TRIAXIAL TESTS FOR TYPICAL WINNIPEG CLAYS

A Thesis

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by

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ABSTRACT

Presented in this thesis are the results of a series of Triaxial tests run on undisturbed samples of a highly plastic Winnipeg clay. The shear strength properties, pore pressure characteristics and the effect of the strain rate were investigated and compared with the available information.

The results indicate the presence of a low angle of shearing resistance and a high cohesion intercept. The nature of the pore pressure change due to applied deviator stress indicates Winnipeg clays as being lightly over-consolidated.

The rate of strain is seen to influence the test results in the consolidated undrained tests. For reliable results in such a test a strain rate with a time to failure of four to six hours or more appears satisfactory.

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CHAPTER I

INTRODUCTION

A knowledge of the properties of a soil is necessary to the Engineer in the design of foundations, highway embankments, dams etc. Soils, unlike most materials encountered in Engineering design, have properties that show wide variation in different geographical locations. Thus it is necessary to determine these properties of different soil types in different regions.

Clays in the Winnipeg area are known to be laminated, highly plastic and compressible. A brief description of the soils in the Winnipeg area are given in Chapter 2. Classification and suitability of these soils for various purposes, from an Engineering point of view, has been dealt with by A. Baracos[1][2] W. F. Riddell[14] and others. An investigation of the shear strength properties of this soil as given by the triaxial test has been done by C.B. Crawford[7], J. D. Mishtak[12] and J. Peters as a part of a yet unfinished M. Sc. thesis at the University of Manitoba. Vane shear test results have been given by F. D. Young[19].

This report contains the results of a series of triaxial tests run on a typical undisturbed clay in the Winnipeg area. The main purpose was to determine the shear strength properties, pore pressure characteristics and the effect of the strain rate. A large number of uniform block samples taken from the Red River Floodway site were available for this purpose.

CHAPTER II

SOILS IN THE WINNIPEG AREA

The soils in the Winnipeg area have been described in previous papers by W. F. Riddell[14], A. Baracos[1], J. A. Elson[9]. A brief summary of these may be given here.

The area that is the City of Winnipeg now was a part of the bed of Glacial Lake Agassiz that covered most of this region thousands of years ago. It is believed that water to a depth of over 500 feet once covered this area. Great quantities of fine sediments from in-flowing rivers were deposited on the bed of this lake. As the ice margins retreated and the waters in the lake discharged through southern and north-eastern outlets the area that is now the City of Winnipeg was formed.

The soils of the Lake Agassiz are predominantly silts and highly plastic laminated clays overlying glacial till. The first few layers of soil up to about 8 or 10 feet depth consists of the more recent deposits of silty and organic soils. Below that is a layer of brown clay about 15 or 20 feet in depth. This clay is locally known as "chocolate" clay. Underlying that is another layer of clay known as "blue" clay. This layer is also about 15 or 20 feet in depth. Both these clays have similar characteristics. The clay content of both has been found to be about 75 or 80 percent. The natural moisture content is usually about 50 percent. The liquid limit of the clays average about 90 and the average plastic limit is about 30. The degree of saturation is usually very high and close to 100%. Many Winnipeg buildings have foundations in this strata and in regions where the moisture content of the soil does not undergo large changes these foundations have been found to be very satisfactory. (W.F. Riddell[14]).

Below these two above mentioned layers is found "hard pan" or glacial till consisting of a mixture of sand, rock flour, gravel and boulders often cemented and below that limestone rock at total depths of about 50 or 60 feet.

The ground water conditions are rather uncertain in the varved clay areas. Most of the soil below a depth of about 8 feet is usually almost fully saturated and a substantial zone of capillary rise is believed to exist.

CHAPTER III

PROPERTIES OF THE SOIL AND SAMPLE PREPARATION

The soil tested was taken from undisturbed block samples taken from the Red River Floodway site. Some of the relevant properties of this soil as determined by tests on the samples are given below.

Depth ft.	Colour	Moisture Content %	Specific Gravity	Liquid Limit	Plastic Limit	Clay Fraction $\mu < .002$ mm
20	grey	58.1	2.76	114.5	40.5	84.
25	"	54.5	2.76	112.2	38.7	81.

TABLE 1 - Some properties of the soil tested

The liquid and plastic limits of this soil were very close to the maximum values reported for soils in the Winnipeg area - Baracos(1). In the samples tested the degree of saturation varied between 95% and 100% (Table 6). The soil was easy to handle and test.

Preparation of test specimens from block samples

Test specimens were carefully cut from blocks. A wire saw and a knife were used to cut the blocks into smaller rectangular prisms. The ends of each prism were then trimmed parallel to each other. The prism was then mounted on a rotary soil trimmer and converted to a cylindrical shape by carefully shaving away all the surplus material using a wire saw. The specimen was finally shortened to the required length by inserting it in a cylinder and trimming off the ends using the edges of the cylinder as guides. The internal diameter of the cylinder used (1.40 inches) gives the diameter of the sample tested. As such when the prisms were converted to cylindrical shapes, the rotary soil trimmer was adjusted to give a diameter as close to this as possible. The height to diameter ratio of the specimen was kept at two so that the specimens tested were 1.40 inches in diameter and 2.80 inches long initially.

During the preparation of the test specimens from the blocks every care was taken to minimize surface evaporation. Temporary resealing was carried out whenever necessary and the samples were wrapped in "saran wrap" and covered with damp rags at all times. The samples not immediately used in testing were wrapped in "saran wrap", covered by aluminum foil and given two coatings of wax. These then were stored in a moisture room. No apparent loss of moisture was found in any of the specimens.

In preparing cylindrical samples attention was paid to the laminations in the natural soil. They were made with the cylindrical axis always perpendicular to the direction of these laminations.

The variation of the moisture content within the block was found to be very small as can be seen from the values in Table 6. The soil samples were easy to handle and test.

<u>Undisturbed soil vs. Remoulded soil</u>

In many investigations of shear strength characteristics of soils, large batches of well mixed remoulded samples are used. There are two main reasons for this. Firstly, it is difficult to obtain perfectly homogeneous undisturbed samples of soil in sufficiently large numbers. The variation that may occur between individual samples is likely to make comparison of test results difficult. Secondly, remoulding enables a control to be exercised over the water content of the soil tested.

Tests on remoulded soils would only help to indicate the pattern of soil behaviour and is essentially suited for a qualitative study. In Winnipeg clays that are known to be laminated, remoulding destroys these laminations.

CHAPTER IV

APPARATUS AND PROCEDURE

Only a brief description of the Triaxial testing apparatus need be given here, as the standard type apparatus described by Bishop and Henkel[3] was used.

The triaxial cells were the standard type used for testing $1\frac{1}{2}$ inch diameter samples. A labelled diagram of this is shown in fig. 1.

The system for controlling the cell pressure was the self compensating mercury control type. This is easy to operate and once set, the cell pressure can be maintained constant over a long period of time.

The pore pressure measuring system was the null indicator type and was manually operated. Originally the pressure connection to the triaxial cell was a polythene tube. On undrained tests run at a rate of strain of approximately 3.5% per hour, the pore pressures were found to be small. The only satisfactory explanation for these very low values of pore pressure on what was believed to be fully saturated normally consolidated soil was the possible expansion of the pressure connection tube. This was replaced by a flexible coiled copper tube as the pore pressure connection from the triaxial cell. More acceptable pore-pressures were recorded on the same soil and these results are discussed elsewhere.

The general set up for an unconsolidated undrained test is illustrated in fig. 2.

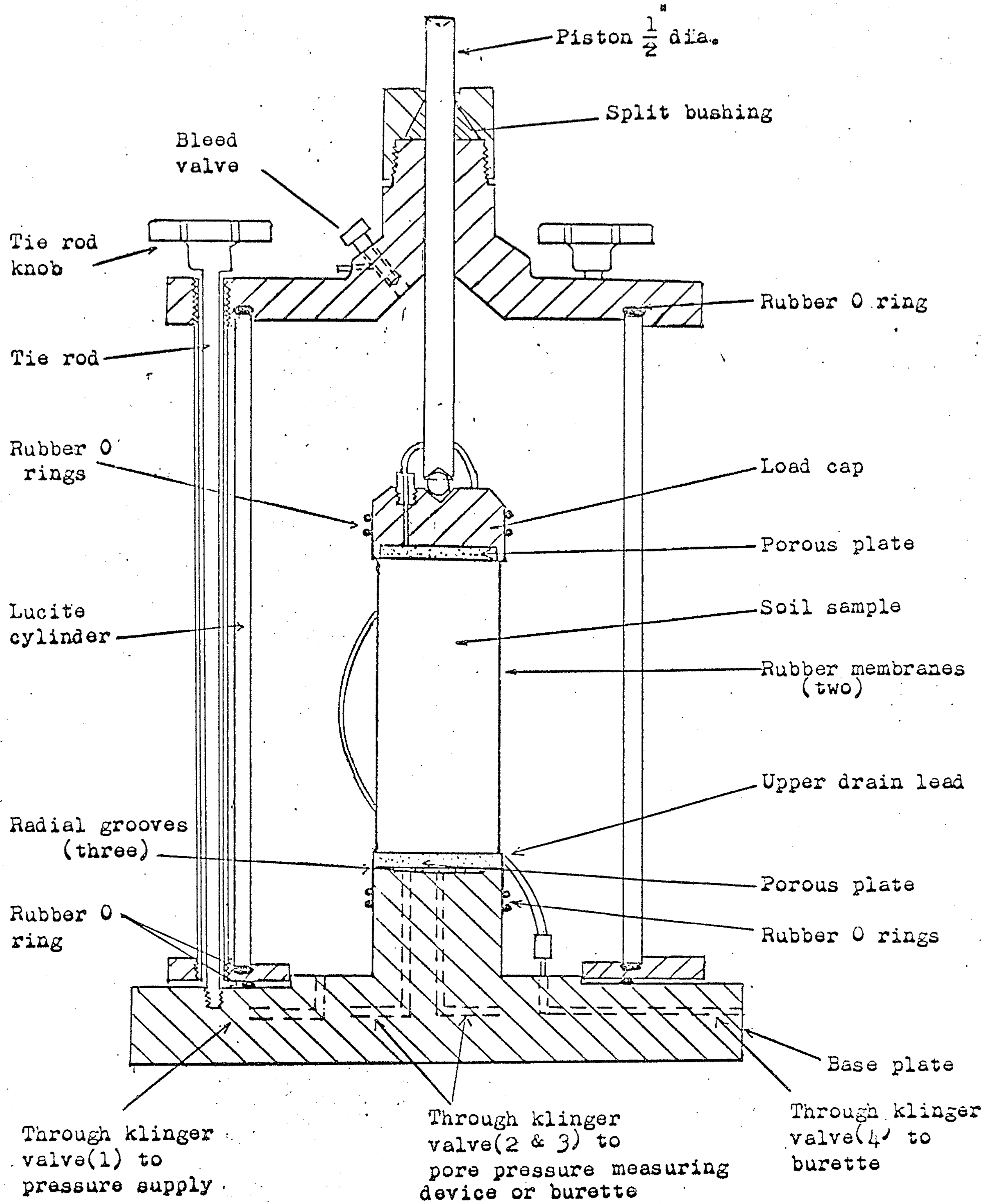

Fig. 1 - Triaxial Cell

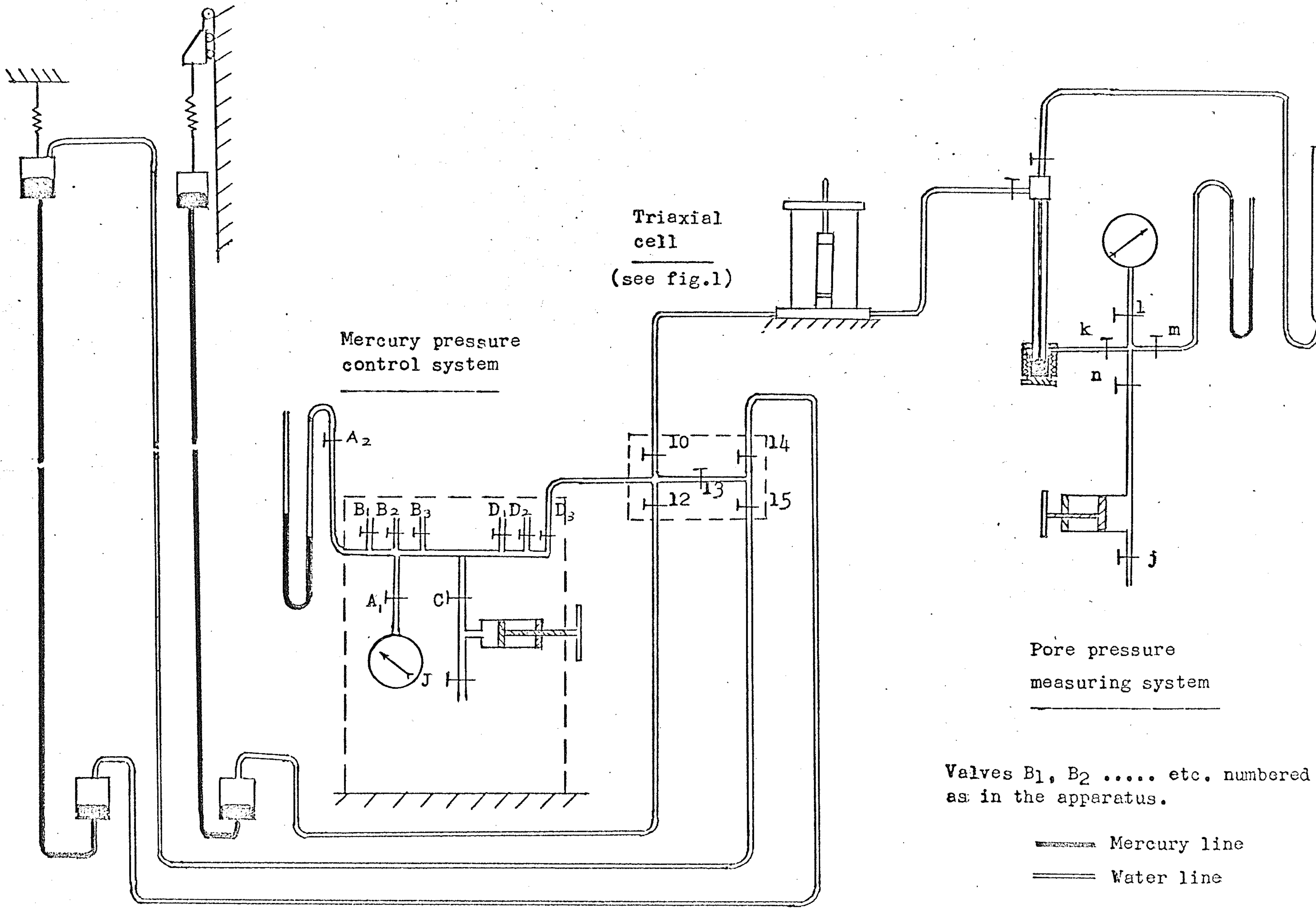

Fig. 2 - The layout of the apparatus for a consolidated undraied test

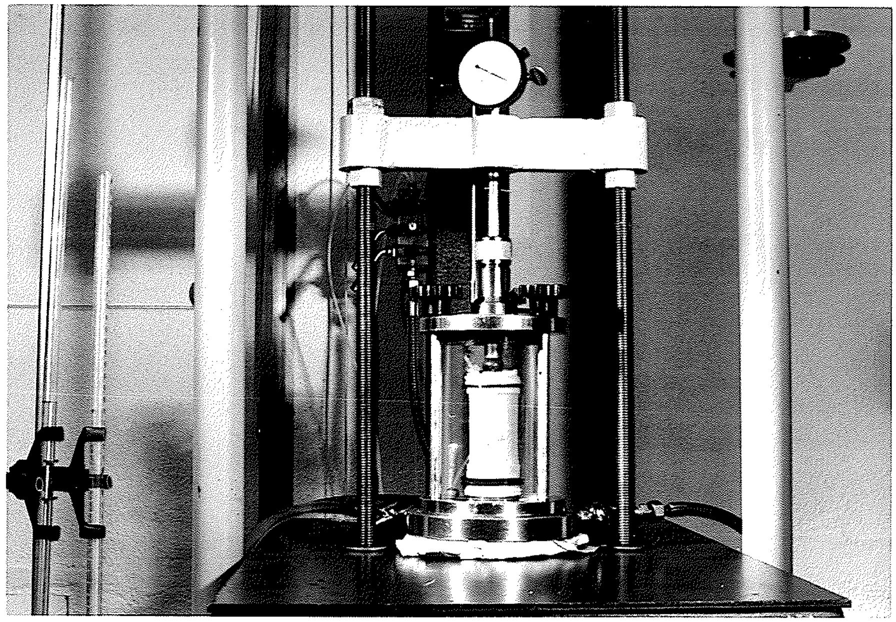

Fig. 3 - Set up for initial consolidation and drained test.

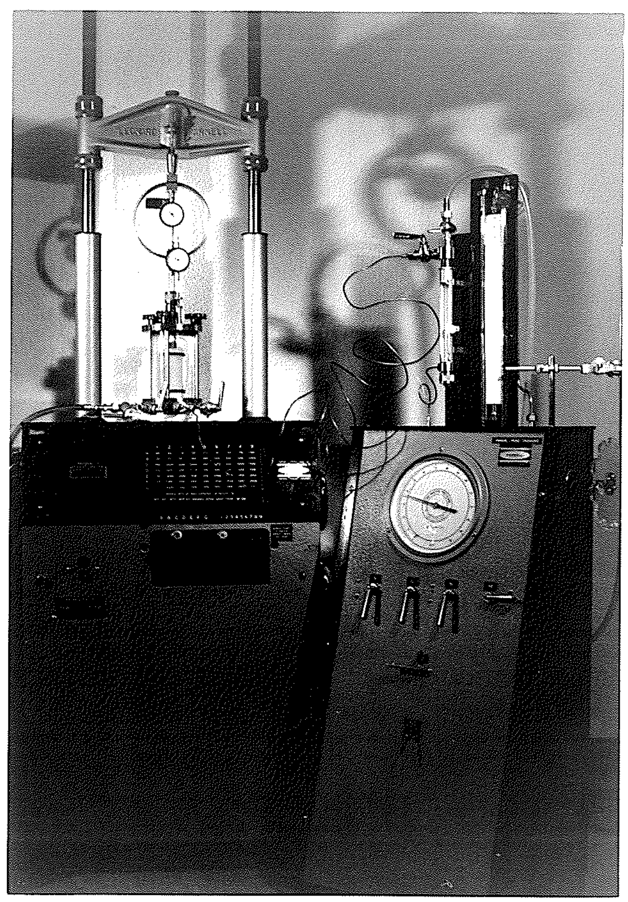

Fig. 4 - Consolidated undrained test in progress.

Procedure

All tests were run on laboratory reconsolidated samples of undisturbed clay. As such, the procedure for setting up of the soil specimens and the initial consolidation process were the same for all the tests. Only the subsequent shear tests varied, depending on how and under what conditions failure was caused. These different types of shear tests are discussed later. Only the initial setting up and the consolidation of the soil samples are described here.

Many of the precautions necessary during the setting up of a sample for a test, are taken to avoid air getting trapped in any of the lines or connections leading to the pore pressure measuring instrument. Entrapped air causes a time lag in the registering of pore pressure; gives rise to erroneous results in the case of the undrained tests with pore pressure measurement.

The base of the cell was immersed in water with the valves 1, 2, 3, 4 (see Fig. 1) open. These were then closed and the cell base was transferred to the loading platform of the testing machine. (Immersing the cell base in water helps to fill up the connecting lines from the valves to the pedestal with water and get rid of any air trapped in them). Two burettes half filled with de-aired water were then connected to the outlets 3 and 4. This was to allow drainage at the top and bottom ends of the sample during the initial consolidation stage--from the top through the porous stone in the load cap, upper drain lead and

Klinger valve 4 into the berette; from the bottom through the Klinger valve 3 into the other burette. The volume of water thus collected is a measure of the volume change in a saturated sample.

