TRIAXIAL TESTS ON CLAYS AND SILTS AT ELEVATED CELL PRESSURES

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ABSTRACT

The thesis is divided into three sections.

In the first section, some of the main equations proposed in the literature to define effective stresses in saturated soils have been examined in the light of results obtained from multi-stage drained compression tests on a clay and calcite. It has been shown that varying both the confining pressure, σ_3, and the pore pressure, u, simultaneously by the same amounts, and within wide limits, had no detectable infuence on the shear strength and volume change characteristics of the soil, provided corrections have been applied for the compressibility C_s of the soil grains. It has been concluded, on the basis of the experimental results, that effective stresses in saturated soils are given by the following equations:

in respect of shear strength,

$$\sigma' = \sigma - u$$

and in respect of changes in volume,

$$\Delta\sigma' = \Delta\sigma - (1 - \frac{C_s}{C}) \Delta u$$

using the familiar soil mechanics notations.

In the next section of the thesis, the results of drained compression tests on clays and silts in the elevated confining pressure range are presented. The shear strength and volume change behaviour of these soils under elevated pressures have been found to follow the patterns previously noted for coarse grained cohesionless soils, and various possible explanations were presented to account for the observed changes in behaviour of the soils. In particular, comparisons between the mechanical behaviour

of calcite and Braehead Silt led to the suggestion that the nature of the clay-sized particles in a fine grained cohesionless soil may have an important influence on its behaviour under elevated pressures.

Results of consolidated-undrained triaxial compression tests performed on remoulded clays are presented in the final section of the thesis. These tests were performed by first consolidating a series of samples isotropically under the same effective stress, then shearing them undrained after the cell pressure had been suddenly decreased to a predetermined percentage of its value during consolidation, failure being attained in about five minutes. It was found that when the initial pore pressure in the sample just before shear was positive, the $\phi = 0$ condition was approximately satisfied, but that when the initial pore pressure was negative, the undrained shear strength was not independent of the cell pressure, but decreased with decreasing cell pressure. The decrease in undrained shear strength between the samples sheared normally consolidated and unconfined was as much as 35 %, and appeared to depend on the quantity of air dissolved in the pore water.

An approximate method for calculating the ambient effective stress at failure and the initial suction P_o'' in unconfined samples, has been presented and the possible practical implications of the results have been discussed.

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PART I

AN EXPERIMENTAL DEMONSTRATION OF THE VALIDITY OF THE TERZAGHI EFFECTIVE STRESS EQUATION.

CHAPTER 1.

A GENERAL REVIEW OF THE LITERATURE ON EFFECTIVE STRESS

1.1 Introduction

The discovery of the principle of effective stress by Terzaghi in the 1920's is generally regarded as the starting point in the systematic development of the science of Soil Mechanics as one of the major branches of Civil Engineering. Although the concept of effective stress was first quantified by Terzaghi, there is evidence in the literature* to suggest that earlier engineers and geologists appreciated some of its underlying principles.

Geologist Charles Lyell wrote in 1871, that the consolidation of ocean botton sediments was due, not to the pressure of the head of water above them, since this was equal to the pore pressure, but to the weight of the material subsequently deposited above them. The gain in strength resulting from the consolidation of soft clay strata was noted by Telford, who reported (Telford, 1830) the preloading, in 1809, of a deep bed of soft clay for a period of about nine months prior to the construction of a part of the Caledonian Canal. The settlement of buildings founded on clay had been reported by Sooy Smith (1892) and also by Shankland (1891) who also published a time-settlement curve covering a period of four and a half years. Both engineers attributed the settlements to the consolidation of the underlying clay materials.

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* For a more detailed discussion of the historical development of the principle of effective stress, see Skempton (1960 (a)).

Most of the shear tests performed around this period for the evaluation of the parameters c, and ϕ (for example, Leygue (1885), Frontard (1914) and Bell (1915)) were essentially undrained tests, even though there is some evidence (Boussinesq, 1876; Reynolds, 1886) that the influence of pore pressure on the shear strength of sands had been realised. It will appear from this that the interrelationship between pore pressure and volume change was not understood at the time.

As a result of his experiments on the consolidation* and shear strength of clays between 1919 and 1925, Terzaghi stated formally that the stress controlling the shear strength and deformation of saturated soils was given by the difference between the total applied stress and the pore pressure. This statement and its subsequent verification by Rendulic (1937) opened up a new era for the science of soil mechanics, which then progressed rapidly after that.

Development of new field and laboratory equipment, coupled with experimental and theoretical work culminated in the evolution of rational design methods, based on the principle of effective stress, and capable of being checked in practice, although most of those design criteria are yet to find a universal application, Bishop (1955), Hilf (1959). While the existence of such a principle is not in dispute, the form of the effective stress relationship suggested by Terzaghi had been criticized (Leonards, 1962) for its inadequacy in accounting for such rheological phenomena as creep and secondary consolidation in clays. The form of the function defining effective stressed in various soils has also been subject to considerable controversy.

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* Some of these experiments formed the basis of his classical theory of consolidation.

1.2 A review of the various equations proposed to define effective stresses in saturated soils.

By definition, the effective stress σ' is that function of the total normal stress σ, and pore pressure u, which controls the changes in volume and shear strength of a soil, consequent upon a change in applied stresses. Terzaghi (1923 and 1932) suggested that for fully saturated soils, this function was of the form

$$\bar{\sigma} = f(\sigma, u) = \sigma - u \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1.1)$$

The validity of this equation had been demonstrated by Rendulic (1937) in respect of shear strength, by showing that there was a close correlation between pore pressures derived theoretically, assuming the validity of the principle and those measured during consolidated - undrained tests on clays and also by Taylor (1944) and Bishop and Eldin (1950). The volume change aspects of the principle of effective stress had been verified by Laughton (1955). On theoretical grounds, however, equation (1.1) can only be rigorously valid (Bishop (1955), Skempton (1960)) if

(i) the soil grains are incompressible

and (ii) the soil particles have the property of a purely cohesive solid, i.e. if the angle of intrinsic friction ψ is zero.

Since real soils satisfy neither of these conditions, various other equations have been proposed in the literature to define effective stresses in soils.

1.2.1 The intergranular stress concept.

If, as is commonly believed (Taylor (1944), Gibbs (1965)) the effective stress controlling the mechanical behaviour of soils is the same as the intergranular

stress *, then by considering the forces acting across an approximately plane surface, always passing through points of contact of soil particles, it can be readily shown (Bishop (1955), Skempton (1960)) that the effective stress will, in this case, be given by

$$\sigma' = \sigma_g = \sigma - u + au \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1.2)$$

where σ_g denotes the average intergranular stress defined as the total normal force divided by the total contact area of the soil grains.

σ denotes total normal stress

u denotes the pore pressure

and a denotes the effective contact area ** of the soil particles per unit area of the plane.

A discussion of the implications of equation 1.2 and its applicability in various soils will be given later.

Even if this definition of effective stress were accepted, it can still be shown, by considering the probable deformations at the contact between two soil grains under fluid pressure, that the stress σ_c controlling the deformation of the soil structure is given by equation (1.1), namely that

$$\sigma_c = \sigma - u \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1.3)$$

1.2.2 Skempton's effective stress equations.

Allowing for the fact that real soils have finite compressibility and angle of intrinsic friction, Skempton

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* Lambe et al (1959) also suggest that failure in granular soils under high confining pressures, when particle fracture occurs, may be controlled by the intergranular stress.

** This term is discussed later.

(1960) derived the following equations to define effective stresses:

$$\sigma' = \sigma - \left(1 - \frac{a \tan \Psi}{\tan \phi'}\right) u \quad \ldots\ldots\ldots\ldots\ldots \quad (1.4)$$

in respect of shear strength, and

$$\sigma' = \sigma - \left(1 - \frac{C_s}{C}\right) u \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1.5)$$

for changes in volume,* where

Ψ denotes the angle of intrinsic friction of the solid particles in the soil.

ϕ' denotes the effective angle of shearing resistance,

C_s denotes the compressibility of the solid particles in the soil,

C denotes the compressibility of the soil skeleton,

u denotes the pore pressure, and

a denotes the contact area ratio.

The relative magnitudes of the various terms, as well as the applicability of the equations to natural soils, concrete and rocks, have been discussed by Skempton (1960) and Bishop et al (1963)

1.2.3 The contact area ratio, a.

If equations (1.2) and 1.4) were true, a knowledge of the magnitude of the contact area ratio 'a' would be necessary for the evaluation of the effective stresses. To date, no direct measurement of the area ratio had been attempted for natural soils, but suggestions have been made in the literature, of its probable magnitude for various soils.

Terzaghi (1936) quoted a value of zero** for sands,

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* This equation was first derived by Bishop in 1953 in a letter to Dr. A.S. Laughton following discussions at Imperial College.

** This value actually represented the contact area ratio between the sand particles and the base of the container rather than between sand grains.

on the basis of "floating tests" and values between zero and 5°/o for clays, from bouyancy tests. Taylor (1944) gave a value of 3°/o as an indication of the magnitude of 'a' for clays Bishop and Eldin (1950) suggested that for coarse-grained soils, a lower limit was imposed on 'a' by the crushing strength of the material making up the grains, and proceeded to show that even in the case of very heavily over-consolidated undisturbed clays, 'a' cannot be much more than 3°/o. Making the assumption that under sufficiently high confining pressures, the slope of the failure envelope for porous materials tended towards the angle of intrinsic friction for the solid particles, Skempton (1960) derived an approximate expression for estimating the value of 'a' under any given confining pressure.

There is some uncertainty regarding the meaning of the contact area ratio in the case of clays. It may be taken as the contact between the rigid adsorbed water layers surrounding the clay particles (Laughton, 1955), or as the contact between the "domains" of oriented particles (Trollope, 1960). The structure of clays and the various types of contact possible in clays will be discussed in more detail later. Lambe and Whitman (1959) indicated that, for clays, 'a' was a function of composition and structure, and suggested that it could be as high as 100°/o for plastic clays remoulded at high water contents. The important influence of interparticle electrical forces, coupled with the possibility of the existence of chemically formed bonds provide further complicating factors in the estimation of the contact area ratio in clays.

1.3 A discussion of the equations defining effective stresses in fully saturated soils.

The apparent validity of the Terzaghi equation for effective stress (equation 1.1) as established by various researchers, has been attributed to the small magnitude of the contact area ratio 'a', in which case both equations 1.2 and 1.4 degenerate into the Terzaghi form. While this argument is superficially attractive, in spite of the discussion above showing that the ratio need not be necessarily small in clays, a closer examination of some of the assumptions made in the derivation of the alternative equations indicates that the errors involved in making these assumptions may be more serious than those due to neglecting 'a'.

It is implicitly assumed in the derivation of equation (1.2) that

(i) the interparticle force is derived solely from externally applied loads

and (ii) that the interparticle force is transmitted between particles only through the interparticle contacts.

While these assumptions may be approximately true for sands and silts, recent work on the microstructure of soils containing a substantial percentage of colloidal particles indicates that these assumptions are far from valid for these soils.

1.3.1 The microstructure of clays.

Rapid advances made in the last decade in understanding the microscopic structure of clays,* though of a

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* Reviewed by Scott (1963) and Meade (1964).

~~quantitative~~ qualitative nature, had led to an appreciation of the influence of electrostatic interparticle forces on the macroscopic behaviour of these soils.

Clay minerals * are platy electrically charged particles having a net negative charge resulting from substitutions in the mineral structure. In the presence of water, this net negative charge causes the particles to be coated with rigid and ordered layers of "adsorbed water" whose properties (e.g. density and viscosity) are different from those of ordinary water, and which is more resistant to shear and normal stresses than ordinary water.

Any two adjacent clay particles are subject to a complex set of forces, the two main ones of which are the van der Waal's forces of attraction and Coulomb's forces of repulsion. The former varies inversely as a power (believed to be the fifth or sixth) of the distance of separation of the particles, while the latter varies inversely as the concentration of electrolytes in the pore fluid. The balance between the forces of attraction and repulsion determines the orientation of the particles in the clay mass, which in turn has a marked influence on the engineering properties of the clay.

The two basic types of particle orientation generally postulated are:

(a) a flocculated structure consisting of a random orientation of particles caused by the attraction of the negatively charged particle surfaces to positively charged ends of other particles (figs. 1(a) and 1(b)). This

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* For a more detailed discussion of the various types of clay minerals, see Grim (1953).

arrangement of particles is typical of material sedimented in a solution of high salt concentration for example, in marine clays (Rosenqvist, 1959).

(b) a dispersed structure in which there is a parallel association of particles within small zones, and is characteristic of clays deposited in fresh water environments (low electrolyte concentration) or prepared in the laboratory using distilled water (fig. 1(c)).

The fabric of clays and the influence of structure on the engineering properties of clays has been discussed in some detail by Mitchell (1956).

There is some experimental evidence (Moum and Rosenqvist, 1961; Salas and Serratosa, 1953; Bolt, 1956; Ladd and Martin, 1967), that both the shear strength and deformation characteristics of clays depend on the chemistry of the pore fluid, which as discussed earlier, in turn influences the interparticle forces. Furthermore, theoretical and experimental work by Marsal (1963) showed that the net interparticle forces derived from the interaction of the van der Waal and Coulomb forces is of the same order of magnitude as the forces derived from externally applied loads. It follows, therefore, that any equation defining effective stresses in clays has to take account of the existence of these interparticle forces.

Lambe (1960) discussed the various types of contact possible in clays, and suggested an extension to equation (1.2) to cater for the net interparticle force, thus

$$\sigma' = \sigma_g = \sigma - u\,(1-a) + R - A \quad \ldots\ldots\ldots\ldots\ldots \quad (1.6)$$

where (R-A) is the result of dividing the net interparticle forces by the areas over which they act. The term (R-A) increases with decreasing particle spacing.

Trollope (1960) proposed a model for clays, based on the diffuse double layer theory, and made up of coarse mineral particles in a matrix of "domains" of oriented particles instead of discrete particles (fig.1(d)). Some form of contact exists between the domains, though it is not clear whether this contact is between adsorbed water layers or clay minerals. However, applying the well-known Bowden-Tabor plastic junction theory (Bowden and Tabor, 1954) to the contact between the domains, Trollope (1960) derived an effective stress law of the form

$$\sigma = \sigma - u + \rho \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1.7)$$

where ρ denotes the normal stress on the failure surface owing to inter-domain forces. The parameter ρ, which is related to the activity A (Skempton, 1953) of the clay, may be computed from the drained strength envelope for saturated clays.

A similar model was discussed by Osterman (1962) who proposed a separation of effective stresses in clays into the following components, namely the intergranular stress in the crystalline solid (quartz or feldspar) phase, σ_g, which is given by equation (1.2); "molecular stresses" σ_m in the clay matrix and stresses due to cementations σ_c .

Thus

$$\sigma' = \sigma_g + \sigma_m + \sigma_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1.8)$$

This equation, however, implies that the coarse grained materials are in contact, which may not always be true. Other effective stress equations for clays have been discussed by Lambe et al (1959) and Whitman (1960), and Suklje (1967).

1.3.2 Interparticle friction.

Skempton (1960), in deriving equation (1.4) assumed the validity of the relationship derived by Caquot (1934) and Bishop (1954) connecting the coefficient of friction at the interparticle contact ϕ_u and effective angle of shearing resistance at constant volume ϕ'_{cv}. This relationship, undrained with p.w.p. measurement
which is of the form,

$$\tan \phi'_{cv} = f(\tan \phi_\mu) \quad \ldots\ldots\ldots\ldots\ldots \quad (1.9)$$

assumes that the mechanism of particle movement during shear was one of sliding, irrespective of the value of the interparticle friction. Recent tests by Skinner (1969) on glass ballotini showed that at low values of interparticle friction, the mechanism of particle movement is indeed one of sliding but that the mechanism changed into one of rolling when the value of the interparticle friction * increased by a factor of 5 as a result of the sample being flooded. Furthermore, this increase in interparticle friction has been shown not to have any appreciable effect on the shear strength of the material. These findings showed clearly that the mechanism of particle movement rather than the effective angle of shearing resistance, changed with changes in interparticle friction, thus contradicting equation (1.9) and therefore the assumption underyling equation (1.4).

As the discussion of the microscopic structure of clays would indicate, the problem of interparticle friction in cohesive soils is far from understood. One has first to decide what sort of interparticle contact exists in these soils. If the view is accepted that the contact is between

* The interparticle friction was measured directly in a specially developed "friction apparatus".

the adsorbed water layers, surrounding either the discrete particles or a 'domain' of particles, then Trollope (1960) and Lo (1966) suggest that provided the yield stress and the shear strength of this bound water layer are considered, the Bowden-Tabor friction law was applicable.

Nothing quantitative can be said with certainty about the properties of the adsorbed water layer, except that its density and viscosity are different from those of ordinary water. Martin (1962) reviewed the available experimental evidence supporting the above view and proceeded to the same conclusion as Michaels (1959) that the adsorbed water layer has anisotropic properties, being rigid in the direction normal to the particle surface, but fluid in the direction parallel to it, thus implying that the adsorbed water layer was capable of withstanding normal stresses but not shear stresses. This point of view conflicts with that held by Grim (1958) and Low (1958) that the adsorbed water layer has a rigid ordered structure which can withstand both shear and normal stresses.

Under normal pressures sufficient to extrude most of the adsorbed water layer* some sort of physical contact may be possible between the mineral particles. Considerations of 'interparticle friction' in cohesive soils are further complicated by the presence of such inert particles as sand and silt, and is therefore hardly amenable to an analytical treatment.

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* Lambe (1958) suggested that pressures necessary to squeeze out the final layers of the bound water may be of the order of thousands of atmospheres.

1.3.3 Effective stresses controlling changes in volume.

Consolidation tests on globigerina ooze, (an ocean bottom sediment) and lead shot (grade 10) by Laughton (1955) showed that the ratio: $(\frac{\sigma - \sigma'}{u})$ remained approximately equal to unity under total applied pressures (σ) of up to 1024 kg/cm^2 (14,560 psi). In the case of the tests on lead shot, microscopic measurements of the facets produced by inelastic deformation showed that the ratio $(\frac{\sigma - \sigma'}{u})$ differed little from unity while the contact area ratio * increased from a value of 0.03 under a pressure of 27 kg/cm^2 to 0.95 under a pressure of 1024 kg/cm^2. While these results may not be considered as establishing conclusively that the effective stress controlling volume change and deformation is given by $(\sigma - u)$** they can, at least, be considered a confirmation of the fact that the contact area term had no influence on relevant effective stress.

1.4 Effective stresses in partly saturated soils.

In the case of soils which have both air and water in their pore spaces, Bishop (1955) proposed an equation defining the effective stress σ' in the form

$$\sigma' = \sigma - u_a + x\,(u_a - u_w) \quad \ldots\ldots\ldots\ldots \quad (1.10)$$

where σ denotes the total applied stress

u_a denotes the pore air pressure

u_w denotes the pore water pressure, which is less than u_a on account of surface tension

and x denotes a parameter which is zero for dry soils and equals unity for fully saturated soils.

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* Necessarily a lower bound value, since some elastic recovery must have occurred on unloading.

** The ratio $(\frac{\sigma - \sigma'}{u})$ varied between a minimum of 0.90 and a maximum of 1.2.

Intermediate values of x depend not only on the degree of saturation, but also on the stress change leading to the particular degree of saturation, the soil structure, and on the cycle of wetting and drying to which the soil has been subjected (Bishop et al, 1960). This equation has been verified for silts by Donald (1961). In the special case when the pore air pressure U_a is equal to the atmospheric pressure, equation (1.10) reduces to

$$\sigma' = \sigma - xu_w \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1.11)$$

where u_w is the gauge pressure.

Noting that in most engineering problems, the degree of saturation S_r of the soil is nearer unity than zero and that pore water pressures rather than pore air pressures are generally measured in the laboratory and in the field, Skempton (1960) rearranged equation (1.10) into the form

$$\sigma' = \sigma - S_x U_w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1.12)$$

where $S_x = 1 - (1 - x)\left(\frac{U_a - U_w}{U_w}\right)$

and $S_x \longrightarrow 1$ when $S_r \longrightarrow 1$

Other expressions defining effective stresses in unsaturated soils have been proposed and discussed by Croney et al (1958) Aitchison (1960), Jennings (1960), Bishop et al (1961) and Jennings et al (1962).

1.5 Summary.

A brief historical account has been provided to show that even though the first quantitative statement of the principle of effective stress had been provided by Terzaghi in the 1920's, the practical implications of the principle had been appreciated by earlier engineers and geologists. The various equations proposed in the literature to define effective stresses in both fully and partly saturated

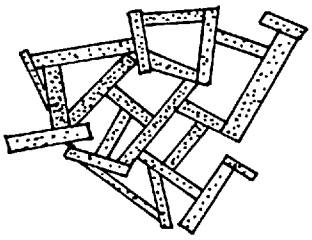
soils have been reviewed. In particular, the validity of the main assumptions made in deriving the various effective stress equations for fully saturated soils have been examined. It is suggested that the proposed refinements to Terzaghi's original equation are based on assumptions, which are at best only approximately true in the case of granular materials. A qualitative discussion of the microstructure of clays and the influence of electrical interparticle forces, has been presented to show that the assumptions may be grossly in error in the case of soils with an appreciable percentage of colloidal particles.

Under the circumstances, it is believed that the experimental method remains the most effective way of demonstrating the influence of the various factors on the shear strength and deformation characteristics of soils.

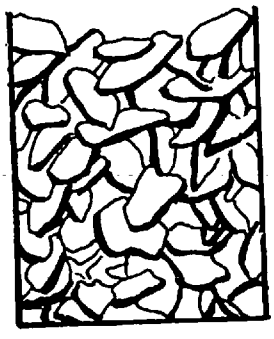
1.6 Scope of work presented.

Multi-stage drained triaxial compression tests on a remoulded silty clay and calcite were described and the results presented. In these tests, the difference between the total applied stresses σ, and the pore pressure u, was maintained constant at a predetermined value while both σ and u were varied in order to study their influence on the shear strength and deformation characteristics of these soils.

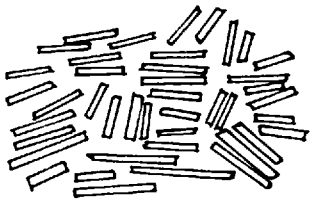
The special apparatus developed for these tests and the method of testing are described in Chapters 2 and 3 respectively. Chapter 4 contains a discussion of the factors likely to influence the accuracy of the results, the method of analysing them as well as a discussion of the results.



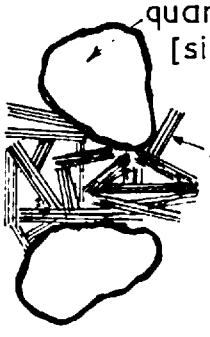
(a) Flocculated edge-to-face structure of clay visualized in two dimensions. (After Lambe, 1958)



(b) Flocculated edge-to-face structure of clay visualized in three dimensions. (After Tan, 1958.)



(c) Dispersed structure of clay visualized in two dimensions. (After Lambe, 1958)



(d) Clay model showing quartz grains in a matrix of 'domains' of oriented clay particles. (After Trollope, 1960)

SCHEMATIC DIAGRAMS OF CLAY STRUCTURE.

FIG. 1.1

CHAPTER 2.

DESCRIPTION OF APPARATUS AND CALIBRATIONS.

2.1 Introduction:

The nature and scope of the testing programme and the degree of accuracy desired necessitated the development of equipment which is either entirely new, or is a modification of a conventional laboratory equipment to enable it to be used in the elevated pressure range (up to 1000 psi) covered by the testing programme. The word 'conventional' as applied to equipment and experimental techniques in this thesis refers to the type of equipment and testing technique generally employed in Soil Mechanics research laboratories, and described by Bishop and Henkel (1962).

The following is a description of the specialized equipment used in the testing programme, as well as a description of some calibration tests performed.

2.2 The balanced ram triaxial apparatus.

This apparatus, which was described briefly by Bishop et al (1965) and in more detail by Skinner (1969), consisted essentially of a specially machined cell base, a stainless steel cell and two cell caps both fitted with rotating bushes (Bishop et al, 1965).

The stainless steel cell was 7 1/2" long, and

had a wall thickness of 3/8", which was reduced to 1/4" from inside at the ends to enable the cell to fit O-rings inserted into 1/4" grooves in the cell base and top cap. One of the cell caps was held over the top of the cell in the usual way, while the other cap was inverted to fit over a specially recessed portion below the cell base (Fig.2.1). Both caps were fitted with 5/8" diameter matched honed stainless steel rams.

The main sample chamber and the space below the cell base were connected by a nylon tubing, which, was itself connected to the constant cell pressure source via a castor oil - water interchange pot (fig.2.3). This ensured that the pressures in the two chambers, and hence the hydraulic thrusts on the two rams, were equal.

The rams were made to bear against the upper and lower beams of a yoke, which was held in position by means of a counter-balance lever system. This system comprised of two short beams, supported on knife edges in the middle, and connected by steel wires to the yoke at one end and to equal brass weights at the other.

The cell base rested on three equal cell supports, which fitted tightly into holes in an aluminium base plate, which, in turn, fitted on to the loading platform of a conventional 5 ton triaxial loading frame.

When the apparatus was not in use, the cell base and the lower cap were held together by three bolts equally spaced round the perimeter of the base. The cell base had connections for filling and emptying the cell, for top and bottom sample drainage leads, and for interconnecting cell pressure and back pressure.

The testing programme called for the variation of cell pressure and back pressure by equal amounts so as to keep the difference between them at a constant predetermined value. For these tests, the balanced ram triaxial apparatus has two main advantages over the conventional apparatus. Firstly, by counterbalancing the considerable upthrust to which the loading ram is subjected, it is possible to measure the strength of soil accurately by using a proving ring of low load capacity and high sensitivity. Secondly, changes in strain rate, which would otherwise result from the compression of the proving ring during the pressure change sequences, were also eliminated. When the apparatus is properly assembled, the net upthrust on the loading ram as a result of changing the cell pressure from zero to 1000 psi was of the order of 1/16 lb.

2.3 The apparatus for continuous variation of cell pressure.

This equipment, which is illustrated in figs. 2.2a and 2.2.b, consisted of a mechanically driven piston moving in a water-filled cylinder. The drive, provided by a single-phase electric motor, through a gear-box, was transmitted to a screw jack by means of a chain. Worm gearing within the screw jack converted the drive into a translatory motion, thus causing the main shaft to move forwards and backwards depending on the gear setting selected. The piston was screwed to one end of the shaft, and was therefore forced back and forth in the cylinder following the shaft movement.

A brass T-piece screwed to the other end of the main shaft operated limit switches situated at either end of travel of the shaft, thus cutting off the power to the

motor. The spacing and positioning of the microswitches were so selected that the piston neither bore against the head of the cylinder, nor came out of it at the limits of travel on the shaft.

A hydraulic over-pressure switch was connected to one end of a tee pipe fitting at the head of the cylinder. The switch, of the type PS/VHT2, manufactured by LONDEX Ltd. of London, had a maximum range of 1200 psi and an adjustable range between 300 psi and 1000 psi. By setting the switch to cut off the power when the pressure in the cylinder attained a value of 1000 psi, the pressure gauges in the line were protected against overload. The other end of the pipe fitting was connected to valve 6 in fig. 2.3.

As the cell pressure rose, increased leakage from the cell made it necessary to increase the rate of pumping at various stages in the test in order to maintain a constant rate of increase of cell pressure. This was done by altering the gear ratio internally by means of a lever on the gear box, and externally by changing wheels A and B (fig.2.2b).

2.4 Constant Cell Pressure Control:

The cell pressure was maintained constant by means of a self-compensating mercury control of the type described by Bishop and Henkel (1962). The Soil Mechanics Laboratories are situated on the 5th floor of the Civil Engineering building, and it was therefore possible to install the lower three stainless steel pots (columns 3, 4 and 5 in fig.2.3) on various floors below the laboratory. The tops of the three upper pots of the system were all connected to a manifold of Klinger valves which made it

possible to put any combination of columns 3, 4 and 5 in series with line A3 in order to obtain any desired pressure up to a maximum of 1,000 psi. The upper pots of the system were periodically replenished with mercury by means of the priming pump provided.

2.5 The constant (σ_3-u) control:

During the tests, it was necessary to maintain the difference between the cell pressure and the back pressure, i.e. (σ_3- u), constant at a predetermined value. This was achieved by connecting the lower pot of column 6 (fig.2.3) to the cell through valves 12 and 13. The top of the upper pot was connected to the sample drainage line through the volume gauge. With valves 12 and 13 open, the cell pressure is transmitted to the lower pot, and the back pressure as taken from the top of the upper pot represented this cell pressure minus the pressure equivalent to the head of mercury in the two pots. By adjusting the position of upper pot, it was possible to obtain any value of (σ_3-u) up to a maximum of 64.5 psi, which represented the maximum head of mercury obtainable in the laboratory.

Unlike the conventional self-compensating mercury control system, the upper pot was not open to the atmosphere. Since this pot formed a closed system with the back pressure line, it was essential to keep it de-aired, especially since there is a danger of cavitation in the line if, for some reason (such as inadvertently opening valves 12 and 13 when testing was not in progress), the pressure transmitted to the lower pot was substantially less than that represented by the head of mercury in the pots. The upper pot and the back pressure line were periodically de-aired by assembling

the cell without a sample, filling it with water, applying a constant cell pressure of 1000 psi and flushing water through column 6 by opening valves 12 and 13 and disconnecting the saran tubing at valve 15 which stayed shut. Care was taken to shut valves 12, 13 and 15 prior to lowering the cell pressure at the end of the test in order to prevent cavitation in the back-pressure line.

2.6 Volume gauge:

The necessity for making very accurate measurement of changes in the volume of the sample, coupled with the small magnitude of the expected volume changes (less than 0.5 cc) ruled out the use of a high pressure variation of the conventional volume gauge containing water and dyed paraffin (Bishop and Henkel, 1962). Furthermore the use of this type of volume gauge would require a "cross-over" system of valves, and this would increase the number of valves and connections in the back pressure line and therefore increase the danger of leakage under the high back pressures used in the experiment. It was therefore decided to replace the volume gauge with a length of 1/8" O.D saran tubing - a technique which had been used successfully by Tan (1968), Son (1968) and Skinner(1969).
The tubing was stretched along a yard scale which was screwed to a vertical board (fig.2.3) and was completely filled with water except for a thread of mercury about three inches long, which was formed into U-shape below the yard scale. The two ends of the mercury column were always adjusted to lie on a horizontal line scribed on the vertical board, so that there is no variation in the back pressure applied to the sample as a result of difference in the levels

of the two legs. The horizontal position was also used as a reference position, since any change in the volume of the sample caused a movement in the legs of the mercury column, and the change (Δl) in the length of tubing between valve 15 and the leading edge of the mercury column multiplied by the relevant tubing constant k (whose determination is discussed later) gave the magnitude of the volume change of the sample.

In order to facilitate the accurate measurement of the change in length (Δl) of the tubing, an ink dot was made on the tubing against the 20 inch marking on the yard rule, and the change in length of tubing necessary to equalize the two legs of the mercury column was then given by the difference between the final reading of the position of the dot and 20 - this difference being determined to two decimal places. A mercury column was preferred to an air bubble which was used by the earlier research workers, since the latter would change in length during the variations in back pressure in the course of the test. The length of 3" for the mercury column was considered sufficient to prevent leakage past the column from one side to the other, thus leading to errors in the determination of sample volume change, and the U-shape was adopted because, when laid horizontally, the mercury column tends to break up into shorter lengths, and this makes it difficult to determine the total movement of the mercury column accurately.

As the back pressure was increased during the test, the bore of the saran tubing (and hence the tubing constant k, which is directly related to the bore) also increased, and the quantity of water contained between

the leading edge of the mercury column and the sample (denoted by V_q) compressed. In order to determine the change in the volume of the sample accurately during the period of varying back pressures, it was essential to determine, by calibration, the variation of the tubing constant k (cc/in) with applied pressure, and to compute the correction for the compressibility of the quantity of water, V_q.

2.6.1 Calibration Tests:

In order to perform the calibration tests, the cell was carefully assembled as described in section 3.2.1.2, with the sample replaced with a solid brass dummy, 1 1/2" high and 3/4" diameter. The back pressure line had been thoroughly de-aired and the filter stones boiled. The cell was filled with water and topped with castor oil. The back pressure pots were adjusted in order to maintain the difference between the cell pressure and back pressure at 10 psi.

2.6.1.1 Variation of tubing constant k with cell pressure.

A thread of mercury, 30" long, was trapped in the tubing(type A) which was otherwise filled with water. One end of the saran tubing was connected to the back pressure line, while the other end was closed by shutting valve 15 (fig.2.3). The tubing was tautly stretched along a yard rule clamped on the bench, thus enabling the length of the mercury thread to be measured under various applied pressures.

Starting from a cell pressure of 50 psi, readings of the lengths of the mercury thread were taken at 100 psi intervals while the cell pressure was increased steadily up to 1000 psi. The readings were repeated with the cell pressure decreasing steadily from 1000 psi to 50 psi. The

readings of the length of the mercury thread taken while the cell pressure (and hence the back pressure) was decreasing were in general lower than those taken under corresponding pressures during the increasing cell pressures phase. The difference between the two corresponding readings was about 0.3" in a length of 30".

After two cycles of cell pressure variation the mercury was carefully flushed out of the tubing into a clean dish, all excess water soaked away with a soft tissue paper, and the weight of the mercury used was determined. The volume of the mercury was calculated, assuming the specific gravity to be 13.6 for mercury. Since this volume was constant during the test, the value of k (cc/in) under each value of cell pressure was calculated by dividing the volume of mercury (in cc) by the average of the four length measurements (in inches) under that pressure.

Two similar calibrations were performed starting with different initial lengths of the mercury thread. The three values of k for each pressure were averaged out.

It became necessary to change the saran tubing used in the first series of tests ($\sigma_3 - u = 10$ psi), because a longer length of tubing was required in order to facilitate the measurement of the anticipated increased volume changes of the calcite samples when tested under the condition $\sigma_3 - u = 64.5$ psi. A second type of tubing (type B) was therefore similarly calibrated for the condition ($\sigma_3 - u = 64.5$ psi). In order to check if the tubing underwent any permanent changes in shape as a result of prolonged use, a further check calibration was performed on the type B tubing after 8 months of constant use, the difference ($\sigma_3 - u$) still being maintained at 64.5 psi,

and the cell pressure being varied between 100 psi and 1000 psi.

Graphs were drawn showing the variation of the tubing constant k with cell pressure for both types of tubing (fig.2.4). Both graphs were straight lines, having a gradient of 2.1×10^{-6} cc/in per psi. The two calibrations performed 8 months apart for tubing type B, gave essentially the same results, thus showing that the tubing retained its shape after almost a year of constant use.

2.6.2 Corrections to the readings of volume change.

During the periods of changing pressures, the readings obtained for the volume change of the sample were corrected for the following:

(a) the change in the bore of the tubing with changes in the back pressure, by using the tubing constant relevant to the magnitude of the back pressure (Sect.2.6.1.1).

(b) the compressibility of the water contained between the leading edge of the mercury and the sample. It can be shown (see Appendix A) that the correction $(\Delta V)_q$ applicable when the back pressure is increased from a pressure p (say 100 psi) to a pressure q is given by

$$(\Delta V)_q = (\Delta X)_q K_{100} - X_q (K_q - K_{100}) \quad \ldots\ldots\ldots \quad (2.1)$$

where $(\Delta X)_q$ denotes the change in the length of the tubing when the pressure is increased from 100 psi to a value q.

X_q denotes the length of tubing in circuit under a back pressure q.

K_q, K_{100} denote the values of the tubing constant under pressures q and 100 psi respectively.

(c) the cubical compressibility of the soil grains as a result of increasing back pressure. This correction is

given by

$$\Delta V_s = C_s V_s \Delta U \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.2)$$

where C_s denotes the compressibility of the soil grains.

V_s denotes the total volume of the soil grains, and

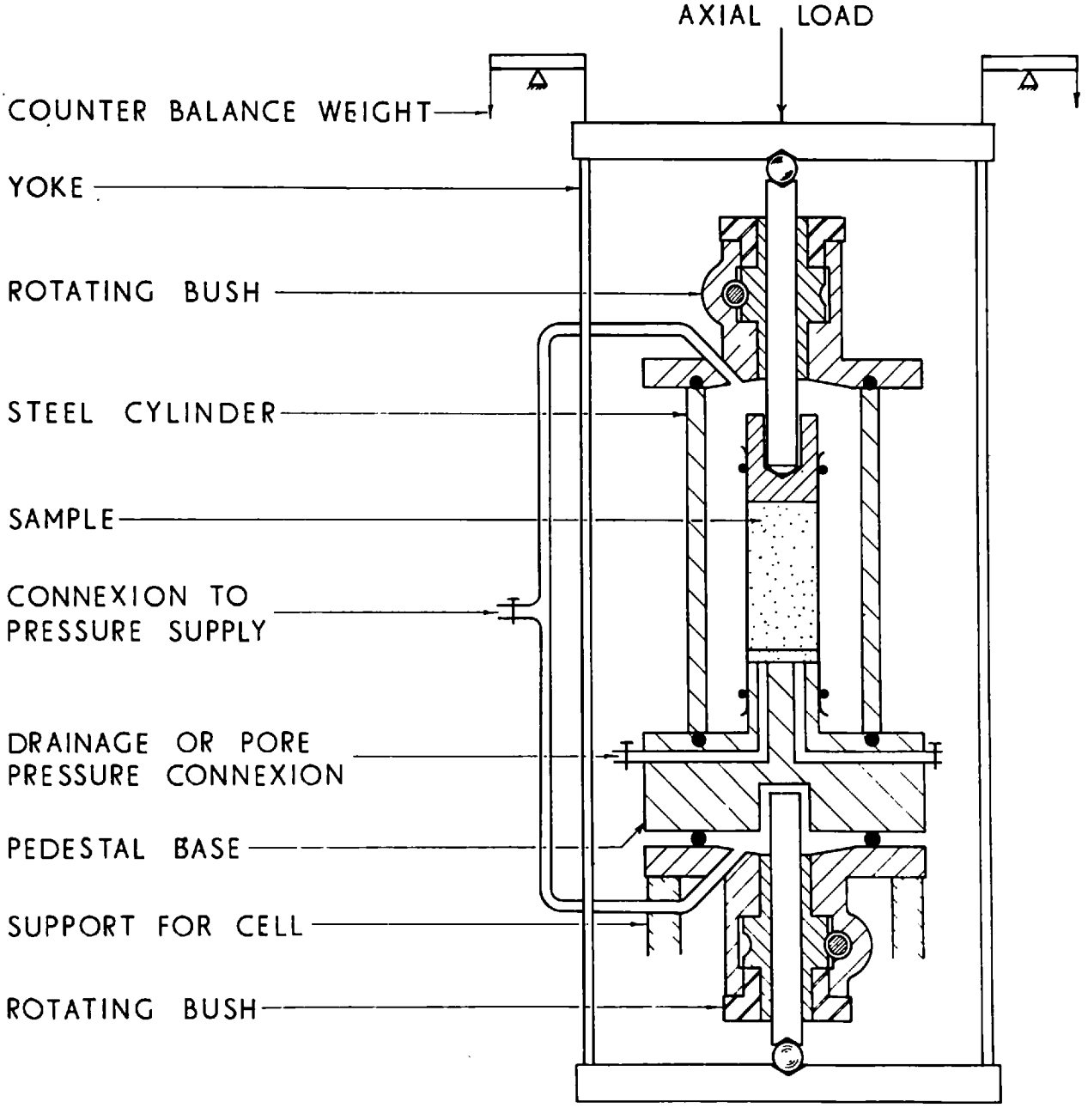
Δ. denotes the change in pore pressure.

Skempton (1960) quotes the value of C_s for marble as 1.4×10^{-6} per kg/cm^2 (2×10^{-5} per psi) and for London Clay as 2×10^{-6} per kg/cm^2 (3×10^{-5} per psi). In the absence of any such data on Wiener Tegel, the value of C_s was taken as approximately equal to that of London clay.

2.7 Measurement of axial loads and sample deformation:

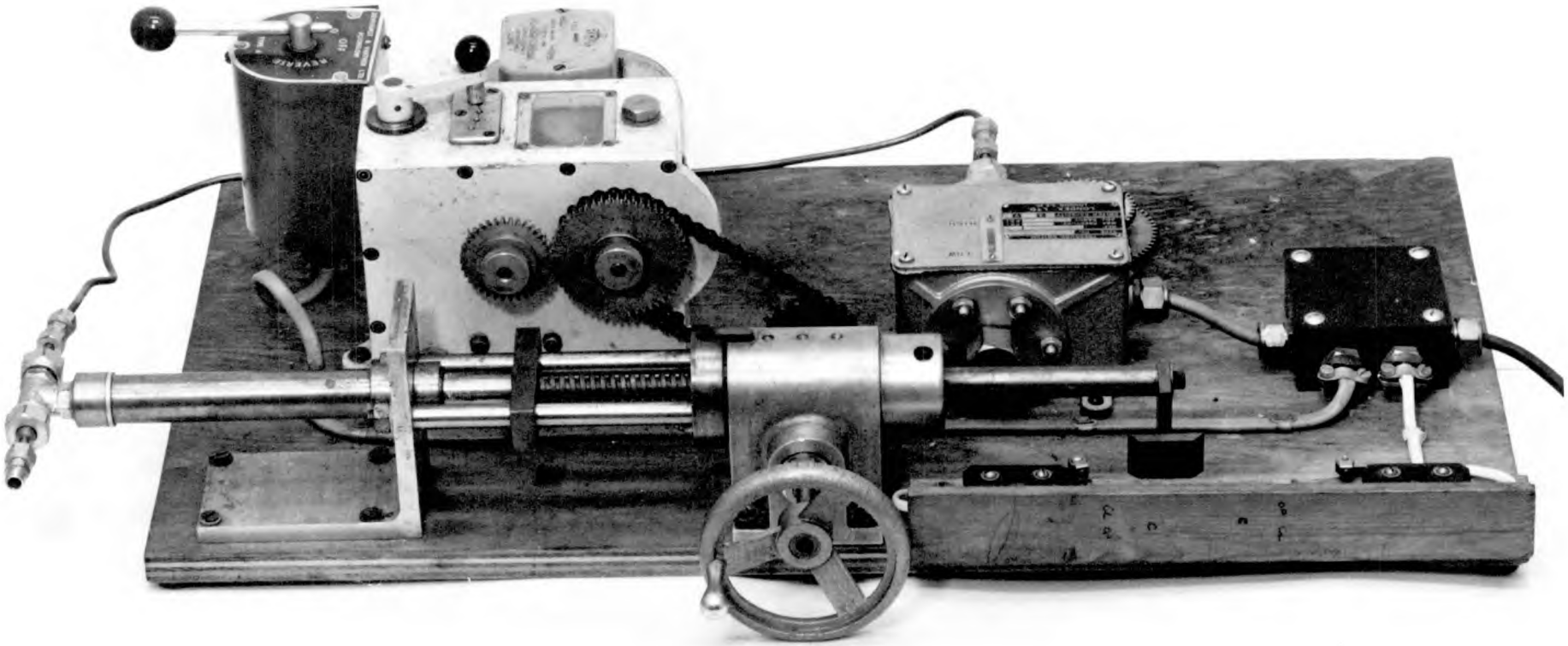
For reasons given in section 2.2, the axial load on the sample was measured directly with a high sensitivity high tensile steel proving ring of low load capacity. The ring had been previously calibrated with an accurate load calibrating device.

The deformation of the sample was measured, in the conventional way, with a dial gauge reading to 1/1000". However, in view of the anticipated changes in the position of the top cell cap during the periods of varying cell pressure, it was decided to support the dial gauge from the base of the cell instead of using the top of the cell as reference, as is done usually.



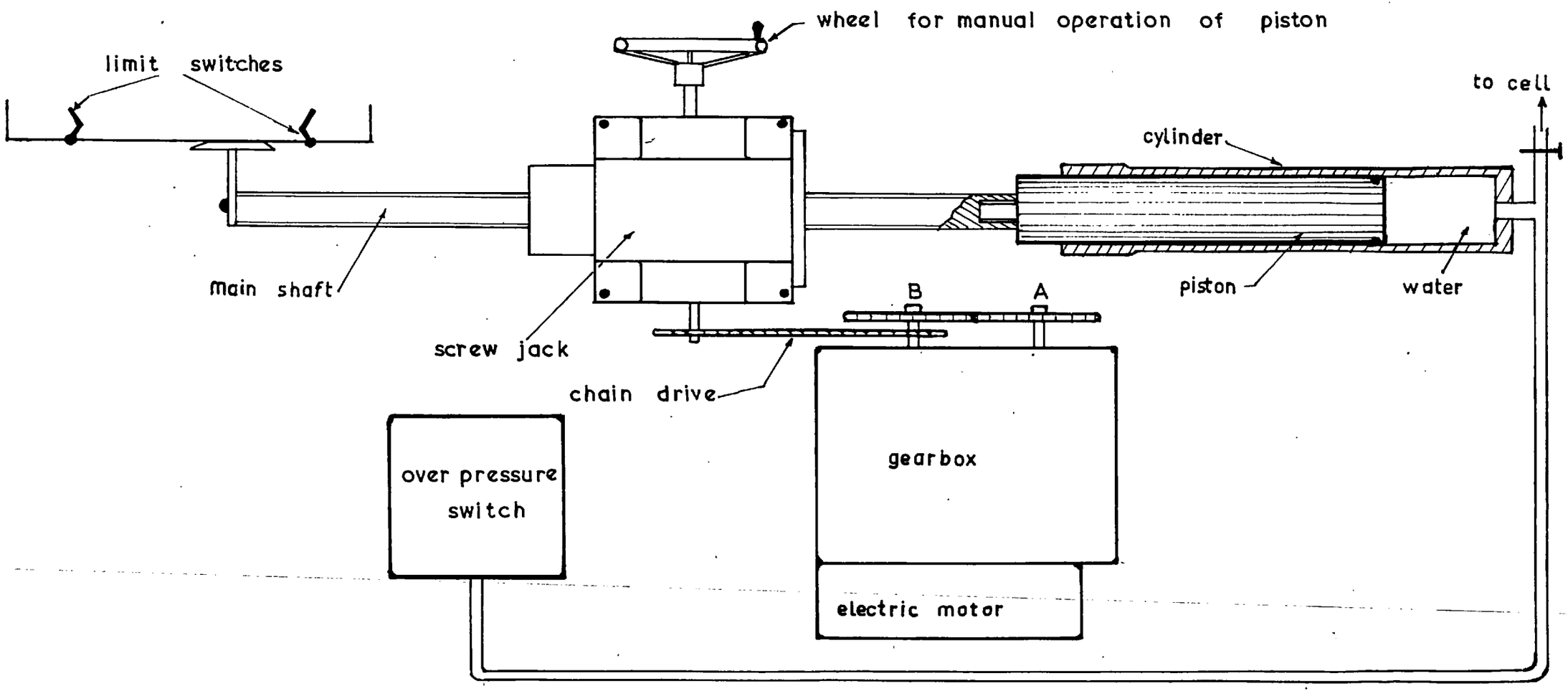
THE BALANCED RAM TRIAXIAL APPARATUS

FIG. 2.1



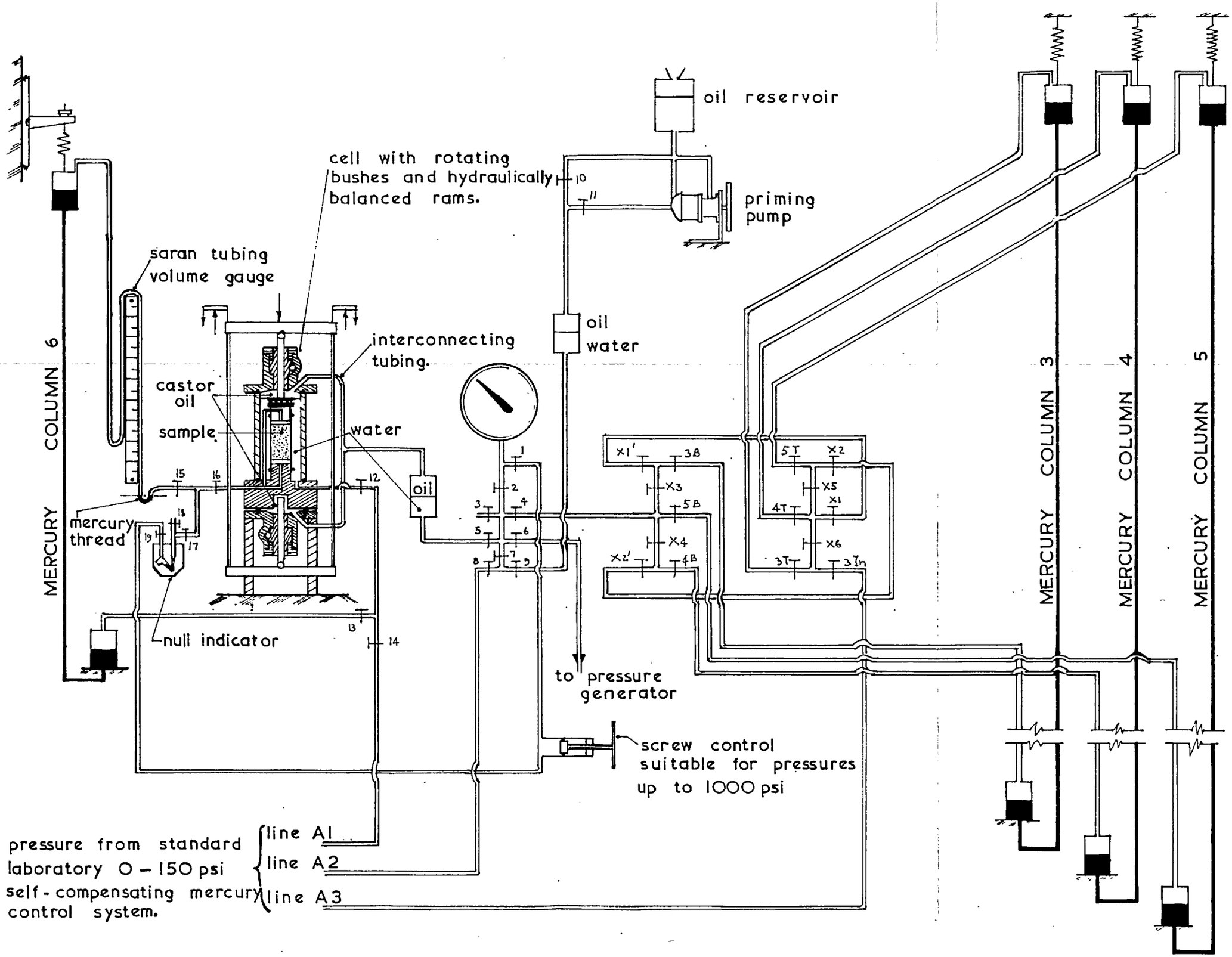
APPARATUS FOR CONTINUOUS VARIATION OF CELL PRESSURE

FIG. 2.2a



A SCHEMATIC DIAGRAM OF THE APPARATUS FOR CONTINUOUS VARIATION OF CELL PRESSURE.

FIG. 2·2b



A GENERAL LAY-OUT OF THE APPARATUS SHOWING THE SELF-COMPENSATING MERCURY CONTROL FOR PRESSURES UP TO 1000 PSI.

FIG. 2.3

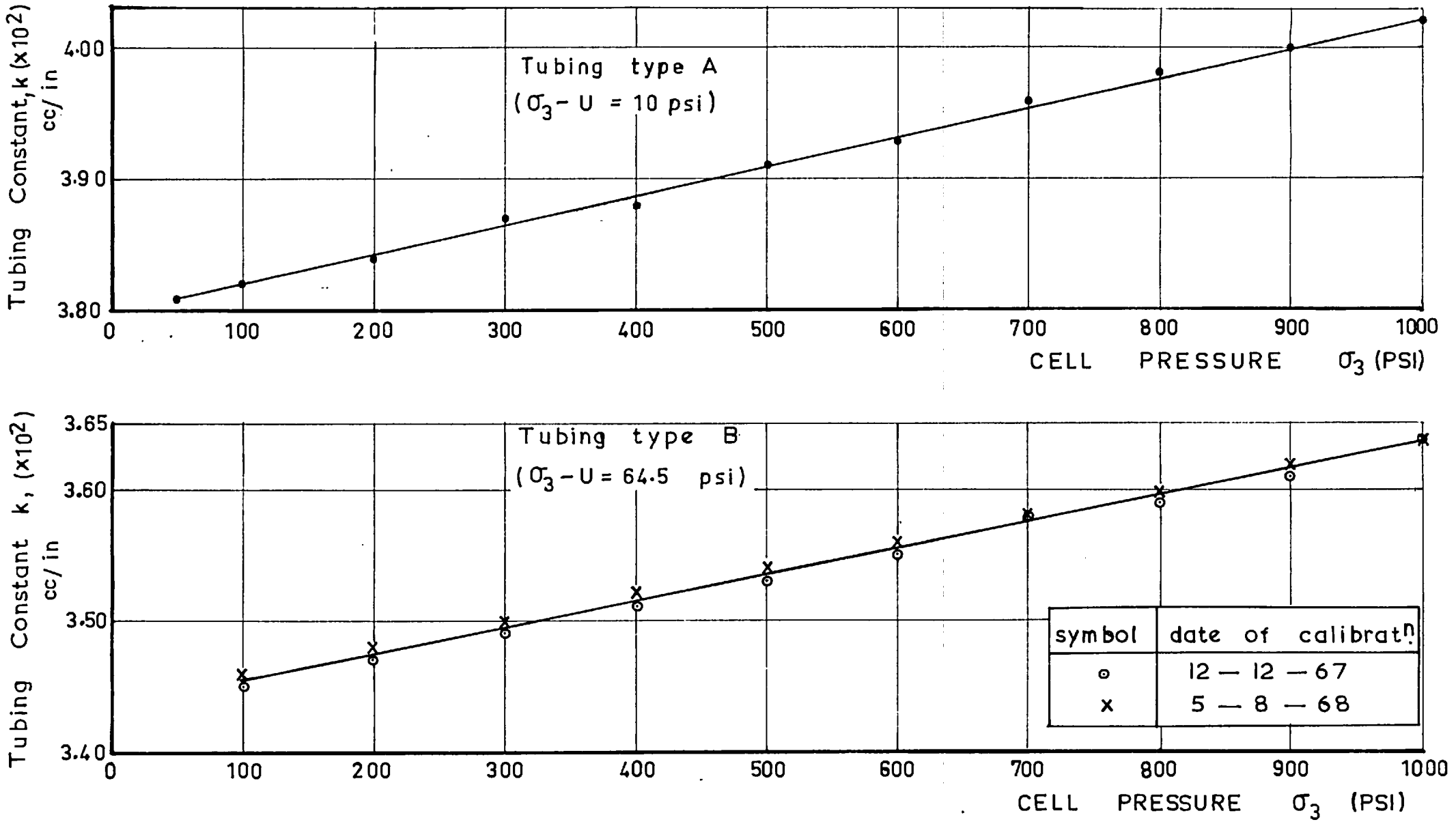


FIG. 2.4

CALIBRATION OF SARAN TUBING.

CHAPTER 3.

A DESCRIPTION OF THE PREPARATION AND PROPERTIES OF THE MATERIALS AND THE TESTING TECHNIQUES.

3.1 Preparation of Materials.

3.1.1 Remoulded Wiener Tegel.

Samples of this material, a silty calcereous clay from the vicinity of Vienna, were air-dried, ground in a mechanical grinder and sieved. The portion passing the No.200 B.S. sieve was made into a slurry at an approximate water content of 120 °/o which was almost two times the liquid limit. After being left to cure overnight, the slurry was transferred into a Hobart Mixer, and kept churned continously for one day, before being poured into a 9" dia. oedometer, the filter stone at the base of which had been well saturated previously.

After the clay particles had settled sufficiently, the excess water was siphoned off, and the soil gently loaded, first with a 9" dia. filter paper, then with the top cap of the oedometer, and finally with dead weights. Each increment of dead load was kept on for about 48 hours which was considered sufficient time for the completion of the consolidation under that load.

When the sample attained sufficient strength, the oedometer was transferred to the platform of a 5 ton triaxial loading frame, which was set running at a speed of 0.00002 in/min. The load applied to the top cap was measured with a 15 ton load cell. During the consolidation process, the sample was kept supplied with water on both faces to prevent

it drying. The top cap was made of aluminium and had perforations to permit drainage from the top face of the sample, in addition to the base drainage.

After loading for three weeks, the final load was read, and the sample slowly unloaded by reversing the motor. The soil sample was then carefully extruded from the oedometer, and the soil "cake" - which was 2" high and of 9" diameter - was cut into four parts and coated with paraffin wax after final moisture content samples had been taken.

The properties of the remoulded clay were as follows:

Initial water content of slurry	= 120 °/o
Final moisture content	= 34 °/o
Final load on sample	= 2.65 ton.
Estimated final consolidation pressure	= 93.3 psi
Liquid Limit	= 52 °/o
Plastic Limit	= 24 °/o
Plasticity Index	= 28 °/o
°/o Clay fraction	= 34 °/o
Activity	= 0.83

3.1.2 Calcite:

Clean marble chips were ground and sieved. The portion passing the No.200 B.S. sieve was used in the testing programme. The specific gravity of the calcite was 2.72, and the liquid limit of the portion passing No. 36 B.S. sieve was 12.4 °/o. It was not possible to perform the plastic limit test on the material, because of the difficulty of rolling 1/8" dia. threads of the material at low water contents.

The grading curves for both materials are given

in fig. 3.1.

3.2 Testing techniques.

3.2.1 Tests on remoulded Wiener Tegel.

3.2.1.1 Sampling and mounting of sample.

Samples were taken from the soil "cake" by pushing a thin-walled sampling tube of internal diameter 3/4" into it. In order to ensure that vertical samples were obtained, the sampling was done by placing the soil "cake" on the platform of a small drilling machine, care being taken to ensure that the face in contact with the platform was flat. A wooden dolly, 3/4" in diameter, was held in the chuck of the machine to act as a guide for the sampling tube. A piece of wood, about 3" square, with a 3/4" diameter hole in the centre, was slipped on to the wooden dolly, followed by the sampling tube (fig.3.2). The platform of the drilling machine was then gently raised till the tips of both the sampling tube and the wooden dolly just touched the top of the soil, and was locked in that position. A gentle pressure on the square piece of wood pushed the tube into the soil, guided by the wooden dolly. The sampling tube was removed from the soil by cutting around it with a spatula, the exposed surface of the soil being recoated with wax immediately afterwards.

The sample was then gently pushed out of the tube, and trimmed to size in a specially made split perspex mould. The two halves of the mould were located in position by means of pins. By using this mould, it was possible to trim the two ends of the sample plane and exactly to the final length of 1 1/2". After trimming, the sample was weighed and initial moisture content samples were taken

from the trimmings.

Prior to mounting the sample on the pedestal* of the triaxial machine, the back-pressure line and the pedestal were thoroughly de-aired by flushing water through them under high pressures (usually 500 psi). It was particularly important for the success of the experiment to have as "hard" a system as possible since any entrapped air in the system will result in erratic volume change readings being obtained during the subsequent changes in back-pressure. Before every test, the pedestal and the back-pressure line were left overnight under a pressure of 1000 psi in order to dissolve any pockets of air in the system - the drainage holes in the pedestal being blocked during this operation. The filter stones were boiled regularly and were kept soaked when not in use. To prevent the clogging of the filter stones and to enable the sample to be easily separated from the filter stones at the end of the test, two filter paper discs, 3/4" dia. were placed between the sample and each filter stone.

The sample was surrounded with a rubber membrane** of 0.008" thickness, and sealed to the pedestal with O-rings. The top drainage lead through the loading cap was well flushed before the loading cap was placed in position and the rubber membrane sealed to it with O-rings. A thin smear of silicone grease was applied to the top of the loading cap to retain 3/32" steel balls in position. These balls made contact with a 1/4" thick brass disc screwed to

---

* A brass adaptor was fitted to the pedestal to enable it to take samples 3/4" diameter instead of the conventional 1 1/2" dia. samples.

** These special rubber membranes suitable for surrounding 3/4" dia. samples were supplied by Precision Dippings Ltd. of Yate, Gloucestershire.

the end of the loading ram when the apparatus is assembled. This arrangement was to ensure free horizontal movement of the loading cap, should the sample fail on a single shear plane, thus minimising the horizontal thrust on the loading ram.

3.2.1.2 Assembly of apparatus.

The stainless steel cell was centrally placed on the O-ring inserted into the base, and was filled with water to within half an inch of the top. The weight of the cell acting on the O-ring was enough to prevent leakage beneath the cell. Three studs, equally spaced round the base, and each with a loose fitting brass spacer tube, were screwed into the base till about an inch of thread protruded beneath base. The top cap was gently placed on the cell so that the O-ring in the top cap rested centrally over the top of the cell, the loading ram being prevented from resting on the sample by means of a perspex spacer. Nuts and washers were screwed on to both ends of the studs and tightened down till the three brass tubes, which were of equal length, were gently wedged between the base and the top cap. This ensured that the top cap and the base of the apparatus were parallel.

The three screws which hold the lower cap to the base when the apparatus is not in use, were then removed and replaced with three more studs which were also tightened down with nuts and washers.

The lower ram was pushed into the recess below the base (fig. 2.1) as far as it could go without touching the base. The yoke without the upper beam was then installed and held in position with counterbalance weights.

The contact between the lower ram and the lower beam of the yoke was through a 1/2" diameter steel ball. The short beams across the knife-edges were then adjusted to be horizontal* by means of two pairs of lock nuts which connected the counterbalance system to the main yoke. The perspex spacer was then removed to enable the loading ram to make contact with the sample, and the upper beam of the yoke set up, adjusted to make contact with a steel ball on top of the loading ram, and locked in that position with lock nuts at each end. The proving ring and the dial gauge for measuring sample deformation, were installed, and the main cell chamber and the recess below the cell base were connected by means of the interconnecting tubing.

The air release valves on the cell caps were opened and castor oil fed in from a water-castor oil interchange pot, to fill space above the water in the main cell as well as the entire lower chamber. The air release valves were then shut and the initial cell pressure of 100 psi was applied in steps and held constant by a conventional Bishop-type mercury system (line A2 in fig.2.3) by opening valves 5, 7 and 8 (fig.2.3). The difference in the levels of the mercury in the back pressure system (mercury column 6 in fig. 2.3) had been set to give a pressure of 64.5 psi, and the cell pressure and the back pressure systems were interconnected by opening valves 12 and 13. During the consolidation and the preliminary shear stages, therefore, the net back pressure would be 35.5 psi which was considered high enough to dissolve any remaining air bubbles in the drainage lines, and also further ensure sample saturation (Bishop and Henkel, 1962).

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* It was essential to keep these beams horizontal throughout the experiment. Any significant departure from the horizontal position results in some load on the proving ring due to upthrust, and therefore errors in determining the load on the sample.

3.2.1.3 Isotropic Consolidation Stage.

Consolidation of the sample was started by opening valves 15 and 16, and during the consolidation phase a graph was drawn of the changes in the length of the saran tubing (which was directly proportional to the volume change of the sample) against the square root of the elapsed time ($\sqrt{t}$) in minutes. The primary consolidation was completed in about three hours on account of the short drainage path (3/4"). From the curve of volume change against $\sqrt{t}$, the theoretical testing rate to ensure negligible excess pore water pressures at the peak deviator stress, was computed according to the usual formula (Gibson and Henkel, 1954) to be of the order of 0.0002 in/min. The actual testing rate selected was 0.0001 in/min, since it was found from experience that the use of this rate of testing enabled the subsequent changes in the cell and back pressures to occur over about 2°/o axial strain, thus giving a convenient separation of points.

3.2.1.4 Preliminary Shear Stage.

After the consolidation stage, the initial length of saran tubing in circuit (that is the length of tubing between valve 15 and the leading edge of the mercury column) was accurately measured using a yard rule, the levels of the two legs of the mercury column having been equalized. To facilitate the accurate determination of changes in the length of tubing in circuit during shear, a small ink dot was made on the tubing against the 20 inch marking on the yard rule against which the tubing ran. Any change in sample volume caused a difference in the levels of the legs of the mercury column, and the change in the

length of tubing necessary to equalize the two legs of the mercury column can be read directly on this scale. The magnitude of the volume change is given by the product of this change in length (Δl) of tubing and a tubing constant whose determination has been described elsewhere (see 2.6.1.1).

The loading platform of the triaxial frame was gently raised manually until the loading ram just made contact with the loading cap on the sample. The point of contact was difficult to judge, since the sample could not be seen. It was deemed to have occurred when there was a deflection on the dial gauge on the proving ring. The zero readings on both the strain and load dial guages were noted, as well as the initial length of tubing in circuit. The sample was then sheared drained, graphs of axial load, deviator stress and volumetric strain being drawn against axial strain as the test progressed.

3.2.1.5 Shear stage incorporating varying cell and back pressures.

When the deviator stress versus strain curve attained a peak value, usually at an axial strain of approximately 9 %, valve 7 (fig.2.3) was shut, thus cutting off the mercury pots maintaining the cell pressure constant, and valve 6 opened thus putting the hydraulic pressure generator into the circuit. The hydraulic pressure generator was switched on and the appropriate gear ratio selected to enable it to increase the cell and back pressures at a suitable steady rate* from 100 psi to 1000 psi in about four hours.

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* The choice of a suitable pumping rate is discussed in Chapter 4.

During this period of increasing cell pressure and back pressure, three sets of readings of axial load, deviator stress and volumetric strain were taken, reduced and plotted. The readings of volume change were corrected for the expansion of the saran tubing, and the compression of water and the soil grains. The cell pressure was maintained constant at 1000 psi, by opening valve 4 which puts lines 3, 4 and 5 of the high pressure self-compensating mercury system in series with line A3 (fig.2.3) to give a maximum pressure of 1000 psi. Two sets of readings were taken while the cell pressure was constant at 1000 psi, after which the cell pressure was steadily decreased to 100 psi, three more sets of readings were taken during the phase of decreasing cell pressure.

Each test involved two such cycles of pressure change.

The test was continued for about 2% axial strain beyond the end of the second pressure change cycle. At the end of the test, the drainage leads were shut, the cell pressure lowered to zero, the cell fluid drained away and the apparatus dismantled. Where the sample failed on a single shear plane instead of barrelling, the inclination of the shear plane to the horizontal was accurately measured with a bevel protractor, and final moisture content samples were taken either from the shear zone, where there is a distinct one, or from the central part of the sample.

Two series of tests were presented on this material. In the first series of tests, the difference between the cell pressure σ_3 and the back pressure u was maintained constant at 64.5 psi and the initial cell pressure was 100 psi. Three results were presented in this series. The first

two (tests 1 and 3) were on samples taken from the main block of material prepared from slurry. The overconsolidation ratio was therefore about 1.5. The third sample (test 2) was prepared in two parts in a brass former of 3/4" dia, and one of whose ends was cut at angle of 55° to the horizontal. The inclined face of each half of the sample was well polished on a glass plate before the two halves were put together in the perspex former and trimmed to size. The presence of the preformed shear plane in the sample was to ensure that the resulting stress-strain curve was reasonably flat over a wide range of axial strain to enable the pressure changes to take place over this range. The strength obtained under these conditions approximates to the residual strength of the material.

In the second series of tests, $(\sigma_3 - u)$ was kept at 10 psi, and the initial cell pressure was 50 psi. Test 4 was on a sample cut from the material prepared from slurry, test 5 was on a sample remoulded in a former, and test 6 was initially heavily overconsolidated, with an O.C.R of 50.

3.2.2 Tests on Calcite.

3.2.2.1 Preparation of sample.

Approximately 150 grams of oven-dried calcite was weighed out, and made into a paste using de-aired water. More water was slowly added as the mixture was thoroughly stirred. In this way, it was possible to obtain an intimate mixture*, which was later de-aired by subjecting it to a vacuum of 28 inches of mercury for three hours.

The sample was sedimented in the usual way in a split former assembled on the pedestal (Bishop and Henkel,

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* Earlier attempts to prepare the slurry by adding water directly to the calcite failed because of the difficulty of getting all the particles coated with water.

