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Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

REPORT
TO

DELEUW CATHER, CANADA LTD.
SUBSURFACE INVESTIGATIONS
PROPOSED SOUTH-EAST CITY
TOWNSHIP OF GLOUCESTER
REGIONAL MUNICIPALITY OTTAWA-CARLETON

VOLUME 3

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PREFACE

H. Q. Golder & Associates Ltd. and K. H. King and Associates Ltd. have been retained by DeLeuw Cather, Canada Ltd. Consulting Engineers to the Ontario Housing Corporation, to jointly carry out a geotechnical investigation at the site of a proposed new community which is being planned for a portion of the Township of Gloucester in the south-east section of the Regional Municipality of Ottawa-Carleton. Previous limited investigations had indicated that subsurface conditions would influence the planning for the new community. The purpose of this investigation was to obtain more detailed information on the subsurface conditions. On the basis of the additional information together with previous published and unpublished data relevant to the proposed site, engineering recommendations are made regarding the geotechnical factors which influence the planning of the proposed development.

The geotechnical work incorporated a variety of shallow and deep boreholes, geophysical tests, trial excavations as well as extensive laboratory testing. For convenience the information has been prepared in the following reports:

- Vol. 1 Subsurface Investigations carried out by K. H. King and Associates Ltd. (Report 312-S2-Vol. 1)
- Vol. 2 Geotechnical Mapping prepared by K. H. King and Associates Ltd. (Report 312-S2-Vol. 2)
- Vol. 3 Subsurface Investigations carried out by H. Q. Golder & Associates Ltd. (Report 73908)
- Vol. 4 Engineering Recommendations prepared by H. Q. Golder & Associates Ltd. (Report 73908-1)

Volumes 1 and 3 contain the factual information which was obtained during the field investigations and associated laboratory testing. This information has been incorporated with earlier information in the preparation of the remaining two reports.

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ABSTRACT

H. Q. Golder & Associates Ltd. have been retained by DeLeuw Cather, Canada Ltd., Consulting Engineers to the Ontario Housing Corporation to co-ordinate a geotechnical investigation at the site of a proposed new community which is being planned for a portion of the Township of Gloucester in the south-east section of the Regional Municipality of Ottawa-Carleton. This report summarizes the factual information which was obtained as a result of field and laboratory testing carried out by personnel from H. Q. Golder & Associates Ltd.

The field work for this investigation consisted of a series of five detailed sampled borings which were put down to bedrock at widely spaced locations across the site. The purpose of these borings was to provide an appreciation of the variation of soil properties with depth and to accurately determine bedrock elevations which could be used to co-ordinate the seismic work. In addition, eight test pits were excavated at various locations in order to obtain block samples for detailed laboratory testing, and to observe the behaviour of excavations.

The results of the field work indicate that underlying a weathered silty clay crust or sand cap is an extensive deposit of highly sensitive silty clay (Leda Clay). The depth to bedrock varies from about 50 ft. to over 160 ft. A blanket of glacial till underlies the Leda Clay deposit and lies directly above the shale bedrock.

The desiccated clay crust was found to be heavily overconsolidated, with the preconsolidation pressure ranging from 1800 to 7200 lb. per sq. ft. in excess of present overburden pressures. The Leda Clay is lightly overconsolidated and weakly cemented with preconsolidation pressures ranging between about 270 and 4200 lb. per sq. ft. in excess of present overburden pressures. The undrained shear strength varies between 300 and 2000 lb. per sq. ft. depending upon elevation and previous desiccation and generally increases with depth according to the relationship $s_u/p = 0.4$. The results of consolidated drained and undrained triaxial tests indicate that the Leda clay is weakly cemented at confining pressures typical of conditions below the weathered crust.

INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by DeLeuw Cather, Canada Ltd., Consulting Engineers to the Ontario Housing Corporation to co-ordinate a geotechnical investigation at the site of a proposed development which is planned for a portion of the Township of Gloucester in the south-east area of the Regional Municipality of Ottawa-Carleton, Ontario.

This report, which constitutes Volume III in the overall series of reports, describes the factual information obtained by H. Q. Golder & Associates Ltd. during the course of their part of the investigation. The results of the field work which consisted of deep borings, test pits and a test trench, are described. The engineering properties of the various strata are presented together with a brief account of the laboratory test techniques used.

SITE DESCRIPTION

The site of the proposed development lies some 5 to 10 miles south-east of the outer limits of the City of Ottawa and covers an area of about 7,000 acres. It is bounded on the north-east by Highway 417 and on the south by Regional Road 8 (Fig. 1). The site, which is sparsely populated, is understood to be marginal agricultural land with areas of dense woodland, particularly on the north-east boundary. The terrain is extremely flat with ground surface elevations lying between 250 and 270 ft. above datum. Drainage is developed by the system of tributaries associated with the main watercourse, Bear Brook, which flows northwards along the eastern flank of the site. In its latter stages within the site boundary, the valley floor in the creek is some 30 ft. to 40 ft. below the surrounding ground surface.

GEOLOGY

During the retreat of the ice sheet which occupied the Ottawa Valley in the late Pleistocene period, the area was inundated by the Champlain Sea in which marine silts and clays were deposited. It is considered that for a time, the ice front formed the northern shoreline a few miles to the east of the site. As the ice retreated, fresh water streams transported clays and silts which were eroded from inland. The consequent inflow of fresh water produced brackish water conditions in which the soils were deposited. As a result of the sedimentary environment, the marine clay deposit is the result of various stages of deposition, erosion and re-deposition cycles. An intermittent sand mantle of varying thickness, which overlies portions of the deep marine deposits, is of deltaic origin and has been built up by the Ottawa River and its tributaries. The complex of deposits described above are generally underlain by glacial deposits and Palaeozoic sandstone, shale, limestone or dolomite.

FIELD WORK PROCEDURES

The field work for this part of the investigation was carried out between January 8 and March 5, 1974. During this time a total of 5 deep borings and 7 test pits were put down at the locations shown on the Boring Plan, Fig. 2. In addition, dynamic cone penetration tests were carried out adjacent to the locations of boreholes 1 and 2. Supplementary vane tests were carried out to depths of about 20 ft. in unsampled borings beside the locations of boreholes 1, 2 and 3. The details of the borings are given in the Record of Borehole sheets which follow the text of this report. A test trench was also constructed in the period between March 11 and March 15, 1974 at a site about 100 ft. east of the location of test pit 503.

Boreholes

Boreholes were advanced through the overburden by wash boring techniques using a trailer-mounted diamond drillrig supplied and operated by F. E. Johnston Drilling Co. Ltd. of Ottawa. The boreholes extended to depths below ground surface ranging between about 59 ft. and 177 ft. and were cased using 6 in., HX, NX and BX casing sizes. On reaching bedrock, the casing was seated into the rock surface and bedrock proved for depths ranging between 3.2 ft. and 9.8 ft. in boreholes 1, 2, 3 and 5. Bedrock was cored in BX size using a core barrel equipped with a diamond studded drillbit. In borehole 4, no rock core was recovered as the bottom of the hole became blocked approximately 2 ft. below the probable rock surface with part of a broken diamond casing shoe.

Samples of the overburden were recovered during drilling operations using a number of techniques. Standard 2 in. O.D. split spoon samples were recovered in the less cohesive surficial sand mantle, which at some locations overlies the deep deposit of marine clay, and in the glacial till stratum which underlies the clay deposit. Within the marine clay deposit, samples were recovered using piston, Osterberg and conventional Shelby tube samples. The type of sampling used is shown on the Record of Borehole sheets. Thin-walled sampling tubes were used in all cases, the majority of which were 2-7/8 in. I.D. These were supplemented at depths in excess of 100 ft. by occasional 2 in. I.D. sample tubes.

The frequency of sampling varied with depth. Samples of the desiccated crust and marine clay were taken almost continuously to depths of between 15 ft. and 25 ft. below ground surface. This was followed by sampling at

about 5 ft. intervals to depths of between 23 ft. and 62 ft. Below these depths, the depth interval between samples was increased to between 7 ft. and 12 ft.

Samples recovered during split spoon sampling operations were placed in air-tight glass jars. Tube samples were waxed and sealed immediately after recovery. Bedrock cores were placed sequentially in conventional core boxes.

During split spoon sampling of the surficial sands and glacial till, standard penetration tests were carried out. In the marine clay deposit, in situ vane shear tests using both BX and NX size vanes were carried out in all boreholes at varying depth intervals to determine the variation of undrained strength with elevation.

Test Pits

Test pits were excavated to allow block samples of the desiccated crust and underlying marine clay to be taken at various elevations. They were dug to depths of between 11 ft. and 20 ft. using either a tractor-mounted backhoe, shovel or gradall. Block samples were sealed using silver foil paper and wax and were packed in wooden boxes lined with styrofoam to minimize disturbance during transportation.

During the period when the test pits remained open, which was generally less than 12 hrs., the stratigraphy was logged and observations made regarding the stability of the side walls of the excavations. The subsoil strata exposed in the test pits is shown on Fig. 3.

No major instability of the side walls of the excavations was observed in test pits 501 - 505 although

failure planes formed along vertical fissures causing blocks of soil to fall into the excavation. Groundwater seepage into these test pits occurred through layers of sand and silt in the weathered clay crust and Leda clay deposit and caused some minor caving of the side walls. In test pits 506 and 507, which were excavated in areas with a thick sand cap, considerable sloughing of the side of walls occurred because of the high groundwater level.

Test Trench

During the installation of municipal services on the site, it may be necessary for a contractor to leave an excavation open over a weekend and similar long periods. While the test pits had remained relatively stable during excavation for block samples, flowing water in the sand seams had produced minor caving which masked any short term changes in the strength of the clay walls. It was therefore decided to excavate a test trench in an area of predominantly clayey soils to determine if there might be any change in clay slope behaviour over a period of three to four days.

A test trench was excavated at a site located about 100 ft. east of test pit 503. The previous test pit indicated that there were no significant sand layers, thus obviating the necessity for continuous pumping of water at the trial stage. (During actual construction for future services, pumping and protective features may be required to satisfy construction safety regulations).

The test trench was excavated in two sections, beginning on March 11, 1974, (Figs. 4 - 6 inclusive). The first section of the trench, which was about 20 ft. by 10 ft. in plan, was taken to a depth of about 11 ft. wide vertical side walls using a tractor-mounted backhoe. About 12 hours

after excavation was complete, the north wall of the excavation experienced some instability with failure planes forming along vertical fissures and blocks of soil falling into the open excavation. This type of failure occurred at a decreasing rate until March 15, when, just before backfilling, the frozen crust at the ground surface was undercut by up to 4 ft., (Photograph 4, Fig. 6). Similar, though less severe, failures occurred on the south face about 24 hours after completion of the excavation. By March 15, the failure zone had extended about 1 ft. laterally into the wall. It is noteworthy that the large majority of the excavated soil had been placed along the top of the north wall, with only minimal surcharge loading above the south face, (Fig. 4).

On March 12, 1974, the test trench was extended using the cross-section shown in Fig. 4. This section was excavated to a maximum depth of approximately 16 ft. using a larger backhoe with a bucket capacity of 1-1/4 cu. yds. (see Fig. 5). There was no major instability along either the north or south walls over the period of three days prior to backfilling on March 15, 1974 as shown in Photographs 1 and 2 in Fig. 5. However minor instability of the type described above was apparent along about half of the vertical section of the north wall. During the time that the test trench remained open, only minor seepage of surface water into the excavation occurred.

General Notes

Groundwater level observations were made in all the open boreholes and test pits. After completion of the drilling, a total of 4 standpipes and 7 piezometers were installed and sealed into the borings to determine the stabilized groundwater level, measure changes in piezometric

head with elevations through the deposit and permit monitoring of future seasonal groundwater fluctuations. Details of the installations are given in the Record of Borehole sheets.

The majority of samples obtained during the field work were brought to our laboratory in Mississauga for detailed examination and testing. The remainder were taken to our laboratory in Ottawa to expedite the completion of the laboratory testing programme.

All the field work described above was supervised throughout by members of our engineering staff who directed the drilling and sampling operations, supervised the in situ testing and construction of test pits and test trench, logged the borings and test pits and cared for the samples obtained.

The borehole, test pit and test trench locations, together with ground surface elevations were determined in the field by DeLeuw Cather, Canada Ltd. It is understood that the elevations are referred to Geodetic origin.

LABORATORY TESTING PROCEDURES

The results of numerous studies, which have been carried out in recent years, into the engineering behaviour of highly sensitive clays have shown that the engineering properties of these soils as measured in laboratory tests are strongly influenced by variations in the test techniques. A number of these studies were carried out on the cemented Leda clays found in Eastern Canada, (Jarrett, 1967; Bozozuk, 1970) and are therefore of particular interest in the present investigation. In view of the results of this work, it was considered important to report the test techniques used in the present study to define the soil parameters.

Index Property Tests

The samples used to measure the Atterberg limits of the various soil strata were not oven-dried prior to testing. It was considered that oven-drying could cause changes to the structure of the constituent clay minerals, which in turn would have significantly affected the measurement of index properties.

Consolidation Tests

In measuring consolidation characteristics of Leda clay a number of factors such as magnitude of load increment, size of test specimen, sampling method and length of sample storage time are known to affect the measured values of preconsolidation pressure. Hamilton and Crawford, (1960), have shown that application of loads using the standard load increment ratio of $\Delta p/p = 1$ during consolidation tests on Leda clay modifies the soil structure and causes a reduction in the measured value of preconsolidation pressure. In the present investigation, a load increment ratio of typically less than 0.5 has been used at stresses in excess of the existing overburden pressure. The actual load increments which were used in the individual consolidation tests are shown on the graphical presentation of the laboratory results (Fig. 18 to 38 inclusive). The load increments were applied at approximately 24 hour intervals to allow sufficient time for consolidation to take place under each load. Primary consolidation at applied loads below the preconsolidation pressure was generally completed within a period of 0.5 to 5 minutes. At loads in excess of preconsolidation pressure the primary consolidation took place within 0.5 to 34 minutes after the loading was applied. Thus the resulting void ratio-pressure curves also include deformations resulting from small secondary consolidation movements.

When the applied stresses reached a value close to the preconsolidation pressure, the majority of samples were off-loaded to stresses below the existing overburden pressure. The subsequent reloading curve gives a more accurate indication of the compression index during reloading than is given by the results of the initial loading cycle.

Two inch and five inch diameter test specimens were used in the present investigation, and the test sample size is indicated in Table 1. A detailed series of tests carried out on tube samples recovered from the site at CFB Gloucester, about 2 miles south west of the area, indicates that 2 inch diameter samples may result in an underestimation of the preconsolidation pressure by less than 6 percent (Bozozuk and Leonards, 1972). Earlier work by Eden (1970) in the comparison of consolidation tests from both block and tube samples has indicated that the preconsolidation pressure as determined from a block sample is up to about 2500 lb/sq. ft. higher than the values measured in specimens trimmed from tube samples obtained from about the same elevation.

All test specimens were trimmed from blocks or sample tubes which had been stored for periods of less than two months.

Triaxial Compression Tests

Triaxial compression tests were carried out on 2 in. diameter samples 4 in. high. Consolidated drained and undrained tests were carried out on samples trimmed from block samples recovered from test pits. Unconsolidated undrained test samples were trimmed from both block and tube samples. All samples were trimmed using a wire saw and soil lathe to minimize disturbance during sample preparation.

A back pressure was applied to samples prior to the consolidation stage to ensure full saturation of the soil. Drainage was facilitated through top and bottom porous stones combined with lateral strip filter drains.

Consolidated undrained triaxial tests were carried out using both isotropic and anisotropic loading during consolidation. A comparison of the test results obtained in an earlier study shows that although there is little difference in undrained strength of the samples having different consolidation history, the strain at failure and porewater pressure response to loading are significantly affected, (Bozozuk and Leonards, 1972).

The shearing stage of consolidated undrained tests was carried out at a strain rate of 0.25 percent/hr. to allow equalization of porewater pressure through the test samples. Porewater pressures were measured using both an electronic pressure transducer and no-volume-change null indicators.

Unconsolidated undrained tests were carried out at various strain rates in order to assess the effect of varying strain rate on undrained strength. In some samples the initial strain rate was reduced after maximum deviator stress was reached to allow a more accurate appraisal to be made of the post peak stress-strain behaviour. All unconsolidated undrained triaxial tests were carried out using lubricated end platens.

Effect of sampling method on measured soil properties

A number of recent studies have examined the variation in soil properties, such as preconsolidation pressure and undrained strength, as measured in the laboratory

tests on samples of cemented clays obtained using different sampling techniques. (Eden 1970; Raymond, Townsend and Lojkasek, 1971). From the results of these tests it is apparent that there is considerably less disturbance associated with block sampling than with other forms of tube samples. In the latter case, it has been shown that Osterberg samples which incorporate a device to prevent overdriving of the sample tube, provide better undisturbed samples than conventional Shelby tube samples.

In the present study all consolidated undrained and drained triaxial tests, together with a limited number of consolidation tests were carried out on specimens trimmed from block samples. However, the soil properties at greater depth were measured using specimens trimmed from 2-7/8 in. dia. tube samples, the majority of which were recovered using an Osterberg sampler.

SUBSURFACE CONDITIONS

The detailed stratigraphy encountered at each of the boreholes is given on the Record of Borehole sheets following the text of this report. Figure 3 summarizes the stratigraphy encountered during excavation of the test pits.

The stratigraphy at each borehole is similar. Underlying an intermittent topsoil cover, a crust of weathered clay or sand cap overlies the predominant deposit at the site which consists of highly sensitive silty clay known as Leda clay. This deposit is underlain by a glacial till layer which is in turn underlain by shale bedrock. The detailed description and soil properties of the individual strata are given in the following sections. A summary of soil properties of the surficial crust deposits, Leda clay and glacial till is given in Fig. 7. Included are plots of Atterberg limits,

natural moisture content, bulk density, undrained shear strength and preconsolidation pressure against elevation.

Surficial Crust Deposits

Underlying dark brown sandy topsoil of average thickness 6 in. is a zone of highly stratified soil consisting of various proportions of weathered, fissured, red-brown silty clay, silts and sands. The actual composition of the crustal material varies from yellow brown silty sand to sand with traces of silt at the locations of test pits 506 and 507 in the northeast corner of the site, to highly weathered and fissured red-brown sensitive silty clay containing layers of grey silt and fine sand at the location of Borehole 1. A comparison of the stratigraphy encountered at borehole locations and in the nearby test pits shows that the composition of the surficial crustal deposits can vary considerably over a short distance, as is the case for example at the location of borehole 1 and test pit 501.

Where sand is the predominant soil type of crust material, it is in a very loose to loose state with 'N' values varying between 2 and 10. A typical grain size distribution curve for the surficial sands is shown in Fig. 8.

The thickness of the weathered clay crust or sand cap was found to be in the range between 3-1/2 ft. to 17 ft. Further information on the predominant soil type in the upper 4.5 ft. and the thickness of the surficial deposits is given in Volume II.

Properties of Desiccated Silty Clay Crust: The variability of the weathered clay crustal zone is apparent

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from an examination of the results of classification tests shown on the Record of Borehole Sheets. The measured plasticity index was found to vary from non-plastic to 58. The results of the Atterberg limit tests plot close to and consistently above the Cassagrande 'A' line on the Plasticity Chart, (Fig. 15).

Further confirmation of the highly stratified nature of the crust deposits is given in the results of classification tests on samples taken from the various layers in a 3 in. depth interval of a block sample recovered from within the crust at the location of test pit 505.

Depth (ft.)	Visual Description	Range of Moisture Content (%)	Liquid Limit	Plastic Limit
5.0 - 5.05	Brown silty Clay	53 - 63	70	25
5.05- 5.15	Red Silty Clay	70 - 80	84	27
5.15	Grey clayey silt	30	-	-

The grain size distribution curves for these three strata are shown on Fig. 9.

The undrained strengths obtained by field vane tests and unconsolidated undrained triaxial tests on samples of the weathered fissured silty clay lie in the range between 480 and 1540 lb. per sq. ft. The general pattern in test results at

each borehole, of decreasing undrained strength through the crust material is typical of desiccated crust strata. The undrained strength of about 900 lb. per sq. ft., which was measured in an undrained triaxial test on a sample which had previously consolidated at about the effective overburden pressure, lies within the same range of values measured in quick triaxial and field vane tests. The results of quick triaxial tests (Fig. 16) carried out on specimens trimmed at varying angles to the horizontal from a block sample of weathered silty clay, indicate that the maximum deviator stress is essentially independent of direction. Thus the undrained shear strength can be considered as isotropic for the purposes of design.

The results of all consolidation tests are given in Figs. 18 to 38 and are summarized in Table I, and Fig. 7. The desiccated crust sample results are differentiated in the summary plot (Fig. 7), and the individual tests are indicated on Figs. 24, 34, 35 and 38. There is possibly some variation in the results as determined between 2 in. and 5 in. diameter samples, but this variation is less than the range between the lowest and highest values which were observed in the 2 inch diameter tests. The desiccated crust is heavily over-consolidated with values of $p_c - p_o'$ lying in the range between 1800 and 7200 lb. per sq. ft. The wide variation in test results is to be expected in a variable and fissured material of this nature. Only small strains accompanied applied stresses up to the preconsolidation pressure with the range of compression index, C_{cr} , in the reload stage typically in the order of 0.03. At stresses above the preconsolidation pressure the strain response to loading increased significantly with values of the compression index, C_c , in the normally consolidated part of the curve lying in the range between 1.18 and 2.58.

Four consolidated drained triaxial tests were carried out on specimens from a block sample of weathered silty clay taken from test pit 501 (Fig. 39). The samples were subjected to loading along predetermined stress paths to define the locus of stress states which caused breakdown of the interparticle cementation bonding.

The soil response to applied stresses lying within this locus is essentially 'elastic'. Values of the principal stress ratio, (σ_1/σ_3) , lying between 0.4 to 0.56 were found to cause only small area changes of less than about 0.5%. The 'elastic' behaviour is further confirmed by the magnitude of the pore water pressure parameter, A, of about 0.3 measured in an undrained test on a sample isotropically consolidated to approximately the effective overburden pressure, (Fig. 40). The results of this test, which simulates the reaction of the soil to applied loading in the field, indicate that the Young's Modulus, E, for this material is of the order of 120 tons per sq. ft. A value of 480 tons per sq. ft. was measured in an earlier study on samples of desiccated crust recovered from the Ottawa area, (Mitchell, Sangrey and Webb, 1972).

Leda Clay

Underlying the weathered clay crust or sand cap across the site is an extensive deposit of highly sensitive grey silty clay, known as Leda clay, which ranges in thickness between 45 ft. and 149 ft. at the borehole locations. This material is stratified at shallow depths below the surficial deposits, containing layers of grey silty sand, sand and silt up to 1 ft. thick. With increasing depth, the deposit becomes less variable although pockets or layers of black organic clay were evident throughout the entire depth of the deposit.

Properties of the Leda Clay: The plasticity index of the Leda clay deposit was found to range between non plastic for sand layers within the deposit to values in excess of 60. These values plot close to and above the Cassagrande 'A' line on the Plasticity Chart, Fig. 15. The natural moisture content of the deposit ranges between 21 and 101 percent resulting in a liquidity index which is typically greater than 1. The results of a detailed series of classification tests carried out on a 4 in. high sample recovered from sample 3, borehole 2 are shown below. The pattern of test results gives a clear indication of the variability of the Leda clay at shallow depths.

Depth (ft.)	Visual Description	Natural Moisture Content %	Liquid Limit	Plastic Limit
5.3	Grey sandy silty clay	61	39	18
5.4	Light grey silty clay	55	64	24
5.5	Red-grey silty clay	74	71	23
5.6	Grey-black organic silty clay	69	-	-
5.7	Light grey silty clay to clayey silt	47	49	22

Typical grading curves for Leda clay and sand strata within the deposit are shown in Figs. 10 to 13. Again the variability of the deposit is indicated by the differences in grain size distributions of the various strata which comprise a 4 in. high sample taken from a depth of 10.3 -10.7 ft. in borehole 1, Fig. 11.

The undrained strength of the deposit was measured using field vane tests and unconsolidated undrained triaxial tests. The results show that the strength increases from a minimum of about 300 lb. per sq. ft. immediately under the crust to values in excess of 2000 lb. per sq. ft. at depths greater than 100 ft. below the ground surface. Values of $\Delta u/p$ ranged between 0.17 and 0.6, with an average value of 0.4. Although an overall strength increase with depth is typical of this type of deposit, it is apparent from the test results summarized in Fig. 7 that there are localized zones of higher shear strength within the deposit.

The results of quick triaxial tests on specimens trimmed from a block sample from test pit 502 at a depth of about 5 ft. below the crust (Fig. 16) indicate that the undrained shear strength is independent of sampling direction. The average value of about 600 lb. per sq. ft. is somewhat higher than the average of the field vane strengths at similar depths and indicates the degree of disturbance which could be associated with the in situ vane tests in this type of material.

In order to examine the effect of rate of testing on the undrained shear strength of the Leda clay, quick tests were carried out at strain rates varying between 0.03 and 1.5 percent per hour. The results indicate that there was no significant influence on undrained strength due to varying strain rates. In a number of quick triaxial tests the strain rate was reduced immediately after maximum deviator stress was attained. The stress strain curves for these tests showed that after failure occurred there is a work softening effect in that the deviator stress decreased from the maximum value at yield. (See also Fig. 17).

The results of consolidation tests carried out on undisturbed samples of the Leda clay deposit are given in Figs. 18 to 38, and summarized in Table I. The test results indicate that the deposit is lightly over-consolidated with values of $p_c - p_o'$ lying in the range of 270 to 4180 lb. per sq. ft. However, when plotted against elevation, Fig. 7, it is apparent that there may be zones within the deposit in which the clay approaches a nearly normally consolidated state. The coefficient of consolidation, C_c , was found to range between 0.52 and 3.45 while the average reload coefficient of consolidation, C_{cr} , is about 0.03. (A similar pattern of results is apparent in the published data from a detailed study carried out on samples recovered from a site about 2 miles to the southwest of the area being considered in the present investigation (Bozozuk and Leonards, 1972)).

A total of 5 consolidated undrained triaxial tests with pore pressure measurements were carried out on undisturbed samples trimmed from block samples recovered from test pit 503 and the results are summarized in Figs. 41 and 42. The undrained strengths measured in these tests lie in the range between 630 and 660 lb. per sq. ft. and are typically greater than the values measured in unconsolidated undrained triaxial and field vane tests at the same elevation. The higher values probably reflect the lower degree of disturbance associated with careful sample preparation and testing techniques. Values of the pore water pressure parameter A_f at failure were found to be in the range between 0.08 and 0.59 with an average value of 0.33. The average value of Young's Modulus, E , as interpreted from the graphs of deviator stress vs. axial strain during cyclically loaded consolidated undrained tests, was about 180 tons per sq. ft.

The locus of stress states which caused breakdown of the cementation bonds in the soil was determined by a series of nine consolidated drained triaxial tests. These samples were subjected to loading along stress paths and at stress levels which are similar to those existing in the field during different phases of construction, Figs. 43 to 45.

Samples reconsolidated at low effective stresses which are representative of the in situ field condition, 'yielded' at relatively low strains prior to rupture. In order to emphasize and illustrate the yield behaviour, the logarithm of the deviator stress has been plotted against axial strain on Figs. 43 to 45. The distinct yielding prior to rupture was also accompanied by a change in pore pressure response at slightly lower strains.

Both the yield and rupture values have been indicated on Fig. 46, which is a summary of all consolidated undrained and drained triaxial tests on the Leda clay. At the lower stress range, there is an apparent curved failure envelope due to cementation in the soil, while at higher stress ranges, the shear strength increases with increase in confining pressure similar to the pattern for normally consolidated soils. Within the low stress range, the average deviator stress is in the order of 650 lb. per sq. ft., while the limited number of tests in the high stress range indicate effective stress parameters of about $c' = 180$ lb. per sq. ft. and $\phi' = 26.5$ degrees.

The cementation envelope is within the same range as the undrained shear strength as determined from block samples (see Fig. 16) but above in situ values as determined by the field vane, or the results of unconsolidated

undrained tests on tube samples, (see Fig. 7). This may be related to the care in sampling and testing of the block samples.

The selection of suitable design strength parameters is discussed in relation to published values from adjacent sites in Volume IV.

Glacial Till

Glacial till, which varies in composition from a very dense red-brown to grey sand and gravel to silty sandy clay underlies the Leda clay deposit at the five borehole locations. At the locations of Boreholes 1, 4 and 5, the till contains numerous cobbles and boulders up to a maximum diameter of 2-1/2 to 3 ft. The thickness of the till blanket was found to vary between 4 ft. and 17 ft. From the results of standard penetration tests, the relative density is generally dense to very dense or hard. Typical grading curves are shown in Fig. 14.

Bedrock

Red-brown and grey shale bedrock of the Queenston and Carlsbad formations respectively underlies the glacial till at each of the five borehole locations. The shale is interbedded with occasional layers of harder dolomite and in boreholes 3 and 5, clay seams up to 2 in. thick were detected. The upper 3 to 7 ft. of the bedrock is in a highly fractured state although becoming sounder with depth.

Groundwater Conditions

Observations of water levels in open borings during drilling and in the standpipes and piezometers installed on completion of the borings indicate that the groundwater

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level across the site was at a depth of between 2 to 3 ft. below ground surface during the period when the field work was carried out. The groundwater level readings, which were taken at periodic intervals during the course of the fieldwork, did not vary significantly with time. From the readings taken in piezometers placed at different elevations through the overburden deposits, it is apparent there is no significant variation in the stabilized groundwater level with elevation.

It was noted that during a period of relatively high air temperatures, water released by melting snow and ice caused widespread ponding of surface water in many areas of the site.

Selection of Representative Design Values

As a part of the laboratory work associated with the shallow boring program (See Vol. I), a series of routine consolidation tests were carried out. In addition, previous investigations at CFB Gloucester and for the structures along Hwy. 417 have also reported the results of detailed strength and consolidation tests. Although these borings are at widely spaced intervals, a single overall pattern in the subsurface parameters can be developed for preliminary planning design.

The results of the individual tests reported in this volume have been combined with these additional results in the selection of typical minimum design values, and this information is discussed in Volume IV.

H. Q. GOLDER & ASSOCIATES LTD.

D. L. Townsend, P.Eng.

J. H. A. Crooks

JHAC/jb
73908

May 17, 1974

Golder Associates

ORIGINAL SIGNED BY

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TABLE 1

SUMMARY OF CONSOLIDATION TEST RESULTS

BOREHOLE OR TEST PIT NUMBER	SAMPLE NUMBER	DEPTH (ft.)	ELEVATION	OVER- BURDEN PRESSURE P_o' lb/sq.ft.	PRECONSOLIDATION PRESSURE		C_{cr}	C_c	e_o	w %	w_L	w_p	γ lb/ cu.ft.
					RANGE P_c lb/sq.ft.	PROBABLE $P_c - P_o'$ lb/sq.ft.							
1	5	10.2	252.8	620	1160-1440	680	0.13	0.52	1.30	48	33	14	110
	8	16.4	246.6	850	1680-1800	890	0.04	2.55	2.27	83	69	25	96
	13	36.5	226.5	1610	2600-2840	990	0.05	2.30	3.00	84	79	27	96
2	7	12.0	257.0	660	1440-1780	1040	0.065	1.44	1.91	71	51	21	99
	11	21.0	248.0	1000	1900-2200	1200	0.022	1.61	2.06	76	73	25	97
	16	49.8	219.2	2100	2540-3080	700	0.02	3.45	2.40	90	87	24	94
3	2	6.5	247.9	400	2200-2600	1800	0.03	1.18	1.89	74	80	21	102
	6	17.0	237.4	810	1600	790	0.035	0.55	1.16	45	86	28	113
	11	48.0	206.4	2070	3300-4000	1230	0.018	1.42	1.40	53	46	20	107
4	6	11.5	252.7	670	1760-2200	1330	0.019	1.12	1.88	68	61	25	100
	9	18.5	245.7	920	1980-2360	1080	0.021	0.42	1.27	47	51	23	112
	13	34.0	230.2	1500	2020-2140	520	0.021	2.74	2.31	77	77	22	94
	19	66.0	198.2	2680	3220-4000	920	0.012	1.35	1.87	67	80	29	98
5	5	16.0	248.8	830	1000-1200	270	0.0	1.59	2.17	79	55	23	97
	13	76.0	198.8	3110	4200-5000	1890	0.018	1.60	1.99	73	69	27	99
	15	96.0	178.8	3870	5300-7000	2130	0.015	0.95	1.50	57	65	24	108
501	1 (2")	5.9	257.3	440	2160-2480	1940	0.041	1.59	2.14	78	80	22	96
	1 (5")	5.9	257.3	440	5500-5900	5060	0.015	2.58	2.02	73	82	26	97
503	2 (5")	9.5	243.3	520	4700-5100	4180	0.0	3.26	2.18	80	-	-	96
	3 (2")	14.1	238.7	700	1940-2040	1300	0.028	3.05	2.20	-	-	-	-
505	1 (5")	5.1	259.2	400	7500-8800	7200	0.01	1.30	1.47	54	70	25	-

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
DS Denison type sample
FS foil sample
RC rock core
ST slotted tube
TO thin-walled, open
TP thin-walled, piston
WS wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer
PH sampler advanced by pressure—pressure, hydraulic
PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

<i>Relative Density</i>	<i>N, blows/ft.</i>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

<i>Consistency</i>	<i>c_u, lb./sq. ft.</i>
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer¹
Q undrained triaxial²
R consolidated undrained triaxial²
S drained triaxial
U unconfined compression
V field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

π	$= 3.1416$
e	$=$ base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	shear strain
ν	Poisson's ratio (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) Unit weight

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index $= (w - w_P) / I_P$
I_C	consistency index $= (w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density $= (e_{max} - e) / (e_{max} - e_{min})$

(c) Permeability

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
k	coefficient of permeability
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change $= -\Delta e / (1+e) \Delta \sigma'$
C_c	compression index $= -\Delta e / \Delta \log_{10} \sigma'$
c_s	coefficient of consolidation
T_v	time factor $= c_s t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_i	sensitivity

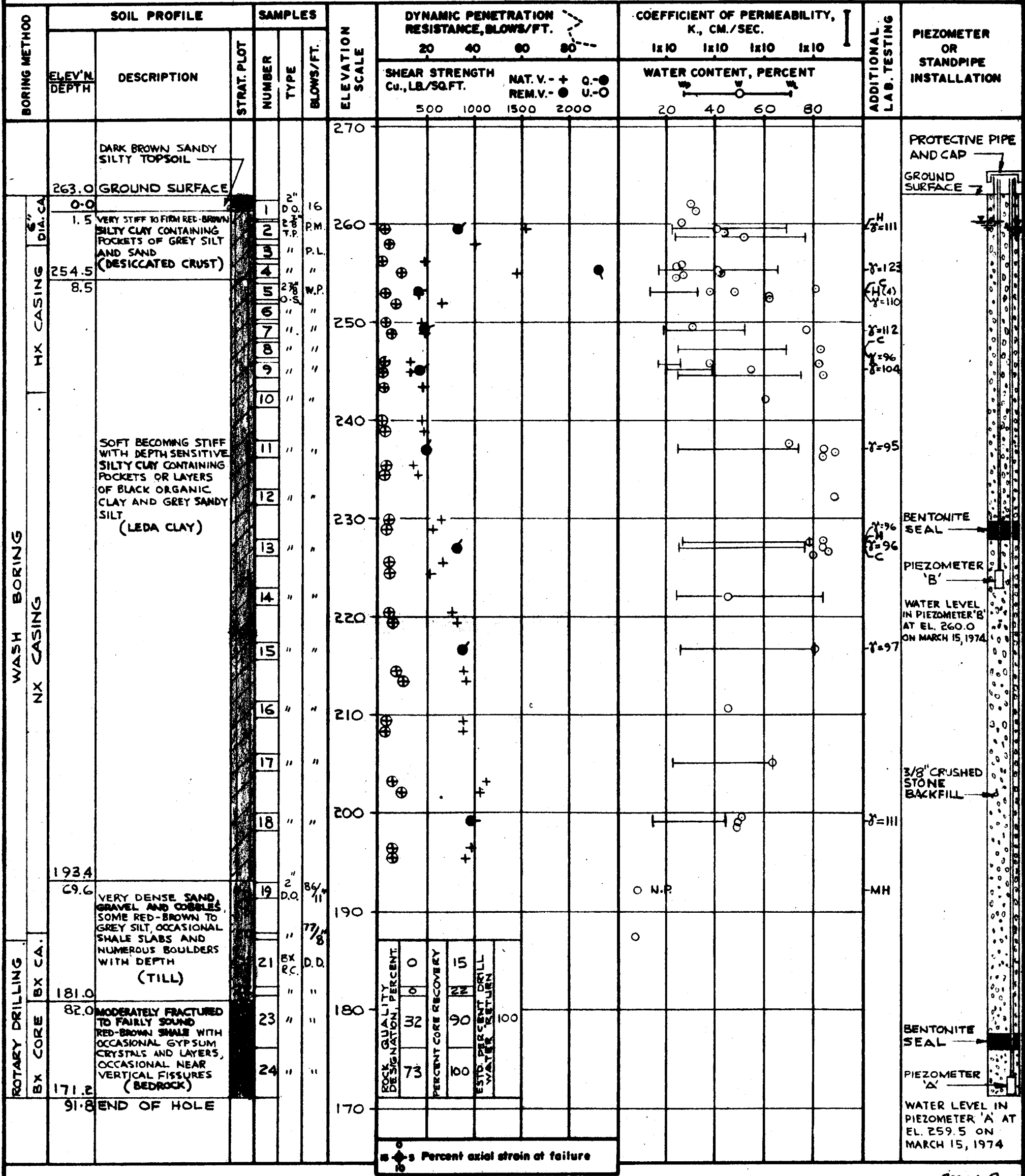
$\left. \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array} \right\}$

$\left. \begin{array}{l} \text{in terms of total stress} \\ \tau_f = c_u + \sigma \tan \phi_u \end{array} \right\}$

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 1

LOCATION See Figure 2 BORING DATE JANUARY 8-16, 1974 DATUM GEODETIC
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



LOCATION

See Figure 2

DRIVING DATE JANUARY 8, 1974

DATUM

GEODETIC

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST No.

VERTICAL SCALE
1 IN. TO 10 FT.

Goldier Associates

DRAWN M. J. B.
CHECKED J. H. C.

