THE EFFECT OF SAMPLING DISTURBANCE ON THE DEFORMATION OF CLAY

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BY

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STATEMENT

This thesis is submitted in candidature for the degree of Doctor of Philosophy at the University of Wales.

Except where specific reference to other investigations is made, the data obtained and the work described in this thesis are the result of the candidate's own efforts.

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DECLARATION

I declare that neither this thesis nor any part of it has been presented for any degree at any other University.

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NOTATION

q'	current shear stress
p'	current mean effective pressure
p'_e	equivalent effective pressure on the compression curve
p'_{pf}	equivalent effective pressure on the intersection of the elastic wall with the critical state line
p'_f	effective pressure at failure
v	specific volume
ε_a	axial strain
ε_v and ε_s	volumetric and distortional strains respectively
ε_v^e and ε_v^p	elastic and plastic volumetric strains respectively
ε_s^e and ε_s^p	elastic and plastic distortional strains respectively
$\dot{\varepsilon}_v^p$ and $\dot{\varepsilon}_s^p$	plastic increments of volumetric and distortional strains respectively
a	pore water parameter as $\Delta u/p'$
A	stress pore pressure parameter as $\Delta u/\Delta q$
S	strain-pore pressure parameter as $\Delta u/\varepsilon_s \cdot p'$

M, λ, κ, N and Γ	soil constants in the critical state theory
η	stress ratio of q'/p'
ψ	slope of the flow rule curve in equation 5.7.
C	constant of the flow rule curve in equation 5.7.
t	wall thickness of a tube sample
t/D	ratio of wall thickness to the inner diameter of the sample

SUMMARY

Disturbance of soil samples is cuased by several factors and each affects different aspects of soil behaviour in different ways. The existing literature on soil sampling disturbance is based largely on empirical comparisons and it is mainly concerned with the effects of disturbance on strength. This thesis considers stress disturbance and mechanical distortions as the main causes of sample disturbance and it deals with their effects on the fundamental behaviour of normally consolidated clays. In addition, the behaviour of virgin and disturbed samples is examined mainly within the concepts of the critical state theories. Other seconday causes of disturbance are also examined and these include the influence of the wall thickness of the tube sampler and the storage period.

Different triaxial compression tests were performed on samples of remoulded kaolin, prepared in different ways to simulate a virgin sample, a perfect sample and a tube sample. All virgin samples were prepared from a slurry and consolidated anistoropically in a triaxial cell. A perfect sample was obtained from a virgin sample by releasing the total stresses in undrained conditions and tube samples were obtained by inserting 1½" diameter tubes into a 4" diameter perfect sample.

The main testing programme on virgin and disturbed samples, included, drained and undrained tests at different stress levels but, in addition, there were other tests to examine aspects of the virgin behaviour and effects of loading conditions and laboratory procedures relevant to the main testing programme.

The results show the behaviour of virgin and perfect samples is mainly in agreement with the concepts of critical state theories, but the stress disturbance alters not only the state, but also the values of the virgin soil parameters. The results show also that tube samples behave as if they are overconsolidated, and their state boundary surface is substantially different to that of virgin samples. It was suggested that the effects of sampling disturbance may be greatly reduced by reconsolidating disturbed samples to a substantially higher stress level, in which case, it is necessary to normalise the test data to the virgin state.

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CHAPTER 1 GENERAL INTRODUCTION

1.1. INTRODUCTION

In geotechnical engineering, laboratory and in-situ testing of soils is undertaken for a number of reasons. For example, engineers test soil to obtain design parameters and research workers test soil to investigate the fundamental behaviour of soils. Laboratory testing is important in this work because of the well defined loading conditions and the accurate measurements of deformations that are possible. Over the last few decades there have been considerable improvements in measuring techniques, in methods for controlling the apparatus and in understanding the soil apparatus interferences but it is still necessary to obtain representative soil samples.

Soil, unlike other building materials such as steel and concrete, is path dependent. A sample of soil obtained by any sampling process must experinece an unavoidable cycle of its stress state and in most cases it suffers additional distortions and both contribute to disturb the sample. If a soil sample is disturbed, its behaviour may not represent the original behaviour even if it is under the same stress state as the soil in the ground.

In this chapter, the problem of sample disturbance is introduced and current sampling techniques are reviewed in general. The main causes of disturbance are discussed and a terminology is suggested to describe the quality of soil

1.2. SOIL SAMPLING IN GEOTECHNICAL ENGINEERING

1.2.1. INTRODUCTION

Natural soils in the field are non-homogeneous in their stress state, composition and structure and thus soil samples can only represent average properties of the in-situ mass of soil. Samples at various depths and locations are necessary to classify soil and to examine its mechanical behaviour. A disturbed sample which has suffered no change in its constituents may be used for classification purposes but a nominally undisturbed sample is necessary to investigate the mechanical behaviour of the soil.

Soil sampling techniques attempt to obtain specimens that preserve the important mechanical behaviour of the in-situ soil and various samples and methods of sampling have been developed to obtain nominally undisturbed samples. The differences between these depend on the purpose of testing, the type of soil, economical and practical restrictions and other secondary factors as described by Hvorslev (1949). The reader is referred to the proceedings of the International Symposium of Soil Sampling (1979) for further details on practices of soil sampling.

Basically there are two types of soil sampling, tube sampling and block sampling. Hand cut block samples are usually considered to be of the best quality but they cannot always be obtained.

1.2.2. BLOCK SAMPLING

A block sample is usually taken from an excavation dug carefully to the required depth. A column of soil, generally square in cross-section is formed by careful digging of the surrounding soil and the sides trimmed to the required size. A box is assembled around the column of soil and the gap between them filled with wax. The top of the box is then screwed to the sides and the base of the column is disengaged with several passages of a wire along the bottom face. Further trimming is required to the bottom of the sample before pouring the wax and assembling the bottom of the box. For transportation, and storage, samples should be wrapped in an inner plastic sheet and shipped in the upright position. Storage of samples is usually in a room of controlled temperature and humidity. During trimming, the block sample is rigidly supported and a thin wire saw used to cut the block sample to the required shape and size. The sample may then be transferred and set up in a triaxial apparatus for further loading.

1.2.3. TUBE SAMPLING

This is the most common method of obtaining soil samples. A tube sampler is a relatively thin walled cylinder and different samples have slightly different geometrical proportions and details. It is attached to a shaft and the idea of tube sampling is to push the sampler into the soil. The sampling process consists of a number of operations.

First a vertical borehole is drilled or an excavation made in the ground to the depth at which the sample is required. There are various methods used in advancing the borehole depending on the regional geology, the equipment available and the experience of the drilling operators. The sides of the hole are often stabilised by casing and the bottom of the hole is cleaned. The sampler is pushed into the soil without rotation by hammer driving, hydraulic pressure or by mechanical jacking. At the end of the drive the sampler is rotated to shear the sample from the parent material and it is jacked from the borehole. In some samplers the vacuum created as the sampler is withdrawn is broken by a vacuum breaking device. For transportation and storage the samples are usually sealed in the sampler or in liners with wax. For testing the tube sample is pushed out manually or by hydraulic pressure, the two ends cut and levelled to the required length and the sample is then set up in laboratory apparatus.

1.2.4. SAMPLERS

Various tube samplers have been developed over the last 50 years or so and many of these are discussed by Kallstenius (1958) and Muhs (1969). Designs vary to suit different soils and the principal objective is to decrease the mechanical distortions of the soil sample. Various arrangements are used to prevent the sample dropping from the other end of the sampler and different sizes of cutting edges at the end of the sampler are provided.

Figure 1.1. shows some typical shapes and defines a number of ratios such as the wall thickness/diameter ratio and the recovery ratio, which seems to describe a sampler and its performance.

Basically, there are two kinds of tube samplers, an open drive sampler and a piston sampler. An open drive sampler consists of a plain section of pipe, levelled at the lower end and secured to the bottom of the drill pipe by means of an adapter containing a vent for escape of air or drilling fluid as shown in figure 1.2. The thin wall open drive sampler has a sharp cutting edge and is crimped to a smaller diameter to allow an inside clearance while a thick wall sampler can be used with a wider variety of soils, and it is provided with a detachable shoe and cutting edge of hardened steel.

A piston sampler is simply a tube sampler with a piston placed at the bottom of the hole as shown in figure 1.2. Piston rods extend from the sampler to the surface through the drill pipe to control the piston's movement. The tube is advanced beyond the piston into the undisturbed soil and thus the piston prevents the entrance of excess soil into the tube and it helps in holding the sample in the tube during recovery.

1.2.5. COMPARISONS OF BLOCK AND TUBE SAMPLES

Among the current techniques of sampling, block sampling gives the best quality sample of soil because of the negligible deformations and distortions to the soil structure if care is exercised in excavation, preservation and preparation of block samples.

The block sampling method is advantageous in cases where large sized samples of soils are required and when soils contain gravel and pebbled size particles. The cost and depth limit of excavations and other practical problems restrict the practice of block sampling.

Tube sampling is a cheaper, easier and faster method where deep samples of soils are required but the samples may suffer an appreciable amount of deformation and mechanical distortions as soil is displaced to make way for the sampler tube. There has been no practical alternative yet to tube sampling. Modified designs of samplers have succeeded in decreasing the mechanical distortions but seem so far to have failed in eliminating them.

The basic differences between block and tube samples lie mainly in the distortions suffered by tube samples and both samples must be handled, trimmed and set up in an apparatus for testing which may cause further disturbance. Presumably, if a sample could be taken with a tube with infinitely thin walls it would be equivalent to a block sample.

1.3. NATURE OF SAMPLE DISTURBANCE

1.3.1. INTRODUCTION

When a sample of soil is transferred from its position in the ground to the laboratory, it will suffer changes in its stress and possibly also to its structure and these changes will alter its behaviour. The term sample disturbance has been used previously in the literature of soil mechanics to cover all changes in the behaviour of the soil without considering the nature and the causes of disturbance or separating their effects.

The disturbance experienced by a sample of soil due to sampling is caused by several factors which are different in nature. The influence of each factor may vary with the type of soil but in general the most important ones are stress disturbance, mechanical disturbance and the period between sampling and testing. Other factors such as the environmental changes in temperature, pressure and the chemistry of the pore fluid may be more significant in ocean floor samples or quick clays.

1.3.2. STRESS DISTURBANCE

An element of soil suffers changes of stress before, during and after a sampling process depending on the method of sampling used. The vertical stress at the bottom of the borehole during drilling is reduced and once the soil is removed the soil layers below the borehole tend to deflect upwards and for large stress reductions and for soils with low shearing resistance there may even be failure and plastic flow of the soil. Figure 1.3. shows some of the forces that may be associated with driving tube samples.

The most important stress change however, is the release of the total overburden stresses. Figure 1.4. shows the state of a sample in the ground under in-situ stresses and the state of the sample under zero total stresses on the laboratory bench.

This involves the change from an anisotropic to an isotropic stress state and hence the removal of the shear stress, together with a reduction of the mean stress which leads to the development of suction in the pore water. Soils with relatively low permeability may be assumed to remain undrained and thus no swelling or consolidation takes place and no change in the overall specific volume, but this will not always be the case. The undrained release of the total overburden stresses is the most significant stress change, unavoidable in any sampling process and hence it will be called stress disturbance.

Axial and lateral deformations at the boundaries and within the sample will occur because of the changes in stresses. Assuming isotropic properties for soils, then in theory, stress disturbance will cause negative axial strain and positive lateral strain for normally consolidated clays where $K_o < 1$ and it will cause positive axial strain and negative lateral strain for heavily overconsolidated clays for which $K_o > 1$ as shown in Figure. 1.4. For the special case of soils under an isotropic stress state in the ground there will be no strains at the boundaries of the sample.

1.3.3. MECHANICAL DISTORTIONS

The amount of mechanical distortions is negligible in block sampling while in tube sampling it depends on the sampler size and wall thickness, the method of sampling and the type of soil.

The sample may suffer different types of mechanical distortions during sampling such as shocks, vibrations, local remoulding and the displacement of soil by the wall of the sampler. Friction between the sampler and the soil causes stress changes and it may cause mechanical distortions for soils with low sensitivity, by inducing failure planes in layers at the edge of the sample. Nevertheless, it seems that the most significant cause of mechanical distortions in tube sampling is due to displacement of soil by the wall thickness. Displacement of soil by the sampler wall may cause shear failure and plastic flow in soils below and within the sampler and the extent of these deformed zones varies with the wall thickness. It may be assumed that there is no overall change in the moisture content and no overall change in volume although,there may be movement of water and hence volume changes within the sample.

1.3.4. TIME EFFECTS

Soil behaviour is time dependent and hence the period between sampling and testing may contribute to disturbing the sample. The sample is usually sealed within its tube or box on site to avoid any immediate loss of moisture content and it is then transferred to the laboratory where a storage period can be extended to a few days or a few weeks before a test is conducted.

Basically, there are two different time dependent phenomena which take place in the sample; equalisation of excess pore pressures induced by stress disturbance and mechanical distortions and secondary effects, such as creep, thixotropy and relaxation. Pore pressure variations within the sample may be due to the variation of structural distortions and local drainage and the equalisation of the pore pressures will take place as time passes. These processes commence immediately after sampling and continue at a diminishing rate until excess pore pressures are negligible. Secondary effects include physical changes in bonding and they usually take place in proportion to time and may depend on the stress state (i.e. whether the disturbed sample is loaded or not, during its storage period). The significance of secondary effects on the behaviour seems to be more apparent in plastic clays with high activity (Mitchell, 1976). There are other changes that may take place, such as oxidation, chemical reactions or biological effects but these are usually of minor importance.

1.4. DEGREE OF SAMPLE DISTURBANCE

1.4.1. INTRODUCTION

The quality of a smaple determines the extent to which it preserves the in-situ properties and thus it is necessary to judge the quality of a sample before testing. So far as the mechanical behaviour of the soil is concerned, Hvorslev (1949), classified soil samples as either non-representative, representative or undisturbed.

A non-representative sample is one which contains soil from other strata or is missing some of the soil constituents, while a representative soil sample is one in which there is no change in soil constituents but whose structure, water content or void ratio may have been altered. An undisturbed sample ideally is one that represents completely the in-situ behaviour.

The term undisturbed sample,is commonly used in soils literature to describe a relatively or an apparently undisturbed sample. In fact, the sample obtained by any sampling method is not really undisturbed and it is questionable whether it is possible to obtain an undisturbed sample at all, because any sample must suffer stress disturbance. This section will introduce several terms that may be used to describe the quality of samples.

1.4.2. VIRGIN SAMPLES

Samples which have suffered no disturbance (i.e. truly undisturbed samples) will be known as virgin samples. A virgin sample may occur either in the field or in the laboratory. A field virgin sample is found only in the field lying in its in-situ position and it cannot be transferred to the laboratory or removed from its position. The behaviour of a field virgin sample cannot be measured directly but it may be found by different methods, such as the back analysis of the full scale behaviour and in-situ tests or by using the normalised behaviour of some disturbed samples in the laboratory after reconsolidation.

A laboratory virgin sample may be prepared in the laboratory by loading a sample along a stress path which copies that of a field virgin sample. This may be done in two ways; either by K_o consolidation of a slurry to the field's stress level and state, or by K_o reconsolidation of a disturbed sample obtained from the field to a higher stress level. The laboratory virgin sample resting in the laboratory has suffered neither mechanical distortions to its structure nor alterations to its virgin stress state and therefore it may be used as reference for investigations of sample disturbance.

It is important to notice that the field virgin sample cannot be duplicated in the laboratory. The K_o consolidation of slurry in the laboratory is conducted on a short time scale and thus the sample lacks structures that may have been developed in the field virgin sample over longer periods. Furthermore, the fabric of a disturbed sample is partially distorted during sampling and it may suffer even further distortions rather than recovery if loaded to higher stress levels.

1.4.3. DISTURBED SAMPLES

Disturbed samples are obtained either during sampling an in-situ virgin sample or from conditioned alterations to a laboratory virgin sample. On the basis of the nature and degree of the disturbance, disturbed samples may be classified into a number of types.

A perfect sample is obtained by releasing the total stresses applied to the virgin sample in undrained conditions. A perfect sample can be obtained in the laboratory and theoretically it may be obtained from a field virgin sample by using an infinitismally thin frictionless tube sampler or by cutting and digging soils without subjecting them to any deformations apart from those caused by releasing the total stresses. Thus a perfect sample is assumed to suffer stress disturbance only with no further stress changes or mechanical distortions.

A block sample represents a sample obtained with a minimum of mechanical distortion and it may be obtained from a virgin sample in the field or in the laboratory by block sampling techniques as described in section 1.2.2. A tube sample represents a sample obtained with some degree of mechanical distortion. The degree of distortion will depend principally on the geometry of the tube sampler and the method of sampling. Exploration samples are those which are taken without much attempt to preserve the mechanical properties of the soil and are usually for identification and classification purposes. They are generally taken when the subsurface conditions are unknown and a survey is required to determine the location and types of soil present. A remoulded sample represents a severly disturbed sample and is achieved by completely remoulding the sample.

Sometimes the sample is dried and remixed with water and sometimes it is remoulded by hand or by machine at its initial water content and this has the effect of completely altering any initial fabric. Thus, a remoulded sample has a dispersed fabric which has no relationship to the fabric of the virgin sample. Remoulded samples are used to obtain soil parameters like the Atterberg limits and they may often be used to obtain soil parameters which are not thought to depend on the fabric of the sample.

1.4.5. MEASUREMENTS OF SAMPLE DISTURBANCE

There are a number of ways of measuring the degree of sample disturbance and apart from pure academic interest, they have practical uses in correcting the observed behaviour of disturbed samples. Engineering parameters such as strength and stiffness, have been used to indicate the degree of disturbance and to correct the value of other engineering parameters. Different causes of disturbance (e.g. stress disturbance) may influence different engineering parameters (e.g. strength, stiffness) differently and it may not be justifiable to correct some aspects of the behaviour of the disturbed sample (e.g. stiffness) on the basis of other aspects of the disturbed behaviour (e.g. strength). The properties of a block sample are often used as a reference since it is impossible to obtain perfect samples for laboratory testing or to measure the properties of the field virgin samples. Alternatively, in-situ testing and back analysis of the full scale behaviour may be used to obtain true values which can then be used as a reference for the

On the other hand the properties of a remoulded sample clearly represent the highest level of mechanical distortion and it may be used as a measure of disturbance.

Two main methods are commonly used to measure disturbance. These are based on measuring the residual effective stress and one-dimensional consolidation curve of a disturbed sample. There are other methods and some of these will be discussed in chapter 3.

1.5. THE BEHAVIOUR OF A DISTURBED SAMPLE IN THE LABORATORY

1.5.1. INTRODUCTION

A disturbed sample may be reloaded and tested following a wide range of stress states, stress paths and loading conditions and additional stress changes and mechanical distortions may be applied to disturbed samples in order to investigate their effects on the measured degree of disturbance. Reloading disturbed samples to higher stress levels may be used to reduce or even eliminate the effects of sampling disturbance or, alternatively, correction parameters based on the measured degree of disturbance may be used in predicting the virgin behaviour from disturbed samples.

1.5.2. RECONSOLIDATING THE DISTURBED SAMPLES

In practice, disturbed samples are prepared for testing either with or without reconsolidation. Theoretically in an unconsolidated undrained test the confining pressure has no influence on the effective stress state of a saturated clay. Isotropic reconsolidation may be carried to any stress level but it is a common practice to reconsolidate the sample to a stress equivalent to the in-situ stresses. It is important to notice that under isotropic stress the sample will be at a different stress state from its anisotropic state in the field and this will lead to further differences in the behaviour. Anisotropic reconsolidation to the field stress state may be conducted in the laboratory and this will eliminate the differences that are caused by different stress states (Davis and Polous, 1967).

It is believed that reconsolidation to a stress level higher than that in the ground will eliminate the effect of stress disturbance (Ladd and Foott, 1974) and may reduce the effects of disturbance and non-homogenity caused by mechanical distortions of tube samples. However, this may not always be the case, since the fabric of some clays may be largely destroyed by volume changes, shear strains and higher stress levels and it is necessary to know by how much the stress level should be raised in order to eliminate the effects of stress disturbance. Reconsolidating the sample to a higher stress level and consequently to a lower specific volume will increase the strength and stiffness and it is necessary to normalise the behaviour of soils by a parameter that describes its specific volume in order to account for the differences.

1.5.3. APPLICATIONS OF ADDITIONAL DISTURBANCE

The effects of distrubance on the virgin behaviour may be estimated empirically by investigating the effects of applying additional disturbances to disturbed samples.

The alterations to the behaviour of the disturbed sample may be correlated with the nature and degree of disturbance and, by extrapolation, used to estimate the behaviour of a virgin sample. This approach requires a series of tests on samples at different degrees of disturbance, but the different causes of disturbance should be considered separately.

This approach has been used to account for the effects of mechanical distortions (Schmertmann 1955) and the stress disturbance (Seed, Norrany and Smith, 1964). The mechanical distortions may be increased by increasing the wall thickness/diameter ratio, t/D, of a tube sample. Similarly, stress distrubance may be increased by increasing the number of undrained unloading cycles and the virgin behaviour may be calculated by extrapolating to zero stress cycles.

1.6. IN-SITU BEHAVIOUR OF SOILS

1.6.1. INTRODUCTION

The behaviour of disturbed samples in the laboratory may be examined in the view of in-situ testing and full scale behaviour to assess the effects of sampling disturbance and furthermore, in-situ testing may be considered as an alternative to laboratory testing. Therefore it is relevant to examine some of the differences between the soil's laboratory behaviour and the field behaviour in order to clarify the need for laboratory testing instead of other approaches.

Apart from sampling disturbances, there are significant differences in the scale of the sample considered, the rate of loading and the loading conditions in each case. The differences in loading conditions include factors such as the influence of the intermediate principal stress and changes of the direction of the principal stress axes during loading. Hence the differences between the behaviour of soils measured in the laboratory and in-situ may be considered as differences in apparent behaviour, rather than differences in fundamental behaviour of soils and further analysis is required to obtain the parameters that represent the fundamental behaviour of the material.

Laboratory testing has the drawback of sampling disturbance, the lack of representation of the mass of soil in the ground by small samples and apparatus interference and all these factors must be taken into account while in-situ tests involve less disturbance and they may be conducted on a relatively larger scale. Nevertheless, laboratory tests are easier, cheaper and quicker and they are conducted under well defined conditions.

1.6.2. FULL SCALE BEHAVIOUR

Engineers are interested in evaluating the parameters and relationships that describe the behaviour of an element of soil to enable them to predict the full scale behaviour of in-situ soils. The behaviour of soil elements may be examined through laboratory tests, in-situ tests and back analysis of full scale behaviour. Full scale testing is seldom possible, it is time consuming and expensive and therefore case histories of constructed structures may be

The samples in the laboratory may be assumed to represent an element of soil under well defined loading conditions, while the overall full scale behaviour is that of a non-homogenous mass of soil with loading conditions defined at the boundaries only. Furthermore, the full scale behaviour may depend on the features that are not represented in a relatively small specimen in the laboratory or in in-situ tests and therefore it is not always appropriate to compare directly parameters measured in the laboratory with those calculated from observation of the overall behaviour of in-situ soils.

Different assumptions and procedures may be used to obtain the fundamental behaviour and properties of soil elements by back analysing the overall behaviour of soil structures. Back analysis involves assumptions of uniformity, homogeneity, stress distribution, drainage paths, idealisation of the soil behaviour and the boundary conditions. There may not be a unique solution for the particular case under examination and therefore back analysis cannot always provide a reference for the virgin behaviour of soil in-situ. Alternatively, the behaviour of soil in the laboratory may be used to predict the overall behaviour of a structure and thus to evaluate the overall errors involved in the calculations. Thus, predictions made by conventional methods of analysis and based on idealisations for soil behaviour are subject to errors which may or may not be compensated by disturbance or other sources of errors.

1.6.3. IN-SITU TESTS

In-situ testing on a relatively small scale sample may be used to examine the behaviour of elements in-situ, to obtain design parameters and to evaluate the degree of disturbance of laboratory samples. For some soils, such as chalk, it is difficult to obtain nominally undisturbed samples and thus, only in-situ testing is possible.

In-situ tests are affected by disturbance and the analysis of the behaviour requires assumptions about the state and loading conditions of the in-situ elements. There are inevitably stress disturbances and mechanical distortions during preparation, such as bedding errors in plate loading tests and displacement disturbance in vane tests. The larger the scale of the in-situ test the more reliable it will be in representing the non-homogeneous features. Many of the differences associated with obtaining soil parameters by back analysis of observations of full scale behaviour, apply also to the interpretation of the in-situ tests.

1.7. PRESENT RESEARCH

The importance of laboratory testing on undisturbed samples was introduced earlier in this chapter and the problem of sampling disturbance was discussed. The research described in this thesis sets out to examine separately, the effects of the different causes of sampling disturbance on the fundamental behaviour of soils.

The work was theoretical and experimental, but it has been restricted to normally consolidated samples. It is appreciated, that most soils in nature are lightly or heavily overconsolidated but the present work on the normally consolidated soils is an important first step in understanding the fundamental nature of sampling disturbance.

All experiments were conducted on reconsolidated kaolin and virgin samples were prepared at one particular stress state. The theoretical work was based on the ideas of critical state soil mechanics as discussed by Schofield and Wroth (1968) and by Atkinson and Bransby (1978), as this theory is the only one in current use which deals at all adequately with the stress-strain behaviour of soils under all conditions.

CHAPTER 2 : THEORETICAL EXAMINATION OF DISTURBANCE

2.1. INTRODUCTION

In examining sampling disturbance, or any other aspects of the soil behaviour, it is not enough to measure some arbitrarily defined parameters such as the change in pore pressure or the stresses at failure and, in order to understand the behaviour of soils, it is necessary to have a conceptual model. The behaviour of soils may be modelled by using a number of different mathematical theories, although none of these models seem to be able to predict accurately, the complete behaviour of all soils at different stress states. The accuracy of predicting mechanical behaviour will depend on the validity of the assumptions, the complexity of the model and the range of applications. Nevertheless, it is essential that a reasonable model should correctly represent the nature of the mechanical behaviour and its important features.

The concepts of failure, elasticity, and plasticity are introduced in this chapter and used to describe important aspects of sampling disturbance and the mechanical behaviour of soils under different loadings and boundary conditions. It is important to notice that the theories of elasticity and plasticity applied to soils, relate strains to effective stresses and are time independent and consequently they exclude all time dependent assumptions and phenomena.

For the present analysis and for triaxial loading, it is convenient to use stress and strain parameters q', p', ε_s and ε_v (Atkinson and Bransby, 1978). Where

$$q' = \sigma_1' - \sigma_3' = \text{deviatoric stress}$$

$$p' = \frac{\sigma_1' + 2\sigma_3'}{3} = \text{mean effective stress}$$

$$\varepsilon_s = \frac{2}{3}(\varepsilon_1 - \varepsilon_3) = \text{Shear strain}$$

$$\varepsilon_v = \varepsilon_1 + 2\varepsilon_3 = \text{volumetric strain}$$

2.2. ELASTIC BEHAVIOUR

2.2.1. INTRODUCTION

The theory of elasticity relates increments of strains through stiffness moduli and it leads to an incremental mathematical model. If the stiffness moduli are constants, then the material is known as linear elastic and large increments may be used in the mathematical equations, otherwise the variations of the stiffness moduli must be accounted for over the range of increment. Yielding and failure may take place after a certain stress level is reached, and further concepts are required to describe the subsequent behaviour of the material.

The fundamentals of elastic behaviour are reviewed in this section and the stress-strain relationships of isotropic and anisotropic materials are presented.

2.2.2. ELASTICITY

There are various definitions of elasticity and a common factor in all of them may be considered as follows: in an elastic material the mechanical behaviour may be distinguished by a unique relationship between the strain increment vector and the stress increment vector. This relationship may be represented in a matrix form as

$$\{\delta \varepsilon\} = [C] \{\delta\sigma'\} \quad \ldots\ldots\ 2.1.$$

Where $\delta\sigma'$ and $\delta\varepsilon$ are the components of the vectors of the effective stress increment and the strain increment respectively and C is a compliance matrix. Equation 2.1. is valid for positive and negative increments. This implies that all deformations are recoverable and therefore the energy stored during loading is recovered during unloading.

Elastic materials are path independent and the principle of superposition is applicable. Therefore, the total deformation associated with a stress change from point A to point B say in figure 2.1., is neither influenced by the stress history before point A nor by the loading path followed between A and B.

Assuming that the principal axis of stress increment and strain increment are co-axial, equation 2.1. written in full becomes:

$$\begin{Bmatrix} \delta\varepsilon_1 \\ \delta\varepsilon_2 \\ \delta\varepsilon_3 \end{Bmatrix} = \begin{vmatrix} \frac{1}{E'_1} & \frac{-\nu'_{12}}{E'_2} & \frac{-\nu'_{13}}{E'_3} \\ \frac{-\nu'_{21}}{E'_1} & \frac{1}{E'_2} & \frac{-\nu'_{23}}{E'_3} \\ \frac{-\nu'_{31}}{E'_1} & \frac{-\nu'_{32}}{E'_2} & \frac{1}{E'_3} \end{vmatrix} \begin{Bmatrix} \delta\sigma'_1 \\ \delta\sigma'_2 \\ \delta\sigma'_3 \end{Bmatrix} \quad \ldots\ldots 2.2.$$

The compliance matrix contains six independent parameters. The parameter E' is Young's modulus and the parameter ν' is Poission's ratio. A Young's modulus E'_i gives the increment of strain $\delta\varepsilon_i$ due to an increment of stress $\delta\sigma'_i$ and a Poission's ratio ν'_{ij} gives an increment of strain ε_i due to an increment of strain ε_j and it can be shown that $\nu'_{ij} = \nu'_{ji}$.

2.3.2. ISOTROPIC ELASTIC MATERIALS

Isotropy means that the material has the same mechanical behaviour in all directions and this simply means that the parameters used to describe the behaviour of the material for a set of axes in a particular orientation is the same for all other orientations. This assumption will reduce the number of independent parameters in equation 2.2. to two, the Young's modulus E' and the Poission's ratio ν' which fully define the stress-strain relationships. Equation 2.2 may be written for an isotropic elastic material as

$$\begin{Bmatrix} \delta\varepsilon_1 \\ \delta\varepsilon_2 \\ \delta\varepsilon_3 \end{Bmatrix} = \frac{1}{E'} \begin{vmatrix} 1 & -\nu' & -\nu' \\ -\nu' & 1 & -\nu' \\ -\nu' & -\nu' & 1 \end{vmatrix} \begin{Bmatrix} \delta\sigma'_1 \\ \delta\sigma'_2 \\ \delta\sigma'_3 \end{Bmatrix} \quad \ldots\ldots 2.3.$$

If the parameters E' and ν' are variables then the material will have a non-linear stress-strain relationship. The consequence of assuming ν' as a constant and E' a variable leads to mathematical difficulties when $E' \to 0$ (Naylor 1978) which are not discussed any further here.

For triaxial conditions $\sigma'_2 = \sigma'_3$, it is more convenient to use the set of invariants and parameters as described in section 2.1. For a triaxial loading, equation 2.1. may be written.

$$\begin{Bmatrix} \delta\varepsilon_s \\ \delta\varepsilon_v \end{Bmatrix} = \begin{vmatrix} \frac{1}{3G'} & 0 \\ 0 & \frac{1}{K'} \end{vmatrix} \begin{Bmatrix} \delta q' \\ \delta p' \end{Bmatrix} \quad \ldots\ldots 2.4a.$$

Where the moduli K' and G' may be expressed in terms of E' and ν' as

$$K' = \frac{E'}{3(1-\nu')}$$

$$\ldots\ldots 2.4b.$$

$$G' = \frac{E'}{2(1+\nu')}$$

Equation 2.4a., demonstrates the decoupling of the components of the increments of shear stress and shear strain from the components of the increments of mean stress and volumetric strain and consequently for any test on an isotropic elastic material there will be unique relationships between shear stresses and strains and between mean stresses and volumetric strains.

For undrained loading $\delta\varepsilon_v = 0$ and provided $\nu' \neq \frac{1}{2}$;

$$\delta p' = \delta p - \delta u$$

$$\delta u = \delta p$$

which is independent of E' and ν' and will hold for any undrained loading path. For a particular drained loading path of slope $D = {}^{\delta q'}/_{\delta p'}$, the strain path parameter defined as $S = \delta\varepsilon_v/\delta\varepsilon_s$ is given by

$$S = \frac{3(1 - 2\nu')}{2(1 + \nu')} D \quad \ldots\ldots \; 2.5.$$

which is independent of E'. Hence for elastic materials with ν' = constant, linear stress paths are uniquely related to linear strain paths as defined in equation 2.5. and as shown by Atkinson (1973).

2.2.4. ANISOTROPIC ELASTIC MATERIALS

The term anisotropic is used throughout this thesis to mean a material with a single axis of symmetry where the properties of the material in any direction normal to the symmetrical axis are identical but different form these along the axis. In general, 5 independent parameters are required to describe the stress-strain relations of anisotropic materials.

For the special case where the axis of symmetry coincides with the principal axis for anisotropic elastic material, equation 2.1 may be written as

$$\begin{Bmatrix} \delta\varepsilon_1 \\ \delta\varepsilon_2 \\ \delta\varepsilon_3 \end{Bmatrix} = \begin{vmatrix} \frac{1}{E'_1} & \frac{\nu'_{13}}{E'_3} & \frac{\nu'_{13}}{E'_3} \\ \frac{\nu'_{13}}{E'_1} & \frac{1}{E'_3} & \frac{\nu'_{33}}{E'_3} \\ \frac{\nu'_{13}}{E'_1} & \frac{\nu'_{33}}{E'_3} & \frac{1}{E'_3} \end{vmatrix} \begin{Bmatrix} \delta\sigma'_1 \\ \delta\sigma'_2 \\ \delta\sigma'_3 \end{Bmatrix} \quad \ldots\ 2.6.$$

Where the stress-strain behaviour is completely defined in terms of the 4 independent parameters E'_1, E'_3, ν'_{13} and ν'_{33} in figure 2.2.

2.3. PLASTIC BEHAVIOUR

2.3.1. INTRODUCTION

Plastic behaviour occurs when strains are partially or totally irrecoverable and theories of plasticity may be used to define certain features of soil behaviour. Plasticity was introduced into soil mechanics by Drucker and Prager (1952), and since then it has been used in analysing soil behaviour, solving some practical problems and developing mathematical models for soils.

The fundamentals of plastic behaviour are introduced in this section and used later in this chapter to describe some mathematical models and to analyse sampling disturbance.

Materials at failure behave plastically, but they will be considered separately in the next section.

2.3.2. PLASTICITY

Following the same line of argument as that used to define elasticity, plasticity may be distinguished by a unique relationship between the plastic strain increment vector and the current effective stress vector and it may be expressed mathematically as

$$\left\{ \delta \varepsilon^{p} \right\} = \left| G \right| \left\{ \sigma' \right\} \qquad ..... 2.7.$$

Where $\delta\varepsilon^{p}$ and σ' are the components of the plastic strain increment vector and the current effective stress vector respectively. Plastic strains occnr when the current stresses exceed stresses defined by yield criterion. A flow rule and a hardening law describe the proportions and determine the magnitude of the plastic strain increments respectively and they will be considered in more detail in the next sections.

Plastic strains are irrecoveable during unloading and thus total strains may be separated into elastic and plastic components. The energy W_p required to cause plastic defomrations is dissipated during loading while the energy W_e required to cause elastic deformations is stored and may be recovered in unloading as follows

$$W_T = W_E + W_p = \text{total energy}$$

$$W_E = \sigma' \, d\varepsilon^{e} = \text{stored energy}$$

$$W_p = \sigma' \, d\varepsilon^{p} = \text{dissipated energy}$$

For a perfectly plastic material, there are no elastic strains, while for an elasto-plastic material yielding and plastic strains will be associated with simultaneous elastic strains that are governed by the elasticity laws.

2.3.3. THE YIELD SURFACE

Materials yield and undergo plastic deformations when they satisfy a specific combination of stresses which may be presented in a mathematical form as:

$$F(\sigma') = 0 \quad \ldots\ldots\ldots 2.8.$$

The yield equation 2.8 may be thought of as representing a yield surface in a stress space and a particular set of yield surfaces plotted in a three dimensional principal stress space are shown in figure 2.3.

For triaxial loading the stress state of the material may be described in terms of two stress invariants q' and p' and the yield equation may be represented as a curve in a two dimensional plane. A state parameter may be introduced as the third axis and hence, the family of yield curves forms a surface known as the state boundary surface and it will define geometrically the limits of all possible states of the material. A typical state boundary surface for soil will have the general shape shown in figure 2.4. Plastic deformations may be introduced as a state parameter and thus following Zienkiewics and Naylor (1971), the state boundary surface may be expressed mathematically as $f(\sigma', \varepsilon^p) = 0$.

The state boundary surface will define geometrically the limits of the elastic region in a stress space and similarly the yield curve will define geometrically the elastic region in a stress plane for a particular state of the material.

2.3.4. HARDENING LAW

Yielding of materials is associated with plastic deformations and consequently,there will be irrecoverable work done on the material. This may cause a change in the state of the material and thus the material will have a larger or a smaller yield surface. If the yield surface gets larger, strain hardening occurs and if it gets smaller, strain softening takes place.

In work hardening materials, the plastic deformations are associated with expansion of the yield surface and the material is transferred to a new state with a larger yield surface. The relationship between the change of the yield surface and the plastic strain increment is known as hardening law,which may be a function of other factors such as, the stress level of the material. This implies that the relationship between the magnitude of the plastic strain increment and the magnitude of the stress increment as

$$\delta\varepsilon^{p} = \alpha\delta\sigma'$$

where α is a hardening parameter.

2.3.5. FLOW RULE

In analysing plastic behaviour, it is required to define the manner in which the plastic strain increment vector is related to the stress vector. This relationship, which is likely to be a function of the stress history of the material, is known as a flow rule.

It is convenient to define a plastic potential such that the plastic strain increment vectors are orthogonal to the plastic potential as shown in figure 2.5. and for a particular state of the material, the plastic potential may be expressed as a function of stress as:

$$Q\ (\sigma') = 0$$

If the plastic potential function is equal to the yield function, the plastic strain increment is normal to the yield curve and the plastic strain increment vector is related to the stress vector through the yield curve. Therefore, the flow rule is said to be associated, the normality condition applies and the plastic strain increment vector is perpendicular to the yield curve as shown in figure 2.5. If the plastic potential and the yield curve do not coincide, then the flow rule is non-associated and a difficulty arises in defining the relationship between the plastic strain increment vector and the stress vector. The G matrix modulus in equation 2.7. may be fully defined by the hardening law and the flow rule and it is a function of several factors such as the stress increment, the stress level and the stress history.

2.4. FAILURE IN SOILS

2.4.1. INTRODUCTION

The deviatoric stress-strain relationships of soils may show a peak followed by a constant stress, where the soil deforms continuously at its critical state and

$$\delta q' = \delta p' = \delta \varepsilon_v = \delta u = 0 \quad \ldots\ldots 2.9.$$

For very large deformations, there may be a further decrease in the stress as the soil approaches its residual stresses as shown in figure 2.6. The effects of sampling disturbance on the peak and critical strengths are of more concern than the effects on the residual strength and hence the behaviour at the residual will not be considered any further.

Failure of materials occurs when the material continues to deform without further loading on its boundaries. Strain increments at failure are wholly irrecoverable and thus materials at failure are considered to be perfectly plastic. Failure in soils is of a frictional nature and hence shearing resistance is a function of the effective normal pressure. The failure envelope of the material is defined as the combination of stresses at which perfectly plastic yielding occurs. Although the failure envelope is part of the state boundary surface, it is not necessarily of a similar geometry to the yield curve. Figure 2.7. shows the peak failure envelope (A) and the yield curve (B) at a particular specific volume.

2.4.2. THE STRENGTH OF SOILS

Strength may be defined as the maximum shearing resistance of soil at its critical state, defined before in equation 2.9. Strength may be measured directly in a shear box test where the normal and shear stresses acting on the failure plane may be calculated from the external forces, but neither the stress state at failure nor the strains are fully described in the shear box test. Alternatively, the strength of soils may be measured in a triaxial cell, where the axial and radial stresses and strains are assumed to coincide with the principal stresses and the principal strains.

The strength of soil ie due to its frictional resistance and its structural resistance. Various equations have been suggested to describe the strength of soils and the most common is the Mohr-Coloumb equation:

$$\tau = c' + \sigma'_n \tan\phi' \quad \ldots\ldots \quad 2.10.$$

At the critical state the strength of remoulded soil is given by:

$$\tau = \sigma'_n \tan\phi_{cs} \quad \ldots\ldots \quad 2.11.$$

and

$$c' = 0$$

Alternatively, following Von-Mises, the failure criterion may be written in terms of invariants of stress and the deviatoric stress is considered to be a function of the mean effective pressure as:

$$q'_f = M p'_f \quad \ldots\ldots \quad 2.12.$$

The Mohr-Coloumb criterion given by equation 2.10 or 2.11 is written in terms of normal and shear stresses on particular planes of failure and it does not include the intermediate principal stress σ'_2. The extended Von-Mises criterion given by equation 2.12 is written in terms of invariants which are functions of all these principal stresses. Thus the two criterions are not equivalent and the frictional parameter M and ϕ'_{cs} are not uniquely related.

For a heavily overconsolidated remoulded soil, the peak strength may be represented by equation 2.10, or by:

$$q'_p = F + Hp' \quad \ldots\ldots \quad 2.13.$$

Where the parameter H is equivalent in meaning to M in equation 2.12 and the parameter F is equivalent in meaning to the parameter c' in equation 2.10.

In a drained test, the soil softens after its peak value until it reaches its critical state, where it continues to deform at a lower strength, while in an undrained test, the soil will fail at its critical state without showing a well defined peak strength value.

For natural soils, there may be an additional structural resistance developed over very long periods of time during the formation of soil in nature. Its effect on the strength of soil may be introduced as an additional parameter c'_s or F_s in equations 2.12 and 2.13, where it contributes to the peak value in heavily overconsolidated soils and it may cause a peak strength value in normally consolidated soils. It is important to distinguish the differences in nature of the three parameters that are contributing to the strength of soils and their influence on the stress-strain relations. The frictional parameters M and H (or ϕ'_{cs} and ϕ'_p) are considered as soil constants, while the parameters F_p or c'_p are functions of the overconsolidation ratio and the parameters c'_s or F_s depend on the different effects of the physical and chemical histories of soil formation.

2.4.3. DEFORMATIONS AT FAILURE

At the critical state, soils deform as perfectly plastic materials where no further hardening or softening takes place and therefore for a material with an associated flow rule, the strain increments may be related to the yield curve at failure. However, for soils the strength equation that describes the failure envelope is not similar to the yield curve at failure, although they may intersect, since the stress combination at the critical state is bound to fall within the yield surface as shown in figure 2.7. Hence care must be taken to determine whether the strain increment vector should be associated with the failure envelope or with the yield curve.

Heavily overconsolidated soils at and beyond their peak strength will deform as elasto-plastic materials, since there are changes in the effective stresses as further softening or hardening takes place until the soil reaches its critical state. In this case, the peak strength envelope defined by equation 2.13. may in theory be used as a yield curve for heavily overconsolidated soils.

Failure may take place over a well defined thin zone and discontinuous slipping will develop as a sample of soil deforms as two rigid bodies separated by a failure zone. Thus, the measured strains at the boundaries may not represent the strains in the slip plane, as shown in figure 2.8. Alternatively, the strains at failure may be assumed homogeneous along a set of planes, such that, everywhere along these planes the mobilised resistance is equal to the strength of soil.

2.5. THE CRITICAL STATE MODEL

2.5.1. INTRODUCTION

Over the past decade the concept of the critical state introduced into soil mechanics by Roscoe, Schofield and Wroth (1958) has been included in a variety of soil models. The first version was the simple Cam clay model (Roscoe et al 1963) outlined in detail by Schofield and Wroth (1968). Further modifications on the model were suggested by other workers but without altering the fundamental concepts of the model (e.g. Roscoe and Burland 1968). The model was developed specifically for the mechanical behaviour of saturated isotropic remoulded clay and it excludes some important aspects of behaviour such as anisotropy and natural

The concepts of the critical state model are presented in this section and some of the recent developments on the critical state model will be introduced. There will be no detailed discussion of those models but reference will be made for further information.

2.5.2. THE STATE BOUNDARY SURFACE

The concept of the state boundary surface was introduced in section 2.3.3. and the critical state models take the specific volume as the state parameter as shown in figure 2.9. Rendulic (1936) and Hvórslev (1937) were the first to show unique relationships between effective stresses and water contents at failure. Henkel (1959) and others confirmed the previous observations and the void ratio was introduced as a state parameter by Roscoe, Schofield and Wroth (1958).

The important features of the state boundary surface are the Roscoe surface, the Hvorslev surface and the critical state line as discussed by Atkinson and Bransby (1978). The Hvorslev surface and the critical state line were described by the failure envelope equations 2.12. and 2.13 in section 2.4. respectively. The Roscoe surface consists of all the yield curves on the wet side of the critical state shown in figure 2.9. or it may be defined by the compression curves at stress ratios q'/p' varying from 0 to M as shown in figure 2.10. Various equations for the Roscoe surface have been suggested based on different assumptions.

For simple Cam clay, Schofield and Wroth (1968) assumed that the work dissipated in yielding is

$$\delta w = Mp' \; \delta\varepsilon^{p} \quad \ldots\ldots\ldots \quad 2.14.$$

Which leads to the yield equations

$$\frac{q'}{Mp'} + \ln \frac{p'}{p_f'} = 1 \quad \ldots\ldots \quad 2.15.$$

Or

$$\frac{q'}{Mp'} + \ln \frac{p'}{p_o'} = 0 \quad \ldots\ldots \quad 2.16.$$

Where p_f' is the value of p' at the intersection of the yield curve with the projection of the critical state line and p_o' is the value of p' at the intersection of the yield curve with the normal compression line as shown in figure 2.11.

For modified Cam clay, Roscoe and Burland (1968) assumed that the work in yielding is dissipated partially through distortional plastic strains ε^{p} and partically through plastic volumetric strains ε_v^{p} as

$$\delta W = p' \sqrt{(\delta\varepsilon_v^{p})^2 + M^2 \, (\delta\varepsilon^{p})^2} \quad \ldots \quad 2.17.$$

Which leads to the yield equation

$$\frac{p'}{p_o'} = \frac{M^2}{M^2 + (q'/p')^2} \quad \ldots \quad 2.18.$$

Or

$$\frac{p'}{p'_f} = \frac{2M^2}{M^2+(q'/p')^2} \quad \ldots\ldots \quad 2.19.$$

2.5.3. COMPRESSION AND SWELLING

The behaviour of saturated soils under isotropic compression and swelling is shown in figure 2.12 and may be approximated to the linear relationships

$$v = N - \lambda \ln p' \quad \ldots\ldots \text{ for compression} \qquad 2.20a$$

$$v = v_k - k \ln p' \quad \ldots\ldots \text{ for swelling} \qquad 2.20b$$

The parameters λ and k describe the slopes of the straight lines and the parameters N and v_k describe their location. For anisotropic compression and swelling, it is assumed that there will be similar curves with the same slope, but different locations and thus they will have different values for N and v_k. It is worth noting that the value of the parameter N depends on the degree of stress anisotropy and the value of v_k on the previous effective maximum pressure. The one dimensional compression curve is considered to have a constant stress ratio q'/p' and thus, it is the special case of anisotropic compression with no lateral strain while for anisotropic swelling the value of q' varies with the overconsolidation ratio. The critical state line shown in figure 2.10 is considered another special case of an anisotropic compression curve with $q'/p' = M$.

Equations 2.14 to 2.20 may be used to derive the mathematical equations of the state boundary surface in simple and modified Cam clay as described by Schofield and Wroth (1968) and Roscoe and Burland (1968) respectively.

This may be written as

$$v = \Gamma + \lambda - k - \lambda \ln p' - \frac{\lambda - k}{M p'} q' \quad ..... \quad 2.21.$$

for simple Cam clay and

$$v = N - \lambda \ln p' - (\lambda - k) \ln \left(\frac{q}{M p'}\right)^2 + 1 \qquad 2.22$$

for modified Cam clay.

The undrained stress path may be derived from the above equation simply by considering v constant and the critical state equation for both models may be written as

$$q'_f = M \exp\left(\frac{\Gamma - v}{\lambda}\right) \quad .... \quad 2.23.$$

As compression proceeds the soil yields and the total volumetric strain increments may be separated into elastic and plastic components. During swelling and recompression the soil expands and contracts along a line where the volumetric strains are assumed to be elastic and may be calculated from

$$\Delta \varepsilon_v^e = \frac{k \, \Delta p'}{v \, p'} \qquad 2.24$$

The total volumetric strain in an elasto-plastic deformation may be calculated from

$$\Delta \varepsilon_v = \frac{\lambda \, \Delta p'}{v p'} \qquad 2.25$$

and the plastic volumetric strain may be calculated as

$$\Delta \varepsilon_v^p = \Delta \varepsilon_v - \Delta \varepsilon_v^e \qquad 2.26.$$

2.5.4. PREDICTION OF SOIL BEHAVIOUR

The state boundary surface defines all the possible statesof soil and separates these into regions of elastic, elasto-plastic and perfectly plastic behaviour. The behaviour of an element at any state within the state boundary surface is assumed to be isotropic and elastic and the state falls on an elastic wall above the swelling line as shown in figure 2.13b. Elasto-plastic deformations take place when the state moves on the state boundary surface where it hardens on the Roscoe surface and softens on the Hvorslev surface. The behaviour of an element of soil on the critical state line is perfectly plastic. Hence, the behaviour of soil may vary qualitatively according to its initial state and loading conditions as discussed in detail by Atkinson and Bransby (1978).

For an undrained loading, the state is bound to fall on a constant volume plane and for drained conditions a linear loading path is bound to fall on a particular stress plane as shown in figure 2.13 a. The state path follows the intersection of those planes with the elastic wall and the state boundary surface. It is important to notice that the current yield curve and elastic wall is governed by the parameter v_k and not by the overall specific volume. Figure 2.12. shows that the soil may undergo changes in its specific volume, along the swelling line without any change in its yield curve, while figure 2.14 shows that the soil hardens or softens in undrained loading under constant specific volume.

For isotropic elastic behaviour, the stress-strain relationships were introduced in section 2.2.3. and for elasto-plastic behaviour it is assumed that the flow rule is associated andisotropic hardening takes place and hence the stress-strain relationship may be fully defined.

2.5.5. RECENT DEVELOPMENTS

In the simple critical state model,stress history appears only as the overconsolidation state. Over the past few years however, further modifications have been introduced to account for other aspects of stress history. These are mainly concerned with anisotropic consolidation and cycling effects.

Stipho (1978), introduced anisotropy into the critical state model for both its elastic and elasto-plastic behaviour. The yield curve was taken to be path dependent and was allowed to rotate and translate during yielding as shown in figure 2.15. Mroz, Norris and Zienkiewcs (1979) and others have used anisotropic hardening to account for cycling effects. They assumed that the material has a consolidation curve and a yield curve as shown in figure 2.16 and several additional parameters are required.

The effects of sampling disturbance will be examined in view of the simple Cam clay model. Anisotropy will be considered simply by assuming yield curves which are symmetrical about the anistotopic consolidation line, as shown in figure 2.17.

2.6. ANALYSIS OF STRESS DISTURBANCE

2.6.1. INTRODUCTION

The critical state model provides a qualitative and a quantative assessment of soil behaviour under different loading conditions. Hence the effects of stress disturbance and any other alterations in the initial state of the sample can, in theory, be predicted from the model.

2.6.2. THE STATE OF A DISTURBED SAMPLE

The state of a normally consolidated sample may be shown as point A in figure 2.18a. In the critical state model, the behaviour of soil for which the state is inside the state boundary surface, is assumed to be isotropically elastic and hence, there is no change of p' on unloading or reloading. Thus, release of the total stresses under undrained conditions shifts the state to a point such as A1 in figure 2.18awhich is inside the state boundary surface and reloading shifts the state back along the same path to A. Thus the critical state model predicts that there are no effects due to stress disturbance alone on the state and behaviour of a perfect sample reloaded to its previous stress state. Alternatively, if soil is not perfectly elastic for states inside the state boundary surface, the unloading and reloading cycle may alter p', although there is still no volume change. and the state of the sample moves to pointB infigure 2.18b. (Yielding may take place during unloading as shown in figure 2.18c. In these cases, the state shifts to another elastic wall and this may result in hardening or softening and the virgin state cannot be recovered by reloading the sample to its previous stresses.

If it is assumed that there is no volume change during and after the release of total stress,the sample preserves its initial water content and saturation and there is no change of effective stresses. However, if there is a change of suction, due to evaporation or cavitation, or due to inflow of water from the surrounding atmosphere or apparatus, then there is a change in volume and effective stresses. In this case, the state of the sample may shift along its previous elastic wall to the point C in figure 2.18d and the sample can be reconsolidated to its virgin state.

2.6.3. THE MECHANICAL BEHAVIOUR OF A DISTRUBED SAMPLE

The critical state model relates q', p' and v at the critical state by relations defined in section 2.5. From equation 2.23 the undrained shear strength ($C_u = \frac{1}{2} q_f$) depends only on the specific volume and thus if the specific volume remains constant during any stress disturbance, the undrained strength and mean effective pressure at failure of virgin and perfect samples must in theory be the same. This is valid as long as equation 2.21. and 2.23. hold, and the values of the parameters k, λ, Γ and M are not altered by stress disturbance.

The effective stresses and the specific volumes of soil at failur are uniquely related in the critical state model and the relationships are history independent, but the deviatoric and volumetric stiffnesses are functions of the stress level, stress state and loading path. Hence disturbed samples must be reloaded to their previous stress state and stress level before testing, in order to eliminate the effects of different initial stress state.

The overconsolidation ratio may be defined as p'_p/p' as shown in figure 2.19 where p'_p is the maximum effective pressure experienced by the sample and p' is the current effective pressure. This may be written as

$$O.C.R. = \frac{\exp\left(\frac{N - v_k}{\lambda - k}\right)}{p'} \qquad 2.27.$$

where

$$v_k = v + k \ln p'$$

As argued in section 2.5.4., changes in the effective stress p' under constant specific volume may result in hardening or softening and samples will have different values of v_k and O.C.R. as shown in figures 2.18 b and 2.18c. In this case, a disturbed sample originally normally consolidated and reloaded to its previous total stress state will behave as an overconsolidated sample.

2.6.4. ANALYSIS OF MECHANICAL DISTORTIONS

The term mechanical distortions was previously used to describe all the disturbances other than stress disturbance. The major cause of mechanical distortions is driving a tube sampler into the soil which results in non-uniform distribution of stresses and displacement of soil by the sampler wall. Such distortions may lead to local remoulding, swelling, cavitation and sometimes, consolidation.

The indentation of a rigid punch into an elasto-plastic material will lead to a number of effects in various elements; some elements are stressed without exceeding the yield stresses, some elements yield and others fail but without overall failure of the punch. The process of driving a tube into natural clays is more complicated due to the shape of the sampler and the complex nature of the deformations that occur in the penetration zone. Figure 2.20 shows a photoelastic analysis of the stress patterns developed during tube sampling and it indicates high radial stresses near the walls of the sampler and bent layers of clays are reported in tube samples (e.g. Hvorslev 1949 and McManis and Arman 1979). Ito and Tanaka (1969) used stereoscopic photographs to assess the zone of disturbance in tube samples and they observed that a cutting edge of 7^{o} caused the least disturbance and at that angle the area of disturbed zones increased proportionally with the increase of the wall thikcness. Further evidence on the variation of the mechanical properties within the tube samples is presented in chapter 3.

A tube sample is subjected to further disturbances such as those caused by forces necessary to extrude the sample from the tube. In addition, a reduction in suction or increase in water content could result from exposing the sample to the air or surplus water.

2.7. NORMALISING PROCEDURES

2.7.1. INTRODUCTION

Some of the differences between the behaviour of samples are caused solely by differences in their initial states and there is the possibility that these different behaviours may be equated by applying suitable normalising procedures to results of laboratory tests. For example, previous observations (Parry 1960, Henkel 1960 and Balasubramaniam et al 1977) showed that samples with the same over-consolidation ratio but of different consolidation stresses exhibit very similar stress-strain relationships when normalised with respect to the consolidation stress. However, normalisation can only be effective if the fundamental behaviour of a virgin soil and that of a disturbed sample is the same (i.e. both are either elastic or elasto-plastic). Clearly it will not be possible to normalise the behaviour of an elastic overconsolidated soil to be the same as that of an elasto-plastic normally consolidated soil.

It is well known that the soil behaviour is controlled by the current effective stresses but it is also controlled by the specific volume.

Thus the behaviour of two samples can only be the same if both their effective stresses and their specific volumes are the same and the state of a sample must be described by both the effective stress and the specific volume. Sampling disturbance alters the effective stress or the specific volume or both and thus in geueral, sampling alters the state of the soil. Furthermore, since the state of samples change during testing,it is not enough simply to normalise with respect to their initial states.

In this section, some of the procedures available to distinguish elastic from elasto-plastic behaviour are introduced and normalising parameters and procedures based on the critical state model are discussed.

2.7.2. FUNDAMENTAL BEHAVIOUR OF SOILS

In order to normalise the behaviour of soils,it is necessary to distinguish between elastic and elasto-plastic behaviour. Although, soils do not behave as ideal elastic, elasto-plastic or plastic materials, their behaviour is often well approximated by these concepts.

In the critical state theory, the behaviour of a lightly overconsolidated soil is represented by the stress and strain paths and stress-strain curves shown in figure 2.21a.. These show a distinctive change in the pattern of behaviour as soil changes from elastic to elasto-plastic. In experimental data this change is often less sharply defined and it may be difficult to distinguish precisely between elastic and plastic behaviour.

Alternatively, the fundamental behaviour of soil may be examined by observing the relationships between stress ratios and strain ratios. For an isotropic elastic soil, increments of stress and increments of strain are related by

$$\delta\varepsilon_s = \frac{2(1+\nu')}{3E'} \delta q' \quad \ldots\ldots 2.28.$$

$$\delta\varepsilon_v = \frac{3(1-2\nu')}{E'} \delta p' \quad \ldots\ldots 2.29.$$

Thus

$$\frac{\delta\varepsilon_v}{\delta\varepsilon_s} = \frac{3(1-2\nu')}{2(1+2\nu')} \frac{1}{\delta q'/\delta p'} \quad \ldots\ldots 2.30.$$

and for a drained test with a linear loading path for which $\delta q'/\delta p'$ is a constant and assuming ν' is a constant, then $\delta\varepsilon_v/\delta\varepsilon_s$ is a constant also and the strain path obtained by plotting ε_v against ε_s is linear and this holds for non-linear elastic soils.

For anistoropic elastic soils in triaxial loading tests, increments of stresses and increments of strains may be related by

$$\delta\varepsilon_1 = \frac{\delta\sigma_1'}{E_1'} - 2\frac{\delta\sigma_3'}{E_3'}\nu_{13}' \quad \ldots\ldots 2.31$$

$$\delta\varepsilon_3 = \frac{\delta\sigma_1'}{E_1'}\nu_{31} - \frac{\delta\sigma_3'}{E_3'}\nu_{33}' + \frac{\delta\sigma_3'}{E_3'} \quad \ldots 2.32.$$

Where E_1', E_3', ν_{13}' and ν_{33}' were previously introduced in section 2.2.

Thus

$$\frac{\delta\varepsilon_v}{\delta\varepsilon_s} = \frac{\frac{3}{2}(\delta\varepsilon_1) + 3(\delta\varepsilon_3)}{\delta\varepsilon_1 - \delta\varepsilon_3}$$

$$\frac{\delta\varepsilon_v}{\delta\varepsilon_s} = \frac{\frac{3}{2}\left(\frac{\delta\sigma'_1}{E_1} - 2\frac{\delta\sigma'_3}{E'_3}\nu_{13}\right) + 3\left(-\frac{\delta\sigma'_1}{E'_1}\nu'_{31} - \frac{\delta\sigma'_3}{E'_3}\nu'_{33} + \frac{\delta\sigma'_3}{E'_3}\right)}{\left(\frac{\delta\sigma'_1}{E'_1} - \frac{\delta\sigma'_3}{E'_3}\nu_{13}\right) - \left(\frac{\delta\sigma'_1}{E_1}\nu_{31} - \frac{\delta\sigma'_3}{E'_3}\nu_{33} + \frac{\delta\sigma'_3}{E'_3}\right)}$$

Dividing through by $\delta\sigma'_1/E'_1$ we have

$$\frac{\delta\varepsilon_v}{\delta\varepsilon_s} = \frac{\frac{3}{2}\left(1 - \frac{X}{Y}\nu'_{13}\right) + 3\left(-\nu'_{31} - \frac{X}{Y}\nu'_{33} + \frac{X}{Y}\right)}{\left(1 - \frac{X}{Y}\nu'_{13}\right) - \left(-\nu'_{31} - \frac{X}{Y}\nu'_{33} + \frac{X}{Y}\right)} \quad \ldots\ldots 2.33.$$

Where

$$X = \frac{\delta\sigma'_3}{\delta\sigma'_1}$$

and

$$Y = \frac{E'_3}{E'_1}$$

Thus, for a drained test with a linear loading path for which $\delta\sigma'_3/\delta\sigma'_1$ is constant and by assuming that the modulus ratio E'_3/E'_1 and the Poission's ratios are constants, $\delta\varepsilon_v/\delta\varepsilon_s$ is a constant also and the strain path obtained by plotting ε_v against ε_s is linear. This holds for non-linear elastic soils provided only that the non-linearity is such that the modulus ratio E'_3/E'_1 remains constant.

It may be shown, following Atkinson (1973a) that for an elasto-plastic material the strain path in a constant $\delta q'/\delta p'$, is non-linear. Thus plotting ε_v against ε_s produces a convenient way of distinguishing elastic from elasto-plastic behaviour.

For undrained tests for which $\varepsilon_v = 0$, from equation 2.29, $\delta p' = 0$ and the effective stress path plotted with axis q' vs p' is linear and vertical. It may be shown following Atkinson (1973a) that for an elasto-plastic material, the effective stress path in an undrained test is non-linear. Thus, effective stress paths in undrained tests provide a second way of distinguishing elastic from elasto-plastic behaviour. Alternatively, the changes of pore pressure during undrained loading of elastic and elasto-plastic soils may be considered. Following Lo (1969) it is convenient to define a normalised pore pressure parameter α as $\delta u/p'$ where p' is the current value of the effective stress and α may be related to the shear strain in an undrained test.

From the principle of effective stress, and for undrained loading of isotropic elastic soil, we have

$$p' = p - u$$

and hence for $\delta p' = 0$

$$\delta p = \delta u$$

$$\frac{\delta p}{p'} = \frac{\delta u}{p'} = \alpha \qquad \ldots\ldots\ 2.34.$$

But for linear loading paths we have

$$\delta p = X_1 \, \delta q$$

$$\frac{\delta p}{p'} = X_1 \frac{\delta q}{p'} \qquad \ldots\ldots \; 2.35.$$

where X_1 is a constant. Thus

$$\alpha = X_1 \frac{\delta q}{p'} \qquad \ldots\ldots \; 2.36.$$

It may be shown (Atkinson and Bransby 1978) that

$$\delta \varepsilon_s = \delta q' \quad \frac{2K \; (1 + \nu')}{9p'v \; (1 - 2\nu')} \qquad \ldots\ldots 2.37.$$

From equation 2.37,

$$v.\delta\varepsilon_s = \frac{\delta q'}{p'} \left[\frac{2K \; (1 + \nu')}{9 \; (1 - 2\nu')} \right] \qquad \ldots\ldots 2.38.$$

Which may be written as

$$\frac{\delta q'}{p'} = \bar{v}.\delta \varepsilon_s . \; Y_1 . \qquad \ldots\ldots 2.39.$$

Where Y_1 is a constant and v is the specific volume which is held constant in an undrained test.

From equation 2.36. and 2.39. we have

$$\alpha = \left\{ X_1 \;\; Y_1 \;\; v \right\} . \delta \varepsilon_s \qquad \ldots\ldots 2.40.$$

and hence

$$\frac{\delta \varepsilon_s}{\alpha} = \text{constant} \qquad \ldots\ldots 2.41.$$

For any elastic soil whether it is anisotropic or non-linear, the effective stress path in an undrained test is given by the intersection of a constant v plane and an elastic wall and this intersection has a linear projection onto the q' : p' plane. Thus, following Henkel and Wade (1965) changes of pore pressures are related to changes of total stresses by an equation of the form

$$\delta u = \delta p + \alpha_1 \delta q \quad .... \quad 2.42.$$

where the parameter α_1 is a constant. For linear loading paths we have

$$\frac{\delta q}{\delta p} = X_2$$

and hence

$$\delta u = (1 + \alpha_1 X_2)\, \delta p$$

$$\frac{\delta u}{p'} = Z \frac{\delta p}{p'}$$

Where Z is a constant. Hence, by following similar substitutions in equations 2.34. to 2.41. it can be shown that the ratio $\delta\varepsilon_s/\alpha$ is a constant for anisotropic elastic soils in undrained tests. Therefore, it may be concluded that for any linear loading path in an undrained test on elastic soils the relationship between q'/p' and $\delta\varepsilon_s/\alpha$ will be a vertical line as shown in figure 2.21h

2.7.3. NORMALISATION OF STATE PARAMETERS

The state boundary surface in figure 2.9. defines all the possible states of a soil. The intersection of the state boundary surface,with an elastic wall defines a yield curve, while its intersection with a constant volume plane defines the stress path in an undrained test.

Figure 2.22. shows yield curves and undrained stress paths at different specific volumes in a two dimensional stress plane. If the yield curves are geometrically the same, a unique yield curve may be obtained in a non-dimensional plane by normalising the stresses with suitable stress parameters. Normalising parameters should be chosen either such that all states at the same specific volume have the same normalising stress or such that all states on the same yield curve have the same normalising stress.

Some suitable normalising parameters are shown in figure 2.23. The parameter p'_e is associated with constant specific volumes and it may be calculated as

$$p'_e = \exp \frac{N - v}{\lambda} \qquad \text{........ 2.43.}$$

Hence normalising stresses on the state boundary surface with respect to p'_e will lead to a dimensionless geometrical shape of the stress path in undrained tests,but it does not lead to a dimensionless yield curve.

Alternatively, stresses of the state boundary surface may be normalised with respect to stresses that are associated with its yield curve, such as the stresses at the intersection of elastic walls with the critical state line (point p'_{fp} in figure 2.23) or the intersection of elastic walls with the normal consolidation line (point p'_p in figure 2.23). If the shape of the state boundary surface is independent of its size, normalising the stresses with respect to either p'_{fp} or p'_p will give a single dimensionless yield curve and both the critical state line and the normal consolidation line appear as single points in figure 2.24.

The parameters p'_{fp} and p'_p may be calculated as

$$p'_{fp} = \frac{\Gamma - v}{\lambda - k} \qquad \text{......... 2.44a.}$$

$$p'_p = \frac{N - v}{\lambda - k} \qquad \text{......... 2.44b.}$$

There are difficulties in measuring both values of Γ and N from tests on soils in general and these difficulties are more pronounced in tests on heavily overconsolidated soils. The position of the consolidation curve in a v-lnp' diagram seems to be time dependent and the position of the critical state line is calculated from the overall measurement of the water content of the sample, rather than the measurement of the water content of the failure zone of the sample.

Nevertheless, it may be assumed that the critical state line is time independent and hence, it is relevant to use the parameter p'_{fp} in normalisation, since it is associated with the critical state line.

The yield curves of simple Cam clay and modified Cam clay in Equations 2.15. and 2.19. respectively may be re-written as

$$q' + Mp' \ln \frac{p'}{p'_{fp}} - Mp' = 0 \quad \text{Simple Cam clay}$$

$$\frac{q'}{p'_{fp}} + M^2 \frac{p'^2}{p'_{fp}} - 2\,M^2\,p' = 0 \quad \text{modified Cam clay}$$

Dividing through by p'_{fp} we have

$$\left[\frac{q'}{p'_{fp}}\right] + M\left[\frac{p'}{p'_{fp}}\right] \ln \left[\frac{p'}{p'_{fp}}\right] - M\left[\frac{p'}{p'_{fp}}\right] = 0$$

$$\left[\frac{q'}{p'_{fp}}\right]^2 + M^2\left[\frac{p'}{p'_{fp}}\right]^2 - 2\,M^2\left[\frac{p'}{p'_{fp}}\right] = 0$$

Which represent normalised yield curves of simple Cam clay and modified Cam clay respsctively.

2.7.4. NORMALISATION OF STRESS-STRAIN RELATIONSHIPS

As argued in section 2.6., the behaviour of soils may be examined within the context of elasticity and plasticity and the stresses and strains measured in a triaxial test may be used to show stress-strain relationships within that context. The etrain invariants ε_s and ε_v are commonly used in soils and they are associated with the stress invariants q' and p' (Atkinson and Bransby 1978).

In general, for axially symmetric loading of an isotropic material stress-strain relationships may be written as

$$\Delta\varepsilon_s = A\ \Delta q' + B\ \Delta p' \qquad \ldots\ldots\ 2.45.$$

$$\Delta\varepsilon_s = D\ \Delta q' + C\ \Delta p' \qquad \ldots\ldots\ 2.46.$$

And as shown by Naylor (1978), if the flow rule is associated then B = D and the stress-strain matrix is symmetric. For isotropic elastic material the parameter B in equation 2.45. is zero and hence the stress-strain relationships are decoupled.

In deriving the simple Cam clay theory, Schofield and Wroth (1968) assumed that the distortional strain was perfectly plastic but it may also be separated into elastic and plastic components (Roscoe and Poorsooshasb 1963). The elastic volumetric strain may be obtained from equation 2.24. It is difficult, however, to obtain an accurate value for κ which may fall into the range of 0.03 to 0.05 for kaolin.

Furthermore, kaolin often shows appreciable changes in the epecific volume due to creep.

For isotropic elastic soil, following Atkinson and Bransby (1978), elastic shear etrains and elastic volumetric strains are given by

$$\delta\varepsilon_s = \frac{2K(1+\nu')}{9vp'(1-2\nu')} \cdot \delta q' \quad \ldots\ldots\ldots \quad 2.47.$$

$$\delta\varepsilon_v = \frac{K\,\delta p'}{vp'} \quad \ldots\ldots\ldots \quad 2.48.$$

$$v\,\delta\varepsilon_s = \left[\frac{2K(1+\nu')}{9(1-2\nu')}\right] \frac{\delta q'}{p'} \quad \ldots\ldots\ldots \quad 2.49$$

$$v\,\delta\varepsilon_v = \left[k\right] \frac{\delta p'}{p'} \quad \ldots\ldots\ldots \quad 2.50.$$

The expressions in square brackets may be assumed constants and hence, for elastic soil the relationship between $v.\ \delta\varepsilon_s$ and $\delta q'/p'$ and between $v.\ \delta\varepsilon_v$ and $\delta p'/p'$ will be constants for any loading path or stress level.

For simple Cam clay the state boundary surface is given by Schofield and Wroth (1968) as

$$\nu = \Gamma + \lambda - k - \lambda \ln p' - \frac{\lambda - k}{Mp'} q' \quad \ldots\ldots \quad 2.51.$$

This equation was obtained for purely theortical considerations and in the analysis it was assumed that $\delta\varepsilon_s^e = 0$. In equation 2.45. however, $\delta\varepsilon_s{}^e$ is not zero but is given by equation 2.47. and the simple Cam clay state boundary surface is not strictly applicable in evaluating the parameters A and C in equations 2.45. and 2.46. However, the state boundary surface given by Schofield and Wroth (1968) is found to be a reasonable approximation of the state boundary surface observed from the results of experiments and for the present purpose of illustrating normalising procedures we will take equation 2.51. as a state boundary surface. Thus equations 2.45 and 2.46 become

$$\delta\varepsilon_s = \frac{\lambda-K}{Mvp'}\left[\frac{\delta q'}{M-q'/p'} + \delta p'\right]$$

$$\delta\varepsilon_v = \frac{-\lambda-K}{Mvp'}\,\delta q' + \frac{\lambda-K}{Mvp'}\,\frac{q'}{p'}\,\delta p' - \lambda\,\frac{\delta p'}{vp'}$$

and these may be written as

$$v.\delta\varepsilon_s = \left[\frac{\lambda-K}{M}\;\frac{1}{M-q'/p'}\right]\frac{\delta q'}{p'} + \left[\frac{\lambda-k}{M}\right]\frac{\delta p'}{p'} \quad \text{... 2.52.}$$

$$v.\delta\varepsilon_v = \left[\frac{\lambda-k}{M}\right]\frac{\delta q'}{p'} - \left[\frac{\lambda-k}{M}\;\frac{q'}{p'}\right]\frac{\delta p'}{p'} \quad \text{. 2.53.}$$

It may be observed that the parameters within the brackets are all constants except for the ratio q'/p' and hence it is possible to obtain stress-strain relationships that are independent of the normal pressure p'. It is interesting to observe that by substituting $\lambda = K$ in equations 2.52 and 2.53, $\delta\varepsilon_s$ becomes 0 and equation 2.53 becomes that of elastic soils.

CHAPTER 3 PREVIOUS WORK ON SAMPLING DISTURBANCE

3.1. INTRODUCTION

A number of different procedures are adopted for measuring, analysing and correcting the effects of sampling disturbance and, in order to outline the approach of the present research, it is necessary to review previous investigations on the problem of sample disturbance.

Since the beginning of this century there has been a wide range of reported observations on the effects of sampling procedures on different types of soils. Some direct investigations have considered the effects of the main casues of disturbance on the strength and deformation properties while other indirect investigations were concerned more about the development of samplers and sampling methods. Although it is difficult to summarise all the published observations, the important approaches to the problem are reviewed in this chapter and the effects of sampling disturbance on the mechanical properties summarised. For further reviews of the literature on sample disturbance, the reader is referred to Hvorslev (1949), Kallstenius (1963) Arman et al (1977) and Broms (1980).

3.2. COMPARATIVE STUDIES ON DISTURBED SAMPLES

3.2.1. INTRODUCTION

Disturbance may be examined with reference to the undisturbed behaviour of soils and, since it is impossible to obtain an undisturbed sample, engineers seek the behaviour of the least disturbed sample. Comparisons between the behaviour of disturbed samples to that of samples with least disturbance may help in assessing the relative disturbance imposed by one technique, compared to that of another. Such comparisons include, differences between in-situ and laboratory testing, block and tube samples and different sizes of tube samples. In this section, some of the published data of this approach are reviewed. The results are usually presented simply as the ratio of some measured parameters for different samples and the data presented in this section will be used later in this chapter, in drawing general conclusions on the effects of sampling disturbances.

3.2.2. COMPARISONS BETWEEN BLOCK AND TUBE SAMPLES

The opportunity of obtaining samples using block sampling and tube sampling has provided valuable information for comparative purposes (e.g. Ward, Samuels and Butler 1959 and Conlon and Isaacs 1971), since there are qualitative differences between them as described before.

The behaviour of tube samples obtained from different samplers may be used to assess the significance of different features in the design of samplers, such as the clearance ratio, the wall thickness and the sampling technique (e.g. Jakobson 1954, Kallstenius 1958, 1963 and Milovic, 1971). An example is shown in figure 3.1., to demonstrate the qualitative differences in stress-strain relationships of block and tube samples and the qualitative similarities between different tube samples.

Some effects of tube sampling are shown in table 3.1. It is necessary to consider the soil type, the sampler type, the method of sampling and the tests performed, since measured parameters are often influenced by such factors. The results indicate that block samples have higher strengths and stiffnesses parameters than tube samples and hence, block samples are considered of a better quality.

3.2.3. DIFFERENT SIZE TUBE SAMPLES

Differences in the behaviour of tube samples of different sizes are partially due to different degrees of disturbance and partially due to differences in representing the fabric of soils. Rowe (1971), reported that the size of samples must represent the fabric of the in-situ soil and Mackinley et al (1975) examined the effects of fissures and stresses on selecting the size of the sample.

It may be argued, that even if the in-situ soil is homogeneous, it will have a non-homogeneous state within the tube sample, due to disturbance and hence, the behaviour of the tube sample in the laboratory represents an overall behaviour of all its disturbed regions. The larger the percentage of the disturbed regions in the sample, the more the overall behaviour is diverted from its in-situ behaviour. Hence, it may be assumed that the larger the diameter of the sample and the lower its wall thickness to diameter ratio and the less will be the influence of the disturbed layers on the overall behaviour.

Disturbed zones in tube samples will have less influence on the behaviour of large tube samples than that of small tube samples of the same wall thickness.

Berre et al (1969) observed that larger tube samples showed more consistent behaviour than that of small tube samples. McManis and Arman (1979) reported that 125mm tube samples had parameters close to those from block samples, while parameters measured from 76 mm tube samples were closer to those of remoulded samples and similar observations were reported by Brenner (1979) and others.

Bozozuk (1971) reported undrained triaxial tests on 1½" samples cut from 2" and 5" tube samples and one-dimensional consolidation tests on 2" and 5" samples. These tests showed that the undrained strengths of samples cut from 5" tube samples were higher than those cut from 2" tube samples and the small sized tube samples showed lower stiffness and pore pressure response.

In a series of tests on tube samples, varying in size from 1" to 5", Conlon and Issacs (1971) observed that disturbance increased as the size of the tube sample decreased.

However, it should be noted that in stiff fissured clays, the strength of the mass will probably be less than the strength of a small sample due to the presence of fissures (e.g. Agarwal 1967 and Marsland 1973b). In this case, it is not easy to describe disturbance by comparing the strengths of different sized tube samples.

3.2.4. IN-SITU BEHAVIOUR AND DISTURBED SAMPLES

As discussed in section 1.6, observations of full scale behaviour and in-situ soil testing cannot wholly replace laboratory testing, but they may be used for reference. It is important to notice that these comparisons are largely empirical and they are influenced by many factors such as differences in sizes of soil samples, uncertainties regarding the boundary conditions and different assumptions in analysing the behaviour.

The undrained strengths of disturbed samples of a variety of soils in undrained triaxial tests are compared to the undrained strengths obtained from in-situ vane tests in table 3.2. and the stiffnesses, calculated as the tangent modulus of stress-strain curves in triaxial tests are compared to those obtained from the load-deformation curves in plate loading tests as shown in table 3.3.

The results vary in different soils and in general it may be observed that parameters from tests on block samples are closer to parameters from in-situ tests and that the stiffness parameters are more altered by disturbance than the undrained strength. In-situ undrained tests gave undrained strengthsthat are higher than those from disturbed samples in triaxial test.

Large scale in-situ shear tests were reported by Karlsrud (1979) and the undrained strengths and stress-strain relationships were compared to that of triaxial tests. In-situ stengths were 20% higher than those measured in the laboratory and the differences were attributed to different boundary conditions.

Burland and Lord (1970) reported good agreements among stiffness parameters obtained from full scale tank tests, in-situ plate loading tests and laboratory tests on block samples of Chalk. All tests showed initial linear load-deformation or stress-strain relationships and the values of Young's modulus approximated from these tests, varied with different grades of Chalk, showing values of E_u in the laboratory which are 20-30% higher than the values in-situ.

Burland and Hancock (1977) predicted the movement for a retaining wall during an excavation of stiff London clay and their predictions were based on stiffness parameters that were derived from previous full scale measurements in London clay. Their predictions showed reasonable agreements with horizontal deformations of the retaining wall during excavation.

The undrained Young's modulus used in the analysis was 5 times that measured in the laboratory and hence, the use of values of Young's modulus calculated from laboratory tests would have led to a gross overestimation of the deformations.

Pennman et al (1971) reported good agreements between the predicted and observed behaviour of an earth dam. Deformations were predicted by a simple linear finite element method and the analysis was based on parameters obtained from tube samples. The uniformity and homogenity of the fill material, the selected equivalent values E' and v' and the state of soils were important factors in the accurate predictions.

Drainage characteristics of in-situ soils were compared to those measured in the laboratory by Bishop and Al-Dhair (1969) and it was found that the values of permeability k and the coefficient of consolidation C_v are underestimated in the laboratory due mainly to inadequate representation of the sample and the non-uniformity of soils as discussed by Rowe (1972).

3.3. STRUCTURAL DISTURBANCE

3.3.1. INTRODUCTION

It is appreciated that fabric and macrofeatures play a significant role in the behaviour of soil such as the contribution of cemented bonds to strength and stiffness (Bishop, 1971) and the influence of particle orientation on stress-strain relationships (e.g. Barden, 1972).

The formation of soil structures in field samples has been discussed by several authorities (e.g. Mitchell, 1976) and it appears that once the natural structure is destroyed, it is impossible to reproduce it in the laboratory. Sampling disturbance may distort some macrostructural features such as mixing layers,or it may produce features that were not present before, such as fissures.

A detailed discussion of previous work on the effects of the fabric on the behaviour of soils is beyond the scope of this thesis but some observations and measurements on disturbed fabrics are presented in this section. Investigations on soils with disturbed structures are not expected to lead to quantative assessments of disturbance or its effects on behaviour, but they are useful in interpreting some of the effects of disturbance and in estimating the variation of disturbance within a sample.

3.3.2. DISTURBANCE OF MACROFEATURES

Knowledge of stratigraphy, joints and fissure patterns is important in shear strength studies and slope stability problems. Permeability and compressibility are influenced by the sizes, spaces and orientation of fissures and other macrofeatures (Rowe, 1972). Such macrofeatures are observed in the ground and in disturbed samples and the technique commonly adopted is to cut the sample and to leave it to dry partially when visual or photographic observations are used to determine the macrostructural nature of the soil.

The use of radiography as a method for determining the nature and extent of distortion on clays, has been demonstrated by Krinitzsky (1970) and used by Arman and McManis (1977) in their investigation on fabric disturbances.

The frequency and nature of joints and bedding planes are ill represented in disturbed samples because of inevitable distortions. A sampling process may cause local pore water flow, local volume changes and these may lead to fissures and weak regions. Fissures have been reported by Martin and Ladd (1975) to open visibly even when care is taken in block sampling and it is difficult to separate natural from inducedfissures or to evaluate the existence of invisible fissures.

Differences between in-situ drainage properties and those measured in the laboratory are due to non-representative samples. All samples are relatively small in comparison to in-situ soils and hence, they do not necessarily represent all fabric of soils in the field.

Fabric studies have indicated that there is an annular zone of disturbance around the outer portion of cylindrical specimen (Ito and Tanaka 1969 and Barden 1971). The effects of drilling and tubing processes on layered soils was reported by McManis and Arman (1979) to produce bent layers with maximum bending near the tube walls and this bending decreases towards the centre.

Kenney and Chan (1972) reported that open tube samples suffer severe cracks during sampling and that extrution may smear the surface material and reduce the extent of those cracks in visual inspection.

3.3.3. MICRO-STRUCTURAL INVESTIGATIONS

The electron microscope is a common method of investigating microfabric of soils and in this method, samples require treatement which inevitably disturbs the sample. Preparation techniques involve the removal, conversion or replacement of the soil water and may involve processes such as cutting or etching. Therefore, such observations are entirely dependent on the surface layers of the soil sample which are most subject to disturbance during preparation.

Kirkpatrick and Rennie (1973), reported that the effects of anisotropic consolidation on soil fabric was to create marked horizontal orientations and remoulding severely distorts soil fabric . Barden (1971) observed that changes in the stress state of soils may lead to the development of preferred orientation of particles. Barden and McGown (1971) investigated the fabric of specimens from the edge, centre, mid-height and top of the tube sample and their results revealed that compaction and re-orientation of particles may vary depending on factors such as the soil type, the sampler type and the size of the sampler. Martin and Ladd (1975) reported that extrusion of samples would lead to fabric disturbance all over the sample.

McGown et al (1974) examinedthe fabric of block samples and 100 mm,150 mm and 250 mm tube samples and the results showed that all elements in 100 mm tube samples were subjectedto strains during sampling and distortions were less in large sized tubes.

3.3.4. VARIATION OF DISTURBANCE WITHIN A TUBE SAMPLE

As argued in section 2.6.4. elements of soil at different locations within a tube sample suffer different degrees of mechanical distortions and further evidence is presented in this section to confirm these patterns of disturbances.

Bromham (1971) and others investigated the variation of water content, bulk density and shear strength from top to bottom of tube samples and the results showed that the minimum disturbance occurred near the centre of the sample and the maximum disturbances were near the top as shown in figure 3.2. Investigations on the fabric of soils (Barden 1971) showed that the outer zone of the tube sample had particles re-orientated parallel to the tube wall and the structure of the inner layers appeared to be compacted, but the particles were not oriented. Apparently the soil in the centre of the sample is subjected to the least amount of deformations while the soil near the wall is subjected to higher stresses and strains. Furthermore, if swelling was to take place, then there will be a flow of water from external layers to the core of the sample and this causes a drop in the overall negative pore pressure.

3.3.5. SOME EFFECTS OF TRIMMING, FRICTION AND REMOULDING

Soil samples are usually subjected to mechanical distortions that may be as little as trimming in block sampling or as severe as complete remoulding.

Remoulding is expected to destroy the original structure of the sample, but not to alter the properties of the material, such as the angle of friction and the Atterberg limits. Soil samples that are remoulded at their initial water content may be used as a reference to assess the effects of remoulding.

Remoulding is considered as the most serious cause of disturbance (e.g. Terzaghi 1941 and Rutledge 1944) and the effects of remoulding on the mechanical behaviour of soils were reviewed by several workers (e.g. Bishop 1971).

Table 3.4. summarizes some of the observations on the effects of remoulding and it can be seen that the stiffness parameters are more influenced than the strength parameters and the effects of remoulding varies for different clays. Figure 3.3 and 3.4 show examples of the effects of remoulding on the stress path and the stress-strain relationships respectively. Alternatively, trimming was reported to have negligible influence on the behaviour (Skempton and Sowa, 1963) of block samples and the effects of trimming were higher in small samples, (Bozozuk 1971).

Holtz (1963) observed that the effects of the remoulded material on the sides of the tube sample may be removed simply by trimming.

Ball, (1962) and Kallstenius (1963 and 1971) considered the external and internal friction on the tube sampler as a major source of disturbance.

Kenney and Chen (1972) observed that frictional forces during tube sampling may cause cracks but extruding tube samples tends to mask the cracks by smearing them.

3.3.6. STORAGE EFFECTS

Samples are usually sealed and stored for some period of time before testing and this delay may cause further alterations to the clay structures as suggested in section 1.3.4. In a partially sealed sample the suctions created by the release of the field total stresses will start to equalise immediately after sampling. Water content may vary over the sample due to local drainage and other time dependent phenomenon may take place at high rates at the beginning of the storage period (Kallstenius 1963).

Bozozuk (1971) observed that a storage period of 15 months did not have much effect on the estimated pre-consolidation pressure from laboratory tests. Anderson and McKinley (1975) observed that a loss of water content by 2% over a storage period of 2 months and on average, stored sample showed an increase in stiffness by 60% and an increase in the undrained strength by 18%.

It is important to notice, that the stresses imposed on the sample during storing has a direct effect on thixotropy or relaxation of the sample (Goodman and Leininger, 1967) and that cohesive forces may develop between the sampler and the clay during storage (Kallstenius 1963).

3.4. DISTURBANCE DUE TO STRESS CYCLING

3.4.1. INTRODUCTION

Since sampling involves a cycle of stresses, it is appropriate to consider cyclic loading tests in this context. Cyclic loading studies usually fall into two categories. The first category is related to pavement design problems, off-shore structure studies and earthquake problems, where soils are subjected to stress cycles of various amplitude and frequencies. The second category includes static unloading and reloading cycles, in order to separate the recoverable from irrecoverable components of deformations or to simulate the stress disturbance in a sampling process.

A static unloading cycle can be used to examine the effects of stress disturbance and to simulate a perfect sample as described in section 1.4. However, samples disturbed by stress cycling do not necessarily represent the effects of mechanical distortions or simulate a tube sample at higher degrees of disturbance. This section reviews the effects of cylcic loading, duration of cycles and the significance of the first cycle on the behaviour of clays.

3.4.2. HYPOTHESIS ON CYCLING EFFECTS

In theory, the problem of cycling effects is one of path and history dependence and hence, for an ideal elastic material which is path and history independent, there are no effects whatsoever. Cam clay models assume that the behaviour of soils below the state boundary surface is elastic and hence, no effects are expected for a stress cycle within the state boundary surface. It is well known that soils are influenced by strees cycling and hence, it is necessary to present a hypothesis on cycling effects.

When soil is loaded, clay particles may slip, re-orientate and readjust their positions to sustain the additional load. If irreversible slipping takes place then the process is accompanied by irrecoverable deformations and dissipation of energy. Alternatively, unloading leads to recoverable deformations and release of stored energy. These readjustments of particles are also time dependent and therefore the periods of loading and unloading are important in considering the new state of the sample. It is relevant to mention that stable measured values of stresses and strains at the boundaries do not necessarily mean that no further structural readjustment is taking place within the body of the sample.

In drained tests, cyclic loading is accompanied by volume changes and distortional strains and in undrained tests, it is accompanied by pore pressure changes and distortional strains. Continuous cycling with stress amplitude below a limit known as threshold, leads to a state of equilibrium, in which no irreversible slipping between particles takes place and hence, recoverable deformations only occur. The deformations and pore pressure gradually approach asymptotes in subsequent cycling as shown in figure 3.5. Alternatively, continuous cycling with shear stress amplitude exceeding the threshold may lead to failure.

3.4.3. MECHANICAL BEHAVIOUR DURING CYCLIC LOADING

There are various ways of subjecting soils to cyclic loading in the laboratory. In a shear box the sample may be subjected to one or two directional strain cycles. Alternatively, the sample may be subjected to cycles of isotropic or anisotropic stresses.

The behaviour of clays subjected to undrained cyclic loading was investigated by several workers, such as Taylor and Bacchus, (1969), Lee and Focht (1976) and Anderson (1976). Typically, a complete cycle of loading,unloading and reloading reveals a hysteresis loop as shown in figure 3.7.and the size of this loop depends on the magnitude of the stress cycle and the area inside the loop represents the energy supplied to the sample during the cycle.

For high amplitudes this energy decreases rapidly with the number of cycles, while for low amplitudes it reaches a relatively small and constant value as shown in figure 3.8. Figure 3.6 shows that the initial recompreseion tangent modulus is nearly constant, while the recompression secant modulus decreases and that irrecoverable strains increase as cycling continues.

3.4.4. SOME EFFECTS OF CYCLIC DISTURBANCE ON THE MECHANICAL BEHAVIOUR OF SOILS

Undrained cyclic loading disturbs the clay structure, changes the effective stresses and consequently alters the mechanical behaviour of soils. Samples subjected to increasing number of undrained loading cycles are assumed to simulate samples at higher degrees of disturbance. Hence, the behaviour of cyclically disturbed samples may be used to indicate some of the effects of sampling disturbance on the mechanical properties of soils.

The effect of cyclic disturbance was reported to decrease the undrained strength of clays (e.g. Okumura, 1971, Lee and Focht, 1976 and Anderson, 1976). The reduction of strength was found to increase with increasing number of cycles and with larger amplitudes. The pore pressure was latered by undrained cyclic loading in two ways. Firstly, there was a permanent increase of initial pore pressure , and secondly the pore pressure response during subsequent static loading was altered. Taylor and Bacchus, (1969) and others reported significant similarities between the stress paths of disturbed samples and overconsolidated samples as shown in figure 3.9 . Cyclic disturbance caused a lower shear modulus, higher strain at failure and a higher volumetric stiffness and the effect of cyclic disturbance on the compression curve is shown in figure 3.10.

If a cyclically distrubed sample is allowed to consolidate, then equilibrium is established at a lower void ratio, which may lead to higher strengths and stiffness (Anderson 1976).

3.4.5. SOME EFFECTS OF STRESS DISTURBANCE ON THE MECHANICAL BEHAVIOUR

Several workers have investigated the effects of stress disturbance on the behaviour of laboratory virgin samples and their results are summarized in table 3.5. Samples wereconsolidated to a virgin state as outlined in section 1.4 and the stress disturbance was imposed either by an undrained release of the axial force in a triaxial cell or by an undrained release of the total stresses in a triaxial or oedometer cell. In theory, the undrained release of the total normal pressure p has no effect on the effective stresses of a saturated sample, provided that its state remains inside the state boundary surface, but this is not necessarily the case.

Perfect samples of different clays were investigated under various loading conditions and the values of strength and stiffness parameters are presented in table 3.5. It may be observed that stress disturbance influences the stiffness more than the strength and the effects are more pronounced in sensitive clays. The effects of stress disturbance on the pore pressure response may be better understood by observing the stress path during unloading and re-loading in figures 3.11 and 3.12, rather than by assessing the changes in the pore pressure parameter A shown in table 3.5. The non-linear stress path during unloading may suggest an elasto-plastic behaviour, while reloading showed a linear stress path which confirms an elastic behaviour. Figure 3.13 shows the volumetric stress-strain relationship before, during and after perfect sampling, and the behaviour may be idealised to that in figure 3.14. The deviatoric stress-strain relationship is influenced by the stress state and the loading conditions (Ladd, 1964) and hence, care must be taken in interpreting it. Figure 3.15. shows that stress disturbance slightly decreases the stiffness in an undrained test. Davis and Polous (1967) observed that reconsolidating a perfect sample to its previous stresses will lead to higher strength and stiffness (See table 3.6).

3.5. CALCULATION OF DISTURBANCE PARAMETERS AND UNDISTURBED BEHAVIOUR

3.5.1. INTRODUCTION

The mechanical properties of soils are altered by sampling disturbance and hence, they can be used to calculate disturbance parameters. The properties of in-situ soils are required as references in calculating disturbance and they can be estimated either by theoretical calculations or by investigating the behaviour of the least disturbed sample. Although disturbance parameters provide quantative measurements of sample disturbance they do not necessarily provide correction factors. Some of the disturbance parameters are reviewed and discussed in this section.

3.5.2. THE RESIDUAL STRESS OF DISTURBED SAMPLES

Since soil is a stress dependent material, changes in its mean effective pressure p' may be considered a convenient parameter in calculating the disturbance factor D_p. An element of soil suffers various changes in its stress state during sampling as shown in figure 3.16. The release of in-situ stresses will transfer the material from state A to state B in figure 3.16 and other causes of disturbance will transfer it to state C. The residual effective stress of a sample may be calculated theoretically (p'_c) and measured in the laboratory (p'_m) and a disturbance parameter may be calculated as follows:

$D_p = p'_m/p'_c$ (Ladd and Lambe 1963)

$D_p = p'_m - p'_c$ (Noorany and Seed 1965)

$D_p = 1 - p'_m/p'_c$ (Okumura 1971
Nelson et al 1971)

There are various problems in calculating the residual effective stress p'_c. Skempton and Sowa (1963) derived equatious to calculate p'_c for lightly overconsolidated and heavily over consolidated soils as

$$p'_c = (\sigma_{vo} - u_o)\left[K'_o + \bar{A}_u (1-K'_o) \right] \quad \ldots\ldots \text{ for } K'_o < 1$$

and

$$p'_c = (\sigma_{vo} - u_o)\left[1 + \bar{A}_u (K'_o - 1) \right] \quad \ldots\ldots \text{ for } K'_o > 1$$

The initial total vertical stress in the ground can be calculated from the unit weights of different layers above the field element and the initial pore pressure in the ground can be measured by a piezeometer or by specifying the water table. The parameter K'_o is defined as the ratio of the horizontal effective stress to the vertical effective stress in the ground and the values of K'_o varies with the stress history, stress state and soil type (Brooker and Ireland 1965). For normally consolidated clays the values of K'_o are often approximated by the equation.

$$K'_o = 1 - \sin \phi' \quad \text{(Jaky 1941)}$$

Where ϕ' is the angel of shearing resistance. For overconsolidated soils the value of K_o' may be larger than one and varies with the overconsolidation ratio and the plasticity index. It is extremely difficult to determine the value of K_o' in the ground. Methods based on soil samples are inaccurate because of sample disturbance and in-situ methods such as these considered by Hughes et al (1977), are relatively new and have not yet gathered such data.

The parameter $\bar{A}_u$ relates the changes in pore pressure to changes in the deviatoric stress during unloading saturated soils as $\Delta u/(\Delta\sigma_1 - \Delta\sigma_3)$ and it is derived from Skempton's general equation for changes in pore pressure (Skempton, 1954). Values of $\bar{A}_u$ depend on the stress level and soil type and vary during loading and unloading (Noorany and Seed 1965). Figure 3.17. shows the possible range of values of some mean effective pressures for different values of $\bar{A}_u$. Therefore, it may be concluded that the calculated residual stress p_c' is subjected to significant errors caused by inaccurate values of K_o' and $\bar{A}_u$.

Furthermore, it is difficult to measure the residual effective stress of a disturbed sample. Direct and indirect methods of measuring the residual stresses were proposed by Skempton (1961) and Ladd (1961) and further discussion on measuring negative pore pressures are introduced in Appendix A.4. Schjetne (1971) measured in-situ pore pressures during sampling and observed that the initial values of negative pore pressure were small and they diminshed as time passed or if the sample was subjected to severe disturbances. The residual stress of a disturbed sample determines whether the sample is on the wet side or the dry side of the critical state line as shown in figure 3.18 and to this extent, it determines the stress path and the stress-strain relationships during loading.

3.5.3. The RECONSOLIDATION CURVE OF A DISTURBED SAMPLE

The compression and swelling behaviour of soils may be approximated to linear relationships as shown in figure 3.19. As argued in section 2.6 if the effect of disturbance is to reduce the effective stress state under a constant specific volume, then the state of soil is shifted from its in-situ state A on the compression curve to state B on the swelling curve as shown in figure 3.20. If the sample is reconsolidated to its previous effective stresses then it will have a lower specific volume and a higher volumetric stiffness at state C on the swelling curve and Noorany and Poormand's (1973) observations showed the above pattern of behaviour in figure 3.21. Thus, it is possible to derive some disturbance parameters based on the reconsolidation behaviour of soils.

Terzaghi (1941) recognised the effects of disturbance on the one dimensional compression behaviour of soils and Rutledge (1944) observed that it shifts the e - log p' curve downwards, decreases its slope and obscures the sharp yield point at the maximum preconsolidation pressure as shown in figure 3.22. Schmertmann (1955) suggested that the e-log p' curve of a remoulded sample and a virgin sample may be used as lower and upper limits respectively to indicate the amount of disturbance. The one-dimensional compression curve of a virgin sample may be constructed theoretically and the maximum preconsolidation pressure may be estimated by several methods (e.g. Casagrande 1936 and Leonards 1962) and the compression curve of a remoulded sample may be observed in the laboratory. The more the sample is disturbed the more its position is shifted towards the remoulded behaviour and Schmertmann (1955) defined this shift at the estimated preconsolidation pressure as the disturbance parameter D_p;

$$D_p = \Delta e / \Delta e \text{ max}$$

Where Δe maximum is the difference in void ratio between the two limit curves at the estimated pre-consolidation pressure and Δe is the difference in void ratio between the virgin behaviour and the disturbed behaviour as shown in figure 3.22b.

