Some Effects of Desiccation of Clays in Relation to Their Compression

A Thesis in Civil Engineering

by

C. B. Kidson
THE CONSOLIDOMETERS WITH THE SUCTION PLATE APPARATUS IN THE BACKGROUND ON THE LEFT
This thesis was prepared at the National School of Engineering, Canterbury University College, New Zealand, under the direction of Mr. P. J. Alley, Senior Lecturer in Civil Engineering. It is presented as a partial requirement for the degree of Master of Engineering, in Civil Engineering.
LABORATORY TESTS OF COMPRESSION, SHRINKAGE UNDER AIR DRYING CONDITIONS AND MEASUREMENTS OF NEGATIVE PORE WATER PressURES WERE MADE ON SOME CLAYS IN THE NATURAL AND REMOULDED STATES, WITH A VIEW TO GAINING INFORMATION ON THE EFFECTS OF DESICCATION ON THE COMPRESSION CHARACTERISTICS OF FINE GRAINED SOILS AND THE RELATIONSHIP BETWEEN NEGATIVE PORE WATER AND EFFECTIVE PressURES.
# INDEX

<table>
<thead>
<tr>
<th>Topic</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>INTRODUCTION</strong></td>
<td>1</td>
</tr>
<tr>
<td><strong>AIMS</strong></td>
<td>2</td>
</tr>
<tr>
<td><strong>BACKGROUND</strong></td>
<td>2</td>
</tr>
<tr>
<td>The Clay-Water System</td>
<td>2</td>
</tr>
<tr>
<td>Air Set</td>
<td>4</td>
</tr>
<tr>
<td>Drainage by Desiccation</td>
<td>4</td>
</tr>
<tr>
<td>States of Unsaturation</td>
<td>6</td>
</tr>
<tr>
<td>Effective Stresses</td>
<td>9</td>
</tr>
<tr>
<td><strong>EXPERIMENTAL PROGRAMME</strong></td>
<td>11</td>
</tr>
<tr>
<td><strong>EQUIPMENT AND TESTING TECHNIQUES</strong></td>
<td>12</td>
</tr>
<tr>
<td>Consolidation</td>
<td>12</td>
</tr>
<tr>
<td>Correction for Machine Strains</td>
<td>17</td>
</tr>
<tr>
<td>Shrinkage Moisture Content Curves</td>
<td>18</td>
</tr>
<tr>
<td>Soil Moisture Tension Measurements</td>
<td>19</td>
</tr>
<tr>
<td>Suction Plate</td>
<td>19</td>
</tr>
<tr>
<td>Temperature Effects</td>
<td>20</td>
</tr>
<tr>
<td>Centrifuge Method</td>
<td>22</td>
</tr>
<tr>
<td>Pressure Membrane</td>
<td>23</td>
</tr>
<tr>
<td>Membrane Material</td>
<td>24</td>
</tr>
<tr>
<td><strong>TEST RESULTS</strong></td>
<td>26</td>
</tr>
<tr>
<td>Standard Tests</td>
<td>26</td>
</tr>
<tr>
<td><strong>DISCUSSION OF RESULTS</strong></td>
<td>29</td>
</tr>
<tr>
<td>Shrinkage Curves</td>
<td>29</td>
</tr>
<tr>
<td>pF versus Moisture Content Curves</td>
<td>33</td>
</tr>
<tr>
<td>General Effects of Desiccation</td>
<td>35</td>
</tr>
<tr>
<td>Void Ratio Versus Log of Neg. Pore Pressure Curves</td>
<td>37</td>
</tr>
<tr>
<td>Preconsolidation Loads</td>
<td>41</td>
</tr>
<tr>
<td><strong>CONCLUSIONS</strong></td>
<td>43</td>
</tr>
<tr>
<td><strong>BIBLIOGRAPHY</strong></td>
<td>45</td>
</tr>
</tbody>
</table>
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My thanks are also due to Mr. F. Downing of the Chemistry Department, who knows no differentiation between creed, colour or University Department when anyone needs help.
INTRODUCTION

The behaviour of fine grained soils under a direct compression force has always been a subject of great interest to soils investigators and practicing engineers. In particular, the preconsolidation load has attracted attention. Casagrande (1), in 1936, proposed an empirical construction to determine this load from the voids ratio versus log of applied load curves. He worked from results obtained from a series of tests on clays which had been naturally compressed under a known depth of overburden, and which had not been exposed to other influences such as desiccation.

The effects of remoulding from the natural state (as would be caused by any disturbance during sampling) have also been subjected to fairly thorough investigation so that the compression curves for samples in the laboratory can be related as nearly as possible to those which could be expected under natural conditions in the field e.g. under a foundation load.

However, very little investigation seems to have been carried out on the effects of desiccation of clays in relation to their preconsolidation load and to the general influence on, and alterations to the shape of the compression curve of decreasing moisture contents.

It is widely known that the internal forces set up in a clay structure during drying can cause effects equal to those due to the external pressure of many feet of overburden. In many cases, investigators have found evidence of preconsolidation loads near the surface, of deep deposits of clay, which exceed those at much
greater depths. Cascasonic (2) noted this on Massena clays (see fig. (1)).

These preconsolidation loads are obviously due to drying, as they decrease with depth in a non-linear manner, and then increase again linearly as the overburden load increases and there have been no moisture change effects.

Thus it seemed that an investigation of the drying effects on the compressibility curves, paying attention to the preconsolidation load, would be interesting and possibly fruitful, as regards extending the knowledge of the behaviour of clays.

**AIMS:**

It was hoped to establish a relationship between the capillary forces in a clay and the alterations to the shape of the voids ratio versus log of applied pressure curves.

**BACKGROUND:**

The Clay-Water System:

The water held at low temperatures in clay (driven off by heating to 100 - 150°C.), as distinct from the OH lattice water, is very important, as its nature and the factors which control its characteristics largely determine the plastic, boning, compaction and suspension properties of the material.
This water may be classed in three categories (Grim ref: (3))

(1) The water in pores, on the surfaces and around the edges of the individual particles of the clay minerals.

(2) Interlayer water, between unit-cell layers of certain minerals e.g. montmorillonite.

(3) Water which occurs in the tubular opening between elongate structural units e.g. sepiolite - attapulgite - palygorskite minerals.

Type (1) water requires little energy to drive off and can be lost at room temperatures.

Types (2) and (3) require more energy, i.e. temperatures at least approaching 100°C, to substantially remove it. In the case of Halloysite the reaction is not reversible, but most other minerals will reform the hydrated state easily if the water has not been lost at too high a temperature. Generally speaking, the pore water is liquid, with a layer of non-liquid adsorbed water on the surface of the pores and between the clay particles.

The exact arrangement of the water molecules adsorbed at the surface of discrete clay particles seems to be a subject of debate among experts, but it is generally agreed that they are bonded in some fixed pattern, that the adsorbed water is at great pressures and may have the structure of ice near the surface of the particles.

Aitchison (4) summarises the states of the water in a clay mass, in the light of the shear forces necessary to displace the water films or produce relative movement of the particles, as follows:

(a) A thin film, not exceeding 10Å in thickness, at the
immediate surface of the particle, possessing the viscosity characteristics of ice.

