

LABORATORY INVESTIGATIONS
OF SEA-BED CLAYS

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David William Hight, M.Sc., D.I.C.

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ABSTRACT

The thesis describes a laboratory investigation into aspects of the behaviour of soils in the foundations to offshore structures. The investigation is set in the context of oil production platforms in the Northern North Sea. There are three parts to the thesis.

In the first part, the soil types, their history and the stress changes to which they are subject, both in the foundation and during sampling, are reviewed. The cyclic and transient nature of the loading and the large component of principal stress rotation in the stress paths are highlighted. Low plasticity glacial clays are selected for study and an attempt is made to justify the use of these in a reconstituted state for the investigation.

The second part of the thesis describes the development of new laboratory equipment required for the work: a hollow cylinder apparatus for investigating the effects of principal stress rotation; a computer system for controlling the hollow cylinder apparatus and for conducting cyclic tests in the hydraulic triaxial cell; a piezometer probe for pore pressure measurement in clays under cyclic and transient loading.

In the third part, the observed behaviour of a low plasticity clay under monotonic, cyclic and transient loading is reviewed. Features of this behaviour are examined in terms of the microfabric of the clay.

Finally the implications of these observations in the interpretation of field and laboratory tests in low plasticity clays are considered. The important part played by wave loading in the stress history of the soils is emphasised.

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There are recurring references in this Thesis to (Gens, pers. comm.). It will be well understood by those who have had the pleasure of working with him that these references, although liberally spread, can in no way reflect the immense contribution that Antonio Gens has made.

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CHAPTER 1

SOME ASPECTS OF FOUNDATION ENGINEERING
OFFSHORE

SYNOPSIS

The essential differences between foundation engineering offshore and on-land are reviewed. Areas of uncertainty are noted. Certain of these have been selected for investigation in a research programme which is to be described in this thesis.

1.1 INTRODUCTION

Exploration for oil and its subsequent production from below the sea bed have led to a massive expansion in civil engineering offshore. The novelty of the offshore situation has presented new challenges and new problems to solve.

For the geotechnical engineer, the design of foundations to offshore structures involves considerable extrapolation from on-land experience. There are essential differences in terms of the structures involved, their loading, the soils with which they interact, the methods by which these soils may be investigated, and, as a natural consequence of the foregoing, the methods and form of analyses which must be undertaken. These differences are summarised in Table 1.1 and considered in more detail below.

Extrapolation inevitably involves uncertainty. In the following review, areas are identified in which these uncertainties arise. Research, set mainly in the context of gravity structures in the Northern North Sea^{*1}, has been carried out into some of these areas and forms the subject of this thesis.

1.2 OFFSHORE STRUCTURES

Offshore structures for oil production currently (1983) take one of two forms, a braced steel jacket structure which is pinned to the sea bed by piles, or a gravity structure which sits directly on the sea bed and relies on its weight to maintain its stability. The two forms of structure are essentially different in form and concept. Piling of the jacket structure leads to integrated behaviour of the structure and the whole soil foundation. The performance of gravity structures, on the other hand, is dominated by the weakest link in the foundation, either pre-existing or which develops during the life of the structure.

Offshore structures cannot be constructed in their final location but must be built elsewhere, launched, transported and installed. These requirements naturally have a major influence on their design. Their installation presents the first challenge to the geotechnical engineer.

*1 The Northern North Sea is considered herein as that part of the North Sea between 56°N and the edge of the continental shelf and between the British coast and the western edge of the Norwegian Trench. Particular attention will be paid to that area in which gravity structures are sited.

Geotechnical conditions at and close to the surface of the sea bed have an important bearing on the installation of gravity structures. Soil strengths and stiffnesses determine the resistance of the sea bed to penetration by the skirts and dowels, which are a feature of this form of structure. Soil variability, in terms of composition and strength, over the area of the foundation, together with sea bed topography, play a part in determining the contact stresses which develop initially at the base of the structure and which may influence design.

Predictions of the driveability of piles during installation of jacket structures is of crucial importance. Dynamic properties of soils over a considerable depth determine, in part, the resistance to pile penetration.

Since the sea-bed cannot be prepared there is an inherent lack-of-fit between a gravity structure and its supporting foundation. Concrete is not cast directly on to the soil and there must be uncertainty over the contact conditions. During site preparation and construction in place on land, further information is often acquired which leads to design modifications being made; this opportunity does not arise offshore.

The installation process itself leads to disturbance of the near-surface soils, both directly by shearing and indirectly by the development of high hydrostatic pressures between the structure base and the sea bed. This disturbance adds to the uncertainty of the contact conditions. Additional disturbance may arise when well drilling is carried out through the structure's base.

Installation of a more-or-less completed structure means that interactive effects between soil and structure take a completely different form, compared with structures ashore. The changing relative stiffness between the soil and structure as construction proceeds in place is no longer present; differential settlements cannot be 'built out'. Piling offshore is carried out after a major proportion of the structure is placed and not before construction, as on land; the distribution of load between the ground supports (raft or mudmat) and the piles is likely to be different.

The scale of offshore structures is related closely to the water depths in which they are deployed. In the Northern North Sea, structures are currently being installed in water depths up to 180m. These structures are vast. Typical gravity structures for depths of 150m have a base area of $10,000\text{m}^2$ and a total weight of 350,000 tonnes. At such a scale, variations in soil properties, both vertically and horizontally, take on an even greater importance in determining performance than for land-based structures. Geotechnical information on a very large volume of soil is required. The scale of the structures, combined with the rates at which they are loaded, also means that undrained events predominate.

1.3 LOADING OF OFFSHORE FOUNDATIONS

Installation involves application of a large proportion of the structural dead load in one step over a short period of time. In the Northern North Sea skirt penetration rates are reported (Lunne and St John, 1979) to have varied between 0.0044 cm/sec and 0.0190 cm/sec . Full dead load is applied in a matter of hours, in contrast to the 2 to 3 year construction period for multistorey structures on land.

In place, offshore structures are subject to a totally different loading regime to their on-land counterparts. Hydrodynamic loading by waves results in large omnidirectional horizontal forces and moments being applied to a structure. In contrast to on-land foundations, except, perhaps, those to storage facilities, road and airfield pavements and railway tracks, live loading represents a major proportion of the total load and plays an important part in determining performance. The magnitude of the live loading is a complex function of wave-structure interaction, depending in part on sea state and sea depth. An essential feature of the loading is its cyclic nature.

Cyclic horizontal forces and moments are filtered and transmitted through the structure to the sea bed. Stress paths result which are not commonly encountered ashore. In the case of gravity structures, the stress paths involve large cyclic rotations in principal stress directions. The nature of these stress paths is examined in Chapter 2.

In the Northern North Sea, the period of a loading cycle ranges between 5 and 25 seconds. For both dead and live loading, then, rates of application are not typical of those encountered on land.

1.4 OFFSHORE SOILS

In the Northern North Sea, Quaternary deposits are up to 1000m thick (Fannin, 1979). With present day sea depths of over 150m it seems probable that some of these soils may have been deposited through water from floating glaciers and calving icebergs. If so, important differences could be expected between soils deposited in such a glaciomarine environment, and soils deposited by lodgement or ablation with which there is experience on land. These differences are explored in Chapter 3, using a variety of techniques: X-ray, electron microscopy and polarizing microscope. The usefulness of such techniques in routine offshore investigations is considered.

Terrestrial glacial deposition is inevitably associated with an uncertain stress history. This is also true for glaciomarine deposition since both grounded ice-sheets and freeze-thaw effects in permafrost contribute to the loading pattern. A special feature of the stress history of offshore soils concerns the cyclic loading they receive from pressure fluctuations on the sea bed, which are produced by the passage of waves and which depend on sea depth and wave height (Wiegel, 1964). This aspect of the stress history is considered in Chapter 3.

In terms of recent (Holocene) history there are further differences between terrestrial and offshore soils. Offshore, subaerial weathering is absent, as too are the effects of seasonal fluctuations in groundwater levels and temperature. However, since currents operating at the sea bed can provide a more powerful transporting agent than wind acting on terrestrial soil, superficial deposits on the sea bed are mobile and their passage contributes to the sea bed's recent history.

The passage of sand waves also presents one of the potential problems which must be considered in siting offshore structures. Others include the development of pockmarks, scour and the stability of submarine slopes.

A unique feature of offshore soils is the presence of gas bubbles under high hydrostatic pressure. These influence the in situ behaviour of the soil and can have a dramatic effect when the soil is sampled. A particular feature of the present environment for soils in the Northern North Sea is the constantly low temperature ($5/6^{\circ}\text{C}$).

1.5 INVESTIGATION OF OFFSHORE SOILS

The first step in any foundation design process is the site investigation. Ashore such an investigation follows a well-established pattern, beginning with the collection of existing information on the site. Geological information forms an essential ingredient, providing a framework within which all observations must be fitted. Then follows reconnaissance of the site, both at ground surface level and, in many instances, from above with the use of aerial photography. Only after these operations does the subsurface investigation begin, making use of boring and pitting, and enabling in situ inspection and sampling to be carried out.

Offshore, existing information for a site is severely limited and geological mapping can only be on a regional scale. Direct reconnaissance is also restricted but geophysical methods, so much better suited to operating in water, provide a valuable replacement tool. Both in situ inspection and large scale sampling are, at present, impossible; macro-fabric and parameters defining en masse behaviour are difficult to assess. Conventional sampling, too, has its difficulties. Many of the mechanical problems associated with working from a drillship with its heaving motion have been reduced with compensators or by the use of drillstring anchors (Zuidberg and Windle, 1979).

There remains, however, the inevitable influence on samples of large total stress reductions, of gas exsolution and possible temperature changes when the samples are taken from deep water sites and brought to the laboratory. These influences are examined in Chapter 4. The value of samples taken from such depths for the determination of design parameters, especially stiffness is questioned. The potential for using reconstituted samples

as a means of determining the in situ behaviour of typical Northern North Sea soils is investigated in the last part of Chapter 4^{*1}.

The laboratory investigation of the mechanical behaviour of offshore soils must take cognisance of the special features of the loading referred to above. To extend the stress path approach (Lambe, 1967) to the prediction of foundation behaviour offshore, two developments are required, namely the ability to investigate the response of soils to cyclic loading and the ability to include principal stress rotation in the stress path.

In Chapter 5 a control system, based on a digital computer, and its associated instrumentation is described. This control system was developed initially to enable cyclic loading to be carried out in the hydraulic triaxial cell (Bishop and Wesley, 1975) on samples which had been subject to appropriate consolidation histories, including K_0 . It also provided rapid data acquisition capability for examining the influence of loading rate on soil behaviour.

An essential requirement for interpretation of soil behaviour is the measurement of pore water pressure. A discussion of pore pressure measurement in cyclic and transient loading tests is given in Chapter 6 and the use of a relatively new piezometer for such measurements is described.

Controlled rotation of principal stress directions is extremely difficult to achieve in elements of soil in the laboratory. The problem is reviewed in Chapter 7 and the design of a new hollow cylinder apparatus for this purpose is described; its ability to simulate the stress paths described in Chapter 2 is demonstrated.

A description of the apparatus developments (Chapters 5, 6 and 7) forms Part B of the Thesis.

^{*1} Chapters 2, 3 and 4 form Part A of the Thesis and provide the necessary background information.

1.6 ANALYSIS OF OFFSHORE FOUNDATIONS

In common with their on-land counterparts, the foundations to offshore structures must be designed both to remain stable and to deform within acceptable limits. In the design process, the models and methods of analysis must take account of the essential differences described above.

In particular, the models used to represent soil behaviour must be capable of allowing for changes in properties which might arise from both cyclic loading and rotation of principal stress directions. The models and methods of analysis must be capable of predicting the potential failure modes associated with the unusual loading patterns, e.g. the failure mode observed by Rowe and Craig (1979) to occur at shallow depth beneath a rocking base and the more deeply-seated failure mode observed by Martins (1980) to occur under combined vertical, horizontal and moment load applied under quasi-static conditions.

Analysis has to be extended to deal with the interaction of structure, soil and sea, and, where the frequency of the cyclic forces justifies it, the analysis must examine the possibility of dynamic amplification of loads. For such analysis, foundation stiffness and damping, and their changes with cyclic loading, are important.

1.7 THE RESEARCH OBJECTIVES

At the inception of this research, the behaviour of soils under cyclic loading, the influence of loading rate on soils typical of the Northern North Sea bed, and the effect on soil of principal stress rotation were not well understood. Improvement in the state of knowledge on these topics was a central aim of the research.

Based on a review in Chapter 3 of the soil types present in the study area, the Lower Cromer Till was selected as an accessible terrestrial till which was sufficiently representative for detailed study. Relying on the similarity in behaviour between intact and reconstituted samples of a similar till from Cowden, reported in

Chapter 4, reconstituted Lower Cromer Till has been used in investigations of undrained behaviour under monotonic, cyclic and transient loading. The results of these investigations are presented in Part C of the Thesis.

Chapter 8 describes the behaviour under monotonic loading. In an attempt to simulate different depositional modes, two methods were used to reconstitute the soil. The observations presented in Chapter 8 form the basis for examining the effects of shearing rate, described in Chapter 9 and the effects of cyclic loading described in Chapter 10.

In Chapter 11, an explanation for several features of the Lower Cromer Till's behaviour is sought in terms of the soil's microfabric which has been examined in detail.

Finally the implications of the research findings (Part C) are reviewed in the light of the background information (Part A). Particular emphasis is placed on:-

- (i) in situ properties: predictions are made which take account of depositional mode, environment and stress history, including that referred to as wave compaction;
- (ii) interpretation of in situ tests: the influence of the soil's behaviour, especially under different rates of shear, is considered;
- (iii) the effects of sampling and the interpretation and conduct of laboratory element tests.

Muir Wood (1979) regards the 'fundamental steps of obtaining reliable and representative data on the distribution, structure and properties of the relevant foundation materials' as representing 'the weak links in the chain of operations' in offshore foundation engineering. This research, begun in 1975, is now presented as a contribution towards strengthening that link.

A	<u>Structure</u>	(i)	Construction Sequence (Installation of prefabricated unit)
		(ii)	Scale
B	<u>Loading</u>	(i)	Rapid Application of Dead Load
		(ii)	High Ratio of Live to Dead Load
		(iii)	Cyclic Horizontal Forces and Moments
C	<u>Soil</u>	(i)	Depositional Mode and Environment
		(ii)	Stress History
		(iii)	Gas Content
D	<u>Ground Investigation</u>	(i)	Absence of Detailed Geological Information
		(ii)	Inability to Inspect in situ
		(iii)	Sampling through Deep Water
		(iv)	Laboratory Testing to Simulate Cyclic Loading and Principal Stress Rotation
E	<u>Analysis</u>	(i)	Prediction of Failure under Cyclic, Inclined and Eccentric Loading
		(ii)	Dynamic Interaction of Soil-Structure and Sea.

Table 1.1. Special aspects of offshore foundation engineering.

PART A

A BACKGROUND TO OFFSHORE STRUCTURES
IN THE NORTHERN NORTH SEA

CHAPTER 2

STRESS PATHS AND PRINCIPAL STRESS ROTATION ASSOCIATED WITH OFFSHORE LOADING

SYNOPSIS

Principal stress rotation is defined and a method for tracking it is proposed. Factors which determine the pattern of principal stress rotation (loading direction, constitutive law and kinematic restraints) are isolated by reviewing the complete stress paths for a number of typical field situations. Stress paths associated with foundations to gravity and piled platforms and with the passage of surface waves are examined; these serve to highlight the significant changes in principal stress direction and in the relative magnitude of the intermediate principal stress which occur.

2.1 INTRODUCTION

The nature of the environmental loading on offshore structures and the stress paths associated with their foundations were cited in Chapter 1 as two of the features which distinguished them from land-based structures. Of the environmental forces, those due to waves dominate; for a typical gravity structure in the Northern North Sea, the horizontal force resulting from interaction between the structure and waves during a 50 year storm amounts to approximately one third of total submerged weight. Both horizontal and vertical forces and overturning moments which reverse in direction are cyclically applied. Concurrently there are cyclic variations in the pressure distribution on the sea floor around the foundation.

Of major importance for both the way in which the structure and soil responds and the manner in which the loading can be simulated in the laboratory is the frequency of loading. A 50 year storm in the Northern North Sea has a wave period between 15 and 20 seconds. For less severe conditions, wave periods reduce to 5 seconds. Such periods are longer than those applying in most other cyclic loading situations for which there is experience of foundation behaviour, e.g. earthquakes, road and rail trafficking. However, these periods are longer than the natural periods of the current structure-foundation systems^{*1} so that large amplification of forces is avoided.

During a cycle, the point of action and direction of the wave force continuously changes. Since the cyclic forces are large compared with the static dead load, there is a corresponding continuous change in direction of the resultant force on the foundation. Within the foundation soil this leads to continuous rotation of the direction of principal stress increments acting on soil elements. Attention is drawn in this Chapter to this aspect of the stress paths associated with wave loading

*1 (Spidsoe et al, 1980, quote natural frequencies of 0.43 secs for translation and 0.63 secs for rotation of gravity structures in 150m of water.)

on offshore structures. Reference is also made to the changes in intermediate principal stress, σ_2 , which accompany wave loading.

As an introduction, rotation of principal stress and principal stress increment direction is defined by considering the changes in the state of stress on an element of soil. A method for tracking changes in principal stress direction and in the magnitude of σ_2 is proposed. Rotation of principal stress directions and changes in σ_2 in simple loading situations are then examined. The complexity of the loading is increased in stages up to that corresponding to the loading on an offshore gravity platform; in this way some of the factors which determine the pattern of principal stress rotation can be identified. The influence of the soil's constitutive law on principal stress rotation is considered and is shown to be of greatest importance when the problem involves kinematic restraints; such restraints determine the behaviour alongside piles.

The potential importance of these two features of the stress paths associated with offshore structures, i.e. principal stress rotation and changes in σ_2 , is considered in Chapter 7; it is shown to justify the development of a new testing apparatus for their simulation.

2.2 DEFINITION OF PRINCIPAL STRESS ROTATION

The general state of stress which can exist on a soil element is shown in Fig. 2.1(a); it comprises six independent stresses, three shear stresses and three normal stresses. This state of stress may be simplified to the state shown in Fig. 2.1(b) in which three principal stresses, σ_1 , σ_2 and σ_3 , act on the element and the directions of their orthogonal axes are defined by angles α , β and γ . For a soil, the state of stress is completely defined by σ_1 , σ_2 , σ_3 , α , β and γ , and by the pore pressure u ; α , β and γ are independent of u .

Changes in any or all of the stresses acting in Fig. 2.1(a) may bring about a change in the magnitude and/or direction of one or more of the principal stresses. These changes in stress can be represented by the stress increments shown in Fig. 2.1(c) which can be simplified

to the state shown in Fig. 2.1(d), in which three principal stress increments, $\Delta\sigma_1$, $\Delta\sigma_2$ and $\Delta\sigma_3$ act on the element with directions defined by α_i , β_i and γ_i . Changes in the direction of the principal stresses arise when the orientations of the principal stress increments which are superimposed, α_i , β_i , γ_i , differ from the initial orientations of the principal stresses, α , β , γ .

When the directions defined by α , β and γ are changed (Fig. 2.1(e)), rotations in principal stress directions are said to have occurred. In the same way, rotations of principal stress increment directions occur when α_i , β_i , γ_i are changed by the addition of further stress increments (Fig. 2.1(f)). For the purposes of presentation, the directions of principal stress or principal stress increments are considered to be positive when measured clockwise from the vertical.

Under conditions of plane strain, principal stress rotations are restricted to one plane, the intermediate principal stress, σ_2 , being fixed in direction as a normal to the plane of zero strain, see Fig. 2.2 (a). Similarly in axisymmetric situations, rotations occur only in radial planes, see Fig. 2.2(b). In general, however, rotations of principal stress directions may occur about three axes.

2.3 PRESENTATION OF STRESS PATHS WHICH INCLUDE PRINCIPAL STRESS ROTATION AND CHANGES IN INTERMEDIATE PRINCIPAL STRESS

The state of stress on an element is usually presented in terms of combinations of the principal stresses, σ_1 , σ_2 and σ_3 . Combinations of principal stresses which are invariants, i.e. their magnitudes are independent of the orientation of the spatial reference axes, are generally used. These may then be related in some form of failure criterion.

The sequence of stress changes experienced by a soil element determines its response. The use of a path in stress space to represent these sequential changes in stress state has been advocated by Lambe (1967). The combinations of principal stress commonly used

to depict stress paths are:-

- (i) $(\sigma_1 - \sigma_3)/2$ and $(\sigma_1' + \sigma_3')/2$
- (ii) $(\sigma_1 - \sigma_3)$ and $(\sigma_1' + \sigma_2' + \sigma_3')/3$

Points in these stress spaces represent only magnitudes of principal stresses. Paths drawn in them cannot convey the complete stress history of a soil element. Information on both principal stress directions and rotations and on the relative value of the intermediate principal stress is not recorded.

To remedy this in a simple manner, the stress path plot advocated by Lambe (1967) has been extended as shown in Fig.2.3 and this new plot is used for presentation purposes herein.

The plot of $(\sigma_1 - \sigma_3)/2$, q_{13} , against $(\sigma_1 + \sigma_3)/2$, p_{13} , is retained. Superimposed on this is a plot of $(\sigma_1 - \sigma_2)/2$, q_{12} , against $(\sigma_1 + \sigma_2)/2$, p_{12} . The method retains the link with Mohr's circle of stress so that by simple construction σ_1 , σ_2 and σ_3 can be defined and the stress path location can be related directly to failure conditions. Alongside, the orientation, α , of the major principal stress to the vertical, is plotted against q_{13} . In the stress paths to be discussed in this Chapter and in the apparatus to be described in Chapter 7, principal stress rotation is restricted to one plane only, i.e. the one containing σ_1 and σ_3 , so that definition of changes in σ_1 , σ_2 , σ_3 and α is sufficient to describe the stress path, provided, of course, that the initial orientation of the stresses is known. For the more general case, when changes occur in the orientation of the three axes of principal stress, α , β , and γ , the plot can easily be enlarged, with β and γ being drawn against q_{13} .

When representing triaxial data in such a plot it should be noted that:-

- (i) for triaxial compression ($\sigma_2 = \sigma_3$), lines q_{13} and q_{12} coincide, and
- (ii) for triaxial extension ($\sigma_2 = \sigma_1$), q_{12} coincides with the horizontal axis of the plot.

2.4 PRINCIPAL STRESS ROTATION AND CHANGES IN INTERMEDIATE PRINCIPAL STRESS ASSOCIATED WITH A VERTICALLY LOADED FLEXIBLE CIRCULAR FOOTING ON AN ISOTROPIC LINEAR ELASTIC FOUNDATION

The stress changes produced at different points beneath a uniform vertically loaded circular area on a semi-infinite linear elastic continuum can be calculated from tabulated solutions (refer Poulos and Davis, 1974). Combining the stress changes, the magnitude and direction of the principal stress increments can be determined and the complete stress path defined. On the axis of symmetry beneath the footing, principal stress increments are vertical and horizontal, refer to element A in Fig. 2.4, and $\Delta\sigma_2$ is equal to $\Delta\sigma_3$. When these principal stress increments are superimposed on an initial geostatic stress distribution, principal stress directions are unchanged (except for cases with K_0 greater than unity when interchange of principal stress directions can occur) and the original stress path plot of Lambe (1967), q_{13} versus p_{13} , is sufficient to describe the stress changes.

For any element off the axis of symmetry, principal stress increments are inclined to the vertical and horizontal, refer to element B in Fig. 2.4, and $\Delta\sigma_2$ is no longer equal to $\Delta\sigma_3$. The direction of the principal stress increment will be determined, in part, by the direction of the shear stress increment, see Fig. 2.4(b). The direction of the shear stress increment will depend, in turn, on the location of the element and on the contact conditions; for a frictionless footing or for a rough footing on an incompressible soil ($\nu = 0.5$), the shear stress on element B will be in the direction shown on the left hand side of Fig. 2.4(b); for a rough footing on a compressible soil, the shear stress direction may be reversed.

In Fig. 2.5 contours have been drawn for the orientation of principal stress increments beneath the footing, for the case of Poisson's ratio equal to 0.45 and of no adhesion over the loaded area. To enable the varying orientation of the principal stress increments to be related to the level of shear stress increment, and the increment of intermediate principal stress, $\Delta\sigma_2$, contours have also been plotted for:-

(i) $(\Delta\sigma_1 - \Delta\sigma_3)/2$, normalised by the surface pressure, p ,
(Fig. 2.6) and

(ii) $\frac{(\Delta\sigma_2 - \Delta\sigma_3)}{(\Delta\sigma_1 - \Delta\sigma_3)}$ (Fig. 2.7)

These plots are used subsequently in a comparison with the stress changes taking place beneath a gravity platform under wave loading.

In Fig. 2.8(a) the complete stress paths are drawn for three elements off the axis of symmetry of the footing; the paths, normalised in terms of the maximum applied surface pressure, p_m , trace the changes in stress state which accompany a build up in vertical pressure, p , on a footing of radius 50m.

The paths have been drawn for the case of an initial isotropic stress ($K_0 = 1$) and for this particular case the directions of the resultant principal stresses and of the principal stress increments coincide.

When the stress increments are superimposed on any other initial stress distribution ($K_0 \neq 1$), the changes in the direction of principal stresses which occur can be seen for the same three elements in Fig. 2.8(b). In Fig. 2.8(b) changes in orientation of the major principal stress, relative to its initial orientation (clockwise rotations considered positive) are plotted against increasing vertical pressure for a range of K_0 values; also plotted are the corresponding changes in deviator stress, $(\sigma_1 - \sigma_3)$.

Figures 2.5 to 2.8 show that for this particular problem:-

- (i) the orientation of the principal stress increments is independent of the magnitude of the applied loading and depends only on the location of the element being considered;
- (ii) a wide range of deviator stress increments is associated with each orientation of the principal stress increments;

- (iii) changes in the parameter $(\Delta\sigma_2 - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$ range between 0 and 0.2;
- (iv) continuous rotations of principal stress directions take place for cases other than K_0 of 1, the magnitude of the rotation varying with applied loading, the initial state of stress and location of the element. Step rotation of principal stress directions occurs only for the case of $K_0 = 1$;
- (v) rotations of principal stress directions occur at a relatively early stage of the loading and may accompany increasing, reducing, or relatively constant deviator stress, depending on K_0 and the location of the element.

2.5 PRINCIPAL STRESS ROTATION ASSOCIATED WITH A VERTICALLY AND HORIZONTALLY LOADED FLEXIBLE CIRCULAR FOOTING ON AN ISOTROPIC LINEAR ELASTIC FOUNDATION

When a horizontal load is superimposed on the vertically loaded foundation the situation is no longer axisymmetric. For the three elements being considered and for the case of K_0 of 1, the complete stress paths take the form shown in Fig. 2.9(a) when an increasing uniform shear stress is added to an existing vertical stress on the footing. The orientation of principal stress increments is no longer fixed; they rotate, together with the resultant applied load, at all locations by amounts which reduce with increasing shear stress. Further rotations of principal stress direction occur for cases of K_0 other than one as illustrated in Fig. 2.9(b). For this particular example principal stress rotations occur at relatively high shear stress levels and accompany further increases in shear stress (see Fig. 2.9(b)) although this is a function of the synchronization of the applied loads.

2.6 STRESS PATHS, INCLUDING PRINCIPAL STRESS ROTATION, FOR AN OFFSHORE GRAVITY STRUCTURE ON AN ISOTROPIC LINEAR ELASTIC FOUNDATION, SUBJECT TO STORM WAVE LOADING.

In many situations the addition of a horizontal component of force is cyclic in nature. Interaction between waves and offshore structures

is one such case. Retaining the same formulation of an isotropic elastic foundation, and taking a Poisson's Ratio of 0.45, the complete stress paths have been calculated for a typical offshore gravity platform in the Northern North Sea.

In reality wave loading is omnidirectional so that any element in the foundation has not only a complex stress path but a complex stress history. For simplicity, however, unidirectional wave loading has been considered. The sequence of wave positions assumed is shown in the upper part of Fig. 2.10.

To calculate the stress paths associated with this loading sequence, the following approach was adopted:-

- (i) Information was obtained from the designers of offshore structures and from published papers (notably, Eide, 1974; Lauritzen and Schjetne, 1976) on the maximum anticipated forces which would act on gravity oil production platforms in the Northern North Sea^{*1}. Average values for these forces and for submerged weight, shown in Fig. 2.11, were selected for subsequent analyses. The variation in these forces and moments and the resulting pattern of displacements with wave position is shown in Fig. 2.10. There is probably a phase difference between the horizontal thrust and net overturning moment which has been ignored; this arises because there are two contributions to the overturning moment, viz:

- (a) the horizontal thrust, and
- (b) the difference in vertical pressure across the top of the base.

This means that the critical position of the wave may not coincide with the crest of the wave at the centre of the structure.

- (ii) The base of the structure was assumed to be circular, with a radius, r_0 , of 50m. To avoid uncertainties over contact

^{*1} Table 2.1 summarises the information that was obtained.

conditions between the base and the sea bed, average contact stresses were determined for the applied forces; these are shown in Fig. 2.12 and have been added to Figs 2.10 and 2.11.

- (iii) Stress changes in the continuum for this combination of applied surface stresses were calculated using solutions published by Gerrard and Harrison (1970). Results for changes in vertical stress, $\Delta\sigma_z$, radial stress, $\Delta\sigma_r$, circumferential stress, $\Delta\sigma_\theta$, shear stresses $\Delta\tau_{rz}$ are plotted against depth in Fig. 2.13(a) and (b) for wave positions II and IV. A plane parallel to the direction of wave travel and through the centre of the platform has been considered; out of this plane, rotations of principal stresses occur in more than one direction. Fig. 2.13 illustrates how the largest fluctuations in stress occur at the base perimeter and how, at shallow depths, reversal in shear stress, $\Delta\tau_{rz}$, only occurs near the perimeter.
- (iv) The pressures which were assumed to act outside the area of the structure's base were calculated for a wave height of 30m, a wavelength of 400m and a water depth of 150m. For these conditions Wiegel (1964) gives a pressure below the wave crest at the sea bed of $3t/m^2$, i.e. 0.15pv, refer Figs 2.10 and 2.11. This surface loading due to the waves was represented by a two-dimensional symmetrical triangular pressure distribution for which stresses within the bed can be calculated using a solution given by Jurgenson (1934). To achieve the net pressure distribution around the base, appropriate balancing stresses over the area of the base were superimposed on the two dimensional stress distribution.

The combined stress changes are presented for wave position II (and hence for wave position IV) as contours of:-

- (i) principal stress increment direction (Fig. 2.14),
- (ii) $(\Delta\sigma_1 - \Delta\sigma_3)/2$ normalised by the vertical surface pressure, p_v (Fig. 2.15),

$$(iii) \quad \frac{(\Delta\sigma_2 - \Delta\sigma_3)}{(\Delta\sigma_1 - \Delta\sigma_3)} \quad (\text{Fig. 2.16}).$$

For a depth of $0.25 r_0$ the sequence of stress changes is summarised in Fig. 2.17.

The complete stress paths for three elements beneath the platform are shown in Fig. 2.18 for the case of K_0 of 1. The principal stress rotations and changes in deviator stress which accompany wave loading are shown for a range of K_0 values in Fig. 2.19.

Comparing the concentrically loaded circular footing and the circular gravity platform base, it is apparent that:-

- (i) The same large range of principal stress increment directions are encountered (Figs 2.5 and 2.14) but while these directions are fixed at any element beneath the concentrically loaded foundation (Fig. 2.8), they continuously change beneath the gravity platform base (Figs 2.14 and 2.18). Rotations of principal stress increments of up to 80° take place in the offshore foundation; they have their largest values near the surface and at the perimeter of the base (Fig. 2.14).
- (ii) The additional forces at the platform base and the surrounding sea bed pressures result in a different distribution of shear stress $(\Delta\sigma_1 - \Delta\sigma_3)/2$ (Figs 2.6 and 2.15). When normalised by the vertical pressure (p for the concentrically loaded foundation and p_v for the gravity platform), the contours beneath the platform take on higher values. These values reach a maximum at the perimeter of the base and continuously change as the directions of the principal stress increments change.
- (iii) The distribution of the parameter $(\Delta\sigma_2 - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$ is totally different beneath the gravity platform (Figs 2.7 and 2.16) and much higher values apply, particularly at the unloaded edge.

- (iv) The resulting stress paths (Figs 2.8(a) and 2.18) show little similarity for the two cases.
- (v) Wave loading on the offshore gravity structure results in a much more complex pattern of principal stress changes in the foundation (Figs 2.8(b) and 2.19). Beneath the platform, the largest rotations of principal stress directions occur near the surface as the wave moves from positions III to IV to V; during this phase of the loading, deviator stresses are decreasing and then increasing well below their maximum value. For both foundations, initial K_0 values of 1.25 are associated with the largest rotations of principal stress directions.

It must be recognised that these stress calculations and stress paths are illustrative only. In reality, the pattern of stress changes, including changes in principal stress direction, will be affected by:-

- (i) initial inhomogeneities in the foundation,
- (ii) inhomogeneities which develop as cycling and consolidation proceeds, particularly near the surface and at the edges of the base,
- (iii) non-uniform contact stresses, which will be determined by topography, base shape and rigidity, and
- (iv) the constitutive laws of the soils present.

2.7 THE INFLUENCE OF CONSTITUTIVE LAW ON PREDICTIONS OF PRINCIPAL STRESS ROTATION

The stress changes calculated so far have been for surface foundations on an isotropic elastic continuum. It is well known that

the soil's constitutive law will affect the distribution and magnitude of principal stress changes; it is necessary to investigate how the conclusions reached above on principal stress rotation are affected by the constitutive law. This investigation has been made using finite element techniques. Since only the case of a surface foundation on a soil of finite depth can be examined, it is necessary first to consider what happens in an elastic bed of finite depth.

(a) Linear elastic foundation of finite depth.

The effect of a rigid layer beneath a uniformly loaded circular area on a linear elastic foundation is to change the orientation of the principal stress increments but this orientation remains constant during loading; this can be inferred from stress changes determined by Milovic (1970). A similar change in the fixed value of the orientation is caused by changes in rigidity of the base. The stress paths for elements (K_0 of 1) beneath a circular rigid footing on a linear elastic foundation of finite depth are shown in Fig. 2.20(a).

(b) Non-linear elastic foundation of finite depth.

For a non-linear elastic foundation, the combined influence of an underlying rigid layer and the constitutive law produces small rotations of principal stress increments, even for the case of an increasing load applied in a fixed direction. This is illustrated in Fig. 2.20(b).

The rotation of principal stress increments, particularly at large radius, is evident and can be compared with the corresponding fixed values for an identical foundation on a linear elastic material.

In his investigation of the stress paths beneath offshore gravity platforms on a non-linear elastic foundation, Davachi (1978) showed that, compared to linear elasticity, non-linearity decreased the rotation of principal stresses beneath the base but increased it outside.

- (c) Strain-hardening plastic foundation
of finite depth.

Gens (1982, pers. comm.) has analysed the case of a circular base on a strain hardening plastic foundation which has a similar shape stress-strain curve as that assumed for the non-linear elastic foundation. The calculated stress paths are presented in Fig. 2.20(c). There is notable similarity between the plastic and non-linear elastic cases with rotation of principal stress increments occurring as surface pressure increases.

It is evident from Fig. 2.20 that the effect of the constitutive law on predictions of principal stress rotation is relatively small for this particular problem^{*1}. Where kinematic restraints are high, however, a different conclusion is reached, as discussed in the next section.

2.8 PRINCIPAL STRESS ROTATION ASSOCIATED WITH A VERTICALLY LOADED RIGID PILE

To date, only the stress paths associated with offshore gravity structures have been considered in detail. The piles supporting offshore jacket structures are cyclically loaded so that rotation of principal stress directions is likely to be a characteristic feature of the stress paths for soil elements alongside the pile.

Vaughan (1980, pers. comm.) has pointed out that the pile provides a kinematic boundary displacement constraint that, together with the soil's constitutive law, has an important influence on the manner in which the soil responds to monotonic loading and on the form of principal stress rotation. This can be demonstrated by considering the state of strain in an element adjacent to a pile, Fig. 2.21. The strain increments represented by point A in the Mohr Circle correspond to the direction of the pile shaft; the extension in this direction must be zero, so that the pole of the Mohr Circle can be constructed. AB then represents the direction of the major principal strain increment.

*1 The effects of anisotropy have not been examined.

For an isotropic elastic soil, i.e. one in which principal stress and strain increments coincide, AB is the direction of the major principal stress increment and there is no normal stress increment in the direction of the pile shaft. For a plastic material, AB is the direction of the major principal stress, assuming that the elastic strains are negligible.

Since events close to the pile resemble those of simple shear (Randolph and Wroth, 1981) it follows that the inclination of AB is 45° when:

- (a) the material is elastic,
- (b) the material is plastic and the dilation is close to zero, as at failure in the Critical State Model.

Lopes (1979) has confirmed this for the case of a rigid pile in elastic soil, his finite element analyses predicting the principal stress increment directions to be fixed at 45° . Potts and Martins (1982) analysed a rigid pile using a Modified Cam Clay Model. The rotations of principal stress directions predicted by them are shown in Fig. 2.21(b) where it can be seen that the direction approaches 45° , as anticipated, and a large proportion of the rotation occurs at an early stage of the loading.

If it is assumed that during cyclic loading of an offshore pile the soil behaves elastically, there will be step rotations of principal stress increments if and when the direction of the shear stress at the pile shaft reverses. Changes in the direction of the principal stress will depend on the changes which take place in the in situ stress state.

2.9 PRINCIPAL STRESS ROTATION ASSOCIATED WITH THE PASSAGE OF SURFACE WAVES

Reference was made earlier in the Chapter to the pressures on the sea bed which accompany the passage of a surface wave. These pressures, acting around the base of an offshore gravity structure, were included in the calculation of stress changes for various elements in the foundation to such a structure.

The importance of these pressure fluctuations on the sea bed was recognised by Henkel (1970) when considering the undrained stability of recent deposits in the Mississippi Delta. The stress changes in the soil accompanying the pressure fluctuations represent an important component of the stress history of the sea bed soils; this is discussed in Chapter 3. The aim here is to consider the form of these stress changes.

The sea bed pressures were assumed to be in phase with the wave and were calculated assuming linear wave theory and a rigid sea bed. In the theory given by Wiegel (1964), in which these assumptions are made, it is shown that the magnitude of the pressure change at the sea bed, Δp , is given by:-

$$\Delta p = \frac{H \rho g}{2} \cdot \frac{\cos 2\pi(\frac{x}{L} - \frac{t}{T})}{\cosh 2\pi d/L}$$

where H is the wave height, measured vertically from trough to crest, (wave amplitude, $a = H/2$)

L is the wave length,

T is the wave period,

d is the depth of water, and

x is the co-ordinate in the direction of wave propagation.

Substituting K , the pressure response factor, for

$$\cosh^{-1} (2\pi d/L)$$

$$\text{then } \Delta p_{\max} = K \cdot \frac{H \rho g}{2}$$

The variation of K with depth d , expressed as the relative depth d/L is shown in Fig. 2.22. $\Delta p_{\max}$ is a function, then, of wave height, H , wave amplitude, L , and sea depth, d . For a sea depth greater than half the wave length, the sea bed pressures become insignificant.

For a particular sea depth, both wave height, H , wave period, T , and wave length, L , will vary, depending on such conditions as wind speed, fetch, storm duration, etc. Hence the maximum pressure change, $\Delta p_{\max}$, operating on the sea bed will also vary.

The sinusoidal variation in pressure along the sea bed, which is positive under the wave crest and negative under the trough as shown in Fig. 2.23, produces stress changes in elements below the bed which vary cyclically as the wave travels along; the situation is akin to that of a rolling wheel load.

The stress paths for soil elements below the sea bed associated with the fluctuations of sea bed pressure do not appear to have been examined in detail. In an approximate analysis, again based on linear isotropic elasticity¹, these stress changes have been determined for the case of an impermeable sea bed. The steps taken were:-

- (i) The pressure wave was simulated by a string of triangularly loaded flexible strips, of base width $2b$ and maximum pressure, $\Delta p_{\max}$, as shown in Fig. 2.23 ($2b \leq L/2$).
- (ii) The resulting stress increments ($\Delta \sigma_z$, $\Delta \sigma_x$, $\Delta \tau_{xz}$) were calculated by superimposing solutions given by Jurgenson (1934) for elements at different locations beneath a triangular strip.
- (iii) The corresponding principal stress increments and their directions were then evaluated.

¹ In view of the lack of kinematic restraints in this situation, this is a reasonable approach.

- (iv) Changes in the magnitude and orientation of the principal stress increments were traced for different positions of the wave train, expressed as a phase angle and defined in Fig. 2.23.

The resulting sequence of changes in principal stress increments and their direction are shown for four depths (z/b of 0.25, 0.5, 0.75 and 1.5) in Fig. 2.24.

It is evident that

- (i) The situation represents an extreme case of rotation of principal stress increments; continuous cyclic rotations between 0° and $\pm 90^\circ$ occurring.
- (ii) The value of the principal stress difference, $(\Delta\sigma_1 - \Delta\sigma_3)$, varies during a cycle reaching the maximum value, $(\Delta\sigma_1 - \Delta\sigma_3)_{\max}$, at a phase angle which depends on depth.
- (iii) The value of $(\Delta\sigma_1 - \Delta\sigma_3)_{\max}$ varies with depth in the manner shown in Fig. 2.25, where $\left[\frac{\Delta\sigma_1 - \Delta\sigma_3}{2\Delta p_{\max}} \right]_{\max}$ is plotted against z/b . A maximum value of $(\Delta\sigma_1 - \Delta\sigma_3)/2$ of $0.33\Delta p_{\max}$ is indicated at a depth of z/b of 0.6 (which is approximately $L/2\pi$).

It is gratifying to note that since carrying out this approximate analysis, Madsen (1978) has published a rigorous analysis of wave-induced pore pressures and effective stresses in a porous bed. For the case of an impermeable bed he predicts a maximum value for $(\Delta\sigma_1 - \Delta\sigma_3)/2$ of $0.368\Delta p_{\max}$ at a depth of $L/2\pi$.

2.10 SUMMARY

Stress paths commonly involve variation in both the magnitude and direction of all three principal stresses acting on a soil element. Continuous rotation of principal stress directions is a general feature of stress paths associated with engineering works. Continuous rotation of principal stress increments only occurs, however, when the direction of the applied loading continuously changes or when kinematic restraints and inelastic behaviour are involved. Stress changes in the sea bed caused by the passage of a surface wave represent an extreme form for rotation of principal stress increments. Stress paths associated with offshore structures are distinguished both by the continuous rotation of principal stress increments which takes place and by the cyclic nature of these rotations; kinematic restraints alongside a pile shaft and immediately beneath the rigid base of a gravity foundation will have a strong influence on the prediction of principal stress rotations. Experience with stress paths involving continuous cyclic rotation of principal stress increments is limited.

Platform details:-

Submerged weight	$150 - 220 \times 10^3 \text{ t}$
Base Area	$8000 - 12000 \text{ m}^2$
(Equivalent diameter	$100 - 125 \text{ m}$)
Natural frequency	$2 - 3 \text{ s}$

Wave details:-

Maximum wave height, H_{max}	30 m
Wave length	$450 - 600 \text{ m}$
Wave period	$14 - 20 \text{ s}$

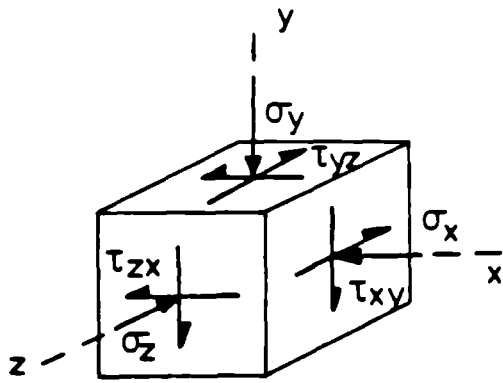
Wave induced forces and moments:-

Maximum horizontal force	$33 - 75 \times 10^3 \text{ t}$
Fluctuating vertical force	$20 - 40 \times 10^3 \text{ t}$
Maximum moment	$1 - 2.25 \times 10^6 \text{ t-m}$

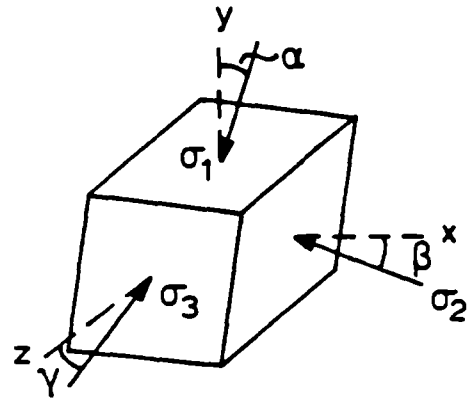
Average contact stresses:-

Maximum vertical stress	$20 - 30 \text{ t/m}^2$
Minimum vertical stress	$0 - 7 \text{ t/m}^2$
Maximum horizontal shear stress	$3 - 8 \text{ t/m}^2$

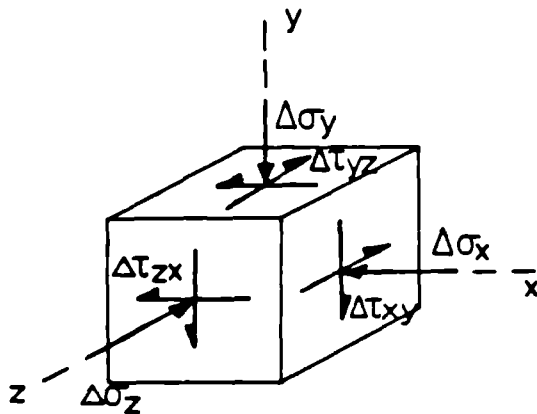
Table 2.1 Range of design details for six gravity platforms proposed for 150m water depth in Northern North Sea.



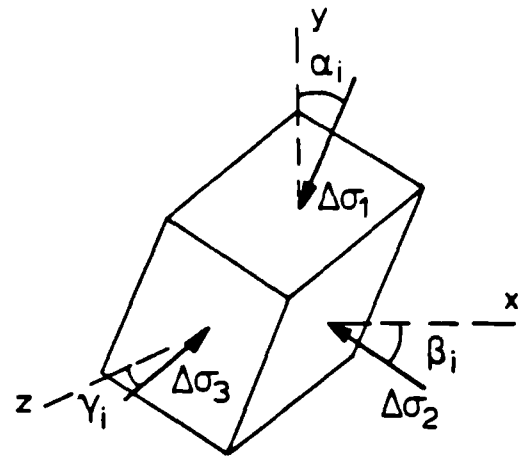
(a) Initial stress state



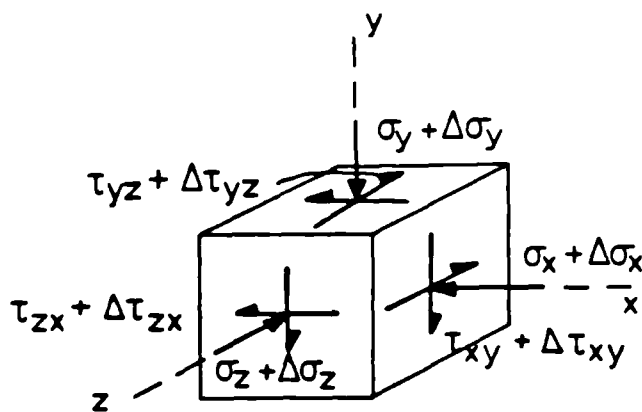
(b) Initial stress state specified by principal stress



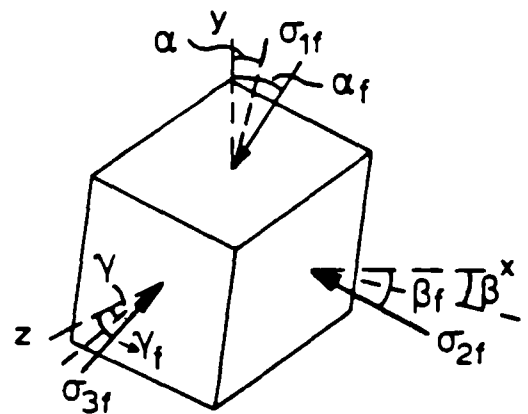
(c) Stress increment state



(d) Stress increment state specified by principal stress increments

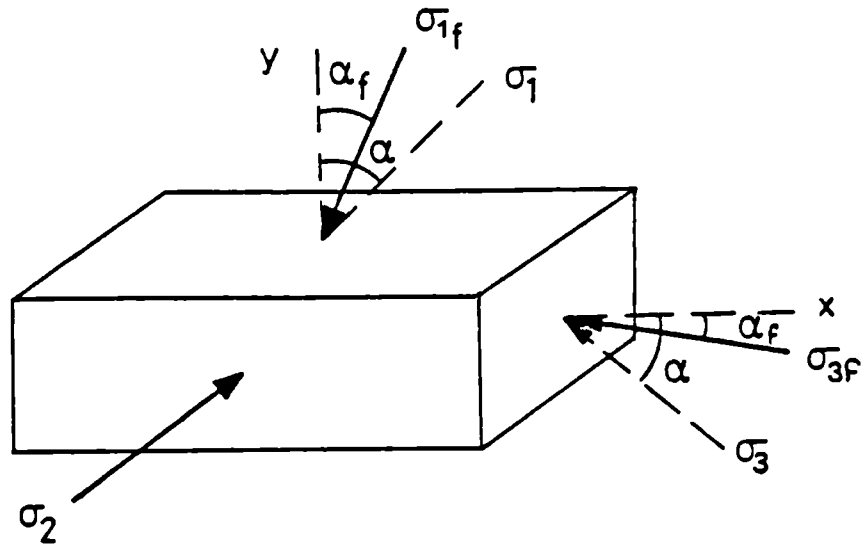


(e) Final stress state

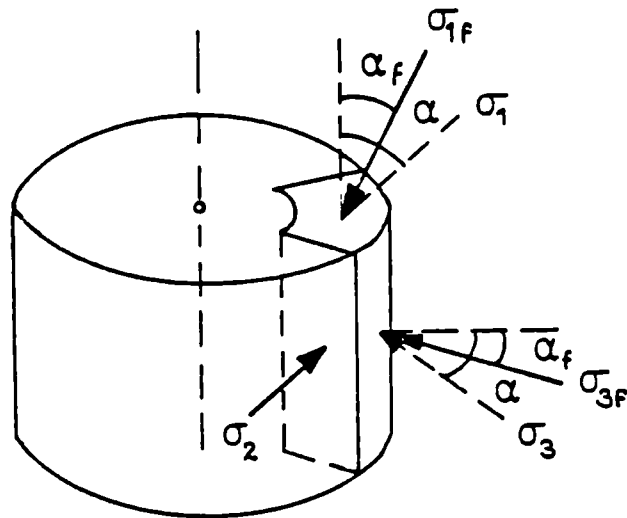


(f) Final stress state showing rotation of principal stress directions

Fig. 2.1 Definitions of principal stress rotation.



(a) Principal stress rotation under conditions of plane strain



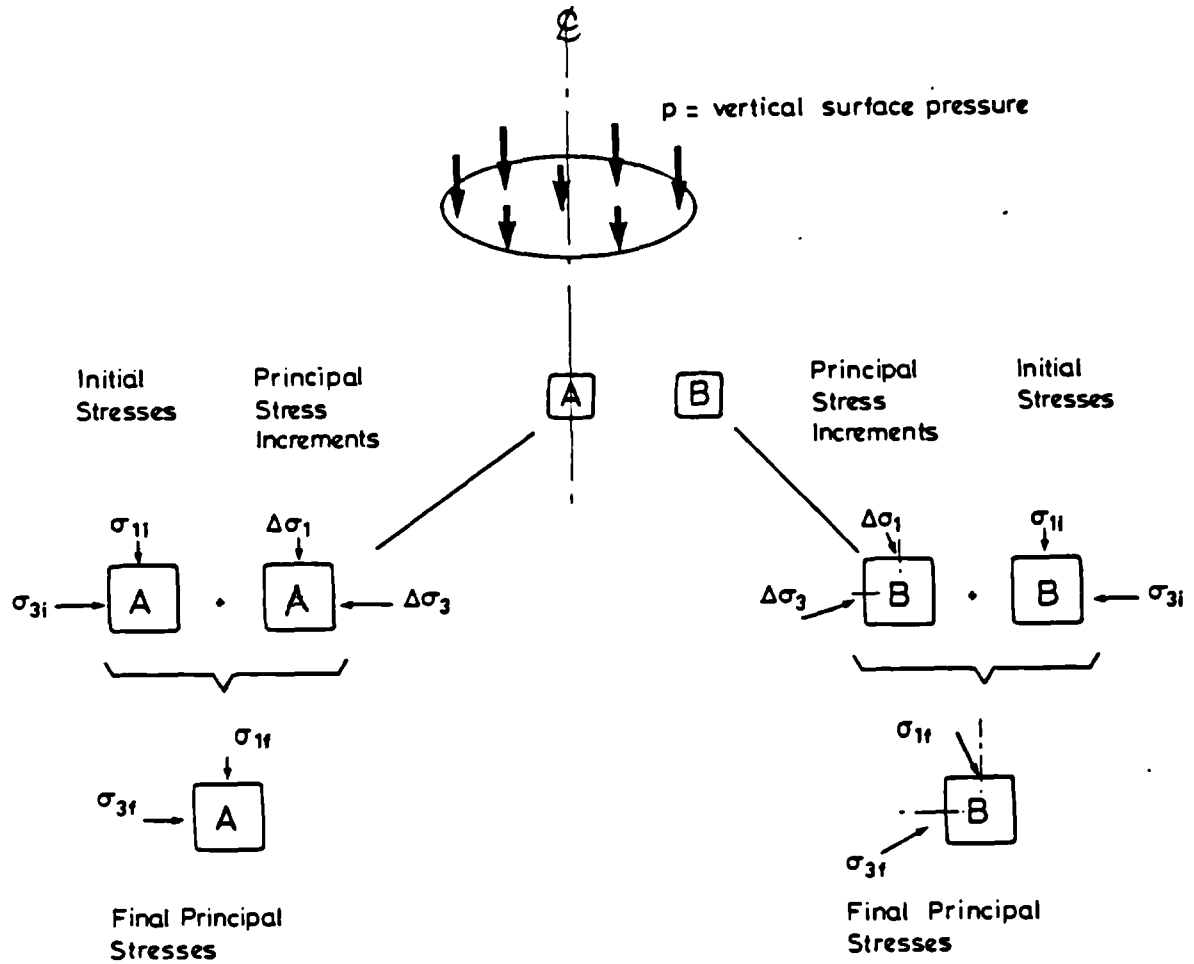
(b) Principal stress rotation under conditions of axial symmetry

Fig. 2.2 Principal stress rotations restricted to one plane.

- (i) construct lines at 45° from A to define σ_1 and σ_3 ,
- (ii) where AB intersects q_{12} , construct 45° to define σ_2 ,
- (iii) project horizontal line from A to A' to define α_A .

α_A and σ_1 may be used to define position of pole.

Fig. 2.3 An extended stress path plot to track changes in intermediate principal stress, σ_2 , and orientation of the major principal stress to the vertical, α .

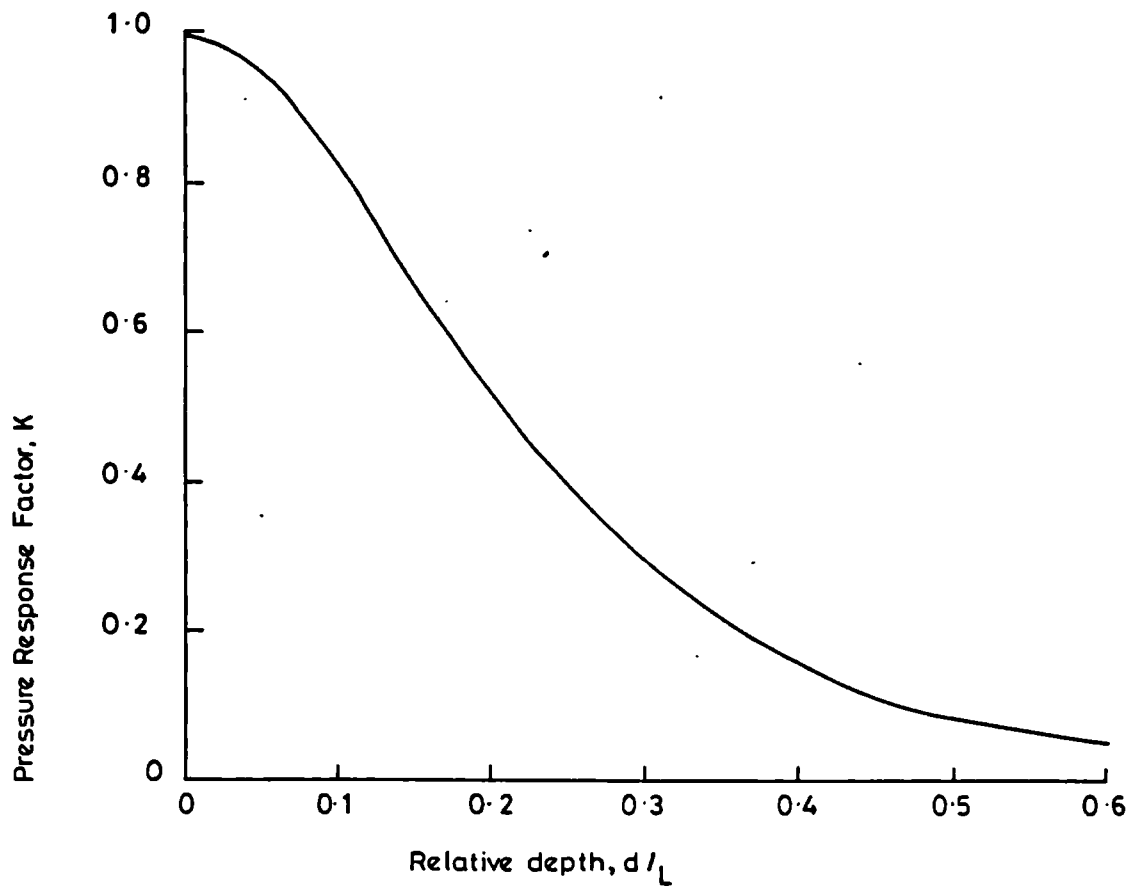


(a) Principal stress directions on and off the axis of symmetry.



(b) Influence of shear stress direction on rotation of principal stresses.

Fig. 2.4 Principal stress rotation beneath a vertically loaded circular foundation



$$\text{Sea bed pressures, } \Delta p_{\max} = K \cdot \frac{H \rho g}{2}$$

where H is wave height
 d is depth of water
L is wave length

Fig. 2.22 Variation of the pressure response factor, K, with depth. (After Wiegel, 1964)

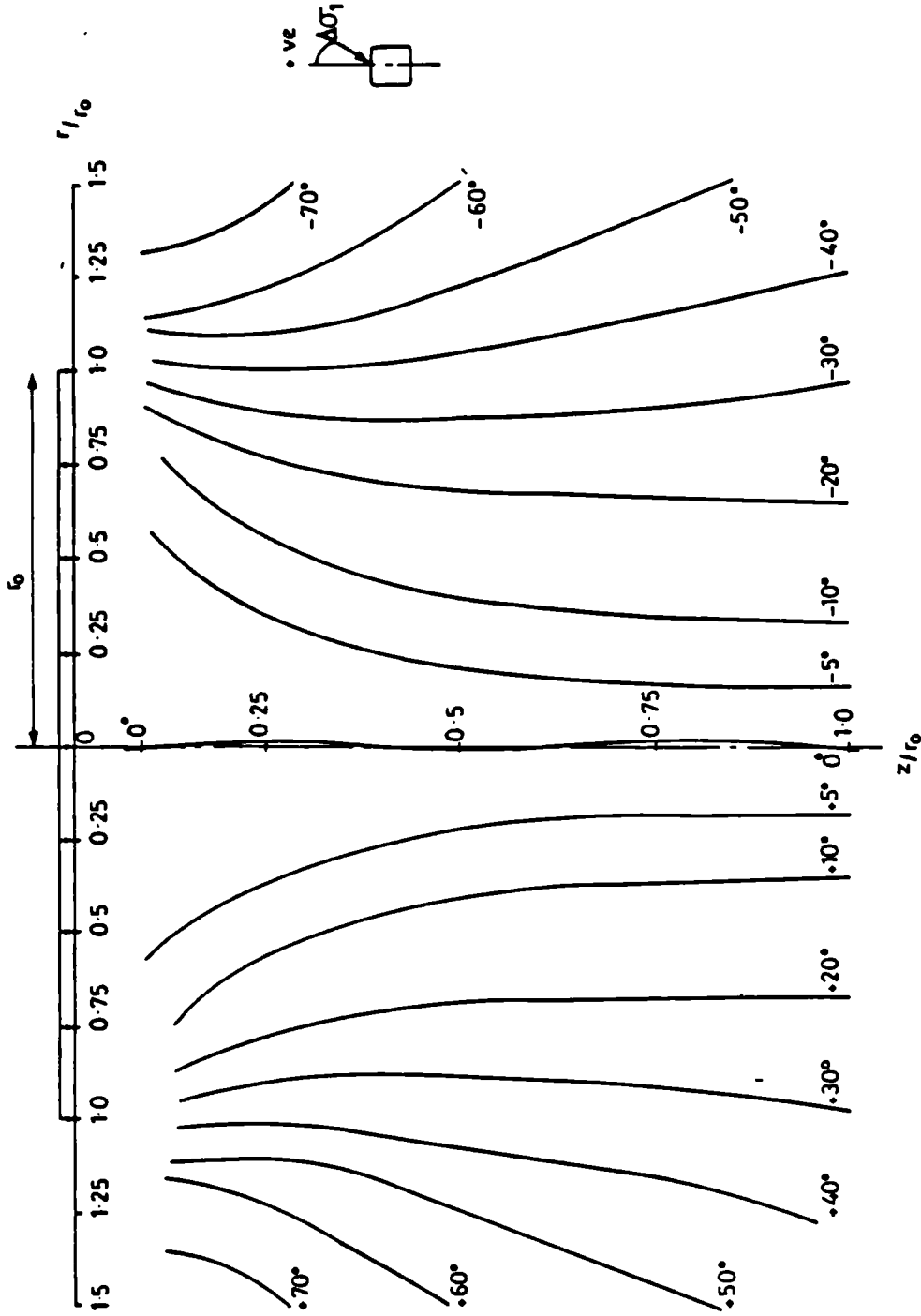


Fig. 2.5 Contours of principal stress increment direction beneath vertically loaded circular footing on isotropic linear elastic foundation ($\nu = 0.45$).

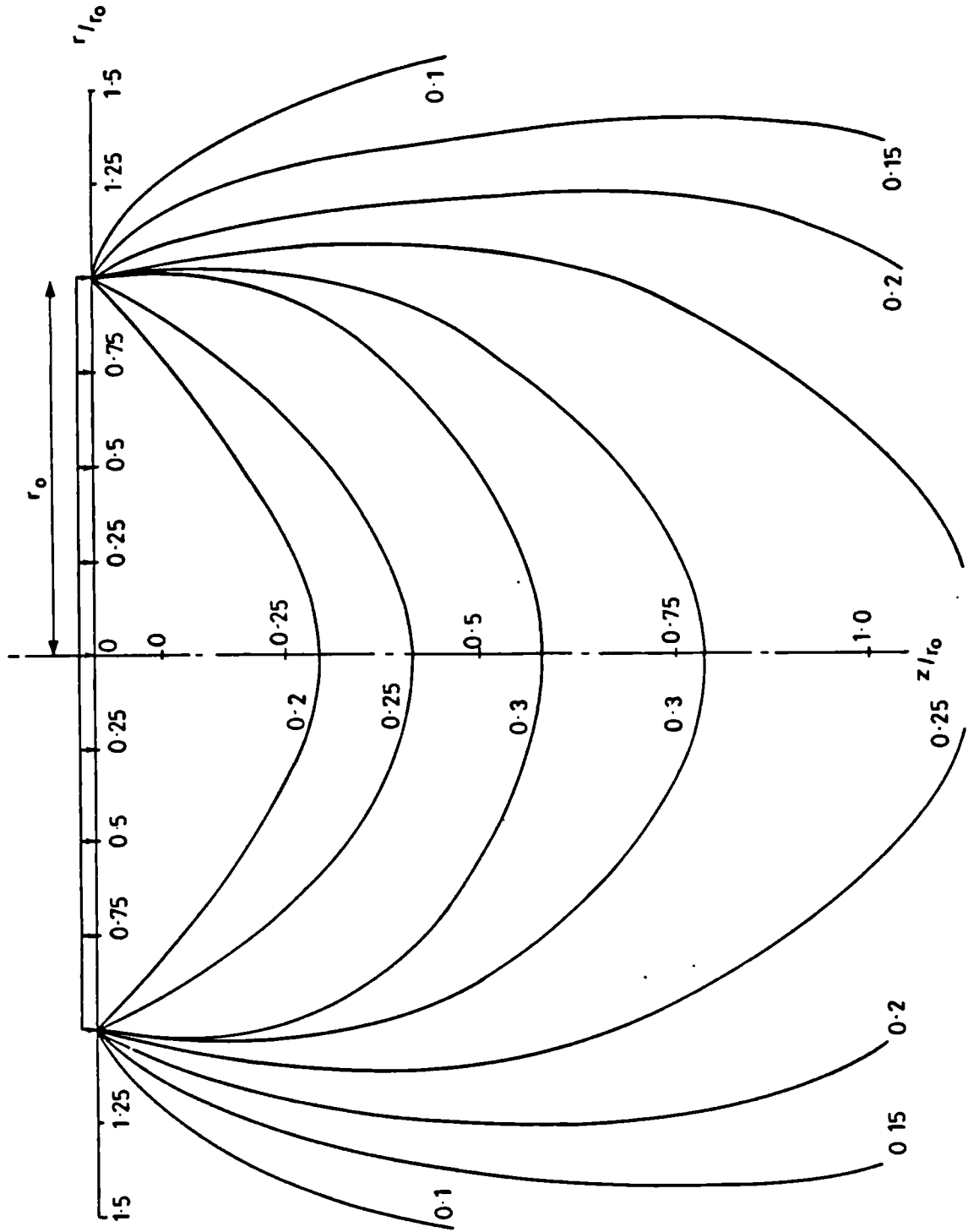


Fig. 2.6 Contours of $(\Delta\sigma_1 - \Delta\sigma_3)/2p$ beneath vertically loaded circular footing on isotropic linear elastic foundation ($\nu = 0.45$).

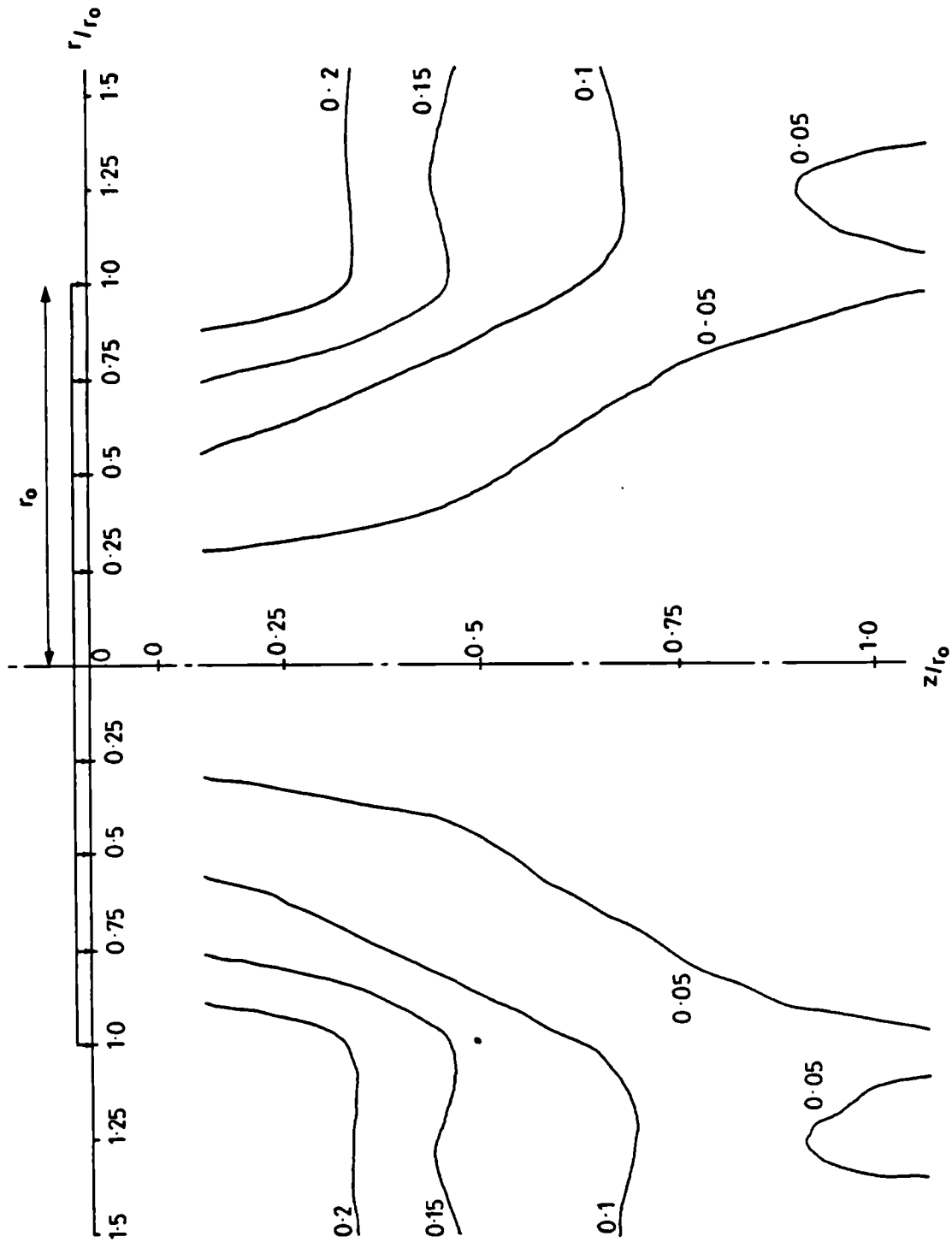
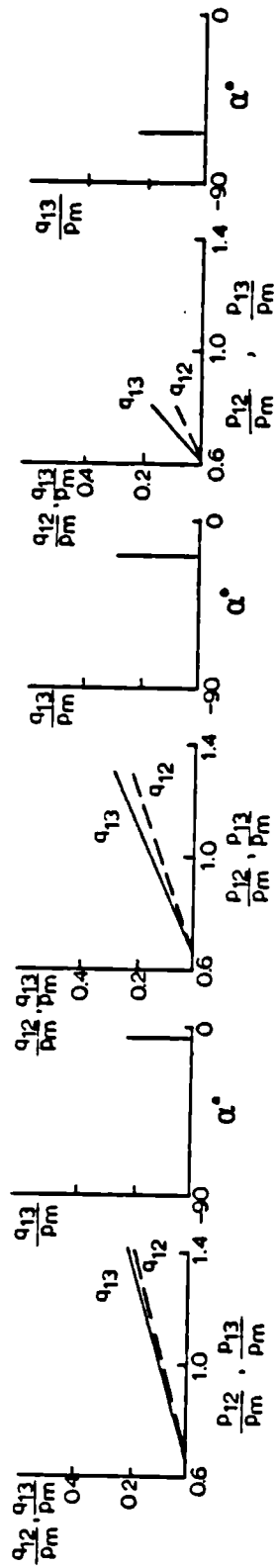
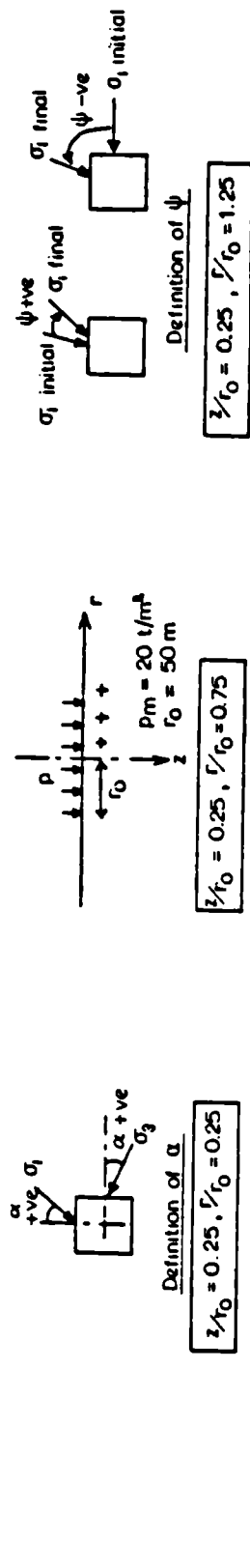
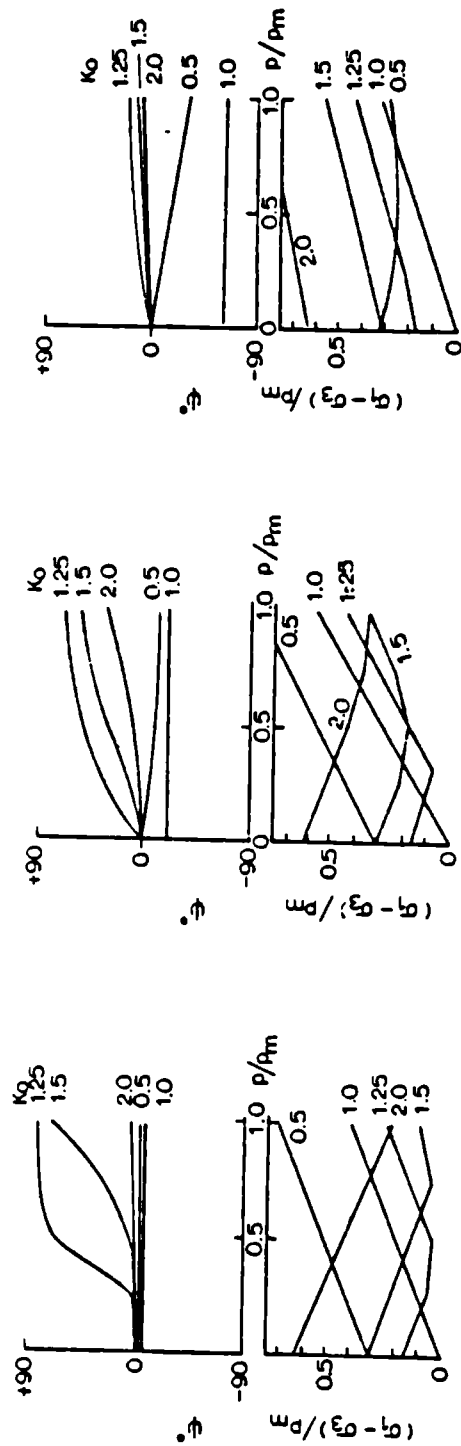


Fig. 2.7 Contours of $(\Delta\sigma_2 - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$ beneath vertically loaded circular footing on isotropic linear elastic foundation ($\nu = 0.45$).



(a) Stress paths for three elements off the axis of symmetry ($K_0 = 1$)



(b) Changes in principal stress direction and $(\sigma_1 - \sigma_3)$ for various K_0

Fig. 2.8 Complete stress paths and principal stress rotations for elements beneath a flexible circular footing under uniform vertical pressure on isotropic linear elastic foundation ($\nu = 0.45$).

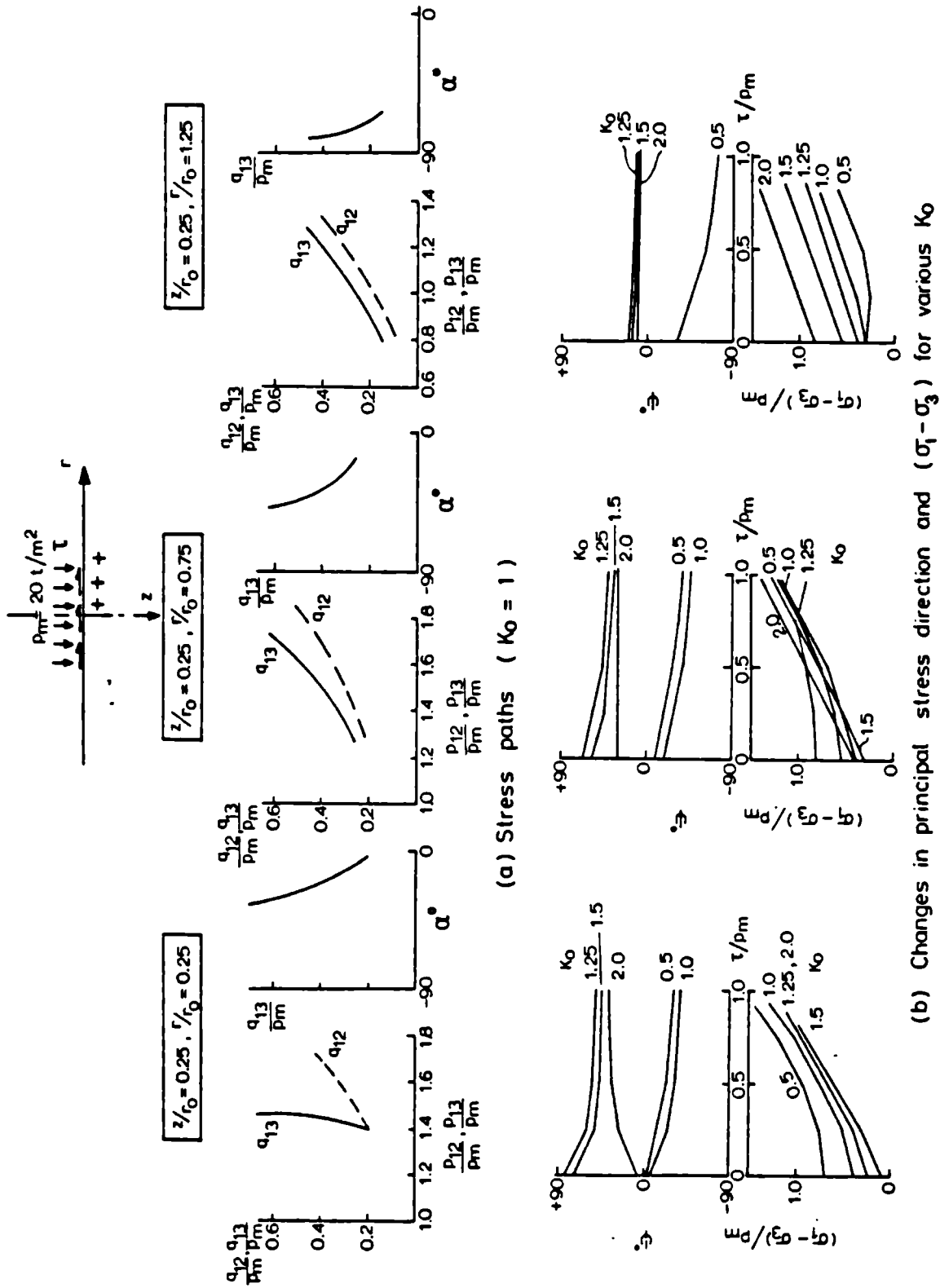


Fig. 2.9 Complete stress paths and principal stress rotations for elements beneath a flexible circular footing under uniform vertical pressure and increasing horizontal shear stress on isotropic linear elastic foundation ($\nu = 0.45$).