RECORD OF BOREHOLE 2

LOCATION See Figure 2

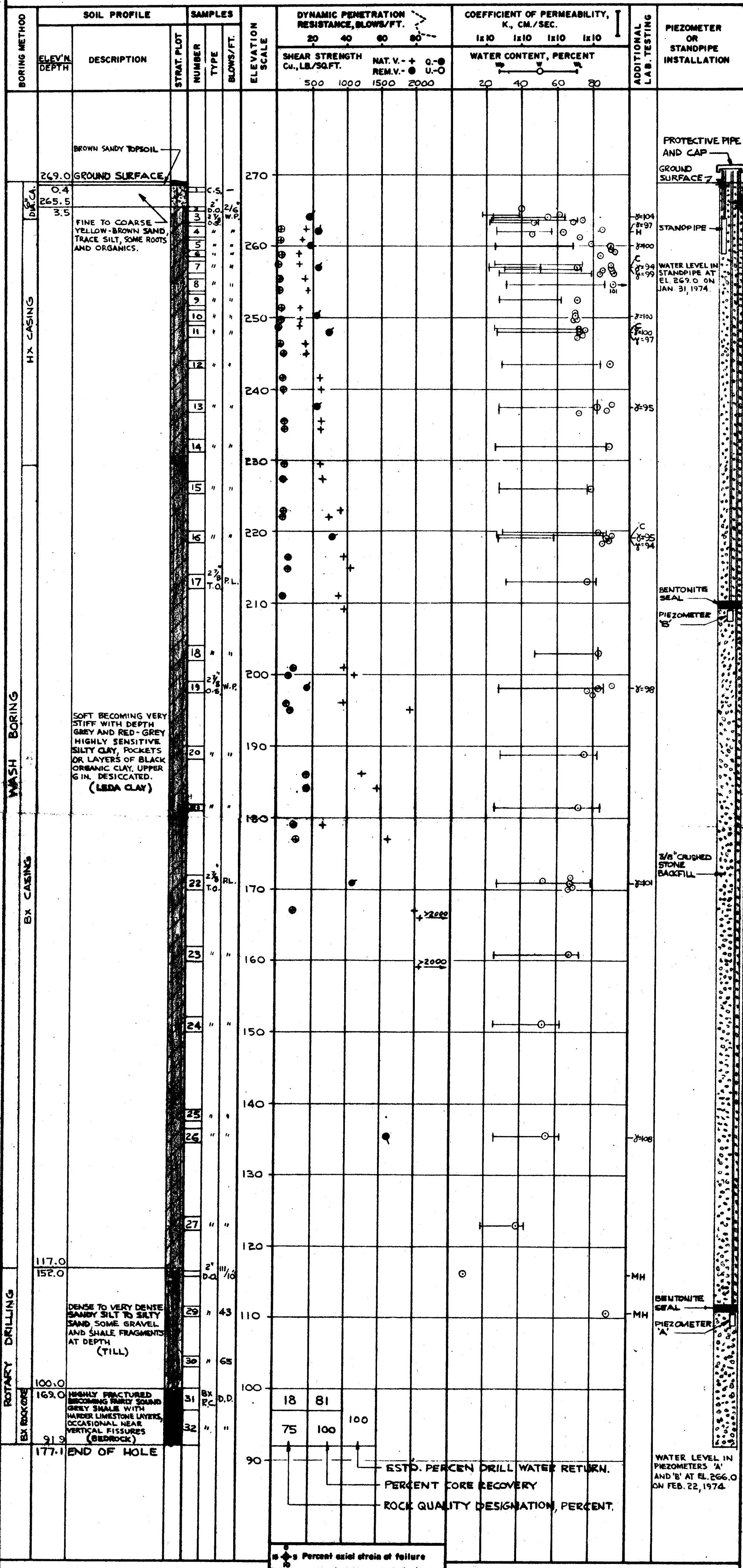
BORING DATE JANUARY 17-31, 1974

DATUM

GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



RECORD OF BOREHOLE 3

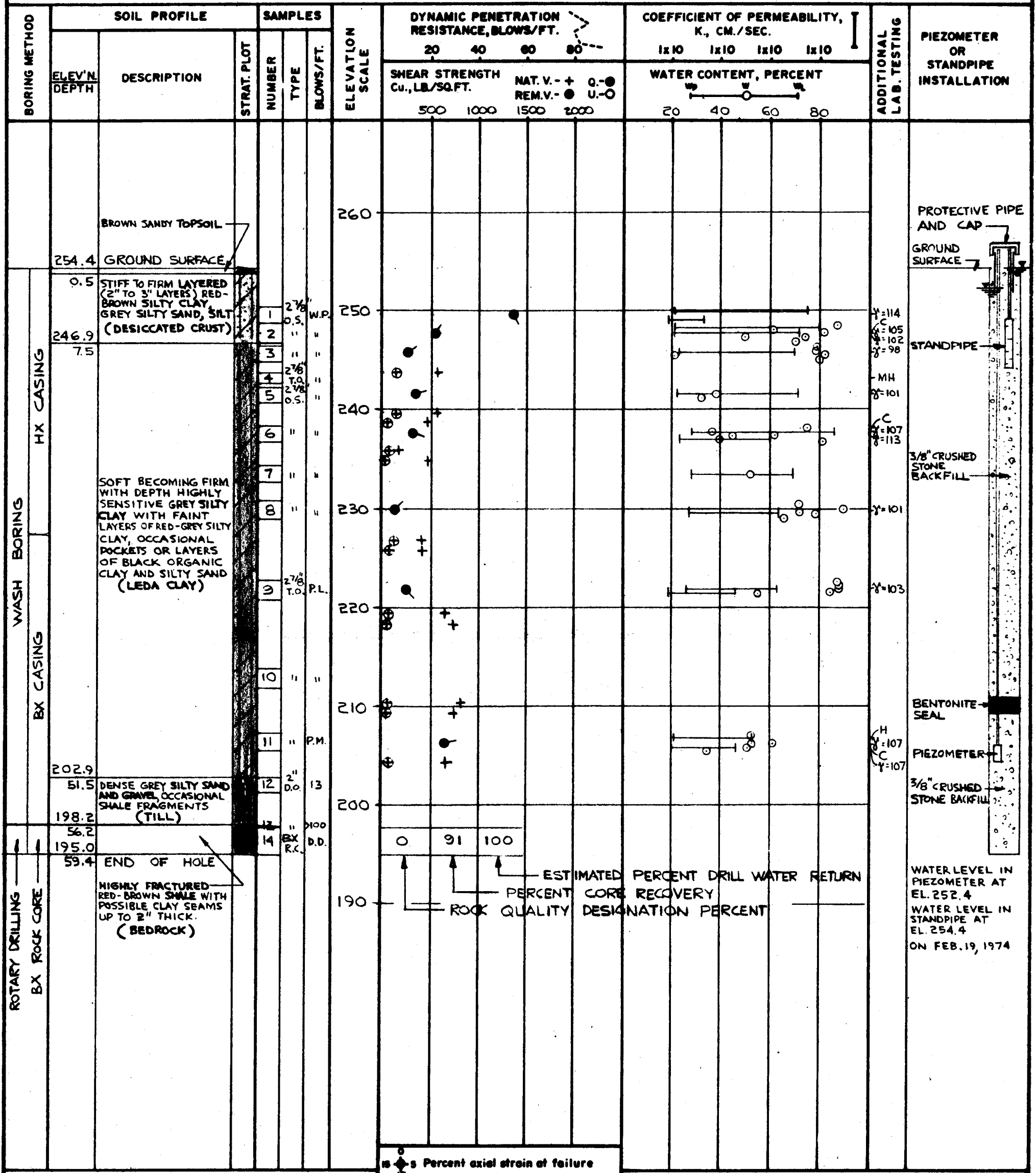
LOCATION See Figure 2

BORING DATE FEB. 18 & 19, 1974

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN D.M.
CHECKED J.M.S.

RECORD OF BOREHOLE 4

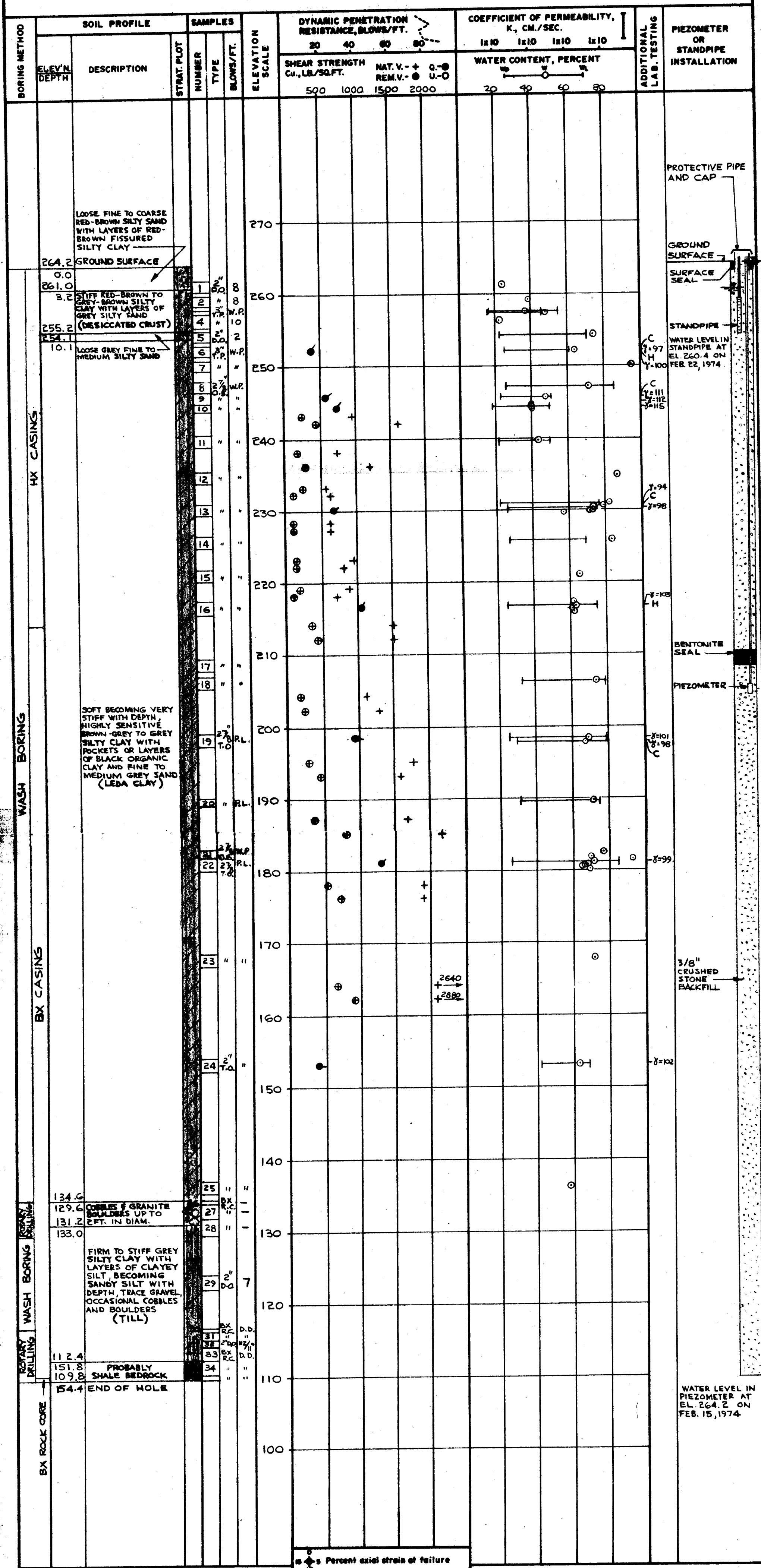
LOCATION See Figure 2

BORING DATE FEB. 4 - 14, 1974

DATUM GEODETIC

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PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



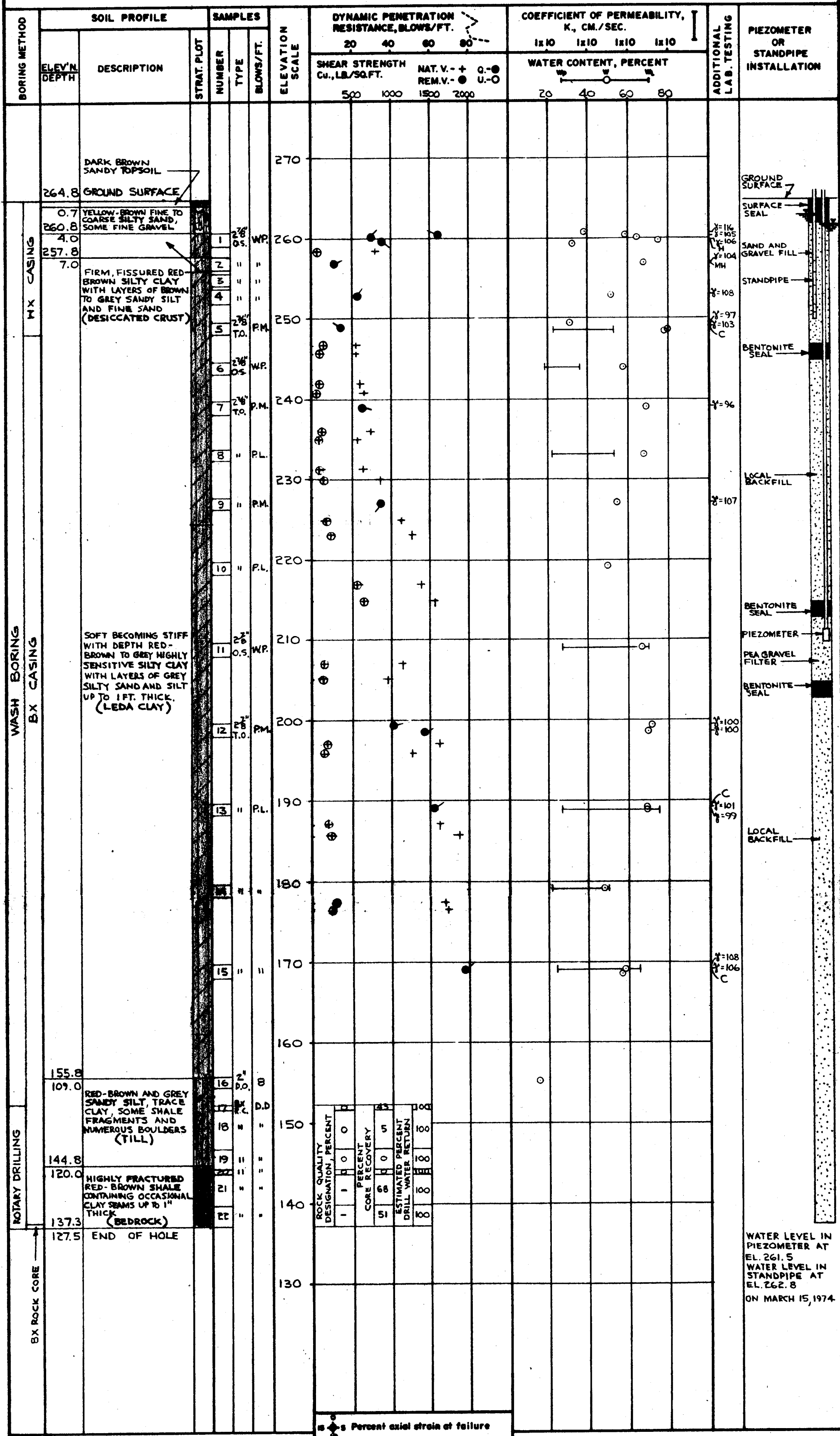
VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN *m.h.b.*
CHECKED *J.H.L.*

RECORD OF BOREHOLE 5

LOCATION See Figure 2 BORING DATE FEB. 20-25, MARCH 2-4, 1974 DATUM GEODETIC
 SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



VERTICAL SCALE
1 IN. TO 10 FT.

Golder Associates

DRAWN *P.M.*
CHECKED *J.H.C.*

KEY PLAN

FIGURE 1



SCALE: 1 INCH TO 3 MILES

Date APRIL 24, 1974

Golder Associates

Drawn *[Signature]*
 Chkd. *[Signature]*
 Appd. *[Signature]*

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BORING PLAN

FIGURE 2

LEGEND

- BOREHOLE IN PLAN
- TEST PIT IN PLAN



SITE BOUNDARY

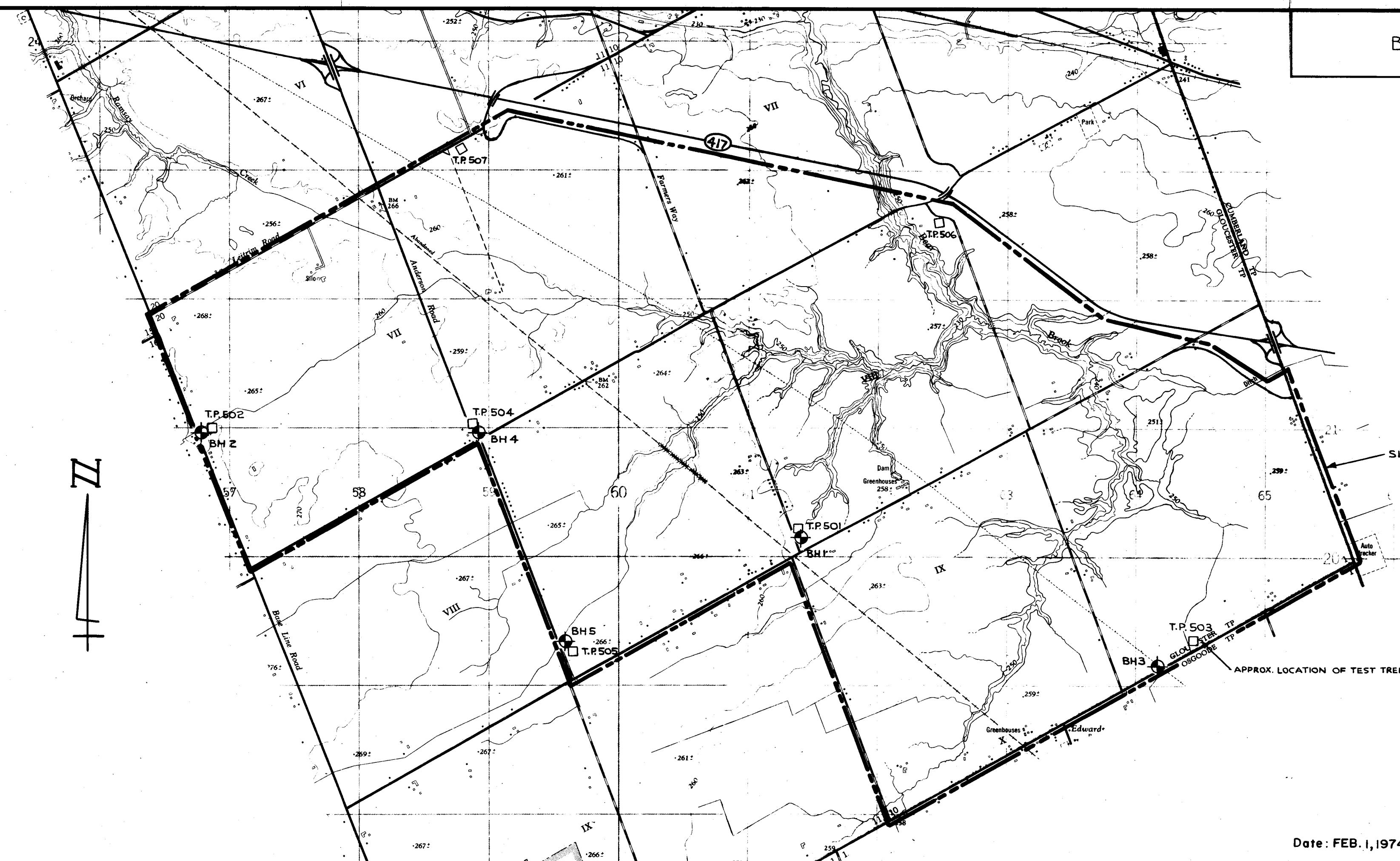
APPROX. LOCATION OF TEST TRENCH

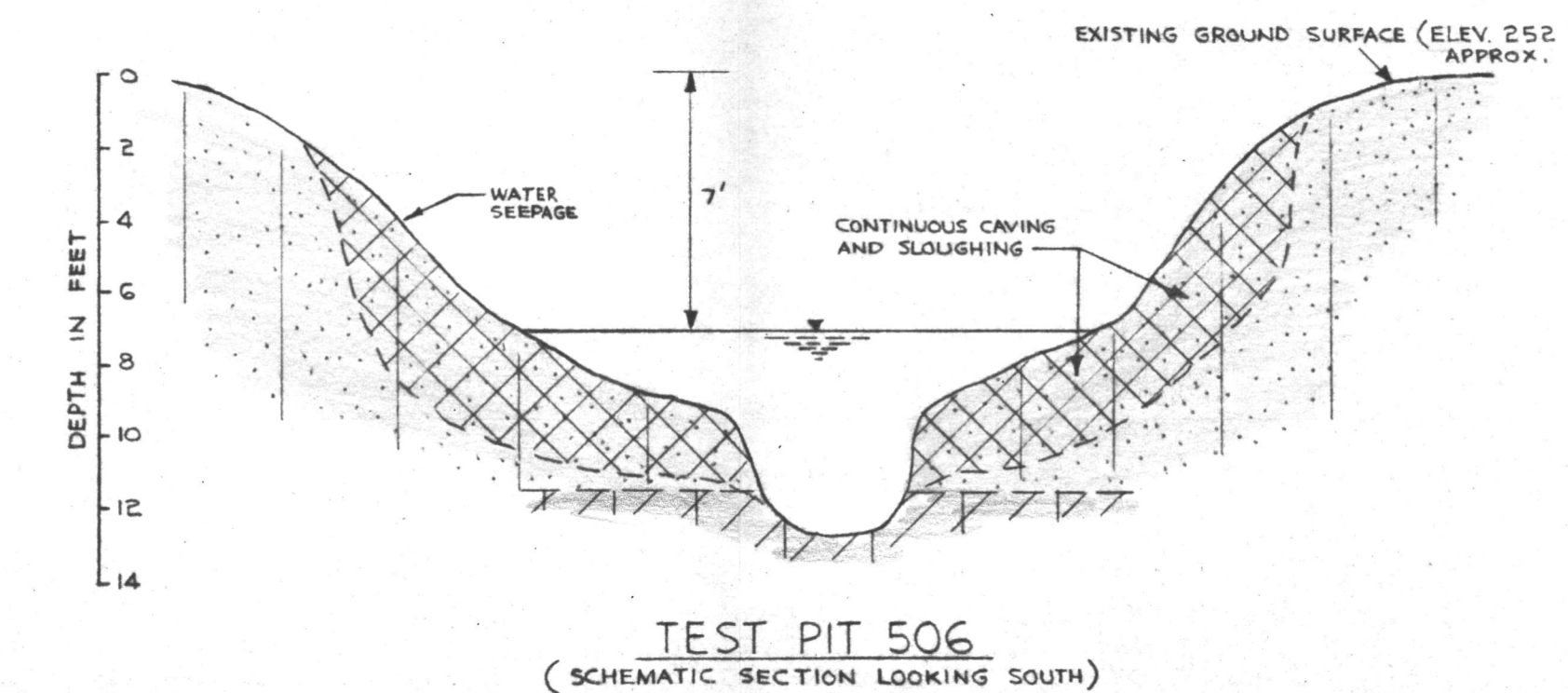
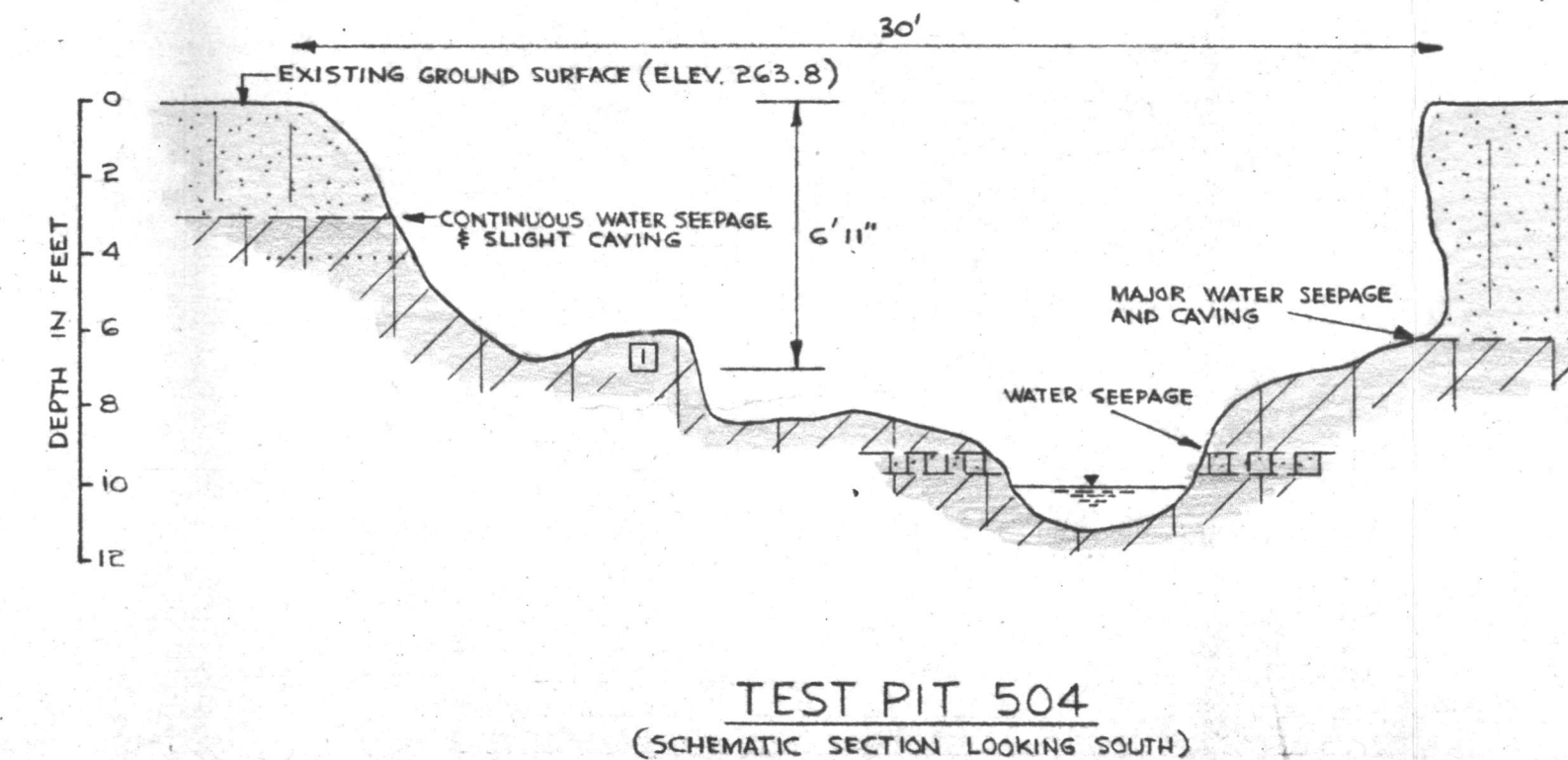
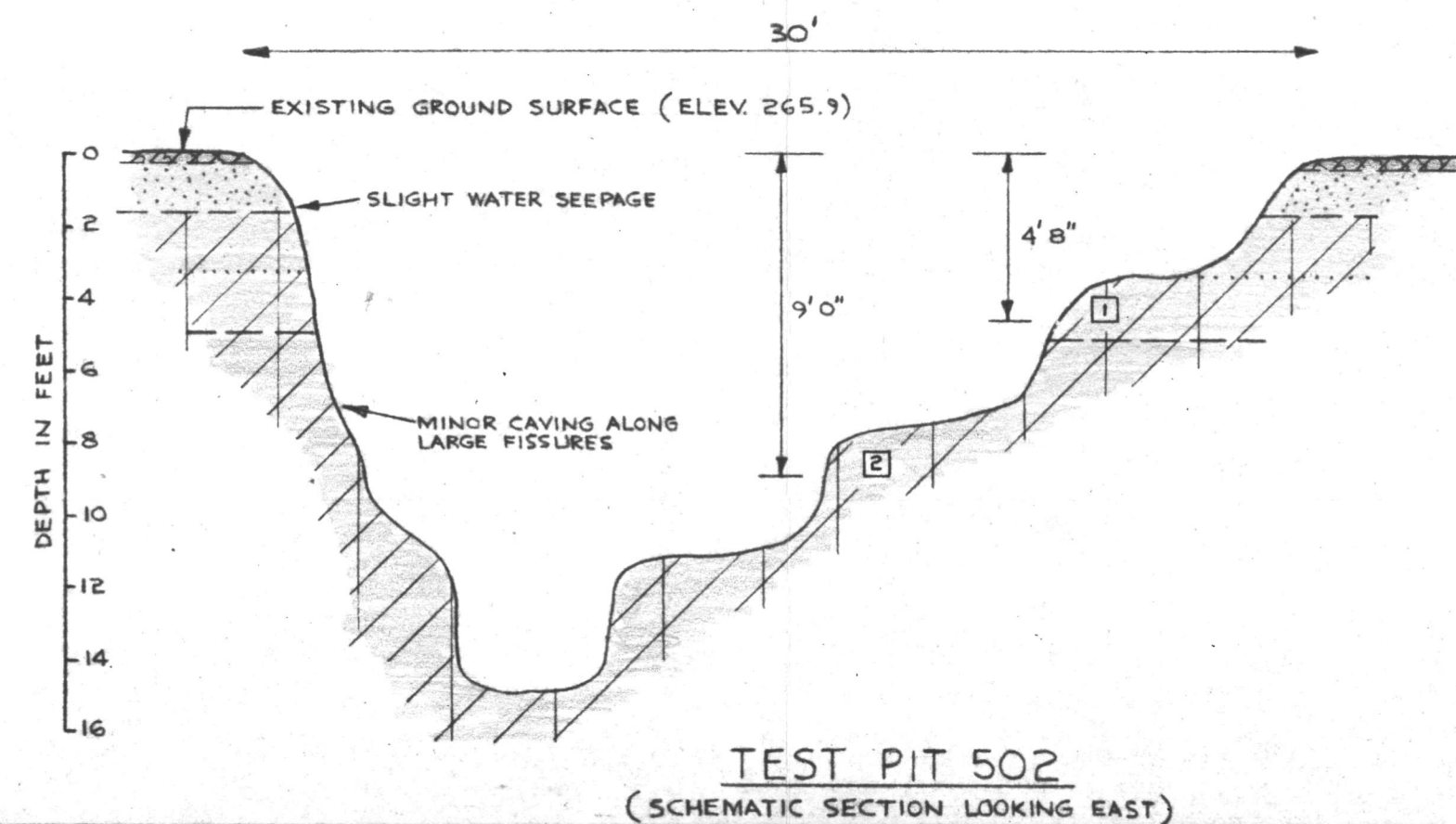
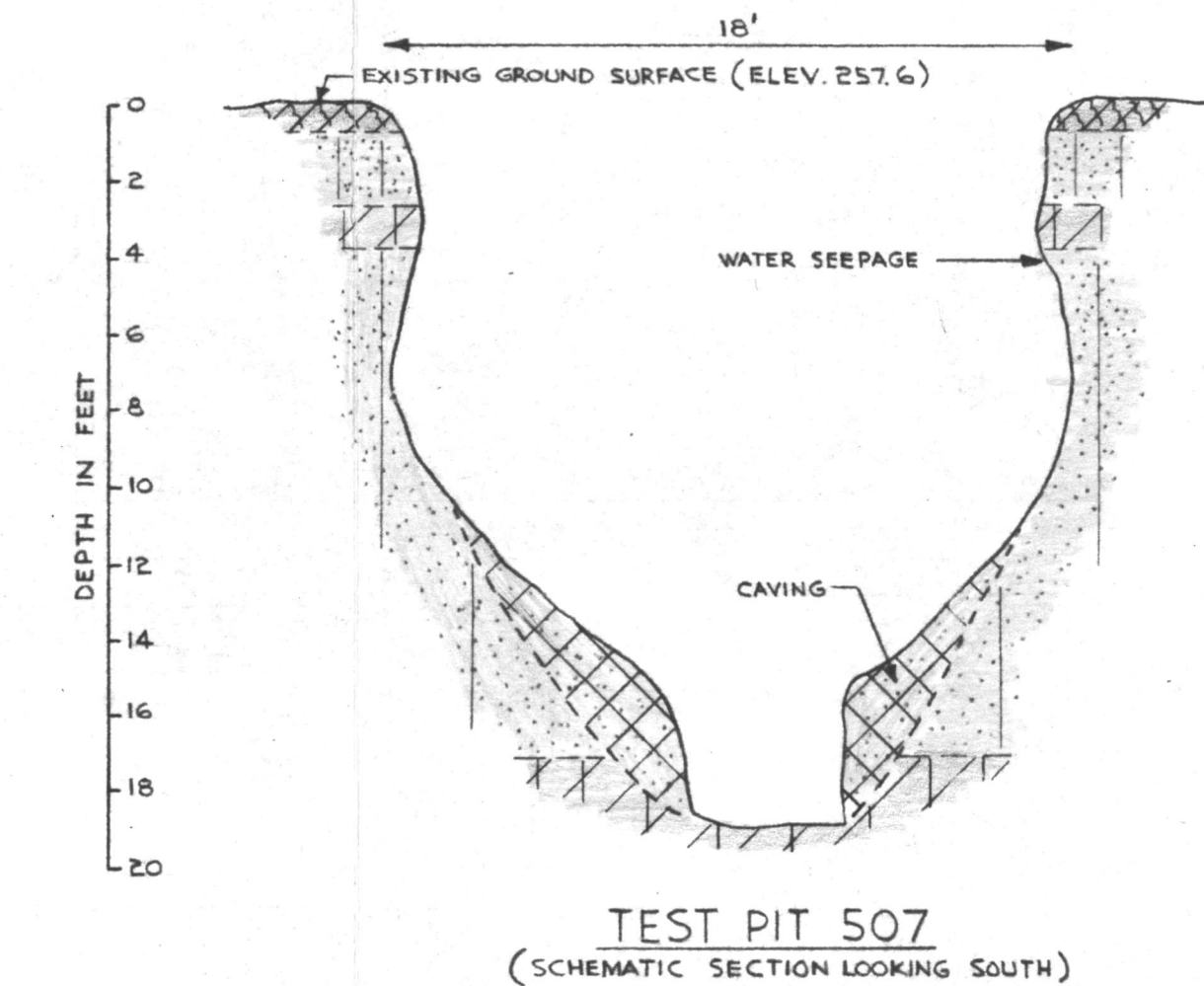
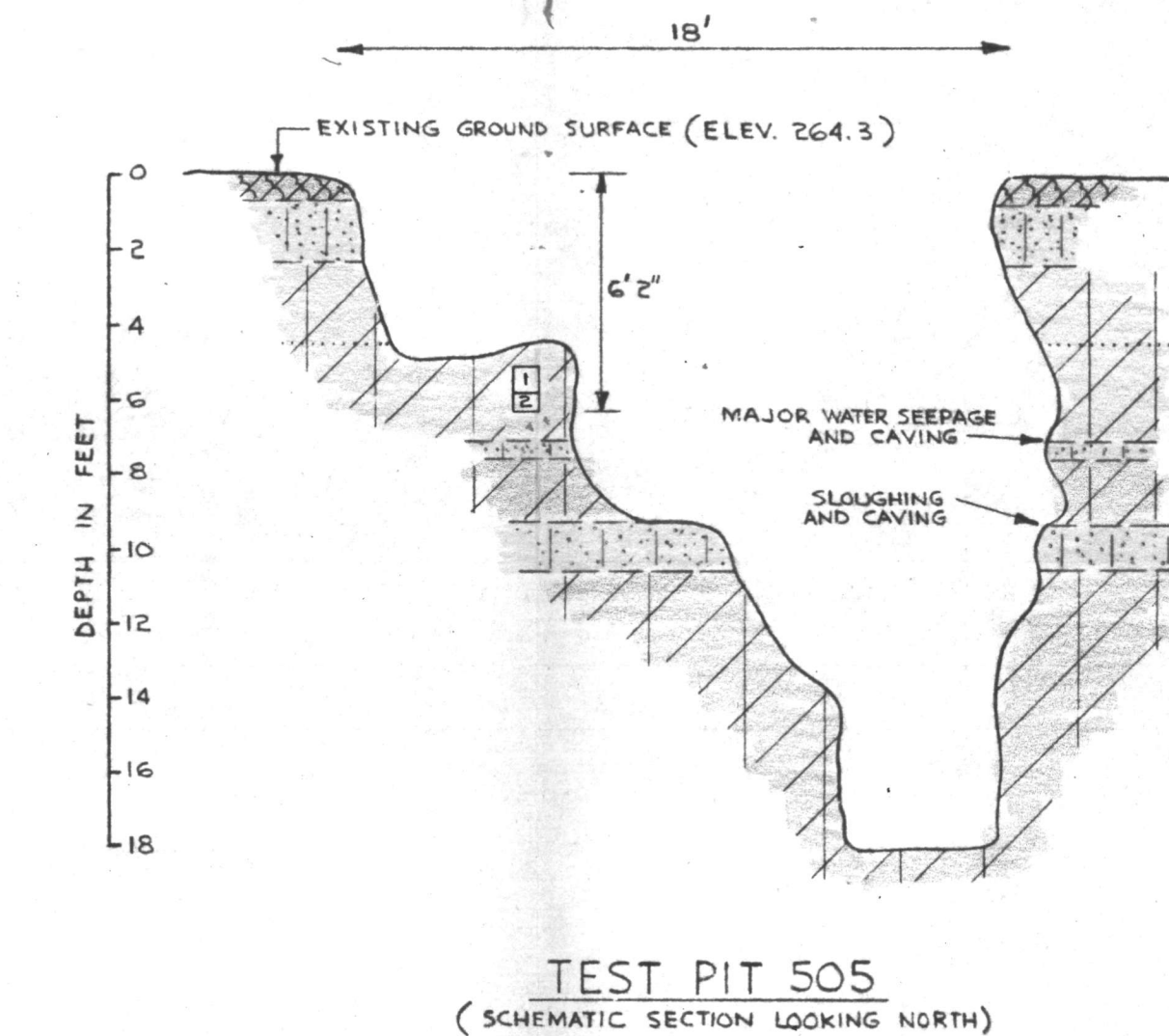
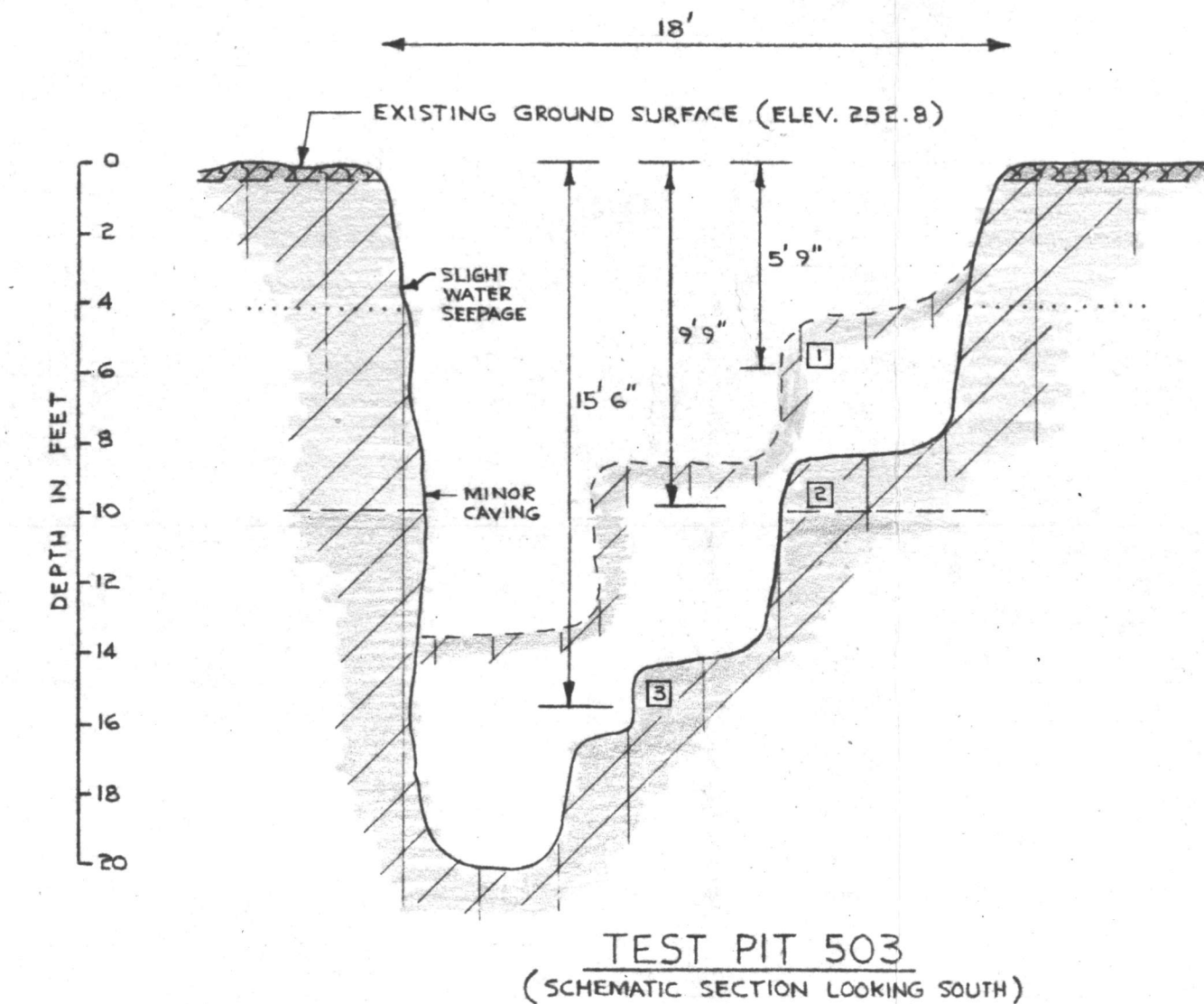
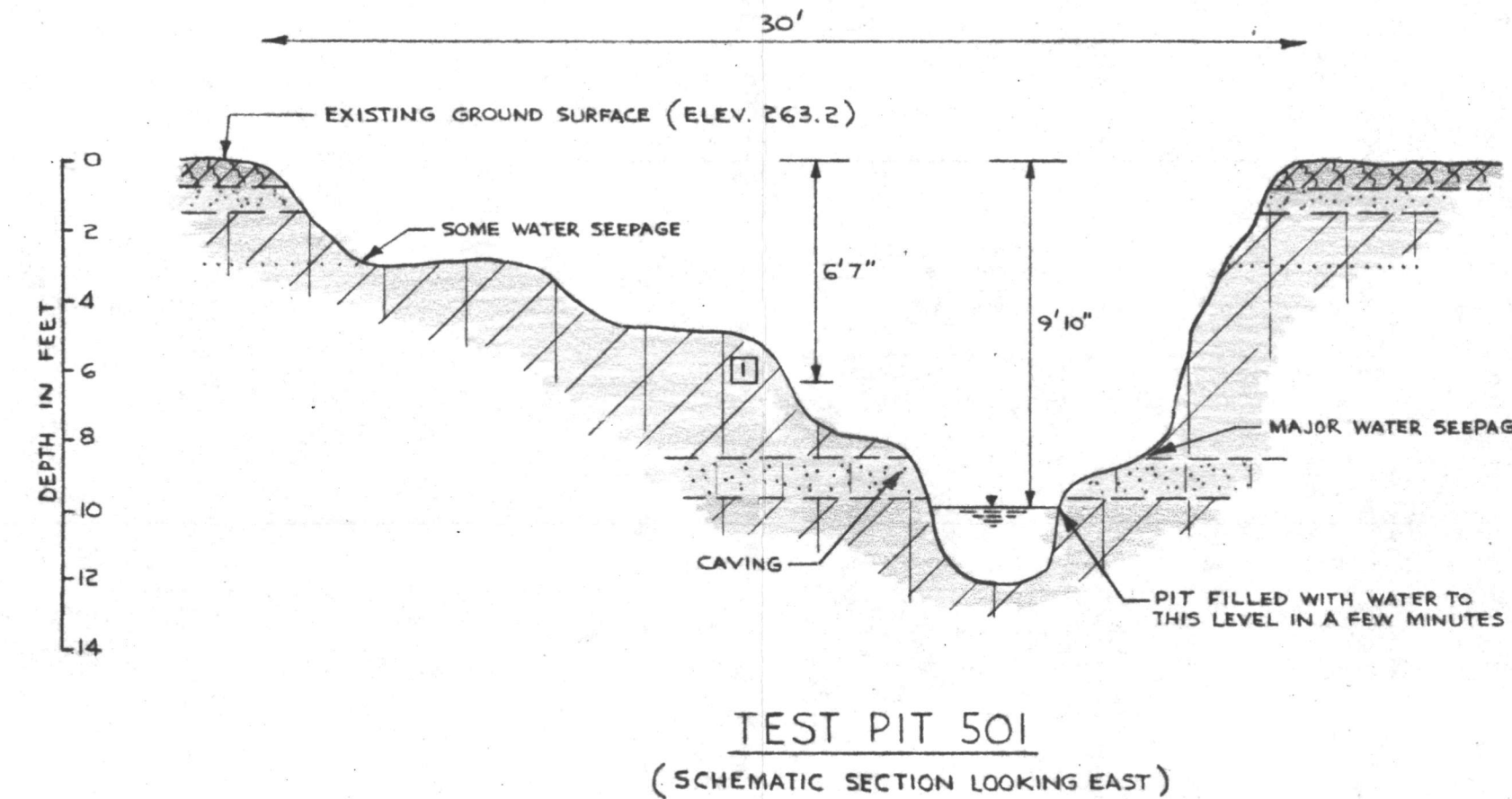
SCALE: 1:25,000