1962). In this case, however, the former had a detatchable perspex extension piece to enable samples to be prepared to a height of more than the final size of 3" and then trimmed to the final height. The de-aired calcite slurry was gently poured into the extension piece and allowed to sediment, the excess water being siphoned off when the particles settled. In order to cause the rubber membrane to adhere to the walls of the split-former and therefore result in cylindrical samples being obtained, it was found necessary to apply a suction to the split-former. When the sedimentation was over, the top of the sample was covered with a 1 1/2" diameter disc of rubber membrane to prevent the entry of air into the sample while a suction of about 2 psi was applied to the base of the sample to stiffen it prior to the removal of the extension piece and trimming the sample level with the top of the split-former, using a spatula. The top filter stone and loading cap were placed in position, the rubber membrane rolled back on to the loading cap and sealed to it with O-rings. The suction was then increased to 5 psi and when all drainage from the sample ceased, the split former was carefully removed and the sample dimensions measured.

3.2.2.2 Measurement of sample dimensions.

The height of the sample was measured at four diametrical points on the perimeter of the loading cap and in the middle, by means of a depth gauge resting on an aluminium cylinder placed on the cell base.

Readings of the diameter of the sample were taken across two perpendicular diameters, and at three levels, using a modification of the conventional dial gauge indicator

(Bishop and Henkel, 1962). Both the depth gauge and the dial gauge indicator had been previously calibrated by taking similar readings on a brass dummy sample which was exactly 3" high and 1 1/2"diameter, the brass dummy being mounted on the pedestal with the same loading cap and filter stones and enclosed in the same thickness of rubber membrane as used in the actual test.

From the zero readings taken on the brass dummy sample and the averages of the readings taken on the actual sample, it was possible to compute the initial height and diameter, and hence the initial volume of the sample.

3.2.2.3 Consolidation and shear stage:

The cell was assembled and filled with water and castor oil as described elsewhere (see section 3.2.1.2). An initial cell pressure of 5 psi was then applied to the sample and the 5 psi suction removed. The cell pressure and back pressure were increased in steps to the starting values, and after the consolidation stage, which took less than fifteen minutes, the test was conducted as described for Wiener Tegel. The rate of strain was 0.001 in/min and the variation of cell pressure and back pressure between the minimum and maximum values took place over one and a half hours.

Two series of tests were performed on the material. In the first series, the difference between the cell pressure and the back pressure ($\sigma_3 - u$) was set at 63.5 psi. The initial cell pressure was 100 psi. Of the three tests presented, two (tests 7 and 8) were on normally consolidated samples and the third (test 9) was on an over-consolidated sample with an over-consolidation ratio (O.C.R) of 6.3. The

initial cell pressure in the second series of tests was 50 psi, and the difference (σ_3 - u) was maintained constant at 10 psi. Tests 10 and 11 were on normally consolidated samples while test 12 was on an overconsolidated sample with an O.C.R. of 5.

3.2.3 Comparative drained tests:

As a comparison with the results obtained from test with varying cell and back pressures, two conventional drained tests were performed on each material. The value of (σ_3 - u) was kept constant throughout the tests at a value corresponding to each used in the main testing programme.

The results of these comparative tests are also presented.

3.3 Computation of various quantities.

3.3.1 Axial loads:

The axial load applied to the sample was determined directly from the dial gauge readings on the high tensile steel proving rings, which had been accurately calibrated against a dead load tester. As explained elsewhere (section 2.2) the provision of the hydraulically balanced ram system enables a sensitive * proving ring to be used, since effect of the upthrust on the loading ram had been counterbalanced. No correction had been applied for the friction between the loading ram and the ~~brushing~~ bushing since a rotating ~~brushing~~ bushing (Bishop et al (1965)) had been used.

3.3.2 Deviator Stresses:

The deviator stress on the sample up to the maximum value was computed by dividing the axial load by the relevant

* The sensitivity of proving rings has been discussed in detail by Bishop and Henkel (1962).

cross-sectional area A_ε , which is given by

$$A_\varepsilon = \left(\frac{1 + \frac{\Delta V}{V_o}}{1 - \varepsilon} \right) A_o \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.1)$$

where ΔV denotes the total change in the volume of the sample corresponding to an axial strain ε

and A_o and V_o denote the initial cross-sectional area and volume respectively.

This relationship was derived on the assumption that the sample deformed as a right cylinder. Owing to the influence the end restraints impose on the sample, the samples generally bulge instead of deforming uniformly. The error involved in using this relationship, in general, increaseswith increasing axial strain, especially in samples which undergo large deformation prior to the attainment of peak strength.

3.3.2.1 Post failure Analysis:

In general, the tests on normally consolidated samples of clay and calcite continued to bulge after the attainment of the peak strength, and in these cases, the relevant cross-sectional area was computed from equation (3.1). In tests on overconsolidated samples of clay (test numbers 1, 3 and 6) and in test 2 which had a preformed shear plane, failure occurred on a single shear plane. Since it is not clear whether the sample continued to bulge after this type of failure, all the post-failure deformation was assumed to result from the relative sliding between the two portions of the sample (see the insert on fig.3.3) and the mean cross-sectional area was then taken as the overlapping area between these two parts. It can be shown (Webb, 1966) that for a sample with a height to diameter ratio of 2,

this area A_c is given by

$$\frac{A_c}{A_p} = \frac{\alpha - \sin\alpha}{\pi} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3.2)$$

where

$$\alpha = 2 \cos^{-1}\left[\left(\frac{2\Delta h}{h_o} \right) \cot\theta \right] \ldots\ldots (3.3)$$

In the above equations

A_c denotes the overlapping area,

A_p denotes the cross-sectional area of the sample at peak deviator stress, and is given by equation (3.1)

Δh denotes the total post-peak vertical displacement,

h_o denotes the initial height of the sample

and θ denotes the inclination of the shear plane to the horizontal.

The exact point of appearance of the shear plane, and hence the value of A_p is subject to some controversy (. LaRochelle, 1967). In this case, the value of A_p was taken as the cross-sectional area at the point of the substantial drop in axial load normally associated with this type of failure.

To make the computation easier, use was made of the graphs showing the variation of the ratio $(\frac{A_c}{A_p})$ with vertical post-failure deformation (fig. 3.3) to calculate the values of A_c .

This type of failure causes a severe distortion of the sample and the rubber membrane and is considered unreliable for post-failure strains in excess of 8 °/o (Webb, 1966).

3.3.3 Volume Change Measurements.

The readings of changes in the volume of the sample, taken during the periods of varying back pressure have been corrected for:

(a) Changes in the bore of the saran tubing used for measuring volume changes.

(b) The compressibility of the volume of water trapped between the sample and the mercury in the saran tubing

(c) The compressibility of the soil grains.

The determination of the calibration curves and the application of the corrections have been discussed in Sections 2.6.1.1 and 2.6.1.2. All the corrections were positive for increasing back pressures and negative for decreasing back pressures.

PARTICLE SIZE DISTRIBUTION

Location _____ Boring No _____ Sample No _____ Date of test _____

Description _____

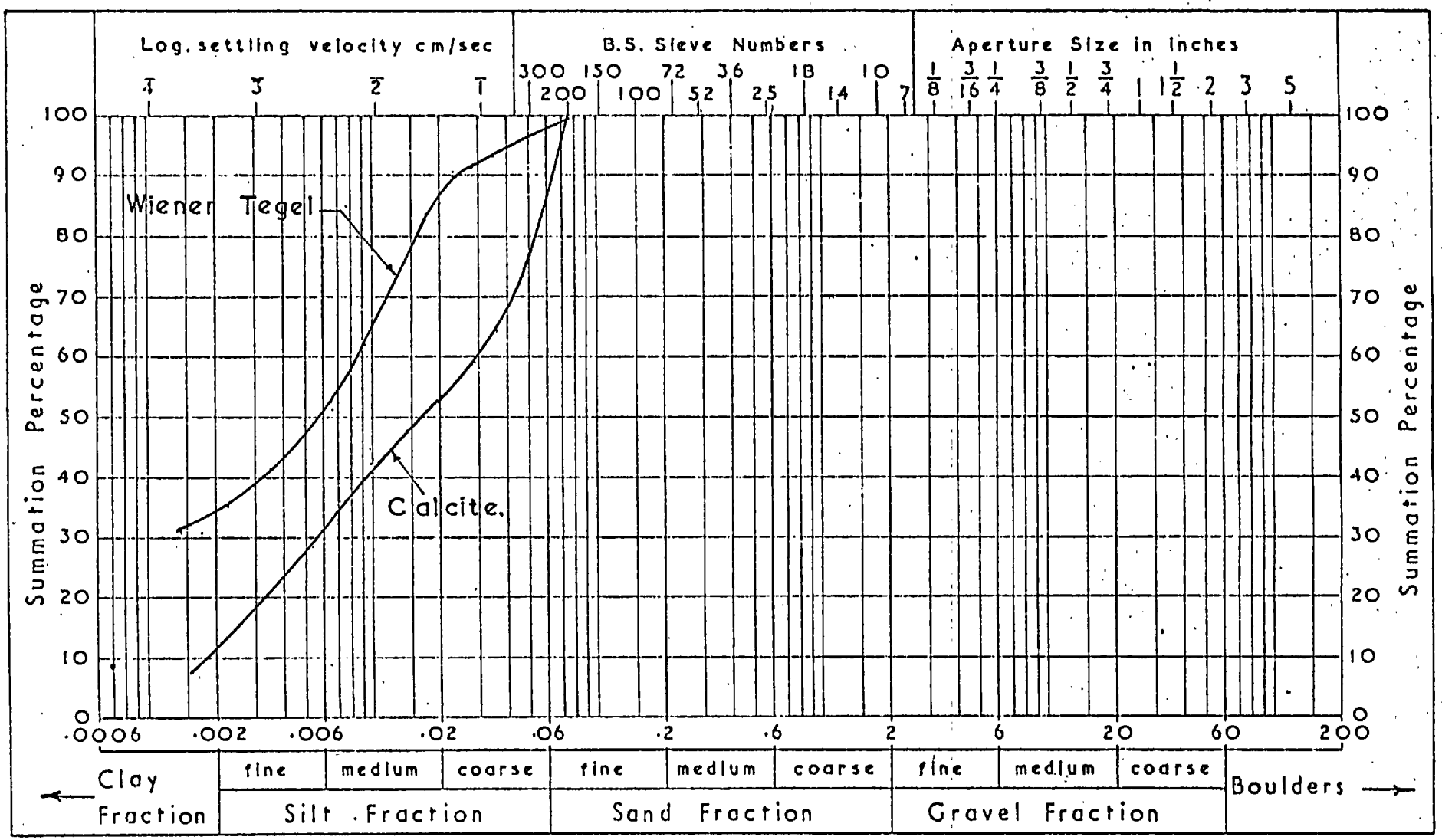
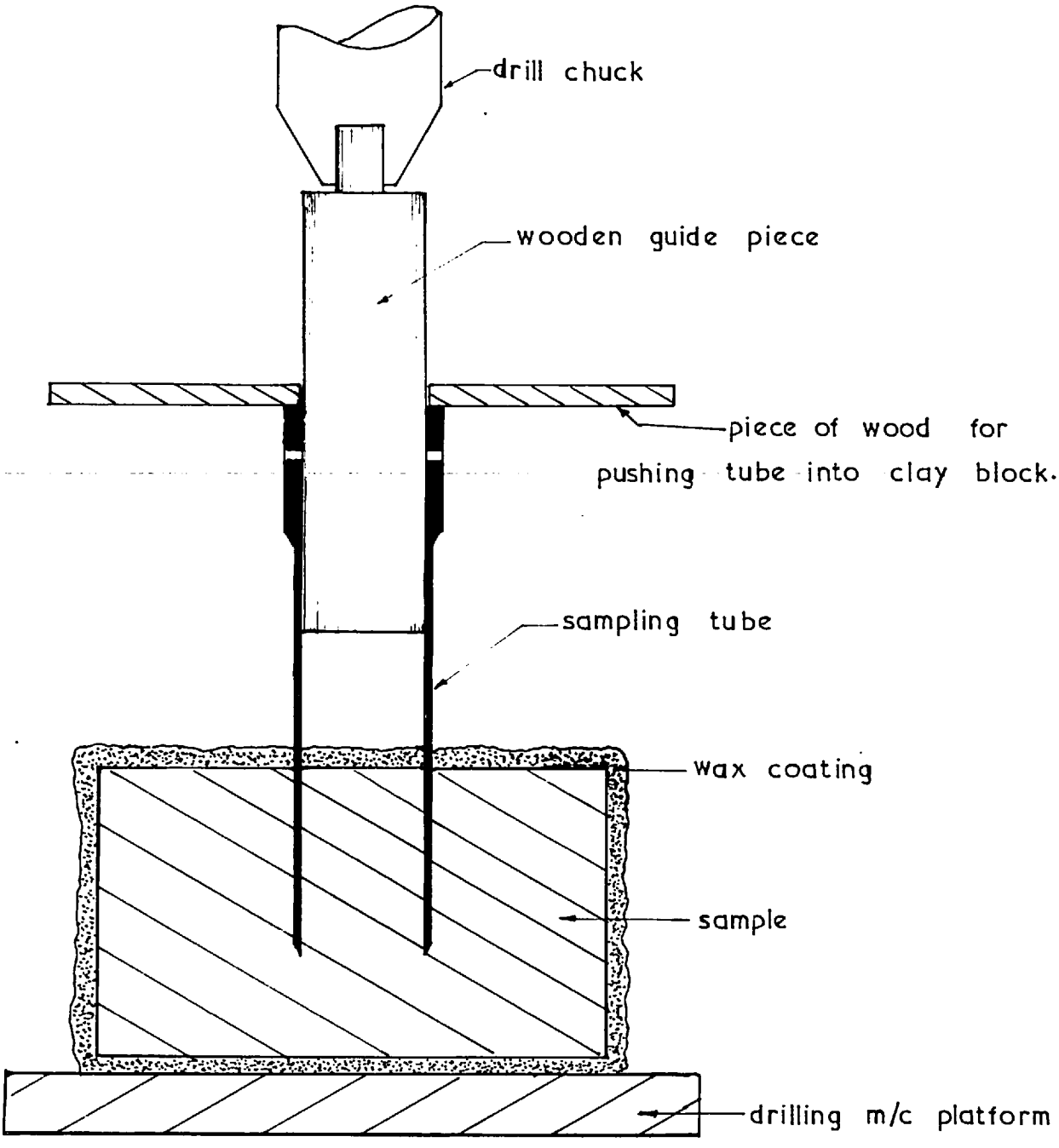


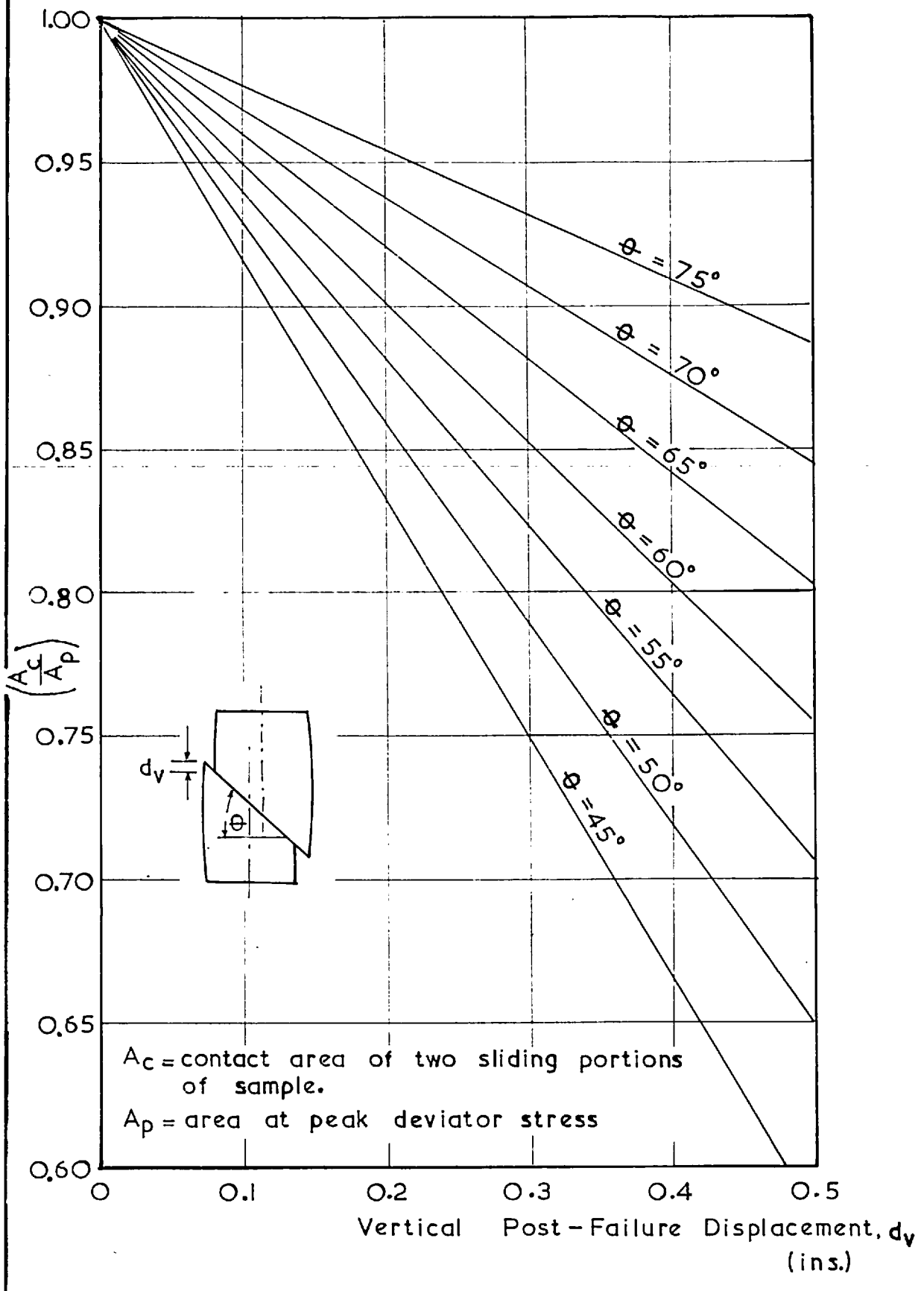
FIG. 3.1

'SOIL MECHANICS DEPT
IMPERIAL COLLEGE



METHOD OF OBTAINING VERTICAL SAMPLES FROM BLOCKS OF CLAY.

FIG. 3.2



POST FAILURE ANALYSIS OF TRIAXIAL SAMPLE. [Webb, 1966]

FIG. 3.3

CHAPTER 4.

ANALYSIS, PRESENTATION AND DISCUSSION OF RESULTS.

4.1 Sources of error in the triaxial test:

In its usual form, the triaxial test is performed on a cylindrical sample of soil enclosed in a rubber membrane, which is sealed to a comparatively more rigid pedestal at the base and loading cap on top. Theoretical analyses by Filon (1902) and Balla (1960) showed that the provision of these rigid end restraints result in an elastic stress distribution which shows a minimum value at the centre of the sample and a maximum at the edges. A similar stress distribution has been measured in experiments, by Shockley et al (1960), and Barden et al (1965).

Other consequences of the provision the rigid platens at the ends of the triaxial sample are:

(a) the bulging of the sample, leading to difficulties in the estimation of the cross-sectional area of the sample (see section 3.3.2), and therefore leading to errors in the computation of the average deviator stress in the sample.

(b) Non-uniform distribution of axial strains in the sample, the variation over the cross-sectional area being in the reverse manner to the variation of axial stress (Duncan et al, 1966).

(c) Non-uniform distribution of volumetric strains. Experimental work by Shockley et al (1960),

and Blight (1963) showed that, depending on the initial placement conditions and stress level, the ends of a sample subjected to drained shear may be consolidating or dilating at a slower rate compared with the dilation rate at the centre of the sample. The volumetric strain, as measured in the triaxial apparatus, therefore represents an average of the volumetric strains at the various parts of the sample. In undrained tests, this non-uniformity of volumetric strains may manifest itself in the internal migration of moisture within the sample, thus invalidating the basic assumption that these tests take place under constant moisture contents.

(d) Non-uniform pore pressure distribution within the sample. This item is discussed in more detail later.

The main methods suggested in the literature to deal with the detrimental effects of rigid end restraints are:

(i) the use of samples with a height-to-diameter (H/D) ratio of 2:1, since it is known that for a sample with this $\frac{H}{D}$ ratio, the stress at mid-height is essentially uniform over the cross-sectional area of the sample. Work by Bishop and Green (1965) and Taylor (1941) demonstrated that the rigid end platens have little influence on the peak strength, for a sample with this $(\frac{H}{D})$ ratio. This method, however, does not prevent the bulging of the sample.

(ii) The use of polished and lubricated end platens, as has been suggested by Rowe et al (1964) and Barden et al (1965). This method has been investigated in some detail by Duncan et al (1966), who concluded that

the advantages obtained from its use did not justify the increased experimental complications it introduced, especially since the elimination of the effects of end restraint has no influence on non-uniformities arising from lack of homogeneity, or the presence of fissures in the sample.

(iii) the use of the segmental sample, made up of a central test specimen and discs of the same material sealed in rubber membranes at the ends. This technique, which is similar to that used by Newman et al (1962) in tests on concrete cubes, had been proposed and discussed by Blight (1965).

The significance of these and other sources of error on the results obtained from the triaxial apparatus have been discussed in some detail by Bishop and Henkel (1962), La Rochelle (1967) and Duncan et al (1965). However, in view of the comparative nature of the tests performed, it is believed that the only sources of error which could seriously affect the results obtained were not those which were common to all triaxial tests, but those whose influence varied from one test to another, and from one part of the test to the other. The only possible source of error which was considered of particular importance to the accuracy of the results obtained, was that due to the non-uniform distribution of pore pressures in the sample.

4.2 Non-Uniform distribution of pore pressures.

Non-uniformities in the distribution of pore pressures in the sample may arise from:

(i) Inherent non-uniformities, such as variable

density and the presence of fissures. This is unlikely to be a serious source of error in this case, since the tests were performed on remoulded samples of clay, prepared with care.

(ii) The application of shear strains to the sample. The errors arising from this source may be overcome by a judicious choice of the rate of application of shear strains (Gibson et al, 1954) to ensure that the excess pore pressures were adequately dissipated in drained tests, or were allowed time to equalize in undrained tests.

(iii) Non-equalization of applied pore pressures. This source of error appears to be the most serious in the testing programme, since at various stages in the shearing process, both the cell pressure σ_3, and the back pressure u were changed by equal amounts so as to maintain (σ_3-u) constant. The problem, with regard to the test on Wiener Tegel, was to determine the most suitable rate of increasing and decreasing the back pressure without causing a substantial pore water pressure gradient in the sample.*

4.2.1 Propagation of pressure states in soils.

If the water and soil grains comprising the skeleton of a fully saturated soil be considered incompressible, any pressure gradient applied to the sample would be instantaneously transmitted throughout the sample. In general, however, both the water and the soil grains have finite compressibility, and hence the movement of a certain amount of fluid is essential for the propagation of pressure

* The case of decreasing back pressure would be analogous to the conditions in the clay core of an earth dam subjected to a rapid draw-down after the establishment of steady state conditions (Paton and Semple, 1960).

states in soils. The time required for the movement of the fluid through the sample, and, therefore, the equalization of the applied pressures will depend on the viscosity of the fluid and the permeability of the soil skeleton.

Mayer and Habib (1955), on the basis of experiments to study the decay of pore pressures in soils initially subjected to a pressure gradient, concluded that the time required for the equalization of pore pressures, was comparable to the time required to saturate the sample. More recently, Barberis (1965) studied the problem further and concluded that

(a) the transmission of pressure states in soils is not instantaneous.

(b) the speed of transmission varies directly as the pressure difference, Δu, and the magnitude of the initial pore pressure u_o, and inversely as the permeability.

and (c) positive pressure differences are propagated faster than negative differences.

In the absence of any quantitative method of assessing the speed of transmission of pressure states in soils, various methods of accelerating the equalization process, have been proposed in the literature. Some of these are the provision of radial drainage by surrounding the sample with a filter paper drain, or the provision of central sample drainage by means of a wet sand column, a wick or some such free-draining material. Neither of these methods was considered suitable in this case, since their use might cause equalization of pore pressures only on the perimeter of the sample. Besides, their influence on the strength and deformation characteristics of the sample may

be difficult to assess, especially in the case of the latter method.

It was, therefore, decided to accelerate the equalization process by providing a short drainage path. This was achieved by using a sample which was 1 1/2" high and of 3/4" diameter, thus maintaining the $(\frac{H}{D})$ ratio of 2:1, and providing for a two-way sample drainage. The effective drainage path therefore was 3/4". A calibration test was therefore performed to determine the rate at which to vary the cell pressure and the back pressure in order to maintain a negligible pore pressure gradient between the ends of the sample and the centre.

4.2.2 Calibration Test:

A sample of clay 3/4 " high and 3/4" diameter, cut from the main block of clay prepared from slurry, was mounted on the pedestal which had been extended by the provision of a perspex piece of the same dimensions as the sample (see insert in fig. 4.1). The drainage connections in the base of the cell and the filter stones had been previously de-aired thoroughly, and the hole in the perspex piece filled with water. The top drainage line was connected to the back pressure line and the sample bottom drainage line connected to a de-aired special Bishop-type null indicator which was capable of withstanding pressures as high as 1000 psi. The difference between the cell pressure and the back pressure $(\sigma_3 - u)$ was set at 64 psi.

The high pressure triaxial apparatus was assembled as described elsewhere (see Sec.3.2.1.2),the cell filled with water and topped with castor oil. An initial cell pressure of 100 psi was applied and full consolidation

permitted under the conditions σ_3 = 100 psi and u = 36 psi, the progress of the consolidation process being monitored by measuring the pore pressure at the base of the sample.

At the end of consolidation, the pressure generator (section 2.3) was set running and both the cell pressure and the back pressure applied to the top of the sample, increased at a steady rate. Regular simultaneous readings were taken of the cell pressure and the pore pressure at the base of the sample, while the cell pressure was increased steadily to 1000 psi, held constant for about half an hour and decreased to the initial value of 100 psi. A graph of the pore pressure at the top of the sample (u_t) against the pore pressure at the base of the sample (u_b) was drawn.

4.2.3 Discussion of the results of the calibration tests.

It was found that a nominal steady pumping rate of 200 psi/hr was necessary to ensure equality between u_t and u_b (fig.4.1) when the drainage path was 3/4" long. A faster rate of increase of u_t caused u_b to lag behind u_t, and a slower rate of increasing u_t posed problems since the substantial leakage occurring from the cell under high pressures reduced the effective pumping rate considerably. Because of this leakage, the rate of change of u_t could not be strictly expected to be constant during the test, even though steps were taken to compensate for the leakage by increasing the rate of pumping as the cell pressure increased.

Values of u_b taken during the unloading phase all plotted above the $u_b=u_t$ line in fig. 4.1. This would tend to confirm the conclusion reached by Barberis (1965)

that positive pressure changes were propagated faster than negative changes. The actual rate of decrease was probably much fas'er than 200psi/hr, since the leakage contributed to it. The rate of unloading was decreased slightly to take account of this. However, since the gearbox on the pressure generator was not infinitely variable, this reduction in pumping rate could only be done in steps and was therefore not very effective.

The calibration was performed in the absence of shear strains. There is some evidence to suggest (Taylor, 1944) that the equalization of non-uniform pore pressures occurs faster when the sample is being sheared, than in the absence of shear strains. On this basis, therefore, the rate of change of the pore pressure of 200 psi/hr which was used in the testing programme would be considered conservative.

4.3 Analysis of results obtained from the multi-stage compression tests.

The equations connecting the axial strain and the load, deviator stress and volumetric strain were assumed to be polynomials of the form

$$y = C_o + C_1x + C_2x^2 + \ldots\ldots C_px^p \quad \ldots\ldots\ldots \quad (4.1)$$

where x denoted the axial strain, which was taken as the independent variable, and y denoted the axial load, deviator stress or volumetric strain. The readings of axial strain and the three dependent variables (axial load, deviator stress and volumetric strain) taken during the periods of constant cell pressure (i.e. portions ab, ef and ij of the curves in fig. 4.2) were fed into an I.B.M 7094 computer which has been programmed to provide

the 'least squares' analysis * of the results, based on a polynomial of the form given in equation (4.1). The values of the polynomial for each dependent variable were computed at intervals of 0.2 °/o axial strain, thus enabling the best-fitting curves for axial load, the deviator stress and volumetric strain to be accurately plotted.

The percentage deviation of each experimental point from the 'least squares' curve was also computed, the percentage deviation being defined as

$$\%\text{ deviation} = \left(\frac{y - y_o}{y}\right) \times 100 \quad \ldots\ldots\ldots \quad (4.2.)$$

where y denotes the adjusted value of the dependent variable, and y_o denotes the value obtained from the experiment.

The data obtained from the comparative drained tests were similarly analysed.

4.4 Presentation of results.

Th 'least squares' curves of axial load, deviator stress, and volumetric strain were drawn against axial strain. The experimental points were plotted on the curves (fig.4.3 to fig.4.14), black dots representing the points taken during periods of constant cell pressure (the points used to determine the shape of the curve) and open circles representing the points taken during the periodsof varying pressures. It was decided to draw the curve of load against axial strain, on account of the uncertainty about the area correction discussed earlier (sect. 3.3.2.1). In addition, graphs showing the variation of cell pressure and percentage deviation with axial strain

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* For a discussion of the principle of least squares and the criterion for the closeness of fit between the experimental data and the curve, see Appendix B.

were also drawn. In general, the percentage deviations of the axial load and deviator stress from the relevant curves were almost identical, and it was decided, for the sake of clarity, to plot an average graph for the two quantities instead of plotting separate graphs.

It is believed that the absolute magnitude of the percentage deviation has no connection with the relative errors in the actual experimental observations. It is only to be considered as a means of comparing the relative departures of the experimental points from the last squares curves.

4.5 Discussion of results:

The aim of the experiment was to study the influence of changing the cell pressure and the pore pressure by equal amounts, on the shear strength and deformation characteristics of calcite and clays. Some of the equations proposed in the literature to define effective stresses controlling shear strength and deformation of soils have been reviewed in Chapter I. Most of the equations relating to effective stresses in clays are of a qualitative nature, since they involve quantities which are not yet measurable. The results obtained from the experiments will therefore be examined in the light of the two main effective stress equations which have been proposed as rigorous expressions defining effective stresses, namely the "intergranular stress" expression and Skempton's (1960) expression.

4.5.1 Effective stress in relation to shear strength.

Rewriting equations (1.2) and (1.4), we have

$$\frac{\sigma'}{(\sigma-u)} = 1 + a\left(\frac{u}{\sigma-u}\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (4.3)$$

and

$$\left(\frac{\sigma'}{\sigma-u}\right) = 1 + \frac{a \tan\Psi}{\tan\phi'}\left(\frac{u}{\sigma-u}\right) \quad \ldots\ldots\ldots \quad (4.4)$$

respectively. For the purposes of argument, some representative values will be assigned to the quantities on the right hand sides of both equations. Skempton (1960) suggested that Ψ for calcite is approximately 8°. For clays, Ψ lies between 5° and 10°. Since Wiener Tegel is a clay with a substantial silt content, it is believed that assuming Ψ to be 8° will not involve much error. From drained tests reported in Section II of this thesis, values of ϕ' for calcite and Wiener Tegel were 38.5° and 23.5° respectively.

As discussed in section (1.2.3), there are divergent views on the magnitude of the contact area ratio, 'a' in soils. However, for the sake of argument a representative value of 'a' will be taken as 3 %, Equation (4.3) then reduces to

$$\left(\frac{\sigma'}{\sigma-u}\right) = 1 + 0.03\left(\frac{u}{\sigma-u}\right) \quad \ldots\ldots\ldots\ldots \quad (4.3a)$$

and equation (4.4) reduces to

$$\left(\frac{\sigma'}{\sigma-u}\right) = 1 + 0.0066\left(\frac{u}{\sigma-u}\right) \text{ for clays } \ldots\ldots \quad (4.4a)$$

or $$\left(\frac{\sigma'}{\sigma-u}\right) = 1 + 0.0102\left(\frac{u}{\sigma-u}\right) \text{ for calcite } \ldots\ldots (4.4b)$$

During the series of tests in which $(\sigma - u)$ was maintained constant at 10 psi, the ratio $\left(\frac{u}{\sigma-u}\right)$ varied

from 4 to 99. If the effective stress were given by the intergranular stress, therefore, the percentage error which would occur in the deviator stress when the cell pressure and the pore pressure were varied between the minimum and the maximum values would be 0.03x95x100 °/o or 285 °/o from equation (4.3a), and if Skempton's expression were valid, the resulting percentage errors would be 63 °/o for tests on calcite and 97 °/o for tests on Wiener Tegel.

Similarly, from the second series of tests (σ_3-u = 64.5)/in which the ratio $(\frac{u}{\sigma-u})$ varied between 0.5 and 14.5, the resulting changes in the deviator stress would be 42 °/o from equation (4.3a) for calcite and clay, and about 18°/o for calcite and 19°/o for clay from equations (4.4a) and (4.4b).

While the foregoing analysis may be open to criticism on certain grounds*, it does at least show that, if these two expressions rather than the Terzaghi expression were valid, one would expect to find distinct discontinuities in the deviator stress-strain curves, since the resulting errors estimated above are generally higher than those normally associated with triaxial tests, namely about 5°/o.

An examination of the deviator stress vs. axial strain curves (fig. 4.3 to fig.4.14) will show that the departures of the points obtained during the periods of changing pressures (open circles) were essentially no

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* For example in the case of the clays, if it is accepted that inter-particle contact is between adsorbed water layers (section 1.3.2), then Ψ should strictly relate to this layer rather than the actual soil particles.

worse than those of points obtained under steady pressures or those of points taken in the comparative tests (Test nos. 13 to 16). In some tests (test numbers 1,3,4,5,7,8,11 and 12) the points taken during the period of changing pressures, plotted above the 'least squares' curves, but in the other tests, they plotted on either side of the curves. In general, the closeness of the points to the curves depended on the validity of the assumption that the stress-strain and the load-strain curves could be represented by polynomials of the form given in equation (4.1). In test numbers 2 and 5 in which the original data yielded curves without sharp changes of slope, the polynomial assumption was probably justified, and a better agreement was obtained between the curve and the points taken under varyinv pressures. The assumption involves some error when applied to results on overconsolidated soils (such as test numbers 4, 6, 9 and 12) which yield stress-strain curves with sharp changes of slope. This probably explains the bigger discrepancy between the points and the curves, obtained in these tests.

Even allowing for these errors, it can still be seen from the graphs that changing the cell pressure and the pore pressure simultaneously by large and equal amounts had no significant influence on the shapes of the resulting stress-strain curves. The ratio $\left(\frac{\sigma'}{\sigma-u}\right)$ therefore remained sensibly constant throughout the test. This means that, all other factors remaining constant, the shear strengths of calcite and clays are controlled, to a high degree of accuracy, by the effective stress as defined by Terzaghi, namely that

$$\sigma' = \sigma - u \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.5)$$

4.5.2 Effective stress in relation to volume changes.

The readings of volume change of the sample were corrected for the compressibility of the soil grains, and analysed in the way described previously (section 4.3). The data obtained during the period of varying pressures plotted either on or close to the curves, and as was the case with the stress-strain and load-strain curves, there seemed to be no particular pattern of deviation of the points from the curve.

It will appear from these results, therefore, that the expression for the effective stress controlling changes in the volume of clays and calcite consequent upon changes in applied stress is of the form which was proposed by Skempton (1960) and which took account of the finite compressibility of the soil grains, namely

$$\Delta\sigma' = \Delta\sigma - \left(1 - \frac{C_s}{C}\right)\Delta u \quad \ldots\ldots\ldots\ldots \quad (4.5)$$

4.6 Summary and Conclusions:

The main sources of error in the triaxial test were discussed. It was concluded that, given the comparative nature of the experiments, the only source of error likely to have any appreciable influence on the results was that derived from the non-equalization of applied pore pressures, and ways of dealing with the problem have been described.

The method of analysing the results has been described and the results obtained were discussed. The results obtained indicated that the effective stresses controlling the shear strength and changes in volume of clays and calcite are given by

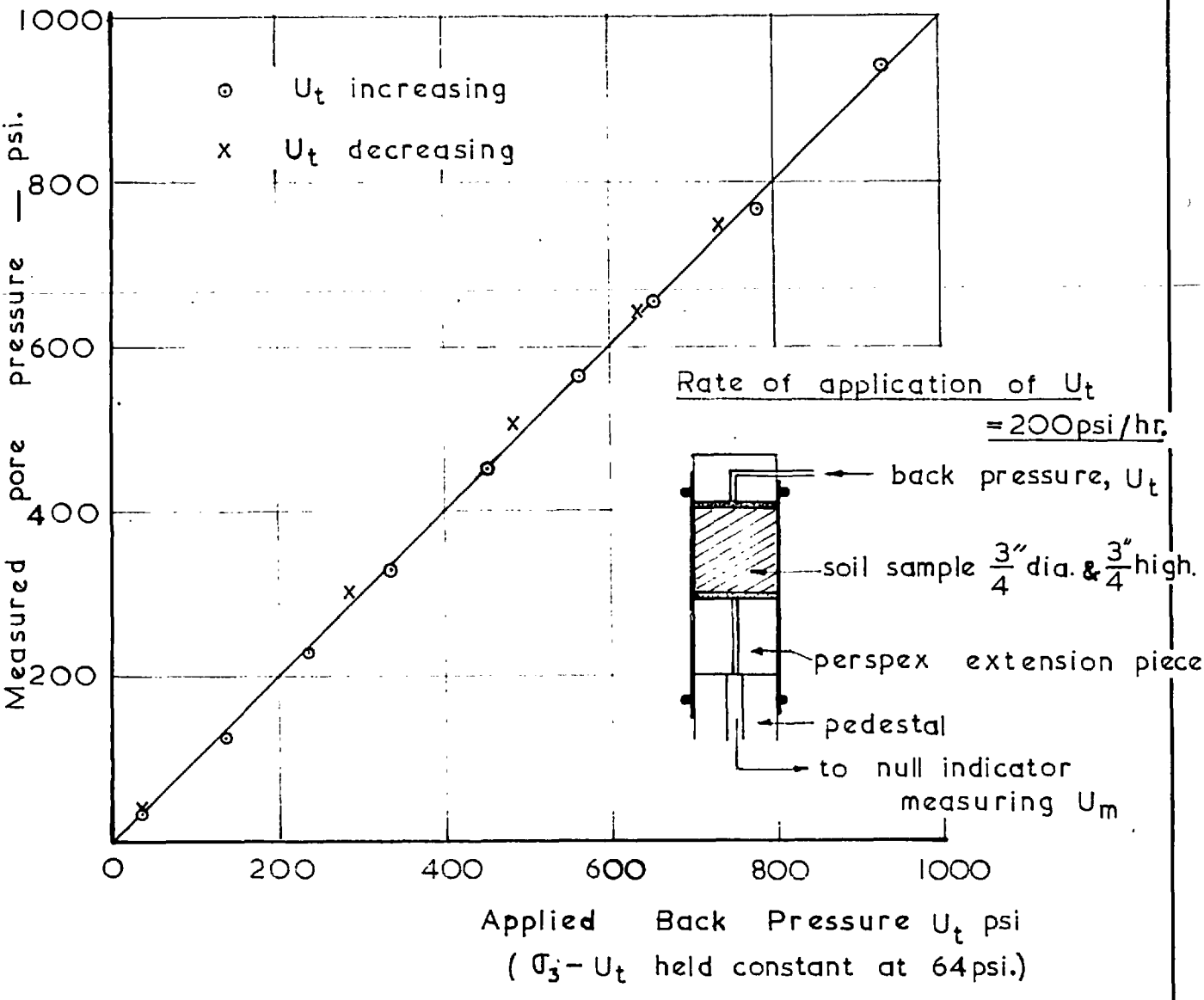
$$\sigma' = \sigma - u$$

for shear strength,

and $\Delta\sigma' = \Delta\sigma - (1 - \frac{C_s}{C})\Delta u$

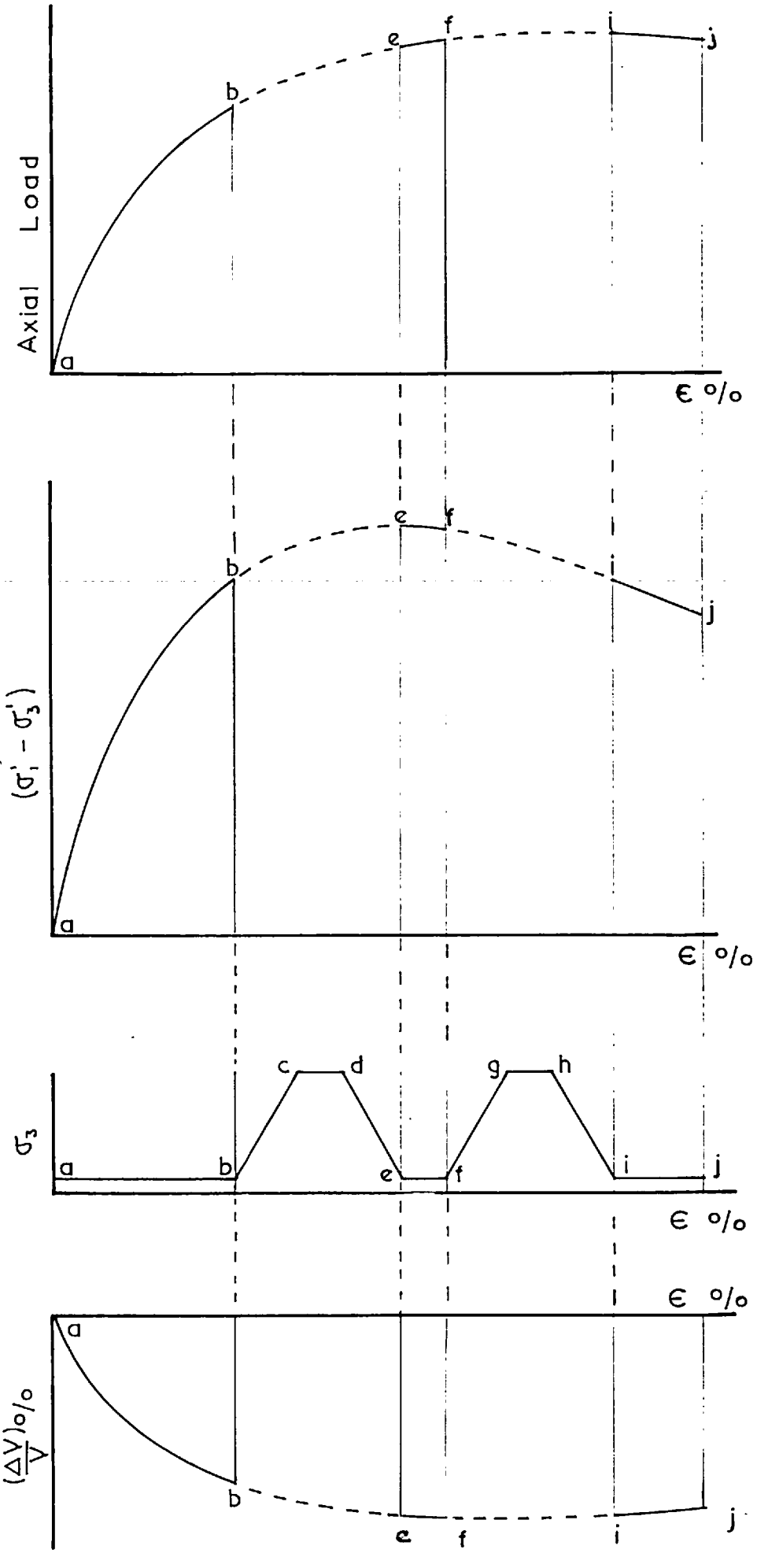
for changes in volume. The influence of $(\frac{C_s}{C})$ was however not important, and its neglect in normal Soil Mechanics calculations was, therefore, not considered serious.

For clays, the influence of such factors as the chemistry of the pore fluid had not been examined.



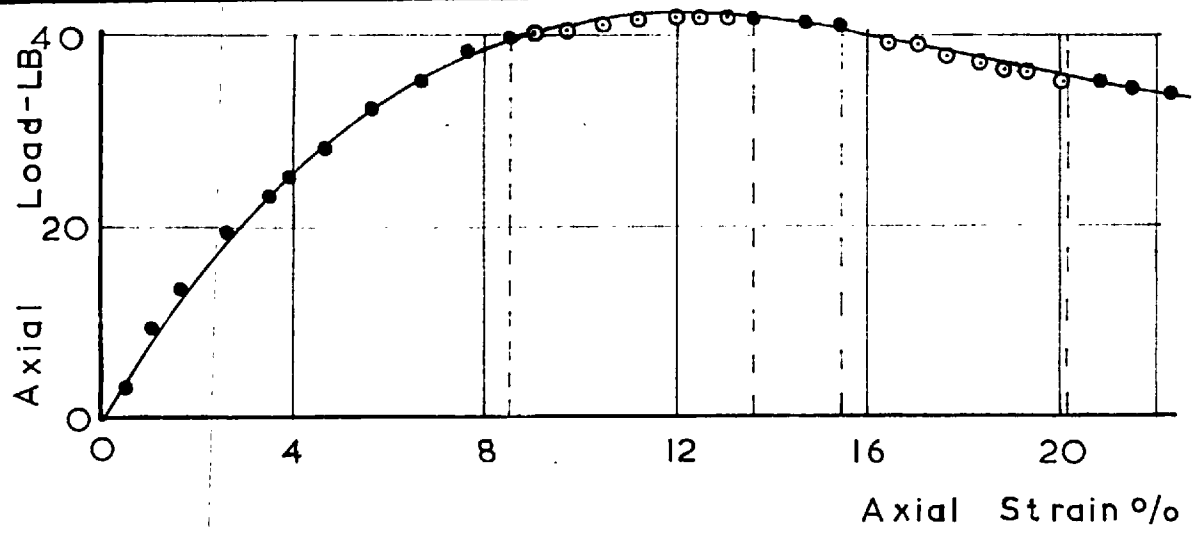
CALIBRATION TO DETERMINE THE MAXIMUM RATE OF INCREASE OF BACK PRESSURE TO ENSURE NEGLIGIBLE EXCESS PORE PRESSURES.

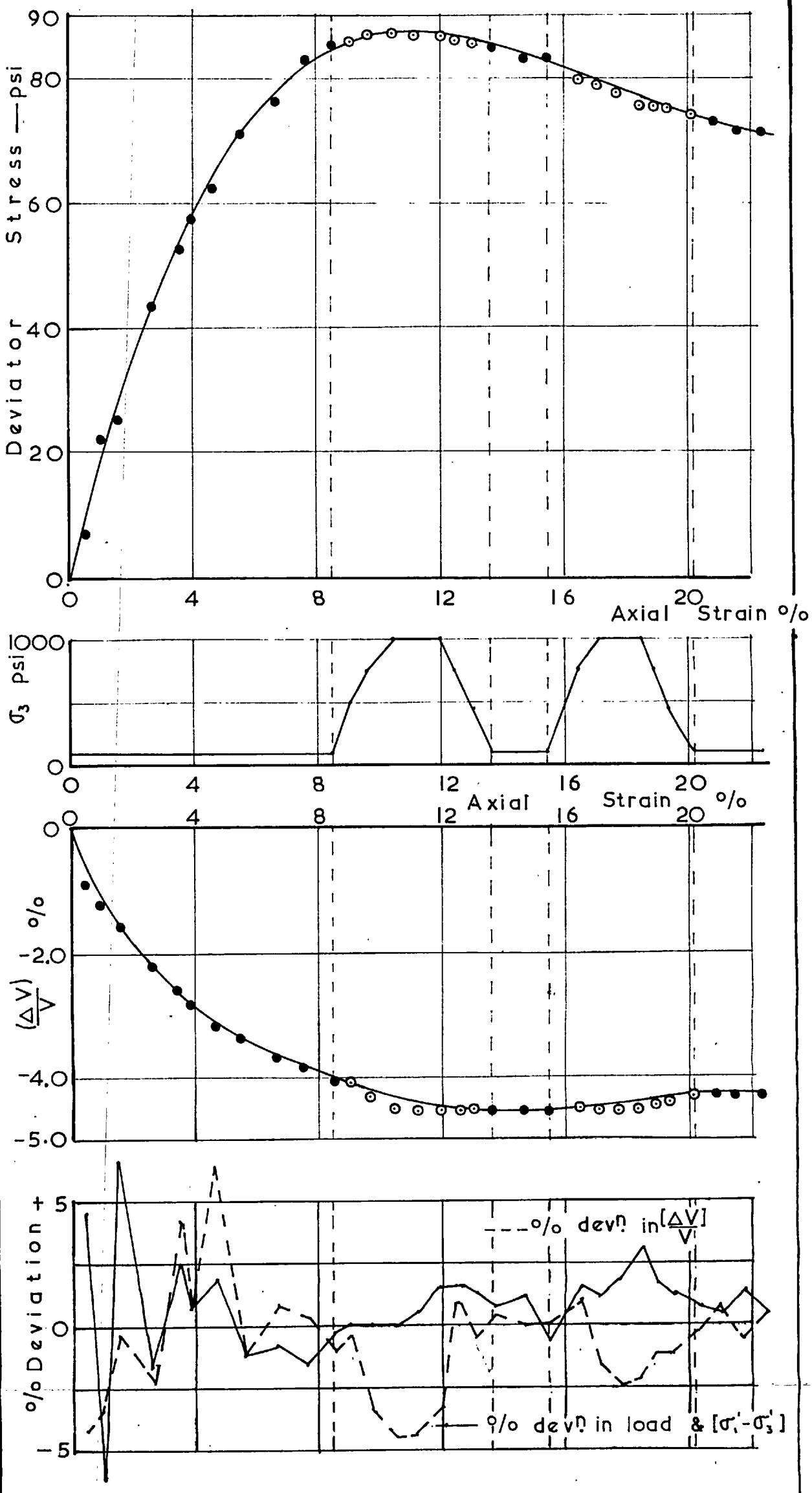
FIG. 4.1



METHOD OF ANALYSING TEST RESULTS.

FIG.4.2



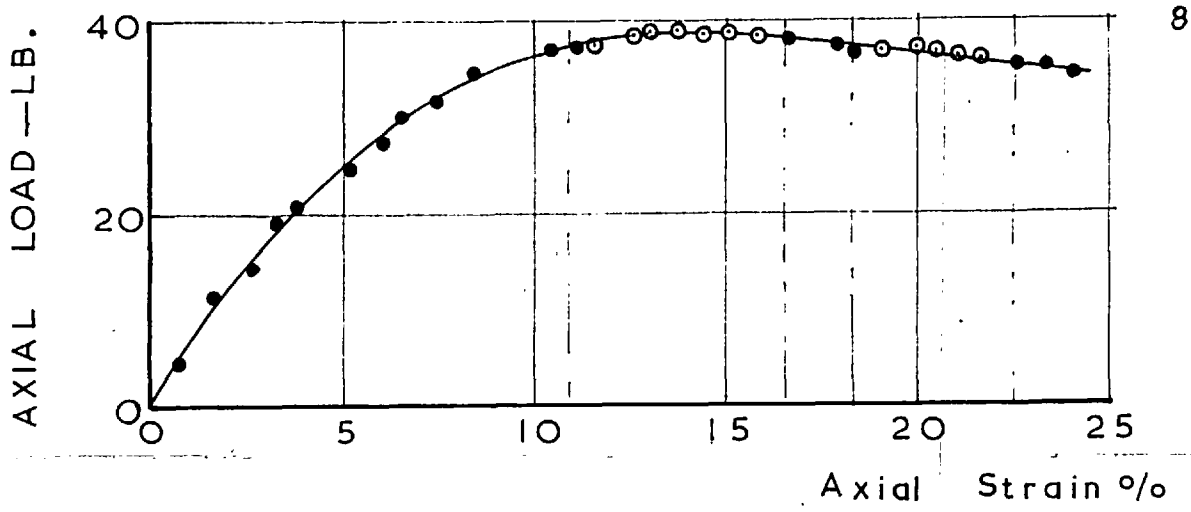


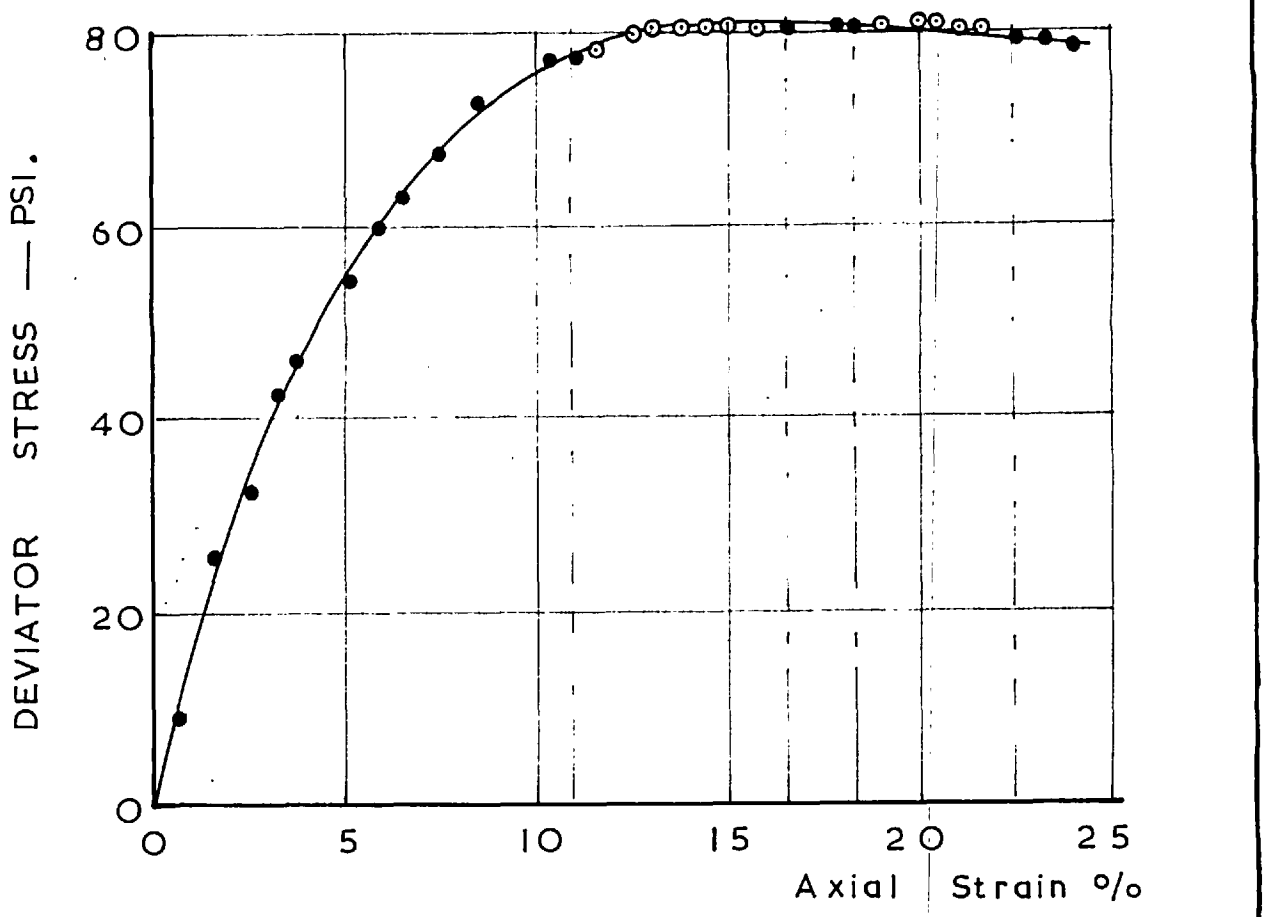
TEST I

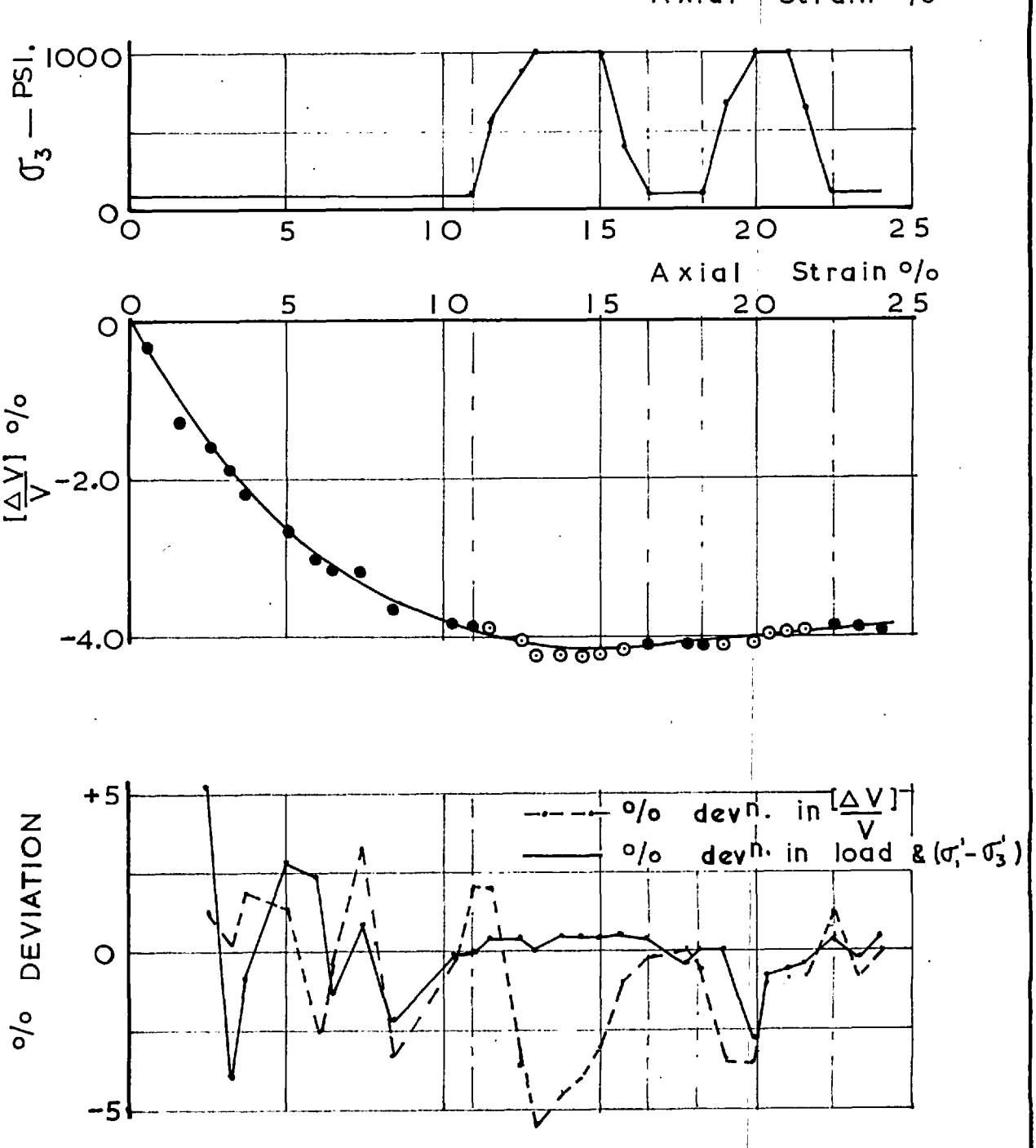
$(\sigma_3 - U)$ maintained constant at 64 5 psi

MULTI-STAGE TEST ON WIENER TEGEL.

FIG. 4.3



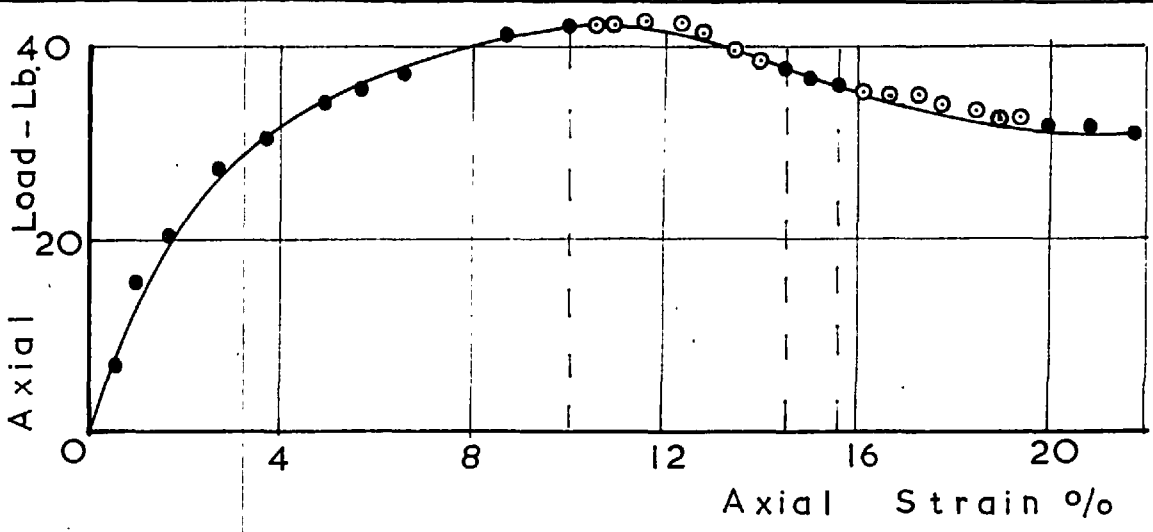


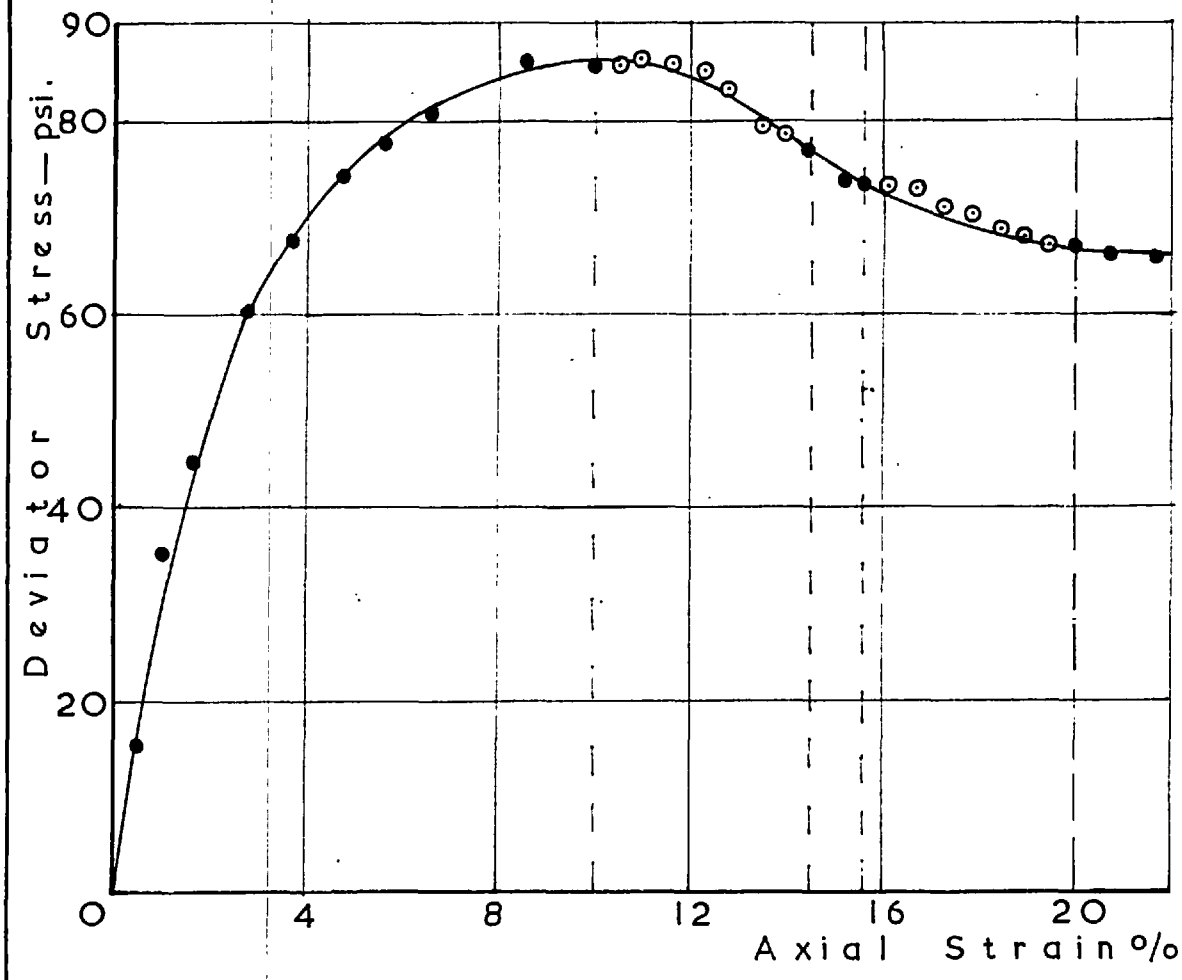


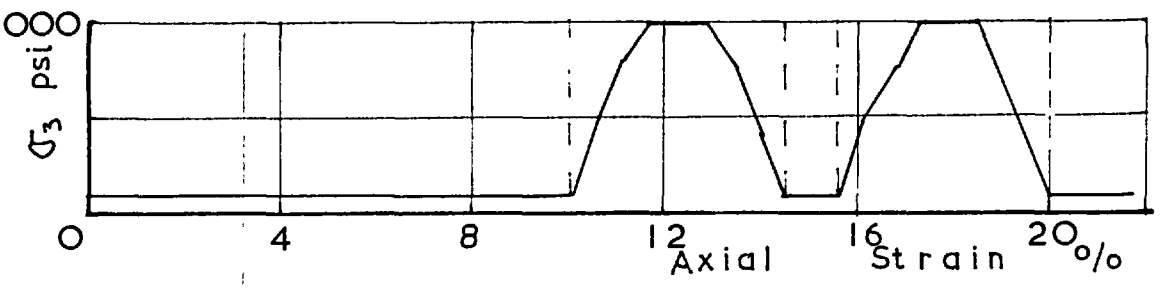
TEST 2. (σ_3-U) maintained constant at 64.5 psi

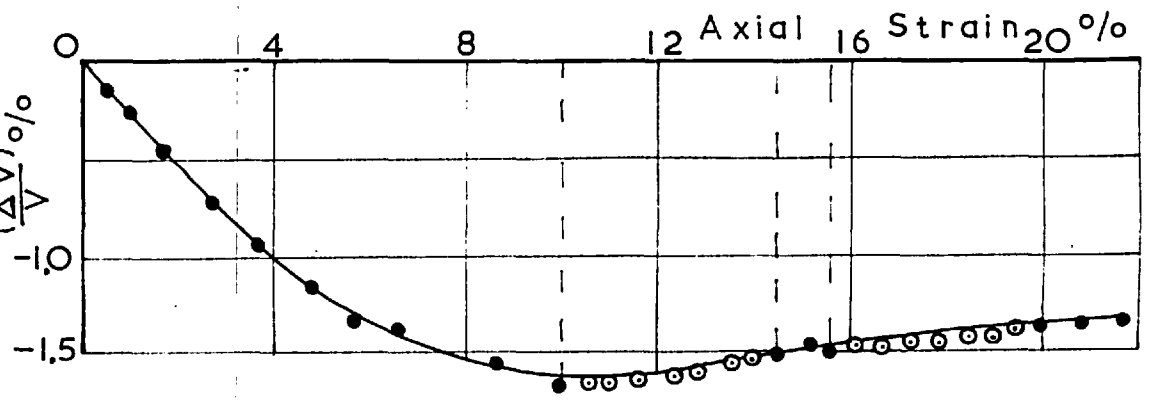
MULTI-STAGE TEST ON WIENER TEGEL.