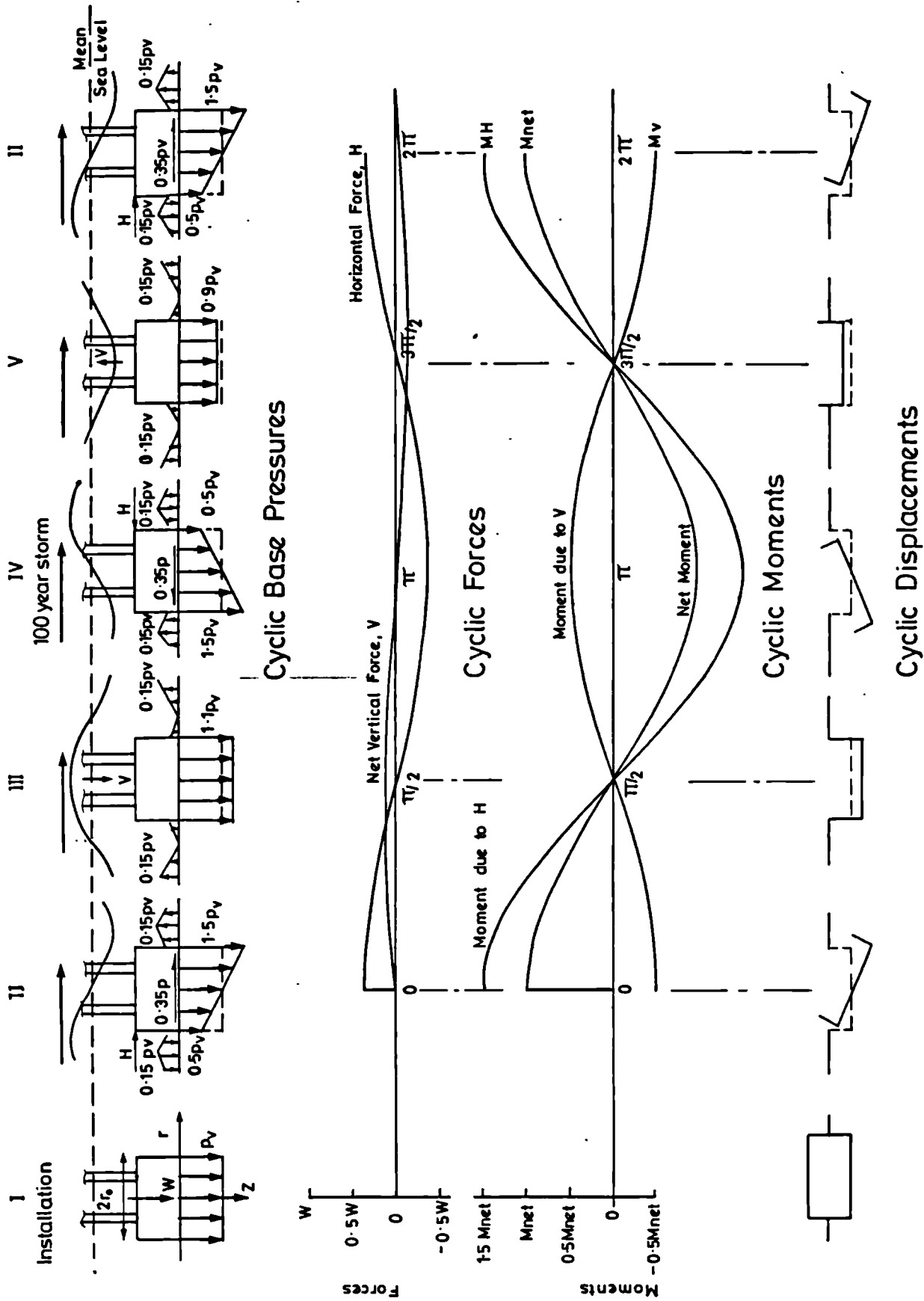
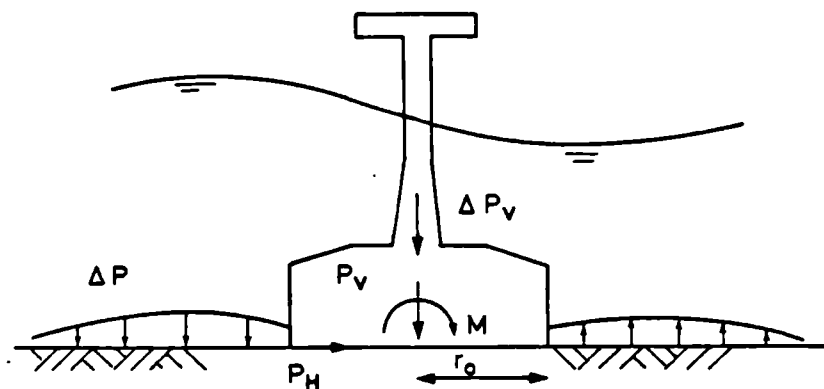


Fig. 2.10 Cyclic forces, base pressures, moments and displacements for a typical gravity structure under design storm wave loading.



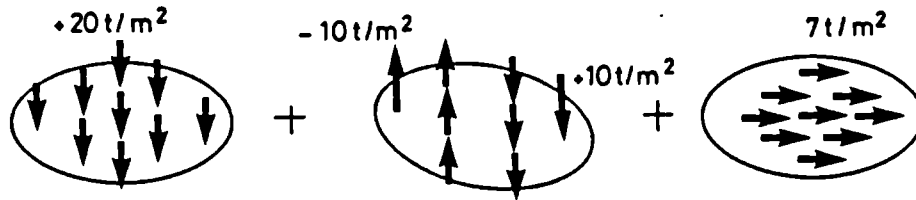
Assumptions :

Contact stresses :

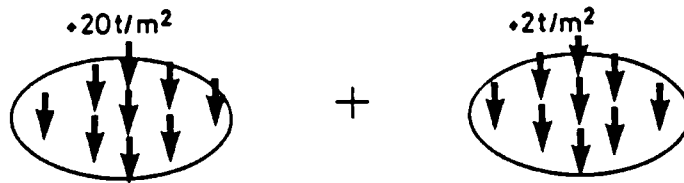
Foundation radius,	r_0	=	50 m	
Submerged weight,	P_v	=	157,000t	$p_v = 20 \text{ t/m}^2$
Cyclic vertical force,	ΔP_v	= $\pm$	17,450t	$\Delta p_{vv} = \pm 2.22 \text{ t/m}^2 = \pm 0.11 p_v$
Cyclic horizontal force,	P_H	= $\pm$	55,000t	$\tau_h = \pm 7 \text{ t/m}^2 = \pm 0.35 p_v$
Overturning moment,	M	=	1,000,000 t·m	$\Delta p_{vm} = \pm 10 \text{ t/m}^2 = \pm 0.5 p_v$
Sea bed pressures	ΔP_{max}	=	3 t/m ²	$= 0.15 p_v$
Wave length		=	400 m	

Fig. 2.11 Average values for forces, submerged weight and dimensions of typical gravity structure in the Northern North Sea.

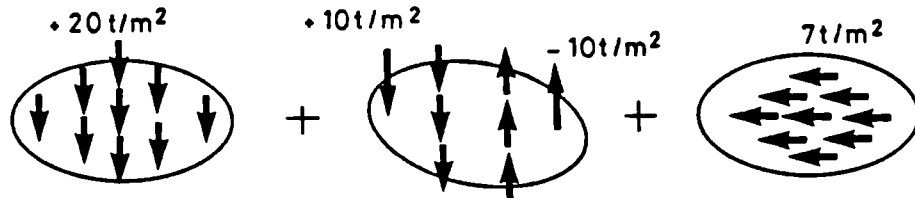
Wave Position II



Wave Position III



Wave Position IV



Wave Position V

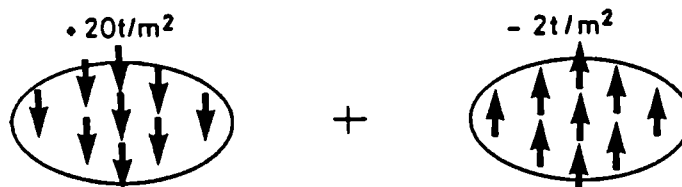


Fig. 2.12 Average contact stresses assumed for calculation of stress changes.

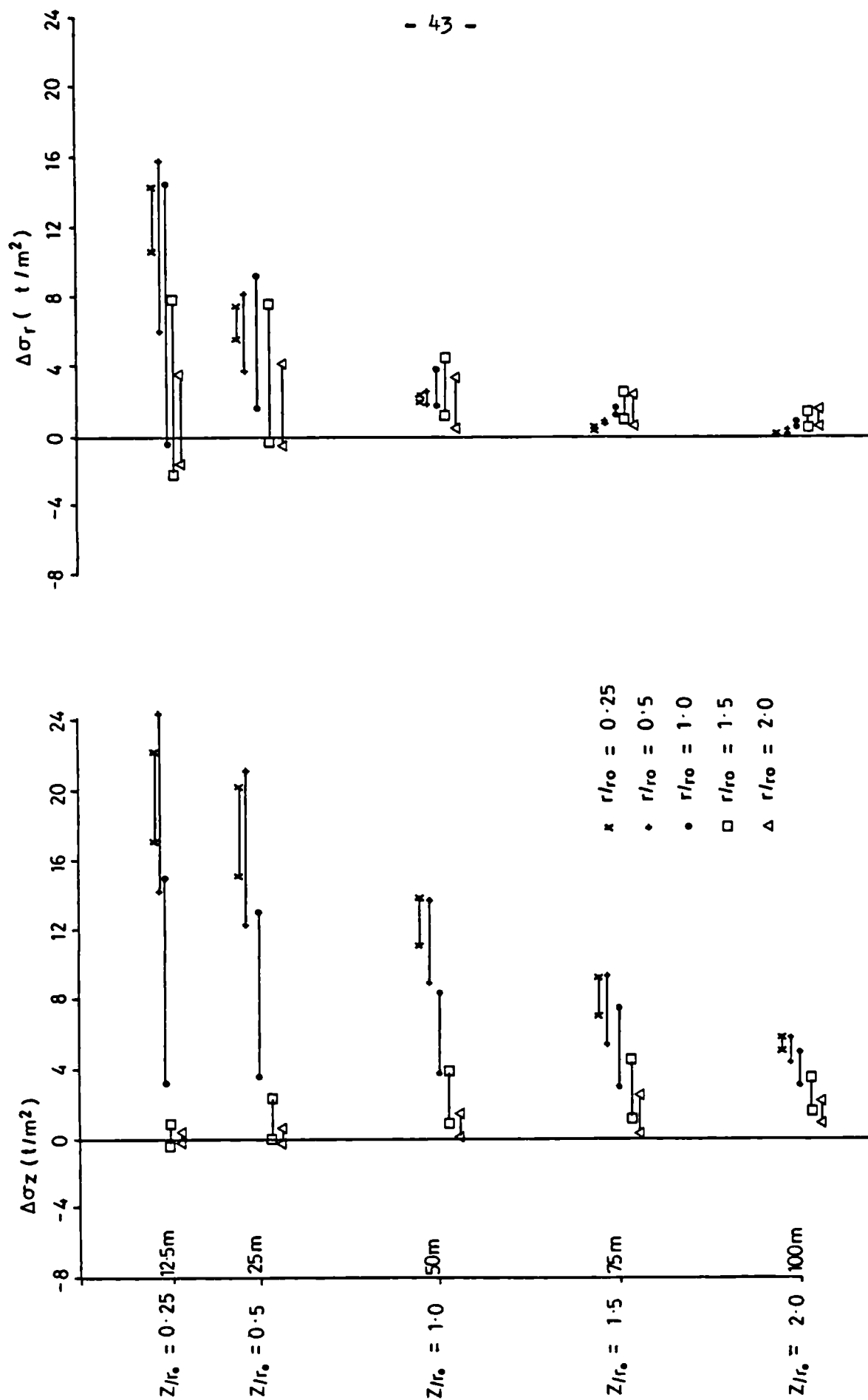


Fig. 2.13(a) Stress changes, $\Delta\sigma_z$ and $\Delta\sigma_r$, beneath typical gravity structure on isotropic linear elastic foundation ($\nu_r = 0.45$). Wave positions II and IV.

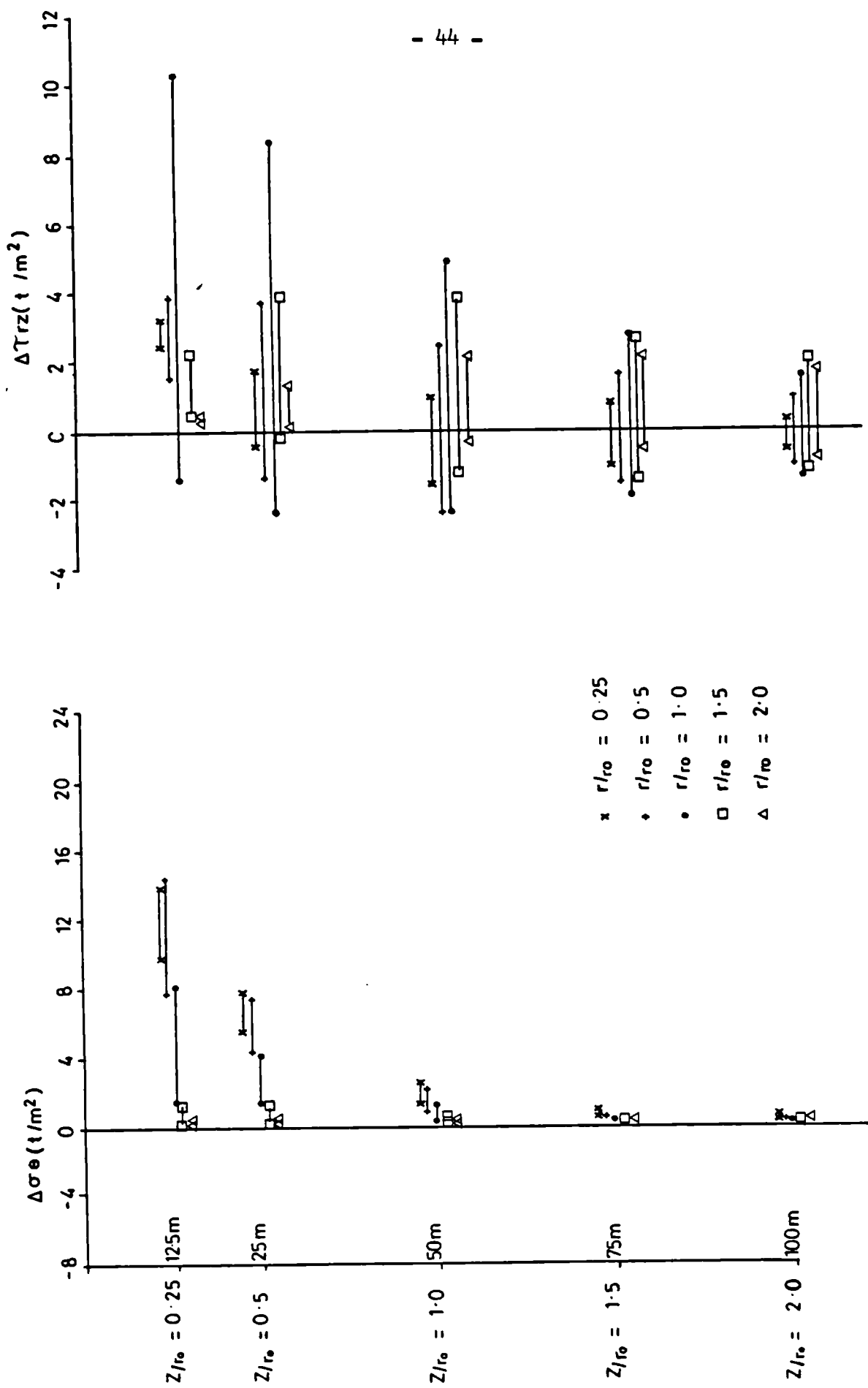


Fig. 2.13(b) Stress changes, $\Delta \sigma_e$ and $\Delta \tau_{rz}$, beneath typical gravity structure on isotropic linear elastic foundation ($\nu = 0.45$). Wave positions II and IV.

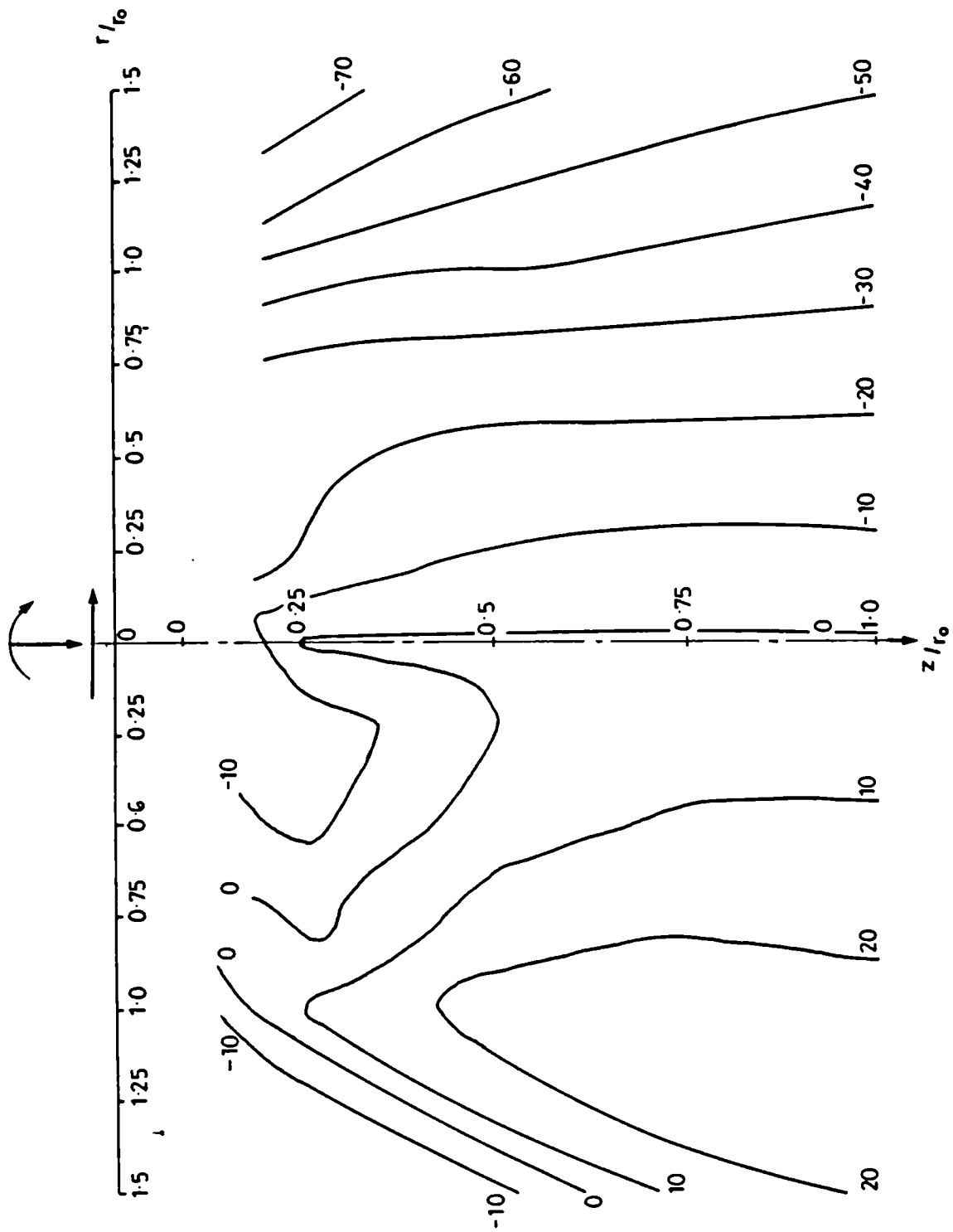


Fig. 2.14 Contours of rotation of principal stress increments in foundation to typical gravity structure.

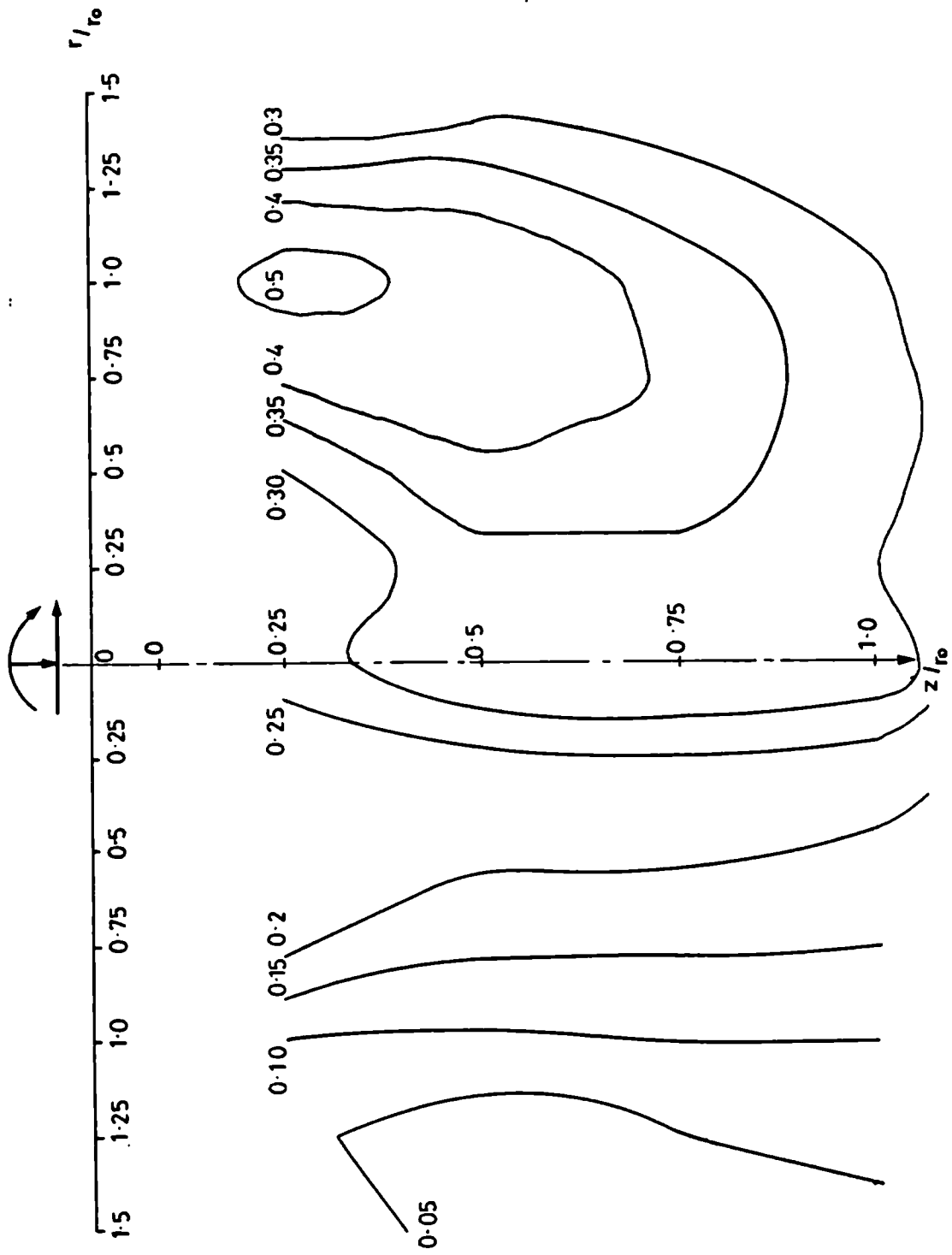
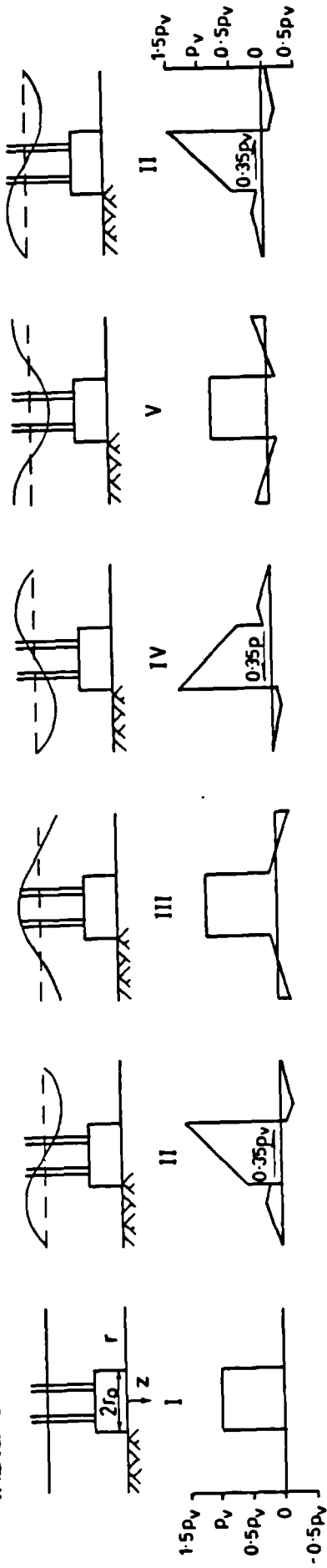


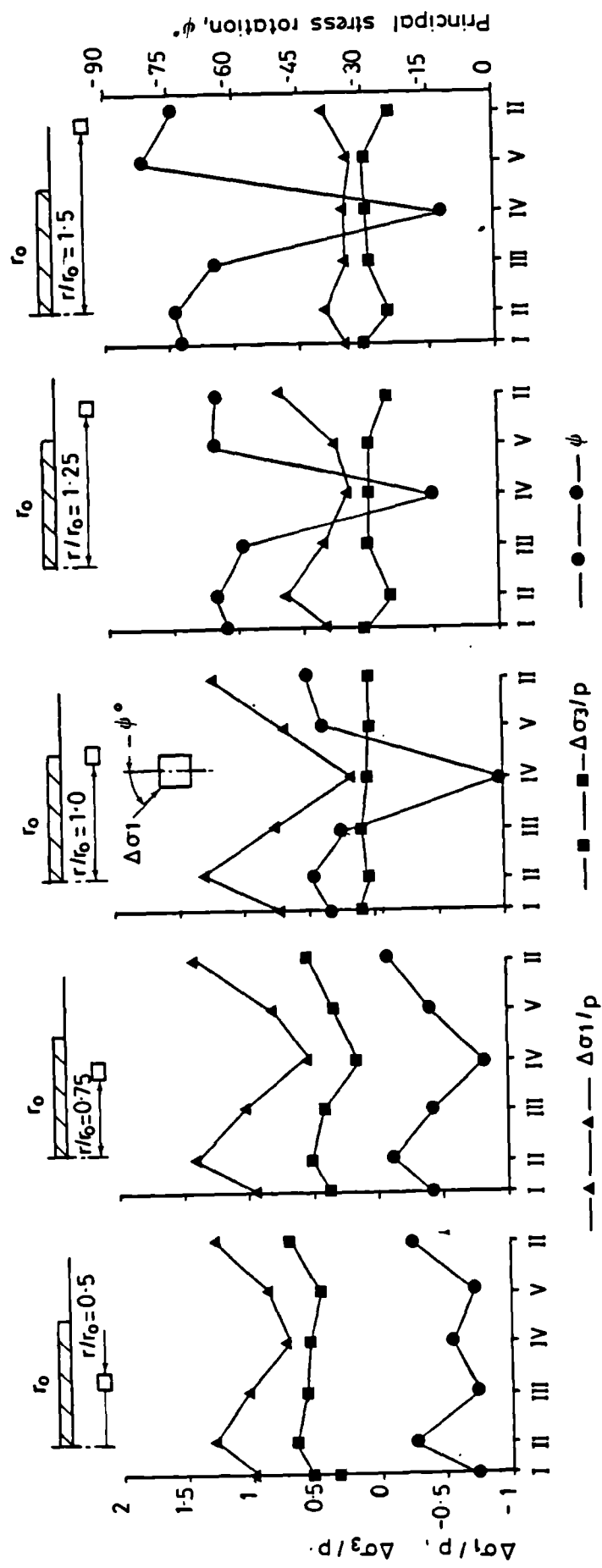
Fig. 2.15 Contours of $(\Delta\sigma_1 - \Delta\sigma_3)/2p_v$ in foundation to typical gravity structure.

Installation

Design Storm Loading



Loading Sequence and Assumed Sea - Bed Pressures



Cyclic Changes in Principal Stress Increments and Orientation at $0.25r_0$ Depth

Fig. 2.17 Sequence of stress changes at depth of $0.25r_0$ in foundation to typical gravity structure.

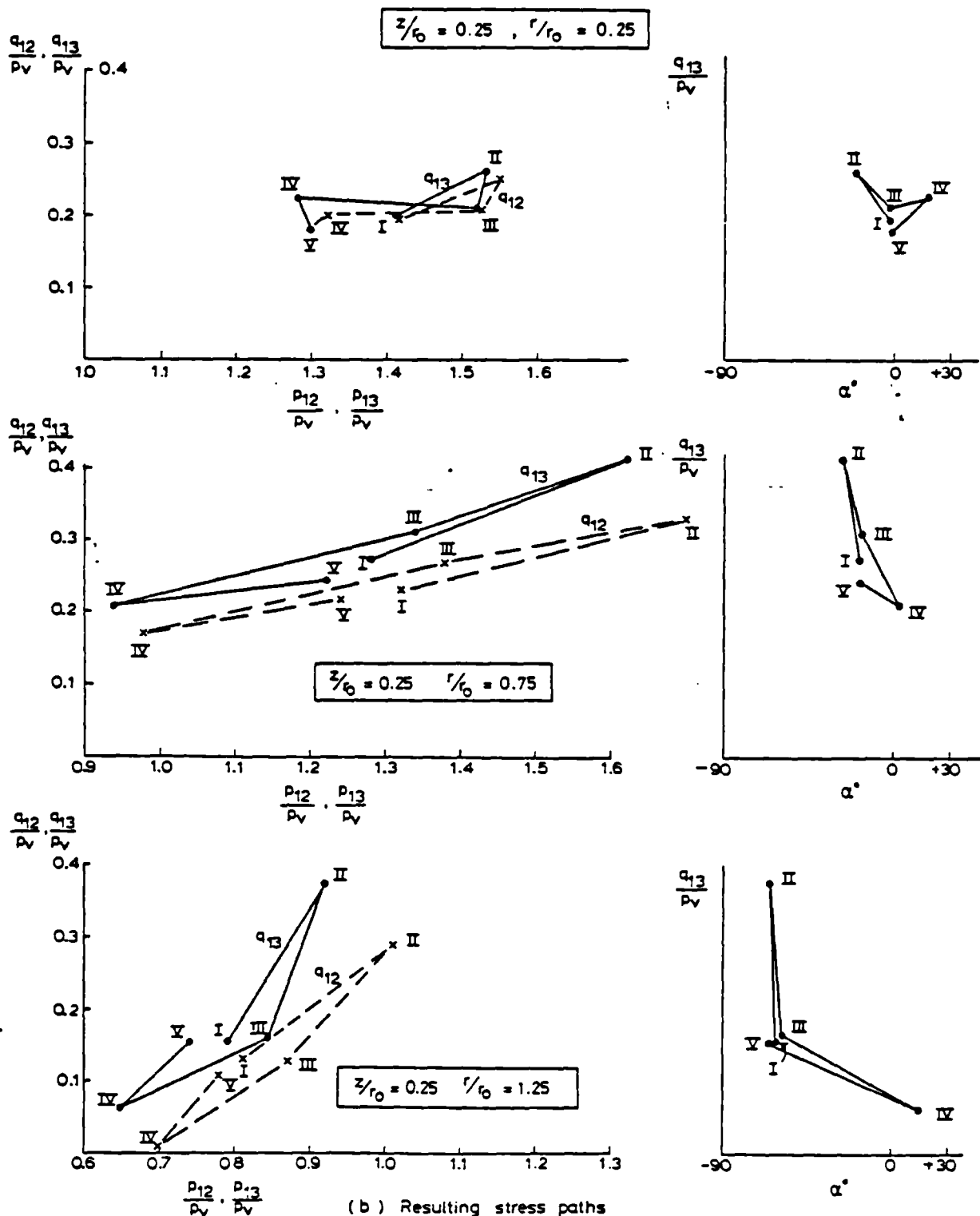
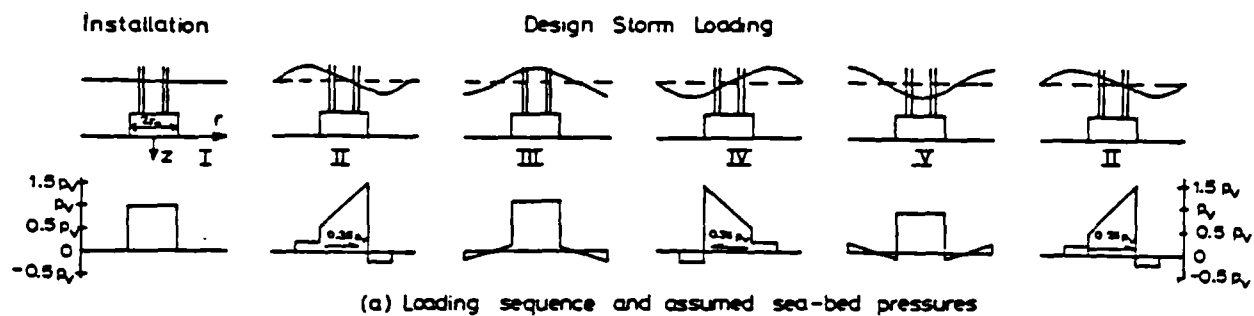


Fig. 2.18 Complete stress paths for three elements beneath a typical offshore gravity platform.

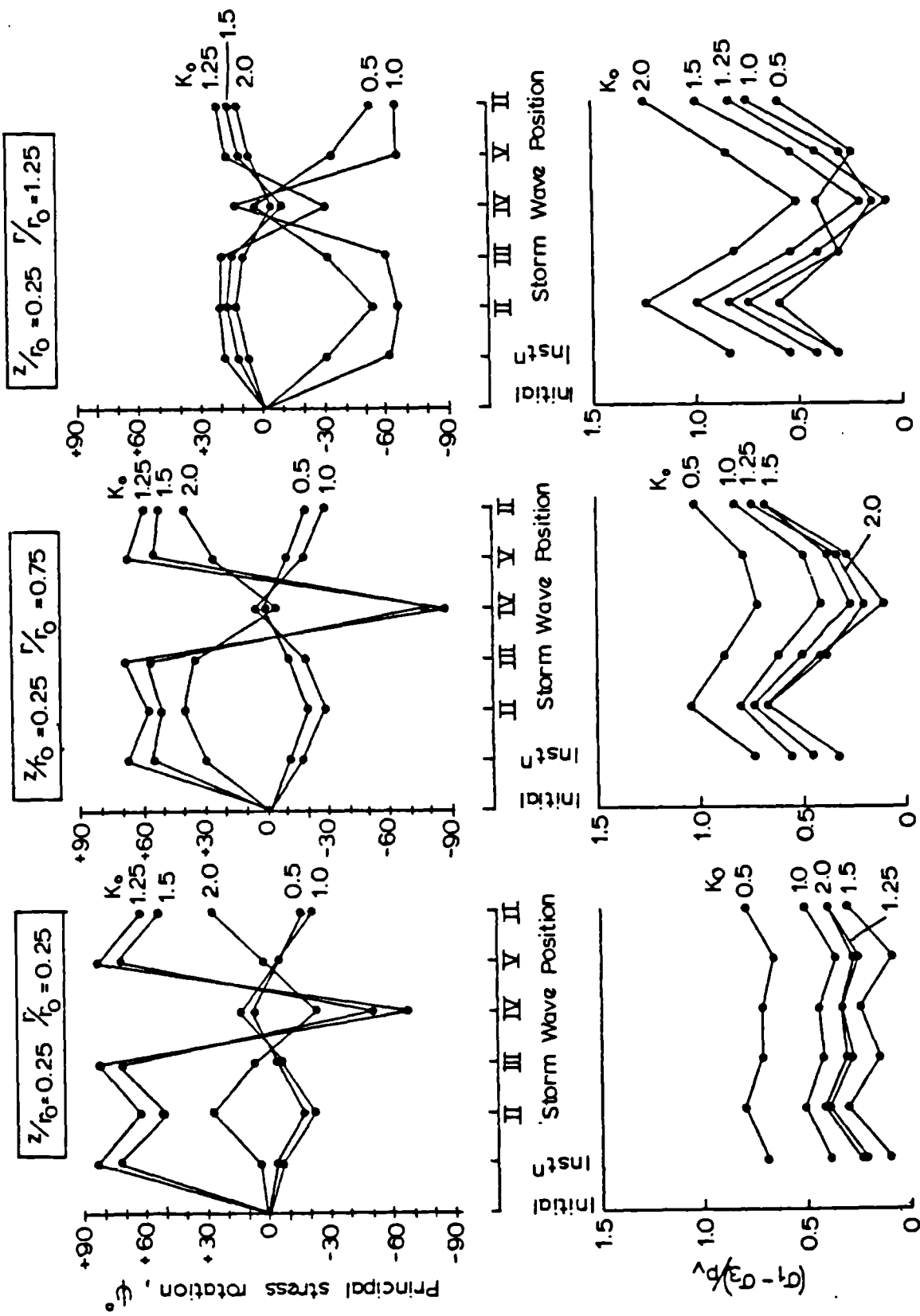
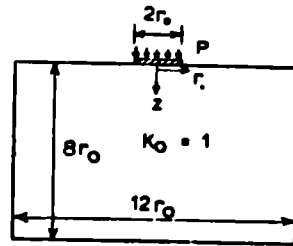
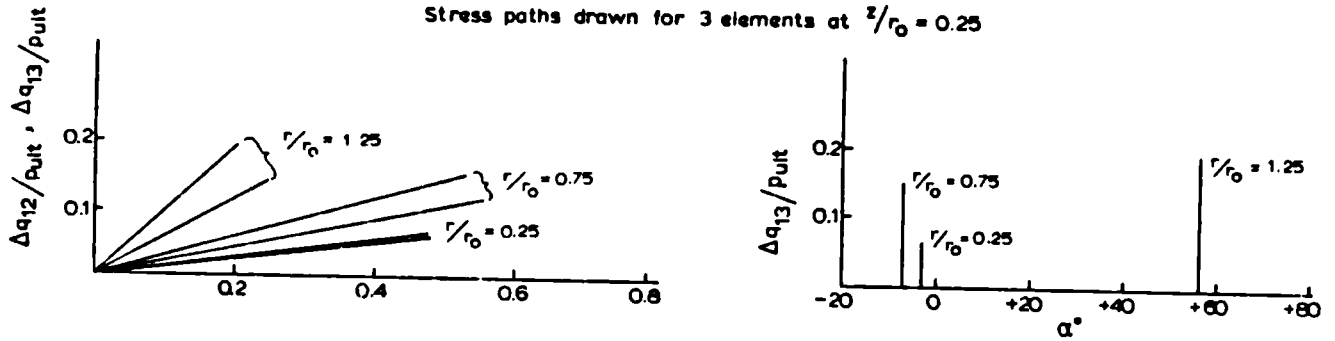


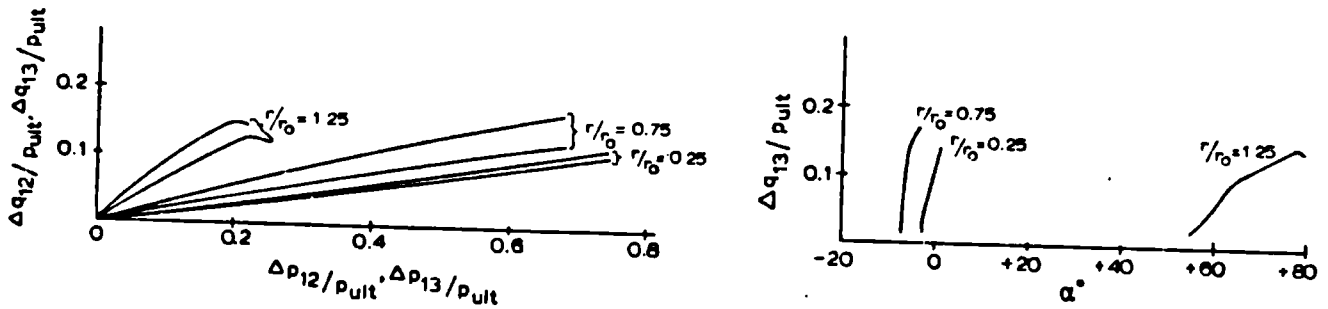
Fig. 2.19 Principal stress rotations and changes in $(\sigma_1 - \sigma_3)$ accompanying wave loading on gravity platform.



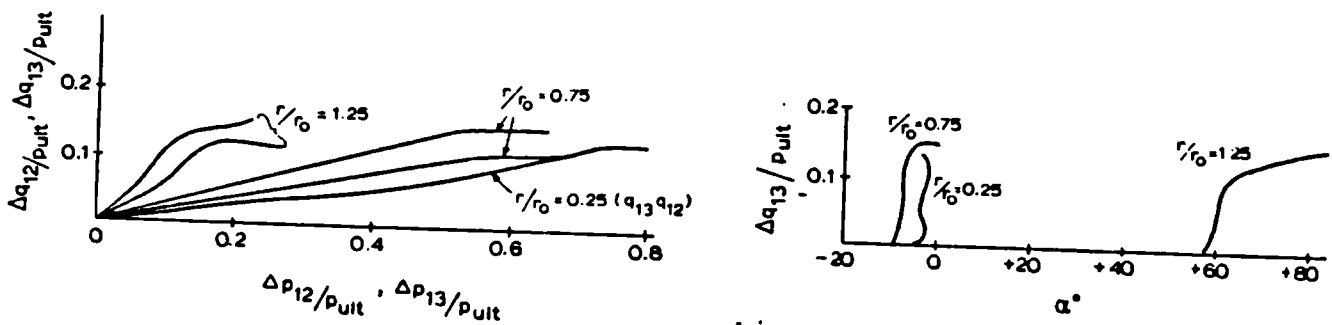
Stress paths drawn for 3 elements at $z/r_0 = 0.25$



(a) Linear elastic foundation

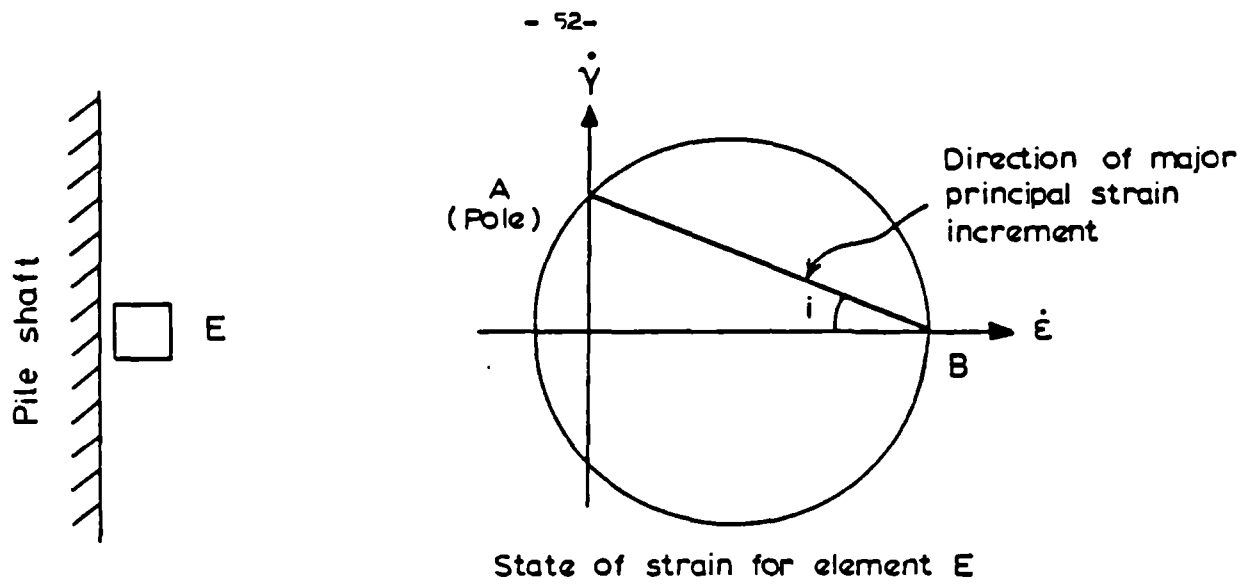


(b) Non-linear elastic foundation

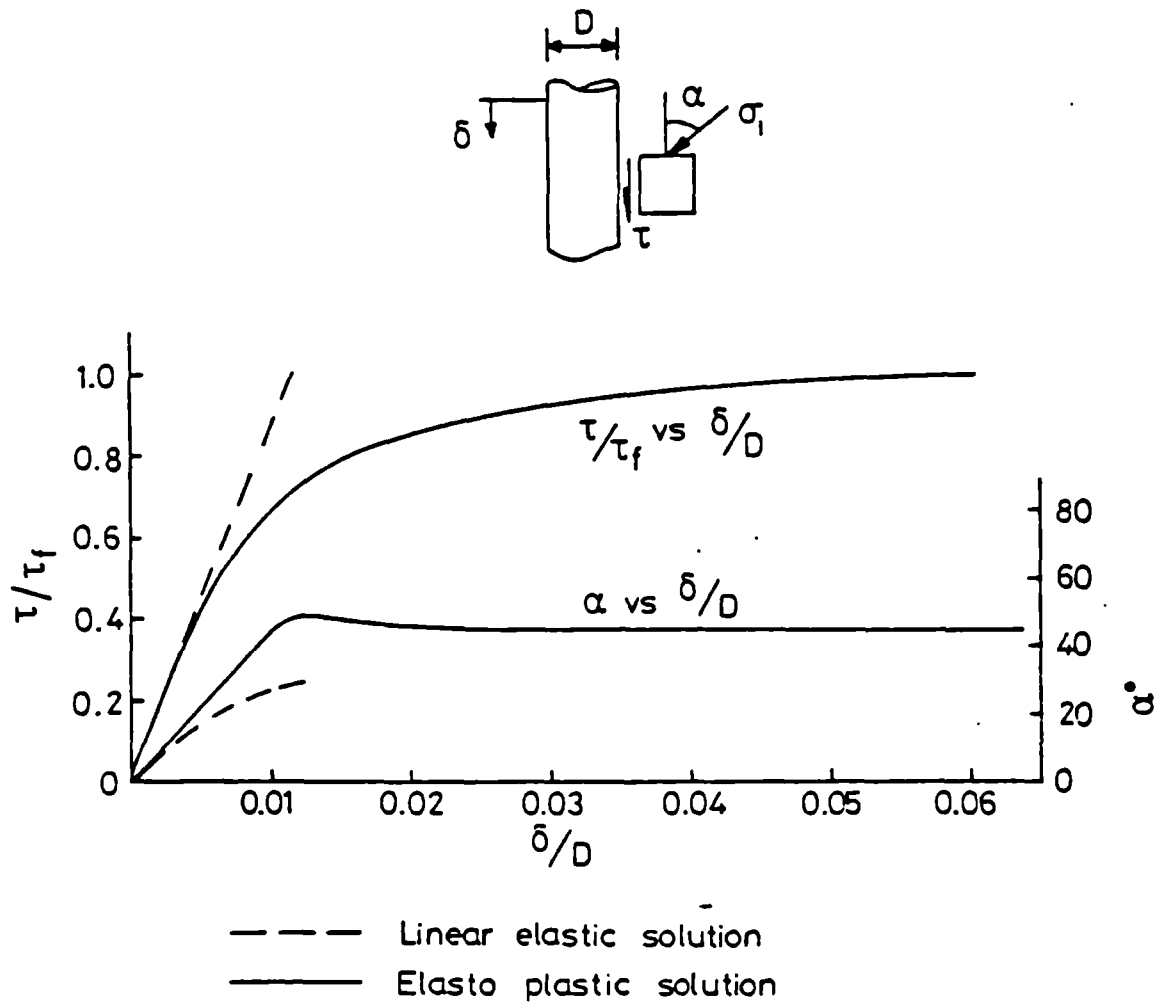


(c) Elasto-plastic foundation

Fig. 2.20 Influence of foundation's constitutive law on the stress paths for elements beneath a rigid circular footing under vertical pressure.

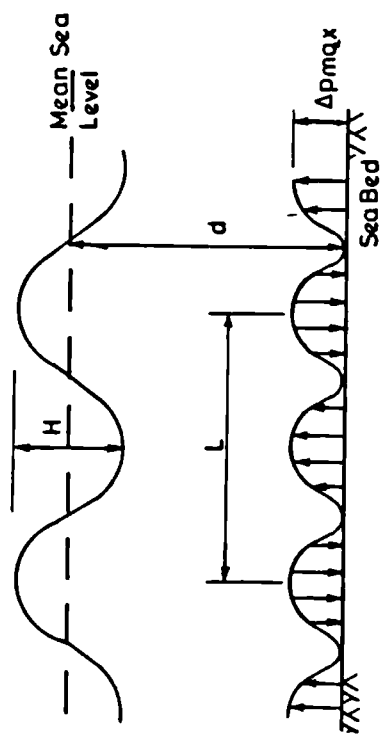


(a) Mohr Circle of strain for element alongside a rigid pile.

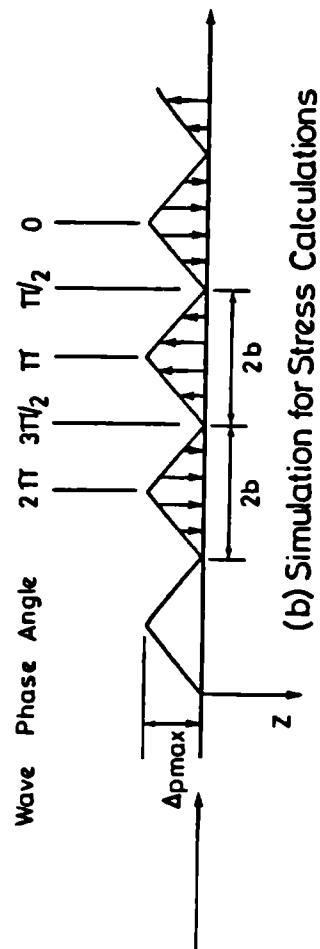


(b) Principal stress rotation during pile loading.

Fig. 2.21 Principal stress rotation alongside a rigid pile



(a) Sea Bed Pressure Fluctuations



(b) Simulation for Stress Calculations

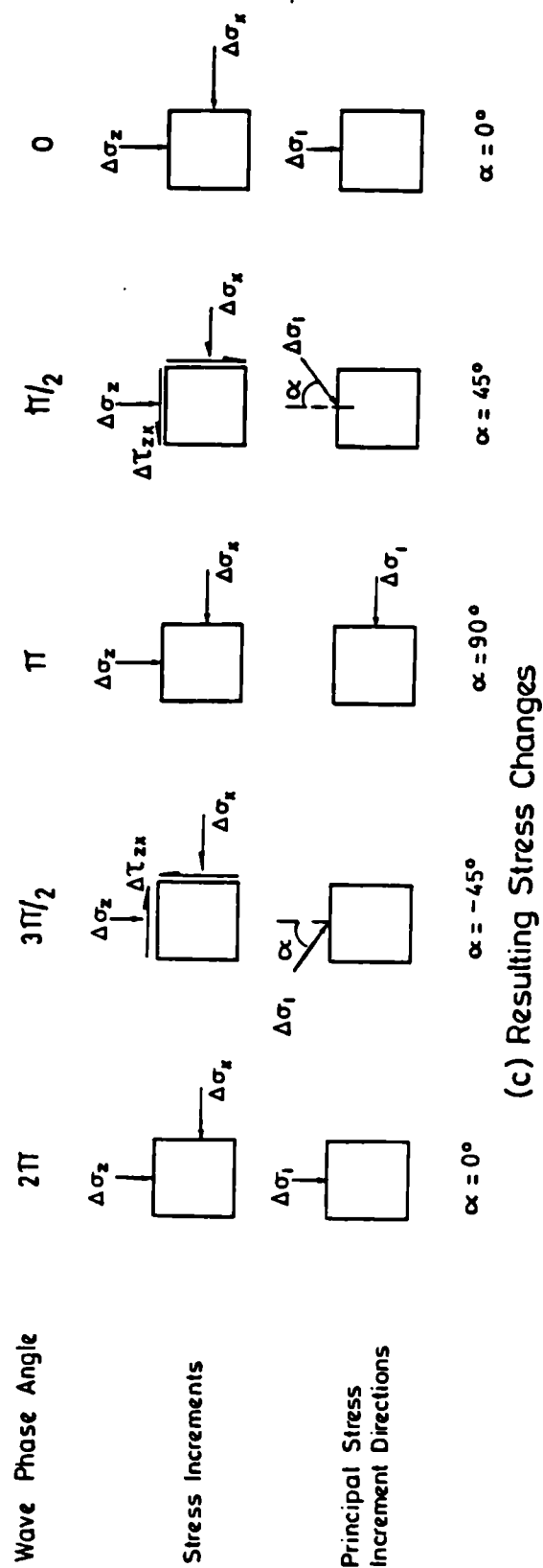


Fig. 2.23 An approximate method for determining stress changes produced by wave pressure fluctuations on the sea bed.

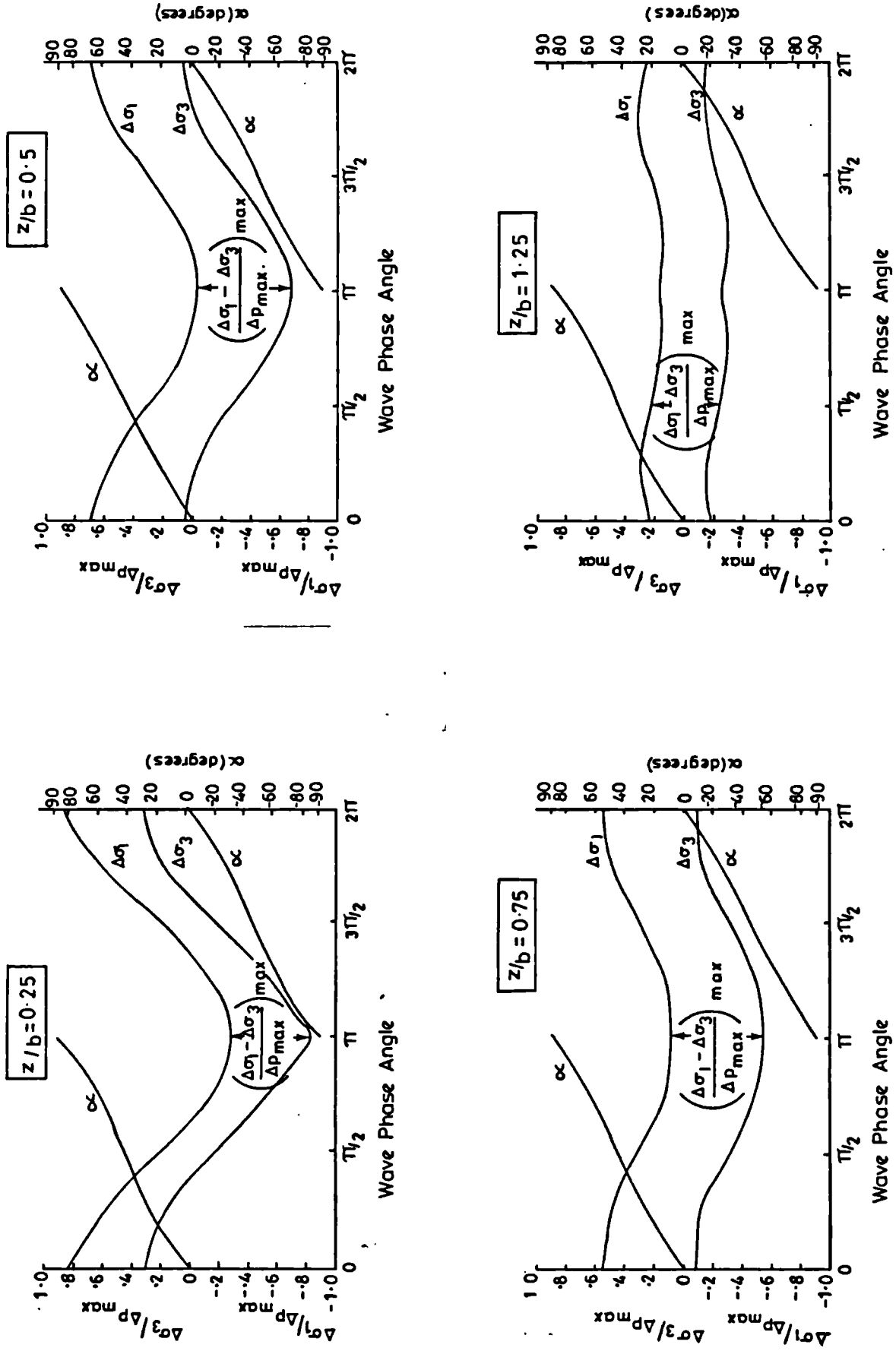


Fig. 2.24 Sequence of changes in principal stress increments caused by wave pressure fluctuations on the sea bed.

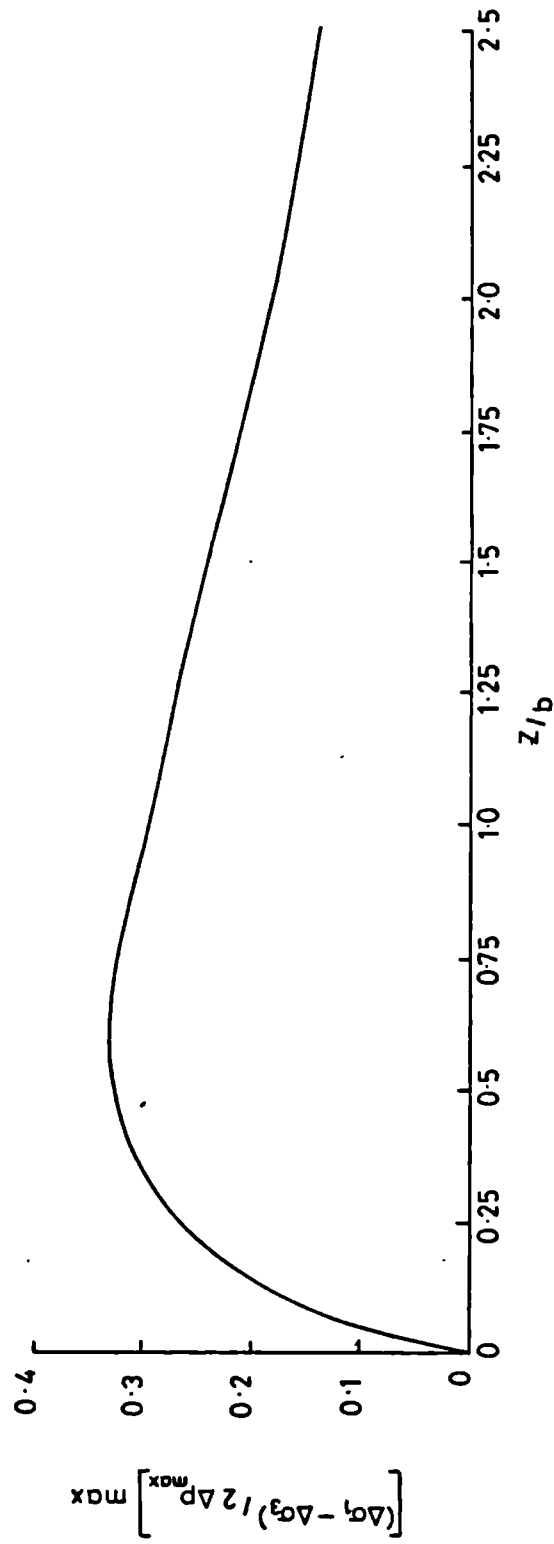


Fig. 2.25 Variation of $[(\Delta\sigma_1 - \Delta\sigma_3)/2\Delta p_{max}]_{max}$ with z/b resulting from wave pressure fluctuations on the sea bed.

CHAPTER 3

DEPOSITIONAL AND STRESS HISTORY, COMPOSITION AND FABRIC OF SEDIMENTS IN THE NORTHERN NORTH SEA

SYNOPSIS

With a view to identifying probable depositional modes and environments and post depositional processes and stress histories, a review has been made of existing information on geological history of the North Sea. This has been supplemented by a detailed examination of the composition and macro- and micro-fabrics of sediments from the Northern North Sea. On the basis of grading and plasticity, sediments classify into 4 types; on the basis of fabric, 6 types are recognised. The techniques for fabric study, which included radiography and optical and scanning electron microscopy, are described and their usefulness is demonstrated.

Glacial marine sedimentation is identified as a probable depositional mode for many of the sediments; the processes involved are examined and differences between glacial marine and terrestrial clay deposits are highlighted. Bonding is shown to be a potential post-depositional process. A wide range of stress histories is described; wave loading and its dependence on a changing sea level are suggested as important components of stress history.

The accumulated evidence serves as a basis for reconstructing North Sea sediments.

3.1 INTRODUCTION

For the research programme to be set in the context of the Northern North Sea it was clearly necessary to:-

- (i) work with the appropriate soils, and
- (ii) have information on the state of the soils in situ.

Furthermore, since it would be necessary to use reconstituted soil which attempted to match that in situ, as will be discussed in Chapter 4, information was sought on factors which might have determined the in situ state of the soil, such as depositional mode and environment and subsequent stress history.

A short investigation was made at the outset of the research with the aims of identifying the range of soil types present in the Northern North Sea, their macro- and micro-fabric, mineralogy and porewater chemistry, and their origins and subsequent history. This involved a review of published information on North Sea geology and soil conditions, a review of glacial marine sedimentation, an inspection of borehole logs, a detailed examination of a limited number of soil samples from the area using X-ray and thin section techniques and both optical and scanning electron microscopy, and a number of geochemical and geotechnical tests on these samples.

The results of this investigation and, where appropriate, the methods used, are described in this chapter. A feature of the stress history of marine soils, namely wave loading, is considered in detail.

It must be emphasised that the investigation, which concentrates on clay soils, was limited in scope and, since the bulk of it was carried out in 1974/75, the findings will have been supplemented by recent, more exhaustive, investigations.

3.2 EXISTING INFORMATION

Prior to exploration for oil, no in situ investigation of the geology of the drift deposits in the Northern North Sea appears to

have been undertaken. Information was restricted to some limited details of bathymetry, partly based on lead-line soundings dating from the eighteenth and nineteenth centuries (Caston, 1974), to a sketchy picture of the distribution of sediments lying on the sea bed, developed using evidence from grab samples and anchoring ships, and to a speculative appraisal of current rates of deposition and erosion. A background geology could, however, be built up by interpreting this information and extrapolating from on-land evidence.

Bathymetry

According to Caston (1974) and as illustrated in Fig. 3.1, the greater part of the Northern North Sea bed lies at depths between 90m and 150m. There is a haphazard pattern of shallower banks, with minimum depths ranging from 60m in the south to 80m in the north, separated by intervening deeper areas in which depths vary between 120m and 180m. There is a broad depression centred on 58°N - 59°N and 0° to 1°E and a series of banks running N-S from 2°E at 61°N to 3°E at 58°N .

The most prominent morphological features are the narrow "linear holes" which occur singly or in groups, e.g. Devil's Hole Deep, Buchan Deep, Fladen Ground Deep (Flinn, 1967, 1973; Caston, 1974; Stubbs, 1974). These features, up to 80 fathoms deep, have a V-shape with a low width to depth ratio and are closed at each end, with no signs of a shallow channel-like continuation. They are similar to tunnel valleys encountered further south in the North Sea (Woodland, 1970; Dingle, 1971; Donovan, 1973).

In his bathymetric survey of the area, Flinn (1967) states that no submarine moraines were recognized as topographic features. Caston (1974) notes that sand waves and banks are restricted to a narrow zone off the Scottish coast, in estuaries, and between the Orkney and Shetland Islands.

Superficial Deposits

It is generally agreed (Bjerrum, 1973; Caston, 1974; Eden, 1975) that in the Northern North Sea a superficial layer of fine sand, often

only a few centimetres thick, is widespread. Eden (1975) describes the flat areas in deep water as usually being occupied by thick incompetent superficial mud; the shallower banks rise above the mud plains and comprise outcrops of tough ill-sorted clay with cobbles and boulders; the intervening gently sloping areas of the sea floor, between the banks and depressions are covered by sand and muddy sand. Eden presents a map, which is reproduced as Fig. 3.2, which shows the approximate distribution of sea bed deposits in the Northern North Sea. The map is based on Admiralty charts, International Fisheries Charts, and on work by IGS, Flinn (1973) and Veenstra (1971); it is similar to maps published by Caston (1974) and Bjerrum (1973).

Bedrock outcrops only along the coasts of Scotland and the Orkney and Shetland Islands. Areas of sandwave activity and iceberg ploughmarks have been added.

Current Rates of Deposition and Erosion

The processes of erosion are more powerful in a submarine environment than in a sub-aerial one. As a consequence, potential rates of erosion, and of deposition, at a site become an important design consideration for offshore structures. Modification of these rates by the structure itself must also be taken into account. Information on recent rates of deposition and erosion is also required in a complete assessment of stress history.

McCave (1973) examined the areas and rates of deposition in the North Sea of mud, i.e. material of organic or inorganic origin, which has an equivalent diameter less than $63\mu\text{m}$. He considered the balance between the supply of material, from rivers, cliff erosion, organic production, etc., and removal of material and discovered that the 13 million tons of mud available for deposition was not compatible with the deposition rates recorded in mud areas. He concluded that:-

- (i) muddy sands on topographically higher areas in the Northern North Sea are being winnowed and acting as the major source of material for deposition in the whole North Sea. (Unfortunately, McCave gives no estimate of this erosion rate.)

- (ii) Deposition does not occur uniformly but is concentrated in sediment traps formed by the deeper troughs. In these, deposition rates may be between 10 to 40cm per hundred years, based on evidence from the Southern North Sea.
- (iii) In the Northern North Sea wave activity at the bed is sufficient during storms to remove any thin deposits from areas other than sheltered depressions, so that deposition is rarely apparent, i.e. mud deposits are absent.

These conclusions are consistent with the information on the nature of the sea bed (Fig. 3.2) and the bathymetry (Fig. 3.1) which shows a correlation between submarine relief and sediment size, the finer-grained deposits occurring in the areas of deeper water and the coarser and more poorly sorted materials being found on the banks. The lack of sedimentation over the last 5,000 years is confirmed by Dyer (1980), who refers to fossil shallow water arctic faunas and floras being found in the top 1 - 2m of the sea bed and to the absence of bedforms which indicate active transport, i.e. ripples, sand ribbons and sand waves; the presence of iceberg ploughmarks is another indicator of little movement of sea bed sediment.

Background Geology of the Drift Deposits

Evidence^{*1} had been available on land for sometime, and certainly prior to investigations offshore, to suggest that ice-sheets which originated in the uplands of Scotland and of Norway extended out into the North Sea and that at various times ice must have covered almost the whole of the North Sea (Redding, 1976).

Evidence then became available in the Southern North Sea to indicate glacial deposition of the drifts; for example, the Dogger

*1 For example, the presence of Norwegian erratics in Scotland and Northern England and the westerly movement of ice across Shetland (Flinn, 1967).

Bank had been interpreted as a glacial feature (Veenstra, 1965; Oele, 1969) while Veenstra (1969) had demonstrated that gravels off the Dogger Bank were of Scottish origin, those off the Cleaver Bank of Scandinavian origin. Detailed investigations for gas production, e.g. Adye et al (1971), had shown that drift deposits, up to 100m thick, were a complex sequence of normal marine and glacial detritus of several ages and composition. Oele (1969) had interpreted the deposits in the Dutch sector of the North Sea as being thin tills overlying fluvioglacial clay and separated from it by an erosion surface.

The "linear holes" known to be present in the Northern North Sea from bathymetric evidence, were interpreted as glacial erosion features at an early date (e.g. Flinn, 1967; Woodland, 1970; Dingle, 1971). Flinn (1967, 1973) linked the information on bathymetry and superficial deposits in the Northern North Sea, pointing out that the deeps were arranged in an arcuate belt partly surrounding the deepest part of the North Sea where there is a patch of mud, and suggesting that the belt of deeps marks the front of an ice sheet at or soon after the last glacial maximum. He went on to postulate that:-

- (i) the area to the east of the deeps was ice-free;
- (ii) the mud was deposited by water flowing out from beneath the ice into this ice-free area;
- (iii) the sea was 90 fathoms lower relative to the land than at present.

Clearly, glacial and inter-glacial deposits associated with the different glaciations of the Quaternary period could be anticipated in the Northern North Sea and, from Flinn's suggestions, the mode of origin of some might involve deposition through water.

The evidence described regarding current rates of deposition and erosion suggests that in the deeps and other sediment traps, deposits have continued to accumulate in post glacial times. The amount of

deposition and reworking was probably greater during the eustatic rise in sea level (Owens, 1977).

This very broad picture of glacial and post-glacial deposits was confirmed by the first detailed investigations in the Northern North Sea to be published, namely for the Forties field (Caston, 1974a; De Ruiter and Fox, 1977). At this site, centred at 57°45'N, 0°55'E, the Quaternary deposits were estimated to be 370m thick. Erosional channels, similar to those described above, were encountered, being cut into a stiff clay, considered to be a till, and being infilled with a fine sand, which was believed to be of fluvioglacial origin. Overlying the till and fine sand in parts was a stony clay, which was interpreted as a moraine and which formed the shallower banks, also referred to above; elsewhere the till was overlain by a homogeneous clay deposited through water. In the intervening deeps, soft post-glacial layered clays and silts, up to 20m thickness, were present, the material being derived by erosion of topographic highs.

With the benefit of widespread geophysical investigations, of both shallow (vibrocore sampling to 8m) and deep borings, of palaeontological studies and carbon dating it is now appreciated (Caston, 1977; Holmes, 1977; McCave et al, 1977; Fannin, 1979) that in the North Sea:-

- (i) Quaternary sedimentation was strongly influenced by older structural patterns. Subsidence in a central trough in the North Sea, the central Graben, was initiated in the Tertiary and has continued up to the present. Average accumulation during Devensian times kept pace with subsidence, but subsequently water depth has increased. Average sedimentation and subsidence rates were exceptionally high in the late Quaternary (up to 3mm/year).
- (ii) Up to 900m of Quaternary sediments have accumulated in the Central Graben; the thickness of sediments reduce to the west and north of the trough as shown in the isopachyte map in Fig. 3.3.

- (iii) The Quaternary comprises a suite of glacial, fluvio-glacial, glacio-marine and marine sediments, mainly Devensian (Weichselian) in age in the North.
- (iv) A series of banks exist along the Norwegian Trench which are interpreted as moraines.
- (v) Erosion features are widespread, occurring at two levels. They represent two periods of cutting, one pre-late Devensian and the other post-glacial or late glacial.

The area of the North Sea between 56°N and 58°N has been intensively studied by the Institute of Geological Sciences (IGS) and the study has culminated in a series of papers published during the course of the research (Thomson and Eden, 1977; Holmes, 1977; Eden et al, 1977).

In Table 3.1 a summary has been attempted by the writer of the Quaternary succession in this area of the North Sea. A cross section, showing the relative disposition of the deposits, is presented in Fig. 3.4. A postulated sequence of events is given in Table 3.2. Eighty per cent or more of the total thickness of Quaternary deposits is thought to be dominantly marine and this forms the lower part of the succession; the upper part is composed of a complex Middle to Late Devensian glacial and marine succession.

At the time of writing, the most detailed account of the recent geology of the North Sea, north of latitude 58°N , is given by Milling (1975). He considers the sediments in the foundation-zone to be piled and gravity structures to be Late Devensian to Holocene in age and, as further south, to be characterized by complex interbedded sequences of glacial, glaciomarine and marine sediments. Deeply incised erosional channels up to 80m deep are again evident, and are infilled with softer sediments. The morainial ridge complex along the Norwegian Trench comprises glacial tills and outwash sands. Milling suggests a subsidence rate of 5mm/year which means that some of the higher standing areas, e.g. the submarine morainial ridge, were probably exposed to subaerial weathering during eustatic lowering of sea-level.

Undoubtedly, many of the conclusions regarding the succession and sequence of events in the IGS study area can be extrapolated to the North, providing that account is taken of the greater intensity of glacial effects further north. Certainly a similar pattern of erosion and deposition occurred; Fannin (1979) refers to seismic evidence between 61°N and 62°N which suggests that eroded material from earlier phases has been dumped over the edge of the continental shelf. Fannin suggests further that an even more complicated sequence of events has affected the sediments close to the sea bed.

Summary

In the Northern North Sea both Holocene and Quaternary soils are present and these range from sands to clays of varying plasticity. More recent soft laminated clays and fine sands, up to 20m thick, occupy depressions and infill deep erosional channels. Elsewhere stiff clays, overlain by a thin fine sand layer, or dense sands occur at foundation level. The clays may be tills or moraines, and, therefore, poorly sorted, or more uniform marine/glaciomarine deposits.

3.3 COMPARISON OF TERRESTRIAL AND MARINE GLACIAL SEDIMENTS

The engineering behaviour of the sediments in the Northern North Sea will be influenced by:-

- (i) composition, i.e. grading and plasticity;
- (ii) mode and environment of deposition;
- (iii) post-depositional processes; and
- (iv) changes in stress state, i.e. stress history.

It seems likely that very different depositional modes and environments existed. The changes in sea depth which occurred during and after the glacial periods have an important bearing on these likely modes and environments, and on stress history and post depositional processes, particularly of the glacial sediments. These changes in depth will have been determined by:-

- (i) eustatic changes in sea-level,
- (ii) isostatic readjustment, and
- (iii) basement subsidence.

Various figures for lowering of sea level have been quoted; for example Bjerrum (1973) suggests a lowering of 100m, Flinn (1973) a lowering of 90 fathoms and Milling (1975) argues for a lowering of up to 90m in Late Middle Devensian time increasing to 120m in Late Devensian.

From consideration of these reductions in sea level (up to 120m) and of present sea depths (up to 180m) it is clear that glacial marine sedimentation will have occurred at times in the Northern North Sea, almost certainly occurring during final retreat of the ice sheets. Both terrestrial and marine glacial deposits are likely in the area of interest; this was confirmed in the review of background geology. Terrestrial glacial deposits include those derived from grounded ice, as lodgement, melt-out and flow tills and moraines, and those derived from meltwater as fluvioglacial and glaciolacustrine sediments. Glacial marine deposits may be derived from grounded ice or from ice floating as a shelf or as icebergs. Major differences can be anticipated between the sediments, both in terms of their engineering properties and their distribution as a result of differences in the four items listed above.

Experience with engineering in terrestrial glacial deposits is extensive; experience with glacial marine deposits is limited. A review of glacial marine sedimentation, with the aim of identifying where differences in engineering properties from terrestrial deposits may

result, is considered to be warranted. Since clay tills appear to be widespread and are likely to prove to be the main concern in foundation performance, the review will concentrate on these.

Mode and Environment of Deposition

Carey and Ahmad (1961) and Anderson (1972) have summarised the principal processes involved when a terrestrial glacier extends into the sea and spreads laterally over the continental shelf. Their models for glacial marine sedimentation have been developed using evidence from modern glacial marine environments, such as the Ross Sea and Weddell Sea. Carey and Ahmad (1961) distinguish between wet-base glaciers, in which the glacier sole is above freezing and meltwater issues from within, and dry-base glaciers, in which the sole is below freezing and from which there is no issue of meltwater. The zones which they recognise for each condition are shown in Fig. 3.5; they comprise:-

- (i) the terrestrial zone, in which the glacier base is grounded above sea-level;
- (ii) the grounded shelf zone where the base is below sea-level but not floating; in this zone in a wet base glacier, the ice is partially bouyant with the load decreasing to zero at the point of flotation; in a dry base glacier this zone is equivalent to the terrestrial zone;
- (iii) the floating shelf zone where the ice is buoyant;
- (iv) an inner iceberg zone, from the ice barrier to the limit of winter pack ice; and
- (v) an outer iceberg zone, beyond the limit of pack ice but within the limit of icebergs.

The process involved outside the terrestrial zone and the sediment types are summarised and compared for wet and dry base marine glaciers in Table 3.3. It is apparent that ice shelves are the primary transporting agents in a glacial marine environment and when these disintegrate there is widespread deposition of debris.