Date: FEB. 1, 1974

Golder Associates

Drawn *myb*
Chkd. *gac*
Appd. *gac*





STRATIGRAPHY

- TOPSOIL
- BROWN TO GREY SAND TO SILTY SAND
- RED-BROWN FISSURED AND LAYERED SILTY CLAY BECOMING LESS FISSURED WITH DEPTH (DESICCATED CRUST)
- GREY HIGHLY SENSITIVE SILTY CLAY (LEDA CLAY)
- GREY SILT TO SANDY SILT
- LOCATION OF UNDISTURBED BLOCK SAMPLE

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

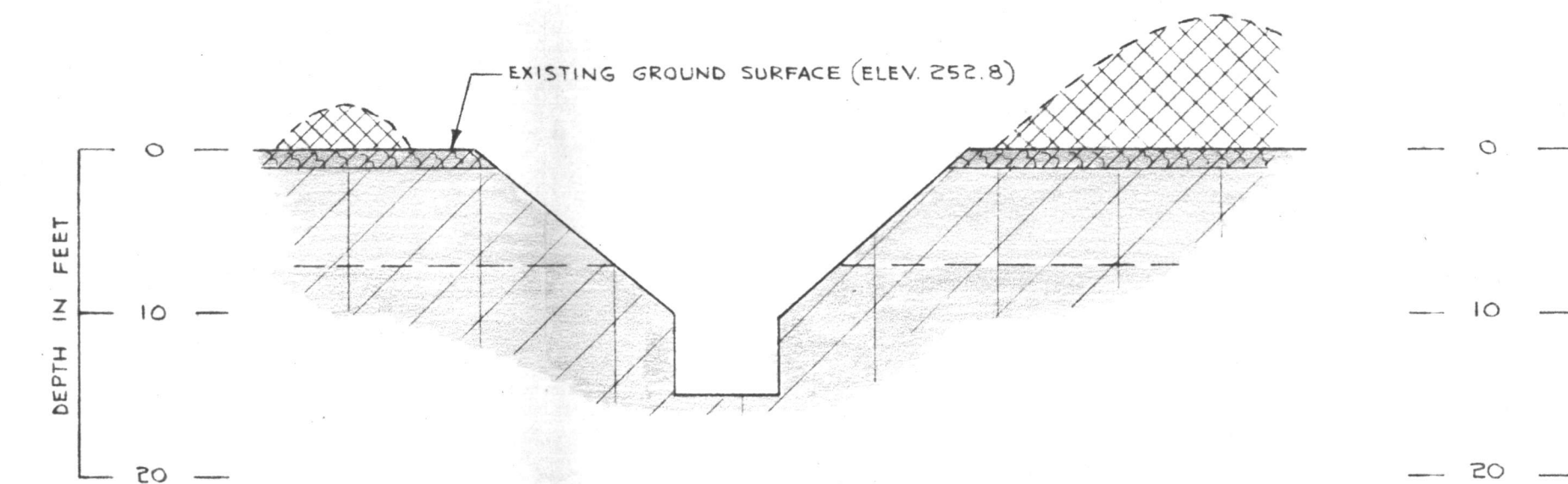
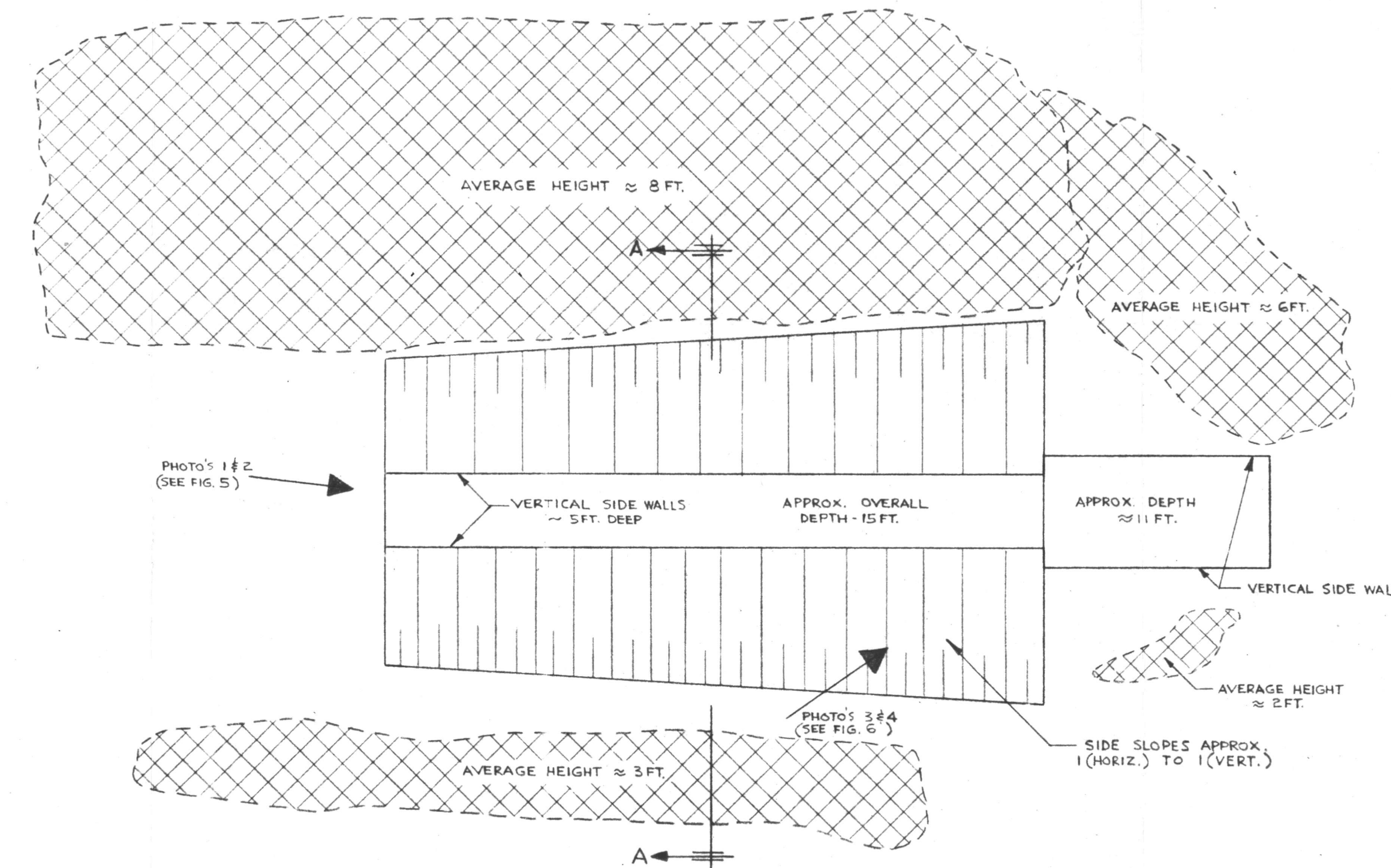
SCALE: 1" TO 5' (APPROX.)

Date APRIL 18, 1974

Golder Associates

Drawn J.A.
Chkd. J.A.
Appd. J.A.

N (APPROX.)



SECTION A-A

SCALE: 1" TO 10'

STRATIGRAPHY

- EXCAVATED MATERIAL
- TOPSOIL
- STIFF TO FIRM RED-BROWN WEATHERED SILTY CLAY WITH LAYERS OF GREY-GREEN SANDY SILT BECOMING SOFT AND LESS DISTINCTLY LAYERED BELOW 10 FT.
- GREY HIGHLY SENSITIVE SILTY CLAY (LEDA CLAY)

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

Date MAY 2, 1974

Golder Associates

Drawn J.A.
Chkd. J.H.
Appd. J.A.

PHOTOGRAPHIC RECORD
OF TEST TRENCH PERFORMANCE

FIGURE 5



PHOTOGRAPH 1 VIEW OF TEST TRENCH LOOKING EAST
SHORTLY AFTER EXCAVATION ON MARCH 12, 1974.
DEPTH OF VERTICAL SIDED SECTION - APPROX. 11 FT.
DEPTH OF SLOPING SIDED SECTION - APPROX. 16 FT.



PHOTOGRAPH 2 VIEW OF TEST TRENCH LOOKING EAST
SHORTLY BEFORE BACKFILLING ON MARCH 15, 1974.

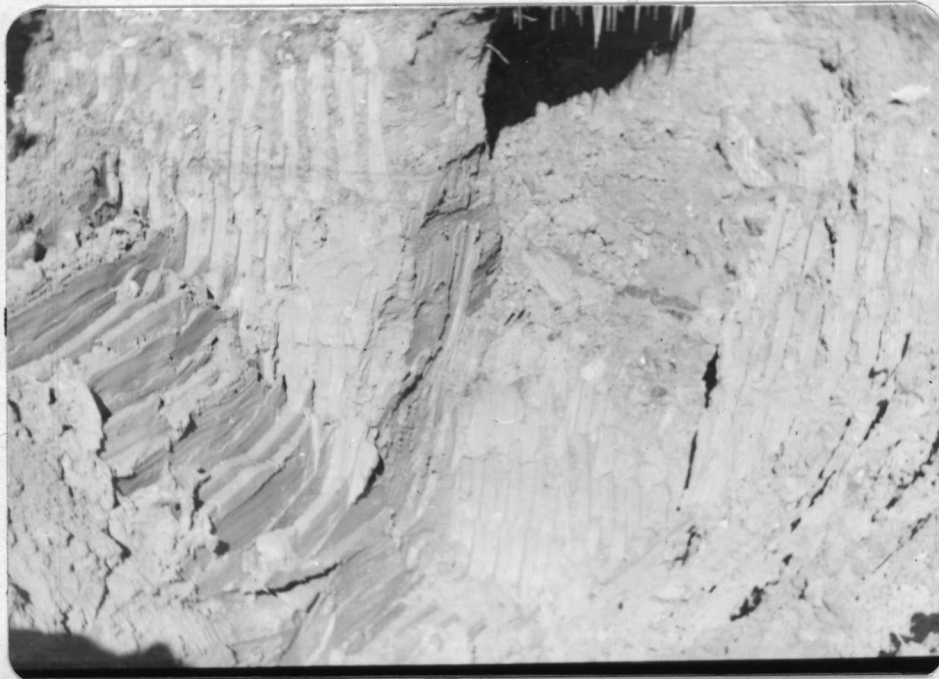
Date MAY 2, 1974

Golder Associates

Drawn J.A.
Chkd. _____
Appd. _____

DETAILS OF SOIL CONDITIONS
AND TEST TRENCH PERFORMANCE

FIGURE 6



PHOTOGRAPH 3 DETAIL OF SOIL CONDITIONS IMMEDIATELY
AFTER EXCAVATION ON MARCH 12, 1974.



PHOTOGRAPH 4 DETAIL OF FAILURE ON THE NORTH
VERTICAL WALL, MARCH 15, 1974.

Date MAY 2, 1974

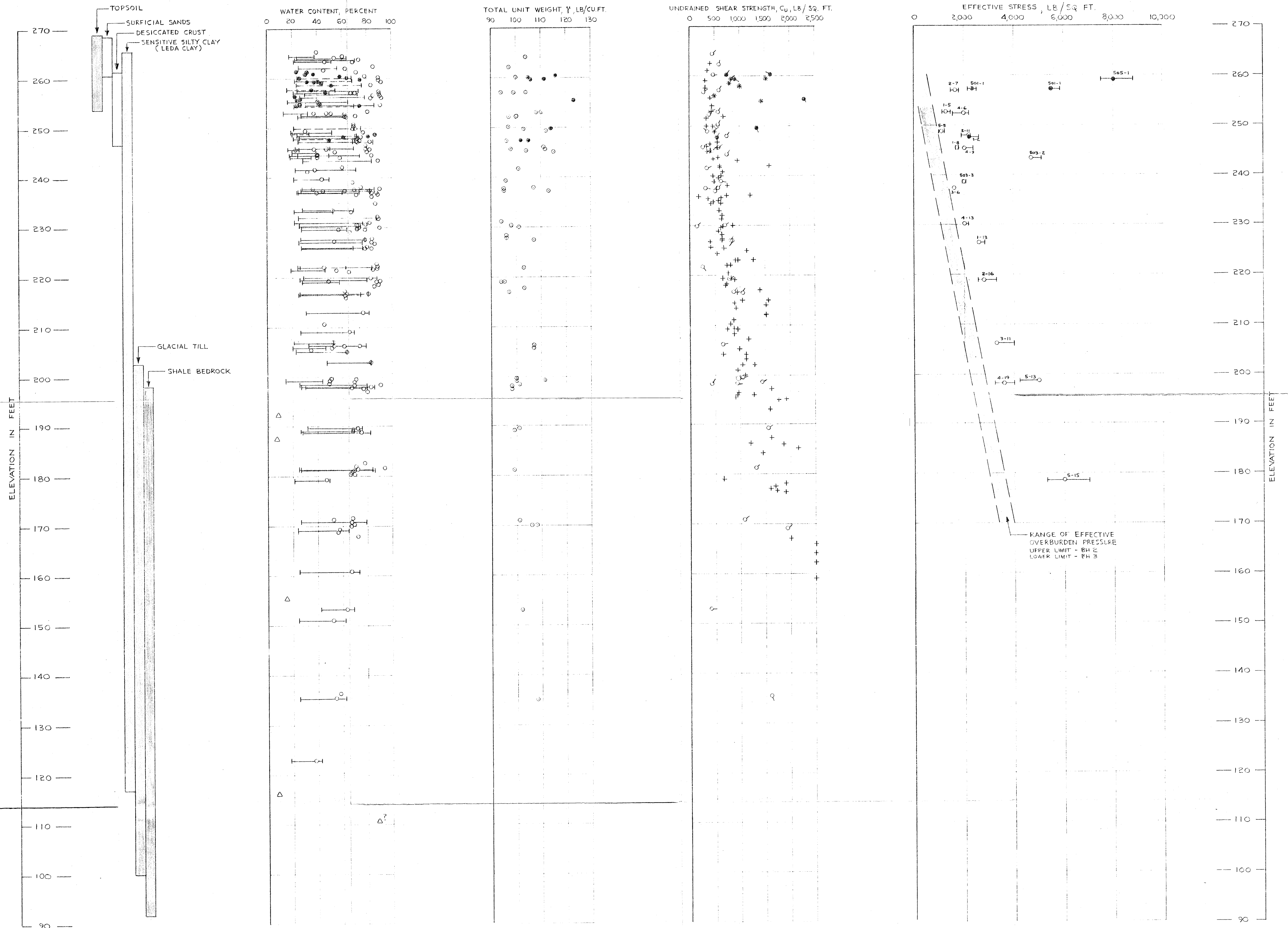
Golder Associates

Drawn J.A.
Chkd. _____
Appd. _____

SIMPLIFIED SOIL STRATIGRAPHY
(SHOWING RANGE IN ELEVATION)

SUMMARY OF
ENGINEERING PROPERTIES

FIGURE 7



W_p — W — W_L (DESICCATED CRUST)
— (LEDA CLAY)
Δ (GLACIAL TILL)

● (DESICCATED CRUST)
○ (LEDA CLAY)

● DESICCATED CRUST } QUICK TRIAXIAL TESTS
○ LEDA CLAY }
* DESICCATED CRUST } IN SITU VANE TESTS
+ LEDA CLAY }

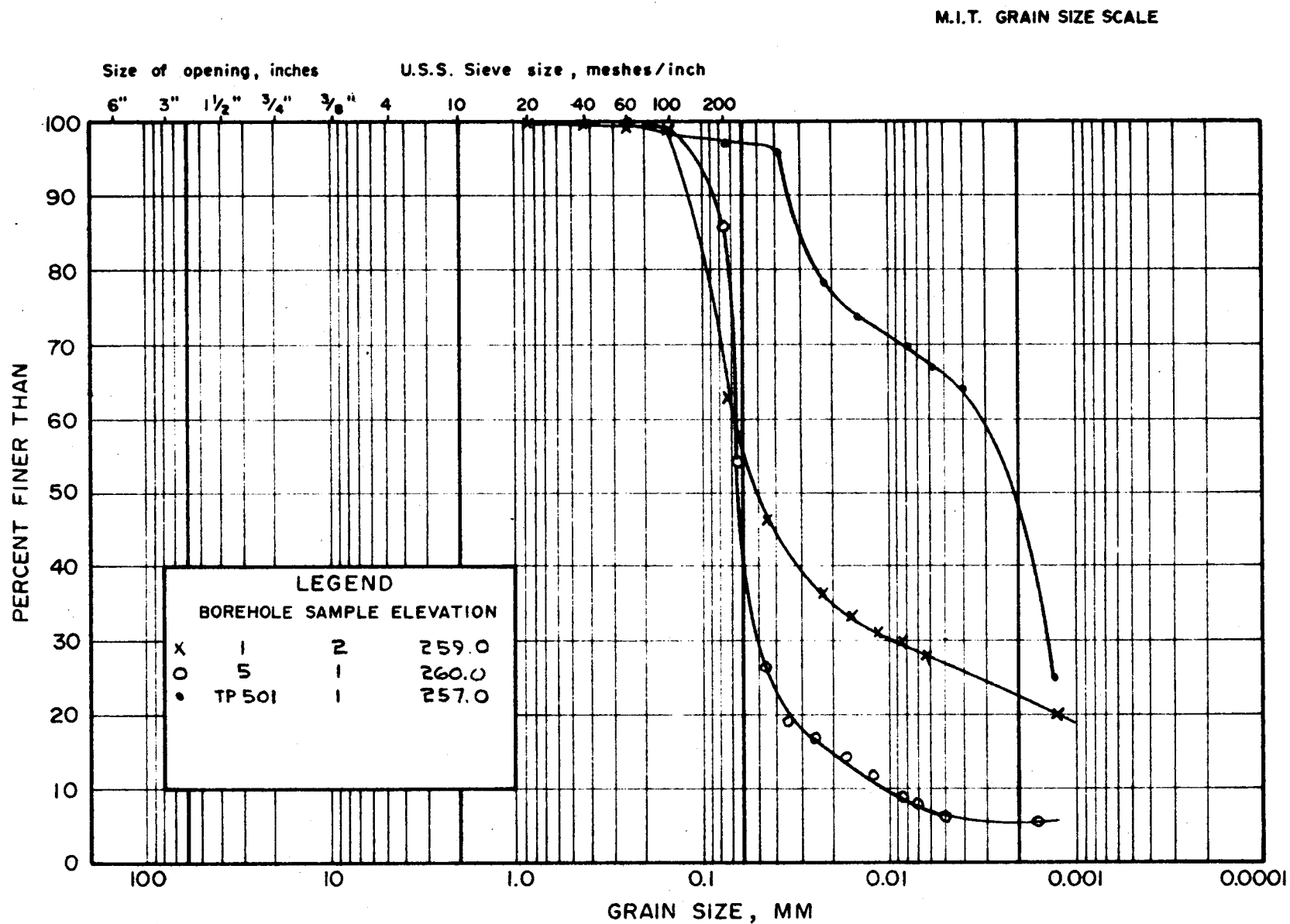
15-10 5 PERCENT AXIAL STRAIN AT FAILURE

1-5 REPRESENTS BORERHOLE OR TEST PIT NO. & SAMPLE NO.
● DESICCATED CRUST
○ LEDA CLAY
— MAXIMUM PRECONSOLIDATION PRESSURE
— MINIMUM

Date: MAY 8, 1974

Golder Associates

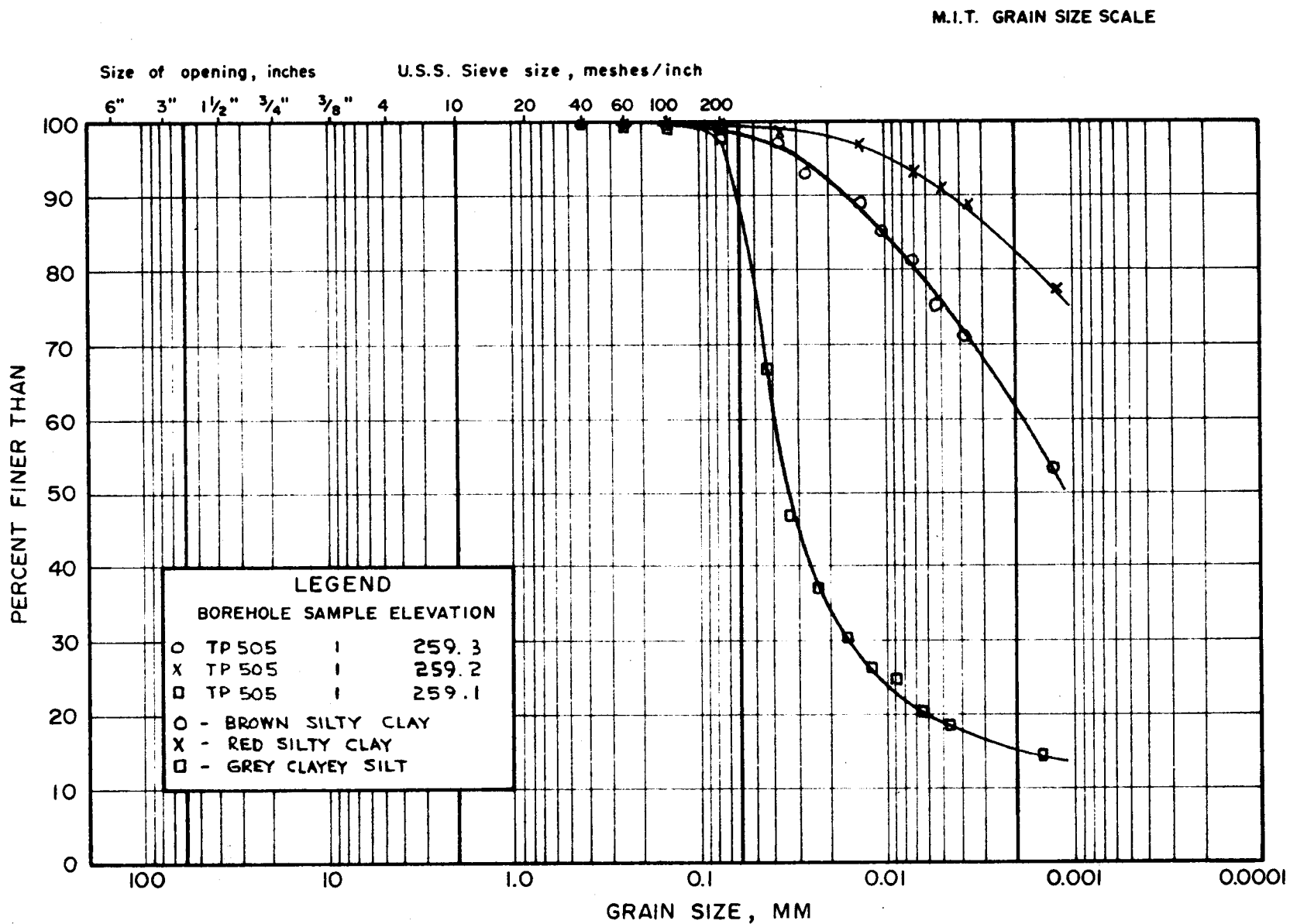
Drawn: J.A.
Chkd: J.L.
Appd: J.L.