Alternatively, Bartlett and Holden (1968) and Bromham (1971) used the slopes of the compression curves of the virgin, disturbed and remoulded samples to calculate a disturbance parameter D_p as

$$D_p = \frac{p'_v - p'_d}{p'_v - p'_r} \times 100$$

Where p'_v, p'_d and p'_r are shown in figure 3.23. In practice, p'_r/p'_v is small and hence, the disturbance parameter may be re-defined as

$$D_p = | 1 - (p'_d/p'_v) | \times 100$$

and this definition has the advantage that a test on a remoulded sample is not required.

3.5.4. THE UNDRAINED STRENGTH OF A DISTURBED SAMPLE

In theory, the effective stresses and specific volumes of soils at failure are uniquely related and,hence, virgin, perfect and disturbed samples are expected to have the same undrained strengths so long as they have the same initial specific volume. However, Noorany and Seed (1965) argued that it is possible for virgin and perfect samples to have different undrained strengths at the same initial specific volume. They argued that different pore pressure responses in virgin and perfect samples may lead to differences in their effective pressures at failure, but this is not necessarily the case, since it is possible to have differences in the stress paths of virgin and perfect samples that lead to the same effective pressures at failure.

Pore pressures at failure in undrained tests have been found to be severely altered by mecahnical distortions (Skempton and Sowa 1963) and thus the undrained strengths were altered even though the specific volume was unchanged.

Undrained strengths of in-situ elements and disturbed samples may be measured in various tests and thus, differences in undrained strengths may be used to indicate the amount of disturbance.

The undrained strength of, for example, an in-situ element in a vane test C_{uv} or that of a perfect sample from a block sample C_{up} or that of a remoulded sample C_{ur} may be used as references to indicate the reduction in strength caused by disturbance. For example Okumra (1971) claculated a disturbance parameter as:

$$D_p = \frac{C_u}{C_{up}}$$

It is important to notice that the undrained strength of a sample can be measured either at its initial water content or at its previous effective stresses. The unconfined compression test and the unconsolidated undrained test show lower strength values than isotropically and anisotropically consolidated undrained strengths, as shown in table 3.5. The former tests preserve the initial water content, but have lower effective stresses, while the latter preserve initial mean effective stresses but have a lower void ratio.
In theory, a perfect sample in an unconsolidated undrained test must have the same strength as a virgin sample in an undrained test.

3.5.5. THE SHEAR STRESS- STRAIN RELATIONSHIP OF A DISTURBED SAMPLE

The effects of disturbance on the behaviour of samples are reflected clearly in the total and effective stress-strain relations of virgin and disturbed samples and hence provide some parameters to indicate the amount of disturbance.

Goodman and Leininger (1967) proposed that the shear stress-strain relationships of block and remoulded samples provide an upper and lower limit to the amount of disturbance respectively. They used the work per unit deformation W as a parameter to indicate disturbance as

$$D_p = \frac{W_B - W_R}{W_B - W_D}$$

Where W_B, W_D and W_R are the work required to deform block, disturbed and remoulded samples respectively in a shear box test to an angular displacement of 40^o as shown in figure 3.26.

3.5.6. CALCULATION OF UNDISTURBED PARAMETERS

Virgin parameters of soils may be obtained from disturbed samples, either by restoring the virgin state of the sample before testing or by correlating disturbance parameters to disturbed parameters and extrapolating the relationship to the undisturbed value.

Ladd and Foott (1974) suggested that reconsolidating the disturbed sample to a higher stress level restores the undisturbed state and Davis and Poulous (1967) suggested that the sample should be reconsolidated to its anisotropic stress before testing. The stress level required to eliminate sampling disturbance varies with different soils and it is likely to increase with increasing initial stress level. Ladd (1969) suggested that the behaviour of disturbed samples at higher stress levels must be normalised and thus, the undisturbed behaviour may be obtained at any stress level by some calculations. Vincent and Massarsch (1979) showed that a unique stress-strain relationship for disturbed and undisturbed samples may be obtained by normalising the values of both stress and strain parameters.

Alternatively, Seed et al (1964) measured the values of the A parameter at various degress of disturbance and deduced the virgin value by back extrapolation. Some other approahces on extrapolation were suggested to calculate the coefficient of consolidation, the maximum preconsolidation pressure (Schmertman 1955), and the undrained strength (Calhoun 1956).

Similarities between disturbed samples and overconsolidated samples were reported by several workers (e.g. Taylor and Bacchus 1969) and Ladd and Lambe (1963) and Nelson et al (1971) assumed that the behaviour of a disturbed sample is similar to an overconsolidated sample with an overconsolidation ratio $D = p'_m/p'_c$.

3.6. SUMMARY AND PRESENT APPROACH

3.6.1. INTRODUCTION

Effects of sampling disturbance on different aspects of soil behaviour vary with different sampling techniques and soils and an attempt has been made in this chapter to present some of the previous approaches to the problem of identifying and quantifying disturbance. The main conclusions on sampling disturbance are summarised in this section and the approach taken in the present research outlined.

3.6.2. VARIATION OF SAMPLE DISTURBANCE IN DIFFERENT SOILS

Soils may be classified according to their composition (grain size distribution and Atterberg limits), stress state (e.g. normally consolidated, lightly or heavily consolidated) and fabirc (e.g. fissured or cemented or layered). The effects of sampling disturbance varies for different soils and hence, it is important to limit observations on sample disturbance to the soil type tested. Some of the previous observations on sample disturbance in different soils may be summarized as follows:

(i) normally consolidated soils suffer more disturbance than heavily overconsolidated soils.

(ii) soils at high stress levels suffer more disturbance than soils at lower stress levels.

(iii) sensitive clays (e.g. cemented or quick clays) are severely disturbed by sampling disturbance in general and by tube sampling in particular,

(iv) in fissured clays, the size of the sample is the most important factor in addition to disturbance.

3.6.3. VARIATION OF SAMPLE DISTURBANCE IN DIFFERENT SAMPLING AND TESTING PROCEDURES

Some of the previous observations on the effects of sampling techniques and testing procedures may be summarised as follows:

(i) block samples are of the best quality and provide the least disturbed parameters with reference to those obtained from in-situ testing,

(ii) the state of a tube sample is non-homogeneous and the ratio of the wall thickness of the sampler to the diameter is an important cause in mechanical distortions, leading to larger disturbance for larger ratios,

(iii) the behaviour of the samples in unconsolidated undrained tests show less disturbance than the behaviour in unconfined compression tests, but both tests underestimate in-situ parameters,

(iv) the behaviour of samples observed in anisotropically reconsolidated undrained tests is least disturbed and may slightly overestimate in-situ parameters, since the samples have lower specific volumes.

3.6.4. Some Effects of Sampling Disturbance on the Mechanical Behaviour

Previous observations of the effects of sampling disturbance on different aspects of the behaviour of soils may be summarised as follows:

(i) the peak strength for drained loading is reduced by different percentages in different soils, due mainly to reduction of the cohesion parameter c'.

(ii) the undrained strength is slightly reduced but the angle of friction at the critical state ϕ'_{cs} is not altered,

(iii) the consolidation behaviour is slightly altered by disturbance showing a lower compression index (C_c) over the range of virgin compression and a higher swelling index (C_s) below the initial stress level.

(iv) the deviatoric stress-strain relationship is severely altered by disturbance showing lower stiffness.

(v) the pore pressure response is seriously altered by disturbance and effective stress paths of disturbed samples are similar to those of

3.6.5. PRESENT APPROACH TO SAMPLE DISTURBANCE

Most of the previous observations on sampling disturbance were concerned with the effects of disturbance on strength and consolidation parameters of soils and were based on comparisons between disturbed samples, block samples and in-situ tests. The different causes of disturbance were recognized but they were not investigated or examined separately and highly disturbed samples were produced by stress cycles which do not necessarily represent disturbance in tube samples.

The present research is aimed at the following:

(i) to produce samples that simulate virgin, perfect and tube samples.

(ii) to investigate the effects of disturbance on the deformation properties of samples in different stress states and loading conditions.

(iii) to examine the behaviour of virgin and disturbed samples within the framework of the fundamental behaviour of soils and the critical state concepts.

(iv) to investigate the effects of different methods of preparing and testing samples on the behaviour of a disturbed sample.

The approaches and procedures for preparing and testing samples will be described in the next chapter and the testing programme and the basic results will be presented.

CHAPTER 4 DESCRIPTION OF APPARATUS AND TESTING PROCEDURES

4.1. INTRODUCTION

In the research described in this thesis, the effects of sample disturbance on the mechanical properties of clay were investigated using triaxial tests by observing the effects produced by imposing an unloading cycle and a tubing process. Each series of triaxial tests was conducted on initially similar homogeneous samples of the same known clay and samples were prepared in three different ways to reproduce virgin samples, perfect samples and tube samples. Preparation of the virgin samples was carried out in three stages, by formation of a slurry, preliminary consolidation and triaxial consolidation. A perfect sample was prepared from a virgin sample by releasing and re-imposing the total stresses under undrained conditions without removing the sample from the apparatus and a tube sample was prepared from the perfect sample by a simple tubing process.

This chapter describes the procedure and the apparatus used for preparing and testing the samples. Some sections in the appendix are devoted to discussing details of the procedures and the interferences of the appratus on the behaviour of soils.

4.2. SAMPLE FORMERS

4.2.1. INTRODUCTION

All samples were prepared by anisotropic compression of kaolin clay from a slurry. This was done in two stages; in the first stage samples were consolidated from a slurry in a sample former to a state where they could just be handled and in the second stage the samples were further consolidated in a triaxial apparatus following approximately the previous stress path to maintain one dimensional consolidation.

Two different sample formers were used to prepare samples 1½" and 4" in diameter and 3" to 8" in leugth respectively and the purpose of this section is to describe the sample formers. An important consideration in the use of the sample formers was side friction and this aspect will be discussed in detail in appendix A1.

4.2.2. THE 1½" SAMPLE FORMER

The sample former was designed by the writer and constructed in the workshop of the Civil Engineering Department at University College, Cardiff. Photograph (1) shows the components of the sample former and photograph (2) shows the sample former in operation. The components shown in photograph (1) are A an internal split cylinder, B an external cylinder and C supplementary loading features.

The internal brass cylinder contained drilled holes filled with porous stones glued in place to facilitate drainage; the size of the holes and the spaces between them are shown in figure 4.1. The internal surface was machined smooth to reduce side friction while the external surface was grooved spirally and vertically to collect the drainage water. It was designed as a split cylinder to avoid having to push out the sample causing additional frictional disturbance. The internal cylinder is held in position by an external aluminium cylinder and an aluminium base, shown in figure 4.2., could be screwed to either end of the external cylinder to allow the loading direction relative to the sample to be reversed. The aluminium base of the sample former was also grooved and drilled to allow drainage. A brass loading platen, drilled and grooved to allow for drainage, a porous stone and a filter paper are used at both ends as shown in figure 4.2.

A piston made of brass slightly less than 1½" in diameter and weighing one pound was used to transfer the dead weights to the slurry through the top platen. A thin bar, 12 inches long was welded to the centre of the brass cylinder and steel cylinders slightly less than 1½" in diameter were used as initial load increments. The weight hanger and the steel cylinders were able to slide easily into the sample former without side friction or locking. The consolidation pressure was applied by dead weights added in increments and the sample former was submerged in water.

4.2.3. THE 4" SAMPLE FORMER

The 4" sample former was designed by Stipho (1978) and constructed in the Civil Engineering Department of University College, Cardiff and its construction and use were described in detail by Stipho (1978). The 4" sample former was originally designed so that the consolidation pressure was applied by a hydraulic pressure system but for the present research it was modified and the stresses were applied by dead weights in a way similar to that used with the 1½" sample former. Photograph (3) shows the components of the sample former and figure 4.3. shows the former being operated in a similar way to the 1½" former.

Figures 4.4. and 4.5. show the details of the 4" sample former and simply it consists of a pressure chamber, a moulding chamber and a top diaphragm. The moulding chamber consists of an internal brass cylinder, an external aluminium chamber and a base plate. The internal brass cylidner was perforated with a network of $\frac{1}{8}$" holes as shown in figure 4.4. and a mixture of sand with plaster of Paris was used as a filler to allow drainage. The inside and outside surfaces of the internal cylinder was machined smooth and it was fitted inside the external aluminium chamber with an $\frac{1}{8}$" gap between the two surfaces. The base of the chamber was grooved and drilled to allow for drainage and connected to a small reservoir elevated above the sample former level.

The pressure chamber functioned as an extension to the moulding chamber, but with no brass sleeve or internal side drainage. This did not make significant differences during the consolidation process, since the sample falls below thepressure chamber at an early loading stage. Other features of the sample former were porous stones and filter papers to allow drainage at both ends and a top diaphragm to transfer the loading pressure to the sample. The rigid brass loading diaphragm is shown in detail in figure 4.5. and it was drilled to allow for top drainage. An aluminium tube was used as an extension to transfer dead weights to the diaphragm and a steel bar was screwed to the centre of the diaphragm to ensure the stability of the dead weights. Loading pressure was applied through the rigid diaphragm by dead weights applied in increments.

4.3. TRIAXIAL CELLS

4.3.1. INTRODUCTION

The apparatus used for anisotropic consolidation and for testing samples was the standard triaxial apparatus and three separate triaxial apparatuses were used in the research. One was a standard 4" cell, another was a standard 1½" cell, both described by Bishop and Henkel (1962) and the third was a stress controlled 1½" cell described by Bishop and Wesley (1975).

Virgin and perfect samples were consolidated in one or the other of the standard 1½" cells and tested without removing them from the cell after consolidation. The 4" triaxial cell was used for consolidating virgin samples which were then transferred as 1½" tube samples to the 1½" cells for testing.

4.3.2. THE 4" TRIAXIAL CELL

The principle features of the 4" triaxial cell were described by Bishop and Henkel (1962). For anisotropic consolidation it is desirable to have stress controlled loading and this was done by providing a Martin air piston acting on the loading ram as shown in figure 4.6 and Photograph 4. Air pressure was supplied from the Bromwade compressor through a Fairchild manostate valve and an air reservoir was supplied in the line to smooth any sudden variation of pressure. The cell pressure and pore pressure were both supplied by air pressure acting in an air/water cylinder as shown in figure 4.6 and Photograph 4; the air pressure was supplied by the Bromwade compressor and controlled by Fairchild manstat valves.

Axial loadswere measured on a proving ring and pore pressures and cell pressures were measured using pressure guages with a sensitivity of 0.2 psi/div.

The 4" sample was provided with side filter paper drains, filter paper and a porous stone at the top and was sealed in a 0.01" thickness rubber membrane with double O rings at both ends.

4.3.3. THE STANDARD 1½" CELL

The standard 1½" triaxial cell described in detail by Bishop and Henkel (1962) was used for consolidating and testing virgin and perfect samples. The cell was provided with an internal axial load cell and with Bell and Howell electrical transducers to measure the cell and pore pressures. A burette and a dial gauge were provided to measure the volume changes and the vertical deflections respectively. The cell was mounted in a strain controlled triaxial loading frame with an air piston between the ram and the loading frame as shown in figure 4.7 so that both stress controlled and strain controlled tests could be conducted. The axial load, cell pressure and pore pressure were supplied separately through an air-water pressure system similar to those of the 4" cell.

The 1½" sample was provided with bottom drainage and side filter paper drains to accelerate consolidation and to assist continuity of the pore water inside the sample and a filter paper was placed on top of the porous stone. The top cap was a rigid perspex with a smooth surface and it was provided with a spherical seating, using a split steel ball in a conical recess.

This system allows tilting but not sideways movements. One rubber membrane, 0.008" in thickness, enclosed the sample and double O rings $1\frac{3}{8}$" in diameter, were used to seal both ends.

4.3.4. THE STRESS CONTROLLED $1\frac{1}{2}$" TRIAXIAL CELL

Some virgin and perfect samples were consolidated and tested in a stress controlled cell described by Bishop and Wesley (1975). The cell was provided with a burette and a dial gauge to measure volume changes and vertical deflections repectively and with electrical resistance transducers to measure the cell pressure and the pore pressure. The $1\frac{1}{2}$" sample was provided with side filter paper drains, filter paper and porous stone at the base and sealed in a rubber membrane with double O rings at both ends. The axial load and the cell and pore pressures were supplied to the cell through manually operated self compensating mercury control systems. The layout of the loading system is sketched in figure 4.8 and the system is shown in photograph (6).

4.4. MEASUREMENTS OF LOADS, PRESSURES AND DIPLACEMENTS IN TRIAXIAL TESTS

4.4.1. INTRODUCTION

In most tests loads and pressures were measured by electircal resistance transducers, displacements by dial gauges and volume changes by burettes and all tests were conducted at a constant temperature. In this section the measurement equipment is described and the accuracy of the measurements is assessed.

4.4.2. MEASURING AMPLIFIER

An automatic Peekel switch box type number 10U was used to connect all transducers to a strain measuring instrument, Peekel type number 581 ONH. The detailed specifications of the switch box and the amplifier are described in the operating and service manual of Peekel instruments. A full bridge configuration was used in all measurements with a bridge voltage of 5 volts and a gauge factor of 2. The amplifier was calibrated using the balance control as calibration source and all micro-strains were measured by the zero method, where a visual meter is simply used as a null-indicator. The meter is balanced to read zero by means of 4 balance controls that are directly calibrated in microstrain. A low sensitivity range was selected for initial balancing, followed by a more sensitive range and re-adjustment to the balancing point if necessary.

4.4.3. LOAD CELLS AND PRESSURE TRANSDUCERS

Internal load cells and pressure transducers were used to measure the axial forces and the cell and pore pressures respectively.

All load cells and transducers were calibrated at least once each six months and there were negligible differences with the passage of time. The maximum forces and pressures used in the calibrations exceeded the maximum axial force and cell pressure that were applied during the course of tests. All the load cells and transducers showed approximately linear calibration curves during loading, unloading and reloading. Table 4.1. shows the accuracy and the sensitivity of each transducer and figures 4.9. and 4.10. show the calibration curves of one transducer and one load cell as examples. Thermal insulation of the transducers reduced the effects of any slight changes in temperature and the transducers were not disconnected at any time during tests to avoid further alterations to the measurements.

The cell pressure transducer was flushed with water and connected directly to the cell chamber. The pore pressure transducer was connected to a brass block that was connected directly to the drainage outlet of the cell so as to allow provision for careful de-airing of the pore pressure connections with a short length of stiff connections.

4.4.4. DISPLACEMENTS AND VOLUME CHANGES

Dial gauages and burettes were used to measure axial deflections and volume changes respectively. The dial gauge was clamped to the external frame of the stress controlled 1½" cell and measured the vertical movement of the base with a sensitivity of .002 mm/div.The dial gauge was clamped to the load ram in the standard 1½" cell and measured the relative movement of the ram towards the base with a sensitivity of .01"/div. The burettes were described by Bishop and Henkel (1962) and they had provision for reversal of the direction flow as required. Each burette was cleaned after each few tests and the paraffin replaced. The sensitivity of the burette was 0.2cc/div but the accuracy of the readings was rather less, due to non-uniformity of the meniscus.

4.5. SAMPLE PREPARATION

4.5.1. INTRODUCTION

All samples were prepared by one dimenasional consolidation of kaolin slurry in two stages, to produce virgin samples and these were used to produce perfect and tube samples. In this section, the materials used, the method of preparation of the slurry and the methods of preparing virgin, perfect and tube samples will be described.

4.5.2. MATERIALS

The properties of kaolin clay are well documentated in soils literature and the critical state models were developed and modified largely as a result of observations of kaolin in triaxial testing.

Hence, kaolin was used in the present investigation although it has a relatively high rate of secondary consolidation.

The kaolin was commercial quality supplied by Whitfield and son Ltd. in a form of a dry powder and was from the same batch as that used by Stipho (1978). The Atterberg limits of the fresh clay were LL=51% and PI=22% and all tests on virgin and perfect samples were carried out using freshly prepared clay. However, there was not enough of the original batch to allow 4" samples to be prepared from fresh clay and thus, reused clay was used to prepare tube samples in preference to using fresh clay from a new batch with different properties. The Atterberg limits of the clay after remoulding, drying crushing and remixing were LL = 52% and PI = 22% which were essentially the same as that of fresh clay. These properties were found to remain the same after further use and it may be assumed that other engineering properties remained basically constant also. The Atterberg limits of fresh clay as measured by Stipho (1978) were LL = 52% and PI = 26% which were only very slightly different than the authors observation. This was either due to the effects of the storage period or to differences in the procedures in which the measurements were taken.

4.3.5. PREPARATION OF SLURRY

Clay slurry with the initial water content well beyond the liquid limit is commonly used as an initial state for sample preparation. Higher initial water contents provide higher degrees of saturation and a higher freedom of particle orientation but require larger initial volumes and longer consolidation periods. A moisture content of approximately twice the liquid limit was used and the slurry had an initial volume of approximately double the volume of the sample required.

To prepare a small quantity of slurry for the 1½" former, 200 gms. of kaolin powder were added to 225cc of water in a plastic jar. The kaolin powder was added in portions and mixed with water over a period of 15 minutes. Each portion was mixed with the water manually using a spatula until a homogeneous slurry was formed before the next portion was added and mixed in the same way. For larger amounts of slurry, 2.5 kgs. of kaolin was mixed with 2.8 litres of water in the container of an electric clay mixer. The mixer was run at various speeds for half an hour with a few intervals of manual shaking and mixing before a homogeneous slurry was formed. The slurry was trasferred to a chamber and left for half an hour under vaccum. Also it was left on a vibrator for a few minutes to ensure the release of any entrapped air and the slurry was allowed to settle for a few minutes before it was transferred to the formers.

4.5.4. PRELIMINARY CONSOLIDATION

In order to obtain samples that can be set up in a triaxial apparatus, the slurry was consolidated in sample formers to a stress level where samples could be handled.

Before transferring the slurry to the sample former the internal chamber of the sample former, the porous stones and the upper loading cap were submerged in water for half an hour. The sample former was then assembled and the saturated porous stones with filter paper on top were placed at the bottom of the former and flooded with water to ensure saturation. The slurry was poured into the sample former slowly and carefully to prevent air being trapped within the sample. After adding each portion the sluury inside the former was tamped slightly and more slurry added until the former was filled to approximately 8" and 18" for the 1½" and 4" formers respectively.

A saturated porous stone with a saturated filter paper beneath were slid up to the slurry in the 1½" sample former and some water was placed on top to ensure saturation, the top platen was slid onto the top of the porous stone and the whole assembly was submerged in water to its mid height. In the 4" sample former, some water was added on top of the slurry and the top cap with a saturated filter paper beneath was slid up to the slurry level.

The base of the former was connected to a small water reservoir and, with porous stones at the top, bottom and sides of each sample, former drainage was allowed in all directions and the sample remained saturated.

After one hour, the sample former was loaded by adding a weight approximately double the weight of the top loading platen. Thereafter, the load increments shown in Table 4.2 were added. It can be seen that each increment was approximately doubled and an interval of 12 hours was allowed for primary consolidation after each increment of load. After the last increment was added the sample former was left for two days.

On completion of loading, the sample was quickly unloaded, the base of the sample former dismantled and the sample pushed out carefully towards the bottom end to sit in a clean lubricated mould of standard size. The sample was then cut to the required length and both ends were carefully levelled using a wire saw. Slices of both ends were saved and their water contents measured. The sample was then ready to be transferred to the triaxial cell.

4.5.5. TRIAXIAL CONSOLIDATION OF VIRGIN SAMPLES

Samples were initially prepared in the formers were non homogeneous and had suffered an unloading cycle and it was necessary to consolidate them to a higher stress level to restore them to a homogeneous state and to eliminate the stress disturbance. All samples were reconsolidated anisotropically, following the previous stress path to a stress state of $\sigma_1' = 200$ kN/m^2 and $\sigma_3' = 175$ kN/m^2, which was more than three times the calculated average effective etresses of the initial sample as shown in figure 4.11. and discussed in Appendix A.1. It is assumed that this procedure eliminates all initial disturbances and experimental evidence in support of this assumption was given by Balasubramaniam (1969) and Ladd and Foott (1974).

Before placing the sample in the cell,all tube valves and pressure supplies were flushed with water and a porous etone was left in boiling water for half an hour. A soaked filter paper was placed on top of the porous stone and water was allowed to flood over the filter paper. The sample was placed carefully on the flooded filter paper and it is likely that some water was absorbed by the sample,due to suction. This did not have a significant effect, since the sample was restored to a homogeneous state at higher stress levels. Side filter drains were soaked with water and placed gently around the sample, overlapping the bottom filter paper, the top cap greased to reduce friction and placed on top of the sample. In the 4" cell, the base of the cell was greased and the top cap was provided with a saturated porous stone and a soaked filter paper. The rubber membrane was stretched along a brass cylinder and was released around the sample in the usual way and O - rings placed at both ends, after greasing the surfaces to seal the sample.

The sample was reloaded to approximately its previous mean effective stress state, with a back pressure of 100 kNm^{-2} and a cell pressure of 150 kNm^{-2}, so that the state was at A in figure 4.11 and water was then flushed through the drainage leads under the applied back pressure to ensure saturation. The sample was allowed to consolidate for 6 hours under these stresses before the estimated previous deviator stress of 30 kNm^{-2} was re-applied and the sample brought to state B in figure 4.11. The sample was then consolidated with a loading path given by $q'/p' = \frac{1}{2}$, shown as BC in figure 4.11 to give approximately one dimensional conditions. Load increments of approximately 17.2 kNm^{-2} in cell pressure and 10.3 kNm^{-2} in the deviator stress were applied simultaneously. A period of 12 hours was allowed for primary consolidation, after each load increment and measurements of stresses and deformations were made, before further increments were applied. Finally, the sample was allowed to rest for 24 hours at its final state at C in figure 4.11. Virgin samples prepared in this way are believed to represent the state of samples in the ground, in that both have essentially the same stress history and, hence, they may be used for further testing or simulated sampling process.

4.5.6. PERFECT SAMPLES

A perfect sample was obtained by undrained release of the total stresses on a virgin sample in the triaxial cell, transferring the sample from state C in figure 4.12. to state D. The process was simply done by closing the drainage valve, releasing the deviator stress and the cell pressure. The 1½" perfect sample remained in its rubber membrane in the cell and it was left for 12 hours to allow equalisation of the pore pressure within the sample, before reloading or testing.

4.5.7. TUBE SAMPLES

Tube samples were obtained by driving 1½" tube samplers into 4" perfect samples. Three sets of tube samplers of different wall thicknesses were used to obtain samples at different degrees of disturbance and to investigate the effects of increasing the wall thickness of tube samples. The tube samples were provided with sharpened ends and their dimensions are shown in Table 4.3.

A 4" perfect sample was carefully slid into a 4" cylinder which confined the sample during the tubing process. Figure 4.13. shows the whole assembly during the tube sampling process, where A represents the 4" sample confined in its cylinder and set up in the assembly.

B is a hydraulic jack operated manually and used to push the 4" sample upwards into a set of tubes shown as C. The appartus is shown also in Photograph (7).

The tubes were lubricated from the inside and were fitted to a steel frame, fixed to the hydraulic piston on top of the sample. The tubes were lowered in line with the 4" sample and the hydraulic jack pushed the 4" sample in steps towards the tubes. After reaching the base of the assembly, the hydraulic pressure was released and the tubes dismantled. Sometimes, the tubes were stored for a few days before testing. During storage, they were wrapped in polythene and sealed, to ensure that there was no change of water content. The effect of short storage periods was found to be negligible, as will be discussed later in this thesis. Before testing, each tube sample was pushed out carefully from the tube into a lubricated standard mould, cut to the required length and the ends were saved for water content measurements. Finally, the 1½" tube sample was set up in the triaxial cell, following the procedure described in section 4.5.5.

4.6. RELOADING OF DISTURBED SAMPLES

4.6.1. INTRODUCTION

Virgin samples were already under total stresses and no further loading was necessary before testing, but perfect and tube samples were initially unloaded and, hence, may be tested or reloaded to different initial states before testing. A number of different reloading paths were applied, to investigate the effects of the initial stress state and different reconsolidation procedures, on the subsequent behaviour.

4.6.2. UNDRAINED RE-LOADING

The initial state of the tube sample in the triaxial cell was difficult to determine, due to the possiblity of swelling during preparation and the difficulty of making accurate measurements of negative pore pressures of perfect samples,as discussed in section 3.5.2. and Appendix A.4. In theory, changes in the pore pressure of saturated samples, are equal to changes in the cell pressure and for isotropic stress changes the sample is assumed to keep its initial effective stresses irrespective of the cell pressure. Hence, by re-applying the previous cell pressure and raising the pore pressure of tube and perfect samples to positive values, it is possible to measure the initial state of both samples more accurately.

Figure 4.14 shows the states of total stresses at A and B, before and after application of cell pressure. Alternatively, the deviator stress may be re-applied at C as shown in figure 4.14.

This addition of deviator stress, is likely to be associated with further changes in pore pressure and. hence, further alterations to the initial effective stress state.

The cell pressure was re-applied in one increment, from A to B in figure 4.14, under undrained conditions and it was left for 12 hours to ensure equalisation of pore pressures within the sample before testing. The deviator stress was applied in two or three increments from B to C , allowing a few hours between each increment, since failure was possible, if the deviator stresses were applied in one increment and a period of 12 hours was allowed before testing the sample at state C.

4.6.3. RECONSOLIDATION PATH C

Disturbed samples may be reconsolidated to their previous stress state following two different stress paths, as shown in figures 4.15 and 4.16. The reloading path C was applied in two stages. In the first stage, the cell pressure was re-applied in one increment, under drained conditions, to B where the cell pressure and the back pressure were 275 kN/m^2 and $100kN/m^2$, respectively. The sample was left for 12 hours to reach an equilibrium state before measurements were taken. In the second stage, the deviator stress was re-applied in three increments, a period of 12 hours followed before readings were taken and the sample was allowed 24 hours to rest, before further testing was carried out. Figure 4.15 shows the total and effective stress paths followed during C reconsolidation.

Although the sample recovered its previous stress state, the specific volume of the sample was inevitably reduced during consolidation.

4.6.4. RECONSOLIDATION PATH K

Some disturbed samples were reconsolidated to their previous stresses, following an alternative path K in figure 4.16. similar to that applied during the approximate one dimensional consolidation of the laboratory virgin sample. Firstly, a cell pressure of 150 kN/m^2 and a back pressure of 100 kN/m^2 were applied and the sample was allowed 12 hours to reach equilibrium at point B in figure 4.16. A deviatoric stress of 30 kN/m^2 was applied, taking the state of the sample to point C and after each 12 hours, an increment of cell pressure 17.25 kN/m^2 and an increment of deviator stress of 10.35 kN/m^2 were applied, until the sample reached its previous stresses at point D in figure 4.16. On completion of the loading path, the sample was left for 24 hours at the required stresses before further testing.

4.7. TESTING PROGRAMME

4.7.1. INTRODUCTION

The testing programme was designed to investigate the effects of the main causes of disturbance on the behaviour of kaolin,by conducting similar tests on virgin and disturbed samples.

The main series of tests included, undrained tests with different loading paths. The results provided data required to examine the fundamental behaviour of disturbed samples and, in addition, in many cases, the loading paths in the tests corresponded closely to those encountered in the field.

The testing programme also investigated other aspects of the behaviour of disturbed samples, such as the storage period, initial swelling, the loading path and stress level, and these aspects were examined in special tests and secondary tests. For the special tests, the methods of sample preparation or testing were slightly different to those employed for the main tests and secondary tests were conducted on samples produced from some main tests. This section describes all tests that were conducted in this research aud presents the results without detailed interpretation, which is left to Chapters 5, 6 and 7.

4.7.2. TESTS DESIGNATION

The testing programme covered a large number of different tests and it involved tests on samples which had a variety of stress histories representing different kinds of samples, tested under different loading conditions. To describe each test, it is convenient to use symbols, in addition to serial test numbers. The system of letters and figures used to identify particular tests in tables and graphs is described in this section and summarised in Table 4.4.

The main series of tests included, drained tests, which are described by three symbols and undrained tests,described by four symbols. For the drained and undrained tests the first symbol is a letter (K for reconsolidation path K or C for reconsolidation path C) which describes how the sample was prepared, followed by another letter (F for virgin, P for perfect, T,M and H for tube samples with thin, medium and thick wall samplers respectively), which describes the type of sample. The third symbol is a figure, which describes the loading path for a drained test (1 to 5 for loading paths shown in figure 4.17) or the stress state for an undrained test (6 to 9 for stress states shown in figure 4.18). All the drained tests in the main series were conducted on samples, initially at the same stress state, while all the undrained tests were by axial loading at constant cell pressure. The fourth symbol in designations of undrained tests is the letter a or b, corresponding to stress controlled or strain controlled respectively. Thus, for example, the test designated KF6a was a stress controlled undrained test , on a virgin sample,that was consolidated anisotropically to the stress state $q' = 105\ kN/m^2$ and $p' = 210\ kN/m^2$.

On completion of some drained tests, the samples were loaded to failure as secondary tests by undrained axial loading and the results are presented,together with some main undrained tests.

These secondary tests were given the serial number of their original drained test, followed by the letter (n) and either a or b to specify it as a stress or strain controlled test respectively. Thus, for example, the test designated 9na was a stress controlled undrained test, on the sample at the end of test 9 and test 9 was KF2.

The testing programme included a series of tests, in which samples were prepared or tested in a slightly different way from the samples in main series. These tests were called epecial tests and they were designated by the symbol S which is followed by a serial number and each special test is described in detail in section 4.7.4.

A few tests were repeated to examine the repeatability of the results and these were designated by the symbol R, followed by the serial number of the original test. Thus, for example, the test designated R63 was identical to test 63. Some other tests were repeated on samples that were stored for short periods, to examine the effects of delay and storage conditions and the letter G was used to describe the tests followed by the serial number of a similar test on an unstored sample. The period of storage in weeks is given in brackets following the dèsignation,. Thus, for example, the test designated G63 (2) was a test in which a sample similar to that in test 63 was stored for two weeks and loaded in a similar way to test 63.

4.7.3. PRESENTATION OF RESULTS

All tests that were conducted in this research were classified into 9 groups and presented in Table 4.5. They include 5 groups of drained tests that were classified according to their loading paths and 4 groups of undrained tests that were classified according to their initial stress states. The drained tests in groups 1 to 5 followed the loading paths 1 to 5 respectively in figure 4.17 and samples in groups 6 to 9 were at initial stress states corresponding to those in figure 4.18 and they were tested under constant cell pressure in undrained conditions.

The basic results of all tests are shown in figures 4.21 to 4.34 and the initial state (q'_o, p'_o, w_o%) of each sample before testing is quoted with each test. The results of drained tests are presented in terms of the relationships $\Delta q' - \Delta\varepsilon_a$, $\Delta p' - \Delta\varepsilon_v$ and $\Delta\varepsilon_a - \Delta\varepsilon_v$ and those of undrained tests were in terms of $\Delta q' - \Delta\varepsilon_a$ and $\Delta u - \Delta\varepsilon_a$.

4.7.4. SPECIAL TESTS

There were some samples in the testing programme that were designed to investigate other aspects of soil behaviour and sampling disturbance. These samples were prepared and tested in a slightly different way from those in the main test and they werecalled special tests.

For these, it is necessary to describe each different method of preparation and the loading conditions of each test.

Samples S1 and S11 were slightly overconsolidated and were tested in a stress controlled and strain controlled undrained conditions respectively. They were consolidated as virgin samples to a stress state of $q' = 115\ kN/m^2$ and $p' = 230\ kN/m^2$ then they were unloaded slowly to a stress state of $q' = 105\ kN/m^2$ and $p' = 210\ kN/m^2$.

Sample S2 involved drained cyclic loading, a virgin sample was consolidated anistotropically to a stress state of $q' = 105\ kN/m^2$ and $p' = 210\ kN/m^2$ then the axial force was released and after 12 hours,drainage was allowed and the sample was reloaded to its previous stress state,by applying increments of the axial load. This cycle of undrained,unloading and drained reloading to the same stress state,was repeated three times and in the fourth cycle,the sample was loaded to failure.

Samples S3 and S4 were intended to examine aspects of path dependency, where two virgin samples were loaded to the stress state of $q' = 145\ kN/m^2$ and $p'=290\ kN/m^2$ by following stress paths abc and adc respectively, as shown in figure 4.19. The two samples were then loaded anisotropically to a higher stress state of $q' = 195\ kN/m^2$ and $p' = 390\ kN/m^2$.

Samples S5 and S6 were perfect samples that were reloaded to their previous total stress under undrained conditions. Sample S5 was tested by stress controlled undrained loading and sample S6 was tested by strain controlled undrained loading.

Sample S7 was a thick walled tube sample, which was stored for one week in its tube. The sample was reloaded to its previous stress state by reconsolidation following path C and was left in the triaxial cell for one week at $q' = 105$ kN/m^2 and $p' = 210$ kN/m^2. The sample was tested by stress controlled undrained loading. This test was intended to examine the effects of storing the sample under its previous stress state.

Sample S8 was a tube sample, that was isotropically reconsolidated to $p' = 210$ kN/m^2 and loaded to failure in strain controlled undrained test. It simply represents a routine test on disturbed samples.

A thick wall tube sample S9 and two thin wall tube samples S10 and S11 were exposed to surplus water before reloading in the cell. The previous cell pressure was re-applied and the samples were allowed to reach their equilibrium state. Sample S11 was loaded in a stress controlled undrained test and samples S10 and S9 were loaded in strain controlled undrained tests. These tests were intended to examine some effects of initial swelling on tube samples with different degrees of disturbance.

4.7.5. SECONDARY TESTS

There were some samples in the main testing programme that were loaded without reaching failure, and thus, it was possible, to carry out further tests that are called here, secondary tests. All secondary tests were undrained loading tests on samples that had different stress histories and,therefore,it is necessary to describe the history of each sample. Seconday tests will be quoted by their serial numbers rather than by their test designations and the serial number is that of the original test followed by the letter (n) to specify it as a secondary test.

In tests 9n, 10n and 12n the virgin sample in the original test KF2, the perfect sample in CP2 and the tube sample in KH2, were consolidated following stress path 2 as shown in figure 4.20 before they were loaded to failure and they represent the tests in group 8 as discussed before.

In tests 17n, 19n and 20n, tube samples of increasing disturbance were consolidated anisotropically to a higher stress level of q' = 150 kN/m^2 and p' = 300 kN/m^2 before the samples were loaded to failure in undrained tests. These were part of group 9 tests and they examine the influence of consolidation to higher stress levels on samples with the same degree of mechanical disturbance.

4.7.6. OTHER TESTS

The testing programme included a few tests to examine the repeatability of the experimental procedures and other tests to show some of the effects of the storage period on the behaviour of tube samples. These tests were designated as R or G respectively followed by the serial number of a similar test. Some tube samples were stored in their tubes for a maximum period of two weeks. They were wrapped in polythene, sealed to prevent any loss in the water content and stored at a room temperature of 25 $\pm$ 1^{o}C, and measurements of water content before and after storage showed negligible losses. The stored samples were reloaded and tested in an identical way to the tests specified by their serial numbers.

4.7.7. TESTING PROCEDURES

At the end of the preparation of different samples, they were loaded following different stress paths in drained tests and they were loaded at a constant cell pressure in undrained tests. All drained tests were stress controlled at a rate of approximately 20 kN/m^2 per 12 hours. Undrained tests were either stress controlled at approximately 20kN/m^2 per 12 hours or strain controlled at a rate of .02 mm/min. The rates in drained tests allowed enough time for primary consolidation and those in undrained tests allowed enough time for dissipation of excess pore pressure. Further discussion on the rates of loading is presented in Appendix A3.

CHAPTER 5.

BEHAVIOUR OF VIRGIN SAMPLES

5.1. INTRODUCTION

A basic idea of the present research is that the behaviour of virgin samples will be used as a reference to assess the effects of sampling disturbances and a virgin sample consolidated to the required stress state and tested without subjecting it to stress or mechanical disturbances can be regarded as an undisturbed sample. Hence, it is necessary to examine the behaviour of virgin samples and it is convenient to do so with the concepts of the critical state model. The behaviour of isotropically consolidated virgin samples of remoulded kaolin has been investigated by, for example Loudon (1967) and Balasubramaniuam (1969) but there is limited data on the behaviour of anisotropically consolidated virgin samples. However, there is enough data to show that there is a state boundary surface for anisotropically consolidated samples, but its shape is rather different to that of isotropically consolidated samples.

There are two main purposes to this chapter. Firstly, it is intended to examine the states of anisotropically consolidated samples, to define more clearly the shape of its state boundary surface and the values of the appropriate parameters. Secondly, it is intended to examine the stress-strain behaviour of virgin samples in the context of the mathematical model for soils based on the critical state theory.

The tests which are relevant to this chapter are the drained tests, with different loading paths KF1, KF2, KF3, KF4 and KF5 and stress and strain controlled undrained tests at the same stress level of the drained tests,KF6a and KF6b. In addition, special tests S1, S2, S3, and S4 were ueed to examine some history effects on virgin samples.

5.2. APPLICATIONS OF ELASTICITY AND ELASTO-PLASTICITY TO THE OBSERVED BEHAVIOUR OF SAMPLES

5.2.1. INTRODUCTION

Virgin samples were loaded in drained and undrained triaxial compression tests. In the drained tests,the samples followed different stress paths and in the undrained tests they had different initial states and in considering the behaviour of these samples it is important to specify whether the loading path was on or within the state boundary surface. In theory, there are various differences in the fundamental behaviour of elastic and elasto-plastic materials and some of these differences that may be used to examine the states of the samples are illustrated in this section and used later in this thesis to examine the behaviour of disturbed samples.

5.2.2. THE STATE OF SAMPLES

It was assumed in Chapter 2, that for normally consolidated soils,there is a state boundary surface in (q',p', v) space and soils whose states are on the surface are known as normally consolidated and their behaviour is elasto-plastic.

Soils whose states are below the surface are known as overconsolidated and their behaviour is purely elastic. Thus, to distinguish between elastic and elasto-plastic behaviour, it is necessary to define the appropriate state boundary surface.

The geometry of the state boundary surface was discussed in Chapter 2 and it needs to be defined separately on the wet side and on the dry side of the critical state line,. but for the present investigation only, the wet side is considered. The state boundary surface on the wet side is limited by the critical state line given by:

$$q'_f = Mp'_f \quad \ldots\ldots\ldots \quad 5.1.$$

$$v_f = \Gamma - \lambda \ln p'_f \quad \ldots\ldots\ldots \quad 5.2.$$

and the normally consolidated line given by

$$q' = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 5.3.$$

$$v = N - \lambda \ln p' \quad \ldots\ldots\ldots \quad 5.4.$$

Where M, Γ and λ are regarded as soil constants. The state boundary surface for the family of Cam clay models is given by:

$$q' = \eta p' \quad \ldots\ldots\ldots\ldots\ldots \quad 5.5.$$

$$v = N_\eta - \lambda \ln p' \quad \ldots\ldots\ldots \quad 5.6.$$

Where N_η is the intersection of a normal consolidation line of stress ratio η with the $p' = 1$ psi as shown in figure 5.1. Although there is sufficient evidence to show that the value of λ is unique for a given soil, there is insufficient evidence to verify that the parameter N_η is simply a function of the stress ratio η.

5.2.3. STRAIN PATHS IN DRAINED TESTS

For elastic materials increments of strain are uniquely related to the increment of stress and are independent of the stress state. Thus, as shown by Atkinson (1973a) linear loading paths produce linear strain paths, provided that Poisson's ratio has a constant value. For plastic materials, plastic strain increments are uniquely related to the stress vector through a flow rule and it turns out that linear strain paths are expected only if the loading path happens to intersect the family plastic potentials at points where they have the same slope. Otherwise the plastic strain path and consequently, the overall strain path is non-linear. Thus, the shapes of the strain path may be used to test the state of soils in drained tests as discussed in section 2.7.

It may be observed in figure 5.2., that the virgin samples in tests KF1 and KF2 had non-linear strain paths and thus are assumed to be elasto-plastic. However, the virgin samples along the consolidation path KFO and KF3 where the effective stress path was linear at $q'/p' = 1/2$ had a linear strain path. This linearity of the strain path of an elasto-plastic behaviour may suggest that the loading path was crossing the plastic potentials at the same slope. The strain paths in tests KF4 and KF5 may be approximated to linear paths by ignoring the strain path near failure and hence it may be assumed that the samples. were behaving elastically.

It is intereeting to notice the behaviour of the virgin sample in loading and unloading cycles. Test S2 in figure 5.2. showed linear etrain paths of approximately the same gradient at each loading cycle and hence, it may be assumed, that the sample was elastic in re-loading cycles.

5.2.4. STRESS-STRAIN RELATIONSHIPS

Although both elastic and elasto-plastic stress-strain curves may be assumed non-linear, the non-linearity of the elastic behaviour is usually significantly less than the non-linearity due to elasto-plastic behaviour. Also, the normalised stress-strain relationships of elastic soils are linear whilst that of elasto-plastic soils, in general, are non-linear as discussed before in section 2.7. Alternatively, the relationship between the stress ratio q'/p' and the strain ratio $\delta\varepsilon_s/\delta\varepsilon_v$ in drained tests and the ratio $\delta\varepsilon_s/\alpha$ inundrained tests, may be used to examine the state of the sample.

Figure 5.3. shows $\delta\varepsilon_s/\delta\varepsilon_v$ and $\delta\varepsilon_s/\alpha$ for drained test S2 and undrained test S1 respectively. During the early part of the tests along the curves ab, the ratios $\delta\varepsilon_s/\delta\varepsilon_v$ and $\delta\varepsilon_s/\alpha$ are constants indicating that the behaviour is elastic but during the later part of the test, along the curves bc the ratios $\delta\varepsilon_s/\delta\varepsilon_v$ and $\delta\varepsilon_s/\alpha$ are not constants indicating that the behaviour is elasto-plastic. The point b represents a yield point separating purely elastic behaviour from elasto-plastic behaviour. Similarly, the normalised stress-strain relationships of tests S1 and S2 in figure 5.4 show linear relationships along the early part of the

tests, ab, indicating an elastic behaviour, but during the later part of the tests along the curves bc, the relationships are non-linear, indicating an elasto-plastic behaviour.

5.2.5. STRESS PATHS

In a similar way, the shapes of stress paths in drained and undrained tests may be used to examine the state of the material. The undrained stress path may be used directly to distinguish elastic from ealsto-plastic behaviour as discussed before in section 2.7., but it is necessary to normalise drained stress paths using an appropriate normalising parameter as discussed before in section 2.7.

Figure 5.5 shows the normalised undrained stress path for test S1 and the normalised drained stress path for test S2. The behaviour along ab is assumed to be elastic and, in theory,for isotorpic materials,normalised stress paths appear as vertical lines in undrained tests and of some slope in drained tests. The behaviour along bc is non-linear and elasto-plastic and hence indicates the state of the sample as elasto-plastic.

5.2.6. STATE BOUNDARY SURFACE

Equations of yield curves and state boundary surfaces were reviewed in Chapter 2 and, as discussed in section 2.5.4, stress paths of normally consolidated soils are bound to follow the intersection of the state boundary surface with the loading path.

Therefore, different stress paths are expected when samples are tested under different levels and loading conditions, as indicated in figure 5.6., but if the material had a unique state boundary surface $F(q', p', v)$ these different stress paths may be normalised to a unique stress path $H(q'/x, p'/x)$ where x is a stress parameter related to the specific volume. Two normalising parameters p'_{fp} and p'_e were introduced in section 2.7.3. and will be used to examine the uniquence and the shape of the state boundary surface.

Figure 5.7. shows the stress path of tests KF1, KF6a, and KF6b normalised by p'_{fp} and p'_e. It may be observed that all these normalised stress paths approximate to unique curves, using either the parameter p'_{fp} or the parameter p'_e and hence, all tests suggest the existence of a unique surface for virgin samples. The $q'/p'_e - p'/p'_e$ curve represents a dimensionless intersection of the constant volume plane, with the state boundary surface, while $q'/p'_{fp} - p'/p'_{fp}$ curve represents a dimensionless intersection of the elastic wall with the state boundary surface.

The results in this section confirm the existence of a state boundary surface for anisotropically consolidated samples. The behaviour of samples was elastic for all states within the state boundary surface and the behaviour was elasto-plastic for all states on the state boundary surface.

5.3. THE BEHAVIOUR OF ANISOTROPICALLY CONSOLIDATED KAOLIN

5.3.1. THE CONSOLIDATION BEHAVIOUR

All virgin samples were consolidated anisotropically following reconsolidation path K, as described in Chapter 4 and the consolidation behaviour of three virgin samples is shown in figure 5.8. and it confirms the repeatability of loading procedures. The consolidation behaviour is replotted in figure 5.9. in terms of v - ln p' and scaling from the figure $\lambda = 0.14$, $k = 0.04$ and $N = 2.458$.

In theory, soils have a constant compression index λ at virgin states and a constant recompression index **k** below the virgin state. Lewin (1970) reported that values of λ may increase slightly at higher stress levels and that values of k suffer wider variations than λ. Nevertheless, the above values of λ, k, and N were adopted for further analysis.

Samples KF3 and KF2 were loaded after 24 hours rest period and both samples showed values of $\lambda = 0.16$ and 0.13 respectively, which are slightly different than the average value of 0.14. The strain path of tests KFO and KF3 in figure 5.8. shows that $\varepsilon_v = \varepsilon_a$ indicating that the consolidation path was very close to one-dimensional consolidation.

5.3.2. THE CRITICAL STATE LINE

The effective stresses and specific volumes at the critical state are given by

$$M = q'_f/p'_f$$

and

$$v_f = \Gamma - \lambda \ln p'_f$$

Where M, Γ and λ are regarded as soil constants.

Figure 5.10a shows the stresses at failure of virgin samples in q' - p' plane. They fall close to a line given by equation 5.1. and scaling from the diagram M = 0.95. The stresses at failure in stress controlled tests KF1, KF4 and KF5, are those within the final stress increment that caused failure and hence, they are not exact measurements of the stresses at the critical state line.

Similarly, figure 5.10b, shows the critical state line in v - ln p' plane. They all fall close to a line given by equation 5.2. and by scaling from the diagram, λ = 0.14 and Γ = 2.429. Some samples such as KF4 and KF5 had specific volumes at failure, rather lower than the average critical state line. These differences are probably due to the measurement of the overall specific volume at failure, rather than that at the failure plane.

Apparently, at the failure plane local swelling takes place in drained and undrained tests and the failing soil may have a specific volume rather greater than the average of the whole sample.

5.3.3. THE STATE BOUNDARY SURFACE ON THE WET SIDE OF THE CRITICAL STATE LINE

Following Atkinson and Bransby (1978), the state boundary surface is defined by the Hvorslev surface on the dry side aud the Roscoe surface on the wet side of the critical state line. In order to examine the complete state boundary surface, it is necessary to conduct compression and extension tests on normally consolidated and over-consolidated soils.

The present investigation provided data to examine the state boundary surface on the wet side between the consolidation path and the critical state line, for tri-axial compression.

Stress paths in tests KF1, KF6a, and KF6b were normalised by p'_e and p'_{fp} and these give normalised stress path and a normalised yield curve as shown in figure 5.7. All the experimental points fall close to one or other curve, indicating that there is a state boundary surface for which the shape is independent of loading conditions or loading paths, at least for the range of tests that were conducted.

The yield curve defined as the intersection of an elastic wall with the state boundary surface, is compared with that of simple Cam clay, modified Cam clay and that proposed by Stipho(1978), in figure 5.11. All the curves in figure 5.11 were plotted to represent the shapes of different yield curves and it was assumed that all yield curves may have the same state at failure but not necessarily the same initial state on the consolidation curve. That implies that all curves had the same values of Γ and λ but different values of N. It may be observed in figure 5.11, that the yield curve of virgin samples is best approximated by Stipho's(1978) equation which gives an ellipse having its apex at the consolidation path rather than at the p' axis, as proposed by Roscoe and Burland (1968).

It may be interesting to observe the normalised stress paths in drained tests KF1, KF2, KF3, KF4 and KF5, as shown in figure 5.12. The stress paths for tests KF1 and KF2, both remain on the state boundary surface and they are approximately symmetrical about the consolidation axis, indicating that the consolidation path may be an axis of symmetry and hence, the yield curve below $q'/p' = 0.5$ may be similar to that above. Further tests are necessary to confirm the suggestion.

Stress paths KF4 and KF5 represent unloading and the states move inside the state boundary surface and move on elastic walls; the stress paths are approximately linear, indicating elastic behaviour. The stress path for test KF3 represents an anisotropic consolidation and it should, in theory, be represented by a single point at the intersection of the state boundary surface with the line $q'/p' = 0.5$;

the slight differences are attributable to taking $\lambda = 0.14$ to calculate p'_e which may not exactly represent the value for KF3 after 24 hours rest period in that particular test.

5.3.4. THE FLOW RULE

A flow rule can be thought of as a material property and hence it should hold disregarding loading conditions. For an associated flow rule the plastic strain increment vector is normal to the yield curve and to examine the flow rule it is necessary to define the yield curve and to calculate the plastic strains. The yield curve is defined as the intersection of an elastic wall with the state boundary surface and the normalised stress path $q'/p'_{fp} - p'/p'_{fp}$ defines the shape of the yield curve. It should be noticed that the normalised stress path $q'/p'_e - p'/p'_e$ defines a unique path similar in shape to that of constant volume and this is not the same thing as a yield curve.

In order to examine the flow rule, the stress ratio q'/p' may be plotted against the plastic strain increment ratio $\dot{\varepsilon}^p_s/\dot{\varepsilon}^p_v$ and for an associated flow rule the above relationship will be identical to that obtained from plotting the stress ratio against the slope of the vector normal to the yield curve. It is possible to scale the slope of the vector normal to the yield curve from figure 5.7 and to plot it against the stress ratio as shown in figure 5.13.

The plastic strain increment ratio may be calculated from both drained test KF1 and undrained test KF6a and the results are plotted against the stress ratio as shown in figure 5.13. The differences in the relationship in drained test KF1 and undrained test KF6a are well beyond errors and any possible variation in the value of k would have had little effect on the calculated values of the plastic strain increment ratio. Similar differences were reported before on the behaviour of kaolin in drained and undrained tests (e.g. Roscoe, Schofield and Thurairijah 1963) and this led to further assumptions to account for the high distortional strain in undrained tests (e.g. Burland and Roscoe 1968). However, for the present discussion, it is assumed that the relationship is the same in both drained and undrained tests and an average curve may be plotted as shown in the figure 5.13. It may be observed in figure 5.13 that the relationship between the stress ratio and the palstic strain increment ratio is different from the relationship between the stress ratio and the slope of the vector normal to the yield curve and hence,it may be concluded that the flow rule is non-associated.

In order to define a flow rule, it is assumed that the relationship between the stress ratio q'/p' and the plastic strain increment ratio $\dot{\varepsilon}^p_s / \dot{\varepsilon}^p_v$ is linear and may be defined as

$$R = C - \eta \tan \psi \quad \ldots\ldots\ 5.7.$$

Where R is the plastic strain increment ratio, $\dot{\varepsilon}^p_s / \dot{\varepsilon}^p_v$, η is the stress ratio q'/p', and ψ and C are constants. The values of ψ and C may be scaled from figure 5.13. as 36^o and 0.7 respectively.

5.3.5. DISCUSSION

Some aspects of the critical state model were examined in this section and the results are consistent with the existence of a critical state line and a state boundary surface for virgin samples. The shape of the yield curve was similar to that of an ellipse with its apex along the consolidation path. The compression and swelling curves were similar to those predicted by simple Cam clay but the relationship between the stress ratio and the plastic strain increment ratio indicated that the flow rule was non-associated.

5.4. OTHER ASPECTS OF THE BEHAVIOUR OF THE VIRGIN SAMPLES

5.4.1. INTRODUCTION

Although this research was not intended specifically to examine different aspects of the behaviour of virgin samples and the validity of the critical state model in predicting the behaviour of soils, but the results allow an examination of some history effects on the behaviour of virgin samples, since stress disturbance is simply a history effect.

Different stresshistories were imposed on four virgin samples and the consequent behaviour is examined in view of elastic and elasto-plastic concepts. They included the behaviour of lightly overconsolidated sample S1, a sample disturbed by cycling loading S2, a virgin sample at different stress state 9nb and virgin samples with slightly different stress histories, S3 and S4.

5.4.2. LIGHTLY OVERCONSOLIDATED SAMPLE

The behaviour of lightly overconsolidated samples of kaolin prepared by isotropic and anisotropic consolidation and swelling was examined by for example, Loudon (1967) and Stipho(1978). In theory, the behaviour of these tests may be predicted by the critical state models but for reference and for comparisons with disturbed samples, tests were carried out to observe the behaviour of lightly overconsolidated samples.

Sample S1 was anisotropically consolidated and swelled to $q' = 105$ kN/m^2, $p' = 210$ kN/m^2 and an overconsolidation ratio of 1.1 and loaded to failure in an undrained test and figure 5.15. shows the stress-strain curve $\Delta q'/p'$ vs. $\Delta v \varepsilon_s$'/. normalised as described in section 2.7. The point b in both figures 5.14 and 5.15 is taken as the yield point at which the state first reaches the state boundary surface, the yield point is rather better defined in the stress-strain curve in figure 5.15 than it is in the stress path shown in figure 5.14. The portion ab in both figures represent an elastic behaviour where the state is on an elastic wall and the position bc represents elasto-plastic behaviour where the state is on the state boundary surface.

The behaviour of the lightly overconsolidated sample was in agreement with the concepts of the Cam clay model.

5.4.3. DEPENDENCY ON THE STATE BOUNDARY SURFACE ON THE PREVIOUS LOADING PATH

In theory, the simple Cam clay and modified Cam clay models predict only the effects of overconsolidation and they do not include other stress history effects such as the loading path and cycling effects on the state boundary surface and the plastic potential. Whether it is necessary or not to include history effects in theoretical predictions, depends on the significance of these effects on the overall behaviour.

Tests S3 and S4 were intended specifically to examine two aspects of path dependency in virgin samples. The two samples were loaded from state A to state B, following different paths as shown in figure 5.16 and were tested at B along the same loading path. The total strains caused by loading the sample from state A to state B were ε_s = 1.5% and ε_v = 1.2% in test S3 and ε_s = .7% and ε_v = 2.2% in test S4 and the results suggest that the behaviour was path dependent. It is interesting to notice that the distortional strains along path S3 were approximately 1.3 times the volumetric strains along the same path, while the volumetric strains along path S4 were approximately 3 times the distortional strains along the same path.

This does not fall in agreement with predictions of strains that are associated with a symmetrical yield curve and further tests are necessary to confirm the above observations.

Firugre 5.17 shows the average plastic strain increment vector of samples S3, S4 and KF3 and the differences may be interpreted as due to the effects of the stress history on the plastic potential. It seems that different stress histories over short loading paths has slight effect on the state boundary surface, but that over long loading paths may have more significant effects and further exploration of history effects on the etate boundary surface is beyond this investigation.

5.4.4. SOME EFFECTS OF DRAINED CYCLIC LOADING

Previous observations on cyclic effects were reviewed in section 3.4 and the problem may be considered as one of history effects. Stress disturbance is the effect of one cycle of undrained unloading of the total stresses followed by either drained or undrained reloading in the laboratory and hence, stress disturbance may be examined in the context of cycles of unloading and reloading.

Test S2 was designed to examine the effects of 4 cycles of undrained unloading followed by drained relaoding of the shear stress as shown in figure 5.18. The strain path $\varepsilon_a - \varepsilon_v$ in figure 4.28 is approximately the same in each cycle and it may be assumed linear and hence, the strain path may suggest an elastic behaviour.

The behaviour of the virgin sample in the first and fourth reloading cycle is shown in figure 5.19., together with the behaviour of a perfect sample in a similar reloading path. However, the initial behaviour of the perfect sample in a similar reloading path showed lower stiffness and a higher distortional strain as shown in figure 5.19. and these differences are attributed to the effects of sampling disturbance. The normalised stress path of the virgin sample in its fourth cycle is shown in figure 5.20 together with that of a virgin sample in test KF1. It may be observed that the initial behaviour of the virgin sample S2 along ab was within an elastic wall while that along bc was on its state boundary surface. Cam clay models do not predict the effects of cyclic loading on the state boundary surface. Although the virgin sample in tests S2 showed an elastic behaviour followed by an elasto-plastic behaviour, the shape of the yield curve of the virgin sample in test S2 seems to have been distorted or rotated due to cyclic loading and further tests are necessary to confirm the above observation.

5.4.5. VIRGIN SAMPLES WITH ADDITIONAL ISOTROPIC COMPRESSION

The yield curves in the modified Cam clay and Stipho's (1978) model were assumed to be ellipses having their apices on the isotropic and anisotropic consolidation paths respectively. In the modified Cam clay model, it is assumed that yielding causes transilation and expansion of the yield curve along the p' axis such that the ellipse always passes through the origin as shown in figure 5.21 while in Stipho's (1978) model the yield curve is assumed to expand, translate and rotate following the loading path as shown in figure 5.22 and it is interesting to examine the effects of increments of isotropic compression on the yield curve.

A virgin sample in test KF2 was loaded from q' = 105 kN/m^2 and p' = 210kN/m^2 to q' = 105kN/m^2 and p' = 290 kN/m^2 and the sample was then loaded to failure in undrained conditions in test 9nb. Figure 5.23 shows the normalised stress path abc plotted together with the normalised stress-strain curve. While it is difficult to draw any conclusions from a single test, the stress path in figure 5.23 may suggest that the yield curve has rotated, expanded and translated from b - c to b - d and further obeervations by other workers (e.g. Balasubramaniam 1969) suggest that long loading paths influence the shape of the state boundary surface. The stress-strain curve shown in figure 5.24. shows a yield point at b and this point is also shown in figure 5.23. It may be assumed that the behaviour along ab is elastic and along bd it is elasto-plastic. The initial elastic behaviour of the virgin sample in test 9nb is not in agreement with predictions of Cam clay models of an initial elasto-plastic behaviour.

5.4.6. DISCUSSION

The tests presented in this section were not enough to establish the history effects on the behaviour of virgin samples, but they may be used to indicate the significance of such effects.

The simple Cam clay theory accounts for the initial elastic behaviour of lightly overconsolidated samples, but it does not account for the effects of different consolidation paths and different loading paths on the shape of the state boundary surface. The tests showed that stress-strain relationships and the shape of the yield curve were influenced by loading paths but it may be assumed that for relatively short loading paths, such influences are negligible. The simple Cam clay theory assumes elastic behaviour for states below the state boundary surface and elasto-plastic behaviour for states on it. The test results confirmed that the behaviour was elastic for states well below the state boundary surface and the behaviour was elasto-plastic for states on it but was difficult to describe the behaviour simply as being elastic for states near the state boundary surface and to separate clearly the elastic from elasto-plastic regions near the state boundary.

CHAPTER 6 BEHAVIOUR OF PERFECT SAMPLES

6.1. INTRODUCTION

The behaviour of perfect samples may be examined within the framework of the critical state model and the effects of stress disturbance assessed qualitatively and quantitatively within a reference to the behaviour of virgin samples. It is assumed here, that stress disturbance does not alter the basic concepts of elastic and plastic behaviour but it may alter the values of some parameters and distort the shape of the state boundary surface. The purpose of this chapter is to examine the fundamental behaviour of perfect samples and to assess the effects of stress disturbance.

6.2. PREDICTION OF THE BEHAVIOUR OF PERFECT SAMPLES

6.2.1. INTRODUCTION

Stress disturbance was analysed in section 2.6. within the framework of the Cam clay model. The theory specifies that the behaviour during undrained unloading is elastic and the model predicts that there will be no disturbance if the sample is reloaded to its previous stress state. The purpose of this section is to predict the behaviour of perfect and virgin samples at different stress levels within the framework of the critical state model, although it was observed in section 5.4 that irrecoverable strains take place during cyclic loading and the yield curve was path dependent and hence, undrained unloading may lead to the sample being in an overconsolidated state, with a distorted yield curve.

6.2.2 INITIAL STATES OF PERFECT SAMPLES IN DIFFERENT TESTS

Figure 6.1a. illustrates the states of virgin and perfect samples before and after reconsolidation. For a perfectly elastic soil, the path of perfect sampling is VEV and the states of virgin and perfect samples are the same at V. For an imperfectly elastic soil, the path of perfect sampling may be VP_1U_1 and the state at U_1 is below the state boundary surface and on an elastic wall. Similarly, if the yield curve of the virgin sample was of the shape shown in figure 6.1b. then the path of perfect sampling may be of VP_2U_2 and the etate at U_2 is below the state boundary surface, but that at P_2 is at the state boundary surface. In both cases, the soil may be reconsolidated to R1 where it is still below the state boundary surface or to R2 where it is again on the state boundary surface.

The initial effective stress states and water contents of perfect samples as measured in different laboratory tests are shown in table 6.1., together with the values predicted by elastic theory. It was not possible to measure accurately the negative pore pressures under zero total stresses, due to the possibility of cavitation in the pore water at large negative pressures. The effective pressure of perfect samples at their previous cell pressure was of the order of 20% less than the values predicted by elastic theory. This may be interpreted as being due to loss of suction, or inelastic behaviour during unloading or to a combination of these effects.

It is important,however, to notice that the change in effective normal pressure during reloading to the previous stress state was found to be equal to the value predicted by elastic theory. The final effective pressure after undrained reloading to the previous total stresses was therefore below the previous effective stress. When the samples were allowed to drain to a back pressure equal to the previous pore pressure,there was a volume change of approximately 1% as the perfect samples recovered their initial effective stresses but at lower specific volumes.

Since the effective stress state after perfect sampling is different to that of a virgin sample, the question arises as to whether it is correct to test the sample at this stress state and at the same initial specific volume,or whether it is correct to reconsolidate the perfect sample to the previous effective stresses of the virgin state or to a higher stress level in which the specific volume will be different from the initial value. If the sample is reconsolidated to a higher stress level, it is possible,in theory, to recover the behaviour of the virgin sample simply by a suitable normalising procedure.

6.2.3. THE PREDICTED BEHAVIOUR OF UNCONSOLIDATED AND RECONSOLIDATED PERFECT SAMPLES

As previously argued, in section 2.6., the Cam clay model predicts that there should be no change in the state of a virgin sample during undrained unloading and reloading, and hence, there should be no need for reconsolidation. However, as shown in the previous section, perfect samples had a lower initial effective stress before reconsolidation or lower initial specific volumes after reconsolidation to the previous or higher stress level.

Assessments of the effects of such changes on the behaviour may be approached in two ways. Firstly, it may be assumed that the state boundary surface does not alter during unloading and reloading, but only the state alters and it follows that perfect samples reconsolidated to their previous stress state have initial states below the state boundary surface. In this case, perfect samples behave as lightly overconsolidated samples and preserve the virgin state boundary surface as shown in figure 6.1c. Secondly, it may be assumed, that both the state and the state boundary surface are altered and hence, a perfect sample at or below its previous stress state will behave as a lightly overconsolidated sample with a state boundary surface different from that of a virgin sample. For a sample which is reconsolidated to a stress level higher than the virgin stress level, the effect of the stress disturbance will be diminished and if it is reconsolidated to a substantially higher stress level, the virgin state boundary surface will be, in theory, recovered. In this case, theoretically, it is possible to obtain the orginal virgin behaviour by a suitable normalising procedure.

The behaviour of perfect samples may be normalised by the parameters introduced in section 2.7 and the same constants, Γ and N, of the virgin sample may be adopted. If the behaviour of a perfect sample show initial elastic behaviour followed by elasto-plastic behaviour similar to that of a virgin sample, then the perfect sample may be thought of as lightly over-consolidated and the effects of stress disturbance were to alter the state of the sample only. However, if the fundamental behaviour of a reconsolidated perfect sample shows initial elasto-plastic behaviour, then the perfect sample may be considered as a virgin sample, but with different state boundary surface and the effects of stress disturbance were to alter the values of some of the soil constants. Alternatively, if the perfect sample showed initial elastic behaviour, followed by elasto-plastic behaviour, which was not identical to that of a virgin sample, then the effects of stress disturbance were to alter both the state and the state boundary surface, and the perfect sample may be considered as a lightly overconsolidated sample with its own state boundary surface.

6.2.4. THE BEHAVIOUR OF PERFECT SAMPLES IN THE LABORATORY

In order to examine either of the previous assumptions, it is necessary to examine the fundamental behaviour of unconsolidated and reconsolidated perfect samples. As discussed earlier in section 5.2., different aspects of the observed behaviour, such as the relationship between the stress ratio and the strain ratio, the stress-strain relationship and the stress path, may be used to examine the fundamental behaviour. The behaviour of virgin and lightly overconsolidated samples introduced in chapter 5. as tests KF6a and S1 repectively, may be used as references for comparisons.

Figure 6.2., shows the rleationship between the stress ratio and the ratios, $\delta\varepsilon_s/\delta\varepsilon_v$ or $\delta\varepsilon_s/\alpha$, for tests KF6a, S1, S5 P7b, CP1 and CP6a. The perfect sample in test P7b was loaded to failure in an unconsolidated undrained test, while those in tests CP6a and CP1 were reconsolidated to their previous stresses and loaded to failure in undrained and drained tests respectively. It was previously argued that an elastic behaviour is associated with a constant ratio $\delta\varepsilon_s/\delta\varepsilon_v$ or $\delta\varepsilon_s/\alpha$ and an elasto-plastic behaviour is associated with varying ratios of $\delta\varepsilon_s/\delta\varepsilon_v$ or $\delta\varepsilon_s/\alpha$. Although it is difficult to trace an initial constant ratio in figure 6.2., for the lightly overconsolidated sample S1, it is possible to see that the initial value of $\delta\varepsilon_s/\alpha$ ratio in test P7b was constant, shown as ab in figure 6.2. Similarly, an initial constant strain ratio may be assumed for the perfect samples in tests CP1 and CP6a, shown aslo as ab in figure 6.2.

Hence, it may be assumed, that the perfect samples showed an initial elastic behaviour.

Stress-strain relationships of reconsolidated perfect samples in drained and undrained tests, CP1 and CP6a, respectively, are shown in figure 6.3. and it is difficult to distinguish elastic from elasto-plastic behaviour. Figure 6.4. shows the stress-strain relationships of reconsolidated perfect samples in drained tests CP2 and CP3 and, the volumetric stiffness in those tests leads to values of $\lambda = 0.1$, which is less than $\lambda = 0.14$ and more than $k = 0.03$ found for virgin samples in section 5.2.

In order to examine the behaviour of reconsolidated perfect samples, it is interesting to compare it with that of a lightly overconsolidated virgin sample, which had the same state. Figure 6.5. shows the stress path of a normally consolidated virgin sample KF6a, a lightly overconsolidated virgin sample S1 and a reconsolidated perfect sample CP6a and it may be observed that the stress path of the perfect sample was different from that of a lightly overconsolidated sample at the same stress state. Although the state of the reconsolidated perfect sample is below the state boundary surface of the virgin sample, it appears as if the perfect sample has a new state boundary surface assoicated with it.

The stress path of the perfect sample in the unconsolidated undrained test P7b is linear and hence, indicates an initial elastic behaviour as shown in figure 6.5., and this confirms that the state of the sample was below the state boundary surface. Clearly, if the sample is reconsolidated to a substantially higher stress state than that of a virgin sample, then the virgin behaviour and the virgin state boundary surface will be recovered.

6.2.5. DISCUSSION

The behaviour of soils before failure may be controlled by one of the two fundamental concepts of elasticity and plasticity and the current state of the sample will determine whether elastic or elasto-plastic concepts will govern the behaviour. There is no reason to assume that stress disturbance will alter the applicability of these fundamental concepts. Thus, it may be possible to consider stress disturbance as an alteration of the state of the sample or as an alteration of some of the parameters that describe the behaviour or both effects may occur together. If stress disturbance only alters the state, then the normalised behaviour of virgin and perfect samples will be identical over their elastic or elasto-plastic ranges. However, it was found that the behaviour of reconsolidated perfect samples over its elasto-plastic range suggests that the geometry of the state boundary surface has been altered by perfect sampling. The question now is to investigate the size and the shape of the new state boundary surface.

6.3. THE BEHAVIOUR OF RECONSOLIDATED PERFECT SAMPLES

6.3.1. INTRODUCTION

The results in the previous section suggest that reconsolidated perfect samples may be thought of as with a state boundary surface, which is different to the virgin state boundary surface and hence, the values of the basic soil parameters N, M, Γ, λ, and k may be different. In this section, the behaviour of reconsolidated perfect samples will be normalised and compared to that of a virgin sample to assess the effects of the stress disturbance on the state, state boundary surface and stress-strain relationships.

6.3.2. THE CRITICAL STATE LINE OF PERFECT SAMPLES

The states of seven perfect samples (CP1, CP6a, CP6b, CP9b, P7b, CP4 and CP5) that were reloaded to failure may be used to construct a critical state line following the procedure described in section 5.3.2. and the data are shown in figure 6.6. The parameter M scaled from figure 6.6a was within the range of values observed in tests on virgin samples, but the data shown in figure 6.6b show that a critical state line may be plotted parallel to that for virgin samples, but slightly shifted from it and scaling from the diagram gives Γ = 2.414 and λ = 0..14. The cause of the apparent shift of the critical state line may be interpreted as being due to stress disturbance.

Sample CP5 departed from the critical state line in a similar way to that of sample KF5 and this was interpreted in section 5.3.2. as being due to differences between the overall water content and that in the failure plane at the end of the test.

The results indicate that the stress disturbance altered the values of Γ from 2.429 to 2.414 but it had no measurable effect on λ and M .

6.3.3. THE STATE BOUNDARY SURFACE ON THE WET SIDE OF THE CRITICAL STATE LINE

Normalised stress paths for normally consolidated and lightly overconsolidated samples were used to establish the state boundary surface of virgin samples in section 5.3. Similarly, the normalised stress paths of reconsolidated perfect samples in five drained tests (CP1 to CP5) and three undrained tests (CP6a, CP6b and CP9b) may be used to examine the state boundary surface of perfect samples. As discussed in section 2.7.3., suitable parameters to normalise stress paths are p'_e and p'_{fp} given by equations 2.43 and 2.44 a. respectively.

A perfect sample has a value of λ slightly lower than that of a virgin sample, because of a gradual change in value from k to λ as an elastic wall approaches the virgin normal consolidation line, but for the time being, it may be assumed that the normal consolidation line for virgin and perfect samples is the same and λ = 0.14

and N = 2.455 for both cases.

The normalised stress path of reconsolidated perfect samples in drained tests CP1 to CP5 are shown in figure 6.7. It seems that the stress paths of the perfect samples in tests CP4 and CP5 were crossing elastic walls. Obviously, the normalised stress paths in tests CP2 and CP3 indicated an elastic behaviour since the stiffness of the perfect sample, $\lambda = 0.11$, was less than the assumed value of $\lambda = 0.14$. The loading paths and initial stress states of perfect samples in tests CP1 to CP5 were similar to those of virgin samples in tests KF1 to KF5. Thus, by comparing figure 6.7 and figure 5.12, it is possible to suggest that the stress disturbance may alter the state of the sample and the shape of the state boundary surface but it does not alter the applicability of concepts of the critical state model to the behaviour of the perfect sample.

Alternatively, it is possible to assume that a perfect sample reconsolidated to its stress state before sampling is a normally consolidated sample with N=2.429 and $\lambda = 0.14$. Figure 6.8a shows two sets of normalised stress paths of reconsolidated perfect samples in tests CP1, CP6a, CP6b, and CP9b. Assuming that N and λ preserve their virgin values, then the normalised stress path of all the above tests fall close to curve aa. However, if it is assumed that N has been reduced in value to 2.429 then the normalised stress path of all the previous tests fall close to curve bb.

The results based on either assumption lead to a unique curve that represents the intersectiou of a constant volume plane with the state boundary surface.

It is difficult at this stage, to confirm the existence of an initial elastic behaviour or an initial over-consolidated state for reconsolidated perfect samples. The normalised stress paths of virgin normally consolidated and lightly overconsolidated samples are compared to that of a perfect sample in figure 6.8b., and it may be observed that the behaviour of the perfect sample was close to that of lightly overconsolidated sample and its state boundary surface was slightly different from that of a virgin sample.

Figure 6.9. shows the reference curve q'/p' vs. N_η, where N_η is the specific volume at the intersection of a normally consolidated line of q'/p' stress ratio with the constant p' section at p' = 1 as shown in Figure 5.1, for virgin and perfect samples. It may be seen in figure 6.9, that an effect of the stress disturbance was to shift the reference curve from aa to bb and hence, distort the state boundary surface. The state boundary surface of the perfect sample was similar in shape to that of a lightly overconsolidated virgin sample in test S1, but in test S1 it had a different value for N.

The normalised stress paths of reconsolidated perfect samples suggest that they are lightly overconsolidated but with different values for the constants N and Γ and that stress disturbance slightly alters the shape of the state boundary surface. This may be thought of as a history effect which can only be predicted by kinematic elasto-plastic models, such as those introduced in section 2.5.5.

6.3.4. THE YIELD CURVE AND THE FLOW RULE

The parameter p'_{fp} was used to normalise stress paths of virgin samples in section 5.3.3. giving yield curves. Similarly, yield curves of perfect samples may be obtained using p'_{fp} calculated from equation 2.44 a, taking $\Gamma = 2.414$, $\lambda = 0.14$ and $k = 0.05$. Yield curves of perfect and virgin samples are shown in figure 6.10 and it may be observed that the yield curve for perfect samples is different to that of a virgin sample. The effects of stress disturbance seemed to be to rotate the yield curve.

Figure 6.11 shows the relationship between the plastic strain increment ratio $\dot{\varepsilon}_v^p/\dot{\varepsilon}_s^p$ and the stress ratio q'/p' for perfect samples in drained tests and undrained tests compared to those predicted from the yield curve for an assoicated flow rule. The results show that the relationship between the plastic strain increment ratio and the stress ratio is different to that predicted from the yield curve and hence, the flow rule is non-associated as was found for virgin samples and discussed in section 5.3.4. The relationship between plastic strain rate and stress ratio for perfect samples shown in figure 6.11 may be compared to that for virgin samples shown in figure 5.13. It may be observed, that the perfect sample had lower strain ratio than that in virgin samples and this implies that the perfect sample had higher distoritional strains or lower volumetric strains or both.

The flow rule in test CP1 showed higher strain rates than that in undrained test CP6a and similar differences in the strain rates in drained and undrained tests were reported in tests on virgin samples in section 5.3.4.

6.3.5. DISCUSSION

In the Cam clay model it is assumed that the behaviour below the state boundary surface is elastic and the model predicts that the state of a perfect sample at its previous stress state is identical to that of a virgin sample. Furthermore, it assumes isotropic hardening and consequently, a unique state boundary surface which is history independent.

It was found that the state of a reconsolidated perfect sample was altered and that the behaviour was not the same as that of a lightly overconsolidated sample. The new state boundary surface for perfect samples, may be assumed to be similar in shape to that of a virgin sample but slightly rotated and shifted.

6.4. THE BEHAVIOUR OF PERFECT SAMPLES WITHIN THE STATE BOUNDARY SURFACE

6.4.1. INTRODUCTION

As observed earlier in this chapter, some perfect samples were loaded within the state boundary surface and, hence, their behaviour may be examined in the context of elastic theory. These were tests on unloaded and reloaded perfect samples in undrained conditions (P7b and S5 respectively), tests on reconsolidated perfect samples where the loading path was directed inside the state boundary surface (CP4 and CP5) and the reconsolidation behaviour of perfect samples. The elastic behaviour of perfect samples will be examined with reference to similar tests on normally consolidated virgin samples, where the loading path was directed inside the state boundary surfaces and tests on overconsolidated samples. The values of some elastic parameters are deduced and used later to assess the effects of stress disturbance.

6.4.2. THE BEHAVIOUR OF PERFECT SAMPLES IN UNCONSOLIDATED UNDRAINED TESTS

Tests P7b and S5 were examined in section 6.2. and analysis of figure 6.2. suggested that the behaviour of these perfect samples was initially elastic before becoming elasto-plastic.

The normalised stress-strain relationships of perfect samples are shown in figure 6.12 but it is difficult from these data to distinguish between elastic and elasto-plastic behaviour. It is assumed for the time being, that the initial behaviour is elastic then it is relevant to scale the elastic stiffness moduli from the stress-strain curves in figure 6.12. The undrained elastic stiffness, E_u', is assumed to be equivalent to the secant moduli at a stress level which is of 50% of the stress increment.

Similarly, E_u' ofsamples S1 and P7b are scaled from figure 6.12 and the results are shown in table 6.2. together with the initial stresses and stresses at failure. It may be observed, that the effects of stress disturbance was to decrease the initial effective pressure p' and the undrained elastic stiffness E_u'. The normalised stress paths of these tests are shown in figure 6.13, showing an initial linear path and, hence, it may be suggested that perfect samples P7b and S5 had an initial elastic behaviour. However, it seems that the perfect sample in test P7b met the state boundary surface on or very near the Hvorslev surface, while the perfect sample in test S5 showed a non-linear stress path, shown as bc in figure 6.13, and it indicates that the loading path met the state boundary surface on the Roscoe surface. The normalised stress paths of tests P7b and S5, showed no distinctive change from elastic to elasto-plastic behaviour and point b in figure 6.13 represents approximately the end of the elastic range.

The rate of loading affects the developed pore pressure and, hence, undrained tests with higher rates of loading are likely to produce stress paths with higher slopes.

6.4.3. THE RECONSOLIDATION BEHAVIOUR OF PERFECT SAMPLES

Before reloading perfect samples, the initial state was $q' = 0$, $p' = 210$ kN m^{-2} and $v = 1.978$ and, hence, they were assumed to be within the state boundary surface. Some perfect samples were reconsolidated to their previous stress state at $q' = 105$ kNm^{-2} and $p' = 210$ kNm^{-2} following reconsolidation paths K or C and, hence, their behaviour during reconsolidation may be used to examine the elastic properties of perfect samples.

Figure 6.14a shows the reload ing paths C,K and the loading path in the first reloading cycle in test S2. The stress-strain relationships and strain paths of perfect samples CP and KP are shown in figure 6.14b together with that of the virgin sample in test S2. Both perfect and virgin samples during reloading showed an initial linear relationship in figure 6.14b and, hence, it may be assumed, that both virgin and perfect samples showed an initial elastic behaviour during reloading. The perfect sample in test CP showed a lower deviatoric stiffness and approximately the same volumetric stiffness of that of the virgin sample in a similar relaoding path in test S2.

This may suggest that the stress disturbance had caused a lower deviatoric stiffness (parameter A in equation 2.45) but did not alter the volumetric stiffness (parameter C in equation 2.46). The behaviour of the perfect sample in reconsolidation path C showed a slightly lower value of volumetric stiffness than that in reconsolidation path κ and this may be interpreted as the gradual change from k to λ in path K. In theory different loading paths will have the same normalised stress-strain relationships ($\Delta q'/vp'$ vs. $v.\Delta\varepsilon_s$ and $\Delta p'/vp'$ vs. $v.\Delta\varepsilon_v$) for isotropic materials and, hence, the differences may be attributed to anisotropy.

6.4.4. THE ELASTIC BEHAVIOUR OF RECONSOLIDATED PERFECT SAMPLES

As observed in section 5.3 and 6.3, the loading paths in tests KF4, KF5, CP4 and CP5 were within the new state boundary surface and it was suggested that their behaviour was elastic. However, the strain paths in tests CP4 and CP5 were non-linear as shown in figure 6.15a and the relationship between the stress ratios and the strain ratios in those tests in figure 6.15b. do not show a constant strain ratio and, hence, they suggest that the behaviour was elasto-plastic. Nevertheless, the effects of stress disturbance may be examined by comparing the behaviour of virgin and perfect samples in similar tests.

Figure 6.16 shows that the perfect sample in tests CP4 and CP5 had lower deviatoric and volumetric stiffnesses respectively, than that of the virgin sample in tests KF4 and KF5 and the value for the parameter k was calculated as 0.04 and 0.03 for perfect and virgin samples respectively. The strain paths in both tests confirm that the perfect sample had a lower deviatoric stiffness than the virgin sample as shown in figure 6.15a.

6.4.5. SOME EFFECTS OF STRESS HISTORY WITHIN THE STATE BOUNDARY SURFACE

In theory, the behaviour of soils within the state boundary surface is elastic and, therefore, history independent but in experiments, soils do show history effects for states within the state boundary surface as discussed in Chapter 5. Two tests on perfect samples CP1 and CP2, were repeated following reconsolidation path K to examine some effects of the stress history on the behaviour of perfect samples. Perfect samples were reconsolidated following path K and then loaded following paths 1 and 2 in tests KP1 and KP2 respectively. The results are shown in figure 6.17, together with that of virgin samples in tests KF1 and KF2 and perfect samples in tests CP1 and CP2. It may be seen in figure 6.17 that the reconsolidation path had negligible influence on the strain path of the perfect sample in loading path 1. However, the effect of slow anisotropic reconsolidation of the perfect sample was to cause an increase in deviatoric stiffness and a decrease in the volumetric stiffness as shown in figure 6.17.

Also, it may be observed that perfect samples in tests KP1 and KP2 fell closer to the virgin behaviour in tests KF1 and KF2 respectively than those in tests CP1 and CP2 respectively.

Although it is difficult to confirm the above observation without further tests, it is possible to assume that the effects of initial swelling and slow anisotropic reconsolidation had brought the behaviour of the perfect sample closer to that of a virgin sample.

6.4.6. PERFECT SAMPLES WITH ADDITIONAL ISOTROPIC COMPRESSION

The behaviour of a virgin sample with additional isotropic compression in test 9nb was examined in section 5.4.3. and a similar test on a perfect sample in test 10na may be analysed and interpreted in a similar way. Test 10na was a termination test to sample KP2 where a perfect sample was loaded from $q' = 105 kNm^{-2}$ and $p' = 210\ kNm^{-2}$ to $p' = 310\ kNm^{-2}$ at the same shear stress level and the sample was then loaded to failure in undrained conditions in test 10na. Figure 6.20 shows the relationship between the stress ratio q'/p' and the ratio $\delta\varepsilon_s/\alpha$ and it may be observed that the sample had an initial constant ratio along ab followed by an increasing ratio along bc. This suggests an initial elastic behaviour followed by elasto-plastic behaviour.

The stress path of test 10na is shown in figure 6.19 and it shows that the stress path was initially linear and then it was followed by a non-linear path which suggests an initial elastic behaviour followed by an elasto-plastic behaviour. Similarly, the stress-strain relationship of test 10na in figure 6.18 may be interpreted as elastic along ab, followed by elasto-plastic behaviour along bc. In theory, the sample should have an initial elasto-plastic behaviour but the observations in both tests 9nb and 10na on virgin and perfect samples respectively seems to indicate otherwise.

6.4.7. DISCUSSION

In this section the behaviour of the perfect sample below its state boundary surface was examined. The results showed that the perfect sample was elastic for states well below the state boundary surface and those which are on or close to the state boundary surface may be interpreted as elasto-plastic. The reloading path had a noticeable influence on the subsequent behaviour of the perfect sample and, hence, it may suggest some form of inelastic behaviour below the state boundary surface. The elastic stiffnesses of the perfect samples were lower in value than those of a virgin sample.

CHAPTER 7. BEHAVIOUR OF TUBE SAMPLES

7.1. INTRODUCTION

It was shown in Chapter 3 that the tube sample is of a less quality than a block sample and the values of soil parameters are more altered in tube sampling than in block sampling. However, it is not obvious whether the tubing process alters the values of basic parameters simply by altering the state of the sample or whether the tube sample is fundamentally different from a block sample.

In this chapter it is assumed that tube samples are non-homogeneous and the main prupose of this chapter is to examine the effects of the tubing process on the state of the sample, the applicability of the critical state concepts to the overall behaviour of tube samples and to examine the effects of increasing the wall thickness to diameter ratio.

7.2. ANALYSIS OF DISTORTIONS IN TUBE SAMPLES

7.2.1. INTRODUCTION

It was suggested earlier in section 2.6.4. that tube samples are non-homogeneous and some evidence was quoted in section 3.3.3. in support of the suggestion. Although the research in this thesis was not directed towards analysing the causes of disturbance in tube sampling, it is necessary to point out the main causes of distortions in tube sampling and to examine theoretically their effects on the behaviour of tube sampling.

In this section it is assumed that the main causes of disturbance in tube samples are the displacement of soil during tubing and loss of suction. The effects of those causes on the behaviour of a tube sample is analysed and the behaviour of a tube sample is assumed to represent an overall behaviour of non-homogenous cylinder of soil which could be examined within the context of the critical state model.

7.2.2. EFFECTS OF HORIZONTAL STRAINS

Assuming that soil is displaced equally into and away from the tube wall the average horizontal strain in the soil in the tube is simply $\epsilon_h' = t/D$ where t is the wall thickness of the tube and D is its inner diameter. However, the horizontal strains may vary across the width of the sample with the largest strains near the wall and the smallest near the centre. Depending on the shape of the edge and the clearance ratio, the horizontal strains may be reduced significantly. Alternatively, the horizontal strains may be reduced simply by reducing the t/D ratio and in theory, there should be no horizontal displacements for an infintisimal wall thickness as illustrated in figure 7.1. For tube samplers with zero clearance ratio, it is assumed that in an undrained tubing process, there will be positive horizontal strains and negative vertical strains.

The changes in stresses associated with these horizontal strains may be calculated through the horizontal stiffness and for normally consolidated or lightly overconsolidated soils the strains will cause yielding and may also cause failure in extension. Figure 7.2. shows a specimen of soil before and after sampling where element A was near the wall of the sampler and element B was at the middle. Element A would have suffered hoizontal strains, α_1 t/D and element B would have suffered horizontal strains α_2 t/D where α_1 and α_2 are coefficients and α_2 is less than α_1. The changes in the stress states of both elements may be shown in figure 7.3. where element A is stressed until failure and element B is assumed to be stressed within the state boundary surface during the tubing processes. At the end of the tubing process and after unloading, both elements return to an isotropic state within the state boundary surfac· but with element A at a lower stress level than element B.

The extent of yielded and failed zones during sampling will depend on the t/D ratio and for increasing values of t/D the failure zone extends further towards the centre. After sampling, the overall behaviour of the soil specimen will represent an average behaviour of overconsolidated elements of soil.

7.2.3. LOSS OF SUCTION

As with perfect samples, tube samples may suffer loss of suction due to possible cavitation, swelling through contact with external water and yielding.

However, it was suggested in the previous section that as the tube is taken relatively quickly the immediate effect will be to produce a non-uniform distribution of excess pore pressure but as time passes with the sample in the tube or extruded from it, there will be equalisation of pore pressure and part of the sample will swell and part will consolidate. Figure 7.4. shows that elements A and B (as introduced before in section 7.2.2) will end up at states C1 and C2 after the equalisation of the pore pressures where element A consolidates and element B swells.

In the present testing programme, tube samples are likely to suffer more loss of suction than perfect samples, since every tube sample was exposed to surplus water at the base of the triaxial cell as they were reassembled. It is likely that tube samples will swell, due to contact with water and suffer further losses in suction and the state of elements A and B within the sample will move from states C1 and C2 in figure 7.4. to state D1 and D2 respectively and point D represents an average state of the sample. The effect of swelling on the behaviour of tube samples is to shift the state of the sample further inside the state boundary surface and to increase the overconsolidation ratio.

7.2.4. PREDICTION OF THE BEHAVIOUR OF TUBE SAMPLES

In laboratory testing, stresses and strains are measured at boundaries of the sample and therefore, the behaviour of a homogeneous sample with frictionless caps may be assumed to represent the behaviour of all elements of soil in the sample (see Appendix A.2).

However, it was suggested earlier in this section that elements of soil at different locations within the tube sample are subjected to different stress paths during tube sampling. Hence, a tube sample is likely to be non-homogeneous in which case, the behaviour measured at the boundaries represents the average behaviour of a non-homogeneous cylinder in which different elements have different disturbances.

There are a number of possible ways in which the behaviour of a non-homogeneous tube sample may be investigated. For example, the stress-strain behaviour of a non-homogeneous cylinder may be caluclated from numerical methods if the properties of all elements within the sample and the boundary conditions are defined. This approach requires many simplifying assumptions and implies knowledge of the effects of disturbance on the behaviour of elements of soil and therefore no attempts were made to follow this approach. Alternatively, the overall behaviour of a tube sample may be examined by observing the stresses and strains at the boundaries of the sample and the effects of disturbance on some overall basic parameters may be assessed.

7.2.5. OVERALL BEHAVIOUR OF TUBE SAMPLES IN LABORATORY TESTS

As with perfect samples, tube samples may be reloaded to their previous in-situ total stresses in drained or undrained conditions. Since the overall effects of tubing are to produce distortions in the sample and to shift the overall state of the tube sample to the point D in figure 7.5, undrained reloading will preserve the initial water content at a lower effective pressure shown as point E in figure 7.5. whilst drained reloading will recover the in-situ effective pressure at a lower specific volume at point F. If a tube sample is reconsolidated to a stress level substantially higher than its previous in-situ stress state, point G in Figure 7.5, then it may be possible to remove the effects of disturbance and hence recover the original behaviour of a stress virgin sample. In this case, the state inevitably is at a higher stress level and test data must be normalised to the virgin stress state.

The overall behaviour of the tube sample may or may not be significantly influenced by the non-homogenity. However, the only test results that are available are those which refer to the average behaviour of samples and it is not always easy to separate the influence of disturbance and non-homogenity.

The behaviour of tube samples must be examined firstly, by examining the fundamental elastic or elasto-plastic behaviour at different stress levels and secondly, by examining the behaviour of tube samples with reference to that of a virgin sample. It is important to notice that the behaviour of a tube sample represents the average behaviour of a non-homogeneous sample, rather than a disturbed element of soil, but the average behaviour of the tube sample can still be examined in the framework of the critical state model.

7.3. THE STATEOF THIN WALL TUBE SAMPLES

7.3.1. INTRODUCTION

The states of virgin and perfect samples in different tests were examined in previous chapters, by observing the values of q', p' and v. Aspects of behaviour such as the normalised stress path, stress-strain relationships and the relationship between the stress ratio and the strain ratio in drained tests and the $\delta\varepsilon_s/\Delta$ in undrained tests were used to distinguish between elastic and elasto-plastic behaviour. In both virgin samples and perfect samples, it was assumed that the samples were homogeneous and hence the behaviour of the samples represented the behaviour of all elements within the sample. A tube sample on the other hand, is probably non-homogeneous and may be at a stress level which is lower than that of a perfect sample. However, the state of the tube sample may still be examined, following methods similar to those used in examining the behaviour of homogeneous samples.

In this section, the behaviour of 4 thin wall tube samples will be examined; the state and behaviour of thick wall tube samples will be examined later in this chapter.

Test T7b was an unconsolidated,undrained test, test S8 was an isotropically consolidated undrained test on a tube sample at a stress level equal to its previous in-situ stress level, test KT6b was an anisotropically consolidated undrained test on a tube sample at its previous in-situ stress state and test 17nb was an anisotropically consolidated undrained test on a tube sample at a stress level 1.5 times its previous in-situ stress level.

7.3.2 THE INITIAL STATE OF TUBE SAMPLES

The initial state of tube samples as measured in the laboratory in different tests are presented in table 7.1. and shown in figure 7.6. together with the value predicted by elastic theory and the values measured for perfect and virgin samples in similar tests. Inspection of these data show that the least disturbed tube sample (i.e. those whose states were close to those of the virgin sample or the perfect sample) were those which the t/D ratio was least and these will be examined first.

The effective pressure of a thin walled tube sample under its previous total pressure was 40% less than the value predicted by elastic theory and 25% less than the effective pressure of the perfect sample. The reduction of the effective pressure of the tube sample may be interpreted following the assumptions in section 7.2 as the effects of yielding during the tubing process and swelling due to absorption of external water before testing and both contribute to reducing the effective pressure. In re-applying the previous total stress of a tube sample, the state of the sample shifts from E to E1 in figure 7.6 and the sample failed at E1. Alternatively, the previous total stress was re-applied under drained conditions.

When drainage was allowed at the previous back pressure, there was an average volume change of 4.6% as the thin walled tube samples recovered their initial effective stress, but at a lower specific volume. However, it is not possible to argue that a tube sample, reconsolidated to its stress state before sampling might be thought of as normally consolidated, since it is likely that some elements within the tube had exceeded their previous stress state and suffered yielding during the tubing process and the sample may be thought of as normally consolidated only if it is reconsolidated to a state which exceeds the greatest state which occurred during tubing.

7.3.3. THE RELATIONSHIP BETWEEN THE STRESS RATIO AND ε_s/α RATIO.

The relationship between the stress ratio and the strain ratio in drained tests and the $\delta\varepsilon_s/\alpha$ ratio in undrained tests were used in chapters 5 and 6 to distinguish elastic from elasto-plastic behaviour. In linear loading paths, elastic soils will have a constant strain ratio in drained tests or a constant $\delta\varepsilon_s/\alpha$ ratio in undrained tests while elasto-plastic soils will show increasing strain or $\delta\varepsilon_s/\alpha$ ratios.

Figure 7.7 shows the relationship between the stress ratio and the $\delta\varepsilon_s/\alpha$ ratio of tube samples in unconsolidated and reconsolidated undrained tests and in spite of some initial scattering due presumably to non-homogeneity, the relationship indicates regions of elastic behaviour over ab and elasto-plastic behaviour over bc.

Tube sample 17nb that was reconsolidated to a stress level 40% higher than its previous stress level had shown an initial elasto-plastic behaviour, whilst tube sample KT6b which was reconsolidated to its previous stress level, showed an elastic behaviour followed by an elasto-plastic behaviour. The un-consolidated tube sample T7b showed a wider scatter of data and it may be approximated to have a constant $\delta\varepsilon_s/\alpha$ ratio of up to $q'/p' = 0.7$ where it starts behaving elasto-plastically and similar behaviour may be observed in test S8, where it starts behaving elasto-plastically at about a stress ratio of $q'/p' = 0.45$.

Although the results are used here to indicate regions of elastic and elasto-plastic behaviour, it is possible that such observations may not be consistent with observations on the normalised stress path or stress-strain relationships due to possible non-homogenity of tube samples in which case different aspects of the behaviour are disturbed to different levels.

7.3.4. NORMALISED STRESS PATHS

For elastic materials the normalised stress path q'/p'_e vs. p'/p'_e is a straight line which will be vertical if the material is isotropic, whereas elasto-plastic behaviour shows a non-linear normalised stress path and therefore in theory it is possible to distinguish elastic from elasto-plastic behaviour.

The normalised stress paths for tube samples T7b, KT6b, S8 and 17nb are shown in figure 7.8 and it may be observed that all the paths may be approximated to vertical lines. It was observed earlier in figure 7.7, that tube samples T7b, KT6b and S8 showed an initial elastic behaviour ab followed by an elasto-plastic behaviour bc and the same points a, b and c are replotted on the stress paths in figure 7.8. The stress paths along bc in figure 7.8 are assumed to represent elasto-plastic behaviour and hence, may be used to represent the intersection of a constant volume plane with the state boundary surface. The stress path of a virgin sample may be compared to that of the tube sample in test 17nb and the results indicate a severe distortion in the shape of the state boundary surface of the tube sample. Similarly, test T7b, S8 and KT6b may be assumed to have elasto-plastic behaviour along paths bc in figure 7.8 and here again the stress paths indicate a state boundary surface which is substantially different from that of a virgin sample.

