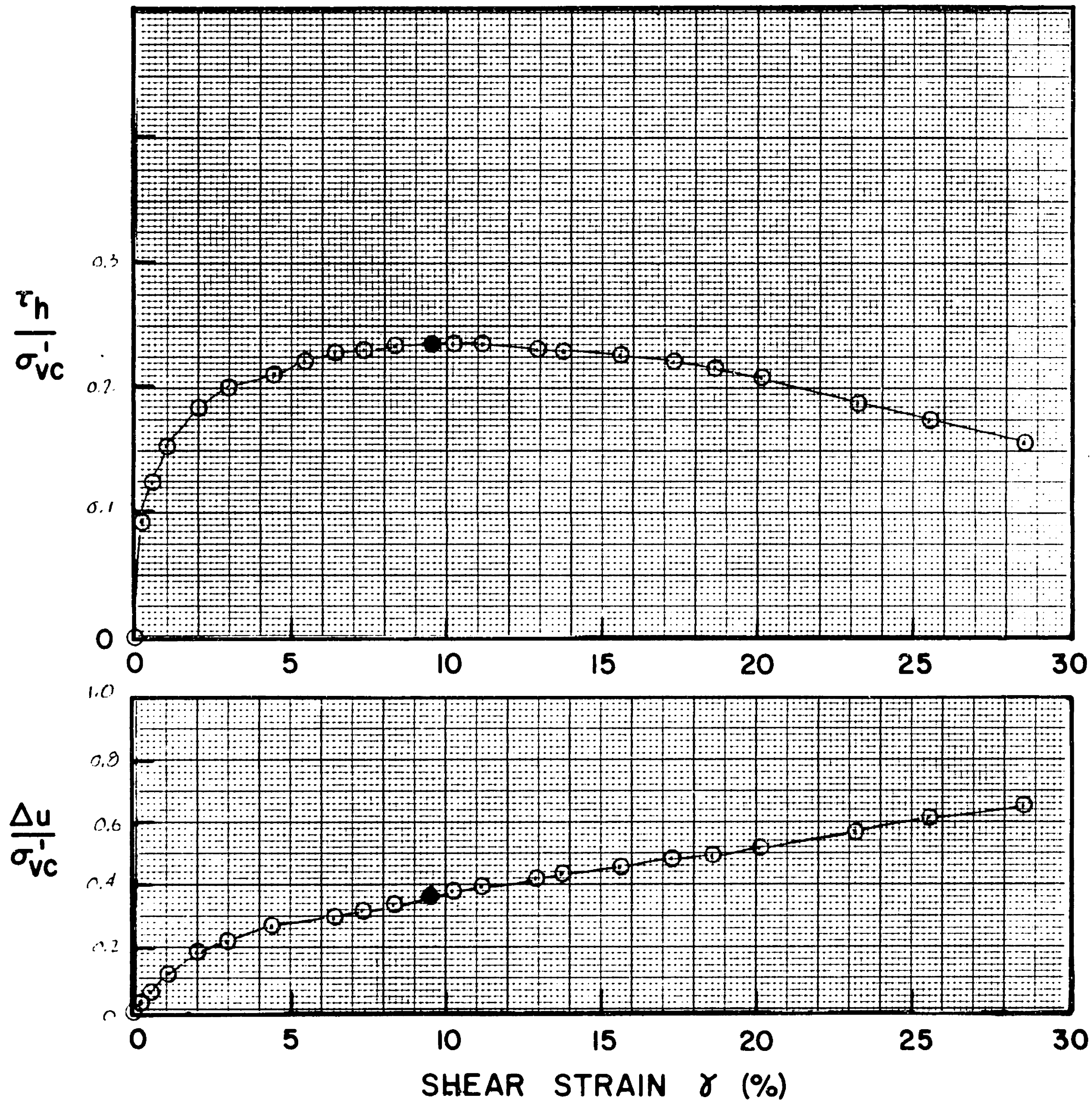
Soil Type ARCTIC SILT w_P(%) 24.3 Estimated σ'_{vo}(KSF) 0.187

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. WDSS2

FIGURE



Sample No. W5B-P3 w_N(%) 36.6 σ'_{vc}(KSF) 16.4 t_c(Days) ____

Depth (RE) 7' w_L(%) 53.5 σ'_p ( ) ____ OCR 1

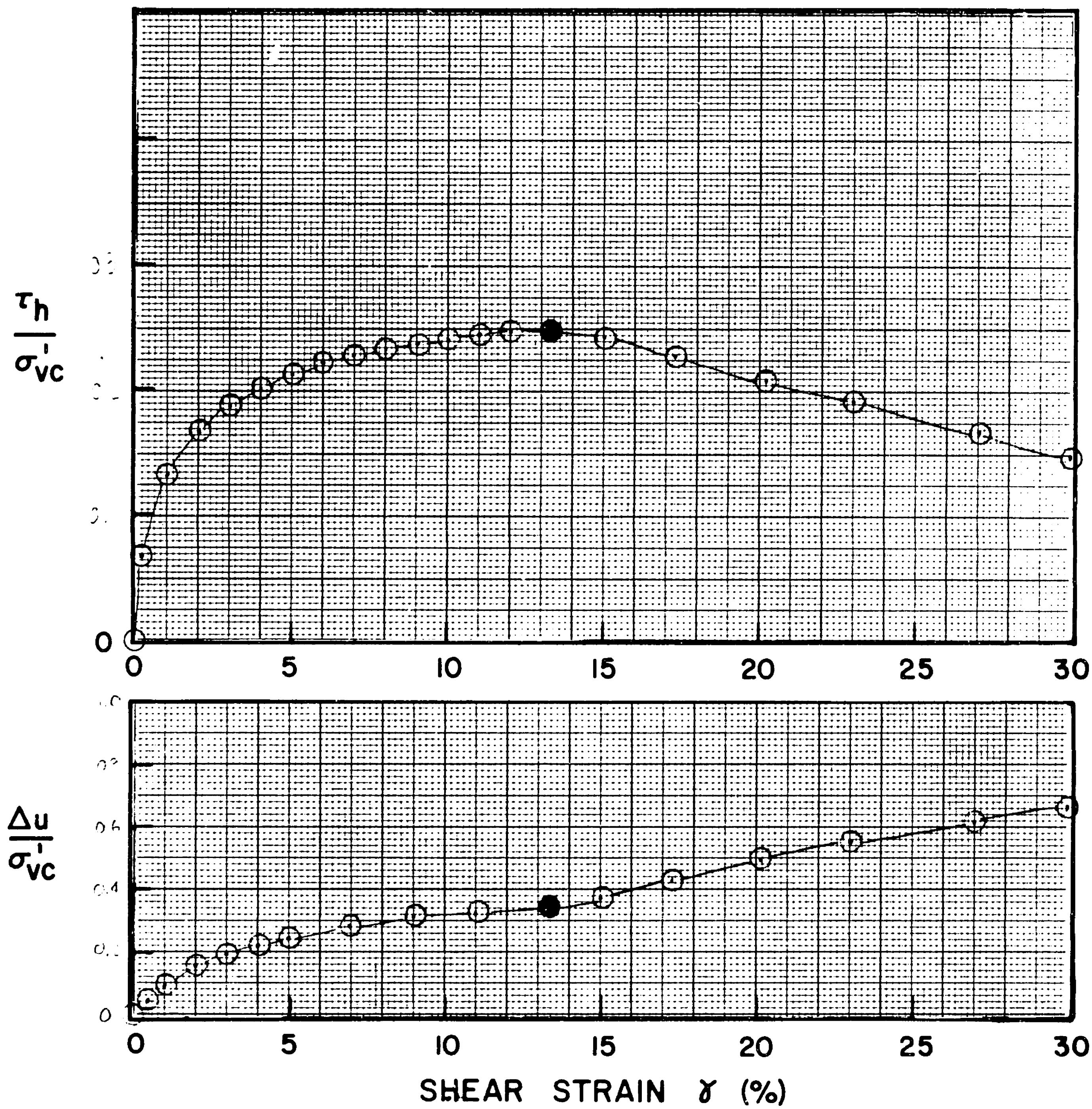
Soil Type ARCTIC SILT w_P(%) 26.4 Estimated σ'_{vo}(KSF) 0.385