Comparing glacial marine and terrestrial sedimentation then:-

- (i) More rapid rates of sedimentation and larger thicknesses of sediments can be anticipated in the marine environment, particularly in the floating shelf and iceberg zones. Rapid sedimentation results in organic material being trapped under anaerobic conditions. Biochemical degradation of this organic material results in the formation of large quantities of methane, carbon dioxide and hydrogen sulphide. Gas or, where water depths are large, gas-saturated pore fluid is likely to be present, as confirmed by Holmes (1977) who describes gas-related features where deposits are thick. Its presence has important potential effects on sampling (Chapter 4), in situ response, and can cause anomalous pore pressures in situ.
- (ii) Both modes of sedimentation will result in a rapidly varying distribution of sediment types; abrupt lateral and vertical changes will occur as a result of sedimentation in the marine environment, especially at the edge of the ice shelf.
- (iii) In the floating shelf and iceberg zones of a marine glacier, some sorting and stratification of the sediments may occur during deposition. There would appear to be an essential difference between sedimentation direct from the base of floating ice and sedimentation from issuing meltwater. For the former case, all particle sizes are present and while particle settling rate is proportional to size, the process may be continuous so that segregation may not be apparent. For the latter case, segregation and layering occurs, with particle size being related to distance and flow speed. Dry base marine glaciers might then be associated with unsorted tills, wet base glaciers with laminated/bedded sediments, presumably similar to terrestrial glaciolacustrine sediments.
- (iv) Very different depositional environments occur in the marine situation particularly because of varying levels of salinity

in the floating shelf zone^{*1}. The interface between fresh and salt water is unstable causing mixing, flocculation of clays, cation exchange and the rapid fall-out of large clay floccs. Differences in initial fabric and placement water content can be anticipated between tills associated with marine and terrestrial glacial sedimentation, except in the grounded shelf zone.

- (v) In the grounded shelf zone, the mode and environment of deposition is similar to that for terrestrial tills. Over-consolidation by ice loading may occur and there will be interbedding with fluvioglacial or glaciomarine sediments.

Post Depositional Processes

There are essential differences, too, between the post-depositional processes which are likely to affect terrestrial and marine glacial deposits. Of the processes identified by Boulton and Paul (1976) as affecting terrestrial tills, only the loss of fines by downward percolating water is likely to be absent in marine glacial sediments. Disturbance by slumping and slope failure will probably be on a larger scale in terrestrial sediments.

After deposition, glacial marine sediments are reworked and redistributed by marine currents and subglacial meltwater streams. A classification of the resulting glaciomarine tills was suggested by Harland et al (1966) and a summary has been made in Table 3.4. A typical sequence of events leading to the formation of these tills is suggested in Table 3.5.

Marine tills are subject to continued wave action. The interaction between wave pressures and sea bed soils will depend on the initial strength and stiffness of the soils, their permeability and their degree of saturation. In soft sea bed clays, wave action can cause thorough remoulding and can induce fabric and increased vertical permeability by causing mud volcanoes (Bohlke and Bennett, 1980; Henkel, 1982). It is also thought to be responsible for the development of a stronger crust in recent delta deposits (Bea and Audibert, 1980).

*1 Saltwater sedimentation - clays flocculate to settle simultaneously with silt and fine sand.
Freshwater sedimentation - clays remain dispersed and settle with lower velocity giving higher degree of parallel orientation.

As a direct consequence of their depositional environment, subsequent chemical changes can lead to bonding in the glacial marine sediments. This is considered in detail below.

Bonding in Glacial Marine Sediments

An important feature of dry base glacial marine sedimentation referred to above is the formation of dense brines beneath the floating ice shelf, which occurs when salts are excluded as sea water freezes on to the base of a newly exposed ice shelf. The brines sink to flow along the bottom as cold salinity currents, either becoming trapped in basins, or flowing out over the continental shelf.

Sea water is nearly saturated in calcium carbonate, CaCO_3 , the solubility of which increases as the temperature of the water drops towards zero. As part of the sea water freezes on to the ice shelf, the concentration of CaCO_3 increases but precipitation does not occur in the cold water. As temperatures rise, the dissolved CaCO_3 , which is at the sea bed and may have permeated it (depending on the presence of permafrost), precipitates, possibly cementing sediments already in place or contemporaneously being deposited.

Watt (1978) has demonstrated indirectly that the precipitation of CaCO_3 can increase the strength of a low plasticity clay. He prepared the following samples:-

- (i) 3 which were saturated for 4 days at room temperature with a brine solution;
- (ii) 3 which were saturated for 1 day at -25°C . and 3 days at room temperature with a concentrated brine solution which had been separated from a normal brine solution taken to -25°C .;
- (iii) 3 which were saturated for 4 days at room temperature with distilled water; and

- (iv) 3 which were saturated for 1 day at -15°C . and 3 days at room temperature with a concentrated brine solution which had been separated from a normal brine solution taken to -15°C .

Samples from group (iv) showed a 20% increase in penetration resistance measured in a fall cone test.

Direct precipitation of calcium carbonate is not the only means by which cementing can occur. The glacial marine sediments are known to contain considerable quantities of shell which may dissolve and recrystallise to form a bond.

Calcium carbonates, together with iron, aluminium or manganese compounds, are the most common cementing agents in bonded clays. Their influence on engineering properties has been investigated by Fischer et al (1978) who found that when 3 - 4% of CaCO_3 was precipitated in normally consolidated Drammen Clay, the liquid limit was reduced, the strength was increased by 35 - 40% and the soil behaved as if overconsolidated. Similar quasi-overconsolidation of cemented normally consolidated marine clays has been reported by Moore et al (1977). The effect of cementing is likely to vary with the stress history at the time of cementing.

Bonding is rarely present in terrestrial glacial sediments.

Stress History

The range of stress histories which may have affected the various sediments in the Northern North Sea, both glacial and marine, is examined in detail in Section 3.4. Considering here just the potential differences in stress history between terrestrial and marine glacial sediments then:-

- (i) although both are equally likely to undergo swelling and reconsolidation caused by erosion and redeposition, the process is likely to be on a larger scale in recent marine than in recent terrestrial sediments;

- (ii) only in terrestrial lodgement tills and in marine tills deposited beneath grounded ice will there be overconsolidation by ice loading, except, of course, where re-advances of ice occur;
- (iii) marine sediments are unlikely to be subjected to wetting-drying cycles, unless the sediments become exposed by lowering of sea-levels; both types of sediment can be subject to cycles of freezing and thawing;
- (iv) only marine sediments are subject to wave compaction unless, of course, terrestrial sediments become submerged.

Summary

Taking account of these differences in mode and environment of deposition, post-depositional processes and stress history, an attempt has been made in Table 3.6 to highlight those differences between terrestrial and glacial marine tills which are likely to influence their engineering behaviour.

Summarising, marine till (compound paratill) is likely to be less well graded and generally more plastic than terrestrial lodgement tills, with sandy clays and clays predominating. It is likely to have been consolidated from higher initial water contents, to have a relatively high gas content and possibly to be bonded. Interbedding and laminations are likely to be more common in marine tills but shear surfaces, shrinkage and unloading joints are likely to be less common. Both marine and terrestrial tills will be overconsolidated but will have followed different, but complex, stress histories.

Additional features which can be used to distinguish marine tills from terrestrial tills include their higher organic content, the occurrence of dropstones and intact shells, their stratification and generally better sorting and the more frequent inclusion of lenses of stratified sands.

3.4 POTENTIAL COMPONENTS IN THE STRESS HISTORY OF SEDIMENTS IN THE NORTHERN NORTH SEA

From the preceding reviews it is evident that there is a large range of possible stress histories for soils in the Northern North Sea, except for the recent soft clays which occupy the depressions in the bed. Stress changes may have been associated with:-

(i) Erosion-redeposition

At least two major sequences of erosion are recognised, see Table 3.2; these are associated with the formation of the Upper and Lower Channels. Erosion of the most recent deposits is only taking place at a very slow rate at present but was more effective when sea levels were lower; Holmes (1977) suggests that the Witch Ground Beds have an erosion surface which has developed since 13,000 BP as a result of repeated storm action. Certainly, an irregular topography would have been left by the retreating ice and modified by slumping and bearing capacity failures; the current, relatively flat, sea bed bears witness to the erosion and re-deposition which has occurred. Unfortunately, there is no information on the thickness of sediments which have been eroded.

(ii) Ice loading

Several advances and retreats of ice sheets occurred during the Quaternary of the N. Sea. Both lodgement tills and ice-push features have been recognised in some areas, indicating that at certain times and in certain locations the ice must have been grounded. Holmes (1977) presents a strong case for ice being grounded at a late stage, at least in the area studied in detail, 56°N-58°N. He considers that the first and deepest succession of Upper Channels correlates with a second phase of late Devensian ice; the readvance causing glacial disruption in the Lower Swatchway Beds and Fisher Beds in the north of the area. Ice-berg plough marks also point to the intermittent grounding of ice.

Consolidation will have occurred beneath the ice sheet unless the sediments were protected by permafrost or, during wet-base glaciation, the ice approached neutral bouyancy. When consolidation occurred, it would have involved a complex sequence of unloading, reloading and changes in piezometric conditions; its progress will have depended on local permeabilities and the time that the ice load acted. Certainly, variations in the degree of consolidation and in the strength reached could be anticipated. There is no reason, however, for the maximum consolidation pressures to have been significantly higher than on typical terrestrial tills.

(iii) Ground freezing-permafrost

The sea bed may have been frozen beneath a dry-base grounded glacier or when the sea water was frozen in the presence of floating ice. Such conditions exist currently in the Beaufort Sea where there are substantial areas of subsea permafrost beneath shallow water depths and floating ice.

Freezing sets up pore water suctions and may be accompanied by the formation of reticulate ice. Two effects remain on thawing:-

- (i) overconsolidation when the pore water pressure returns to its original value (Nixon and Morgenstern, 1973), and
- (ii) the presence of fissures and other characteristic fabrics (a blocky macrofabric, Williams (1968); separation of fine and coarse particles, Williams (1968); a pedal microfabric, Smart and Tovey (1981)).

The size of the suction and hence the magnitude of the preconsolidation pressure will depend on such factors as temperature gradients, permeability, and pore water chemistry (e.g. salinity determines the freezing point and may restrict permafrost development). Williams (1966) has demonstrated that the pore pressure at the freezing front, u_i , can be related to the air entry value for the soil ($p_a - u_a$) by:-

$$u_i = p_i - (p_a - u_a) \cdot \frac{\sigma_{iw}}{\sigma_{aw}}$$

where

p_i	=	pressure of ice	
σ_{iw}	=	surface tension, ice-water	
σ_{aw}	=	surface tension, air-water	
p_a	=	pressure of air	} in measurement of air entry value.
u_a	=	pressure of water	

To estimate air entry value, data has been collected on the relationship between air entry value and permeability. For permeabilities of the order of 10^{-10} m/sec, which could be anticipated for Northern North Sea sediments, air entry values would approach 600 kPa and, taking σ_{iw}/σ_{aw} as 0.4, suctions up to 1500 kPa could develop on freezing.

Sutherland and Gaskin (1973) have presented data which indicates that while William's expression is supported in silts, it leads to an under-estimate by a factor of at least two of suctions caused by freezing in clays. It is evident that substantial preconsolidation pressures could develop this way in the Northern North Sea sediments. However, there is no reason to expect higher preconsolidation pressures resulting from subsea permafrost than terrestrial permafrost; in fact the pressures may be less because of the higher salinity of the pore water.

In the Beaufort Sea, preconsolidation pressures between 90 and 1850 kPa in 12m of marine sediments are ascribed to the effects of freezing (Sellmann and Chamberlain, 1979). It is interesting to note that in this area the heavily overconsolidated samples occur at isolated locations and are associated with strengths less than 260 kPa.

(iv) Subaerial weathering - dessication

Both Milling (1975) and Holmes (1977) refer to the possibility of subaerial weathering. The likelihood of this is, of course, closely related to the changes in sea depth which have occurred since the last glaciation. The previously quoted estimates of a lowering of 120m compared with the present depth of 150m would suggest that only the shallower areas of sea bed will have been subject to dessication and to overconsolidation by this means.

(v) Sand wave activity

Although there is little evidence of sand wave activity at present in the area of interest, movement of sands may have taken place when sea depths were lower and current velocities higher. The passage of a sand wave could have produced a small amount of over-consolidation.

(vi) Wave loading

Bjerrum (1973) has ascribed the very dense condition of the sand in areas of the Northern North Sea to the continual stress reversals to which it is subjected by wave action. Similar sands deposited in relatively calm waters in the Norwegian Fjords are normally found in a very loose condition. Part of the compaction is considered to result from the surface shear stresses exerted by waves and by currents.

Reference was made in Chapter 2 to the sea bed pressure fluctuations which accompany the passage of a surface wave and to the stress changes at depth associated with the pressure fluctuations. It was suggested that these stress changes form a part of the stress history of soil below the sea bed and it was noted that for a fixed sea depth, the maximum pressure and maximum stress changes at any depth would vary with time.

As discussed in this Chapter, sea depth has itself changed in recent time, following an uncertain path, such as illustrated in Fig. 3.6.

For sea depths below a critical value, both wave height, H , and wave length, L , become a function of sea depth, d . The maximum value of sea bed pressure, $\Delta p_{\max}$, being a function of H , L and d , will also have changed with rising sea level and hence with time, adding a new dimension to the stress history.

The exact manner in which the maximum height, $H_{\max}$, and wave length, $L_{\max}$, and hence sea bed pressure, $\Delta p_{\max}$, will have changed

with sea depth cannot be determined. It will have taken the form indicated in Fig. 3.7. This plot has been derived by assuming that, as the sea depth has increased, the maximum wind speed and fetch which operate at present have applied, i.e. the wind speed and fetch which give rise to $H_{\max}$ of 30m, $L_{\max}$ of 410m, $T_{\max}$ of 15.85 secs in a sea depth of 150m have been used.

The maximum stress changes with depth below sea bed associated with the passage of surface waves will also have varied as sea level has risen. Assuming the variations in $H_{\max}$, $L_{\max}$ and hence $p_{\max}$ with d shown in Fig. 3.7 then the variation of $((\Delta\sigma_1 - \Delta\sigma_3)/2)_{\max}$ with sea depth could be found for a number of different depths below sea bed; the results are presented in Fig. 3.8.

At any depth below sea bed, it is apparent that $(\Delta\sigma_1 - \Delta\sigma_3)_{\max}$ reached a peak value at a certain sea depth and has since decreased. The variation of $(\Delta\sigma_1 - \Delta\sigma_3)_{\max}$ and of $(\Delta\sigma_1 - \Delta\sigma_3)_{\max}/2\sigma_v'$ with depth is shown in Fig. 3.9. It is apparent that in the upper 15 to 25m, the ratio of $((\Delta\sigma_1 - \Delta\sigma_3)/2\sigma_v')_{\max}$ reaches a significant value when compared to the ratio of c_u/σ_v' for a normally consolidated soil, of, say, 0.25.

Discussion

The events which may have contributed to the stress history of the sediments in the Northern North Sea involve, in some cases, different stress paths. Referring to Fig. 3.10, path A may be associated with erosion and re-deposition or ice-loading, path B with wave compaction. While the processes of dessication and freezing may imply a stress path similar to A, there must be a departure from this path when the soil becomes discontinuous, i.e. shrinkage cracks form or reticulate ice develops.

In view of the stress path dependency of soil behaviour, then important differences in, say, stiffness, response to cyclic loading, etc. can be expected for soil in situ at the same water content and vertical effective stress, σ_v' , but which has followed a different

stress history. Important differences in horizontal effective stress, σ_h' , are also likely.

The most probable stress history of the near surface soils is considered further in Chapter 12.

3.5 INVESTIGATION OF SOIL COMPOSITION, GEOCHEMISTRY AND FABRIC

Introduction

At the time of carrying out the initial review of existing information on the Northern North Sea (1974/75), it was evident that there was little published in a detailed form on the nature and geotechnical properties of the clays, particularly at the sites where gravity structures were being proposed. At these sites the soft recent clays found at the Forties Field and beneath the areas of superficial mud shown in Fig. 3.2 are absent. By definition, they are suitable as sites for gravity structures by virtue of the stiff clays and dense sands present both at the surface and at depth.

In view of this lack of data an investigation was undertaken to determine, for the range of clays present, those features which, together with stress history, mode and environment of deposition and temperature, determine engineering behaviour, viz. composition, mineralogy, porewater chemistry and macro- and micro-fabric. Special emphasis was placed on establishing a typical prototype soil and its mode of deposition which could be matched subsequently in laboratory prepared soil samples. It was hoped that the investigation would provide further information on sedimentological and stress history; it was felt that fabric might hold the key to understanding these processes. Certainly, it was important to determine fabric in order to extrapolate from element to full-scale behaviour. It appeared that a detailed petrological examination of the coarser material was unlikely to provide useful information on depositional environment and so was not pursued; however, the opportunity was taken to examine in detail any marine fossils and their favoured habitat.

Oil companies and the Institute of Geological Sciences (IGS) were approached and a number of organisations co-operated in making available borehole logs and soil samples. Fig. 3.11 shows the location of sites for which logs have been provided; gravity structures have been placed subsequently at all but two of the sites. Intact and disturbed soil samples were made available from Ninian and Statfjord; one intact sample was provided from the IGS boring, SIN 33, made through a pockmark at Forties; more recently intact samples have been obtained from Magnus.

Soil Composition

The borehole logs, together with the data on classification testing contained in them, were used primarily to establish the range in soil grading and plasticity.

The analysis of the data should not be considered as rigorous. In the first place the depths penetrated in the borings varied considerably so there is a bias towards material in the top 30 - 40m. Secondly, the methods used for particle size analysis and measurement of plasticity were not consistent, the pretreatment and the effects of lamination varying in the case of the former and the chemistry of the pore water varying in the latter. The conclusions reached should only be regarded as applying to the area for which logs were made available.

(a) Grading

On the basis of grading curves, it was found to be possible to divide the clays into four groups, A - D, each with its own distinctive grading envelope; these envelopes are shown in Fig. 3.12.

(b) Plasticity

The plasticity indices of samples in each group were examined, both on an individual location basis and on a cumulative basis; the cumulative data is shown in histogram form in Fig. 3.13 and a typical Casagrande Plasticity Chart is shown for one of the locations in Fig. 3.14; all samples plot above the A-line.

There is little overlap in terms of plasticity between the groups suggesting that:-

- (i) the subdivisions based on grading are well-founded, and
- (ii) there is similarity in mineralogy between the groups; this is confirmed by
 - (a) the activity for each group, which is listed, together with the plasticity data, in Table 3.7, and which can be assessed from activity charts presented in Fig. 3.15 for different sites, and
 - (b) data on mineralogy presented below.

Soils in groups A and B classify as sandy clays, those in C and D as silty clays.

Clay Mineralogy

The mineralogy of five of the clay samples from the Northern North Sea was determined by X-ray diffraction techniques; these give a qualitative identification of the crystalline minerals, both clay and non-clay, that are present. The results are summarised in Table 3.8.

All samples were similar and in each illite was found to be the major clay mineral with varying proportions of chlorite, kaolin, calcite and smectite. These clay minerals are typical of terrestrial glacial deposits in Europe; no unusual or 'problem' clay minerals were found. The data is consistent with the activity charts in Fig. 3.15.

Data on mineralogy has been published by, for example, Løken (1976) and Milling (1975) which confirms the types and proportions given in Table 3.8.

Pore-water Chemistry

The pore-water chemistry of seven samples from the Ninian field and of the sample from the IGS boring, SLN 33, was analysed using the following techniques:-

- (i) Chloride: titration by the classical Mohr method using standard silver nitrate solution.
- (ii) Sodium and potassium: flame emission spectrophotometry.
- (iii) Calcium and magnesium: flame activated atomic absorption spectrophotometry.
- (iv) Sulphates: classical gravimetry, recovery as barium sulphate.

The results are summarised in Table 3.9. Data on pore water chemistry of soil samples from the Northern North Sea does not appear to have been published elsewhere.

Of particular interest are the chloride and sodium concentrations. The values are relatively high, indicative of deposition in a marine environment with probable dilution by glacial meltwater. They are remarkably consistent, both with depth at the Ninian site and between the two sites. The lack of variation with depth between 10 and 64m suggests no major change in depositional environment and no significant diffusion subsequently.

There are no anomalies in the profile of geochemical data for Ninian. Higher concentrations of potassium, calcium and magnesium and of sulphates were found in the sample from 64m depth; this sample immediately underlies a 16m layer of sand which suggests that it may have a different origin from the clays above; the high sulphates content indicates a lack of sulphate-reducing bacteria. The SLN 33 sample is similar in all respects to samples above 36m at Ninian.

Soil Fabric

Techniques

Soil fabric has been observed at different levels, adopting the following techniques:-

- (i) radiography of the whole core, before and after extrusion, the split core and, in some cases, slices from the core;
- (ii) visual and photographic observation of the split cores;
- (iii) optical microscopy of thin sections formed by Carbowax impregnation of soil specimens;
- (iv) scanning electron microscopy of soil specimens.

The methods used are summarised below.

(a) Radiography

Scanning of unopened or partly opened sediment cores by X-ray radiography is now a routine procedure in oceanography. Pioneering work with the technique by Hamblin (1962), Bouma (1964) and Krinitszky (1970) has demonstrated its usefulness as a supplement to other methods in interpreting depositional environment, since it reveals details of primary and secondary structures which are not readily visible during direct observation. The technique can be used to produce stereo pairs.

The process is simple, rapid and, of course, non-destructive and non-disturbing; it involves transmitting X-rays through the sediment onto radiographic film, pre-sealed in a lightproof envelope. The film density (i.e. darkening) which results is a function of exposure time and energy of radiation arriving at the film. Gradients in density across a film reflect variations in absorption and transmission properties in the sediment produced by, inter alia, differences in composition, grain size, packing density, particle orientation, etc. Since X-rays follow straight paths, the image produced on the film is a projection of the absorption properties of all particles lying behind each other in the path of the rays. Film density also, then, depends on sample thickness.

Many sedimentation and structural features have been identified in radiographs. These include:-

- (i) Stratification/lamination/lithology. Absorption decreases with decrease in grain size and increase in water content so that on the radiographs clays appear darker than sands;
- (ii) bioturbation, i.e. disturbance by burrowing animals;
- (iii) fossil remains, e.g. shells and shell fragments in their undisturbed positions;
- (iv) organic matter, including root patterns;
- (v) concretions/secondary mineralization;
- (vi) erosion surfaces;
- (vii) dessication fractures;
- (viii) shear zones, fissuring and fractures;
- (ix) voids;
- (x) gas bubble and gas expansion structures;
- (xi) mechanical disturbances produced by coring;
- (xii) stone and gravel content, including their orientation and location relative to proposed test specimens.

In sedimentological studies, it is usual for slices, 1cm or less in thickness to be prepared for radiography. However, exposures can be made of cylindrical cores, either extruded or still encased in steel or plastic tubes. Even though considerable detail may be lost in the radiography of whole or even split cores, particularly if compensations are not provided for the cylindrical shapes of the cores, there are distinct advantages in scanning cores before extrusion, especially when selecting samples for mechanical and classification testing.

As a calibration of the technique, two samples, not from the North Sea, but with distinct sedimentary features were radiographed. The results are shown in Plate 3.1. In the upper part of the plate, an X-ray^{*1} of a split core of a glaciolacustrine laminated clay from Teesside is shown alongside a photograph and thin section of the soil. The ability of the X-ray to detect the fine laminae is well illustrated. In the lower part of the plate, an X-ray of a slice of London Clay is presented; fissuring in the clay is shown clearly.

The potential usefulness of X-radiography is convincingly demonstrated by the evidence presented in Plate 3.1. Since fabric can be identified non-destructively and prior to extruding a sample, the technique has obvious advantages as an adjunct to programming appropriate laboratory tests. This advantage is even greater in offshore investigations where the cost of sampling is high and the number of samples available is limited.

(b) Photography of Split Cores

Each core was split, logged and photographed at natural scale in colour and black and white.

(c) Optical Microscopy

Thin sections of samples from the North Sea were prepared using the technique described by Tchalenko (1967). They were viewed through an optical petrographic microscope with polarizer, analyser and camera attachment; magnifications up to ninety times were achieved. All observations were carried out with crossed polarised light and the stage carrying the thin section was rotated to give either maximum or minimum light intensity transmitted through the discontinuities.

*1 X-ray film has been used as a negative to produce the photographs; light areas in the photographs indicate materials with high absorption coefficients.

To take photographs of the whole thin section, with a magnification of two, a polarising camera set-up was used; this comprised a source light beneath a polarising sheet on which the thin section was mounted, an analyser and the camera. These combined techniques permitted a study to be made of silt and sand grain shapes, texture, weathering and mineralogy.

(d) Scanning Electron Microscopy

Samples were prepared by air drying and fracturing before being coated under vacuum with gold palladium. They were mounted on stubs using 'Durofix' and the sides of the 1cm³ samples were painted with silver before being viewed in a Cambridge Stereoscan 600 scanning electron microscope, capable of magnification up to fifty thousand times.

Results of Macrofabric Study

The samples made available from the Ninian Field could be divided into 6 types in terms of macrofabric. Examples of each macrofabric are shown in Plates 3.2 and 3.3 alongside an X ray and short description. The distribution of the fabric types with depth is shown in Fig. 3.16 together with the commercial borehole log. Depths at which changes in fabric occur correspond approximately to changes in description of soil type in the log; the inappropriateness of some of the sample descriptions is apparent.

Macrofabrics similar to some of those shown in Plates 3.2 and 3.3 have been observed in samples obtained from both Statfjord and Magnus; there were also variants on these fabrics. In Plate 3.4 examples are shown of Fabric Types 2, 3, 4 and 5 seen in samples from Statfjord. Fabric Type 2 was the most extensive at Statfjord, extending more-or-less without interruption from sea-bed to a depth of 25m, to be succeeded by Types 3 and 5, which were interbedded between 25m and 35m, and then a continuous bed of Type 5 to 55m.

Plate 3.5 shows examples of Fabric Types 1, 2, 3 and 4 which were recognised at Magnus. Sediments with Fabric Type 3 appear to be the

most extensive below 20m; occasionally the fabric becomes more platy and tends towards that of Fabric Type 4. Clays in the upper 20m generally exhibit Fabric Type 2.

These fabric types are not restricted to the area in the Northern North Sea for which samples were made available. In Plate 3.6 a selection of the photographs given to the author by IGS have been reproduced; the samples were taken from borehole SLN33, bored through a pockmark in the Forties Field. Examples of Fabric Types 3 and 4 are indicated. Also of interest are:-

- (i) the vesicular muddy sand sample which gives evidence of the gas release problem discussed in Chapter 4, and
- (ii) the well-defined fissuring evident in some samples. (No evidence for fissuring was found in the samples which were examined from other sites.)

Published descriptions of soil types from other sites in the Northern North Sea are consistent with categories in this classification of fabric types. As an example, Durning et al (1978) describe two clay soil types at Heather field; one appears to be similar to Fabric Type 6 with successive layers of silty sand, a few centimetres to 2m thick, interbedded with a sandy silty clay; the other, a sandy silty clay with low clay content and plasticity index which is similar to soils with Fabric Types 2 and 5.

Results of Microfabric Study

The microfabrics associated with five of the macrofabrics which have been identified are illustrated in Plates 3.7 to 3.15.

The essential features of each fabric type at macro- and micro-level are summarised below, observations being given, where possible, on depositional environment, placement water content and post-depositional changes.

Fabric Type 1 (Plates 3.2 and 3.5)

This fabric was the least widespread in the study that was made. In the sediment there are harder, drier lumps within a slightly softer clay matrix. There are numerous clasts and broken shells. In thin section, sand particles are seen to be angular. A contorted fabric with poor sorting is evident suggesting turbulence during rapid deposition, which would be consistent with glacial deposition from grounded ice as an orthotill (Table 3.4).

Fabric Type 2 (Plates 3.2, 3.4, 3.5, 3.7, 3.8 and 3.9)

This fabric appears to be the most widespread among the Northern North Sea sediments which have been examined. It is found in soils which exhibit a wide range of plasticities (LL 21% to 51%, PL 13% to 22%) and transcends the classification into groups based on grading and plasticity. It is not a fabric routinely encountered on land.

On a macro-scale the fabric is frequently featureless but occasionally shows an apparent stratification. The stratification is not evident in X-rays or thin section; it is suggested that the stratification results from mechanical disturbance, in particular from tension which may develop when using a short jarring link. Similar stratification has been observed by the writer in samples of London Clay. There is no evidence of freeze-thaw features in the macrofabric and no 'drag structures' (Chapter 4) which would indicate remoulding or subglacial shearing at low water content.

The proportion of quartz evident in thin section varies, of course, with plasticity. In the clays of lowest plasticity the sand is very densely packed (note the compatibility of sand grains in Plate 3.7(b)) and the clay forms a thin skin around the grains (e.g. Plate 3.8(a)). With increasing plasticity there is increasing sparseness of quartz and increasing thickness to the clay skin around the sand grains - there is less distinction between the clay at the contacts between the sand grains and the clay filling the voids. At the highest plasticity there is some evidence of banding (e.g. Plate 3.7(g)) which is not the same as varving in glaciolacustrine deposits. The quartz grains are angular and sub-angular.

The relatively uniform distribution of coarser material throughout the matrix, seen in both the X-rays and thin section, suggests a nearly continuous "rain" of unsorted material during accumulation of the sediment, such as might occur from ice rafts. Griggs and Kulm (1969) suggest ice rafting as a depositional mechanism for material fitting a similar description, i.e. a uniform distribution of sand grains and pebbles in a silty clay matrix. Furthermore, the sediment is similar to material sampled after recent deposition in Icelandic fjords (Boulton, 1976, pers. comm.); material from the same source but deposited in an inland lake shows a different fabric and points to the importance of a saline environment. The intact shells, the possible bioturbation and the organic content are all consistent with glacial marine sedimentation. Relatively fast sedimentation rates could be anticipated leading to gas generation. The high organic content would rule out deposition beneath permanent sea-pack ice since this is associated with low organic productivity; iceberg activity is indicated. This would be consistent with the low silt contents since the bergs would account for the presence of sand sizes and above while the clays may have travelled horizontally in meltwater streams from which the silt had previously settled out. The featureless macrofabric would indicate long still periods of deposition.

Sediments with such a glaciomarine origin could represent the most recent glacial deposits at a site, being associated with recession of the main ice sheets. In this case they may not have been subject to ice loading and possibly not to permafrost; an alternative explanation for their high strengths and apparent overconsolidation is offered in Chapter 12; in this respect the very high density of the clay matrix (e.g. Plate 3.9(f)) is to be noted. Such sediments can, of course, be buried at a site as a result of subsequent re-advances of ice sheets.

Fabric Type 3 (Plates 3.2, 3.4, 3.5, 3.6, 3.10 and 3.11)

Although having a very similar microfabric to Fabric Type 2, this sediment shows a distinctly different distribution of quartz in thin section and a different macrofabric. The quartz may be banded with the

bands exhibiting contortions (Plate 3.10(c)) while the silt-sized material often occupies pockets. Within the coarser quartz bands there is orientation of the longer grains (Plate 3.10(d)). The clay is oriented and there is some evidence of fine shear surfaces.

The shear surfaces, the orientation of the clay and long quartz grains and the general pattern of the fabric suggest 'flow' or glacial shearing. The absence of 'drag structures' (Chapter 4) would rule out shearing/remoulding at low water content. Origin as a flow till is suggested. The foliation evident at natural scale has been reported previously for flow tills (Boulton, 1972).

An alternative explanation for the foliation at natural scale lies in the effects of freeze-thaw leading to ice segregation.

Gas exsolution features are evident in the coarser areas in thin section (see the small black circular holes in, for example, Plate 3.10(c))^{*1}.

Fabric Type 4 (Plates 3.3, 3.4, 3.5, 3.6, 3.11 and 3.12)

This distinctive platey or 'fishscale' fabric has been observed in samples from Ninian, Statfjord, Magnus and the IGS boring at Forties, yet it is rarely encountered on land. In X-rays the peds are discernible in the Ninian sample as changes in tone and, therefore, indicative of changes in water content; they are barely discernible in X-rays of the Forties sample (Plate 3.6). In thin sections of samples from different sources which show this fabric there is a notable feature, depicted in Fig. 3.17: clay peds appear to be coated by a thin layer of quartz particles (Plates 3.12(c), (d), (f) and (h)); the size of the clay peds varies. Within the peds, there is marked orientation in the clay and the orientation varies from ped to ped. There is little quartz within the peds.

Gas exsolution features (see the small black holes in Plates 3.12(b), (c), (d) and (h)) are prevalent. There is evidence, too, of concentrated organic content (e.g. Plate 3.12(e)) and, occasionally, of calcite (Plate 3.11(h)). High organic

*1 Jardine (1982, pers. comm.) pointed out these features and suggested they represented gas bubbles.

concentrations and calcite bonding often seem to occur in combination.)

It has been suggested that this fabric has developed as a result of ground freezing (e.g. Marsland et al, 1982). The observations in X-ray and thin section indicate, however, that the peds were developed at the time of deposition with the quartz coating possibly being picked up at the sea bed. (The quartz coating also rules out its origin as an unloading feature.) The pedal fabric may well be related to the unstable conditions referred to previously at a fresh-water, saline-water interface which lead to flocculation; it is of interest that the ped fabric has been retained despite the subsequent stress changes, suggesting a low initial water content for the peds. (The silt/sand coating may also have contributed to the structure being retained.) A glaciomarine origin is consistent with the presence of dropstones in the sediment.

Fabric Type 5 (Plates 3.3, 3.4 and 3.13)

This fabric is very similar to that of Type 2, being distinguished at macro-scale by the much higher sand content. The sediment is distinctly organic, indicating a fast rate of deposition. A similar depositional mode and environment to those for Type 2 can be envisaged, possibly with stronger currents being associated with the high fine sand content and shells being preserved in an upright position.

Fabric Type 6 (Plates 3.3, 3.14, and 3.15)

The layering and sand rippling which can be observed at natural scale in this fabric type are indicative of a marine depositional environment. The thickness of the clay layers indicates the possibility of large annual rates of sedimentation which would be consistent with glacial marine sedimentation. The sedimentary features at the sand/clay interface appear like bearing capacity failures suggesting the the clay was at a high liquidity index when the sand was deposited. 'Roll' features suggest small-scale slumping and, in thin sections, shears can be seen running across the bedding.

Within the clay layers there is no strong orientation of the clay; quartz grains occur either generally at varying densities in a sandy clay seam or in undulating bands or lenses. Quartz grains exhibit a range of shapes from angular to rounded. Gas exsolution features can be seen in thin section (e.g. Plates 3.14(e) and (f)) and possibly in electron micrographs (e.g. Plates 3.15(a) and (c)).

There is a distinct banding within the clay beds (Plate 3.15); the denser clay which occupies the central part of Plate 3.15(a) is shown in detail in Plate 3.15(b); two forms of bonding can be observed which are shown in greater detail in Plates 3.15(e) and (f). The long 'string-like' bonding is thought to be chlorite; the more general amorphous bond is thought to be calcite. Both forms of bonding may be associated with the conditions conducive to calcium carbonate bonding which were described in Section 3.3. (Similar bonding has been noted only in Fabric Type 4.) The coarser clay to the left and right of Plate 3.15(a) is shown in detail in Plates 3.15(c) and (d); there appears to be much less bonding (although it is not absent) but the silt is highly angular.

It should be re-emphasised that this set of fabric types is not exhaustive and that there will be further variants of each type. However, it is worth noting that similar fabrics to Types 2, 4 and 6 are not widely reported for terrestrial soils.

Marine Fossils

Palmer (1976, pers. comm.) has inspected the intact shells and indicated that they originate from animals which exist completely buried or partially buried in clayey sediments which are incoherent enough for small, not very powerful, animals to push aside; this would suggest a high liquidity index for the sediments originally. Furthermore, the shell types are indicative of fairly quiet conditions with oxygen available; not all the shells are indicative of cold water conditions.

Summary

Evidence re depositional mode and environment

The following general observations may be made:-

- (i) For all fabric types, with the possible exception of Fabric Type 1, a marine or glaciomarine depositional environment has been suggested. Table 3.10 summarises the suggested depositional modes and environments for each fabric type.
- (ii) Only Fabric Type 1 exhibits fabric features which might be interpreted as resulting from lodgement deposition. However, it does not show a platy fabric, which has been reported for lodgement tills (Boulton and Paul, 1976), or 'drag' structures which will be suggested in Chapter 4 as a good indicator of subglacial plastering; an orthotill is suggested. (It is of interest to note that in the grading envelopes summarised in Fig. 3.12, none is typical of terrestrial lodgement tills.) Nevertheless, tills deposited by grounded ice as lodgement have been suggested by many authors and may well exist outside the areas which have been examined by the writer.
- (iii) All samples were grey when extruded, oxidising rapidly to grey-brown; this might suggest that the material had not previously been exposed to air, i.e. having been accumulated in a non-oxidising environment.
- (iv) Fabric types do not correlate with classifications based on plasticity and grading demonstrating the importance of depositional mode and environment. It will be shown that fabric differences in soils of the same composition (which result from different depositional modes) influence engineering behaviour (Chapters 4 and 12).
- (v) There is good evidence that a number of sediment types have involved placement and consolidation from relatively high water

contents (Fabric Types 2, 5, 6). Others may have involved placement of at least components of the sediment at relatively low water content (Fabric Types 1 and 4).

Evidence re post-depositional processes
and stress history.

- (i) In the fabric study no evidence has been found for the effects of weathering.
- (ii) Fabric Type 4 exhibits features which might be caused by ground freezing. Smart and Tovey (1981) present a scanning electron micrograph which shows a ped fabric caused by freezing and report that freezing increases the density of the matrix clay. Williams (1968) describes flaky macrofabrics and separation of fine and coarse particles being associated with freezing-thawing cycles. However, simple experiments have shown that in each sediment type which was distinguished on the basis of grading and plasticity, freezing leads to a blocky macrofabric in which there has been separation by ice lenses. (At high silt contents the fabric became more platey.) The fabrics developed by freezing depended to some extent on the liquidity index at the time of freezing but did not resemble those of Fabric Type 4. The general absence of a blocky fabric indicates that freeze-thaw has not been a major component in the history of most of the sediments.
- (iii) Bonding has been observed in Fabric Types 4 and 6.
- (iv) Unloading (stress-relief) features and fissuring are not evident in the six fabric types, with the possible exception of Type 3. However, samples showing lithnic surfaces, consistent with heavy overconsolidation, were located in the IGS boring at Forties (Plate 3.6). Overconsolidation by erosion would tie up with the erosion surfaces which have been reported.

- (v) A post-depositional process associated with the sampling has been gas exsolution which, in these stiff deposits, does not appear to have led to significant fabric disturbance, but to the formation of small spherical voids.

3.6 DISTRIBUTION OF SEDIMENTS

The distribution of sediments on a regional scale could not be assessed by examining a series of borehole logs in isolation. Some evidence on the form of this variation is presented in Fig. 3.4. However, a clear feature which did emerge was the rapid lateral and vertical variation in sediment type which occurred at some sites. This is illustrated in Fig. 3.18 which shows alongside each other a series of simplified borehole logs for site B (Fig. 3.11) which cover a horizontal distance of only 500m. Sediment types have been subdivided into the four categories A - D based on plasticity and grading. Similar rapid lateral variations in sediment type have been reported by Caston (1974). Rapid vertical variations in sediment type are indicated in the borehole log from the same site shown in Fig. 3.16.

As discussed in Section 3.3., such variability can arise in a glaciomarine environment at the ice front. The heterogeneity presents problems of idealisation in foundation analysis and requires particular care in the design of gravity structure foundations.

3.7 UNDRAINED STRENGTH OF SEDIMENTS IN THE NORTHERN NORTH SEA

Published information on undrained strength of the sediments at the sites of gravity platforms in the Northern North Sea was not examined in detail because of its questionable reliability; this results from both the effects of sampling, discussed in Chapter 4, and from the methods used in its measurement, including pocket penetrometers and torvane (Andresen et al, 1979). Nevertheless, a number of important features were noted which must be accounted for by the origin, post-depositional processes and consolidation history of the sediments. Two of the features were:-

- (i) a characteristic profile of undrained shear strength with depth, and
- (ii) the occurrence of very high strengths near the surface at several locations.

Figure 3.19 summarises the undrained strength profiles at a number of platform sites in the Northern North Sea. In most cases, the profiles indicate a higher strength crust; strengths increase to a maximum at a depth generally between 5m and 10m; thereafter they remain constant over a short depth, decreasing to a minimum, generally between 10m and 20m.

This profile of strength can be compared in Fig. 3.19 with that for a normally consolidated deposit for which a C_u/σ'_v of 0.25 has been assumed. High overconsolidation ratios, OCRs, are implied near the surface. (Although data from oedometer tests has been examined, no supporting conclusions on OCR could be drawn, preconsolidation pressures being poorly defined.) The soils appear to approach a normally consolidated condition at some sites at depths of 40m to 50m.

The most notable example of very high undrained strengths being measured in samples taken from near the sea bed is the Heather field where C_u up to 800 kPa is reported at depths of around 5m (Durning et al 1978; Parry, 1978). These strengths do not generally correlate well with their liquidity index i.e. C_u is not given by

$$C_u = 170e^{(-4.6LI)} \text{ kPa} \quad (\text{Wroth, 1979}).$$

This can be inferred from data given by Durning et al (1978) and is confirmed also by data tabulated by Wroth (1979) for other North Sea sediments in which C_u predicted by the above expression underestimates the measured C_u .

The highest near-surface strengths appear to be associated with soils which show Fabric Type 2 and, therefore, of low plasticity. The clearest evidence for this is provided by data from Magnus where strengths of up to 400 kPa have been measured in this type of soil in

the top 15m. Indirect evidence can be found in data published for the Heather field (Durning et al, 1978). Soils having undrained shear strengths in excess of 800 kPa are described as well-graded with a clay content between 10% and 20%, a plastic limit between 12% and 14% and a liquid limit between 28% and 30%.

3.8 SUMMARY

Sediments in the Northern North Sea are glacial, glacial marine or marine in origin. The Quaternary sediments are of variable thickness, reaching a maximum of 900m; they are often very stiff to hard close to the sea bed. Recent (Holocene) marine sediments are up to 20m thick and occupy deeper areas (up to 150m sea depth); they are usually soft to firm. Open and buried channels exist in the Quaternary sediments.

The widespread occurrence of glacial marine sediments is notable because of lack of experience with them as foundation soils and because, compared with terrestrial glacial sediments, differences in mode and environment of deposition and post-depositional processes can be expected to lead to differences in engineering behaviour. Glacial marine and terrestrial glacial deposits differ in composition, plasticity (the former tend to be of low PI, less well-graded than terrestrial tills with low silt contents), fabric, pore water chemistry, placement water content and stress history. In glaciomarine sediments a mechanism for bonding exists and bonding has been observed.

Glacial marine sediments might have been anticipated from a comparison of present sea depths and estimates of sea lowering. They have been recognised by the presence of dropstones and intact shells, by characteristic features (or lack of features) in their fabric and by the occasional occurrence of calcium carbonate bonding which would have been associated with the formation of dense brines. The high salinity of the pore water is consistent with marine or glacial marine sedimentation.

Based on plasticity and grading alone, four sediment types have been distinguished. These have plasticity indices above and below

25% so that both brittle and non-brittle behaviour can be expected; differences in behaviour are likely to extend to cyclic response. Rapid lateral and vertical changes in sediment type can occur; in glaciomarine sediments these changes are related to the varying position of the ice front; methods of analysis dealing with heterogeneity are an obvious requirement.

Six fabric types have been recognised; these do not correlate well with the subdivisions based on plasticity and grading, but appear to reflect more closely the mode and environment of deposition and post-depositional processes; fabric studies have proved useful in identifying these processes. The variety in fabric, and its effect on engineering behaviour, adds to the heterogeneity.

Fabric Type 2 appears to be widespread; it is associated with clays of low plasticity and low silt content and often has a high undrained shear strength; it does not appear to have a land-based counterpart. This fabric will be the prototype for reconstruction.

The mineralogy of the sediments is consistent between the sediment and fabric types and shows that no unusual types or "difficult" clay minerals are included.

Much of the sediment has been deposited through water; this is supported by evidence from the fabric studies which show rippling, bedding and laminations. Rates of sedimentation, especially in a glaciomarine environment, were rapid; subsequent organic decay will have led to gas-saturated pore water and to free gas. Features related to gas exsolution have been observed in the fabrics.

Many of the sediments have been consolidated from a relatively high water content. This is revealed by fabric features at the interface between sands and clays and by bioturbation in the radiographs; it is consistent with the environment required by the remains of marine life which have been identified.

The sediments have been subject to a complex stress history in which subaerial drying, ice-loading, ground freezing, erosion-redeposition (erosion features at two levels have been identified) may have played a part. It is suggested that wave loading has been a major component of the stress history, particularly as its intensity has changed with changing sea depth; this has been demonstrated by calculation of stress changes caused by wave loading. It will be suggested (Chapter 12) that wave loading is responsible, at least in part, for the high undrained strength of some of the near surface soils and the characteristic strength profile which has been described. Fabric features indicate that in some of the sediments freezing may have occurred; fissuring and jointing associated with ice unloading and dessication are not widespread.

Erosion of the shallower deposits with sedimentation in the deeper water has continued since glacial times but only at a slow rate recently. There is no evidence of sand wave activity at present.

<u>Unit Name</u>		<u>Soil Description</u>	<u>Erosional and Other Features</u>
Witch Ground Beds	0-20m	Very soft to firm sandy and silty clays, well laminated. Carbonaceous.	Unfilled and filled pockmarks. Erosional surface in parts. Holocene infilling to sea bed hollows.
Upper Channel Deposits	Varies	Silty fine sands and soft to firm clays.	Infill to Upper Channels*
Marr Bank Beds (Southern & Western Part) or Upper Swatchway Beds	15-20m Varies	Sands and bedded clays. Firm to stiff silty clays with occasional clay-supported dropstones. Gassy.	Surface channelling. Shallow arctic marine adjacent to ice sheet. Shallow marine deposition from floating ice in hollows formed in Lower Swatchway Beds.
Fisher Beds (Eastern & Southern Part) or Lower Swatchway Beds (Northern Part)	 20-70m	Layered silty sands and very stiff to hard sandy and silty clays. Gassy. Stiff to hard gravelly clays with boulders.	Deep channels partially filled with soft silty clays or open (e.g. Devil's Hole), 240m deep (Upper Channels). Ice tectonics and confused bedding. Ice push and morainic features especially at junction with Fisher Beds. Confused bedding. Form banks and ridges at sea bed. Shallow arctic marine, glacio-marine, glacio-fluvial.
Lower Channel Deposits	Varies	Gravel, sand and clays.	Infill to Lower Channels.
Aberdeen Ground Beds	180-300m	Stiff to very stiff silty and sandy clays, sometimes fissured, with occasional dropstones, and thin layers of sands, sometimes gasified. Carbonaceous.	Eroded surface with incised channels, 200m deep (Lower Channels). Channels infilled with fine sands, silts and clays. Shallow arctic marine.
Basal Beds	50-100m	Not examined in detail.	*under brackish arctic conditions

Table 3.1 Quaternary succession in the North Sea, 56°N - 58°N
(Based on Holmes, 1977, Eden et al, 1977, McCave et al, 1977)

Approximate Time BP

Event

2.5m/2m	Deposition of Basal Beds and subsequent partial erosion during Anglian/Wolstonian sequence of glacial events or Early Devensian.
↓	
130,000/72,000	Deposition of Aberdeen Ground Beds in shallow cold water associated with ice during Early Devensian.
↓	
36,000	Large scale erosion in Late Devensian by action of ice and water leading to formation of Lower Channels in present surface of Aberdeen Ground Beds. Subsequent infilling of channels by sands and clays with boulders.
↓	
23,200	Deposition of Fisher Beds in shallow water arctic marine conditions and of Lower Swatchway Beds as moraines or till during Late Devensian.
↓	
23,000	Major erosion accompanying re-advance of ice, marked by Swatchway Channels and Upper Channels.
↓	
22,000	Deposition of arctic sands (Marr Bank Beds) and of marine clays with dropstones (Upper Swatchway Beds) in shallow water between two ice fronts. Channel formation and infill in proximity of ice sheets. Marine and subaerial erosion of islands of Lower Swatchway Beds.
↓	
18,000	Ice withdrawal accompanied by infill of Upper Channels and deposition of muds (Witch Ground Beds).
↓	
10,300	Flandrian deposition in estuaries and hollows with little net deposition in Central North Sea. Development of widespread semi-mobile layer derived from glacial detritus with significant biogenic content.
↓	
Present	

Table 3.2 A suggested Quaternary history of the Central North Sea
(Based on Eden, Holmes and Fannin, 1977)

	<u>Wet base glaciers</u>	<u>Dry base glaciers</u>
Grounded Shelf Zone	Till deposited under conditions of low vertical load and large horizontal shear giving rise to dragged and rolled structures. Meltwater sediments and submarine slumping at point of flotation. Interbedding when glacier temporarily retreats and floats.	Little sedimentation. No flow tills. Sediment carried to bouyancy line.
Floating Shelf Zone	Rapid sedimentation direct from floating ice and from meltwater density currents. Laminar, thickly bedded and well developed stratification in finely ground rocks and minerals of sand to clay grade. Unfossiliferous sediments because of lack of light. Large volumes of meltwater discharged leading to reduced salinity.	Sediment largely trapped by sea water freezing on to base. No meltwater currents. Poor stratification with many erratics. Dense brines formed leading to precipitation of calcites and sulphates and to thick-shelled fossils.
Iceberg Zone	Little sediment left in calving icebergs. Fossiliferous sediments now possible.	Sediment rich icebergs. Fossiliferous sediments now possible.

Table 3.3 Sedimentary processes associated with wet and dry base glaciers in a glacial marine environment

<u>Sediment Type</u>	<u>Sediment Description</u>	<u>Deposition</u>
Orthotills	Non-stratified, non-sorted clastic debris of sand and gravel in silt and clay matrix which forms 50 - 75% of total sediment. Clasts generally <5cm and no preferred orientation. Marine fossils absent.	Rapid deposition by disintegration of large grounded ice shelves. (Lack of bedding implies absence of marine currents.)
Residual Paratills	Non-stratified, poorly sorted gravelly lags with sandy matrix.	Sediments shed by floating ice shelves and exposed to marine currents which remove finer fraction.
Compound Paratills	Stratified with fewer clasts than Orthotills. Matrix comprises more than 80% of total sediment and is sorted. Elongate pebbles parallel to bedding.	Fine sediments winnowed from lag deposits and mixed with debris from floating ice. Stratification increases and clast density decreases with increasing distance from ice front.
Subglacial Channel and Distributary Fan Deposits	Gravels and sands which are sorted and may contain dropstones.	Subglacial meltwater deposits.

Table 3.4 Classification of glaciomarine sediments
(After Harland et al, 1966; Anderson and Clark, 1977).

Glacial advance	Dry base ice-shelf carves way onto continental shelf, eroding or consolidating material deposited in earlier floating shelf and iceberg zone. Sea water freezes onto base forming dense brines (salinity $>34.5\%$). Minimal sedimentation near point of flotation. Sedimentation in iceberg zone of paratills.
Stationary ice sheet or periodic advance and retreat	Sedimentation in floating shelf and iceberg zone of paratills.
Glacial retreat	Ice shelves become wet base and grounded ice becomes partially buoyant. Orthotills deposited in grounded shelf zone. Fresh water generated reducing salinity ($<34.5\%$). Sediment charged meltwater flows as density currents over shelf, reworking orthotills to form residual paratills and fines combine with sediments being deposited in floating shelf zones to form compound paratills. Eventually valley glaciers terminate at the sea with sediment charged icebergs calving off.
Isostatic uplift	Deposition of shallow water marine sediments.

Table 3.5 Typical sequence of events in glacial marine sedimentation

Terrestrial Till			Glacial Marine Till		
	Lodgement	Melt-out	Flow	Ortho-till	Residual Paratill
Grading*	Widely graded (Clay fraction < 30%) (often rock flour)	Sometimes lacking silt and clay	Variable because of segregation		Compound Paratill
Plasticity*	Low (PI < 25%)	Often non-plastic	Variable		Low silt content Higher clay content Sandy clays common
Placemnt	Low	High	High		Low to medium
Water Content					Variable. Blocks of low water content in higher water content matrix possible in floating shelf some.
Gas Content	Low	Low	Low		High
Depositional Mode and Environment	Slow accretion by ice plastering beneath active glacier (Subglacial)	Slow melting-out of stagnant ice brings suspended particles into contact	Accumulation on glacier surface, subject to flow as low strength mass (Supraglacial)		As lodgement or melt-out till
Post-depositional Processes	Freezing-thawing. Fluctuations in water table. Subsequent ice loading causing resolidation. Water percolation and wind action.	Drying	Drying. Disaggregation by stream erosion.		Winnowing by marine currents to leave coarse material in place.
Stress History	High effective consolidation pressures possible, from ice loading.	Sometimes normally consolidated. Usually lightly overconsolidated by drying and freezing.	Overconsolidated by subaerial drying.		Limited ice loading possible. Submarine erosion and redeposition and wave loading producing overconsolidation
Fabric	Platoy. Zones of parallel fabric bent around clasts. Non-stratified. Poorly sorted. Efficient packing.	Unsorted and highly disturbed. Open microfabric.	Stratified horizons. Sometimes foliated.		Angular grains in open framework. Poorly sorted. Contorted where slumping occurred.
Discontinuities	Shears and fissuring. Jointing associated with unloading.	Generally unjointed.	Shear surfaces and drying out cracks.		Shear surfaces where slumping involved. Absence of shrinkage and unloading joints.

*Depends on source rocks

Table 3.6 Differences between terrestrial and glacial marine tills which are likely to influence their engineering behaviour. (Based on Flint, 1971; McGown & Derbyshire, 1971; Boulton, 1972, 1975; Carey and Ahmed, 1961)

Sediment Type	Full Range				Mean Range			Mean Activity
	PL	LL	PI	% Clay	PL	LL	PI	
A	10/18	20/33	9/20	21/25	12/14	26/30	13/15	0.61
B	10/23	28/41	12/26	28/45	14/17	35/40	18/23	0.56
C	11/23	40/52	19/33	45/55	17/20	42/48	25/28	0.52
D	15/26	48/65	25/43	48/58	18/22	50/55	31/34	0.62

Table 3.7 Summary of clay soil types in Northern North Sea based on grading curves and Atterberg Limits.

Sample	Whole Rock			Clay Fraction		
Depth	Major (>30%)	Minor (10-30%)	Trace (<10%)	Major (>30%)	Minor (10-30%)	Trace (<10%)
Ninian 5.2A	Quartz	Calcite Felspar	Mica ⁺ Clay Minerals	Illite	Chlorite Kaolin Calcite	Smectite Felspar Quartz
Ninian 7B/37	Quartz	Felspar Calcite	Mica ⁺ Clay Minerals	Illite	Smectite Kaolin Chlorite Calcite	Felspar Quartz
Statfjord NB2 S3	Quartz	Felspar Calcite	Mica ⁺ Clay Minerals	Illite	Chlorite Kaolin Smectite Calcite	Felspar Quartz
Statfjord NB4 S4	Quartz	Felspar Calcite	Dolomite Mica ⁺ Clay Minerals	Illite	Chlorite Kaolin Calcite Smectite	Felspar Quartz
Statfjord BA6/S21	Quartz	Calcite Felspar Clay Minerals	Mica ⁺	Illite	Chlorite Kaolin Calcite	Smectite Felspar Quartz
⁺ Micas are dominantly of muscovite type but some biotite could also be present						

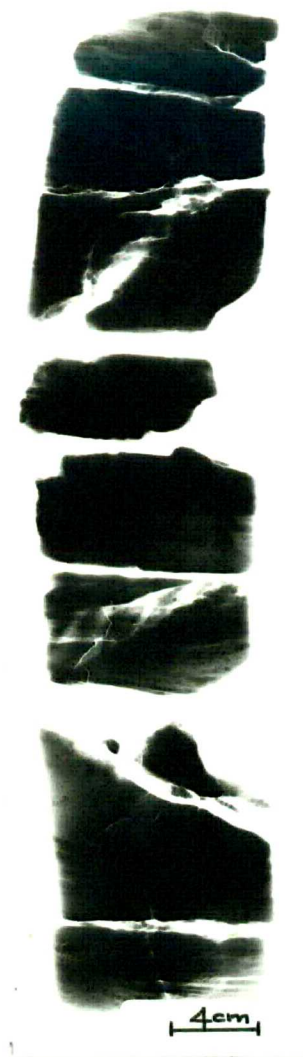
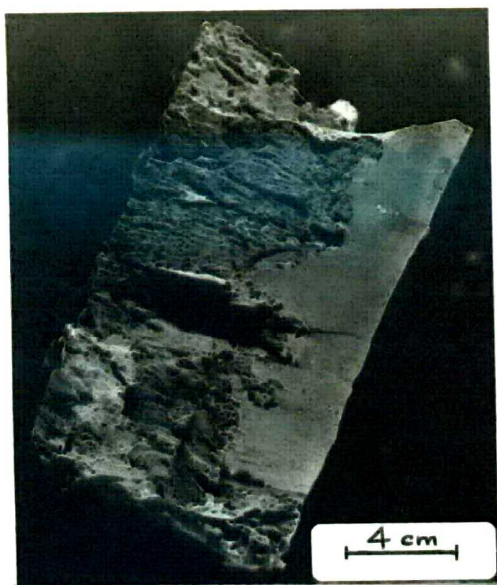
Table 3.8 Results of X-ray diffraction analyses of clay samples from Northern North Sea sites.

Sample Depth (m)	Cl ⁻ (g/l)	Na (g/l)	K (mg/l)	Ca (mg/l)	Mg (mg/l)	SO ₄ ²⁻ (g/l)	
Ninian	10 DB7/11	17.41	8.88	606.20	567.20	277.10	2.51
	10 DB7/11	17.84	8.84	616.33	642.19	314.60	2.80
	18 DB7/18	20.52	10.99	791.80	484.80	298.96	3.07
	36 DB7/30	18.35	10.40	743.00	520.10	297.20	2.67
	36 DB7/30	18.81	10.94	820.80	153.90	205.20	2.57
	64 DB7/42-1	17.33	10.38	1285.00	989.30	889.80	10.38
	64 DB7/42-2	16.82	10.08	1329.00	778.90	870.20	13.75
IGS Boring SLN/33	17.67	10.57	811.60	305.00	426.00	5.70	

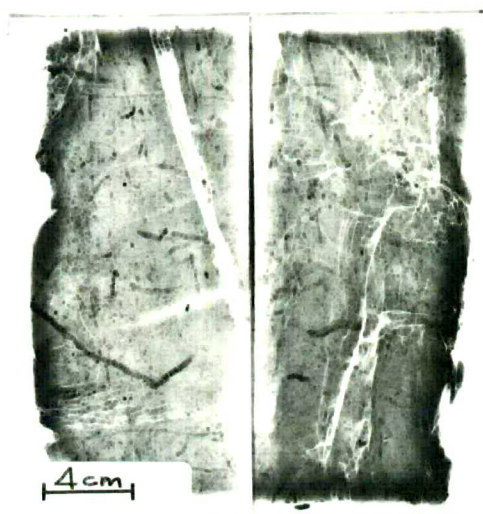
Table 3.9 Results of chemical analysis of pore water from clay samples from two sites in Northern North Sea.

<u>Fabric Type</u>	<u>Depositional Environment</u>
1	Rapid deposition by disintegration of grounded ice as orthotill.
2	Glacial marine in iceberg zone (from dry base glacier).
3	Flow-till, possibly submarine.
4	Glacial marine in floating shelf zone, dry base. (Flocculation of clays occurring at interface between fresh and salt water.)
5	Glacial marine in iceberg or floating shelf zone (faster currents c.f. Type 2).
6	Glacial marine in floating shelf zone, wet base. (Bonding from subsequent carbonate deposition.)

Table 3.10 Suggested depositional environments for sediments exhibiting Fabric Types 1 to 6.

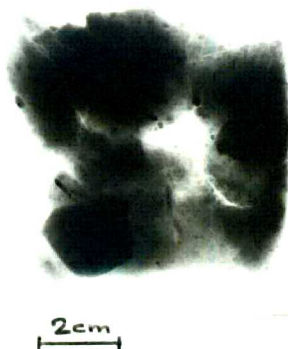


GLACIOLACUSTRINE LAMINATED CLAY FROM TEESSIDE.
SPLIT CORE, THIN SECTION AND RADIOGRAPH.



RADIOGRAPH OF FISSURED LONDON CLAY

Plate 3.1 Radiographs and photographs of glaciolacustrine laminated clay from Teesside and of fissured London clay.



FABRIC TYPE 1

(DB7C - 43)

CONTORTED. HARD LUMPS IN SOFT MATRIX.
NUMEROUS CLASTS AND BROKEN SHELLS.

LL = 36%, PL = 18%

DEPTH: 14 - 25m



FABRIC TYPE 2

(DB7 - 30, DB7 - 11, DB5 - 2A, DB7B - 34)

GENERALLY FEATURELESS. INTACT AND BROKEN
SHELLS AND CLASTS WELL DISTRIBUTED. SOME
ORGANIC CONTENT AND EVIDENCE OF BURROWING.

LL = 21% - 51%, PL = 13% - 22%

DEPTH: 2 - 14m, 36 - 42m



FABRIC TYPE 3

(DB7C - 67, DB5 - 50)

SLIGHTLY FOLIATED. OCCASIONALLY PLATY.
OCCASIONAL SHELLS, CLASTS AND INCLUSIONS
OF FINE SAND. EVIDENCE OF BURROWING.

LL = 36%, PL = 18%

DEPTH: 100 - 118m

Plate 3.2 Macrofabrics observed in samples from Ninian. Types 1, 2 and 3.



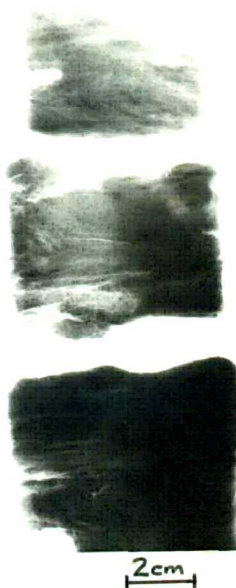
FABRIC TYPE 4

(DB7C - 60)

FINE PLATY FABRIC. NUMEROUS RANDOM CLASTS AND SHELLS.

LL = 35%, PL = 18%

DEPTH: 64 - 100m



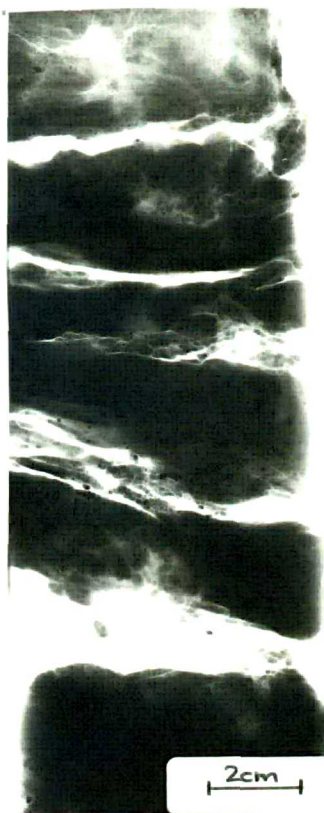
FABRIC TYPE 5

(DB7B - 37)

STRATIFIED AND VERY FINELY LAMINATED SANDY CLAY AND FINE SAND. ORGANIC. OCCASIONAL SHELLS.

NON PLASTIC

DEPTH: 25 - 36m, 42 - 48m



FABRIC TYPE 6

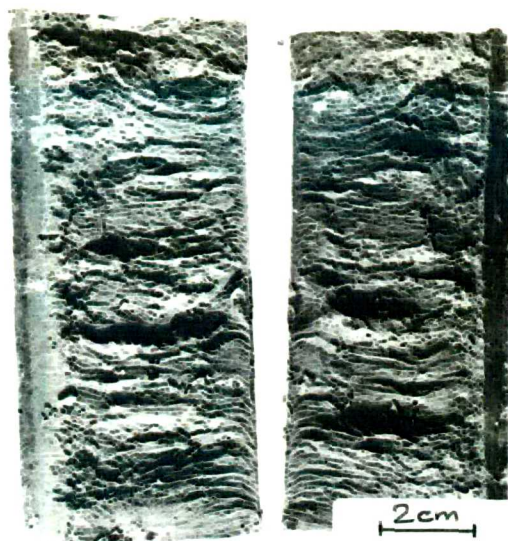
(DB7 - 42)

INTERBEDDED LAYERS OF SILTY FINE SAND AND MEDIUM PLASTICITY CLAY. LAMINAE 12 - 40 MM THICK. CLAY BEDS OCCASIONALLY CONTAIN CLASTS AND SHOW EVIDENCE OF BURROWING. SAND BEDS SHOW RIPPLING AND SLUMPING. ORGANIC.

LL = 41%, PL = 19%

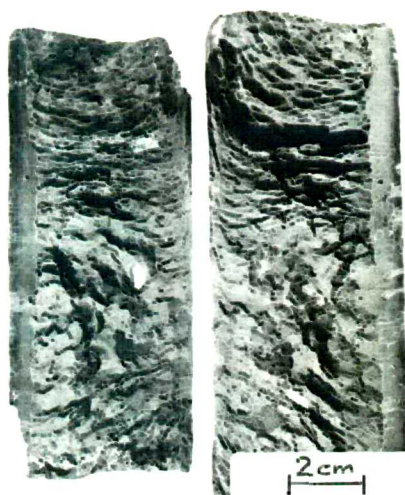
DEPTH: 48 - 64m

Plate 3.3 Macrofabrics observed in samples from Ninian. Types 4, 5 and 6.



FABRIC TYPE 2

LL = 40% - 45%, PL = 15% - 20%
DEPTH: 0 - 25m



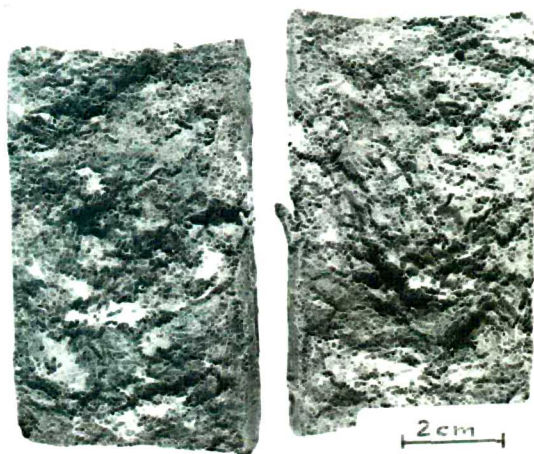
T Y P E 3

T Y P E 4



FABRIC TYPES 3 AND 4

LL = 45% - 50%, PL = 15% - 20%
DEPTH: 25 - 30m



FABRIC TYPE 5

LL = 30%, PL = 15%
DEPTH: 30 - 55m

Plate 3.4 Examples of Macrofabric. Types 2, 3, 4 and 5 at Statfjord.



FABRIC TYPE 1

LL = 38% - 40%, PL = 18% - 20%

DEPTH: 35 - 46m



FABRIC TYPE 2

LL = 35% - 40%, PL = 10% - 15%

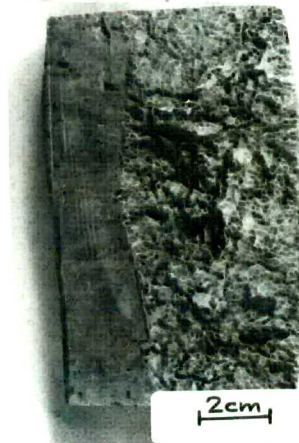
DEPTH: 4 - 7m, 11 - 17m



FABRIC TYPE 3

LL = 40% - 50%, PL = 15% - 20%

DEPTH: 19 - 28m, 50 - 61m, 75 - 85m



FABRIC TYPE 4

LL = 44%, PL = 20%

DEPTH: 85 - 110m

Plate 3.5 Examples of Macrofabric. Types 1, 2, 3 and 4 at Magnus.



FABRIC TYPE 4
(ABERDEEN GROUND BEDS)
DEPTH 135m

FABRIC TYPE 4
LL = 53%, PL = 24%
DEPTH 148m



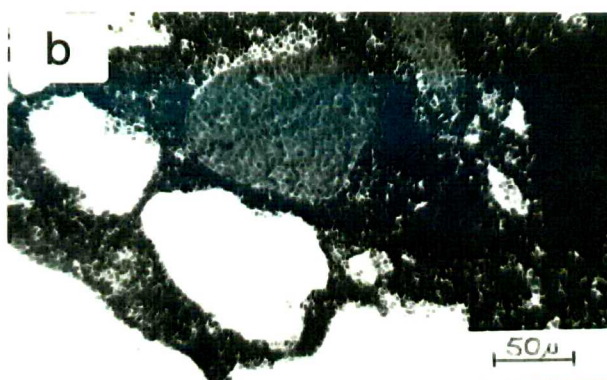
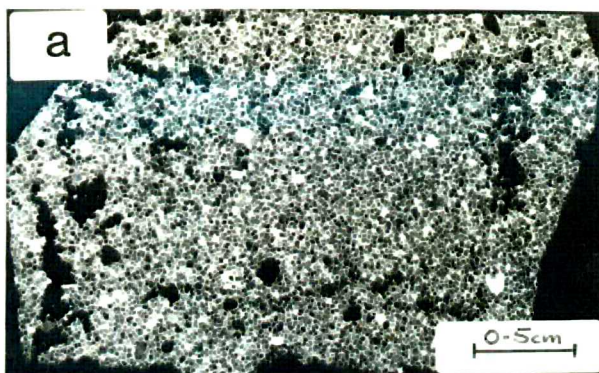
FABRIC TYPE 2
(ABERDEEN GROUND BEDS)
DEPTH 148m



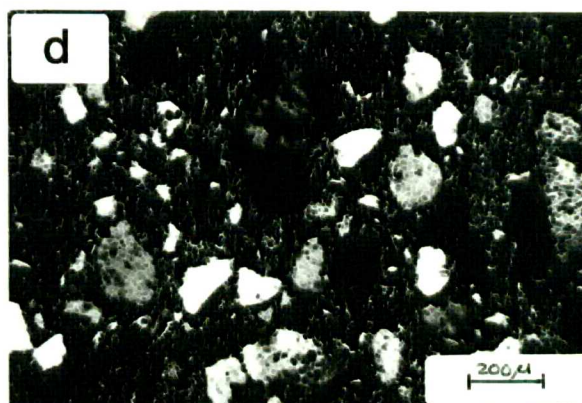
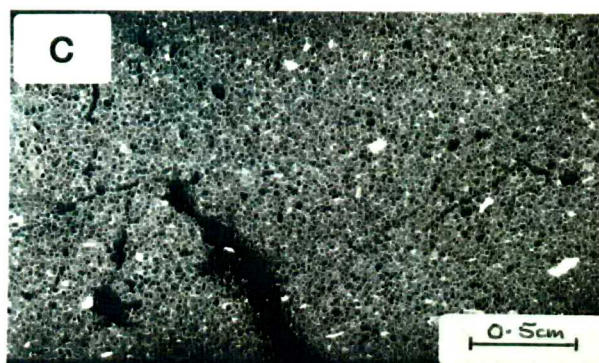
FABRIC TYPE 2
(ABERDEEN GROUND BEDS)
DEPTH 148m



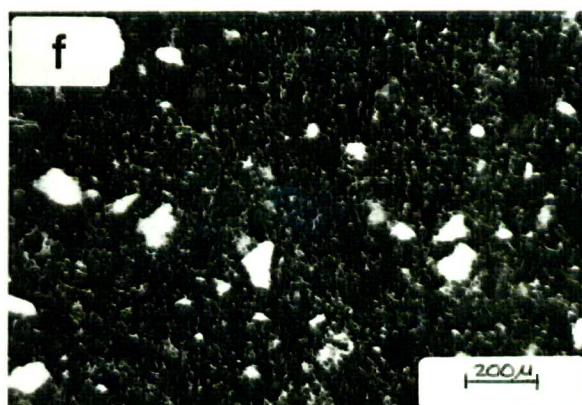
FABRIC TYPE 2
(ABERDEEN GROUND BEDS)
DEPTH 148m



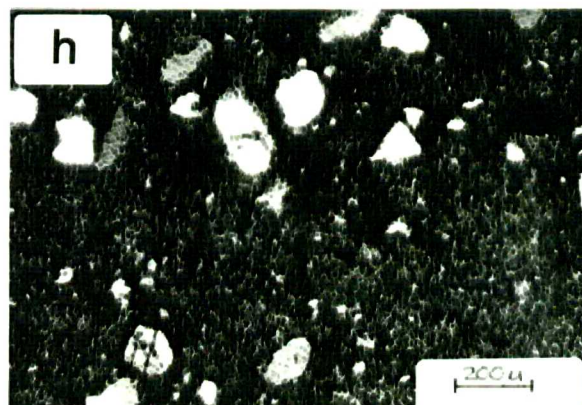
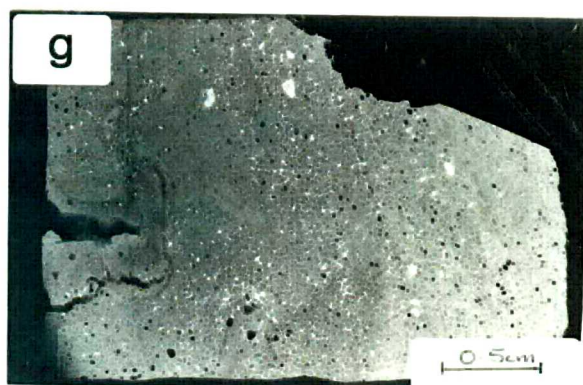
DB7 - 30 (LL = 29%, PL = 14%)



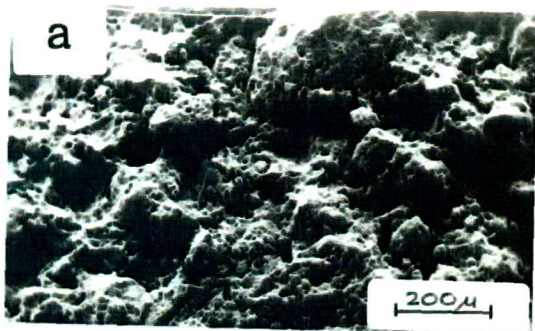
DB7 - 11 (LL = 39%, PL = 18%)



DB5 - 2A (LL = 51%, PL = 22%)



DE7E - 34 (LL = 21%, PL = 13%)



DE7 - 30 (LL = 29%, PL = 14%)

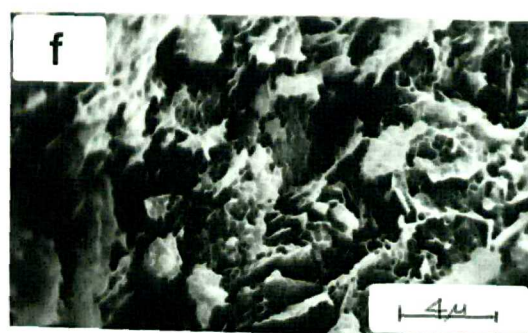
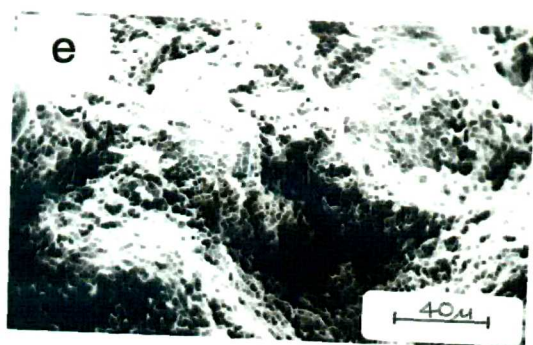
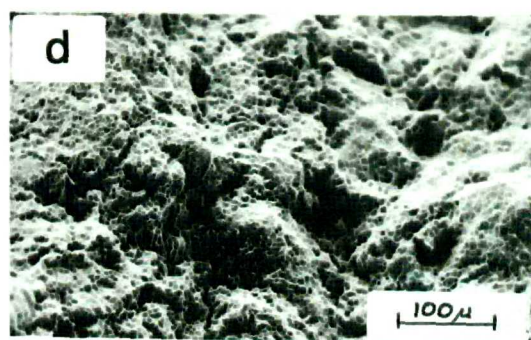


Plate 3.8 Fabric Type 2. Scanning electron micrographs.

DB5 - 2A (LL = 51%, PL = 22%)

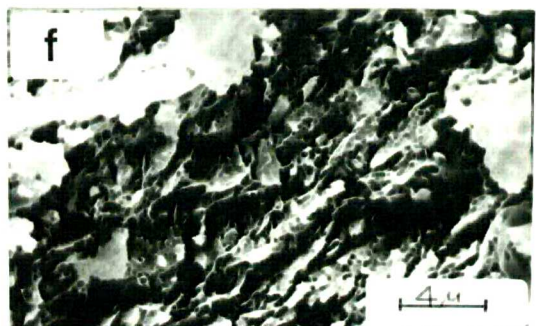
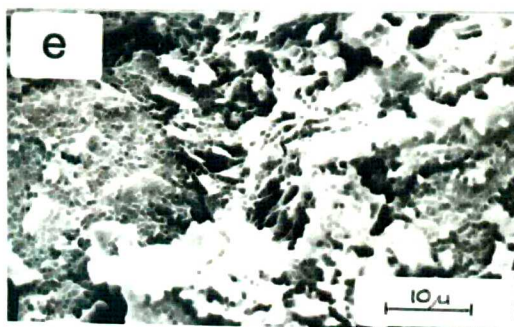
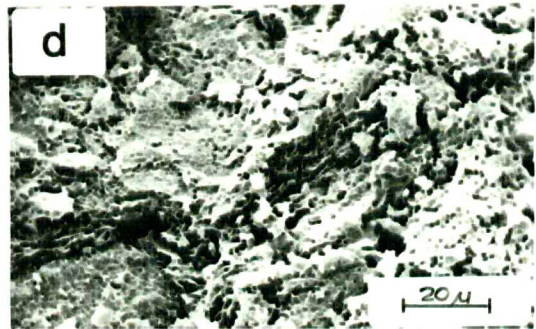
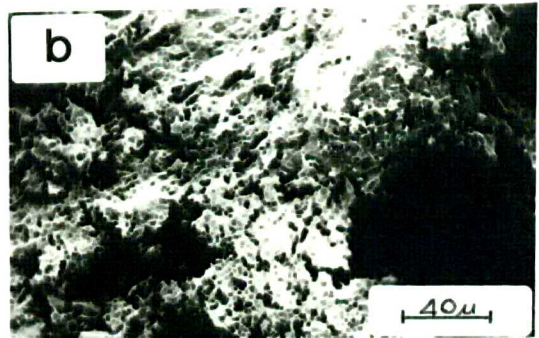
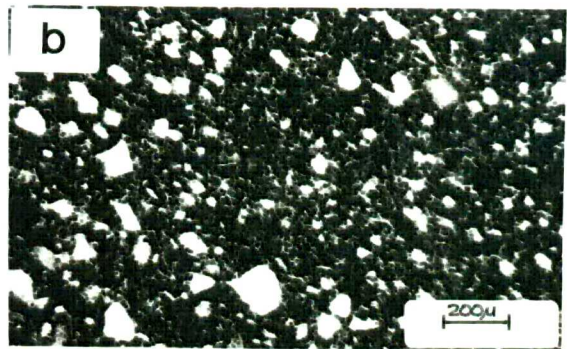


Plate 3.9 Fabric Type 2. Scanning electron micrographs.