7.3.5. NORMALISED STRESS-STRAIN RELATIONSHIPS

In theory the normalised deviatoric stiffness of elastic soils G'/vp' (where the shear modulus G' is given by equation 2.4b) is constant while for elasto-plastic soils it decreases as the stress ratio increases. The stress-strain relationships in undrained tests on tube samples may be normalised as $\Delta q'/p'$ vs. $v\varepsilon_s$ and the variation of stiffness may be used to examine the change of state of the tube sample.

Figure 7.9. shows the normalised stress-strain behaviour of the tube samples in tests T7b, KT6b, S8 and 17nb and the same points a, b and c are replotted on the normalised stress-strain relationship to correspond to those on the stress paths and stress-strain relationships in figures 7.8 and 7.7 respectively. It may be observed in figure 7.9 that tests S8 and T7b showed an initial constant stiffness along ab and the stiffness was decreasing along bc which is consistent with the behaviour in figure 7.7. However, it is difficult to describe the initial behaviour along ab in test KT6b as elastic and point b does not indicate clearly a yield point corresponding to the yield point in figure 7.7. Test 17nb shows a non-linear stress-strain relationsip along bc which is consistent with figure 7.7 and hence, confirms an elasto-plastic behaviour.

7.3.6. DISCUSSION

It was suggested in section 7.2, that tube samples are non-homogeneous and their overall behaviour at or below their previous in-situ stress state is controlled by elastic elements at different overconsolidation ratios. The results in this section show that tube samples, both unconsolidated and reconsolidated to a stress level at or below their previous stress level, were initially elastic and later in the test behaved elasto-plastically. However, not all data showed the change in state from elastic to elasto-plastic as clearly as observed in the relationship between the stress ratio and the ratio $\delta\epsilon_s/\alpha$. The observed behaviour of tube samples was not consistent in indicating elastic or elasto-plastic behaviour and this was probably due to the non-homogenity of tube samples.

The state boundary surface of the tube samples shown in figure 7.8 as normalised stress paths q'/p_e' vs. p'/p_e' was substantially different from that of a virgin sample. Although reconsolidating the tube samples to a higher stress level in test 17nb seemed to have restored elasto-plastic behaviour, the state boundary surface was different to that for a virgin sample. It seems that the behaviour of virgin samples is substantially disturbed by the tubing process and hence, it seems necessary to examine both the fundamental behaviour of tube samples and the values of measured average soil parameters.

7.4. OTHER ASPECTS OF TUBE SAMPLES

7.4.1. INTRODUCTION

The results in the previous section suggest that tube samples are non-homogeneous and their behaviour appears to be initially elastic and later plastic. The state boundary surface of a tube sample was different to that for virgin samples and perfect samples and hence, other aspects of the behaviour of a tube sample are likely to be different from that of a virgin sample. However, it may still be possible that some fundamental relationships in the critical state model such as the critical state line and elastic behaviour are the same in tube and virgin samples.

7.4.2. THE CRITICAL STATE LINE

Tube samples were loaded to failure in drained and undrained tests and the overall states of tube samples at failure are measured in the laboratory and shown in figure 7.10.

It may be observed that there was a wider scatter of data than that observed for virgin and perfect samples, but the average states at failure for all tube samples fall close to a straight line and scaling from the diagram, $M = 0.98$, $\Gamma = 2.383$ and $\lambda = 0.14$ compared with $M = 0.95$, $\Gamma = 2.429$ and 2.414 and $\lambda = 0.14$ for virgin and perfect samples respectively. Thus, the main effect of tubing has been to shift the critical state line and the value of M was only slightly higher than that measured from virgin and perfect samples as shown in figures 7.10a and 7.10b. Similar observations on the effect of sampling disturbance on the $w_f'/$ - log p'_f were reported by Calhoon (1956).

It may still be argued that the overall specific volume at failure does not represent the specific volume on the failure plane or planes and therefore, the change in the measured value of Γ must be considered as an overall parameter, rather than a material constant.

7.4.3. RECONSOLIDATION BEHAVIOUR OF TUBE SAMPLES

In theory, soils have a constant compression index λ at virgin states and a constant swelling index k below their state boundary surfaces. However, it was observed earlier in section 6.3, that as the perfect sample was reloaded from an overconsolidated to normally consolidated state the volumetric stiffness changed gradually from λ to k in the v - ln p' diagram and this represents a deviation from the ideal behaviour of the model.

Since tube samples were found to be more disturbed than perfect samples and possibly non-homogenous, having zones of overconsolidated elements and failure planes, its behaviour during reconsolidation is expected to depart even more from the simple ideal model.

The overall behaviour of tube samples following paths K and C are compared to those of perfect samples and virgin samples in similar reconsolidation paths as shown in figures 7.11 and 7.12. It appears that perfect and tube samples have approximately the same deviatoric stiffness over their elastic range but the tubing process seems to have reduced the overall volumetric stiffness and shifted the consolidation path downwards in figure 7.13 and scaling from the diagram, the recompression index was 0.08. It seems more difficult to identify the change from elastic to plastic behaviour in tube samples.

7.4.4. PATH INDEPENDENCY

Some effects of stress histories on the behaviour of perfect and virgin samples were examined in section 5.4 and 6.4. respectively, where it was observed that the behaviour of virgin samples and perfect samples were slightly influenced by the consolidation path. Similarly, there were tests to examine whether the behaviour of tube samples is path independent in the elastic region.

Two sets of tube samples were reconsolidated to the same stress level following different stress paths.

The first set of samples included drained tests KT1 and KT3 and undrained test KT6b and these tube samples were prepared, follwoing reconsolidation path K. The second set of tube samples were prepared following reconsolidation path C and they repeated the tests T1, T3 and T6b in tests CT1, CT3 and CT6b. The results are shown in figures 7.14a and 7.14b. It may be observed that the reconsolidation path had negligible effects on the test reeults in all cases and hence, the behaviour of tube samples below their previous stress states was path independent. It may be suggested that the tube sample was overconsolidated and its state was well below the state boundary surface.

7.5. EFFECTS OF ADDITIONAL DISTURBANCE ON THE BEHAVIOUR OF TUBE SAMPLES

7.5.1. INTRODUCTION

Previous observations on the behaviour of tube samples were based on samples obtained from thin walled tube sampler with a t/D ratio of 0.039. The perfect sample may be thought of as a tube sample from a tube sampler that has a t/D ratio equal to zero and hence the effects of increasing the t/D ratio from zero to 0.039 were to cause the sample to be non-homogeneous and overconsolidated, and its overall behaviour was severely distorted. It might be supposed that further increases in the t/D ratio of the tube will increase the degree of disturbance in the tube sample and the purpose of this section is to examine the effects of increases in the t/D ratio on the furdamental behaviour of soils and on the values of some soil parameters.

7.5.2. STATES OF HIGHLY DISTURBED SAMPLES AT DIFFERENT STRESS LEVELS.

It was assumed earlier that the statesof tube samples at different degrees of disturbance are basically elastic and it may be necessary to examine this assumption before further analysis. The same approach as that adopted in section 7.3 may be used to examine the states of highly disturbed samples and arguments similar to those used earlier may be used to interpret the behaviour.

Firgure 7.15 shows the relationship between the stress ratio and the $\delta\varepsilon_s/\alpha$ ratio at different stress levels for tube samples T, M and H. It may be observed that the behaviour of all tube samples in unconsolidated undrained tests had initial elastic behaviour, shown as ab in figure 7.15a followed by an elasto-plastic behaviour shown as bc and all tube samples that were reconsolidated to a higher stress level had an initial elasto-plastic behaviour shown in figure 7.15c. where the relationshipsbetween q'/p' and $\delta\varepsilon_s/\alpha$ were non-linear. However, tube samples that were reconsolidated to their previous stress state showed some differences in their behaviour as shown in figure 7.15b. The highly disturbed sample in test KH6b showed a non-linear relationship and hence indicates an initial elasto-plastic behaviour, the sample in test CM6a showed an initial scattered data followed by a non-linear rleationship, while the least disturbed tube sample in test KT6b showed an initial constant value of $\delta\varepsilon_s/\alpha$ ratio which indicates an initial elastic behaviour.

In order to confirm the above observations, it is necessary to look at other aspects of the behaviour, such as the stress paths and stress-strain relationships. Figures 7.16a, 716b and 7.16c show the normalised stress paths of tube samples (T, M and H) below, at and above their previous in-situ stress states respectively.

Figures 7.16b and 7.16c show that tube samples reconsolidated to stresses equal and above their previous in-situ stress state respectively had similar normalised stress paths which may be approximated to a vertical line and it is difficult to see clearly a yield point. The stress path of all tube samples indicate a state boundary surface which is very different to that of a virgin sample. The normalised stress paths of tube samples in unconsolidated undrained tests in figure 7.16a show more pronounced differences between thick and thin walled tube samples. Data of thick walled tube sample are more scattered and the shape of its yield curve suggests that its state was on the wet side of the critical state line.

Normalised stress-strain relationships of the above tests are shown in figure 7.17. The behaviour of tube samples at and above their previous in-eitu stress state in figures 7.17b and 7.17c respectively show that the normalised stress-strain relationships of thick walled tube samples divert slightly from that of thinner walled tube samples. Tube samples that were reconsolidated to a higher stress level in tests 17n, 19n and 20n showed similar normalised stress-strain relationships in thin and thick walled tube samples as shown in figure 7.17c.

The stress-strain relationship was non-linear and hence, in agreement with an elasto-plastic behaviour, at higher levels.

There is no reason to assume that the states of tube-samples T, M, and H are different, i.e. all tube samples showed similar regions of elastic and elasto-plastic behaviour in similar tests. However, thick walled tube samples showed more scattered data and more disturbed behaviour than thinner walled tube samples in similar tests. The results also showed that different aspects of the behaviour of thick walled tube samples were inconsistent with each other in a similar way to that observed in thin walled tube samples in section 7.2. The relationship between the stress ratio and $\delta\varepsilon_s/\alpha$ ratio showed clearly zones of elastic and plastic behaviour, while it was more difficult to see that in normalised stress paths or normalised stress-strain relationships in the same tests.

7.5.3. THE OVERALL BEHAVIOUR OF HIGHLY DISTURBED SAMPLES AT DIFFERENT STRESS LEVELS

It was observed in the previous section that the normalised stress paths and stress-strain relationships were approximately the same for thin and thick walled tube samples. All tube samples showed similar patterns of behaviour under similar loading conditions. However, the effects of increasing the mechanical distortions on the values of some basic soil parameters may be more significant in some test than others.

The purpose of this section is to look at the overall behaviour and parameters of tube samples and to assess the effects of increasing the wall thickness diameter ratio on their values.

The behaviour of unconsolidated samples and samples reconsolidated to their previous in-situ stress level and to a higher stress level are shown in figures 7.18, 7.19 and 7.20, respectively. The values of strength, stiffness and pore pressure parameters are scaled from the diagrams and presented in table 7.2.

All tube samples at a stress level higher than their previous in-situ stresses showed similar stress-strain relationships and pore pressure response and hence, it seems, that reconsolidating tube samples to a higher stress level helps to eliminate the differences that are caused by different degrees of mechanical distortions. However, the highly disturbed sample in test H6b that was reconsolidated to its previous in-situ stress level showed slightly higher strength and stiffness than the least disturbed tube sample in T6b in a similar test. In the unconsolidated undrained test, thick walled sample H7b showed significantly lower strength and stiffness and its stress-strain behaviour diverted from that of thinner walled tube samples T7b and M7b.

It seems that differences in the behaviour of tube samples of different degrees of mechanical distortions are most obvious at lower stress levels and reconsolidating them to higher stress levels eliminates these differences.

7.5.4. PATH INDEPENDENCY IN HIGHLY DISTURBED SAMPLES

It was observed earlier, that samples taken with thin walled tubes were path independent at stress states less than their previous in-situ stresses, in the sense that their behaviour was indepeudent of the previous loading path and hence, their reconsolidation path had uo effect on the subsequent behaviour . However, it is interesting to examine whether the highly disturbed samples' taken with thick wall tubes were also path independent, since the overall behaviour of reconsolidated failure zones may be elasto-plastic and hence, may influence the overall path independency.

Highly disturbed samples were reconsolidated following paths K and C to their previous stress state, as explained before and they were tested under similar conditions. They included drained tests KH1, CH1, KH3 and CH3 and undrained tests KH6a and CH6b , these results are shown in figures 7.21 and 7.22. For the sake of such an inquiry, it seems relevant to compare the behaviour of thick tube samples in different tests and to compare the values of some overall basic parameters since the fundamental behaviour of thick tube samples is similar to that of thin tube samples as suggested in the previous section.

It may be observed that samples taken with thick wall tubes, that were reconsolidated following path K had higher stiffnesses, strengths and slightly higher pore pressure response and the values of some of the parameters were scaled from the diagrams and presented in table 7.3.

Comparing values of stiffness and strain ratios of different tube samples show some differences and the observations indiacte a form of path dependency. Apparently the wider zones of remoulded elements behaved elasto-plastically and influenced the overall behaviour of the tube sample. It may be suggested that not all elements in highly disturbed samples recover their virgin behaviour during reconsolidation.

7.5.5. SOME EFFECTS OF STORAGE ON THE BEHAVIOUR OF TUBE SAMPLES

Tube samples are usually subjected to additional causes of disturbance, such as storage and it was suggested earlier, in this thesis, that these effects may be of secondary importance. It was interesting within the framework of this research to conduct a few tests to examine the significance of the effects of the storage period and initial swelling. Tests on tube samples that were allowed to swell, S9 and S11 and tests on tube samples that were stored for a few days before testing G30 (½), G30(1) and S7, were described in chapter 4 and the results are compared to the behaviour of tube samples in similar tests in CT6b and T7b as shown in figures 7.23 and 7.24.

It may be observed in figure 7.23b that the stress-strain relationship indicates slightly lower strength and stiffness as the storage time increases. Test G30(1) shows that reconsolidating a tube sample to its previous in-situ stresses does not necessarily eliminate the effects caused by storage.

The results suggest that a storage period of less than three days had no significant effect in this particular test while that of a few weeks may cause considerable reductions in strength and stiffness and hence, it agrees with previous recommendations noted in Chapter 3 on testing samples immediately after sampling.

The tube sample in test S7 was reconsolidated to its previous in-situ stresses and stored for one week in the triaxial cell before testing. Figure 7.23a. shows the stress-strain curve and pore pressure response of sample S7 compared to that in test KH6a. The results indicate that a storage period of one week under the previous in-situ stresses had led to a considerable increase in stiffness and it is difficult to confirm this observation without further tests.

In both tests S9 and S11, water wae flushed through the base of the sample in the triaxial cell before application of the cell pressure, to cause initial swelling to take place. Both samples were tested undrained at their previous in-situ pressure and the results are compared to those in similar tests,without allowing initial swelling.

It may be noticed from the stress-etrain curves in figure 7.24 that swelling caused a decrease in strength, stiffness and pore pressure response since the swelled eample had a lower effective pressure and a higher specific volume. In theory, the sample should maintain the same undrained strength at the same w/s . The highly disturbed sample in test S9, was less influenced by initial swelling and this may confirm that the sample was more disturbed and had less initial suction than that in a sample taken with a thin wall tube. The results suggest that exposing samples to surplus water for only a few minutes may lead to considerable reduction in suction and an increase in the overconsolidation ratio.

7.6. EFFECTS OF TUBE DISTURBANCE ON SOME VIRGIN PARAMETERS

7.6.1. INTRODUCTION

Although it is important to examine the fundamental behaviour of soils and the effects of disturbance on that behaviour still, practising engineers are mostly concerned with the effects of disturbance on design parameters. Hence, it is valuable to compare directly the stress-strain relationships and the common engineering strength and stiffness parameters of virgin and disturbed samples. This engineering approach is similar to that used previously and a review of such work was given in Chapter 3.

Values of some engineering parameters were calculated from the basic data presented in figures 4.21 to 4.34 in Chapter4. The parameters of disturbed samples were compared to those of virgin samples in similar tests as shown in tables 7.4 to 7.7. These results are used in this section to assess the effects of disturbance and laboratory procedures on the behaviour of virgin samples in different tests.

7.6.2. EFFECTS OF DISTURBANCE ON STIFFNESS

Table 7.4. shows the undrained secant modulus E'_u of virgin and disturbed samples at a stress level which is 50% of the total stress increment.

It may be observed that the stiffness of tube samples in the unconsolidated undrained test (test 7 in table 7.4) was significantly reduced by increasing the wall thickness/diameter ratio of the sampler. However, the effects of reconsolidation was to reduce the differences between thin and thick walled tube samples (test 6 in table 7.4) and the stiffness parameters of different tube samples were approximately the same at higher stress levels (test 9 in table 7.4). The overall stiffness parameters of thin walled tube samples that were re-consolidated to their previous in-situ stress level were approximately equal to that of a virgin sample. However, the perfect sample at its previous in-situ stresses (test 6 in table 7.4) showed a lower stiffness parameter than that of a virgin sample.

Table 7.5 shows the deviatoric and volumetric stiffness parameters of virgin and disturbed tests for various loading paths. The effects of disturbance on the stiffness parameters varied in different loading paths and in general it may be observed that both the tube sample and the perfect sample had higher volumetric stiffness parameters and lower deviatoric stiffness parameters than that of a virgin sample.

It seems difficult to pass general comments on the effects of perfect and tube sampling on stiffness parameters, since the effects seem to vary at different stress levels and loading paths.

7.6.3. EFFECTS OF DISTURBANCE ON THE STRENGTH OF VIRGIN SAMPLES

The effects of sampling disturbance on the critical state line were examined in section 6.3 and 7.4. However, it may be interesting to compare the undrained strengths of virgin samples with those of disturbed samples. Table 7.6 shows the undrained strengths of disturbed and virgin samples in similar tests. It may be observed, that the undrained strengths of disturbed samples in the unconsolidated undrained tests are lower than that of a virgin sample. However, the undrained strength of the perfect sample that was reconsolidated to its previous stress state was approximately equal to that of a virgin sample, while that of a tube sample was greater than that of a virgin sample.

The results in table 7.6 are not normalised and hence, they do not take into account differences in initial water contents or effective pressures of virgin and dieturbed samples. In theory, samples with lower water contents have higher undrained strengths and the results in tests series 6 and 9 in table 7.6 confirm this. Samples with different initial effective pressures and the same water content must, in theory, have the same undrained strengths. This seems to indicate that the effects of both swelling and tubing in thick walled tube samples led to higher water contents, lower effective stresses and lower undrained strengths.

7.6.4. EFFECTS OF DISTURBANCE ON STRAIN PATHS AND PORE PRESSURE RESPONSES

Ratios of strains and plastic strain increments were used in section 5.2 to examine aspects of the fundamental behaviour of soils. The effects of disturbance on the strain behaviour of soils may be assessed by comparing the strain behaviour of virgin and disturbed samples.

It was reported in the previous section, that disturbed samples had lower deviatoric stiffness and higher volumetric stiffness parameters than those of virgin samples. Hence, in general, ratios of deviatoric to volumetric strains are higher in disturbed samples. Strain paths of virgin and disturbed samples along different loading paths in drained tests are shown in figure 7.21. The differences between strain paths of virgin samples and those of disturbed samples varied in different tests. Loading paths of higher $\Delta q'/\Delta p'$ ratios showed higher ratios of deviatoric to volumetric strain ratios. The above observations are empirical and it is difficult to use them in assessing different degrees of sampling disturbance.

The pore pressure response is used in calculating the effective stress path and hence, in examining the behaviour of samples. However, it is possible to examine the effects of disturbance on the pore pressure response by comparing the values of some pore pressur parameters of virgin and disturbed samples.

Two pore pressure parameters are used in this section to examine the effects of disturbance; parameter A_f is calculated at failure as

$$A_f = \Delta u_f / \Delta q_f$$

and parameter S_{50} is calculated at a stress level of 50% of the stress increment as

$$S_{50} = \Delta u / \Delta \varepsilon_a \cdot p'$$

Values of A_f and S_{50} for virgin and disturbed samples may be scaled from figure 4.29 to 4.34 and the results are summarised in tables 7.7a and 7.7b. For isotropic elastic soils, A_f is $\frac{1}{3}$ and it may be observed in table 7.7a that tube samples had values of A_f approximately equal to $\frac{1}{3}$ in all tests. Virgin and perfect samples showed higher values of A_f than those in tube samples. For elasto-plastic behaviour on the wet side of the critical state line, A_f has higher value and virgin and perfect samples showed values of A_f, that are approximately 2 and 3 times that of tube samples respectively. The A_f parameter does not seem to give a clear indication of the degree of disturbance.

Similarly, the parameter S_{50} is not expected to give a clear indication of the degree of disturbance and table 7.7b shows that it is less influenced by tubing disturbance in test 6. It seems that disturbance influences parameters S_{50} and A_f in different ways and these parameters do not give clear indications of the effects of sampling disturbance.

8. CLOSING REMARKS

8.1. SUMMARY

It appeared from the literature, that previous approahces to the problem of sampling disturbance were mainly concerned with its effects on the values of some soil parameters and thus, the differences between block samples and tube samples were recognised quantatively on empirical basis. The main prupose of this thesis was to examine the effects of sampling disturbance on the fundamental behaviour of soils. The major causes of disturbance were recognised as the stress disturbance and the mechanical distortions and their effects on the behaviour of samples were approached and investigated separately.

Different triaxial compression tests were performed on kaolin which was prepared in three different ways, to simulate a virgin sample, a perfect sample and a tube sample. All virgin samples were prepared from slurry in two stages; in the first stage, the slurry was loaded in a sample former to form a 1½" sample at a low stress level and in the second stage, the sample was loaded anisotropically in a triaxial cell to a substantially higher stress level. A perfect sample was obtained from a virgin sample, simply by releasing the total stresses in undrained conditions and tube samples were obtained by inserting 1½" tube samplers in 4" perfect sample.

The testing programme included drained tests on disturbed samples, reconsolidated to their previous stress state and undrained tests on disturbed samples at different stress levels. In addition, there were tests to examine some aspects of the virgin behaviour, effects of loading conditions and other secondary causes of disturbance that were relevant to the main testing programme.

8.2. MAIN CONCLUSIONS

The research set out to distinguish between the effects of the main causes of sampling disturbance on normally consolidated soils and to examine their effects on the fundamental stress-strain behaviour of soils. The behaviour of the anisotropically consolidated kaolin used in the research, was shown generally to follow the concepts of the critical state soil mechanics and these theories were used as a reference to assess the effects of sampling disturbances.

The investigations in this research, showed that the two main causes of disturbance, stress disturbance and mechanical distortions, were basically different and must be approached in different ways.

Stress disturbance appeared to be a form of stress history effect that altered both the state of the virgin sample and the values of some soil parameters. A perfect sample of a normally consolidated soil at or below its previous stress state, may be thought of as anoverconsolidated sample, while a perfect sample reconsolidated to a higher stress state may be thought of as a normally consolidated sample. Although the Cam clay model predicted no effects of stress disturbance, the behaviour of the perfect sample followed the concepts of the critical state model with a state boundary surface and some parameters that were slightly different from those of a virgin sample.

The main cause of mechanical disturbance was the wall thickness of the tube and it appeared that increasing the ratio of the wall thickness to the diameter of the tube, caused a quantitative increase in the disturbance, rather than a qualitative change in the state of the sample. The behaviour of the tube sample did not follow the concepts of the Cam clay model, due to its non-homogeneity, but its overall behaviour was similar to that of overconsolidated sample .By reconsolidating a tube sample to a moderately high stress level, the elasto-plastic behaviour was recovered but this was not the same as the behaviour of the virgin sample.

8.3. OTHER OBSERVATIONS

The series of tests in this research made it possible to comment on some other aspects of sampling disturbance and laboratory testing. These are mostly empirical observations and they are summarised as follows:

The properties of virgin samples that were most influenced by stress disturbance and mechanical distortions were the deformation properties. The stress disturbance caused a lower deviatoric stiffness and a higher volumetric stiffness, whereas the mechanical distortions decreased both the deviatoric and volumetric stiffnesses. Sampling disturbance reduced the pore pressure response of virgin samples and both the perfect sample and the tube sample had approximately the same strain-excess pore pressure relationship.

The disturbed sample had a cirtical state line similar to that of a virgin sample, but it was slightly shifted downwards in v - ln p' plane having the same values of M and λ but a different value of Γ. The disturbed sample had a lower effective pressure under zero total stresses and a lower undrained strength. The tube sample that was reconsolidated to its previous stress level had a lower specific volume and higher strength than that of a virgin and perfect sample.

The deviation of the values of some parameters of the disturbed sample from those of a virgin sample varied at different states and loading conditions. Strength and stiffness parameters of disturbed samples at their previous stress states were in better agreement with those of virgin samples, than those of disturbed samples in unconsolidated undrained tests. The parameters of the tube sample at its previous stress state overestimates those of a virgin sample.

A storage period of up to 3 days produced no significant alteration to the behaviour of disturbed samples of kaolin. Initial swelling produced a serious reduction in the strength and stiffness of tube samples but swelling is expected to have negligible effects on the behaviour of disturbed samples reconsolidated to their previous stress levels.

8.4. FURTHER RESEARCH

The research has investigated the behaviour of perfect and tube samples, but it did not fully establish mathematical or experimental procedures which recover the behaviour of virgin samples. However, there were some preliminary investigations on the effects of reconsolidation on the behaviour of disturbed samples and there are reasons to suggest that if disturbed samples were reconsolidated to a substantially higher stress level, then the normalised behaviour will be identical to that of a virgin sample. It is not clear to what stress level the perfect and tube samples should be reconsolidated before they recover their virgin behaviour. It may be assumed that the required stress level will depend on the degree of disturbance and the initial stress state.

Tube samples with t/D greater than 0.039were found to have little relationship to virgin and perfect samples. However, there were reasons to suggest that the central core of tube samples of a lower t/D ratio is similar to a perfect sample and, hence, trimming a tube sample with a low t/D ratio and a large diameter may give a perfect sample. It is not clear to what t/D ratio and to what diameter this will be applicable.

All tests in this research were conducted on samples of remoulded kaolin that were prepared from a slurry and anisotropically consolidated in a triaxial cell. The fabric of natural soils may have a significant influence on the behaviour of soils and, hence, further research is required to extend the previous observations to cover the behaviour of natural soils.

A.1. SIDE FRICTION IN THE SAMPLE FORMER

A.1.1. INTRODUCTION

The function of the sample former is simply to produce a homogeneous sample which is stiff enough to allow it to be mounted in a triaxial cell. Since the sample was further consolidated in the cell to a stress level at least 4 times that in the former, the precise state in the former is not important.

The sample was prepared by loading slurry in a thick tube of height to diameter ratio larger than 2 and friction errors are expected at low stress levels. The magnitude of these errors is unimportant, except insofar as they may affect the homogeneity of the sample, in spite of further triaxial consolidation and for this reason, it is necessary to investigate their magnitude and how they may be reduced. Some of the previous investigations into the development of side friction in oedometer tests are reviewed and further investigations led to a better procedure for obtaining uniformity throughout the sample. In order to assess the significance of side friction on the homogenity of the sample, a number of tests were carried out, in which samples were prepared in different ways and the distribution of water contents along the lengths were measured.

A.1.2. PREVIOUS INVESTIGATIONS ON EFFECTS OF SIDE FRICTION

The observed behaviour of the soil in an oedometer is influenced by several factors among which, the side friction is of importance (Rowe 1959). The standard oedometer cell has a height to diameter ratio usually less than 0.5, which is very different from that of the sample former, used in the present research. The effects of side friction on the distribution of stresses and strains within an oedometer cell are basically similar to those in the sample former and, although the magnitude is less, similar procedures may be used to reduce the effects of side friction in the sample former.

Leonards and Girault (1961) observed that side friction varied linearly with depth and was not affected by the load increment ratio. However, higher side friction is developed at lower stress levels (Taylor, 1942) and Burland and Roscoe (1969) observed non-uniform distributions of vertical and horizontal strains at low stress levels.

Different techniques have been suggested to reduce side friction. The work of Thompson (1962), Barden and Barry (1965), Burland and Roscoe (1969) and others indicates that by smearing the side walls of an oedometer with silicone grease, wall friction can be reduced to a very small magnitude.

Northey and Thomas (1965) however, suggested that soft, low friction coatings are of little more effect than other smooth surfaces and in addition, it is difficult to keep the lubricants in place for long durations. Other measures to reduce the effects of side friction such as the use of the floating ring and using a low height/diameter ratio are not relevant to the case of the sample former and hence, they will not be discussed any further.

A.1.3. TESTS TO EXAMINE THE EFFECTS OF SIDE FRICTION

The side friction in the sample formers was expected to be appreciable, due to the large H/D ratio and the existence of the porous stones along the inner wall. No attempt was made to measure the side friction along the sampler directly and hence, it was not possible to calculate the stresses throughout the sample. However, the effects of the side friction and the effectiveness of techniques to reduce the side friction may be assessed by the measurements of water content along the length of a sample after consolidation in a sample former.

Figure A.1. shows the variation of water content with depth in samples prepared in the sample former. In tests A and B the samples were loaded from the top and the water content was found to vary approximately linearly with depth. In test C where the load had reached about 60% of its required value, it was quickly removed, the sample and the former inverted and the load reapplied, and increased to the required value.

This procedure had the effect of reducing the observed variation of water content as shown in figure A.1. This technique has not reduced the magnitude of side friction, but because of the change of direction, it reduced its effect on the uniformity of the sample. In spite of some initial uon-homogenity in samples produced by the sample formers, it was observed that samples which were subjected to further triaxial consolidation to an effective vertical stress level of 2.75 kN/m^2 had a uniform water content of 37.5 $\pm$ .5%.

A.2. SOME EFFECTS OF THE APPARATUS ON THE MEASURED BEHAVIOUR

A.2.1. INTRODUCTION

The measurements of stresses, pore pressures and strains at the boundaries of a homogeneous sample in the triaxial cell are meant to represent those in all elements within the sample, but, in triaxial tests, there are several causes of non-uniform distribution of stresses and strains and it is necessary to reduce or account for these interferences. Some of the major cuases of non-representative measurements are introduced in this section, with some correction procedures. The behaviour of samples that buckle are assumed to be non-representative and these tests were repeated rather than corrected.

A.2.2. END FRICTION

The top cap used in the triaxial tests in this research was a rigid smooth perspex and the sample was based on a filter paper on top of a porous stone. The top cap was smeared with silicone grease to reduce friction, but apparently, this did not last for the full two weeks duration of tests and there were probably shear stresses developed between the sample and the cap in most tests in the final stages.

The base of the cell was unavoidably rough, since it was not possible to smear the filter paper with silicone grease and preserve its drainage function at the same time.

The effects of friction at both ends are to restrain the lateral deformation near the ends, rotate the axes of principal stresses, cause a variation of the cross sectional area and the stresses along the sample (Perloff and Pombs 1969). A rigid top cap leads to a uniform distribution of strains across the top face, except near the edges (Burland and Roscoe 1969). Atkinson (1973) observed that different lubricants did not seem to be effective in reducing the friction for long durations. Crawford (1963) reported that one effect of frictional ends was to cause non-uniform distribution of pore pressure within the sample and Blight (1963) suggested a very slow rate of loading to allow equalisation of pore pressure within the sample.

Balasubramanium (1969) reported that the deformations of samples with frictional ends were uniform along the sample up to 50% of the strain at failure. These effects are examined in more detail by other workers (e.g. Atkinson 1973) with further procedures on reducing their effects. In the present research, two measures were used to reduce the effects of the end restraint. Firstly, the top cap was smeared with silicone grease, in order to minimise the friction, especially in the early stages. Secondly, tests were performed with very slow rates of loading to allow time for dissipation or equalisation of pore pressures.

A.2.3. CORRECTION OF DEVIATORIC STRESSES

Deviatoric stresses were calculated simply as the axial force divided by the current cross sectional area, given by Bishop and Henkel (1962) as

$$A = A_o \quad \frac{1 - \Delta v/v_o}{1 - \Delta \ell/\ell_o}$$

The initial weight of the top cap was deducted from the overall axial load. The triaxial cells that were used in this research were supplied with internal load cells and hence, excluded the effects of any ram friction.

A.2.4. THE RUBBER MEMBRANE

The procedure used in sealing the sample within rubber membrane is commonly used in routine and research tests.

The rubber membrane may influence the measurements of stresses and strains in two ways. Firstly, it may fail to seal the sample for long durations, and secondly, it may influence the measured deviatoric stress during testing.

Leakage through the rubber membrane and past the bindings may cause serious errors in measurements of pore pressures,for long duration undrained tests, but it has negligible errors on short term tests. The problem was investigated in detail by Polous (1964) and different measures were suggested to account for and reduce the effects of leakage on the pore pressure and volume changes. Among the procedures that were used by several workers (e.g. Davis and Poulos 1963) are double membranes with oil between, replacing the cell water by oil and applying grease to the ends under the membrane. Polous (1964) assumed a linear relationship between leakage rates and pressure and hence, estimated the errors caused by leakage in drained and undrained tests.

In the present research, a single membrane, with greased ends and double O - rings was used. Some samples were left in the cell at the end of the test, to observe further changes in the pore pressure or the volume. Negligible changes in pore pressures and volume were observed for periods up to 3 days and hence, no corrections were employed to account for leakage.

Although the membranes are usually thin, it was observed by Henkel and Gilbert (1952), that membranes cause an incrase in measured strnegth. The effects of the rubber membrane on the deviatoric stress was examined by Bishop and Henkel (1962) and estimated as

$$\Delta q = \frac{\pi D_o M \ \varepsilon_a(1-\varepsilon_a)}{A_o}$$

where D_o is the initial diameter of the sample and M is the compression modulus of the rubber membrane per unit width (1.7 lb/in for 0.008 in thick membrane). This equation was used in the present research to correct the results of drained and undrained test where ε_a was taken as the axial strain.

A.2.5. VARIATION OF TEMPERATURE

Temperature variations may affect both the apparatus and the sample and hence, cause errors. Thermal effects on the walls of the cell, the base and the tubes are negligible, but they may alter the measurements of stresses and loads in the transducers. Henkel and Sowa (1963) found that cycles of temperature variation appeared as a hysterisis loop in the observed pore pressure and led to an accumulation of excess pore pressure.

The influence of temperature variations on soil behaviour may be approximated by theoretical analysis and experimental observations as reviewed by Campanella and Mitchell (1968).

All tests reported in this thesis were conducted at an approximately constant temperature of 25 $\pm$ 1^{o}C in a temperature controlled laboratory. There were negligible differences in the measurements of the transducers and load cells and thus no corrections were necessary to account for the minor variations of temperature.

A.2.6. EFFECTS OF SIDE FILTER DRAINS

Bishop and Henkel (1962) reported that restrains imposed by filter paper drains were difficult to estimate and they proposed a correction of 2 psi at strains above 2%. This proceudre may be satisfactory in correcting strengths but it does not seem satisfactory to correct stress-strain relationships simply by deducing a constant value of 2 psi throughout the test.

In the present research, it did not seem possible to estimate accurately the effects of filter drains on the stress-strain relationships and no correctious were used to account for their effects.

A.3. SOME EFFECTS OF DIFFERENT RATES OF LOADING

A.3.1. INTRODUCTION

In general, soils do not respond immediately to changes in total stresses and boundary conditions and hence, it is important to examine the effects of the rate of loading on drained and undrained tests. In theory a drained test has a rate of loading where no excess pore pressure is allowed to develop and an undrained test has a rate of loading where no excess pore pressure is allowed to dissipate. However, in the laboratory, samples are loaded in drained and undrained tests by simply preventing or permitting drainage at the boundaries but it is difficult to control local drainage within the sample. Hence, the loading rate may still influence the measured behaviour.

In general triaxial tests are either stress controlled or strain controlled. In a stress controlled test, increments of stresses are applied and thus the stresses are raised as steps, and after each step the sample is allowed some time to reach equilibrium. The rate of loading is usually presented in terms of stress increments per period of time and thus the rates may be reduced either by increasing the duration or by decreasing the increments or by both. In a strain controlled triaxial test the sample usually deforms axially at a constant rate in terms of deflection per unit time and a range of rates is usually provided by the triaxial loading machine.

A.3.2. SOME EFFECTS OF THE RATE OF LOADING

The rate of loading may affect measurements of stresses, strains and pore pressures in two basic ways. Firstly, the rate of loading may be larger than that of primary consolidation in drained tests or the rate of equalisation of pore pressure and the response of the pore pressure transducers in undrained tests and therefore, the measurements misrepresent the state of samples. Secondly, the rate of loading may be much slower than that of primary consolidation and equalisation of pore pressure and, therefore, creep effects may contribute significantly to misrepresent the strains from those caused by changes of effective stresses.

Higher rates of loading have been observed to cause higher strengths (e.g. Crawford 1963, Richardson and Whitman 1963 and Blight 1963). It is difficult to interpret these effects since the pore pressures at failure were measured at the boundaries and they do not represent those at the failure plane. It may be argued that slower rates of loading will cause some local drainage and hence, reduce the effective pressure at the failure plane. Northey and Thomas (1965) showed that longer durations are required to achieve equilibrium at higher stress levels. Bishop and Henkel (1962) suggested a method to estimate the time required to bring the sample to failure based on the coefficient of consolidation at the stress level of the sample before testing.

Secondary consoldiation takes place under constant effective stresses and secondary volumetric strains were reported to be as much as 25% of primary volumetric strains for some kaolin samples (Davis and Poulos 1967).

In order to estimate the period of time for primary consolidation, measurements of axial deflection were plotted versus the time for both a drained and undrained increment of load and the results are shown in figure A.2. A period of 12 hours seemed to be sufficient for primary consolidation in both drained and undrained conditions and was used in all stress controlled tests in this research.

A.3.3. EXPERIMENTAL INVESTIGATION ON DIFFERENT RATES OF LOADING

Part of this research consisted of undrained tests on virgin and perfect samples that were conducted at different slow rates of loading. In the strain controlled tests, samples were brought to failure within 8 hours at a loading rate of .02 $mm/_{min}$ while the stress controlled tests required up to 3 days at a loading rate of 20 kN/m^2 per 12 hours. From calculations based on

$$t_f = \frac{20h^2}{\eta C_v} \qquad \text{(Bishop and Henkel 1962)}$$

more than 90% dissipation of the excess pore pressure occurred in the strain controlled tests but because of the slower rate it is believed that secondary effects occurred in the stress controlled tests.

The rate of loading in stress controlled tests may be calculated as the total strain increment divided by the period of test.

Two sets of tests may be used as examples to show the effects of different rates of loading in undrained tests. In the first set of tests, virgin, perfect and lightly over consolidated samples were loaded to failure, KF6a, CP6a and S1 respectively, in stress controlled tests and in the second set of tests, similar samples were loaded to failure in KF6b, CP6b, and S11 in strain controlled tests and the results are shown in figure A.3.

It may be observed, that the stress-strain relationships were not influenced by the differences in loading rates, but the pore pressure response was higher at lower rates of loading in stress controlled tests. However, for the loading rates used in this research, the differences in the observed behaviour were within a narrow zone and had no significant effect on the test result.

A.4. SOIL WATER RELATIONSHIPS

A.4.1. INTRODUCTION

Engineers are more interested in the effects of the soil water relationship on the mechanical behaviour of soil than the physical and chemical nature of that relationship. The mechanical behaviour of soil is influenced by the degree of saturation, the water content and the drainage conditions and it is necessary to measure some parameters that describe these. The purpose of this Appendix is to examine the errors involved in the measurements of some soil water parameters.

A.4.2. SATURATION

For fully saturated soils, the effective stresses are claculated simply by deducting the pore pressure from the total stresses, but for partially saturated soils it is not simple to measure the pore pressure and calculate the effective stresses. For fully saturated soils, an increase in the mean total stress under undrained conditions will lead to an equal increase in the pore pressure and this simple test is commonly used to examine the degree of saturation and known as the B test (Skempton 1954). Measurements of the pore pressure in the triaxial cell may be easily influenced by air bubbles trapped in the pore water system outside the sample, between the sample and the membrane, within the porous stone and along the drainage path leading to the transducer. Hence, in addition to ensuring that the sample is saturated, it is important to ensure saturation of the rigid system for pore pressure measurements.

Previous observations on the problem of saturation (e.g. Bishop and Henkel 1962,Black and Lee 1973 and Nadarajah 1973) have led to a number of recommendations to ensure saturation of soils. These included preparation of samples from a slurry at a water content higher than the liquid limit, de-airing the slurry, subjecting it to vibrations and de-airing it again, care to avoid trapping air while spooning the slurry into the sample former, care to ensure initial saturation of the porous stone, filter paper and the drainage path, application of a back pressure more than 100 kNm^{-2} and flushing pressurised de-aired water through the pore pressure system before testing.

All these procedures were employed in the present research.

It is difficult to obtain 100% saturation in remoulded soils and the above techniques may lead to a saturation degree within the range of 90 to 99%. It is even more difficult to ensure saturation in undistrubed samples in undrained tests and higher back pressure is suggested to ensure saturation in drained tests. In the series of tests presented in this thesis, it was found that values of the parameter B in the B test were 90 to 95%.

A.4.3. SUCTION

Pore pressure is usually measured through an external body of water which is assumed to be in equilibrium with the pore water. However, this assumption may not be strictly true for negative pore pressures and it is difficult to specify accurately the pore water soil relationship (Child 1969). At high negative pore pressures, air or water vapour bubbles may occur and reduce the suction and the meniscuses near the external surface of the sample may fail as a result of the differential pressure. Hence, the negative pore pressure measured at the boundaries of soils may not necessarily represent that of the pore water (Donald 1961).

Casagrande (1936) suggested that suction would cause internal swelling, where pore water is transferred locally from some elements to another.

The effects of imposing negative pore pressure to soils may be the same as compression. Bishop (1975) et al observed thatthe internal meniscuses of water can hold very high suctions which may be measured under high total pressure. Although there is evidence that water can sustain high negative pore pressures (Marshall 1959) the ability of pore water in clay to carry high suctions is not necessarily the same (Donald, 1961).

In the present test, it was not possible to measure accurately negative pore pressures of disturbed samples under zero total pressure and it was necessary to re-apply high total pressure and calculate the effective stress state of the sample.

A.4.3. WATER CONTENT

The state of soil is usually described in terms of the effective stresses and the specific volume and aspects of soil behaviour may be normalised by parameters that depend on the specific volume. Hence, it is necessary to examine the procedures used to calculate the specific volume.

For saturated soils the specific volume v, the void ratio e and the water content w are related as

$$v = 1 + e = 1 + w\,G_s$$

Where G_s is the specific gravity. The initial specific volume v is calculated from the initial water content before loading, the current specific volume v during testing is calculated from the observed volume change simply

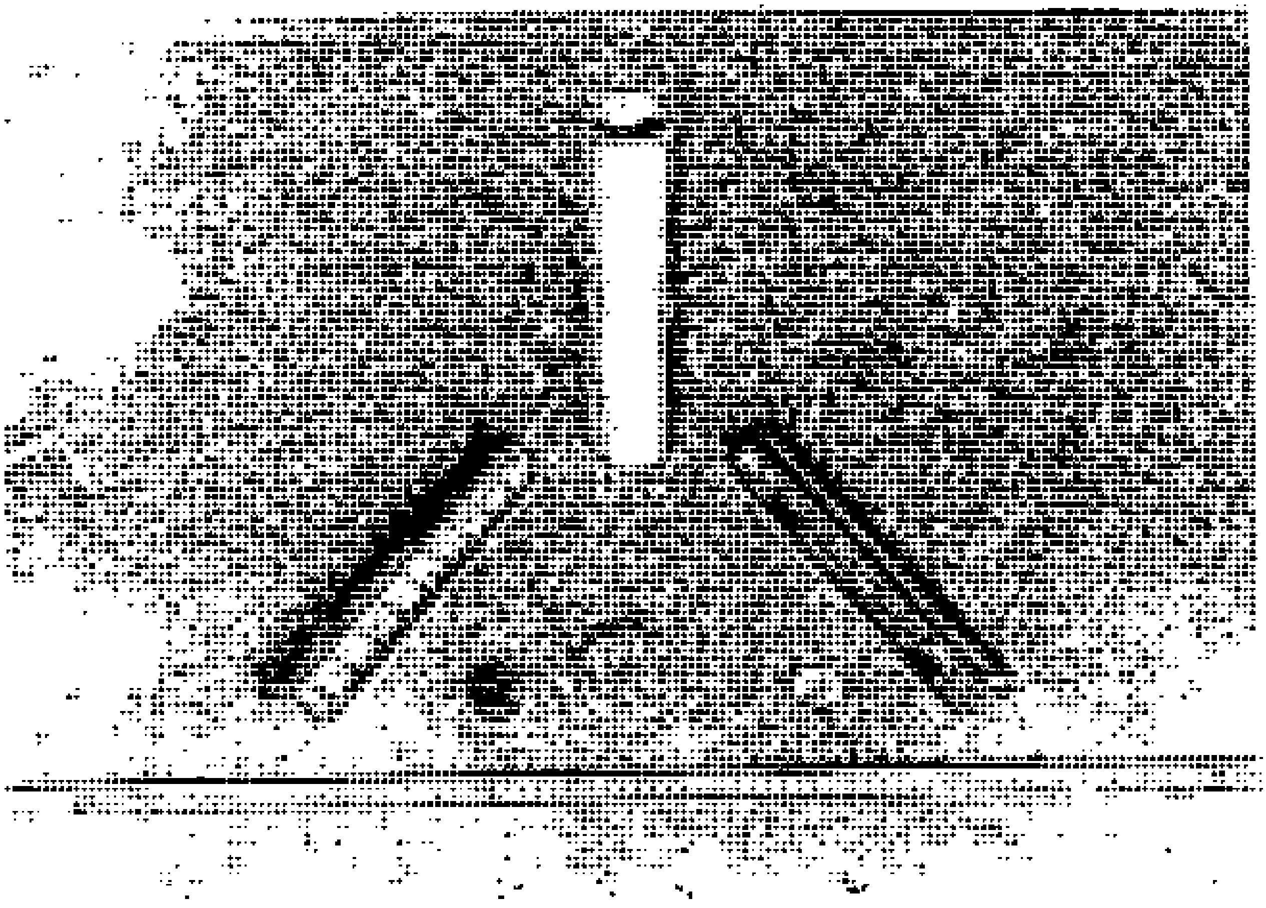
$$v = v_o\,(1 - \Delta V/V_o)$$

Where V_o is the initial volume of the sample and ΔV is the change in volume and is +ve for compression and -ve for swelling and the water content at the end of the test may be compared to the calculated value. The author observed that the measured water content at failure was usually different from the calculated value, but in general, they were within ± 2%. Henkel and Sowa (1963) observed that the measured volume of water during consolidation was larger than the corresponding decrease in weight of the specimen amounting to a difference of 1.5%in water content. They attributed this error to the amount of water taken in by the sample during unloading and they found that the higher the suction, the larger the error. Ward, Marsland and Samuels (1965) observed that the initial water content of block samples were 5% less than those in-situ and those may be attributed to suction in a similar way.

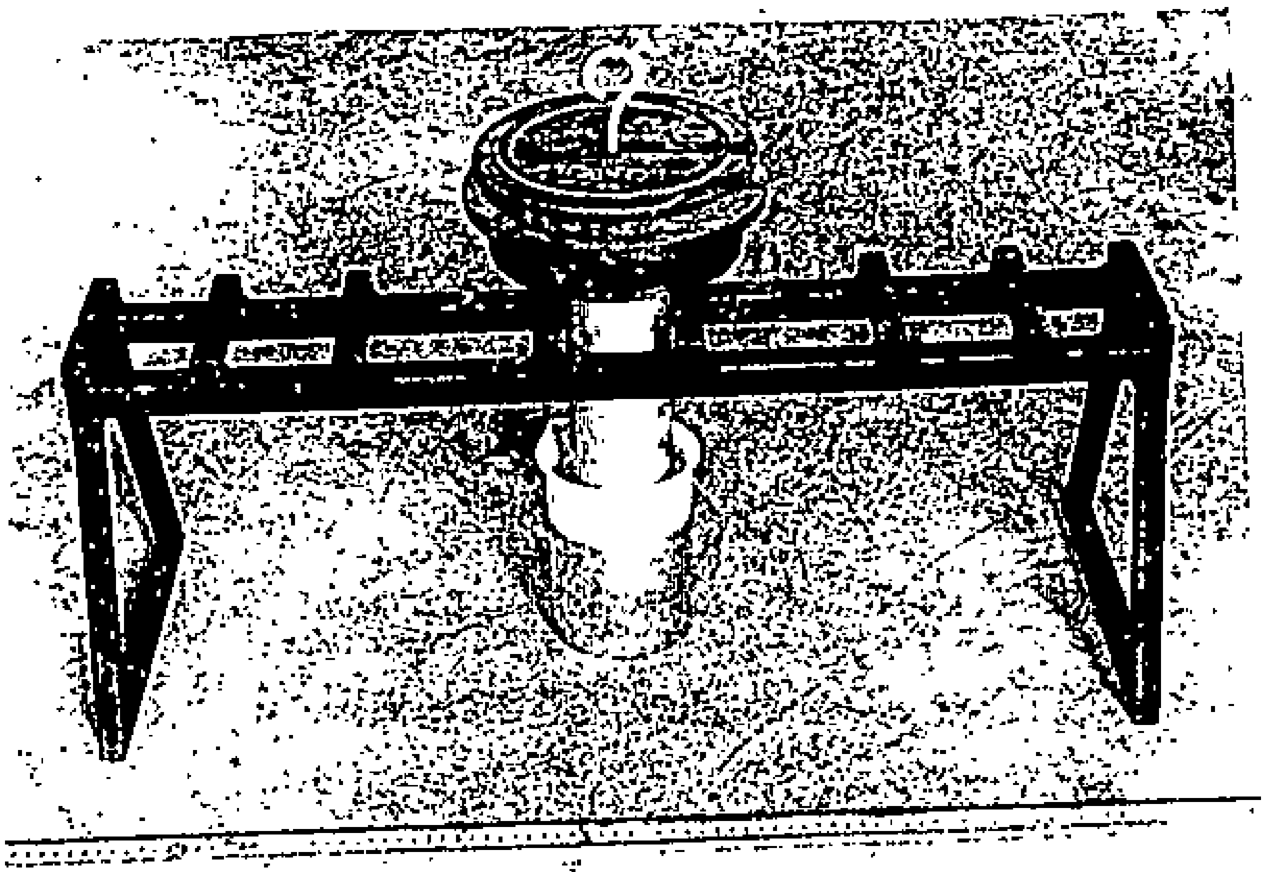
In the present research, values of specific volumes were based on volume changes and initial water contents rather than on the water contents at failure.

Although it was shown in Appendix A.1. that the initial water content of samples before testing was homogeneous, there were reasons to suggest that it may not be so, as the sample approahces failure. As the failure plane starts to develop, non-homogeneous deformations and local drainage may take place and neither the overall change in v nor the overall water content can be used to calculate the specific volume at failure, in the failure plane.

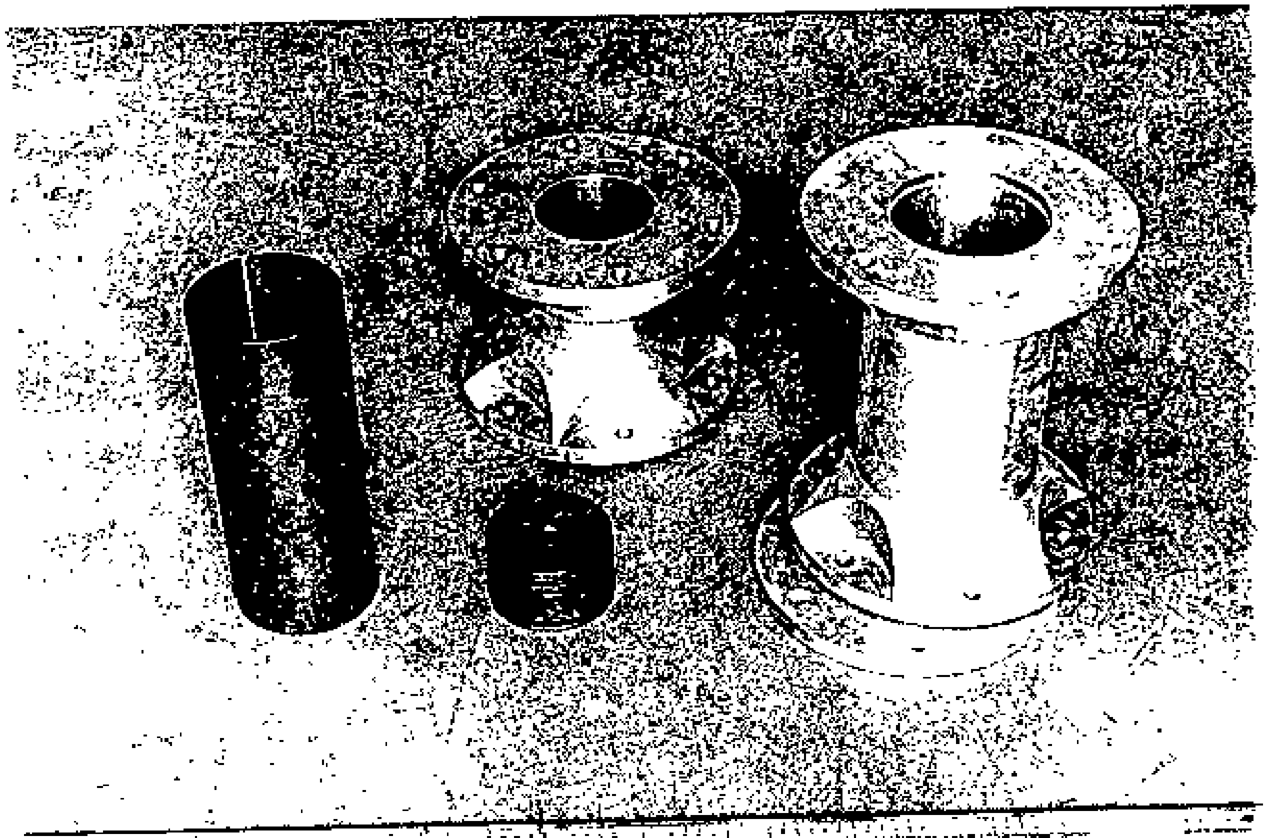
To account for such effects on the stress path and stress-strain relationships, the behaviour of an element before failure is estimated by extending the uniform relationships up to the failure state.



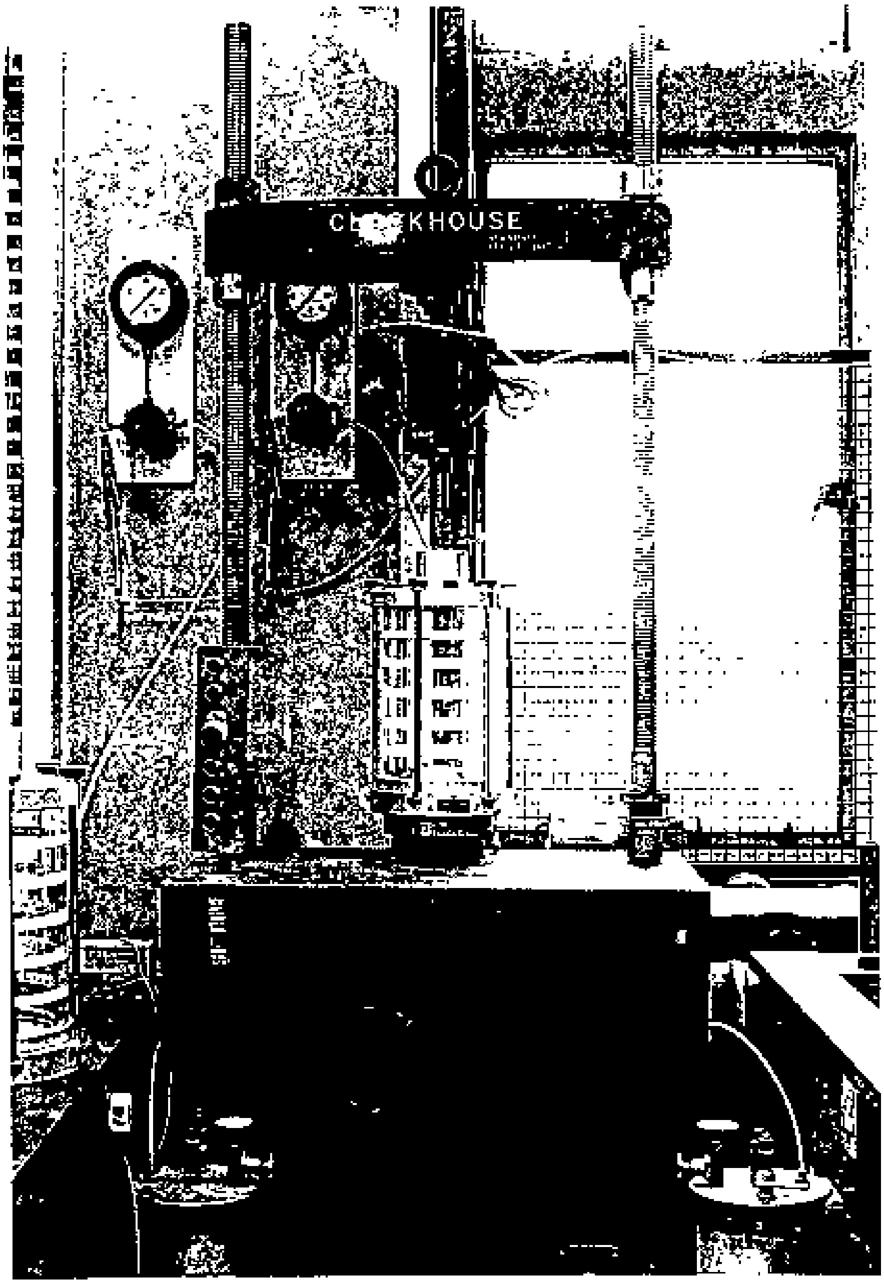
PHOTOGRAPH 1 : THE COMPONENTS OF THE 1½" FORMER



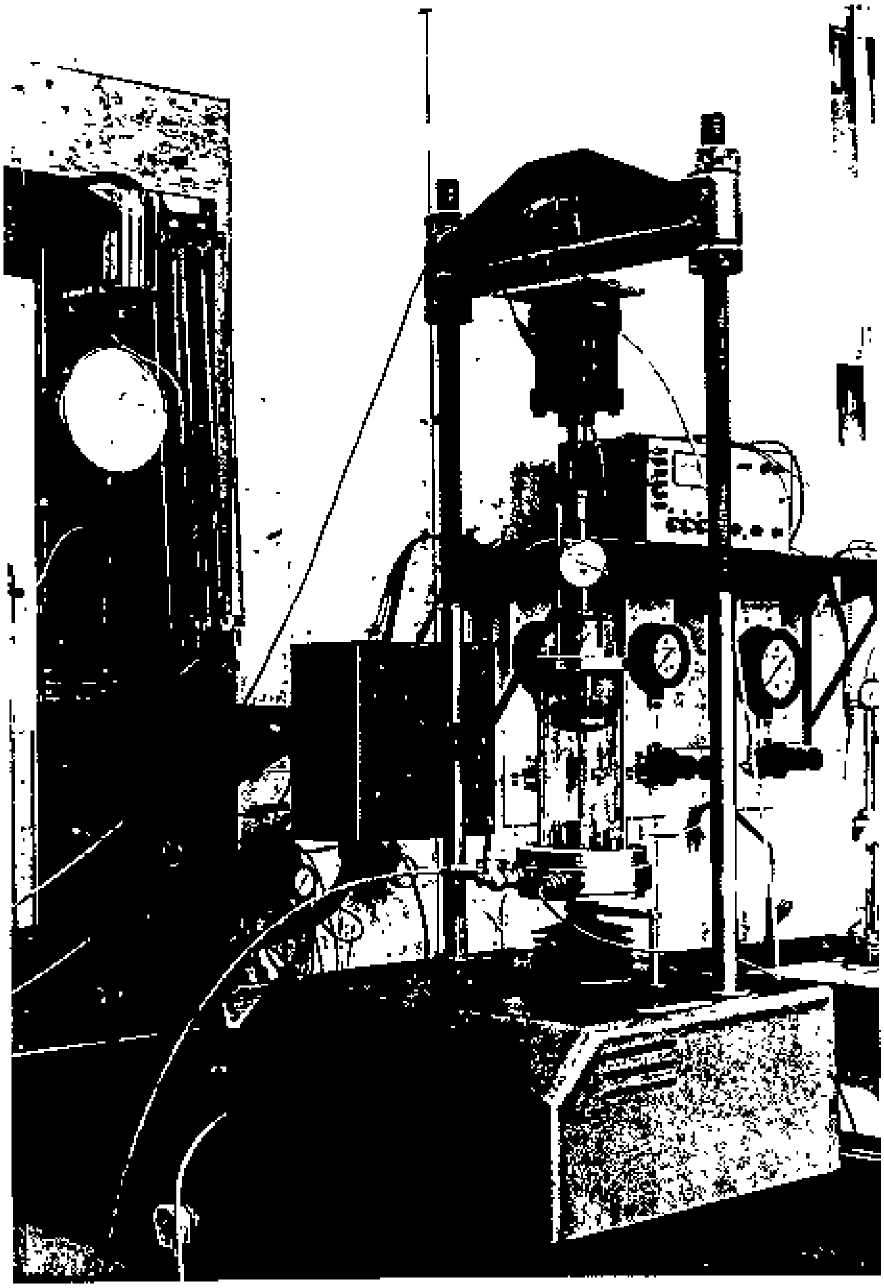
PHOTOGRAPH 2 : THE 1½" SAMPLE FORMER IN OPERATION



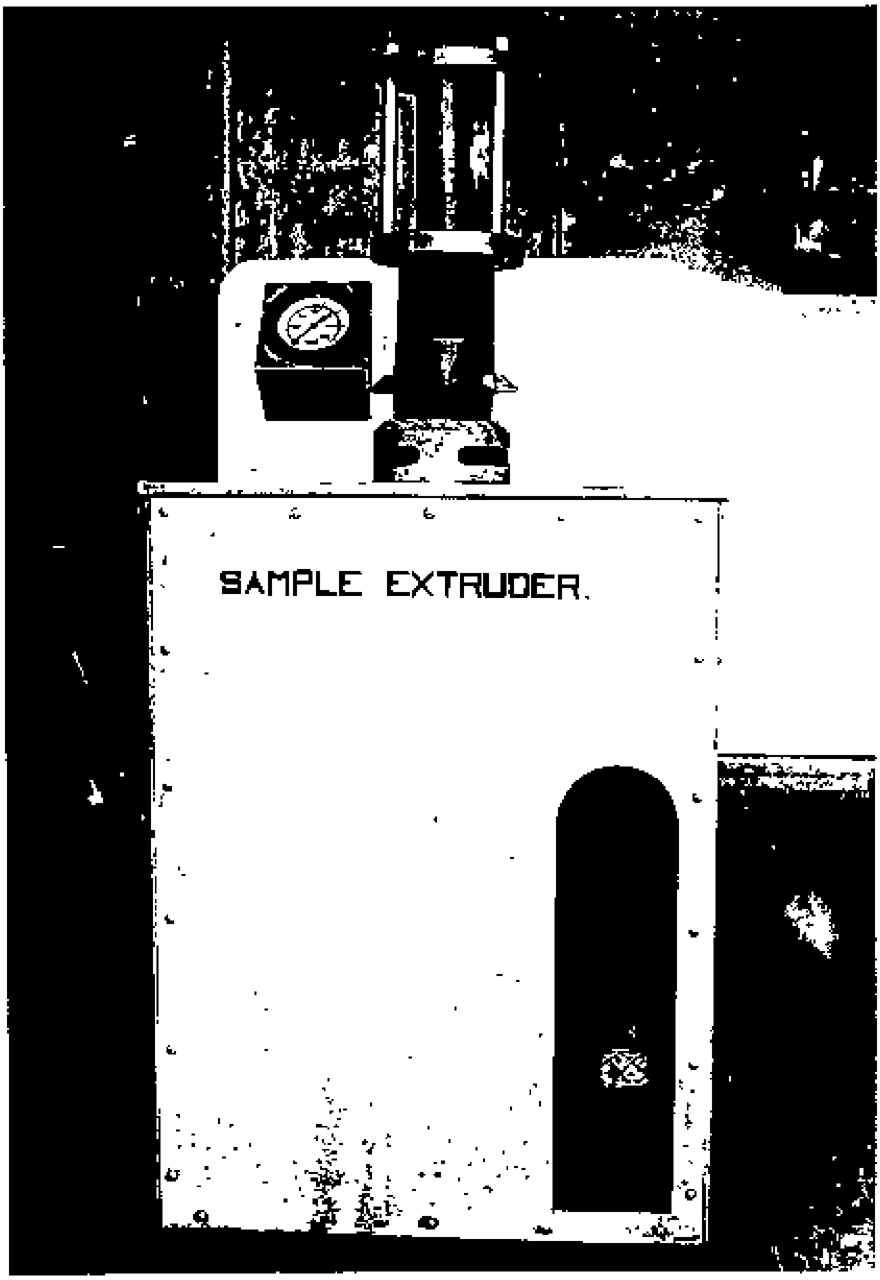
PHOTOGRAPH 3 : THE COMPONENTS OF THE 4" FORMER



PHOTOGRAPH 4 : THE 4" TRIAXIAL CELL



PHOTOGRAPH 5 : THE $1\frac{1}{2}$" CELL IN A TRIAXIAL LOADING MACHINE



PHOTOGRAPH 7 : APPARATUS FOR TUBE SAMPLING

Soil Type	Sampler Type	Laboratory Test	Ratio of strength	Ratio of stiffness	Reference
London Clay	$1\frac{1}{2}$" tube	unconfined compression	0.61	0.29	Ward et al 1959
Overconsolidated clay	Piston	unconsolidated undrained	0.42	-	Eden 1971
sensitivity=20 LL=60, PL=23	Piston	unconfined compression	0.54	0.5	Rochelle and Lefebvre 1971
fissured clay	Open tube	unconsolidated undrained	-	0.2	Lo et al 1971
sensitive clay LL=56%, PL=25	3" tube	unconsolidated undrained	0.62	-	Conlon and Isaac 1971
	5" tube		0.78		
sensitivity=10 LL=69, PL=25	Piston	consolidated undrained	0.86	0.84	Milovic 1971
	Open tube		0.71	0.72	
Leda cemented clay	Open tube	unconsolidated undrained	0.30	0.18	Raymond et al 1971
	Osterberg		0.55	0.52	

NB. Measurements on block samples were used as a reference in all above ratios

TABLE 3.1 EFFECTS OF THE SAMPLER TYPE ON SOME MECHANICAL PROPERTIES

Soil Type	Disturbed Sample	Laboratory Test	Ratio of strengths	Reference
sensitivity=5 LL=68, PL=38	5" tube sample	unconfined compression	0.7	Nelson et al 1971
Glacial deposit	2" tube sample	unconsolidated undrained	0.7	Adams et al 1971
Marine Clay	5" tube sample	consolidated undrained	0.9	Bozozuk 1971
sensitive clay	square tube sample	unconfined compression	0.91	Wilson 1969
stiff fissured clay	2" tube sample	unconsolidated undrained	0.32	Lo et al 1971
	block sample		0.62	
San Francisco Bay	6" tube sample	unconfined compression	0.75	Hall 1963

NB. Measurements of in situ vane tests were used as a reference in all above ratios
The ratio of strengths above are given as $\frac{C_u \text{ of disturbed sample}}{C_u \text{ vane test}}$

TABLE 3.2 EFFECT OF SAMPLING DISTURBANCE ON IN-SITU STRENGTH

Soil Type	Disturbed Sample	Laboratory Test	Ratio of stiffness.	Reference
London clay	tube sample	unconsolidated undrained	0.30	Marsland 1973b
London clay	block sample	unconsolidated undrained	0.33	Marsland 1973a
London clay	block sample	unconsolidated undrained	0.85	Ward et al 1965
chalk	block sample	unconfined compression	2.40	Burland and Lord 1970

NB. Measurements of in situ plate loading tests were used as a reference in all above ratios
The ratio of stiffness above are given as $\frac{E_u \text{ of sample}}{E_u \text{ of in-situ test}}$

TABLE 3.3 EFFECT OF SAMPLING DISTURBANCE ON IN SITU STIFFNESS

Type of clay	Sensitivity $\frac{C_u \text{ undisturb.}}{C_u \text{ remoulded}}$	Stiffness ratio $\frac{E_u \text{ undisturb.}}{E_u \text{ remoulded}}$	Reference
Marine	14	25	Casagrande
Marine	8	6.	Casagrande
Volcanic	5	12	Rutledge
Glacial	4	33	Peck
Glacial	3	17	Rutledge
Glacial	3	5	Rutledge
Glacial	2	7	Rutledge

TABLE 3.4 EFFECTS OF COMPLETE REMOULDING ON COMPRESSIVE STRENGTH AND UNDRAINED STIFFNESS MODULUS (RUTLEDGE 1944)

Stress Disturbance	Soil Type	Laboratory Test	% of change in strength	% of change in undrained stiffness	% of change in drained stiffness	% of change in the parameter A_f	Reference
Undrained release of deviator stress	Weald Clay sensitivity = 2	Unconsolidated undrained	-1			-67	Skempton and Sowa 1963
	Boston Blue Clay	Unconsolidated undrained	-7				Ladd and Lambe 1963
	Honmoku Marine Clay, sensitivity	Unconsolidated undrained	-10	-30		-66	Okumra 1971
Undrained release of total stresses	San Francisco Bay sensitivity	Unconsolidated undrained	-6			-46	Noorany and Seed 1965
	Kaolin Clay sensitivity = 3	Unconfined compression	-18				Davis and Polous 1967
		Reconsolidated undrained		-15	+5		

NB. Measurements of laboratory virgin samples were used as a reference in all above percentages

TABLE 3.5 SOME EFFECTS OF PERFECT SAMPLING ON THE MECHANICAL PROPERTIES

Type of Sample	Laboratory virgin sample	K_o reconsolidated perfect sample	K_o reconsolidated disturbed sample	K_o reconsolidated remoulded sample	Isotropically reconsolidated perfect sample
E_u	1723	1464	1577	1876	7160
$\frac{E_u(\text{virgin})}{E_u(\text{disturbed})}$	1.00	1.17	1.09	0.91	0.24
E'	505	530	542	704	2590
$\frac{E'(\text{virgin})}{E'(\text{disturb})}$	1.00	0.95	0.93	0.71	0.16
ν'	0.395	0.390	0.380	0.354	0.075
$\frac{\nu'(\text{virgin})}{\nu'(\text{disturb})}$	1.00	1.01	1.04	1.11	5.26
C_{v3}	0.0613	0.0567	0.0482	0.0410	0.0337
$\frac{C_{v3}(\text{virgin})}{C_{v3}(\text{disturb})}$	1.00	0.28	0.33	0.40	0.48

TABLE 3.6 EFFECTS OF RECONSOLIDATION ON DEFORMATION PARAMETERS
(Davis and Polous 1967)

		sensitivity	accuracy
Modified cell	load cell	3.17div/KNM^{-2}	$\pm$ 0.3 KNM^{-2}
	cell pressure transducer	33.1 div/KNM^{-2}	$\pm$ 0.5 KNM^{-2}
	back pressure transducer	35.5 div/KNM^{-2}	$\pm$ 0.5 KNM^{-2}
Standard cell	load cell	1.02 div/KNM^{-2}	$\pm$ 0.5 KNM^{-2}
	cell pressure transducer	35.8 div/KNM^{-2}	$\pm$ 0.5 KNM^{-2}
	back pressure transducer	38.1 div/KNM^{-2}	$\pm$ 0.5 KNM^{-2}

TABLE 4.1 SENSITIVITY AND ACCURACY OF LOAD CELLS AND TRANSDUCERS

Time hours	$1\frac{1}{2}$" sample former		4" sample former	
	increment of load (lbs)	total load (lbs)	increment of load (lbs)	total load (lbs)
0	1	1	5	5
6	2	3	10	15
18	5	8	20	35
30	10	18	40	75
42	15	33	40	105
54	-	-	40	145
66	-	-	40	195

TABLE 4.2 PERIODS AND INCREMENT OF LOADS IN PRELIMINARY CONSOLIDATION

Tube	Wall thickness t (mm)	$\frac{t}{D}$ ratio
T	1.48	.039
M	2.73	.072
H	3.99	.105

TABLE 4.3 DIMENSIONS OF THE TUBE SAMPLES

Desig.	Drainage Conditions	initial state		ratio of load increments
		$q'(kNm^{-2})$	$p'(kNm^{-2})$	
1	drained	105	210	$\Delta p'/\Delta q' = 0$
2	drained	105	210	$\Delta q'/\Delta p' = 0$
3	drained	105	210	$\Delta q'/\Delta p' = \frac{1}{2}$
4	drained	105	210	$\Delta q'/\Delta p' = -2/1$
5	drained	105	210	$\Delta q'/\Delta p' = 0$
6	undrained	105	210	$\Delta q/\Delta p = 3/1$
7	undrained	0	< 210	$\Delta q/\Delta p = 3/1$
8	undrained	105	> 210	$\Delta q/\Delta p = 3/1$
9	undrained	> 105	> 210	$\Delta q/\Delta p = 3/1$

(a) Description of figures designations

K	Anisotropic consolidation with $q'/p' = \frac{1}{2}$
C	Reconsolidation of disturbed samples; the cell pressure is reapplied first then followed by the axial pressure
S	Special preparation or a special test
R	Repeated test
G	Stored sample
n	Termination test
F	Virgin sample
P	Perfect sample
T	Tube sample; thin wall thickness and t/D = .039
M	Tube sample; medium wall thickness and t/D = .072
H	Tube sample; thick wall and t/D = .105

(b) Description of letters designations

TABLE 4.4 DESCRIPTION OF DESIGNATIONS

Serial No.	Desig	Figure	Group Description
39	KF6a	4.29	1-Undrained tests on virgin and perfect samples
40	CP6a	4.29	2-Initial stress state of $q'=105$ kNm^{-2} and
41	KF6b	4.29	$p' = 210$ kNm^{-2}
42	CP6b	4.29	
43	P7b	4.31	1-Undrained tests on disturbed samples
44	M7b	4.31	2-Initial stress state of $q'=0$ and $p'<210$ kNm^{-2}
45	H7b	4.31	
46	T7a	4.31	
47	T7b	4.33	
48	9n	4.32	1-Undrained tests on disturbed samples
49	10n	4.32	2-Initial stress state of $q'=105$ kNm^{-2} and
50	12n	4.32	$p' > 210$ kNm^{-2}
51	KF9b	4.34	1-Undrained tests on virgin and disturbed
52	CP9b	4.34	samples
53	KT9b	4.34	2-Initial stress state of $q'>105$ kNm^{-2},
54	17n	4.34	$p'>210$ kNm^{-2} and $q'/p'=\frac{1}{2}$
55	19n	4.34	
56	20n	4.34	
57	S1	4.29	Special Tests see section
58	S2	4.28	
59	S3	4.25	
60	S4	4.25	
61	S5	4.29	
62	S6	4.29	
63	S7	4.30	
64	S8	4.32	
65	S9	4.31	
66	S10	4.33	
67	S11	4.31	
68	R57	4.29	Other Tests
69	R34	4.30	
70	R14	4.23	
71	R15	4.23	
72	R22	4.26	
73	G47([illegible])	4.33	

TABLE 4.5 TESTING PROGRAMME

Serial No.	Design	Figure	Group Description
1	KF1	4.21	1-Drained tests on virgin and disturbed samples
2	CP1	4.21	
3	KP1	4.21	2-Initial stress state of q'=105 kNm^{-2} and
4	KH1	4.21	p'=210 kNm^{-2}
5	KT1	4.21	3-Ratio of stress increments $^{\Delta p'}/_{\Delta q'}$=0
6	CT1	4.2	
7	CM1	4.21	
8	CH1	4.21	
9	KF2	4.22	1-Drained tests on virgin and disturbed samples
10	KP2	4.22	2-Initial stress state of q'=105 kNm^{-2} and
11	CP2	4.22	p'=210 kNm^{-2}
12	KH2	4.22	3-Ratio of stress increments $^{\Delta q'}/_{\Delta p'}$=0
13	KT2	4.22	
14	KF3	4.23	1-Drained tests on virgin and disturbed samples
15	CP3	4.23	2-Initial stress state of q'=105 kNm^{-2} and
16	KH3	4.23	p'=210 kNm^{-2}
17	CT3	4.24	3-Ratio of stress increments $^{\Delta q'}/_{\Delta p'}$=½
18	G17	4.24	
19	CM3	4.24	
20	CH3	4.24	
21	KF4	4.26	1-Drained tests on virgin and disturbed samples
22	CP4	4.26	2-Initial stress state of q'=105 kNm^{-2} and p'=210 k
23	CT4	4.26	3-Ratio of stress increments $^{\Delta q'}/_{\Delta p'}$=$^{2}/_{-1}$
24	KH4	4.26	
25	KF5	4.27	1-Drained tests on virgin and disturbed samples
26	CP5	4.27	2-Initial stress state of q'=105 kNm^{-2} and p'=210
27	KT5	4.27	3-Ratio of stress increments $^{\Delta q'}/_{\Delta p'}$=0
28	KH5	4.27	
29	KT6b	4.30	1-Undrained tests on tube samples
30	CT6b	4.30	2-Initial stress state of q'=105 kNm^{-2}
31	G30(1)	4.30	and p'= 210 kNm^{-2}
32	G30(1)	4.30	3-Ratio of stress increments $^{\Delta q}/_{\Delta p}$ = $^{3}/_{1}$
33	KT6a	4.30	
34	CT6a	4.30	
35	CH6b	4.30	
36	R35	4.30	
37	CM6a	4.30	
38	KH6a	4.30	

Test	q'_o kNm^{-2}	p'_o kNm^{-2}	calculated w_o%	p'_o calculated as elastic	Disturbance factor $1 - p'/p'_o$
unconsol-idated undrained	0	145 ± 5	37.5	175	.18
reconsol-idated	105	210	36.5	210	0.0
reloaded undrained	105	170 ± 5	37.5	175	0.03

TABLE 6.1 THE INITIAL STATE OF THE PERFECT SAMPLE IN DIFFERENT TESTS

Test	[illegible]	q'_f kNm^{-2}	p'_f kNm^{-2}	p'_o kNm^{-2}	q' kNm^{-2}
S1	.33	175	175	210	[illegible]
S5	.07	160	175	[illegible]	[illegible]
P7b	.17	145	158	[illegible]	0

TABLE 6.2 STRENGTH AND ELASTIC STIFFNESS OF VIRGIN AND PERFECT SAMPLES

sample type	unconsolidated undrained			reconsolidated to previous level		
	p'_o kNm^{-2} (measured)	p'_o kNm^{-2} (elastic)	$1 - \frac{p'_o \text{ measured}}{p'_o \text{ elastic}}$	q'_o (kNm^{-2})	p'_o (kNm^{-2})	w_o %
T	115 ± 5	175	.34	105	210	33.5 ± 1
M	100 ± 5	175	.42	105	210	33 ± 1
H	80 ± 5	175	.55	105	210	33 ± 1.5
P	145 ± 5	175	.17	105	210	36.5
F	–	–	–	105	210	37.5

TABLE 7.1 THE INITIAL STATE OF THE TUBE SAMPLES

Sample	$\Delta q'_f$ kNm^{-2}	E'_u	$\frac{\Delta u_f}{\Delta q_f}$
T	102	30	.31
M	96	25	.31
H	73	15	.13

(a) Unconsolidated Undrained Tests

Sample	$\Delta q'_f$ kNm^{-2}	E'_u	$\frac{\Delta u_f}{\Delta q_f}$
T	210	30	.30
M	210	30	.30
H	189	20	.38

(b) Reconsolidated Undrained Tests

Sample	$\Delta q'_f$ kNm^{-2}	E'_u	$\frac{\Delta u_f}{\Delta q_f}$
T	267	28	.36
M	273	31	.50
H	283	33	.27

(c) Samples Reconsolidated to a higher stress level

NB. All E'_u are secant moduli at 50% of $\Delta q'_f$ and in [illegible]$^{-2}$

TABLE 7.2 PARAMETERS OF TUBE SAMPLES IN DIFFERENT TESTS

Sample	$\Delta q'_f$ kNm^{-2}	E'_u kNm^{-2}	$\frac{\Delta u_f}{\Delta q_f}$
KH6	208	25	.3
CH6	186	19	.38

Sample	$\Delta q'_f$ kNm^{-2}	E'_s kNm^{-2}
KH1	187	25
CH1	165	18

Sample	E'_v kNm^{-2}	E'_s kNm^{-2}
KH3	67	22
CH3	50	16

NB. All E'_u and E'_s are secant moduli at 50% of $\Delta q'_f$

TABLE 7.3 PARAMETERS OF TUBE SAMPLES
WITH DIFFERENT STRESS HISTORIES

Test	virgin	t/D = 0 perfect	t/D = .039 Tube T	t/D = .079 Tube M	t/D = .104 Tube H
7	-	44	30	23	15
6	32	20	30	30	20
8	48	81	-	-	90
9	25	16	28	31	33

NB. All measurements are secant moduli at 50% of $\Delta q_r'$ and in MNm^{-2}

TABLE 7.4 THE UNDRAINED STIFFNESS OF VIRGIN AND DISTURBED SAMPLES IN DIFFERENT UNDRAINED TESTS

loading path	virgin	perfect	Tube T	Tube M	Tube H
1	26	20	16	16	16
3	19	20	37	23	19
4	32	22	13	-	-

(a) Deviatoric stiffness E'_s measured as a secant modulus at 50% of Δq_f and in kNm^{-2}

loading path	virgin	perfect	Tube T	Tube M	Tube H
2	41	55	40	-	60
3	29	53	61	53	56
5	-104	- 80	- 144	-	- 170

(b) Volumetric stiffness E'_v measured as a secant modulus and in kNm^{-2}

TABLE 7.5 STIFFNESSES OF VIRGIN AND DISTURBED SAMPLES IN DIFFERENT DRAINED TESTS

Test	Virgin Sample	Perfect Sample	Tube T	Tube M	Tube H
7	-	155	102	96	73
6	167	159	210	210	189
8	300	268	-	-	284
9	193	182	267	273	283

NB. All measurements are q_f' and in kNm^{-2}

TABLE 7.6 THE UNDRAINED STRENGTH OF VIRGIN AND DISTURBED SAMPLES IN DIFFERENT UNDRAINED TESTS

Test	virgin	perfect	Tube T	Tube M	Tube H
7		8.2	3.77	5.72	5.81
6	4.66	8.47	8.61	8.58	7.05
8	10.45	5.57	-	-	6.34
9	-	7.43	10.05	6.86	8.53

NB. All measurements are $\frac{\Delta u}{p'.\varepsilon_s}$ at 50% of Δq_f

The S_{50} parameter of virgin and disturbed samples

TABLE 7.7a THE PORE PRESSURE PARAMETERS OF VIRGIN AND DISTURBED SAMPLES IN DIFFERENT UNDRAINED TESTS

Test	virgin	perfect	Tube T	Tube M	Tube H
7	-	.18	.31	.31	.13
6	1.03	.66	.30	.30	.38
8	.19	.45	-	-	.35
9	.83	.91	.36	.50	.27

NB. All measurements are $\Delta u_f/\Delta q_f$ at failure

TABLE 7.7 b THE A_f PARAMETER OF VIRGIN AND DISTURBED SAMPLES

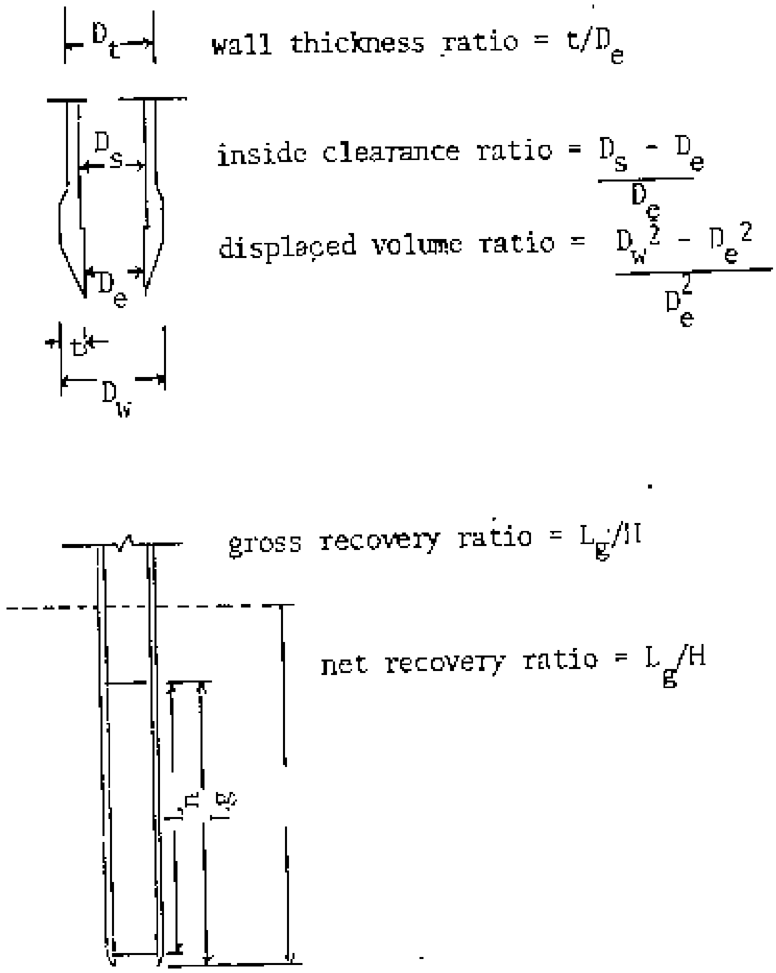


FIG. 1.1 DIMENSIONS, MEASUREMENTS AND RATIOS IN TUBE SAMPLERS

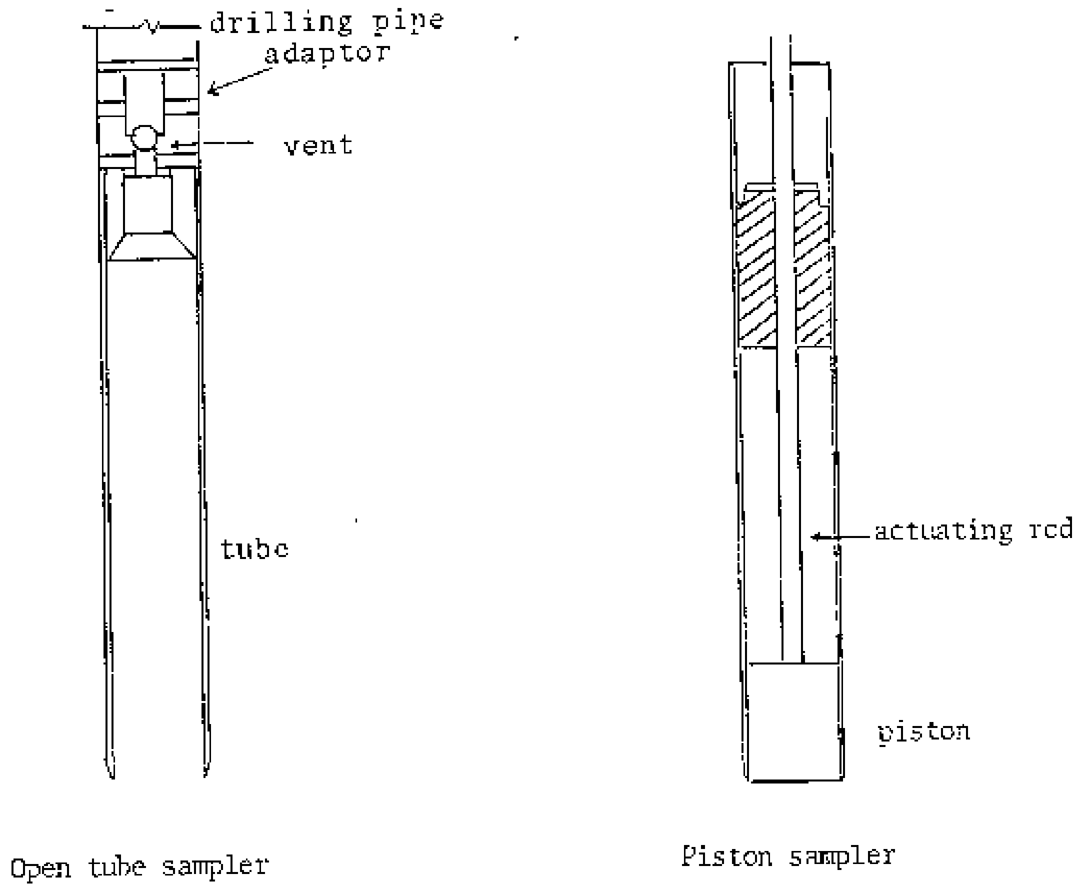


FIG. 1.2 MAIN TYPES OF TUBE SAMPLERS

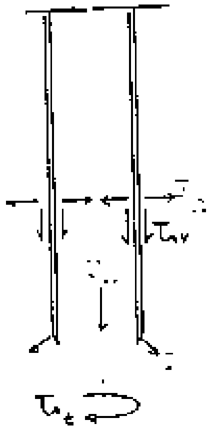
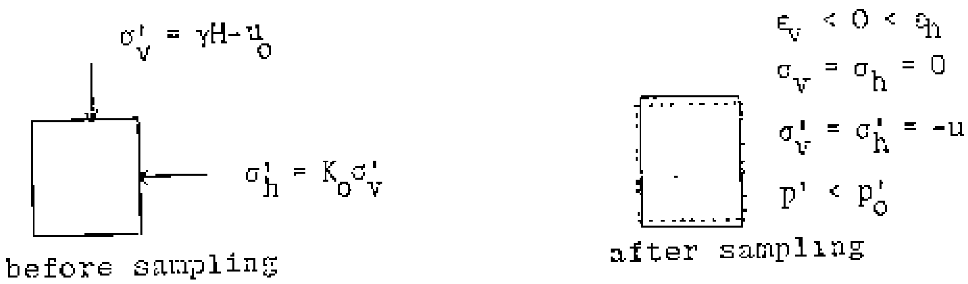
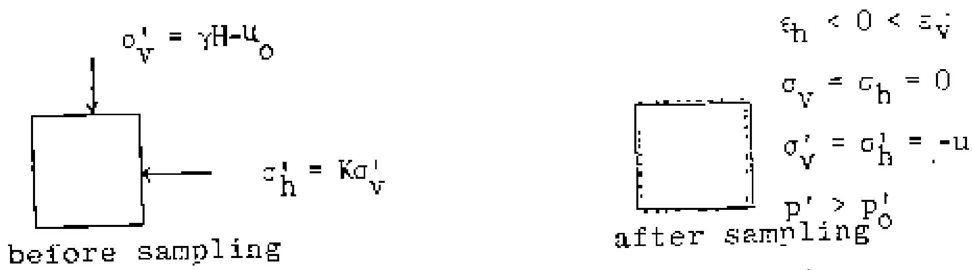