(b) A thicker film extending outwards from the adsorbed film (a), for a distance of up to 200 Å, possessing viscosity characteristics changing from those of ice at its inner boundary to those of free water at its outer extreme.

(c) All water more than 200 Å from a particle surface, having the characteristics of free water (pore water).

Air Set:

Grim and Cuthbert (5) have shown that when certain clay-water mixtures are allowed to stand under certain conditions, there is an increase in compression strength without loss of water. They call this "air-set". The explanation appears to be that a certain amount of time is required for the water to penetrate some of the surfaces of the clay-mineral particles and for the water molecules to assume their orientation. This increases the strength, as these authors state that a clay is dependant on the development of non-liquid water for its compressive strength.

Drainage by Desiccation:

The general theory of drainage by desiccation, as described by Terzaghi (6), can be explained as follows:

On exposure to air, the discharge velocity of the pore water in a clay is equal to the rate of evaporation. Now when the clay-water system is in equilibrium, the neutral stress due to the weight of the water is negligible. Hence the neutral stress must be
either very slightly positive or else negative. Yet on exposure
of the surface the water drains from the interior. Now there is
no increased pressure in the interior to cause this hydraulic
gradient. Hence it must be caused by surface tension at the
exterior.

In order to visualize this state of stress in a partly
desiccated mass of clay, an analogy has been drawn between the particle
structure of a fine grained soil and a bundle of highly compressible
capillary tubes of radius "r" (See fig. (2)).

![Figure 2]

Saturated

After evaporation- tubes
compressed and menisci at
radius "r" of capillaries

In reality,

of course, these
tubes would con-
sist of a very
tortuous channel

of voids. However, the effect would be the same, as the forces
are acting at the free surface when all pore spaces are full. The
maximum angle of contact $\theta$ of the pore water is assumed equal to
90° when the soil is completely saturated. As evaporation proceeds,
the water tends to be withdrawn into the pores, but this tendency
is resisted by the surface tension causing the surface layer of water
to adhere to the tubes, giving rise to a negative pore water pressure.
The menisci become cupped and their radius decreases. The minimum
value of this radius is equal to the radius "r" of the tubes.
The tensile stress in the water in the tubes, $p''$, can be found
by equating hydrostatic and surface tension forces:

$$2 \pi r T_s \cos \theta + \pi r^2 p'' = 0$$
\[ p^a = -\frac{2T_s \cos \alpha}{r} \]

where \( T_s \) = surface tension.

(Of course, in time the water will evaporate enough to bring about the state where the pores are not all filled, only the smaller ones containing water at a greater negative pressure, and eventually the free water remaining will be of a lenticular nature between particles).

Now in a soil, the actual air-water interface is not a simple one with a single value \( r \) for the radius of curvature in all directions. Hence we relate the negative pore water pressure, \( p^w \), to the equivalent rise in a capillary tube to a height \( h \) cms. above a free water surface

\[ i.e. \quad -p^w = \gamma g h \quad (\gamma_w = \text{density of water}), \]

as \( h \) can be measured experimentally.

The most convenient measure of the pore water pressure deficiency seems to be that proposed by Schofield - his \( p^F \) scale.

\[ p^F = \log_{10} h \quad \text{where } h \text{ is in cms.} \]

This has been used by the majority of workers in this field.

Aitchison (4) points out that if the \( p^F \) scale is to be used as a measure of the free energy, then it should only be applied to soils in which the osmotic effects of dissolved salts are negligible, as this affects the usual methods of measuring this quantity (e.g. suction plate or pressure membrane).

**States of Unsaturation**

To define the condition of soil with respect to its moisture
content, Aitcheson (4) recognises the following states of saturation or unsaturation.

A. Complete Saturation
B. Primary Unsaturation
C. Secondary Unsaturation
D. Partial Saturation
E. Modified primary Unsaturation
F. Modified Secondary Unsaturation.

The following parameters are needed to define these states:

\[ p'' = \text{pore water pressure} \]

(where \( p'' < 0 \) it is equal to the negative pore water pressure)

= soil moisture tension or pressure deficiency in the pore water

Aitcheson (4) regards \( p'' \) as being \( > 0 \) if it is negative. Here it is regarded as algebraically equal to the pore water pressure, the zero datum being atmospheric pressure.)

\[ p_g = \text{the pressure in the adsorbed gas or occluded air bubbles in the soil.} \]

\[ S_r = \text{degree of pore space saturation} \]

\[ \Delta w = \text{soil moisture deficit - i.e. the volume of water per unit area which must be supplied to the soil to achieve a condition of zero soil moisture tension.} \]

\[ n_r = \text{the relative compressibility of the pore fluid.} \]

\[ = \frac{C_r}{C_w} \]
8.

\[ C_p = \text{coefficient of compressibility of the pore fluid} \]
\[ \text{(it may contain air)} \]

\[ C_w = \text{coefficient of compressibility of air-free water} \]

Using the above parameters:

A. **Complete Saturation** - All pore spaces filled with water (no air bubbles)

\[ S_r = 100\% \quad \Delta_w = 0 \]
\[ n_r = 1 \quad p'' > 0 \]

B. **Primary Unsaturation** - This is the first stage in drainage by desiccation of an initially saturated soil. There are no air bubbles and all the voids are water filled, but there is a negative pore water pressure. This pF value is caused by the curvature of the air-water interface.

\[ p''_d < p'' < 0 \quad p''_d \text{ is the critical pressure deficiency at which the largest pore will drain} \]

\[ \Delta_w > 0 \quad S_r = 100\% \quad n_r = 1 \]

C. **Secondary Unsaturation** - In the second stage of desiccation some of the pores have drained

\[ p'' \leq p''_d \]

\[ \Delta_w > 0 \quad S_r < 100\% \quad n_r > 1 \]

\[ p_g = 0 \text{ as the air in the drained pores is not occluded or adsorbed and is therefore at atmospheric pressure.} \]

The three states D, E, and F are parallel to those A, B, and C for a soil which has some adsorbed or occluded air in the pore fluid to begin with. It therefore follows that we can define the states
thus:

D. Partial Saturation:

\[ \Delta_w = 0 \quad S_r < 100 \quad p'' \geq 0 \]
\[ n_r > 1 \]

i.e. there is no negative pore pressure, but the soil cannot be
regarded as saturated, as it contains air. \( S_r \) normally exceeds 90%.

E. Modified Primary Unsaturation: Case parallel to B

\[ p''_d < p'' < 0 \quad \Delta_w > 0 \quad S_r < 100\% \]
\[ n_r > 1 \quad p_g < 0 \]

F. Modified Secondary Unsaturation: Case parallel to C

\[ p'' < p''_d < 0 \quad \Delta_w > 0 \quad 100 > S_r > 0 \]
\[ n_r > 1 \quad p_g \text{ may be a complex function with varying}
\]
pressures in different phases of the soil

Effective Stresses:

One of the basic equations of soil mechanics is the relationship
between pore water pressure, applied, and effective stresses.

i.e. \( p' = p - p'' \) .................. (2)

where \( p' \) = effective pressure on the soil structure

\[ p = \text{applied pressure} \]

\[ p'' = \text{pore water pressure} \]

This law as stated by Terzaghi (6) is the basis soil mechanics
involving positive values of pore water pressures.

