SOME ASPECTS OF PORE PRESSURE MEASUREMENT
IN TRIAXIAL STRENGTH TESTS OF SOILS

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ABSTRACT

It has been known for some time that the fluid in the voids of a soil mass exerted a pressure, but the importance and effect of this pressure (called pore water pressure) on soil strength, until recently, was not fully realized. In this research program some aspects of pore water pressure were studied and laboratory techniques developed to facilitate measurements of pore pressure.

For purposes of this investigation, a Farnell triaxial machine was used. It consisted of a triaxial compression chamber, a loading device, a control unit and a null indicator for measuring pore water pressure. Due to a failure in the feed valve for the loading device, most of the loading was done on a Chicago Soil Test loading apparatus which, although normally used in the consolidation test, with some modification served the purpose.

Consolidated drained and undrained tests were run on sand and silt. Pore pressures were recorded, on undrained specimens, and a Mohr's rupture envelope plotted for effective stresses at a strain where the ratio of the effective vertical stress to effective horizontal stress was a maximum. Mohr's rupture envelope for drained tests was plotted at maximum deviator stress.

Test results indicated that excess hydrostatic pore pressures are dependent on the volume change tendencies of a soil under loading. If positive pressures are developed during shear, the strength will increase if the pressure dissipates with time. If negative pressure is developed during shear the soil will lose strength if the pressure

is dissipated. In practice, if pore pressures are expected to develop, allowance may be made in design and field supervision exercised to ensure that pore pressures do not reach sufficient magnitude to cause failure.

ACKNOWLEDGEMENTS

The author is deeply indebted to all those who have contributed or assisted in making this investigation possible. Special recognition is given to the following who played a major role in this research program.

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INTRODUCTION

Until recently, pore pressure was a relatively unexplored field of soil mechanics, yet it is of major importance, since knowing this pressure permits a more comprehensive analysis of soil strength and subsequent determination of a stable design. In cohesive soils, pore water bears a certain amount of the applied loading until consolidation (which results from the dissipation of pore water and hence pore pressure) is complete. The process in granular material is similar, but time of consolidation much shorter.

In order to visualize pore pressures, consider an increasing load applied to a saturated soil. As the load becomes higher the soil tends to compress, placing the liquid under additional pressure. This pressure is what is known as excess hydrostatic pore water pressure. If it can dissipate, the soil will consolidate and strengthen. If the pressure cannot be dissipated, intergranular (effective) stresses will not increase and strength will be lower. In certain cases, the pore pressure may be due to capillary rise and thus negative. In such an instance, compression would cause dissipation of negative pressure, resulting in a weakening of the structure.

With an understanding of the causes of pore pressure, theoretical and laboratory analyses are more readily understood. The program followed in this thesis was intended to do the following:

1. Set up pore pressure measuring equipment in the laboratory.

2. Develop techniques suitable for the measurement of pore pressure.

3. Determine some aspects of pore pressures on local soils.

In this thesis the following symbols will be used:

Total vertical stress σ_1

Total lateral stress σ_2 and σ_3

Since $\sigma_2 = \sigma_3$ and is applied simultaneously, as a rule only σ_3 will be used.

Effective vertical stress $\bar{\sigma}_1$

Effective lateral stress $\bar{\sigma}_3$

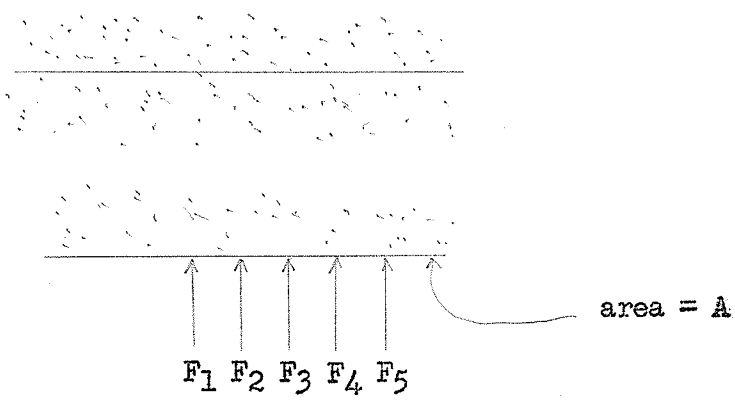
Pore water pressure u

Unit weight γ

POREWATER PRESSURE

Porewater pressure[1], as applied to soil mechanics, is the pressure of the fluid in the voids of a soil mass. This pressure, as will be indicated in subsequent paragraphs, is very important in soils engineering and may be either positive or negative, as shown in the following analogy.

Consider a horizontal section through a mass of soil.



Let $F_1 F_2 F_3$ be equal vertical intergranular forces.

Intergranular pressure[2] (effective pressure, or 0)

$$= \frac{F_1 + F_2 + F_3 + + +}{A}$$

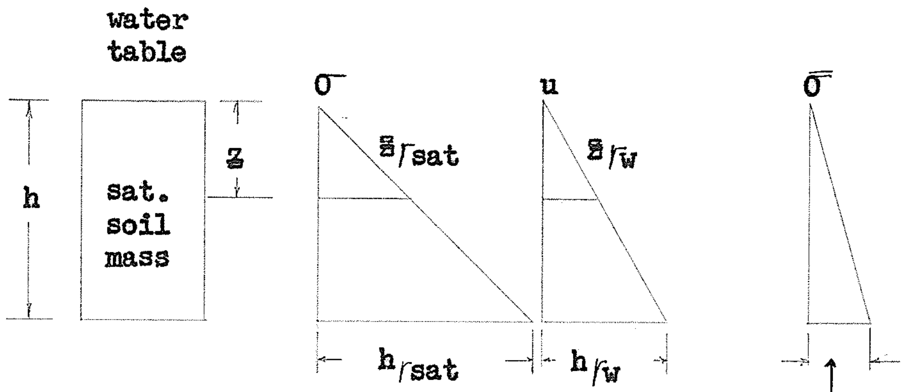
Let u = pore pressure, which is the pressure of the fluid in the voids and acts in all directions.

Then the total pressure 0 = 0 + u.

In a saturated soil with no capillary rise and the water table at the surface 0, 0 and u could be represented schematically as follows:

Note: $_{sat}$ = unit weight of saturated soil

$_{w}$ = unit weight of water

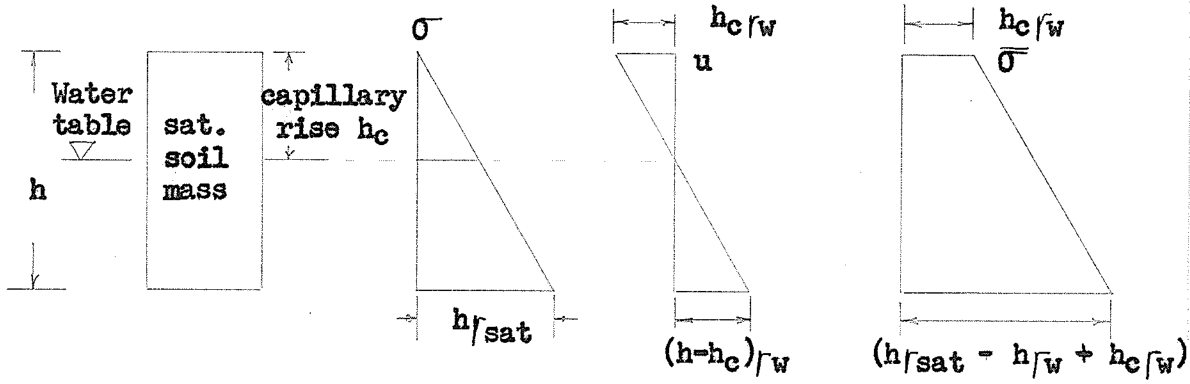


from above equation $\bar{\sigma} = \sigma - u$

$= h\gamma_{sat} - h\gamma_w$

In this instance u is positive.

Now consider the case of a saturated soil with capillary rise to the surface and the water table below the surface.



In this case u is negative in the zone of capillary rise and holds the soil particles together.

Pore pressure is the controlling factor in the relationship between normal stress and volume change. For an equal all-round change in stress this is expressed quantitatively for a saturated soil by the relationship

$$\frac{\Delta V}{V} = C_c \, (\Delta\sigma - \Delta u)$$

Where $\frac{\Delta V}{V}$ = change in volume per unit volume of soil

$\Delta\sigma$ = change in total normal stress

Δu = change in pore pressure

C_c = compressibility coefficient of the soil skeleton

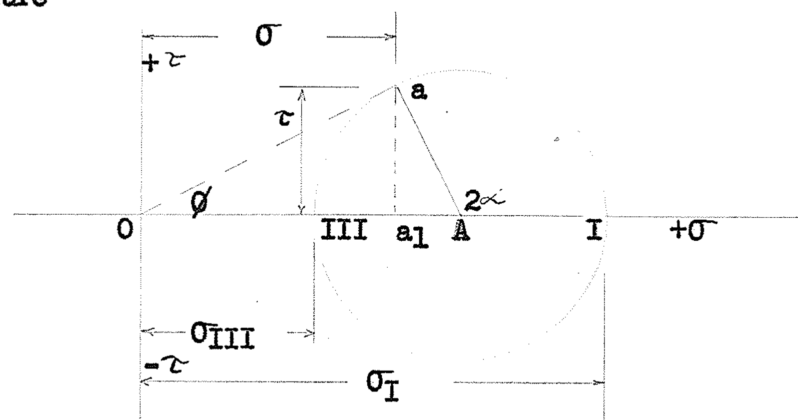
$\sigma - u$ = effective stress

It will be noted that the volume will change if the pore pressure changes.

This is the cause of the settlement of a building and also explains additional settlements caused by ground water lowering.

The shear strength of soils is largely determined by the frictional forces arising during slip at the contacts between the soil particles. These are a function of the stress carried by the solid skeleton rather than the total normal stress.

The values of the stresses σ and τ can be computed by introducing the numerical values for σ_I, σ_{III}, and α into equations. However, we can also determine these values by means of the graphical procedure[15]



In this diagram the compressive stresses (positive) are plotted on a horizontal axis from the origin 0 to the right and the positive shearing stresses on a vertical axis from point 0 in an upward direction. Hence positive values of the angle ϕ appear above the horizontal axis. The horizontal axis is reserved for the principal stresses because the corresponding shearing stress is equal to zero. In order to determine the values σ and τ for any plane forming an arbitrary angle α with the principal plane, we make 0 III = σ_{III}, 0 I = σ_I, trace a circle with a diameter I III = $\sigma_I - \sigma_{III}$, whose center A is located halfway between I and III and trace through A a line Aa which makes an angle 2α with AI. For geometrical reasons the abscissa of the point "a" thus obtained is equal to the normal stress σ, and its ordinate is equal to the shearing stress τ. The distance 0a represents the resultant stress on the inclined section through the specimen.

Hence the circle of which I III is a diameter represents the locus of all the points which are defined by equations $s = c + \sigma \tan \phi$ and $s = \sigma \tan \phi$. For this reason the circle is commonly called the circle of stress.

The failure plane, which represents the maximum resistance to shear, may be written as $\tau = S_n \tan \phi + c$

S_n = normal stress

ϕ = angle of failure plane

c = cohesion

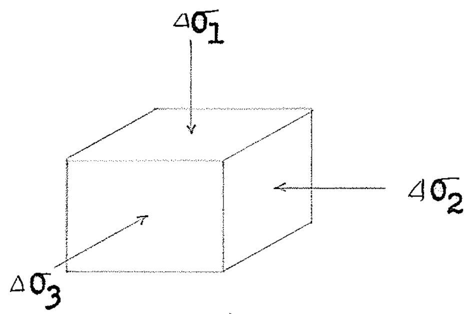
If pore pressure measurements are made, then

$$S_n = \sigma - u$$

$$= \bar{\sigma}$$

$$\text{and} \quad \tau = \bar{\sigma} \tan \phi + c$$

Parameters A and B



Δu = increase in pore pressure

$\Delta\sigma_1$, $\Delta\sigma_2$, $\Delta\sigma_3$, change V by an amount = $-\Delta V$

Increase in effective stresses

$$\Delta\bar{\sigma}_1 = \Delta\sigma_1 - \Delta u$$

$$\Delta\bar{\sigma}_2 = \Delta\sigma_2 - \Delta u$$

$$\Delta\bar{\sigma}_3 = \Delta\sigma_3 - \Delta u$$

Decrease in volume of soil skeleton

$$-\Delta V = V\left(\frac{1 - 2\mu}{E}\right)(\Delta\bar{\sigma}_1 + \Delta\bar{\sigma}_2 + \Delta\bar{\sigma}_3)$$

E = Young's modulus

μ = Poisson's ratio

If no drainage occurs

$$-\Delta V = nVC_w\Delta u$$

n = initial porosity

C_w = compressibility of fluid

The decrease in volume of soil skeleton is almost entirely due to the decrease in volume of voids.

Therefore $nC_W \Delta u = \frac{1 - 2\mu}{E}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)$

But $\Delta\sigma_2 = \Delta\sigma_3$

Therefore $\Delta u = \frac{1}{1 + \frac{nC_W}{C_c}} (\Delta\sigma_3 + 1/3(\Delta\sigma_1 - \Delta\sigma_3))$

Since $C_c = 3\left(\frac{1 - 2\mu}{E}\right)$ the compressibility coefficient of the soil skeleton.

C_W is constant only in fully saturated soils. The corresponding changes in pore pressure are therefore expressed in terms of two empirical parameters A and B, where

$$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3))$$

For fully saturated soils the value of C_W - that of water alone - is so small that B = 1 to within the limits of experimental accuracy. The value of A depends very largely on whether the soil is normally consolidated or overconsolidated.

Pore pressure is an independent variable[3] and, unless conditions of drainage are such that the fluid in the pore space can be freely expelled, an excess pressure will result from the stress change. The rate at which this pore pressure dissipates depends on the permeability of the soil. For a thick clay strate or impervious rolled fills, the time required may be many years. During this period the pore pressure is a function of

(i) initial stress change

(ii) coefficient of consolidation

(iii) the distance of the soil element from a surface at which drainage can occur

Pore Pressure Measurement

There are three principal types of triaxial tests and these are classified according to conditions of drainage obtained during each stage.

(i) Undrained tests - No drainage and hence no dissipation of pore pressure is permitted during the application of the all-round stress. No drainage is allowed during the application of the deviator stress.

(ii) Consolidated Undrained-Drainage is permitted during the application of the all-round stress so that the sample is fully consolidated under this pressure. No drainage is allowed during the application of the deviator stress.

(iii) Drained tests - Drainage is permitted throughout the test so that full consolidation occurs, and no excess pore pressure is set up during the application of the deviator stress.

It will be noted that when measuring pore pressures, 100% consolidation under the confining pressure must take place first, otherwise the confining pressure will influence pore pressure, rendering ambiguous results.

The only test which could give effective stress directly is the drained test, and on a sample of low permeability drainage may take a considerable time; it would therefore, in some instances, be more convenient to measure the pore pressure directly during shear. If the magnitude of the pore pressures during shear are known, then the intergranular stresses may be computed and enable the determination of the true strength envelope.

Undrained Test on Saturated Cohesive Soil[4]

If pore pressure is measured during the test, the effective stresses at failure can be determined. Both effective stresses are independent of the applied cell pressure, thus only one effective Mohr's stress circle is obtained and the shape of the failure envelope in terms of effective stress cannot be determined (pore pressure if measured would be equal to confining pressure).

Undrained Tests on Partly Saturated Cohesive Soils[5]

The deviator stress at failure is found to increase with cell pressure. This increase becomes progressively smaller as the air in the voids is compressed and passes into solution. The failure envelope expressed in terms of total stress is non-linear. If the pore pressure is measured, the failure envelope can be determined in terms of effective stress and is found to be a straight line over a wide range of stress.

Consolidated-undrained tests, expressed in terms of total stress, can be applied in practice only to a very limited extent. If the pore pressure is measured during the undrained stage of the test, the results can however be expressed in terms of effective stress. The values of c and ϕ thus obtained can be applied to a wider range of practical problems.

In the standard test consolidation takes place under an equal all-round pressure, and the sample is then sheared by increasing the axial load at a sufficiently slow rate to prevent any build-up of excess pore pressure. The minor principal stress σ_3 at failure is thus equal to the consolidation pressure; the major principal stress σ_1 is the axial stress. Since the pore pressure is zero, the effective stresses are equal to the applied stresses, and the strength envelope in terms of effective stress is obtained directly from the Mohr's stress circles at failure.

TEST EQUIPMENT

For purposes of this investigation, a Farnell triaxial machine was used. It consisted of a triaxial compression chamber, a hydraulic loading cylinder, a control unit and a pore pressure measuring device. The basic design, component parts and function of each of the above will be dealt with in the following paragraphs.

TRIAXIAL TEST

Advantages:

Drainage may be controlled and pore pressure measured more expeditiously than by any other practical means. Failure to appreciate the significance of excess pore pressure in strength measurement has been perhaps the greatest single factor in the late development of Soil Mechanics as a systematic branch of Civil Engineering[6].

Disadvantages:

1. In the test, σ_2 is equal to σ_3, but in many practical problems this is not so.
2. In the test, the principal planes are fixed in relation to the axis of the specimen. In problems where the direction of the major principal stress changes constantly under applied stresses, this restriction limits the accuracy with which pore pressure can be predicted.
3. Friction between the loading cap and specimen restrict lateral strain by friction.
4. In a drained test it is difficult to obtain uniformity of volume change, and in an undrained test it is difficult to obtain uniformity of pore pressure.
5. The duration of a test may be a considerable time but no account is taken of the phenomena of creep in soils[7].

(a) TRIAXIAL CHAMBER

The cell (Drawing No. 1) consists of three principal components - the base, which forms the pedestal on which the sample rests and incorporates the various pressure connections; the removable cylinder and top cap, which enclose the sample and enable fluid pressure to be applied; and the loading ram, which applies the deviator stress to the sample.

The principal features to be noted are:

(1) The base.

(2) The removable cylinder and top cap.

(3) The loading ram.

(4) Loading caps.

(5) Rubber membrane.

The base is machined from a single bronze forging and is plated. There are provisions for four pressure connections, as shown in Drawing No. 2:

1. The connection for filling the cell with the fluid, usually water, to apply the all-round pressure. This connection also serves to empty the cell at the end of the test and should therefore be made sufficiently large for rapid emptying.

2 & 3. The connections with the base of the sample: these provide for drainage in a drained test, and for pore pressure measurement in undrained or consolidated-undrained tests.

Four shallow radial grooves in the top of the pedestal provide access to the base of the porous disc in drained or pore pressure tests. The disc should be inert, rigid and accurately flat.

4. The connection with the loading cap: this is used only if it is necessary to pass water through the sample during the test (to saturate it or to measure permeability); or in a dissipation test when the coefficient of consolidation is determined by measuring the rate of decrease in pore pressure at the base of the sample while drainage is permitted from its upper end.

This connection also serves as a means of pore pressure measurement at the top of the sample. Such measurements are made in samples of low permeability.

The removable cylinder and top cap: a transparent Perspex cylinder is used, which facilitates the setting up of the test and enables the mode of failure to be observed. Strain observations can also be made optically. The 4-inch diameter cylinder used is $\frac{1}{4}$ inch in wall thickness and is normally taken up to a pressure of 150 pounds per square inch.

The cylinder is permanently fitted between O-rings as seals; the only joint to be made during testing is between the lower brass collar and an O-ring set in the base of the cell. For this the hand tightening of three nuts is sufficient.

The top cap, to which the cylinder is fixed, is a bronze casting, and the central boss forms the bush through which the stainless-steel ram slides. An air release valve is fitted, and there is also an oil filler hole for adding oil before long duration tests.

The loading ram poses the mechanical problem of how to combine minimum friction and leakage with a ram stiff enough to carry a wide range of axial loads and with a travel sufficient to permit large strains.

A $\frac{3}{4}$-inch diameter ground stainless steel ram runs in a lapped or honed bronze bush and is lubricated with oil or grease. No packing or sealing rings are used. With the clearance normally used (0.0003 in.) the ram, if wiped clean of oil, falls freely through the bush, and if oiled, falls slowly under its own weight.

Leakage is controlled in short duration tests (up to about half an hour) by applying a film of thin grease or oil to the ram immediately before testing. For tests of longer duration, the cell is filled with water to within about one half inch of the top, and the remaining space filled with machine oil, which floats on the water. The viscosity of the oil reduces leakage past the ram to negligible proportions, and tests of several days duration can be run on one charging of oil. Efficient lubrication of the ram is also ensured.