FIG. 1.3 FORCES ACTING DURING SAMPLING OPERATIONS



(a) element of normally consolidated soil before and after sampling



(b) element of heavily overconsolidated soil before and after sampling

FIG. 1.4 CHANGES OF STRESSES AND STRAINS IN PERFECT SAMPLING

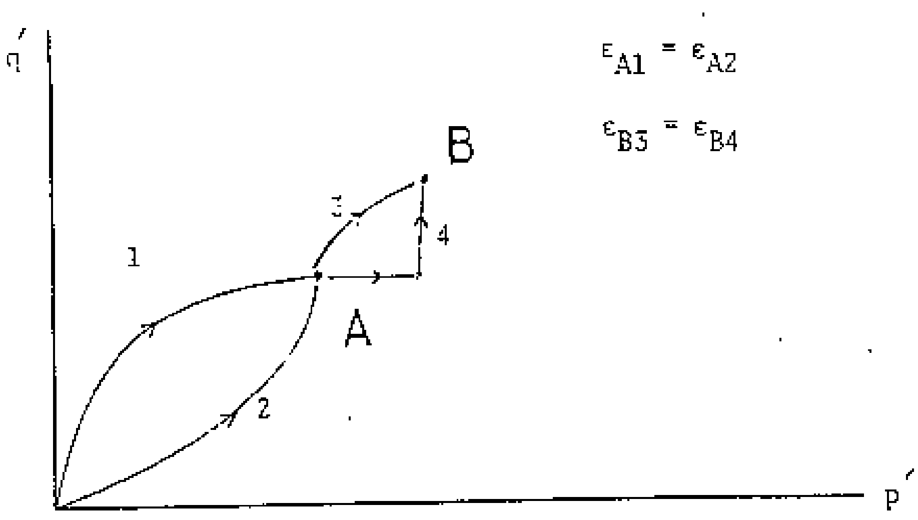


FIG. 2.1 PATH INDEPENDENCY IN ELASTIC MATERIALS

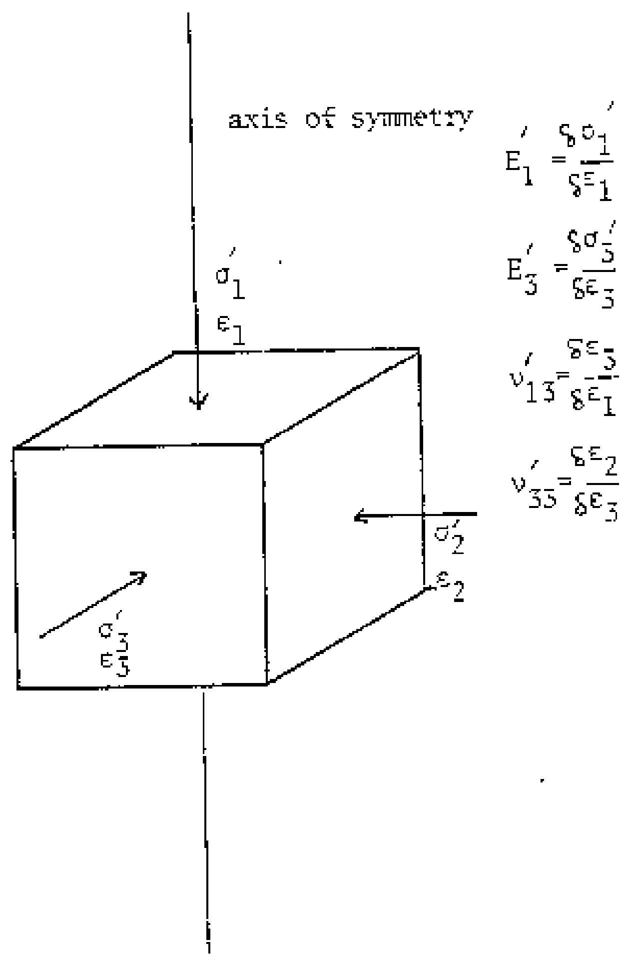


FIG. 2.2 STRESSES AND STRAINS IN AN ELEMENT OF ANISOTROPIC ELASTIC MATERIAL

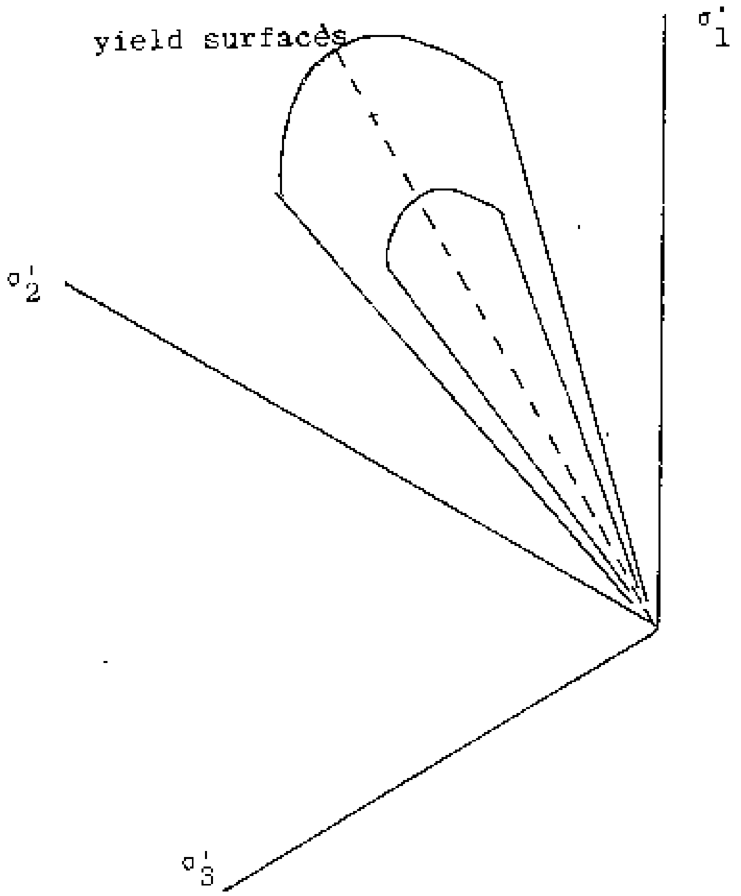


FIG. 2.3. YIELD SURFACES IN A THREE DIMENSIONAL STRESS SPACE

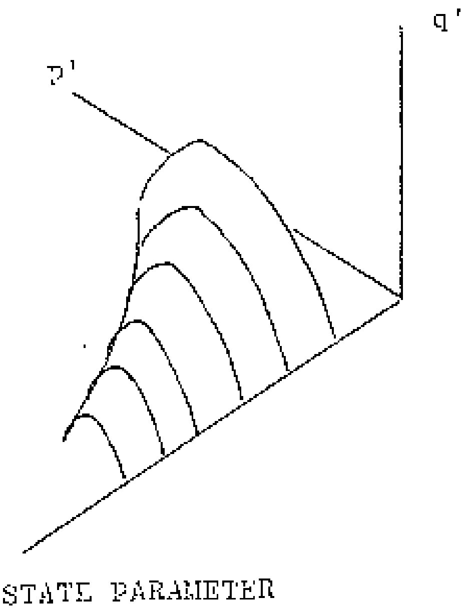


FIG. 2.4. HYPOTHETICAL STATE BOUNDARY SURFACE

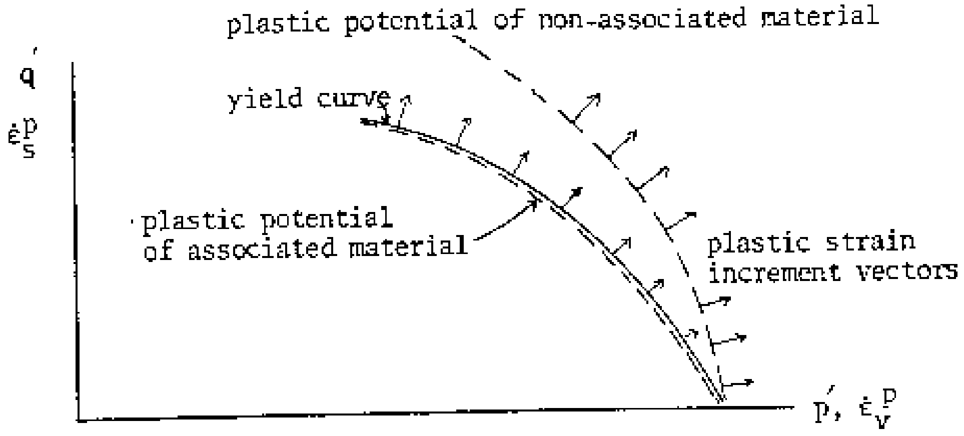


FIG. 2.5 PLASTIC POTENTIALS OF ASSOCIATED AND NON-ASSOCIATED MATERIALS

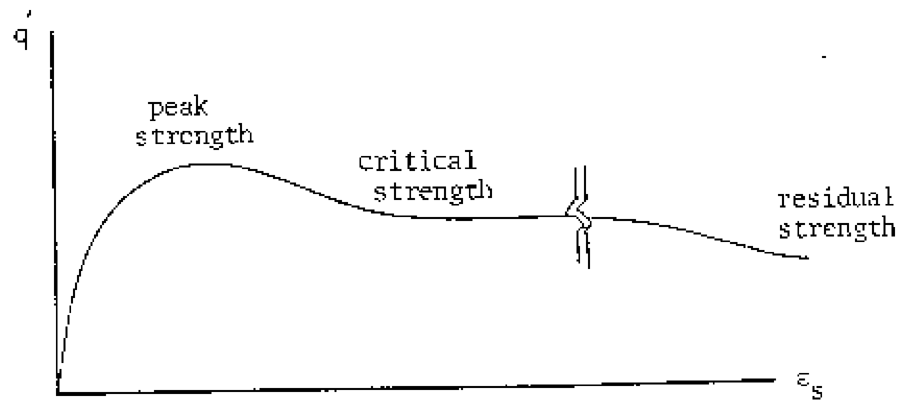


FIG. 2.6 THE STRENGTH OF SOILS

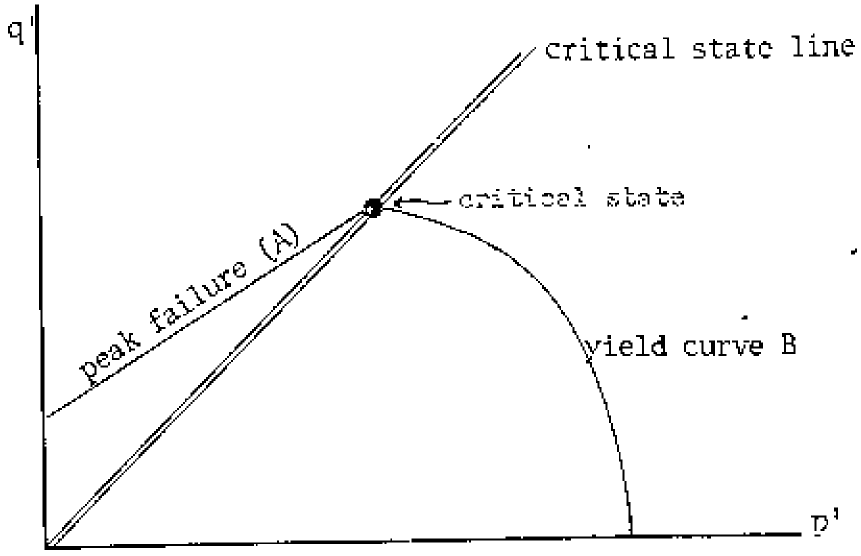


FIG. 2.7 FAILURE AND YIELD CURVES

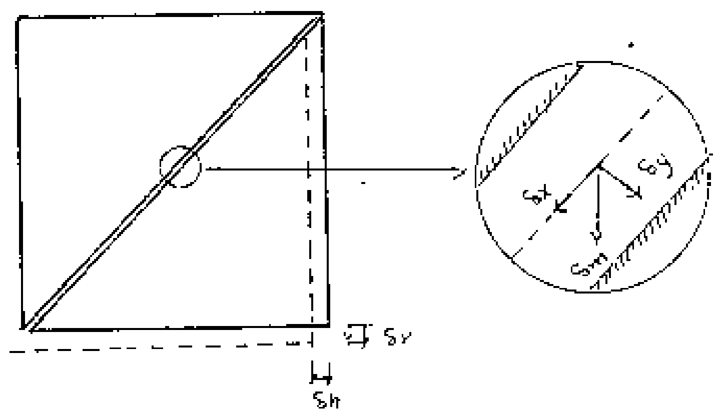


FIG. 2.8 DEFORMATION AT THE BOUNDARIES AND WITHIN THE SLIP ZONE OF AN ELEMENT OF SOIL

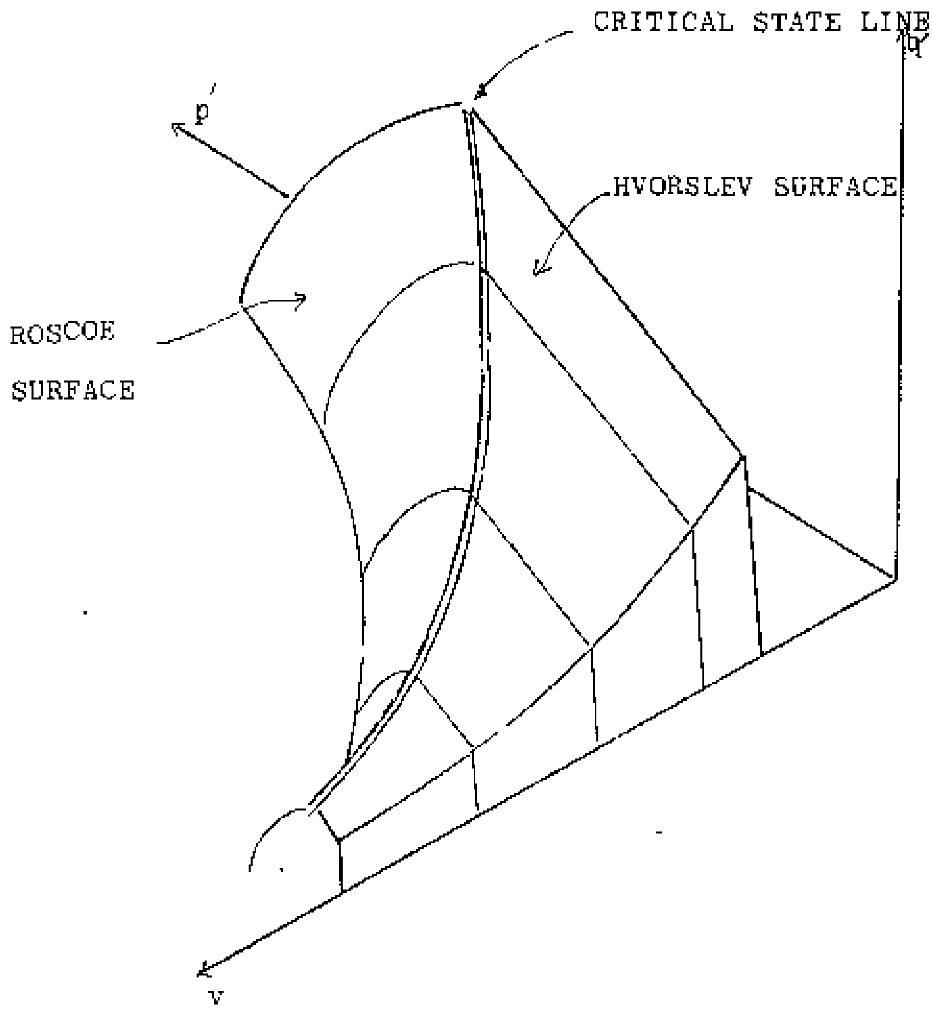


FIG. 2.9 STATE BOUNDARY SURFACE FOR SOILS

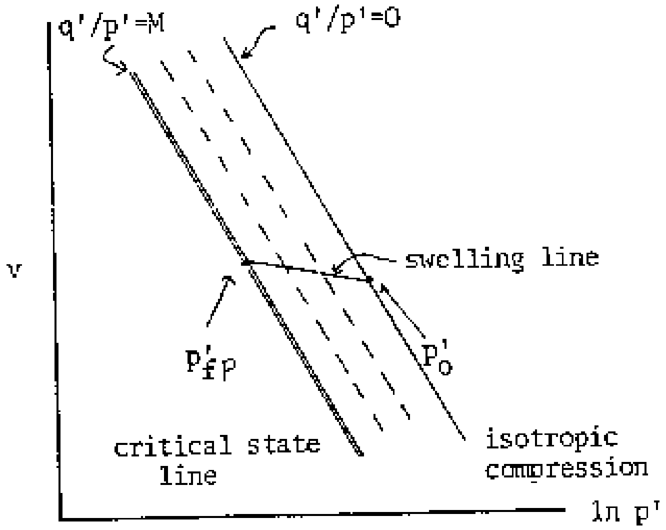


FIG. 2.10 COMPRESSION CURVES IN SOILS

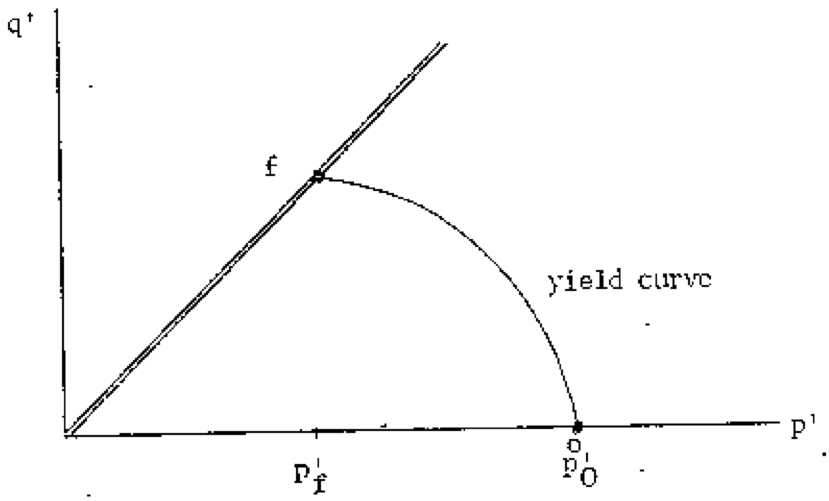


FIG. 2.11 INTERSECTION OF THE YIELD CURVE WITH THE ISOTROPIC COMPRESSION AND THE CRITICAL STATE LINES

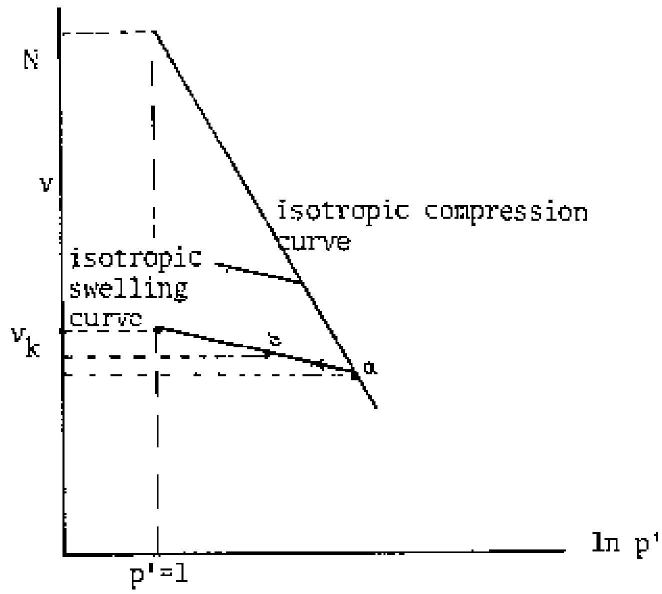


FIG. 2.12 COMPRESSION AND SWELLING CURVES

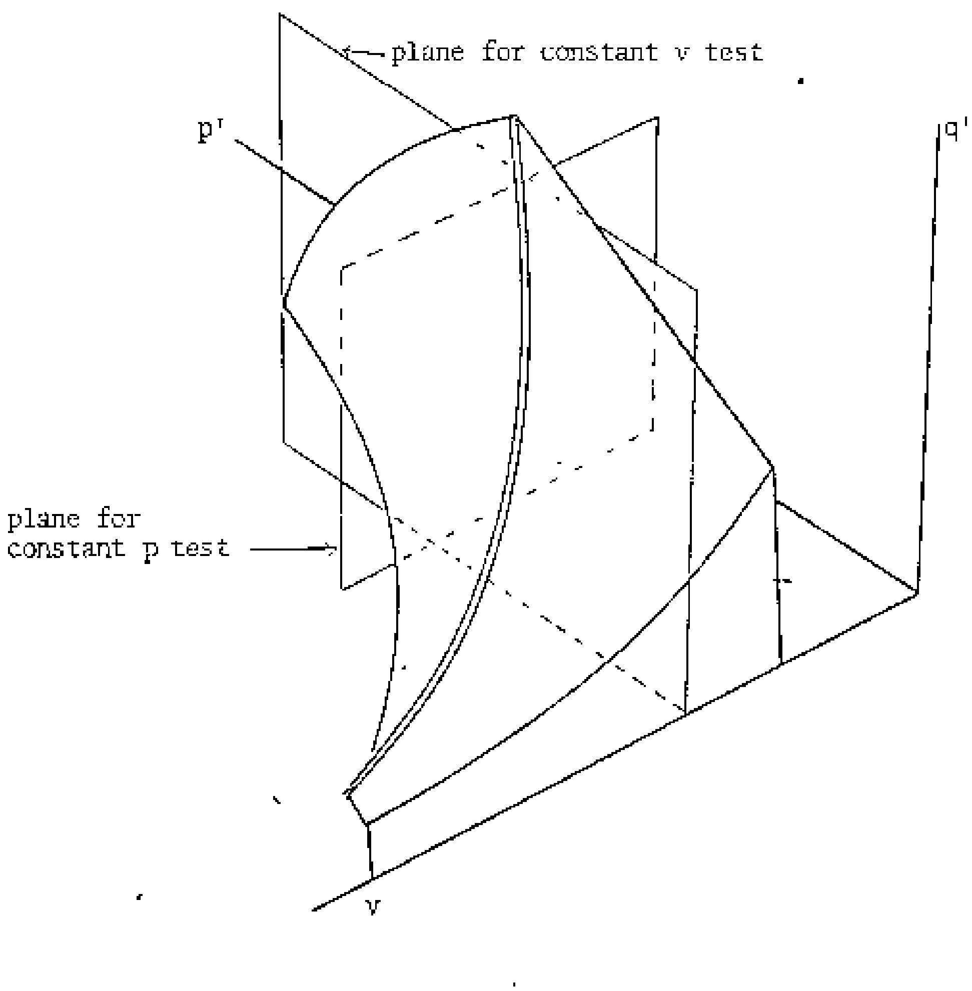


FIG. 2.13(a) LOADING PLANES AS BOUNDARY CONDITIONS

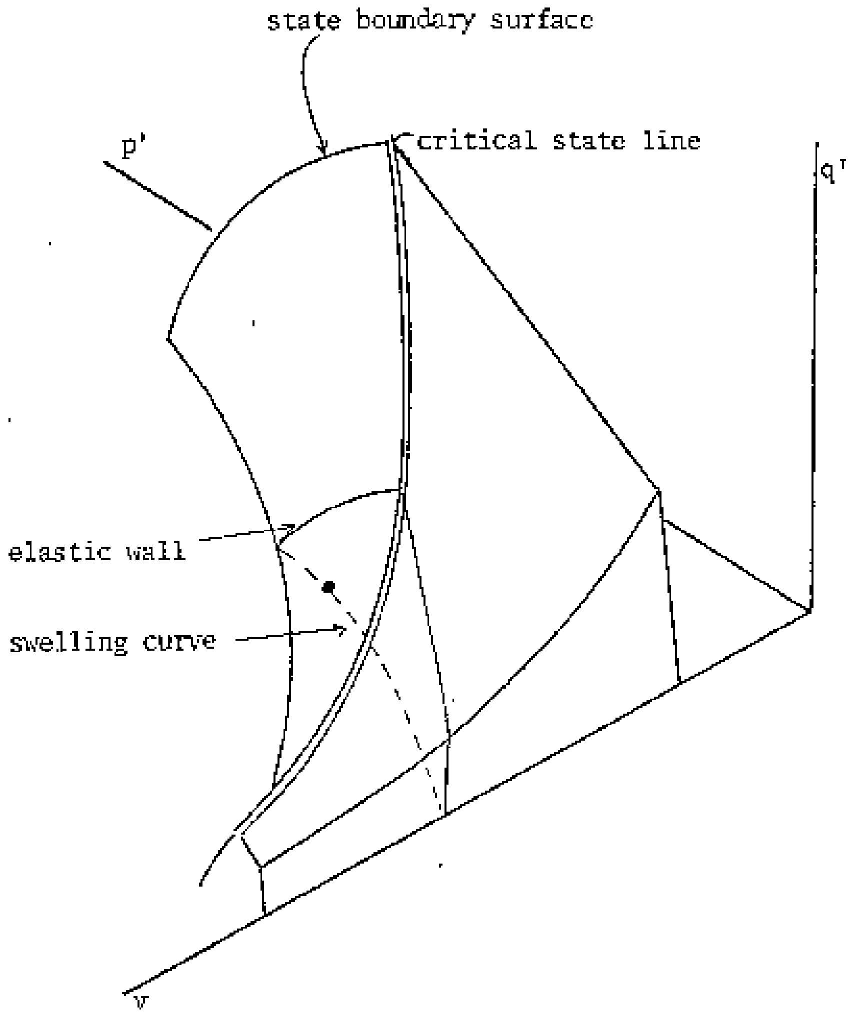
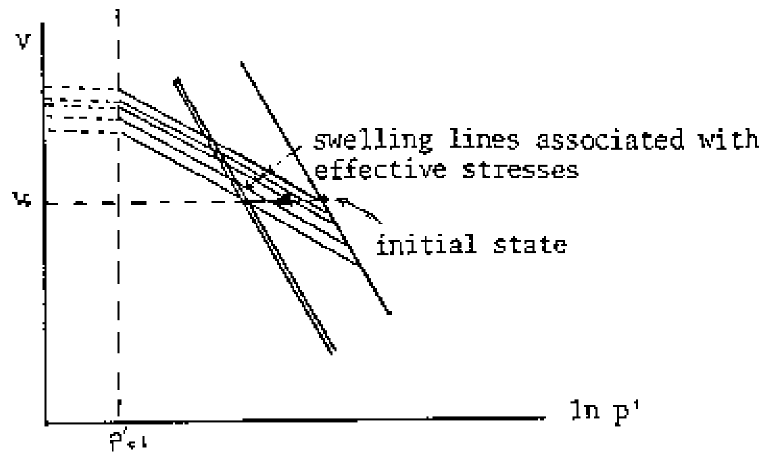
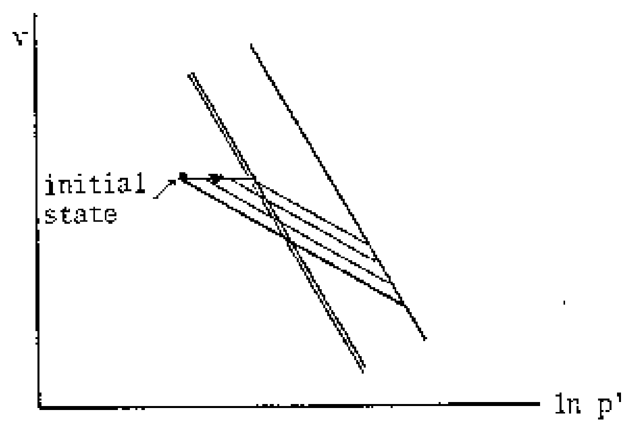


FIG. 2.13(b) ELASTIC WALL WITHIN THE STATE BOUNDARY SURFACE



(a) Normally consolidated element hardens in an undrained test



(b) Heavily overconsolidated element softens in an undrained test

FIG. 2.14 HARDENING OR SOFTENING IN UNDRAINED TESTS

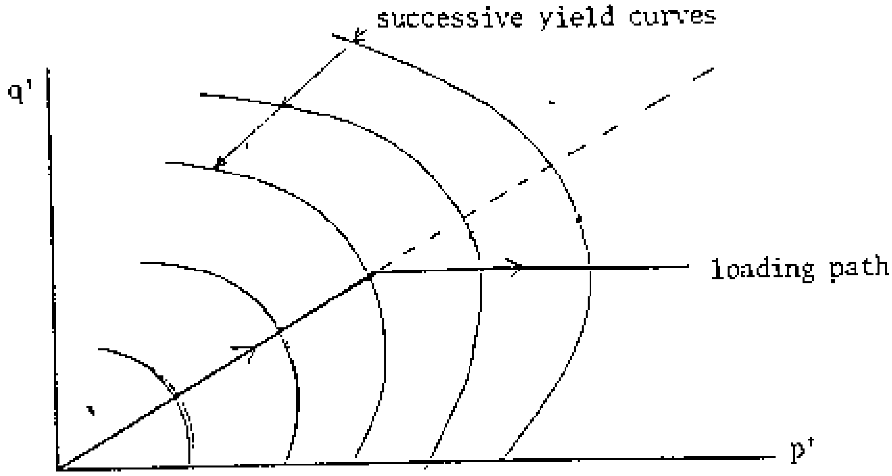


FIG. 2.15 KINEMATIC HARDENING

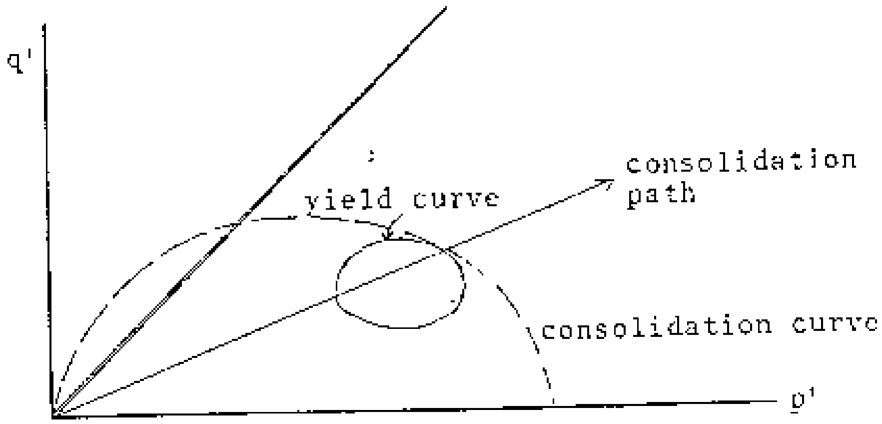


FIG. 2.16 DOUBLE YIELD CURVE (After Mroz et al 1979)

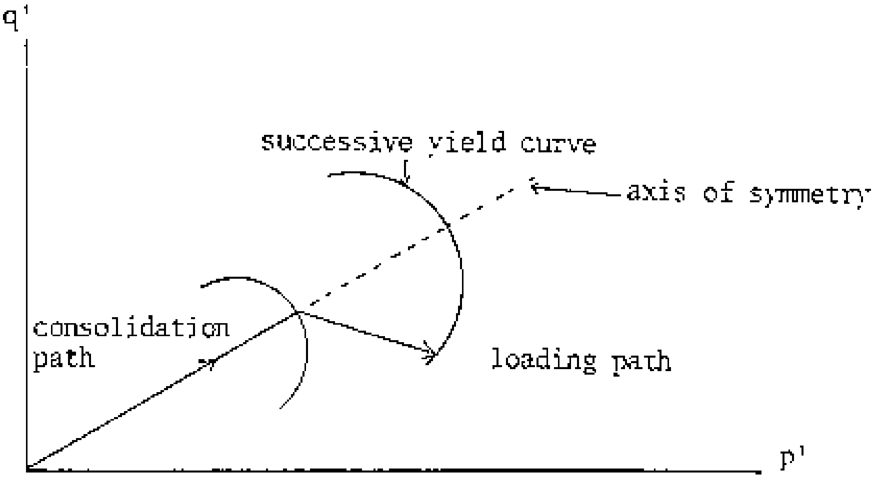
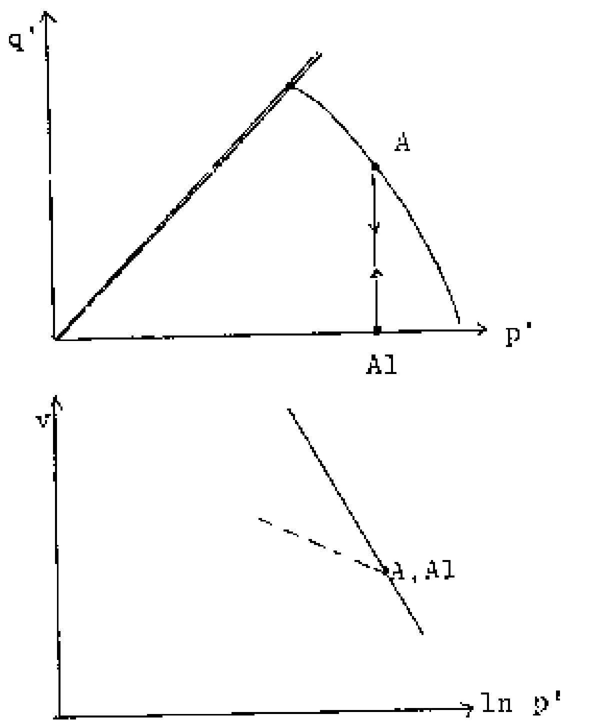
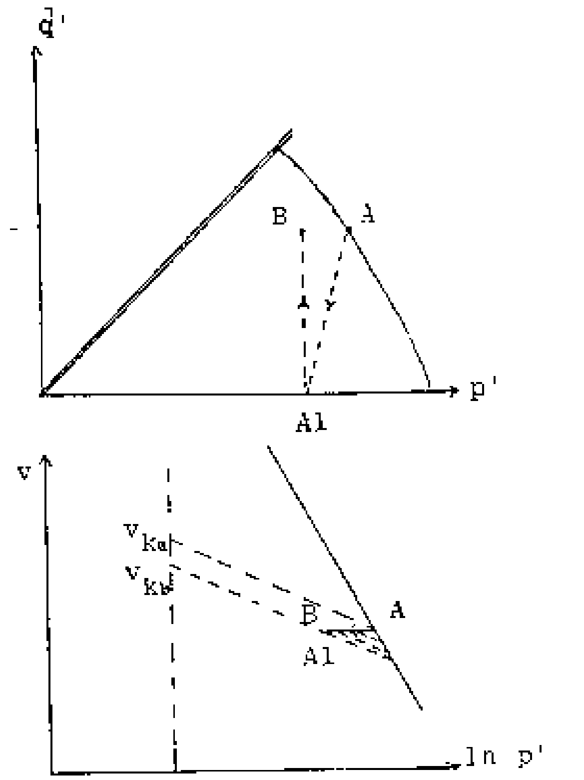


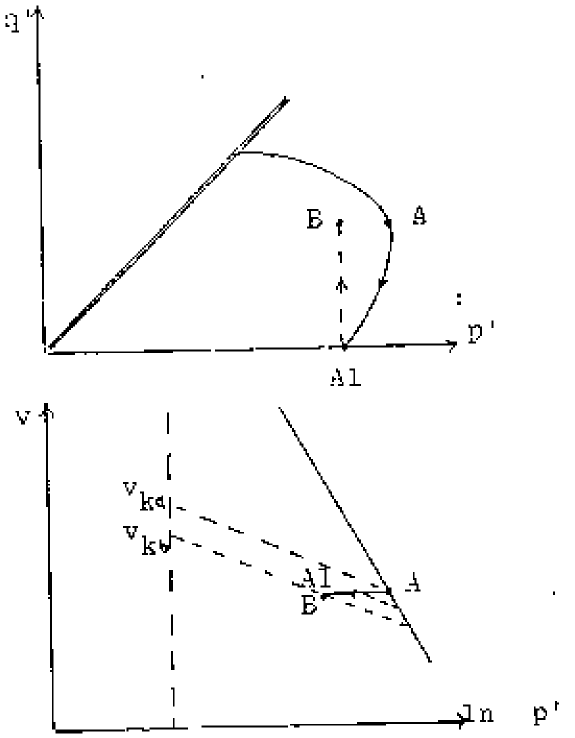
FIG. 2.17 THE EFFECTS OF THE CONSOLIDATION PATH AND THE LOADING PATH ON THE SHAPE OF THE YIELD CURVE



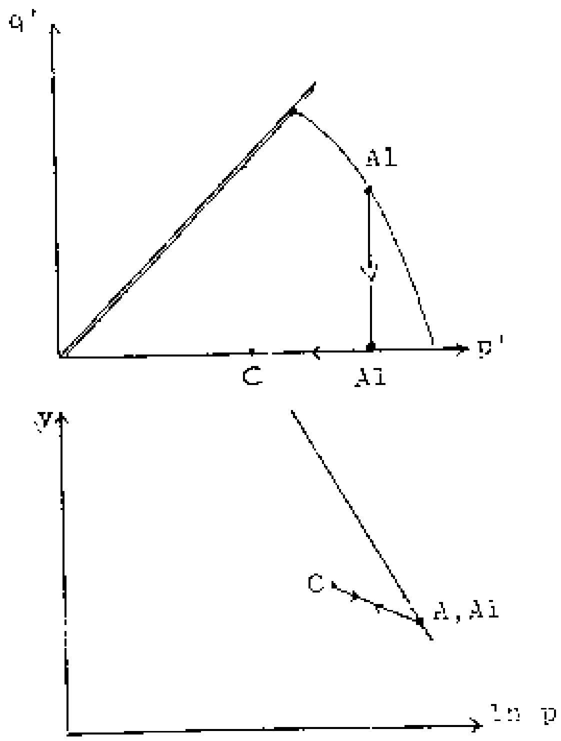
(a) Elastic behaviour in simple Cam clay



(b) In-elastic behaviour within the state boundary surface



(c) Elasto-plastic behaviour in a modified Cam clay



(d) Elastic behaviour in swelling

FIG. 2.18 CHANGE OF STATE OF NORMALLY CONSOLIDATED SAMPLES IN UNLOADING AND RELOADING

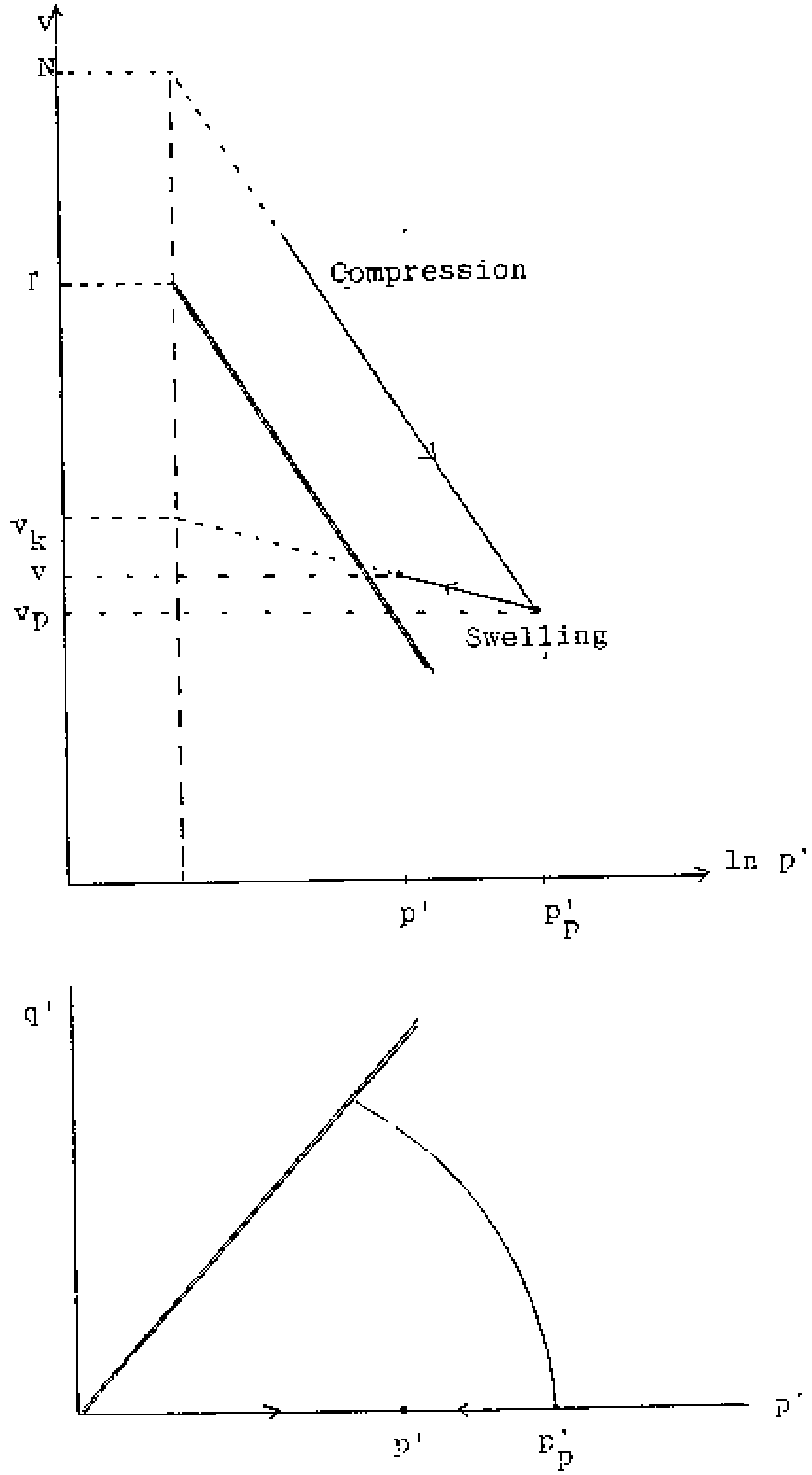


FIG. 2.19. THE STATE OF AN OVERCONSOLIDATED SAMPLE

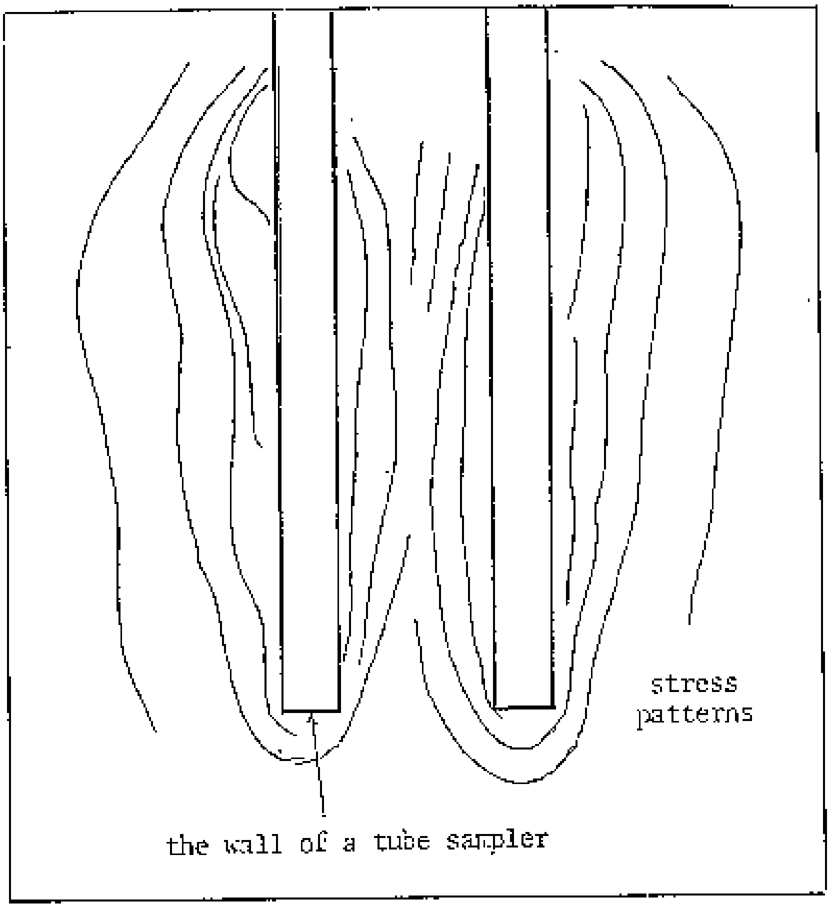


FIG. 2.20 A SKETCH OF THE STRESS PATTERNS DEVELOPED DURING TUBE SAMPLING (After Arman and McManis 1977)

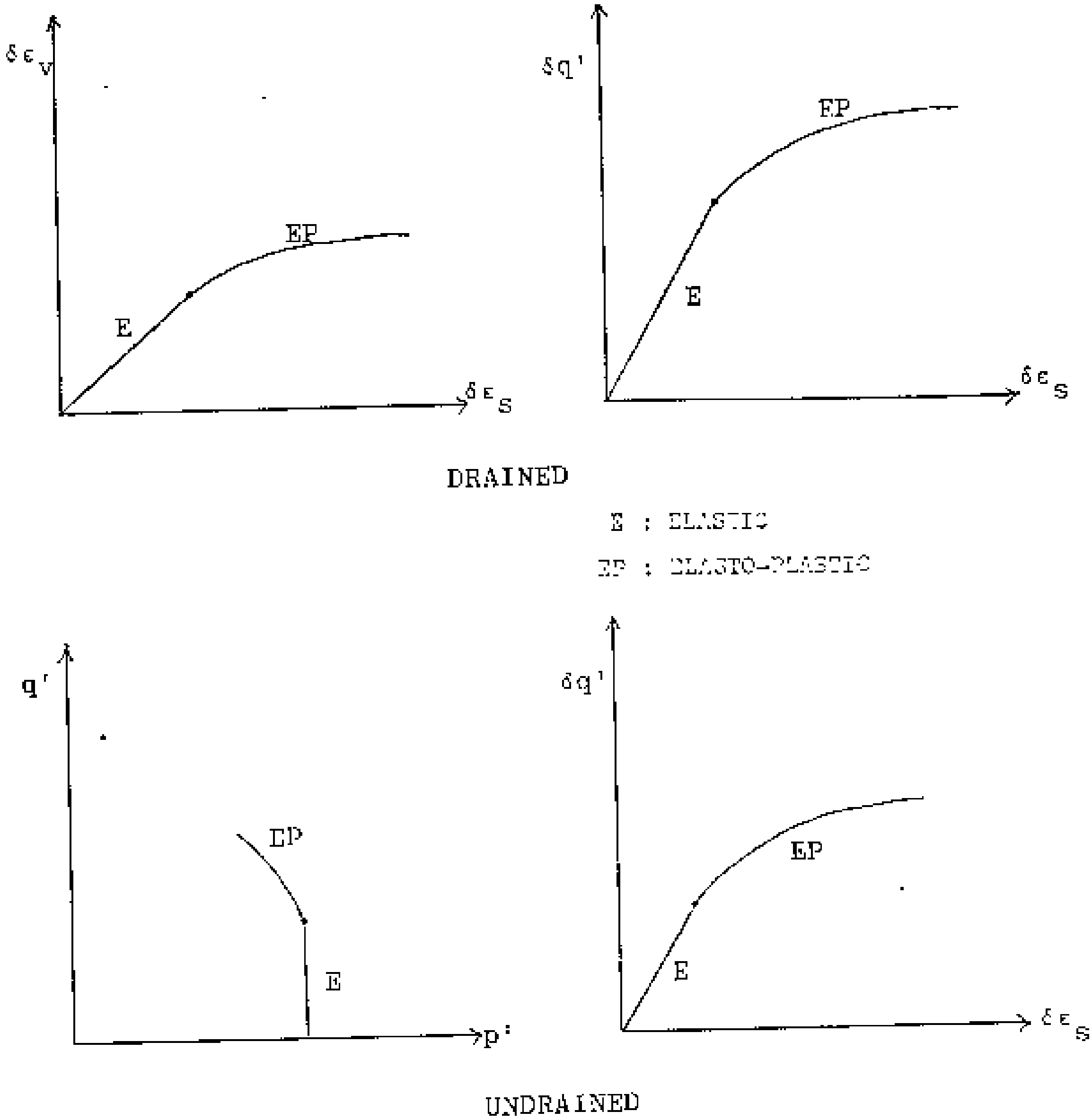
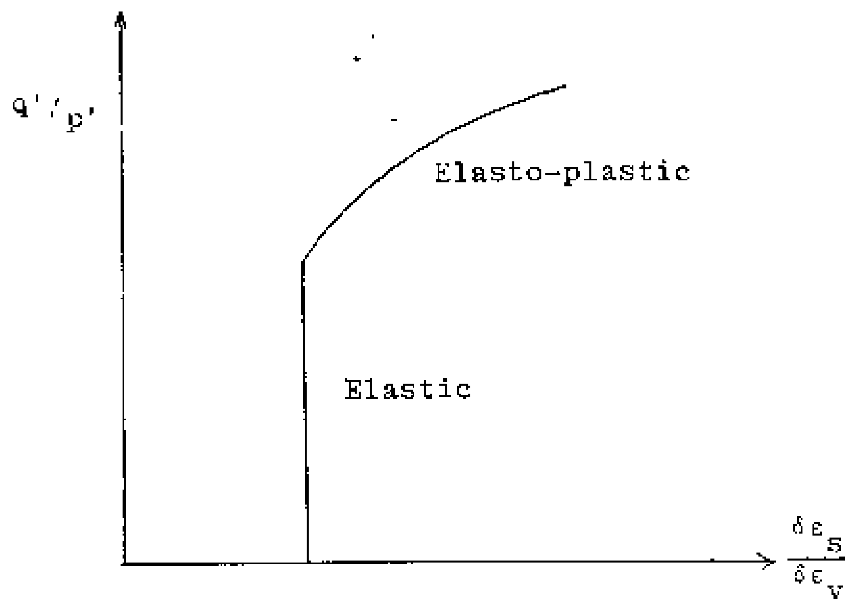
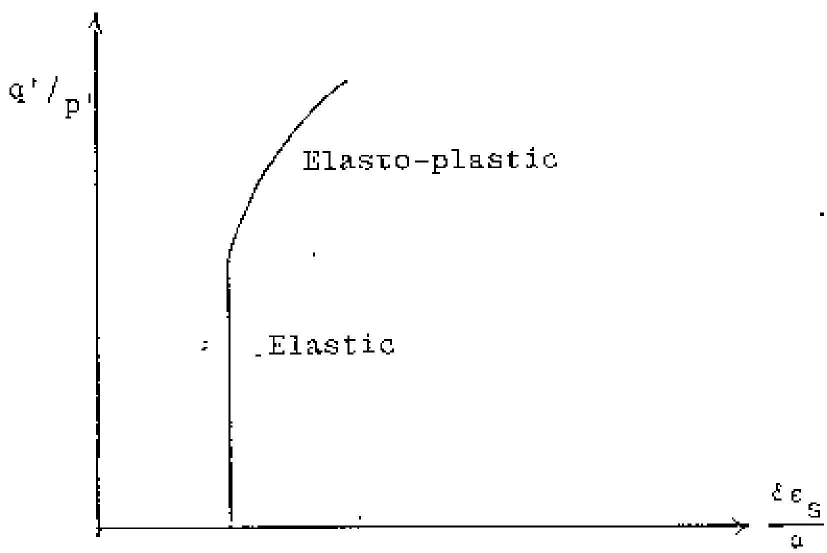


FIG. 2.21 (a) IDEALISED BEHAVIOUR OF AN OVERCONSOLIDATED SAMPLE



DRAINED



UNDRAINED

FIG. 2.21 (b) IDEALISED BEHAVIOUR OF AN OVERCONSOLIDATED SAMPLE

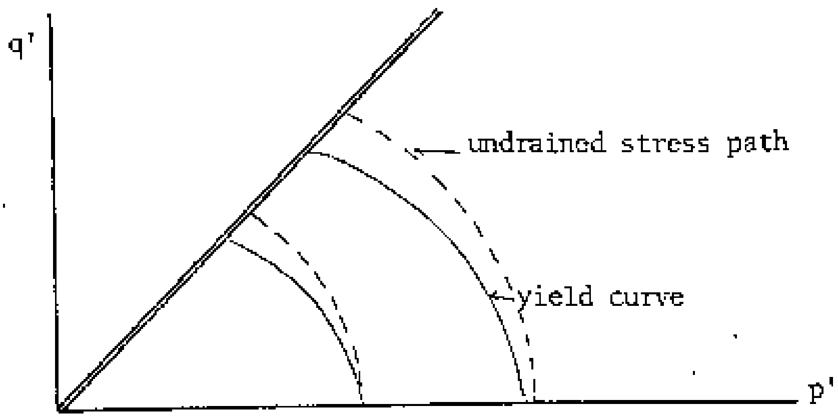


FIG. 2.22 YIELD CURVES AND STRESS PATHS AT DIFFERENT STRESS LEVELS

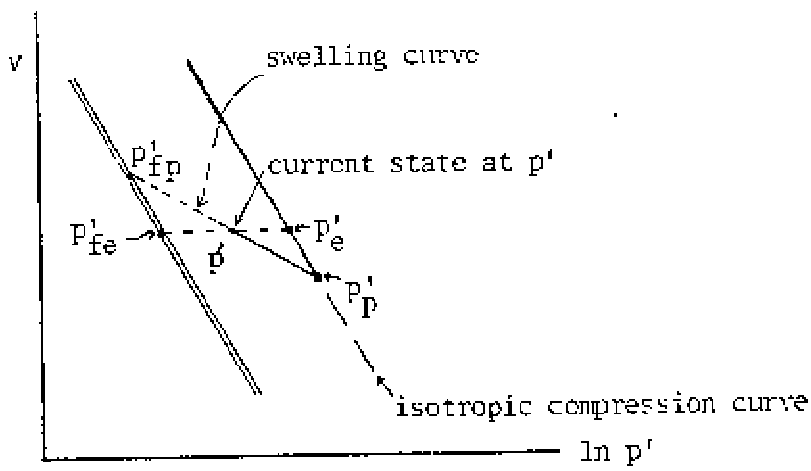


FIG. 2.23 DIFFERENT NORMALISING PARAMETERS

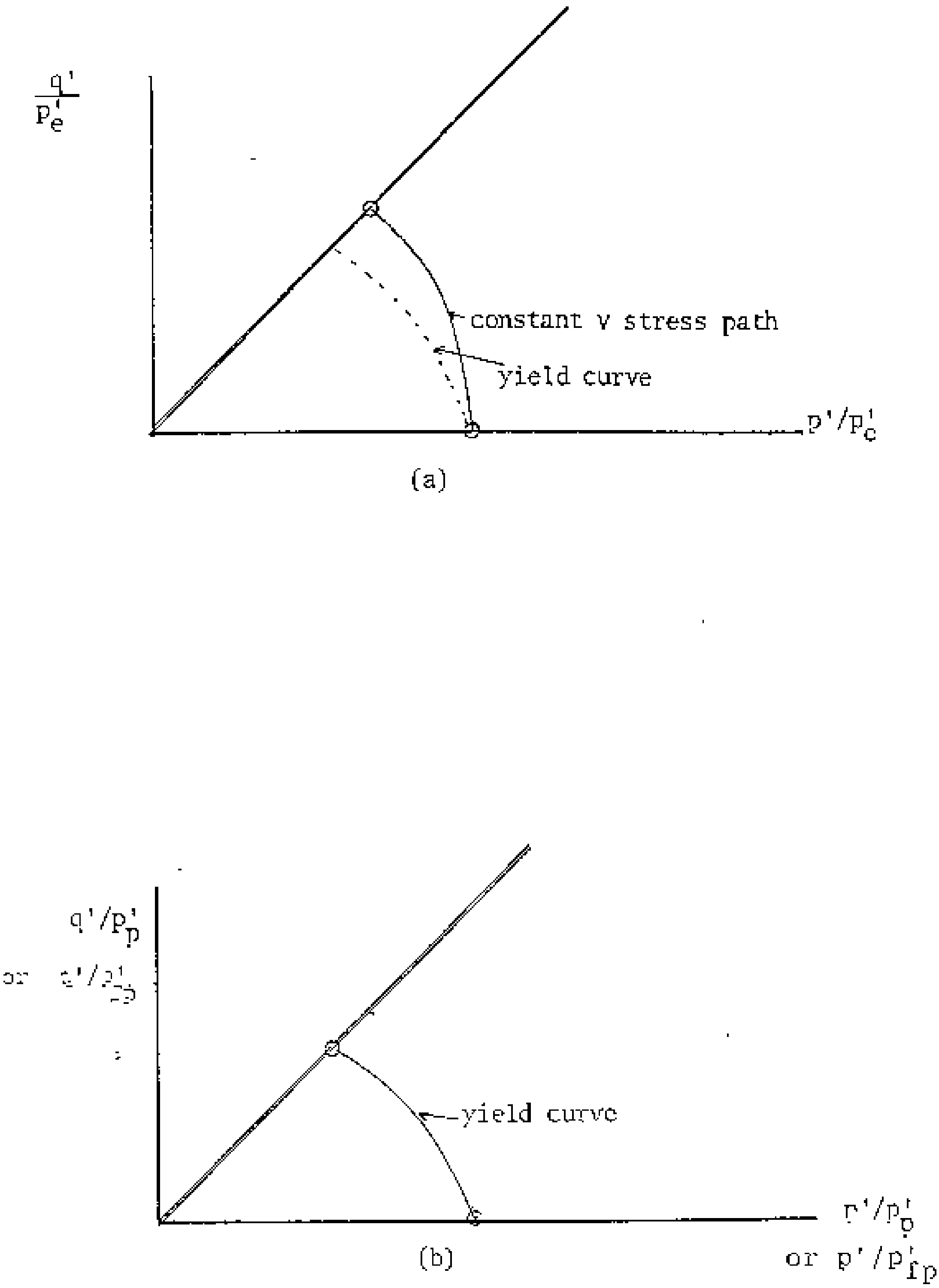
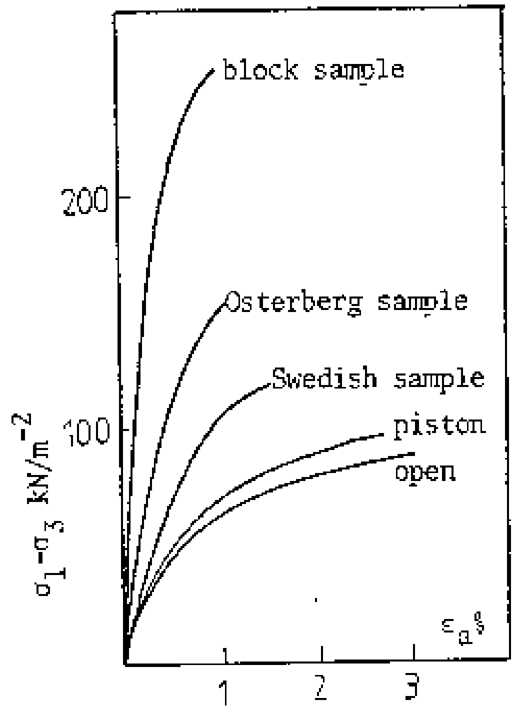
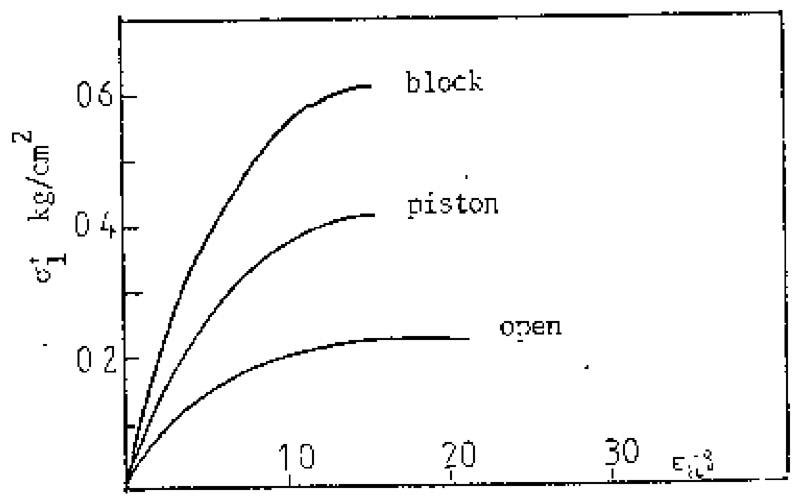


FIG. 2.24 NORMALISED STRESS PATHS IN DRAINED AND UNDRAINED TESTS

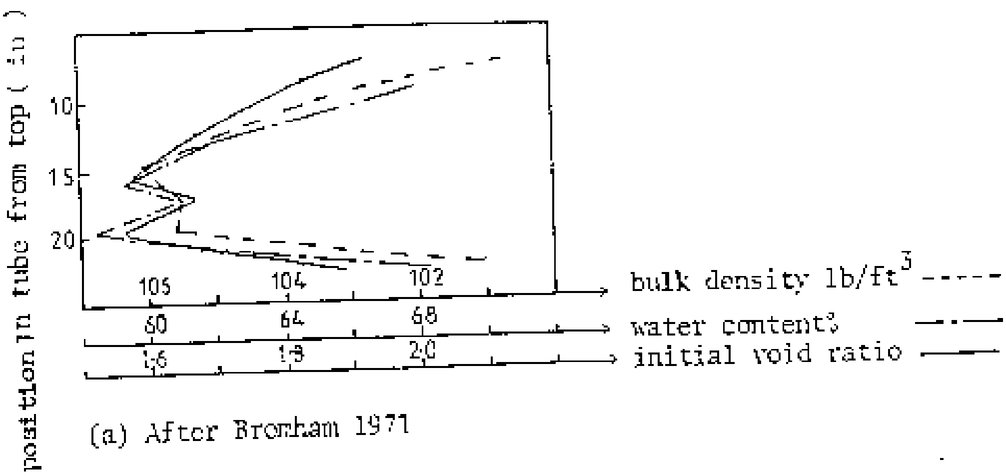


(a) After Raymond et al 1971

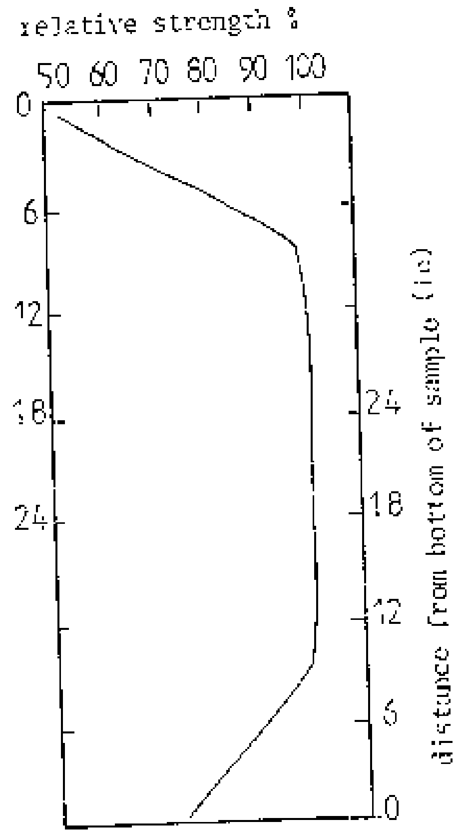


(b) After Milovic 1971

FIG. 3.1 UNCONFINED COMPRESSION TESTS ON BLOCK AND TUBE SAMPLES



(a) After Bromham 1971



(b) After Hvorslev 1949

FIG. 5.2 VARIATION OF DISTURBANCE ALONG TUBE SAMPLES

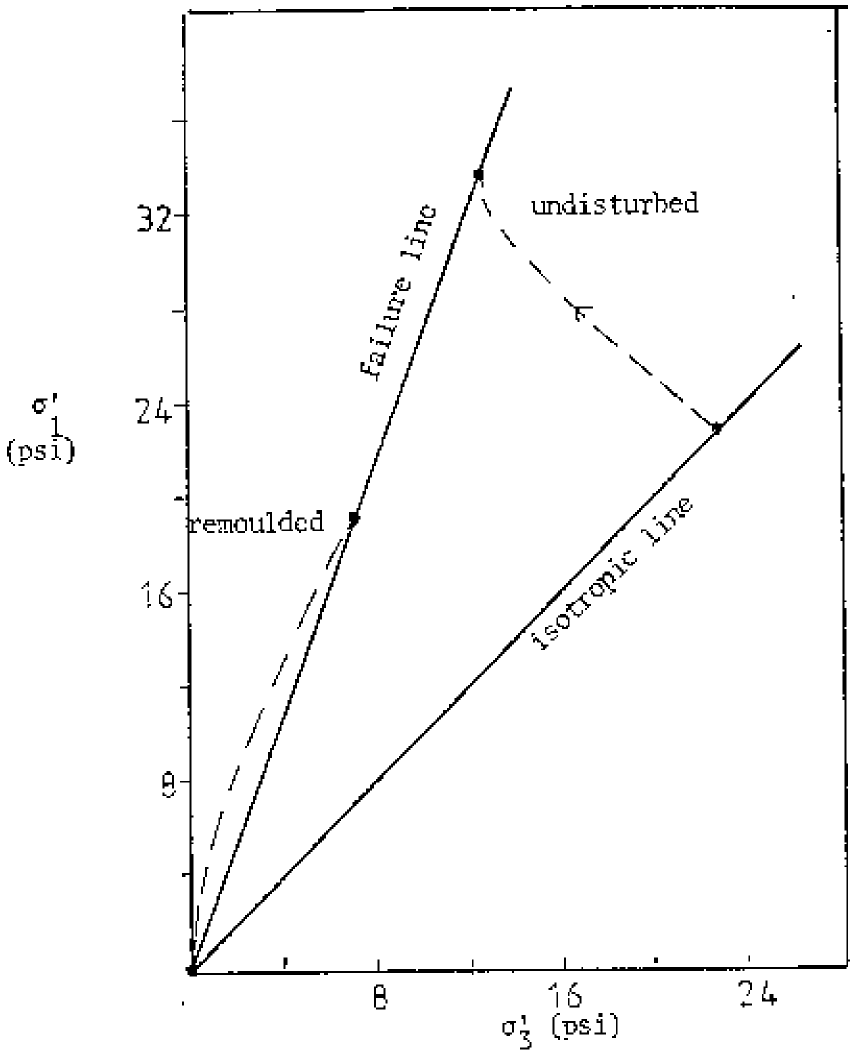


FIG. 3.3 STRESS PATHS OF BLOCK AND REMOULDED SAMPLES
(After Skempton and Sowa 1963)

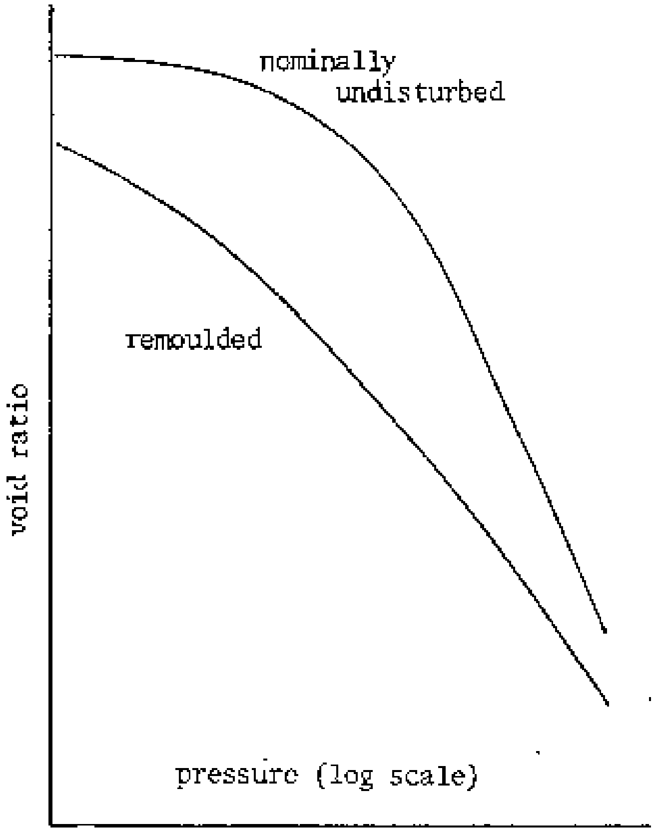


FIG. 3.4 CONSOLIDATION BEHAVIOUR OF TUBE AND REMOULDED SAMPLES (After Schmertmann 1955)

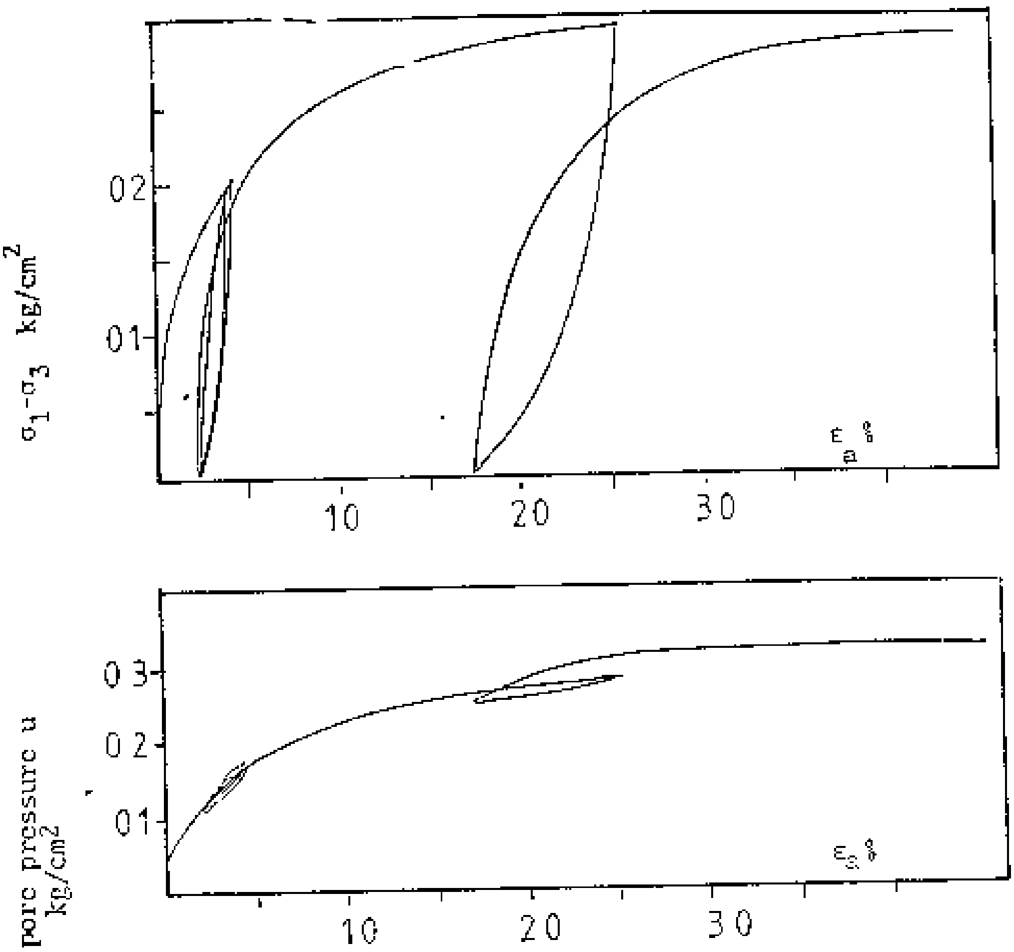


FIG. 3.5 RESULTS OF CYCLIC LOADING ON UNDISTURBED CLAY
(After Lo 1961)

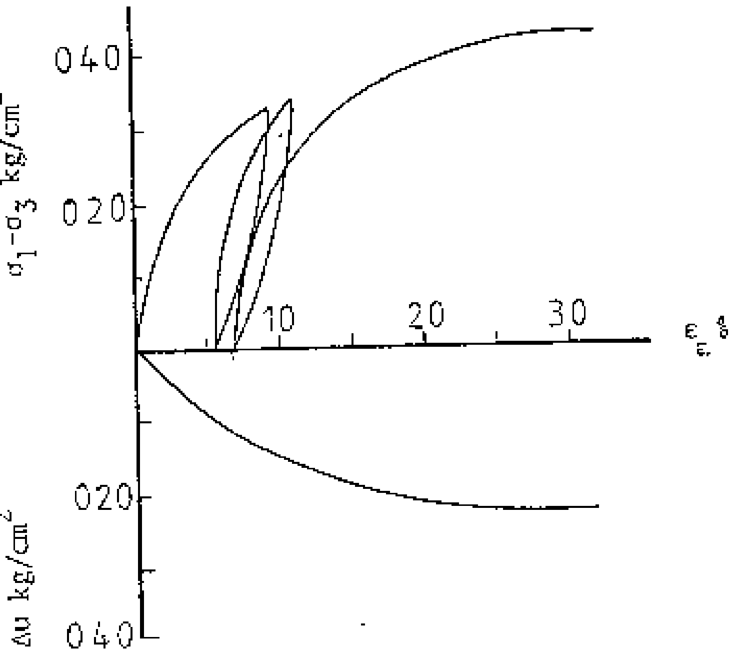


FIG. 3.6 RESULTS OF CYCLIC LOADING ON AN UNDISTURBED SAMPLE (After Lo 1961)

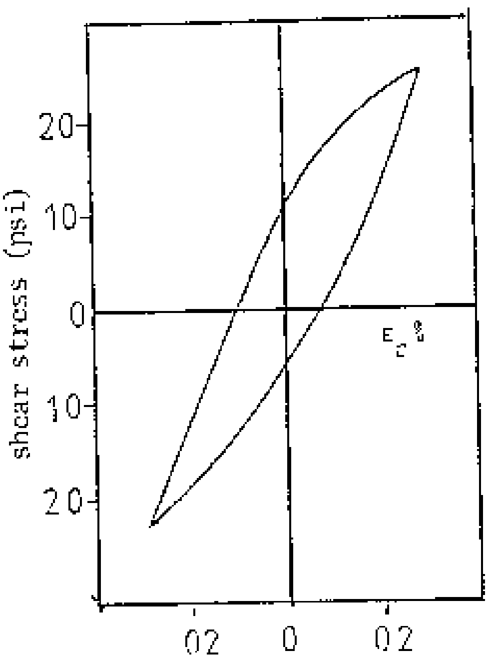


FIG. 3.7 TYPICAL HYSTERISIS LOOP IN CYCLIC LOADING (After Taylor and Bacchus 1969)

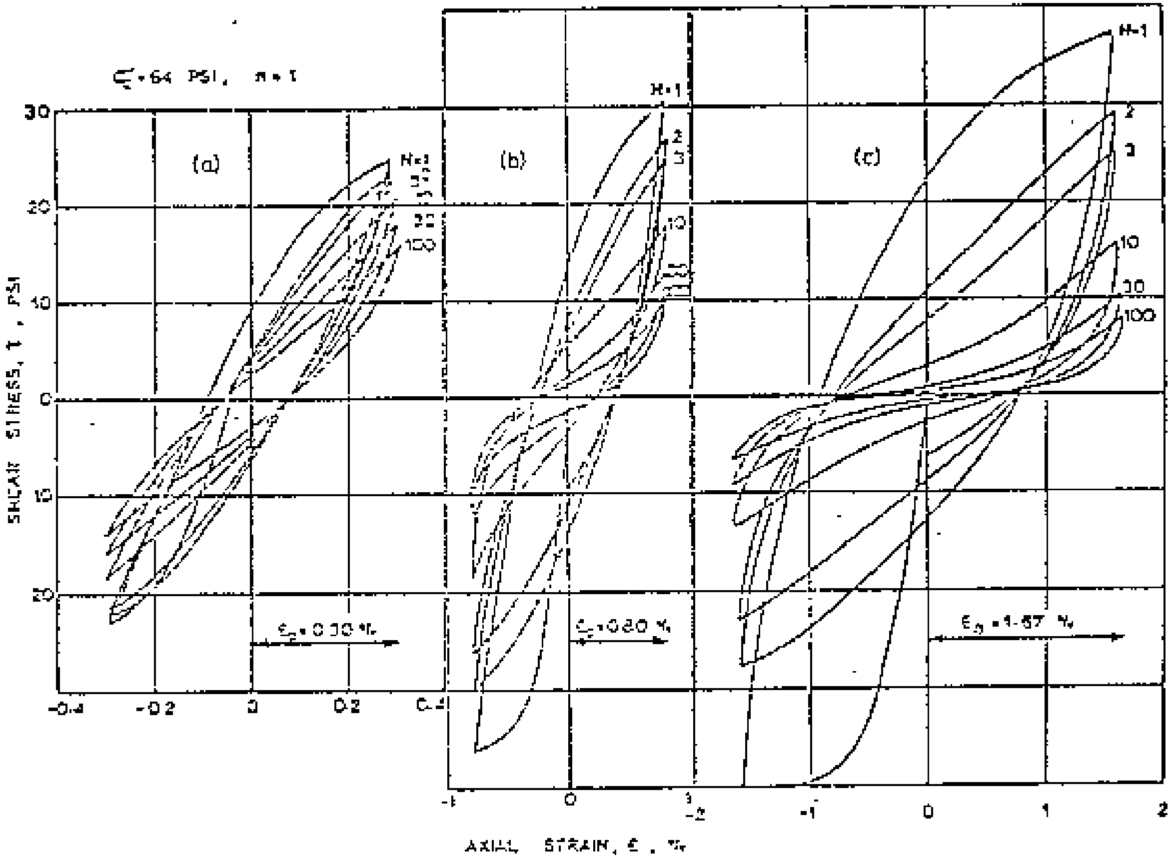


FIG. 3.8. CYCLIC LOADING AT DIFFERENT AMPLITUDES
(After Taylor and Bacchus 1969)

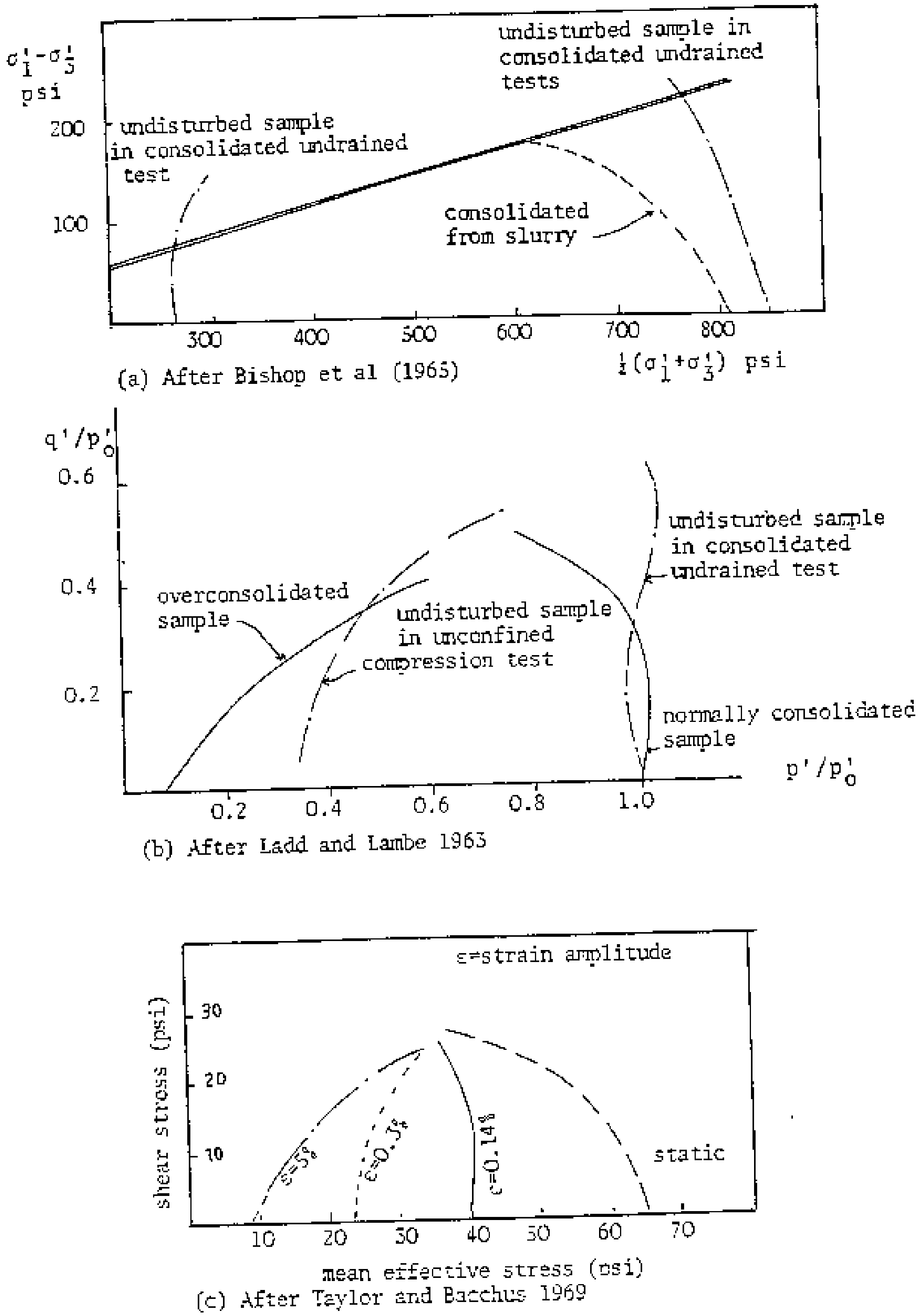


FIG. 3.9. STRESS PATHS OF DISTURBED SAMPLES AND VIRGIN SAMPLES IN DIFFERENT TESTS

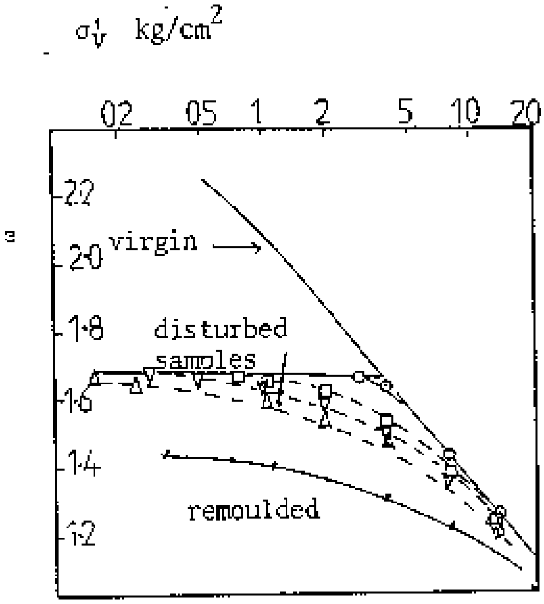


FIG. 3.10 CONSOLIDATION BEHAVIOUR OF CYCLICALLY DISTURBED SAMPLES (After Okumura 1971)

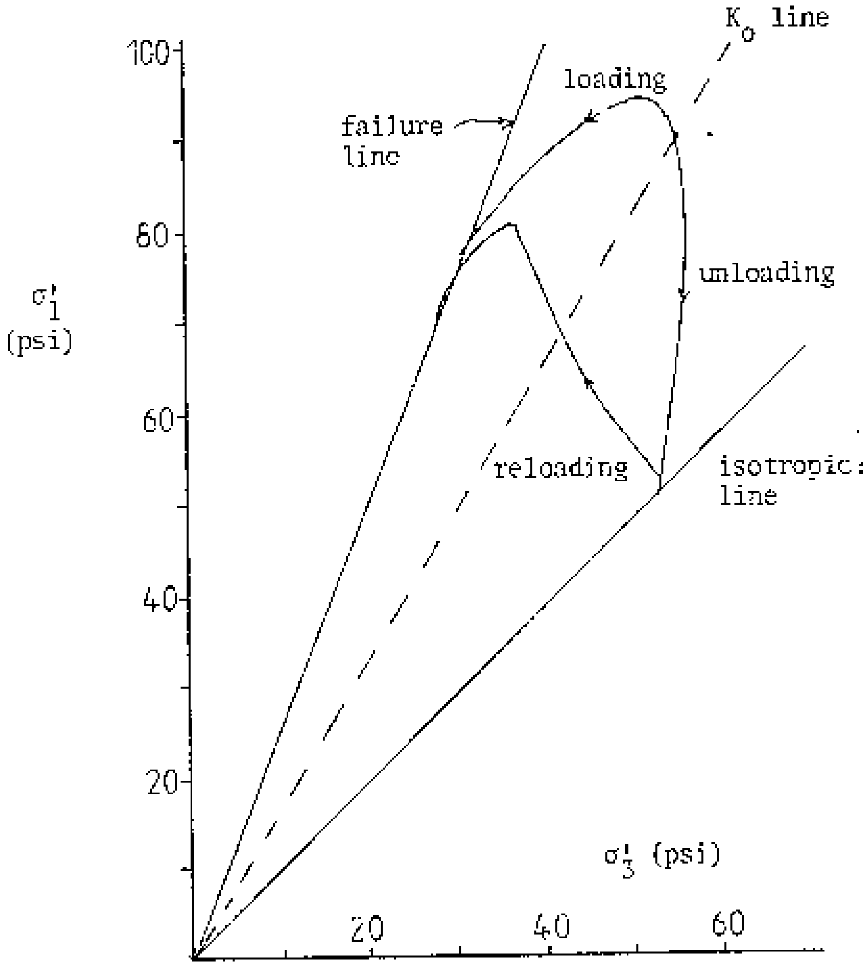
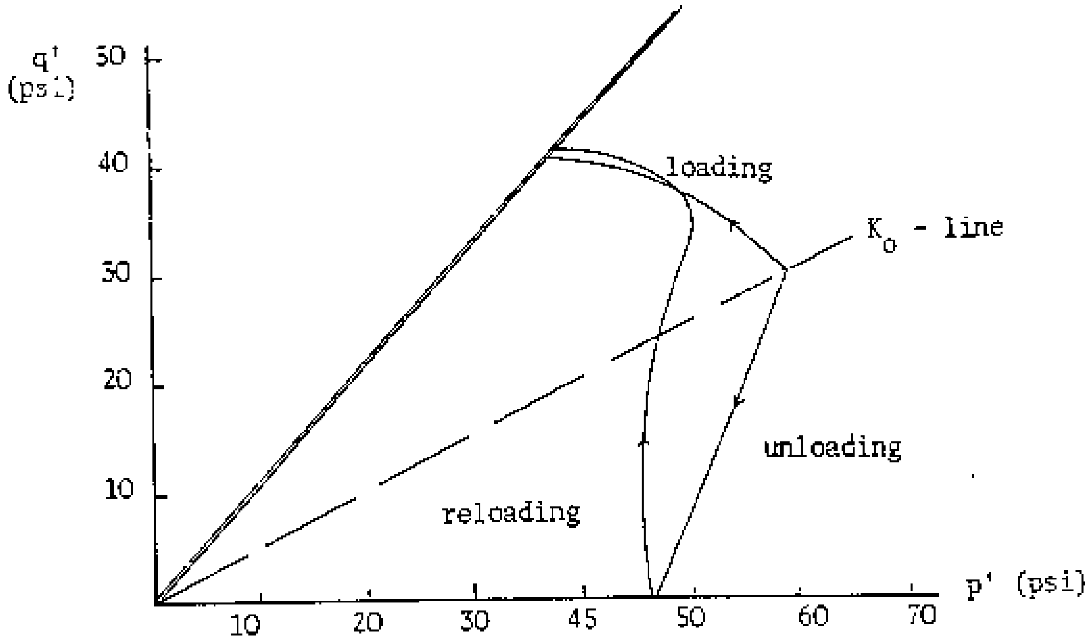
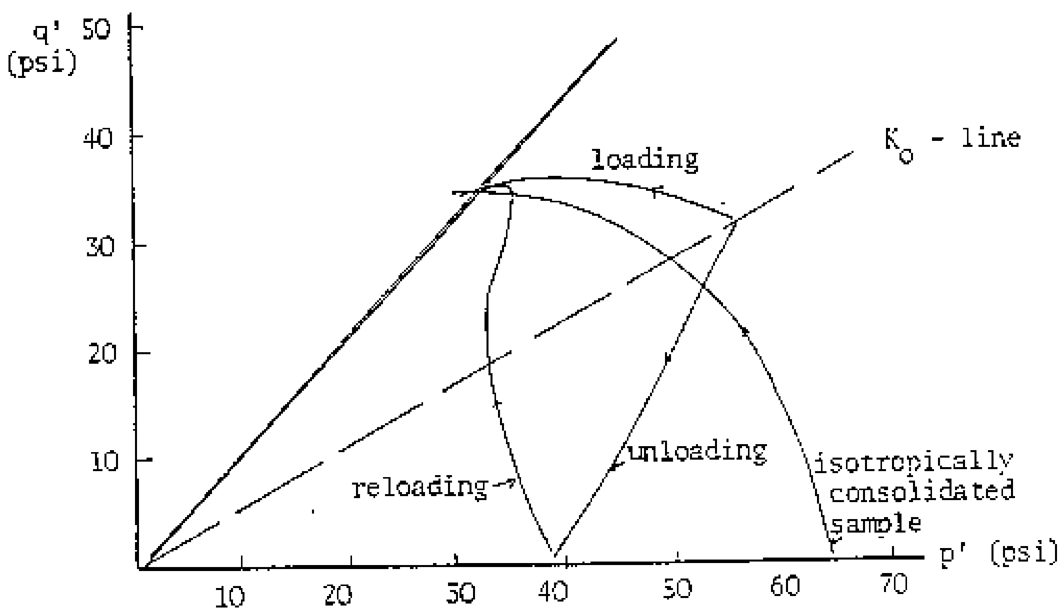


FIG. 3.11 STRESS PATHS IN PERFECT SAMPLING (After Skempton and Sowa 1963)



(a) Tests on block samples



(b) Tests on slurry samples

FIG. 3.12 STRESS PATHS IN PERFECT SAMPLING
(After Adams and Radhakrishna 1971)

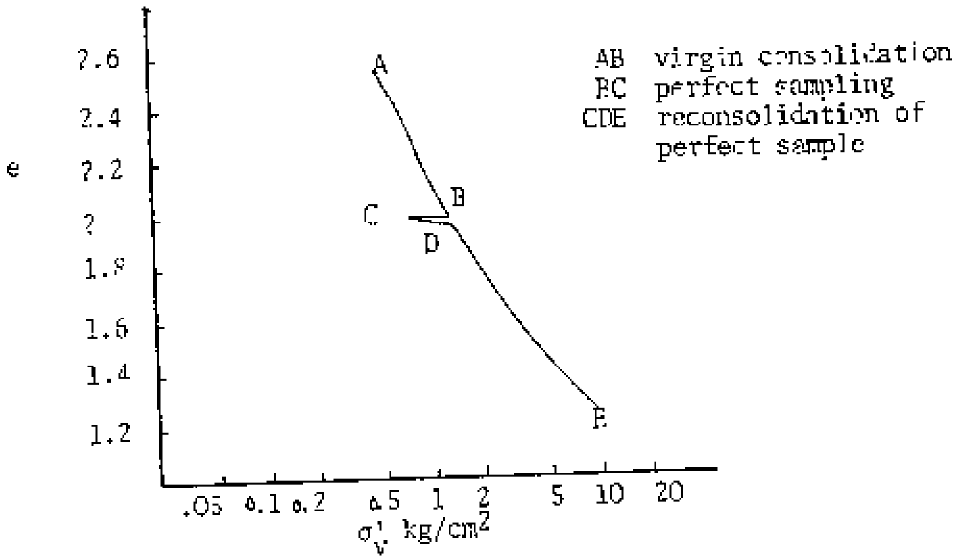


FIG. 3.13 CONSOLIDATION BEHAVIOUR OF A PERFECT SAMPLE (After Noorany and Poormand 1973)

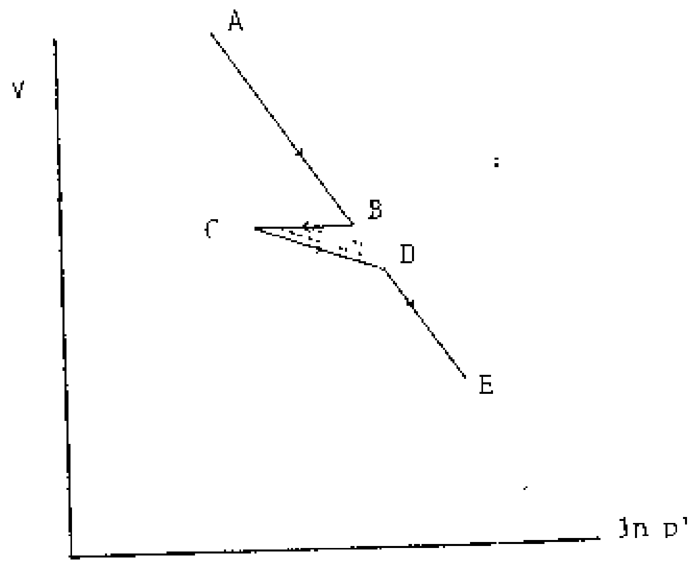


FIG. 3.14 PREDICTED BEHAVIOUR OF A PERFECT SAMPLE IN CONSOLIDATION

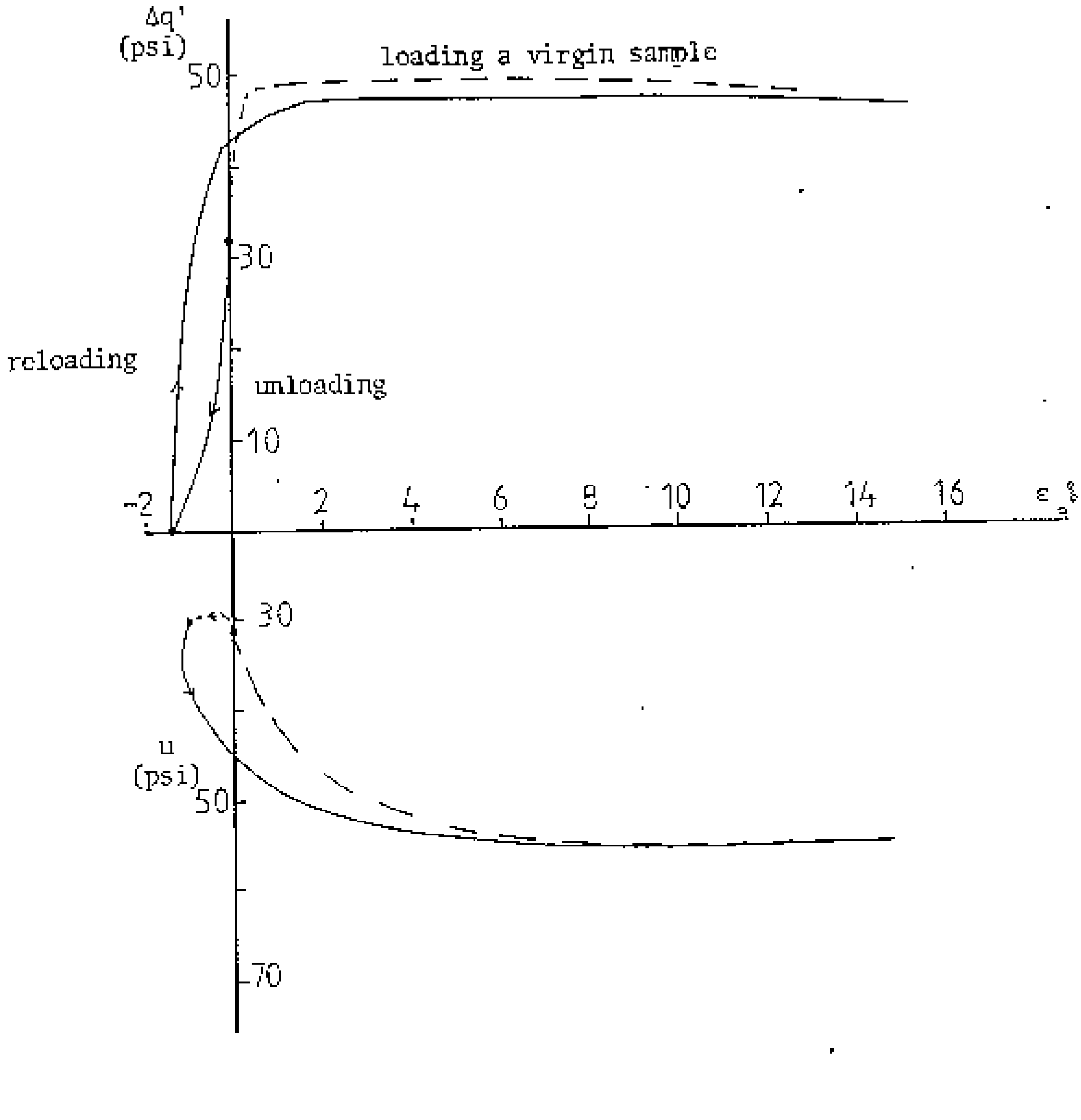


FIG. 3.15 STRESS STRAIN BEHAVIOUR IN UNLOADING AND RELOADING TESTS (After Skempton and Sowa 1963)

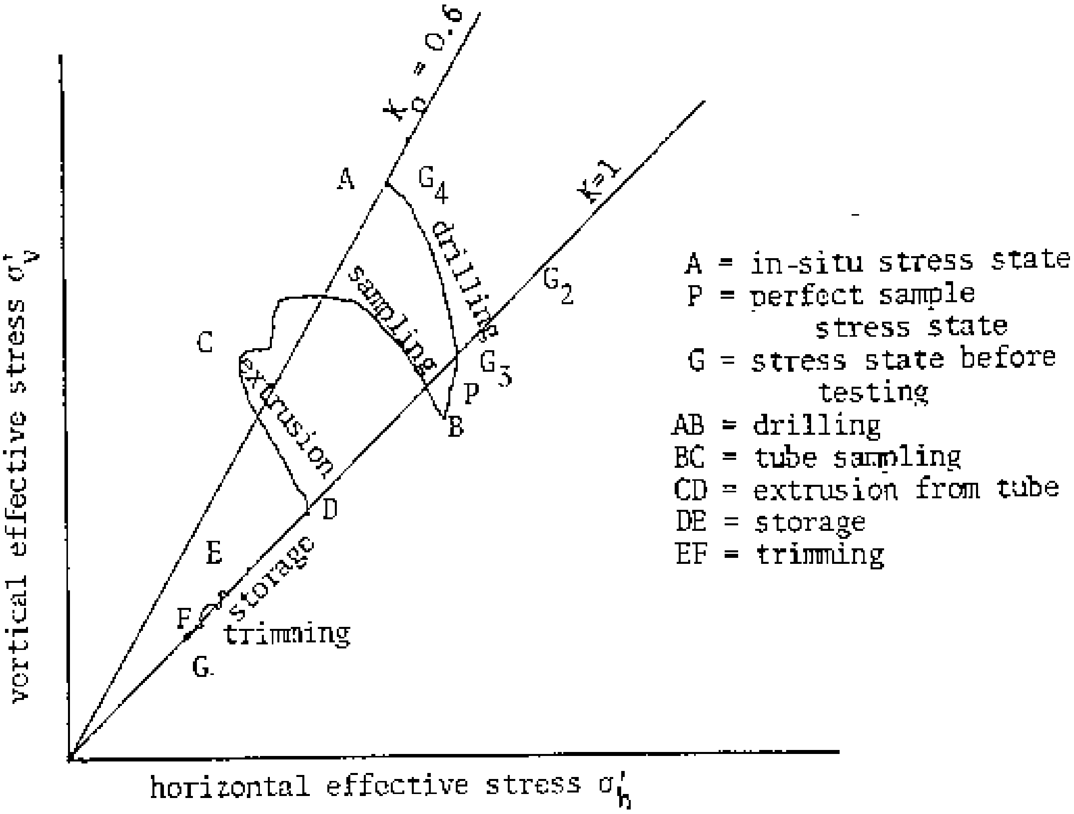


FIG. 3.16 CHANGES IN EFFECTIVE STRESSES DURING SAMPLING
(After Ladd and Lambe 1963)

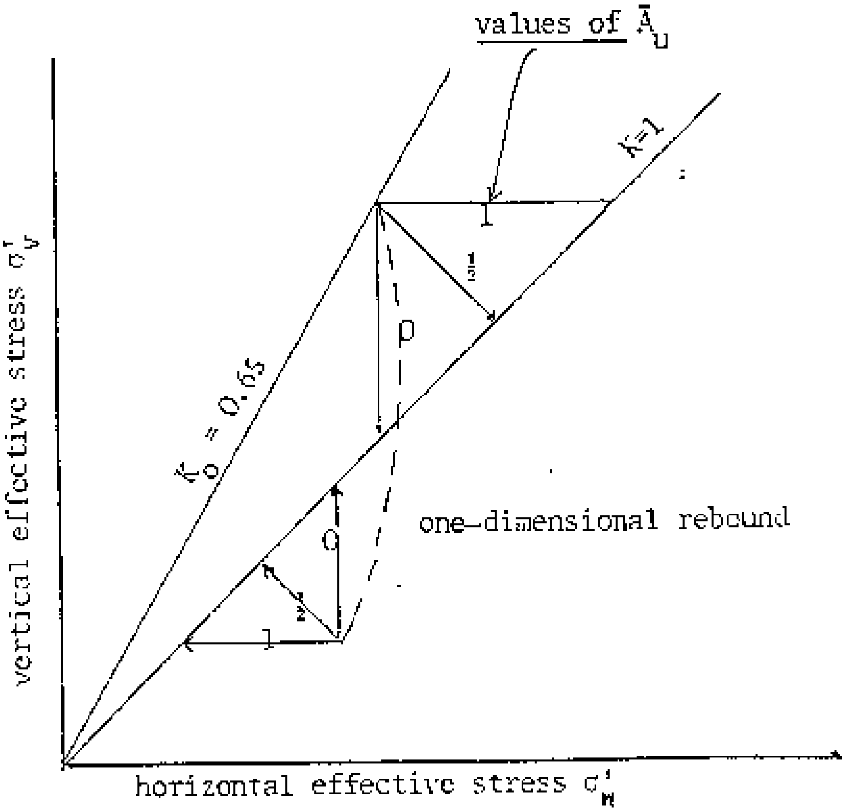


FIG. 3.17 POSSIBLE RANGE OF EFFECTIVE STRESSES DURING SAMPLING
(After Ladd and Lambe 1963)