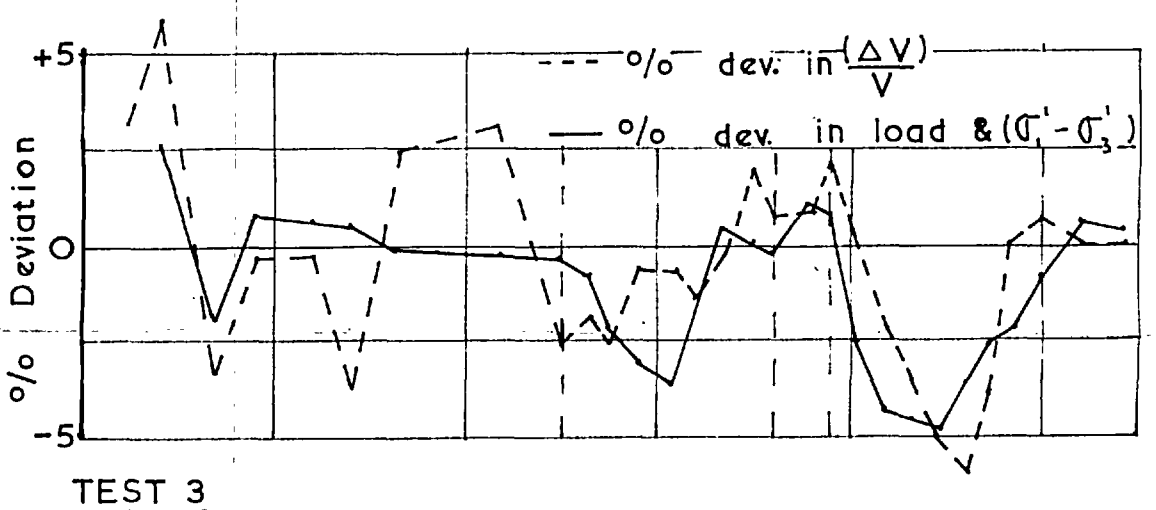
FIG. 4.4









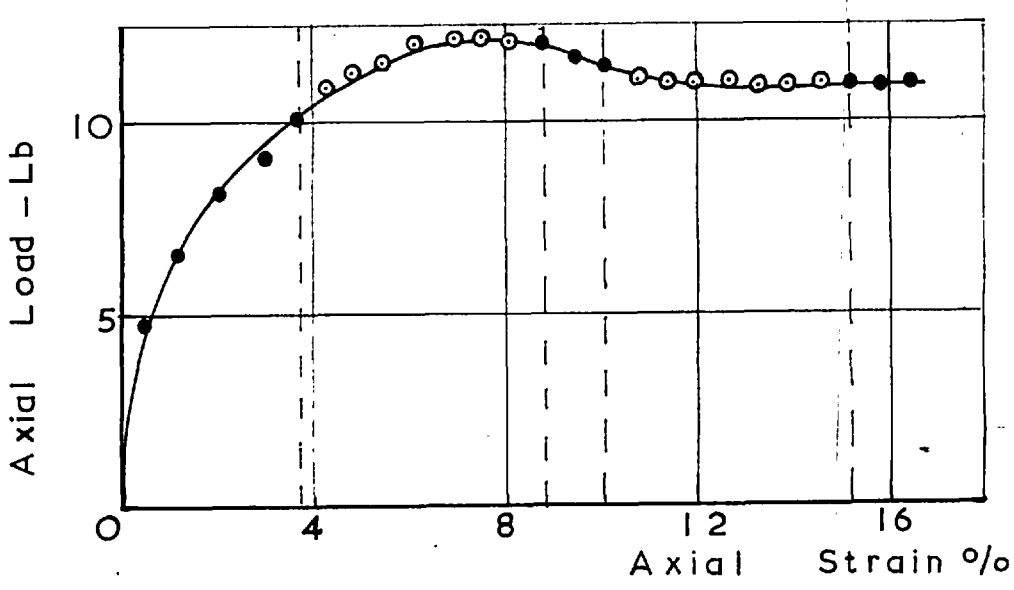


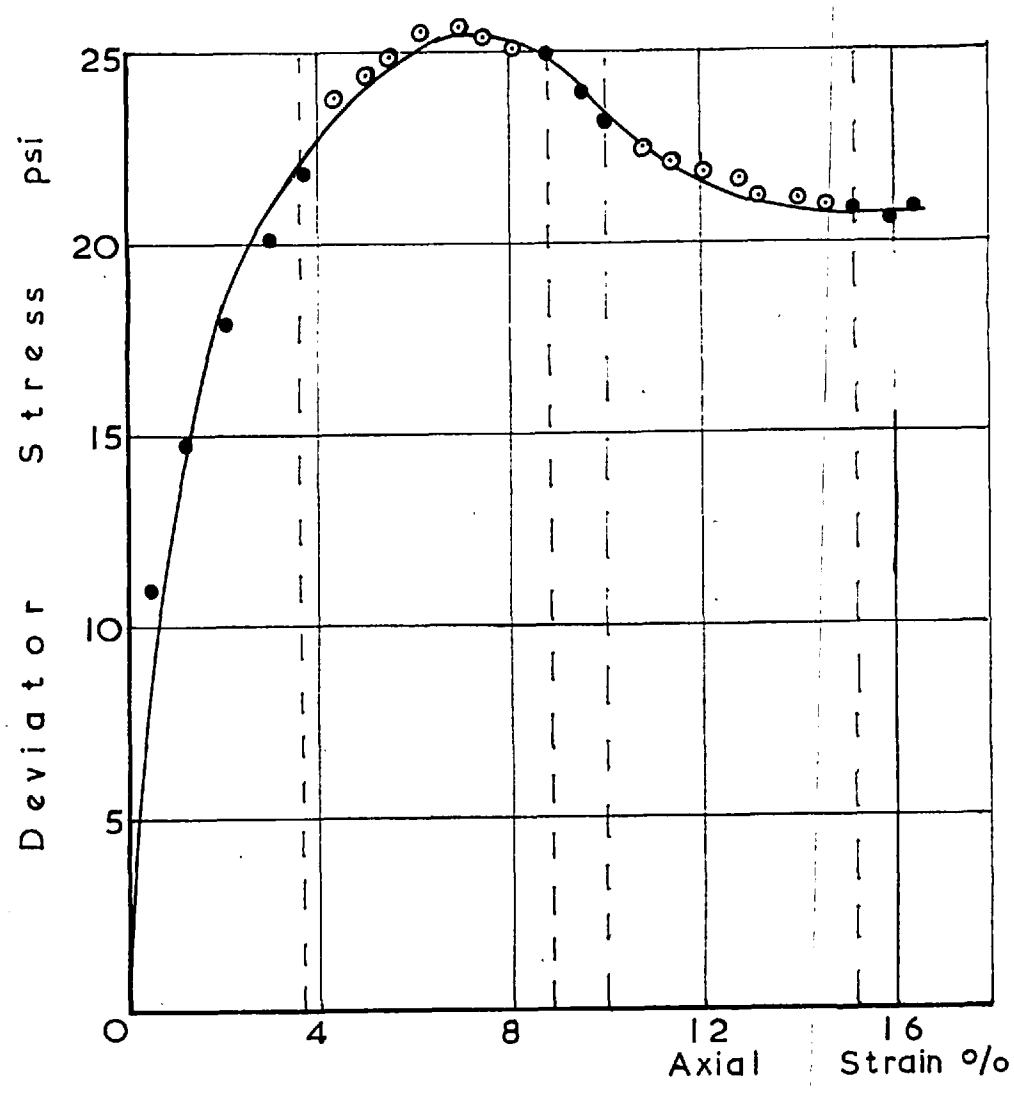
TEST 3

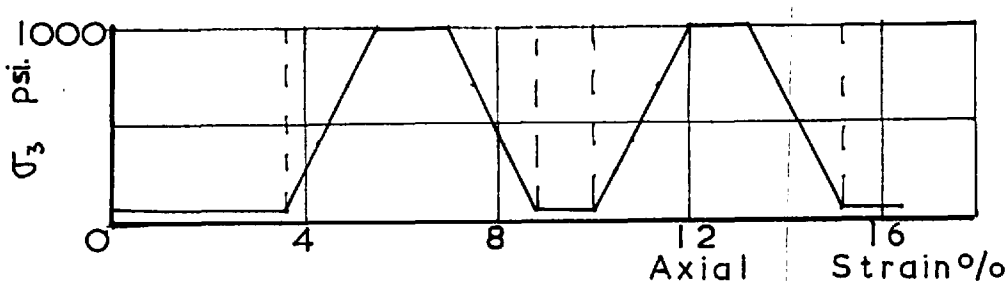
$(\sigma_3 - U)$ maintained constant at 645 psi.

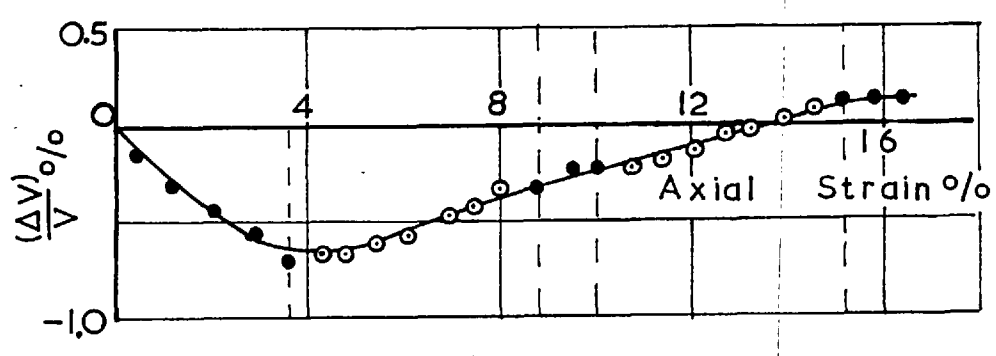
MULTI-STAGE TEST ON WIENER TEGEL.

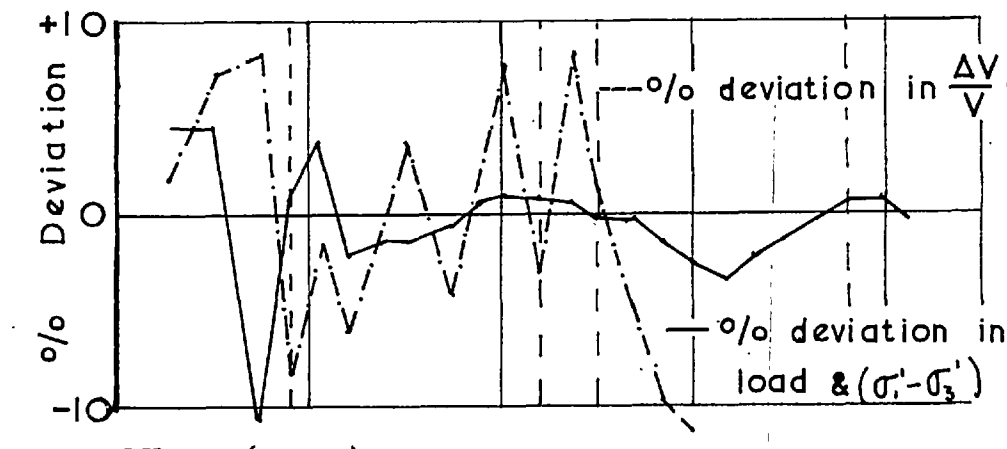
FIG. 4.5









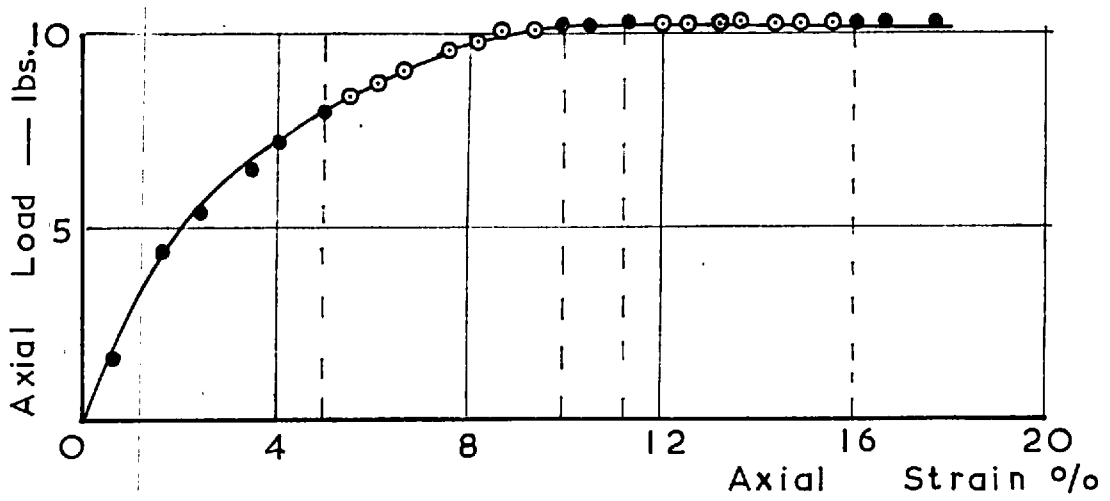


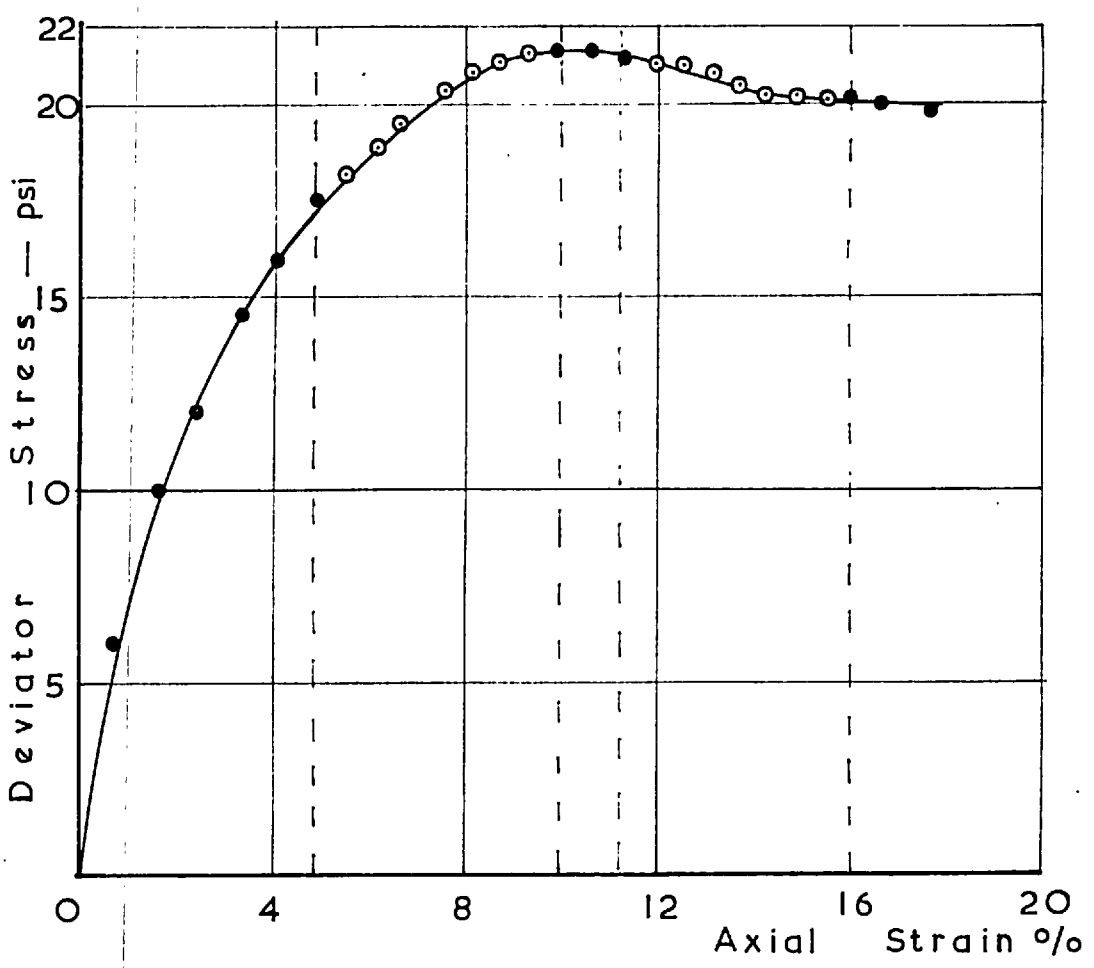
TEST 4 $(\sigma_3 - U)$ maintained constant at 10 psi.

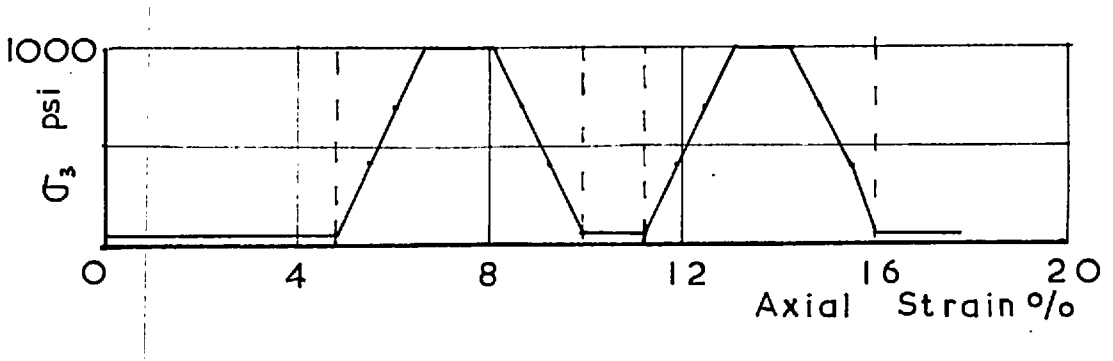
O.C.R. = 9.3

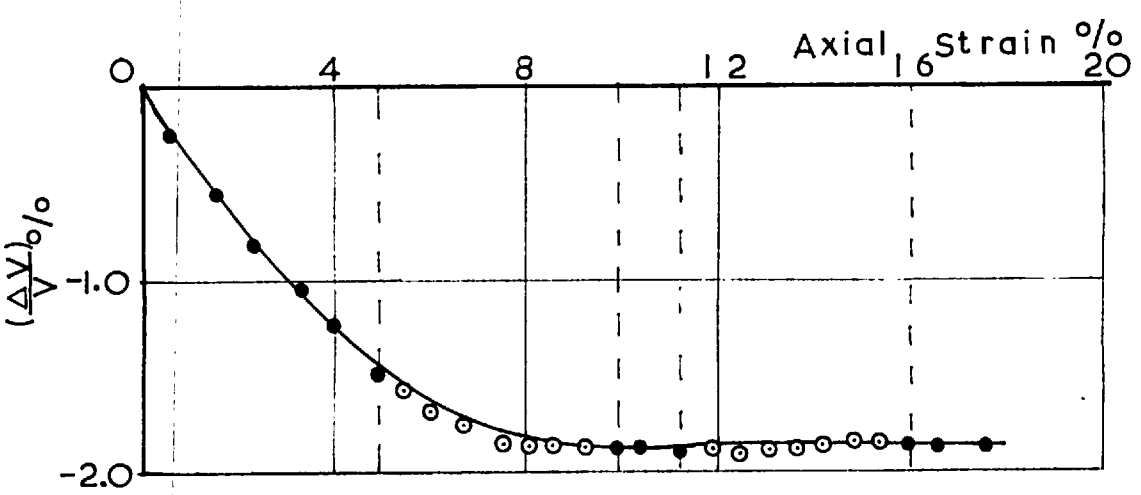
MULTI-STAGE TEST ON WIENER TEGEL.

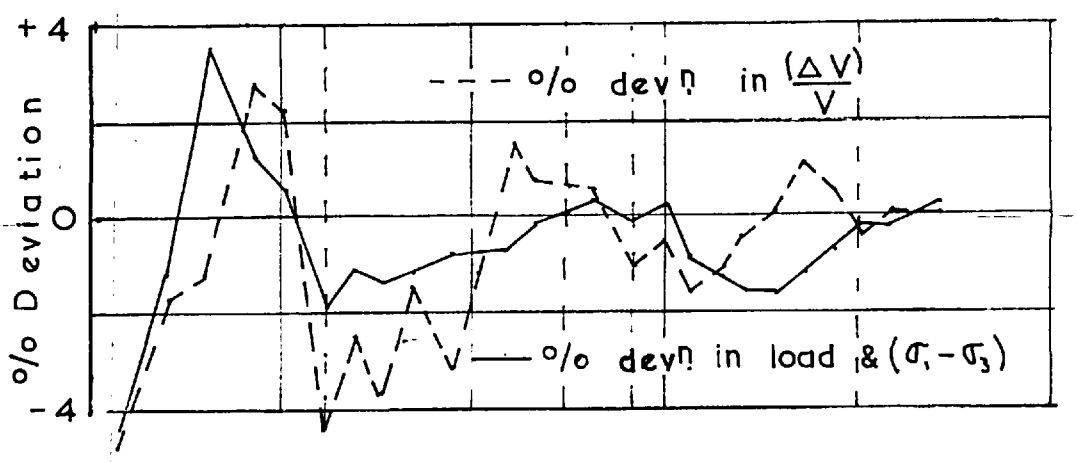
FIG. 4.6







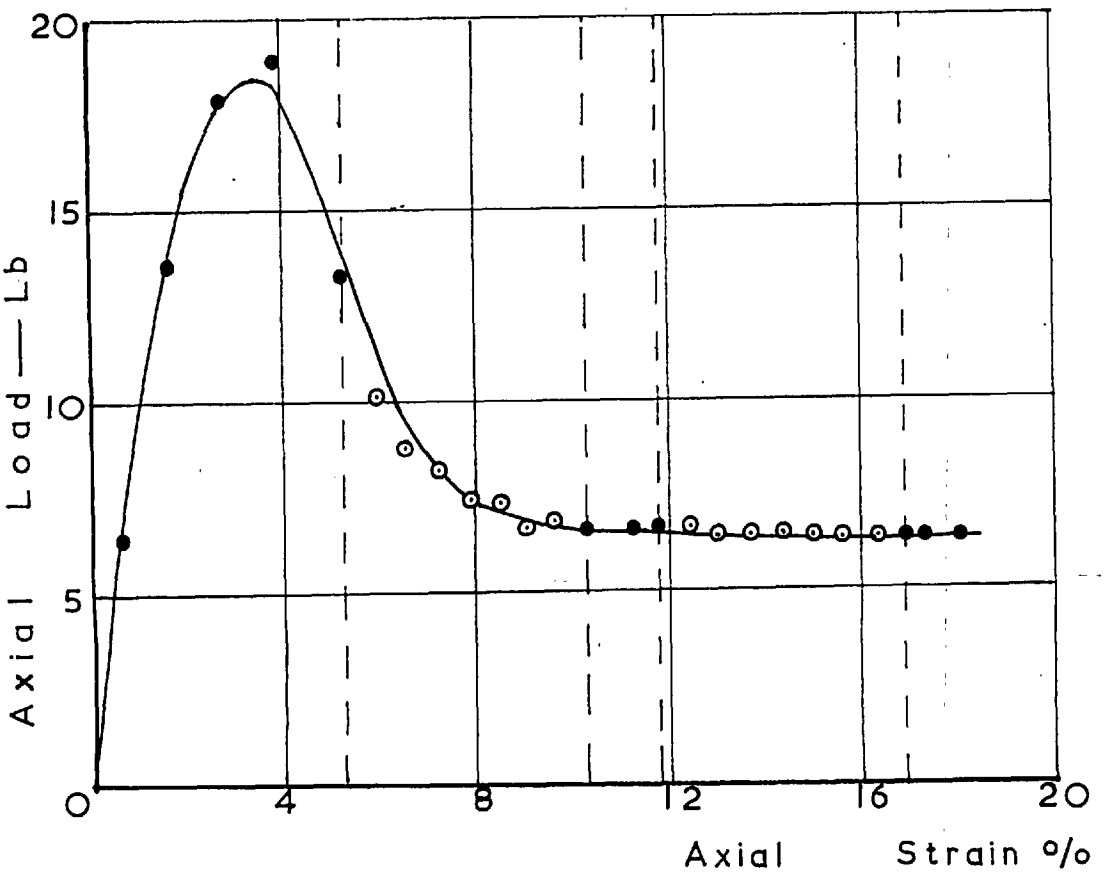


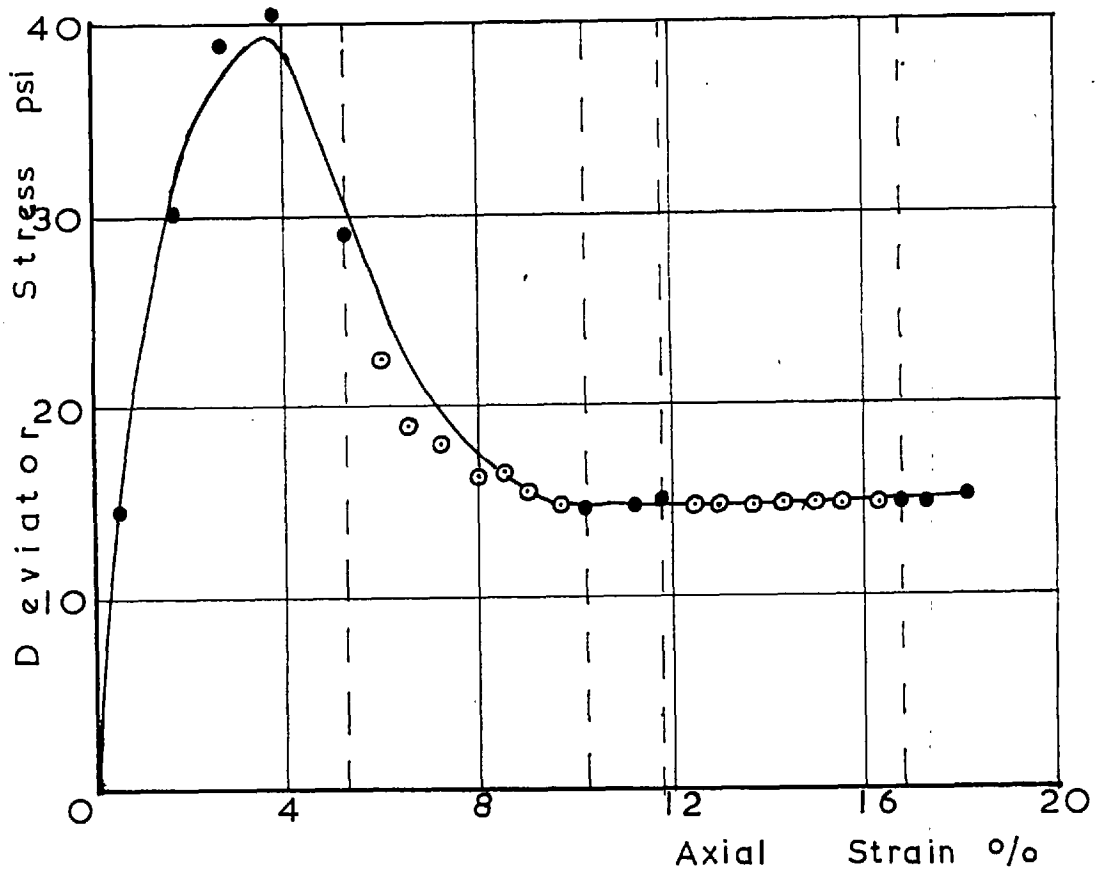


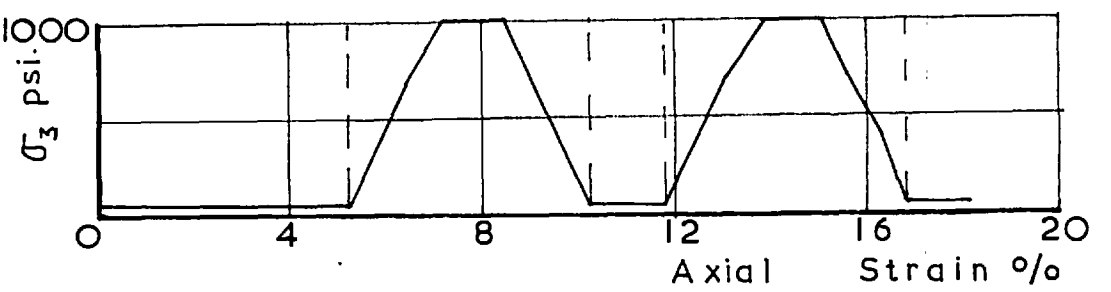
TEST 5 — $(\sigma_3 - U)$ maintained constant at 10 psi

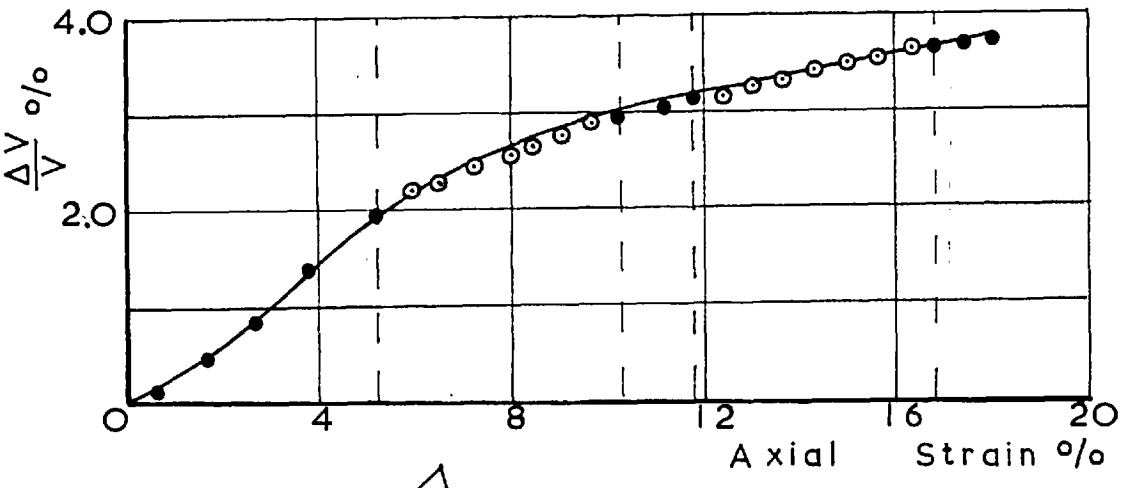
MULTI-STAGE TEST ON WIENER TEGEL.

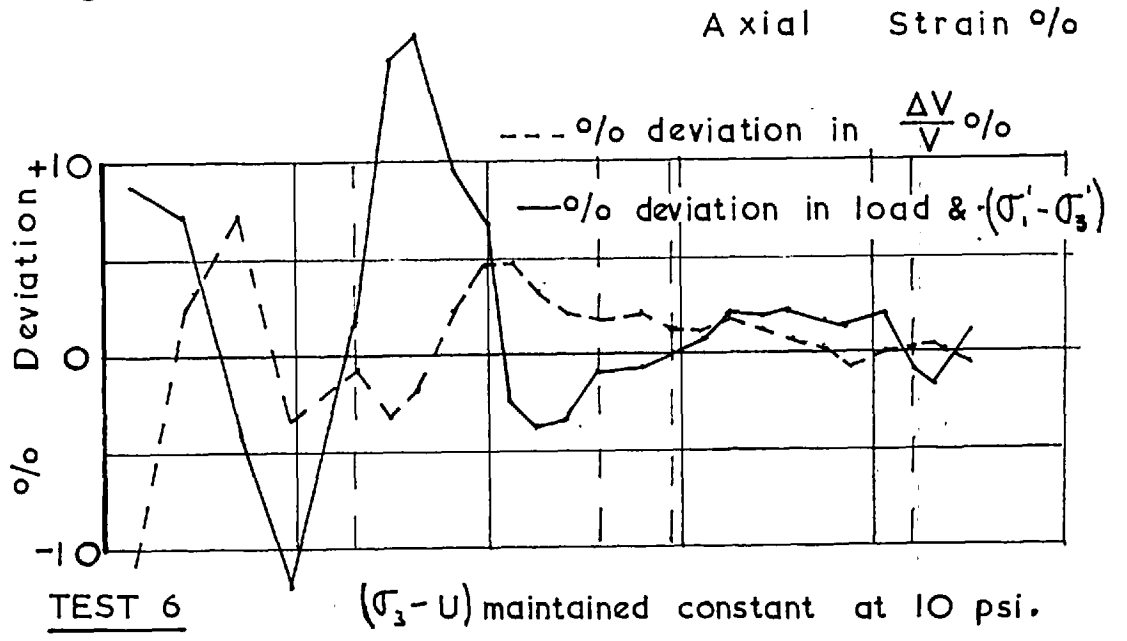
FIG. 4.7









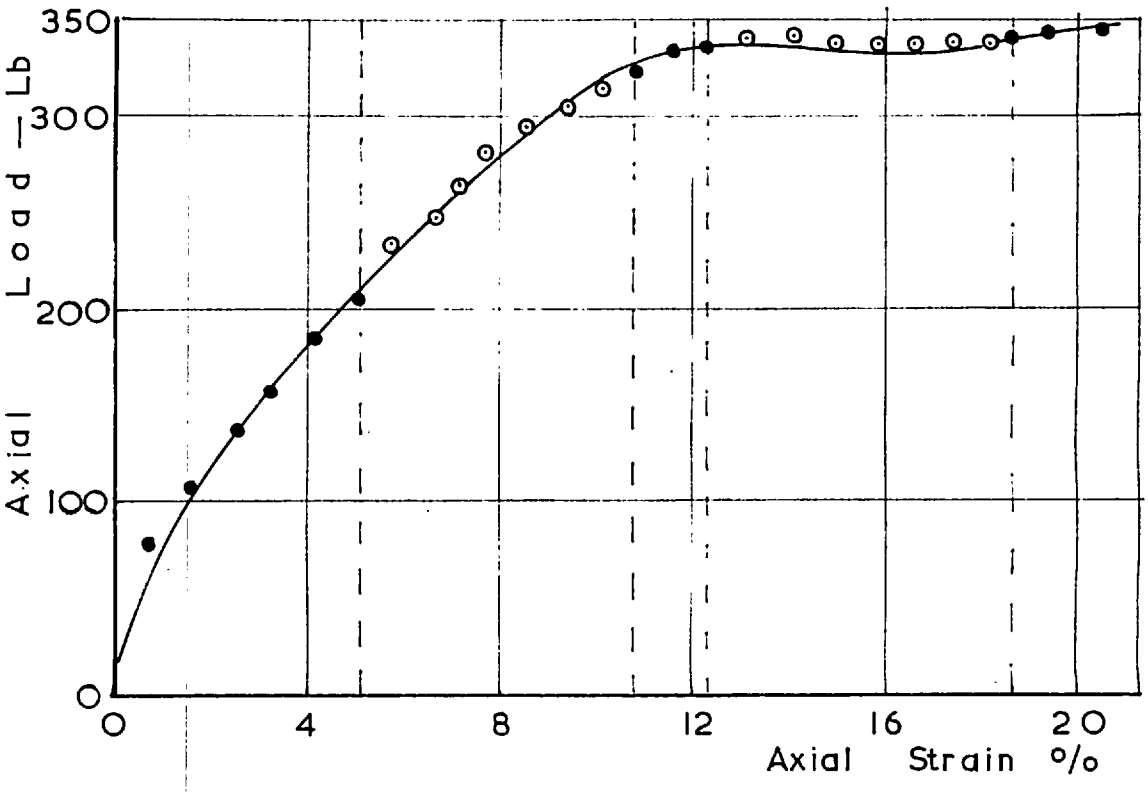


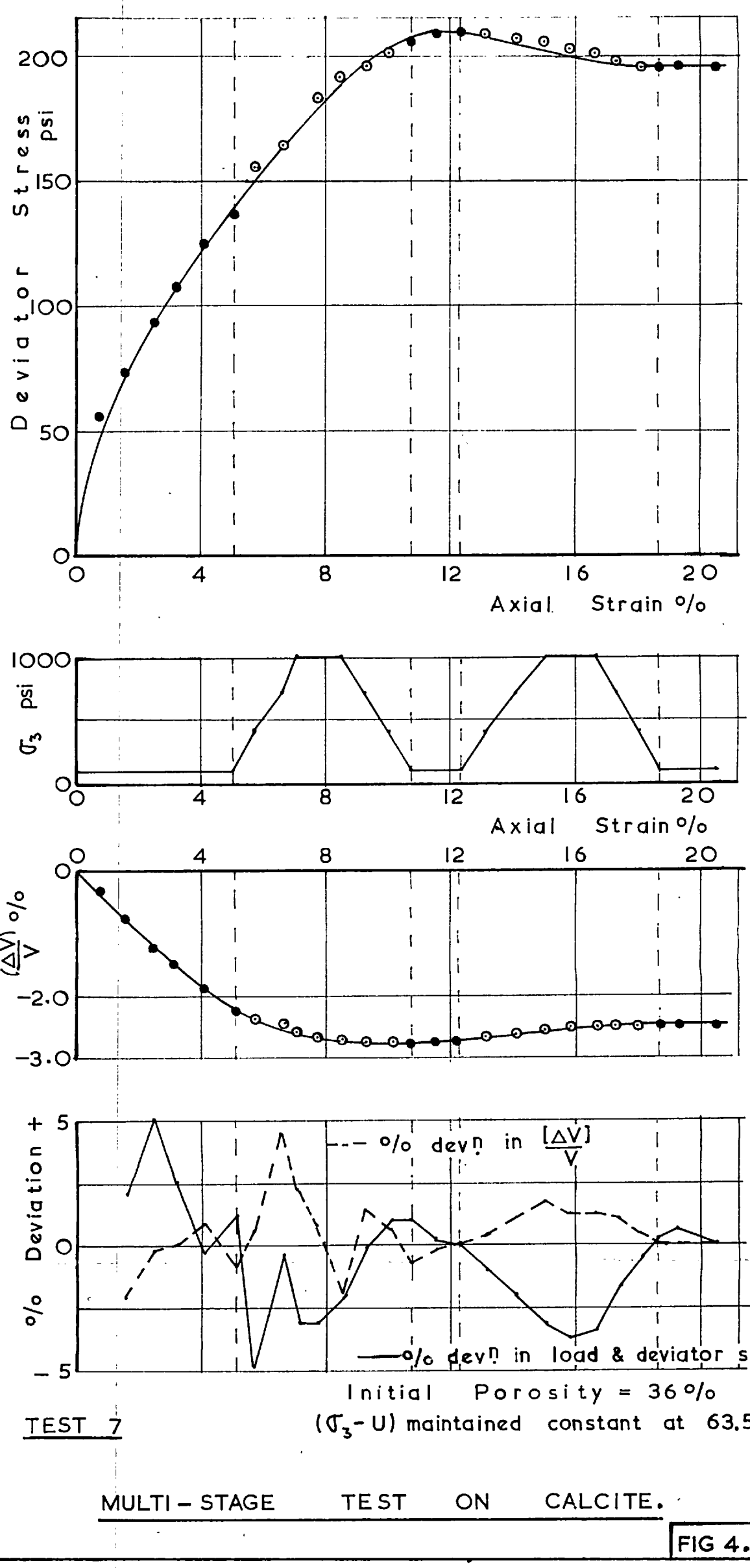
TEST 6 $(\sigma_3 - U)$ maintained constant at 10 psi.

The sample was initially heavily over-consolidated. O.C.R.= 50

MULTI-STAGE TEST ON WIENER TEGEL.

FIG. 4.8





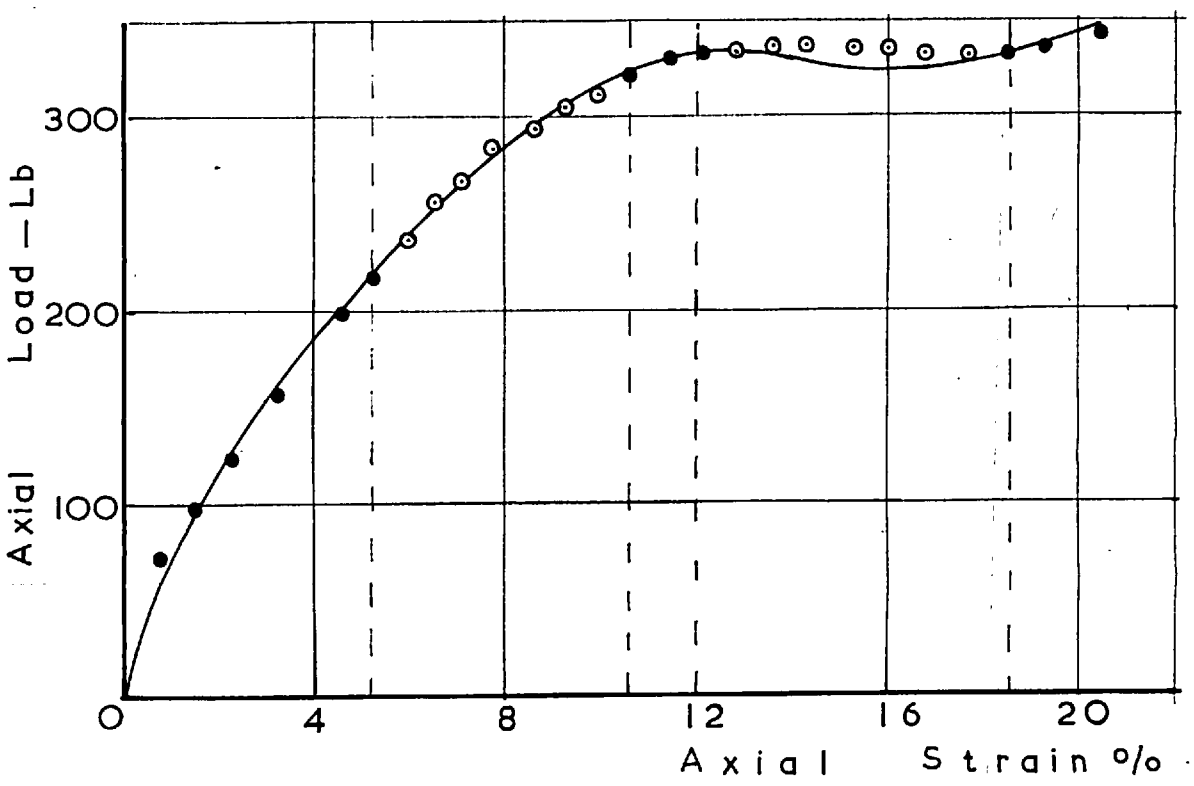
Initial Porosity = 36 °/o

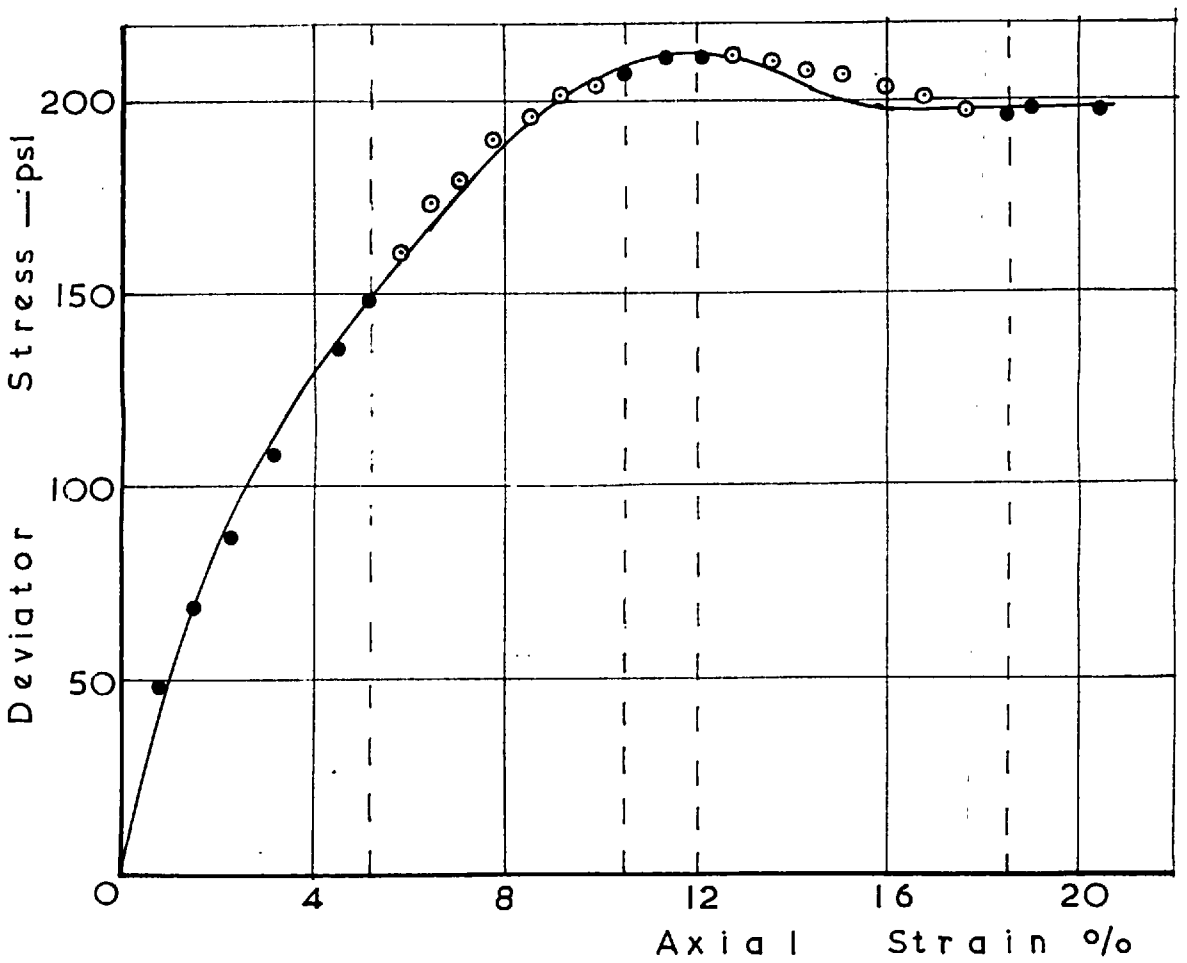
TEST 7

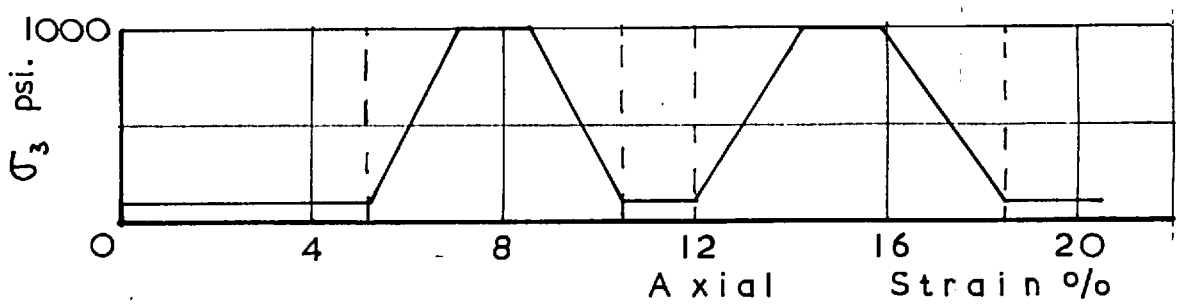
$(\sigma_3 - U)$ maintained constant at 63.5 psi.

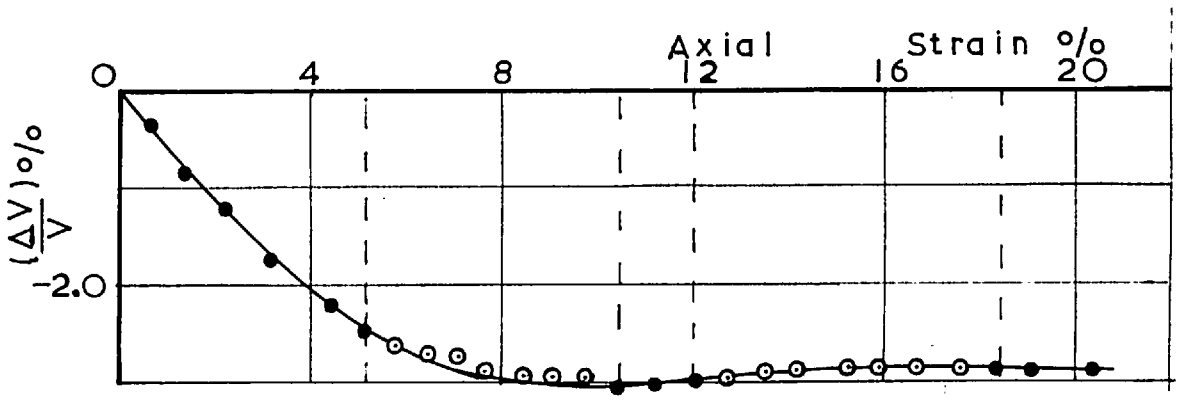
MULTI – STAGE TEST ON CALCITE.

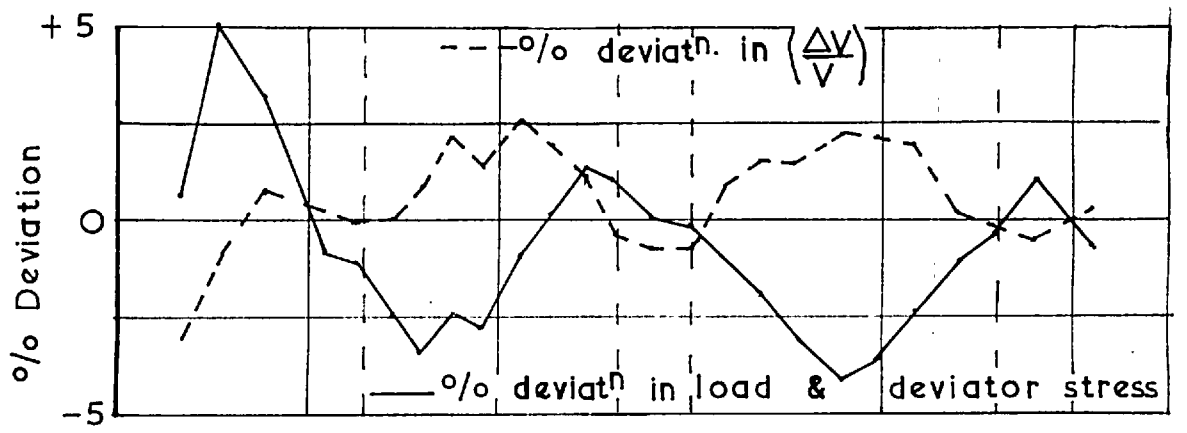
FIG 4.9







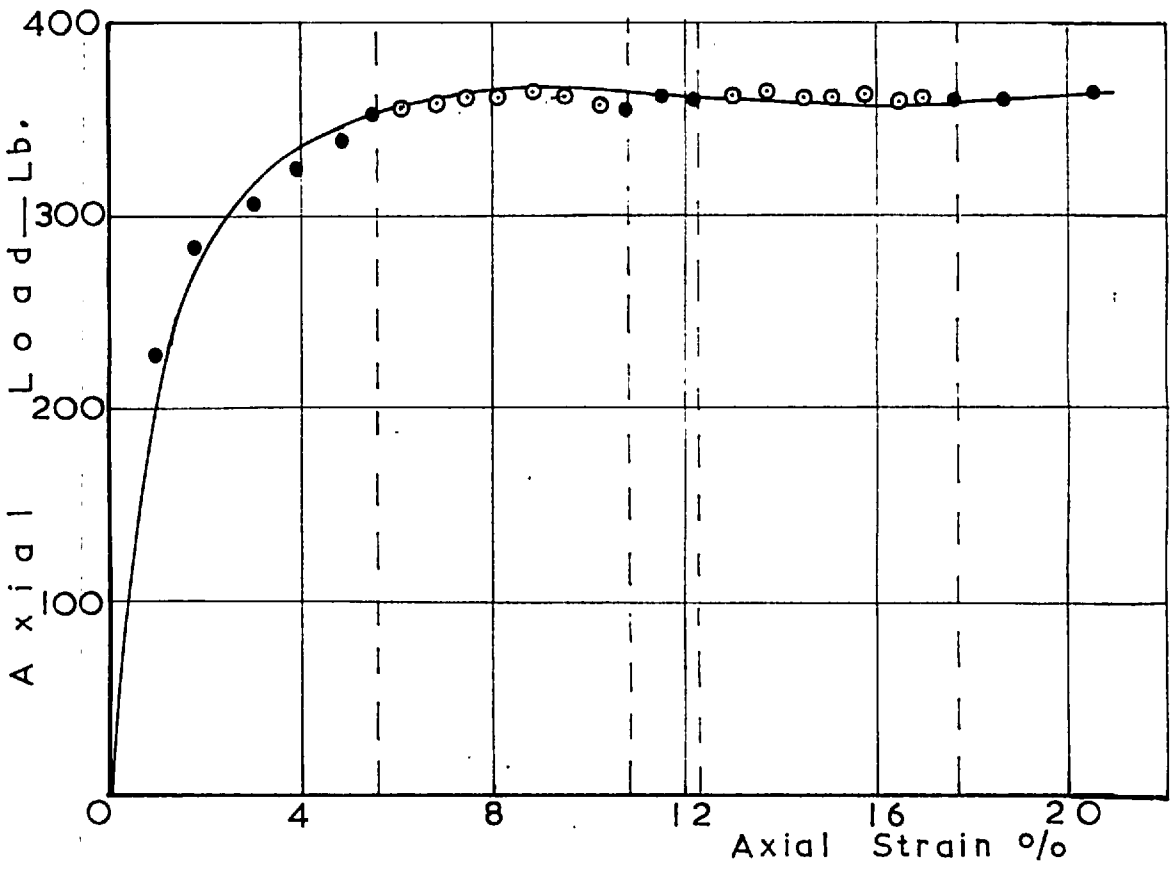


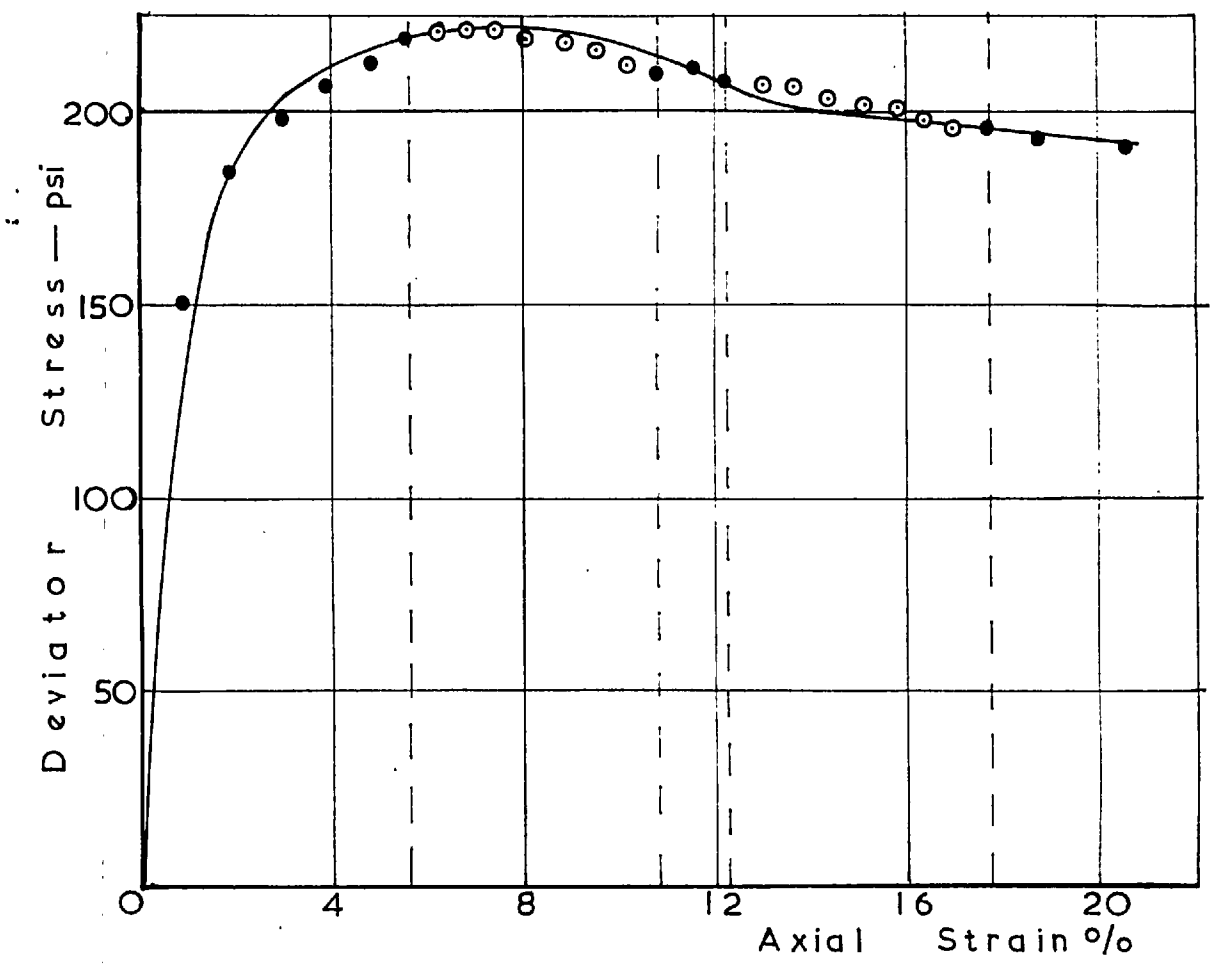


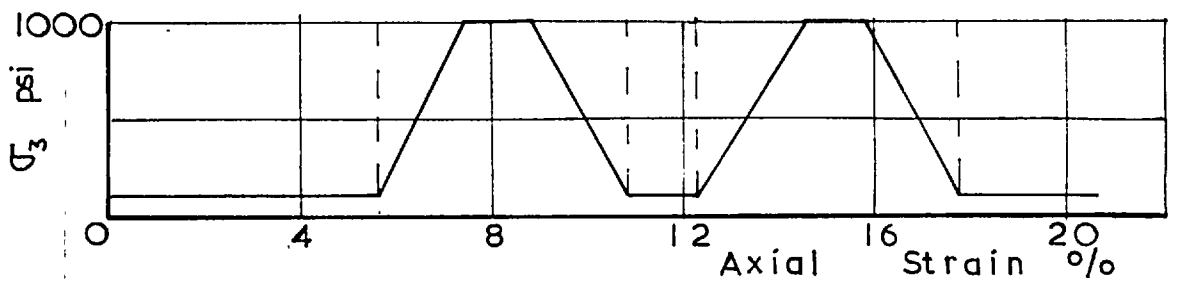
TEST 8 — $(\sigma_3 - U)$ maintained constant at 63.5 psi.
Initial porosity of sample = 36.5 %

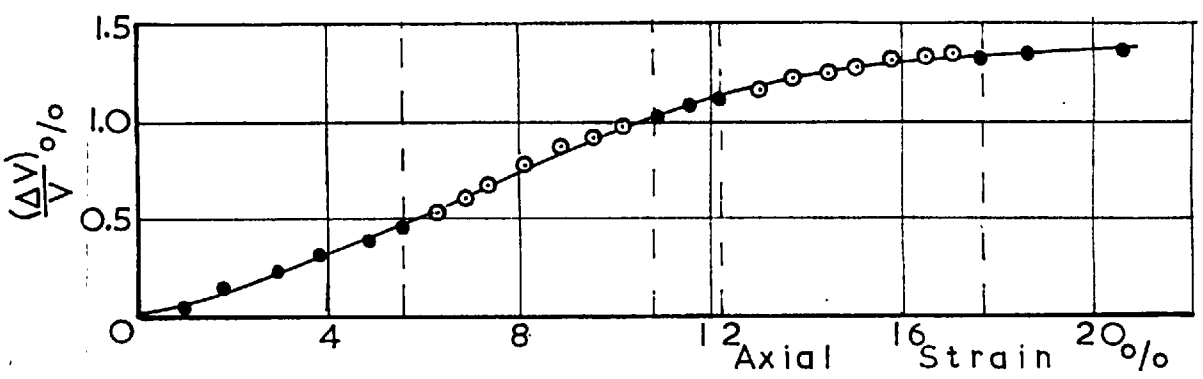
MULTI-STAGE TEST ON CALCITE.

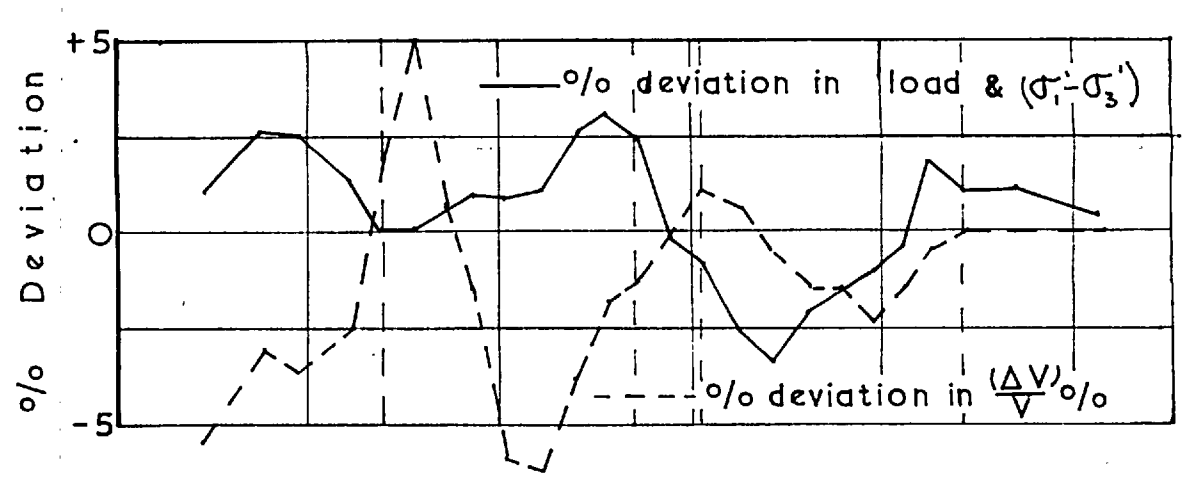
FIG. 4.10









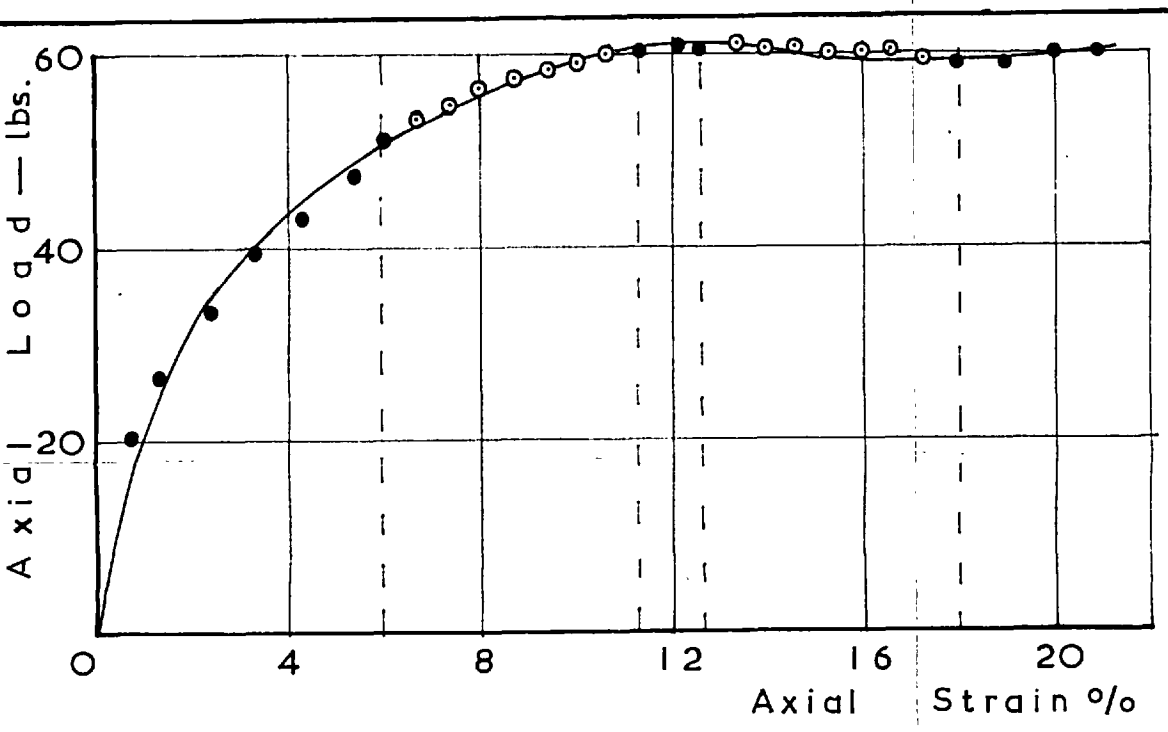


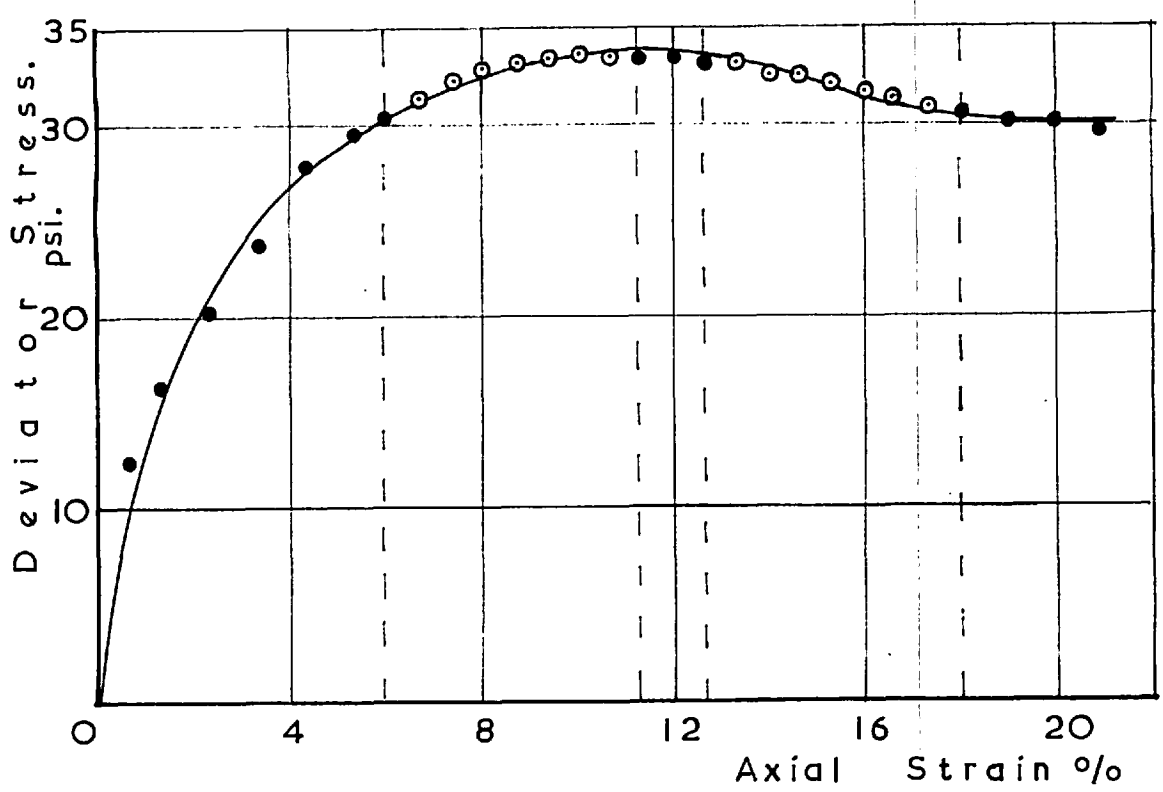
TEST 9 $(\sigma_3 - U)$ maintained constant at 63.5 psi

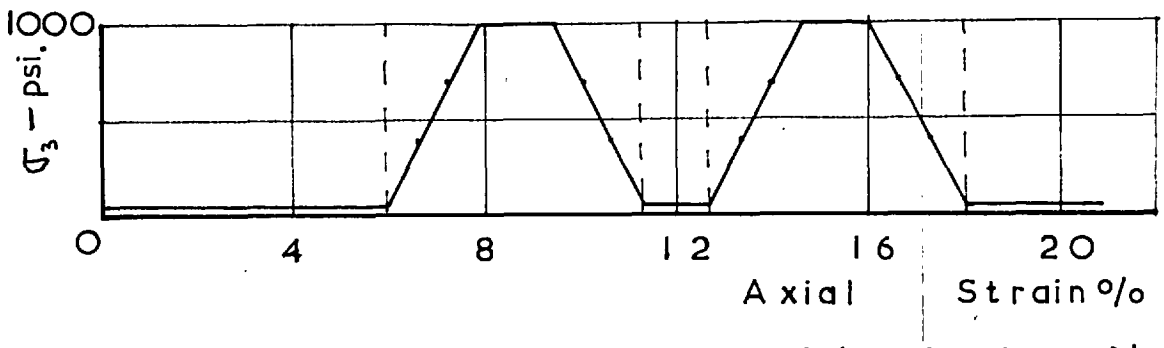
The sample was initially over-consolidated. O.C.R.=6.3

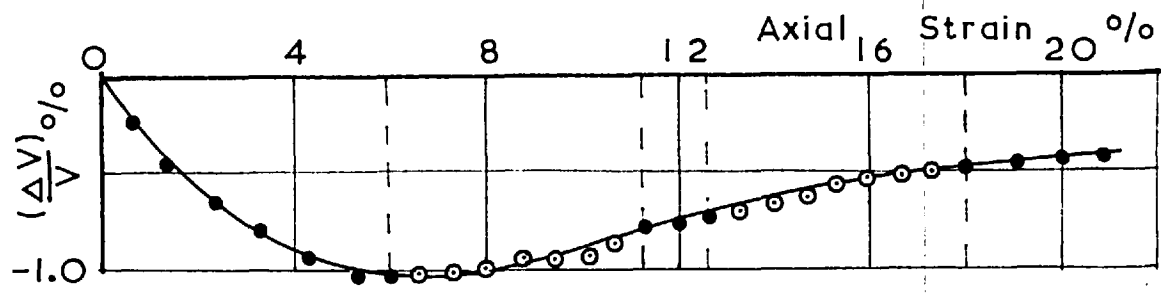
MULTI-STAGE TEST ON CALCITE.