It seems reasonable to assume that this concept can be extended
to embrace negative values of the pore water pressure, \( p'' \), in a fine
grained soil when the pores are water filled.

Equation (2) applies strictly, of course, only to a saturated soil in which there is perfect point contact between the particles, there being no reduction in the effect of the hydrostatic forces on the particles due to an appreciable area of contact.

The idea behind the experimental work then, was to investigate the extension of equation (2) in the case of clays, by measuring the magnitude of these soil moisture forces, for various moisture contents and through the moisture content—shrinkage and applied load-compression curves, relate these forces to those applied directly in a consolidometer.
EXPERIMENTAL PROGRAMME

It was desired to reproduce in the laboratory the conditions which would exist as an alluvial deposit of clay was desiccated under air drying conditions. Hence a certain amount of each sample was dried, the lumps broken up and then the soil was slurried well past the liquid limit to a consistency which, while being definitely liquid, would not allow appreciable segregation of the particles to take place before the mixture re-gelled. This slurry was then allowed to air dry and was used as a remoulded sample.

The following tests were carried out:

**Standard:** Natural moisture content

- Liquid Limit
- Plastic Limit
- Specific Gravity
- 24 Hour Hydrometer

**Consolidation:**

- On natural sample
- On remoulded as near to virgin curve as possible and at decreasing moisture contents.

**Moisture Content:**

- On natural and remoulded samples.

**Shrinkage:**

Shrinkage - moisture content curves for natural and remoulded samples.
Consolidation:

The machines used for this were penetrometers of the type designed by Bishop and manufactured by Mykesh Farrance Engineering Ltd. Eventually there were three machines; the second arriving in April and the third towards the end of the year in October.

Each test was carried out in the standard manner using loads of 0.5 to 16.0 tons/sq.ft. applied at twenty four hourly intervals. The sample, on the unloading cycle, was not allowed to swell under zero load, but the final reading of the dial gauge was made with the 0.5 tons/sq.ft. load in place. The water in the sample cell was drained rapidly out, the sample removed and a moisture content taken. It was considered that in this way a more positive value of the final height of the specimen could be obtained. No change in the reading of the specimen thickness was noted as the surrounding water was removed, indicating that the moisture content remained virtually constant.

A rigid concrete slab was constructed with its own foundation completely independent of the floor of the laboratory, giving a vibration free base for the consolidometers.

The sample ring was always slightly greased to minimize the wall friction on the specimen, although van Zelst (7) showed that this was only the cause of small errors compared factors such as remoulding of the sample when trimming the top and bottom of the disc.
On these machines there is slightly too much clearance around the upper porous plate, and the centring of this plate on the sample is not positive enough.

The top porous plate is pivoted against the rigid frame of the machine and the load is applied upwards through a hanger frame on which the sample cell containing the sample and bottom porous plate rests (see fig. (3)). The hanger is only loosely confined at its lower end. If at this point it was positioned by a very short bearing, then the centring would
be automatic.

Some tests were ruined because the upper plate tilted when not quite central, and hence stuck e.g. fig. (4). This was highly aggravating when it occurred, as the remoulded samples were losing moisture all the time and there was no chance of repeating the test at that moisture content.

With reduced clearance on the upper plate, the sample would not have the same tendency to squeeze out through the annular space left. This introduces an error in all tests and although in tests on natural samples this is small, it is impossible to run a test on a remoulded sample such above the liquid limit.

Assume x cu. ins. of the sample is squeezed out. The sample ring diameter is 3". Hence loss in sample height.

\[ Sh = \frac{4x}{9\pi} \]

\[ = 0.144x \]

In most tests

\[ Se = \frac{1 + e}{Sh} \]

\[ \approx 2.5 \]

This can alter the voids ratio reading by

\[ Se \approx 2.5 \times 0.144x \]

\[ 0.35x \]

Now average radius of sample ring (See fig. 5.)

\[ r_1 = 1.498" \]

Average radius of top porous plate

\[ r_2 = 1.483" \]
Compressive curve showing the effects of the upper porous plate sticking due to inaccurate centring.
Now if the sample is squeezed into this annular space to a height

\[ h = 0.2^n \]

\[ x = h \pi (r_1^2 - r_2^2) \text{ cu. ins.} \]

\[ = 0.02 \pi (1.496^2 - 1.43^2) \text{ cu. ins.} \]

\[ = 0.028 \text{ cu. ins.} \]

An apparent change in voids ratio

\[ S_e = 0.01 \]

Which is \( \frac{1}{100} \) of the initial voids ratio. In extreme cases it would be much more.

The method of calculating the voids ratio during the test was that given in Capper and Cassie (8).

That is, the sample was carefully levelled off and the initial height assumed to be exactly that of the sample ring. The dial gauge reading was then taken. The loading and unloading cycle was carried out, and after the final reading was taken the sample was removed and a moisture content taken. Now at this stage the sample was fully saturated.

\[ e_2 = \frac{m}{g} \quad e_2 = \text{final voids ratio} \]

\[ m = \text{moisture content} \]

\[ g = \text{specific gravity} \]

\[ h_1 = \text{initial height} \]

(assumed: that of the sample ring)

\[ h_2 = \text{final height} \]

\[ h_1h_2 = \text{change in dial gauge reading}. \]

Now the relationship between change in voids ratio and change in sample height under different loads, as recorded by the
dial gauge is

\[ \frac{Se}{Sh} = \frac{1 + e_2}{h_2} \]

and from this the values of

\[ \frac{Se}{Sh} \]

\[ = 0.775 - \Delta h \]

\[ = 0.775 - 0.1 \]

\[ = 0.675 \]

\[ = 2.37 \]

assuming height is that of the sample ring.

\[ \frac{Se}{Sh} = \frac{1 + e}{h} \]

\[ = 0.755 - \Delta h \]

\[ = 0.755 - 0.1 \]

\[ = 0.655 \]

\[ = 2.44 \]

for correct initial height.

This gives an error of nearly 3.0%. Now this error is operative throughout the whole of the graph, although it is due only to one incorrect reading. It is therefore better to adopt the system whereby a measured steel block is placed on the bottom porous plate, and the dial gauge is set to read directly the height of the sample, rather than to adopt the height of the sample ring as a datum. This means that any errors in levelling
the sample will be confined to the compression reading for the initial load, and will not affect the most important part of the graph i.e. the slope of the straight line portion of the e/log p curve.

**Correction for Machine Strains:**

It was found that there were inherent strains in the consolidometers which added to the dial gauge readings giving a false figure for the compression of the sample. To correct for this, four loading and unloading cycles were carried out on each machine with no soil in them - the load being applied directly to the porous plates. The errors are set out in table (1). All readings are to be subtracted from those obtained during the test.