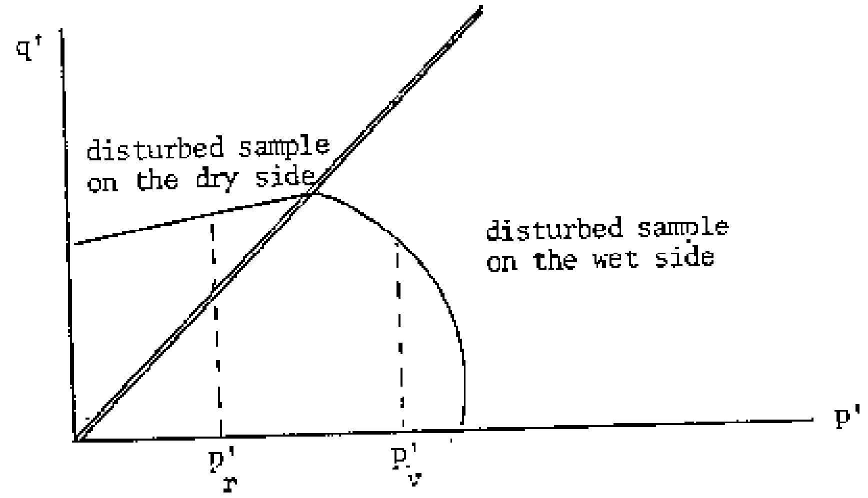


FIG. 3.18 THE EFFECTS OF THE RESIDUAL STRESS ON THE SUBSEQUENT BEHAVIOUR

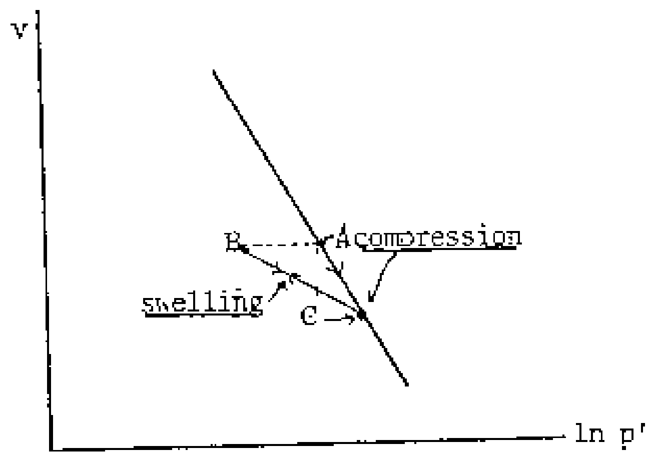


FIG. 3.19 COMPRESSION AND SWELLING CURVES

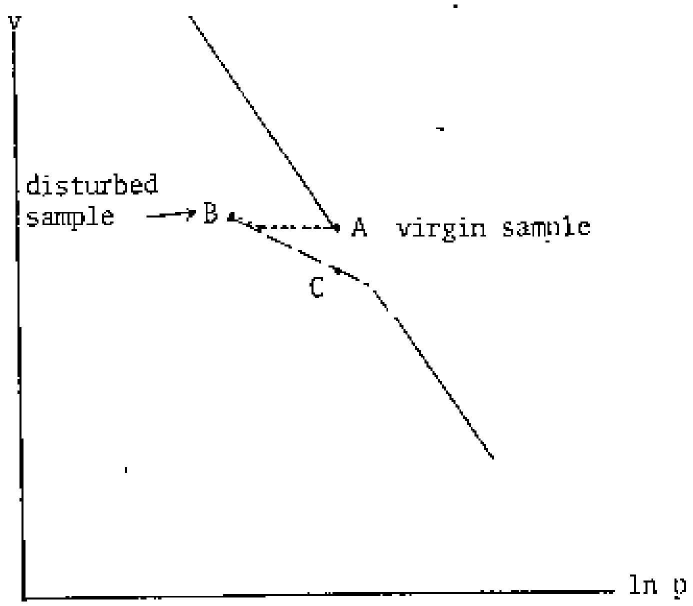


FIG. 3.20 PREDICTED BEHAVIOUR OF A DISTURBED SAMPLE

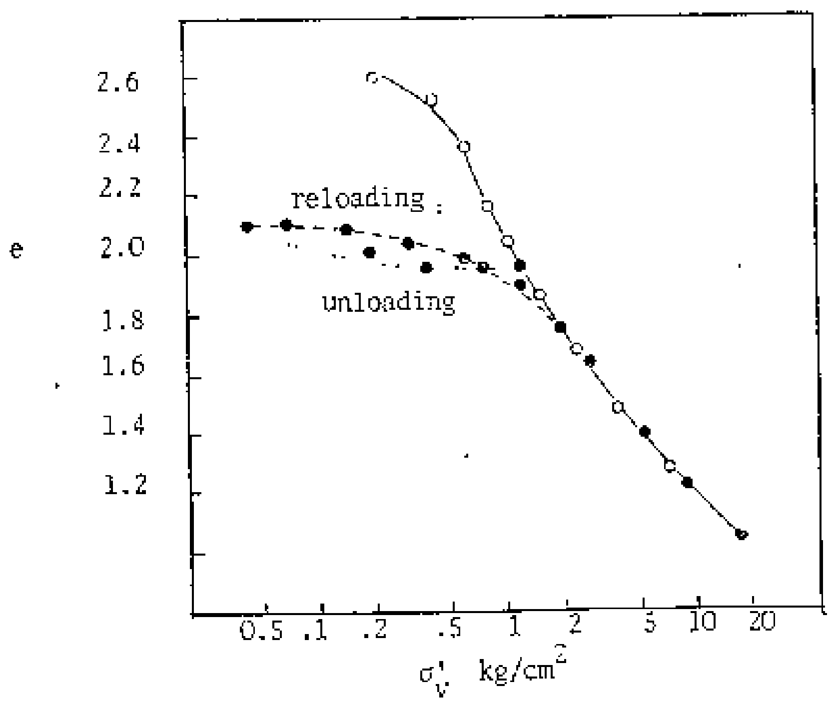
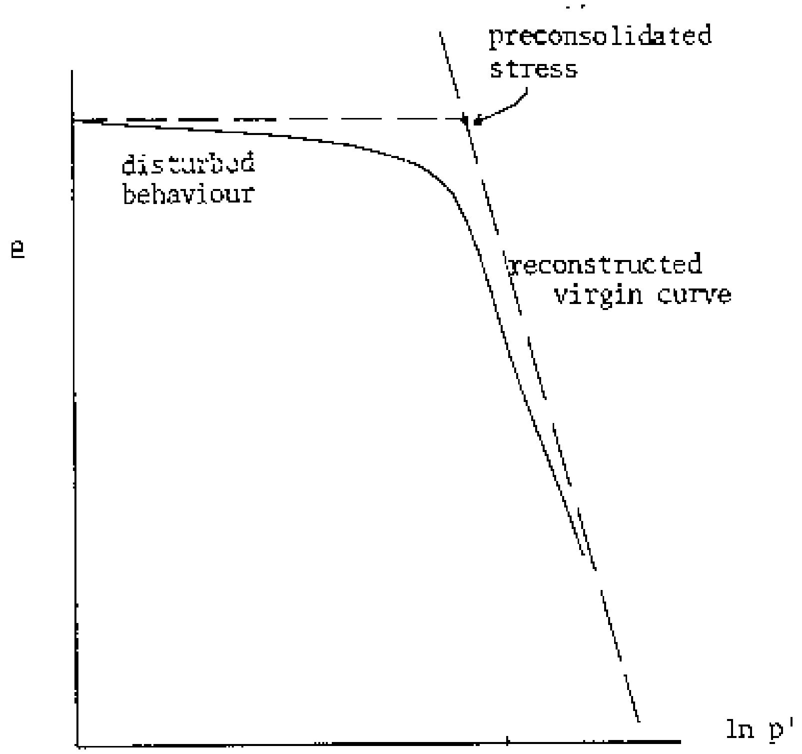
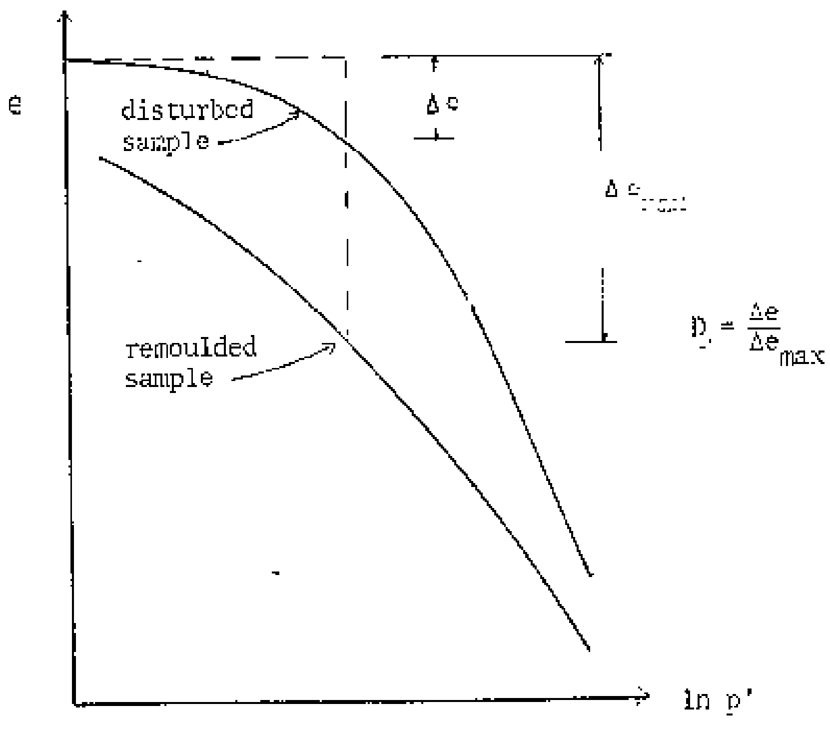


FIG. 3.21 CONSOLIDATION BEHAVIOUR OF A DISTURBED SAMPLE
(After Noorany and Poormand 1973)

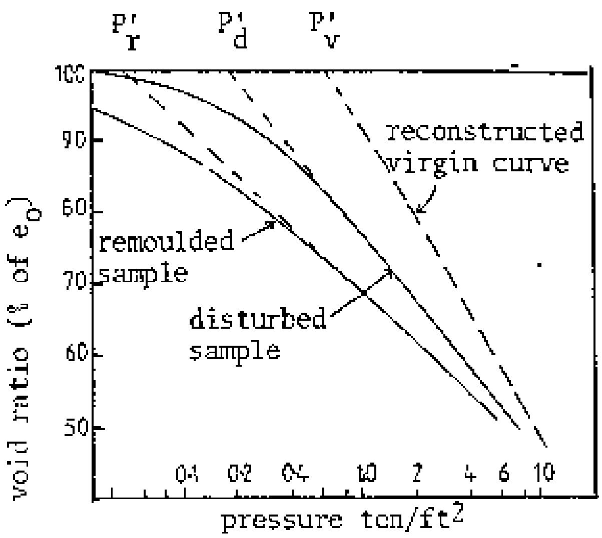


(a) Effects of disturbance on the consolidation
behaviour (after Schmertmann 1955)



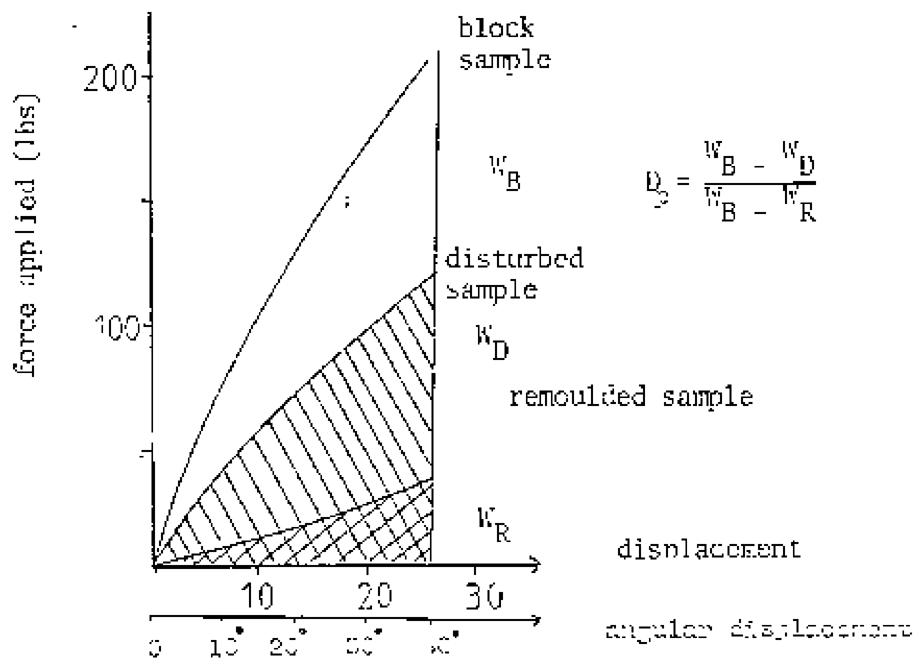
(b) Disturbance parameter based on the
consolidation curve (after Schmertmann 1955)

FIG. 3.22 CONSOLIDATION BEHAVIOUR OF DISTURBED SAMPLES



$$D_p = \frac{P'_v - P'_d}{P'_v - P'_r} \times 100$$

FIG. 3.23 DEFINITION OF DISTURBANCE FACTOR
(After Bromham 1971)



$$D_s = \frac{W_B - W_D}{W_B - W_R}$$

FIG. 3.24 DISTURBANCE PARAMETER BASED ON ENERGY
(After Goodman and Leininger 1967)

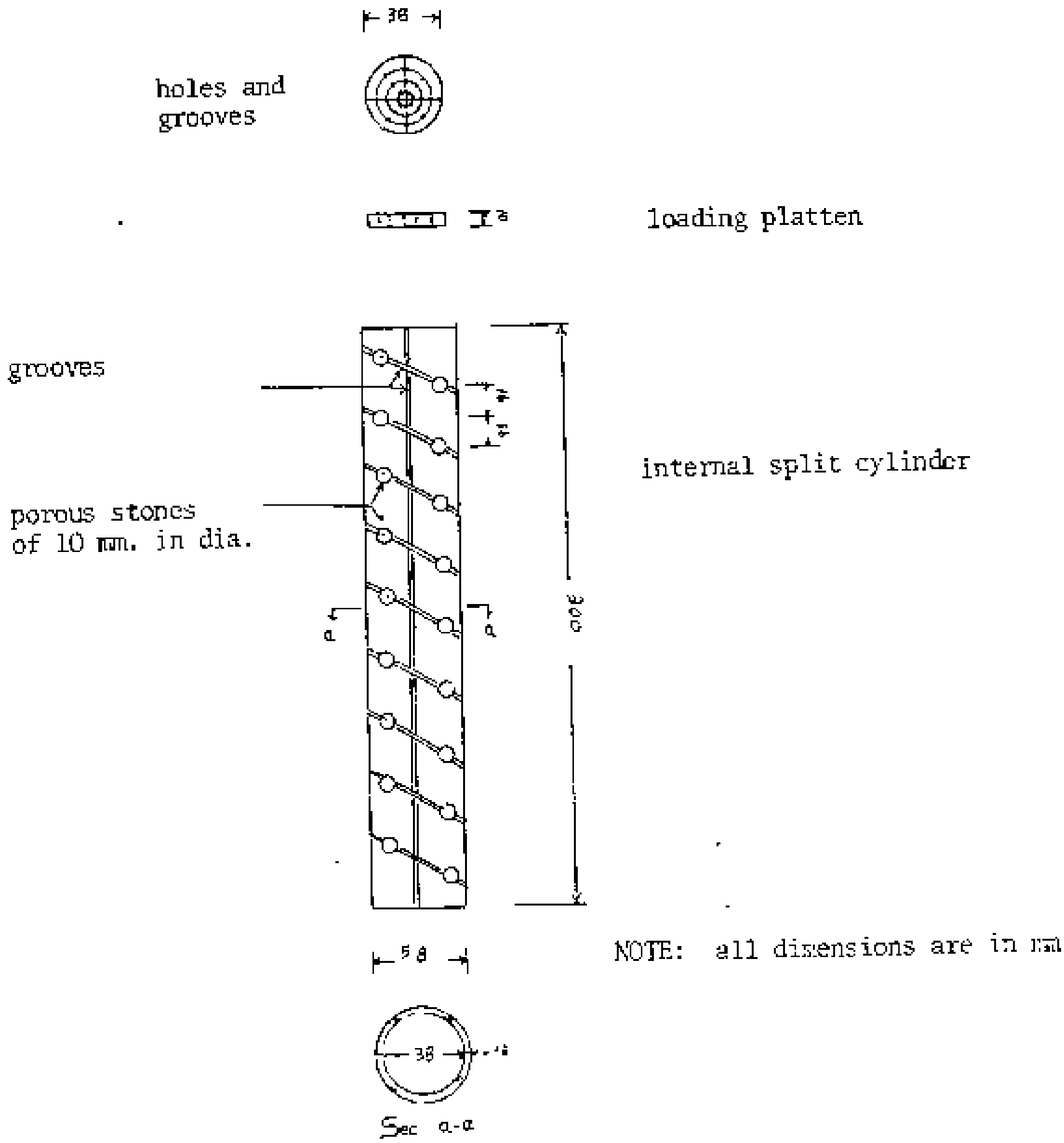


FIG. 4.1 DETAILS OF THE INTERNAL CYLINDER OF THE $1\frac{1}{2}$"
SAMPLE FORMER

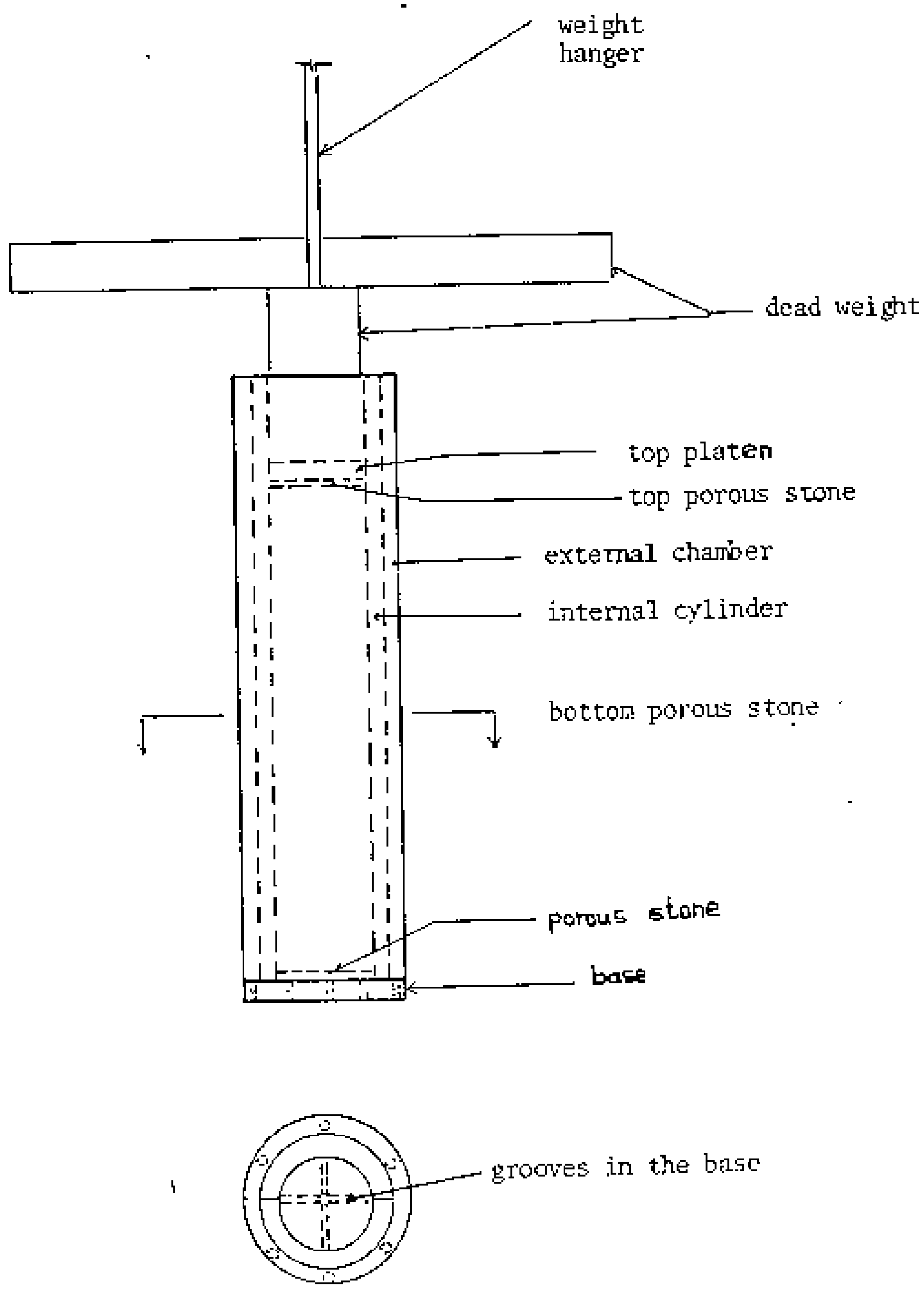


FIG. 4.2 ASSEMBLED 1½" SAMPLE FORMER

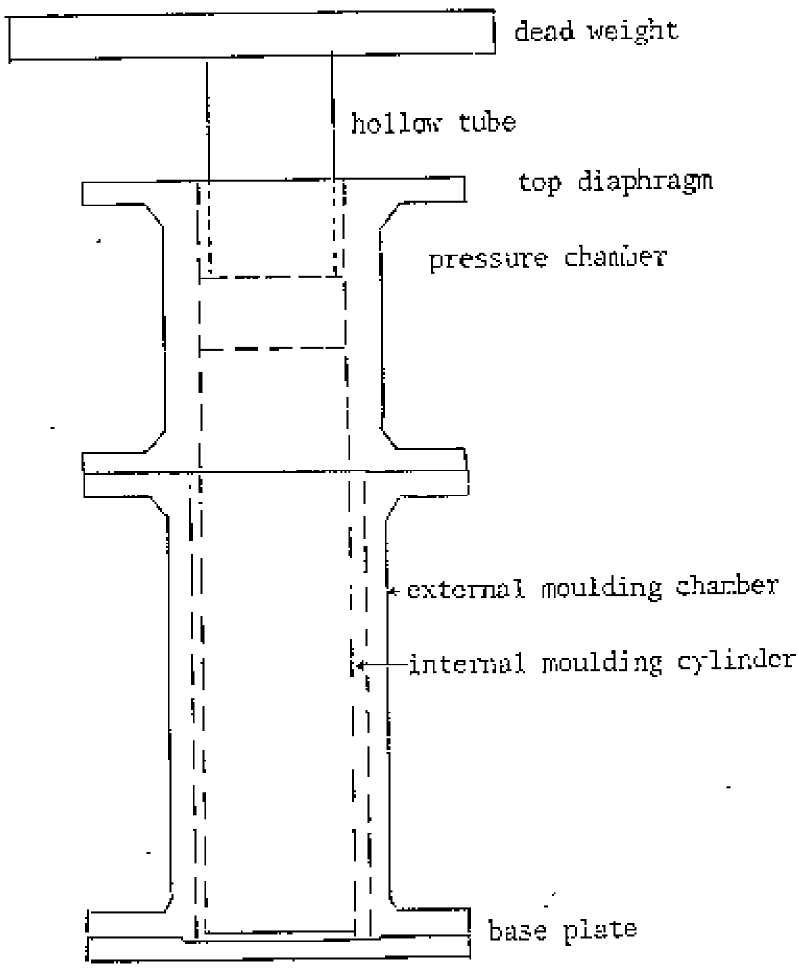


FIG. 4.5 ASSEMBLED 4" SAMPLE FORMER

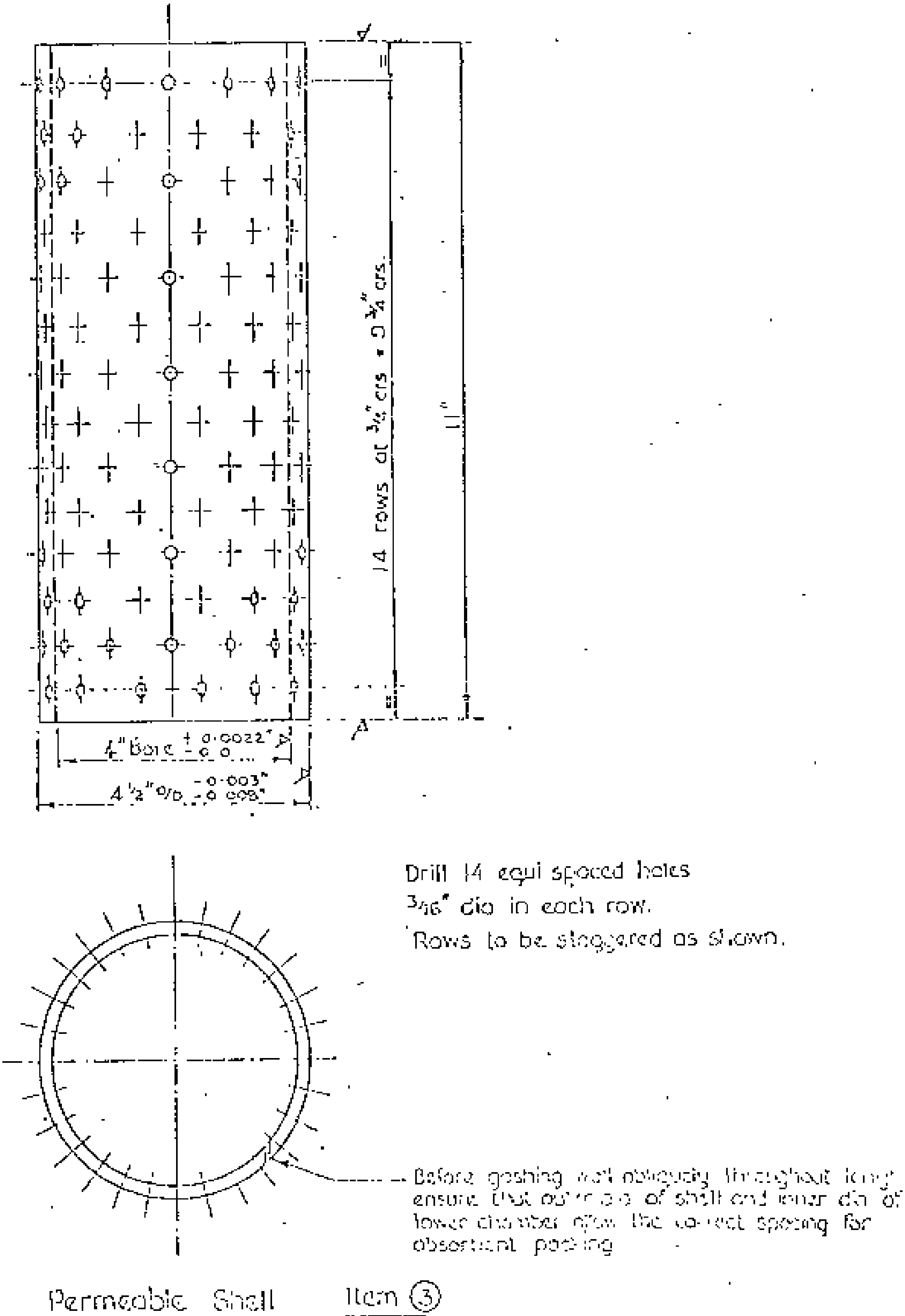
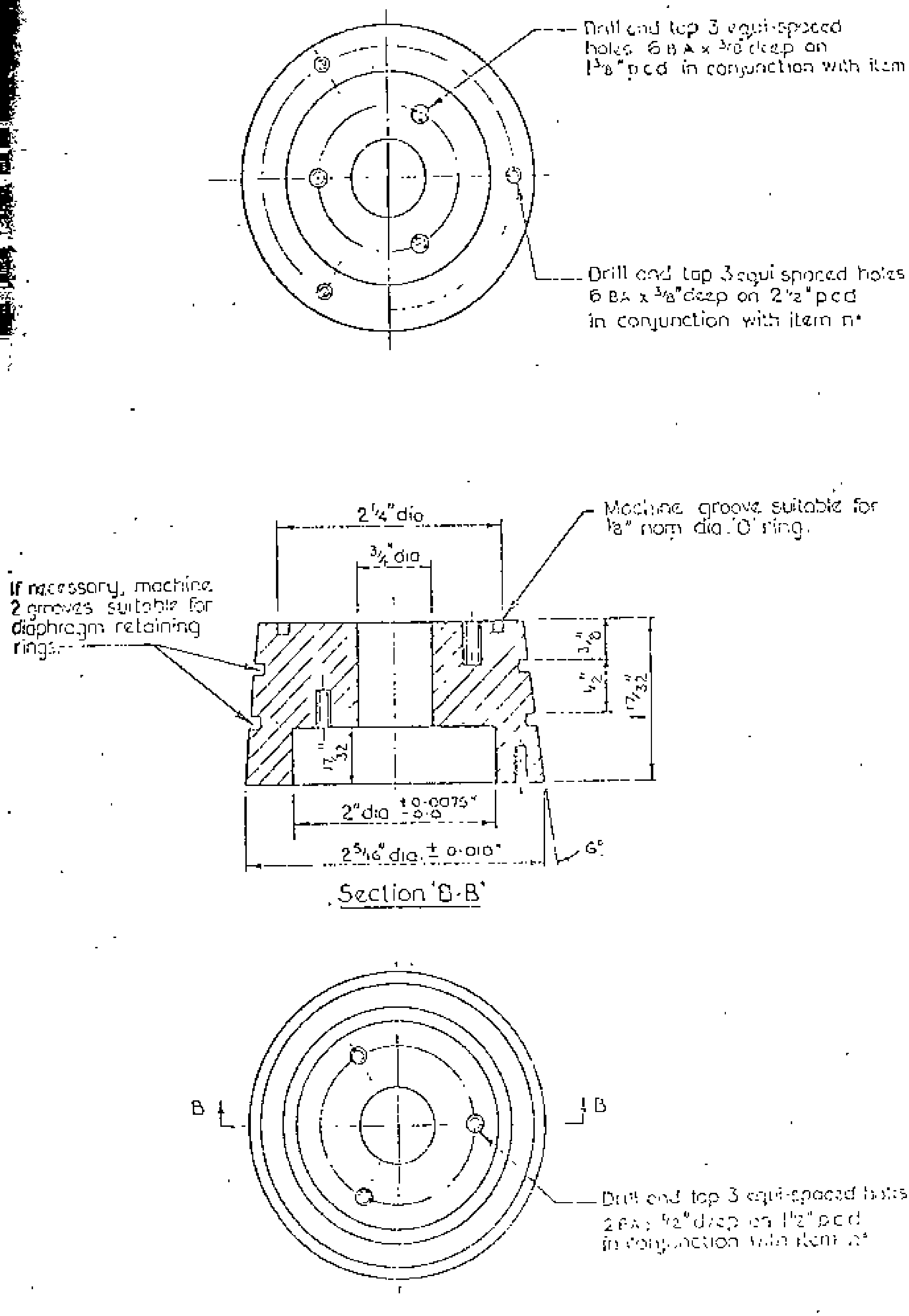


FIG. 4.4. Details of the Moulding cell (brass sleeve).
(from Stipho 1978)



Item (7)

FIG. 4.5a : Details of the Moulding Cell (top drainage details).

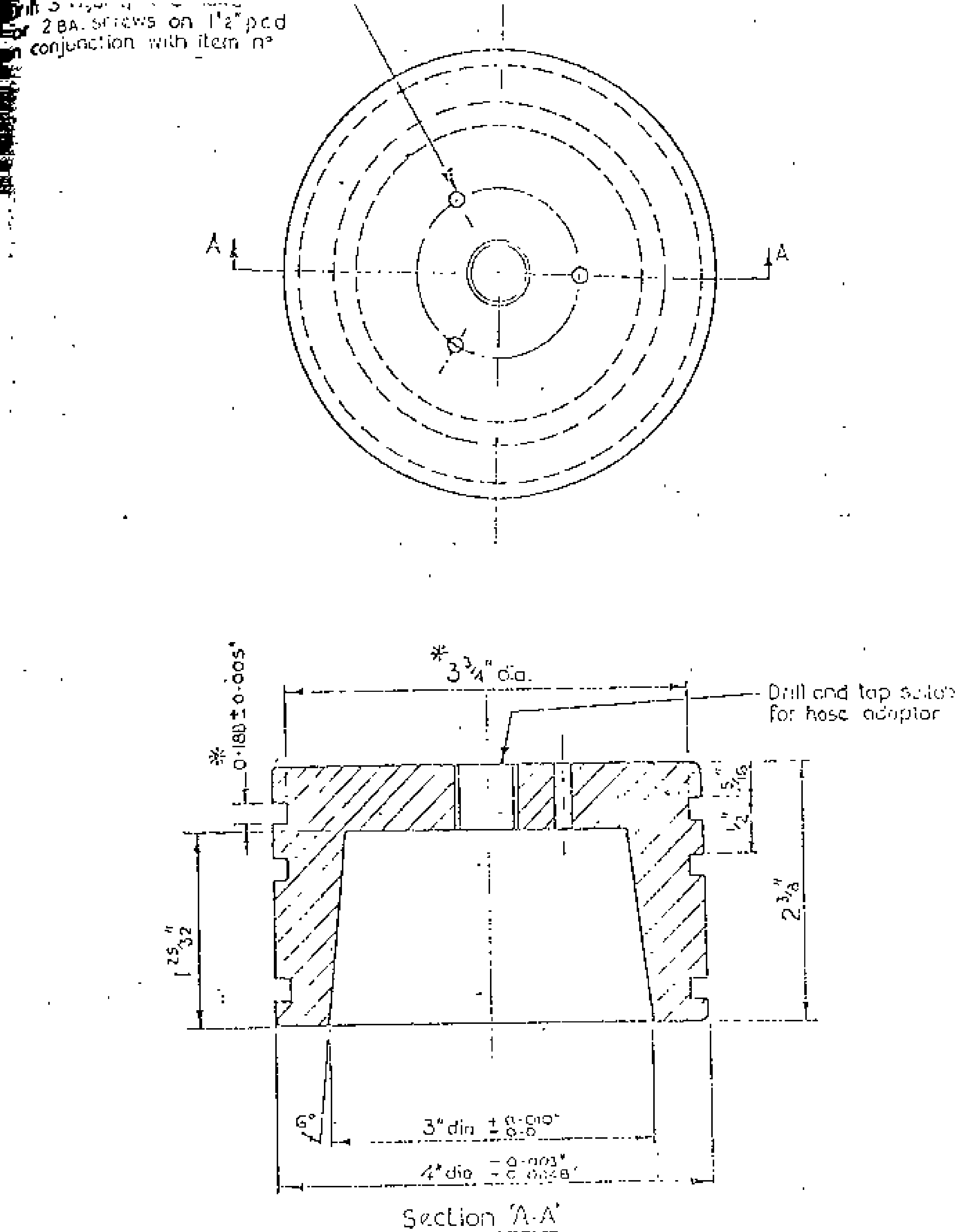


FIG. 4.5b. : Details of Moulding Cell (pressure diaphragm).

(from Stipho 1978)

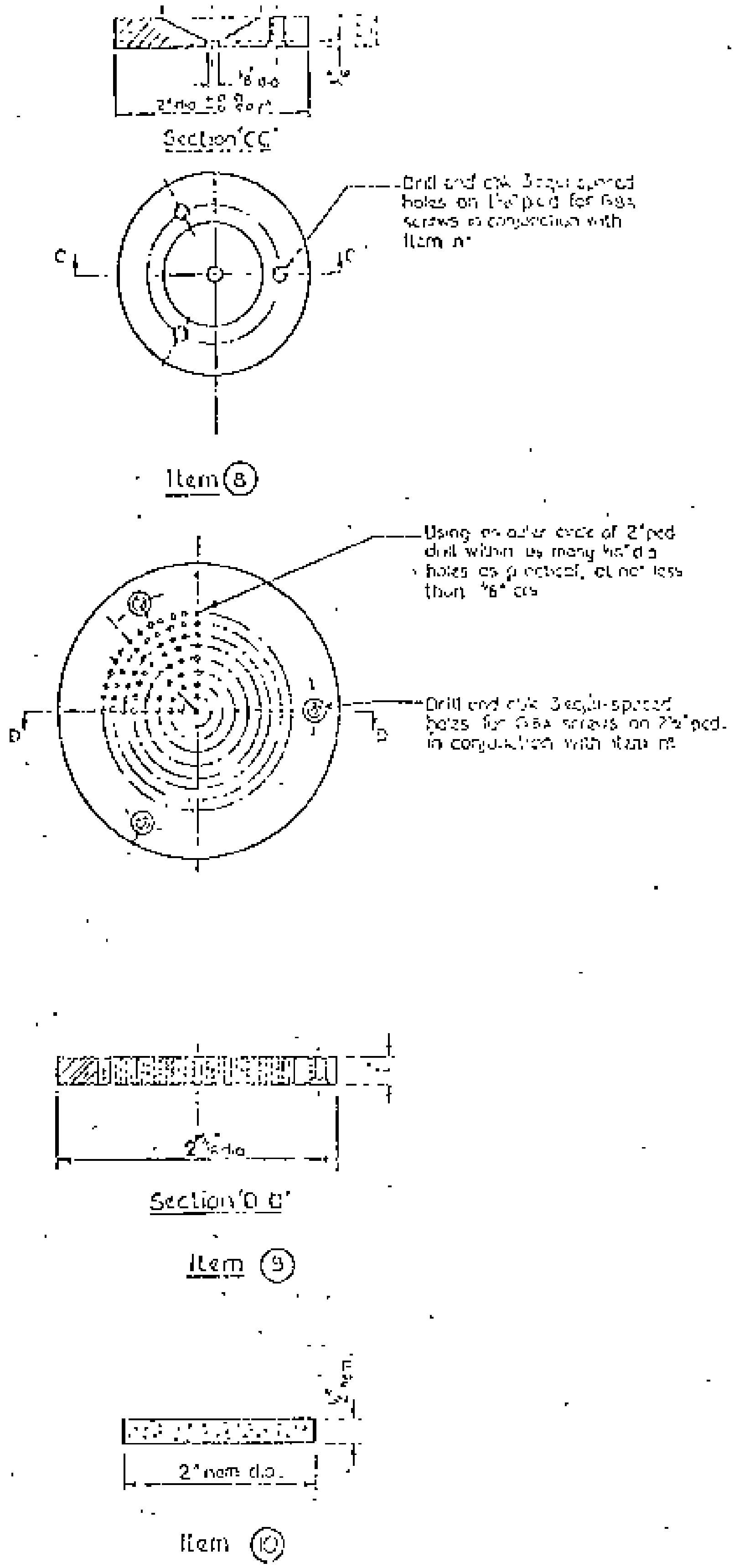


FIG. 4.5c : Details of the Moulding Cell (pore pressure chamber at the centre of the pressure diaphragm).

(from Stipho 1978)

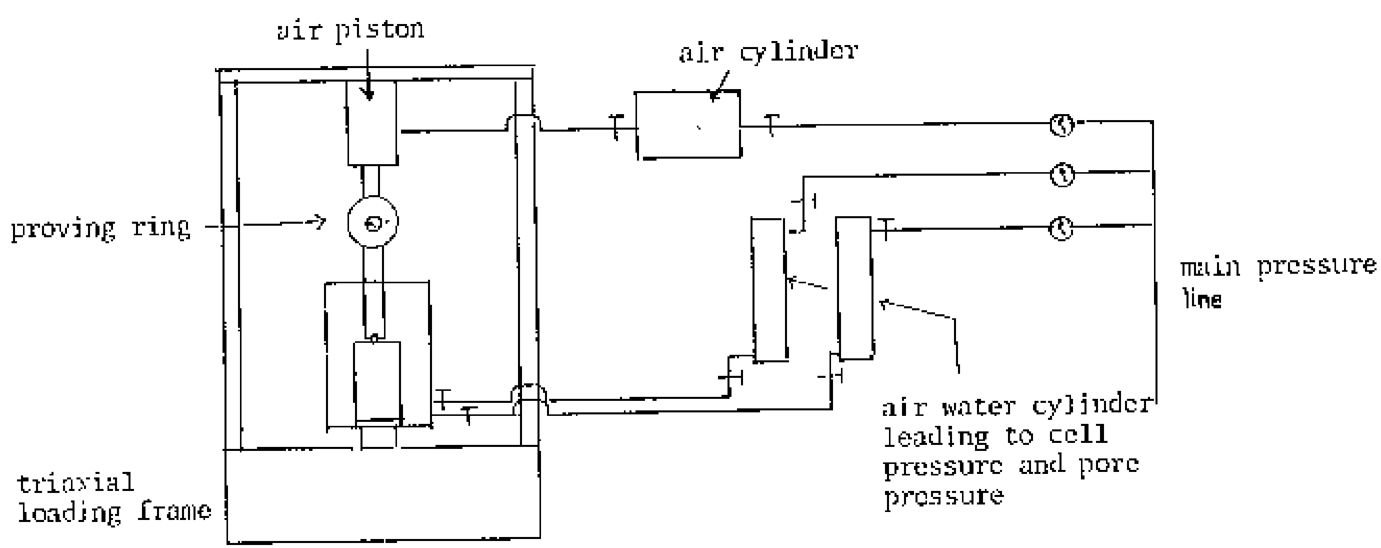


FIG. 4.6 LAYOUT OF THE 4" TRIAXIAL CELL SYSTEM

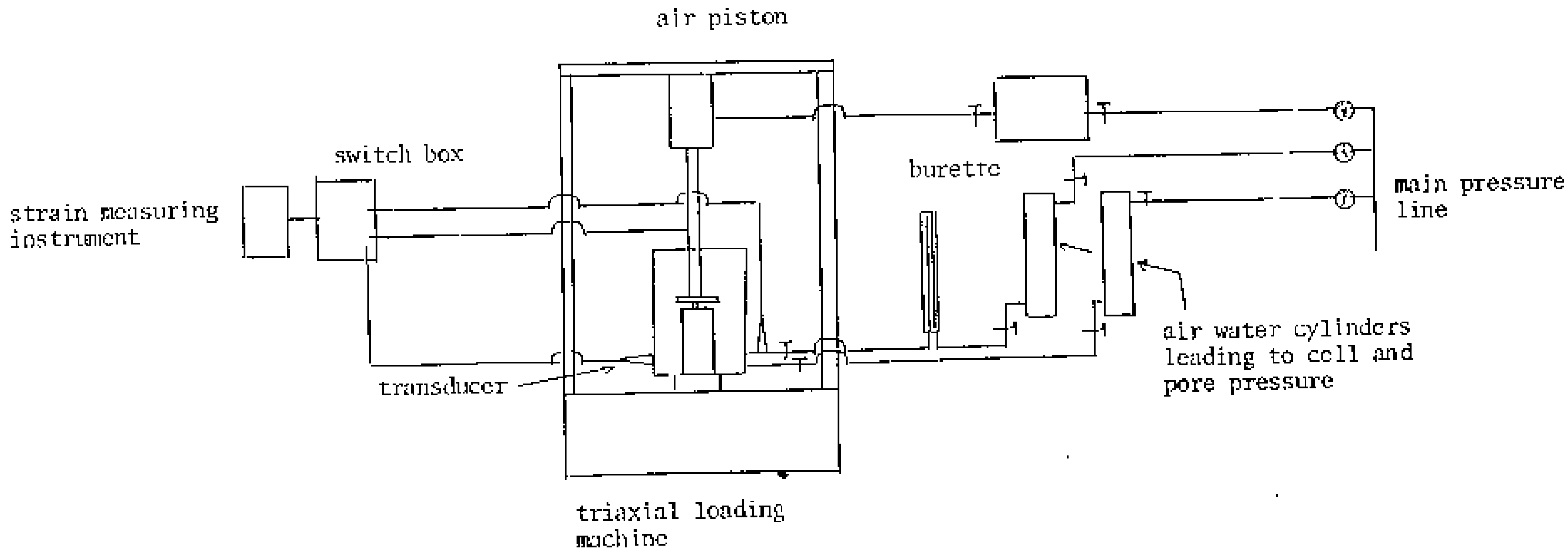


FIG. 4.7 LAYOUT OF THE $1\frac{1}{2}$" TRIAXIAL CELL SYSTEM

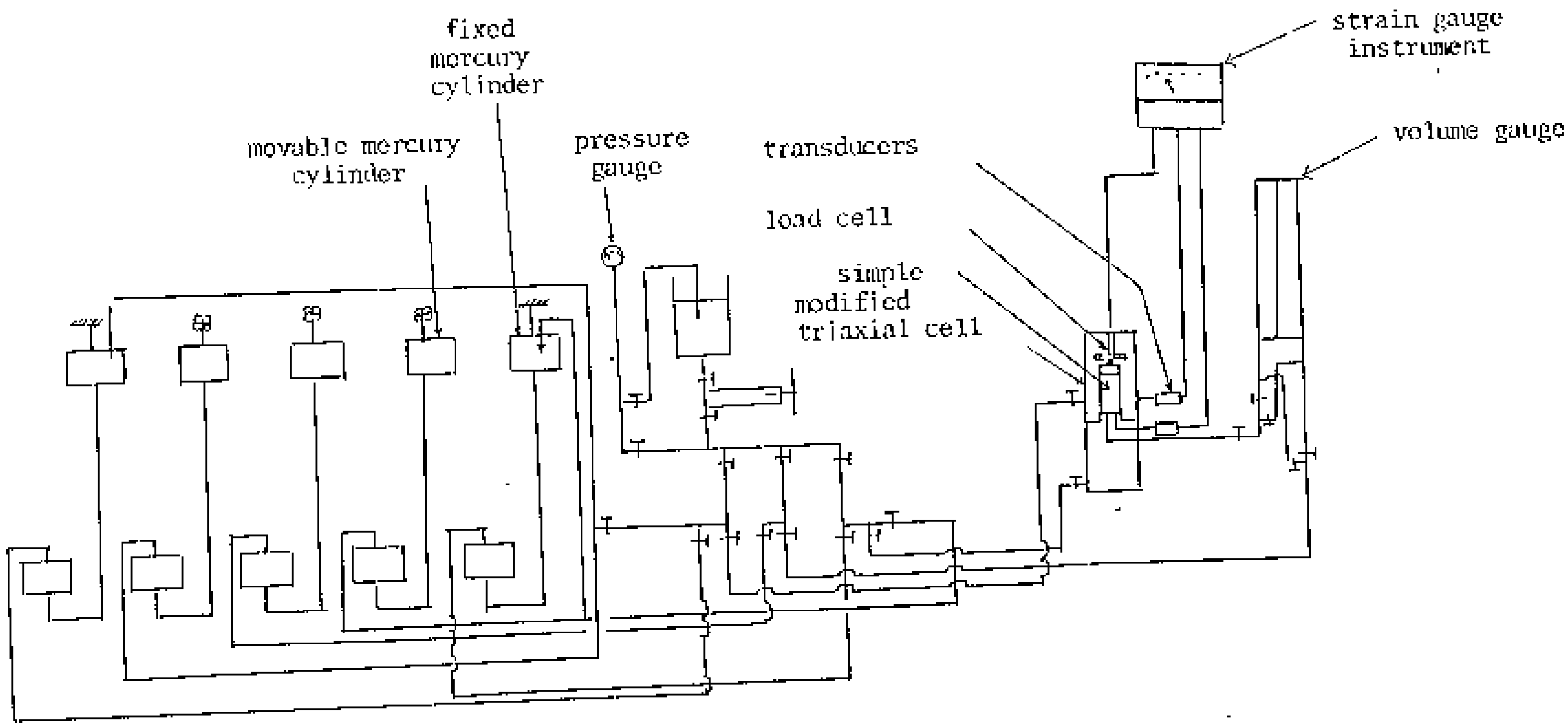


FIG. 4.8 LAYOUT OF THE $1\frac{1}{2}$" STRESS CONTROLLED TRIAXIAL CELL SYSTEM

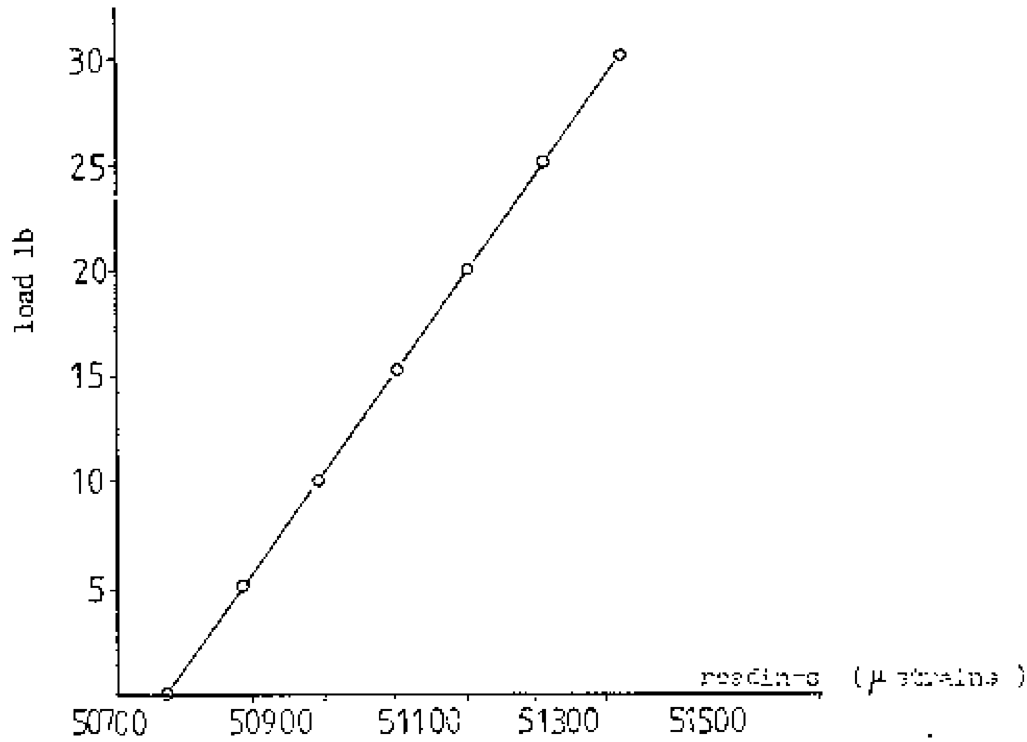


FIG. 4.9 CALIBRATION OF A LOAD CELL

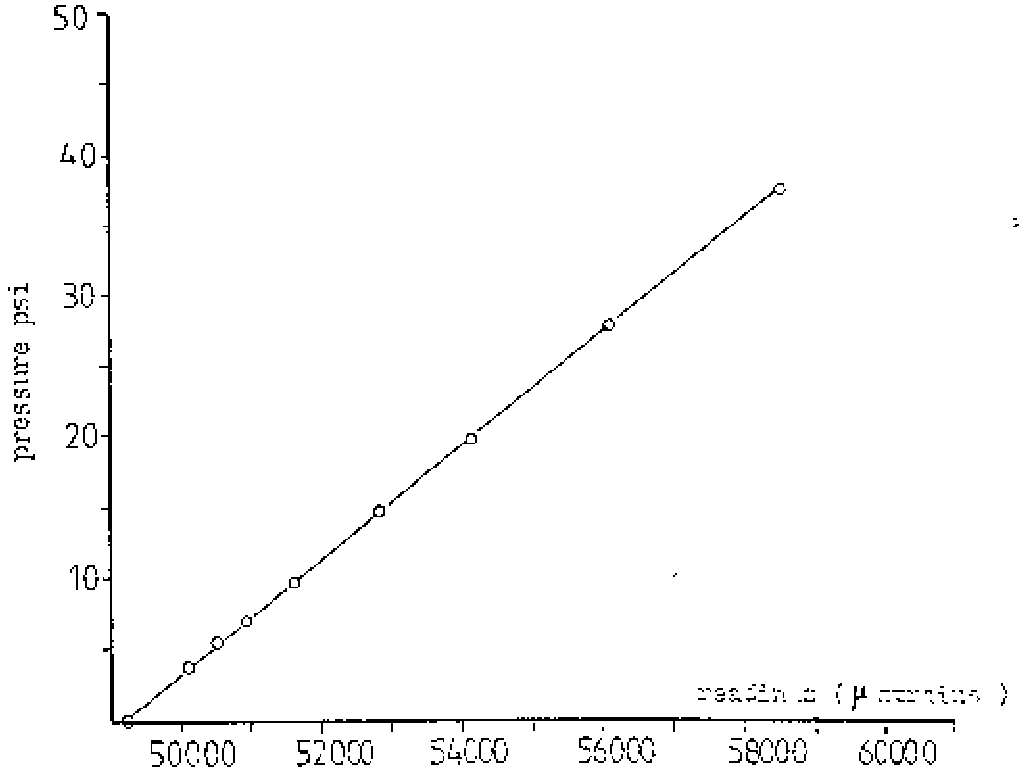


FIG. 4.10 CALIBRATION OF A TRANSDUCER

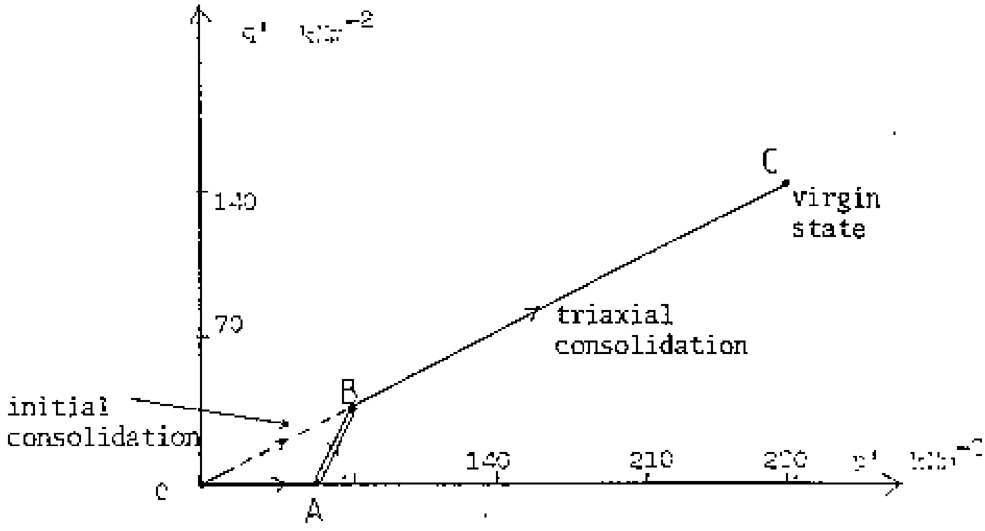


FIG. 4.11 STRESS PATH IN THE TRIAXIAL CONSOLIDATION

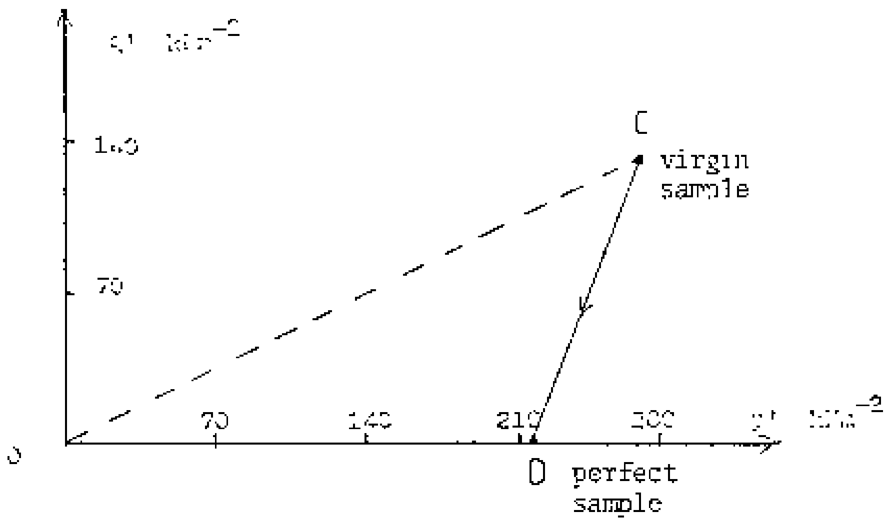


FIG. 4.12 STRESS PATH IN PERFECT SAMPLING

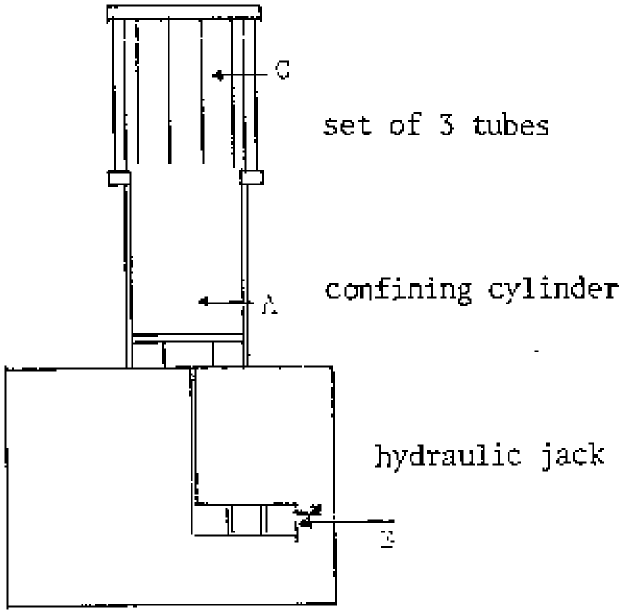


FIG. 4.13 TUBE SAMPLES EXTRUDER

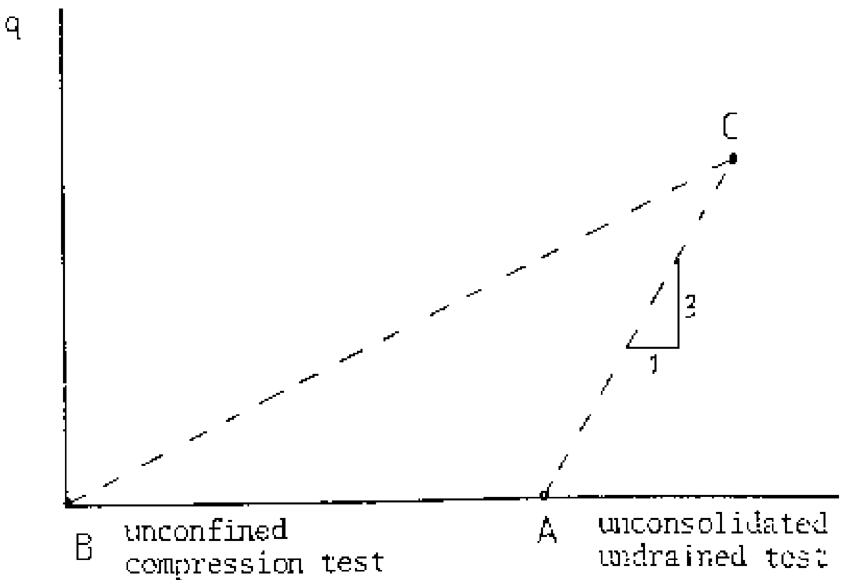


FIG. 4.14 STRESS STATES OF THE PERFECT SAMPLE
IN DIFFERENT TESTS

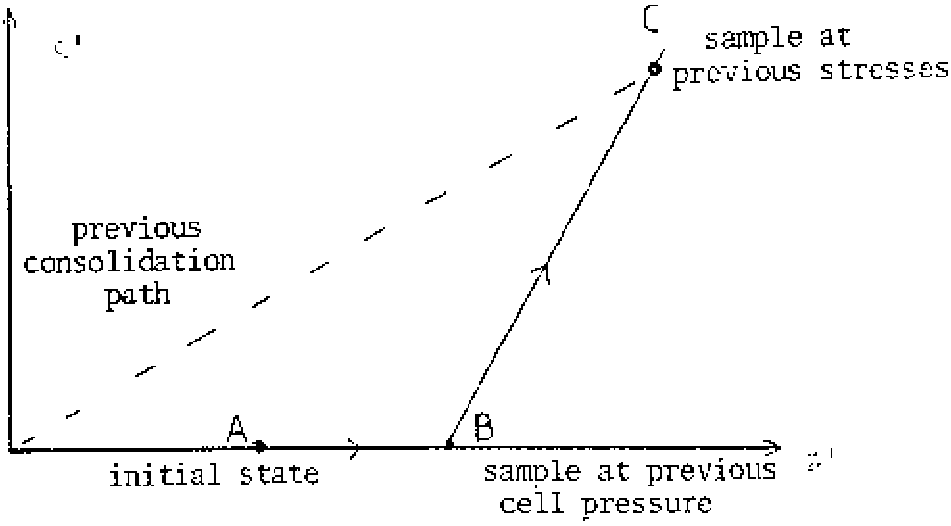


FIG. 4.15 RECONSOLIDATION PATH C

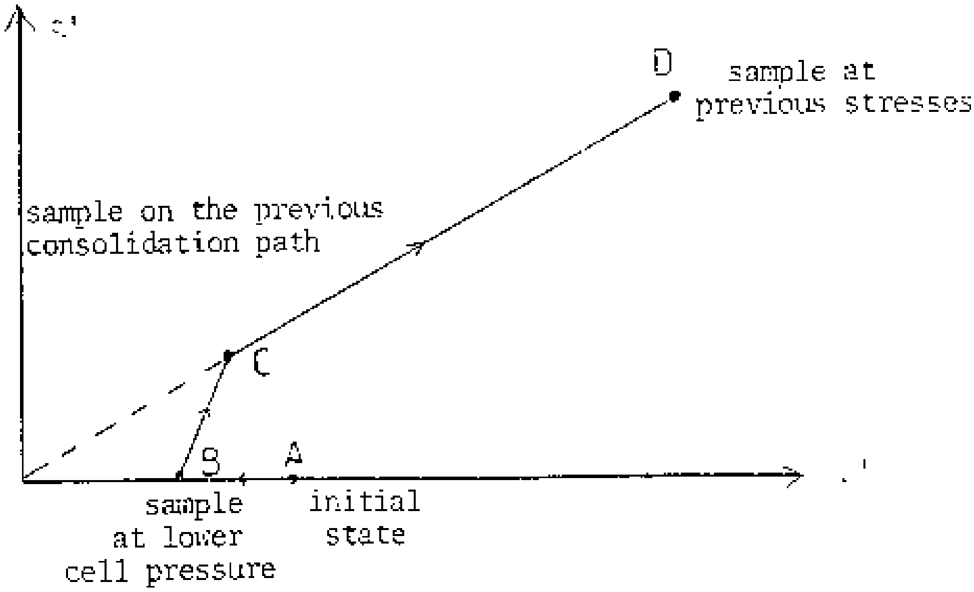


FIG. 4.16 RECONSOLIDATION PATH K

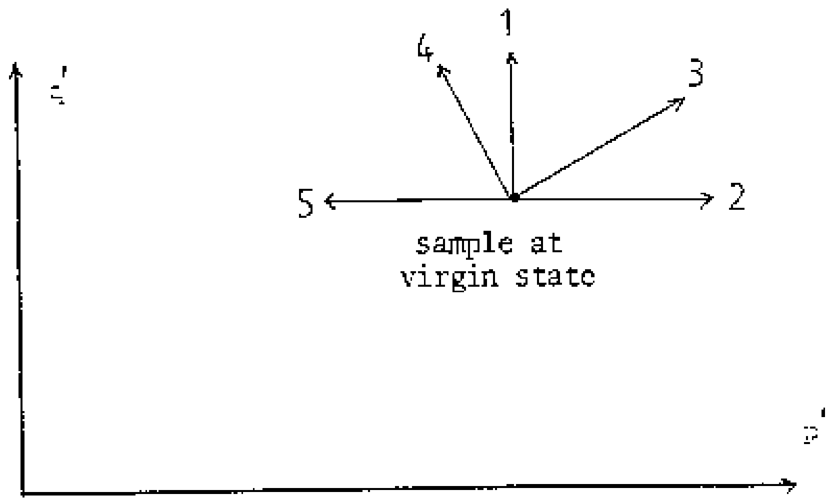


FIG. 4.17 LOADING PATHS IN DRAINED TESTS

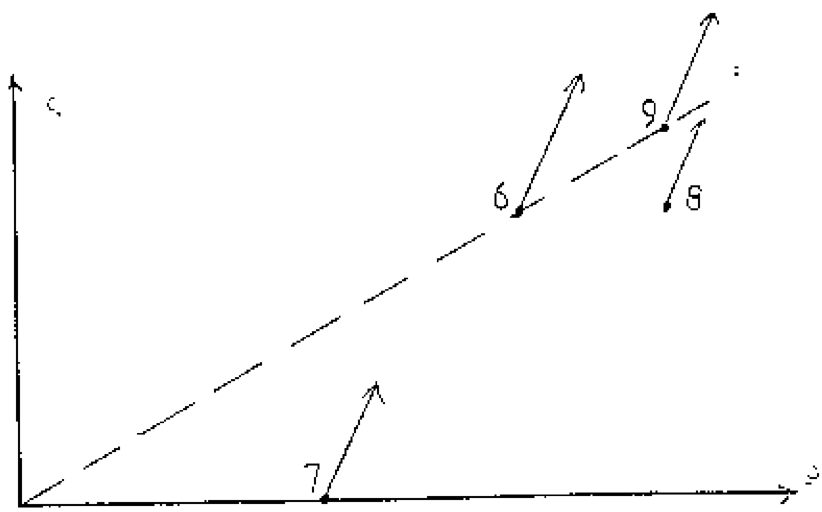


FIG. 4.18 INITIAL STRESS STATES IN UNDRAINED TESTS

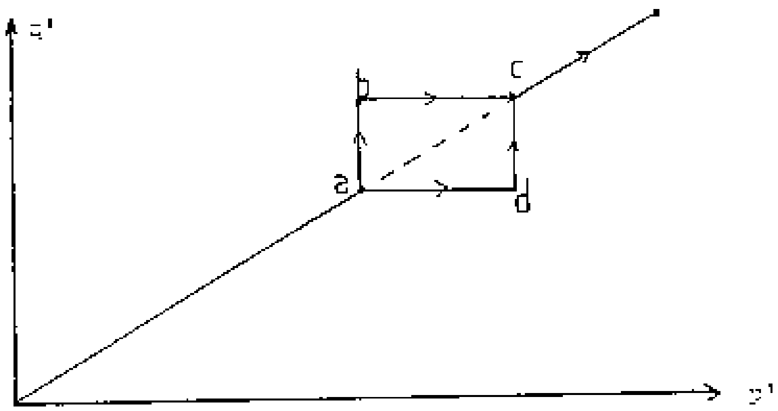


FIG. 4.19 LOADING PATHS IN SPECIAL TESTS
S3 AND S4

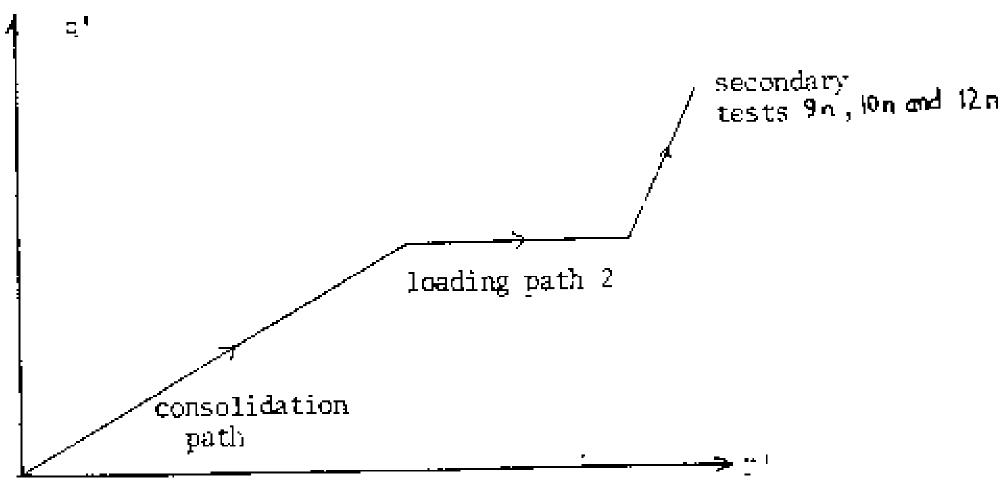
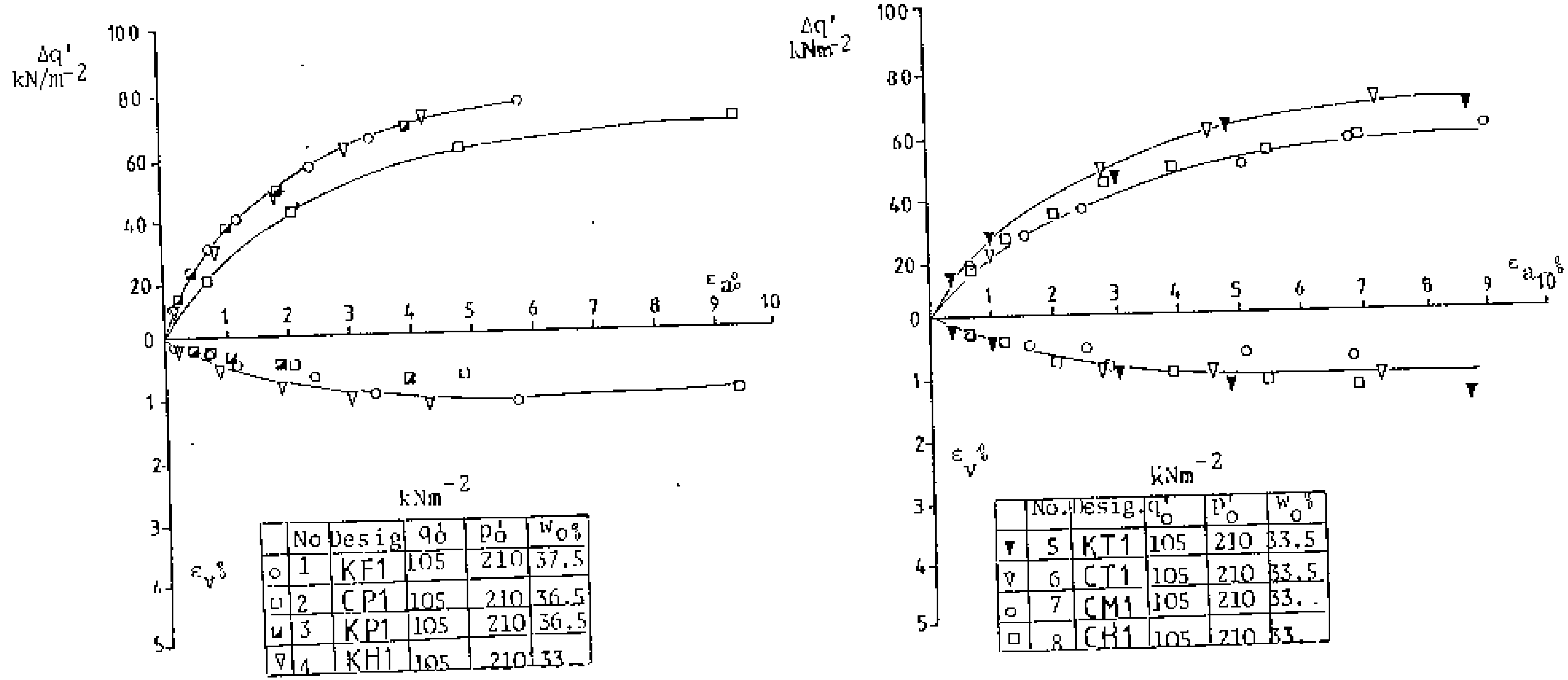


FIG. 4.20 LOADING PATH IN SECONDARY TESTS



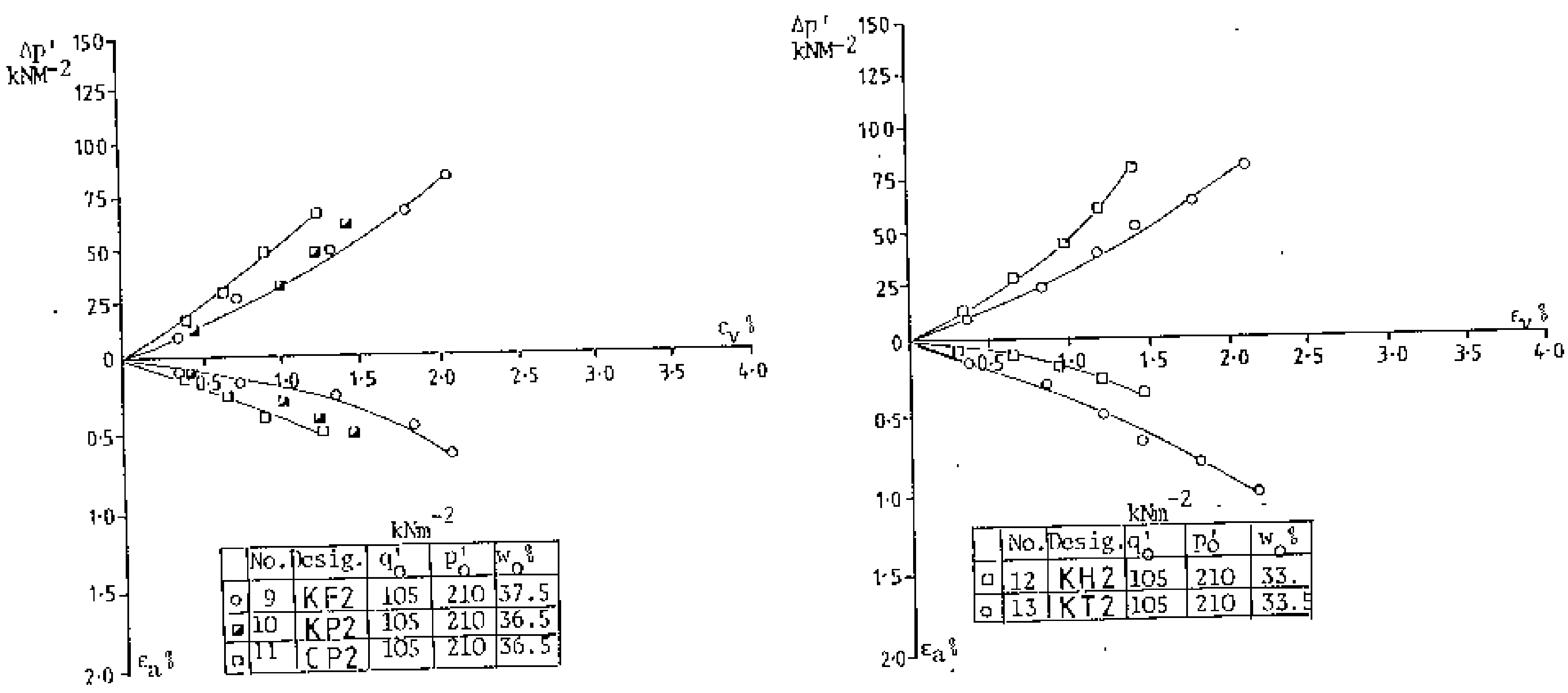
kNm^{-2}

	No	Desig	q'_0	p'_0	w_0%
○	1	KF1	105	210	37.5
□	2	CP1	105	210	36.5
◪	3	KP1	105	210	36.5
▽	4	KH1	105	210	33

kNm^{-2}

	No.	Desig.	q'_0	p'_0	w_0%
▼	5	KT1	105	210	33.5
▽	6	CT1	105	210	33.5
○	7	CM1	105	210	33.
□	8	CH1	105	210	33.

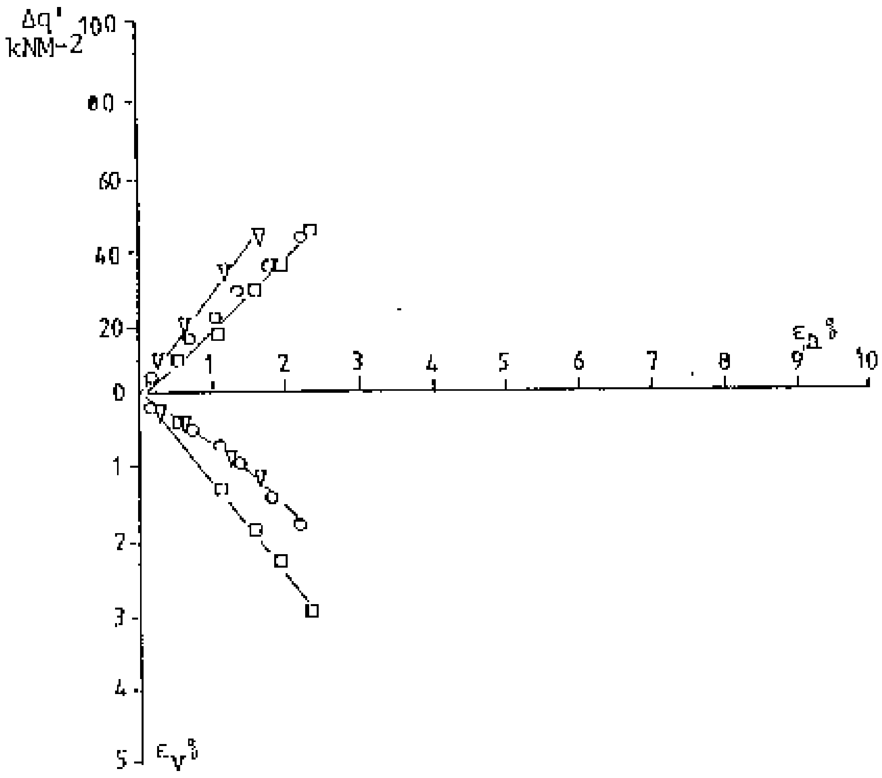
FIG. 4.21 BEHAVIOUR OF SAMPLES IN CONSTANT p' TEST

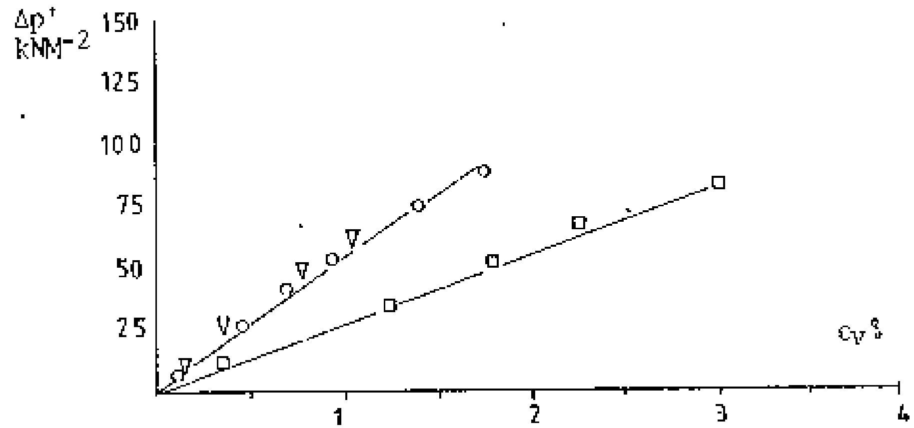


	No.	Desig.	q'o	p'o	wo %
○	9	KF2	105	210	37.5
■	10	KP2	105	210	36.5
□	11	CP2	105	210	36.5

	No.	Desig.	q'o	p'o	wo %
□	12	KH2	105	210	33.
○	13	KT2	105	210	33.5

FIG. 4.22 BEHAVIOUR OF SAMPLES IN CONSTANT q' TEST

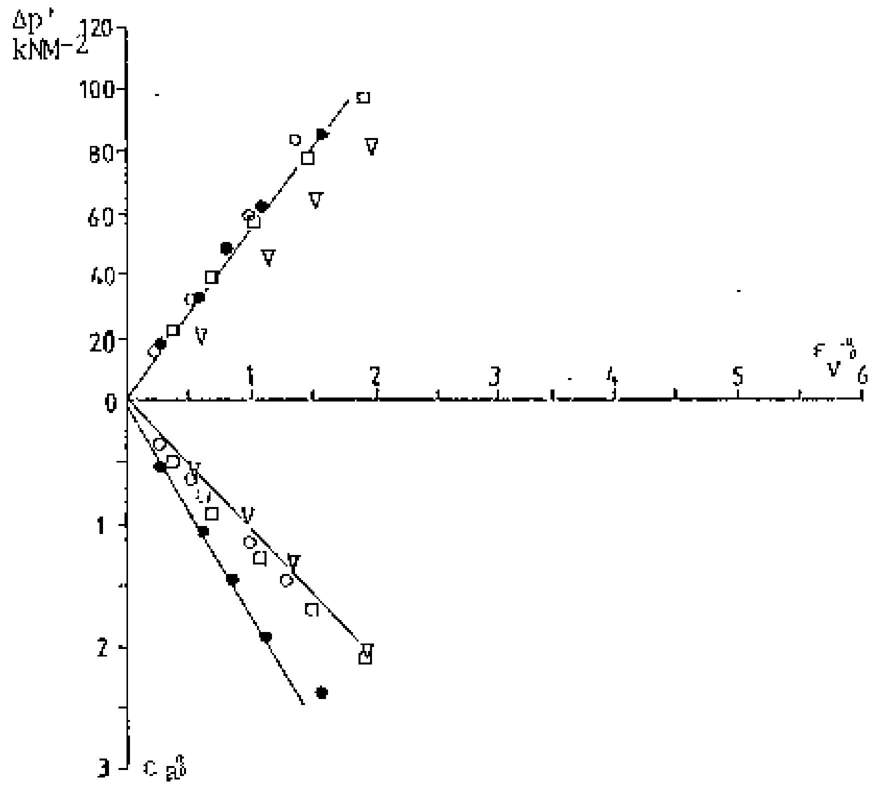


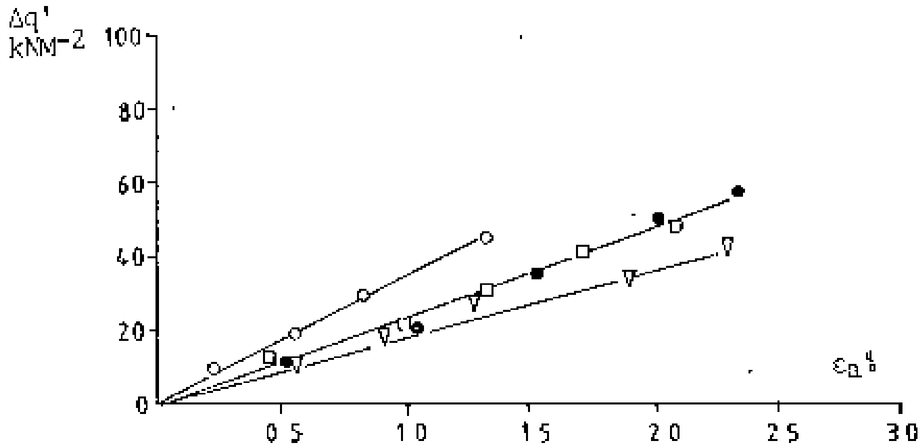


kNm^{-2}

	No.	Desig	q_o'	p_o'	w_o%
□	14	KF3	105	210	37.5
o	15	CP3	105	210	36.5
∇	16	KH3	105	210	33.

FIG. 4.23 BEHAVIOUR OF SAMPLES IN ANISOTROPIC LOADING

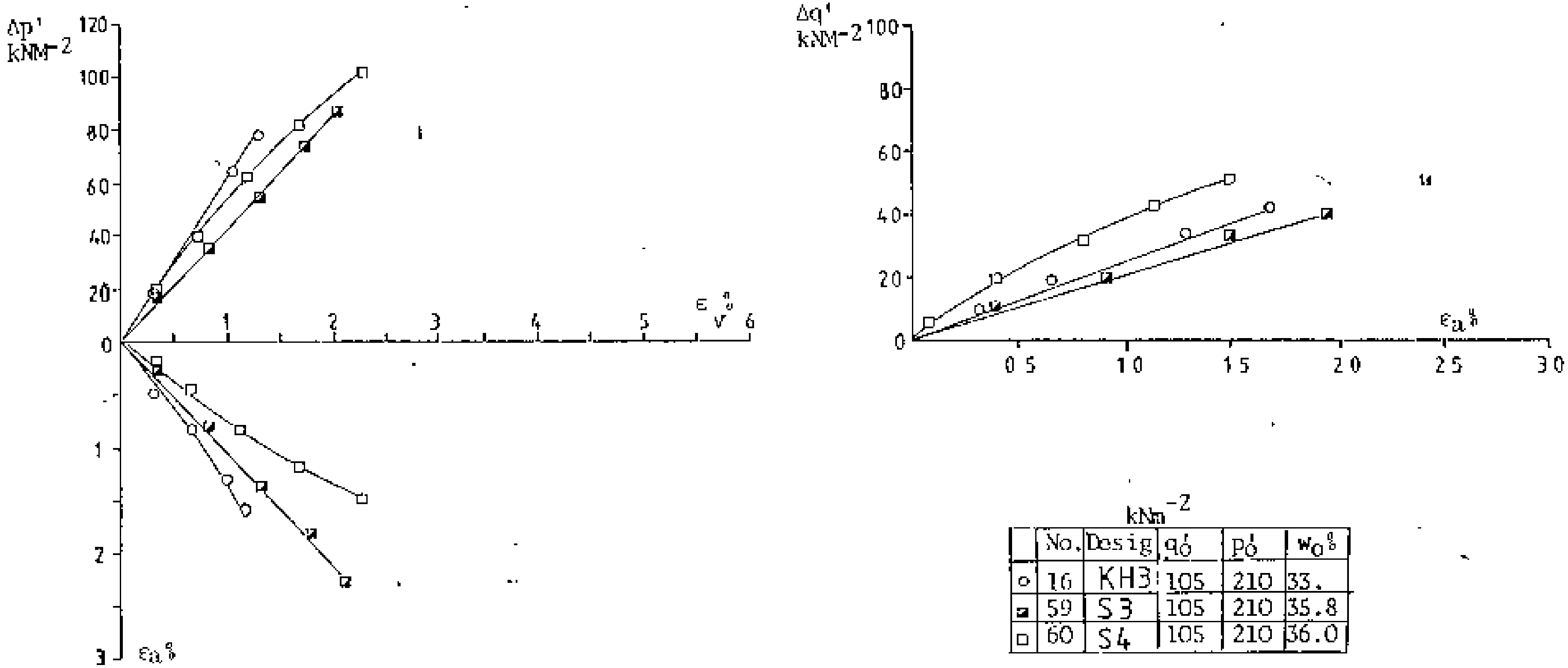




kNm^{-2}

	No.	Desig.	q'_o	p'_o	w_o %
○	17	CT3	105	210	33.5
●	18	G17	105	210	33.5
□	19	CM3	105	210	33.
∇	20	CH3	105	210	33.

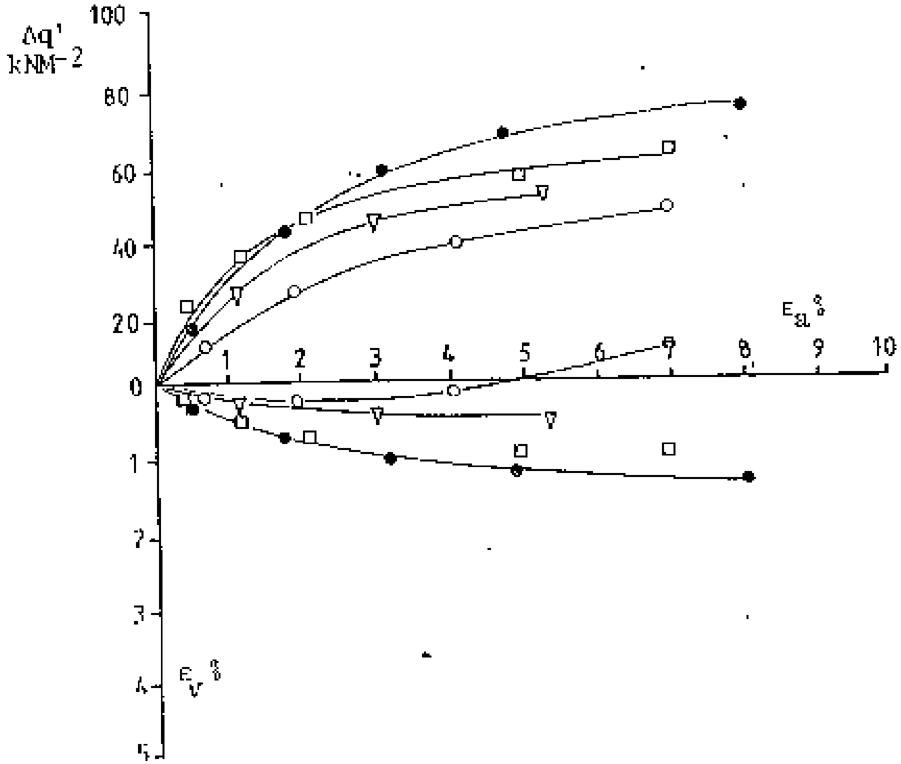
FIG. 4.24 BEHAVIOUR OF TUBE SAMPLES IN ANISOTROPIC LOADING



kNm^{-2}

	No.	Desig	q_0'	p_0'	w_0%
○	16	KH3	105	210	33.
◪	59	S3	105	210	35.8
□	60	S4	105	210	36.0

FIG. 4.25 BEHAVIOUR OF SPECIAL SAMPLES AND A TUBE SAMPLE IN ANISOTROPIC LOADING



kNm^{-2}

	No.	Desig	q_o'	p_o'	w_o%
□	21	KF4	105	210	37.5
▽	22	CP4	105	210	36.5
○	23	CT4	105	210	33.5
●	24	KH4	105	210	33.

