THE MECHANICAL PROPERTIES OF SATURATED AND PARTLY SATURATED SOILS WITH SPECIAL REFERENCE TO THE INFLUENCE OF NEGATIVE PORE WATER PRESSURES

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ABSTRACT

The effective stress law used for describing the behaviour of saturated soils with positive pore-water pressures is investigated for its validity when used with negative pore water pressures or suctions. An extension of the law to include partly saturated soils is discussed theoretically and applied to investigations of some mechanical properties, particularly shear strength and volume change phenomena, of cohesive and non-cohesive soils.

Particular attention is paid to the parameter, χ which relates suction and effective stress in partly saturated soils. A series of tests on Braehead silt which demonstrates the effect on χ of type of test, values of pore-air and pore-water pressures, degree of saturation and wetting-drying hysteresis is described. The possible range of values of the parameter is also treated.

Errors arising from putative undrained tests on partly saturated samples using standard techniques are indicated and a series of truly undrained tests on compacted clay described. These tests, performed under mercury, were used to investigate strength and pore-pressure phenomena in undrained compression. Experimental verification is given of the method normally used for calculating pore-air pressure changes from volume changes in compacted soils.

The effective stress law is applied to a number of problems involving pore-water suctions including pore-pressure parameters for

saturated and partly saturated soils, suctions in undisturbed saturated samples, volume changes in drying clays and shear strength characteristics of remoulded clay.

The techniques used for some of the tests were new to soil mechanics and the development of apparatus suitable for use with triaxial testing equipment is treated in some detail.

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NOMENCLATURE

The principal symbols used in this thesis are summarised below.

$\sigma_1, \sigma_2, \sigma_3$	major, intermediate and minor principal total stress	lb/in^2
$\sigma'_1, \sigma'_2, \sigma'_3$	major, intermediate and minor principal effective stress	lb/in^2
τ	shear stress	lb/in^2
* u_w	pore-water pressure	lb/in^2
* u_a	pore-air pressure	lb/in^2
σ_g	intergranular stress	lb/in^2
p_c	equivalent consolidation pressure	lb/in^2
T	surface tension of air-water interface	dyne/cm
* l	sample length	in.
* A	sample cross sectional area	in.^{2}
* V	sample volume	cm^3
ε	axial strain = $\frac{-\Delta l}{l_o}$ x 100	%
v	volumetric strain = $\frac{\Delta V}{V_o}$ x 100	%
v_w	$\frac{\Delta V_w}{V_o}$ x 100 (ΔV_w = vol. of water removed during test)	%
c'	apparent cohesion w.r.t. effective stresses	lb/in^2
ϕ'	angle of shearing resistance	degrees

* subscript o means initial value

c_{cu}	consolidated undrained apparent cohesion	lb/in^2
ϕ_{cu}	" " angle of shearing resistance	degrees
c_r	true cohesion (undrained tests)	lb/in^2
ϕ_r	angle of internal friction (undrained tests)	degrees
s	sensitivity = $\frac{\text{undisturbed strength}}{\text{remoulded strength}}$	
ψ	angle of intrinsic friction of soil mineral	degrees
w	water content = $\frac{W_w}{W_s} \times 100$ (W_w = wt. of water, W_s = wt. of dry soil)	%
$^*V_v$	void volume	cm^3
V_w	vol. of water in sample	cm^3
$^*S_r$	degree of pore space saturation = $\frac{V_w}{V_v} \times 100$	%
$^*e$	void ratio = $\frac{V_v}{V_s}$	-
G_s	specific gravity of soil mineral	gm/cm^3
V_s	solids volume = $\frac{W_s}{G_s}$	cm^3
$^*n$	porosity = $\frac{V_v}{V} \times 100$	%
γ_d	dry density of soil mass = $\frac{W_s}{V}$	lb/ft^3
γ_w	density of water	gm/cm^3
a	area of intergranular contact	in^2
P.L.	plastic limit	%
L.L.	Liquid limit	%
L.I.	Liquidity index = $\frac{w - P.L.}{L.L. - P.L.}$	-

H	Henry's co-efficient of gaseous solubility (= 0.02 vols. air/vol. water at 20°C.)
k_o	co-efficient of earth pressure at rest $= \dfrac{\text{lateral effective stress}, \sigma_h'}{\text{vertical effective stress}, \sigma_v'}$
χ, A, B	empirical parameters

CHAPTER 1

THE EFFECTIVE STRESS LAW AND GENERAL SOIL-WATER RELATIONSHIPS

(1.1) Introduction

Until quite recently soil mechanics research had confined itself almost exclusively to problems of saturated soil behaviour. The development of quantitative methods for appreciating the strength and deformation characteristics of these soils had to wait for Terzaghi's realisation of the effective stress principle, and even then the two phase material proved difficult enough to handle theoretically. In fact a large family of saturated soils - those in which the pore water pressures were considerably below atmospheric, though the pore spaces were completely filled with water - was treated very scantily.

The great development of soil mechanics began in the 1920's but, since the end of the 19th century, soil physicists had been working to put general soil-water relationships on a sound rational basis. Today a vast literature exists on the subject and its influence is being increasingly felt in soil mechanics research. By subjecting theories based on the effective stress principle to critical analysis in the light of soil physics knowledge much can be done to solve some outstanding engineering problems.

The shrinking and swelling of clay soils in some climates and the effect on structures and pavements is one such problem which is beginning to yield to this treatment. These soils may be fully or partly saturated but there are also many engineering works which depend for their stability on partly saturated soils, the most obvious being embankments of freshly compacted fill.

Analysis of the mechanical behaviour of the three phase (water-air-mineral) soil system presents formidable complexities, yet much can be done, and this thesis is an attempt to clarify some aspects of the mechanics of soils, saturated as well as partly saturated, which may be loosely described as possessing pore water suction.

The first chapter will be devoted to an exposition of the current forms of the effective stress law and those principles of soil-water physics which are most pertinent to the immediate needs of soil mechanics. The remainder of the thesis will deal with theoretical and experimental methods, some of them new to soil engineering, for describing and predicting some of the more important relationships between pore pressure and stress-strain phenomena in soil.

(1.2) The effective stress law in saturated soils

The stimulus for most of the current work in soil mechanics undoubtedly came from Terzaghi's realisation of the effective stress principle. (Terzaghi 1923). As formulated by Rendulic (1936), this states:

"The mechanical behaviour of a clay depends only on the effective stress in the clay". The effective stress is given by:

$$\sigma' = \sigma - u_w \qquad \text{........ (1.1)}$$

σ' - normal effective stress

σ - normal total stress

u_w - pore water pressure

The influence of finite contact areas between soil particles has been treated by Bishop (1950), Lamb and Whitman (1959) and Skempton (1960). The average intergranular stress may be shown to be:

$$\sigma_g = \sigma - (1-a)\, u_w \qquad \text{......... (1.2)}$$

where a = effective contact area of the soil particles per unit area of the soil,

but this is not necessarily the stress controlling the overall behaviour of the soil.

For shear strength considerations Skempton showed that experimental evidence for soils and rocks correlates well with the effective stress defined by:

$$\sigma' = \sigma - \left(1 - \frac{a \tan \psi}{\tan \phi'}\right) u_w \qquad \text{........ (1.3)}$$

where ψ = angle of intrinsic friction of soil mineral

ϕ' = angle of shearing resistance of soil.

For most clay minerals ψ ranges from 5° to 10°, and ϕ from

20°-30°, $\frac{\tan \psi}{\tan \phi'}$, falling between 0.15 and 0.25. Hence, even for moderate values of a, equation (1.3) reduces to equation (1.1) on neglecting second order effects.

For volume changes the following expression holds for an increment $\Delta\sigma$ in total stress:

$$\Delta\sigma' = \Delta\sigma - (1 - \frac{C_s}{C})\Delta u_w \qquad \text{.......... (1.4)}$$

where C_s = compressibility of soil mineral;

C = " " " skeleton.

For soils $\frac{C_s}{C}$ is negligible and equation (1.4) also reduces to (1.1). As discussed by Bishop (1959) there is no evidence, theoretical or experimental, to disprove the validity of Terzaghi's effective stress law for either shear strength or volume change phenomena in soils.

Rendulic (1937), Taylor (1944) and Henkel (1958) have presented evidence for the validity of equation (1.1) as defining the effective stress controlling shear strength in clays while Laughton (1955) has demonstrated its validity for the volume change of lead shot and Globigerina ooze up to pressures of 1,000 kg/cm^2.

(1.3) Critical Appraisal

For coarse grained soils a simple model consisting of a skeleton of rigid grains in particle to particle contact completely surrounded by ordinary liquid water (i.e. obeying the laws of hydro-dynamics) is sufficiently accurate for the validity of eqn. (1.1) to be accepted without question on theoretical grounds. For clays, as pointed out by Lambe and Whitman, such a model may be grossly inaccurate, and they question the validity of eqn.(1.1) for either volume change or shear strength relationships. As examples they cite the phenomenon of "secondary compression" or volume change of many clay soils which occurs long after the excess pore water pressure has become zero, and the decrease in shearing strength of these soils at very slow rates of loading. These examples do not, however, invalidate the law but merely emphasise that any correlations must be made with allowance for the time dependence of some of the variables.

Many saturated* soils in the field and most at some stage of their existence between sampling and laboratory testing exhibit a soil moisture deficit - i.e. they would imbibe water supplied at atmospheric pressure unless subjected to a change in their external stress system. As pointed out by Aitchison (1960) the soil water is referred to as being in a state of suction which implies the existence of a pressure deficiency or tension in at least some parts of the soil water. If eqn.(1.1) is to be used for such cases there is a tacit assumption that

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*Saturated in as much as $S_r = 100\%$. For a more detailed subdivision of the states of saturation, see Aitchison (1956).

the action of a negative pore water pressure is not altered by a change in the physical state of the pore fluid. (Aitchison and Donald [1956]).

Hence, before we can extend our principles to encompass unsaturated soils, we must examine critically the significance of the variables in eqn.(1.1) for saturated samples.

(1.4) The meaning of pore water pressure

Pore water pressure, as measured and used by the engineer, could be defined as the hydrostatic pressure in the water in any real or imaginary macro-pore in the soil. It is only a directly measurable quantity however if defined as the pressure in a water system external to the soil specimen but in equilibrium with the soil water. Atmospheric pressure is taken as datum and sub-atmosphere pressures are referred to, perhaps somewhat contrarily, as negative pore pressures. No lower limit is set to these negative pore water pressures, and the laws of capillarity are assumed applicable. The influence of surface forces from the soil grains is usually ignored, as indicated by the term "macro-pore" in the definition above. The validity of this approach will be examined below in the light of some soil science evidence.

The pore water pressure, u_w, and the total stress, σ , are the only directly measurable variables in the effective stress equation. The effective stress σ', which has been defined simply as the stress which controls the mechanical behaviour of a clay, may then be regarded either as a stress determined by the forces acting at intergranular contacts and calculable from a soil model or simply as a parameter equal to the difference between total stress and pore water pressure without any attached physical significance. The first alternative is satisfactory for sands or medium fine silts, for which surface tension considerations are certainly valid (Schofield, 1935). For clays the second alternative is preferable for, as will be seen, the measured pore water pressure may have no physical significance as an actual pressure exerted isotropically in the pore fluid.

These considerations do not, however, vitiate the effective stress law provided it is considered as an empirical relationship between two measurable quantities and a third variable which may be correlated satisfactorily with mechanical properties such as strength and volume change. Indeed at present there is no other workable engineering system.

(1.5) Soil Suction

Many expressions are in current use to describe the state of the water in unsaturated soils or saturated ones with sub atmospheric pore pressures. The most common terms are soil suction, pressure deficiency, capillary potential, pore water tension and negative pore water pressure. In some cases many of these are identical, or nearly so, and it is instructive to examine the differences between them and their relevance to the actual physical state of the pore water. It must be added that this is not a closed subject as there seems to be considerable controversy in soil science literature on this very important and fundamental topic.

As pointed out recently by Aitchison (1960 b), the term "suction" is vague and ill-defined. According to Croney (1960 b), the accepted road engineering definition of suction is "... the total free energy depression per unit volume of water of the soil-water system", usually referred to free water at atmospheric pressure as datum. Aitchison, while recognizing the usefulness of this approach proposes that, for engineering purposes, "suction in a soil-water-air system is the pressure deficiency in the water phase of the pore fluid with respect to the soil air pressure under defined conditions of external stress and temperature." This is also the definition used by Bishop (1960 b), except that Aitchison's 'pressure deficiency' symbol, $\bar{p}''$, is replaced by the more self-explanatory expression $(u_a - u_w)$, where u_a is the measured pore air pressure. It will be shown that this term is a dominant influence in the behaviour of unsaturated soils, regardless of the absolute value of u_a, and it will be used throughout this thesis.

The soil science view as summarised by Gardner (1960) is essentially in accordance with this definition. He points out that the total potential of soil water can be expressed as the sum of three components:

(1) gravitational potential

(2) capillary potential

(3) osmotic potential.

Bolt and Miller (1958) add a fourth term, the adsorption potential due to surface force fields. Gardner's suction term represents only part of the total potential and is the component measured by tensiometer, suction plate or pressure membrane. The total potential may be measured by vapour equilibrium, or freezing point depression.

Schofield (1960) defines suction as the free energy or Gibb's potential of the soil water relative to pure water at the same temperature and pressure rather than as a pressure deficiency. For high suctions in swelling clays this is probably the best approach but it is of limited engineering usefulness. The Gibbs potential, Φ , was also used by Low (1958) as a measure of water holding forces in soil and, for some purposes he divided it into two components:

$$\Phi = \mu + \Theta \qquad \text{........ (1.5)}$$

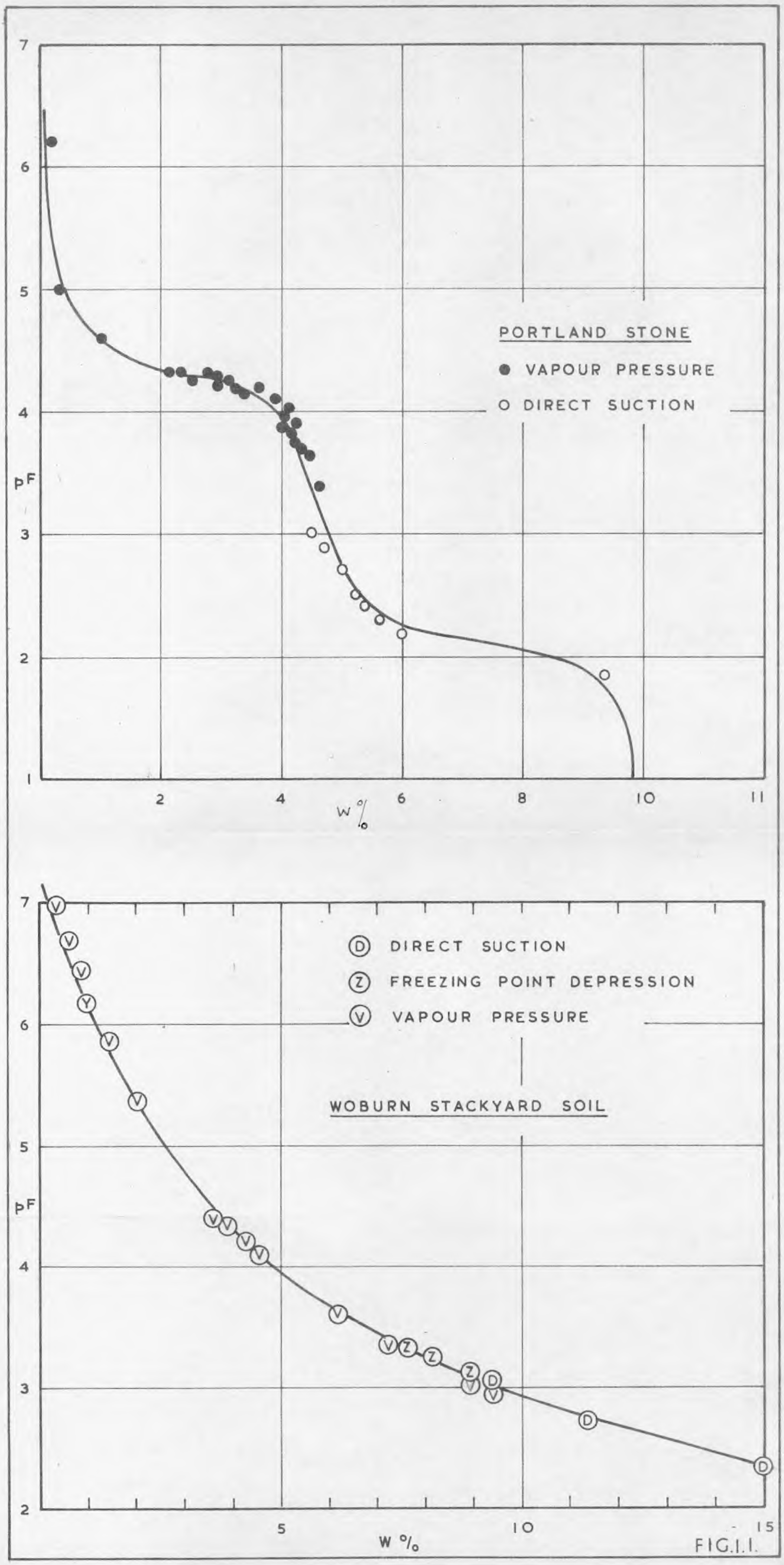
μ = chemical potential

Θ = positional potential.

A similar division has been made more recently by Marshall (1959) who uses the terms matrix suction and solute suction, the former being the equilibrium value measured by tensiometer and pressure membrane and the latter being principally an osmotic phenomenon. There does appear to be some confusion on this point as Schofield (1960) has said "... Gibbs potential of the water at all points within the swollen clay is the same as that of the pure water in equilibrium with it through the pressure membrane." This seems to imply a semi-permeable membrane as otherwise equilibrium would require solute diffusion into the water beneath the membrane.

It is important to investigate the errors resulting from using matrix suction as a substitute for total potential. Much experimental data is now available for suction-water content relationships of soils determined by a variety of methods. (For example, Croney, Coleman and Russam (1953); Coleman (1959)). A very satisfactory degree of overlap has been obtained between vapour pressure measurements and pressure membrane determinations, Coleman (1959) finding no appreciable differences at suctions approaching 1,000 atmospheres.

By using a novel technique for vapour pressure measurements Davidson and Schofield (1942) were able to demonstrate the practical equivalence of the two approaches down to suctions of about one atmosphere, i.e. in the tension plate range. Fig. (1.1) shows some of their results on soil and building stone. The complete equivalence of the methods used in the low suction range has also been established (see, for example, Croney, Coleman and Bridge [1952]).



PORTLAND STONE
VAPOUR PRESSURE
O DIRECT SUCTION
pF
W %
DIRECT SUCTION
FREEZING POINT DEPRESSION
VAPOUR PRESSURE
WOBURN STACKYARD SOIL
pF
W %

FIG.1.1.

Full details of these methods and their ranges of application may be found in publications by Croney, Coleman and Bridge (1952), Aitchison (1956) and Penner (1959), the last two being directed specifically to an engineering audience.

Finally, the precise meanings of the terms often used as synonyms for soil suction must be given.

Capillary Potential - This concept was originally due to Buckingham (1907) and represents the total potential in the absence of significant gravitational or osmotic effects. It has the dimensions of work per unit mass.

Soil Moisture Tension.

This is the height of a suspended column of free water in equilibrium with the soil water. In the absence of osmotic effects it is the capillary potential expressed as a height, h cm. of water. The term $pF = \log_{10} h$ was introduced by Schofield (1935) and is widely used as a convenient scale to cover the range of suctions from saturation to oven dryness.

Pressure Deficiency

This represents the decrease of water pressure below the air pressure with which it is in equilibrium across a curved interface. Its value is calculated according to the classical laws of surface tension and, in this thesis, will be indicated by $(u_a - u_w)$.

We have also the relationship

$$h = \frac{(u_a - u_w)}{\gamma_w} \qquad \text{.........} \ (1.6)$$

$(u_a - u_w)$ is exactly equivalent to Aitchison's p".

Negative Pore Water Pressure

This represents any sub-atmospheric pore water pressure measured in water equilibrated with the sample. It does not imply absolute tension until it numerically exceeds one atmosphere, i.e. $u_w < -14.7$ lb/in^2 at sea level. In the many cases where the soil air is at atmospheric pressure the negative pore water pressure is numerically equal to the pressure deficiency and the soil suction as defined earlier.

(1.6) Absolute tensions in water

The acceptance of the concept of negative pore water pressure implies that for $u_w < -14.7$ lb/in^2 the pore water exists in a state of absolute tension. There is ample experimental evidence to show that in carefully cleaned and de-aired systems water can sustain tensions of at least several hundred atmospheres without rupture and even in the presence of some dissolved air some tens of atmospheres may be supported. The available evidence has been reviewed by Marshall (1959). Following what appears to be somewhat of a tradition in this field the writer, in collaboration with G.E. Blight, arranged a simple laboratory demonstration which was able to produce 3½ atmospheres absolute tension after a few minutes of preparation.

Terzaghi (1925) realised the significance of the high tensile strength of water as a possible explanation for volume change in drying clays if the effective stress law was not invalid and capillary phenomena could still be invoked in the high stress range involved. Comparison of water contents produced by suction or applied pressure (Aitchison and Donald [1956] and some evidence in this thesis) has established that Terzaghi's approach is compatible with soil macro-behaviour even if the mechanism can be seriously questioned on fundamental grounds.

In spite of the incontrovertible evidence for the tensile strength of water, there has been considerable reluctance by soil scientists to admit that absolute water tensions are likely in actual soils. Gardner (1960) says that because of the number of mechanisms contributing to capillary potential, the actual pressure inside the soil water may not be less than atmospheric and for most soils the capillary model probably fails at suctions above one atmosphere.

Bolt and Miller (1958), Marshall (1959) and Greacen (1959) have all expressed doubts as to the ability of soil water to carry tension, a limit of about one atmosphere being set to the actual negative water pressure in the pores. However, if the soil pores are fine enough to support a $(u_a - u_w)$ greater than one atmosphere there seems no reason why the capillary approach should suddenly fail at one atmosphere suction. It is quite possible that the choice of this limit independently by many workers has been controlled by the regularity with which direct suction measuring apparatus breaks down in the vicinity of one atmosphere suction, but the physical dimensions of the measuring system bear no closer relationship to the soil pores than those of the apparatus used for measuring the tensile strength of water.

While recognizing the grounds for controversy on this point it must be stated that, at least in the moderate stress ranges of interest to soil engineers, there appears to be no mechanical behaviour incompatible with the assumption of absolute tension in the soil water, though in fairness it must be said that such water stresses have never been directly demonstrated.

(1.7) Validity of the 'capillary' approach

It has already been shown that the pressure deficiency approach to suction problems requires the extension of capillary laws into the range of absolute water tensions. Some writings which cast doubt on this have been referred to above, but the philosophy behind this simple and very useful tool has been recently restated by Aitchison (1960) as follows:

"While it is not suggested that the capillary model can be used to explain fundamental soil-water interactions, it is postulated that relationships of macro-physical quantities - each dependent upon unexpressed fundamental properties - can be developed by calculations based upon such elementary models."*

The parallel with the idea of surface tension itself is obvious.

Schofield (1960) has adduced evidence to show that, allowing for surface effects, the hydrostatic pressure midway between two montmorillonite platelets, 8.8mμ apart, in a pore fluid with 0.04 molar sodium ion concentration is about one atmosphere greater than the pressure of water in equilibrium with it through a pressure membrane. Obviously, for such a case the capillary hypothesis is not applicable, except possibly in a wholly empirical way.

Winterkorn (1953) discussed the possible effects of surface adsorption forces on the structure of the adjacent pore water and concluded that it would be severely modified. He quoted the case of a dense clay which contracted on freezing, the reason given being that the water structure, even at room temperature, was virtually that of ice, and the volume decrease on cooling was normal thermal contraction.

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* The use of the term 'capillary model' is taken to mean that the physical condition of the pore water is no different from that of water in a capillary tube of sufficient diameter for the simple laws of capillarity to apply. In other words, the pore water is assumed to be normal water, a fluid whose properties have been adequately summarised by Dorsey (1940). There need not necessarily be a curved air-water meniscus at each surface pore of a saturated sample if it is subjected to a confining pressure through a membrane but the behaviour of the soil may still be described by reference to a capillary model.

Lambe (1953) has suggested that the layer of strongly oriented water around a clay particle may be up to 10 molecules thick and the influence of orientation may be noticed at up to 200 molecules thickness. Low (1958) has demonstrated experimentally the decrease in density of water adjacent to clay particles up to distances of 84 A° due to orientation of the water molecules. Lambe and Whitman (1959) assert that virtually all the water in a highly plastic clay is under the influence of particle surface forces.

In the face of these opinions it may appear rash to persist with the capillary model approach but Marshall's (1959) review of the available evidence establishes its validity for predicting the principal soil water relationships up to suctions of at least 200 atmospheres - i.e. within the rupture strength of pure water.

Schofield (1960b) agrees with this conclusion and states that he does not think that orientation phenomena reach many Angstrom units out from the surface. He also states that for engineering purposes values of u_w as measured by normal techniques may be correlated with soil behaviour even though the actual water pressure in the pores may be different.

Hence although the actual limit of applicability of the capillary model is still very uncertain, the method may be applied with confidence to moderately coarse grained soils, with caution to unexceptional clays in the typical engineering stress range, and with hesitation to highly plastic clays with low water contents.

(1.8) Basic relationships of moisture stresses and capillarity

Having elected to use the capillary analogy, it is necessary to investigate the mathematical relationships implied by it. The results will not be discussed in great detail, as they are all extremely well known and many excellent summaries are available (see, for example, Aitchison [1956], Alpan [1959]).

(a) Pressure difference across a curved air-water interface.

The stability of an air-water interface requires a pressure difference of:

$$(u_a - u_w) = T\left(\frac{1}{R_1} + \frac{1}{R_2}\right) \qquad \text{............ (1.7)}$$

where R_1, R_2 = principal radii of curvature of the meniscus;

T = surface tension of water;

= 75.7(1 - 0.002t) dyne/cm.

t = temperature in ^{o}C.

This pressure difference may be expressed as an equivalent height of capillary rise:

$$h = \frac{2T\cos\alpha}{rg\gamma_w} \qquad \text{........... (1.8)}$$

where r = radius of capillary tube (cm.)

g = acceleration due to gravity (cm. sec^{-2})

γ_w = density of water (gm.cm^{-3})

α = contact angle of water to soil mineral, usually assumed zero;

h = soil moisture tension (cm.)

We have also, by an earlier definition:

$$pF = \log_{10} \left(\frac{2T}{rg\gamma_w} \right) \qquad \text{(1.9)}$$

(for $\alpha = 0$)

(b) Soil water vapour pressure.

For equilibrium conditions the vapour pressure above a curved interface is related to the soil moisture tension by the Kelvin equation:

$$h = -\frac{R\Theta}{Mg} \cdot \ln \left(\frac{H}{100} \right) \qquad \text{(1.10)}$$

where R = universal gas constant

M = molecular weight of water

H = relative humidity of soil atmosphere

Θ = absolute temperature.

Combining equation (1.8) with the definition of pF leads to:

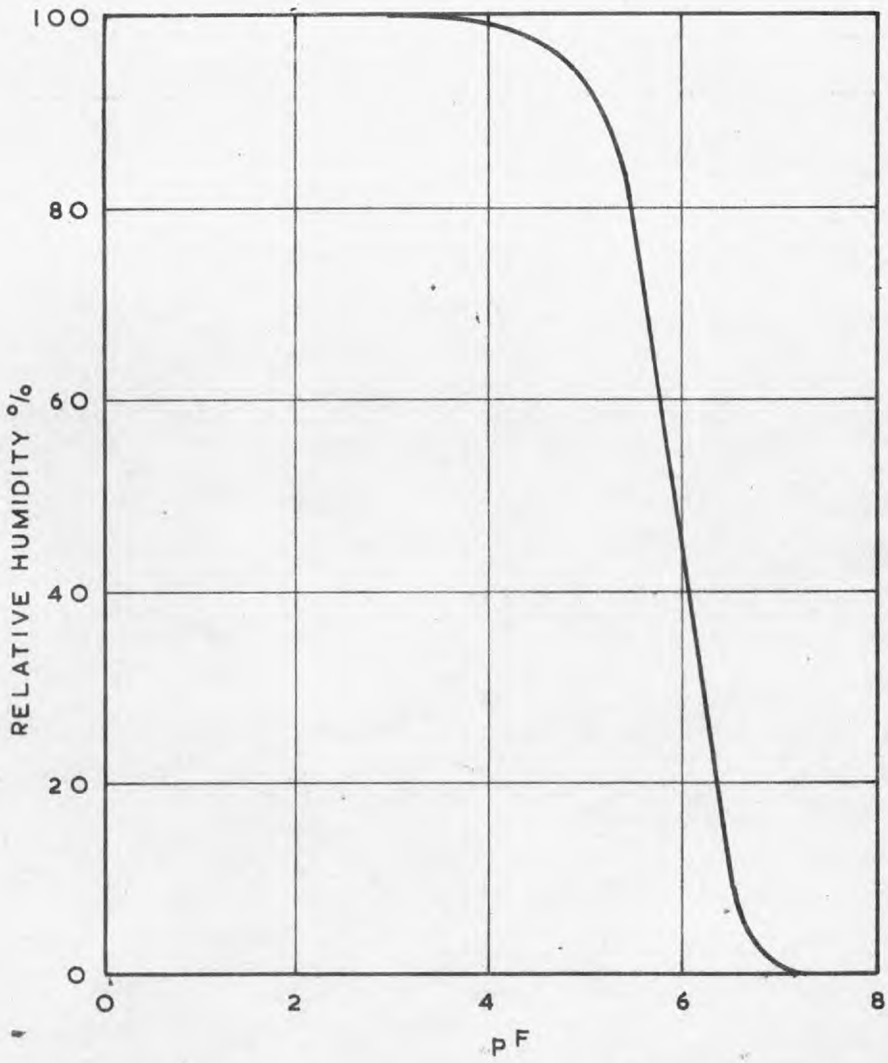
$$pF = 6.5 + \log_{10} (2 - \log_{10} H) \qquad \text{(1.11)}$$

This simple but very important relationship enables us to plot the variation of suction or pF with humidity, as in Fig (1.2). To keep an engineering perspective on these values Table 1.1 has been prepared. The normal engineering stress range, certainly for laboratory work, is found above 98% humidity and the use of the pF symbol in this thesis will be restricted to discussion of results taken directly from soil science literature.

(1.9) Moisture characteristic curves

The most convenient way of representing the moisture-holding capacity of a soil is a plot of water content (gravimetric) as abscissa against suction (or its equivalent expressions) as ordinate. Two such soil moisture characteristics curves, so-called after Childs (1940), have already been given in Fig 1.1. There is no unique relationship for any soil and the effect of grain structure and wetting-drying hysteresis has been adequately dealt with, qualitatively, by Croney and Coleman (1954) and others. For theoretical purposes it is sometimes more convenient to plot degree of pore space saturation, S_r, in place of water content. Such a graph will be referred to as a modified moisture characteristic curve.

The main features of a moisture characteristic curve can be predicted from considerations of possible capillary configurations in the pore space of an ideal soil, a collection of uniform spheres arranged in an ordered packing. This geometrical approach was fruitfully pursued in a series of papers by Haines. His conclusions were



RELATIONSHIP BETWEEN pF AND RELATIVE HUMIDITY AT 20°C

FIG. 1.2.

TABLE 1.1

RANGE OF MOISTURE STRESSES

(UP TO OVEN DRYNESS)

pF	h cm.	equivalent water head in ft.	Pressure Difference $(u_a - u_w)$	
			lb/in^2	kg/cm^2
0	1	3.28×10^{-2}	1.42×10^{-2}	0.001
1	10	3.28×10^{-1}	1.42×10^{-1}	0.01
2	10^2	3.28×1	1.42×1	0.1
3	10^3	3.28×10	1.42×10	1
4	10^4	3.28×10^2	1.42×10^2	10
5	10^5	3.28×10^3	1.42×10^3	10^2
6	10^6	3.28×10^4	1.42×10^4	10^3
7	10^7	3.28×10^5	1.42×10^5	10^4

neatly summarised by Keen (1931). The implications of soil moisture characteristic curves, particularly with regard to pore size distributions, have been dealt with by Childs (1940), and information on actual and idealised pores in soils and other porous materials has been summarised in a recent symposium of the Colston Research Society (1958)

CHAPTER 2

THEORETICAL CONSIDERATIONS

(2.1) Introduction

Because of the complexity of the three-phase unsaturated soil system, theoretical studies on the mechanical properties of such soils are few. General soil-water relationships for both saturated and unsaturated soils have been studied thermodynamically (e.g. Edlefsen and Anderson [1943], Bolt and Miller [1958]) but the results are not usually in a form readily usable by engineers. This chapter attempts to summarise the engineering approach to problems of pore pressure changes, shear strength and volume change phenomena when the pore water has initial negative values. Pore pressure prameters will be treated first as they represent, in some ways, the more tractable aspects of the problem.

(2.2) Pore Pressure Parameters in Saturated Soil

The change in pore water pressure in a saturated clay subjected to maximum principal stress, σ_1 , and intermediate and minor principal stresses $\sigma_2 = \sigma_3$ has been given by Skempton (1954) as:

$$\Delta u = B\left(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right) \qquad \text{.............. (2.1)}$$

The form of the equation was derived from elastic theory with modified co-efficients to allow for real soil behaviour.

The parameter B represents change in pore water pressure for a change in all-round stress and, for saturated soils, its value was shown to be 1 (neglecting second order effects). The parameter A allows for the effect of shear stresses on the soil structure and hence on the pore water pressure.

Hence, for saturated soils

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \qquad \ldots\ldots\ldots\ldots (2.2)$$

For three independent principal stresses, (an unusual laboratory condition at the present), Henkel (1958) has suggested that a more convenient form of eqn. (2.1) is:

$$\Delta u = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3} + \beta\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2} \qquad \ldots\ldots\ldots\ldots (2.3)$$

Within the framework of assumptions as to the properties of soil water when under suction forces, as detailed in the previous chapter, there is no reason why eqns (2.1), (2.2) or (2.3) should not also apply to moderately negative values of initial pore water pressure.

(2.3) Pore Pressure Parameters in Partly Saturated Soil

(a) Under equal all-round stress.

The engineering problem of predicting pore water pressures in loaded compacted earth fills has stimulated many investigators (e.g., Brahtz, Zanger and Bruggerman [1936], Hamilton [1939], Bishop [1950], Hilf [1956]) into attempting theoretical solutions. The common approach adopted by these investigators was to calculate changes in pore air pressure in a sealed mass of soil of known compressibility by applying Boyle's law to the calculated volume changes under load, and making an allowance for solubility of the pore air in the pore water according to Henry's law of gaseous solubility. This could not give the pore water pressure directly, the $(u_a - u_w)$ term being ignored by the earlier investigators. Hilf (1956) realised this limitation and Bishop and Henkel (1957 - p.180) have queried the usual assumption of initial atmospheric pore air pressure u_a.

The results of the analysis may be presented in many ways, e.g.

(1) After Bishop:

$$\frac{\Delta p}{p_o} = \frac{-\dfrac{\Delta V}{V_o}}{\dfrac{\Delta V}{V_o} + n_o(1 - S_o + S_o H)} \qquad \ldots\ldots\ldots\ldots (2.4)$$

V_o = initial sample volume

S_o = initial degree of saturation $\left(\dfrac{V_w}{V_{v_o}}\right)$

V_{v_o} = initial void volume

p = absolute air pressure

n_o = initial porosity = $\dfrac{V_{v_o}}{V_o}$

H =Henry's coefficient (0.02 vols. of air per unit volume of water at 20°C)

(2) After Hilf

$$\Delta u_a = \frac{P_o \cdot \Delta V}{V a_o - \Delta V + 0.02\, V_w} \qquad \text{.............} (2.5)$$

u_a = pore air pressure (gauge)

P_o = initial absolute pressure

Va_o = initial air volume

V_w = volume of pore water.

(3) After Hamilton:

$$P = P_o \left(\frac{V_{a_o} + 0.02V_w}{Va_c + 0.02V_w} \right) \qquad \text{.............} (2.6)$$

P, P_o = absolute air pressures

V_{a_o}, V_{a_c} = initial and compressed air volumes

V_w = volume of pore water.

Hamilton's form of the equation possesses the most apparent relationship to the physical processes involved and can indeed be written straight down without calculation. It is the simplest to use when merely checking the validity of the theory but Bishop's eqn (2.4) is more adaptable for investigating other consequences of the solution process (Bishop and Eldin, 1950).

A thermodynamic approach was made to the problem by Coleman and Croney (1952). Their basic equation is:

$$u = \alpha p + s \qquad \ldots\ldots\ldots\ldots\ldots\ldots (2.7)$$

where p = applied pressure

u = pore water pressure under load

s = suction, measured on an unconfined sample (negative).

Equation (2.7) cannot be applied to undrained compression but only to the constant water content condition where u_a is allowed to dissipate to atmospheric pressure. In all but its most recent statement eqn.(2.7) has been assumed valid for natural conditions. In Croney and Coleman (1960) p is replaced by $\frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$ for conditions where k_o, the ratio of lateral to vertical effective stresses, differs from 1. σ_1, σ_2 and σ_3 are the major, intermediate and minor total stresses respectively. For saturated soils with $\alpha = 1$ the modified approach to eqn (2.7) is at variance with eqn.(2.3) for which experimental conformation exists. (Henkel, 1958)

Croney and Coleman showed that the shape of the soil shrinkage curve, $\frac{dV_s}{dV_m}$, (where dV_s = overall volume change, dV_m = volume of water removed) was a measure of α, but the thermodynamic argument implies reversibility which is certainly not a property of most soils on first compression.

For saturated soils, of course, $\alpha = B = 1$, though α is not a strict counterpart of B. (see later).

Bishop (1960, b) has extended Skempton's parameters to unsaturated soils under all-round stress.

$$\Delta u_a = B_a . \Delta\sigma \qquad \text{.............. (2.8)}$$

$$\Delta u_w = B_w . \Delta\sigma \qquad \text{.............. (2.9)}$$

B_a and B_w both have values between 0 and 1 and are treated as empirical parameters, though B_a can be calculated as shown above if the soil compressibility is known.

The Croney-Coleman eqn. (2.7) was derived for atmospheric pore air pressure. For the case of undrained compression of a partly saturated soil Bishop has extended it to:

$$\Delta (u_w - u_a) = \alpha . \Delta (\sigma - u_a) \qquad \text{................ (2.10)}$$

where α = an empirical parameter

σ = total applied stress.

It immediately follows that:

$$\Delta (u_a - u_w) = \Delta\sigma (B_a - B_w) = \alpha . \Delta (\sigma - u_a).$$

$$\therefore \quad B_a - B_w = \alpha (1 - \frac{u_a}{\Delta\sigma}) = \alpha (1 - B_a)$$

$$\text{i.e. } B_w = B_a + \alpha (1 - B_a) \qquad \text{..............(2.11)}$$

It has been claimed that the α and B coefficients are exactly equivalent, but (2.11) shows that $\alpha = B_w$ only if $B_a = 0$, i.e. $\Delta u_a = 0$, which is impossible in an undrained test.

(2.3) (b) Stress systems with $\sigma_1 > \sigma_2 = \sigma_3$

Of greater interest than the case discussed above is the stress system applied by normal triaxial testing equipment with two equal principal stresses. Bishop has extended Skempton's eqn (2.1) for a single pore fluid to:

UNDRAINED TESTS

$$\Delta u_w = B_w \left[\Delta\sigma_3 + A_w(\Delta\sigma_1 - \Delta\sigma_3) \right] \quad \text{.............. (2.12)}$$

$$\Delta u_a = B_a \left[\Delta\sigma_3 + A_a(\Delta\sigma_1 - \Delta\sigma_3) \right] \quad \text{...............(2.13)}$$

and the Croney-Coleman equation to:

CONSTANT WATER CONTENT

$$\Delta(u_w - u_a) = \alpha . \left[\Delta(\sigma_3 - u_a) + \eta(\Delta\sigma_1 - \Delta\sigma_3) \right] \quad \text{...(2.14)}$$

The coefficients A_a, A_w, B_a, B_w, α and η are empirical parameters determined from appropriate compression tests. The distinction between undrained (i.e. totally sealed) tests and constant water content tests is that in the latter the pore air is allowed to drain at a constant back pressure (not necessarily atmospheric). This is the test condition under which Croney and Coleman measure their α, which is therefore deprived of generality. The particular advantages of constant water content tests will become clear at a later stage of this thesis. As for all round stresses, Δu_a can be calculated from (2.4), (2.5) or (2.6) if volume changes are known but $\Delta (u_a - u_w)$ has to be found experimentally.

(2.4) <u>The effective stress law for negative pore water pressures</u>

(a) Saturated Soils.

It was pointed out in Chapter 1 that if the capillary hypothesis was accepted as valid for moderate negative pore water pressures, then the effective stress laws for shear strength and volume change remained unaltered

i.e. $\sigma^1 = \sigma - u_w$ (volume change)

$$\sigma^1 = \sigma - (1 - a\frac{\tan\psi}{\tan\phi^1})\, u_w \quad \text{shear strength}$$

with $\sigma^1 = \sigma - u_w$ .... (1.1)

serving practically for both cases.

Many clays remain sensibly 100% saturated at high suctions (e.g. Holmes, 1955, gives shrinkage data for a black clay with $S_r = 99\%$ at pF 5, i.e. at $u_w = -1{,}420\ lb/in^2$) and Aitchison (1957) has postulated that extension of the effective stress law into this range accounts for the very high strengths found for dessicated soils in the field in arid and semi-arid climates. The range of suctions found at shallow depths under such conditions has been experimentally determined by Aitchison (1956). For saturated clays Aitchison arrives at the expression

$$C = K. (\sigma'_a)^m \qquad \text{.............. (2.15)}$$

where K, m constants

C = ½ (unconfined compressive strength)

σ'_a = ambient effective stress.

For consolidated undrained triaxial tests at moderate ambient pressures, the apparent cohesion, C, is found to be a linear function of the consolidating pressure (equivalent to σ'_a), but the values quoted by Aitchison range between m = 0.3 and m = 0.5. The assumptions in his theory are valid for the low stress range so that the m values should all be close to 1 while the soils remained saturated. The reason for the difference may be that Aitchison's m values are average figures over a wide range of degrees of saturation, though this could not explain all the discrepancy for his heavier clay soils.

(b) Unsaturated soils.

Apart from the works of Haines (1925, 1927, 1930) little theoretical treatment of the strength of partly saturated soils had been done until quite recently. Aitchison and Donald (1956) published a method for calculating the change in effective stress with $(u_a - u_w)$ allowing for changes in the degree of pore space saturation. This theory, based on an ideal soil model and valid for relatively incompressible non-cohesive soils has been further developed and will be presented at the end of this chapter.

The recent Conference on Pore Pressure and Suction in Soils brought together several variations on the theme of a modified effective stress law for partly saturated soils. The most general expression was that originally put forward by Bishop (1955), viz:

$$\sigma' = \sigma - u_1 - x.(u_2 - u_1) \quad \ldots\ldots\ldots\ldots (2.16)$$

where u_1, u_2 = pressures in two pore fluids separated by curved menisci

x = a parameter (equal to 1 in saturated soils and 0 in dry soils) depending primarily on the degree of pore space saturation.

The other expressions of the law were all for the special case of $u_a = 0$ as their authors had been concerned mainly with natural soil deposits for which this is a reasonable approximation.

These expressions were:

Croney, Coleman and Black (1958) $\sigma' = \sigma - \beta' u$ .... (2.17)

Aitchison (1960) $\sigma' = \sigma + \psi . p''$ .... (2.18)

Jennings (1960) $\sigma' = \sigma + \beta . p''$ .... (2.19)

Remembering that p'' was a pressure deficiency, i.e. a numerically positive quantity, these three expressions are seen to be identical with Bishop's equation for $u_1 = u_a = 0$ and

$$\chi = \psi = \beta = \beta' \qquad \text{........ (2.20)}$$

(Note - Croney and Coleman's symbol for total stress, p, has been altered to conform with standard soil mechanics nomenclature).

Such a profusion of equivalent parameters was highly undesirable and Aitchison and Bishop (1960) proposed a final form of the law, viz:

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \qquad \text{............. (2.21)}$$

It is instructive to look into the derivations of the laws given above. Croney, Coleman and Black (1958) applied the thermodynamics of reversible processes to compression of a partly saturated soil element and arrived at the equation

$$\sigma' = p - \beta . u \qquad \text{............. (2.22)}$$

Once again this is open to question on the grounds of the reversibility of the process except perhaps for overconsolidated soils and small stress changes. By analogy with this expression the equation

$$\sigma' = \sigma - \beta' . u \qquad \text{............. (2.17)}$$

was proposed as a definition of the effective stress controlling shear strength. Although shear strains are very far removed from reversibility the expression arrived at, partly by intuition, is, surprisingly, identical with the Bishop, Aitchison and Jennings formulae derived from a mechanistic approach.

Coleman (1960) extended the thermodynamic argument to include air pressures other than atmospheric. This led to the equation:

$$\sigma_v' = (\sigma - \pi) - \beta(u - \pi) \quad \ldots\ldots\ldots\ldots\ldots\ldots (2.23)$$

i.e. the only result is to shift the datum from which pore pressure and applied pressure was measured. This is a very important principl the basis of pressure membrane technique and the "translation of origin" method of suction measurement used by Hilf (1956). The form of eqn. (2.23) is then the same as Bishop's eqn. (2.16).

Jennings, adopting the traditional "wavy plane" approach of soil mechanics, considered that in partly saturated soil with some air voids connecting to the atmosphere such a plane, passing through poin of interparticle contact, would intersect a proportionate area β of water and $(1 - \beta)$ of air. Hence if p" was the pressure deficiency

$$\sigma' = \sigma + \beta . p'' \quad \ldots\ldots\ldots\ldots\ldots (2.19)$$

and also he defined the pore water pressure by

$$u = -\beta . p'' \quad \ldots\ldots\ldots\ldots\ldots (2.24)$$

Eqn. (2.24) was also given by Aitchison and was an attempt to leave the Terzaghi effective stress law unchanged, viz.:

$$\sigma' = \sigma + \beta\, p'' = \sigma - u \quad \ldots\ldots\ldots\ldots\ldots (1.1)$$

However, in this expression, with the Aitchison-Jennings definition of u, σ is the only measurable variable and the equation is not usable.

Although Jennings arrived at the common formulation of the law, his model was oversimplified, particularly in its neglect of the "tensile pull" exerted by each air-water interface. However, if β is treated as an empirical parameter and not an exact area this difficulty disappears.

Aitchison's approach was similar to Jennings', except that he stated that the area over which the water pressure was effective depended on S_r. Aitchison also gave the formula

$$\sigma' = \frac{1}{100} \left(S_r . p'' + \sum_{0}^{p''} 0.3\ p'' . \Delta S_r \right) \qquad \ldots\ldots\ldots (2.25)$$

for the effective stress caused by suction forces.

This effective stress is greater than $\frac{S_r . p''}{100}$ which is the contribution according to Jennings' model, at least for moderately high S_r. The significance of eqn. (2.25) will be dealt with at the end of this chapter.

Bishop does not give details of the derivation of his equation but χ is obviously treated as an empirical parameter to allow for the diminished effectiveness of the pore water pressure when air has entered the soil structure. The inclusion of the air pressure u_a (or u_1 in eqn 2.16) serves merely to move the datum for pressure measurements.

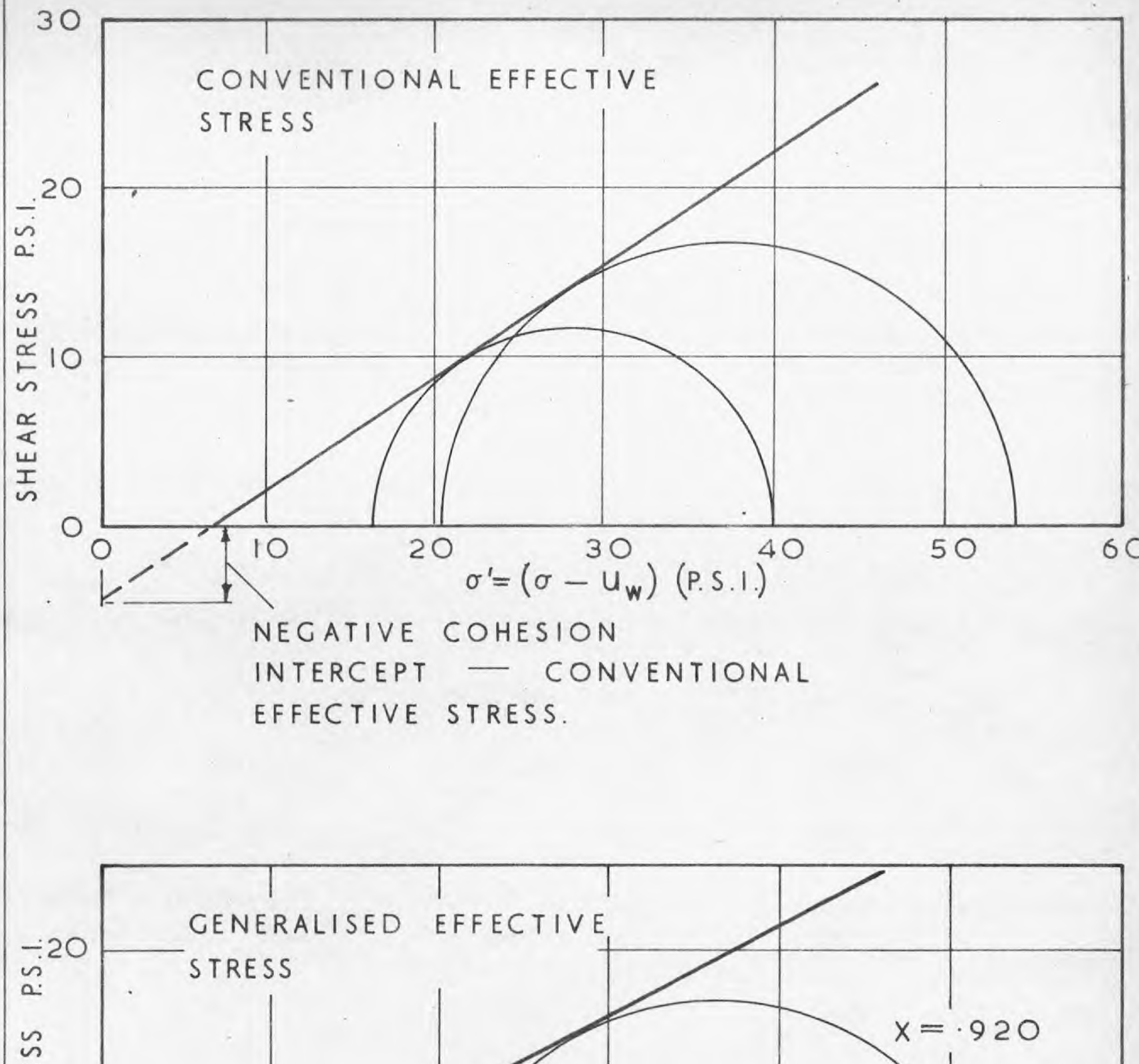
A detailed mechanical treatment based on the Bishop approach was given by Alpan (1959), allowing for finite contact areas between particles. However, in spite of an attempt to allow for some of the errors inherent in the usual idealised model soil the only workable equation to emerge from the analysis was that $\chi \doteq S_r$, an assumption which represents the simplest approach to the problem.

Hilf (1956), although he appreciated that large differences could exist between u_a and u_w in compacted soils, nevertheless used the effective stress eqn. (1.1) with his measured value of pore water pressure. As pointed out by Bishop (1960, b) this, the 'conventional' approach, leads to an overestimate of effective stress of $(1 - \chi)$ x $(u_a - u_w)$. The result, on a conventional Mohr circle plot, is a spurious 'negative cohesion'. Fig. 2.1 (after Blight [1961]) presents experimental confirmation.

(2.5) Graphical construction for χ.

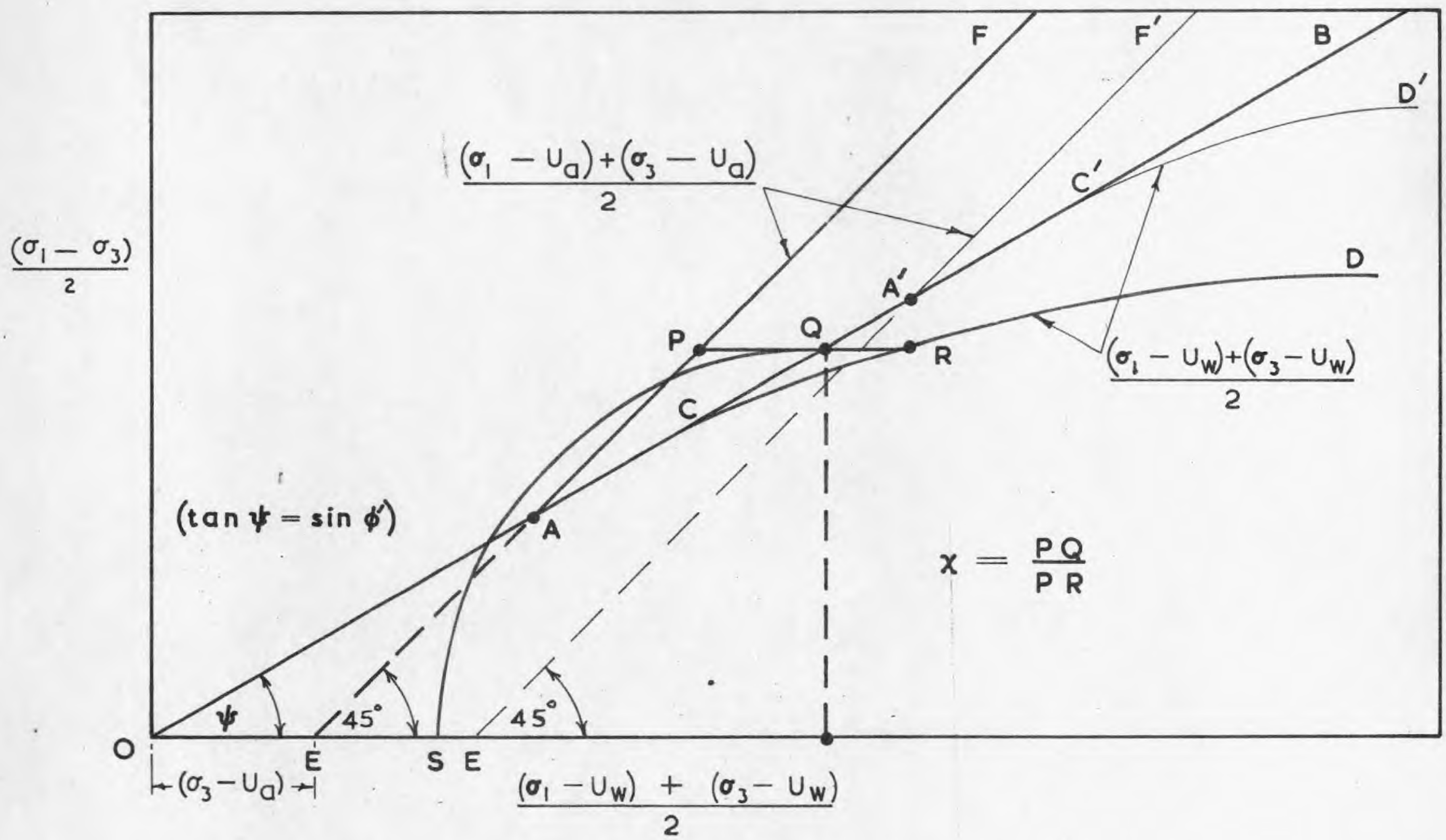
Blight (1961) has also shown the graphical significance of the χ factor as it applies to a range of tests on compacted soils. Some details have also been given by Bishop et al (1960). A short description will be given here of the method and its significance for the series of compression tests on Braehead silt described later in this thesis.

The plot shown in Fig. 2.2 represents a slight variation of the Mohr envelope. The abcissa $\frac{(\sigma_1 + \sigma_3)}{2} - u_w$ is the centre of the



TWO UNDRAINED SHEAR TESTS ON A COMPACTED SHALE SHOWING AN APPARENT NEGATIVE COHESION INTERCEPT WHEN PLOTTED CONVENTIONALLY

FIG. 2.1.



GRAPHICAL SIGNIFICANCE OF χ FACTOR

(TESTS WITH $\sigma_3 - U_a$ CONSTANT)

FIG. 2.2.

conventional Mohr circle and the ordinate, $\frac{(\sigma_1 - \sigma_3)}{2}$, the radius of the circle. Hence OAB is the envelope of the tops of the Mohr circles and not the tangent points. By simple geometry we have

$$\tan \psi = \sin \phi' \qquad \text{.......... (2.26)}$$

where ϕ' = angle of shearing resistance.

For a series of drained or constant water content tests with $(\sigma_3 - u_a)$ = constant, saturated samples will fall along OAB (for cohesionless soil such as the silt tested) and partly saturated samples along CD. Line EAF, at 45° to each axis represents $\frac{(\sigma_1 + \sigma_3)}{2} - U_a$ versus $\frac{(\sigma_1 - \sigma_3)}{2}$.

$$\text{Since } \frac{\sigma_1 + \sigma_3 - 2u_a}{2} = \frac{\sigma_1 - \sigma_3 + 2\sigma_3 - 2u_a}{2}$$

$$= \frac{(\sigma_1 - \sigma_3)}{2} + (\sigma_3 - u_a) \qquad \text{....... (2.27)}$$

then OE = $(\sigma_3 - u_a)$, which is held constant for the test series.

For a given $(\sigma_3 - u_a)$ there is a minimum sample strength which can be measured, as u_w cannot be greater than u_a. Hence the minimum effective stress in the sample for any series is $(\sigma_3 - u_a)$.

For point A we have, (if S = 100%)

$$\frac{(\sigma_1 - u_w) + (\sigma_3 - u_w)}{2} = \frac{\sigma_1' + \sigma_3'}{2} = OE + \frac{EA}{\sqrt{2}} \qquad \text{...... (2.28)}$$

$$\therefore \frac{\sigma_1' + \sigma_3'}{2} = (\sigma_3 - u_a) + \frac{\sigma_1' - \sigma_3'}{2} \quad \text{.......... (2.29)}$$

$$\text{i.e. } \sigma_3' = (\sigma_3 - u_a) \quad \text{.......... (2.30)}$$

Therefore EA is shown dotted as this part of EAF is experimentally unobtainable. OA has been drawn in full as saturated tests with $u_a = u_w = 0$ can be used to define this part of the true effective stress envelope.

Blight has pointed out that, on this type of plot, χ at any point is given by the ratio $\frac{PQ}{PR}$. For a sample represented at failure by point R, the true effective stress to give the same strength on a saturated sample is that required to produce Q as the top of a Mohr circle. A quadrant QS of this circle has been drawn and, from simple geometry, PQ = ES.

$\therefore$ True lateral effective stress $\sigma_3' = OS$

i.e. $\sigma_3' = OE + ES = (\sigma_3 - u_a) + PQ$

$\therefore (\sigma_3 - u_a) + \chi(u_a - u_w) = (\sigma_3 - u_a) + PQ$

or $$PQ = \chi(u_a - u_w) \quad \text{.................. (2.31)}$$

$$\text{Now } PR = \left[\frac{(\sigma_1 + \sigma_3)}{2} - u_w\right] - \left[\frac{(\sigma_1 + \sigma_3)}{2} - u_a\right] = (u_a - u_w) \quad \text{... (2.32)}$$

$$\therefore \frac{PQ}{PR} = \chi\frac{(u_a - u_w)}{(u_a - u_w)} = \chi \quad \text{................. (2.33)}$$

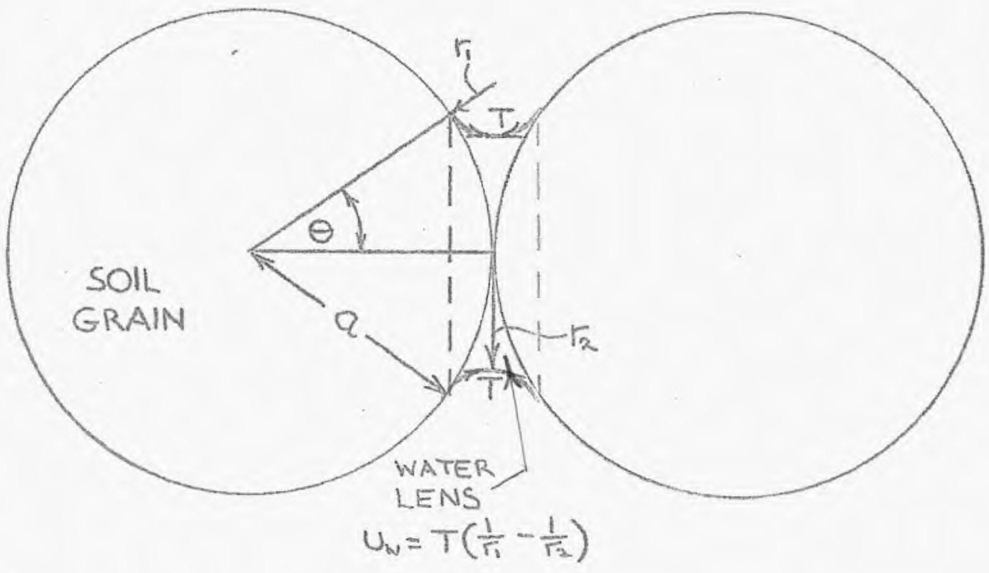
Also shown on Fig. 2.2 are points A^1, C^1, D^1, E^1, F^1, corresponding to another series of testsaat a higher constant $(\sigma_3 - u_a)$. Provided that the χ-S relationship is not affected by the higher initial effective stress, then $F^1A^1C^1D^1$ should be geometrically similar to FACD but displaced a distance AA^1 along the true effective stress line.

(2.6) Effective stresses calculated from ideal soil geometry

The work of Haines on 'cohesion' in ideal soil masses was followed up by Aitchison and Donald (1956) with a graphical solution for converting a $(u_a - u_w)$ versus S_r plot into a σ^1 versus S or $(u_a - u_w)$ plot. The treatment was carried slightly farther in a previous thesis by the writer (Donald, 1957) and a summary of the results will be presented here, as they form the basis for Aitchison's eqn (2.25) and some new analytical extensions of it.

(2.6.1) Haine's equations

In a series of papers Haines (1925, 1927, 1930) assisted by some criticism from Fisher (1926) derived the basic relationships between interparticle stresses and water content for ideal soil of uniform spherical grains in open and close packed array.



BASIC IDEALISED UNSATURATED SOIL UNIT

FIG. 2.3

The force between two spherical grains due to the pressure deficiency in the water and the surface tensile pull (FIG 2.3) is found as follows:

$$F = T\left(\frac{1}{r_1} - \frac{1}{r_2} \right) x \pi r_2^{\,2} + 2\pi r_2 T$$

(The second term represents the tensile pull of the meniscus resolved at the mid-plane of the diagram).

Expressing r_1 and r_2 in terms of Θ, the half angle subtended at the grain centre by the water lens, the above expression reduces to:

$$F = \frac{2\pi aT}{1 + \tan\frac{\Theta}{2}} \qquad \text{.................. (2.34)}$$

Transferring this interparticle force into a stress for both open packed and close hexagonal packed spheres the following expressions emerge:

$$\sigma' = \frac{\pi T}{2a(1 + \tan\frac{\Theta}{2})} \qquad \text{... open packing .... (2.35)}$$

$$\sigma' = \frac{\pi\sqrt{2}\,T}{a(1 + \tan\frac{\Theta}{2})} \qquad \text{... close packing ...... (2.36)}$$

Fisher (1926) considered the exact capillary curve, the "Nodoid" of Plateau, the blind experimentalist. Table 2.1 presents some typical values, the water contents being calculated for a particle specific gravity $G_s = 2.65$.

