THE EFFECTS OF SAMPLING A NATURAL SILTY CLAY

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SCOPE AND CONTENTS:

The effects of sampling on the strength and preconsolidation load of a normally consolidated natural silty clay have been studied by means of laboratory simulations of "perfect" samples, "ground" samples, "tube" samples and "bottom of failed borehole" samples.

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CHAPTER 1

SAMPLING EFFECTS

1.1 INTRODUCTION

The process of obtaining soil samples from the field causes deformation of the obtained specimens. Changes in the engineering properties of the soil are to be expected as a result of this deformation. The degree of change varies with the type of soil and is a function of the mechanical properties of the natural material being sampled.(Hvorslev,1949)

The clay type of soils are the most problematic in considerations of the stability of engineering earthworks.

The behaviour of clay soils as influenced by their structure was first dealt with by Casagrande [1932]. The structure of the clay soils, as reflected by the sensitivity, has been found to be an important consideration. The idea of "sensitivity" of soils was introduced and defined* by Terzaghi [1944].

Relevant properties of clay soils which can be affected by sampling deformations are shear strength, preconsolidation load, permeability, compressibility, sensitivity, etc. The shear strength and preconsolidation

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* Soil Sensitivity $= \frac{\text{Unconfined compressive strength undisturbed}}{\text{Unconfined compressive strength remolded}}$

load are the most significant properties for consideration of stability of earthworks in clay soils.

1.2 DEFORMATIONS DUE TO SAMPLING

1.2 (a) Introduction

The most commonly used method of obtaining soil samples, consists of drilling a borehole into the soil to the levels from which samples are required, and then driving metal tubes into the soil at the bottom, twisting, and withdrawing the tubes with the soil inside. Refinements in the sampling technique allow samples to be obtained which have had no change in water content (i.e. undrained).

The problems of subsurface exploration and sampling of soils were studied by Hvorslev [1949].

Hvorslev makes a distinction between "avoidable" and "unavoidable" types of disturbance to which samples may be subjected prior to testing.

The "avoidable" types of disturbance, which would induce large deformations in the samples, would include sampling from a borehole with bottom failure, dropping obtained samples and similar cases of mishandling.

The "unavoidable" types of disturbance are said to be deformations induced by sampling tubes and deformations caused by total stress changes.

Consideration is given especially to the "unavoidable" deformations and their effect on strength and preconsolidation load of clays.

Problems with borehole bottom failures are so often encountered (especially when compensation for hydraulic head differences is neglected) that some attention, as well, is given to the study of data on samples from these boreholes.

1.2 (b) Borehole Bottom Failure

Hvorslev [1949] interpreted the problem of borehole bottom failure, as consisting of a plastic flow of soil from the bottom of the hole. This plastic flow is caused by a large reduction in total stresses (e.g. by neglecting to load the borehole bottom with water or drilling mud) compared to the shear resistance of the material.

Hvorslev states that the bulb of soil affected, may have a "depth of approximately three times the diameter of the hole....It is probable that the actual disturbance may reach much greater depths, when a large amount of soil flows into the hole".

Strength

Studies have been made of the effect of large deformations on the shear strength of clay soils, such as those which may be induced by borehole bottom failures.

Ladd [1965] made a laboratory study of the problem of bottom failure of a circular excavation. He used extension test results to represent the strength of the material at the bottom of the borehole, at incipient failure.

Rutledge [1944] investigated the effects of remolding on the shear strength of natural clay samples. He reported radical changes in the shape of stress strain curves and in the pore pressure behaviour of the material, as well as a significant drop in strength, due to remolding (as would be expected by the definition of "sensitivity" for natural soils).

Preconsolidation Load

The problem of the effects of large deformations on preconsolidation load of clays, has also been studied by Rutledge [1944]. Large differences in the shape of void ratio-pressure curves, and consequently in preconsolidation load values, were reported.

Rutledge's conclusions on general effects of sample disturbance (e.g. remolding) on the void ratio-pressure diagram and preconsolidation load of clays, were derived from observations of laboratory oedometer consolidation tests. Some load losses occur in the latter type of consolidation tests, due to friction in consolidometer rings. Leonards and Girault [1961] found that the rate at which side friction develops in steel or brass rings, is dependent on rate of strain and rate of pore pressure dissipation, for a certain pressure increment. The ratio of side friction loss to vertical pressure values seemed to decrease with increasing pressure. These factors would probably affect, to a certain degree, the actual significance of some of the observations made by Rutledge, since the work of Leonards and Girault showed that some shifts and changes in shape of consolidation curves can be attributed to load losses in the rings.

Conclusions

Sampling from a failed borehole in sensitive soil, would probably yield samples consisting mainly of remolded material.

It has been suggested by S.W. Smotrych that the problem of sampling from a failed borehole, could be studied through a laboratory investigation of the strength and preconsolidation load of undisturbed and remolded samples of a natural sensitive clay.

Side friction in consolidometer rings and its related problems, are factors which are absent in triaxial K_o consolidation tests (see Chapter 2). Hence, a comparative study of results from these and oedometer tests, might be useful in reassessing Rutledge's observations and the significance of testing procedures for accurate preconsolidation load determination.

1.2 (c) Tube Sampling Deformations

The insertion of tubes into the soil, inevitably causes disturbances in the obtained samples.

The degree of disturbance depends on the manner in which the tube is forced into the soil and on the dimensions of the tube. The greatest disturbance is caused by driving the sampler into the soil by successive blows of a hammer.

The best results can be obtained if the sampler is pushed into the ground at a high and constant speed.

The above observations were made by Hvorslev [1949] who also found that for a given internal diameter sampling tube forced into the soil by the same process, the degree of disturbance depended on the area ratio,

$$A_r(\%) = 100 \frac{D_e^2 - D_i^2}{D_i}$$

in which D_e is the external diameter, and D_i the internal diameter of the tube.

With regard to the effects of different degrees of deformation on various sampled soils, Hvorslev commented that "some plastic soils can withstand a strain of several percent without an appreciable change in physical properties, whereas a strain of less than one percent may cause serious disturbance of brittle soils".

It is common practice to find the shear strength and preconsolidation load of a field deposit through testing of tube samples.

<u>Strength</u>

Ladd and Lambe [1963] developed a method by which the undrained strength of tube samples was corrected to an approximate field strength value (i.e. "perfect" sample strength: see 1.3 (b)).

The method was based on readings of pore water suction in the samples obtained, and on a correlation with overconsolidation ratios based on the shape of effective stress paths from undrained tests. Using this method the authors found that 20 to 50% lower strengths were obtained from tube samples of various materials.

Preconsolidation Load

Casagrande [1936], Terzaghi and Peck [1948] and Schmertmann [1953] have investigated the problem of determining the field preconsolidation load from data for tests on tube samples. Casagrande developed a graphical solution (see Chapter 3, Fig.17) to the problem, on the basis of a laboratory study of rebound and recompression curves, obtained from oedometer tests on undisturbed clay specimens. This author assumed that these drained laboratory sampling simulations were applicable to the field sampling problem, during which swelling of the samples presumably occurred. Swelling of samples with today's sampling methods may be avoided.

The Casagrande method is commonly applied indiscriminately for overconsolidated* and normally consolidated soil.

Terzaghi and Peck's method of field preconsolidation load estimation (see Chapter 3, Fig.17) for normally consolidated soil, makes use of the observation that field

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overconsolidated ≡ field preconsolidation load is greater than present effective (see p.37) overburden pressure

normally consolidated ≡ field preconsolidation load equal to present effective overburden pressure sampling does not alter the water content of samples. The value estimated however, is only a "minimum possible" preconsolidation load value since compensating for the effects of undrained deformations is not considered.

The method described by Schmertmann was designed also for a "minimum possible" preconsolidation load value. The method, as applied to normally consolidated soil, was essentially the same as Terzaghi and Peck's. Solutions were extended to overconsolidated soils, by introducing a "field" recompression line.

Conclusions

Tube sampling deformations cause significant changes in properties of clay soils, as illustrated by Ladd's results on strength.

It has been suggested by S.W. Smotrych that tube sampling may cause failure in extension of sensitive, brittle soils.

It is thought that the use of the Casagrande method for samples of normally consolidated soils, may lead to an overestimation of preconsolidation load values. Undrained field sampling processes and the absence of geological rebound (i.e. overconsolidation), are the reasons for raising this question, since the method was derived on the basis of drained sampling cycles on normally consolidated material.

The use of the other methods for normally consolidated soils does not compensate for the effects of undrained deformations. Consequently, no accurate solutions would seem to exist for the problem of finding field preconsolidation load from tests on undisturbed normally consolidated clay samples.

1.2 (d) Deformations Due to Total Stress Changes

The smallest degree of deformation which can be induced on a soil specimen, by any sampling process, is that which is caused by the reduction of its total stresses to zero (i.e. to atmospheric pressures). This idea that unloading of total stresses causes deformation in the soil, was originally introduced by Hvorslev [1949].

The stress system acting on "in-situ" normally consolidated clays is anisotropic, (i.e. the horizontal stresses are smaller than the vertical stresses).

The ratio of the horizontal and vertical effective stresses (see p.37), corresponding to the condition of zero lateral strain (which exists for sediments in almost horizontal layers of considerable extent), is usually termed the coefficient of earth pressure at rest, K_o (see Bishop [1958]).

Deformations are induced by trimming processes of samples obtained for testing. Even with the highest possible quality samples (e.g. block samples from shallow deposits) it is impossible to obtain a sample for laboratory strength and preconsolidation load testing, which has only suffered deformations caused by unloading of total stresses.

However, the process of unloading total stresses is one which easily lends itself to laboratory simulations, and so the effect of this "unavoidable" disturbance on strength and preconsolidation load of soils can be isolated.

1.3 "PERFECT" SAMPLING

1.3 (a) Introduction

Bishop and Henkel [1953] did the first laboratory study involving unloading of stresses. These researchers made an investigation of the "influence of anisotropic consolidation on the interpretation of test data". In the laboratory study, anisotropically consolidated samples were subjected to an undrained cycle of reduction of total vertical stress to total radial stress values (i.e. a change to an isotropic stress state). Bishop and Henkel termed this process an "ideal sampling" cycle. A comparison of compression test results on "ideal" samples, to those of isotropically consolidated samples, was used to verify the validity of undrained tests for "in-situ" strength data.

The term "perfect" sampling has subsequently been adopted by most authors, when referring to the process described by Bishop and Henkel.

1.3 (b) "Perfect" Sampling and Strength

The significance of "perfect" sampling was fully appreciated when Skempton and Sowa [1963] and Ladd and

Lambe [1963] published the results of simultaneous investigations of the effects of "perfect" sampling on "in-situ" strength of normally consolidated clays.

A "perfect" sample was then defined as one that has experienced no disturbance other than that associated with the release of "in-situ" shear stresses. These laboratory studies of the effects of "perfect" sampling on "in-situ" strength consisted of the following. Sample pairs of the same material were similarly prepared and K_o consolidated in the triaxial cell. Subsequently the sample chosen to represent the "ground" condition, was tested in undrained compression. The other specimen, which was to represent the "perfect" sample, was unloaded under undrained conditions to an isotropic stress state, as described by Bishop and Henkel, and then tested in undrained compression.

Skempton and Sowa reported a 1-1/2% difference in strengths (in favor of "ground" samples) for tests on remolded Weald clay, with a plasticity index = 24% and a sensitivity of 2. The significance of these results is however considered as being limited, since reconsolidated remolded materials will necessarily differ in many characteristics and properties from natural materials.

Ladd and Lambe [1963], from tests on natural Kawasaki clays, found that "perfectly" sampled specimens gave strengths 0 to 15% lower than non-sampled specimens.

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* See discussion of these results in Chapter IV

These clays had a plasticity index from 31 to 43% and a sensitivity of 10.

The same range of differences (see Ladd and Bailey [1964]) was obtained with tests on remolded samples (prepared from a slurry) of Boston Blue clay, which had a PI = 14% (sensitivity of 5 as a natural deposit). The work however is open to the same type of criticism as mentioned for Skempton and Sowa's investigation.

In 1965, Seed and Noorany investigating the effects of perfect sampling on "in-situ" strength of clays, found a 6% difference for tests on a natural material (San Francisco Bay Mud) with a PI = 45% and a sensitivity of 8.

1.3 (c) "Perfect" Sampling and Preconsolidation Load

"Perfect" sampling investigations should be very useful for predicting the susceptibility of different materials to changes in physical properties, because of sampling deformations. No work has however been reported on what effect "perfect" sampling might have on the void-ratio pressure diagram of any type of clays.

As described in 1.2 (c), the Casagrande graphical construction method for determining the preconsolidation load of normally consolidated clays, was not derived on the basis of undrained sampling cycles (e.g. "perfect" sampling). Since undrained conditions prevail with actual field sampling methods, it would appear that the validity of the construction for normally consolidated soils could be tested through a "perfect" sampling type of laboratory investigation.

1.4 RESEARCH SUGGESTED BY REVIEW

Despite the importance of the remarks made in 1949 by Hvorslev on the effect of small deformations on physical properties of brittle materials, it is observed that no work has been done to study the effect of "perfect" sampling on shear strength of natural silty clays (e.g. PI<15%) of moderate to high sensitivity.

An investigation of the effects of "perfect" sampling on preconsolidation load of normally consolidated clays, and of the applicability of Casagrande's construction to the results is suggested.

Hvorslev on tube sampling deformations indicates that a sensitive, brittle soil, such as a normally consolidated natural silty clay, would be appropriate for an investigation of the effect of extension failure on the compression strength of samples.

A borehole bottom failure investigation is also thought to be relevant for this type of clays. Conclusions on Rutledge's observations can be derived from this investigation.

Since lean clays (i.e. silty) are often encountered in soils engineering problems it was decided that investigations as mentioned above, would be conducted in order to fill in these obvious gaps in the spectrum of sampling studies.

CHAPTER 2

SAMPLING AND STRENGTH OF NORMALLY CONSOLIDATED SILTY CLAYS

2.1 INTRODUCTION

Following the conclusions and suggestions in Chapter 1 on the sampling of clays, the necessary steps were considered for obtaining undisturbed samples of a natural lean clay.

Important factors, such as sensitivity and softness of the material, indicated that a normally or slightly overconsolidated shallow deposit of silty clay would probably be a good source for the required samples.

Accordingly, a preliminary survey of lacustrine deposits in the Hamilton Bay area was made, through the use of borehole records.

2.2 MATERIAL

2.2 (a) Location

After some probing in the Hamilton Bay front area, the desired material was found in the fore bay of H.M.C.S. Star.

2.2 (b) Field Operation

Field sampling and testing of the deposit was carried out during the period of May 28 to June 7, 1969.

In order to ensure that the best possible samples would be obtained, it was decided that a fixed piston sampler should be used. This type of sampler was used, together with 4-3/4 inch diameter sampling tubes and 6 inch diameter casing. The sampling tubes had a wall thickness of 1/8 inches and a length of 5 feet.

Precautions were taken to see that the tubes were gradually pushed into the soil rather than dynamically driven, and that the borehole was filled with water at all times.

Vane tests were done on the deposit, in a borehole adjacent to the one from which the samples were obtained.

2.2 (c) Properties

Some variations are to be expected in the properties of material obtained from a natural deposit. In this case, some of the variations are shown in Figures 1 and 2, for the material used in the laboratory testing program.

It will be seen in Figure 1 that essentially two kinds of material were used, one having a PI $\approx$ 15 and LL $\approx$ 37 and the other having a PI $\approx$ 11 and LL $\approx$ 29.

On the Casagrande plasticity chart the first material would be located in the "silty clays of medium plasticity" region, and the second material would be located in the "silty clays of low plasticity" region. Henceforth, the first material will be referred to as silty clay and the second as clayey silt.

The specific gravity of the soil was found to be 2.73.

Figure 2 shows the results of grain size analyses on typical samples of the two materials.

The field vane tests indicated that the material had a moderate sensitivity (4 to 7), and tests on samples labelled 1 to 3 in Figure 1 showed that it was lightly over-consolidated.

Typically the "silty clay" had a natural water content of 33% and the "clayey silt" had a natural water content of 25%.

2.3 SAMPLE PREPARATION

In order to prepare samples for the laboratory testing program the following procedure was followed.

Firstly, a length of clay was measured at the bottom or cutting end of the tube, equal to 2-1/2 times the tube diameter. Then the same was done for the clay at the other end, this time measuring 2-1/2 times the diameter of the casing. These lengths of clay at the ends, were considered as being part of any failure bulbs that might have developed in drilling the hole and pulling or twisting the tube out of the soil.

Subsequently, two 9 inch sections were cut out of the middle or usable part using a band saw, and cutting only through the metal (the soil was subsequently cut with a wire saw). The ends of these sections were sealed, using saran sheeting, aluminum foil paper and a micro-crystalline wax,

(Esso Micro Van 1400)

When samples were required 4-1/2 inch lengths of the material were extruded from the sections, and the remainder was resealed and stored again. The 4-1/2 inch lengths were then cut into four equal vertical slices with a fine wire saw. Slices not to be immediately used were sealed and stored in a humid room.

Samples 1.40 inches in diameter by 3.50 inches in height were carefully trimmed from each of the slices, using a fine wire saw and an appropriately precise lathe. The use of samples of this size was justified because of the substantial shortening to be expected from the K_o consolidation stages prior to undrained shear testing. Also, these had to conform to the size of the pedestal of the triaxial cell base which was 1.40 inches in diameter.

2.4 APPARATUS

The laboratory measurement of shear strength of clay soils under controlled conditions of drainage and of deformation characteristics, is generally accomplished with triaxial test apparatus.

The principal features of a triaxial test apparatus are as follows. The sample of cylindrical shape, is enclosed in a watertight cover (rubber membrane) and placed in a chamber that can be filled with fluid under pressure. An additional axial stress can be applied to the top of the sample through a rigid cap by means of a ram through a bushing in the top of the chamber. Water may enter or leave the sample through a porous stone in the bottom, seated on the pedestal of the chamber base which has two drainage holes for this purpose. An undrained condition can be imposed by closing a valve on the discharge line and the pressure of the water in the sample may be measured by means of a pressure gauge connected to this line. An extensometer (dial gauge) is provided to measure the strain of the sample in the vertical direction.

The triaxial test apparatus used in this investigation is shown in Figure 3. A short description of some of its features follows.

The cell pressures were applied through the use of self-compensating mercury manometers of the Bishop[‡] type. These pressures were read on the Bourdon-tube type of pressure gauge, which was calibrated* in-situ before performing any tests. The manual control pressure cylinder provided a source of pressures to maintain a no-flow condition at the null, when reading pore pressures in undrained tests. These pressures were also read on the Bourdon gauge.

The volume-change gauge, of the type described by Bishop and Henkel, 1962 (p. 208), had a 25 c.c. burette which could be read with an accuracy of about 0.02 c.c..

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‡ see Bishop and Henkel 1962

* Rotating Ram Dead Weight Calibrator

The vertical rod, which is shown fixed on to the base of the cell, was used for attaching the datum for measurement of vertical deformation of the samples. The dial gauge used for these measurements had an accuracy of 0.0001 inches. Since straining of the tie rods would occur as the cell pressures changed, it was not considered desirable to have the datum fixed on to the top of the cell (the usual arrangement). Hence, the above set up made possible an accurate monitoring of the K_o consolidation stages (see Procedure).

A perspex type of null indicator of the type described by Bishop and Henkel [1962] was used. This null was clamped on to the cell base, through a very short and rigid arm which provided a link with a drainage hole in the pedestal and the base of the sample. The no drainage condition was controlled by maintaining a no movement requirement of the mercury inside the perspex body of the null. The perspex body used had been shrunk and anealed in order to reduce changes in volume during use and remove residual stresses.

One of the most outstanding features of this particular apparatus is the motor and rotating bushing system, which reduces friction on the ram through which vertical loads are applied. These loads were measured using a 300 lb proving ring, which was calibrated with static weights. The results of this calibration were used in the form of point to point calibration factors, as given by a graph. It is estimated that in this way loads could be read with an accuracy of 0.05 lbs.

The constant rate of strain for the strain tests was supplied by the motor and gear box system, as shown in the figure.

Top caps for the samples were also specially designed. The top caps used for the compressive strength tests were made of perspex and had a centered guide hole for the ram. In order to avoid eccentricity in vertical loading of the specimens, the ram made contact with the top of the sample through a flat stainless steel disc at the end of this guide hole. For the extension test a brass top cap was used, which had slots on the vertical sleeve guide. A small rotation was used to engage the pins on the sides of the ram extension piece in these slots. Then, by lowering the cell, an upward force could be applied to this top cap.

2.5 EXPERIMENTAL PROCEDURE

2.5 (a) Mounting Sample

The assembly of the prepared cylindrical specimens, for the triaxial tests was as follows.

After the sample weight was recorded, side drains consisting of wet Whatman's No. 54 filter paper with vertical slashes at 1/4 inch intervals (see Appendix I) were put on.