A little water was allowed to flow back from the burette to cover the pedestal. A saturated porous disc was slid onto the top of the pedestal. The sample of clay whose initial dimensions and weight were noted was placed on this porous disc. To accelerate consolidation saturated filter paper drains are placed round the specimen before it is placed on the pedestal. (The filter paper used was 1/4" wide and were placed at 1/4 inch intervals, lengthwise along the samples. Two pieces of filter paper, circular in shape were placed at each end between the end of the sample and the porous disc.) Two rubber membranes were placed over the sample using a membrane stretcher, and the lower part of the membrane was sealed to the pedestal with two "O" rings. Before the load cap was placed in position and the upper end sealed off, valve 3 was opened and some water was allowed to flow upwards between the specimen and the rubber membrane. (The air trapped between the soil specimen and the membrane is driven out this way.) The load cap was carefully placed in position next and the upper part of the rubber membrane was sealed to the load cap with O-rings. The burette connected to valve 3 was then lowered with valve 3 open, so that a few inches of negative head was acting on the sample. (The excess water is removed this way and the slight negative pressure ensures that the sample is sitting firmly on the pedestal). After a few minutes this valve was closed and the burette returned to a convenient position with the level of the water in it in line with the center of the sample.

The porous stones were cleaned and saturated by leaving them in boiling water, before use. It is good practice to connect the load cap (with the saturated porous disc in place) to the upper drain lead, immerse it in de-aired water and apply a suction pressure at the end of the burette connected to valve 4. This ensures a clean air-free drainage line.

The top part of the cell was next positioned in place and the base of the cell was assembled. De-aired water was run into the cell through the opening 1, with the bleed valve kept open. When the cell was nearly full about 1/4 inch depth of oil was introduced. More water was admitted and any remaining air was driven out through the bleed valve and this was then closed. The loading arrangement was set up, and the cell was connected to the Mercury pressure system through valve 1. The pressure was adjusted to the desired value and valve 1 was opened. Simultaneously valves 3 and 4 were opened and the burette readings were taken at suitably spaced intervals. The change in height of the specimen was obtained by using a dial gauge set to zero initially with loading ram in contact with the ball on the loading cap. A graph of volume change against time was plotted to ascertain if consolidation was complete. Usually the consolidation was complete after about 48 hours.

The setting up and the initial consolidation process were the same for all the tests. The drained tests could simply be started after this; the vertical load increased or the cell pressure decreased depending on how the shear failure is brought about.

In the case of the undrained tests it was necessary to transfer the cell to the testing machine and for the pore pressure measuring system to be connected to the base of the cell. To do this, when the consolidation was complete valves 3 and 4 were closed and the pressure in the cell was temporarily reduced to zero. The cell and the burettes were then removed as a unit and transferred to the testing machine. The cell pressure was applied once again. The connection to the burette at 3 was removed.

A little water was passed upwards through valve 2, under a small pressure of about 1 or 2 lb/sq. in. and allowed to drain out through 3, while connecting the pore-pressure measuring system to the base of the cell at valve 3. It was necessary to ensure that the pore-pressure measuring system was free of air. The procedure for this was standard. (Bishop and Henkel[3]). The undrained test with measurement of pore-pressure was then carried out with the chosen rate of strain.

Consolidated Undrained Tests

The sample was set up, allowed to consolidate under a chosen cell pressure, transferred to the testing machine and connected to the pore-pressure measuring system as described earlier.

The dial gauges that measure the load (deflection of proving ring) and strain were set to zero with no load acting. The machine was set to the desired speed or rate of strain and the test was started. The dial gauge readings and pore pressure were taken at equal intervals of time, until failure. The choice of a suitable rate of strain was trial and error. It should be slow enough to allow for equalization of the pore-pressure within the sample. This is discussed elsewhere.

Over-consolidated Undrained Tests

This test was run the same way as the consolidated undrained test except that the soil was 'laboratory over-consolidated' before causing shear failure. This was done by first allowing the sample to consolidate at a high cell pressure (120 lb/sq. in. was chosen for these tests) for about 48 hours and then allowing it to swell back at a reduced pressure, for about another 48 hours. The shear test was carried out the same way as in the case of the consolidated undrained test.

Consolidated Drained Test (minor principal stress σ_3 constant, major principal stress σ_1 increasing)

The sample was set up as described earlier and allowed to consolidate for about 48 hours under a chosen cell pressure. When consolidation was complete the deviator stress $(\sigma_1 - \sigma_3)$ which was initially zero was increased in steps until failure occurred. The load increments used were 10% of the estimated failure load at the start of loading and smaller increments as the failure load was reached. After each increment of loading the sample was allowed to drain and consolidate for about 24 hours. Dial gauge and burette readings were taken at the start and the end of each load increment and at suitably spaced intervals. This test resembles a continuation of the initial consolidation process with the addition of vertical load increments on the sample as the only difference.

<u>Consolidated Drained Test</u> (σ_3 decreasing, σ_1 constant)

The setting up of the sample and the initial consolidation process was the same as in the rest of the tests. After consolidation was complete failure of the sample was brought about by keeping the vertical stress on the sample at the same value of the initial all round pressure and decreasing the cell pressure in steps, until failure. The cell pressure was decreased in steps of 10% of the estimated total cell pressure drop necessary to cause failure, at the start of the test and smaller decrements were used as failure was reached. As the cell pressure was decreased it was necessary to make two corrections to the vertical load on the sample to keep σ_1 constant due to the following reasons.

(i) An additional load is required to balance the decrease in the stress (σ_1) due to the decrease in the cell pressure. This is calculated by multiplying the area of the sample before the cell pressure is reduced by the amount by which the cell pressure is reduced ($\Delta\sigma_3$) at each stage.

(ii) The upward force on the loading ram is reduced when the cell pressure is lowered and therefore a corresponding decrease in the vertical load is necessary for balance. The reduction in the load on the ram due to

(ii) this effect is obtained by merely multiplying the cross-sectional area of the ram ($\frac{1}{2}$ inch diameter) by $\Delta\sigma_3$.

To adjust the vertical load for (i) and (ii) the valve 1, connecting the cell to the mercury pressure system, was first closed. The cell pressure was lowered to the new value. The necessary load to keep σ_1 constant, was added to the loading arm of the platform scale and the valve 1, was opened simultaneously. At each stage the sample was allowed to drain and consolidate for about 24 hours.

In all these tests after shear failure, the load was removed, the cell pressure was reduced to zero, the water in the cell was run out and water content tests were done on the soil sample. The water content at the top, bottom and the failure region, was determined.

Pore Pressure vs. Hydrostatic Confining Pressure

A method for evaluating the behaviour of the pore-pressure measuring system is to determine the variation in pore pressure with increase of hydrostatic pressure. (Casagrande and Hirschfield[5]).

At ordinary pressures the pore water and the soil grains in a sample of soil can be considered to be incompressible. i.e. the volume of a saturated soil and consequently the void ratio will remain constant under conditions of no drainage. Therefore if a sample of soil is assumed to be 100% saturated an increase in the confining pressure should cause an equal increase in the pore pressure. In an ideal system for measuring pore pressure this increase will be registered as an immediate rise in the pore pressure to its new value. Hence the variation of pore pressure with cell pressure is linear and has a gradient equal to one. This corresponds to B = 1, (Skempton[18]).

In problems concerning the undrained shear strength of soils the pore pressure change ($\triangle u$) which occurs under changes in the principal stresses $\triangle\sigma$, and $\triangle\sigma_3$, has been expressed by the equation,

$$\triangle u = B\left[\triangle\sigma_3 + A\,(\triangle\sigma_1 - \triangle\sigma_3)\right] \quad \text{----Skempton}^{(18)}.$$

where A and B are referred to as the pore pressure coefficients or pore pressure parameters.

In the derivation of the above expression the application of the stresses $\triangle\sigma_3$ and $\triangle\sigma_1$ have been considered in two stages; firstly, an equal all-round increment $\triangle\sigma_3$ and, secondly, a deviator stress ($\triangle\sigma_1 - \triangle\sigma_3$). The change in pore pressure has been considered separately corresponding to each of these stages. In other words,

$$\triangle u = \triangle u_a + \triangle u_d,$$

where $\triangle u_a$ = pore pressure change due to the application of $\triangle\sigma_3$.

$\triangle u_d$ = pore pressure change due to the application of ($\triangle\sigma_1 - \triangle\sigma_3$)

The coefficient B, has been shown to be given by the expression,

$$\frac{\triangle u_a}{\triangle\sigma_3} = B = \frac{1}{1 + \frac{nC_v}{C_c}}$$

where
n = porosity of the soil
C_v = compressibility of the pore fluid (air and water)
C_c = " " " soil structure

In the case of a fully saturated soil, $\frac{C_v}{C_c}$ is approximately equal to zero since the compressibility of the water is negligible compared to that of the soil skeleton. In other words, B = 1 or the pore pressure change is equal to the change in the all round pressure in the case of a fully saturated soil under conditions of no drainage.

Δu_d has been given by the expression,

$$\Delta u_d = \frac{1}{1 + \frac{nC_v}{C_c}} \cdot A \cdot (\Delta\sigma_1 - \Delta\sigma_3)$$

$$\text{i.e., } \Delta u_d = B \cdot A \cdot (\Delta\sigma_1 - \Delta\sigma_3)$$

where A is a coefficient that depends on the elastic properties of the soil. For a material that behaves in accordance with the elastic theory A = 1/3. In soils A varies greatly from this value and has to be determined experimentally. This is discussed elsewhere.

In the case of an unsaturated soil the condition B = 1, does not apply because of the presence of air in the voids. Initially B is less than 1, but as the confining pressure increases and consequently the pressure in the pore water, the theoretical value B = 1 for a saturated soil, is approached. As the pore pressure increases, the volume of air in the voids decreases and much of it goes into solution thereby increasing the degree of saturation.

In the case of an undisturbed sample of soil that has not been allowed to re-consolidate there is always an initial tension or a negative pore pressure in the pore water. Hence the curve representing the variation of pore pressure with confining pressure will show a negative intercept. This negative pore pressure does not alter the linear variation or the B = 1 condition for a saturated soil.

The results of such a test are shown in figure 6. The sample was set up as described earlier but with direct connection to the pore pressure measuring device. No consolidation was allowed. The cell pressure was increased from zero to 20 lb. per sq. in. and thereafter in steps of 10 lb. per sq. in. After each increment the pore pressure was measured allowing 5 minutes between each increment of cell pressure. The sample tested showed a degree of saturation of 98% initially but at cell pressures shown it can be assumed to be fully saturated. The graph showing the variation of pore pressure with cell pressure, shown in fig. 6(a) approximates very closely to the perfect theoretical variation to be expected in such a test. The negative intercept obtained by extrapolation was -4.0 lb. per sq. in. Tabulated below are the corresponding values of $\Delta\sigma_3$ and Δu_a along with the value of B for each increment.

$\Delta\sigma_3$ lb.per sq. in.	Δu_a lb. per sq. in.	B
0	-	-
20.0	15.0	
30.0	25.0	1.000
40.0	34.0	0.900
50.0	43.9	0.990
60.0	53.6	0.970
70.0	63.0	0.940
80.0	72.5	0.950
90.0	82.2	0.970

Table 2 - Pore pressure parameter B

Fig. 6 (b) shows the typical variation of pore pressure with time during a 10 lb. per sq. in. increment in cell pressure. Theoretically the pore pressure should increase by an equal amount immediately. The total time allowed for the measurement of pore pressure before a new increment of cell pressure was applied, was 5 minutes. As the curve in fig. 6 (b) shows the pore pressure measuring system was sensitive and almost the entire increase in the pore pressure was quickly shown.

Almost no decrease in the height of the sample was observed. This condition is not unusual in a fully saturated sample when there is no drainage.

This test provides a check on the reliability of the pore pressure measuring system and helps to verify that the pore pressure parameter B = 1 for saturated soils.

CHAPTER V

RESULTS AND DISCUSSION

The results are presented graphically in the form of

(i) Stress-strain curves
(ii) Pore pressure-strain curves
(iii) Mohr circles

A typical set of results for a consolidated undrained test along with the necessary calculations are shown in Data sheets (i) and (ii). In the undrained tests the cross-sectional area (a_c) of the sample after initial consolidation was calculated assuming the sample to remain perfectly cylindrical in shape.

$$a_c = a_o \cdot \frac{(1 + \Delta v/v_o)}{1 - \varepsilon}$$

where a_o= initial average cross-sectional area

Δv = change in volume

v_o = initial volume

ε = axial strain

During the shearing stage when no change in volume occurs,

$$a_c = \frac{a_o}{1 - \varepsilon}$$

The same formula, $a_c = a_o \cdot \frac{(1 + \frac{\Delta v}{v_o})}{1 - \varepsilon}$, was used to calculate the average cross-sectional area of the sample after each stage of load increment, during the consolidated drained tests.

No correction was made for the restraint imposed by the rubber membranes or the vertical filter paper drains. The corrections for these have been given as 0.6 lb. per sq. in. for a standard membrane (0.008 in. thick) and approximately 2 lb. per sq.

in. for filter paper drains at 15% strain. - Bishop and Henkel[3]. These are small enough to be considered negligible.

Also in the presentation of the results Stress History paths were drawn in the case of the undrained tests with pore pressure measurement. (Figs., 13 (b), 14 (b), 15 (b)).

Stress History Paths

The Mohr circles can be used to represent the conditions of stress at any particular stage and not necessarily failure. The shear stress and the normal stress on the final plane of failure or any other plane can be determined at any particular stage of loading, knowing the principal stresses σ_1, and σ_3.

A stress history path may be defined as the locus of points, whose co-ordinates are the shear stress and the normal stress on the plane on which failure is assumed to take place, plotted from the start of loading until failure, in a triaxial test.

As the definition implies it is necessary to assume the ultimate failure plane. In the case where the results are represented by the Mohr envelope the failure plane is taken as that plane which is inclined at an angle (α_f) to the direction of the plane on which the major principal stress acts, where

$$\alpha_f = 45+\phi'/2$$

(i.e., the plane on which the stresses at failure are represented by the point where the Mohr envelope touches the failure circle).

This assumption is theoretical but is usually a close enough approximation to the direction of the actual failure plane.

Plotting of Stress history paths

The stress history paths can be plotted either graphically or using the following equation.

$$\bar{\sigma} = \sigma'_3 + (\sigma_1 - \sigma_3) \cos^2 \alpha_f$$

$$\tau = (\sigma_1 - \sigma_3) \sin \alpha_f \cos \alpha_f$$

where $\bar{\sigma}$ and τ are the effective normal and shear stresses, on the assumed plane of failure, for any corresponding values of σ'_1 and σ'_3. (See figure 3 (a)).

Graphically, Mohr circles are plotted for corresponding sets of values of σ'_3 and σ'_1, for each shear test. Tangents to these circles are drawn, parallel to the Mohr envelope for the total stress circles and these tangent points are joined to obtain the stress history path (Fig. 5(b)). On the Mohr circles these tangent points represent the shear stress and normal stress on the assumed failure plane. In the case of the total stress circles the stress history paths are straight lines as all stress circles for any particular shear test have the same common starting point (σ_3, 0). Each line is inclined at an angle of α_f to the positive direction of the σ axis and also represents zero pore pressure. Therefore, the horizontal distance between the stress history path for the effective stress circles and a line inclined at an angle α_f to the positive direction of the σ axis and passing through (σ_3, 0) is a measure of the pore pressure.

Stress history paths — (for consolidated undrained tests)

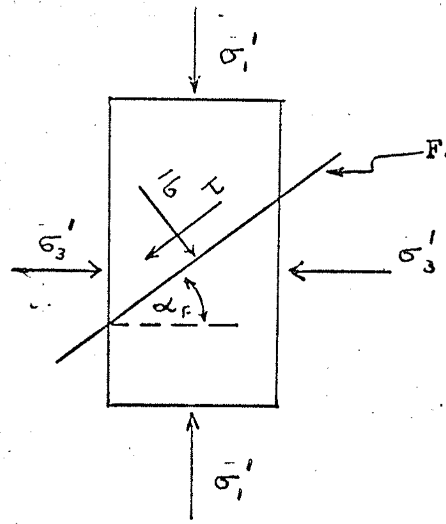

Fig. 5 (a)

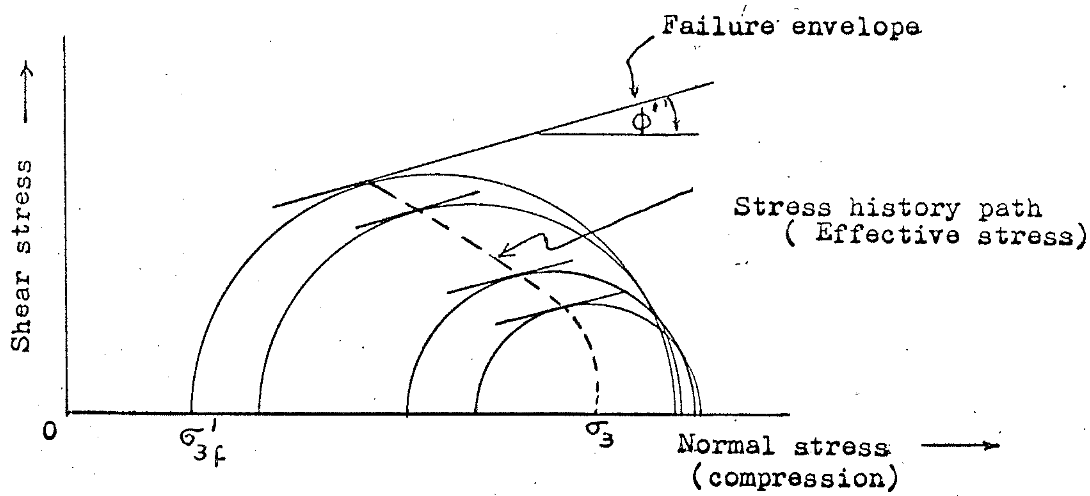

Fig. 5 (b)

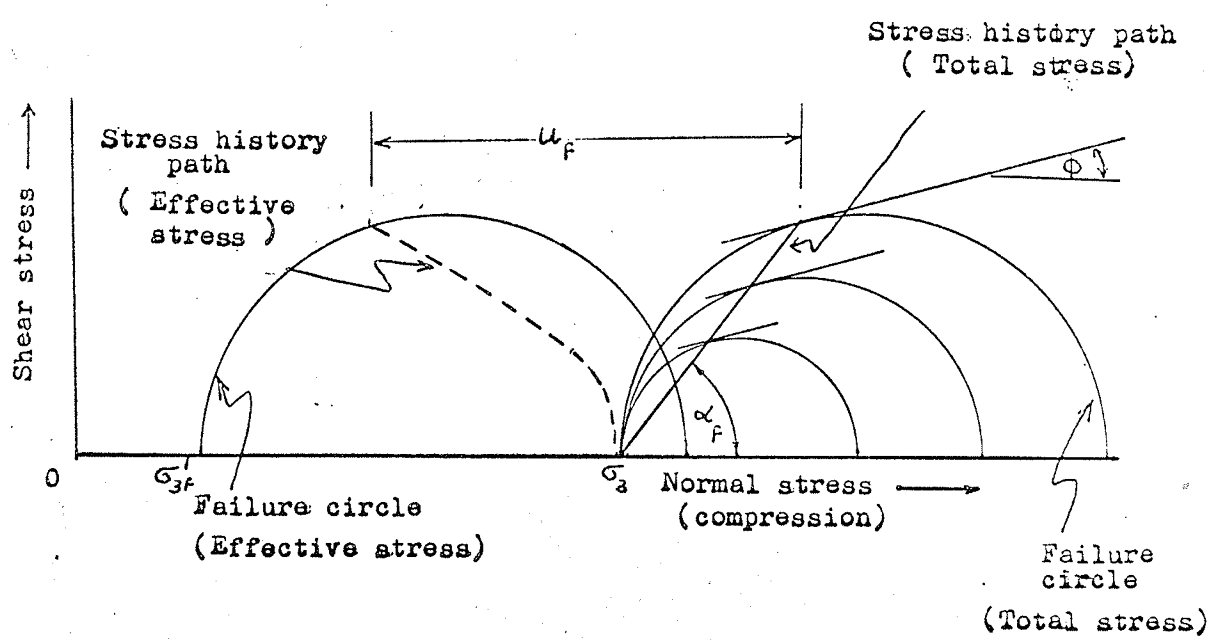

Fig. 5 (c)

In other words the horizontal distance between the stress history paths for the total and effective stresses for any one test at any particular stage represents the pore pressure at that stage. The inclination of the tangent to the stress history paths with the positive direction of the σ axis gives the rate of increase of pore pressure with deviator stress.