<table>
<thead>
<tr>
<th>Load T/sq. ft</th>
<th>Dial Gauge No. 1</th>
<th>Error in Inches No. 2</th>
<th>Error in Inches No. 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.0021</td>
<td>0.0025</td>
<td>0.0021</td>
</tr>
<tr>
<td>0.5</td>
<td>0.0007</td>
<td>0.0009</td>
<td>0.0007</td>
</tr>
<tr>
<td>1.0</td>
<td>0.0008</td>
<td>0.0007</td>
<td>0.0008</td>
</tr>
<tr>
<td>2.0</td>
<td>0.0011</td>
<td>0.0012</td>
<td>0.0011</td>
</tr>
<tr>
<td>4.0</td>
<td>0.0015</td>
<td>0.0016</td>
<td>0.0015</td>
</tr>
<tr>
<td>8.0</td>
<td>0.0024</td>
<td>0.0025</td>
<td>0.0024</td>
</tr>
<tr>
<td>16.0</td>
<td>0.0019</td>
<td>0.0020</td>
<td>0.0019</td>
</tr>
<tr>
<td>8.0</td>
<td>0.0012</td>
<td>0.0013</td>
<td>0.0012</td>
</tr>
<tr>
<td>4.0</td>
<td>0.0008</td>
<td>0.0017</td>
<td>0.0008</td>
</tr>
<tr>
<td>2.0</td>
<td>0.0005</td>
<td>0.0008</td>
<td>0.0005</td>
</tr>
<tr>
<td>1.0</td>
<td>0.0004</td>
<td>0.0006</td>
<td>0.0004</td>
</tr>
<tr>
<td>0.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TABLE I**
Shrinkage Moisture Content Curves:

The volume change as the soil shrank under air drying conditions was measured. This was accomplished by a mercury displacement method. From this, a plot of voids ratio versus moisture content was made, providing a link between the compression and soil moisture tension curves.

A glass dish with the rim ground flat was filled with mercury (see fig. 6), and a perspex lid, with a \( \frac{1}{16} \)" hole bored in it, placed on top. A standard weight (4 lbs) was applied in the centre of the perspex cover and the excess mercury escaped through the hole in the cover or around the sides. Placing a fixed weight on top was found more definite than just pressing on the lid with the hand, and assured a constant volume of mercury.

A soil pat of about 20c.c. volume was trimmed making sure there were no sharp projections or surface pockets, so that the mercury would follow the contours of the sample. It was then submerged in the mercury by the three prongs on the lid of the apparatus, and the weight of liquid displaced taken to \( \pm 0.001 \) lbs, representing an error of \( \pm 0.5\% \).

A series of volume measurements was taken for each soil pat, weighing it after each immersion and letting it air dry between. It generally took about two days for the sample to dry to the point where it contained about 5% of water, after which it was oven dried and the weight of solids found. Knowing this and the specific gravity of the soil the moisture content and corresponding voids ratios were then calculated.

Soil Moisture Tension Measurements:
Soil Moisture Tension Measurements:

Three methods of finding the pF's of the samples were tried over the following ranges.

<table>
<thead>
<tr>
<th>pF</th>
<th>Cms Hg</th>
<th>lbs/sq.in.</th>
<th>T/sq.ft.</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.80</td>
<td>5.0</td>
<td>0.06</td>
<td></td>
<td>Suction Plate</td>
</tr>
<tr>
<td>2.13</td>
<td>10.0</td>
<td>0.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.43</td>
<td>20.0</td>
<td>0.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.74</td>
<td>40.0</td>
<td>0.50</td>
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<td></td>
</tr>
<tr>
<td>3.00</td>
<td>50.0</td>
<td>0.62</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.15</td>
<td></td>
<td>20.0</td>
<td>1.29</td>
<td>Centrifuge</td>
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<tr>
<td>3.45</td>
<td></td>
<td>40.0</td>
<td>2.57</td>
<td>(Unsuccessful)</td>
</tr>
<tr>
<td>3.62</td>
<td></td>
<td>60.0</td>
<td>3.86</td>
<td>Pressure Membrane</td>
</tr>
<tr>
<td>3.85</td>
<td></td>
<td>100.0</td>
<td>6.43</td>
<td></td>
</tr>
</tbody>
</table>

(1) Suction Plate:

In principle this consists of a saturated porous plate, on top of which the sample is placed while the underside is in contact with a column of water at a pressure less than atmospheric.

The diagram (fig. 7) shows the apparatus used for the Laboratory. The set up is essentially the same as that used by Cronie, Coleman and Bridge (9).

The porous plates used could not be obtained locally and had to be sent from England. The pore size must be very uniform and of maximum diameter not greater than 1.5 microns to support a pressure difference of 50 cms. of Hg between faces without the moisture films breaking down and allowing the passage of air. These plates were available only in small sizes (3.2 cms. in diameter) as they are very difficult to manufacture. Those used were in cut down Buchner funnels.

The sample of soil was trimmed to 0.5 cms. in thickness and
put in position, making sure that the plate was fully saturated, and that the sample was in intimate contact with it. The suction was then applied and an airtight plastic cover (similar to those used on jam jars) was secured over the Buchner funnel by means of a rubber band (see fig. 7). This prevented evaporation from the upper surface of the sample affecting the final moisture content, as when equilibrium is reached, the vapour pressure inside this enclosed space must attain a value consistent with the vapour pressure over the curved surface of the capillary water at that particular radius "r". This is less than that over a plane surface of a liquid by an amount:

\[ S_p = \frac{\frac{T}{\rho}}{\rho - \sigma} \]

where \( T \) = surface tension
\( \rho \) = density of the liquid
\( \sigma \) = density of the saturated vapor pressure.

This fact is used in the measurement of high values of \( p_F \) between 4.5 and 7. (See Crossey et al.\(^{(9)}\)).

The plastic cover over the sample has also the advantage of being flexible enough to allow the pressure of the air around the sample to remain the same as that at the open end of the mercury manometer tube. Hence the sample is kept airtight, but the pressure difference across the suction plate is still unaffected by changes in barometric pressure.

**Temperature Effects:**

As there was no constant temperature room available, the
SUCTION PLATE APPARATUS

Mercury Manometers

Cut Down Buchner Funnel

Thick Pyrex Flask

Suction Pump

Sample
Porous Glass Disc
Plastic Cover

Suction Plate
The apparatus was subject to the daily variations of the laboratory temperature. The change in pressure of the air between the mercury manometer and the water in the flask could cause considerable variations in the applied suction. Consider the case when there was a variation between minimum night and maximum day temperature of 20°C. If the maximum temperature was 35°C, and the corresponding minimum 15°C, then as the volume of air in the apparatus remains practically constant, we can say that:

\[
\frac{P}{T} = \frac{P + SP}{T + ST}
\]

(The only change in volume would be that due to the different position of the mercury in the manometer, and as the tubing was only 2 mm in diameter, this volume change would be small compared with the total volume which consists mainly of the air in the flask).

\[
T + ST = 273 + 35 = 308
\]

\[
T + ST = 273 + 15 = 288
\]

\[
\frac{P + SP}{P} = \frac{T + ST}{T}
\]

\[
\frac{SP}{P} = \frac{ST}{T} = 20 \div 7%
\]

To avoid this effect as much as was possible, the sample was placed in the apparatus in the late afternoon and the suction applied. In the morning the manometer reading would have changed slightly but was thereafter kept at that value by adjusting the suction throughout the day.

Each sample was left in the apparatus for 24 hours. Cronen et al (9) found that for a sample of this size equilibrium was reached
in this period. The soil was then removed, weighed to 0.001 of a g, oven dried, reweighed and the moisture content for that particular negative pore water pressure calculated.