The sample was then set on a saturated porous stone on top of the pedestal, a top porous stone and top cap were added and the sample was enclosed by a prophylactic (see Appendix II). Two rubber "o" rings were snapped on at each end (i.e. at pedestal and at top cap) to ensure a watertight condition for the enclosed specimen. The cell was filled with deaired water and a cell pressure of 10 psi was applied for a period of approximately 12 hours, to allow air trapped between the membrane and the sample to diffuse through the membrane and dissolve in the deaired cell water. After this period of time the cell was disassembled and silicone grease was smeared on the membrane, as part of a procedure required to reduce water leakage through membranes (see Appendix II). A second membrane and "o" rings were placed over the original one and again a period of approximately 8 hours, under 10 psi cell pressure, was allowed to remove air trapped between the two membranes.

2.5 (b) Cell Pressure - Pore Pressure Test

A cell pressure-pore pressure test was done on the assembled sample. This consisted of increasing the cell pressure in a series of 10 psi increments and reading corresponding pore pressures, allowing time for equalization of these pressures inside the sample at each stage (see Appendix III).

The time allowed for equalization was 120 minutes for each cell pressure increment, and readings of the pore pressure were taken against elapsed time in order to study the response times of the system (see Appendix III). A back pressure value (i.e. pore water pressure required in the sample to ensure 100% saturation) was selected on the basis of when the value of B* first equalled unity for the given increment.

Figure 4 shows the results of a typical cell pressure-pore pressure test. A back pressure of 30.0 psi was used for all tests. (see Appendix III).

2.5 (c) K_o Consolidation

Generally field deposits consist of almost horizontal layers of considerable extent, such that consolidation which takes place due to the weight of succeeding strata occurs under conditions approximating zero lateral yield.

The laboratory simulation of this process, which is known as K_o consolidation, has been described by Bishop [1958] for triaxial test specimens.

The Bishop method is based on the fact that the volume of water expelled from a cylindrical specimen in axial compression is equal to the change in length multiplied by the

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* $B = \dfrac{\text{pore pressure increment}}{\text{cell pressure increment}}$ (equal to unity for 100% saturation - see Skempton [1954])

initial cross sectional area, if the lateral strain is zero. Hence, increments of axial stress and cell pressure are adjusted to maintain this condition throughout the test. For the present investigation a slight modification was made in the method of application of stresses, although the same principle for lateral yield was used.

A table was made up for the required ram movement (or change in sample length) for each 0.1 c.c. of water expelled by the sample. Changes in sample length were read with an accuracy of 0.0001 inches and the volume of water expelled was read with an accuracy of 0.02 c.c., so that the requirements of the method were very closely maintained.

Under no drainage conditions, the cell pressure was increased by a increment equal to the radial effective stress of the sample, and maintained at this value. The drainage was subsequently opened and the vertical strain was increased gradually by manual control of the ram movement, as prescribed in the table for the observed volume of water expelled. Readings of volume change were recorded at chosen elapsed time intervals and the obtained "volume change vs $\sqrt{\text{time}}$" relationships were plotted. Generally 500 to 900 minutes were allowed for each consolidation step done in this manner, although primary consolidation rarely took more than 120 minutes.

After the first consolidation step, care was needed when applying the cell pressure increment for each of the subsequent consolidation stages because some of the ram load would be removed in the process, due to further compression of the proving ring. Compensating for this load removal was then done simultaneously with the application of the cell pressure increment, by using a cell pressure-proving ring load calibration curve. A check was maintained using the vertical deformation dial.

2.5 (d) Strength Testing

1) Ground Samples

The drainage was closed off after the sample was K_o consolidated to the desired stress level. These levels were selected to satisfy the conditions that field preconsolidation load had to be exceeded (normally consolidated clay requirement) and that maximum effective radial stresses were dictated by the range in cell pressures (= 122 psi) and the use of a back pressure (= 30 psi). The strength testing levels, in terms of effective radial stresses, were then 23.0, 46.0, 72.0 and 92.0 psi.

A strain rate was selected (see Appendix III) and the sample was tested by increasing the ram load, under undrained conditions. Readings of pore pressure, proving ring load and vertical deformation of the sample were taken at regular time intervals.

2) Perfect Samples

The drainage was closed off at the end of consolidation, as previously described, and the ram load on the sample was decreased to zero in four steps. For each step, an increment of load was taken off instantaneously, and the pore pressure and vertical deformation readings were taken for a period of approximately 120 minutes. It was observed that this time was sufficient to allow equalization of pore pressures, because readings reached asymptotic values with time (however some very slight increase was still detected; probably creep). For the final step, which took the sample to an isotropic stress state, approximately 12 hours were allowed before the vertical deformation and pore pressure readings were completed.

Following this "sampling" procedure the specimen was tested in compression as described for the ground sample strength (see Appendix III for selection of strain rate of testing).

3) Extension Sample

The procedure followed for this sample was the same as for the "perfect" sample, up to the end of sampling to isotropic stress conditions. After this the ram was secured to the top cap (as described in 2.4) and the cell pressure was increased sufficiently (by combining two mercury manometer systems) to create an upward thrust on the ram capable of reducing vertical stresses on the sample to cause an extension failure. The necessary reaction to this upward force was supplied by the proving ring. A constant rate of strain was selected for the gearbox, and the sample was then tested under controlled strain conditions. By moving the cell down, the proving ring load, and consequently the axial load on the sample, was decreased.

When failure was detected (see Figure 10) the test was immediately stopped, and an unloading stage to isotropic stress conditions was started. The procedure followed was the same as that for the "sampling" stages, except that in this case the vertical load was increased in increments. When no ram load on the top cap was attained and the sample was in an isotropic stress state, the sample was tested to failure in compression, as in the previous cases (i.e. using the gear box, set to an appropriate strain rate, which in this case was the same as that used for the "perfect" samples).

4) Remolded Samples

These samples, which had already been tested as "ground" samples, were now remolded and again tested for compression strength at the same water content as before (i.e. no consolidation performed).

2.6 RESULTS

The results of the laboratory investigation of the effects of sampling on the strength of lean clays, are summarized in Table I.

Undrained shear strength is defined as "one half of the maximum deviator stress" [i.e. $\frac{1}{2}(\sigma_1 - \sigma_3)_{max}$].

Failure is defined on the basis of maximum deviator stress.

2.6 (a) "Ground" Strength - "Perfect" Sample Strength

The deviator stress and porewater pressure behaviour with strain is illustrated in Figures 5 to 8. From these figures it is seen that "ground" samples failed at 1 to 1-1/2% strain, whereas "perfect" samples failed at 2 to 3% strain. Pore pressure curves for the "perfect" samples generally assumed a flat shape much earlier than those for the "ground" samples.

The "perfect" samples of silty clay, i.e. PSS4 and PSS6 showed 0.8% and 1.0% strains due to "perfect" sampling, whereas the clayey silts PSS5 and PSS7 showed 0.6% and 0.7% strain respectively.

The effective stress paths obtained for the "ground" and "perfect" samples are shown in Figure 9. Although the stress paths for each sample of a pair are radically different in shape, it is noted that they almost coincide in the region of the failure envelope.

2.6 (b) "Ground" Strength - "Extension" Sample Strength

It should be mentioned here that filter strips were left on the "extension" sample, on the assumption that it would have failed in extension by the time it reached its initial length (3.5 inches), such that full benefits of their use (see Appendix III) can be obtained. However this did not happen. As seen in Figure 10, interference from the filter strips (see Appendix I) obscured the results after the original length was attained.

In order to arrive at the deduced stress path shown in Figure 11, the assumption was made that a pore pressure parameter, here conveniently expressed as the ratio $\Delta u/\sigma_m'$ (where m = minor principal effective stress coordinate at the point where unloading is started) would be constant for both unloading paths (i.e. XS and EY in Figure 10). Numerically this produced:

$$\frac{\Delta u_{XS}}{\sigma_{3c}'} = \frac{2.35}{23.0} \approx 0.1 \quad \text{hence} \quad \frac{\Delta u_{EY}}{\sigma_v'} = 0.1 \quad \text{and} \quad \sigma_v' = 6.0 - \Delta u_{EY}$$

Solving the above two equations simultaneously:

$$\Delta u_{EY} \approx 0.55 \text{ psi}, \quad \sigma_v' = 5.45 \text{ psi}$$

Extending the $\sigma_v' = 5.45$ line to intersect with the stress path obtained up to initial length of the sample, the point E was obtained which is the best approximation that can be used to describe the state of stess which the sample was carried to in extension.

The "ground" sample used for comparison of behaviour and strength was GSS4R.

The effective stress paths (see Figure 11) show that a great reduction in effective stresses took place in the sample which was subjected to extension, prior to compression testing. The stress path for compression testing of this specimen was very much different from that of the "ground" specimen but similar to the remolded samples.

The strain to failure, in compression, of the "extension" sample was about 13%, compared to 0.8% for the "ground" sample.

2.6 (c) "Ground" Strength - Remolded Sample Strength

The results of tests on remolded samples are shown in Figures 12 and 13. As explained previously [section 2.5 (d) 4)] the samples were made from the material of "ground" samples GSS4R and GSS7.

There are very obvious differences between the curves obtained for the undisturbed material and those for the remolded material. Aside from great reductions in the deviator stress and pore pressure values, complete remolding also caused great changes in the shape of pore pressure curves, as well as in the shape of effective stress paths.

The ratio of the strengths of the samples showed that the material had a sensitivity of 5, as measured in the laboratory.

2.7 (d) Failure Envelopes

The failure envelope for the samples of silty clay, as seen in Figure 14, shows that this material had a friction angle ϕ', equal to 27.8°.

The failure envelope for the clayey silt, shows in turn that the friction angle ϕ', for this material was 29.9°. (Figure 15)

There is a unique failure envelope for cases where correspondence in peak deviator stress and peak principal effective stress ratio is observed. This correspondence existed for the "perfect" and the remolded samples but was not observed for "ground" samples.

It has been shown in Appendix III, that the degree of pore pressure equalization at peak deviator stress has an influence on the correspondence of the two failure criteria. Consequently the pore pressures and effective stresses as obtained for "ground" samples at peak deviator stress are considered to be slightly in error, which is reflected in the results plotted in Figures 14 and 15.

2.7 DISCUSSION OF RESULTS

It was found, for the silty clay from Hamilton Bay that "perfect" sampling had only a small effect on the "in-situ" strength [see Table I]. The average decrease in strength due to "perfect" sampling was 1%, while 4-1/2% was the highest value recorded.

It is thought that actual sampling (i.e. tube sampling) of this clay may lower its "in-situ" strength by as much as 35.5%, as reflected by the results of the "extension" sample. It should be noted that this is probably a conservative estimate, since the "ground" sample used for comparison was a medium plasticity, high water content sample (see Table 1), from which a lower value of strength could be expected than for the case of a low plasticity, low water content sample.

The percent differences in the "perfect" sampling case were also conservative, since final water contents (and effective vertical stress values) were generally lower in the case of the sampled specimen, which would tend to increase slightly the value obtained for strength. In the case of the number 7 pair (see Table I) the sampled specimen gave a strength 2.6% higher than the "ground" specimen, a fact which is thought to be in part due to differences in final water contents of the two samples. This problem seemed to have been accentuated for the siltier pairs.

The low plasticity samples (5 & 7) suffered less vertical deformation but developed more pore pressure than the medium plasticity samples (4 & 6), due to the "perfect" sampling process (see Figures 5 to 8).

From the results of the remolded samples it would appear that sampling from a failed borehole in this type of deposit, would originate a 400% difference in "in-situ" strength, as determined by undrained compression tests.

Also the results can be interpreted as meaning that the laboratory samples GSS7 and GSS4R, had a sensitivity of 5.

ELEV. 40'00"

Sample	LL	PL	PI
PSS1	30.8	18.6	12.2
GSS2	29.2	17.6	11.6
GSS3	31.2	19.0	12.2
PSS3	32.2	19.3	13.0
PSS2			
GSS4			
PSS4	35.5	21.5	14.0
PSS5	36.6	22.3	14.3
GSS5	36.1	21.8	14.3
GSS4R	37.3	22.3	15.0
GSS6	37.0	21.5	15.5
PSS6			
GSS5R	28.5	17.9	10.6
GSS7	31.0	18.8	12.2
PSS7	26.8	16.3	10.5

<u>SAMPLING TUBE I</u>

DATA:

4-3/4" inside diameter

1/8" wall thickness

Sampled From: 39' to 42'

Water Table @ 0 feet

Soil Surface @ 29 feet

NB. Dotted Line Separates 4-1/2" sections of clay as extruded

ELEV. 50'00"

Sample	LL	PL	PI
GSS5RII	29.4	18.3	11.1
EI	30.4	18.5	11.9
PSS5R	30.3	19.1	11.2

<u>SAMPLING TUBE II</u>

DATA:

4-3/4" inside diameter

1/8" wall thickness

Sampled From: 49' to 52'9"