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

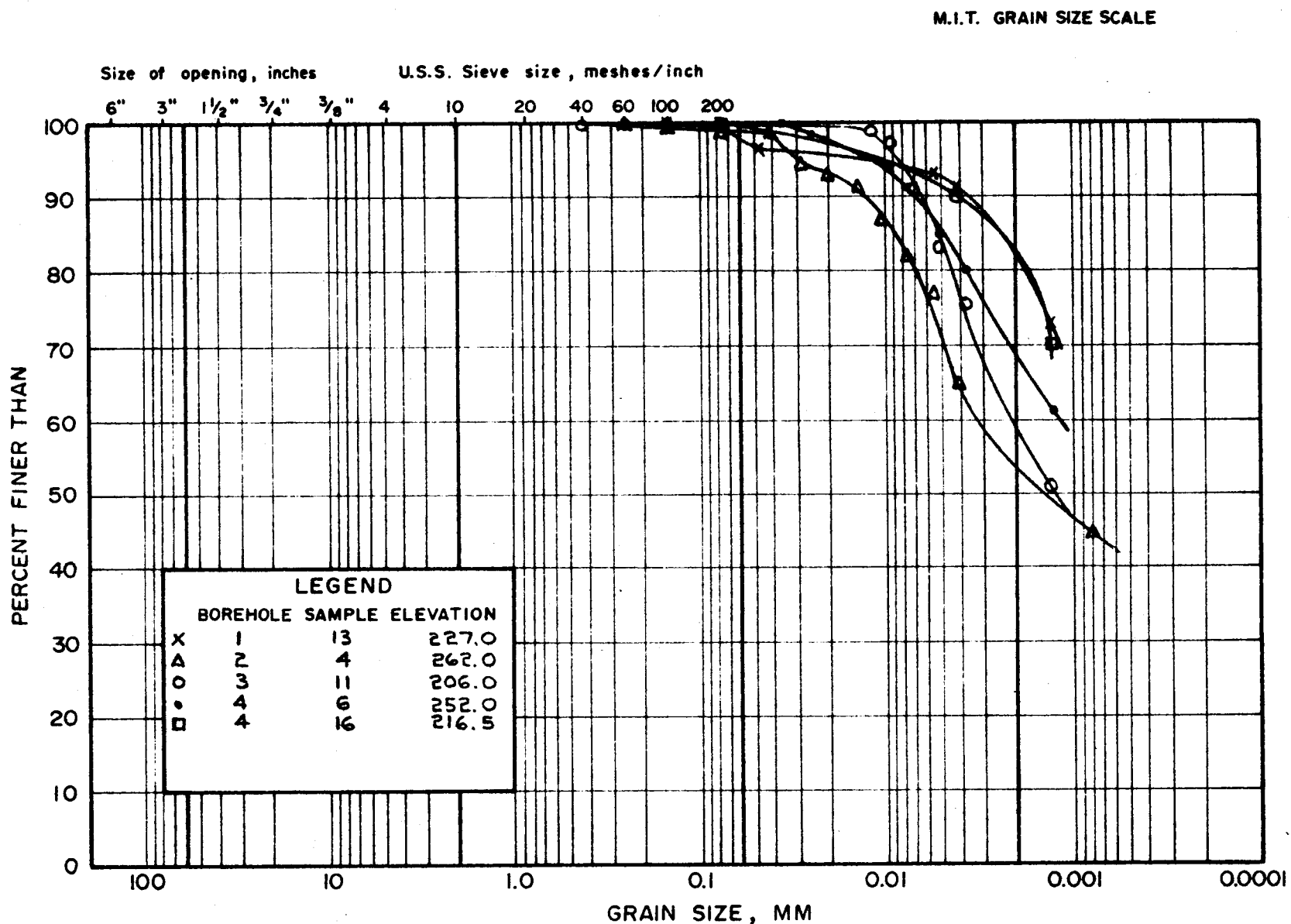
GRAIN SIZE DISTRIBUTION
DESICCATED CRUST

FIGURE 8



GRAIN SIZE DISTRIBUTION
DESICCATED CRUST

FIGURE 9

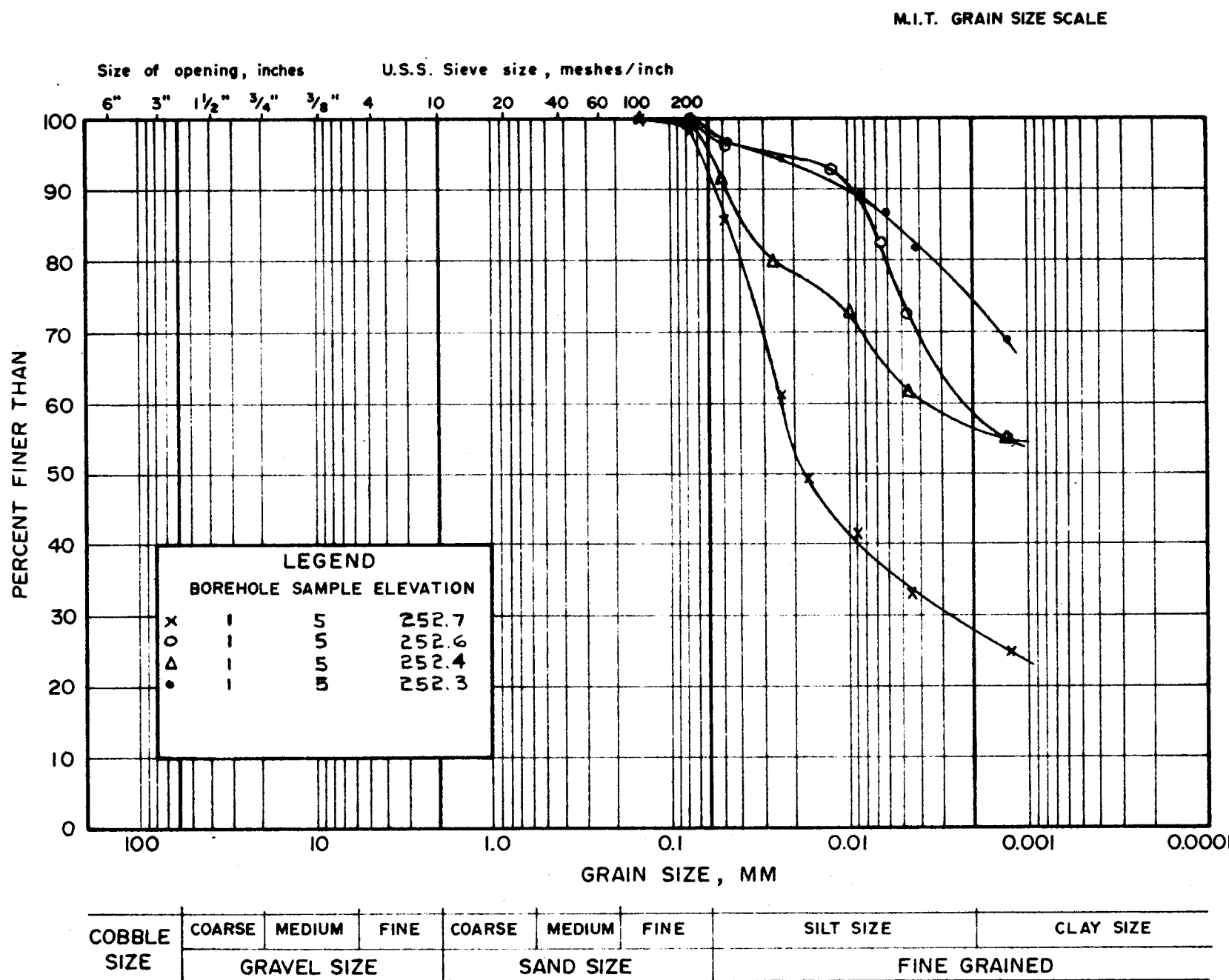


COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			

GRAIN SIZE DISTRIBUTION
LEDA CLAY

FIGURE 10

Golder Associates



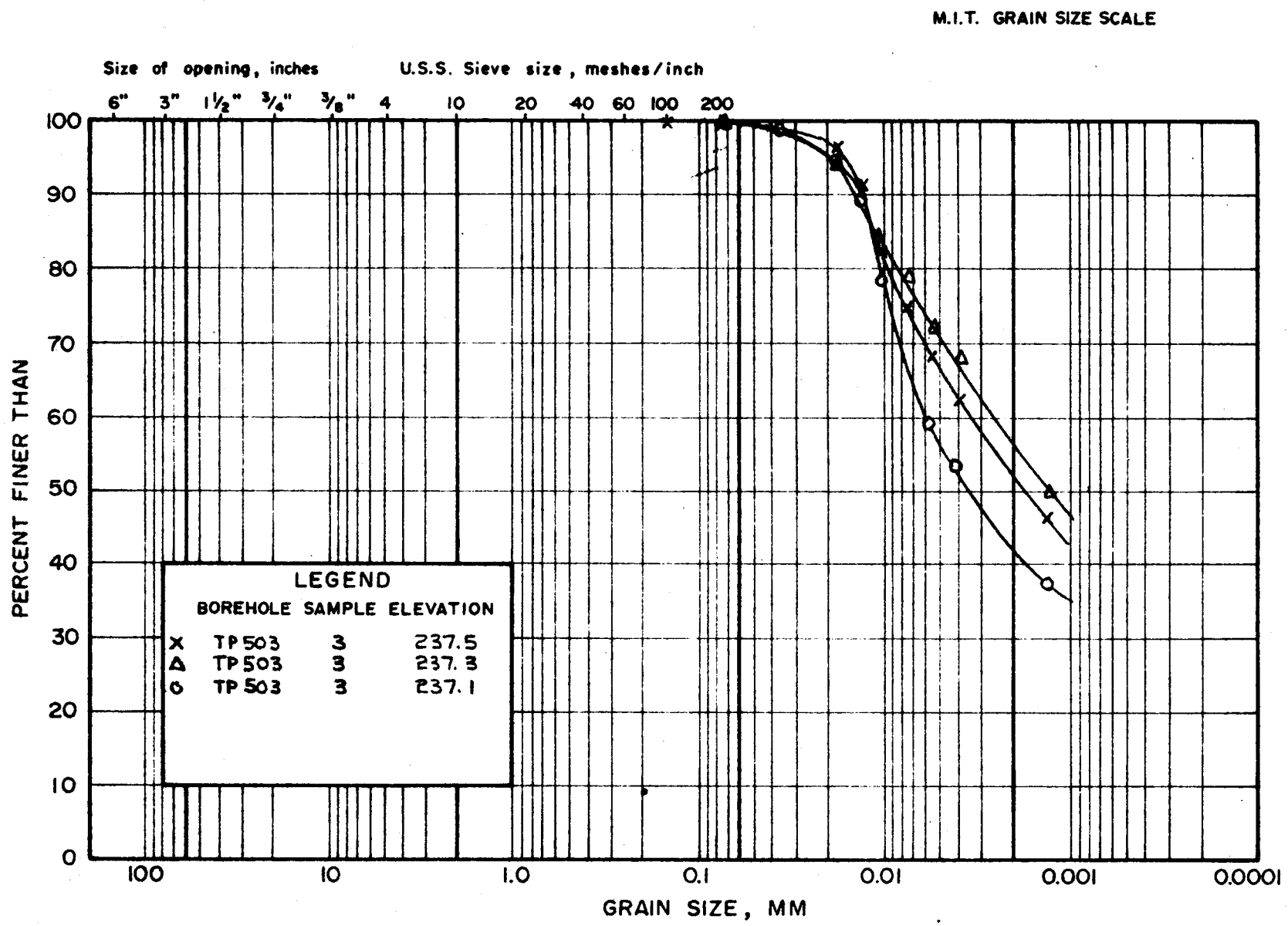
GRAIN SIZE DISTRIBUTION
LEDA CLAY

FIGURE 11

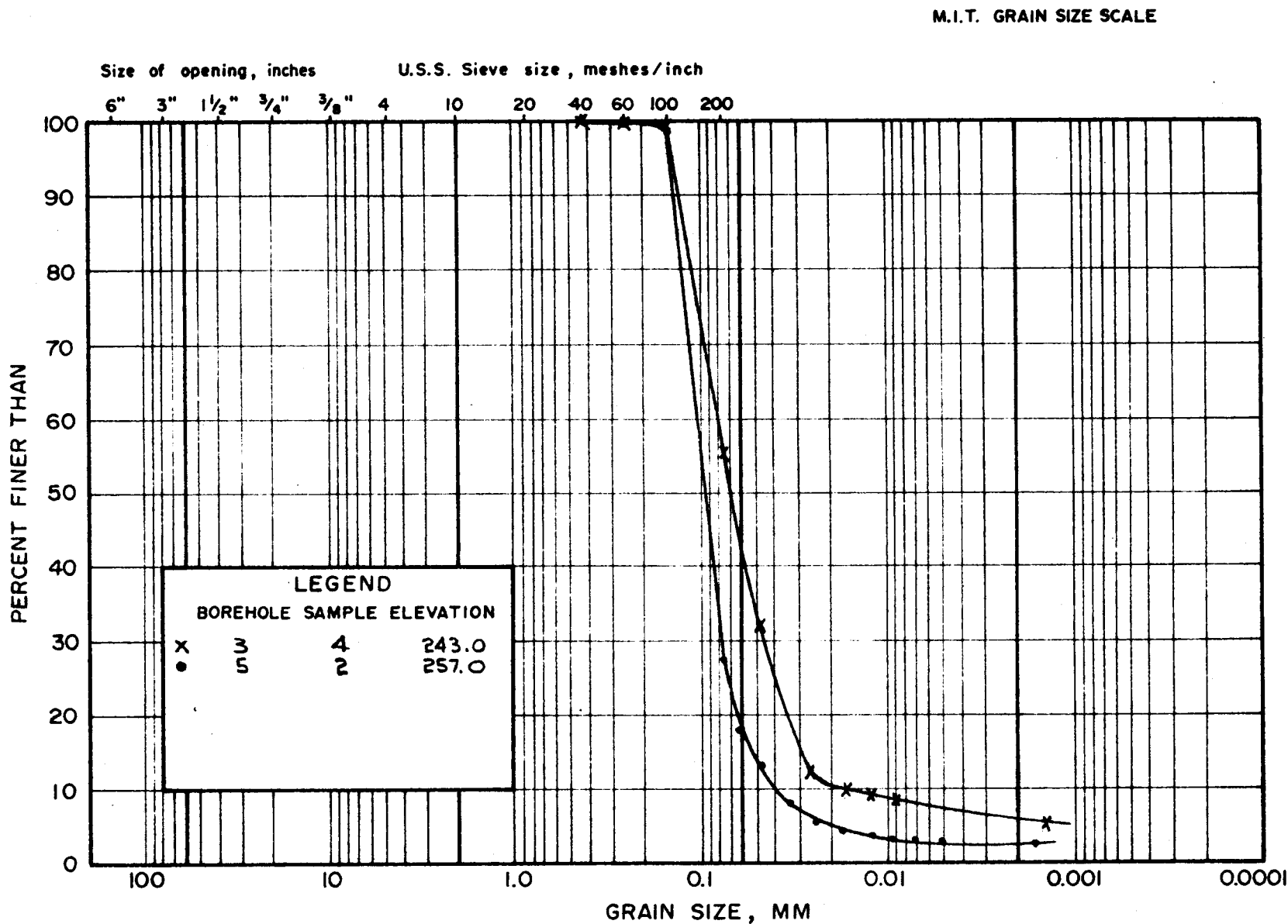
Golder Associates

GRAIN SIZE DISTRIBUTION
LEDA CLAY

FIGURE 12



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			



GRAIN SIZE DISTRIBUTION
SAND LAYERS IN LEDA CLAY

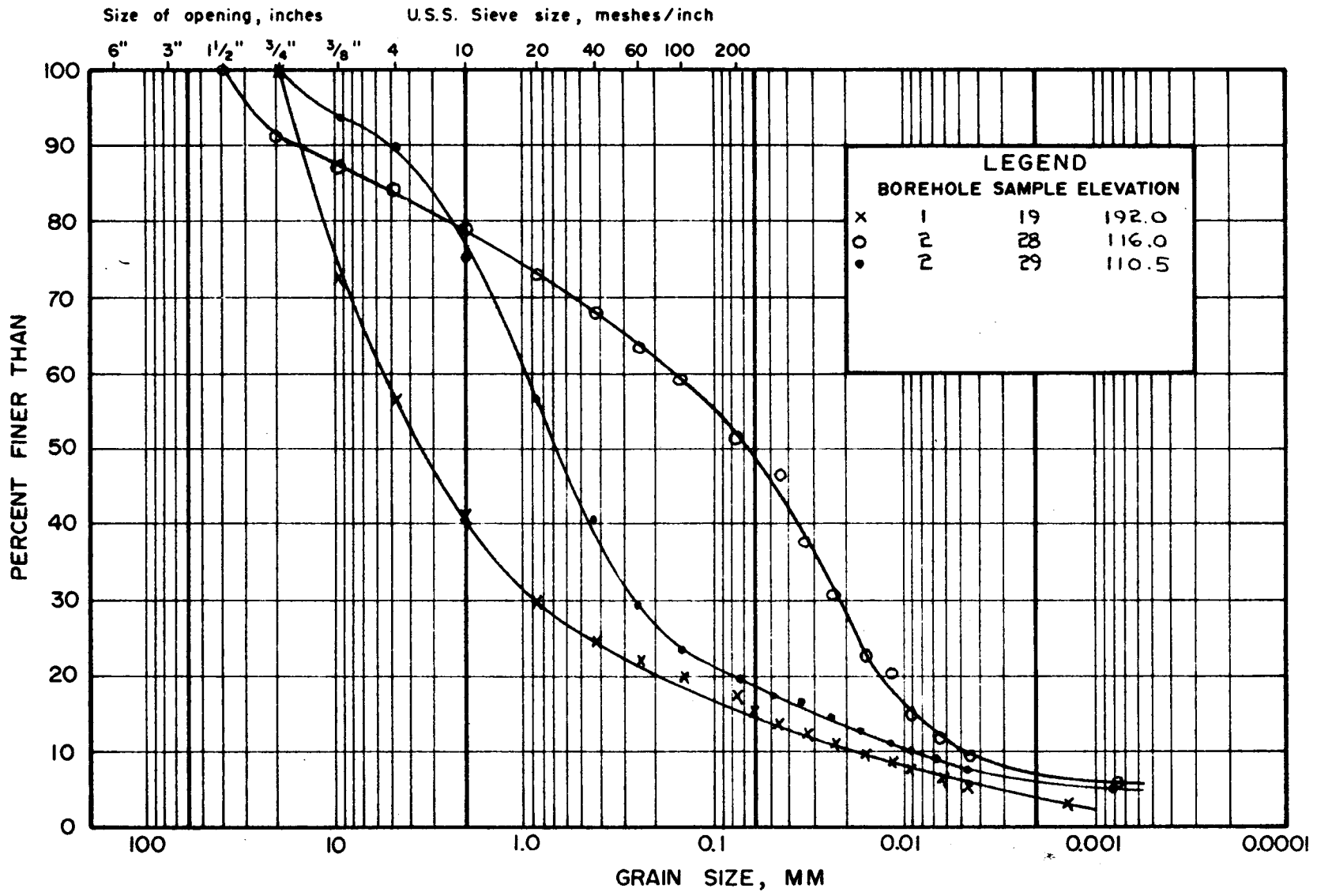
FIGURE 13

M.I.T. GRAIN SIZE SCALE

GRAIN SIZE DISTRIBUTION
TILL

FIGURE 14

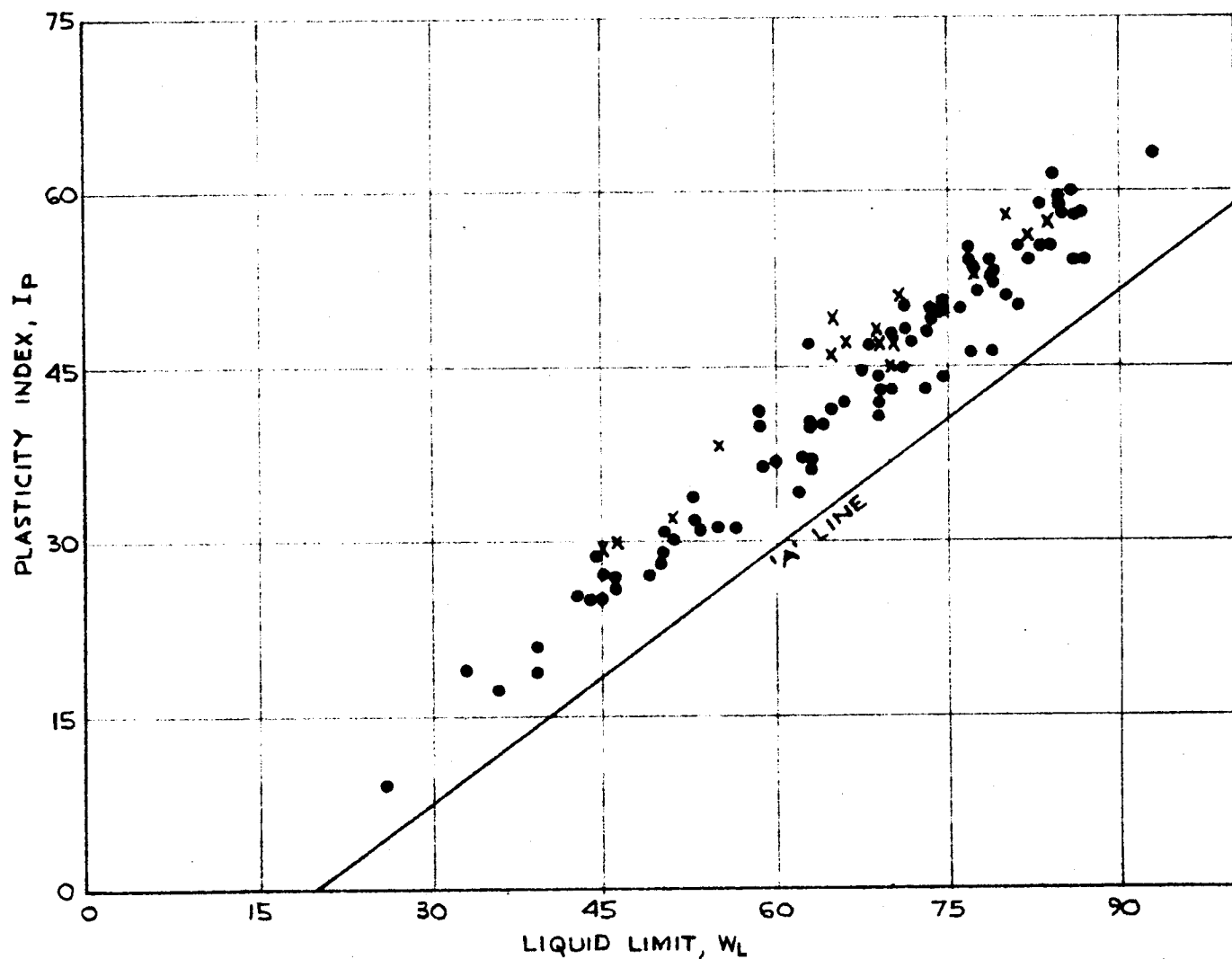
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COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

PLASTICITY CHART

FIGURE 15



LEGEND

- X DESICCATED CRUST
- LEDA CLAY

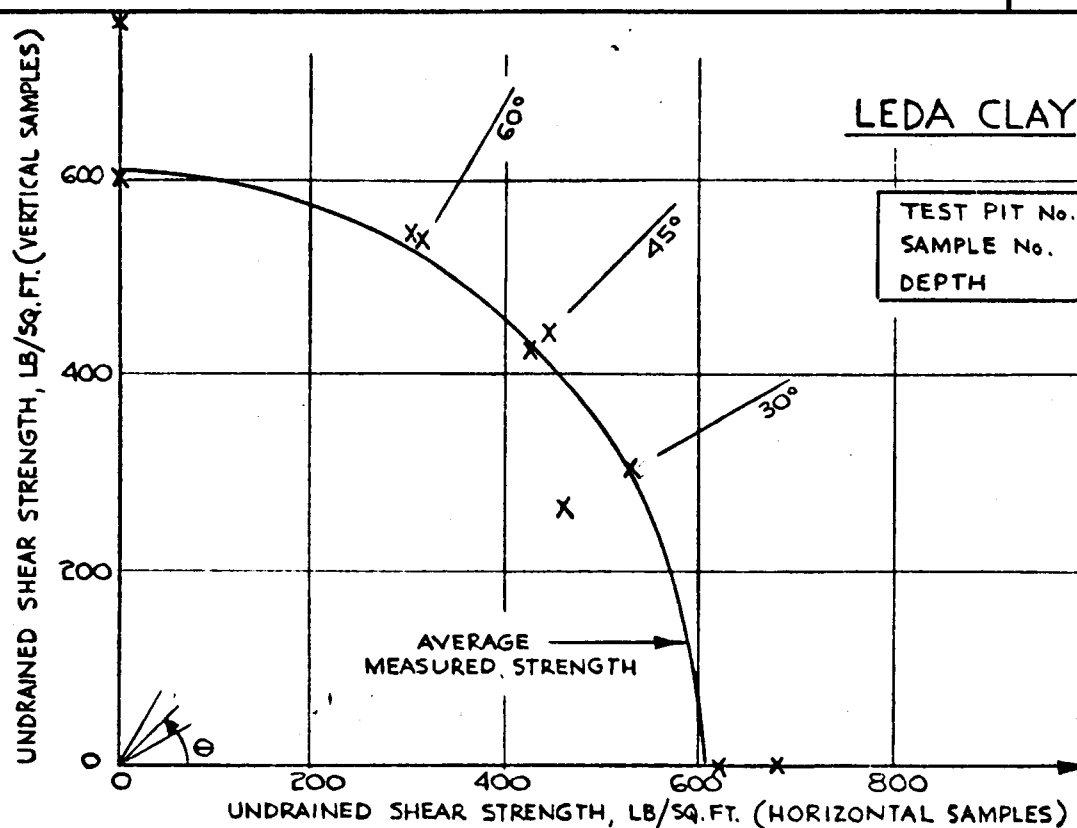
Date MAY 3, 1974

Golder Associates

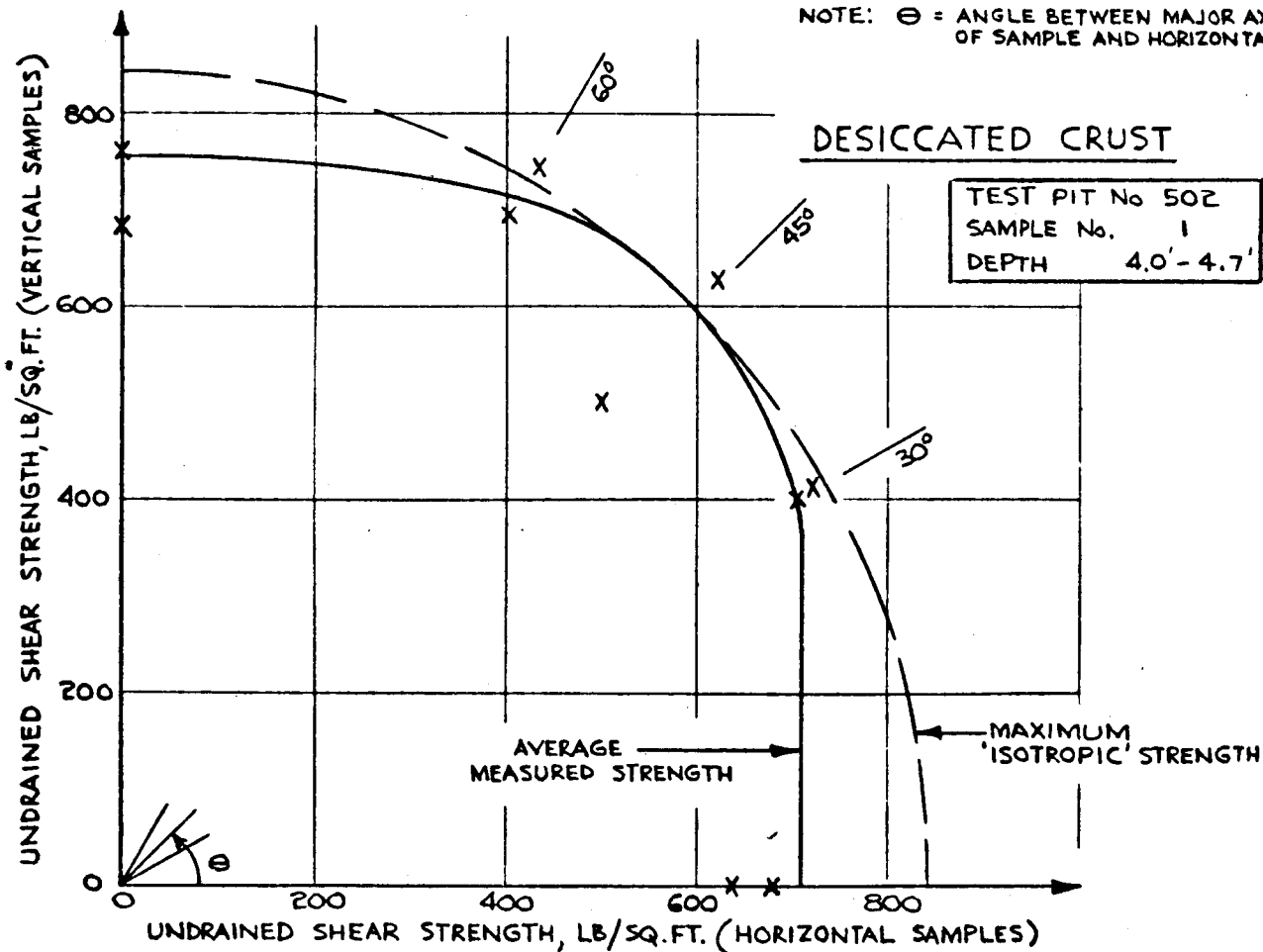
Drawn J.A.
 Chkd. 7/8/74
 Appd. 5/1/74

VARIATION IN UNDRAINED SHEAR STRENGTH WITH DIRECTION - DESICCATED CRUST AND LEDA CLAY

FIGURE 16



NOTE: Θ = ANGLE BETWEEN MAJOR AXIS OF SAMPLE AND HORIZONTAL



Date MAY 2, 1974






Golder Associates

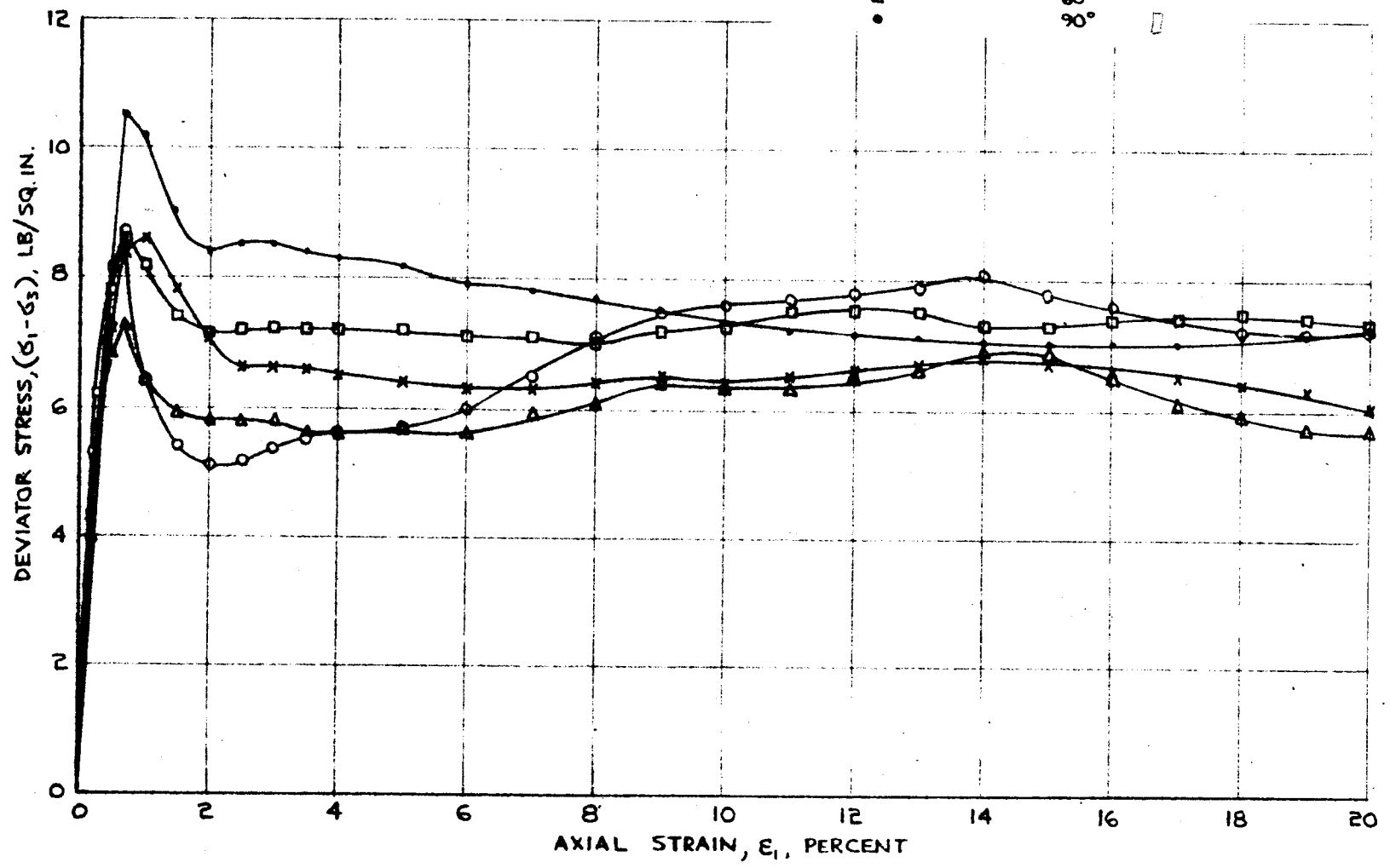
Drawn J.A.
Chkd. J.A.
Appd. J.A.

QUICK TRIAXIAL COMPRESSION TESTS
TYPICAL STRESS-STRAIN CURVES - LEDA CLAY

FIGURE 17

LEGEND

TEST PIT 502, SA. 2		
SYMBOL	SAMPLING DIRECTION *	
x	0°	
Δ	30°	
○	45°	
□	60°	
●	90°	



NOTE: TESTS CARRIED OUT ON SPECIMENS TRIMMED FROM BLOCK SAMPLE, STRAIN RATE $\approx 1\%$ /MIN.
* ANGLE BETWEEN AXIS OF SAMPLE AND HORIZONTAL

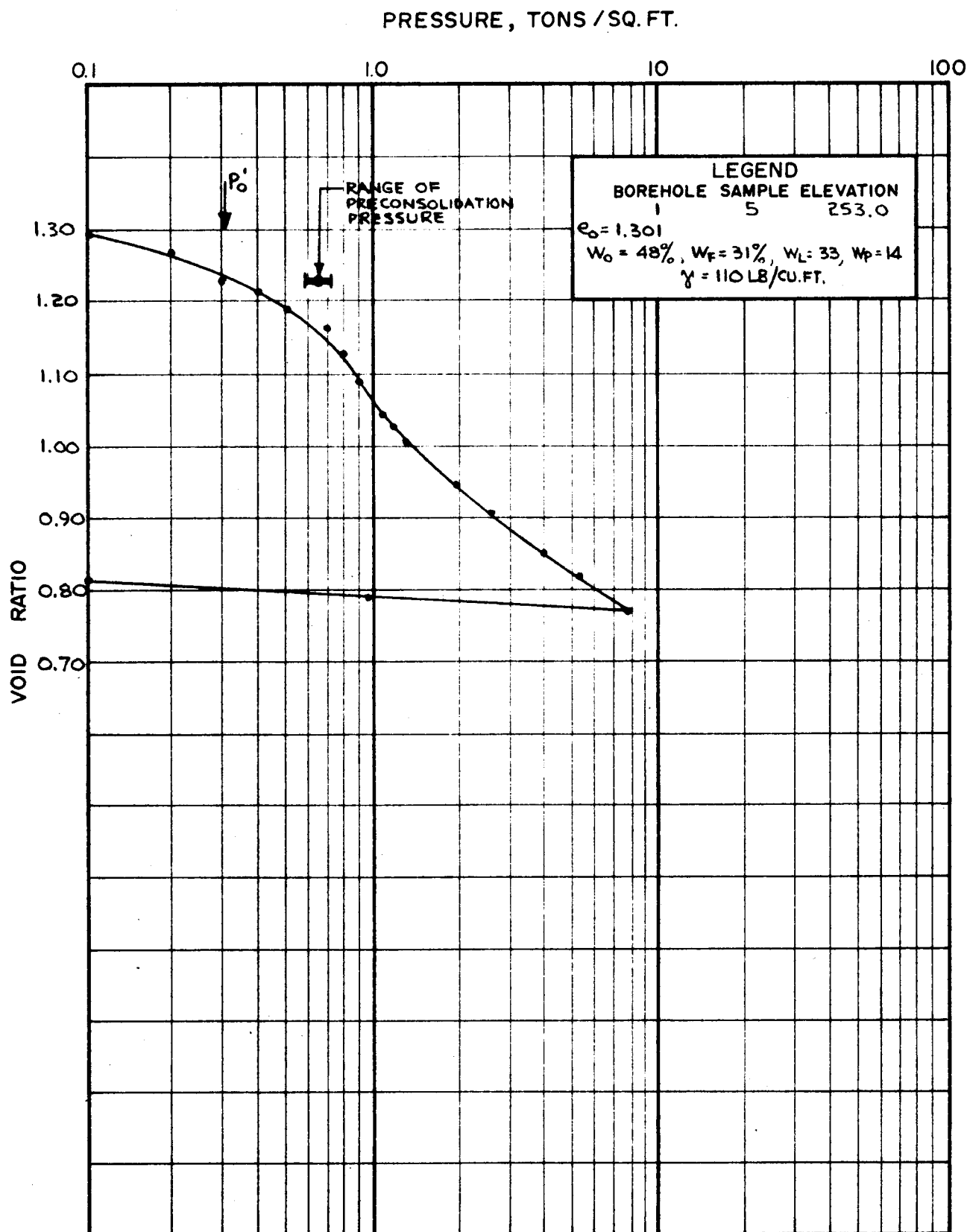
Date MAY 3, 1974

Golden Associates

Drawn: J.A.
Chkd: J.A.
Appd: J.A.
Date: 5/17/74

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

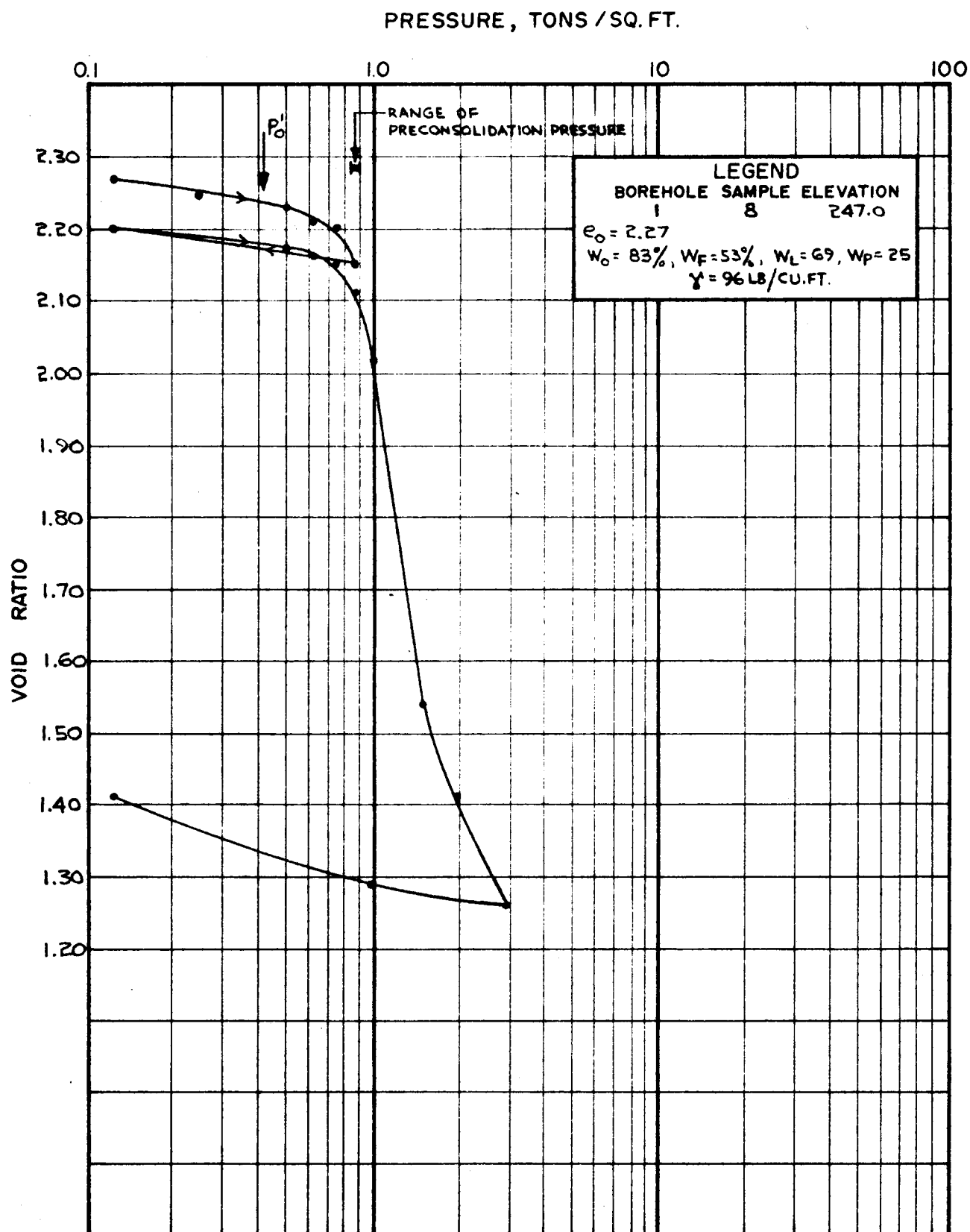
FIGURE 18



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

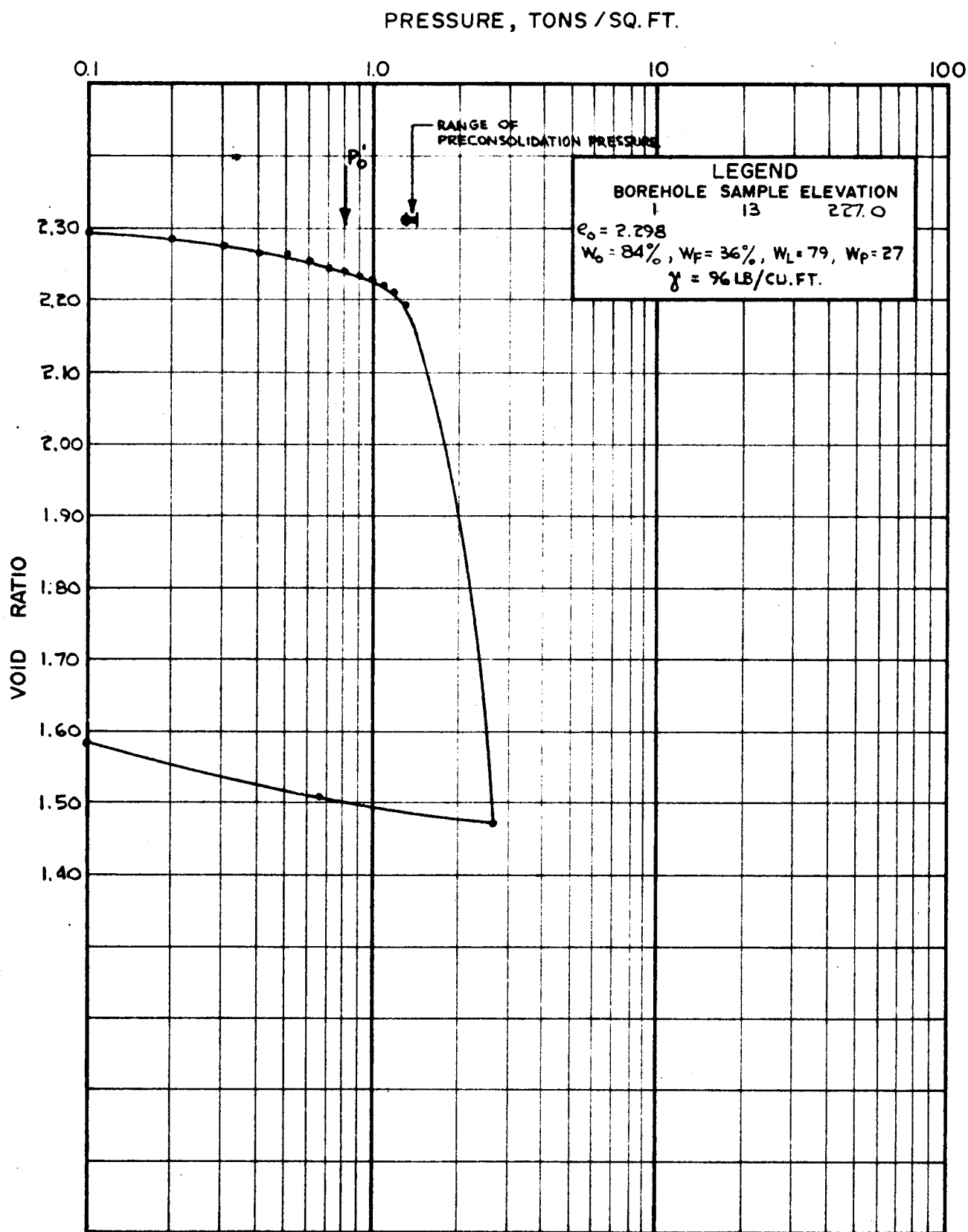
FIGURE 19



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

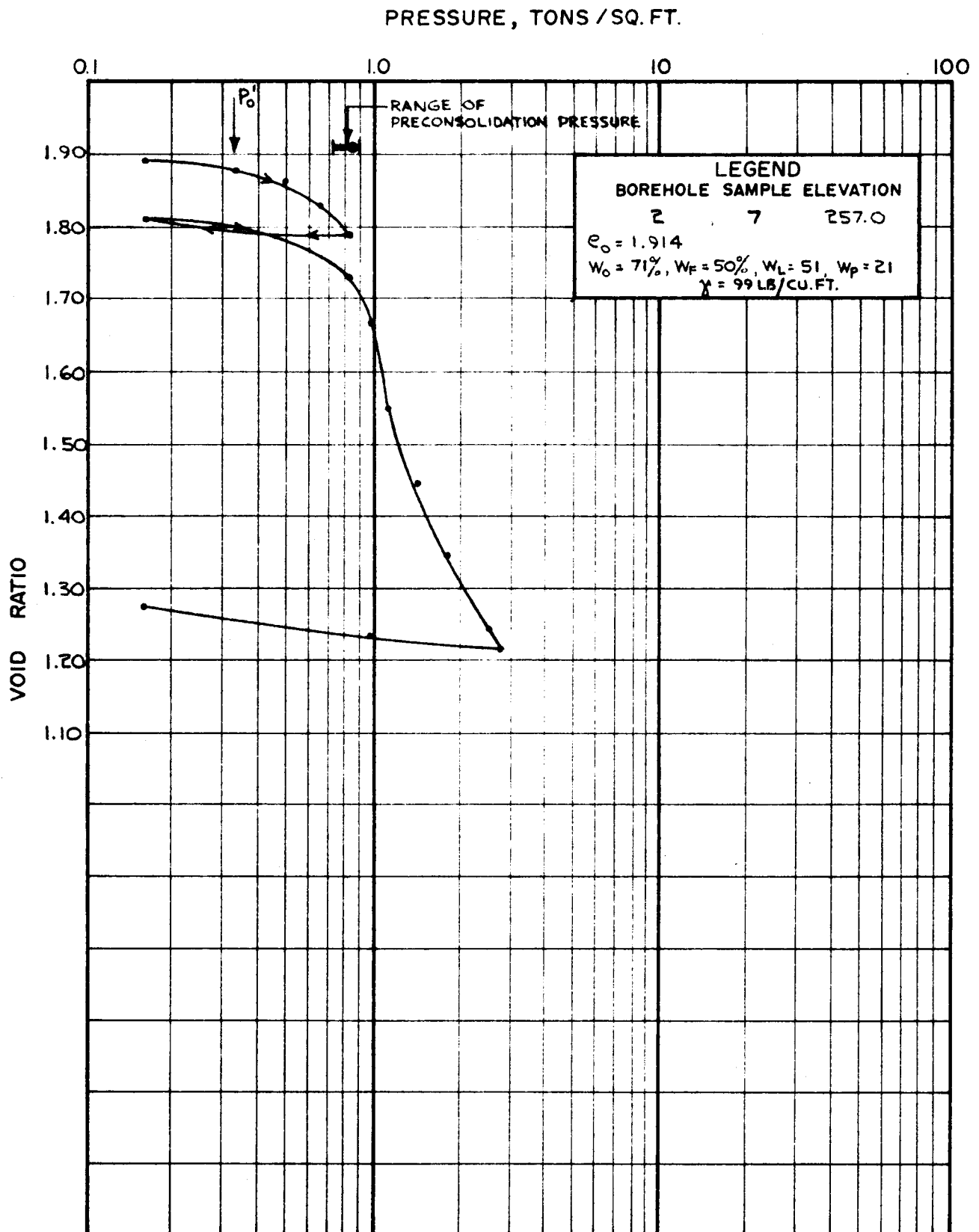
FIGURE 20



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

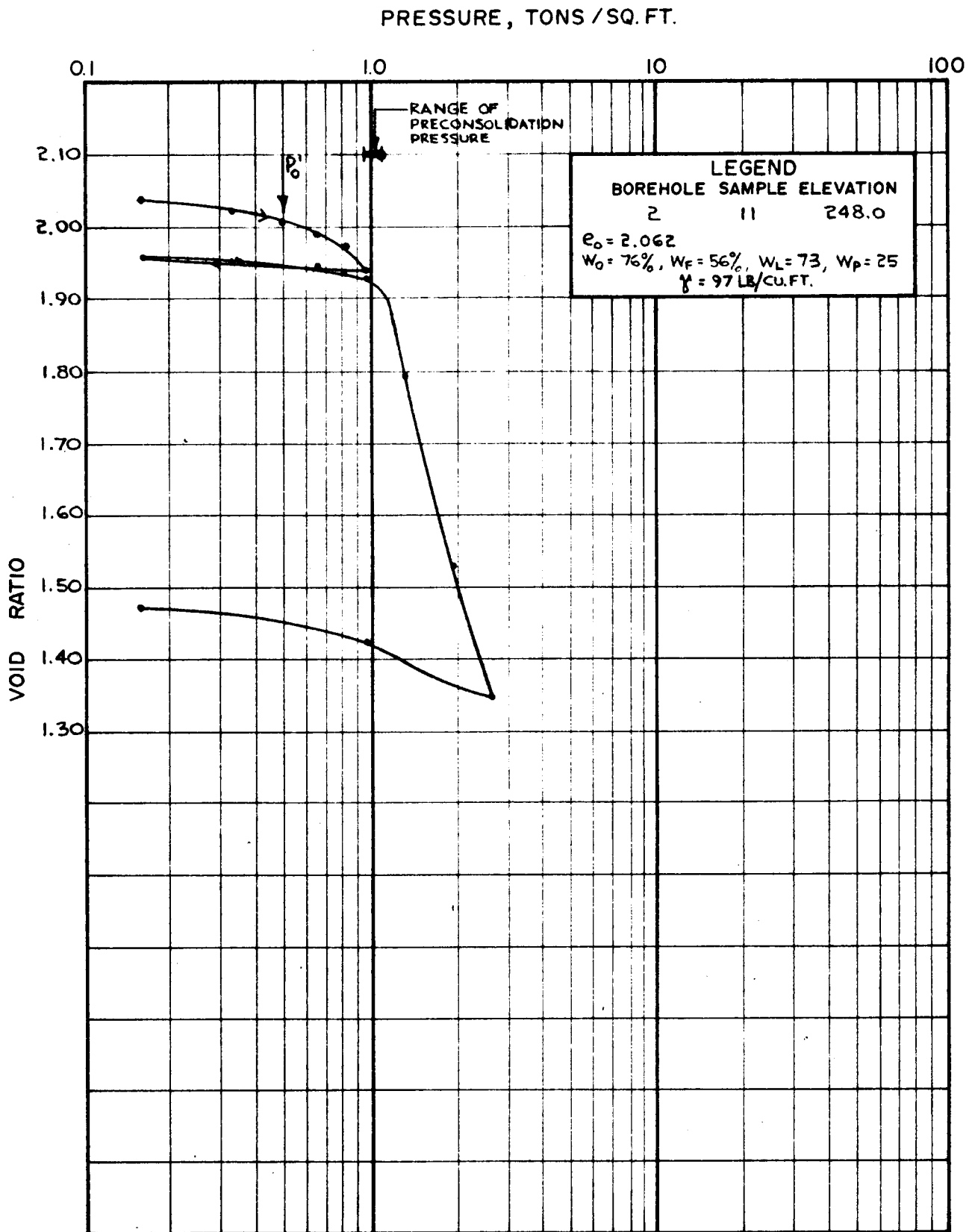
FIGURE 21



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

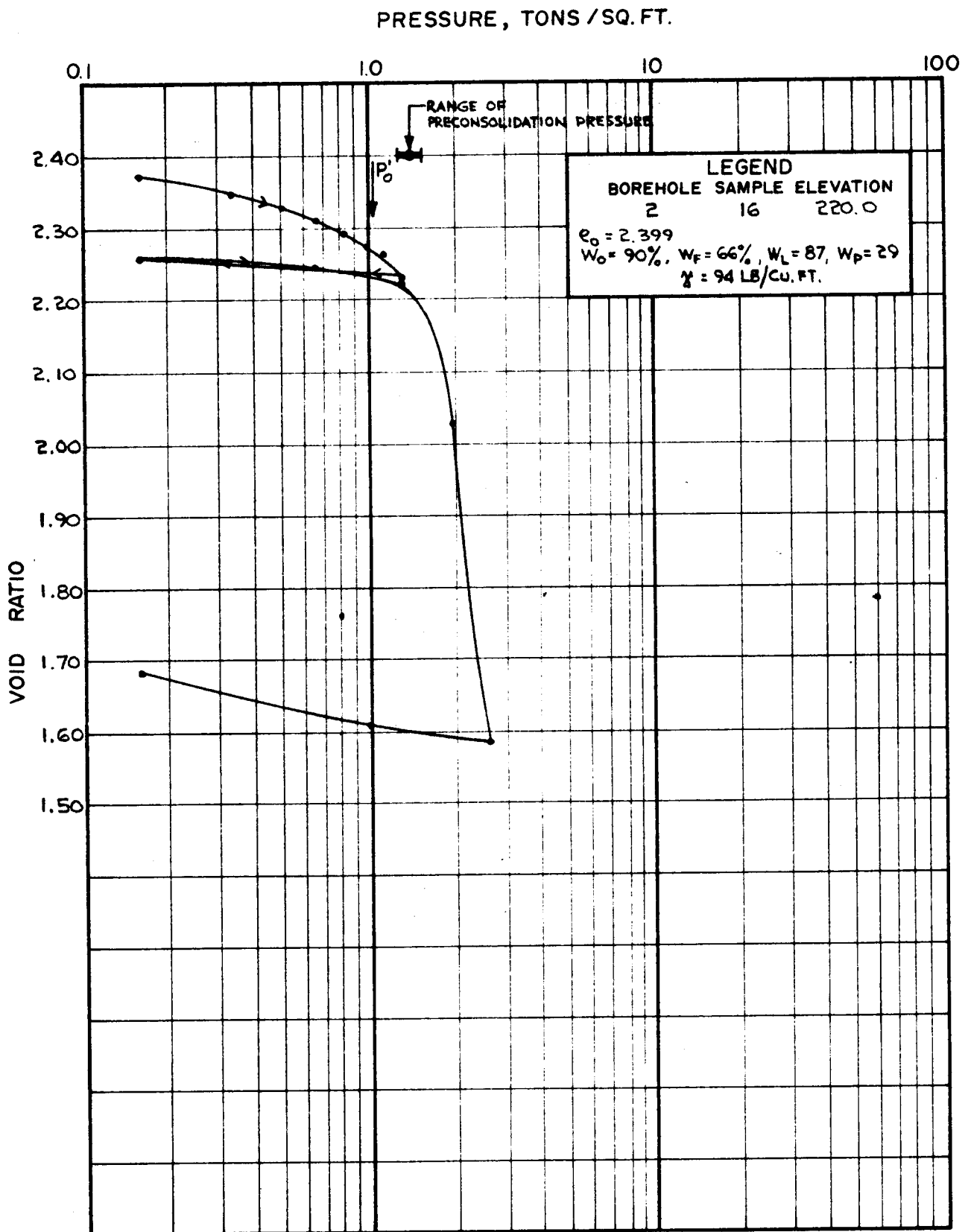
FIGURE 22



Golder Associates

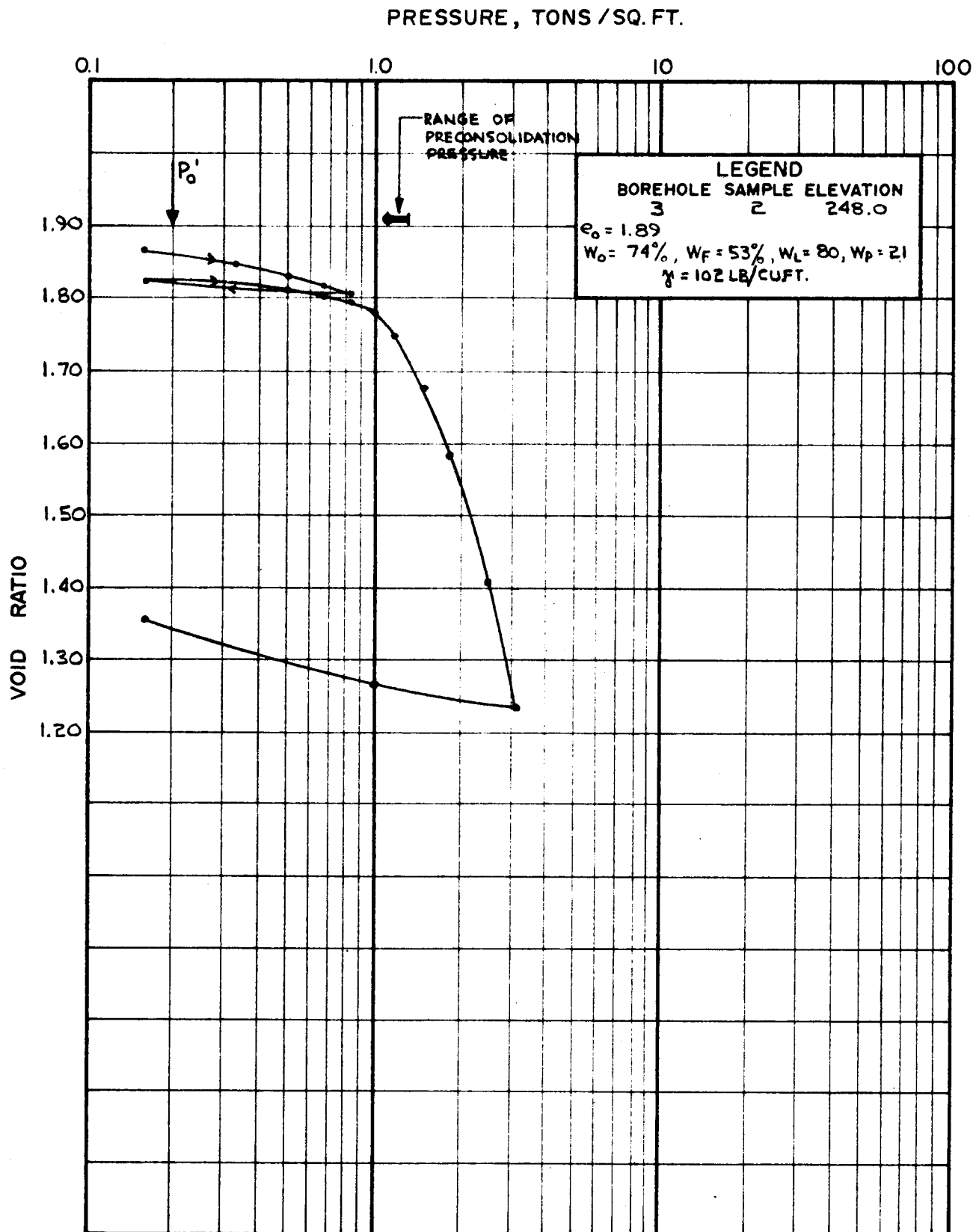
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 23