1-10 - 10 (L1 = 3, H1 = 10)



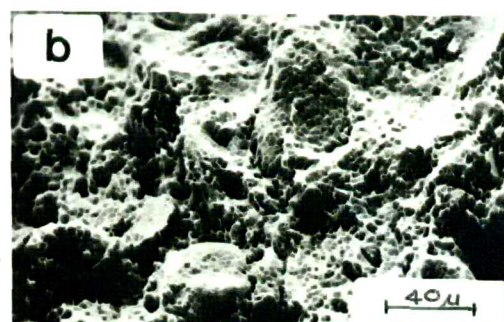
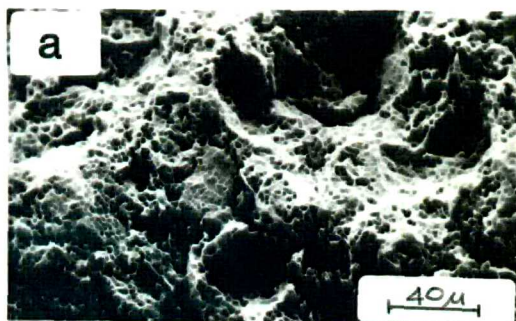
1-15 - 50



Plate 3.10 Fabric Type 3. Thin sections.

DB7C - 67 (LL = 36%, PL = 18%)

TYPE 3



SIN 33 (LL = 53%, PL = 24%)

TYPE 4

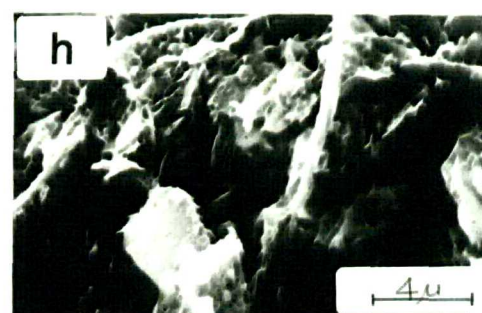
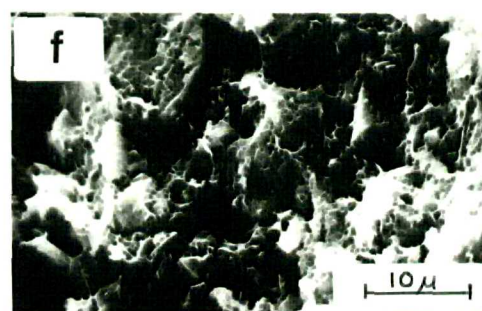
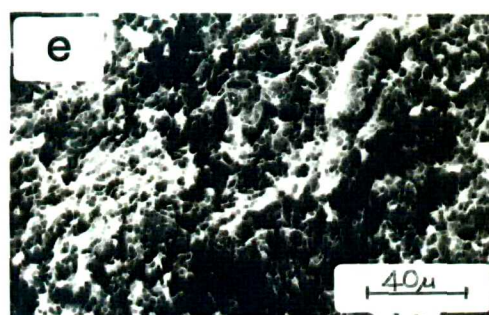
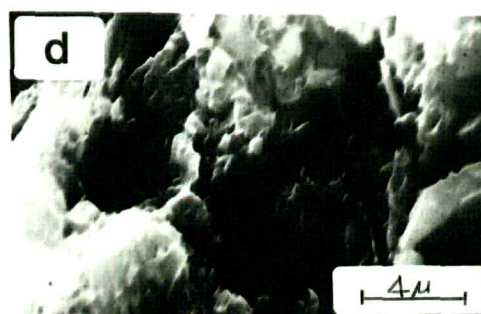
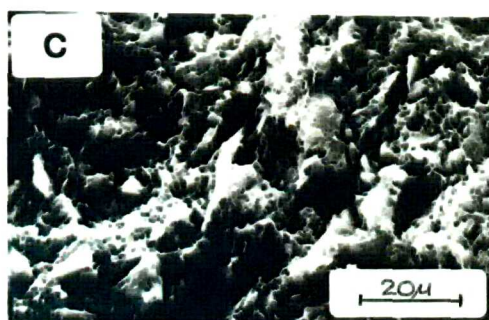
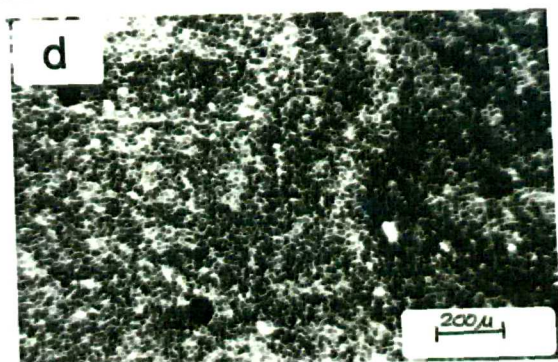
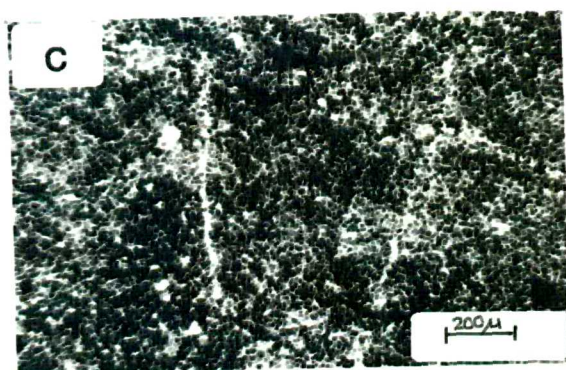
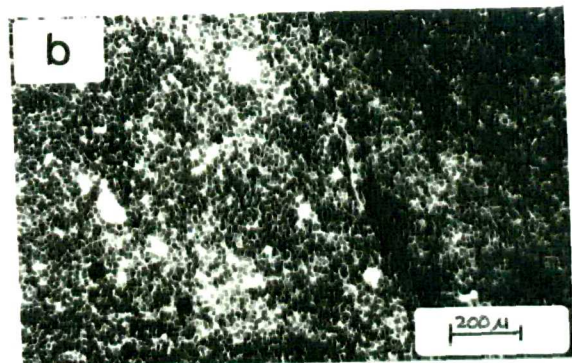
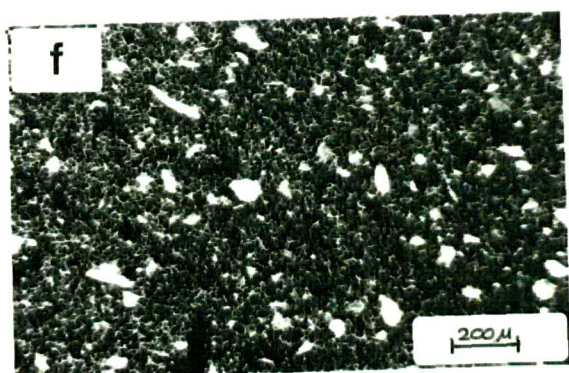


Plate 3.11 Fabric Types 3 and 4. Scanning electron micrographs.

PA6 - 321 (LL = 13, PL = 24)



PA6 - 321



PA1 - 318 (LL = 4.5%, PL = 21%)

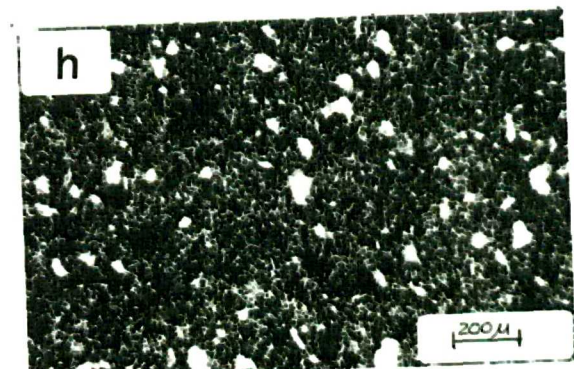
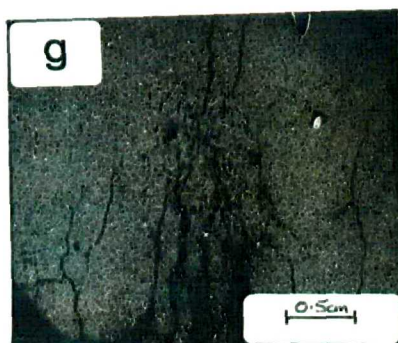
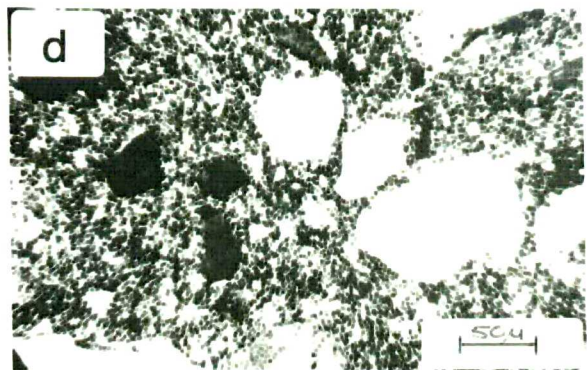
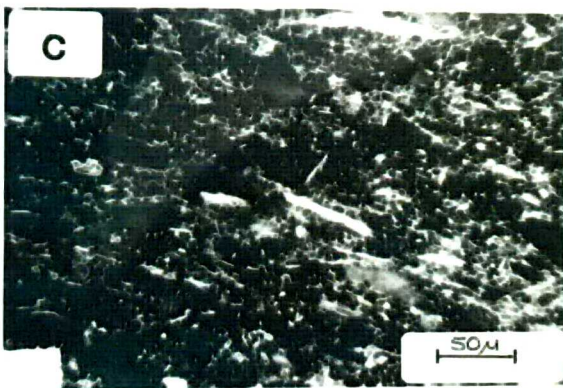
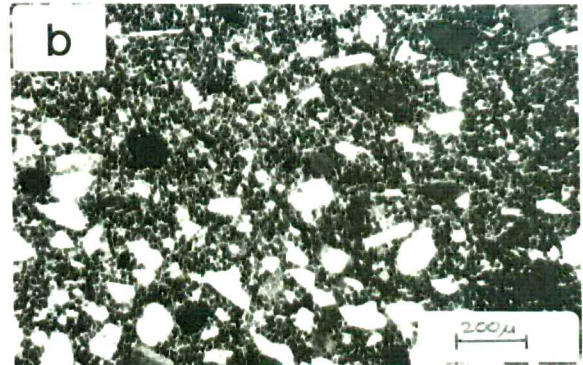
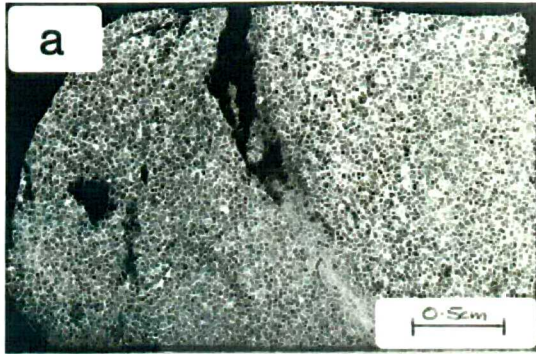


Plate 3.12 Fabric Type 4. Thin sections.

DE2 - 439



DE7C - 40

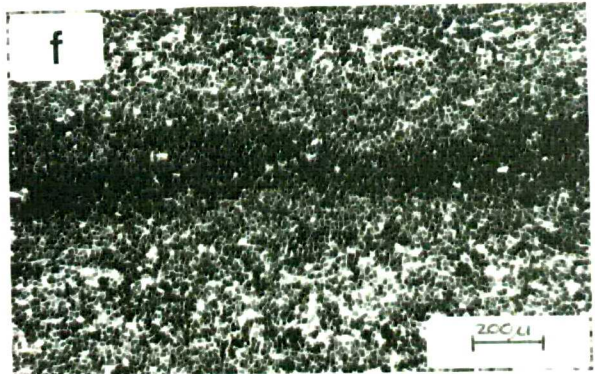
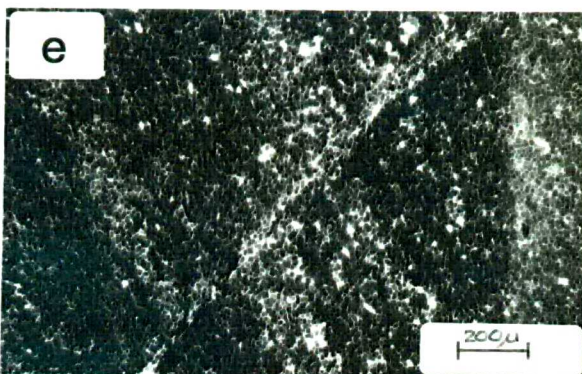


Plate 3.13 Fabric Type 5. Thin sections.

187 - 182 (IL = 41%, PL = 19%)

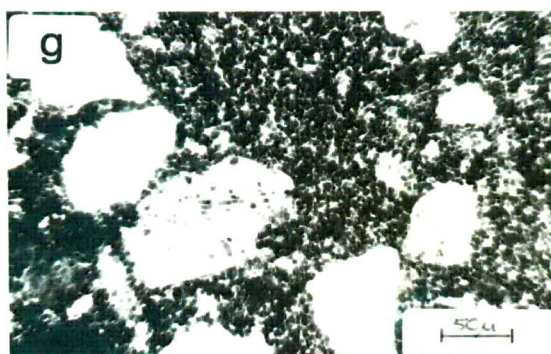
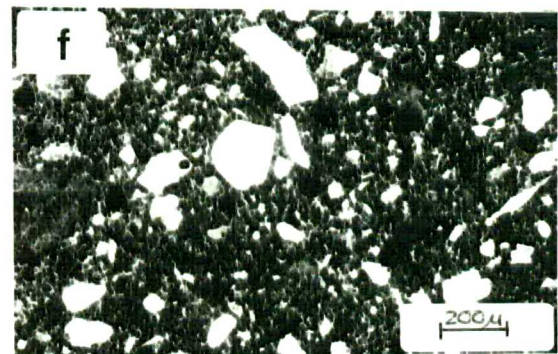
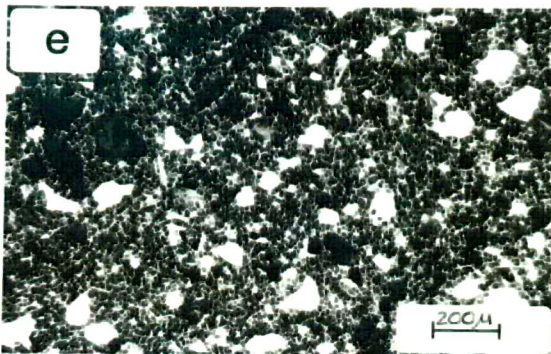
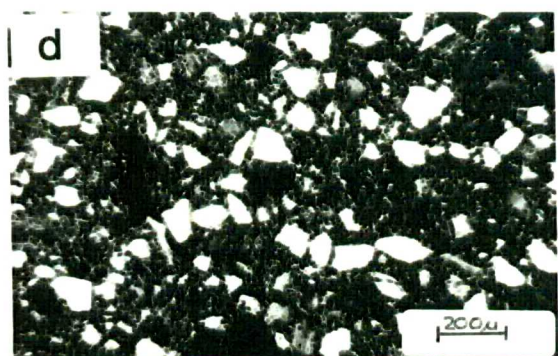
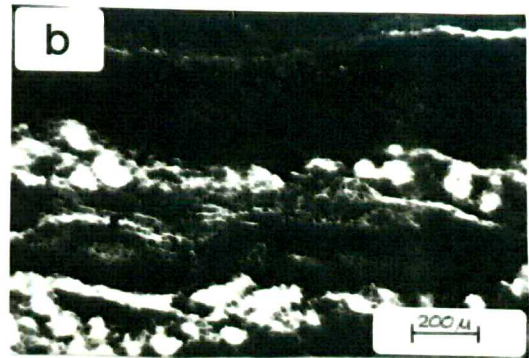
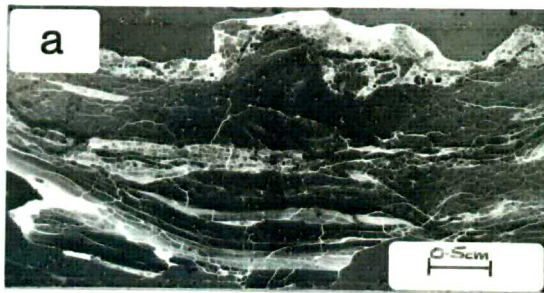


Plate 3.14 Fabric Type 6. Thin sections.

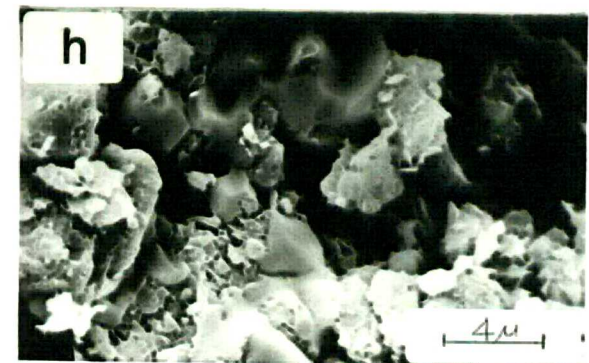
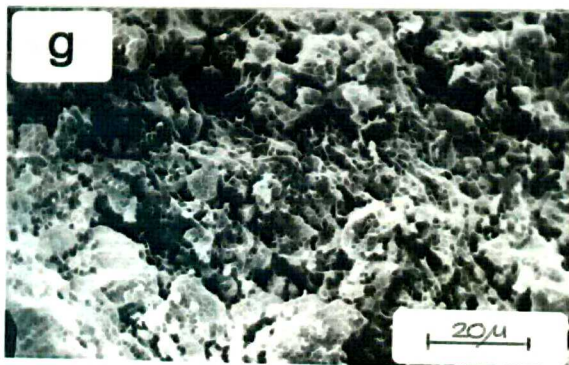
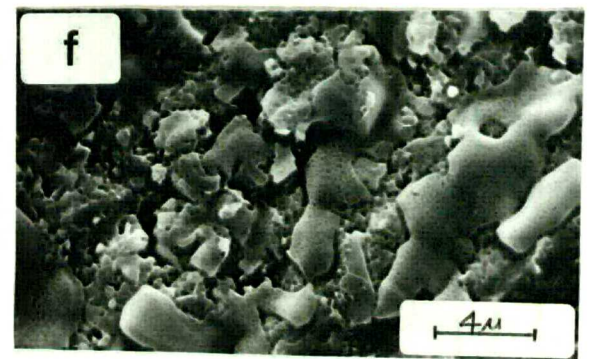
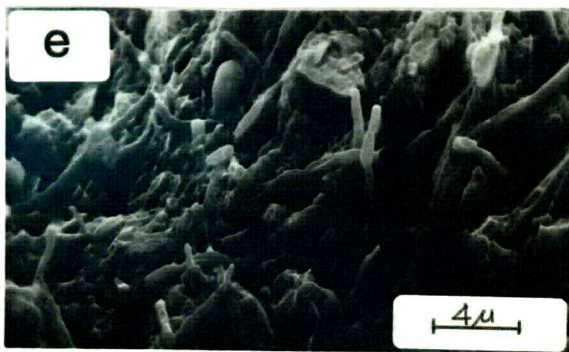
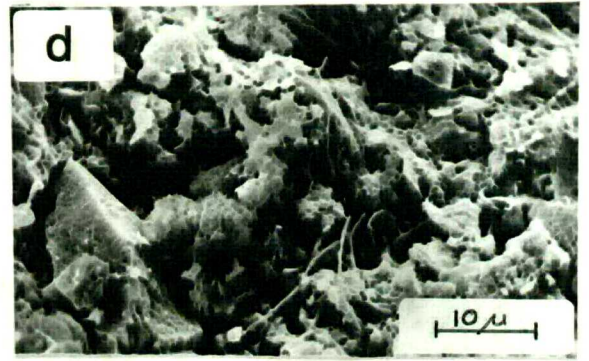
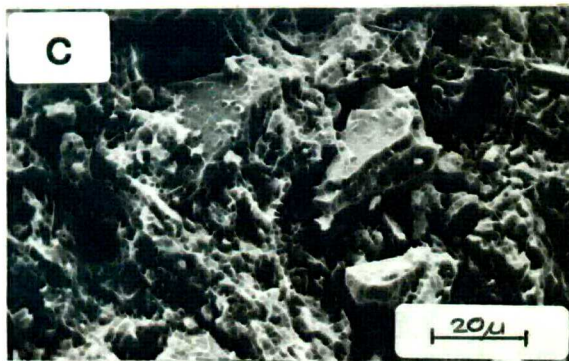
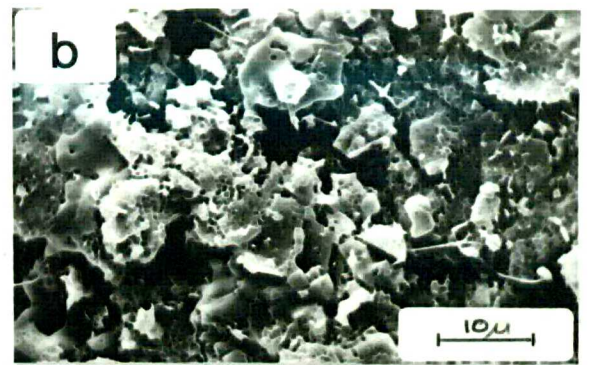
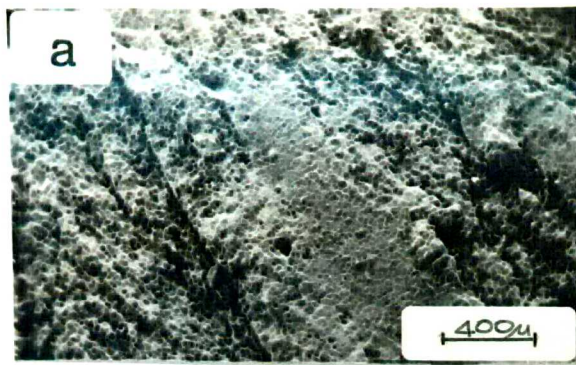


Plate 3.15 Fabric Type 6. Scanning electron micrographs.

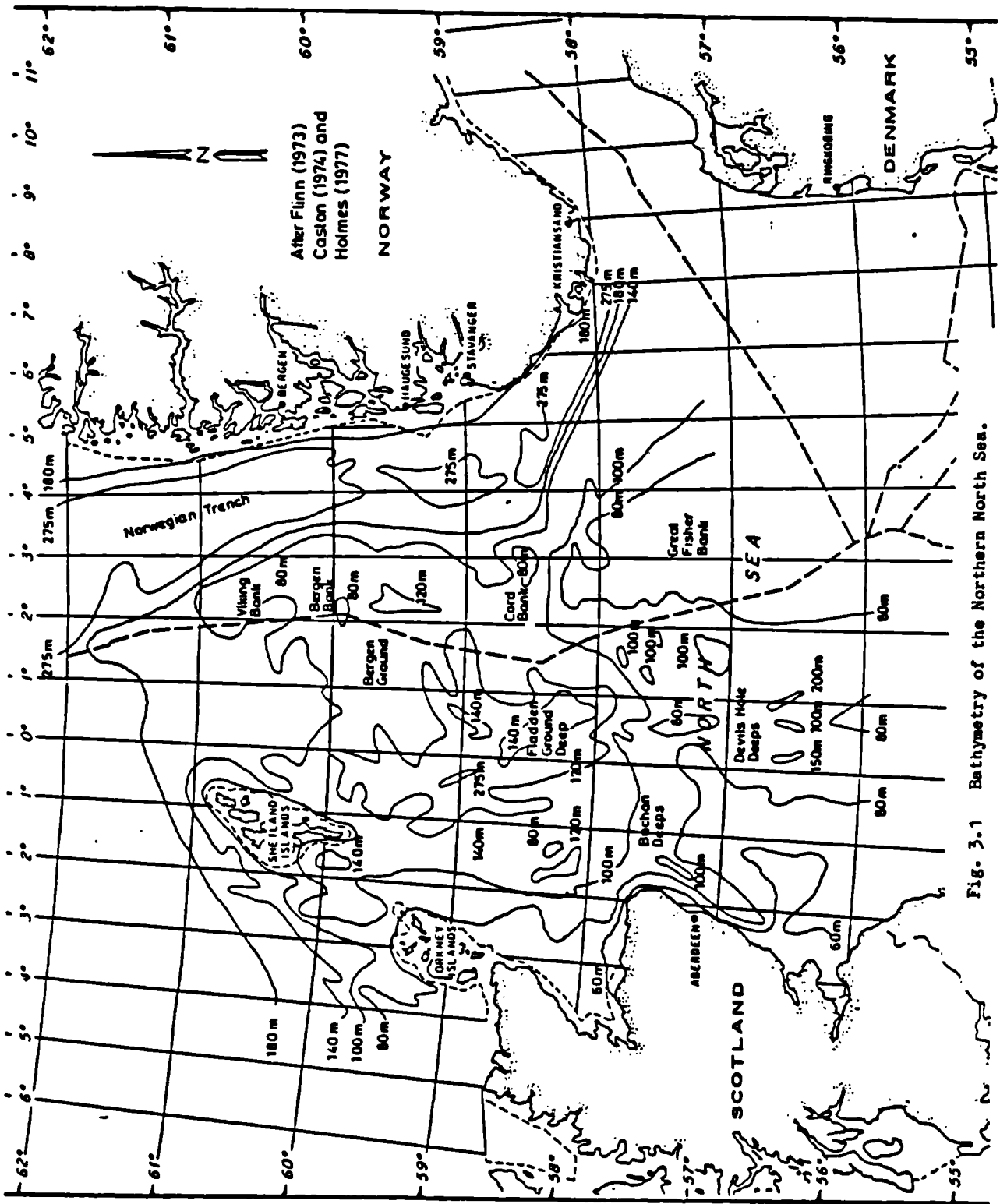
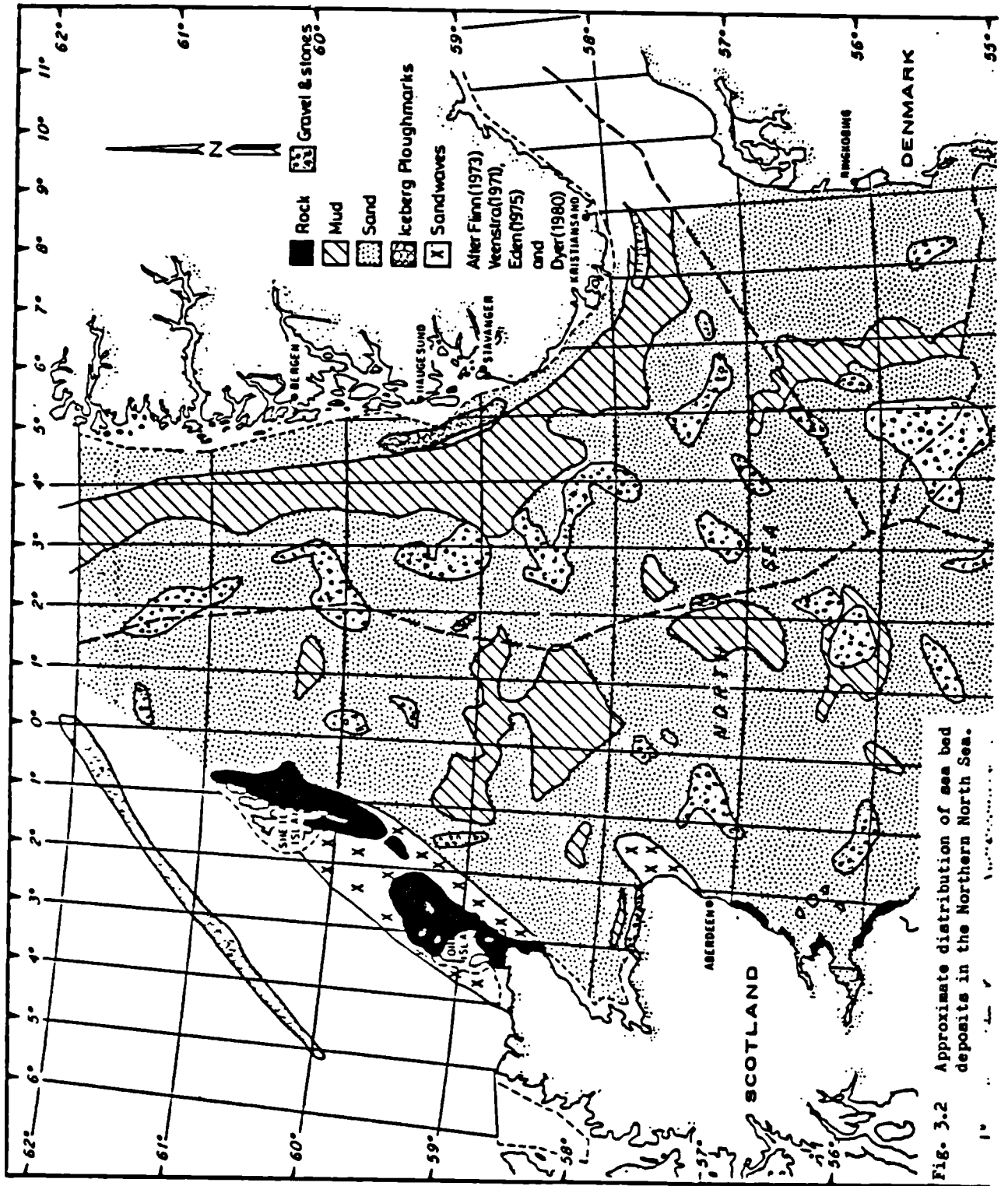


Fig. 3-1 Bathymetry of the Northern North Sea.



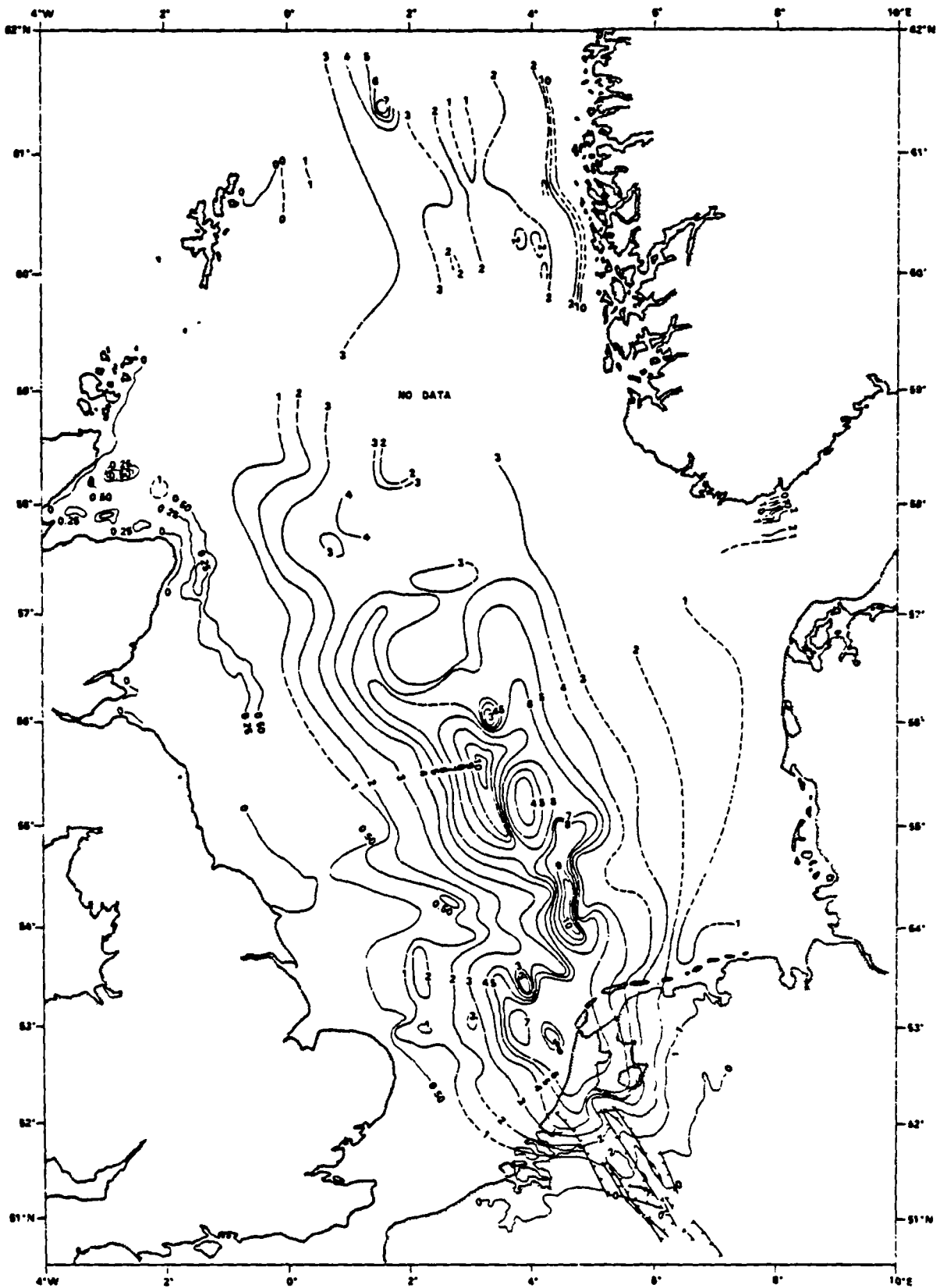
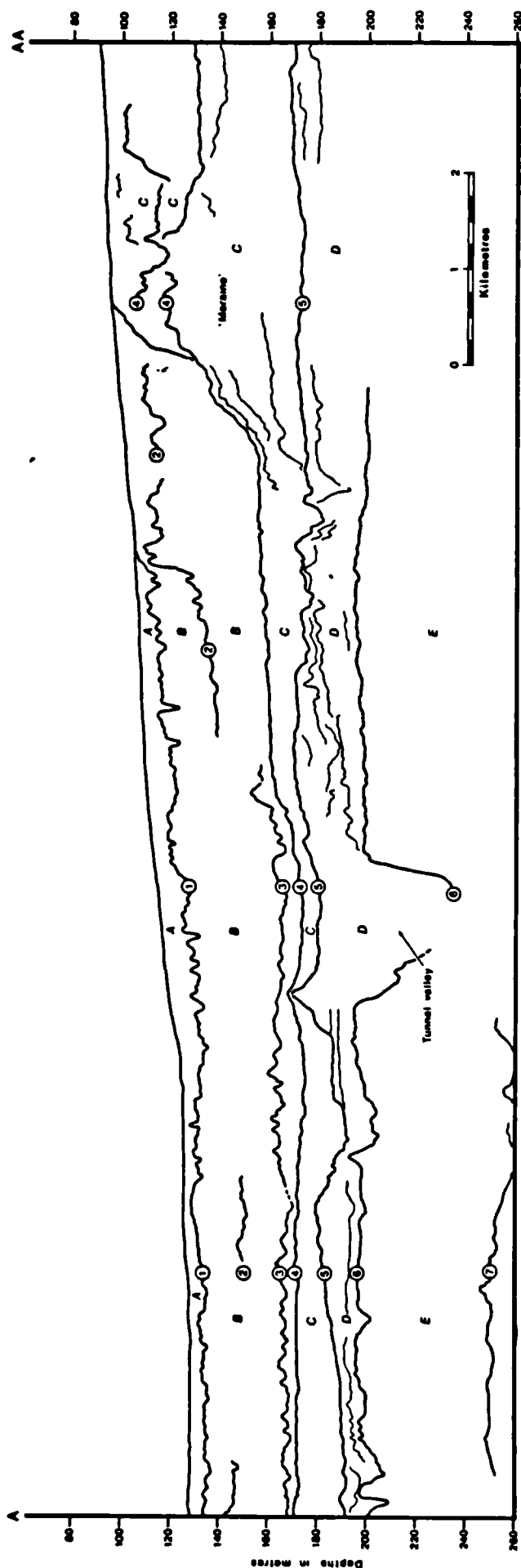
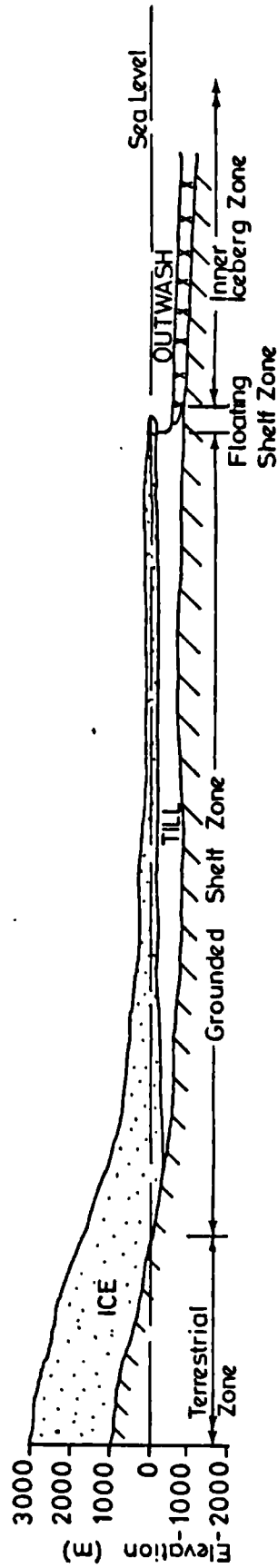


Fig. 3.3 Isopach map of total Quaternary thickness in North Sea (Caston, 1977).

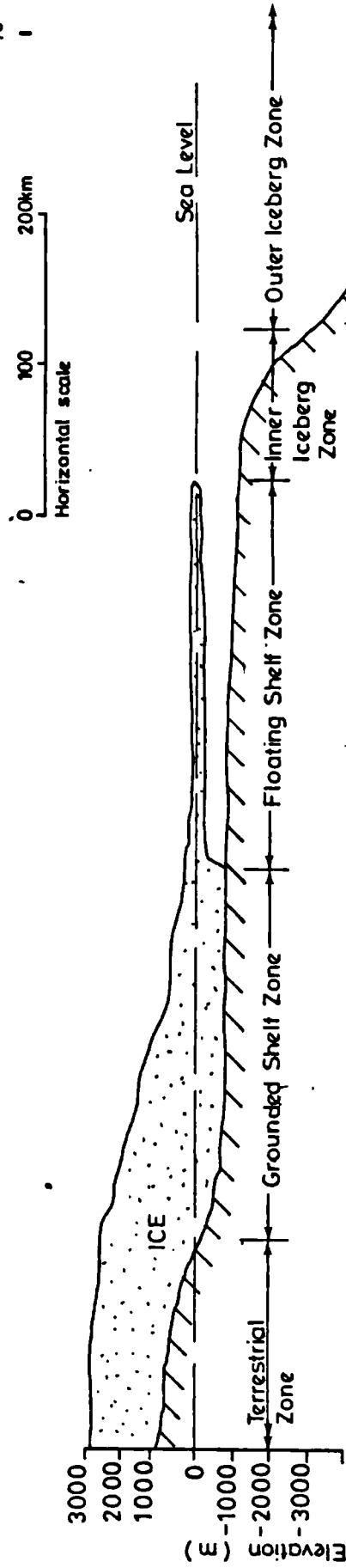


- | | |
|---------|---------------------------------|
| LAYER A | Witch Ground Beds |
| LAYER B | Fisher and Lower Swatchway Beds |
| LAYER C | |
| LAYER D | Lower Channel Deposits |
| LAYER E | Aberdeen Ground Beds |

Fig. 3.4 Cross section showing distribution of glacial and marine sediments in area of Forties Field (after Caston, 1977).



(a) Profile of wet-base glacier (based on Ross Barrier)



(b) Profile of dry-base glacier (based on Pencksokka glacier)

Fig. 3.5 Environments for glacial marine sedimentation (after Carey and Ahmad, 1961).



Fig. 3.6 Variation in sea depth with time (after Mörner, 1971, and others).

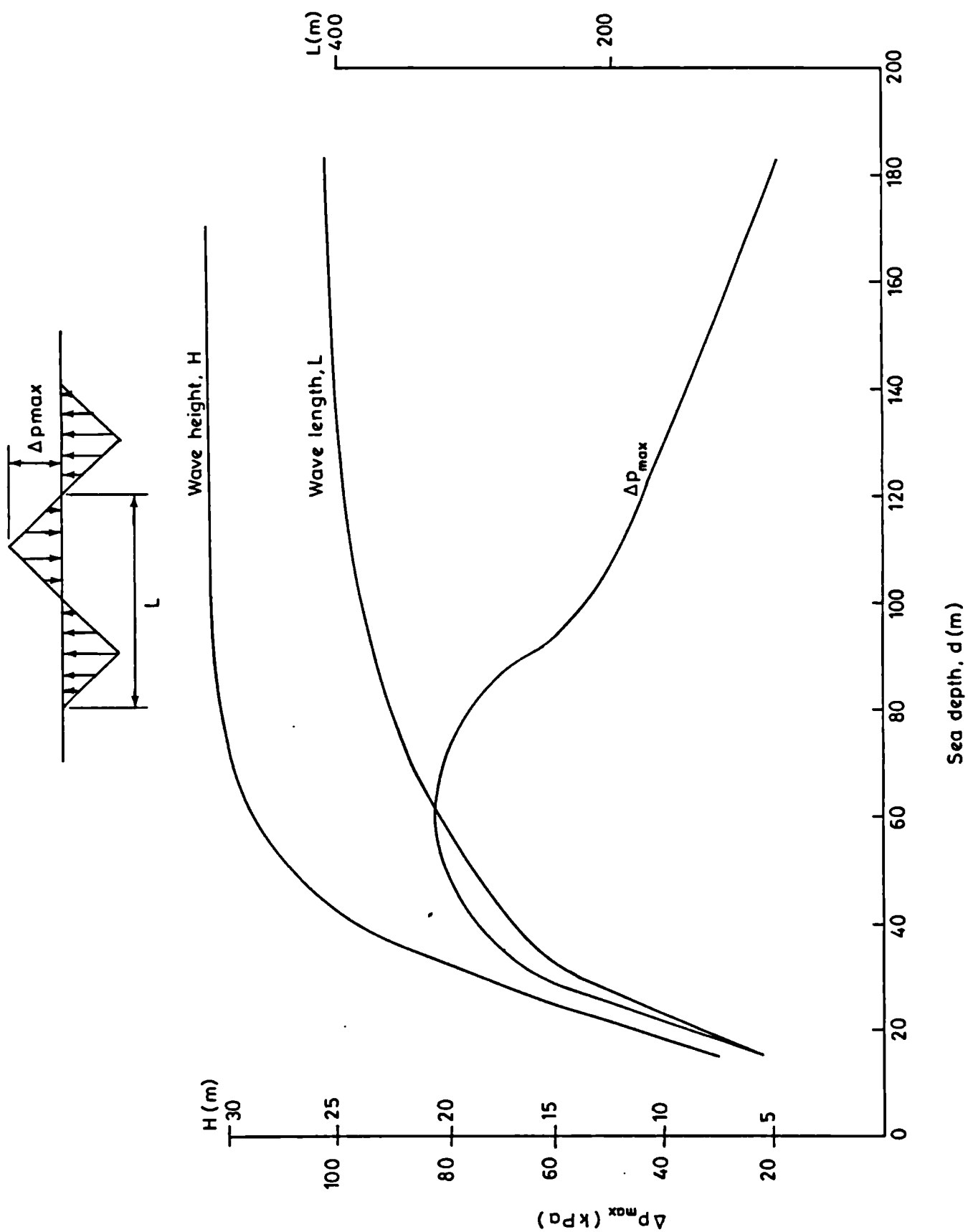


Fig. 3.7 Variation in wave height, wave length and maximum sea bed pressure with sea depth.

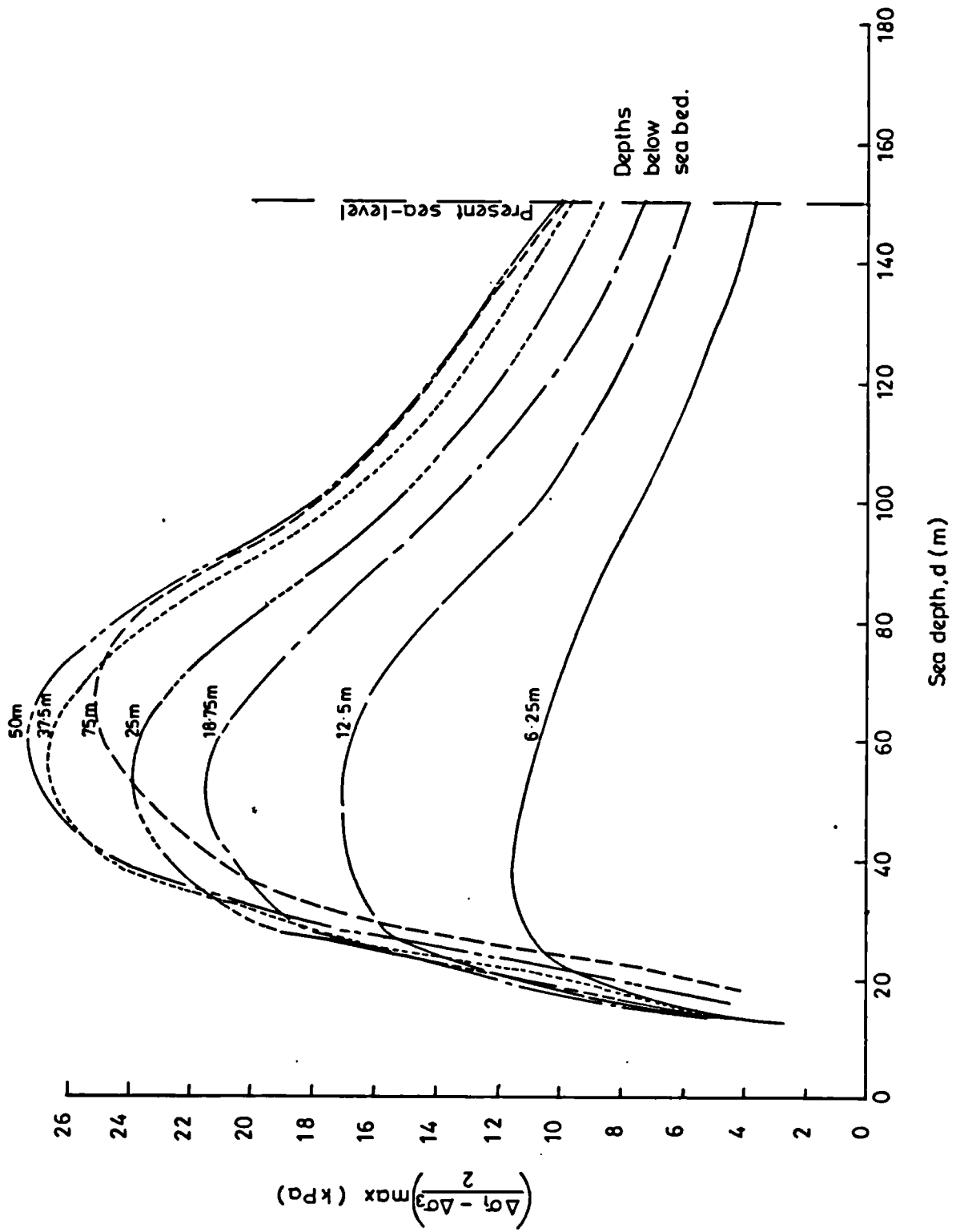


Fig. 3.8 Variation in $\left(\frac{\Delta\sigma_1 - \Delta\sigma_3}{2}\right)_{\max}$ with sea depth at different depths below sea bed.

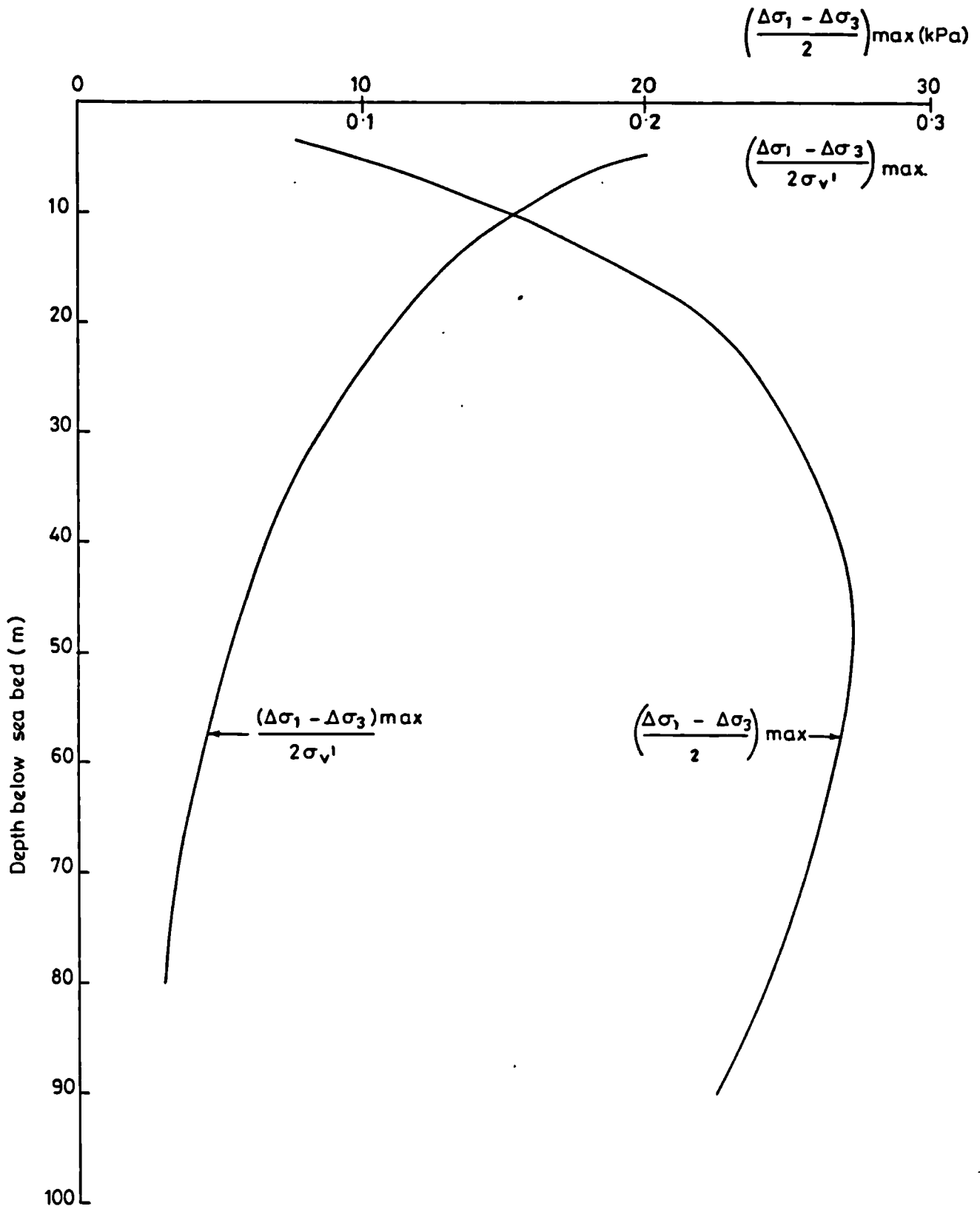
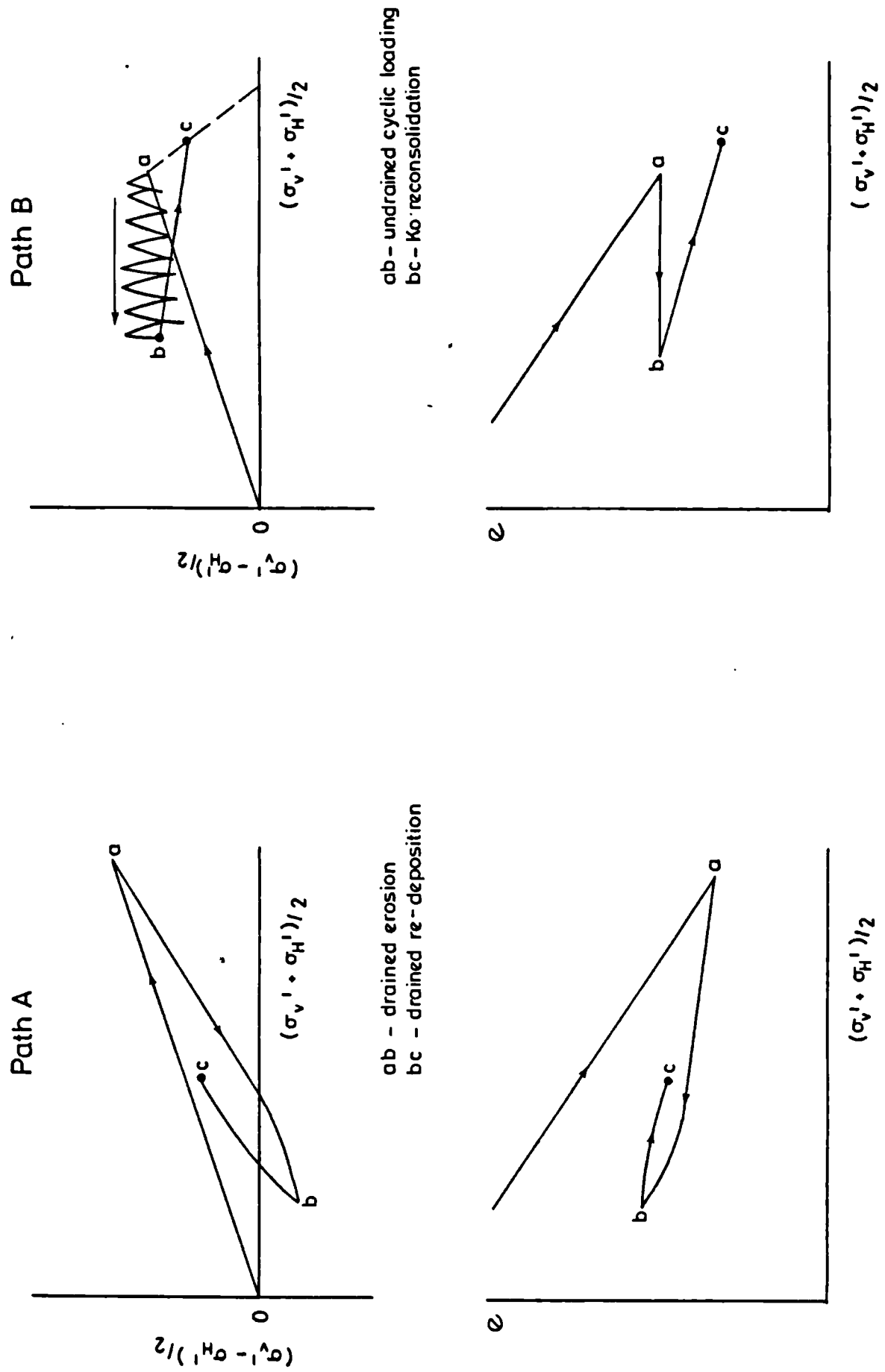


Fig. 3.9 Variation in $\left(\frac{\Delta\sigma_1 - \Delta\sigma_3}{2}\right)_{\max}$ and $\left(\frac{\Delta\sigma_1 - \Delta\sigma_3}{2\sigma_v'}\right)_{\max}$ with depth below sea bed.



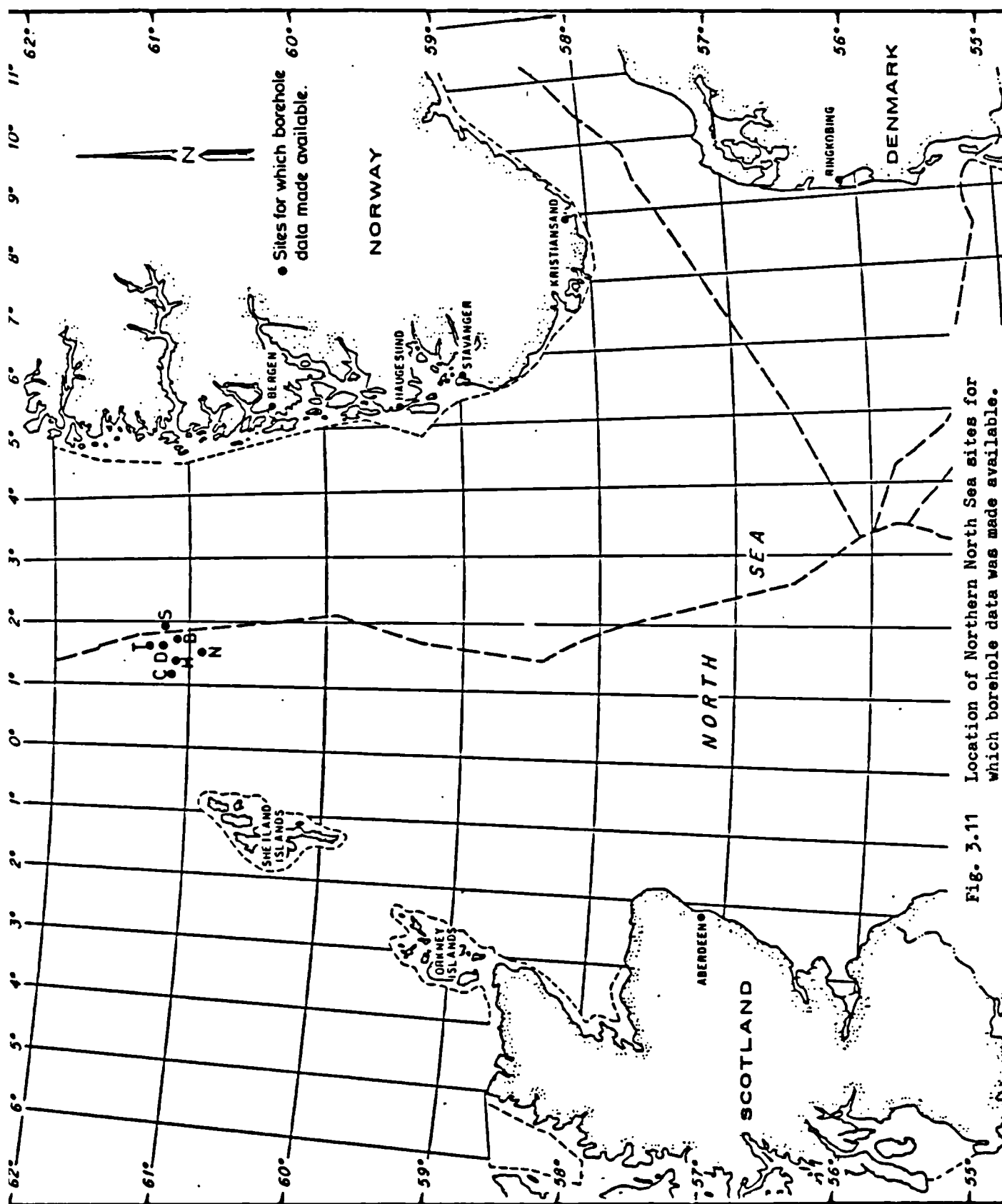


Fig. 3.11 Location of Northern North Sea sites for which borehole data was made available.

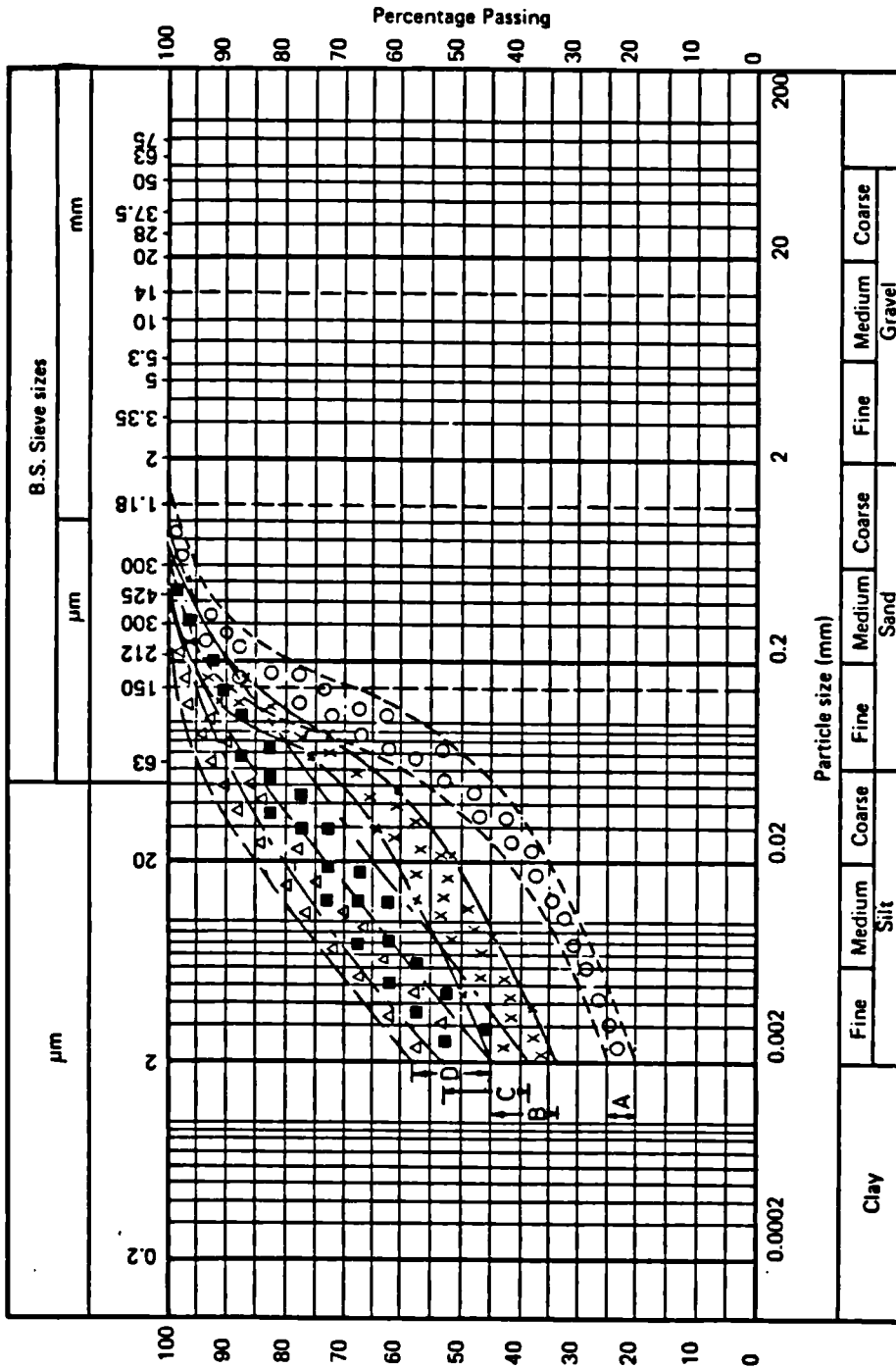


Fig. 3.12 Grading envelopes for sediments in the Northern North Sea.

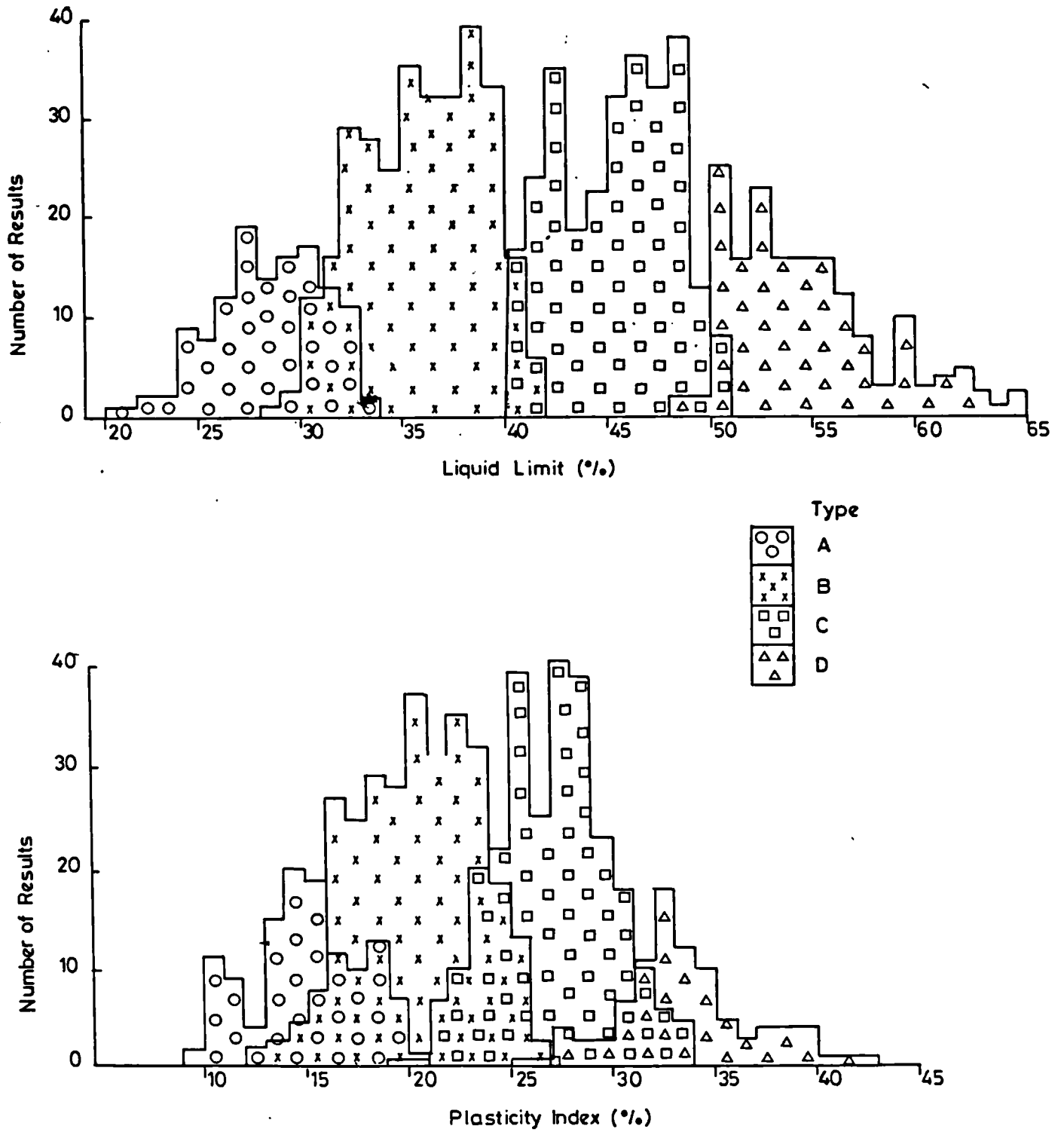


Fig. 3.13 Histograms of plasticity data.

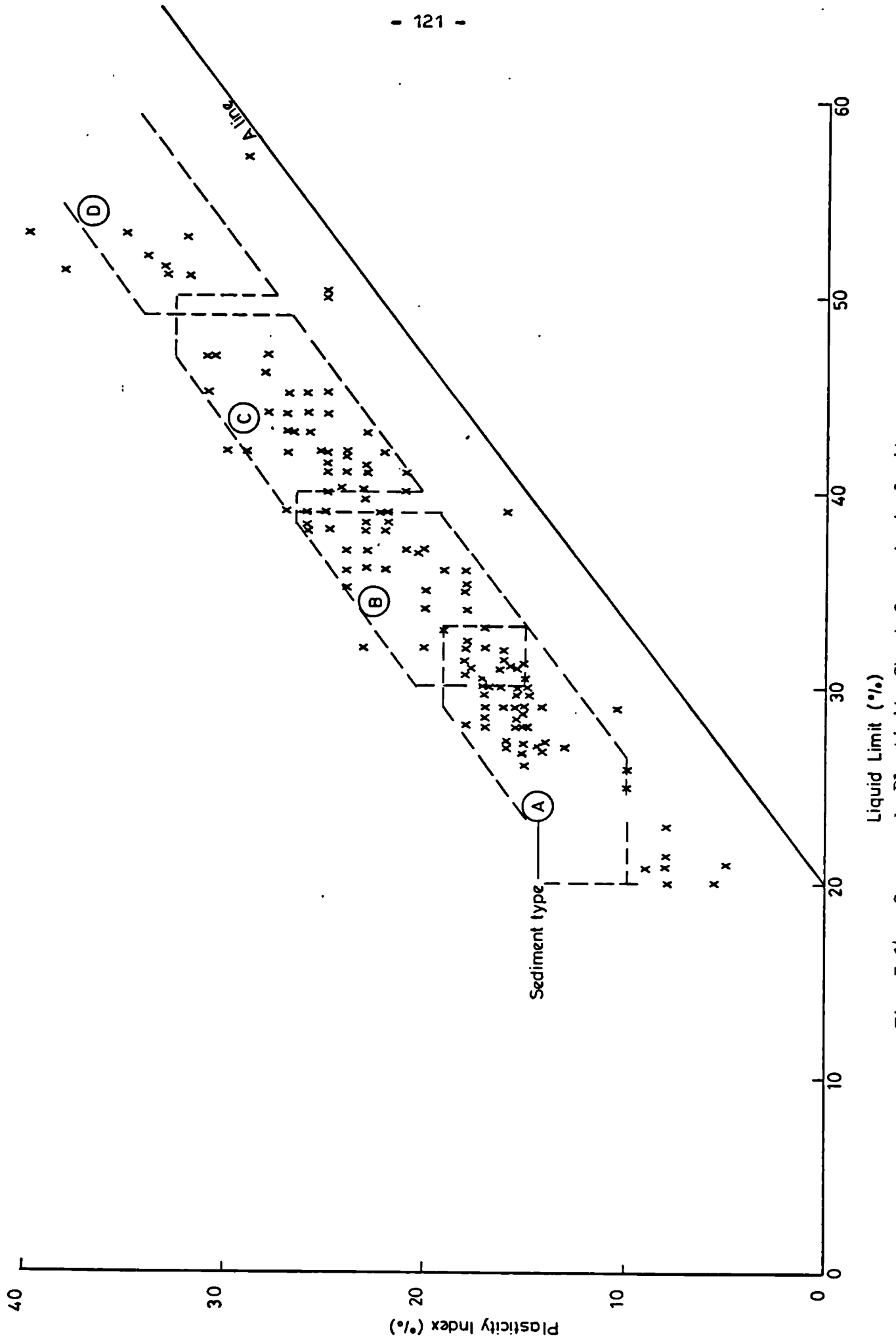


Fig. 3.14 Casagrande Plasticity Chart for a typical site.

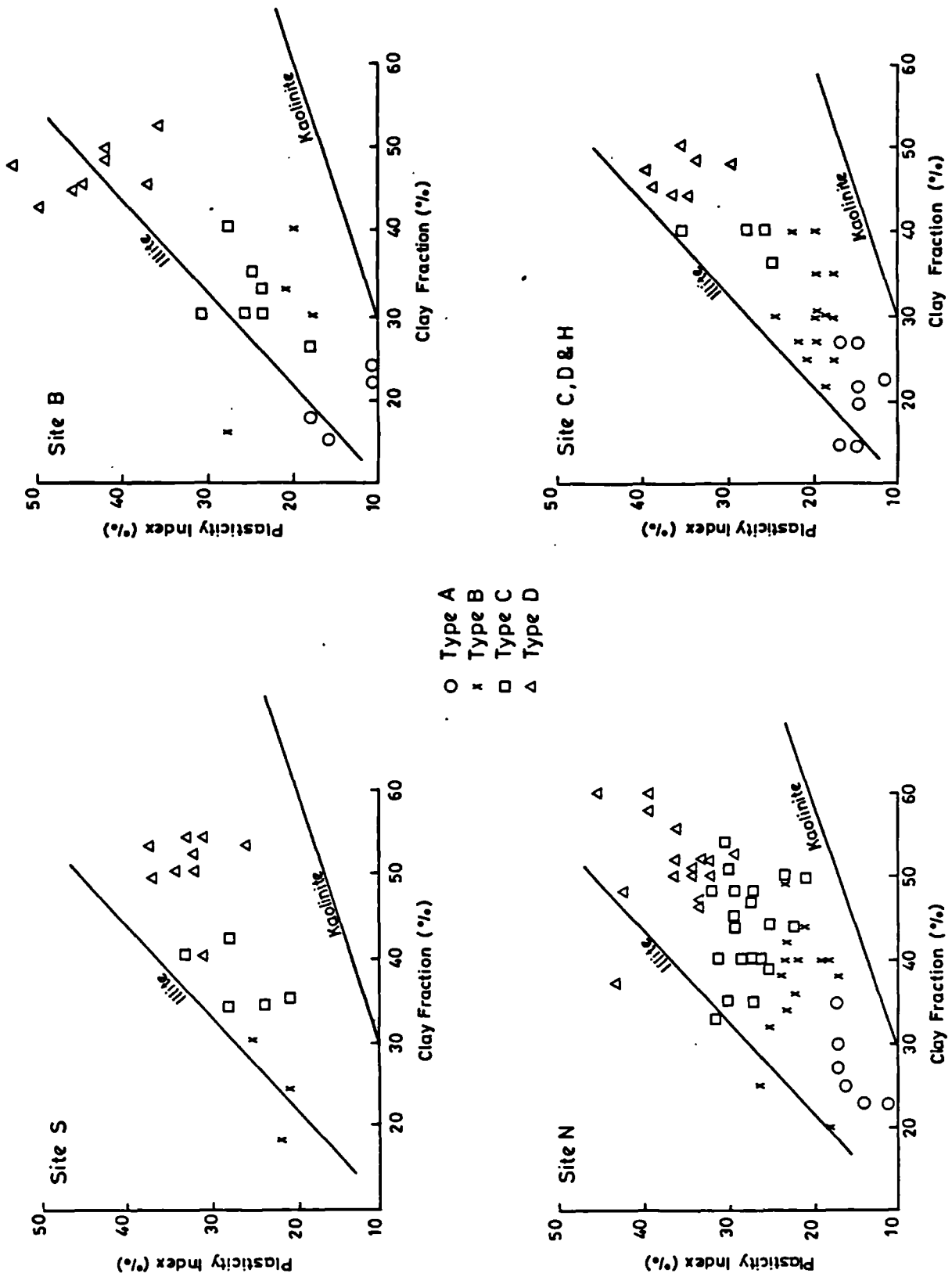
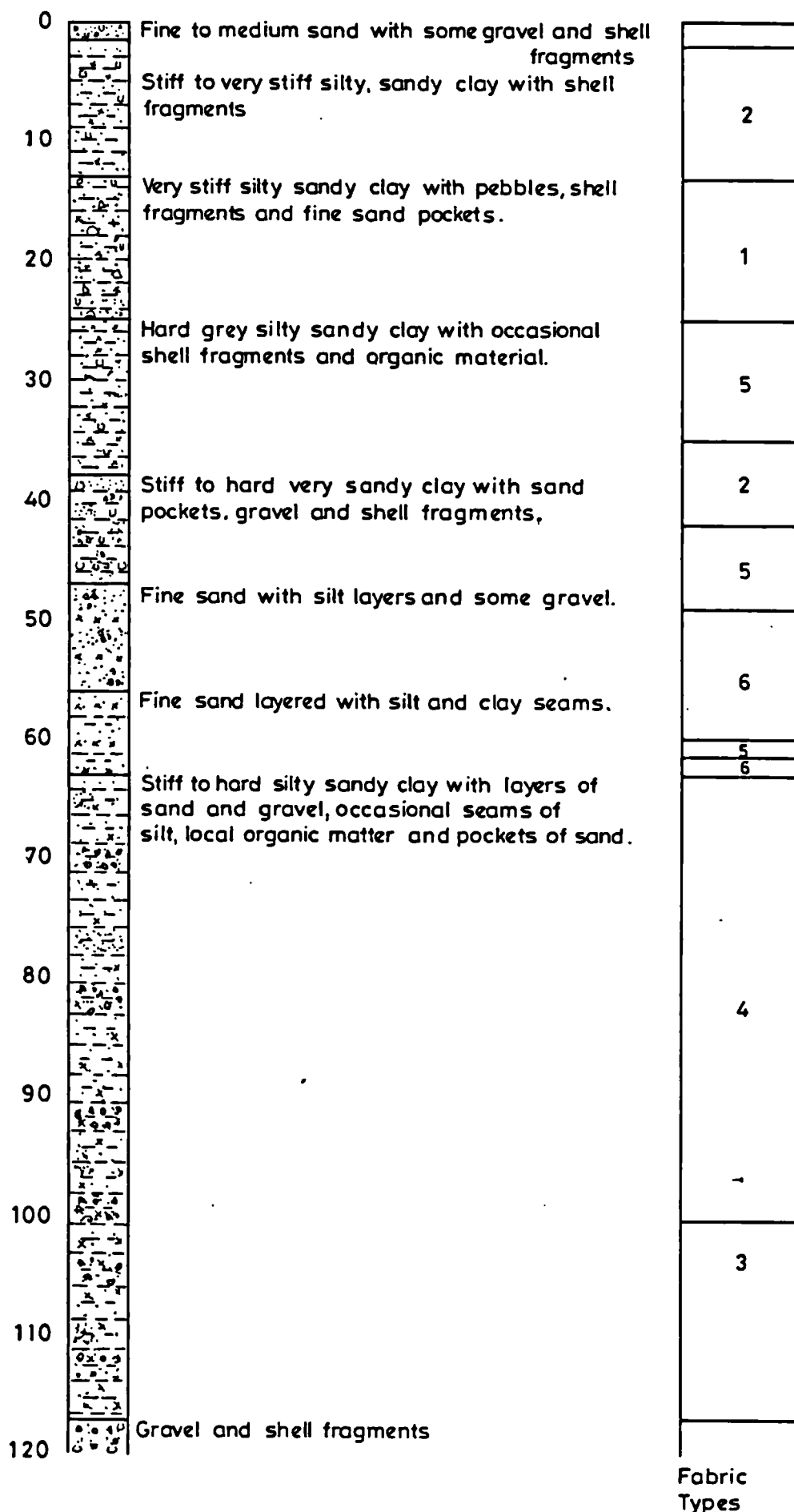


Fig. 3.15 Activity charts for different sites.
(Activity ratios after Skempton, 1953.)