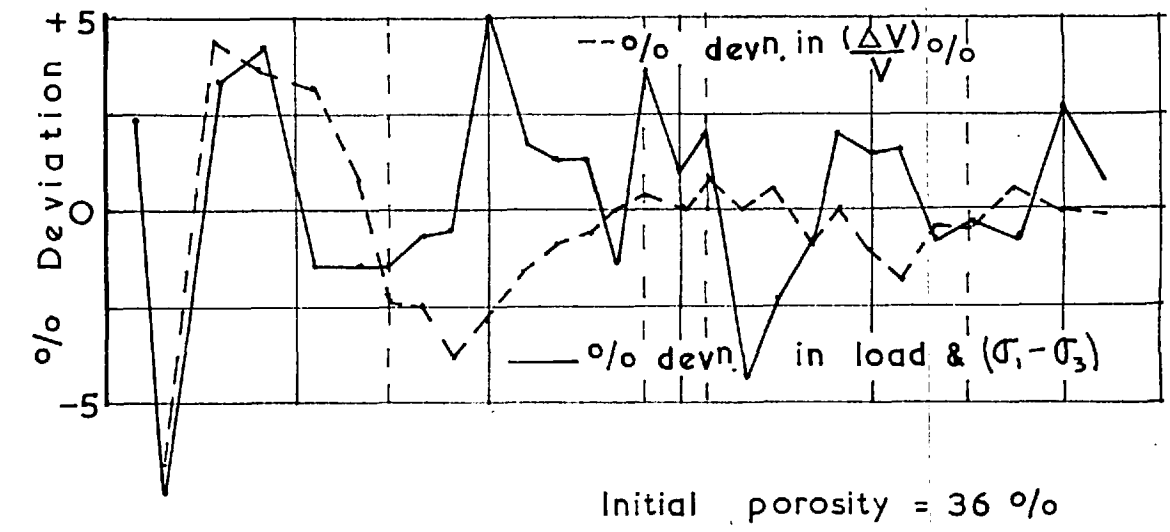
FIG. 4.11









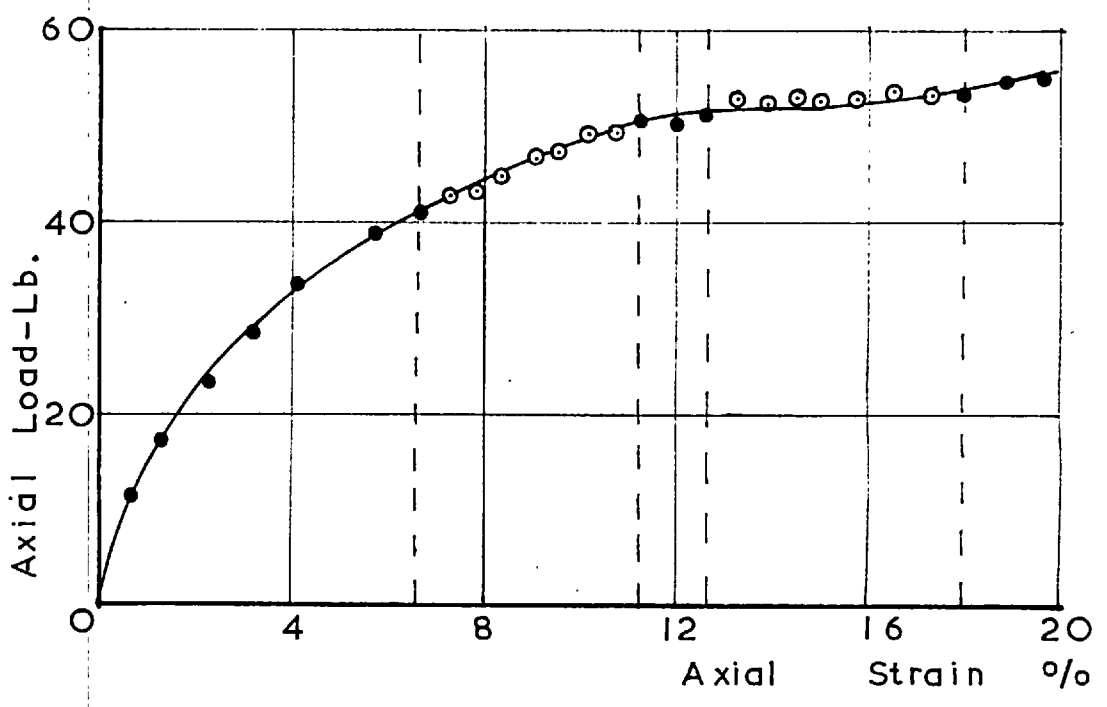


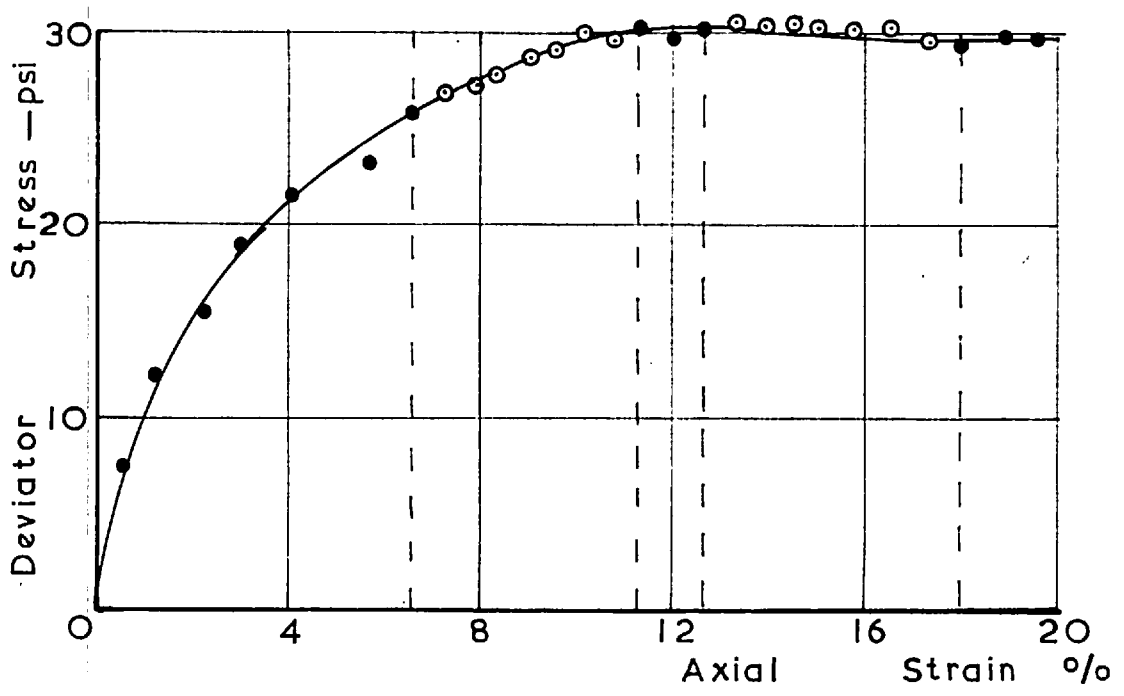
Initial porosity = 36 %

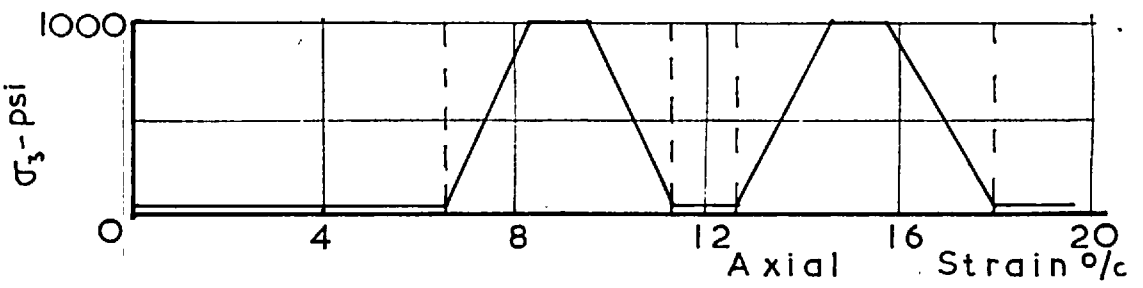
TEST 10. (σ_3-U) maintained constant at 10 psi.

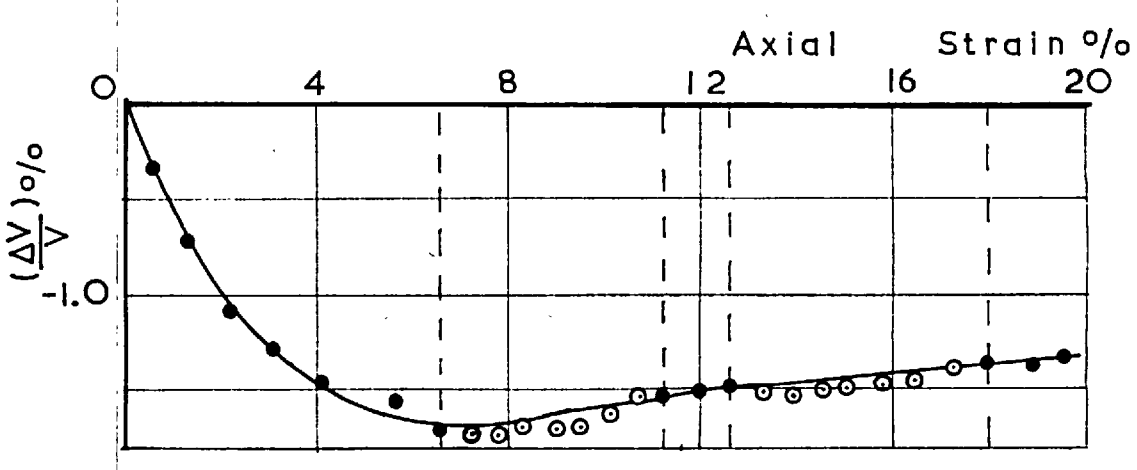
MULTI-STAGE TEST ON CALCITE.

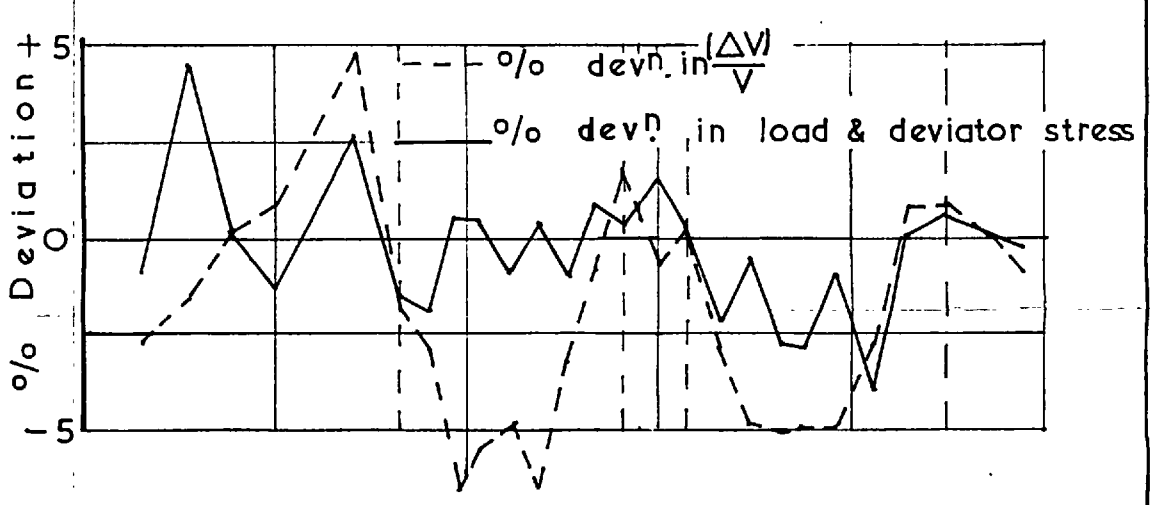
FIG. 4.12









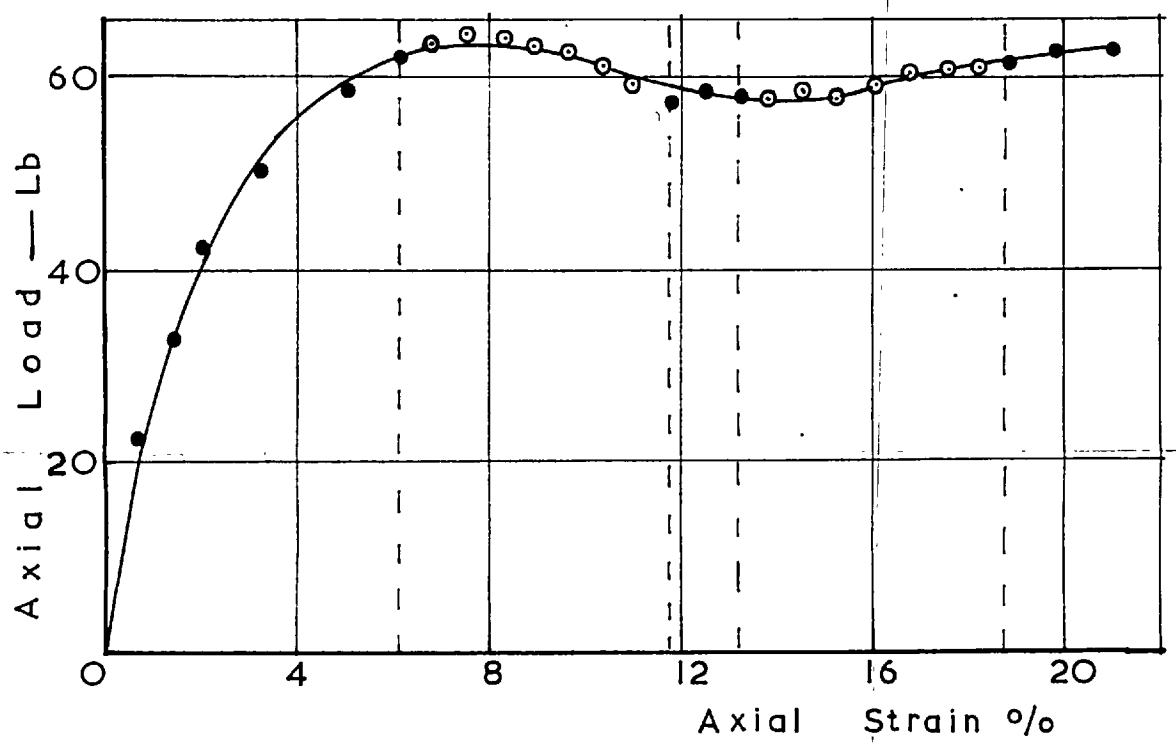


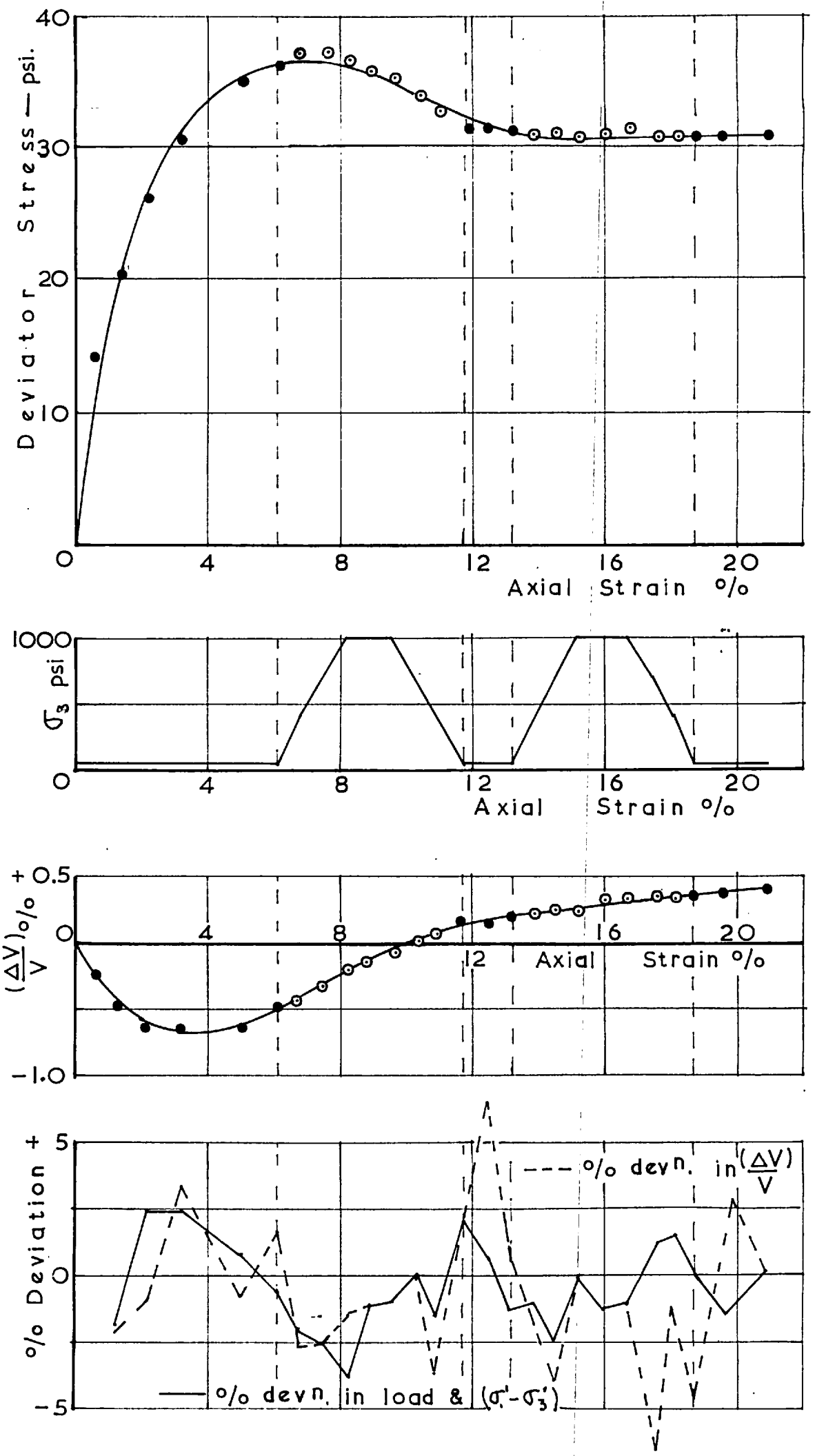
TEST II

$(\sigma_3 - U)$ maintained constant at 10 psi
Initial porosity of sample ≈ 35 %

MULTI-STAGE TEST ON CALCITE.

FIG. 4.13





TEST 12 $(\sigma_3 - U)$ maintained constant at 10 psi

Sample was initially over-consolidated with O.C.R.= 5

Initial porosity = 35 %

MULTI-STAGE TEST ON CALCITE.

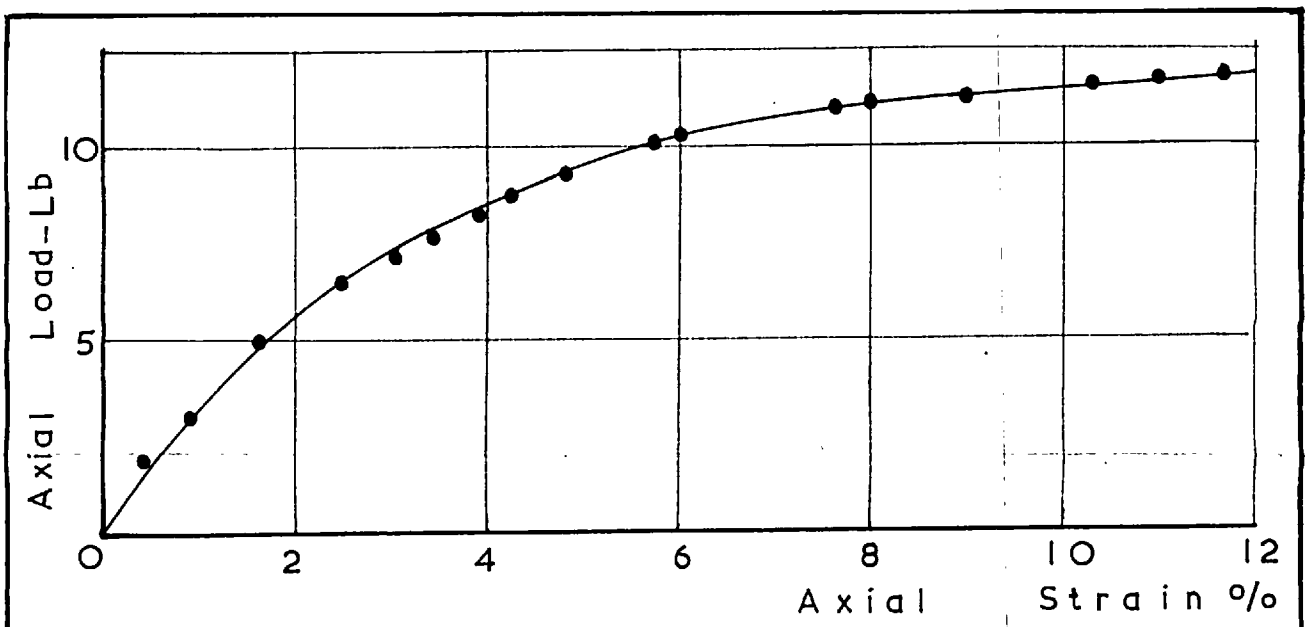
FIG. 4.14

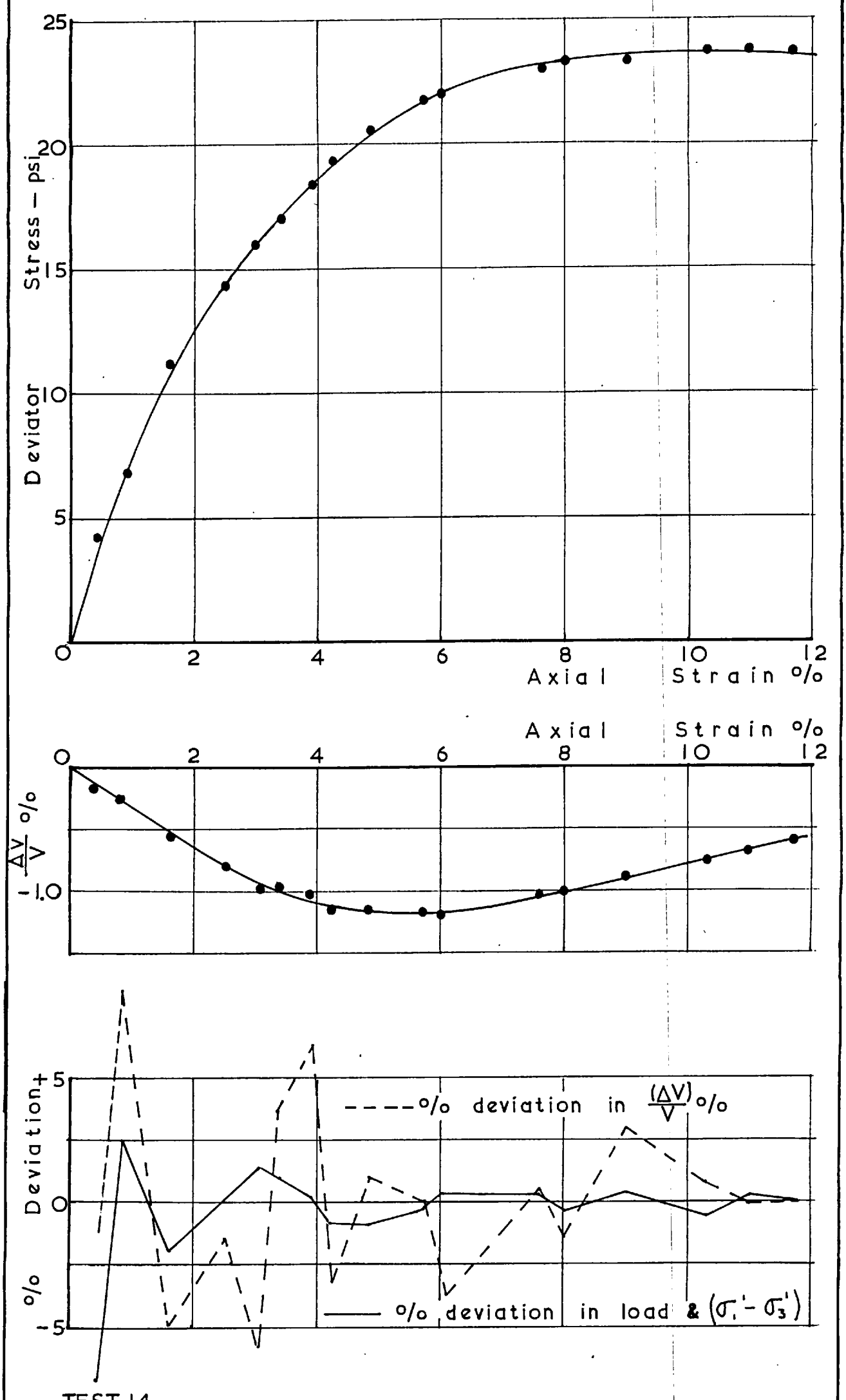
TEST 13

The test was conducted under a constant cell pressure of 100 psi. & back pressure of 35.5 psi. $(\sigma_3 - U) = 64.5$ psi.

DRAINED TEST ON WIENER TEGEL.

FIG. 4.15



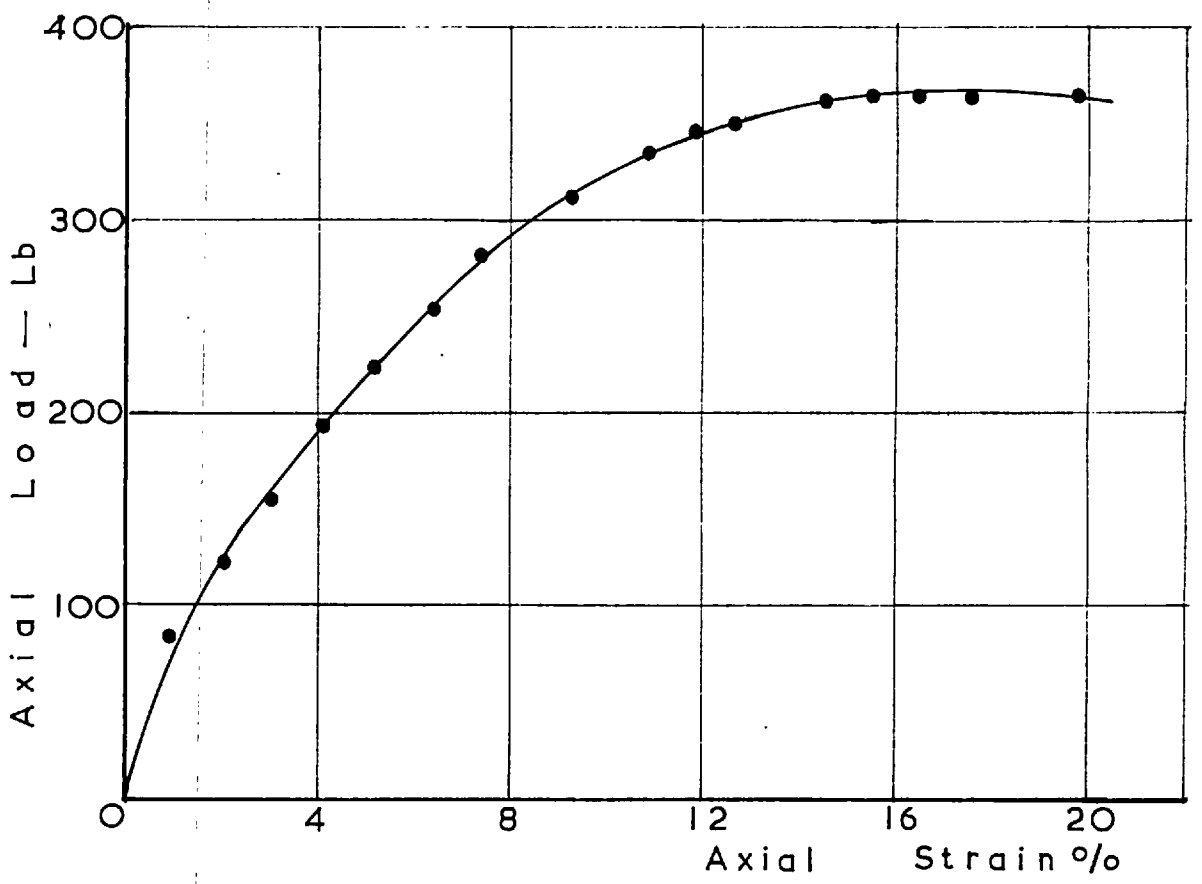


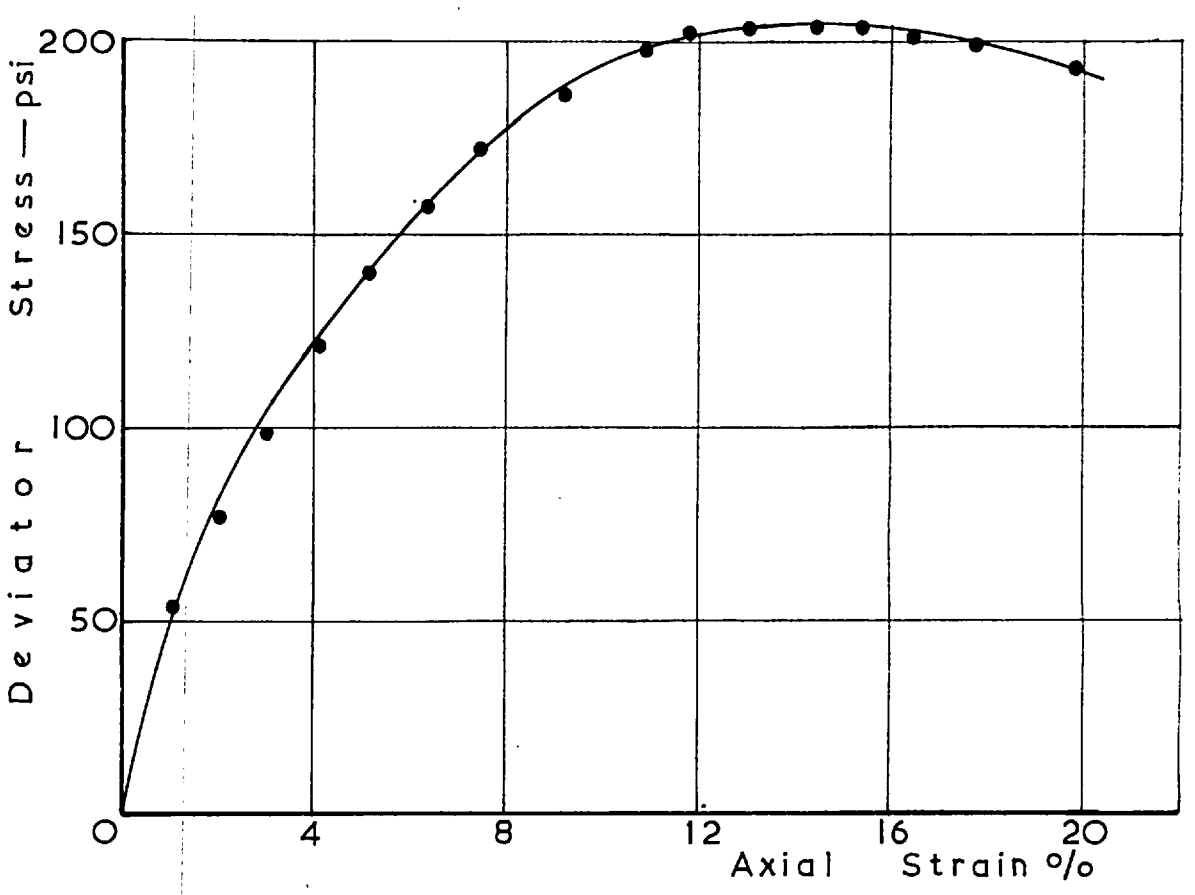
TEST 14

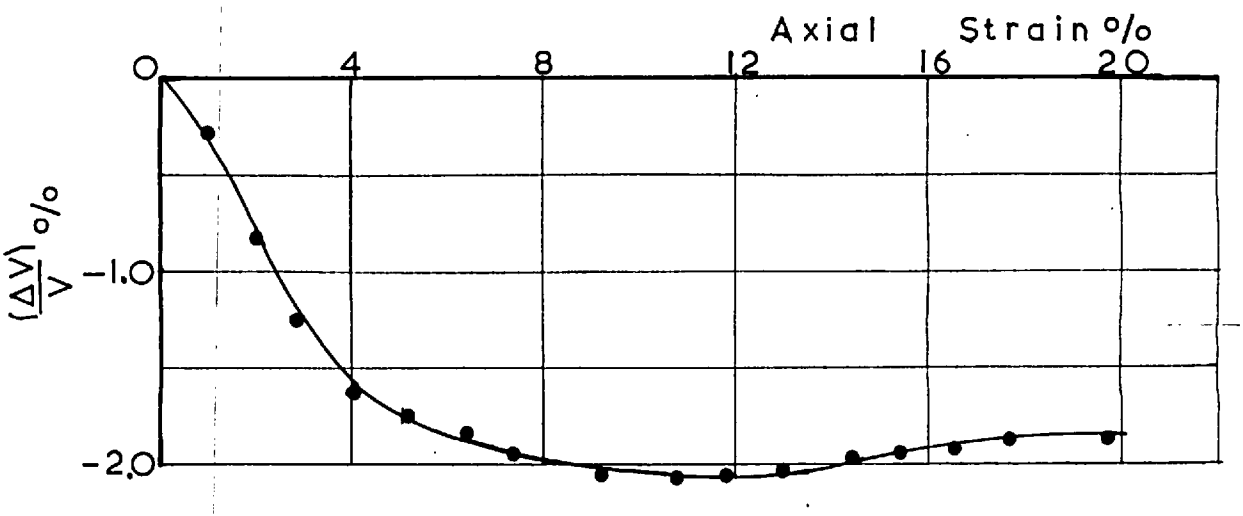
The test was conducted under a constant cell pressure of 50 psi & back pressure of 40 psi. $(\sigma_3 - U) = 10$ psi.

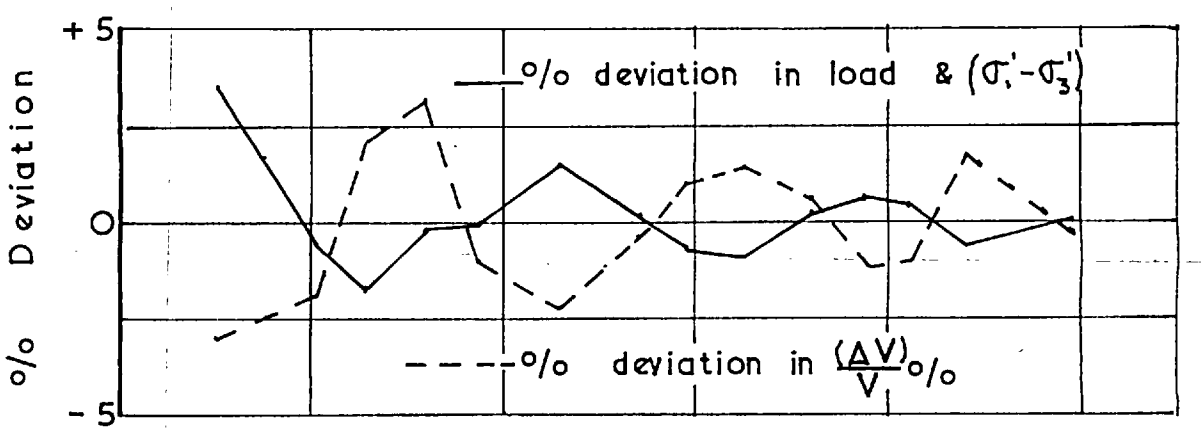
DRAINED TEST ON WIENER TEGEL.

FIG. 4.16







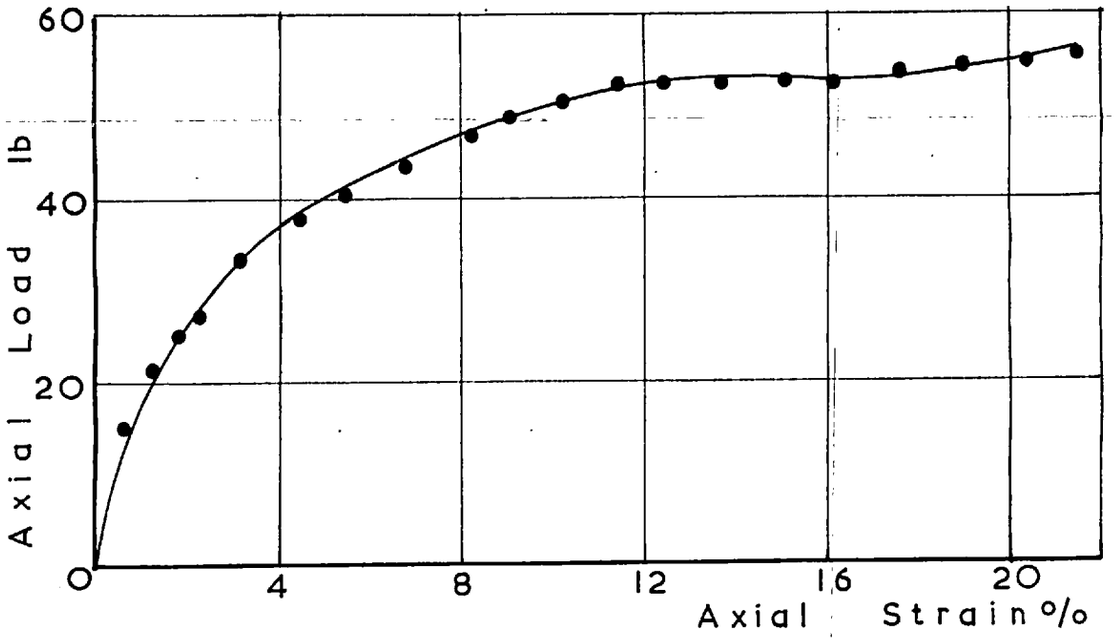


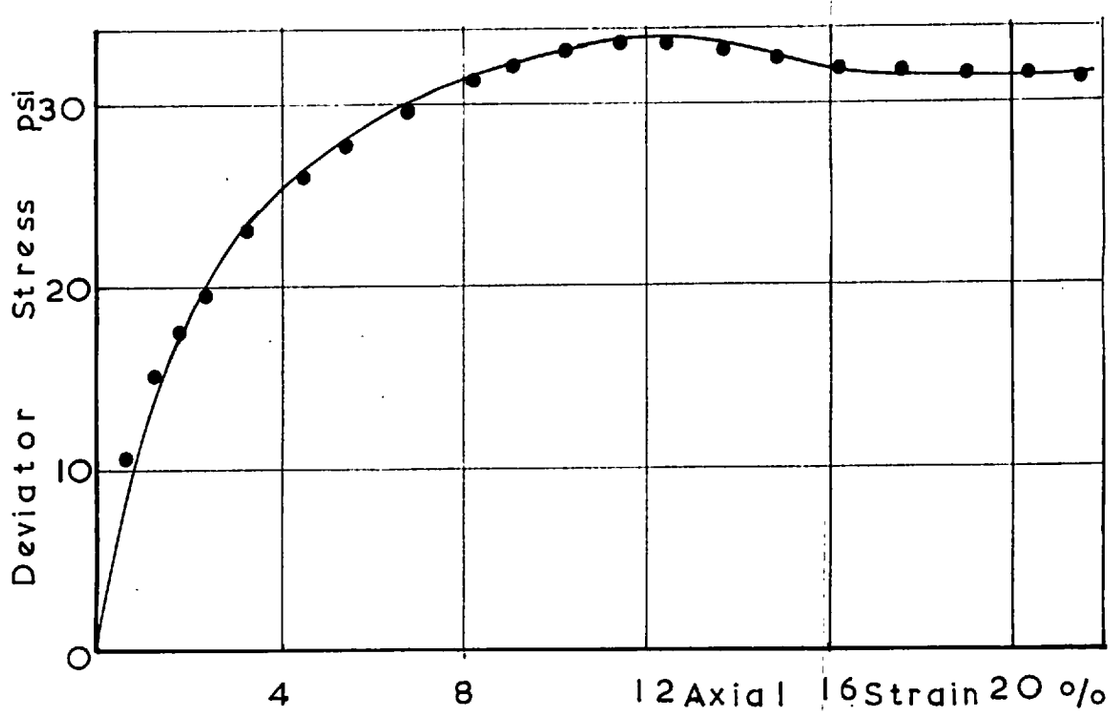
TEST 15.

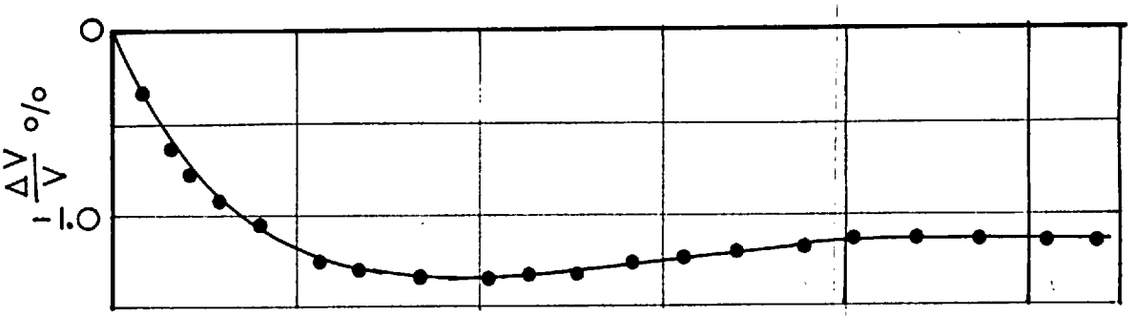
The test was under a constant cell pressure of 100 psi and back pressure of 36.5 psi. $(\sigma_3 - U) = 63.5$ psi
Initial porosity = 36.9 %

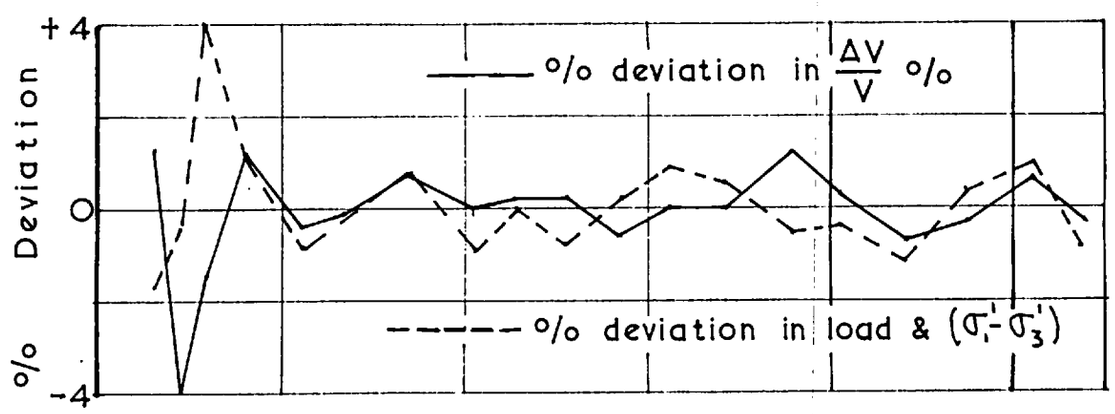
DRAINED TEST ON CALCITE.

FIG. 4.17









TEST 16

The test was conducted under a cell pressure of 50 psi and a back pressure of 40 psi. $(\sigma_3 - U) = 10$ psi.

DRAINED TEST ON CALCITE.

FIG. 4.18

PARTICLE SIZE DISTRIBUTION

Location ——— Boring No ——— Sample No ——— Date of test ———

Description Miscellaneous.

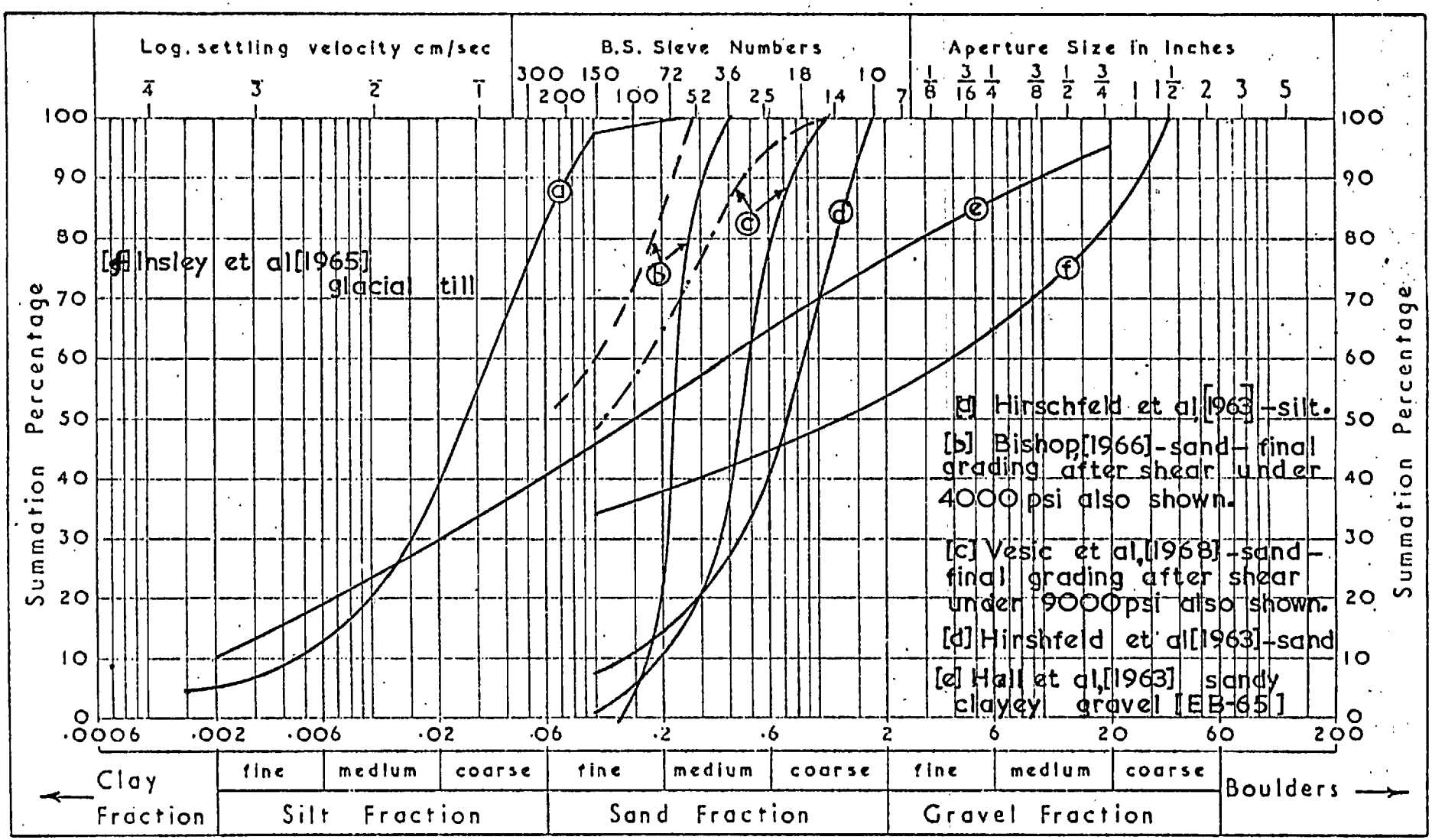


FIG. 5.1

— Particle size in m.m. —

SOIL MECHANICS DEPT
IMPERIAL COLLEGE

PART II

DRAINED COMPRESSION TESTS ON CLAYS

AND

SILTS UNDER ELEVATED PRESSURES.

CHAPTER 5

REVIEW OF THE LITERATURE ON THE SHEAR STRENGTH AND DEFORMATION CHARACTERISTICS OF SOILS UNDER ELEVATED PRESSURES.

5.1 Introduction.

Until a few years ago, most research on the shear strength and deformation properties of soils had been restricted to the range of confining pressures attainable in the conventional triaxial apparatus, namely about 150 psi. However, the increasing tendency towards the construction of very high earth and rockfill dams * in recent years has stimulated research into the mechanical behaviour of soils under pressures comparable to those obtaining under these dams as well as under deep foundations and around deep tunnels.

Since the expression "high pressure" is relative, it is considered pertinent here to define the following ranges of confining pressure:

Low pressures:	0 -150 psi
Elevated pressures:	150 - 1500 psi
High pressures:	1500 - 15,000 psi

According to the above classification, therefore, the work presented in this section falls within the elevated pressure range, and any further use of the term "high pressure" should be treated as purely descriptive. The following is a brief review of recent work on the mechanical behaviour of soils under elevated pressures.

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*Such as the 980' high Nurek dam in the U.S.S.R and the Oroville dam (about 735' high) in the U.S.A.

5.2 The behaviour of granular soils under elevated pressures.

In connection with the preliminary design studies on the Oroville dam, Hall and Gordon (1963) performed drained and consolidated-undrained tests on poorly- to well-graded gravelly materials under confining pressures of up to 650 psi (about 50 kg/cm^2). Hirschfeld and Poulos (1963) described drained tests on compacted sand and an undisturbed silt under pressures of up to 570 psi (40 kg/cm^2). Vesic and Barksdale (1963) reported tests on sands under maximum confining pressures of 9000 psi, in connection with studies on the bearing capacity of deep foundations in sands and the cratering of deeply buried nuclear devices. The conclusions and postulates arising from this work were later verified by Vesic and Clough (1968).

Other recent work on the mechanical behaviour of granular materials under high confining pressures were reported by Bishop (1965) and Bishop (1966) on Ham River sand up to 4,000 psi confining pressure, Insley and Hillis (1965) who described triaxial compression tests on glacial till (the proposed core material of the 650' high Mica Creek dam) up to pressures of 450 psi, and by Tombs (1969) on rockfill. All the results presented for coarse-grained cohesionless materials (e.g. sands and gravels) showed that the shear strength and deformation characteristics of these soils were seriously modified under elevated confining pressures. In particular, the Mohr failure envelopes showed marked curvatures. Some of the factors known to influence the shear strength characteristics of these soils under elevated confining pressures are discussed below.

5.2.1 Particle degradation.

It is generally agreed that the principal cause of the curvature of the failure envelope is the particle break-down which occurs mainly under shear strains, and to a minor extent during compaction and consolidation. Conclusive evidence for this phenomenon has been provided by the increased percentage of fines which occur in the grading of the material after shear, compared with the initial grading (Bishop, 1966; Vesic et al, 1968). Some of the factors which have been known to influence the tendency of a soil to particle degradation under shear are (i) increasing uniformity of the soil, and angularity of the particles (ii) decreasing strength of the individual particles and (iii) increasing confining pressure and shear stress. The major cause of particle break-down are probably the initial grading of the soil and the strength of the individual particles.

Tests within the elevated confining pressure range by Insley et al (1965) on a well-graded glacial till (a material which had previously undergone particle degradation under natural causes) and by Hall and Gordon (1963) on sandy clayey gravel both yielded essentially linear effective stress envelopes. Fig. 5.1 compares the initial grading curves of these materials with those of narrowly graded sands and silts which were subject to particle fracture under shear. The fact that the post-shear gradings of some of these initially narrowly graded soils (for example b, c) approximated to the wide gradings associated with Insley et al's glacial till led Bishop (1965) to suggest that there might be an ideal

grading for these materials incorporating an optimum percentage of fines to inhibit particle break-down under shear without causing too excessive a reduction in ϕ'.

While the phenomenon of particle break-down is not necessarily limited to high stress levels - some amount of particle fracture had been reported in narrowly graded sands at confining pressures as low as 40 psi (Bishop and Green, 1965), it becomes more intense at higher pressures. There is some evidence (Vesic and Clough, 1968) that particle break-down is most intense up to a certain level of mean normal stress, which they called the "break-down stress" σ_B. They postulated that below this stress level, shear stresses are accompanied by particle break-down and the resulting failure envelopes are curved. Above the "break-down stress", the sand behaved as a linearly deformable material with a constant angle of friction ϕ'. They suggested that for a given material σ_B was a function of mineral composition, initial gradation and particle shape, and reported that for their Chatahooche River Sand, σ_B was in excess of 1422 psi (100 kg/cm^2). It is interesting to note that tests by Mr. A.E. Skinner on Han River sand published by Bishop (1966) also showed the tendency that the failure envelope might become linear for confining pressures in excess of 4,000 psi. It is quite possible therefore that the so-called 'break-down stress' σ_B mentioned above may be the minimum value of mean normal stress necessary to alter the initial grading of the sand to that at which no further particle degradation can occur, and above which the failure envelope is linear.

Other factors which determine the degree of particle break-down are the strength and deformation character-

istics and the shape of the particles, as well as the initial density of the soil - the weaker, more compressible or more angular the particle the more particle degradation occurs under a given mean normal stress. Also the higher the initial density, the higher the degree of particle break-down which occurs under shear stresses.

5.2.2 Volumetric Strains.

One main feature of shear behaviour of granular soils under high pressures is that the sample is generally still consolidating when the peak deviator stress is attained. Since the volume change measured in a triaxial test is the sum of the volume changes due to increases in the effective major principal stress σ_1', and shear stresses, it is suggested (Hirschfeld et al, 1963; Bishop, 1966) that this difference in volume change behaviour of soils at elevated pressures may be the result of the predominance of the shear stress component of the volume change at elevated pressures, possibly as a consequence of particle break-down.

The marked influence of the rate of volume change at failure on the shape of the Mohr failure envelope had been demonstrated by Bishop (1966) by showing the close correlation between ϕ' and the rate of volume change at failure $\left(\frac{d\varepsilon_v}{d\varepsilon_1}\right)_f$ for varying confining pressures and initial porosities, $d\varepsilon_v$ and $d\varepsilon_1$ denoting corresponding changes in volumetric and axial strains at failure respectively. Similar conclusion was reached by Hirschfeld et al (1963) by observing that the curved failure envelopes they obtained for compacted sand and silt, became linear when based on deviator stresses corrected for the work done on the sample by the cell pressure during the volume decrease

at failure. The correction of the deviator stress was based on Rowe's dilatancy equation, namely

$$(\sigma_1' - \sigma_3')_{cv} = \frac{\sigma_1'}{1 + \left(\frac{de_v}{de_1}\right)_f} - \sigma_3' \quad \ldots\ldots\ldots\ldots\ldots (5.1)$$

where $(\sigma_1' - \sigma_3')_{cv}$ is the corrected deviator stress. This approach is, however, invalid when applied to tests performed under high confining pressures, since it ignores the work done in crushing the particles. This problem has been discussed in more detail by Osterman (1962). A further evidence, though indirect, of the connection between the rate of volume change at failure and the shape of the Mohr failure envelope has been provided by Vesic et al (1968) who reported that rate of volume change at failure for tests under mean normal stresses in excess of the 'break-down stress' was zero, and as discussed earlier, the failure envelopes were linear for tests in this pressure range.

5.2.3 Axial Strains at failure.

The axial strain at failure has been noted to increase with increasing confining pressure. Generally speaking, the better graded the soil was initially the lower the failure strain. Irrespective of the initial density of the soil, failure under elevated pressures is of the ductile nature normally associated with low pressure tests on initially loose samples.

5.3 The behaviour of cohesive soils under elevated confining pressures.

The mechanical behaviour of cohesive soils under the influence of high confining pressures has not been investigated to the same extent as that of granular soils. There are probably two reasons for this. Since interest in the behaviour of soils under high pressures was initiated mainly as a part of design studies for very high earth and rockfill dams [e.g. Hall and Gordon (1963), Insley et al (1965)] attention was naturally focussed on the properties materials relevant to the studies. Besides, it was relatively easier, in the case of granular soils, to examine such phenomena as particle break-down quantitatively.

The most detailed investigation published to date on the behaviour of cohesive soils under confining pressures in the elevated pressure range had been reported by Bishop, Webb and Lewin (1965) who performed triaxial compression tests on slurried and undisturbed samples of London clay, extending to confining pressures of 1000 psi. Some of their observations and main conclusions are outlined below.

(a) Tests on undisturbed London Clay in the low to intermediate confining pressure range (0-250 psi) were characterised by brittle failure, in which the axial load sometimes decreased to less than half its value at the peak. The stress-strain curves for tests in the elevated pressure range, however, showed ductile failure characteristics.

(b) In general, samples of undisturbed London clay dilated strongly when sheared under low confining pressures, but were still showing decreases in volume at peak deviator stress, when tested under elevated pressures. They suggested that this difference in volume change behaviour probably indicated a breakdown of particle aggregates under high pressures.

(c) The effective stress failure showed a marked change in slope in passing from the low to the elevated confining pressure range. The average values of the effective stress strength parameters c' and ϕ' for undisturbed London clay were 18 psi and 26° in the low pressure range (0-100 psi) and 113 psi and 10° respectively in the elevated pressure range. The curvature in the strength envelope for remoulded London clay was less marked, the drop in ϕ' being typically from 21° to 16°.

(d) They suggested that the drop in ϕ' with increasing confining pressure may be due to the orientation of flaky particles in the direction of shear - a similar mechanism to that invoked by Skempton (1964) to explain the decrease in ϕ' from peak to residual values when clays are subjected to large shear strains at low pressures. This view was reinforced by the closeness of ϕ' for London clay under a pressure of 1000 psi, to the value quoted by Skempton (loc. cit.) on the basis of shear box tests to large strains.

Lee and Haley (1968) reported studies, extending to confining pressures of 1,000 psi, on the relative strength and deformation properties of two compacted clays having different initial structures. They found that the initial structure had no influence on the effective stress failure envelope, but that when total stresses were considered, two separate curved envelopes were obtained - the failure envelope for initially dispersed soils lying above that for samples which had initially flocculated structures.

Tests on stabilized and untreated soils by Wissa et al (1965) showed that provided samples having the same age of curing were considered in the case of stabilized soils, the effective stress failure envelopes for both materials remained linear up to confining pressures of about 850 psi (60 kg/cm^2). Like the glacial till tested by Insley et al (1965) both the untreated and the stabilized soils were well-graded, containing about 10 °/o clay sized particles.

The findings of Insley et al and Wissa et al may be contrasted with those of Bishop et al (1965) and Webb (1966) from high pressure tests on London Clay which contains about 57 °/o of clay sized particles, and also of Lee et al (1968) whose commercial kaolinite contained almost 100 °/o clay sized particles. This would suggest that the percentage of clay sized particles has some influence on the shape of the failure envelope, and that there is probably a minimum percentage of clay sized particles below which ϕ' remains insensitive to confining pressure.

Olson and Kane (1965) presented the results of very fast unconsolidated undrained tests done in connection

with studies of the design of buried structures to resist blast loadings. The tests which were on unsaturated compacted clay, covered the range of confining pressures from 100 psi to 1000 psi. The times to failure ranged from 3 milliseconds to 100 seconds. They attributed the curvature in the total stress failure envelope to pressure saturation of the sample under high pressures.

5.4 Summary.

A brief review was presented on the mechanical behaviour of granular and cohesive soils under elevated confining pressures. Some of the factors which are known to influence the shear strength and deformation properties of these soils under high pressures have been enumerated and discussed.

5.5 Scope of work presented:

Drained triaxial compression tests were performed on slurried and undisturbed intact London Clay, slurried Wiener Tegel, calcite and Brcehead Silt, extending to confining pressures of 1000 psi. The experimental techniques and the preparation and properties of these materials are described in Chapter 6, while the results obtained are discussed in Chapter 7.

CHAPTER 6

DESCRIPTION OF THE EQUIPMENT, THE PREPARATION OF MATERIALS, AND TESTING TECHNIQUES.

6.1 Introduction.

The triaxial apparatus used in these series of tests was a modification of the standard design described by Bishop and Henkel (1962) for tests at conventional pressures (0-150 psi). Only those modifications, which were to enable the apparatus to be used for tests in the elevated pressure range, will be described.

6.1.1 The Triaxial Apparatus.

This equipment consisted of a stainless steel cell, which was 7 1/2" long, and had an internal diameter of 3 1/4", and a wall thickness of 3/8". The thickness of the cell was reduced at the ends to 1/4" from the inside, in order that it can fit over O-rings inserted into 1/4" grooves in the base and the top cap. The top cap was fitted with rotating bushes (Bishop et al, 1965(a)). The cell base, which was machined from a solid brass forging, had four pressure connections fitted with Klinger valves (AB 10), and two other connections into which were sweated 1/8" gas Simplifix couplings, through which saran tubings could be run for top drainage connection, or pore pressure measurement. The connections fitted with Klinger valves were used for filling and emptying the cell, and for providing connections to the constant pressure sources.

The pedestal on the cell base was fitted with a brass adaptor to facilitate the testing of samples of 3/4" diameter instead of the conventional 1 1/2" diameter samples. When the apparatus is assembled, an effective seal was provided between the O-rings at either end and the steel cylinder, by tightening down six 1/4" diameter stainless steel tie-rods which were equally spaced round the cell base, and screwed through both the top cap and the cell base.

6.1.2 The Constant High Pressure Source.

During the earlier tests on clay, the cell pressures up to a maximum value of 1000 psi were maintained constant by means of the self-compensating mercury system described earlier (see Section 2.4). It often became necessary to replenish the upper mercury pots during the tests, because of the high leakage rates occuring under these pressures. This led to some tests being lost overnight as a result of the cell pressure dropping.

Later, a hydraulic constant pressure source employing spring-loaded Ansler bleed valves, and designed by Mr. A.E. Skinner, became available and was used instead of the self-compensating mercury system. Since this apparatus worked by pumping a light machine oil to replace the leakage losses, it became necessary to instal an oil-water interchange pot between it and the cell.

6.1.3 The Screw Control:

For increasing the cell pressure up to 150 psi, the conventional screw control (Bishop and Henkel, 1962) was used. For higher pressures, the cylinder and piston on the hydraulic pressure generator (Section 2.3) was used

as a screw control, by putting the gear lever in the neutral position and driving the piston manually by means of the wheel (fig.2.2(b)).

6.1.4 Volume Change Measurement.

Changes in the volume of the samples of clay, calcite and silt under low pressure were measured by noting the displacement of a column of mercury trapped in a 1/8" bore Saran tubing (see Section 2.6). The volume changes occuring during the tests on silt under confining pressures in excess of about 750 psi, were, however, too large to be accurately measured by this technique, and therefore, in these tests, the volume changes were measured directly by means of a 10 cc burette.

6.2 Preparation of Materials.

6.2.1 Undisturbed London Clay.

Intact lumps of undisturbed London Clay were obtained from a depth of about 20' from a freshly cut platform at a reservoir site in Wraysbury in Middlesex. The lumps were marked for orientation and placed in sealed plastic bags to prevent them drying up. Back in the laboratory, the lumps were carefully trimmed to a thickness of between 2" and 3", with plane and parallel top and bottom faces, in order to facilitate easier sampling later. The clay blocks were coated with about 1/8" thickness of paraffin wax, and were stored in a plastic bag in a temperature-controlled sample room. The trimmings were used to determine the field moisture contents.

The geological history of London Clay in the Thames Valley had been discussed in some detail by Skempton

and Henkel (1957) and by Bishop et al (1965 b), who estimated that the preconsolidation pressure of London Clay at the nearby Ashford Common was about 600 psi (39 t/ft^2).

6.2.2 Remoulded London Clay.

The preparation of this material from slurry was similar to the preparation of Wiener Tegel described in Section 3.1.1. In this case, however, the sample was consolidated to an estimated final load of 4 1/2 tons over three months.

6.2.3 Remoulded Wiener Tegel and Calcite.

The method of preparation of these materials had been described in some detail elsewhere (see Section 3.1.1 and 3.1.2).

6.2.4 Braehead Silt.

Samples of this material were prepared from a batch stored in the laboratory for teaching purposes. The silt and clay fractions were separated by means of sedimentation. X-ray diffraction analyses performed on the two portions of the material revealed that the silt fraction was made up of angular to sub-angular particles of quartz and flaky particles of mica. The approximate mineralogical composition of the clay (minus 2μ) fraction is given below in Table 6.1

Table 6.1

Mineral	Approximate %
Quartz	40
Illite (mica)	35
Kaolinite	20
Chlorite	5

The classification and other properties of the materials are given in Table 6.2, and the grading curves are shown in fig. 6.1.

6.3 Testing Techniques.

6.3.1 Tests on Clays.

The preparation and setting up of the samples of clay were carried out as described in Section 3.2.1.1. In the tests on undisturbed London Clay, the samples extracted from the blocks were closely examined for any evidence of fissuring, the samples with any traces of fissures being rejected. Since the final sample size was only 1 1/2" high by 3/4" diameter, it was relatively easier to obtain intact samples than would be the case if samples of the conventional size were being tested. The sampling from the undisturbed blocks of clay was done in such a way as to obtain samples with vertical axes.

The sample was trimmed to size, weighed and set up on the pedestal of the triaxial apparatus in the standard manner.

The triaxial apparatus was then assembled and filled with water, and topped with castor oil. The cell pressure and back pressure were then applied and the sample allowed to consolidate. Readings were taken of the elapsed time, t, and the change in sample volume (ΔV), to enable the most suitable testing rate to be determined in the usual manner from a plot of $\sqrt{t}$ against (ΔV) (Gibson et al, 1954).

The sample was then sheared under drained conditions. The time to failure varied, in general, between four days and ten days depending on the type of clay and the magnitude of the confining pressure - for a given clay,

Table 6.2

Results of classification & other tests.

MATERIAL	L.L %	P.L %	P.I %	% Clay (<2 μ)	Activity A	S.G	Initial w.c %	Preconsolidn Pressure, P_c (psi)
Undisturbed London Clay.	74	28	46	57	0.81	2.68	29.0	600
Remoulded London Clay							28.9	170 *
Wiener Tegel	52	24	28	34	0.83	2.68	34.3	70
Calcite	12.4	—	—	12+	—	2.72	—	—
Braehead Silt	29	23	6	15	—	2.71	—	—

+ This percentage represents calcite flour rather than clay minerals.

* Approximate.

the time to failure increased with increasing confining pressure.

At the end of the shearing process, the drainage lines were shut, the cell pressure reduced to zero, the water drained from the cell, and the triaxial apparatus dismantled. Where the sample failed on a single shear plane, the inclination of this plane to the horizontal was accurately measured with a bevel protractor. The final moisture content specimen was taken either from the shear zone (if there was one) or from the middle part of the sample. In general, the moisture content at the ends of the sample would be expected to be unrepresentative of the moisture content corresponding to conditions at the end of the test, since the sample must have imbibed some water from the filter stones at either end, on removal of the confining pressure.