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

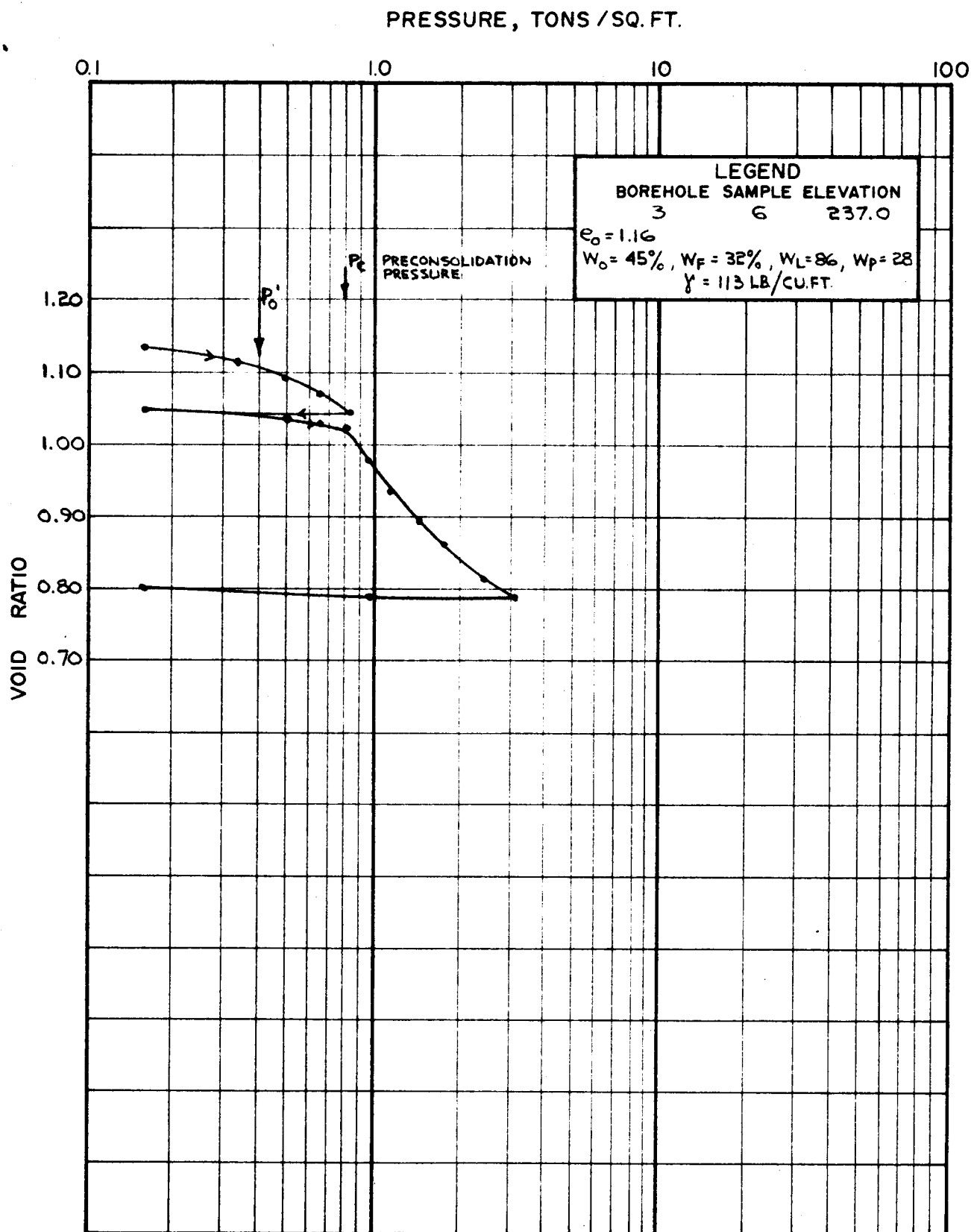
FIGURE 24



Golder Associates

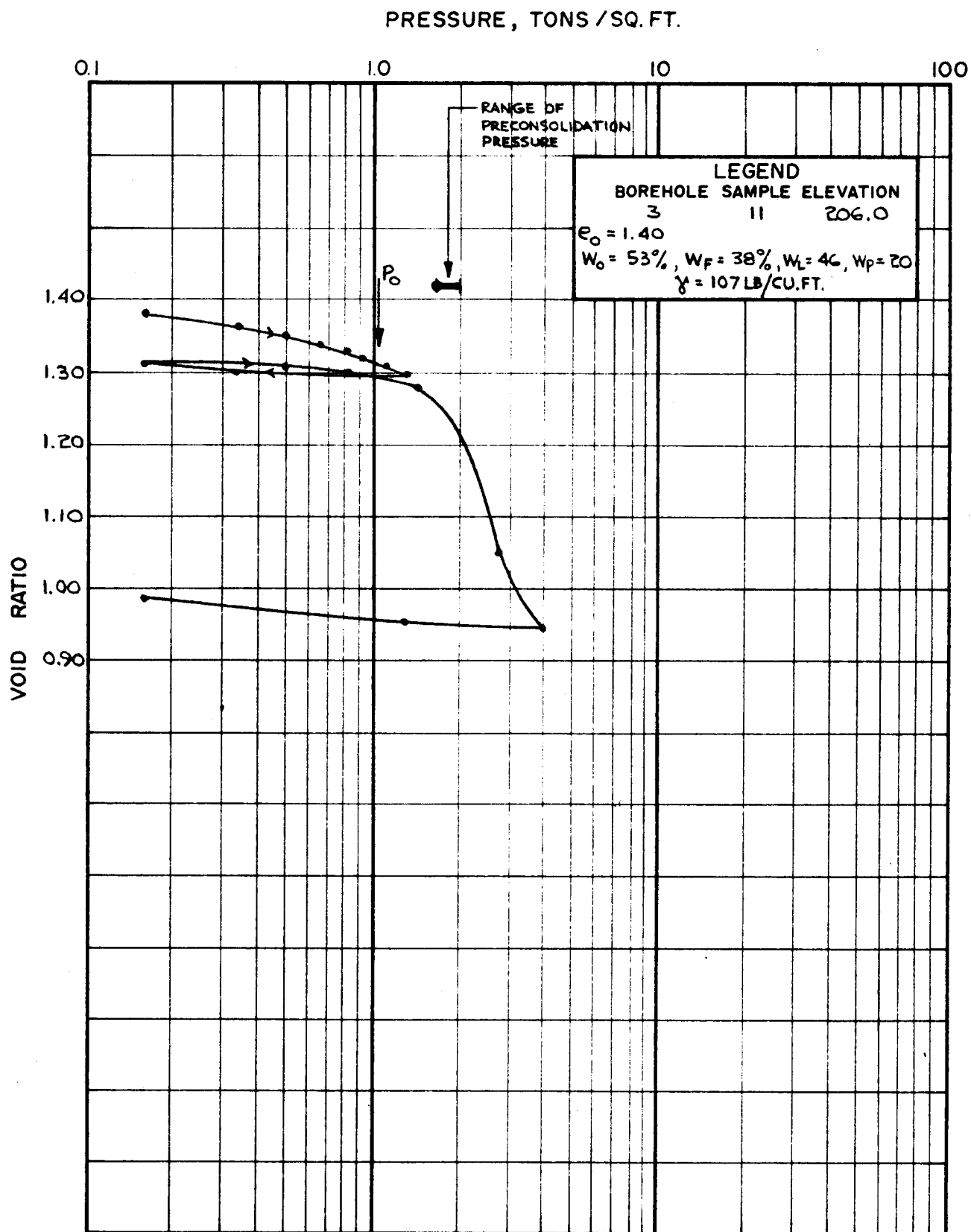
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 25



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

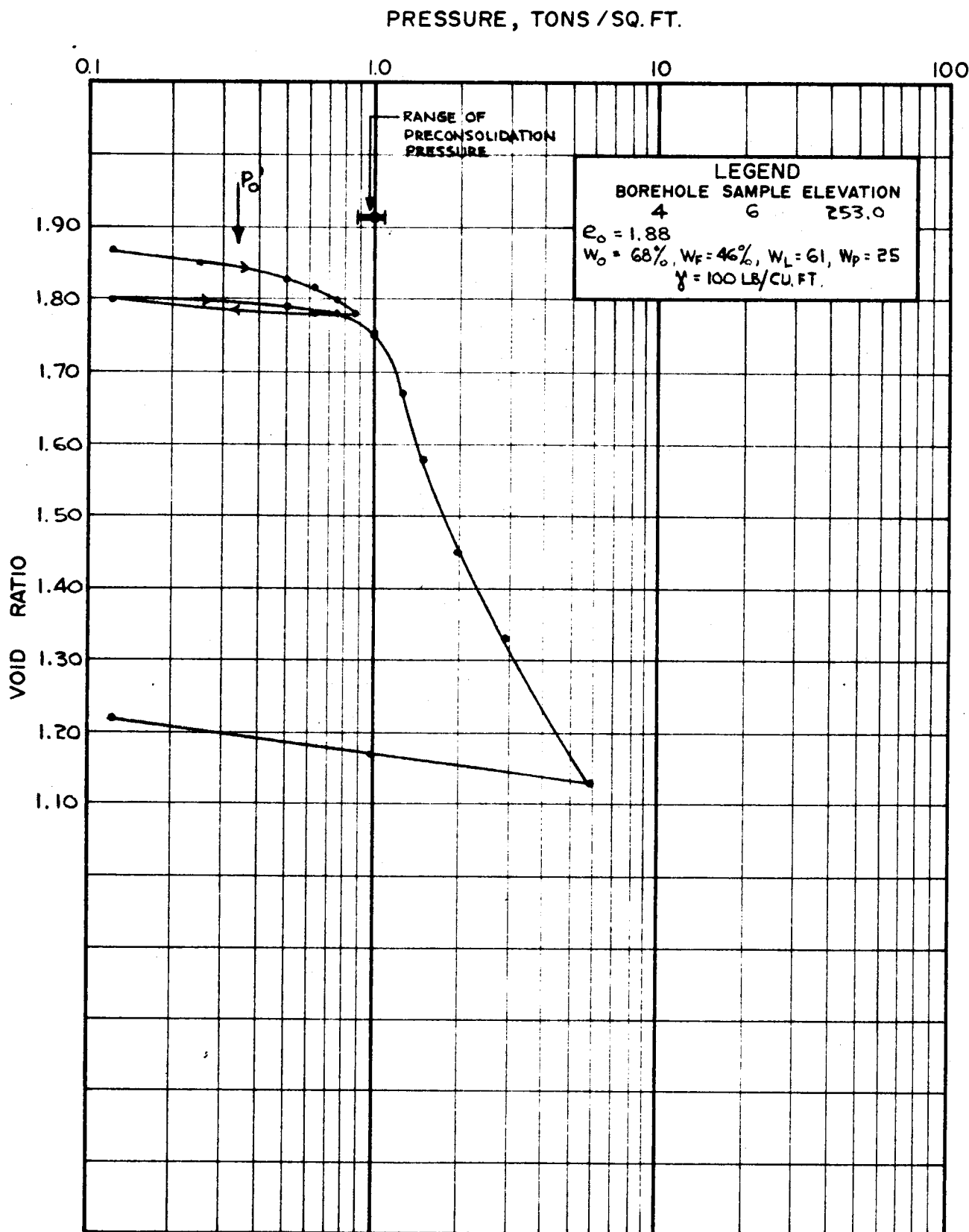
FIGURE 26



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

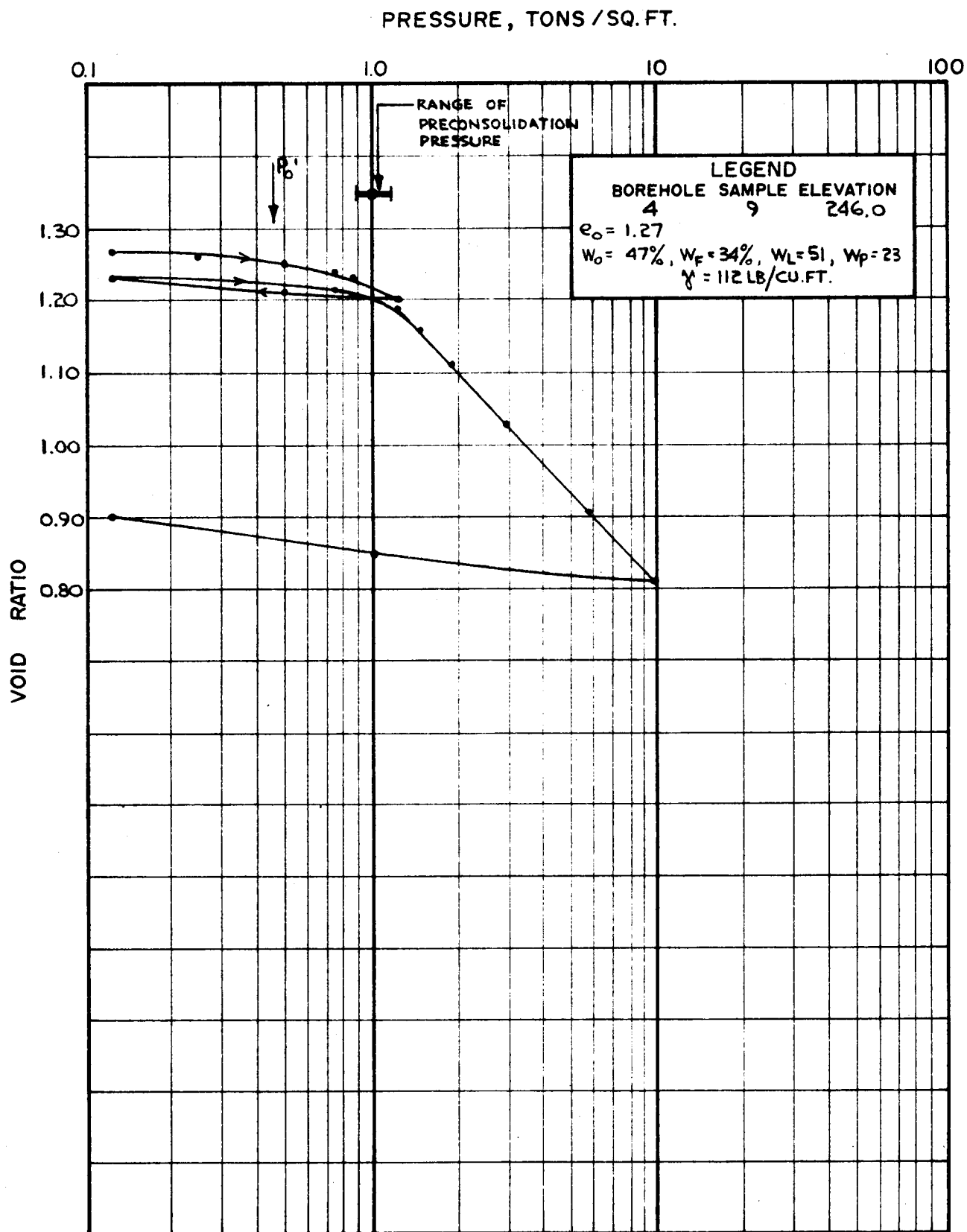
FIGURE 27



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

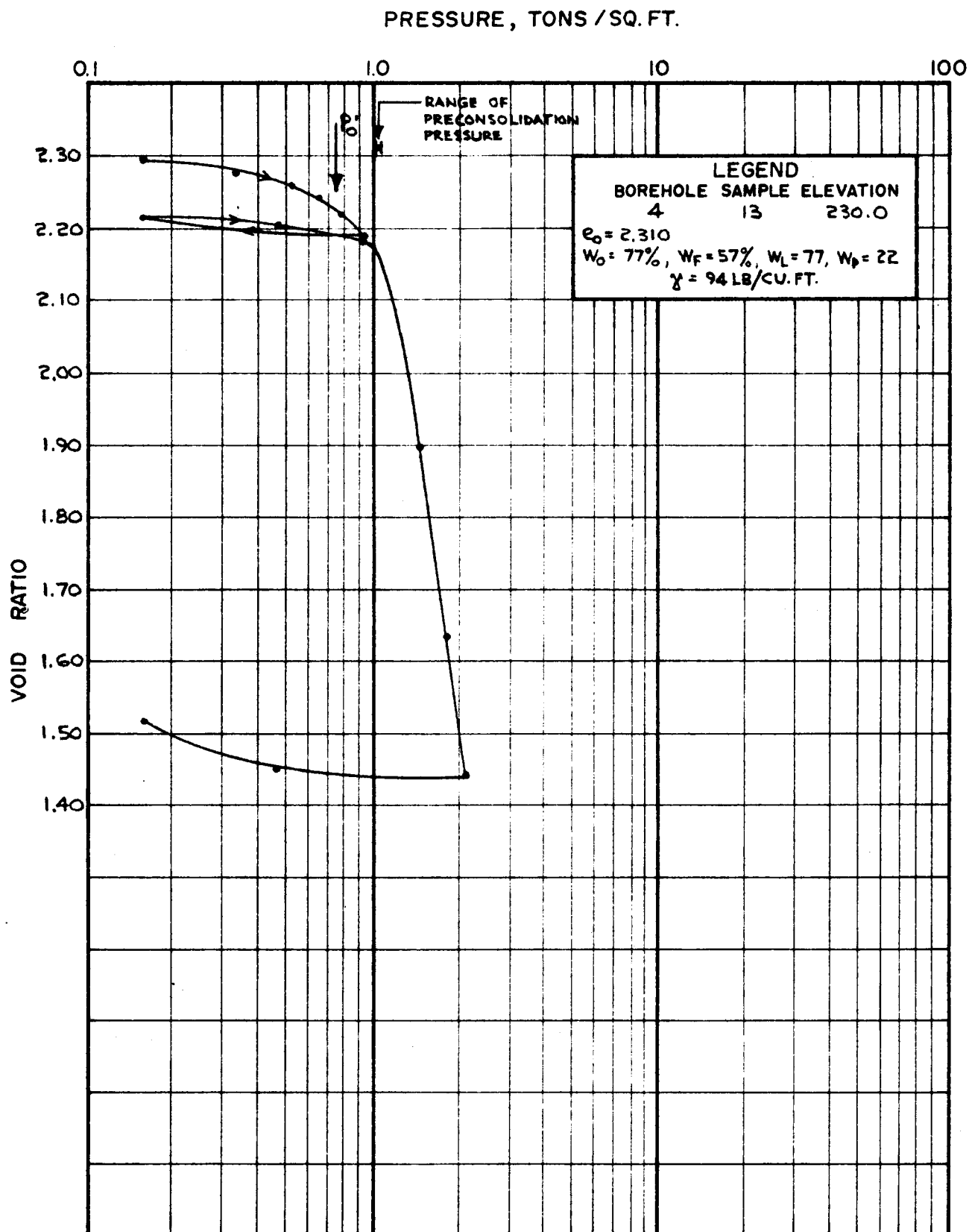
FIGURE 28



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

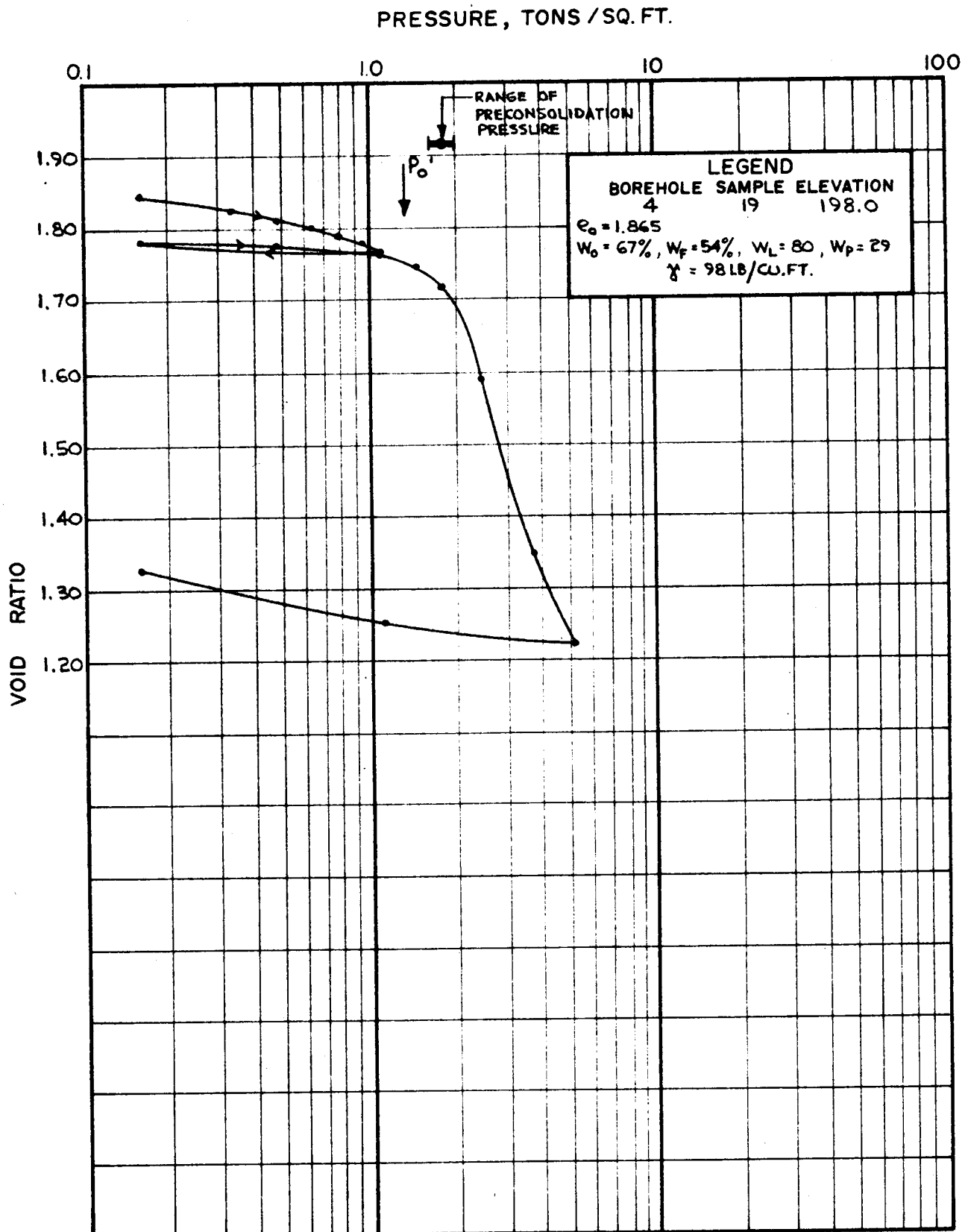
FIGURE 29



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

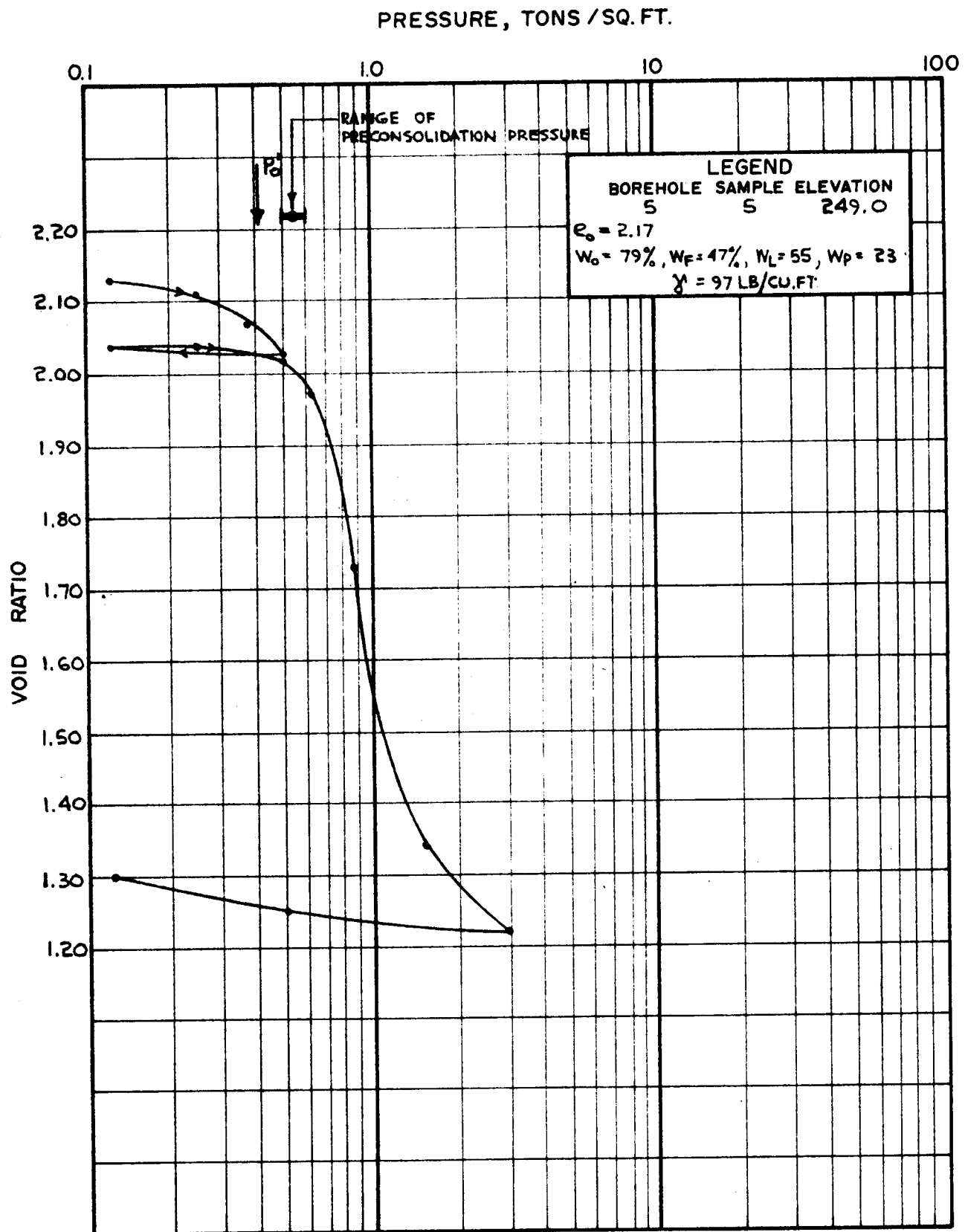
FIGURE 30



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

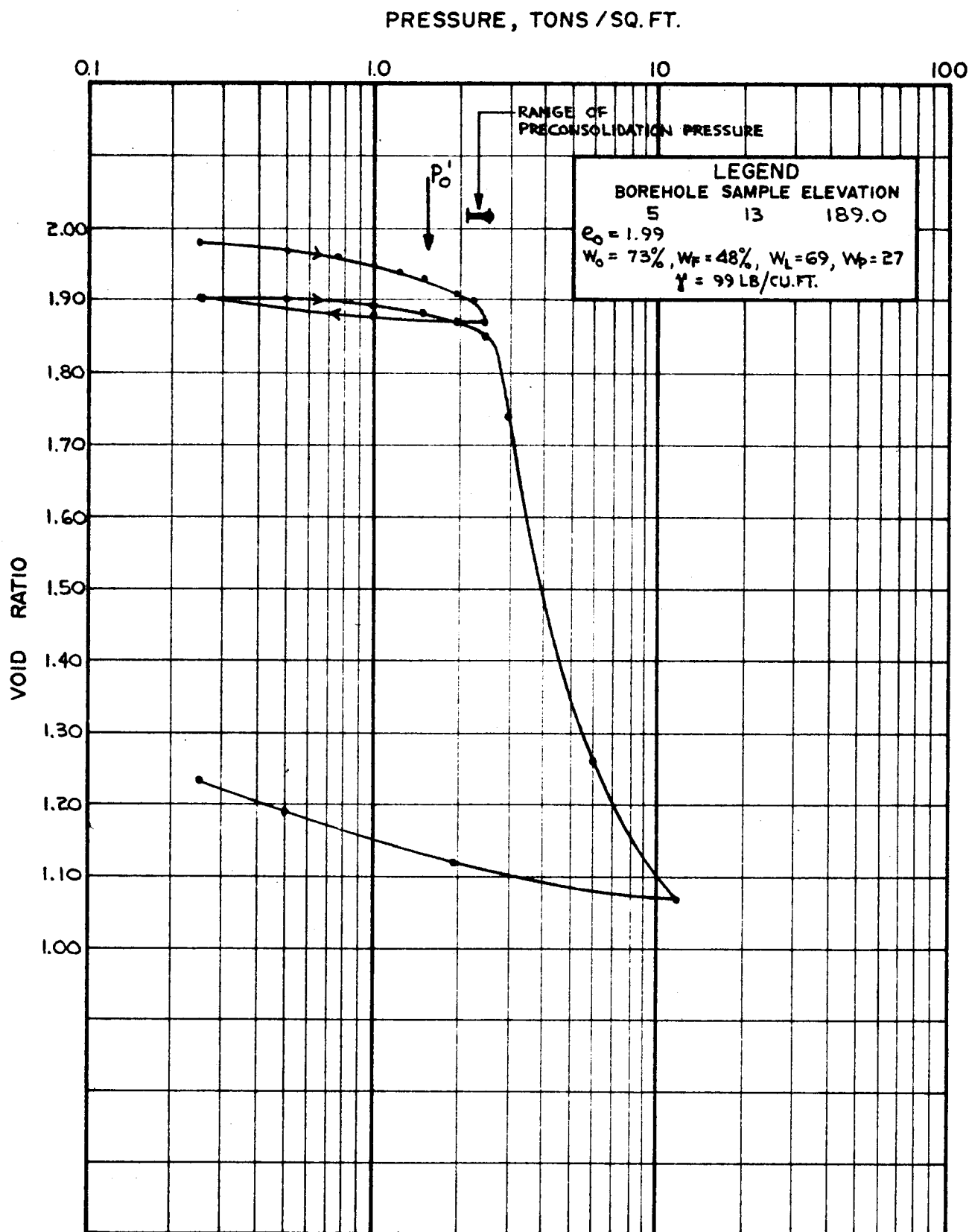
FIGURE 31



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

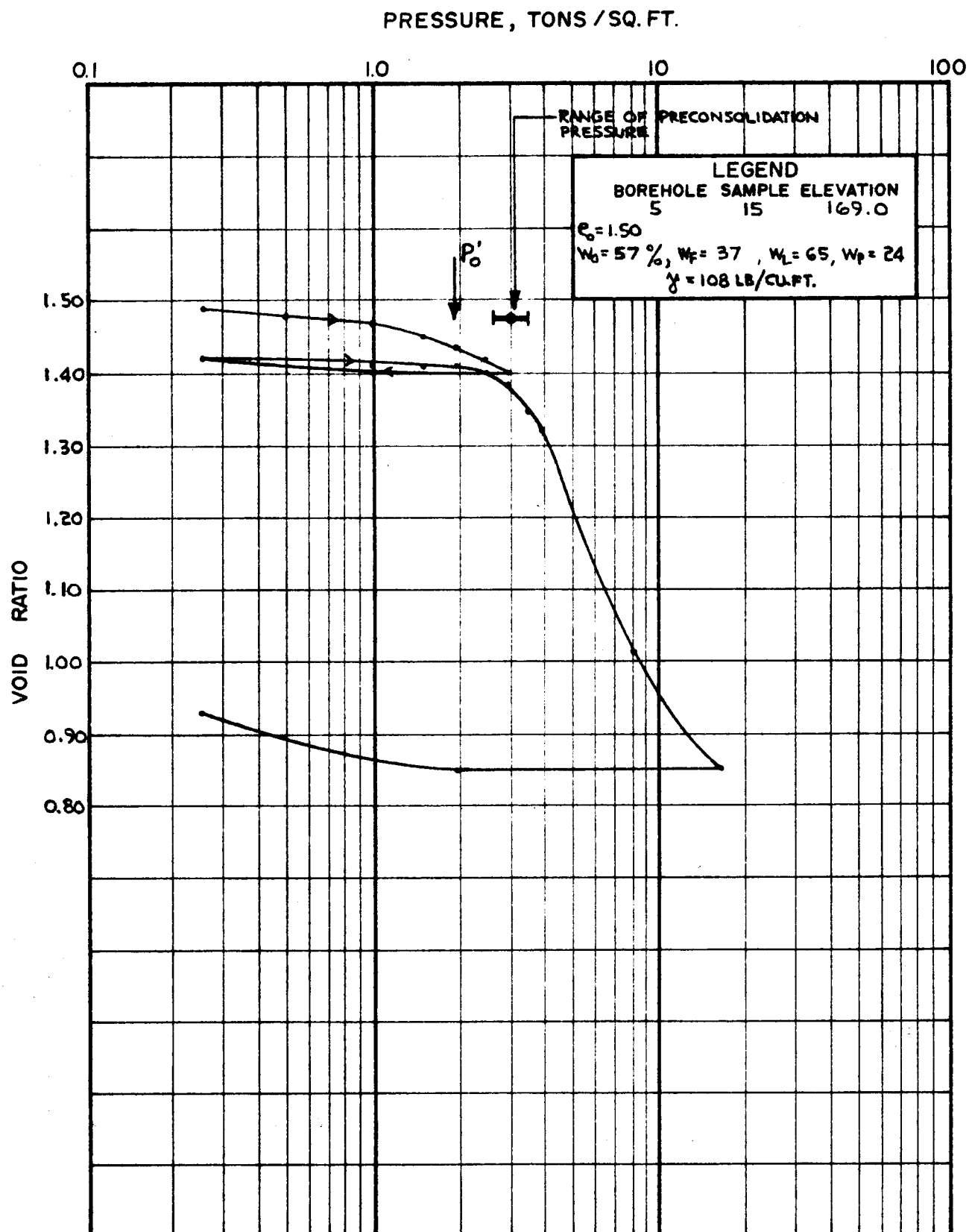
FIGURE 32



Golder Associates

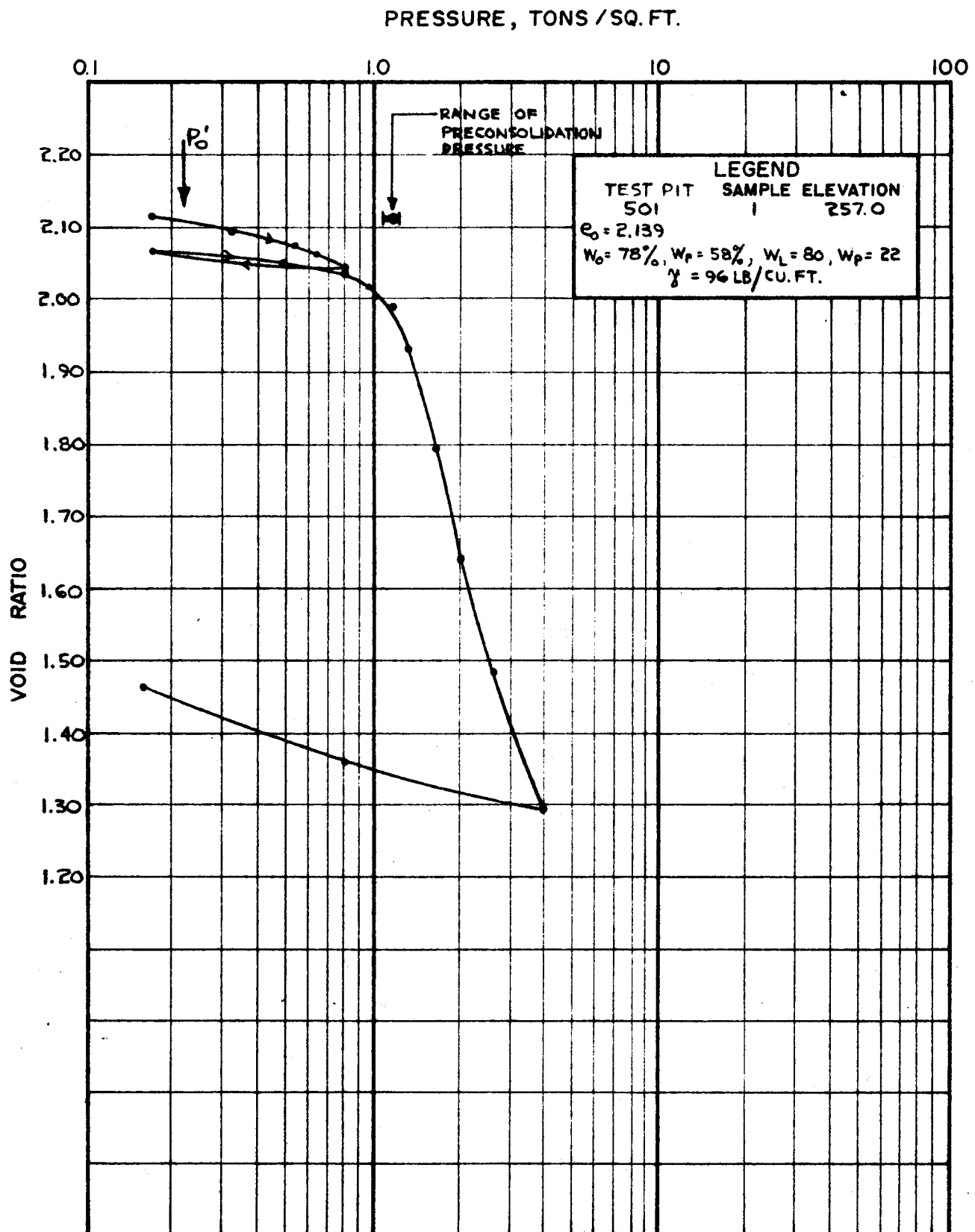
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 33