The stress history paths have the advantage of showing on one plot, the development of pore pressure and the pore pressure characteristics of a soil.

The stress history paths for the consolidated undrained tests are shown in figures 13(b), 14(b), 15(b) along with the Mohr's circles. Those for the over-consolidated undrained tests are shown in figure 20. As can be seen from figures 13(b), 14(b) and 15(b) the stress history paths follow a similar pattern in all the undrained tests run at different rates of strain. They lie to the left of corresponding stress paths (straight lines) for the total stress because the pore pressure was positive. The inclination of the tangent to the stress history paths with the positive direction of the σ axis is seen to increase throughout the duration of each test and reaches a maximum value when failure is imminent. As this "inclination" shows the rate of increase of pore pressure with deviator stress it means that, this rate was always less than unity $\left(\frac{du}{d(\sigma_1 - \sigma_3)} < 1\right)$ at 'low' strains and approached unity as the strain increased and failure became imminent. (When the pore pressure increases at the same rate as the deviator stress, the stress history path for effective stress is at right angles to the direction of the corresponding stress history path for total stress. For higher inclinations, $\frac{du}{d(\sigma_1 - \sigma_3)} > 1$ and for lower inclinations $\frac{du}{d(\sigma_1 - \sigma_3} < 1]$. As can be seen from the curves of deviator stress vs. strain and pore pressure vs. strain in figures 7, 8, and 9 the deviator stress increased rapidly during the initial $2\frac{1}{2}$ or 3% strain whereas the corresponding rise in pore pressure was much less rapid. Approximately after the initial 3% strain the rate of increase of deviator stress as well as pore pressure, with strain, became smaller than during the initial 3% and finally both the deviator stress and pore pressure attained a constant value before failure occurred. Therefore the portion of the stress history paths having the greatest curvature (see figures 13(b), 14(b), 15(b)) corresponds to a strain in the sample of approximately 3%.

In the case of the samples tested at a confining pressure of 20 lb. per sq. in. the stress history paths are seen to be flat and close to the vector curve for the total stress. This may be due to the fact that the confining pressure of 20 lb. per sq. in. was only slightly greater or perhaps less than the maximum effective over-burden pressure the soil has been subjected to. The soil tested was from depths of 20 and 25 feet. An exact value for the effective overburden pressure cannot be estimated due to the uncertainty of the ground water conditions. A rough estimate of the total overburden pressure may be made using the moist density of the soil. At a depth of 25 feet this gives an approximate total overburden pressure of 19 lb. per sq. in. or 1.29 tons per sq. foot. Due to the variations in the water table and possible effects of dessication it is possible that the soil was subject to pressures somewhat higher than these. Hence in the case of the samples tested at a confining pressure of 20 lb. per sq. in. the soil may have behaved in the manner of a slightly over consolidated sample.

The stress history paths for the laboratory over consolidated samples are shown in figure 20.

Stress-strain curves and failure criteria

The stress-strain curves shown in figures 5 to 10 show a great deal of consistency and similarity in the case of the consolidated undrained tests, run at rates of strain of 1.5% per hour and 0.9% per hour. It can be seen that in the normally consolidated samples the pore pressure reached a maximum when the deviator stress attained its maximum value and remained constant thereafter. The two failure criteria used in triaxial tests are the maximum deviator stress and the maximum principal effective stress ratio $\left(\frac{\sigma_1'}{\sigma_3'}\right)$.

The effective major principal stress (σ_1) can be expressed as,

$$\sigma_1' = (\sigma_3 - u) + (\sigma_1 - \sigma_3)$$

Hence the principal effective stress ratio can be given by

$$\frac{\sigma_1'}{\sigma_3'} = 1 + \frac{(\sigma_1 - \sigma_3)}{(\sigma_3 - u)}$$

From this it can be seen that where the deviator stress and the pore pressure attain their maximum values at the same strain and remain constant the two failure criteria coincide. This being generally the case with the soils tested, both failure criteria gave the same stress circles and consequently the same values of ϕ' and C'. The same was true of the laboratory overconsolidated samples the only difference being a little falling off of the pore pressure from a maximum value that was reached before the deviator stress attained its maximum value.

In many triaxial tests on normally consolidated clays, the two failure criteria give different shear strength envelopes. From a practical point of view these differences may be unimportant but they reflect certain properties of the different soil types.

A summary of the results of many triaxial tests on normally consolidated Norwegian clays and some other clays has been given by L. Bjerrum and N. E. Simons(4) with a comparison of the ϕ' values obtained by using the different failure criteria. In the case of most of these soils the deviator stress has been observed to reach a maximum at a fraction of the strain at which the pore pressure reached a maximum. In such cases the maximum principal effective stress ratio gives a higher shear strength envelope than the maximum deviator stress failure criterion. Similar results have been given by N.E. Simons[16] for an Undisturbed Drammen Clay, with a comparison of both failure criteria. These need not be reproduced here but two of the factors that can be seen to influence this difference appear to be sensitivity and the strain at which the deviator stress reaches a maximum. The difference in the ϕ' value given by the two failure criteria, seems to increase with increasing sensitivity. (Sensitivity is the ratio of the undisturbed to remoulded strength of a soil, at constant water content. The sensitivity of Winnipeg clays has been given as 2.0 to 3.3 by F.D. Young[19] after many in situ tests using the Vane shear apparatus.) The same difference appears to become smaller as the strain at which the deviator stress reaches its maximum value, becomes larger. This latter observation appears to be true when applied to Winnipeg soils. The maximum deviator stress was reached at the maximum strain the samples underwent, before failure planes developed.

Also results of several consolidated undrained tests on normally consolidated clays have been given by T.C. Kenney[11]. In these he has compared the values of ϕ' as obtained by using the two failure criteria. He uses the Mohr circle to show that the deviator stress acting on an element of soil to be given by

$$\sigma_1 - \sigma_3 = 2 \quad \frac{C'_{mo} \operatorname{Cos} \phi'_{mo} + (\sigma_3 - u) \operatorname{Sin} \phi'_{mo}}{1 - \operatorname{Sin} \phi'_{mo}}$$

where c'_{mo} and ϕ'_{mo} denote the mobilized values of the shear strength parameters and u denotes the pore water pressure, σ_1 and σ_3 being the total major and minor principal stresses, respectively. Using this concept of "ϕ' mobilized" he concludes that at maximum deviator stress the amount of ϕ' mobilized is not necessarily the maximum, as the degree of mobilization of the shear strength parameters depend not only upon the magnitude of the applied shear stresses but also upon the magnitude of the induced pore pressures. Also from test results ϕ' has been found to depend on the stability of the soil structure and the sensitivity of the soil. Curves showing the variation of $\dfrac{\tan \phi'_{max(\sigma_1 - \sigma_3)}}{\tan \phi'_{max\left(\frac{\sigma_1'}{\sigma_3'}\right)}}$ with natural sensitivity for some normally consolidated clays as given by T.C. Kenney (11) and also for some Norwegian clays as given by L. Bjerrum and N. E. Simons (4) are reproduced in fig. 23 . These show that the difference in the value of ϕ' as given by the two failure criteria is zero for soils of low sensitivity but increases somewhat with increasing sensitivity.

The results of consolidated undrained tests on a soil that tends to dilate when sheared have been given by A.W. Bishop and D.J. Henkel (3). In this case values of ϕ' and C' close to their maximum have been found to be mobilized at a small fraction of the strain required to produce the maximum deviator stress. The increase in deviator stress at higher strains has been attributed to a drop in pore pressure due to the tendency of the soil to dilate when sheared.

In the case of the Winnipeg clays tested there was no ambiguity about the failure point. ϕ' at maximum deviator stress can be said to be its "fully mobilized" value. This perhaps is a reflection of the low sensitivity of the Winnipeg soils. Strains of the order of 5 to 6% were reached before failure occurred and in most cases failure planes started to develop after the maximum deviator stress was reached. Therefore in the case of this soil it may not be possible to increase the cell pressure to a new value when the maximum $\frac{\sigma_1'}{\sigma_3'}$ ratio is reached, and thus use the one sample method to define a satisfactory Mohr envelope.

In the case of the consolidated drained tests much higher strains were encountered and the deviator stress at failure was higher than in the case of the consolidated undrained tests. Strains as high as 16 to 18% were reached before failure. The samples strained uniformly and without non-uniform bulging. As failure loads were reached, the strain increased and at failure load the strain continued to increase until failure planes developed. There was no buckling or any distortions at high strains and consequently it was not difficult to judge the failure loads. The stress-strain curves were found to be somewhat irregular in shape (Figures 10 and 12) and this undoubtedly reflects the procedure of using step increments of load.

The stress-strain curves for the over-consolidated samples were very similar to those of the normally consolidated samples under undrained conditions. The pore and pressure vs. strain curves however differed in that the pore pressure reached a maximum value before the deviator stress reached its maximum and started to fall slowly while the deviator stress kept on increasing. No negative pore pressures were observed. This indicates that Winnipeg clays have little tendency to dilate when sheared. From this it is not implied that the clays in the Winnipeg area have no tendency to swell. The tendency of the Winnipeg clays to swell with increase of moisture content is well known.

Pore pressure parameter A

The concept of pore pressure parameters (explained earlier) introduced by A. W. Skempton(17) is a convenient way to express the pore pressure response to different changes in the applied stress. The significance of the parameter B which shows the response of the pore pressure to an increase in the all round stress has been mentioned earlier. The values obtained for parameter "A" at failure (A_f) are shown in Table 6. The "A_f" value was often less than 0.5 and in the case of the normally consolidated samples varied from 0.252 to 0.710 the latter value being the one for a sample tested at a confining pressure of 120 lb. per sq. in., which was the highest cell pressure used. These values of "A_f" are somewhat lower than those reported for soils in the Winnipeg area having similar properties. Given below are these values for the parameter "A_f" for soils in the Winnipeg area as obtained from the consolidated (isotropically) undrained tests.

- about 0.6 to 0.7 and close to 1.0 at confining pressures higher than 100 lb. per sq. in. - J. D. Mishtak[12].
- approximately 1.0 - P.F.R.A., Soil Mechanics and Materials Division[13].
- average of 0.72 - Norwegian Geotechnical Institute Publication No. 35. 1960[4].
- approximately 1.0 - C.B. Crawford[7].

The question arises as to whether the "low" values of "A_f" as summarized in table 6, could be due to faulty measurement of pore pressure.

It is seen from the curves of pore pressure vs strain that the pore pressure clearly attained a maximum value when failure was reached. The variation of the pore pressure with strain was small close to failure. Therefore the possibility of any time lag in the response of the pore pressure measuring device, to an increase in pore pressure can be discounted. The tests were generally run at slow enough speeds to allow for the equalization of pore pressure within the sample. Also from table 6, it is seen that the average value of ϕ' obtained from the consolidated undrained tests agrees closely with the value of ϕ' as given by the consolidated drained tests. All this points to the fact that in the consolidated undrained tests "A" at any stage represented the true change in the pore pressure due to the change in the deviator stress.

In most normally consolidated clays "A_f" has been found to be close to unity but this is not necessarily so. The table 3 gives an approximate range of A_f values (A at failure) to be expected for different types of soils as given by A.W. Skempton[18].

Type of Clay	A_f
Clays of high sensitivity	$+3/4$ to $+1\frac{1}{2}$
Normally Consolidated Clays	$+\frac{1}{2}$ to $+1$
Compacted Sandy Clays	$+1/4$ to $+3/4$
Lightly Over-Consolidated Clays	0 to $+\frac{1}{2}$
Compacted Clay gravels	$-1/4$ to $+1/4$
Highly Over-Consolidated Clays	$-\frac{1}{2}$ to 0

Table 3 - Pore pressure parameter A in different soil types (Skempton)

According to the typical values shown above the clay tested falls in the category of lightly overconsolidated clays.

There are many factors that affect the "A" value. In the derivation of the expression for the pore pressure change due to changes in the all round and deviator stresses, considered separately, parameter A has been introduced as a stress-strain constant and as such it varies with the factors that affect the stress-strain properties even for any one soil. Reproduced in tables 4 and 5, are some typical values of A_f for different soils, tiven by A.W. Biship and D.J. Henkel(3) and L. Bjerrum and N. Simons(4), respectively.

Type of Soil		Plasticity Index	Value of A_f
Normally consolidated—	marine clay: undisturbed	60	+1.3
	London clay: remoulded	52	+0.97
	Weald clay: remoulded	25	+0.94
	alluvial sand clay: undisturbed	18	+0.47
	loose sand	-	+0.08
	dense sand	-	-0.32
Over-consolidated—	Weald clay: undisturbed	25	-0.62
	Weald clay: remoulded over-consolidation ratio = 8	25	-0.22
	London clay: remoulded over-consolidation ratio = 8	52	-0.11

Table 4
(Typical values of A_f for various soil types - A.W. Bishop and D.J. Henkel)

Location	Plasticity Index	Sensitivity	A_f at σ_1'/σ_3' max.		A_f at $(\sigma_1 - \sigma_3)$ max	
			I.C.	A.C.	I.C.	A.C.
Drammen	15	8	1.32	1.67	1.18	0.91
Oslo	18	5	1.07	4.35	1.06	0.84
Oslo	18	8	1.11	1.76	1.00	1.13

Table 5
(Typical values of A_f for normally consolidated Norwegian clays - L.Bjerrum and N. E. Simons)

I.C. = Isotropic Consolidation
A.C. = Anisotropic Consolidation

The above values illustrate the factors that influence the "A_f" value. In over-consolidated clays "A_f" tends to be low and is sometimes negative in the case of heavily over-consolidated clays. In certain sensitive clays in which the two failure criteria give different shear strength envelopes the "A_f" value will obviously be different for the two failure criteria. Anisotropically consolidated samples show values of "A_f" somewhat different from samples consolidated isotropically prior to shear test.

From table 6 it is seen that for the over-consolidated samples tested "A_f" was lower than in the case of the normally consolidated samples. The "A_f" values varied from about $+1/8$ to $+1/4$ depending on the over-consolidation ratio. The variation of "A" with over-consolidation appears to be less in the case of Winnipeg clays than with other clays. For example, graphs showing the variation of "A_f" with over-consolidation ratio for London clay and Weald clay have been given by D.J. Henkel[10]. A similar curve for an undisturbed Oslo clay has been given by N. E. Simons[17]. These curves are very similar and show considerable variation in the "A_f" value of these clays with increasing over-consolidation ratio. In the case of the London clay this variation has been found to be from approximately $A_f = 1$ at an over-consolidation ratio (O.C.R.) of 1 through $A_f = 0$ at an O.C.R. of 4.5 to $A_f = -0.20$ at an approximate O.C.R. of 25. Therefore in Winnipeg clays the very limited variation of the pore pressure parameter A_f would justify its use in the analysis of practical problems. Of course in the ground where large areas are involved the soil is consolidated under conditions of no lateral yield. No tests were done on anisotropically consolidated samples. It may be assumed that the build up of pore pressure due to an increase in the vertical pressure will not be very different under these conditions.

Rate of Strain

The consolidated undrained tests were run at different rates of strain, as mentioned earlier. Three different rates of strain were chosen primarily to represent a wide range of speeds used in ordinary laboratory testing. These three rates of strain used are given below.

(a) 0.0015 inches per minute

This is about 3.3% per hour strain on a sample 2.7 inches in height before testing. The time to failure varied from 1 to 2 hours.

(b) 0.00067 inches per min.

This represents a strain of 1.5% per hour. The time to failure ranged from 2 to 4 hours.

(c) 0.00040 inches per min.

This is about 0.9% per hour strain. The time to failure was about 6 hours.

Figures 7, 8 and 9 show the stress-strain and pore pressure-strain curves for these tests. The Mohr rupture envelopes and stress history paths are shown in figures 13, 14 and 15.

The effect of the rate of strain, mainly on the pore pressure in the consolidated undrained tests, has been dealt with by A.W. Bishop and D.J. Henkel[3] and others. The influence of the strain rate has been attributed to three main causes.

There is always a time lag in the response of the pore pressure device to changes in pore pressure. The time t that is required for small out of balance pressure Δp to lead to a displacement Δx of the water-mercury surface in the null indicator has been given as,

$$t = \frac{\pi}{4} \gamma_\omega \frac{1}{K\, C_c} \left(\frac{d}{D}\right)^4 \left(\frac{\Delta x}{\Delta p}\right)^2$$

where γ_ω = density of water

K = coefficient of permeability

C_c = " " compressibility under an equal all round stress

d = diameter of small-bore tube in null indicator

D = diameter of sample.

This expression has been derived assuming a saturated sample and not considering filter strip drains. The presence of filter strip drain paths causes a reduction in the time t. The above expression shows the relative importance of the soil properties permeability and compressibility in estimating the sensitivity of the null indicator type of pore pressure device. In soils where the permeability and compressibility are both not very low this type of device has been found to be sufficiently sensitive. The Winnipeg clays have low permeability but high compressibility. The influence of the rate of strain, due to time lag, can be ignored in the case of the soils tested. As shown earlier when discussing the test to determine the pore pressure parameter B, the pore pressure measuring device appeared sensitive and reliable. Also from the pore pressure-strain curves it is seen that the pore pressure practically in all cases attained a maximum value as failure was approached and the variation of pore pressure close to failure was very small. Therefore the effects of a time lag, if any, would be negligible.

More important than time lag is the pore pressure distribution within the sample. Non uniform pore pressures exist within a sample due to the end restraints and other factors that give rise to non-uniformity in the applied stresses. In undrained triaxial tests the pore pressure equalization within a sample will depend on permeability, sample dimensions and the rate of strain. The use of filter strips provides drain paths over the surface of the sample and accelerates the equalization of pore pressure. However, for any particular test the sample dimensions and conditions of drainage being roughly the same, the pore pressure gradients within the samples will depend primarily on the strain rate. Where the pore pressure is measured at the base of the sample it is necessary that the measured pore pressure be equal to the pore pressure in the failure zone. Therefore in the undrained test with pore pressure measurement the rate of strain used should ensure equalization of pore pressure within the sample. Both theoretical considerations and practical results have shown that the degree of equalization of the pore pressure within a sample increases as the time to failure is increased.

The deviator stress vs. strain curves for the Winnipeg clay tested as shown in figures 7, 8 and 9 are similar in shape for all three rates of strain. Where the fastest of the three strain rates (3.5% per hour) was used the deviator stress at failure was less than in the case of the two slower rates of strain. This is different from what has been observed for a number of soils where the maximum deviator stress has been found to increase somewhat with increasing rate of strain - A. Casagrande and S.D. Wilson[6], C.B. Crawford[8], A.M. Richardson and R.V. Whitman[15], T. C. Kenney[11]. But for strain rates (b) and (c) the maximum deviator stress at failure is very nearly equal in both cases for any one particular confining pressure. Whatever slight differences that exist might very well be due to slight differences in the soil samples.

The pore pressure vs. strain curves for the two rates of strain (b) and (c) are similar. The maximum pore pressure (at failure) for the same confining pressure is somewhat different in the two cases. At a confining pressure of 80 lb. per sq. in. the maximum pore pressure recorded was 20.2 lb. per sq. in. for strain rate (b) and 24.2 lb. per sq. in. for (c) whereas at 40 lb. per sq. in. confining pressure the maximum pore pressure was higher in the case of strain rate (b). These differences don't show any particular relationship to the rate of strain. The pore pressure vs. strain curves in these two cases are close enough to attribute any differences to a slight non-uniformity of the soil samples. Where the strain rate (a) was used the pore pressure vs. strain curves were different from those of the other two cases. The maximum pore pressure recorded (at failure) was higher than when strain rates (b) and (c) were used. This again is different from what has been observed in the case of many soils, where the magnitude of the induced pore pressures has been found to increase as the rate of strain decreased.