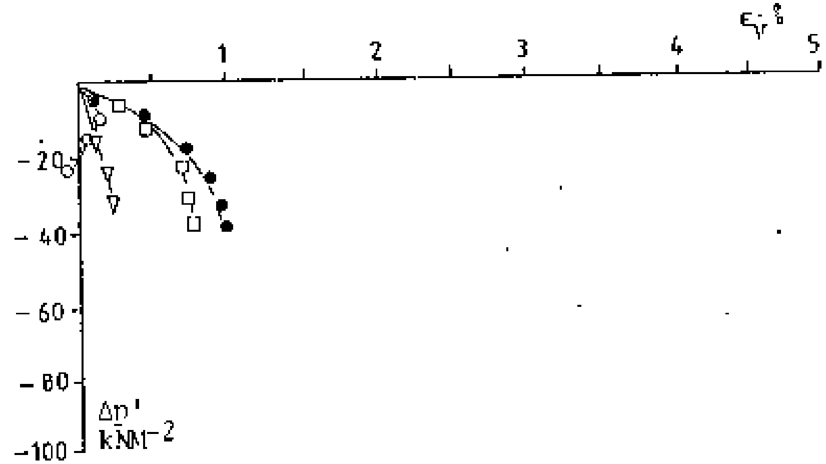


FIG. 4.26 BEHAVIOUR OF SAMPLES IN LOADING PATH 4

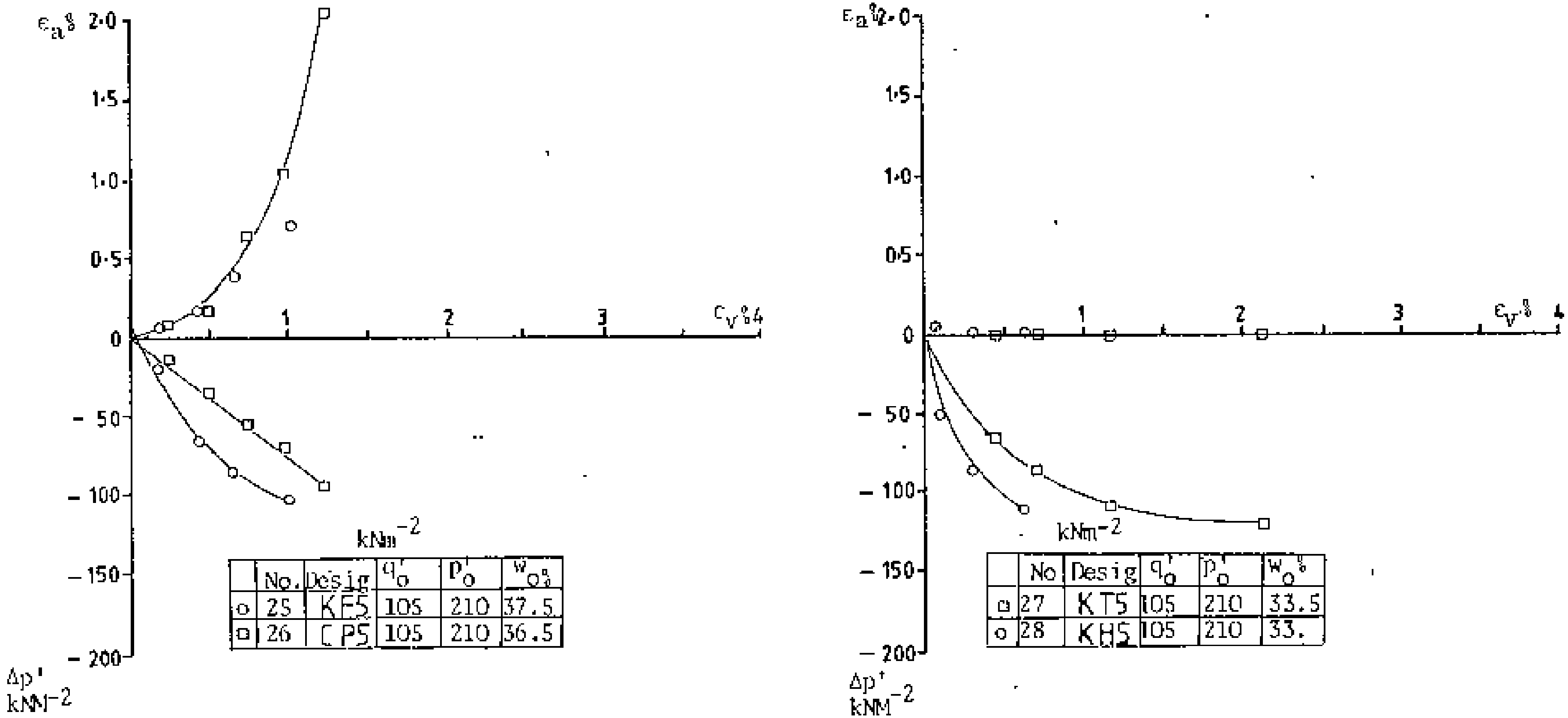


FIG. 4.27 BEHAVIOUR OF SAMPLES IN UNLOADING PATH 5

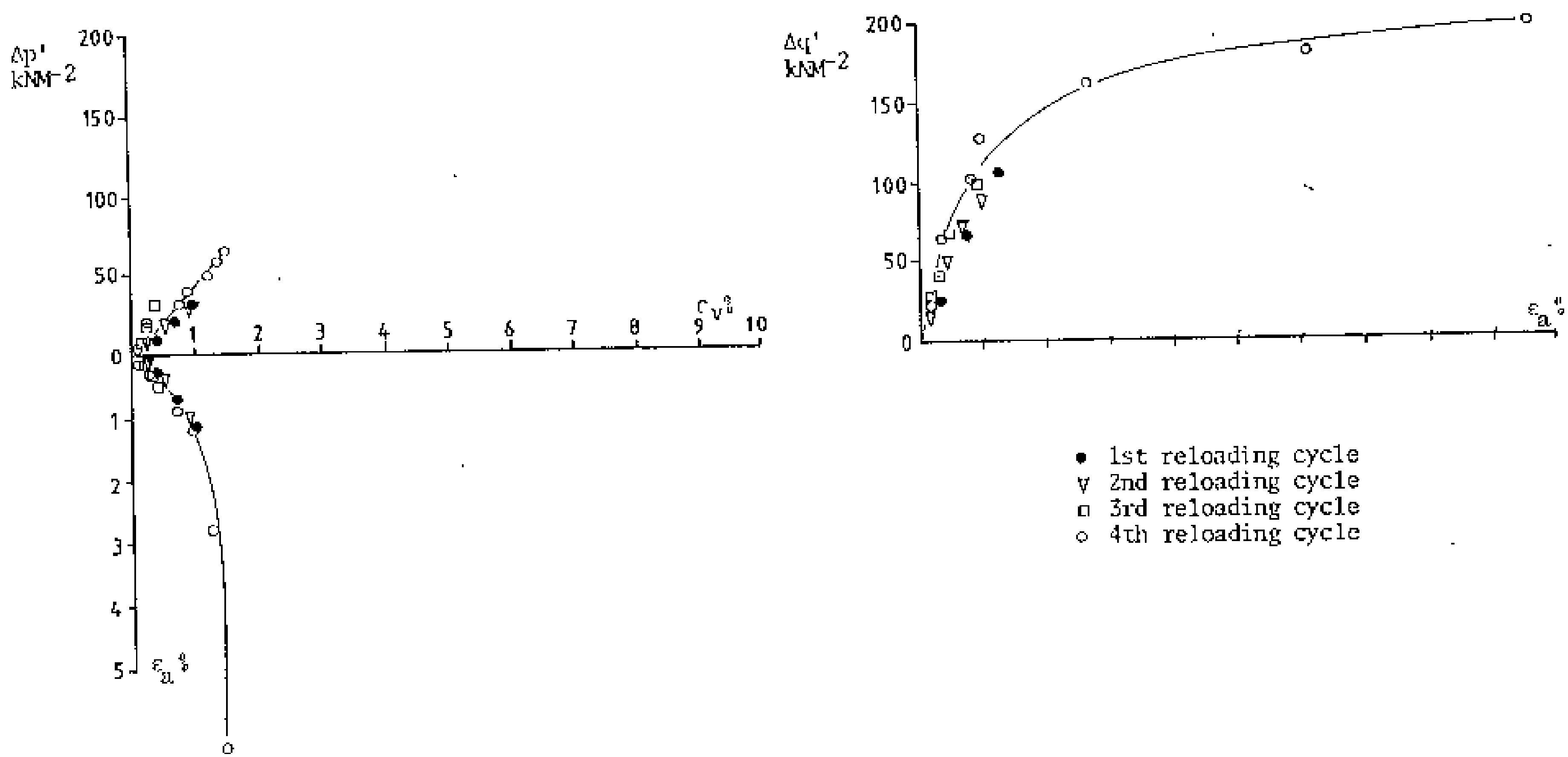
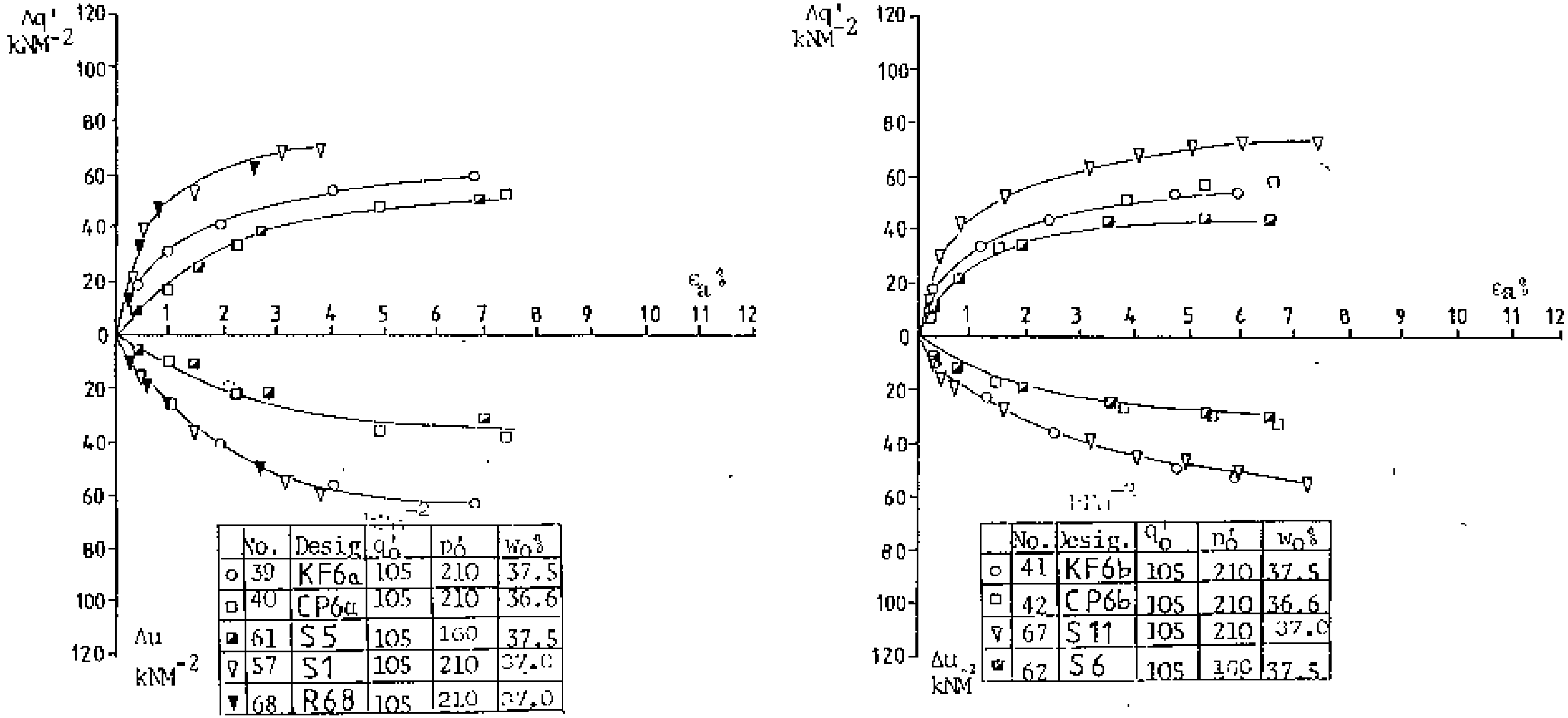


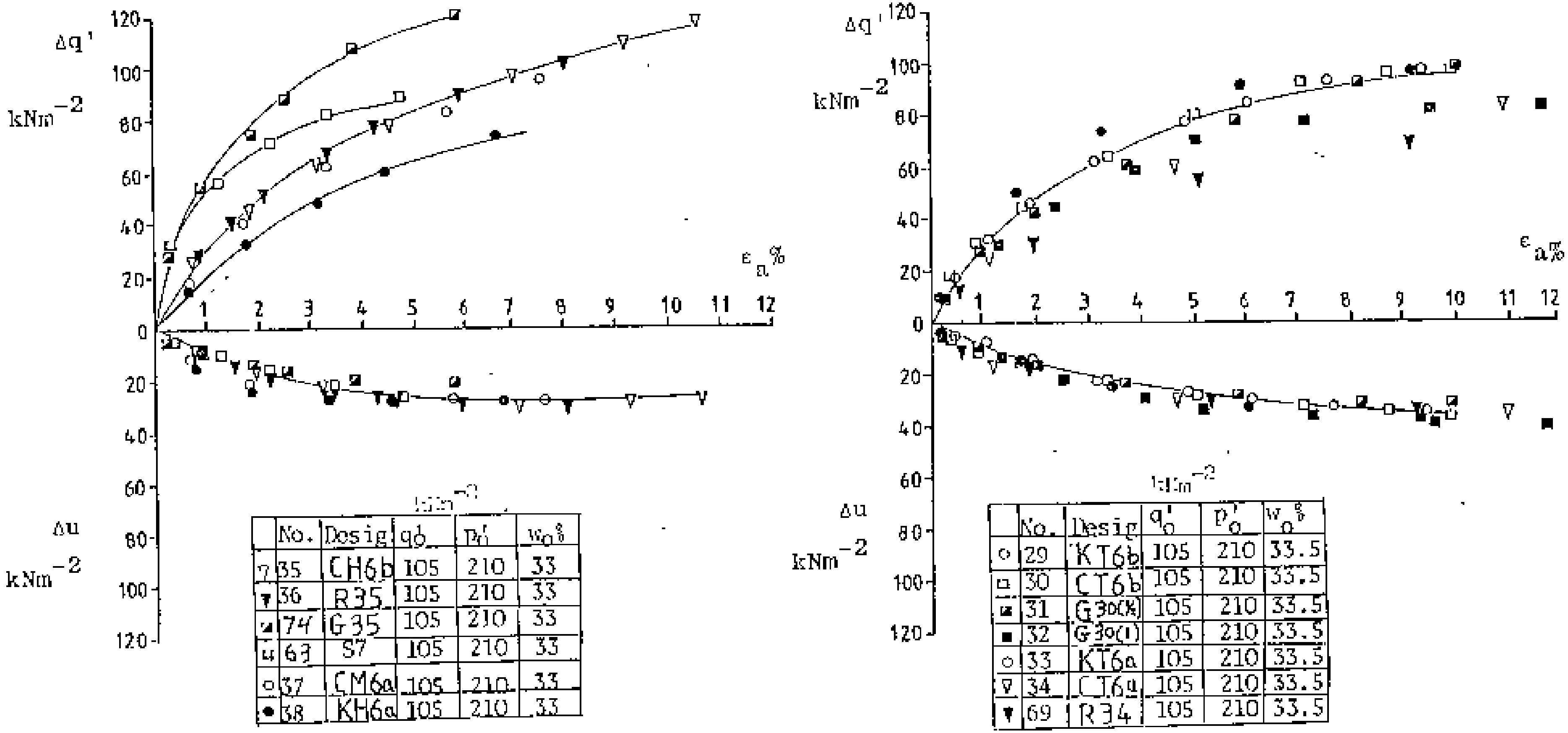
FIG. 4.28 BEHAVIOUR OF VIRGIN SAMPLE IN CYCLIC LOADING TEST S2



	No.	Desig.	q'_0	p'_0	w_0%
○	39	KF6a	105	210	37.5
□	40	CP6a	105	210	36.6
◪	61	S5	105	160	37.5
▽	57	S1	105	210	37.0
▼	68	R68	105	210	37.0

	No.	Desig.	q'_0	p'_0	w_0%
○	41	KF6b	105	210	37.5
□	42	CP6b	105	210	36.6
▽	67	S11	105	210	37.0
◪	62	S6	105	160	37.5

FIG. 4.29 BEHAVIOUR OF VIRGIN, PERFECT AND SPECIAL SAMPLES IN UNDRAINED LOADING



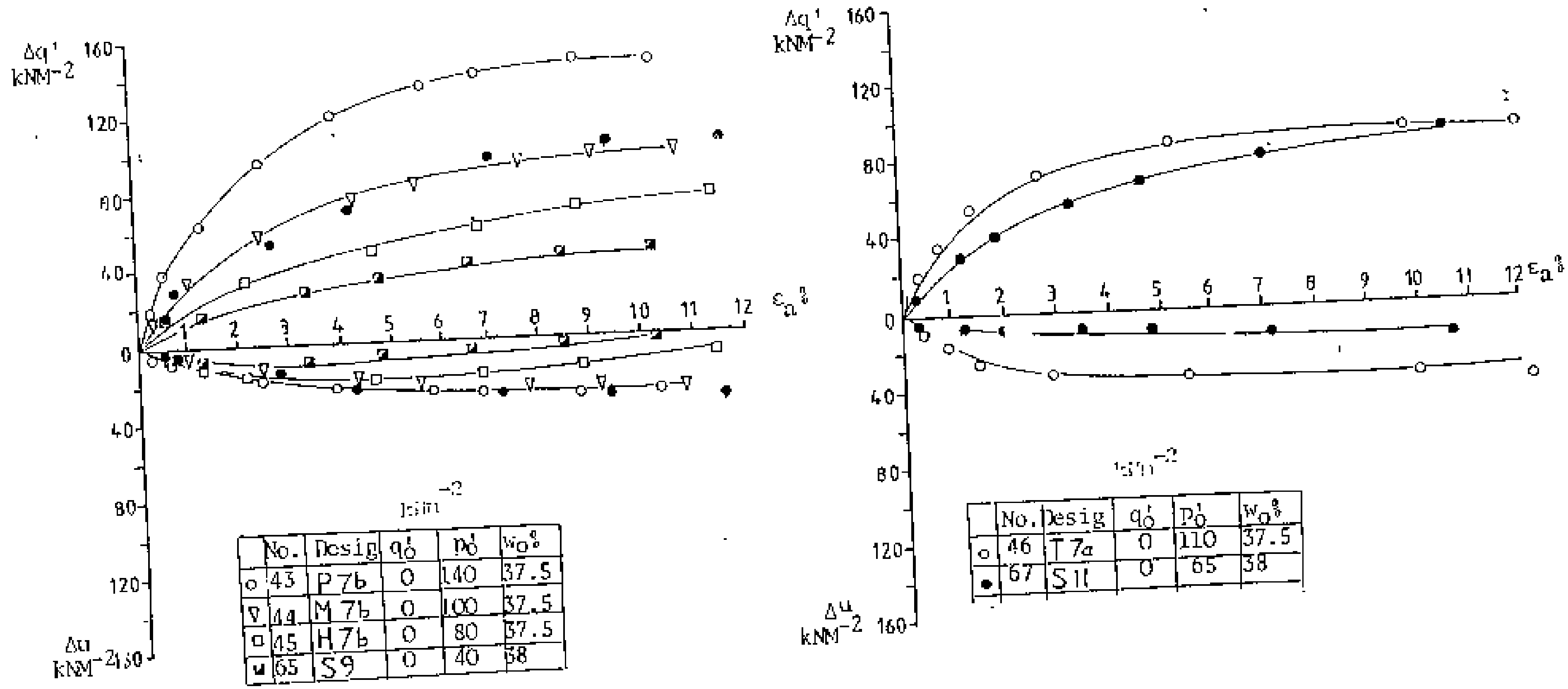
kNm^{-2}

	No.	Desig	q'_o	p'_o	w_o%
∇	35	CH6b	105	210	33
▼	36	R35	105	210	33
◪	74	G35	105	210	33
⊔	63	S7	105	210	33
○	37	CM6a	105	210	33
●	38	KH6a	105	210	33

kNm^{-2}

	No.	Desig	q'_o	p'_o	w_o%
○	29	KT6b	105	210	33.5
□	30	CT6b	105	210	33.5
◪	31	G30(K)	105	210	33.5
■	32	G30(1)	105	210	33.5
○	33	KT6a	105	210	33.5
∇	34	CT6a	105	210	33.5
▼	69	R34	105	210	33.5

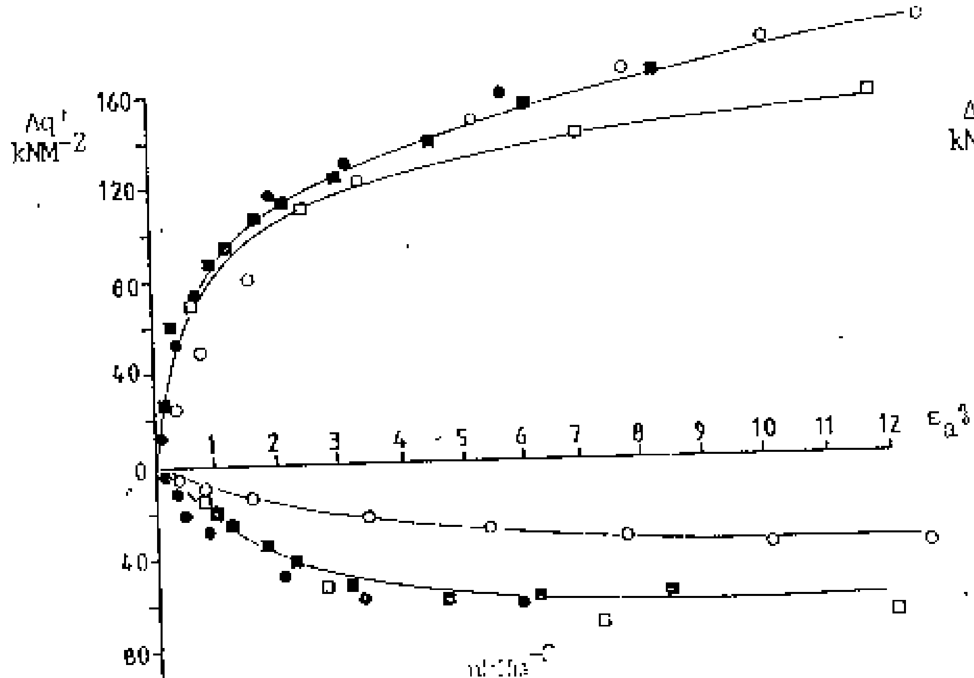
FIG. 4.30 BEHAVIOUR OF TUBE SAMPLES IN UNDRAINED LOADING



	No.	Desig	q_0'	p_0'	w_0%
○	43	P 7b	0	140	37.5
▽	44	M 7b	0	100	37.5
□	45	H 7b	0	80	37.5
■	65	S9	0	40	38

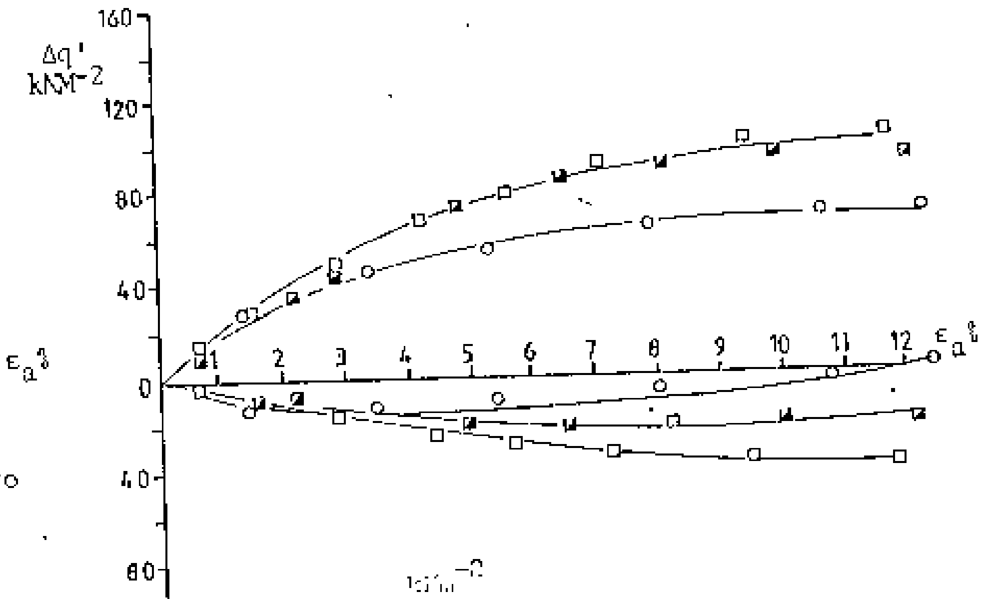
	No.	Desig	q_0'	p_0'	w_0%
○	46	T 7a	0	110	37.5
●	67	S11	0	65	38

FIG. 4.31 BEHAVIOUR OF DISTURBED SAMPLES IN UNCONSOLIDATED UNDRAINED TESTS



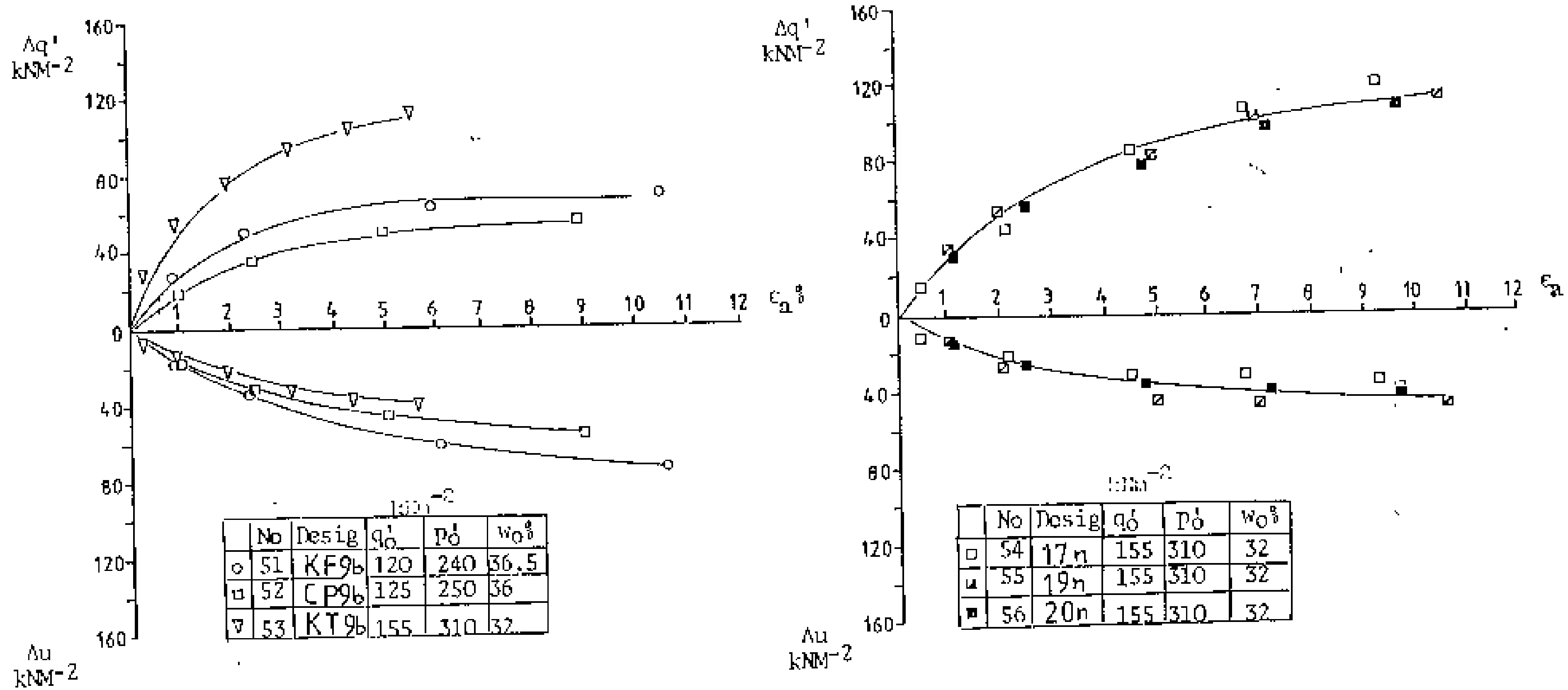
	No	Desig	q_o'	p_o'	w_o%
○	48	9n	105	290	36
□	49	10n	105	310	35
■	50	12n	105	310	32
○	64	S 8	0	280	32

FIG. 4.32 BEHAVIOUR OF SAMPLES AFTER ADDITIONAL ISOTROPIC CONSOLIDATION



	No	Desig	q_o	p_o'	w_o%
□	47	T7b	0	110	37.5
◪	73	G47	0	40	37.5
○	66	S10	0	65	38

FIG. 4.33 BEHAVIOUR OF TUBE SAMPLES IN UNCONSOLIDATED UNDRAINED TESTS



	No	Desig	q'_o	p'_o	w_o%
○	51	KF9b	120	240	36.5
□	52	CP9b	125	250	36
▽	53	KT9b	155	310	32

	No	Desig	q'_o	p'_o	w_o%
□	54	17n	155	310	32
⊠	55	19n	155	310	32
■	56	20n	155	310	32

FIG. 4.34 BEHAVIOUR OF SAMPLES AFTER ADDITIONAL ANISOTROPIC CONSOLIDATION

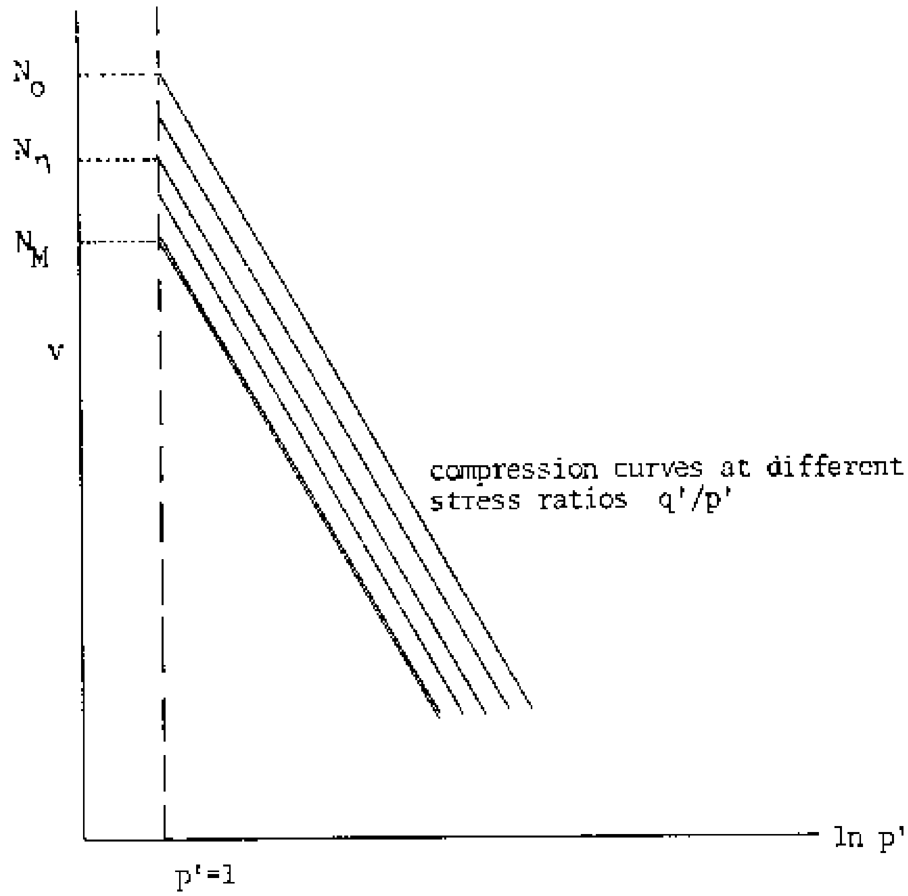
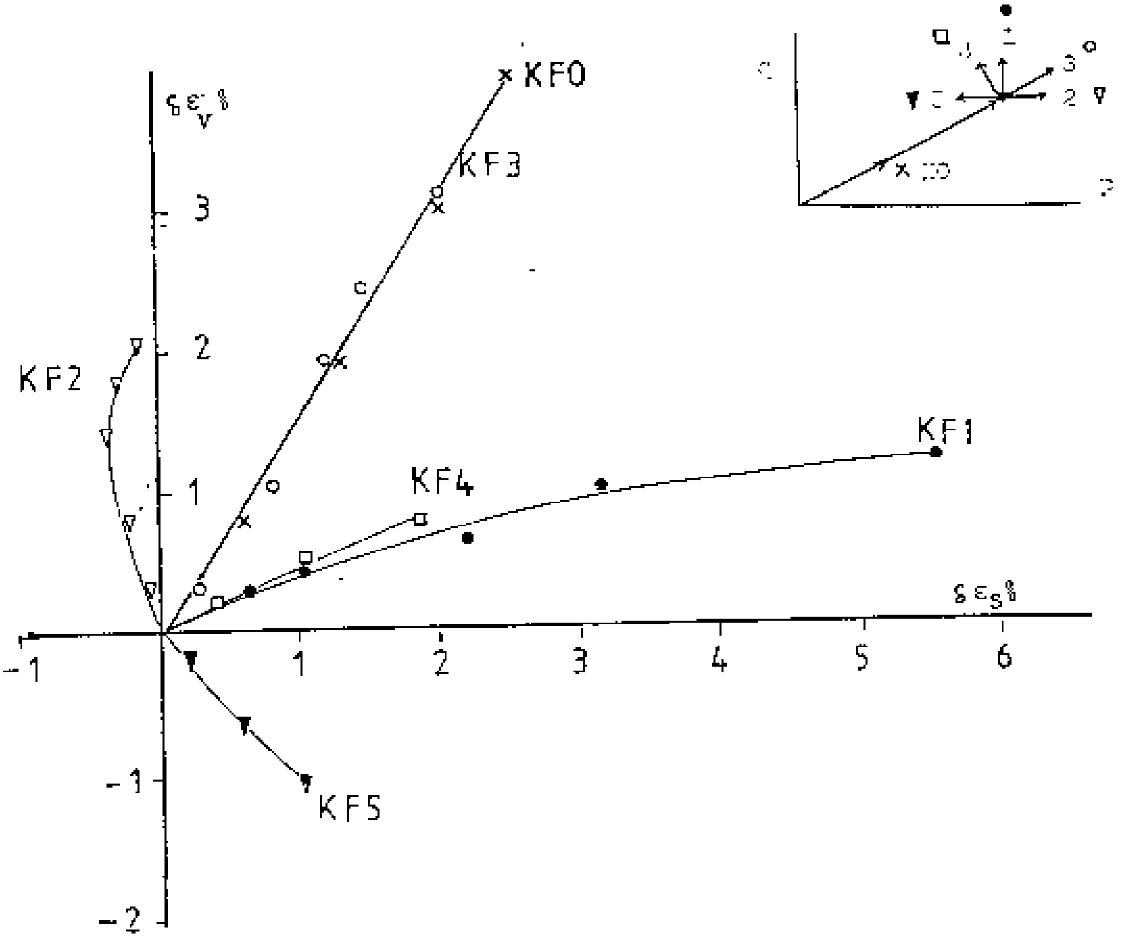
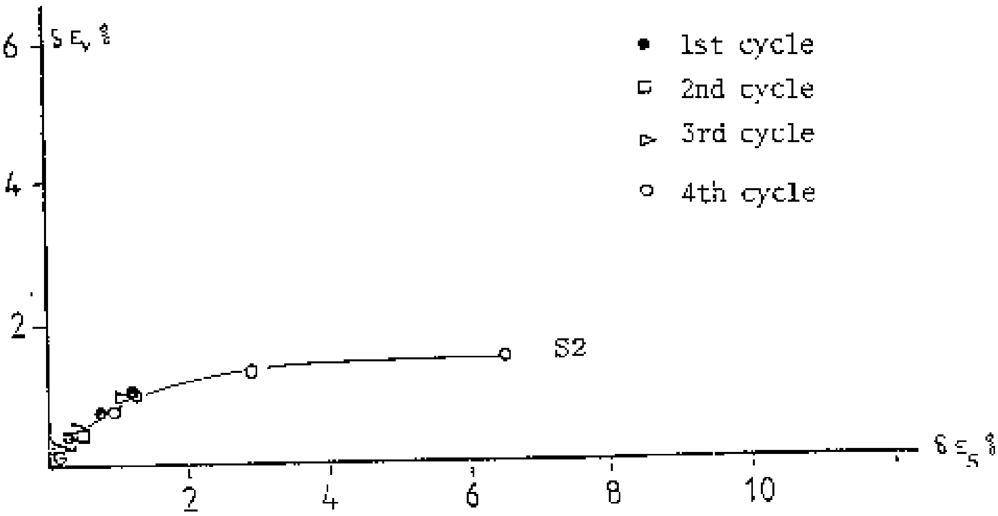


FIG. 5.1 LOCATION OF THE COMPRESSION CURVE AND THE N_η PARAMETER



(a) Virgin sample in different loading paths



(b) Cyclic loading in test S2

FIG. 5.2 STRAIN PATHS OF VIRGIN SAMPLES

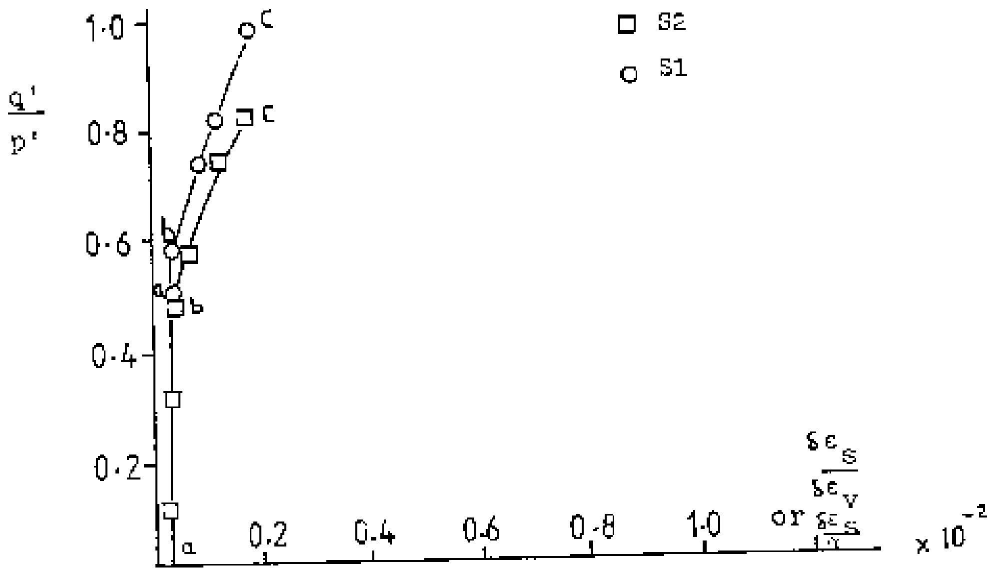


FIG. 5.3. RELATIONSHIPS BETWEEN STRESS AND STRAIN RATIOS OF OVERCONSOLIDATED SAMPLES

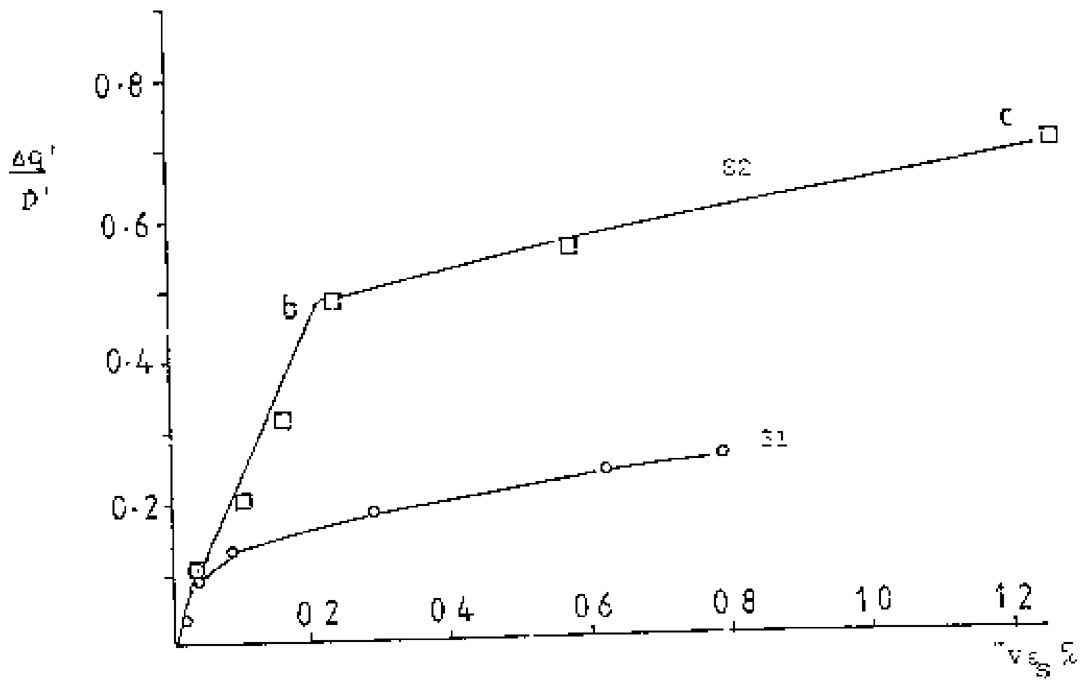


FIG. 5.4. NORMALISED STRESS-STRAIN RELATIONSHIPS OF OVERCONSOLIDATED SAMPLES

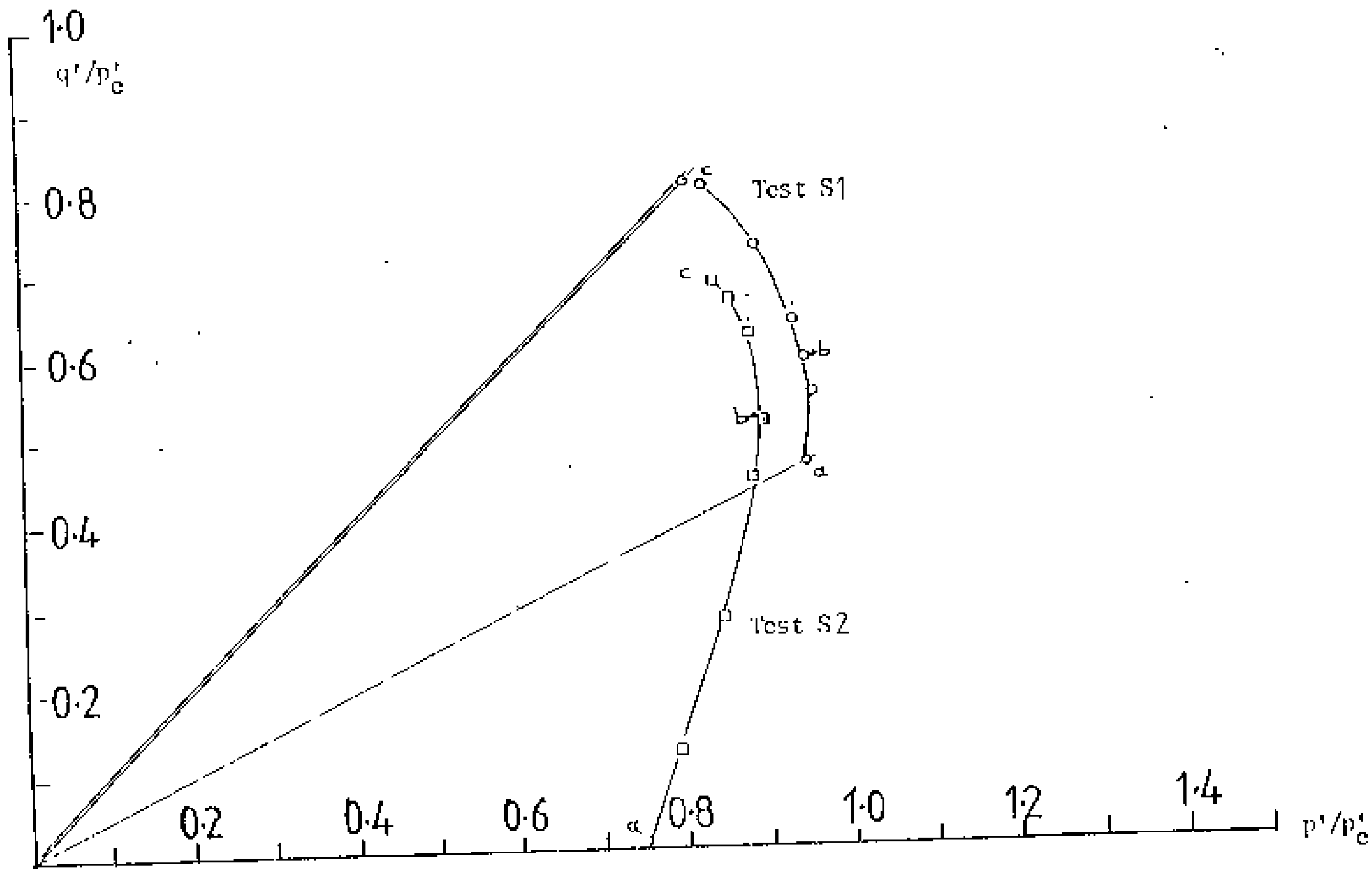


FIG. 5.5 NORMALISED STRESS PATHS FOR ELASTIC AND ELASTO-PLASTIC BEHAVIOUR

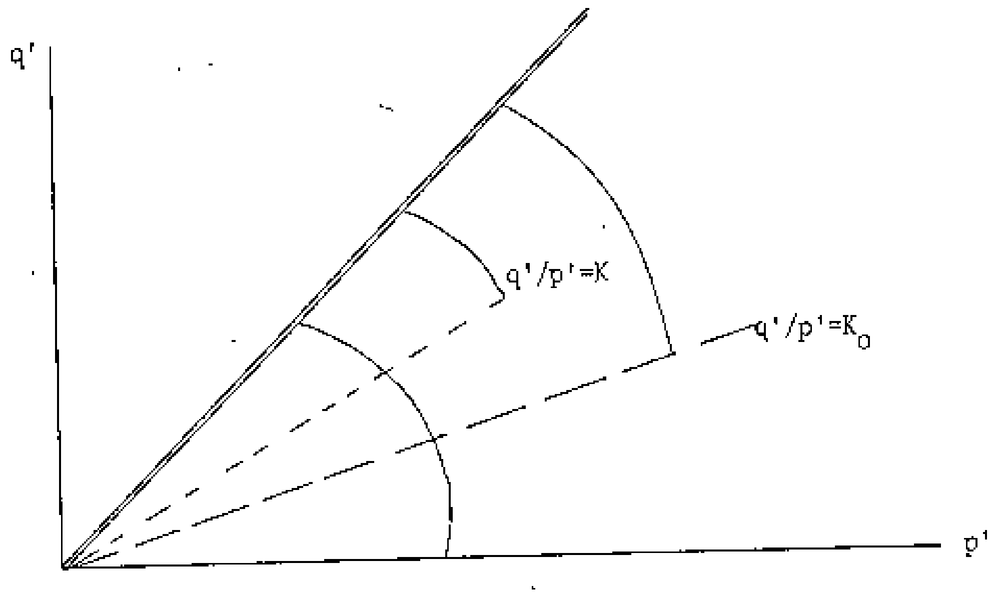


FIG. 5.6 DIFFERENT STRESS PATHS IN DIFFERENT TESTS

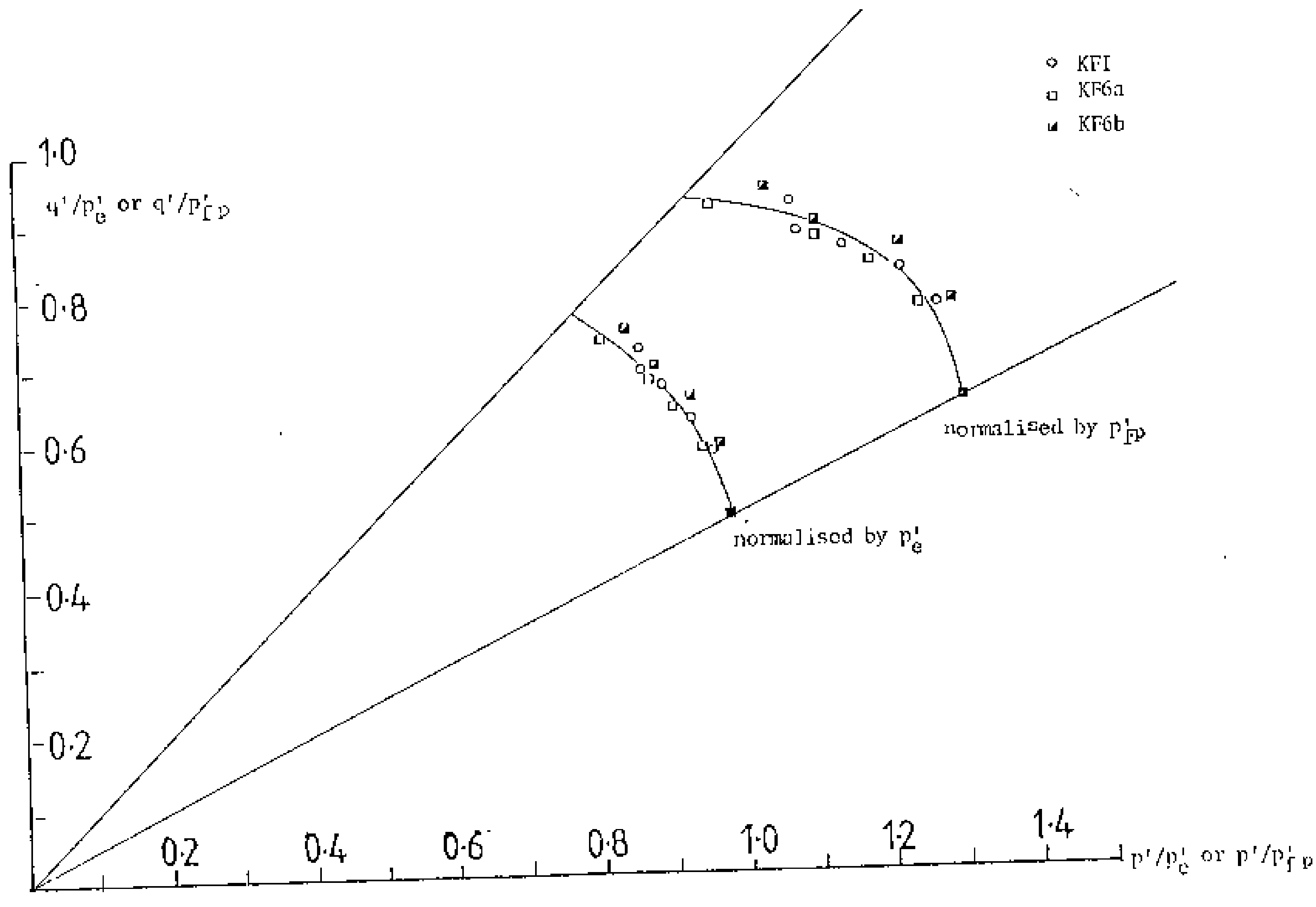


FIG. 5.7 STRESS PATHS NORMALISED BY DIFFERENT PARAMETERS

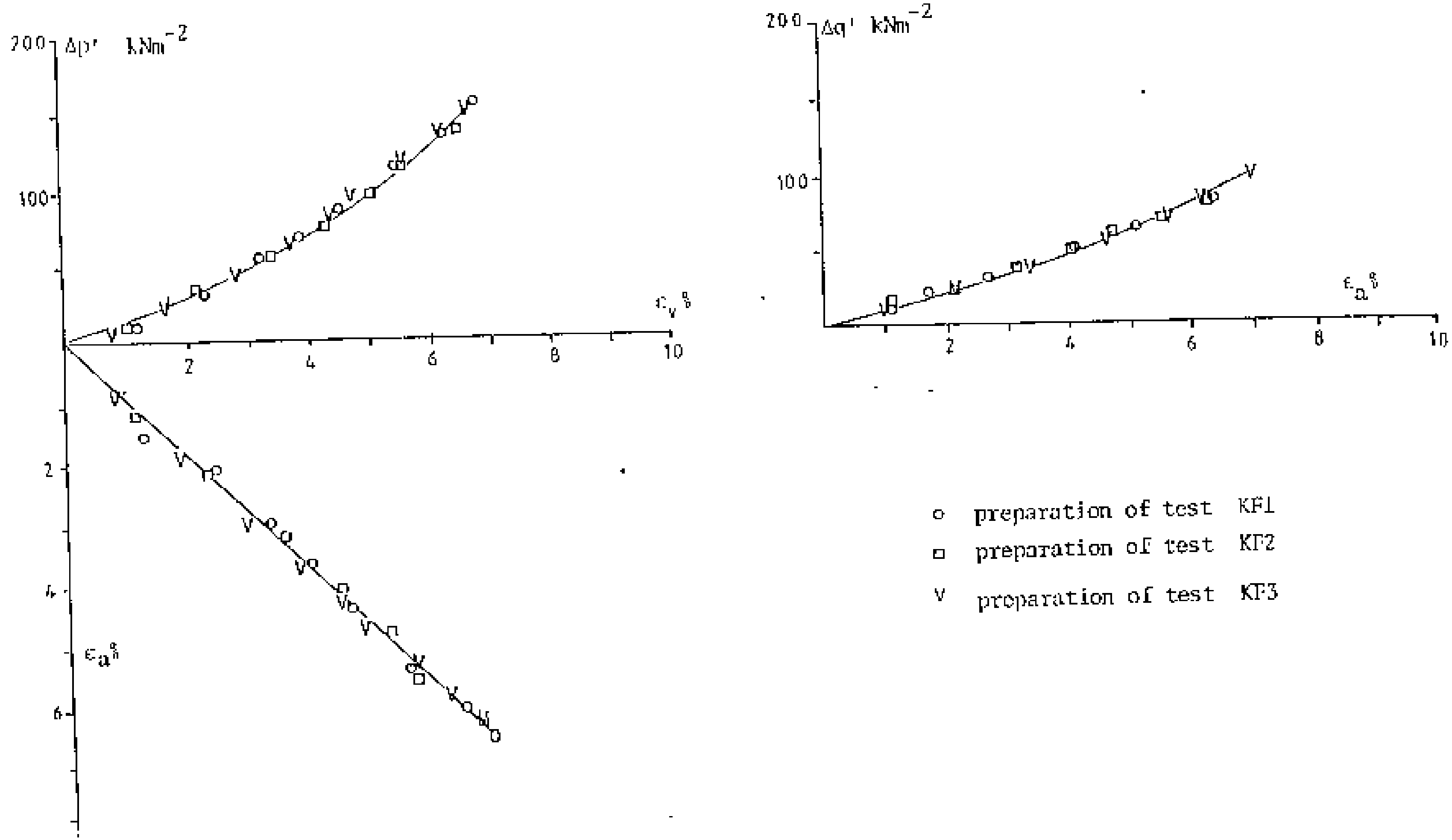


FIG. 5.8 THE CONSOLIDATION BEHAVIOUR OF VIRGIN SAMPLES

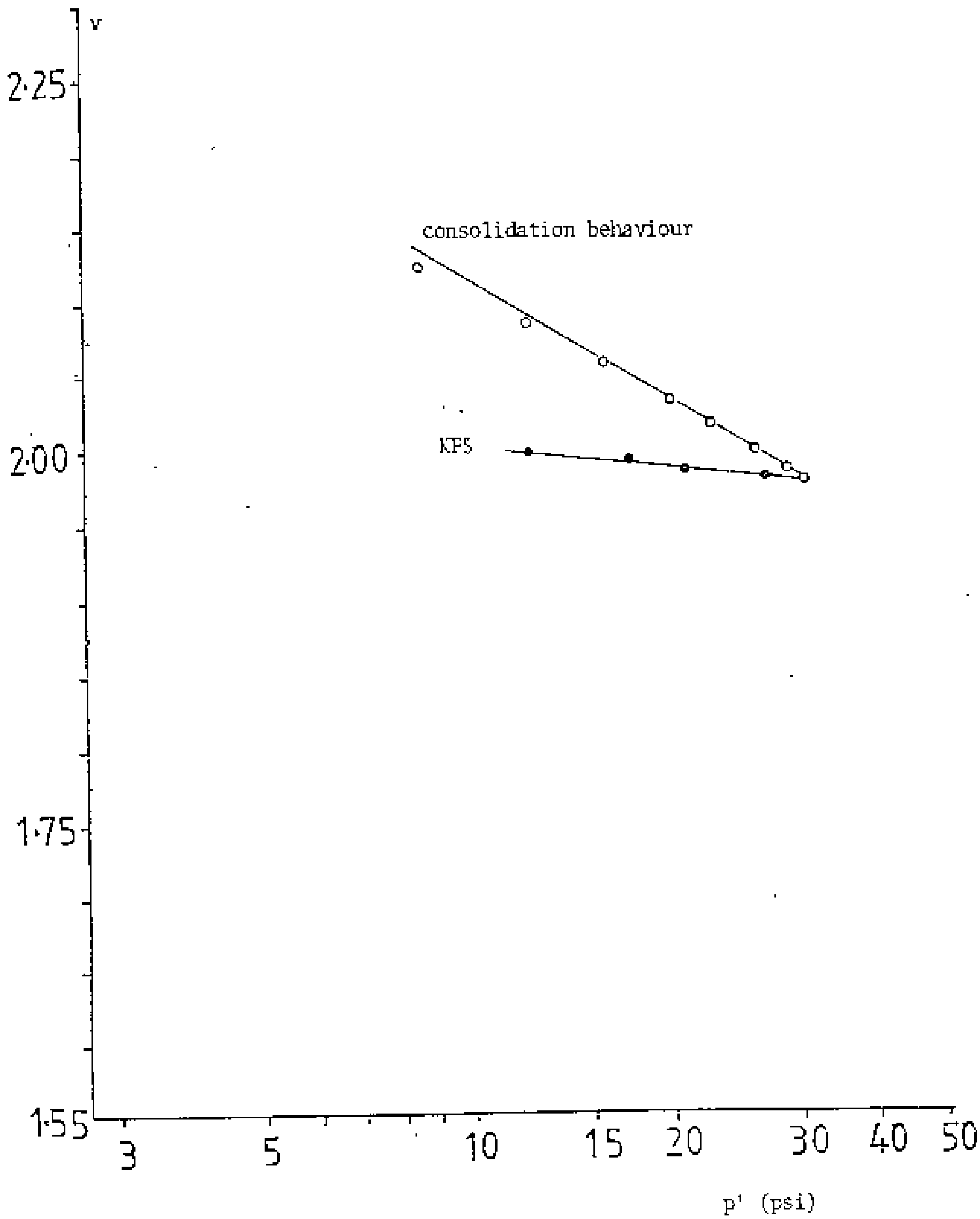


FIG. 5.9 COMPRESSION AND SWELLING CURVES OF VIRGIN SAMPLES

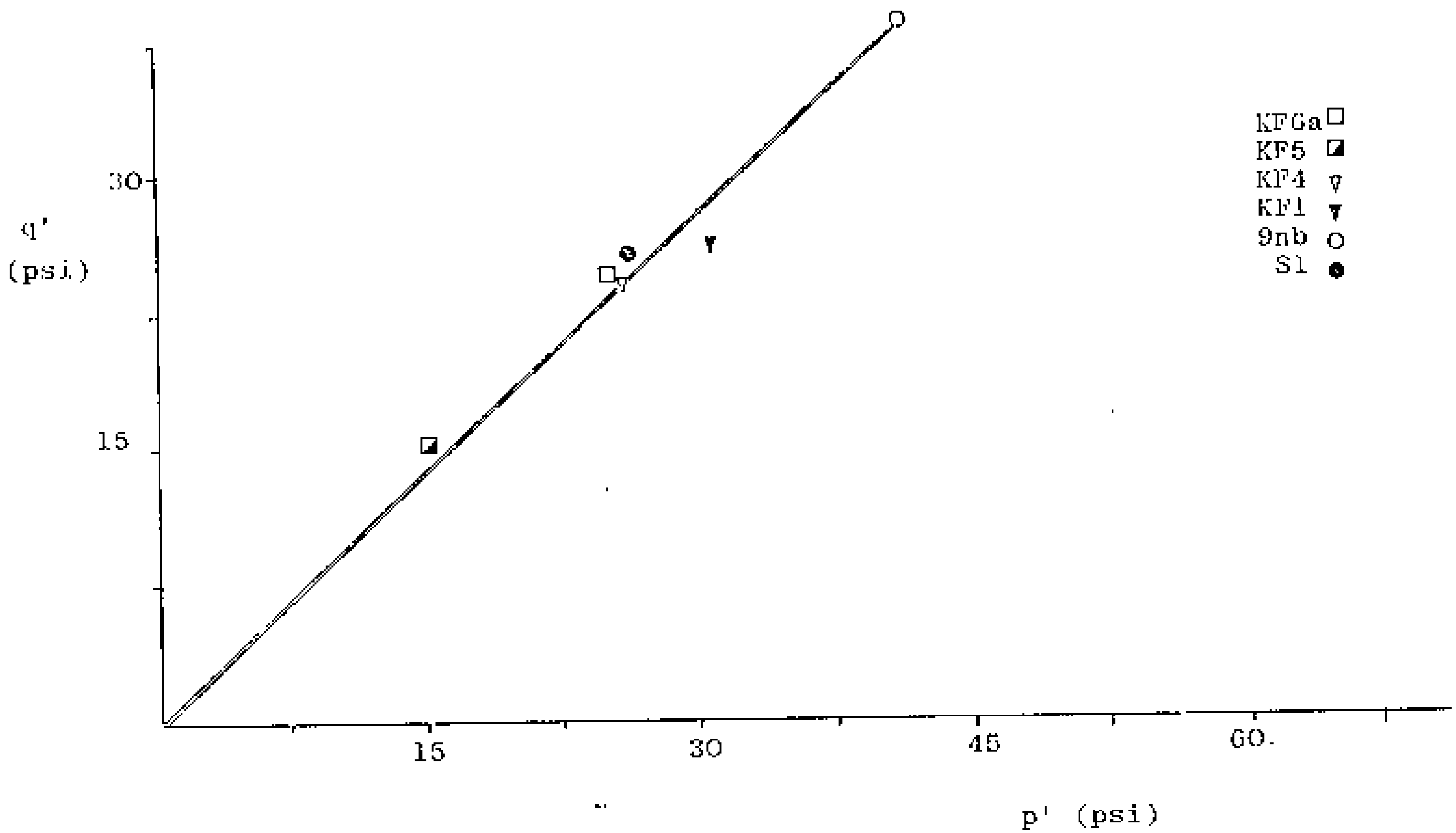


FIG. 5.10a. STRESS STATES OF VIRGIN SAMPLES AT FAILURE

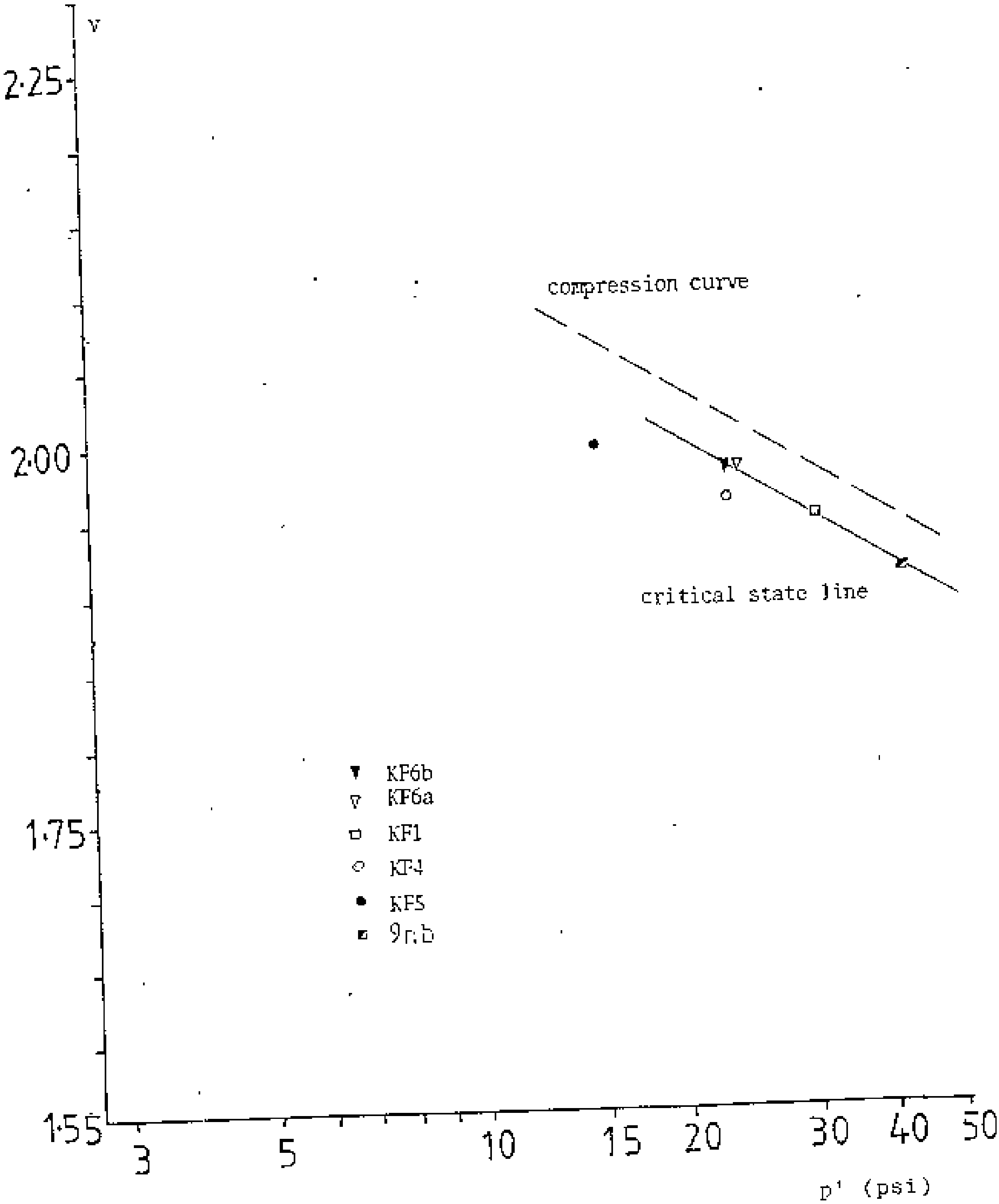


FIG. 5.10b THE CRITICAL STATE LINE

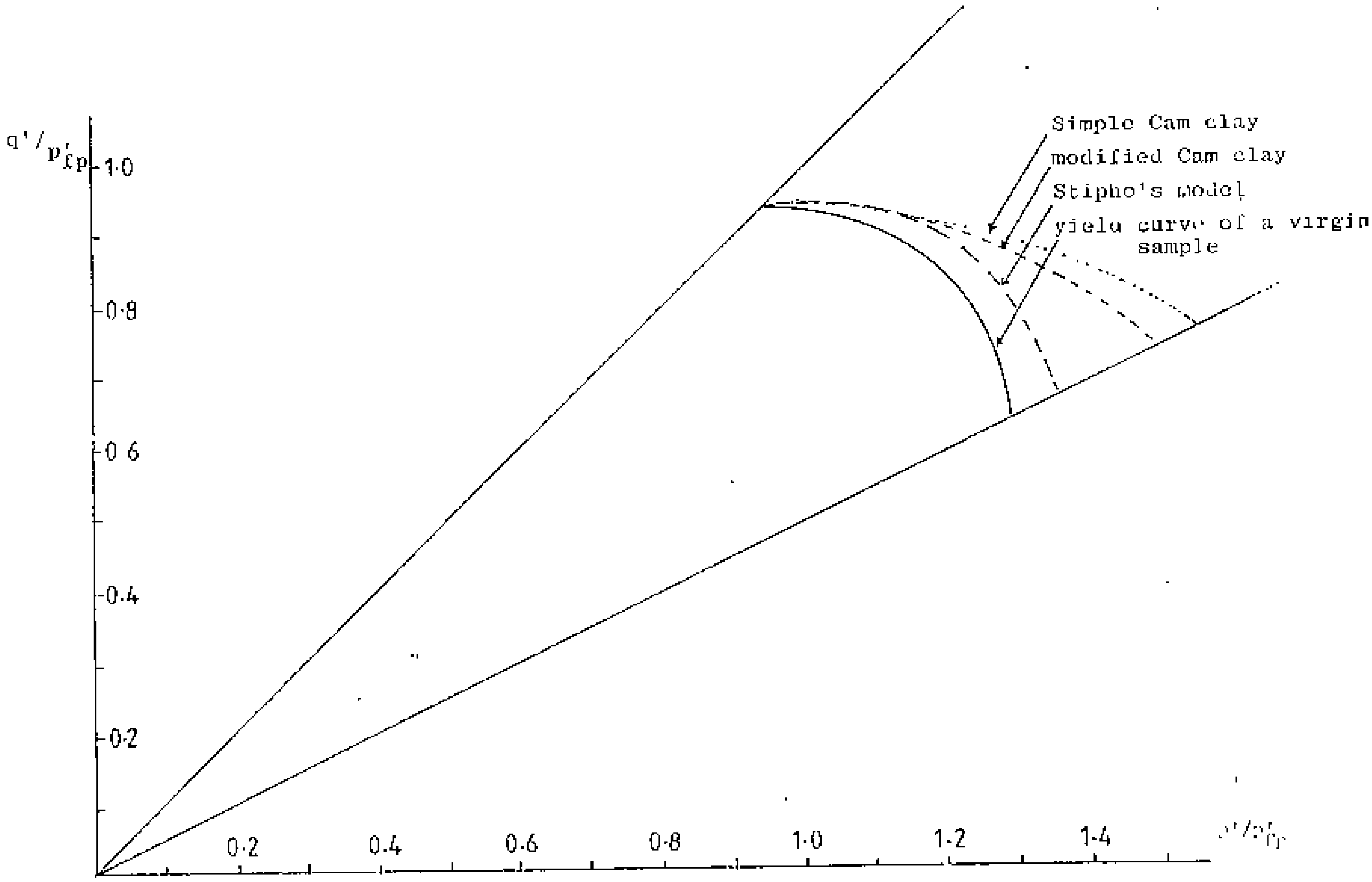


FIG. 5.11. THE YIELD CURVE OF VIRGIN SAMPLES COMPARED TO SOME ELASTO-PLASTIC MODELS

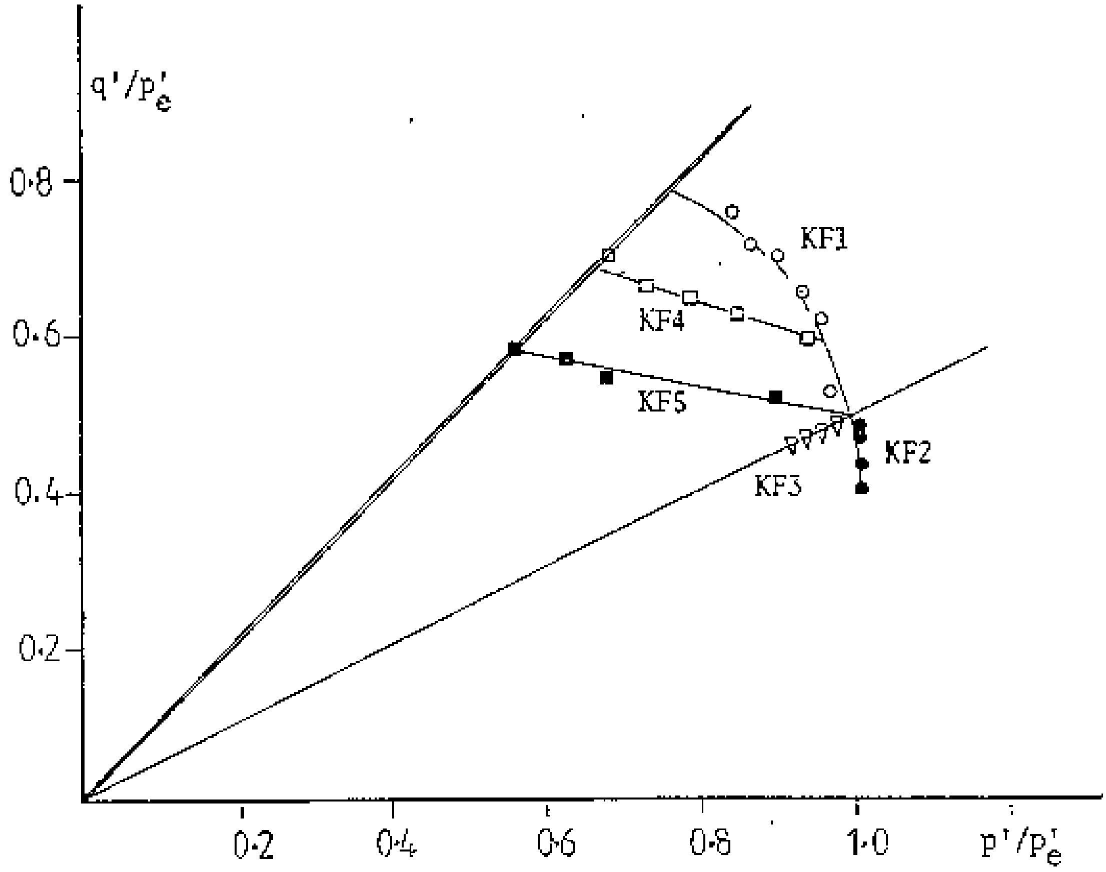


FIG. 5.12 NORMALISED STRESS PATH IN DRAINED TESTS

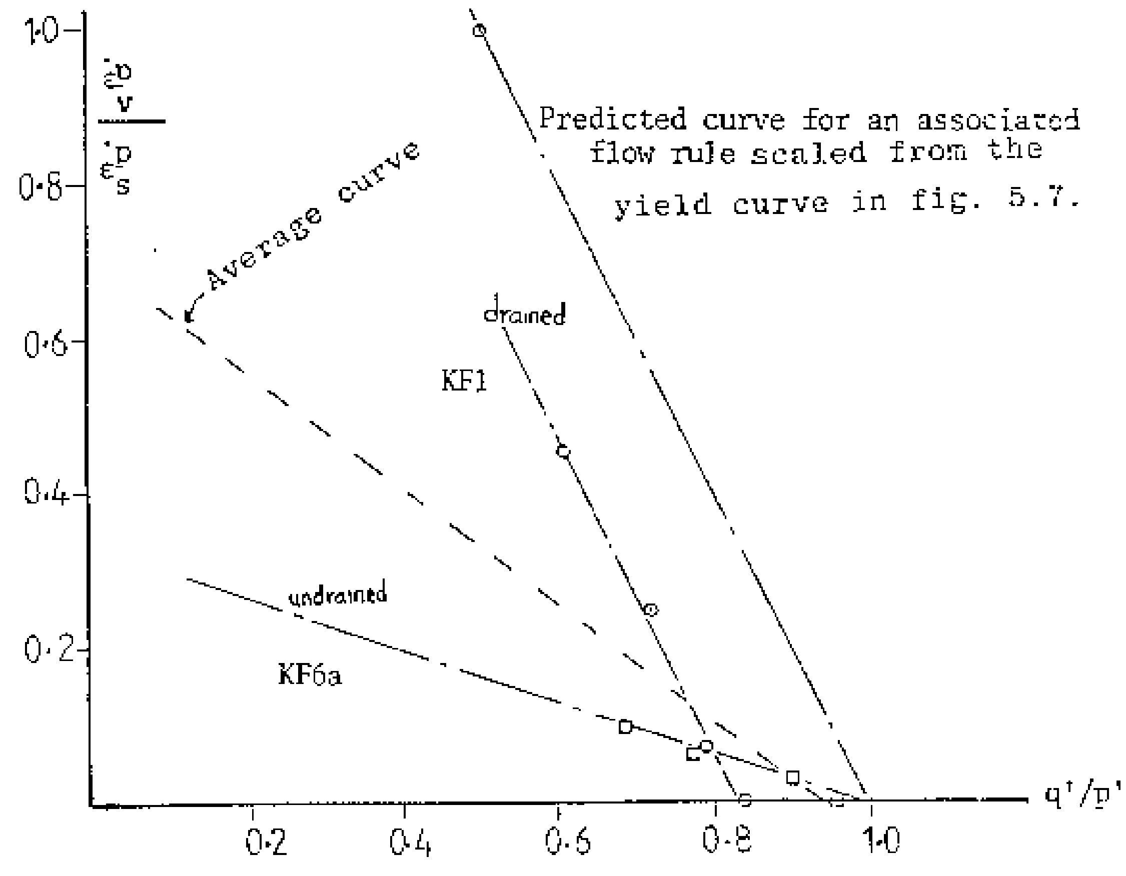


FIG. 5.13 RELATIONSHIP BETWEEN PLASTIC STRAIN INCREMENTS AND STRESS RATIO

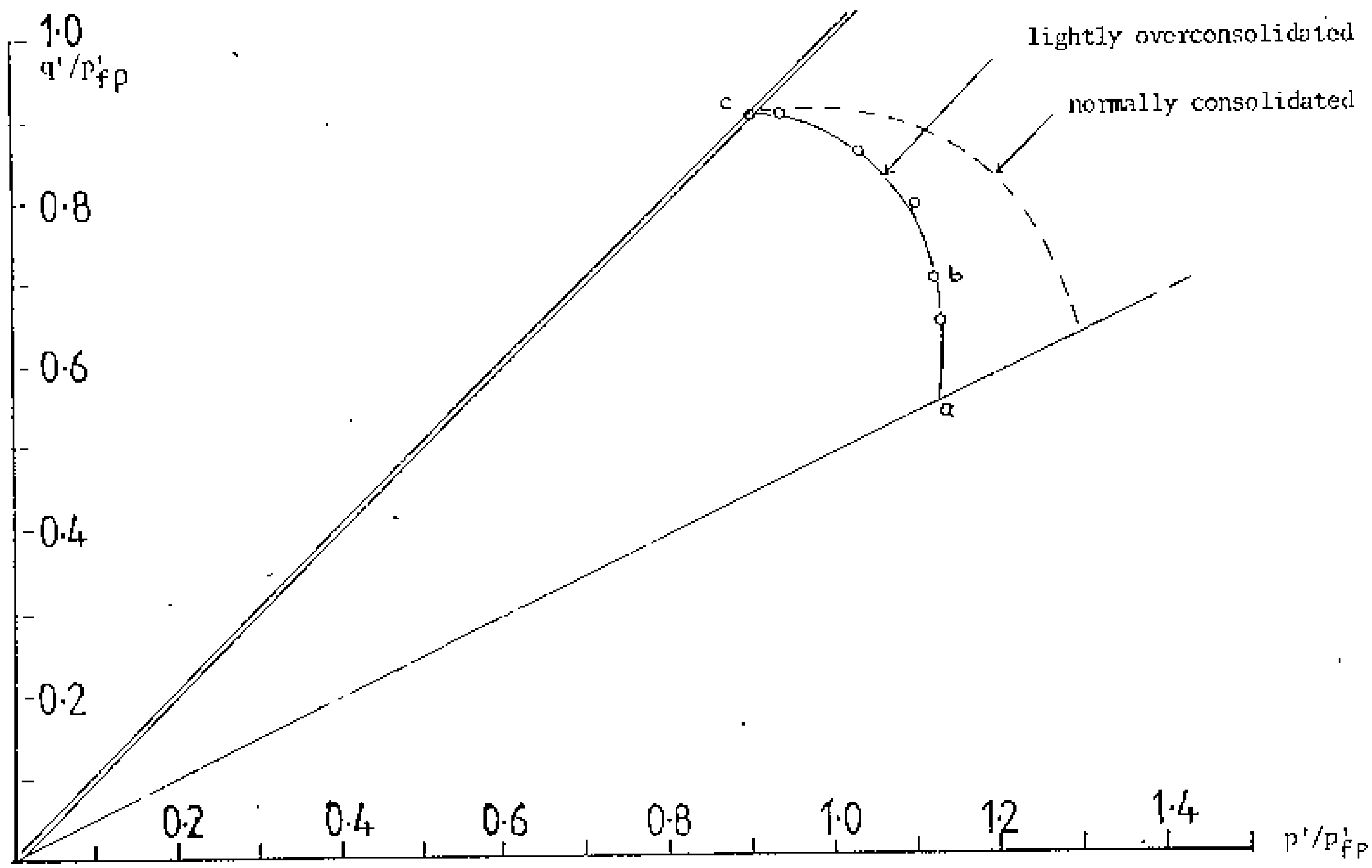


FIG. 5.14 NORMALISED STRESS PATH OF LIGHTLY OVERCONSOLIDATED SAMPLE

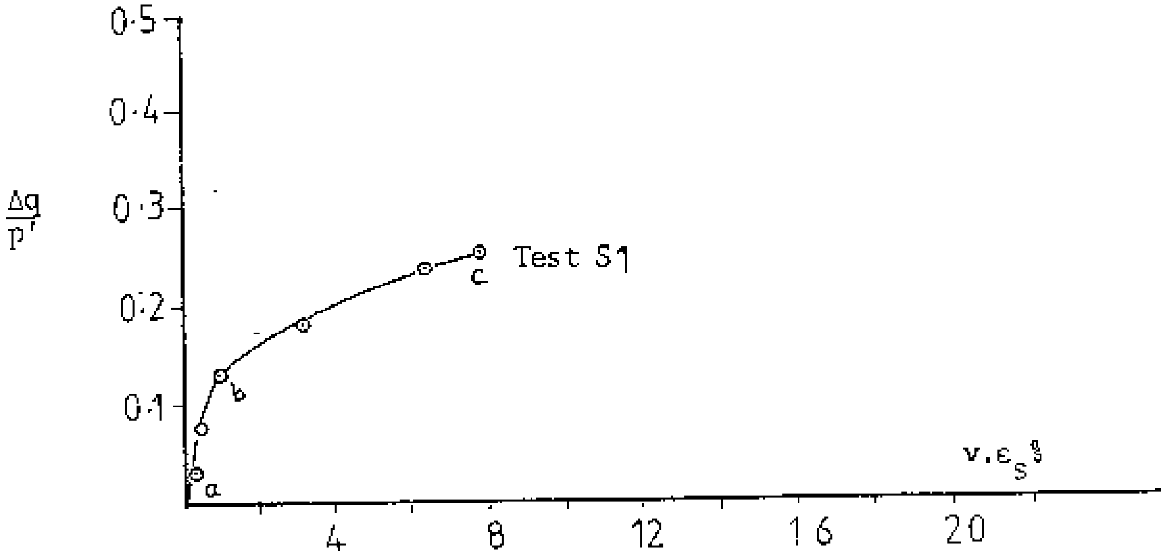


FIG. 5.15 NORMALISED STRESS STRAIN RELATIONSHIP OF LIGHTLY OVERCONSOLIDATED SAMPLE

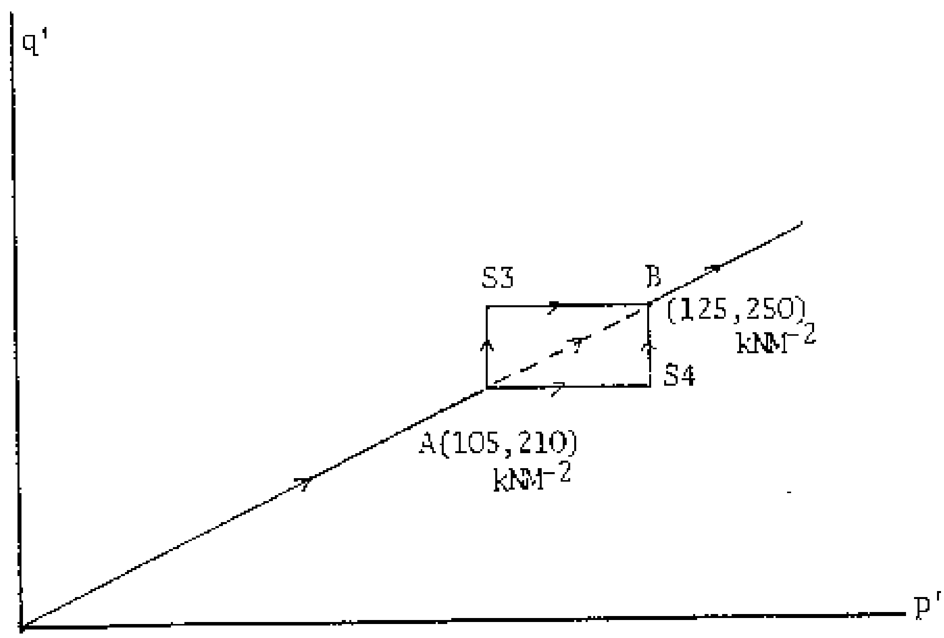


FIG. 5.16 PREPARATION AND LOADING PATHS OF TESTS S3 AND S4

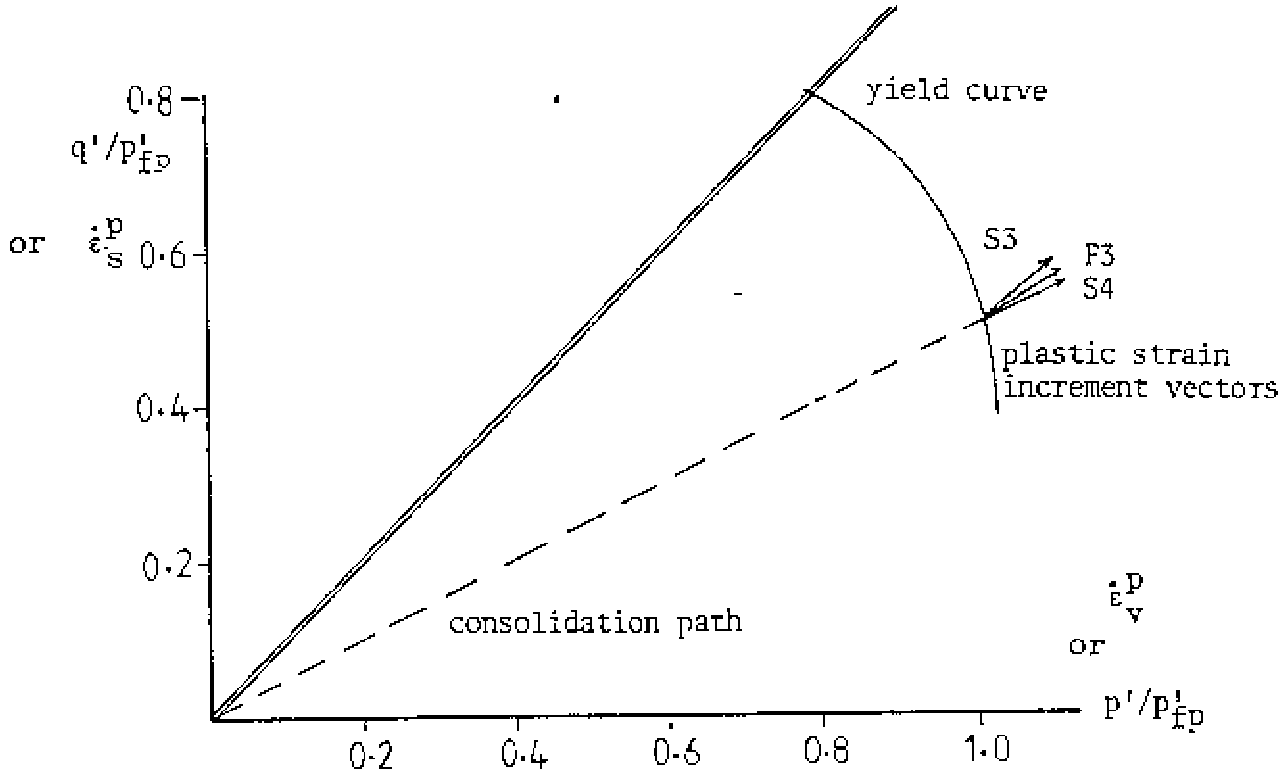


FIG. 5.17 PLASTIC STRAIN INCREMENT VECTORS IN VIRGIN SAMPLES WITH DIFFERENT STRESS HISTORIES

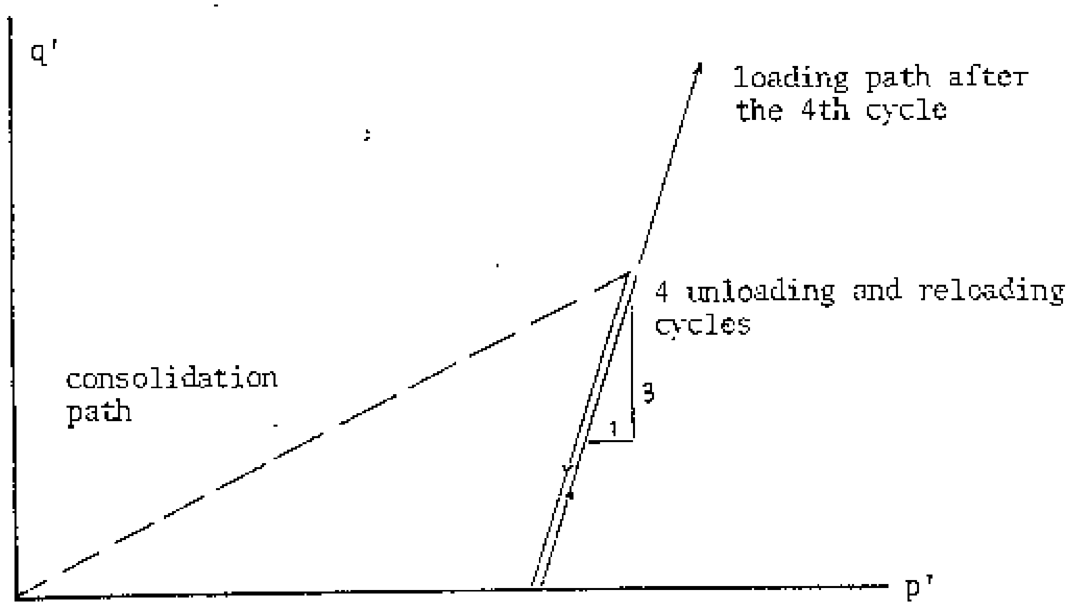


FIG. 5.18 STRESS PATH IN TEST S2

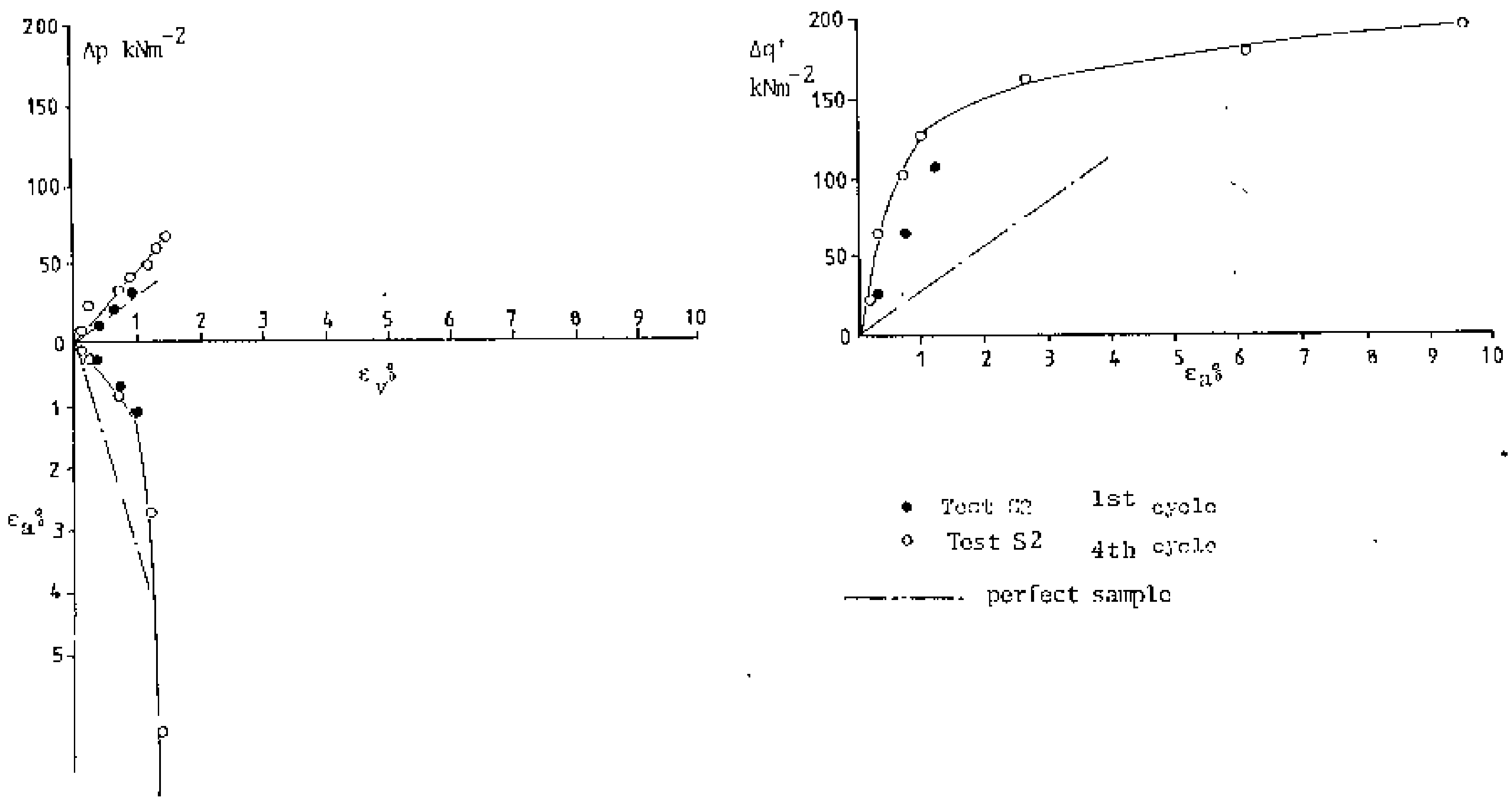


FIG. 5.19 THE RELOADING BEHAVIOUR OF A PERFECT SAMPLE AND A CYCLICALLY DISTURBED SAMPLE

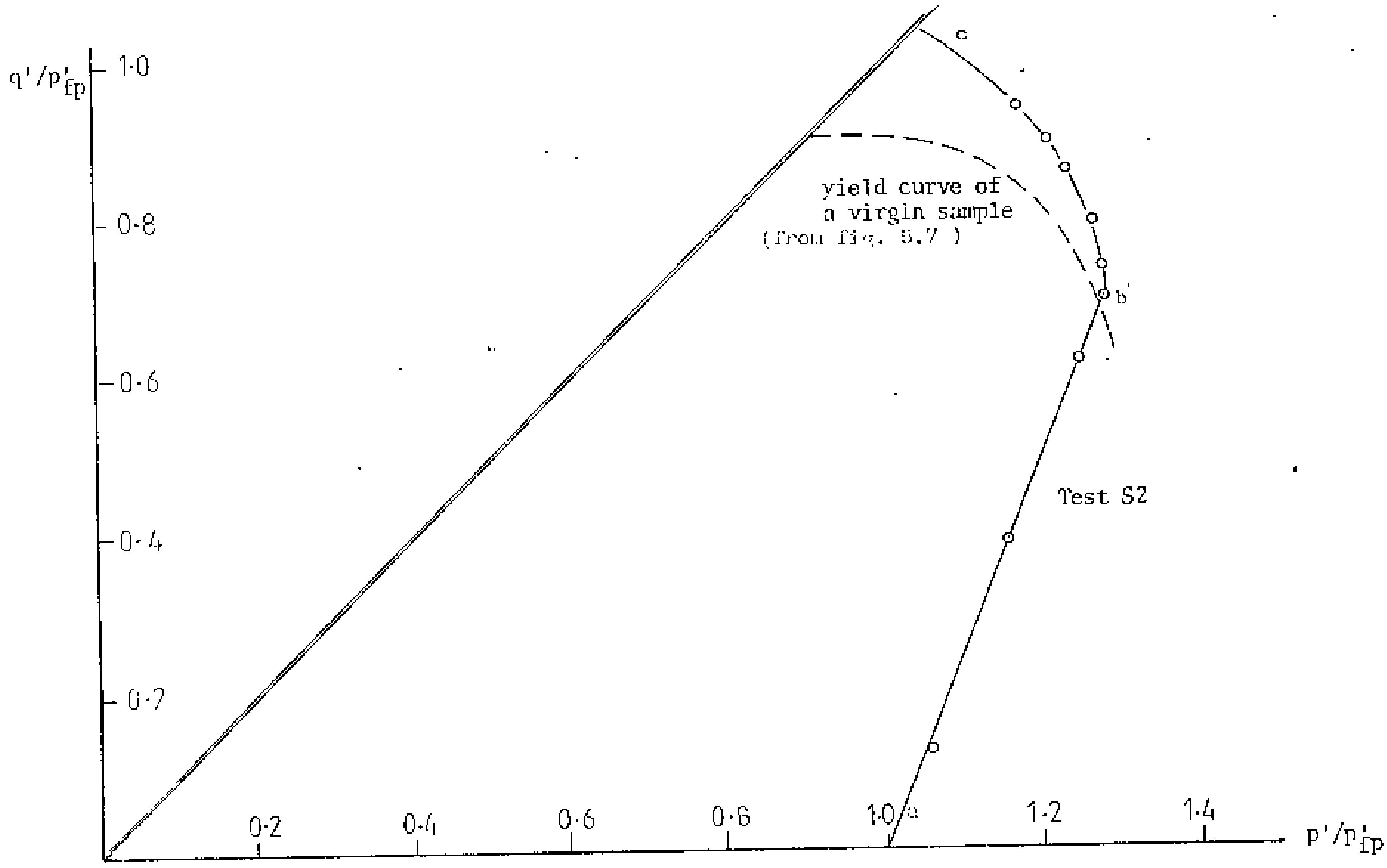


FIG. 5.20 NORMALISED STRESS PATHS OF VIRGIN AND CYCLICALLY DISTURBED SAMPLE

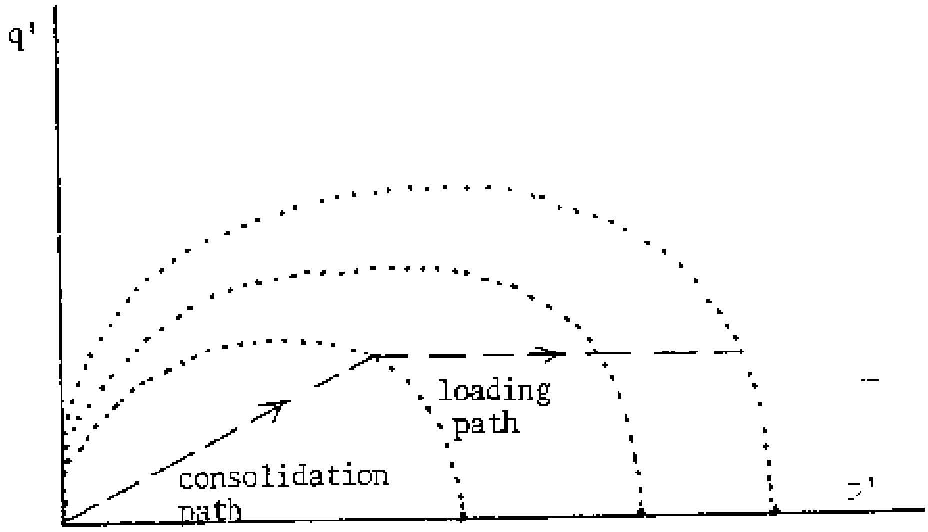


FIG. 5.21 ISOTROPIC HARDENING

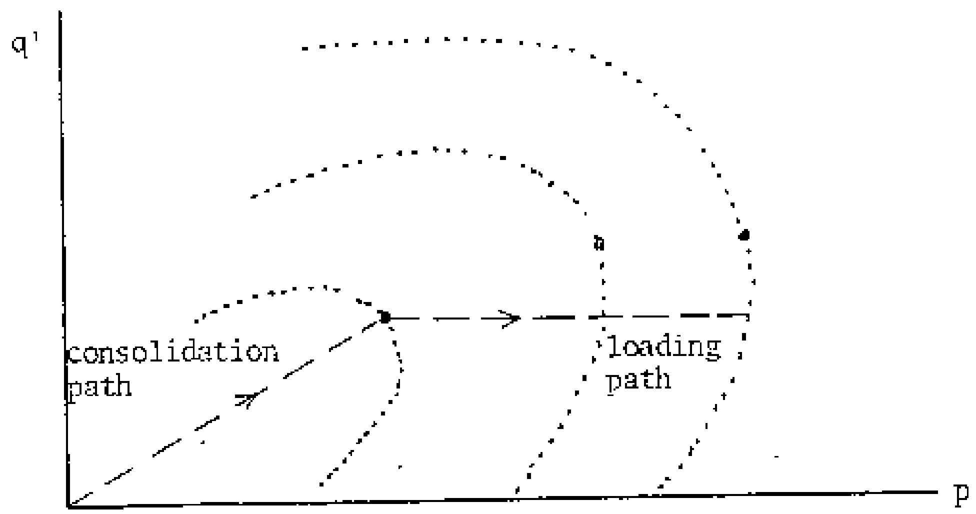


FIG. 5.22 KINEMATIC HARDENING

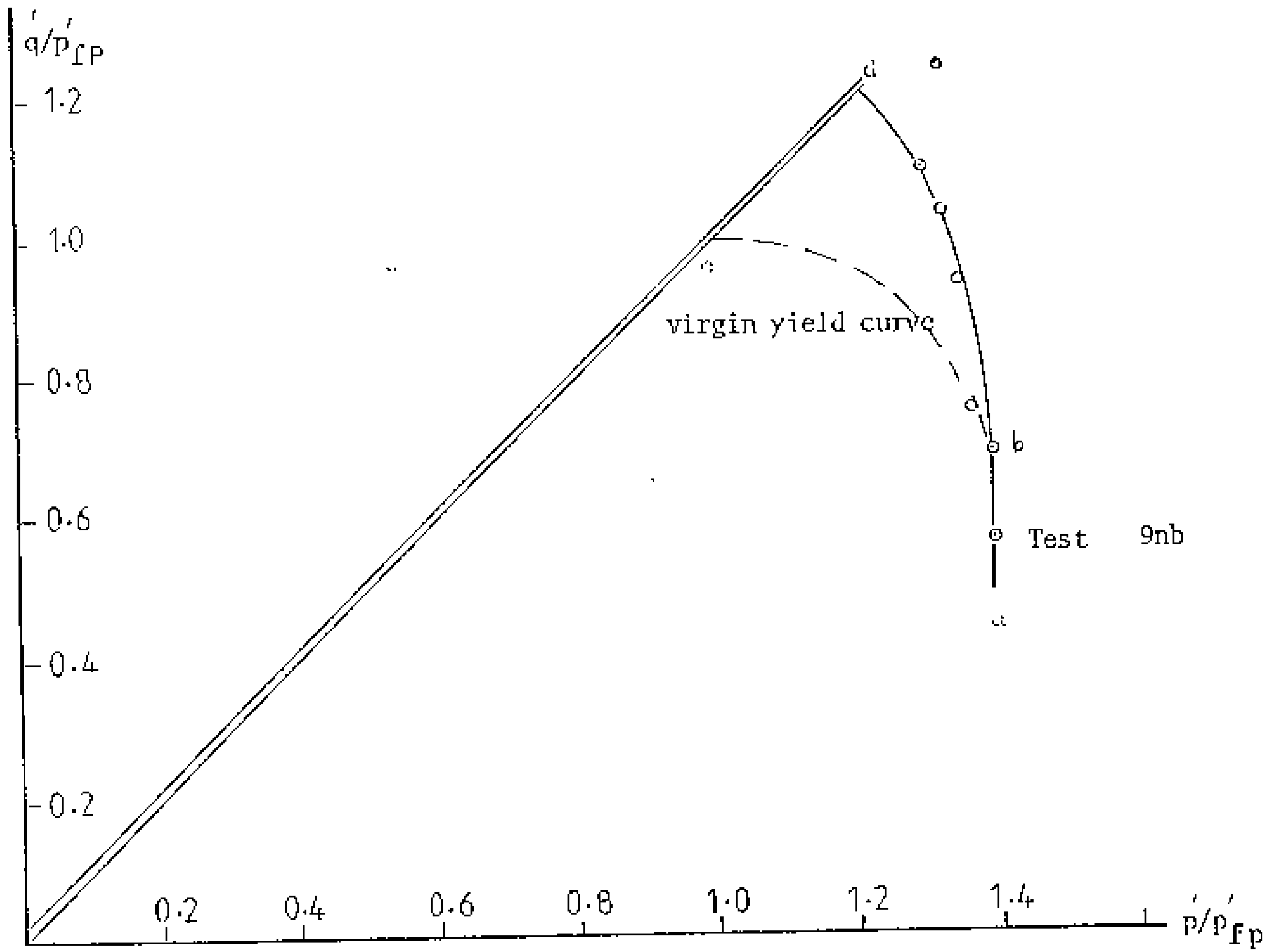


FIG. 5.23 EFFECTS OF ADDITIONAL ISOTROPIC LOADING ON THE YIELD CURVE OF AN ANISOTROPIC VIRGIN SAMPLE

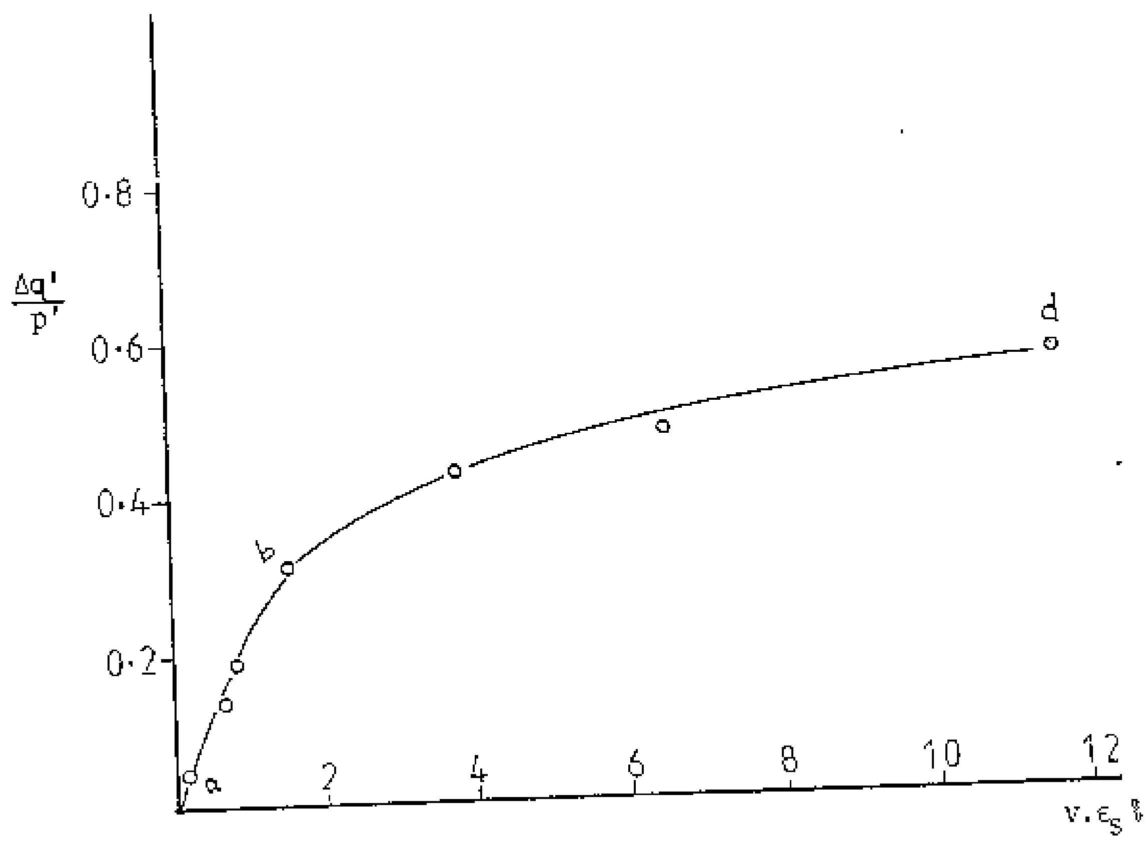
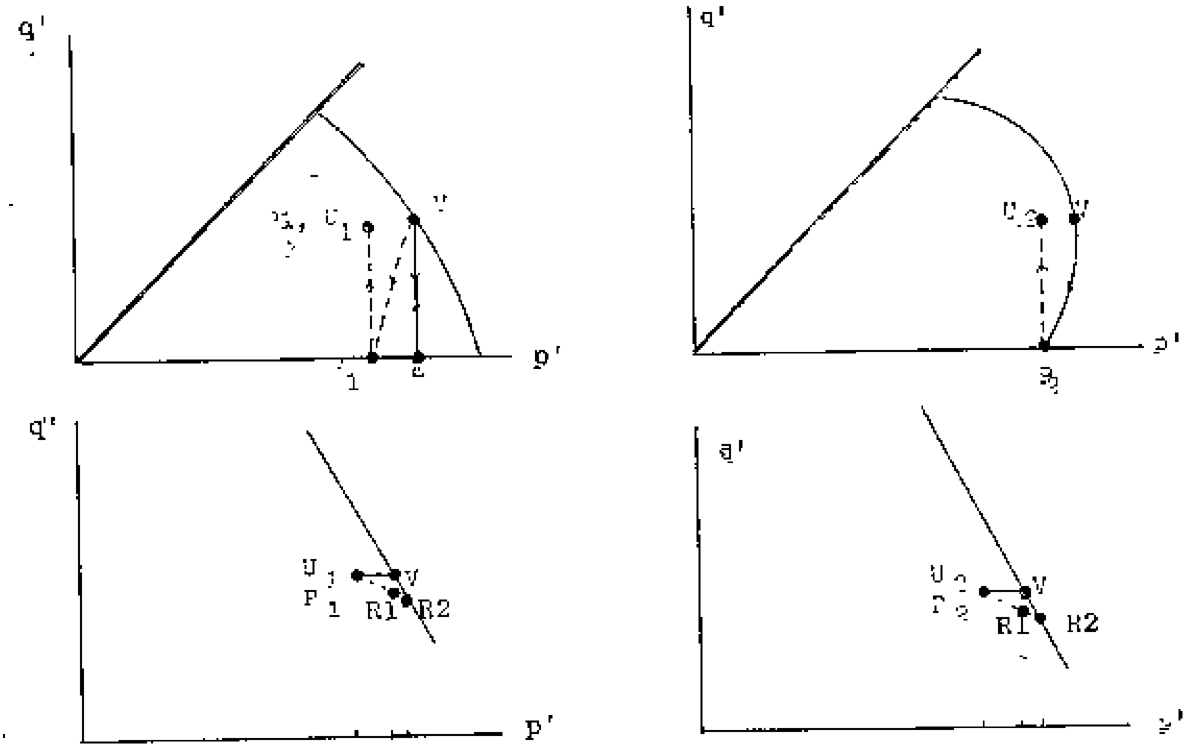
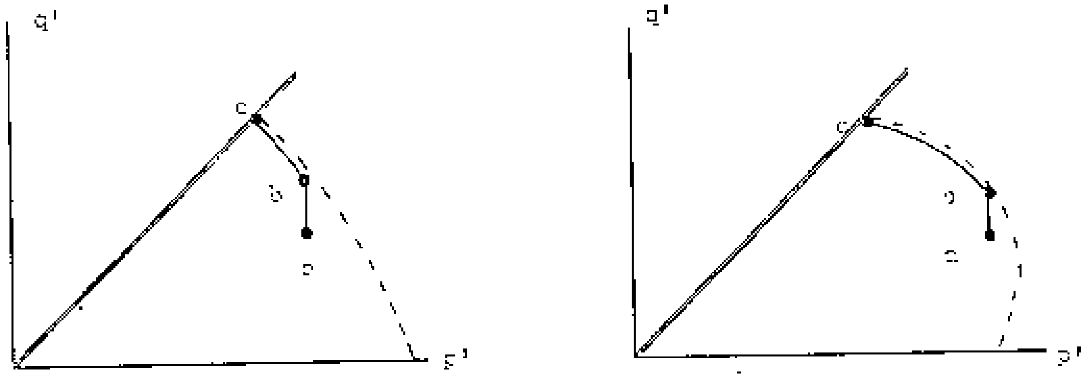