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST (2" SAMPLE)

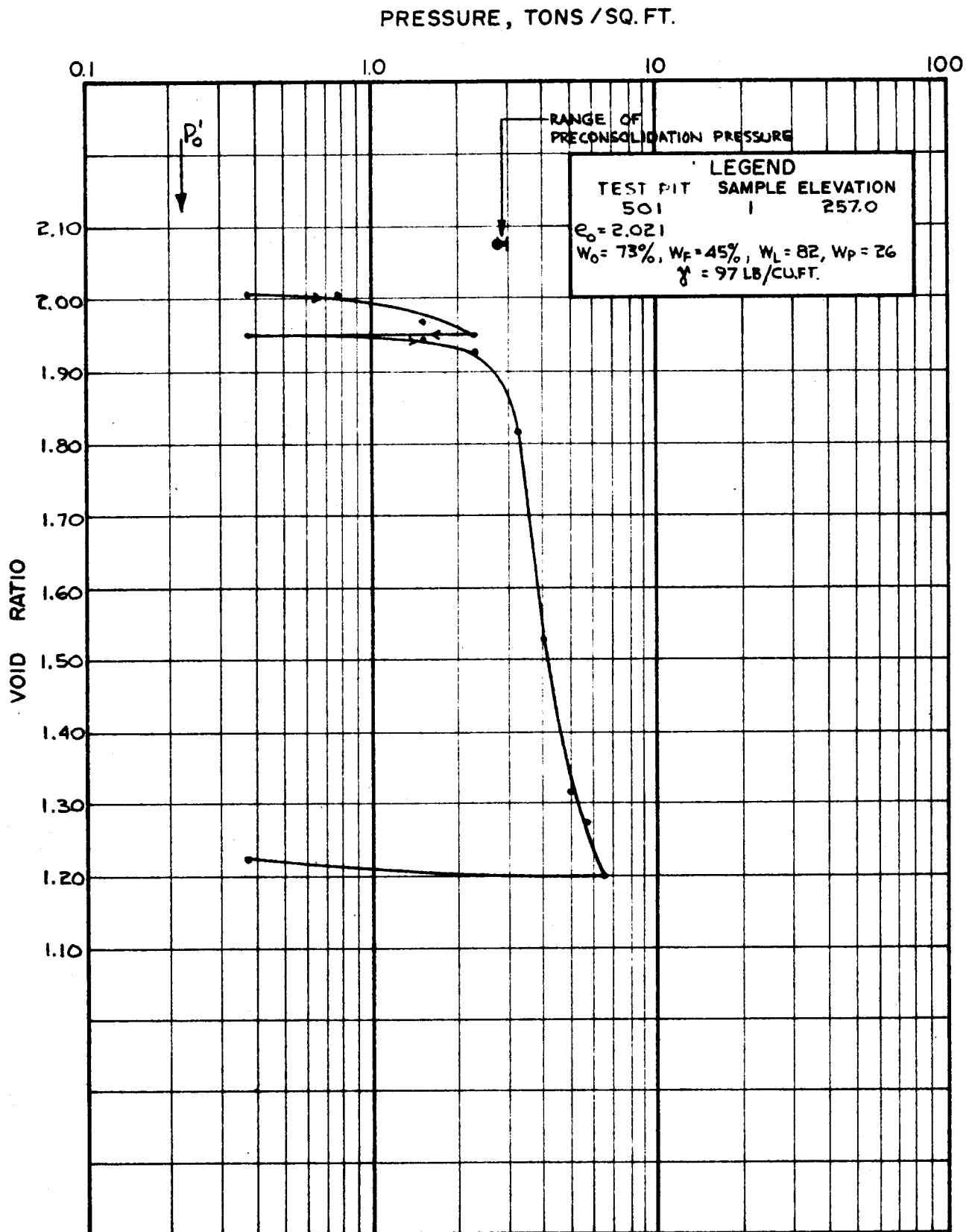
FIGURE 34



Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST (5" SAMPLE)

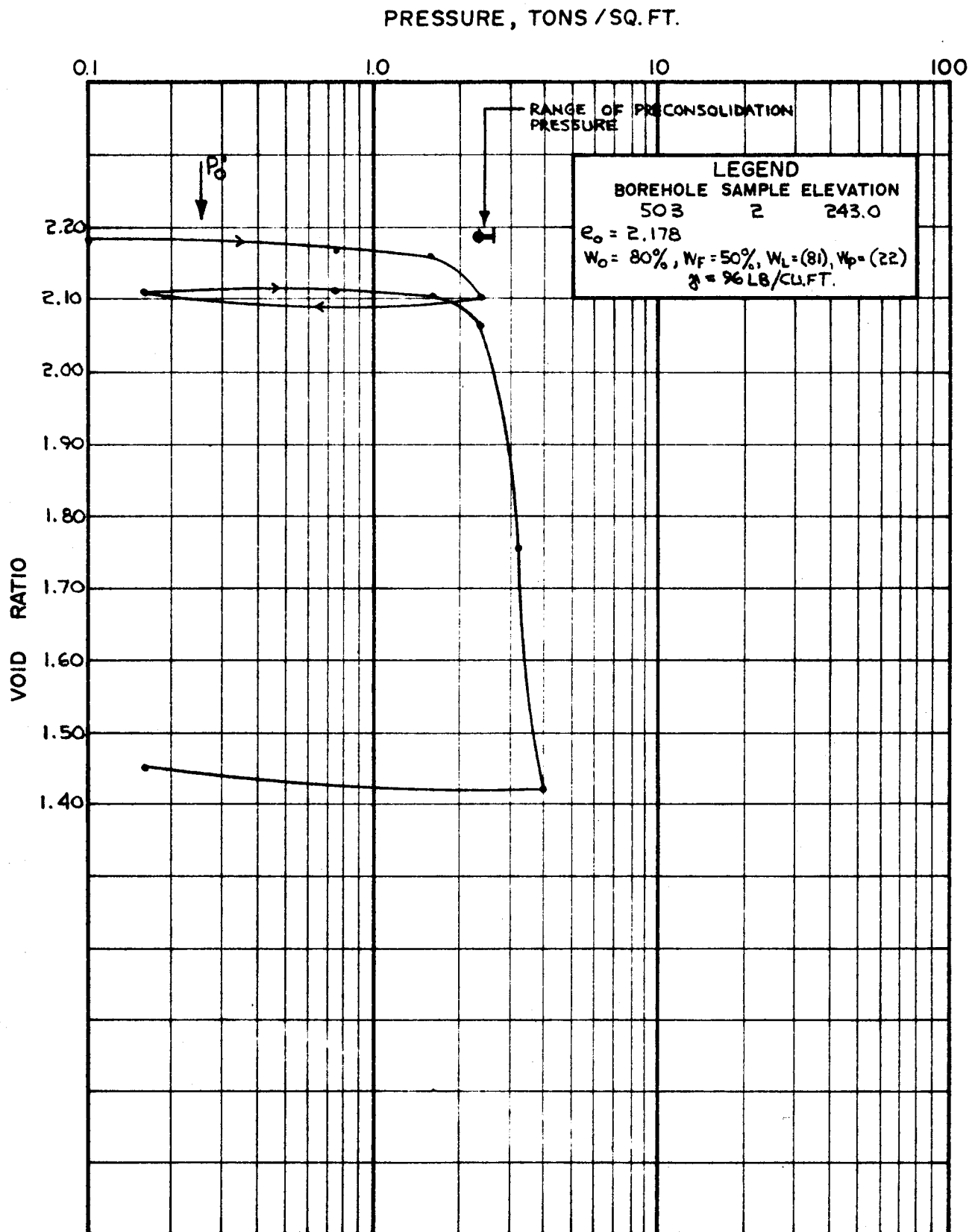
FIGURE 35



Golder Associates

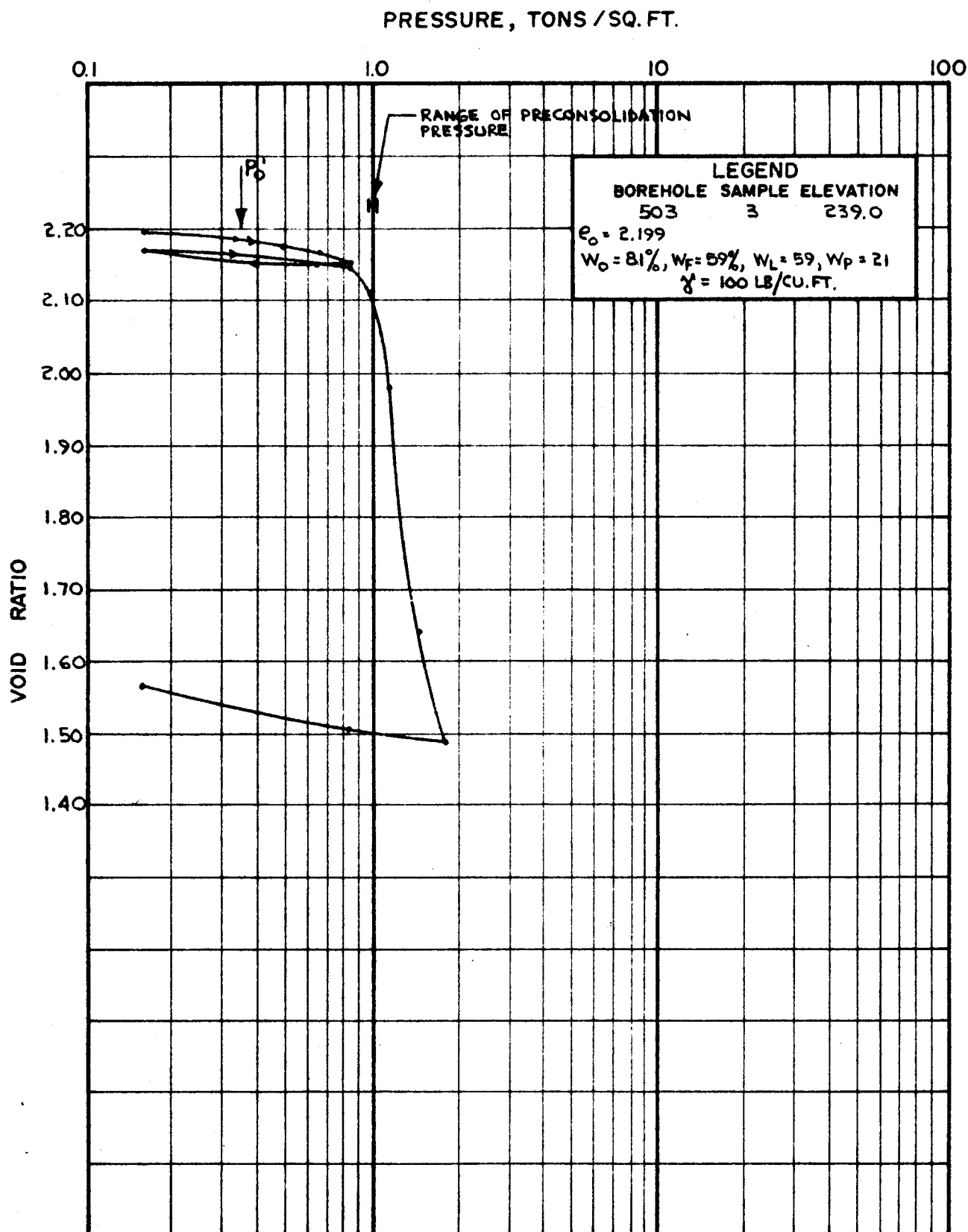
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST(5" SAMPLE)

FIGURE 36



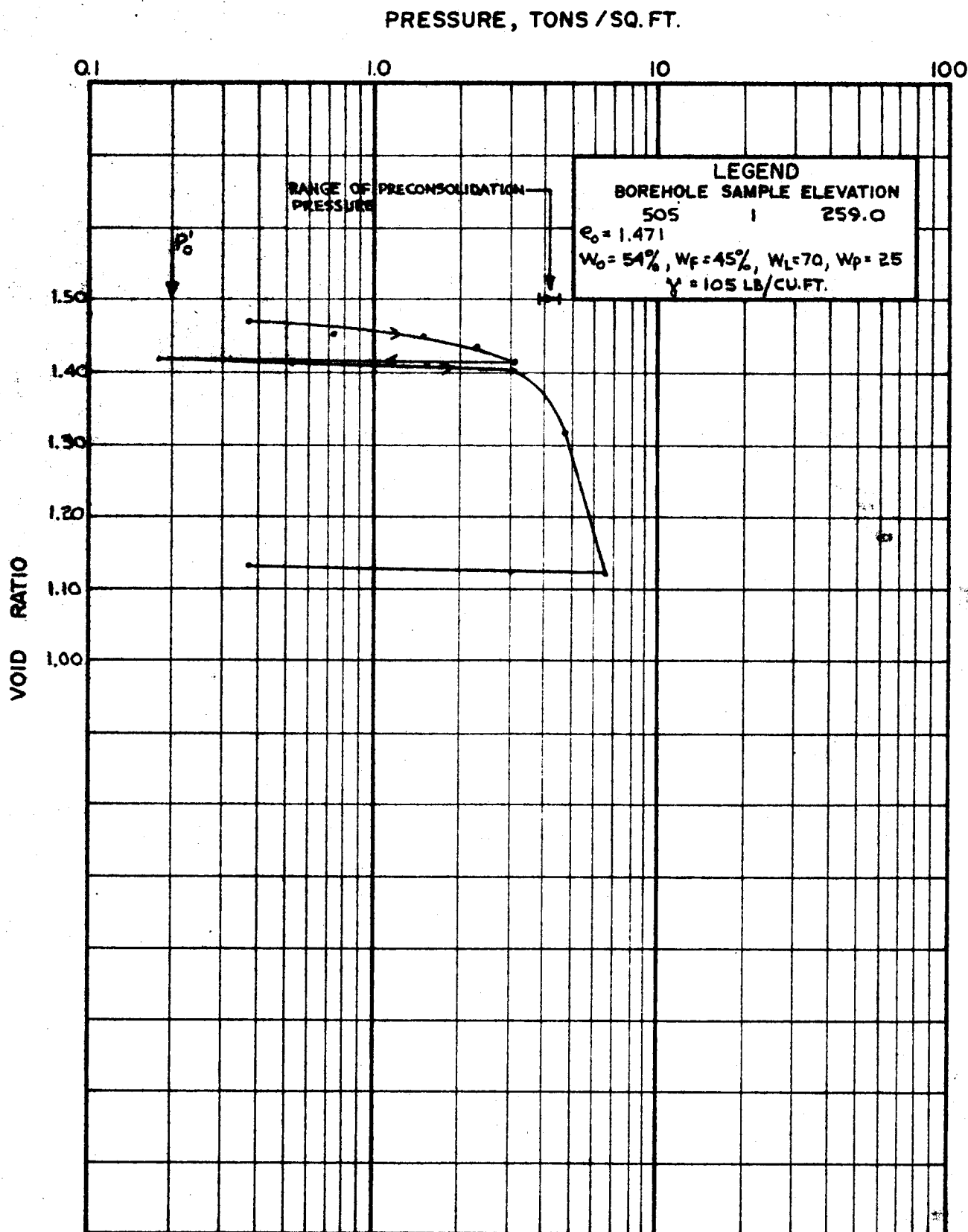
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 37



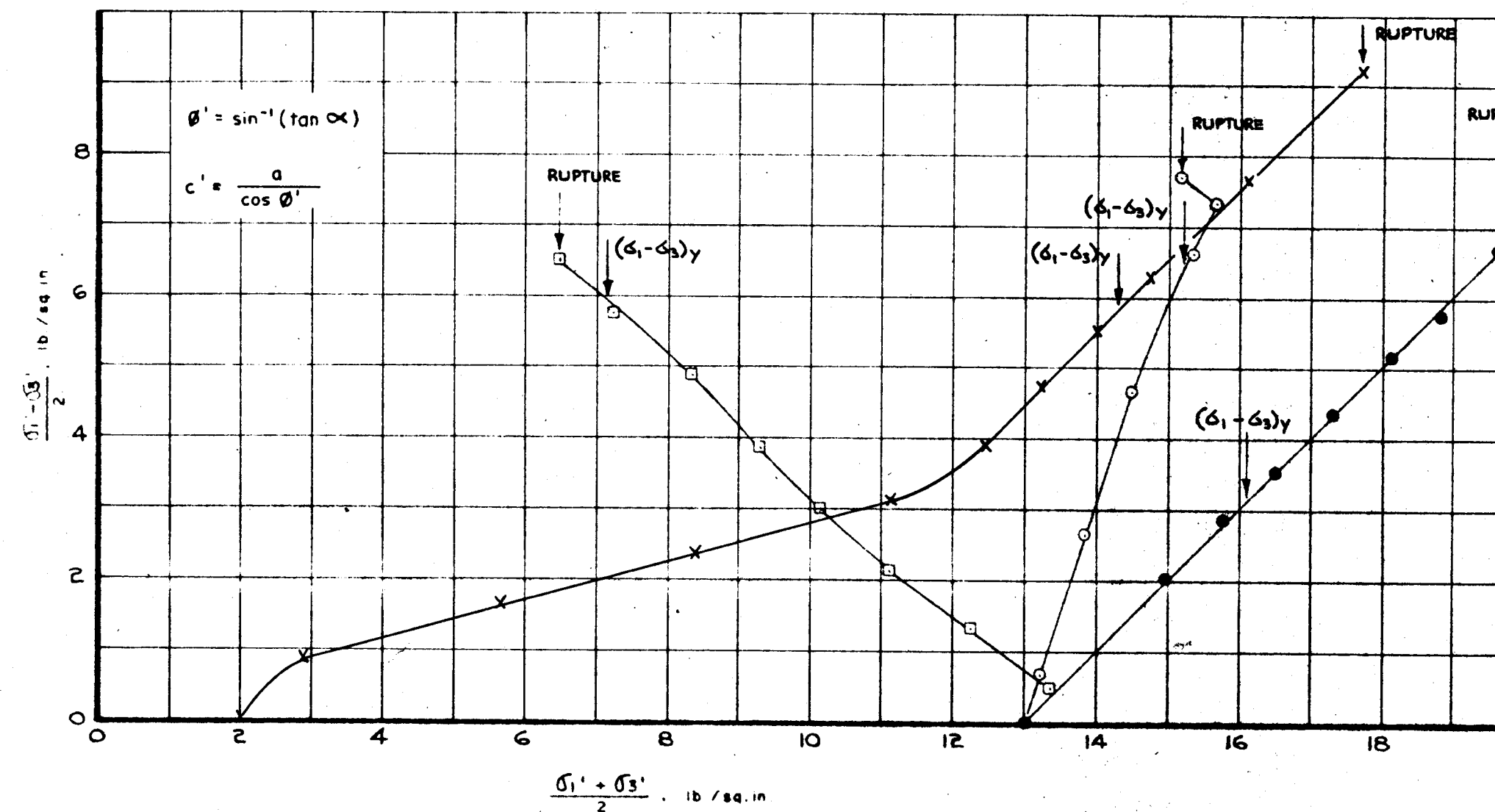
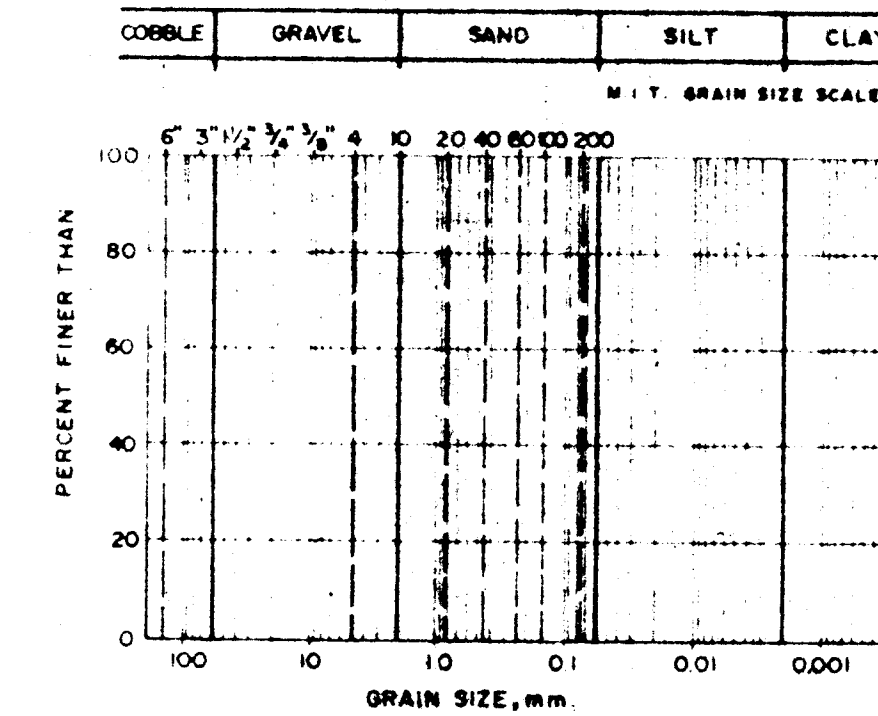
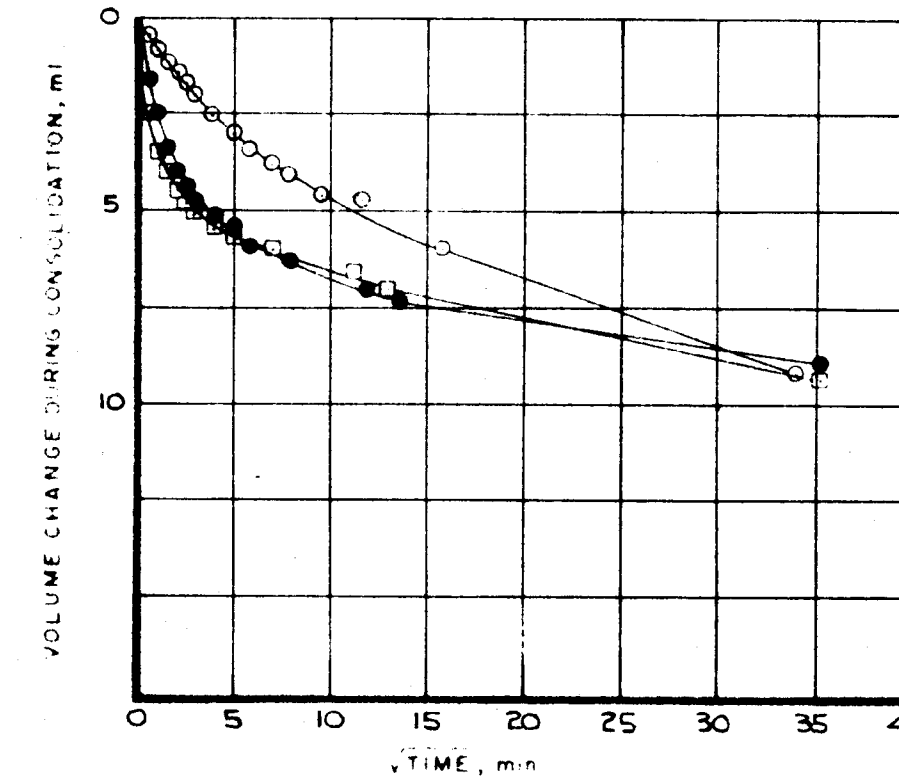
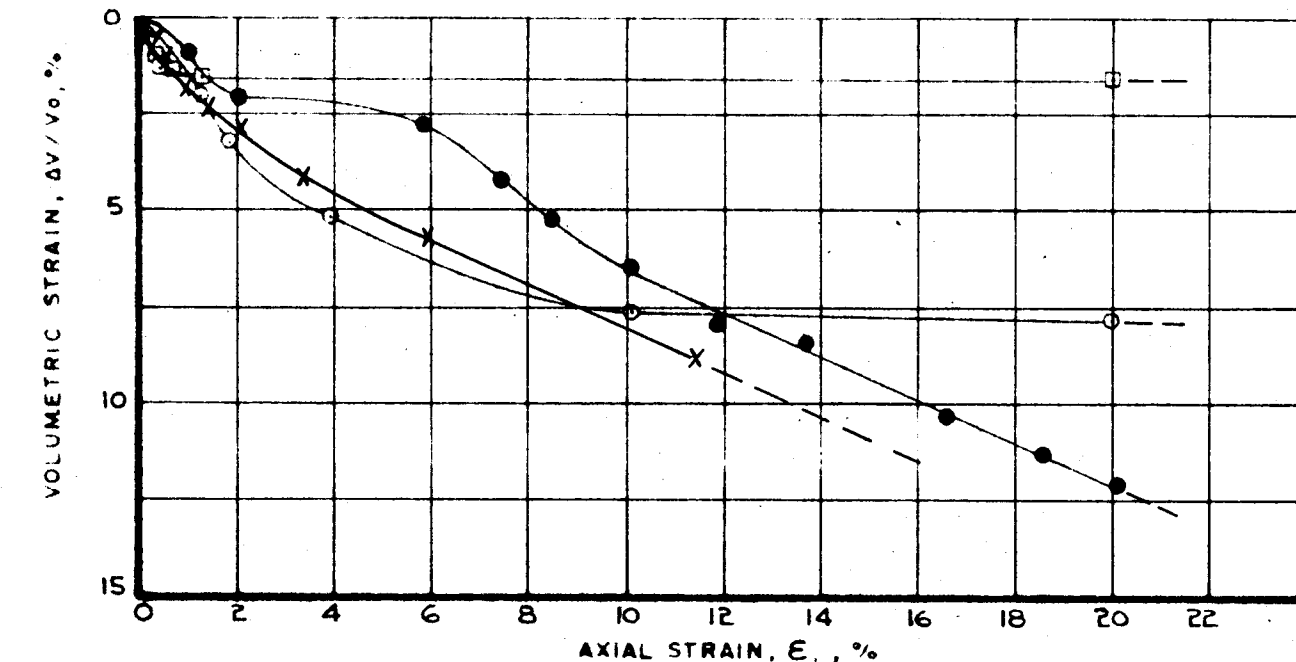
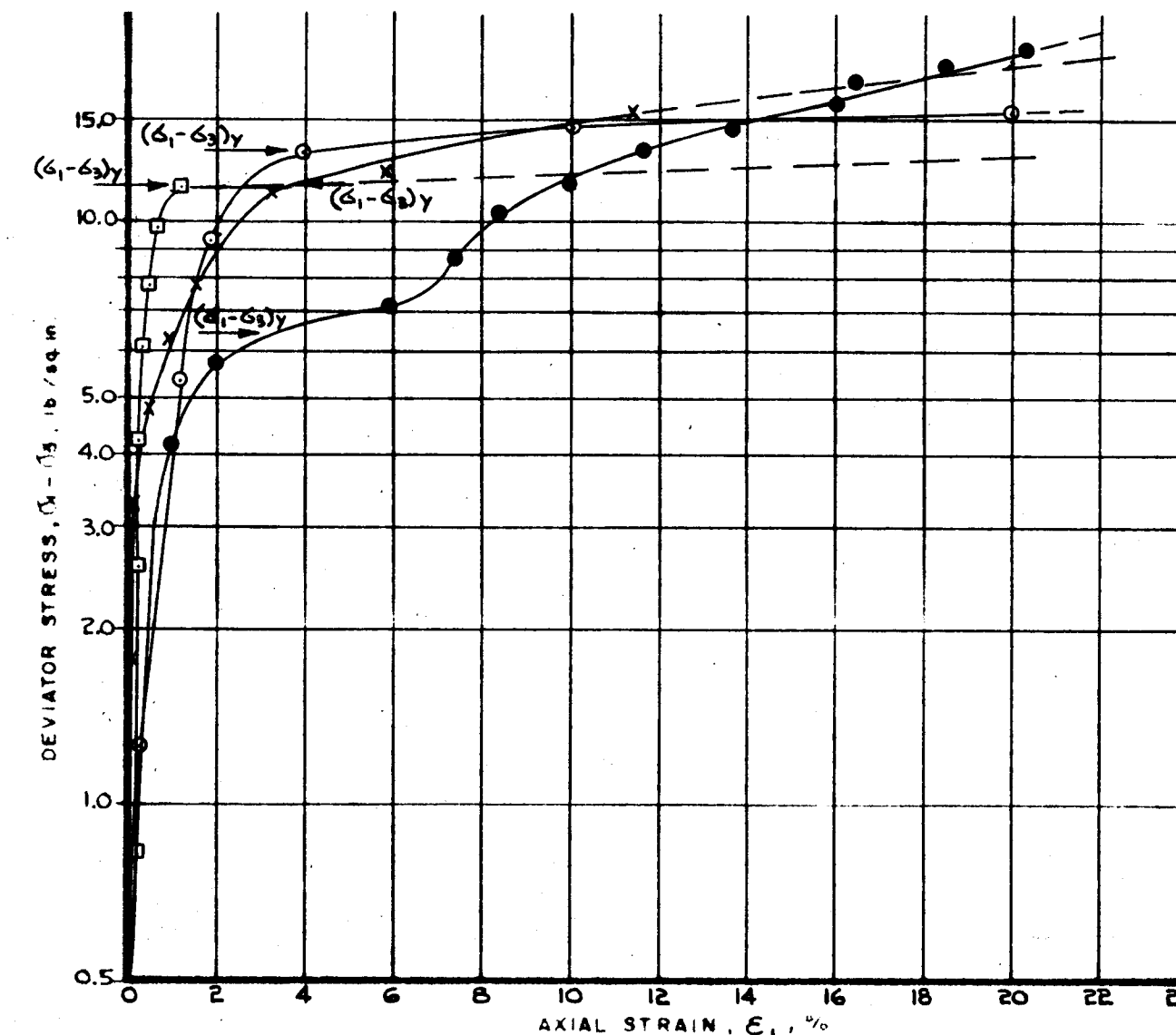
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 38



S TESTS CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TESTS DESICCATED CRUST

FIGURE 39



	a	b	c	d
BOREHOLE NUMBER	501	501	501	501
SAMPLE NUMBER	1	1	1	1
SAMPLE DEPTH, ft	6.2	6.2	6.2	6.0

SPECIMEN DIAMETER, in.	2.01	2.0	2.0	2.0
SPECIMEN HEIGHT, in.	4.0	4.0	4.0	4.0

WATER CONTENT, BEFORE CONSOLIDATION, %	62	62	67	65
CELL PRESSURE, σ₃, lb./sq. in.	13	22	22	8.5
BACK PRESSURE, lb./sq. in.	0	9	9	0
PURE PRESSURE PARAMETER 'B'	0.78	0.98	0.97	—
CONSOLIDATION PRESSURE, (σ₃)ᵧ, lb./sq. in.	13	13	13	8.5
VOLUME CHANGE DURING CONSOLIDATION, ΔV, ml	-9.3	-9.4	-9.0	—
WATER CONTENT, AFTER CONSOLIDATION, %	60	57	63	—
AVERAGE RATE OF STRAIN, % / hr.	—	—	—	—
AVERAGE LOAD INCREMENT, lb./sq. in.	4	2	2	1.5
AVERAGE LOAD DURATION, hr.	24	24	24	24
TIME TO FAILURE, days	6	7	12	8
WATER CONTENT, AFTER TEST, %	55	58	49	54

MAX. DEVIATOR STRESS (σ₁ - σ₃)ᵧ, lb./sq. in.	15.4	13.2	20.0	17.7
AXIAL STRAIN AT (σ₁ - σ₃)ᵧ, %	20.0	20.0	20.3	11.6
MAX. EFFECTIVE PRINCIPAL STRESS RATIO (σ₁' / σ₃')ᵧ, lb./sq. in.	—	—	—	—
AXIAL STRAIN AT (σ₁' / σ₃')ᵧ, %	—	—	—	—
DEVIATOR STRESS AT YIELD, (σ₁ - σ₃)ᵧ, lb./sq. in.	13.2	11.5	6.1	11.8
AXIAL STRAIN AT (σ₁ - σ₃)ᵧ, %	2.4	1.2	2.5	4.0

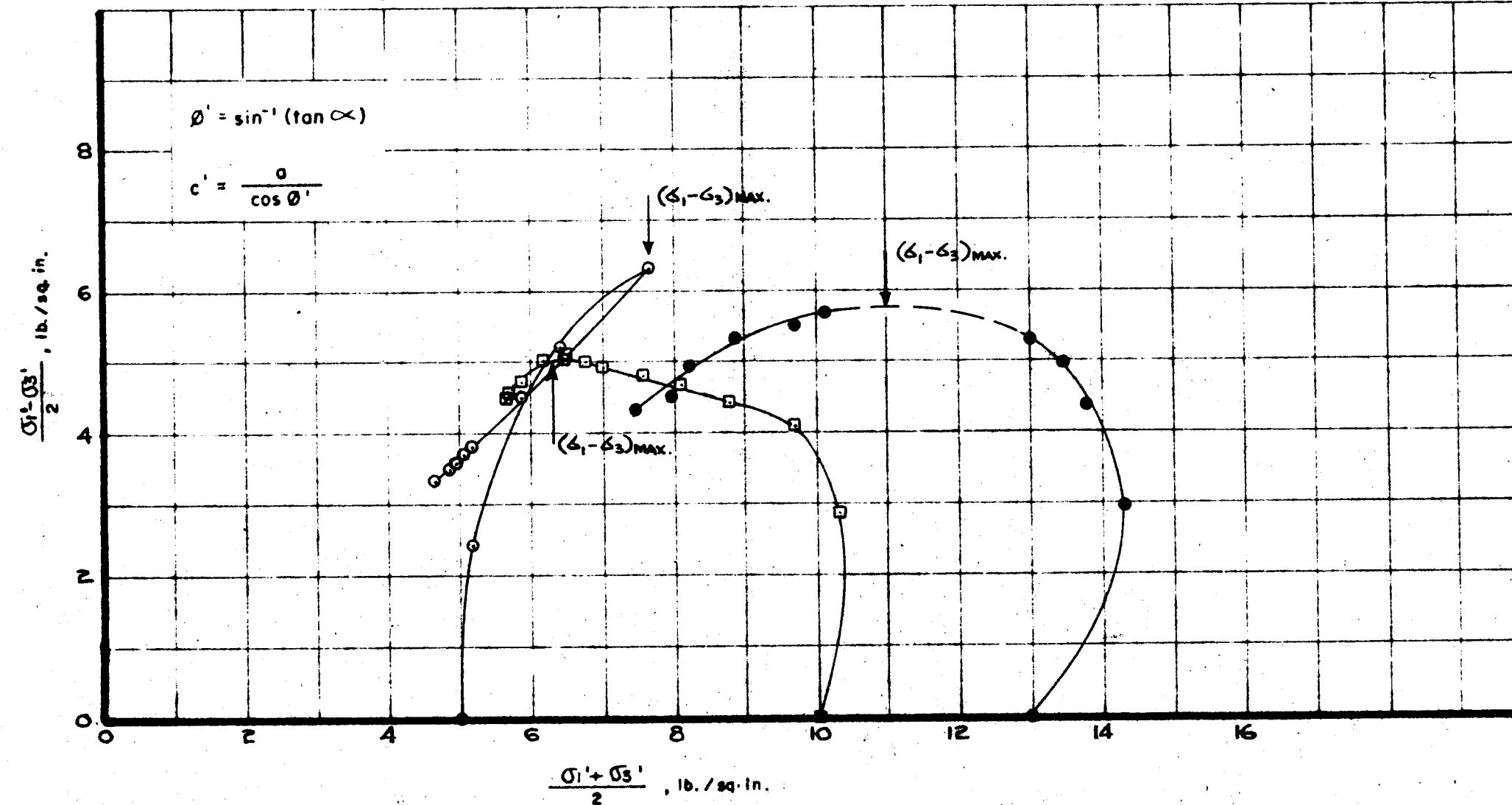
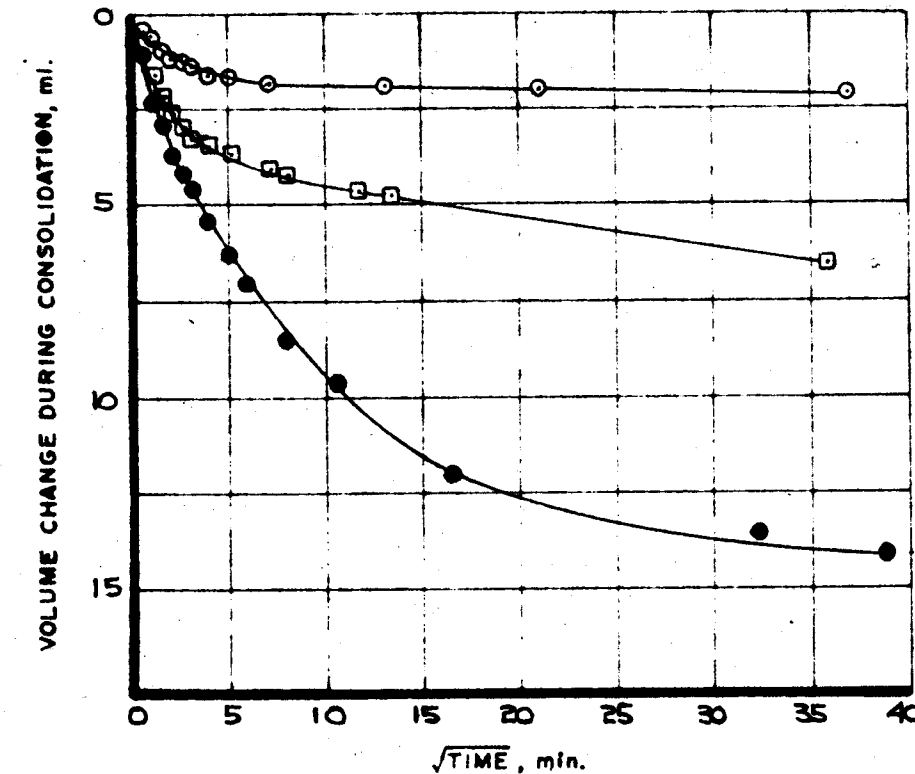
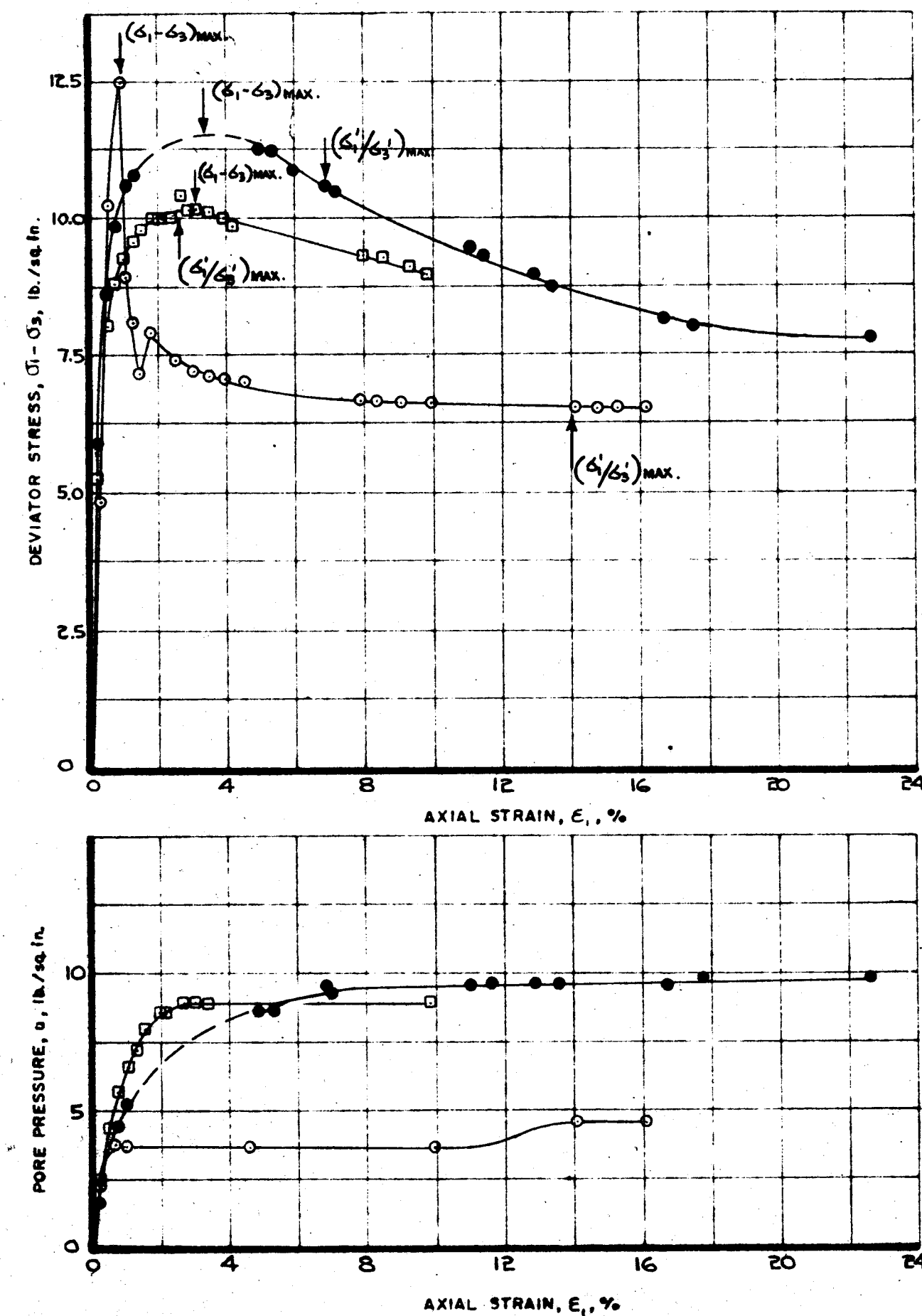
NATURAL WATER CONTENT, w, %	62	60	66	65
LIQUID LIMIT, wₗ, %	66	69	71	63
PLASTIC LIMIT, wₚ, %	19	21	20	22
UNIT WEIGHT, γₜ, lb./cu. ft.	103	102	103	102

REMARKS
SAMPLES a, b, & c ISOTROPICALLY CONSOLIDATED AT CELL PRESSURE ≈ p₀ + 10 LB/SQ. IN.
SAMPLE d ANISOTROPICALLY CONSOLIDATED

Date APRIL 25, 1974

Golder Associates

Drawn J.A.
Chkd J.H.R.
Appd J.H.R.