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. WDSS3

FIGURE



Sample No. W5B-P4 w_N(%) 28.9 σ'_{vc}(KSF) 24.6 t_c(Days) 1.0

Depth ft 9.2' w_L(%) 44 σ'_p ( ) ____ OCR 1

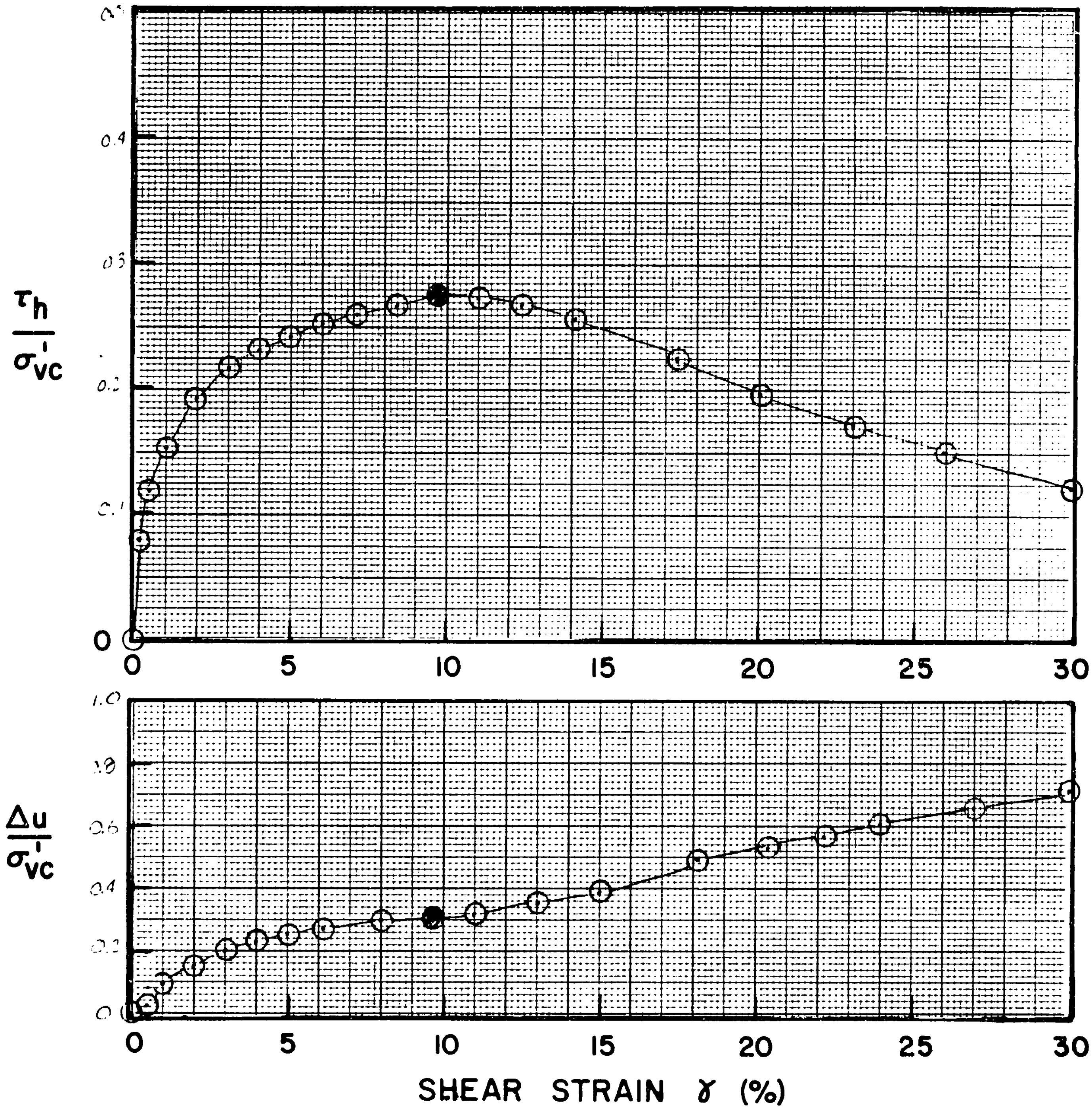
Soil Type ______ w_P(%) 24 Estimated σ'_{vo}(KSF) 0.506

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. WD554

FIGURE



Sample No. W5B-P4 w_N(%) 29.4 σ'_{vc}(KSF) 16.4 t_c(Days) 1.0

Depth (KE) 4.3' w_L(%) 44 σ'_p ( ) ____ OCR 1

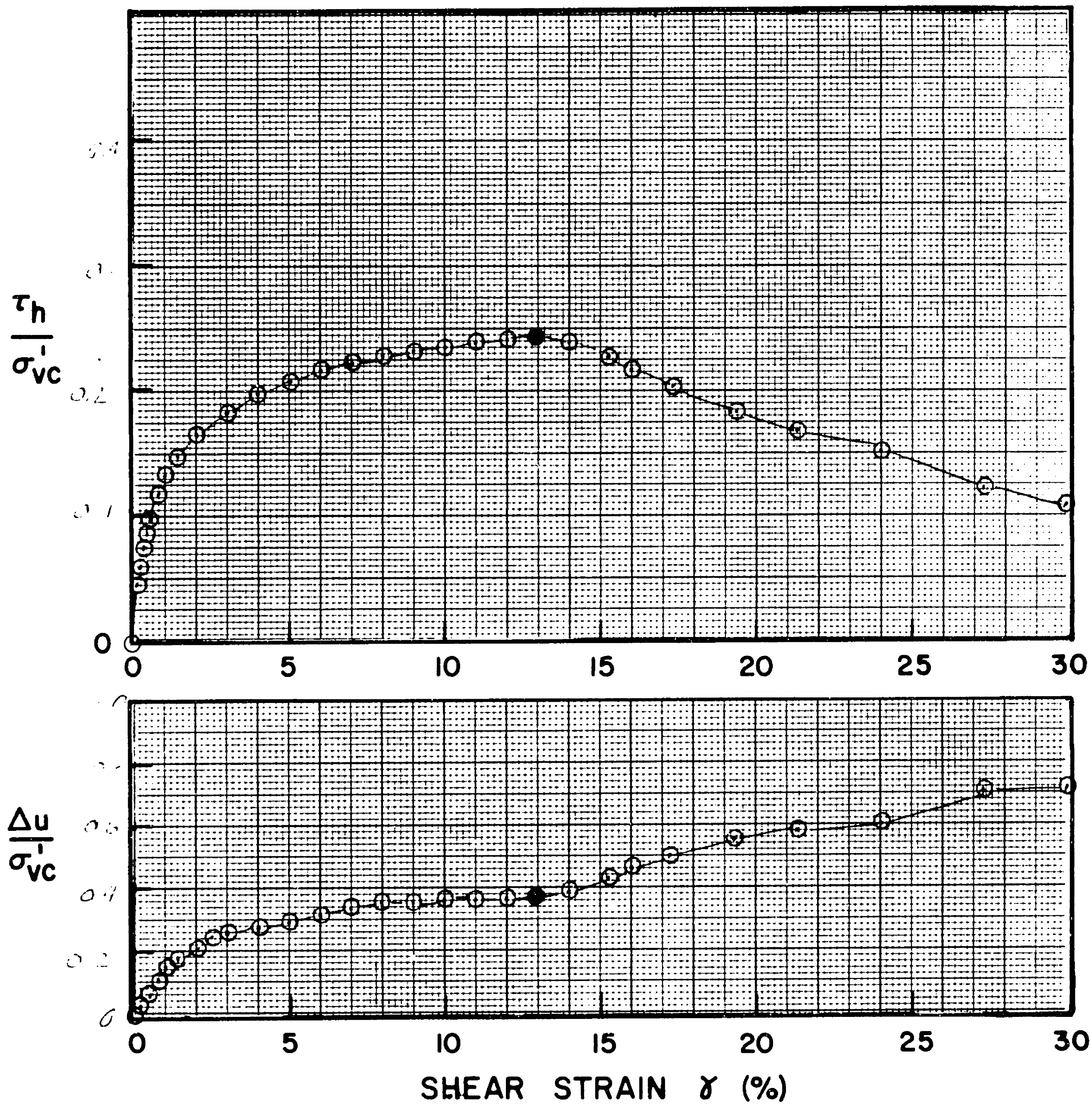
Soil Type ARCTIC SILT w_P(%) 24 Estimated σ'_{vo}(KSF) 0.512