		OPEN PACKING			CLOSE PACKING		
Θ degrees	$(U_a - U_w)^{\frac{T}{xa}}$	w %	S_r%	$\sigma'(x\frac{T}{a})$	w %	S_r%	$\sigma'(x\frac{T}{a})$
5	250	0.002	0.01	1.505	0.004	0.03	4.257
10	58.6	0.03	0.09	1.444	0.06	0.45	4.086
15	24.1	0.14	0.41	1.388	0.27	2.04	3.926
20	12.25	0.39	1.13	1.335	0.78	5.91	3.777
30	4.10	1.60	4.60	1.239	3.22	24.4	3.504
40	1.40	4.23	12.32	1.152	LIMIT OF DISCRETE		
45	0.71	6.24	18.15	1.111	WATER LENSES		

TABLE 2.I

EFFECTIVE STRESSES IN IDEAL SOILS

Following Haines' theory the important data of Table 2.II are also obtained, viz:

RELATIONSHIPS OF WATER CONTENT, SOIL MOISTURE TENSION AND EFFECTIVE INTERGRANULAR STRESS IN UNSATURATED IDEALIZED COHESIONLESS SOILS

Condition	S_r %	w% Close Packing	w% Open Packing	$(u_a - u_w)$ Close Packing	$(u_a - u_w)$ Open Packing	σ' Close Packing	σ' Open Packing	θ Degr
1. Saturated in equilibrium with free water	100	13.3		0		0		-
	100		34.1		0		0	-
2. Pore spaces filled with water at limiting moisture tension - about to drain	100	13.3		12.9T/a		12.9T/a		-
	100		34.1		4.8T/a		4.8T/a	-
3. Pore spaces drained to lenticular state at same value of moisture tension as condition 2	6.0	0.8		12.9T/a		3.8T/a		20
	4.7		1.6		4.8T/a		1.2T/a	30
4. Discrete water lenses on point of fusion = wet limit of lenticular phase	24.3	3.2		4.1T/a		3.5T/a		30
	18.2		6.2		0.7T/a		1.1T/a	45

Table 2.II

(2.6.2) Extension to graded soils.

In extending Haines' theory to graded soils the writer assumed that the soil could be represented by a model consisting of groups of pores draining at any particular suction, each group being represented by one diameter ideal spherical particles in open or close packing. The relative proportions of each particle size present in a unit cube of soil were chosen so that the model gave the appropriate S_r versus $(u_a - u_w)$ curve.

For changes ΔS_r and $\Delta(u_a - u_w)$ the change in effective stress, σ', was calculated from the data in Table 2.II assuming that the ΔS_r represented drainage of a group of ideal soil spheres from saturation to condition 3 of the table. $\Delta(u_a - u_w)$ was assumed effective on the proportion of the soil which remained saturated. The effective stress in previously drained groups was assumed to remain constant for increasing $(u_a - u_w)$, an assumption justified by Table 2.I. The solution finally arrived at was:

$$\sigma' = \frac{1}{K_2}\left\{(1-K_1)\Big[(u_a-u_w)f(u_a-u_w)\Big]_{(u_a-u_w)_d}^{(u_a-u_w)} + K_1\int_{(u_a-u_w)_d}^{(u_a-u_w)} f(u_a-u_w)\,d(u_a-u_w) - (100-K_2)\left([\bar{u}_a-u_w]-[\bar{u}_a-u_w]_d\right)\right\}$$

$$+ (u_a-u_w)_d \qquad \ldots\ldots\ldots\ldots (2.37)$$

where K_1 = 0.25 (open packing), 0.29 (close packing)

K_2 = 95.3 (" "), 94 (" ")

and $S_r = f(u_a - u_w)$ .... (2.38)

$(u_a - u_w)_d$ = value of suction at which the coarsest group of pores drains.

By assuming a simplified 'modified moisture characteristic curve' as in FIG. 2.4 it was possible to draw some general conclusions from eqn. (2.37).

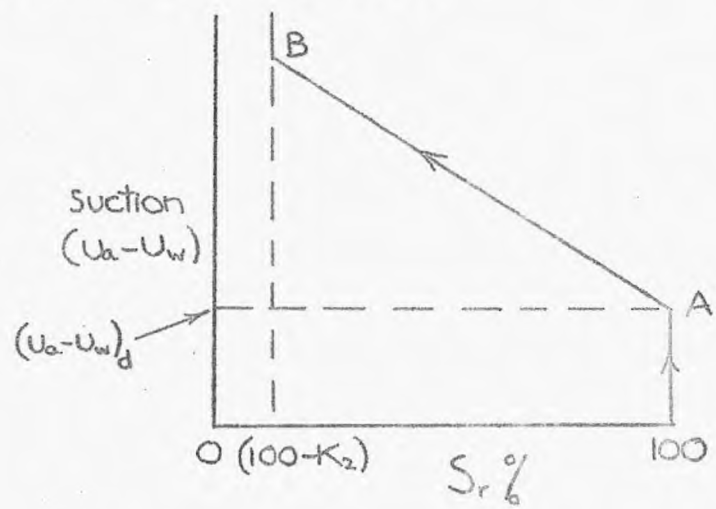


FIG.2.4 IDEALISED MOISTURE CHARACTERISTIC

The equation to AB is

$$S_r = -b.(u_a - u_w) + \underbrace{(100 + b[u_a - u_w]_d)}_{=C} \qquad \text{........... (2.39)}$$

Insertion in (2.37) gave:

$$\sigma' = \frac{1}{K_2}\left\{ b\left(\frac{K_1}{2} - 1\right)(u_a - u_w)^2 + (C + K_2 - 100)(u_a - u_w) + \frac{3K_1 b}{2}(u_a - u_w)^2_d \right\} \quad \text{...(2.40)}$$

From this it was deduced that the σ' versus $(u_a - u_w)$ plot was parabolic, convex upwards, and with the maximum effective stress occurring at a degree of saturation less than 100% only if

$$b < \frac{K_2}{(u_a - u_w)_d (1-K_1)} \quad \text{or } C < \frac{K_2}{(1-K_1)} + 100 \qquad \ldots\ldots\ldots (2.41)$$

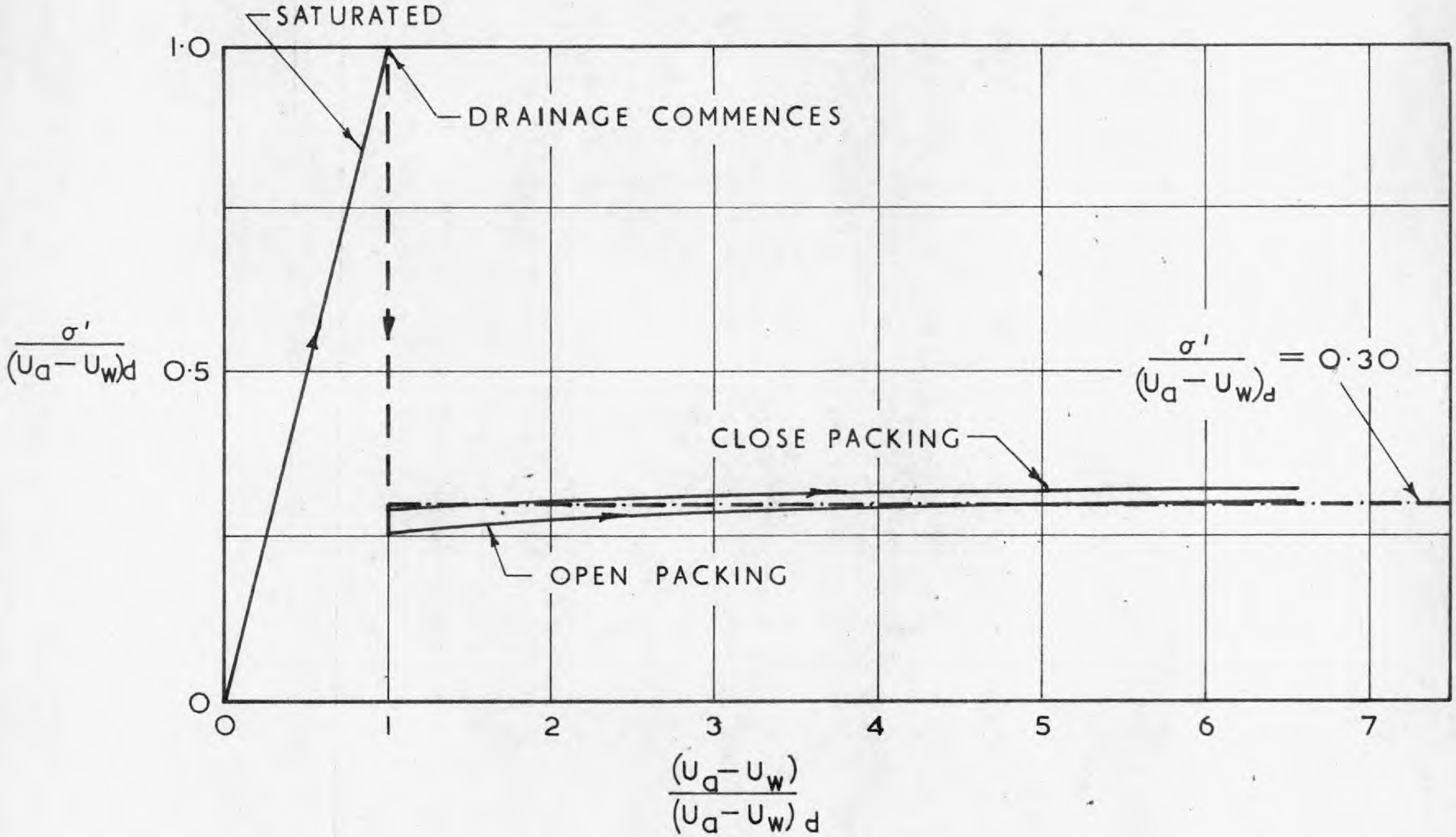
Hence, the steeper the moisture characteristic curve, the more likely is the maximum effective stress to be found at low degrees of pore space saturation.

(2.6.3) Simplified theory.

Equation (2.37), though accurate within the limits of the assumptions made for the theory, is difficult to apply other than as a step by step process. Even then the accuracy of the result is unlikely to justify the effort involved as the model soil bears only a general resemblance to any practical case. An approximate approach has recently been developed and leads to a somewhat surprising result.

The effective stress in an ordered array of mono-sized spheres is presented as a function of suction, $(u_a - u_w)$, in a non-dimensional plot in FIG. 2.5. There is very little difference between open and close packing and, over a wide suction range

$$\sigma' \simeq 0.3\,(u_a - u_w)_d \qquad \ldots\ldots\ldots\ldots\ldots (2.42)$$



EFFECTIVE STRESSES CAUSED BY DRAINAGE OF UNIFORM SPHERES

FIG. 2.5.

This leads us immediately to a form of Aitchison's eqn. (2.25), viz:

$$\sigma' = S.(u_a - u_w) + 0.3\sum_{0}^{S}(u_a - u_w).\Delta S \qquad \text{.......... (2.43)}$$

Here S is expressed, for simplicity, as a ratio between 0 and 1 and the total volume of water remaining in any drained pores is taken as negligible. This is sufficiently true for all but low values of S. The first term represents the contribution to the effective stress of those pores which remain saturated at the specific $(u_a - u_w)$, the volume porosity being assumed equal to the area porosity (see, for example, Leliavsky 1958. This is generally known as Dupuit's principle). The second term represents the contribution of all pores which have drained up to the particular value of $(u_a - u_w)$. The change in degree of saturation, ΔS_r, for an increment in $(u_a - u_w)$, is taken as a measure of the proportionate area of a wavy plane through the soil occupied by the particular sized pores draining in that increment (from Dupuit's principle).

For a continuous S_r versus $(u_a - u_w)$ variation, we have:

$$\sigma' = S.(u_a - u_w) + 0.3\int_{0}^{S}(u_a - u_w).dS \qquad \text{......... (2.44)}$$

This can be combined with eqn.

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \qquad \text{......... (2.21)}$$

to derive a relationship between χ and S.

To simplify the algebra we may take both σ and u_a equal to zero, then

$$\chi.(-u_w) = S.(-u_w) + 0.3 \int_0^S (-u_w).dS \qquad \text{.........}\ (2.45)$$

$$\text{i.e.} \quad \chi = S + \frac{0.3}{(-u_w)} \int_0^S (-u_w).dS \qquad \text{.........}\ (2.46)$$

The solution has been studied for the three idealised moisture characteristic curve shapes in FIG. 2.6.

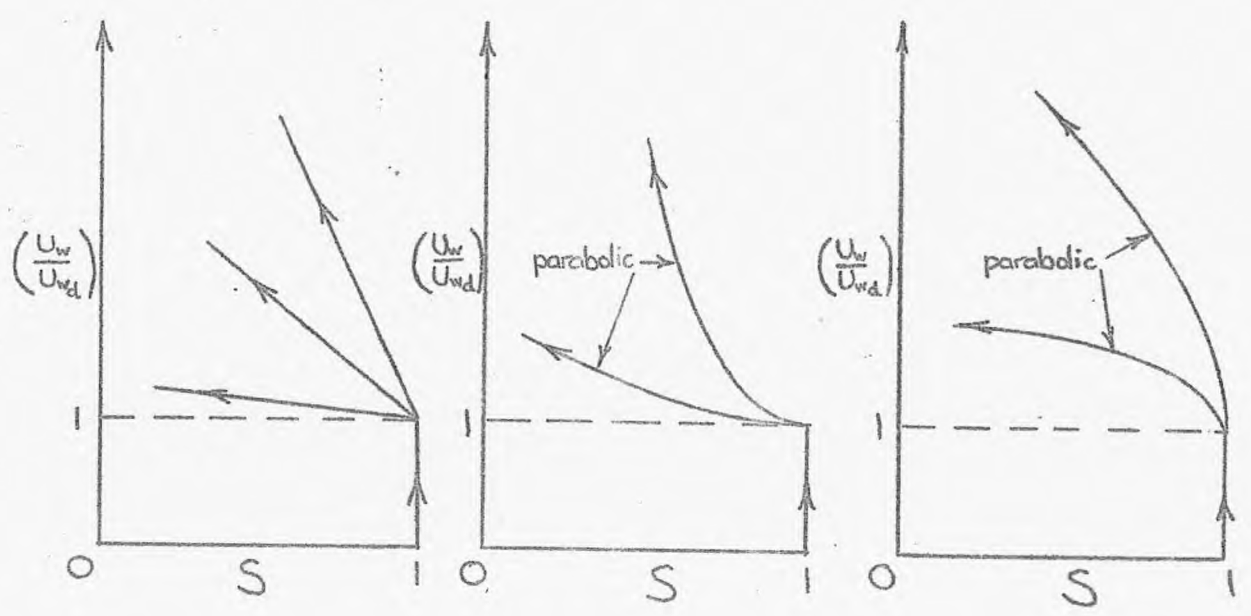


FIG. 2.6

MODIFIED MOISTURE CHARACTERISTIC CURVES - IDEALISED

The equations to the dimensionless moisture characteristic curves are:

Straight line $\left(\frac{u_w}{u_{w_d}}\right) = -m.S + (1 + m)$ ....(2.47)

Parabola Concave up $\left(\frac{u_w}{u_{w_d}}\right) = m.S^2 - 2m.S + (1 + m)$ ....(2.48)

Parabola Concave down $S = m\left(\frac{u_w}{u_{w_d}}\right)^2 - 2m.\left(\frac{u_w}{u_{w_d}}\right) + (1 + m)$ ....(2.49)

where m = an arbitrary constant

u_{w_d} = pore water pressure at incipient drainage.

The parabolas have been chosen tangent to either the horizontal or the vertical at S = 1.

Inserting (2.47), (2.48) and (2.49) in (2.46) and carrying out the integration,the following results emerge:

Straight line: $\chi = S - 0.15(S-1)\ \frac{(mS-m-2)}{(mS-m-1)}$ ....(250)

Parabola (1) $\chi = S - 0.1(S-1)\left[\frac{2}{1 + (mS^2-2mS+[1+m])}\right]$ ....(2.51)

Parabola (2) $\chi = S - 0.1(S-1)\ \left[2 + \left(\frac{u_{w_d}}{u_w}\right)\right]$ ....(2.52)

We cannot say what the relation between χ and S is without knowing the constant m for a particular curve. We can, however, derive limiting values for either very flat or very steep curves by letting m tend to zero or infinity. These limits are:

Straight line	(1)	$\chi = 0.30 + 0.70S$
	(2)	$\chi = 0.15 + 0.85S$
Parabola (1)	(1)	$\chi = 0.30 + 0.70S$
	(2)	$\chi = 0.10 + 0.90S$
Parabola (2)	(1)	$\chi = 0.30 + 0.70S$
	(2)	$\chi = 0.20 + 0.80S$

There is a very small range covered by all of these equations and, with little error, we can write for all curves (using average co-efficients)

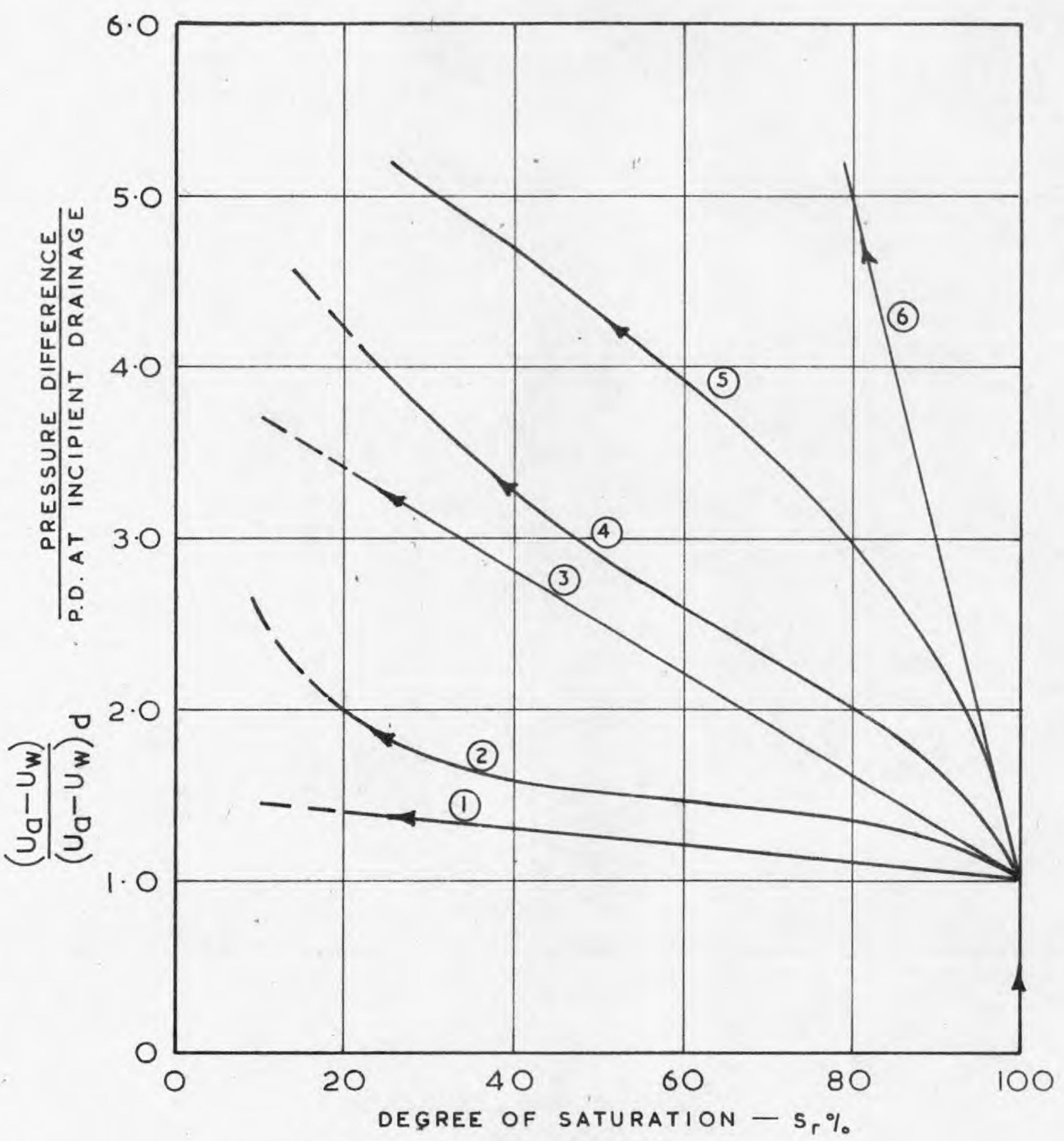
$$\chi = 0.22 + 0.78S \qquad \text{...........} \quad (2.53)$$

It was assumed that curves other than the limiting ones would also yield - S relationships approximating to eqn (2.53). Taking the straight line at 45°, i.e. m = 1 gives:

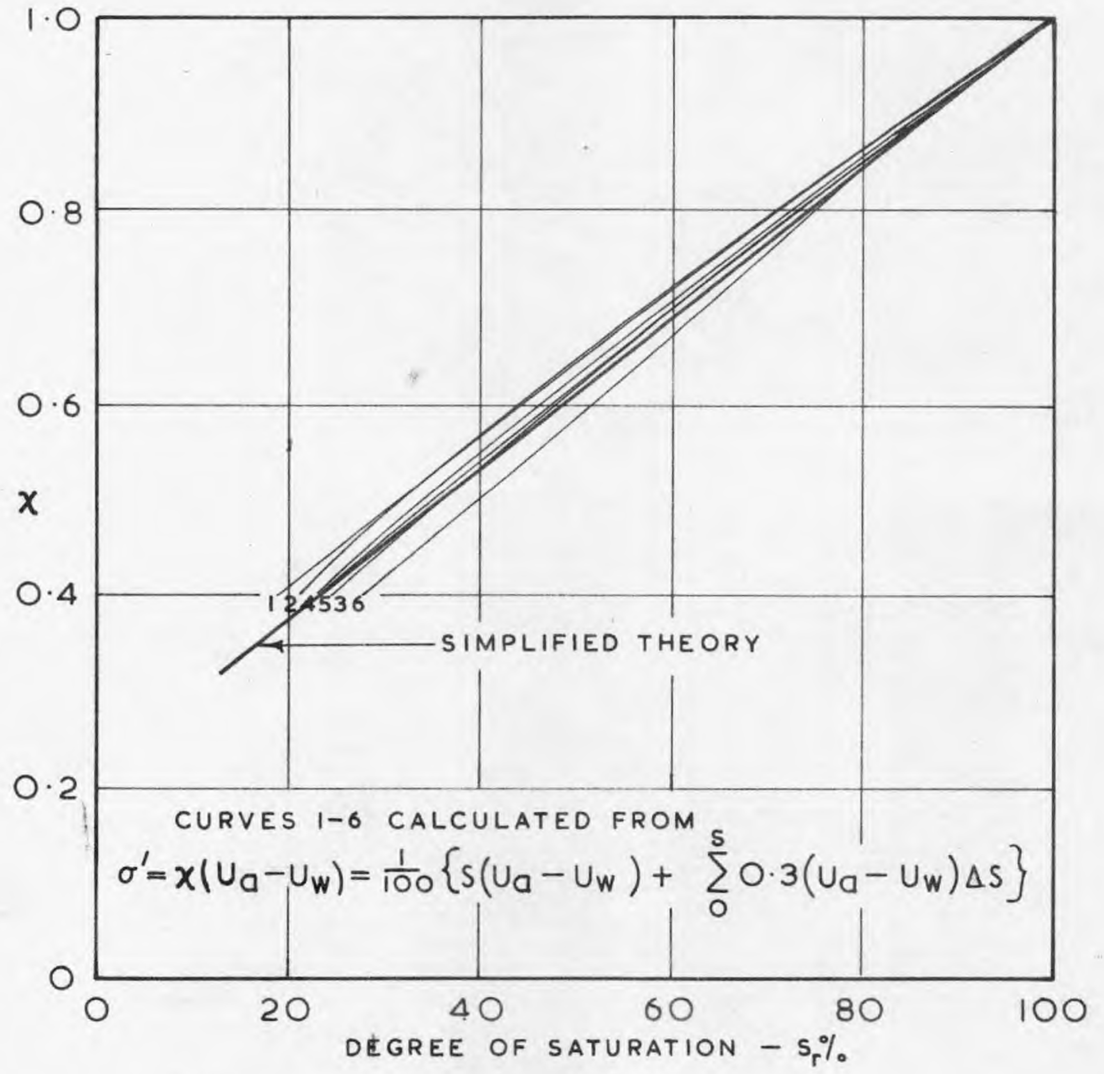
$$\chi \simeq 0.22 + 0.80S - 0.02S^2 \qquad \text{...........} \quad (2.54)$$

which helps to justify the assumption.

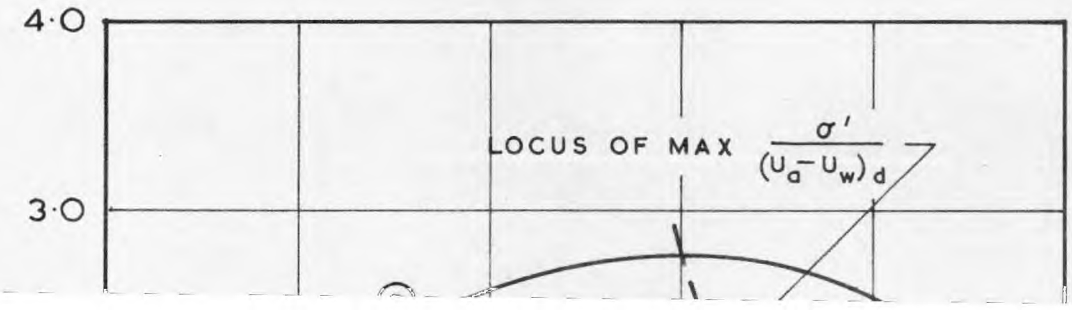
FIG 2.7 shows six modified moisture characteristic curves covering a wide range and their χ-S plots calculated arithmetically from eqn. (2.45). The narrow band of curves justifies the adoption of a unique



(a) TYPICAL & IDEALISED MODIFIED MOISTURE CHARACTERISTIC CURVES.



(b) $\chi - S_r$ RELATIONSHIPS



χ-S relationship regardless of the actual S vs. $(u_a - u_w)$ plot. Equation (2.53) gives values of χ to within $\pm$ 0.04 over the range 20% = S = 100%. For greater accuracy a parabola was fitted as a mean of the six χ-S curves and yielded eqn. (2.54), which may be taken as the final statement of the unique χ-S relationship. This relationship is only valid for relatively incompressible soils draining from an initial S_r = 100%. Fig. 2.7 also shows $\frac{\sigma^1}{(u_a - u_w)_d}$ versus S plots for curves (2), (4) and (5). The shapes of these curves illustrate the general conclusions of section (2.6.2).

(2.6.4) General conclusions on χ.

The parameter χ has been shown to be a convenient variable for expressing the effective stress law for partly saturated soils. For the special case dealt with above its empirical nature was replaced by quantitative theory. It cannot be expected that the unique relationship deduced will hold good for all partly saturated soils, regardless of the mechanisms by which they have reached a given degree of pore space saturation. Consideration of the effect of entrapped air bubbles is sufficient to clarify this point. A partly saturated soil may contain any number of air bubbles, all of equal size at equilibrium. Any wavy plane through the soil mass may be made to pass around the bubbles so that, if all the air is present in this form, the soil is effectively saturated for effective stress calculations, i.e. χ = 1 independent of S. In compacted soil some air will be present as isolated bubbles and some as interconnected voids. The estimation of χ for such a system is difficult.

No mention has been made of the possible differences between volume change and shear strength effective stress laws. Croney and Coleman offered two parameters β and β', with the suggestion that experiment might differentiate between them. In a qualitative way it may be argued that intergranular stresses caused between grains by isolated water lenses will be normal stresses only, giving rise to increased shear resistance but contributing very little to volume change. Hence we could expect two parameters χ_v and χ_{τ}, with $\chi_v < \chi_{\tau}$, at least for low values of S_r.

Before the 'generalised' effective stress equation (2.21) can be applied, experimental confirmation of its correctness is necessary. The remainder of this thesis will be devoted to details and analysis of techniques and tests necessary for investigating the conclusions of this chapter on the behaviour of partly saturated soils and saturated soils with negative pore water pressures.

CHAPTER 3

APPARATUS DEVELOPMENT

3.1. Introduction

The value of any set of data is minimised if the techniques used in compiling it are questionable or unreliable. Triaxial compression testing procedures for saturated soils are now well established and results involving the measurement of positive pore water pressures may be used with confidence, if careful consideration has been given to testing rates. However, when the pore water pressures are below atmospheric experimental difficulties increase and it is now apparent that in the past a considerable volume of testing, particularly on compacted soils, has been done with insufficient technique to ensure accurate pore pressure measurement, the errors being serious when the difference between air and water pressures in the pores is large. A comprehensive survey of apparatus and techniques used in triaxial testing has been given by Bishop and Henkel (1957).

Soil science literature contains a large body of information on the measurement and control of suction in soil, usually unstressed externally, and the techniques established there are readily adaptable to triaxial testing. The apparatus described by Bishop and Henkel has been modified, and where desirable replaced, to permit the measurement and control of soil suction during strength and volume change tests. Any new apparatus has been designed to operate with the ancillary equipment described by Bishop and Henkel as the testing techniques are primarily those of soil mechanics. However, in view of the opening statement of this section, the writer feels that a rather detailed treatment of techniques is justified as the methods are still moderately unfamiliar in soil mechanics.

(3.2) <u>The fine porous disc triaxial cell base.</u>

The air in the pore space of a partly saturated sample is at a higher pressure than the water due to surface tension. To transmit the water stresses to the measuring system correctly the following somewhat conflicting properties are required of the porous medium -

(1) It must have an air entry value in excess of the difference between the air and water pressures in the soil at its surface.

(2) Its permeability should be high for rapid reading and, in some tests, ease of drainage of water from the sample.

(3) It must have sufficient mechanical strength to support the loaded sample and resist any stress systems to which it might be reasonably subjected accidentally (e.g. unbalanced back pressure).

A suitable material has been found in "Aerox" ceramic discs, Celoton Grade VI, some properties of which are presented in Table 3.I.

TABLE 3.I

Porosity	46%
Air entry value	34 lb/in^2 (short term) approx 20 lb/in^2 (long term)
Permeability	$k = 3 \times 10^{-6}$ cm/sec.

Blight (1961) has made ceramic discs with air entry values of over 60 lb/in^2 and this figure seems to improve with time, possibly because of gradual clogging.

Bishop (1960) has described triaxial cell bases fitted with fine porous discs for both 1½" dia. and 4" dia. samples. The 1½" dia. bases were used throughout this project and details are shown in Figs. 3.3 and 3.6.

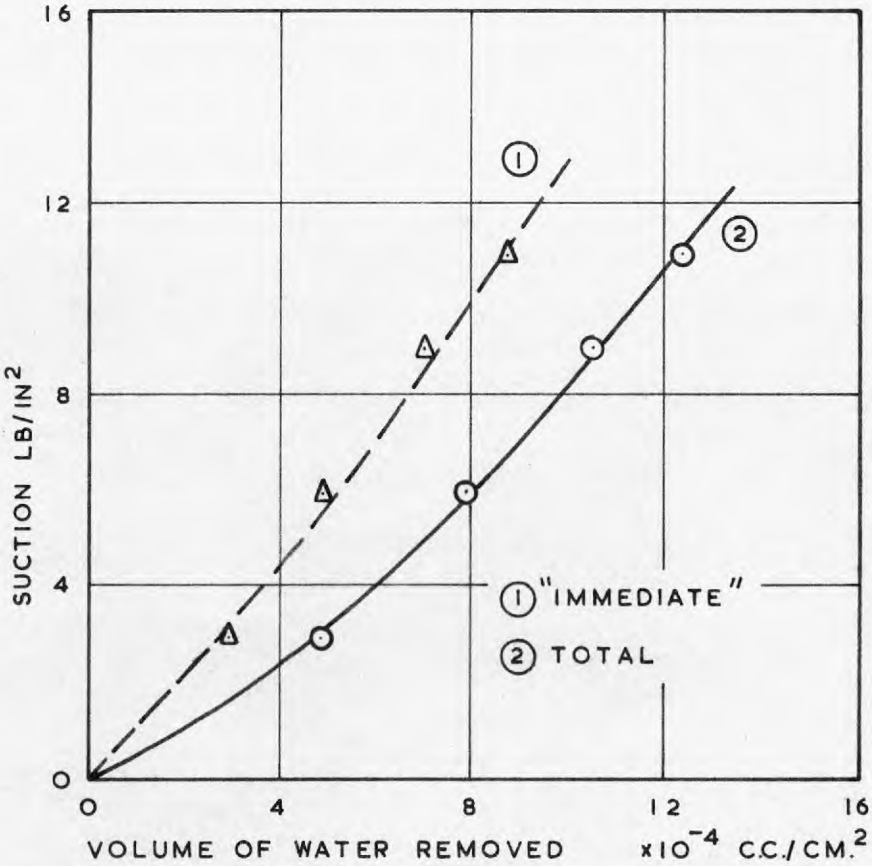
The disc is sealed into a recess machined in the cell pedestal with "Twinbond", an Epoxy resin adhesive. This effected a leak proof joint but after some weeks' use the Twinbond had swollen slightly above the surface of the stone and had to be trimmed back. Maintenance was required several times during the early life of each base, but the swelling ensured continued effectiveness of the seal. The assembly could support transient excess back pressures of at least 30 lb/in^2.

Ideally, no water should drain from the porous stone at the values of suction being measured if time lag in the measuring device is to be avoided. Fig. 3.1 shows the extent of the drainage from one of the stones used, calculated from the movement of the meniscus in the capillary section of a null indicator (Fig. 3.2) attached to the cell base, the top of the stone being wiped free from water at zero applied suction. More than half the volume change was immediate and could be partly accounted for by bad contact between the underside of the porous disc and the base pedestal. Drainage was complete in approximately three minutes. Less drainage would be expected with a soil sample in contact with the stone.

Air can only enter a successful fine porous base by diffusion through the water in the porous disc. In certain circumstances, this does happen and the recess shown in the underside of the stone in Fig. 3.3 permits a small volume of free air to collect without completely breaking water continuity in the apparatus.

(3.3) 'Perspex' Null Indicator

For the measurement of pore pressures in undrained tests a null indicator of the type described by Bishop and Henkel (1957) is used. With negative pore pressures it is desirable to modify the apparatus to minimise the volume of water between the porous stone and the reference meniscus and to eliminate all unnecessary joints as possible sources of leakage. A simple indicator made from a single block of



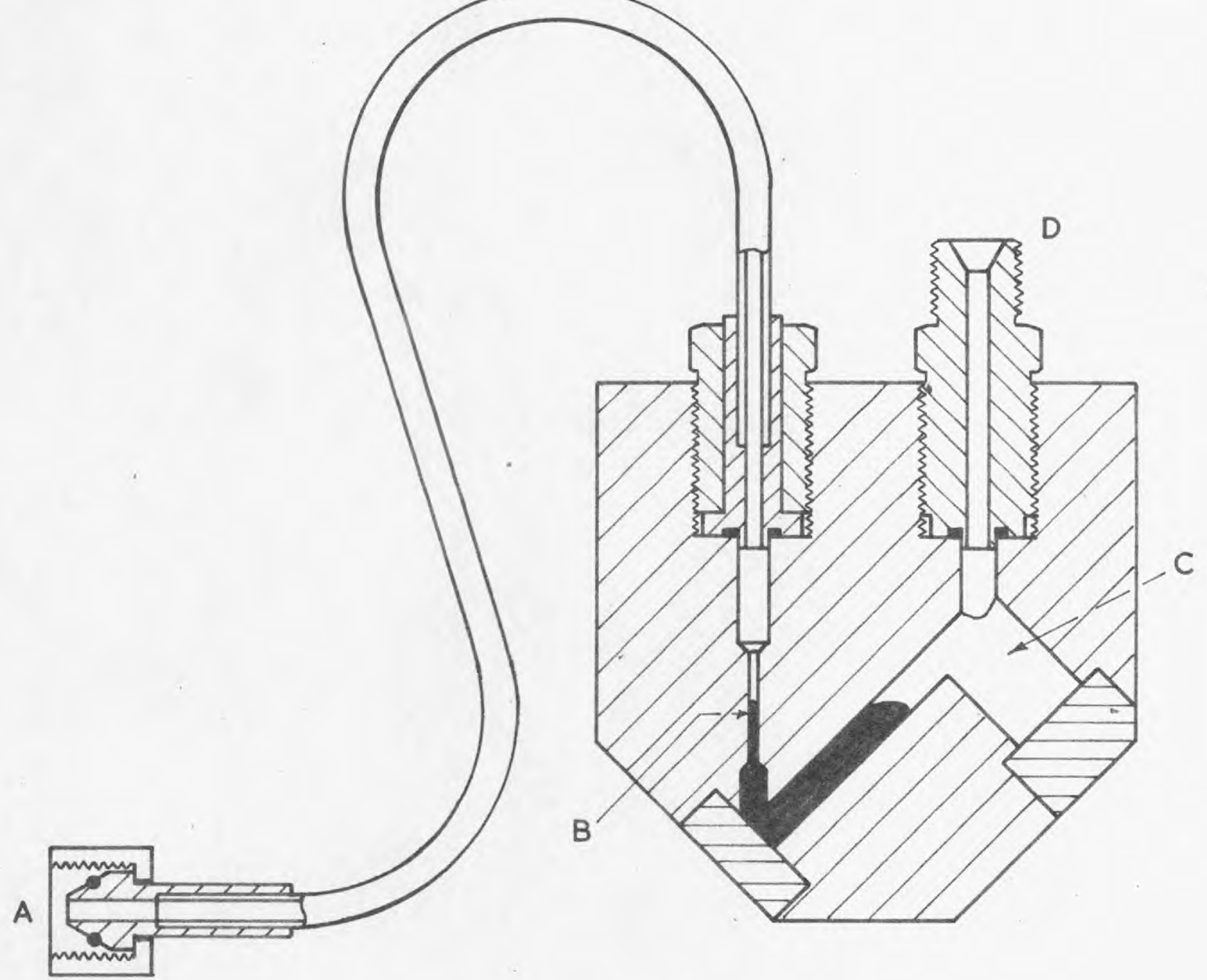
DRAINAGE PROPERTIES OF FINE POROUS CERAMIC.

FIG. 3.1

'Perspex' and designed by Dr. A.W. Bishop is shown in Fig. 3.2. The cone fitting, A, joins directly to the cell base through an "O" ring seal. The reference meniscus in the capillary bore, B, is observed through a low power telescope and, to facilitate flushing, the apparatus can be tilted to run the mercury into the chamber C. Water may then be freely passed through in either direction. When filled with the correct amount of mercury the indicator imposes no head correction on the applied pressure. Connexion to the source of variable badk pressure and measuring gauges is made at D through an ON-OFF zero displacement piston value (Bishop and Henkel).

(3.4) Flushing Procedure

It is essential to remove all free air from the porous stone, cell base and capillary side of the null indicator or whatever extra apparatus is joined to the pore pressure outlet. The same flushing procedure was used for all cases and will be described as for the null indicator. The triaxial cell was connected to a source of water pressure, usually a screw ram, but occasionally a mercury cylinder constant pressure control, both of which are described in detail in Bishop and Henkel (1957). The null indicator was connected to another screw controlled ram (or mercury cylinder control) connecting with a manometer and pressure gauge. The cell was completely filled with de-aired distilled water and the pressure increased to 100 lb/in^2.



'PERSPEX' NULL INDICATOR

FIG. 3.2

The null indicator was tilted to run the mercury into its reservoir and water was permitted to flow through the porous disc until the back pressure had also built up to 100 lb/in^2. This back pressure was reduced several times with the aid of the screw ram to draw more fresh de-aired water into the apparatus, the cell pressure being adjusted continuously to 100 lb/in^2. Finally, the apparatus was left to stand overnight under pressure for any remaining air to dissolve. Next morning a little more water was drawn through under a small gradient and then cell pressure and back pressure were reduced simultaneously to zero. When the mercury cylinder pressure controls were used one maintained the cell pressure at 100 lb/in^2 and the other controlled the back pressure at, say, 98 lb/in^2 so that fresh water was drawn through for some hours at a pressure sufficient to dissolve all the air.

After emptying the cell, the mercury in the null indicator was tipped bac k into the sloping section of the bore and a small back pressure employed to bring the meniscus to a suitable level in the capillary section. The indicator was then isolated by means of the valve at D (Fig. 3.2)

The success of the flushing was tested by wiping the surface of the porous disc free from water, setting a very small back suction on the indicator with the screw ram and opening the isolating valve. The suction was then increased to about 12 lb/in^2 and the movement of the mercury meniscus observed. In a well de-aired base the movement was less than 2 mm.

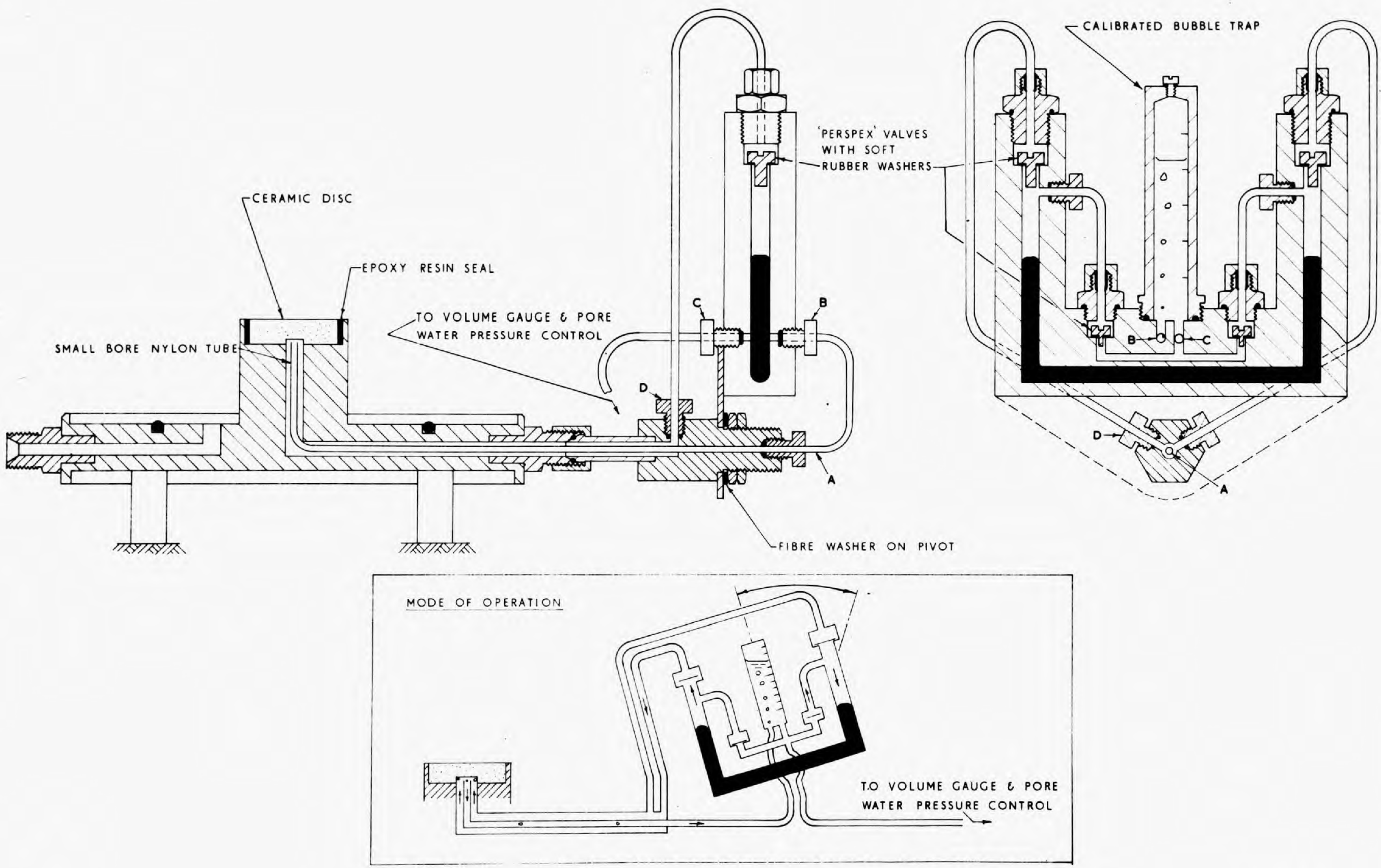
When measuring suctions greater than about 9 lb/in^2 it is desirable to reflush the apparatus after every test of one or more days duration. When not in use the porous stone was kept submerged to prevent evaporation and subsequent cavitation in the cell base water. If some free air has collected in the apparatus its removal is greatly facilitated in the early stages of the flushing process by applying a back suction of 12 lb/in^2 through the screw ram before allowing the back pressure to build up.

(3.5) Bubble Pump

Drained tests with controlled negative pore water pressures were carried out by applying a constant suction to the water in the cell base. During the first tests it became apparent that air was diffusing through the water in the porous disc from the sample and coming out of solution under the influence of the reduced pressure. This made it difficult to measure accurately the volume of water drained from the sample. The presence of a large air bubble immediately under the porous stone also lowered its efficiency as a water transmitter, particularly for flow of water from the base back into the sample. In tests where initial drainage is to be followed by undrained compression, it is essential to remove all free air from the system for the null indicator to operate accurately.

To remove the air from the cell base and allow its volume to be continuously measured during test, without significantly disturbing the suction in the water, the closed circuit bubble pump and air trap, shown fitted to a cell base in Fig. 3.3 was designed in conjunction with Dr. Bishop.

This uses the pressure difference in a tilted U tube containing mercury to circulate the water in the system. The U tube is built up from three 'Perspex' blocks cemented together after machining and the complete assembly pivots on a brass connector fitted to the pore pressure outlet from the cell base. Nylon tubing is used for the flexible connexions, joints being made with O ring seals or standard "Ermeto" pipe fittings. Four 'Perspex' valves with soft rubber washers enable the mercury to act as a piston in a double acting pump, when the whole assembly is rocked back and forth. Any air bubbles under the stone or in the base leads are collected by the flow and removed to the calibrated 'Perspex' bubble trap via a $1/_8$" O.D. Nylon tube inserted up the pore pressure lead until it contacts the underside of the porous disc. With this diameter tubing in the $3/_{16}$" dia. pore pressure lead there is no tendency for bubbles to stick in the horizontal section and gentle pumping for half a minute is sufficient to remove all the free air. The pressure at the top surface of the porous disc was found, experimentally, to be raised temporarily by about 0.15 lb/in^2 by this process. The action



$1\frac{1}{2}''$ DIA. CELL BASE AND BUBBLE PUMP

FIG. 3.3

of the pump is indicated diagrammatically in the inset figure.

The bubble trap has a 5 c.c. capacity and the true volume of water drained from the sample is found by subtracting the air volume from the total fluid volume removed from the cell base, as measured by the gauge described in the following section. A blow-off valve in the top of the trap is useful for removing the air after, or sometimes during, a test. At the end of a slow test under moderately high suctions (say, $u_w = -10$ lb/in^2) several cubic centimetres of air may have collected. The pump assembly can safely withstand 100 lb/in^2 pressure and therefore the cell base plus pump can be de-aired by the technique already given. In some tests a null indicator was joined to C by a short length of Nylon tube and everything flushed together.

3.6. Volume Gauge

The testing programme required the measurement of the following volumes:

(a) water entering or leaving the soil sample under both positive and negative pressures to permit calculation of the degree of saturation of the sample.

(b) water entering or leaving the triaxial cell under positive pressure - as a measure of sample volume change for degrees of saturation, S_r, less than 100%.

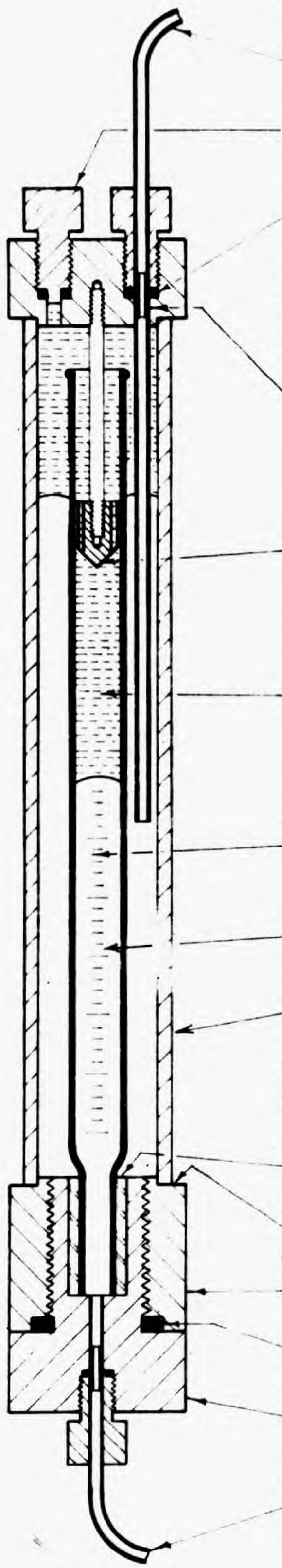
Two suitable devices are described by Bishop and Henkel (1957) but are complicated by the need to compensate for a changing mercury level if pressure changes are to be minimised. A simpler and cheaper apparatus is shown in Fig. 3.4. It was designed with Dr. A.W. Bishop and uses a water-paraffin meniscus as its reference level.

The water to be metered enters at the base of the tube and the pressure or suction control is connected to the top lead. The glass burette is under all-round pressure and its calibration is therefore not pressure dependent. Any volume change in the 'Perspex' tube is made good by the pressure control. Any amount of water may be passed upwards through the gauge without removing the paraffin, which is dyed dark red with "Waxoline Red O.S." (Hopkin and Williams).

The difference (0.2) between the specific gravities of water and paraffin ensures that a volume change using the full length of the burette alters the pressure in the system by less than 0.1 lb/in^2. The burette is treated with Silicone "Repelcote" to prevent sticking of the meniscus and volume changes of 0-25 c.c. can be read to 0.02 c.c.

For use at sub-atmospheric pressures, the gauge is best filled initially with de-aired water and paraffin, although the removal of all traces of air from the top of the tube is not essential for correct operation. Excess air is blown out through the filler plug between tests.

Nylon tubing is used with suctions as it is impermeable to air. However, it is permeable to water vapour (Penman 1960) and provides



VOLUME CHANGE GAUGE

FIG. 3.4

a source of creep in the reading if appreciable lengths of tubing are used. Thus 3'6" of Nylon $1/_8$" O.D. 'Polypenco' tubing was found to lose up to 0.2 cc in one week, depending on the laboratory humidity. Where possible tubes were shortened to the minimum convenient length, otherwise a correction to the gauge reading was made, based on the average loss measured over one month. When working with positive pressures only, polythene tubing is preferable as it is practically impermeable to water (Penman) and the end fittings are simpler to make.

(3.7). Pressure Controls

(a) Water Pressure

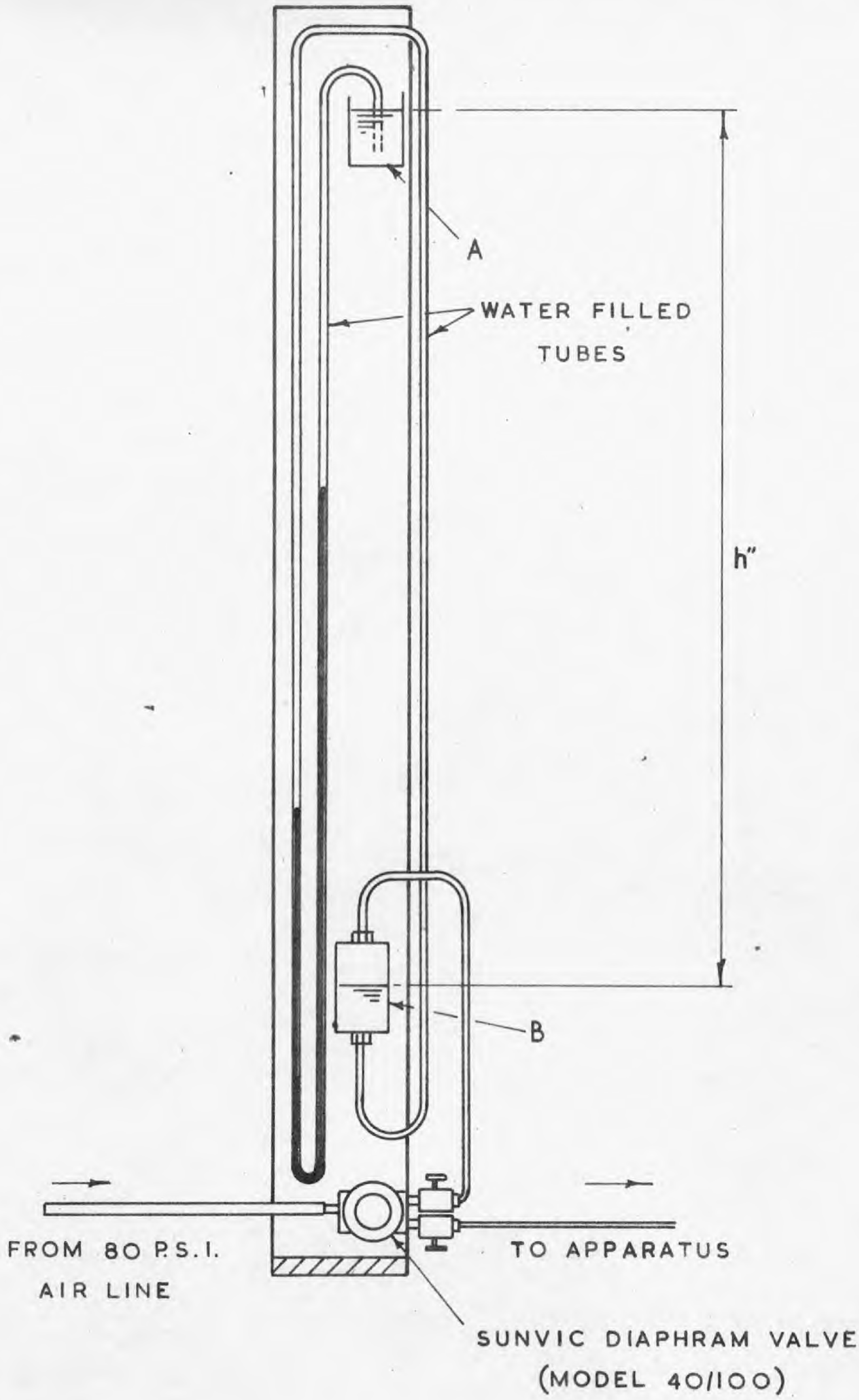
A liquid filled pressure system used in conjunction with the self-compensating mercury control (Bishop and Henkel 1957) provides a convenient way of accurately maintaining pressures for long periods without the problems of safety and dissolved gas inherent in a controlled air pressure system. With very minor modifications the system described by Bishop and Henkel can be used to control suctions in the 0-1 atmosphere range.

The P.V.C. tubing joined to the bottom of the lower mercury cylinder (in the positive pressure system) is permeable to air and allows leakage under suction. The first three feet of this tubing was replaced by $3/_{16}$" O D. 'Polypenco' nylon tubing. The lower cylinder was raised to bench level so that the movable cylinder could be set lower, inducing a sub-atmospheric pressure in the system. Rubber O ring seals were used at all joints subjected to water suctions. The arrangement is illustrated diagrammatically on the right hand side of Fig.4.2. At suctions greater than 10 lb/in^2 some air may come out of solution in cylinder 2b, but the effect on the controlled suction is negligible.

(b) Air Pressure.

For suctions greater than about 10 lb/in^2 it is more convenient to use an elevated air pressure around or in the pores of the sample. The apparatus used for control and measurement of pore pressure is shown in Fig. 3.5. Basically, it is a sensitive diaphragm reduction valve, but the diagram has been included to demonstrate the method of manometer reading which differs from that described by Bishop and Henkel.

Both sides of the mercury-glass manometer are water filled and connect with 'Perspex' cylinders A and B. These act as essentially constant level controls, and the height, h inches, of A above B, considered as a head of water, is chosen to give a round figure of pressure, in this case 1.25 lb/in^2. No scale is attached to the manometer itself, but one portable scale is used for any number of such instruments. This scale is calibrated to transfer inches of mercury head into lb/in^2 pressure, using a buoyant density (in water) of 12.55 gm/cm^3. The pressure is then read directly from this



AIR PRESSURE CONTROL

FIG. 3.5

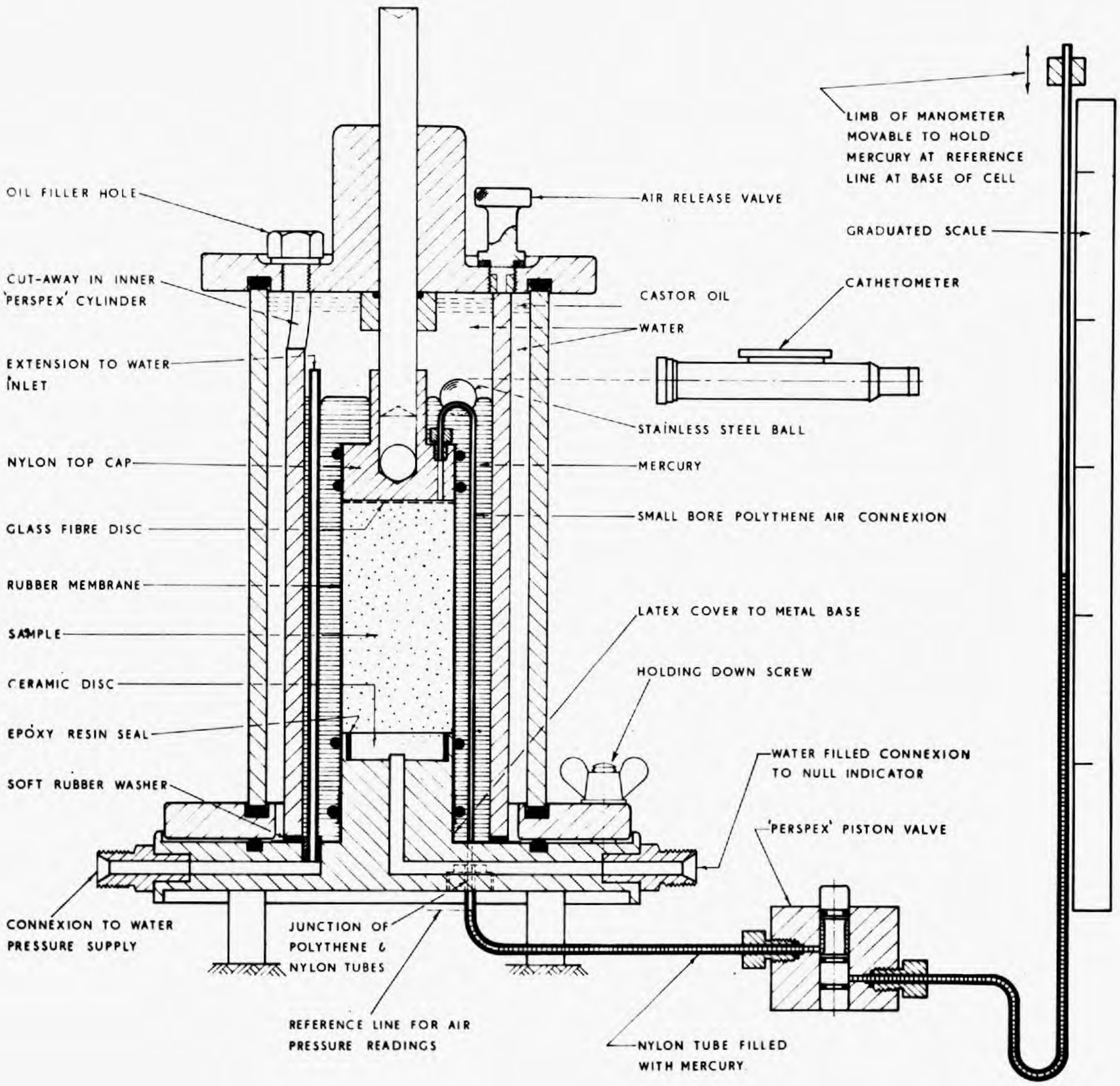
scale by holding the zero level with the lower limb of mercury and the constant correction of 1.25 lb/in^2 is added. The technique obviates calibration of every manometer and ensures accurate readings to better than 0.1 lb/in^2. For air pressures beyond manometer range the line marked "to apparatus" is tapped and connected to any of the 0-150 lb/in^2 pressure gauges in the system (See Fig. 4.2).

(3.8.) The mercury cell

Transfer of air between the pore space of the soil and the water in the triaxial cell by diffusion through the rubber membrane takes place at an appreciable rate, unless the cell water is air saturated at the appropriate pressure. A truly 'undrained' test lasting more than a few hours is thus impossible with partly saturated soil in the usual triaxial apparatus (Blight 1960, Bishop et al, 1960). This phenomenon also undoubtedly accounts for the low initial air pressures in compacted soils (-5 to -6 lb/in^2) reported by Alpan (1960).

To eliminate this error the triaxial cell for 1½" dia. samples has been modified to surround the sample completely with mercury, in which air has negligible solubility. (Fig. 3.6). All parts contacting the mercury are made of Nylon, Polythene or Perspex or else protected by a latex sheet.

The main feature of the cell is the addition of a castellated



LIMB OF MANOMETER MOVABLE TO HOLD MERCURY AT REFERENCE LINE AT BASE OF CELL
OIL FILLER HOLE
AIR RELEASE VALVE
GRADUATED SCALE
CUT-AWAY IN INNER 'PERSPEX' CYLINDER
CASTOR OIL
CATHETOMETER
WATER
EXTENSION TO WATER INLET
STAINLESS STEEL BALL
NYLON TOP CAP
MERCURY
GLASS FIBRE DISC
SMALL BORE POLYTHENE AIR CONNEXION
RUBBER MEMBRANE
LATEX COVER TO METAL BASE
SAMPLE
CERAMIC DISC
HOLDING DOWN SCREW
EPOXY RESIN SEAL
WATER FILLED CONNEXION TO NULL INDICATOR
SOFT RUBBER WASHER
'PERSPEX' PISTON VALVE
CONNEXION TO WATER PRESSURE SUPPLY
JUNCTION OF POLYTHENE & NYLON TUBES
REFERENCE LINE FOR AIR PRESSURE READINGS
NYLON TUBE FILLED WITH MERCURY

Fig. 3.6

inner 'Perspex' cylinder sealed by a rubber washer to the cell base by pressure from the upper part of the cell as it is screwed down. Pore water pressure is measured through the cell base and pore air pressure is measured at the top of the sample. A coarse glass fibre disc transmits this air pressure to a small bore polythene tube connecting the sample cap with an adjustable nylon manometer via a gland in the cell base and a 'Perspex' valve. The datum line for air pressure readings is just beneath the cell base and at no point is the permeable polythene tubing exposed to water. The volume of the fine polythene tube introduces a slight lag into air pressure measurements and a small departure from undrained conditions.

Another polythene tube is used to raise the point of entry of the water supply to the cell above the maximum mercury level. A latex annulus protects the top of the cell base and the lower half of the sample pedestal from mercury attack. It is perforated to fit neatly over the two polythene tubes.

Tests have been carried out to examine the drop in air pressure due to diffusion using a rigid porous ceramic cylinder in place of a sample. The results given in Table 3.2 indicate the importance of this effect in the absence of a mercury seal. (The figures for loss through the mercury are too high as a small leak at the polythene-nylon tube junction was discovered after this test).

Measurements of volume change for partly saturated soils are usually made by gauging the flow of water to or from the cell. Accurate measurements depend upon complete air-free filling of the cell, allowance for oil leakage past the ram, and possession of a reliable calibration of cell volume change with pressure. For compacted soils however the cell volume change may be several times the sample volume change and the probable error is of the same order of magnitude as the variable being measured.

The inner "Perspex" cylinder does not change in volume with change of cell pressure, hence the variation in mercury level in this cylinder is a measure of sample volume change. A plastic or stainless steel ball is floated on the mercury and observed with an accurate cathetometer. A line scribed on the sample cap allows its movement to be followed also and, during shear tests when the cell is moving, readings are taken on a datum line on the cell body. With the dimensions of the current apparatus if

h_1 = height of ball above any datum (cm)

h_2 = " " sample cap above same datum (cm)

h_3 = " " cell " " " (cm)

then sample volume change, dV (cc) is given by:

$$dV = 29.0\,(\Delta h_1 - \Delta h_3) + 2.85\,(\Delta h_2 - \Delta h_3) \qquad \text{......} \; (3.3)$$

The levels can usually be read to $\pm$ 0.002 cm. which corresponds to an accuracy of better than 0.1 cubic centimetres.

Measurements of pore air and water pressures made in the mercury cell represent a close approach to a true undrained condition. However, there is an extra 0.3 cc of air in the air leads and sample cap, and 7 cc of water in the porous disc and null indicator leads which, for some purposes, will act as complementary to the pore fluids.

The lateral stress gradient down the sample is only important for soft soils and low cell pressures. The mean pressure exerted on the sample by the mercury is 1.2 lb/in^2. The range of the apparatus is limited to a maximum suction of about 10 lb/in^2 in slow tests as air diffusion through the water in the porous disc vitiates the null indicator technique and artificially raised air pressures cannot be used in undrained tests.

(3.9) General Comments

The advantages and disadvantages of nylon tubing have been mentioned in the text. It is quite satisfactory for general plumbing where suctions will be encountered but, if a length is sealed off from a source of constant pressure for several days, evaporation may be sufficient to cause cavitation in the tube.

Finally, any new 'Perspex' apparatus involved in supposedly constant volume systems should be wetted a month before use as this material was found to absorb from 1-2% of moisture by weight, most of it in the first few weeks.