6.3.2 Tests on Calcite.

The preparation of the samples of calcite prior to testing had been described in some detail in Section 3.2.2. The nominal size of the samples tested was 3" high by 1 1/2 " diameter.

It was found that the most effective way of preparing fully saturated samples of the material was to deposit it under water, and only the range of initial porosities corresponding to this mode of deposition was investigated. Since the calcite was essentially free-draining, - the consolidation stage being completed in 15 minutes - a convenient strain rate of 0.002 in/min. was selected, leading to failure in between two and three hours.

6.3.3 Tests on Braehead Silt.

Samples with two ranges of initial porosity were tested. A relatively loose packing corresponding to an initial porosity of about 45 °/o, was achieved by sedimenting the silt under water using the same technique as employed in the tests on Calcite. The mixture of silt and distilled water had been previously de-aired by boiling it for about ten minutes. The process of sedimentation was accelerated by the application of a suction of 2 psi to the lower filter stone. After all the particles settled, the excess water was siphoned off and the top filter stone and cap were placed on the end sealed to the rubber membrane with O-rings. The suction was then increased to 5 psi and when the sample attained sufficient strength, the split formers were removed, the sample dimensions measured and the tests conducted in a manner similar to that described for calcite.

Samples with initial porosities of about 40°/o were prepared by compacting silt at a moisture content of about 10 °/o into the rubber membrane enclosed in the split former. The sample was compacted in 1/2" layers, using a special brass tamper. The sample was trimmed level with the top of the split formers and the top filter stone and the top cap which had a drainage connection, were installed and sealed to the rubber membrane in the usual way. The split formers were removed, and after the sample dimensions were measured, the cell was assembled and filled with water and castor oil.

The sample was saturated in situ by applying a cell pressure of 30 psi and back pressures of 23 psi and 28 psi to the base and top drainage lines respectively, and

permitting water to flow through the sample for over 24 hours prior to testing. In order to further ensure saturation, all tests, with the exception of those conducted under a confining pressure of 750 psi and 1000 psi, were performed with drainage against a back pressure of 100 psi. Saturation times of 48 hours were allowed for tests under confining pressures in excess of 750 psi, since the necessity for measuring the volume change of the sample directly with a burette ruled out the possibility of applying back pressures.

A testing rate of 0.02 in/min. was used in all the tests, and the dry weight of the material used in each sample was determined by recovering the sample after the test and weighing it after oven-drying it. This weight generally compared well with the weight determined by the difference between the total dry weight of soil used in making the original mixture and the dry weight of the soil remaining after the preparation of the sample.

PARTICLE SIZE DISTRIBUTION

Location _____ Boring No _____ Sample No _____ Date of test _____

Description _____

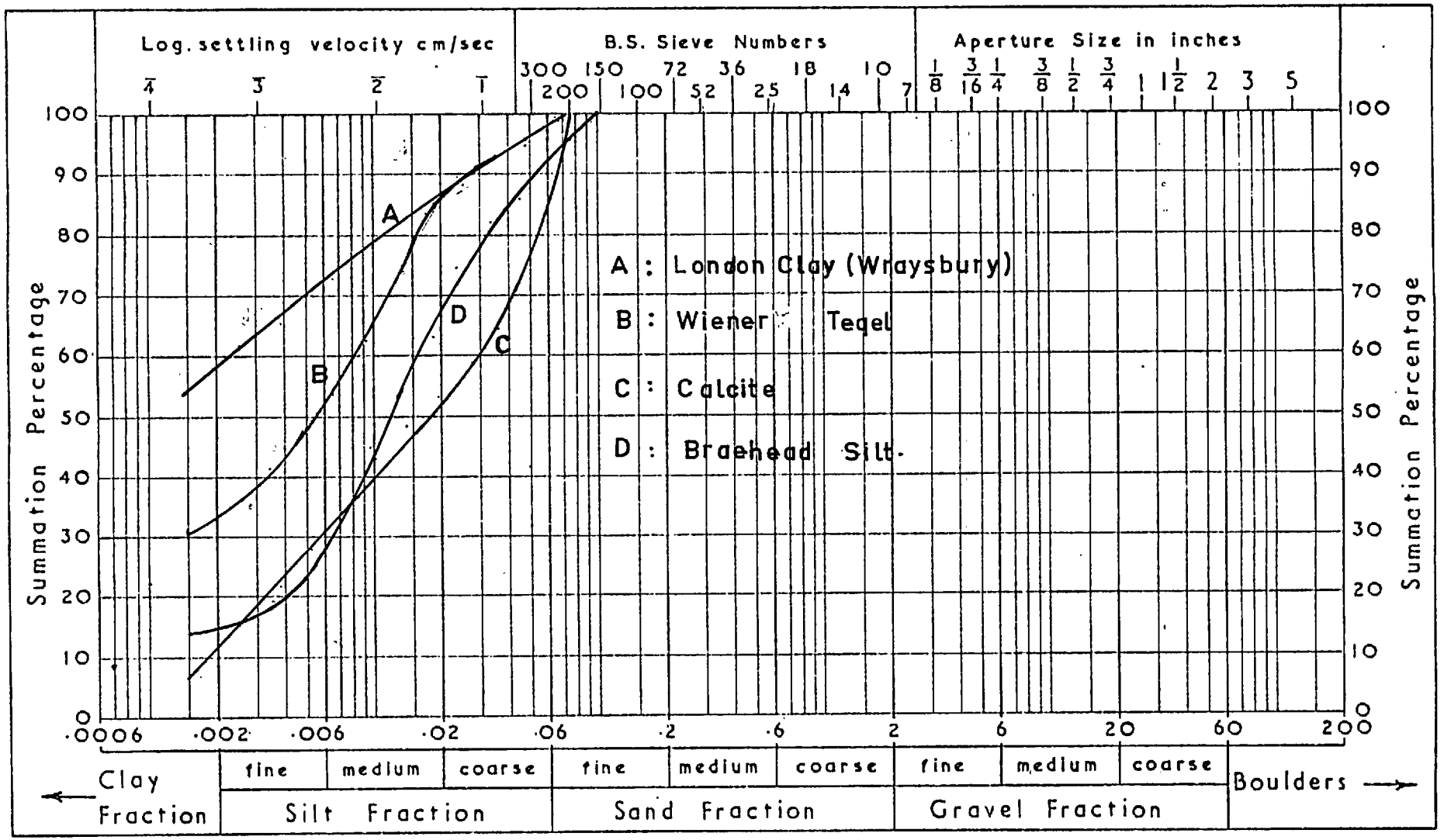


FIG. 6.1

SOIL MECHANICS DEPT
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CHAPTER 7

PRESENTATION AND DISCUSSION OF THE RESULTS OF DRAINED COMPRESSION TESTS.

7.1 Failure criteria.

The choice of the type of failure criterion which best describes failure conditions in soils had been subject to some controversy, for example, Haythornthwaite (1961); Geuze (1963) with discussion, Wu et al (1963) and more recently, Bishop (1966). Bishop examined the three main criteria, namely the Mohr-Coulomb, extended Tresca and the extended Von Mises criteria in the light of available experimental evidence and concluded that, of these failure criteria, the Mohr-Coulomb criterion is the most applicable to define the failure condition in granular soils. Even though there is some controversy about its validity when applied to the failure conditions in cohesive soils (Geuze, 1963) the writer believes that, in the absence of any conclusive evidence to the contrary, the Mohr-Coulomb criterion provides the best method currently available for analysing test results in both granular and cohesive soils. The results of the drained tests performed will, therefore, be analysed on the basis of the Mohr-Coulomb failure criterion, failure being defined as the maximum deviator stress condition.

7.2 Presentation of Results.

The deviator stresses were computed on the basis of the average cross-sectional area of the sample as

discussed elsewhere (Section 3.3.2).

Curves of deviator stress and volumetric strain against axial strain were presented for all the materials tested. In addition, conventional and modified Mohr envelopes were also presented for each material covering both the low and the elevated pressure range. The modified Mohr diagram, in which $\left(\frac{\sigma_1' + \sigma_3'}{2}\right)$ is plotted against $\left(\frac{\sigma_1' - \sigma_3'}{2}\right)$, has the advantage of clarity over the conventional Mohr failure envelope, in that the stress circles are replaced by points, and can therefore be more easily averaged, and a more representative failure envelope drawn. The usual effective stress parameters, C' and ϕ' are obtained from the slope ψ and the intercept d on the $\left(\frac{\sigma_1' - \sigma_3'}{2}\right)$ axis by means of the expressions

$$\phi' = \text{Sin}^{-1} (\tan \psi) \quad \ldots\ldots\ldots\ldots \quad (7.1)$$

$$\text{and} \quad C' = d \tan \phi' \cot \psi \quad \ldots\ldots\ldots\ldots \quad (7.2)$$

The results obtained for undisturbed London Clay were compared with those obtained by Webb (1966) from tests on London Clay from nearby Ashford. Hvorslev parameters K and ϕ_e were also determined for the clays.

7.3 Discussion of results.

7.3.1 Drained Tests on Clays.

7.3.1.1 Stress-Strain Curves.

Drained tests on intact London Clay under conventional (0-150 psi) confining pressures showed sudden brittle failures, with the sample failing along a single shear plane. The mode of failure changed into a ductile one as the confining pressure increased. Similar tests on remoulded Wiener Tegel and London Clay resulted in smooth stress-strain curves over the whole elevated pressure range (fig.7.1 to fig. 7.7).

In general, the failure strain increased with increasing confining pressure, thus leading to considerable barelling of the sample, and a possibility of errors being introduced into the computation of the cross-sectional areas. The values of deviator stress were not corrected for the confining effect of the rubber membrane (0.008" thickness) since this effect was deemed insignificant compared with the magnitude of the deviator stresses being measured.

7.3.1.2 Volumetric Strain Characteristics.

All the three types of clay decreased in volume initially on the application of the deviatoric stress. At failure, however, the samples of intact London Clay, tested under low confining pressures showed strong dilatation characteristics, while those of the remoulded clays continued to consolidate till the end of the test. All samples tested between about 200 psi to 1000 psi decreased in volume throughout the shearing process.

The curves of the rate of volume change of the sample at failure against confining pressure are given in fig. 7.21. From the curves, it appears as if failure at constant volume occurs under a confining pressure of about 250 psi for intact London Clay, and about 140 psi for Wiener Tegel.. The curves for intact and remoulded London Clay coincide for tests under confining pressures in excess of about 550 psi. A possible explanation of this phenomenon is discussed later (Section 7.3.1.3d).

In general, the volume change behaviour of the clays tested followed closely the sort of volume change behaviour normally associated with shear in granular ma-

terials under comparable or higher confining pressures (Bishop, 1966; Vesic et al, 1968) and is generally attributed to the effects of particle break-down. If one accepts the 'domain' type of clay structure (Trollope, 1960) discussed in Chapter 1, and if one further accepts the suggestion (Morgan, 1967) that an increase in confining pressure results in 'domain growth' as a consequence of the closer packing of smaller domains, then one can expect the behaviour of cohesive soils under elevated pressures to be similar to that of granular soils under comparable confining pressures. On this basis, the volune change characteristics can be explained by the break-down of the enlarged particle 'donains' in a nanner similar to the break-down of single particles in granular soils.

7.3.1.3 The relationship between normal effective Stress and shear strength.

The effective stress Mohr failure envelopes and the curves of $\left(\frac{\sigma_1' + \sigma_3'}{2}\right)$ against $\left(\frac{\sigma_1' - \sigma_3'}{2}\right)$ are given for the three types of clay in figs. 7.12 to fig. 7.17. All the strength envelopes showed marked curvatures in passing from the low to the elevated pressure range. The following observations can be nade fron the curves:

(a) Departure from the straight line failure envelopes obtained under low confining pressures occurred in all three types of clay between confining pressures of 200 psi and 300 psi. The curvature in the envelope for intact London Clay was nore pronounced than that in the envelope for renoulded London Clay (fig.7.17a).

(b) There was an indication that the slope of the Mohr envelope for undisturbed London Clay increased again for confining pressures in excess of about 700 psi. Similar trend was noted by Webb (1966) from tests on samples of London Clay from nearby Ashford Common. The failure envelope obtained by Webb from tests in vertical samples obtained from a depth of about 66' was plotted on fig. 7.16 for comparison. Unfortunately, the limitations of the apparatus did not permit the performance of tests under confining pressures in excess of 1000 psi in order to confirm this trend. However, it is interesting to note that the average preconsolidation pressure of London Clay in this area has been estimated at about 600 psi (Bishop et al, 1965, b) and it is quite possible that this increase in ϕ' resulted from the fact that the sample behaved as a normally consolidated one, for confining pressures in excess of the preconsolidation pressure.

A possible alternative explanation may be deduced by a direct analogy with the behaviour of other porous media, in which continued increase in confining pressures results in a progressive elimination of voids and plastic deformation of the mass of material, with the resulting increase in strength. However, if this mechanism were operating, it would be reasonable to expect to detect a similar trend in the envelope for remoulded London Clay, since the magnitude of the confining pressures would be such that all effects of the initial stress history of the material would be destroyed.

(c) Advantage was taken of the fact that the effective stress cohesion intercept C' is zero for normally conso-

lidated soils, to determine the values of ϕ' for confining pressures in excess of the estimated preconsolidation pressures. The variation of ϕ' with confining pressure for clays is shown in fig. 7.22 (c & d). From this figure, it can be seen that ϕ' decreases with increasing confining pressure for both the remoulded and undisturbed clays. However, while the curves for the remoulded clays appeared to be tending to a constant value, the curve for intact London Clay decreased to within 0.5° of the curve for remoulded London Clay at a pressure of about 900 psi, and then began to rise again.

(d) The convergence of the volume change characteristics at failure (fig.7.21) and the strength properties (fig. 7.22 (d)) for intact and remoulded London Clay at elevated pressures would tend to suggest that the structures of the two clays before, and possibly during, shear were identical. Undisturbed London Clay is known to have a structure with a high degree of particle parallelism (Tchalenko, 1968).

The method of preparing the remoulded material from slurry was such that it could be expected to produce a certain degree of preferred particle orientation in a direction perpendicular to the applied stress. Work on pure kaolin (Tchalenko, 1967) showed that the maximum degree of orientation is nearly attained under an effective normal stress as low as 1.6 psi ($0.1 t/ft^2$). Considerations of the relative sizes and intensity of surface activity of the mineral particles of Kaolinite and illite (which is the predominant clay mineral in London Clay) would lead one to expect that the corresponding pressure would be much higher for London Clay. Indeed, there is

experimental evidence to suggest (Tchalenko, 1968) that the original 'undisturbed' structure of London Clay is only re-established when the slurried material is consolidated one-dimensionally under pressures comparable to the in-situ pre-consolidation pressure.

Experimental information on the influence of isotropic consolidation pressures on the structure of clay is scanty. Meade (1964), however, suggests that an increase in confining pressure increases the degree of particle orientation, especially where the initial structure of the material had a partial preferred orientation. The foregoing discussion would, therefore, lead one to suppose that a combination of an initial one-dimensional consolidation and a subsequent isotropic consolidation together with the effects of shear stresses would produce comparable high degrees of particle orientation in the remoulded and undisturbed samples of London Clay under elevated pressures. In addition to accounting for the convergence of properties, this increase in degree of orientation of particles with increase in confining pressure may also explain the closeness of the origin ϕ' under elevated pressures to the residual angle of shearing resistance ϕ'_r obtained in the shear box (Skempton, 1964).

7.3.1.4 Hvorslev Parameters.

Hvorslev (1937 and 1960) postulated that the shear strength of saturated normally consolidated clays may be represented by the relationship *

$$\tau_f = c_e + \sigma'_n \tan \phi_e \quad \ldots\ldots\ldots\ldots \quad (7.3)$$

* This relationship applies strictly to remoulded clays with high initial moisture contents. Hvorslev (1960) suggests that modifications may be necessary when the equation is applied to undisturbed soils on account of natural cementations, or when applied to remoulded soils with low moisture content, as occur during shear under /cont...

where C_e and ϕ_e are the true cohesion and angle of friction respectively. The parameter ϕ_e remains constant for a given clay, but C_e is a function of the moisture content in the failure zone, and, to a lesser extent of the initial structure of the soil.

A direct experimental determination of the parameters C_e and ϕ_e involves the difficult problem of ensuring that all the samples have the same void ratio (or water content) at failure. Various ways of overcoming this problem have been proposed and discussed by Terzaghi, (1938) Bjerrum, (1954) Bishop and Henkel (1962), Schmertmann (1963) and by Gibson (1953) who also discussed the modification to equation 7.3. to take account of the energy dissipation as a result of volume changes during shear.

The following method, will be used to analyse the results. Based on the Mohr-Coulomb failure criterion, it can be shown that the relationship between the deviator stress at failure in a drained triaxial test and the Hvorslev parameters C_e and ϕ_e is of the form

$$\frac{1}{2}(\sigma_1' - \sigma_3')_f = \frac{C_e \cos \phi_e}{1 - \sin \phi_e} + \sigma_{3_f}' \frac{\sin \phi_e}{1-\sin \phi_e} \quad \ldots\ldots(7.4)$$

If C_e is put equal to $(K \sigma_e)$* , where K is an arbitrary constant, and σ_e is the 'equivalent consolidation pressure', defined as the pressure on the virgin consolidation curve corresponding to the water content

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F/n. cont. from p. 125.
elevated cell pressures.

* This form of relationship between C_e and σ_e has been shown by Bjerrum (1954) to be incorrect for clays remoulded at low moisture contents.

or void ratio at failure, then equation (7.4) reduces to

$$\left(\frac{\sigma_1' - \sigma_3'}{2\sigma_e}\right) f = \frac{K \cos \phi_e}{1 - \sin \phi_e} + \frac{\sigma_{3_f}' \sin \phi_e}{\sigma_e (1-\sin \phi_e)} \quad \ldots(7.5)$$

If a graph of $\left(\frac{\sigma_1'-\sigma_3'}{2\sigma_e}\right)_f$ is plotted against $\left(\frac{\sigma_3'}{\sigma_e}\right)$, therefore, the slope of the resulting straight line would be $\frac{\sin \phi_e}{1-\sin \phi_e}$ and the intercept on the $\left(\frac{\sigma_1' - \sigma_3'}{2\sigma_e}\right)$ axis would be $\left(\frac{K \cos \phi_e}{1-\sin \phi_e}\right)$, hence both K and ϕ_e can be determined. The graphs of $\left(\frac{\sigma_1'-\sigma_3'}{2\sigma_e}\right) f$ against $\left(\frac{\sigma_{3_f}}{\sigma_e}\right)$ for both Wiener Tegel and London Clay are given in fig. 7.23. No energy corrections have been applied to the results. Values of σ_e were derived from fig. 7.24 based on the assumption that the virgin consolidation curves for all the clays are identical with the curves of water content at the end of consolidation against consolidation pressure (fig.7.24). The virgin consolidation curve for intact London Clay was assumed to be the same as that for remoulded London Clay.

The average values of ϕ_e and K are given in Table 7.1. The amount of scatter in the results for London Clay made any accurate determination of the parameters difficult. Also the separation of the points was not large enough for the influence of confining pressure, if any, on the parameters to be detected.

7.3.2 Drained Tests on Calcite and Braehead Silt.

7.3.2.1 Introduction.

The stress - strain and volumetric strain - axial strain curves for both materials are given in figs. 7.8 to 7.11. The general shape of these curves indicated

ductile failures over the full elevated pressure range, irrespective of the initial porosity.

The volumetric strain - axial strain curves showed that the volume change behaviour of these soils during shear followed the same pattern as was observed for clays under similar confining pressures, namely dilatation under low confining pressures and consolidation under elevated pressures.

7.3.2.2 Strength Relationships.

The conventional and modified Mohr failure envelopes for both materials are presented in figs. 7.18 and 7.19, and 7.20 and the low pressure envelopes for Braehead Silt are given in Fig. 7.13 (a). The Mohr envelope for calcite remained essentially linear over the full elevated pressure range, while the envelope for Braehead Silt showed a marked curvature over the same confining pressure range, the curvature being more pronounced in the case of samples which were initially dense. Over the elevated pressure range, the effective angle of internal friction ϕ' decreased from 33.6° to 27.7° for initially dense samples and from 30.8° to 27.3° for initially loose samples (fig. 7.22 (b)). Allowing for the scatter in the results for calcite, the corresponding decrease in ϕ' was only 0.3° (a drop from 38.8° to 38.5°) (fig. 7.22a).

7.3.2.3 Rate of volume change at failure $(\frac{de_v}{de_1})_f$

The plots of the rate of volume change at failure $(\frac{de_v}{de_1})_f$ against the effective confining pressure for both materials is given in fig. 7.21. In the low pressure range (0-200 psi), the initially dense samples

of the silt dilated more strongly during shear than did the samples which were initially loose. The two curves converged with increasing confining pressure in the same fashion as was noted for clays and sands, and may be attributed to the progressive destruction of the effects of initial porosity of the sample.

The curve for calcite showed that the material continued to dilate during shear under confining pressures as high as 700 psi, at which stress level all other materials are already showing negative volume changes during shear. As mentioned elsewhere (Section 3.2.2) it was extremely difficult to wet the calcite during the preparation of the sample. This probably indicates some sort of particle surface activity which could conceivably account for this 'atypical' volume change behaviour during shear under elevated pressures.

Alternatively, this volume change behaviour could be taken as a possible evidence of the absence of any substantial degree of particle break-down during shear under confining pressures of up to about 700 psi.

A fairly close correlation was obtained between $(\frac{de_v}{de_1})_f$ and ϕ' (fig. 7.25b) for Brachead Silt, irrespective of the initial density or effective confining pressure, σ'_3. Bishop (1966) also reported a similar correlation for Ham River Sand over a much wider range of confining pressure (up to 4000 psi) and suggested that it indicated that the curvature in the Mohr failure envelope was largely accounted for by the decrease in the rate of dilatancy at failure with increasing stress.

A similar plot for calcite showed some scatter (fig.7.25a) but a linear trend was detectable. It is

worth noting, further, that the start of the slight decrease in ϕ' with increasing confining pressure in fig. 7.22a corresponded closely with the start of the decrease in $(\frac{de_v}{de_1})_f$ with cell pressure in fig. 7.21; this probably indicates a link between $(\frac{de_v}{de_1})_f$ and ϕ'.

7.3.2.4 Particle break-down.

In coarse grained cohesionless soils, the decrease in the rate of dilatancy $(\frac{de_v}{de_1})_f$ is usually attributed to the combined effects of particle break-down and increase in shear stresses, the former effect being the more important. It is believed, however, that the phenomenon of particle break-down is probably not very important in fine grained cohesionless soils at the stress levels considered, and that the volume change behaviour of the silt during shear in the elevated pressure range was due principally to the effects of the increase in shear stresses. This is because, firstly, the increased number of interparticle contacts in fine grained cohesionless soils would reduce the stresses at the contacts considerably, and, secondly, it is possible that a substantial percentage of fines could form a matrix which could effectively reduce the number of contacts between larger granular particles.

An indirect evidence of the absence of particle break-down in the silt was obtained by plotting the volumetric strains resulting from isotropic consolidation alone and also from the combined effects of isotropic consolidation and the application of shear stresses against the average principal stress in fig. 7.26. Similar curves for Ham River Sand (Bishop, 1966) are also plotted on the

diagram for comparison. It can be seen from this diagram that, for the silt, the volumetric strain due to the application of shear stresses was much smaller than that due to isotropic consolidation pressures - a behaviour which is contrary to that observed from tests under comparable pressures on Ham River Sand which underwent particle break-down during shear. It is suggested that the smaller magnitude of the volumetric strain during shear of Braehead Silt compared with that of Ham River Sand probably indicates the absence of particle break-down during shear.

Values of the volumetric strain resulting from isotropic consolidation were not available for a similar comparison to be made for calcite. However, the relative invariance of ϕ', for this material, with confining pressure over the elevated pressure range, coupled with the volume change behaviour during shear discussed earlier, may be taken as evidence of the absence of any substantial degree of particle degradation during shear under high pressures.

On the available, admittedly indirect, evidence, therefore, it can be concluded that, though the mechanical behaviour of fine-grained and coarse grained cohesionless soils under elevated pressures are similar, the phenomenon of particle break-down, which is accepted as being principally responsible for the modification of the properties of coarse grained cohesionless soils under these pressures is probably not very important in fine-grained cohesionless soils.

7.3.2.5 Comparison between the behaviour of calcite and Braehead Silt under elevated pressures.

As has been mentioned in Section 7.3.2.2, over the elevated pressure range, the shear strength properties of the calcite remained essentially unchanged, while the ϕ' for the Braehead Silt was strongly dependent on confining pressures. Some factors which may possibly account for this difference in shear behaviour are examined below.

7.3.2.6 Initial Grading.

The initial grading curves (fig.6.1) showed that the two materials had essentially identical grading, except that the calcite contained slightly more material in the medium to coarse silt fraction range. It is not believed that this minor difference in initial grading would significantly account for the observed differences in shear behaviour under elevated pressures.

7.3.2.7 Particle Shape.

The coarse (silt sized) particles in the Braehead Silt were separated from the clay-sized particles by sedimentation techniques, and were mounted in Canada Balsam for viewing under an optical microscope. Particles of the calcite were also sprinkled on a thin section glass plate, and were covered with a cover glass plate. Magnified 500 times, the coarse particles in the silt were seen to be angular to sub-angular fragments of quartz and feldspar and flakes of mica, while the calcite particles were seen to be rounded.

In coarse grained granular materials in which the particles are in direct contact, angular particles generally

undergo more particle break-down and therefore more reduction in ϕ', than rounded particles, as a result of shear under high pressures. However, as explained earlier, direct interparticle contact between the silt sized particles may be either reduced or prevented by a matrix of fines, and therefore angular particles in fine grained cohesionless soils may not necessarily lead to particle break-down. Also, it has already been suggested that the phenomenon of particle degradation may probably, not be invoked to explain the modification in the shear strength properties of the silt under elevated pressures. Differences in particle shape are therefore not considered important as far as the differences in shear behaviour are concerned.

7.3.2.8 The nature of the clay-sized particles.

The two materials contained approximately the same percentages of fines. However, the fines in the calcite were calcite flour which was relatively inactive compared with the fines in the Braehead Silt which were identified as containing a substantial amount of clay minerals (Section 6.2.4). It is suggested that the observed differences in shear behaviour between calcite and Braehead Silt over the elevated pressure range may be accounted for by considering the relative activity of the fines. This may also partially explain the insensitivity of ϕ' with increasing confining pressure noted for a glacial till by Hirschfeld et al (1965) since the fines would be predominantly inactive rock flour.

7.3.2.9 Summary.

The results of drained compression tests on clays and fine-grained cohesionless soils over the elevated pressure range are presented. It has been found that the shear strength properties of the clays and the Braehead Silt were modified in the manner already noted for sands and gravels, and the possible reasons for this behaviour have been discussed.

The behaviour of the silt has been compared with that of calcite whose shear strength parameters remained constant over the elevated pressure range. It has been suggested that this difference in shear strength behaviour may be explained by considering the relative activity of the fines in each material.

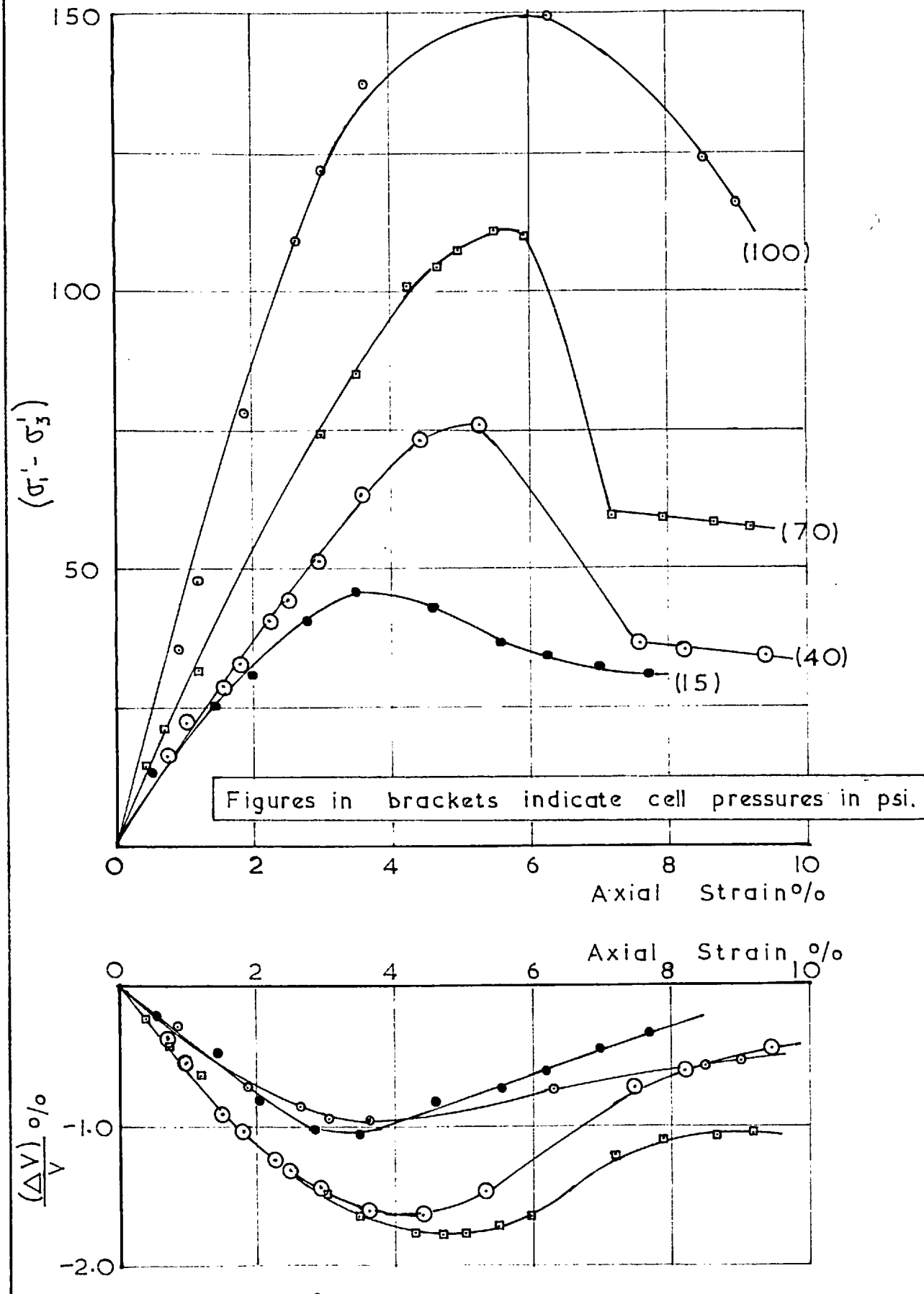
Table 7.1

STRENGTH PARAMETERS.

MATERIAL	Intact London Clay					Remoulded London Clay					Remoulded Wiener Tegel				
STRENGTH PARAMETERS / PRESSURE RANGES (PSI)	C' (psi)	ø' °	ø'orig.+ °	K	øe °	C' (psi)	ø' °	ø'orig.+ °	K	øe °	C' (psi)	ø' °	ø'orig.+ °	K	øe °
Low Pres. Range 0 — 150	10	22	—	0.069	20.1	0	16.5	—	0.04	11.9	2.5	24	—	0.10	13
Medium Pres. Range 150 — 500	62	10.5	16	"	"	18	12.5	13.5	"	"	21	18	20	"	"
Elevated Pressure Range 500 — 1000	70	10	13.7	"	"	20	12	13.2	"	"	21	18	19.1	"	"

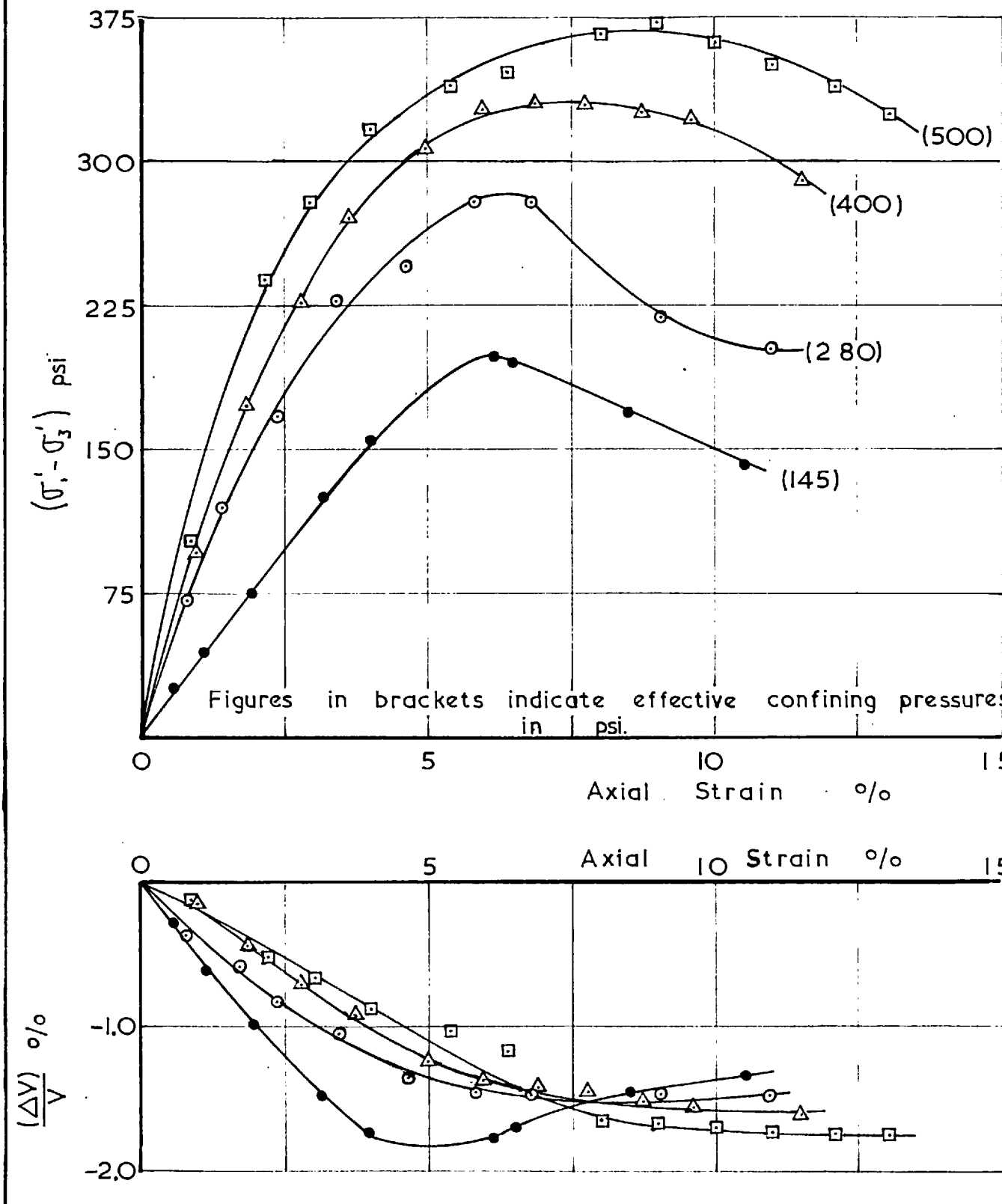
+ $\phi'_{orig.}$ was obtained from the tangent from the origin to the furthest stress circle.

Other values of C' & ø' were obtained by extrapolation of the relevant section of the failure envelope.



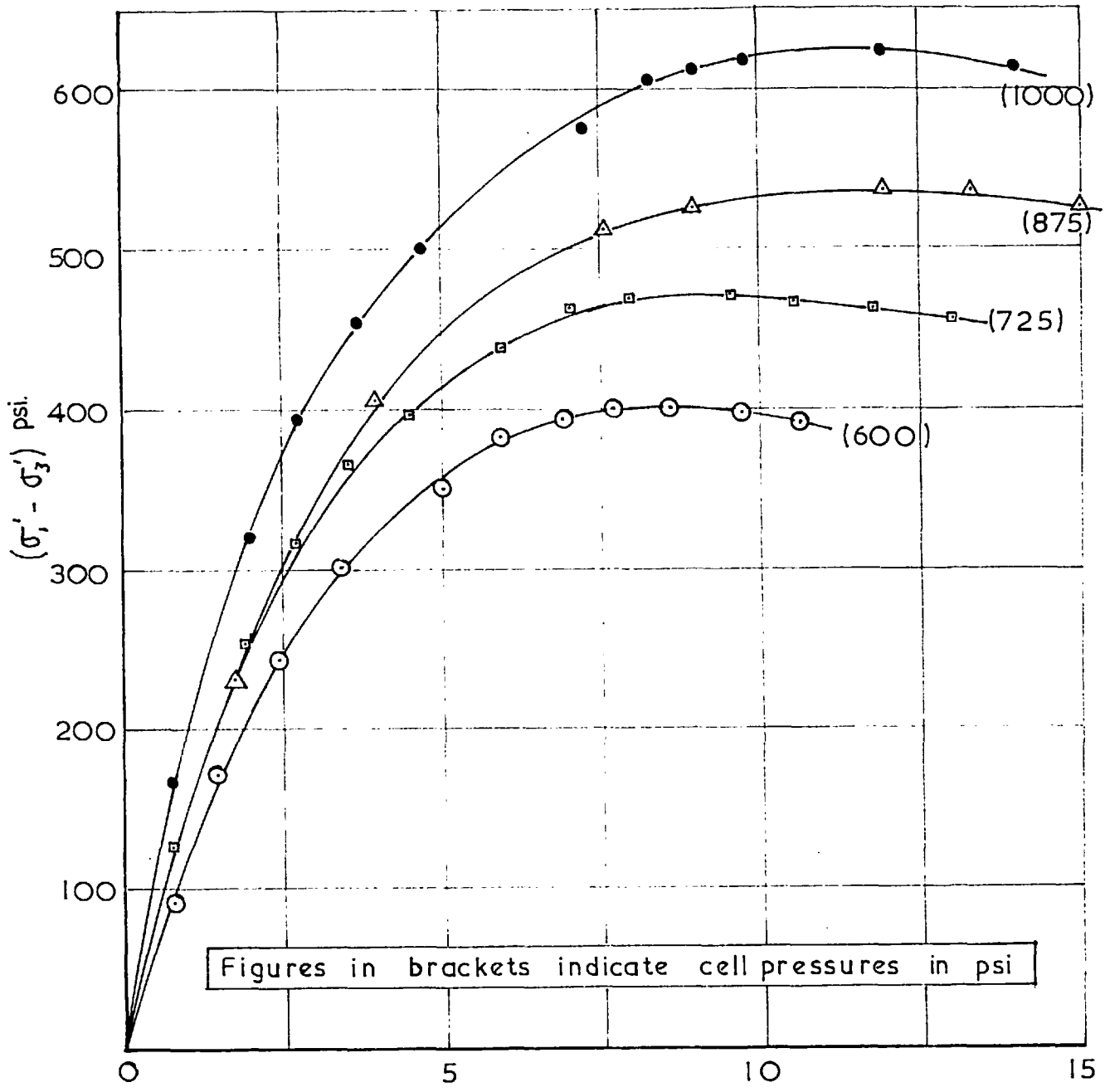
DRAINED TESTS ON INTACT SAMPLES OF UNDISTURBED LONDON CLAY.

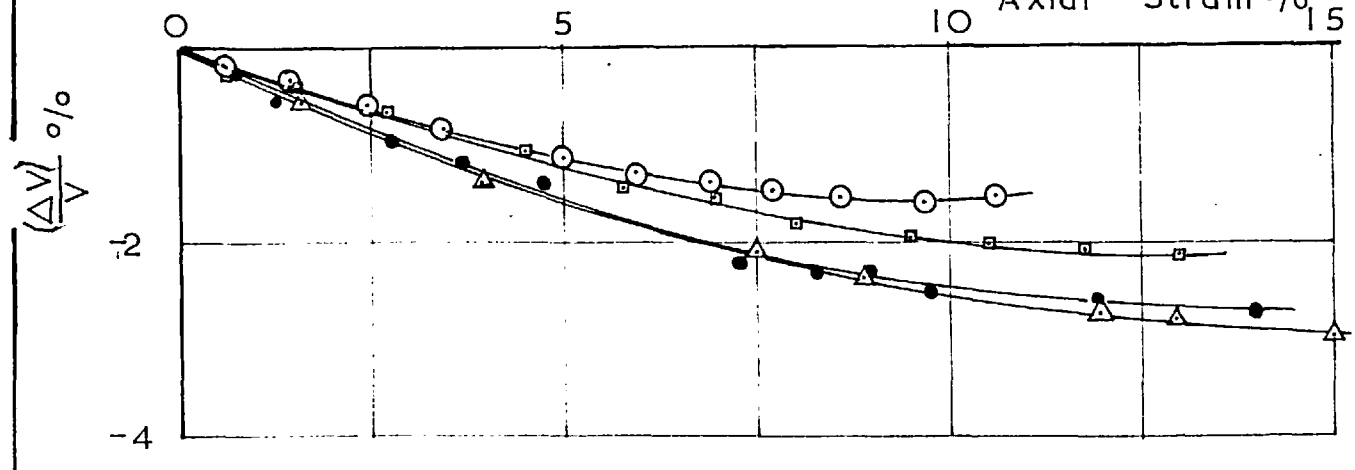
FIG. 7.1



DRAINED TESTS ON INTACT SAMPLES OF UNDISTURBED LONDON CLAY.

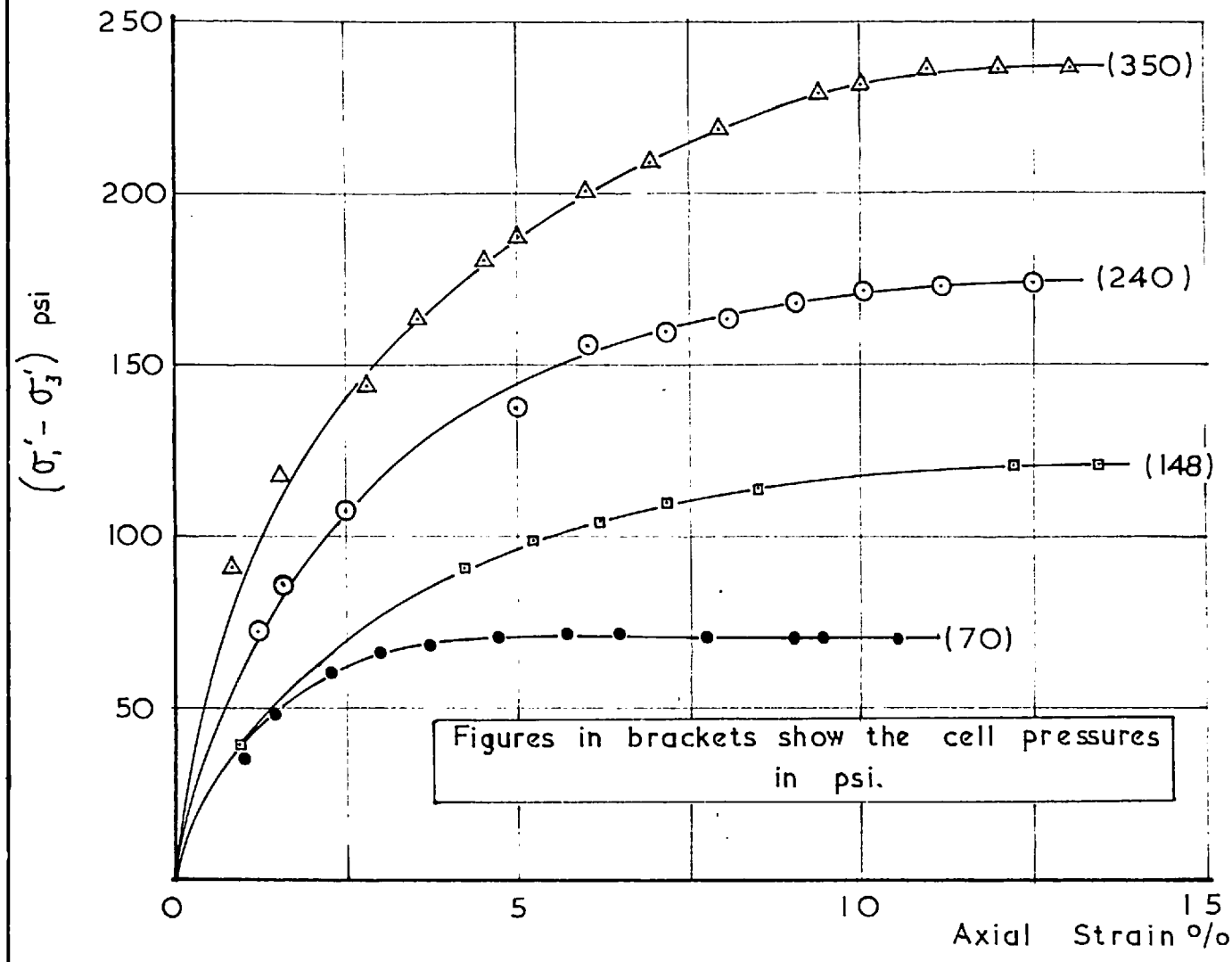
FIG. 7.2

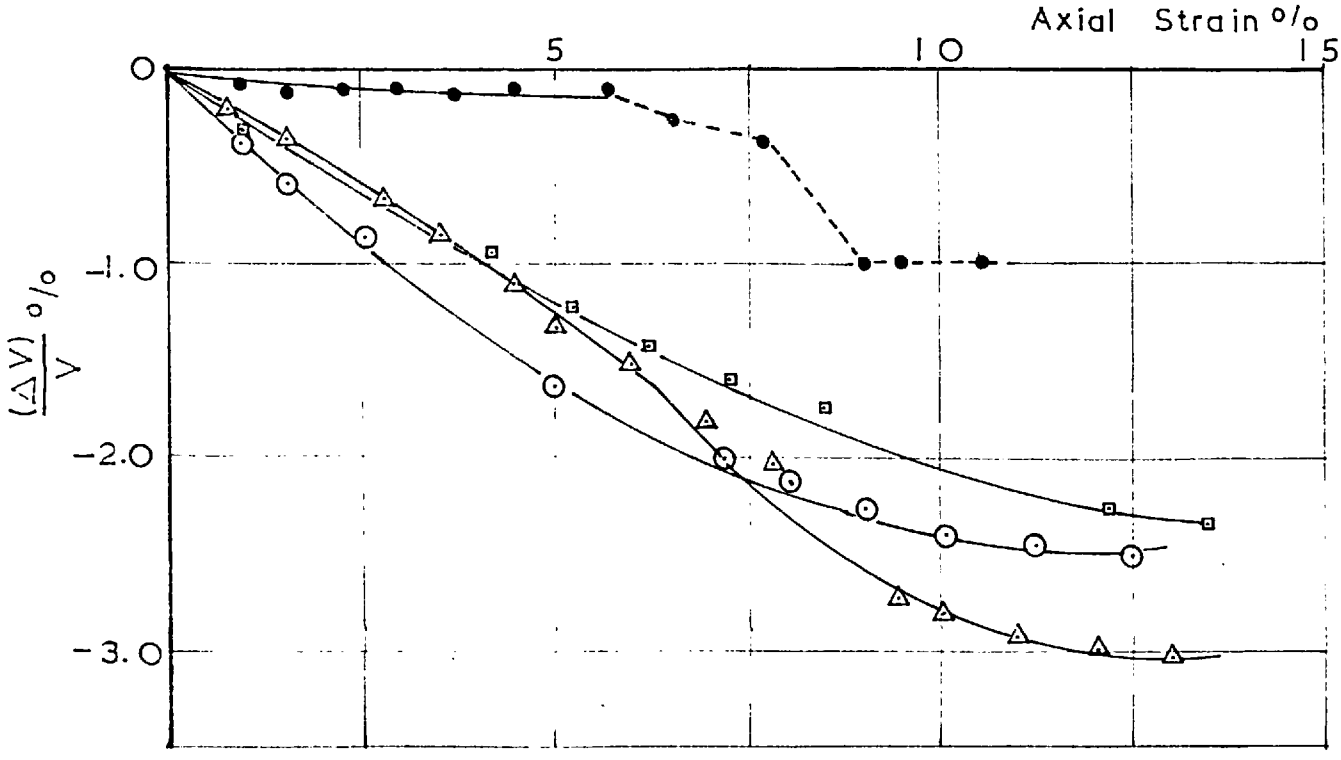




DRAINED TESTS ON INTACT SAMPLES OF UNDISTURBED LONDON CLAY.

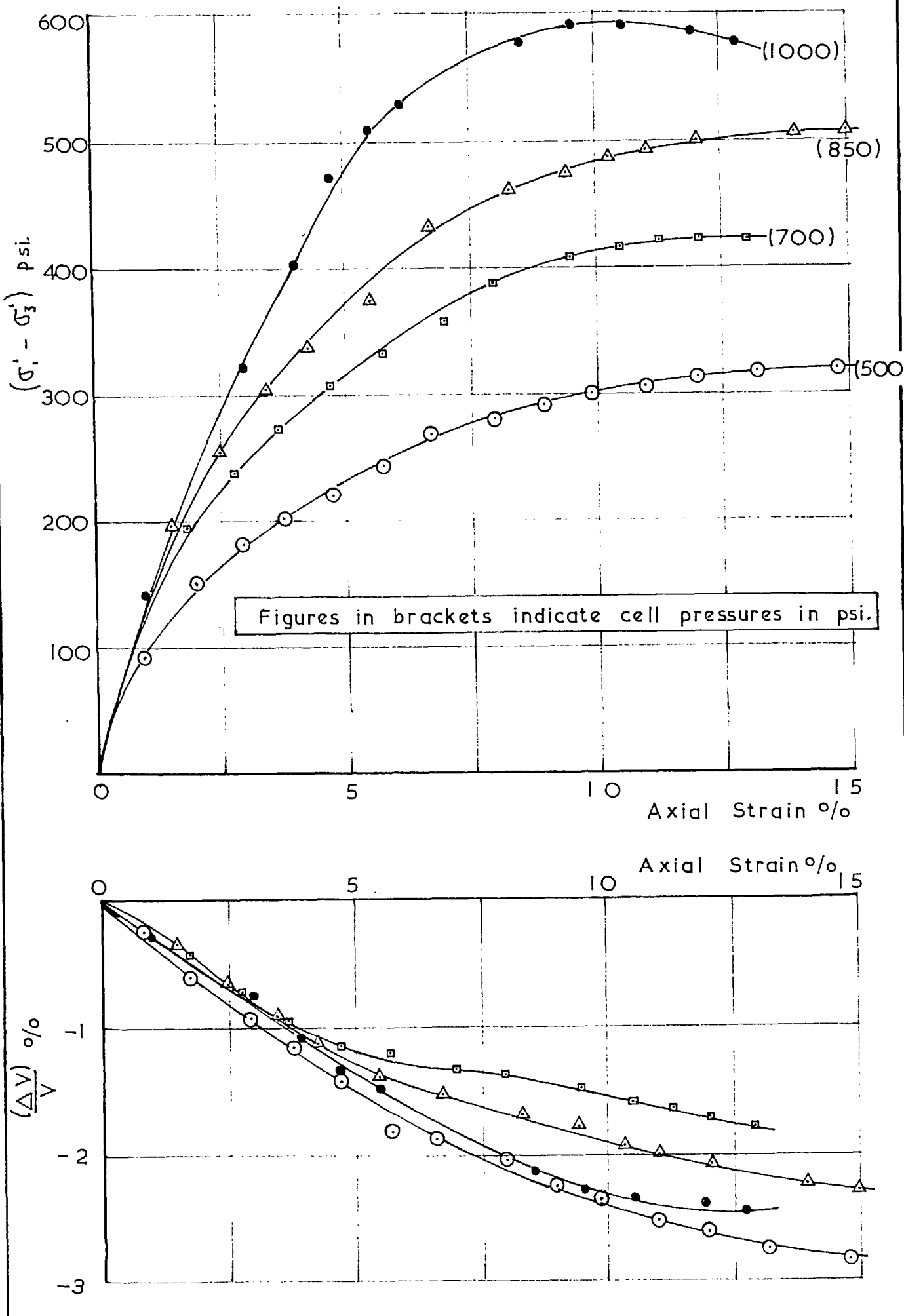
FIG. 7.3





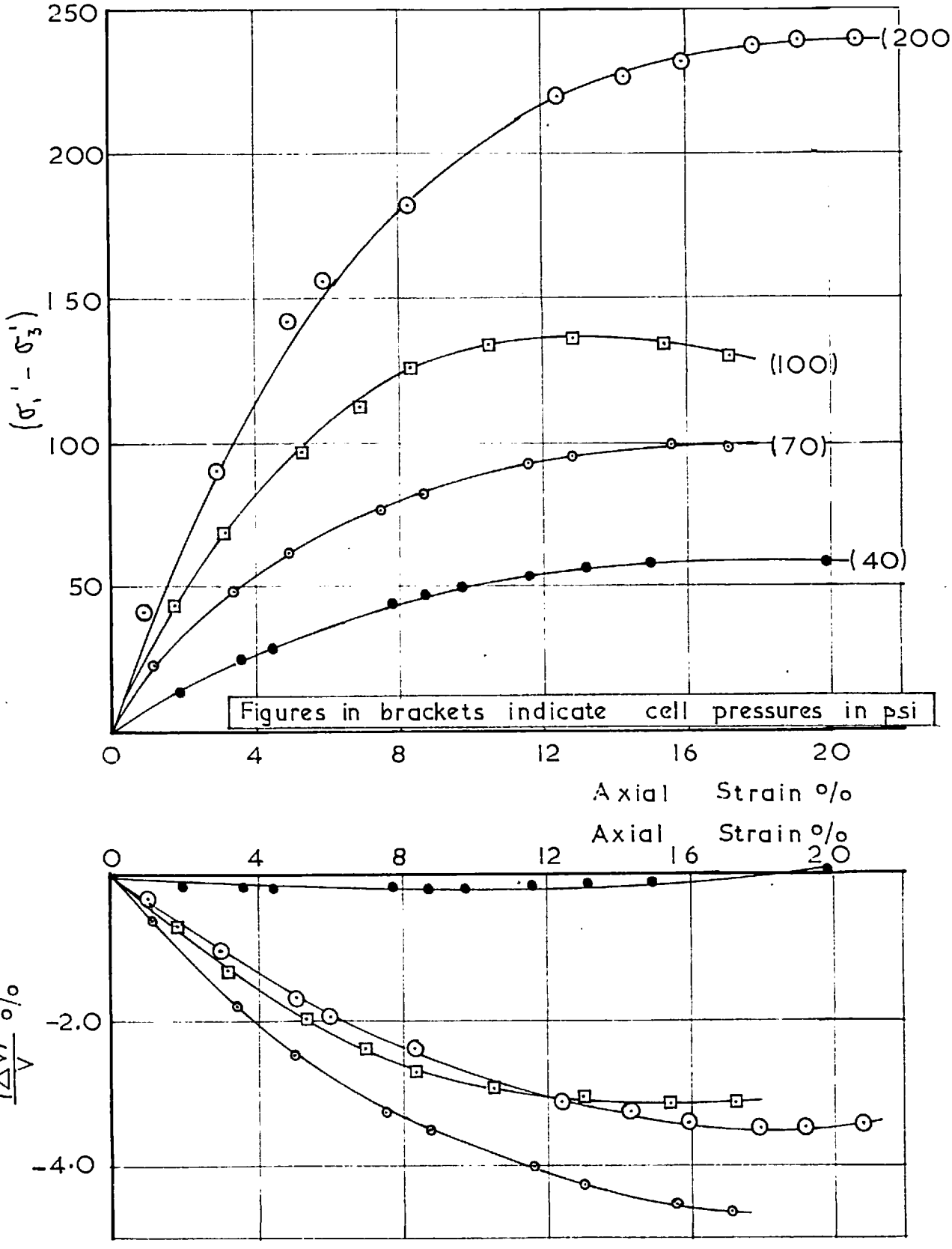
DRAINED TESTS ON REMOULDED LONDON CLAY.

FIG. 7.4



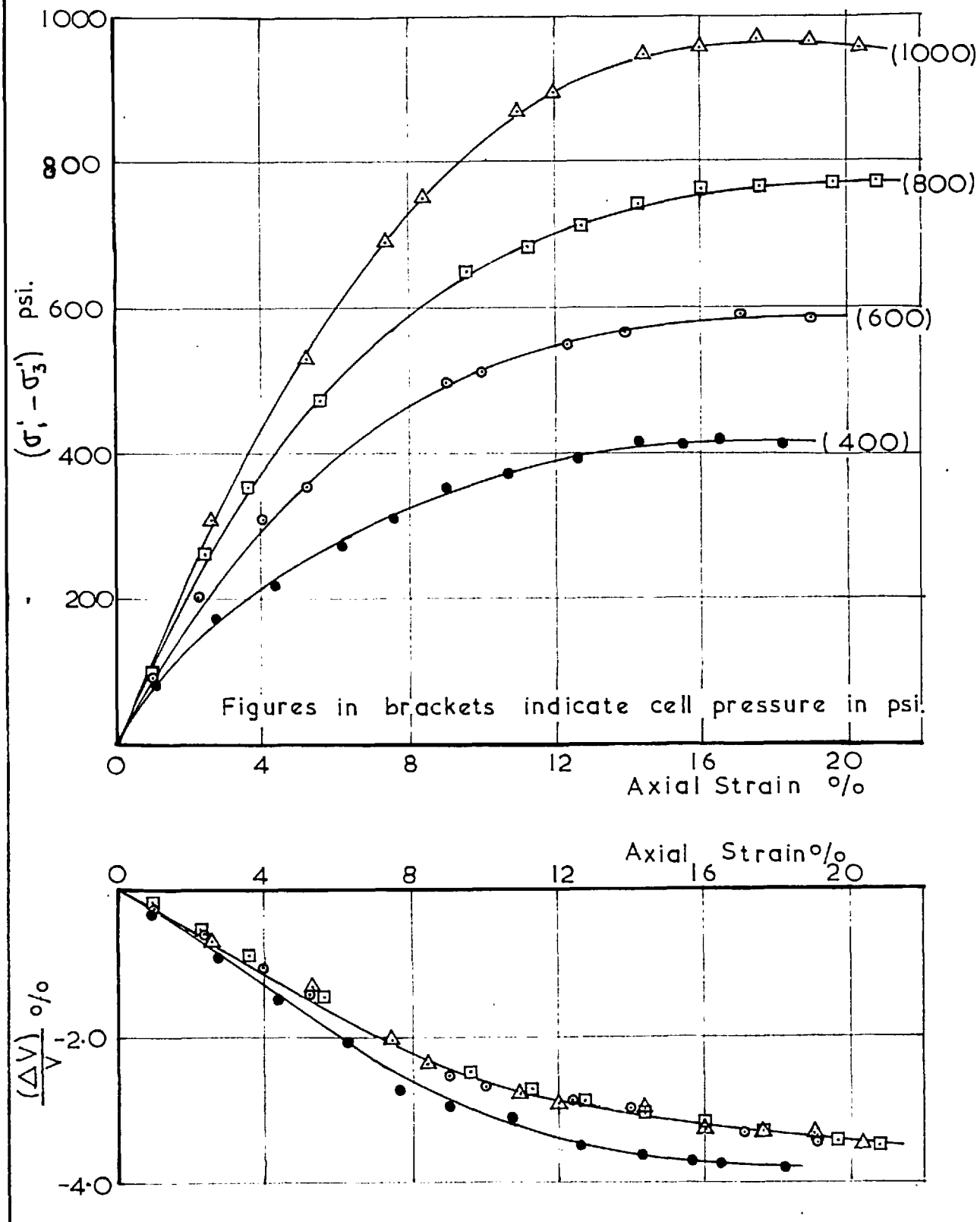
DRAINED TESTS ON REMOULDED LONDON CLAY.

FIG. 7.5



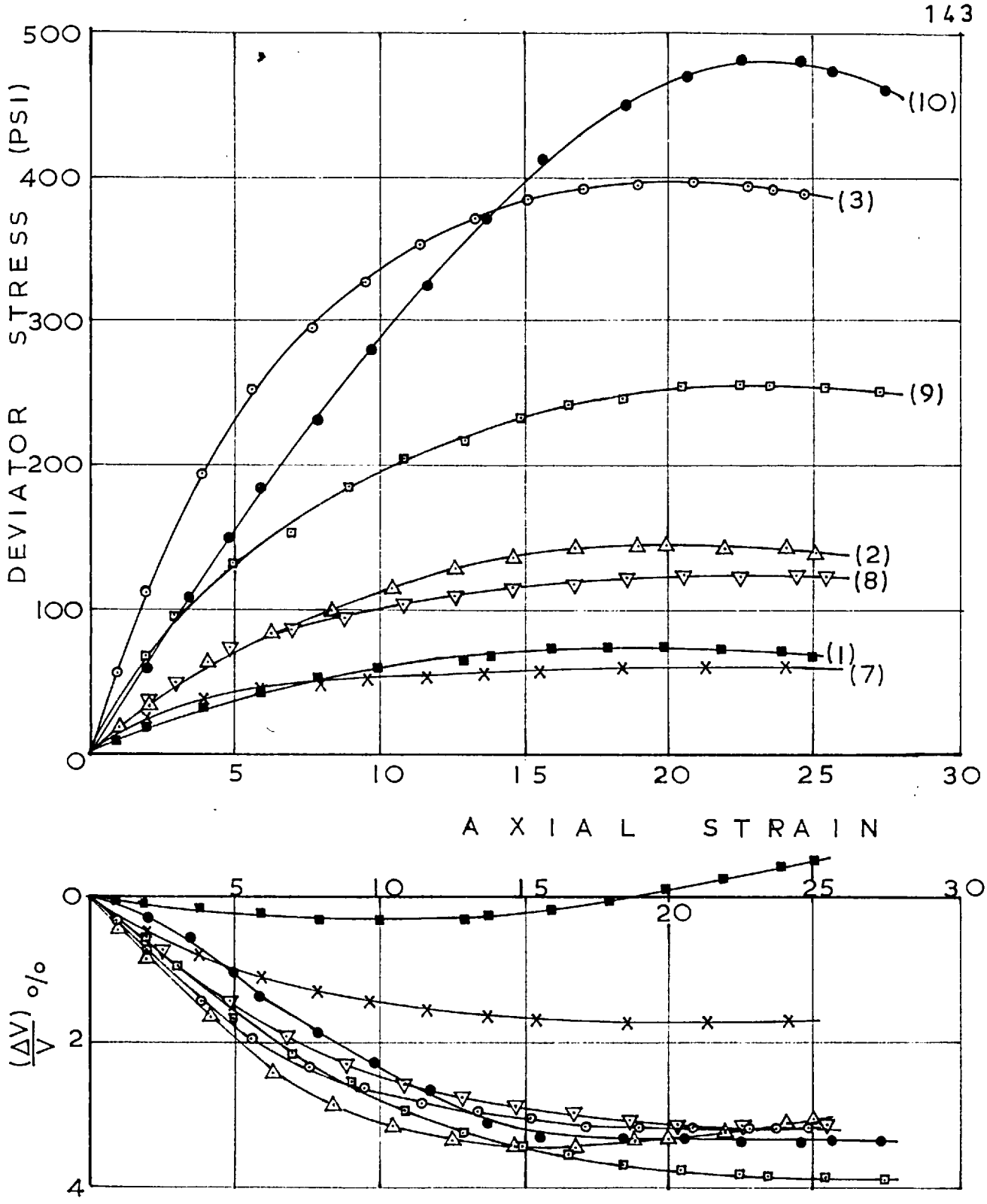
DRAINED TESTS ON REMOULDED WIENER TEGEL.

FIG. 7.6



DRAINED TESTS ON REMOULDED WIENER TEGEL.

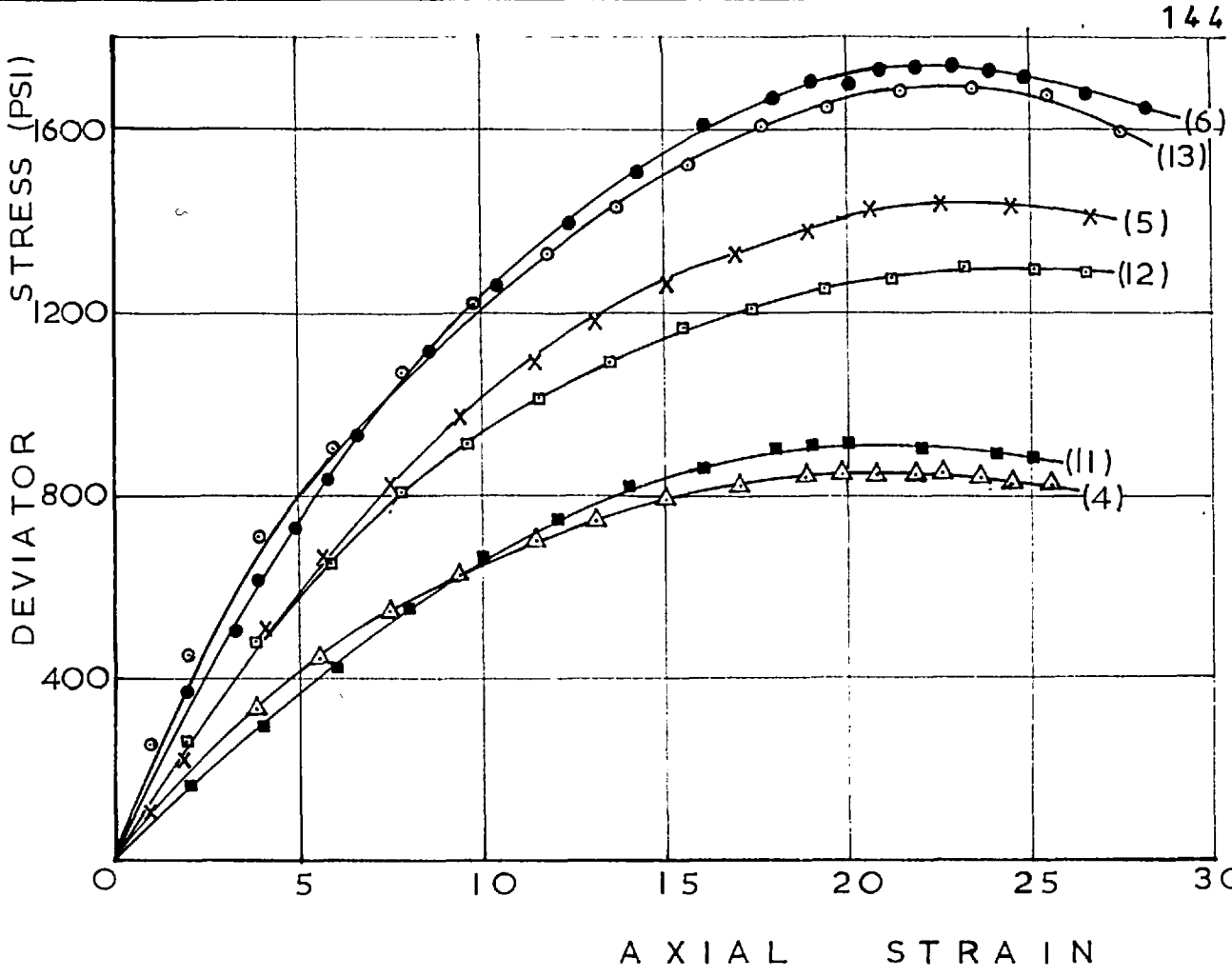
FIG. 7.7

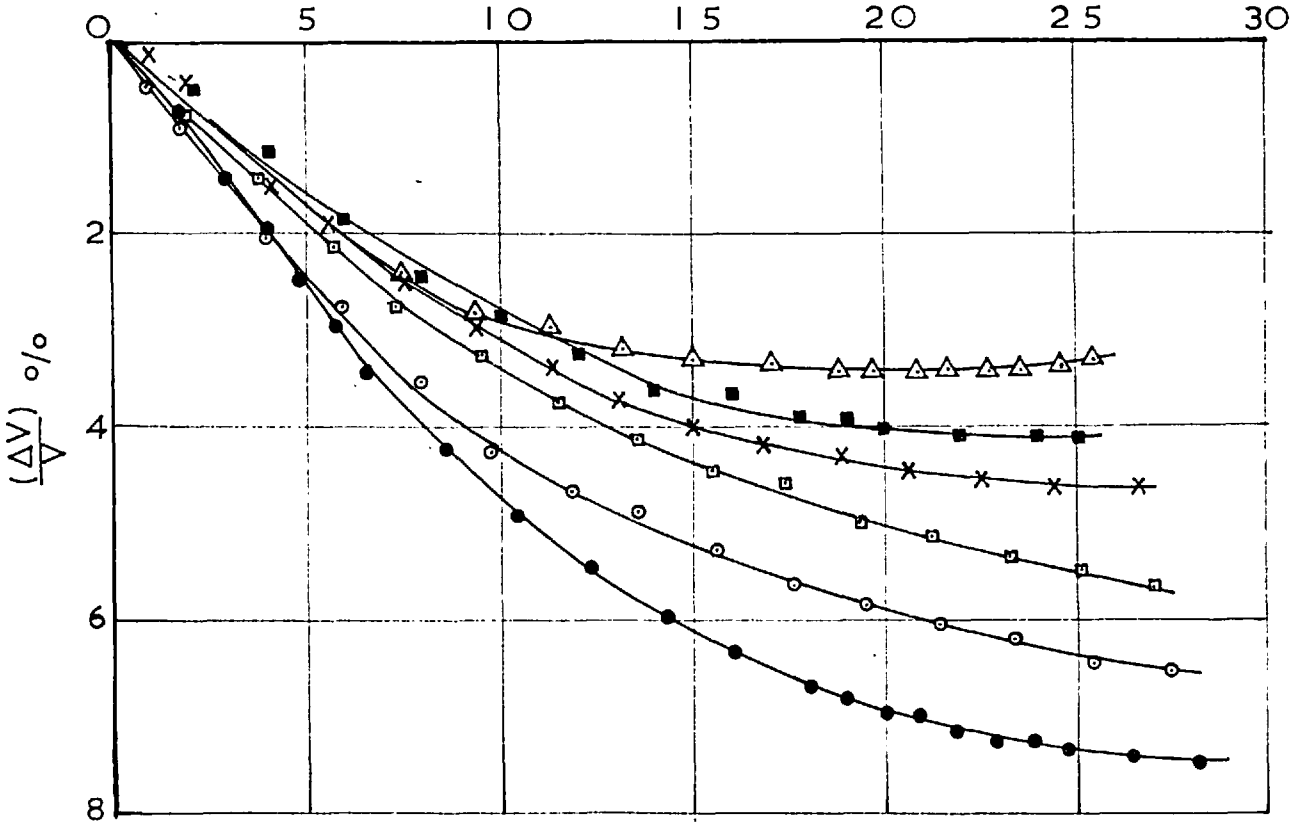


Test No.	1	2	3	7	8	9	10
Initial Porosity, %	39.5	40.0	40.5	44.7	43.6	44.1	43.6
Confining Pressure, (psi)	30	60	175	30	60	125	250

DRAINED TESTS ON BRAEHEAD SILT.