R TESTS CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENT

FIGURE 40

DESICCATED CRUST

	a	b	c	d	FAILURE SKETCH
BOREHOLE NUMBER	501	501	501		
SAMPLE NUMBER	1	1	1		
SAMPLE DEPTH, ft	6.2	6.2	6.2		

SPECIMEN DIAMETER, in.	2.0	2.0	2.0
SPECIMEN HEIGHT, in.	4.0	4.0	3.99

WATER CONTENT, BEFORE CONSOLIDATION, %	62	63	64
CELL PRESSURE, σ_3 , lb./sq. in.	14	19	25
BACK PRESSURE, lb./sq. in.	9	9	12
PORE PRESSURE PARAMETER 'B'	.99	.95	.91
CONSOLIDATION PRESSURE, σ'_c , lb./sq. in.	5	10	13
VOLUME CHANGE DURING CONSOLIDATION, Δv , ml.	2.0	7.9	16.6
WATER CONTENT, AFTER CONSOLIDATION, %	61	59	56
AVERAGE RATE OF STRAIN, % / hr.	25	25	25
AVERAGE LOAD INCREMENT, lb./sq. in.	—	—	—
AVERAGE LOAD DURATION, hr.	—	—	—
TIME TO FAILURE, hr.	4	12	12
WATER CONTENT, AFTER TEST, %	60	59	56

MAX. DEVIATOR STRESS $(\sigma_1 - \sigma_3)_{max}$, lb./sq. in.	12.6	10.1	11.6
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)_{max}$, %	0.9	3.1	3.4
MAX. EFFECTIVE PRINCIPAL STRESS RATIO $(\sigma'_1 / \sigma'_3)_{max}$, lb./sq. in.	14.2	9.67	3.94
AXIAL STRAIN AT $(\sigma'_1 / \sigma'_3)_{max}$, %	14.1	2.6	6.9
PORE PRESSURE PARAMETER, A_f	.285	.280	.675

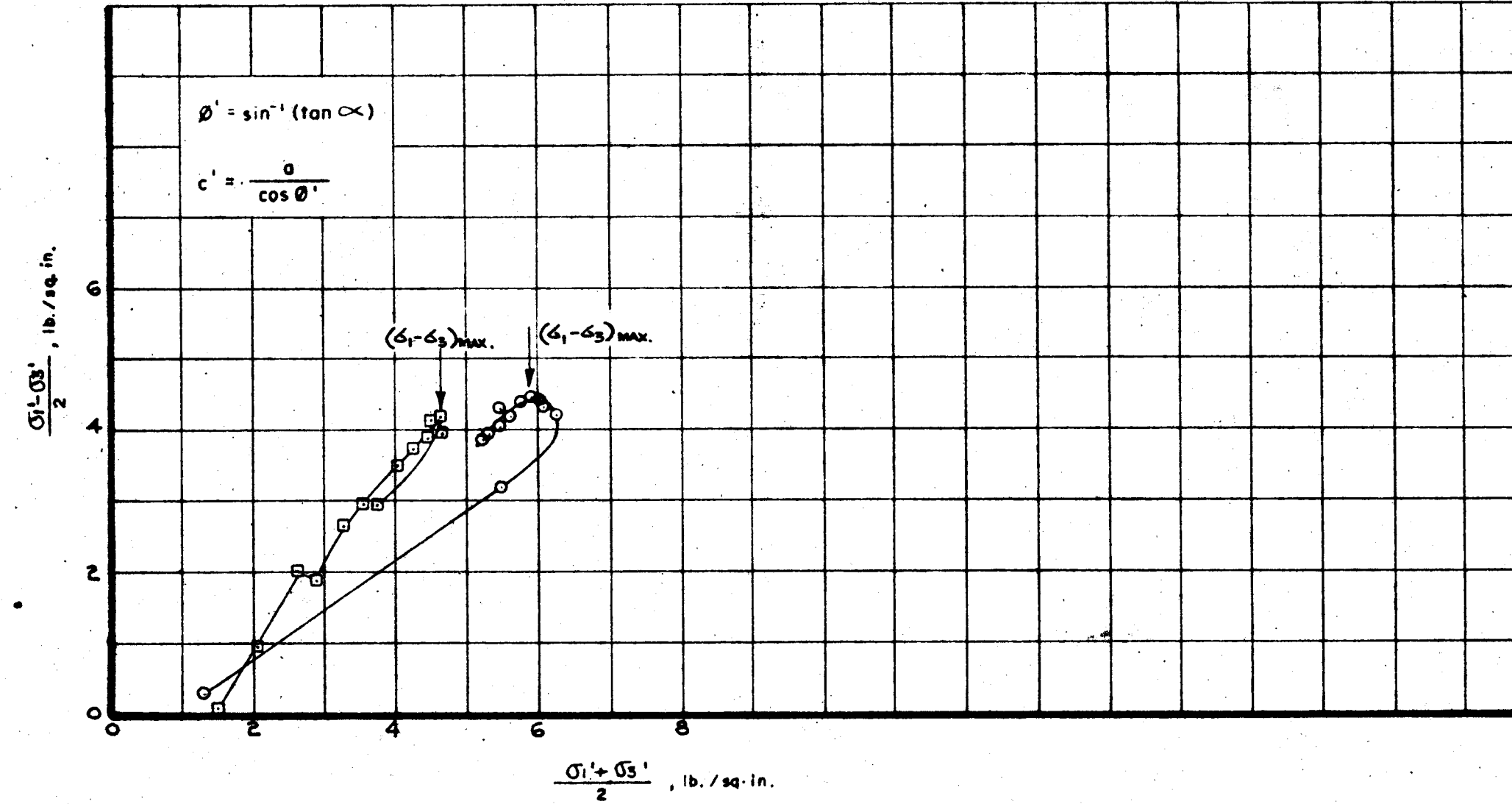
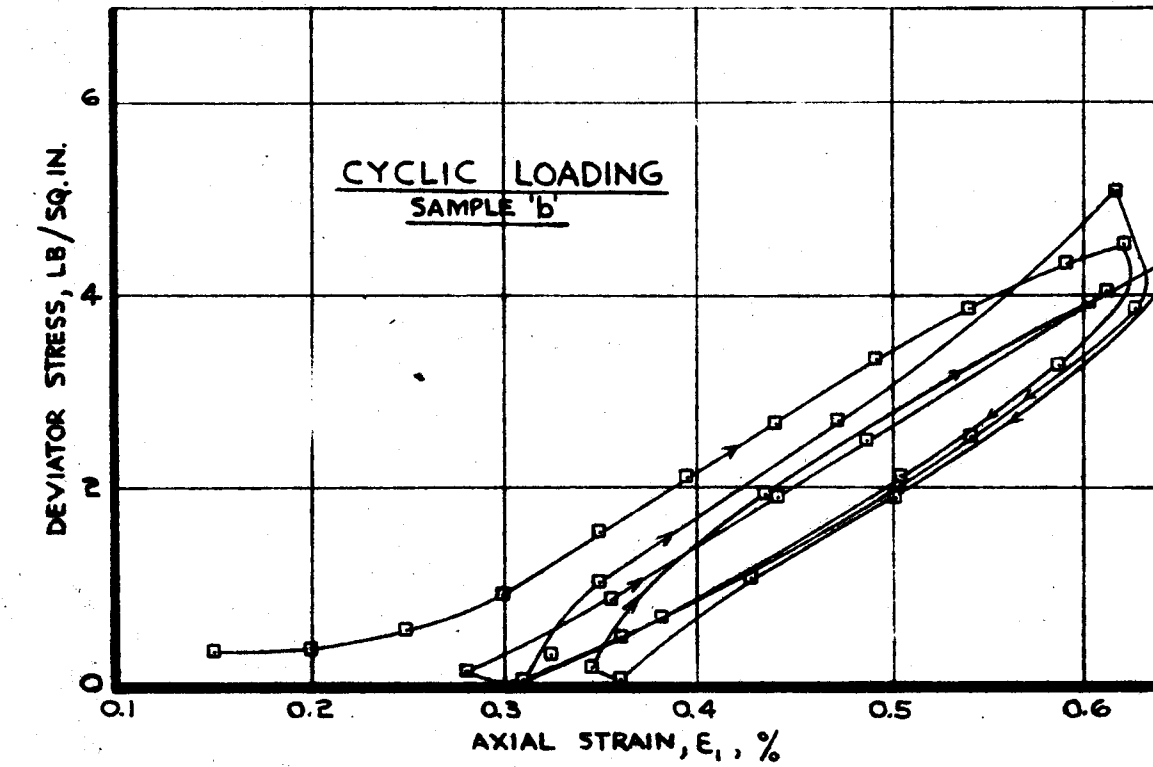
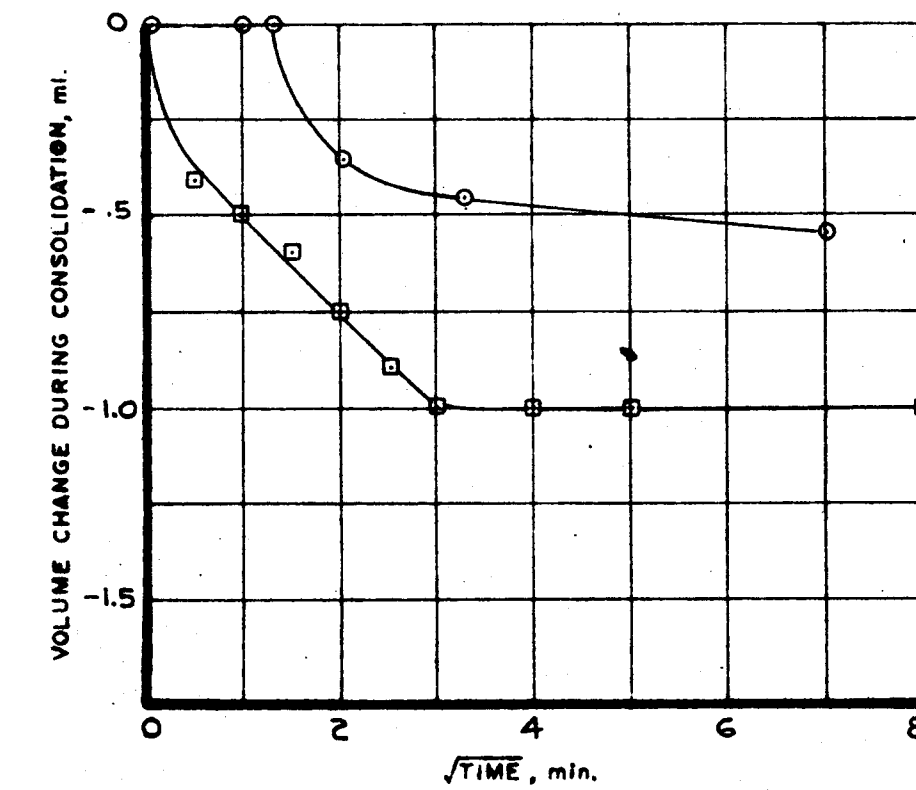
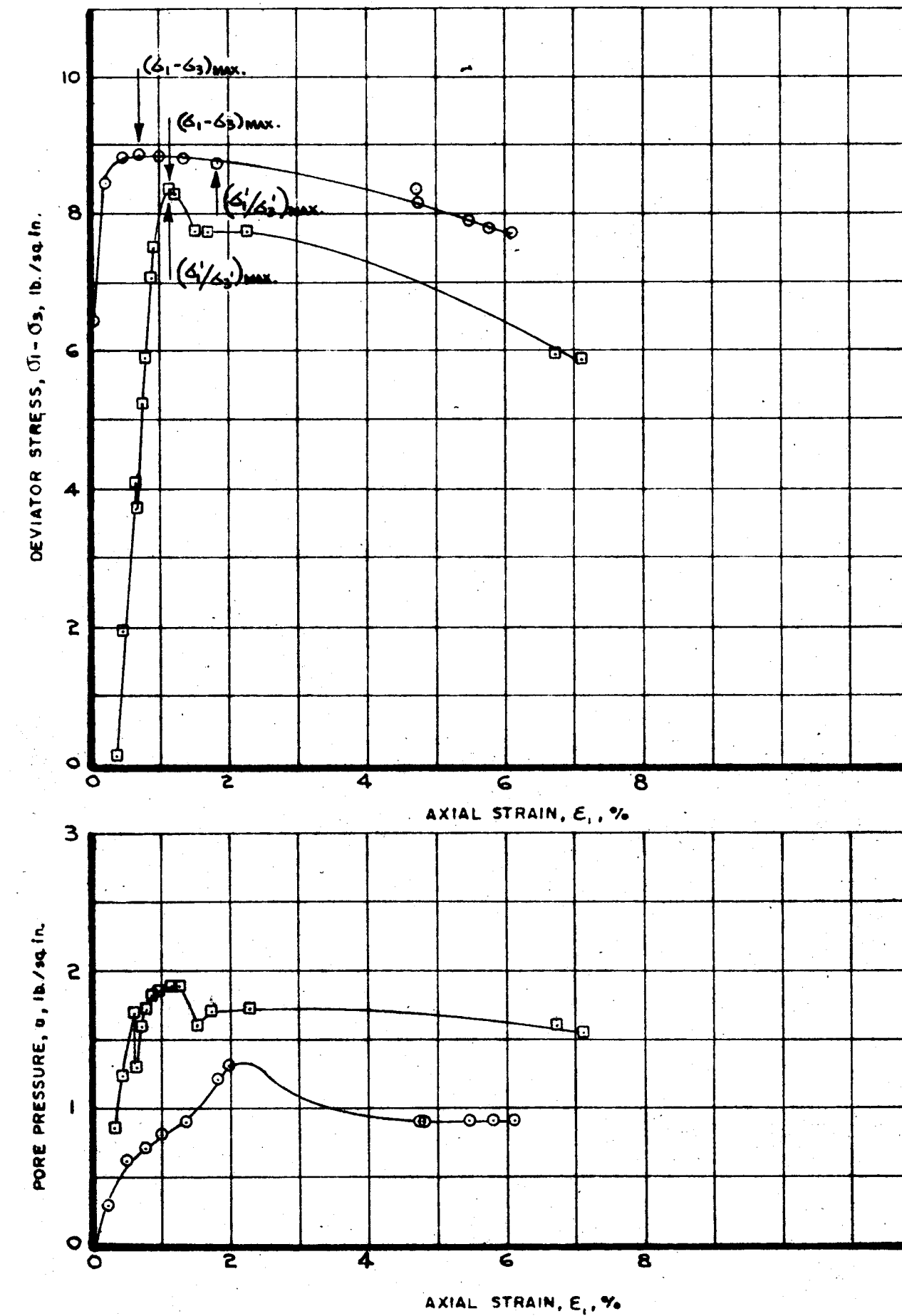
NATURAL WATER CONTENT, w , %	60	62	62
LIQUID LIMIT, w_L	69	65	65
PLASTIC LIMIT, w_p	22	16	19
UNIT WEIGHT, γ_t , lb./cu. ft.	104	103	103

REMARKS
SAMPLE a CONSOLIDATED AT $P'_0 + 2$ P.S.I.
SAMPLE b CONSOLIDATED AT $P'_0 + 7$ P.S.I.
SAMPLE c CONSOLIDATED AT $P'_0 + 10$ P.S.I.

Date APRIL 29, 1974

Golder Associates

Drawn J.A.
Chkd. HRC
Appd. [Signature]



R TESTS CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENT

FIGURE 41

LEDA CLAY

	a	b	c	d
BOREHOLE NUMBER	503	503		
SAMPLE NUMBER	2	2		
SAMPLE DEPTH, ft	9.4	9.4		

FAILURE SKETCH

SPECIMEN DIAMETER, in	2.0	2.0		
SPECIMEN HEIGHT, in	4.0	4.0		

WATER CONTENT, BEFORE CONSOLIDATION, %	70	80		
CELL PRESSURE, σ_3 , lb./sq. in.	2.3	2.3		
BACK PRESSURE, lb./sq. in.	0	0		
PORE PRESSURE PARAMETER B	.98	.78		
CONSOLIDATION PRESSURE, σ_c , lb./sq. in.	2.3	2.3		
VOLUME CHANGE DURING CONSOLIDATION, Δv_c , ml.	.6	1.0		
WATER CONTENT, AFTER CONSOLIDATION, %	69	80		
AVERAGE RATE OF STRAIN, % / hr.	.25	.25		
AVERAGE LOAD INCREMENT, lb./sq. in.	—	—		
AVERAGE LOAD DURATION, hr.	—	—		
TIME TO FAILURE, hr.	3	—		
WATER CONTENT, AFTER TEST, %	60	70		

MAX. DEVIATOR STRESS $(\sigma_1 - \sigma_3)_{max}$, lb./sq. in.	8.8	8.4		
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)_{max}$, %	.75	1.17		
MAX. EFFECTIVE PRINCIPAL STRESS RATIO $(\sigma_1'/\sigma_3')_{max}$, lb./sq. in.	9.73	20.0		
AXIAL STRAIN AT $(\sigma_1'/\sigma_3')_{max}$, %	1.95	1.17		
PORE PRESSURE PARAMETER, A _f	0.08	.22		

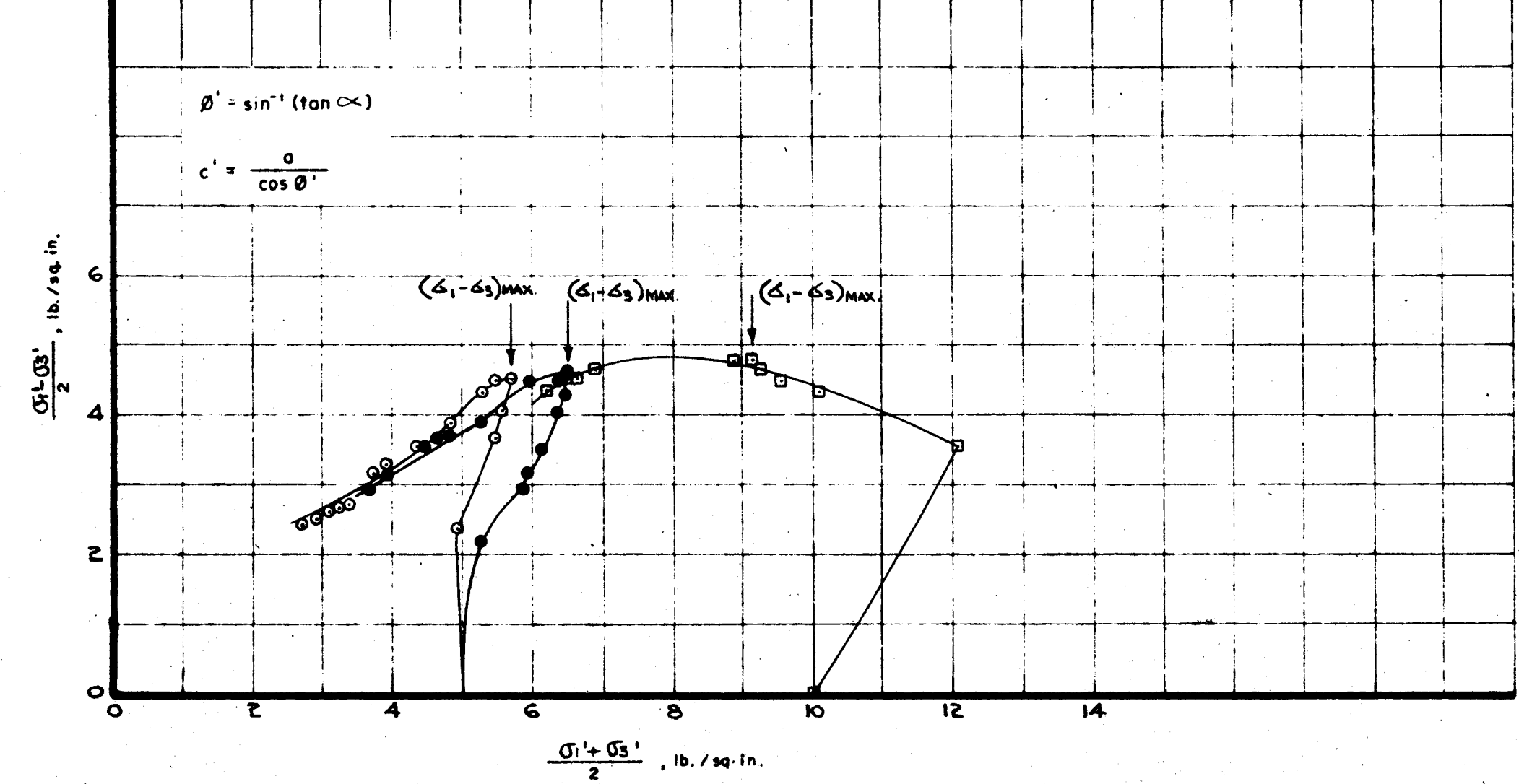
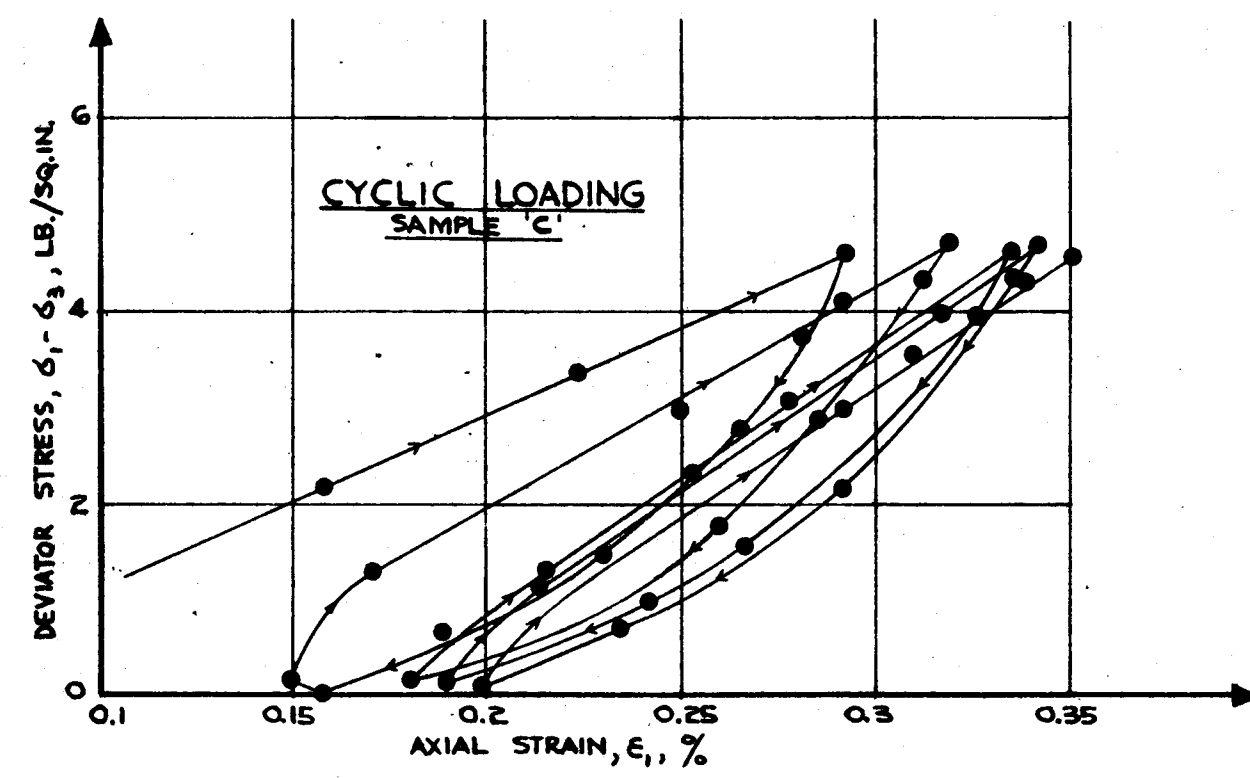
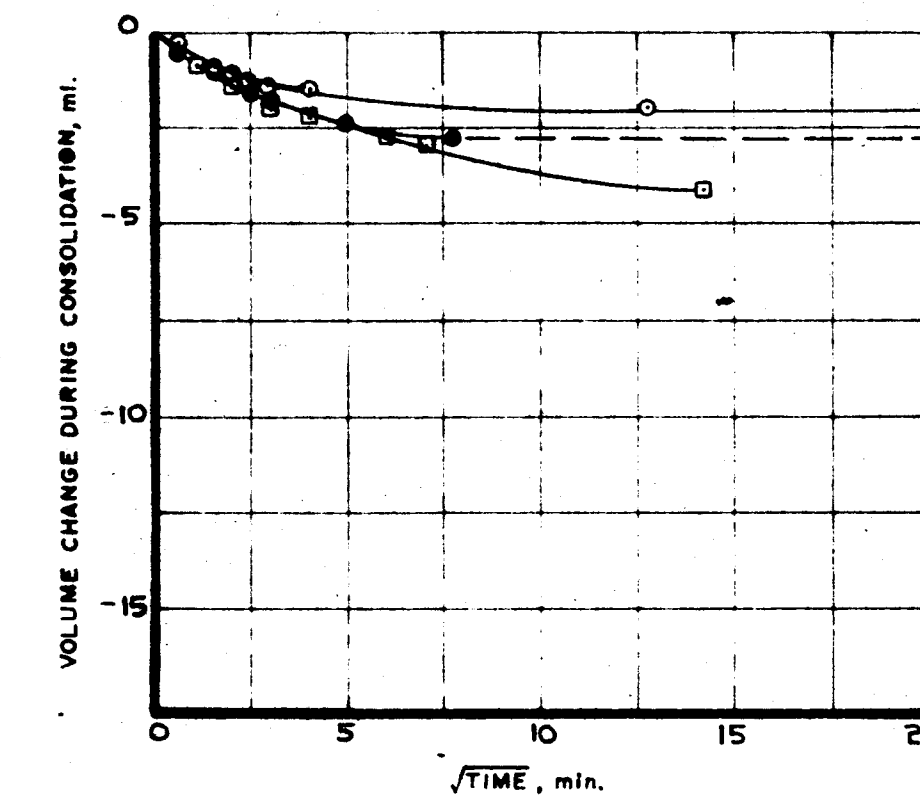
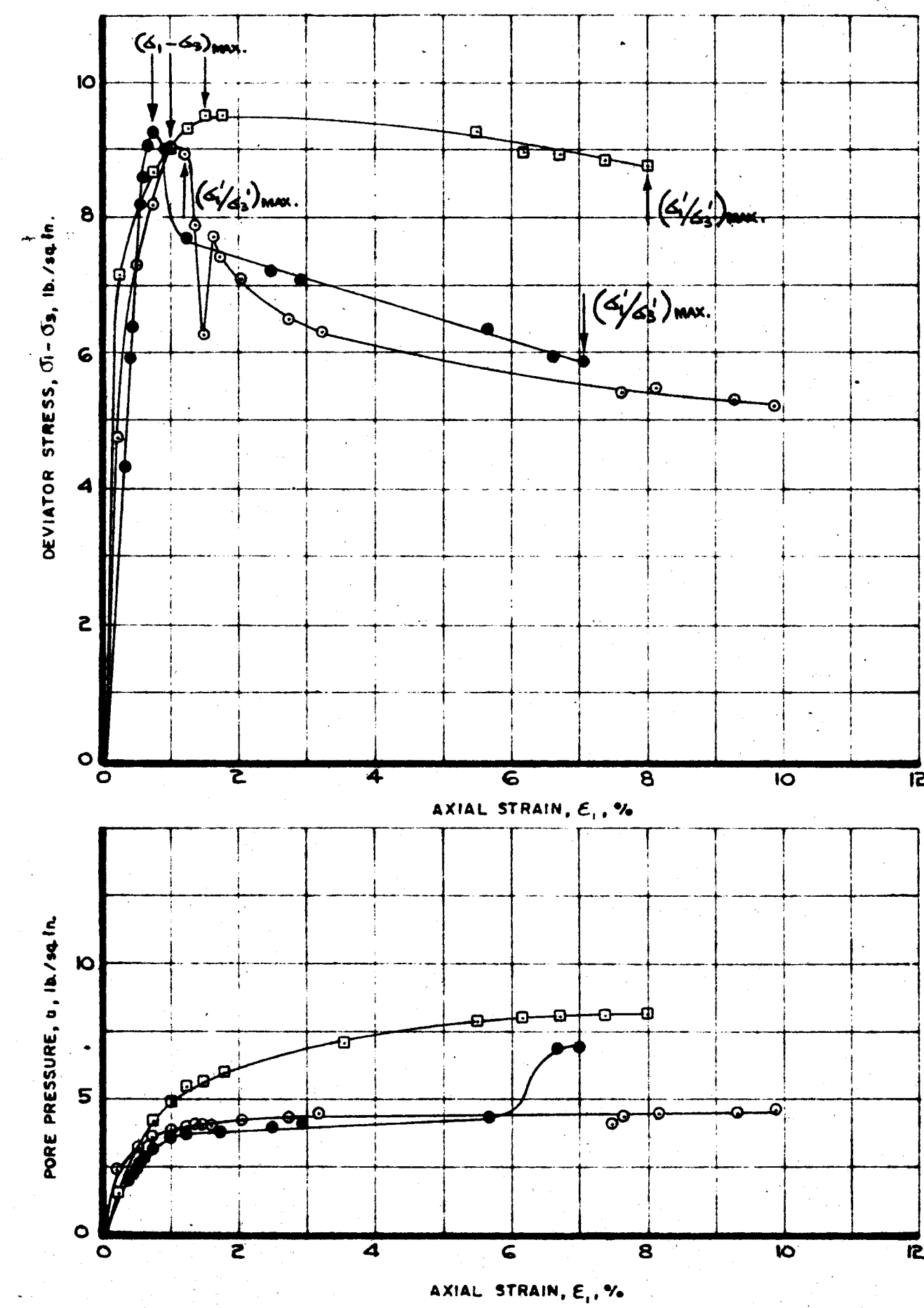
NATURAL WATER CONTENT, w , %	SEE FAILURE SKETCHES			
LIQUID LIMIT, w_L				
PLASTIC LIMIT, w_p				
UNIT WEIGHT, γ_t , lb./cu. ft.	105	107		

REMARKS
SAMPLE b ANISOTROPICALLY CONSOLIDATED AT $(\sigma_1/\sigma_3) \approx 0.26$

Date APRIL 30, 1974

Golder Associates

Drawn
Chkd
Appd



R TESTS
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS
WITH PORE PRESSURE MEASUREMENT

LEDA CLAY

FIGURE 42

	a	b	c	d	FAILURE SKETCH
BOREHOLE NUMBER	503	503	503		
SAMPLE NUMBER	3	3	3		
SAMPLE DEPTH, ft	15.0	15.0	15.0		
SPECIMEN DIAMETER, in.	2.0	2.0	2.0		
SPECIMEN HEIGHT, in.	4.0	4.0	4.0		

TEST CONDITIONS	a	b	c
WATER CONTENT, BEFORE CONSOLIDATION, %	59	59	64
CELL PRESSURE, σ_3 , lb./sq. in.	20	28	5
BACK PRESSURE, lb./sq. in.	15	18	0
PORE PRESSURE PARAMETER 'B'	.96	.97	.95
CONSOLIDATION PRESSURE, σ'_c , lb./sq. in.	5	10	5
VOLUME CHANGE DURING CONSOLIDATION, Δv_c , ml.	-1.4	-11.9	-2.7
WATER CONTENT, AFTER CONSOLIDATION, %	58	49	62
AVERAGE RATE OF STRAIN, %/hr.	.25	.25	.25
AVERAGE LOAD INCREMENT, lb./sq. in.	—	—	—
AVERAGE LOAD DURATION, hr.	—	—	—
TIME TO FAILURE, hr.	4	6	3
WATER CONTENT, AFTER TEST, %	53	54	62

TEST RESULTS	a	b	c
MAX. DEVIATOR STRESS $(\sigma_1 - \sigma_3)_{max}$, lb./sq. in.	9.0	9.6	9.2
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)_{max}$, %	1.0	1.5	.76
MAX. EFFECTIVE PRINCIPAL STRESS RATIO $(\sigma'_1/\sigma'_3)_{max}$, lb./sq. in.	9.96	5.87	9.8
AXIAL STRAIN AT $(\sigma'_1/\sigma'_3)_{max}$, %	1.25	7.99	7.0
PORE PRESSURE PARAMETER, A_f	+4.20	+5.86	+3.41

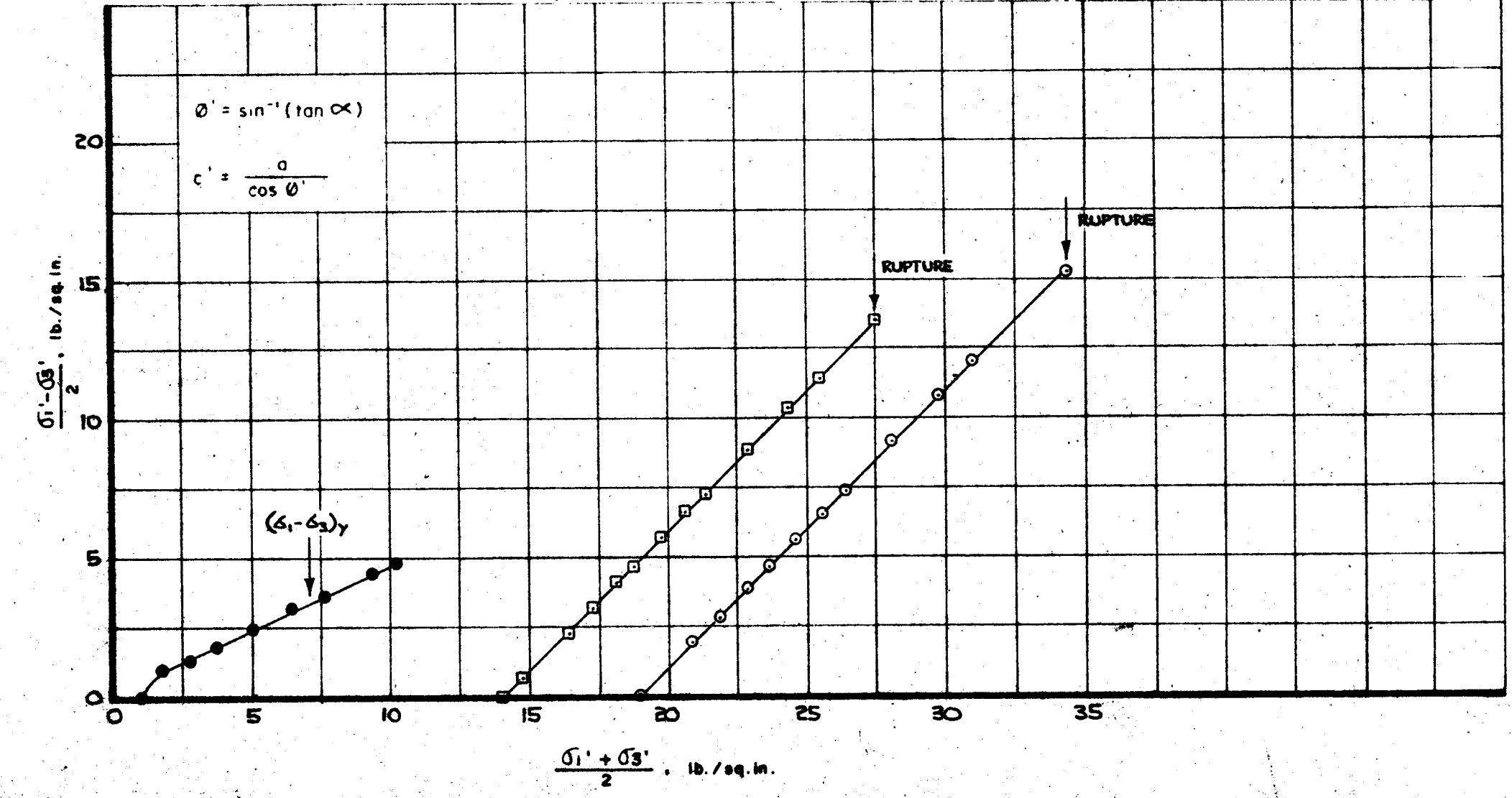
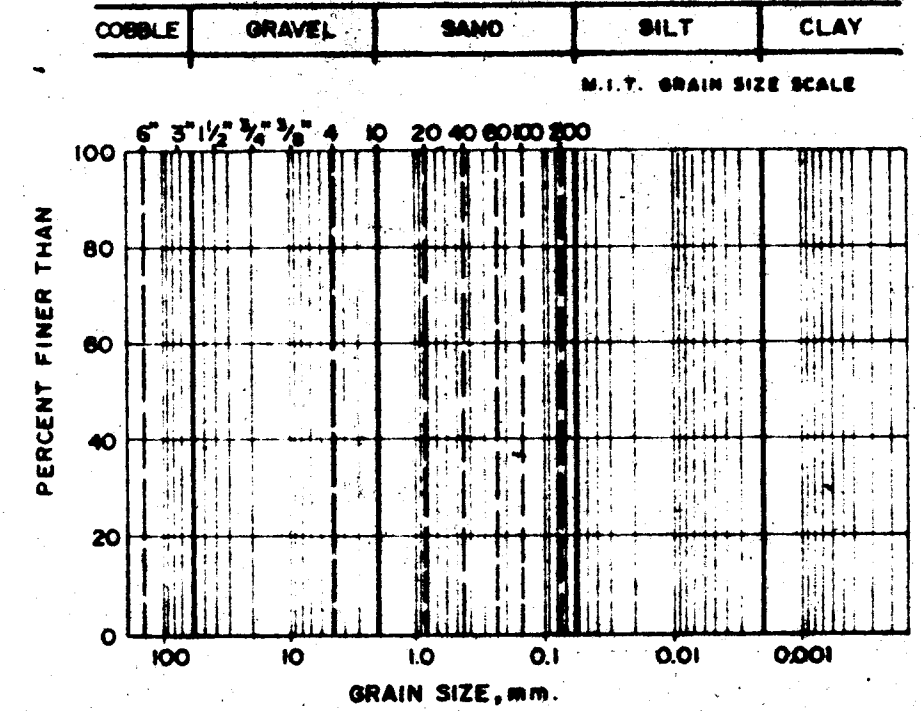
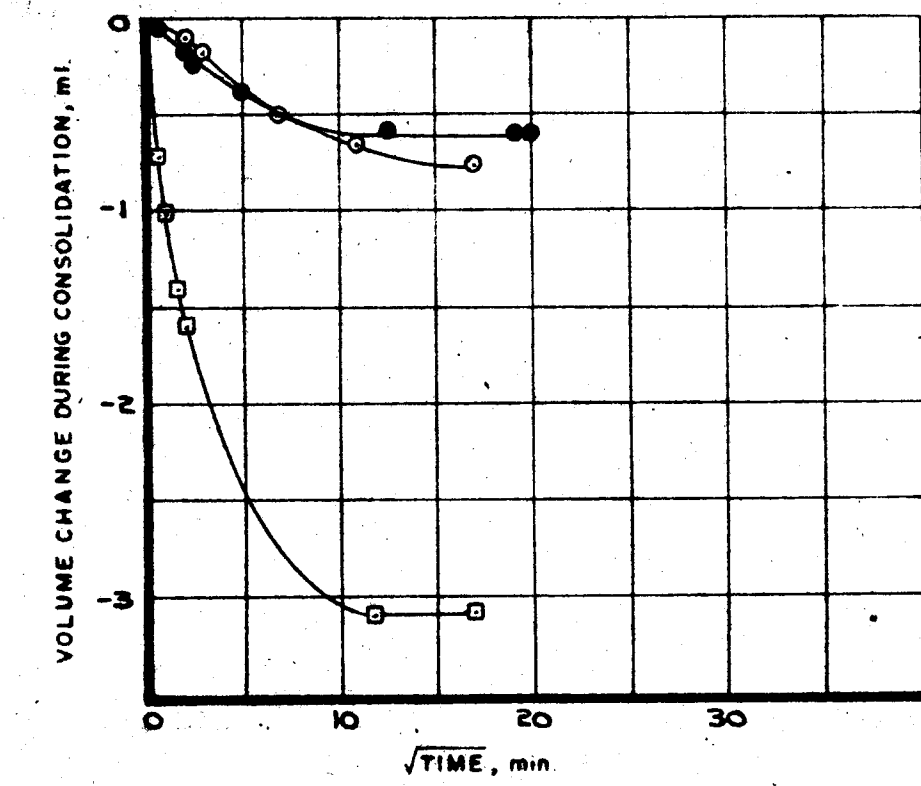
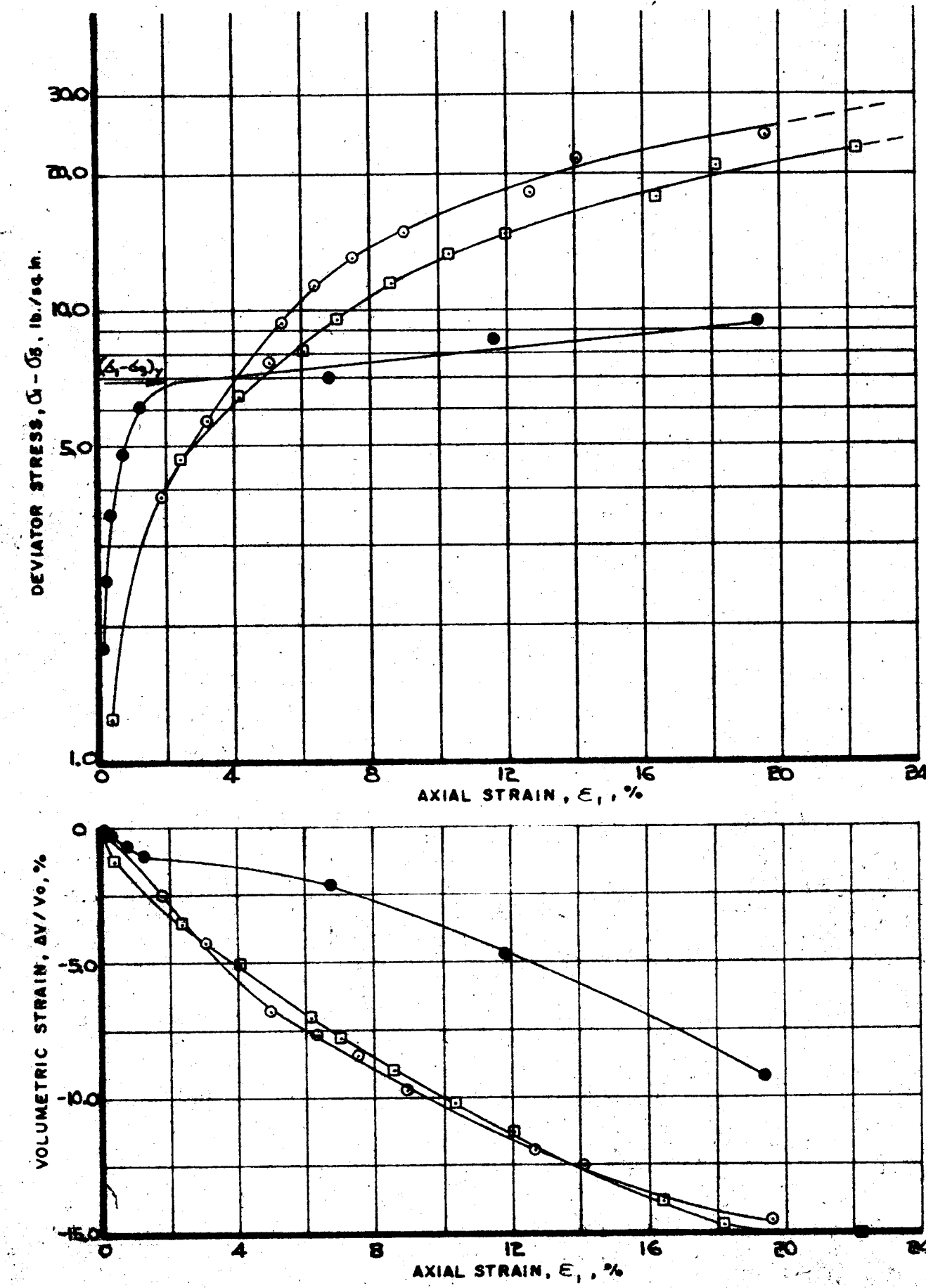
	a	b	c
NATURAL WATER CONTENT, w , %			
LIQUID LIMIT, w_L			
PLASTIC LIMIT, w_p			
UNIT WEIGHT, γ_f , lb./cu. ft.	103	105	103