The Mohr rupture envelopes for the clay at the three different rates of strain are shown in figures 13, 14 and 15. For tests run at strain rate (a) ϕ' is less than that for the strain rates (b) and (c); 8.8° for (a) as compared with 12.9° and 13.2° for (b) and (c) respectively. The values of ϕ' at strain rates (b) and (c) are more compatible with the results of the consolidated drained tests, than ϕ' at strain rate (a). Consolidated drained tests show ϕ' to be about 14° to 15°.

From all these it is evident that a strain rate of 3.5% per hour or a time to failure of 1 to 2 hours was not suitable for the consolidated-undrained test on the Winnipeg clay tested. A rate of strain of about 1.5% per hour or less may be considered suitable. It is difficult to state what caused lower deviator stresses and higher pore pressures at a faster strain rate than at slower rates of strain. It is possible that the pore pressure at the base of the sample was higher than in the middle when a strain rate (a) was used. A third factor that influences pore pressure and deviator stress as the rate of strain is varied may be mentioned in this connection. It is the modification in the behaviour of the soil structure at different rates of strain.

The fact that ϕ' and C' were somewhat different at the faster strain rate (a) from the values at the slower strain rates (b) and (c) might be an indication of this.

So far in the discussion the possibility of pore pressures being affected by leakage of water from the cell chamber into the sample, was not considered. The usual precautions taken in triaxial testing to prevent this (mentioned when describing "procedure") were taken. These have been found to be satisfactory and the possiblity of any significant leakage may be discounted.

Where undisturbed samples of clay are used to compare the effect of different strain rates the necessity to use specimens from the same block sample to assure uniformity in the clay samples, places a restriction on the number of tests that could be run. Fortunately in the case of the soil tested fairly uniform block samples were available and a few tests could be used with enough justification to observe the effects of different rates of strain in the consolidated undrained tests. The conclusion that a strain rate that gives a time to failure of about 4 to 6 hours or more is suitable in the case of the Winnipeg clays, appears reasonable on the basis of the results.

Mohr envelopes

These are shown in figures 13 to 19. Neglecting the one case of the set of consolidated undrained tests where the strain rate of 3.5% per hour can be considered too rapid to give reliable results, all other tests give Mohr envelopes that vary very little. The consolidated undrained tests at strain rates of approximately 1.5% per hour and 0.9% per hour give ϕ' values of 12.9° and 13.2° respectively, which from an applied engineering point of view, are the same. Average ϕ' from the overconsolidated undrained tests was 12.0°. In the consolidated-drained tests where the minor principal stress σ_3 was kept constant and the major principal stress σ_1, was increased ϕ' values of 14.0° and 14.8° were obtained for soil from depths of 20 ft. and 25 ft., respectively; where σ_1, was kept constant and σ_3 decreased ϕ' was equal to 15.2°, for the clay from 25 ft. depth.

The presence of a high cohesion intercept can be seen from the results. C' is about 9.0 lb. per sq. in. from the consolidated-undrained tests and about 6.0 lb. per sq. in. average, from the consolidated-drained tests. Thus it is seen that for the Winnipeg clays a higher cohesion intercept in terms of effective stresses is obtained from the consolidated-undrained tests than from the consolidated-drained tests. This agrees with what has been observed by J.D. Mishtak[12] in the case of a Winnipeg clay.

The average shear strength envelope for effective stresses as obtained by consolidated-undrained tests is shown in figure 19 along with those reported by J.D. Mishtak[12] and C.B. Crawford[7], for the same type of soil, samples of which were taken from the Red River Floodway site. Also included in table 7, and figure 21 are the results of some triaxial tests done on a Winnipeg clay by J. Peters as a part of his still unfinished

M. Sc. thesis, at the University of Manitoba. In all cases the tests on the soil from the Floodway site were done on specimens taken from block samples. The tests on the soil from the Searle Grain Elevator site were done on specimens taken from shelby tube samples.

All results clearly indicate that the angle of shearing resistance ϕ' for Winnipeg clays is somewhat low. They also show the presence of a high cohesion intercept.

Results obtained by J. Peters from tests run at a rate of strain of 0.9% per hour show almost a constant value of ϕ' and for all purposes no variation in ϕ' with depth (Table 7) ϕ' equal to 14.9° obtained for soil samples from a depth of $7\frac{1}{2}$ to 9 feet can be ignored because at this shallow depth the soil may not be sufficiently uniform in its properties or representative of the clay strata in the Winnipeg area. The results obtained from tests run at different rates of strain (Crawford, Samarasingha, - table 7) show a variation of ϕ' with the rate of strain from 8.8° at 3.5% strain per hour to 13.2° $\not+$ 0.9% strain per hour. Assuming that ϕ' values may be compared for different Winnipeg clays under identical rates of testing, a graph of ϕ' against rate of strain may be plotted (figure 22). This roughly shows an increase in ϕ' with decreasing rate of strain.

No such comparison can be made in the case of "cohesion" C', as it depends on the amount of pre-consolidation and the degree of disturbance etc.

CHAPTER VI

CONCLUSIONS

The results were discussed in detail in the previous chapter. On the basis of these results the following conclusions may be summarized, about the soil tested which was a typical clay in the Winnipeg area.

There is a low angle of shearing resistance and a high "cohesion intercept". The angle of shearing resistance is about 13° as given by the consolidated-undrained test and only slightly lower by 1 to 2°, than that as given by the consolidated drained test. Where ϕ' is required under drained conditions it is possible to use the undrained test with pore pressure measurement rather than the drained test which takes several days.

The pore pressure parameter A_f (at failure) lies between $\frac{1}{4}$ and $\frac{1}{2}$. From the pore pressure characteristics, vector curves etc., the clay can be described as being lightly over-consolidated. For the over-consolidated clays (maximum over-consolidation ratio = 8) A_f varied between $\frac{1}{8}$ and $\frac{1}{4}$. The limited variation of A_f under different conditions would justify its use in practical problems, where it is measured in the laboratory under conditions similar to those in the field problem.

The rate of strain affects the test results in the consolidated undrained tests. Higher deviator stresses and lower pore pressures result from decreasing the strain rate. For reliable results a strain that gives a time to failure of 4 to 6 hours or more appears satisfactory.

The two failure criteria, the maximum deviator stress and the maximum principal stress ratio coincided in all tests. The maximum deviator stress was usually reached about 6% strain and failure planes developed after that. Therefore it is not possible to use the multi-stage test using one sample to obtain a Mohr rupture envelope.

A comparison of the test results with other reported results of triaxial tests on clays in the Winnipeg area shows that these clays have fairly uniform properties.

Type of drainage:- Top — Bottom —— Filter strip ——

Cell no. 3 Frame no. 1

CONSOLIDATION DATA

Date	Time	Elapsed Time	Pan Load(lb)	Cell Pressure (p.s.i)	Temp. F	Bottom Drainage Water(cc)	Bottom Drainage Air	Top Drainage Water(cc)	Top Drainage Air	Dial Reading in.
26-1-65	1520	0		40		41.20	—	42.70	—	.5000
		1 m				40.90		42.30		.5030
		2 m				40.80		42.20		.5040
		5 m				40.60		42.00		.5070
		10 m				40.40		41.80		—
		30 m				40.00		41.40		.5140
		1 h				39.60		41.10		—
		2 h				39.40		41.00		
27-1	1520	24 h				39.00		40.70		.5290
28-1	"	48 h				39.20		40.40		.5300

SAMPLE DIMENSIONS

Diameter, Top in. 1.43
Middle in. 1.42
Bottom in. 1.40
Average in. 1.42
Average Height in. 2.83
Cross-Sectional Area (a_o) in^2 1.58

Specific gravity -------- 2.76
Volume of sample ---- cc 73.4
Volume of soil solids cc 27.6
Volume of voids ----- cc 45.8
Degree of saturation -- 98
Density Moist lb / cu ft. 103
Density Dry
Void Ratio --------- 1.66

Volume change (Δv) = 4.30 cc
Length change (Δl) = 0.03 in.
Corrected area(a_c) = $a_o \dfrac{(1 - \Delta v/v)}{(1 - \Delta l/l)}$ in^2

SKETCH AT FAILURE

$\alpha \simeq 45^\circ$

MOISTURE CONTENTS

Sample Location	After Failure: Top	After Failure: Failure Plane	After Failure: Bottom	After Failure: Entire	Before Consolidation: Entire Sample
Container no. ----------------	2	Z	1	1	1
Wt. container + moist sample gm	97.24	62.91	58.60	155.80	158.92
Wt. container + dry sample -- gm	76.05	62.91	51.10	—	—
Wt. moisture -------------- gm	21.19	12.70	7.50	—	44.81
Wt. container ------------- gm	36.25	39.70	37.42	37.42	37.42
Wt. dry sample ------------ gm	39.80	23.21	13.68	—	Σ = 76.69
Moisture content --------- %	53.4	55.0	54.8		58.3

Sample Description
grey clay, easy to handle

Sample no.
Depth ft. 20
Test hole no.

Project Thesis

Tested by N.S. Date 26/1/65

SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

$L_c = 2 \cdot 80''$

ELAPSED TIME IN min. sec.	CHAMBER PRESSURE IN lbs./sq. in. OBSERVED	CORRECTED	COUNTER	PROVING RING DIAL IN .0001 in.	LENGTH CHANGE, ΔL IN in.	STRAIN ε IN %	AREA, A, IN sq. in.	AXIAL LOAD P IN lbs.	AXIAL PRESSURE, $p = \frac{P}{A}$ IN lbs./sq. in.	PORE PRESSURE OBSERVED cm Hg	OBSERVED IN lbs./sq. in.	CORRECTED IN lbs./sq. in.	$\bar{\sigma}_3$ IN lbs./sq. in.	$\bar{\sigma}_1$ IN lbs./sq. in.	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	τ_{ff}
0		40		0	.0000	0	1.505	0	0	—		0	40	40	—	—
600				15	.0033	0.118	1.506	—	—			—	—	—		
1200				23	.0061	0.218	1.509	6.4	4.3			—	—	—	—	
1800				55	.0088	0.315	1.510	15.3	10.1							
2400				73	.0110	0.394	1.510	20.3	13.5			1.5	38.5	42.0	—	
3600				99	.0172	0.615	1.514	27.5	18.2			3.2	36.8	55.0		
4800				119	.0240	0.860	1.519	33.1	21.8			4.0	36.0	57.8	1.61	
6000				138.5	.0320	1.14	1.522	38.5	24.6			4.6	35.4	60.0		
7200				150	.0385	1.38	1.526	41.6	27.3			5.6	34.4	61.7	1.79	
8400				159.5	.0460	1.64	1.531	44.4	28.9			6.2	33.8	62.7		
9600				168	.0531	1.90	1.534	46.8	30.5			6.9	33.1	63.6	1.93	
10800				174.5	.0610	2.18	1.539	48.5	31.5			7.5	32.5	64.0		
12000				181	.0692	2.47	1.542	50.4	32.6			8.3	31.7	64.3	2.03	
13200				189	.0780	2.78	1.548	52.5	33.9			8.8	31.2	65.1		
14400				194	.0850	3.04	1.552	54.0	34.8			9.4	30.6	65.4	2.14	
15600				198	.0940	3.36	1.556	55.0	35.3			10.0	30.0	65.3		
16800				202	.1020	3.64	1.562	56.1	35.9			10.3	29.7	65.6	2.21	
17400				203.5	.1065	3.80	1.564	56.5	36.1			10.7	29.3	65.4		
18000				205	.1110	3.96	1.567	57.0	36.4			10.9	28.9	65.2	2.24	
18600				206.5	.1151	4.11	1.569	57.6	36.6			11.1	28.9	65.5		
19200				208	.1191	4.28	1.571	57.9	36.8			11.3	28.7	65.6	2.28	
19800				209.2	.1240	4.40	1.573	58.1	36.9			11.8	28.2	65.1		
20400				210.6	.1285	4.59	1.578	58.6	37.2			12.0	28.0	65.2	2.32	
21000				—	—	—	—		—			—	—	—		
21600				211.5	.1390	4.96	1.585	58.8	37.2			12.2	27.8	64.9	2.33	
22000				213.5	.1425	5.08	1.587	59.4	37.4			12.4	27.6	64.9		
22,800				214.3	.1470	5.25	1.589	59.6	37.5			12.5	27.5	65.0	2.36	
23,400				216	.1520	5.43	1.590	60.0	37.6			12.6	27.4	65.1		
24,000				217	.1565	5.59	1.596	60.3	37.8			12.7	27.3	65.1		
24,600				217.5	.1615	5.76	1.598	60.4	37.8			12.7	27.3	65.1	2.38	
25,200				218	.1680	6.00	1.601	60.6	37.9			12.7	27.3	65.2	2.38	

PROJECT: Thesis.

SAMPLE NO.

PLOTTED: ______ DATE: 29/1/65

REMARKS: Rate of strain= 0.00040 in per min

CHECKED: ______ DATE:

SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

Depth ft.	Type of test	Moisture contents w_i	w_t	w_{fp}	w_b	S %	Rate of strain in. per min.	σ_3 lb / sq in	$(\sigma_1 - \sigma_3)$ lb /sq in	u_{max} lb./sq in	e_f %	t_f mins.	ϕ'	c'	A_f
20	Consolidated Undrained	59.5	58.0	57.3	57.0	97.6	0.0015 (3.3 % / hr)	20	27.7	10.2	4.10	70	8.8	10.8	.365
		58.7	55.2	54.0	53.1	100		40	33.1	20.8	4.08	80			.605
		58.0	51.0	53.5	53.0	97		60	34.8	14.0	4.86	90			.402
		58.9	46.9	48.0	46.1	96.0		80	45.0	24.5	8.40	140			.550
20	"	57.8	55.8	56.5	56.0	95.0	0.00067 (1.5 % / hr)	20	29.8	8.0	2.80	110	12.9	9.3	.268
		57.9	55.0	56.4	54.9	96.0		40	37.5	16.2	5.28	225			.405
		58.0	54.0	57.5	57.0	96.0		60	36.6	11.5	6.30	220			.315
		59.1	56.0	55.1	54.8	96.0		80	58.8	20.2	5.73	225			.348
20	"	57.9	56.2	55.1	54.9	97.9	0.00040 (0.9 % /hr)	20	29.0	7.3	4.04	290	13.2	9.0	.252
		58.3	53.4	55.0	54.8	98.0		40	37.9	12.7	6.00	420			.335
		57.8	51.9	48.3	48.9	98.4		80	53.0	24.2	5.94	450			.457
20	Consolidated Drained (constant)	59.5	56.0	57.0	57.5	97.6		20	Failed by buckling						—
		57.6	47.0	48.9	48.4	97.0		40	43.6	—	9.55		14.0	7.0	—
		58.0	48.9	49.4	49.1	98.0		60	56.7	—	13.60				—
		58.0	44.0	43.0	45.0	96.0		80	69.7	—	16.80				—

Table 6 - Summary of test results

Depth ft.	Type of test	Moisture contents w_i	w_t	w_{fp}	w_b	S %	Rate of strain in. per min.	σ_3 lb/sq in.	$(\sigma_1 - \sigma_3)$ lb/sq in.	u_{max} lb/sq in.	e_f %	t_f	0	C' lb/sq in	A_f
25	Consolidated Drained (σ_3 constant)	55.2	51.4	53.7	49.8	97.0		20	25.4	—	4.5		14.8	5.0	—
		54.3	49.4	51.0	50.8	97.9		40	35.0	—	13.0				—
								80	57.8	—	16.0				—
								120		—	17.0				—
25	Consolidated Drained (σ_3 decreasin	53.8	44.2	42.0	42.1	95.0		102 to 50	52	—	10.6		15.2	6.5	—
		54.2	45.5	47.0	44.8	96.0		80 " 36	44	—	5.2				—
		54.2	48.1	51.0	53.8	95.0		60 " 28	32	—	4.5				—
		54.0	53.1	51.5	52.4	95.0		40 " 14	26	—	3.3				—
25	Over - consolidated Undrained	55.0	51.0	52.0	52.5	96.0	0.00040 (0.9 % / hr)	120 to 15	22.8	3.0	6.0	400	12.0	7.5	.131
		53.7	47.6	45.5	44.7	95.0		" 20	26.3	3.4	4.92	320			.131
		54.4	46.4	45.9	46.8	97.0		" 30	31.2	5.4	4.74	320			.173
		54.0	45.2	46.4	45.9	97.0		" 40	39.9	5.0	6.0	420			.125
		54.5	43.9	41.6	43.3	96.9		" 80	50.8	13.2	5.15	360			.260
		53.8	45.4	42.6	45.2	97.0		" 120	59.1	41.0	6.69	460			.710

w_i = initial water content
w_t = water content at the top, after failure .
w_b = " " " " bottom after " .
w_{fp} = " " in the failure zone.
S = degree of saturation.
e_f = strain at failure.
t_f = time to failure.