TABLE 3.2

Effect on pore air pressure of diffusion through the rubber membrane

(Sample:- Hollow ceramic cylinder $\frac{\text{air volume}}{\text{total volume}} = 77\%$)

Test Condition	Initial Cell Pressure (absolute)	Initial Air Pressure (absolute)	Pressure drop after									
			2 hours		12 hours		24 hours		7 days		14 days	
	lb/in^2	lb/in^2	lb/in^2	%	lb/in^2	%	lb/in^2	%	lb/in^2	%	lb/in^2	%
Rubber membrane surrounded by de-aired water	52.7	43.1	0.4	0.9	2.2	5.1	4.05	9.4	12.4	28.8	13.8	32
Rubber membrane surrounded by mercury	39.7	29.7	0.1	0.3	0.1	0.3	0.1	0.3	0.35	1.2	-	-

CHAPTER 4

EXPERIMENTAL TECHNIQUES

(4.1) Introduction.

This section includes details of most of the types of test used during the project. In later sections where test results and their significance are being discussed an excess of detail would interfere with the course of the argument and it is probably better to keep all such necessary but distracting information in one place for easy reference.

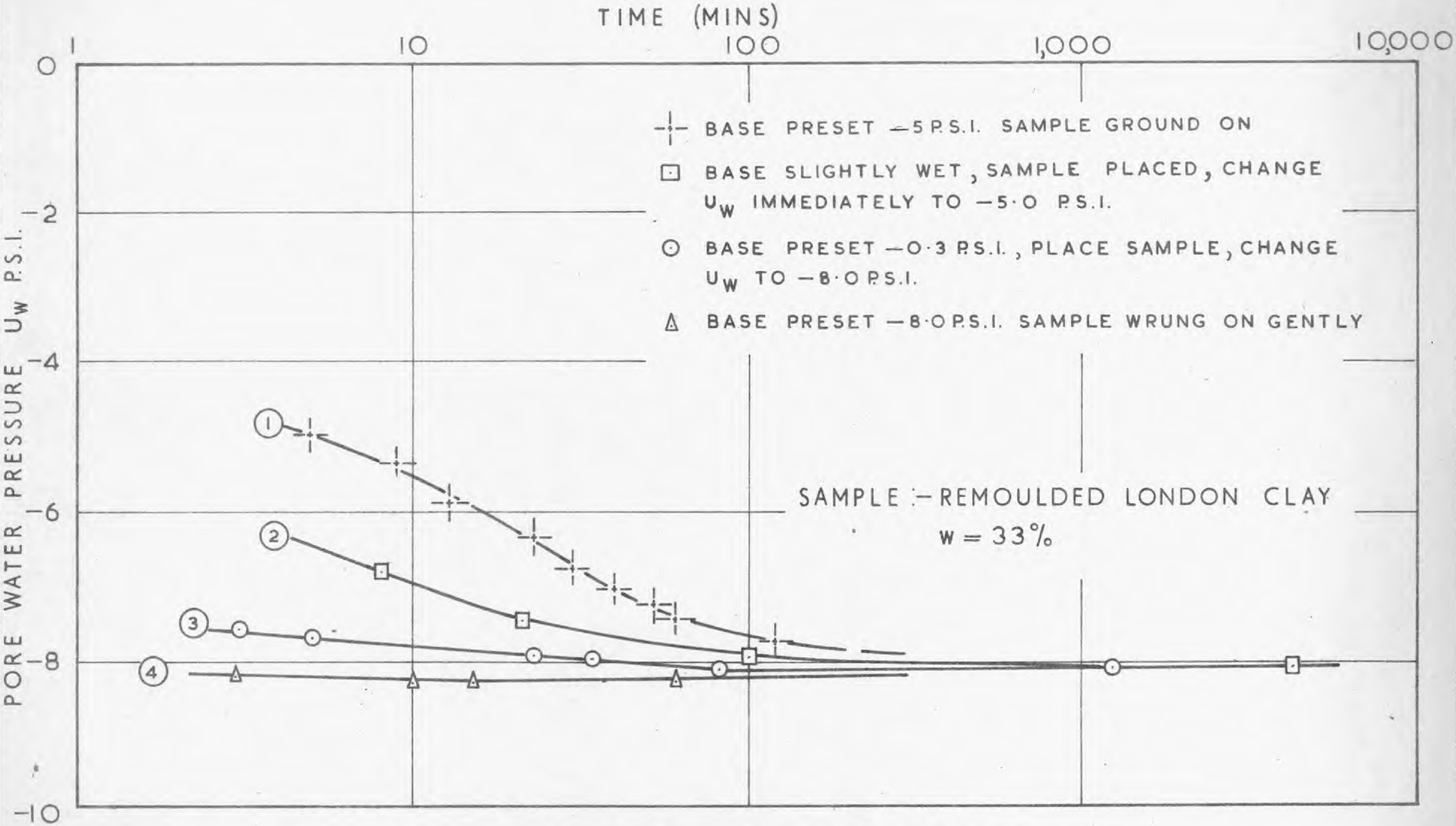
(4.2) The measurement of suction.

Much of the experimentation was concerned with the accurate measurement of pore water pressure in the range 0 to -15 lb/in^2. This proved to be not quite such a simple matter as was at first thought and, at the risk of being pedantic, it will be presented in some detail as it was of fundamental importance to the investigation. The use of no flow methods in conjunction with suction plate apparatus has been described in some detail by Croney, Coleman and Bridge (1952), but their method is fairly slow and requires flow of a definite quantity of water to or from the soil. This can result in incorrect measurement, particularly in heavy clay soils.

In principle the technique used at Imperial College consisted of placing a sample on the fine porous stone of a well de-aired base and null indicator and observing the movement of the mercury meniscus with a low power telescope. The slightest tendency of the meniscus to move from the telescope cross hair was corrected by adjusting the back pressure (negative for unconfined samples) to the null indicator until movement ceased. With saturated clays if the back pressure was within 0.1 lb/in^2 of the true pore pressure an observable creep was still obtained in one or two minutes. Compacted soils take smewhat longer as it is difficult to establish effective contact without loading the sample.

However, as shown in Section 3.2., the fine porous stone loses a small amount of water by drainage at suctions in the 0-1 atmosphere range and the act of placing a sample may temporarily disturb the pore pressure at its base. The effect of various placing techniques on the response time of the apparatus is illustrated by Fig. 4.1. The only technique giving sensibly immediate response is that in which the stone is allowed to equilibrate at approximately the correct suction for two to three minutes before the sample is very gently, but firmly, slid on.

If the initial suction estimate is very incorrect, the back pressure is set at a new estimate, one or two minutes allowed for extra stone drainage, and the creep of the meniscus again observed. Readings converge quite rapidly on to the true value. For accurate



RESPONSE TIME OF POROUS STONE

FIG. 4.1

work in the low stress range the following points are relevant to the apparatus as used at Imperial College:

(a) Each reading is rapid enough to avoid temperature fluctuations.

(b) During compression tests the null indicator rises with the loading platform and makes reading difficult at rapid strain rates.

(c) The back pressure, as read on a manometer, should be checked before and after a reading as there is often a drift subsequent to changing the pressure. This is caused mainly by water sticking temporarily to the glass manometer tube and by creep in the polythene tube connections to the manometer. Immediately after completing a reading the null indicator valve is closed and the back pressure system returned to positive pressure.

(d) Should a small volume of free air come out of solution in the null indicator leads (usually overnight) an approximate suction reading may be rapidly obtained as the meniscus is now extremely sensitive to back pressure variations. However, if the air is immediately under the stone and completely filling the small cavity (Fig. 3.3), then the reading may not be representative of the sample.

(3) Partly saturated soils with low degrees of saturation, and samples with uneven ends give greatly improved pore pressure response when loaded vertically.

(4.3) Measurement of suctions greater than one atmosphere.

The above technique is limited to suctions of about 13 lb/in^2 for quick tests, or 10 lb/in^2 for tests of several days' duration. If the suction is beyond this range but within the blow-through limit of the porous stone the pore water pressure can be elevated into the measurable range by surrounding the soil sample with air under pressure. The justification for this was dealt with in section 2.4.(b). Similar techniques have been used by Hilf (1956) and, by modifying a pressure membrane apparatus, by Croney and Coleman (1954). Blight (1961) has measured suctions of 70 lb/in^2 in compacted soil using this method.

When using 1½" dia. by 3" high clay samples the fine porous stone is allowed to equilibrate at a suction of, say, 10 lb/in^2 and the sample gently wrung on. Five or six 5" lengths of fibre glass yarn are placed longitudinally on the sample to overlap the base pedestal and the sample cap. A thin pre-saturated rubber membrane is then placed around the sample, but not sealed with O rings in the customary fashion.

The cell top is screwed down and air at the desired pressure admitted through the cell pressure fitting. The pore pressure reading is then made as before. A little free water is kept in the cell and the membrane prevents evaporation from the sample while allowing the air pressure to surround the soil rapidly from the fibre glass threads. Response is usually somewhat slower than for samples with suctions less than 12 lb/in^2 but improves markedly upon sample loading. It is desirable to elevate the pore water pressure to a moderate positive value as any air which diffuses through the stone then remains small in volume.

(4.4) Control of suctions greater than one atmosphere.

To investigate suction-water content relationships in the range 0-50 lb/in^2, the cell base may be used as a pressure plate (Richards, 1938). A cell base fitted with a bubble pump is desirable, the bubble pump being supplied with water at atmospheric pressure. The wet soil (at liquid limit say) is formed into a short cylinder and placed in intimate contact with the stone. The cell is assembled and air blown in at the desired pressure. A volume gauge connected to the bubble pump indicates when drainage is complete. The cell is then dismantled and the top quickly slided off the sample for moisture content determination. The cell is then re-assembled and a higher air pressure applied. In this way a range of suctions may be applied to the one sample. A porous disc with an air entry valve of 35 lb/in^2 when exposed, supported 50 lb/in^2 pressure difference when in contact with a clay sample.

(4.5) Tests on remoulded clays ($S \doteq 100\%$).

(a) Sample preparation

An 8 kilogram batch of ground oven-dried London Clay (provided by R.R.L) was mixed with distilled water to a consistency just dry of the liquid limit (68%) and stored in a stainless steel airtight bin to 'mature' for several months. When a sample was required at a specified water content a 4" dia. oedometer was used to consolidate the clay under the pressure associated with that water content on the oedometer consolidation curve.

The consolidated sample was then tied in a close fitting rubber membrane and remoulded by hand without loss of water. Samples were formed by packing the clay in thin layers into a 1½" dia. sample tube against a disc supported by a wooden dolly. After trimming to 3" length and extrusion from the tube samples were measured, weighed and placed on the fine porous stone for suction measurement before shearing.

(b) Compression tests

Most remoulded samples were tested unconfined apart from an unsealed pre-soaked membrane to prevent evaporation losses. In long term tests a little free water was left on the cell base. Drier soils, with remoulded suctions greater than 10-12 lb/in^2, were placed using fibre glass threads and elevated air pressure in the cell. A third group of samples was tested by sealing the rubber membrane to the base pedestal and sample cap with rubber O rings and subjecting the soil to an all-round water pressure.

The measurement of axial strain and compressive load during test followed closely the technique given by Bishop and Henkel (1957). Pore water pressures were measured throughout test with the technique described previously.

The deviator stress $(\sigma_1 - \sigma_3)$ at any axial strain was calculated from

$$(\sigma_1 - \sigma_3) = \frac{P.(1 - \varepsilon)}{A_o} \qquad \text{............ (4.1)}$$

where

P = axial load on sample (from proving ring)

A_o = initial area of cross-section of sample

$\varepsilon = -\dfrac{dl}{l_o}$ = axial strain

l_o = initial sample length.

A correction for strength due to membrane action was made only for the confined tests with sealed membranes. Following Bishop and Hehkel a correction varying linearly with strain from zero to 0.6 lb/in^2 at 15% strain was used.

Tests were continued until maximum deviator stress had been clearly defined. The sample was then removed for water content determination. Invariably these remoulded samples failed along many slip planes and so the sample was simply sliced horizontally into three or four equal parts for moisture content tests.

(c) Long term storage

Some samples were kept for periods of up to a year after remoulding before shearing. To preserve the moisture content but allow easy, disturbance free, removal for testing each sample was placed between two 1½" dia. by ½" thick wax discs and surrounded by a pre-wet membrane. This assembly was then closely wrapped in a sheet of aluminium cooking foil and dipped in a wax - 50% paraffin wax, 50% vaseline. This mixture had been proven to be completely water-tight and the maximum loss of weight suffered by a sample over one year was 0.01 gm. Pre-soaked membranes were used as the normal rubber membrane for 1½" dia. samples absorbs 0.3 - 0.4 c.c. of water in about five days. Waxed samples were kept in a cool dark bin until removed for the types of test described above.

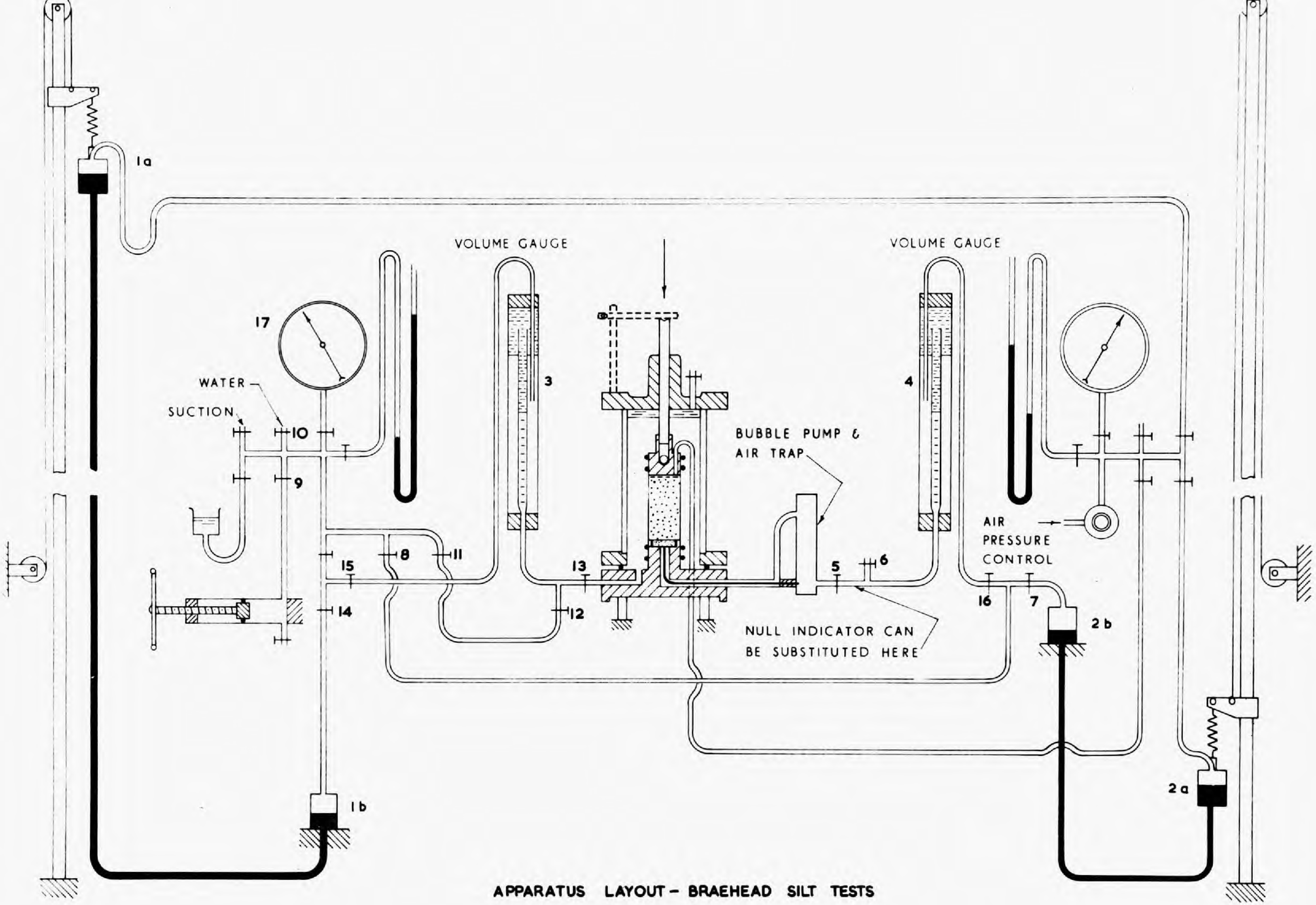
(d) Normally consolidated and overconsolidated undrained compression tests

These tests were performed on London Clay remoulded at an initial water content of 62%. The technique and calculations were precisely those given by Bishop and Henkel (1957).

(4.6) Tests on Braehead Silt

(a) Drained tests with controlled pore air and water pressures.

A diagrammatic layout of the apparatus used for silt tests is given in Fig. 4.2. The arrangement shown is for the most general



APPARATUS LAYOUT - BRAEHEAD SILT TESTS

Fig. 4.2

case of dependent or independent variation of cell pressure σ_3, pore air pressure u_a, and pore water pressure u_w at any stage of a test. Mercury cylinders 1a and 1b control the cell pressure and the 'reversed' system 2a and 2b controls the pore water pressure.

The first series of tests was performed with $u_a = 0$ (i.e. atmospheric) and $(\sigma_3 - u_a) = 2.0$ lb/in^2. This low value was chosen simply to keep the membrane in intimate contact with the sample throughout test for accurate volume change measurements. In such tests the air pressure control and its connexions to cylinders 1a and 2a and the sample cap are not used. The fine polythene lead from the sample cap is then left open to atmosphere through the cell base.

Later tests were done with elevated pore air pressures but still keeping $(\sigma_3 - u_a) = 2.0$ lb/in^2 so that nearly all the strength of the sample was derived from the effect of the suction, $(u_a - u_w)$. The detailed technique will be given for such a test and minor variations summarised at the end of the section.

The fine porous disc, cell base, bubble pump and volume gauge (4) were flushed under pressure as described. A thin (0.01") rubber membrane was clamped to the base with two O rings and held to cylindrical shape by a split vacuum former. A little free water was placed on the porous stone and a suction of 1 lb/in^2 applied to the cell base from cylinder 2b.

A known oven dry weight of silt had been mixed with distilled water, boiled under vacuum and allowed to cool. This was now carefully spooned into the membrane until level with the top of the former and the residue dried and weighed to give the total sample dry weight. As the slurry settled excess free water was drawn off the top and the sample was then left overnight to consolidate under the 1 lb/in^2 suction.

Next morning the surface of the sample was gently scratched in a criss-cross pattern with a fine wire. Unless this was done there was a risk of a layer of fines on top of the silt producing an erroneously high air entry value. A coarse filter paper disc and a coarse porous ceramic disc were next gently placed on the sample. Two O rings having been previously passed over it, the sample cap was now placed, the membrane eased up around it from the former, and the O rings snapped into place from a split metal ring. A stainless steel ball was lowered into the water filled guide in the sample cap and an hour or so allowed for further sample drainage. The split former was then gently removed and the sample measured. The meniscus in volume gauge (4) was returned to the bottom of its burette before removing the former by manipulating values (5), (6), (7), (8) and (9) and applying pressure from the screw ram. The volumetric equilibrium of the sample could then be checked before measuring its dimensions.

The sample diameter was measured at three heights (½", 1½", 2½") on each of two directions at right angles with an accurate cathetometer. After allowing for the membrane thickness an average diameter was computed. The height was measured to 0.01" with a steel rule or the cathetometer. Penman (1952) could claim an accuracy of only ± 10% in porosity for 1½" dia. silt samples measured mechanically. The optical method gave reproducible results to within ± 2-3%.

The sample was wet with a wash bottle, particularly under the O rings, and the cell top screwed home with the ram withdrawn almost to the top of its travel. Water was now admitted to the cell, using valves (10), (11), (12) and (13). The preliminary wetting prevented air bubbles from being trapped under the O rings. The cell was filled completely, overflow being allowed to collect in the grooved cell top. Any air bubbles trapped under the cell cap were removed by sucking through a bent, fine bore polythene tube. A quantity of castor oil was poured onto the cell top and introduced into the cell by sucking some of the water out. In this way a completely air-free cell was obtained. The cell ram was gently eased down into the sample cap guide and the oil filler plug tightened in the cell cap.

Mercury cylinders 1a and 1b were set to 2 lb/in^2 and the cell blow-off valve shut. To prevent the ram from being blown out the strain gauge pillar (shown dotted) was used to hold the ram down onto a collar and O ring forced against the top of the ram guide. The cell pressure was applied through valves (13) (14) and (15), the 'immediate' drop in volume gauge (3) being noted.

Readings of volume gauges (3) and (4) were taken until drainage ceased. As the silt remained 100% saturated at this small suction (1 lb/in^2) the two volume changes should have been equal (after gauge (3) had been corrected for the 'immediate' cell volume change). The agreement was usually within 0.1 c.c. unless some air had been entrapped in the sample, in which case gauge (3) was taken as the correct volume decrease.

The air pressure u_a was now increased to its test value, the flexible connexions to the tops of cylinders 1a and 2a ensuring equal changes in σ_3 and u_w. The only observable result of this operation was an increase in the cell volume. One hour was allowed for the cell to creep and the new reading of gauge (3) noted.

The suction $(u_a - u_w)$ was now set to its final test value by lowering cylinder 2a. Further consolidation and drainage of the soil took place and was measured by the gauges. Depending on the actual values of u_a and $(u_a - u_w)$, the pore water pressure could be either positive or negative. If $(u_a - u_w)$ exceeded the air entry valve of the silt greater volume change was recorded by gauge (4) than gauge (3), after subtracting from gauge (4) the volume of air collected by the bubble pump. For high $(u_a - u_w)$ gauge (4) had to be reset during the test. The gauge readings enabled the sample volume and degree of saturation at the end of drainage to be computed. The new dimensions were calculated assuming isotropic consolidation.

The sample was set up for compression in the normal manner, gauge (3) being affected by the entry of the ram into the cell to make contact with the stainless steel ball. During compression the collar and O ring were slid up to the top of the ram, thus permitting oil to leak from the ram bushing. This oil leak was measured regularly by mopping up with weighed blotting paper. The sample volume decrease at any time after commencing compression was given by:

$$dV = dV\,(\text{gauge } 3) + dV_{ram} - dV_{oil\ leak} \quad \ldots\ldots\ldots\ldots\ (4.1)$$

$dV_{ram} = 3.22\ dl$ (cubic centimetres)

dl = axial compression (inches)

$dV_{oil} = 1.03 \times$ weight of oil (castor oil).

Readings were taken regularly of compressive load, axial compression, volume gauges, air trap and oil leakage. The deviator stress, $(\sigma_1 - \sigma_3)$, was calculated, allowing for change in cross sectional area, from the formula

$$(\sigma_1 - \sigma_3) = \frac{P.\,(1 - \varepsilon)}{A_o.\,(1 + \frac{dv}{V_o})} \quad \ldots\ldots\ldots\ldots\ (4.2)$$

where P = axial load on sample

ε = axial strain = $-\frac{dl}{l_o}$

l_o = sample height prior to axial compression

A_o = cross sectional area prior to compression

$v = \frac{dV}{Vo}$ = volumetric strain

dV = sample volume increase during axial compression.

A correction to $(\sigma_1 - \sigma_3)$ was made for membrane action as before. A correction was also made for the work done by sample dilatation, but this will be dealt with in the section on analysis of results. The variation of degree of saturation with axial strain was also calculated.

Once peak deviator stress had been clearly defined, the test was stopped, valve (5) shut and cylinder 2a wound up level with 2b. The air pressure was reduced to zero, followed by the cell pressure, and the sample removed. The bubble trap was checked to see how much water the sample had imbibed during the dismantling process.

The sample was weighed and cut into three or four slices for moisture content determination. The filter paper and porous disc on top of the sample were checked for water absorption. From the final water content and the volume measurements, the degree of saturation as initially placed was calculated. This usually came to between 98% and 100%, thus indicating the measurement of initial dimensions.

For tests with $u_a = 0$ and u_w moderately negative, the technique was almost precisely the same, except that there was no trouble with cell volume change. During one test σ_3, u_a and u_w were varied greatly throughout the test but keeping the differences $(\sigma_3 - u_a)$ and $(u_a - u_w)$ constant. This was done merely by altering u_a very slowly, the connexions to the mercury cylinders ensuring equal changes in σ_3 and u_w. For tests of several days' duration some air always came out of solution, even when σ_3, u_a and u_w were all positive. The greatest volume of air was collected by the pump when u_a was moderately high and u_w appreciably negative.

(b) 'Overdrained' tests

To investigate the effect of drying-wetting hysteresis a series of 'overdrained' tests was performed. The first part of the test was carried out exactly as above, the draining suction being 25 lb/in^2. When drainage was virtually complete $(u_a - u_w)$ was decreased to its test value, keeping σ_3 and u_a at their initial values, and water allowed to re-enter the sample until equilibrium was reached. This generally took 3-4 days. The remainder of the test was performed as above.

The values of $(u_a - u_w)$ were chosen to give roughly equal spacing to the degrees of saturation at failure.

(c) Constant water content tests

(1) $(\sigma_3 - u_a) = 2.0\ lb/in^2$.

For reasons given in section 3.8, it was impossible to carry out a true undrained test with respect to both pore fluids in the normal triaxial cell. This series of tests was therefore undrained only with respect to the pore water; hence the differentiation between undrained and constant water content tests.

The apparatus was the same as that in Fig. 4.2. with the inclusion of a "Perspex" null indicator just behind valve (5). During the setting up and initial consolidation and drainage of the sample the mercury was kept out of the way in its reservoir. In this series the air pressure was kept sufficiently elevated so that u_w was never much below atmospheric pressures. $(\sigma_3 - u_a)$ was kept at 2.0 lb/in^2 throughout so that these tests could be directly compared with the drained series.

Once drainage was complete any air which had collected in the bubble pump was quickly blown out by momentarily releasing the valve in the top of the air trap (If u_w was negative it was made temporarily positive with the screw ram to expel the bubbles). The process occupied but a few seconds, so that the sample remained unaffected. The mercury was then tipped into the sloping bore of the null indicator and allowed to rise into the capillary section by releasing a little more water from the bubble trap (For negative u_w valve (5) was closed and a positive pressure applied to force water out of the null indicator top connexion which was carefully loosened.) The u_w control was isolated through valve (7) and the pore water pressure read, setting the back pressure from the screw ram through valves (8) (9) and (16). Valve (5) was closed after each pore pressure reading.

The compression test was carried out as before with axial load, axial compression, volume gauge (3), oil leakage and pore water pressure recorded during test. The deviator stress was calculated from eqn. 4.2. which is also valid for this case.

(ii) $(\sigma_3 - u_a) = 15.0$ lb/in^2.

A short series of constant water content tests was run at $(\sigma_3 - u_a) = 15.0$ lb/in^2. The procedure was as for the $(\sigma_3 - u_a) =$ 2.0 lb/in^2 test as far as consolidation and drainage under $(\sigma_3 - u_a)$ = 2.0, $(u_a - u_w)$ = desired initial test value. This stage was reached with an elevated air pressure sufficient to keep u_w positive when $(\sigma_3 - u_a)$ was changed to 15 lb/in^2, σ_3 being kept constant to avoid error due to cell volume change. The increase in $(\sigma_3 - u_a)$ was made by slowly reducing the air pressure, u_a, and raising mercury cylinder 1a to keep σ_3 constant, as determined by the pressure gauge (17). $(u_a - u_w)$ of course was unchanged by this process. The remainder of the test was run as for the samples with lower $(\sigma_3 - u_a)$.

(d) <u>Normally consolidated drained tests</u>

These control tests were performed on the saturated silt slurry, consolidation being by single end drainage through a coarse porous stone at the base of the sample. The technique was normal soil mechanics routine as detailed by Bishop and Henkel. From these tests the coefficient of permeability of the silt was also calculated as:

$$k = 3.3 \times 10^{-5} \text{ cm/sec}$$

from $k = \gamma_w . m_v . c_v$

where c_v = coefficient of consolidation

m_v = " of volume decrease

$= \frac{de}{(1+e)dp}$

e = void ratio

p = pressure

(4.7) Mercury Cell Tests

These were performed on Talybont boulder clay for which a considerable amount of data exists from constant water content tests. (Alpan, 1959, Blight 1961). The soil as used for 4" dia. compacted samples contained 38% coarser than No 10 B.S. sieve. This fraction was removed as likely to damage the thin walled 1½" dia. sampling tubes used in the sample preparation.

The soil was compacted in a 4" dia. by 4" high mould using 25 blows of the 5.5 lb. hammer dropped from 12 inches on each of three layers. A 3" x 1½" dia. sample was cut from the centre of the compacted specimen with a thin walled tube, weighed, and accurately measured with a vernier caliper.

The cell base and null indicator having been previously deaired the sample was placed on the porous stone and enveloped by the rubber membrane (a dummy top cap being used). The membrane was placed to overlap the latex protective sheet and both sealed to the pedestal with an O ring, a further ring being added higher up the pedestal. Entrapped air was eased out the top of the membrane which was then folded back from the dummy sample cap. Two O rings were placed over the nylon loading cap, the fibre glass disc centred on top of the sample and the cap sealed into place with the aid of a split metal ring for O ring placing.

Before adding the two Perspex cylinders any air under the latex sheet was displaced by water and the re-entrant space under the O rings wetted with a wash bottle. After assembling the two cells and the ball float the space immediately around the sample was half filled with water and mercury run in through the oil filler hole using a long stemmed funnel. In this way entrapped air was almost completely eliminated. Sufficient mercury was added to cover the air pressure lead, the remainder of the cell space being filled with water and a thin layer of castor oil.

The cathetometer was set up on a rigid table and levelled accurately so that any drifting of the ball float would not alter its reading.

(a) <u>All round compression tests</u>

After the mercury had been added ten minutes were allowed to elapse before taking any cathetometer readings so that water sticking to the inside of the inner cell could rise to the surface of the mercury, assisted by gentle tapping of the cell.

Readings were then taken on the floating ball, the sample cap and a suitable fixed datum not on the triaxial cell. This avoided any errors due to cell deformation on applying a pressure increment. The pore water and pore air pressures and the barometric pressure were recorded. Readings were taken at regular intervals until equilibrium was reached, usually in less than two days.

The air release valve in the cell top was closed and the pressure increased by a suitable increment using a screw ram, finally connecting to a mercury pot pressure control. Levels and pressures were again read until equilibrium was indicated, the process being repeated until maximum desired cell pressure had been reached.

The cell pressure was released and the mercury and water siphoned off through the oil filler hole before dismantling the cell and removing the sample for moisture content determination.

Sample volume changes were calculated from the cathetometer readings using eqn. 3.1.

$$\Delta V = 29 (\Delta h_1 - \Delta h_3) + 2.85 (\Delta h_2 - \Delta h_3)$$

The pore air and water pressures were corrected for changes in barometric pressure, a drop in atmospheric pressure of 1 inch of mercury reducing u_a and u_w by 0.5 lb/in^2. As the initial readings had been taken with a mean pressure of 1.2 lb/in^2 on the sample, equilibrium values for the unconfined sample were obtained by extrapolation of the data to zero applied pressure.

(b) True undrained compression tests

The compacted sample was set up as above and the desired cell pressure applied, volume change and pore pressures being measured until equilibrium was reached. The cell was now set up in the loading frame and prepared for compression in the normal manner. As soon as the ram contacted the sample the levels of the cap and ball were re-read, together with a new datum scribed on the cell top. This datum, which was necessary to allow for the rise of the loading platform during compression, was not used earlier as the cell changed height slightly when the cell pressure was applied.

During compression (strain rate 0.0029% per minute) readings were taken regularly of the proving ring, axial compression, barometric pressure, u_a, u_w, and levels of ball, sample cap, and datum. If the sample cap showed any tendency to tilt during test the difference $(\Delta h_2 - \Delta h_3)$ was checked against the axial compression of the sample as measured by a dial gauge mounted on the bottom of the proving ring. A slight difference was expected at high axial loads due to compression of the nylon immediately under the stainless steel ball in the sample cap.

Deviator stress at any strain was again calculated from

$$(\sigma_1 - \sigma_3) = \frac{P.\ (1 - \varepsilon)}{A_o(1 + \frac{dv}{Vo})} \qquad \text{.........} \quad (4.2)$$

Measured air and water pressures were corrected for the barometric pressure prevailing at the start of the test. Volume changes were calculated from

$$dV = 29\,(\Delta h_1 - \Delta h_3) + 2.85\,(\Delta h_2 - \Delta h_3) \qquad \ldots\ldots\ (3.1.)$$

and, from the final measured water content, the change in degree of saturation S_r with axial strain $\mathcal{E}$ was computed.

During an early test with high axial load the fibre glass disc on the sample top was compressed over the end of the air lead, cutting it off from the air pressure changes in the sample. A later test was carried out with a small cross grooved out of the sample (to a depth of $^1/16''$) and filled with layers of fine and coarse sand, the sample cap being placed with the air lead directly over the sand.

This test also gave slow u_a response with a large axial load on the sample as did a further one with a fibre glass top disc with two tongues descending ½" down the side of the sample. For samples near the optimum water content sheared under moderately high lateral stresses end measurement of pore air pressure does not appear to be satisfactory at the strain rates used, which were much slower than normal testing rates. The significance of the errors in u_a is dealt with in Section 6.5 (3).

CHAPTER 5

EXPERIMENTAL RESULTS AND ANALYSIS - SATURATED SOILS

(5.1) Introduction

Many of the principles governing the effects of negative pore water pressures on soil behaviour may be demonstrated quite adequately on saturated soils. These principles may also be used to throw some light on aspects of soil macro behaviour which have previously been treated on a strictly empirical basis. Partly saturated soils introduce further complications into analysis which possibly confuse rather than clarify the points at issue and this chapter will therefore be confined to tests on soils with 100% pore space saturation. A few tests on saturated samples which relate directly to the partly saturated tests as controls will be held over until the following chapter.

(5.2) Description of Soils

The main properties of the soils used are summarised in TABLE 5.1.

SOIL	L.L.%	P.L.%	P.I.	% clay fraction
LONDON CLAY (R.R.L.)	66	27	39	55
LONDON CLAY (WALTON)	69	25	44	-
PEATY CLAY (LAGOS)	170	60	110	-
BRADWELL CLAY	92	35	57	53

TABLE 5.1. BASIC SOIL PROPERTIES

(5.3) Pore Pressure phenomena

(a) In section (2.4)b it was shown that for the general case of a partly saturated soil with specific values of σ, u_a and u_w the important pressures controlling soil behaviour should be the differences ($\sigma - u_a$) and ($u_w - u_a$) and hence if the absolute value of u_a is regarded as the convenient datum for stress measurements, the measured σ and u_w will automatically give these differences.

This principle, called the 'Translation of Origin' by Hilf (1956), was the basis of the method used for measuring suctions greater than one atmosphere and of course is also the basis for operation of the pressure membrane apparatus.

Experimental verification of the principle for two unconfined saturated clays - one natural and one fully remoulded - is given in FIG. 5.1. The Bradwell clay had an initial pore water pressure of - 29 lb/in^2, but the invariance of ($u_w - u_a$) for values of u_w in the directly measurable range enabled the extrapolation to atmospheric ambient air pressure to be made with confidence. The remoulded London clay had measurable pore water pressures at all stages of the test.

(b) The pore pressure parameter B

The effect of applied mechanical load on an initial negative pore water pressure is shown in Fig. 5.2. The parameter B is very nearly 1.0, which is its established value for saturated soils of all types with positive pore water pressures. Hence in the one atmosphere

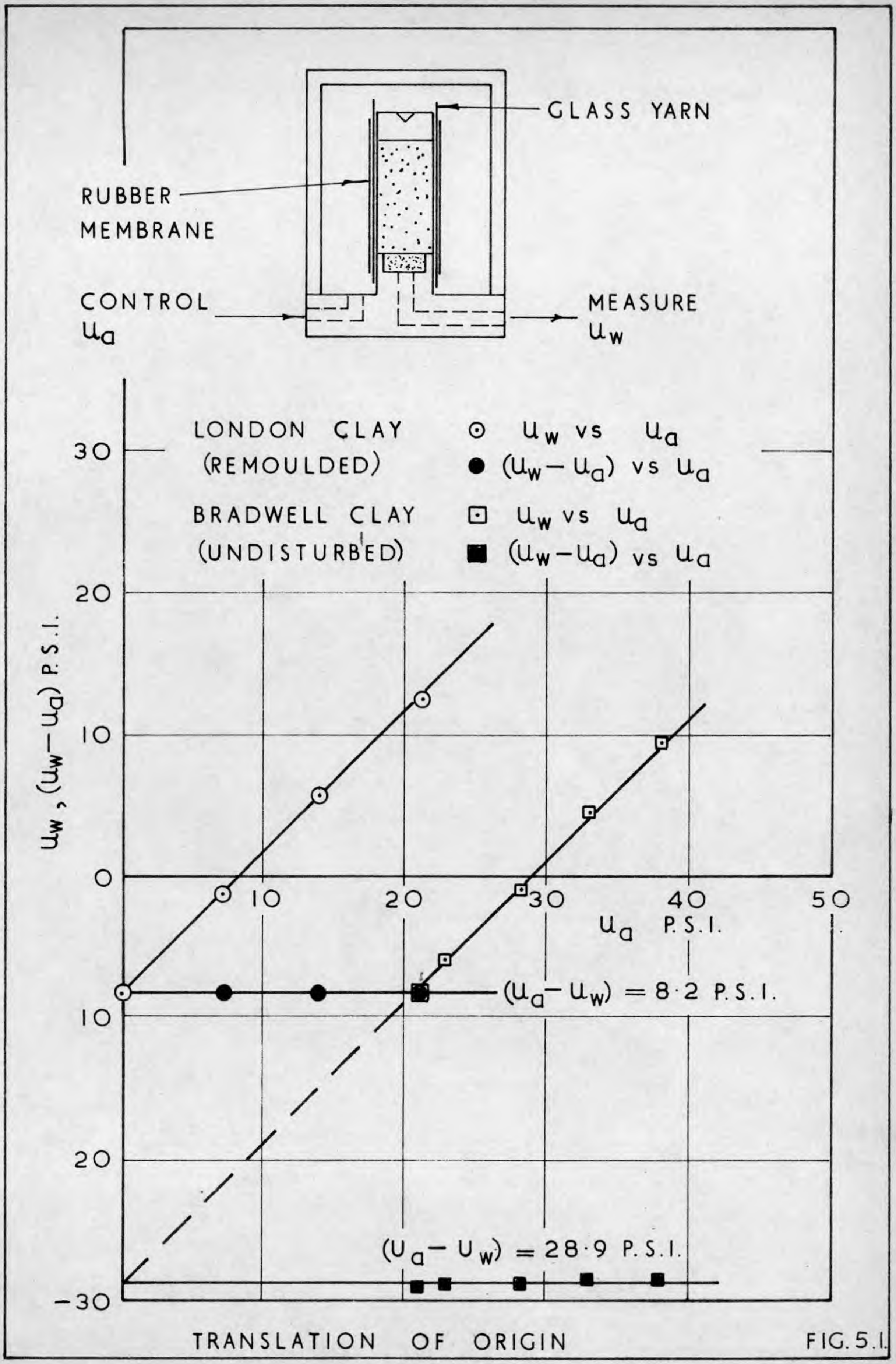
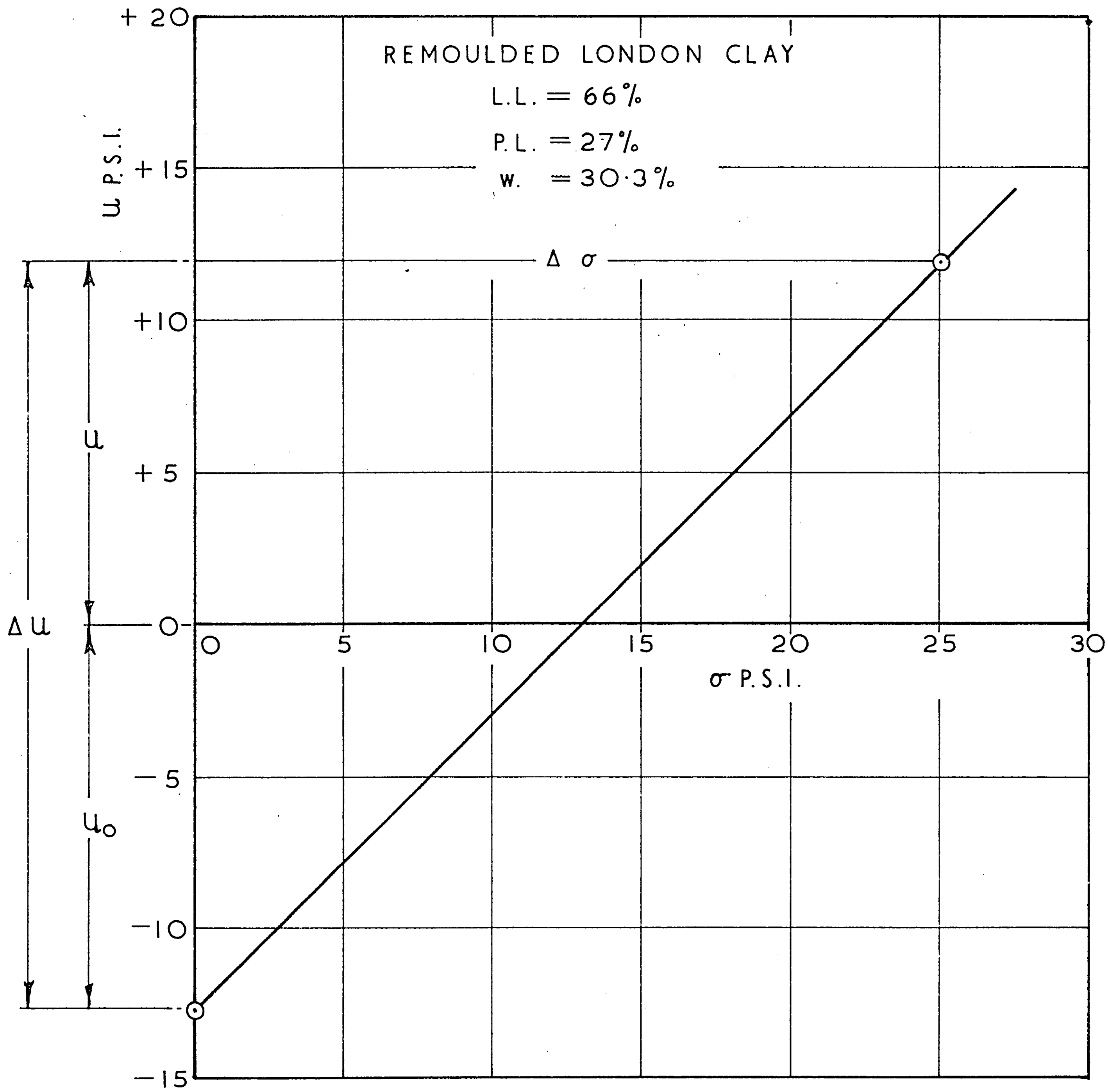

GLASS YARN
RUBBER MEMBRANE
CONTROL u_a
MEASURE u_w
LONDON CLAY (REMOULDED)
u_w vs u_a
$(u_w - u_a)$ vs u_a
BRADWELL CLAY (UNDISTURBED)
u_w vs u_a
$(u_w - u_a)$ vs u_a
u_w, $(u_w - u_a)$ P.S.I.
u_a P.S.I.
$(u_a - u_w) = 8 \cdot 2$ P.S.I.
$(u_a - u_w) = 28 \cdot 9$ P.S.I.
30
20
10
0
10
20
−30
10
20
30
40
50
TRANSLATION OF ORIGIN

FIG. 5.1.



$$U = u_0 + \Delta u$$

$$B = \frac{\Delta u}{\Delta \sigma} = \frac{u - u_0}{\Delta \sigma}$$

$$= \frac{12{\cdot}0 - (-12{\cdot}6)}{25{\cdot}0}$$

$$= 0{\cdot}98$$

INCREASE IN PORE WATER PRESSURE WITH INCREASE IN ALL ROUND PRESSURE UNDER UNDRAINED CONDITIONS.

FIG. 5·2.

suction range our normal soil mechanics concepts still apply. The initial negative pore pressure existing in almost every laboratory sample when unconfined has often been neglected, with consequent erroneous B values for the first increment of applied stress.

FIGS. 5.1 and 5.2 also demonstrate the equivalence for saturated soils of mechanical pressure applied through a membrane and air pressure applied through the capillary action of the pore water. A more convincing proof of this equivalence involving measurements of water content changes (and hence also volume changes) is given in section 5.4. The principle is important as consolidation under moderate pressures has been used to study suction-water content relationships. The oedometer has been used for this purpose (Croney, Coleman and Bridge) although it does not subject the soil to an isotropic stress system. Differences in water content produced by oedometer and triaxial consolidation may be gauged by reference to Fig.5.5. (later).

(c) Suctions in undisturbed samples

The suction in an unconfined laboratory sample is related to the principal stresses to which the soil was subjected in the ground but is not necessarily equal to the effective overburden stress as is often assumed. Bishop and Henkel (1953) simulated consolidation (under conditions of zero lateral yield, $K_o = 1$) followed by sampling in a triaxial cell for a sample of silty clay. For an effective overburden stress of 25 lb/in^2 (and lateral effective stress 10.6 lb/in^2) they found an isotropic effective stress after sampling of only 8.9 lb/in^2, i.e. only 36% of the overburden stress. The difference was attributed to the effect of releasing a deviator stress of 14.4 lb/in^2.

'Undisturbed' samples of normally consolidated peaty clay from Lagos and overconsolidated clay from Bradwell became available and the unconfined suctions were measured. Values of suction are presented in TABLE 5.2 and compared with the overburden effective stresses.

NORMALLY CONSOLIDATED LAGOS CLAY L.L 170% P.L. 60%					OVERCONSOLIDATED BRADWELL CLAY L.L. 92% P.L. 35%			
Depth ft.	σ_v' (lb/in^2)	$-u_w$ (lb/in^2)	$\frac{-u_w}{\sigma_v'}$	$-u_w$ (remoulded)	Depth ft.	σ_v' lb/in^2	$-u_w$ lb/in^2	$\frac{-u_w}{\sigma_v'}$
12	2.0	0.6	0.30	0.1	19	9	18.1	2.01
22	3.0	1.75	0.58	0.25	30	13	29	2.23
32	3.9	1.1	0.28	-		TOO STIFF FOR		
45	5.3	2.5	0.47	-		REMOULDING		

σ_v' = vertical effective stress in situ;

$-u_w$ = suction in unconfined sample.

TABLE 5.2. THE EFFECT OF SAMPLING ON PORE WATER PRESSURES IN SOILS

The ratio $\frac{-u_w}{\sigma_v'}$ is seen to be very different from 1 in both cases. A satisfactory though somewhat incomplete analysis of these figures is possible and is given in Appendix 2. The small suctions, and hence effective stresses, in remoulded Lagos clay indicate that this is a fairly sensitive soil.

(5.4) Effective stresses and volume change

The simplest method of checking the effective stress law for negative pore water pressures is to compare volume changes caused by direct mechanical load and internal suction. Strictly only isotropic mechanical stress can be compared with suction stress but, so far as the writer is aware, the published results compare water contents produced in the oedometer with those obtained by normal suction techniques (Croney, Lewis and Coleman - 1950, Aitchison and Donald - 1956). Exact equivalence cannot be hoped for with this approach and so a comparison of water contents produced by triaxial consolidation and suction was made. (FIG. 5.3). The soil moisture characteristic curve marked R.R.L was kindly supplied by the Road Research Laboratory. It was obtained by a combination of suction plate, pressure membrane and vapour equilibrium techniques. The lower curve was obtained at Imperial College from a triaxial consolidation test with $u_w = 0$. Unfortunately 53% was the limiting water content at which the triaxial sample could be successfully handled and hence the two curves remained

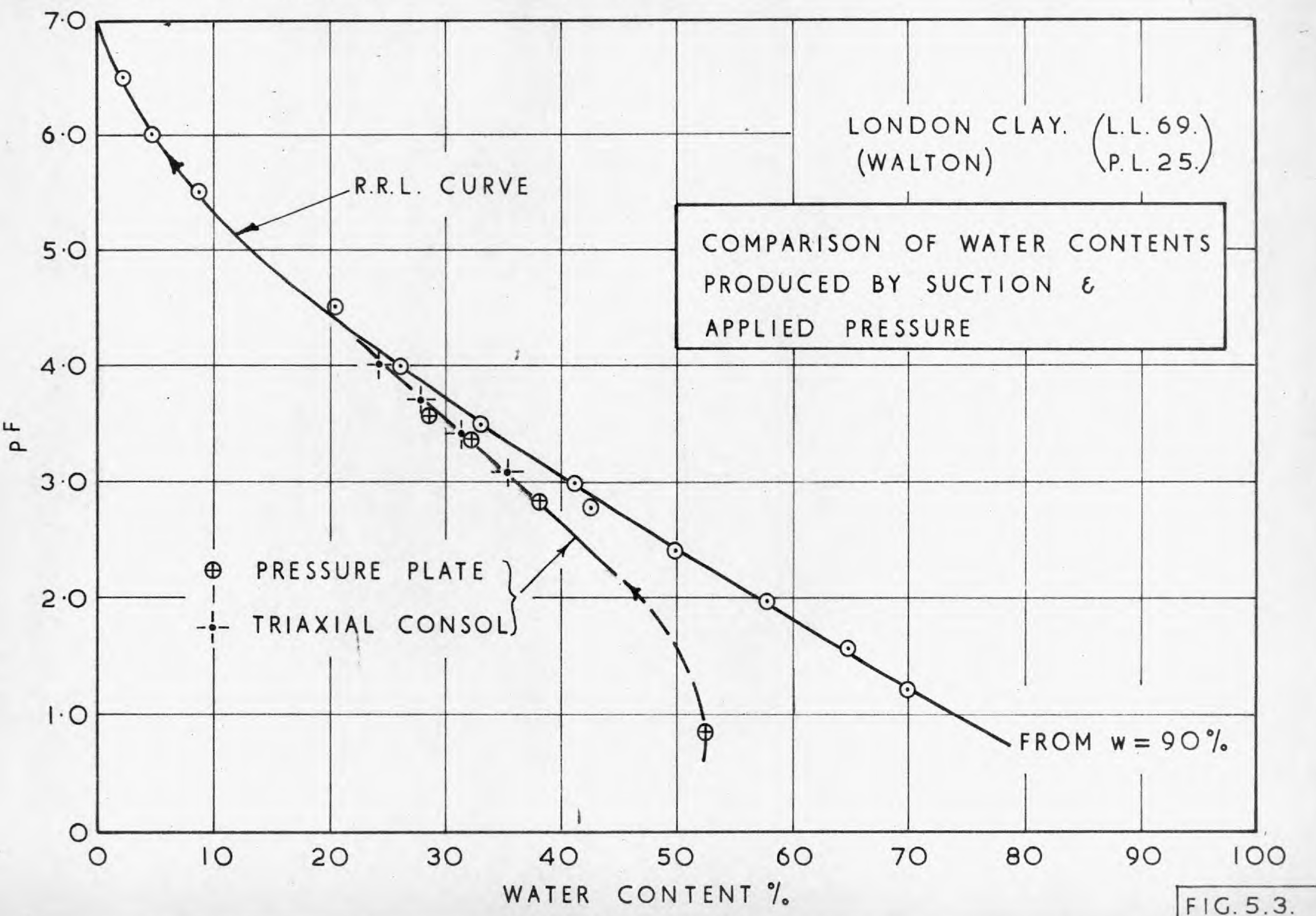

LONDON CLAY. (WALTON)
(L.L. 69. P.L. 25.)
COMPARISON OF WATER CONTENTS PRODUCED BY SUCTION & APPLIED PRESSURE
R.R.L. CURVE
PRESSURE PLATE
TRIAXIAL CONSOL
FROM w = 90%
pF
7·0
6·0
5·0
4·0
3·0
2·0
1·0
0
0
10
20
30
40
50
60
70
80
90
100
WATER CONTENT %

FIG. 5.3.

separate over the range studied. It is quite clear however that the initial water content difference is rapidly disappearing and the lines would probably coincide before pF5, unless the suction specimens had begun to desaturate.

To establish the equivalence beyond all doubt some pressure plate readings were taken, starting from the same initial water content as the triaxial sample. These points fell accurately on the triaxial compression line and, as section (1.4) has established the practical equivalence of pressure plate or membrane methods with other suction techniques then the mechanical equivalence of suction and applied pressure has been established, at least up to suctions of pF4.

This equivalence enables a quantitative attack to be made on problems of shrinkage, heaving, and swelling pressures in some heavy clay soils. The range of suctions (and hence an estimate of the range of swelling pressures) found in the field in semi-arid climates has been studied experimentally by Aitchison (1956).

(5.5) Shear strength and effective stresses

(a) Previous work

The validity of the Terzaghi effective stress law has been well established for saturated cohesionless soils in the range $0 > u_w > -13$ lb/in^2. Bishop and Eldin (1950) tested sand in undrained compression with pore water pressure measurement down to about -13 lb/in^2.

Fine porous discs were not required for this work as the whole sample - cell base - pore pressure measuring system was air free. Penman (1954) performed similar tests on Braehead silt. Childs (1955) and Donald (1956) tested sand samples with controlled negative pore water pressures. Childs' samples were unconfined columns while Donald's tests were carried out in a direct shear box. In all of these tests, provided that no cavitation occurred in the apparatus, the soil when fully saturated behaved as if the negative pore water pressure was equivalent to an applied external stress.

The data for clays is somewhat confusing. Aitchison (1957) and Towner (1958) equilibrated samples with various suctions and then carried out rapid unconfined compression tests without pore pressure measurements. This is equivalent to a series of consolidated undrained tests plotted in terms of total stresses. The scatter in strengths at any one suction was very large and this raises some doubts as to the ability of the pressure membrane to control the suction in a sample accurately for a long time. At the higher suctions a small change in water content is reflected as a large change in effective stress, but most checks on membrane operation have been done on a water content basis.

Towner's failure envelope from suction tests gave him the same ϕ_{cu} (consolidated undrained angle of shearing resistance with respect to total stress) as the value obtained from triaxial tests but the apparent cohesion c_{cu} was measured as 4.2 ± 1 lb/in^2 compared with 2.4 lb/in^2 obtained in the triaxial test. Both values are higher than usually obtained for truly normally consolidated clays. Aitchison did not attempt to compare his results with triaxial tests as pressures of several hundred atmospheres would have been necessary. Some discrepancies in his results have already been discussed in section 2.4.

A similar series of tests on undisturbed samples of London clay was reported by Croney and Coleman (1960). The soil remained saturated up to a suction of pF 5 and the unconfined compression strength increased steadily with log. initial suction but again there were no tests under applied external pressure for comparison. ϕ_{cu} appeared to decrease with increasing suction.

(5.5) (b) Shear Strength of remoulded clay

(i) As an alternative to equilibrating samples with a known suction, triaxial specimens with a range of initial suctions were prepared by remoulding London clay at water contents between the liquid and plastic limits. The main purpose of these samples was to study the equivalence of suction and external stress as generators of shear resistance in soils but, as remoulded soils have usually been studied on a total stress basis, several collateral investigations were made.

(5.5) (b) (ii) Pore pressure changes during remoulding

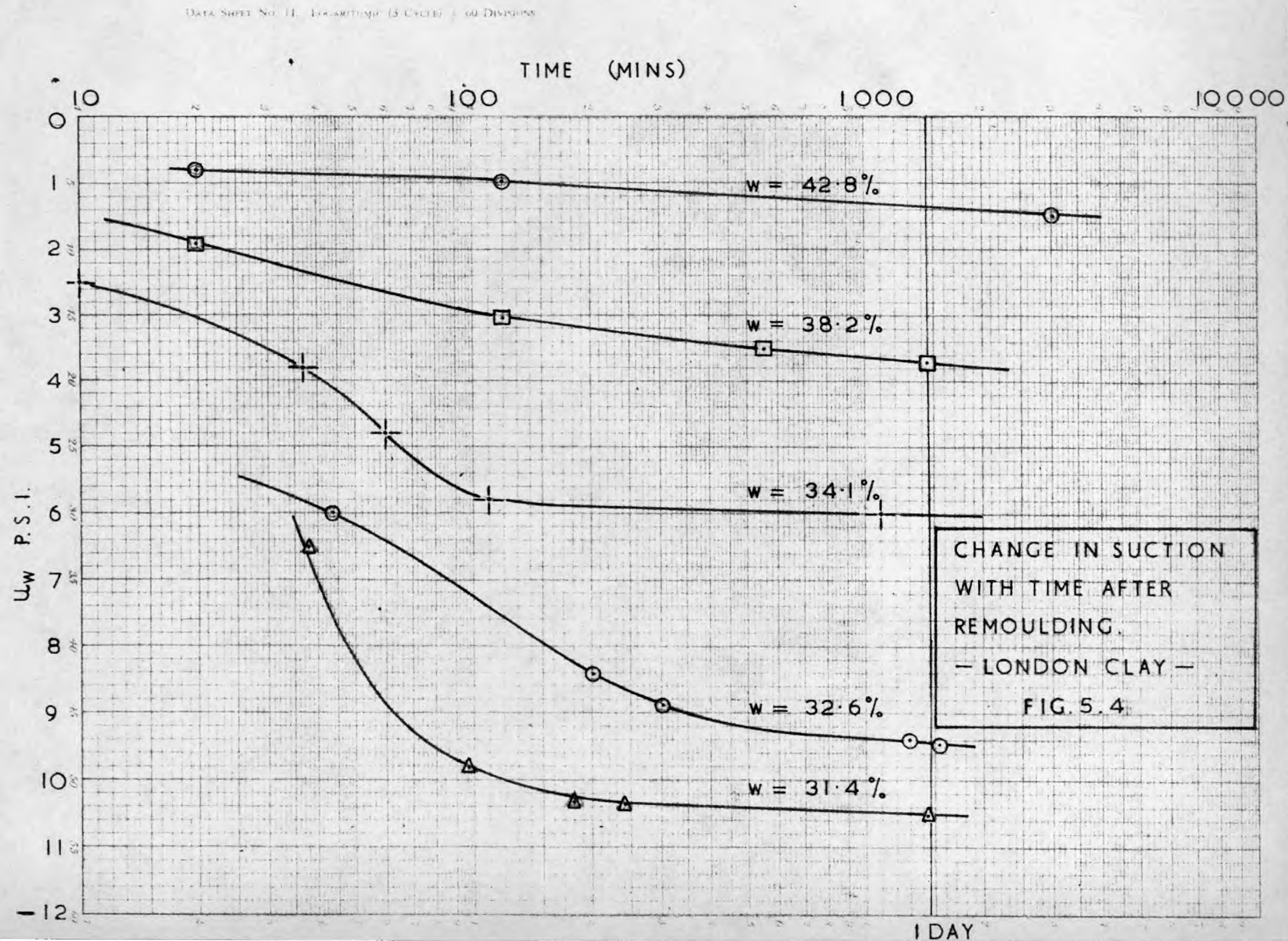
The clay, as described in section (4.5) was normally consolidated in the oedometer before remoulding and forming into 1½" dia. by 3" long samples. After trimming and placing the sample on the cell base, the suction was followed until its rate of change had become sufficiently slow for testing to commence.

Changes in remoulded suction with time for soils near the liquid limit have been given by Day (1956) and Low (1959) but there is, to the writer's knowledge, no published data for soils with moderately low liquidity indices (L.I.)

$$\text{where} \quad L.I. = \frac{w - P.L.}{L.L. - P.L.} \qquad \ldots\ldots\ (5.1)$$

The remoulding process, which may be regarded as very severe shearing, is assumed to take place at constant volume for saturated soils. This assumption may be in error if second order effects are included and some tests were performed to check its validity. These are discussed in Appendix I.

Typical suction versus log. time curves for a range of water contents are given in Fig. 5.4. The effective stress-moisture content relationships for oedometer and triaxial consolidation, together with the values for remoulded samples one day after remoulding are given in FIG. 5.5. The negative pore pressure in a saturated



CHANGE IN SUCTION WITH TIME AFTER REMOULDING.
— LONDON CLAY —
FIG. 5.4

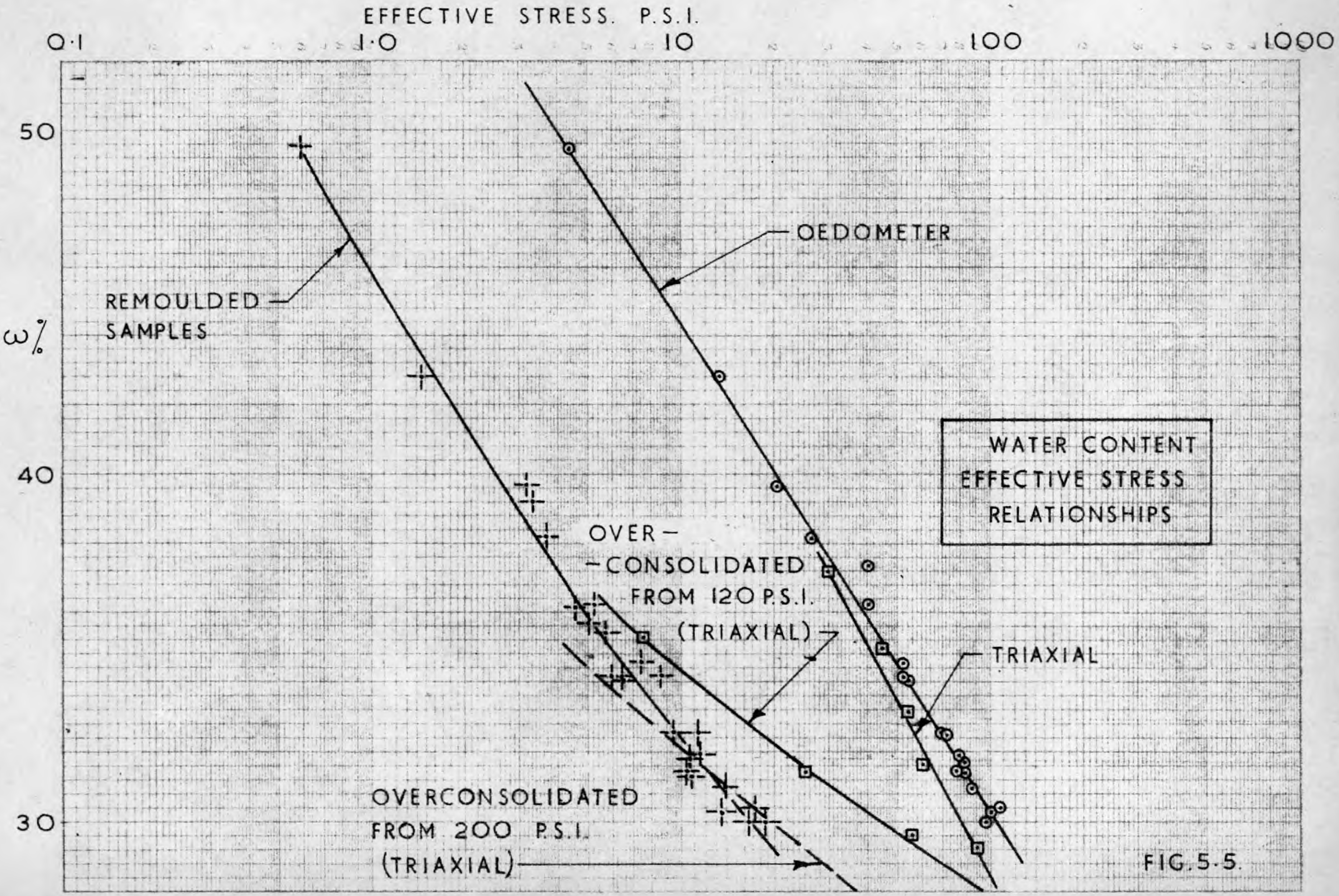

EFFECTIVE STRESS. P.S.I.
0·1
1·0
10
100
1000
ω%
50
40
30
OEDOMETER
REMOULDED SAMPLES
WATER CONTENT EFFECTIVE STRESS RELATIONSHIPS
OVER-CONSOLIDATED FROM 120 P.S.I. (TRIAXIAL)
TRIAXIAL
OVERCONSOLIDATED FROM 200 P.S.I. (TRIAXIAL)

FIG. 5·5.

sample which has been isotropically consolidated and then released from external stress may be assumed numerically equal to the consolidating pressure. On remoulding a large change in pore pressure occurs (See table 5.3).

w water content %	$-u_{w_o}$ (lb/in^2) (before remoulding)	$-u_{w_r}$ (lb/in^2) (after remoulding)	$\frac{u_{w_o}}{u_{w_r}}$
40	20	2.5	8.0
35	40	5.9	6.8
30	80	16.5	4.8

TABLE 5.3. LONDON CLAY:- CHANGE IN UNCONFINED PORE WATER PRESSURE ON REMOULDING. (BEFORE CONSOLIDATION w = 62%)

An attempt was made to simulate remoulding by alternate undrained compression and extension tests inside a triaxial cell but the samples 'necked' severely after two or three stress reversals and the effective stresses at this stage were still considerably different from the fully remoulded values. Murayama and Hata (1957) have shown that appreciable pore pressure changes were still occurring after several hundred stress reversals on a clay sample in a 'simple shear' box and it would seem that hand remoulding is the only simple and effective technique.