FIG. 5.24 NORMALISED STRESS-STRAIN RELATIONSHIPS OF TEST 9nb



(a) Elastic and in-elastic behaviour in unloading and reloading

(b) Elasto-plastic and elastic behaviour in unloading and reloading



(c) Elastic and elasto-plastic behaviour of reloaded samples

FIG. 6.1. THEORETICAL BEHAVIOUR OF PERFECT SAMPLES

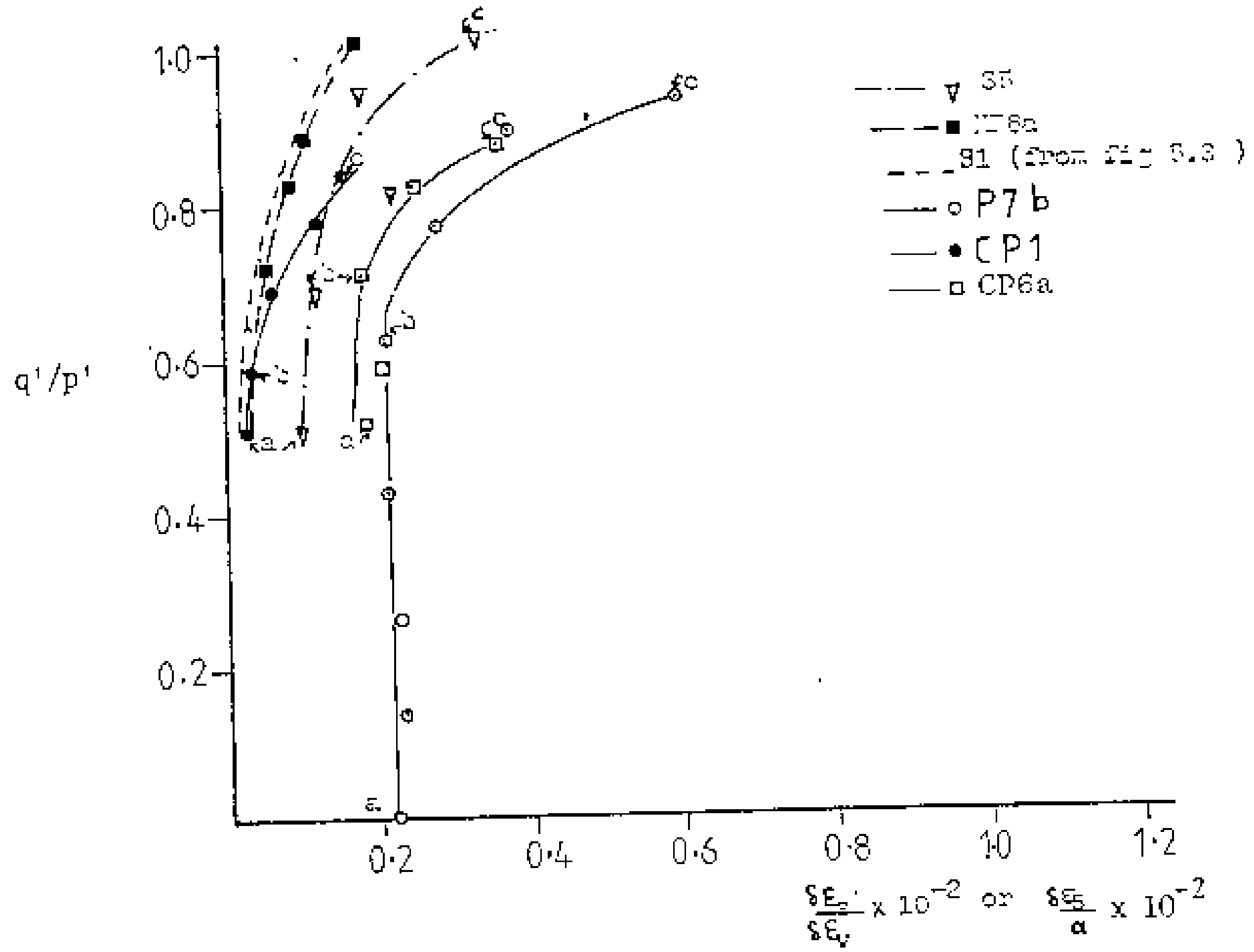


FIG. 6.2 RELATIONSHIPS OF STRESS RATIOS AND STRAIN RATIOS IN DIFFERENT TESTS

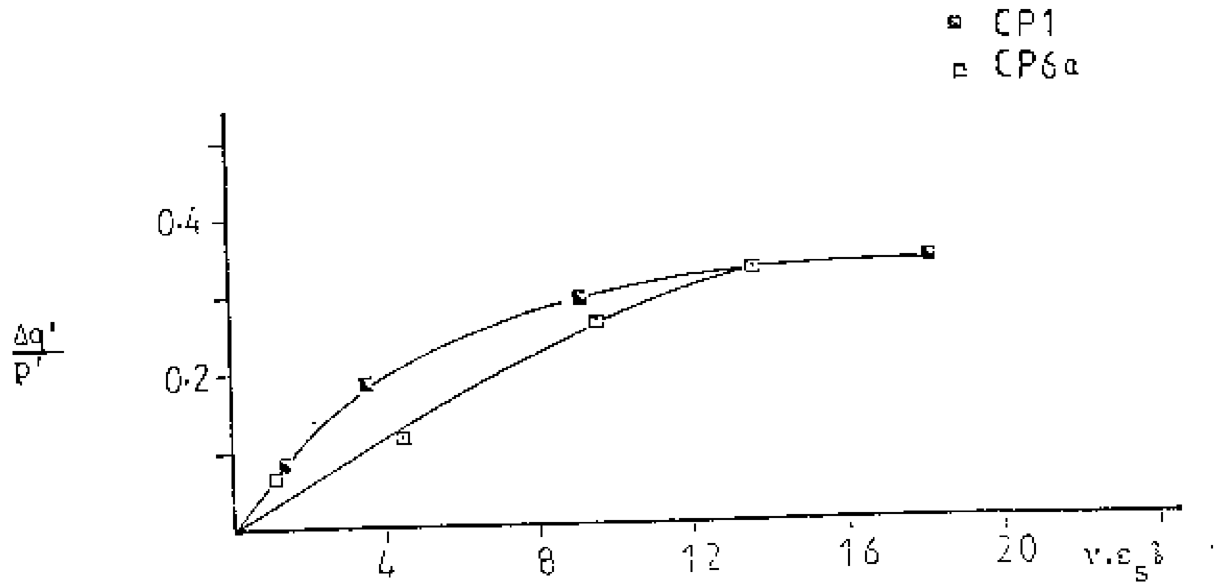


FIG. 6.3 DEVIATORIC STRESS-STRAIN RELATIONSHIP OF RECONSOLIDATED PERFECT SAMPLE

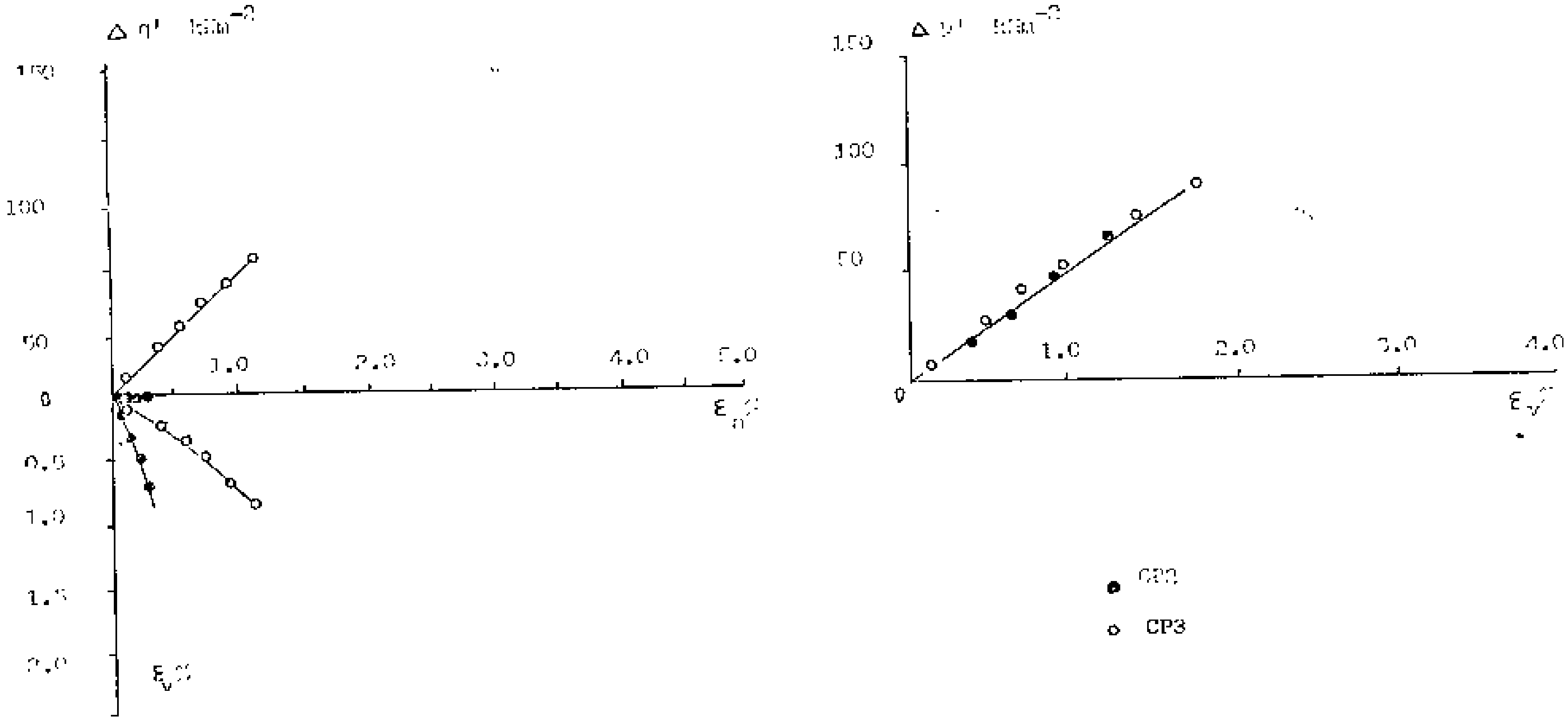


FIG. 6.[illegible] BEHAVIOUR OF PRECONSOLIDATED PERFECT SAMPLES IN TESTS CP2 AND CP3

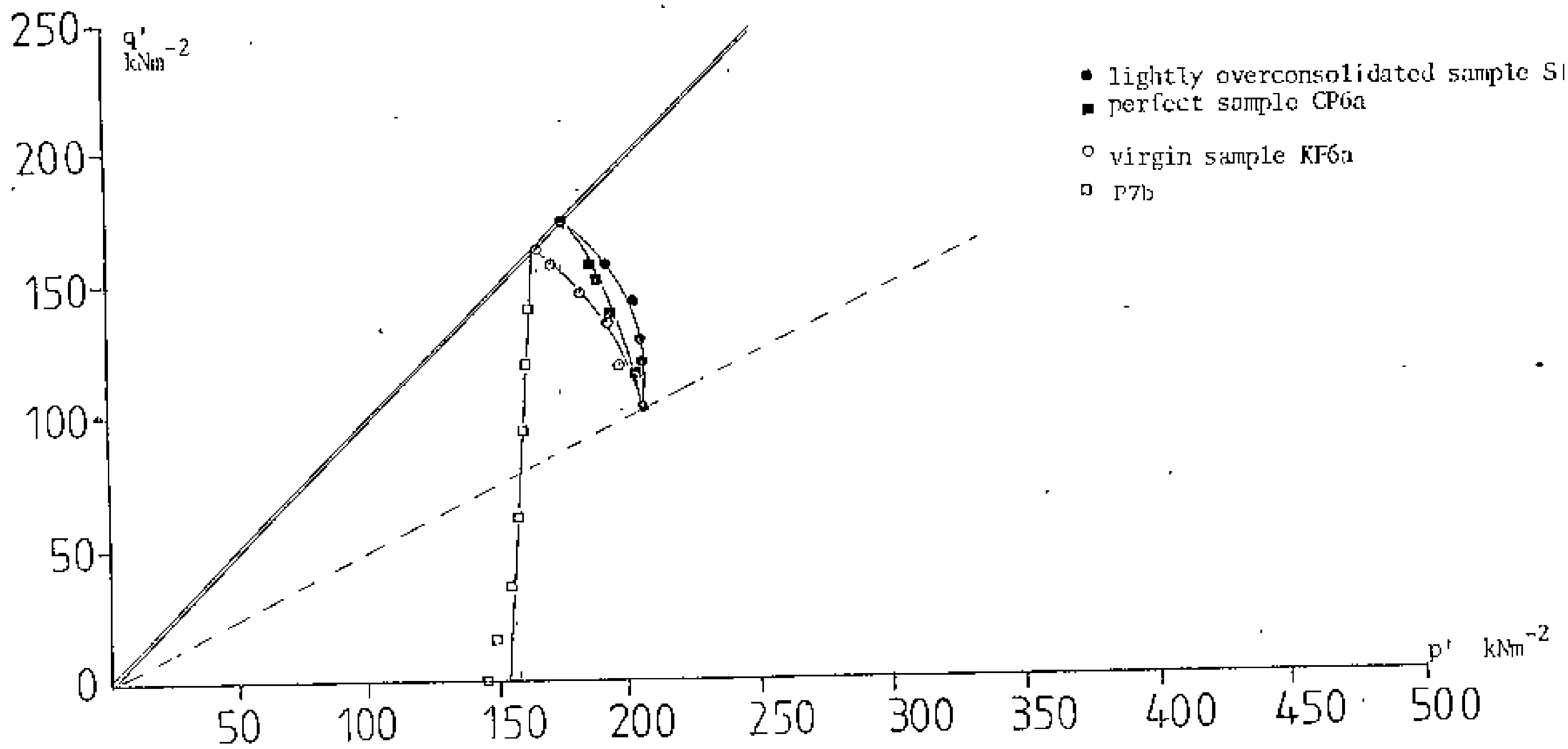


FIG. 6.5 STRESS PATHS OF VIRGIN AND PERFECT SAMPLES

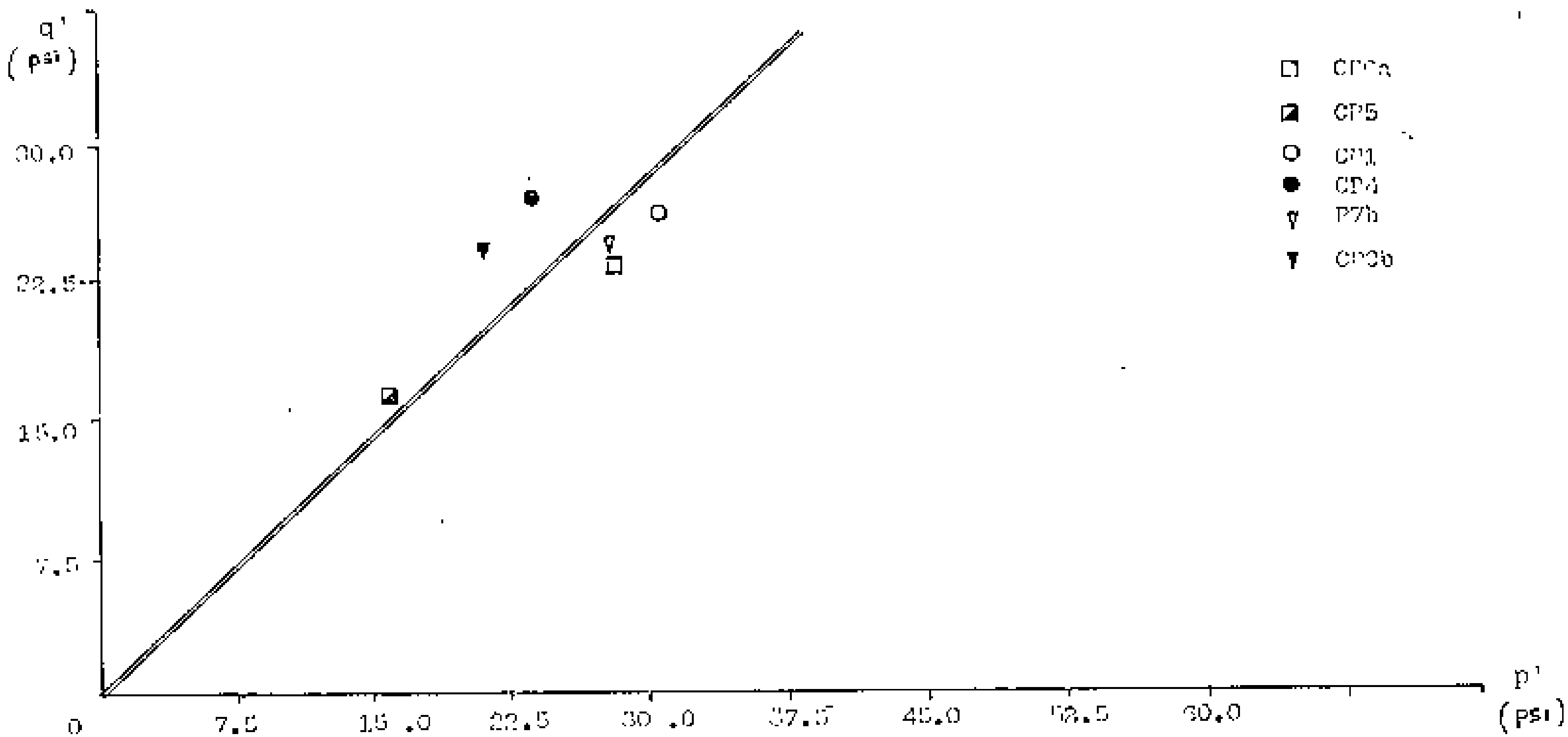


FIG. 6.6a THE CRITICAL STATE LINE OF PERFECT SAMPLES

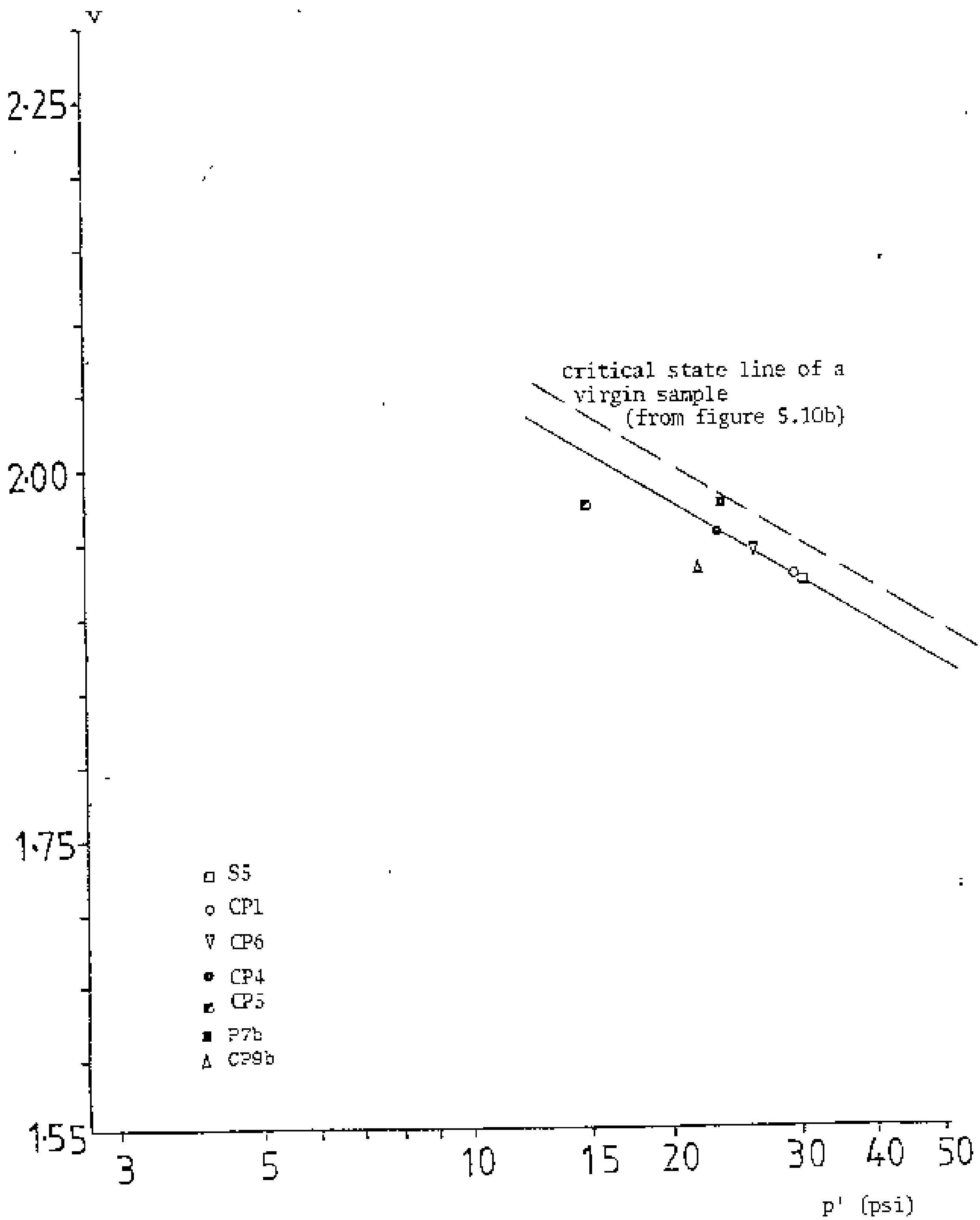


FIG. 6.6b THE CRITICAL STATE LINE OF PERFECT SAMPLES

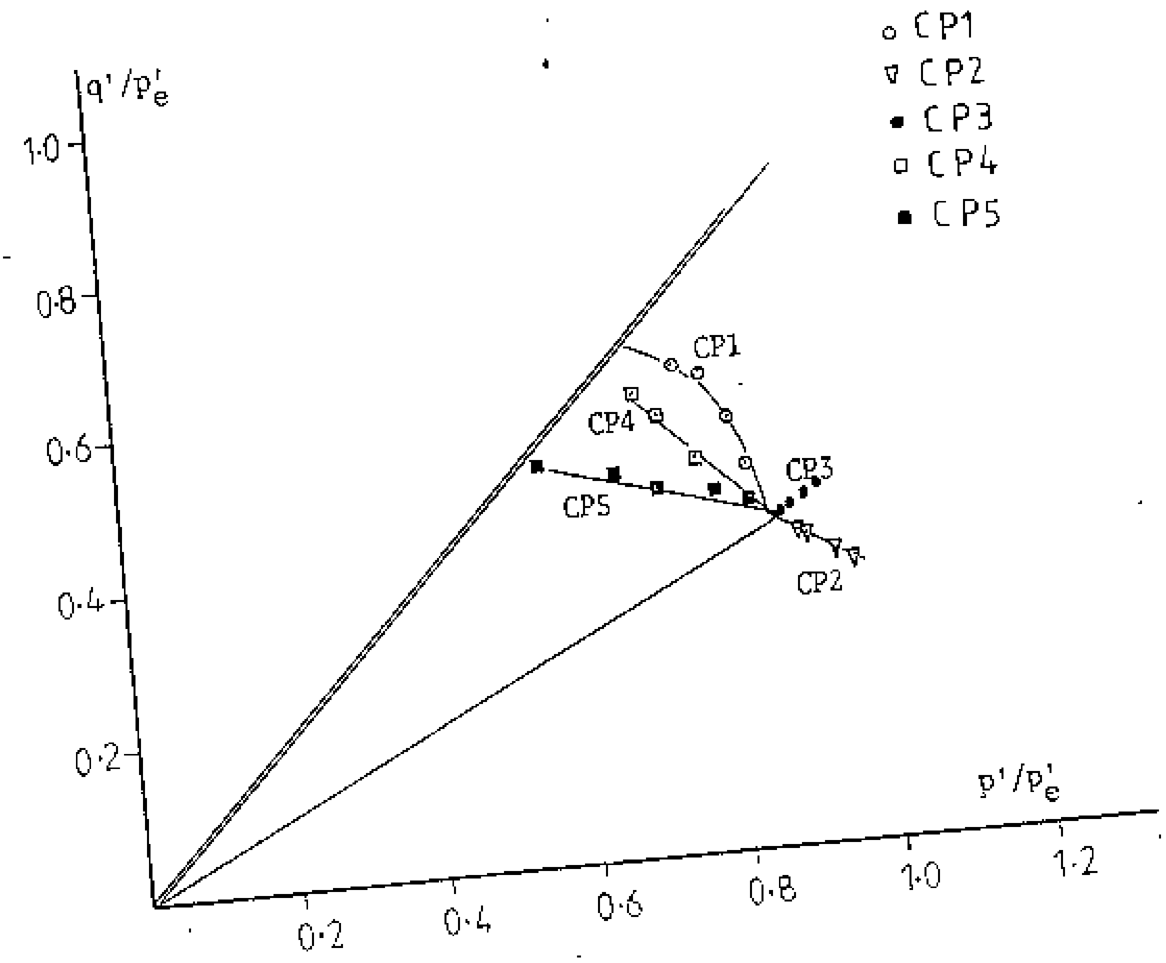


FIG. 6.7 NORMALISED STRESS PATHS OF PERFECT SAMPLES IN DRAINED TESTS

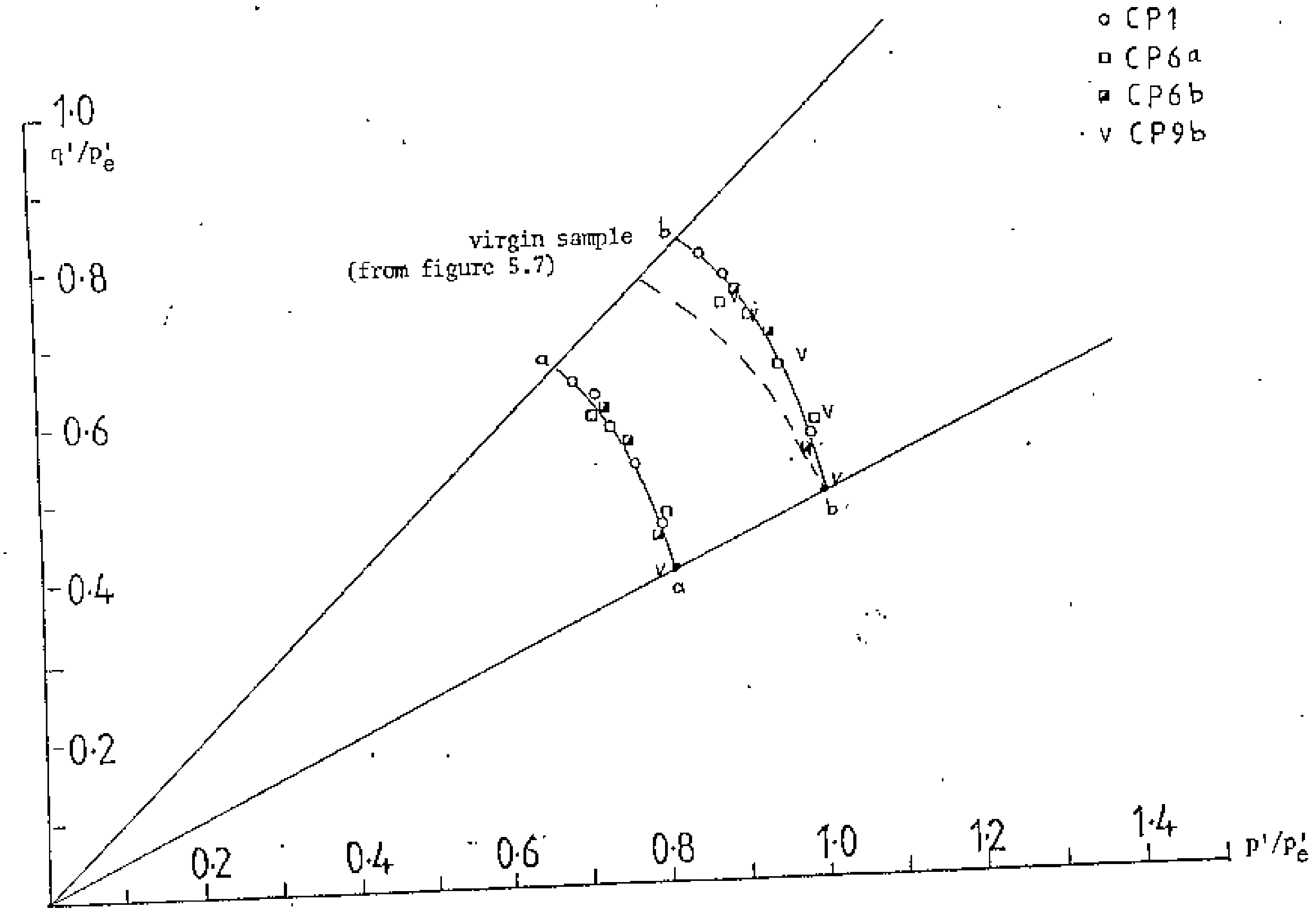


FIG. 6.8a THE STRESS PATH OF PERFECT SAMPLES NORMALISED FOLLOWING DIFFERENT ASSUMPTIONS

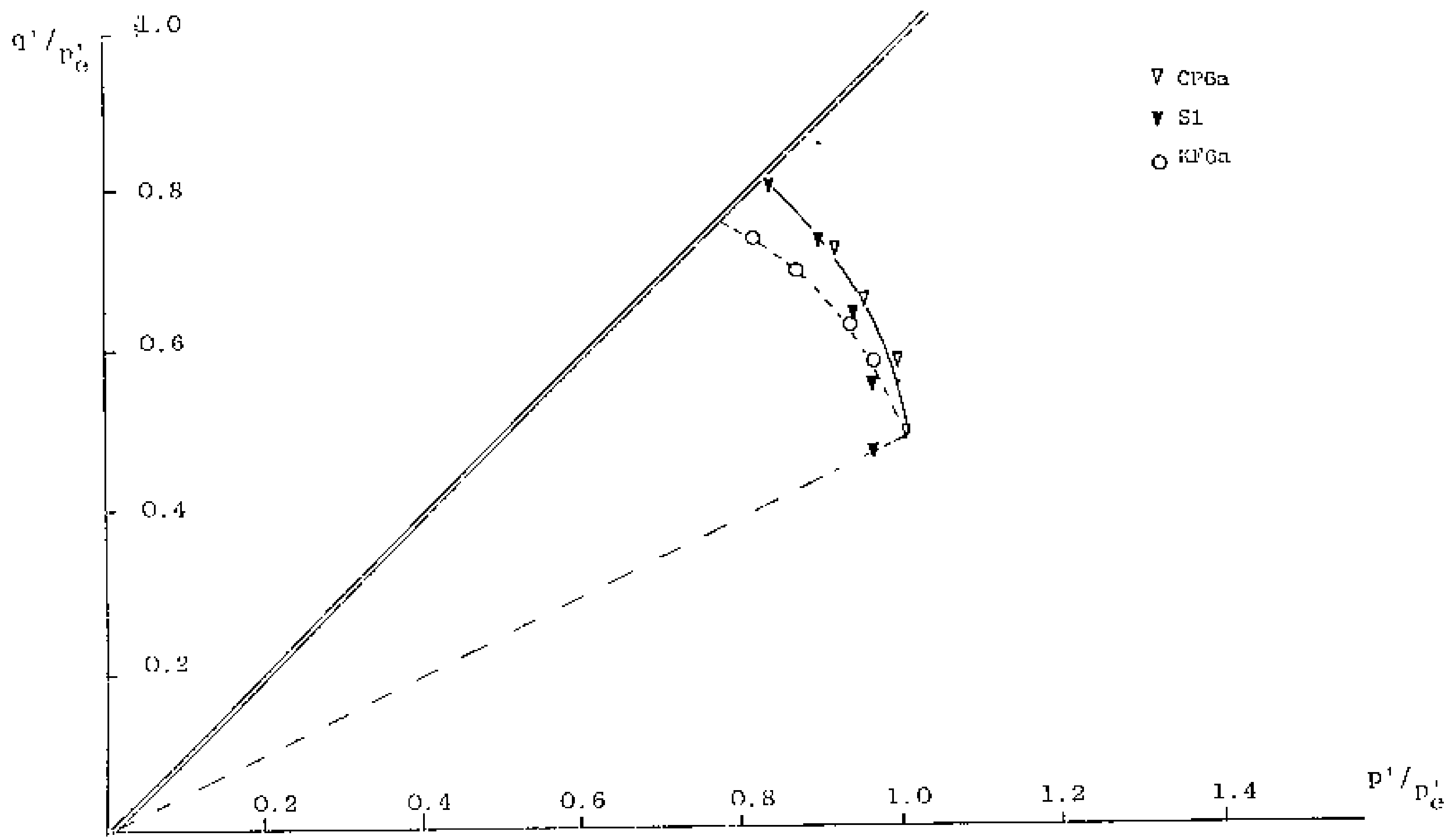


FIG. 6.8 b. NORMALISED STRESS PATHS OF PERFECT AND VIRGIN SAMPLES

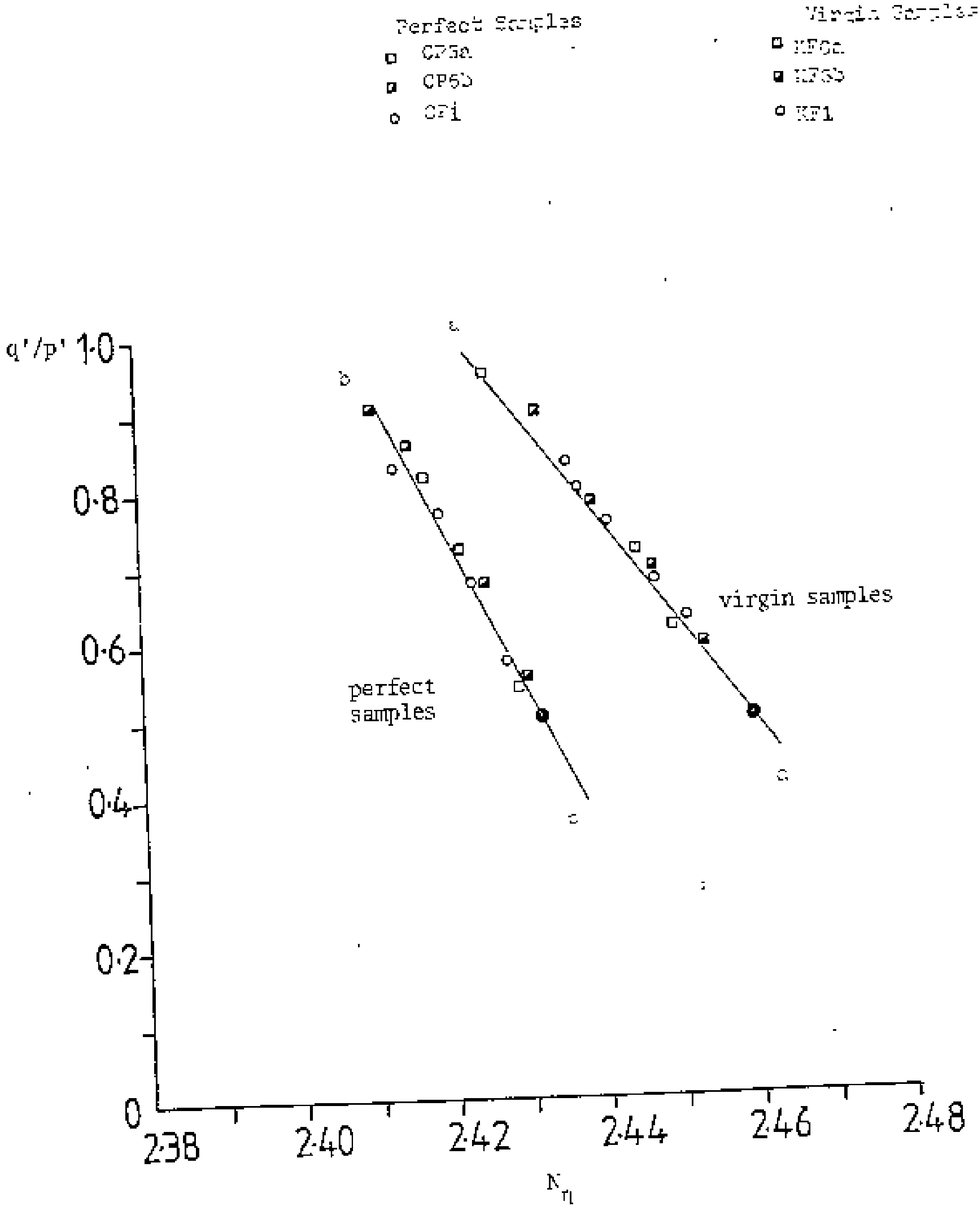


FIG. 6.9 REFERENCE CURVES FOR PERFECT AND VIRGIN SAMPLES

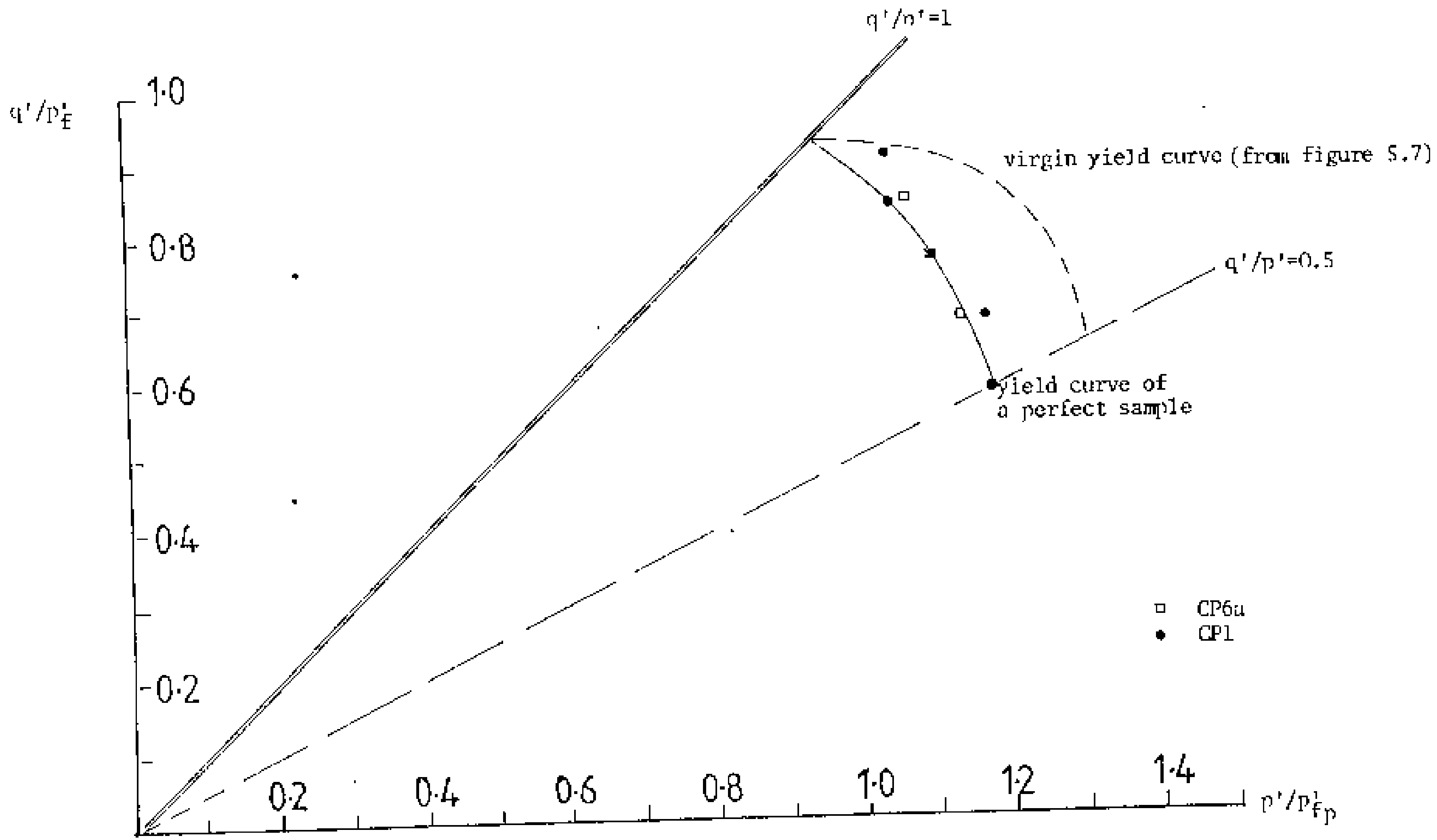


FIG. 6.10 YIELD CURVE OF A PERFECT SAMPLE

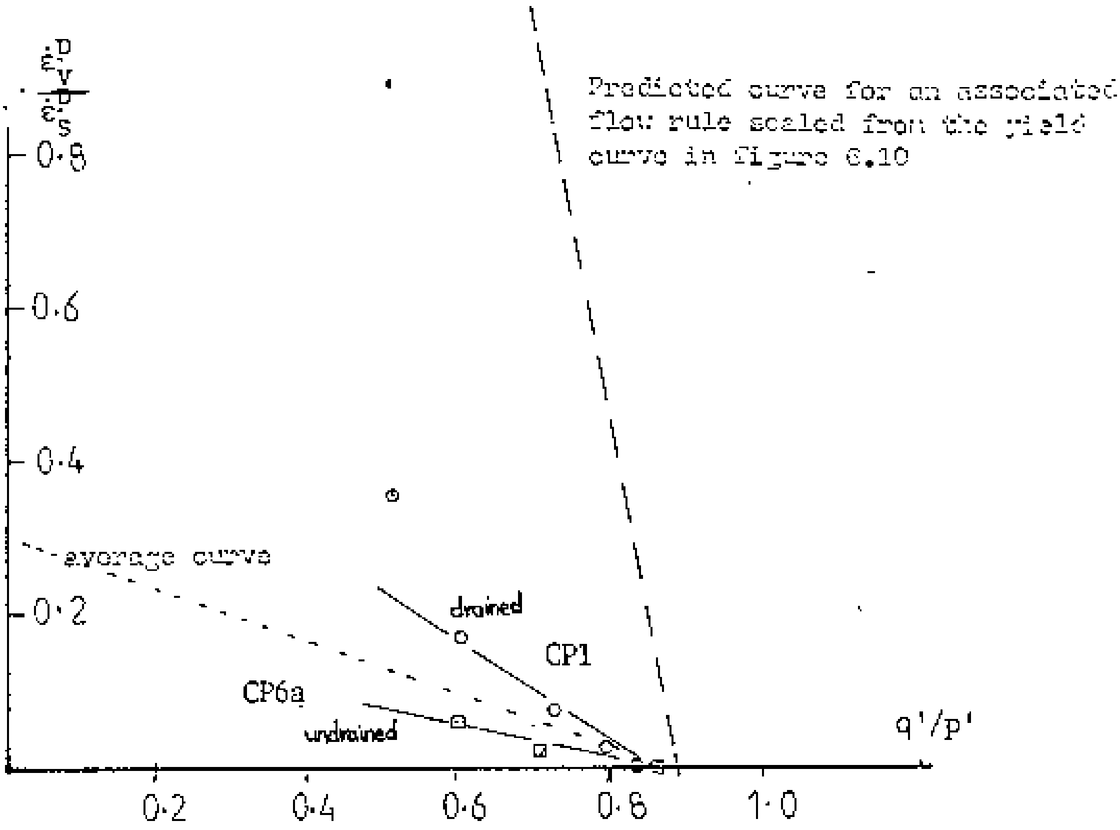


FIG. 6.11 RELATIONSHIPS BETWEEN PLASTIC STRAIN INCREMENTS AND STRESS RATIOS

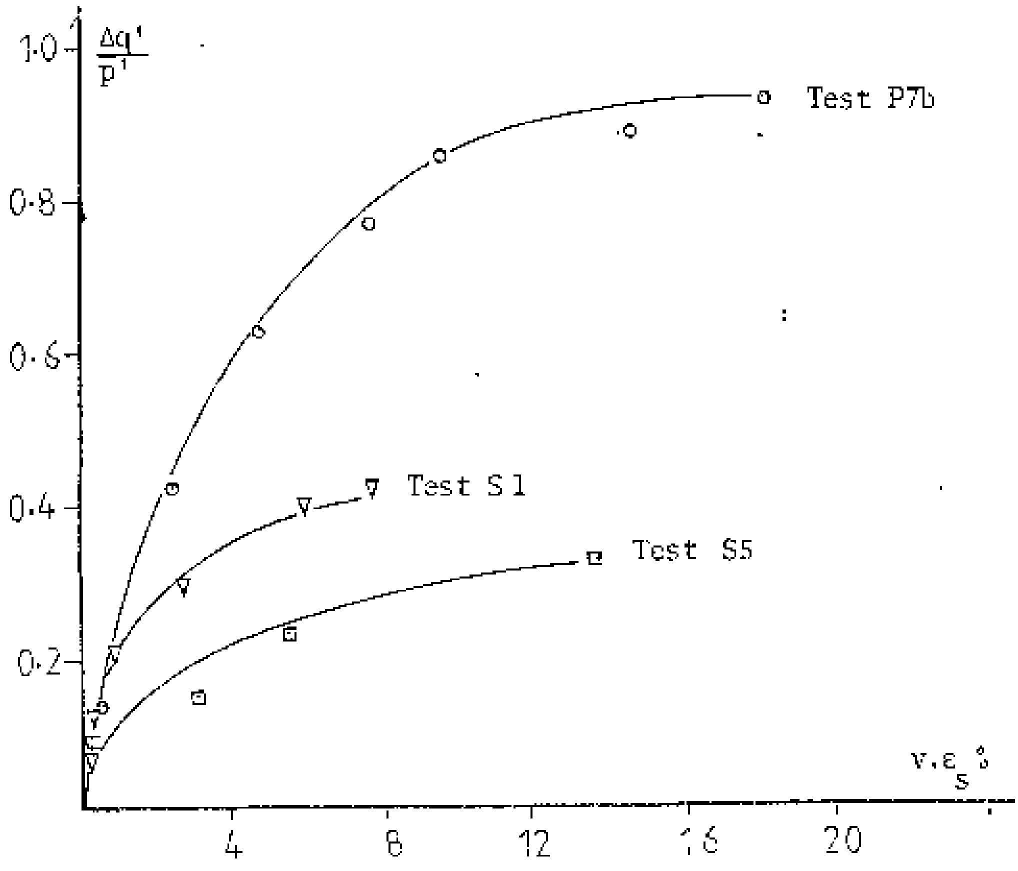
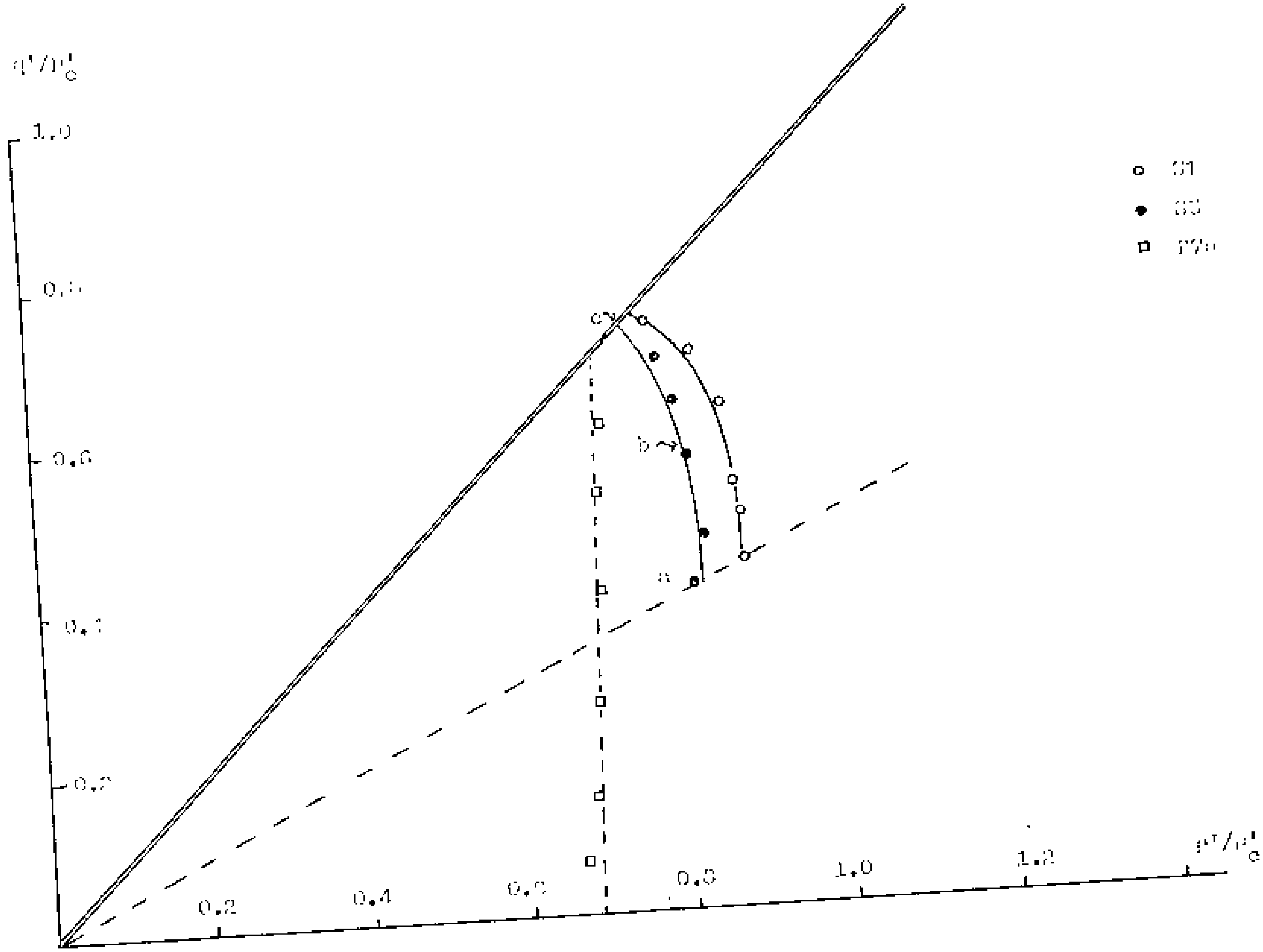


FIG. 6.12 NORMALISED STRESS STRAIN RELATIONSHIPS OF PERFECT SAMPLES



[illegible] STRESS PATHS OF [illegible] SAMPLES

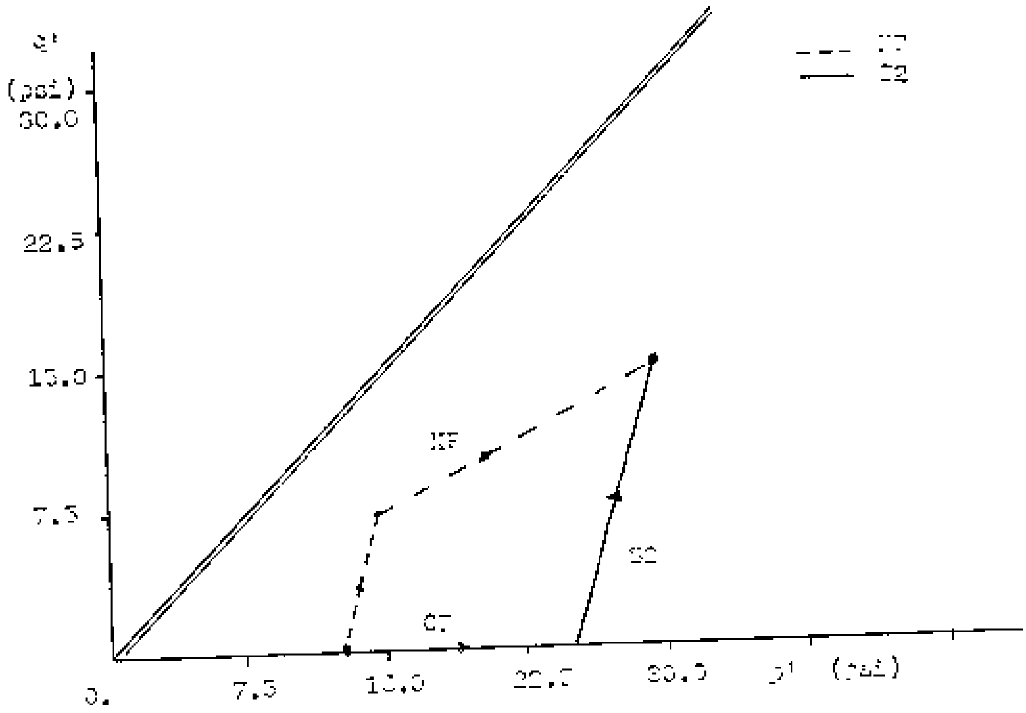


FIG. 6.14a STRESS PATHS IN KP, CP AND S2

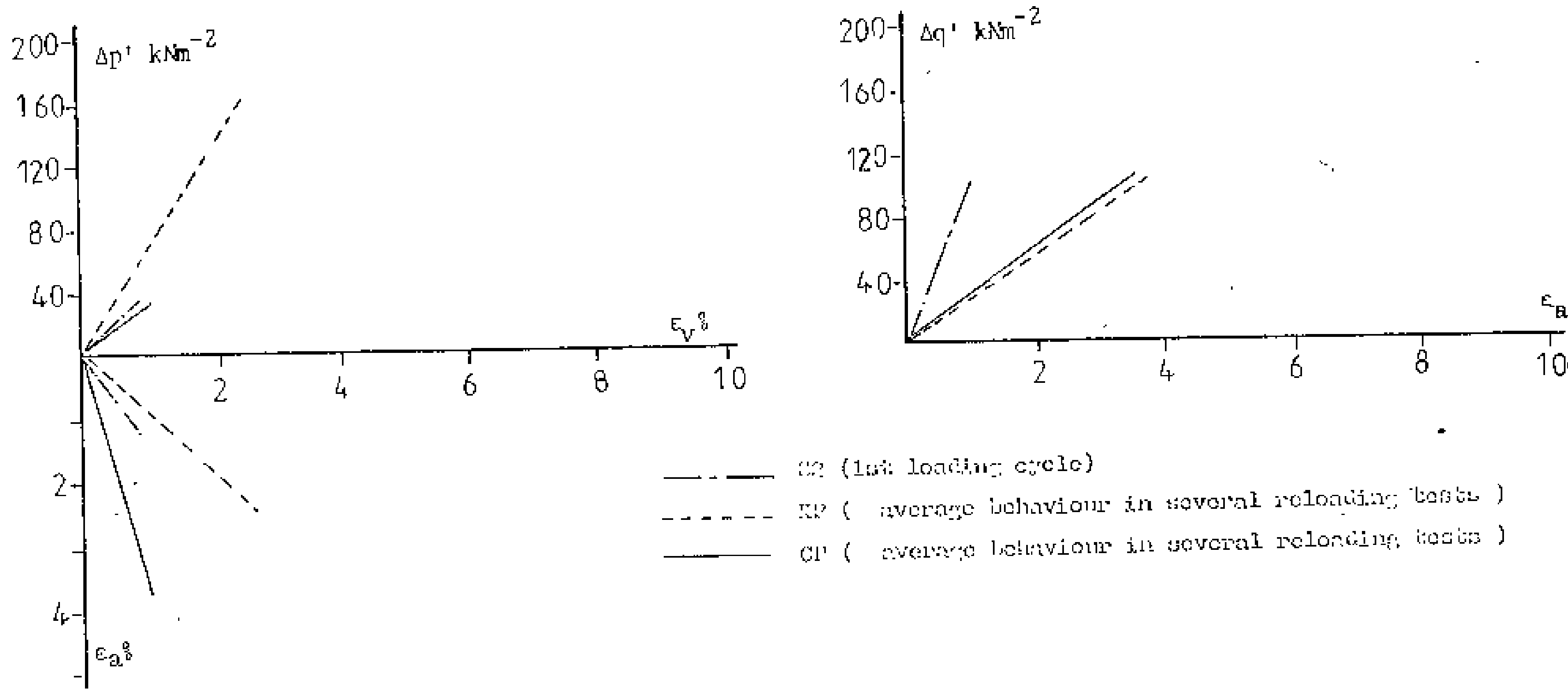
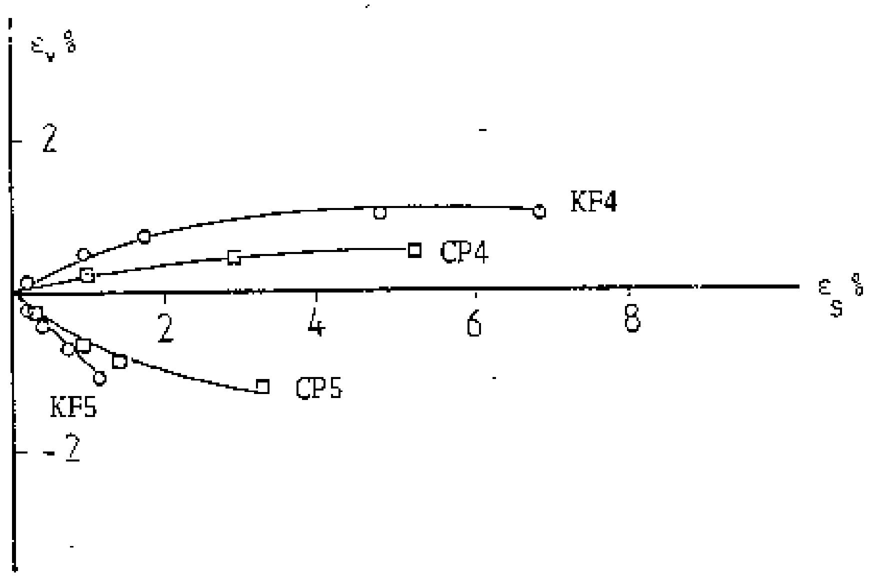
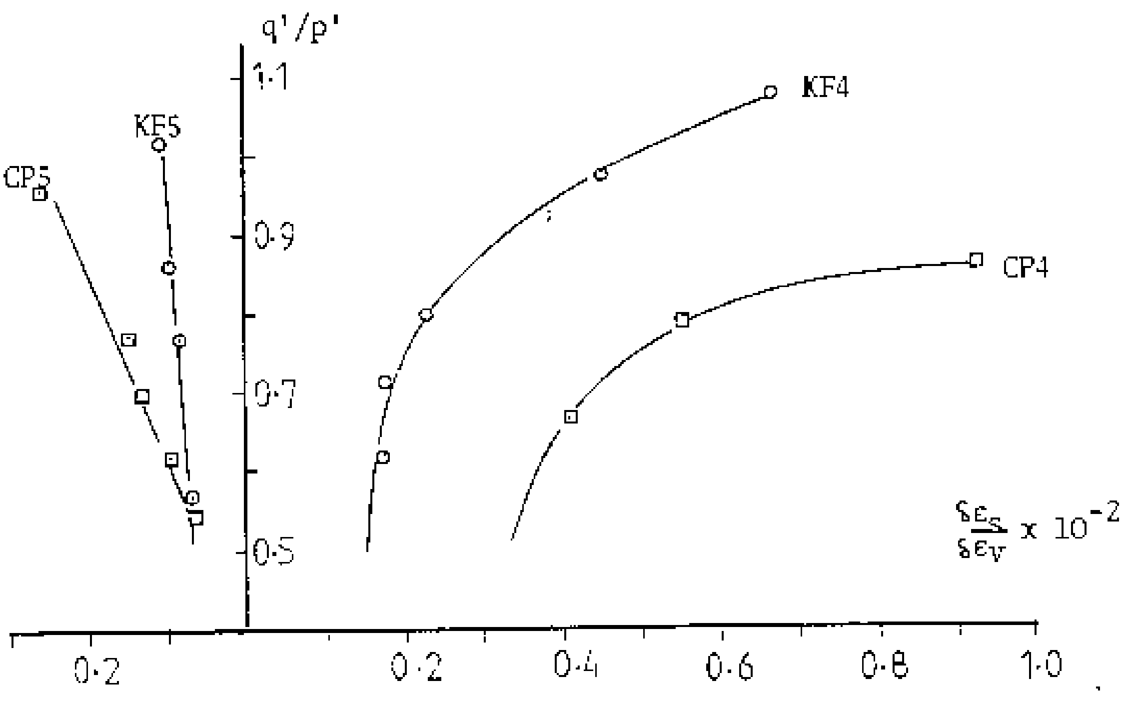


FIG. 6.14b THE RELOADING BEHAVIOUR OF VIRGIN AND PERFECT SAMPLES

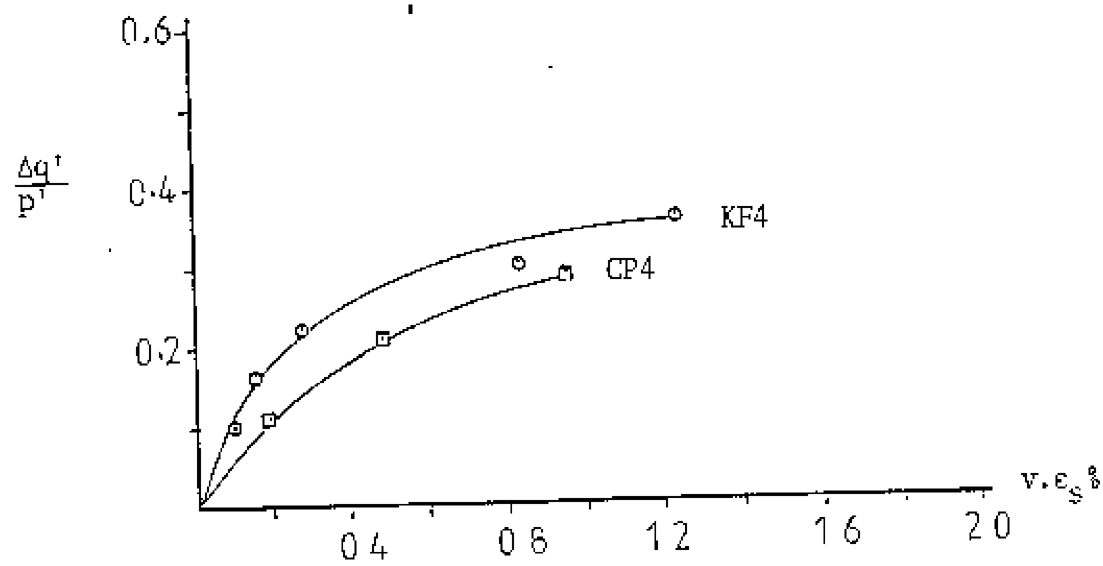


(a) Strain paths of perfect and virgin samples



(b) Relationships between stress ratios and strain ratios

FIG. 6.15 THE BEHAVIOUR OF PERFECT SAMPLES AND VIRGIN SAMPLES IN LOADING PATHS 4 AND 5



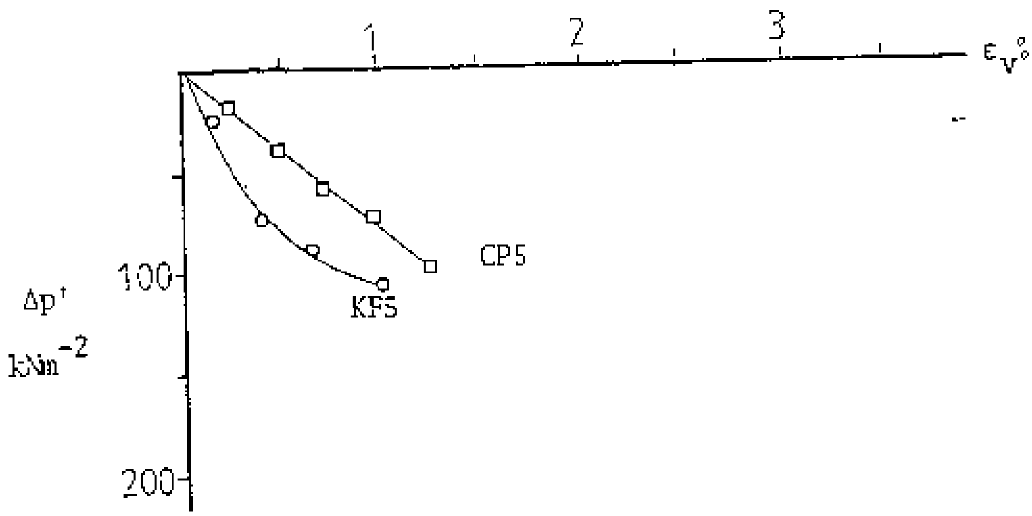


FIG. 6.16 STRESS-STRAIN RELATIONSHIPS OF PERFECT AND VIRGIN SAMPLES IN LOADING PATHS 4 AND 5

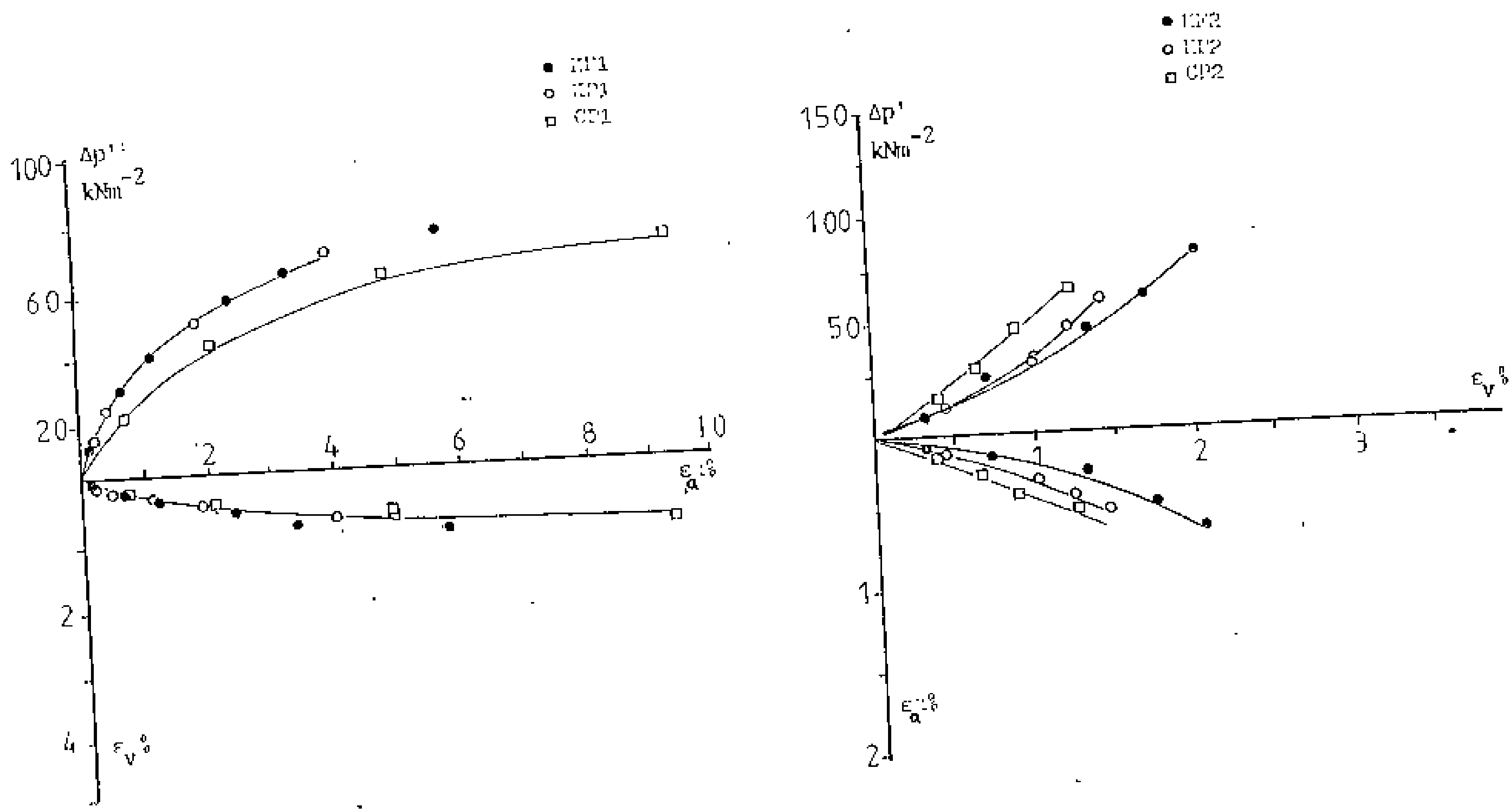


FIG. 6.17 THE EFFECTS OF STRESS HISTORY ON THE BEHAVIOUR OF PERFECT SAMPLES

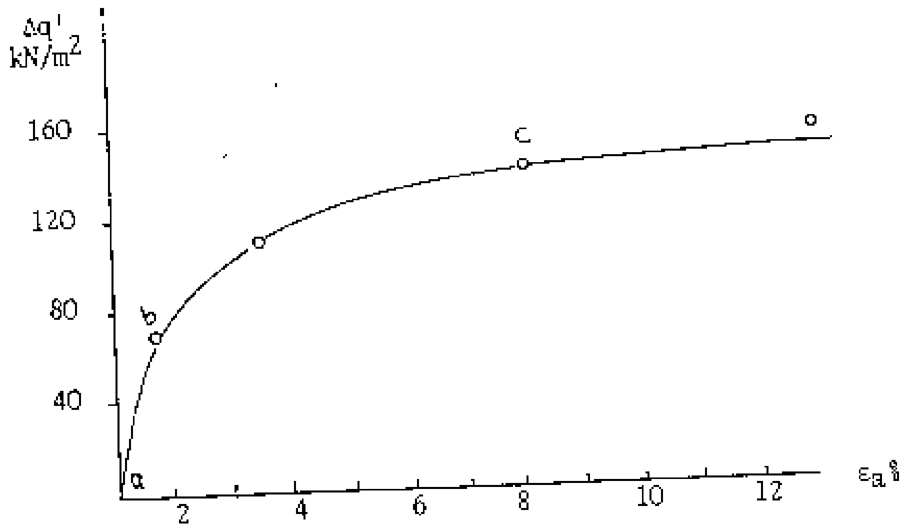


FIG. 6.18 BEHAVIOUR OF A PERFECT SAMPLE IN TEST 10na

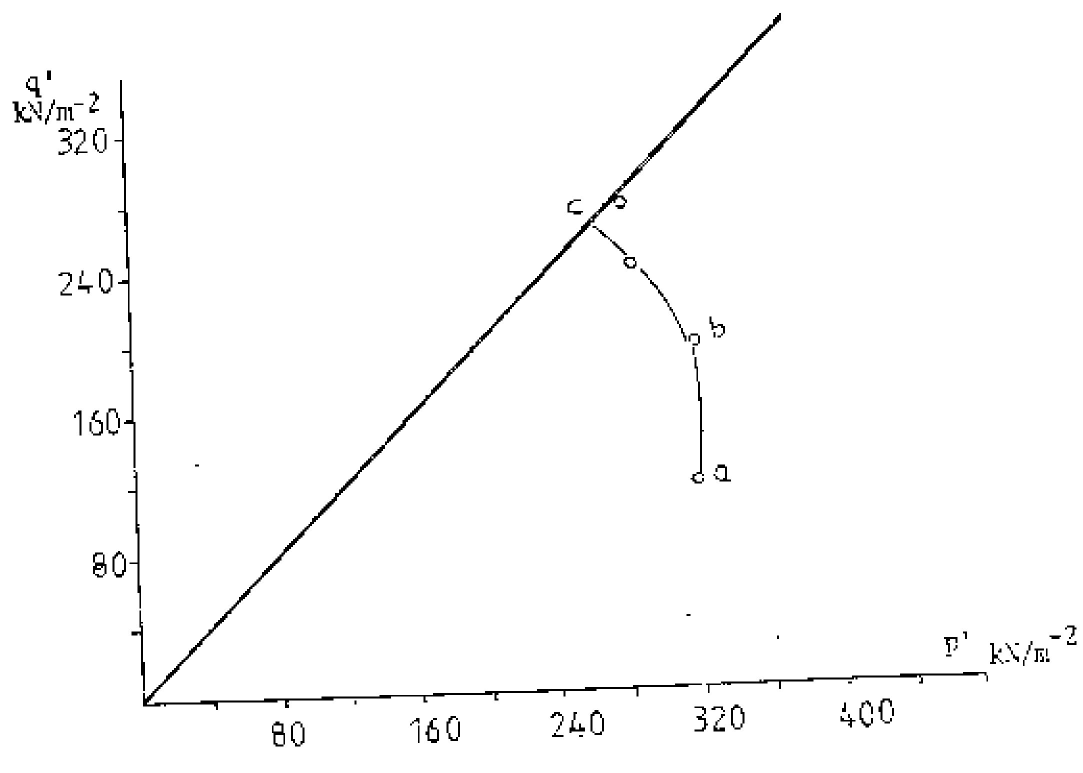


FIG. 6.19 STRESS PATH OF A PERFECT SAMPLE IN TEST 10na

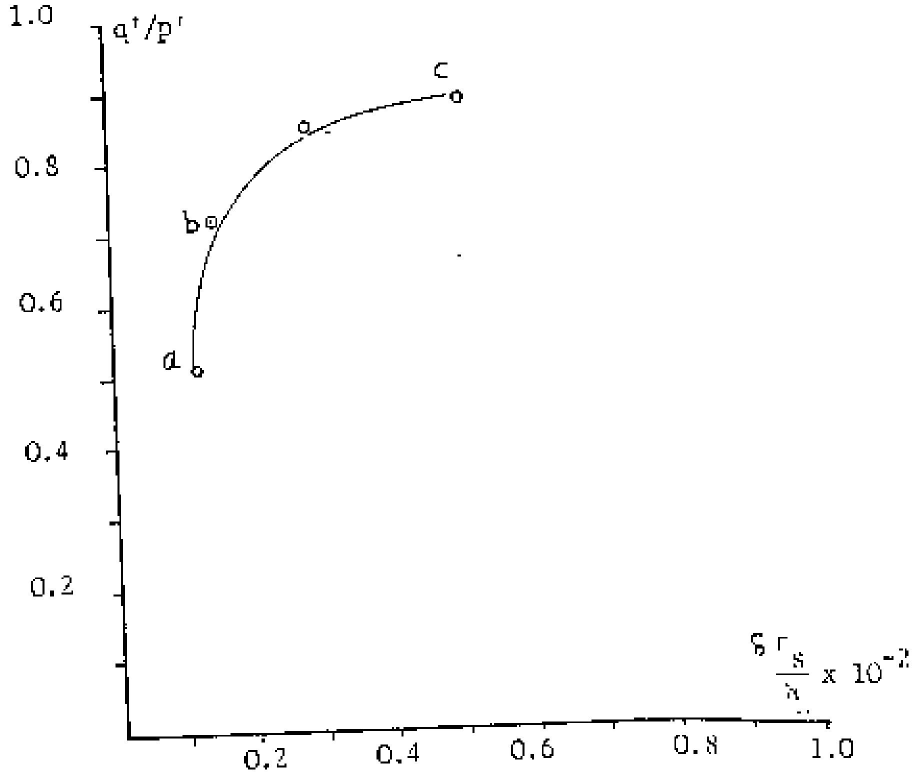
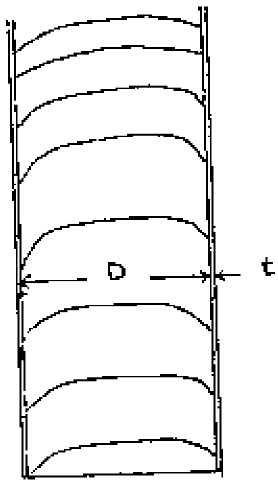
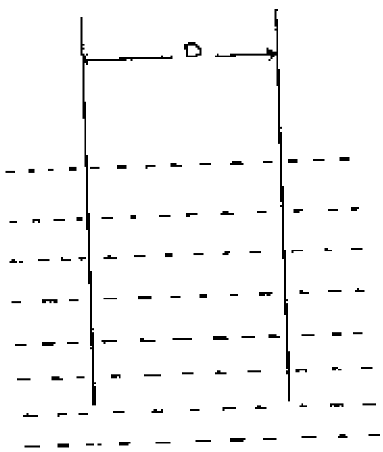


FIG. 6.20 RELATIONSHIP BETWEEN THE STRESS RATIO AND STRAIN RATIO IN TEST 10na

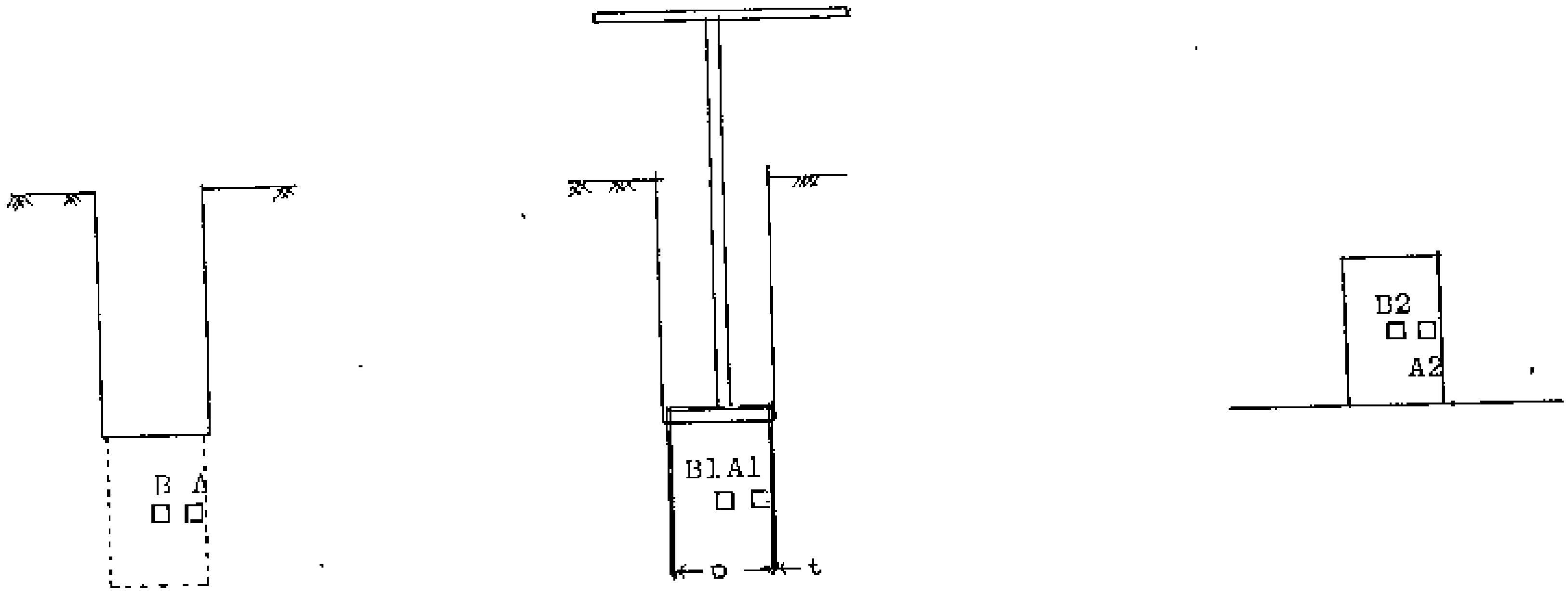


(a) EXAMPLE FROM LITERATURE SHOWING BENT LAYERS IN A TUBE SAMPLE (HVORSLEV 1949)



(b) HYPOTHETICAL CASE OF SAMPLING WITH FRICTIONLESS TUBE SAMPLER AND ZERO WALL THICKNESS

FIG. 7.1. DISTORTIONS CAUSED BY THE WALL OF A SAMPLER



before sampling

during sampling

sample after extrusion

FIG. 7.2. ELEMENTS OF SOIL AT DIFFERENT STATES BEFORE AND AFTER SAMPLING

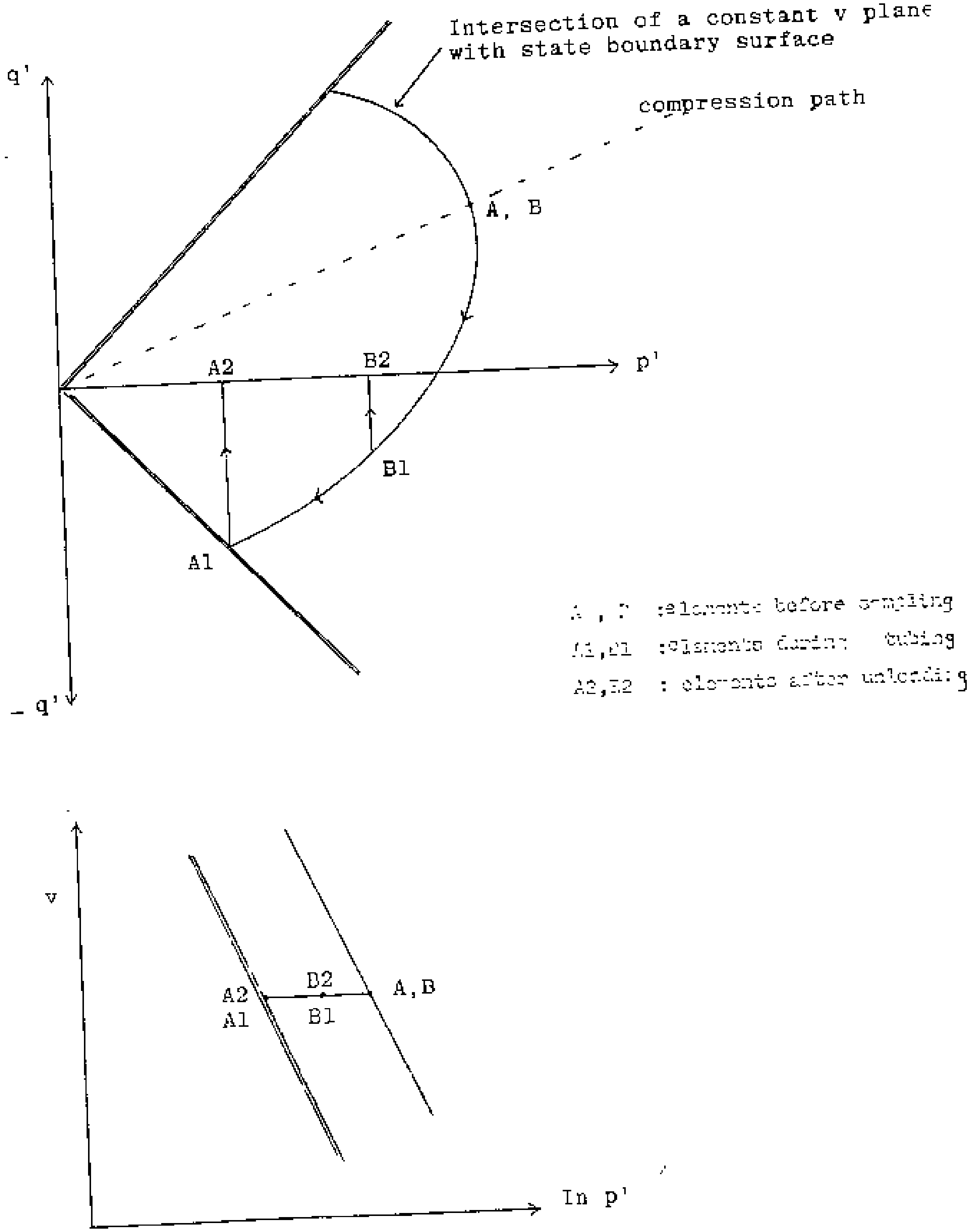


FIG. 7.3. STATES OF DIFFERENT ELEMENTS OF SOIL BEFORE AND AFTER SAMPLING

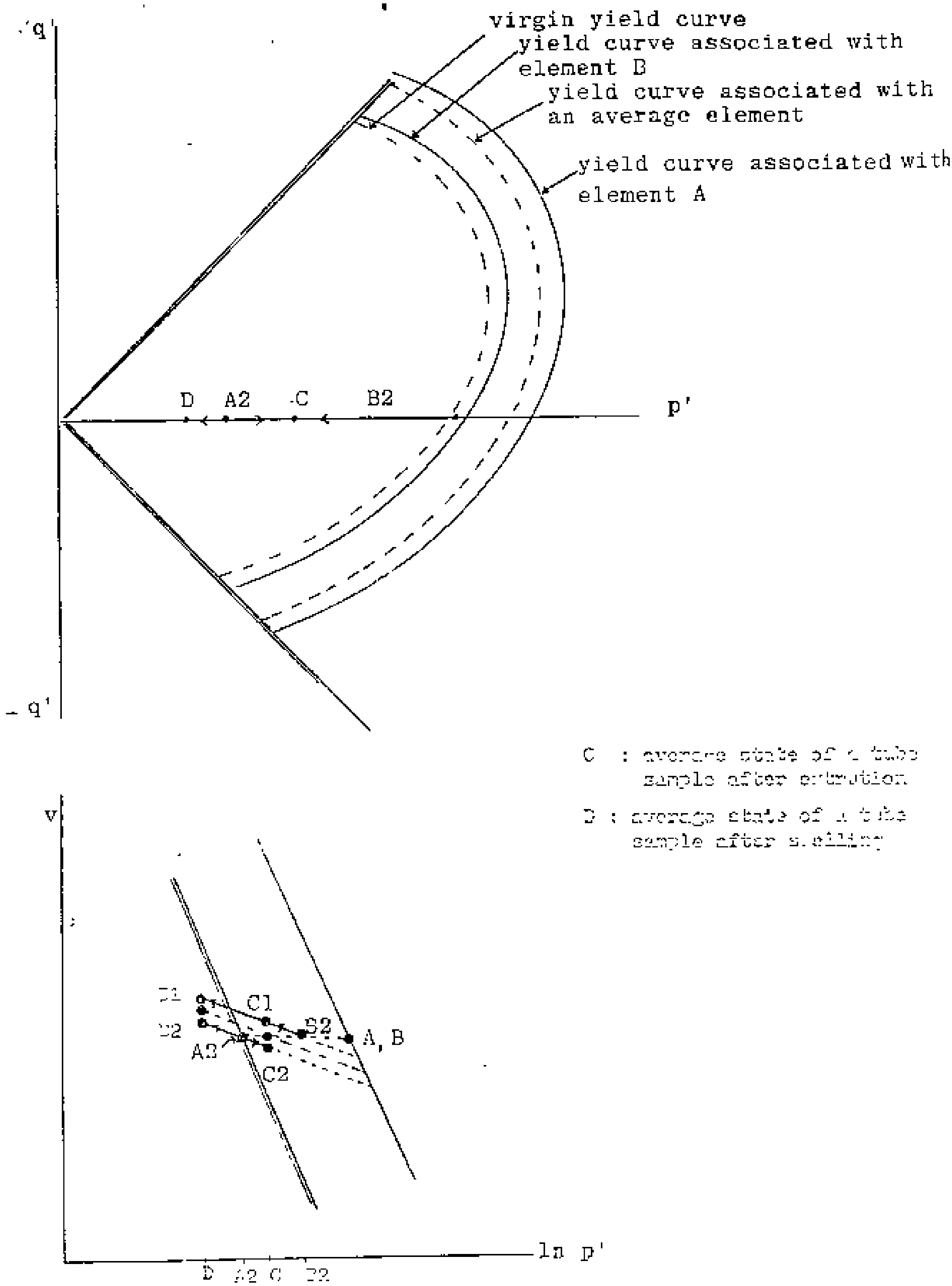
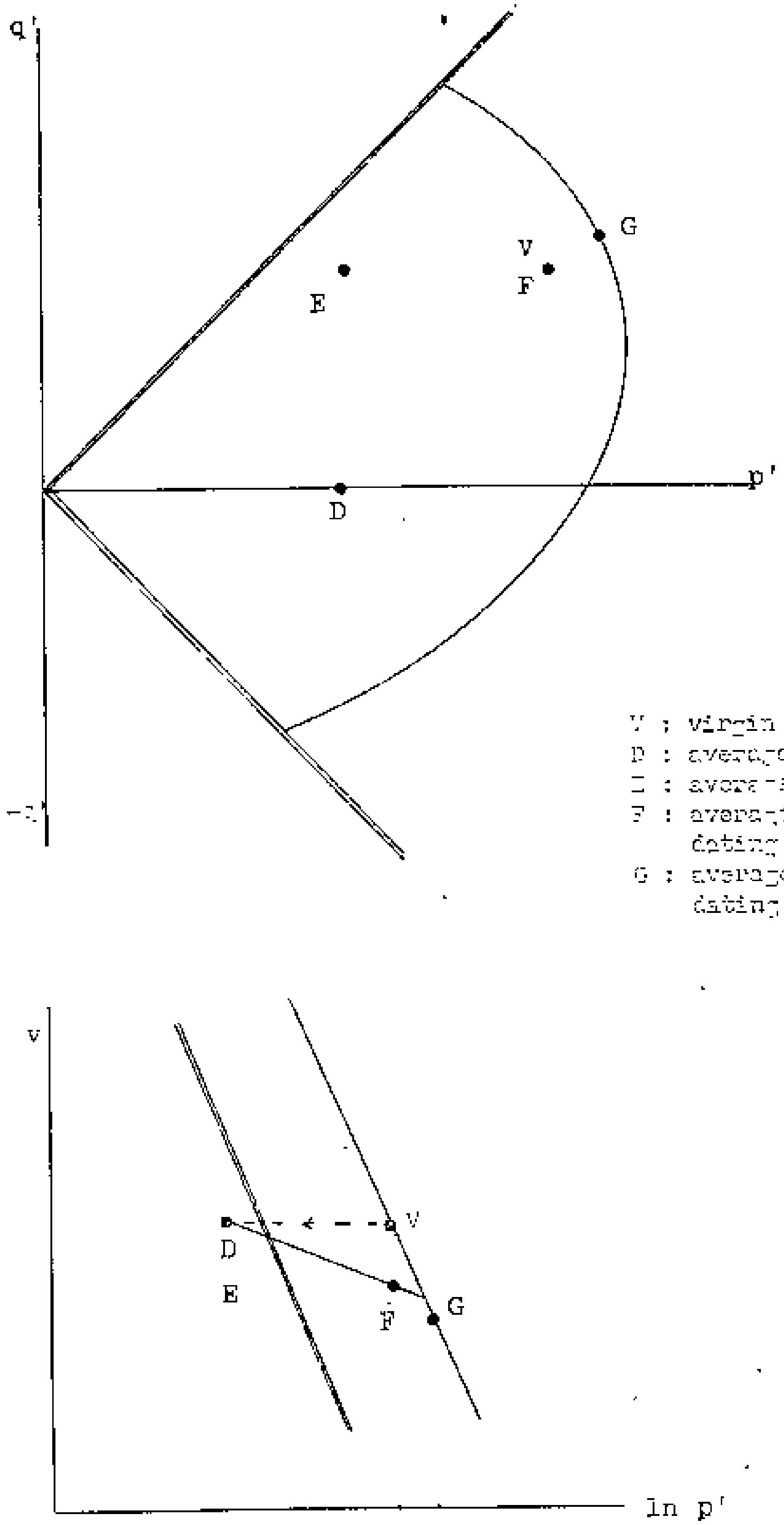


FIG. 7.4. STATES OF DIFFERENT ELEMENTS OF SOIL AFTER SAMPLING AND SWELLING



V : virgin state before sampling
D : average state after sampling
E : average state after reloading
F : average state after reconsolidating
G : average state after reconsolidating to a higher level