Table 6 (ctd.) - Summary of test results

PORE PRESSURE vs(1) CELL PRESSURE

(2) TIME (During cell pressure increase)

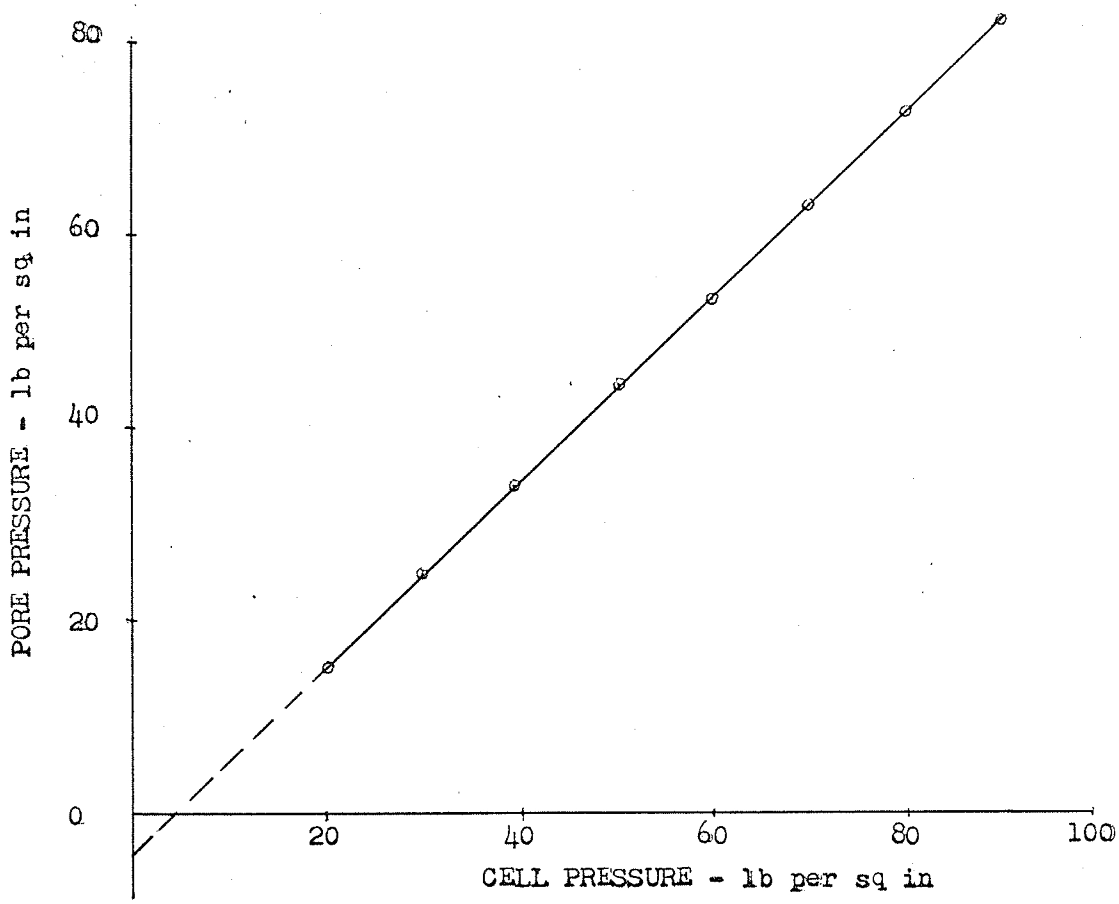

Fig. (6a)

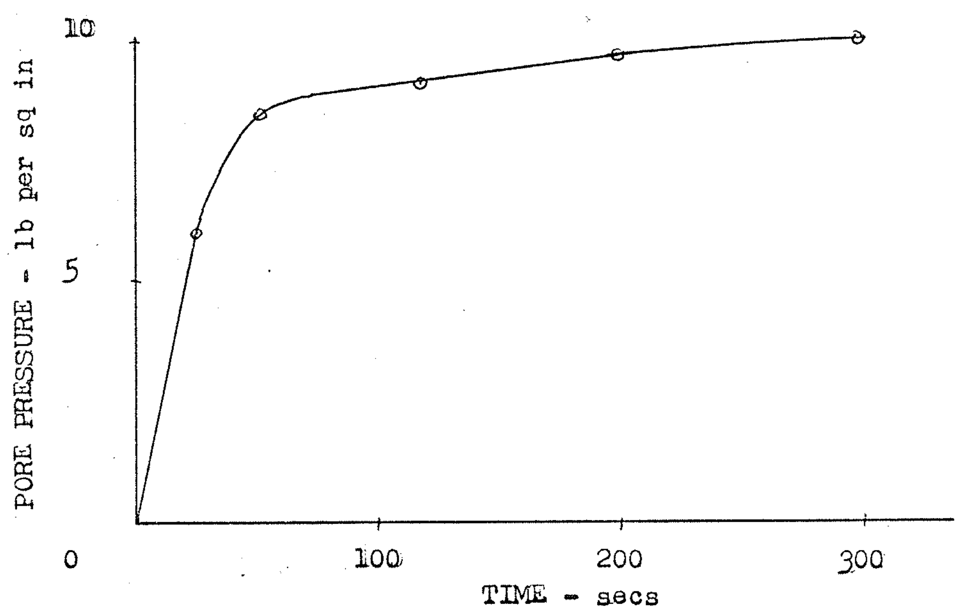

Fig. (6b)

DEVIATOR STRESS
VS. STRAIN
PORE PRESSURE

(Consolidated undrained test - rate of atrain 0.0015 in per min)

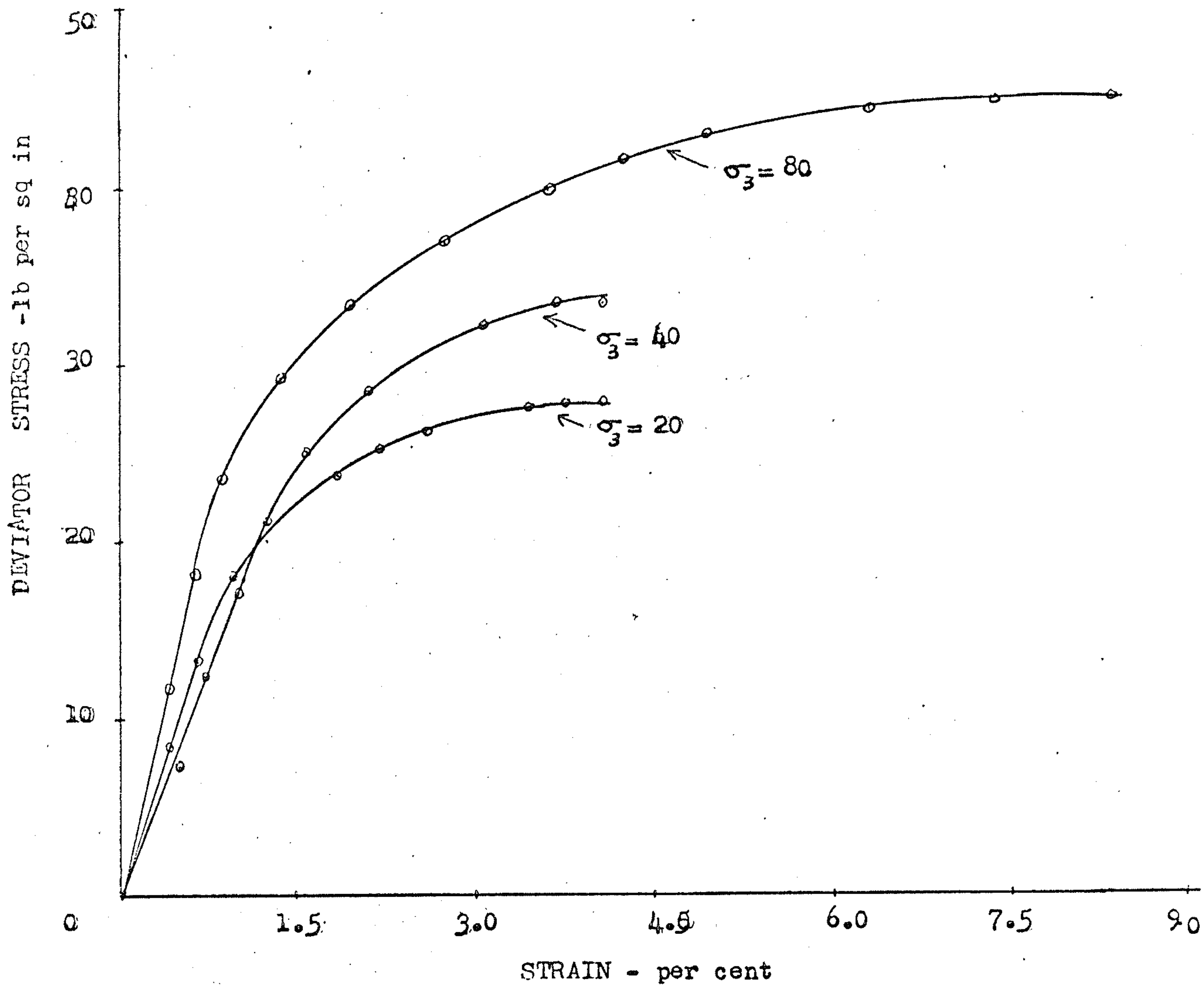

Fig. 7 (a)

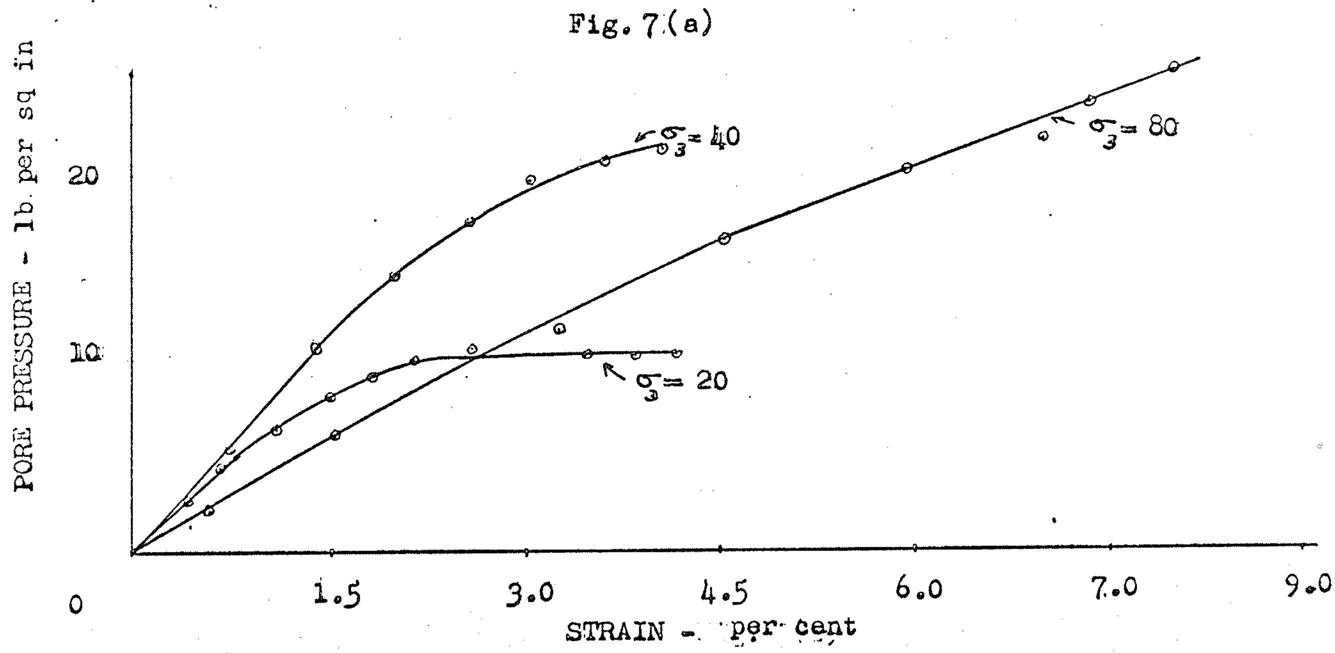

Fig. 7 (b)

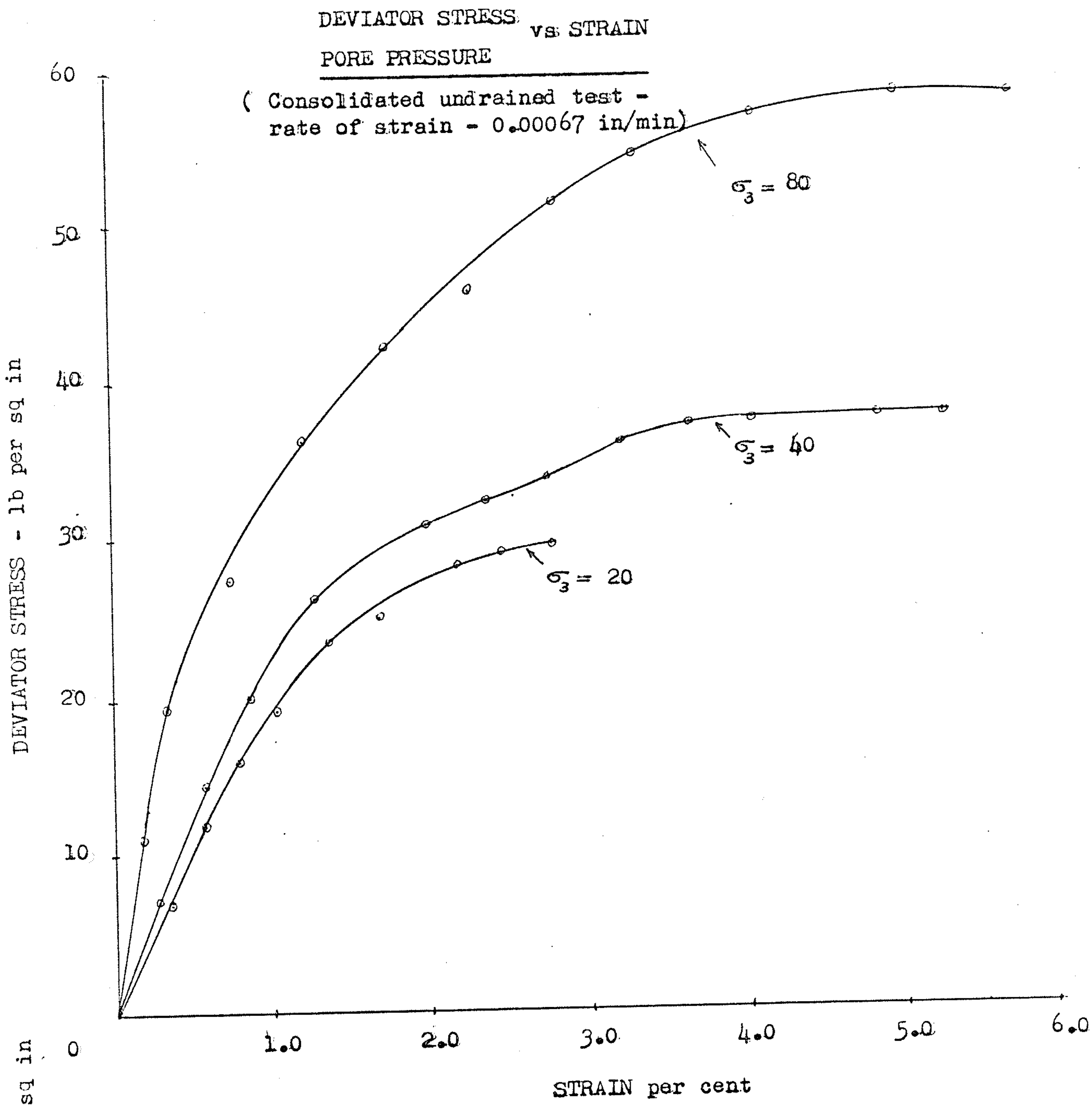
DEVIATOR STRESS vs STRAIN
PORE PRESSURE
(Consolidated undrained test - rate of strain - 0.00067 in/min)
$\sigma_3 = 80$
$\sigma_3 = 40$
$\sigma_3 = 20$
DEVIATOR STRESS - lb per sq in
60
50
40
30
20
10
0
1.0
2.0
3.0
4.0
5.0
6.0
STRAIN per cent

Fig. 8(a)

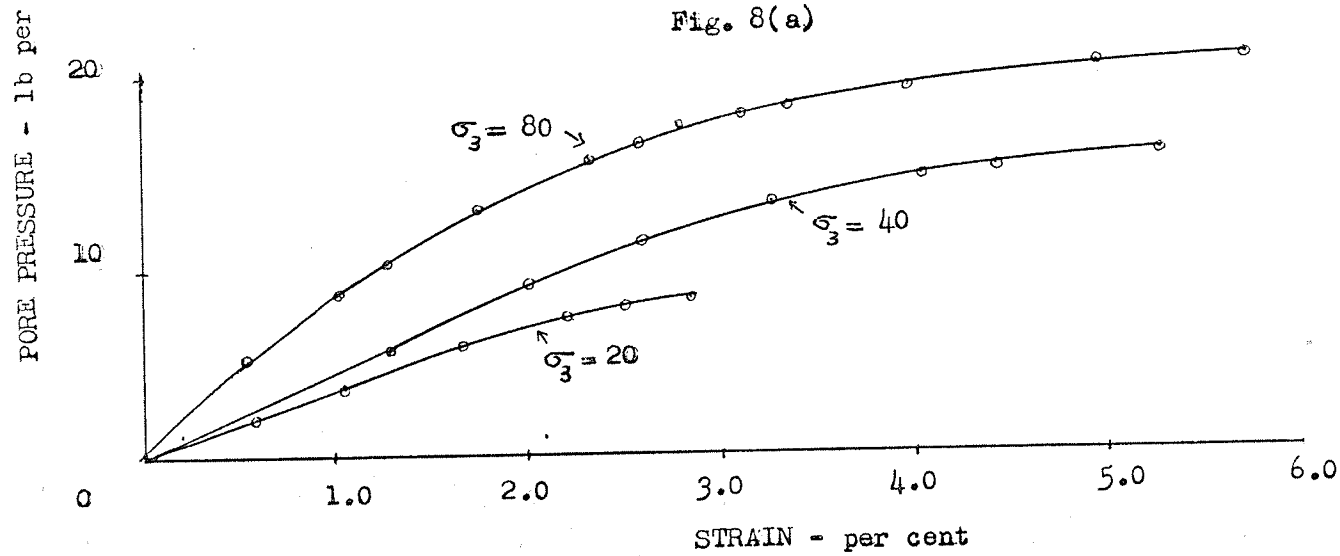
$\sigma_3 = 80$
$\sigma_3 = 40$
$\sigma_3 = 20$
PORE PRESSURE - lb per sq in
20
10
0
1.0
2.0
3.0
4.0
5.0
6.0
STRAIN - per cent

Fig. 8(b)

DEVIATOR STRESS VS STRAIN
PORE PRESSURE

(Consolidated undrained test
rate of strain - 0.00040 in/min)

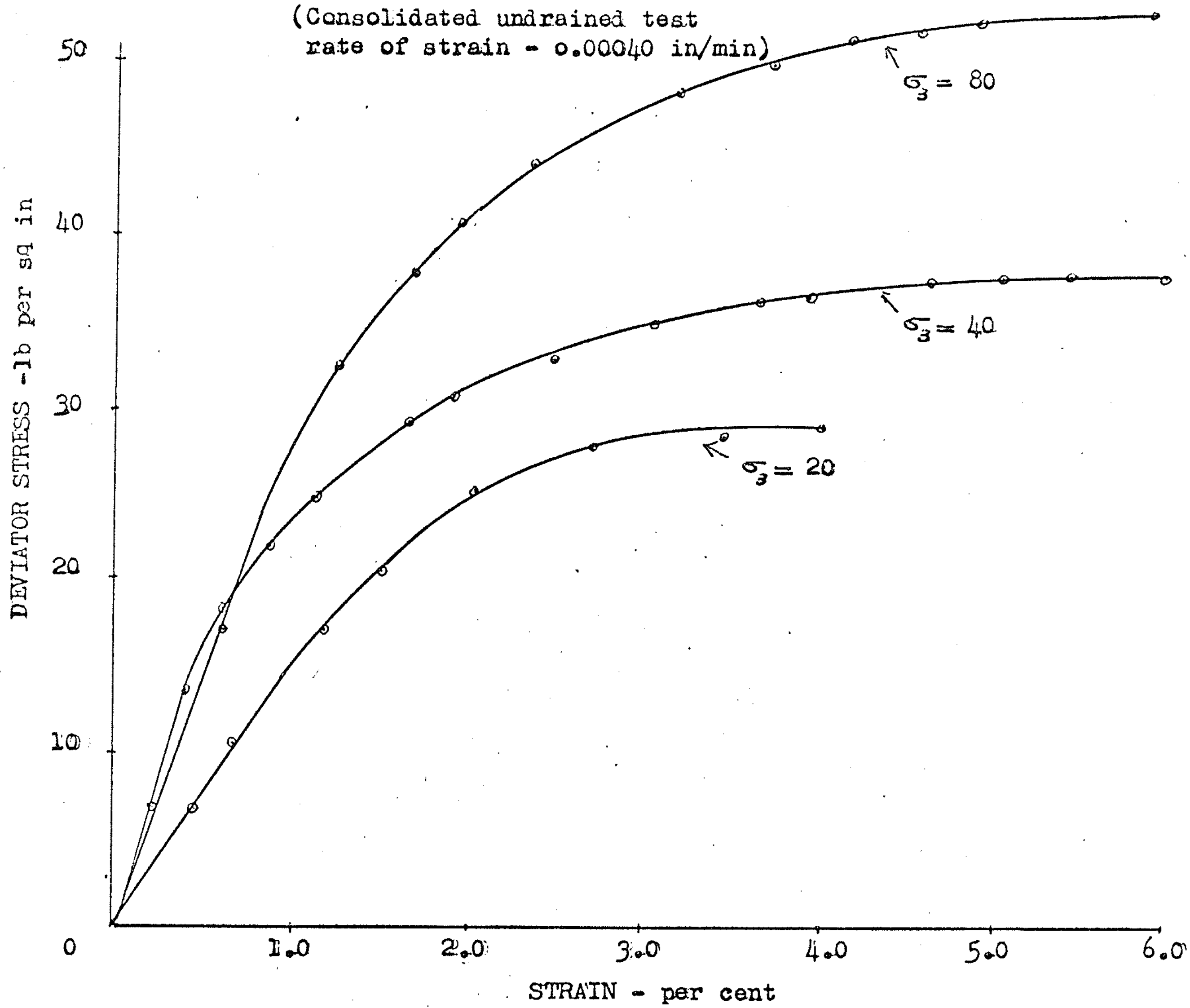

Fig. 9(a)