For tests run at a constant rate of strain it is generally sufficient to obtain a zero reading on the proving ring with the testing machine running at the specified rate of strain, but with the ram out of contact with the top of the sample.

The ram[8] is fitted with an adjustable collar as a stop to prevent it from being blown out of the cell when not in position in a testing machine. The cell can thus be used as an independent unit during the consolidation stage of the test, and transferred to the machine on a flexible pressure lead only when the axial load is to be applied.

Loading caps: the load from the ram is transmitted to the sample by various alternative types of loading caps. The loading caps for this machine are quite thick due to the large axial loads. Both are aluminum alloy, one being solid, the other recessed on the bottom and fitted with a porous disc to permit drainage. A drainage gland in the top of the cap is connected by means of a polythene tube to the "upper pore pressure" gland in the base plate. The purpose of this cap has been described previously under "Connections". Both caps have a knob on top (of the same material) $\frac{3}{4}$ inch in diameter by 1/8 inch high and rounded on top. This knob acts as a seat for the loading ram.

Rubber membrane: the sample is enclosed in a thin rubber tube or membrane 5 to 6 inches in length which may be obtained commercially or made in the laboratory from self-vulcanizing latex. The membrane should apply the minimum restraint to the sample consistent with providing a reliable barrier to leakage. The thickness is usually 0.01 - 0.015 for special tests on soft clay.

The rubber membrane is sealed against the smooth surface of the loading cap and of the pedestal by rubber O-rings under tension, sprung into place from the end of a metal tube. One ring at each end is sufficient if the surfaces are clean and the membrane is a close fit, but it is generally a useful precaution to use several rings, particularly in long duration tests.

(b) DETAILS OF APPARATUS FOR CONTROLLING THE CELL PRESSURE

The use of an air reservoir: the simplest method is to have an air reservoir, of large capacity compared with possible volume changes in the sample or leakage from the cell. The air acts on the surface of the water supply to the cell and is itself supplied by a motor-driven compressor.

A typical layout is illustrated in Drawing No. 3. The air supply passes through a reducing valve, and its admission to the reservoir is controlled by another valve. The blow-off valve permits the reduction of pressure, either when making fine adjustments to its magnitude, or at the end of a test.

(c) LOADING DEVICE

The hydraulic cylinder is supplied with liquid from the control unit. A schematic diagram of this unit indicates the system's operation. Oil from the reservoir is pumped up to the bypass valve and feed valve. The bypass valve permits the excess oil to return to the reservoir without going to the cylinder. Oil that passes the feed valve goes to the open-close valve and, depending on the setting of this valve, proceeds to the hydraulic cylinder or back to the oil reservoir. The feed valve permits the cylinder to rise at a maximum rate of one inch per minute and may be slowed down to zero. The open-close valve permits us to release the cylinder without changing the adjustment on the feed valve or shutting off the motor.

Just prior to commencement of laboratory work the feed valve for the Farnell hydraulic loading cylinder broke and the Chicago Soil Test loading device was used for most of the work. This device was normally used for the consolidation test but, with some modification (see photograph page 36), served the purpose extremely well. The loads were applied manually in increments of one to five pounds to the end of a lever, which gave a mechanical advantage of eight at the load yoke. With the Chicago Soil Test apparatus, no correction was required in calculations for weight of piston and force necessary to overcome confining pressure, as a balance was obtained with small load increments prior to testing.

(d) DETAILS OF MEASURING PORE PRESSURE

The usual laboratory methods of measuring pressure - the mercury manometer and the pressure gauge of the Bourdon-tube type - cannot be applied directly to the measurement of pore pressure in a small sample of soil, owing to the volume of pore water which would have to flow from the sample to cause the instrument to register. This amounts, for example, to 0.013 cubic inches for each pound per square inch rise in pressure for an 1/8 inch bore manometer; for a sensitive Bourdon gauge (total range 0 - 150 lb/sq. in.) the volume change averages 0.025 cu. in. per lb/sq. in. over the range 5 - 15 lb/sq. in.

This flow of pore water has two undesirable results. It modifies the actual magnitude of the pore pressure existing in the test specimen, which it is the purpose of the apparatus to measure; this is particularly important in soils of low compressibility. In addition, in soils of low permeability, the flow of pore water leads to a serious lag in the attainment of a steady reading on the pressure gauge or manometer.

These difficulties can be entirely avoided, however, by the use of a null method of pressure measurement which has many advantages for general laboratory use.

The null indicator consists of a glass tube immersed in a mercury trough, as shown in Drawing No. 4. The pore pressure line from the sample is completely filled with water and meets the mercury at some point in the capillary tube. The line from the mercury trough to the manometer or Bourdon gauge is also completely filled with water. Thus the smallest volume change in the sample is transmitted to the pressure gauge.

Control Cylinder

One other major item needs mention before referring to the apparatus as a whole, and this is the control cylinder (Drawing No. 5). It consists of a cylinder approximately $1\frac{1}{2}$ inches inner diameter, a brass piston and a stainless steel piston rod. The main features are brought out in the accompanying figure, with the exception that the piston rod bears on the piston by means of a recessed deep groove ball bearing packed with grease. The piston has a total travel of about five inches.

Layout

The complete layout of the apparatus is shown diagrammatically (Drawing No. 6). It will be noted that in addition to the pressure gauge (0 - 150 lb/sq. in. in gradations of 1 lb/sq. in.) a mercury manometer is fitted. This is used (i) for negative pore pressures, (ii) for the accurate measurement of low positive pore pressures, and (iii) for checking the zero error of the pressure gauge. It is calibrated directly in centimeters of mercury. Failure to close the isolating valve "m" when passing into the high-pressure range appears to be sufficiently frequent to justify fitting a mercury trap at the top of the manometer.

The graduated tube "h" connected to the valve "f" is used for determining the gauge and manometer readings corresponding to zero pore pressure. In the case of fully saturated samples this graduated tube can also be used to measure volume change during the consolidation stage of tests in which drainage is permitted through the base of the specimen. This procedure is not permissible with partly saturated samples as it results in the accumulation of air in the connections used for the measurement of pore pressure during the undrained stage of the tests.

Filling the Pore Pressure Apparatus

The manufacturer suggested that the following procedure be adhered to when filling the apparatus.

1. Remove the mercury trap from the base of the null indicator and fill the centre portion with re-distilled mercury. Screw it back

on to the null indicator approximately one revolution.

2. Connect the copper capillary tube to the upper end of the null indicator.

3. When scale is in lowest position fill the manometer to zero level with redistilled mercury making sure that both columns are continuous.

4. Connect outlet at "j" to a supply of de-aired distilled water and check that the pump is screwed right in.

5. Connect a vacuum pump of the water ejection type to the back of the glass cock on the pressure side of the manometer.

6. Close valves "l", "k" and "j", turn glass cock "o" at top of the manometer so that it connects the vacuum pump to "m".

7. Open valves "m" and "n" and pull vacuum at "o".

8. While a vacuum is being pulled at "o" open "j". Water will thus flow through "j" into the water pump and the connections up to "o".

9. Close valve "n" and screw out water pump to full extent.

10. Close valves "j", "m" and "f". Open valves "n" and "k" and pump water through the null indicator until it is issuing through the capillary tube at the point where it connects to the cell. While water is flowing connect to cell at point marked "p.w.l." and close valve "a".

11. Open valve "f" and continue pumping de-aired distilled water through into the burette until this is half full, ensuring that no air is trapped in the rubber connecting pipe.

12. Close valve "n", open "j" to distilled water and refill pump by screwing outward. Close valve "j".
13. Turn "o" from "m" to the pressure limb of manometer and close valve "k".
14. Close "m" and generate a pressure of approximately 5 pounds p.s.i. with the pump with "l" open.
15. Pull a vacuum at "o" and when the mercury has reached a maximum, open mercury pressure limb to "m" and open "m".

Repeat operations 14 and 15 until water has united.

De-airing and Cleaning Pore Pressure Apparatus

In a new apparatus, or one not used for some time, small bubbles of air or other gas tend to adhere to the inside of the various tubes and fittings. The procedure for removing them is outlined below:

1. The mercury trough "g" is screwed to the lower limit of its travel, so that the connection to the lower end of the glass tube no longer dips below the mercury surface. Water may then be freely circulated through the system without carrying mercury away from the trough.
2. The triaxial cell is disconnected, and the end of the copper tube is immersed in a dish of freshly boiled water with the valve "a" open. A rubber tube connected to a vacuum line operated by a water ejector is fitted to the tube "h", and with the valves "f", "k" and "n" open, and "l", "m" and "j" shut, water is drawn through the system while the piston is screwed in and out. Rapid opening

and closing of valve "a" at this stage facilitates the removal of air from the valve itself. The vacuum lead may then be transferred to valve "j"; with this valve open and valve "f" shut the process may be repeated. Alternatively, valve "a" can be closed and water may be drawn in through valve "f" from the tube "h". Finally, with valves "f" and "j" closed and valve "a" open, water may be forced out under pressure through the copper tube.

3. To ensure that the system is now air-free, valve "a" is closed and valves "f" and "l" are opened (valves "k" and "n" being open and valve "j" shut). The mercury trough "g" is screwed into its upper position. As this operation displaces water, the mercury may rise up the capillary tube unless the control cylinder "e" is screwed back at the same time. The control cylinder is then screwed in until mercury rises to a convenient level in the capillary tube. The pressure gauge "d" indicates an initial reading.

4. The valve "f" is now closed and the pressure increased by screwing in the control cylinder. A further rise in the mercury level in the capillary tube indicates either expansion of the apparatus, compression of air bubbles still remaining between the mercury surface and valves "a" and "f", or leakage. If the control is adjusted to maintain constant pressure on the gauge "d", steady creep of the mercury level indicates leakage, provided the apparatus has been allowed to cool before checking. A large rise

which is not fully reversible generally indicates air bubbles, which pass into solution at higher pressures.

Calibration

It is seldom convenient to set the pressure gauge exactly at the same level as the sample in the testing machine. This, in addition to the difference in level between the mercury surfaces in the capillary tube and in the trough, necessitates the use of a zero pore-pressure reading on both the pressure gauge and the manometer. It is, in fact, of some practical advantage to maintain a considerable difference in mercury levels, as this ensures that the pressure gauge is normally under a small positive pressure, even if placed at eye level.

To obtain the zero reading, after the apparatus has been checked for trapped air, valve "f" is opened (after reducing the pressure to zero) and the graduated tube "h" adjusted so that the level of the water in it corresponds with the mid-height of the sample under test. The screw-control is adjusted to bring the mercury level in the capillary tube to a convenient height, which is marked, and the pressure gauge and manometer readings are noted. These are the zero readings corresponding to atmospheric pressure at the mid-height of the sample, and all changes in pore pressure are measured with reference to them.

The zero readings obviously depend on the mercury level chosen. In general this level is chosen to give a whole number on the pressure gauge scale; the manometer scale is then adjusted to read zero for this level, so that no correction is necessary.

Provided the level of the specimen being tested remains approximately constant (a 3-inch change in level represents an error of approximately 0.1 lb/sq. in.), the zero settings should remain constant. The few seconds taken to check them before each test are, however, well justified, as they will indicate the presence of air in the manometer, drift in the calibration of the pressure gauge or loss of mercury from the system.

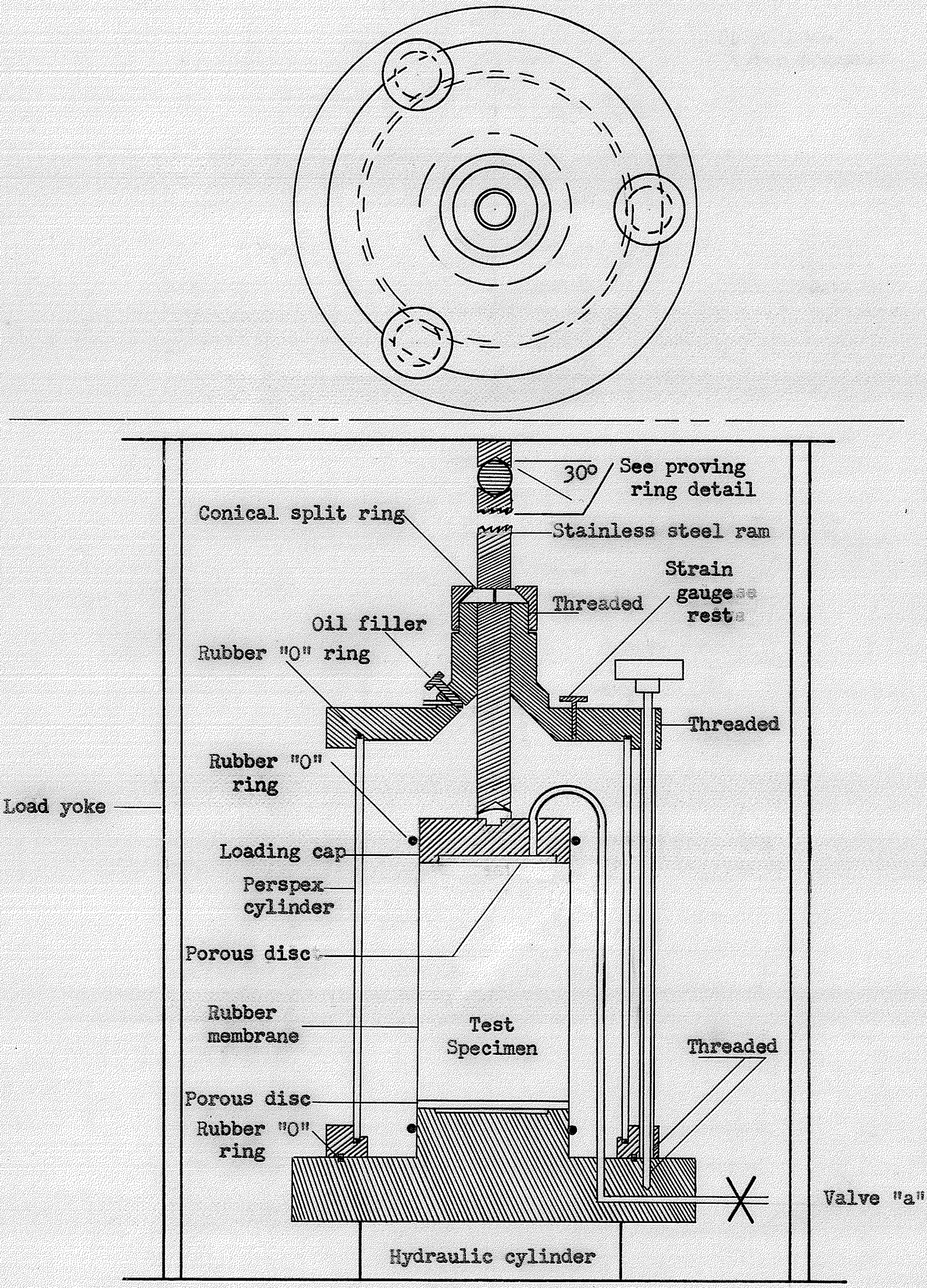
One additional point is of importance when both cell pressure and pore pressure are large and the effective stress, which is the difference between them, has to be known accurately. A direct calibration of the pore-pressure system against the cell-pressure gauge may be made by connecting up to a triaxial cell full of water but without a test specimen. The pressure read on the pore-pressure system is thus equal to the cell pressure. The use of this direct calibration minimizes the error which might otherwise be introduced into the small difference of two large quantities.

Pore Pressure Measurement

When the equipment is set up as shown and precautions taken to eliminate errors as previously described, the test may be started and pore pressure variations observed. For this purpose valve "j" is opened, "n" kept closed, and the control cylinder piston screwed out to permit the cylinder to fill with water. Valve "j" is then closed, valve "f" is kept closed, valves "a", "k", "n" and, for the start of the test, "m" are opened.

The first sign of pore pressure will register in the null indicator by the mercury changing its level. By suitable manipulation of the control cylinder piston, the mercury in the null indicator is brought back to its original or "zero" level. The change in pressure may now be observed on the mercury manometer "c" but it will be remembered that the sample has not been permitted to change volume, i.e. lose water, because the null indicator is still at zero. (In theory the water in the line may be compressed, but for all practical purposes this may be neglected, because the pressures normally handled are too low.)

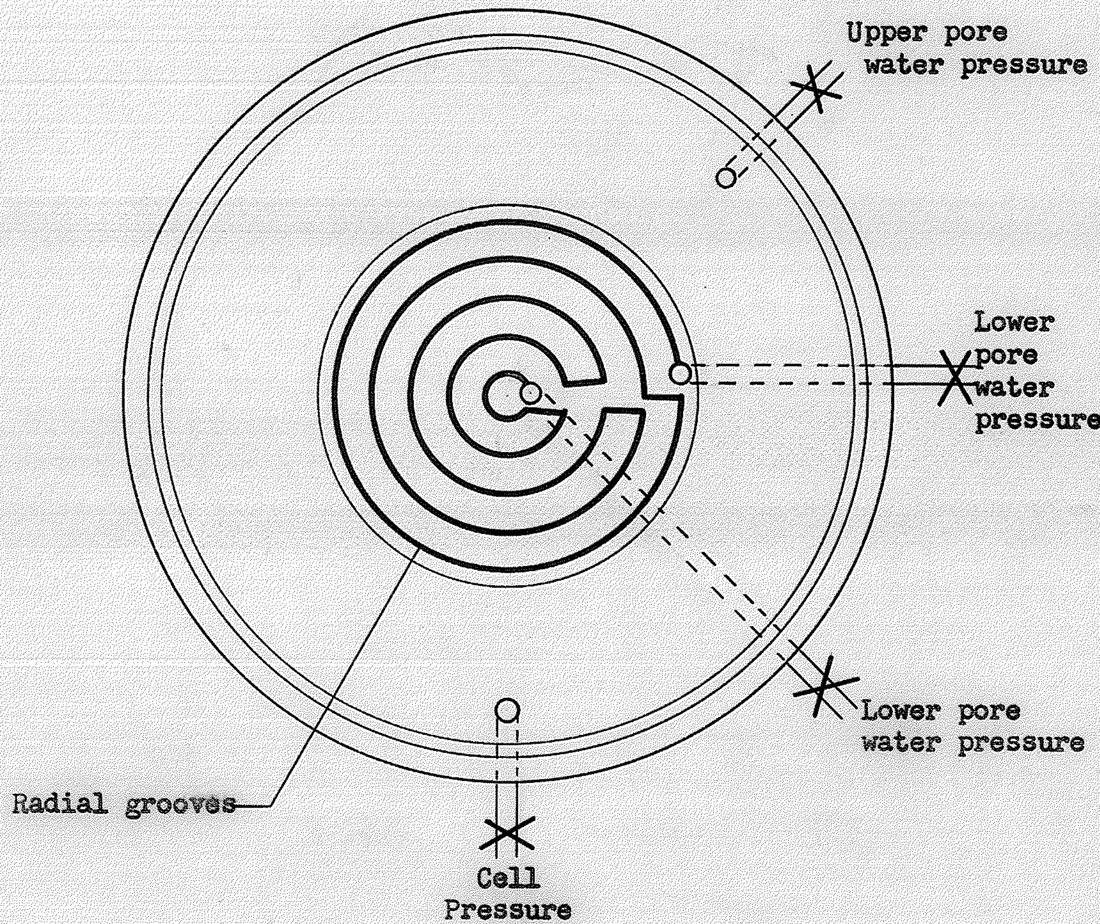
As the pore pressure increases to the capacity of the mercury manometer the pressure may be transferred to the Bourdon gauge. This would be accomplished by first closing valve "k" and then "m". Valve "l" would now be opened and by manipulating the control cylinder piston the same pressure placed on the Bourdon type gauge as previously showed on the manometer. Valve "k" could then be reopened and the test proceeded with as before. It should be noted that if valve "l" were opened before closing valve "k", the void created by the Bourdon gauge would result in a distortion to the sample with subsequent deleterious effects on the final results. A possible modification of the above would be to leave both valves "l" and "m" open, shutting off "m" at a suitable time. Attention at this time is brought to the fact that the control cylinder may be replenished with water by closing valve "n" and opening "j". Further, since pore pressures may be either positive or negative the piston on the control cylinder should, at the beginning of each run, be suitably placed.