Simplified Borehole Log.

Fig. 3.16 Distribution of fabric types with depth at Ninian.

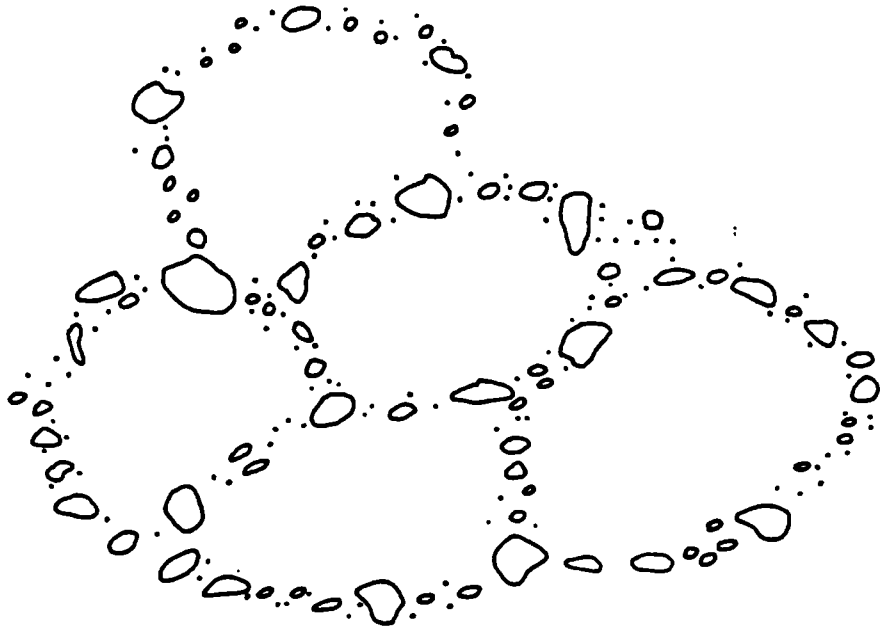
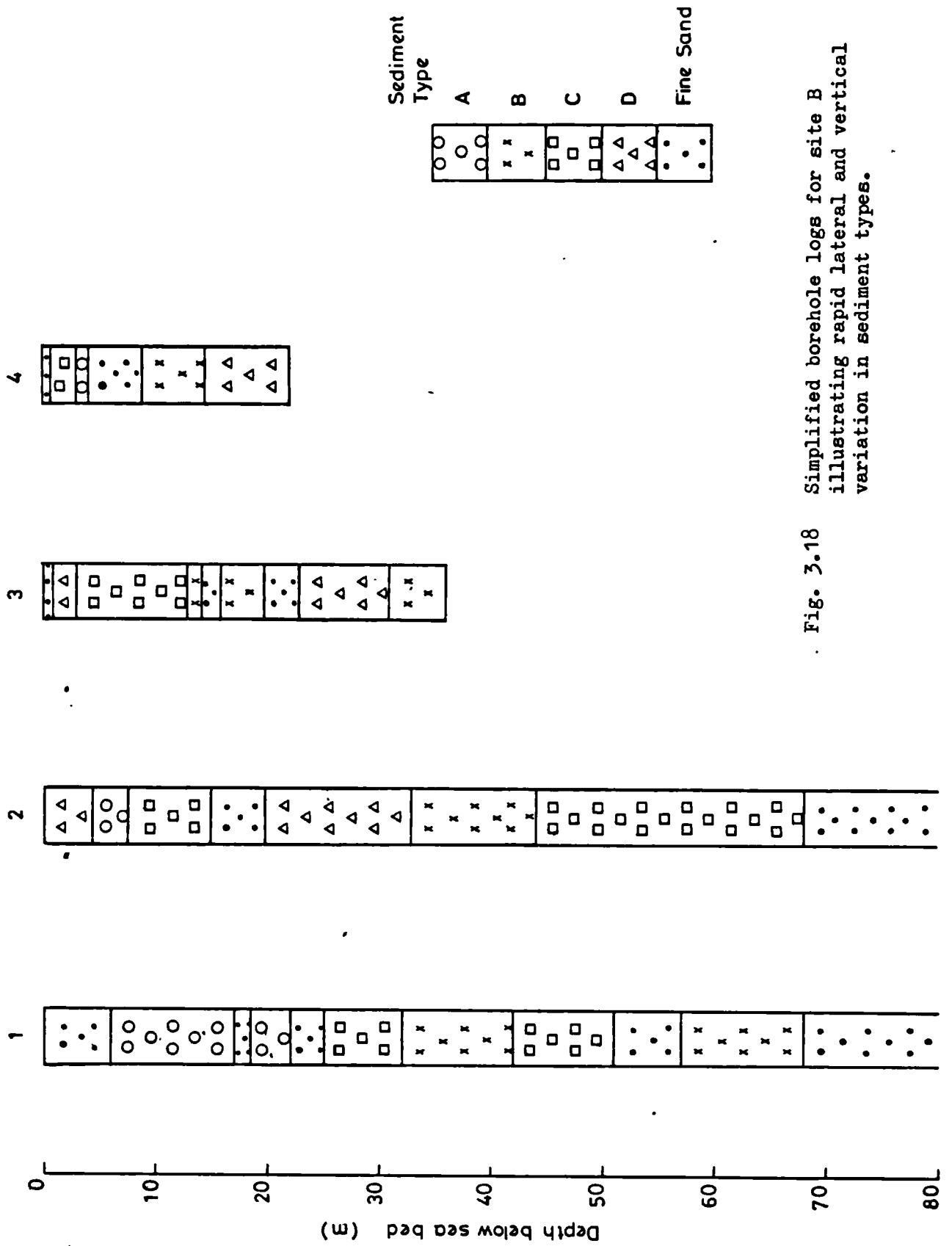


Fig. 3.17 Detail of Fabric Type 4



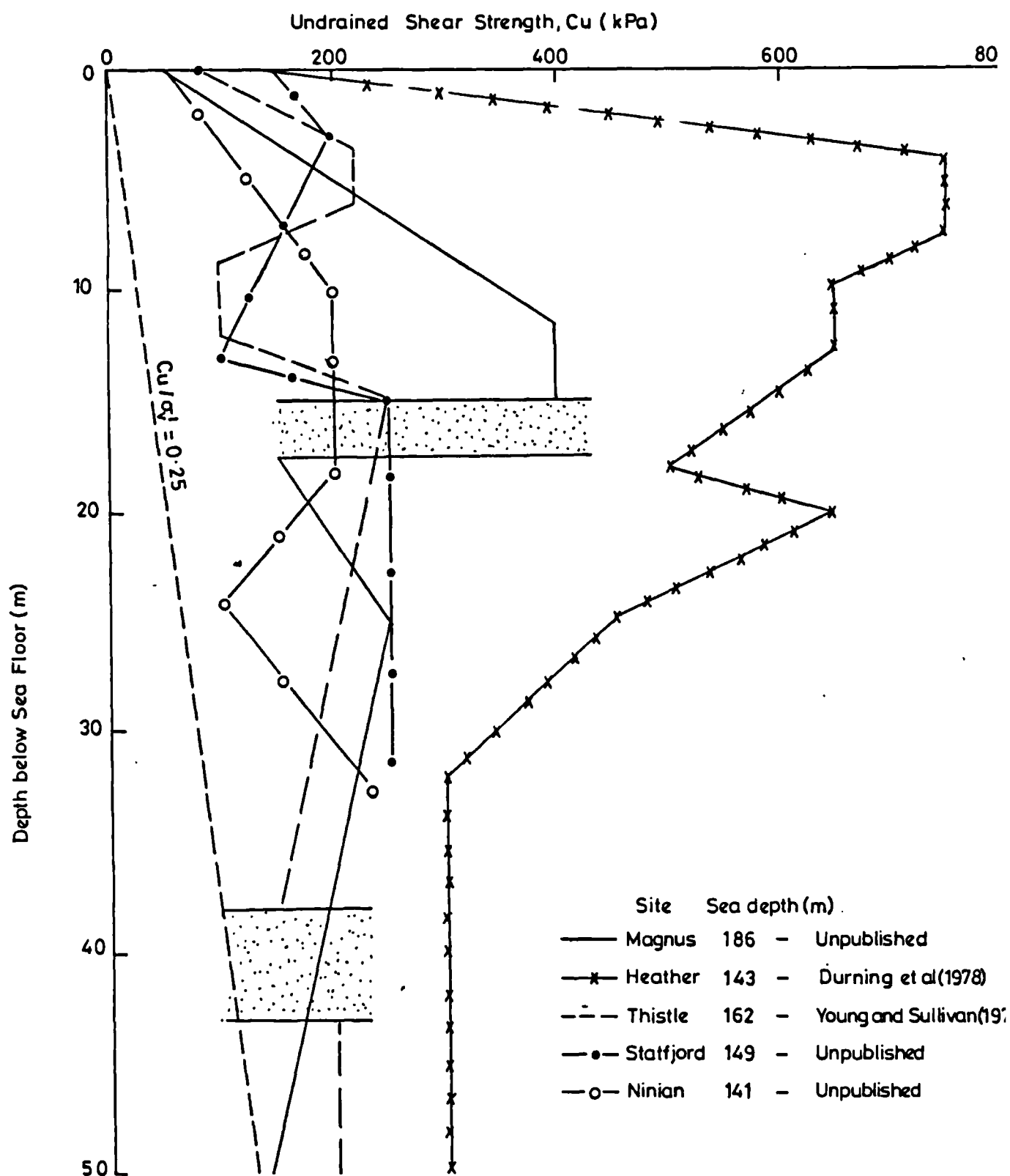


Fig. 3.19 Summary of undrained strength profiles.

CHAPTER 4

SAMPLING, SIMULANTS FOR NORTH SEA SEDIMENTS
AND RECONSTITUTED SOIL BEHAVIOUR

SYNOPSIS

The Chapter comprises three parts. In the first the special effects of sampling offshore, large total stress release, temperature change and gas exsolution, are examined. These effects are shown to preclude the use of intact samples for a fundamental investigation of the behaviour of soils from the Northern North Sea. The second part describes two terrestrial glacial tills which were selected to simulate some of the Northern North Sea sediments; they are shown to provide a match in terms of composition, mineralogy and microfabric. In the final part the usefulness of mechanical tests on reconstituted samples of these two soils is considered. It is demonstrated that for these low plasticity soils the pre-failure behaviour of intact samples in undrained compression can be matched by reconstituted samples which have been given an anisotropic consolidation history and brought to the same water content and initial effective stress. Two methods for reconstituting samples are used: mixing as a slurry and moulding at low water contents. Evidence, including that from microfabric studies, is presented which shows that these methods may simulate deposition from floating glaciers and by lodgement. The shear strength-water content relationship for the soils is found to depend on the simulated mode of deposition.

4.1 INTRODUCTION

The advantages of investigating soil behaviour using good intact samples is obvious; only in such samples can the effects of stress history, aging, bonding, fabric (both macro and micro) be correctly represented. The provision of suitable intact samples from the Northern North Sea bed for a fundamental investigation of behaviour was considered to be untenable. Their natural variability in terms of water content, plasticity, grading and microfabric has been described in Chapter 3 and makes them unsuitable for research. The possibility of obtaining sufficient quantity of material from one horizon did not exist. The limited range of sample sizes that could be taken and the obvious inability to inspect in situ exposures meant that the representativity of a sample in relation to its surrounding could not be assessed. In addition, glacial soils may have high stone contents making sampling in them particularly difficult. However, perhaps the over-riding consideration in rejecting the use of intact samples was the special effects of sampling offshore which were cited in Chapter 1: total stress release, temperature change and gas exsolution.

In addition to the usual effects of mechanical disturbance^{*1} (block samples cannot, of course, be taken), a sample retrieved from the sea bed undergoes an inevitable large release of total stress and change in temperature. The total stress release is considerably higher than in conventional on-land site investigations, simply because of the depth of overlying water, and results in the expansion of pore water and the possible release of gases. The effects are general and not specific to any soil. In the first part of this Chapter the magnitude of the effects of the total stress and temperature change are examined for offshore samples in terms of the percentage change in effective stress in the sample. It is demonstrated that at deep water sites, samples from shallow depths can be subject to a substantial reduction in effective stress; this will affect their response in subsequent testing (Chapter 12). The exsolution of gas which is liable to occur and the

¹ Evidence of mechanical disturbance is available in some of the photographs presented in Chapter 3.

reversibility of the process is discussed. Other effects of sampling, specific to low plasticity soils, are considered in Chapters 8 and 12.

As an alternative to the use of intact samples, the approach which was adopted for the research programme was to locate a terrestrial glacial clay which could be used as a simulant of one of the Northern North Sea sediment types. The second part of this Chapter describes in detail such a clay, the Lower Cromer Till. Similarity is demonstrated in terms of plasticity, grading, mineralogy and microfabric. The effects of differences in pore water chemistry are shown to be insignificant in this type of soil. A second terrestrial till, the Cowden Till, has also been used, partly to explore the generality of the findings. A description of the Cowden Till is also given in the second part of the Chapter.

To remove the effects of natural variability in the North Sea simulant and to impart a range of consolidation histories, it has been necessary to work with reconstituted soil. Two methods for reconstituting the soil were employed: in the first, air-dried batches of the soil were mixed with distilled water to form a slurry with a liquidity index of 1.5. The slurry was consolidated anisotropically or isotropically to a range of overconsolidation ratios (OCR). This soil will be referred to as the reconstituted soil and is considered to simulate glaciomarine sedimentation and subsequent consolidation. The second method involved mixing the soil with distilled water at relatively low water contents; this soil will be referred to as the remoulded soil and will be taken to represent deposition as lodgement till.

It remained to justify the use of reconstituted soil by showing that its behaviour matched that of the intact soil from which it was derived. The third part of this Chapter makes a detailed comparison between the behaviour of intact, reconstituted and remoulded samples of Cowden Till which demonstrates the agreement that can be achieved.

4.2 THE EFFECTS OF TOTAL STRESS AND TEMPERATURE CHANGE ON SAMPLES RETRIEVED OFFSHORE

The total stresses acting on a soil element in the ground are reduced to zero as a result of sampling. The precise sequence of total stress changes will vary but may be simplified as :-

- (i) a release of the total stress difference, $\sigma_v - \sigma_h$, and
- (ii) a release of an isotropic stress, σ_h , where σ_v and σ_h are the in situ vertical and horizontal total stresses respectively.

The values of σ_v and σ_h offshore exceed those at a corresponding land site by an amount equal to $\gamma_w d$ where d is the sea depth. When $\sigma_v - \sigma_h$ is released there is no difference for a sample taken offshore or on land from an equivalent depth. However, the release of isotropic stress can be much larger offshore; for a large change in isotropic stress, $\Delta\sigma$, departures of the pore pressure parameter, B , from unity can become significant.

The pore pressure set up by changes in the state of isotropic stress under undrained conditions can be predicted from a consideration of the relative compressibilities of the porous soil skeleton, C , of the pore fluid filling the voids, C_w , and of the solid material comprising the skeleton, C_s , together with the porosity, n . Bishop (1973) presented the derivation from first principles for:-

$$\frac{\Delta u}{\Delta \sigma} = \frac{1}{1 + n(C_w - C_s)/(C - C_s)} \quad 4 - 1$$

By similarly considering volume changes in each component due to both temperature and pressure an expression can be derived to relate the pore pressure set up by changes in temperature under undrained conditions. Adopting Bishop's notation and assumptions and drawing on work by Mitchell (1976), the derivation is now made.

Consider an element of volume V of a saturated porous medium, of porosity n . The volume of the pore fluid is nV and the volume of the

solid material forming the skeleton is $(1 - n)V$. Apply a temperature change of Δt under conditions of constant total stress and let Δu be the value of the pore pressure change under conditions of zero drainage, i.e. zero flow of fluid across the boundary of the element, and no cavitation.

Consider first the volume changes resulting from Δu . Following Bishop and Eldin (1950) the strains and volume changes may be considered to be the sum of those obtained by imposing:-

- (a) a total stress and pore pressure change equal in magnitude to Δu , and
- (b) an additional total stress at constant pore pressure, of magnitude $-\Delta u$.

For stage (a) the following terms must be considered:-

- (i) The decrease in volume of the pore fluid

$$(nV)C_w \Delta u \quad 4 - 2$$

- (ii) The decrease in volume of the solid material

$$(1 - n)VC_s \Delta u \quad 4 - 3$$

- (iii) The decrease in overall volume of the element, which is

$$VC_s \Delta u \quad 4 - 4$$

as the components of strain along any continuous path across the solid skeleton within the element are equal to those within the solid material of the skeleton.

For stage (b) there are two terms:-

- (iv) The decrease in overall volume of the element

$$- VC_s \Delta u \quad 4 - 5$$

- (v) A decrease in the volume of the solid components of the skeleton due to the excess of the total stress over the pore pressure given by

$$(1 - n)VC_s \frac{1}{(1 - n)} (-\Delta u) = -VC_s \Delta u \quad 4 - 6$$

Consider next the volume changes resulting from Δt . The changes are:-

- (i) The increase in volume of the pore fluid

$$\alpha_w(nV) \Delta t \quad 4 - 7$$

where α_w is the thermal expansion coefficient of soil water.

- (ii) The increase in volume of the solid material

$$\alpha_s(1 - n)V \Delta t \quad 4 - 8$$

where α_s is the thermal coefficient of cubical expansion of the mineral solids.

- (iii) The increase in overall volume of the element which is made up of two components. If relative movement of soil grains does not occur, the volumetric strain of the soil mass is equal to the volumetric strain in the soil grains

$$V \alpha_s \Delta t \quad 4 - 9$$

However, the change in temperature may induce changes in inter-particle forces, cohesion, and frictional resistance which results in soil grain movement and reorientation. Mitchell (1976) relates this volume change to α_{ST} , the physico-chemical coefficient of structural volume change, giving

$$V \alpha_{ST} \Delta t \quad 4 - 10$$

For zero drainage, the sum of the separate volume changes of the soil constituents due to both temperature and pressure change must equal the sum of the volume changes of the element due to both temperature and pressure changes.

Taking decreases in volume as positive we have for the changes in overall volume, equations 4 - 4, 4 - 6, 4 - 9 and 4 - 10, and for the changes in the constituents, equations 4 - 2, 4 - 3, 4 - 5, 4 - 7 and 4 - 8. These lead to:-

$$\begin{aligned} VC_s \Delta u - VC \Delta u - V \Delta t (\alpha_s + \alpha_{ST}) \\ = nVC_w \Delta u + (1 - n)VC_s \Delta u - VC_s \Delta u \\ - nV \alpha_w \Delta t - (1 - n)VC_s \Delta t \end{aligned} \quad 4 - 11$$

which simplifies to:-

$$\frac{\Delta u}{\Delta t} = \frac{\alpha_{ST} + n(\alpha_s - \alpha_w)}{(C_s - C) + n(C_s - C_w)} \quad 4 - 12$$

Equations 4 - 1 and 4 - 12 can be used to examine the effects of total stress release and temperature change on the effective stress in samples taken from the sea bed. It is necessary first to consider the probable range for the expressions for $\Delta u / \Delta \sigma$ and $\Delta u / \Delta t$. Values for physical constants involved are given in Table 4.1.

The influence of the value of the compressibility C and of the porosity n on $\Delta u / \Delta \sigma$ have been considered by Bishop and Hight (1977). Fig. 4.1(a), modified from their paper, illustrates that in the higher range of compressibilities B is significantly less than unity, but generally exceeds 0.9. In the range of compressibilities encountered in rocks and soils at high consolidation pressures, the value of B will be greatly reduced and in the limit approach zero as the value of C approaches C_s .

Unfortunately values for the expression for $\Delta u / \Delta t$ are dependent on the parameter α_{ST} and there is little published experimental data on

its value for different soils. Mitchell (1976) quotes a value for remoulded illite of about $-0.5 \times 10^{-4}/^{\circ}\text{C}$. Attempts to backcalculate values from the temperature-induced pore pressure changes tabulated by Mitchell (see his Table 13.8) are thwarted by a lack of accurate data on n and C for the soils. However, negative values are indicated, suggesting limited structural collapse with increasing temperature.

Inspection of equation 4 - 12 and of the constants in Table 4.1 reveals that the effect of a negative α_{ST} is to increase $\Delta u / \Delta t$. Lower bounds to $\Delta u / \Delta t$ can be derived by setting α_{ST} to zero. This has been done to derive Fig. 4.1(b). As compressibility reduces the parameter T increases markedly.

The effects of total stress release, $\Delta \sigma$, and temperature change, Δt , can now be assessed for a number of situations. In Fig. 4.2(a) the percentage loss in effective stress has been plotted against water depth for two values of B and for two sampling depths, d , below sea bed level. For reasonable values of B , which allow for the low compressibilities applying to unloading, losses in effective stress approaching 100% can occur.

Losses in effective stress of the same order can result from temperature change as shown in Fig. 4.2(b). Plots are presented for two values of T and for the same sampling depths, 10m and 50m.

In both cases, the percentage change in effective stress is greatest in the soils of low compressibility (i.e. low B , high T) at shallow depths below sea bed in deep water.

Only the effect of temperature change during sampling on effective stress has been examined. Reference will be made subsequently to the effect of temperature on gas solubility in pore water and in Chapter 9 some tentative conclusions will be made regarding the effect of temperature on response to transient loading. Temperature will have an important influence on other aspects of soil behaviour but these have not been investigated.

4.3 GAS EXSOLUTION IN OFFSHORE SAMPLES

The evidence presented in Chapter 3 indicates that several of the soil types in the Northern North Sea were sedimented rapidly in shallow water, trapping organisms. Subsequently there was the generation of biogenic gas which was forced into solution by the simultaneous increase in sea depth and hence pore pressure. As a direct consequence, the soils can be expected to be associated with large gas concentrations in the pore water. It is feasible that the pore water is gas-saturated and that undissolved gas is present; in fact, there is geophysical evidence for the presence of free gas in the form of acoustic blanketing (Holmes, 1977).

Exsolution of these gases occurs when the pore fluid pressure reduces below the liquid - gas saturation pressure and bubbles form in the pore space. The process is time dependent and continues until there is equilibrium between the liquid and gas pressures and gas concentrations in the bubble and in the liquid (Morgenstern, 1981).

As a result of sampling, and particularly because of the large reduction in pore pressure and of the high gas concentrations which are involved, gas exsolution is a high probability in samples from the Northern North Sea. Indeed, cavitation, or gases coming out of solution, will not necessarily depend on the development of negative pore water pressures.

The process by which gas comes out of solution from a large body of water when its pressure is reduced is well-understood. In theory, the process is reversible, i.e. gases re-dissolve when the pressure is increased.

The process by which gas comes out of solution in the pores of an initially saturated soil (i.e. the process by which cavitation occurs) does not appear to have been investigated in detail. Of interest, here, is whether this process is reversible, i.e. whether, if gases come out of solution on the release of total stresses during sampling, they are forced back into solution and the sample returns to its

original volume when the total stresses are reimposed. This is equivalent to enquiring whether changes in the pore pressure parameter B are reversible.

As gases exsolve they must occupy voids previously occupied by water or occupy voids enlarged by a build-up in gas pressure. The displaced water is imbibed by the soil surrounding the void which is newly filled by gas. While the swelling may only occur locally, there is an overall increase in the sample volume, as depicted in Fig. 4.3. Since the movement of water is involved, there is an additional time dependency which is related to local permeabilities.

In sands there will be relatively rapid removal of pore water by exsolving gas, which is equivalent to the sand having a low blow-through pressure; gas can become continuous quite quickly. In clays, low permeability will restrict the movement of water and promote the development of enlarged voids, the growth of which will be determined by the stiffness and strength of the surrounding soil. The process will continue until the gas can drain or exsolution is complete. In low permeability soils undrained gas expansion will lead to disruption of the fabric as voids are enlarged. Evidence for fabric changes caused by gas exsolution is presented in Chapter 3 and by Fannin (1979).

The pressure at which gases exsolve is obviously dependent on temperature and the initial volume of dissolved gas. For constant values of these, it seems likely that the pressure at which cavitation occurs will vary with void size in the manner shown in Fig. 4.4. (Bishop et al (1975) provide evidence for the dependence of cavitation pressure on void size.) For a uniform distribution of gas in solution, exsolution of gases will take place first in the larger voids.

As water is made available to the soil surrounding a void which becomes filled by gas and as swelling takes place, pore water suctions are reduced. The reduction in suction (Δp_F) will be related, in part, to the volume of water which becomes available, i.e. to the void size which is evacuated. Since we are postulating that the larger voids are evacuated first, then the reduction in suction will tend to be larger initially; this is illustrated in Fig. 4.5.

Combining the postulates of Figs 4.4 and 4.5 then a stage will be reached at which the suction is reduced to a level equal to the cavitation pressure in the largest remaining water-filled voids. Beyond this there will be no further cavitation, Fig. 4.6.

The extent to which cavitation will occur will depend closely on the range of void sizes, and hence on the fabric, present in the soil. In coarse-grained soils with large void sizes, cavitation may cause partial drainage of the soil; in fine-grained soils, cavitation may not take place. In soils containing fine and coarse fabrics, as in many of the examples shown in Chapter 3, cavitation in the coarser material will allow swelling of the fine-grained soil.

When samples taken from the sea-bed are tested in the laboratory, the total stresses which acted in situ may be fully or partially re-imposed. In a sample which remains saturated, application of a sufficiently large isotropic stress enables an estimate to be made of the initial suction in the sample, p_k' , via a measurement of the initial effective stress, p_o' , see Fig. 4.7.

In samples in which gases exsolve, the correct effective stresses will only be re-established if changes in the pore pressure parameter B are reversible. There are several reasons to suppose that this will not be the case:-

- (i) Reimposition of total stresses on a de-saturated sample must involve re-consolidation of the swelled soil, with water draining towards the gas-filled voids. It is well-established that this process is not reversible in saturated soils.
- (ii) The evacuation of water from voids as gas exsolves and its subsequent return when total stresses are applied is analogous with the wetting and drying cycle experienced by a soil when its suction-water content is established. There is hysteresis in this process as shown in Fig. 4.8. In other words, although cavitation will occur first in the larger voids, solution of gases may well take place first in the smaller voids.

Two additional factors contribute to the lack of reversibility of the process:-

- (i) the change in solubility of gas with change in temperature, and
- (ii) the change with time of the distribution of gas within the pores as a result of diffusion.

As discussed in Chapter 3, temperatures at the sea bed average around 5°C to 6°C. During testing in the laboratory, temperatures may be between 15°C and 20°C. The change in solubility over this temperature range for a number of gases is summarised in Table 4.2. There is a marked reduction in solubility with increase in temperature.

Following the hypothesis presented above, the exsolved gases will occupy a range of the larger void sizes. As a result of surface tension effects, the pressure of the gas in voids of different size will vary, the smaller gas-filled voids being associated with the higher pressures. Pressure differentials will give rise to mass transfer of gas by diffusion from smaller bubbles at higher pressure to larger bubbles at lower pressure. Larger bubbles will grow in size while smaller bubbles will reduce in size and may collapse. Williams (1967) has observed this phenomenon.

Changes in the volume and distribution of gas and of bubble size lead to changes in the overall compressibility of the pore fluid (water + gas) and therefore changes in the value of the pore pressure parameter, B . Both the reduced solubility of the gases at the elevated temperatures in the laboratory, and the growth in bubble size with diffusion would point to a lower B for reloading than for unloading. The irreversibility is illustrated in Fig. 4.9.

The consequences of a B value different for unloading and reloading would appear to be as follows:-

- (i) Initial effective stress, p_o' , cannot be related directly to the suction in the sample, p_k' , (Fig. 4.7) and so an estimate cannot be made of the effective stresses existing after sampling.

- (ii) For B reloading $\leq B$ unloading, samples, particularly stiff ones, will be difficult to saturate even when the full total stresses are re-imposed. The time required to achieve full saturation may be long.
- (iii) Non-reversible changes in fabric may occur when reimposing total stresses.

Shearing a sample which is not fully saturated will, in turn, result in an error in the measured C_u . The sign and magnitude of the error will depend on the shape of the effective stress path, i.e. on the pore pressure parameter A (Fig. 4.10). In unconsolidated undrained samples which have not been fully saturated, additional scatter in data on C_u and stiffness will be introduced. Even if saturation is achieved, the lower pore pressures on reapplication of total stress will mean that cavitation may occur during dilation. Based on data from Bishop et al (1975) this may be reflected in increased brittleness in the stress-strain curves.

It may be concluded that exsolution of gases during and subsequent to sampling will lead to:-

- (i) expansion of the soil and possibly disruption of its fabric which is additional to that caused by mechanical disturbance; this will continue until gas drainage occurs;
- (ii) swelling local to the gas-filled voids which produces water content variations in the soil on a microscale.

The process is probably not reversible, in other words, reimposition of the total stresses will not force all gas back into solution; an initially saturated sample may no longer be saturated (Fig. 4.10). This arises because of:-

- (i) the non-reversibility of the local swelling-reconsolidation process around the gas-filled voids;

- (ii) changes in gas solubility with temperature; and
- (iii) changes with time in the distribution of gas in the soil and hence changes in the compressibility of the pore fluid and, therefore, of the pore pressure parameter, B.

The process is time-dependent, involving both the flow of water and diffusion of gas. There appears to be good advantages, then, in keeping the time between sampling and testing to an absolute minimum and reimposing the full total stresses.

The arguments put forward in Sections 4.2 and 4.3 indicate that gas exsolution alone will make samples retrieved from deep water sites unsuitable for research. However, even if gas exsolution does not occur, the combined effects of total stress release and temperature change on effective stresses in saturated samples is unsatisfactory. The alternative approach of locating a simulant for North Sea sediments to avoid these problems and those posed by natural variability appears to be justified.

4.4 SIMULANTS FOR GLACIAL SEDIMENTS IN THE NORTHERN NORTH SEA

To produce material which might match the behaviour of sediments in the Northern North Sea, it would appear necessary to ensure similarity in terms of:-

- (i) composition,
- (ii) method of deposition/forming,
- (iii) environment of deposition - e.g. chemistry and temperature,
- (iv) post-depositional changes - e.g. bonding and fissuring,
- (v) aging, and
- (vi) drained and undrained stress history,

or to show that any of these factors is unimportant.

In Chapter 3, the composition of the glacial sediments in the areas and depths of interest was reviewed. The range of depositional processes and environments which operated and of subsequent stress histories were also described. This information has been used to identify soil types and reconstitution methods which might successfully reproduce intact material behaviour.

Soil composition and depositional mode and environment determine initial soil fabric. This initial fabric is modified by stress history and post-depositional changes. Similarity of fabrics between the Northern North Sea sediments and material selected to represent them might indicate that items (i) to (iv) above are satisfactorily dealt with. Considerable attention is paid, therefore, to a comparison of the microfabric of Northern North Sea sediments presented in Chapter 3 and the microfabric of soils chosen to simulate them.

Reproduction of the fabric of intact soil may be difficult to achieve. The level of difficulty might relate to the level of ordering which can develop in the clay fraction. With reducing clay content both should reduce. There would appear, then, to be reasonable prospects for matching behaviour in the low plasticity clays offshore. This approach has been tried for soils with Fabric Type 2 which have been seen to be widespread, to have a featureless macrofabric and to have a relatively simple and regular microfabric; furthermore there is no reason to expect any effects of weathering on their fabric.

The Lower Cromer Till

The characteristics of several terrestrial glacial deposits described in the literature were examined in the search for a match with North Sea sediments with Fabric Type 2. One of the best matches appeared to be provided by a low plasticity sandy clay, the Lower Cromer Till. Its geological history and engineering geology had been discussed by Kazi and Knill (1969). In common with the Northern North Sea sediments it was Pleistocene in age, having been deposited during the Cromer Advance. It was reported to be a dense, stiff overconsolidated grey or dark grey sandy clay which was unstratified and non-laminated, but jointed. It

contains pebbles of flint, quartz, schist, gneiss and igneous rocks of Scottish and Scandinavian origin. The pebbles are irregularly shaped, generally less than 50mm and distributed sporadically.

The till was readily accessible and ample soil could be taken for the full research programme, including for the preparation of soil beds for the large model test facility (Herbertson, 1977, Martins, 1980). It was sampled from the base of the cliffs at Happisburgh, Norfolk (National Grid ref. TG383311); a site location plan is shown in Fig. 4.11. Both block samples and large disturbed samples, totalling 2 cu.m., were taken from the cliff face.

The soil showed considerable natural variability in grading. A grading envelope based on the results of individual samples is shown in Fig. 4.12. To reduce variability in the reconstituted samples large batches of soil were prepared; the small percentage of gravel in the natural soil, generally less than 4%, was eliminated from the batches. The resulting average grading for the reconstituted soil is shown in Fig. 4.12. Comparison with Fig. 3.12 shows that the grading is very similar to that of group A of the Northern North Sea sediments. The low silt content, and large proportion of fine sand, i.e. gap grading, should be noted.

The Lower Cromer Till has a clay fraction of approximately 15% and average liquid and plastic limits of 25% and 12% respectively, giving an activity of 0.86. This low plasticity and activity is again typical of the Northern North Sea sediments in group A (c.f. Table 3.7). In terms of plasticity the soil represents a lower bound to the offshore sediments. The behaviour of such low plasticity clays did not appear to have been investigated previously.

The mineralogy of the Lower Cromer Till was determined by X-ray diffraction analysis and the results are presented in Table 4.3. The similarity with Northern North Sea sediments is evident (c.f. Table 3.8).

No claims are made regarding similarity in stress history of the Lower Cromer Till and North Sea sediments. Overconsolidation in the

Lower Cromer Till is regarded as resulting from ice loading and from unloading by erosion and cliff instability. In fact, the undisturbed material was not used in a detailed study of behaviour because of the uncertain stress history, particularly associated with exposure in the cliff face. As for mode of deposition it will be suggested that the Lower Cromer Till is of glaciomarine or glaciolacustrine origin. Evidence will be presented shortly showing similarity in microfabric between the intact and reconstituted soil, i.e. that consolidated from slurry. In Chapter 8 it will be shown that strengths of the intact soil correlate with those measured in the reconstituted soil as opposed to the remoulded soil. The low silt content is also indicative of a lack of subglacial shearing and, therefore, of a glaciomarine origin. It is encouraging that after selecting the Lower Cromer Till as the North Sea simulant, Redding (1976) commented on the similarity between the drift deposits along the Norfolk coast and the younger glacial deposits in the Northern North Sea.

The pore water chemistry of the intact Lower Cromer Till is shown in Table 4.3. Obviously this chemistry is changed in reconstituting the soil and will be different from that in the Northern North Sea sediments (Table 3.9). As a simple check on the potential influence of pore water chemistry on the behaviour of these low plasticity clays, the Atterberg Limits for a sample of the more plastic Lower Cromer Till were found:-

- (i) using distilled water, and
- (ii) using saline water (NaCl added at 35gm/litre).

The recorded values were:-

	<u>Liquid Limit (%)</u>	<u>Plastic Limit (%)</u>
Distilled Water	28	13
Saline Water	27	14

The effects of pore water chemistry appear to be small and can reasonably be neglected. (By avoiding the use of saline water, potential problems with osmosis are also avoided.)

The Cowden Till

The samples of Cowden Till used in the research programme were taken from a depth between 4.0 and 5.9m at the BRE test-bed site at Cowden on the Yorkshire Coast. The water table was approximately 1m below ground level. The till is a red-brown glacial clay with up to 12% gravel and pebble content. Chalk is the predominant clast (>30%) with sandstone, shales, flint, siltstone and limestone also present; clasts are subangular to rounded. The clay is reckoned to be a terrestrial till deposited subglacially.

A grading for the soil is presented in Fig. 4.12. The clay content, 24%, is considerably higher than in the Lower Cromer Till and the sand content, 28%, is considerably less. An important contrast lies in the silt content of the Cowden Till, 41% c.f. <25% in the Lower Cromer Till. The clay has a liquid limit of 32% and plastic limit of 16%; its mineralogy is summarised in Table 4.4 and the similarity with Lower Cromer Till is clear.

4.5 FABRIC OF THE NORTH SEA SIMULANTS

The techniques for fabric studies were the same as those described in Chapter 3. For the SEM, samples were again prepared by air drying and fracture. In the low plasticity sandy clays which were examined, the fracture surface is likely to show a preference for following the clay. This leads to the appearance of a surprisingly large clay content which should be contrasted to the low clay content evident in thin sections. In the surfaces examined in the SEM, sand grains are frequently hidden by a clay coating and the fracture surface shows sockets left by sand grains which have been plucked out.

Intact Lower Cromer Till

Texturally the Lower Cromer Till is homogeneous with little or no macrofabric features, refer Plate 4.1 and note the resemblance with

North Sea sediment types 2 and 5 (Plates 3.2 and 3.3).

Shell fragments can be seen in the X-ray to be randomly distributed and clasts to be infrequent; there is no evidence of stratification. The thin sections emphasise the homogeneity at macroscale.

The microfabric of the intact Lower Cromer Till observed in the scanning electron microscope and in thin sections under the polarising microscope is illustrated in Plates 4.2, 4.3 and 4.10(a)-(d). The essential features to note are:-

- (i) Sand grains are not in contact but are separated by domains of oriented clay which will be referred to as clay bridges (e.g. Plates 4.2(d)-(f), 4.3 (b) and (c)).
- (ii) In some clay bridges, the clay appears to have fused (Plates 4.2(e) and (f), 4.3(d)); these crystalline forms represent contact bonds (Katti, 1981, pers. comm.).
- (iii) The clay bridges can be seen between sand grains (Plates 4.2(c) and (d)) and as the boundary to holes left where sand grains have been plucked out during sample preparation (Plate 4.3(b)).
- (iv) Clay filling the voids between the sand grains is much more open than the clay in the bridges (Plates 4.2(g) and (h)) and with occasional edge to face contact it develops a honeycomb fabric; this clay will be referred to as the matrix clay.
- (v) The clay bridge - clay matrix fabric is evident at magnifications of 500 times.

Comparison of this microfabric with that of Northern North Sea Fabric Type 2 presented in Chapter 3 shows that there is remarkable similarity. Of course this was the aim, but similitude had only been shown to this point in terms of grading, plasticity and mineralogy.

Reconstituted Lower Cromer Till

The microfabric of the Lower Cromer Till reconstituted by anisotropic consolidation from a slurry ($LI = 1.5$) is illustrated in Plates 4.4 and 4.10(g) and (h). There appears to be the desired similarity between this microfabric and that of the intact soil shown in Plates 4.2 and 4.10(a)-(d). There is the same distinct clay bridge - clay matrix structure. Clay bridges are illustrated in Plates 4.4(a) to (c), (e) and (f); clay matrix is shown in Plates 4.4(g) and (h), and Plate 4.4(d) shows an area of clay matrix disturbed by a silt grain. Sand grains are spread more or less regularly. There is an absence of voids (c.f. the remoulded soil which will be described). There is variation in the width of the clay bridges (c.f. Plates 4.4(a) and (f)) and this is reflected in the packing of the clay within the bridges; in narrow bridges the clay is dense and highly oriented; in wide bridges the clay takes on the character of the clay in the matrix. No contact bonds have developed which may indicate that this is a time related process or requires higher stress levels.

The thin sections of the intact and reconstituted Lower Cromer Till in Plate 4.10 confirm the absence of grain to grain contacts and show some orientation of clay between the grains. In the reconstituted soil (Plates 4.10(e)-(h)) there is a more uniform distribution of the sand fraction than in the intact soil (Plates 4.10(a)-(d)). (Non-uniform distribution of the sand fraction was a feature of the intact samples of North Sea sediments illustrated in Chapter 3.) Comparing the samples reconstituted by anisotropic (Plates 4.10(g) and (h)) and isotropic (Plates 4.10(e) and (f)) consolidation there is evidence of orientation of the granular fraction in the anisotropically consolidated soil, with elongated sand grains oriented perpendicular to the major principal stress direction during consolidation. In both the intact soil and in the anisotropically consolidated reconstituted soil there is indication of load trajectories (load-carrying struts) having developed (e.g. Plate 4.10(b)).

The similarity of the intact and reconstituted soil fabrics suggests that the intact soil has a glaciomarine or glaciolacustrine

origin since either would imply a high placement water content as used for the reconstituted soil.

In these fabrics it is of interest to note how the sand grains contribute to a densification of the clay, locally reducing water content and creating areas of increased density; the homogeneity observed at macroscale clearly does not persist at microscale - this will be considered further in Chapter 11.

Remoulded Lower Cromer Till

The microfabric of the Lower Cromer Till, when remoulded at a water content of 15% and examined before consolidation, is shown in Plate 4.5. There is a distinct fabric for the remoulded soil which is quite different from that of the intact and reconstituted soil:-

- (i) Voids are present (especially adjacent to sand grains) which do not have the shape of a sand grain and do not represent the socket left by a sand grain which has been plucked out (Plates 4.5(c) and (d)); they appear to be a remnant feature of the remoulding process associated with the previous rotation (sliding of the granular component as it has ploughed through the clay (Fig. 4.13)).
- (ii) No well-developed clay bridges are found but there are 'whorls' within the clay (Plates 4.5(e)-(g)), presumably evidence of where the clay has been dragged along by a sand grain during the remoulding process (Fig. 4.13). It is suggested that this feature of the microfabric in an intact low plasticity soil might be used as an indicator that lodgement was the probable mode of deposition.
- (iii) Apart from this local preferred orientation, there is no other evidence of ordering in the clay which is open and which can be said to be random (c.f. the ordered clay matrix in the intact and reconstituted soil).

- (iv) There tends to be segregation with pockets where sand grains have congregated and are in grain-to-grain contact (Plates 4.5(a) and (b)). The distribution of the sand grains is generally irregular.

This open and unordered fabric is entirely consistent with the generally 'soft' feel of the remoulded soil.

Intact Cowden Till

Over the depth from which the Cowden Till was sampled the soil shows nearly no macrofabric features (Marsland and Powell, 1979). The microfabric (Plate 4.6) on the other hand is particularly interesting:-

- (i) A clay bridge-clay matrix fabric is evident to a degree but is not as well-developed as in the Lower Cromer Till. Silt is present within the clay bridges (e.g. Plate 4.6(e)) and within the clay matrix (e.g. Plates 4.6(g) and (h)). Clay coats both the sand and silt grains making recognition difficult and often giving the general appearance of a clay.
- (ii) 'Whorls' of clay, i.e. thin clay seams or ribbons, are present (e.g. Plates 4.6(b) and (c)). These 'drag' structures were noted in the fabric of the remoulded Lower Cromer Till and will be seen again in the remoulded Cowden Till. As discussed, these will be taken as an indication that the placement involved a remoulding process, i.e. as lodgement till.
- (iii) Clay particles are generally aggregated with occasional edge to face contacts. A notable feature is the honeycomb formed by clay plates, e.g. Plate 4.6(d); these may represent the skin of clay around grains which have been removed - if so they indicate how thin this skin may be.
- (iv) Again, as in the remoulded Lower Cromer Till, there is occasional grain to grain contact (e.g. Plate 4.6(b)).
- (v) Details of the microfabric are only discernible at magnifications greater than 1000 times (c.f. 500 times for the Lower Cromer Till).

Reconstituted Cowden Till

The microfabric of reconstituted Cowden Till, anisotropically consolidated from slurry, is illustrated in Plate 4.7. There is no strong similarity with the intact soil. The clay bridge-clay matrix fabric is probably better developed than in the intact soil (see Plates 4.7(e) and (f)). As in the Lower Cromer Till, the thickness of the clay bridges is variable and the orientation in the clay reduces as the thickness increases. The increased presence of silt (c.f. Lower Cromer Till) and its effect on the development of the fabric is evident in Plates 4.7(a) and (b). Clay coats to sand grains are more prevalent than in the intact Cowden. There are no clay 'whorls' and no inter-weaving clay chains or honeycomb patterns.

Remoulded Cowden Till

The microfabric of the Cowden Till moulded at a water content of 20% is illustrated in Plates 4.8 and 4.9. The similarity with the fabric of the intact soil is quite remarkable. The clay 'whorls' or ribbons and honeycomb pattern are much in evidence, e.g. Plates 4.8(a)-(d) and Plates 4.9(a) and (b), (c.f. Plates 4.6(c)-(f) in the intact soil). Again, the clay 'whorls' or drag structures, are almost certainly a relic of the remoulding process. Areas of matrix clay are relatively open, e.g. Plates 4.8(f) and (g), (c.f. Plates 4.6(g) and (h) in the intact soil) and there is interaction between the clay and silt. There is occasional segregation as in the intact soil and remoulded Lower Cromer Till, with grain to grain contact, both between sand grains and silt grains, e.g. Plate 4.8(h) and Plate 4.9(d). Possibly as a result of the large number of silt and sand grains and the low moulding water content there is occasionally a lack of clay 'coat' to these grains. In general terms the fabric of the remoulded Cowden Till is more uniform than that of the remoulded Lower Cromer Till.

In summary, it is evident that:-

- (i) The fabric of the intact Lower Cromer Till matches well sediments from the Northern North Sea having Fabric Type 2.

- (ii) For Lower Cromer Till the microfabric of the intact soil is matched by that of the soil reconstituted by anisotropic consolidation from a slurry. This, together with the absence of 'drag' structures in the intact soil would be consistent with a glaciomarine or glaciolacustrine origin for the till.
- (iii) For Cowden Till the microfabric of the intact soil is matched by that of the remoulded soil. 'Drag' structures are present in both but not in the reconstituted soil. This could indicate that the Cowden Till is a lodgement till.
- (iv) There is no evidence of bonding in the intact samples of either the Lower Cromer Till or Cowden Till.

4.6 MATCHING INTACT AND RECONSTITUTED SOIL BEHAVIOUR - COWDEN TILL

There are few precedents for the successful matching of intact soil behaviour by reconstituted soil. Figure 4.14 shows the results of tests on undisturbed and remoulded samples of a normally consolidated and an overconsolidated clay. In both cases the difference in behaviour between remoulded and undisturbed samples is very large. Even reconstitution of plastic sedimentary clays by consolidation from a slurry fails to produce matching behaviour. Bishop, Webb and Lewin (1965) have compared the undrained shear strength and effective stress path in compression of intact samples of London Clay and samples reconstituted from slurry; their results, presented in Fig. 4.14, suggest that an ordered structure was developed in the reconstituted soil but that there was little similarity in behaviour between the two types of sample. Presumably this was because effects induced by aging and bonding had not been correctly simulated.

In contrast to plastic clays, evidence for the agreement between undrained shear strengths measured on undisturbed and remoulded samples of well-graded, low plasticity glacial tills, i.e. evidence for a sensitivity of one, has been provided by Skempton and Bishop (1954), Skempton and Brown (1961) and Vaughan et al (1975). In these types of soil, shear strengths tend only to be fully mobilised at large

strains, by which stage initial soil fabrics are largely destroyed and, therefore, agreement in shear strength does not necessarily imply agreement in pre-failure behaviour. However, the matching of fabrics for intact and reconstituted low plasticity soil which has been described encourages the belief that similarity of pre-failure behaviour might also be obtained.

In this section a laboratory investigation is described which had the aim of ascertaining the requirements that reconstituted samples should meet in order that their behaviour, both at and prior to failure, matches that of undisturbed material. This has been achieved by comparing the behaviour in undrained triaxial compression of undisturbed samples and of samples reconstituted in various ways and with various stress histories. The investigation concentrated on the use of Cowden Till since high quality undisturbed samples, 98mm diameter, were available.

Testing Programme on Cowden Till

The testing programme on Cowden Till is summarised in Table 4.5. In series A and B unconsolidated undrained tests were carried out on the samples of intact soil. In series C the intact soil was remoulded and tested without consolidation; two methods were used for remoulding: manual kneading and static compaction after thorough breakdown of the intact soil. For series D to G air dried material was mixed with water contents of 23% (LI = 0.44, series D and F) and 37.5% (LI = 1.34, series E and G); these samples are referred to as reconstituted. The samples mixed to a slurry (LI = 1.34) were consolidated either isotropically (series E) in the triaxial cell using a technique described by El-Ghamrawy (1978) or anisotropically (series G). Anisotropic consolidation was carried out initially in a 228mm oedometer to low stress levels; samples were cut from the resulting block and the anisotropic consolidation continued in the triaxial cell. The drier samples were also consolidated both isotropically (series D) and anisotropically (series F) in the triaxial cell.

The intact samples showed variations in water content and initial effective stress, p_o' . The reconstituted samples were prepared in

such a way as to cover this range. Isotropically consolidated samples were given different overconsolidation ratios using a maximum consolidation pressure of 800 kPa. The aim in anisotropic consolidation was to obtain samples with water contents similar to the average for the intact samples under the same in situ effective vertical pressure (estimated to be 80 kPa). Anisotropic consolidation approximately maintained K_0 conditions following stress paths suggested by Wroth (1975) and shown in Fig. 4.15. To provide a range in p_0' for the anisotropically consolidated samples, and, in particular to match p_0' in intact samples A3 and A1, sample F2 was consolidated and F3 was swelled isotropically after undrained release of the shear stress, i.e. perfect sampling, refer Fig. 4.15.

Finally, series H was undertaken to provide information on the effects of sampling. Soil mixed at a water content of 37.5% was consolidated in a 228mm diameter oedometer to a vertical stress which, on swelling back to the estimated in situ vertical stress, would provide a block at a similar shear strength to the intact samples. 38mm diameter samples were cut from the block for testing.

Testing equipment

The hydraulic triaxial cell (Bishop and Wesley, 1975) was used for testing samples which were consolidated anisotropically. Other tests were performed in conventional 100mm and 38mm triaxial apparatus. No lubricated ends were provided.

Pore pressures were measured at the base of the 38mm diameter samples and at the mid-height of the 98mm diameter samples using a probe. The same strain rate was used in all the tests to eliminate any possible rate effect and it was such that at least 90% equalisation was achieved when the axial strain reached 0.3%

Effect of gravel content

The gravel content which can present problems in taking intact samples of glacial tills also presents problems in reconstituting them.

To ensure uniformity both within a sample and between samples and to facilitate their preparation, it is preferable to eliminate some of the coarser fraction. In doing so, a balance should be achieved between the following requirements:

- (i) the material eliminated should not constitute a large enough proportion of the soil to affect its behaviour, and
- (ii) the maximum particle size included should be small compared to the sample size.

For the material used in the investigation, the 38mm diameter by 75mm high reconstituted and remoulded samples had a maximum particle size of 2mm. The percentage larger than 2mm, i.e. the gravel fraction, ranged between 5.2% and 12.4% in the intact samples. To check whether the elimination of this proportion affected behaviour, four samples were formed with gravel contents of 0%, 3.66%, 7.07% and 11.70%. After isotropic consolidation to give a p_o'/C_u similar to that of the intact samples, the four samples were sheared undrained. Fig. 4.16 shows that no significant differences were produced by varying the gravel content, up to the level found in the intact samples.

Table 4.6 shows that although the shear strength in these samples is similar, the overall water content for each varies considerably because of the presence of the gravel. The difference in water content between samples 1 and 4 is 1.2%; in terms of the shear strength-water content relationship, found for the reconstituted soil, this difference would imply a shear strength change of 50%. By measuring the saturation moisture content of the gravel fraction, the water content of the less-than-2mm fraction, which is referred to herein as the matrix, could be determined. Table 4.6 shows that the four samples had similar matrix water contents. The matrix, which has approximately the same grading in all the samples, provides, therefore, a useful basis for comparison between samples with different gravel contents.

Undrained Shear Strength, C_u

As the stress-strain curves did not usually show a peak, the shear stress at an axial strain of 21% has been taken as the undrained shear

strength, C_u , for all samples. At 21% strain the curve was approaching the horizontal so that the values are close to the ultimate undrained shear strength, $C_{u \text{ ult}}$. Comparisons between samples have been made in terms of a shear strength-water content relationship. In Fig. 4.17 there is a good correlation between C_u and water content for the reconstituted samples (series D to H). No significant differences are observed between the samples formed at different water contents and then consolidated. This is contrary to the findings for Lower Cromer Till in which differences in shear strength have been found between samples consolidated from slurry and those consolidated after remoulding at lower water contents (Chapter 8).

The relationship for intact samples, based on the overall water content is poorly defined, there being, for example, differences in shear strength of about 40% in samples of roughly similar overall water contents. However, using the matrix water content as recommended above, the scatter in the data reduces but it is still larger than for reconstituted samples. This probably reflects variation in composition of the intact samples, a variation which was removed from the reconstituted samples since they were formed from the same homogeneous batch of soil.

The corrected results for the intact samples plot below the shear strength-water content line drawn for reconstituted samples in Fig. 4.17. the average difference being 0.37% in terms of water content.

Water contents for the unconsolidated remoulded samples also had to be corrected for gravel content. The relationship between C_u and corrected water content is similar to that for intact samples and again different to that for the reconstituted soil.

Effective Stress Failure Envelope, ϕ' .

In Fig. 4.18 the values of $(\sigma_1 - \sigma_3)/2$ and $(\sigma_1' + \sigma_3')/2$ at the end of each test have been plotted to define the effective stress failure envelope. All results are contained between lines of slope 25.5° and 27° . This confirms that ϕ' can be defined from tests on remoulded or reconstituted samples, see Vaughan et al (1978).

Pre-failure Behaviour

Fig. 4.19 shows the stress-strain curves and effective stress paths for all the intact samples tested in series A and B. These provide the standard to be matched by remoulded and reconstituted samples. The variations in p_o' and shear strength which are apparent in the figure are discussed subsequently.

(a) Undrained deformation modulus, E_u .

To account for the non-linearity in the stress-strain curves, the Young's moduli chosen for comparison were the secant modulus at shear stresses equal to 25% and 50% of C_u . Since pre-failure behaviour was found to be dependent on p_o' , the comparisons have been made between E_u , normalized by C_u , and p_o' , also normalized by C_u , refer Fig. 4.20.

This dependence of E_u on p_o' in the reconstituted and intact samples is immediately apparent from Fig. 4.20. There is no significant difference between the moduli found for the different kinds of reconstituted samples at either stress level and the range of results obtained for these has been indicated in the figure to facilitate comparisons. The moduli for the intact samples are of the same order of magnitude, though generally slightly lower. The trend for the relationship between E_u/C_u and p_o'/C_u shown in Fig. 4.20 is valid only for the range of p_o'/C_u investigated and should not be extrapolated to higher values.

The deformation moduli for unconsolidated remoulded samples are significantly lower than those for reconstituted and intact samples at the same value for p_o'/C_u , except during the early stages of shear in statically compacted samples.

(b) Effective stress paths

Given the experimental difficulties in accurately determining the deformation modulus, the effective stress path becomes the more sensitive yardstick for comparison of intact and reconstituted behaviour. Fig. 4.21

represents the normalized effective stress paths for intact and reconstituted samples; comparisons can be made for different ranges of p_o'/C_u .

There is very close agreement between the stress paths for the intact and anisotropically consolidated reconstituted samples and these are different to the stress paths for isotropically consolidated reconstituted samples. The difference decreases with the value of p_o'/C_u and becomes almost negligible with p_o'/C_u between 0.35 and 0.42.

It is clear, therefore, that anisotropic consolidation of reconstituted samples is necessary for simulation of the behaviour of this intact material; the isotropically consolidated samples fail to match intact behaviour.

Only the results of samples formed initially at a water content of 23% have been plotted in Fig. 4.21. The samples mixed at a water content of 37.5% gave practically the same results.

Fig. 4.22 shows the normalized stress paths for the unconsolidated remoulded samples. Comparisons are made difficult by the variation in p_o'/C_u which depend on the remoulding procedure. However, it appears that the statically compacted and manually kneaded samples have different initial stress paths, which do not coincide with those for the intact or reconstituted samples.

Effects of Sampling

Tests F2 and F3 were designed to provide information on sampling effects and should be compared with the results for samples A3 and A1. Figs 4.21(a) and (c) show the normalized stress paths and Fig. 4.20 the corresponding stiffness in terms of E_u/C_u . Naturally the stress paths depend on the final consolidation state but it is interesting to note that the stress path for the sample that was swelled, F3, is similar to that of the isotropically consolidated reconstituted sample, see Fig. 4.21(c); on the other hand the stress path of the sample that was consolidated, F2, remains different from the isotropically consolidated reconstituted sample, see Fig. 4.21(a). The changes in stiffness which result from the changes in water content are larger than the changes in shear strength;

consequently the E_u/C_u value increases for F2 and reduces for F3. The intact samples, A1 and A3, had similar stress paths to samples F2 and F3 and E_u/C_u values which were well below and above the average value. Since A1 and A3 also gave significantly different shear strengths to the other intact samples, it suggests that they underwent some moisture changes during or subsequent to sampling and this may account for some of the scatter mentioned earlier.

The effects in this material of sampling, including limited total stress release, handling and trimming, can be assessed from the tests in series H. Samples H2, H4 and H5 were tested without consolidation after sampling while samples H1 and H3 were swelled to the effective stress existing after perfect sampling, p_g' , observed in tests F1 and G1; H1 and H3 increased in water content by 0.14% and 0.31% respectively. In Fig. 4.23 the normalized stress paths of H1 and H2, which are typical, are presented together with those for F1 and F2. The similarity is remarkable even for F2 and H2 in which it is likely that different mechanisms have led to the increased initial effective stress. The C_u -water content points and Young's moduli agree well with those of the other samples. It may be concluded, therefore, that no major effects are associated with sampling in which there is limited mechanical disturbance, although pre-failure behaviour is strongly influenced by the value of the isotropic stress at the start of the undrained shearing. Small water content changes, such as those observed in the swelling of H1 and H3, seem to have little effect.

Concluding Remarks

The pre-failure behaviour observed in unconsolidated undrained compression tests on an intact terrestrial till from Cowden can be reproduced by reconstituted samples given an anisotropic consolidation history and brought to the same water content and isotropic effective stress before shear, p_o' . Despite the matching of intact and reconstituted material pre-failure behaviour, it is unlikely that the stress history of the intact soil has been followed. Indeed this is unlikely to be the case when the depositional process and complex stress history associated with glacial soils is considered. The agreement suggests, instead, that this material is relatively insensitive to the precise

anisotropic stress history. Furthermore, factors such as bonding, aging and placement water content appear to be relatively unimportant in determining pre-failure behaviour of the intact material.

Ultimate undrained strength in compression, $C_{u \text{ ult}}$, is related to water content. The $C_{u \text{ ult}}$ -water content relationship for intact soils is matched only by that for samples remoulded at low water contents; for samples reconstituted at higher water contents ($w_c = 37.5\%$ and 23%) and then consolidated, a higher $C_{u \text{ ult}}$ is measured at a given water content. For Lower Cromer Till it will be shown in Chapter 8 that different $C_{u \text{ ult}}$ -water content relationships also exist for the remoulded and reconstituted soil; however, for Lower Cromer Till the intact strength appears to correlate with that of the reconstituted soil and not that of the remoulded soil.

In this respect, the comparison of microfabrics referred to earlier is of considerable interest. For Cowden Till, the microfabric of the intact soil was found to match that of the soil remoulded at low water content - this corresponds to the agreement found between the $C_{u \text{ ult}}$ -water content relationships for the intact and remoulded soil. For Lower Cromer Till, the microfabric of the intact soil matched that of the reconstituted soil and not the remoulded soil; in the same way agreement was found between the strengths of the intact and reconstituted soil at the same water content. The apparent independence of pre-failure behaviour on method of reconstituting the samples would imply that the total fabric is not dominant in this stage; it will be suggested in Chapter 11 that contact conditions largely determine pre-failure behaviour.

For Cowden Till, the combined evidence from microfabric and $C_{u \text{ ult}}$ water content relationships suggests placement at low water content, probably as a lodgement till. (Note that for placement at higher water contents, differences between methods of placing are eliminated by consolidation.) For Lower Cromer Till, the evidence suggests placement at high water content as a glaciomarine or glaciolacustrine clay. (In this soil differences between methods of placing are not eliminated, except by consolidation to high stresses.) It would appear that

microfabric studies can play a useful role in distinguishing depositional mode, an important parameter in determining soil behaviour.

The angle of friction, ϕ' , is common to all types of sample in the Cowden Till, irrespective of the method of forming, consolidation history and p_o' . (The same conclusion is reached for Lower Cromer Till in Chapter 8).

4.7 EXTENSION TO LOWER CROMER TILL AND TO DIFFERENT CONSOLIDATION HISTORIES

In view of the similarity in grading and plasticity between the Cowden and Lower Cromer Tills, it might be assumed that the conclusions on the matching of intact and reconstituted soil behaviour can be applied to the Lower Cromer Till. A similar detailed comparison between intact and reconstituted samples of Lower Cromer Till could not be made because of the very uncertain stress conditions at the cliff face, referred to previously. However, indirect evidence of matching behaviour in the overconsolidated soil is provided on Fig. 4.24 where the results of an unconsolidated undrained test on the intact material can be compared with the results of a similar test on a sample cut from a block consolidated from slurry. The block had an overconsolidation ratio of 7; there was no attempt to match water contents in the two samples, so the results presented in Fig. 4.24 have been normalized by dividing by undrained shear strength. The agreement in effective stress paths is encouraging; there is agreement, too, in terms of the $C_{u \text{ ult}}$ -water content relationship which is described in Chapter 8 and in terms of ϕ' . The less stiff response of the intact sample shown in Fig. 4.24 and the lack of brittleness evident in its stress path may be related to lateral expansion that could have occurred at the cliff face.

The apparently successful matching of intact and reconstituted behaviour has been observed with the low plasticity clays in an overconsolidated state. It has been inferred that, in this state, the effects of aging and thixotropy must be insignificant in triaxial compression. (Ladd and Edgers (1972) also suggested that this may be the

case for low plasticity clays.) To determine whether this inference can be extended to soil in a normally consolidated state the effects of aging in the normally and over-consolidated reconstituted Lower Cromer Till have been examined.

The results of four tests by Gens (1982, pers. comm.) are presented in Fig. 4.25. Four reconstituted samples were prepared by anisotropic (K_o) consolidation from a slurry, one pair of samples was brought to a normally consolidated state and one pair to an OCR of 7. For each pair, one sample was sheared undrained 24 hours after consolidation and the second sample was sheared undrained 35 days after being maintained with anisotropic stresses under drained conditions.

It is evident that:-

- (i) for the overconsolidated samples the effects of aging are relatively small; there is virtually no change in stiffness; a slight increase in peak strength (reached by dilation) and a smaller increase in ultimate strength; the pore pressure response is slightly altered;
- (ii) for the normally consolidated soil, the effects of aging are more significant; there is a substantial increase in peak strength and much less pore pressure is generated; post-peak behaviour, when the sample is contracting, is, however, similar with only a slight difference in ultimate strength being found between the 'aged' and 'young' samples.

It would appear, then, that for examining the behaviour of the low plasticity clays, reconstituted samples will provide a good indication of the behaviour of the intact soil when in an overconsolidated state. For soil in a normally or lightly overconsolidated state, the effects of aging will be absent in the reconstituted soil; in the Lower Cromer Till, at least, these effects may be significant. However, it is reassuring to find that data presented by Koutsoftas (1981) for a low plasticity marine clay shows excellent agreement with the pattern of behaviour for reconstituted Lower Cromer Till presented in Chapter 8, even when both are normally or lightly overconsolidated.

Differences in behaviour between intact and reconstituted low plasticity soil may well occur when the soil is bonded. Evidence for bonding of North Sea specimens was described in Chapter 3 but bonding does not appear to be widespread.

To relate the observations on small reconstituted samples to field behaviour, scale effects in this type of material must be taken into account. The effect of discontinuities, without more plastic clay infill, on mass behaviour is discussed in Chapter 8; discontinuities would appear to have a small effect on shear strength parameters. In Chapter 12 data will be presented which shows that the in situ behaviour of the Cowden Till can be correctly predicted using results of tests on reconstituted samples; this indicates that macrofabric and sampling effects in this particular till are not significant.

4.8 SUMMARY

Samples taken directly from the sea bed of the Northern North Sea are unlikely to be suitable for a fundamental investigation of soil behaviour. They are likely to show a wide range in composition, depositional mode and stress history. To the effects of mechanical disturbance on the samples are added those of a large total stress release and temperature change. The magnitude of these two latter effects has been evaluated in terms of the pore pressure parameter, B , and a temperature parameter, T . Pore pressure changes are related to total stress changes by B and to temperature changes by T . Both parameters are highly dependent on the soil skeleton compressibility, C , and to a lesser extent on porosity, n . As compressibility reduces, B becomes less than unity and this gives rise to an additional drop in effective stress on sampling from the sea bed. The greatest reduction in effective stress resulting from total stress and temperature change occurs in low compressibility soils at shallow depths and it is these soils at such depths which are of greatest interest to the designers of gravity structures.

The combination of large total stress release and sedimentary history lead to a high probability of gas exsolution in samples taken

from the Northern North Sea. It has been suggested that the process is time dependent and this, together with the effect of temperature change, make the process non-reversible. Significant changes in fabric may arise when total stresses are reimposed, and effective stresses are different from those which would exist without gas exsolution.

The pre-failure behaviour of low plasticity clay has been shown to be strongly influenced by the initial effective stresses prior to shear. Unless the effective stresses are restored to those existing after perfect sampling, the measured response will not be representative. Restoration of effective stresses has been shown to work only when changes in effective stress are small and there is limited mechanical disturbance including that caused by gas exsolution. Neither restriction is likely to be met in offshore samples.

A terrestrial glacial sediment, the Lower Cromer Till, has been located which is similar in composition, mineralogy and microfabric to some of the low plasticity clays present in the Northern North Sea. Effects of pore water chemistry on its behaviour appear to be small.

The Lower Cromer Till has a relatively homogeneous macrofabric but distinctly inhomogeneous microfabric. Two clay phases are distinguished in the microfabric: dense oriented clay between the sand grains-clay bridges - and open clay occupying the void spaces-clay matrix. In the intact Lower Cromer Till contact bonds have developed in the clay bridges. The microfabric of the intact Lower Cromer Till is matched by reconstituted soil which has been anisotropically consolidated from slurry. There is agreement, too, between the $C_{u\text{ ult}}$ -water content relationships for intact and reconstituted samples. For remoulded Lower Cromer Till there is a different microfabric and $C_{u\text{ ult}}$ -water content relationship. Similitude between intact and reconstituted soil is taken as evidence of a glaciomarine or glaciolacustrine origin for the Lower Cromer Till.

A second low plasticity clay, the Cowden Till, has been used in the investigation. For this till the microfabric of the intact soil

is matched only by that of soil remoulded at low water content and both exhibit a characteristic drag structure. There is agreement, too, between the $C_{u \text{ ult}}$ -water content relationships for intact and remoulded soil. This will be taken as evidence that the Cowden Till was deposited by lodgement. Soil reconstituted from slurry has a different microfabric and $C_{u \text{ ult}}$ -water content relationship than the intact soil.

Pre-failure behaviour of intact samples of the Cowden Till can be matched by reconstituted samples of the soil which are given an anisotropic consolidation history and brought to the same water content and initial effective stress. This agreement implies that in these low plasticity clays:-

- (i) effects of aging are small, except in the normally consolidated soil,
- (ii) the precise form of the anisotropic consolidation history may not be important when the soil is overconsolidated,
- (iii) the microfabric is relatively easily reproduced, presumably because of the limited scope for ordering in the clay fabric.

In making comparisons between intact and reconstituted soil, allowance must be made for differences in gravel content - the use of matrix water content is suggested as a basis for comparison.

In view of the successful matching of behaviour in intact and reconstituted samples of these low plasticity clays, and of the apparently infrequent occurrence of bonding in the North Sea sediments, it is proposed to examine in detail the behaviour of the anisotropically consolidated reconstituted Lower Cromer Till. In part C of the thesis its response to monotonic, transient and cyclic loading is described.

C_w ,	Compressibility of water	$48 \times 10^{-8}/\text{kN/m}^2$
C_s ,	Compressibility of solid grains	$2.7 \times 10^{-8}/\text{kN/m}^2$
α_s ,	Thermal coefficient of cubical expansion of solid grains	$3 \times 10^{-5}/^\circ\text{C}$
α_w ,	Thermal coefficient of expansion of water	$21 \times 10^{-5}/^\circ\text{C}$

(Based on Kaye and Laby (1973))

Table 4.1 Physical constants required for evaluation of $\Delta u/\Delta \sigma$ and $\Delta u/\Delta t$.

	<u>5°C</u>	<u>20°C</u>	<u>0°C</u>
CO ₂	1.4	0.85	1.713
Hydrogen sulphide	3.8	2.5	4.67
Air	0.0257	0.0188	0.029
Nitrogen	0.0212	0.0156	0.0239
Oxygen	0.0425	0.0302	0.049
Methane	-	0.002	0.030

Table 4.2 Henry's coefficient of solubility for various gases in water at different temperatures.

Mineralogy

<u>Whole rock</u>	Major (>30%)	:	quartz
	Minor (10-30%)	:	calcite, felspar, clay minerals
	Trace (<10%)	:	mica (muscovite type)
<u>Clay fraction</u>	Major (>30%)	:	calcite, illite
	Minor (10-30%)	:	smectite, kaolin, chlorite
	Trace (<10%)	:	quartz, felspar

Pore water chemistry

Cl ⁻	0.66	g/l
Na	0.4	g/l
K	20.28	mg/l
Ca	40.56	mg/l
Mg	233.22	mg/l
SO ₄ ⁻²	0	

Carbonate content 4 - 8%

Table 4.3 Details of Lower Cromer Till from Happisburgh, Norfolk.

<u>Clay fraction</u>	Major	:	illite, kaolinite
	Minor	:	chlorite, vermiculite

Table 4.4 Mineralogy of Cowden Till.

Series	Tests	Type of sample	Type of consolidation prior to undrained shear	Symbol
A	A1,A2 A3,A4	98mm intact	Unconsolidated	●
B	B1,B2	38mm intact	Unconsolidated	○
C		Remoulded directly from intact material	Unconsolidated	
	C1,C2	Manually kneaded		⬮
	C3,C4	Statically compacted after remoulding		⬮
D		Soil mixed at W/C = 23%	Isotropically consolidated	△
E		Soil mixed at W/C = 37.5%	Isotropically consolidated	▽
F	F1,F2 F3	Soil mixed at W/C = 23%	Anisotropically consolidated	▲
G	G1	Soil mixed at W/C = 37.5%	Anisotropically consolidated	▼
H	H1,H3	Soil mixed at W/C = 37.5% and consolidated	Consolidated to 'perfect' sampling stresses	✱
	H2,H4 H5		Unconsolidated	+

Table 4.5 Summary of testing programme.

Sample	Gravel content (%)	Shear strength (kPa)	Water Content	
			Whole sample (%)	Matrix (%)
1	0	157.2	16.81	16.81
2	3.66	160.3	16.49	16.90
3	7.07	161.1	16.07	16.86
4	11.70	158.9	15.61	16.93

Table 4.6 Shear strengths and water contents in samples with varying gravel content.

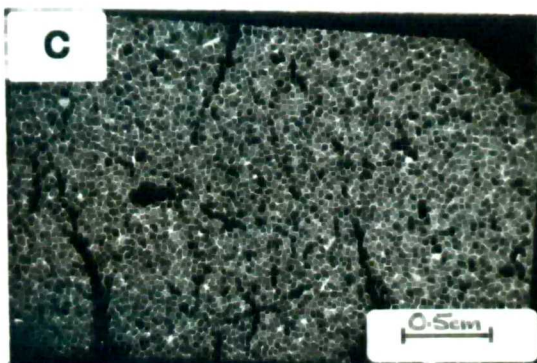
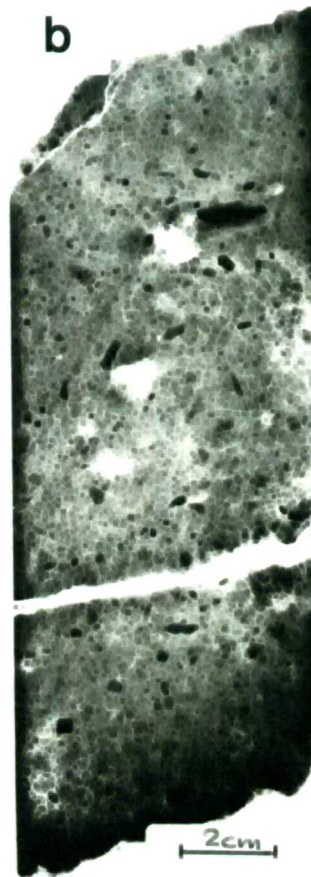


Plate 4.1 (a)-(d) Lower Cromer Till. Macrofabric of intact soil.

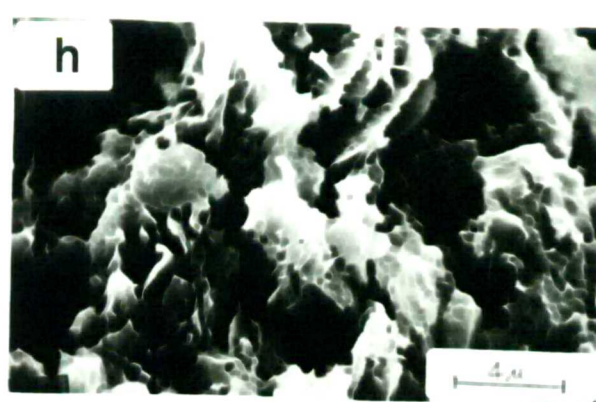
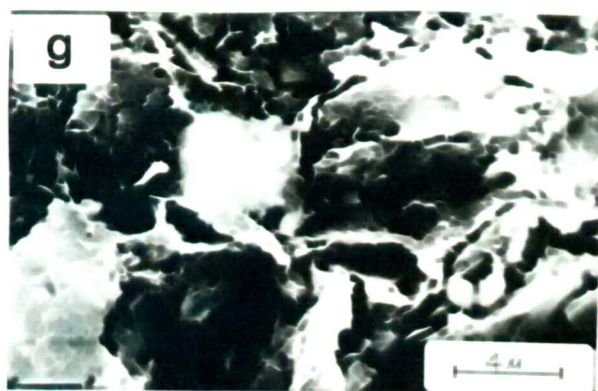
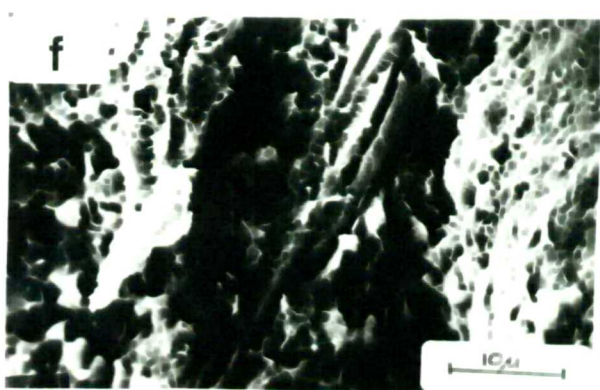
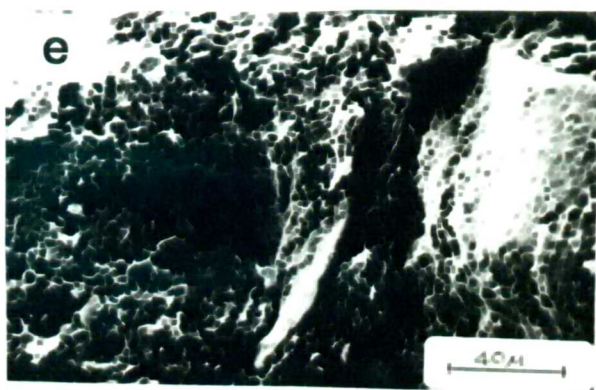
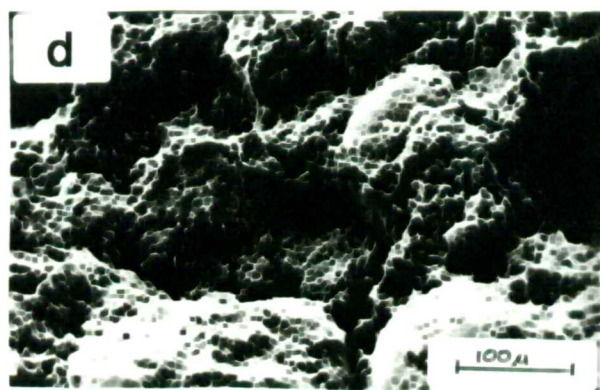
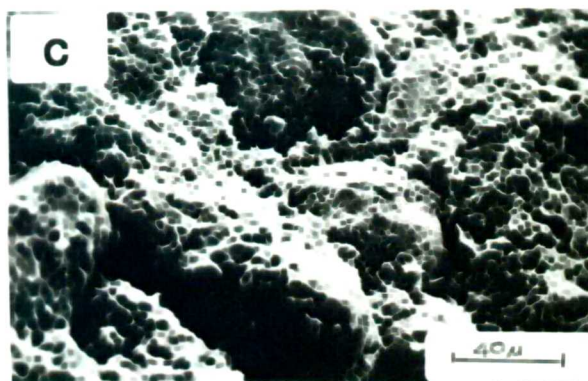
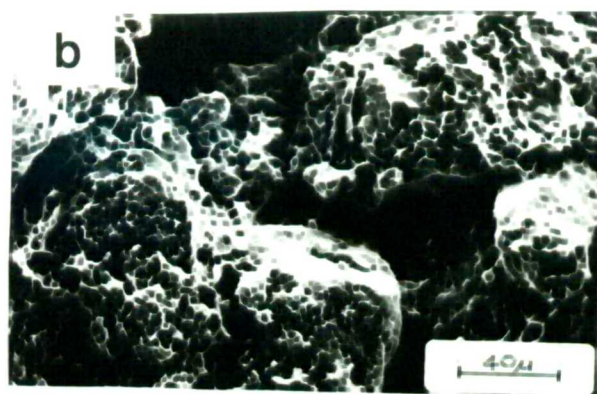
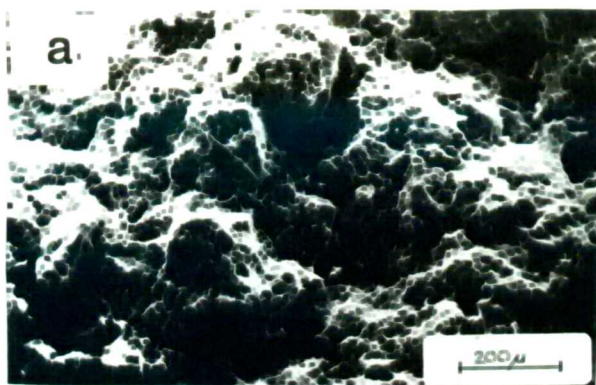


Plate 4.2 (a)-(h) Lower Cromer Till. Microfabric of intact soil.