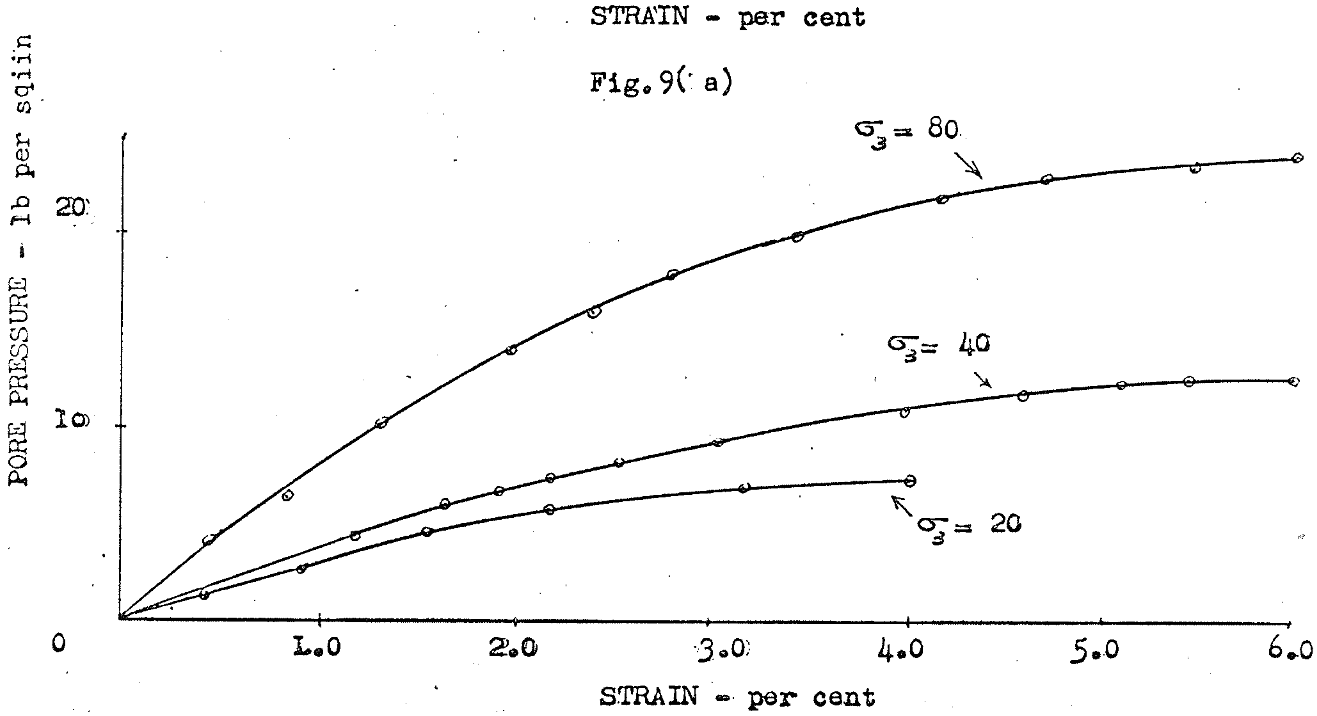

Fig. 9 (b)

DEVIATOR STRESS VS STRAIN

(Consolidated drained test)

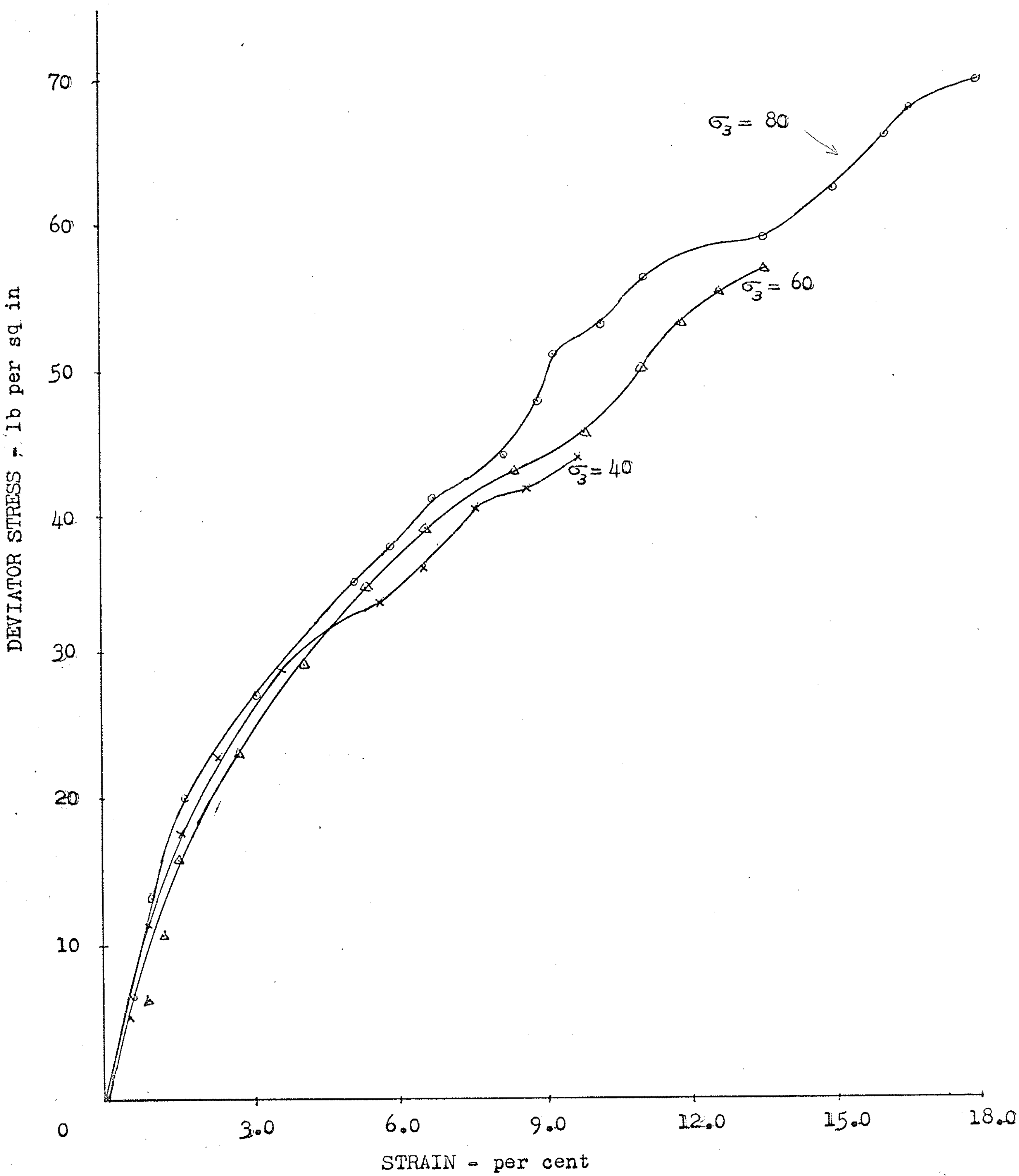

Fig. 10

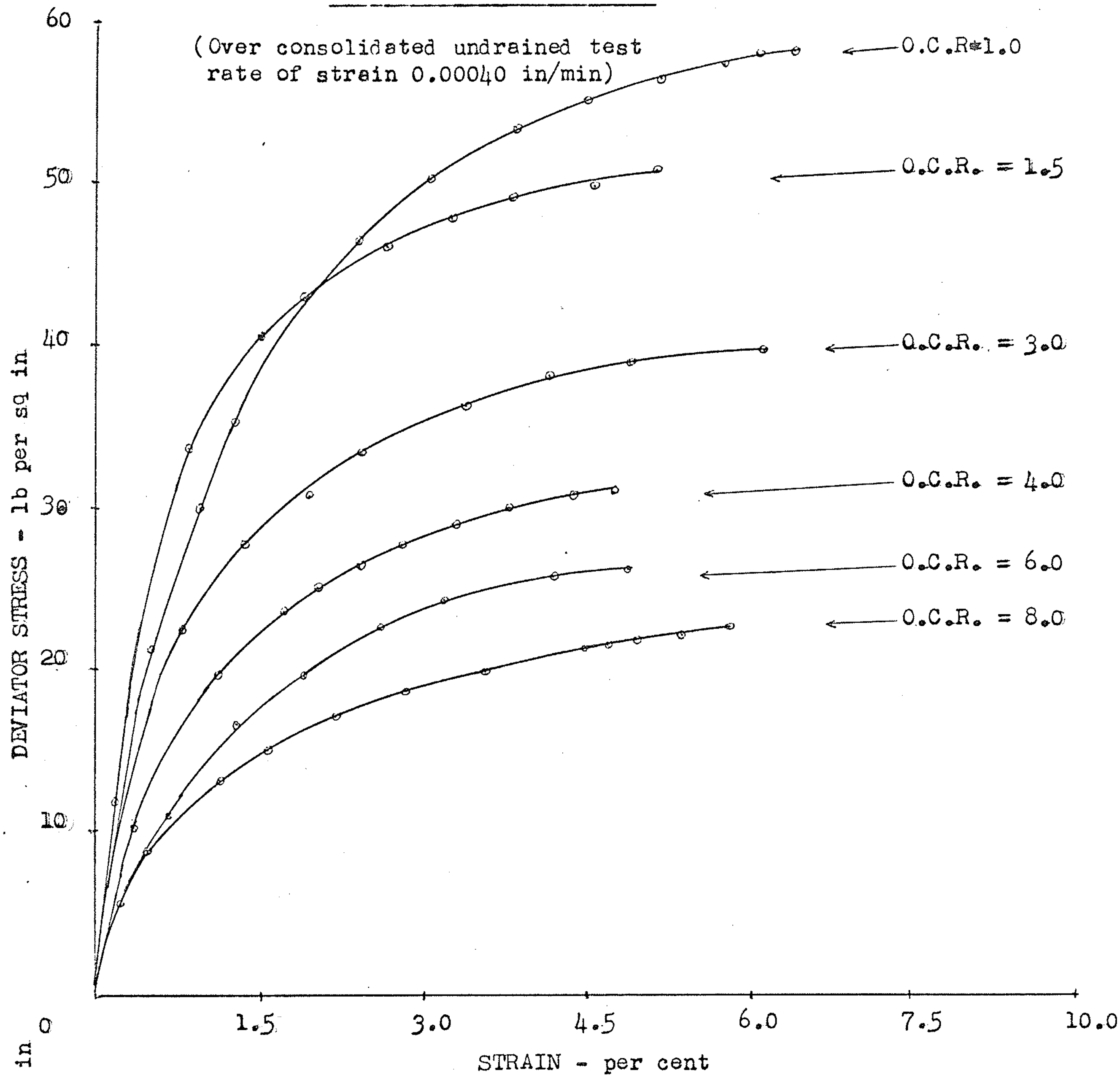

Fig.11 (a)

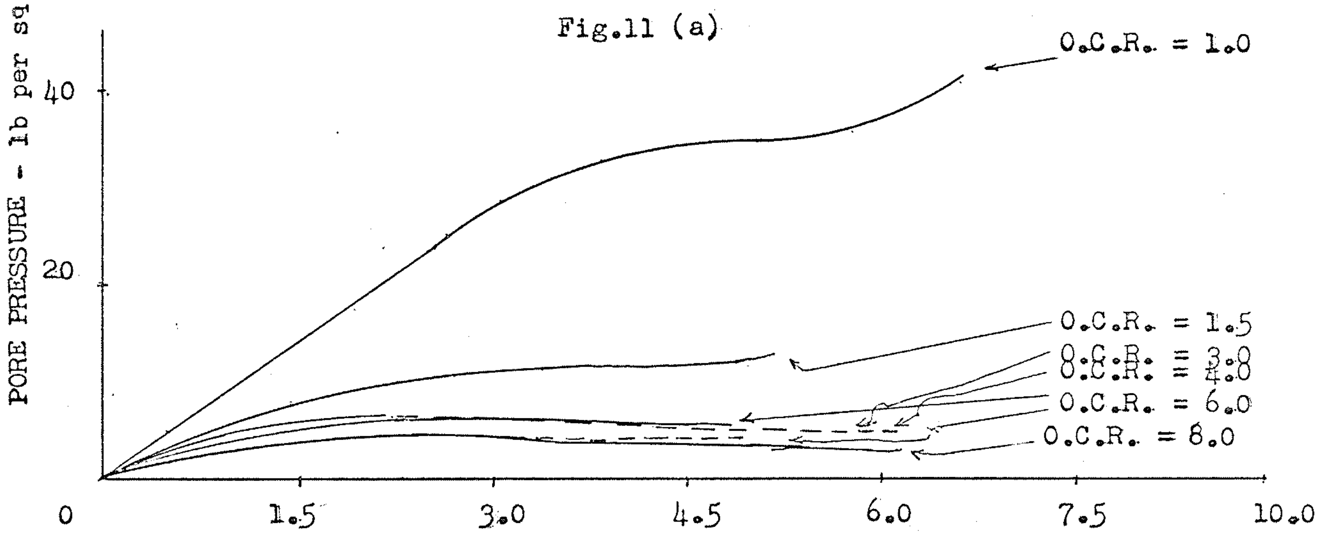

Fig.11(b)

DEVIATOR STRESS VS. STRAIN

(Consolidated drained test)

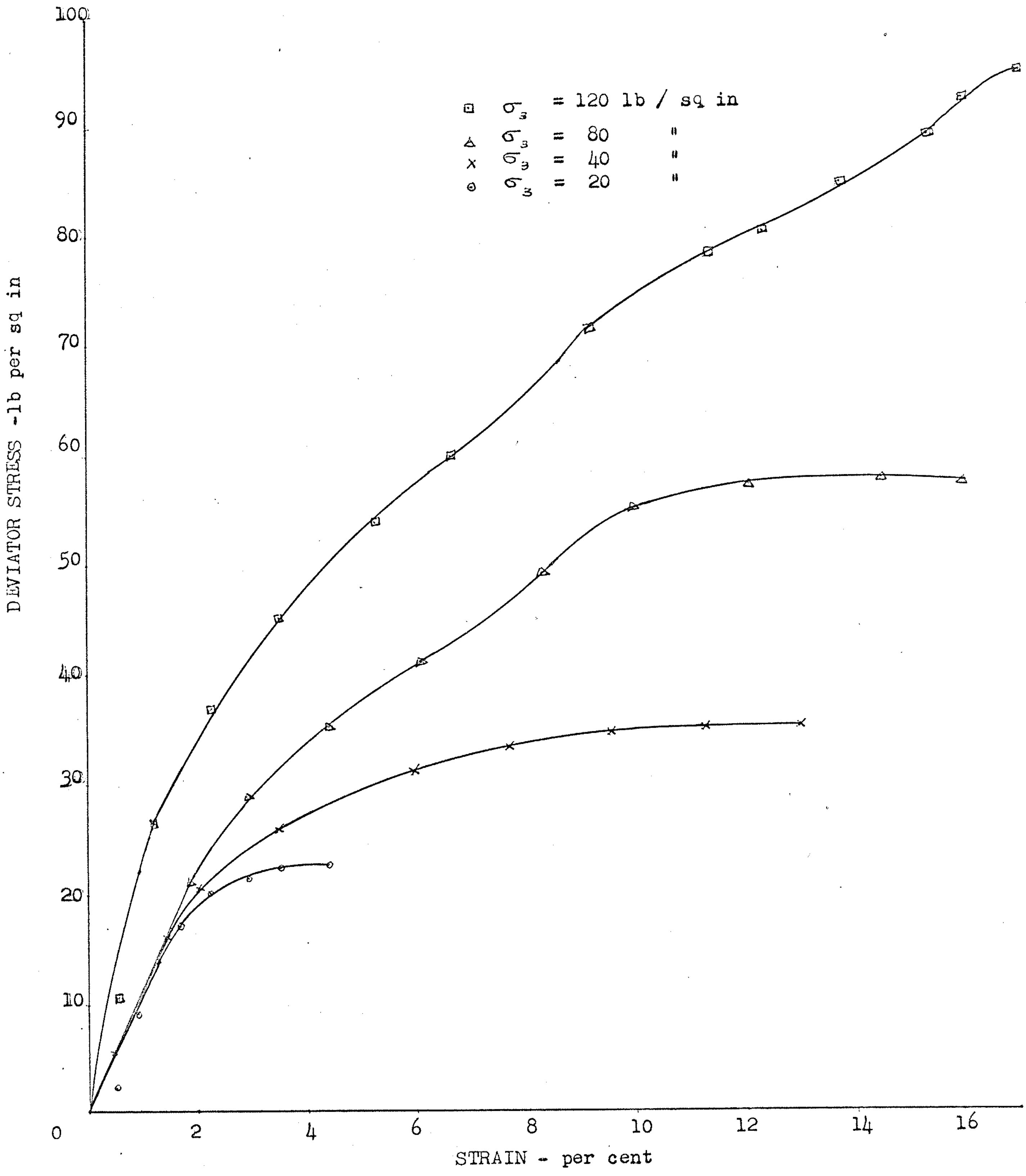

Fig.12

Mohr Rupture Envelope

(Consolidated undrained test - rate of strain 0.0015 in / min)

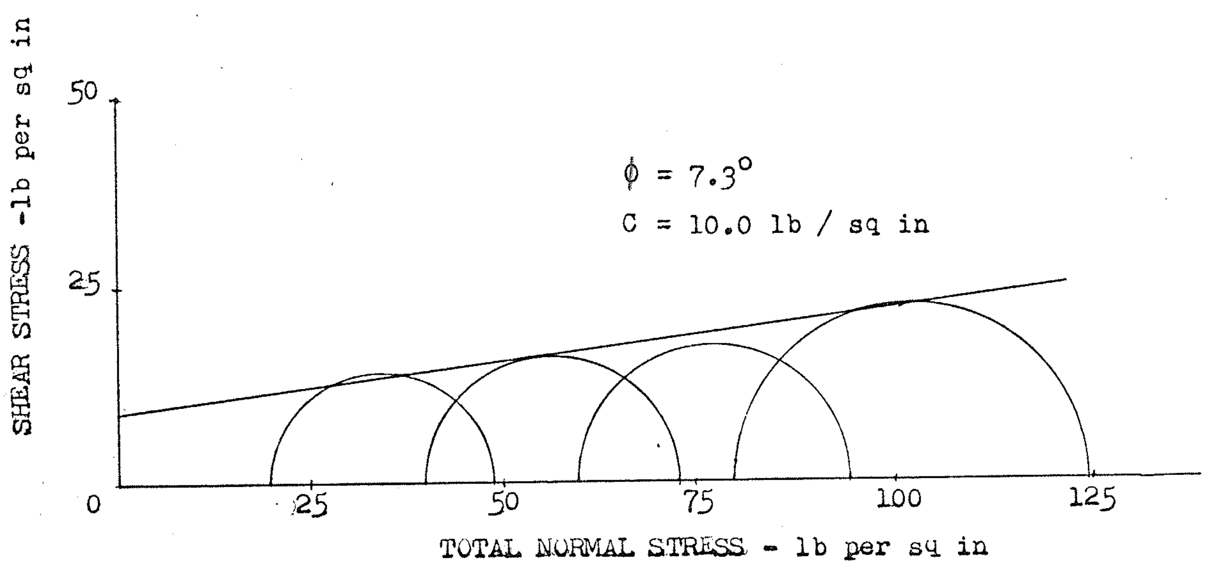

Fig. 13(a)