FIG. 7.8

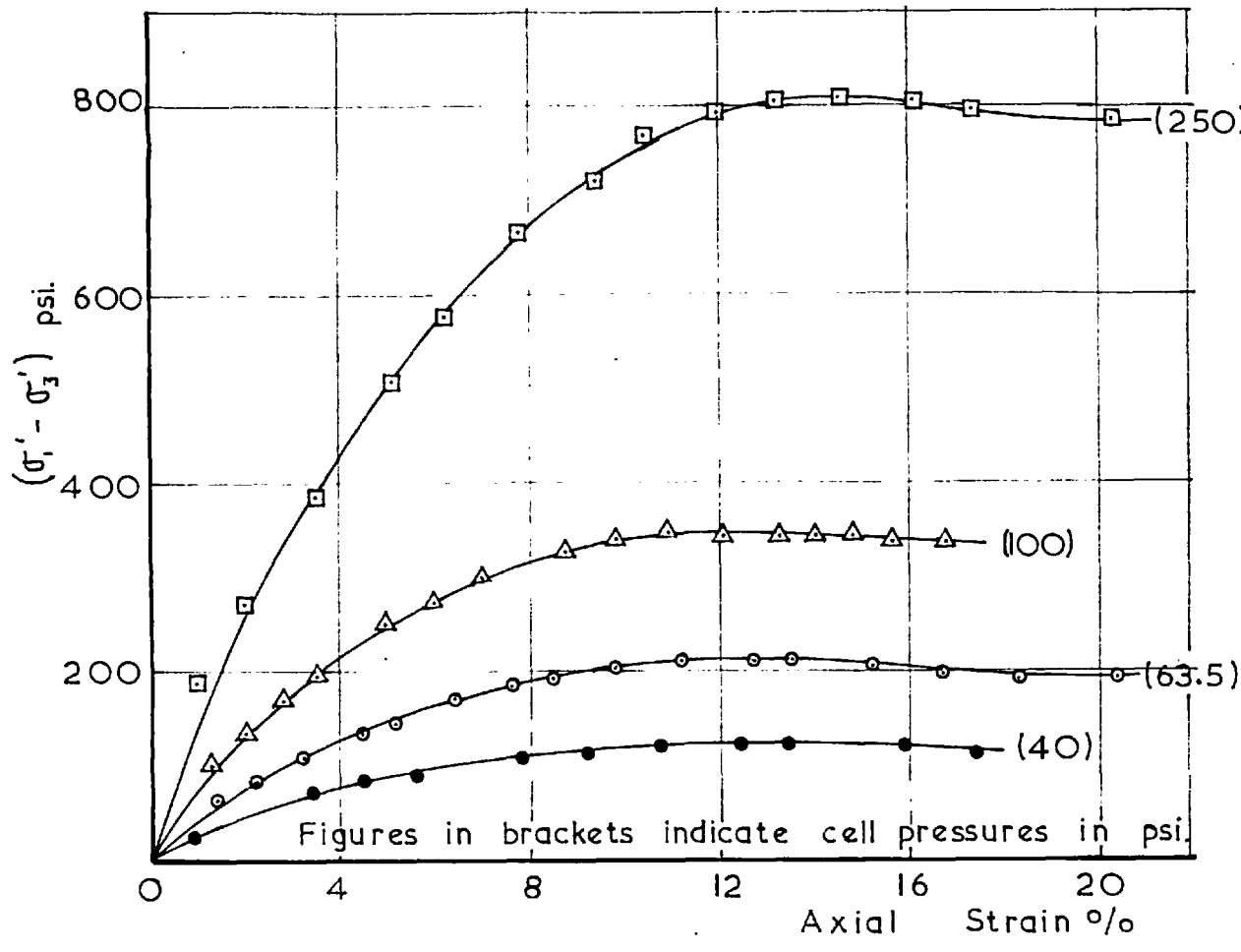


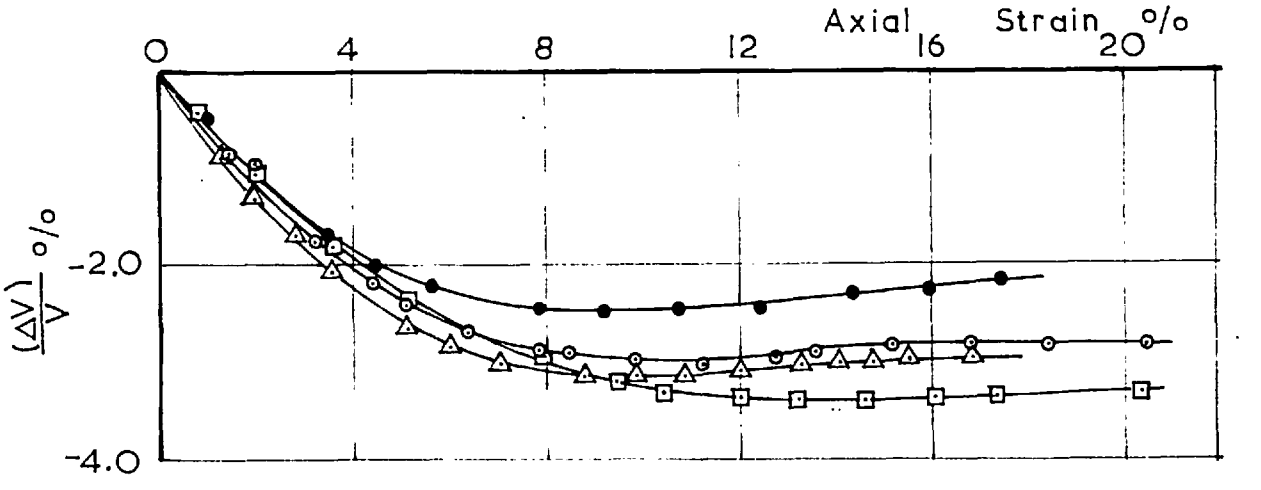


Test No.	4	5	6	11	12	13
Initial Porosity, %	40.0	40.5	40.9	43.9	43.7	44.3
Confining Pressure, (psi)	400	750	1000	500	750	1000

DRAINED TESTS ON BRAEHEAD SILT.

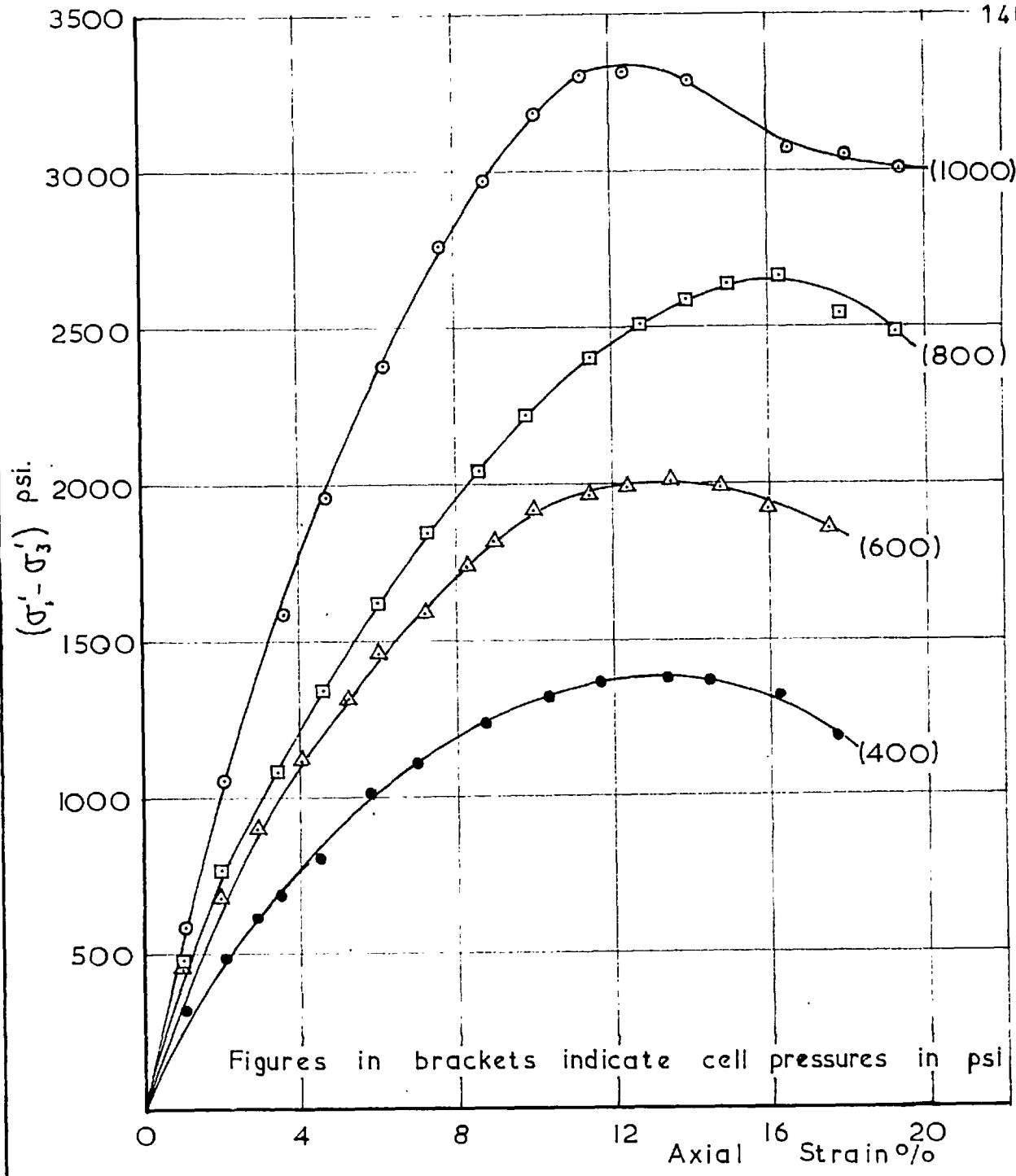
FIG. 7.9





DRAINED TESTS ON CALCITE.

FIG .7 .10



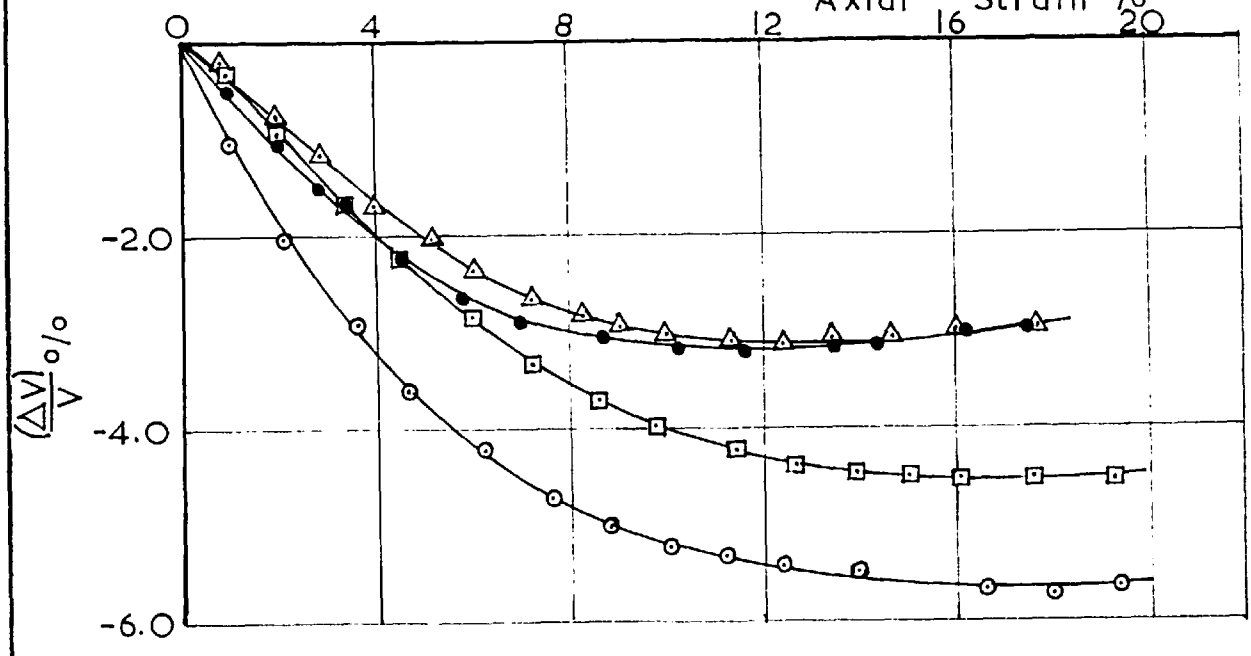
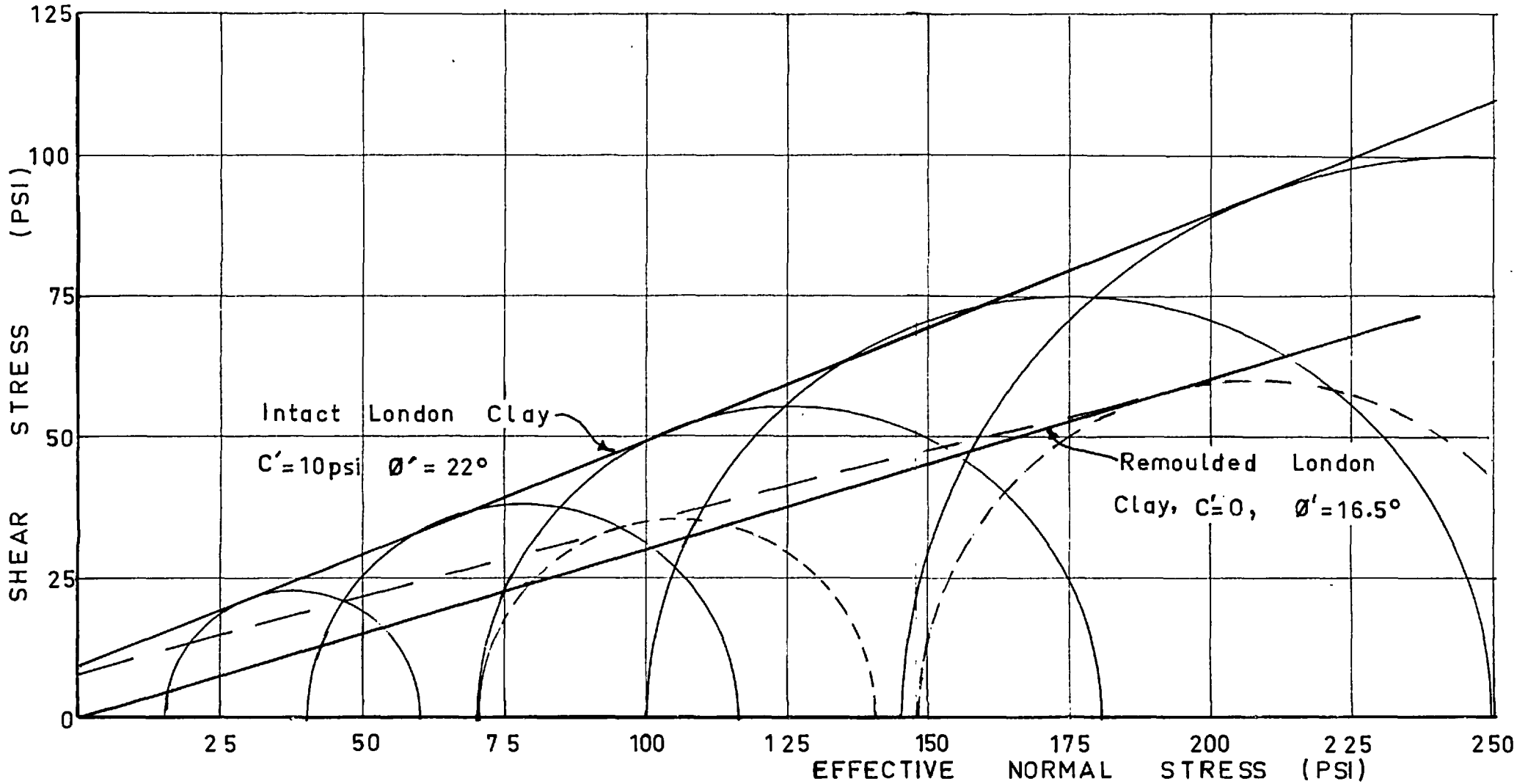


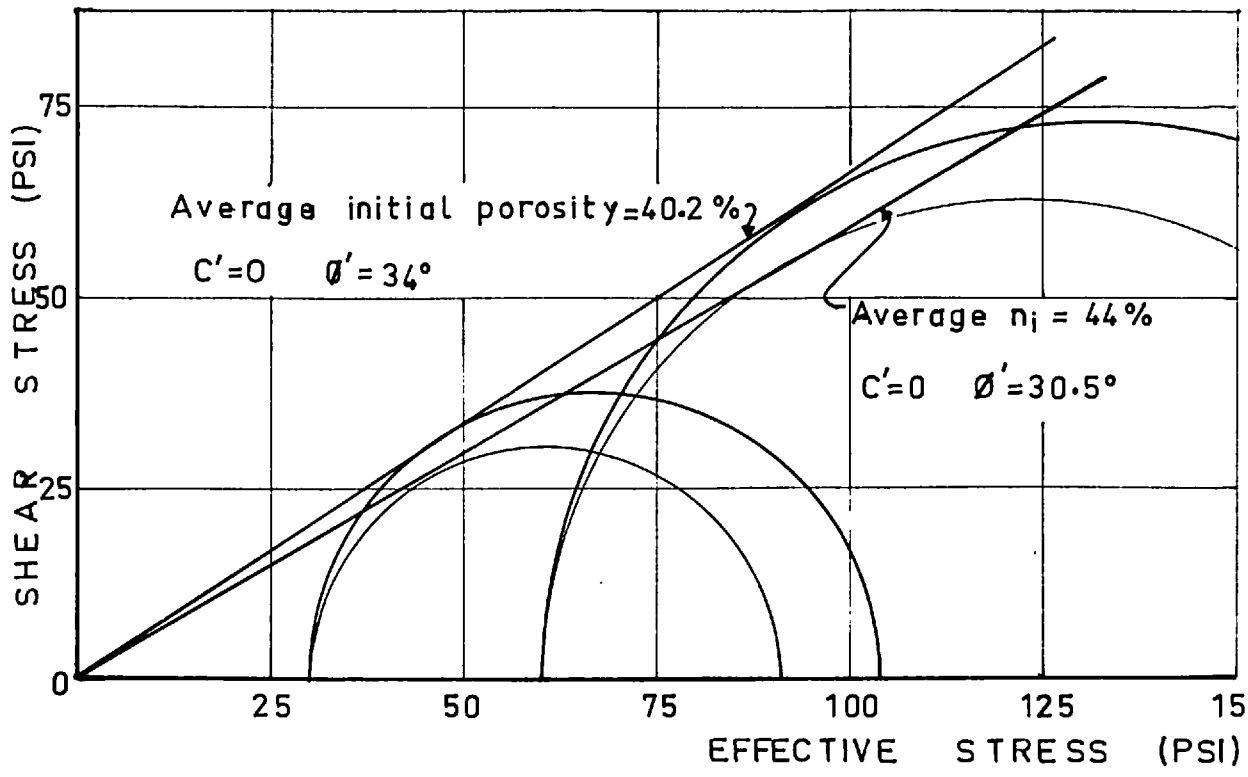
FIG. 7.11

DRAINED TESTS ON CALCITE.

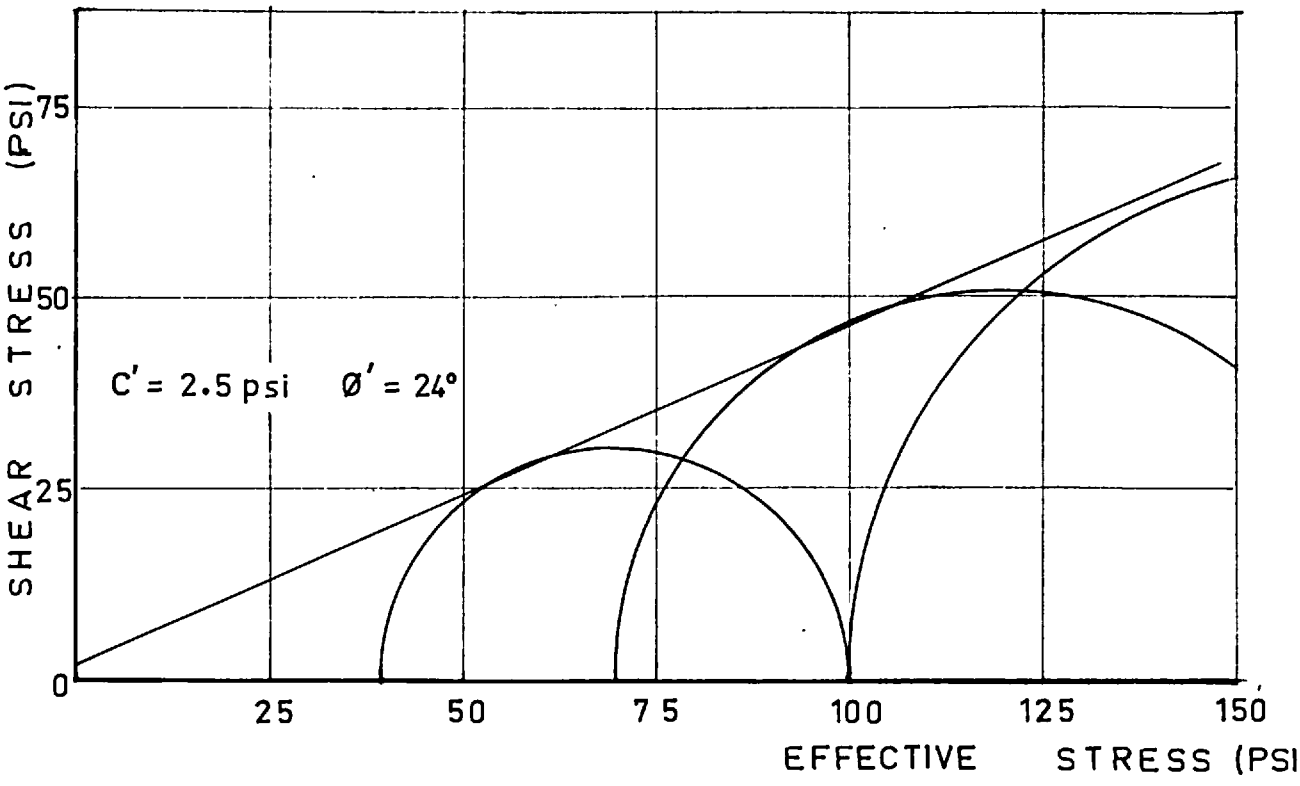


LOW - PRESSURE EFFECTIVE STRESS MOHR FAILURE ENVELOPE FOR LONDON CLAY.

FIG. 7. 12



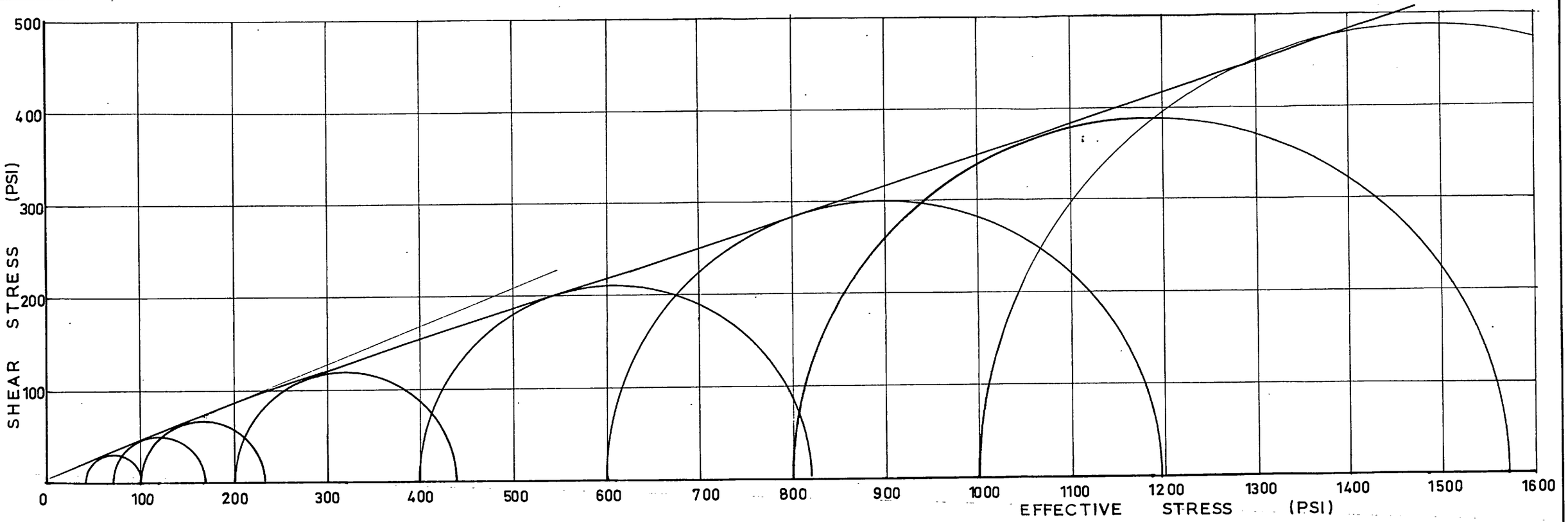
(A) BRAEHEAD SILT.



(B) WIENER TEGEL.

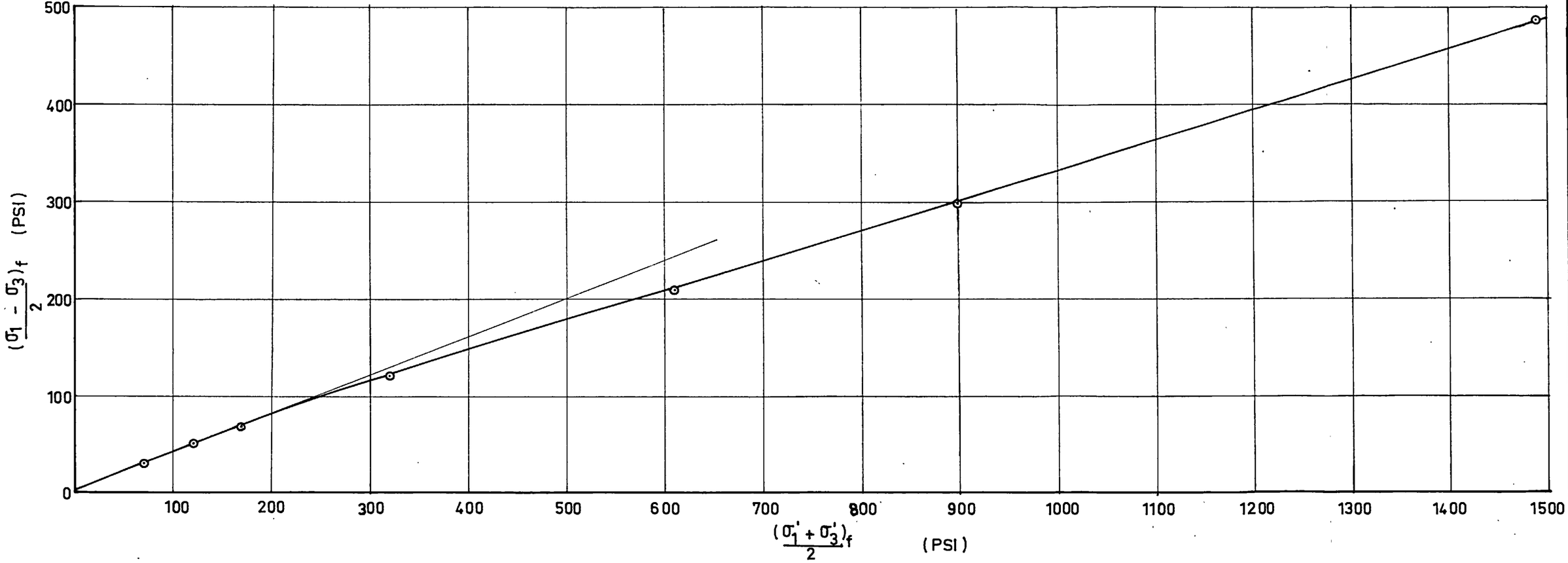
LOW PRESSURE MOHR FAILURE ENVELOPES.

FIG. 7.13



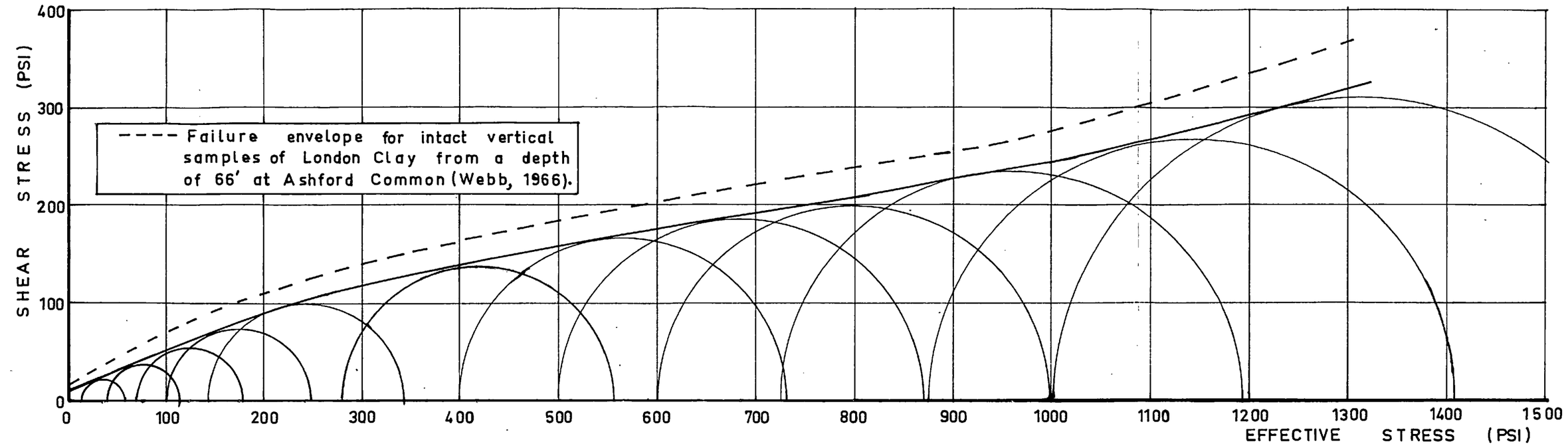
DRAINED TESTS ON REMOULDED WIENER TEGEL.

FIG. 7.14

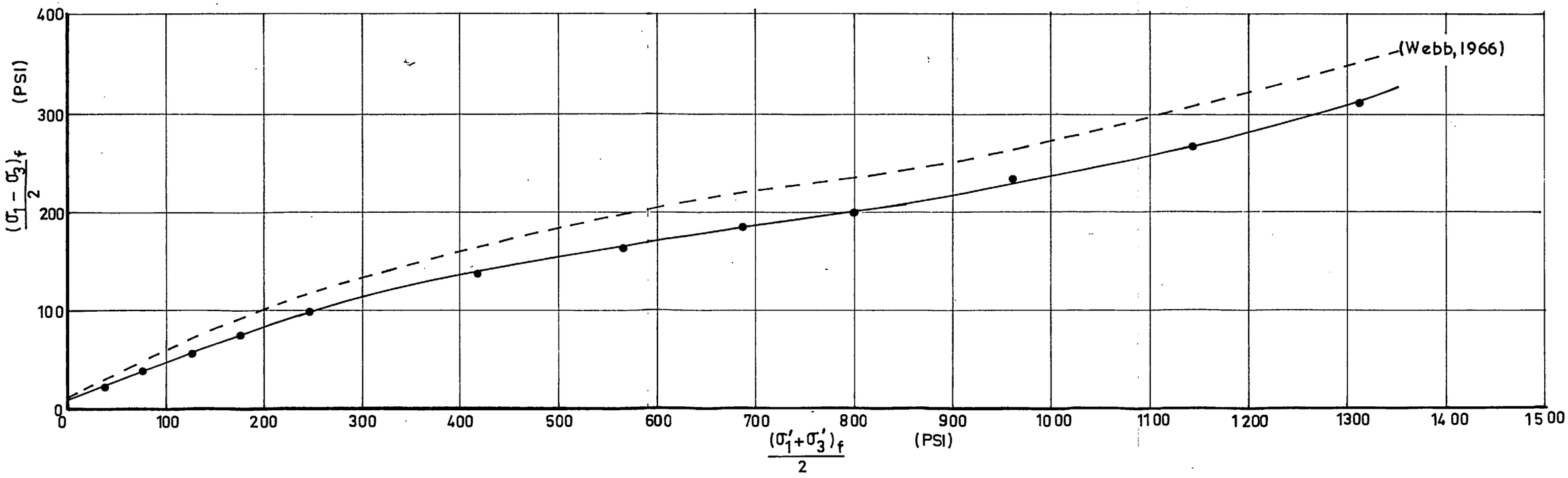


DRAINED TESTS ON REMOULDED WIENER TEGEL.

FIG. 7.15

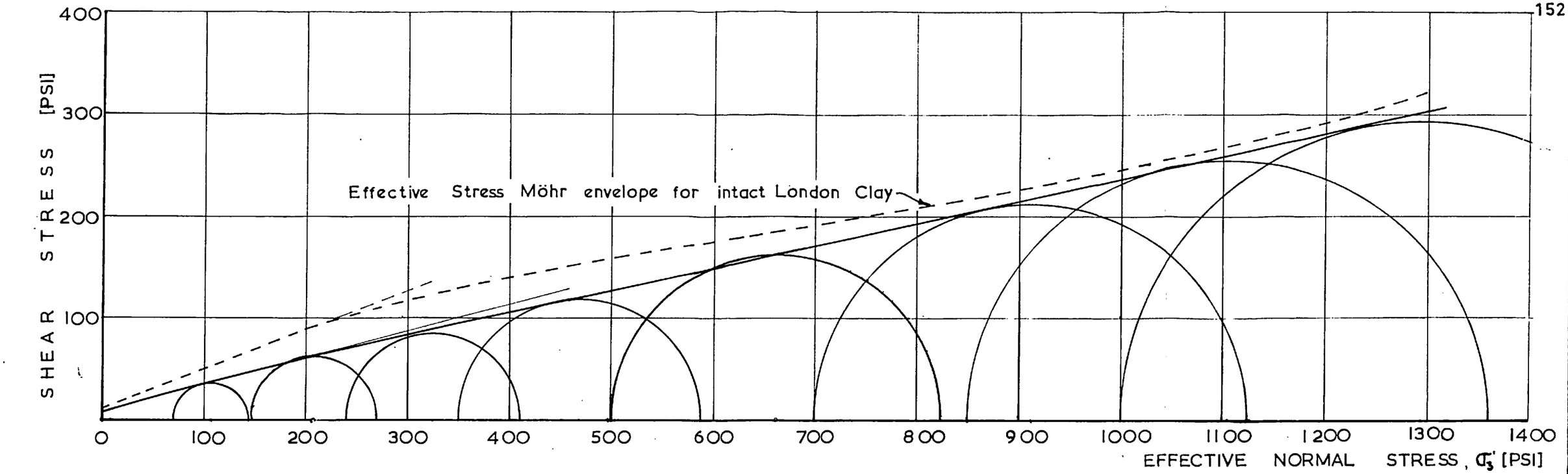


MOHR FAILURE ENVELOPE.

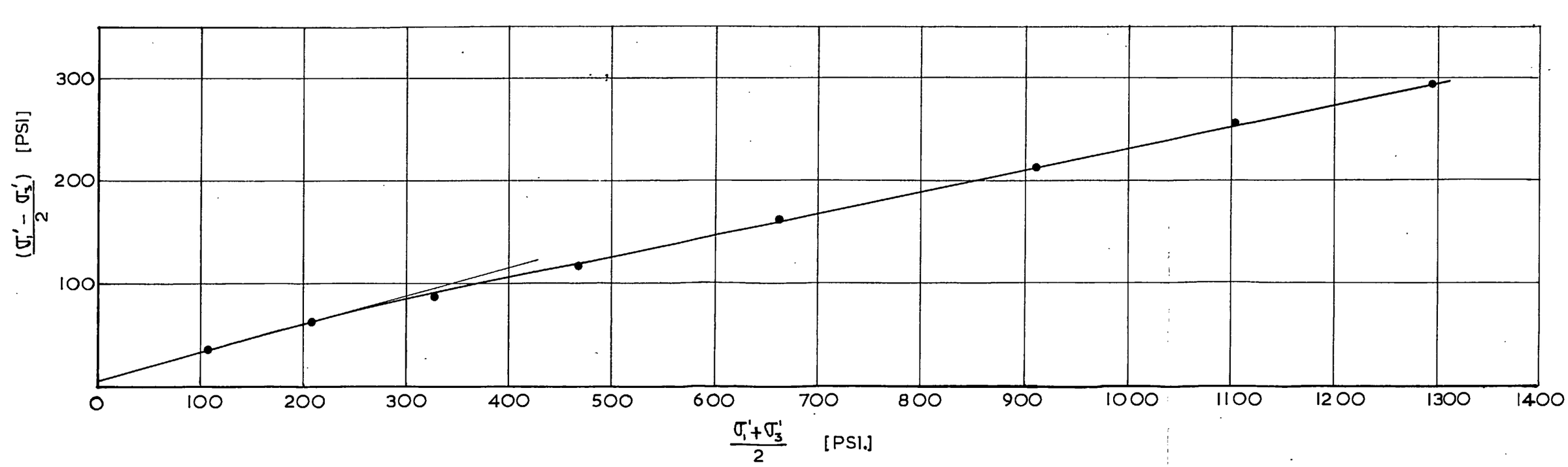


DRAINED TESTS ON INTACT SAMPLES OF LONDON CLAY.

FIG. 7.16



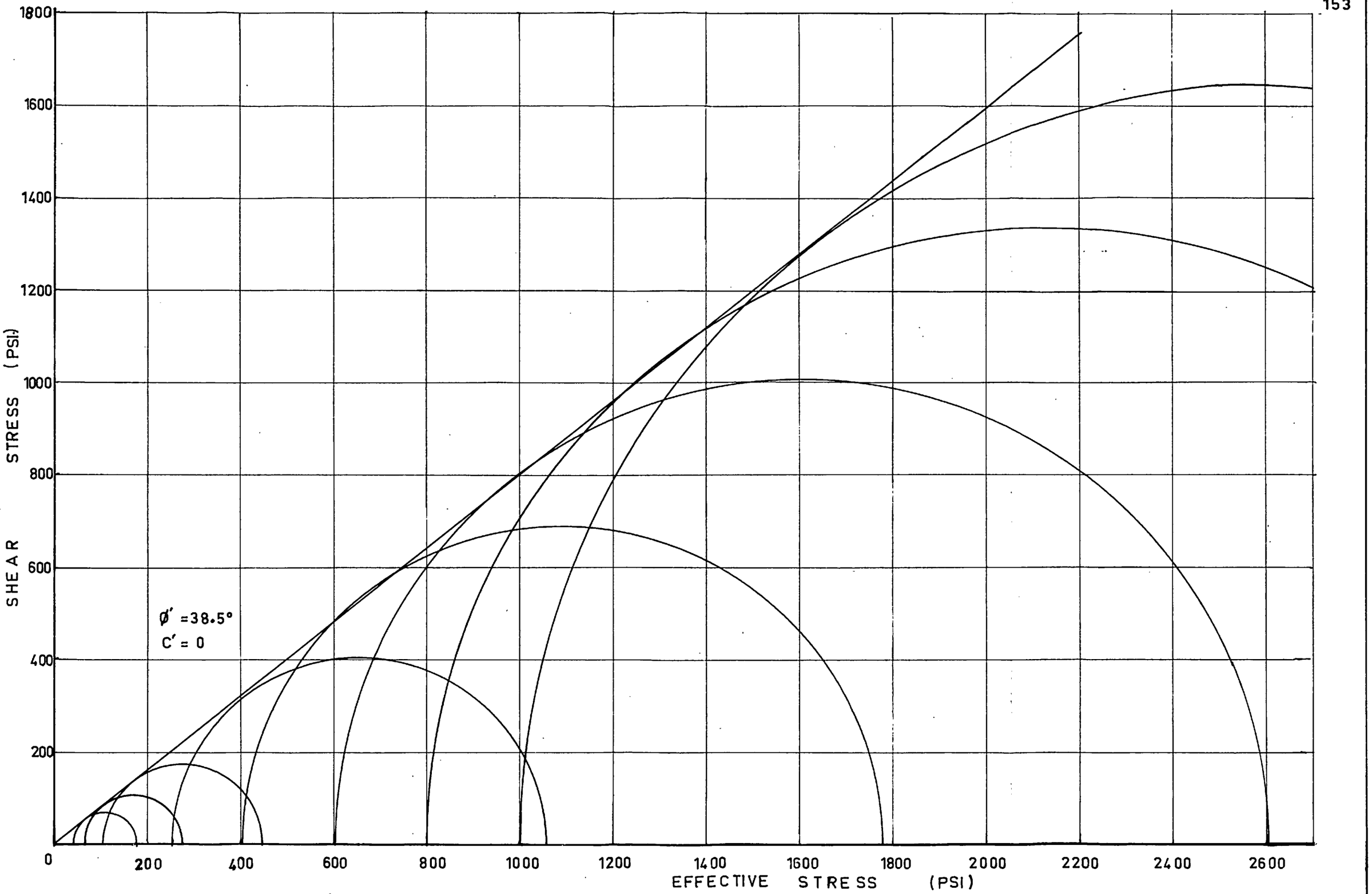
(a) MOHR ENVELOPE.



(b) MODIFIED MOHR ENVELOPE.

DRAINED TEST ON REMOULDED LONDON CLAY.

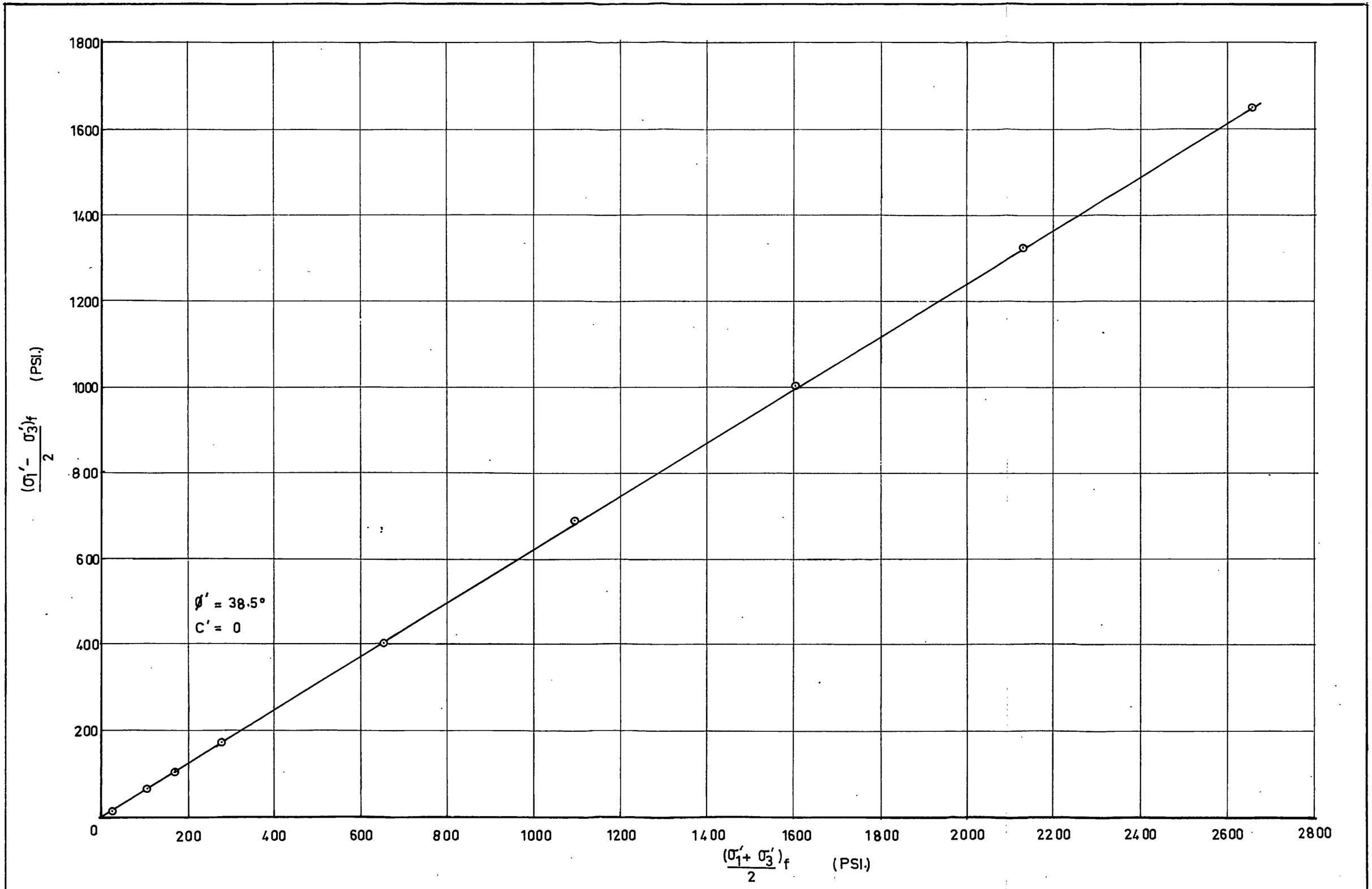
FIG. 7.17



EFFECTIVE STRESS MOHR ENVELOPE.

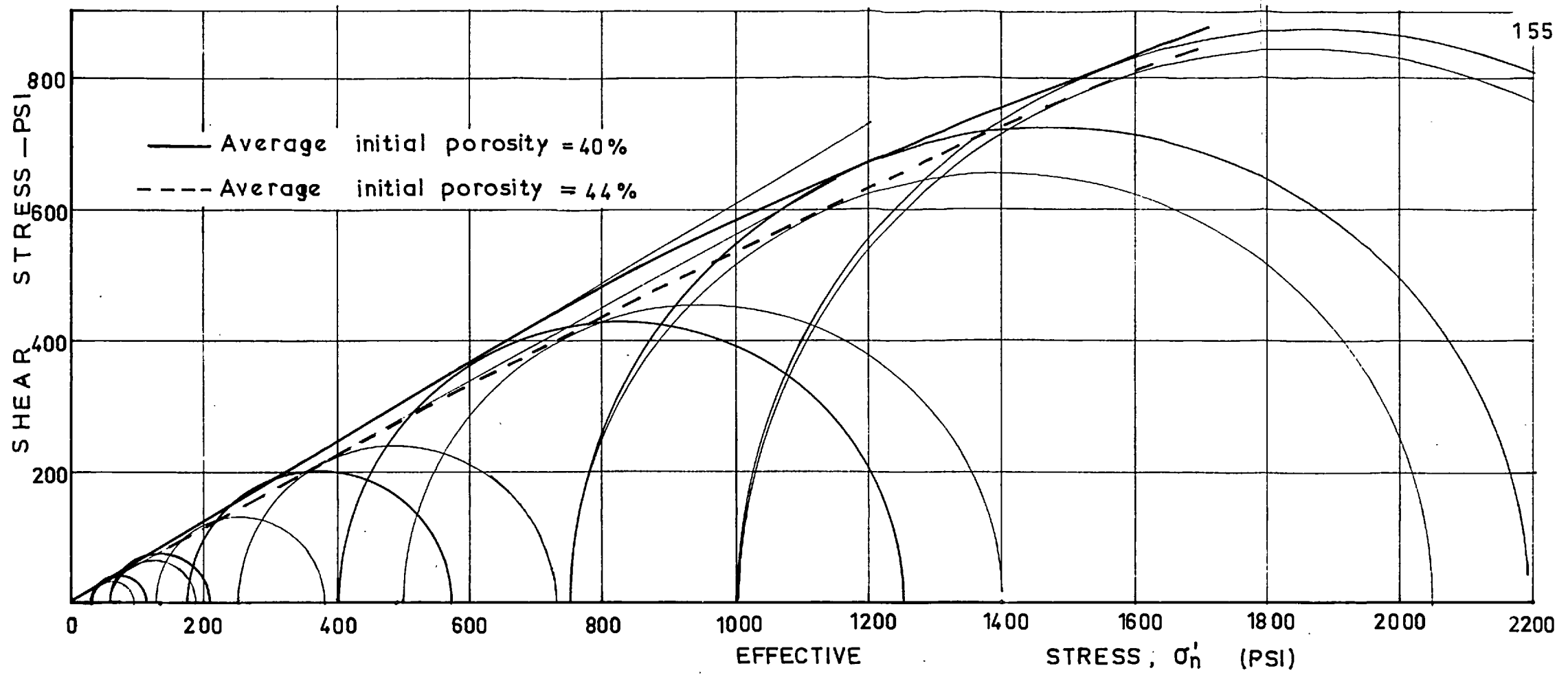
DRAINED TESTS ON CALCITE.

FIG. 7.18

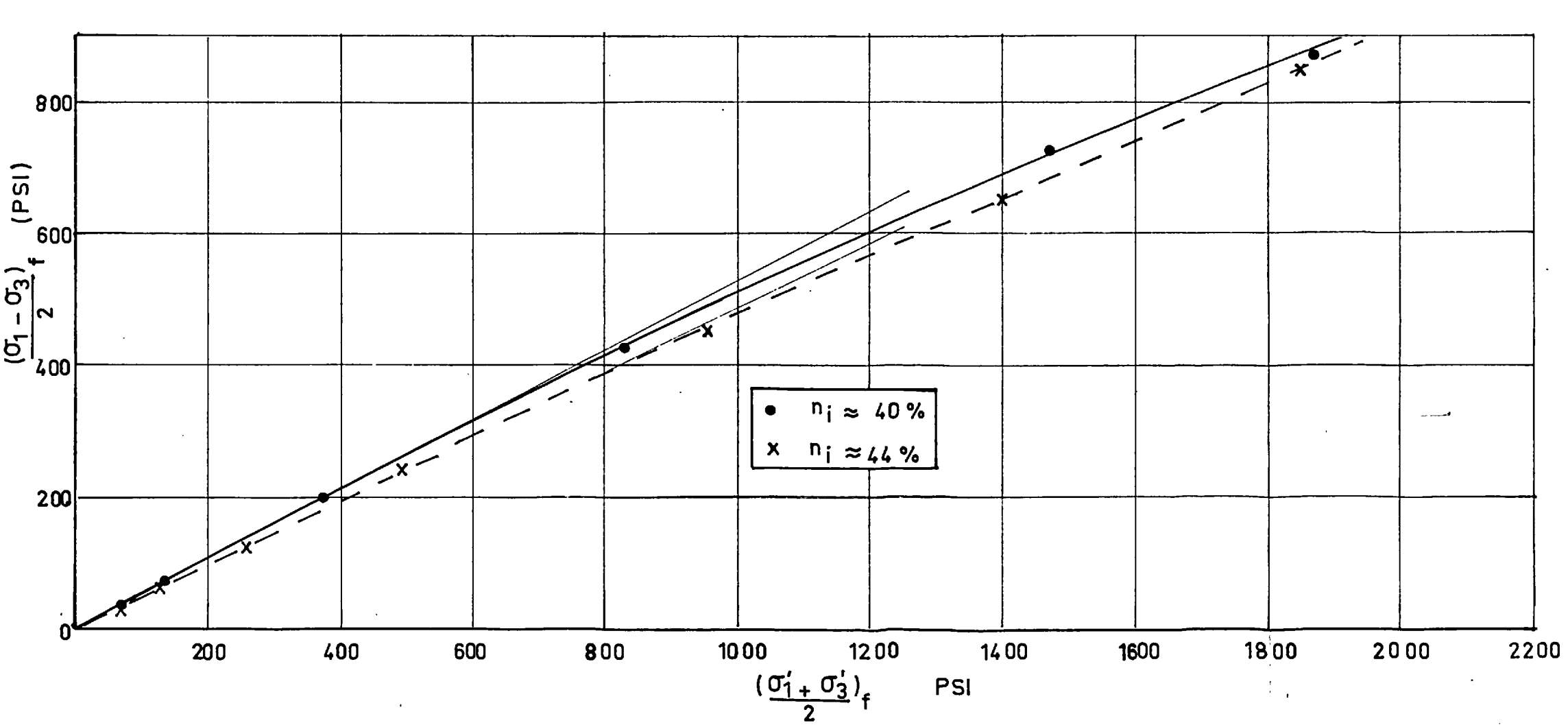


DRAINED TESTS ON CALCITE.

FIG. 7.19

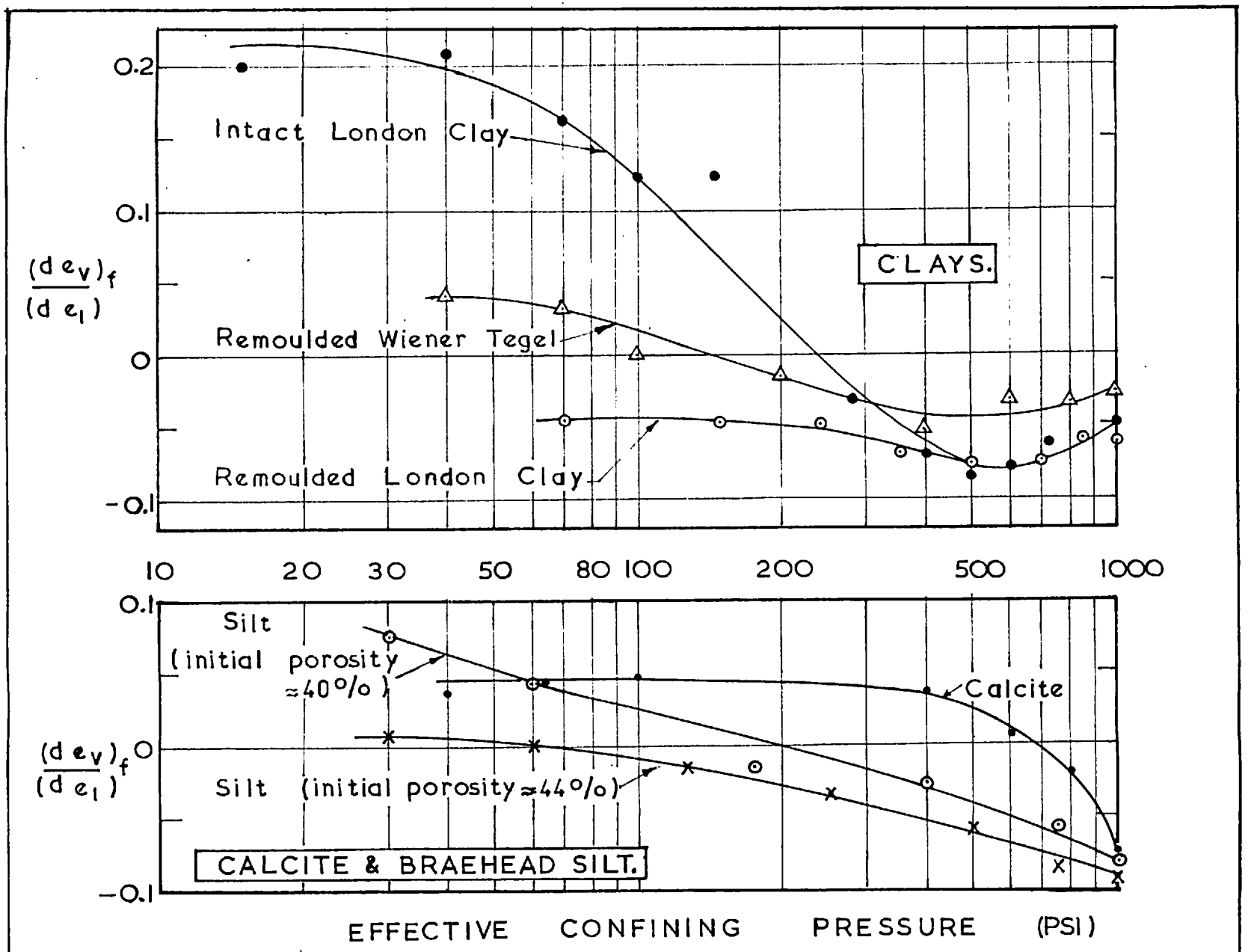


MOHR FAILURE ENVELOPE.



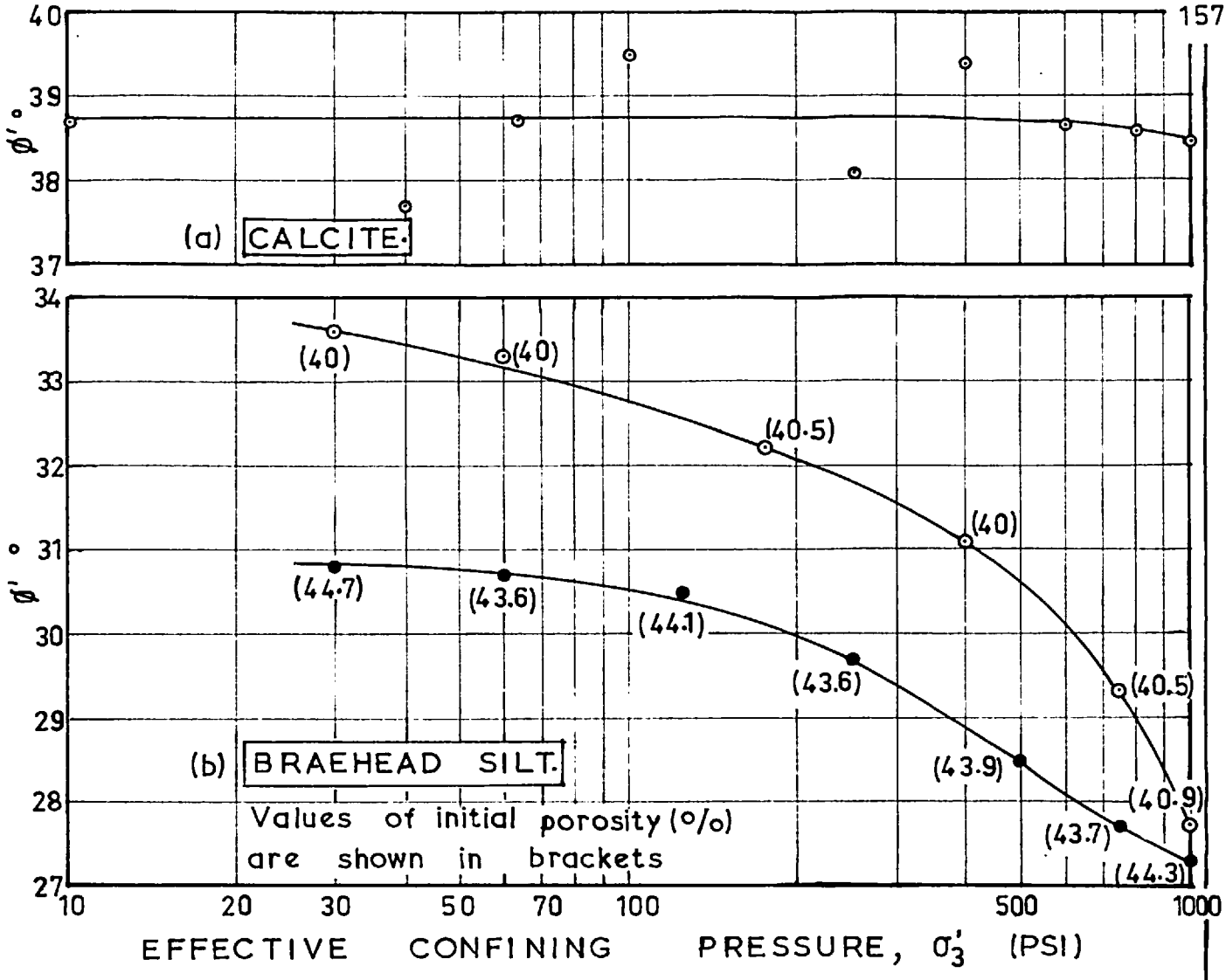
DRAINED TESTS ON BRAEHEAD SILT.

FIG. 7.20



VARIATION OF THE RATE OF VOLUME CHANGE AT FAILURE $\frac{(d e_v)_f}{(d e_1)}$ WITH CONFINING PRESSURE.