The remoulded suction line in FIG. 5.5 agrees moderately closely with the 'continuously disturbed suction line' of Croney and Coleman (1954) for a similar clay. An interesting feature of figure 5.5 is that non-remoulded samples would require very high overconsolidation ratios to lie on the remoulded suction line. (The overconsolidation ratio is taken as $\left\{\frac{\text{maximum consolidation pressure}}{\text{pressure at final equilibrium}}\right\}$). The dotted line for overconsolidation from 200 lb/in^2 has been drawn according to the dimensional method of Lundgren (1957) by analysis of the experimental data for rebound from 120 lb/in^2.

From FIG. 5.4 it is apparent that there may be continued long term suction changes. Also from the evidence quoted above, complete remoulding may not always be achieved. To check on the required remoulding time several samples at each of two water contents were remoulded for times varying from one to forty minutes and the suction accurately followed for some days. Small differences in initial water contents made it difficult to compare the results directly but the suction-log. time curves all followed the same general shape - low initial values (1 to 3 lb/in^2) immediately after remoulding with a gradual increase to full value (5 to 9 lb/in^2) within 200 - 300 minutes. There was no appreciable further increase for times up to 50,000 minutes. Allowing for the water content differences, increased remoulding time produced progressively smaller final suctions, but beyond 20 minutes remoulding the changes were small. The difficulty in remoulding drier samples may account for the bend to the right at the lower end of the remoulded suction curve (FIG. 5.5). Croney and Coleman's 'continuously disturbed' curve for Gault Clay (Croney and Coleman, 1960) shows the same phenomenon very markedly.

(5.5) (b) (iii) Compression tests on remoulded London clay

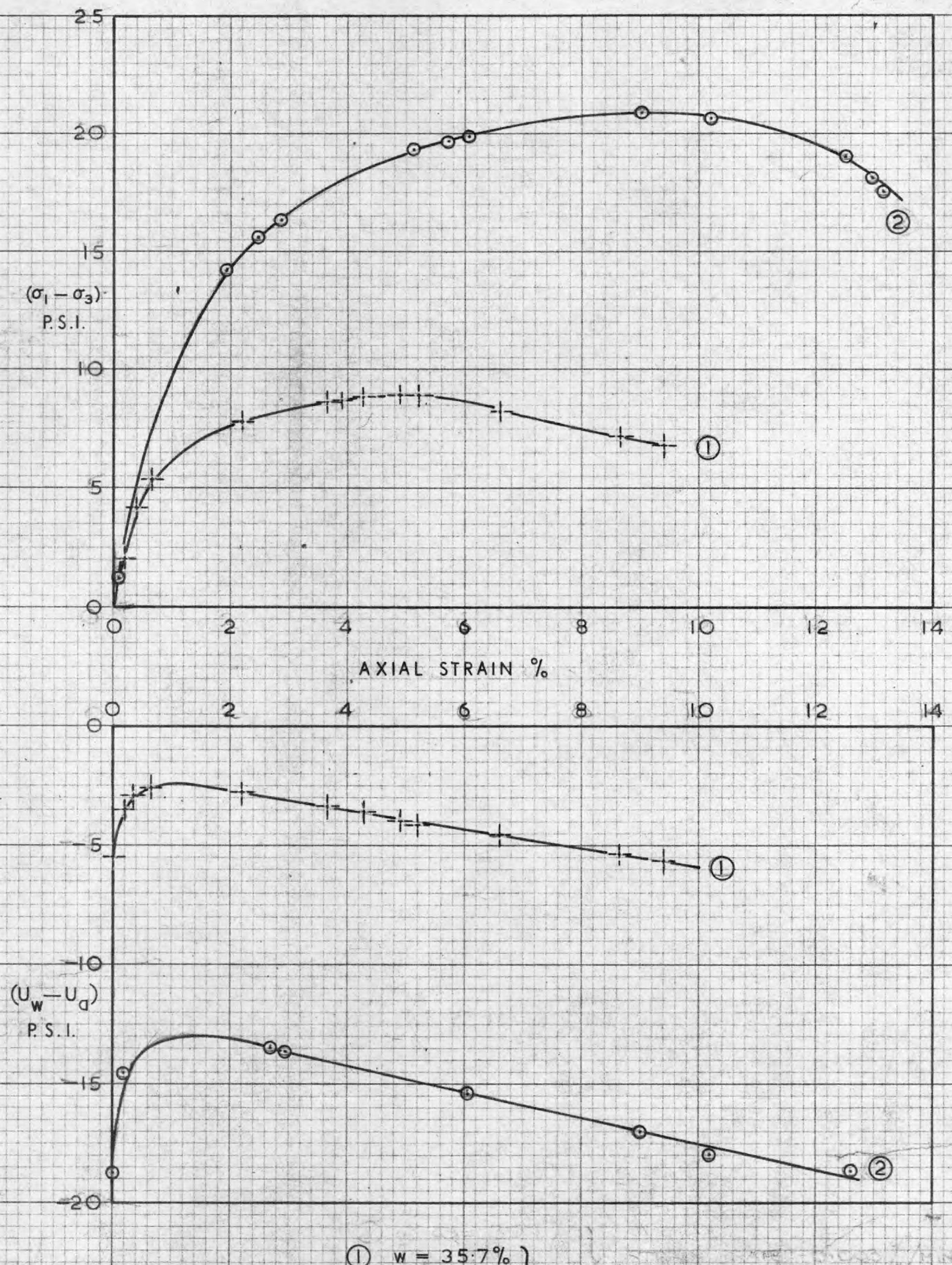
Remoulded samples were failed in axial compression either unconfined, unconfined with elevated ambient air pressure u_a, or confined with positive cell pressures. Maximum deviator stress was taken as the failure criterion and all samples failed with a clearly defined peak strength and many failure planes distributed throughout the specimen. A correction for strength caused by the rubber membrane was made only for the confined tests. Water contents were determined for the middle and ends of each sample after compression. In general the middle third of the sample was 0.2 - 0.4% wetter than the ends.

Two typical stress-strain curves are shown in FIG. 5.6. The complete test series is summarised in Table 5.4. For all samples the pore water pressure rose sharply during the first 1 per cent strain and then gradually returned to somewhere near its initial value by failure, the average value of A at failure being + 0.1 which is typical of a moderately overconsolidated clay.

TABLE 5.4 TEST RESULTS - LONDON

$G_s = 2.75$ L.L = 66% P.L = 2

Test No. L.C.	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	21	22	23	24	25	26	27	T.1
	← REMOULDED, UNDRAINED TESTS																								←
σ_3 - cell pressure LB/IN^2	0	0	0	0	0	0	0	0	0	25	0	0	0		0	0	0	10	10	20	20	20	15	15	0
U_a - pore-air pressure LB/IN^2	0	0	0	0	0	0	0	0	0	0	0	0	0		0	0	0	0	0	0	0	0	15	15	0
U_w pore-water pressure (initial) LB/IN^2	-3·4	-3·2	-5·7	-10·5	-0·6	-10·7	-5·3	-6·0	-1·45	+11·6	-11·5	-11·2	-6·5		-3·7	-9·5	-10·7	+5·3	+2·5	+8·6	+5·9	+2·8	-1·7	+6·4	-4·6
U_w " " " (failure)	-2·1	-2·45	-4·5	-8·5	-0·65	-9·2	-4·0	-4·75	-1·4	+12·5	-9·8	-8·9	-4·9		-2·8	-7·4	-11·0	+7·0	+3·6	+11·5	+6·2	+3·9	-1·4	+6·85	-4·05
$(\sigma_1 - \sigma_3)_f$ - at failure, LB/IN^2	6·2	6·08	9·92	16·5	2·65	15·95	9·47	11·3	4·46	21·0	17·0	16·9	10·9		6·95	13·7	18·1	7·5	12·45	15·8	19·5	21·9	22·7	13·75	8·9
$\frac{(\sigma_1 - \sigma_3)_f}{2}$ LB/IN^2	3·1	3·04	4·96	8·25	1·32	7·97	4·73	5·65	2·23	10·5	8·50	8·45	5·45		3·47	6·85	9·05	3·75	6·22	7·90	9·75	10·95	11·35	6·87	4·95
σ'_{1f} - major princ. eff. stress, LB/IN^2	8·3	8·53	14·42	25·0	3·30	25·15	13·47	16·05	5·86	33·5	26·8	25·8	15·8		9·75	21·1	29·1	10·5	18·85	24·3	33·1	38·0	39·1	21·9	12·95
σ'_{3f} - minor " " "	2·1	2·45	4·5	8·5	·65	9·2	4·0	4·75	1·40	12·5	9·80	8·90	4·90		2·80	7·40	11·0	3·0	6·40	8·50	13·8	16·1	16·4	8·15	4·05
$\frac{(\sigma'_1 + \sigma'_3)_f}{2}$ LB/IN^2	5·20	5·49	9·46	16·75	1·97	17·2	8·73	10·4	3·63	23·0	18·3	17·35	10·35		6·27	14·25	20·05	6·75	12·62	16·4	23·45	27·05	27·75	15·02	8·50
ω_f - water content at failure, %	39·3	39·7	35·4	31·4	49·5	31·8	36·2	34·1	42·8	30·3	31·8	31·8	34·0		38·2	32·6	31·4	36·1	34·6	32·6	31·0	30·4	30·0	34·15	36·0
p_e - equivalent consol^n. pressure, LB/IN^2	24	23	39·5	67·5	-	64	36	46·7	16	79	64	64	47		28	57	68	36	44	57	71	78	82	46·5	
$\frac{(\sigma_1 - \sigma_3)_f}{2 p_e}$	·129	·132	·1255	·122	-	·124	·1315	·121	·1395	·133	·133	·132	·116		·124	·120	·133	·104	·1415	·1385	·135	·140	·138	·1475	←
$\frac{\sigma'_{3f}}{p_e}$	·0875	·106	·113	·126	-	·144	·111	·1015	·088	·158	·153	·139	·104		·100	·130	·162	·0835	·146	·149	·1945	·206	·200	·175	
σ'_1 - oedometer consolidation, LB/IN^2		20·2	40·2	80·5	4·4	80·5	40·2	52·5	13·1	100·5	78·8	78·8	52·5	52·5	26·2	70·0	78·7	26·2	52·5	70·0	87·5	105	96	52·5	
ω (" "). %			37·1	31·9	50·5	32·9	36·8	35·4	43·8	30·2	33·4	32·5	35·8	35·6	38·9	33·3	32·4	38·8	34·6	33·2	32·0	31·0		33·8	
A_f - A at failure	·21	·11	·12	·12	-·02	·09	·14	·11	·01	·04	·10	·13	·15		·13	·15	·02	·23	·09	·18	·00	·05	·01	·03	·08
ε_f - strain at failure %	7·2	7·0	8·2	7·8	7·0	7·2	6·6	6·9	8·3	10·8	7·2	5·4	5·5		4·7	5·0	6·8	6·2	8·6	8·7	12	10	8·8	7·2	6·3
Strain rate, % per minute ($\times 10^{-2}$)	·57	2·8	2·8	2·8	2·8	·57	·57	·57	·57	·57	·57	·12	·12		·12	·12	·12	·57	·57	·57	·57	·57	·57	·57	·30



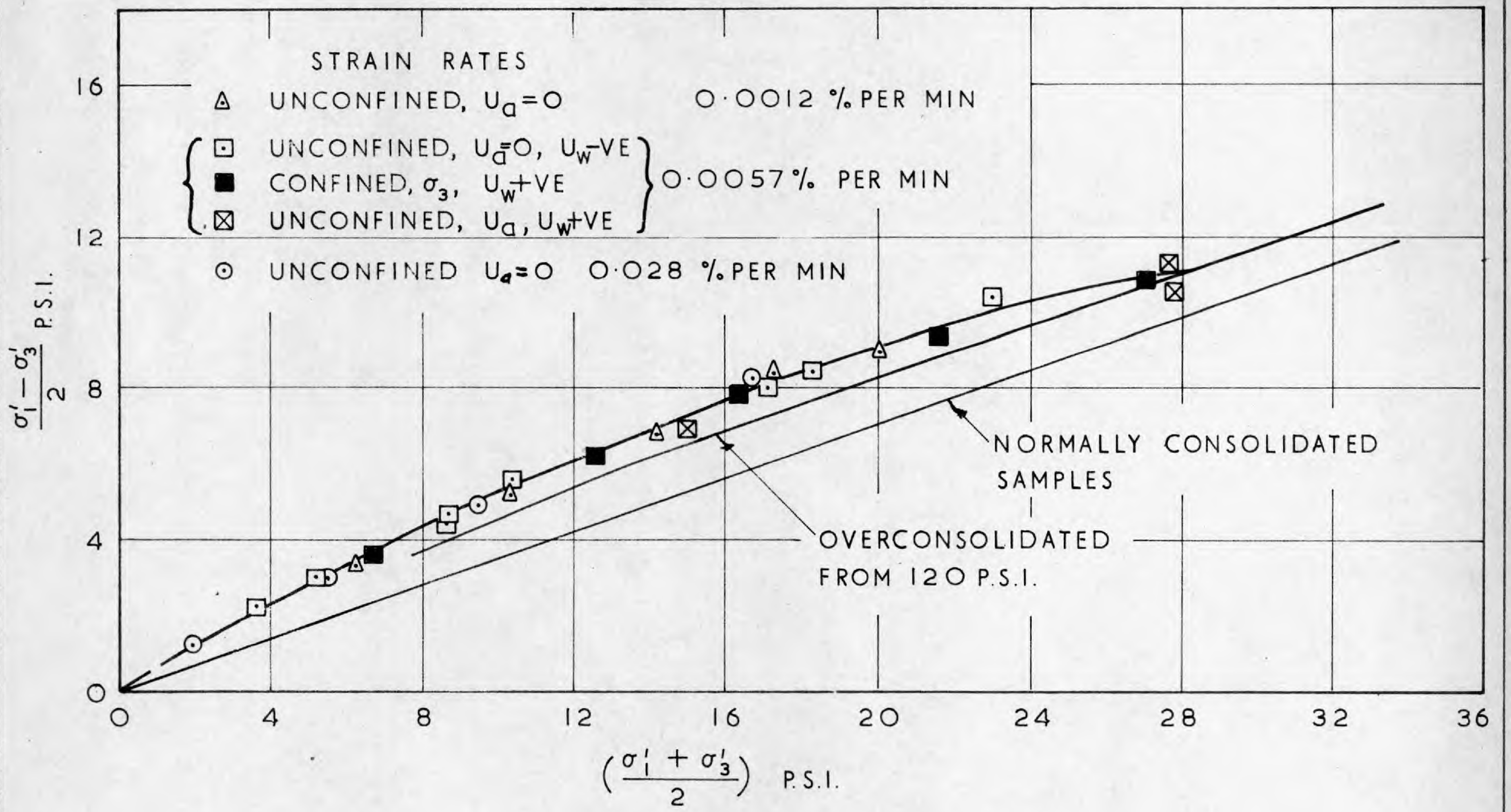
UNDRAINED COMPRESSION TESTS – REMOULDED CLAY

FIG 5.6.

The correct strain rate for satisfactory pore pressure equalization was determined by running tests at three different rates - from failure in 24 hours to failure in 7 days. There is little point in using filter paper drains on samples with negative pore water pressures and the published data on testing rates refers to such accelerated equalization of positive pore pressures. However, when plotted as in Figs. 5.7 and 5.8 no appreciable differences were found between tests done at any of the three strain rates. Hence most samples were strained at 0.0057 per cent. per minute, a convenient rate for completing tests within two days and getting accurate pore pressure readings for most of the test.

In FIG. 5.7 the results are plotted as one half the sum of the effective principal stresses at failure against one half of the deviator stress. FIG. 5.8 is the more usual Mohr plot, but with many circles this tends to become rather confused and FIG. 5.7 which is in effect the envelope of the top points of the Mohr circles, is much clearer and the envelope is easier to fix. The relationship between the slopes of the envelopes in FIGS. 5.7 and 5.8 has been dealt with in section 2.5. FIG. 5.9 shows the failure envelopes for the same clay normally consolidated and overconsolidated from a maximum pressure of 120 lb/in^2. These envelopes are also shown on FIG. 5.7.

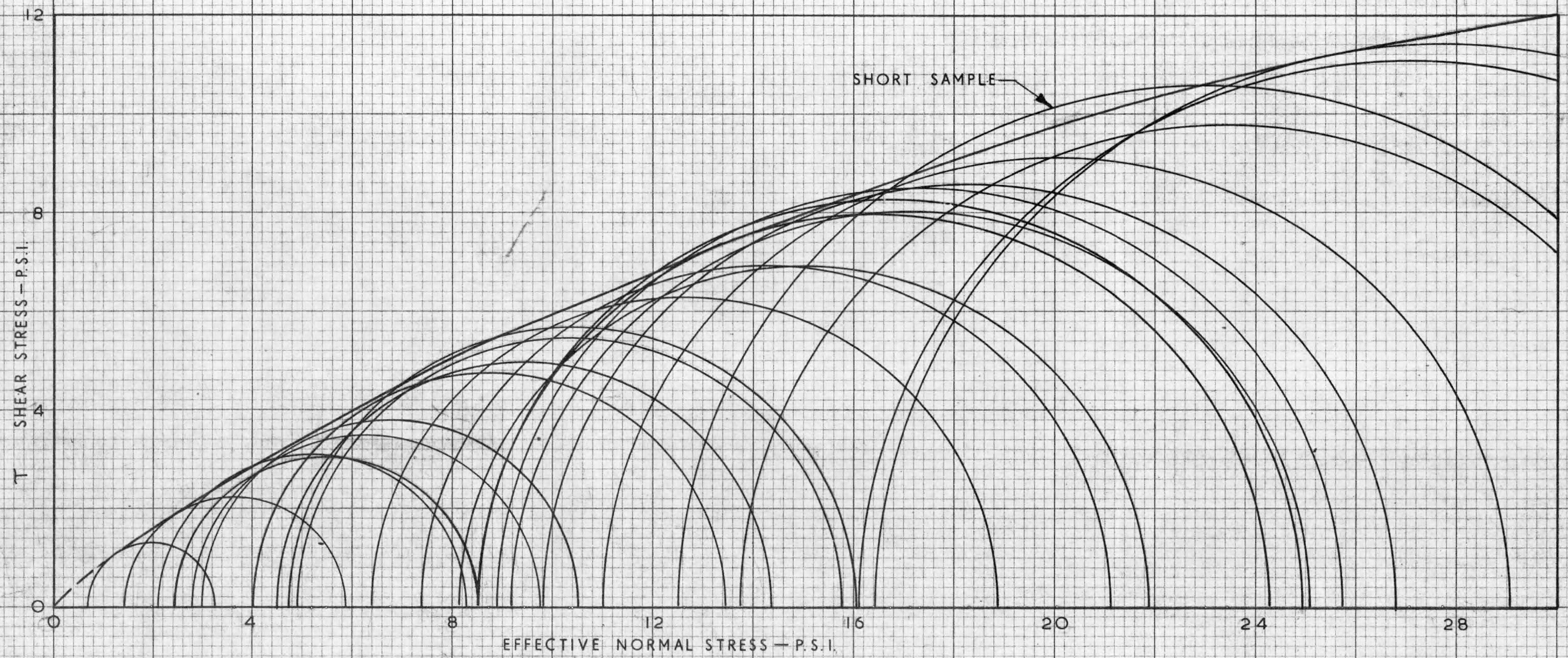
The failure envelope for remoulded samples is gently curved but



LONDON CLAY

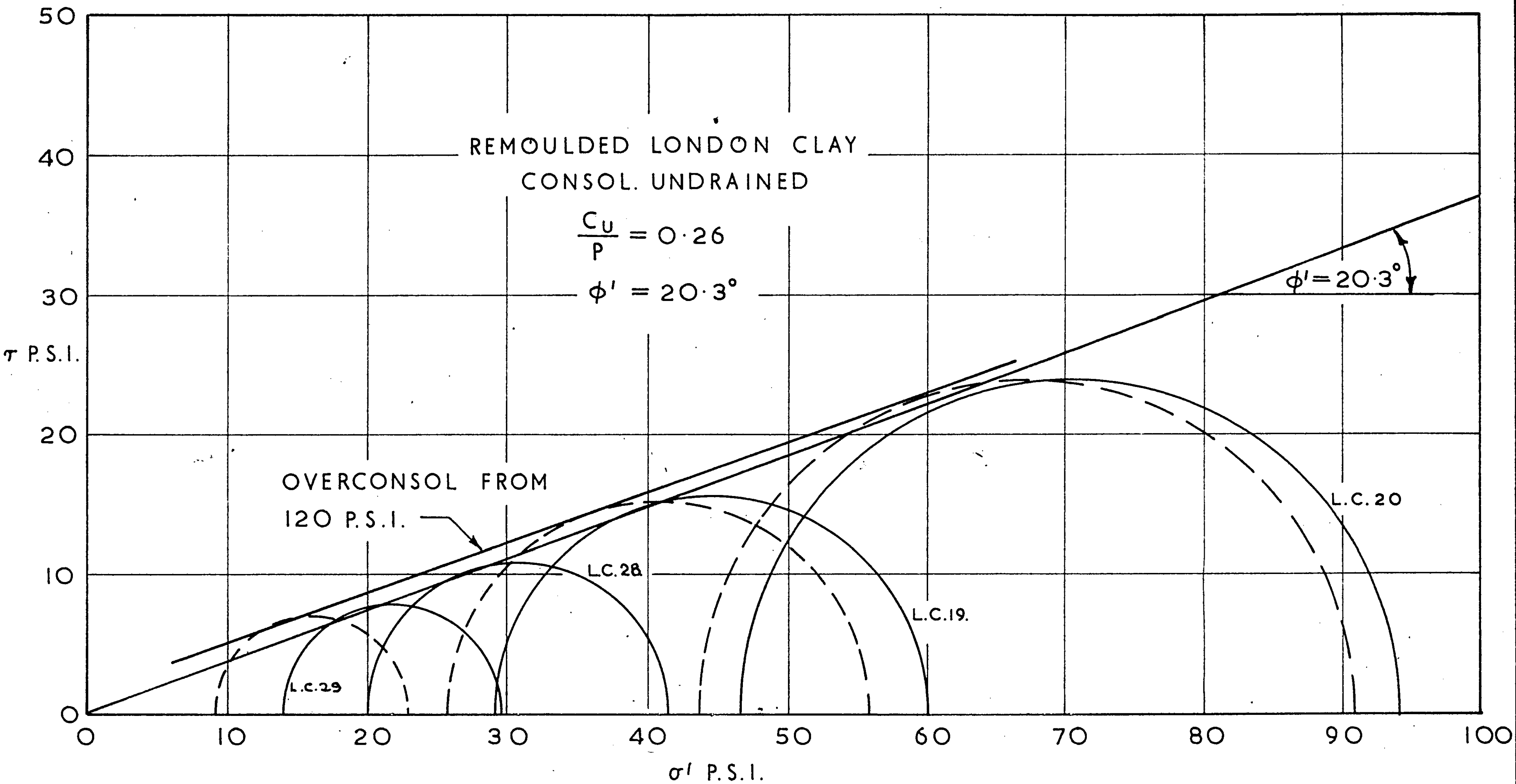
FAILURE ENVELOPE FOR REMOULDED SATURATED SAMPLES

FIG. 5.7



MOHR FAILURE ENVELOPE (REMOULDED LONDON CLAY)

FIG 5.8.



FAILURE ENVELOPES FOR LONDON CLAY

(1) NORMALLY CONSOLIDATED

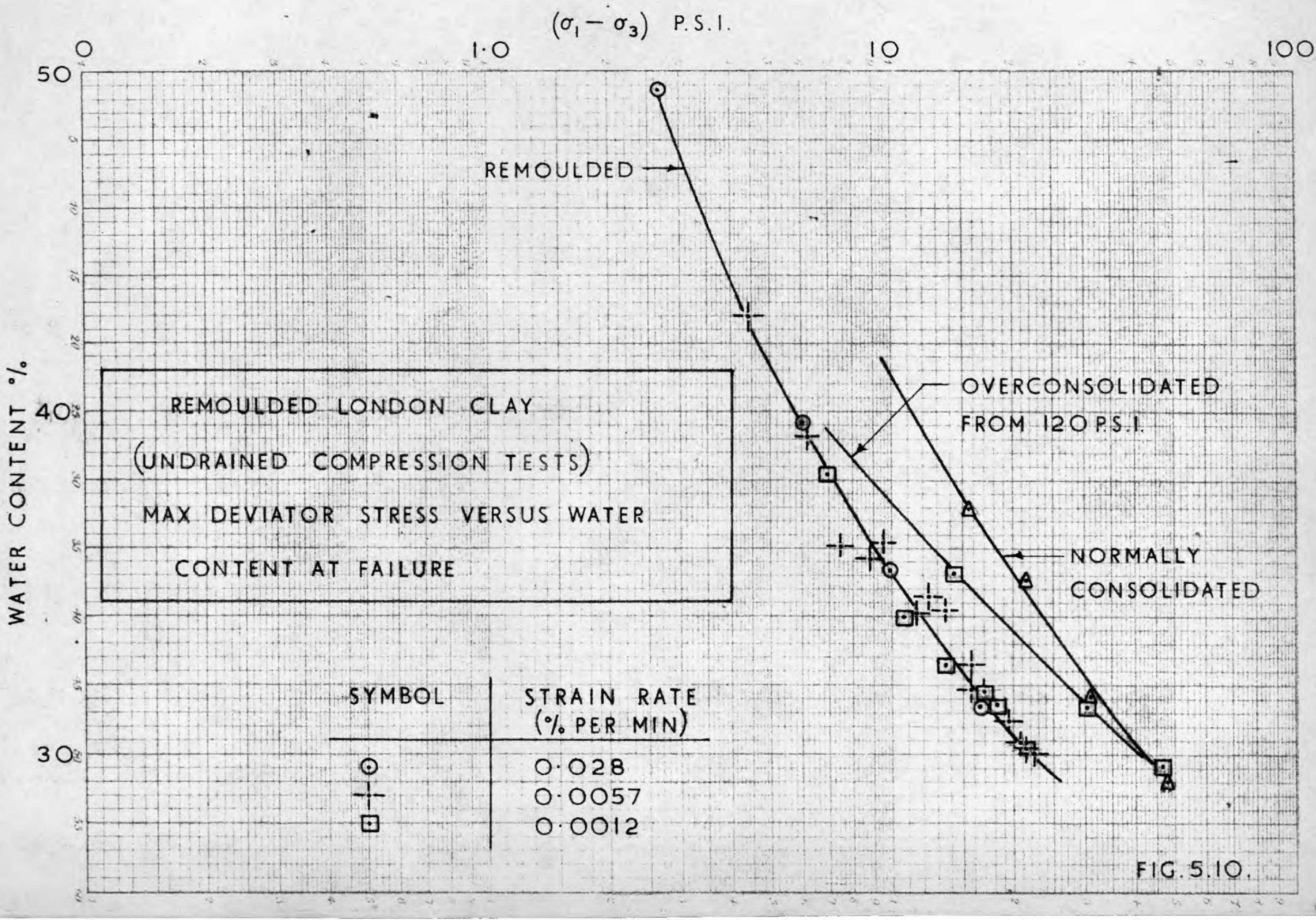
(2) OVER-CONSOLIDATED FROM 120 P.S.I.

FIG. 5. 9.

apparently passes through or near the origin. The points for drier samples may turn towards the normally consolidated line because of incomplete remoulding as mentioned earlier. In a general way the samples may be said to be behaving like highly overconsolidated non-remoulded specimens in that their strengths are all considerably above the normally consolidated envelope. However the A values are not strictly comparable as these may be appreciably negative for heavily overconsolidated clay. This reflects the differences in structure prior to shearing.

Probably the most interesting single feature of the failure envelope in FIG. 5.7 is that the points all lie accurately on the line regardless of whether the pore water pressure is negative, positive due to elevated air pressure or positive as a result of confinement and applied cell pressure. Hence, in the range 0-20 lb/in^2 we may take the concepts outlined in Chapter 2 as valid.

In FIG. 5.10 the deviator stress at failure is plotted against the overall sample water content for normally consolidated and remoulded tests. The points representing remoulded samples lie on a unique curve regardless of the type of test (i.e. with or without elevated pore water pressure) or the rate of strain. Some of the scatter evident in the central part of the curve may have been due to the capricious behaviour of the laboratory drying ovens which had to be replaced some time after the commencement of the project.



(σ₁ − σ₃) P.S.I.
0
1·0
10
100
50
40
30
WATER CONTENT %
REMOULDED
OVERCONSOLIDATED FROM 120 P.S.I.
NORMALLY CONSOLIDATED
REMOULDED LONDON CLAY
(UNDRAINED COMPRESSION TESTS)
MAX DEVIATOR STRESS VERSUS WATER
CONTENT AT FAILURE
SYMBOL
STRAIN RATE (% PER MIN)
0·028
0·0057
0·0012

FIG. 5.10.

Although the pore pressure changes during shear of normally consolidated samples lead to an appreciable drop in effective stress and partial remoulding of the specimen, at failure normally consolidated and fully remoulded samples have distinctly different strengths. If, following Terzaghi (1944), we define the sensitivity of a clay as follows:

$$S = \frac{\text{undisturbed strength}}{\text{remoulded strength}} = \frac{c}{c_r} \qquad \ldots\ldots\ldots (5.2)$$

where c, c_r - apparent cohesion of undisturbed and remoulded clay respectively,

then treating the normally consolidated specimens as 'undisturbed' an average sensitivity of 1.95 is found for $38\% \geqslant w \geqslant 30\%$.

It is also instructive to plot the maximum deviator stress versus water content for samples overconsolidated from a maximum pressure of 120 lb/in^2. This has been done in FIG. 5.10. There is obviously no simple correlation between the water content of a clay and its strength, the latter falling anywhere between the normally consolidated and fully remoulded lines and possibly even below the remoulded line for overconsolidated clays at high water contents, depending on the stress history of the soil. This subject has been treated in some detail by Henkel (1957).

Peck, Hanson and Thornburn (1953) have stated that the sensitivity of highly overconsolidated soils is approximately one. Such soils are often found with field moisture contents close to the plastic limit and the general behaviour depicted in FIG. 5.10 is compatible with their statement. The failure curve for samples originally consolidated under pressures of the order of 300 lb/in^2 would then have to become practically coincident with the remoulded strength curve at water contents above about 28%.

Hvorslev Parameters

The most fundamental parameters used in soil mechanics for describing the shear behaviour of soils are those proposed by Hvorslev (1937). As modified by Gibson (1953) these parameters are

ϕ_r = true angle of internal friction

$\frac{c_r}{p_e}$ = the ratio of true cohesion to the equivalent consolidation pressure,

where p_e is the isotropic consolidation pressure which will produce in an initially slurried sample the same water content as in the specimen being investigated. The shear strength of a clay is then given by:

$$\tau_f = c_r + \sigma_n' \tan \phi_r \qquad \text{.............} (5.3)$$

where τ_f = shearing strength

σ_n' = effective normal stress on failure plane.

c_r, ϕ_r - refer to undrained tests or drained tests corrected for dilatation.

The technique normally used for determining these parameters has been given by Bishop and Henkel (1957) and requires a minimum of six tests - three normally consolidated and three overconsolidated. The difficulty lies in ensuring an appreciable difference in effective stress for samples at the same water content. However, remoulding of normally consolidated samples has been shown to effect up to an eightfold decrease in effective stress at constant water content and it was decided to analyse the test results to see if remoulding altered the parameters appreciably.

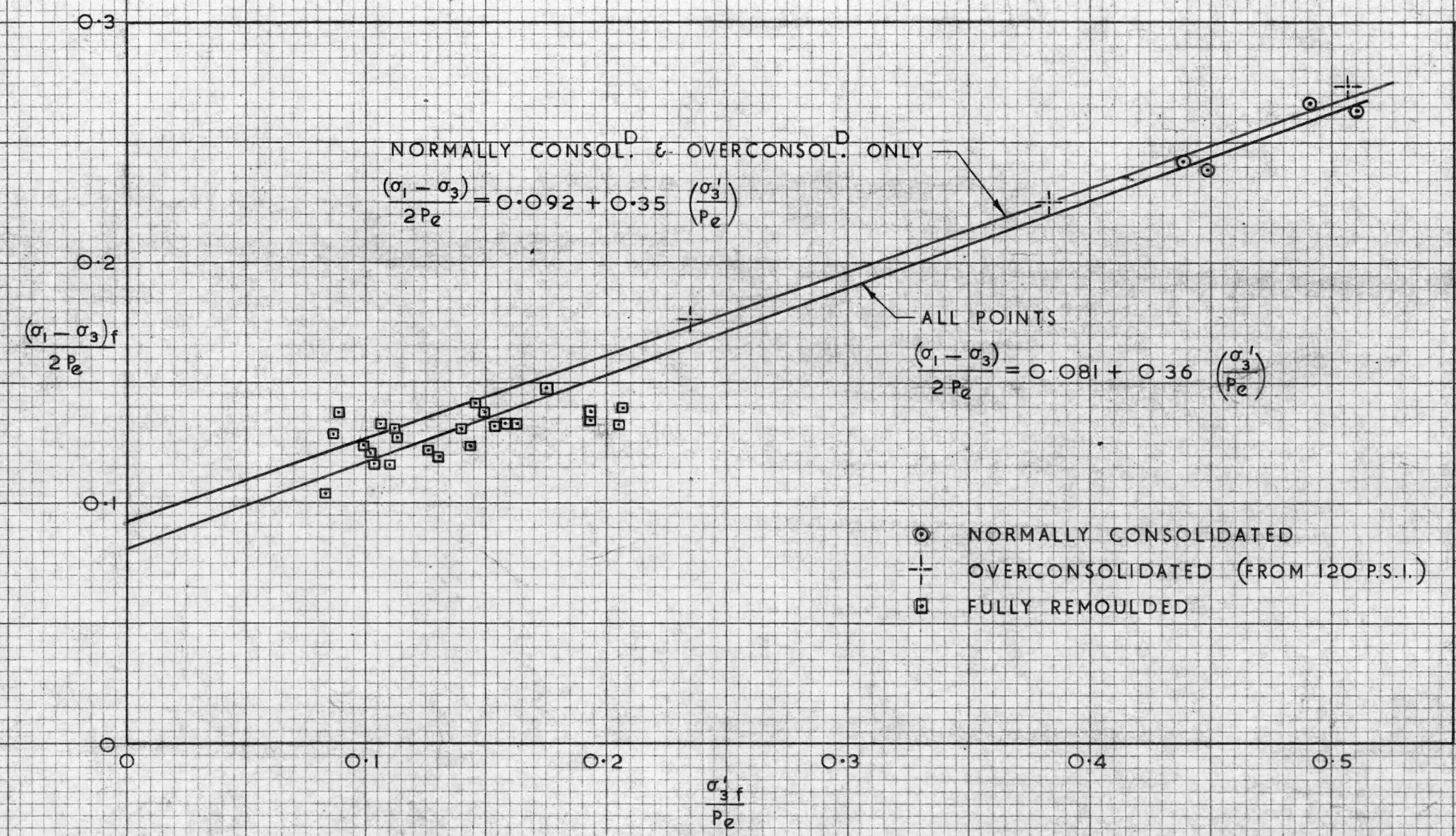
Bjerrum (1954) in an extensive investigation of these parameters used samples consolidated from different initial water contents to obtain adequate separation of failure strengths at a given water content. However his driest remoulding water contents were well above the plastic limit (Liquidity Indices 0.44 - 0.52) whereas the driest samples in the present series had a Liquidity Index of 0.05.

Following Gilbert (1954) the test results for remoulded, normally consolidated and overconsolidated samples have been plotted in FIG. 5.11 as $\frac{(\sigma_1 - \sigma_3)f}{2p_e}$ against $\frac{\sigma_3'{}_f}{p_e}$,

where $\sigma_3'{}_f$ - minor principal effective stress at failure

$(\sigma_1 - \sigma_3)f$ - maximum deviator stress.

Two straight lines were fitted to the results by the method of least squares, one ignoring the fully remoulded tests; the other



DETERMINATION OF HVORSLEV PARAMETERS

FIG 5.11.

including all tests.

Normally consolidated and overconsolidated tests $\frac{(\sigma_1'-\sigma_3')}{2p_e} = 0.092 + 0.35 \frac{\sigma_3'}{p_e}$ ..... (5.4)

All tests $\frac{(\sigma_1'-\sigma_3')}{2p_e} = 0.081 + 0.36 \frac{\sigma_3'}{p_e}$ ..... (5.5)

Gilbert has shown that the slope of the line is $\frac{\sin \phi_r}{(1-\sin \phi_r)}$ and the intercept on the vertical axis is $\frac{c_r \cos\phi_r}{p_e(1-\sin\phi_r)}$.

The values given in Table 5.5. may then be calculated.

TABLE 5.5. HVORSLEV PARAMETERS

	ϕ_r (degrees)	$\frac{c_r}{p_e}$
Without remoulded tests	15.0°	0.070
All tests	15.4°	0.062

The true angle of internal friction is estimated quite closely when the remoulded tests are included and the $\frac{c_r}{p_e}$ ratio is correct to within a tolerable 13%. Bjerrum has shown that unless a clay has been consolidated from near its liquid limit the true cohesion is no longer directly proportional to the equivalent consolidation pressure p_e but is given by

$$c_r = c_o + k.p_e \qquad \text{............ (5.6)}$$

The different structure in remoulded samples, as compared with normally consolidated ones, undoubtedly accounts for the error in true cohesion when estimated by the above method. However, when a rapid estimate of ϕ_r and $\frac{c_r}{p_e}$ is required the effect of remoulding enables it to be made with virtually one sample. The soil is first tested normally consolidated then remoulded without loss of water and again sheared, with pore-pressure measurements. Two points with very good separation are then obtained on a plot like FIG. 5.11. The method would, of course, have to be tested on a wide range of soils and might be seriously in error with some clays.

(5.5) b (iv) Long term effects of remoulding

When the change in remoulded suction with time after remoulding was first noticed it was decided to store some remoulded samples for periods of up to one year before shearing them. Skempton and Northey (1952) had measured a thixotropic strength regain in London clay of 85% in 200 days.

The percentage thixotropic regain is defined by the expression

$$\left(\frac{c_t - c_r}{c_r}\right) \times 100$$

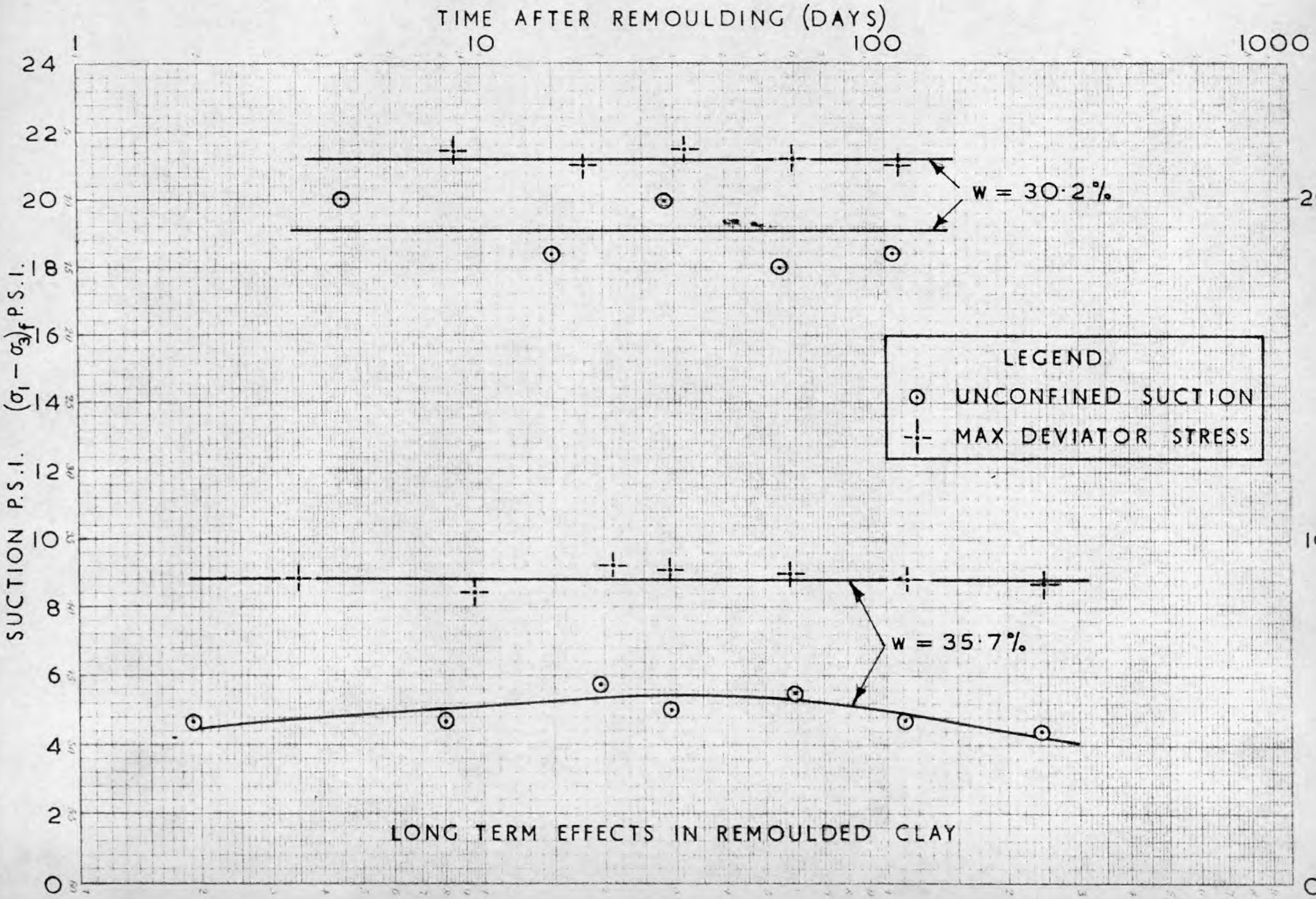
where c_t, c_r = apparent cohesion at time t after remoulding and immediately after remoulding respectively.

For undrained tests on saturated clay

$$c = \frac{(\sigma_1 - \sigma_3)}{2} \qquad \ldots\ldots\ldots\ldots (5.7)$$

Samples were stored, after remoulding, at water contents of 35.7% and 30.2%, i.e. liquidity indices of 0.22 and 0.08. Skempton and Northey presented data for liquidity indices of 1 and 0.55, the strength regain dropping by about 12% over this range. By extrapolation, at a liquidity index of 0.22 the probable percentage thixotropic strength regain is about one half of the value at the liquid limit, i.e. $\frac{85}{2} \simeq 43\%$ It was hoped that such a strength regain would be compatible with long term suction increases in the soil but this was not the case. Fig. 5.12 shows maximum deviator stress and suction prior to compression as functions of the time since remoulding. At the higher water content although the suction changes are small there does seem to be a trend towards an increase in suction for the first month followed by a gradual decrease. The drier samples show more scatter, probably due to the difficulty of completely remoulding the stiff clay, and there cannot be claimed any trend for these points.

For both water contents the failure strengths were independent



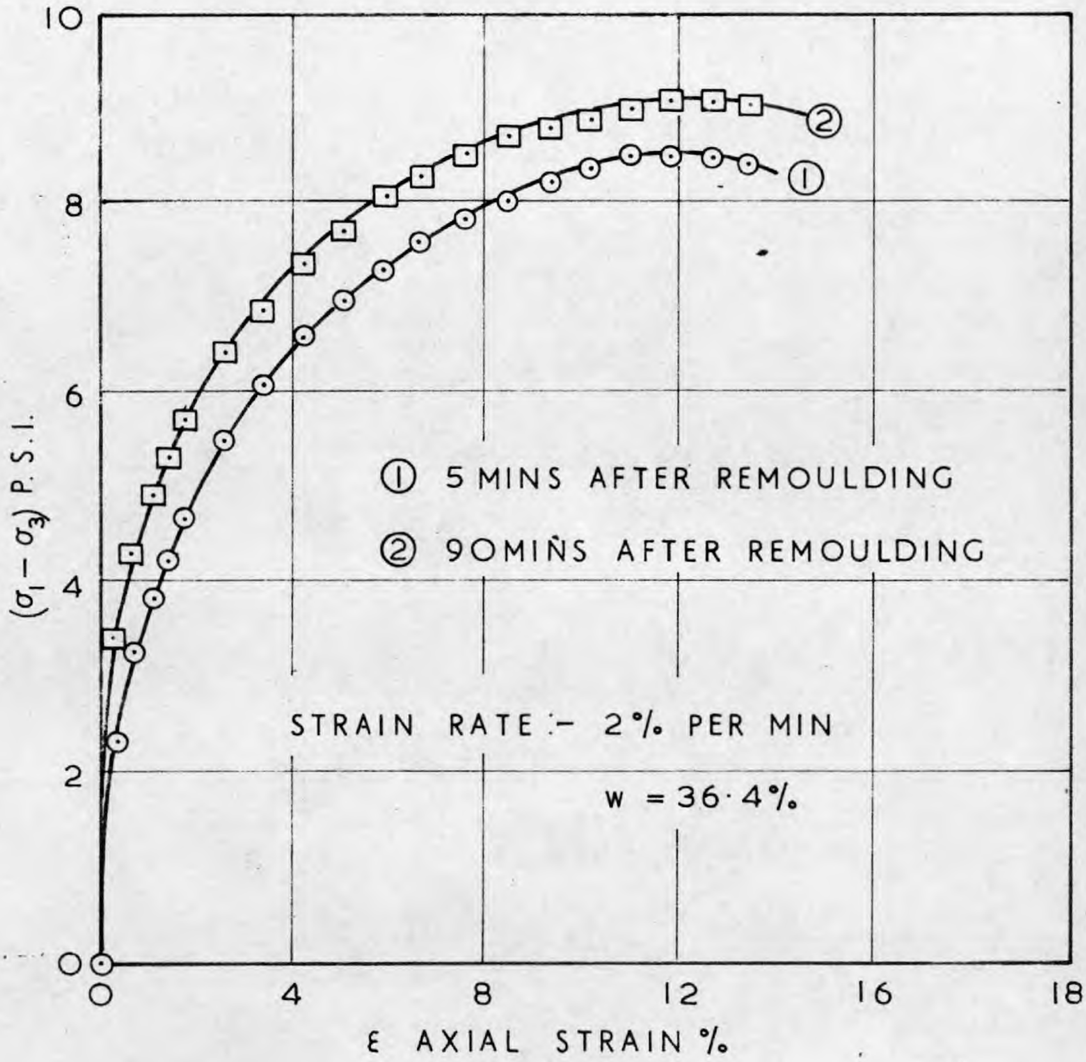
TIME AFTER REMOULDING (DAYS)
1
10
100
1000
24
22
20
18
16
14
12
10
8
6
4
2
0
20
10
0
SUCTION P.S.I. $(\sigma_1 - \sigma_3)_f$ P.S.I.
W = 30·2%
W = 35·7%
LEGEND
UNCONFINED SUCTION
MAX DEVIATOR STRESS
LONG TERM EFFECTS IN REMOULDED CLAY

FIG. 5.12.

of time after remoulding. Any structural changes which may have occurred in the soil would be partly destroyed by the shearing process, but the effect of such changes would be most noticeable at low axial strains. However, the stress-strain and pore pressure-strain curves for all tests at each water content were practically identical and a typical curve from each series has already been presented in FIG. 5.6.

The data of FIG. 5.4 indicates that the strength immediately after remoulding may be temporarily low. Two samples were remoulded from a batch of clay at 36.4% water content, one being sheared five minutes after remoulding, the other 90 minutes after. Each test took approximately seven minutes to complete. The stress-strain curves are plotted in FIG. 5.13. The difference in maximum deviator stresses is small, 0.65 lb/in^2, but appreciable. At failure roughly 12 minutes had elapsed since remoulding the first sample so that even curve (1) was onlyan approximation to an 'immediately after remoulding' test.

For this particular clay there appears to be negligible strength regain at any time of practical interest after remoulding, certainly at field water contents. The reasons for the different behaviour of Skempton and Northey's clay are not obvious. However, their method of storage did not preclude ionic diffusion between the soil and the water in which the samples were stored and this may have influenced their results. No chemical data was given for their soil so comparisons cannot be made on this basis.



LONDON CLAY (REMOULDED)

RAPID UNDRAINED TESTS

FIG. 5.13.

(5.6) Summary - Saturated Soils.

Tests on saturated clays have demonstrated the main principles involved in measuring negative pore pressures in soils and assessing their changes with changing external stress and the effect on soil macro behaviour.

Negative pressures in the range 0 to -13 lb/in^2 can be measured accurately using direct suction techniques and greater suctions can be measured by translating the datum for pressure measurements. The suction $(u_a - u_w)$, is a constant for a soil held at constant $(\sigma_3 - u_a)$ independent of the value of ambient air pressure, u_a. The effective stress - or stress which controls the mechanical behaviour of the soil - is likewise a constant under these conditions.

In saturated soil negative pore water pressures behave exactly the same as positive pore water pressures when the soil is subjected to external stress, i.e. an increase in ambient mechanical pressure produces an equal increase in pore water pressure.

In the generalised effective stress law

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \qquad \text{........ (2.21)}$$

if u_a is regarded as an ambient pressure, then $\chi = 1$ for both volume change and shear strength considerations.

$$\text{i.e.} \quad \sigma' = \sigma - u_w \qquad \text{........ (1.1)}$$

and the conventional effective stress law is valid for negative pore pressures. At present no limit can be set to the suction for which this statement remains true. Experimental evidence from consolidation tests and undrained axial compression tests was adduced in support of eqn. (1.1).

The effective stress law was applied to a number of problems involving negative pore water pressures with considerable success. These studies covered suctions and effective stresses in undisturbed samples, the effect of remoulding on soil strength - including long term effects, and the possibility of determining Hvorslev parameters with a minimum of testing.

It should be emphasised that none of the tests performed actually proved the existence of absolute tensions in the pore water or, indeed, gave any direct expression to the state of the water in the pores. What was shown was that no behaviour investigated was in any way inconsistent with the assumption of normal water in the soil pores, exerting hydrostatic pressure on the soil particles.

CHAPTER 6

EXPERIMENTAL RESULTS AND ANALYSIS - PARTLY SATURATED SOILS

(6.1) Introduction

The previous chapter dealt with tests on saturated soils and some of the results presented could have been predicted from previous knowledge using the single extra assumption that the effective stress law was valid for negative pore water pressures. The more interesting, general and unpredictable problem of the behaviour of soils with degree of pore space saturation, S_r, less than 100% is treated in this chapter. The main emphasis is on testing the validity of Bishop's effective stress law:

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \qquad \text{.......... (2.21)}$$

particularly as it applies to shear strength phenomena, but volume change-suction relationships and general pore pressure phenomena are also investigated.

(6.2) Soil Properties

Two soils were used in these investigations and their properties are summarised in Table (6.1) and FIG. 6.1.

RICHARD COSTAIN LTD. SOIL MECHANICS DEPT.

Particle Size Distribution

Location .... Boring No. .... Sample No. .... Date of Test ....

Description ① BRAEHEAD SILT. ② TALYBONT CLAY

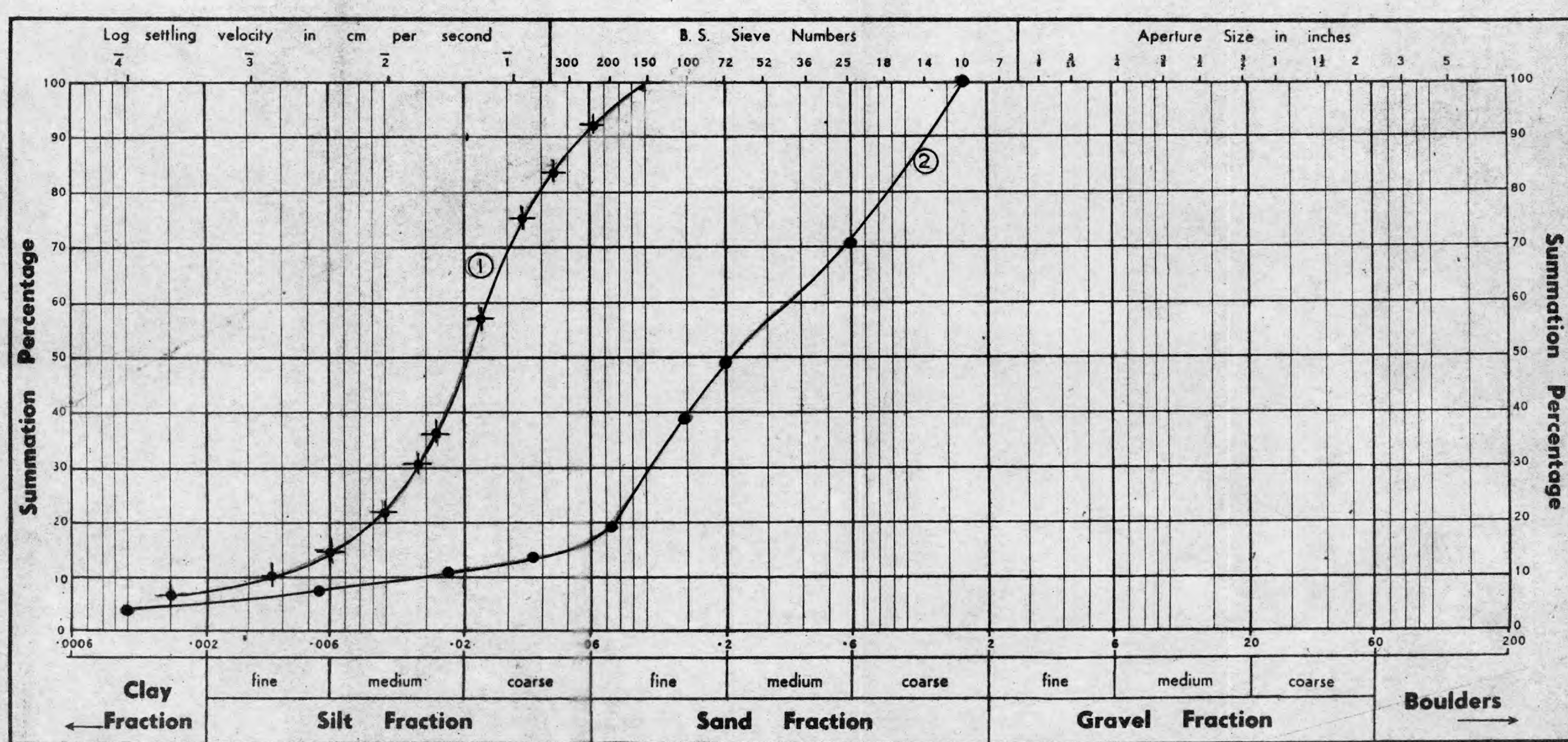


FIG. 6.1.

TABLE 6.1 INDEX PROPERTIES

	L.L.%	P.L.%	P.I.
Braehead Silt	29	23	6
Talybont Boulder Clay	22	-	-

Capillary effects are inversely proportional to particle diameter and a silt was chosen for intensive testing as it retained an appreciable amount of water at a suction of one atmosphere while its lack of true cohesion simplified the result analyses. The behaviour of a similar silt when saturated has been studied by Penman (1953).

The Talybont Boulder Clay was used for studying pore pressure phenomena during all round compression and true undrained compression with constant lateral stress σ_3 in the triaxial cell. The same soil (but including the fraction retained on B.S. No.10 sieve) had been extensively tested by Alpan (1959) but some queries had arisen concerning certain of his results and they were checked with improved techniques. Further data on the soil as tested by Alpan has been obtained by Blight (1961).

(6.3) Pore Pressure Phenomena

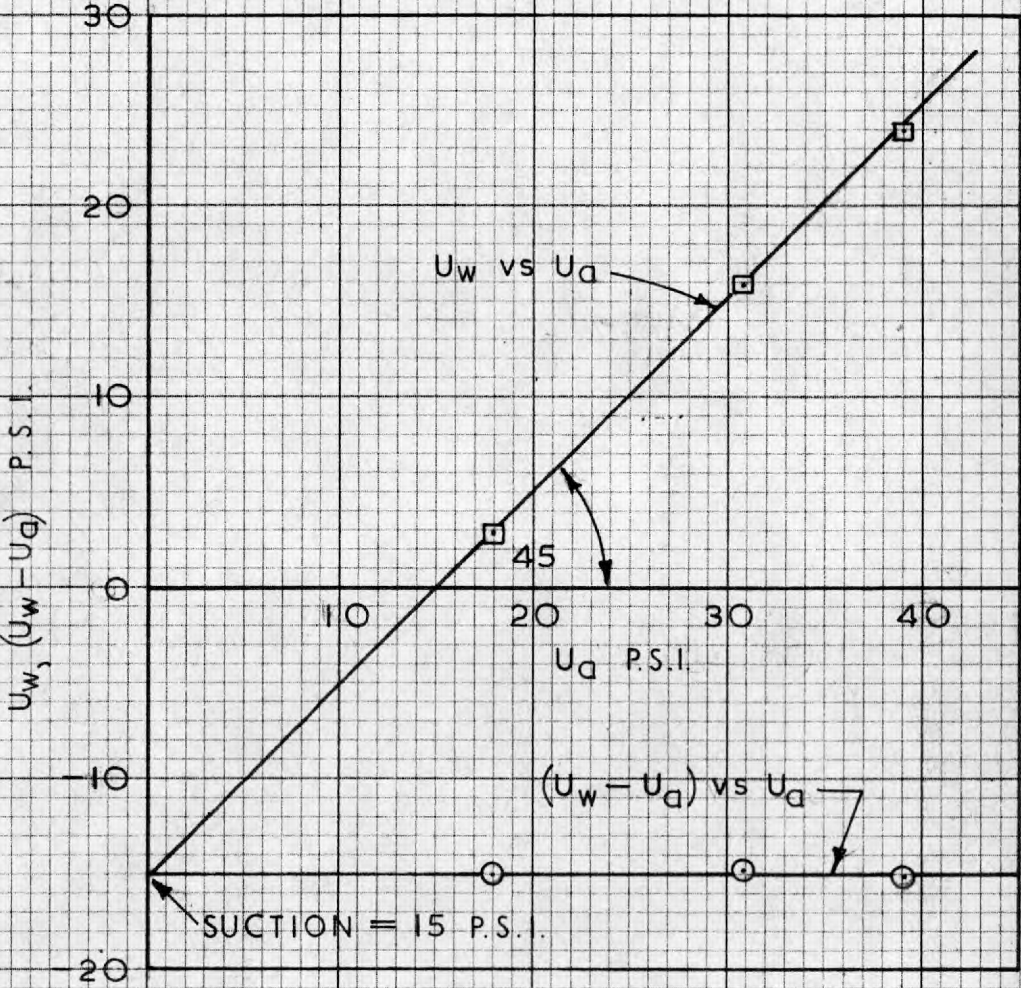
(1) Translation of Origin.

In section (5.2) the effect of an increase in ambient air pressure u_a on a saturated soil was shown to be an equal increase in pore water pressure. In FIG. 6.2. the results of a test on Braehead silt with S = 41% are presented. The soil was initially at equilibrium under σ_3 = 20 lb/in^2, u_a = 18 lb/in^2 and u_w =3lb/in^2. The air pressure was slowly increased in two increments to 39 lb/in^2, ($\sigma_3 - u_a$) being kept constant at 2 lb/in^2 and u_w measured at the base of the sample. The pore water pressure changed by an amount equal to the increment in air pressure, i.e. ($u_a - u_w$), the suction, remained constant.

The use of elevated air pressures for measuring suctions greater than one atmosphere was pioneered in soil mechanics by Hilf (1956), who tested compacted soils. Alpan (1959) used the method to keep the pore water pressures in compacted Talybont clay in the measurable range for the duration of a triaxial compression test.

The data of FIG. 6.2 provides indirect evidence for the correctness of the form of equation (2.21). If ($\sigma_3 - u_a$) and ($u_a - u_w$) are both constant in spite of changes in σ_3, u_a and u_w then the effective stress σ' should also be constant. Any change in effective stress in a compressible soil would result in a change of suction, but such a change was not observed in the test.

Hence it has been proved that for both saturated and partly saturated soils the effect of an increase in u_a, $\sigma_3 - u_a$ being kept constant ($\sigma_3 = u_a$ for the saturated samples) is merely to raise the



BRAEHEAD SILT $S_r = 41\%$

TRANSLATION OF ORIGIN

FIG 6.2.

datum from which stresses are measured.

(6.3) 2. Initial pore pressures in compacted soil

Until the work of Hilf (1956) the differences between pore air and pore water pressures in compacted soils were neglected or assumed zero. Hilf himself expressed surprise at measuring a moderate suction on inserting a pore pressure probe into his first unconfined sample, though his surprise would probably be surprising itself to a soil physicist!

Most soil mechanics measurement of pore pressure prior to 1956, and a good deal of it since, was done using coarse porous discs at the sample ends. It is now apparent that for all but the very wettest soils the apparatus must have been recording pore air pressure. As will be shown the errors in effective stress calculated from

$$\sigma' = \sigma - u_w \qquad \text{............ (1.1)}$$

using u_a in mistake for u_w are appreciable except for wet soils at moderately high cell pressures and much reported data on compacted soils will have to be regarded with suspicion.

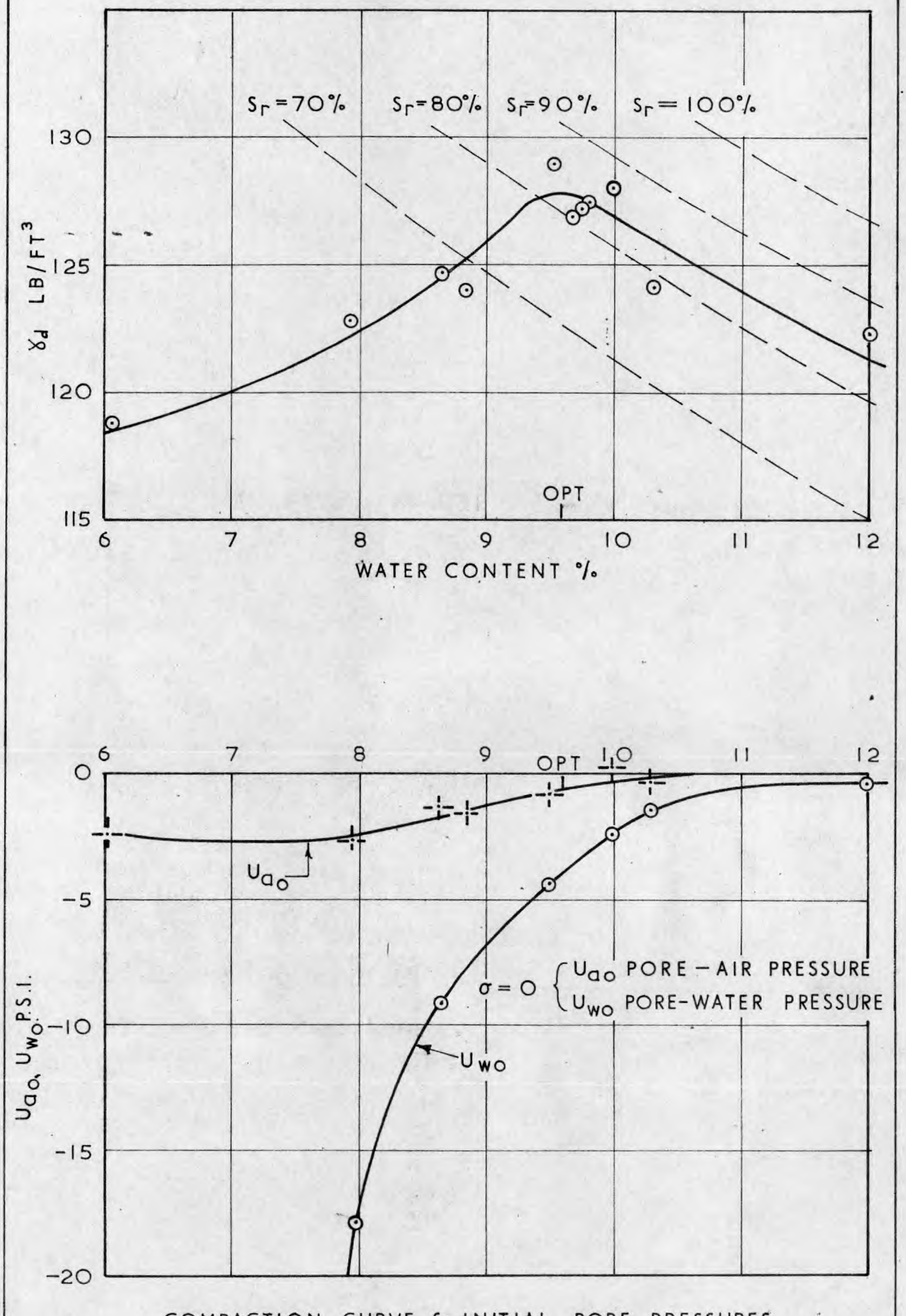
Hamilton (1939), investigating the effect of gaseous solubility on pore pressure changes under all round compression assumed that the initial pore pressure, air or water, was zero. Alpan (1959) has quoted values for initial equilibrium air pressures in unconfined samples of -5.5 lb/in^2 at water contents two percent dry of the optimum, which could help to explain why Hamilton had difficulty in matching his experimental results with theory.