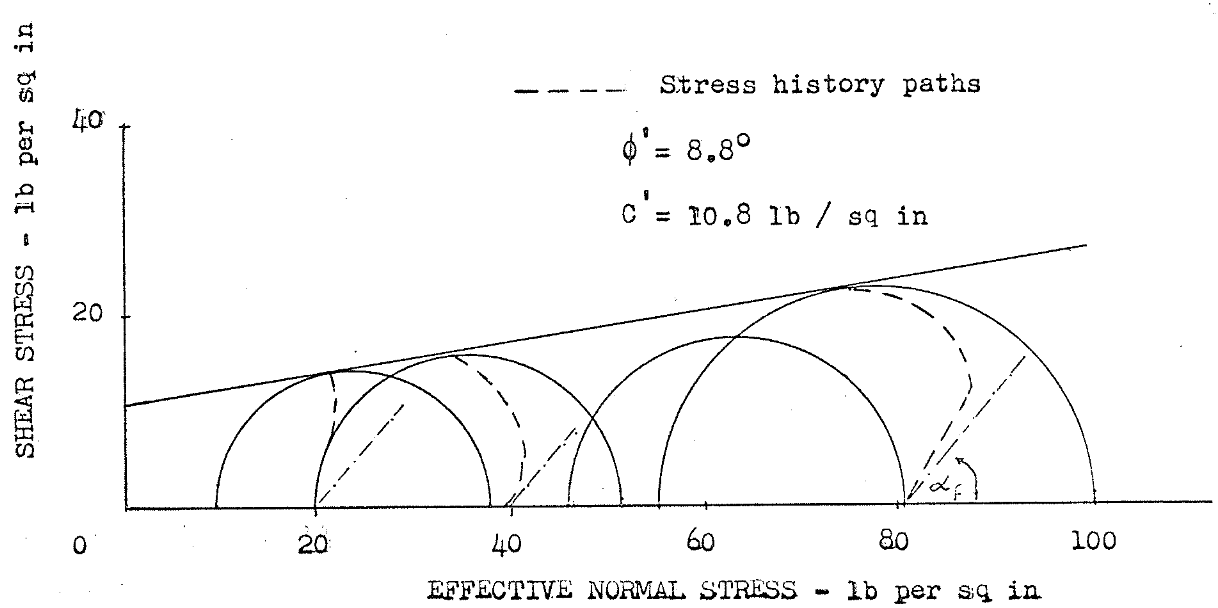

Fig. 13(b)

Mohr Rupture Envelope

(Consolidated undrained test - rate of strain 0.00067 in / min)

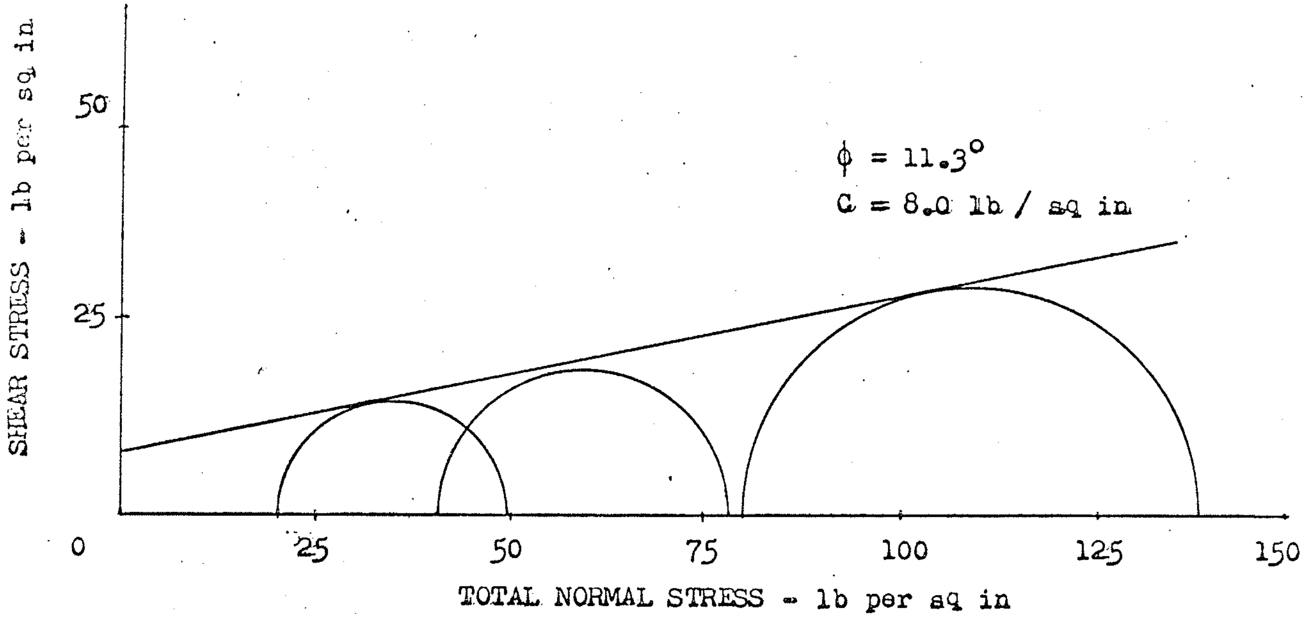

Fig. 14 (a)

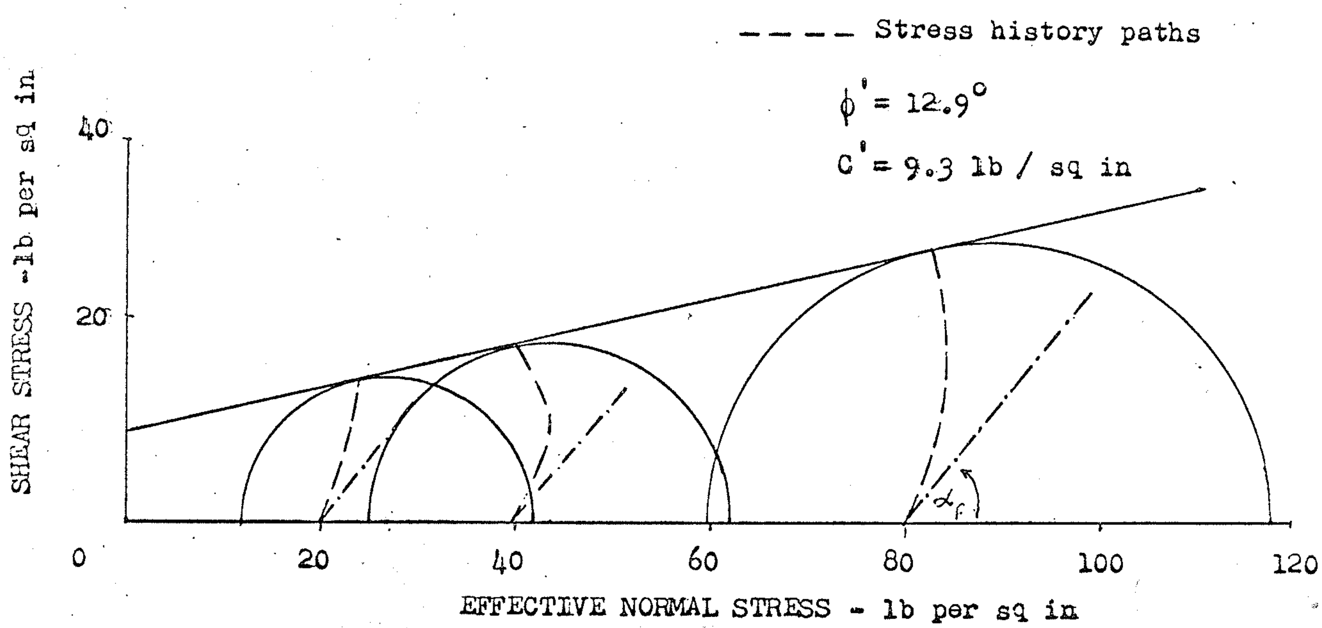

Fig. 14 (b)

Mohr Rupture Envelope

(Consolidated undrained test - rate of strain 0.00040 in per min)

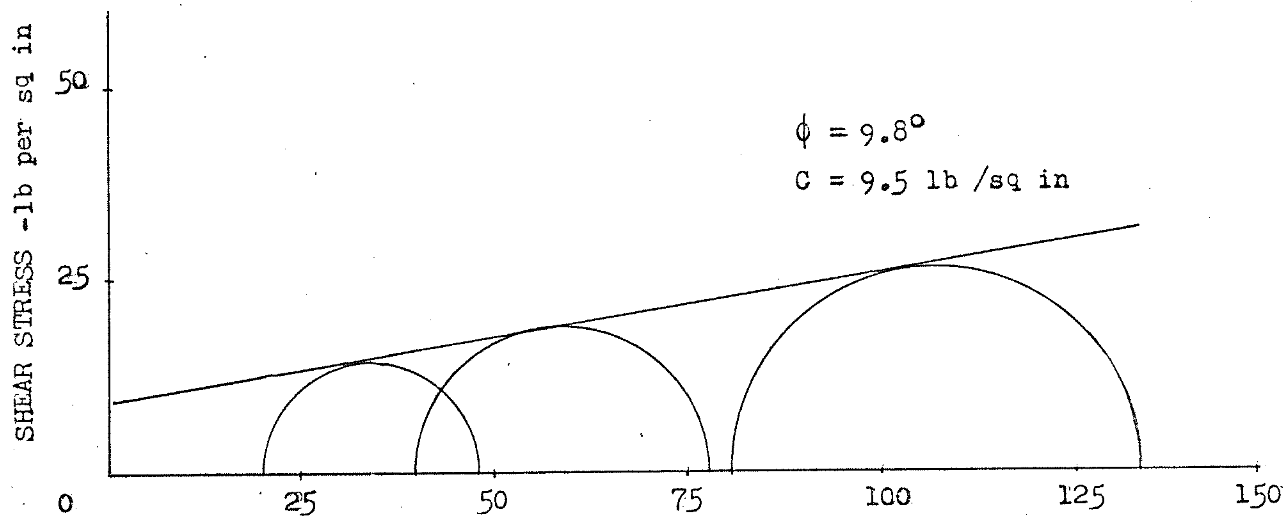

Fig.15 (a)

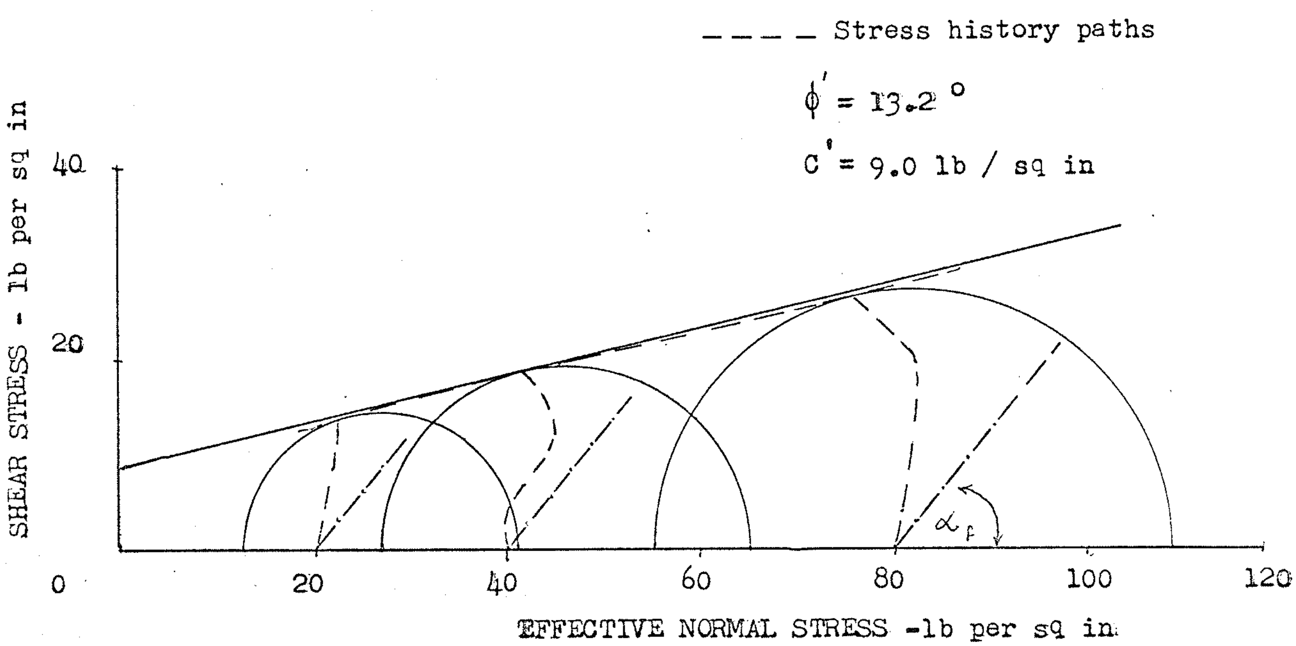

Fig. 15 (b)

Mohr Rupture Envelope

(Consolidated drained test)

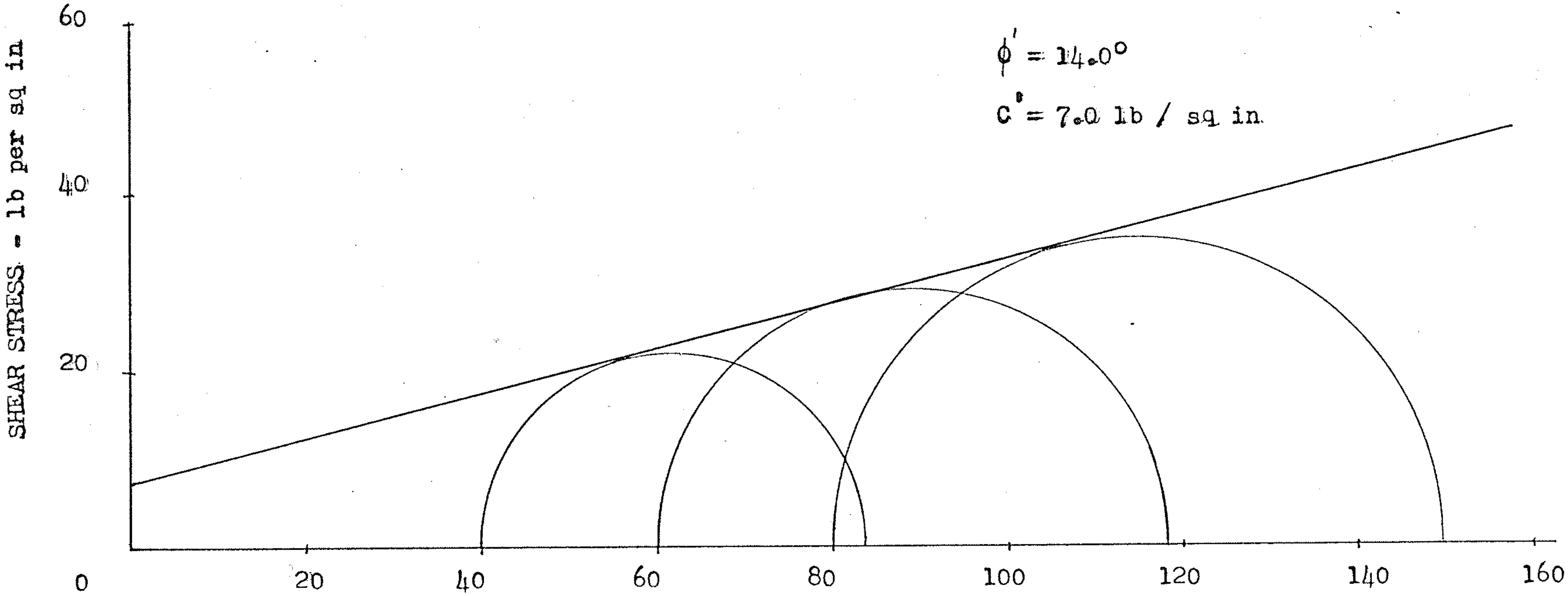

Fig. 16

Mohr Rupture Envelope

(Consolidated drained test)

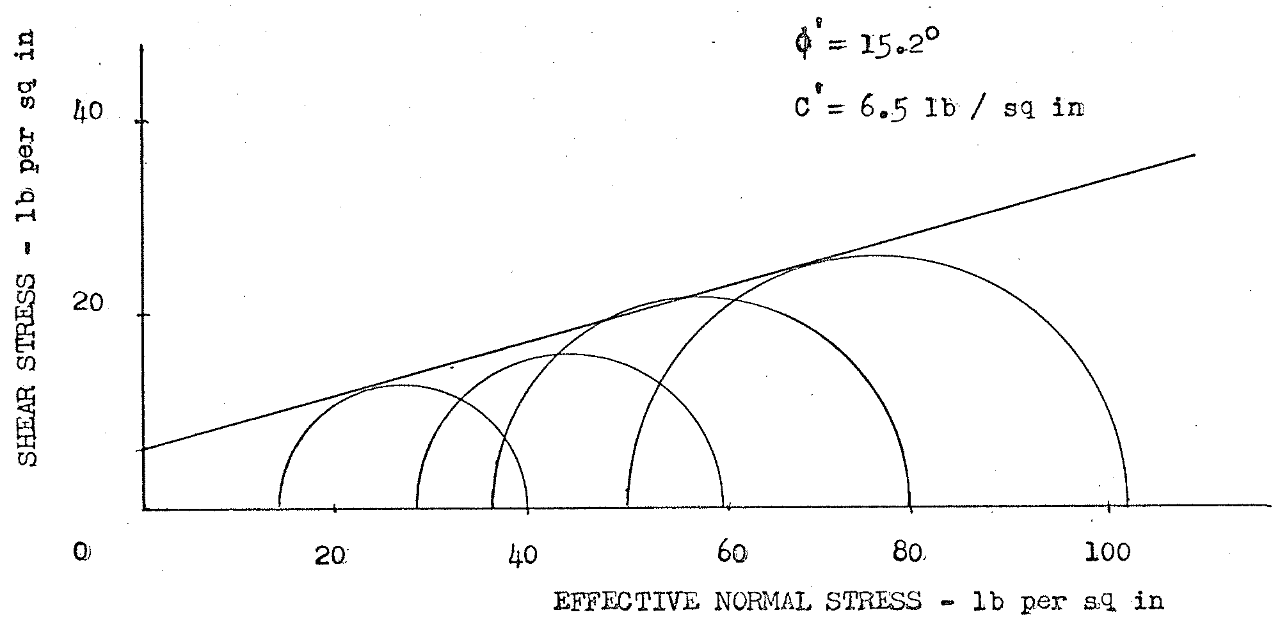

Fig. 17

Mohr Rupture Envelope

(consolidated drained test)