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. WDSS5

FIGURE



Sample No. WEB-14 w_N(%) 30.0 σ'_{vc}(KSF) 2.45 t_c(Days) 1.0

Depth (KE) 9.4' w_L(%) 44 σ'_p ( ) ____ OCR 1

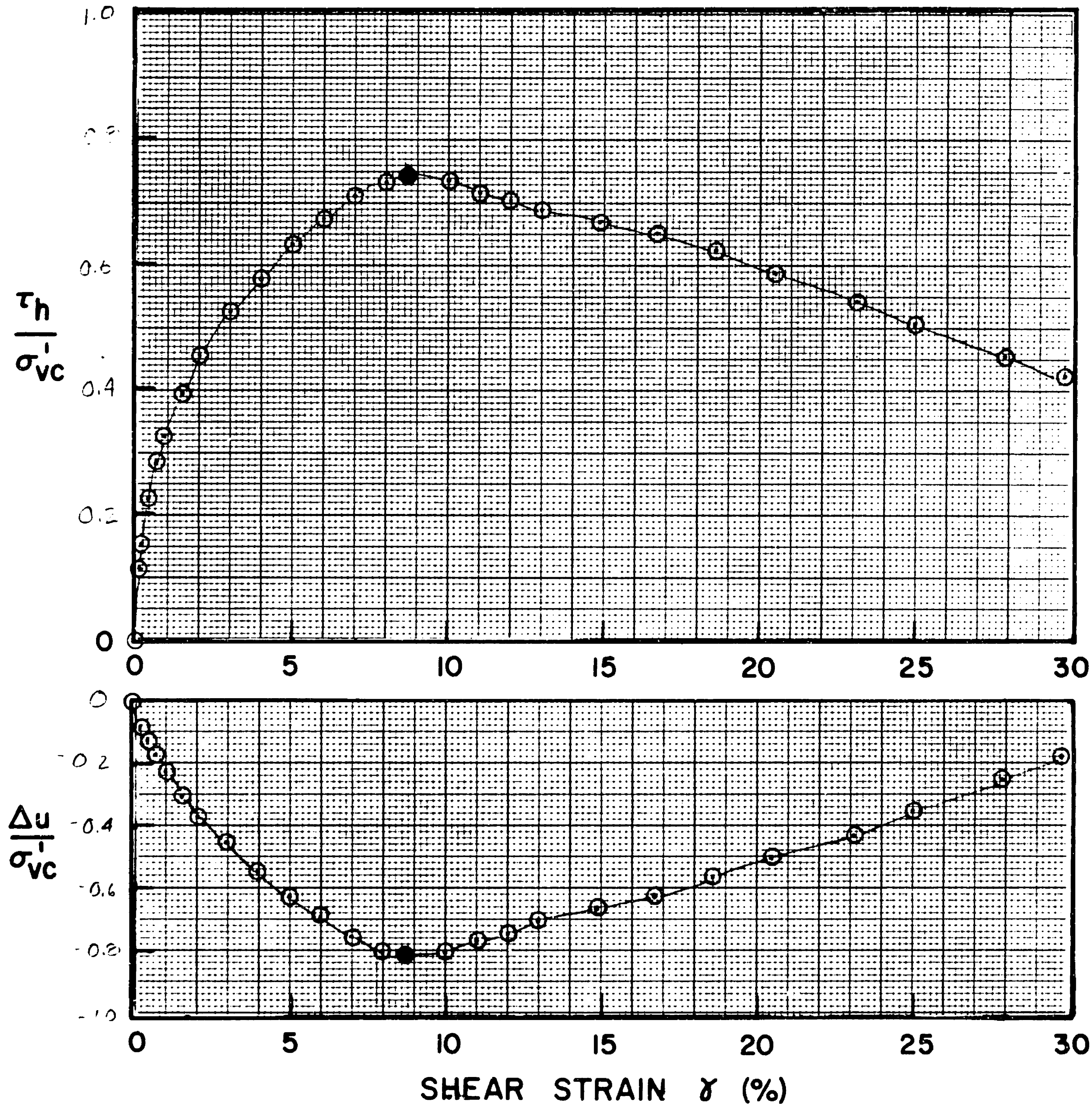
Soil Type Arctic Silt w_P(%) 24 Estimated σ'_{vo}(KSF) 0.517

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. WDSS6

FIGURE



Sample No. W5B-P4 w_N(%) 29.4 σ'_{vc}(KSF) 4.92 t_c(Days) ____

Depth (KE) 9.5' w_L(%) ____ σ'_p (KSF) 24.6 OCR 5

Soil Type Arctic Silt w_P(%) ____ Estimated σ'_{vo}(KSF) 0.523

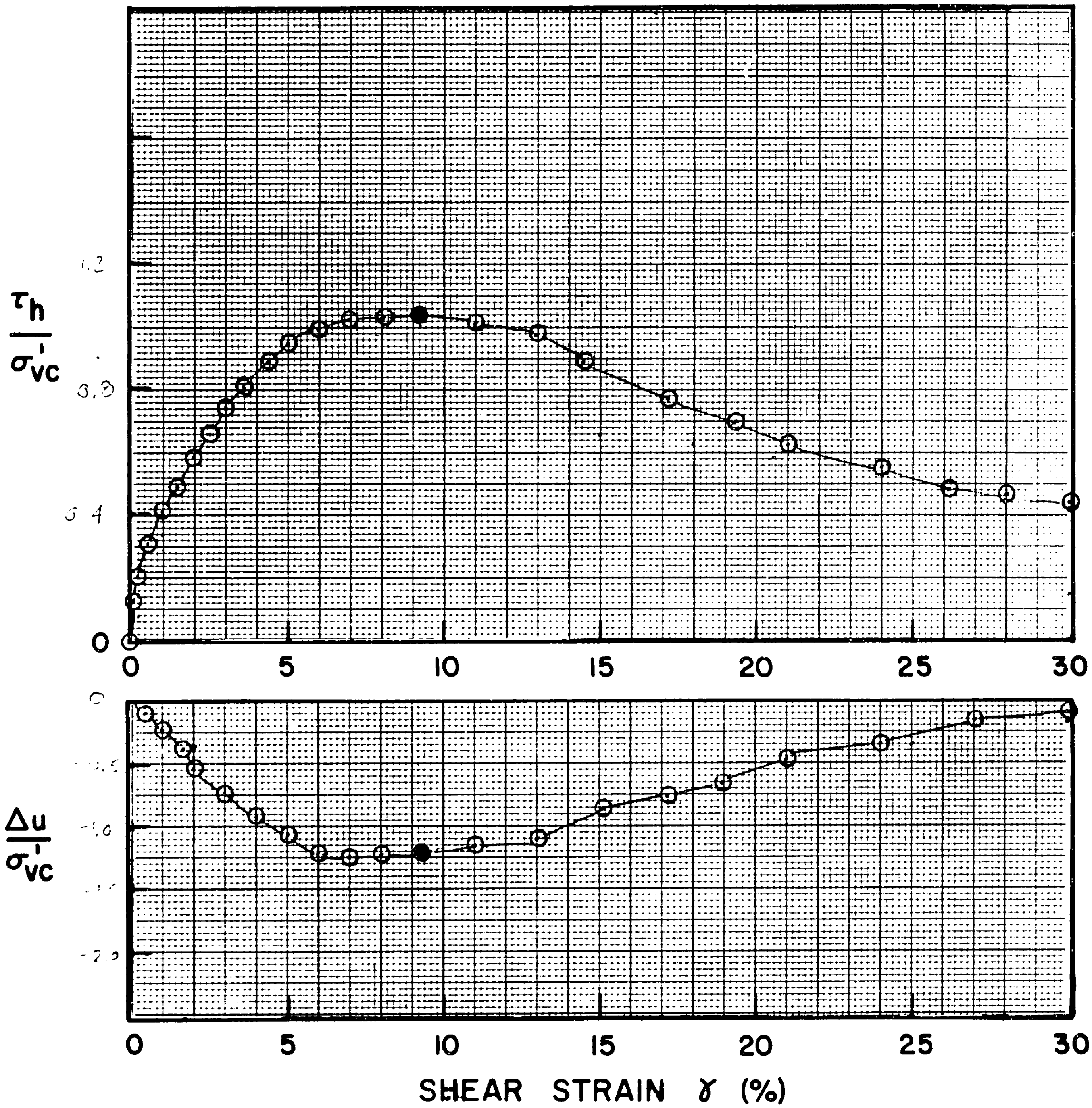
Silty Clay (CL)