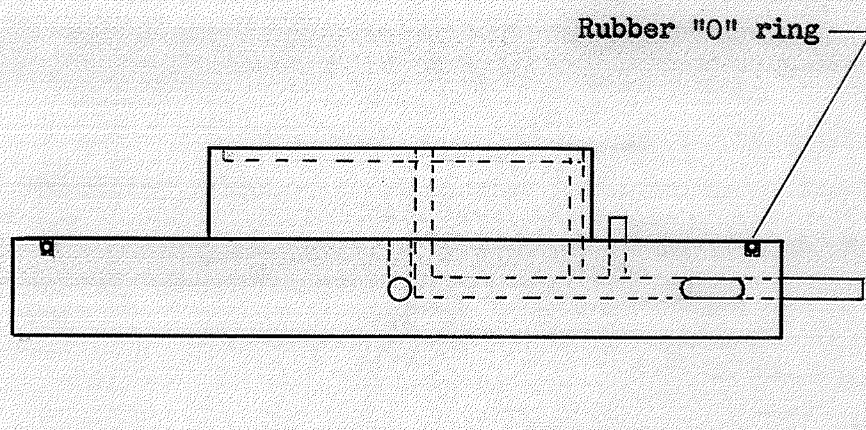


TRIAXIAL CHAMBER

DH

Drawing No. 1

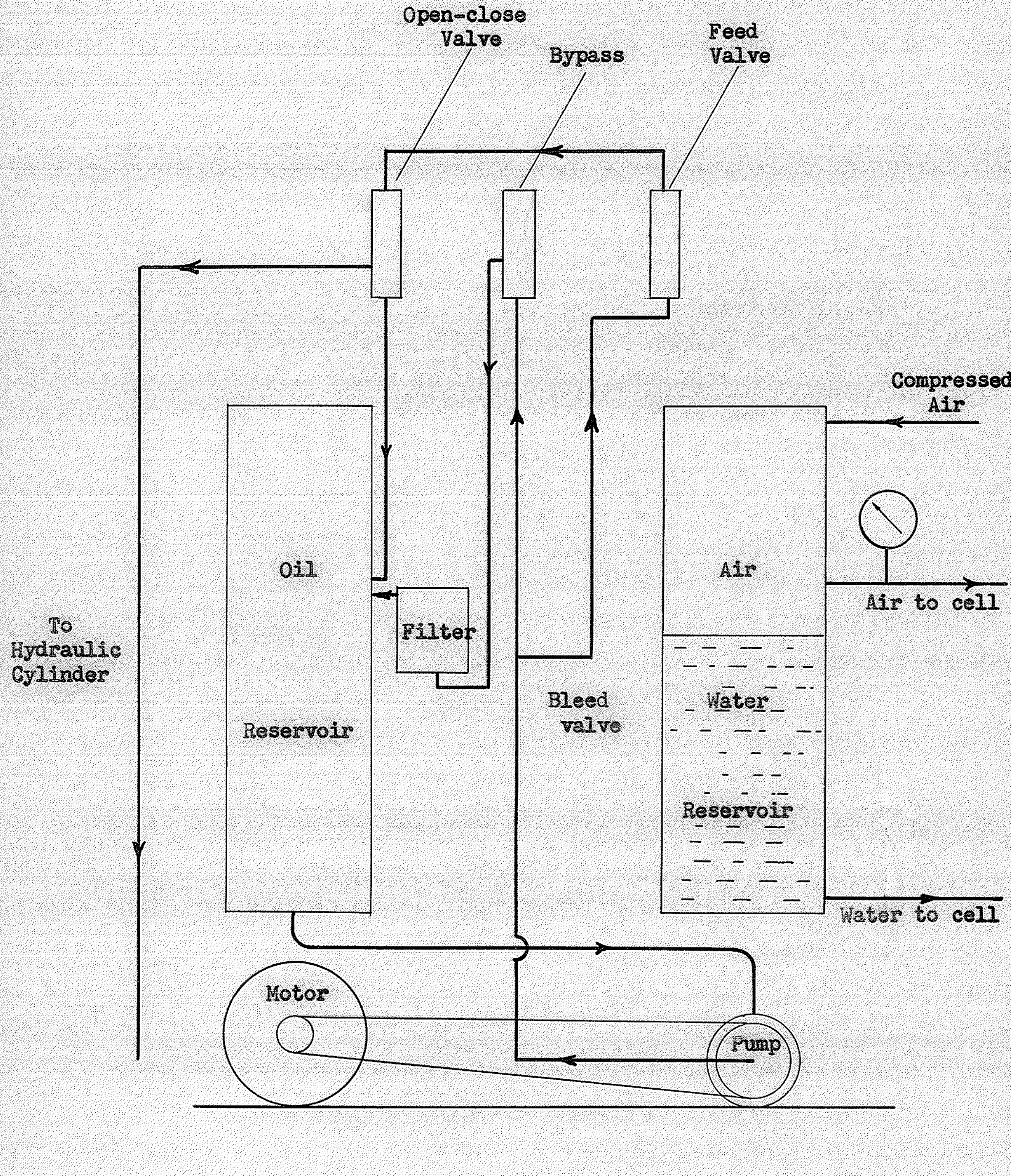




BASE PLATE DETAIL

DH

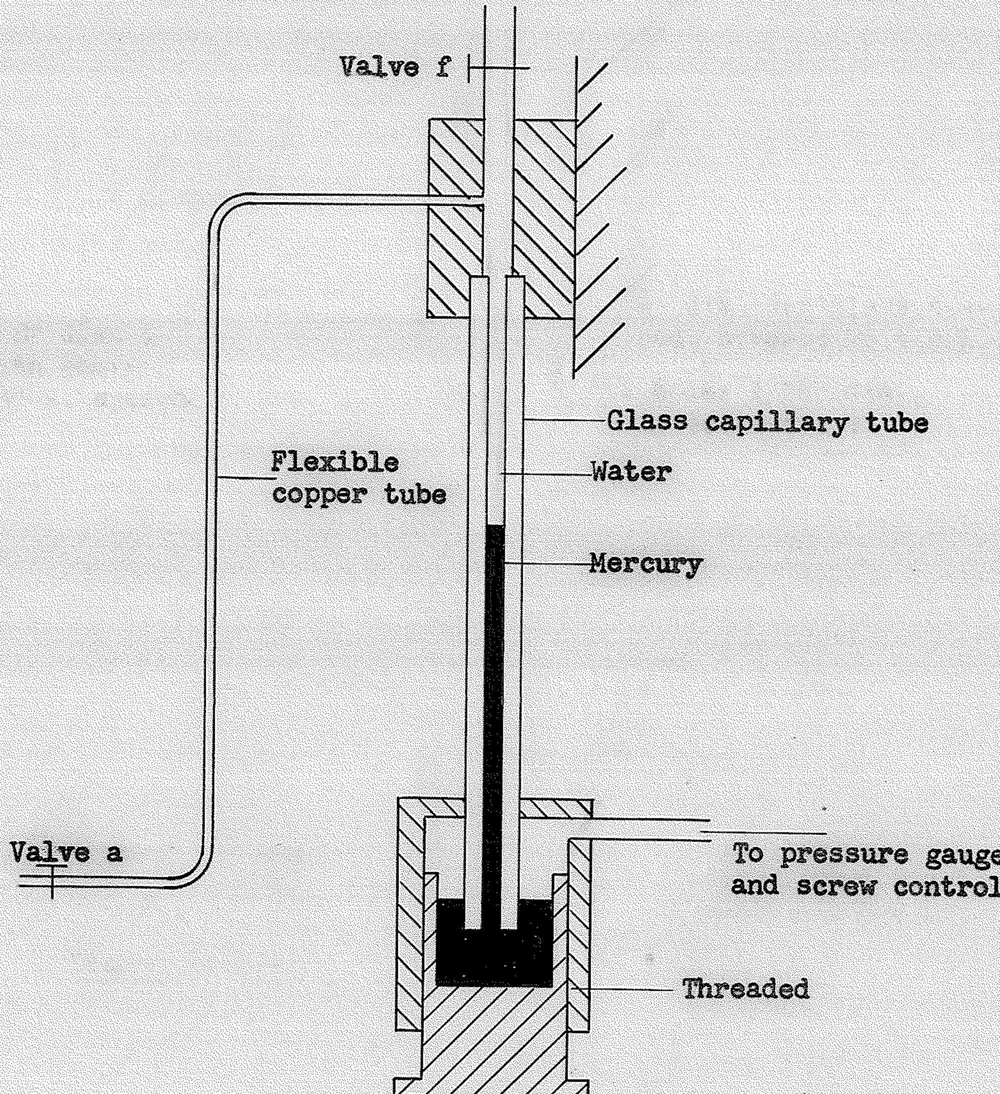
Drawing No. 2



C O N T R O L   U N I T

DH

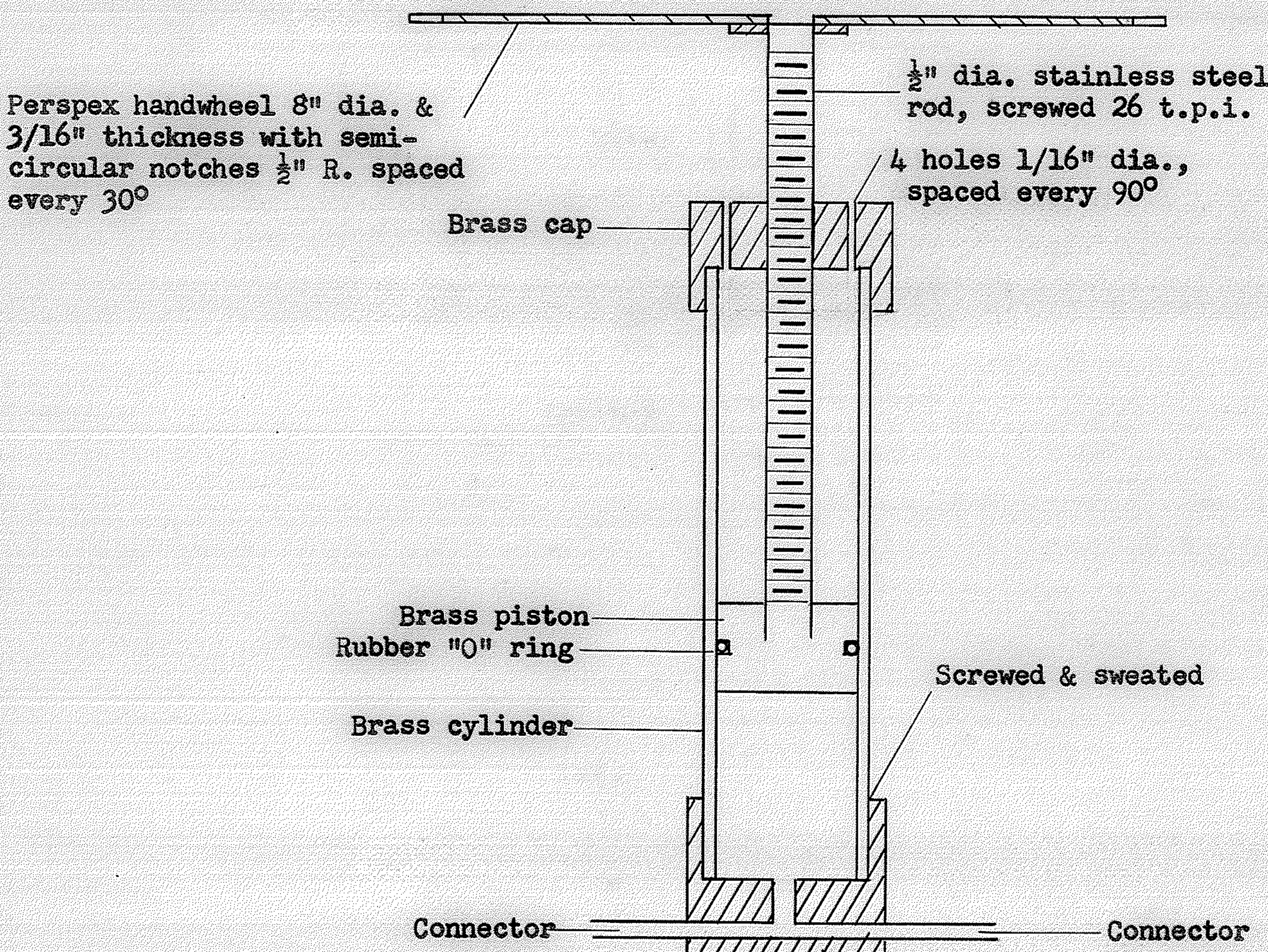
Drawing No. 3



NULL INDICATOR

DH

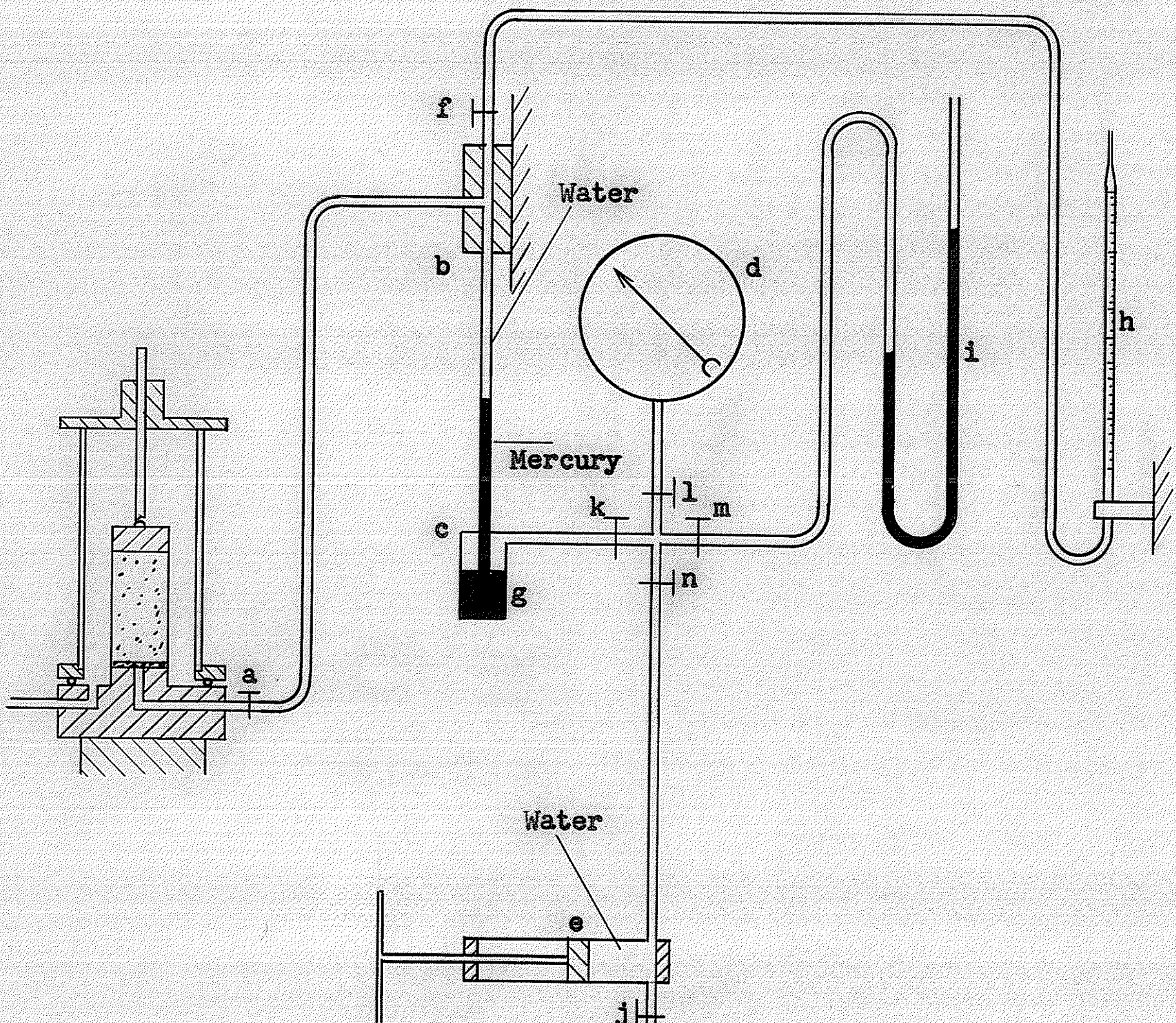
Drawing No. 4



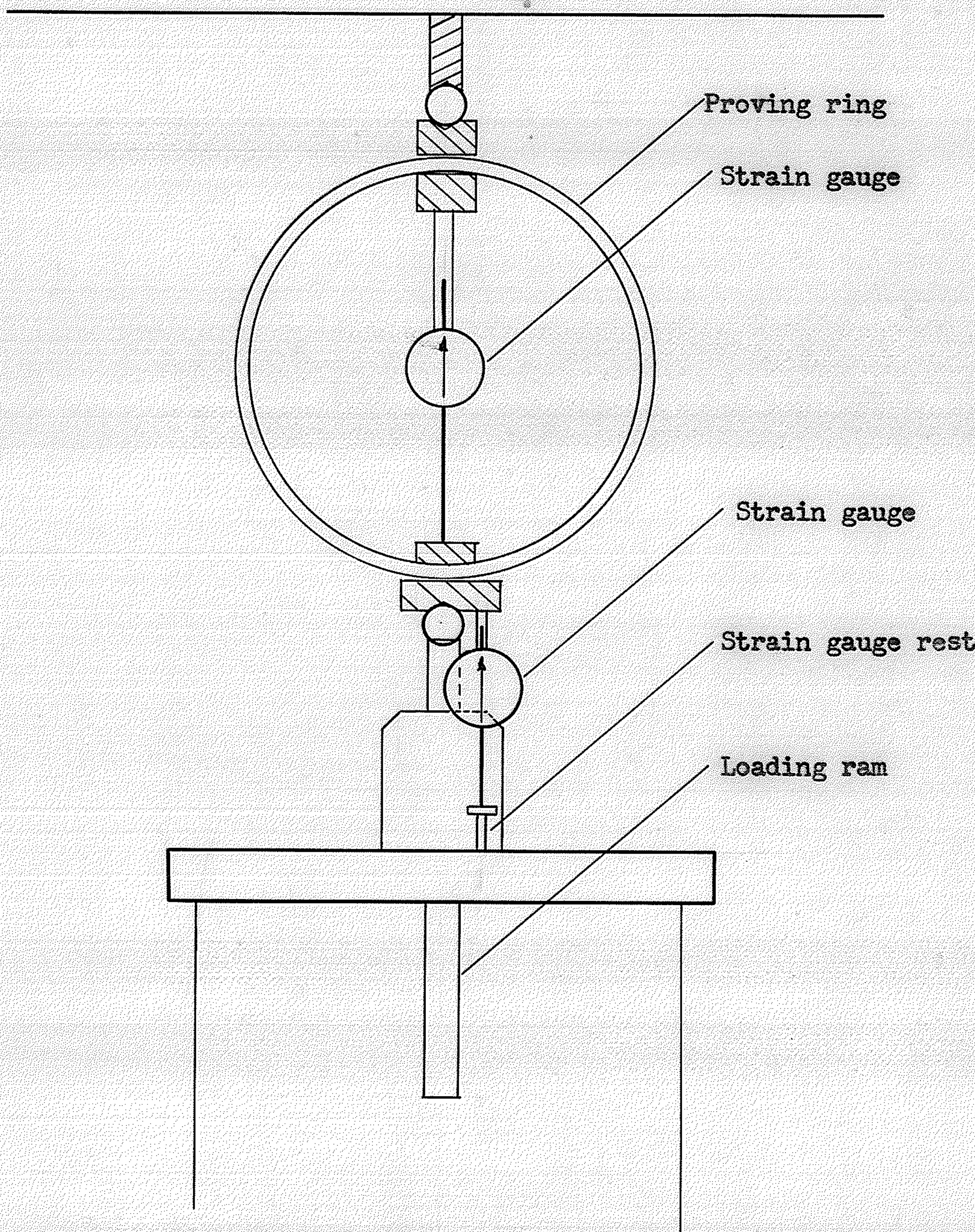
CONTROL CYLINDER DETAILS

DH

Drawing No. 5



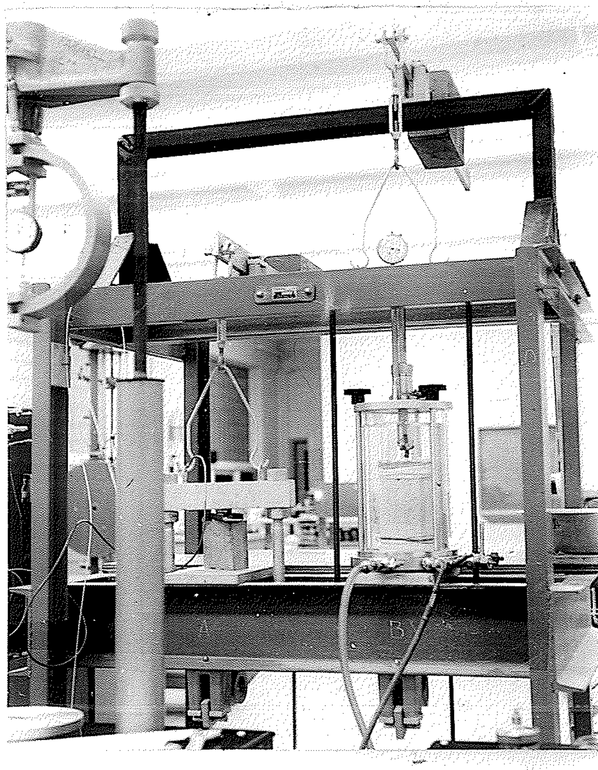
SCHEMATIC LAYOUT OF APPARATUS
FOR MEASURING PORE PRESSURE



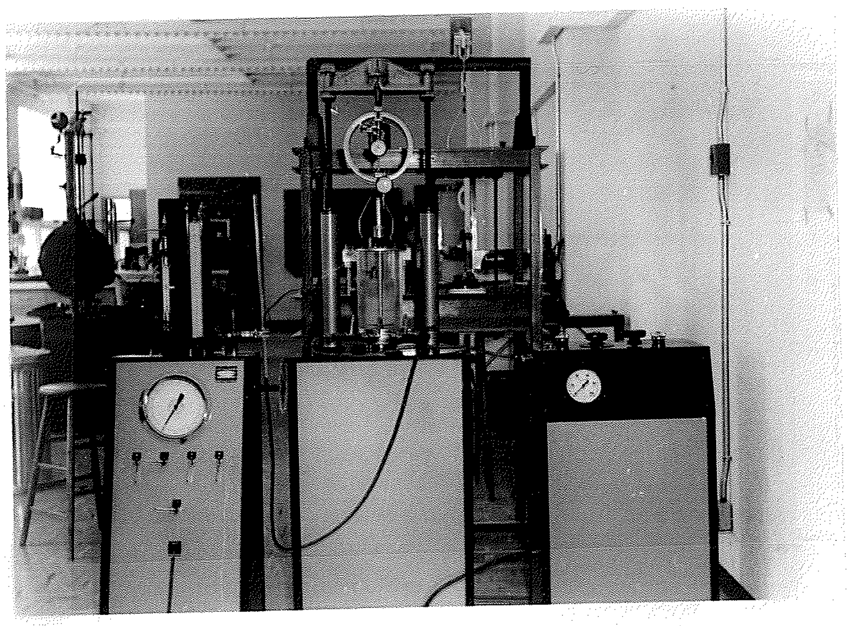
P R O V I N G   R I N G

DH

Drawing No. 7



Triaxial chamber positioned for specimen
loading on Chicago Soil Test device



Triaxial chamber positioned for specimen loading on
Farnell Machine
Pore Pressure device (left), Hydraulic Loader (centre),
Control Unit (right)