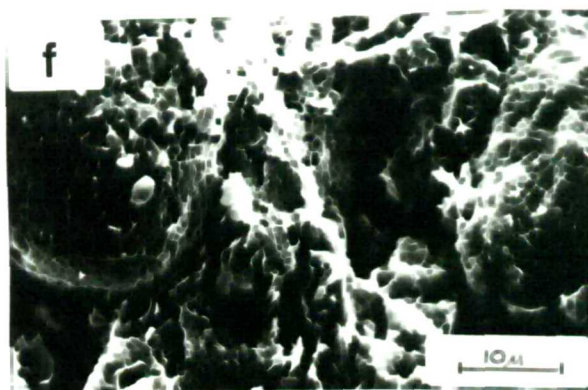
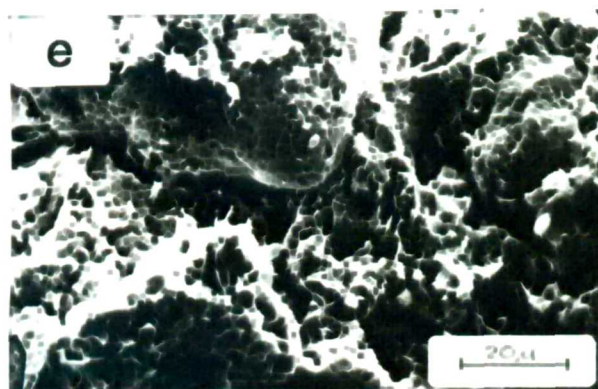
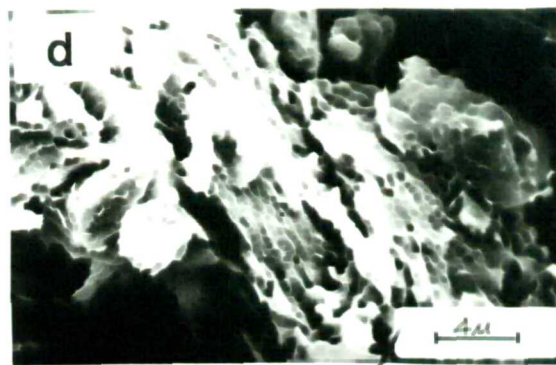
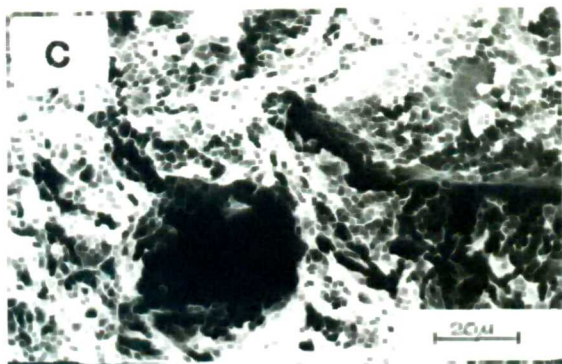
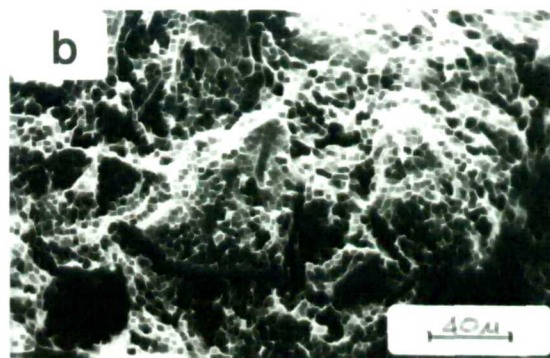


Plate 4.3 (a)-(f) Lower Cromer Till. Microfabric of intact soil.

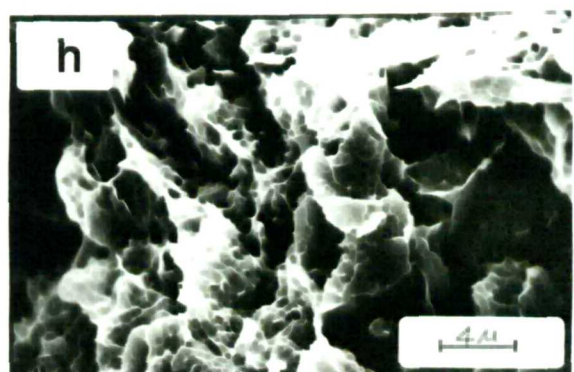
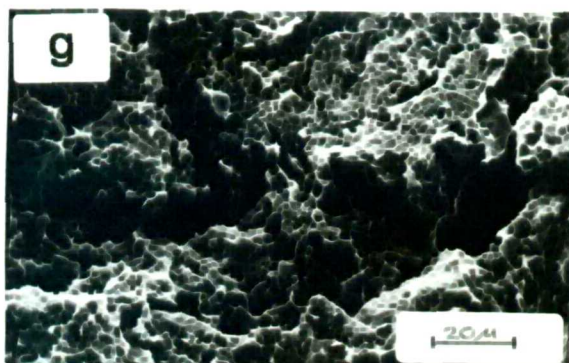
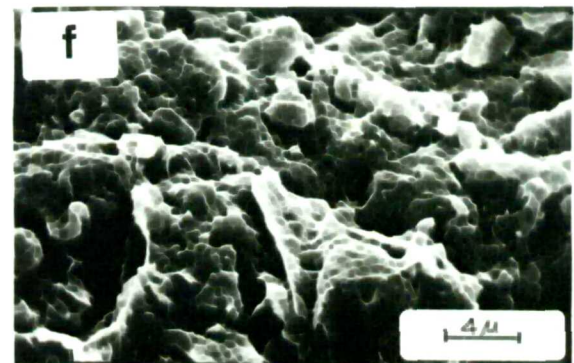
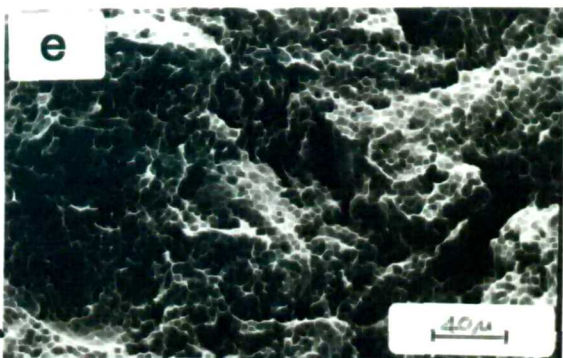
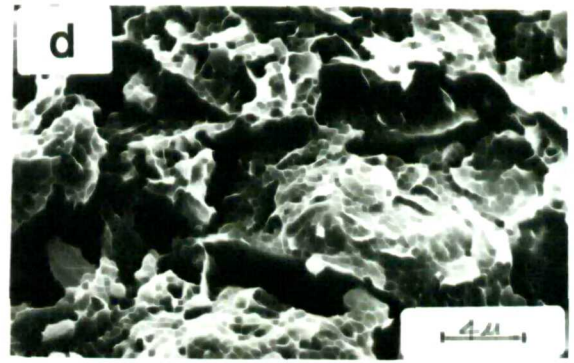
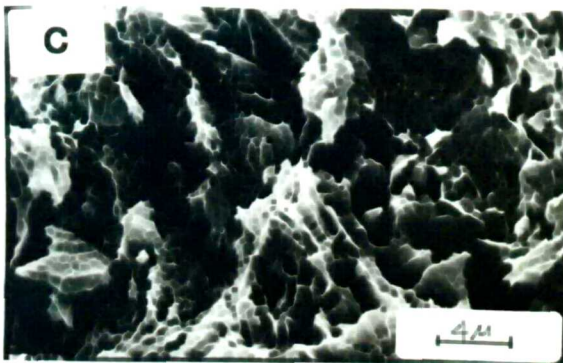
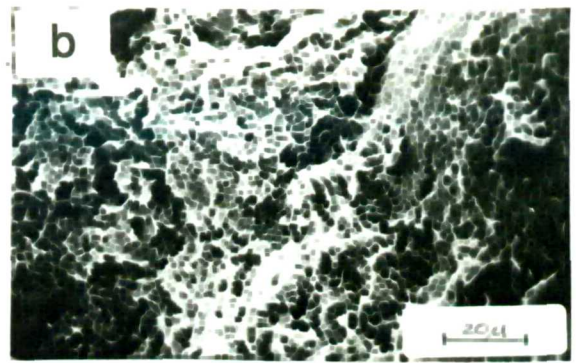
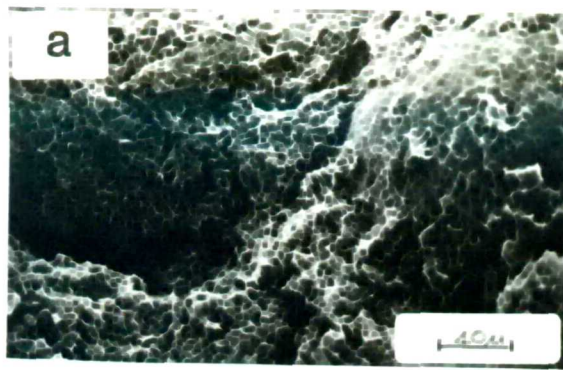


Plate 4.4 (a)-(h) Lower Cromer Till. Microfabric of anisotropically consolidated reconstituted soil.

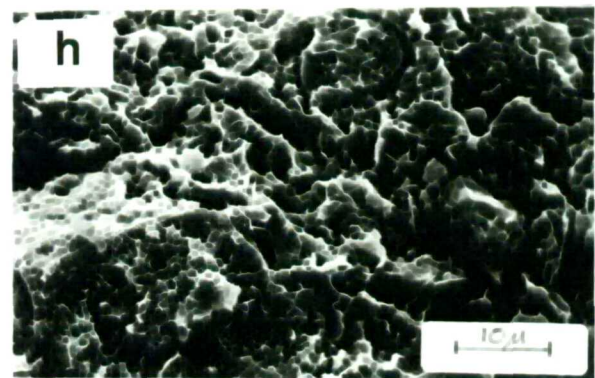
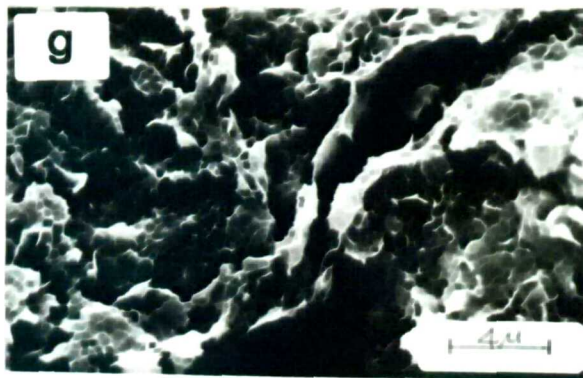
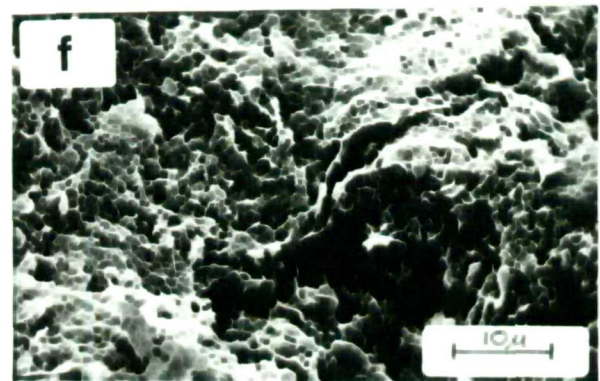
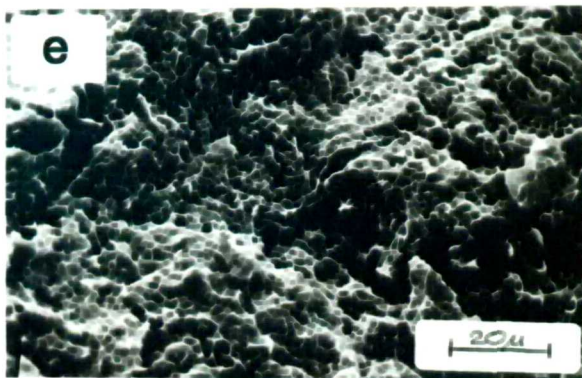
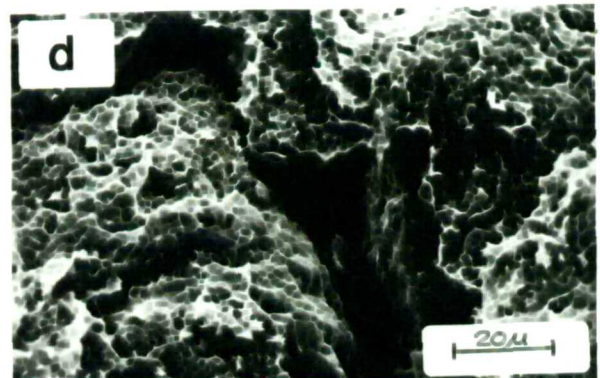
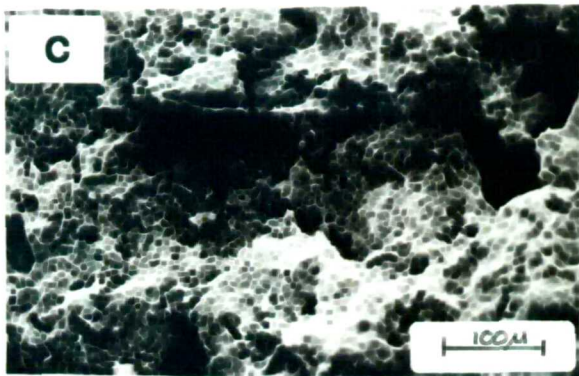
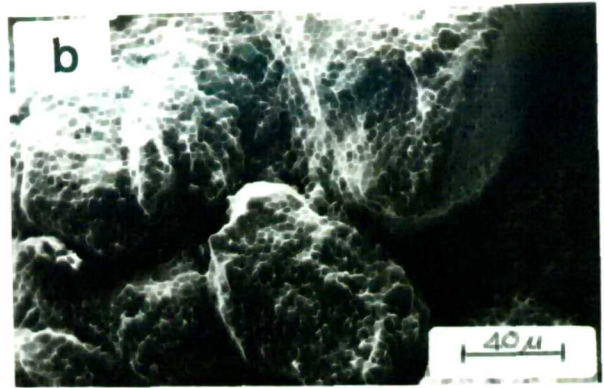
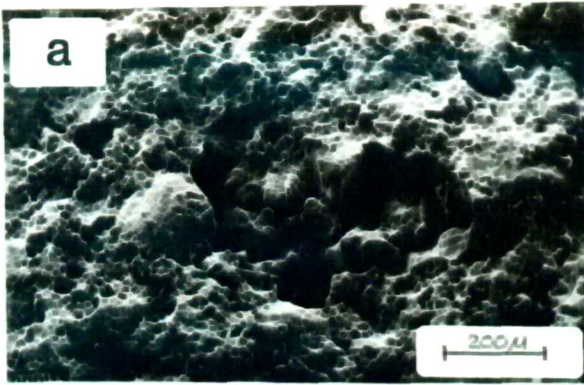


Plate 4.5 (a)-(h) Lower Cromer Till. Microfabric of remoulded soil.

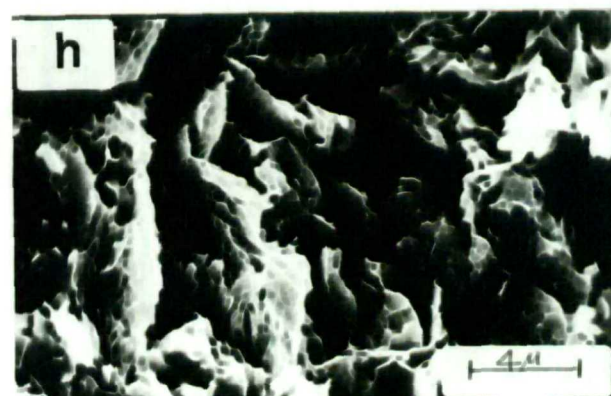
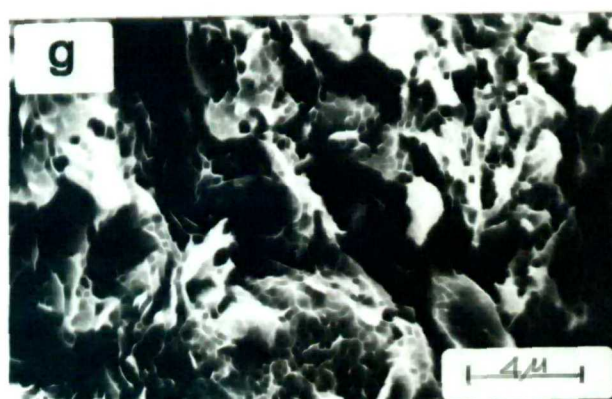
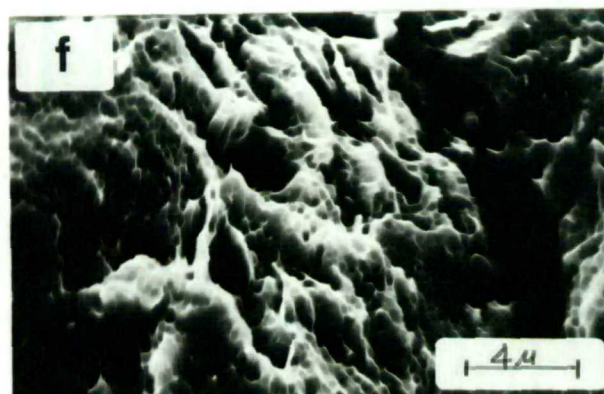
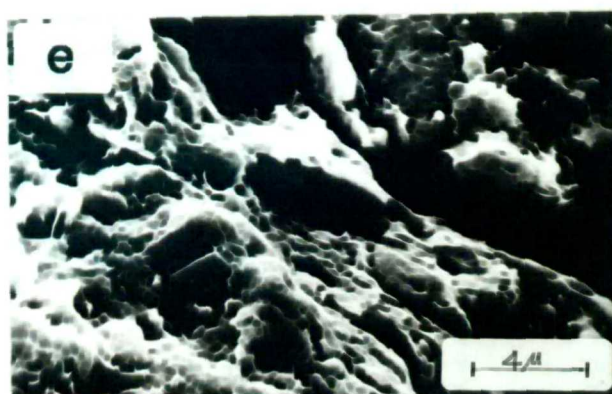
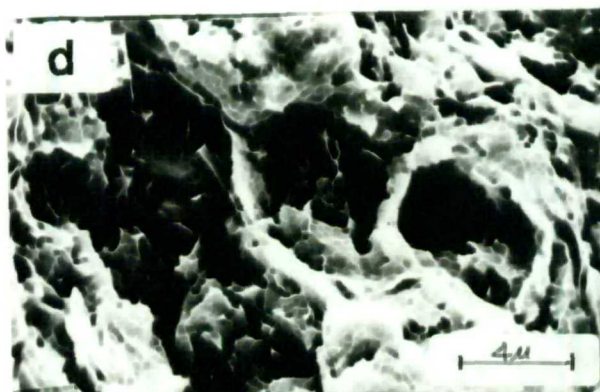
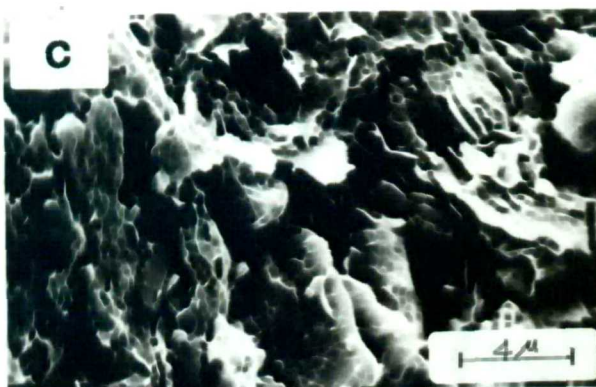
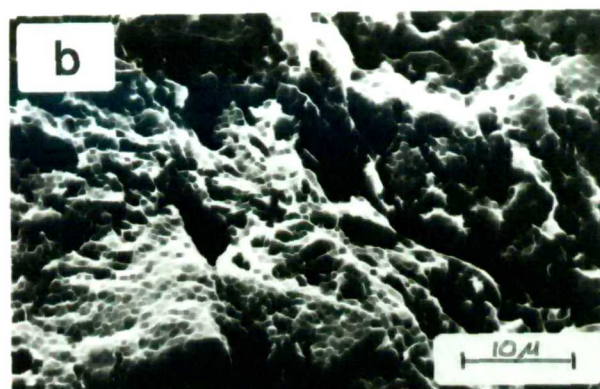
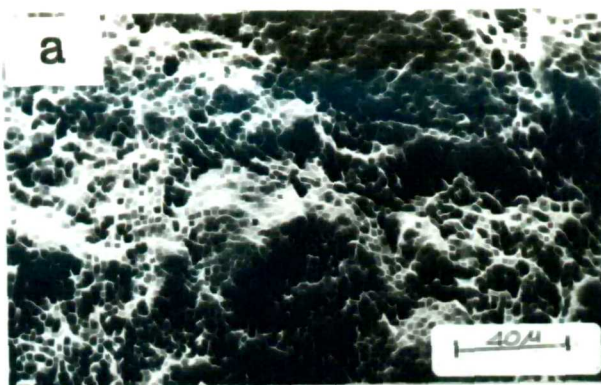


Plate 4.6 (a)-(h) Cowden Till. Microfabric of intact soil.

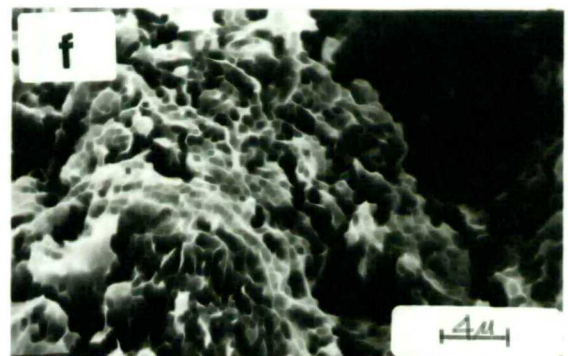
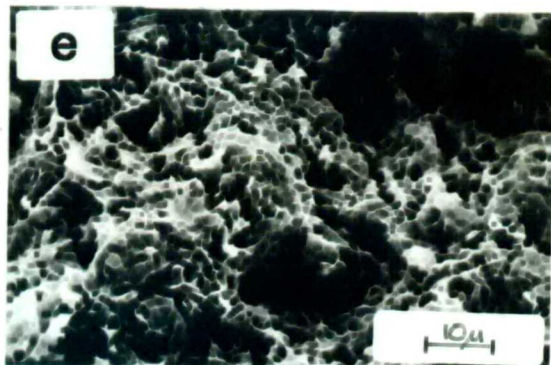
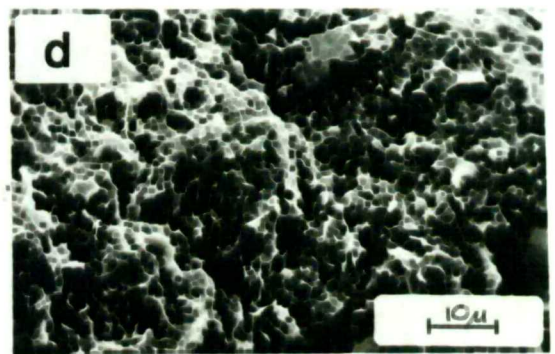
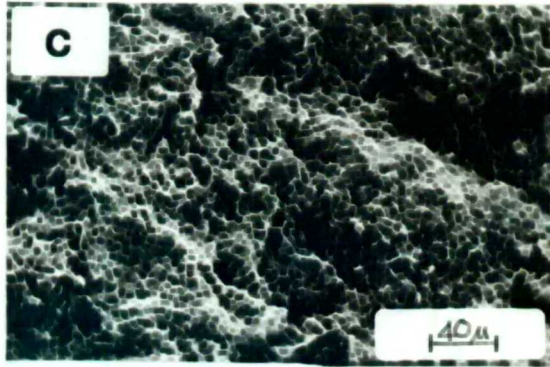
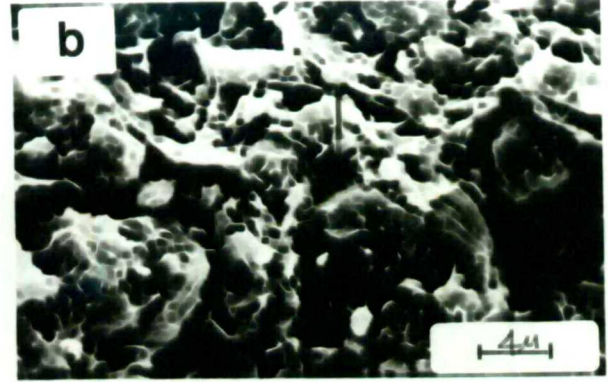
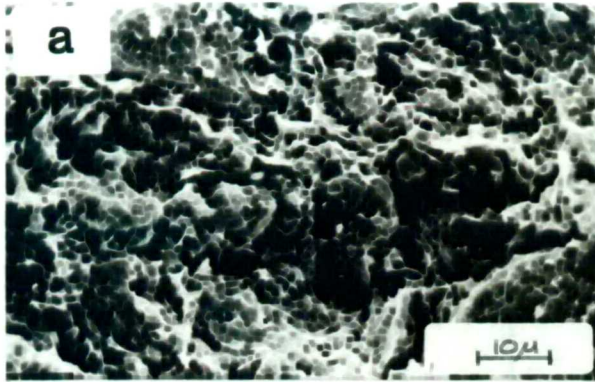


Plate 4.7 (a)-(f) Cowden Till. Microfabric of anisotropically consolidated reconstituted soil.

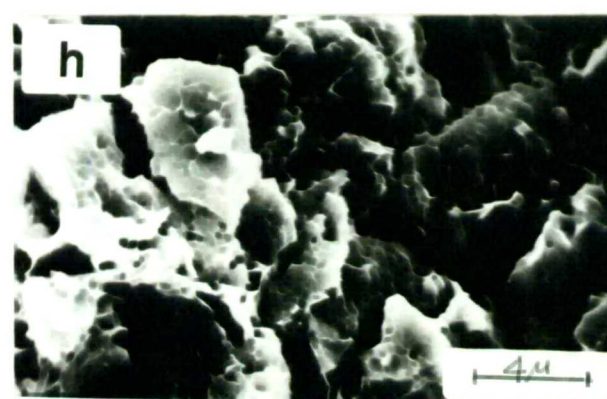
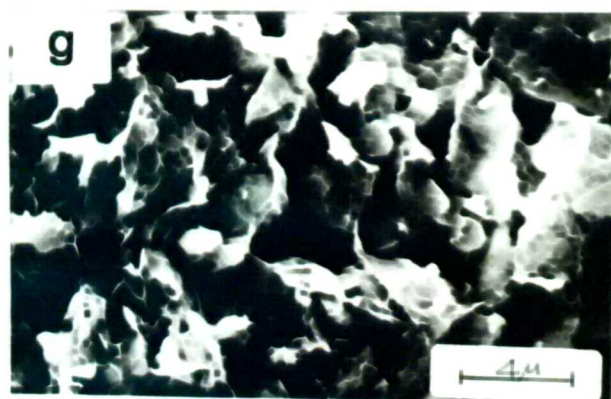
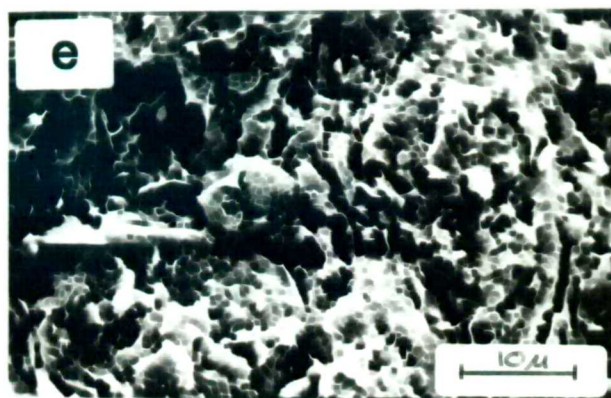
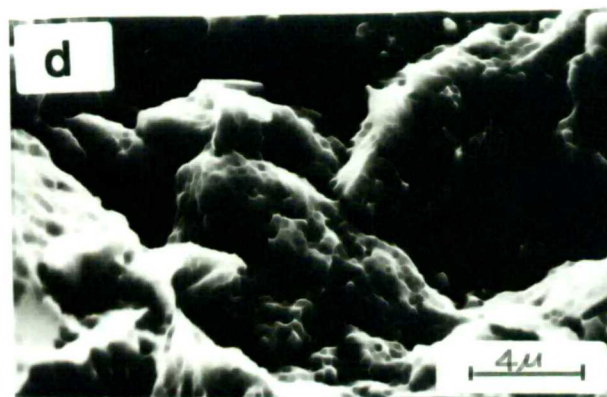
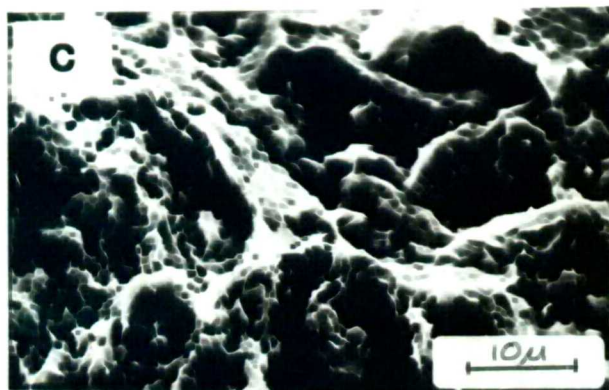
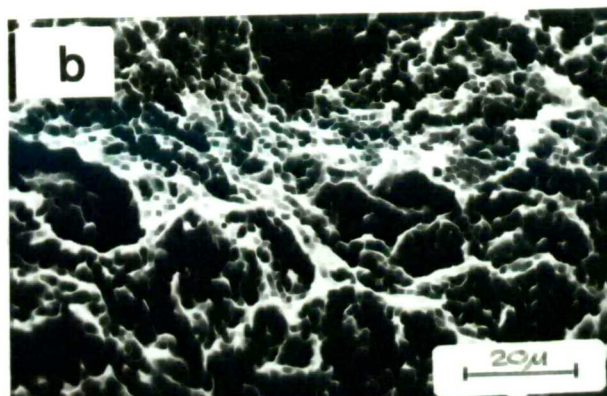
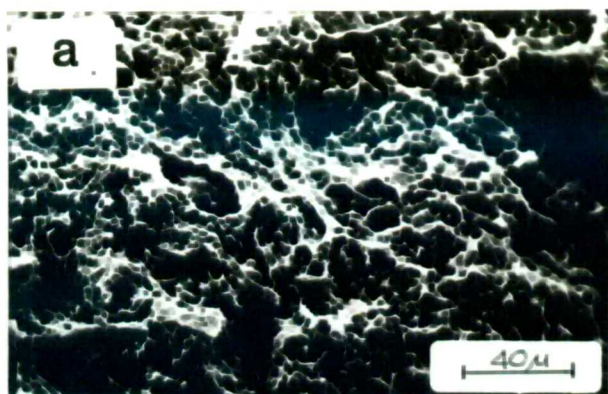


Plate 4.8 (a)-(h) Cowden Till. Microfabric of remoulded soil.

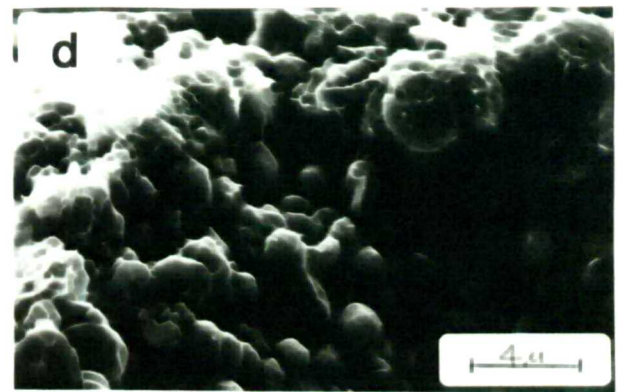
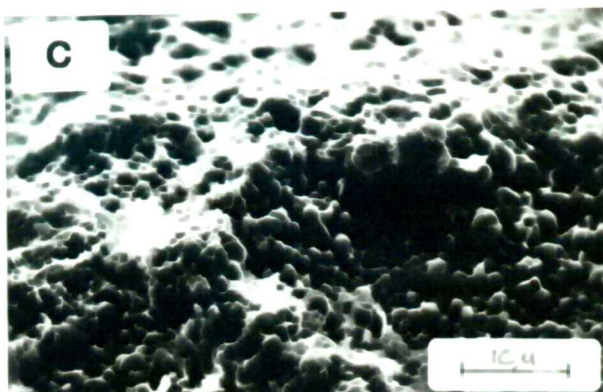
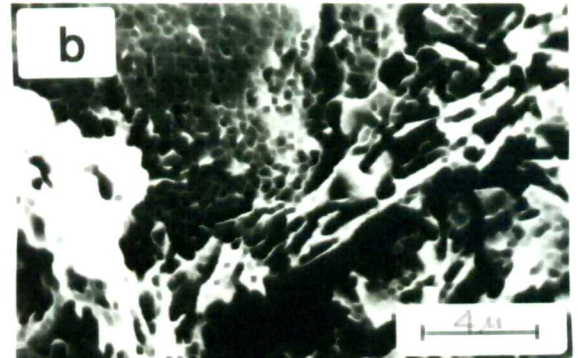
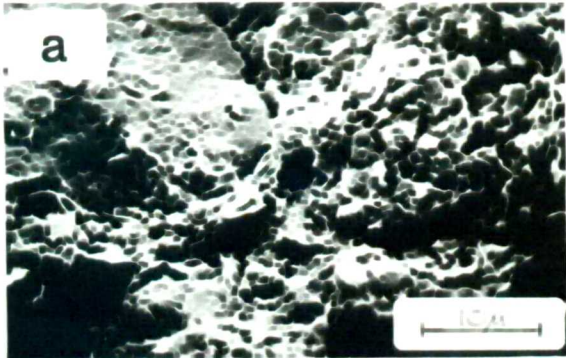
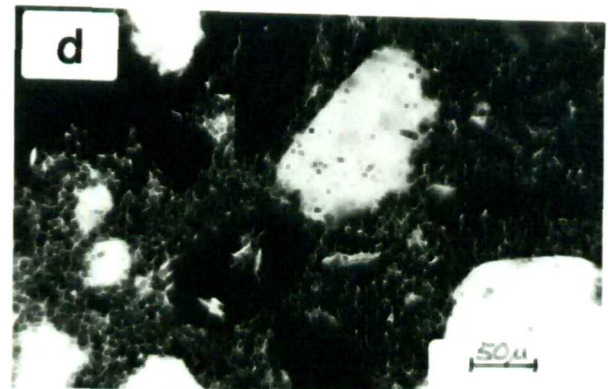
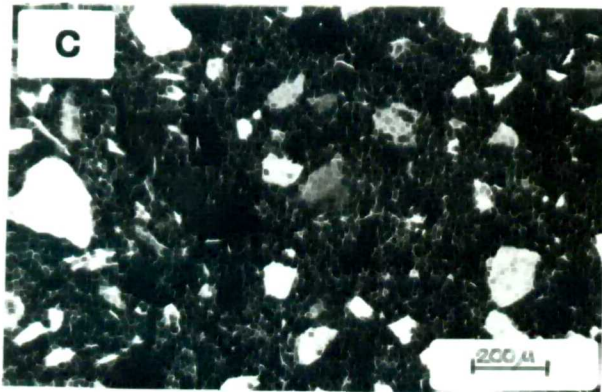
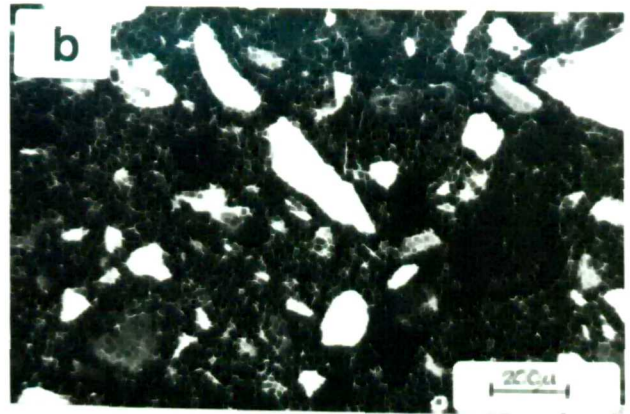
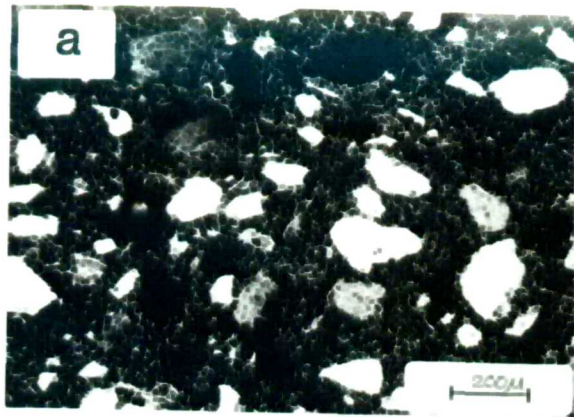
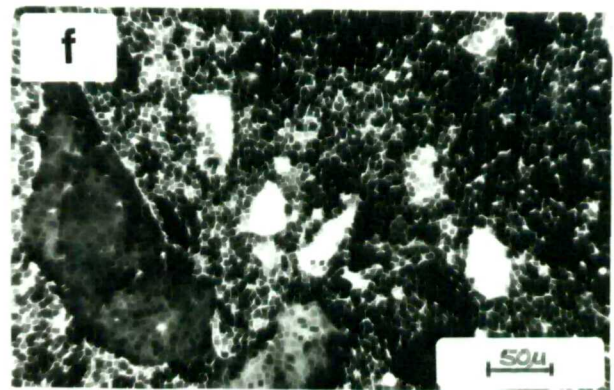
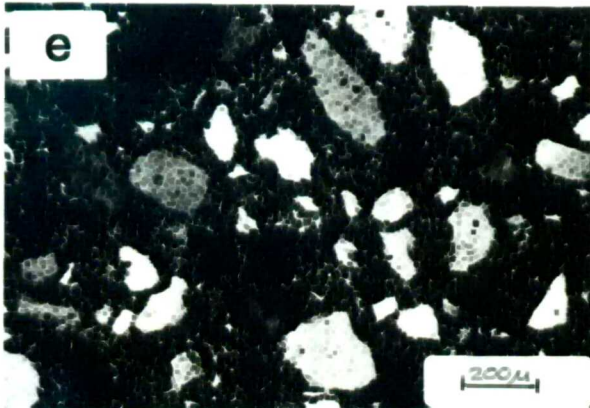


Plate 4.9 (a)-(d) Cowden Till. Microfabric of remoulded soil.



Reconstituted: isotropic



Reconstituted: anisotropic

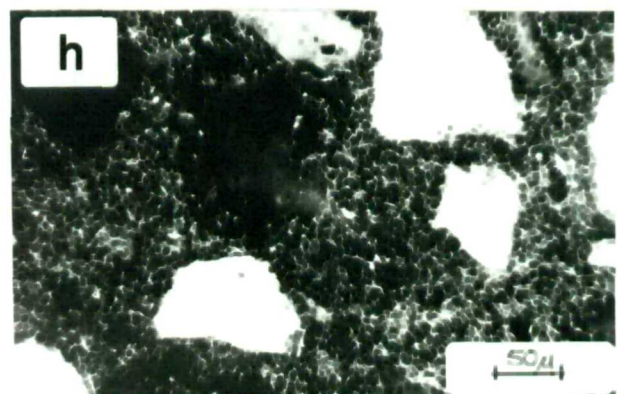
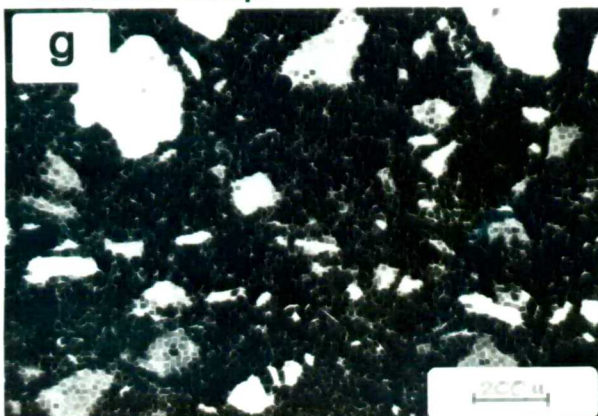
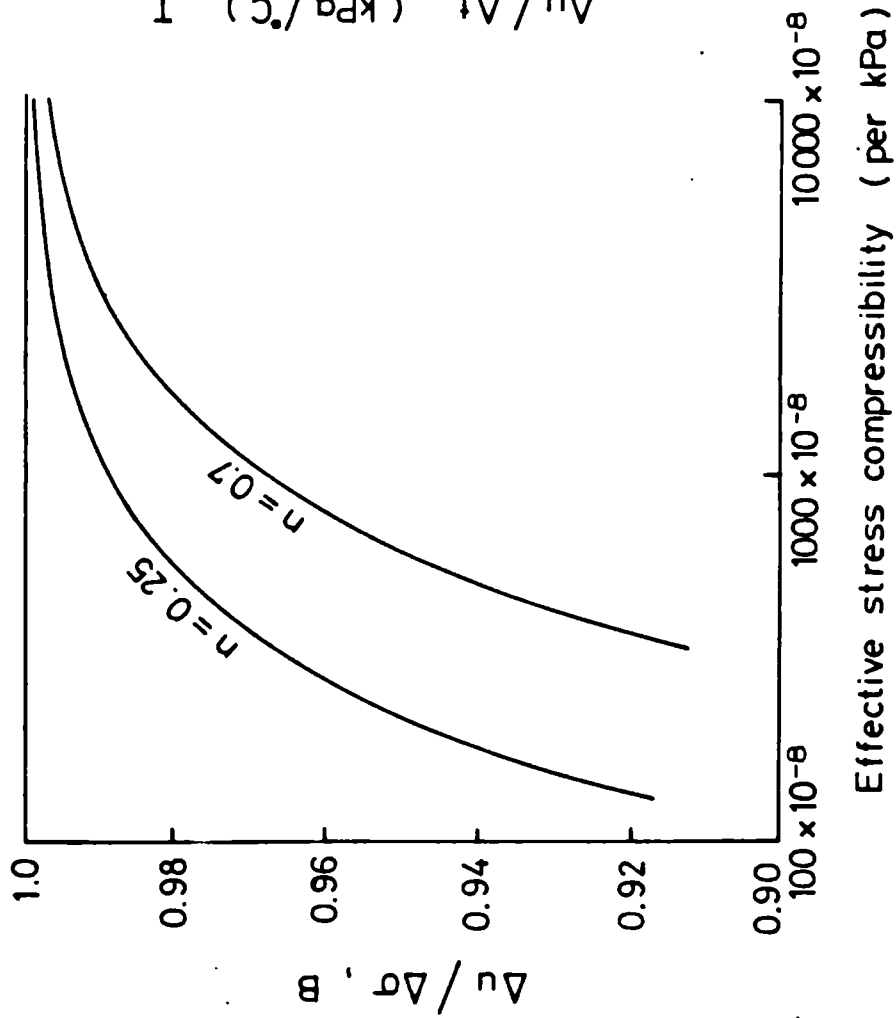
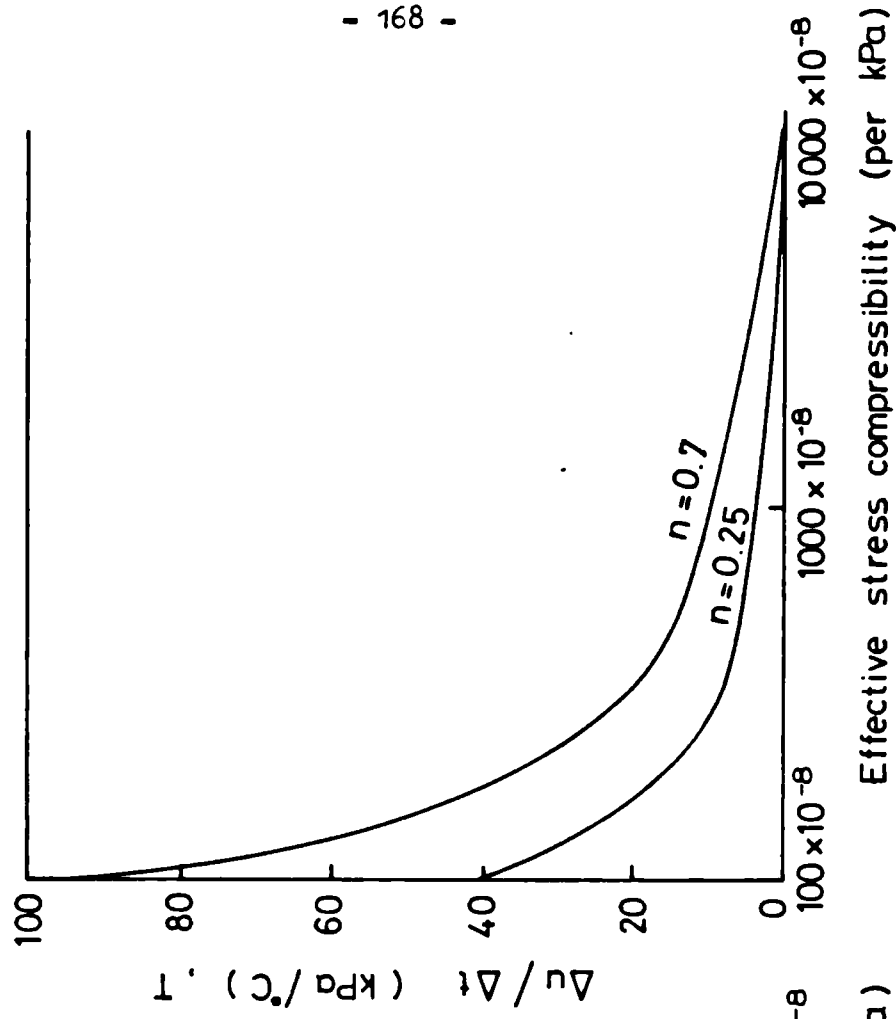


Plate 4 10 (a)-(h) Lower Comer Till. Microfabric of intact and reconstituted soil. (Thin sections).

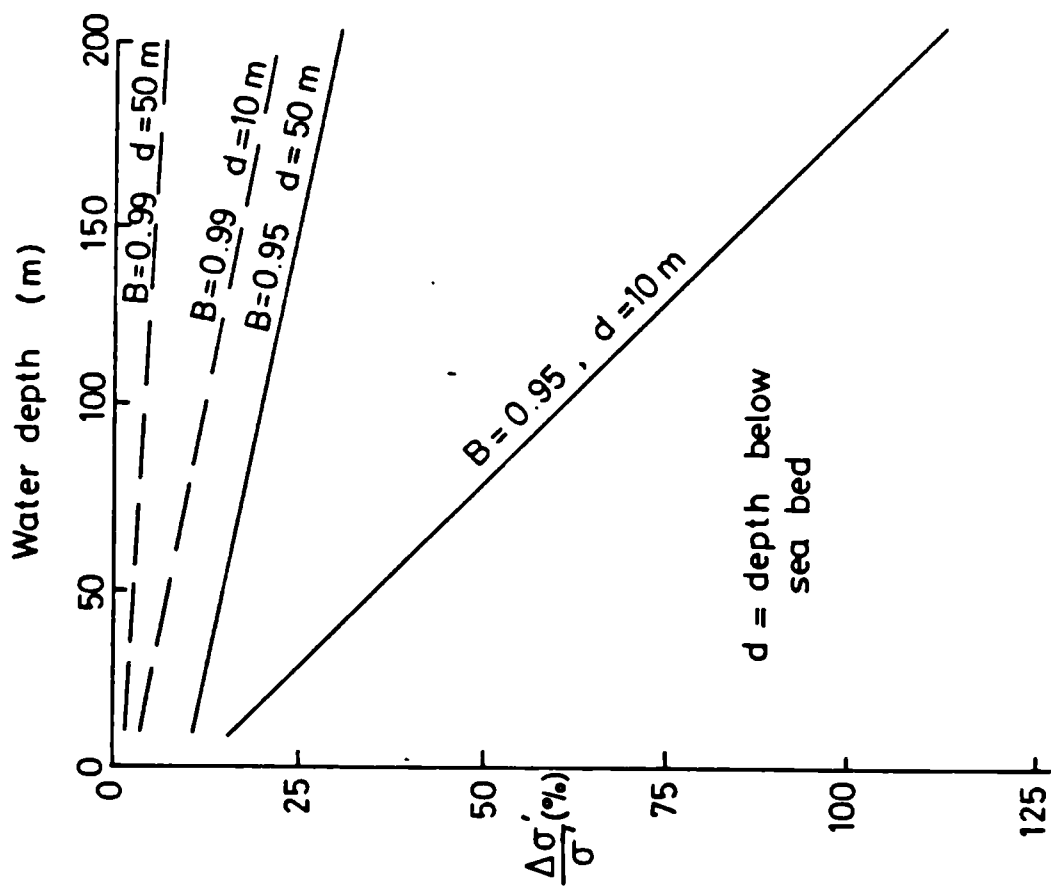


(a) Pore pressure parameter B

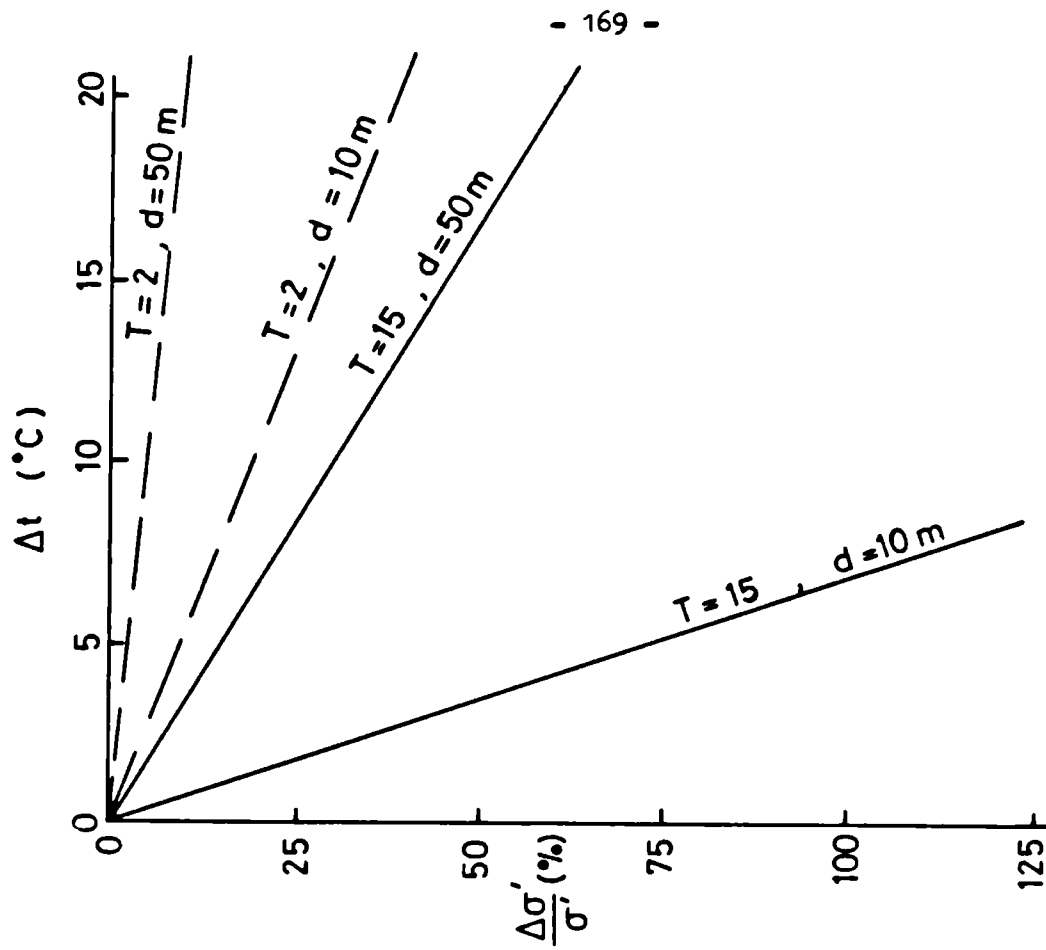


(b) Pore pressure parameter T

Fig. 4.1 Variation of pore pressure parameters B and T with soil compressibility.



(a) Effects of total stress release



(b) Effects of temperature change

Fig. 4.2 Change in effective stress with total stress release and temperature change.

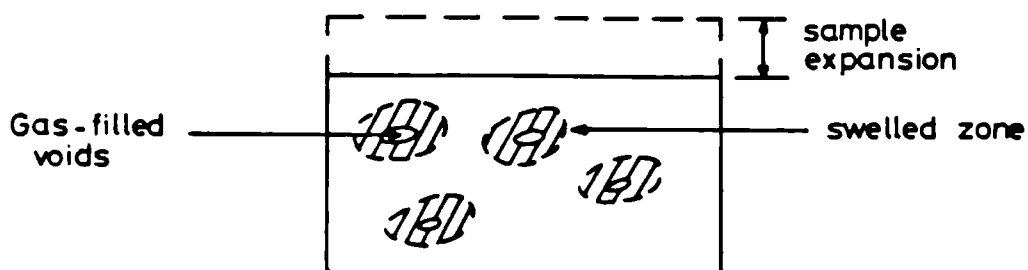


Fig. 4.3 Localised swelling where gas exsolution occurs.

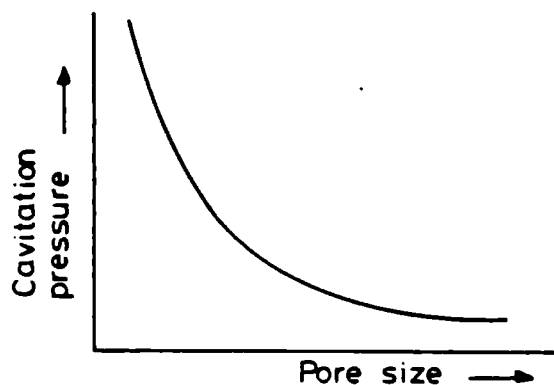


Fig. 4.4 Form of variation in cavitation pressure with pore size.

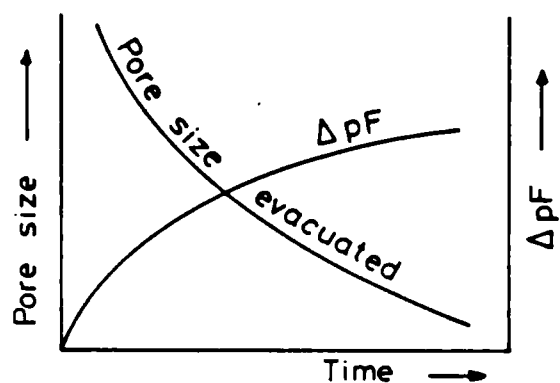


Fig. 4.5 Change in suction with time.

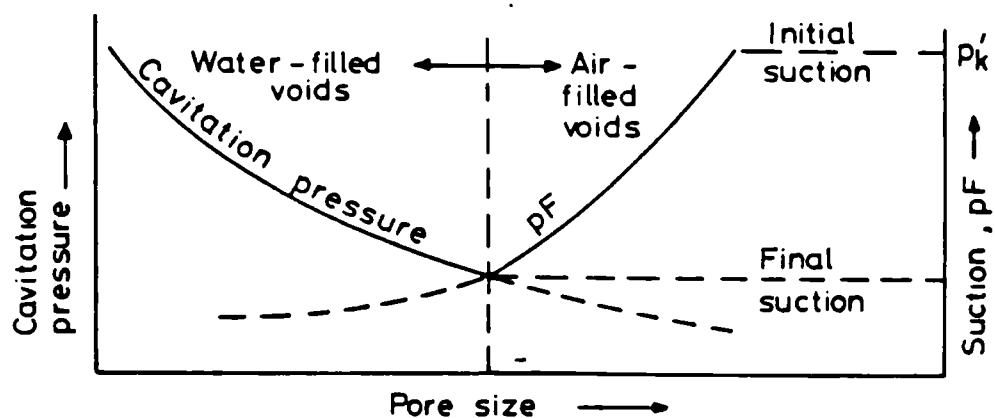


Fig. 4.6 Equilibrium between suction and cavitation pressure.

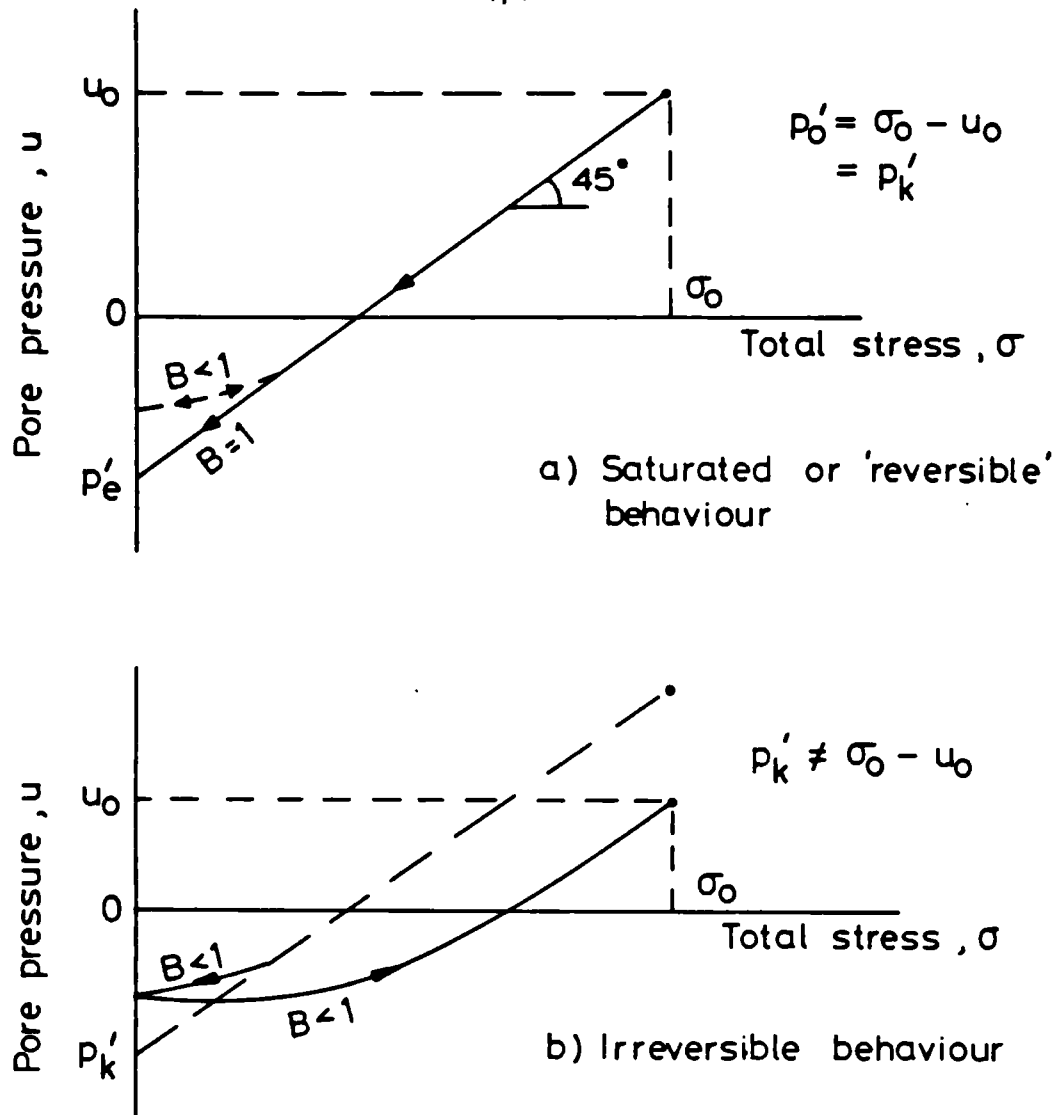


Fig. 4.7 Changes in pore pressure on isotropic unloading and reloading.

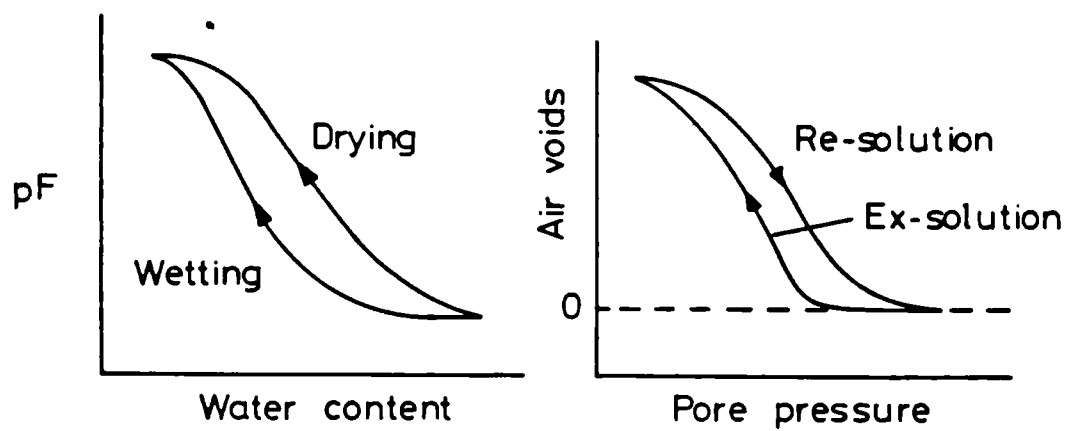
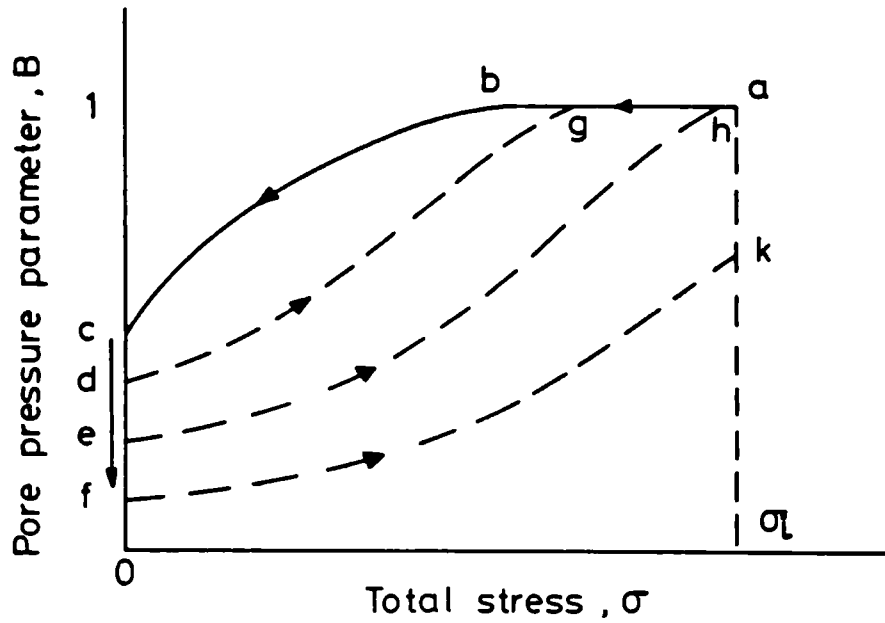


Fig. 4.8 Analogy of hysteresis in pF -water content and in gas exsolution-resolution.



- a b c - total stress unloading
- b - gas exsolution
- c d e f - gas diffusion / temperature change
- d g a - total stress reloading after short storage
- g - resaturation
- e h a - total stress reloading after intermediate storage
- f k - total stress reloading after long storage
- k - $B < 1$

Fig. 4.9 Irreversibility of changes in pore pressure parameter B.

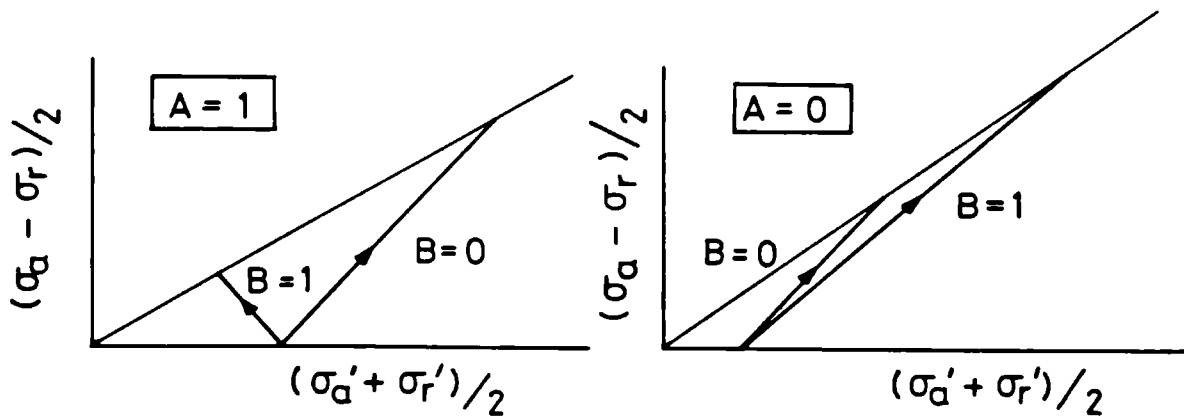


Fig. 4.10 Effect of lack of saturation on measurement of undrained strength.

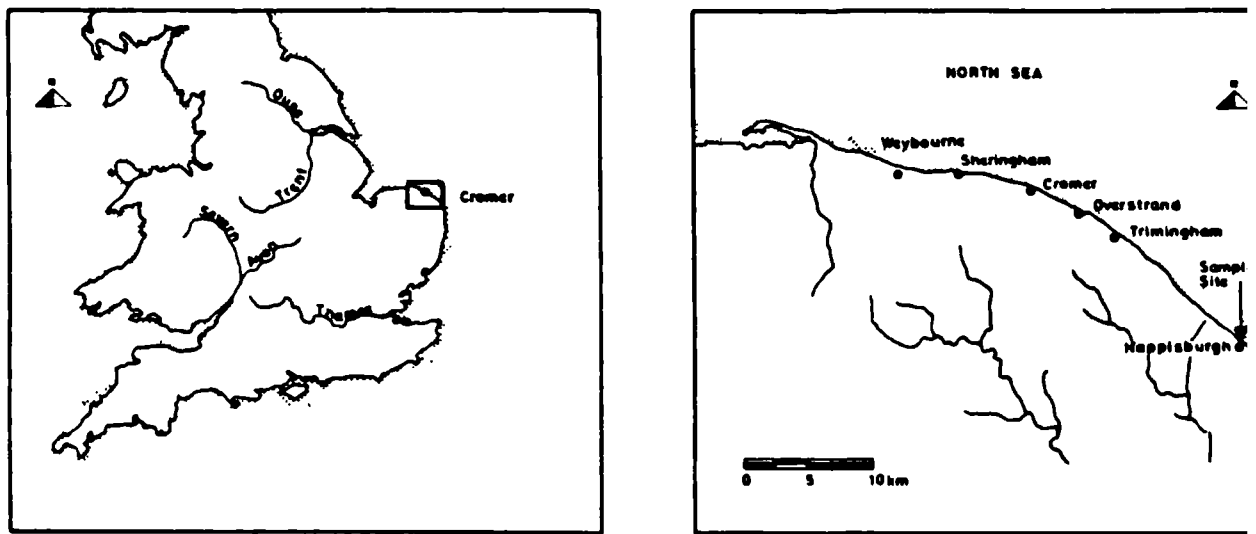


Fig. 4.11 Location of site for sampling Lower Cromer Till.

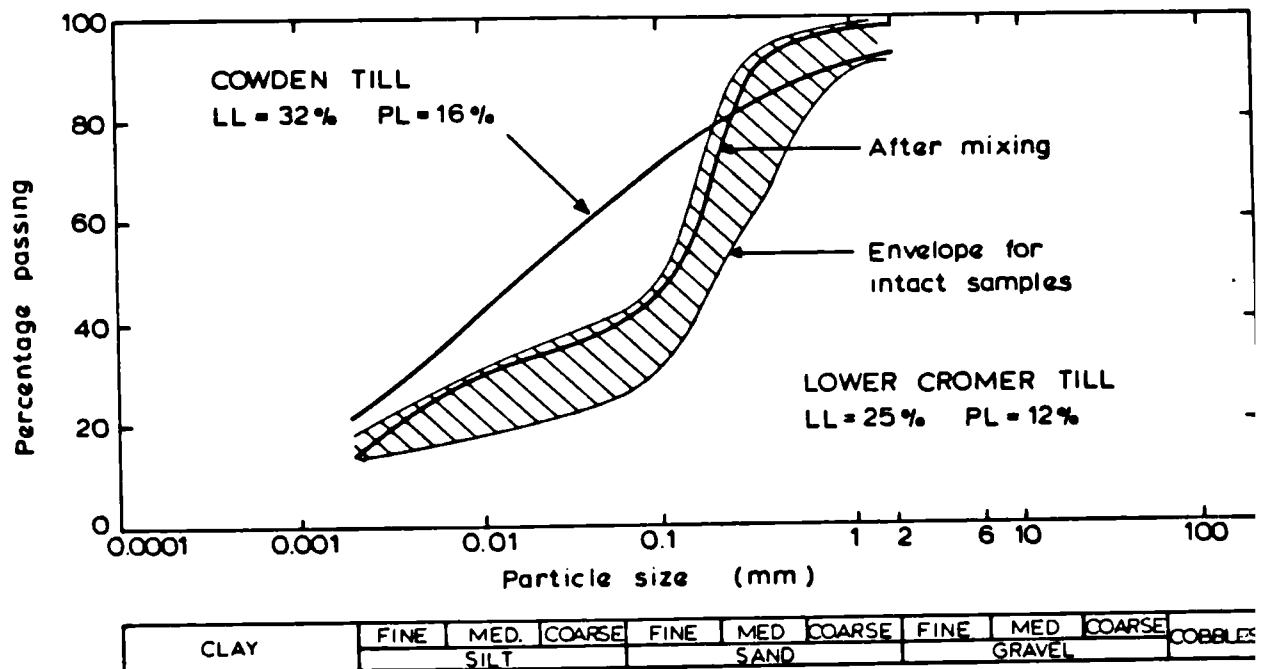


Fig. 4.12 Grading envelope for Lower Cromer Till and Cowden Till used in the laboratory investigation.

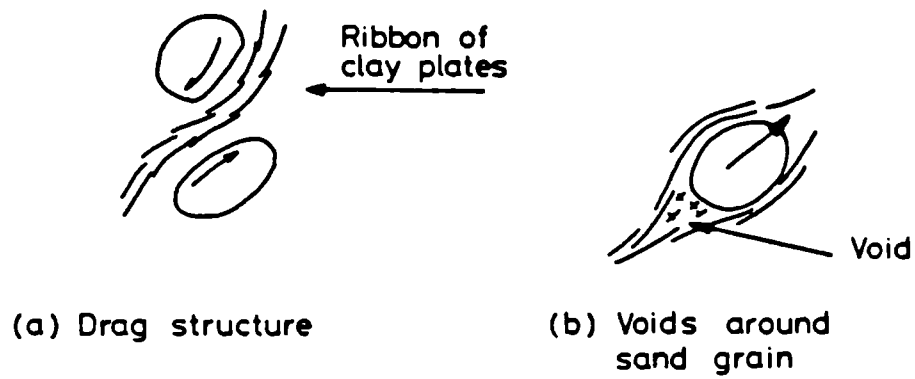


Fig. 4.13 Details of microfabric in remoulded Lower Cromer Till.

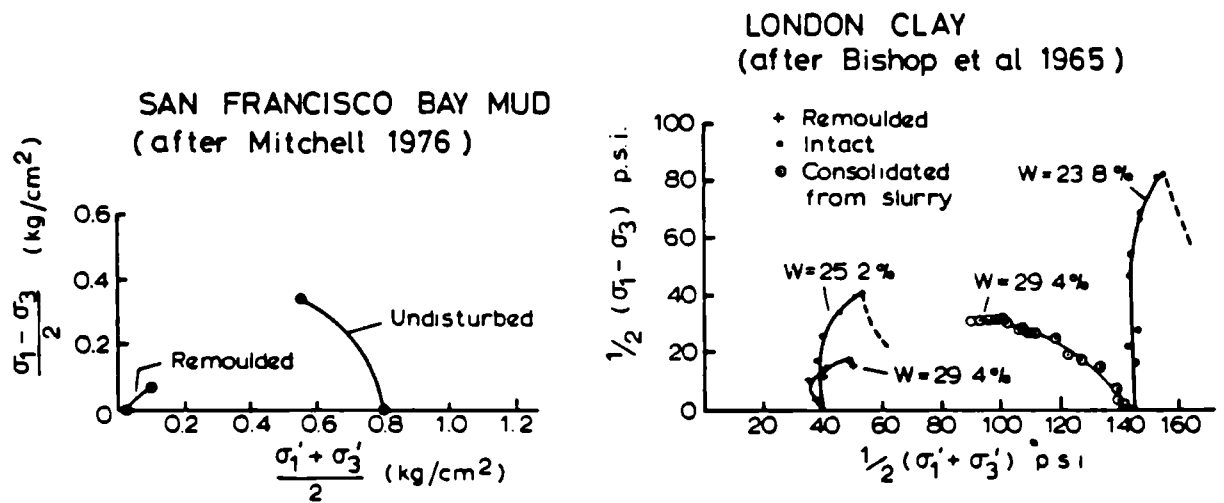
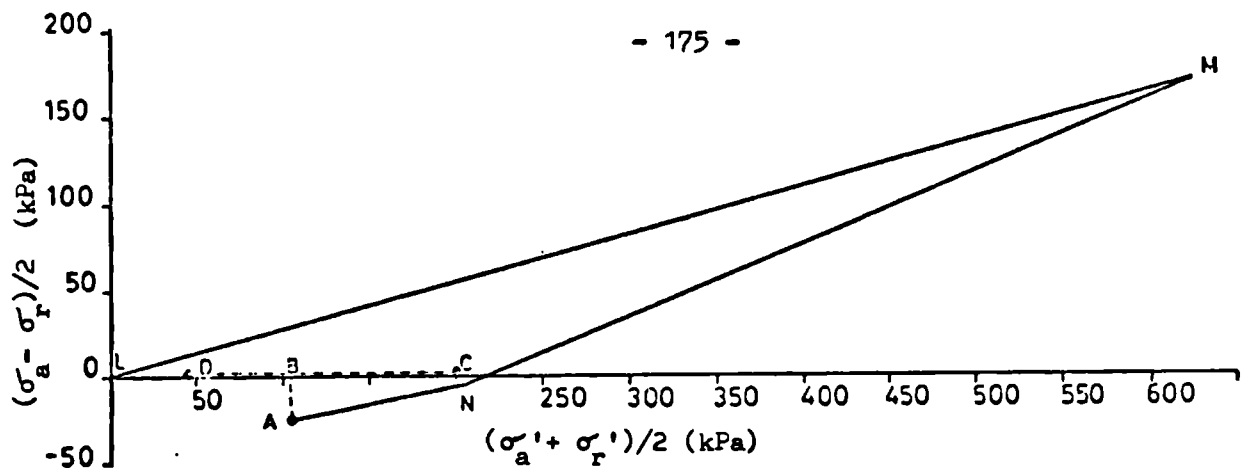


Fig. 4.14 Stress paths for undisturbed and reconstituted sedimentary clays.



LMNA = Anisotropic consolidation stress path
 AB = Undrained 'perfect' sampling
 BC, BD = Isotropic consolidation and swelling

A = Final consolidation point for tests F1, G1
 C = Final consolidation point for test F2
 D = Final consolidation point for test F3

Fig. 4.15 Anisotropic consolidation path used in the investigation.

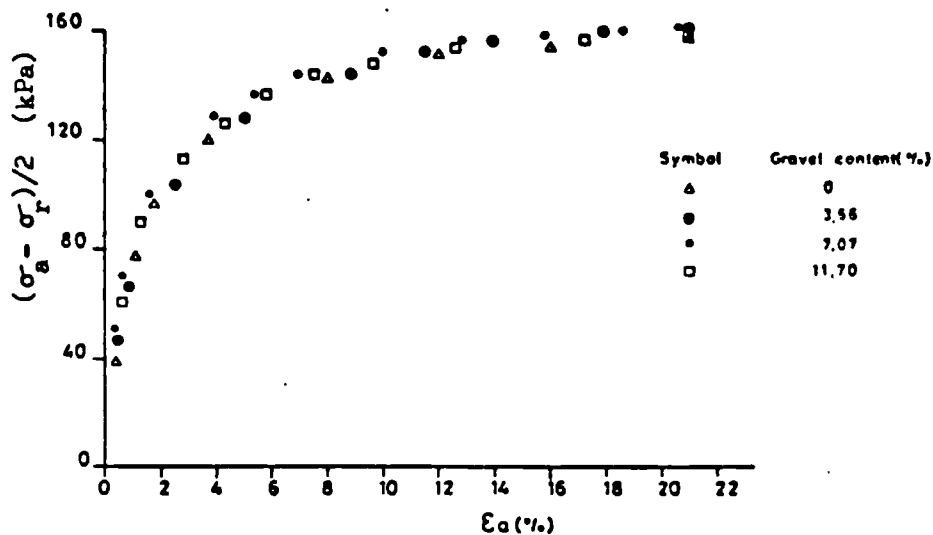
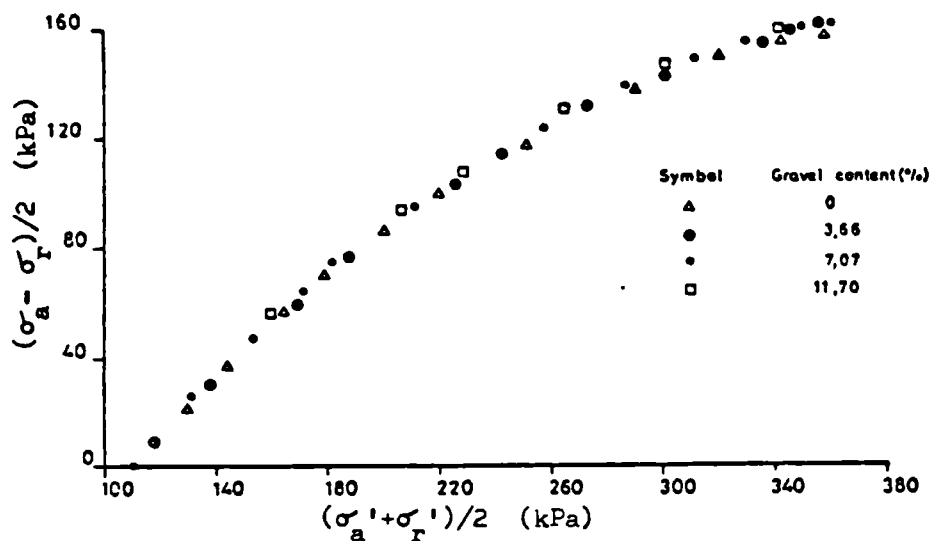


Fig. 4.16 Undrained compression behaviour of the material with different gravel contents.