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. WDSS7

FIGURE



Sample No. W5B-P4 w_N(%) 29.0 σ'_{vc}(KSF) 2.44 t_c(Days) 1.0

Depth (RE) 9 6' w_L(%) ______ σ'_p (KSF) 24.6 OCR 10.09

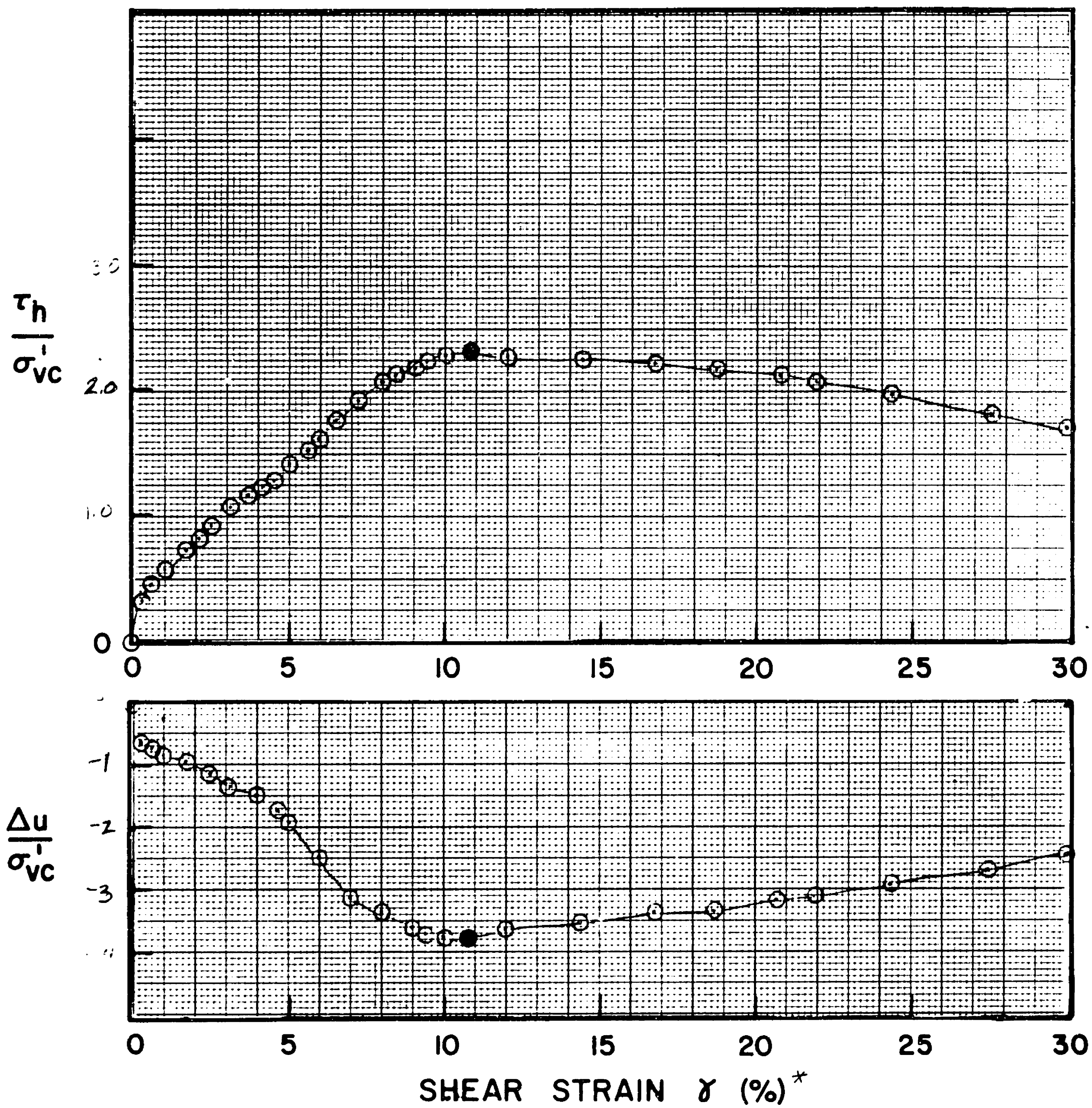
Soil Type ARCTIC SILT w_P(%) ______ Estimated σ'_{vo}(KSF) 0.528

GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. WDSS8

FIGURE



Sample No. W53-P4 w_N(%) [illegible] σ'_{vc}(ksf) 1.22 t_c(Days) ____

Depth (FE) 9.7 FT. w_L(%) 44.0 σ'_p (ksf) 24.6 OCR 20.

Soil Type Arctic Silt w_P(%) 24.0 Estimated σ'_{vo}(ksf) 0.534

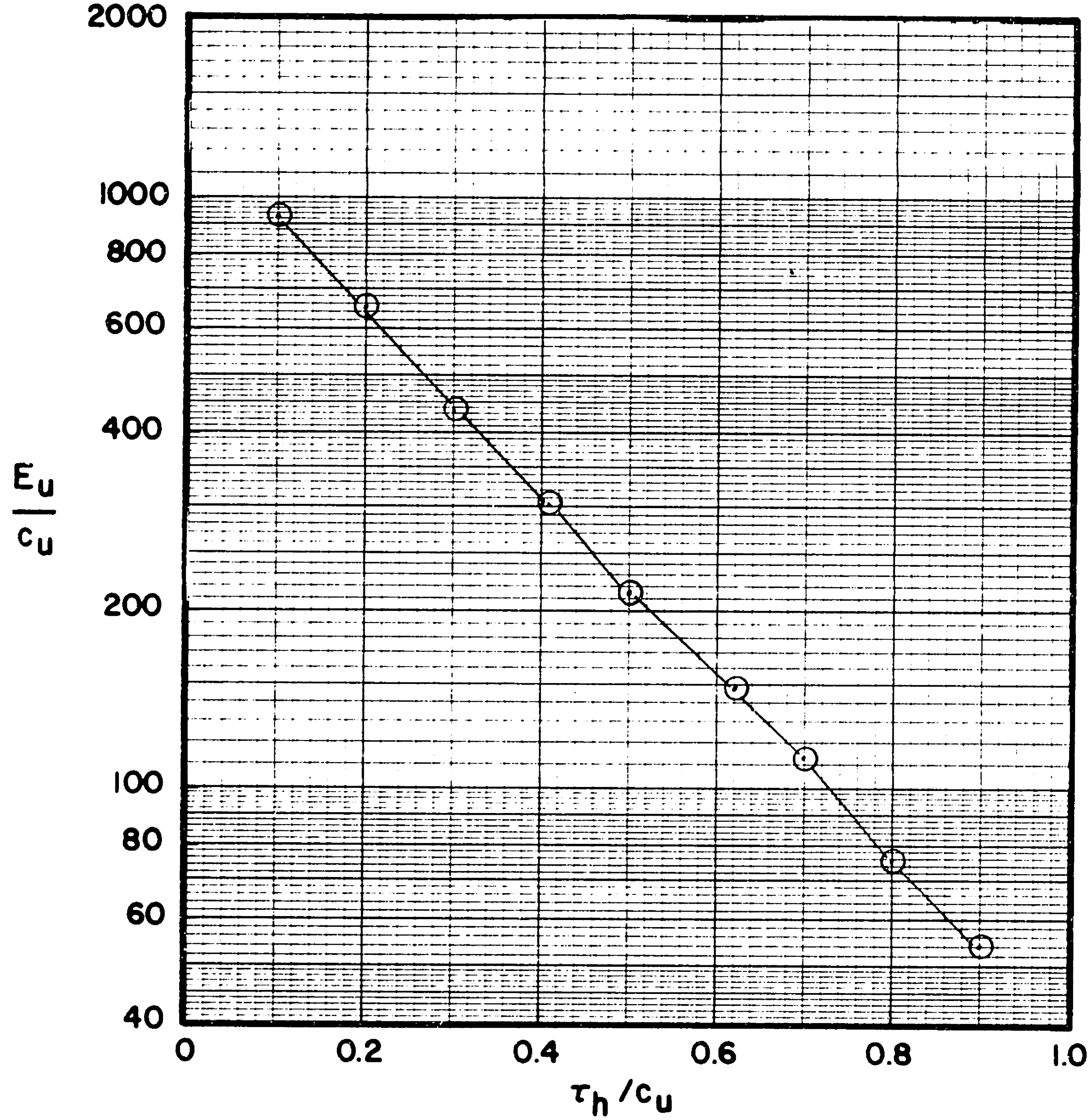
GEOTECHNICAL LABORATORY
DEPT. OF CIVIL ENGR.
M.I.T.