Water Table @ 0 feet

Soil Surface @ 29 feet

<u>NOTE:</u>

GSS ≡ Ground Strength Sample

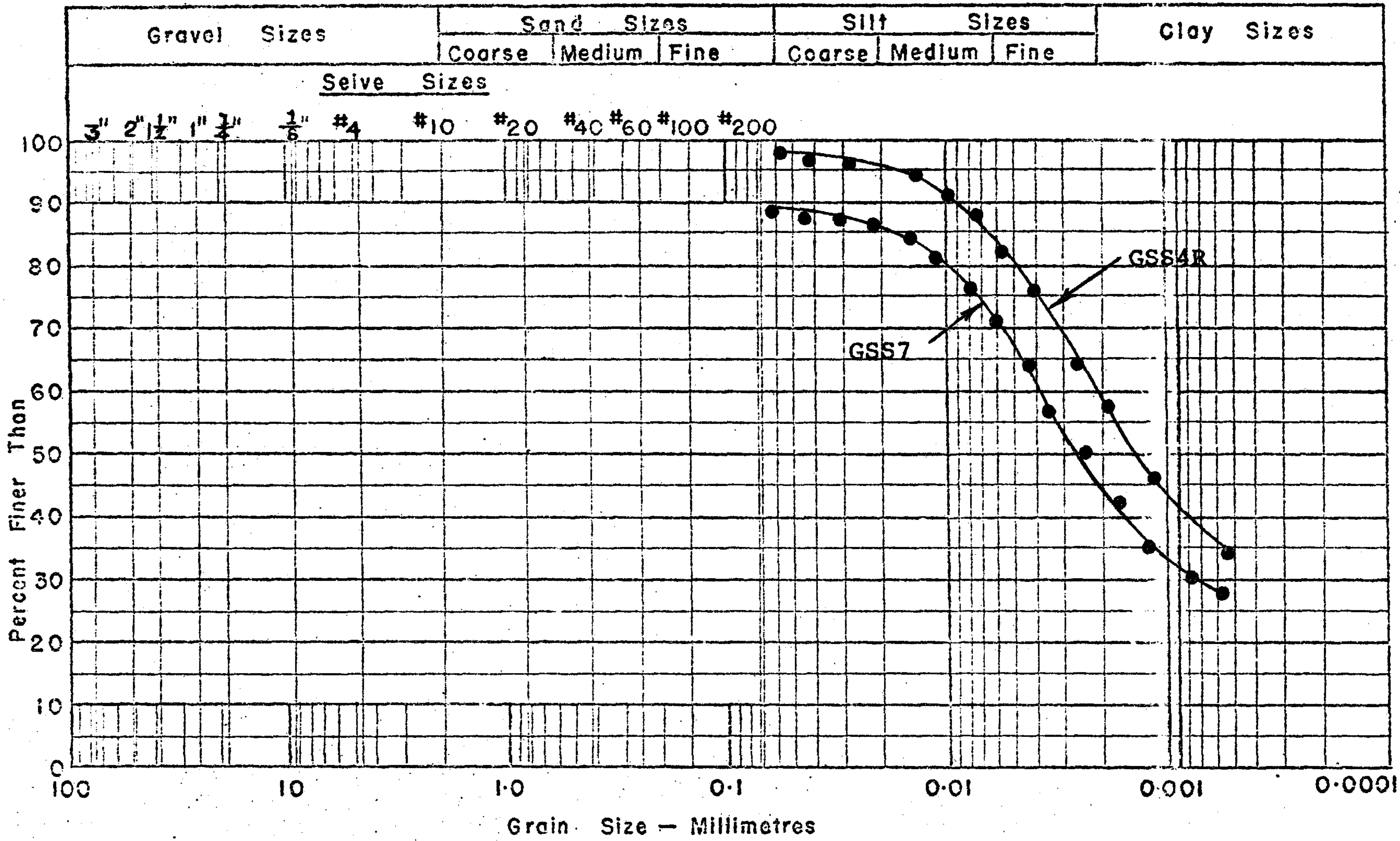
PSS ≡ Perfect Strength Sample

Numbers 1 to 7 identify the pair

An additional R means a repeat sample

E ≡ Extension Test Sample

<u>FIGURE 1 LOCATION AND CHARACTERISTICS OF MATERIAL TESTED</u>

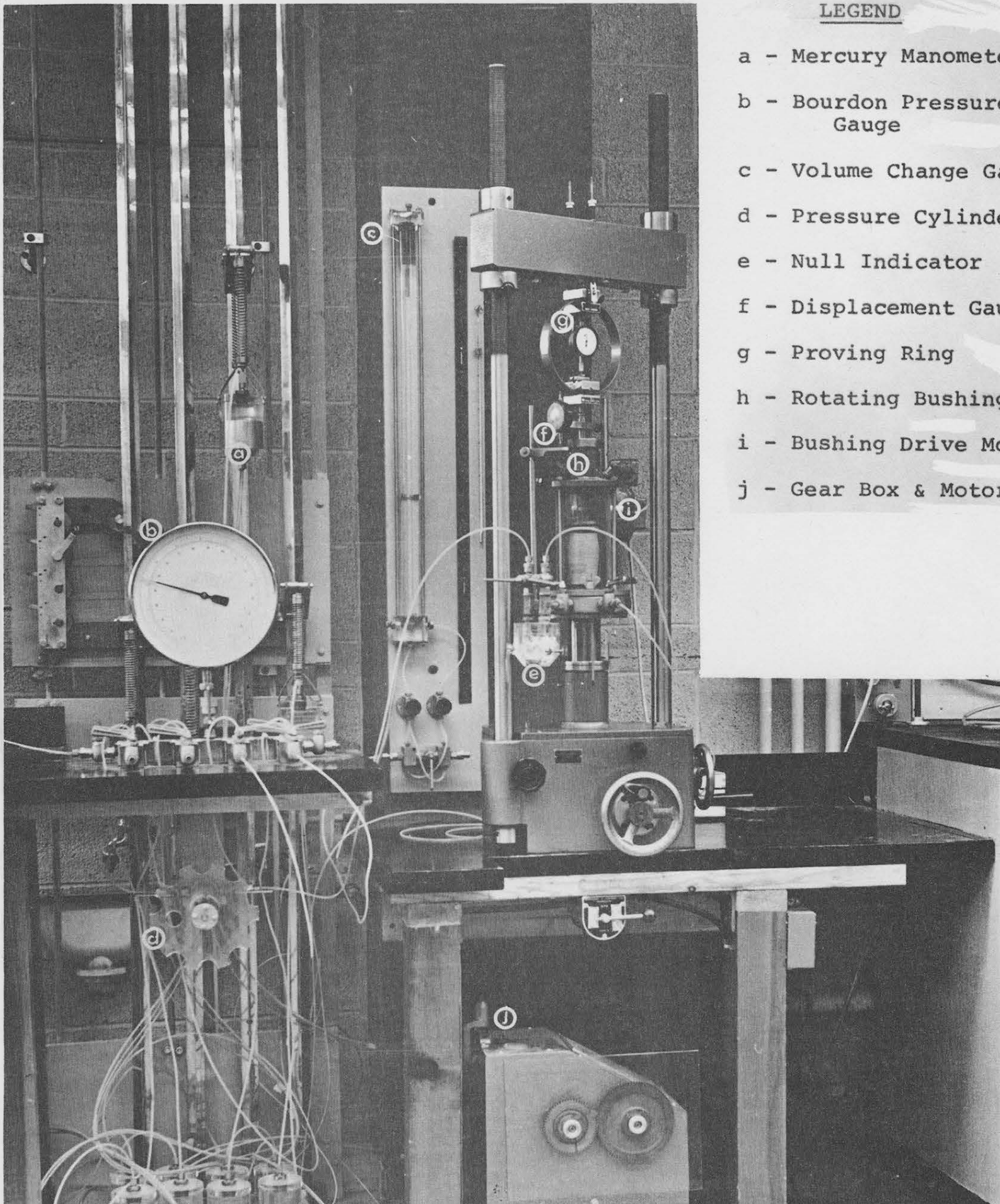


Remarks: GSS7	GSS4R
LL = 31.0	LL = 37.3
PL = 18.8	PL = 22.3
PI = 12.2	PI = 15.0

D_{10} = ______ mm.
D_{60} = ______ mm.
C_U ______

Note: M·I·T Grain Size Scale

FIGURE 2 GRAIN SIZE DISTRIBUTION - TWO TYPICAL SAMPLES



LEGEND

a - Mercury Manometer

b - Bourdon Pressure Gauge

c - Volume Change Gauge

d - Pressure Cylinder

e - Null Indicator

f - Displacement Gauge

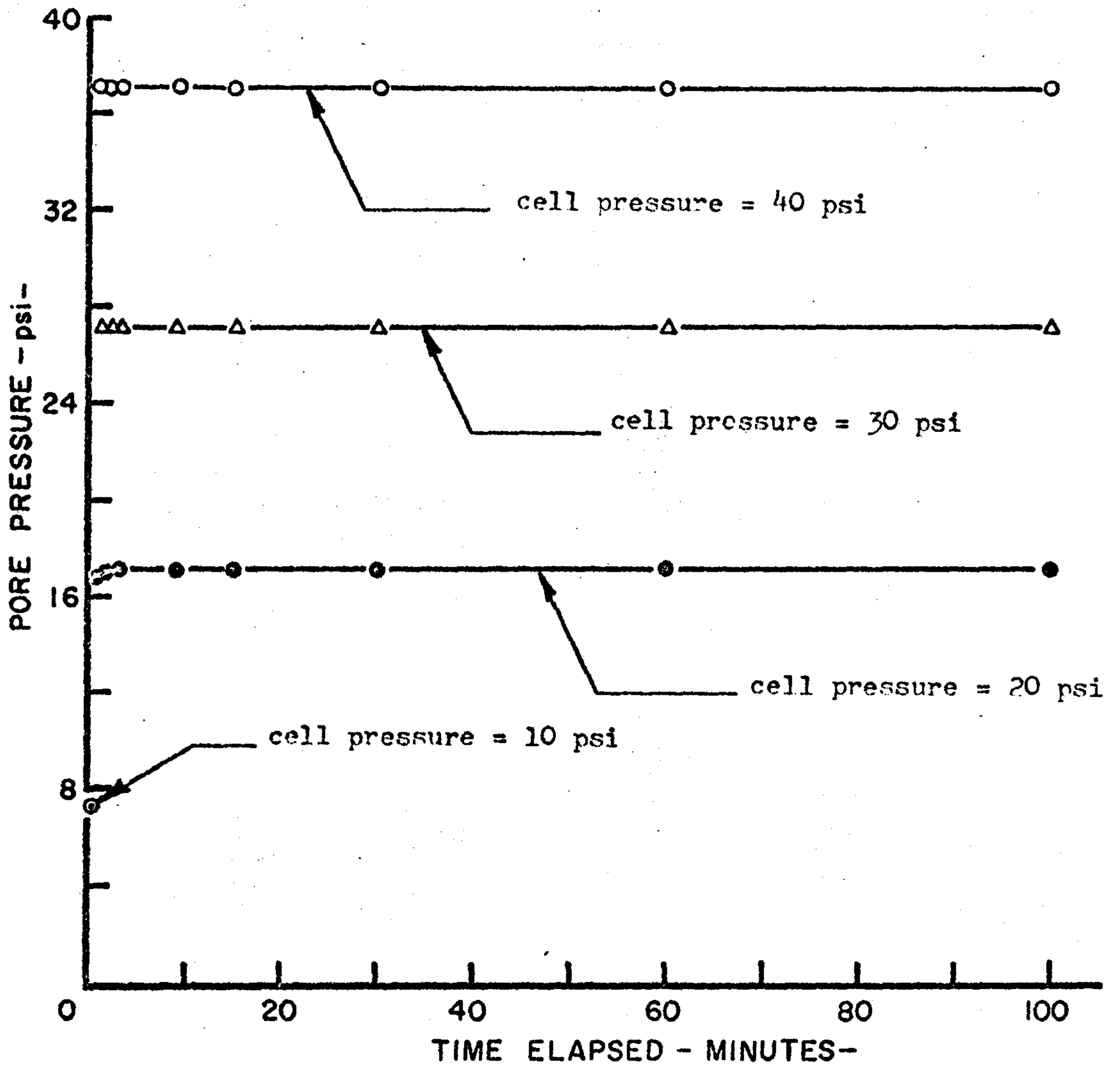
g - Proving Ring

h - Rotating Bushing

i - Bushing Drive Motor

j - Gear Box & Motor

FIGURE 3 TRIAXIAL TEST APPARATUS



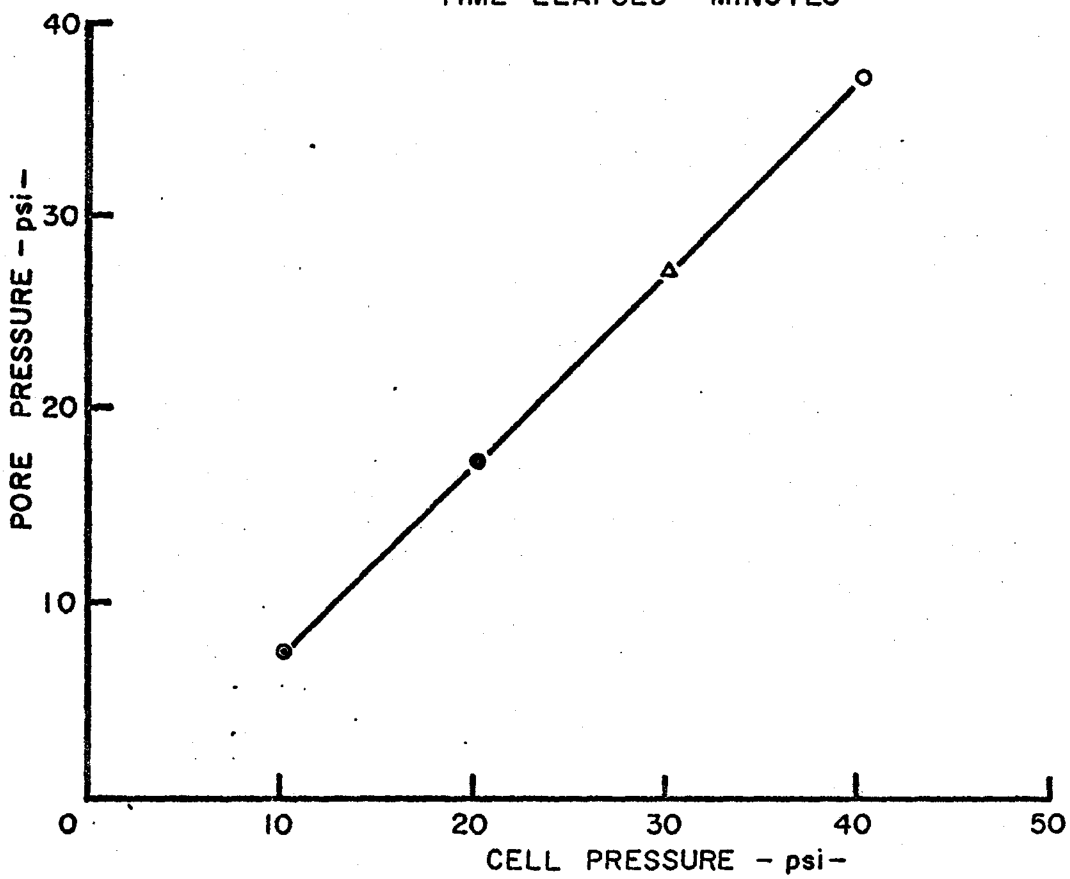


FIG. 4 CELL PRESSURE – PORE PRESSURE TEST :

SAMPLE TYPE	SYMBOL	INITIAL* WATER CONTENT	PLASTICITY INDEX	EFFECTIVE** VERTICAL CONSOLIDATION PRESSURE	EFFECTIVE** RADIAL CONSOLIDATION PRESSURE	COEFFICIENT OF EARTH PRESSURE AT REST λ	FINAL‡ WATER CONTENT	$\frac{(\sigma_1-\sigma_3)_{max}}{2}$ ∂	PERCENT DIFFERENCE IN STRENGTH
		w_1-%	PI-%	σ'_{1c}-psi	σ'_{3c}-psi	K_o	w_f-%	C_u-psi	$(\frac{\text{Ground Cu}}{\text{Sampled Cu}} - 1)100.$
GROUND	GSS4R	33.0	15.0	45.8	23.0	0.50	30.1	13.0	+ 4.5 %
PERFECT	PSS4	33.3	14.0	45.5	23.0	0.50	30.1	12.4	
EXTENSION	EI	25.3	11.9	45.6	23.0	0.50	22.3	9.6	+ 35.5 %
REMOLDED	GSS4R	30.1	15.0	-	-	-	30.1	2.6	+ 400 %
GROUND	GSS5RII	25.0	11.1	89.6	46.0	0.51	19.5	24.8	- 0.6 %
PERFECT	PSS5R	25.4	11.2	90.7	46.0	0.51	19.4	25.0	
GROUND	GSS6	33.1	15.5	137.1	72.0	0.52	24.7	35.7	+ 2.7 %
PERFECT	PSS6	32.5	-	138.6	72.0	0.52	23.8	34.7	
GROUND	GSS7	26.7	12.2	184.1	92.0	0.50	18.7	49.6	- 2.6 %
PERFECT	PSS7	24.6	10.5	186.3	92.0	0.49	18.1	50.9	
REMOLDED	GSS7	18.7	12.2	-	-	-	18.7	10.2	+ 396 %

FOOTNOTES

* Initial water contents were calculated on basis of initial weight of the sample and oven dry weight of the sample found at the end of testing

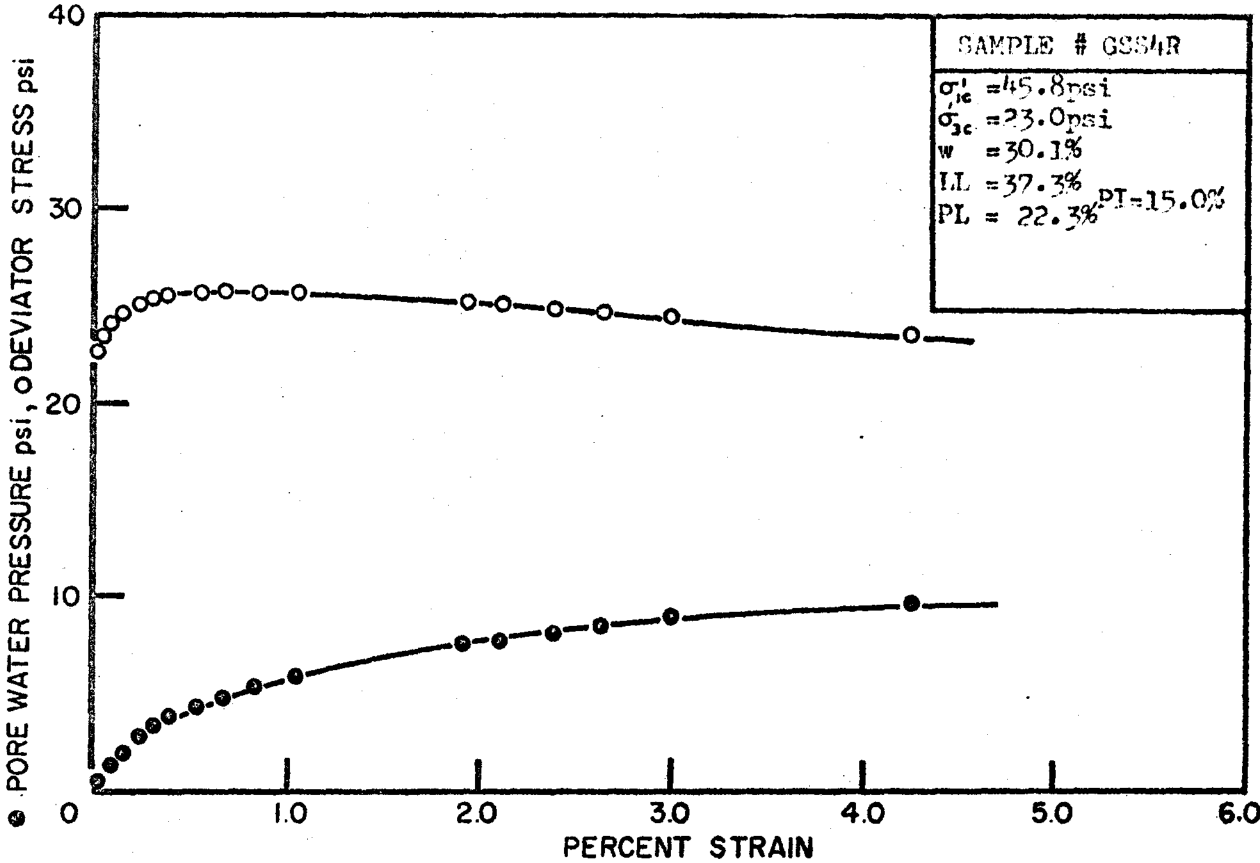
‡ Final water contents were calculated on basis of recorded volume changes in consolidation and above initial water content data

∂ Undrained shear strength is defined as half the maximum principal stress difference, i.e. $\frac{(\sigma_1-\sigma_3)_{max}}{2}$