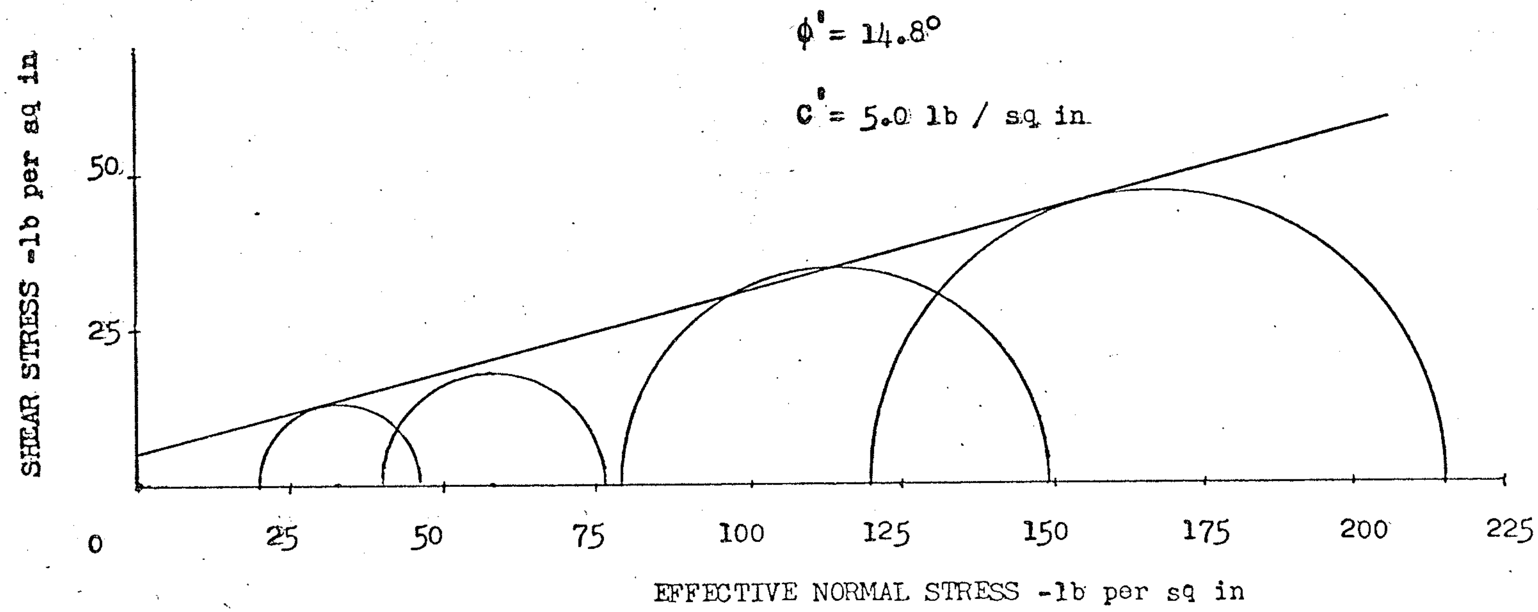

Fig. 18

Mohr Rupture Envelope

(over-consolidated undrained test- rate of strain 0.00040 in/mi

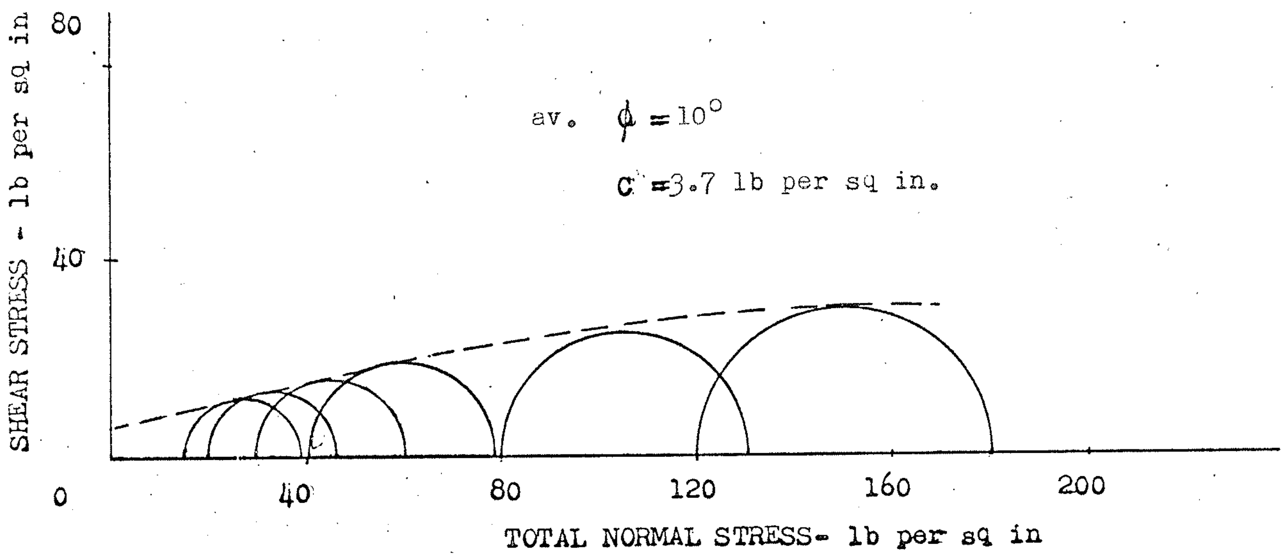

Fig. 19 (a)

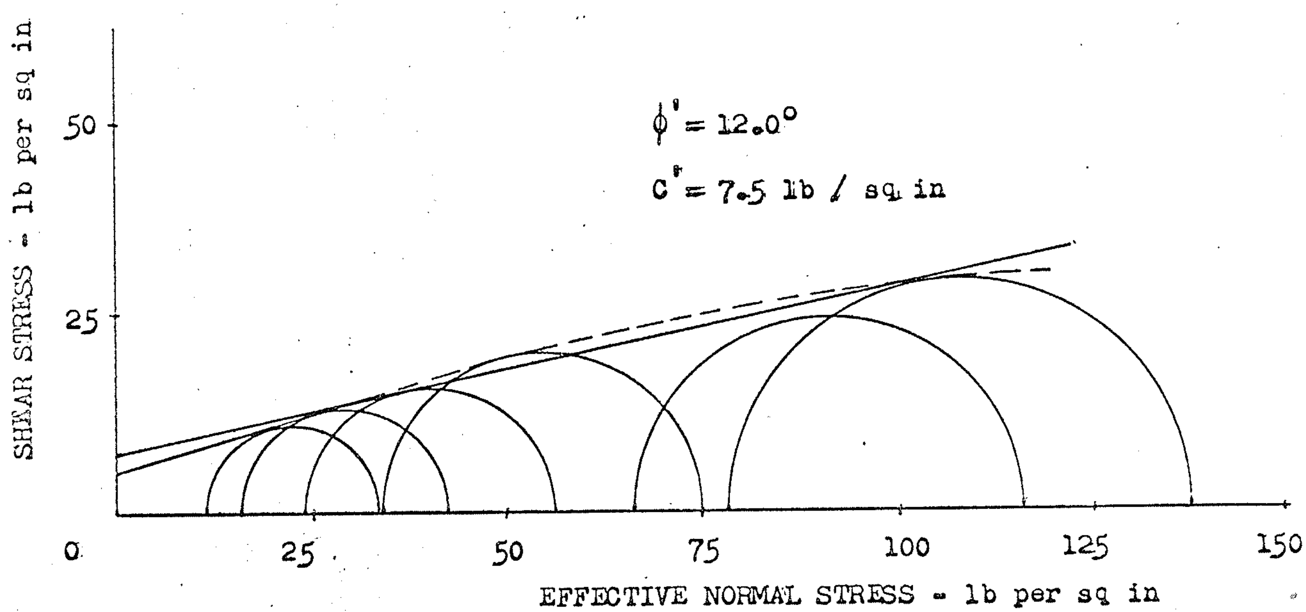

Fig. 19 (b)

(Stress history paths)

(Over-consolidated undrained tests)

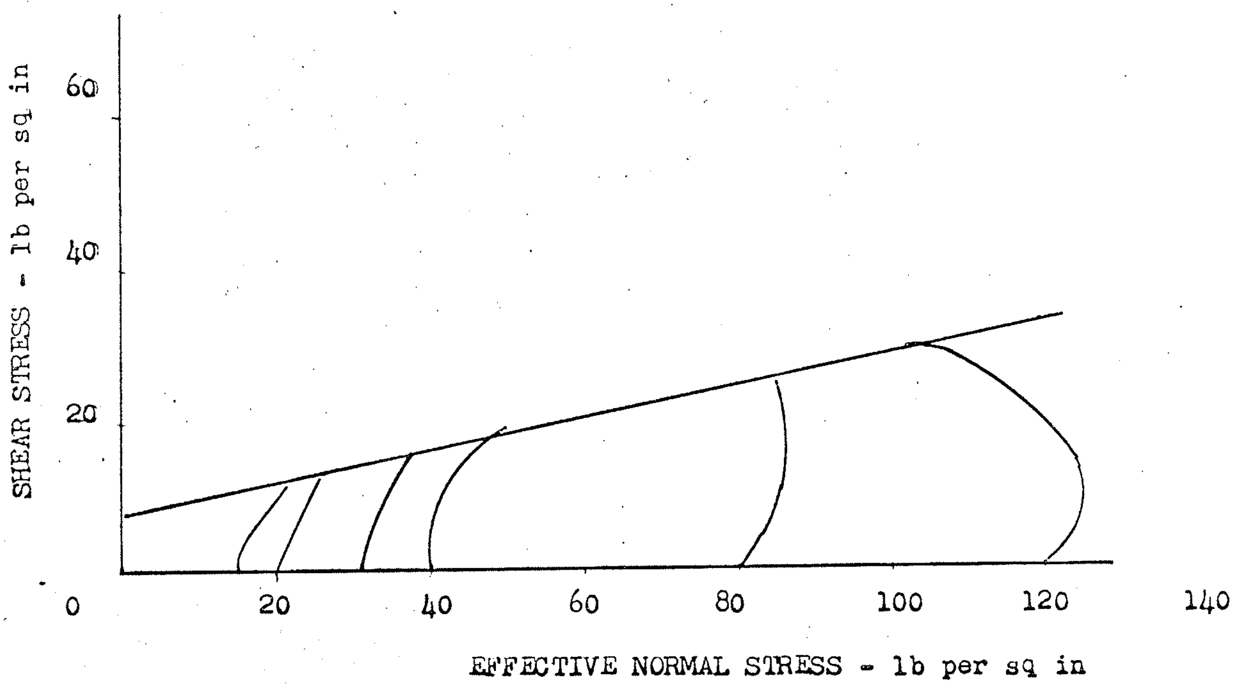

Fig. 20

	Site	Depth ft.	Average natural moisture content %	Liquid limit	Plastic limit	Clay fraction %	Rate of strain % per hr	Consolidated undrained case	
								ϕ'	c'
Mishtak	Red river floodway	30	54.8	95	21	-	-	12.0	6.5
Crawford	"	30	53.6	94	34	83	2.0	9.0	8.3
Peters	Searle grain elevator	7.5 - 9.0	42.1	96	30.1		0.9	14.9	4.3
		17.5 - 19	58.7	98.7	31.7		0.9	12.4	6.3
		27.5 - 29	58.0	76.4	25.8		0.9	11.8	5.2
		37.5 - 39	64.0	85.6	31.4		0.9	11.6	3.0
		47.5 - 49	62.0	81.7	32.4		0.9	11.5	3.7
Samarasingha	Red river floodway	20	58.1	114.5	40.5	84	0.9	13.2	9.3
		20	"	"	"	"	1.5	12.9	9.0
		20	"	"	"	"	3.3	8.8	10.8

Table 7 - Comparison of test results on Winnipeg clays.

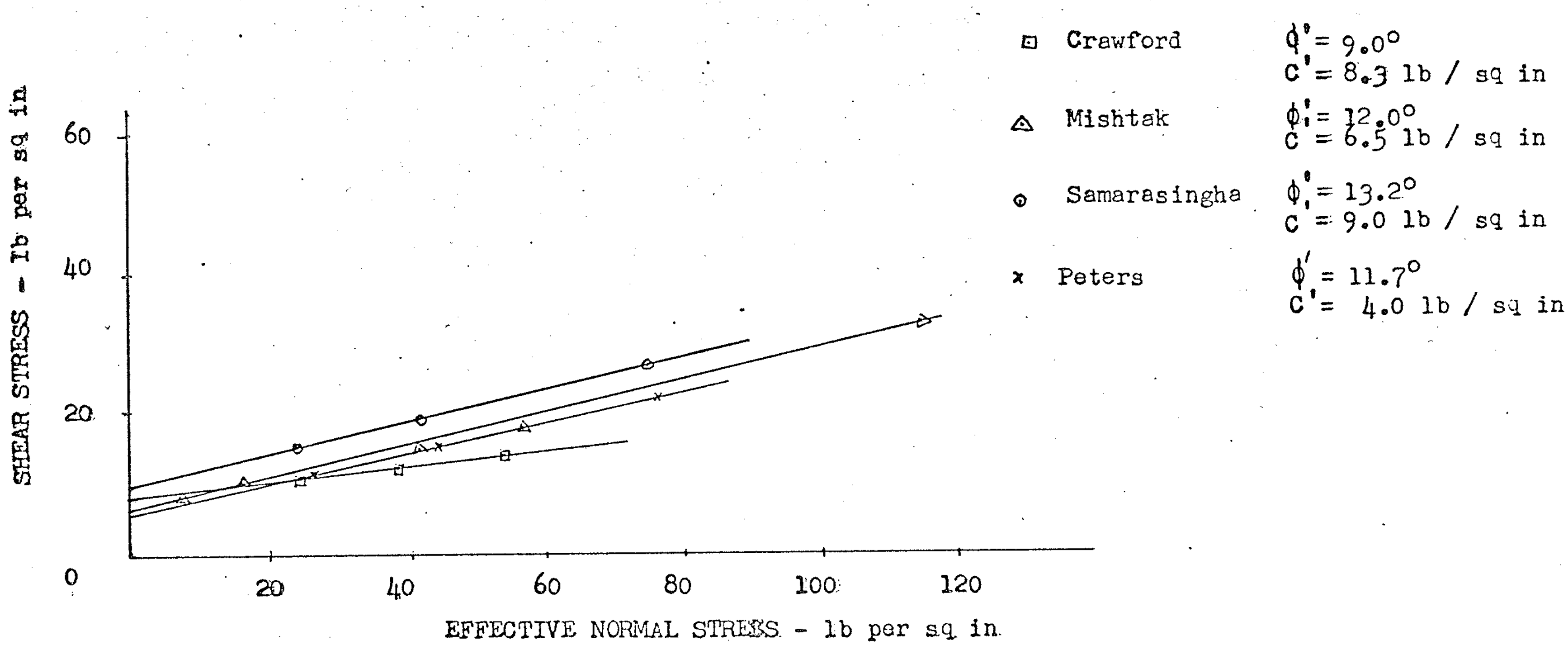

Fig. 21 - Comparison of shear strength envelopes for Winnipeg clays

ϕ' VS. RATE OF STRAIN

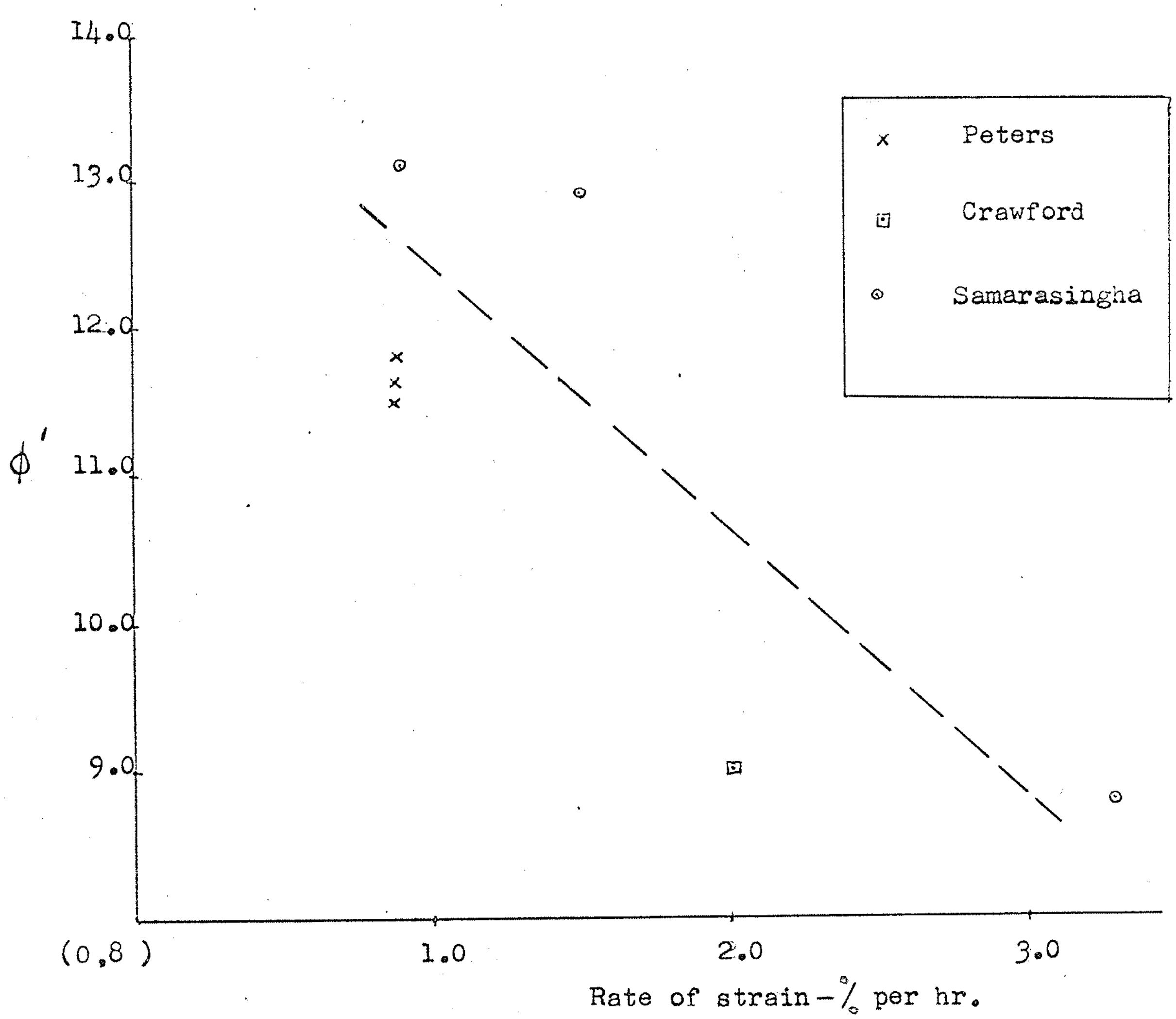

Fig 22

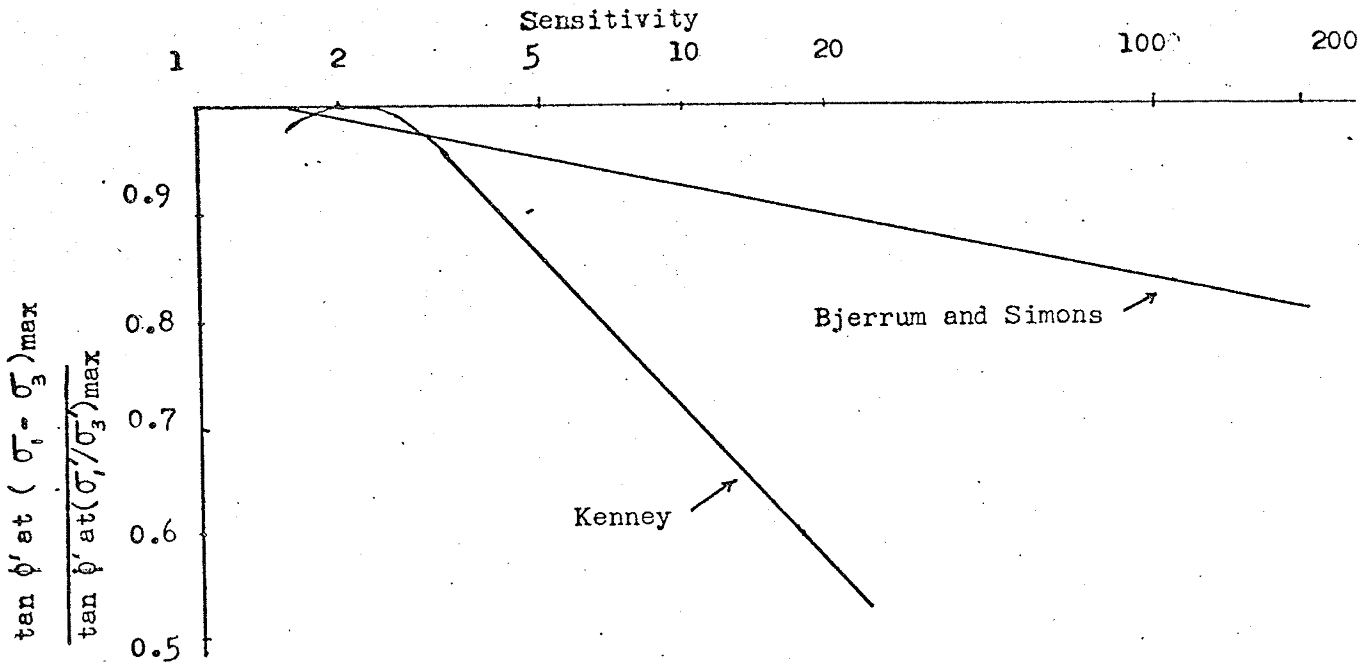

Fig. 23 - Relationship between tan ϕ' at $(\sigma_1 - \sigma_3)_{max}$ to tan ϕ' at $(\sigma_1'/\sigma_3')_{max}$

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