NORMALIZED STRESS VS STRAIN

CK_oUDSS TEST NO. WDSS9

* based [illegible]

FIGURE



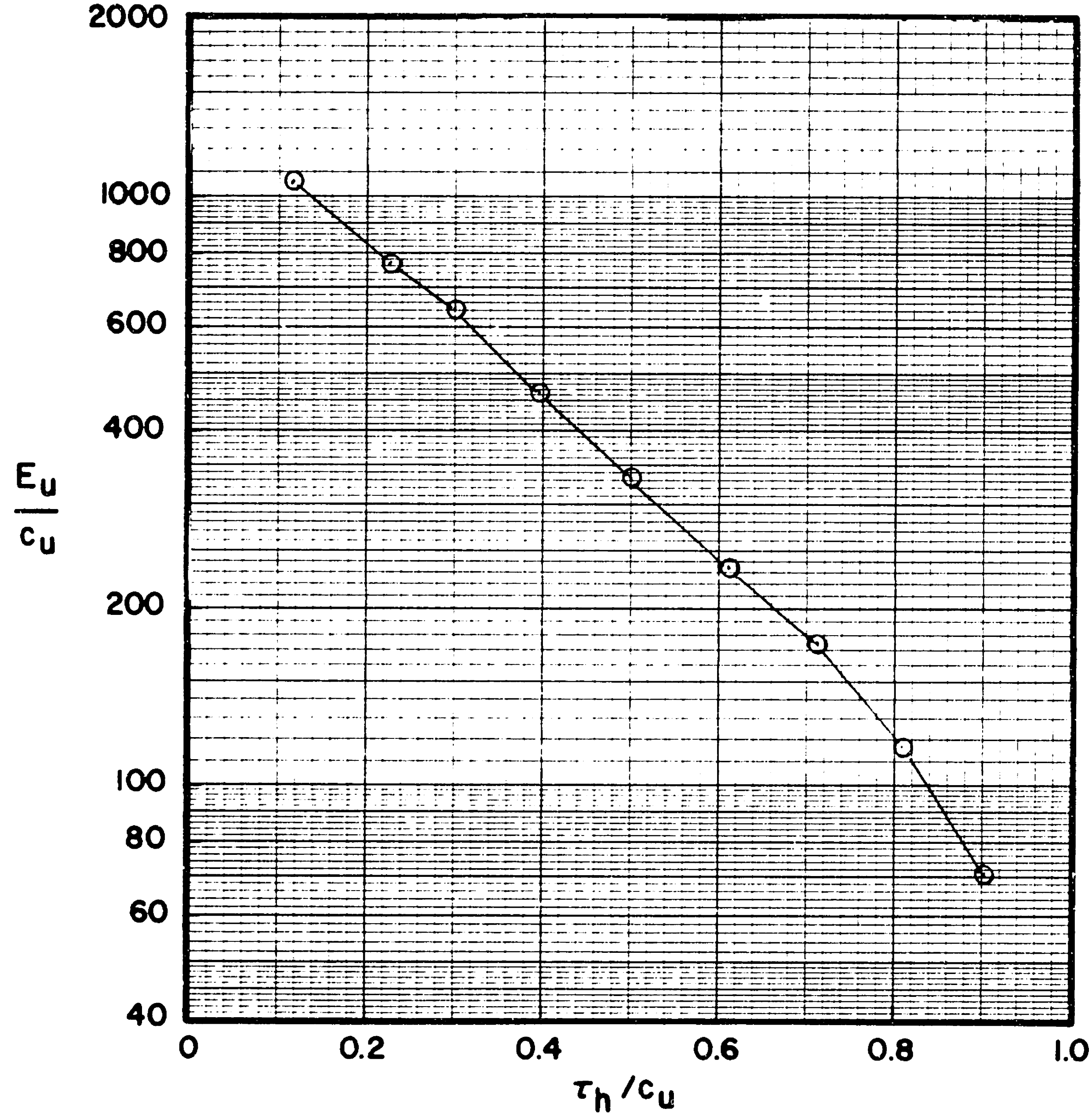
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WD391	W5B-~~P5~~	14.2'	29.2	16.4	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING W5B **SOIL TYPE** ARCTIC SILT

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



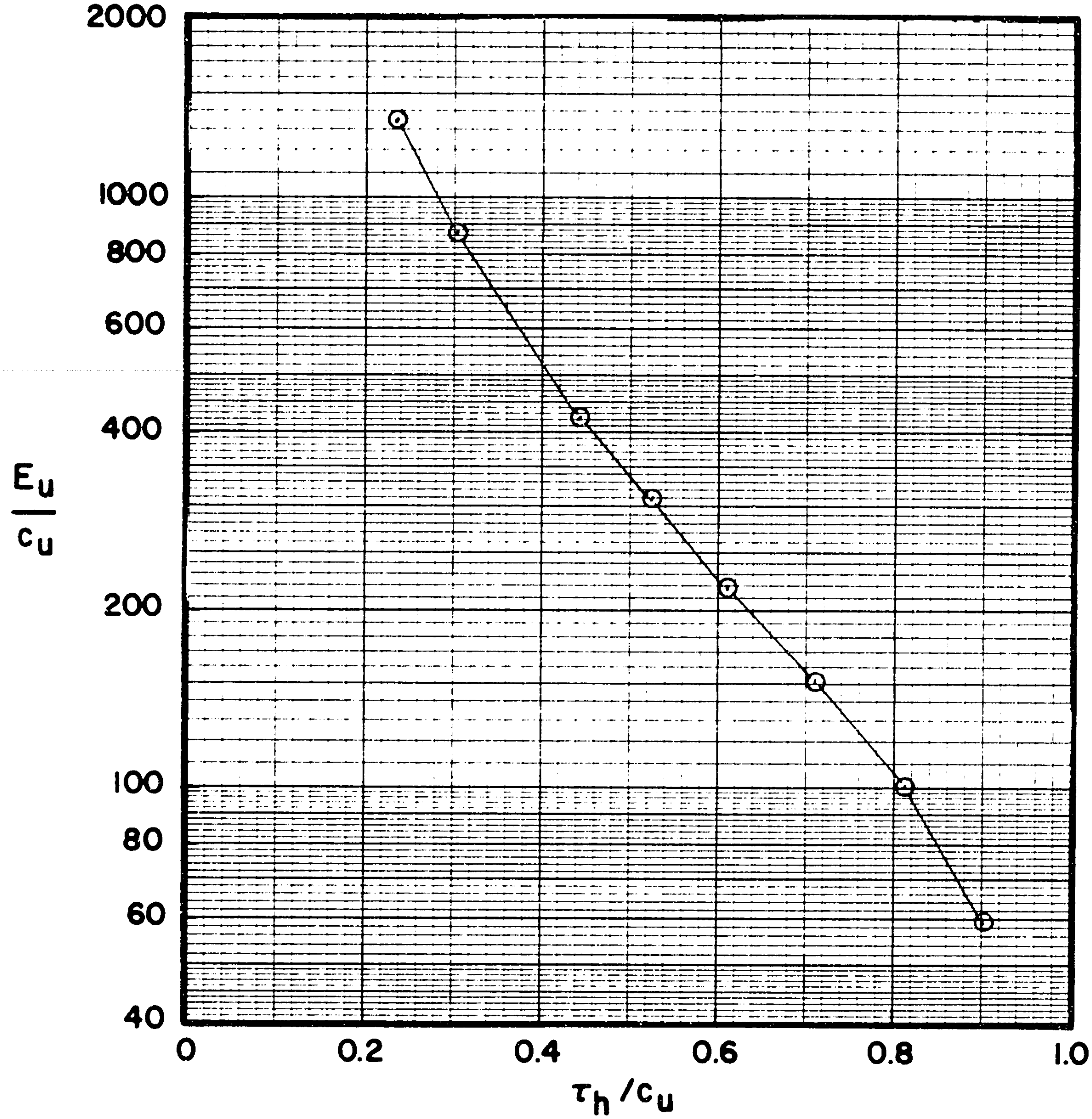
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS2	W5PV-S1	34'	41.7	9.09	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING W5B SOIL TYPE ARCTIC SILT