The sample had to be weighed rapidly as otherwise there was an appreciable loss of moisture during the process.

(2) Centrifuge Method:

The apparatus for the centrifuge method was as shown in fig. (8).

The principle is virtually the same as that of the suction plate. Instead of having a column of water under a suction of h cm. of H₂O in contact with the pore water of the sample, we have a suction exerted on the soil moisture.

\[ h = \frac{x}{g} \]

where \( h \) is height from the centre of the sample to the water table

and \( xg \) is the centrifugal field.

A pair of centrifuge cups were made, having screw tops and a small hole (0.06" D) at radius \( r_2 \) to allow excess water to escape while keeping the water table at a constant level.

The porous cylinders on which the samples were placed
These were turned on the soil lathes, balanced and shellacked on the sides to prevent flaking under constant wetting.

The centrifuge speeds were measured by means of a stroboscope consisting of disc patterned to indicate the speeds required when used with a fifty flashes per second mercury vapour lamp.

Now if \( r_1 \) and \( r_2 \) were as shown in fig. (8) then the centrifugal field

\[
x = \frac{r_2 + r_1}{\omega^2 \frac{g}{2}}
\]

where \( \omega \) = angular velocity in radians/second.

Now suction in cms. of \( H_2O \) exerted on the sample

\[
= (r_2 - r_1) \left( \frac{r_2 + r_1}{2} \right) \frac{\omega^2}{g}
\]

and

\[
pF = \log_{10} \left( \frac{r_2^2 - r_1^2}{2} \frac{\omega^2}{g} \right)
\]

Using the centrifuge available in the laboratory and running up to speeds of 3000 r.p.m. it was hoped to obtain \( pF \)'s up to 3.8. Unfortunately the machine was too small for the job and could not maintain the speeds required for 24 hours without heating up to such a degree that the water in the cups evaporated through the small hole. In place of this, the pressure membrane apparatus was made and used.

(3). Pressure Membrane:

A diagram of the apparatus is shown in fig (9). The apparatus was an adoption of that used by Richards (11).

The soil sample (weighing 10 - 15 gms. and less...
SECTIONAL ELEVATION - EXPLODED VIEW

FIGURE 9
than I cm. thick) is placed on the saturated membrane, the cell assembled and the pressure of the air surrounding the sample to that required (as indicated by the Bourdon gauge) by use of a compressed air cylinder. It is left in for 24 hours to attain equilibrium.

The principle of the pressure membrane apparatus is exactly the reverse of that of the suction plate. The air pressure forces from the voids of the soil any water which is not normally at a greater negative pressure than the positive air pressure applied. The membrane must be so finely porous that the moisture films will not allow the free passage of air, but will let the water which is forced from the sample pass through. The membrane, on the side away from the sample, is in constant contact with water from the reservoir so that it remains saturated at all times. The porous disc backing the membrane gives it support, while allowing free passage either way for the water. It was made of unglazed pottery, filed down to the correct shape.

Sintered glass discs were made and tried, but were found to crush and crack with pressures of over 60 lbs./sq.in. The unglazed pottery discs eventually cracked too, after about three weeks use, and it is recommended that a sintered brass disc would be superior as it would be stronger, more porous and would probably be more smoothly faced, hence being less liable to perforate the membrane.

Membrane Material:

Richards (14) used a membrane of a cellophane commercially available in the U. S. A. Several types of wrapping cellophane
and some plastics including polyvinyl chloride were tried, but were found to be of no use. The plastics generally were not porous enough whereas the cellophane was too porous and was also easily damaged. Finally, some Visking sausage casing was obtained from the Soil Bureau Experimental Station by courtesy of Mr. R. Packard. As a membrane permeable to water but not to air under high pressures, this has been used successfully at pressures of up to 250 lbs/sq.in. This membrane only requires soaking in water for about five minutes before the test to become completely saturated. It is fairly strong and not liable to be damaged during handling. It was found eminently satisfactory for the pressure membrane apparatus.
Four clays had the complete series of tests carried out on them. Only this number could be completed as each took at least six weeks and some of the equipment did not arrive till late in the year.

The soils tested had the following properties:

**Sample No. 6:**

Danedin - a yellow-orange weathered marl deposit.

<table>
<thead>
<tr>
<th>Depth of Sample</th>
<th>Surface</th>
<th>Hydrometer Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nat. M/C</td>
<td>24.0%</td>
<td></td>
</tr>
<tr>
<td>L.L.</td>
<td>38.9%</td>
<td></td>
</tr>
<tr>
<td>P.I.</td>
<td>22.2%</td>
<td></td>
</tr>
<tr>
<td>P.I.</td>
<td>16.7</td>
<td></td>
</tr>
<tr>
<td>S.G.</td>
<td>2.62</td>
<td></td>
</tr>
</tbody>
</table>

| (% Coarse Sand | 2.0mm-0.25mm = | ) | 19.1% |
| (% Fine Sand   | 0.25mm-0.05mm = | ) | 49.9% |
| Silt           | 0.05mm-0.005mm = | | 31.0% |
| Clay           | 0.005mm = | | 12.0% |
| Colloids       | 0.001mm = | | 0.056 |

U.S. Bureau of Soils Classification = Silty Clay

Natural C = 0.056

Average Remoulded C = 0.077

**Sample No. 7:**

Waipara - clay from limestone country.

| Depth of Sample | = 1' 6" - 3' 0"
| Nature of Overburden | Heavy black soil
| Nat. M/C | 24.0%
| L.L. | 51.8%
| P.I. | 22.1%
| P.I. | 29.7
| S.G. | 2.59

Hydrometer Analysis:

| (% Coarse Sand | 2.0mm-0.25mm = | 9.0% |
| (% Fine Sand   | 0.25mm-0.05mm = | 14.5% |
| Silt           | 0.05mm-0.005mm = | 27.7% |
| Clay           | 0.005mm = | 51.8% |
| Colloids       | 0.001mm = | 38.0% |
27.

U.S. Bureau of Soils Classification = Clay
Natural \( C_0 \) = 0.063
Remoulded Average \( C_0 \) = 0.106

**Sample No. 8:**

Christchurch City - Taitapu series silty clay (T.4)

<table>
<thead>
<tr>
<th>Depth of Sample</th>
<th>0'-1' Top Soil 1'-6' Heavy Silty Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nature of Overburden</td>
<td>= 28.3%</td>
</tr>
<tr>
<td>Nat. M/C</td>
<td>= 35.6%</td>
</tr>
<tr>
<td>L.L.</td>
<td>= 27.4%</td>
</tr>
<tr>
<td>P.I.</td>
<td>= 8.2%</td>
</tr>
<tr>
<td>S.G.</td>
<td>= 2.56</td>
</tr>
</tbody>
</table>

**Hydrometer Analysis:**

| % Coarse Sand | 2.00mm-0.25mm | = 4.0% |
| % Fine Sand | = 1.0% |
| % Silt | = 51.0% |
| % Clay | = 44.0% |
| % Colloids | = 22.8% |

U.S. Bureau of Soils Classification = Silty Clay
Natural \( C_0 \) = 0.037
Remoulded Average \( C_0 \) = 0.037