** Effective Stress = Total Stress Minus Pore Water Pressure ($\sigma' = \sigma - u$)

λ $K_o = \sigma'_{3c}/\sigma'_{1c}$

TABLE 1



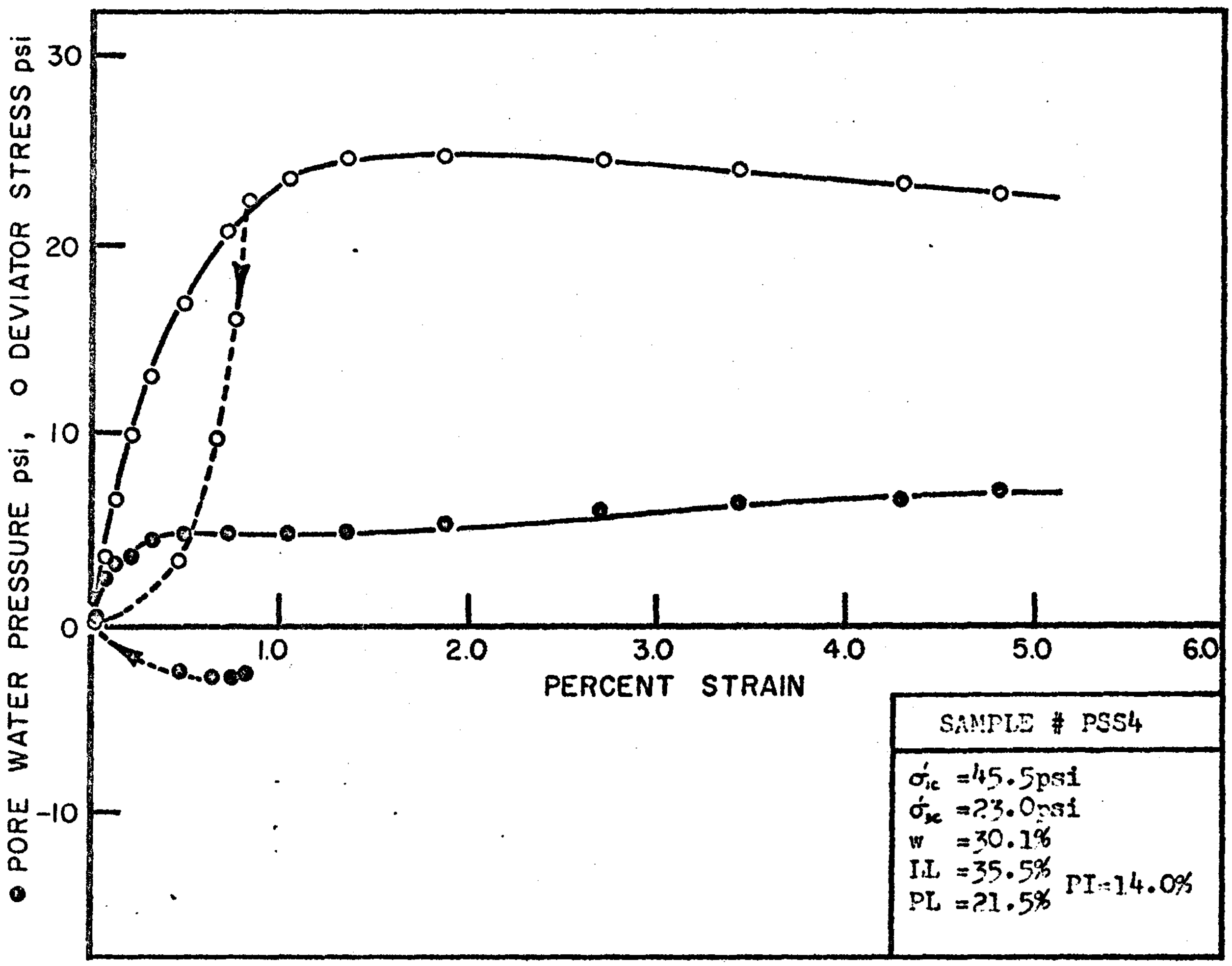


FIG. 5. STRESS-STRAIN RELATIONSHIPS — NO. 4 PAIR —

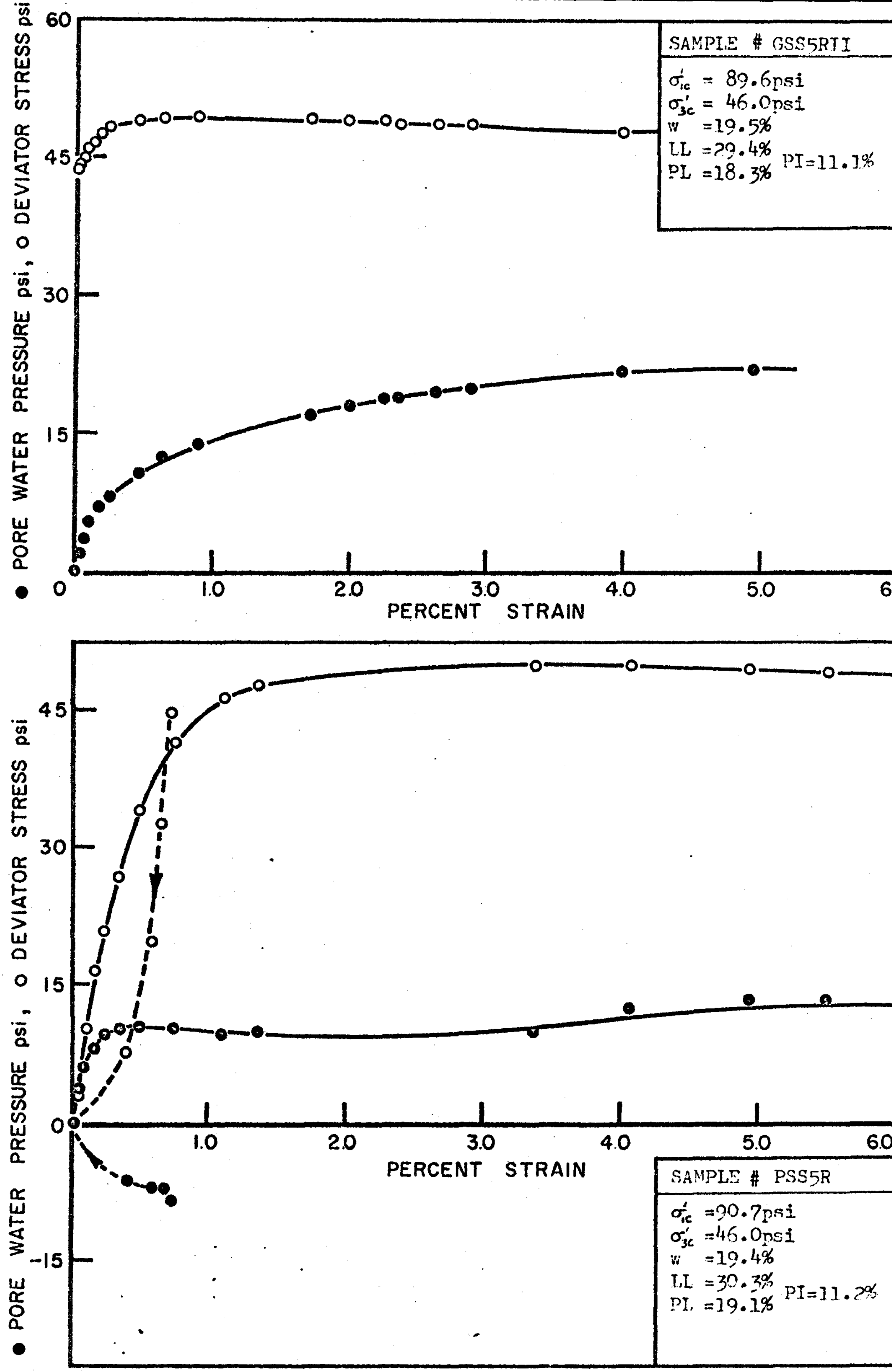


FIG. 6. STRESS–STRAIN RELATIONSHIPS — NO. 5 PAIR —

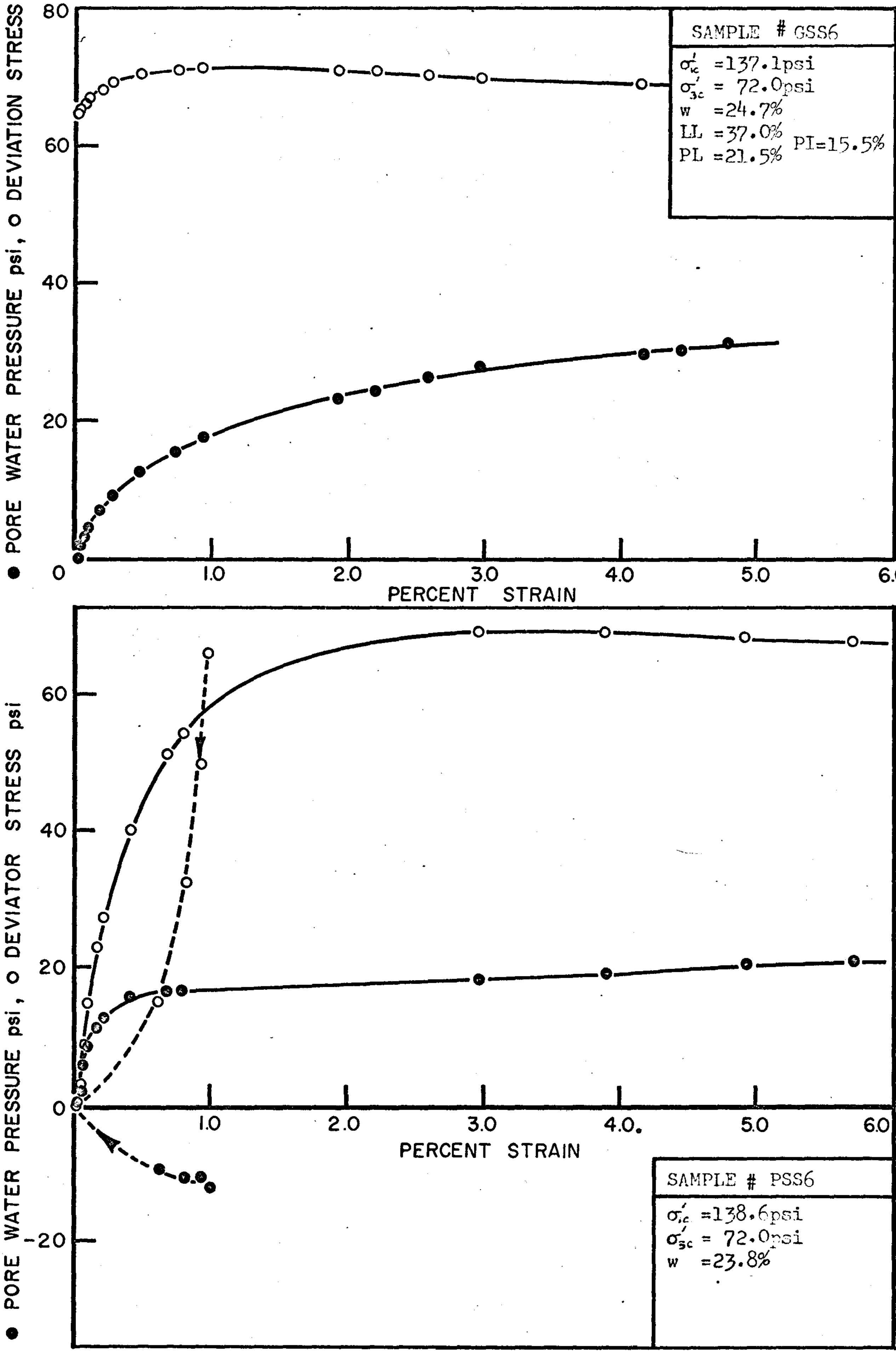


FIG. 7. STRESS-STRAIN RELATIONSHIPS — NO. 6 PAIR —

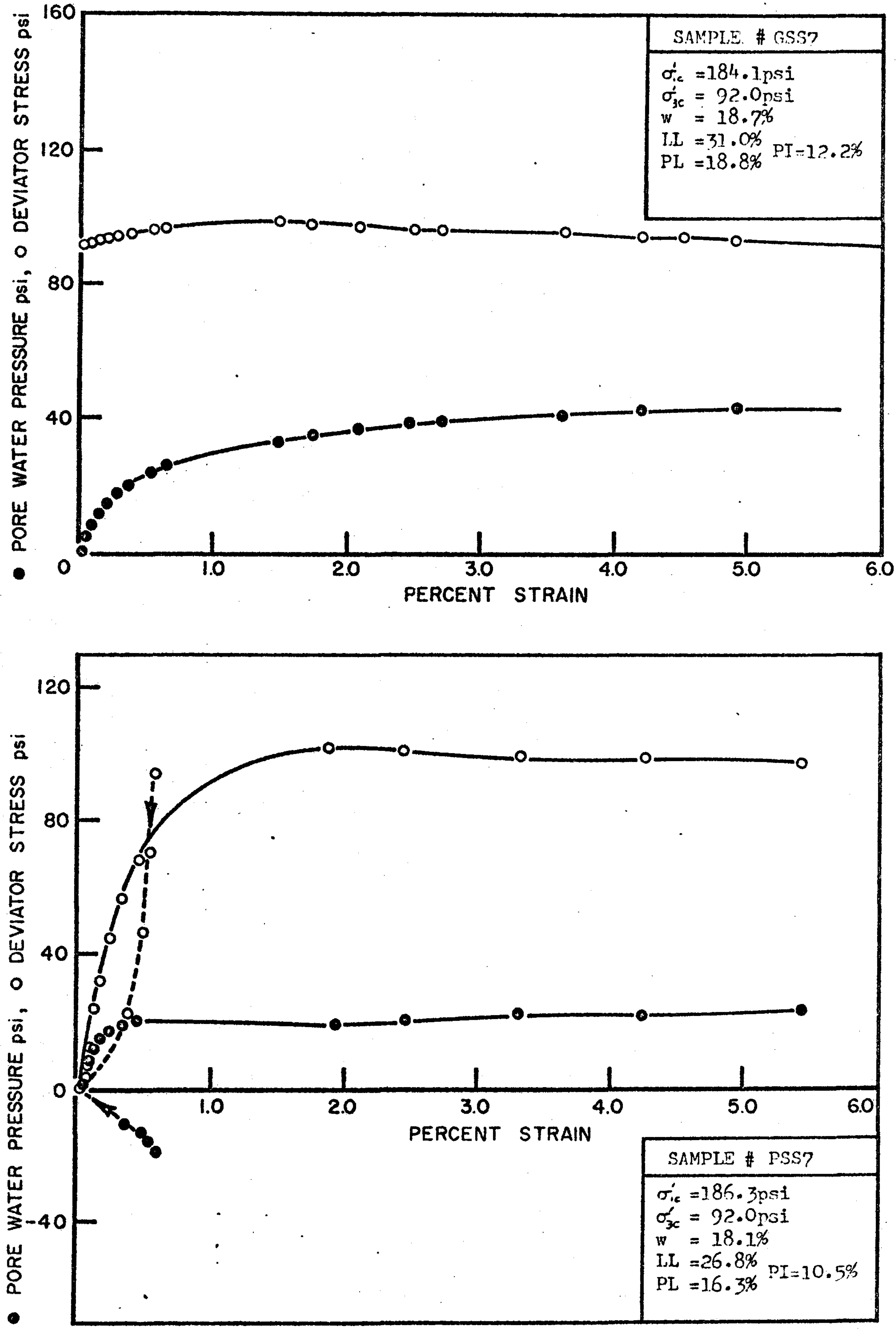


FIG. 8. STRESS-STRAIN RELATIONSHIPS - NO. 7 PAIR-

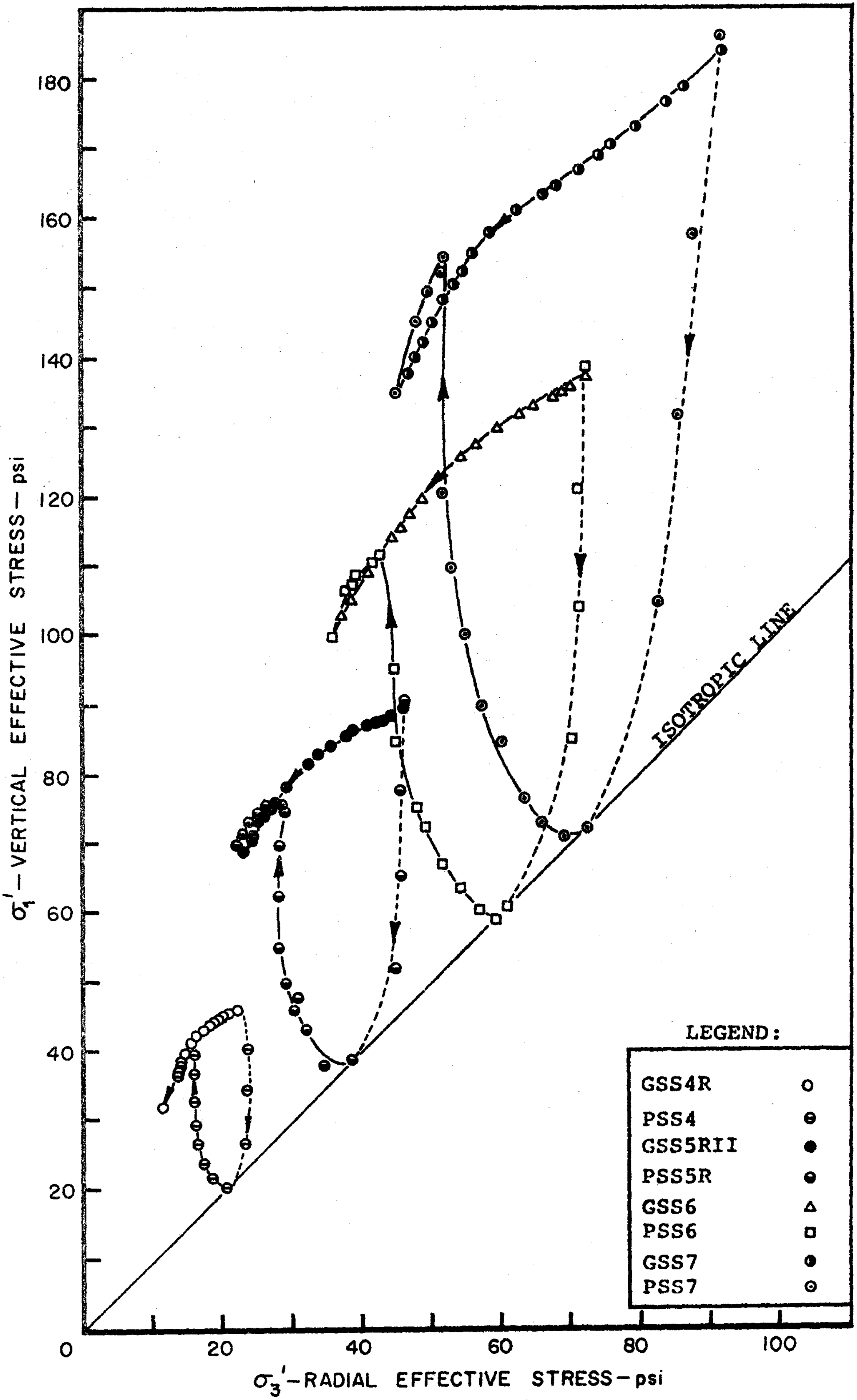


FIG. 9. "GROUND" and "PERFECT. SAMPLE STRESS PATHS.

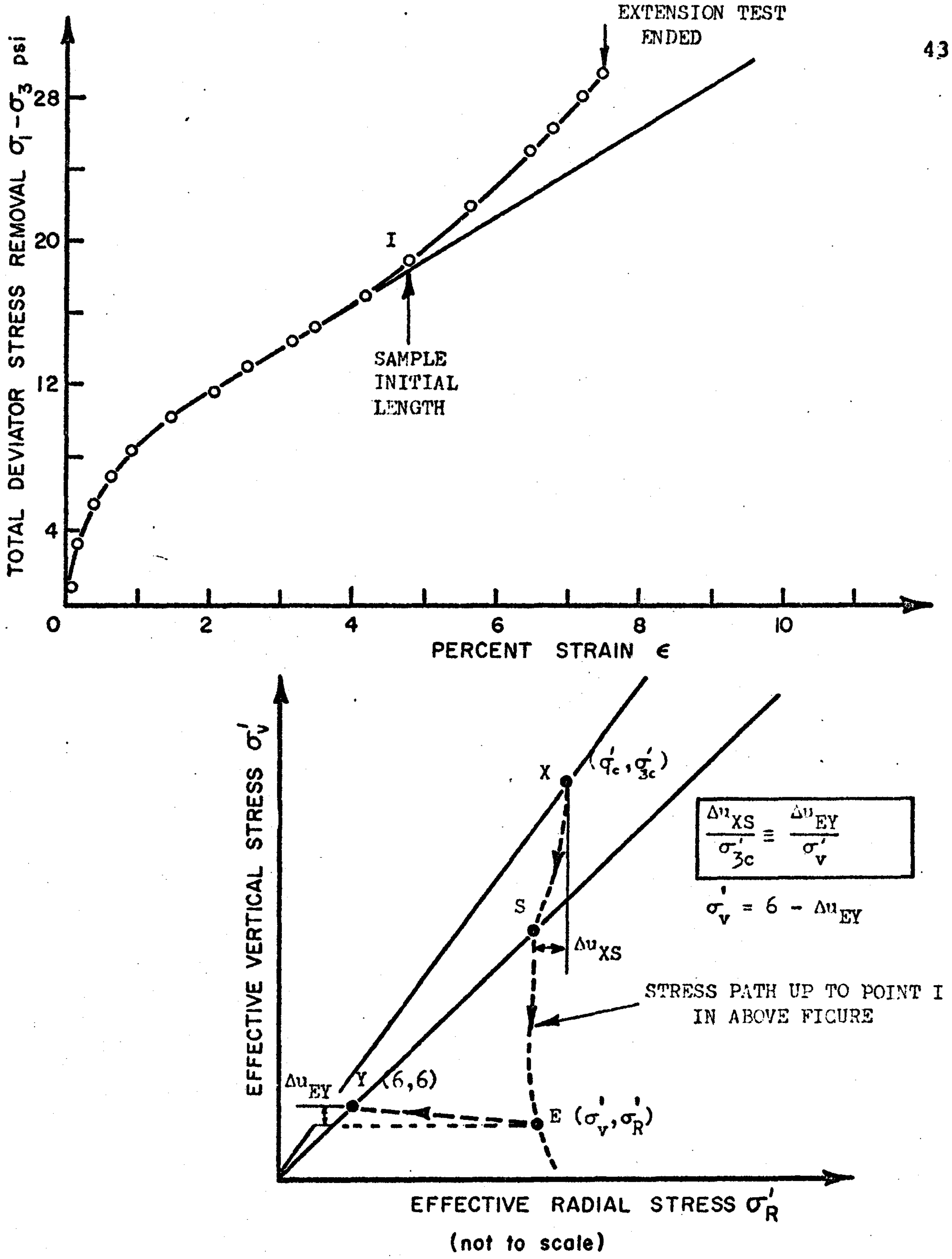


FIG. 10. EXTENSION TEST

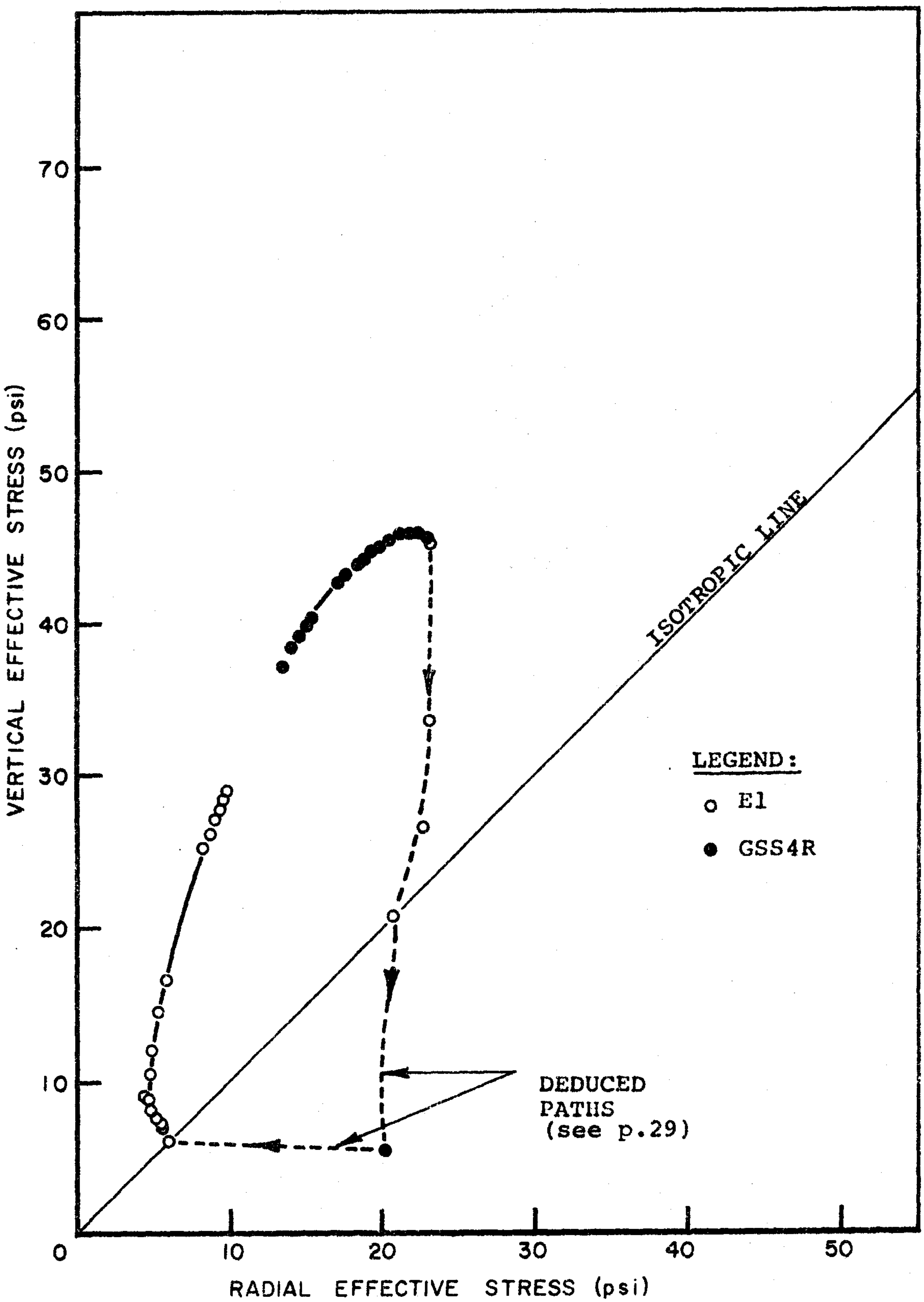


FIG. 11. EFFECTIVE STRESS PATH OF "EXTENSION" and "GROUND" SAMPLES.