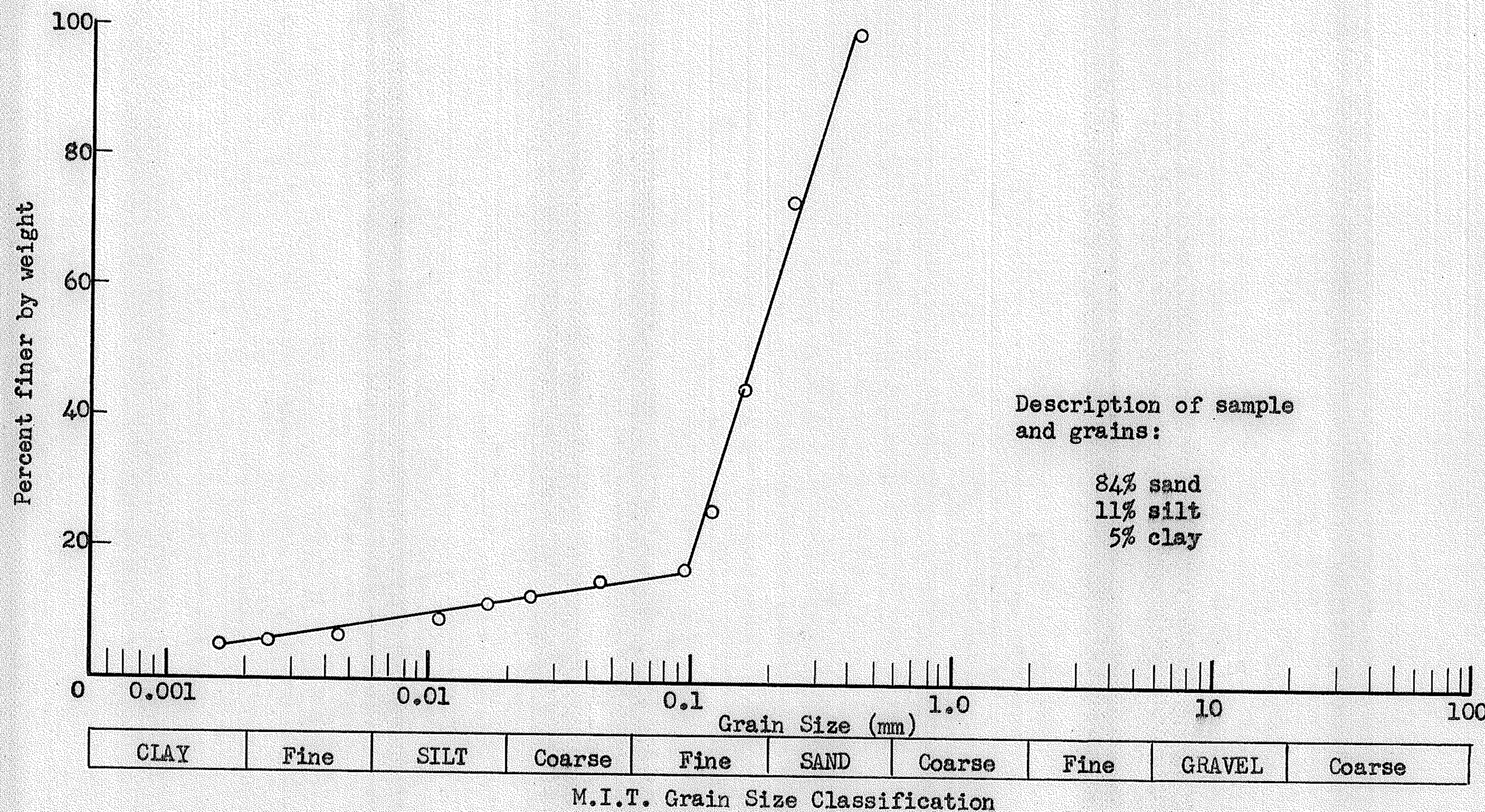
MATERIALS USED

As one of the primary purposes was to develop techniques for pore pressure measurement tests, the tests were limited to sands and silts that could be tested in the time available.

The sand sample came from a test bore at Shilo, Manitoba and was quite dilatent when wet. It was a fine sand passing a number forty sieve. Its grain size analysis showed that it was made up of 84% medium and fine sand, 11% silt and 5% clay. When made into test specimens, it proved to be consistent even though it was dried and used over again several times. The moisture content, dry density and void ratio were consistent, varying slightly with confining pressure, which seems reasonable. Saturation was also consistent but never exceeded 86%.

The silt used was taken from an excavation in Fort Garry. It was a tan silt with a grain size analysis as follows: 11% fine sand, 76% silt, and 13% clay. It was quite friable even when wet and compacted, which made it difficult to trim into test size.

Grain size curves for these materials are shown on Drawings Nos. 8 and 9.

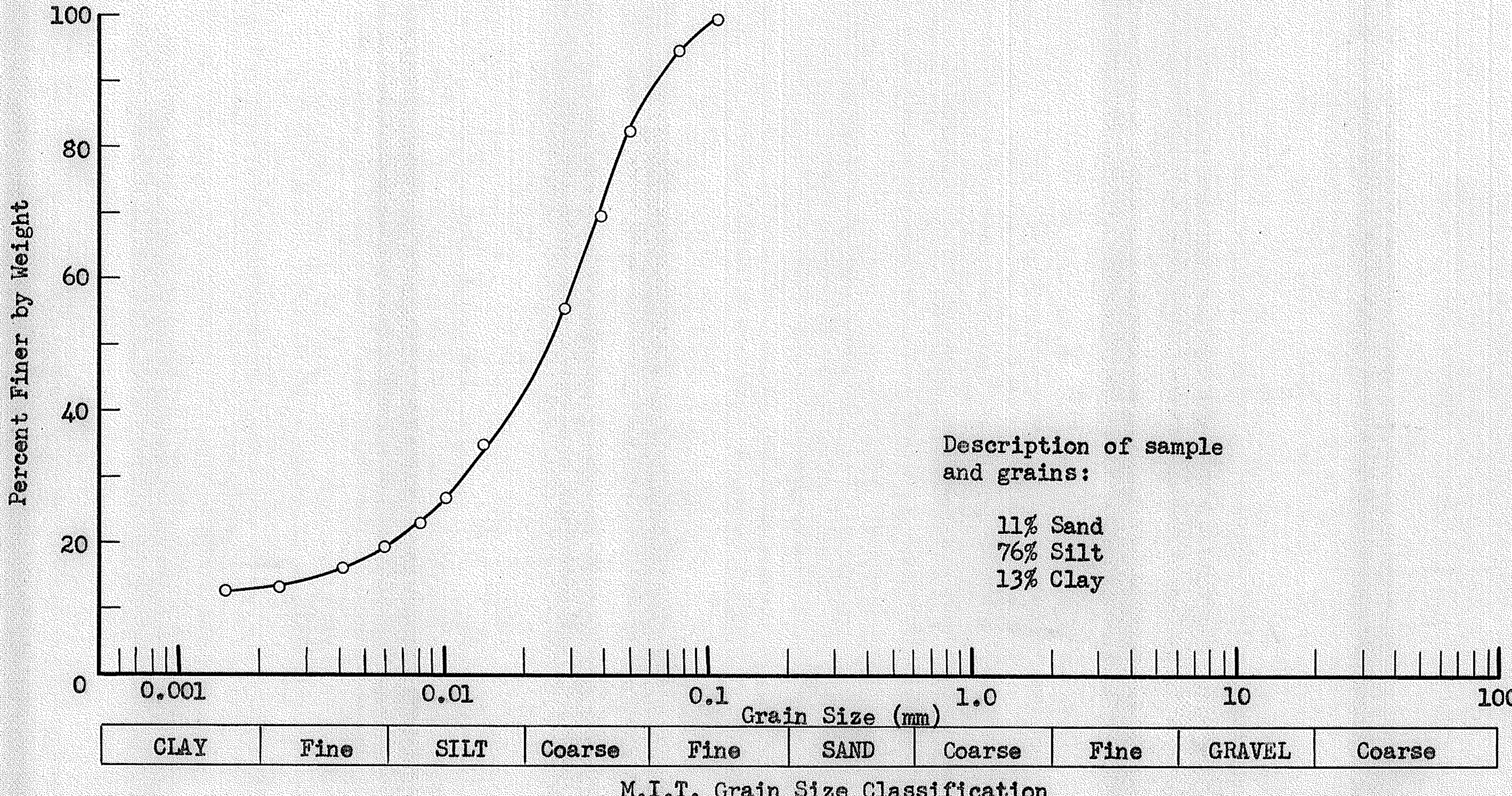


MECHANICAL ANALYSIS OF SOILS

DH

Drawing No. 8

University of Manitoba
Department of Civil Engineering
Graduate Studies 1959



MECHANICAL ANALYSIS OF SOILS

Drawing No. 9

University of Manitoba
Department of Civil Engineering
Graduate Studies 1959

DH

TESTING PROCEDURE

Considerable difficulty was experienced in setting up the apparatus and running the first test. As a result of the many obstacles encountered, a routine procedure was developed to expedite testing. A general pattern will first be described, which will apply to all tests for this thesis, followed by details used for various types of samples.

First and foremost, it was essential that the membranes were leakproof. It was found that new membranes, not even used once, could be perforated. It was difficult and sometimes impossible to locate holes in a membrane prior to running a test, but two methods were found that rendered reasonable results. The first - the membrane was inflated with air and held under water while the presence and source of any air bubbles was carefully observed. The second - a sample of fine sand was set up, and after saturation, it was allowed to consolidate under about 50 p.s.i. If the water ceased to flow from the sample (and rise in the burette) after an hour, it was concluded that the membrane was leakproof. Neither of the above were attempted prior to visual examination while the rubber was stretched gently with the fingers. Membranes were handled with care and every endeavour made to prevent them from being punctured. Sharp articles were kept away and, when placed on a mold, work was performed on top of a cloth to prevent abrasion.

After the membrane was placed on the sample, rubber rings were fitted around the base and the loading cap to ensure a closer fit.

All air had to be removed prior to measuring pore pressure. This aspect of the test will be discussed later.

The chamber was then positioned. The loading ram was well lubricated with a fine grease and the contact surfaces between the cell and the base thoroughly cleaned, which facilitated a tight seal and prevented damage to the easily-scored metal. After the cell was tightened on it was filled with water. It was found here more expedient and just as satisfactory to apply pressure to the cell in the form of air rather than water. Thus the cell was full of water, which eliminated any danger of explosion in case it burst, and the pressure was applied with air, which rendered unnecessary a layer of oil on the water surface (to act as a seal) and an aspirator to catch leakage.

The sample was left to consolidate until the water ceased to rise in the burette. The confining pressure was so regulated as to remain constant for the duration of the test. With respect to drainage, the burette connection contained a continuous column of water, and when the tube was connected to the base, caution was exercised to ensure a continuous column to the base of the sample. Readings were taken prior to the commencement and subsequent to the end of consolidation, and the change in volume of the sample was recorded.

The same base connection was used to connect the pore pressure equipment. This was accomplished by shutting the valve on the base, disconnecting the burette line and attaching the pore pressure line, again taking care to ensure a continuous column of water between the base of the sample and the mercury surface of the null indicator.

Loading was then commenced and continued until the sample failed. With the Farnell equipment, the rate of load application was set on the feed valve to the hydraulic cylinder and recordings made of the load, the deflection, and the pore pressure. This equipment increased the load continuously, and no time was allowed for the sample to consolidate. The Chicago Soil Test loading device enabled the consolidation of the sample under each load increment and at any time the exact load on the sample could be determined. As with the Farnell, recordings were made of load, deflection and pore pressure.

Some techniques were developed to facilitate the setting up of samples. The following routine was developed and found satisfactory for a disturbed cohesionless soil. The membrane was fitted on a mold and placed over the base. The mold was filled with dry material and the loading cap placed. A small vacuum was applied to the sample to make it rigid while the mold was removed. After the membrane was secured with rubber rings, de-aired, distilled water was drawn through the sample to saturate it. This was accomplished by applying a vacuum to the loading cap drainage gland and giving the pore pressure connection in the base of the sample access to water.

For cohesive samples, the method of setting a sample was more elaborate. After the sample was trimmed (when necessary) it was weighed. Filter paper shields were placed on both ends and longitudinal strips of filter paper equally spaced around the sample. The strips were in sufficient number as to adequately drain the sample, but not so numerous as to strengthen it appreciably. (A silty sample may have needed only three or four such strips.) The specimen was then placed on the base and fitted with a membrane. By means of a vacuum applied to the loading cap drainage gland, water was flushed around the sides and ends of the specimen until all air was removed. The sample was then ready for testing.

Sand samples consolidated very rapidly and drained in about an hour, but cohesive materials took considerably longer, and for this reason, some method was required to determine when consolidation had taken place. This information was best obtained from a time consolidation curve. Plastic soils will never completely consolidate, and for such samples the time consolidation curve was the only means of determining when "one hundred" percent consolidation had occurred. Two graphs were drawn on semi-log paper (see drawing page 50), one showing vertical deflection in relation to elapsed time and the other showing volume change in relation to elapsed time. On each graph elapsed time was on a log scale and consolidation was considered complete when the curve tended to level off. This method was relied on when consolidating the silt samples, although none of the silt specimens were tested until vertical deflection was less than 0.0125% over a period of three hours.

In the tests on silt, some difficulty was encountered in saturating the specimens, and further problems arose in obtaining uniformity of samples. Remolded specimens were within limits, compacted to the same density at the same moisture content. To achieve this, the soil was oven-dried, screened through a number 4 sieve, and thoroughly mixed. It was then weighed and sprinkled and worked with such a quantity of water as to give it a moisture content of about 20%. The last operation was done quickly to prevent evaporation and thus assure a homogeneous material. Only enough material for one mold was left out, the rest placed in the moist room. The soil was quickly compacted, trimmed, wrapped in foil and dipped in wax before more samples were compacted.

The remolded samples were not saturated as required for the measurement of pore pressure and the following technique was devised to obtain saturation. Therefore, in addition to procedures already outlined for cohesive materials, the following steps were also adopted for the silt specimens.

A few grams of material were shaved off the ends of the test specimen and its moisture content determined by a rapid test. For the accuracy required here and because of the desiccated condition of the soil, it was found that a quick moisture content test, employing about one hour of drying at 105°C, was sufficient. After the moisture content was determined the soil was returned to the oven and reweighed 24 hours later as a check. Reweighing never indicated a significant change. The specimen was then measured and weighed prior to its placement on the pedestal. While it was saturating, its original moisture content was determined. Its original degree of saturation was also determined and from this information the weight of the test specimen at 100% saturation was calculated. Water was drawn up through the sample by vacuum, and after about two hours the specimen (remaining on the base) was weighed to ascertain its degree of saturation.

After the sample was set up in the triaxial chamber and during consolidation under confining pressure, vertical deflections and volume changes were recorded against time. Here another problem arose, the possibility of a leaking membrane. If an excessive amount of water was discharged from the sample, the operation was halted long enough to place another membrane over the existing membrane and to remeasure the sample. The measured change in volume and the recorded change in volume were compared. Further, at the end of a test the sample was reweighed (before removal from the base) and its weight change compared with its change in volume. However, it was found that excessive discharge did not always indicate a ruptured membrane. Therefore, in the instance of a cohesive soil being consolidated where a disruption of consolidation might have upset the results, the test was carried out and checks for leaks were left until the end. In such instances, 100% consolidation was based on vertical deflections, as described.

Based on the vertical deflections, silt samples seemed to consolidate in about five or six hours. A 2.8 inch diameter specimen four inches high did not change volume by more than 1 cc. per hour at this 100% consolidation.

Drained and undrained tests were carried out. In both tests, vertical loading commenced after consolidation under confining pressure was complete. For the undrained test, loading took place at the rate of 16 pounds per minute; measurements of pore pressure and vertical deflections were recorded at definite time intervals (one minute) after the application of a new load. No drainage from the sample was permitted. In the drained test, vertical deflections and volume changes were recorded when consolidation from each load increment was complete. Zero pore pressures at the end of consolidation for each load increment made pore pressure measurements unnecessary.

DATA AND GRAPHS

Date: 24 March, 1959 Sample & Test No. 3

TRIAXIAL COMPRESSION TEST Sheet No. 1

DATA & SUMMARY SHEET SOIL MECHANICS LABORATORY

Description of Sample: Clayey sandy silt

Before Test

Wt. Specimen Wet + Membrane + Filter Strips 16124 gms.
Wt. Membrane + Filter Strips 15272 gms.
Wt. Specimen Wet = W_c 820 gms. After Sat. 852 gms.

Saturation

Method Vacuum
Pressure -14 #/in^2
Period 2 hrs.

Measured Dimensions

	Diam. in.	Net Diam. D_o in.	Area in^2
Top	2.82	2.80	A_t = 6.15
Cent	2.80	2.78	A_c = 6.08
Bot	2.86	2.84	A_b = 6.35

Thickness of Membrane = .01 in.
(Deduct twice to obtain net diam)

$$\frac{A_t + 2A_c + A_b}{4} = A_o = 6.17 \text{ in}^2$$

A_e = 5.90 in^2

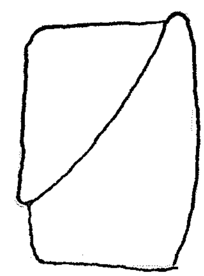
Height of Specimen = H_o = 4.20 in.
H_e = 4.06 in.

Vol. of Specimen V_o = 425 cc.
V_e = 388 cc.

After Test

Tare No. B 22a Weight = 39.9 gms.
Wt. Specimen Wet + Tare = 141.7 gms.
Wt. Specimen Dry + Tare = 125.6 gms.
Wt. Specimen Wet = W_e = 101.8 gms.
Wt. Specimen Dry = W_s = 85.7 gms.