However it now appears that Alpan's initial pore pressures were at least partly controlled by diffusion of pore air through the rubber membrane into the cell water. The order of magnitude of the possible error in pore air pressure may be judged by reference to FIG. 12 of the paper by Bishop et al (1960). In this shear test although the sample volume decreased continuously the pore air pressure rose to a maximum and then fell steadily. Without air loss through the membrane u_a should have increased steadily as a reflection of the volume change. The probable error in true undrained u_a at failure is about 6 lb/in^2. For samples equilibrating under zero confining pressure the errors will undoubtedly be smaller but will depend on the degree of air saturation of the cell water at the true equilibrium value of u_a.

The mercury cell apparatus described in chapters 3 and 4 was used to check Alpan's values and the results are given in FIG. 6.3.

The compaction dry density - water content curve differs slightly from Alpan's as a result of the lower clay content and higher coarse fraction in his sample. The optimum water content was 9.6% compared with Alpan's 9.3%.

The initial pore air and water pressures u_{a_o} and u_{w_o} could not be



COMPACTION CURVE & INITIAL PORE PRESSURES
FOR UNCONFINED TALYBONT CLAY

FIG. 6.3.

obtained directly as the minimum agerage lateral pressure in the mercury cell was 1.2 lb/in^2. Most samples were subjected to several increments of all round pressure to provide the results of the next section and the pore pressures at zero confining pressure were obtained by extrapolation. Two days were found to be sufficient for equilibrium after initial placing of the sample in the cell.

The initial pore air pressures are definitely below atmospheric but only by roughly half the amount reported by Alpan. When first placed under mercury the samples exhibited air pressures between 0 and +1.5 lb/in^2 and the approach to equilibrium occurred with a very slight overall volume decrease. The reason for the equilibrium initial negative u_a is obviously not release of compaction stresses which would require a volume increase.

A tentative explanation is that, immediately after compaction, at least some of the air is present in the pores in the form of isolated bubbles. The initial air pressure in each bubble will depend on its diameter and the pressure in the surrounding water. FIG. 6.3 shows that the water pressure is considerably below atmospheric at most water contents and hence the pressures in all but the smallest bubbles will also be sub-atmospheric. The air pressure measuring system will record at first the pressure in the coarser interconnected air filled voids but, as time passes, the air pressure will equalise throughout the sample by diffusion and the final pressure must be negative (gauge).

The rate of solution of air bubbles and the possible equilibrium configurations of air in partly saturated soils were briefly investigated in collaboration with G.E. Blight and some results are given in Appendix 3.

The pore water pressure decreases very rapidly with decreasing water content and at only 1½% dry of optimum it is already beyond the limit of direct measurement. The $(u_w - u_a)$ of -15.2 lb/in^2 for the sample with w = 7.95% was measured in a triaxial cell, without mercury, using an elevated pore air pressure. The equilibrium air pressures for this and the drier sample were measured in the mercury cell with the base of the sample resting on a solid metal disc to prevent cavitation in the water in the cell base. The vertical difference between the u_{a_o} and u_{w_o} curves at any water content is a measure of the unconfined soil suction.

Several interesting features emerge from FIG. 6.3. The most important is the magnitude of the suction or pressure difference, $(u_a - u_w)$, which had been neglected in earlier soil mechanics work. The curves shown in FIG. 6.3 differ appreciably from Alpan's and, apart from the influence of air diffusion through the membrane, Alpan's results appear to give suctions that are numerically too small at water contents lower than about 1% dry of optimum. The possible reason is a breakdown in his measuring system at $u_w \simeq -15$ lb/in^2. The compaction water content versus unconfined suction curve obtained by Blight (1961) on Alpan's material is essentially in agreement with FIG. 6.3. Using very fine porous ceramics Blight has measured suctions of up to 70 lb/in^2 in compacted materials. One soil with 20% clay fraction had a suction of 40 lb/in^2 even at the optimum water content and it is quite obvious that almost all early workers on pore pressures in compacted soils must have been measuring pore air pressures and not pore-water pressures as intended. Alpan (1959) who did some of his tests on coarse porous stones provided experimental proof of this.

The appreciable value of initial negative pore air pressure also represents a source of error in some previous work (e.g. Hamilton, 1939) and limits the lowest value of water content at which truly undrained tests may be carried out without breakdown of the measuring system.

The pressure difference is only 3.3 lb/in^2 at the optimum water content dropping to 1.0 lb/in^2 at 1% wet of optimum and approaching zero asymptotically at higher water contents.

It is instructive to examine the errors in effective stress for this soil arising from using the air pressure instead of the water pressure, or from ignoring the u_a term altogether. For the purpose of these calculations the χ factor has been taken numerically equal to S (ratio).

TABLE 6.2

EFFECTIVE STRESSES IN UNCONFINED COMPACTED BOULDER CLAY

(i.e. $\sigma = 0$)

w%	u_{a_o} lb/in²	u_{w_o} lb/in²	$u_a - u_w$	$\chi(u_a - u_w)$	$(\sigma - u_w)$	$(\sigma - u_a)$	$\sigma - u_a + \chi(u_a - u_w)$
8	-2.5	-16.8	14.3	8.4	16.8	2.5	10.9
9	-1.4	- 6.9	5.5	3.2	6.9	1.4	4.6
10	-0.4	- 2.5	2.1	1.2	2.5	0.4	1.6

Taking the last column as the true effective stress in the soil the unwitting use of $(\sigma - u_a)$ is seen to underestimate the effective stress by a factor of approximately four at all water contents. The $(\sigma - u_w)$ expression, as used by Hilf (1956), is also seriously in error at all water contents if considered as a percentage of the true effective stress. The unconfined case is obviously the worst possible and the error in effective stress will decrease with increasing cell pressure for two reasons - the contribution of the suction forces is now only a part of the total effective stress and the $(u_a - u_w)$ term will decrease with increase in all round stress as described in the next section.

(6.3) 3. Pore Pressure Parameters B_a and B_w.

As mentioned in Chapter 2 the pore pressure parameter B has two counterparts in partly saturated soil, B_a and B_w, where

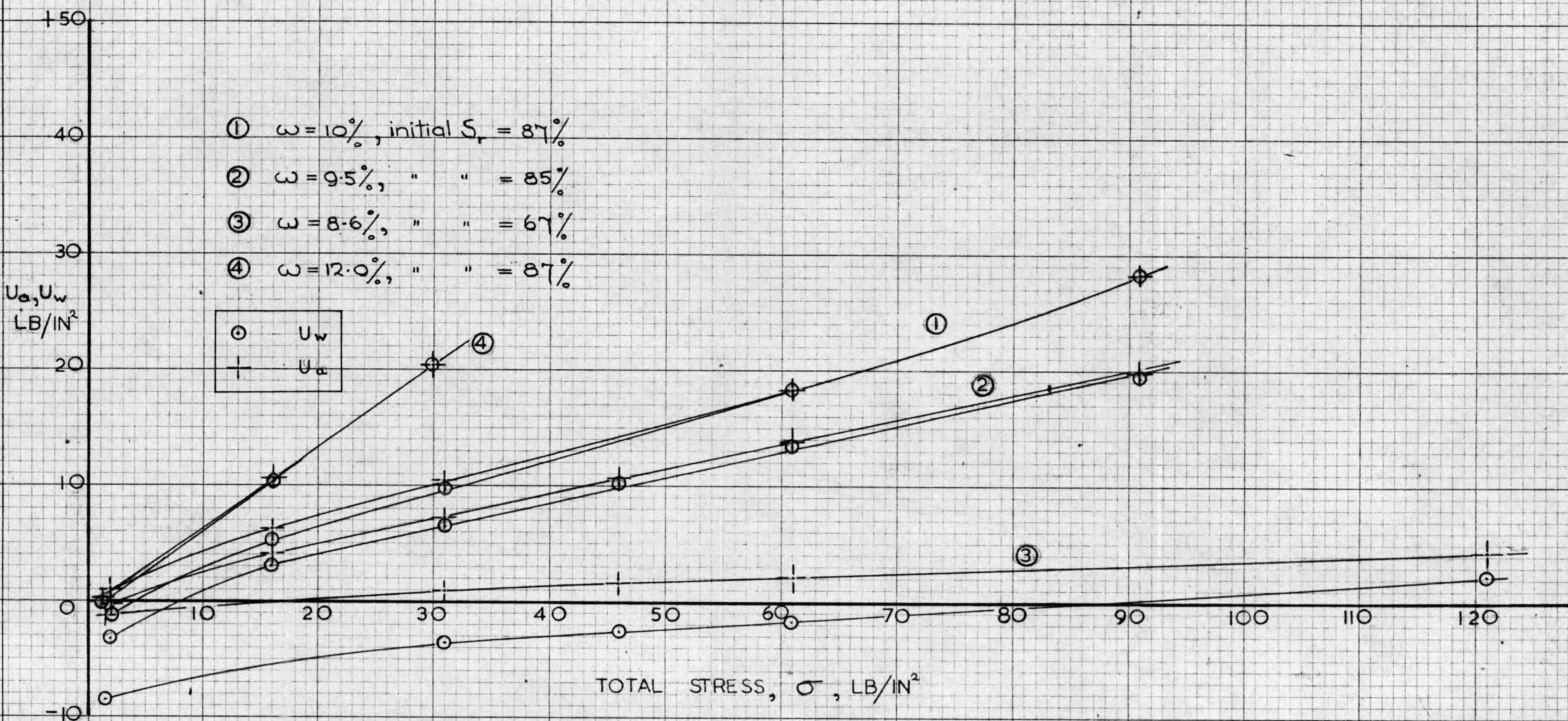
$$B_a = \frac{\Delta u_a}{\Delta \sigma} \qquad \text{.............} (2.8)$$

$$B_w = \frac{\Delta u_w}{\Delta \sigma} \qquad \text{.............} (2.9)$$

An attempt to measure these parameters for compacted Talybont clay was made by Alpan (1959) but; as most of his pressure increments were left overnight, the diffusion process reduced his measured changes and gave fallaciously low B values, about 0.065 at the optimum water content.

The results of some all round compression tests on compacted Talybont clay are given in FIG. 6.4 and the corresponding B values are plotted in FIG. 6.5, calculated for a total stress increment from 0 to 30 lb/in^2. The pressure difference ($u_w - u_a$) is plotted as a function of ($\sigma_3 - u_a$) in FIG. 6.6 for tests at two water contents.

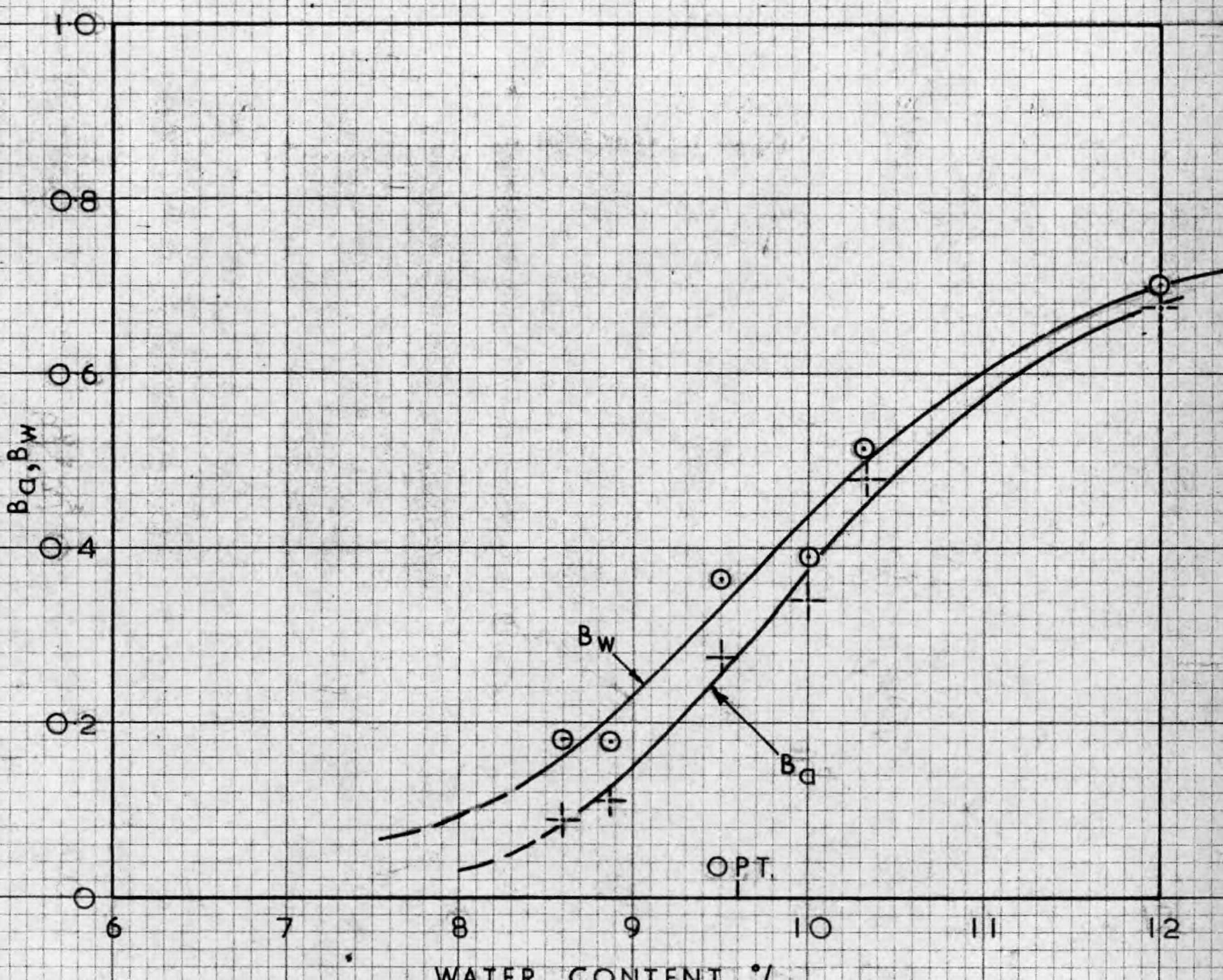
FIG. 6.4 shows that for samples dry of optimum much of the initial difference ($u_a - u_w$) is maintained even at high cell pressures, but for the sample 0.4% wet of the optimum the difference has disappeared by the time the cell pressure reaches 60 lb/in^2. This does not mean that the soil has become saturated, for B does not become 1.0, but



TALYBONT BOULDER CLAY

PORE PRESSURE CHANGES UNDER UNDRAINED ISOTROPIC COMPRESSION

FIG. 6.4



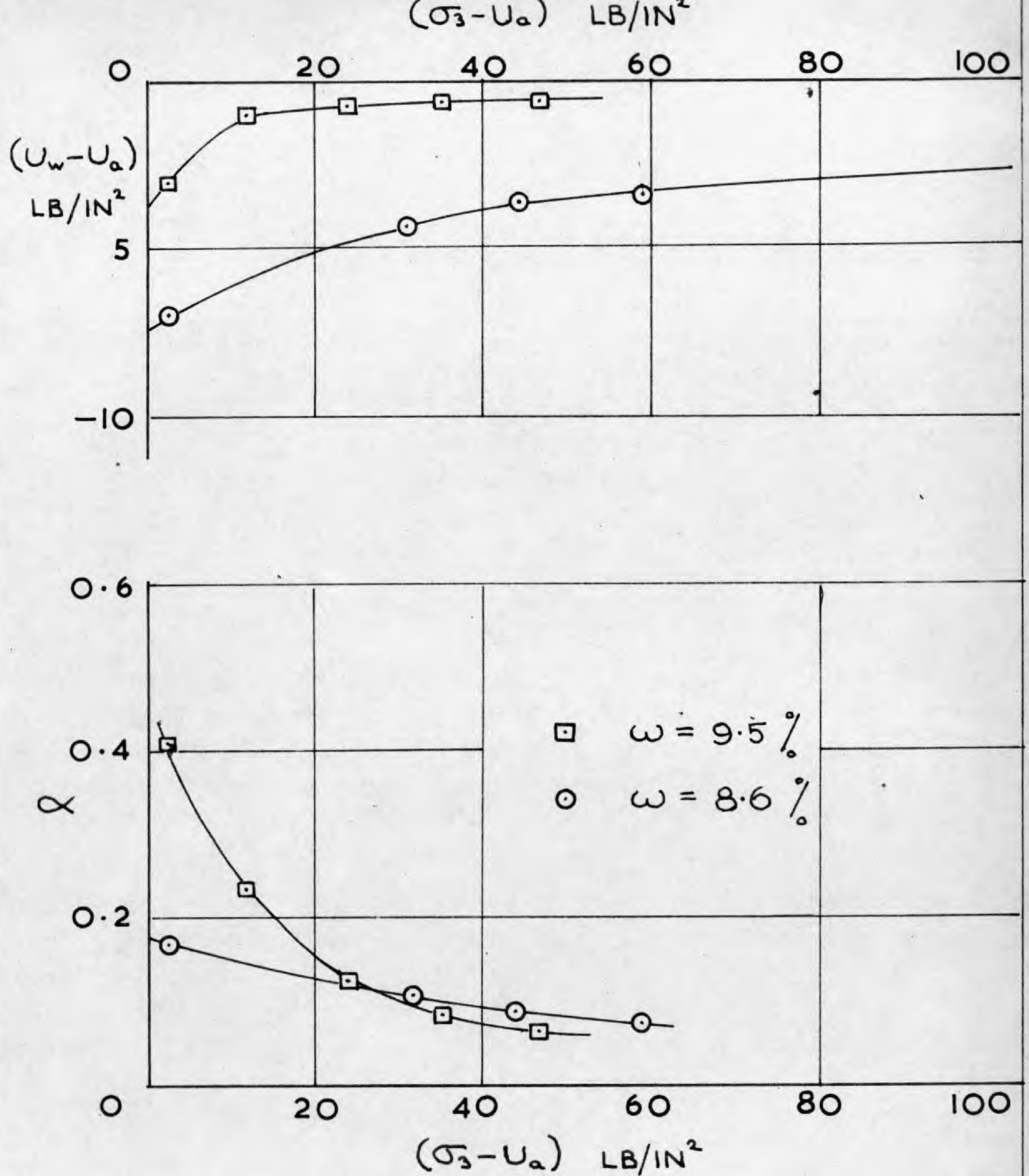
TALYBONT CLAY

PORE PRESSURE PARAMETERS

B_a & B_w, ISOTROPIC COMPRESSION

PRESSURE INCREMENT 0 – 30 P.S.I.

FIG 6.5.



TALYBONT — VALUES OF α IN

$$\Delta(U_w - U_a) = \alpha \cdot \Delta(\sigma_3 - U_a).$$

FIG. 6.6

water has entered the air pressure connexion and the air remaining in the sample is in isolated bubbles and voids at a pressure very close to that of the pore water. The low α values over the whole $(\sigma_3 - u_a)$ range for the drier sample in FIG. 6.6 also indicate that $(u_w - u_a)$ is insensitive to changes in cell pressure for moisture contents appreciably below the optimum.

Alpan, in his FIG. 6.3, shows values of B_a and B_w with B_a greater than B_w at dry water contents and the two values converging at the optimum water content. It is difficult to justify this on logical grounds as $(u_a - u_w)$ undoubtedly decreases with increasing lateral stress and Δu_w must therefore always be greater than Δu_a, i.e. $B_w > B_a$. This is consistent with the behaviour depicted in FIG. 6.5. B_a approaches B_w on the wet side of optimum water content but can really never equal it if there is a measurable initial difference $(u_a - u_w)$. Even in samples considerably wet of the optimum B is appreciably less than its maximum value of 1 which is only reached at complete saturation. The combination of fine porous disc for pore water pressure measurement and mercury cell for the prevention of air loss by diffusion has made it possible to carry out these tests, which are quite possibly the first truly undrained tests to be done within the field of soil mechanics.

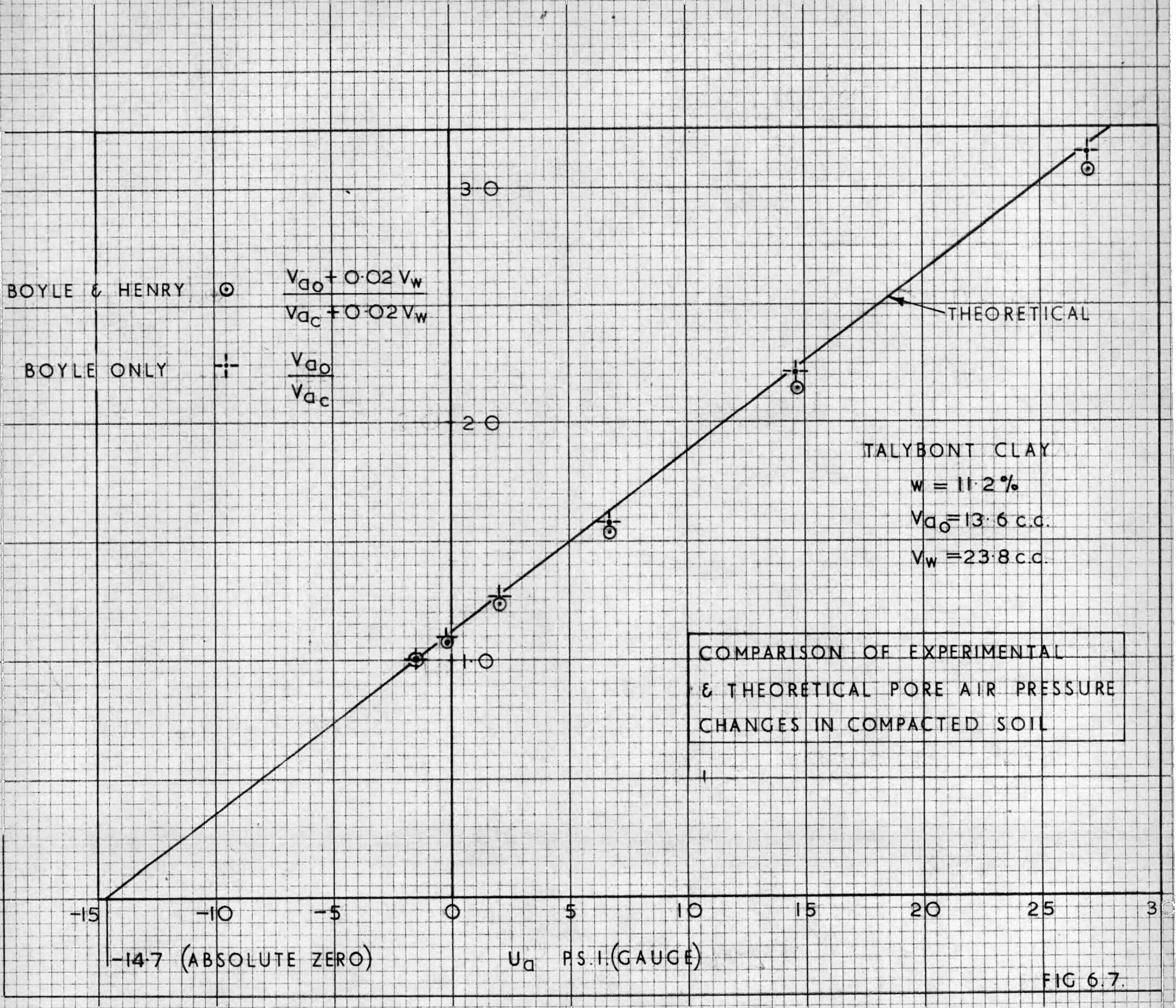
(6.3). 4. Calculation of B_a and experimental verification

In section (2.3) the various methods for calculating changes in pore air pressure given the sample volume change and initial void ratio and degree of saturation were discussed. Hamilton (1939) attempted to verify his result experimentally but failed for several reasons - his apparatus could not prevent air diffusion, he assumed initial atmospheric pressure in the voids and neglected the difference between air and water pressures.

The mercury cell has now made it possible to check the theory more accurately and several tests were done in which samples of Talybont clay were subjected to increments of total stress, the pore pressure and volume changes being recorded. The method of result presentation in FIGS. 6.7 and 6.8 is the same as that used by Hamilton.

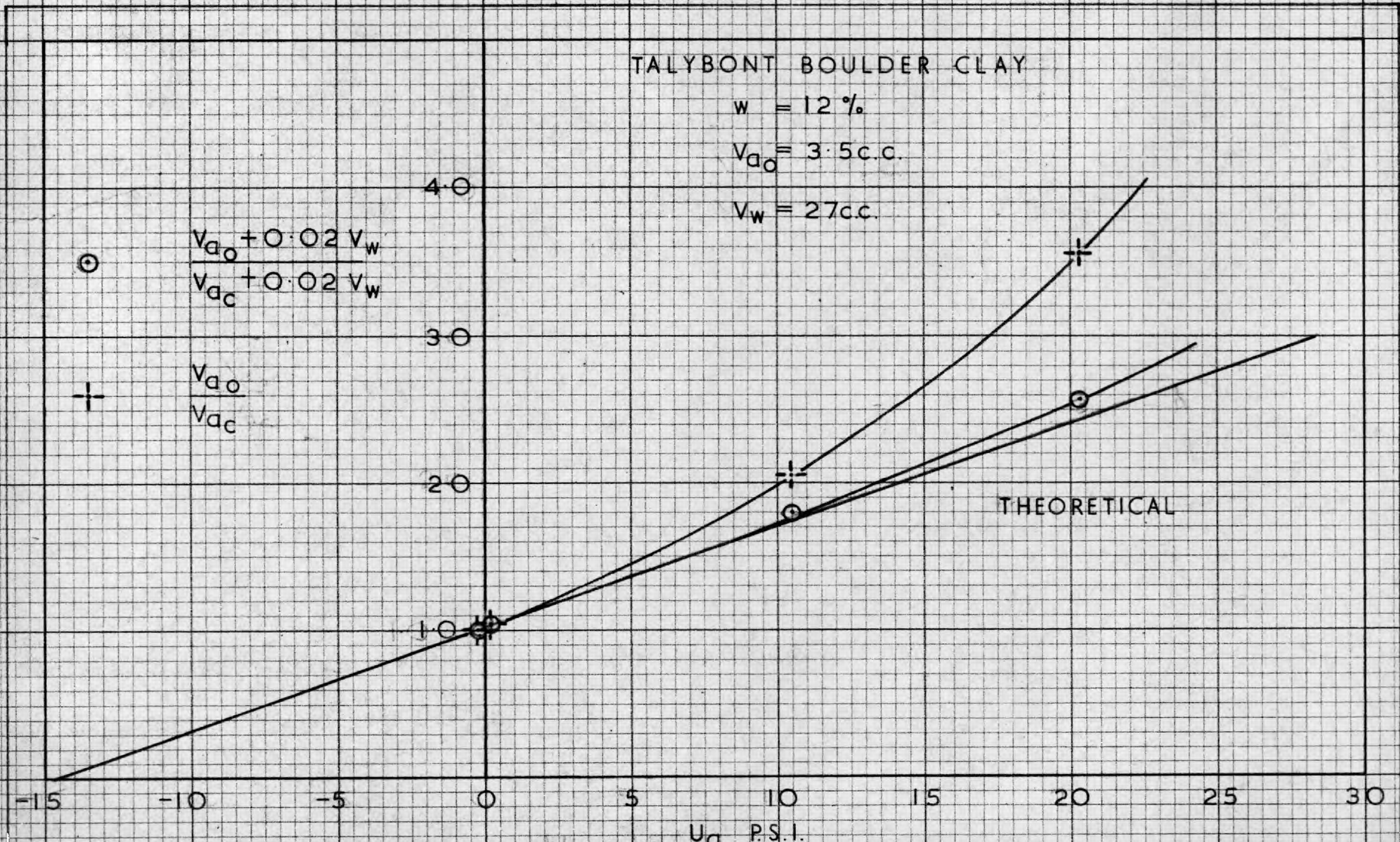
The theoretical line passes through absolute zero pressure (-14.7) lb/in^2 on the u_a axis and the point representing the initial sample conditions (i.e. u_o from extrapolation to σ_3=0) for which both $\frac{V_{a_o} + 0.02\, V_w}{V_{a_c} + 0.02\, V_w}$ (Boyle's law and Henry's law) and $\frac{V_{a_o}}{V_{a_c}}$ (Boyle's law only) are equal to 1.0. The same theoretical line is obtained on this type of plot whether or not the effect of gaseous solubility (Henry's law) is included in the argument.

FIG. 6.7 represents a sample in which the initial air volume was of the same order of magnitude as the pore water volume. This could



3·0
2·0
1·0
BOYLE & HENRY ⊙ $\frac{V_{a_o} + 0{\cdot}02\,V_w}{V_{a_c} + 0{\cdot}02\,V_w}$
BOYLE ONLY $\frac{V_{a_o}}{V_{a_c}}$
THEORETICAL
TALYBONT CLAY
w = 11·2 %
V_{a_o} = 13·6 c.c.
V_w = 23·8 c.c.
COMPARISON OF EXPERIMENTAL & THEORETICAL PORE AIR PRESSURE CHANGES IN COMPACTED SOIL
-15
-10
-5
0
5
10
15
20
25
3
-14·7 (ABSOLUTE ZERO)
U_a P.S.I. (GAUGE)

FIG 6.7.



THEORETICAL & EXPERIMENTAL AIR PRESSURE CHANGES not have been obtained using normal compaction techniques and this sample was lightly hand compacted into a 1½" dia. tube. Naturally, the sample surface contained many large voids and the initial air volume, calculated from extrapolation of data, must be regarded as approximate only. However, the results show that there is little error caused by neglecting the solubility term as both sets of points fall close to the theoretical line.

FIG. 6.8 represents the opposite extreme where the air void volume is very small compared with the pore water volume. This was obtained by using the compaction technique described in Chapter 4. In this case the solubility term has a profound effect on the pore pressures, calculations based on Boyle's law only grossly ~~under~~overestimating u_a at high lateral pressures. The agreement between theory and experiment is satisfactory for calculations based on the Boyle and Henry laws as the small initial air volume is difficult to measure in 1½" dia. x 3" high samples and the value of $\dfrac{V_{a_o} + 0.02V_w}{V_{a_c} + 0.02V_w}$ is very sensitive to small errors in V_{a_c} at high pressures.

FIGS. 6.7 and 6.8 may be said to justify the theory normally used for predicting pore pressures in compacted fills (for example, see Gould, 1959) with the reservation that due consideration be given to the $(u_a - u_w)$ term and its changes with increasing total stress. If the initial air volume is comparable with the water volume then simpler calculations based on Boyle's law only are acceptable.

To round off this section an extension of TABLE 6.2 is given below to indicate the probable errors in effective stress for Talybont Boulder clay subjected to typical values of total stress σ.

TABLE 6.3

w%	σ lb/in^2	$(\sigma - u_a)$ lb/in^2	$(\sigma - u_w)$ lb/in^2	$(\sigma - u_a) + \chi(u_a - u_w)$ lb/in^2
8.6	0	1.6	9.0	6.5
	30	29.1	33.7	32.2
	60	58.0	61.8	60.6
9.5	0	1.0	4.6	4.1
	30	23.0	23.8	23.7
	60	46.4	47.1	47.0

* taking χ = S.

For the drier sample both $(\sigma - u_a)$ and $(\sigma - u_w)$ are appreciably in error as compared with the final column, the 'generalised' effective stress - although at pressures greater than 30 lb/in^2 $(\sigma - u_w)$ gives a sufficiently accurate value for non research testing. The sample compacted close to the optimum water content shows good agreement between all three calculated stresses except at very low applied pressures. This explains why some workers obtained reasonable results even though they were probably measuring air pressures in mistake for pore water pressures. However, it must be noted that Talybont boulder clay has really only a low percentage of clay sized particles and many compacted soils will exhibit greater $(u_a - u_w)$ values at all water contents with correspondingly greater errors in effective stresses calculated from the 'conventional' $(\sigma - u_w)$ formula.

The errors in effective stress calculated during shear are even more important and a short series of undrained compression tests (under mercury) was carried out on Talybont clay. These tests were done at the very end of the experimental programme and a brief account is given in Section 6.5.3.

(6.4) Effective stresses and volume change

(6.4) 1. Although Terzaghi (1925) realised that volume decreases in desiccated soils could probably be correlated with effective stress changes if the large negative pore water pressures could be measured there appears to be little experimentation reported in the literature. Evidence for this correlation in saturated clays has been presented in section 5.4. Very high suctions are necessary to cause appreciable unsaturation in clay soils and no direct experiments were performed for this thesis. However, data collected by Croney, Coleman and Russam (1953) enables indirect estimates of the χ factor to be made and the method is discussed below.

(6.4). 2. <u>The χ factor from volume changes</u>

The principle of inferring effective stresses caused by suction forces in soils may be outlined as follows:

(a) The relationship between void ratio and effective stress for equal all-round pressure is derived from consolidation tests on saturated soil with zero or positive back pressure in the pore water.

(b) For any other mechanism which produces volume change in the soil the effective stress at any void ratio, provided the soil is at equilibrium, is taken as the effective stress associated with that void ratio in the consolidation test (a).

Allowance must be made for stress history if the soil has been subjected to cycles of loading and unloading or drying and wetting.

The Road Research Laboratory data presents soil moisture characteristic curves and shrinkage curves for natural samples of a wide range of clay soils. The curves for Gault clay are reproduced at the top of FIG. 6.9. The method of calculation of the χ factor is indicated in the lower diagram. While the soil remains saturated the void ratio - pF curve is a straight line and, since χ is 1 over this range, this curve may also be taken as the consolidation e - log σ'

relationship. This initial straight section of the plot has been extrapolated to higher pressures and assumed to represent the true e - log σ' relationship for the soil.

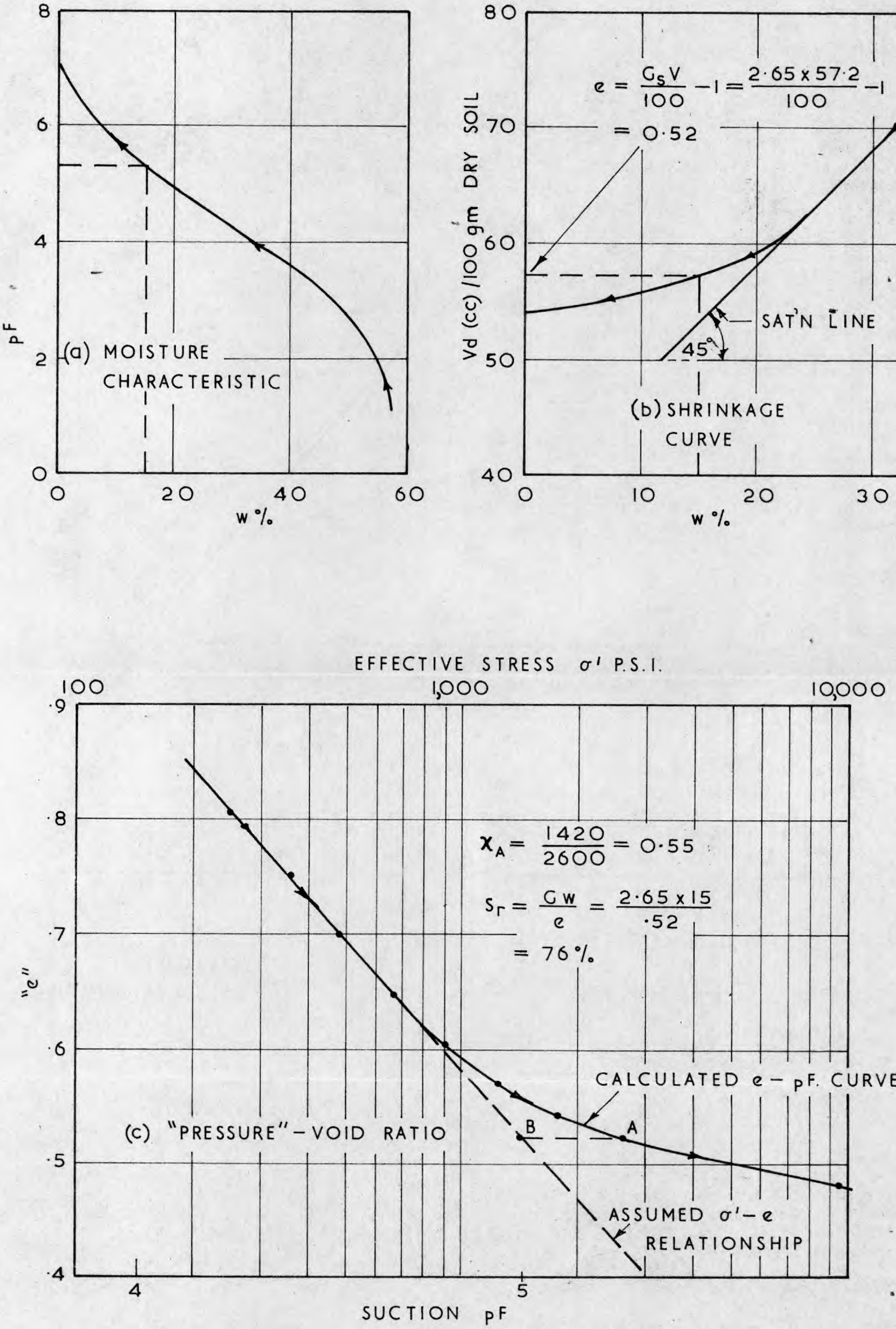
The justification for this has been provided by the work of Parasnis (1960) who showed that the void ratio - log. effective stress plots for Globigerina Ooze, Terrigenous mud (both deep sea sediments), Gault clay and Keuper marl were virtually linear up to pressures of 700 atmospheres.

The χ - S_r relationships deduced for four typical clays from the R.R.L. data are shown in FIG. 6.10. The most striking feature of these graphs is the low values of χ for all but the highest degrees of saturation. There is a surprising unanimity between the curves on this point. The general shapes of the χ - S plots are very different from the unique theoretical relationship derived in Chapter 2.

It is probable that once some air has entered the soil pores much of the further shrinkage takes place in regions which have remained saturated, creating some larger pores but having little effect on the overall volume. Similar χ - S curves to those in FIG. 6.10 were obtained by Blight (1961) from shear tests on compacted soils, though for a smaller range of degrees of saturation.

(6.4). 3. <u>Volumetric test - Braehead Silt</u>

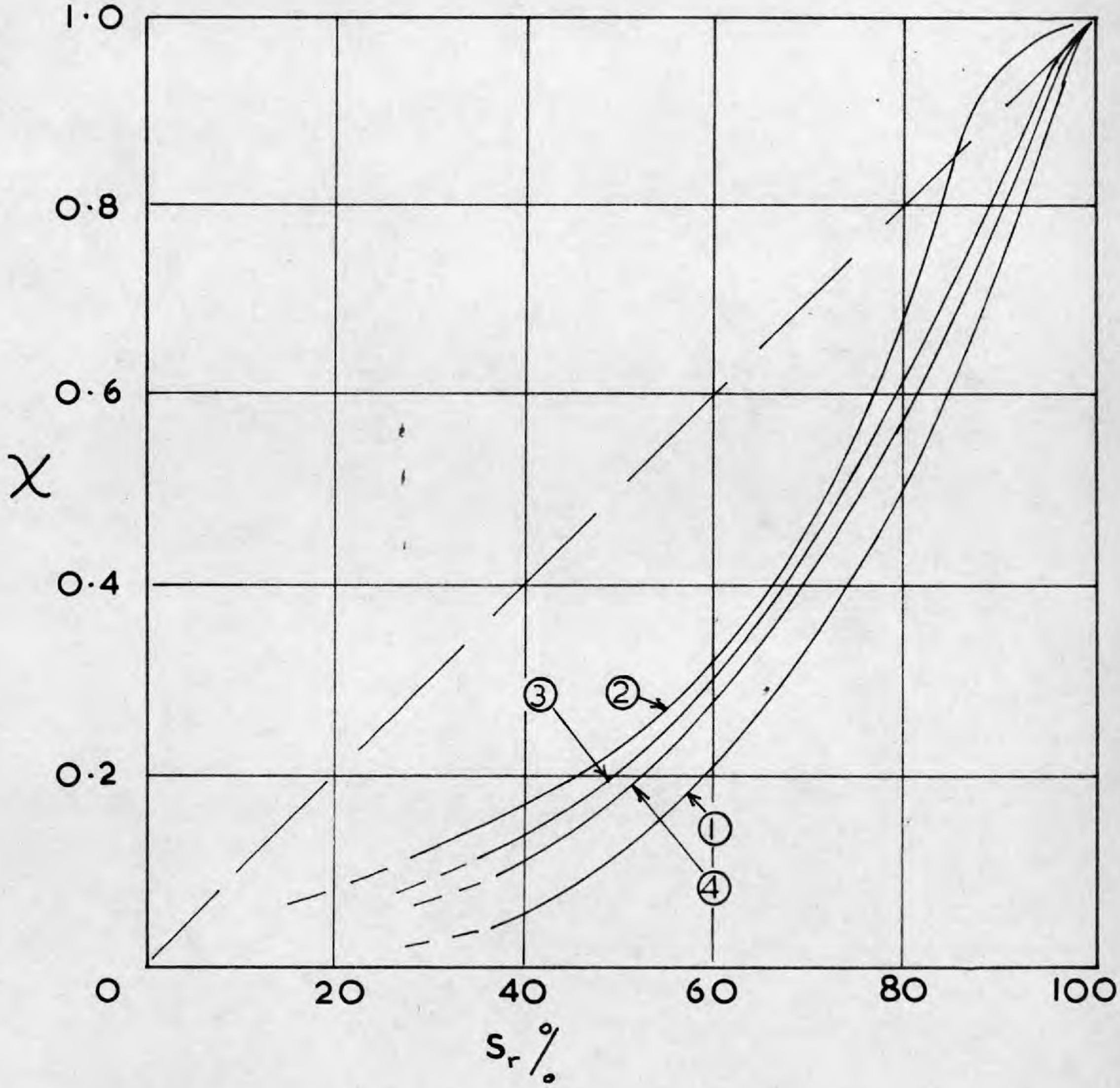
The results of a volume change test on Braehead silt are given in FIG. 6.11 and the method of χ calculation is indicated on the diagram.



GAULT CLAY SHAVES WOOD

DETERMINATION OF χ—Sr RELATIONSHIP.

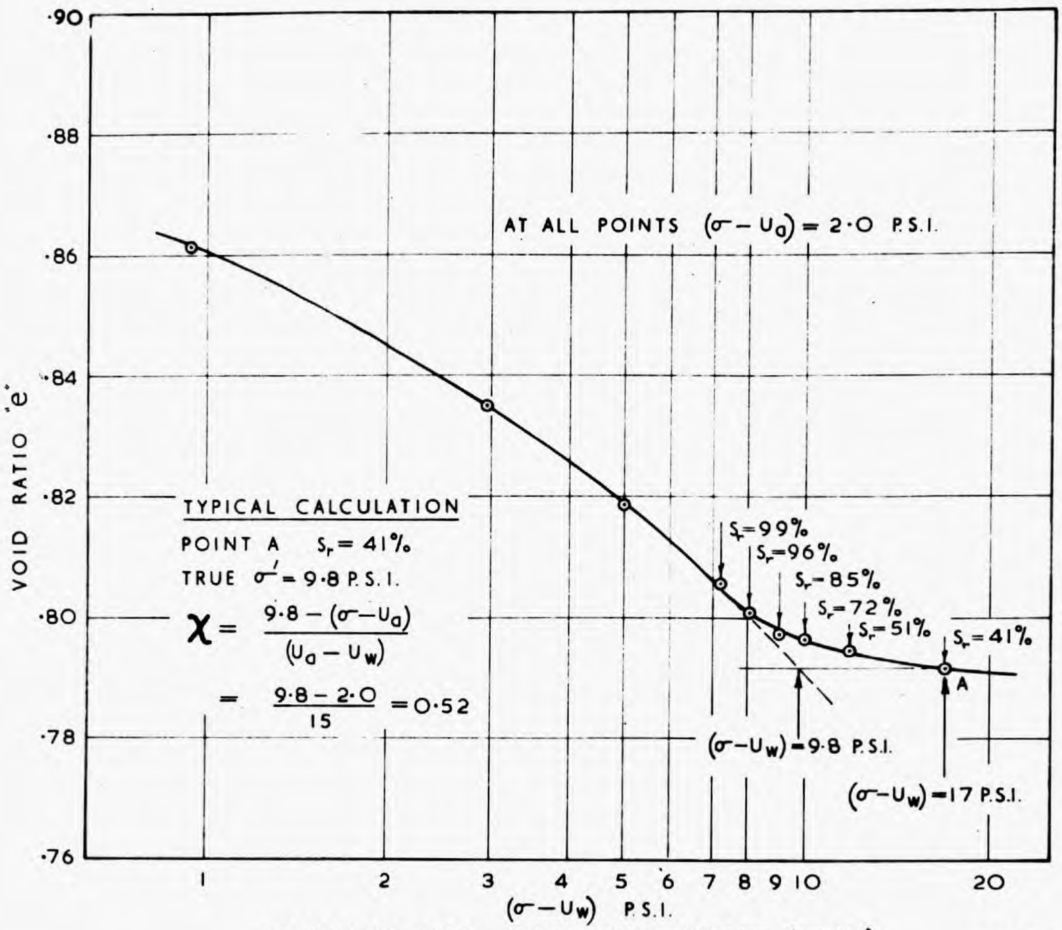
FIG 6.9



① GAULT CLAY Nº 4
② LONDON CLAY
③ OXFORD CLAY
④ WEALD CLAY

VOLUMETRIC χ-S RELATIONSHIPS
FOR CLAY SOILS

FIG. 6.10



BRAEHEAD SILT – VOID RATIO VERSUS $(\sigma - U_w)$

FIG. 6.11

I

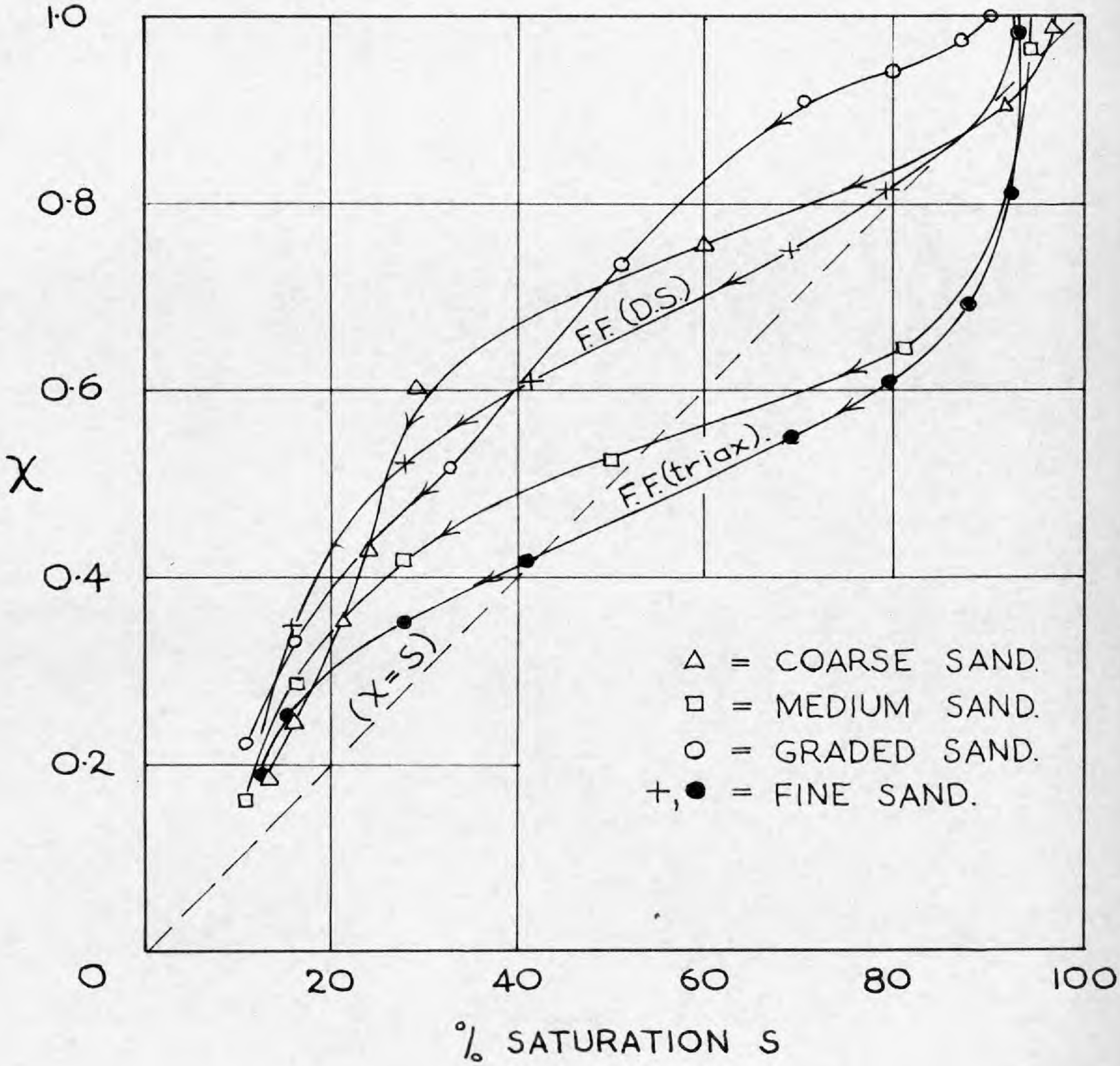
This was a relatively insensitive test as very small volume changes were involved but the χ - S values obtained agreed very well with those deduced from shear tests. The values are summarised below and plotted later in FIG. 6.24 and for this particular soil there appears to be no difference in the partly saturated effective stress law for shear strength or volume change phenomena.

S %	96	85	72	57	41
χ	.99	.95	.83	.71	.52

(6.5) Effective stresses and shear strength

(6.5) 1. Other work.

For correlations between effective stress and strength in partly saturated soils there are few tests available in the literature which have been done with sufficient control and measurement of variables or reliable technique. Aitchison and Donald (1956) reported reasonable agreement between measured and predicted strengths for medium fine and fine sands but the suction range used for the tests was only 0-½ atmosphere. χ values calculated from drained direct shear box tests with controlled negative u_w, reported by Donald (1956), are plotted as fuctions of S_r in FIG. 6.12. The range of χ for any value of S_r is considerable and may be partly due to the mechanism of failure in the direct shear box. Most samples failed along a curved surface and



χ-S RELATIONSHIPS DEDUCED FROM DIRECT SHEAR TESTS ON FOUR SANDS

FIG. 6.12.

not the horizontal mid-plane of the shear box, hence the full contact area may not have been operative throughout shear. At values of S below about 50%, χ appears to be larger than S as required by the theory.

A considerable amount of testing has been reported for compacted soils but, for reasons already outlined, much of it is too suspect to be used for research purposes. Recent investigations on the effect of rate of strain on the pore water pressure distribution in compacted soils by Blight (1961) has shown that much previous testing of compacted soils has been done too rapidly.

Alpan (1959) attempted to rationalise the behaviour of compacted Talybont clay using the 'generalised effective stress' of equation 2.21. He showed that the use of this equation gave reasonable failure envelopes for wetter samples but some of his pore pressure measurements on samples more than 1% dry of the optimum are open to question. Alpan used a χ factor assumed equal to S, but Blight (1961), using improvements on Alpan's techniques, deduced χ factors for a range of compacted soils, including Alpan's Talybont clay. Some of these results have been published by Bishop et al (1960).

(6.5). 2. Shear tests on Braehead Silt

(i) It was decided that the most suitable way to study the effect of degree of saturation, wetting-drying hysteresis and method of test on the χ factor was to test a soil whose complete stress history was known. Braehead silt was chosen because it was cohesionless and gave an excellent range of degrees of saturation for suctions which were within the limits of the apparatus.

By placing the silt as a saturated slurry (Chapter 4) and then subjecting it to controlled stresses only, it was possible to eliminate the unavoidable unknown factors which arise when testing natural or compacted soils.

The silt was used at a low initial density for two main reasons - it was quite easy to reproduce the initial porosity in all samples, within close limits, and the value of ϕ', the angle of shearing resistance, is insensitive to small changes in initial porosity for very loose samples (Bishop and Eldin, 1953).

The method of inferring effective stresses, and hence χ factors, from shear tests is very similar to that described in section (6.4).2.

(a) The relationship between deviator stress and lateral effective stress is established from standard drained compression tests on saturated samples

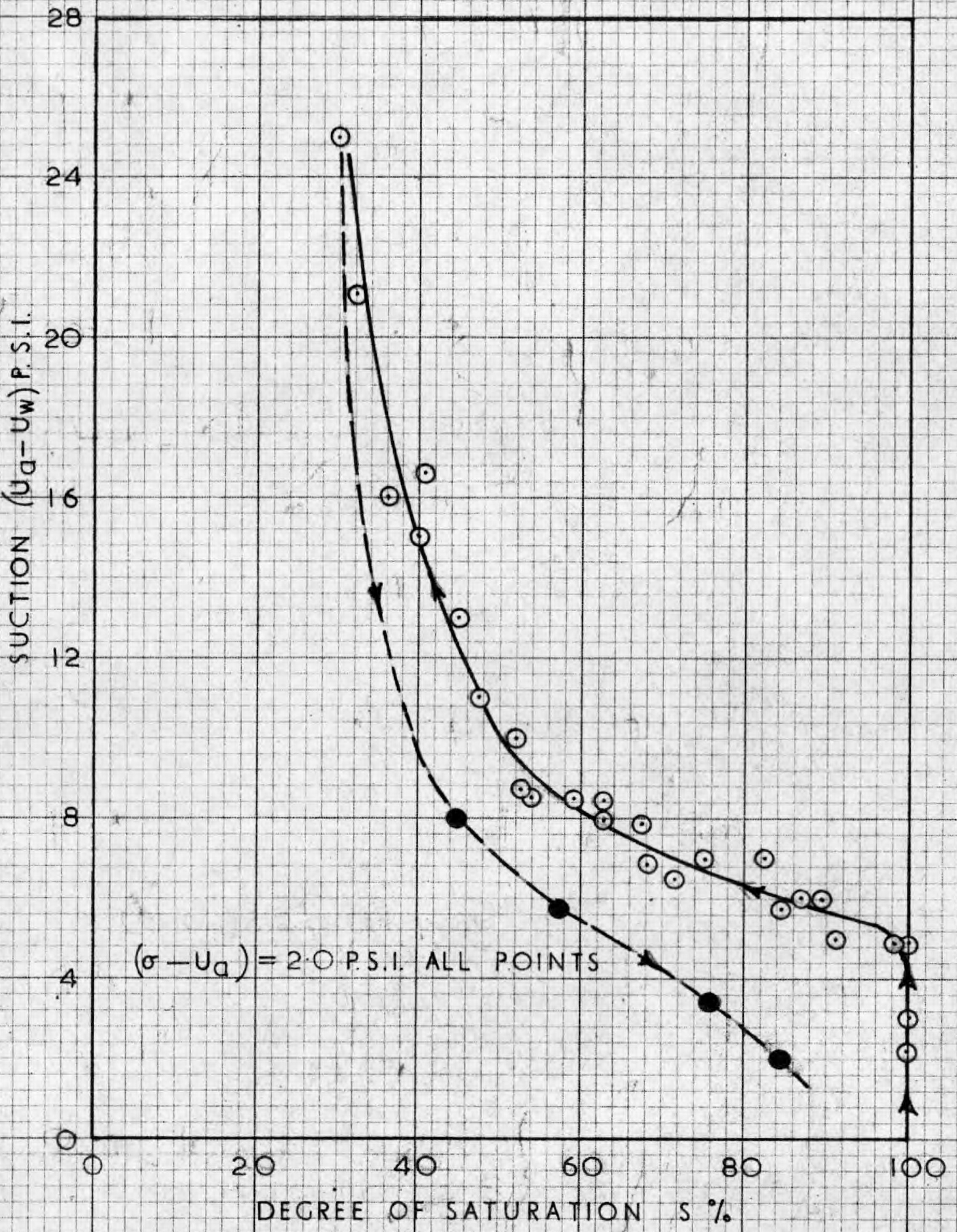
(b) The lateral effective stress, σ_3', in a partly saturated sample with maximum deviator stress $(\sigma_1 - \sigma_3)$ is taken as that lateral effective stress which would cause the same deviator stress at failure in a saturated sample.

(c) Knowing σ_3', σ, u_a and u_w, χ is deduced from $\sigma' = (\sigma - u_a) + \chi(u_a - u_w)$

This approach, simple in theory, proved somewhat difficult to apply as the partly saturated samples showed very different volume changes during shear and maximum deviator stress was reached at different strains for comparable saturated and partly saturated samples. Details of the calculation of χ values from tests on Braehead silt are given in (6.5.2) iii.

(ii) Moisture retention data

The modified soil moisture characteristic curves for Braehead silt are shown in FIG. 6.13. The drying curve is for draining from initial saturation at an average porosity of 46.5% (under $\sigma_3 - u_a$ = 2.0 lb/in^2), and the wetting curve starts from a suction of 25 lb/in^2 or a degree of saturation of about 30%. A linear scale has been used for the suction, $(u_a - u_w)$, as the total range involved is relatively small. The silt remains virtually saturated up to a suction of 5 lb/in^2. One or two points fall well off the curve but this is understandable as small differences in initial porosity will exert a large effect on the degree of saturation in the early stages of draining. Reference to TABLE 6.5 (later) shows that the $(u_a - u_w)$ values were produced by various combinations of u_a and u_w, the air pressure u_a often being greater than atmospheric. However the degree of saturation, and hence the water content also, is seen to depend only on the pressure difference.



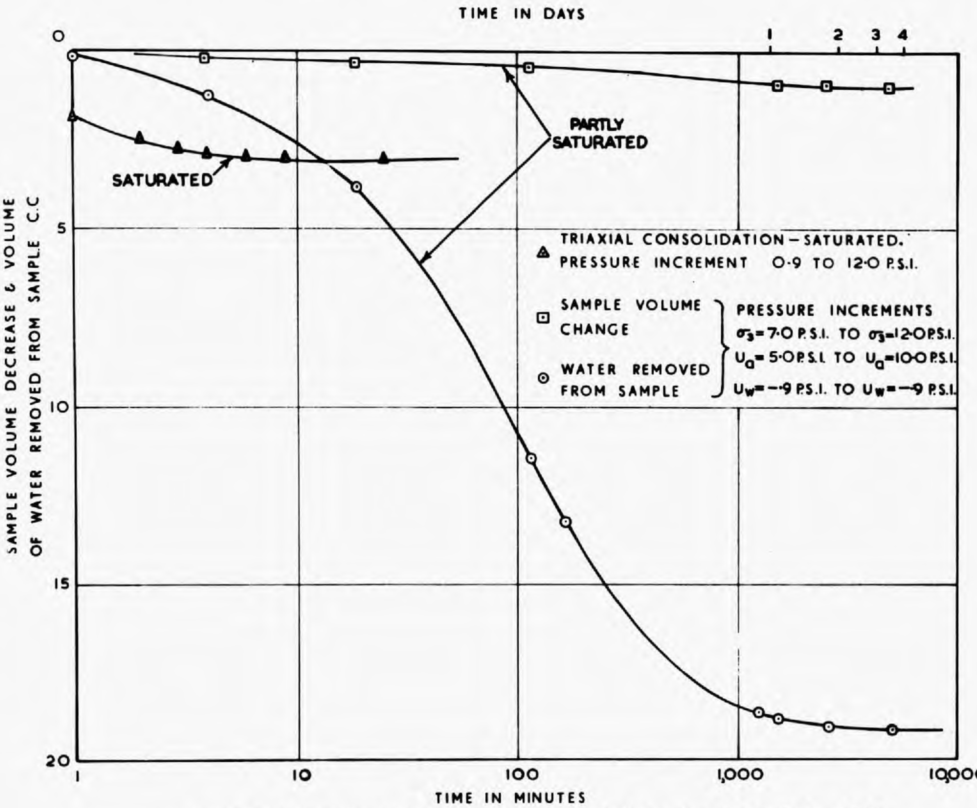
BRAEHEAD SILT MOISTURE CHARACTERISTIC CURVES FOR SLURRY INITIALLY CONSOLIDATED UNDER $(\sigma - U_a) = 20$ P.S.I.

FIG 6.13.

(iii) Drained Compression Tests

The technique used for these tests has been described in section 4.6. Detailed stress-strain curves are given in Appendix 6. FIG. 6.14 shows typical curves for the volume of water removed from a sample and the volume decrease of the sample during the consolidation or drainage stage before axial compression. For consolidation of a saturated sample under a positive cell pressure these two curves are the same and equilibrium is reached in a very short time, say 15 minutes. However, if a suction sufficient to cause air entry into the soil pores is applied to the base of the sample then the two volume changes are far from equal and both continue for an appreciable time - at least four days for the sample shown.

The reason for the slowness of drainage is that the coefficient of permeability, k, decreases rapidly with decreasing degree of pore space saturation (Childs and Collis-George, 1950 , Gardner 1958, Marshall 1960). This meant that tests on partly saturated samples could not be run as quickly as those on fully saturated ones if the pore water pressure was to be in equilibrium with the suction applied to the porous disc throughout the compression test. The determination of a suitable rate of strain presented some difficulties as there was no published data to use as a guide. A rough estimate was made using the method given by Bishop and Henkel (1957) for drained tests on saturated soils. The percentage of the total volume of water drained



TIME EFFECTS IN CONSOLIDATION AND DRAINAGE OF BRAEHEAD SILT

FIG. 6.14

versus log. time curve for a sample with S_r approximately equal to 50% was fitted to the theoretical drainage curve for consolidation of saturated clay to give c_v, the 'equivalent coefficient of consolidation' and hence the time to failure for any desired degree of pore pressure equalisation, 95% being taken as a suitable value. The experimental and theoretical curves can be matched at any point and the required time to failure was calculated for matches at 35, 50, 65 and 80 per cent drainage or consolidation. The calculated times to failure ranged from 3,500 to 8,300 minutes.

As this method can only be regarded as a rough approximation tests were carried out on three samples drained at a suction of 8.5 lb/in^2 using strain rates given in TABLE 6.4. The data of TABLE 6.4 supports the approximate calculations outlined above and the remainder of the

TABLE 6.4

EFFECT OF RATE OF STRAIN ON SHEAR STRENGTH

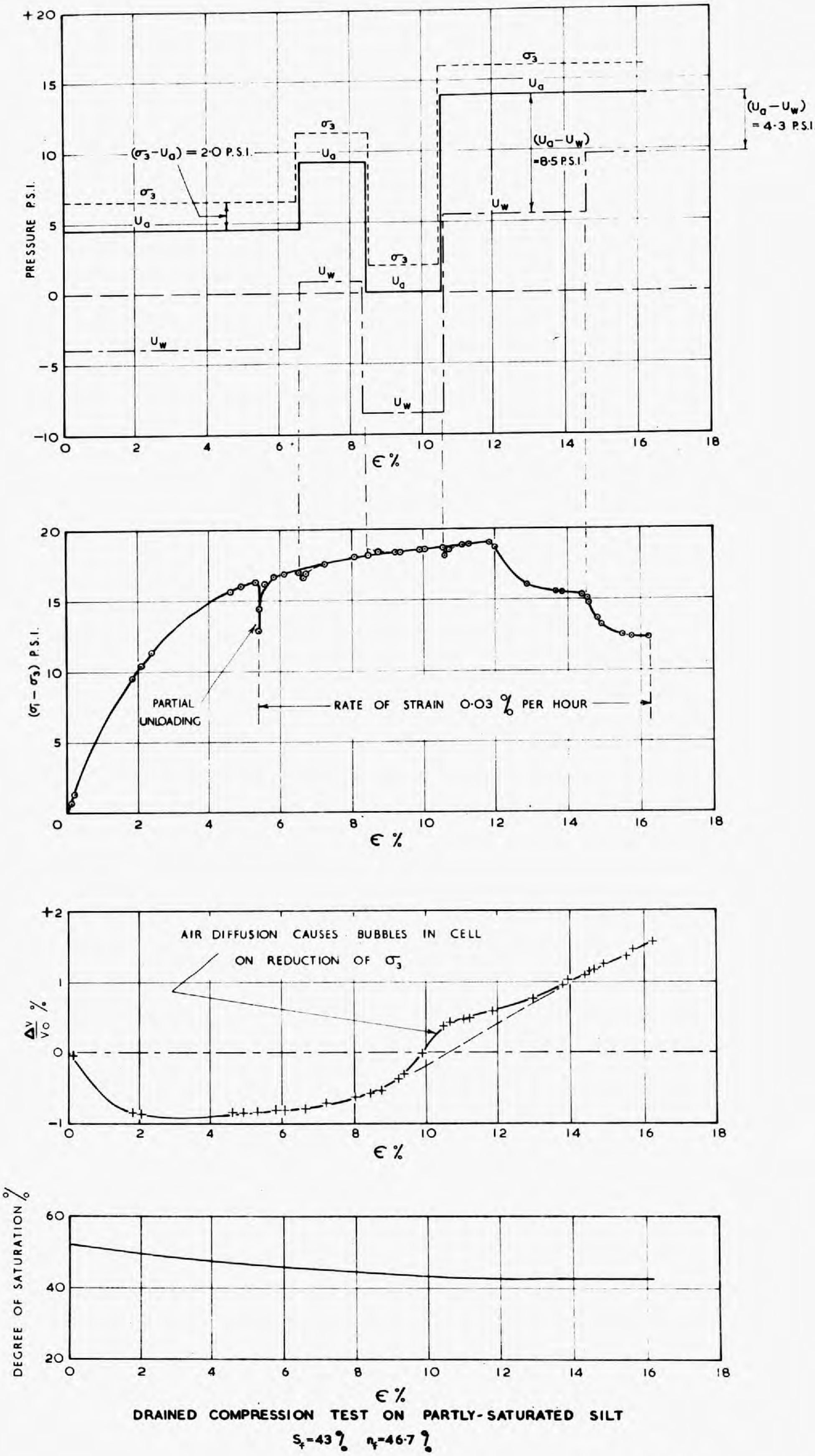
TEST NO.	STRAIN RATE % per min.	TIME TO FAILURE mins.	$(\sigma_1 - \sigma_3)$ LB/IN^2
4	0.00286	3,900	19.45
5	0.00130	9,000	19.4
6	0.00477	2,500	17.7

tests were compressed axially at a rate of 0.0029% per minute except some of the fully saturated samples which were failed in 6-8 hours.