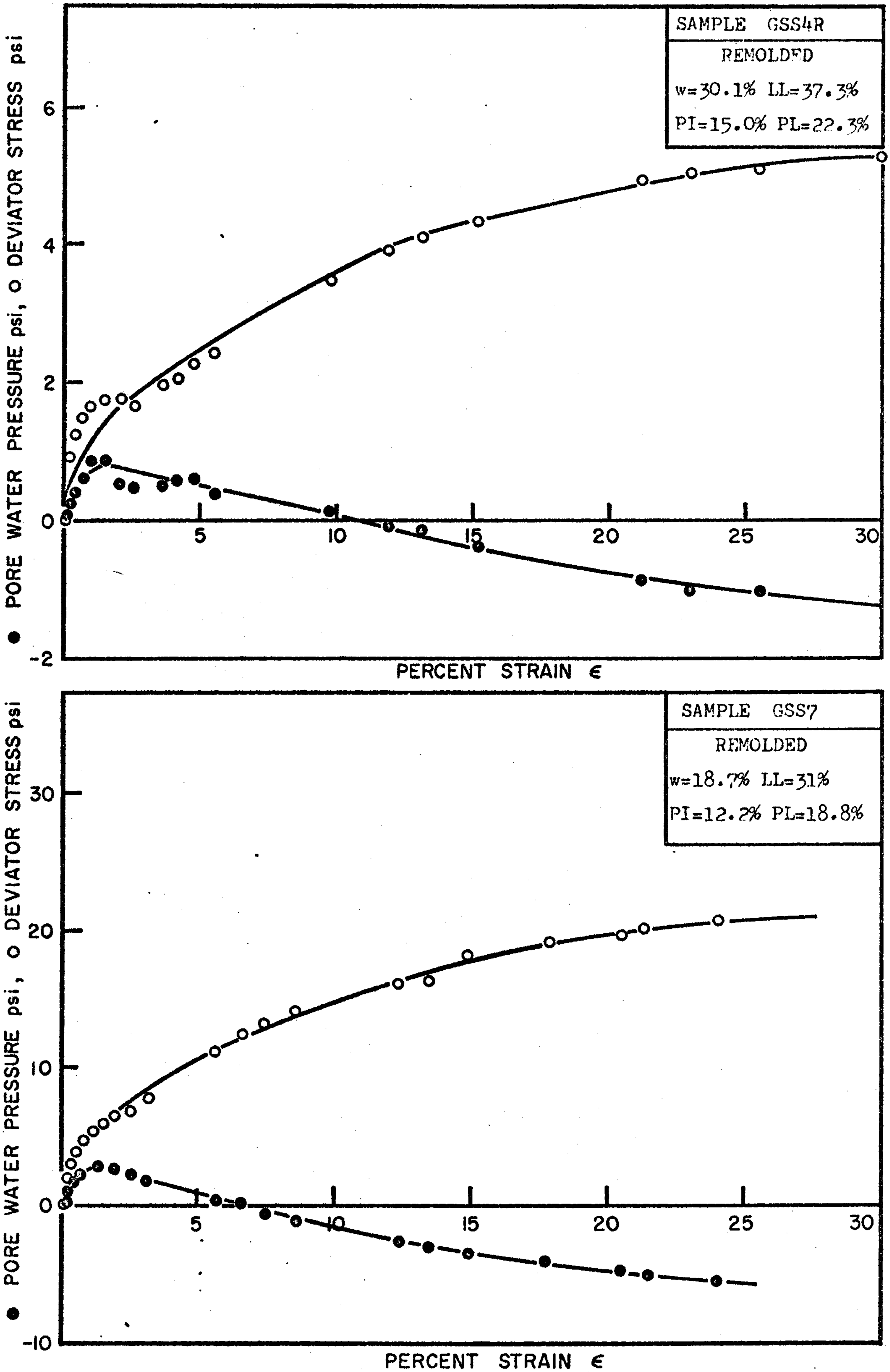


FIG. 12 STRESS-STRAIN RELATIONSHIPS-REMOLDED SAMPLES-

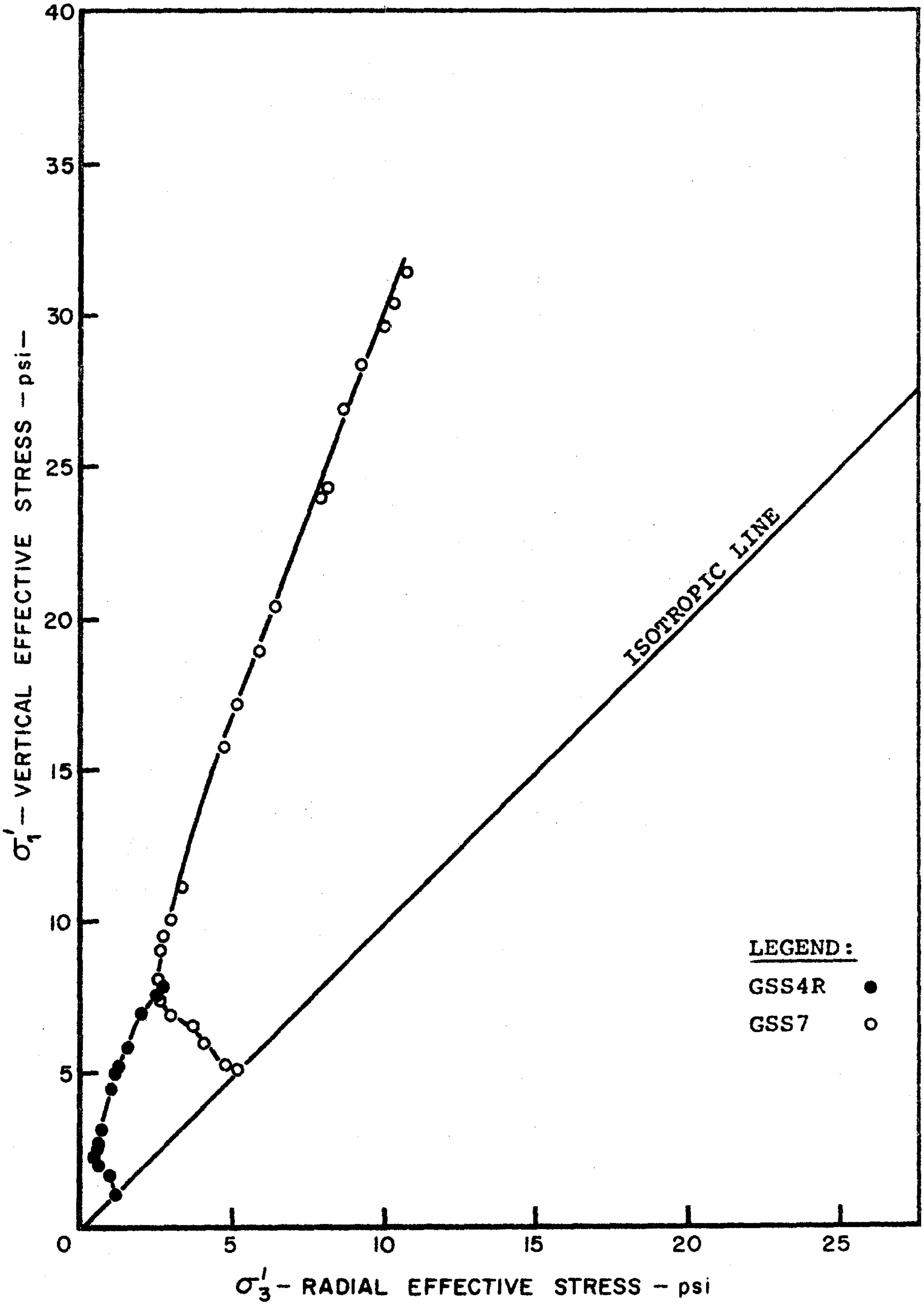


FIG. 13. EFFECTIVE STRESS PATH OF REMOLDED SAMPLES.

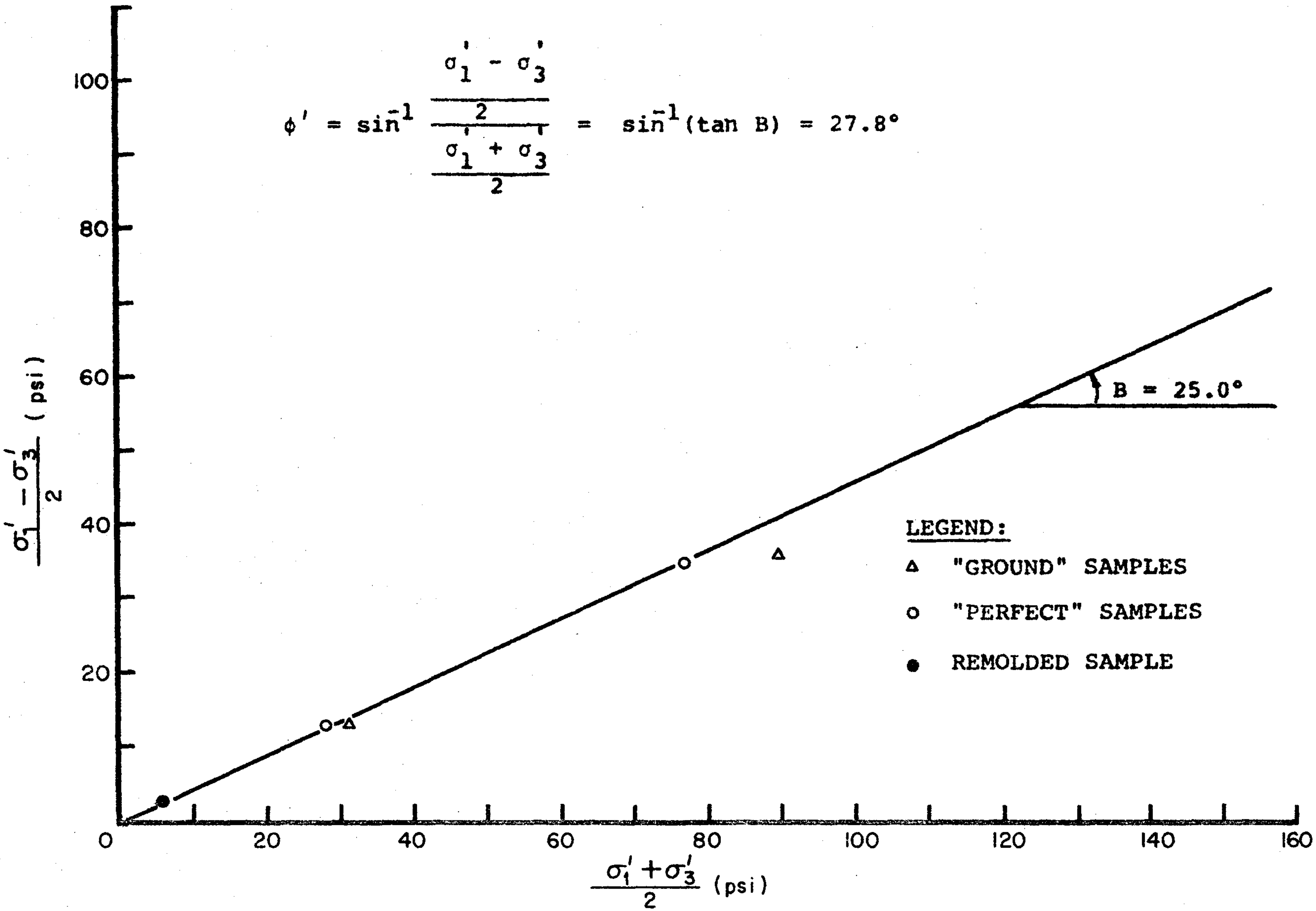


FIG. 14. FAILURE ENVELOPE OF "SILTY CLAY" at $(\sigma_1 - \sigma_3)$max .

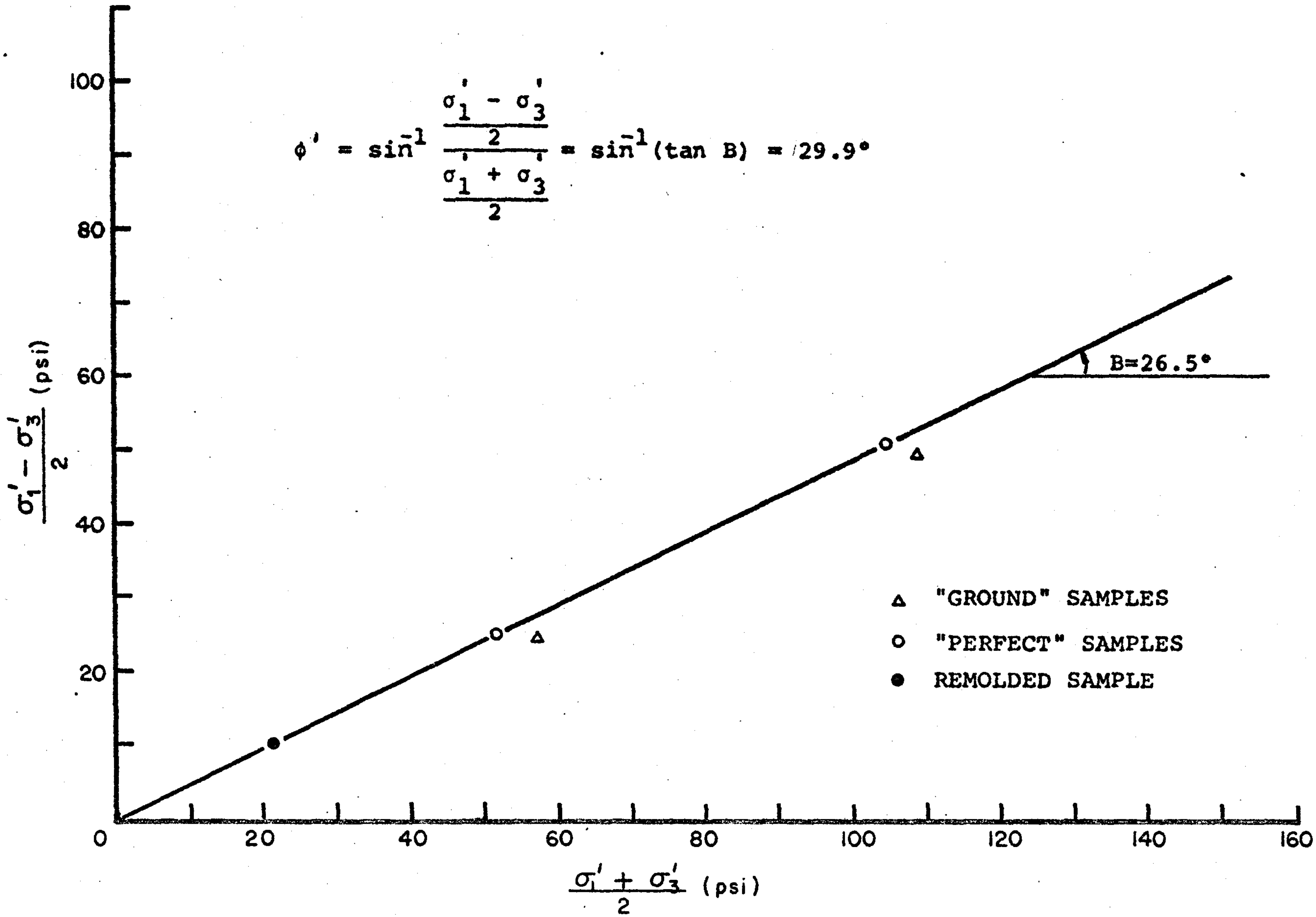


FIG. 15. FAILURE ENVELOPE OF "CLAYEY SILT" AT $(\sigma_1 - \sigma_3)_{max}$.

CHAPTER 3

SAMPLING AND PRECONSOLIDATION LOAD OF A SILTY CLAY

3.1 INTRODUCTION

As suggested in Chapter 1, a number of questions related to sampling and preconsolidation load determination of clays will be investigated, using a natural silty clay.

The void ratio-pressure relationships obtained from tests, will be expressed in the conventional semi-log method (i.e. void ratio versus log vertical pressure).

In summary, the aspects to be investigated are:

(a) The applicability of the currently used graphical methods of preconsolidation load determination (i.e. Casagrande [1936], Terzaghi & Peck [1948] and Schmertmann [1953]) by means of a triaxial K_o consolidation test of a "perfect" normally consolidated natural silty clay sample.

(b) The influence of load loss in oedometers on the preconsolidation load, as given by the results of triaxial K_o and oedometer consolidation tests on undisturbed samples of a natural silty clay.

(c) The influence of borehole bottom failure on preconsolidation load, as determined by the results of oedometer consolidation tests on remolded and undisturbed samples of a natural silty clay (Rutledge, 1944).

3.2 MATERIAL

Samples obtained from Hamilton Bay, as described in 2.2, were used for these investigations.

The two samples for the "perfect" sampling investigation were obtained from the top layer of tube #II (same layer as GSS5RII, see Figure 1). This was a "clayey silt" of PI = 11.7% and natural water content = 25%.

The material for the oedometer tests came from the top 1 inch of the bottom layer in tube #1 (same layer as GSS7) and so it was also of the "clayey silt" type. In this case the PI = 12.2% and the natural water content was 26.7%.

3.3 SAMPLE PREPARATION

The triaxial samples for the "perfect" sampling investigation, were prepared in the manner described in 2.3.

The preparation of oedometer samples was as follows. A one inch thick layer was cut, using a fine wire saw, from the top of an extruded section of the material, sealed between two flat discs and stored in the humid room.

Trimming of the undisturbed specimen to the appropriate oedometer ring size (see 3.4) was accomplished by using flat end discs for the lathe, carefully machined to the size of internal diameter of the rings.

The obtained sample was carefully fitted into the ring and excess material was trimmed off the top and bottom of the ring.

The remnants were remolded and another sample of this material was placed into another ring, taking care to avoid air pockets in this process.

3.4 APPARATUS

The triaxial test apparatus used in the "perfect" sampling investigation has already been described in 2.4.

The oedometer consolidation test equipment is shown in Figure 16. This apparatus was of the fixed-ring container type (see T.W. Lambe [1965]), with a lever system for vertical load application.

The oedometer rings were nickel plated and highly polished to diminish side friction effects. The rings were 2.434 inches internal diameter by 0.70 inches in height.

Drainage for these samples was provided by the use of top and bottom porous stones, while the rigid metal ring was used for maintaining the "no lateral yield" condition (i.e. K_o condition) during consolidation.

An accurate dial gauge (0.0001 inches accuracy) was used for measuring the vertical deformation of the samples.

3.5 EXPERIMENTAL PROCEDURE

3.5 (a) Mounting of Samples

Samples for the "perfect" sampling investigation were mounted as described in 2.5 (a). Mounting of the oedometer test samples went as follows. The prepared samples, which had been placed in the rings, were now seated on the bottom porous stone inside the container and a top porous stone was subsequently added. The lever arm was adjusted using the counterweight at the end, so that no initial load was applied to the sample, and sat on the polished steel sphere through which load was transmitted to an end platten and to the porous stone on top of the sample. The dial gauge was set in position as shown in Figure 16 and the datum for settlements was recorded.

<u>3.5 (b) "Perfect" Sampling - Preconsolidation Load Tests</u>

The procedures followed in these tests were the same as described in 2.5 (b) and 2.5 (c), up to the end of the undrained "perfect" sampling cycle. The two specimens tested were K_o consolidated up to an effective radial stress of 23.0 psi, before sampling.

After this point, the usual K_o consolidation procedure was resumed using a first cell pressure increment approximately equal to the change in radial effective stress due to sampling. Subsequent increments of cell pressure were made double each previous increment, until enough points were obtained to define a recompression curve and a virgin curve (within the available range of cell pressures). Account was taken of the fact that a slight reduction in cross sectional area occurred due to "perfect" sampling, so that K_o conditions were controlled thereafter on the basis of this area.

3.5 (c) Oedometer Consolidation Tests

The procedure followed in the oedometer tests consisted, briefly, of adding weights every 24 hours to the carrier at the end of the lever arm, and recording vertical settlements at chosen time intervals. Water was added to the bath container immediately following application of the first load, to ensure full saturation of the samples throughout the test.

Each load increment added was equal to the weight already on the carrier. A maximum weight of 32 Kg. could be used with this set up. Hence the vertical consolidation pressures used were 1.78, 3.56, 7.12, 14.24, 28.48, 56.96 and 111.4 psi, as calculated using the measured lever arm ratio. The first pressure increment applied was approximately equal to the suction pressure typically obtained with the "clayey silt" samples.

3.6 RESULTS

The results of the investigation on the effect of "perfect" sampling on the preconsolidation load of a normally consolidated natural "clayey silt" are shown in Figures 17 and 18. These are the "e-log p" relationships obtained for the two samples tested.

The obtained recompression curves showed a small inclination and there was a gradual transition to the virgin line obtained after sampling.

The range of values tabled for the preconsolidation pressure, as predicted by the Casagrande method, corresponds to the range of choices for a point of maximum curvature in the recompression curves.

The minimum preconsolidation load value obtained by the Terzaghi & Peck method was the existing effective "overburden" pressure (i.e. the induced preconsolidation load value) since the upward extension of the obtained virgin curve goes through this point. The application of the Schmertmann method would have given the same result, since the procedure is the same as for Terzaghi & Peck's method in the case of normally consolidated clays.

The results of the investigation on the influence of load loss in oedometers on the preconsolidation load, are shown in Figure 19.

It is seen that the "e-log p" curve obtained for the triaxial K_o sample falls between the undisturbed and the remolded oedometer sample curves. A constant load loss is observed from a comparison of oedometer and triaxial K_o results for the undisturbed samples.

Preconsolidation load values predicted from triaxial K_o consolidation tests were 24% lower than those predicted from oedometer tests on undisturbed specimens of the same material.

The influence of borehole bottom failure (i.e. remolding) on preconsolidation load is illustrated by the results of the oedometer tests as seen in Figure 19.

The remolded curve was displaced downward from the curve for the undisturbed oedometer sample, as observed by Rutledge [1944]. However, the virgin line obtained for the remolded sample does not show a lesser slope than the virgin line for the undisturbed sample, as observed by Rutledge.

Remolding of the material lowered the preconsolidation load value by 59%, as given by the oedometer tests.

The Casagrande construction was used for these values since the natural material was lightly overconsolidated.

3.7 DISCUSSION OF RESULTS

The results of the "perfect" sampling investigation showed that the Casagrande method (1936) gave erroneously high values for the preconsolidation load of a normally consolidated silty clay. The Terzaghi and Peck method gave exact estimates of preconsolidation load.

The oedometer and triaxial K_o consolidation tests on the undisturbed samples of silty clay showed that load losses in oedometer rings account for a 24% difference in preconsolidation load. The results of Figure 19 also showed that friction in oedometer rings probably was a significant factor in observations made by Rutledge. No convergence of virgin lines was observed from these tests.

The pattern of load losses with the type of rings used here, is illustrated by the differences observed between "oedometer" and "triaxial K_o" curves. There seemed to be an approximately constant percent load loss at all pressure levels, within the range of the tests. It is possible that different load loss patterns developed with the brass rings used by Rutledge.

Leonards and Girault [1961], as previously mentioned, found that percent load losses decreased with increasing pressure, for brass and steel rings. Also, these losses were a function of strain rate (e.g. undisturbed $\neq$ remolded sample strain rates). Leonards and Girault subsequently recommended the use of greased, teflon lined consolidometers to avoid these problems.