Failure Sketch



Specific Gravity of Solids = G = 2.70

Volume of Solids = V_s = W_s/G = 253 cc.

Volume of Voids = V_v = V - V_s V_{vo} = 172 cc.
V_{ve} = 135 cc.

Weight of Water in Specimen = W - W_s W_{wo} = 170 gms.
W_{we} = 133 gms.

Degree of Saturation = V_w/V_v S_o = 99.0 %
S_e = 98.5 %

Void Ratio = V_v/V_s e_o = .680
e_e = .534

Water Content = W_w/W_s w_o = 24.9 %
w_e = 18.8 %

Dry Unit Weight = γ_{dry} = 110 lbs/ft^3

Remarks: Specimen weighed on base.

Sample & Test No. 3

TRIAXIAL COMPRESSION TEST
PRELIMINARY CONSOLIDATION
DATA SHEET

Sheet No. 2

SOIL MECHANICS LABORATORY

Increment of σ_c = 0 to 60 lbs/in^2

Date	Time	Elapsed Time Min.	Temp. C	Dial Reading R	Pipette Reading cc.
24 March/59	9:25	0	68	800	50
		00:05"		790	49
		00:10"		775	48
		00:20"		770	46
		00:40"		764	
		00:60"		756	43
		02:00"		747	40
	9:30	05:00"		730	37
	9:35	10:00"		705	30
	9:45	20:00"		695	28
	10:05	40:00"		675	25
	10:25	60:00"		667	22
		120:00"		661	15
		210:00"		659	
		240:00"		659	
		300:00"		659	13

Conditions at End of Consol. Increment

Change in Vol. of Water = ΔV_w = 37 cc

Volume of Voids = V_{vc} = 135 cc

Consolidated Void Ratio = e_c = .534

Consolidated Area = A_c = 5.90 in^2

If $\Delta V_w = \Delta V_v$: $V_{vc} = V_{vo} - \Delta V_w$ and $A_c = \dfrac{V_o - \Delta V_w}{H_o - \Delta H_c}$ $e_c = V_{vc}/V_s$

If $\Delta V_w \neq \Delta V_v$: $A_c = A_o \dfrac{H_o - 2\Delta H_c}{H_o}$; $V_c = H_c \times A_c$; $V_{vc} = V_c - V_s$

TRIAXIAL TEST - DATA SHEET

PROJECT M.Sc. Thesis SAMPLE NO. 3 Silt DATE 24 March, 1959

CELL PRESSURE 60 p.s.i.

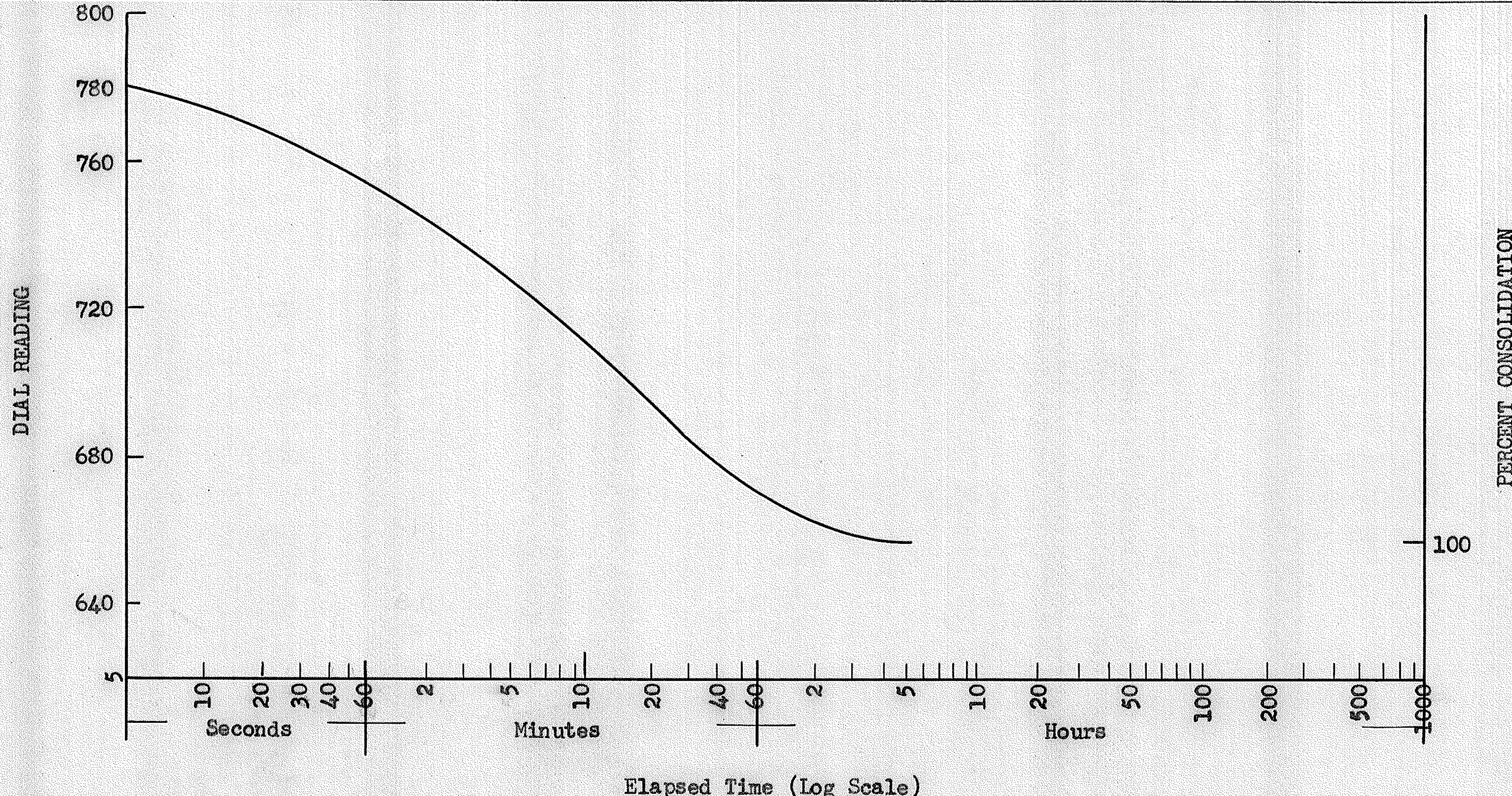
Time	Elapsed Time (Min.)	Load	h ins. Strain Dial	Unit Strain %	1 minus unit strain	Corrected Area	Axial Stress	Burette Reading cm^3	Pore Water Press. lb/in^2	Eff. Lat. Stress	Eff. Vert. Stress
2:30	0	0	0	0	1.0	5.90	60.0		0	60.0	60.0
PM	2½	40	.002	.049	.999	5.90	67.3		0	60.0	67.3
	5	80	.006	.144	.999	5.90	74.0		1.0	59.0	73.0
	7½	120	.010	.300	.997	5.92	81.1		1.8	58.7	79.3
	10	160	.018	.49	.995	5.93	88.0		2.5	57.5	85.5
	12½	200	.038	.94	.990	5.95	94.8		4.5	55.5	90.3
	15	240	.058	1.39	.986	5.97	101.6		6.4	53.6	94.2
	16	256	.060	1.49	.985	5.98	104.3		7.2	52.8	97.1
	17	272	.077	1.85	.981	6.02	107.0		8.4	51.6	98.6
	18	288	.089	2.14	.979	6.03	109.4		15.8	44.2	93.6
	19	304	.091	2.19	.978	6.03	112.2		19.2	40.8	93.0
	20	320	.098	2.38	.976	6.04	113.0		20.8	39.2	92.2
	21	336	.098	2.38	.976	6.04	116.0		24.0	36.0	92.0
	22	352	.252	6.21	.938	6.28	116.0		24.0	36.0	92.0
	23	368	.344	8.50	.915	6.45	117.0		24.7	35.3	92.3
	24	384	.499	12.30	.877	6.73	117.0		24.9	35.1	92.1
	25	400	.706	17.40	.826	7.14	116.0		25.0	35.0	91.0
	26	416	.945	23.30	.767	7.70	114.0		25.0	35.0	89.0
	27	432	1.120	27.60	.724	8.15	113.0		25.0	35.0	88.0

Tested DH	Date: 24 March, 1959	Soil Mechanics Laboratory
Computed DH	Date: 24 March, 1959	Faculty of Engineering
Checked DH	Date: 24 March, 1959	University of Manitoba

Sample No. 3

CONSOLIDATION TEST TIME CURVE

Cell Pressure = 60 p.s.i.



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Graduate Studies 1959

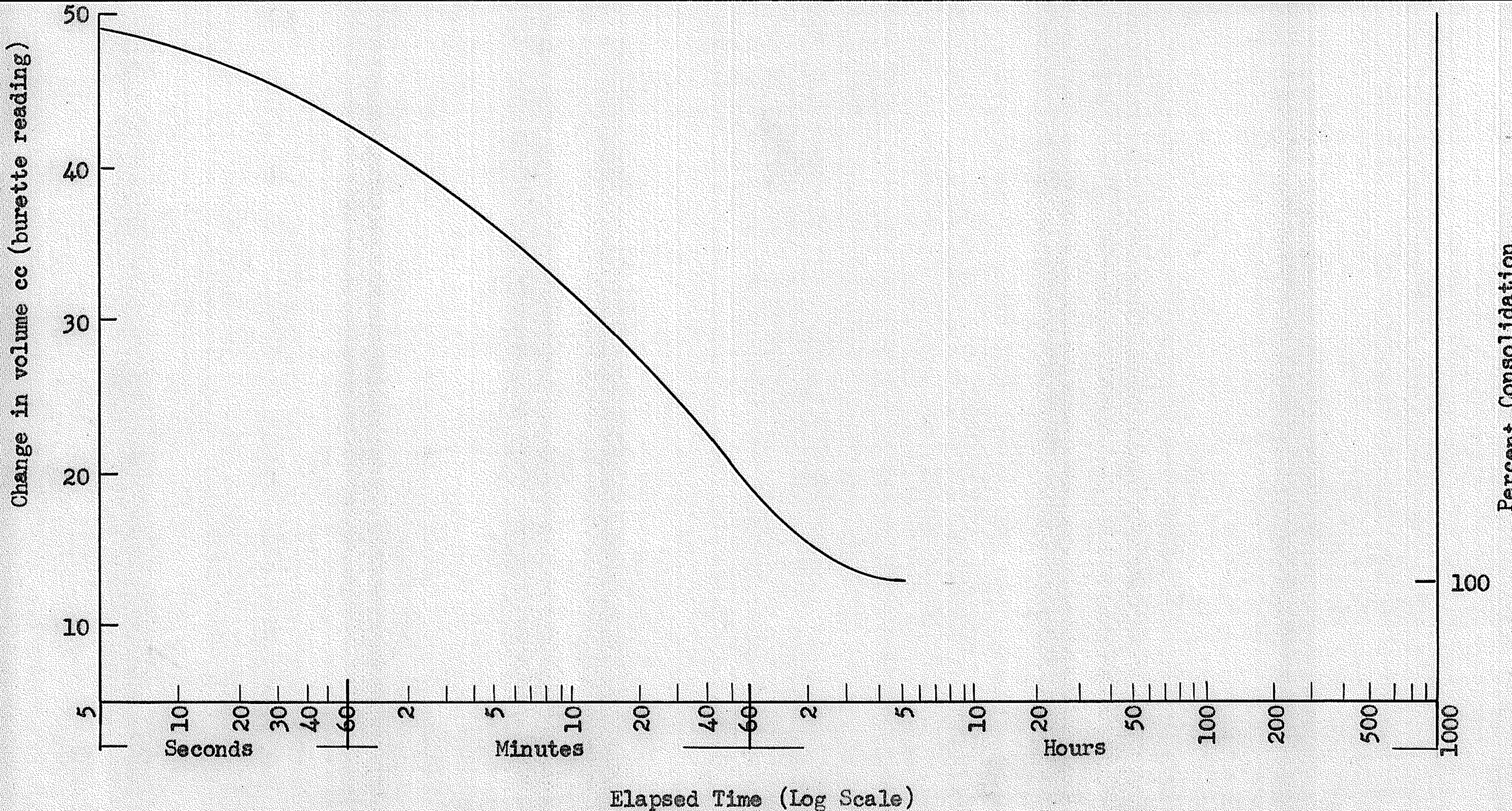
DH

Drawing No. 10

Sample No. 3

CONSOLIDATION TEST TIME CURVE

Cell Pressure = 60 p.s.i.



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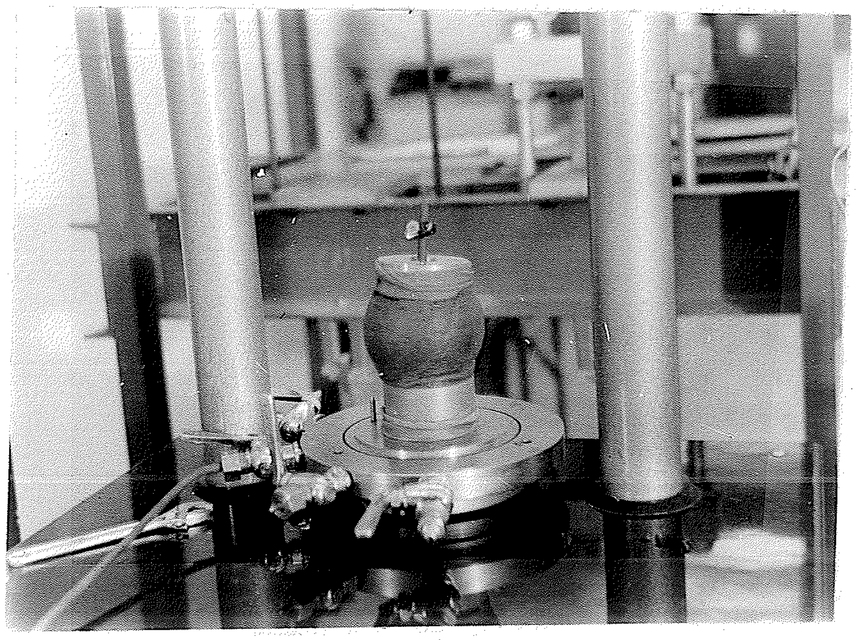
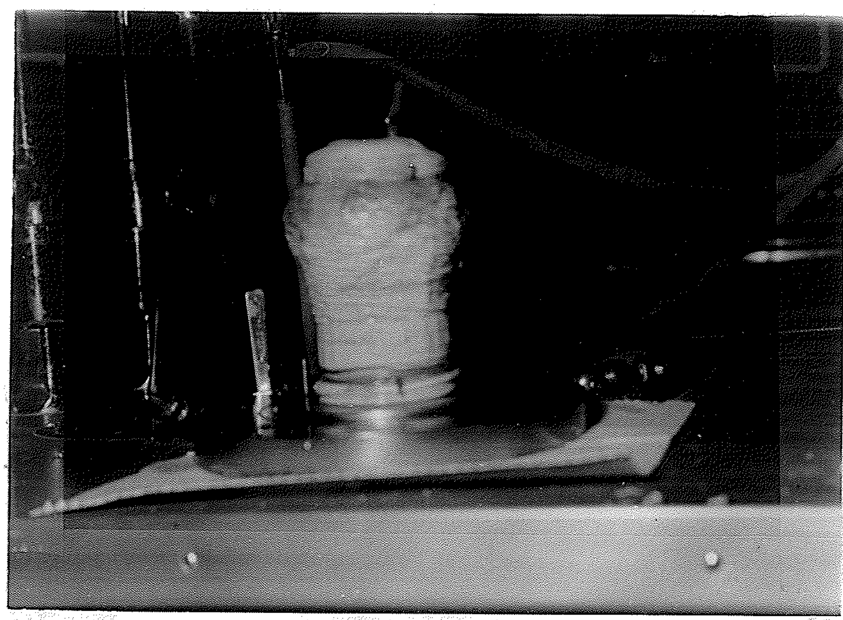
DH

Drawing No. 11

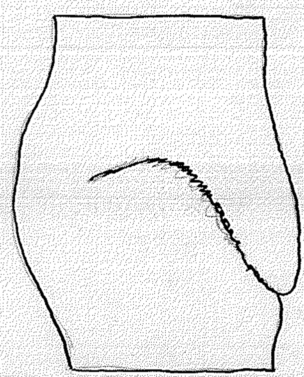


Typical Sand Failure

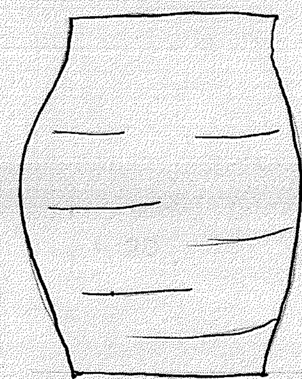


Typical Silt Failure

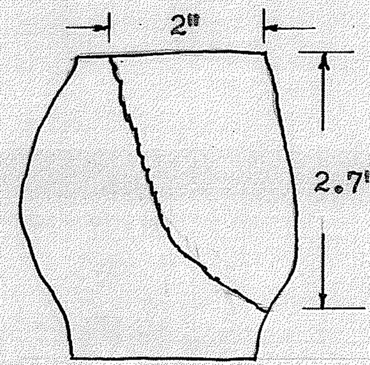
U. of M.
Dept. of C.E.
Grad. Studies 1959



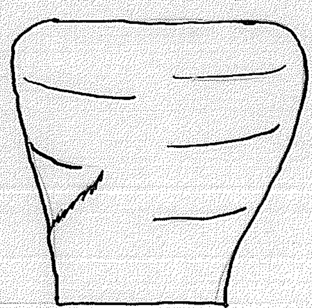
Drained sand
σ_3 = 30 p.s.i.



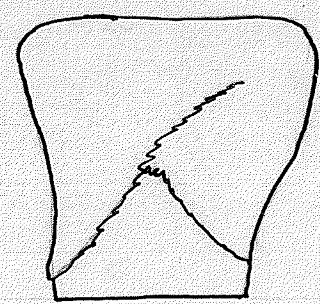
Undrained sand
σ_3 = 20 p.s.i.



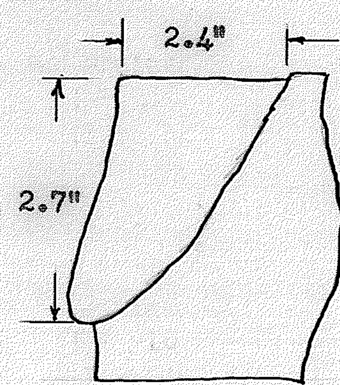
Drained sand
σ_3 = -10.8 p.s.i.
Failure Angle = 53.4°



Undrained silt
σ_3 = 30 p.s.i.

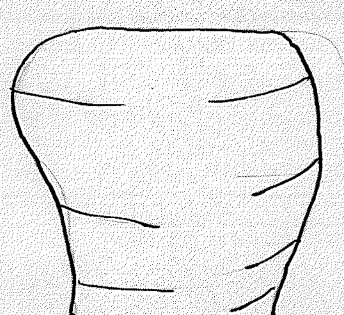


Undrained silt
σ_3 = 45 p.s.i.

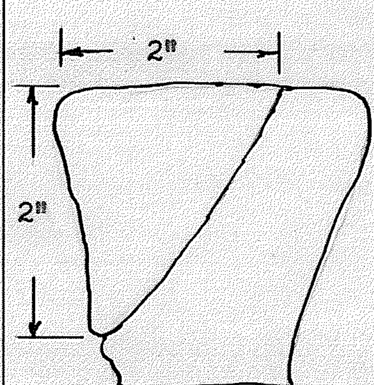


Undrained silt
σ_3 = 60 p.s.i.

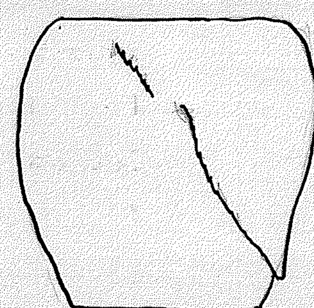
Failure Angle = 48.3°



Drained silt
σ_3 = 30 p.s.i.