The behaviour of a partly saturated sample during compression is shown in FIG. 6.15. This was a special test and the top diagram may be ignored for the moment. The deviator stress ($\sigma_1 - \sigma_3$), always showed a clear peak and samples failed by initial bulging with the appearance of a shear plane at about the time of reaching maximum strength. The sample volume decreased during the early stages of compression, as would be expected in loose normally consolidated samples, but long before failure the direction of volume change had reversed and the samples were dilating strongly after the fashion of dense saturated sands and silts.

Another reversal of normal saturated soil behaviour is that although the overall volume is increasing for most of the test, extra water drains from the sample at the controlled value of suction. This is a reflection of the lower water retaining potential of coarser pores. The combined effect of additional drainage and volume increase is that the overall degree of saturation drops continuously during compression. This is shown at the bottom of FIG. 6.15. This effect made it very difficult to carry out drained compression tests with degrees of saturation at failure greater than about 80%.

At the end of each test the sample was cut into three or four slices



DRAINED COMPRESSION TEST ON PARTLY-SATURATED SILT

$S_f = 43$ % $n_f = 46\cdot7$ %

FIG.6.15

for water content determination. The centre of the sample was drier than the ends by up to 3% but as the shear planes cut through most of the height of each sample the overall water content and degree of saturation were taken as acceptable values for correlating results. The value of S_r on the failure plane would have been exceedingly difficult to estimate.

The test, No. 19, shown in FIG. 6.15 was run more slowly than most so that u_a, u_w and σ_3 could be changed during test. The controlled air pressure u_a was coupled to the mercury cylinder controls for σ_3 and u_w so that $(\sigma_3 - u_a)$ and $(u_a - u_w)$ remained constant regardless of the absolute value of u_a. A smooth variation of $(\sigma_1 - \sigma_3)$ with axial strain was obtained in spite of wide variations in the three variables. This may be taken as convincing proof of the correctness of the general form of the effective stress equation.

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \qquad \text{........ (2.21)}$$

since S is varying smoothly throughout test and hence also χ, if it is taken as a function of S. A change in the $(u_a - u_w)$ term alone produced a sharp kink in the stress strain curve. The small deviations from the smooth curve at each change of pressure were due to the change in thrust on the end of the ram temporarily altering the axial load on the sample.

This test also provided visual evidence of the loss of air from the sample pores by diffusion through the membrane, free bubbles appearing in the cell on the reduction of pressure during the third stage of the test.

Volumetric changes at failure are plotted in FIG. 6.16 as functions of the degree of saturation at failure. FIG 6.16 (a) shows the rate of sample volume increase at failure with respect to strain, expressed as a fraction of the total volume at the commencement of shear plotted against S_r at failure. This graph also includes results from constant water content tests, 'overdrained' tests and standard drained triaxial tests on saturated samples. The marked increase in dilatation with decrease in S_r will be noted.

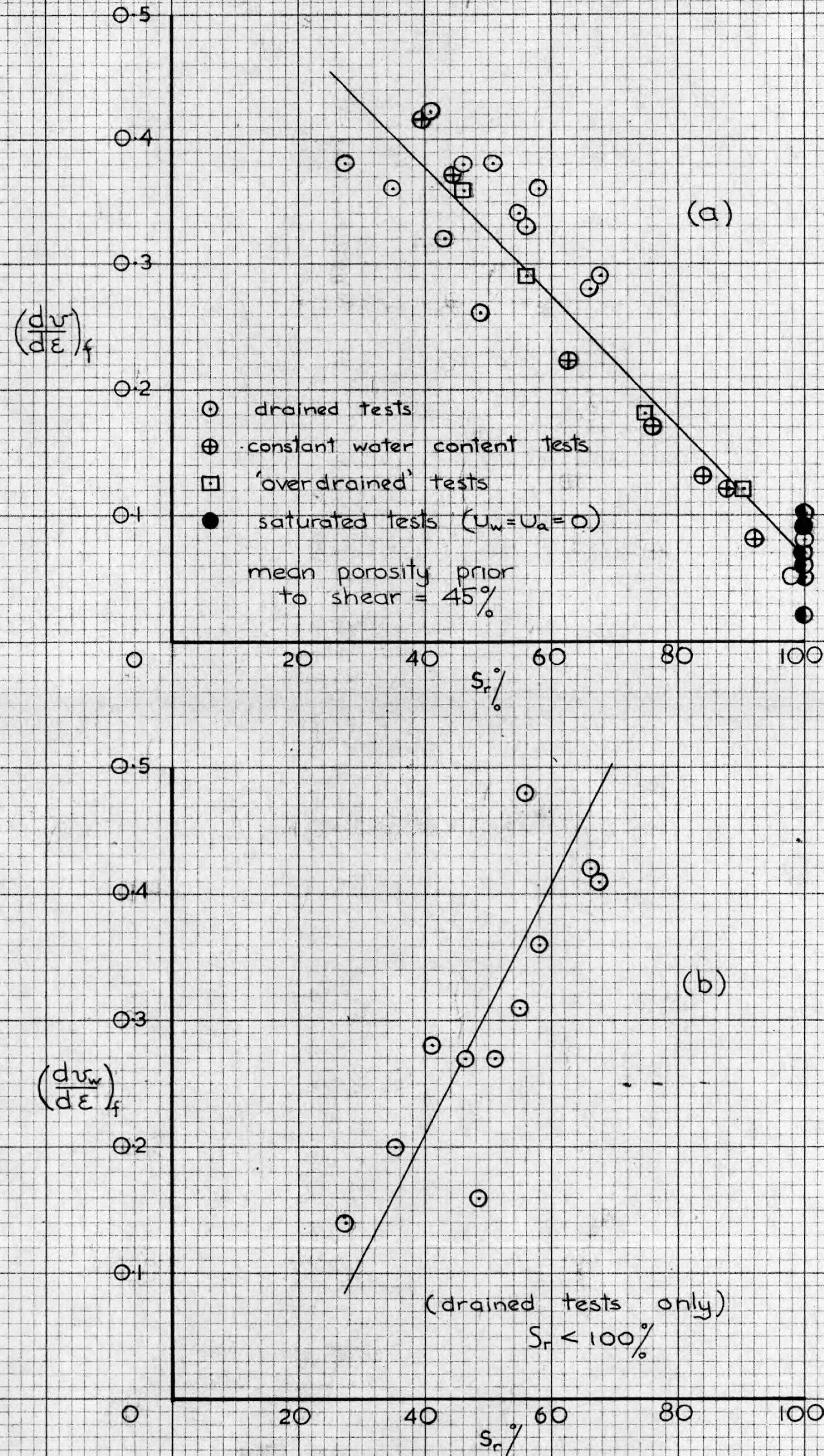
FIG. 6.16 (b) shows the rate of drainage of water from normally consolidated samples at failure, expressed as $\frac{dv_w}{d\varepsilon}$ where $v_w = \frac{\Delta V_w}{V_o}$

ΔV_w = volume of water drained since commencement of shear

V_o = sample volume at start of shear.

Both graphs in FIG. 6.16 may reasonably be approximated to by straight lines. All of the partly saturated tests plotted in FIG. 6.16 were done with $(\sigma_3 - u_a) = 2.0$ lb/in^2, i.e., nearly all the effective stress was due to the pore water suction.

It is not clear exactly why the degree of saturation should have such a marked influence on the volume change. For saturated samples subjected to external pressure the intergranular stresses before



STRAIN RATE OF CHANGE OF VOLUMETRIC STRAIN, $\frac{dv}{d\varepsilon}$, AT FAILURE AND STRAIN RATE OF CHANGE OF WATER DRAINAGE FROM SAMPLE (EXPRESSED AS % OF TOTAL INITIAL VOLUME), $\frac{dv_w}{d\varepsilon}$, AT FAILURE, AS FUNCTIONS OF OVERALL DEGREE OF SATURATION AT FAILURE, S_f.

FIG. 6·16

shearing possess both normal and transverse components, but for samples in which most of the intergranular stresses are caused by isolated water lenses around the particle contacts these stresses will lack a strong transverse or shear component. This difference undoubtedly affects the volume change characteristics of the soil but the mechanism involved is not completely understood. The large dilatations observed were not functions of triaxial testing technique as Donald (1956) noted similar volume increases for loose sands tested in direct shear.

These volume change phenomena made the inference of effective stresses from comparisons of partly and fully saturated tests rather more difficult than expected. Drained tests on fully saturated samples gave $\phi' = 33.4^{\circ}$ $c' = o$ for samples normally consolidated from slurry. The value of $\left(\frac{dv}{d\varepsilon}\right)_f$ was positive but small :- 0.05 to 0.10. However, as shown by Bishop (1954) dilatation against a cell pressure produces an additional component of deviator stress and therefore the partly saturated tests had to be corrected for the work done in dilating before they could be compared with the (corrected) saturated control tests.

The correction to $(\sigma_1 - \sigma_3)$ for saturated samples has been given by Bishop (1954) as

$$\text{correction to } (\sigma_1 - \sigma_3) = -\sigma_3 . \left(\frac{dv}{d\varepsilon}\right) \qquad \ldots\ldots\ldots (6.1)$$

where σ_3 = total minor principal stress

$v = \frac{\Delta V}{V_o}$ = volumetric strain

ΔV = volume change during shear

V_o = volume at start of shear

ε = axial strain $= \frac{-\Delta l}{l_o}$

Eqn. (6.1) is only strictly true for small strains. As shown in Appendix 4, a more general form of the equation is:

$$\text{correction} = -\sigma_3 \cdot \frac{dv}{d\varepsilon} \cdot \frac{A_o}{A} \qquad \text{............ (6.2)}$$

where A_o = sample cross sectional area at start of shear

A = sample area at strain ε.

The correction for partly saturated soils is more difficult to derive and possible approaches are dealt with in Appendix 4. The correction finally used was:

$$\text{correction to } (\sigma_1 - \sigma_3) = -(\sigma_3 - u_a) \cdot \frac{dv}{d\varepsilon} \cdot \frac{A_o}{A} \qquad \text{........ (6.3)}$$

The justification for using this equation, which is not rigorously correct, is dealt with in the appendix.

Calculation of χ factor

For a cohesionless soil the relationship between deviator stress and lateral effective stress, σ_3' is given by:

$$(\sigma_1 - \sigma_3) = \frac{2 \sin \phi'}{1 - \sin \phi'} \cdot \sigma_3' \qquad \text{......... (6.4)}$$

or, for $\phi' = 33.4^\circ$, $(\sigma_1 - \sigma_3) = 2.42\, \sigma_3'$ .... (6.5)

Hence, for a partly saturated sample, at failure

$$\sigma'_{3\,f} = \frac{(\sigma_1 - \sigma_3)_f}{2.42} \qquad \text{............ (6.6)}$$

where $(\sigma_1 - \sigma_3)_f$ = peak deviator stress corrected for dilatation and membrane effects.

We have also, from section (2.4)

$$\sigma'_3 = \sigma_3 - u_a + \chi(u_a - u_w) \qquad \text{........... (2.21)}$$

$$\therefore \chi = \frac{(\sigma_1 - \sigma_3)_f - 4.84}{2.42\,(u_a - u_w)} \qquad \left(\text{for } (\sigma_3 - u_a) = 2.0\right) \qquad \text{....... (6.7)}$$

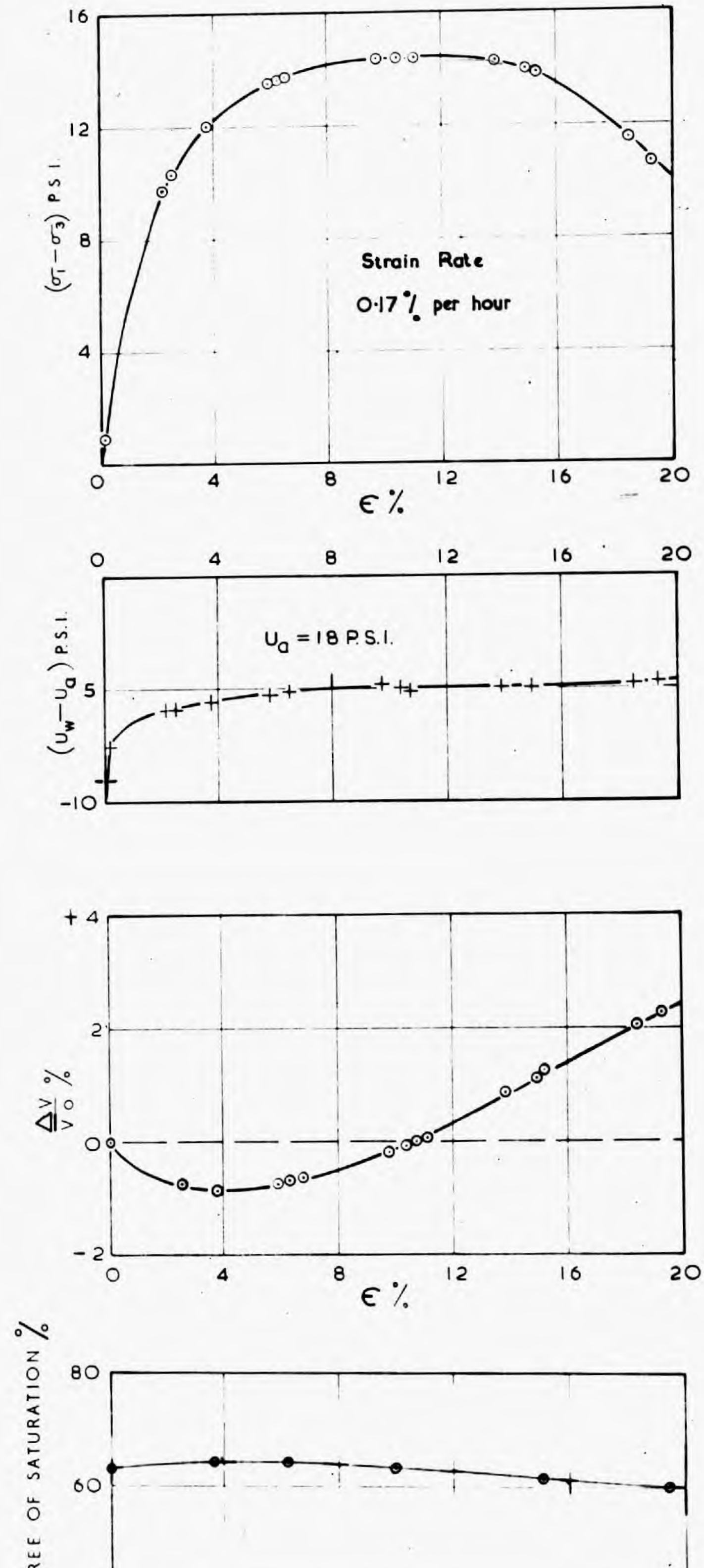
Values of χ calculated from drained tests with controlled $(u_a - u_w)$ are tabulated in TABLE 6.5 which gives all pertinent details of these tests as well as the other types of test to be described next. Analysis of the results will be given after the various test details have been discussed.

(iv) Constant water content tests

The initial purpose of these tests was to fill in the gap between 80% and 100% saturation which could not be covered by the drained tests. Later the series was extended to overlap the drained test results to study the effect of different types of compression on the χ factor. As these tests were not performed under mercury the pore air pressure u_a was controlled at atmospheric pressure or greater.

A typical test result is given in FIG. 6.17. The complete set of stress-strain curves is given in Appendix 6. During compression the pore water pressure rose rapidly at first and then remained practically constant for the remainder of the test. The strain rate chosen was the same as for the drained tests and the mechanism of failure was also the same - bulging followed by the appearance of a shear plane or planes at maximum deviator stress. The volume change curves were also very similar to those for drained tests and, in fact, values of $\frac{dv}{d\varepsilon}_f$ for this series have already been given in FIG. 6.16. As no overall water content changes were allowed during shear the degree of pore space saturation varied very little with axial strain. Water contents measured at the end of each test again showed the centre of each sample drier than the ends though the differences were generally less than for the drained tests. The χ factor was calculated as for drained tests after $(\sigma_1 - \sigma_3)_f$ had been corrected for dilatation and rubber membrane effects.

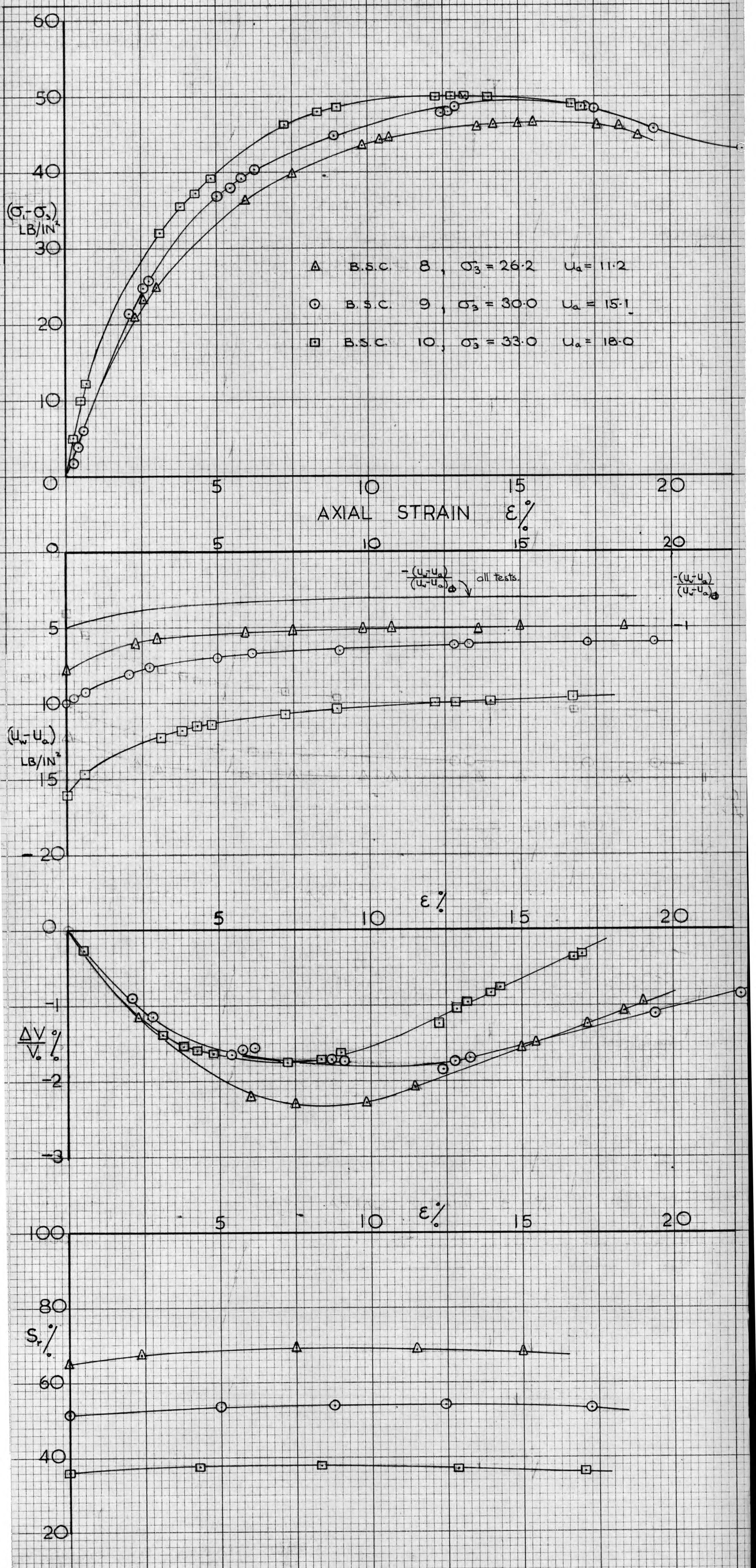
The main series of constant water content tests was carried out with $(\sigma_3 - u_a) = 2.0$ lb/in^2 for all tests. A further short series of tests was done with $(\sigma_3 - u_a) = 15.0$ lb/in^2 in an attempt to inhibit the volumetric expansion of the soil and study the effect on the χ factor of this slightly different failure mechanism. The test results are given in FIG. 6.18. The initial volume decreases were greater than



CONSTANT WATER CONTENT TEST – BRAEHEAD SILT

$S_f = 63$ $n_f = 45{\cdot}9$

FIG. 6.17



BRAEHEAD SILT — CONSTANT WATER CONTENT TESTS, $(\sigma_3 - u_a) = 15{\cdot}0$ LB/IN² for the tests with $(\sigma_3 - u_a) = 2.0$ lb/in^2, and although the samples were dilating at failure the rates of dilatation were less than for the earlier tests. If the pore pressure differences are plotted dimensionlessly as $\frac{(u_w - u_a)}{(u_w - u_a)_o}$ (where $(u_w - u_a)_o$ is the value at the start of shear) against axial strain ε %, then a unique curve is obtained. The χ values from these tests are plotted later with the results from all tests but it may be said here that no significant differences were noted between the χ values for the two series of constant water content tests. It was more difficult to determine χ accurately from these tests as the suction was providing less than one half of the lateral effective stress.

(v) 'Overdrained' tests

By analogy with the term 'overconsolidated' used with some saturated soils, 'overdrained' refers to a sample which has been subjected to a greater $(u_a - u_w)$ at some time in the past than the value obtaining at the time of test. A series of four drained tests was carried out by draining the samples initially under a suction of 25 lb/in^2 and then allowing them to absorb water at reduced suctions chosen to give a good coverage of the range of saturation from 30% to 90%. The purpose of these tests was to investigate the effect of the hysteresis in the soil moisture characteristic curve, FIG. 6.13, on the χ - S relationship. Even at very small suctions the degree of saturation did not rise above 90% and the wetting process obviously entrapped an appreciable volume of air in the pores. For a sand Haines found that 15% of the voids remained air filled even after the suction had been reduced to zero.

The test results are given in FIG. 6.19. The strain rate was the same as for the earlier 'normally drained' tests and full details of the results are given in Table 6.5. The main observable difference between the two types of drained test was that the degree of saturation stayed more nearly constant during shear for the overdrained samples. In fact for B.S.O. 2, with $(u_a - u_w) = 2.0 \text{ lb/in}^2$ during test, the degree of saturation increased and water re-entered the sample during compression. The correction for dilatation used on this sample was the normal one for saturated soils, viz:

$$\text{correction to } (\sigma_1 - \sigma_3) = (\sigma_3 - u_w)\,\frac{dv}{d\epsilon}\cdot\frac{A_o}{A}$$

but for the other samples with water draining from the soil at failure the correction in equation (6.3) was used. χ values at failure were calculated from the corrected deviator stress as already described. The values obtained from all tests are analysed in the next section.

(vi) <u>Result analysis</u>

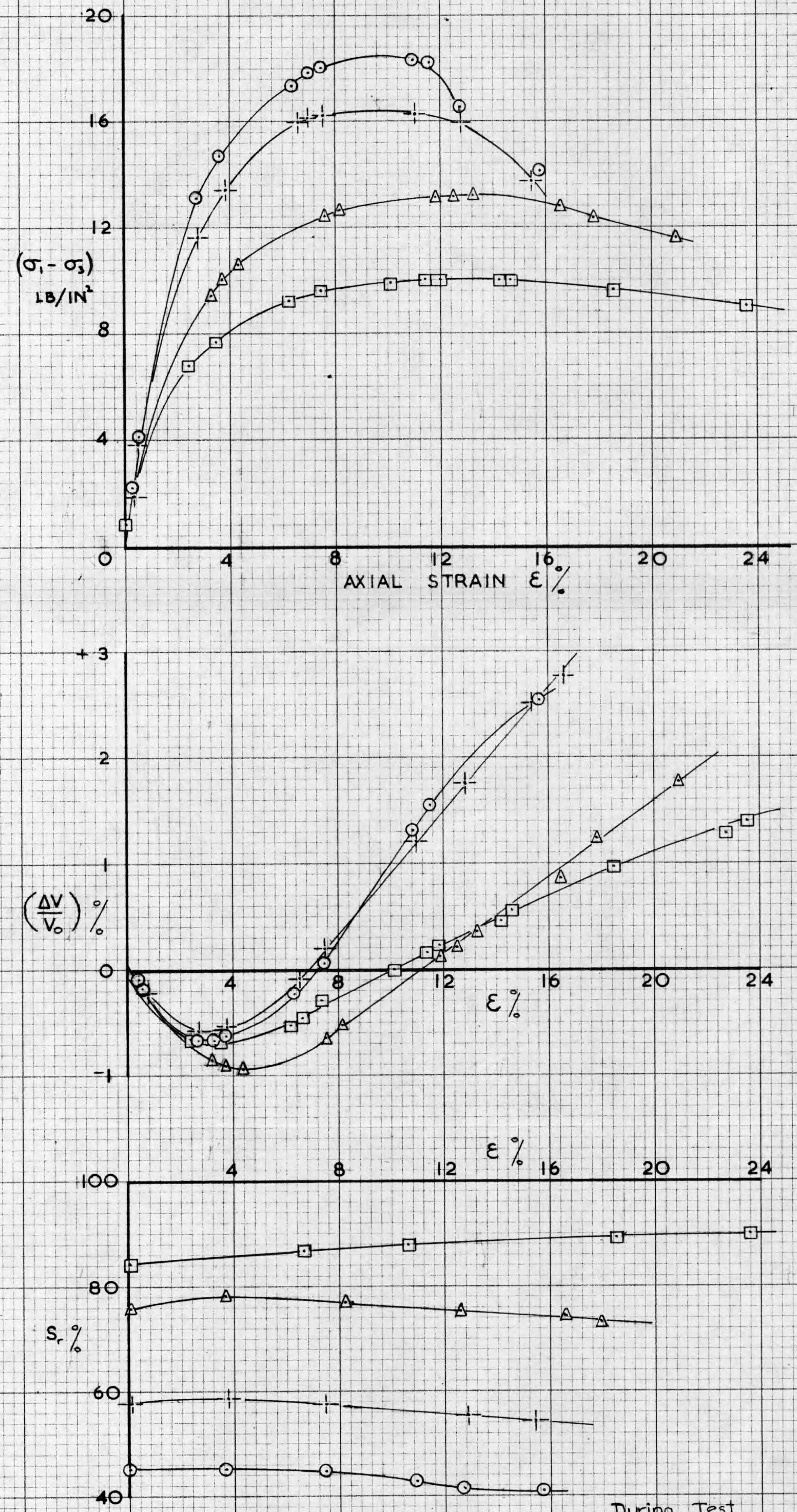
In FIG. 6.20, $(\sigma_1 - \sigma_3)_f$, the deviator stress at failure is

TABLE 6.5 TEST RESULTS - BRAEHEAD SILT

$G_s = 2.76$ L.L. = 29% P.L. = 23%

Test N°. B.S.	4	5	6	11	12	13	14	15	16	17	18	19	20	21	22	26
	DRAINED TESTS $(\sigma_3 - U_a) = 2.0$ lb/in²															
Cell pressure, σ_3, LB/IN²	2.0	2.0	2.05	2.1	2.1	2.0	2.1	2.0	2.08	12.0	2.0	6.85	2.0	2.05	17.0	16.25
U_{af} - air pressure (failure)	0	0	0	0	0	0	0	0	0	10.0	0	4.85	0	0	15.0	14.2
U_w - LB/IN² (start of shear)	-8.5	-8.4	-8.5	-3.0	-2.0	-5.95	-6.85	-7.90	-6.45	-1.0	-5.75	-3.7	-5.0	-4.86	-6.1	-0.8
U_{wf} - LB/IN² (at failure)	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
$(U_a - U_w)$ at start of shear	8.5	8.4	8.5	3.0	2.0	5.95	6.85	7.90	6.45	11.0	5.75	8.5	5.0	4.86	21.1	15.0
$(U_a - U_w)$ at failure LB/IN²	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
Porosity (start of shear)	0.46	0.455	.455	.445	.450	.448	.455	.445	.452	.451	.450	.464	.459	.450	.460	.450
Porosity, n, at failure	0.46	.455	.463	.440	.442	.439	.457	.444	.454	.452	.446	.467	.456	.437	.455	.454
Water content (failure), ω %	14.85	15.3	14.2	28.8	28.2	20.9	16.7	16.2	17.3	12.3	19.3	13.7	21.0	27.6	8.9	10.5
$(\sigma_1 - \sigma_3)_f$ LB/IN²	19.45	19.4	17.7	12.55	10.05	18.6	16.45	17.5	16.45	19.1	15.04	18.6	14.9	17.0	25.1	23.2
$\frac{(\sigma_1 - \sigma_3)_f}{2}$ " "	9.72	9.7	8.85	6.27	5.02	9.3	8.23	8.75	8.22	9.55	7.52	9.3	7.45	8.50	12.55	11.6
$\frac{(\sigma_1 + \sigma_3)_f}{2}$ " "	11.72	11.70	10.9	8.37	7.12	11.3	10.33	10.75	10.30	21.55	9.52	16.15	9.45	10.55	29.55	27.85
$\frac{(\sigma_1 + \sigma_3)_f}{2} - U_w$ " "	20.22	20.10	19.40	11.37	9.12	17.25	17.18	18.65	16.75	22.55	15.27	19.85	14.45	15.41	35.65	28.65
χ	.71	.705	.63	1.0	1.0	1.0	.68	.67	.72	.53	.74	.67	.83	1.0	.40	.50
Degree of satⁿ (failure), S_f %	48.5	51	46	100	100	100	54.8	56	58	40.6	66	43	67.5	99	29	35
S %, (start of shear)	58.8	62.5	53.8	"	"	"	68	67.8	71.5	47.6	84.5	52.2	91	99	32	40
$\left(\frac{dv}{d\varepsilon}\right)_f$, ($v = \frac{\Delta V}{V_0}$ $\varepsilon = -\frac{\Delta l}{l_0}$)	.26	.38	.38	.10	.08	.05	.34	.33	.37	.42	.28	.32	.29	.05	.38	.36
$\left(\frac{dv_w}{d\varepsilon}\right)_f$, ($v_w = \frac{\Delta V_w}{V_0}$)	.16	.27	.27	-.1	-.08	-.05	.31	.48	.36	.28	.42	-	.41	.25	.14	.19
U_a, LB/IN², initial drainage																
U_w, LB/IN², " "																
$(U_a - U_w)$ " "																

Test N°. B.S.	O.1	O.2	O.3	O.4	C.1	C.2	C.3	C.4	C.5	C.6	C.7	C.8	C.9	C.10	D7	D8	D9	D10	D24	D25
	OVERDRAINED $(\sigma_3 - U_a) = 2.0$ lb/in²				CONSTANT WATER CONTENT $(\sigma_3 - U_a) = 2.0$ lb/in²							$(\sigma_3 - U_a) = 15.0$			DRAINED TESTS $U_a = U_w = 0$					
Cell pressure, σ_3, LB/IN²	21.5	21.55	21.5	21.6	20.0	2.0	2.1	11.1	18.0	7.1	21.2	26.2	30.0	33.0	8.0	4.0	12.0	10.0	10.0	5.0
U_{af} - air pressure (failure)	19.5	19.6	19.6	19.6	18.0	0	0	9.0	16.0	5.1	19.2	11.2	15.1	18.0	-	-	-	-	-	-
U_w - LB/IN² (start of shear)	11.5	17.6	16.1	13.9	10.0	-7.0	-6.0	2.0	3.0	-0.9	2.6	3.2	5.1	2.0	0	0	0	0	0	0
U_{wf} - LB/IN² (at failure)	"	"	"	"	13.0	-3.25	-3.0	4.85	7.3	1.7	7.8	6.2	9.0	8.0	"	"	"	"	"	"
$(U_a - U_w)$ at start of shear	8.0	2.0	3.5	5.7	8.0	7.0	6.0	7.0	13.0	6.0	16.6	8.0	10.0	16.0	-	-	-	-	-	-
$(U_a - U_w)$ at failure LB/IN²	"	"	"	"	5.0	3.25	3.0	4.15	8.7	3.4	11.4	5.0	6.1	10.0	-	-	-	-	-	-
Porosity (start of shear)	.454	.452	.463	.444	.460	.452	.456	.455	.447	.459	.445	.440	.445	.452	.464	.470	.452	.462	.449	.468
Porosity, n, at failure	.455	.453	.464	.449	.459	.447	.453	.453	.446	.452	.448	.430	.435	.447	.452	.458	.440	.448	.437	.457
Water content (failure), ω %	13.2	26.1	22.4	16.7	19.4	24.6	26.5	21.5	13.4	27.55	11.75	18.3	15.0	10.75	29.4	30.5	29.8	29.1	30.3	32.7
$(\sigma_1 - \sigma_3)_f$ LB/IN²	17.7	10.0	12.7	15.8	13.9	12.4	11.4	14.3	19.0	12.45	21.8	45.0	48.2	47.6	19.5	9.44	29.45	23.28	25.0	11.9
$\frac{(\sigma_1 - \sigma_3)_f}{2}$ " "	8.85	5.0	6.35	7.9	6.95	6.2	5.7	7.15	9.5	6.22	10.9	22.5	24.1	23.8	9.75	4.72	14.72	11.64	12.5	5.95
$\frac{(\sigma_1 + \sigma_3)_f}{2}$ " "	30.35	26.55	27.85	29.5	26.95	8.0	7.8	18.25	27.5	13.32	32.1	48.7	54.1	56.8	17.75	8.72	26.72	21.64	22.5	10.95
$\frac{(\sigma_1 + \sigma_3)_f}{2} - U_w$ " "	18.85	8.95	11.75	15.6	13.95	11.25	10.8	13.4	20.2	11.6	24.3	42.5	45.1	48.8	"	"	"	"	"	"
χ	.66	.99	.95	.80	.75	.93	.91	.89	.65	.92	.61	.75	.77	.47	-	-	-	-	-	-
Degree of satⁿ (failure), S_f %	43.5	88.5	75	56	63	84	88	76	45.2	92	40	68	54	37	100	100	100	100	100	100
S %, (start of shear)	45	84.5	75.8	57.7	62.8	82.3	87	75.5	45.0	89.5	40.5	65	51.8	36.2	"	"	"	"	"	"
$\left(\frac{dv}{d\varepsilon}\right)_f$, ($v = \frac{\Delta V}{V_0}$ $\varepsilon = -\frac{\Delta l}{l_0}$)	.36	.12	.18	.29	.22	.13	.12	.17	.37	.08	.42	.14	.11	.19	.10	.05	.07	.06	.09	.02
$\left(\frac{dv_w}{d\varepsilon}\right)_f$, ($v_w = \frac{\Delta V_w}{V_0}$)	very small	-.21	very small	very small	-	-	-	-	-	-	-	-	-	-	-.10	-.05	-.07	-.06	-.09	-.02
U_a, LB/IN², initial drainage	19.5	19.6	19.6	19.6																
U_w, LB/IN², " "	-5.5	-5.5	-5.5	-5.4																
$(U_a - U_w)$ " "	25.0	25.1	25.1	25.0																



		Initial $(U_a - U_w)$	During Test σ_3	U_a	U_w	$U_a - U_w$
⊙	B.S.O.1	25 LB/IN^2	21·5	19·5	11·5	8·0
⊡	B.S.O.2	25	21·5	19·6	17·6	2·0
△	B.S.O.3	25	21·5	19·5	16·0	3·5
+	B.S.O.4	25	21·6	19·6	13·9	5·7

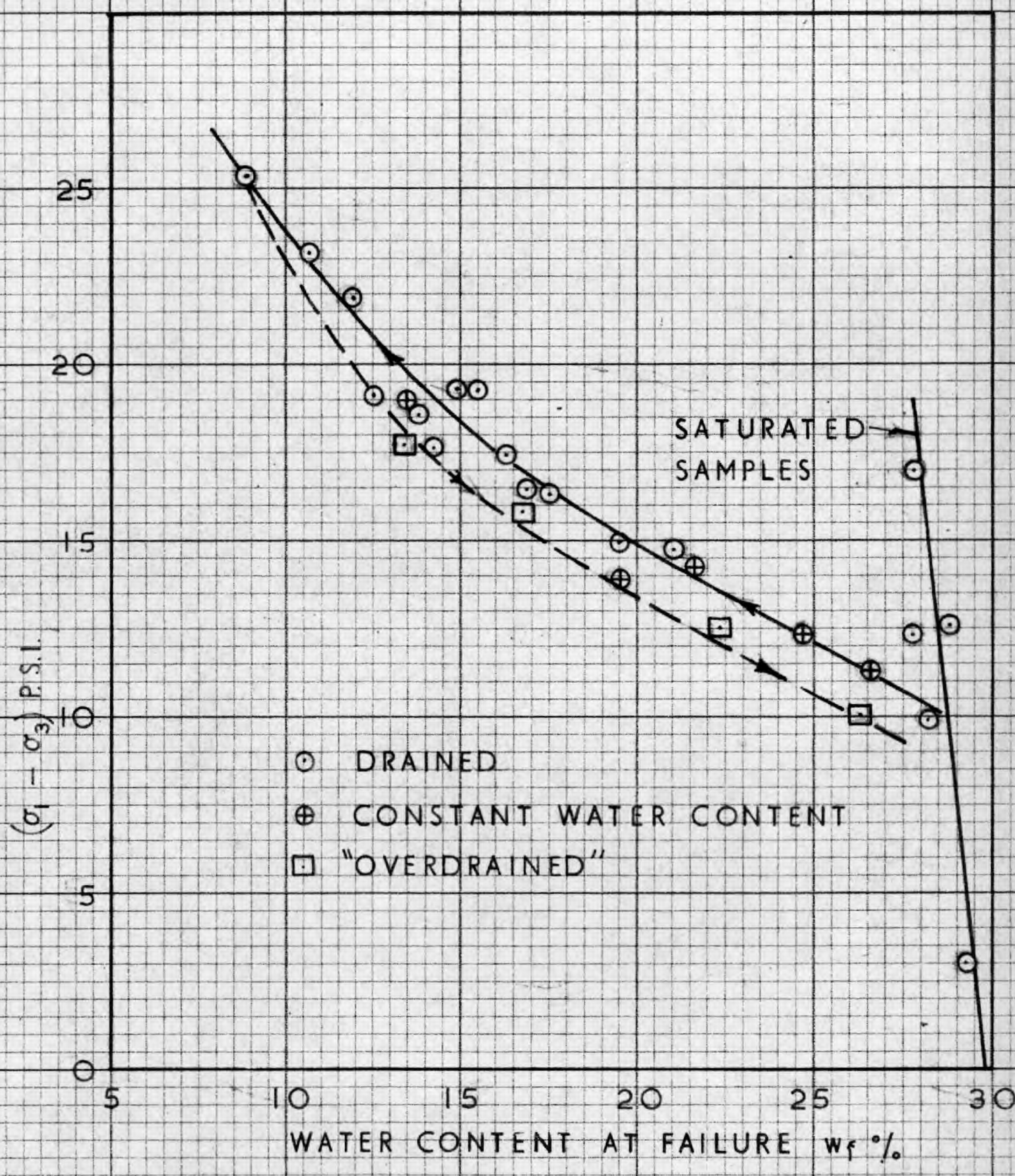
BRAEHEAD SILT 'OVERDRAINED' TESTS

FIG. 6.19

plotted against the overall water content at failure, w_f for $(\sigma_3 - u_a) = 2.0\ lb/in^2$. For normally drained samples, i.e. those taken direct from the slurried condition to their final values of $(\sigma_3 - u_a)$ and $(u_a - u_w)$, there appears to be a unique relationship between the strength and the water content regardless of the type of test. This statement holds at least for drained tests and constant water content tests at a range of cell pressures, failed in axial compression with $(\sigma_3 - u_a)$ constant during test. There is some hysteresis in the relationship when overdrained tests are included. FIG. 6.20 may be taken as a partial extension to unsaturated soils of the so-called 'American Working Hypothesis' for saturated soils (Rutledge, 1947) which, in effect, states:

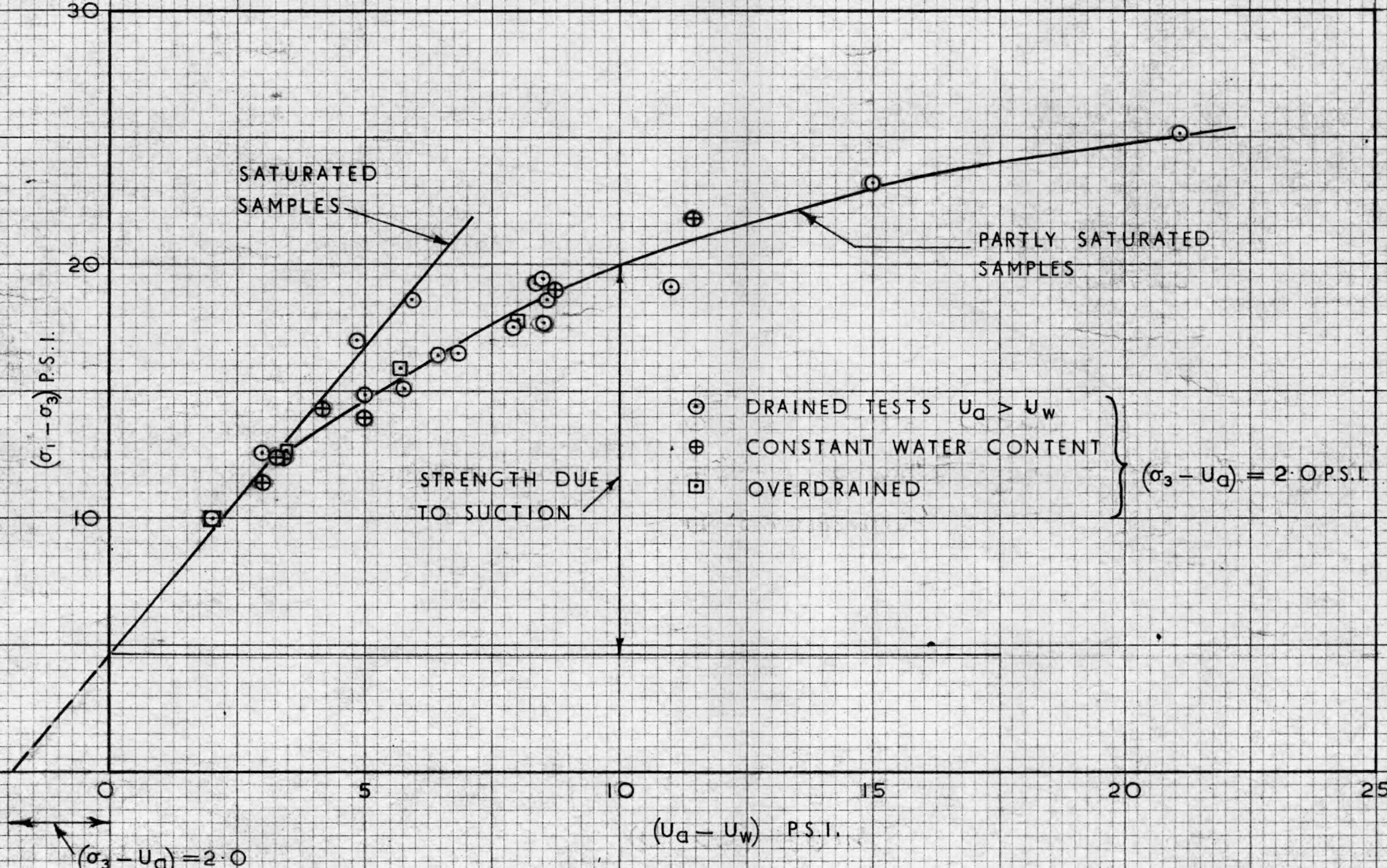
"For a normally consolidated soil the compression strength at failure depends only on the water content and is independent of the method of test".

FIG. 6.20 and the moisture characteristic curve, FIG. 6.13, imply a relationship between $(\sigma_1 - \sigma_3)$ and $(u_a - u_w)$ for $(\sigma_3 - u_a)$ constant. This is plotted in FIG. 6.21. Two clear relationships are seen. All saturated samples plot on a straight line cutting the $(u_a - u_w)$ axis at $-2.0\ lb/in^2$. The strength at zero suction is caused by the constant $(\sigma_3 - u_a) = 2.0\ lb/in^2$ for all tests. The partly saturated tests lie on a common line regardless of type of test or absolute values of u_a and u_w at failure. The strength component directly caused by the



STRENGTH OF PARTLY SATURATED SILT AS A FUCTION OF OVERALL WATER WATER CONTENT AT FAILURE

FIG 6.20.



MAXIMUM DEVIATOR STRESS VERSUS SUCTION $(U_a - U_w)$ AT FAILURE

FIG. 6. 21.

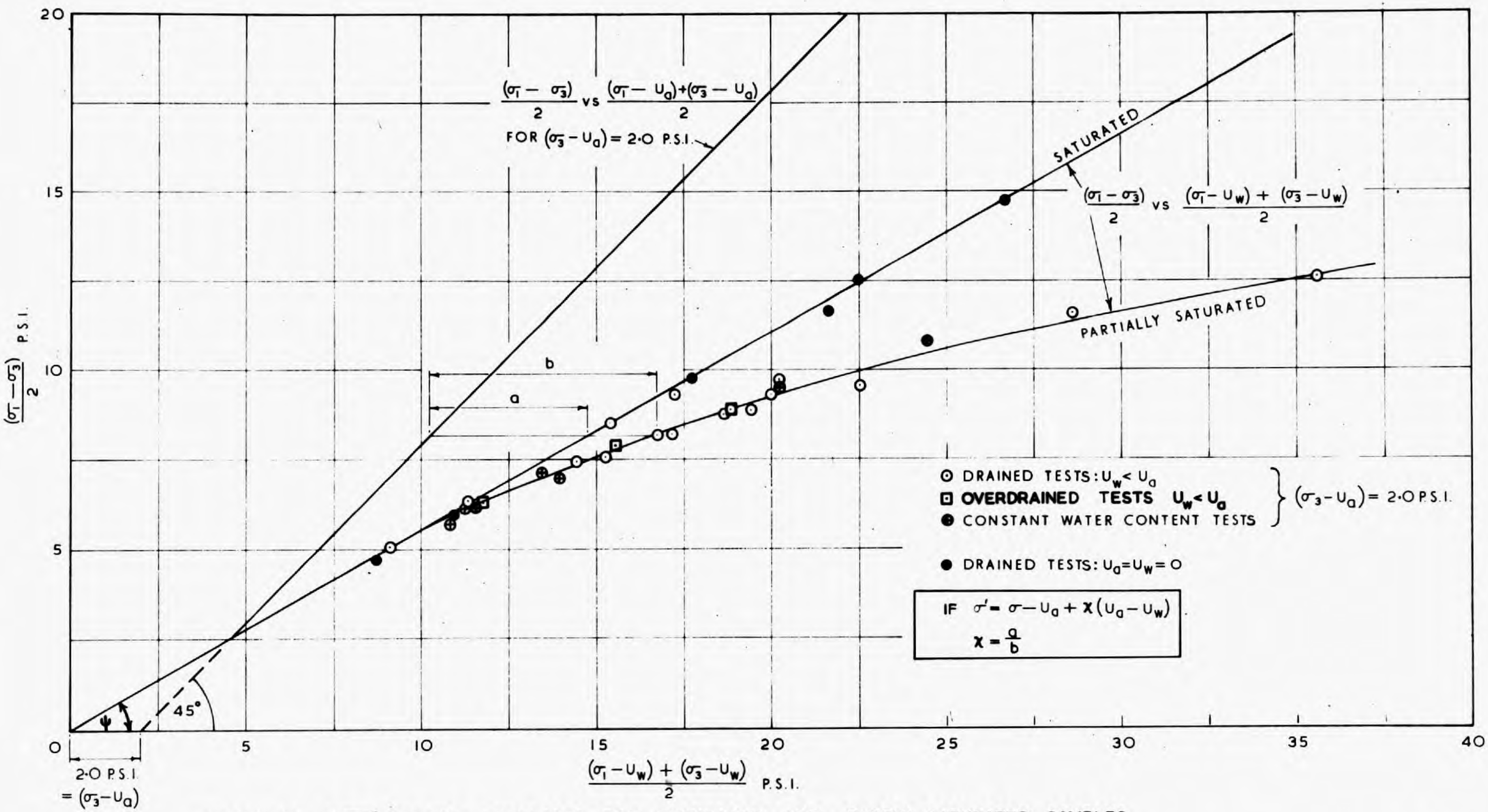
capillary effect is seen to depend only on the magnitude of the pressure difference or suction, $(u_a - u_w)$. A maximum value of peak deviator stress was not reached in this series of tests but examples may be found in Donald (1956) and Blight (1961).

FIG. 6.22 is a modified Mohr plot of the type described in section 2.5. The points plotted as $\frac{(\sigma_1 - \sigma_3)}{2}$ against $\frac{(\sigma_1 - u_w) + (\sigma_3 - u_w)}{2}$ represent the tops of the Mohr circles according to the conventional (saturated) effective stress equation. The points for saturated samples, corrected for volume change, fall on a straight line through the origin making an angle ψ with the horizontal. Remembering that

$$\sin \phi' = \tan \psi \qquad \text{............ (2.26)}$$

then ϕ' for the normally consolidated silt is found to be 33.4°.

The points representing the partly saturated samples corrected for volume change define a curved failure envelope departing increasingly from the line for saturated samples, the true effective stress failure envelope, with decreasing degree of saturation. Once again the position of the line appears to be independent of the type of test, even the hysteresis samples fitting into the picture quite accurately. The graphical significance of the χ factor is indicated on the diagram though the values used in later figures were calculated by the more accurate method outlined earlier. It is interesting to note that the partly saturated soil failure envelope, if considered as approximately

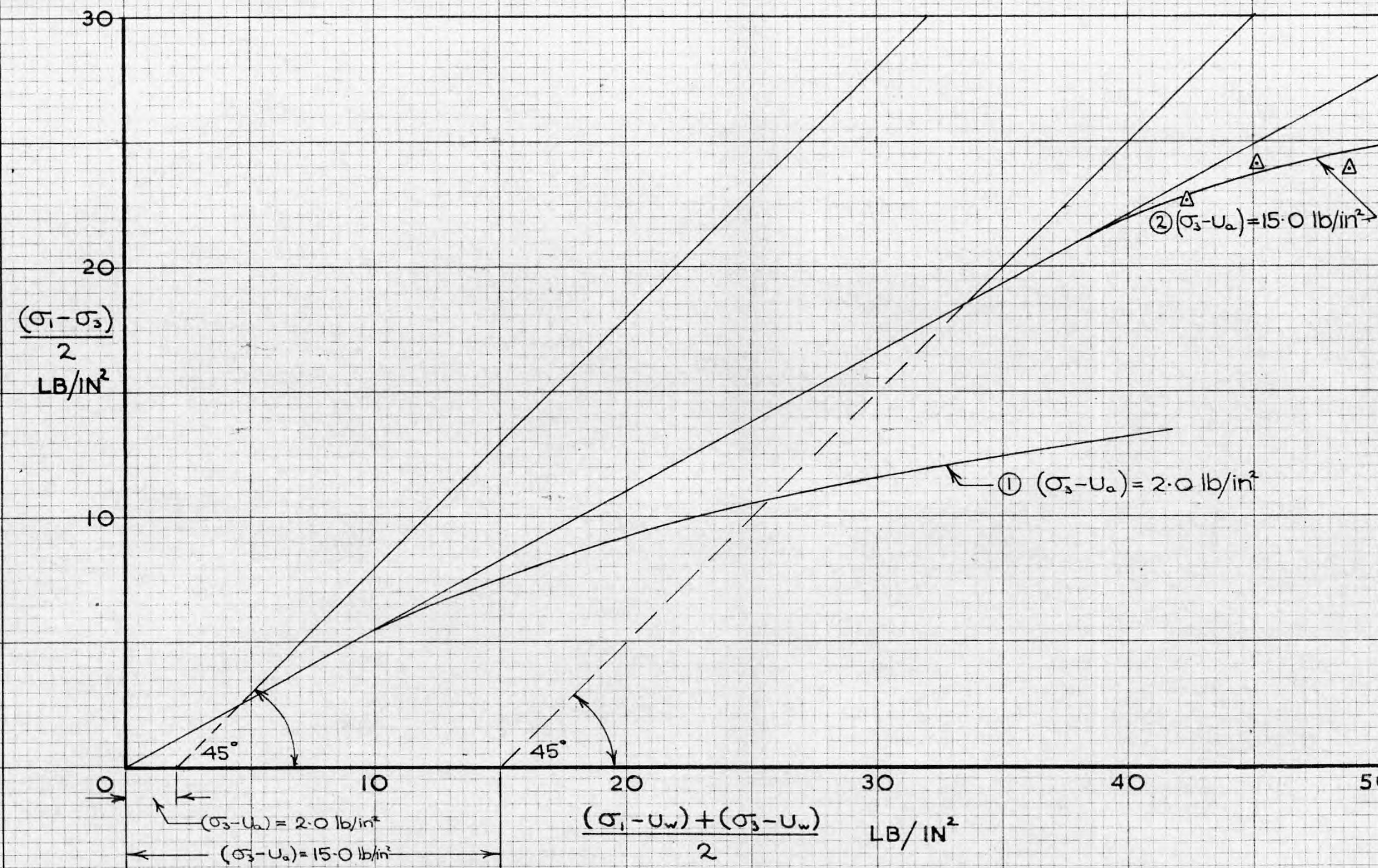


BRAEHEAD SILT — FAILURE ENVELOPES FOR SATURATED AND PARTLY-SATURATED SAMPLES

FIG. 6.22

straight over any short length, gives an appreciable intercept on the $\frac{(\sigma_1 - \sigma_3)}{2}$ axis. Hence unsaturated samples tested at constant $(\sigma_3 - u_a)$ give a spurious positive cohesion when plotted in terms of conventional effective stresses. The 'negative cohesion' mentioned in section 2.4 and FIG. 2.1 is obtained from tests at constant water content and different $(\sigma_3 - u_a)$. FIG. 6.23 presents the results of the three constant water content tests run with $(\sigma_3 - u_a) = 15.0$ lb/in^2. Although there are insufficient points to fix an accurate envelope the general effect is seen to be a translation of the failure envelope along the true effective stress line, the envelope being also somewhat shortened.

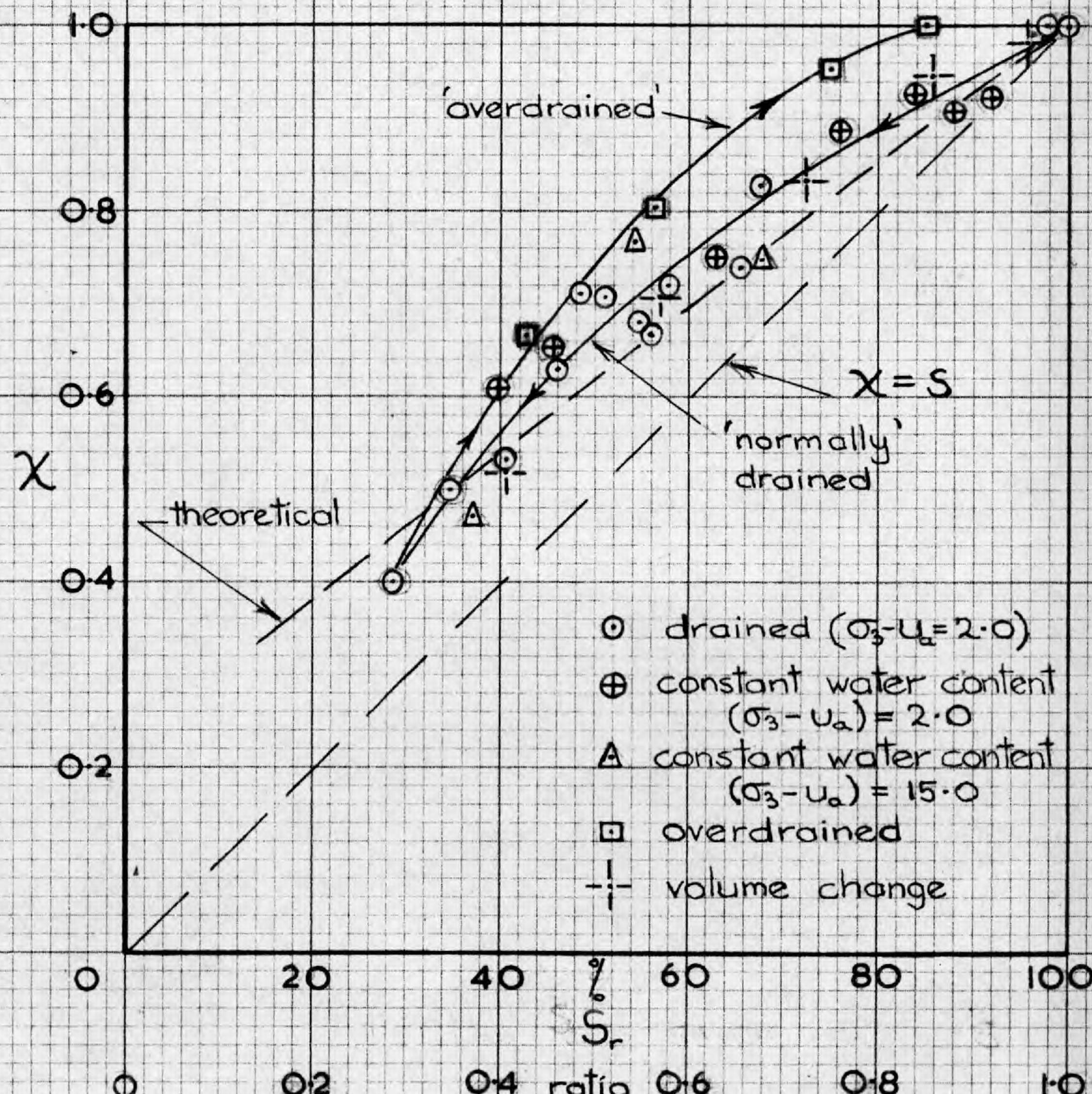
Probably the most interesting graph to be derived from the test results is the $\chi - S_r$ relationship shown in FIG. 6.24. Values of χ calculated from test results for drained tests, constant water content tests at two values of $(\sigma_3 - u_a)$ and the volumetric test described in section (6.4).3. fall quite accurately about a common line. The tests covered a wide range of actual values of u_a, u_w and σ_3 but the fact that a common plot like FIG. 6.24 is obtained when χ values are calculated from the Bishop modified effective stress equation does much to verify the equation and establish the pressure difference $(u_a - u_w)$ as one of the main factors controlling shear strength in partly saturated soils, regardless of the separate absolute values of u_a and u_w.



BRAEHEAD SILT

FAILURE ENVELOPES FOR TWO VALUES OF (σ_3-U_a).

FIG. 6.23



BRAEHEAD SILT

χ-S RELATIONSHIPS

FOR $\sigma' = \sigma - u_a + \chi(u_a - u_w)$

FIG. 6.24

In FIG. 6.24 the dotted line is the theoretical relationship, equation (2.54), derived in chapter 2. This slightly underestimates the measured χ values at high degrees of saturation but gives excellent agreement for the low end of the range. Considering the assumptions involved in both theory and experiments the agreement is very satisfying over the complete range of saturation studied.

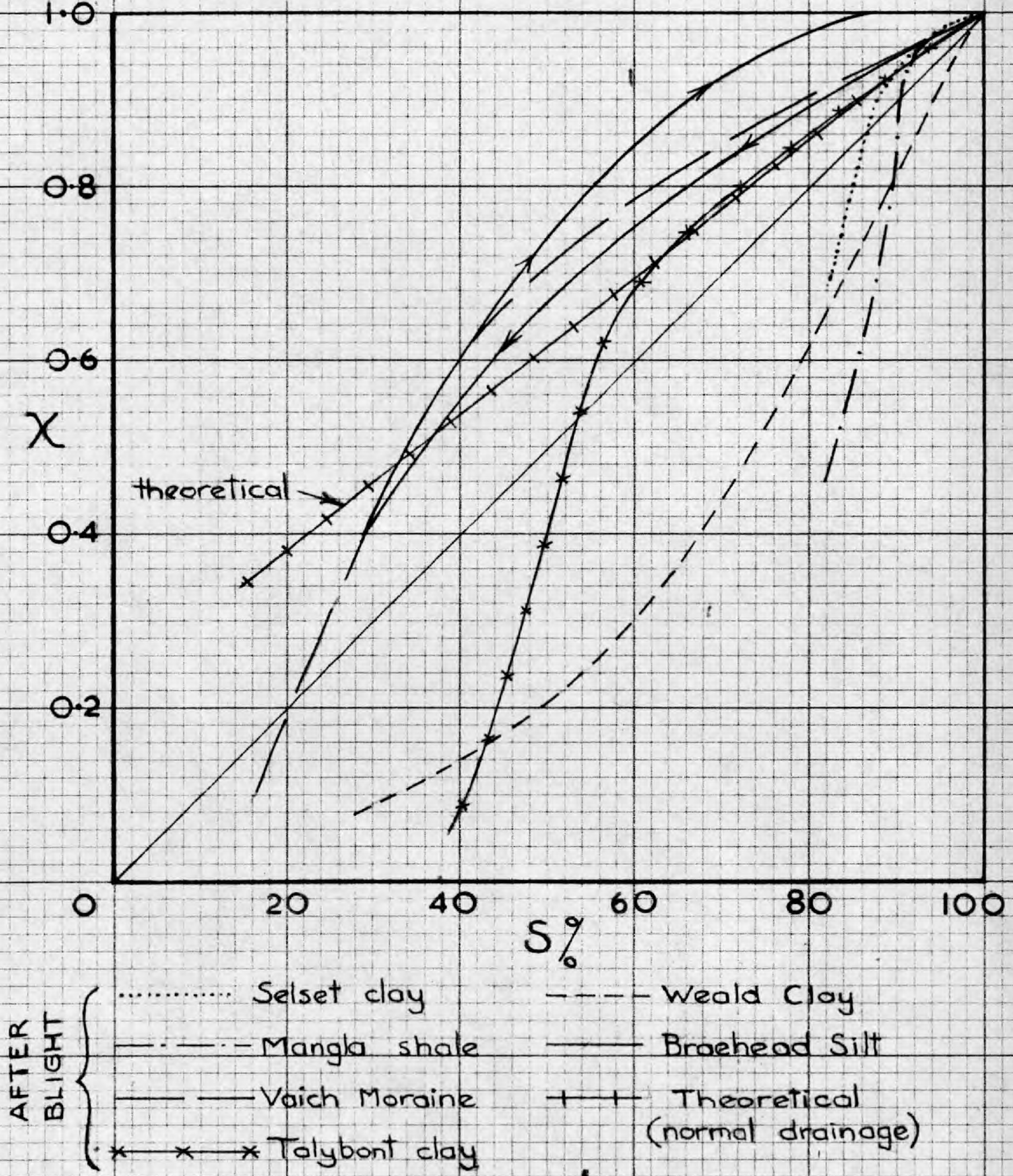
The 'overdrained' or hysteresis tests give larger χ values at a given S than normally drained samples, the difference being very marked at high degrees of saturation. This is quite reasonable as it was pointed out in section 2.6.4. that isolated air bubbles do not invalidate the conventional effective stress law and χ can be 1.0 even though S is less than 100%. During wetting under reduced (u_a-u_w) some air is trapped in the pores and these pores will then behave as virtually saturated ones for effective stress calculations. Hence χ is greater for a given S than in a comparable normally drained sample in which presumably none of the air is present in isolated bubbles.