The results also indicate that borehole bottom failure may lower preconsolidation load values for obtained samples, by as much as 59%.

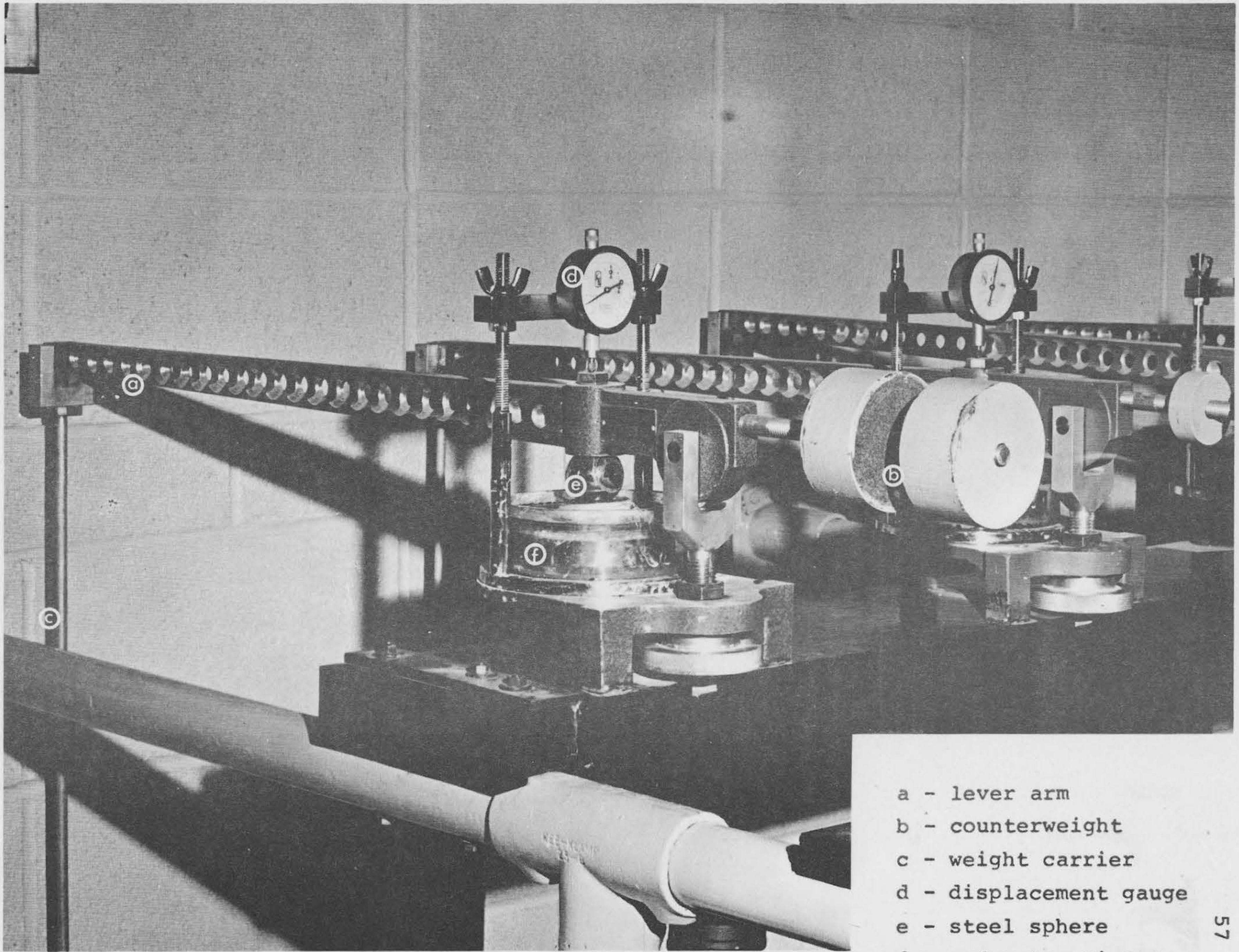


FIGURE 16 OEDOMETER TEST APPARATUS

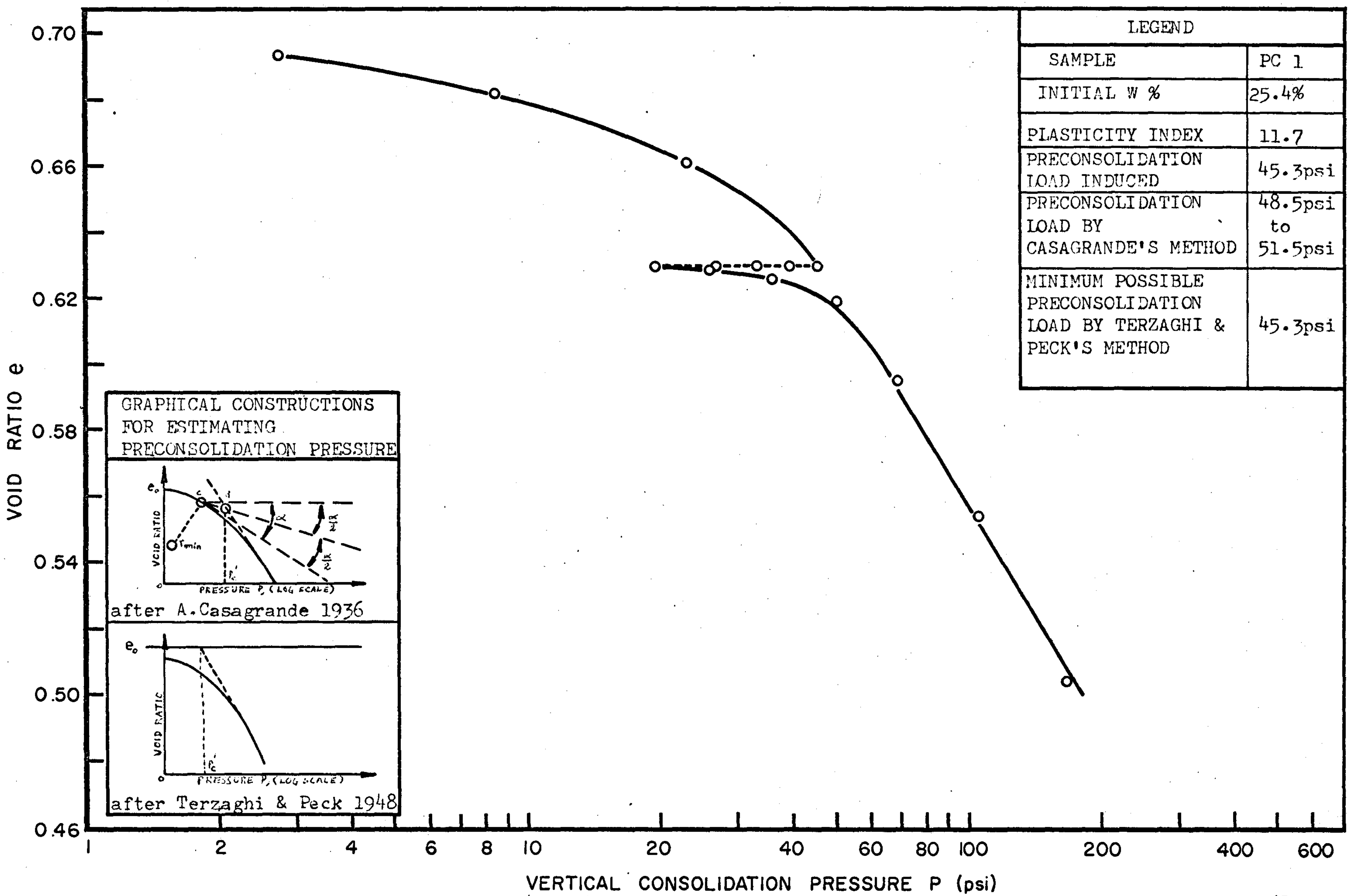


FIG. 17. EFFECT OF "PERFECT" SAMPLING CYCLE — TRIAXIAL K_o CONSOLIDATION—

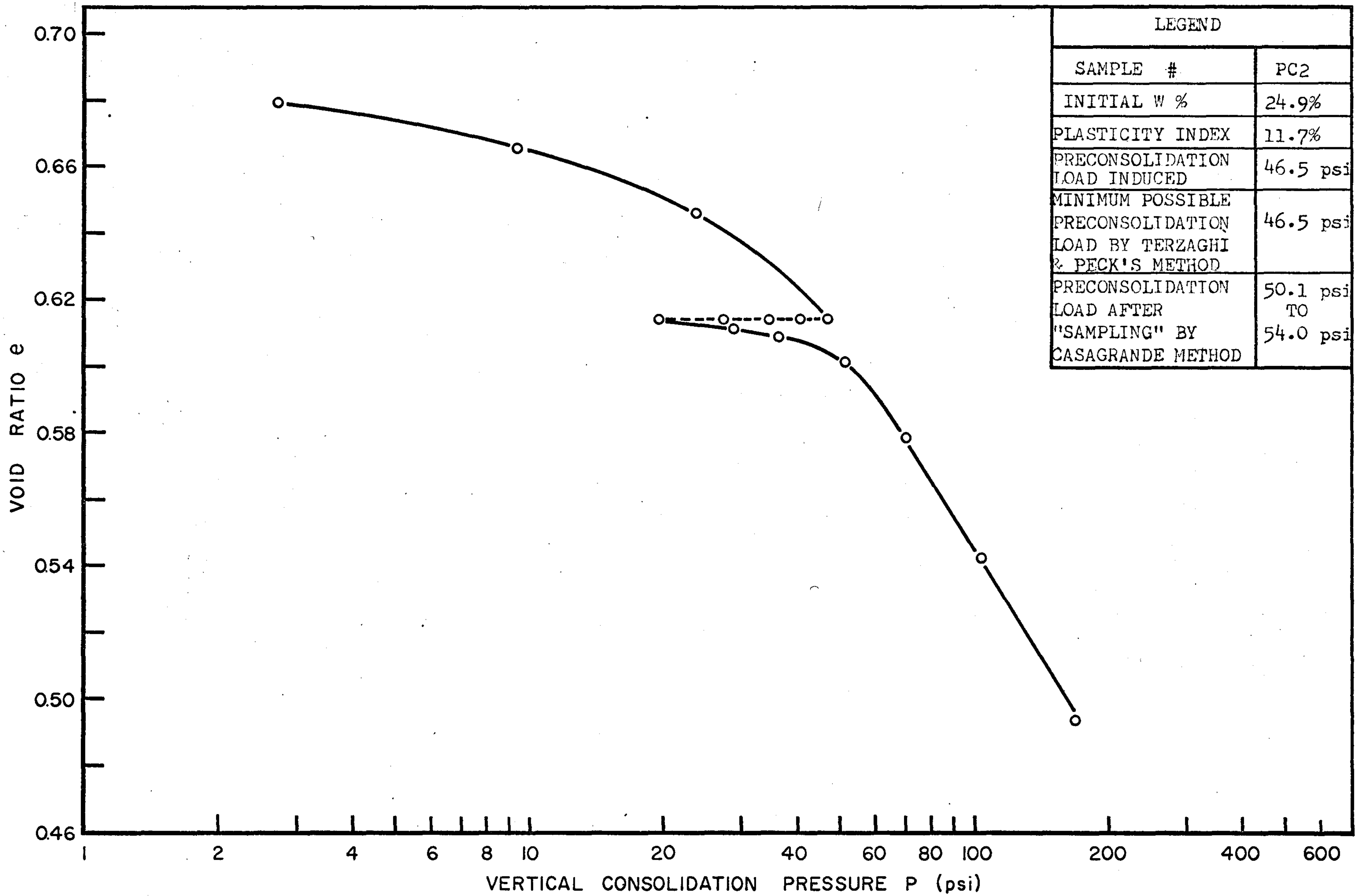


FIG. 18. EFFECT OF "PERFECT" SAMPLING CYCLE —TRIAXIAL K_o CONSOLIDATION —

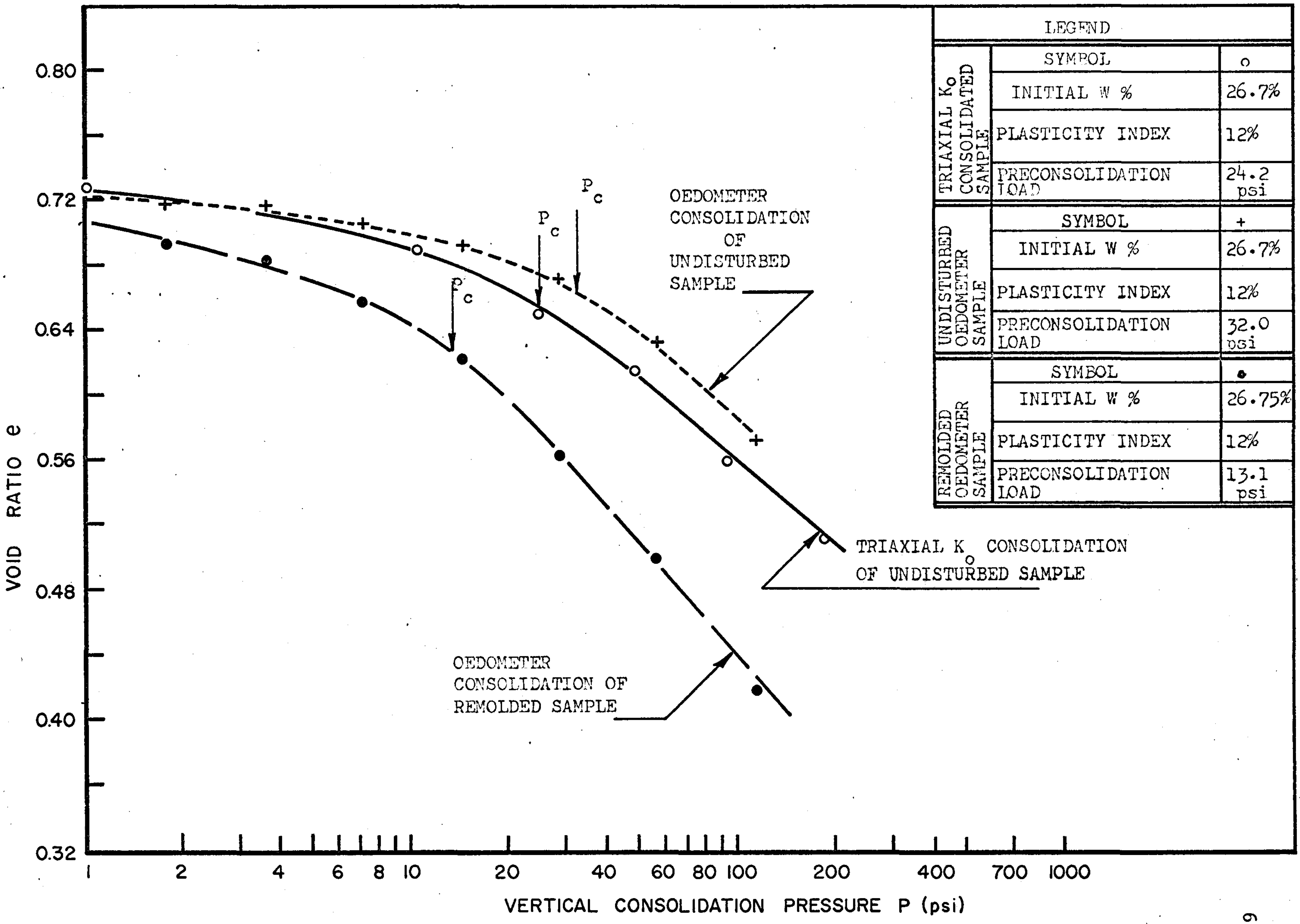


FIG. 19 . FACTORS INFLUENCING PRECONSOLIDATION LOAD DETERMINATION.

CHAPTER 4

THE EFFECTS OF SAMPLING A NATURAL SILTY CLAY

4.1 LIMITATIONS OF LABORATORY SAMPLING STUDIES

The results of laboratory sampling studies of natural clays may be of limited applicability to the actual sampling situations.

Firstly, disturbance of the natural soil prior to laboratory testing reduces its strength and consequently, this may influence the percentage difference observed between the simulated "ground" and "sampled" specimens in the laboratory.

Secondly, the consolidation rates used in the laboratory are radically different from field consolidation rates, which are measured in terms of years or even centuries. The very much faster rates used for the laboratory consolidation, most probably, results in soil structures which are very different from those in situ. Consequently, the behaviour of the laboratory samples is most probably very different from that in situ.

4.2 EFFECTS OF SAMPLING ON THE STRENGTH OF A NATURAL SILTY CLAY

4.2 (a) "Perfect" Sampling

From the results of the laboratory investigation on "perfect" sampling of a normally consolidated natural silty clay, shown in Figures 5 to 9 and Table I, it appears that the removal of the total stresses has little effect on the strength.

The "ground" samples and the "perfect" samples tested in undrained compression at the lower water contents, gave the lowest percentage difference in strength. It is thought that this trend indicates that for a natural silty clay the influence of "perfect" sampling on the strength decreases as the water content decreases.

It is thought that the 15% differences reported by Ladd and Bailey [1964], for "perfect" sampling of natural Kawasaki clay and remolded Boston Blue clay are questionable, in that they did not compensate for difference in rates of strain for "ground" and "perfect" samples (see III.2 (c)). The shape of the "ground" sample stress paths obtained by Ladd and Bailey indicates that the testing rate used was too fast to allow 25% pore pressure equalization for most of the stress path (see Figure 25).

<u>4.2 (b) Tube Sampling</u>

The approach suggested by S.W. Smotrych for the problem of tube sampling, that samples deform corresponding to extension during sampling, is based on the assumption that some of the soil displaced by the insertion of the sampling tube into the bottom of a borehole enters the sampling tube.

Failure in extension may or may not occur, depending on the type of soil (e.g. sensitive, brittle, etc.) and on the sampling tube used (i.e. area ratio) (Hvorslev, 1949). The tube sampling simulation by extension to failure is then applicable only to those cases where failure does take place. It is interesting to note that the 38% strength loss obtained for the "extension to failure" sample (see Figures 10 and 11 and Table I) is in agreement with the 20 to 50% losses reported by Ladd and Lambe [1963] for tube samples of various clay soils.

4.2 (c) Borehole Bottom Failure

The bottom failure of a borehole leading to remolding of the material being sampled was thought to be a possible occurrence for sensitive silty clay deposits.

In this context, the laboratory comparison, as suggested by S.W. Smotrych, with remolded and undisturbed material presents a realistic degree of strength loss to be expected.

A 400% strength loss was obtained, which reflects the "sensitivity" of the natural silty clay tested (see Table I and Figures 12 to 14).

4.3 EFFECTS OF SAMPLING ON THE PRECONSOLIDATION LOAD OF A NATURAL SILTY CLAY

4.3 (a) "Perfect" Sampling

The Casagrande [1936] method, for determining preconsolidation load, was shown not to be applicable for "perfect" samples of normally consolidated silty clays (see Figures 17 & 18). Casagrande's method gave overestimates of 6 to 16% for the preconsolidation load.

Terzaghi and Peck's [1948] method gave an exact estimate of the preconsolidation load for "perfect" samples of normally consolidated natural silty clay (see Figures 17 & 18).

4.3 (b) Oedometer Determination

It will be seen from the results in Figure 19, that the oedometer test produced a 24% overestimate of preconsolidation load, with respect to the value obtained with a triaxial K_o consolidation test.

Leonards and Girault [1961] have reported that the percentage load losses decrease with increasing pressure, for steel and brass rings. A constant load loss of 35% was observed at all pressures from a comparison of the oedometer and triaxial K_o consolidation tests on undisturbed samples of silty clay (see Figure 19).

4.3 (c) Borehole Bottom Failure

The results of the oedometer tests on the undisturbed and remolded silty clay show a 59% decrease in "in-situ" preconsolidation load as would be obtained for samples from a borehole with bottom failure. (see Figure 19).

4.3 (d) Rutledge's Observations

The results of oedometer tests on undisturbed and remolded samples of natural silty clay did not confirm Rutledge's (1944) observation that virgin lines for undisturbed and remolded samples converge (see Figure 19). Within the range of testing pressures, the obtained virgin lines were essentially parallel.

It is thought that perhaps the load loss patterns were different from those which existed for Rutledge's tests using brass rings.

4.4 CONCLUSIONS

1) Laboratory "perfect" sampling lowers the undrained shear strength of natural silty clays by 4-1/2% at the most (see Table I and Figures 5 to 9).

2) The extension to failure test, simulating tube sampling of natural silty clays, lowered undrained strength by 36% (see Table I and Figure 11).

3) Samples of natural silty clays from borehole with bottom failure may have the undrained compression strength reduced by as much as 400% (See Table I and Figures 12 and 13).

4) The Casagrande [1936] method overestimates the preconsolidation load of "perfect" samples of normally consolidated natural silty clay by 6 to 16% (see Figures 17 and 18).