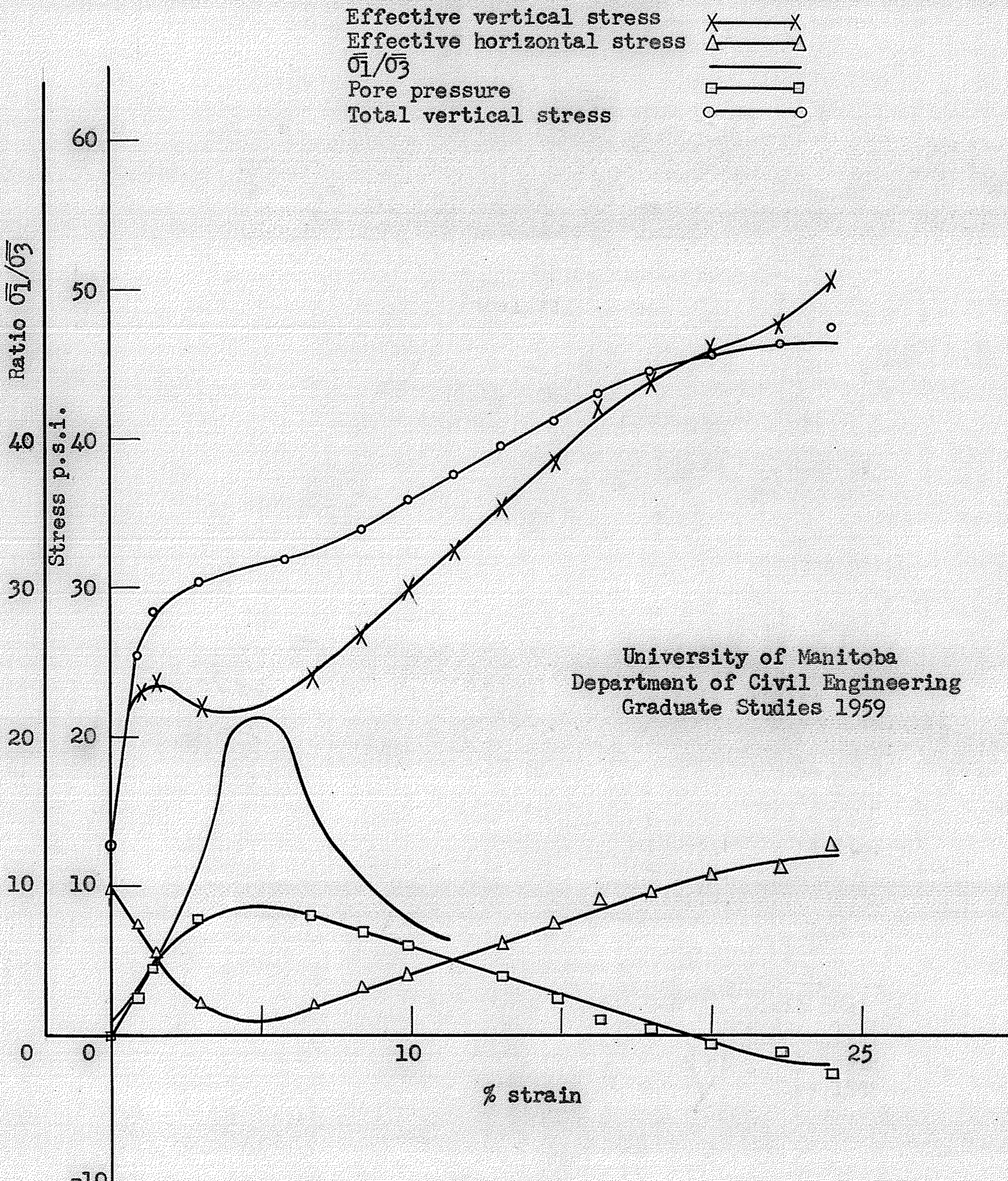
Drained silt
σ_3 = 45 p.s.i.
Failure Angle = 45°



Drained silt
σ_3 = 60 p.s.i.

Curves showing total stress, effective stress, pore pressure and $\overline{\sigma_1}/\overline{\sigma_3}$ vs strain for an undrained sample of sand.

Cell pressure 10 p.s.i.



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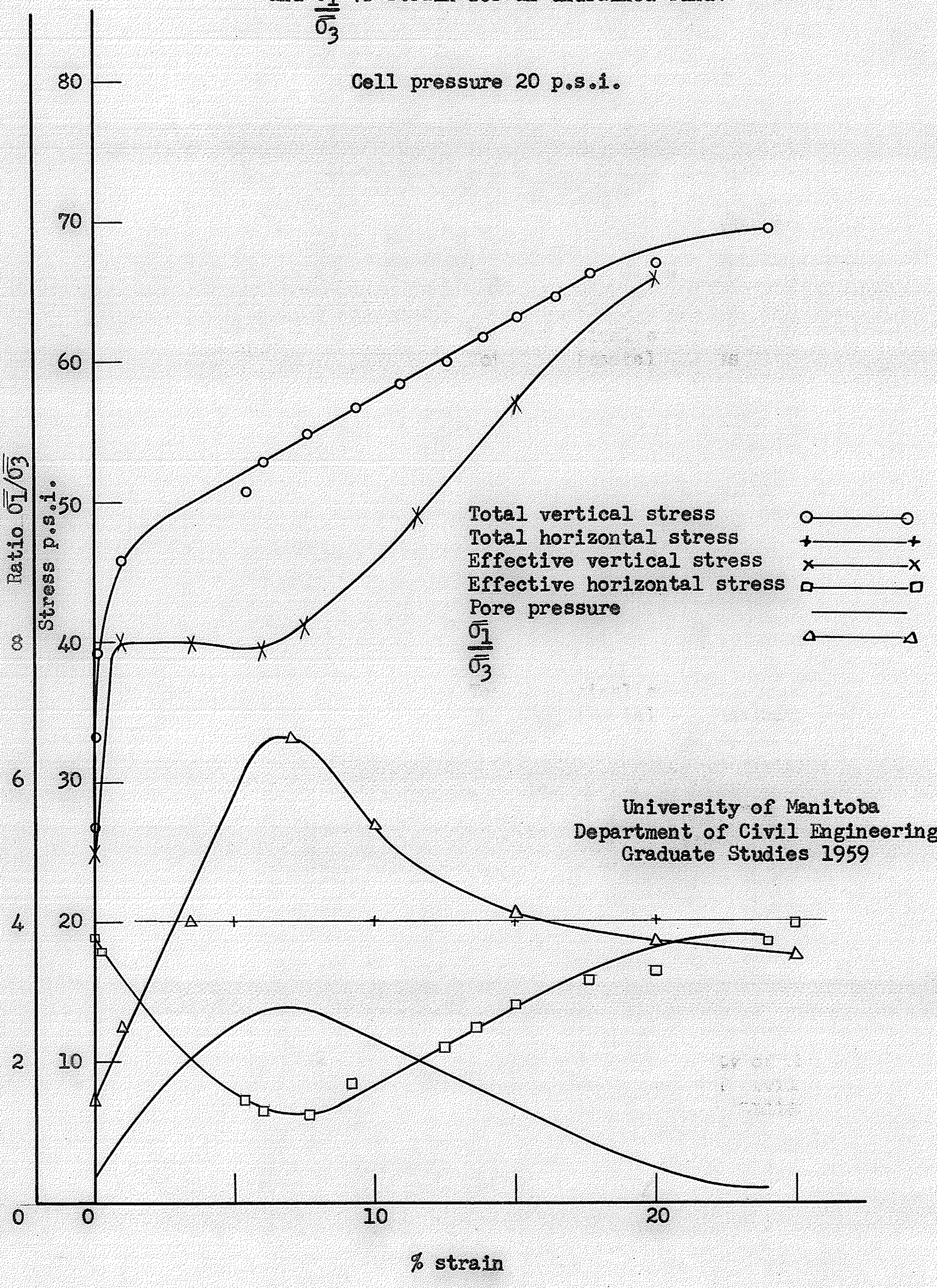
Curves showing σ_1, σ_3, $\bar{\sigma}_1$, $\bar{\sigma}_3$, pore pressure and $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$ vs strain for an undrained sand.



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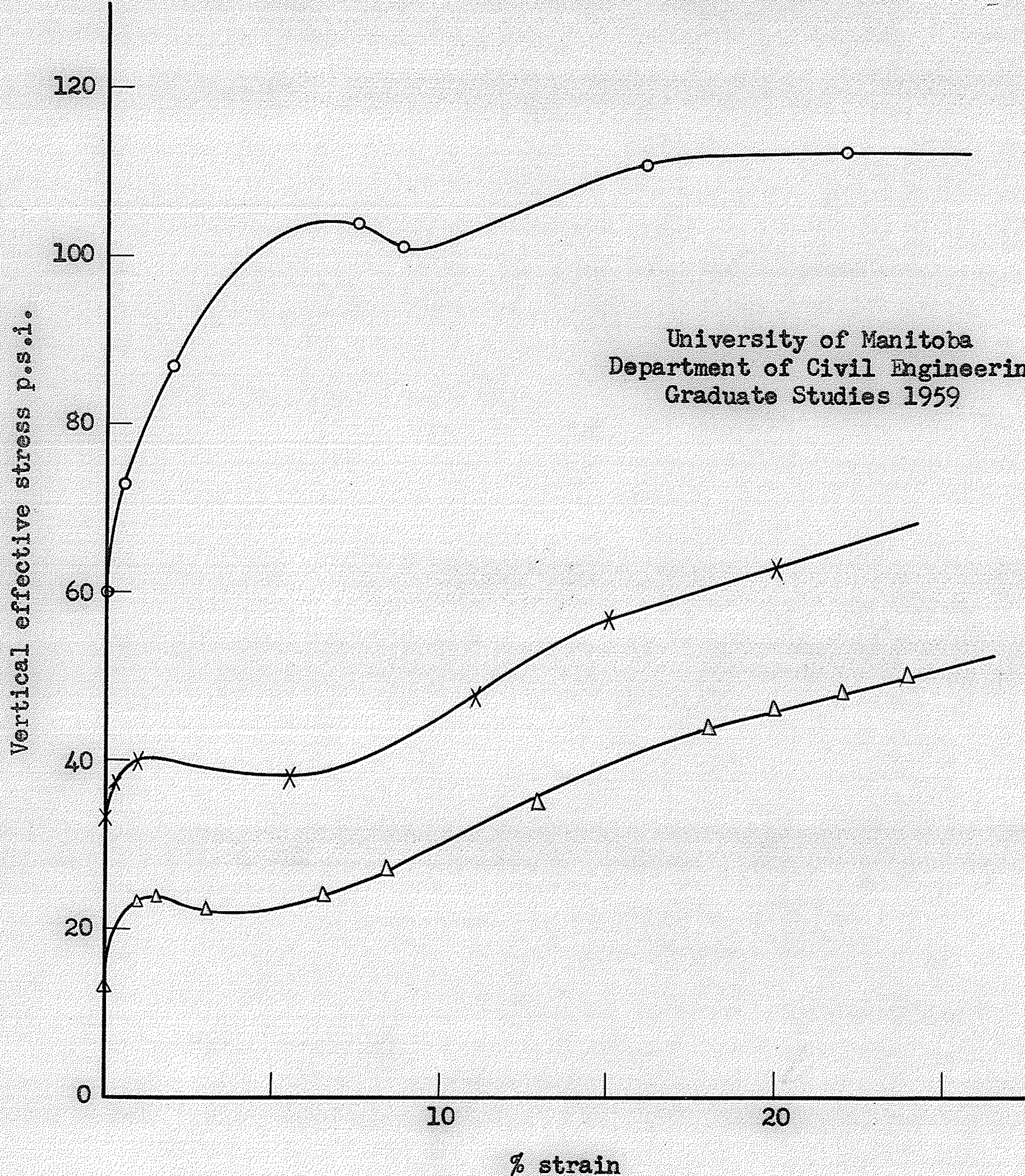
Drawing No. 14

Curves showing effective vertical stress
vs strain for undrained samples of sand.

σ_3	G	S start	S end	e start	e end	MC start	MC end	γ_{dry} #/ft^3	
10	2.65	86.1	85.5	.747	.720	24.3	23.3	96.5	△——△
20	2.65	86.0	85.2	.741	.703	24.1	22.7	97.2	×——×
30	2.65	86.3	85.0	.748	.692	24.5	22.4	97.7	○——○



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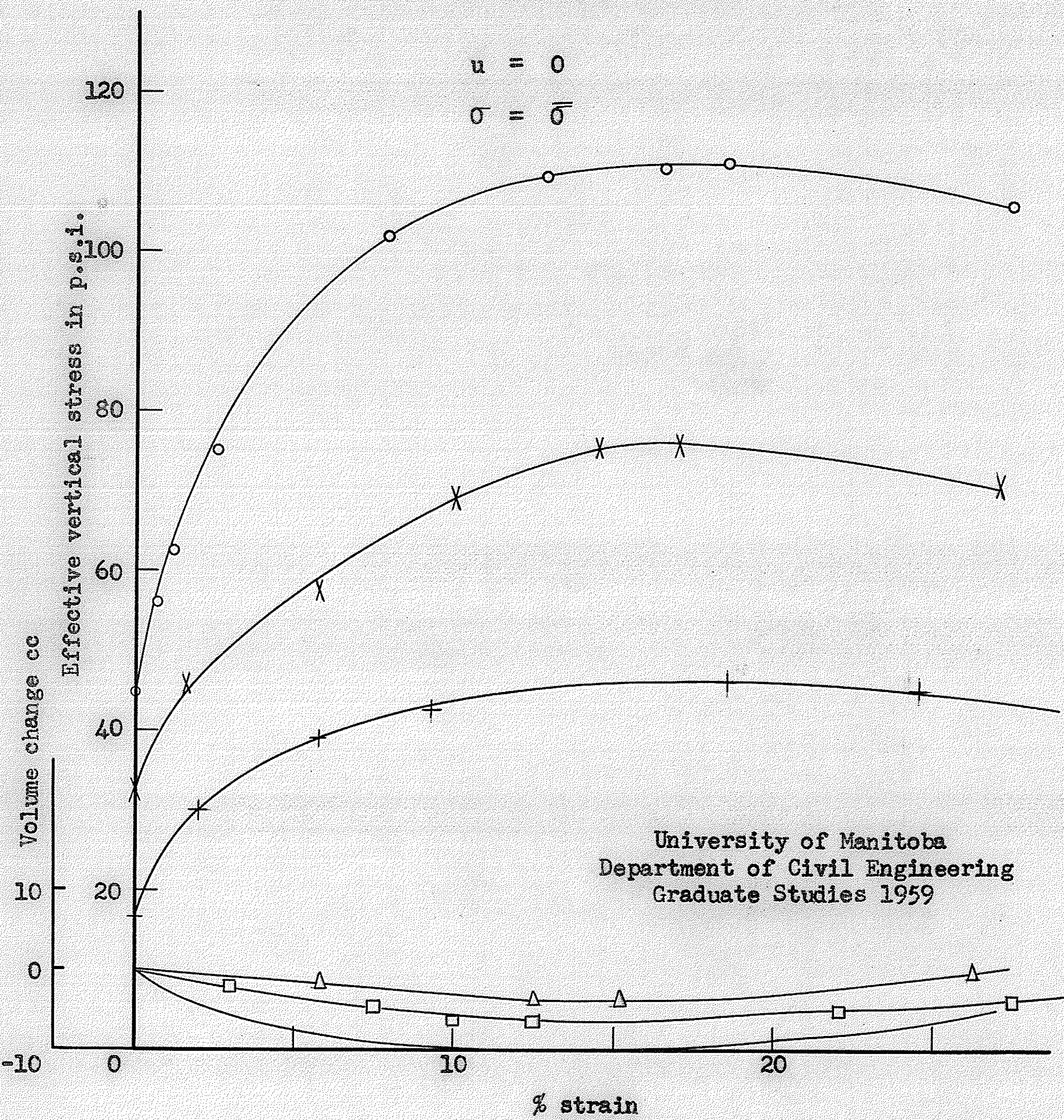
Drawing No. 15

Curves showing effective vertical stress and volume change vs strain for drained samples of sand.

σ_3	G	S start	S end	e start	e end	MC start	MC end	γ_{dry} #/ft^3	
30	2.65	86.3	84.7	.746	.681	24.4	21.8	98.7	o——o
20	2.65	85.9	85.0	.740	.700	24.0	22.7	97.2	X——X
10	2.65	86.1	85.2	.747	.700	24.2	22.6	97.1	+——+

Volume change cell pressure 10 p.s.i. △——△
Volume change cell pressure 20 p.s.i. □——□
Volume change cell pressure 30 p.s.i. ——



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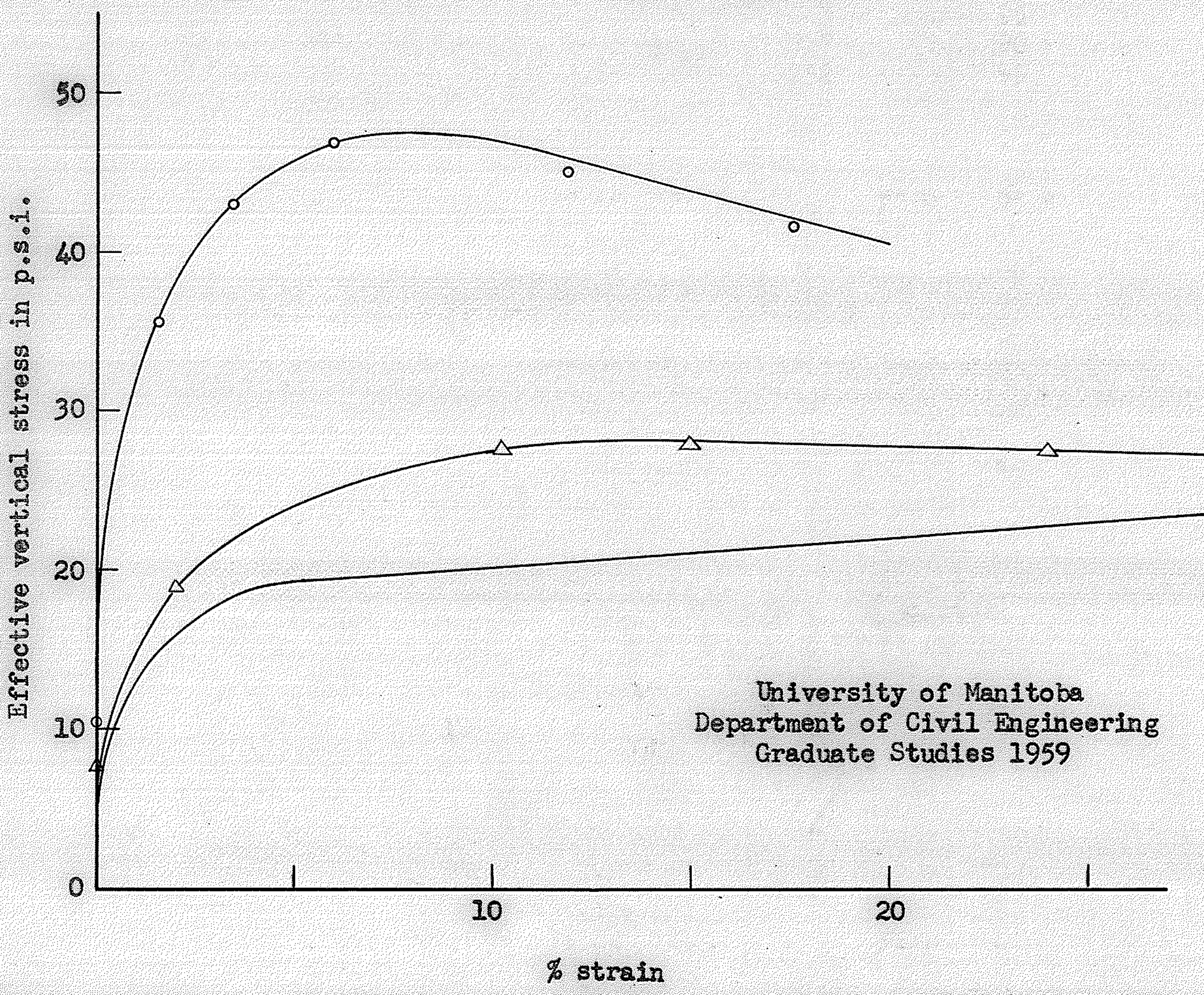
Curves showing vertical effective stress vs strain for sand samples with negative pore pressure