**Sample No. 9:**

Heathcote - Taitapu series silty Clay

<table>
<thead>
<tr>
<th>Depth of Sample</th>
<th>Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nat. M/C</td>
<td>= 27.0%</td>
</tr>
<tr>
<td>L.L.</td>
<td>= 33.7%</td>
</tr>
<tr>
<td>P.I.</td>
<td>= 26.1%</td>
</tr>
<tr>
<td>S.G.</td>
<td>= 2.62</td>
</tr>
</tbody>
</table>

**Hydrometer Analysis:**

| % Coarse Sand | 2.00mm-0.25mm | = 0.0% |
| % Fine Sand | 0.25mm-0.05mm | = 3.1% |
| % Silt | 0.05mm-0.005mm | = 63.2% |
| % Clay | 0.005mm-0.001mm | = 33.5% |
| % Colloids | 0.001mm | = 17.2% |

U.S. Bureau of Soils Classification = Silty Clay
Natural \( C_0 \) = 0.030
Remoulded Average \( C_0 \) = 0.060

These tables give the standard properties of the soils tested. The shrinkage, pF versus moisture content, and compression
tests are all represented graphically in figures 10-18.
DISCUSSION OF RESULTS

A. Shrinkage Curves:

The voids ratio versus moisture content relationship for any mixture of solid particles, water and air is

\[ e = \frac{V_v}{V_s} = \frac{V_a + V_w}{V_s} \]

where

- \( V_v \) = Total Volume of voids
- \( V_a \) = Volume of air
- \( V_w \) = Volume of water
- \( V_s \) = Volume of solids
- \( M \) = moisture content
- \( G \) = Specific gravity of solids.

Hence for a completely saturated mixture, the plot of voids ratio against moisture content is a straight line, of gradient \( \tan \theta = g \), passing through the origin.

Thus the shrinkage curves for soils in the range of "complete saturation" and "primary unsaturation" follow this straight line, diverging only when the state of "secondary unsaturation" is reached.

A soil which is initially in a state of "partial saturation" will have its shrinkage curve lying to the left of this straight line the vertical distance above the line \( e = mg \) representing the proportion of voids occupied by adsorbed and occluded air.

For any shrinkage curve there seem to be three important characteristics.
(1) Voids ratio at which the curve departs from the $S_p = 100\%$ line, which we will define as the "breakaway" point.

(2) The shape of the curve after it leaves the "breakaway" point, i.e., whether it is curved along its length or nearly straight.

(3) The slope of the curve after "breakaway".

These characteristics must depend on:

(a) The compressibility

(b) The size of the voids i.e., maximum pore size.

(c) The uniformity of pore size.

Position of "Breakaway":

For a compressible soil, the shrinkage curve will follow the straight line throughout the stage of "primary unsaturation". During this period, the whole of the soil mass will be shrinking under the effects of the negative pore water pressure. The capillary openings will be becoming smaller and the radius of curvature of the air-water interfaces will be able to attain lower values with consequently greater negative pore water pressures without the value of $S_p$ dropping below 100%. This will obviously give a low value for the "breakaway" voids ratio.

The "breakaway" point will be high if there are some very large pores in the soil. In this case, the curve will diverge slightly from the line as soon as these pores lose their water by evaporation, (the state of "secondary unsaturation" having been reached) and if the soil is compressible, the slope will be steep until the compression due to the negative pressure in the smaller diameter voids has been undergone.

The Shape after "breakaway":

The shape after "breakaway":
A sudden breakaway into another straight line occurs when the pore size is very uniform, as all the pores drain at the same time and the compression in the state of "secondary unsaturation" is negligible. As has already been stated, a less uniform pore size distribution produces a curve, the length of which is dependent on the compressibility of the soil.

The Slope of the Curve:

The slope of the curve after leaving the line of saturation is dependent almost entirely on the compressibility of the soil. A very incompressible soil would follow the line \( e = 0 \) for the state of complete saturation; the period of primary unsaturation would be very short as the pore diameter would not decrease, then the curve would be very flat between where it left the saturation line and reached zero moisture content.

The test results for the four soils can be seen in figs. 10-13.

Sample No. 6 (fig. 10)

The shrinkage curves for this sample are peculiar in that the remoulded curve lies above the natural one i.e. the breakaway point occurs at a higher voids ratio although the compression index \( C_e = 0.077 \) for the remoulded sample as compared with 0.056 for the natural sample.

This was probably due to the remoulded sample having a few large voids which drained before the rest. The remoulded curve shows more compression after breakaway than the natural one but still does not attain such a low value of voids ratio at zero moisture content as the larger pores which drained first would remain uncompressed.
SAMPLE NO. 6

SHRINKAGE CURVES

VOIDS RATIO

MOISTURE CONTENT

LINE E = M.G FOR COMPLETE SATURATION

E = \frac{V_v}{V_s} = \frac{V_a + V_w}{V_s} = \frac{V_a}{V_s} + M.G

FIG. 10
SAMPLE NO. 7

SHRINKAGE CURVES

MOISTURE CONTENT

VOIDS RATIO

FIGURE 11
SAMPLE NO. 8

SHRINKAGE CURVES

MOISTURE CONTENT

VOIDS RATIO

FIGURE 12
SAMPLE NO. 9

SHRINKAGE CURVE FOR REMOULDED STATE ONLY—NATURAL SAMPLE WAS TOO BRITTLE

MOISTURE CONTENT

FIG. 13
Sample No. 7 (fig. 11)

Here the natural sample is obviously in the state that Aitchison (4) described as "partial saturation". It is reasonable to expect that most soils will have some adsorbed or occluded air in the pore fluid in the natural state. In fact some authors such as Dawson (12) claim that they have never found an undisturbed sample with all the voids completely filled with water. This may be so, but in a lot of cases the percentage of air is probably so small that it would not be obvious using this method of measuring shrinkage whether the soil is in a state of saturation or of partial saturation.

This clay is very compressible and must have small pores (not only has it a high percentage of clay size particles, but it has 38\% of colloids), hence it reaches the low values of $e = 0.25$ for the natural sample and $e = 0.20$ for the remoulded state when the moisture content is zero, and the remoulded curve does not break away from the line $S_f = 100\%$ until $e = 2.6$.

The straight line portion of the natural shrinkage curve represents the condition of "modified primary unsaturation", the value of $S_f$ being near-or above the 90\% mark as Aitchison (4) suggests.

Sample No. 8 (fig. 12)

The natural sample seems to be completely saturated. Pore size must be very uniform as both the natural and remoulded samples go from breakaway to zero moisture content in straight flat lines. This soil is the least compressible of the four in the consolidometer, but is more prone to compression by desiccation than sample 6 for example, by virtue of the fact that its uniformity of pore size ensures that the state of primary unsaturation exists until the moisture
evaporates from nearly all the pores at once. This means that the negative pore pressure has an over all compressive effect for a far greater range.

Sample No. 9 (Fig. 13):

This soil is very similar to sample 8, belonging to the same silty clay group, but has a greater silt fraction (63.2%) and hence a greater maximum pore size. This is the reason why it has a higher breakaway voids ration. It is not nearly as compressible as samples 6 and 7 as can be seen from the flat curve after breakaway.