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WD553	W5B-P3	7'	36.6	16.4	1	

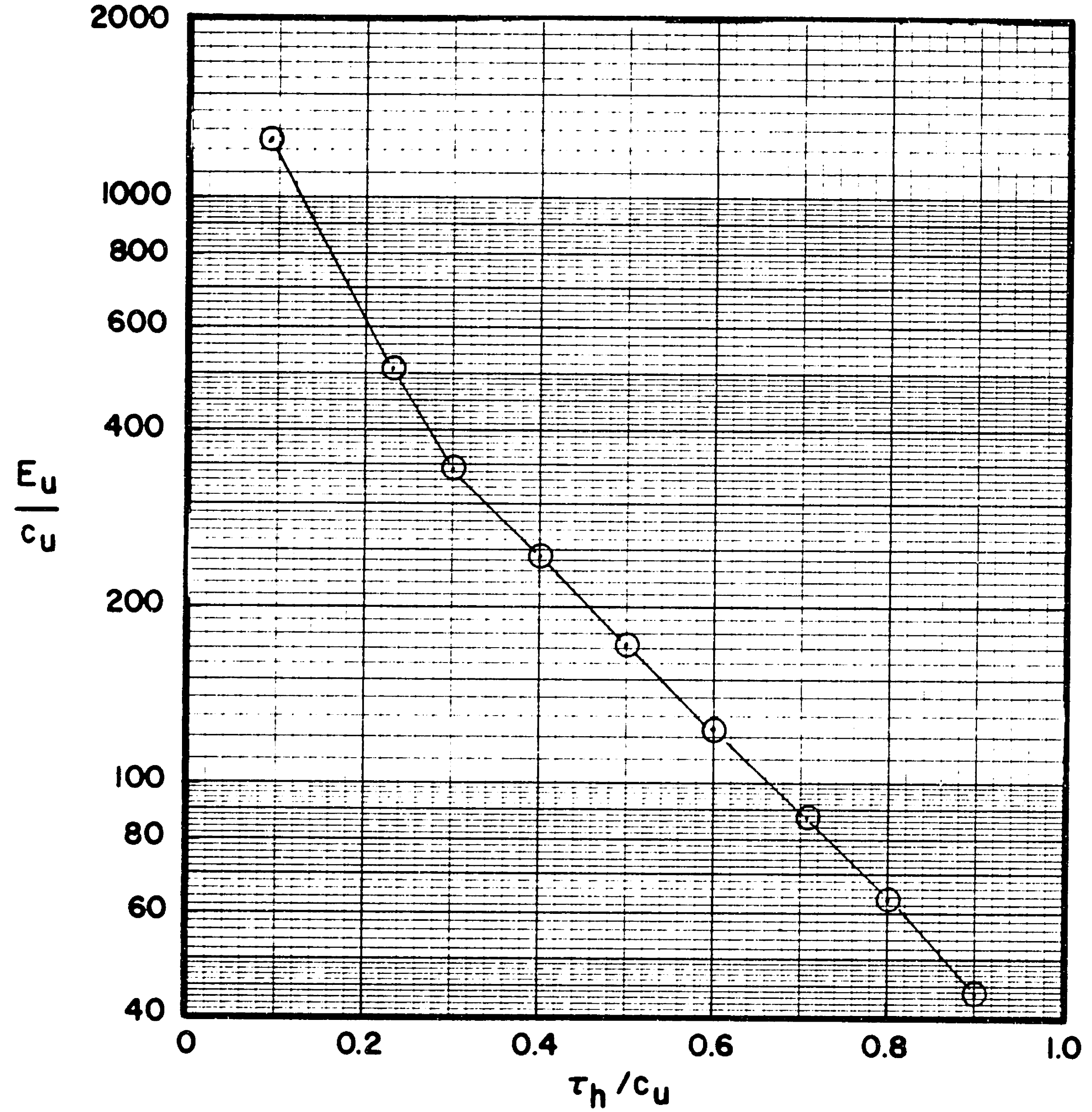
NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING W5B **SOIL TYPE** ARCTIC SILT

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

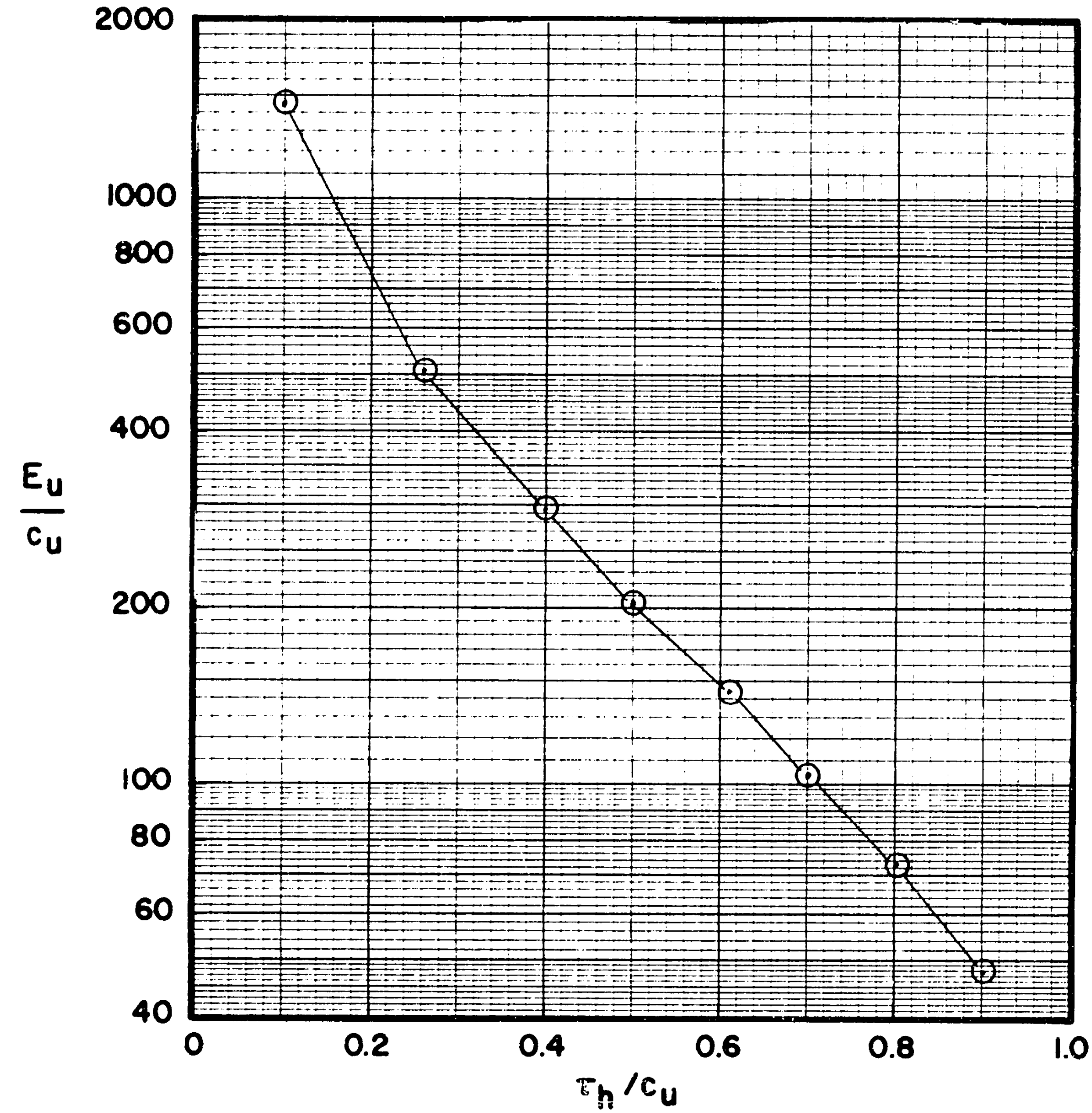


Test No.	Sample No.	Depth (FT)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS4	W5B-P4	9.2'	28.9	26.4	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING W5B-P4 **SOIL TYPE** ARCTIC SILT