σ_3	G	S	e	MC	Γ_{dry} #/ft^3	
-5.6	2.65	82.0	.743	23.1	95.0	———
-7.4	2.65	84.5	.717	22.9	96.8	△——△
-10.8	2.65	85.0	.752	22.6	95.0	○——○

$$\bar{\sigma}_1 = \sigma_1 + u$$
$$\bar{\sigma}_3 = \sigma_3 + u$$



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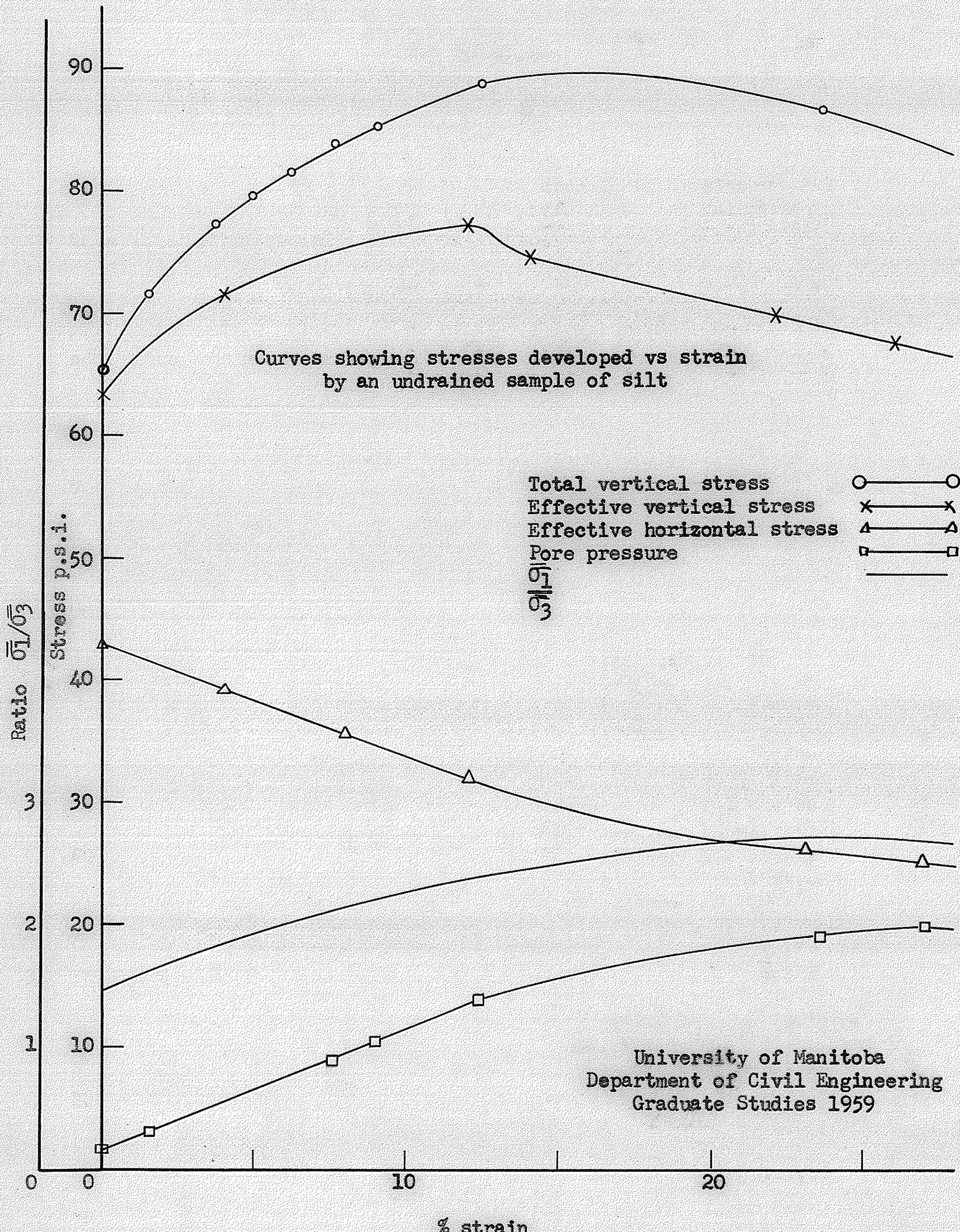
Drawing No. 17

Sample No. 2

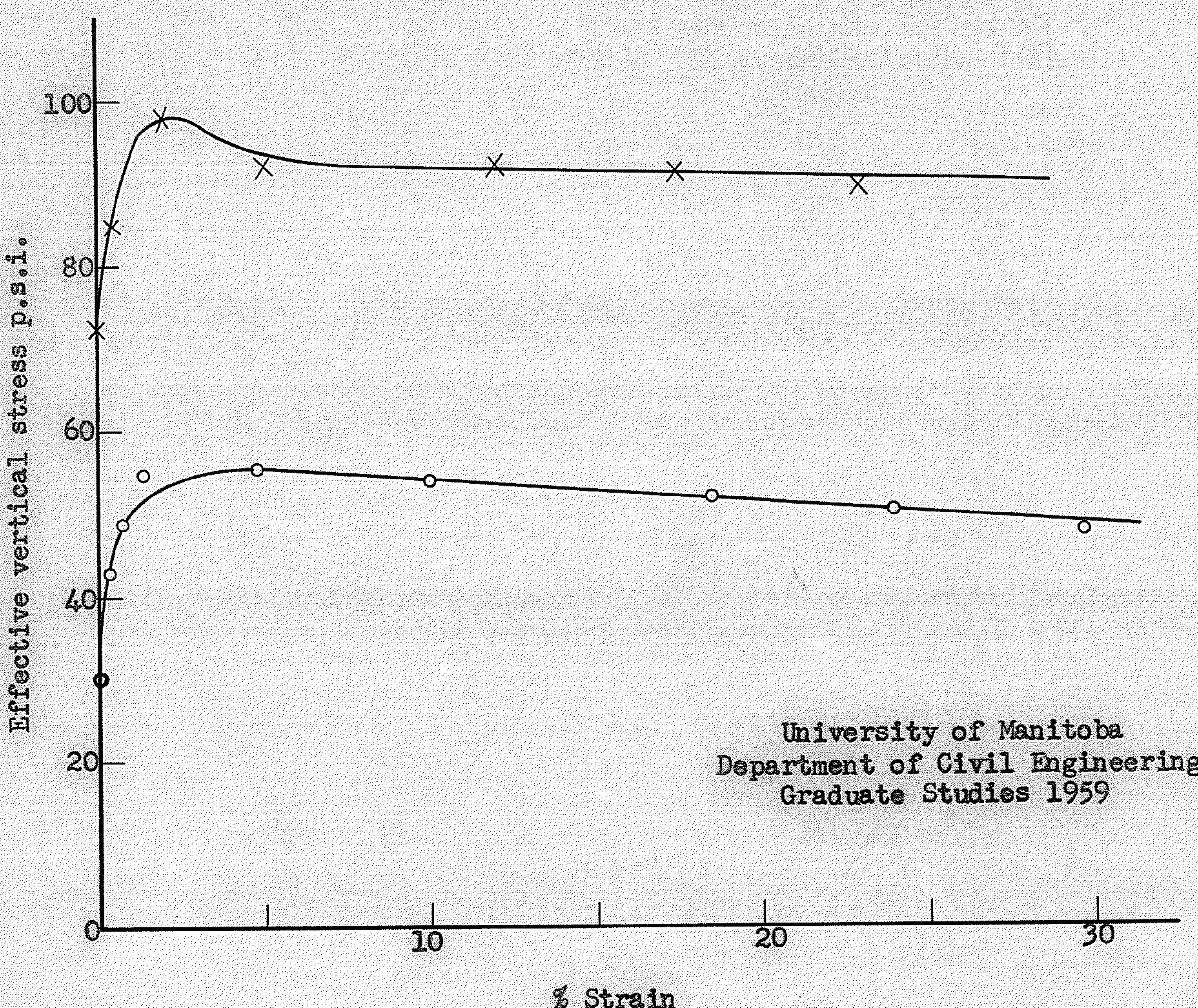
	G	S	e	MC	γ dry
start	2.70	91.9	.775	26.4	
end	2.70	90.0	.662	23.0	104



Effective vertical stress vs strain
for undrained samples of silt

$$\bar{\sigma} = \sigma - u$$

σ_3	G	S start	S end	e start	e end	MC start	MC end	γ_{dry} #/ft^3	
30	2.70	99.75	99.5	.747	.546	27.7	20.1	109	o———o
60	2.70	99.00	98.5	.680	.534	24.9	19.3	110	x———x

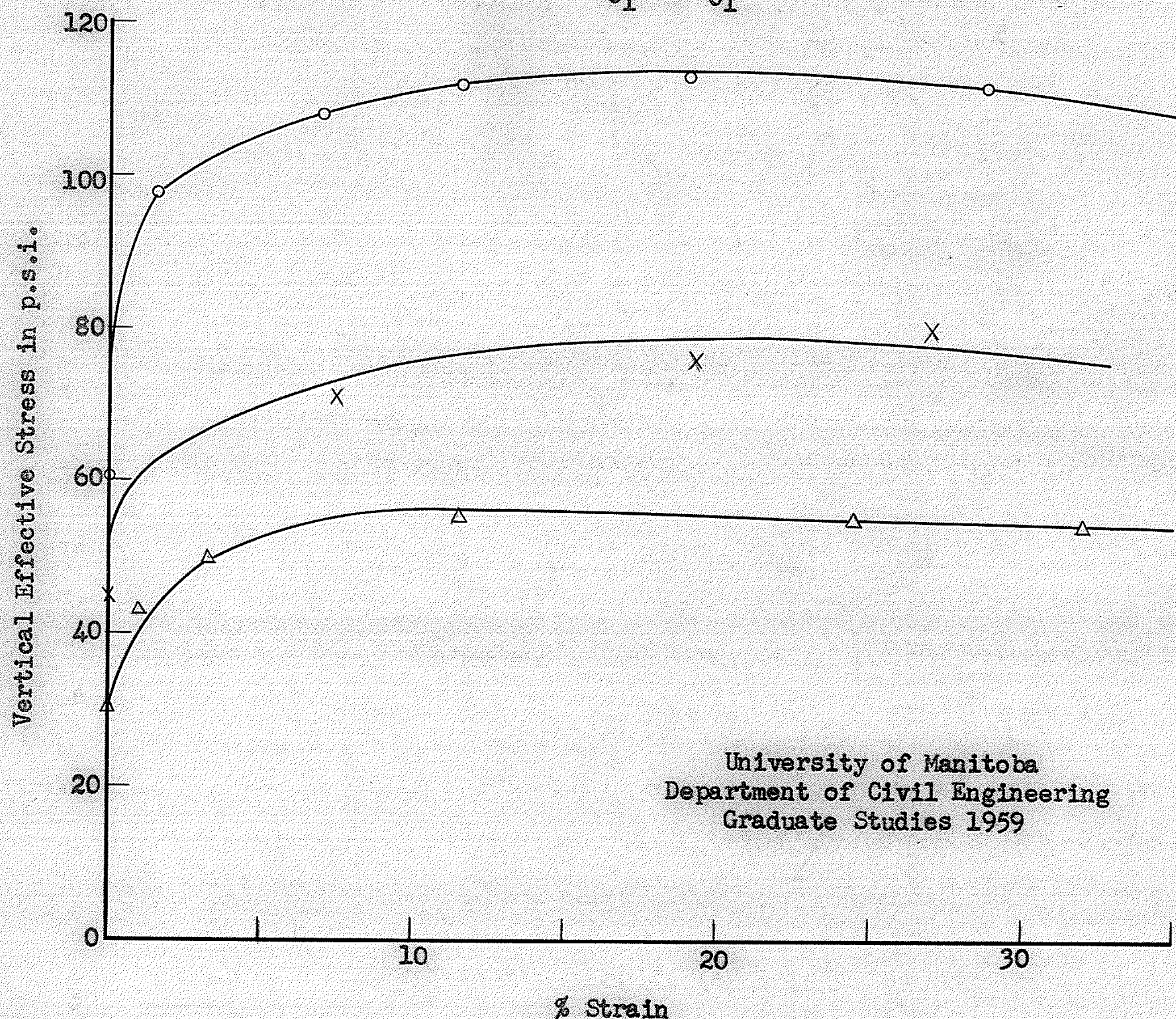


Vertical effective stress = total stress
Pore pressure = 0

Curves showing vertical effective stress vs strain at three different confining pressures for drained samples of silt.

σ_3	G	S start	S end	e start	e end	MC start	MC end	γ_{dry} #/ft^3	
30	2.70	91.0	89.5	.717	.621	24.2	20.7	104	△——△
45	2.70	93.2	92.5	.674	.605	23.3	20.7	105	×——×
60	2.70	93.8	92.0	.748	.576	26.0	19.6	107	○——○

$\bar{\sigma}_1 = \sigma_1$



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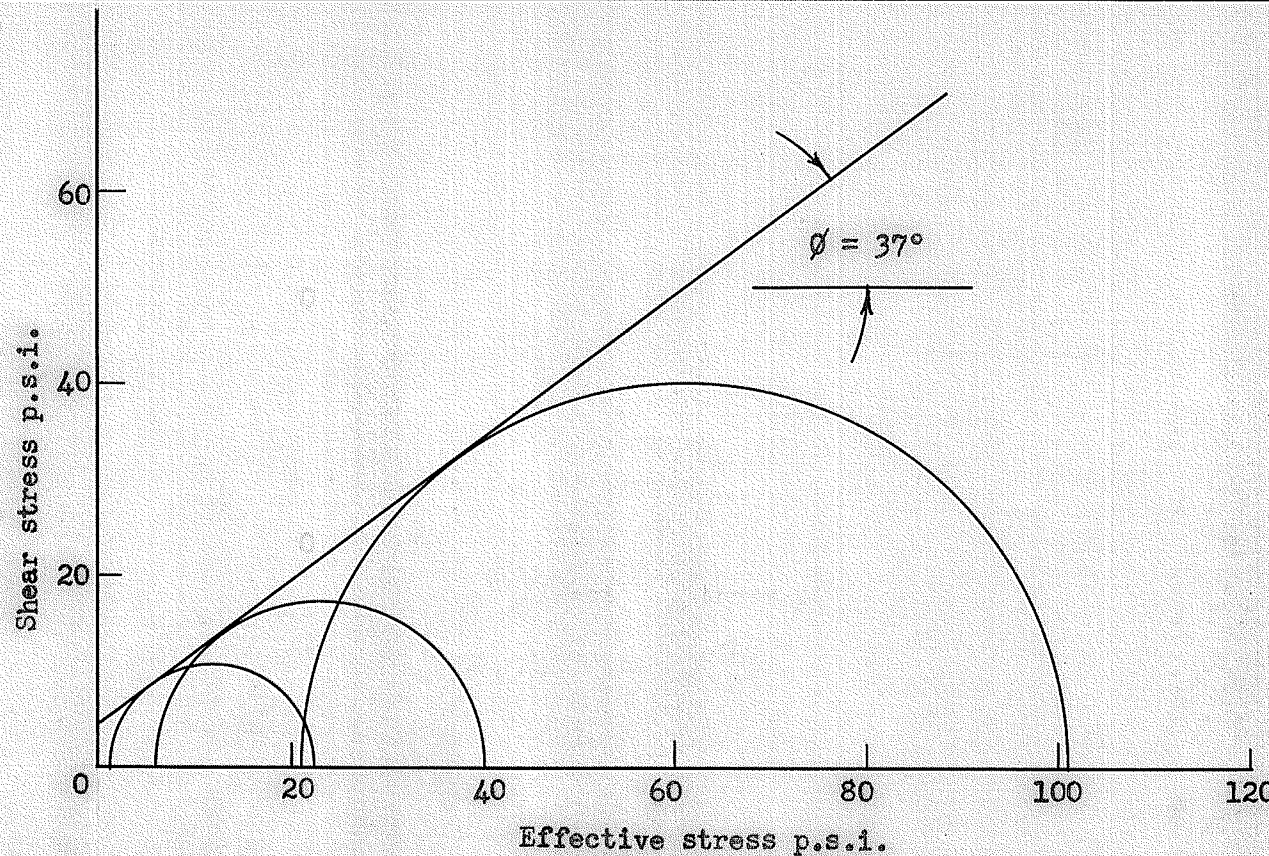
Drawing No. 20



Mohr's rupture envelope for effective stresses for undrained samples of sand at maximum $\frac{\overline{\sigma_1}}{\overline{\sigma_3}}$

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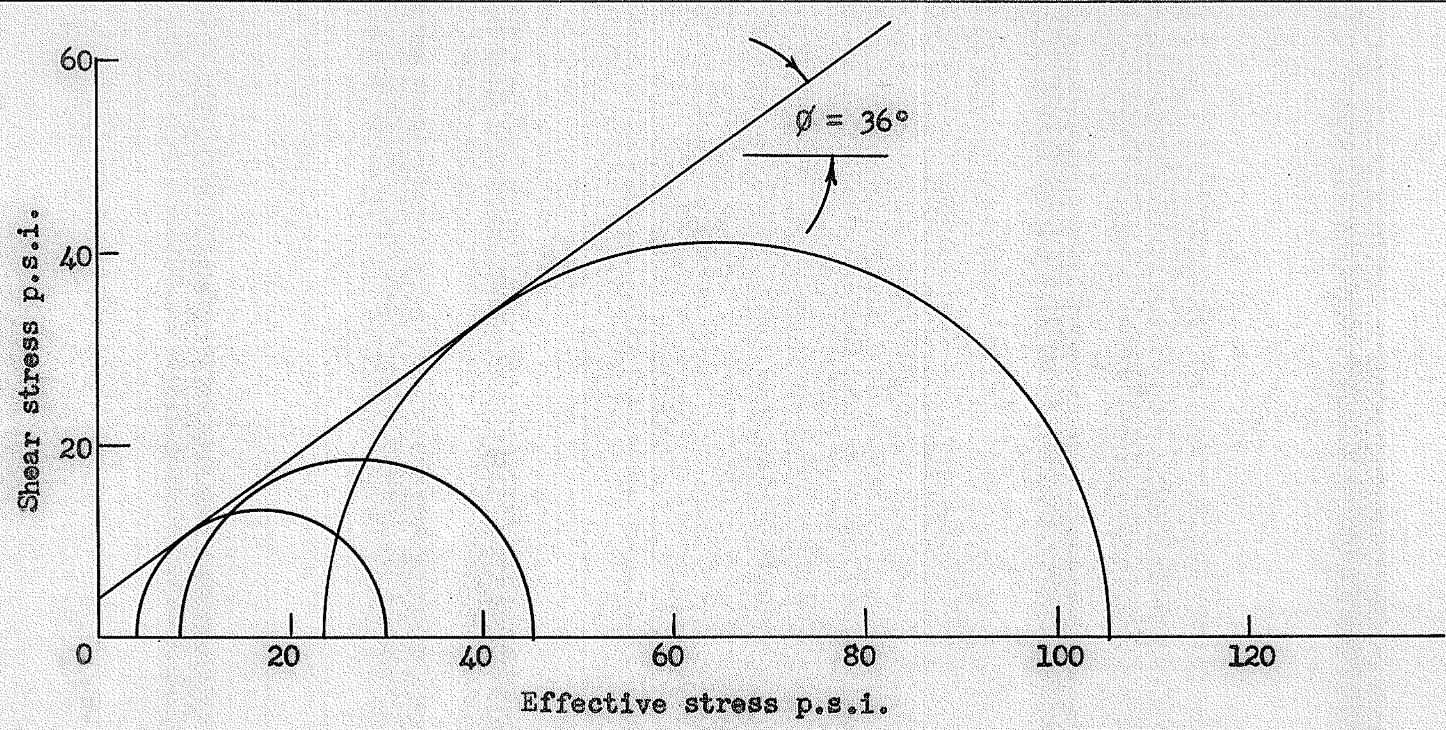
Drawing No. 21



Mohr's rupture envelope for effective stresses for undrained samples of sand at 10% strain.