Unfortunately the natural sample was too brittle to carry out the shrinkage tests on as the prongs by which the sample is submerged in the mercury caused the soil pat to split. This sample was taken from the surface and was obviously in the "partially saturated" state.

pF Versus Moisture content curves (See fig. 14):

As far as the validity of the equation

$$p' = p - p'' \quad (p' = \text{effective pressure on a plane in the soil structure})$$

$$p = \text{total normal load on a plane}$$

$$p'' = \text{pore water pressure}$$

goes, the only portion of the pF/moisture content curves over which this can be expected to hold is that which represents either the state of "primary unsaturation" or with some slight variations the state of "modified primary unsaturation", as the assumption that it is true for any positive value of $p''$ let alone negative ones depends on the pore pressure operating over the whole of the plane.

If some of the pores were drained it is obvious that the
above relationship would not hold if it was impossible to pass a surface through the soil at right angles to the direction of p without it cutting some of these drained pores. In this case the increase in effective pressure would be proportional to the negative pressure and to the area of undrained pores cut by this surface.

The general shape of the pF/moisture content curves can be divided into three parts:

A. This part rises sharply giving a comparatively large increase in pF with little loss in moisture. This then curves over into part B, which has a more gradual increase in pF value for an equivalent decrease in moisture content.

B. The curve once again rises sharply.

The section A represents the transition from saturation or partial saturation to "primary" or "modified primary" unsaturation when evaporation is causing the air water interfaces to become cupped, and although little water is lost there appears a sharp rise in the pF value. It should be remembered though, that the negative pore pressure is plotted to a logarithmic scale and that this increase in pF at these low values is not as great as it appears at first sight.

It is obvious from the shrinkage curves that section A lies within the primary unsaturation or modified range.

Although in the case of samples 6R 9R and probably 8R the moisture content at breakaway from the shrinkage curve (6R, 22.0%; 8R 15.0%; 9R 22.5%) coincides with the beginning of section A.
the $p'$ curve, it does not do so in the case of samples 7R and 7N. It would be reasonable to expect that this rapid increase in $p'$ with little loss of water would be caused by the change to the state of secondary unsaturation when the larger pores drain and the water is held only in the smaller pores and in lenticular form.

The length of section $E$ seems to depend on the compressibility of the sample under negative pore pressure as $7R > 6R > 5R > 9R$.

The General Effects of Desiccation:

The consolidation tests are shown graphically in figs. 15-18. They show the natural and remoulded log of pressure curves and the $p'$ versus voids ratio curves. While quite a number of tests have been carried out by workers in the soil mechanics field on remoulded samples to the same or lower moisture contents than the natural, these have been mostly in connection with sample disturbance and other sources of testing error and the clays have not been allowed to reequilibrate after remoulding.

The general equation of the straight line portion of the voids ratio versus log of pressure curve is

$$e = e_0 - C_e \log \frac{\sigma' + \pi}{\sigma'_0}$$

where $\sigma'_0$ = preconsolidation pressure,

$e_0$ = voids ratio at preconsolidation pressure,

$C_e$ = compression index

Therefore as Skempton (13) points out $e_0$ and $C_e$ are the
defining constants and for plots of $C_\varepsilon$ versus $e$ he obtains straight lines for a single clay at varying initial moisture contents. The only sample for which this holds is No. 9. In general, the change in slope is so small that $C_\varepsilon$, over the range measured, does not change by more than 10%. Hence the errors that it has been seen can creep into the consolidometer work, especially with samples at high moisture contents, could mask any existing pattern.

Rutledge (14) did work on remoulded samples in connection with undisturbed testing. This work is different as he disturbs the structure of the clay where I am allowing it to build up. Naturally enough, on his remoulded samples he found that they all showed lower values of voids ratio for the same load than they did in the natural state. With the clay given a chance to regel this was not the case.

It generally appeared the clays, as they lost moisture, were tending to revert to the characteristics of the natural sample. This can be seen in Figs. 10, 11, and 13 for samples 6, 7 and 9. Here the soil at initially higher moisture content than the natural sample has a greater value of voids ratio for the same load. The curves are displaced downwards towards the natural, as the initial moisture content decreases.

Sample 8 is one case where the clay has had a structure which it does not regain completely on regelling. The natural moisture content was 28.8% and the remoulded sample with 29.9% initial moisture content is displaced downwards considerably. This clay may be similar to halloysite which, after dehydration, does not reform the same configuration of interlayer adsorbed water between the unit cell layers of the clay fraction.
---- Indicates a back-loading curve
SAMPLE No.1
COMPRESSION CURVES

LOG OF PRESSURE (TONS/SQ. FT.)

FIGURE 18
SAMPLE No3
COMPRESSION CURVES

LOG OP PRESSURE (TONS/ SQ. FT.)

FIGURE 17
In general it is noticeable that although the remoulded clays show a slight increase in resistance to consolidation pressures over the straight line portion of the curve, it is not very marked for quite wide variations in the initial moisture content i.e. Although, as the clay dries, the curve is at a lower voids ratio throughout its length and although "air-set" causes a slight flattening of the curve, $C_0$ does not decrease greatly for correspondingly large changes in initial moisture content.

The 27.7% initial moisture content remoulded curve for sample 7 is an interesting one. To more clearly define the first part of the curve, this sample was loaded with 0.2 T/sq. ft. to give an extra point on the curve. Under this load swelling took place. This is regarded as being due to the adsorption of more interlayer water (Drin (3)). The final part of the curve then lay between the 40.7% and the 43.7% ones. The absorbed water appears to be one of the major controlling factors in the strength of a clay structure.

**The Void Ratio versus Log of Negative Pore Pressure Curves:***

If equation (2) for relating effective, applied and pore pressures for a soil structure is true for negative values of $p''$, then the voids ratio versus log of negative pore pressure curve plotted to the same scale as the voids ratio versus log of applied pressure graph should take the same form. It can be seen that the soil moisture tension and applied pressures can only be equated if the sample in the consolidometer has exactly the same initial condition (i.e. has the same voids ratio) at the beginning of the compression curve as the sample shrinking under air drying conditions. This means bringing the remoulded consolidometer sample to the
moisture content at the end of portion (A) of the pF/moisture content curve before testing. Then the consolidometer compression curve is started on the soil at the point at which it enters the range of "primary unsaturation", which is when the surface tension forces begin to compress the soil i.e. $r = 100\% \quad p'' < 0$.

This moisture content can be assessed, but great difficulty was found in attaining it exactly. If, as was done, a piece of soil was cut from the edge of the sample to be tested, the moisture content taken and the remainder weighed and left in a scale pan until it reached the correct calculated weight, it was invariably found that the moisture content at the centre of the piece of soil (where the consolidometer sample was taken) was different to that calculated.

Hence the results can be viewed only from a qualitative angle.

The values of moisture content, pF and negative pore water pressure calculated from Fig. (14) are shown in Table (2).