5) The virgin lines of the "e-log p" curves obtained for the undisturbed and remolded silty clay, within the range of testing pressures used, did not converge as observed by Rutledge [1944](see Figure 19).
6) Oedometer tests overestimate the preconsolidation load of natural silty clays by 24%, as compared with triaxial K_o consolidation tests (see Figure 19).
7) Bottom of borehole failure in a silty clay would reduce the preconsolidation load by 59%, as shown by the results of oedometer tests on undisturbed and remolded samples of a a natural silty clay (see Figure 19).

APPENDIX I

CORRECTIONS FOR STRENGTH OF RUBBER MEMBRANE AND FILTER PAPER DRAIN

I.1 RUBBER MEMBRANE CORRECTION

In order to apply the external hydraulic pressures onto the triaxial sample, and produce at the same time the desired internal stress and water content conditions, a thin rubber membrane is generally used to enclose the sample.

The rubber membrane, however, imposes a restraint on the specimen.

I.1 (a) Henkel and Gilbert Method

Experiments were carried out by Henkel and Gilbert (1952) to investigate the effect of the membrane on the measured strength of triaxial specimens of 1-1/2 inches in diameter. These consisted of a comparison between the undrained strengths of remolded samples measured with and without a rubber membrane. A theory for calculating the correction from the properties of the rubber membrane was developed which gave results in substantial agreement with the measured values (Bishop and Henkel [1962]).

This theory assumed that the sample deformed as a right cylinder, and so, assuming a Poisson's ratio for rubber of 0.5, axial deformation produced compensating radial expansion so that the membrane exerted no confining stress on the sample.

The information required for the use of the theory was obtained through an "extension modulus test" (Bishop and Henkel [1962]) on a 1 inch strip of the membrane used in each case.

Henkel and Gilbert [1952] reported a value of extension modulus, M, of 0.80 lb/in. for a commercial type of "thin" membrane of 0.008 inch thickness.

I.1 (b) Tests and Results

The membranes used in the present investigation (prophylactics) had a measured thickness of 0.008 inches, however it was considered desirable that modulus tests be done here to account for possible differences in the types of rubber.

Figure 20 shows a schematic diagram of the arrangement used for the test.

It was found that the prophylactics used had an extension modulus of 0.45 lb/in. from two tests, the results of which are shown in Figure 20.

It should be noted that the results shown in the above figure are accurate within the range of application of corrections (i.e. up to 20% strain). Deviations from the straight line obtained in these tests after this point, are explained by the fact that "necking" in the rubber strip occurs due to friction at the glass rods (see Figure 20).

The computed corrections (see Figure 22) are then valid under the previous assumption that the sample remains straight (i.e. no bulging) up to failure.

I.1 (c) Conclusions

Duncan and Seed [1967] have produced a theory for membrane corrections, taking bulging into account (e.g. after failure). As reflected by the much lower extension modulus (0.45 lb/in.) of the prophylactics as compared to that of the membranes tested by Henkel and Gilbert (M = 0.80 lb/in), the corrections obtained for the former type (see Figure 22) are so small as to make such a refinement (i.e. corrections for bulging) impractical in this investigation. Consequently, the membrane strength corrections developed are applied only to measured deviator stress (i.e. membrane assumed to exert no radial stress on the specimen).

Two membranes with a smear of silicone grease between them were used to minimize leakage (see Appendix II). It was observed, however, that the outside membrane buckled immediately following the application of the deviator stress.

It is thought that the silicone grease acts as a lubricant, thus making the outer membrane ineffective for load carrying purposes. Hence the measured deviator stress is corrected only for the strength of the membrane which is in contact with the soil.

I.2 FILTER PAPER DRAIN CORRECTION

I.2 (a) Introduction

The use of side drains consisting of filter paper strips is recognized as being advantageous in triaxial testing of clays. The advantages provided are completion of consolidation and equalization of pore pressure in undrained shear stages in reasonable amounts of time (see Appendix III).

The consequence of using a filter paper drain is the restraint imposed on the specimen.

The effect of filter paper strength on measured deviator stress has been investigated by Bishop and Henkel [1962], for the case where 1-1/2 inch and 4 inch diameter samples are covered over half their surface area with Whatman's No. 54 paper (i.e. 1/4 inch wide strips at 1/4 inch intervals).

Their investigation consisted of testing 1-1/2 inch diameter specimens with and without the above type of drains. The correction obtained was a constant 1.5 psi, after buckling of the drains occurred at 2 to 3% strain.

I.2 (b) Filter Paper Drain Correction For Firm Samples

In the present investigation the samples were covered over their entire surface by side drains of Whatman's No. 54 paper. Vertical slits were cut at 1/4 inch intervals in order to reduce the resistance to applied vertical loads. This arrangement was suggested by Bishop and

Gibson [1963] to provide the maximum drainage surface area. As a first approximation the correction for this case of the fully covered specimen could be taken as 3.0 psi, that is, twice the Bishop and Henkel correction of 1.5 psi for the half covered case.

The general problem of corrections of strength for filter paper drains has been studied by Duncan and Seed [1967]. Their work followed that of Olson and Kiefer [1963] on Whatman's 1 and 50 paper.

The formula developed by Olson and Kiefer for triaxial test conditions is,

$$\Delta\sigma_{afp} = -K_{fp}\left(\frac{P}{A_s}\right) = \text{ Maximum Drain Strength Correction}$$

where: K_{fp} = load carried by filter paper covering a unit length of specimen perimeter

P = length of perimeter covered by filter paper

A_s = specimen x-sectional area

Duncan and Seed calculated the value of K_{fp} for Whatman's 54 paper using Bishop and Henkel's $\Delta\sigma_{afp}$ = 1.5 psi with p = 1/2. The value obtained was 0.19 kg/cm^2, which compared with a value of 0.13 Kgm/cm^2 they obtained experimentally for plane strain test conditions. They argued that the difference was reasonable since "the plane strain drains carry load only through column action", whereas under triaxial test conditions there exists a "confining effect".

The direct use of the "Olson and Keifer" formula gives for fully covered 1.4 inch diameter triaxial specimens a value of $\Delta\sigma_{afp} = 0.19 \left(\frac{2\pi R}{\pi R^2}\right) = 0.214$ Kg/cm^2 = 3.0 psi which is also the value obtained using the "first approximation" argument.

For the application of correction it was thought that the correction would increase linearly to 3.0 psi up to a strain of 2-1/2% at which buckling of the drains was noticed,during preliminary tests. The correction was taken as being constant after this point (see Figure 22). It should be noted, that both the membrane and the filter paper corrections were applied from the beginning of the K_o consolidation stages.

I.2 (c) Filter Paper Drain Correction for Soft Samples

The use of the preceeding corrections produced irregularities in the deviator stress versus strain results of tests on soft remolded material (samples GSS4R and GSS7, Chapter 2).

Bishop and Henkel [1962] noted that the filter strip correction for "cell pressures of less than 5 psi", should be reduced and could in fact change sign, since the filters might tend to buckle into the soil and cause premature failure. "Cell pressure" as used here by Bishop and Henkel was interpreted to mean "initial effective stress".

It was taken that at an effective stress of zero the filter strip correction for soft soils would be zero and that the correction would then increase proportionately to the full value of 3.0 psi at an initial effective stress of 5.0 psi.

Two undrained tests on remolded samples of the same material, one with and one without filter strips, were carried out to check these assumptions. The results are shown in Figure 21.

It will be seen that the correction at 29% strain is 0.32 psi. On the basis of the Bishop and Henkel criterion this correction should have been 0.25/5.0 X 3.0 = 0.15 psi.

For this soil, contrary to Bishop and Henkels experiment, the filter strip correction seems to be fully operative at initial effective stresses greater than 2.5 psi.

Corrections were then made accordingly for soft remolded samples GSS4R and GSS7.

I.2 (d) Filter Paper Drain Corrections in Extension Tests

Filter strips are much stronger in tension than in compression (buckling reduces the strength in compression to a very small value). Olson and Kiefer [1963] quote tensile strengths of 298 psi for soaked and 3570 psi for air dry Whatman's No. 50 paper.

The order of the correction would obviously be so great compared to the strength of the soil, that this then is an instance where the disadvantage of using filter paper far outweigh the advantages of their use.

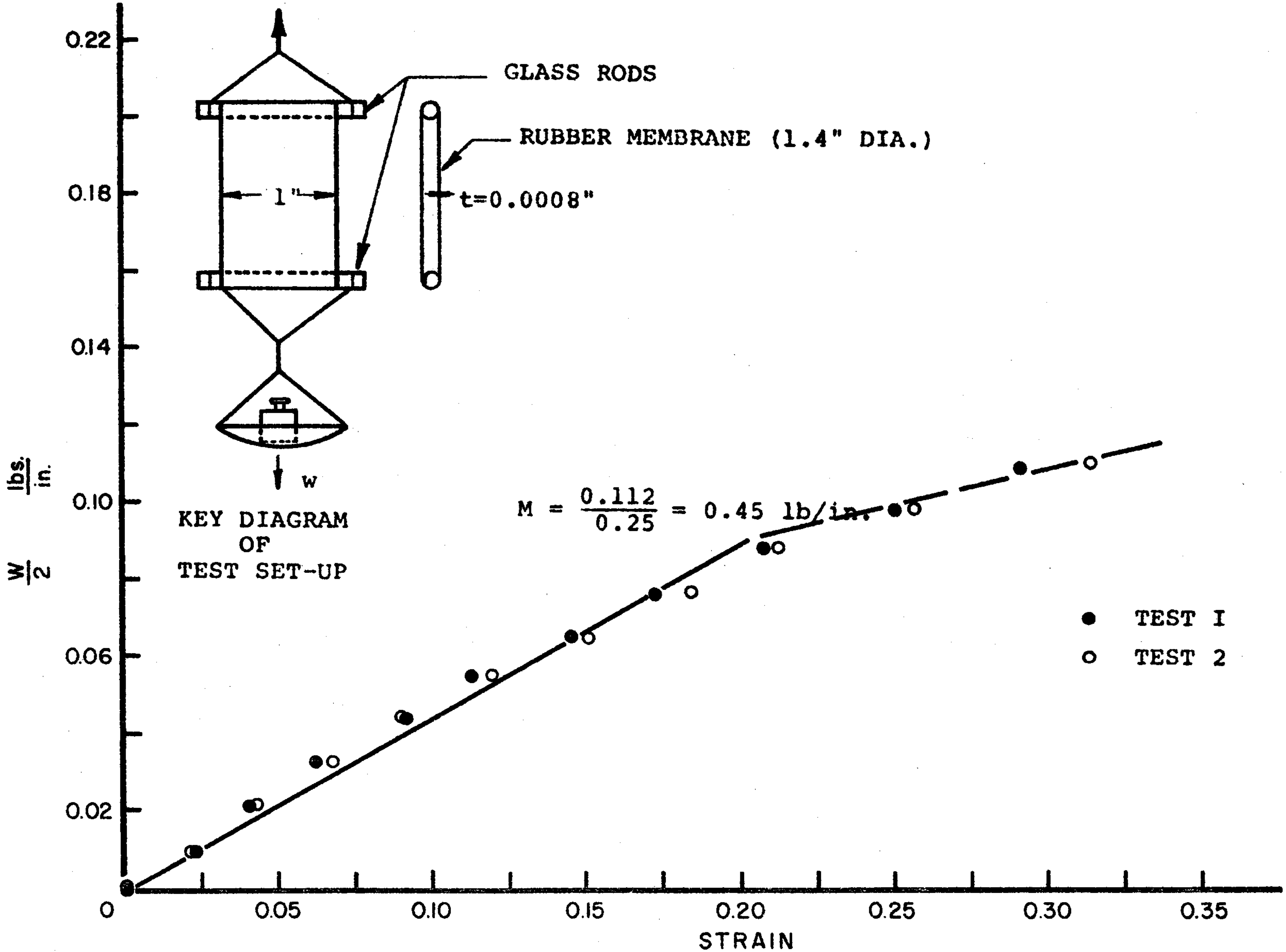


FIG. 20. MEMBRANE (PROPHYLACTIC) EXTENSION MODULUS TEST.

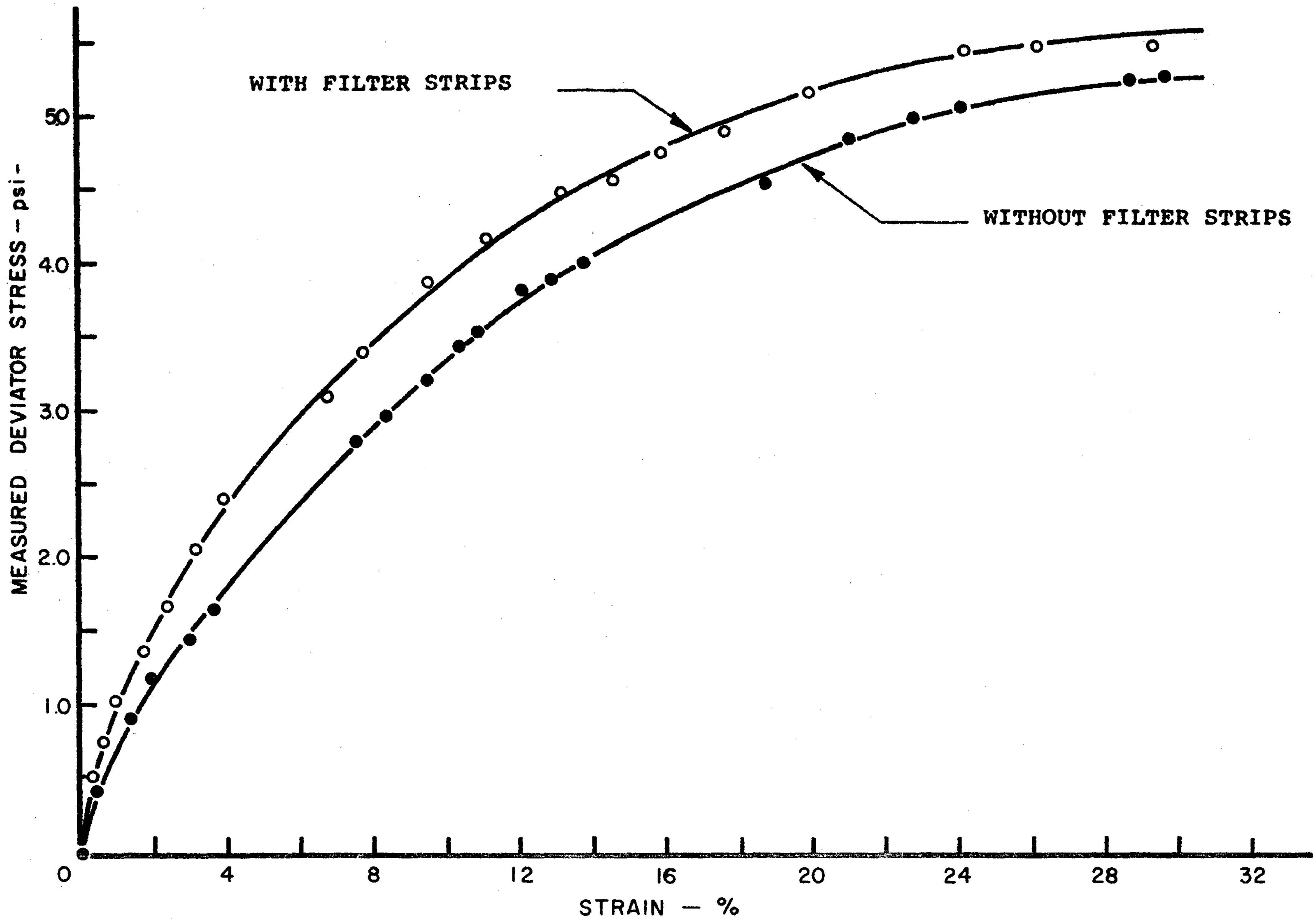


FIG. 21. STRESS-STRAIN CURVES FOR SOFT SOILS — WITH and WITHOUT FILTER STRIPS —

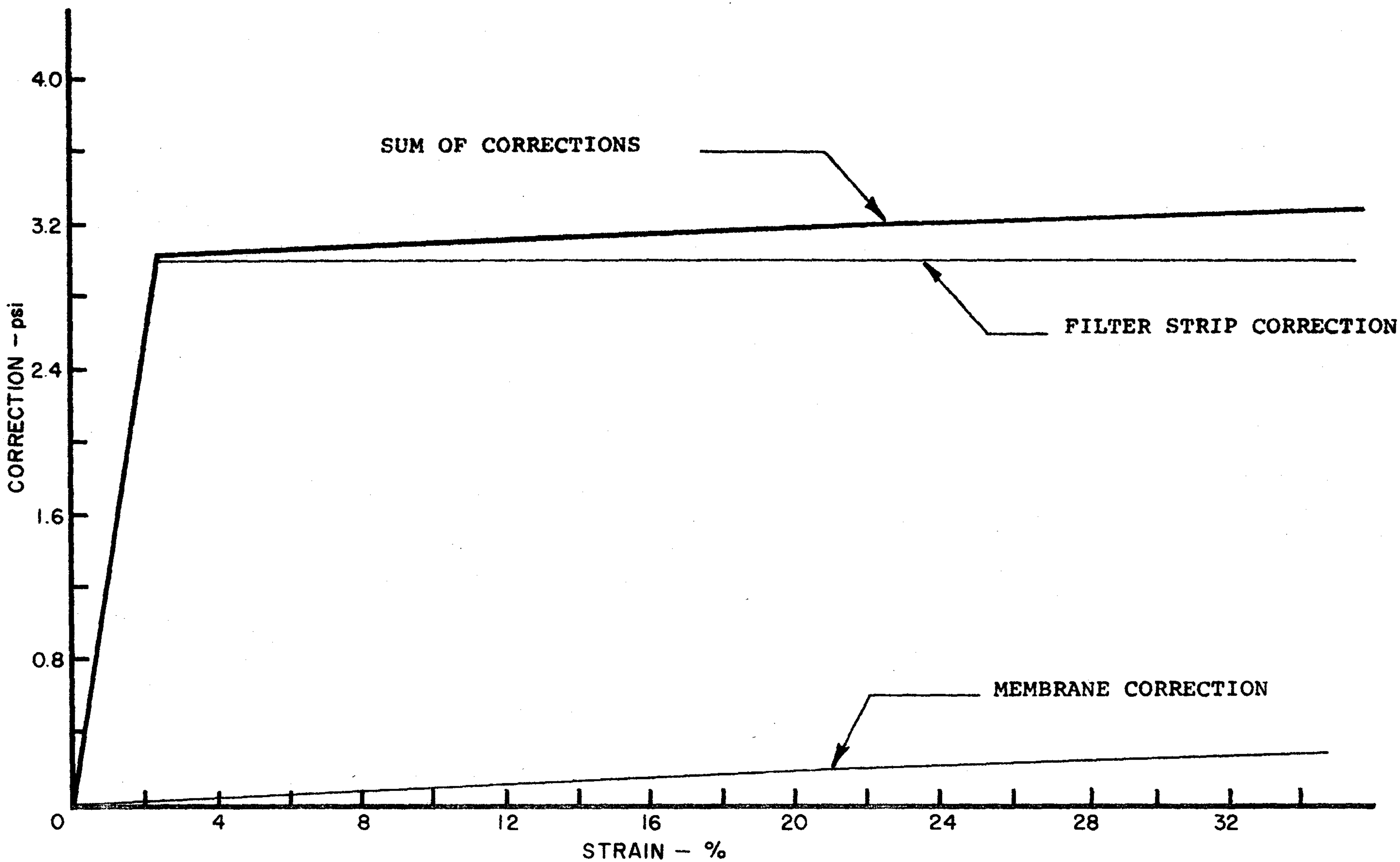


FIG. 22. DEVIATOR STRESS CORRECTIONS.

APPENDIX II

LEAKAGE THROUGH RUBBER MEMBRANES

II.1 INTRODUCTION

During the preliminary tests of triaxial samples enclosed by a single prophylactic, it was observed that pore pressures increased when drainage was closed following isotropic consolidation.

It was thought that this could be explained by the leakage of cell water through the membrane into the sample, due to the high pressure gradients existent across the membrane.

The problem of permeability of rubber membranes has been considered by Poulos [1964].

Bjerrum, Simmons and Torblaa [1958] reported the pore pressure increases with a single membrane, in a series of long term tests designed to investigate strain rate effects on shear strength of clays. They did not at the time find the cause of the problem. These and other observations must have contributed significantly to the subsequent use of two membranes with silicone grease (a vacuum sealant) smeared between them, in the assembly of triaxial test samples in Norway.

II.2 TESTS AND RESULTS

For the purpose of this research it was only required that a membrane be found which reduced the leakage, to a degree where the effect of leakage could not be detected for the period of the undrained triaxial strength test.

The tests consisted of enclosing a stack, 2-1/8" high, of 1.4" diameter porous stones with the various membranes and measuring the pore pressure increase with time for each case. High cell pressures were used as well as an initial back pressure, which were allowed to act overnight to ensure full saturation of the porous stones. Measurements of pore pressure commenced in the morning when drainage was closed.

The membranes tested included a single commercial type membrane, a single prophylactic and a double prophylactic with silicone grease. The results are shown in Figure 23.

The "double membrane with silicone grease" proved to give the lowest pore pressure increase with time for this rigid and highly responsive system.