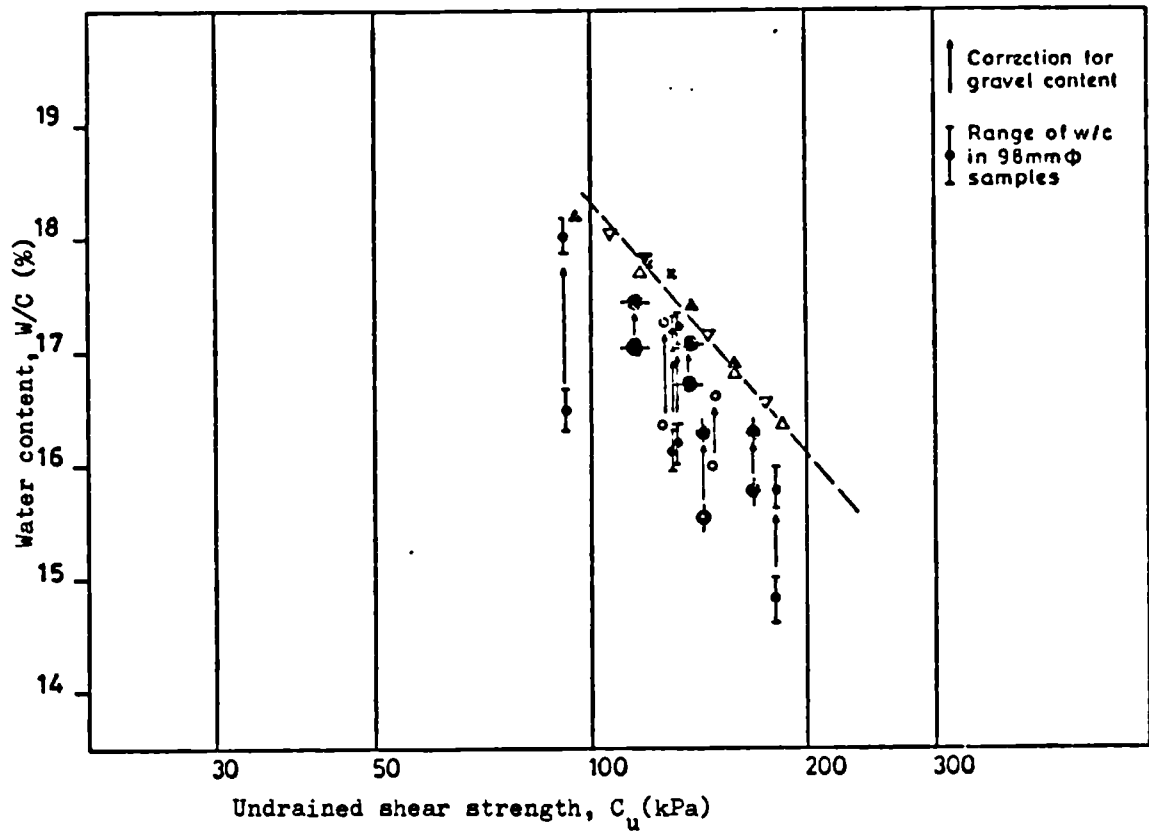


Fig. 4.17 Undrained shear strength-water content relationship

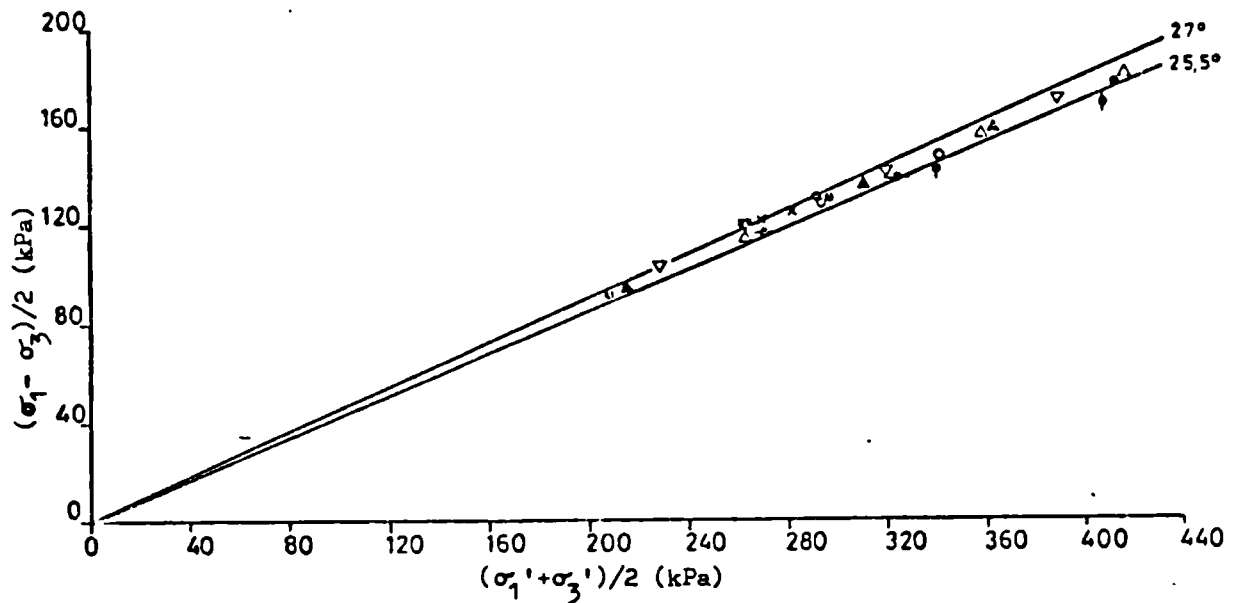


Fig. 4.18 Ultimate strengths from triaxial tests in terms of effective stress.

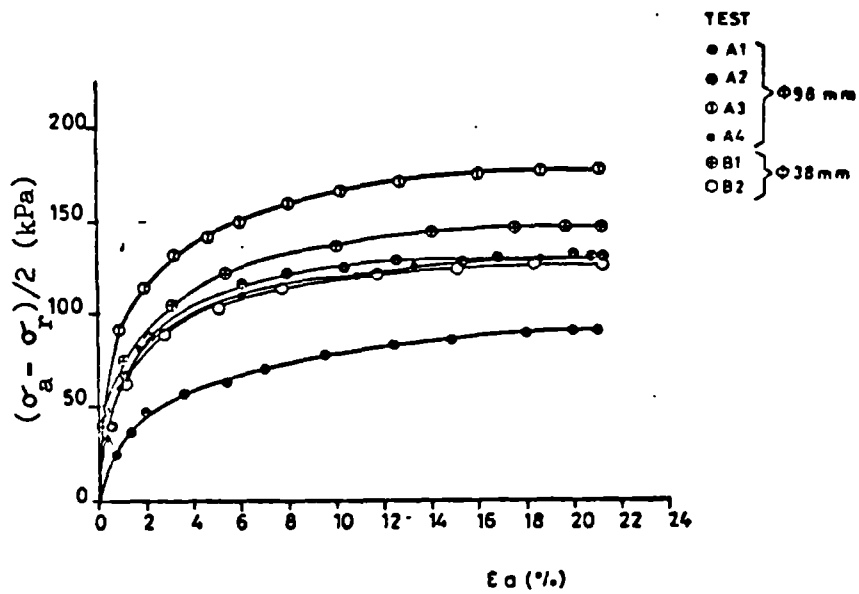
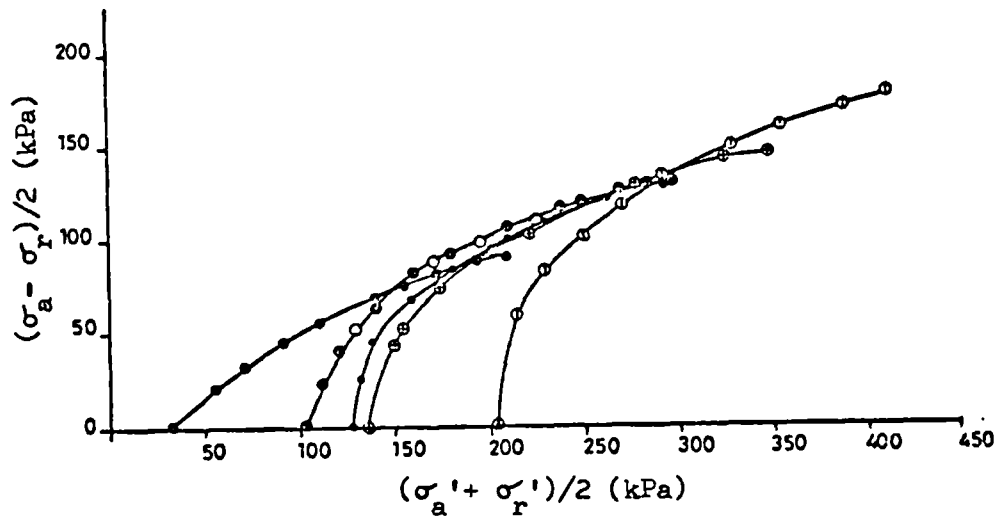
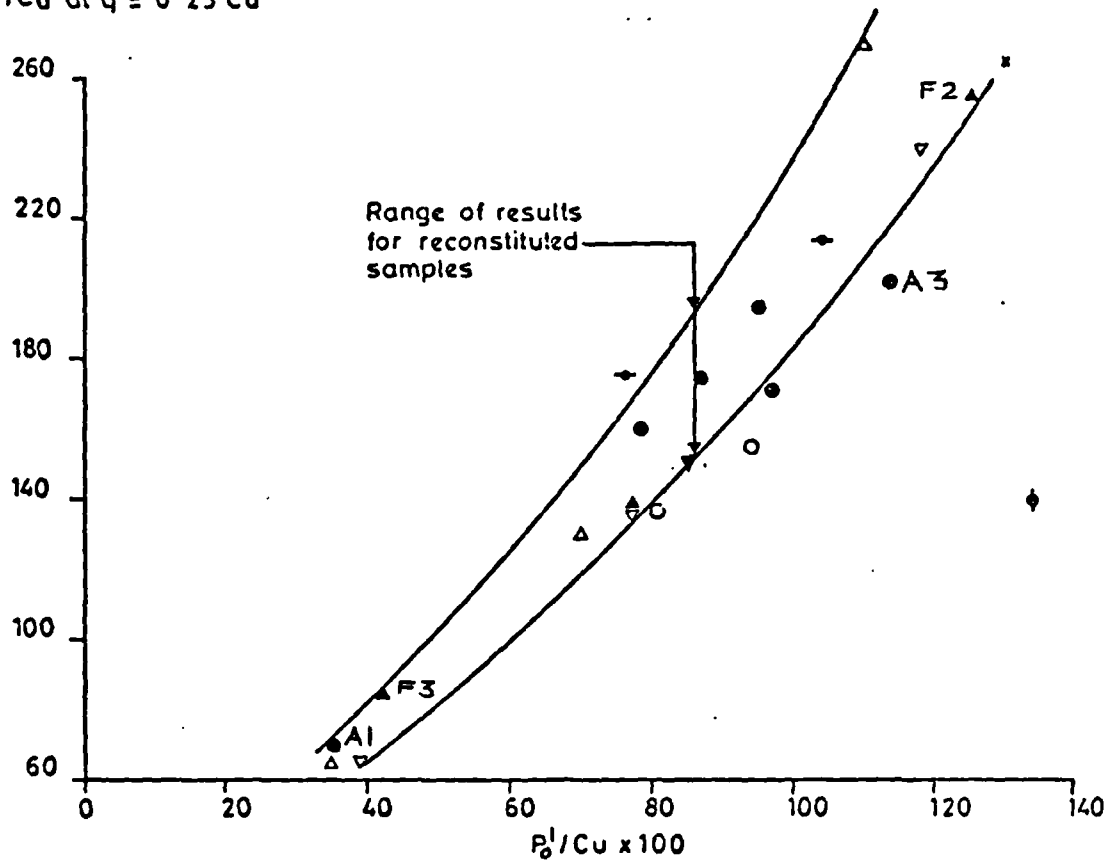


Fig. 4.19 Undrained compression behaviour of intact samples

Eu/Cu at $q = 0.25 \text{ Cu}$



Eu/Cu at $q = 0.5 \text{ Cu}$

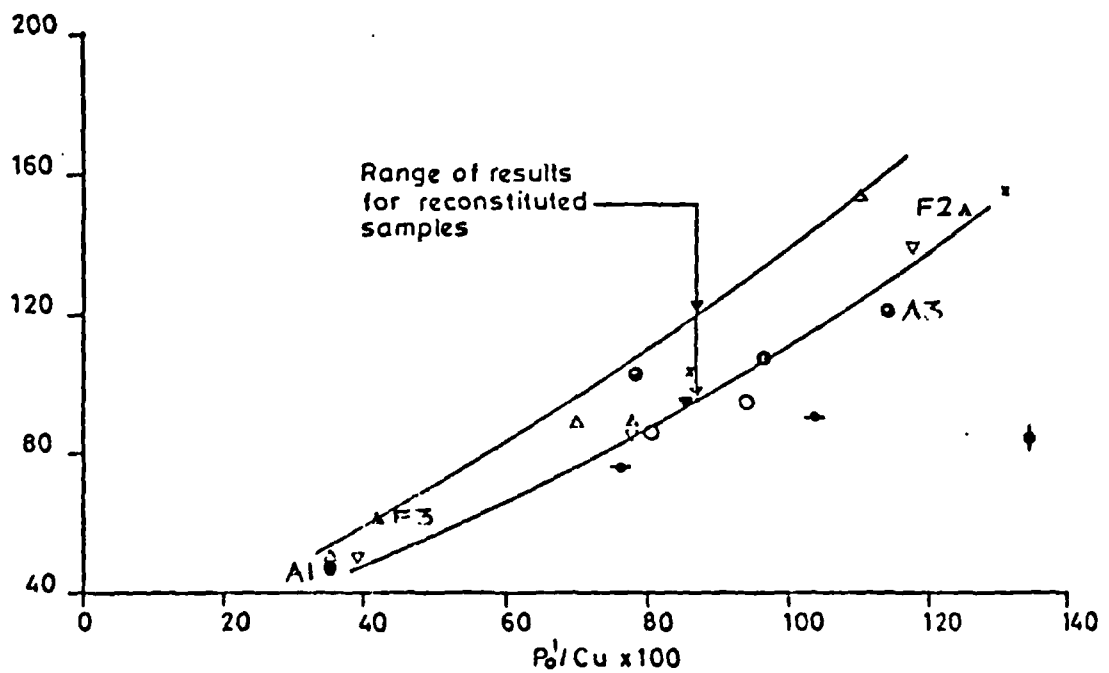


Fig. 4.20 Undrained deformation moduli.

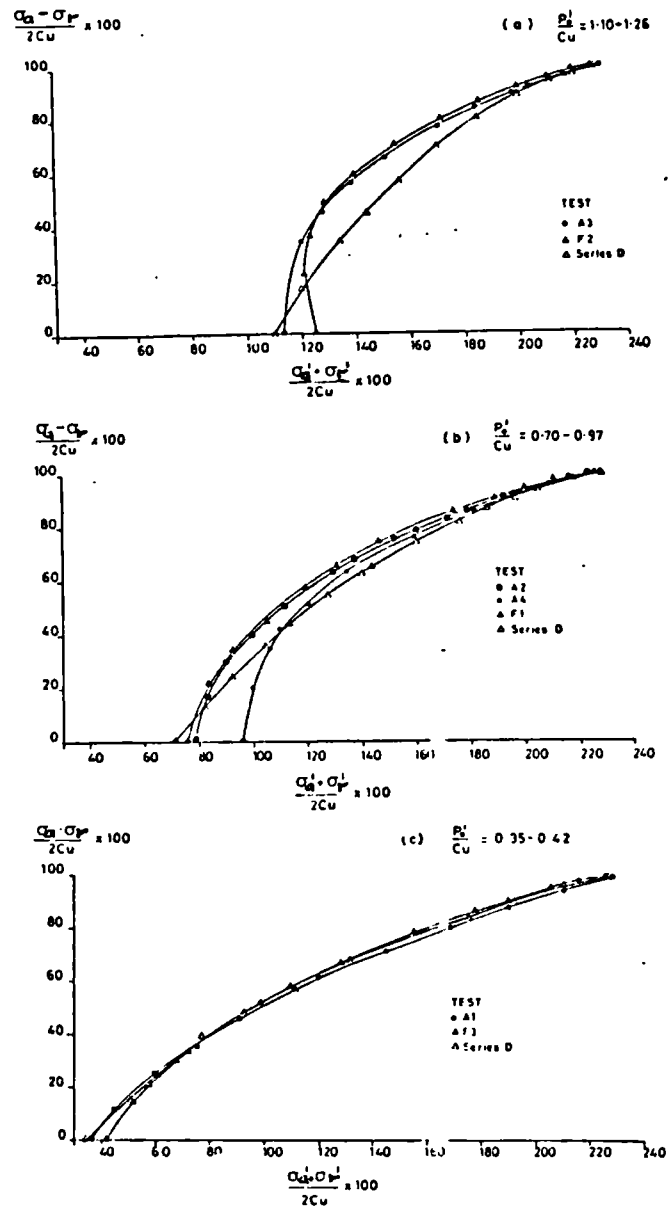


Fig. 4.21 Normalised stress paths for Series A, D and F.

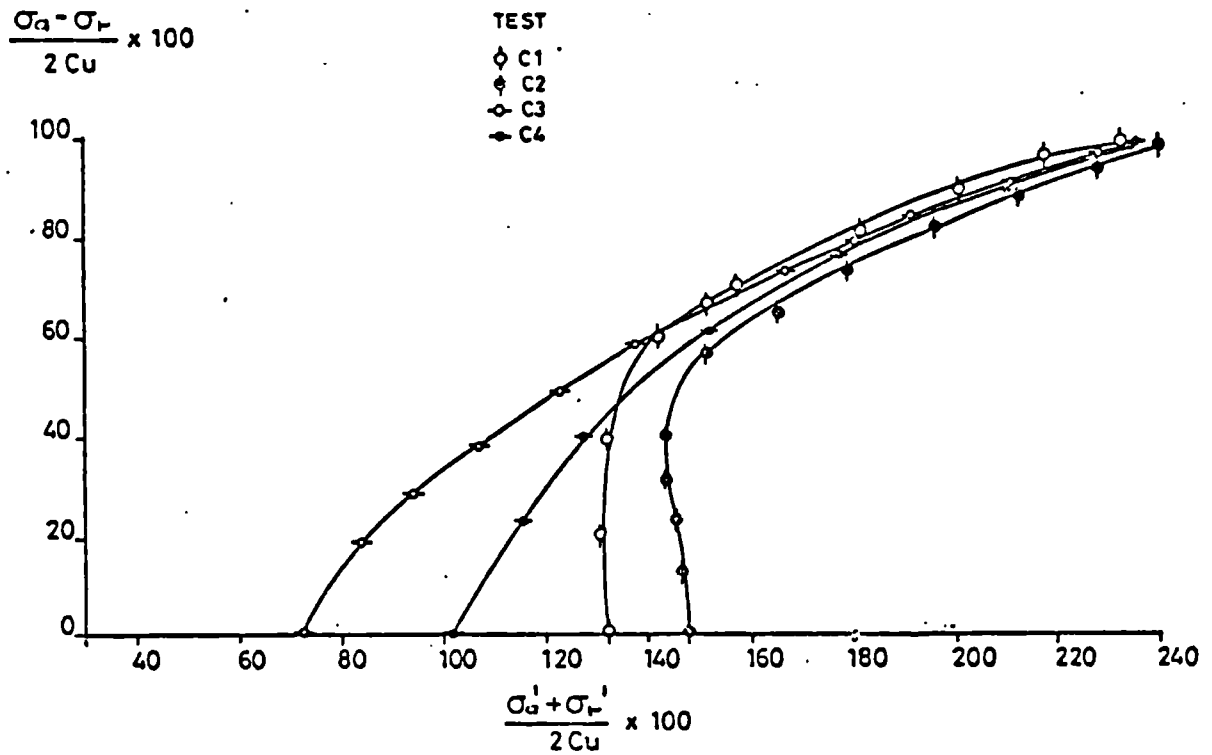


Fig. 4.22 Normalised stress paths for unconsolidated remoulded samples.

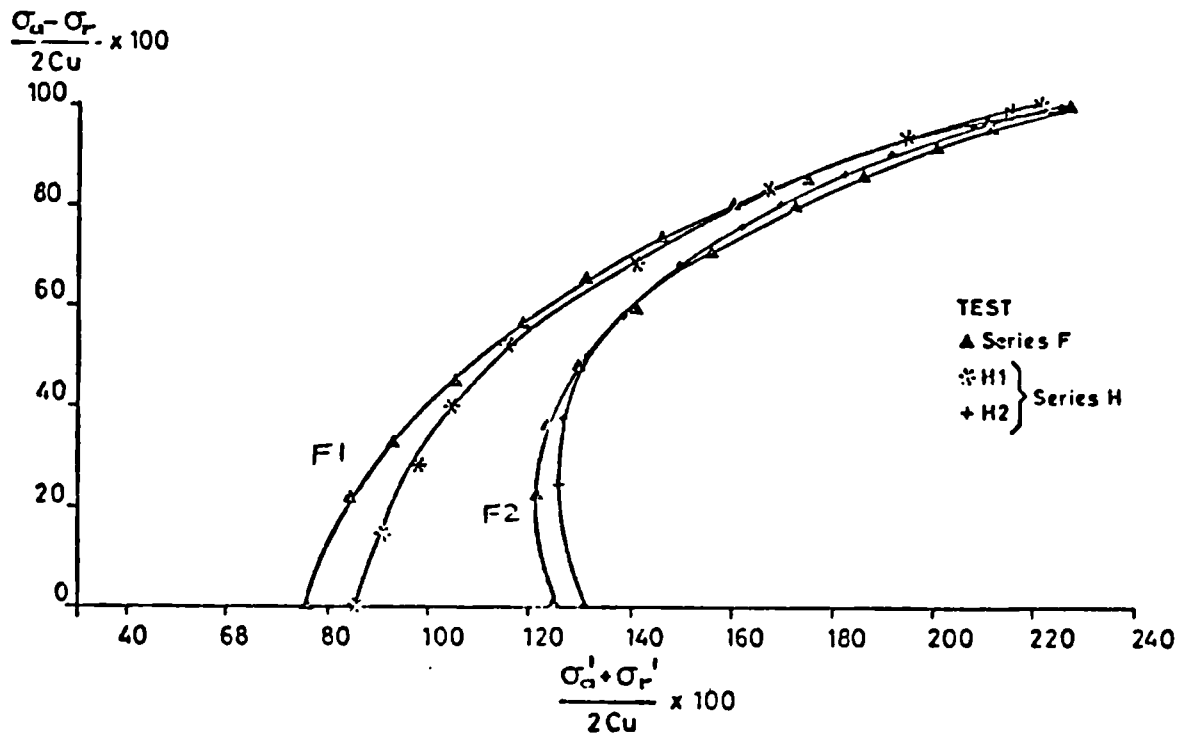


Fig. 4.23 Normalised stress paths for Series F and H.

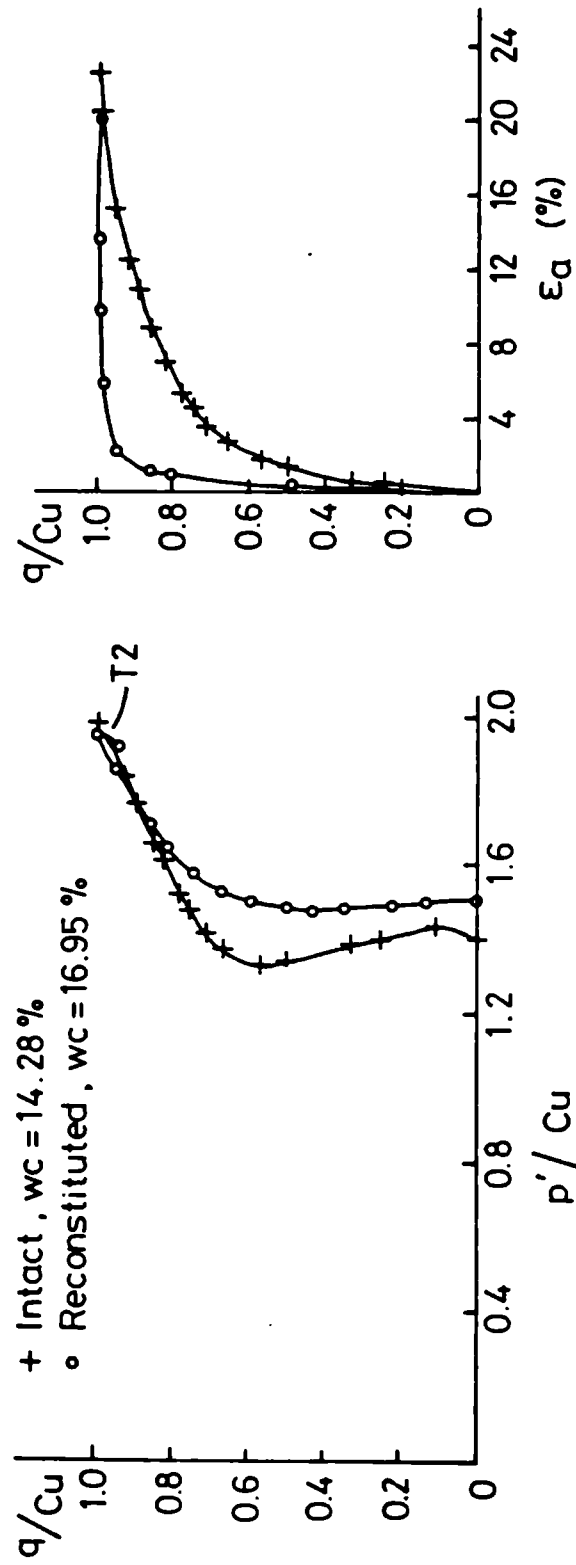


Fig. 4.24 Comparison of intact and reconstituted material behaviour in unconsolidated undrained compression tests on Lower Cromer Till.

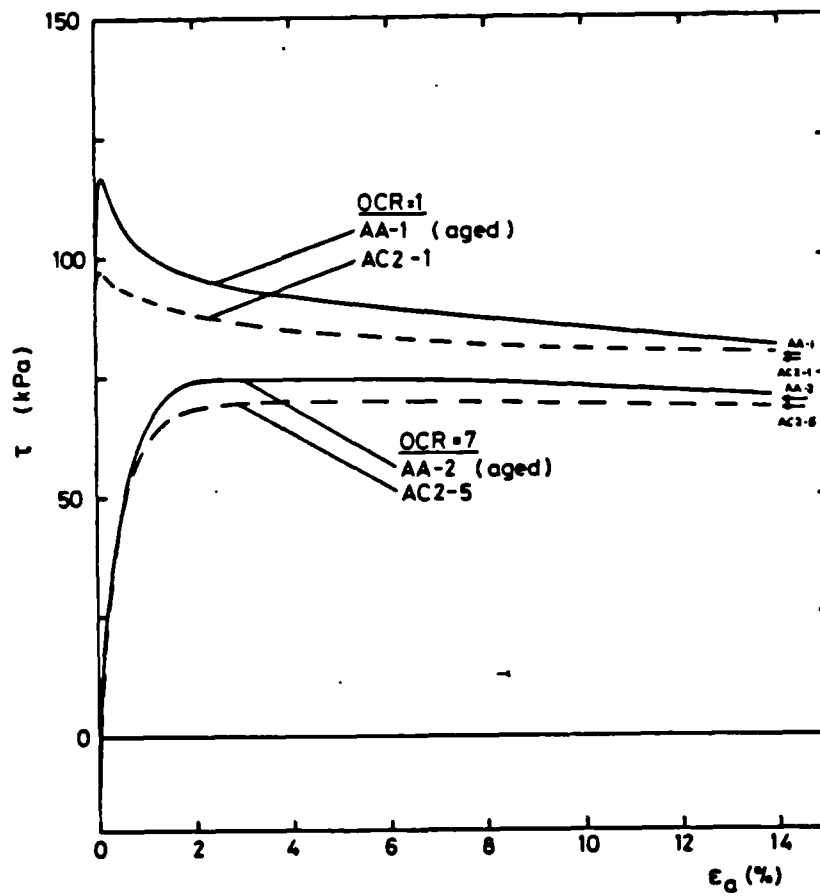
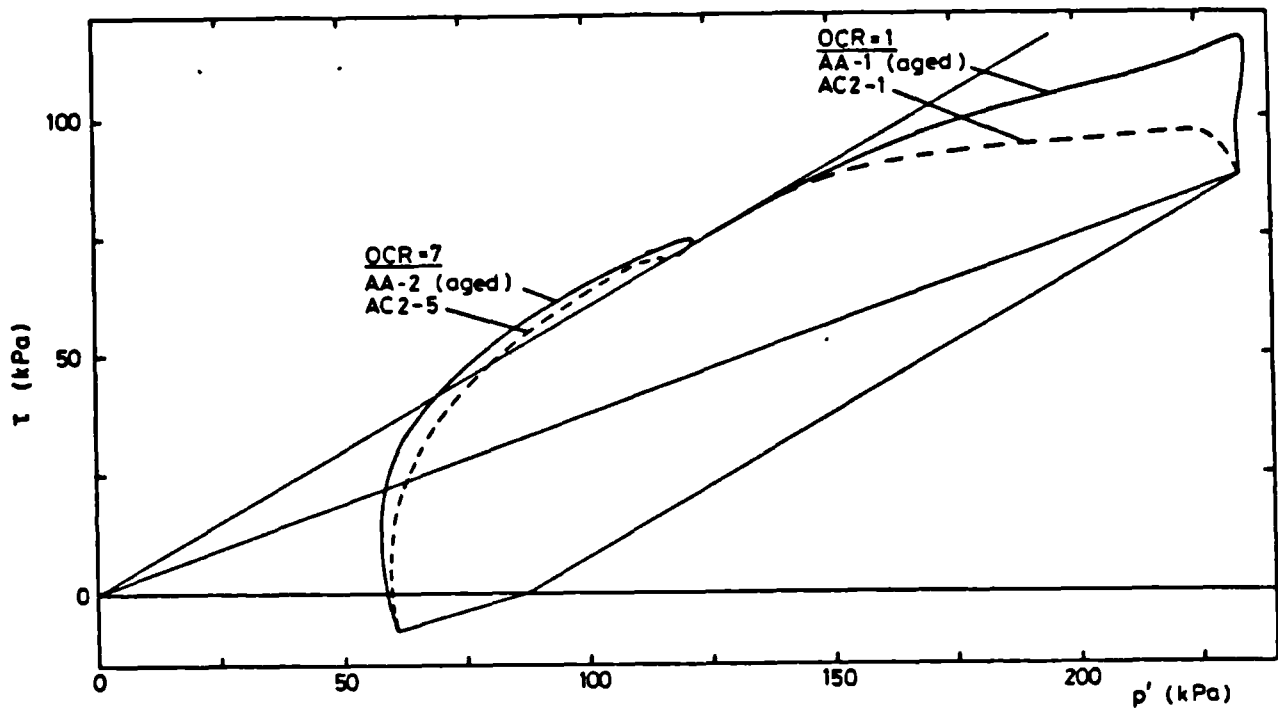


Fig. 4.25 Effects of aging in reconstituted samples of Lower Cromer Till OCR 1 and 7.

PART B

LABORATORY APPARATUS DEVELOPMENTS

CHAPTER 5

A COMPUTER-BASED CLOSED-LOOP CONTROL SYSTEM FOR
SOIL TESTING EQUIPMENT

SYNOPSIS

A control system has been developed which is capable of applying precisely specified sequences of stresses or deformations to soil samples in various pieces of testing equipment. The closed-loop control is based on a digital computer which both analyses data from transducers monitoring the test's progress and the soil's response and directs servo-systems operating pressure and displacement regulators. The system, and the instrumentation on which it depends for feedback, is described and the range of tests which can be performed with it, is discussed. Some results obtained using the system to control the hydraulic triaxial apparatus are presented. Components of the control system can operate independently of the computer to provide some automation in testing; an example is given of their use for K_0 consolidation and swelling.

5.1 INTRODUCTION

The strength and deformation characteristics of soil are dependent on the conditions under which they are loaded and deformed. Prediction of field behaviour involves, therefore, subjecting soil elements in the laboratory to loading and deformation conditions which simulate the particular field situation under consideration, i.e. subjecting them to the appropriate stress or strain path.

No single testing apparatus is capable of simulating all aspects of field loading or deformation. Various pieces of apparatus have, therefore, evolved, each capable of a particular range of stress or strain paths.

The value of each of these items of equipment is enhanced when systems can be introduced to control the applied loads, stresses or displacements. Various analogue devices have been used for control of individual pieces of testing apparatus. For example, Cullingford (1969) describes the features of a servo-controlled hydraulic testing machine capable of applying programmed stress changes with time to cylindrical soil samples. A servo-system of this type for dynamic triaxial testing of soils is outlined by Cullingford et al (1972). An electro-pneumatic control system for the triaxial test was introduced by Lamb and Anderson (1973). A particularly sophisticated analogue control system was devised by Saada (1968) who describes a pneumatic computer which performs the operations necessary for testing thin hollow cylinders of soil at any desired rate of stress change.

Wood (1972) appears to have been the first to introduce digital control into soil testing when he developed a digital computer system for controlling the Cambridge True Triaxial Apparatus. In this apparatus, stresses are applied to the faces of a cuboidal soil specimen through rigid plattens, opposite pairs of which are driven by motors. In the control system a PDP8/E computer monitors the applied stresses via load cells in the plattens and ensures that these stresses follow a pre-programmed sequence by directing the motor speeds.

This Chapter describes a new control system based on a digital computer. Unlike other control systems which have been developed, both analogue and digital, this system is not dedicated to one piece of testing equipment but is sufficiently versatile to control the stresses or strains applied, for example, in the hydraulic triaxial apparatus (Bishop and Wesley, 1975), the plane strain apparatus (Atkinson, 1973) and apparatus described in Chapter 6 for testing hollow cylinders of soil. Examples of its use with the hydraulic triaxial apparatus are presented together with a discussion of the range of tests which can be performed. The emphasis has been on developing a control system capable of performing undrained cyclic tests in the hydraulic triaxial apparatus and of simultaneously manipulating 6 parameters, as required in the new hollow cylinder apparatus.

5.2 DIGITAL COMPUTER CONTROL

Principles

The essential feature of any automatic control system is an error-determining device which compares input and output quantities and seeks to eliminate any difference between them by controlling, through some machine, the flow of energy from an external source to a load. Comparison may be made either between analogue or digital signals.

In a common form of analogue control system a servo-amplifier compares two electrical signals, a command signal and a feedback signal, from either a transducer or tachogenerator. To eliminate any difference between these two signals, the amplifier controls the energy supplied from a power pack to a motor which either drives directly the tachogenerator or varies indirectly the parameter being measured by the transducer, see Fig. 5.1. Use is made of this principle in components of the digital control system described herein. However, it is insufficient on its own for a fully automated test to be run since a varying command signal is required. This is supplied in a fully analogue control system by, for example, a programmable electronic signal generator or by a stepping motor and pulse counter.

In a digital control system, a digital computer has, amongst its rôles, those of the error-determining device and the command signal generator. The computer fulfills its rôle as an error-determining device by acquiring, storing and retrieving data from transducers which are monitoring the progress of a test; it processes the data so that it can make comparisons between physical quantities, such as stress, strain, deformation, etc. Any differences between measured and desired values are corrected by calculation and transmission of command signals to units generating the particular parameter. The computer fulfills its rôle as a command signal generator by calculating and transmitting the command signals required to bring about the desired progressive changes in these parameters. These changes may be related to time by reference to an internal clock in the computer.

Digital control systems have a number of advantages over analogue systems which, for certain applications, compensate for their higher cost.

- (i) As an error-seeking device the computer is not limited in the number of comparisons that it can make at any instant. Control can be based on feedback from any transducer in any combination. (In analogue systems only two electrical signals can be compared simultaneously; there is no control link between stresses, a separate analogue circuit being required for each desired stress link.) This leads to a much greater degree of flexibility in control by digital systems. Stress-controlled tests can be conducted under both undrained and drained conditions. (Compensation for changes in volume in drained tests cannot be made in analogue systems, Cullingford et al, 1972, so that only load-controlled tests are possible under these circumstances.) Rates of stress or strain change can be controlled directly or related to the progress of pore pressure equilibration or dissipation.
- (ii) Independent control of stresses (or strains) means that the full range of stress (or strain) paths possible in any piece of apparatus can be generated and followed.

- (iii) The flexibility of a digital system results in its not being dedicated to one piece of apparatus.
- (iv) Greater precision can be achieved in digital control since comparisons can be made in terms of physical units derived using signals resolved to microvolts and converted using calibration factors which take account of non-linearity and hysteresis in the transducers. Measurement errors, such as drift in the energising of transducers or compliance of the transducers, can be compensated for by storing information on them in the control program. There is greater precision, too, in the stress (or strain) path, and its variation with time, generated in a digital system, compared to that set by an electronic signal generator.
- (v) Output data can be selected and processed giving the convenience of immediate feedback on a test's progress.

Control Algorithms

The control algorithm which is implemented by the digital computer both generates the stress or strain path which a sample is to follow and ensures that this path is followed. Comparison of the measured and desired state of a sample at a particular time or point in stress and/or strain space is, then, an essential part of the algorithm. The path may be related to the time directly or indirectly by observation of the progress of, for example, consolidation using measured pore pressure differences. The algorithms rely on calibration factors for the control systems, for example factors which relate the rate of stress change to command signal, but these calibration factors are continuously updated as the test proceeds.

During the course of this research a number of control algorithms and control programs have been designed; these cater for anisotropic consolidation, including K_0 consolidation, controlled gradient consolidation, variable and constant strain rate tests, and undrained cyclic triaxial tests in which stress levels are controlled. It would

be inappropriate to describe these herein. However, to illustrate the features of a typical control algorithm, Appendix 5.1 to this Chapter describes the algorithm designed for undrained cyclic loading and used extensively to produce the data described in Chapter 10.

5.3 DESCRIPTION OF THE CONTROL SYSTEM

The System Configuration

The closed-loop control system in which the digital computer operates is shown as a block diagram in Fig. 5.2. For this example, the system is shown controlling two stresses, σ_a and σ_r , in the hydraulic triaxial apparatus. Since the computer can only deal with digital input and output it is immediately interfaced with an analogue-to-digital converter (ADC) and with a digital-to-analogue converter (DAC). The former, the digital voltmeter, enables digital information to be obtained from transducers which monitor the applied load or displacement and the response of the soil element; the transducers are sampled by a scanner at any instant dictated by the control algorithm. The DAC enables digital signals, calculated by the computer, to be converted to an analogue signal suitable for operating the pressure or displacement control systems.

The hardware comprises an assembly of largely standard components which are commercially available (see Table 5.1 for information on suppliers). Brief details of the computer system are given in Table 5.2, and of the servo-system in Table 5.3.

The Computer and Peripherals

The control system is based on a DEC PDP 11-05 minicomputer which is directly addressable through its switch register. In its basic form, the system require only 8k of core memory. However, this has been enlarged to 24k to increase the quantity of data that can be stored in long duration cyclic loading tests. Provided within the central processor is a real-time clock which generates interrupts at mains frequency.

The computer is interfaced directly with the devices shown in Fig. 5.2 and in more detail in Fig. 5.3. Each module becomes an extension of the PDP 11 and is operated by the computer and cannot communicate directly with any other module so that the system can only be used under program control from the computer. The devices comprise:-

(i) analogue scanner (channel selector)

The scanner is the analogue signal switching unit which can handle signals from transducers in any sequence. Fifty channels have been provided each with three reed switches which are operated through coils. The interface receives channel address information from the computer and controls the input selectors. A constant current power supply is linked to the scanner to energise the selected transducer.

(ii) analogue to digital converter (digital voltmeter)

In the control system the rôle of the digital voltmeter is to digitize analogue signals in the form of DC voltages. The voltage ranges and reading rates which are available are presented in Table 5.2; the required range and rate are selected remotely in the control algorithm. In the 0-100mV range and at a rate of 25rdgs/s, the instrument has a sensitivity of $1\mu\text{V}$; this provides for very good resolution with the transducers in use. Sensitivity is reduced at faster reading rates.

(iii) digital-to-analogue converter (DAC)

Six digital-to-analogue converter channels are provided in the system; each has a 10-bit twos-complement binary input, loaded in two bytes, and gives outputs from -10 to +10 volts. The stability of the DAC has been found to be extremely good, although during control, the signal from the DAC is monitored as a check. Analogue signals can be set to an accuracy of +5mV.

(iv) teleprinter

This provides the communication link in the system enabling there to be dialogue between the operator and the computer. The control algorithm may be presented to the computer via the teleprinter, together with various pieces of information required at the start of a test. The teleprinter provides a hard copy of the output from an experiment.

(v) high speed paper tape reader and punch

These peripherals provide an alternative means of presenting information to the computer and receiving data. Control programs may be listed on paper tape and entered into the computer memory in this form using the reader; data output on paper tape is suitable for presenting to other computers for re-analysis and plotting.

Although not used in the present control system, the computer is interfaced with a further module which comprises 32 channels, each capable of switching up to 30V at 200mA. Since these channels can be controlled remotely, they may be used in the algorithm to operate switches which could, for example, switch from stress to strain loading or to reverse drive directions.

The Servo-systems

The voltage set up by the computer via the DAC provides the command signal to the servo-systems which regulate either load, pressure or displacement. These servo-systems comprise a servo-amplifier, a form of analogue control system referred to earlier and servo-motors.

The servo-amplifiers are capable of operating in either a 'velocity' mode or a 'positional' mode. In the former, the amplifier uses the signal from the tachogenerator on the motor it is driving as feedback, and compares this with the command signal; differences between the command and feedback signals are eliminated by alteration of the power

supplied to the motor and hence its speed of rotation. For a fixed command signal, therefore, the servo-amplifier operating in the 'velocity' mode rotates a motor at a constant speed, despite changes in the load on the motor; the speed is directly proportional to the input voltage and its direction depends on the polarity of the signal, i.e. a bidirectional drive is achieved.

In the positional mode, the servo-amplifier is used to compare the command signal and a feedback signal, not from the tachogenerator, but from a transducer monitoring either a load, pressure or displacement. When any difference between the two signals is detected, power is supplied by the amplifier to the motor controlling the appropriate parameter until the difference is sensibly zero, the null position being attained without overshoot. For a fixed command signal, then, the servo-amplifier maintains either constant load, pressure or displacement. The velocity with which the motor drives to a null position and the ramp down to the null position may be varied.

The methods by which velocity and positional control are obtained are shown in Fig. 5.4.

Three types of servo-amplifier are used, differing in their input power and in their output torque and speeds; they vary, therefore, in the size of servo-motor which they are capable of controlling. Details of the amplifiers, EM83, EM100 and EM100A are given in Table 5.3. The minimum signal voltage that the amplifiers can discriminate, $\pm 5\text{mV}$, is compatible with the accuracy and stability of the DAC.

Four different servo-motors have been used in the development of the control systems. For the operation of air valves an EM83 amplifier has been used in conjunction with a 10801 motor (using armature feedback) and a 28PLT motor which incorporates a tachogenerator for feedback and control of velocity. For operation of the winches of a self-compensating mercury pot system, an EM100 amplifier has been used with a G9M4T motor. For operation of the displacement regulator, an EM100A amplifier has been used with a G12M4T motor. Details of these motors are given in Table 5.3.

The measured performances of the servo-systems (servo-amplifier and servo-motor) are summarised in Figs 5.5 to 5.8. The resulting range of available servo-speeds is shown in Fig. 5.9. To provide a sufficiently wide range of rates of stress change and displacement rates, the motors are connected to various reduction gear boxes.

Pressure Control Systems

Early development work was with a servo-drive connected to the standard self-compensating mercury control units (Bishop and Henkel, 1962, Figs 28 and 29); this replaced the variable speed differential drive described by Bishop et al (1973). The particular servo-drive, the EM100 (refer Table 5.3), operates in the 'velocity' mode through a 2400:1 reduction gearbox, giving a rate of stress change variable between 0.13kPa/hour and 1300kPa/hour in 1000 discrete steps. The servo-drive is also currently used in an open-loop control system for anisotropic consolidation and other drained stress paths. (Buri, 1978, El Ghamrawy, 1978, Gens, 1982).

The self-compensating mercury control units are eminently suitable for slow rates of testing. For faster stress-controlled tests, when flow of mercury through the connecting lines is too slow, an alternative pressure control system has been developed based on a servo-operated air valve. The unit can be seen in Plate 5.1. Its position in the air system is shown in Fig. 5.10. The air valve spindle is rotated by a servo-motor operating through a reduction gearbox. Pressures can be set to an accuracy of ± 0.6 kPa, provided that a coarse air control valve is introduced on the upstream side of the servo-valve. The servo-valve is extremely stable, even when there are fluctuations in the supply pressure. There is a reasonably linear relationship between regulated pressure and angular rotation of the valve spindle which is useful in a control system. With a 500:1 reduction gearbox, stresses can be changed at rates variable between 0.6kPa/min to 600 kPa/min in steps of 0.6kPa/min.

In the hydraulic triaxial cell, two pressure regulators are used for control of axial stress, σ_a , and radial stress, σ_r . In principle the same system can be used to control back pressures.

Displacement Control System

As described by Bishop and Wesley (1975), constant rate of strain tests are run in the hydraulic triaxial apparatus by connecting the loading pressure chamber to a standard control cylinder with a screw-controlled piston which is then rotated by the variable speed drive unit. In the closed-loop computer system, the drive unit has been replaced by a servo-system, the EM100A, refer Table 5.3, which has been connected to a compact form of the screw-controlled piston and cylinder, see Fig. 5.11. The servo-amplifier operates in the 'velocity' mode and is capable of maintaining a constant displacement rate without feedback from an axial displacement transducer. Feedback is used in tests with controlled changes in strain rate and the motor is under the direct control of the computer system.

5.4 INSTRUMENTATION

The instrumentation used to monitor progress of a test in the hydraulic triaxial apparatus and to measure the soil's response is described. Similar instrumentation is suitable for the other testing devices. Three developments have been undertaken to provide the feedback data which is essential for the control system: measurement of radial strain (for K_0 tests), measurement of pore pressure at mid-height (for control of pore pressure gradients) and measurement of volumetric strains.

The geometry of the sample is derived from its initial dimensions and from changes in height and diameter measured by displacement transducers. To date, axial displacements have been measured to $5\mu\text{m}$ with an externally-mounted resistive transducer with 25mm stroke. Allowances for load cell deflection are made within the control program. Monitoring based on LVDTs mounted internally over a central gauge length, used, for example, by Daramola (1978), is perfectly feasible. Diametral changes have been measured with a new radial strain belt (Fig. 5.12). It consists simply of a foil-strain-gauged strip of spring steel soldered between two rigid arms which are supported by a pulley system. The device is capable of resolving to $\pm 3\mu\text{m}$ with a drift of $2\mu\text{m}/\text{day}$. For

increased resolution of radial strains, semi-conductor strain gauges were introduced; these gave resolutions of $\pm 1 \mu\text{m}$ but problems with their stability have not been overcome. On samples larger than those of 38mm diameter tested in the hydraulic triaxial cell, lateral strain gauges of the type described by El Ruwayih (1976) or Menzies (1976) would be suitable.

The radial stress acting on the sample, σ_r , is the cell pressure measured by an unbonded wire pressure transducer which is continuously energised to avoid the transient heating effects in the wire which are evident at the rapid pulsing and reading times involved in the logging operation. The axial stress on the sample, σ_a , is derived from a measurement of the deviator load on an internal load cell, from σ_r and from the geometry of the sample. The internal load cell was designed by Dr A. E. Skinner and is described by Bishop et al (1975).

In the hydraulic triaxial apparatus, pore water pressures are measured both at the base of the sample and at its mid-height using a probe which is mounted flush with the sample's cylindrical surface. This piezometer probe is described in detail in Chapter 6. The base measurement is conventional - a fine porous stone, of 28mm diameter, has been let into the pedestal so that their top surfaces are flush and has been fixed rigidly around its periphery so that the effects of bedding on measured pore pressures, refer Bishop and Henkel (1962), are non-existent. Two pieces of stainless steel hyperdermic tubing connect the base pedestal to an unbonded resistive strain-gauged pressure transducer.

The use of two pore pressure measurements has a double advantage, as discussed subsequently. In high speed testing, the pore pressure probe monitors pore pressure changes unaffected by end restraint (Chapter 6). In drained and consolidation stages, the rate of testing can be controlled to limit the pore pressure difference which develops between the middle of the sample and the base, where the back pressure is recorded.

For automatic measurement of volume change two approaches have been used. In the first direct measurement of the pore fluid transferred

to or from the sample has been made using the transducer shown in Fig. 5.13. In the second, volume changes have been calculated on the basis of the axial and radial strain measurements.

For cyclic work, two features for the transducers were essential: repeatability, especially at low stresses, and lack of hysteresis. The standard internal load cell (Fig. 5.14) exhibits considerable rate-independent hysteresis, mainly as a result of changes in bedding of the sensor on its support points (refer to Appendix 5.2 in this Chapter for a fuller discussion); Fig. 5.15 shows the level of hysteresis involved. This was brought to acceptable limits by:-

- (i) ensuring no change in contact position between the sensor and its supports, and
- (ii) chamfering the edges of the sensor nibs to ensure that there was no contact between it and the housing.

The improved performance can be judged from Fig. 5.15. The maximum hysteresis in the pressure transducers amounted to 1kPa in an excursion from 70 to 550kPa; the maximum hysteresis in the displacement transducers was $25\mu\text{m}$ in an excursion of 12.5mm.

Linearity between the change in a transducer's output and the change in the parameter that it is monitoring is an obvious advantage in a control system, although, of course, non-linear calibration factors can be handled by the computer. For the pressure transducers, linear behaviour was obtained by operating them at pressures above 200kPa.

To accommodate the additional internal transducers required to provide feedback to the control system, the cell top on the standard hydraulic triaxial apparatus was enlarged (see Plate 5.1). The enlarged top conveniently housed the piezometer probe and radial strain belt, and outlets for the electrical leads from each transducer and for top cap drainage could be provided in the base of the cell.

5.5 RANGE AND PERFORMANCE OF CONTROL SYSTEM

The ability to vary independently and at any desired rate the stresses (and strains) applied to a soil element in a testing apparatus and to monitor changes in the element's geometry, volume and pressures, means that it is theoretically possible with digital computer control to follow any drained or undrained stress (or strain) path, subject to the limitations imposed by the particular apparatus.

Consolidation or swelling under various stress ratios can be conducted. Rates for consolidation or swelling can be specified in the control algorithm, based on an assumed C_v or C_g ; alternatively, the pore pressure gradient in the sample may be monitored and the rates of change of stresses may be controlled to ensure that the gradient does not exceed a pre-selected value. Controlled gradient consolidation (refer Lowe et al, 1969; Wissa et al, 1971) is also possible since the back pressure, i.e. the pore pressure at the base of the sample during consolidation, can be one of the controlled parameters.

K_0 consolidation is achieved by monitoring radial strain either directly from the radial strain belt or indirectly from the volume change and axial strain. Radial stresses are varied to maintain zero radial strain while the rates of both axial and radial stress change can be controlled to ensure full pore pressure dissipation. The accuracy with which K_0 conditions can be maintained is a function of the resolution and stability of the radial strain belt. Very high resolution is required for unloading since lateral strains are small for a wide range of unloading stress path directions (Lewin and Burland, 1970). Certainly, the best resolution can be obtained in a digital system and allowances for drift can be made within the control program.

Drained stress paths to failure present no additional problems in control, beyond those for consolidation under constant stress ratio. At failure there is the potential to switch from stress to strain control which would avoid runaway failure and would permit investigation of stress path induced strain softening.

Monotonic stress changes leading to undrained failure are relatively straightforward to control. Stresses may be calculated in triaxial tests on the usual assumption of deformation taking place as a regular right cylinder; alternatively the sample shape may be monitored and allowances made for barreling. Since the pore pressure gradient which exists in the sample as a result of end restraint can be monitored, the rate of testing can be varied either to ensure equilibration throughout the sample or to limit equilibration (which may be desirable when strains are being determined over a central gauge length).

Cyclic loading tests, particularly those which involve a switch from compression to extension, require careful programming to minimise backlash and inertia effects and to eliminate errors resulting from compliance in the load measuring transducers.

Since its development, the control system has been used almost exclusively for investigations in the hydraulic triaxial apparatus of the effect of undrained cyclic loading and rate of undrained loading on the Lower Cromer and Cowden Tills. The system is particularly suited to this type of investigation because of its ability to:-

- (i) provide truly stress-controlled cyclic loading,
- (ii) consolidate under K_0 conditions prior to and between sequences of cyclic loading,
- (iii) process the large volume of data generated in cyclic testing and recognise and report on significant events, and
- (iv) log at high speeds.

The overall accuracy achieved with the control system is determined by many factors. These include the accuracy and stability of:-

- (i) the digital to analogue conversion,
- (ii) the servo-amplifiers,

- (iii) the tachogenerators on the servo-motors,
- (iv) the air pressure regulators,
- (v) the transducers providing essential feedback.

Each of these was assessed and improved where possible. The overall accuracy that was achieved in stress control was approximately $\pm 1.0 \text{ kPa}$.

Two examples of the system's performance in conducting stress- and strain-controlled tests are presented. In the first, data is taken from an undrained stress-controlled compression-extension test on Lower Cromer Till. Figure 5.16 compares the target sinusoidal stress wave and the measured stress changes that were applied. In the second example, Fig. 5.17 shows the results of a test in which the strain rate was alternated between $0.025\%/\text{min}$ and $0.35\%/\text{min}$ at predefined axial strains. This test formed part of an investigation into the effect of rate of shearing on the behaviour of a glacial till; this type of test was first advocated by Richardson and Whitman (1963). The change in strain rate was achieved virtually instantaneously without overshoot so that the measured soil response was unaffected by the usual interruptions involved in changing gears, etc. The measured stress-strain curve is shown in Fig. 5.18.

5.6 USE OF SERVO-SYSTEMS FOR AUTOMATIC TESTING WITHOUT COMPUTER CONTROL

The fact that the servo-amplifiers which have been incorporated in the control system are capable of operating in 'positional' mode enables them to be used independently of the computer, for example, to maintain constant cell pressure or constant deviator load, or constant back pressure. A command signal is derived from a zenner diode within the amplifier, being varied through a dial potentiometer.

The 'positional' mode is particularly useful for following K_0 consolidation and swelling paths in the triaxial apparatus. The radial strain belt (see Fig. 5.12) is continuously energised and provides the feedback to the servo-amplifier which drives the motor operating the cell

pressure regulator. At the start of the test, the initial reading on the radial strain gauge is taken and this voltage is set as the command signal. Alternatively, a path to the estimated K_0 line may be followed before operating in 'positional' mode; the radial strain transducer reading at this point is then set as the command signal. The axial stress on the sample is then increased slowly. Any tendency for the sample to increase in diameter is registered by the strain belt and is counteracted by an increase in cell pressure under control of the servo-amplifier.

The use of a radial displacement transducer of the form shown in Fig. 5.12, i.e. one making a measurement on a discrete diameter, means that for K_0 conditions to be maintained throughout the height of the triaxial sample, the state of effective stress on that diameter must be the same as on all diameters at every level. Since the total radial stress is the same at all levels on the flexible boundary of the triaxial sample (end effects not coming into play to disturb σ_r if there is no radial strain) then pore pressures must be uniform throughout the sample, i.e. the sample must be fully drained. (In contrast, in the oedometer, maintenance of K_0 conditions implies a varying total stress along the height of the wall as drainage proceeds following a step change in loading.)

This requirement for full drainage sets strict limits on the rates at which axial and radial stresses on the sample can be changed. With the control system it is possible to control the rate of axial stress change so that excess pore pressures above a pre-scribed value are not generated. This is achieved by using the transducer monitoring mid-height pore pressures, i.e. the pore pressure probe, as feedback to the servo-amplifier which drives the motor operating the axial load regulator. By setting the command signal to the amplifier at a value slightly above the initial reading on the pore pressure probe (or below in the case of swelling), the axial stress is increased (or decreased for swelling) until the mid-height pore pressure reaches the value corresponding to the command signal. As dissipation of pore pressure proceeds, the mid-height pore pressure is maintained by continuous increases (or decreases) in axial stress.

The circuit diagram used for K_0 -control together with control over pore pressure gradient is shown in Fig. 5.19. Preliminary work by the author on 38mm diameter samples showed that the system was capable of following K_0 paths during loading. For dense Ham River Sand, similar K_0 paths to those observed by Daramola (1978) were followed by the control system. For unloading the radial strain device was not sufficiently sensitive for automatic control. For long term tests on clay, the device was insufficiently stable. It is hoped that radial strain devices based on electrolevels will overcome the problems of resolution and stability.

From the circuit diagram for the servo-amplifier operating in positional mode, shown in simplified form in Fig. 5.4, it can be seen that the device is designed such that a null position is never achieved. Even when command and feedback signals (a and b at the amplifier) are balanced, a signal reaches the velocity amplifier via the proportional band (cde); the size and polarity of this signal depends on the initial setting of the radial displacement transducer. Such an 'internal' disturbance is not strictly necessary in a K_0 test since an 'external' disturbance to the null position exists in the form of a continuously changing axial or radial stress.

While direct elimination of the proportional band would remove the internal disturbance it would result in a velocity for correcting imbalance between command and feedback signals which reduced as the difference between these signals reduced. This is undesirable in terms of the resulting stress path. Instead, the proportional band has been retained, but the initial signal from the radial displacement transducer is set at zero. In this way, when there is balance between the command and feedback signals, zero voltage exists across the proportional band and no power is supplied to the motor. When differences exist between the command and feedback signals, the proportional band ensures that a constant velocity results while the difference is being eliminated. A potentiometer powered from the supply within the servo-amplifier has been introduced to bring the initial feedback signal from the radial displacement transducer to zero.

5.7 POTENTIAL MODIFICATIONS TO THE COMPUTER CONTROL SYSTEM

The closed-loop servo-system which has been described was developed in the early part of this research (1975-76). Significant developments have taken place in hardware in the interim. Possibly the most important ones concern the availability of:-

- (i) Stepper motors to control the pressure and displacement regulators; these operate directly on a digital signal from the computer obviating the need for digital to analogue conversion. Since each step of the motor is under the control of the computer, the rotation rate is almost infinitely variable and gearboxes can be eliminated.
- (ii) Larger data storage facilities such as floppy discs.
- (iii) More sophisticated compilers which allow more rapid data acquisition and manipulation.

5.8 SUMMARY

The closed-loop computer-based control system and the associated instrumentation which have been developed have been essential for conducting the research which is described in this thesis and in theses by Takahashi (1981) and De Campos (1983). The system and instrumentation have:-

- (i) enabled cyclic loading to be carried out in the hydraulic triaxial cell on samples subject to a range of anisotropic consolidation histories;
- (ii) provided rapid data acquisition capability for transient tests on samples subject to a similar range of anisotropic consolidation histories;
- (iii) offered the potential for simultaneous variation of six parameters in testing hollow cylinders of soil to produce controlled rotation of principal stress directions.

The system utilises both pressure and displacement regulators and is capable of controlling either stress- or strain-controlled tests in a range of pieces of apparatus. It has been proven to be capable of:-

- (i) anisotropic consolidation, K_0 ,
- (ii) controlled gradient consolidation,
- (iii) variable strain rate control,
- (iv) undrained cyclic loading.

It has the merits of:-

- (i) good accuracy in the control of stress or strain paths,
- (ii) versatility and convenience in use, particularly over data handling,
- (iii) more efficient use of testing equipment.

<u>Equipment</u>	<u>Company</u>	<u>Address</u>
Computer-logger, (including DAC, ADC + I/O devices)	Solartron Schlumberger	Farnborough, Hampshire.
Servo-amplifiers and servo-drives	McLennan Servo Supplies Ltd.	Yorktown Industrial Estate, Doman Road, Camberley, Surrey.
Servo air-valves	John Watson & Smith Ltd.	Leeds, England.

Table 5.1 Supplies of Hardware for Computer Control System

Control Computer:

PDP11 - 05
Word length : 16 bits
Memory cycle time : $1.2\mu\text{s}$
Memory : 24K

Analogue Scanner:

Number of channels : 50

Reed switching: Operate - Reeds settle to bounce free
state in $<6\text{ms}$

Release - Reeds release within $<2\text{ms}$

Constant Current Bridge Power Supply: Range - $400\mu\text{A}$ to 30mA
Stability - $0.005\%/24$ hrs in
constant temp.
environment

Digital Voltmeter:

Nominal Ranges : 0 - 100mV, 0 - 1V, 0 - 10V, 0 - 100V, 0 - 1kV

Reading Rates : 25 rdgs/s for 4 nines (i.e. 10^{-4}V sensitivity for
0 - 100mV range)

10 rdgs/s for 5 nines (i.e. $1\mu\text{V}$ sensitivity for
0 - 100mV range)

Accuracy : 24 hrs $\pm 1^\circ\text{C}$. 0.002%rdg
0.001%fs

90 days $\pm 5^\circ\text{C}$. 0.004%rdg
0.001%fs

Digital-to-Analogue Converter:

Number of channels : 6

Output voltage : - 10V to 10V

Accuracy : $\pm 5\text{mV} \pm 0.1\%$ of peak-to-peak voltage range

Output impedance : 68 ohms

Maximum output current : 15mA each channel

Rate of voltage change : 0.5V/ms

SERVO AMPLIFIERS					SERVO MOTORS				
Type	Output Voltage (volts)	Continuous Current (Amps)	Velocity	<u>Resolution</u> Position	Type	Nominal Voltage (Volts)	Nominal Speed (Rpm)	Nominal Torque (oz-in)	Max Input Power (Watts)
(a) Stress control - self compensating mercury systems									
EM100 (SM 8020)	+24	4.0	10000:1	1500:1	G9M4T	18.0	3700	20.0	40.0
(b) Stress control - air valves									
EM83 (SM 8004)	+15	0.7	100:1	100:1	9904- 120-10801	24.0	1075	1.4	4.2
(SM 8022)			(Armature feedback) 1000:1	1000:1	28PLT	12.0	1790	2.16	6.0
(c) Displacement control									
EM100A	+24	8.0	10000:1	1500:1	G12M4T	30.0	2225	75.0	125.0

Table 5.3 Components used in the servo-systems

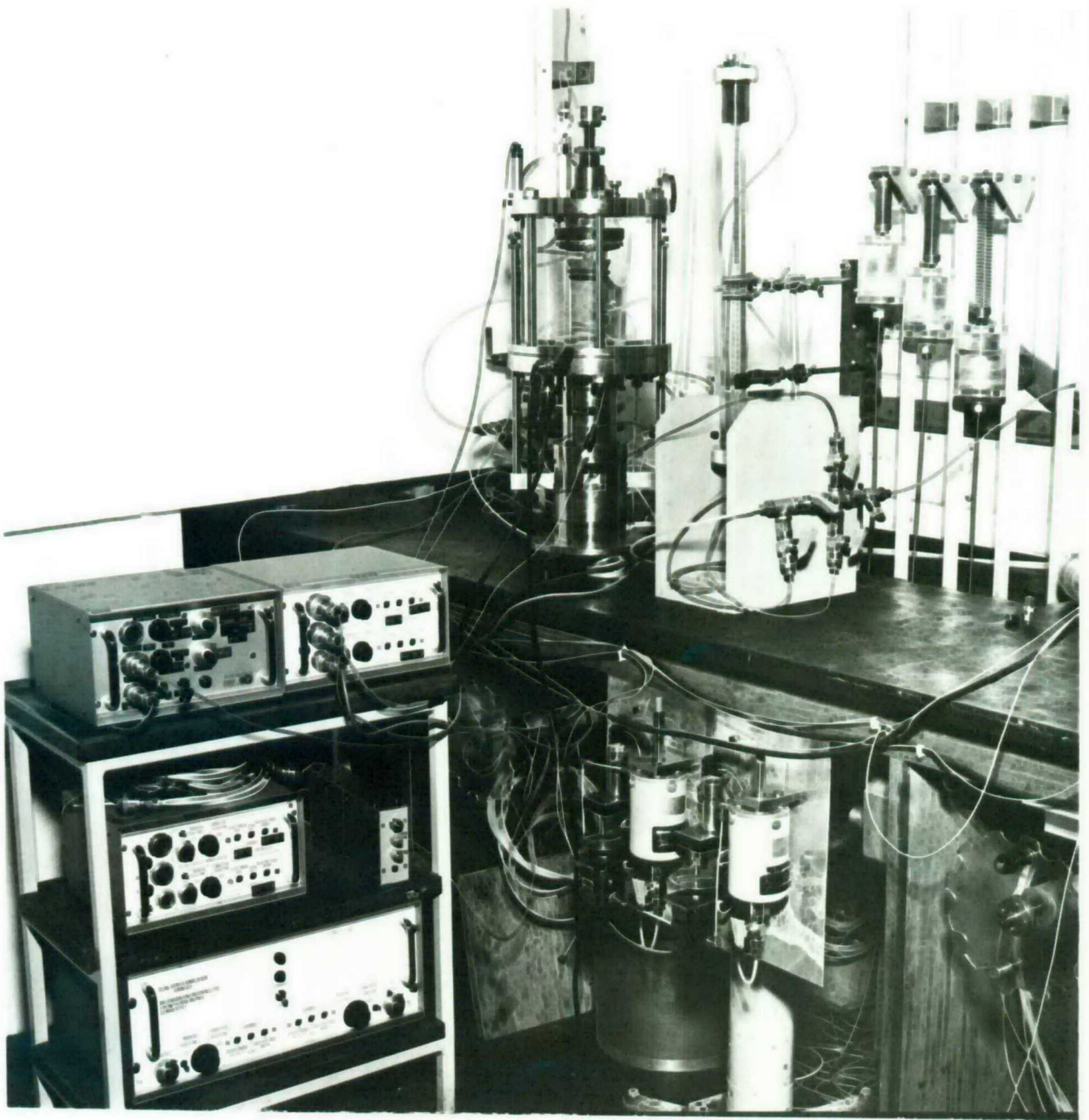


Plate 5.1 Computer-based closed-loop control system operating hydraulic triaxial apparatus.

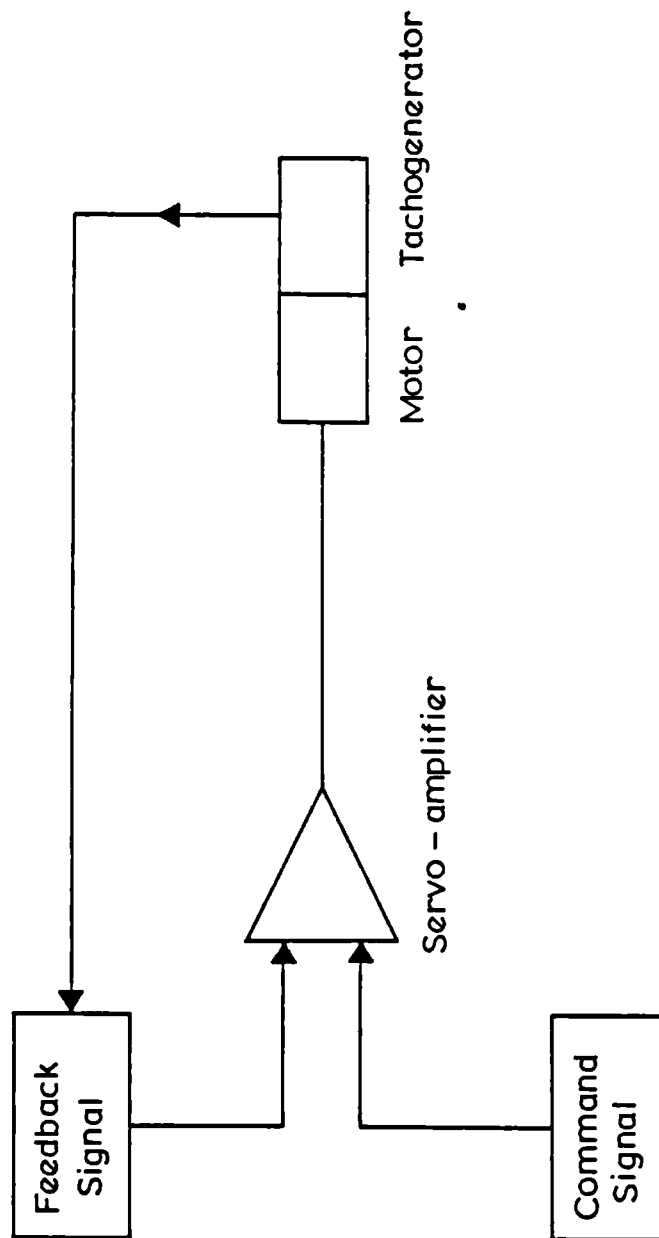


Fig. 5.1 Principles of analogue control system.

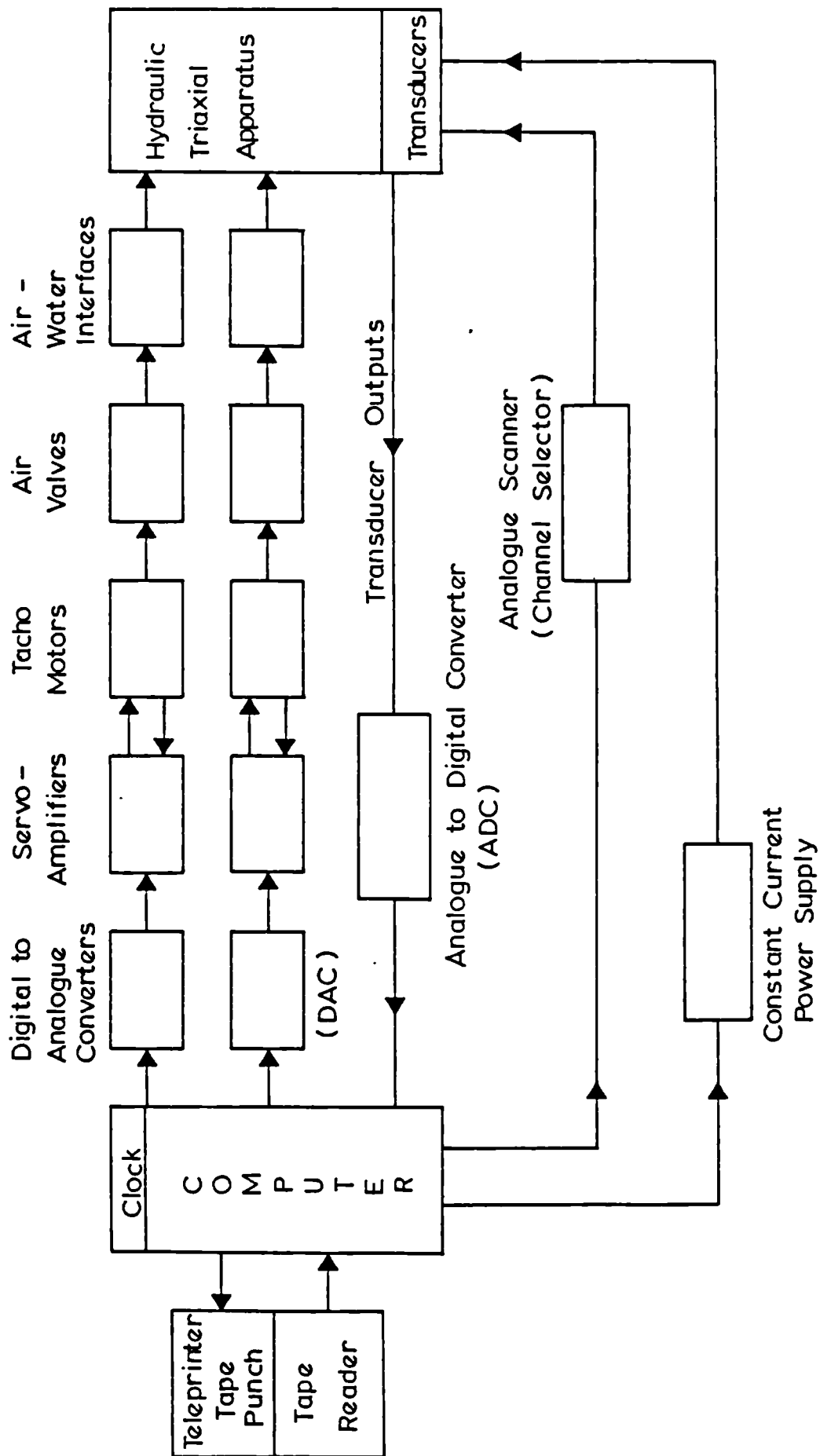


Fig. 5.2 Closed-loop computer control system.

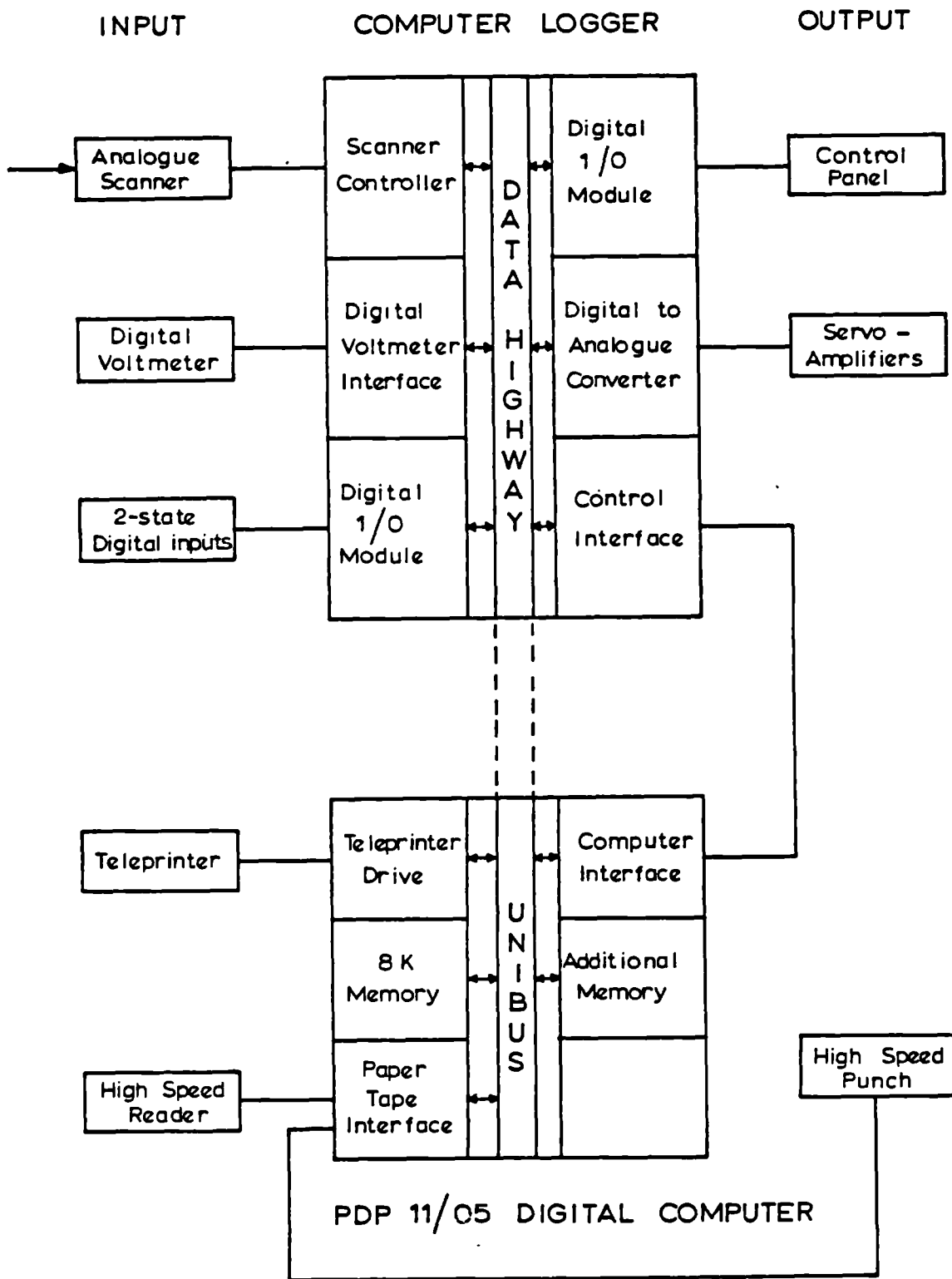


Fig. 5.3 Computer interface modules.