FIGURE



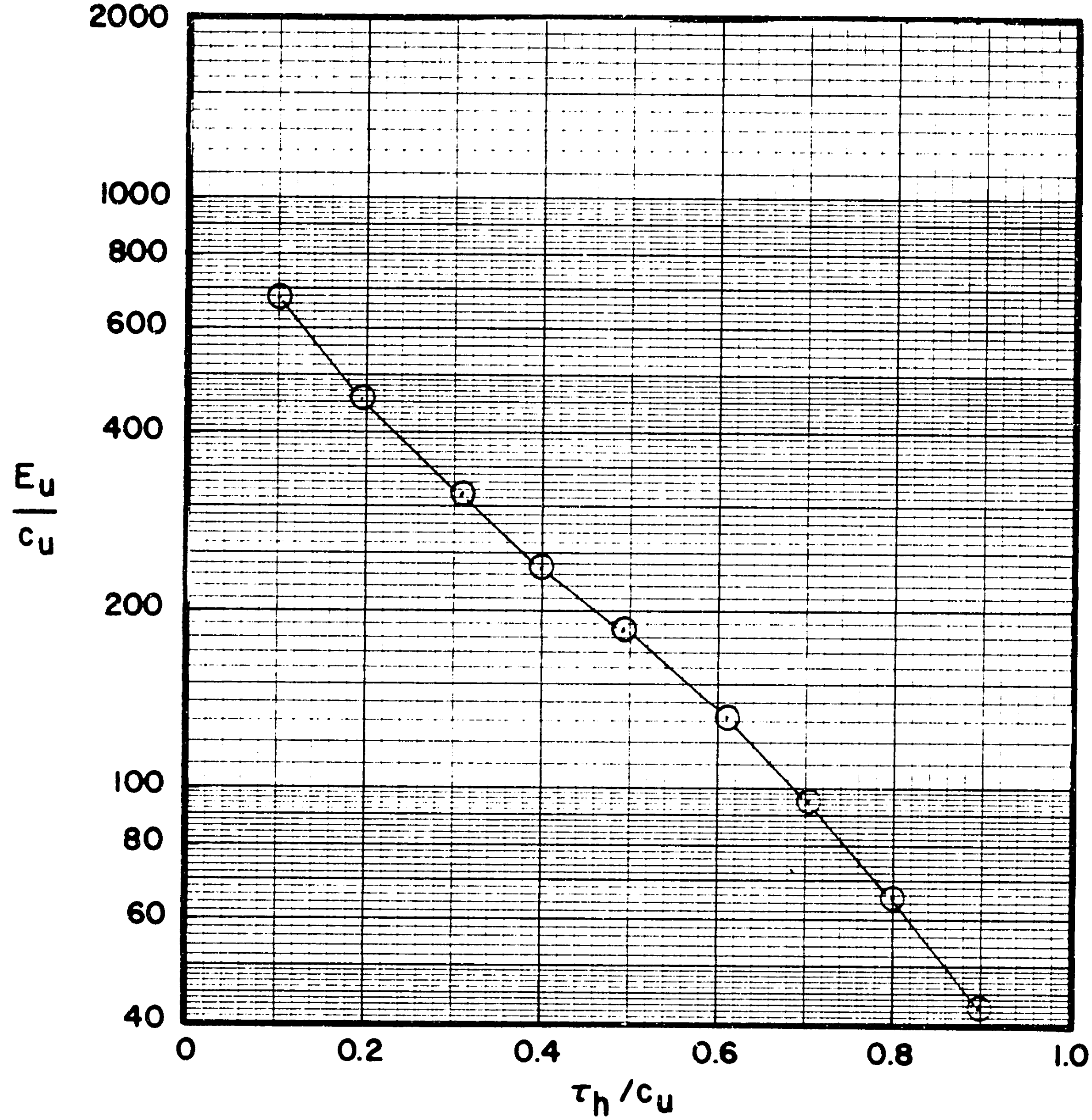
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WD335	W5B-P4	9.3′	29.4	16.4	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING W5B-P4 **SOIL TYPE** ARCTIC SILT

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



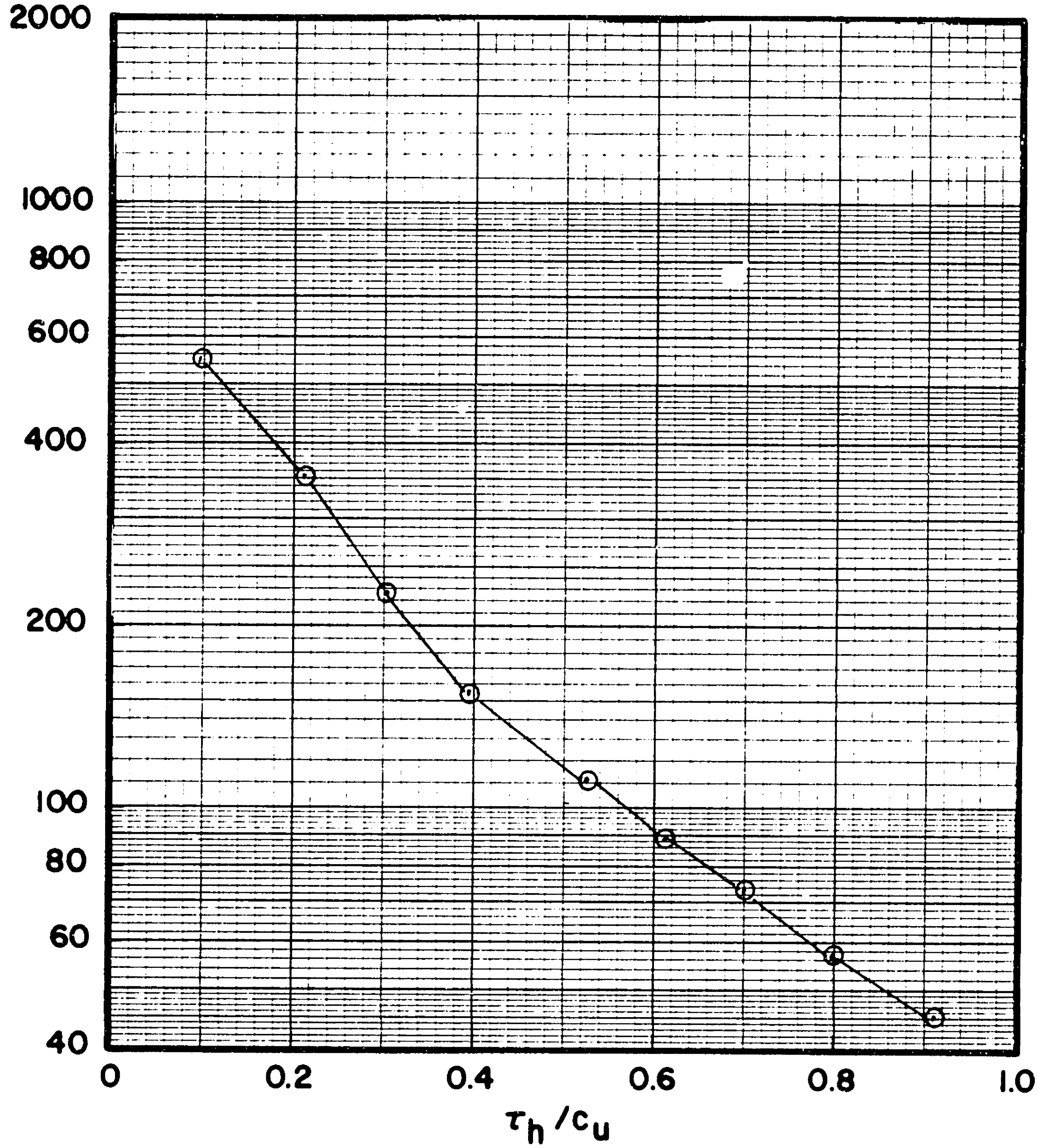
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (ksf)	OCR	Symbol
WDSS 6	W5B-P4	9.4'	30.0	24.5	1	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING W5B-P4 **SOIL TYPE** ARCTIC SILT