The agreement between theory and experiment for the strictly controlled tests on Braehead silt is heartening, but to present a more general picture of the present state of knowledge of the behaviour of partly saturated soils FIG. 6.25 has been prepared. This graph contains representative samples of all χ-S plots which may be deduced from tests on compacted and normally consolidated soils done recently at Imperial College, and shrinkage tests on natural samples done at the

Road Research Laboratories. The χ values refer to both volumetric and strength change tests.

The first impression given by FIG. 6.25 is one of considerable disorder. However, on inspection several features emerge. For high degrees of saturation all curves except the volumetric relationship for Weald Clay have χ values slightly higher than S (expressed as a ratio) as expected from the general considerations given earlier in this thesis.

However all curves show a tendency to drop towards the $\chi = S$ line, four of them crossing it between wide limits of S. The low χ values obtained for the lower sections of the curves are probably due to the influence of the soil structure on the mechanism of failure in partly saturated soils (Blight, 1961) and there is a rough correlation between the shape of the χ-S curve and the composition of the soil. FIG. 6.25 (a) shows the relationship between the % clay fraction and the values of S for which $\chi = 0.75$.



SUMMARY OF X-S RELATIONSHIPS

FIG. 6.25

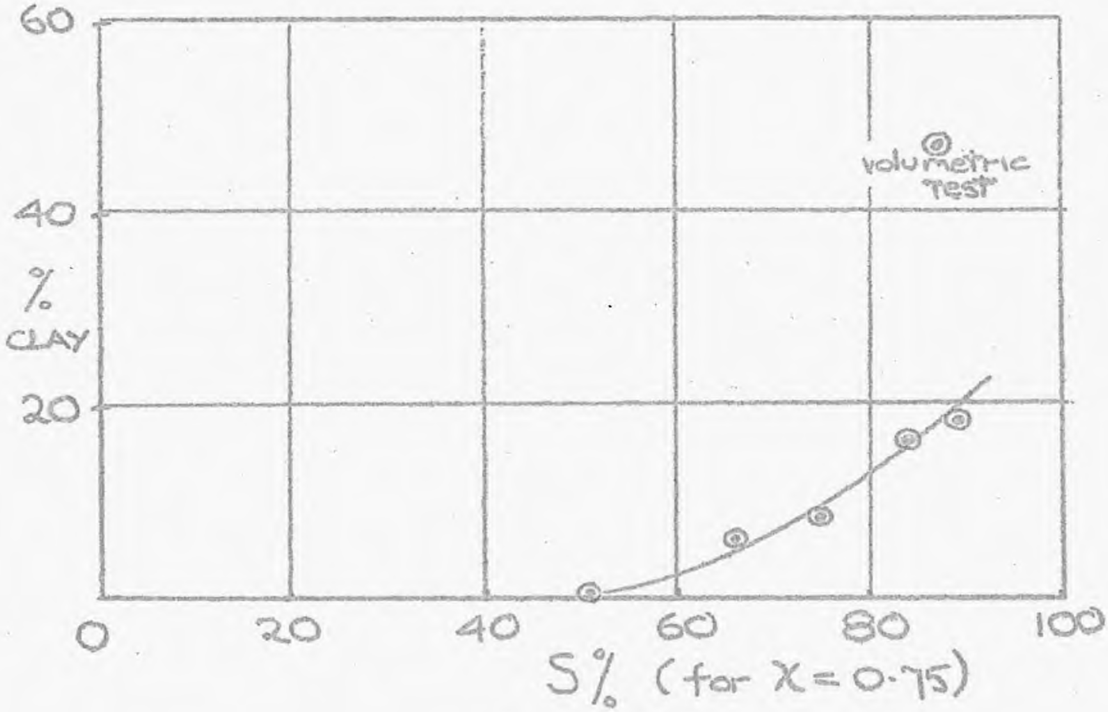


FIG. 6.25 (a)

Hence an estimate of χ can be made if the % clay fraction of a soil is known, either from a series of correlations like FIG. 6.25(a) or from reference to FIG. 6.25. Tests on a wider range of soils are obviously desirable to establish any correlations firmly but as they are lengthy and difficult this may require some time.

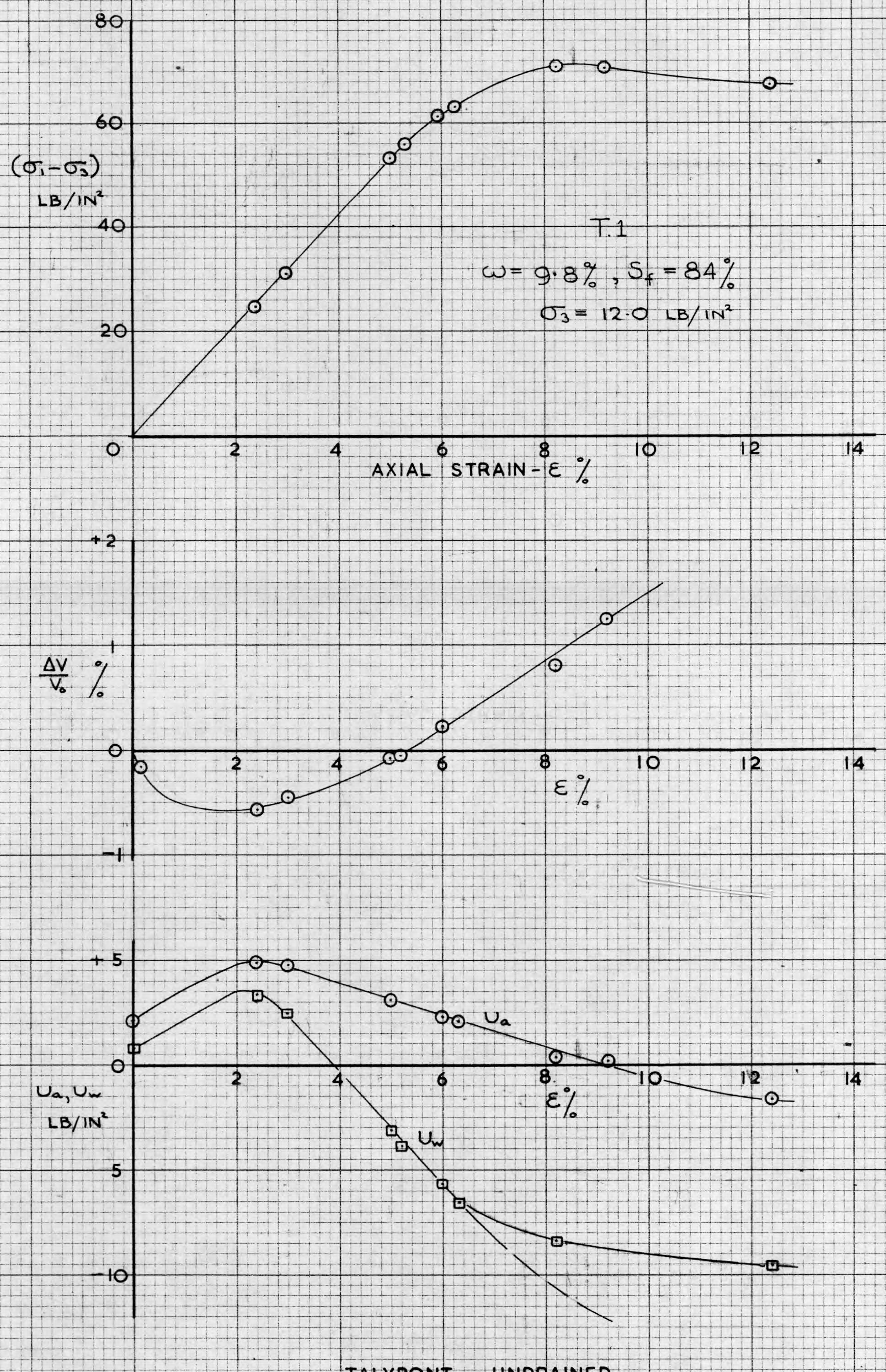
(6.5).3. <u>Shear tests on Talybont Clay</u>

No completely undrained compression tests on partly saturated soils have been reported in soil mechanics literature. As the development of the mercury cell had made them possible, tests were carried out on Talybont boulder clay to investigate the true pore pressure changes during shear and compare them with changes in constant water content tests.

The technique used for these tests has already been given in section (4.7). All tests were done at approximately the same water content (average 9.75% = optimum + 0.15%). This water content was chosen as the lowest which could be tested without the pore-water pressure reaching the apparatus limit. This is a limitation to fully undrained tests and the decrease in pore water pressure during shear means that, at present, many compacted soils cannot be tested except at water contents somewhat wet of the optimum water content.

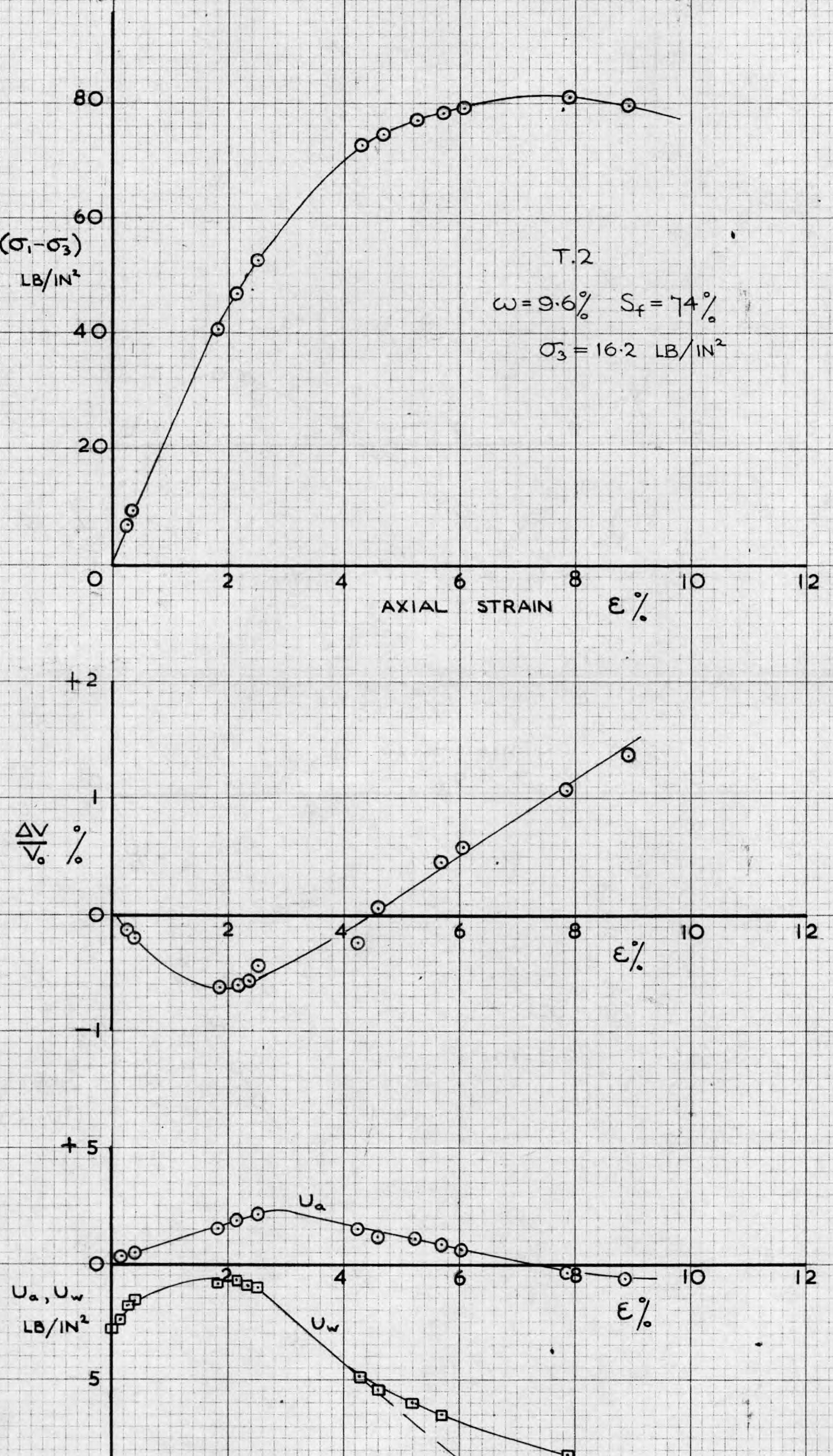
The results of the tests are given in FIGS. 6.26 - 6.30. All samples were compressed at 0.000085 inches per minute. In all tests the deviator stress built up to a well defined maximum value, the strain at max. deviator stress increasing with increase in cell pressure σ_3. The sample volume decreased for the first 2-3 per cent strain and then increased steadily until past failure. Higher cell pressures slowed the rate of volume increase but, at failure, all samples were less dense than at the start of compression. Failure was accompanied by the appearance of shear planes after initial bulging.

The pore air pressure changes should approximately mirror the shape of the volume change curve. This was observed for T_1 and T_2 (with cell pressures of 12.0 and 16.2 lb/in^2 respectively) although there was a slight time lag in reading - as evidenced by the failure to return to the exact initial air pressure when the overall volume change had returned to zero. However for T_3, T_4 and T_5 the measured air pressure



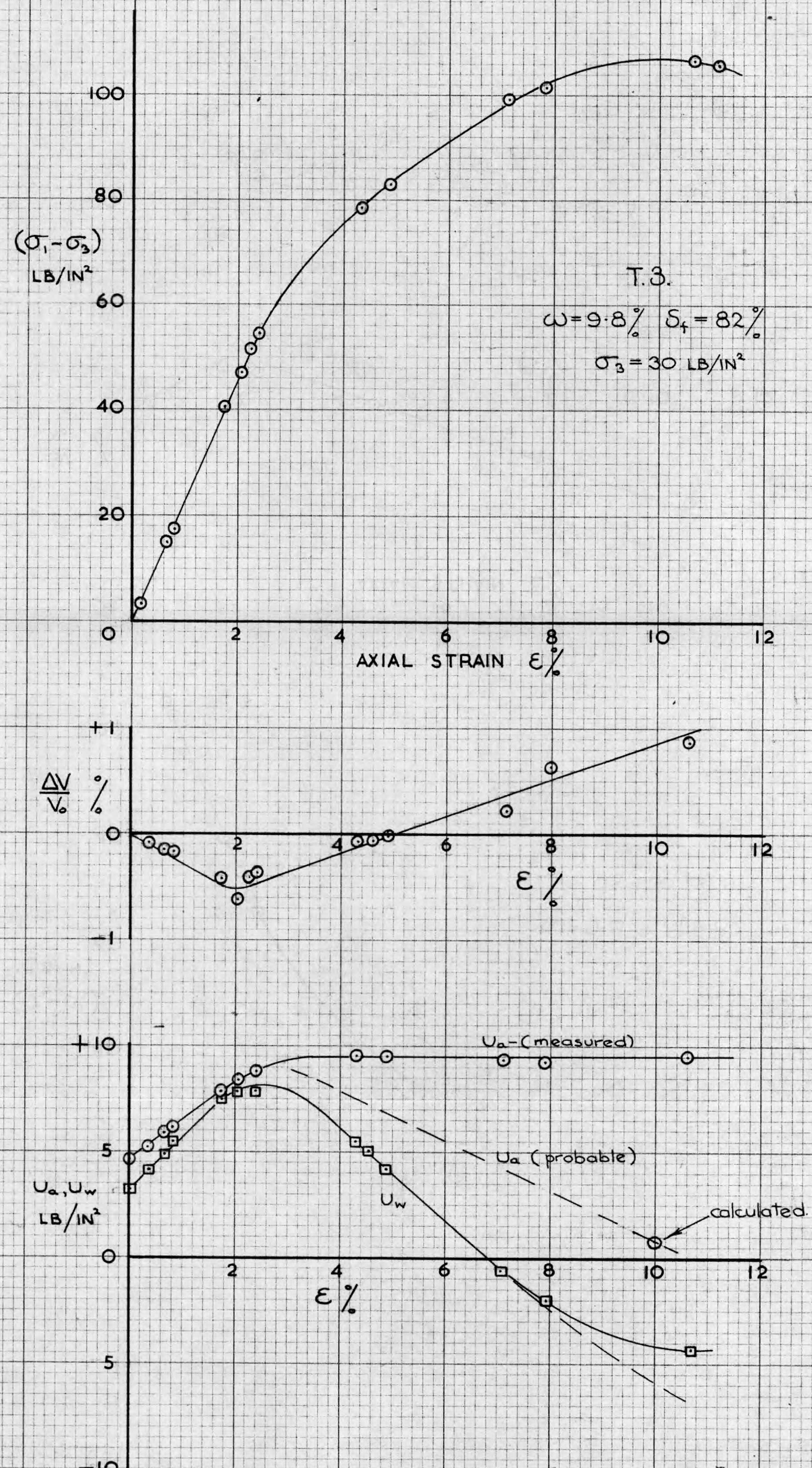
TALYBONT — UNDRAINED
COMPRESSION TEST
(UNDER Hg)

FIG. 6.26



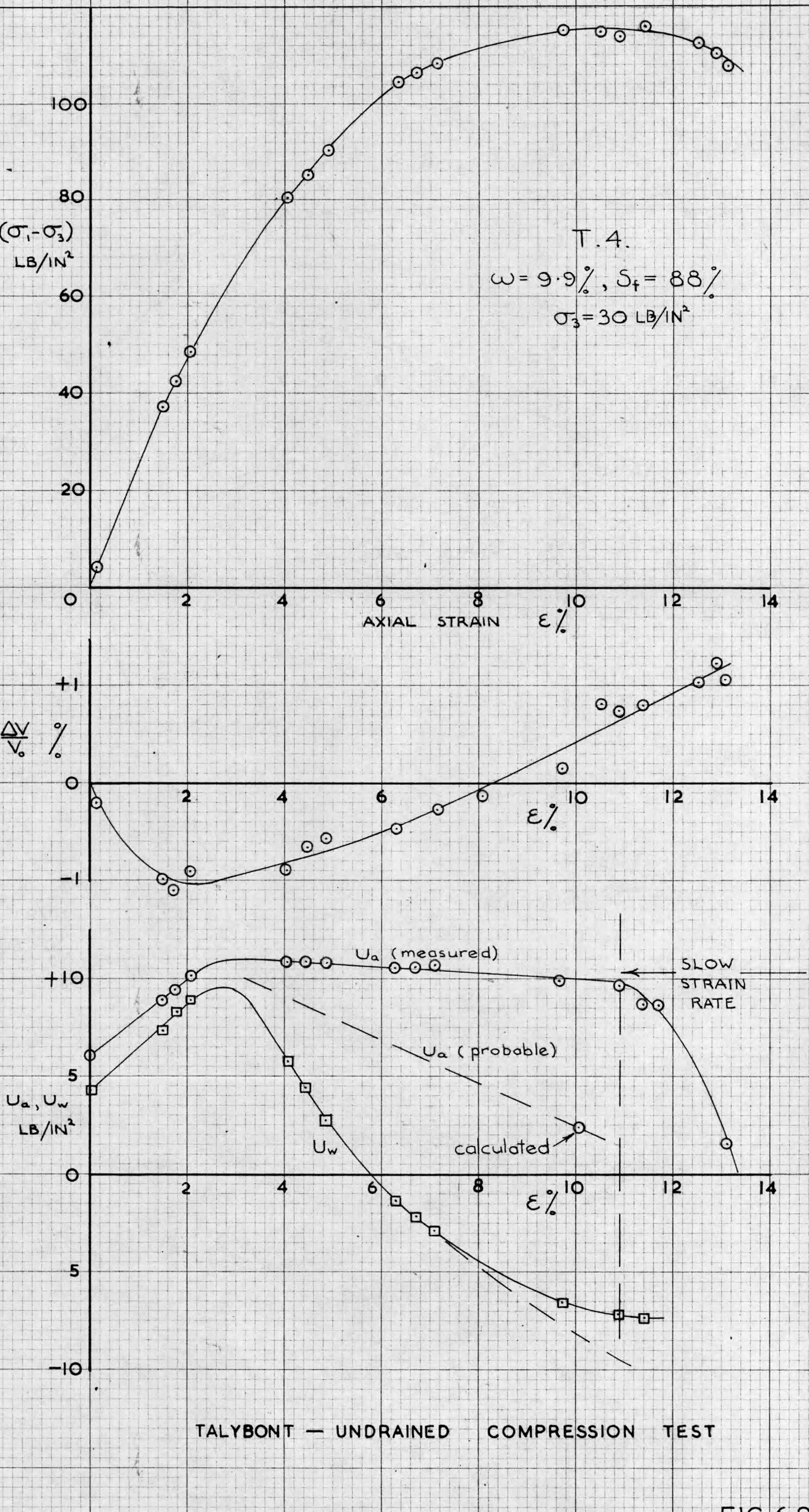
TALYBONT – UNDRAINED COMPRESSION TEST

FIG. 6.27



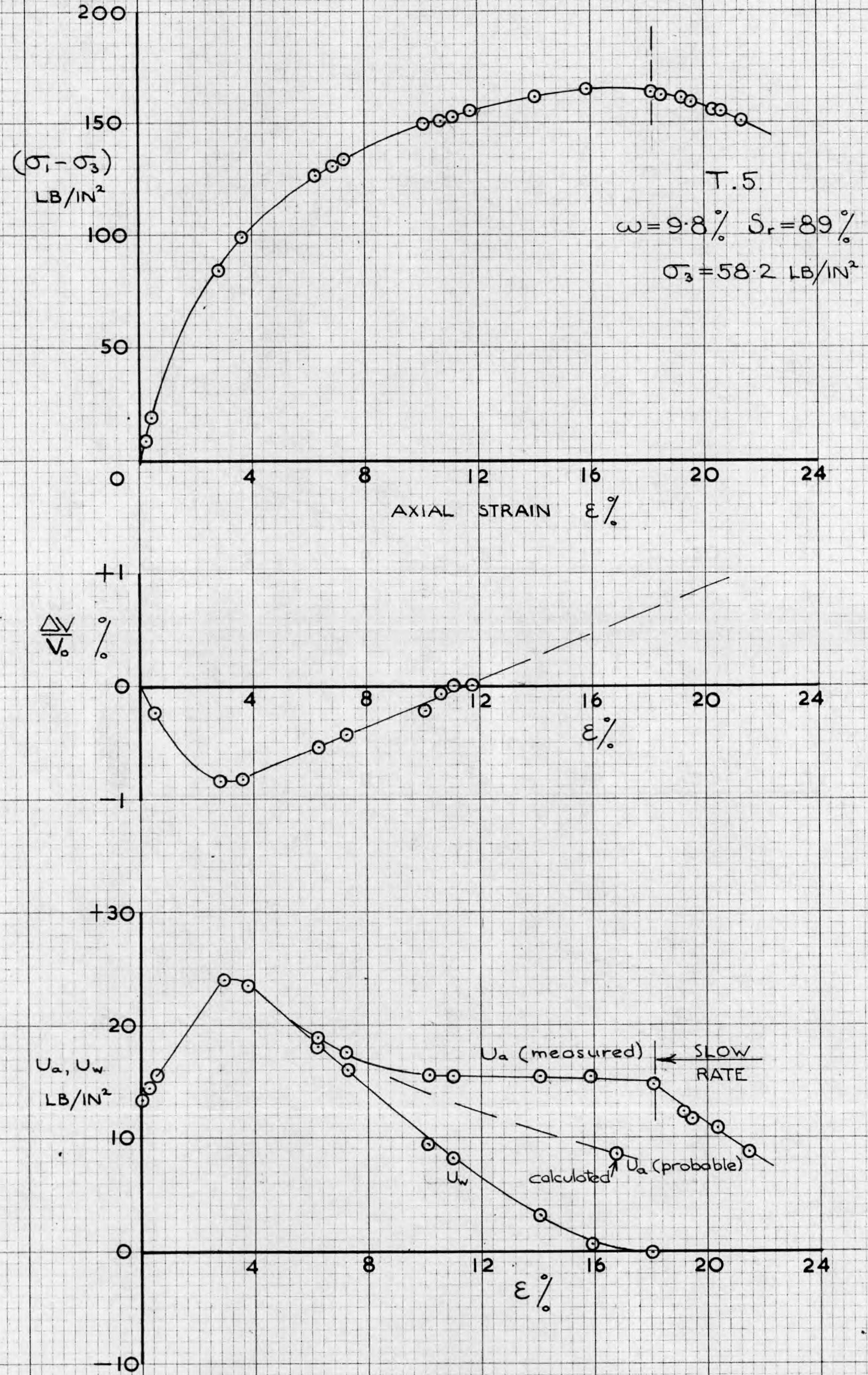
TALYBONT — UNDRAINED COMPRESSION TEST

FIG. 6.28



TALYBONT — UNDRAINED COMPRESSION TEST

FIG. 6.29



TALYBONT — UNDRAINED COMPRESSION TEST

FIG. 6.30

tended to remain constant after reaching its maximum value (or for T_5 after separating from the water pressure). These tests had cell pressures of 30, 30 and 58.2 lb/in^2 respectively with correspondingly higher axial loads than T_1 and T_2. Under high stresses it is possible that the soil pores adjacent to the air pressure measuring system became virtually closed, trapping some air in the system at a pressure bearing no relationship to the value in the soil voids at a later stage in the test.

After reaching maximum deviator stress sample T_4 was put onto the slowest obtainable strain rate and the air pressure gradually dropped to near its value calculated from the volume change. The same technique was used for T_5 with similar results. For T_3, T_4 and T_5 the correct pore-air pressures at failure were calculated from the volume changes using the Hamilton method described in section 2.3.

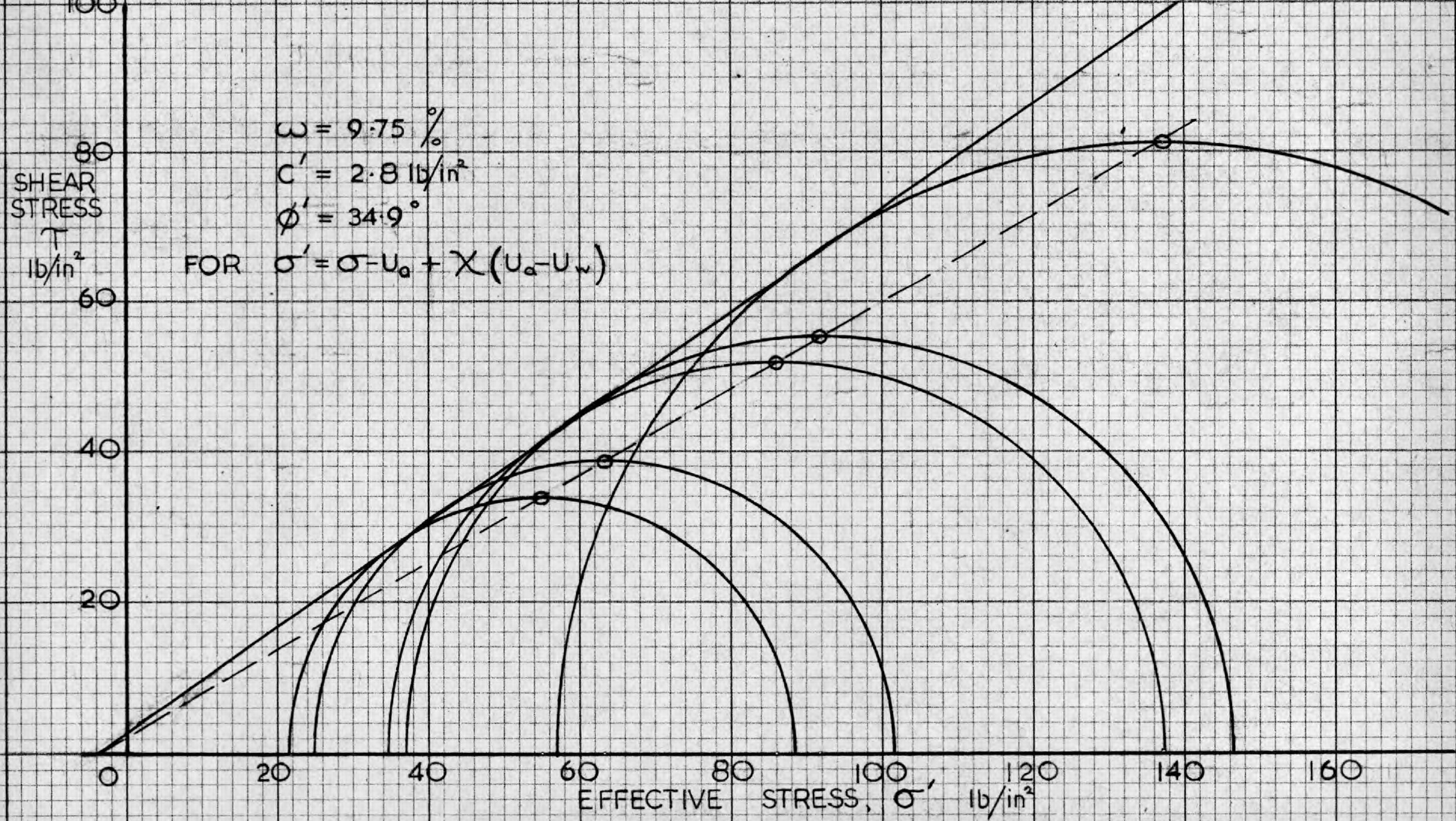
The pore-water pressure changes also roughly mirrored the volume change curves but, in contrast to the air pressures, measured water pressures were more reliable for the tests at high cell pressures. The samples had been standing on the porous discs for over a week before failure giving ample time for the pore-air to diffuse into the de-aired water in the pore pressure system. Samples with low confining pressures tend to generate large negative pore water pressures during shear and it is difficult to avoid breakdown of the pore pressure measuring system. Once the pore pressure measuring device had become insensitive the measured suctions were probably too small and an estimate of the probable u_w in the sample was made by extrapolating the earlier reliable readings. The values of u_a, u_w and $(\sigma_1 - \sigma_3)$ used for plotting the results are summarised in TABLE 6.6. Measured values of $(\sigma_1 - \sigma_3)$ were corrected for dilatation, using a correction of $(\sigma_3 - u_a).\frac{dv}{d\varepsilon}.\frac{A_o}{A}$ as for the silt tests.

TEST NO.	C σ_3 LB/IN²	T $\frac{(\sigma_1-\sigma_3)}{2}$ LB/IN²	U failure u_a LB/IN²	failure u_w LB/IN²	S_f	(u_a-u_w) LB/IN²	GENERALISED $\sigma_3' = (\sigma_3-u_a) + S(u_a-u_w)$*	GENERALISED $\frac{\sigma_1'+\sigma_3'}{2}$ LB/IN²	CONVENTIONAL $\sigma_3' = \sigma_3-u_w$	CONVENTIONAL $\frac{\sigma_1'+\sigma_3'}{2}$ LB/IN²
T_1	12.0	33.6	0	-11.2	0.84	11.2	21.4	55.0	23.2	56.8
T_2	16.2	38.5	-1.8	-11.0	0.74	9.2	24.8	63.3	27.2	65.7
T_3	30.0	51.3	+1.4	- 6.0	0.82	7.4	34.7	86.0	36.0	87.3
T_4	30.0	55.0	+2.5	- 8.2	0.88	10.7	36.9	91.9	38.2	93.2
T_5	58.2	80.0	+8.7	0	0.89	8.7	57.2	137.2	58.2	138.2

* Taking χ = S.

TABLE 6.6. CONDITIONS AT FAILURE FOR UNDRAINED TESTS ON COMPACTED CLAY

In FIGS. 6.31 and 6.32 the results are presented as Mohr failure envelopes. FIG. 6.31 is based on the generalised effective stress



TALYBONT — GENERALISED EFFECTIVE STRESS CIRCLES

FIG. 6.31

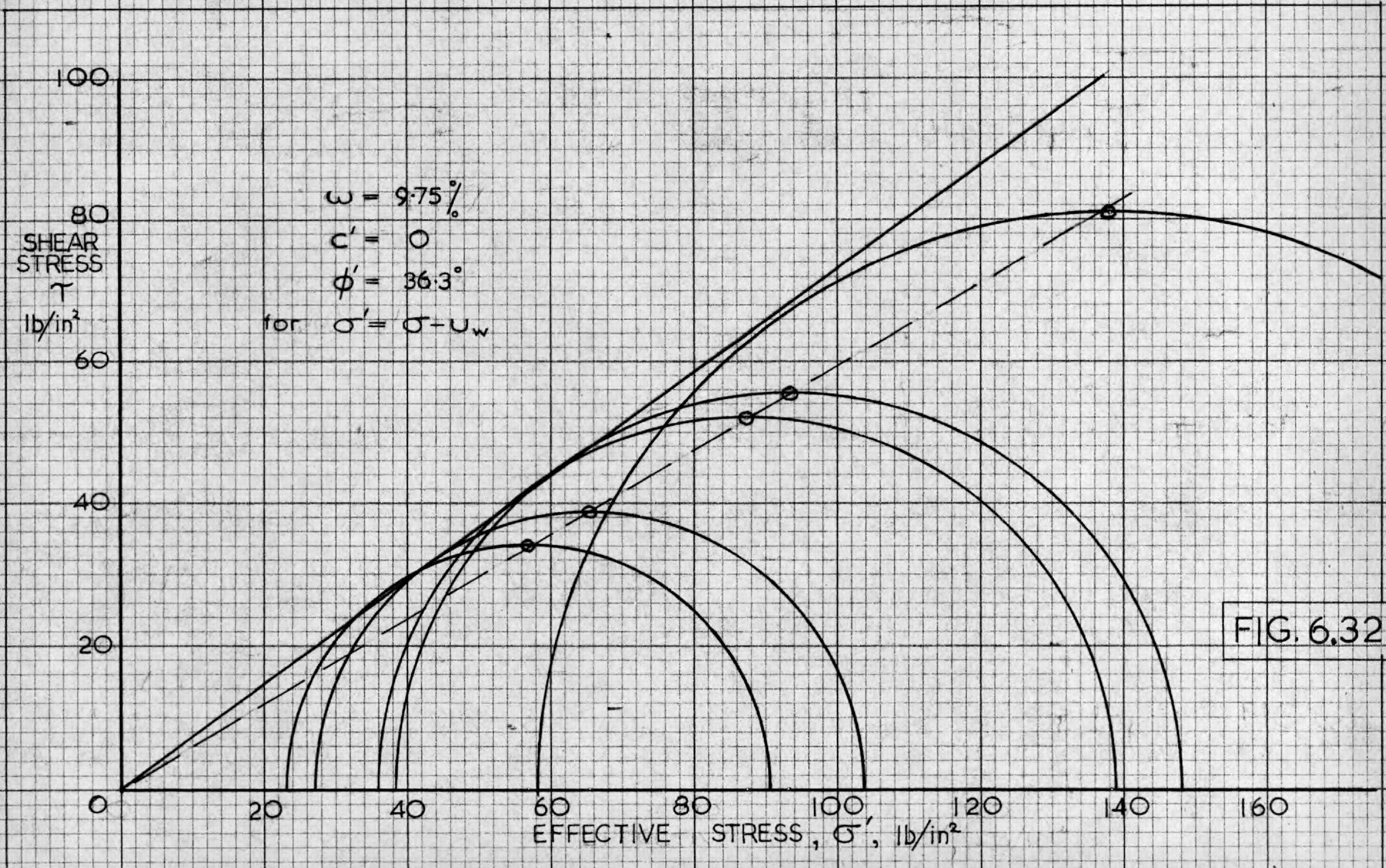


FIG. 6.32

TALYBONT — CONVENTIONAL EFFECTIVE STRESS CIRCLES values in TABLE 6.5, taking account of the χ factor and the air-water pressure difference. FIG. 6.32 is plotted from 'conventional' effective stress calculations, i.e. assuming $\sigma' = \sigma - u_w$.

To assist in drawing the failure envelopes the point at the top of each circle was marked and the line of best fit drawn as on the figures. The intersection of this line with the σ' axis must also lie on the tangent envelope which can then be drawn more easily without tending to weight the influence of the larger circles. The following values were obtained:

'generalised' stress	$c' = 2.8$ lb/in^2; $\phi' = 34.9^\circ$
conventional stress	$c' = 0$; $\phi' = 36.3^\circ$.

The method of result presentation is seen to have a marked effect on the apparent cohesion and an appreciable effect on the angle of shearing resistance. Blight (1961) tested samples of Talybont clay compacted at the optimum water content and then saturated under back pressure. The effective stress parameters found were $c' = 0$ and $\phi' = 36.0^\circ$. Blight's χ factors in FIG. 6.25, for Talybont clay, were all greater than S for the degrees of saturation in TABLE 6.6, though still less than one. However, his results were corrected for dilatation against the full generalised effective stress and application of the dilatation correction used for the Braehead silt yields χ values of 1.0 at degrees of saturation above about 80%. The χ-S curve for this correction has been given by Bishop et al (1960). $\chi = 1$ is then consistent with the values of c' and ϕ' obtained from the 'conventional effective stress' plot for both saturated samples and true undrained tests on partly saturated samples. For soils with higher clay content the differences in effective stress parameters determined from conventional and generalised effective stresses will be greater than for Talybont clay (Blight, 1961).

Constant water content tests on Talybont clay could be used to define the effective stress failure envelope just as accurately as true undrained compression tests, but the only reliable way of assessing the undrained strength of a particular soil at a given water content and condition of total stress is to perform a truly undrained compression test.

(6.6) Summary - Partly saturated soils

The techniques for the measurement of suction in partly saturated soils are the same as for saturated soils, the 'translation of origin' method being equally valid in either case for suctions beyond the directly measurable range.

The initial pore-air and pore-water pressures in compacted soil were measured as functions of water content. Both pressures were found to be sub-atmospheric for unconfined samples. The responses of pore pressures to changes in applied stress were investigated using the mercury cell, a device for ensuring true undrained conditions. In partly saturated soil both pore pressures changed by less than the applied pressure and parameters B_a and B_w were evaluated for a range of water contents.

It was shown that changes in u_a calculated from a theory based on Boyle's and Henry's laws of gaseous behaviour were consistent with experiment, though changes in u_w and hence in the suction $(u_a - u_w)$ had to be measured experimentally. Errors in effective stress arising from incorrect measurement or use of the pore pressures were discussed.

The parameter χ in the Bishop effective stress law was investigated for volume changes by analysis of shrinkage data. For clay soils low χ values were obtained for most degrees of saturation.

Axial compression tests on partly saturated silt established the correctness of form of the equation:

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \qquad \text{.........} (2.21)$$

and enabled χ to be investigated under carefully controlled conditions. The parameter was found to depend on the drainage history of the soil but was independent of the type of test - drained at constant suction, constant water content with low $(\sigma_3 - u_a)$, constant water content (moderate $\sigma_3 - u_a$), or volume change test. The χ-S relationship for 'normally drained' silt agreed reasonably well with theory.

Consideration of all available χ- S data indicated the importance of the parameter but also highlighted its wide range of possible values for a given degree of saturation.

Finally a short series of fully undrained compression tests on compacted Talybont clay was described. In spite of difficulty in measuring the pore air pressure under high axial stresses the pore pressure changes during shear were consistent with the volume changes. The effective stress failure envelope was plotted in terms of both generalised and conventional effective stresses, appreciable differences being found in c' and ϕ'.

CHAPTER 7

CONCLUDING REMARKS

The experimental data in the two preceding chapters has shown how the generalised effective stress equation

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \qquad \ldots\ldots\ (2.21)$$

can be applied to the macro-behaviour of saturated and partly saturated soils. For saturated soils χ was shown to be 1.0 and eqn. (2.21) then reduces to the familiar effective stress equation

$$\sigma' = \sigma - u_w \qquad \ldots\ldots\ (1.1)$$

The parameter χ was found to have a wide range of possible values at any degree of saturation S, but for any one soil prepared with a standardised technique, χ showed a smooth variation with degree of saturation, S. For samples draining from initial saturation the χ - S relationship was predictable from theory with fair accuracy, at least for relatively incompressible soil. Initially saturated clay soils gave low experimental χ values, deduced from volumetric changes, even at high S values, and extensions to the theoretical approach would be desirable to cover this important group of soils. The χ - S plots for soils prepared by dynamic compaction showed wide variations but rough correlations between clay content and the value of

S for a given χ were observed. Future tests may serve to support these correlations or, by introducing other factors besides particle size, replace them by more accurate methods for predicting χ.

The values of χ, as they apply to shear strength phenomena are very important, particularly for soils with high clay content and low external applied stresses. Under these conditions the effective stress parameters c' and ϕ' depend greatly on the form of the effective stress law used.

It is also essential to measure the pressure difference or suction $(u_a - u_w)$, accurately - a requirement which has not been met by much previous soil mechanics work. The experiments described in this thesis have shown the importance of the $(u_a - u_w)$ term in determining effective stresses, and hence macro behaviour, in both saturated and partly saturated soils. When u_a is small, u_w may have large negative values at all degrees of saturation. For some soils this implies an absolute tension in the pore water and, although this tension cannot be measured directly, the behaviour of the soils tested was compatible with its existence.

In an experiment planned with G.E. Blight and performed by him (Blight, 1961) two clay samples compacted at the same water content were sheared - one with u_a elevated sufficiently to make u_w positive, the other with $u_a = 0$. The initial $(u_a - u_w)$ was 63 lb/in^2 and both samples gave identical stress-strain curves, i.e. the effective stresses were the same even though, according to the capillary hypothesis, the water in the second sample was in a state of tension for the duration of the test.

The measurement of pore-water pressures in the 0 to -1 atmosphere range also presents some difficulties, particularly in long term tests, and there is a definite need for a device which can measure suctions between, say, 0 and 100 lb/in^2 quickly and reliably. This range would cover many problems of engineering interest. True undrained tests on compacted soils are very restricted in range at the moment by the lack of such a device.

Finally it is believed that the application of some established soil physics techniques to soil mechanics measurements, particularly of shear strength, presented in theses by Alpan (1959), Blight (1961) and the present volume has helped to clarify many aspects of soil macro-behaviour. Although the ultimate aim is complete understanding of the processes involved, much useful work remains to be done at the semi-empirical level of the work reported in these theses.

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Dr. A.W. Bishop supervised the project throughout and his continued interest, encouragement and suggestions played a major part in shaping its course. Acknowledgement is also made of helpful discussions on special topics with Drs. D.J. Henkel, R.E. Gibson and N.N. Ambraseys of the staff of the department.

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APPENDIX 1

REMOULDING OF SATURATED CLAYS AT CONSTANT WATER CONTENT AS A CONSTANT VOLUME PROCESS

In section 5.5 (b) remoulding of a saturated clay without change in water content was assumed to take place at constant volume. Undrained shear tests on saturated soils are also assumed to take place without volume change. However, it was pointed out in Chapter 1 that at least a part of the pore water in clay soils is affected by surface forces emanating from the clay particles and Low (1958) has measured water densities of 0.97 - 0.98 $gm.cc.^{-1}$ at distances of approximately 10 $\mathring{A}$ from particle surfaces. The density was appreciably affected at distances of up to 80 $\mathring{A}$.

If remoulding is vigorous enough to disturb the adsorbed water then a second order volume change might occur as the percentage of highly oriented water is reduced. The effect was investigated for the London Clay used in the tests described in Chapter 5.

A 4" diameter mould was filled with clay slurry made by depositing ground oven dry clay under water in thin layers and evacuating after each layer. The bottom of the mould was connected to a source of 1½ lb/in^2 suction and the soil allowed to consolidate for several weeks. Two 1½" dia. samples were then cut with thin-walled sampling tubes and triaxially consolidated under pressures of 15 and 30 lb/in^2

respectively.

From each consolidated specimen four rough spheres, of approximately ¾" dia. were cut with a wire saw and immediately painted with a latex coating. A thin wire hook was stuck to the coating and the spheres given three dippings in latex. After drying, the spheres were placed in distilled water in a constant temperature bath and left for four or five days. After this time the gain in weight of a sphere in a further week was less than 1 milligram.

Before remoulding, each sphere was weighed submerged in water at 20^{o}C, the wire hook supporting it from the balance arm. The balance used was sensitive to one milligram. The sphere was then remoulded for ten minutes without damaging the latex envelope, the remoulding being done manually with constant dippings of the sphere in water to prevent drying out. After remoulding the sphere was again submerged in water and its buoyant weight observed until it had become constant. Three spheres from each batch were remoulded, the fourth being kept as a control.

Although there was appreciable variation in the results two typical sets are presented in the table below:

	SERIES 1	SERIES 2
Water Content	w = 40.3 %	w = 36.5 %
Sample volume (c.c)	6.036	9.027
Submerged weight (gm) (before remoulding)	4.752	6.896
Submerged weight (gm) (40 secs. after remoulding)	4.747	6.890
Submerged weight (15 mins. after remoulding)	4.752	6.896
Final volume change	$<$ 1 in 6,000	$<$ 1 in 9,000

In each series the sample appeared to increase in volume immediately after remoulding but within 15 minutes it had returned to its original buoyant weight ($\pm$ 0.001 gm). The decrease in density may be mainly a temperature effect as the work done in vigorous remoulding will undoubtedly elevate the temperature inside the sphere. One sphere from Series 2 was weighed submerged in water chilled to 14°C (i.e. equivalent to a 6°C temperature rise inside the soil). The immediate loss of buoyant weight was 0.006 gm. which is the loss for the sample quoted in the table, and 0.005 gm were regained in 15 mins. as the temperature gradient evened out.

Hence, for this particular clay, remoulding appears to alter the volume of the soil by less than 1 part in 9,000 if time is allowed for temperature equilibrium, and the normal 'no volume change' assumption is valid to a high degree of accuracy. For vigorously remoulded samples some of the change in pore pressure with time after remoulding (see Chapter 5) may be due to temperature gradients.

It would be instructive to repeat the remoulded sphere tests on a range of clays, particularly those exhibiting large secondary stress-strain phenomena. A brief attempt was made to test a Na-Bentonite but the water content was too high for the latex to set as an impervious envelope.

APPENDIX 2

SUCTION IN UNDISTURBED SAMPLES

In section (5.3)(c) values of suction measured in unconfined undisturbed samples of two clays were given. In all but the most recent statement of eqn. (2.7), viz:

$$u = \alpha p + s$$

(with $\alpha = 1$ for saturated soil)

it has been implied that the suction should numerically equal the effective overburden stress which was removed by sampling. Table 5.2 showed that the unconfined suction could be much less or much greater than the overburden stress depending on whether the soil was normally consolidated or over consolidated.

Skempton (1961) has recently treated the problem from a semi-empirical viewpoint, measuring the parameter A_s, which is similar to A in eqn. (2.1) but represents stress release on sampling. Skempton finds:

$$K_o < 1 \qquad -u_w = \sigma_v' \left[K_o - (A_s - \tfrac{1}{3})(1-K_o)\right] \quad .... \text{(A2.1)}$$

$$K_o > 1 \qquad -u_w = \sigma_v' \left[K_o + A(1-K_o)\right] \quad .... \text{(A2.2)}$$

where σ_v' = vertical effective stress in ground

$$K_o = \frac{\sigma_h'}{\sigma_v'}$$

σ'_h = horizontal effective stress in ground.

The peaty clay from Lagos, Nigeria, was fairly sensitive and had undoubtedly been a little disturbed by sampling, although visual inspection of the structure gave no evidence of this. A reasonable mean value of $\frac{-u_w}{\sigma'_v}$ for this clay would be about 0.5, which is obviously inconsistent with eqn (2.7) which may be rewritten for saturated soils as

$$p - u = \sigma'_v = -u_w \qquad \text{.........} \quad (A2.3)$$

$$\text{i.e. } \frac{-u_w}{\sigma'_v} = 1.0$$

If we assume $K_o = 0.75$, a reasonable value for normally consolidated clay, then from (A2.1)

$$-\frac{u_w}{\sigma'_v} = K_o - (A_s - \tfrac{1}{3})(1 - K_o) = 0.75 - 0.25(A_s - \tfrac{1}{3}) = 0.5$$

$$\text{i.e. } A_s = +1.3$$

And if $K_o = 0.7$* $\qquad A_s = +1.0$

This value of A_s is quite comparable with values of A measured

---

* Some typical K_o values have been given by Bishop (1958).

in undrained triaxial compression test for soft normally consolidated clays. Current work at Imperial College is investigating the range of values of A and A_s and there seems no reason for them to be widely different.

The more recent (1960) version of eqn. (2.7) replaces p by $\frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3}$ though Croney and Coleman do not give any indication of how the stress invariant is to be estimated.

The Croney-Coleman expression may be rewritten:

$$-s = \sigma_v' \frac{(1 + 2K_o)}{3} \qquad \text{.........} \quad (A2.4)$$

which, for $\frac{-u_w}{\sigma_v'} = \frac{-s}{\sigma_v'} = 0.5$,

gives $K_o = 0.25$, an unrealistically low value.

The figures for overconsolidated clay from Bradwell are even more interesting, as the high suctions can only be explained by the presence of high lateral stresses in the ground; i.e. $K_o > 1$. There are theoretical reasons for expecting high K_o values in heavily overconsolidated clays and analyses of slope failures in these materials help to confirm their existence. (La Rochelle, 1960). Although the suction data given is very scanty, if we assume $\frac{-u_w}{\sigma_v'} \simeq 2.1$, then equation (A2.2) gives, for A = + 0.25 (an average value from shear tests) $K_o = +2.5$.

This is also the value derived by La Rochelle from slope stability analyses.

Equation (A2.4) gives $K_o = 2.65$ which, though higher, is still compatible with the above value.

The geological significance of the high in-situ lateral stresses has been dealt with by Skempton (1961).

APPENDIX 3.

EQUILIBRIUM OF AIR IN SOILS

In section 6.3 the change in pore-air pressure with time was attributed to equalisation of an initially non-uniform pressure distribution in the bubbles throughout the sample. If there are bubbles in a sealed sample at equilibrium then they must be all of the same pressure (and hence diameter) otherwise a concentration gradient of air dissolved in the water exists and flow will take place by diffusion until the air pressure is equal everywhere in the sample - both in bubbles and interconnected voids. This also means that air bubbles cannot permanently exist in a submerged sample as the concentration of air in the water is controlled by the atmospheric pressure and any bubbles must have a greater air pressure which will be gradually reduced by diffusion.

The disappearance of a free bubble under such conditions has been treated by Epstein and Plesset (1950) and Liebermann (1957). Although this does not relate directly to air pressure changes in sealed samples it is instructive to investigate the time required for complete solution of a bubble of any given diameter. With bubbles of many sizes in a sealed soil it is likely that some will disappear completely while others may grow slightly but at equilibrium all must have the same curvature and internal pressure.

The most complete solution available is that due to Epstein and Plesset (1950). In their theory at $t = 0$ a bubble of radius R_o is placed in a solution of concentration c_i. The governing equation was found to be:

$$\frac{dR}{dt} = \frac{K\,(c_i - c_s)}{\rho(\infty) + \frac{2T}{3R}} \left\{ \frac{1}{R} + \frac{1}{\sqrt{\pi K t}} \right\} \qquad \text{......... (A3.1)}$$

where

R = bubble radius at time t

K = diffusivity of gas in liquid

c_i = solution concentration at distance from bubble

c_s = solution concentration at surface of bubble - saturated at pressure in bubble

T = surface tension

$\rho(\infty)$ = density of air in bubble at same conditions of pressure and temperature with an interface of zero curvature, (i.e. with $u_a = u_w$)

For water initially air saturated at atmospheric pressure Epstein and Plesset's approximate solution of (A3.1) was modified to the following form:

$$\frac{t}{t_{R=o}} = 1 - \frac{\frac{R_o p_L}{2T}\left(\frac{R}{R_o}\right)^3 + \left(\frac{R}{R_o}\right)^2}{\left(1 + \frac{R_o p_L}{2T}\right)} \qquad \text{........ (A.3.2)}$$

p_L = water pressure; T = surface tension.

This dimensionless equation eliminates the diffusion coefficient - a parameter to which it appears difficult to assign an exact value. In FIG A. 3.1 the approximate theoretical solution is compared with experiment for a bubble with $R_o = 0.68 \times 10^{-2}$ cm. This bubble was trapped in water on the underside of a glass plate and its diameter measured with a microscope. The time for complete solution was measured as 73 minutes. The experimental curve agrees moderately well with the approximate theory. A numerical solution of A 3.1 was calculated for the first 10% diameter decrease of an 0.01 cm. radius bubble and agreed almost exactly with the approximate solution. No details will be given here but the approximate solution is most likely to be in error during the early stages of solution and this check was quite heartening.

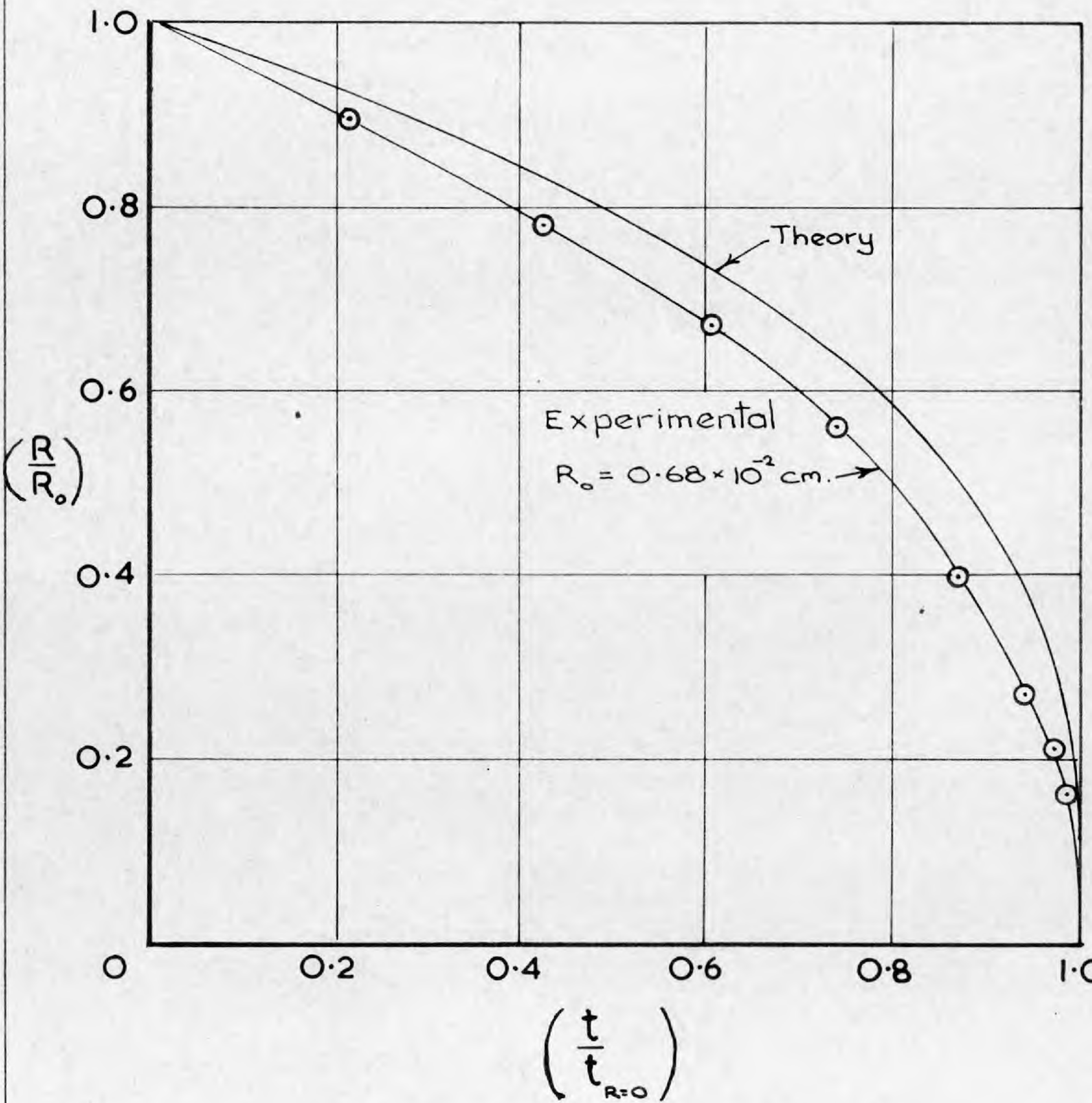
TABLE A. 3.1 gives some typical values for times of complete

TABLE A.3.1

R_o - Radius (cm.)	$t_{R=0}$
	(a) Theoretical
10^{-3}	6.6 sec.
10^{-2}	98 mins.
10^{-1}	67 days
	(b) Measured
0.22×10^{-2}	12 mins.
0.68×10^{-2}	73 mins.
1.19×10^{-2}	364 mins.

solution calculated for $K = 2 \times 10^{-5}$ cm^2 sec^{-1} and measured in the laboratory under the microscope. The measured values are consistent with the theory though no attempt was made to investigate large bubbles!

In a sealed soil sample not all the air can dissolve in the pore water unless the sample is infinitely compressible. Theoretical solution of any realistic model is different, though equilibrium of initial pore-air conditions could be a factor in any time dependent changes of pore water pressure or strength. This is too large a subject to pursue here and the information in this Appendix should be considered merely as a guide to the orders of magnitude of times required for air equilibria in simple systems.



SOLUTION CURVES FOR AIR BUBBLE IN AIR SATURATED WATER

FIG.A3.1

APPENDIX 4

THE DILATATION CORRECTION FOR PARTLY SATURATED SOILS

In section (6.5)(iii) brief mention was made of a correction to the measured value of deviator stress, $(\sigma_1 - \sigma_3)$, to allow for the dilatation of the sample. Such a correction was found necessary (Bishop and Eldin (1953)) before drained and undrained tests on dense sand could be compared. No correction is necessary for undrained tests on saturated soil as no volume change can occur but, with partly saturated samples, appreciable volume changes can occur during shear of both drained and undrained samples. These changes have already been plotted as functions of the degree of saturation, S, for Braehead silt in FIG. 6.16.

The derivation of the correction is an extension of the method of Bishop (1954) for drained tests on saturated soils.

Fig. (A 4.1) shows a typical plot of volumetric strain, $v = \frac{\Delta V}{V_o}$, against axial strain $\epsilon = \frac{\Delta l}{l_o}$. The sample dimensions are shown at the start of compression and at the beginning and end of a small increment in strain, $d\epsilon$, at any stage of the test. If the test is a drained one, during the increment $d\epsilon$ a volume of water dV_w may drain from the soil and a volume of air dV_a enter the soil pores. If dV is

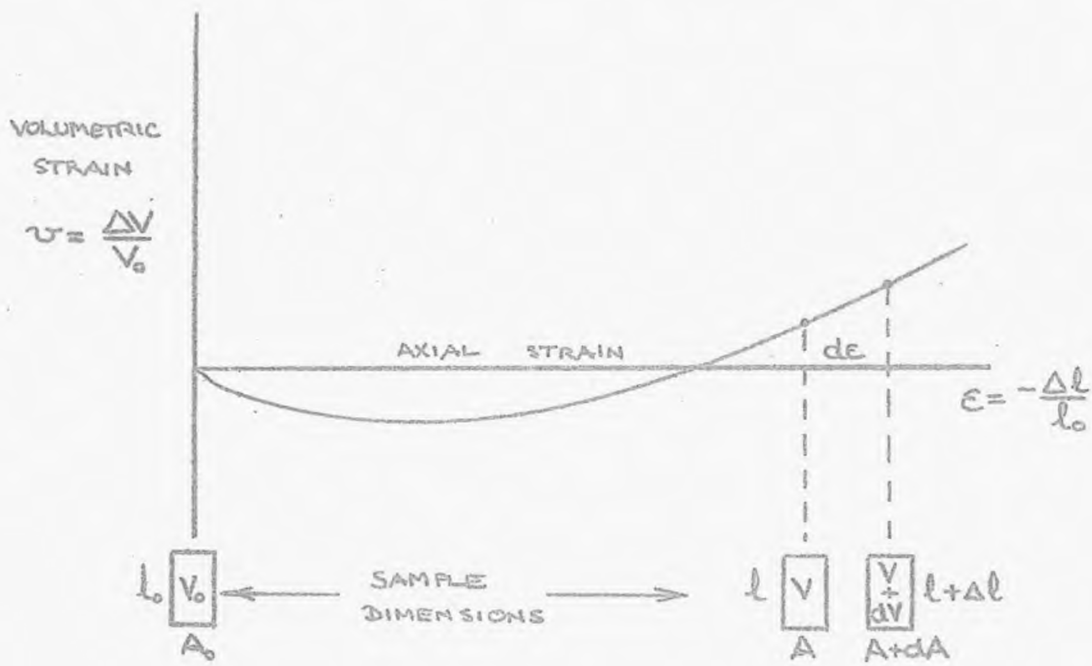


FIG (a 4.1)

the total sample volume increase then:

$$dV = dV_a - dV_w \qquad \text{............. A 4.1.}$$

The stresses are controlled at the following values

cell pressure σ_3

pore-water pressure u_w

pore-air pressure u_a.

Consider the work balance of the sample during the strain increment:

Work done by axial loading mechanism

$$dW_p = -A.(\sigma_1 - \sigma_3).dl = A(\sigma_1 - \sigma_3)\ l_o d\varepsilon \qquad \text{...... (A 4.2)}$$

Work done by cell pressure control

$$dW_\sigma = -\sigma_3.dV \qquad \text{......(A.4.3)}$$

Work done by pore-water pressure control

$$dW_w = -u_w.dV_w \qquad \text{............(A 4.4)}$$

Work done by pore-air pressure control

$$dW_a = u_a.dV_a \qquad \text{.......(A 4.5)}$$

The work done in overcoming true frictional resistance in the sample may be written as

$$dW_f = -(\sigma_1 - \sigma_3)_f.A.dl = (\sigma_1 - \sigma_3)_f\ A.l_o d\varepsilon \qquad \text{.......(A 4.6)}$$

As a first approximation this term may be assumed to absorb any imbalance of work done by the external stresses. This leads us to the following expression.

$$A(\sigma_1 - \sigma_3) l_o d\varepsilon - \sigma_3 dV - u_w dV_w + u_a dV_a = (\sigma_1 - \sigma_3)_f A l_o d\varepsilon \qquad \text{.......(A 4.7)}$$

i.e. $(\sigma_1 - \sigma_3)_f = (\sigma_1 - \sigma_3) - \frac{(\sigma_3 dV + u_w\, dV_w - u_a dV_a)}{A\, l_o d\varepsilon}$ ...... (A 4.8)

Introduce the notation

$$v = \frac{\Delta V}{V_o} \; ; \quad v_w = \frac{\Delta V_w}{V_o}$$

where ΔV, ΔV_w are total changes in V and V_w since the start of compression.

then $dv = \frac{dV}{V_o}$ and $dv_w = \frac{dV_w}{V_o}$ ....(A 4.9)

Eqn (A 4.8) may be rearranged, using (A 4.1) as

$$(\sigma_1 - \sigma_3)_f = (\sigma_1 - \sigma_3) - \frac{\left[(\sigma_3 - u_a)dV - (u_a - u_w)dV_w\right]}{A l_o d\varepsilon} \quad \text{.........(A 4.10)}$$

Now $\frac{dV}{A l_o d\varepsilon} = \frac{dV}{V_o} \cdot \frac{V_o}{A l_o d\varepsilon} = \frac{dV}{V_o} \cdot \frac{A_o l_o}{A l_o d\varepsilon} = \frac{A_o}{A} \cdot \frac{dv}{d\varepsilon}$ ....(A 4.11)

Similarly $\frac{dV_w}{A l_o d\varepsilon} = \frac{A_o}{A} \cdot \frac{dv_w}{d\varepsilon}$ ....(A 4.12)

Therefore $(\sigma_1 - \sigma_3)_f = (\sigma_1 - \sigma_3) - (\sigma_3 - u_a)\frac{A_o}{A} \cdot \frac{dv}{d\varepsilon} + (u_a - u_w) \cdot \frac{A_o}{A} \cdot \frac{dv_w}{d\varepsilon}$ ...(A 4.13)

For small strains and drained, saturated samples ($\left[u_a - u_w\right] = u_a = u_w = 0$) this reduces to Bishop's expression

$$(\sigma_1 - \sigma_3)_f = (\sigma_1 - \sigma_3) - \sigma_3 \cdot \frac{dv}{d\varepsilon} \quad \text{.............(A 4.14)}$$

In this equation the $-(\sigma_3 . \frac{dv}{d\varepsilon})$ term is regarded as a correction to be applied to the measured deviator stress $(\sigma_1 - \sigma_3)$ to reduce it to the deviator stress $(\sigma_1 - \sigma_3)_f$ required in a non-dilating sample at the same void ratio for overcoming frictional resistances between grains. For partly saturated samples the equivalent correction appears to be:

$$\text{correction to } (\sigma_1 - \sigma_3) = -(\sigma_3 - u_a)\frac{A_o}{A}\frac{dv}{d\varepsilon} + (u_a - u_w)\frac{A_o}{A}.\frac{dv_w}{d\varepsilon} \quad \ldots\ldots(A\ 4.15)$$

Note that the form of this expression once again stresses the importance of the pressure differences $(\sigma_3 - u_a)$ and $(u_a - u_w)$.