FIG. 7.21



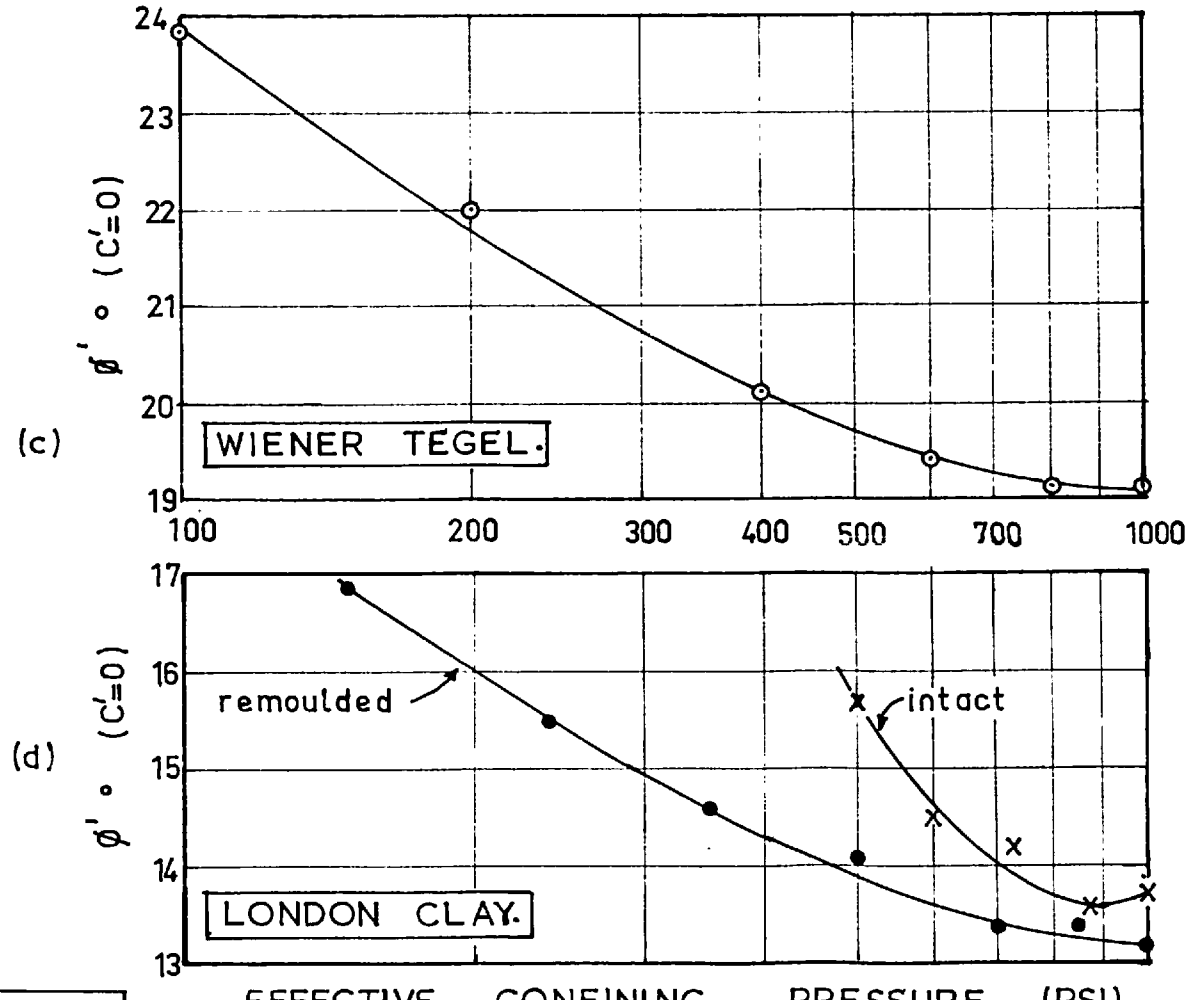
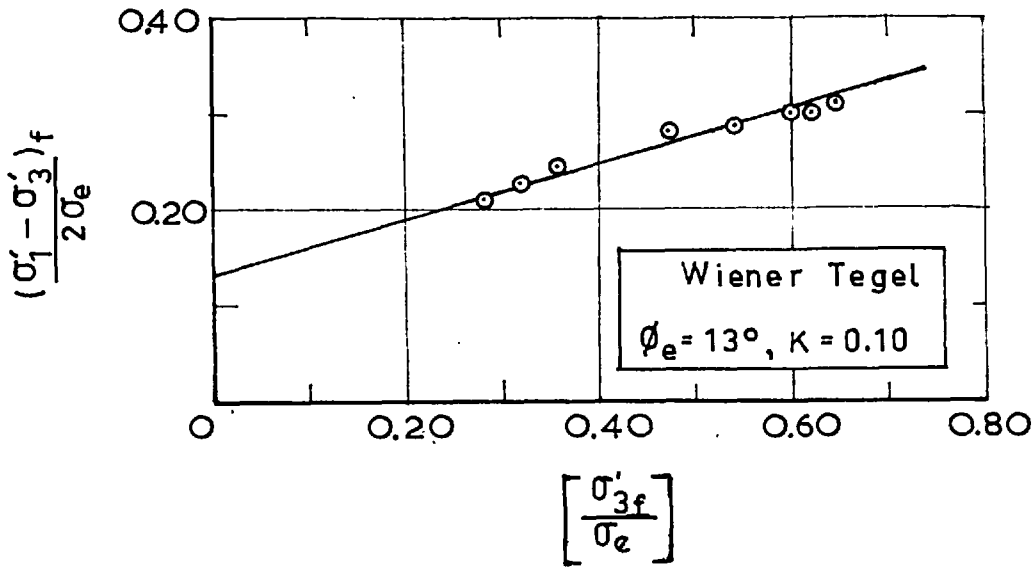
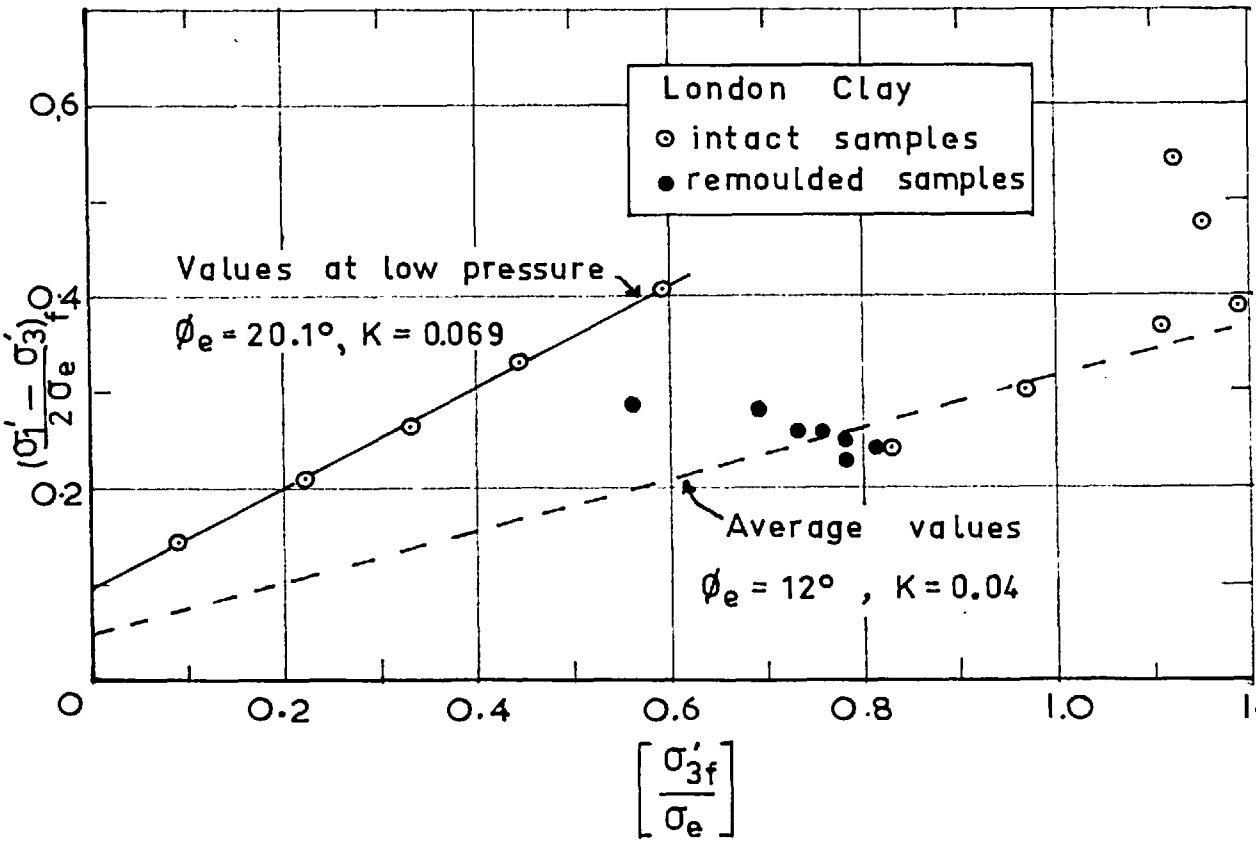


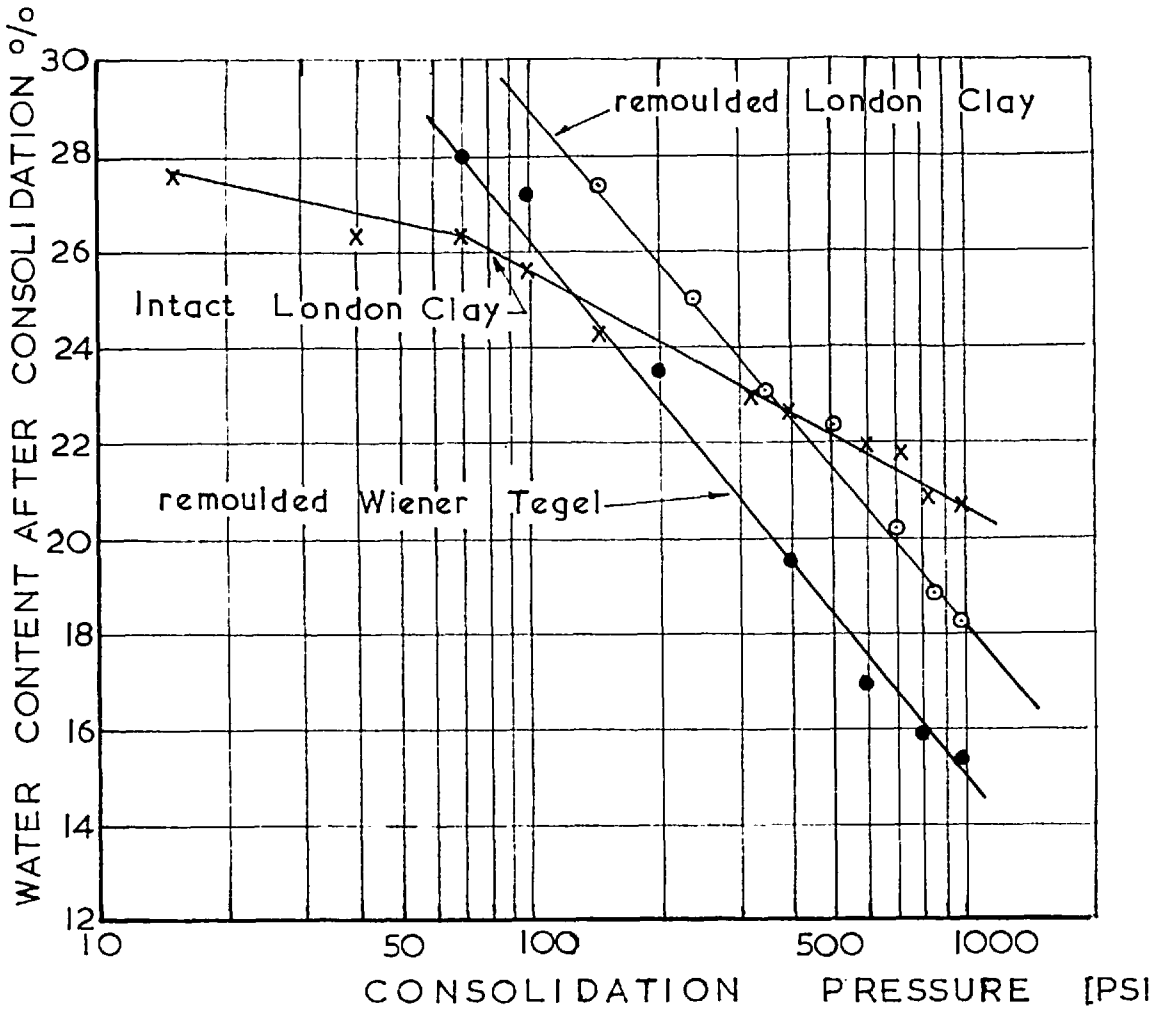
FIG. 7.22 VARIATION OF φ' WITH CONFINING PRESSURE

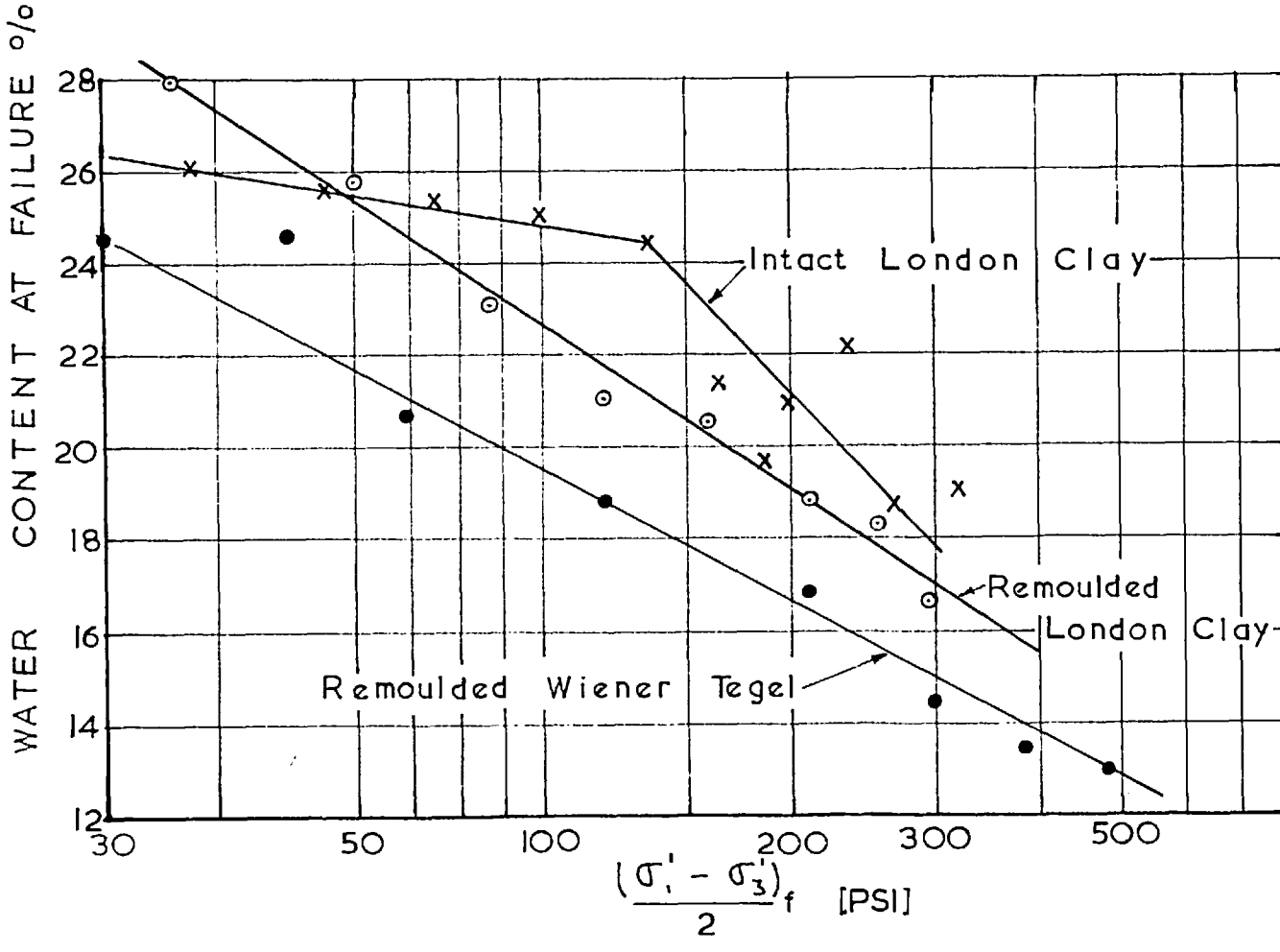




DETERMINATION OF HVORSLEV PARAMETERS.

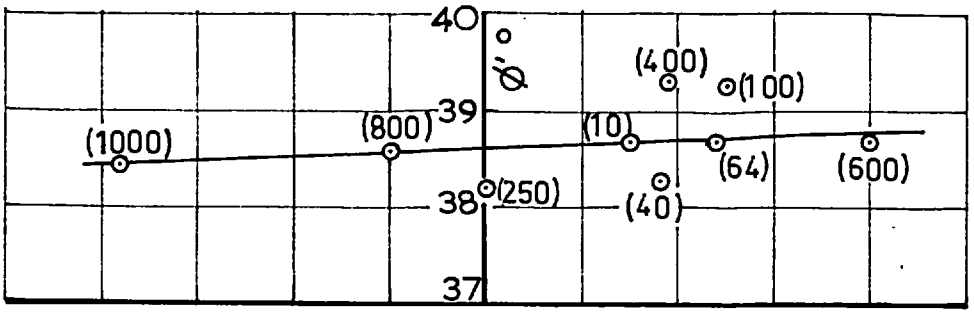
FIG. 7·23



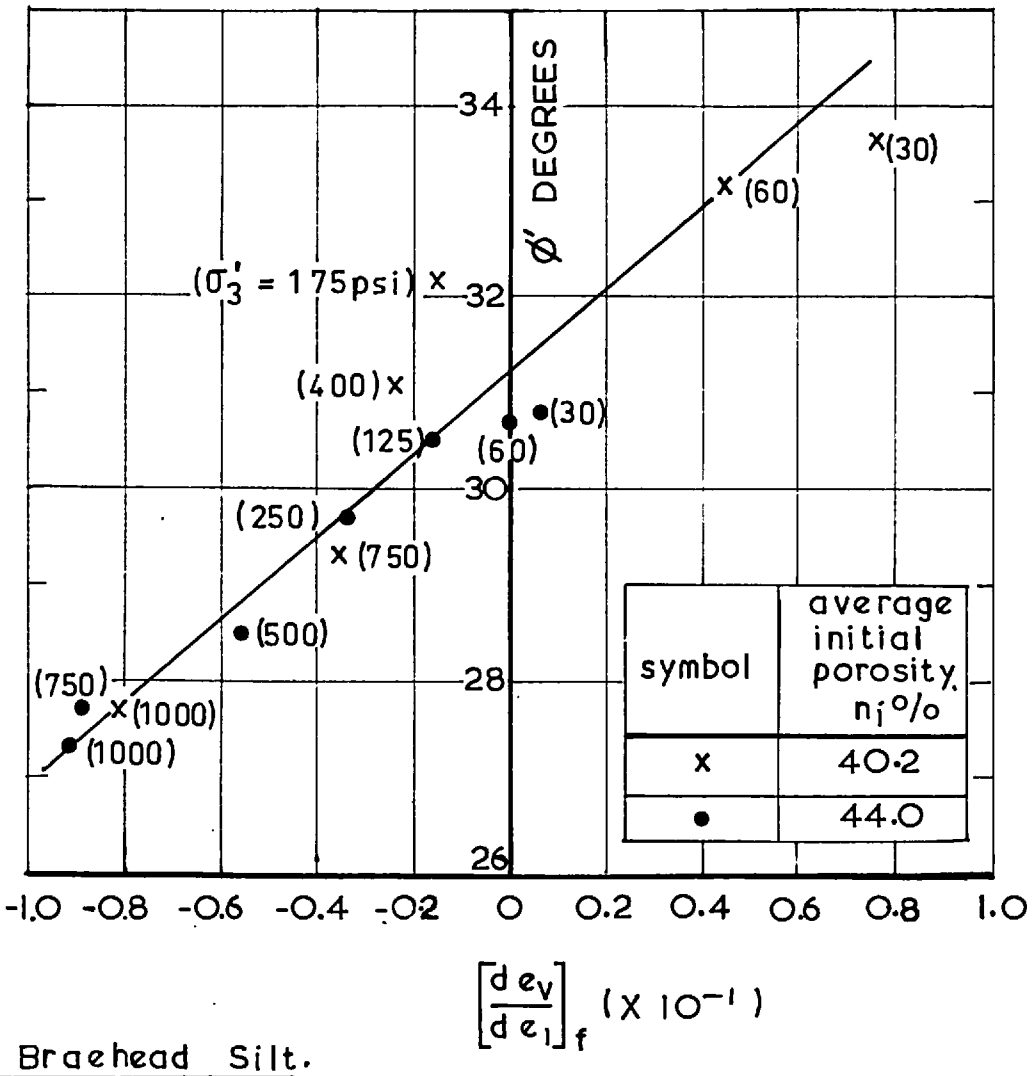


GRAPHS OF WATER CONTENT AGAINST PRESSURE.

FIG. 7.24



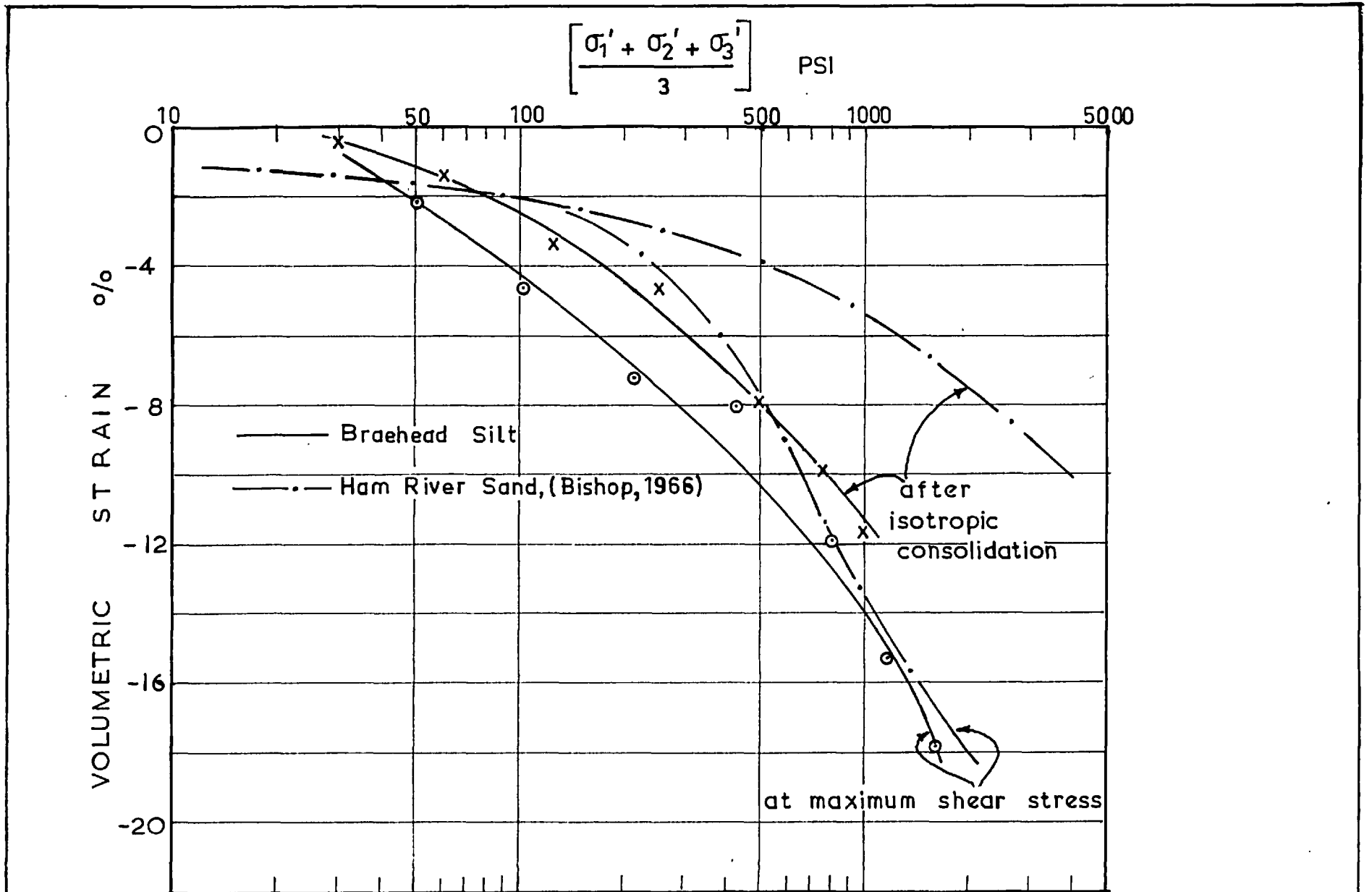
(a) Calcite.



(b) Braehead Silt.

THE RELATIONSHIP BETWEEN Ø' AND THE RATE OF VOLUME CHANGE AT FAILURE, $\left[\frac{de_v}{de_1}\right]_f$.

FIG. 7·25



RELATIONSHIP BETWEEN THE VOLUMETRIC STRAIN AND THE AVERAGE PRINCIPAL STRESS AFTER CONSOLIDATION AND AT FAILURE.

FIG. 7. 26

PART III

CONSOLIDATED-UNDRAINED TESTS

ON

REMOULDED LONDON CLAY.

CHAPTER 8.

DESCRIPTION OF THE PREPARATION OF MATERIALS, APPARATUS AND EXPERIMENTAL TECHNIQUES.

8.1 Preparation of materials.

Three batches of remoulded London Clay (from Wraysbury) were prepared from slurry for use in the tests.

8.1.1 Preparation of Remoulded London Clay Batch I.

The preparation of this material had been described elsewhere (Section 6.2.2). During the preparation of the material, it was consolidated in a 9" diameter oedometer to a final effective stress of 160 psi. The unloading of the sample was done in one operation, thus preventing the sample from swelling gently back to the zero effective stress condition.

Furthermore, the first tests in this series were performed on the material almost nine months after it was prepared. It was suspected that the sample might become partially saturated through the release of gases as a result of the sudden unloading operation and the long storage, a suspicion which had been apparently reinforced by the results obtained from the first series of test on the material. It was therefore decided to prepare a fresh batch of material.

8.1.2 Preparation of Remoulded London Clay Batch II.

This batch of remoulded clay was prepared in much the same way as batch I, except that the maximum pre-consolidation load was much smaller, and the sample was

unloaded over a week by reversing the direction of drive of the triaxial loading frame, and reducing the rate strain. The sample was kept supplied with water on both faces during the unloading operation.

The material was tested within a week of preparation.

8.1.3 Preparation of Remoulded London Clay Batch III.

Since there was a possibility of air becoming entrained in the slurry at various stages during the preparation (such as when the slurry was being poured from the Hobart Mixer into the oedometer) it was decided to prepare a third batch of clay under vacuum conditions.

To do this, a perspex cylinder, of 3" I.D and 6" long was made, sealed at both ends with end covers into which O-rings were inserted. A 3" dia. coarse porous stone was inserted into the base of the cylinder, which had a connection to a container with a freshly boiled water. The top cover of the cylinder had connections for a suction line and an opening for a stirrer, which was driven by a portable electric drill set to a slow speed.

About 4" depth of dry clay powder was placed in the cylinder after the porous stone at the base had been soaked. The clay powder was then subjected to a suction of 25 inches of mercury from a vacuum pump for three hours, the dry powder being prevented from being drawn into the suction line by means of perforated filter paper discs which were inserted into a recess at the entrance to the suction line. The powder which was kept stirred during the application of the vacuum, and was suddenly flooded

from the base with freshly boiled water till the powder was covered with about an inch of water. The water supply was then cut off and the vacuum pump shut down, while the stirring continued for another hour.

The mixture was then allowed to sediment, the excess water being drawn off. The top cover was later removed so that the sample can be consolidated in the same way as for the two previous cases.

The index properties of London Clay were given in Table 6.1. Table 8.1 below gives the final preconsolidation pressure P_c and water contents of the materials from the three batches of remoulded clay.

TABLE 8.1

Material	Initial n.c. of slurry (%)	Final n.c (%)	Pre-consolidation pressure (psi)
Batch I	160	28.9	160 *
Batch II	112	43	41
Batch III	-	41	45

* Approximate

8.2 Apparatus.

The apparatus used for these series of undrained tests was the same as those used for the tests on clay in Sections A and B, and described in detail in those sections.

8.3 Experimental Techniques.

8.3.1 Tests on material batch I.

The samples taken from this batch of remoulded clay were set up for testing the manner described in detail elsewhere (Section 3.2.1.1). The samples were consolidated isotropically under various confining pressures between 300 psi and 1000 psi, that is, under consolidation pressures higher than the estimated preconsolidation pressure on the material.

After consolidation, the drainage connections from the sample were closed, and the cell pressure suddenly decreased to a predetermined fraction of the isotropic consolidation pressure and the sample was sheared undrained at the relatively fast rate of 0.02 in/min, the peak strength being attained in about 5 minutes.

The test was halted after about 14% axial strain, the apparatus quickly dismantled, and the sample recovered. The inclination of any shear plane was measured quickly, and the sample was then cut into three sections in order to study the moisture content distribution in the top, middle and lower portions of the sample. This point is discussed more fully later.

All the samples tested in this series were consolidated against zero back pressure, and hence the isotropic consolidation pressure σ_3 was fully effective at the end of the consolidation process. Five samples were consolidated under each pressure, and sheared under the following fractions of the consolidation pressure: σ_3, $0.75\sigma_3$, $0.50\sigma_3$, $0.25\sigma_3$ and zero.

Tests were also done on samples consolidated under pressures less than the estimated preconsolidation

pressure. In these cases, the samples were prepared by remoulding the soil at the same moisture content as the main material, and packing the soil into a mould against a wooden dolly. Only two samples were consolidated isotropically under each pressure, and were sheared, in one case under a confining pressure equal to the maximum isotropic consolidation pressure and in the other case under zero confining pressure. The results obtained from this series of tests are given in Table 9.1.

8.3.2 Tests on material batch II.

Three main series of tests were conducted on this batch of material, all the samples being consolidated isotropically under an effective stress of 500 psi.

In the first series of tests, the samples were consolidated under a cell pressure of 1000 psi and a back pressure of 500 psi. After consolidation, the samples were sheared undrained as described above. Six tests were done in which the cell pressure during shear was lowered from 1000 psi to 500 psi in steps of 100 psi, and in a further three tests, the cell pressure during shear were 300 psi, 150 psi and zero.

In the second series of tests, the samples were consolidated isotropically under an effective stress of 1000 psi, using a cell pressure of 1000 psi and draining the sample against the atmosphere. A back pressure of 500 psi was then applied, and the sample allowed to swell back to the condition σ_3 = 1000 psi and u = 500 psi. The samples in this state were overconsolidated, with an overconsolidation ratio (O.C.R) of 2. The sample was sheared undrained under the various confining pressures

give in Table 9.2.

A third series of undrained tests were performed on samples consolidated under a cell pressure of 500 psi and zero pore pressure, the cell pressures (in psi) during shear being 500, 400, 300, 200, 100 and zero. A further special test was done in which after consolidation to an effective stress of 500 psi, the drainage lines were shut, the cell pressure suddenly decreased to zero and maintained at that value for half an hour, and was then increased again to 500 psi, after which the sample was sheared undrained.

After the tests, some of the samples were trimmed as described below for impregnation with carbowax. The trimmings were used to determine the average moisture content of the sample after the test.

8.3.2.1 Preparation of thin sections.

After the tests, some of the samples used in the undrained tests, as well as some used in drained tests, were carefully trimmed parallel to the main axis and on either side of a diameter until a final thickness of 1/4" remained. The direction of trimming was so chosen as to include any traces of shear plane produced. The sample was then immersed in molten Carbowax 5000 in an oven maintained at a temperature of 60°C for three days, after which the sample was air-dried and later stored in a plastic bag, away from moisture.

The impregnated samples were later ground on one face on three grades - coarse, medium and fine - of emery paper. During the grinding and subsequent operations, the samples were lubricated or washed with paraffin

instead of water, since the latter would dissolve away the carbowax. After the grinding operation, the ground face of the sample was polished on a clean glass plate, using a mixture of an optical smoothing powder ("Aloxite") and paraffin, care being taken that there were no quartz grains in the powder to scratch the polished surface. The samples were then washed thoroughly with paraffin and later wiped clean with a polishing cloth and air-dried for a day.

The dry samples were mounted on thin section glass plates which had been previously cleaned with alcohol to remove traces of grease and were inspected to ensure that there were no scratches on them. A thin uniform smear of Araldite solution was applied to one face of the glass plate, and the ground face of the sample was then placed on it, care being taken to ensure that no air bubbles were trapped between the glass plate and the sample. The Araldite solution was then allowed to set for two days, after which the second face of the sample was ground and polished as described earlier. The polishing operation was continued till the sample on the glass plate was just transparent, and then the slide was washed with paraffin and left to dry. The excess dried Araldite was carefully removed from the face of the slide with a blade, and a cover slide stuck on the newly ground face of the sample, using a fresh Araldite solution. After the setting of this Araldite, the slide was ready for viewing.

8.3.3 Tests on material batch III.

Three more undrained tests were conducted on samples cut from this batch of soil; two of the samples were consolidated isotropically under a cell pressure of 1000 psi and a pore pressure of 500 psi, and sheared undrained with the cell pressures equal to 1000 psi and zero respectively. The third sample was consolidated against the atmosphere under a cell pressure of 500 psi. The cell pressure during shear was also 500 psi.

The results of the tests on samples cut from remoulded London Clay batches II and III are summarized in Table 9.2.

CHAPTER 9.

PRESENTATION AND DISCUSSION OF THE RESULTS OF THE CONSOLIDATED UNDRAINED TESTS ON REMOULDED LONDON CLAY.

9.1 Introduction.

The undrained shear strength of saturated normally-consolidated intact samples, consolidated under a given pressure, is, within the limits of experimental accuracy, assumed to be sensibly independent of confining pressure during shear. This fact forms the basis of the $\phi = 0$ analysis (Skempton, 1948), which is used in the computation of the bearing capacity and earth pressures, and in the assessment of the stability of slopes in clays, in situations when there is little opportunity for changes in water content, for example, at the end of construction.

Advantage is taken of the independence of undrained shear strength, C_u, of confining pressure to determine C_u quickly and easily by means of a simple unconfined compression apparatus. In the case of such tests on undisturbed over-consolidated clays (Glossop, 1947; Skempton and Henkel, 1957) and on some normally consolidated clays (Skempton, 1953) the unconfined compressive strength so obtained had been found to be up to 20 % lower than the values of C_u determined in the triaxial apparatus under confining pressures in excess of the estimated overburden pressure. This decrease in undrained strength is generally assumed to be the result of the opening up of fissures or micro-fissures in the

samples, although Skempton (loc.cit.) reported such decreases in undrained strength in normally consolidated Tilbury clay, which had no visible evidence of fissuring.

The results presented in this chapter lead to another possible explanation based on the influence of the initial pore pressure in the sample.

9.2 Possibility of the existence of absolute tensions in pore water.

When a fully saturated sample of clay, which had attained equilibrium water content under externally applied pressures is subjected to a sudden decrease in applied stresses without changes in water content, the resulting expansion in the sample causes a decrease in pore pressures. If the initial pore pressure in the sample is either small or zero (gauge) large negative pore pressures * may be set up. For negative pore pressures/(i.e. more negative) than one atmosphere (i.e. about -14.7 psi at sea level) the pore water would be/absolute tension, and if the pore water is capable of withstanding the tension, the sample remains saturated ** and the decrease in pore pressure is then directly equal to the decrease in total applied stresses (Aitchison, 1960; Donald, 1961). Assuming that the Terzaghi effective stress law for fully saturated soils can be extrapolated into the range of negative pore pressures, it follows that the effective stress on the sample remains unchanged.

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* Croney et al (1960) recommended the use of this term to describe sub-atmospheric pressures in confined samples, and the use of the term 'suction' for pressures in unconfined samples. For a detailed discussion of these and other related terms used in the literature to describe sub-atmospheric pore pressures, see Donald (1961).

** Aitchison (1960) used the term "quasi-saturated" to describe the state of a soil having a degree of saturation of 100 %o and sub-atmospheric pore pressures.

There is, however, considerable controversy surrounding the ability of soil water to support large tensions without rupture. Work by Green (1951) indicated that the tensile strength of de-aired water is about 2,800 psi, and Aitchison (1960) suggested that even water containing moderate amounts of dissolved air could support tensions of up to 40 atmospheres. Other available evidence on the ability of water to sustain tension has been reviewed by Marshall (1959) who, together with Bolt and Miller (1958) and Greacen (1959), questioned the ability of soil water, as distinct from ordinary water, to withstand tensions of any appreciable magnitude.

Various reasons were advanced to justify the view that soil water could have different properties from ordinary de-aired water. Bolt and Miller (1958) reasoned that soil water, which invariably contains some dissolved gases, would have little tensile strength since when the absolute pressure approached the vapour pressure of the water, bubbles of water vapour would be released. They, therefore, concluded that soil water must have the same limitations on its shear strength as the water in the measuring apparatus, namely that it may not be expected to withstand tensions lower than one atmosphere. Furthermore, soil water is under the influence of such factors as the adsorptive forces associated with the fields of force of the particle surface forces (Lambe, 1953; Winterkorn, 1953) and osmotic pressures due to any dissolved salts. Olson and Langfeller (1965) attempted to explain the low negative pore pressures (less than -250 psi)

they measured in compacted clays by considering the total head, h, in the pores of a soil as being made up of a pressure head ($\frac{u}{\gamma_w}$), an osmotic head ω, an adsorptive head α and a position head, z. Thus

$$h = \frac{u}{\gamma_w} + \omega + \alpha + z \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9.1)$$

They stated that the actual limiting negative pore pressure in the soil was one atmosphere, and that, if lower pore pressures were recorded, it was because the total head was depressed by the predominating influence of the adsorptive and the osmotic heads. They suggested that the above reasoning may also explain the large negative pore pressures (much larger than the tensile strength of de-aired water) which, calculations based on the capillary model reveal, exist in the pores of drying soils (e.g. Terzaghi, 1925). The above approach is, however, one of the soil physicists, and probably has little relevance to the problem from the point of view of the soil mechanics worker who is essentially interested in relating measured pore pressures to soil macrobehaviour.

From the preceding review, it is clear that opinions are divided on the subject of the ability of soil water to withstand high negative pore pressures without cavitating. Some indirect evidence will be provided in this section of the thesis to indicate that soil water may be incapable of supporting negative pore water pressures equal in magnitude to the consolidation pressure under all conditions.

9.3. Presentation and discussion of results.

9.3.1 Tests on material batch I.

9.3.1.1 Final Water contents.

At the end of the tests on this batch of remoulded clay, the cell fluid was drained, the cell quickly dismantled and the sample recovered and cut into three vertical sections in order to study the distribution of final moisture contents. The moisture contents of the end pieces could be expected to exceed that of the middle part of the sample, since the decrease of cell pressure at the start of the shear stage, and also the removal of the shear stresses at the end of the test would inevitably lead to the inbibition of some water by sample from the filter stones. In order to lessen the possibility of this happening, fine ceramic stones were used at the ends of the sample and the length of drainage lead between the ends of the sample and the nearest valve was kept to the minimum, in order to cut down on the amount of water available for imbibition.

The results of the moisture content determinations are given in Table 9.1 and fig. 9.1 shows graphs of the undrained shear strength $(\frac{\sigma_1 - \sigma_3}{2})$ against the water content of the middle third of the sample, for samples sheared normally consolidated and unconfined.

In general, the moisture content of the central part of a sample was, as expected, less than those of the ends, the average difference between the maximum and the minimum values of water content being of the order of 1.2 °/o. For samples consolidated under the same ambient pressure, the final moisture contents of the three sections increased with decreasing confining pressure

before shear. These observations can probably be explained by the inbibition of water during the decrease of the cell pressure prior to shear, and the removal of shear stresses at the end of the test, as well as the inherent non-uniform moisture distribution in the triaxial sample. Similar variations in water content after undrained shear had been noted and discussed by Henkel and Sowa (1963) and Olson (1963).

9.3.1.2 Strength Relationships.

For samples consolidated isotropically under the same effective pressure (P_c') and sheared undrained under various cell pressures σ_3 (less than or equal to P_c'), the undrained shear strength was found to decrease with increasing difference between the consolidation pressure P_c' and the total ambient pressure σ_3 during shear i.e. $(P_c' - \sigma_3)$. Table 9.1 summarizes the results obtained from tests on this material, and the stress-strain curves are presented in figs. 9.2, 9.3 and 9.4. In general, the initial slope of the stress-strain curve, which is a measure of the stiffness of the sample decreased, and the failure strain decreased with increasing magnitude of the difference $(P_c' - \sigma_3)$. The brittle type of failure noticeable from the curves for the unconfined samples is also indicative of the existence of large negative pore pressures at failure (Whitman, 1960).

If the undrained shear strength of the normally consolidated sample ($P_c' = \sigma_{3s}$) is represented by (C_{uo}), and the undrained shear strength under any other confining pressure (σ_{3s}) is denoted by C_u, a non-dimensional plot of $\left(\frac{C_u}{C_{uo}}\right)$ against the ratio $\left(\frac{\sigma_{3s}}{P_c}\right)$ can be made. Such

a plot is presented for this batch of remoulded clay in fig. 9.8(a), and from it, it can be seen that the points corresponding to the various consolidation pressures plotted close to a unique curve, and that the value of the ratio $\left(\frac{C_u}{C_{uo}}\right)$ for samples sheared unconfined was 0.65, indicating that there was a 35 °/o decrease in undrained shear strength between samples sheared unconfined, and with the cell pressure equal to the effective consolidation pressure P_c'. Possible reasons for this decrease in strength will be discussed later.

The curve of $\left(\frac{C_u}{P_c'}\right)$ against $\left(\frac{\sigma_{3s}}{P_c'}\right)$ is shown in fig. 9.9. For isotropic consolidation pressures in excess of 500 psi, presumably when the initial effects of the oedometer consolidation have been destroyed, the points plotted on a unique curve, which, for values of $\left(\frac{\sigma_{3s}}{P_c'}\right)$ equal to or greater than unity, was asymptotic to the line $\left(\frac{C_u}{P_c'}\right) = 0.22$. The same value of $\left(\frac{C_u}{P_c'}\right)$ was obtained by plotting C_u against P_c' for normally consolidated samples, fig. 9.10, and this value may be compared with that computed from Skempton's (1957) relationship for undisturbed normally consolidated clays, namely,

$$\left(\frac{C_u}{P}\right) = 0.11 + 0.0037\ (W_{PI}) \quad \ldots\ldots\ldots\ldots \quad (9.2)$$

The above relationship would yield a value of 0.28 for $\left(\frac{C_u}{P}\right)$ when $W_{PI} = 46$ °/o. Bishop et al (1965) also reported values of $\left(\frac{C_u}{P}\right)$ ranging from 0.20 to 0.24 from tests over a comparable confining pressure range on remoulded samples for London Clay from nearby Ashford Common, and suggested that the fact that these values were lower than those predicted from Skempton's (loc.cit.) relationship

indicated the development of an ordered structure in the samples consolidated from slurry.

The decreases in undrained shear strength discussed above may, to a minor extent, be attributed to the increased water content of the ends of the sample due to the inhibition of water as a result of the sudden decrease in confining pressure. Indeed, in earlier tests, in which there was a long length of tubing between the sample and the nearest Klinger valve, and in which sample drainage was through coarse filter stones, the water content of the ends of the sample increased so much on decreasing the cell pressure, that the sample failed by splitting at either end, while the central part remained intact. The results of these tests were rejected as unrepresentative. Steps taken to deal with this problem included the replacement of coarse filter stones with fine ones, and the installation of the valves as near to the sample as practicable, thus reducing the quantity of water available to be sucked in by the sample. In spite of these measures, some increase in water content occurred, as can be seen from the final water content distributions presented in Table 9.1, and it is conceivable that this may be partially responsible for the observed decrease in undrained strength.

It is believed, however, that the major cause of the decrease in undrained shear strength was the release of dissolved gases from the pore water following the sudden decrease in confining pressure prior to shear.

9.3.2 Results of tests on remoulded clay batch II.

The results obtained from tests on samples of this material are summarized in Table 9.2, and the

stress-strain curves are given in figs. 9.5 and 9.6. As a comparison, the relevant section of Table 9.1 has been reproduced in Table 9.2. Figure 9.8 (b) gives a plot of $(\frac{C_u}{C_{uo}})$ against $(\frac{\sigma_{3s}}{P'_c})$ based on tests on this material. The curve for tests on material batch I is also shown (broken) as a comparison.

The results derived from the two series of tests on samples having an effective stress of 500 psi after consolidation plotted close to a single curve (fig.9.8(b)). In series A, the samples were consolidated isotropically under a cell pressure of 500 psi, and zero pore pressure, while in series B, the cell pressure and pore pressure during consolidation were respectively 1000 psi and 500 psi. In this latter series of tests, when the initial pore pressure in the sample was positive (i.e.when the total drop in cell pressure was less than or equal to 500 psi), there was little change in undrained shear strength - the 1-2 °/o variation obtained being within the limits of possible experimental errors. However, when the difference between the confining pressures during consolidation and shear exceeded 500 psi, in which case the initial pore water pressure was negative, there was a decrease in undrained shear strength similar to those observed from tests on material batch 1 and from tests series A on this material. The maximum decrease in undrained strength between the normally consolidated and the unconfined samples was only 17 °/o compared with the 35 °/o decrease in strength obtained from tests on material batch I.

It is interesting to note that a lot of care was taken during the preparation of the material from

slurry and the consolidation stage in the oedometer, to ensure that as little air as possible became entrained in the pore fluid, and that the sample was tested immediately after preparation. Compared with material batch I, therefore, this batch of remoulded clay should be able to remain fully saturated over a wider range of negative pore pressures, and hence the lower percentage drop in strength.

A special test (A7) was performed in which the sample was consolidated under σ_3 = 500 psi, and u = 0, then with the drainage leads shut, the cell pressure was dropped to zero, maintained at that value for half an hour, and increased again to 500 psi before the sample was sheared undrained. The resulting undrained shear strength showed a decrease of 3 % compared with the undrained shear strength of the normally consolidated sample having the same consolidation history. Shortage of material prevented the repetition of this test in order to establish whether this drop in strength was the result of any residual negative pore pressures, or just the normal scatter of experimental results.

9.3.2.1 Tests on over consolidated samples.

(Tests C1 to C6)

The stress-strain curves for the tests on over-consolidated samples (overconsolidation ratio = 2) are given in fig. 9.6, and the points ($\frac{\sigma_{3s}}{p'_c}$, $\frac{c_u}{\sigma_{uo}}$) are plotted on fig. 9.8 (b). As can be seen, these points plotted close to the unique curve for material batch II. Since the pore pressure behaviour during undrained shear for these samples is likely to be different from those of

samples which were initially normally consolidated (on account of the pore pressure coefficient A_f at failure being dependent on overconsolidation ratio), this would tend to suggest that the decrease in undrained strength was governed by the magnitude of the initial negative pore pressure in the sample rather than the pore pressure in the shear zone at failure. This conclusion should, however, be considered only tentative, since there was not enough material to enable tests to be performed on a series of samples with higher over consolidation ratios, and which may themselves develop substantial negative pore pressures under shear stresses.

9.3.3 Results of tests on remoulded clay batch III.

The results of this series of tests are also summarized in Table 9.2 and the stress-strain curves are given in figs. 9.7. As explained earlier, this batch of material was prepared under vacuum conditions, so that there was little likelihood of any air being entrained in the sample at any stage during the preparation.

Only four tests, in which the samples were initially consolidated under an effective stress of 500 psi, could be performed on this batch of material, and of these three, D1, E1 and E2, which had either zero or positive initial pore pressures gave identical values of the undrained shear strength, thus confirming a similar trend detected from the tests on material batch II. The fourth test, E3, which was performed unconfined, yielded an undrained shear strength 13% less than the shear strength of the normally consolidated sample.

9.4 Effective Stress due to negative pore pressures.

9.4.1 Introduction.

Effective stresses resulting from negative pore pressures may be deduced either by direct comparison of the mechanical behaviour of samples with known suctions with those of samples with externally applied effective stresses, or by assuming them to be equivalent to the magnitude of the negative pore pressure in the sample. The first method requires a dependable technique of inducing and measuring negative pore pressures in samples - various methods available for this purpose have been reviewed by Croney et al (1960) - and may, in the low negative pore pressure range, require very large confining pressures in order to produce the same effect as the negative pore pressures. Undrained compressive tests have been done on samples equilibriated under known suctions (Aitchison, 1957; Croney et al, 1954; Towner, 1958) but these tests had not been compared directly with any tests on similar material under known effective stresses, so no firm conclusions can be drawn regarding the magnitude of effective stresses due to negative pore pressure The second method, which had been suggested by Hilf (1956) is likely to be valid only for small negative pore pressures or suctions, but could conceivably involve considerable error when applied to samples with high suctions, when degree of saturation could differ significantly from unity, on account of cavitation.

In the absence of any established method of computing effective stresses resulting from negative pore pressures, the following approximate method had been adopted.

9.4.2 Computation of initial negative pore pressures and effective stresses.

It was not possible to measure any pore pressures before or after the tests owing to the need to shear the samples at a fast rate in order to attain failure before the sample had time to suck any water from the filter stones. It is, however, possible to compute approximate values of the initial negative pore pressures (or suctions in unconfined samples) and the effective ambient pressure at failure, σ'_{3f}, subject to the following assumptions:

(a) that the pore pressure coefficient A_f at failure (Skempton, 1954) remains constant for samples consolidated under the same ambient effective stress and sheared under different confining pressures,

(b) that the conventional Terzaghi effective stress law for saturated soils can be extrapolated into the range of negative pore pressures, and that any decrease in the degree of saturation of the sample as a result of the decrease in ambient pressure is not sufficient to invalidate its use.

(c) that the effective stress failure envelopes derived from drained and consolidated-undrained tests are identical.

Assumption (a) would probably involve some error since the pore pressure parameter, A_f, is known to be sensitive to soil structure and stress history. Since a sudden decrease in confining pressure in the absence of water would

be expected to cause an expansion of the soil structure (and hence the establishment of negative pore pressures), this parameter would strictly not be a constant for a series of samples having the same consolidation pressure but different degrees of unloading prior to shear, especially since the size of any bubbles formed as a result of cavitation would increase with increasing degree of unloading, resulting in a rupture of the soil structure, particularly in unconfined samples.

Aitchison and Donald (1956), on the basis of comparisons between void ratio changes induced in unsaturated clays by externally applied mechanical stresses in the oedometer and negative pore pressures or suctions, concluded that the Terzaghi effective stress law, namely,

$$\sigma' = \sigma - u \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9.1)$$

(using the familiar soil mechanics notations) was valid as a determinant of shear strength in such soils up to suctions of about 30 kg/cm^2. It is, therefore, believed that assumption (b) would not involve much error, even though it is recognized that the correct form of the effective stress law for unsaturated soils is as proposed and discussed extensively in the literature (see Section 1.4).

The equivalence of ϕ' and ϕ'_d, the angles of internal friction derived from consolidated undrained tests with pore pressure measurement, and from drained tests respectively has been discussed by Skempton and Bishop (1954), Henkel (1960) and Bjerrum and Simons (1960) among others. Conflicting conclusions had been drawn regarding the relative magnitudes of ϕ' and ϕ'_d, but it is

generally agreed that the two major factors contributing to any differences between the effective stress failure envelopes derived from the two types of test were the differences in the strain rates employed in the two types of tests, and possible volume changes during shear in the drained tests. Provided corrections are applied for these, the two envelopes should be almost identical. In general, the undrained shear strength decreases by approximately 5 °/o for a tenfold increase in the time to failure, (Bishop and Henkel, 1962). Using the values of the undrained shear strength derived from these very fast undrained tests in conjunction with effective stress failure envelopes derived from slow drained tests could, therefore, lead to an underestimate of the pore pressure at failure, and possibly of A_f.

9.4.2.1 Method of Calculation.

The pore pressure, u_f, in the failure zone of the sample during the undrained shear was estimated graphically as illustrated in fig. 9.11, the effective stress failure envelopes for the material (batch 1) having been determined previously from drained tests at elevated cell pressures (Chapter 7). Since, for a sample consolidated at a given pressure against the atmosphere and sheared undrained at the same pressure, the pore pressure was set up solely under shear stresses, it was possible to estimate the value of the pore pressure parameter, A_f, from the relationship:

$$A_f = \frac{u_f}{(\sigma_1 - \sigma_3)_f} \quad \ldots\ldots\ldots\ldots \quad (9.2)$$

and this value of A_f was assumed constant for all samples

initially consolidated under similar stresses. It was therefore possible to calculate the pore pressures set up during shear in each case from the product of A_f and the deviator stress at failure.

The initial negative pore pressure $-u_i$ * in the sample was then given by the difference between u_f determined graphically and the pore pressure resulting from shear stresses. From a knowledge of u_f, it was also possible to estimate the ambient effective stress on the sample at failure, σ'_{3f} .

All calculated quantities are given in Tables 9.1 and 9.2. Figure 9.12 (a) and 9.12 (b) show graphs of the maximum (total) deviator stress at failure (2Cu) against the initial suction, P''_o in unconfined samples and the ambient effective stress at failure σ'_{3f} respectively. Both graphs were linear on log-log plots, showing that the undrained shear strength C_u is related to both P''_o and σ'_{3f} by relationships of the form

$$C_u = K_1 (\sigma'_{3f})^{n_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9.3)$$

and $$C_u = K_2 (P''_o)^{n_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9.4)$$

where K_1, K_2, n_1, n_2 are constants.

The line obtained for material batch II is also shown broken in fig.9.12(b). Similar relationships were also

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* In confined samples, the negative pore pressure -u, was represented by the notation P''_o for suction.

suggested by Aitchison (1957) for quasi-saturated and unsaturated soils.

The ratio, β , between the initial negative pore pressure and the decrease in confining pressure causing it, is also given in Table 9.1. While too much emphasis may not be placed on the accuracy of the values quoted for this ratio, it is believed that the fact that the ratio differed significantly from unity probably indicates that the samples were becoming progressively unsaturated as a result of the release of dissolved gases on decreasing the consolidation pressure prior to shear. It should also be emphasised that the close correlation obtained between the various quantities as a result of this analysis, does not necessarily provide conclusive proof of the validity of the method of analysis.

9.4.3 Relationship between undrained shear strength and overconsolidation ratio (O.C.R).

Another possible explanation of the results obtained may be based on the progressive change in the state of the samples from a normally consolidated to an overconsolidated state with increasing magnitude of $(P_c' - \sigma_3)$. It can be shown that the final ambient effective stress, σ'_{3o} , on a sample which had been previously consolidated against the atmosphere under an ambient pressure of σ'_{3i} , and then subjected to a sudden decrease $(\Delta\sigma_{3i})$ in ambient pressure under conditions of constant water content, is given by

$$\sigma'_{3o} = \sigma'_{3i} - (1-\beta)\Delta\sigma_{3i} \quad \ldots\ldots\ldots\ldots \quad (9.5)$$

where $\beta = (\frac{\Delta u}{\Delta\sigma_{3i}})$; Δu being the negative pore pressure

resulting from the decrease in the ambient pressure.

If for reasons mentioned in the preceding section, β is less than unity, then there is a net decrease in the effective stress on the sample compared with the effective stress before the change in ambient pressure. The sample can, therefore, be considered as being overconsolidated just before the start of the shear stage, the overconsolidation ratio ($\frac{\sigma'_{3i}}{\sigma'_{3D}}$) increasing with the magnitude of the decrease in ambient pressure ($\Delta\sigma_{3i}$). Consolidated-undrained tests on samples of a marine clay with various O.C.R's, reported by Nakase (1967) showed that the undrained shear strength decreased with increasing overconsolidation ratio, and that the ratio between C_u for the overconsolidated and normally consolidated samples, ($\frac{C_{uoc}}{C_{unc}}$), was uniquely related to the reciprocal of the overconsolidation ratio. If this approach is correct, then the earlier assumption about the pore pressure parameter A_f being constant for samples with the same σ'_{3i} would involve a further error since A_f is strongly dependent on O.C.R.

9.5 Possible practical significance of the results.

As discussed earlier, undrained tests on undisturbed samples of clay under confining pressures less than the estimated in-situ effective vertical stress, have been known to yield strengths which are substantially lower than those obtained from tests on samples of the same material under confining pressures in excess of the in-situ stress. This decrease in strength has been generally put down to the opening up of fissures.

There is ample evidence that samples of clay (especially overconsolidated clay) extracted from great depths have large residual negative pore pressures or suctions (Skempton, 1961; Skempton and Sowa, 1963, Bishop et al, 1965(b)). While evidence of direct measurement of such suctions is scanty, various methods of estimating them have been proposed by Skempton (1961) and Bishop et al (1965 b) who reported estimates as high as 100 psi for the suction in undisturbed London Clay (Ashford Common) taken from a depth of 114 feet. It is conceivable, on the basis of the work presented, that where a sample is tested undrained with a residual suction, some decrease in strength could result, and this fact may probably explain the decrease in strength in an intact normally consolidated clay, reported by Skempton, and to which reference has already been made. The criterion , on this basis, for the establishment of a $\phi = 0$ condition, will, therefore, be the destruction of any residual suction in the sample, rather than the closing of fissures. It is relevant to mention here that Skempton and La Rochelle (1965) reported obtaining constant undrained strengths from tests on samples of undisturbed London Clay taken from a depth of 24', under confining pressures as low as 4 psi. This pressure is hardly sufficient to ensure the closing of fissures, but may be enough to destroy any residual suction in the sample.

While it is recognized that the conclusions drawn from the work reported in this section may not be directly applicable to undisturbed samples on account of differences in stress system and history, the writer is, nevertheless, of the opinion that the above approach offers

a possible explanation for the observed decreases in undrained shear strength.

9.6 Thin Sections.

The preparation of the thin sections for viewing under an optical microscope has been described in Section 8.3.2.1. The impregnation technique using Carbowax 5000 did not work satisfactorily, probably because of the low final water contents resulting from isotropic consolidation under high pressures. Only a thin outer shell was impregnated, and it was therefore not possible to obtain any information regarding the soil structure resulting from the shearing process, or the orientation of particles in the vicinity of the shear zones. Similar difficulty was experienced by Sutherland and Singh (1967) who proposed an alternative technique based on pre-drying the samples.

9.7 Summary.

Some experimental evidence has been presented to show that the undrained shear strength of clay samples with initial negative pore pressures decreases with increasing magnitude of the negative pore pressure. It is suggested that this decrease in strength may be explained by postulating a decrease in degree of saturation of the sample as a result of the release of dissolved gases from the pore fluid. This would tend to imply that pore water is probably incapable of withstanding appreciable negative pore pressures without cavitation, at least at certain locations in the soil pores.

An approximate method of computing the values of the initial negative pore pressure, $-u_i$ or suction, P''_o ,

and the ambient effective stress at failure σ'_{3f} has been presented. Calculations based on this approximate method indicated that the relationship between the undrained shear strength, C_u, of quasi- or unsaturated soils, and P''_o and σ'_{3f} was of the form suggested by Aitchison (1957), namely,

$$C_u = K_1 (\sigma'_{3f})^{n_1}$$

and

$$C_u = K_2 (P''_o)^{n_2}$$

Possible practical significance of the results has been discussed.

The carbowax impregnation technique for the preparation of thin sections has been found unsuitable for samples having very low moisture contents.

TABLE 9.1

Consolidated Undrained Tests on Remoulded London Clay Batch I

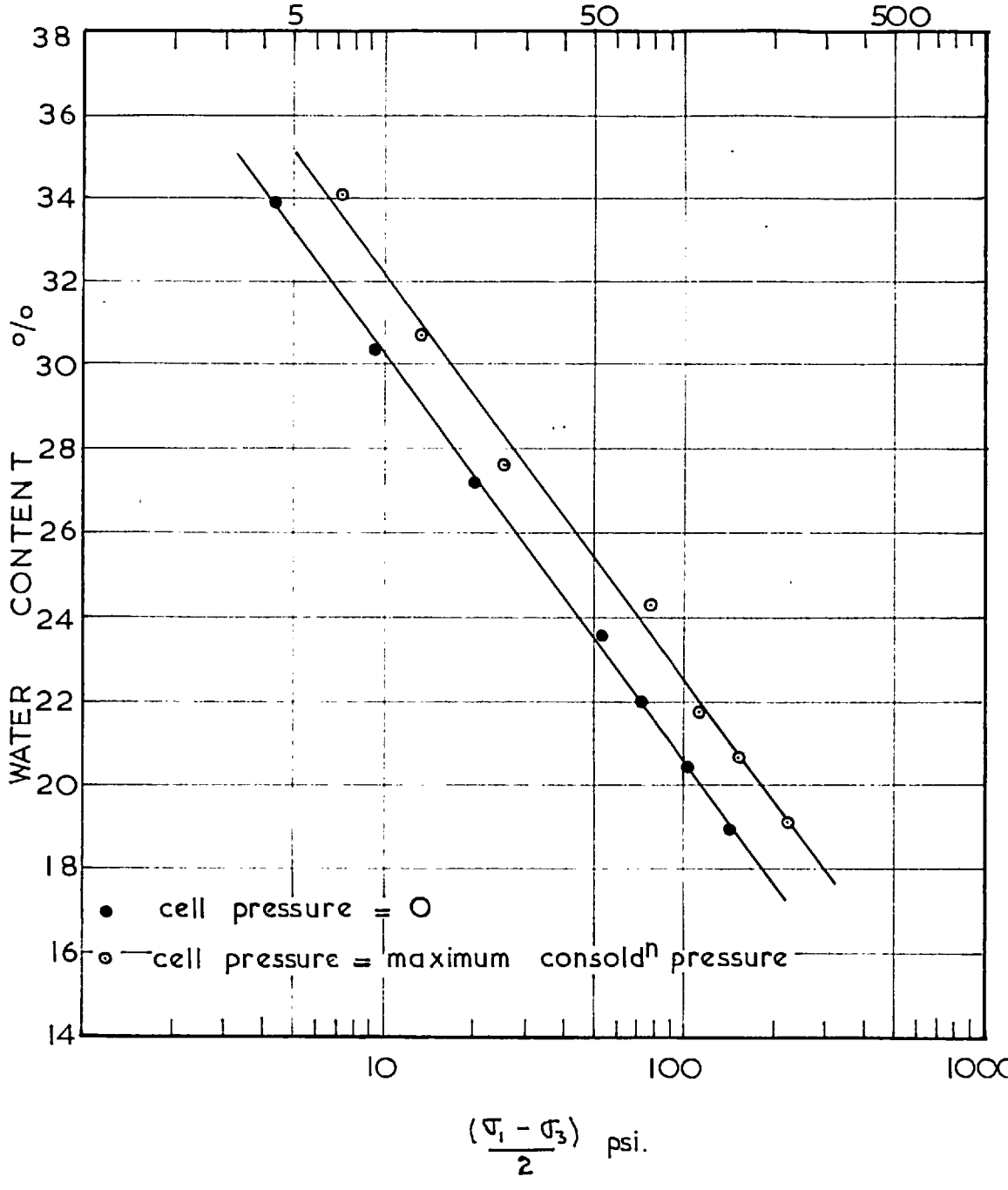
Test No.	σ_3' (=P_c') during consolidn (U = 0) psi	σ_{3s} during shear psi	Initial water cont. o/o	Final water content o/o			Peak $(\sigma_1-\sigma_3)$ psi	$\frac{C_u}{\sigma_1-\sigma_3}{2}$ psi	$\frac{C_u}{C_{uo}}$	$\frac{\sigma_{3s}}{P_c'}$	$\frac{C_u}{P_c'}$	U_f psi	A_f	$A_f(\sigma_1-\sigma_3)$ psi	U_i psi	σ_{af}' psi	$\beta\left[\frac{\Delta U}{\Delta \sigma_3}\right]$
				Top	Mid.	Bot.											
1	25	25	37.4	34.4	34.2	35.2	14.4	7.2	1.00	1.00	0.29	+27	–	–	–	–	–
2	25	0	36.2	35.8	33.9	34.8	9.6	4.8	0.67	0	0.19	+9	–	–	–	–	–
3	50	50	35.2	31.6	30.8	30.7	26.3	13.2	1.00	1.00	0.26	+35	+1.33	+35	0	–	–
4	50	0	35.0	32.3	30.5	31.5	19.0	9.5	0.72	0	0.19	-5	–	–	-30.3	–	0.61
5	100	100	33.9	26.9	26.9	27.7	51.3	25.7	1.00	1.00	0.26	+52	+1.01	+52.0	0	48	–
6	100	0	29.6	27.2	26.0	27.2	39.7	19.9	0.77	0	0.20	-32	+1.01	+40.2	-72.2	32	0.72
7	300	300	28.1	25.6	24.3	26.1	154	77	1.00	1.00	0.26	+105	+0.68	+105	0	195	–
8	300	225	"	24.8	23.8	24.4	144.1	72.1	0.94	0.75	0.24	+50	"	+98.2	-48	175	0.64
9	300	150	"	25.4	24.0	23.0	142.8	71.4	0.93	0.50	0.24	-22	"	+97.4	-119.4	172	0.80
10	300	75	"	25.6	24.0	25.2	125.7	62.7	0.81	0.25	0.21	-75	"	+85.7	-160.7	150	0.71
11	300	0	"	24.1	23.6	24.0	107	53.5	0.70	0	0.18	-121	"	+73	-194	121	0.65
12	500	500	"	21.8	20.9	20.5	227	113.5	1.00	1.00	0.23	+171	+0.75	+171	0	329	–
13	500	375	"	22.2	21.9	21.2	218	109	0.96	0.75	0.22	+83	"	+164.2	-81	292	0.65
14	500	250	"	21.2	21.2	22.3	212	106	0.93	0.50	0.21	+43	"	+160.1	-117	207	0.48
15	500	125	"	23.0	22.0	23.3	187	93.5	0.82	0.25	0.19	-136	"	+140.9	-276.8	261	0.74
16	500	0	"	26.1	22.1	23.1	146.5	73.3	0.65	0	0.15	-187	"	+110.4	-297.4	187	0.60
17	750	750	"	20.7	19.4	20.2	318	159	1.00	1.00	0.21	+240	+0.75	+240	0	510	–
18	750	563	"	21.1	19.4	20.4	310.8	155.4	0.98	0.75	0.21	+69	"	+234.6	-165.6	494	0.89
19	750	375	"	21.2	19.6	20.9	297	148.5	0.93	0.50	0.20	-85	"	+224.2	-309.2	460	0.82
20	750	188	"	21.5	20.2	22.0	276.2	138.1	0.87	0.25	0.18	-234	"	+208.5	-442	422	0.79
21	750	0	"	21.0	20.5	21.4	202.4	101.2	0.64	0	0.13	-290	"	+152.8	-442.8	290	0.59
22	1000	1000	"	19.1	18.2	18.8	438.3	219.2	1.00	1.00	0.22	+279	+0.63	+279	0	721	–
23	1000	750	"	19.3	18.6	19.5	420.6	210.3	0.96	0.75	0.21	-60	"	+267.7	-207.7	690	0.83
24	1000	500	"	18.2	19.5	19.5	391.7	195.9	0.89	0.50	0.20	-135	"	+249.3	-384.3	635	0.77
25	1000	250	"	20.5	18.3	20.1	366.6	183.3	0.84	0.25	0.18	-347	"	+233.4	-580.9	597	0.77
26	1000	0	"	20.1	19.3	19.0	290.9	145.5	0.66	0	0.14	-444	"	+185.2	-629	444	0.63

TABLE 9.2

Consolidated – Undrained Tests on Remoulded London Clay.

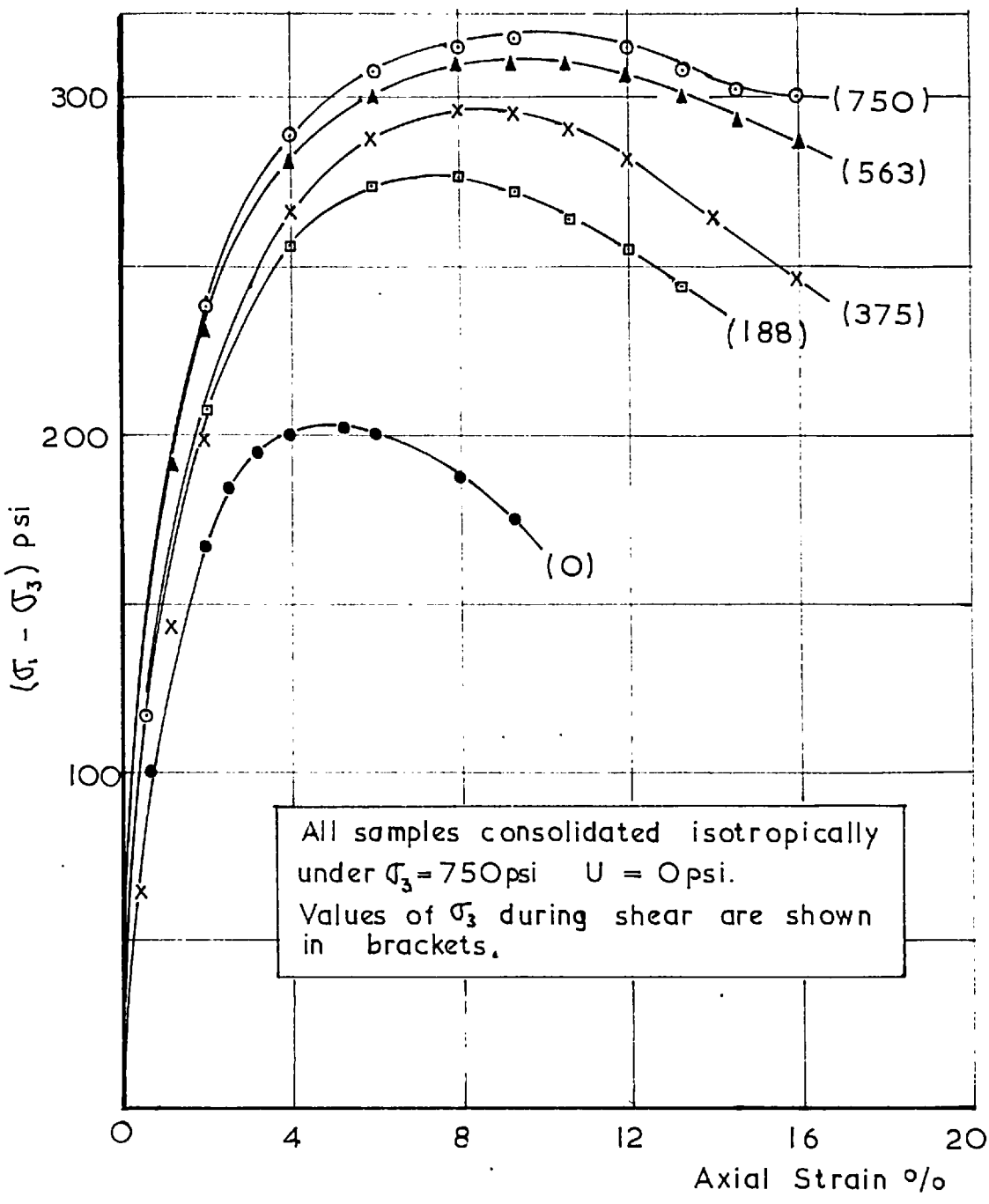
Sample Batch	Test No.	Isotropic Consolidn. σ_3 psi	Isotropic Consolidn. U psi	σ_{3s} during shear psi	* Final w.c. %	Peak $\sigma_1-\sigma_3$ psi	$\frac{C_u}{C_{uo}}$	% Change in strength.	$\frac{\sigma_{3s}}{P'_c}$	U_f psi	A_f	$A_f(\sigma_1-\sigma_3)$ psi	U_i psi	σ'_{af} psi	$\beta \left[\frac{\Delta U}{\Delta \sigma_3}\right]$
I	12	500	0	500	21.8	227	1.00	0	1.00	+171	0.75	+171.0	0	329	–
	13	500	0	375	22.0	218	0.96	-4	0.75	+83	"	+164.2	-81	292	0.65
	14	500	0	250	22.3	212	0.93	-7	0.50	+43	"	+160.0	-117	207	0.48
	15	500	0	125	22.0	187	0.82	-18	0.25	-136	"	+140.9	-277	261	0.74
	16	500	0	0	22.1	147	0.65	-35	0	-187	"	+110.4	-297	187	0.60
II	A1	500	0	500	22.1	201	1.00	0	1.00	+187	0.915	+184.0	0	316	–
	A2	500	0	400	22.1	196.9	0.98	-2	0.80	+91	"	+180.6	-90	309	0.90
	A3	500	0	300	22.5	193.6	0.96	-4	0.60	0	"	+177.6	-178	300	0.89
	A4	500	0	200	22.8	189.5	0.94	-6	0.40	-85	"	+173.8	-259	285	0.86
	A5	500	0	100	22.7	175.0	0.87	-13	0.20	-186	"	+160.0	-346	286	0.86
	A6	500	0	0	22.6	164.1	0.81	-19	0	-258	"	+151.0	-409	258	0.82
	A7	500	0	500	22.2	194.8	0.97	-3	1.00	+200	"	+178.8	+21	300	–
	B1	1000	500	1000	22.1	195.3	1.02	+2	2.00	+696	1.00	+196.0	0	304	–
	B2	1000	500	900	22.4	192.8	1.01	+1	1.80	+600	"	+192.8	+407	300	0.93
	B3	1000	500	800	22.4	192.9	1.01	+1	1.60	+500	"	+192.9	+307	300	0.96
	B4	1000	500	700	22.1	193.6	1.01	+1	1.40	+400	"	+192.6	+206	300	0.98
	B5	1000	500	600	21.5	190.7	1.00	0	1.20	+305	"	+190.7	+114	295	0.96
	B6	1000	500	500	22.4	190.5	1.00	0	1.00	+205	"	+190.5	+14	295	0.97
	B7	1000	500	300	22.4	183.9	0.97	-3	0.60	+15	"	+184.0	–	285	–
	B8	1000	500	150	23.1	171.1	0.90	-10	0.30	-107	"	+171.0	–	257	–
	B9	1000	500	0	22.8	161.9	0.83	-17	0	-258	"	+162.0	–	258	–
	C1	1000	500	1000	20.8	267	1.00	0	2.00	–	–	–	–	–	–
	C2	1000	500	750	20.5	267	1.00	0	1.50	–	–	–	–	–	–
	C3	1000	500	500	20.3	267	1.00	0	1.00	–	–	–	–	–	–
	C4	1000	500	300	20.4	257	0.96	-4	0.60	–	–	–	–	–	–
	C5	1000	500	150	20.6	251	0.94	-6	0.30	–	–	–	–	–	–
	C6	1000	500	0	20.8	219	0.82	-18	0	–	–	–	–	–	–
III	D1	500	0	500	24.5	188.7	1.00	0	1.00	–	–	–	–	–	–
	D2	1000	500	1000	24.0	189.6	1.00	0	2.00	–	–	–	–	–	–
	D3	1000	500	500	23.8	188.9	1.00	0	1.00	–	–	–	–	–	–
	D4	1000	500	0	24.0	165.2	0.87	-13	0	–	–	–	–	–	–

* From the shear zone or the middle part of the sample.



RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND WATER CONTENT AT FAILURE FOR REMOULDED LONDON CLAY.

FIG. 9.1



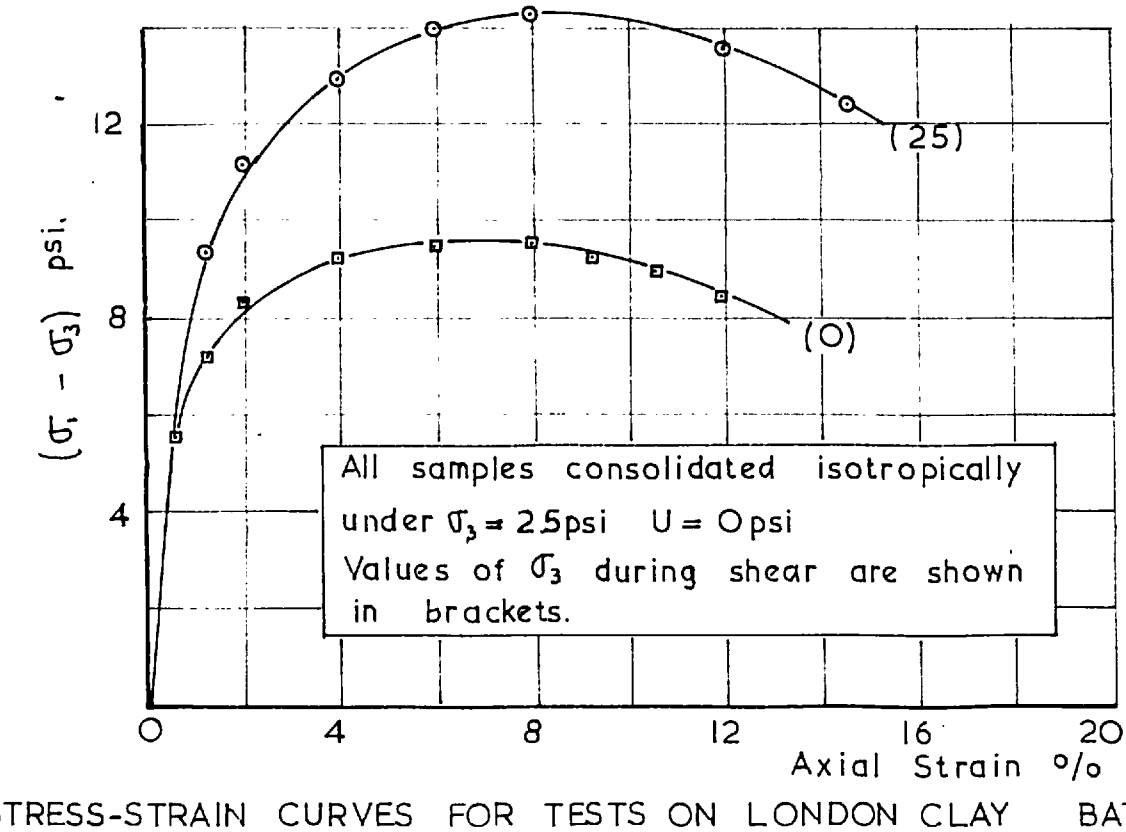
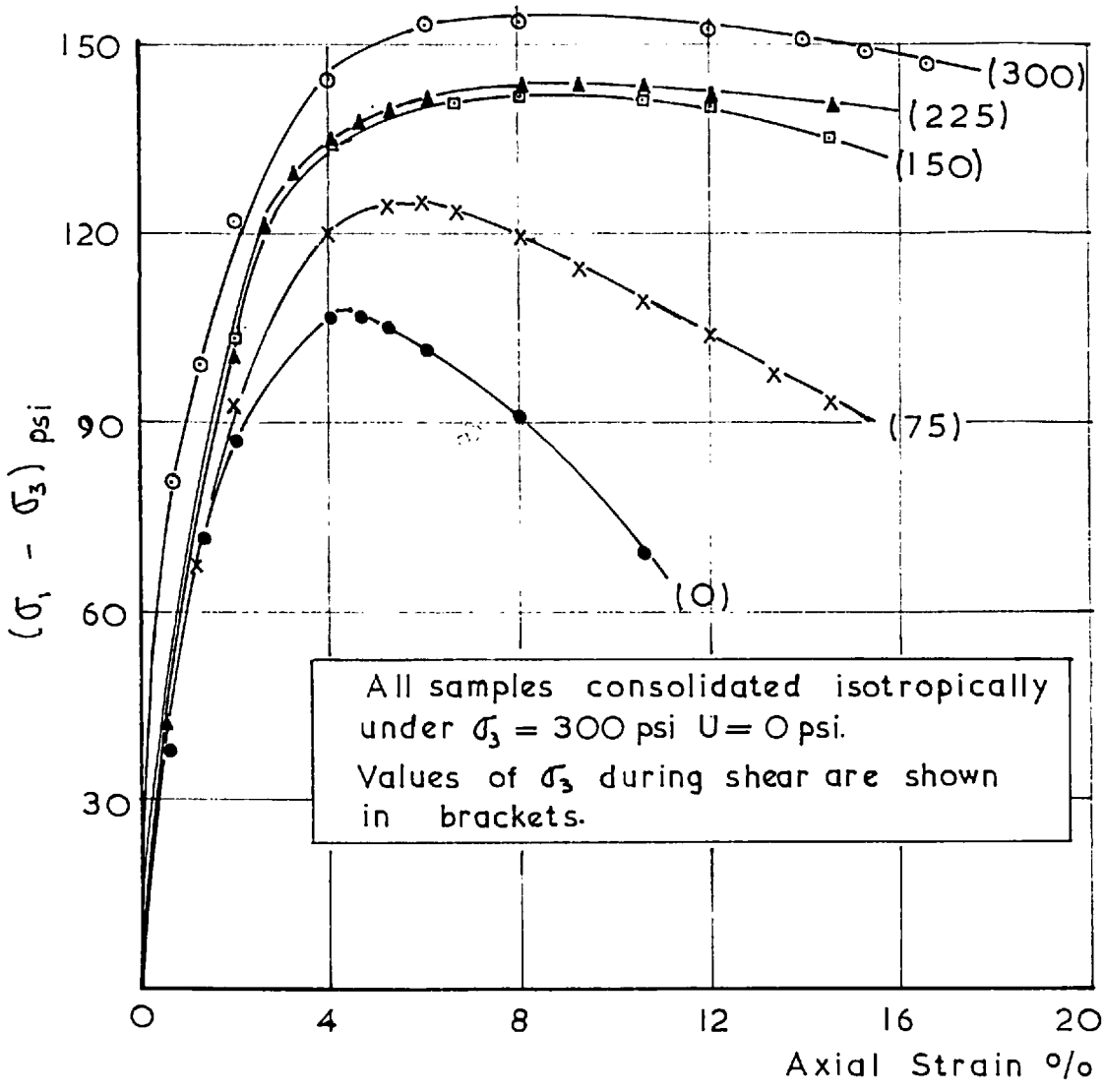


FIG. 9.2

STRESS-STRAIN CURVES FOR TESTS ON LONDON CLAY BATCH I



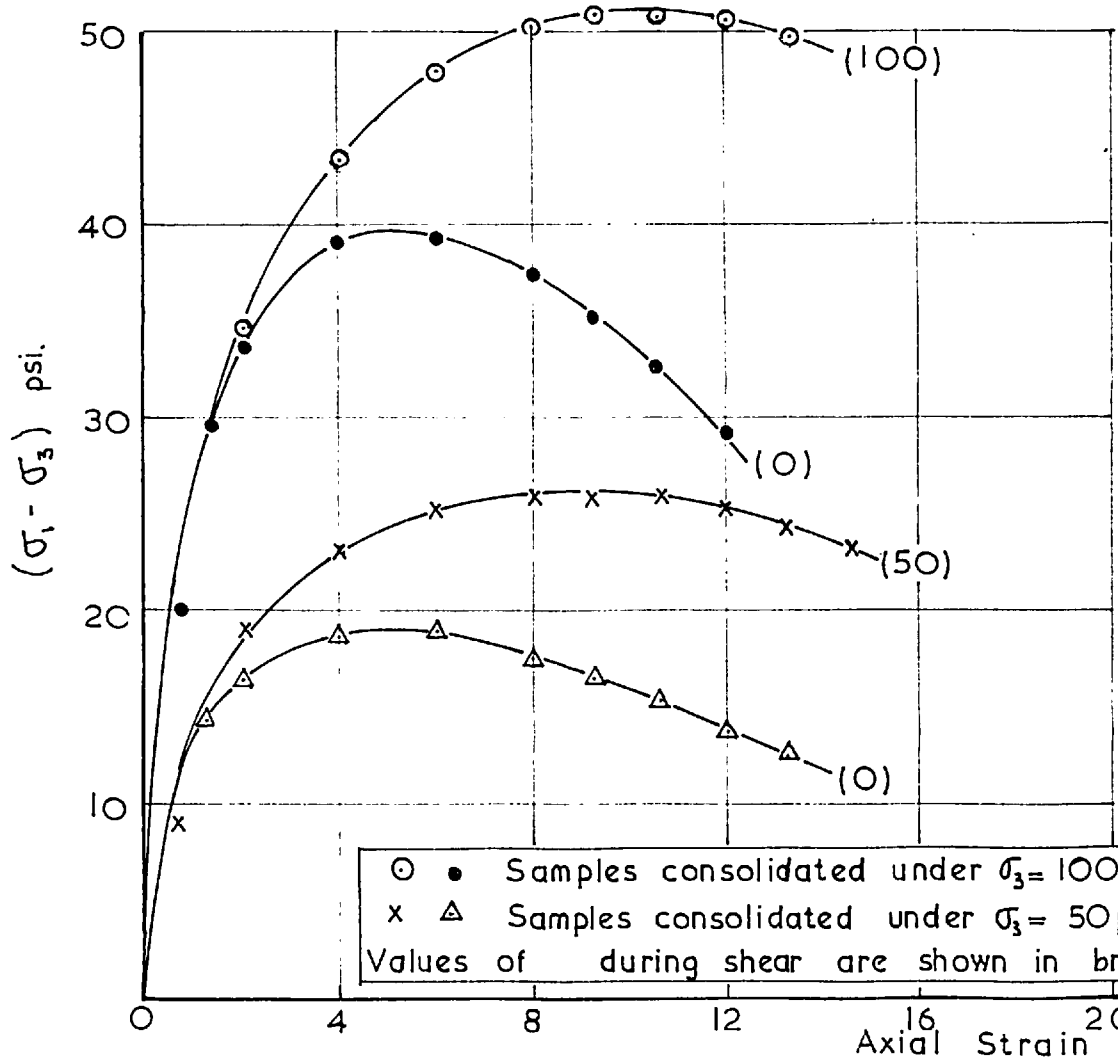
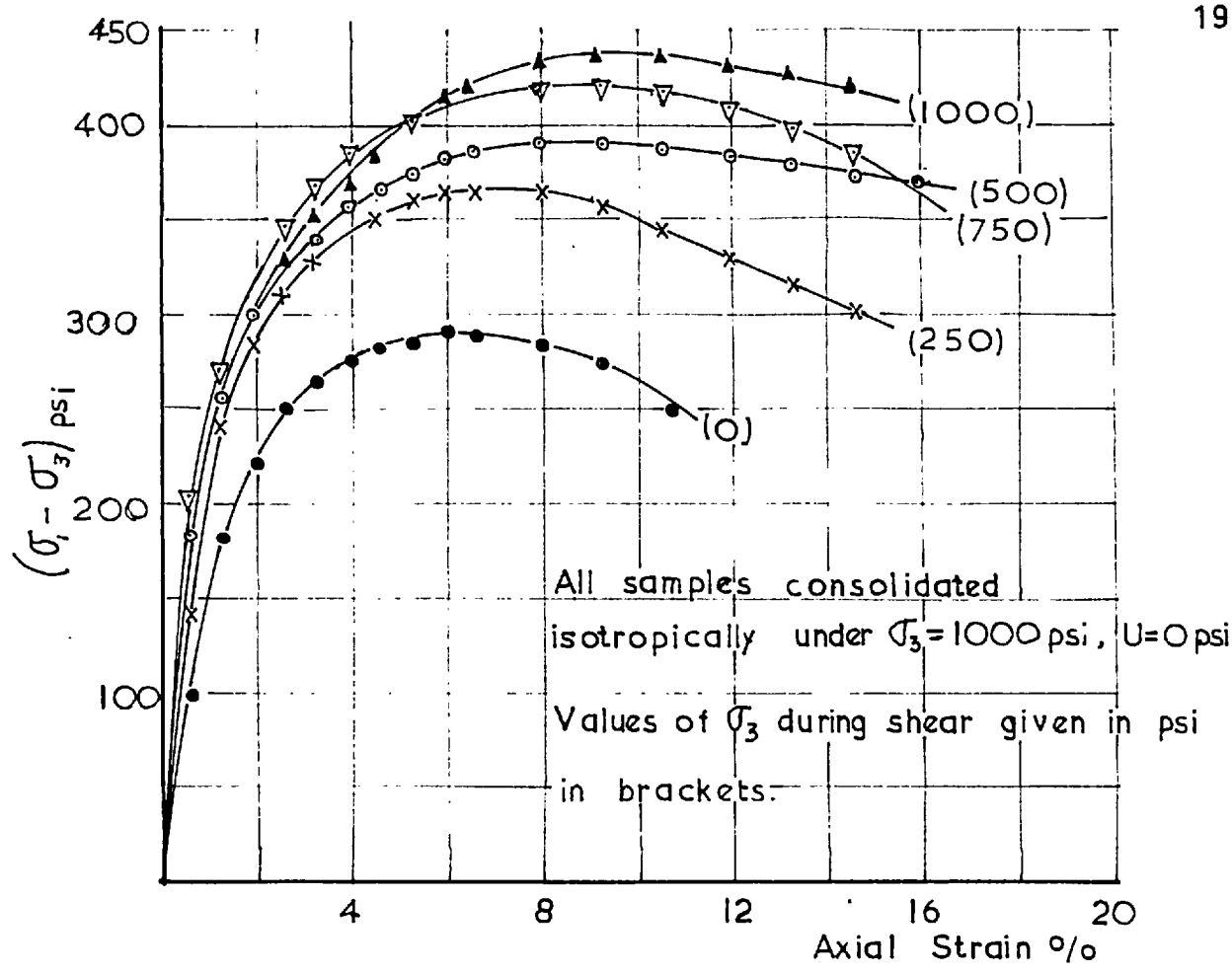
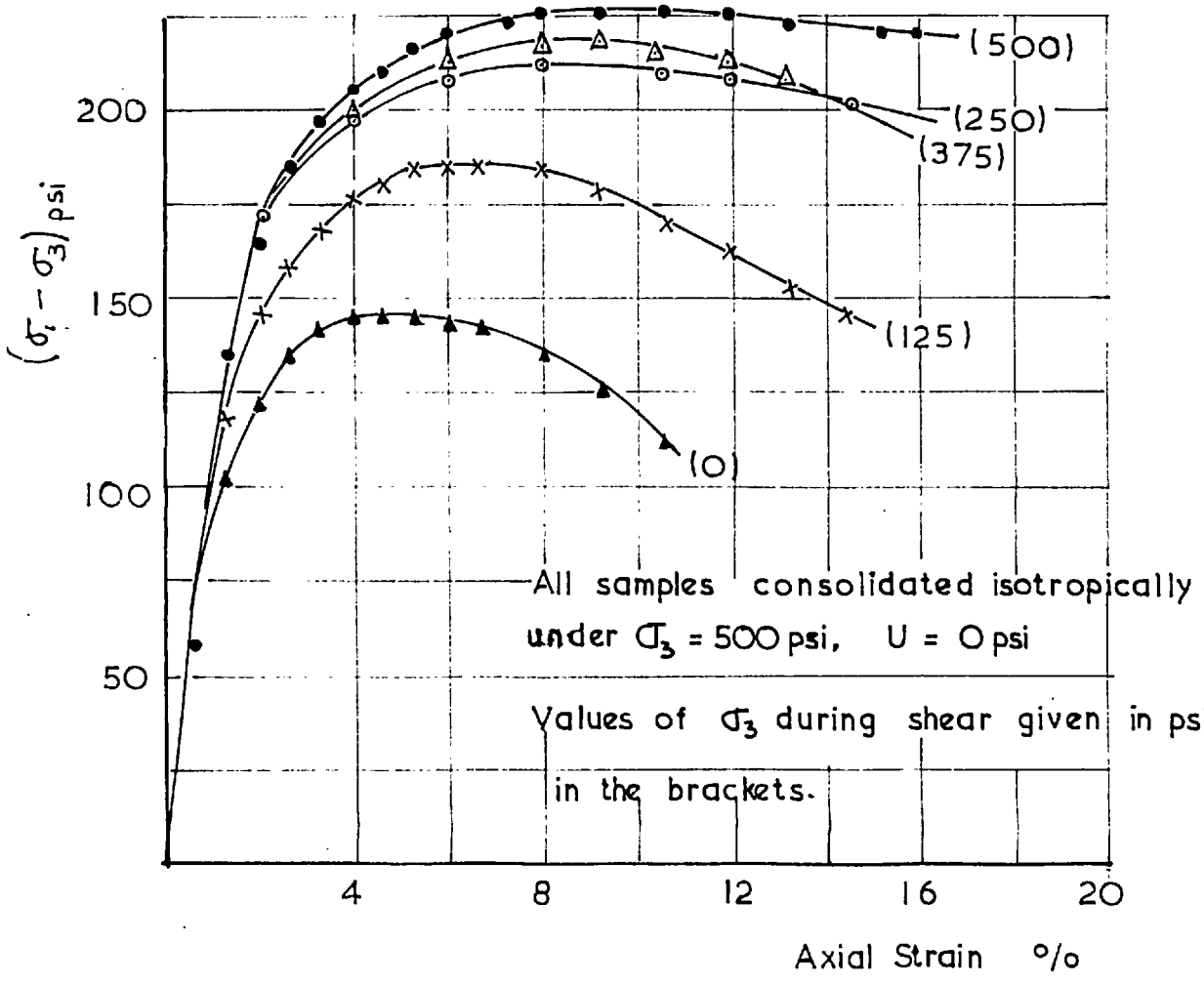


FIG. 9.3

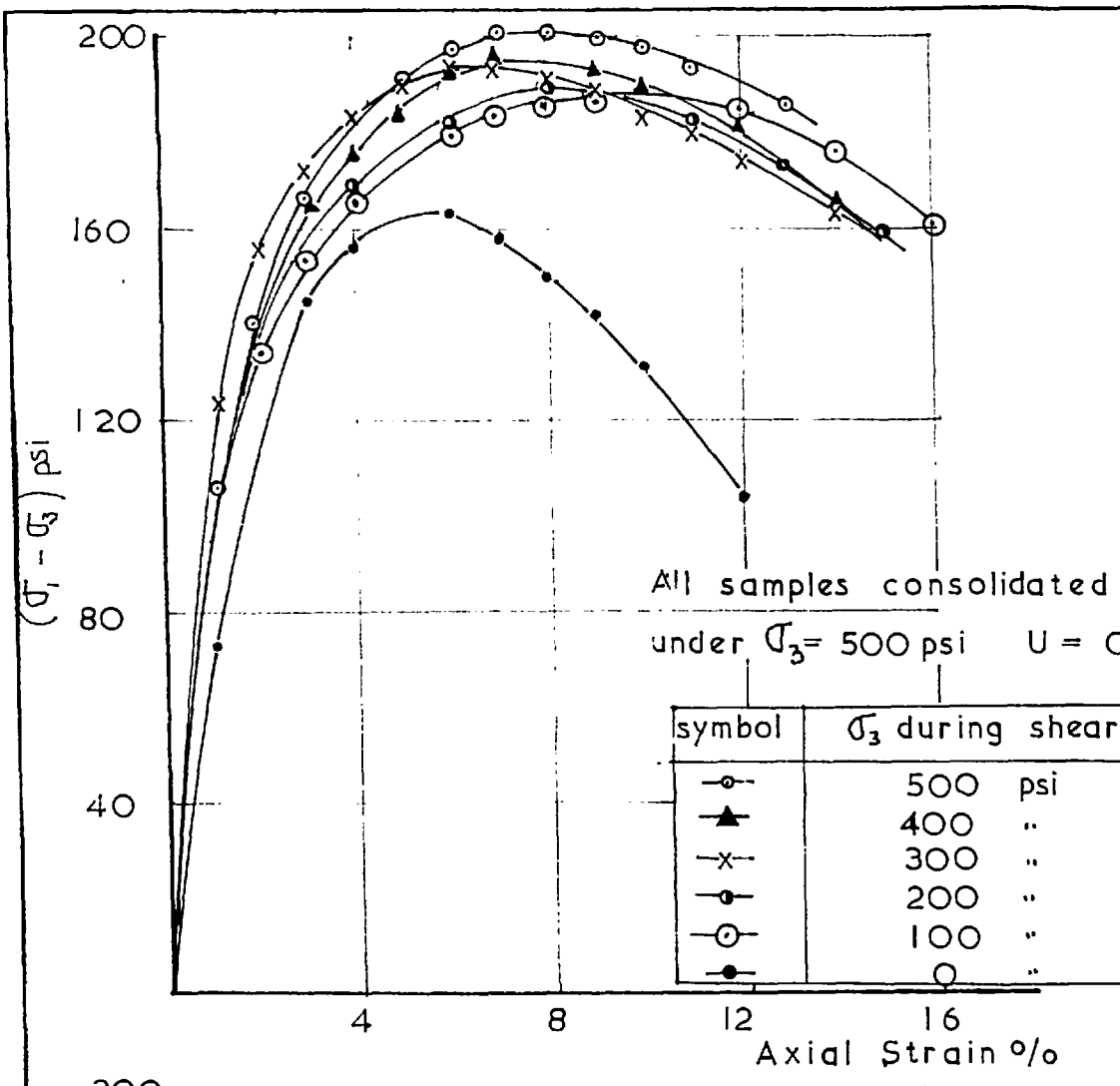
STRESS—STRAIN CURVES FOR TESTS ON LONDON CLAY— BATCH I





STRESS-STRAIN CURVES FROM CONSOLIDATED UNDRAINED TESTS ON REMOULDED LONDON CLAY. BATCH 1

FIG. 9.4



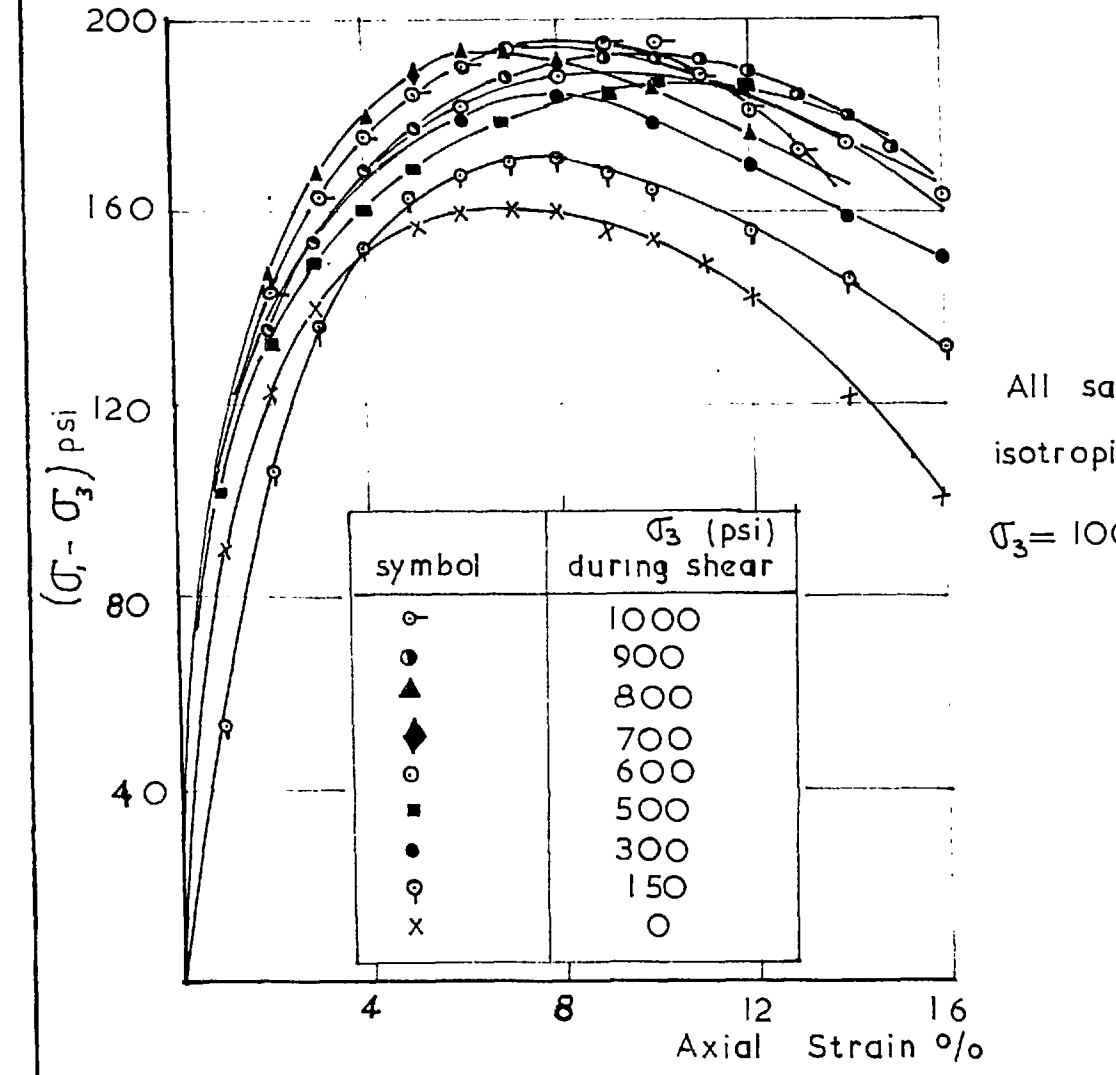
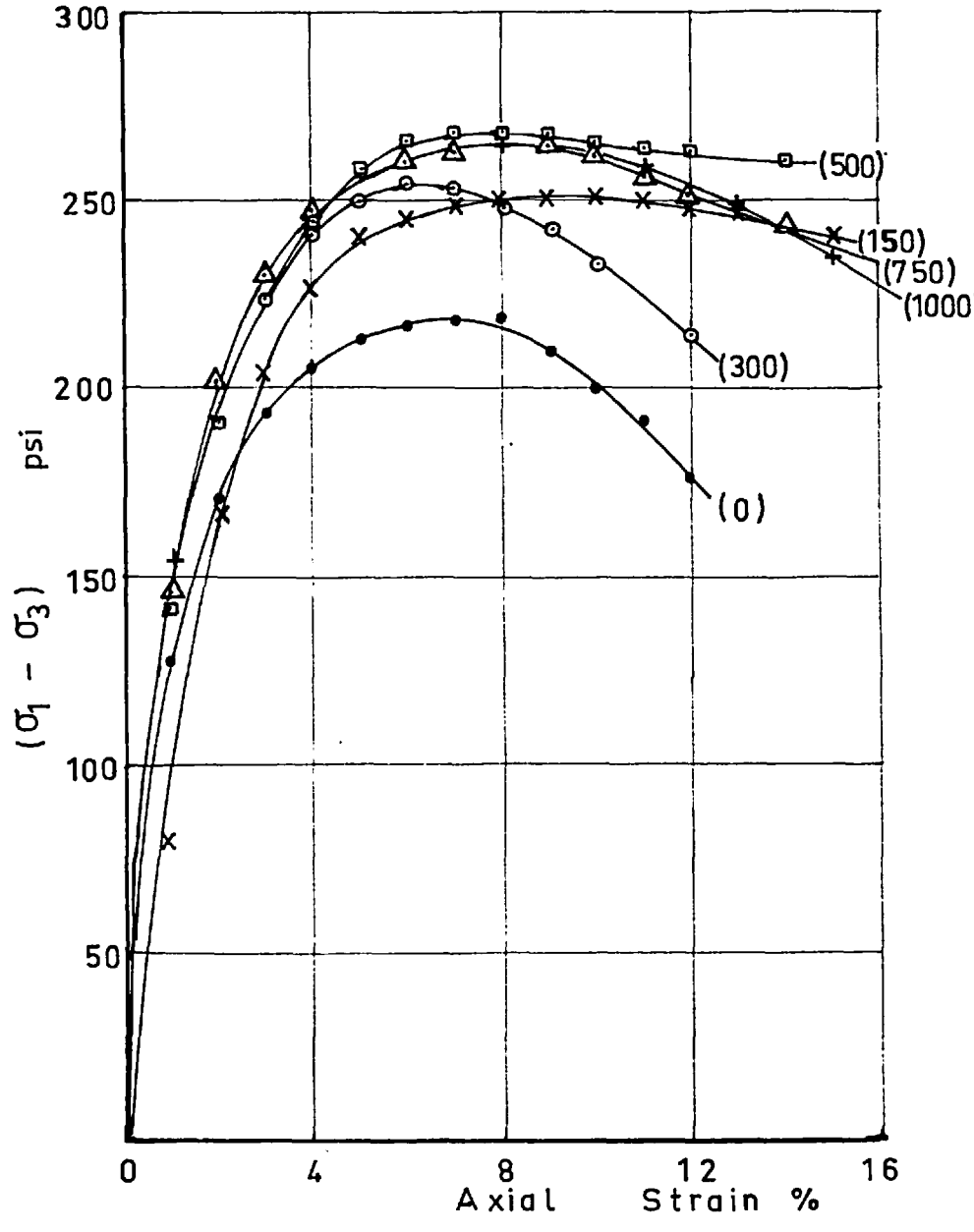


FIG. 9.5

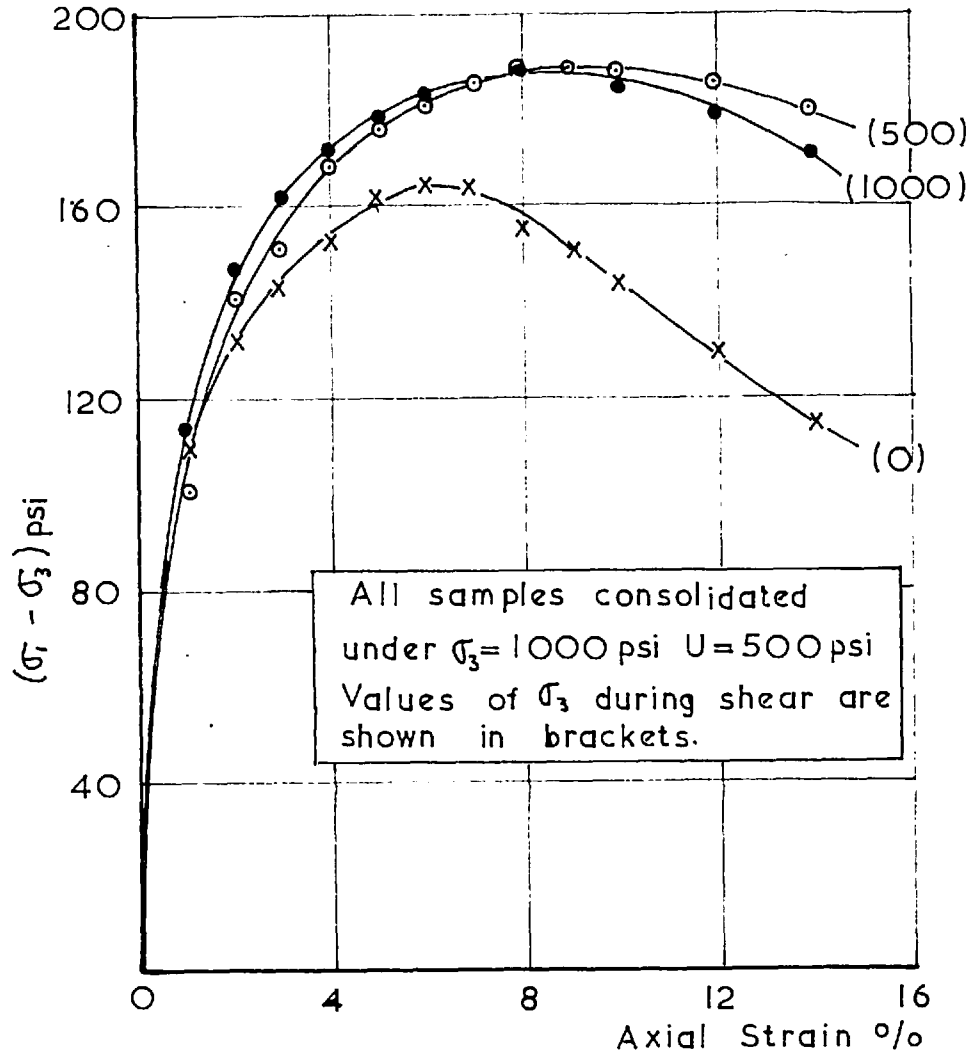
STRESS–STRAIN CURVES FROM CONSOLIDATED UNDRAINED TESTS ON REMOULDED LONDON CLAY. BATCH 2

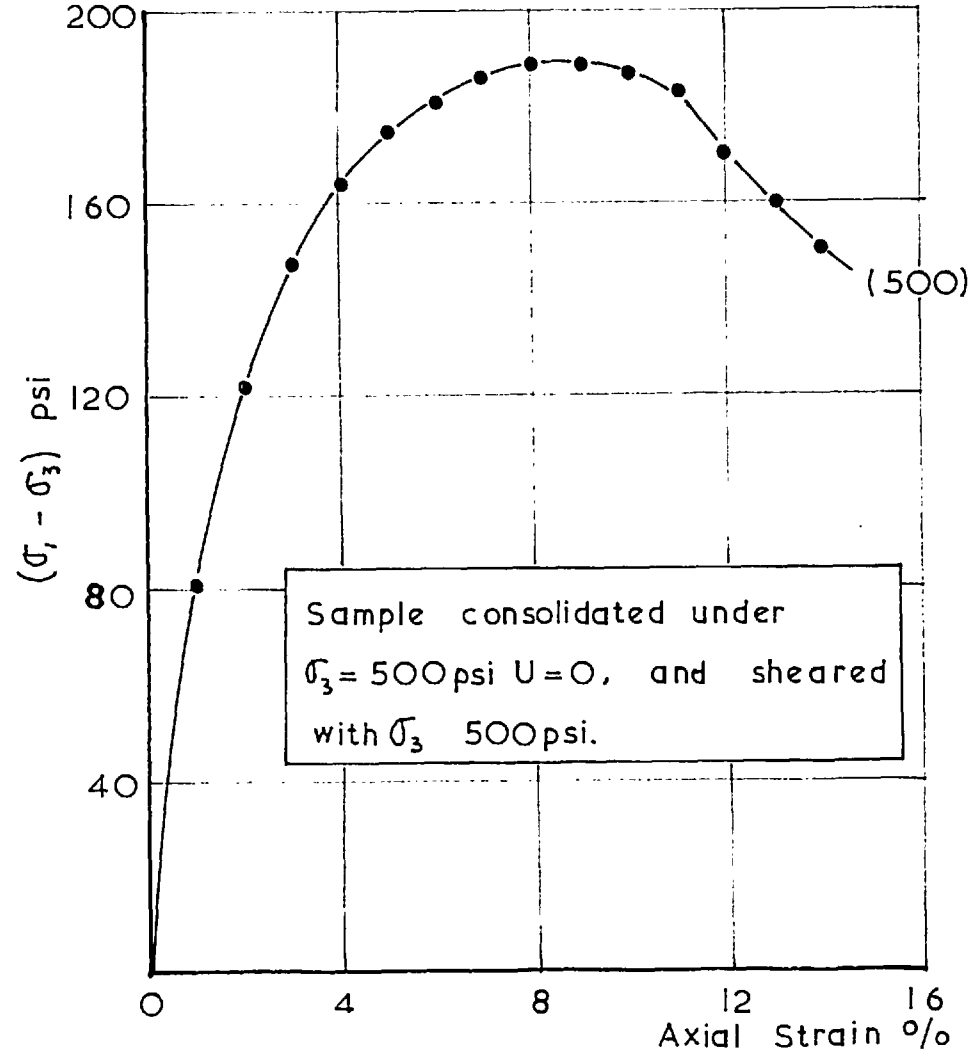


All samples were consolidated isotropically under $\sigma_3 = 1000$ psi, U = 0, and were later allowed to swell back to the condition $\sigma_3 = 1000$ psi & U = 500 psi (O.C.R. = 2). Confining pressures during shear (psi) are given in brackets.

CONSOLIDATED UNDRAINED TESTS ON REMOULDED LONDON CLAY BATCH 2.

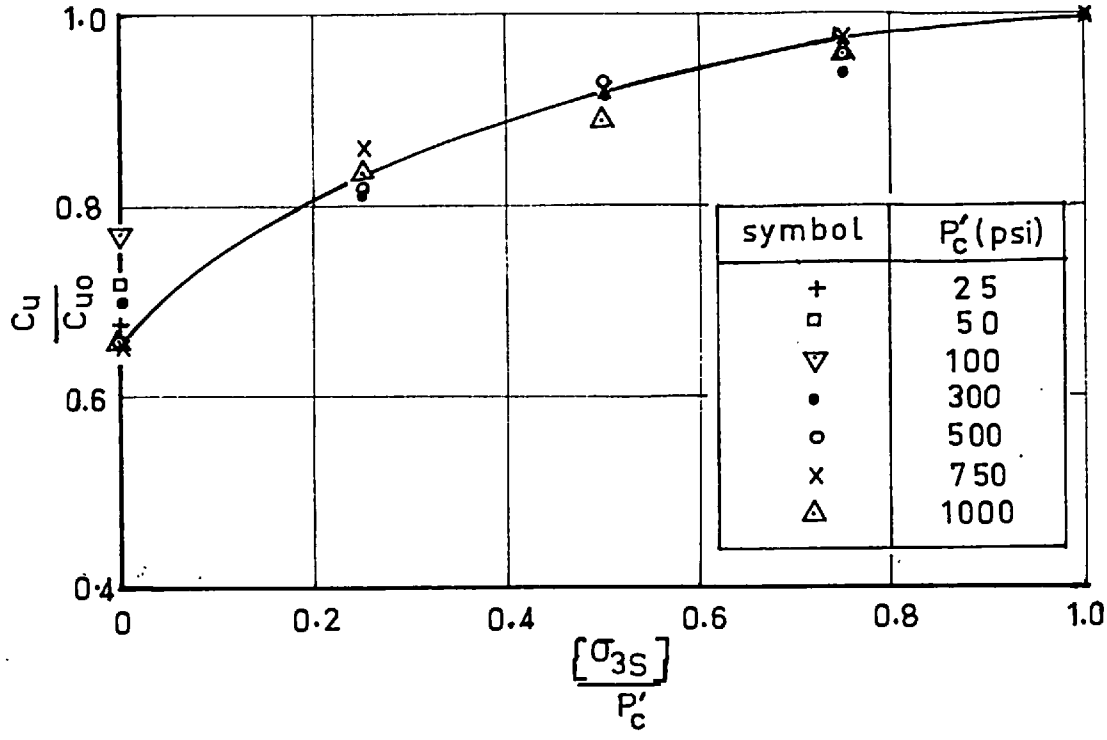
FIG. 9.6



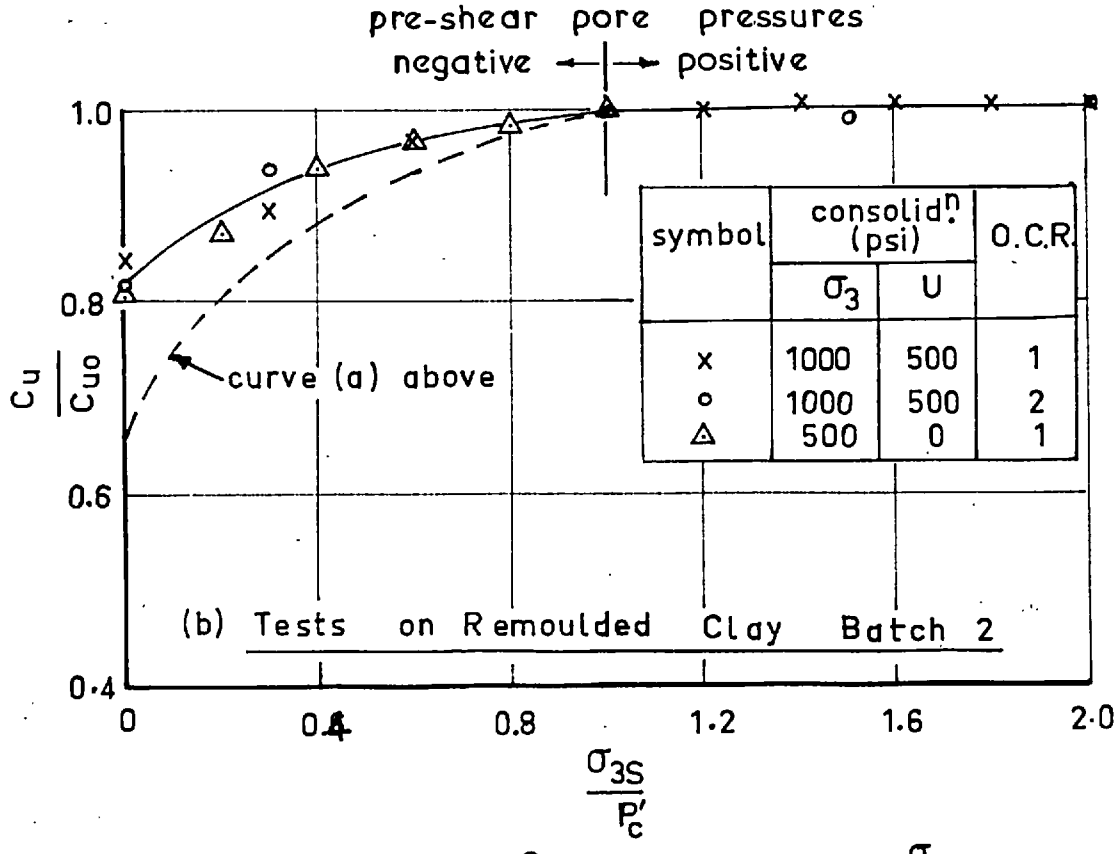


CONSOLIDATED UNDRAINED TESTS ON REMOULDED LONDON CLAY — BATCH 3

FIG. 9.7



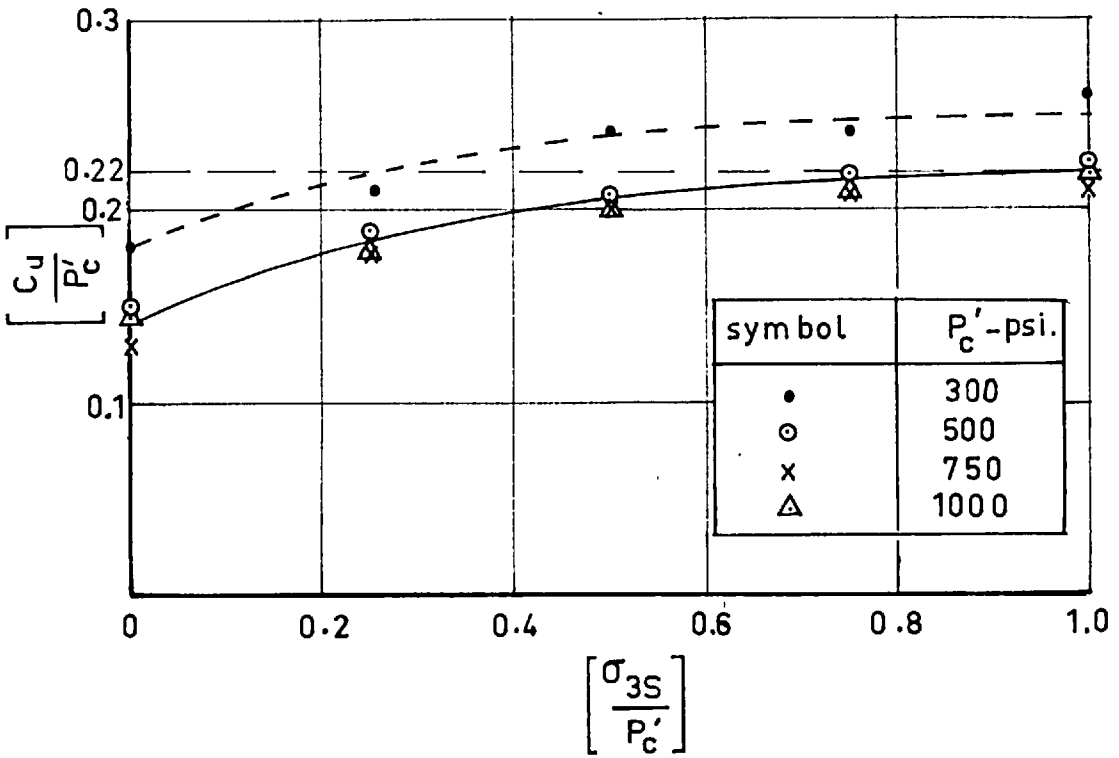
(a) Undrained Tests on Remoulded London Clay Batch 1



(b) Tests on Remoulded Clay Batch 2

GRAPHS OF $\frac{c_u}{c_{uo}}$ AGAINST $\frac{\sigma_{3S}}{P'_c}$.

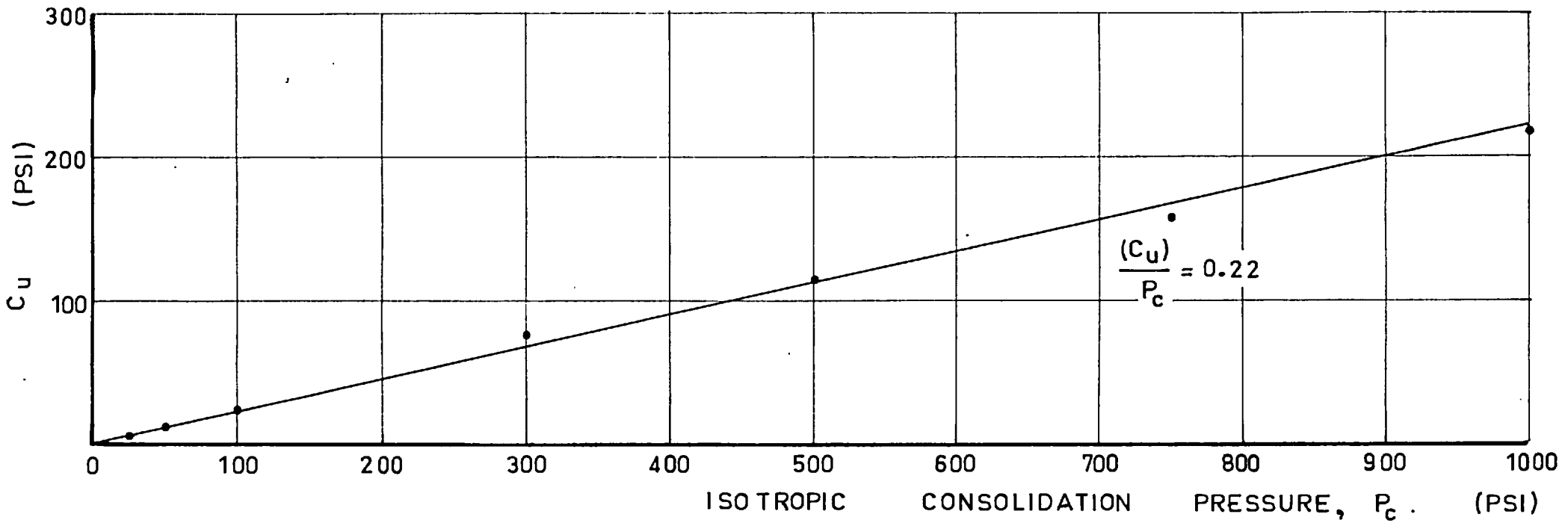
FIG. 9·8



VARIATION OF $\left[\frac{C_u}{P_c'}\right]$ WITH $\left[\frac{\sigma_{3S}}{P_c'}\right]$

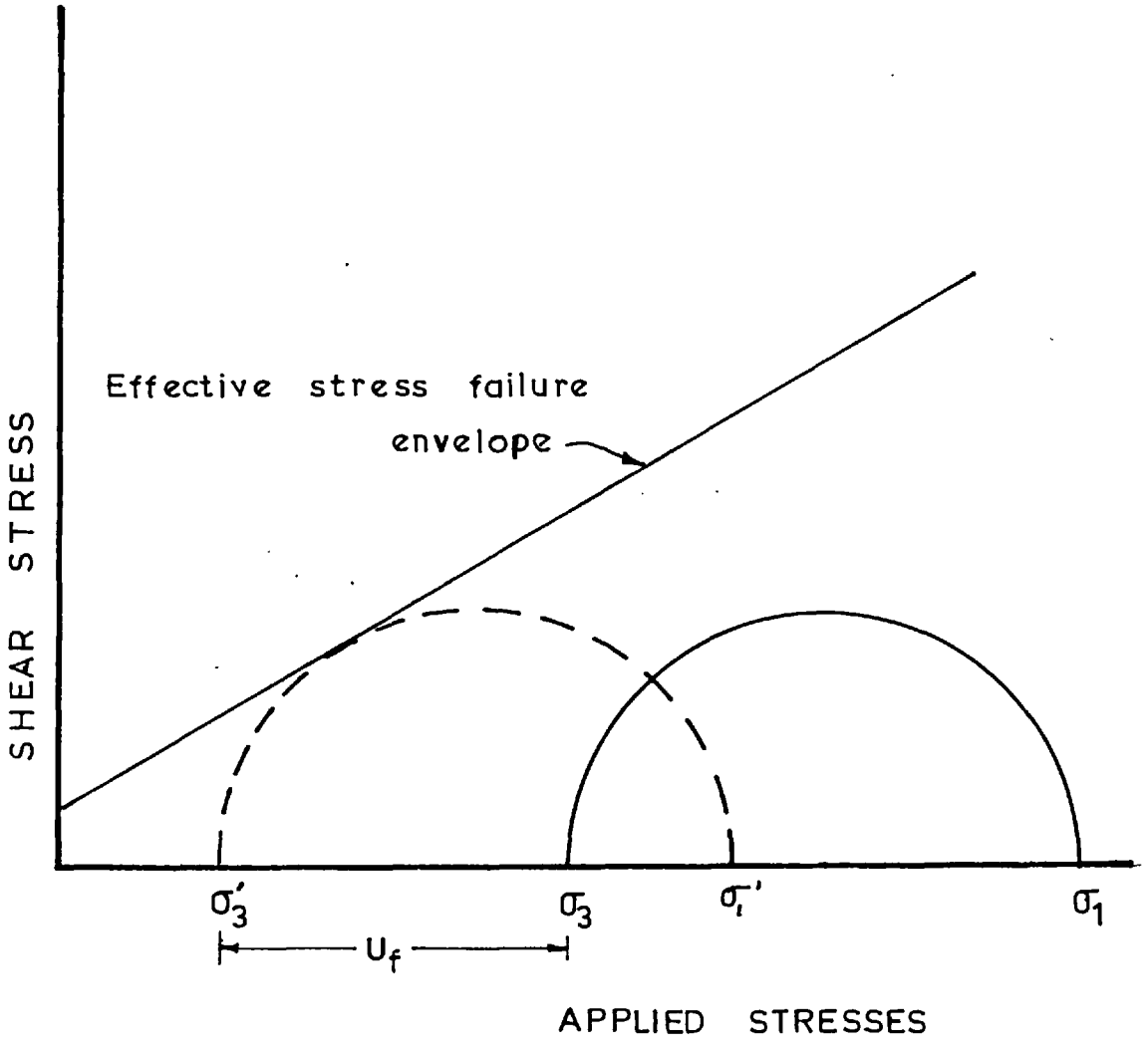
CONSOLIDATED UNDRAINED TESTS ON REMOULDED LONDON CLAY BATCH 1.

FIG. 9.9



CONSOLIDATED UNDRAINED TESTS ON REMOULDED LONDON CLAY BATCH 1.

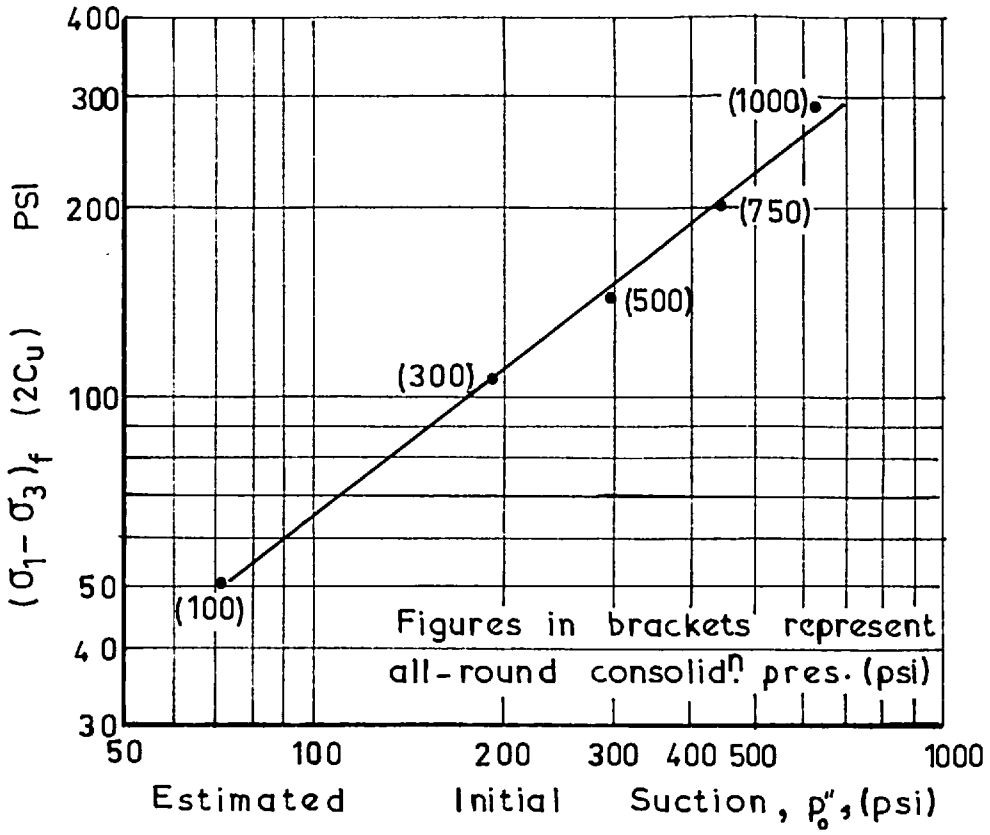
FIG. 9·10



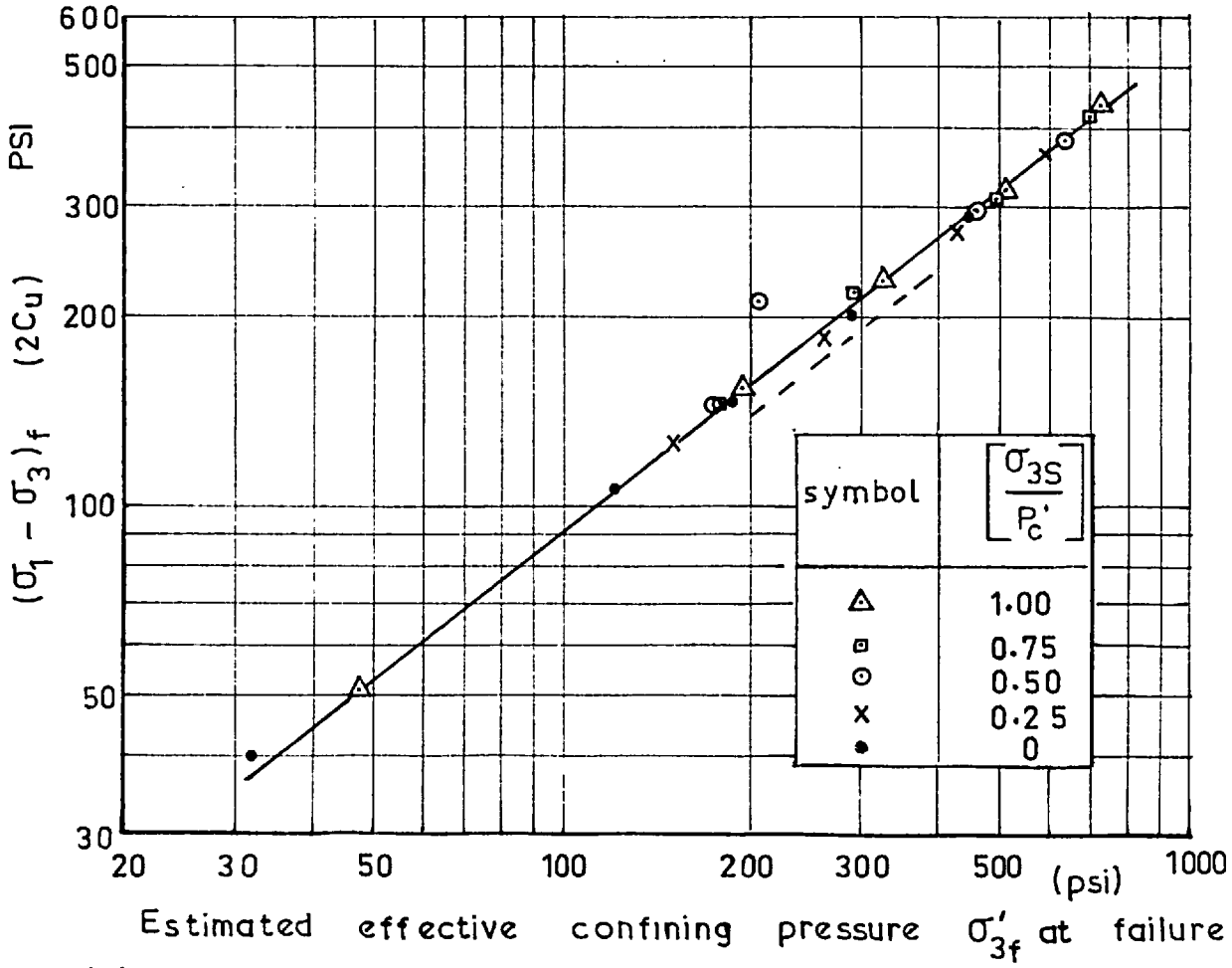
GRAPHICAL DETERMINATION OF THE PORE PRESSURE AT FAILURE.

CONSOLIDATED UNDRAINED TESTS ON LONDON CLAY.

FIG. 9.11



(a) RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH & INITIAL SUCTION IN UNCONFINED SAMPLES.



(b) RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH & σ_{3f} FOR SAMPLES WITH INITIAL NEGATIVE PORE PRESSURES.

FIG 9.12

CHAPTER 10

CONCLUDING REMARKS.

Work presented in the first four chapters of this thesis showed that the best approach to the determination of effective stresses in clays and fine-grained cohesionless soils is probably the experimental one. It has also been shown that for clays (as represented by Wiener Tegel) and fine-grained cohesionless soils (as represented by Calcite), the effective stress in relation to shear strength is given by the Terzaghi equation, and that effective stress in relation to changes in volume is given by an expression which takes account of the compressibility of the soil grains and soil structure. However, this compressibility correction is, in general, so small that very little error would result if the Terzaghi form is used in the normal engineering stress range.

The results presented relate strictly to the influence of the difference between the external applied stresses σ and pore pressure u, i.e. $(\sigma-u)$, on the mechanical properties of these materials, all other factors remaining constant. As mentioned in Chapter 1, the fact that the effective stress concept does not seem to take account of the creep behaviour of soils, has been regarded as a possible shortcoming of the principle. Recent work (Lovenbury, 1969) on creep, though of a limited nature, indicated that the influence of creep may be exaggerated in a stiff overconsolidated clay, like London Clay * ,

* Long term drained tests on London Clay showed a decrease in drained strength of 15% compared with the 5-day strength. This decrease is within the normal scatter of experimental results on the material.

but that it may be important in soft, normally consolidated clays. The writer is of the opinion that the existence of creep does not invalidate the results obtained. Apart from the external applied stresses, other factors such as changes in the chemistry of the pore fluid, have been known to influence the mechanical behaviour of clays. It is conceivable, therefore, that creep could be the result of possible changes in the pore fluid chemistry in the long term, after the applied stresses have become fully effective. Any future research concerning effective stresses in cohesive soils should also include an investigation of the influence of changing the nature of the pore fluid under constant fully effective applied stresses on the mechanical behaviour of these soils. These tests could, if combined with the results presented, lead to a formulation of an all-embracing definition of effective stresses to take account of all the known phenomena in clays.

The results of drained compression tests at elevated cell pressures, on remoulded and undisturbed clays and silts showed that the shear strength and deformation behaviour of these soils are modified in a manner similar to that already noted for sands and gravels by other workers. It has been suggested that the nature of the clay-sized particles may be just as important as their percentage, in determining the mechanical behaviour of fine-grained cohesionless soils under elevated pressures.

Consolidated-undrained tests on samples of remoulded London Clay with large induced negative pore pressures showed that the undrained shear strength decreased with increasing magnitude of the induced initial

* from here

negative pore pressure in the sample. The decrease in undrained shear strength appeared also to depend on the quantity of air dissolved in the pore fluid. The results were explained by postulating a decrease in the degree of saturation of the sample, resulting from a release of dissolved gases in the pore fluid. This was taken as an indirect evidence of the inability of pore water to sustain large induced negative pressures.

On a sudden decrease of the cell pressure under undrained conditions, it is possible that a negative pore pressure response equal in magnitude to the decrease in cell pressure may be obtained momentarily, but this would decay slowly as dissolved gases are released from solution. The pore pressure generated by the application of shear stresses would be superimposed on this negative pressure, resulting in a complex pore pressure pattern. An approximate method has been presented for evaluating the pore pressure at failure, and hence the ambient effective stress at failure and the initial negative pore pressure in the sample at the start of undrained shear. From these, an approximate relationship has been deduced between the undrained shear strength and the two latter quantities.

Owing to the absence of any rapid and reliable method of measuring large negative pore pressures, the evidence presented above regarding the negative pore pressure or suction was, inevitably, indirect. The development of suitable methods of measuring large negative pore pressures would be an asset to further work involving large negative pore pressures. In addition to providing

conclusive evidence regarding the ability of pore water to sustain absolute tensions, it would also facilitate work on the extension of the effective stress concepts developed for positive pore pressures into the large negative pore pressure ranges.

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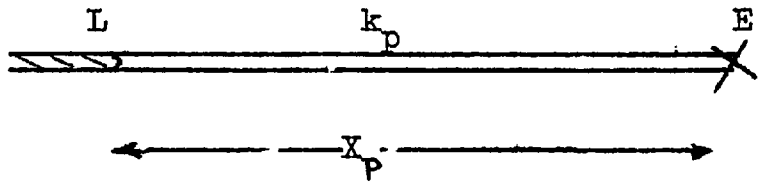
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APPENDIX A

Expression for the correction (ΔV_c) to the readings of volume change.



Let k_p, k_q denote the values of the tubing constant k under pressures p and q respectively.

X_p, X_q denote the lengths of saran tubing LE in circuit under pressures p and q respectively.

$$(\Delta X)_q = X_p - X_q$$

and C denote the compressibility of water, and is by definition equal to $(\frac{dv}{dp} \cdot \frac{1}{v})$

Under pressure P the volume V of water in the length LE of tubing is given by

$$V = k_p X_p \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (A.1)$$

If the pressure is increased to q, there is a decrease (ΔV_c) in the volume V of water given by

$$(\Delta V_c) = C.dp.V = C\ (q-p)\ k_p X_p \quad \ldots\ldots\ldots\ldots\ldots \quad (A.2)$$

New volume of water V_q under pressure q is given by

$$V_q = X_q\ k_q = V-(\Delta V_c) = k_p X_p - C\ (q-p) k_p X_p$$

$$X_q = \frac{k_p X_p\ [\ 1 - C\ (q-p)\ ]}{k_q} \quad \ldots\ldots\ldots\ldots\ldots \quad (A.3)$$

The decreased length X_q of tubing under an increased pressure q therefore reflects the combined effect of the expansion of the tubing and the compression of the water

contained therein.

It can also be written that

$$\Delta V_c = X_p k_p - X_q k_q$$

and since $X_p = (\Delta X_q) + X_q$,

$$\Delta V_c = k_p(\Delta X)_q + X_q k_q - X_q k_q$$

$$\therefore \quad \Delta V_c = k_p(\Delta X)_q + X_q(k_q - k_p) \quad \ldots\ldots\ldots\ldots \quad (A.4)$$

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and when $p = 100$ psi,

$$\Delta V_c = (\Delta X)_q \, k_{100} + X_q \, (k_q - k_{100}) \quad \ldots\ldots\ldots \quad (A.5)$$

The correction ΔV_c is positive for increasing pressures and negative for decreasing pressures.

APPENDIX B

The Principle of Least Squares.

The best curve to fit m sets (x_i, y_i) i=1,2,3,..m, of experimental observations may be obtained by using the principle of least squares*, which yields a curve such that the sum of the squares of the deviations of the experimental points from it, is a minimum. In the absence of any specific indication of the type of mathematical function to fit the experimental data the function is usually assumed to be a polynomial of degree n ($<$m) of the form

$$y(x) = C_o + C_1x + C_2x^2 + \ldots\ldots\ldots C_nx^n \ldots\ldots (B.1)$$

The closeness of fit between the observed data and the "least squares" curve is judged by the Gauss Criterion, which states that, for the best fit, the quantity

$$\Omega = \frac{(y_o - y)^2}{(m - r)} \text{ is a minimum } \ldots\ldots\ldots\ldots (B.2)$$

where (y_o-y) denotes the residual at the point y_o i.e. the difference between the observed value of the parameter, y_o, and the value y, of the same parameter computed from the curve,

m denotes the total number of pairs of experimental data, and

v denotes the total number of arbitrary constants in the polynomial. From equation (B.1) therefore, r = n+1.

The Gauss Criterion was, therefore, used to determine the degree of the polynomial in equation (B.1) to give the best fit.

The computation, which would have been laborious if done by hand, was performed on an I.B.M.7094 computer.

* For a detailed discussion of this and other methods of analysis of experimental observations, see Worthing and Geffner (1960).