FIG. 7.5 OVERALL STATE OF A DISTURBED SAMPLE AT DIFFERENT STRESS LEVELS

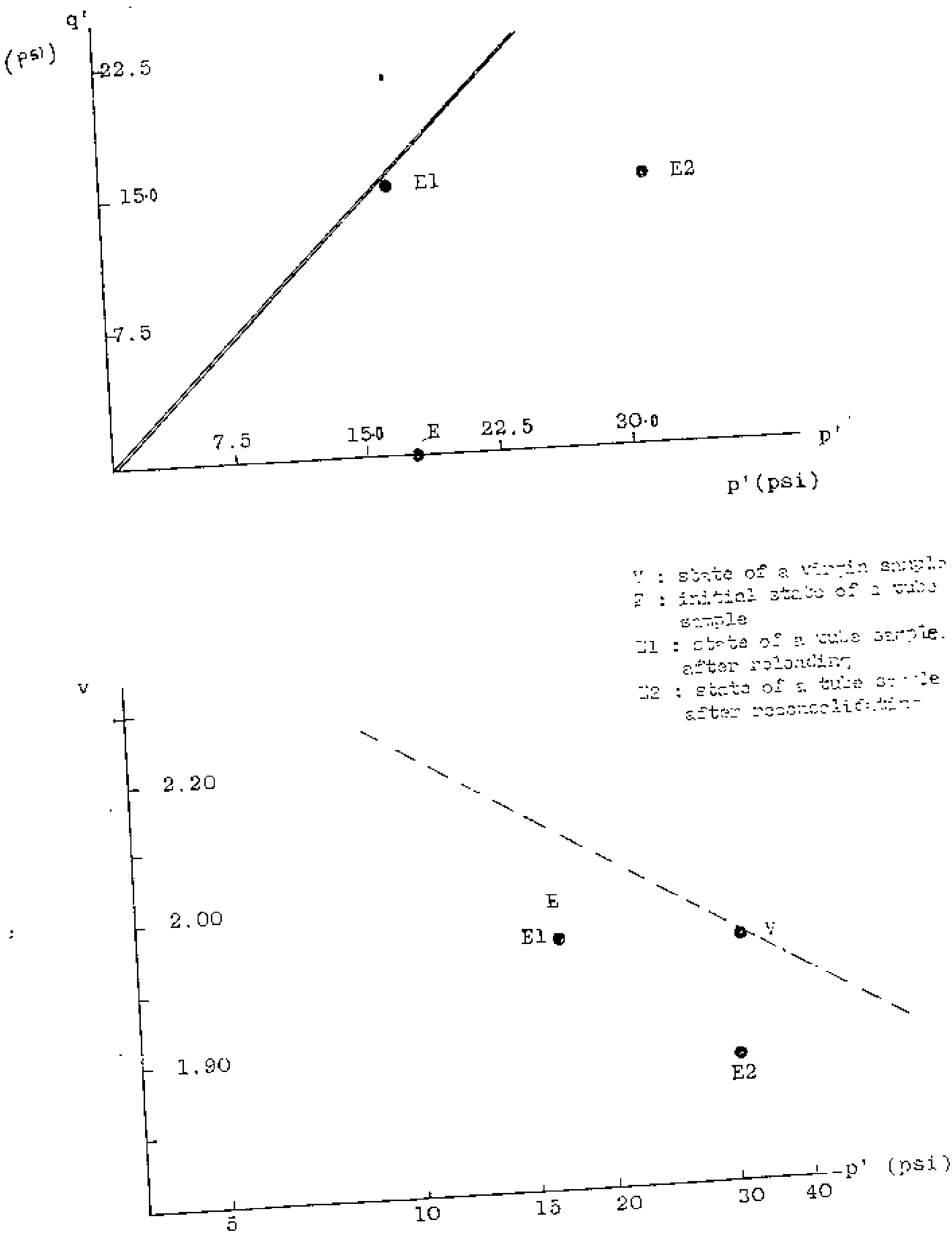


FIG. 7.6 STATES OF A TUBE SAMPLE AT DIFFERENT STRESS LEVELS

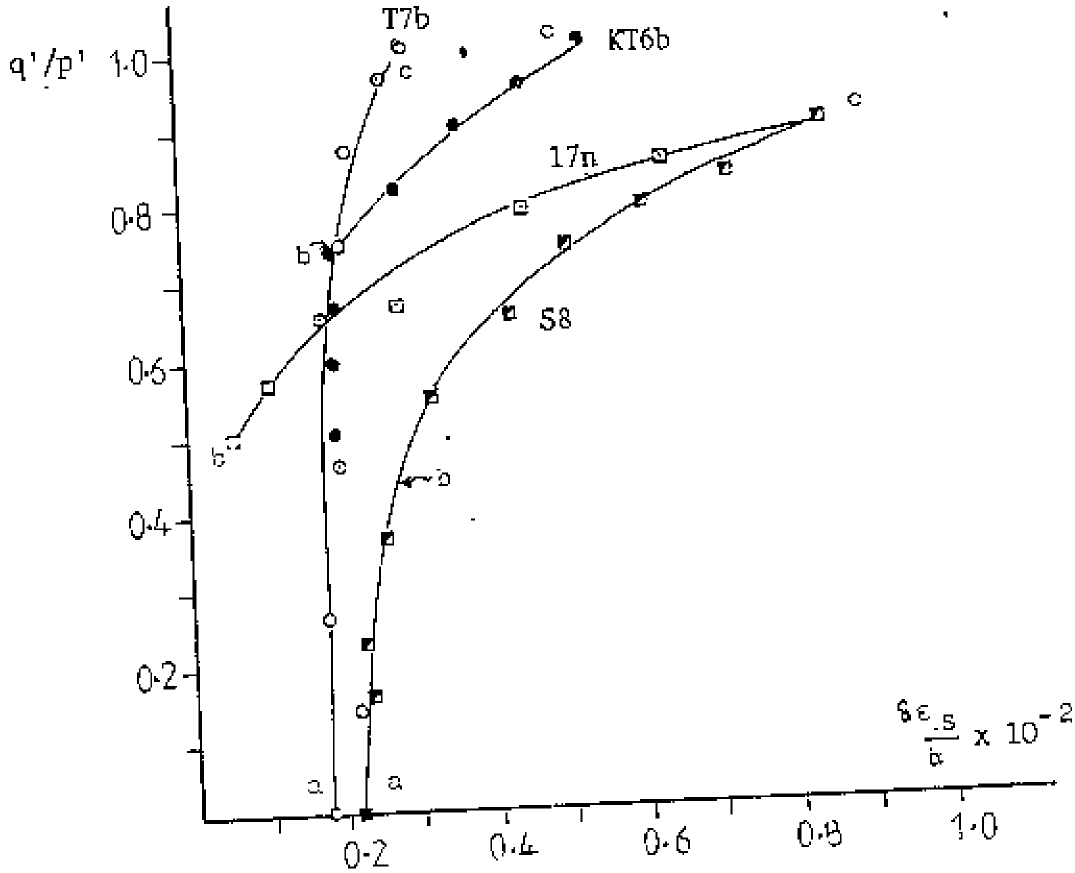


FIG. 7.7 RELATIONSHIPS BETWEEN STRESS RATIOS AND STRAIN RATIOS OF TUBE SAMPLES IN DIFFERENT TESTS

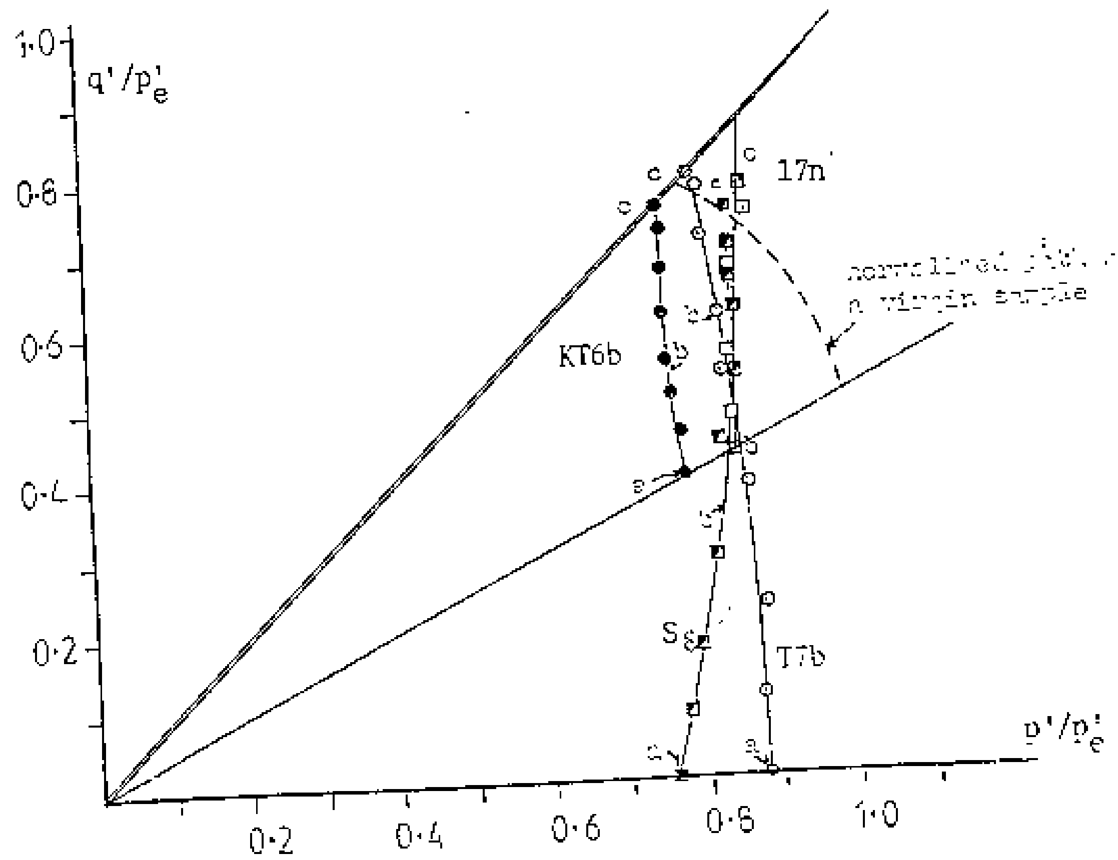


FIG. 7.8 NORMALISED STRESS PATHS OF TUBE SAMPLES IN DIFFERENT TESTS

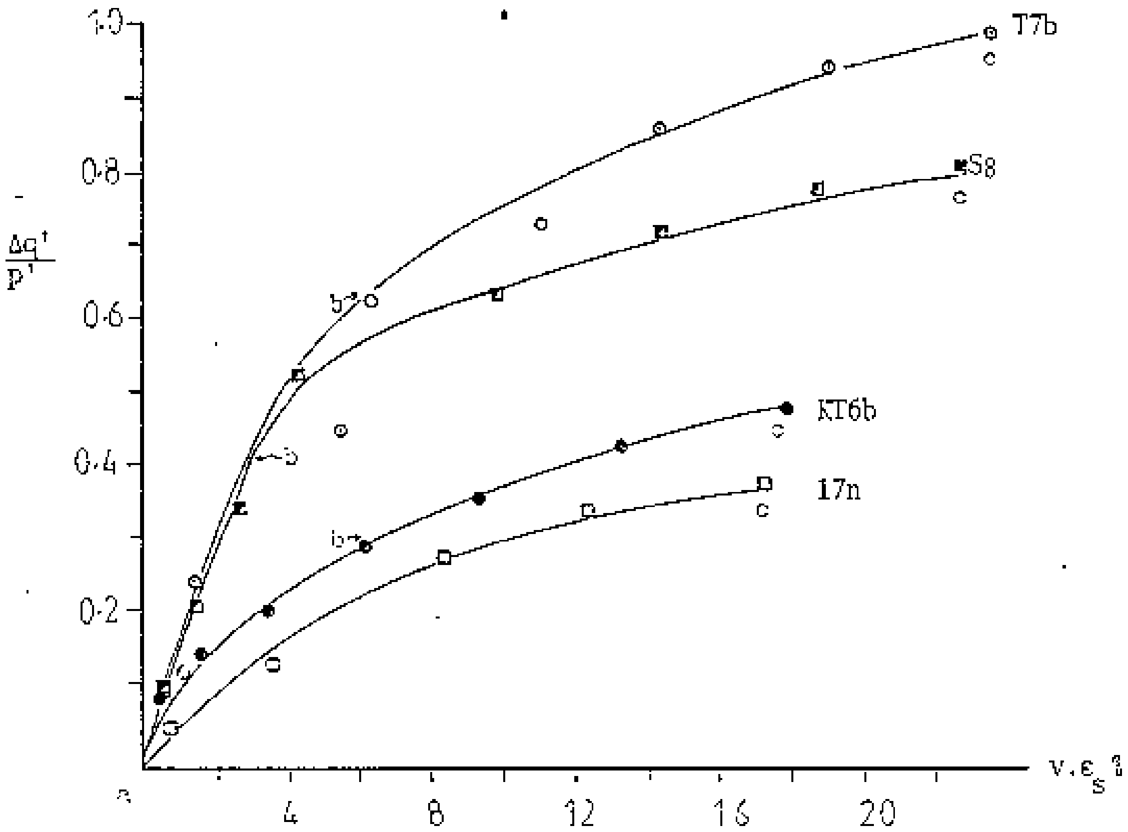


FIG. 7.9 NORMALISED STRESS STRAIN RELATIONSHIPS OF TUBE SAMPLES

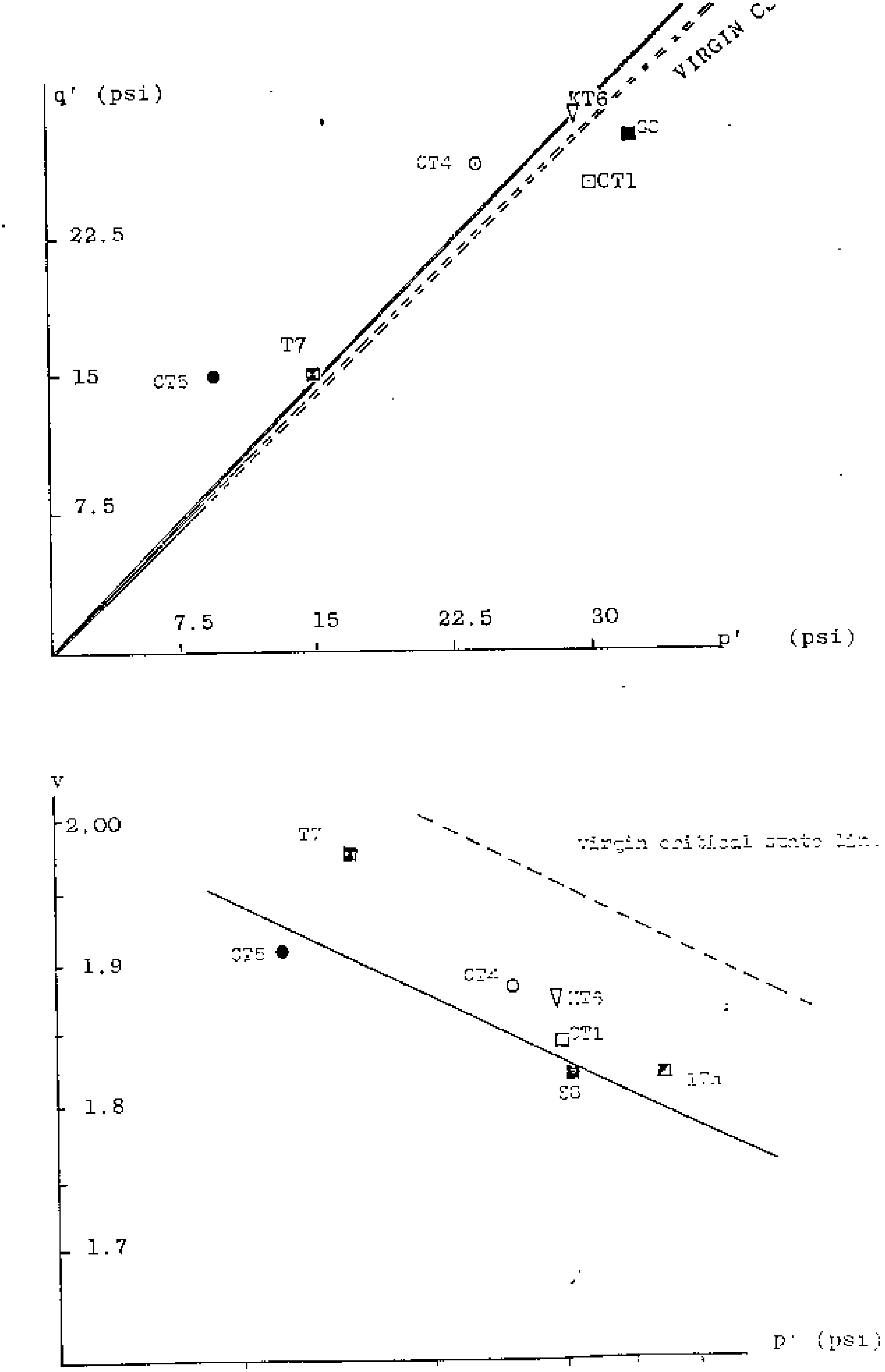


Fig. 7.10. THE STATE OF TUBE SAMPLES AT

FAILURE

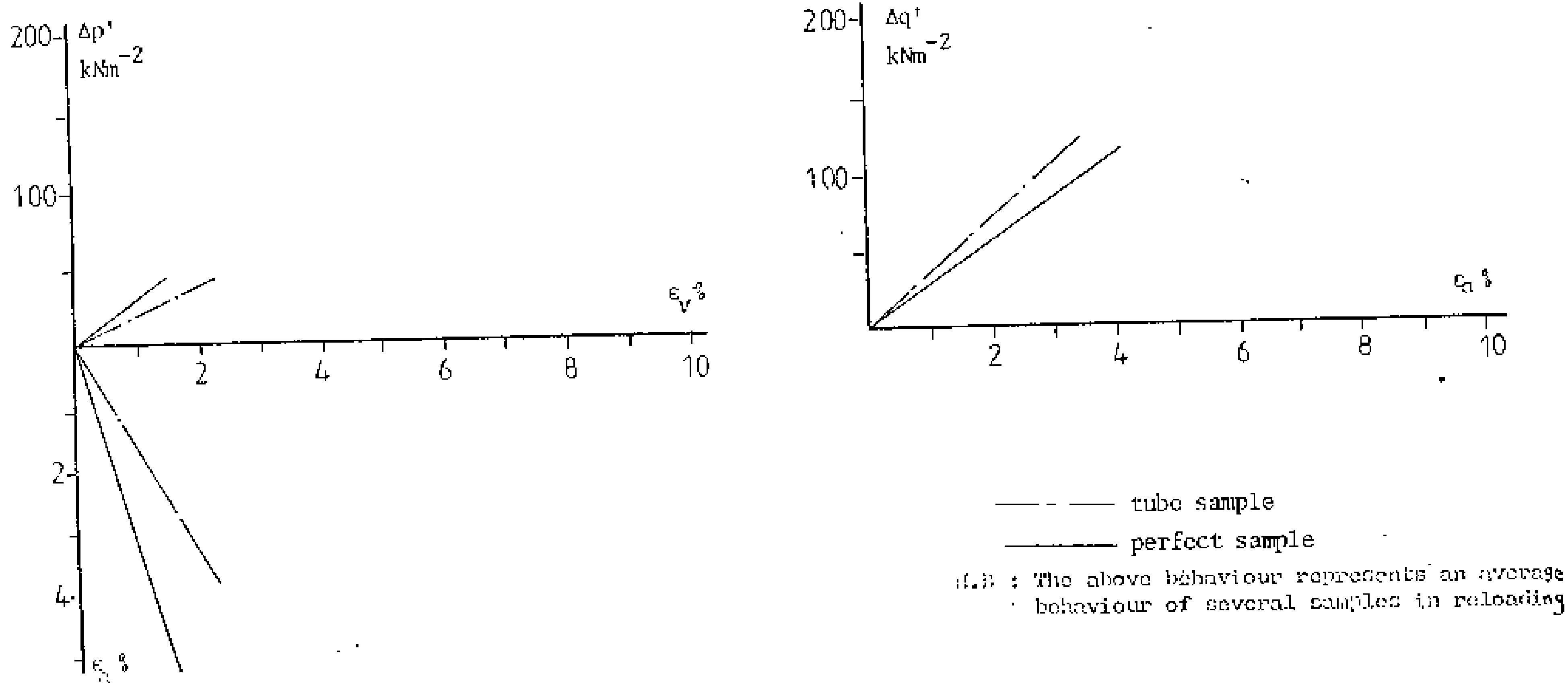


FIG. 7.11 THE RELOADING BEHAVIOUR OF PERFECT AND TUBE SAMPLES FOLLOWING PATH C

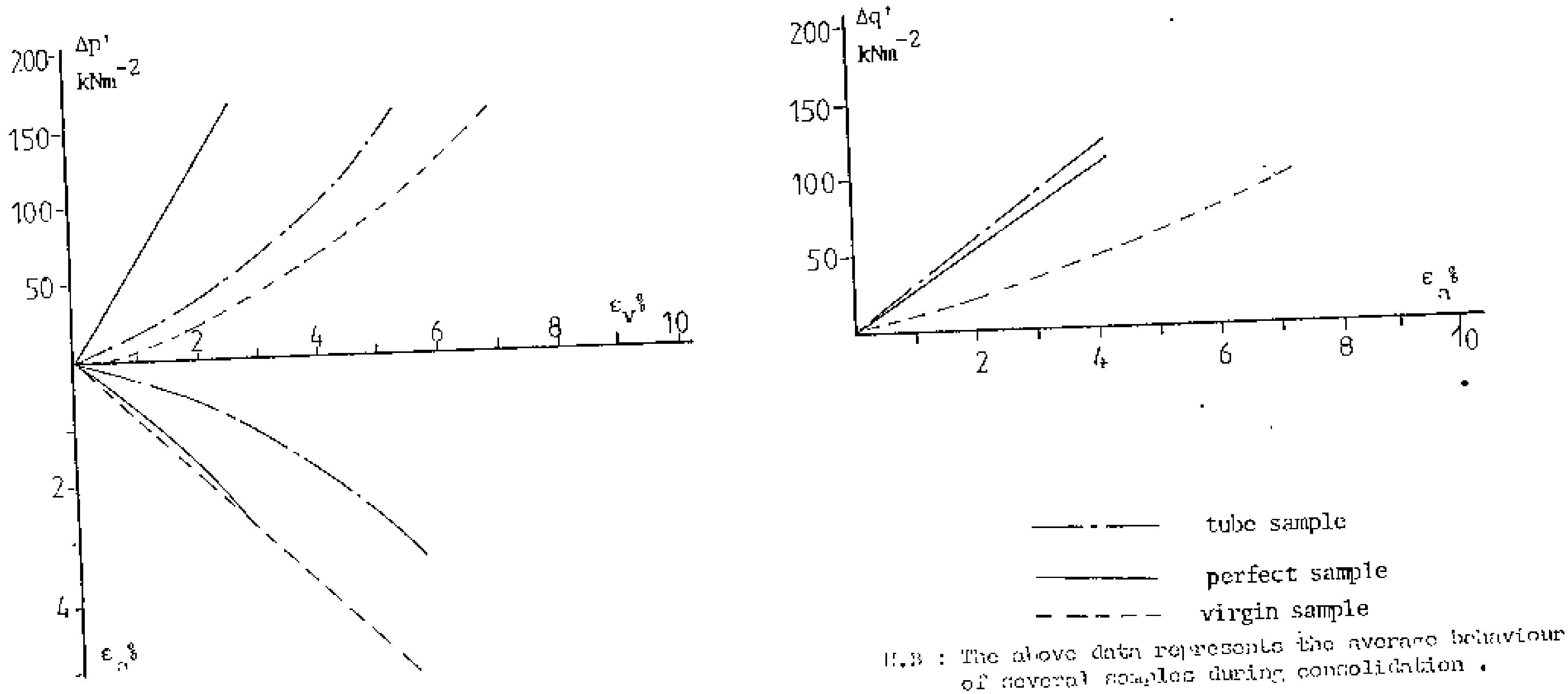


FIG. 7.12. THE RELOADING BEHAVIOUR OF PERFECT AND TUBE SAMPLES FOLLOWING PATH K

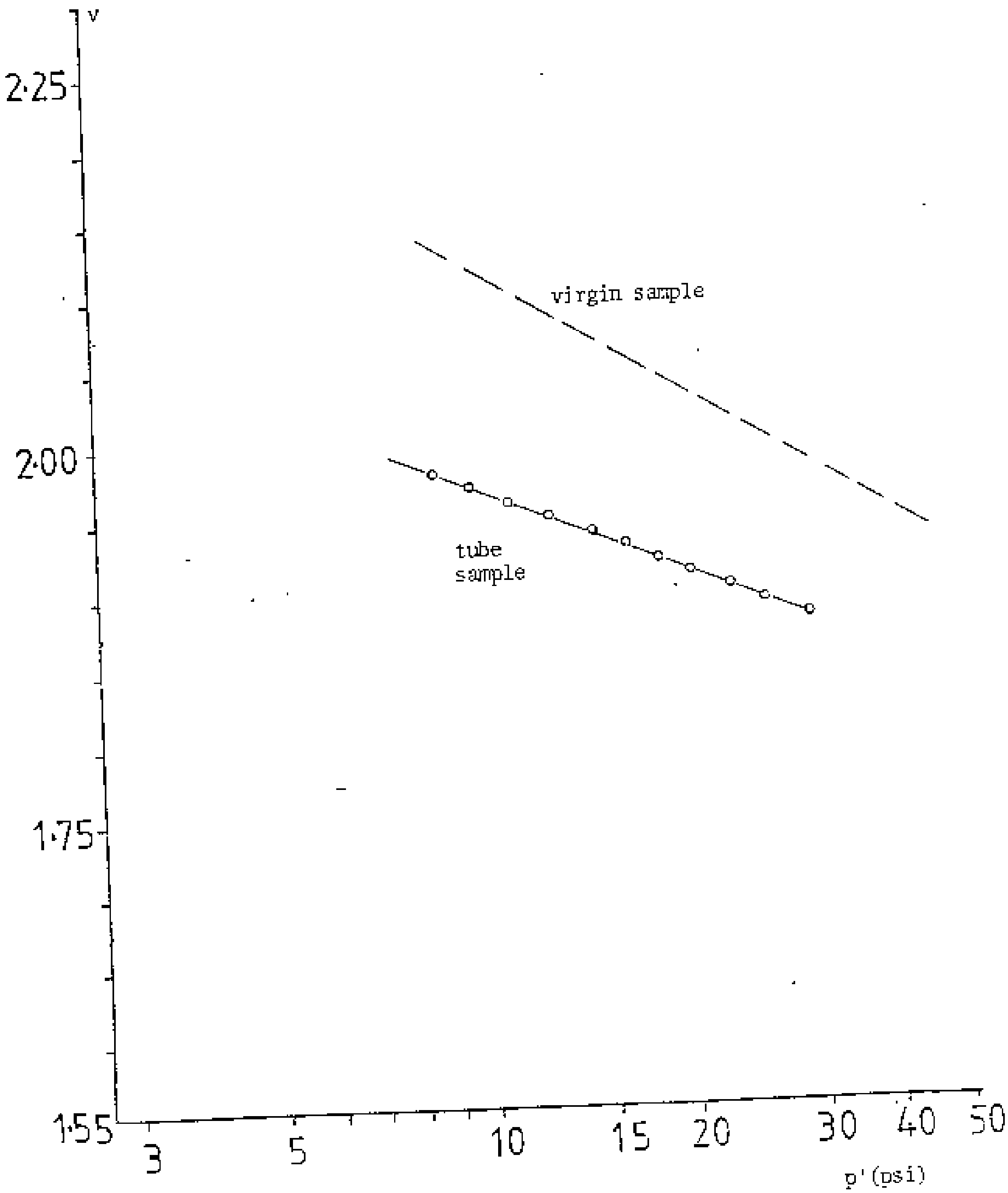


FIG. 7.13 RECOMPRESSION CURVE OF A TUBE SAMPLE

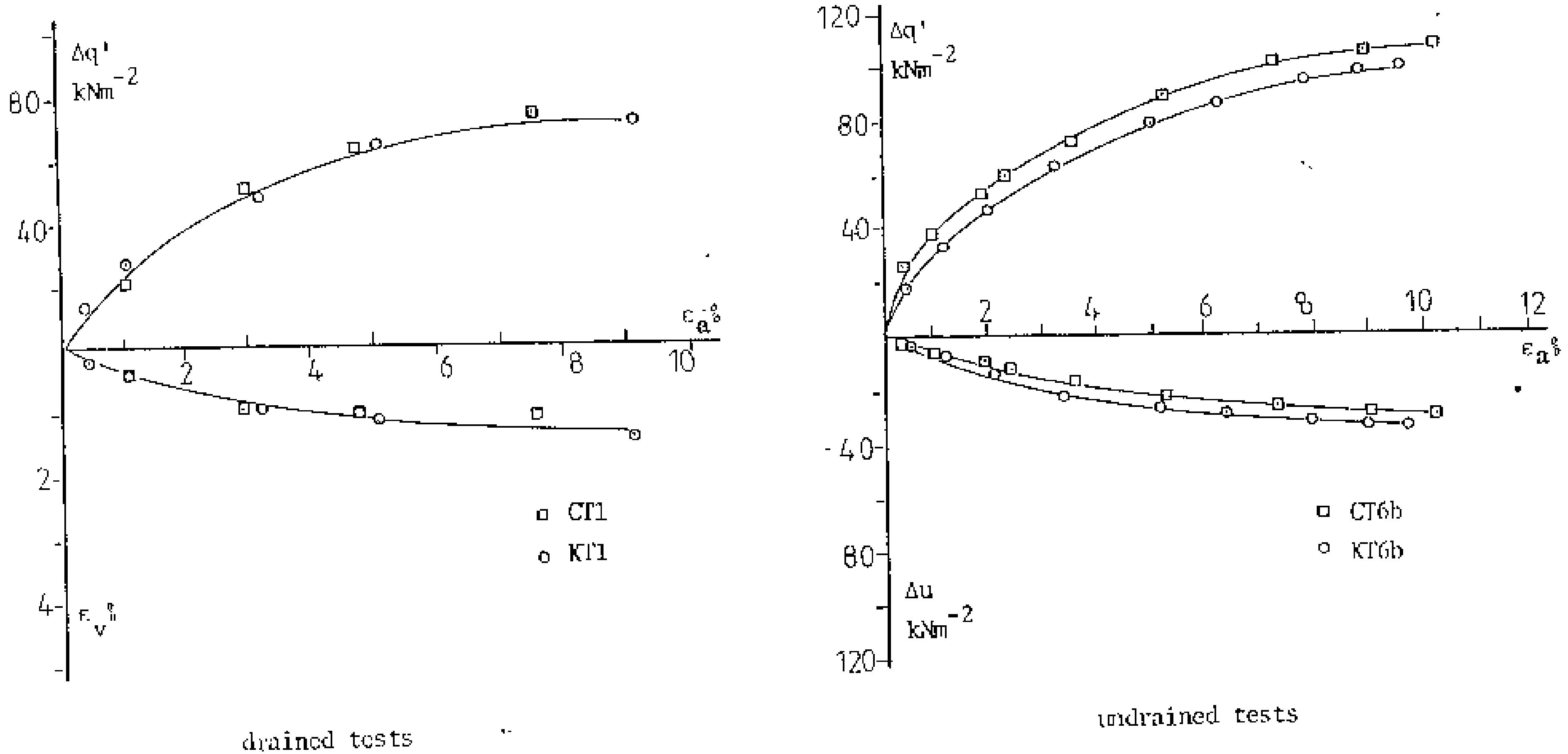


FIG. 7.4.1(a) SOME EFFECTS OF STRESS HISTORY ON THE BEHAVIOUR OF TUBE SAMPLES

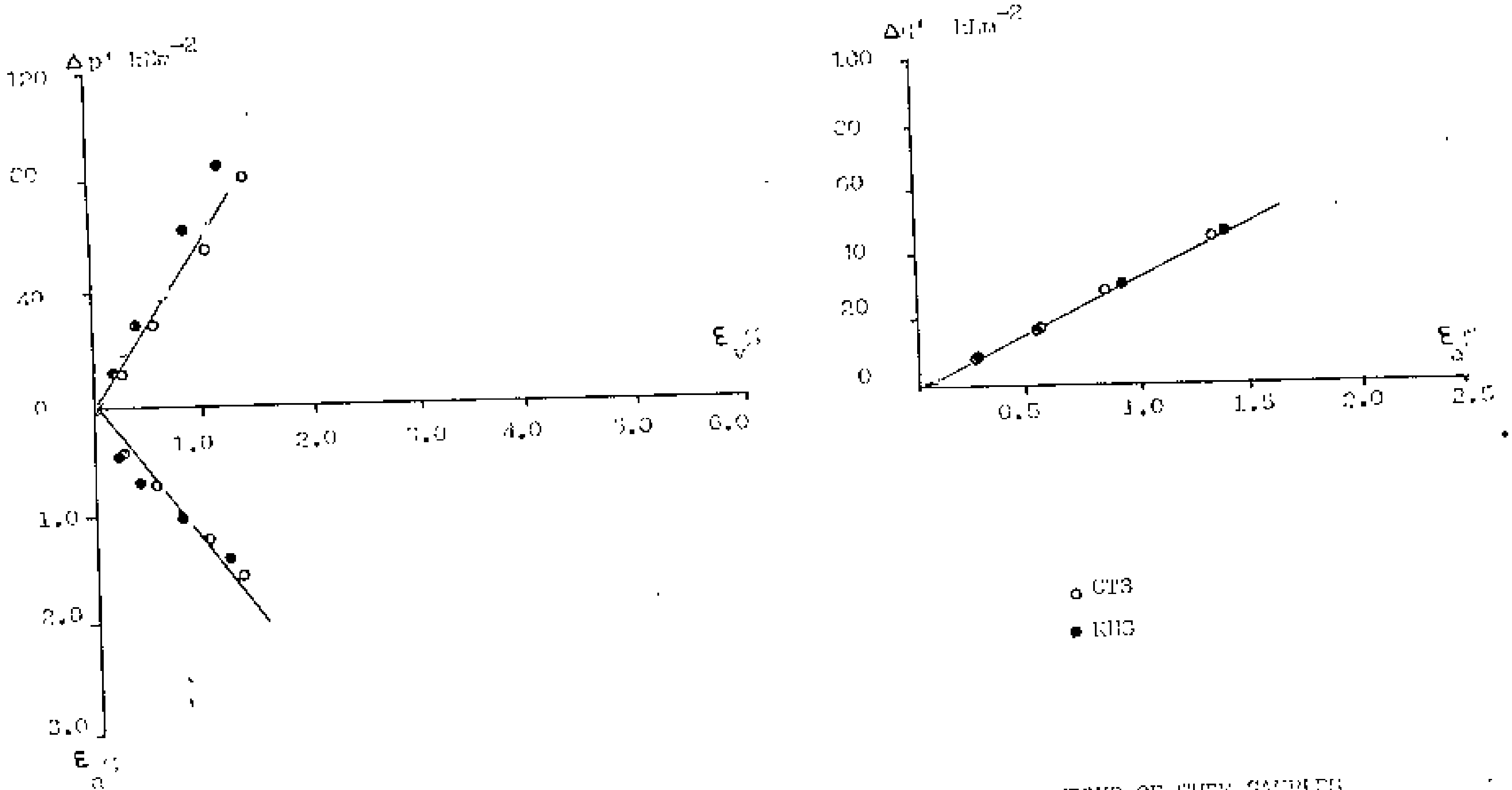


FIG . 7.14(b) .SOME EFFECTS OF THE RECONSOLIDATION PATH ON THE BEHAVIOUR OF TUFF SAMPLES

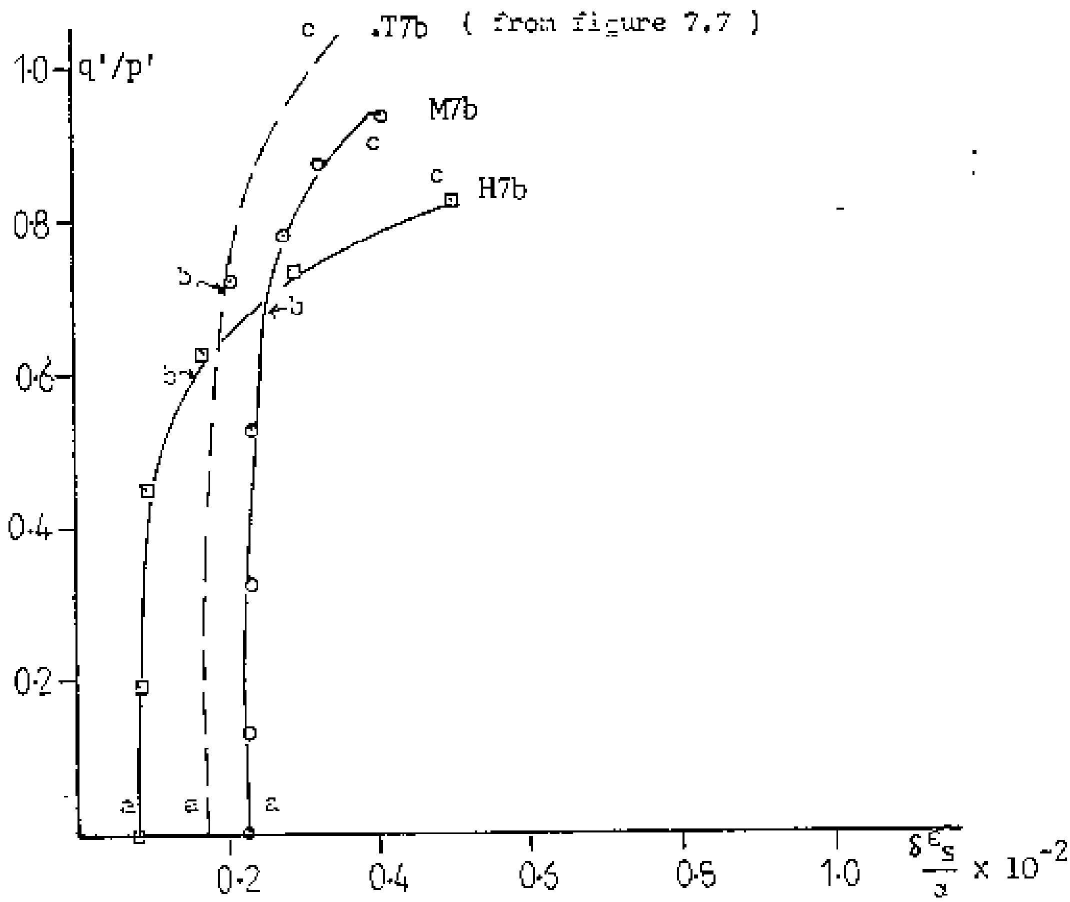


FIG. 7.15(a) BEHAVIOUR OF TUBE SAMPLES IN UNCONSOLIDATED UNDRAINED TESTS

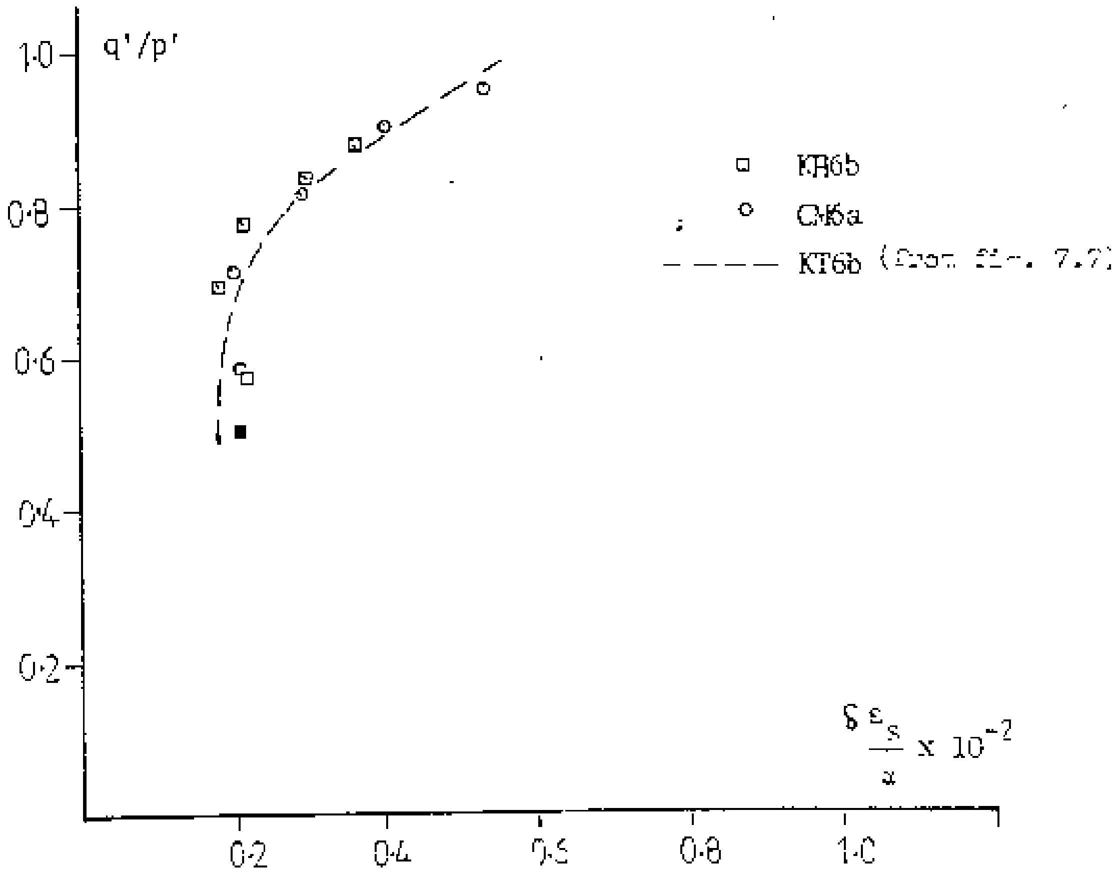


FIG. 7.13(b) BEHAVIOUR OF TUBE SAMPLES RECONSOLIDATED TO THEIR PREVIOUS STRESS STATE

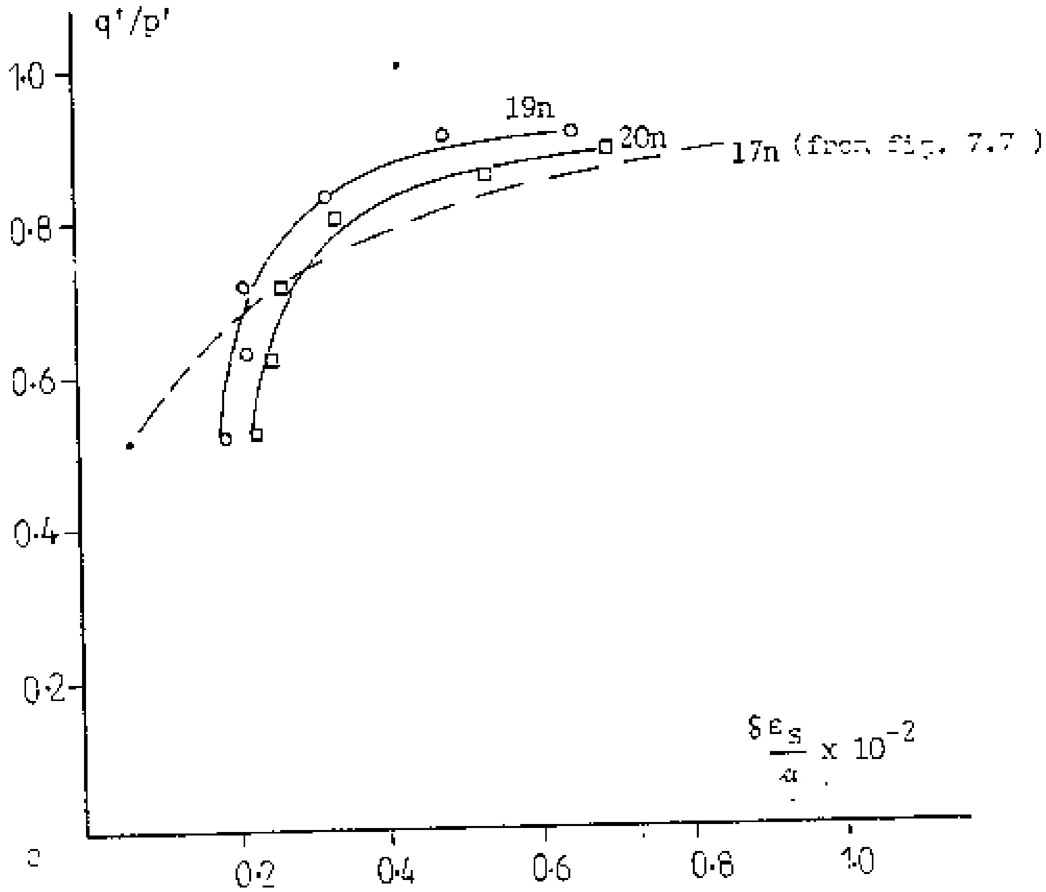


FIG. 7.15(c) BEHAVIOUR OF TUBE SAMPLES RECONSOLIDATED TO A HIGHER STRESS LEVEL

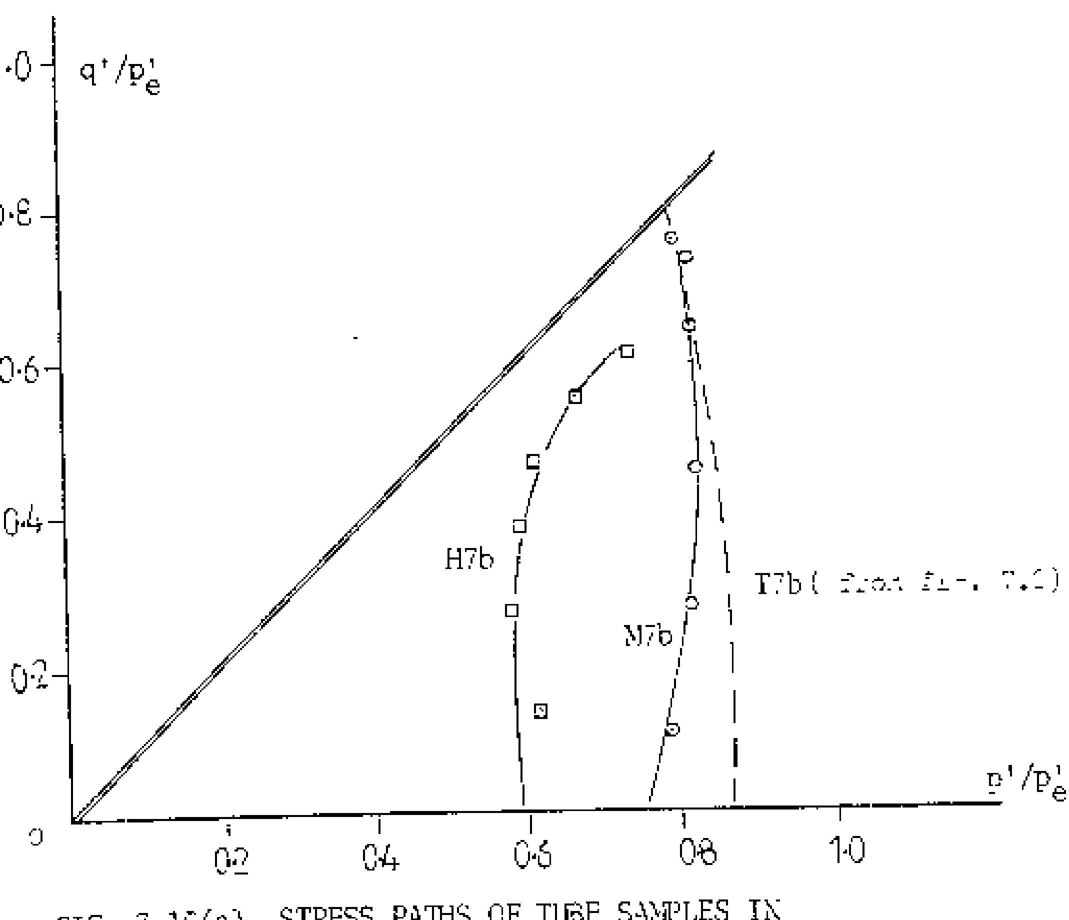


FIG. 7.16(a) STRESS PATHS OF TUBE SAMPLES IN UNCONSOLIDATED UNDRAINED TESTS

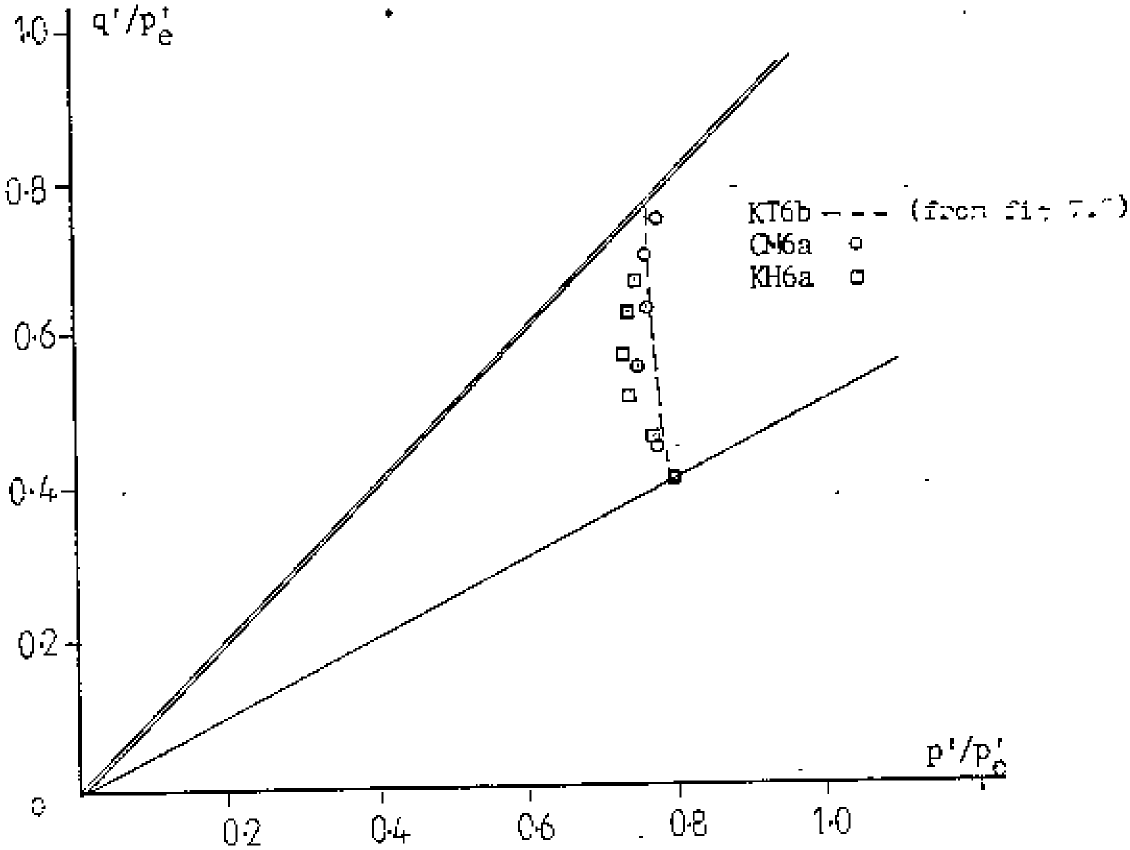


FIG.7.16(b) STRESS PATHS OF TUBE SAMPLES RECONSOLIDATED TO THEIR PREVIOUS STRESS STATE

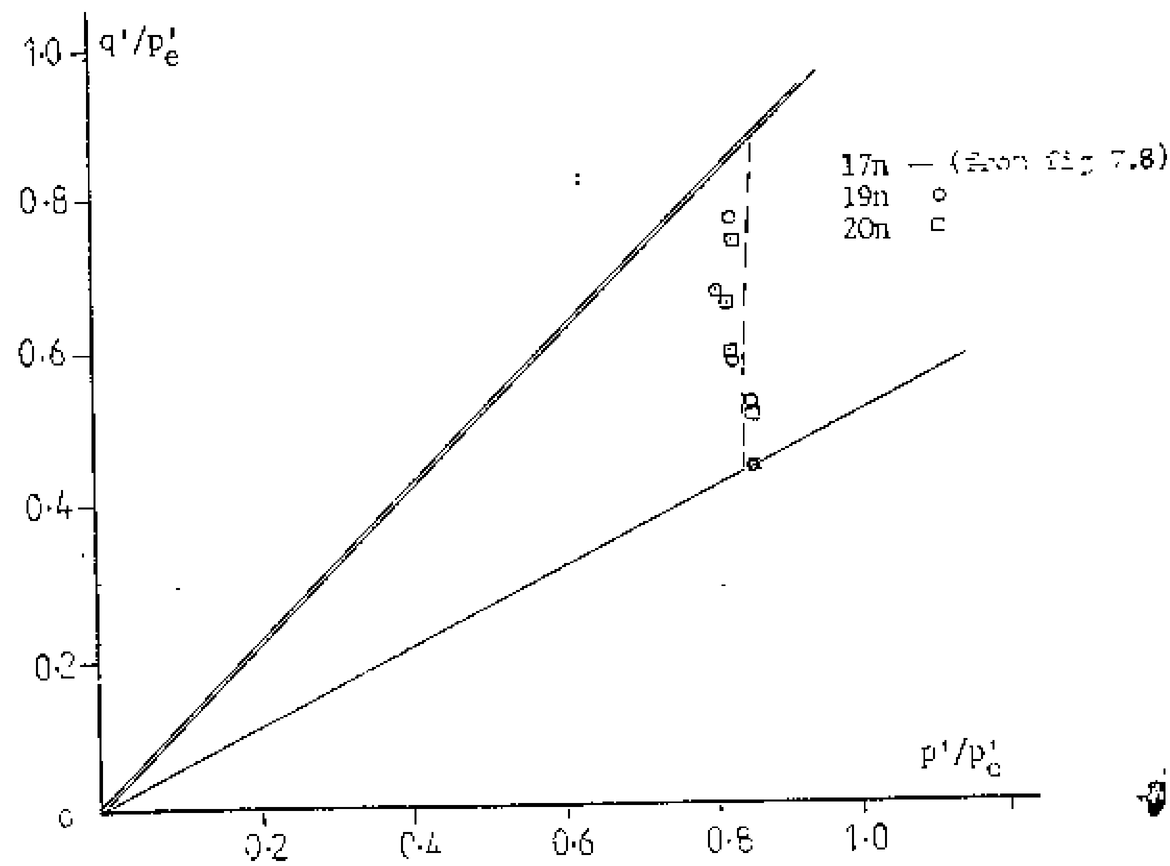


FIG.7.13(c) STRESS PATHS OF TUBE SAMPLES RECONSOLIDATED TO A HIGHER STRESS LEVEL

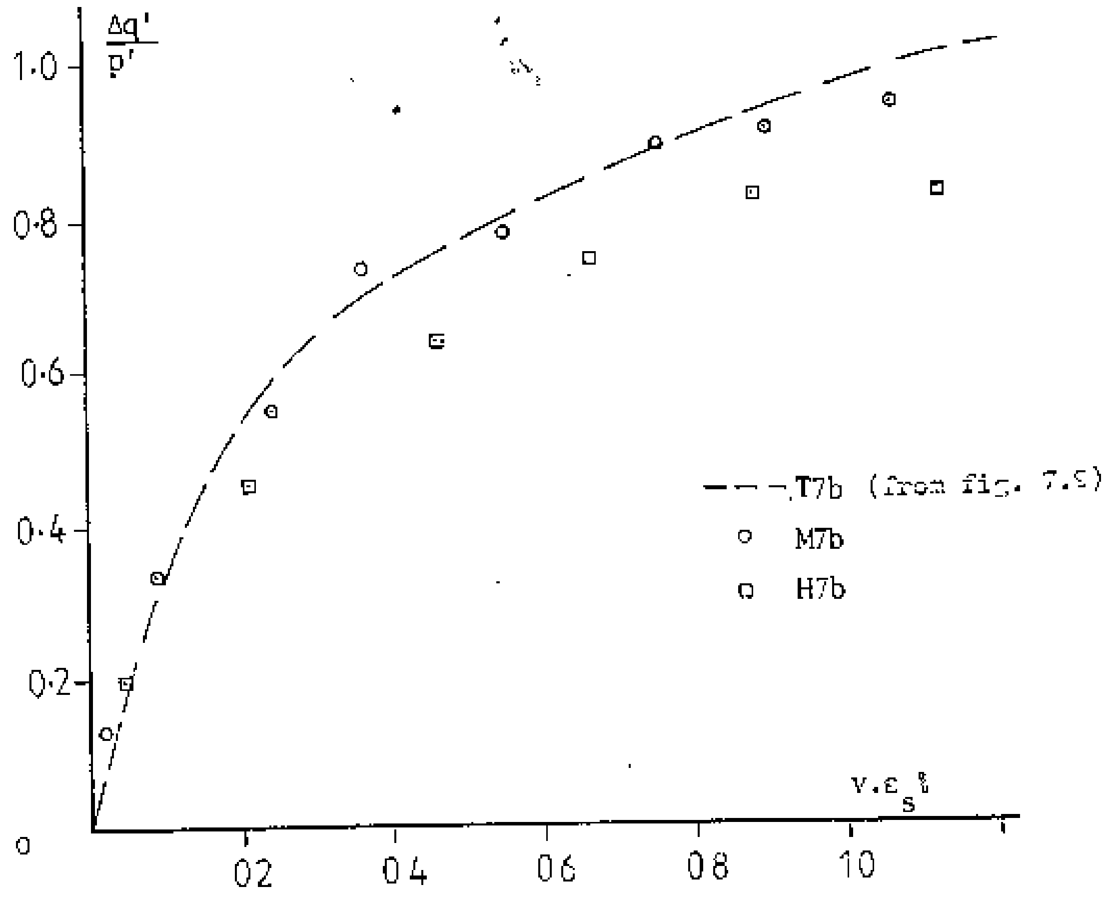


FIG.7.17(a) NORMALISED STRESS-STRAIN RELATIONSHIPS IN UNCONSOLIDATED UNDRAINED TESTS

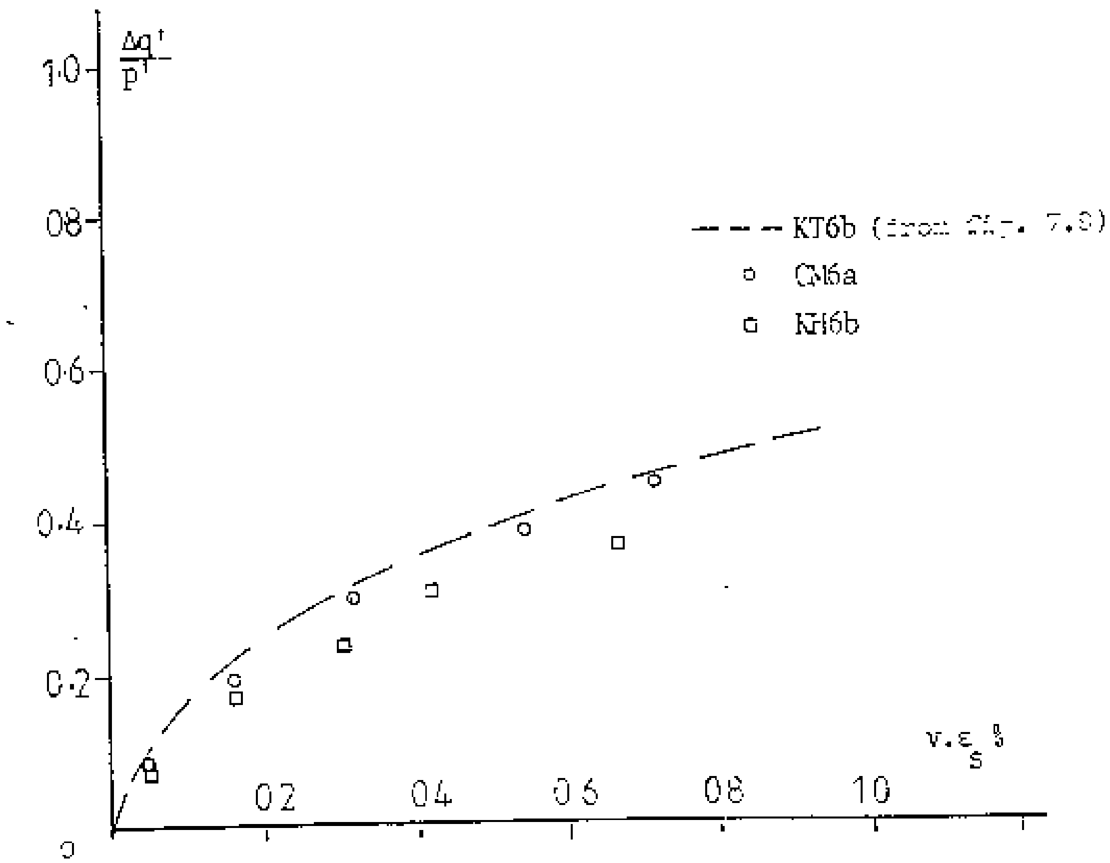


FIG.7.17(b) NORMALISED STRESS-STRAIN RELATIONSHIPS IN RECONSOLIDATED UNDRAINED TESTS

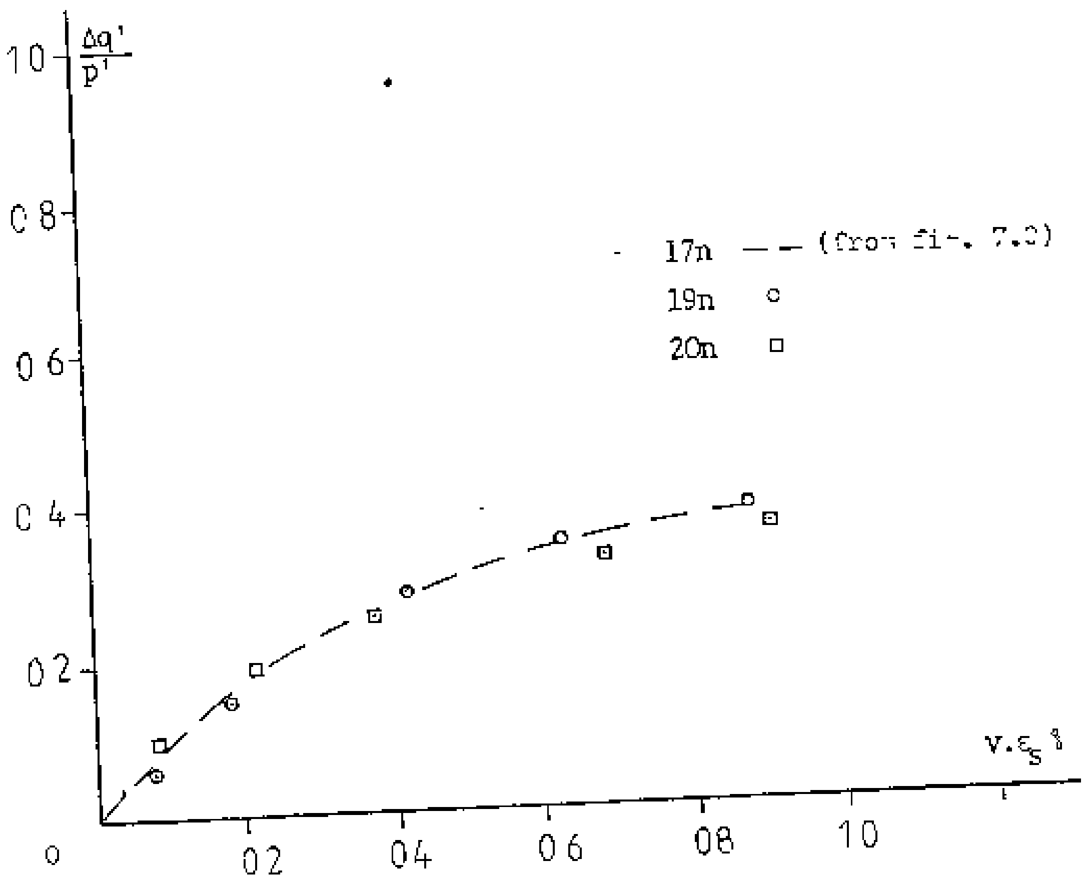


FIG. 7.17(c) NORMALISED STRESS-STRAIN RELATIONSHIP OF TUBE SAMPLES RECONSOLIDATED TO A HIGHER STRESS LEVEL

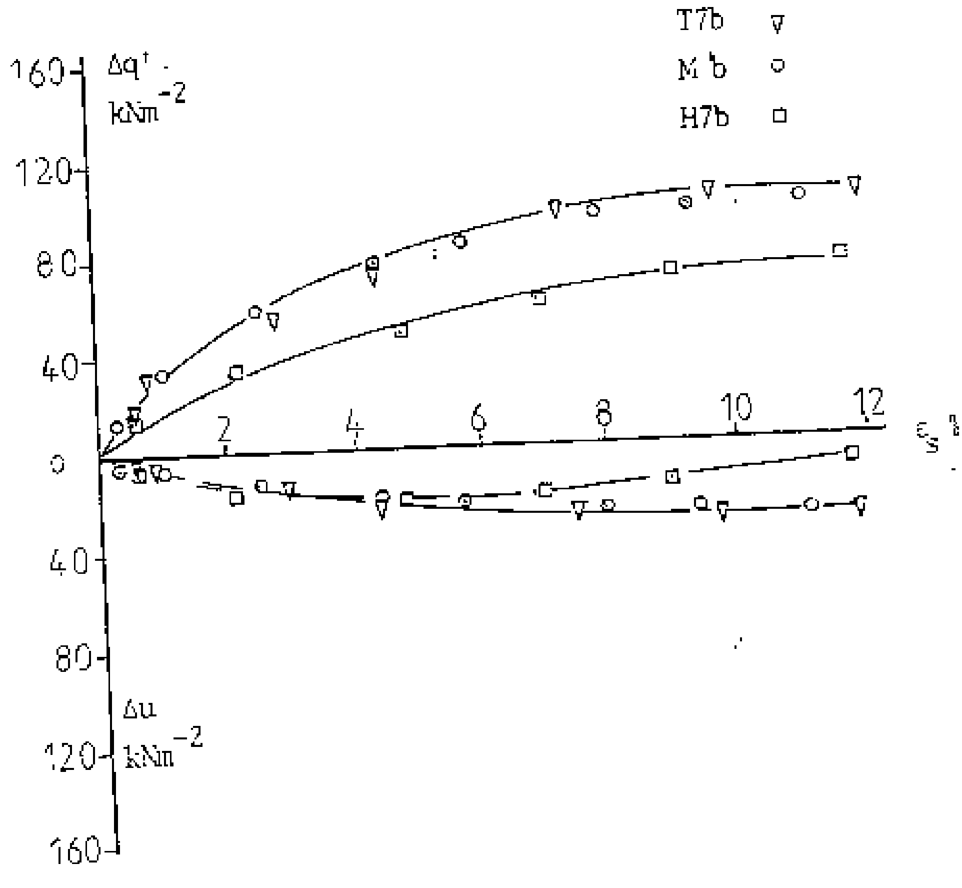


FIG. 7.18 BEHAVIOUR OF TUBE SAMPLES IN UNCONSOLIDATED UNDRAINED TESTS

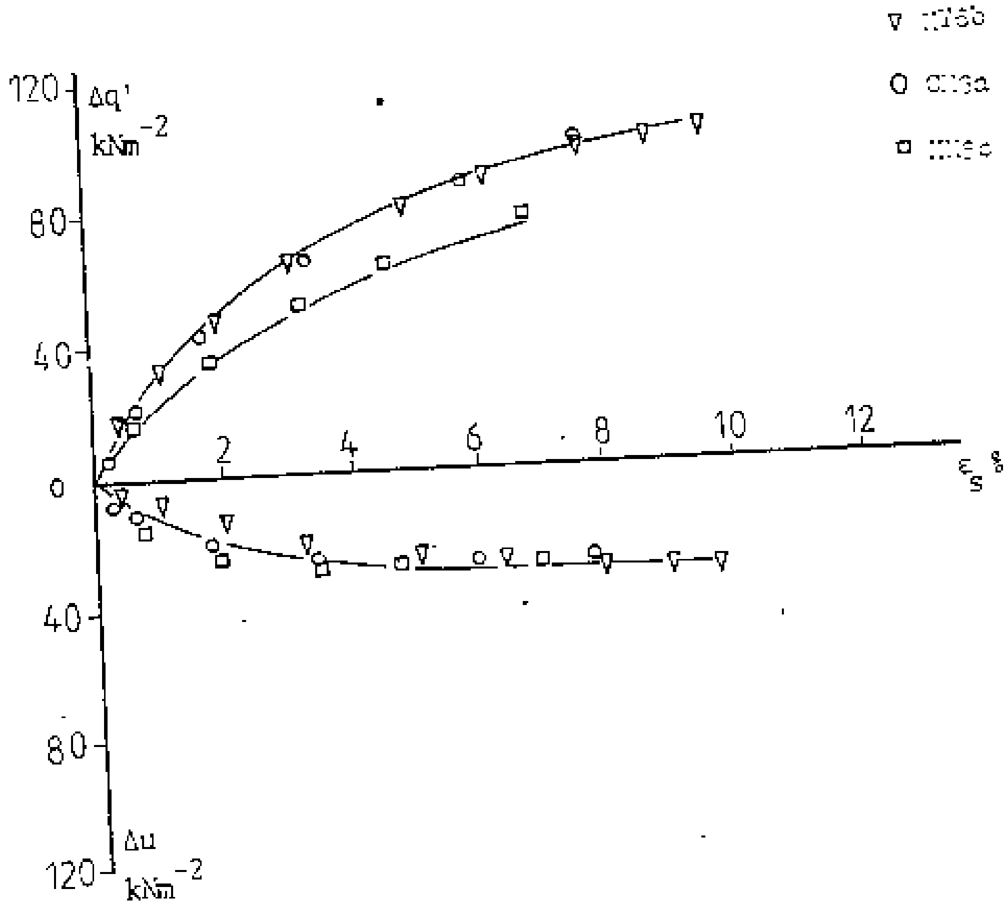


FIG. 7.19 BEHAVIOUR OF TUBE SAMPLES RECONSOLIDATED TO THEIR PREVIOUS STRESS STATE

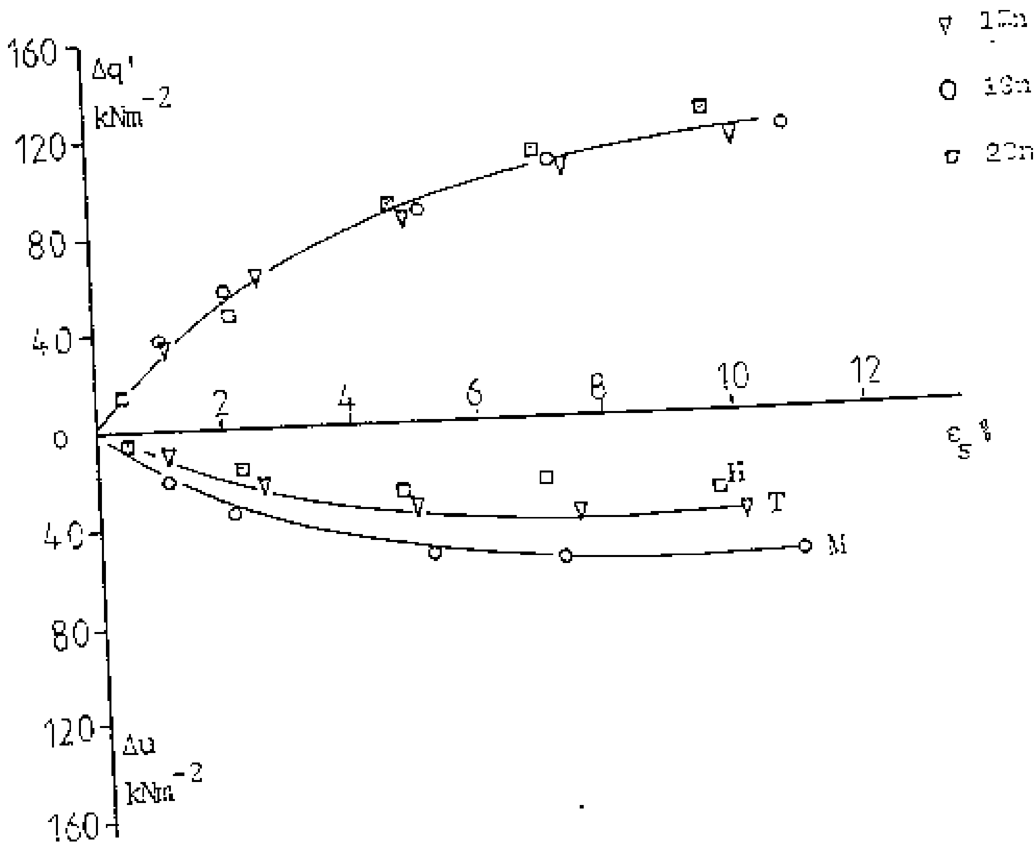


FIG. 7.20 BEHAVIOUR OF TUBE SAMPLES RECONSOLIDATED TO A HIGHER STRESS LEVEL

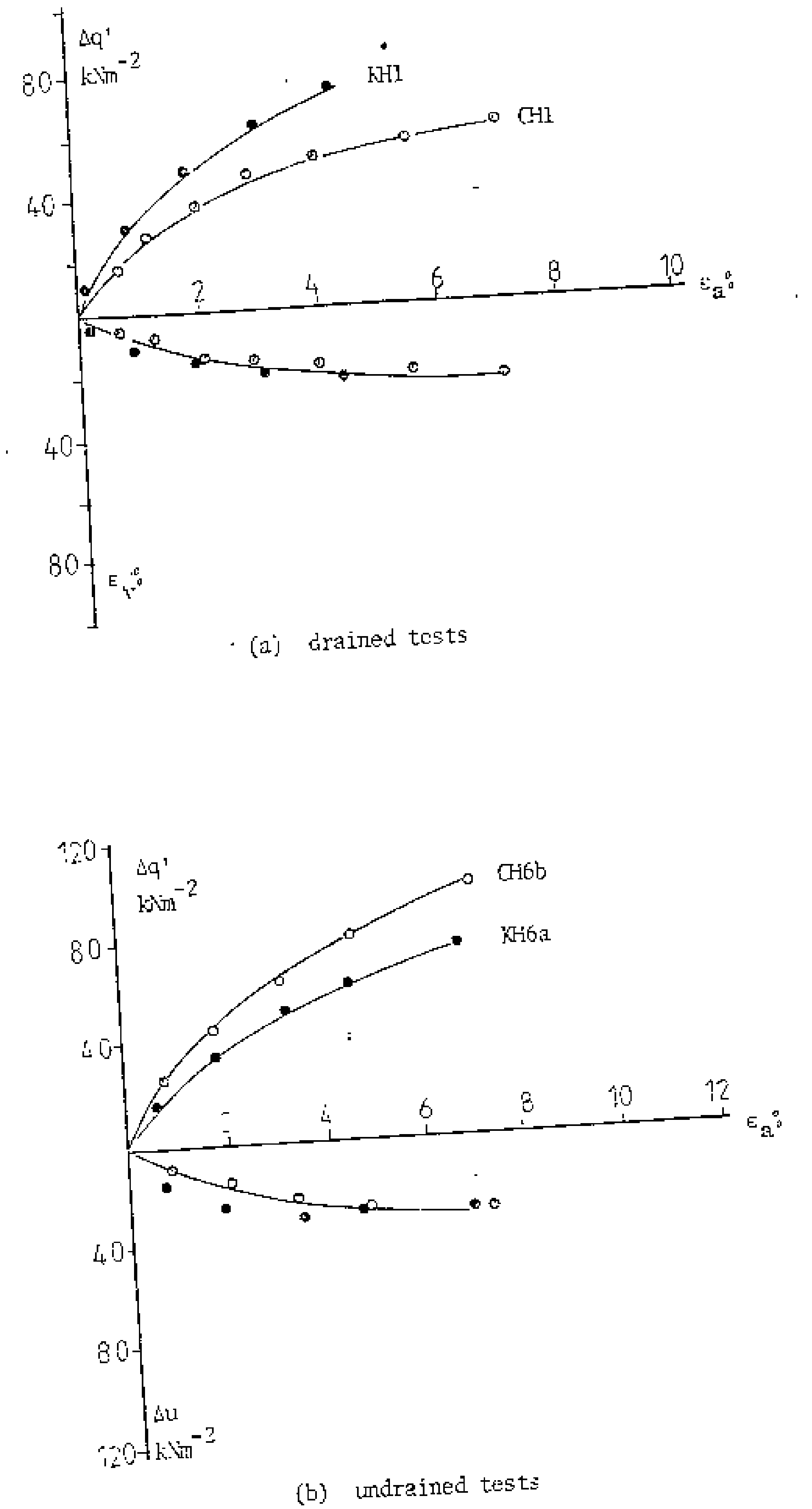


FIG. 7.21 EFFECTS OF STRESS HISTORY ON THE BEHAVIOUR OF HIGHLY DISTURBED SAMPLES

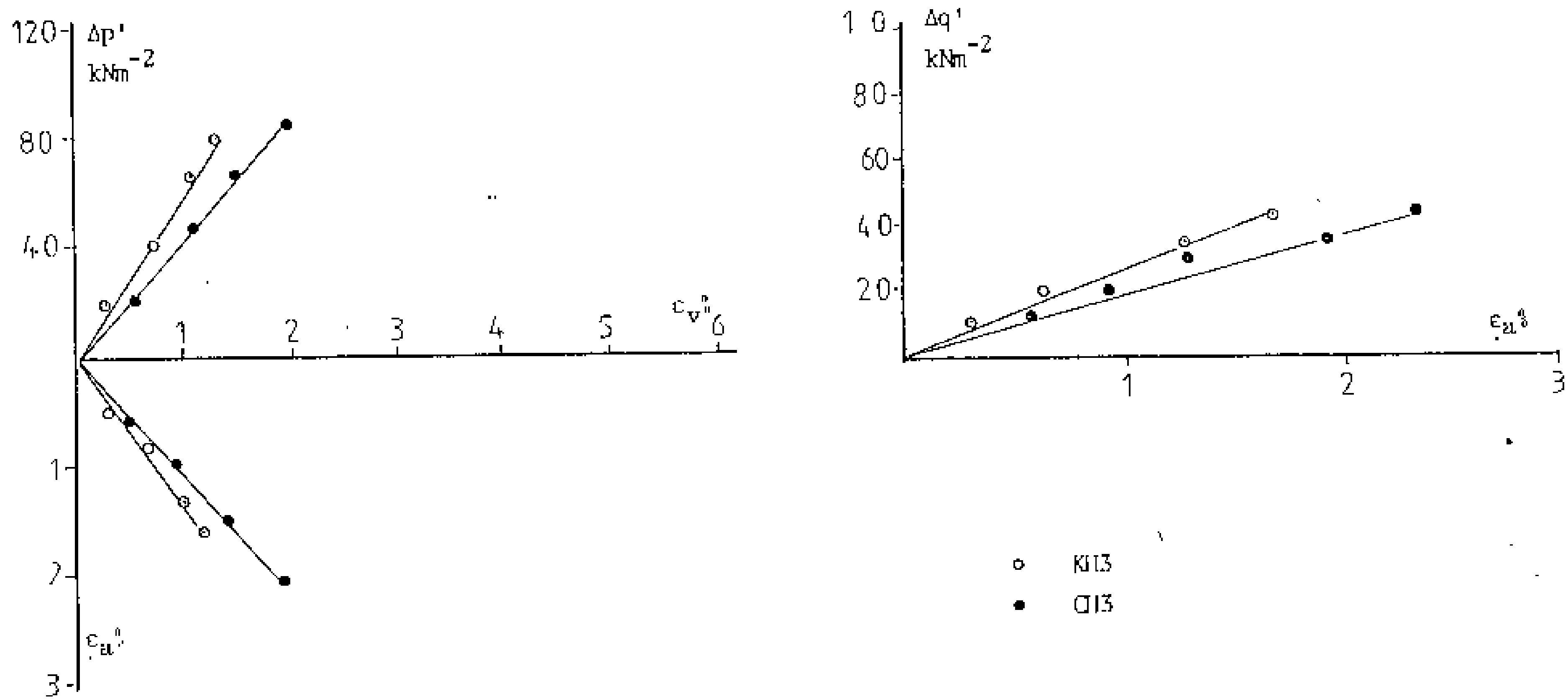


FIG. 7.22 EFFECTS OF STRESS HISTORY ON THE BEHAVIOUR OF HIGHLY DISTURBED SAMPLES .

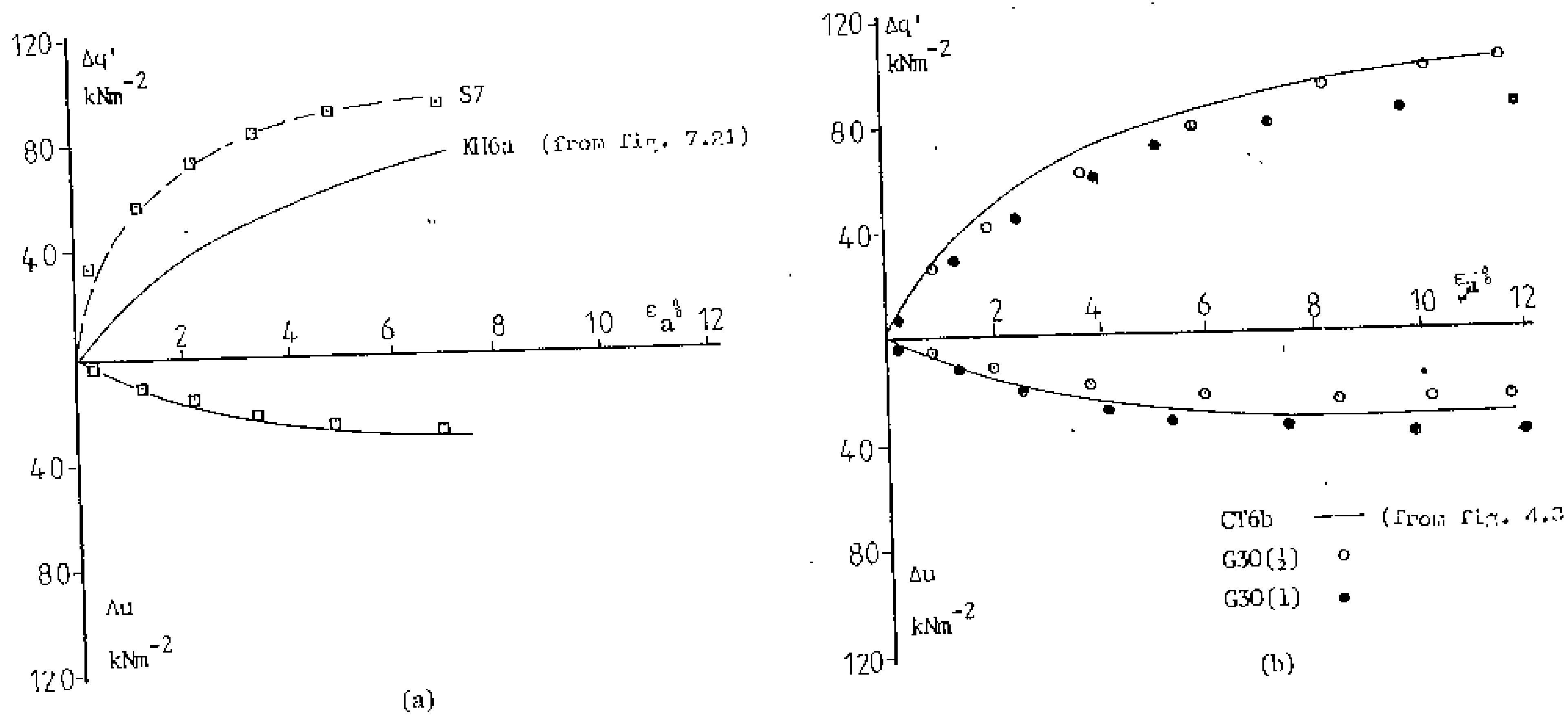


FIG. 7.23 EFFECTS OF STORAGE PERIODS ON THE BEHAVIOUR OF TUBE SAMPLES

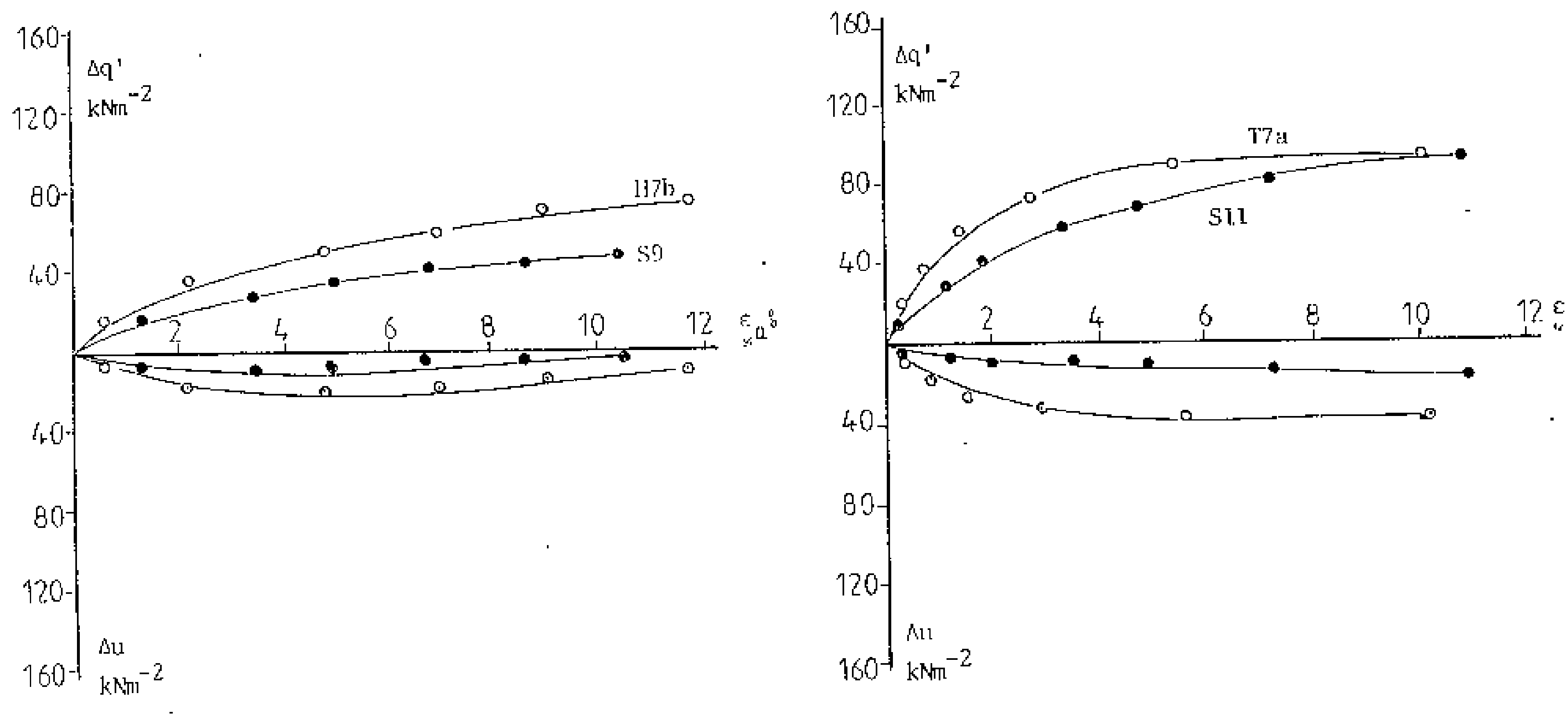


FIG. 7.24 EFFECTS OF INITIAL SWELLING ON THE BEHAVIOUR OF TUBE SAMPLES

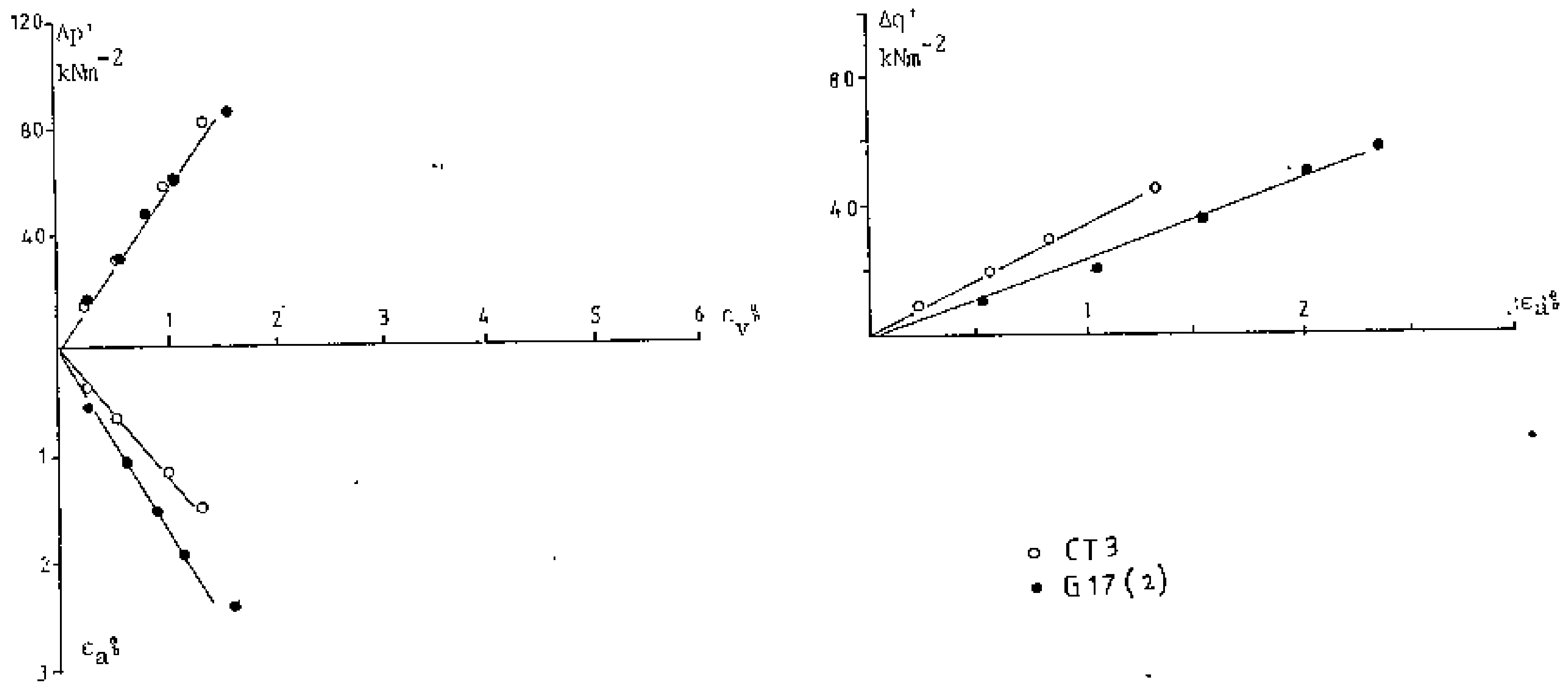


FIG. 7.25 EFFECTS OF THE STORAGE PERIOD ON COMPRESSION BEHAVIOUR

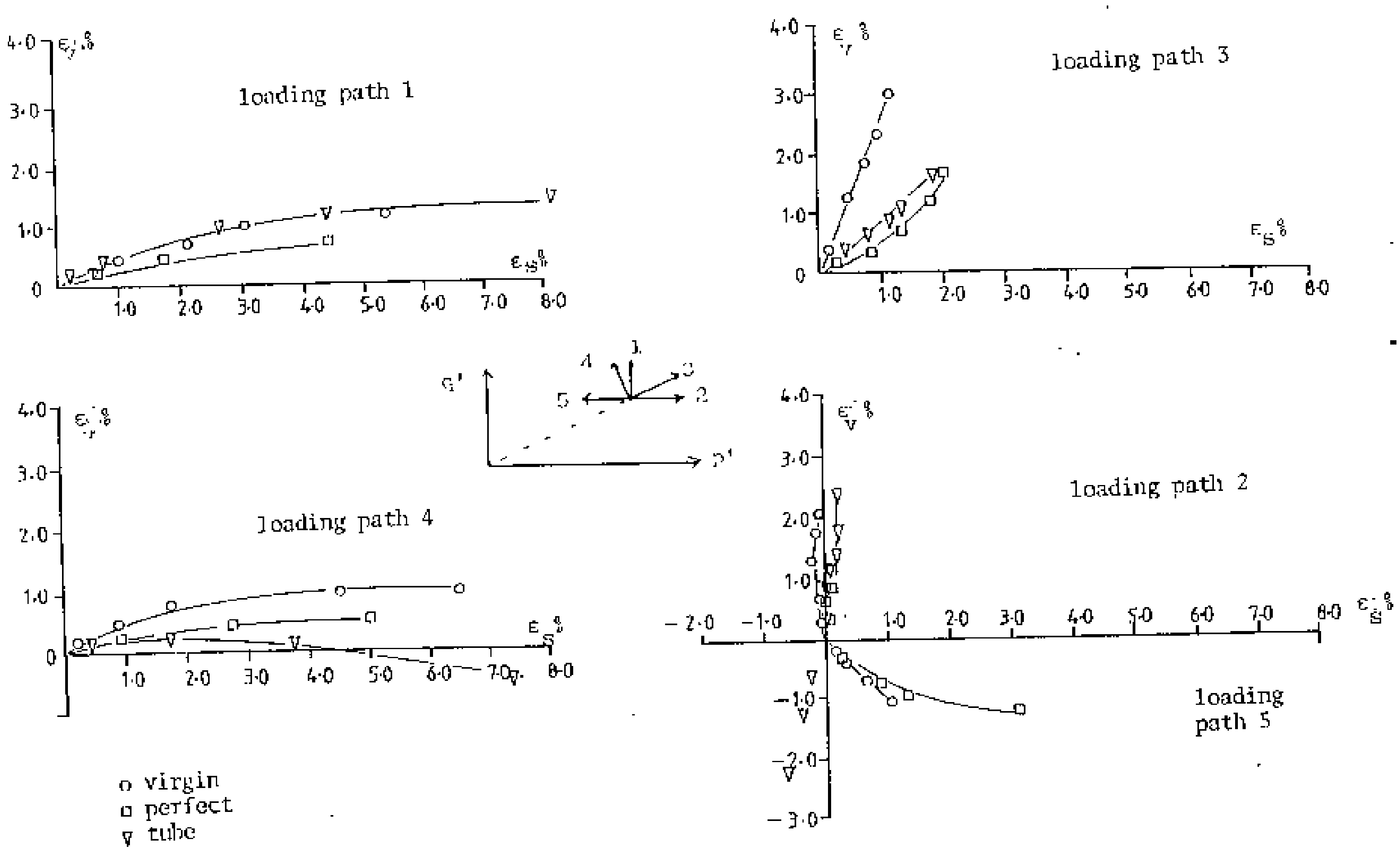


FIG. 7.26 STRAIN PATHS OF VIRGIN AND DISTURBED SAMPLES

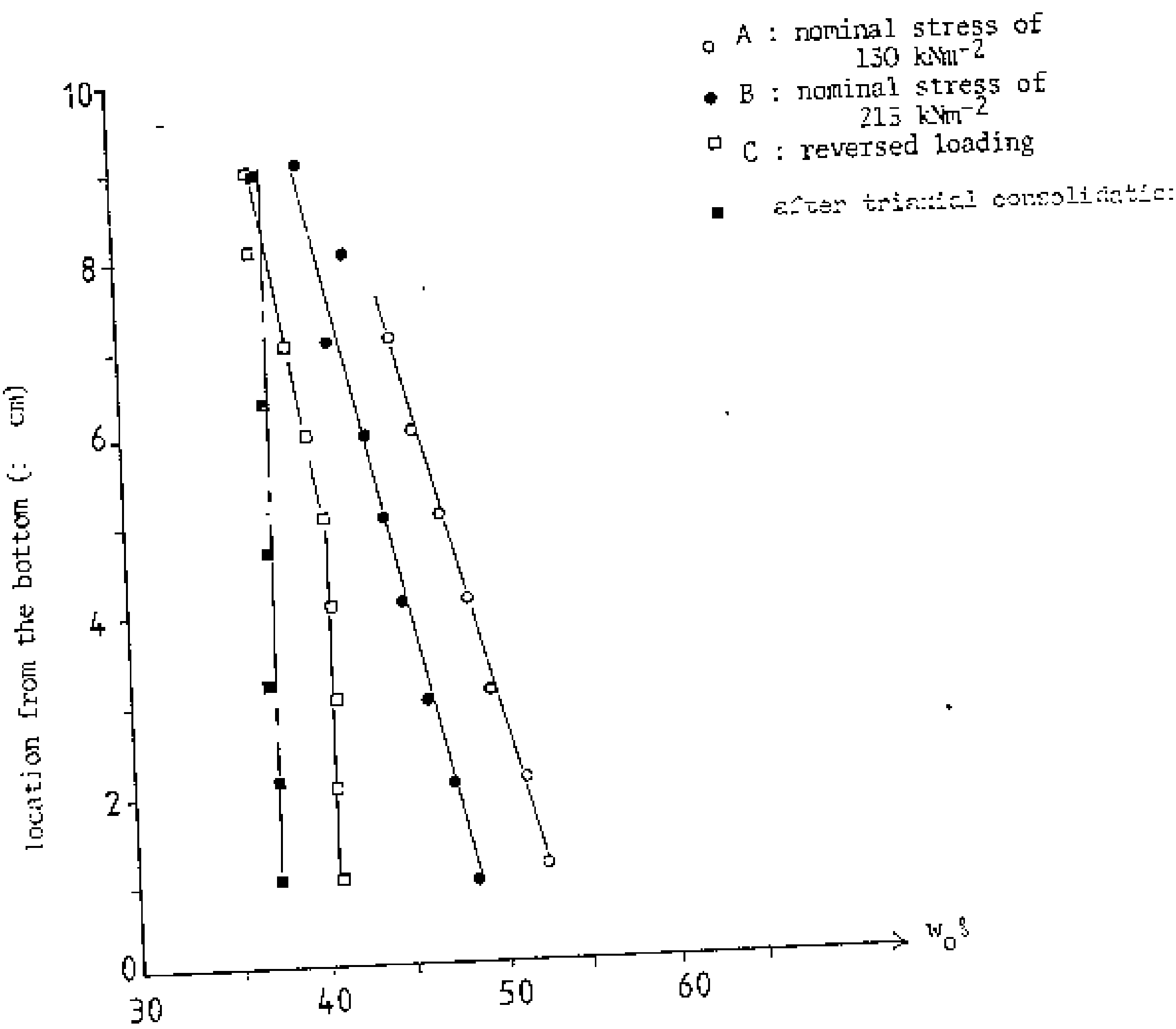


FIG. A.1 EFFECTS OF DIFFERENT TECHNIQUES IN REDUCING THE EFFECTS OF SIDE FRICTION

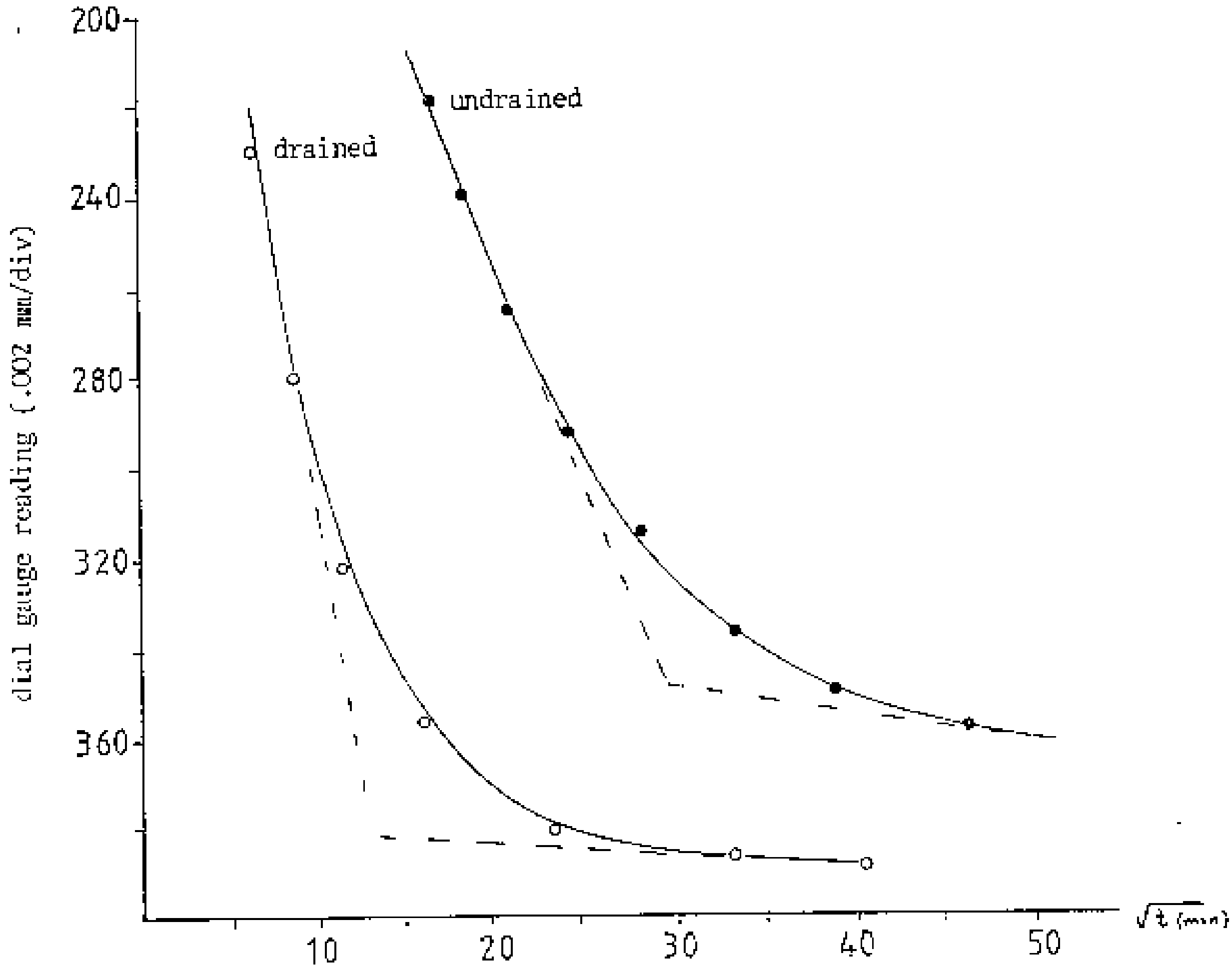


FIG. A.2 PRIMARY CONSOLIDATION AND EQUALISATION OF PORE PRESSURE

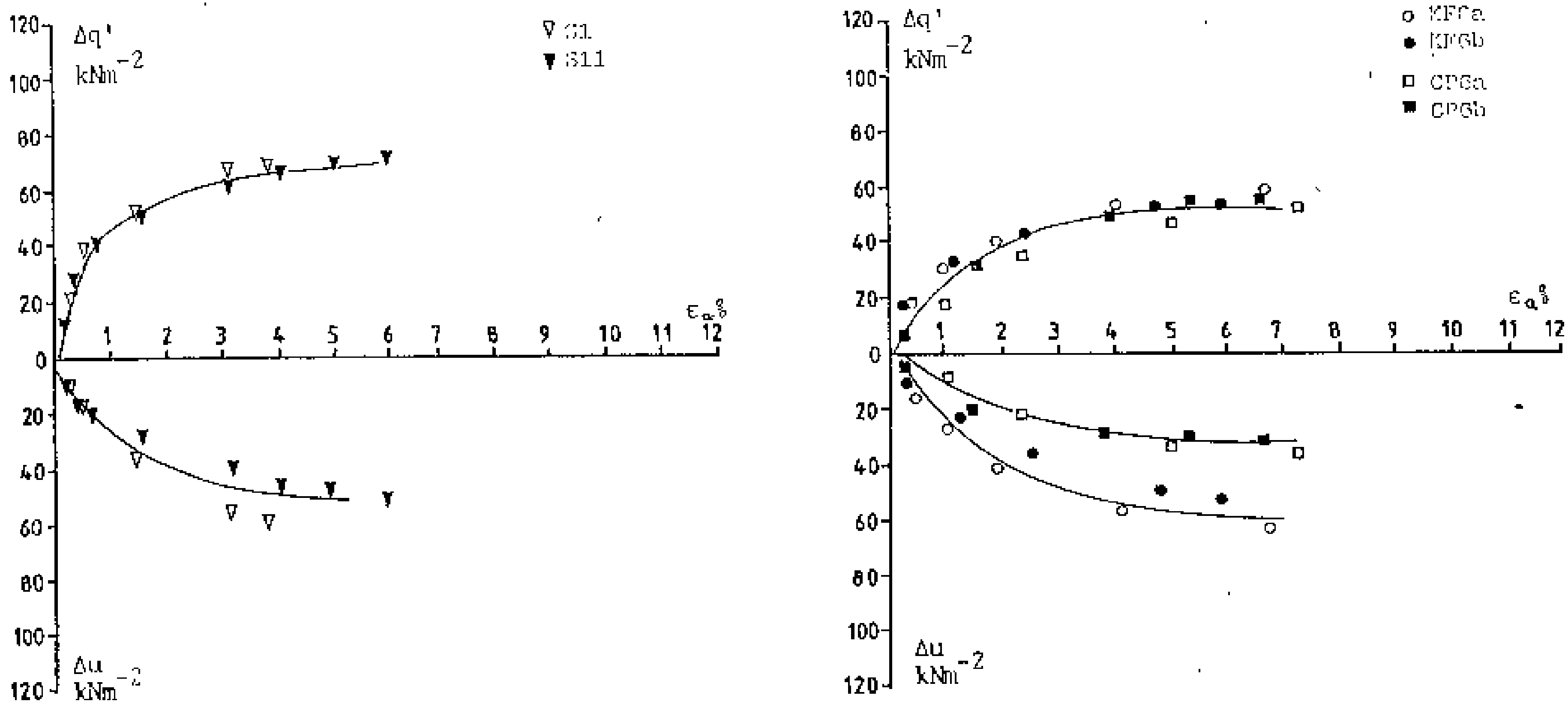


FIG. A.3 EFFECTS OF LOADING RATES IN UNDRAINED TESTS

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