Readings of air temperature were also made. This was due to a drop in the initial pore pressure which was tentatively attributed to an observed air temperature drop. The air temperatures were then fluctuated intentionally through use of the air conditioning system (see Figure 23). It will be seen that the pore pressures vary with the temperature. Over the length of the test a pore pressure increase is evident. With this rigid system even a very slight leakage would result in a significant pore pressure increase. Perhaps some of this pore pressure increase was also due to the cycling of temperatures as observed by Henkel and Sowa [1963].

It was then decided that the best and worst membranes would be tested with the soft system obtaining with a clay specimen. The temperature was kept constant to avoid the Henkel and Sowa effect, during this and subsequent series of tests.

The results are shown in Figure 24. It will be seen that with the single membrane pore pressures increased significantly.

With the double prophylactic with a smear of silicone grease there was no detectable increase in pore pressure. Consequently this was the membrane used in all undrained triaxial testing.

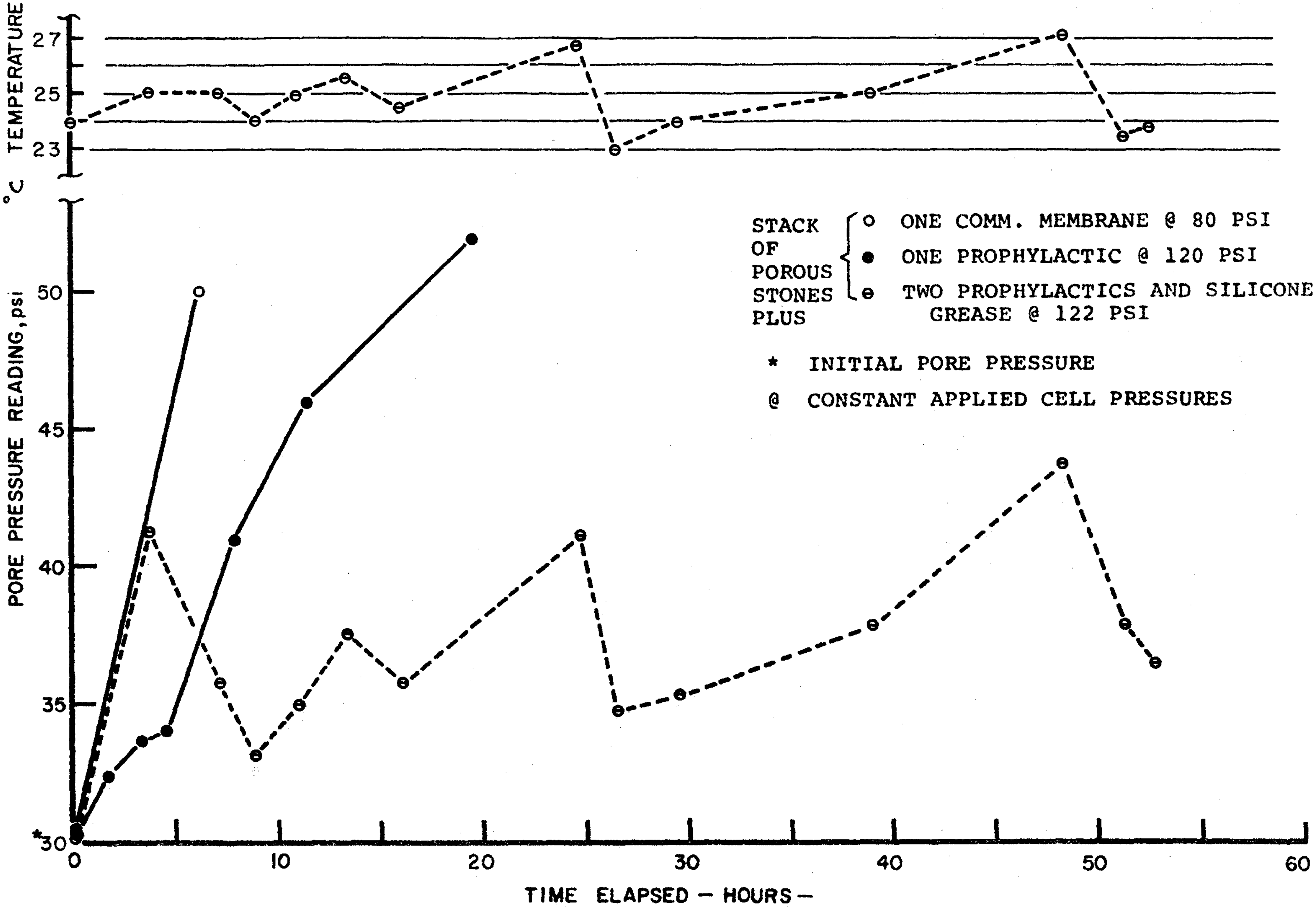


FIG. 23. OBSERVATIONS OF PORE PRESSURE CLIMB — with SATURATED POROUS STONES —

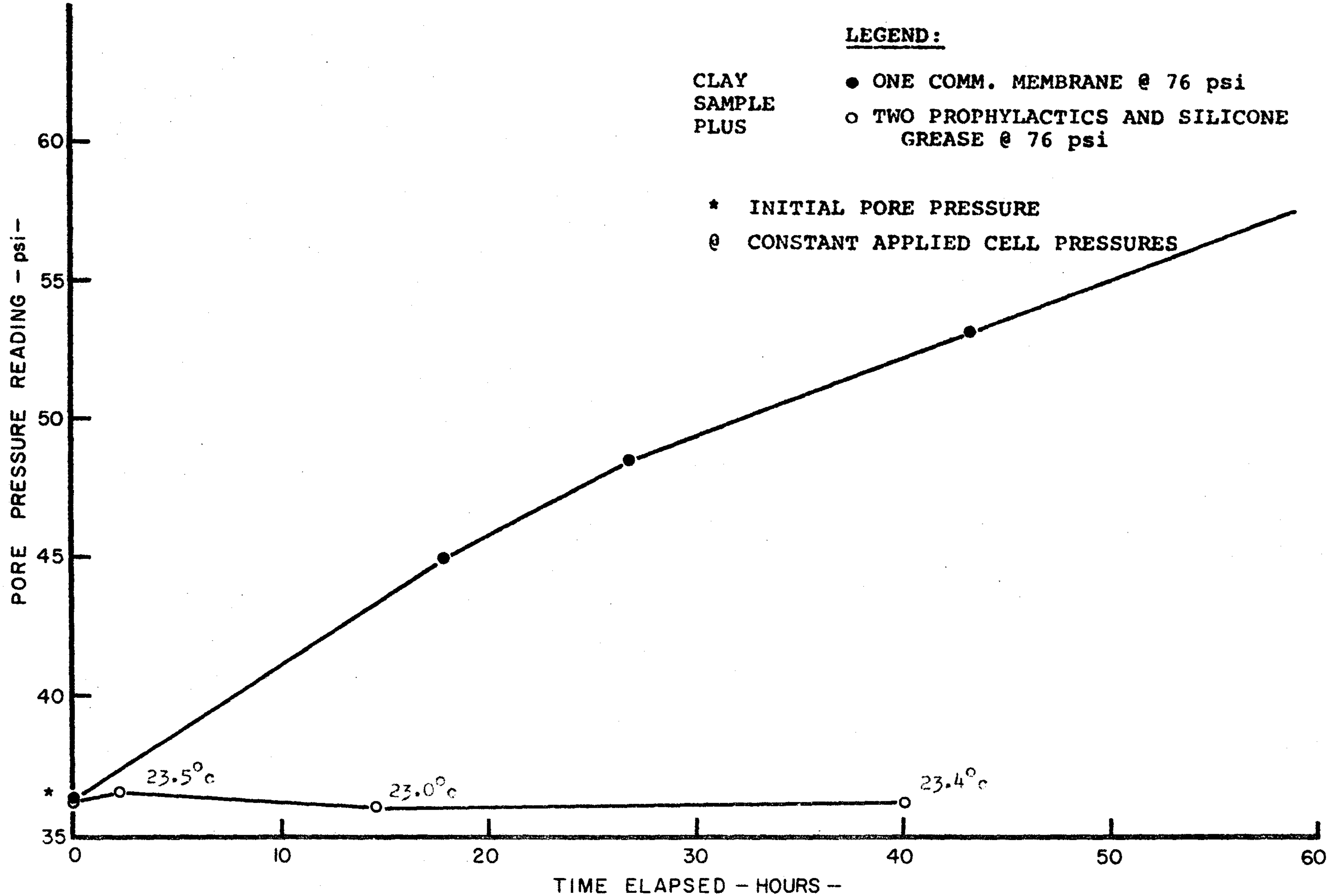


FIG. 24 . OBSERVATIONS OF PORE PRESSURE CLIMB - with a SOIL SPECIMEN -

APPENDIX III

TESTING RATES

III.1 INTRODUCTION

In the undrained strength testing of soils with a triaxial apparatus consideration has to be given to the selection of a suitable strain rate since testing rates have an influence on the values of strength and pore pressures measured (see Bishop and Henkel [1962]). There are a number of requirements which must be satisfied.

III.2 SELECTION OF RATES

III.2 (a) Time Lag in Pore Pressure Measuring System

For undrained tests with pore pressure measurements the time lag in the pore pressure measuring system has a marked influence on pore pressures observed during the undrained stage. For this reason some care must be exercised in selection of a testing rate compatible with the observed time lag.

A number of factors are recognized as having a decidedly benefic effect in accelerating the response of the pore pressure device (see Bishop and Henkel [1962]). The most significant of these is the volume compliance of the null system. The null used in the present investigation (see Chapter 2, Section 2.4), which was clamped on to the cell base through a very short and rigid arm, had a very small volume compliance.

It has been found that the use of filter paper side drains is very effective in reducing the response time for clay soils (see Gibson and Henkel [1954]).

In the present investigation drains were used consisting of Whatman's No. 54 filter paper (see Appendix I).

The time lag can also be minimized by the use of a back pressure high enough to drive all entrapped air into solution.

The check on the response time of the system consisted of a cell pressure versus pore pressure test in which pore pressure readings were taken against time for each cell pressure value (see Figure 4). The pore pressure readings reached asymtotic values in less than one minute. A back pressure of 30 psi was used. (See 2.5 (b))

Consequently the time lag in pore pressure readings had comparatively little influence on the selection of testing rates.

III.2 (b) Pore Pressure Equalization

It has been observed (Bishop and Henkel [1962]) that in undrained triaxial tests the non-uniformity of pore pressures results from non-uniform stress and strain conditions imposed by the end restraint on samples. Readjustment of pore pressures through migration of pore water is known as pore pressure equalization. In this investigation pore pressures were measured at the bottom of the sample. Hence, optimum conditions for pore pressure equalization were considered essential for reliable determination of effective stresses.

The rate at which pore pressure equalization occurs depends on the permeability of the sample, its dimensions, the rate of testing and the boundary conditions for drainage. With respect to the latter, the use of filter side drains is of considerable help in facilitating the migration of pore water to accelerate pore pressure equalization. Since porous stones were used both at the top and bottom of the specimen, optimum conditions* were established for the progress of equalization.

On the basis of work done by Gibson (see Bishop and Henkel [1962]), a strain rate was calculated using C_v (coefficient of consolidation) values of the system, as obtained from the last K_o consolidation stages. This calculated value satisfied the chosen criterion of 95% equalization for the first significant reading at 0.1% strain. The rate used for both samples was 0.0002 inches/min..

From these preliminary tests it was found that the values of maximum deviator stress did not correspond to the

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* See table of η values, Bishop and Henkel [1962]
Note that η = 40.39 instead of 35, for drainage from both ends and radial boundary, Bishop and Gibson [1963]

values of maximum principal effective stress ratio for the "ground" samples, but did for the "perfect" samples. It was noted that the proving ring compressed different amounts for the two samples, during the test, (see Appendix III.2 (c)) and that the strain at failure was greater for the "perfect" sample. It was thought, based on S.W. Smotrych's testing experience, that the difference in effective stress behaviour of the samples was a reflection of differences in pore pressure equalization due to the difference in rates of compression to failure. It was decided to lower both test rates to maintain compatibility and to gain the desired correspondence. The rates used were 0.000128"/min for the "perfect" sample and 0.00004"/min for the "ground" sample.

The results of this decision are best illustrated by Figures 25 and 26, which show how the difference in lag between pore pressure readings and deviator stress applied, for samples tested at two different rates, produced the expected shift in deviator stress peak to a point closer to the stress ratio peak.

Presumably, lowering both test rates still further would have produced the perfect correspondence in values sought. This was attempted for another "ground" sample using a rate of 0.000008"/min and this was still not achieved. It was considered that at such small rates, temperature fluctuation problems limited the experimental accuracy. The deviator stress strain curves tend to flatten out with lowering of strain rates obscuring the point were maximum deviation occurs.

III.2 (c) Rate Effects on Strength

The observation that different amounts of deformation went into the proving rings for each sample (i.e. "ground" and "perfect" sample respectively), showed that the resulting strain rates (i.e. sample deformation rates) were different for the pair. The deformation rate for the "ground" sample was about five times higher than the deformation rate for the "perfect" sample.

Taylor [1943] and Casagrande [1951] have both observed that lowering the rate of sample deformation has the effect of lowering the failure strength. A 5% decrease in strength of clay soils, can be expected for a tenfold decrease in deformation rate (Bishop and Henkel [1962]).

On the basis of these results, estimates were then made of the deviator stress at failure for each sample. Appropriate allowances were made for the difference in proving ring compression during testing of both samples.

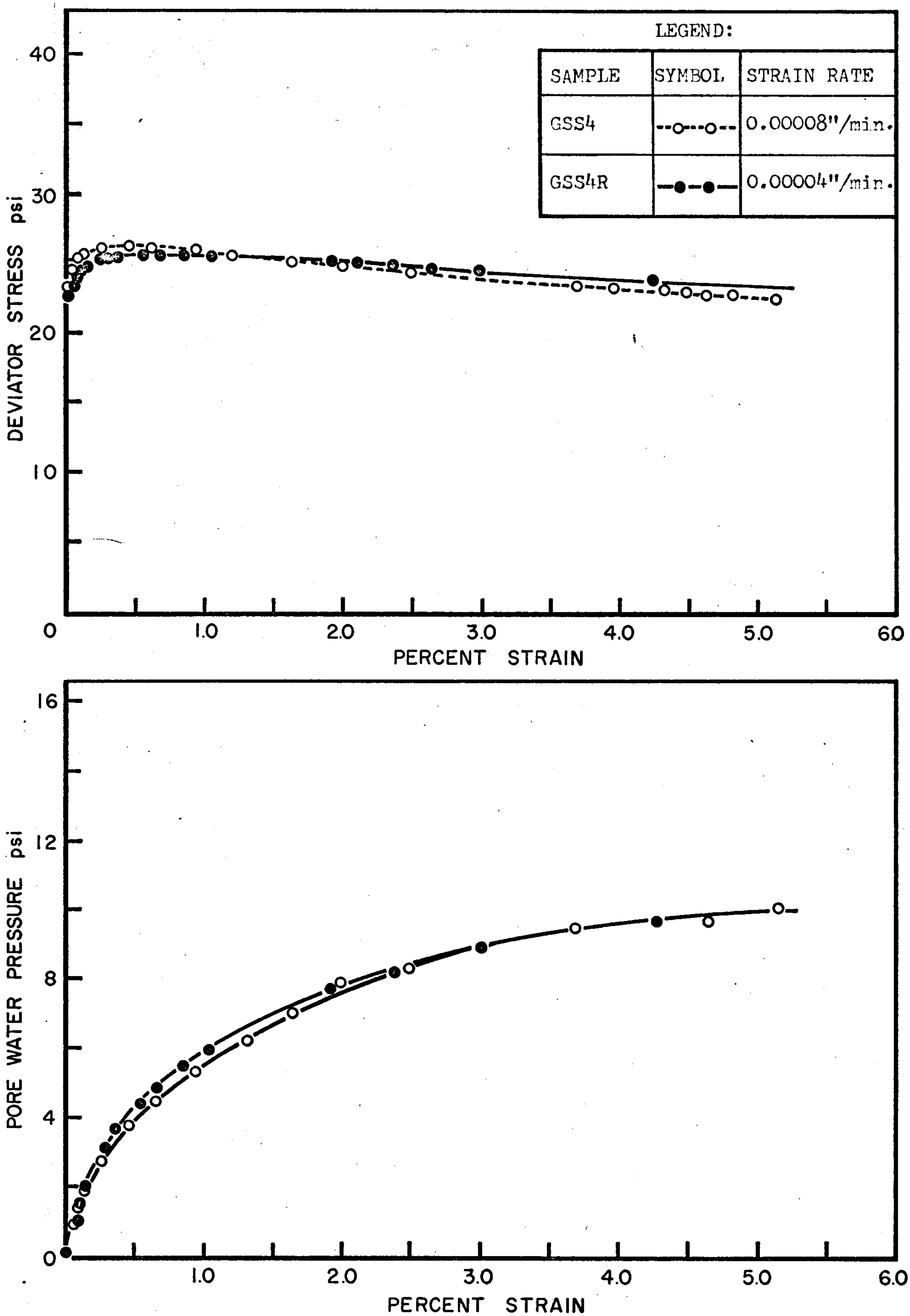

FIG. 25 . EFFECTS OF STRAIN RATE ON STRESS-STRAIN RELATIONS.

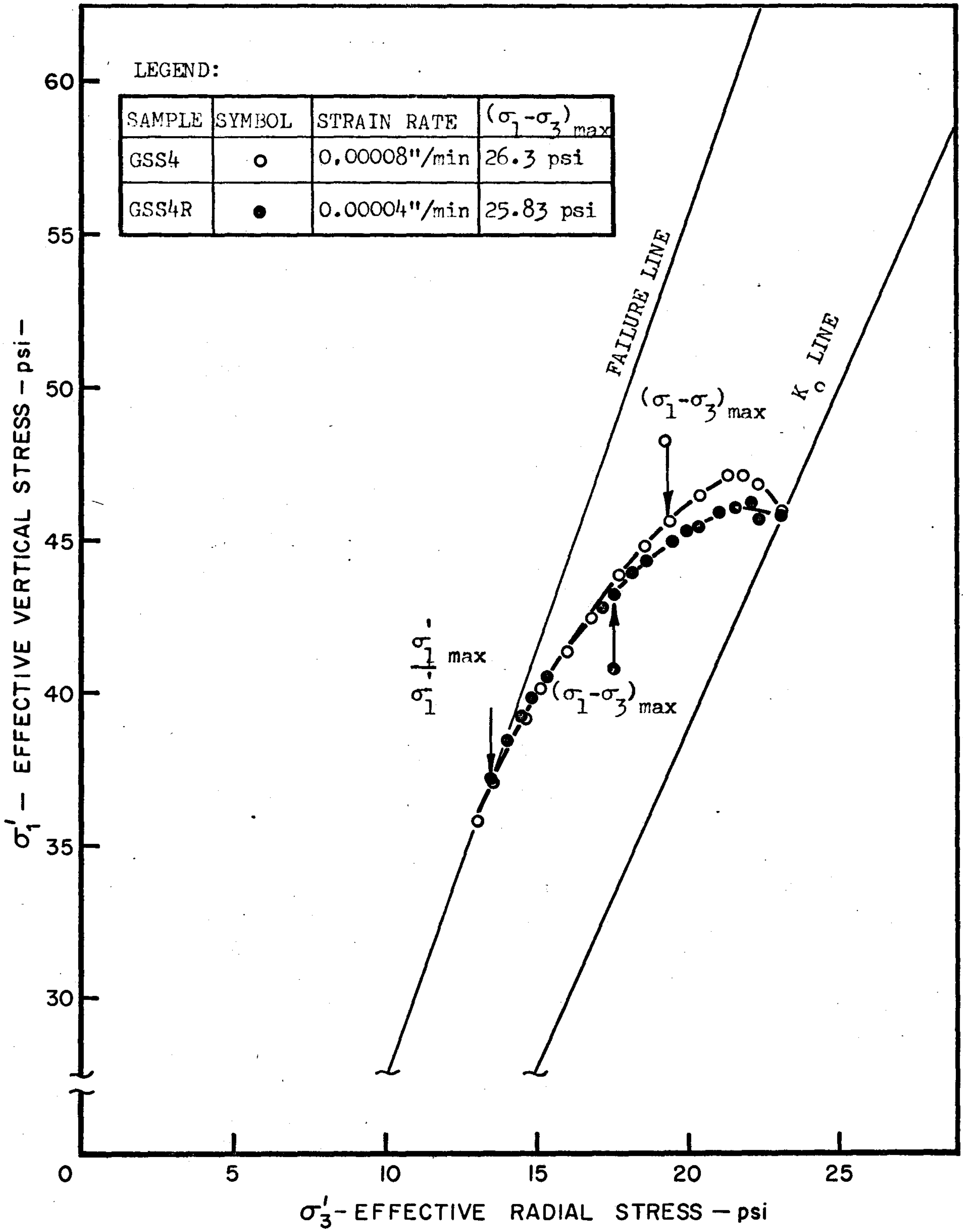

FIG. 26. THE EFFECT OF STRAIN RATE ON THE EFFECTIVE STRESS PATH.

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