However, the derivation of the correction (A 4.15) is open to serious criticism in that it does not recognise any forms of energy loss or storage within the sample - e.g. heat losses or changes in surface area of air-water menisci. In fact for some clays drained and undrained tests do not correlate any better after correction than before (Henkel, 1958).

Originally the full correction (A 4.15) was applied to the tests on Braehead silt before calculating the χ factors as described. However, for degrees of saturation greater than about 70%, χ values rising well above 1 were obtained! These values were obviously wrong and were attributed mainly to a very optimistic addition to $(\sigma_1 - \sigma_3)$ arising from the $(u_a - u_w)\frac{A_o}{A}.\frac{dv_w}{d\varepsilon}$ term in the correction.

For an incompressible soil draining under the influence of an applied suction the work done by the suction control is accounted for by an increase in air-water surface area in the soil, surface tension being in fact the energy required to create unit surface area. This is the basis of a method for determining specific surface areas of soil particles (Payne, 1953).

There was no way of telling how much the air-water surface area was changing with strain but, as the other extreme to applying the full correction, it was assumed that all the energy represented by the water removed from the soil under suction was stored in increased meniscus area. This left a correction of $-(\sigma_3-u_a)\,\frac{A_o}{A}.\,\frac{dv}{d\varepsilon}$. As described in Chapter 6, results from all types of test gave consistent χ values when corrected in this fashion.

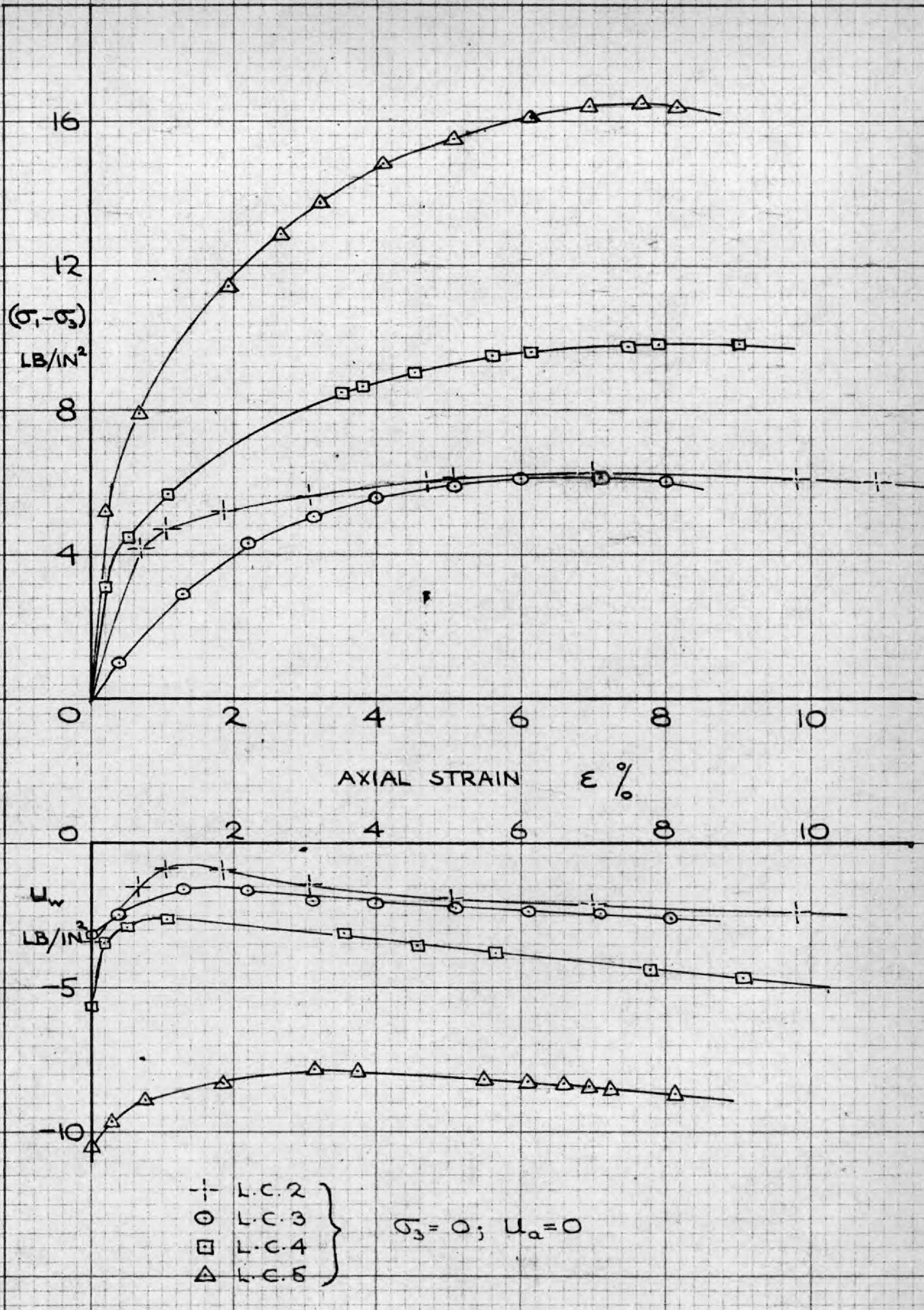
For constant water content tests the dv_w was automatically zero but, as the samples were increasing in volume at failure, there were undoubtedly internal changes in meniscus configurations. This effect was neglected in the energy balance and the correction of $-(\sigma_3-u_a).\,\frac{A_o}{A}.\,\frac{dv}{d\varepsilon}$ was applied to the measured $(\sigma_1-\sigma_3)$. The correction theory is then admittedly incomplete but the expression used can partially be justified theoretically and its effectiveness in giving consistent and reasonable correlations between the various tests was gratifying.

A P P E N D I X 5

TEST RESULTS

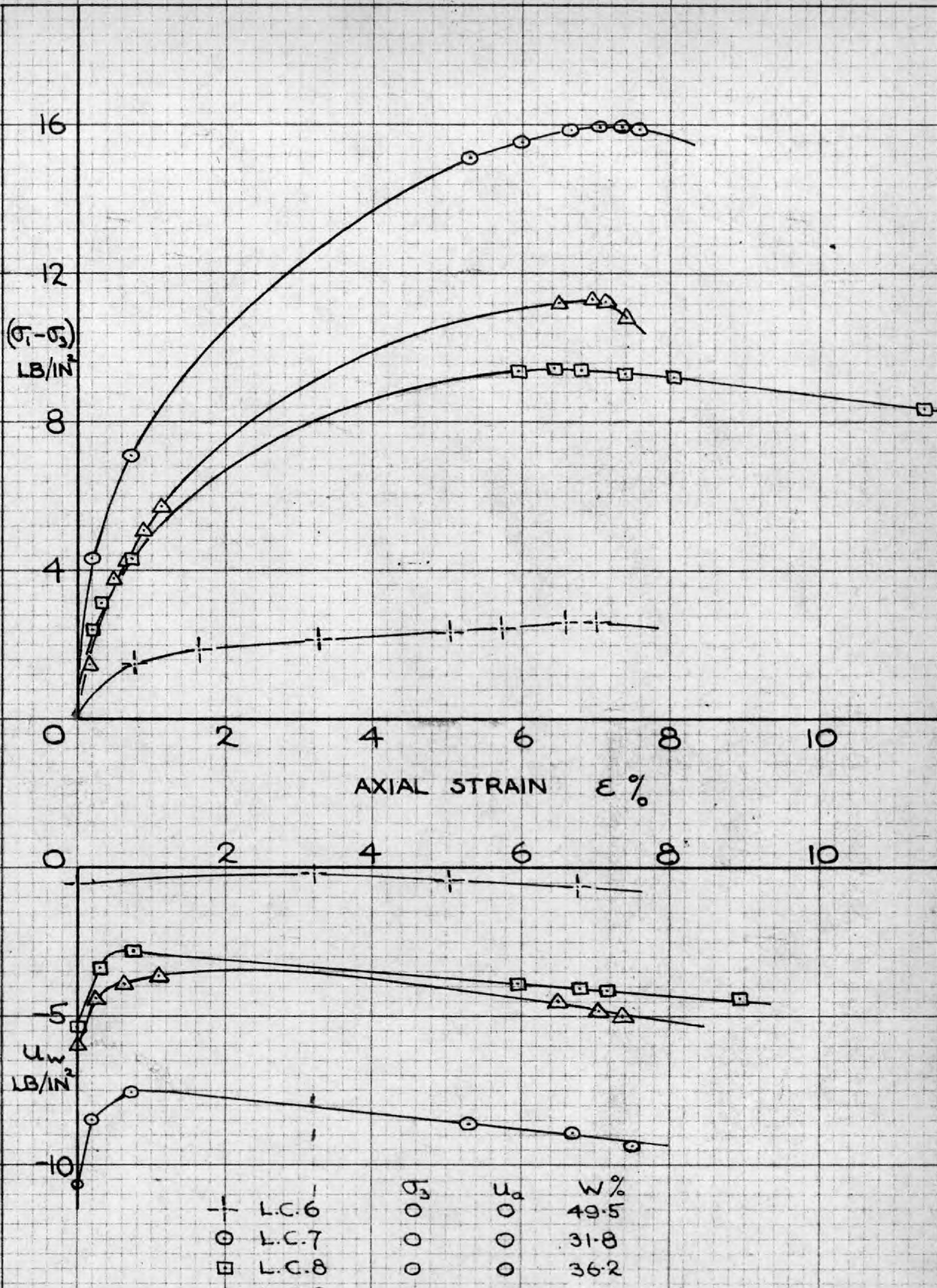
LONDON CLAY

(Corrected for Membrane Effect)



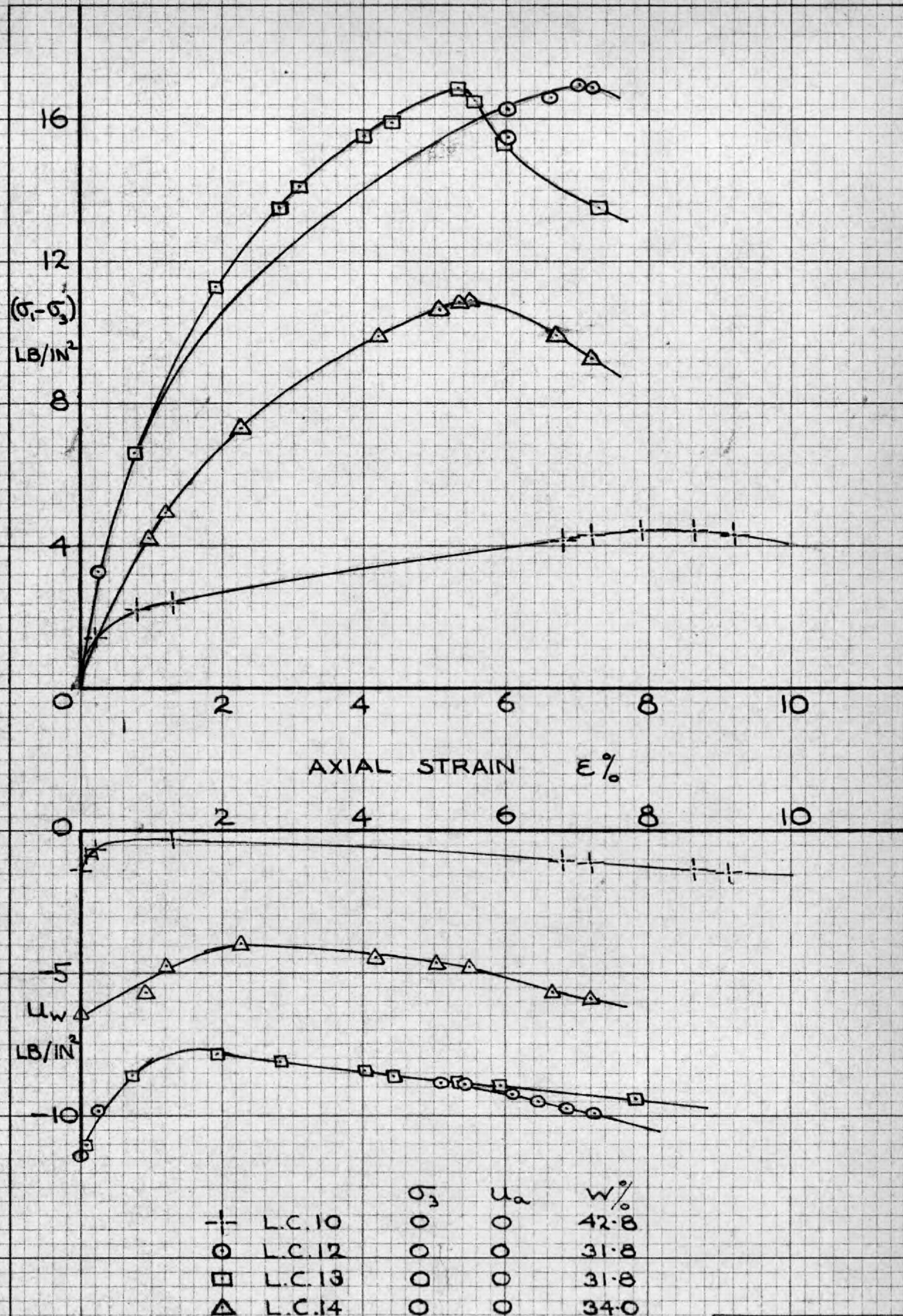
LONDON CLAY - REMOULDED UNDRAINED TESTS

FIG. A5-1



LONDON CLAY - REMOULDED UNDRAINED TESTS

FIG. A5-2



LONDON CLAY-REMOULDED UNDRAINED TESTS. FIG. A5-3

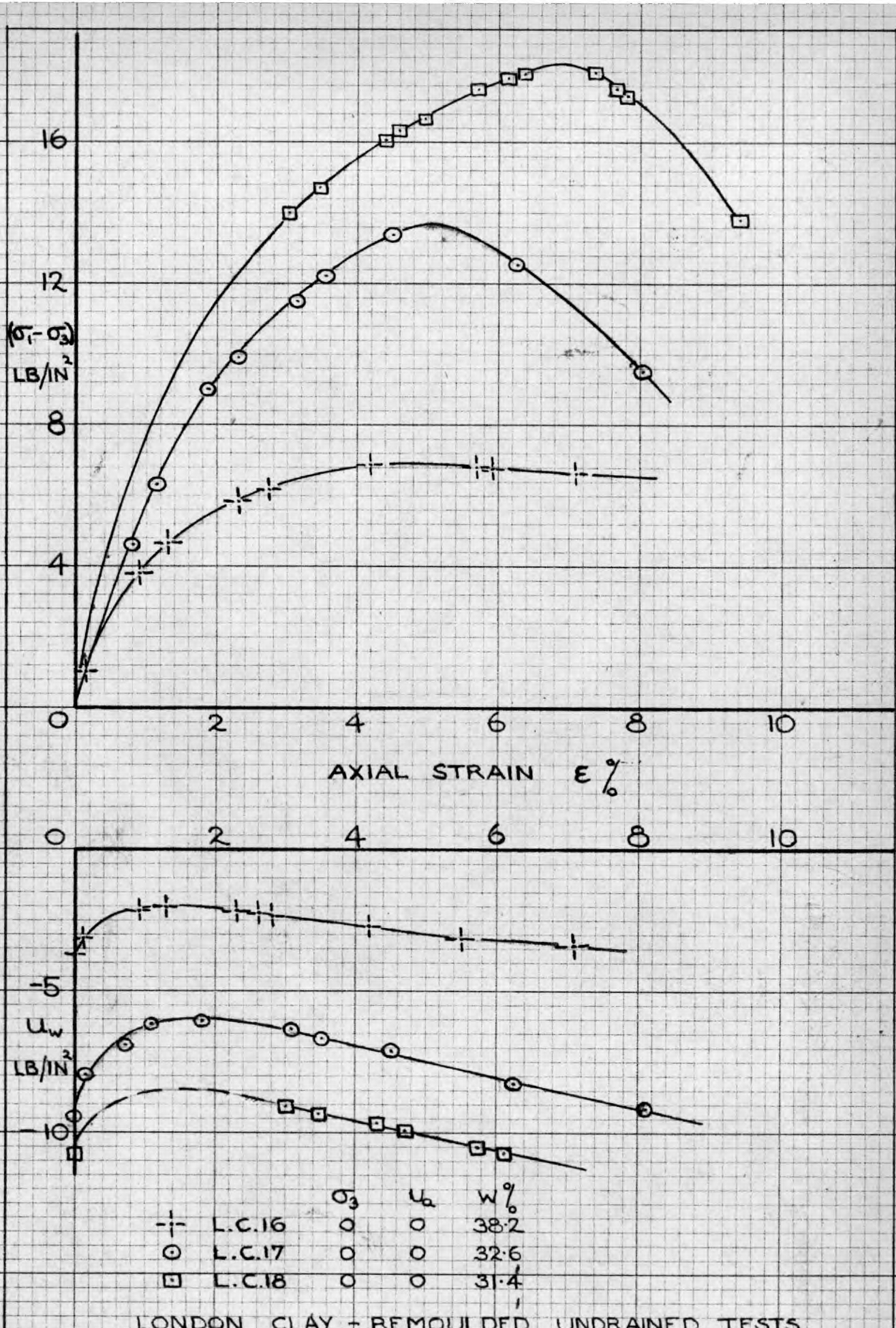
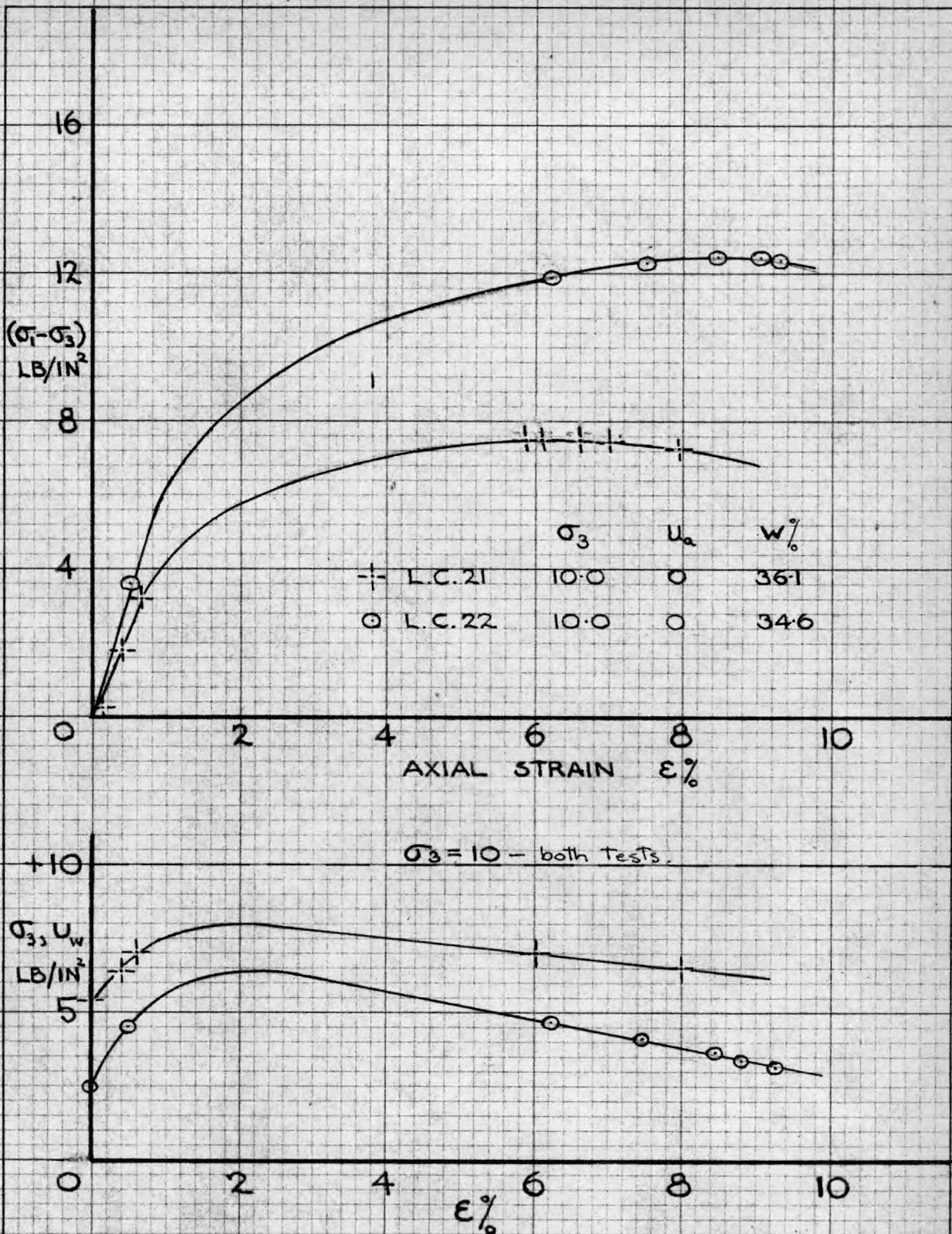
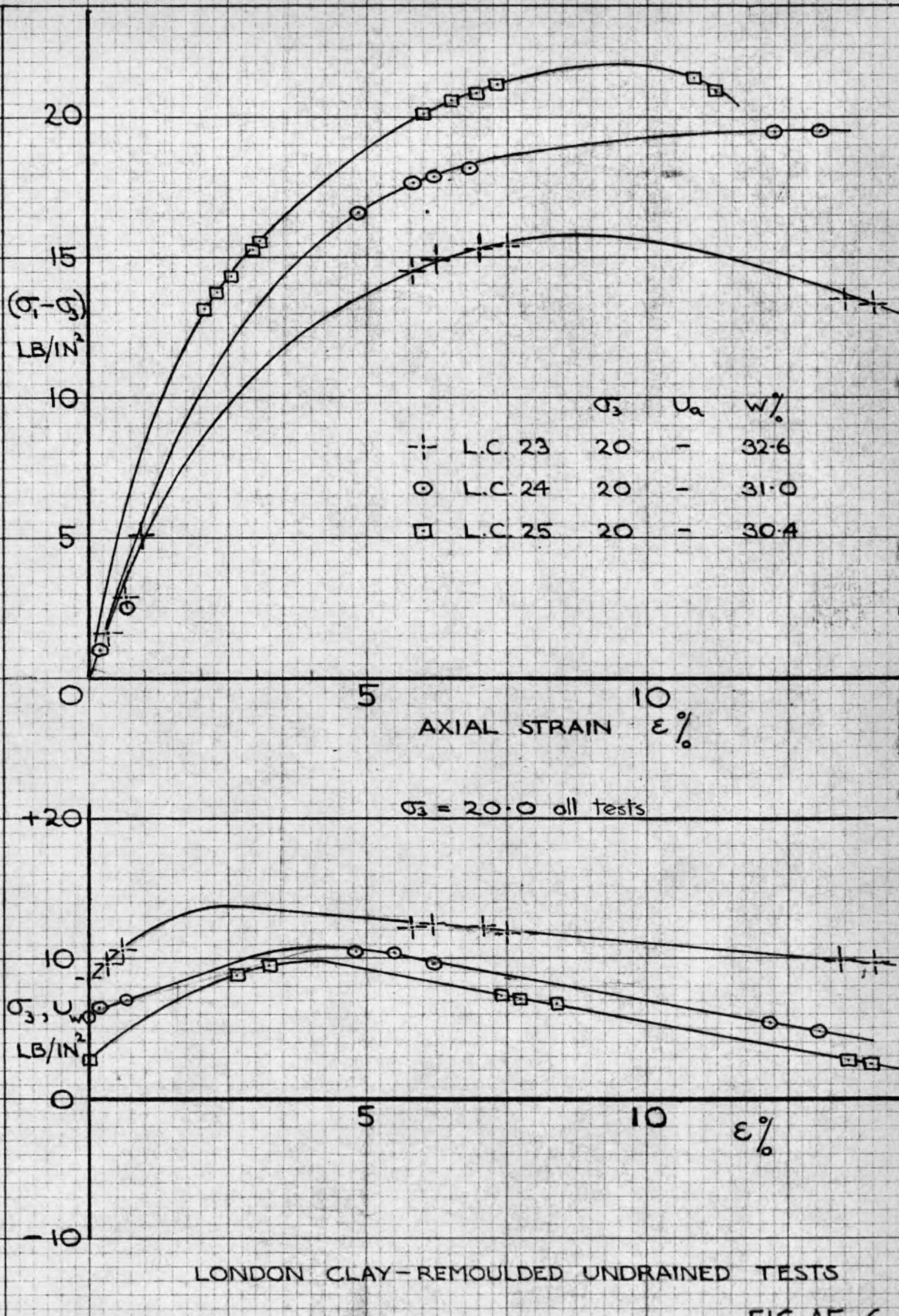

16
12
8
4
0
$(\sigma_1 - \sigma_3)$
LB/IN²
2
4
6
8
10
AXIAL STRAIN ε%
0
2
4
6
8
10
-5
-10
u_w
LB/IN²
σ_3 u_a W%
L.C.16 0 0 38·2
L.C.17 0 0 32·6
L.C.18 0 0 31·4
LONDON CLAY - REMOULDED UNDRAINED TESTS

FIG. A5-4



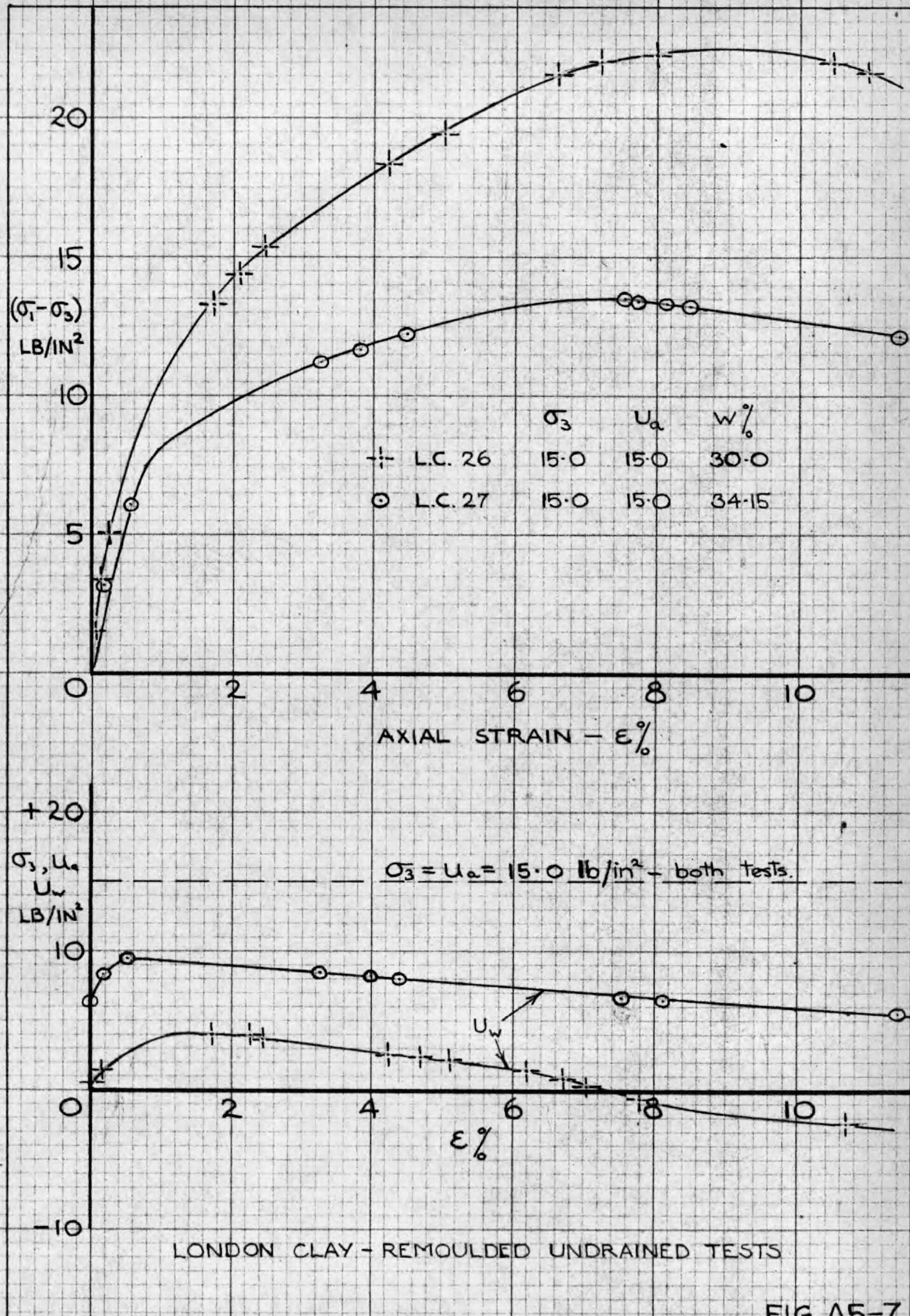
LONDON CLAY – REMOULDED UNDRAINED TESTS

FIG. A5-5



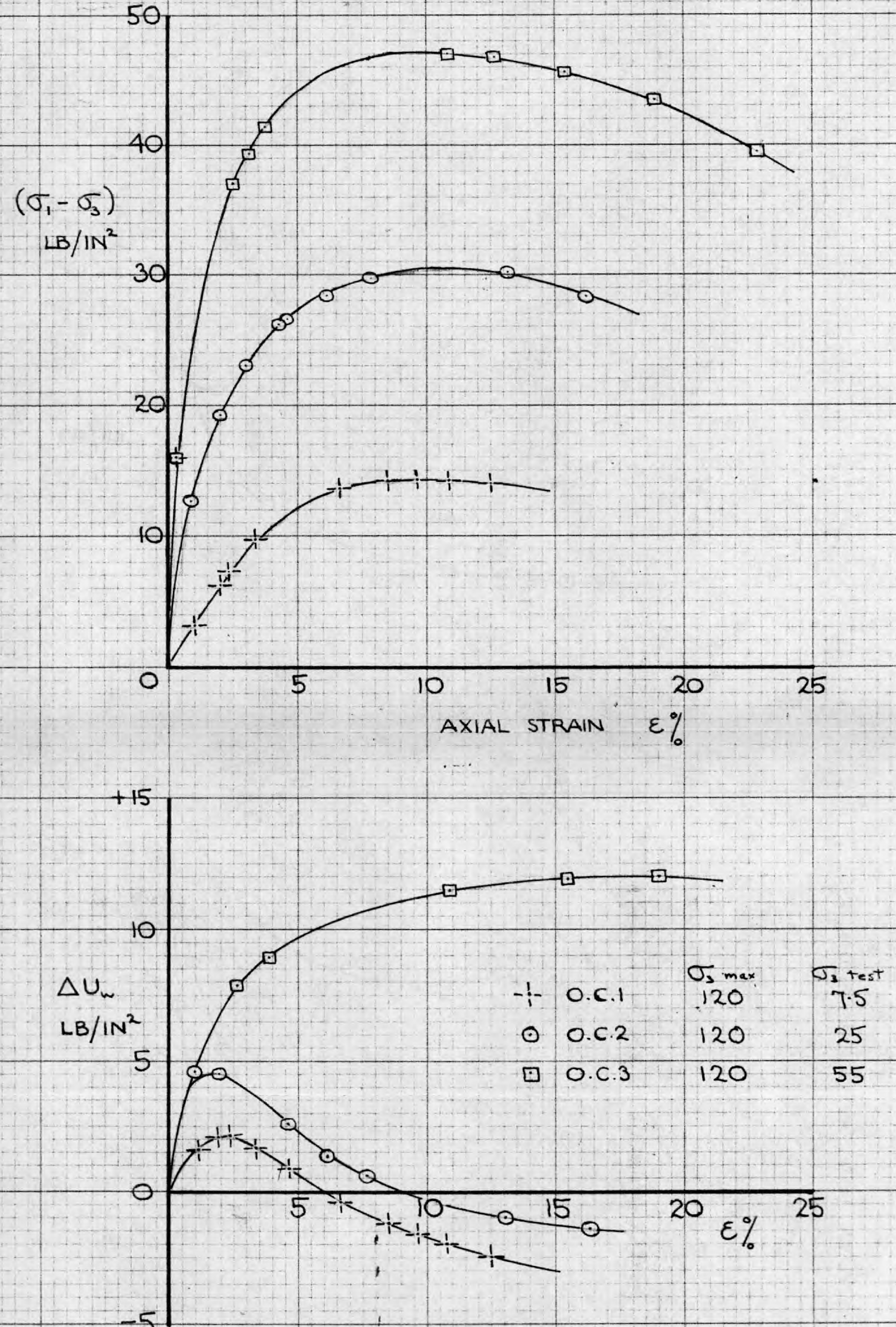
LONDON CLAY - REMOULDED UNDRAINED TESTS

FIG. A5-6



20
15
$(\sigma_1-\sigma_3)$
LB/IN²
10
5
0
2
4
6
8
10
σ_3 | U_a | W%
L.C. 26 | 15·0 | 15·0 | 30·0
L.C. 27 | 15·0 | 15·0 | 34·15
AXIAL STRAIN – ε%
+20
σ_3, U_a
U_w
LB/IN²
$\sigma_3 = U_a = 15 \cdot 0$ lb/in² – both tests.
10
U_w
0
−10
ε%
LONDON CLAY – REMOULDED UNDRAINED TESTS

FIG. A5-7



LONDON CLAY – OVERCONSOLIDATED, UNDRAINED TESTS

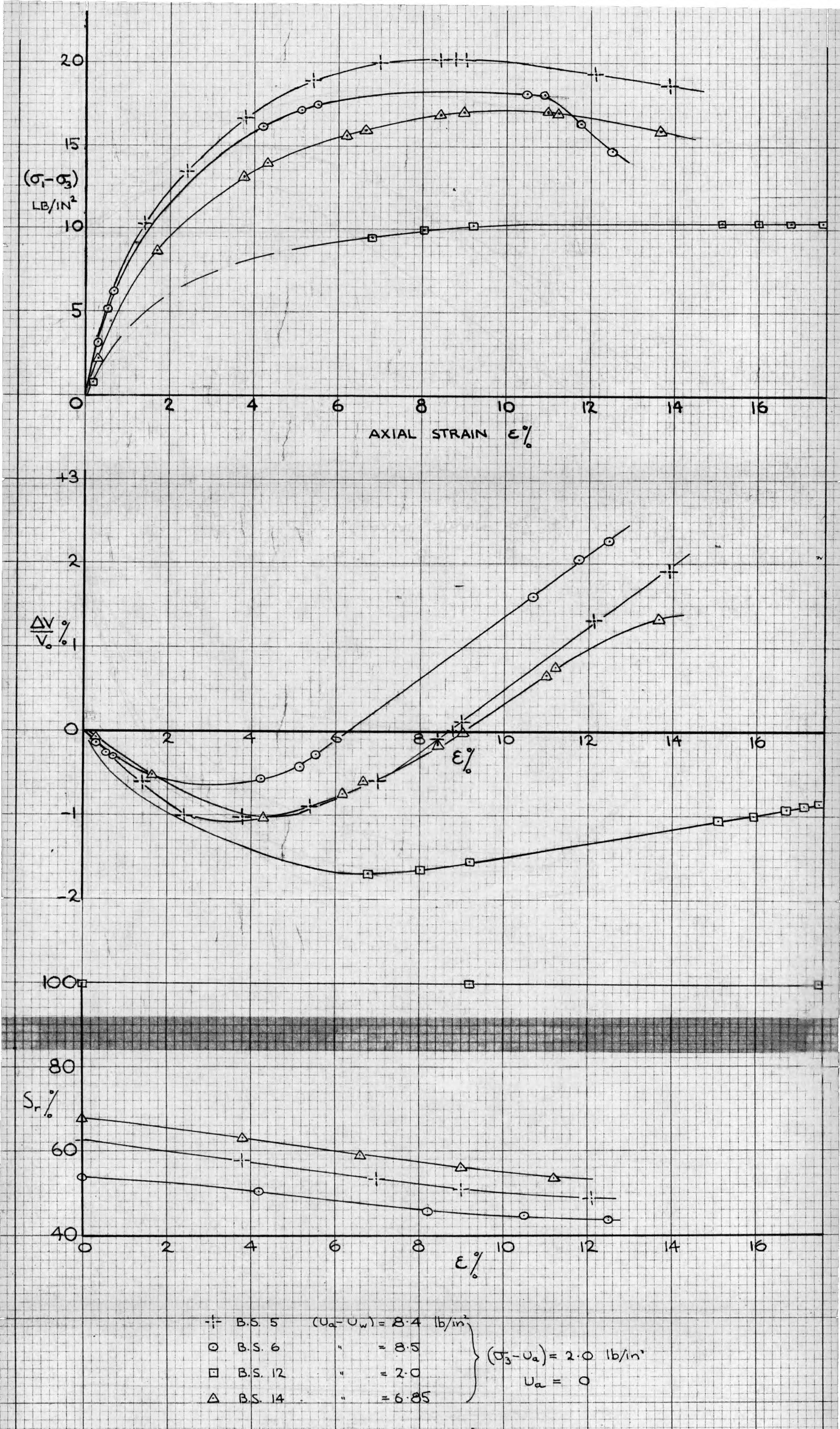
FIG. A5-8

A P P E N D I X 6

TEST RESULTS

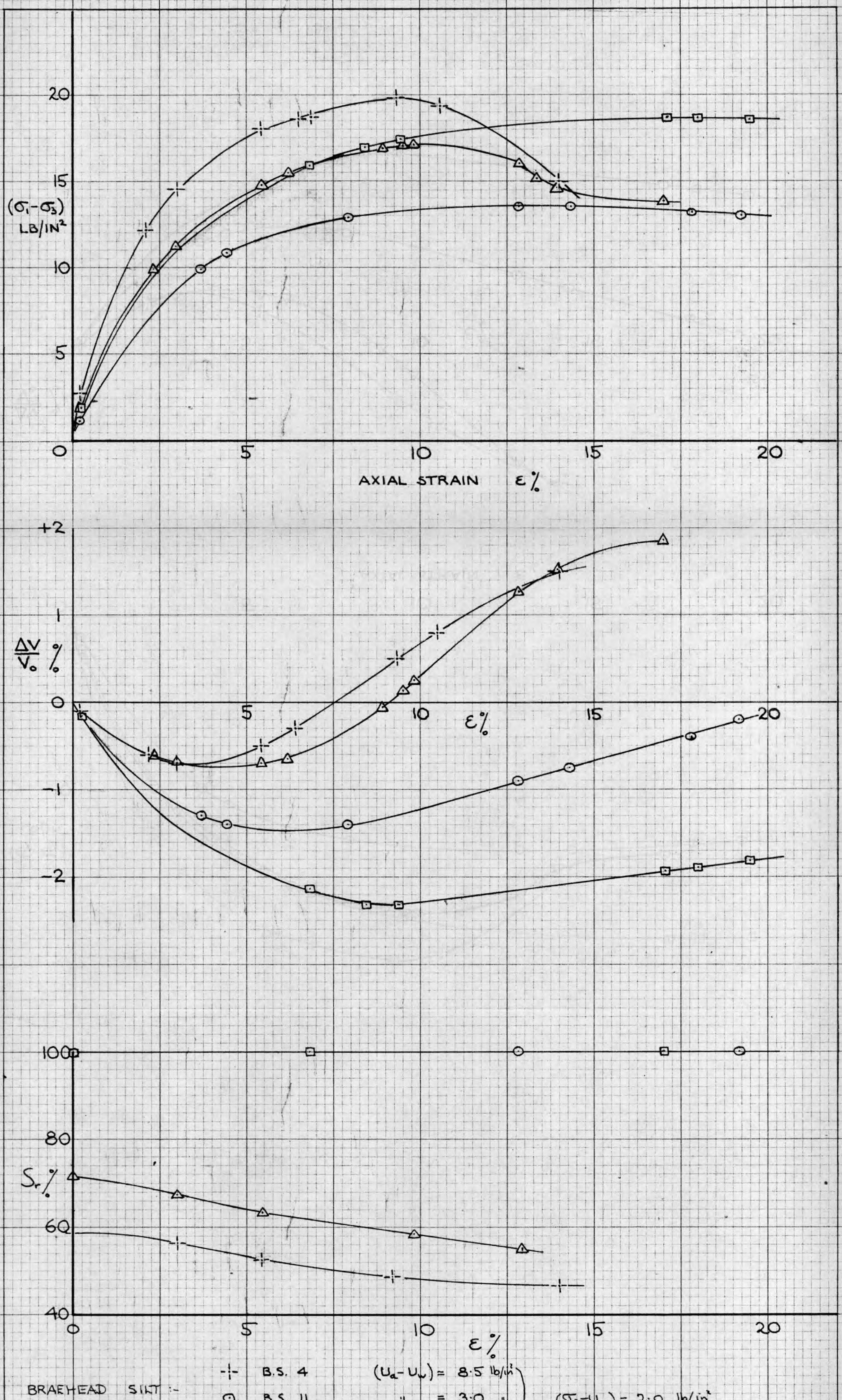
BRAEHEAD SILT

(Corrected for Membrane Effect)



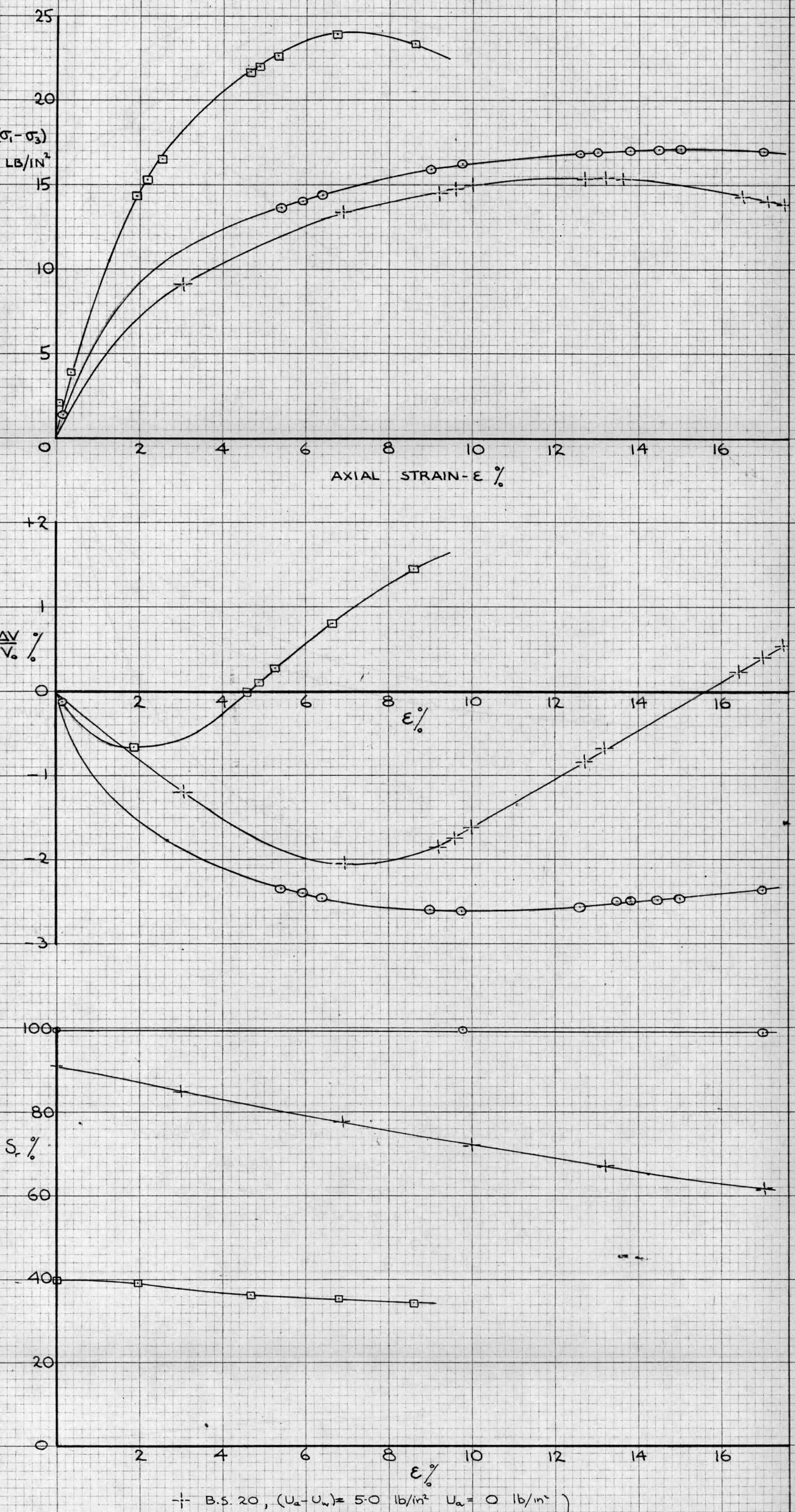
BREAHEAD SILT – DRAINED TESTS AT CONSTANT $(U_a - U_w)$

FIG. A6-1



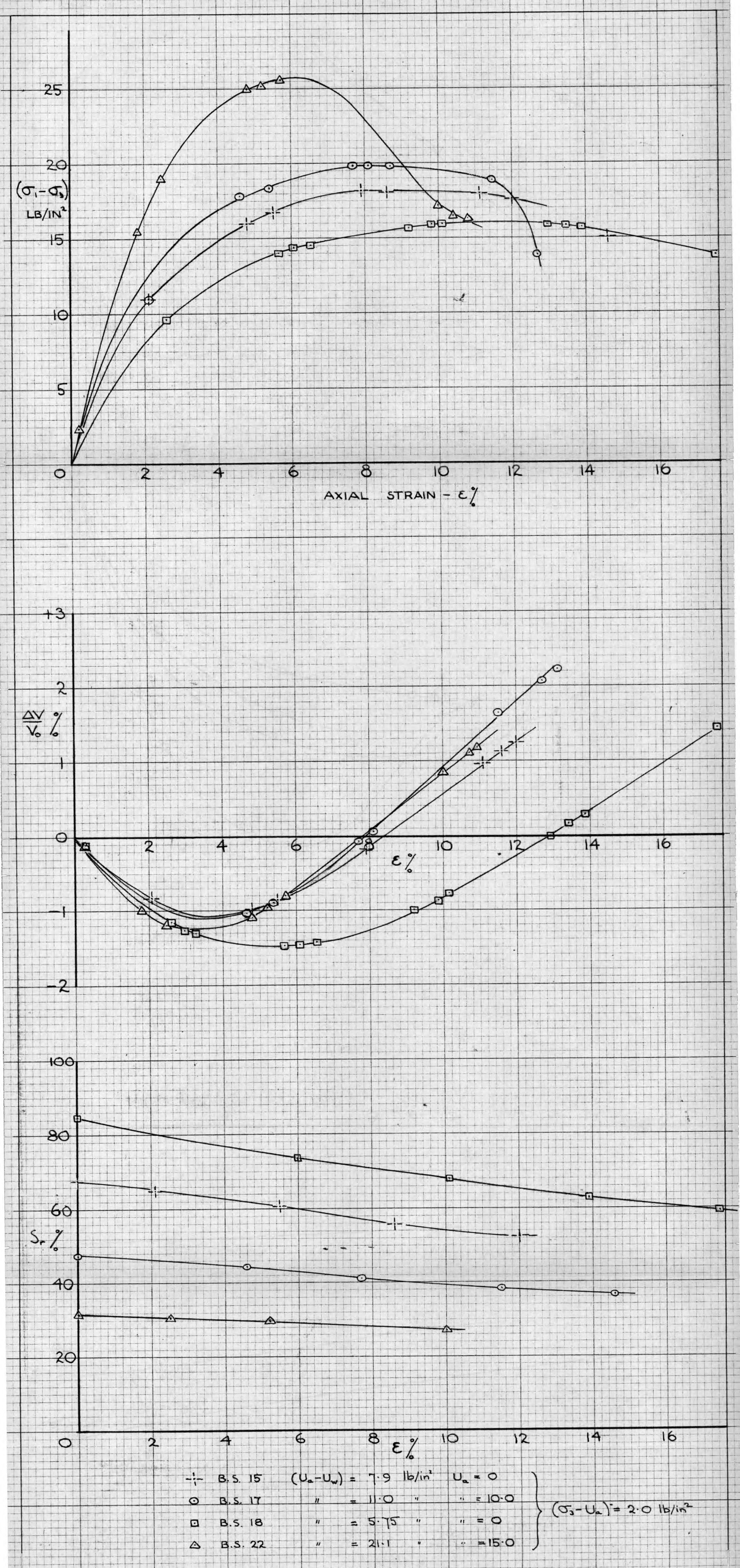
20
15
10
5
0
(σ1-σ3) LB/IN²
0 5 10 15 20
AXIAL STRAIN ε%
+2
1
0
-1
-2
ΔV/Vo %
5 10 15 20
ε%
100
80
60
40
Sr %
0 5 10 15 20
ε%
BRAEHEAD SILT :-
DRAINED TESTS (CONSTANT Ua-Uw)
B.S. 4 (Ua-Uw) = 8·5 lb/in²
B.S. 11 " = 3·0 "
B.S. 13 " = 5·95 "
B.S. 16 " = 6·45 "
(σ3-Ua) = 2·0 lb/in²
Ua = 0

FIG. A6-2

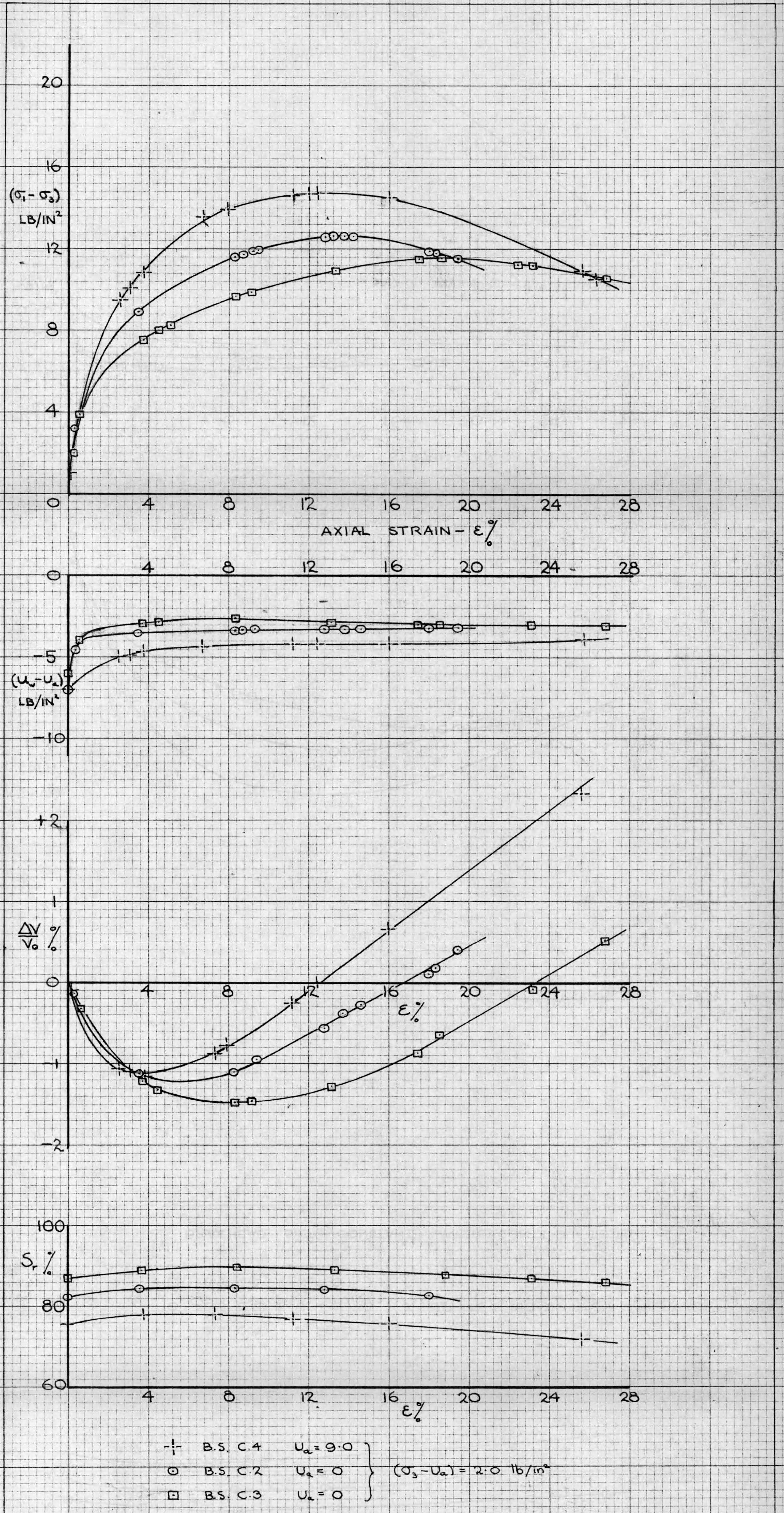


BRAEHEAD SILT – DRAINED TESTS, CONSTANT $(U_a - U_w)$

FIG. A6-3

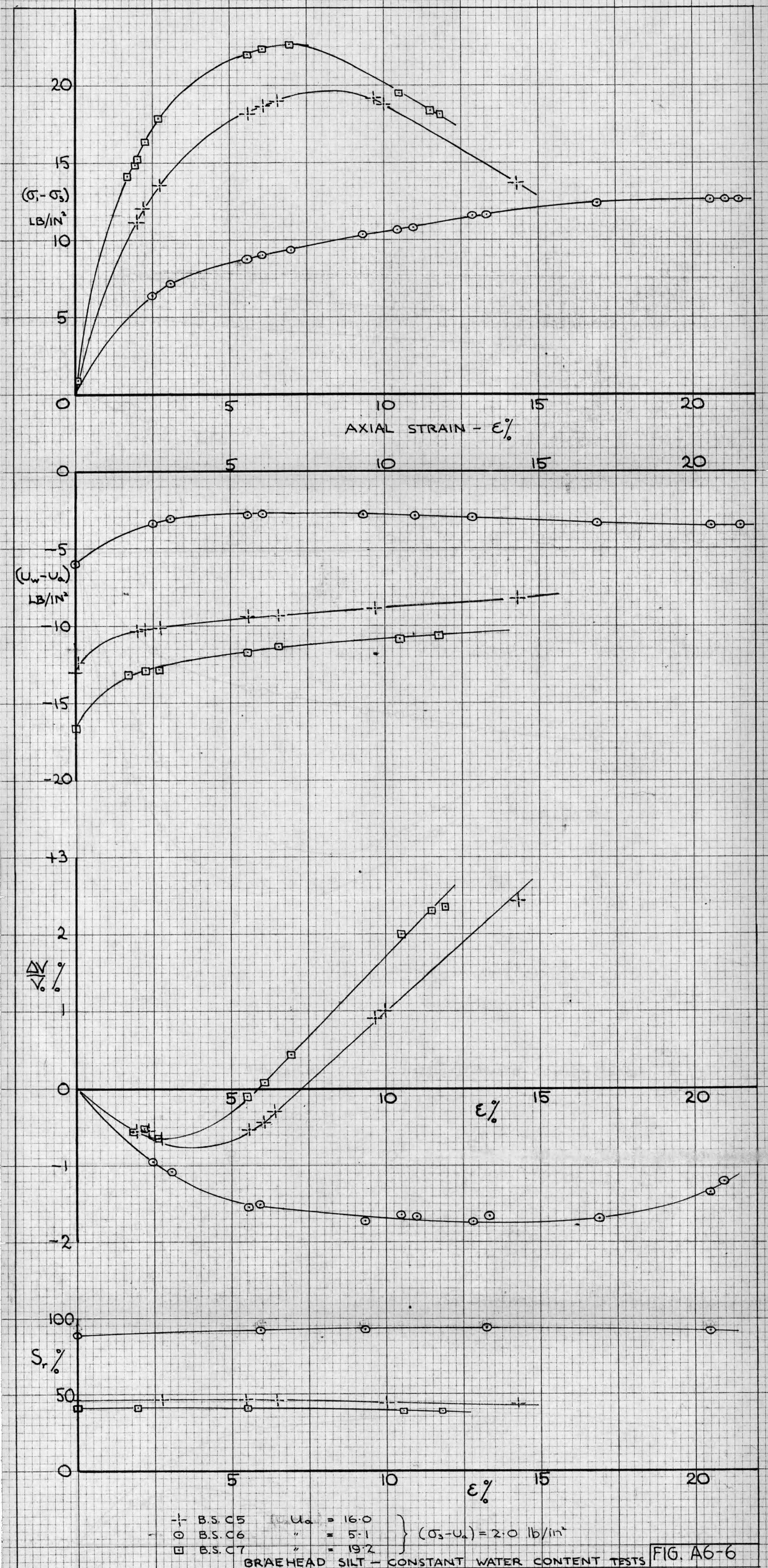


BRAEHEAD SILT – DRAINED TESTS, CONSTANT $(U_a - U_w)$ FIG A6–4

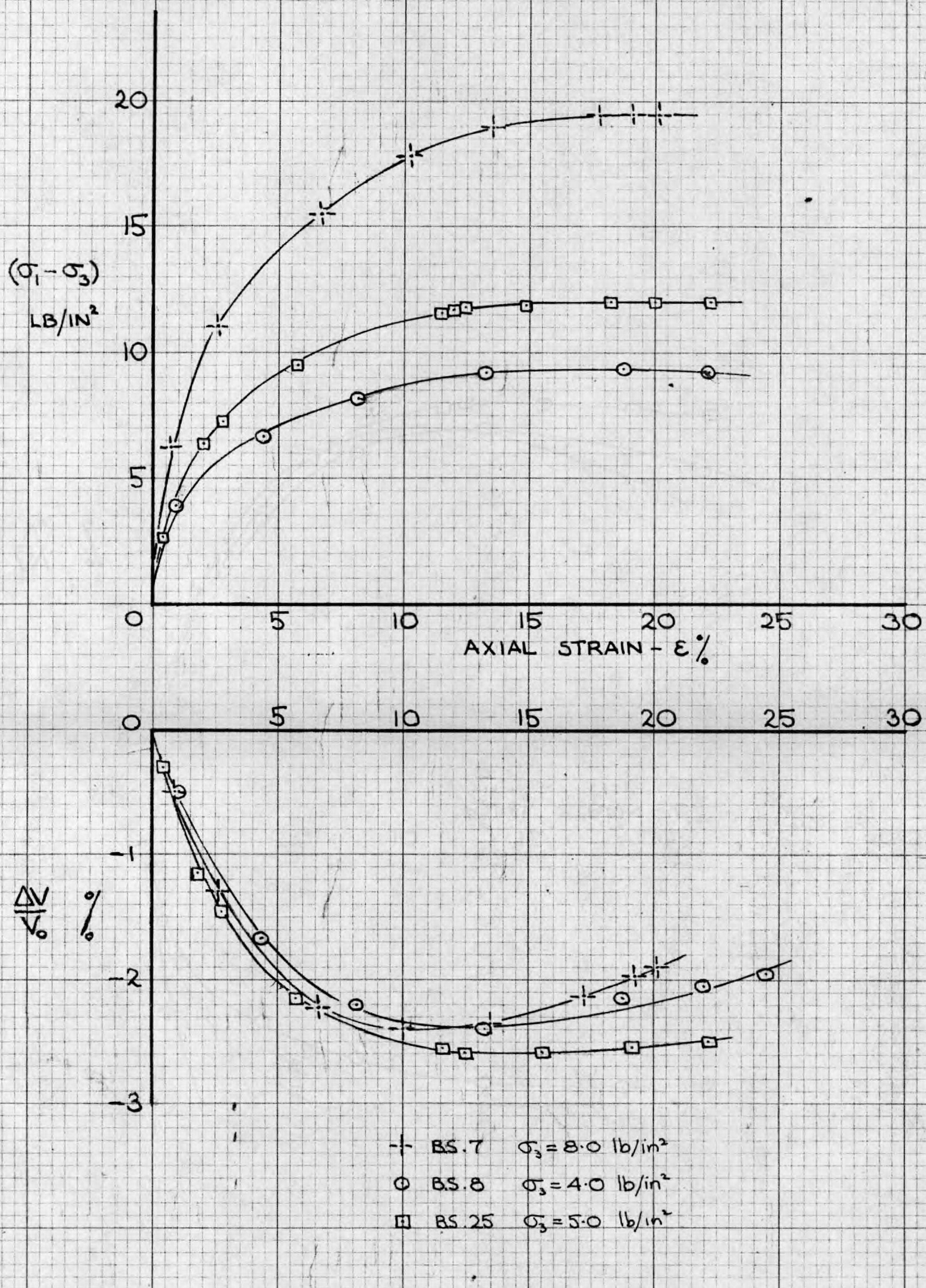


BRAEHEAD SILT - CONSTANT WATER CONTENT TESTS

FIG. A6-5

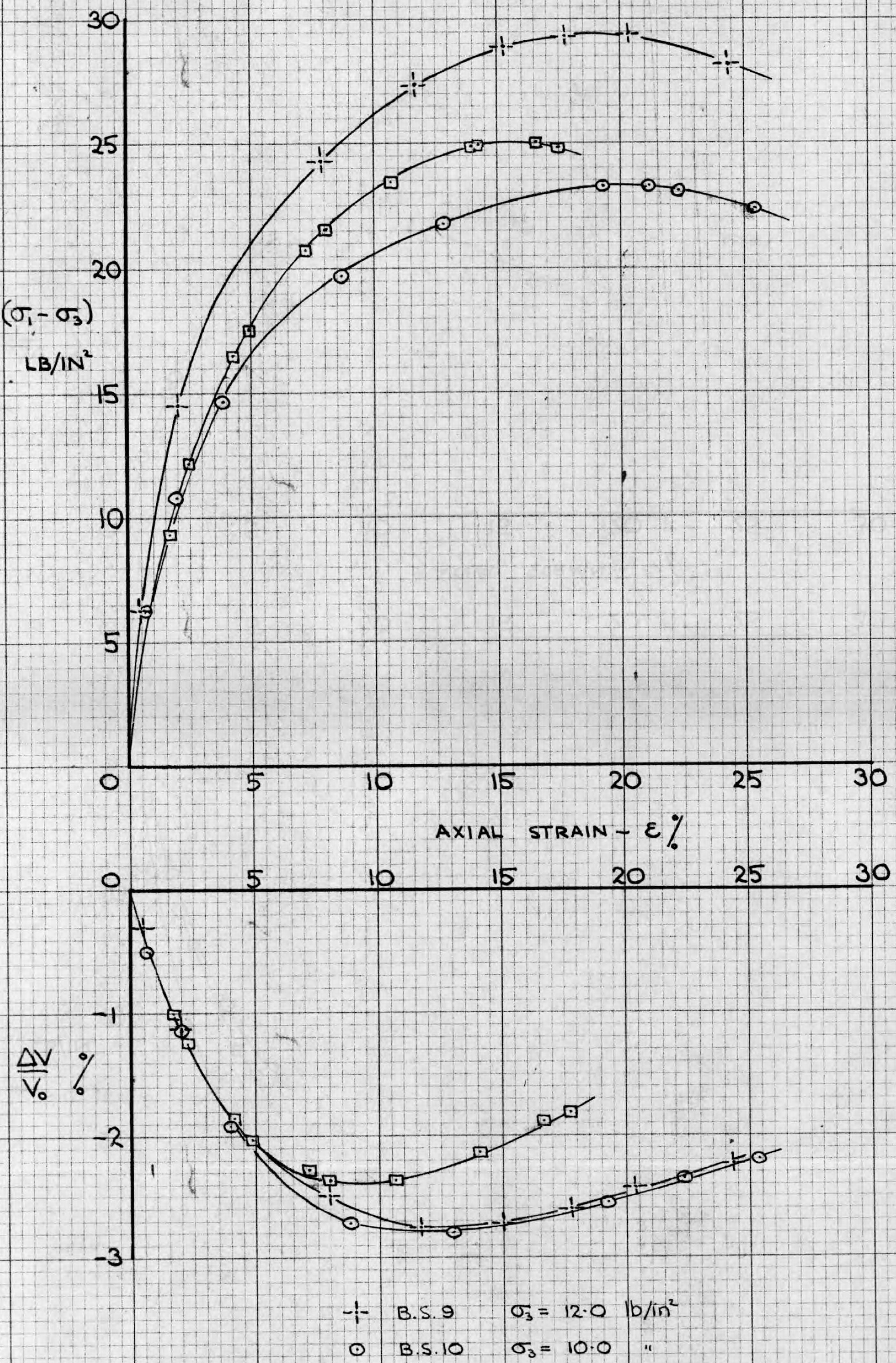


BRAEHEAD SILT — CONSTANT WATER CONTENT TESTS FIG. A6-6



BRAEHEAD SILT DRAINED TESTS
$(u_a = u_w = 0)$

FIG. A6-7



BRAEHEAD SILT – DRAINED TESTS

$(U_a = U_w = 0)$

FIG. A6-8