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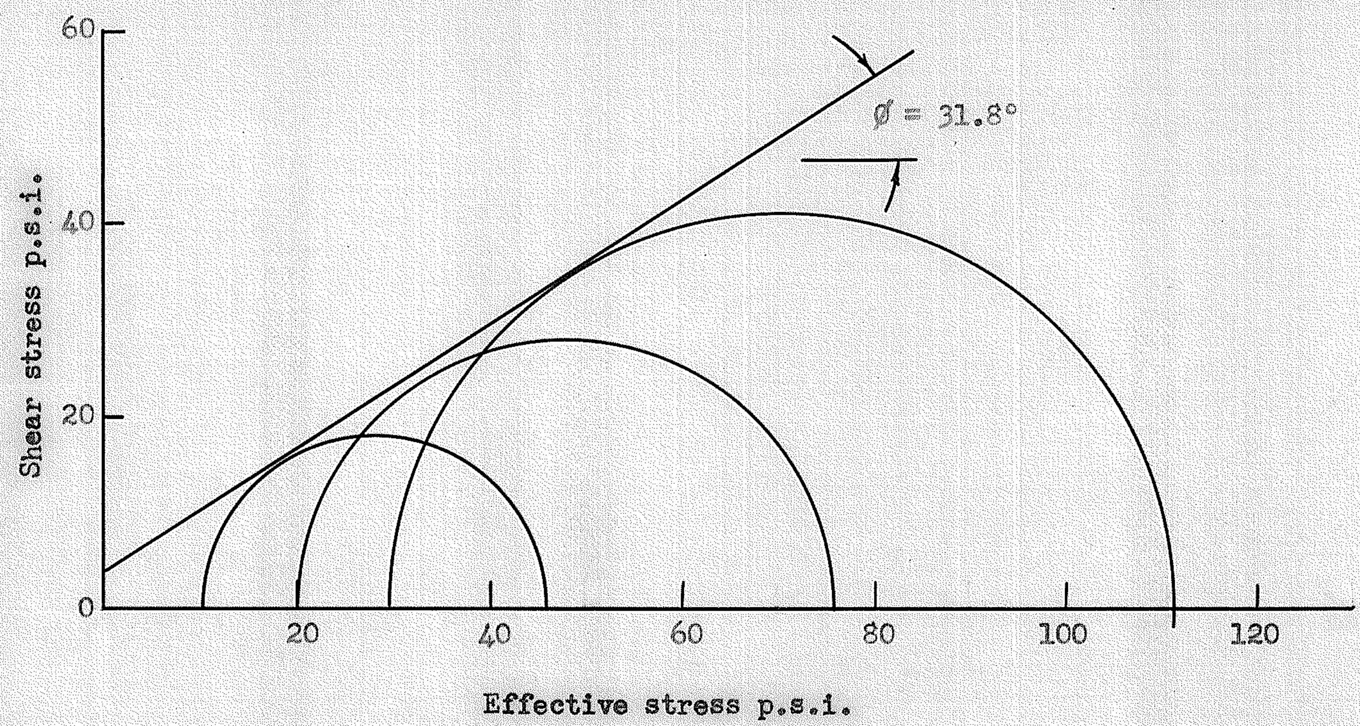
Drawing No. 22



Mohr's rupture envelope for drained sand at 16% strain = maximum strain. Total stress = effective stress.

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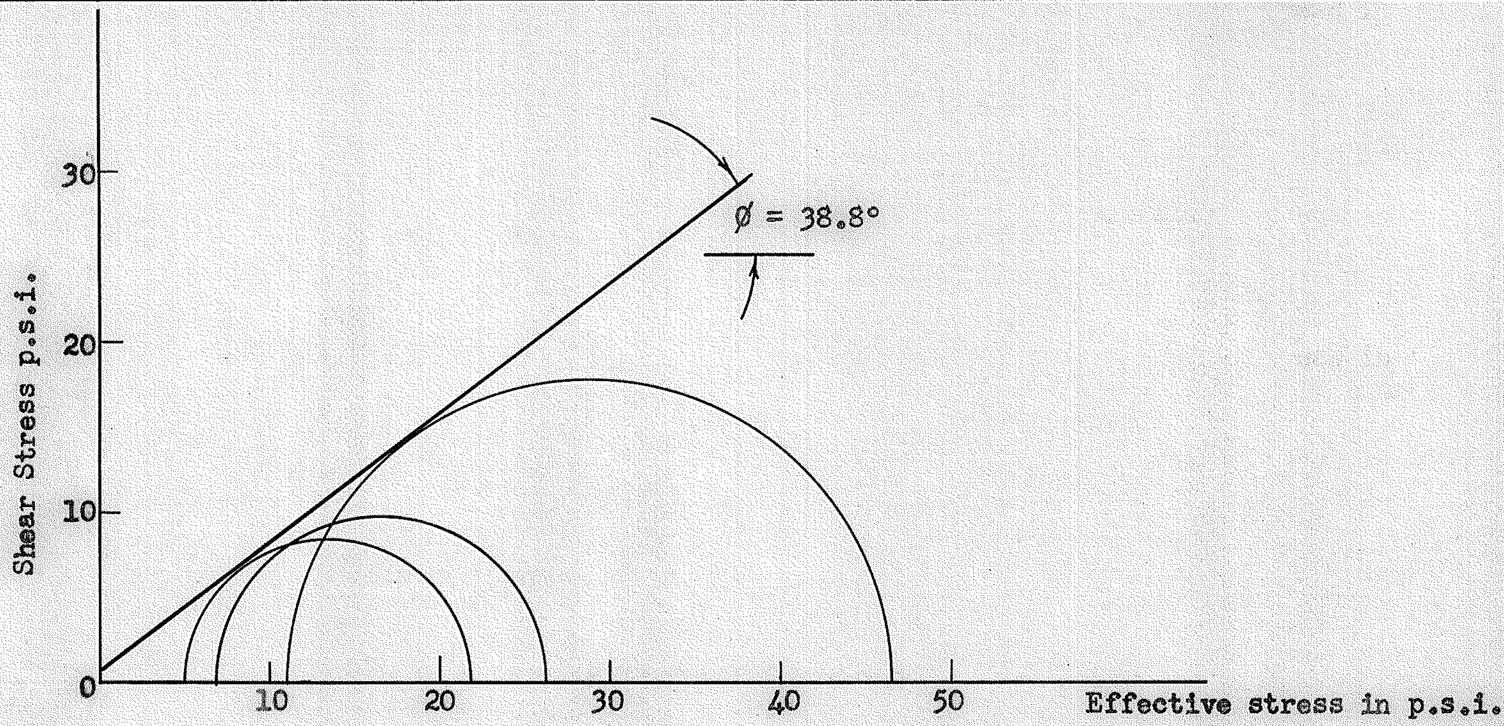
Drawing No. 23



Mohr's Rupture Envelope for Sand Samples
with Negative Pore Pressure
Effective Stress = Total Stress

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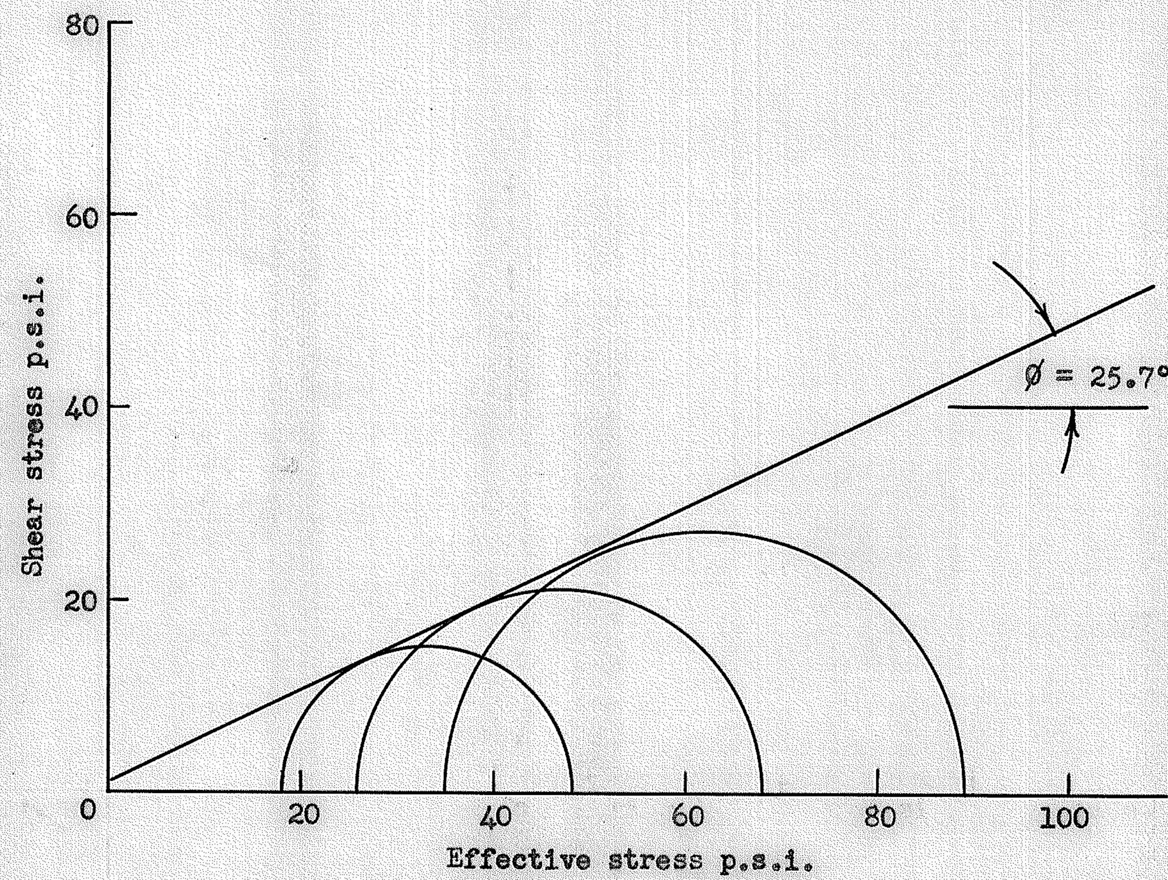
Drawing No. 24



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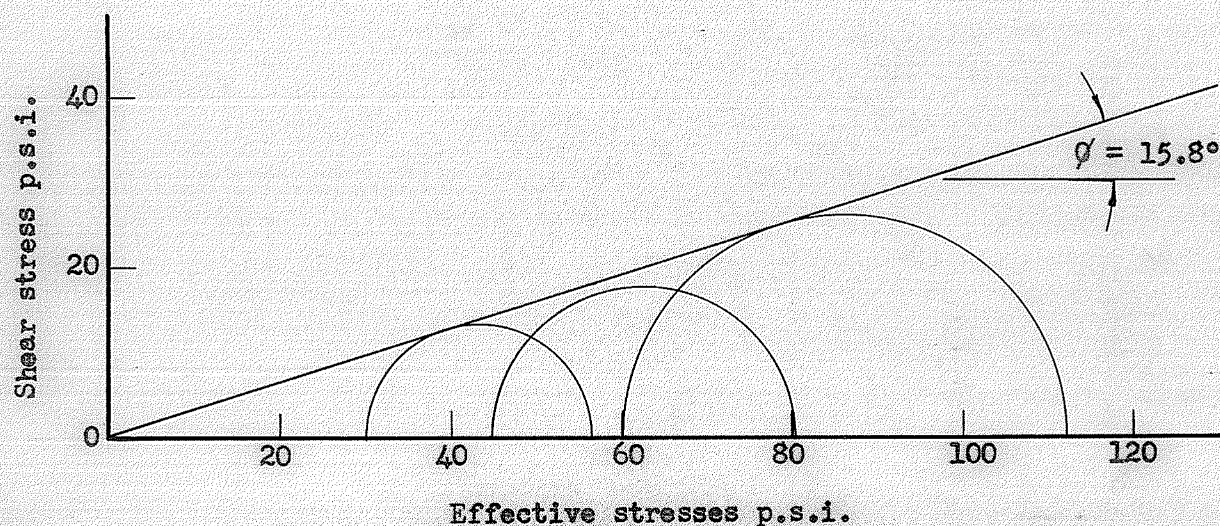
Mohr's rupture envelope for effective stress for undrained samples of silt at maximum $\frac{\overline{\sigma_1}}{\overline{\sigma_3}}$

DH

Drawing No. 25



Mohr's rupture envelope for effective stresses
on drained samples of silt

$$u = 0$$

$$\sigma = \bar{\sigma}$$

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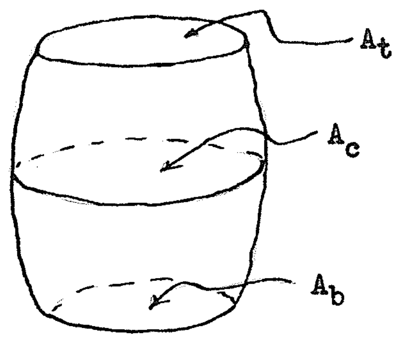
Drawing No. 26

Calculations

The volume of the sample was determined by multiplying its measured height (H_o) by its average measured area (A_o). The area was determined by the following method.



Average area between top and centre $= \dfrac{A_t + A_c}{2}$

Average area between bottom and centre $= \dfrac{A_c + A_b}{2}$

Average area (A_o) through the top and bottom sections $= \dfrac{\dfrac{A_t + A_c}{2} + \dfrac{A_c + A_b}{2}}{2}$

$$2(2\,A_o) = A_t + A_c + A_c + A_b$$

$$= A_t + 2A_c + A_b$$

$$A_o = \frac{A_t + 2A_c + A_b}{4}$$

After the sample had been consolidated, its new volume (V_e) was obtained by subtracting its volume change according to the burette, and its new height (H_e) was obtained by subtracting the vertical deflection from H_o. Then the area after consolidation (A_e) was equal to $\dfrac{V_e}{H_e}$.

In the case of a drained test, since the volume was changing during loading, a new area was calculated for each volume change. For undrained tests, where volume was constant, the area was determined as follows

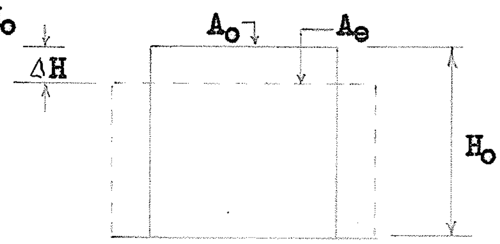
$$A_o = \text{area at load commencement}$$

$$A_e = \text{area when vertical deflection for load taken}$$

$$V = A_o H_o$$

$$V = A_e (H_o - \Delta H)$$

$$\text{Therefore } A_e (H_o - \Delta H) = A_o H_o$$



$$A_e = \frac{A_o H_o}{H_o - \Delta H}$$

$$= \frac{A_o}{\frac{H_o - \Delta H}{H_o}}$$

$$= \frac{A_o}{1 - \frac{\Delta H}{H_o}}$$

$$\frac{\Delta H}{H_o} = \text{unit strain}$$

$$\text{Therefore } A_e = \frac{A_o}{1 - \text{unit strain}}$$

DISCUSSIONS AND CONCLUSIONS

For undrained tests on sand, failure occurred shortly after the pore pressure reached a maximum[9], although this is not evident from the vertical effective stress-strain curves. At failure the sample expanded and, sand being quite porous, dissipated the excess internal pressure, which resulted in the drop in pore pressure. The sample confined at 10 p.s.i. indicated negative pore pressure at the end of the test, indicating that the volume had so increased as to place the pore water in tension.

The effective stress-strain curves for drained tests on sand (Drawing No. 16) indicated failure more positively. In each test, the sample lost volume continuously until failure, and then gained volume. It was noticed that the minimum volume occurred at maximum vertical effective stress. For the undrained tests, the relation between strain and $\bar{\sigma}_1/\bar{\sigma}_3$ was drawn and Mohr's rupture envelope drawn for the maximum point on the curve. It was also drawn for 10% strain but the angle of internal friction was almost the same in each case, but slightly less for the latter. This agrees closely with results obtained by Taylor[10].

The tests using negative pore pressures were quite unique, in that they were drained, and yet pore pressure measurements were made. In these tests, negative pore pressure increased, accompanied by expansion of the sample until failure occurred, and after failure the negative pressure decreased with accompanying sample contraction.

The values of vertical effective stress correspond closely with their counterparts in the drained test, but the angle of internal friction, 38.8°, was much higher.

The actual degree of saturation for the sand samples was probably much higher than recorded. Most of the discrepancy probably arose from the fact that a considerable amount of sand and water was lost in handling the sample. Because of its porosity, water drained out of the sample when it was removed from the base and sand adhered to the porous disc on the pedestal, loading cap and membrane. To eliminate this error (subsequent observations indicated that as much as 30 grams of sand and 20 cc of water could have been lost), sand samples should be weighed while still on the base, then removed to an evaporating dish, and all solids clinging to the apparatus washed off into the evaporating dish.

It was found that the pore pressure in silt specimens did not drop after failure, but tended to remain constant. This was probably due to the cohesive and relatively impervious nature of silt. With sand, the sample not only became less dense but was also quite porous, readily permitting the dissipation of pore pressure. The silt grains, being much finer, would not exhibit these properties to the same extent and therefore would have a tendency to retain the pore pressure, even after failure.

One source of possible error was ensuring 100% consolidation. If consolidation was not complete when the null indicator was connected, the confining pressure causing consolidation would be transmitted to the pore pressure device and render ambiguous results. Observations were made with regard to pore pressure change prior to loading a specimen and consolidation continued until the pore pressure tended to remain constant. Another possible source of error was in obtaining a suitable load increment. With drained tests, it was found that an excessive load increment would create pore pressures that could not be dissipated fast enough to prevent premature failure (several specimens failed very obviously in this fashion and therefore the data recorded therefrom could not be used). When a premature failure was obvious, the results could be disregarded, but when such a failure was not obvious but had commenced, the maximum vertical stress for the specimen was not reached. It was found that load increments of 40 pounds could be used until the axial load was approximately three times the confining pressure and 16 pound increments thereafter.

The angle of internal friction based on effective stresses for the undrained test on silt (25.7°) was much greater than for the drained test (15.8°). This generally agrees with observations made by J. E. Roberts[11] on fine grained soils.

Generally the results agreedclosely with those observed by Taylor[12] and brought out the major aspects of porewater pressure measurements. Silt, being much finer than sand, was not as easy to work with but, since it was a cohesionless material, behaved in a similar fashion. It gave some indication as to methods of testing other fine grained soils such as clays. The information gathered from the series of tests pointed out the importance of pore pressure studies from a theoretical viewpoint, and indicated the technical procedure to be taken. Excess hydrostatic pore pressures are dependent on the volume change tendencies of a soil under loading. In designing earth structures and foundations, excess hydrostatic pore pressure is highly significant.

1. If positive pore pressure is developed during shear, the strength will increase if pressure dissipates with time, but will decrease if it does not.
2. If negative pore pressure developed during shear, the soil will swell if the negative pore pressures are dissipated with time and the strength will decrease.

From the preceding information it is obvious that pore pressures are of major importance. When a soil is loaded rapidly, excess hydrostatic pressures are developed too quickly to permit consolidation and intergranular stresses may not be sufficiently mobilized to prevent failure.

If drainage can take place fast enough the soil will develop a higher strength. This is the practical aspect of pore pressure measurements. From laboratory analysis similar to that carried out in this study, the drained and undrained strength of a soil may be determined, and design based on the applicable strength. Further, the possibility of excess hydrostatic pressures developing may be allowed for in the design by studying pore pressures.

The laboratory results may be combined with pore pressure measurements in the field[13] to assure that hydrostatic pore pressures never reach sufficient magnitude as to cause failure. For example, the rate of construction of a high earth fill may be adjusted to permit sufficient consolidation of the supporting soils or drainage provided[14] to assist more rapid consolidation.

A number of devices have been developed for measuring pore pressures in the field and the following description is a general one. Prior to construction, a hole is drilled down to the elevation in the natural soil where it is later desired to measure pore pressure. A pipe sealed at its lower end, with perforations on the last two inches of the pipe, is inserted. The space between the pipe and the walls of the hole is backfilled with granular material for a distance of 12 inches from the bottom. Higher up that space is filled with a clay slurry that must be of the highly impervious type, since otherwise the device will measure the highest pore pressure in any one of the soil layers intersected by the pipe instead of the pressure at its lower end.

The top end of the pipe, above the natural soil surface, is connected to a double line of tubing (to permit flushing gas bubbles or air from tubing) which is carried through a specially dug and backfilled trench.

A number of such devices are installed but usually are connected by tubing to one terminal well, where a system of pipes and valves permit their connection to two pressure gauges which are used for the measurement of pore pressures.

The pressures are observed, recorded and compared with laboratory results from the particular area to which they apply. If the hydrostatic pressures appeared to be excessive, remedial action as previously mentioned would be taken. If the hydrostatic pressures were within the limits indicated by laboratory results, then construction could proceed in the normal fashion.

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2. Soil Mechanics in Engineering Practice, by Terzaghi and Peck, page 52.

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4. Bishop and Henkel, page 10.

5. Bishop and Henkel, page 12.

6. Bishop and Henkel, page 26.

7. Bishop and Henkel, page 30.

8. Bishop and Henkel, page 36.

9. Fundamentals of Soil Mechanics, by Donald W. Taylor, page 341.

10. Taylor, page 337.

11. Shear Strength, a lecture given by Professor J. E. Roberts, Massachusetts Institute of Technology, September 4, 1958.

12. Taylor, Chapter 14.

13. Soil Mechanics, Foundations, and Earth Structures, by G. P. Tschebotarioff, Article 17 - 6.

14. Tschebotarioff, Article 19 - 4.

15. Theoretical Soil Mechanics, by Karl Terzaghi, page 17.