Refer to Page 39:

**TABLE 2**

<table>
<thead>
<tr>
<th>Sample No. 6:</th>
</tr>
</thead>
<tbody>
<tr>
<td>From figure (15) it can be seen that the voids ratio versus log of negative pore water pressure follows a curve practically parallel to the remoulded compression one as down to a value of $p'' = -2.0$ Tons/Sq. ft.</td>
</tr>
<tr>
<td>$p'' = -2.0$ Tons/Sq. ft.</td>
</tr>
<tr>
<td>$p'' = pF 3.34$</td>
</tr>
<tr>
<td>The curve starts from the point where a remoulded consolidometer sample of initial moisture content 30.6% would commence. After the value of $p'' = -2.0$ tons/ sq. ft. it deviates from the general form</td>
</tr>
<tr>
<td>$p^F$</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>2.039</td>
</tr>
<tr>
<td>2.340</td>
</tr>
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<td>3.039</td>
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<td>3.817</td>
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<tr>
<td>3.942</td>
</tr>
<tr>
<td>4.039</td>
</tr>
</tbody>
</table>

$S_T < 100\%$ Below
of the consolidation curves, flattening out. From table (2) we can see that after this point the soil is in the state of "secondary unsaturation".

It would appear that up to this point equation (2) holds true. It should also be noted from fig. (14) that behind this moisture content the pF curve 6R reverses its curvature, i.e. embarks on section (C) of the pF curve.

Sample No. 7. (fig. 16):

The natural negative pore pressure and direct compression curves do not appear to parallel one another to any marked extent. The natural sample is initially in a state of "partial saturation" ($\psi = 0$ but $S_r < 100\%$). Hence the effects of soil moisture tension and direct compression do not seem analogous.

In the case of the remoulded sample, the curve due to compression by surface tension forces once again follows the form of the remoulded compression curves until just after the point where $p'' = -2.0$ tons/sq. ft. which is, as in the case of sample No. 6, where the pF/moisture content curve becomes concave upwards.

Sample No. 8. (fig. 17):

Here the pF curve has not reached section (C), and neither has the soil entered into the range of "secondary unsaturation", i.e. $S_r < 100\%$, anywhere along the range from $p'' = 0$ to $p'' = -10.0$ tons/sq. ft. The curve follows the expected form for the whole of its length.

Sample No. 9. (fig. 18):

It does not appear that $p = -p''$ in this case. This ties in with the pF curve which in this case has a peculiar
form. The section (B), over which most of the compression of
the soil takes place, is practically non existent. This is
not surprising when the shrinkage curve (fig. 13) is referred to,
as it can be seen from this that the clay does not compress much at all
under the action of soil moisture tensions.

Preconsolidation Loads:

In figs. 19–22 some of the remoulded samples showing definite
preconsolidation loads are illustrated. The preconsolidation load
\( p_0 \) is calculated using Casagrande's \((1)\) method of an empirical
construction. This method of estimating the load is not accepted
by all investigators as being consistent with field observations,
but is still very widely used.

Below is given a list of samples, initial moisture contents
and the preconsolidation loads \( (p_0) \) obtained graphically.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Moisture Content</th>
<th>( p_0 ) (Tons/sq. ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9R</td>
<td>36.6%</td>
<td>0.59</td>
</tr>
<tr>
<td>6R</td>
<td>29.9%</td>
<td>0.96</td>
</tr>
<tr>
<td>7R</td>
<td>27.7%</td>
<td>1.60</td>
</tr>
<tr>
<td>6R</td>
<td>35.2%</td>
<td>0.86</td>
</tr>
<tr>
<td>6R</td>
<td>32.2%</td>
<td>1.02</td>
</tr>
</tbody>
</table>

Refering to the \( p_0 \) versus moisture content curves (fig. 14.)
we see that, at these specific moisture contents, the only sample
to have had any possible preconsolidation from negative pore
pressure is 7R 27.7%. There has been no other stress history so
that except for the one sample there should be no sign of pre-
consolidation.

The only explanation that comes to mind for this would tie
in with a hypothesis put forward by Terzaghi in 1941, in which
he states that the natural consolidation curve for a clay is
very flat under slow loading, as would occur in nature, as "the
SAMPLE No. 7
COMPRESSION CURVE SHOWING APPARENT
PRECONSOLIDATION LOAD

P₀ = 1.60 Tons/sq. ft.
SAMPLE No. 8

COMPRESSION CURVE SHOWING APPARENT
PRECONSOLIDATION LOAD

Pc = 1.50 Tons/sq. ft.

LOG OF PRESSURE (TONS/ SQ. FT.)

FIGURE 21
SAMPLE No. 3
COMPRESSION CURVE SHOWING APPARENT
PRECONSOLIDATION LOAD

$P_0 = 0.59$ Tons/ sq. ft.

LOG OF PRESSURE (TONS/ SQ. FT.)  

FIGURE 22
rigid bond between solid parts of adsorbed water films surrounding clay particles" was not broken under these conditions. Now if by "air-set" the clay gains this inherent strength due to the development of non-liquid water, as mentioned by Grim and Guthbert (5), these bonds may not be immediately dislocated under small loads, but with the large loads they may be broken and be unable to reform with the rapid loading, giving the illusion of a small preconsolidation load. It appears that it would be interesting to load a clay slowly in small increments in the laboratory and see if any change in the shape of the compression curve occurs.

The sample 7R 27.7% had been subjected to a negative pore water pressure of 2.2 tons/sq.ft. = pF 3.38 as can be seen from fig. (14). The preconsolidation load actually shown is less than this, being only 1.60 tons. This would indicate some recovery of the soil on swelling, as this sample has been allowed to do. Even so, it would require nearly 33 feet of overburden weighing 110 lbs. per cu. ft. to produce such a preconsolidation, which gives an idea of the magnitude of the effect produced by even this small amount of desiccation.
CONCLUSIONS:

(1) The voids ratio versus moisture content curves give a good general picture of the compression of a clay under the action of negative pore water pressures.

The three important characteristics of this relationship are "breakaway" voids ratio, curvature and slope of the line after breakaway and from these we can get a qualitative picture of the compressibility by desiccation, maximum pore size and uniformity of pore size.

(2) The pH versus moisture content curves can be divided into three sections (A), (B) and (C) - (see discussion of results P.34) and it is along the section (B) that a clay is compressed by soil moisture tension. Hence the more compressible the soil, the longer the section (B), which represents the state of "primary Unsaturation", will be.

(3) The general effects of desiccation on the compressibility of the soil are:

(a) To lower the voids ratio at which any particular load will be carried.
(b) To decrease the compression index.
(c) To induce a preconsolidation load which can be quite considerable in magnitude.
(d) In most cases the clay on regelling and drying tends to revert to the same compression characteristics as the undisturbed sample.
Any pattern of behaviour which may have been represented mathematically, involved such small variations that the tendency was obscured by small experimental errors.

(h) The effective pressure was equal to the negative pore water pressure throughout the state of "primary unsaturation" unless the pF/moisture content curve for the soil embarked upon the concave upwards portion of its shape i.e. section (i).

Hence we can say that \( p = -p'' \) for moisture contents that lie within the "primary unsaturation" range and are greater than that at which the pF/moisture content curve reaches section (c).

For soils which are definitely in the "modified primary unsaturation" range this does not seem to hold. Hence care should be taken in applying equation (2). In cases such as finding the equilibrium moisture contents under large pavements dissection should be used.

(5) Small preconsolidation loads of magnitude up to a ton/sq. ft. can appear, due probably to "air-set", where there is no previous stress history for the soil.

Soil moisture tension forces can induce large preconsolidation loads, equal to those caused by many feet of overburden.
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