REMARKS

1) SAMPLE 'a' TESTED AT EFFECTIVE OVERBURDEN PRESSURE.

2) SAMPLE 'b' TESTED AT EFFECTIVE OVERBURDEN PRESSURE + 8 P.S.I.

3) SAMPLE 'c' - CYCLIC LOADING TEST AT EFFECTIVE OVERBURDEN PRESSURE.



S TESTS CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TESTS LEDA CLAY

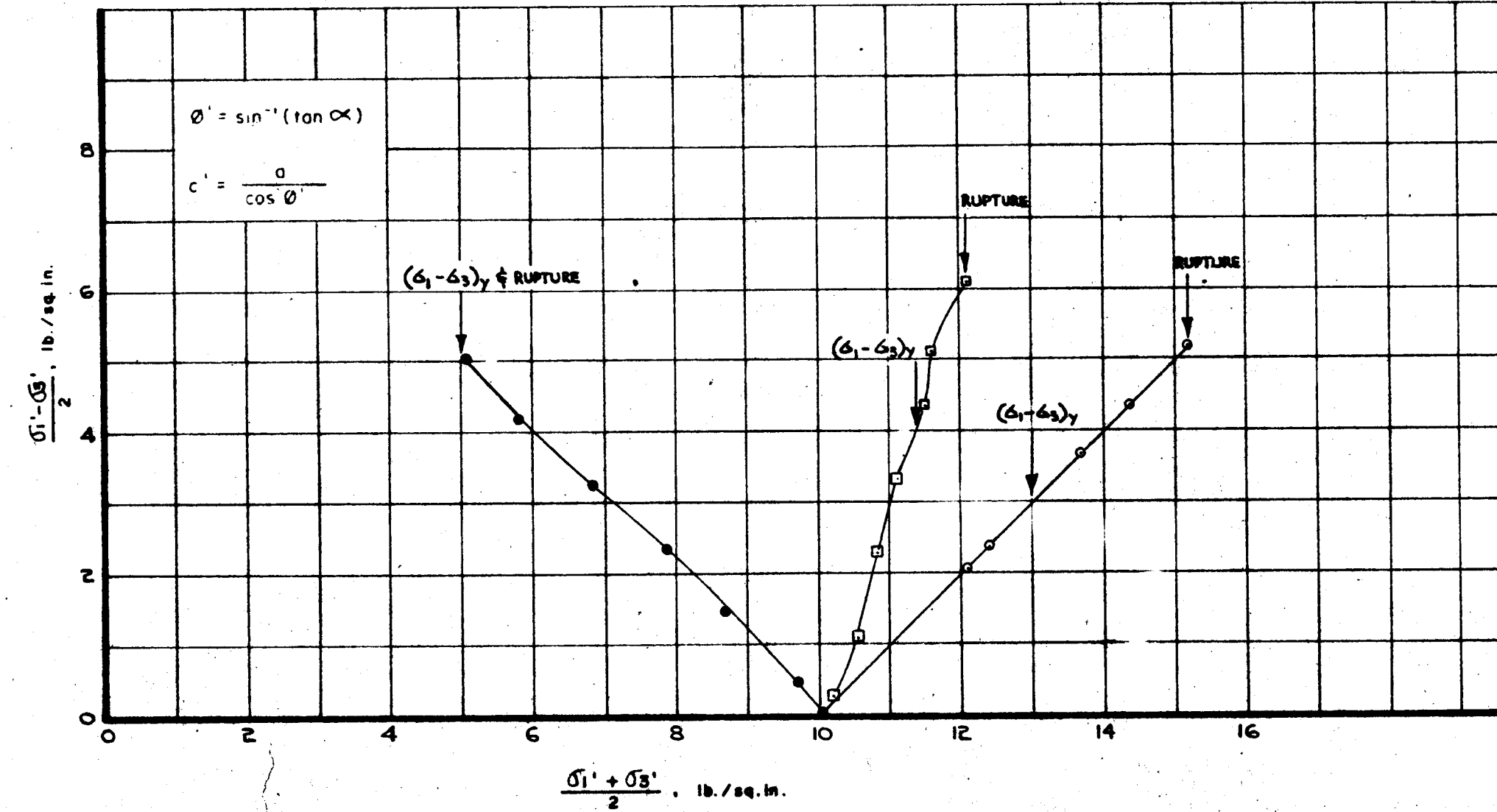
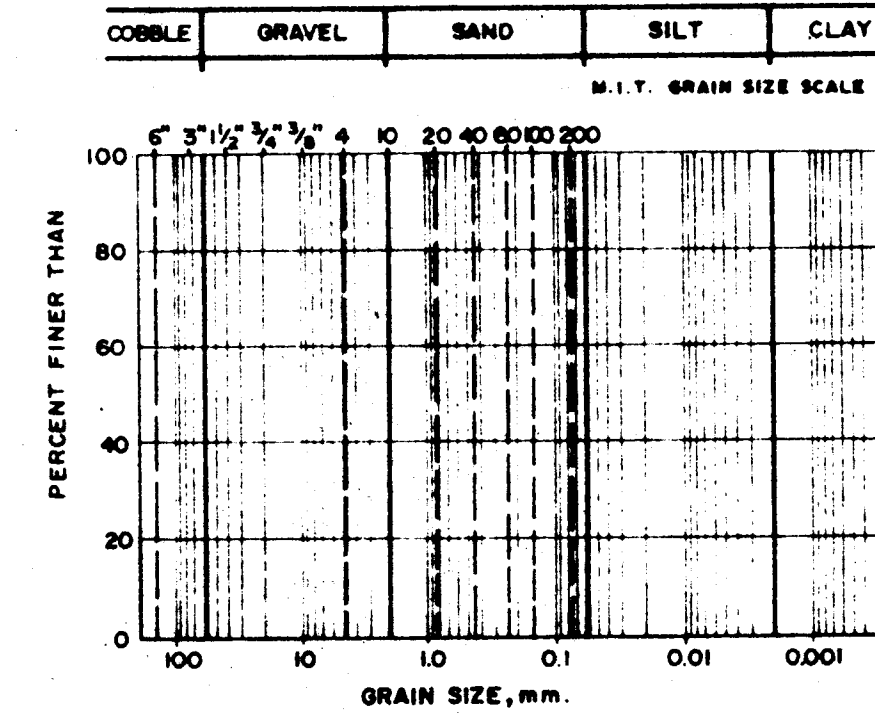
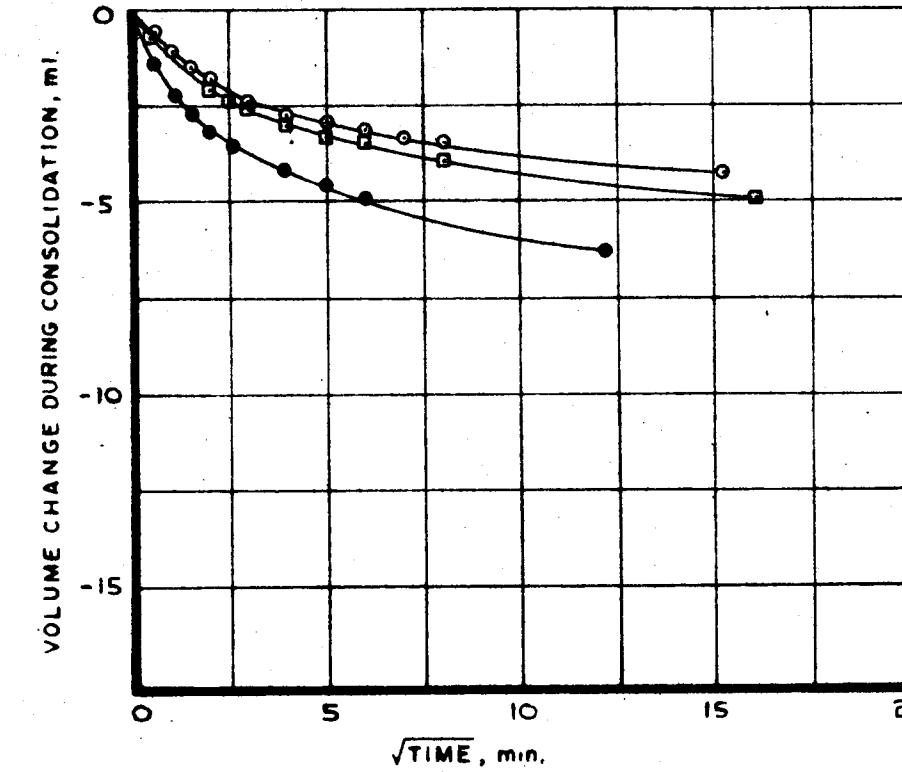
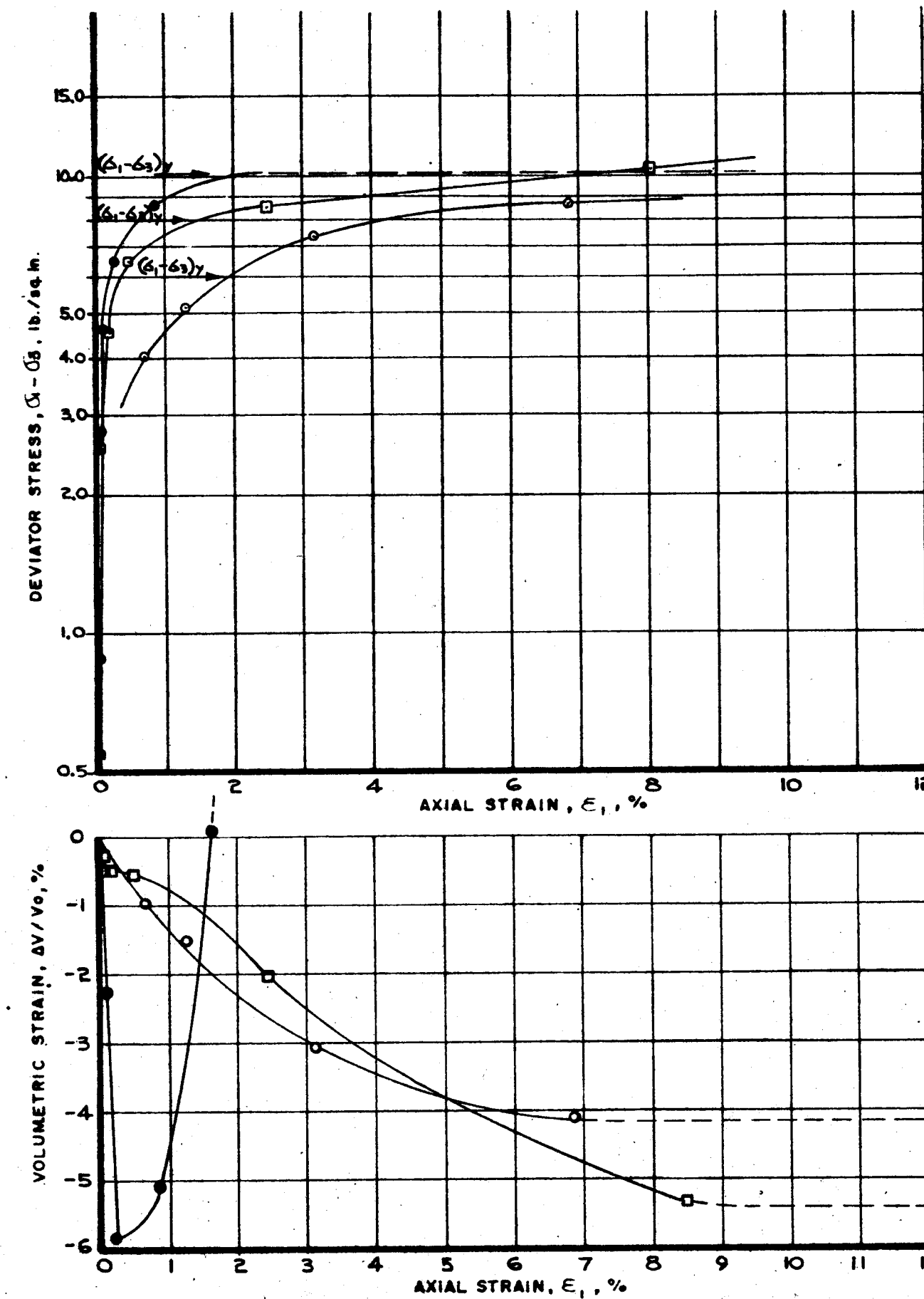
	a	b	c	d	FAILURE SKETCH
BOREHOLE NUMBER	503	503	503		
SAMPLE NUMBER	2	2	2		
SAMPLE DEPTH, ft.	9.4	9.4	9.4		
SPECIMEN DIAMETER, in.	2.0	2.0	2.0		
SPECIMEN HEIGHT, in.	4.0	4.0	4.0		

WATER CONTENT, BEFORE CONSOLIDATION, %	72	—	67	
CELL PRESSURE, σ_3 , lb./sq. in.	19	14	1.5	
BACK PRESSURE, lb./sq. in.	0	0	0	
PORE PRESSURE PARAMETER B	.55	.83	—	
CONSOLIDATION PRESSURE, σ_c , lb./sq. in.	19	14	1.5	
VOLUME CHANGE DURING CONSOLIDATION, ΔV_c , ml.	37.4	20.6	—	
WATER CONTENT, AFTER CONSOLIDATION, %	52	—	67	
AVERAGE RATE OF STRAIN, % / hr.	—	—	—	
AVERAGE LOAD INCREMENT, lb./sq. in.	2	1.5	1.3	
AVERAGE LOAD DURATION, hr.	24	24	24	
TIME TO FAILURE, days	10	11	6	
WATER CONTENT, AFTER TEST, %	43	—	53	

MAX DEVIATOR STRESS ($\sigma_1 - \sigma_3$) max, lb./sq. in.	30.8	27.0	9.3	
AXIAL STRAIN AT ($\sigma_1 - \sigma_3$) max, %	19.6	22.6	19.4	
MAX EFFECTIVE PRINCIPAL STRESS RATIO (σ_1' / σ_3') max, lb./sq. in.	—	—	—	
AXIAL STRAIN AT (σ_1' / σ_3') max, %	—	—	—	
DEVIATOR STRESS AT YIELD, ($\sigma_1 - \sigma_3$) _y , lb./sq. in.	—	—	6.9	
AXIAL STRAIN AT ($\sigma_1 - \sigma_3$) _y , %	—	—	2.0	

NATURAL WATER CONTENT, w, %	(72)			
LIQUID LIMIT, w _L	SEE FAILURE SKETCHES			
PLASTIC LIMIT, w _P	—			
UNIT WEIGHT, γ_t , lb./cu. ft.	100	103	102	

REMARKS
a - CONSOLIDATED AT $P_0' + 15$ P.S.I.
b - CONSOLIDATED AT $P_0' + 10$ P.S.I.
c - ANISOTROPIC CONSOLIDATION



S TESTS CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TESTS LEDA CLAY

FIGURE 44

	a	b	c	d	FAILURE SKETCH
BOREHOLE NUMBER	503	503	503		RED
SAMPLE NUMBER	3	3	3		RED
SAMPLE DEPTH, ft.	15.0	15.0	15.0		RED

SPECIMEN DIAMETER, in.	2.0	2.0	2.0
SPECIMEN HEIGHT, in.	4.0	4.0	4.0

WATER CONTENT, BEFORE CONSOLIDATION, %	60	55	62
CELL PRESSURE, σ_3 , lb./sq. in.	23	14	10
BACK PRESSURE, lb./sq. in.	13	4	0
PORE PRESSURE PARAMETER 'B'	.96	.99	.94
CONSOLIDATION PRESSURE, $\bar{\sigma}_c$, lb./sq. in.	10	10	10
VOLUME CHANGE DURING CONSOLIDATION, ΔV_c , ml.	-9.4	-9.6	-12.3
WATER CONTENT, AFTER CONSOLIDATION, %	57	51	51
AVERAGE RATE OF STRAIN, % / hr.	—	—	—
AVERAGE LOAD INCREMENT, lb./sq. in.	2	2	2
AVERAGE LOAD DURATION, hr.	24	24	24
TIME TO FAILURE, days	5	7	6
WATER CONTENT, AFTER TEST, %	52	47	59

MAX DEVIATOR STRESS $(\sigma_1 - \sigma_3)_{max}$, lb./sq. in.	10.3	12.5	10.1
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)_{max}$, %	—	—	—
MAX EFFECTIVE PRINCIPAL STRESS RATIO $(\sigma_1' / \sigma_3')_{max}$, lb./sq. in.	—	—	—
AXIAL STRAIN AT $(\sigma_1' / \sigma_3')_{max}$, %	—	—	—
DEVIATOR STRESS AT YIELD, $(\sigma_1 - \sigma_3)_y$, lb./sq. in.	6.0	8.0	10.1
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)_y$, %	2.0	1.5	2.0

NATURAL WATER CONTENT, w , %	60	55	62
LIQUID LIMIT, w_L	SEE FAILURE SKETCH 9		
PLASTIC LIMIT, w_P	SEE FAILURE SKETCH 9		
UNIT WEIGHT, γ_t , lb./cu. ft.	104	105	104

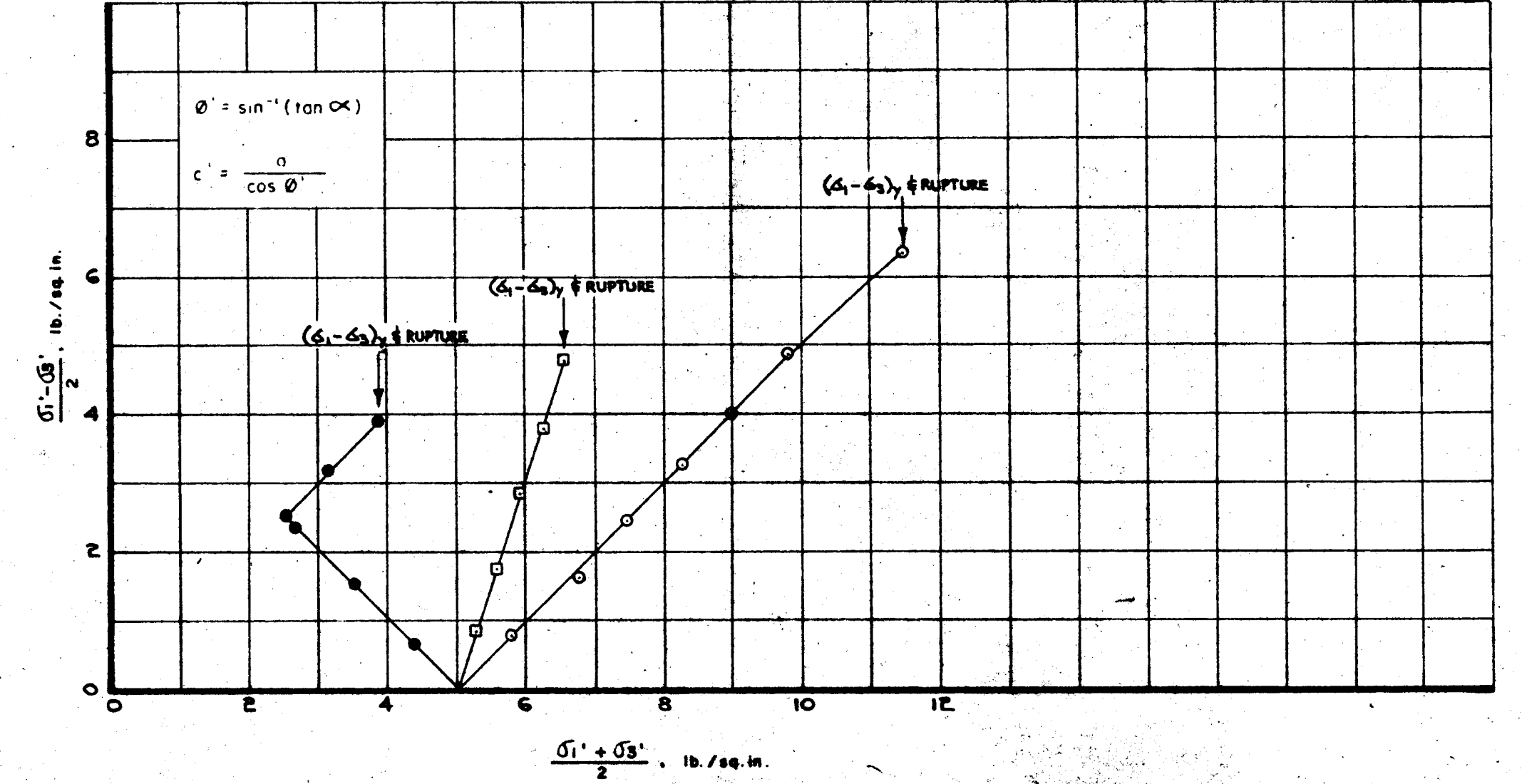
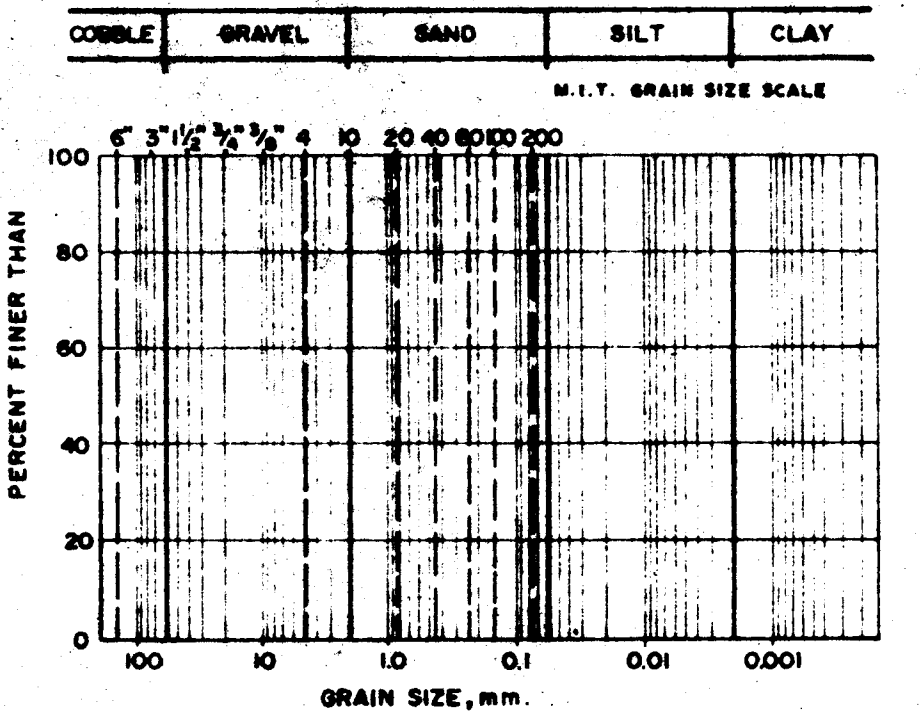
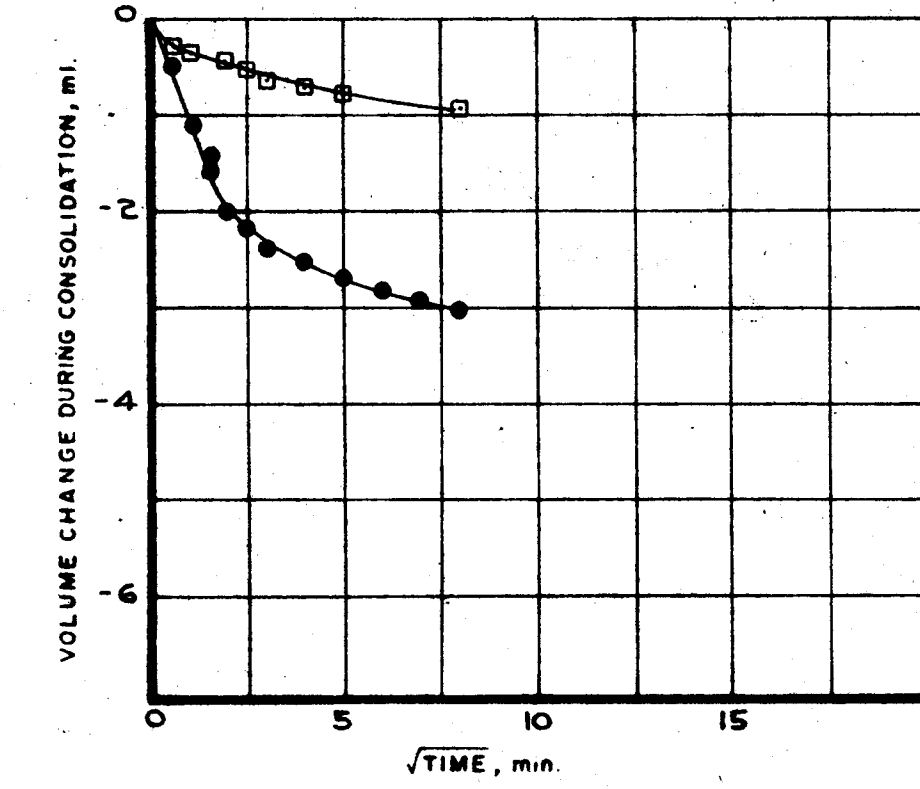
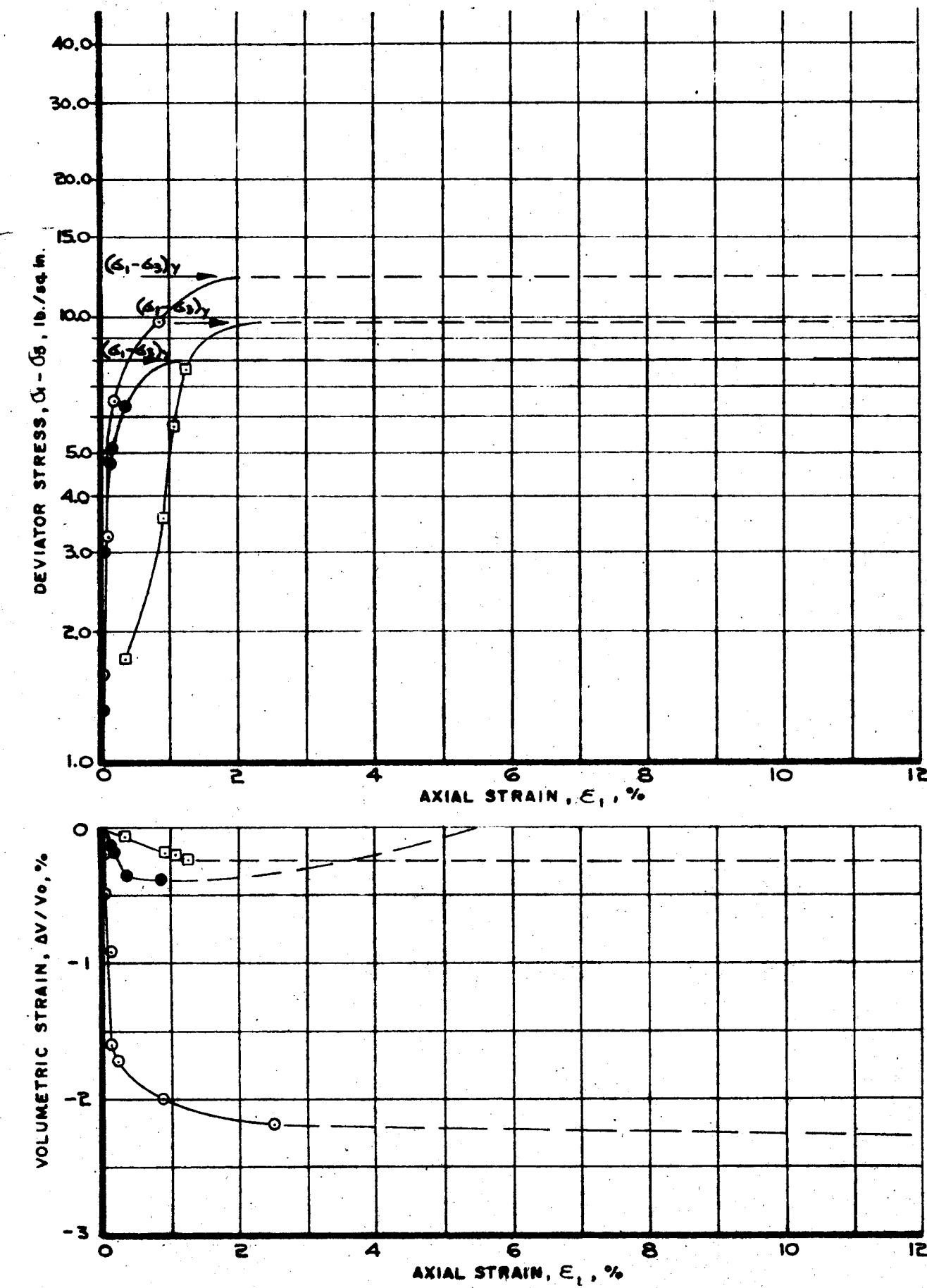
REMARKS

ALL SAMPLES CONSOLIDATED AT EFFECTIVE OVERBURDEN PRESSURE + 5 P.S.I.

Date: APRIL 24, 1974

Golder Associates

Drawn
Chkd
Appd



S TESTS CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TESTS LEDA CLAY

FIGURE 45

	q	b	c	d	FAILURE SKETCH
BOREHOLE NUMBER	503	503	503		GREY
SAMPLE NUMBER	3	3	3		RED
SAMPLE DEPTH, ft.	15.0	15.0	15.0		GREY

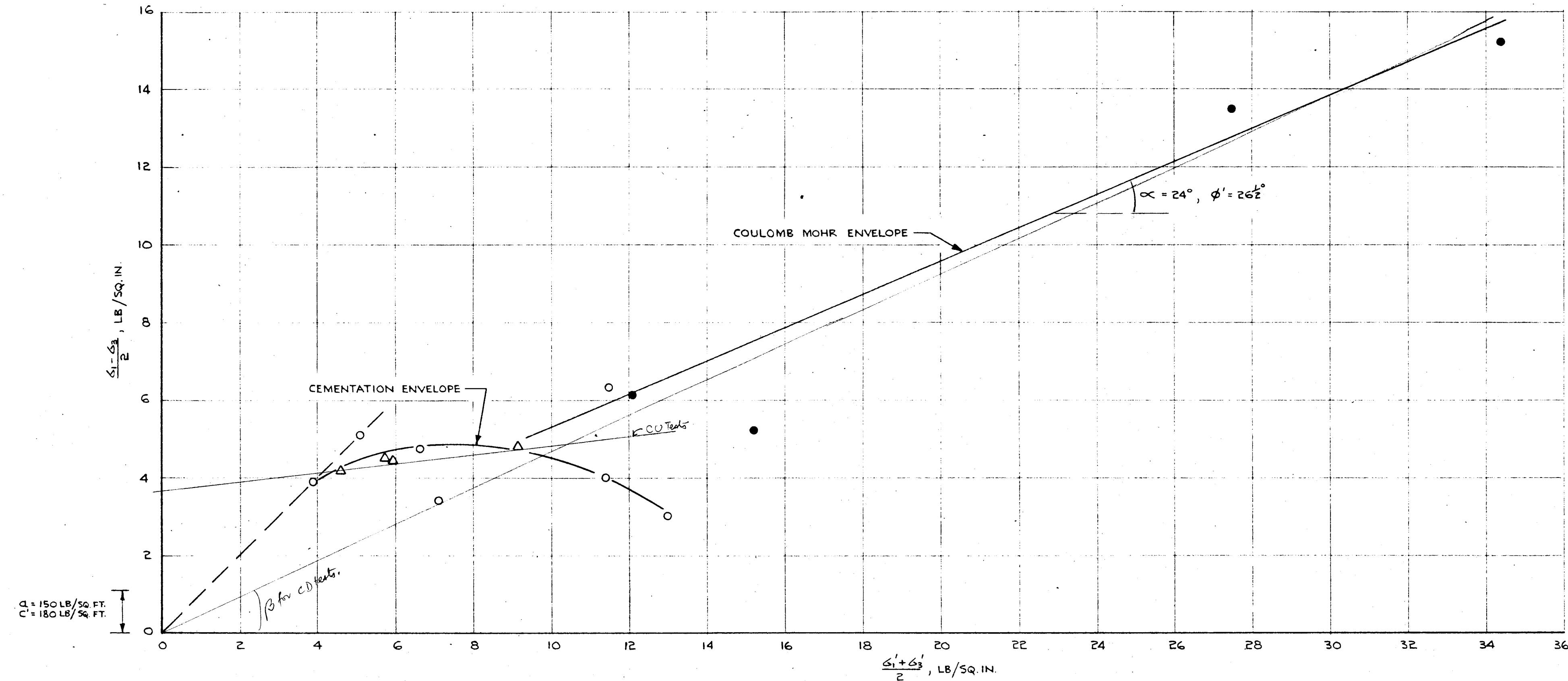
SPECIMEN DIAMETER, in.	2.0	2.0	2.0
SPECIMEN HEIGHT, in.	4.0	4.0	4.0

WATER CONTENT, BEFORE CONSOLIDATION, %	56	64	57
CELL PRESSURE, σ_3 , lb./sq. in.	5	5	13
BACK PRESSURE, lb./sq. in.	0	0	8
PORE PRESSURE PARAMETER 'B'	.69	.50	.99
CONSOLIDATION PRESSURE, σ_c , lb./sq. in.	5	5	5
VOLUME CHANGE DURING CONSOLIDATION, $\Delta V/V_0$, %	-7	-1.0	-4.8
WATER CONTENT, AFTER CONSOLIDATION, %	53	63	55
AVERAGE RATE OF STRAIN, %/hr.	—	—	—
AVERAGE LOAD INCREMENT, lb./sq. in.	1.5	2.0	2.0
AVERAGE LOAD DURATION, hr.	24	24	24
TIME TO FAILURE, days	6	5	6
WATER CONTENT, AFTER TEST, %	53	63	56

MAX DEVIATOR STRESS $(\sigma_1 - \sigma_3)_{max}$, lb./sq. in.	13.0	9.6	7.8
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)_{max}$, %	—	—	—
MAX. EFFECTIVE PRINCIPAL STRESS RATIO $(\sigma_1' / \sigma_3')_{max}$, lb./sq. in.	—	—	—
AXIAL STRAIN AT $(\sigma_1' / \sigma_3')_{max}$, %	—	—	—
DEVIATOR STRESS AT YIELD, $(\sigma_1 - \sigma_3)_y$, lb./sq. in.	13.0	9.6	8.0
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)_y$, %	2.0	2.0	1.1

NATURAL WATER CONTENT, w , %	56	63	57
LIQUID LIMIT, w_p	SEE FAILURE SKETCH		
PLASTIC LIMIT, w_p	SEE FAILURE SKETCH		
UNIT WEIGHT, γ_t , lb./cu. ft.	107	103	106

REMARKS
ALL SAMPLES TESTED AT EFFECTIVE OVERBURDEN PRESSURE.



Date MAY 10, 1974

Golder Associates

Drawn J.A.
Chkd. J.H.R.
Appd. J.S.