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

FIGURE



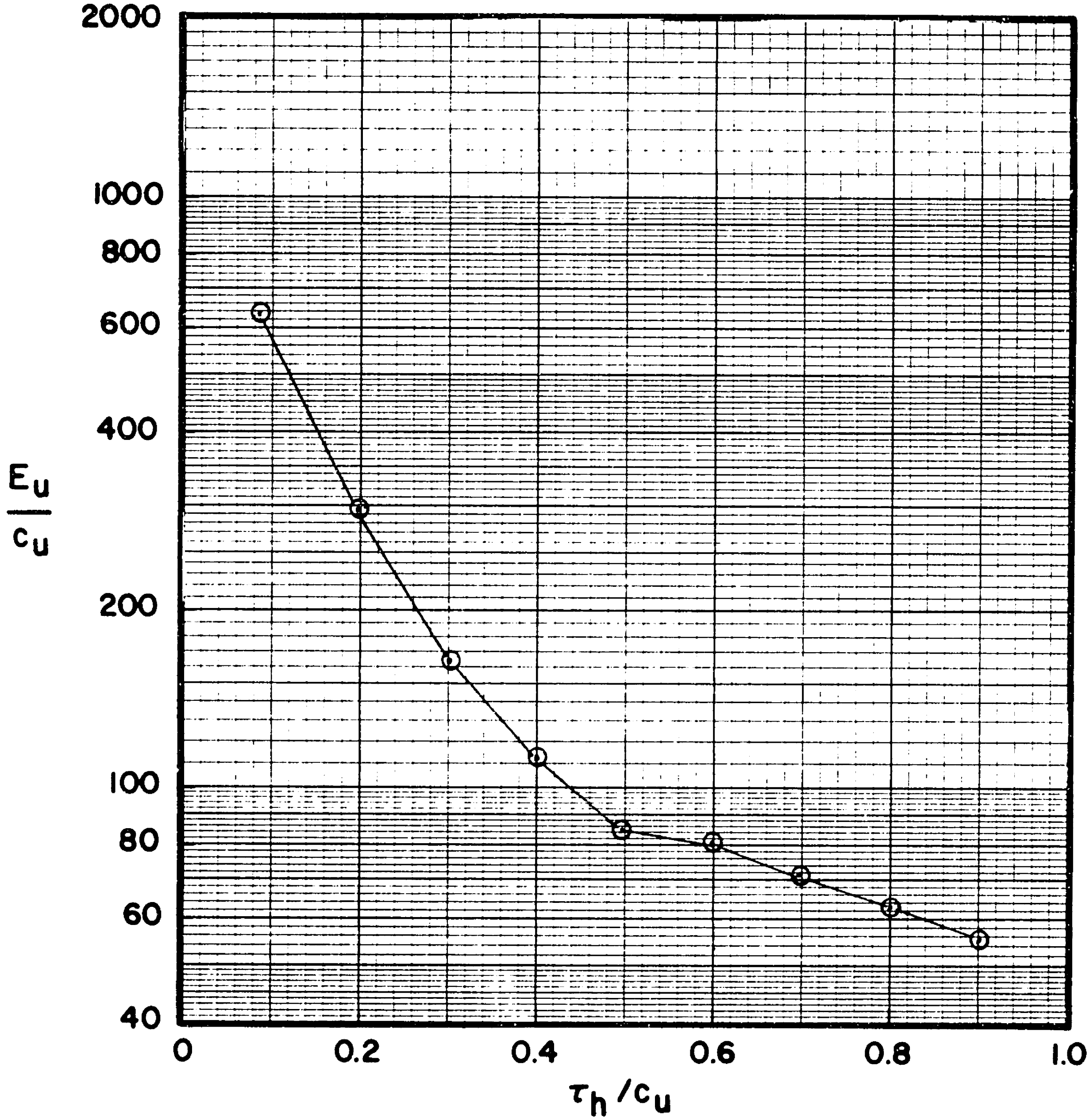
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS7	W5B-P4	9.5'	29.4	4.92	5	

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING W5B-P4 **SOIL TYPE** Arctic silt

FIGURE



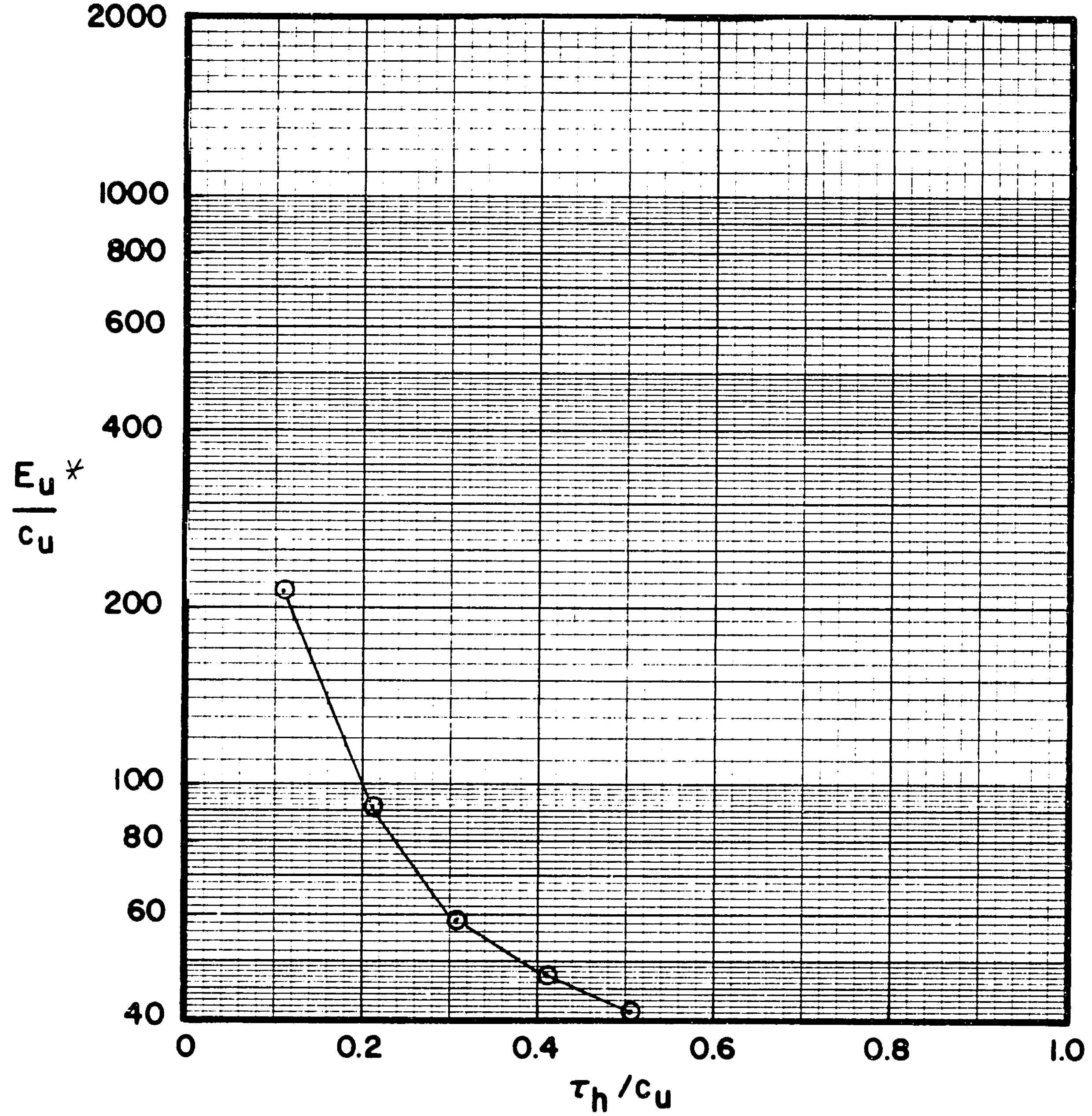
Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS8	W5B-P4	9.6'	29.0	2.44	10.09	

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING 5B-P4 SOIL TYPE ARCTIC SILT

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.



Test No.	Sample No.	Depth (RE)	w_N (%)	σ'_{vc} (KSF)	OCR	Symbol
WDSS9	WSB-P4	9.7'	28.9	1.22	20.2	

* corrected for [illegible]

NORMALIZED MODULUS FROM CK_oUDSS TESTS

BORING __________ SOIL TYPE __________

FIGURE

GEOTECHNICAL LABORATORY, DEPT. OF CIVIL ENGR., M.I.T.