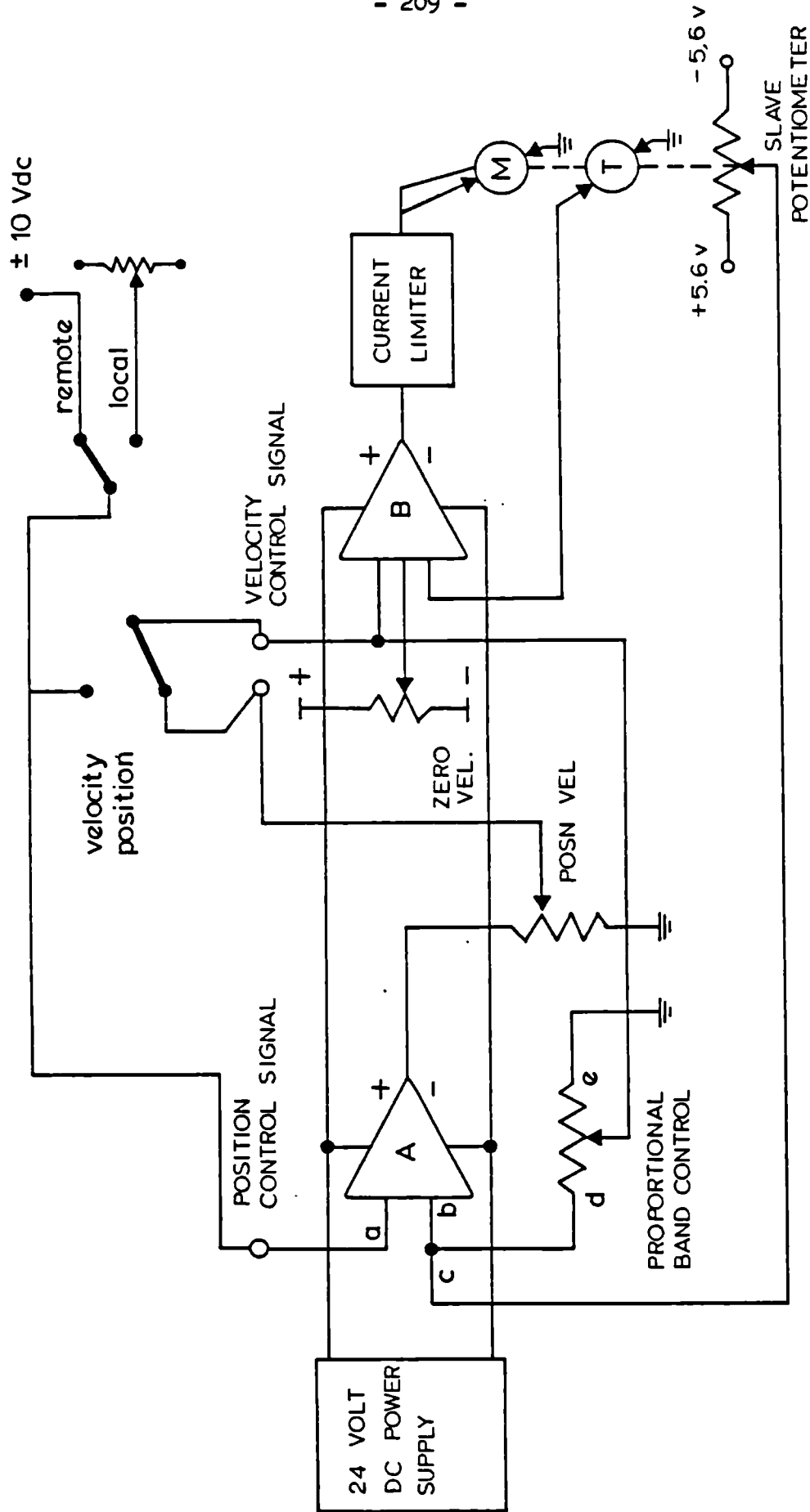
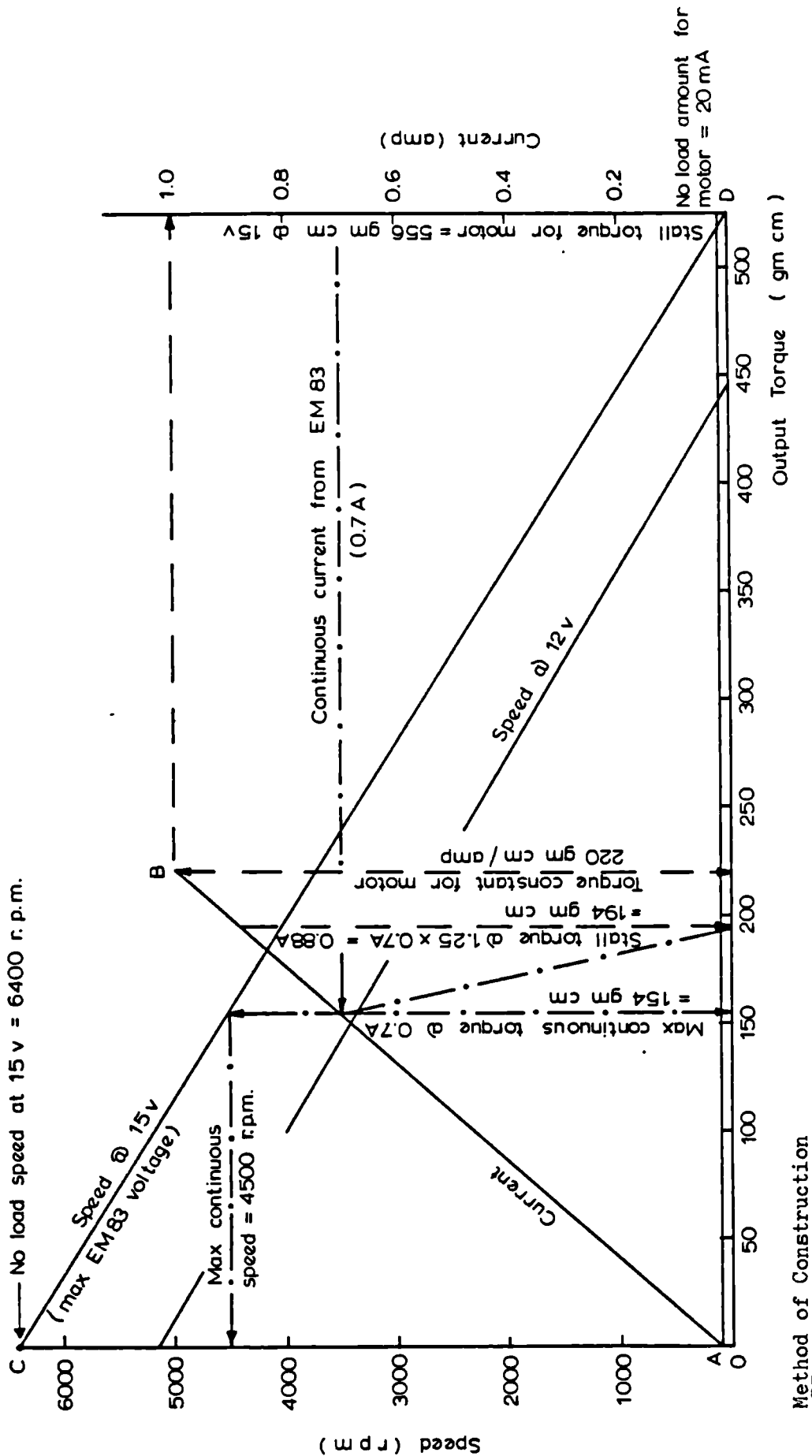


Fig. 5.4 Layout of servo-amplifiers for velocity and position control.



- 1 Establish current line using A - av. no-load current (20mA) and B - torque constant (220gm cm/A).
- 2 Establish speed line at max. voltage from amplifier (15v) using C - no-load speed @ 15v (6400 rpm) and D - stall torque at 15v (556gm cm). [Note speed and stall torque are directly $\propto$ to voltage.]
- 3 Enter max. continuous current from amplifier to find max. continuous torque and speed.

Fig. 5.5 Servo-system performance curves for EM 83 + 28 PLT motor.

1 oz in = 71.43 gmf cm

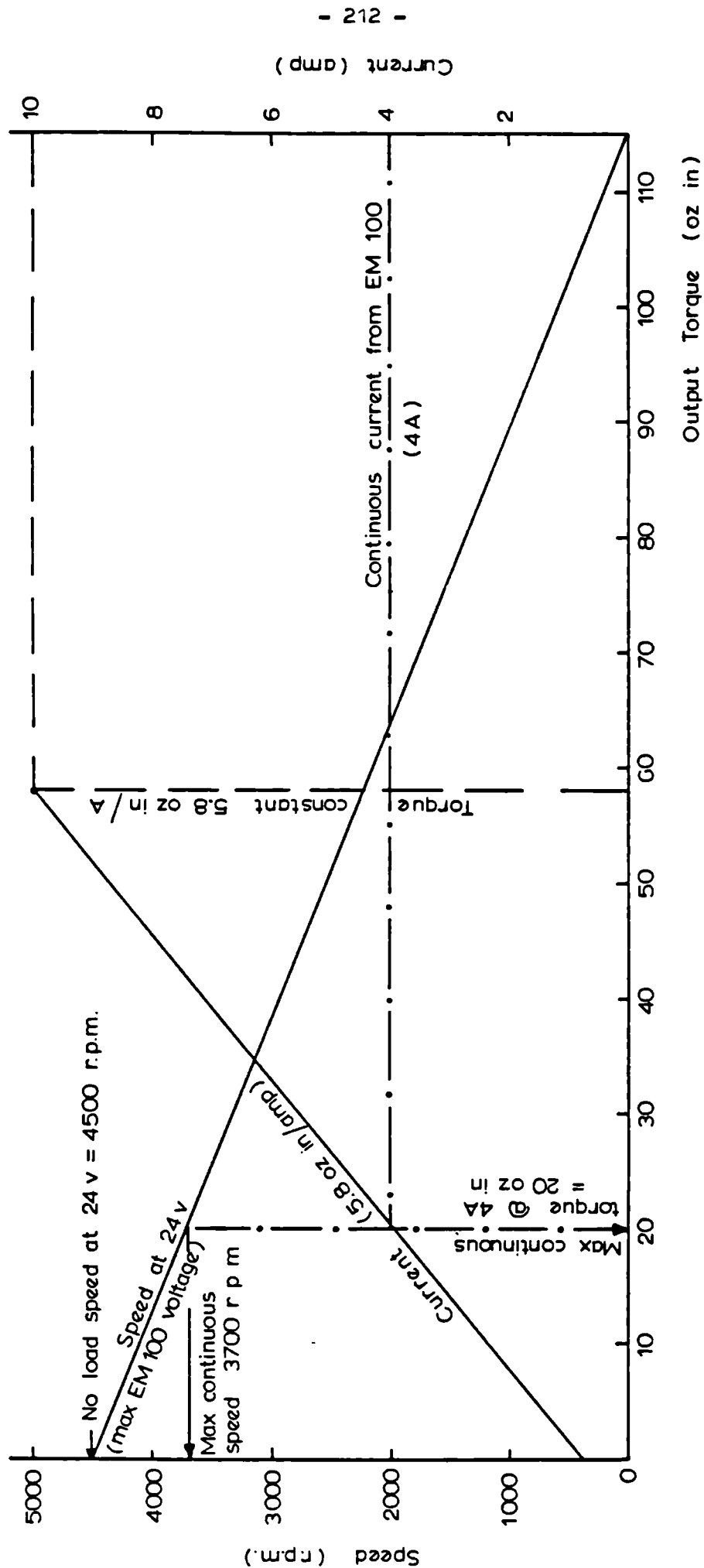


Fig. 5.7 Servo-system performance curves for EM 100 + G9M4T motor.

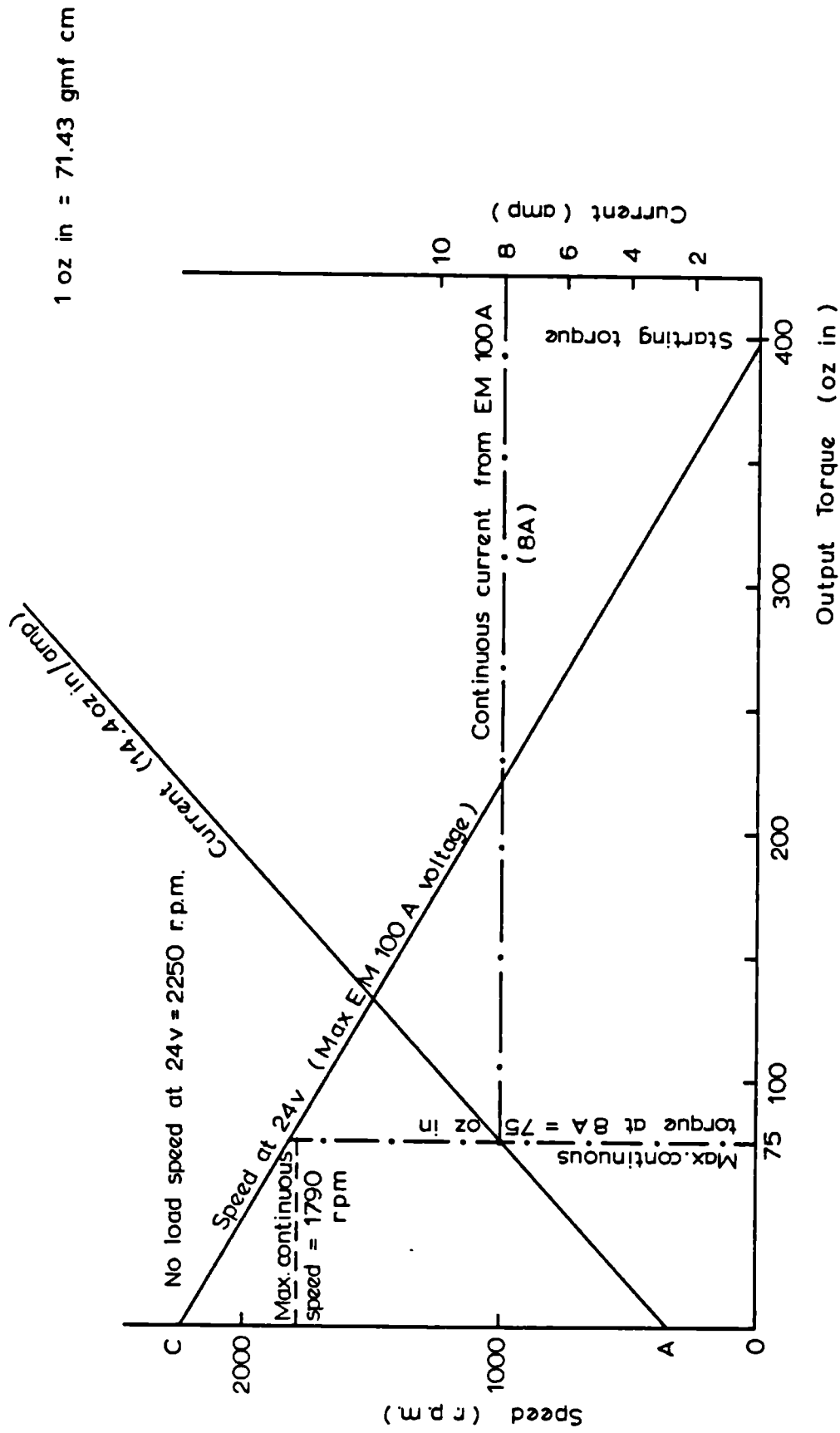


Fig. 5.8 Servo-system performance curves for EM 100A + G12M4T motor.

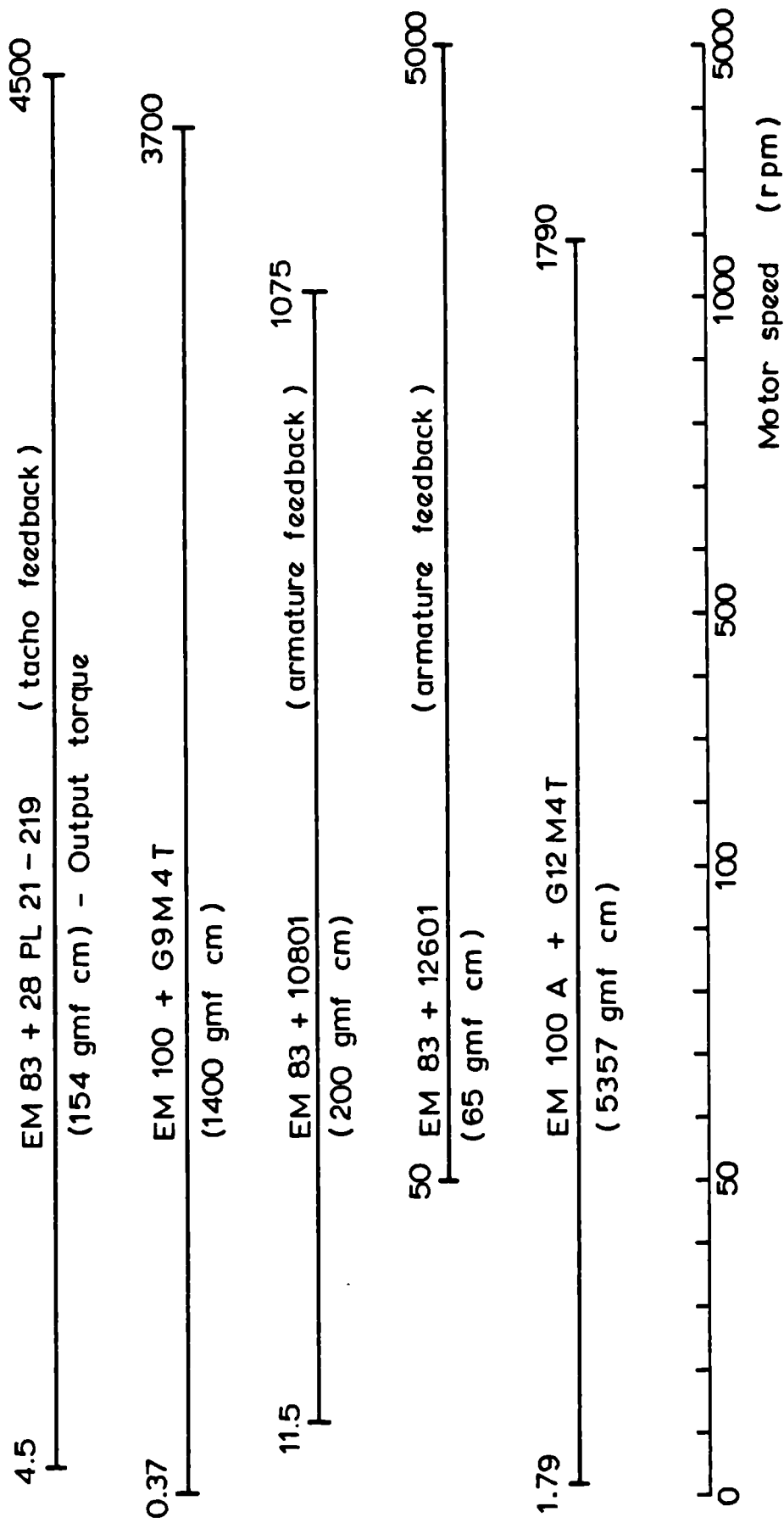
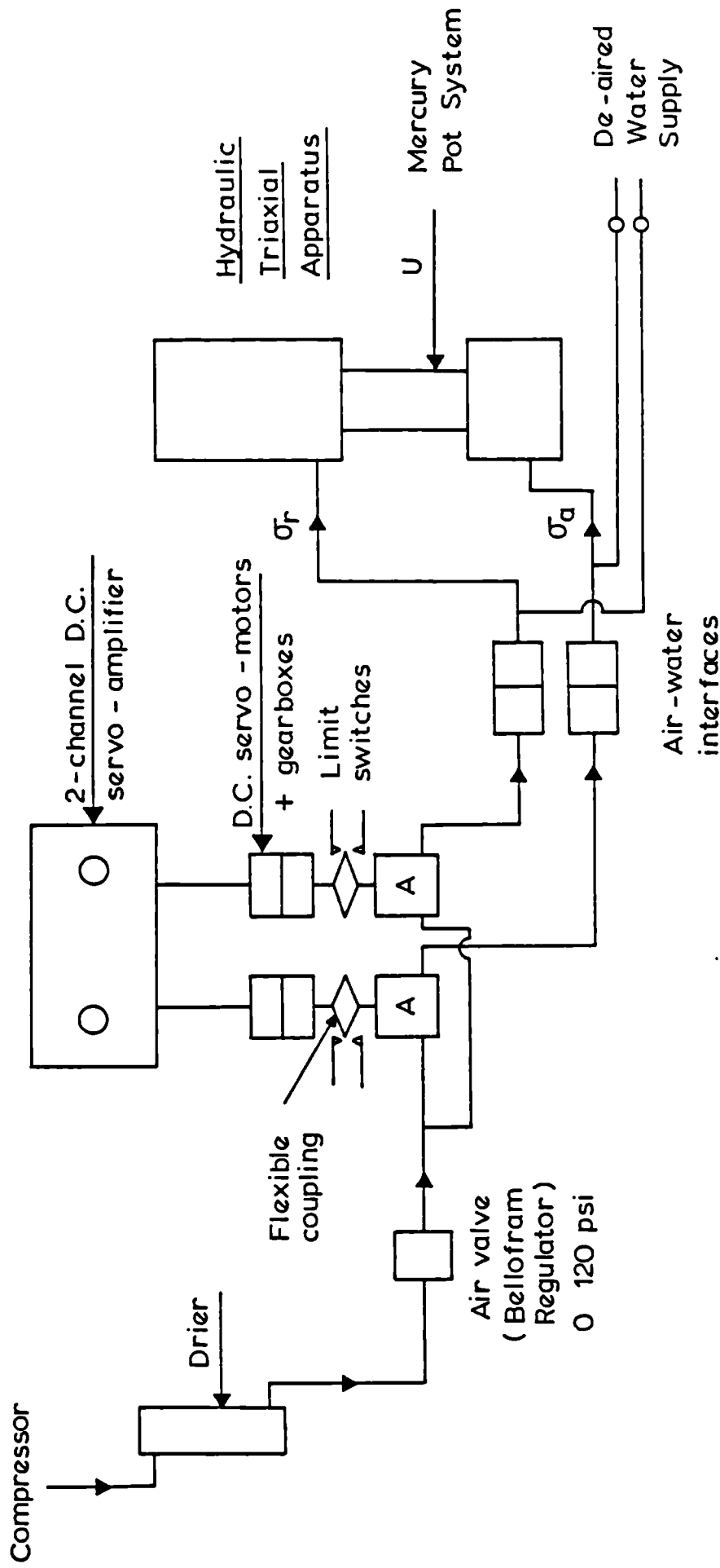


Fig. 5.9 Range of servo speeds available.



A - Precision Capsule Operated Pressure Regulator
'Electromanostat'

Fig. 5.10 Layout of air pressure system for pressure control.

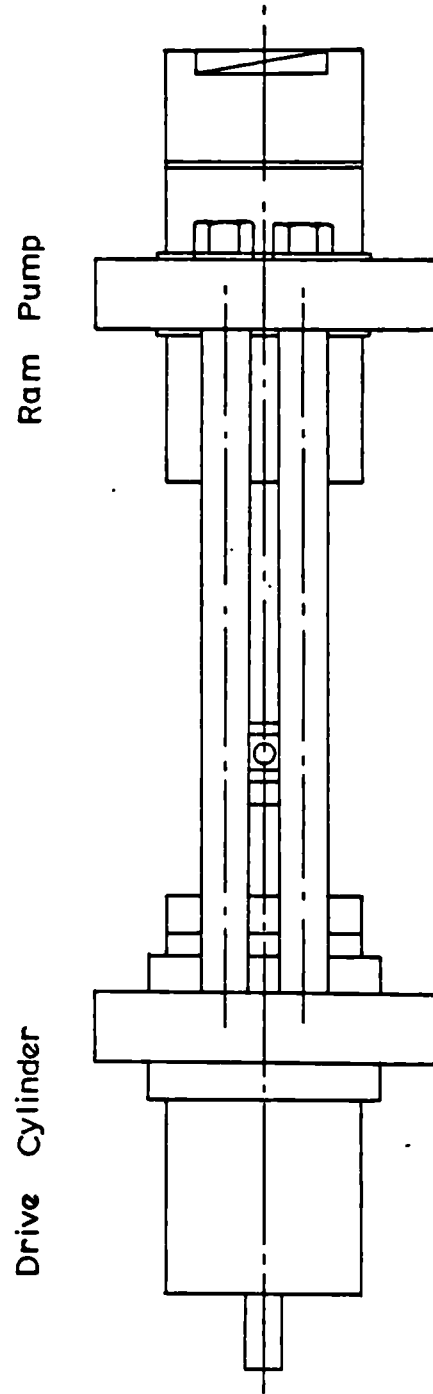
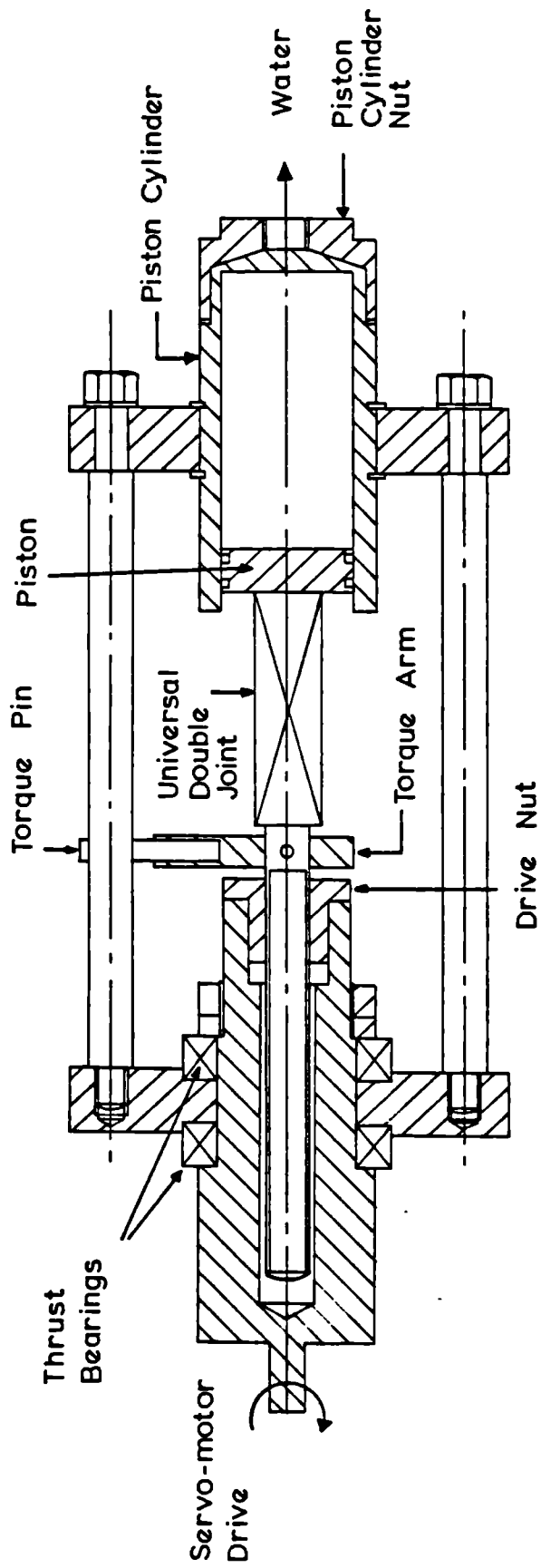


Fig. 5.11 Displacement regulator.

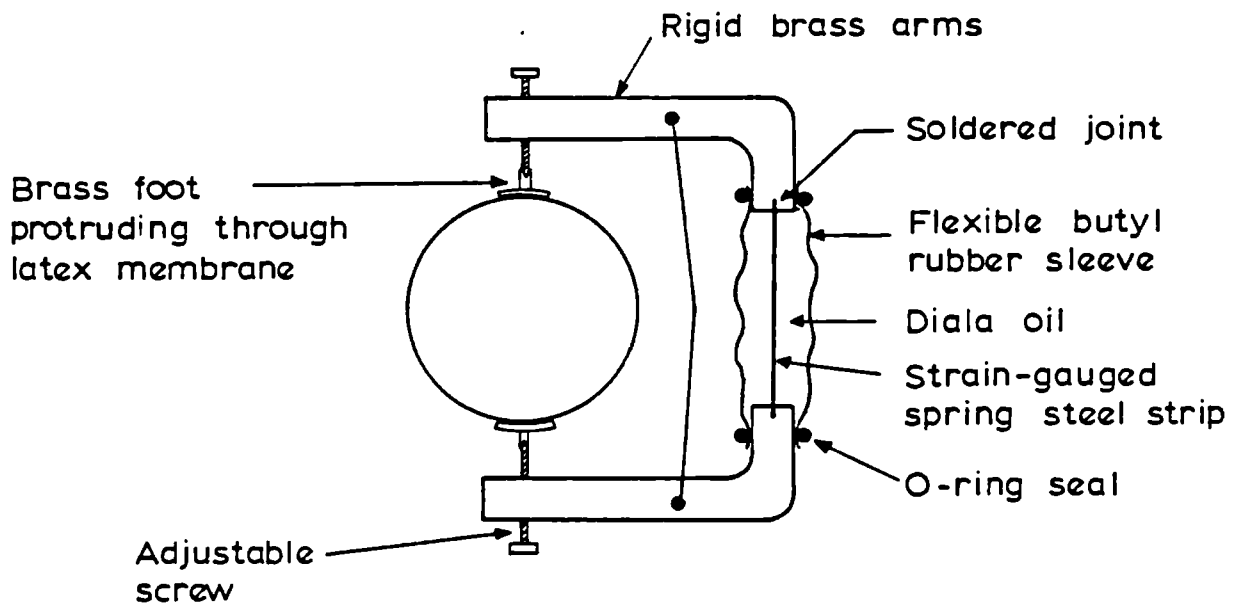
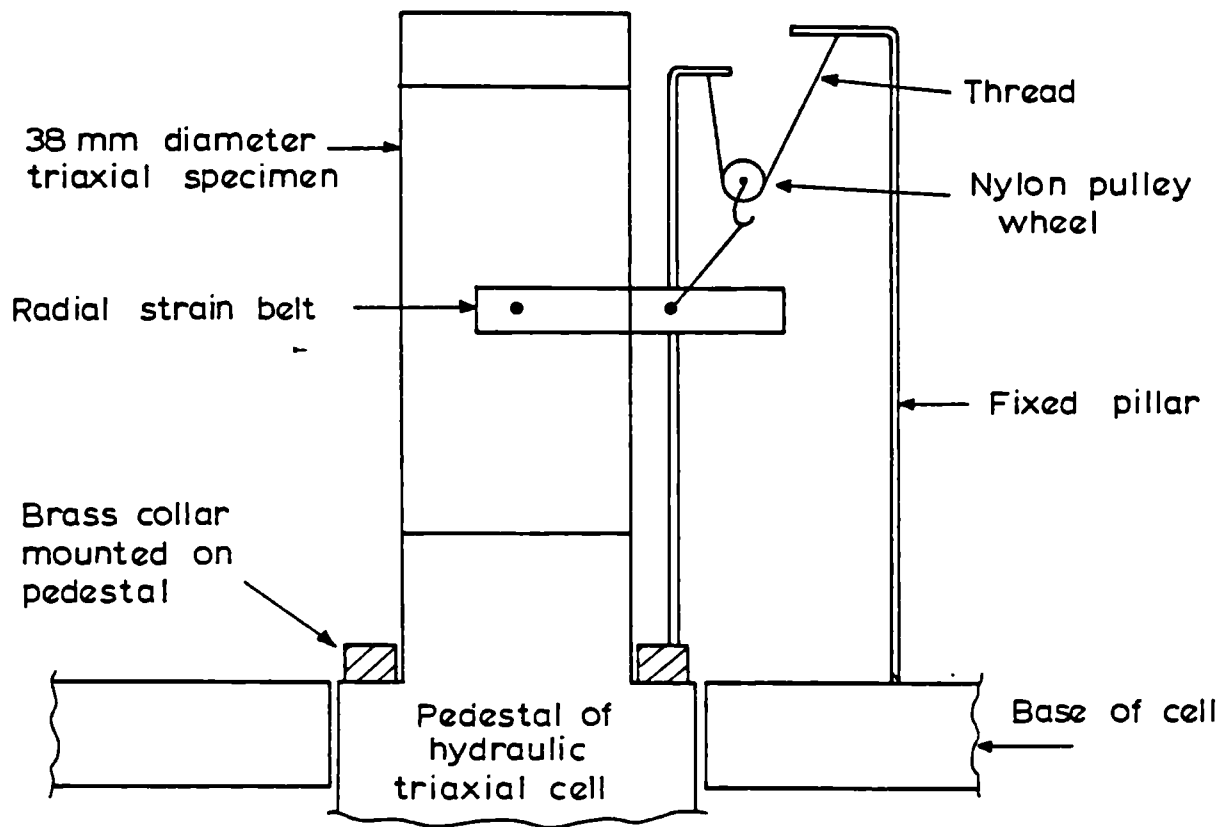


Fig. 5.12 Radial strain belt for 38mm diameter samples.

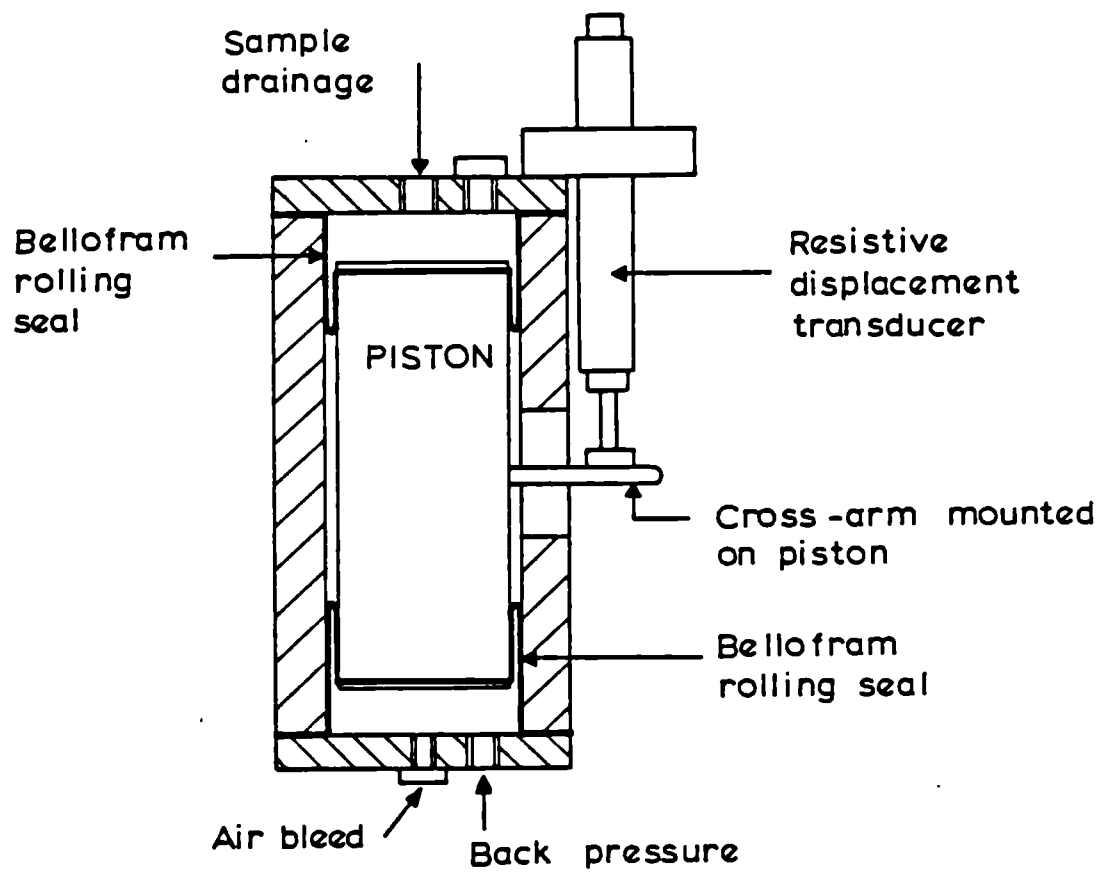


Fig. 5.13 Volume change transducer.

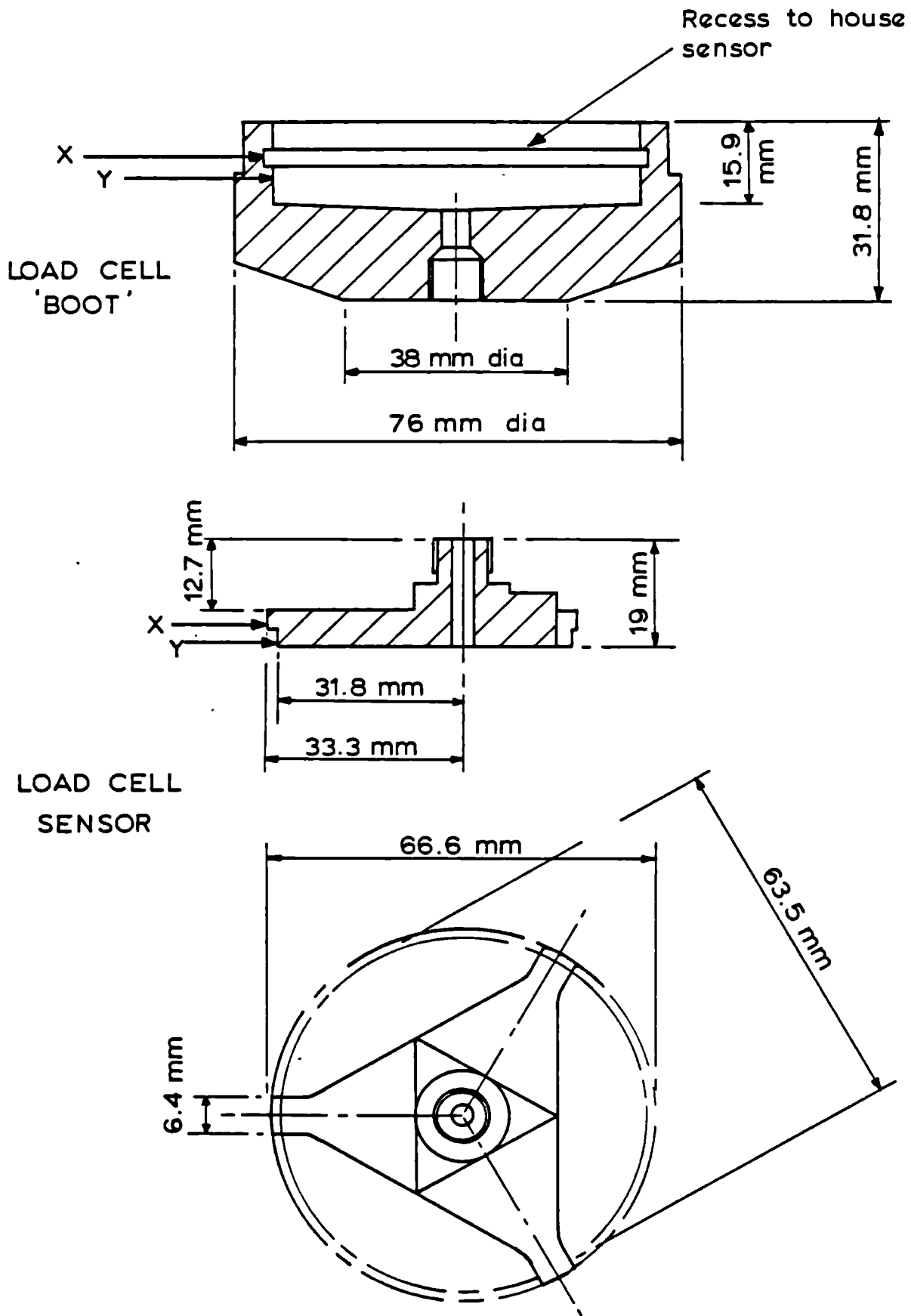


Fig. 5.14 Internal load cell.

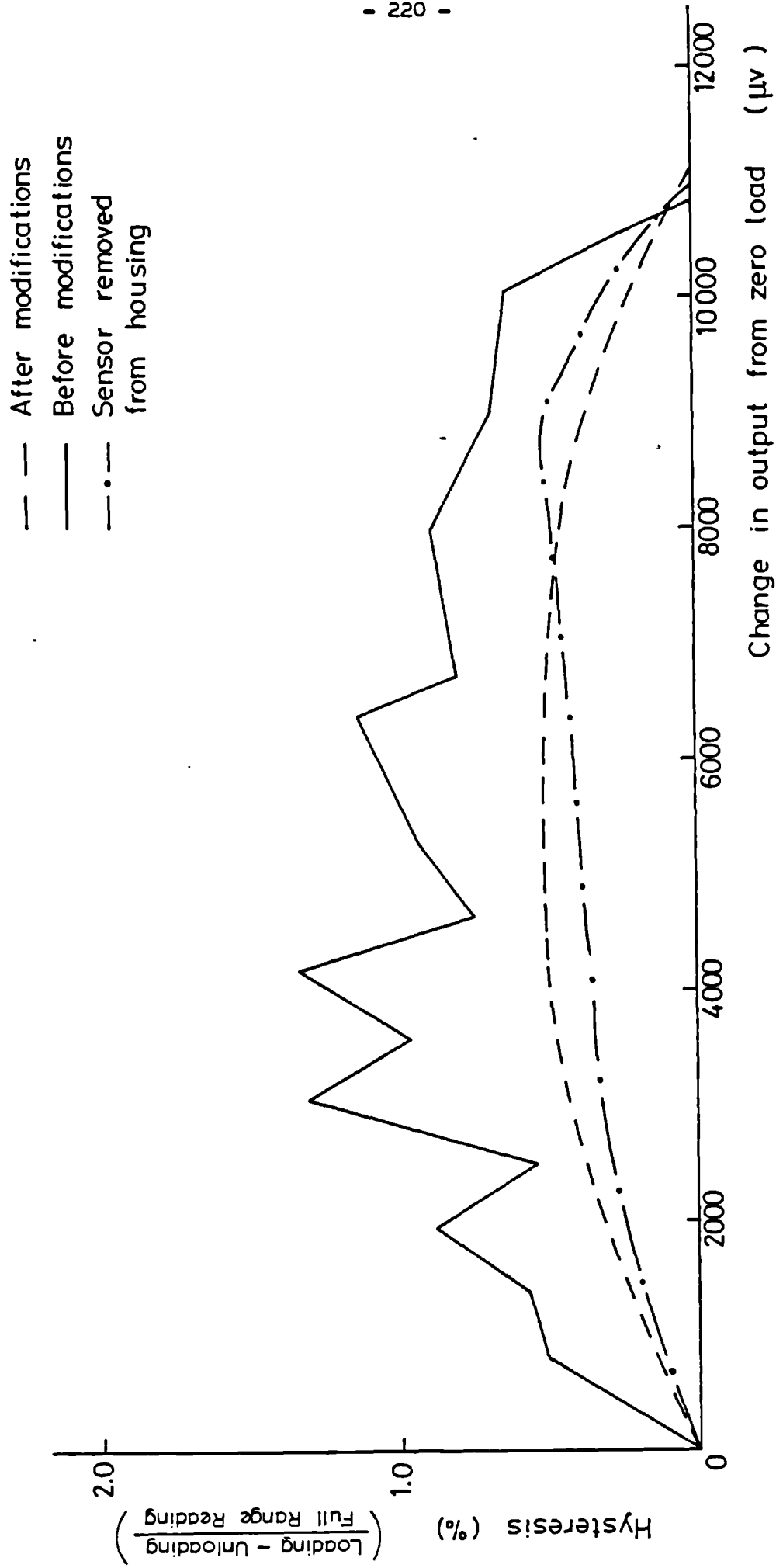


Fig. 5.15 Hysteresis in load cells, before and after modification.

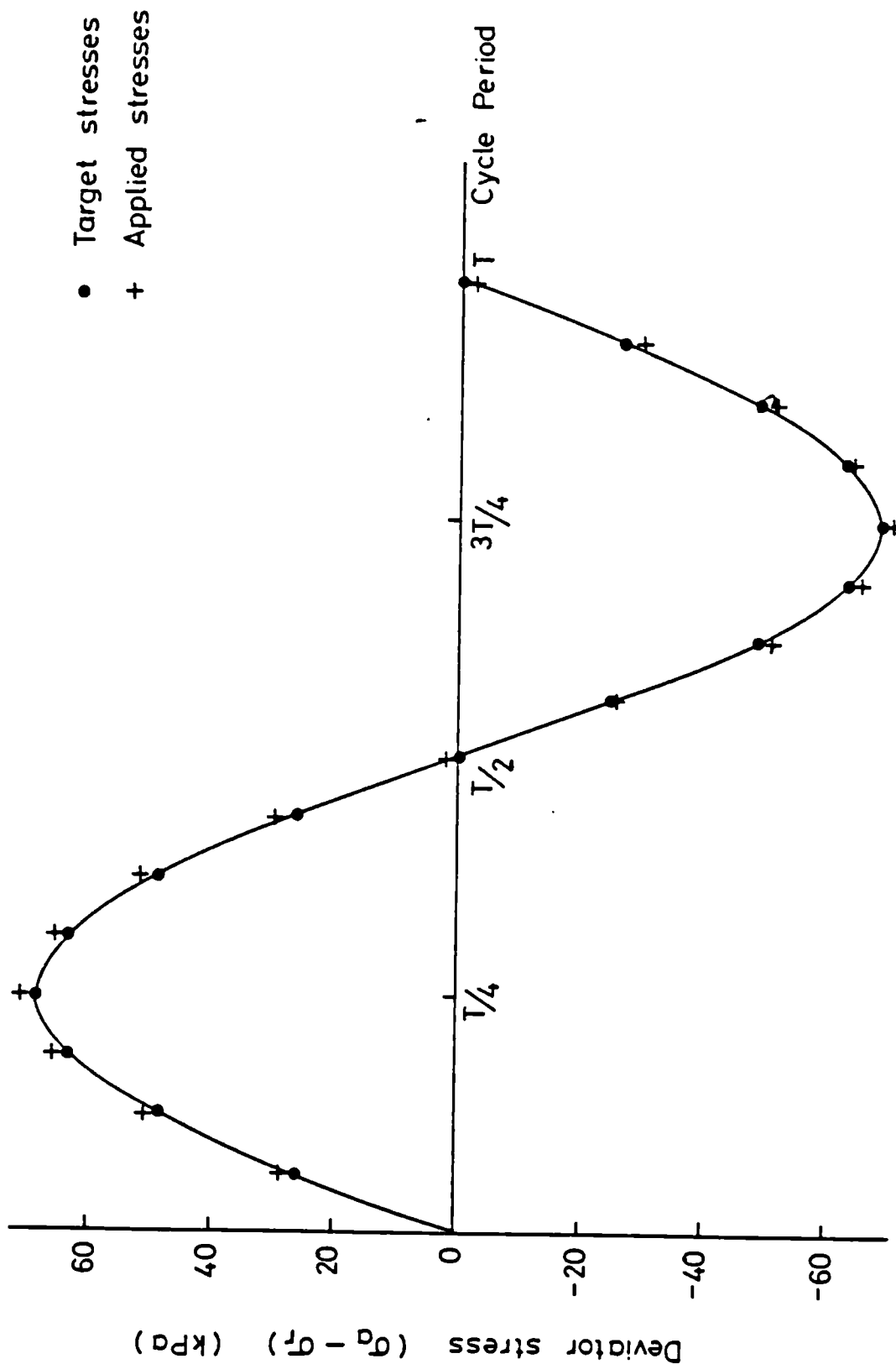


Fig. 5.16 Performance of computer control system : undrained cyclic compression-extension in the hydraulic triaxial cell.

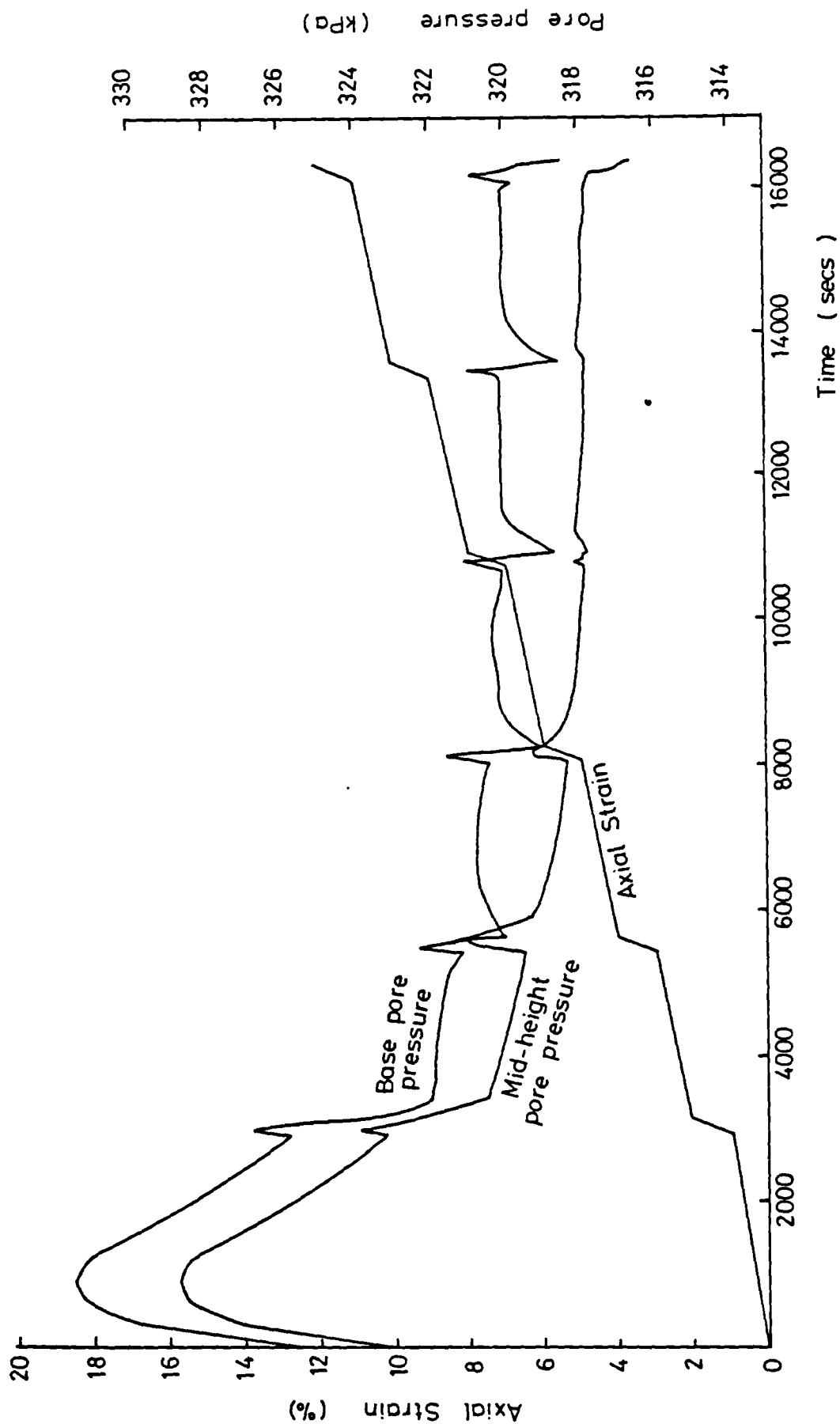


Fig. 5.17 Performance of computer control system : variable strain rate test.

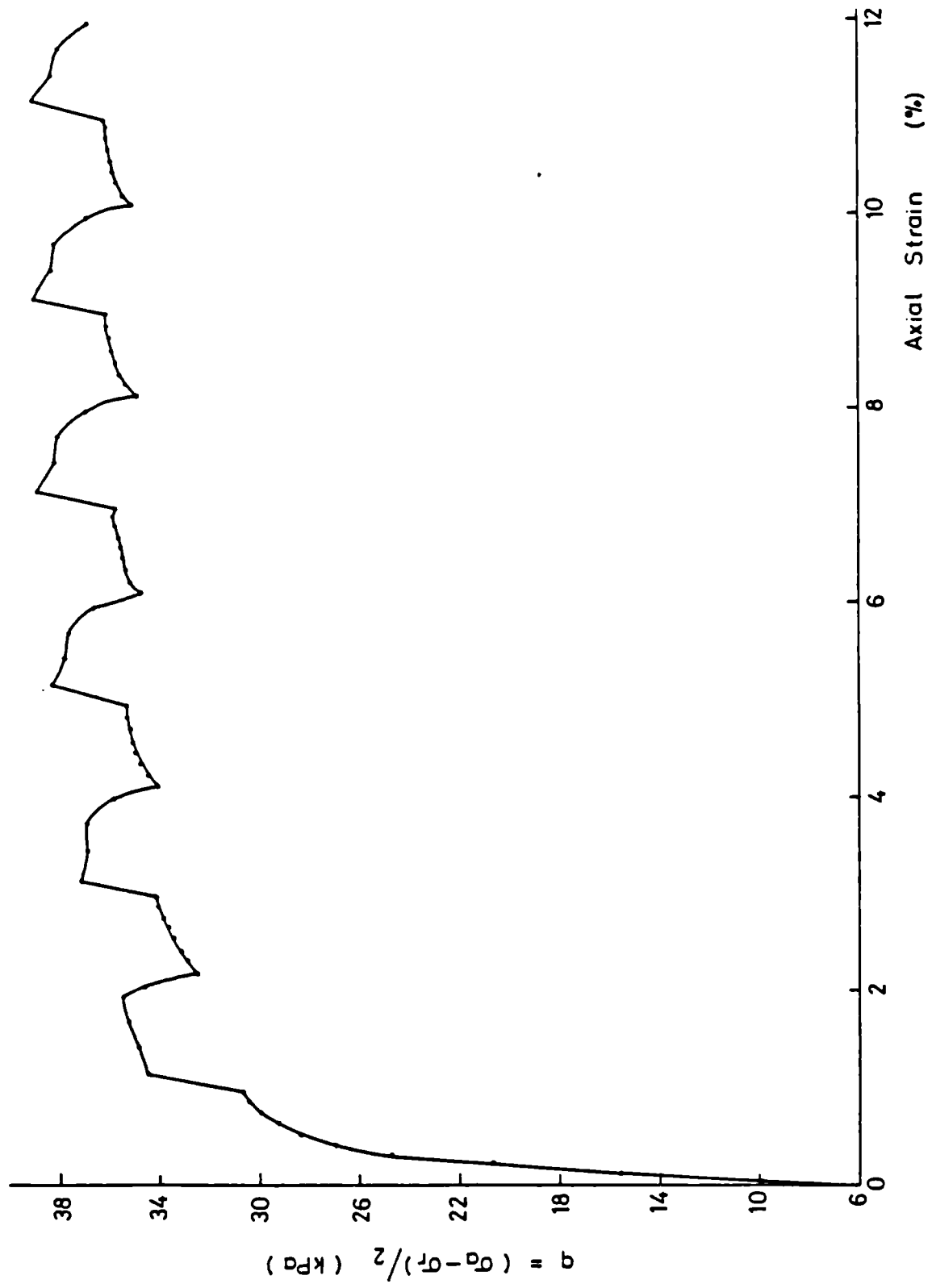


Fig. 5.18 Stress-strain curve observed in variable strain rate test.

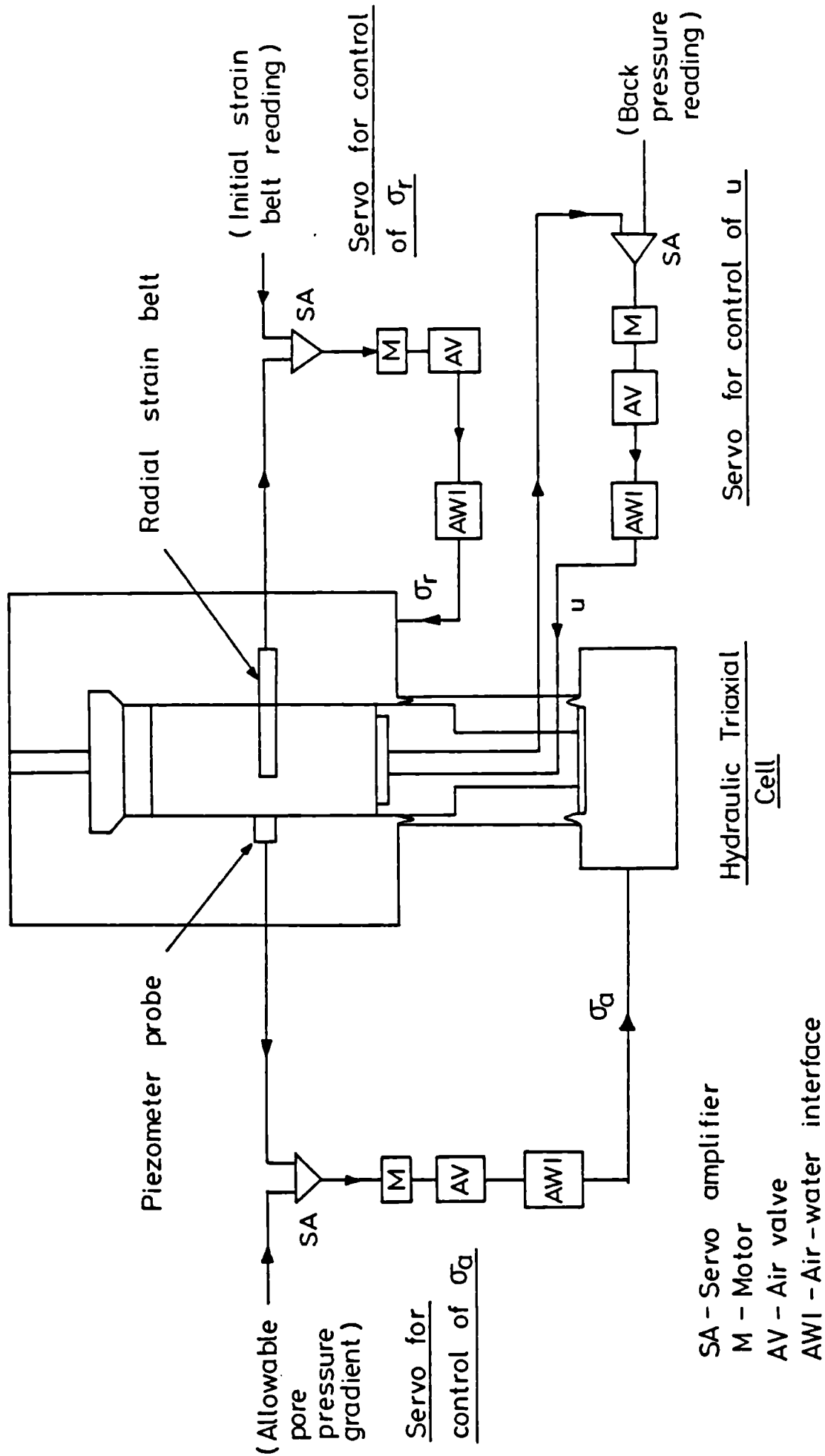


Fig. 5.19 Automatic control of K_0 consolidation and swelling.

APPENDIX 5.1

Algorithm and Control Program for Undrained Cyclic Loading in the Hydraulic Triaxial Cell

As an illustration of a typical control algorithm, this Appendix describes the algorithm for control of undrained cyclic loading in the hydraulic triaxial apparatus; it relies on the instrumentation described previously. The algorithm is drawn in Fig. 5. A1. 1. It enables a sinusoidally varying deviator stress to be applied to the sample. The essential steps in the algorithm are:-

- (1) All command signals are set to zero.
- (2) Data on transducer zeros, sample dimensions, test requirements (number of cycles, stress levels, frequency, number of target stresses to be set, etc.) are requested. (The program contains calibration factors, which may be non-linear, details of hysteresis in the transducers and other correction factors.)
- (3) The arrays to store data from the test are dimensioned.
- (4) The current stresses acting on the sample (i.e. after consolidation or after previous undrained shear) are established by measurement and calculation.
- (5) The first set of target stresses are calculated together with the voltage signals (V) required to be sent to the servo-amplifiers. (These signals are calculated on the basis of calibration constants pre-determined for each system.)
- (6) The signal voltages are set up, the computer clock set to zero and the test is started.
- (7) The clock is continuously interrogated until it is time to make the first comparison between the current stress state and the target stress state. In this interim period the cell

pressure in the hydraulic triaxial apparatus is being maintained constant by continuous adjustment of a pressure regulator.

- (8) At the pre-defined time (T2), all transducer readings are taken and stored. The current rate of change of deviator load with respect to time and the current rate of change of sample area with respect to deviator load (Q) are calculated. The point in the stress cycle is established.
- (9) The current stress state and the next target stress state are calculated and the new command signals are determined, either using initial calibration factors or updated versions of these.
- (10) The new command signals are set.
- (11) The routines are repeated until the test is complete, at which time all processed data is printed out. The end of the test may be determined by the number of cycles or some pre-defined limit on strain or pore pressure.

Within the control algorithm, subroutines are introduced to enable the friction which develops in the loading ram of the hydraulic triaxial apparatus to be overcome when loading directions are reversed. This is particularly important in slow cyclic tests.

For completeness a listing of this program together with a namelist are attached. In cyclic tests, the efficiency of the program, in terms of computer operating times, becomes important and much effort was devoted to optimising and timing the various routines.

The control system cannot be used to carry out cyclic loading at periods shorter than approximately 8 minutes; this restriction arises from the gearing in the drive between the motors and air valves, friction in the air-water interfaces, friction in the hydraulic triaxial cell and, to some extent, the compiler in the PDP-11 computer.

To assist in handling the large quantities of data generated in cyclic tests, computer plotting programs have been developed. Examples of two computer plots are presented in Figs 5. A1. 2 and 5. A1. 3.

CONTROL PROGRAM FOR UNDRAINED CYCLIC LOADING

LIST

```
1 FOR C=21 TO 25:LET V=DAS(1,C,3,2):OUT @0#9F6,V,:NEXT C
10 REM -INPUT DATA
20 LET C1=21:LET Z1=0.000281:LET I1=10180:LET E1=0:LET R1=12500
25 LET C2=22:LET Z2=0.000562:LET I2=9740:LET E2=0:LET R2=11500
30 LET C3=23:LET S3=23000:LET Z3=-0.003772
35 LET C4=24:LET S4=163.9
40 LET C5=25:LET Z5=0.035640:LET I5=495:LET E5=0.04:LET R5=35
45 LET O1=4:LET O2=5
50 LET A2=0.041:LET B2=0.000032
60 LET T1=960:LET T5=40
61 LET Q0=10:LET T0=3600:LET T3=150
65 LET Q1=16:LET N1=1
70 LET D0=1.48:LET H0=2.95
71 LET Z4=DAS(1,C4,3,2)
72 LET J1=T1/T5:LET V0=0.785*D0+2*H0
73 LET R=0
74 LET K1=0.00246:LET K2=0.000121:LET K3=0.0272
200 REM -DIMENSION ARRAYS
205 GOSUB 1000
206 REM -ESTABLISH BASE STRESSES
210 GOSUB 1500
215 REM -TAKE INITIAL READINGS
220 GOSUB 1800
300 REM
301 REM -START OF FIRST CYCLE
305 LET J=1:LET K=1:LET E=0:LET J2=0:LET T2=T5
306 LET Z=DAS(2,0,0,0,0)
307 LET W=J*6.28/J1:LET Q8=Q7+Q1*SIN(W)
308 IF J=J1/4+1 THEN 310
309 GOTO 315
310 GOSUB 2500
311 IF Q8>Q(J-1) THEN LET X2=DAS(8,02,10,0):GOTO 350
312 GOTO 340
315 IF J=3*J1/4+1 THEN 317
316 GOTO 340
317 GOSUB 3000
318 IF Q8<Q(J-1) THEN LET X2=DAS(8,02,10,0):GOTO 350
340 GOSUB 1900
350 GOSUB 2000
355 IF Q9<Q6 THEN GOSUB 3500
420 LET J=J+1:IF J>J1 THEN 486
421 GOTO 307
486 LET K=K+1
487 IF (K+10*E)>N1 THEN 800
488 LET A(0)=A(J):LET L(0)=L(J):LET Q(0)=Q(J):LET J=1
489 IF K>10 THEN 495
490 GOTO 307
495 IF E=0 THEN 600
500 GOTO 700
599 REM
600 REM -OUTPUT RESULTS AND CONTINUE
605 LET X2=DAS(8,02,10,0):LET X1=DAS(8,01,10,0)
610 GOSUB 4000
655 LET K=1:LET E=E+1:LET J=1:LET Z=DAS(2,0,0,0,0)
660 GOTO 307
```



```
700 REM -OUTPUT PEAK VALUES + 10*(E+1) CYCLE
705 LET X2=DAS(8,02,10,0):LET X1=DAS(8,01,10,0)
710 GOSUB 4500
715 LET K=1:LET E=E+1:LET J=1:LET Z=DAS(2,0,0,0,0)
720 GOTO 307
799 REM
800 REM -OUTPUT RESULTS AND STOP
805 LET X2=DAS(8,02,10,0):LET X1=DAS(8,01,10,0)
810 IF E>0 THEN 900
815 GOSUB 4000
819 OUT #9F6,P1,P2,Z4,*6F2,H1,↑
820 STOP
825 END
900 GOSUB 4500
904 OUT #9F6,P1,P2,Z4,*6F2,H1,↑
905 STOP
910 END
1000 REM-DIMENSION ARRAYS
1005 DIM G1(50,10),G2(50,10),G3(50,10),G4(50,10),G5(50,10),G6(50,10)
1010 DIM Y(50),Q(50),N(50),A(50),U(50),L(50)
1100 RETURN
1500 REM -ESTABLISH BASE STRESSES
1515 GOSUB 1800
1520 LET K=1:LET J=1:LET J2=T0/T3:LET I=1:LET T2=T3
1522 LET Z=DAS(2,0,0,0,0)
1525 LET Q8=Q7+Q0*J/J2
1530 GOSUB 1900
1535 GOSUB 2000
1536 IF Q9<Q6 THEN GOSUB 3500
1540 LET J=J+1:IF J>J2 THEN 1559
1555 GOTO 1525
1559 LET X2=DAS(8,02,10,0):LET X1=DAS(8,01,10,0)
1560 OUT #0↑,"UNDRAINED APPLICATION OF DEVIATOR STRESS",↑,↑
1565 GOTO 4015
1571 FOR J=1 TO J2
1575 GOSUB 5000
1580 NEXT J
1581 REM -MONITORING PORE PRESSURES
1585 LET Z=DAS(3,D,H,M,S):LET V2=DAS(1,C2,3,2):LET V5=DAS(1,C5,3,2)
1590 LET T4=(H*60+M)*60+S:LET T=T4
1591 LET Z=DAS(2,0,0,0,0):LET M1=5
1595 OUT #5X,"TIME(SECS)",#10X,"PORE PRESSURE(KN/M2)",↑
1596 OUT #27X,"BASE PROBE",↑
1597 LET S2=I2-R2*(V2-E2)
1598 LET S5=I5-R5*(V5-E5)
1600 OUT #5X,#71,T,#13X,#7F2,(V2-Z2)*S2*6.895,#1X
1601 OUT #7F2,(V5-Z5)*S5*6.895,↑
1602 IF M1>60 THEN RETURN
1605 LET Z=DAS(3,D,H,M,S)
1610 IF (M+H*60)<M1 THEN 1605
1611 LET V3=DAS(1,C3,3,2):LET D3=(V3-Z3)*S3*0.004448
1620 LET V2=DAS(1,C2,3,2):LET V5=DAS(1,C5,3,2):LET M1=M1+5
1621 LET T=(H*60+M)*60+T4
1622 LET V4=DAS(1,C4,3,2):LET D4=(V4-Z4)*S4-K1-K3*D3
1623 LET A4=0.00064516*V0/(H0-D4):LET H8=D3/(A4*2)
1624 IF (H8-H1)>0.5 THEN GOSUB 6000
1625 IF (H8-H1)<-0.5 THEN GOSUB 6500
1626 GOTO 1597
```

```

1800 REM -TAKE INITIAL READINGS
1805 LET G1(0,0)=DAS(1,C1,3,2):LET G2(0,0)=DAS(1,C2,3,2)
1810 LET G3(0,0)=DAS(1,C3,3,2):LET G4(0,0)=DAS(1,C4,3,2)
1811 LET G5(0,0)=DAS(1,C5,3,2)
1815 LET P1=G1(0,0)+0.00001:LET P2=P1-0.00002
1816 LET S1=I1-R1*(G1(0,0)-E1):LET S2=I2-R2*(G2(0,0)-E2)
1817 LET S5=I5-R5*(G5(0,0)-E5)
1820 LET P0=4.56*S1*(G1(0,0)-Z1)
1825 LET L(0)=(G3(0,0)-Z3)*S3
1826 LET H7=H0-(G4(0,0)-Z4)*S4
1827 IF L(0)>0 THEN LET H7=H7-K1-K2*L(0)
1830 LET A(0)=V0/H7
1835 LET Q(0)=L(0)/A(0):LET Q7=Q(0):LET A7=A(0)
1840 RETURN
1900 REM -CALCULATE AND SET VOLTAGE
1905 LET Q4=Q8-Q(J-1)
1910 LET N(J)=Q(J-1)+0.5*Q4:LET A(J)=A(J-1)+R*Q4/2
1915 LET U(J)=N(J)*A(J)+P0:LET Y(J)=A2+B2*U(J)
1920 LET U2=(Q4*(A(J-1)+R*Q4))/(T2*Y(J))
1925 LET X2=DAS(8,02,10,U2)
1930 RETURN
2000 REM-CLOCK,MEASURE+MAINTAIN CONSTANT CELL PRESSURE ROUTINE
2010 LET P3=DAS(1,C1,3,2):IF P3>P1 THEN 2025
2015 IF P3<P2 THEN 2030
2020 LET X1=DAS(8,01,10,0):GOTO 2035
2025 LET X1=DAS(8,01,10,-5):GOTO 2035
2030 LET X1=DAS(8,01,10,5)
2035 LET Z=DAS(3,D,H,M,S):LET T=(60*(24*D+H)+M)*60+S
2040 IF T<((K-1)*J1+J)*T2 THEN 2010
2045 LET G1(J,K)=DAS(1,C1,3,2):LET G2(J,K)=DAS(1,C2,3,2)
2050 LET G3(J,K)=DAS(1,C3,3,2):LET G4(J,K)=DAS(1,C4,3,2)
2060 LET G5(J,K)=DAS(1,C5,3,2):LET G6(J,K)=T
2075 LET L(J)=(G3(J,K)-Z3)*S3
2080 LET A(J)=V0/(H0-(G4(J,K)-Z4)*S4)
2085 LET Q(J)=L(J)/A(J):LET Q9=Q(J)-Q(J-1):LET Q9=ABS(Q9)
2086 LET Q6=ABS(Q4):LET Q6=0.2*Q6
2095 LET U(J)=(L(J)+L(J-1))/2+P0
2098 IF A(J)=A(J-1) THEN 2110
2100 LET R=(A(J)-A(J-1))/(Q(J)-Q(J-1))
2105 RETURN
2110 LET R=0:RETURN
2500 REM -REVERSE FRICTION
2505 LET F0=DAS(1,23,3,2)
2510 LET X2=DAS(8,02,10,-10):LET X1=DAS(8,01,10,0)
2511 LET P3=DAS(1,C1,3,2):IF P3>P1 THEN 2515
2512 IF P3<P2 THEN 2516
2513 LET X1=DAS(8,01,10,0):GOTO 2520
2515 LET X1=DAS(8,01,10,-5):GOTO 2520
2516 LET X1=DAS(8,01,10,5)
2520 LET F1=DAS(1,23,3,2)
2525 IF (F0-F1)>0.000005 THEN 2535
2530 GOTO 2511
2535 LET X2=DAS(8,02,10,0):LET X1=DAS(8,01,10,0)
2550 RETURN

```

```

3000 REM -REVERSE FRICTION
3005 LET F0=DAS(1,23,3,2)
3010 LET X2=DAS(8,02,10,10):LET X1=DAS(8,01,10,0)
3011 LET P3=DAS(1,C1,3,2):IF P3>P1 THEN 3015
3012 IF P3<P2 THEN 3016
3013 LET X1=DAS(8,01,10,0):GOTO 3019
3015 LET X1=DAS(8,01,10,-5):GOTO 3019
3016 LET X1=DAS(8,01,10,5)
3019 LET F1=DAS(1,23,3,2)
3020 IF (F1-F0)>0.000005 THEN 3030
3025 GOTO 3011
3030 LET X2=DAS(8,02,10,0):LET X1=DAS(8,01,10,0)
3045 RETURN
3500 REM -FRICTION BOOST
3505 IF Q4>0 THEN 3520
3510 GOSUB 2500
3515 RETURN
3520 GOSUB 3000
3525 RETURN
4000 REM -OUTPUT DATA
4005 OUT @0+,+
4006 LET J2=0
4015 OUT @0"INITIAL SAMPLE AREA:      ",#8F2,A7*645.16,"MM2",+
4017 OUT @0"INITIAL SAMPLE HEIGHT:    ",#6F2,H7*25.4,"MM",+
4020 OUT @0"BASE CELL PRESSURE:        ",#7F2,P0*1.512,"KN/M2",+
4025 OUT @0"BASE DEVIATOR STRESS:      ",#7F2,Q7*6.895,"KN/M2",+
4026 LET P4=(G2(0,0)-Z2)*S2*6.895:LET P5=(G5(0,0)-Z5)*S5*6.895
4027 OUT @0"BASE PORE PRESSURE:        ",#7F2,P4,"KN/M2",+
4028 OUT @0"PROBE PORE PRESSURE:       ",#7F2,P5,"KN/M2",+
4029 OUT @2#8F2,A7*645.16,#6F2,H7*25.4,#7F2,P0*1.512,Q7*6.895,P4,P5,+
4030 OUT @0" TIME RADIAL AXIAL      BASE  MID HT      Q      P"
4031 OUT @0"      P'    AXIAL    RADIAL",+
4035 OUT @0"      STRESS STRESS    PWP      PWP      "
4036 OUT @0" STRAIN  STRAIN",+
4040 OUT @0" SECS KN/M2  KN/M2    KN/M2    KN/M2    KN/M2  KN/M2"
4041 OUT @0"  KN/M2    %      %",+
4042 OUT @0+
4043 IF J2>1 THEN 1571
4045 FOR I=1 TO (K-1)
4050 OUT @0"CYCLE NO:","#5I,I+E*10,+
4051 OUT @2#5I,I+E*10,+
4055 FOR J=1 TO J1
4056 GOSUB 5000
4060 NEXT J
4061 OUT @0+
4065 NEXT I
4070 RETURN
4500 REM -OUTPUT PEAK VALUES
4505 FOR I=1 TO (K-2)
4510 OUT @0"CYCLE NO:","#5I,I+E*10,+
4511 OUT @2#5I,I+E*10,+
4515 FOR J=J1/4 TO J1 STEP J1/4
4520 GOSUB 5000
4525 NEXT J
4530 NEXT I
4535 LET I=K-1:OUT @0+
4540 OUT @0"CYCLE NO:","#5I,I+E*10,+
4541 OUT @2#5I,I+E*10,+
4545 FOR J=1 TO J1
4546 GOSUB 5000
4550 NEXT J
4560 RETURN

```

```
5000 REM -CALCULATE OUTPUT DATA
5001 LET S1=I1-R1*(G1(J,I)-E1)
5002 LET S2=I2-R2*(G2(J,I)-E2)
5003 LET S5=I5-R5*(G5(J,I)-E5)
5005 LET D1=(G1(J,I)-Z1)*S1*6.895:LET D2=(G2(J,I)-Z2)*S2*6.895
5010 LET D3=(G3(J,I)-Z3)*S3*0.004448:LET D4=(G4(J,I)-Z4)*S4
5011 LET D5=(G5(J,I)-Z5)*S5*6.895
5012 IF D3>0 THEN LET D4=D4-K1-K3*D3
5020 LET A4=0.00064516*V0/(H0-D4):LET H1=D3/(A4*2):LET H2=2*H1+D1
5025 LET H3=(D1+H2)/2:LET H4=H3-D5
5026 LET H5=D4*100/H0
5030 OUT @0#*5I,G6(J,I),#1X,**6F2,D1,#1X,**6F2,H2,#1X,#7F2,D2,#1X
5031 OUT @2#*5I,G6(J,I),**6F2,D1,**6F2,H2,#7F2,D2,†
5035 OUT @0#7F2,D5,#1X,**6F2,H1,#1X,**6F2,H3,#1X,**6F2,H4,#1X,#7F3,H5,
5036OUT @2#7F2,D5,#7F3,H5,†
5045 RETURN
6000 REM -REDUCE DEVIATOR STRESS TO REQUIRED LEVEL
6005 LET X2=DAS(8,02,10,-1)
6006 LET V3=DAS(1,C3,3,2):LET D3=(V3-Z3)*S3*0.004448
6007 LET P3=DAS(1,C1,3,2):IF P3>P1 THEN 6010
6008 IF P3<P2 THEN 6011
6009 LET X1=DAS(8,01,10,0):GOTO 6012
6010 LET X1=DAS(8,01,10,-5):GOTO 6012
6011 LET X1=DAS(8,01,10,5)
6012 LET V4=DAS(1,C4,3,2):LET D4=(V4-Z4)*S4-K1-K3*D3
6015 LET A4=0.00064516*V0/(H0-D4):LET H8=D3/(A4*2)
6020 IF H8<H1 THEN 6030
6025 GOTO 6006
6030 LET X2=DAS(8,02,10,0):LET X1=DAS(8,01,10,0)
6031 LET V1=DAS(1,C1,3,2):LET D1=(V1-Z1)*S1*6.895
6035 OUT @0"SHEAR STRESS:",**6F2,H8,#5X,"CELL PRESSURE:"
6036 OUT @0#*6F2,D1,†
6040 RETURN
6500 REM -INCREASE DEVIATOR STRESS TO REQUIRED LEVEL
6505 LET X2=DAS(8,02,10,1)
6506 LET V3=DAS(1,C3,3,2):LET D3=(V3-Z3)*S3*0.004448
6507 LET P3=DAS(1,C1,3,2):IF P3>P1 THEN 6510
6508 IF P3<P2 THEN 6511
6509 LET X1=DAS(8,01,10,0):GOTO 6512
6510 LET X1=DAS(8,01,10,-5):GOTO 6512
6511 LET X1=DAS(8,01,10,5)
6512 LET V4=DAS(1,C4,3,2):LET D4=(V4-Z4)*S4-K1-K3*D3
6515 LET A4=0.00064516*V0/(H0-D4):LET H8=D3/(A4*2)
6520 IF H8>H1 THEN 6530
6525 GOTO 6506
6530 LET X2=DAS(8,02,10,0):LET X1=DAS(8,01,10,0)
6531 LET V1=DAS(1,C1,3,2):LET D1=(V1-Z1)*S1*6.895
6535 OUT @0"SHEAR STRESS:",**6F2,H8,#5X,"CELL PRESSURE:"
6536 OUT @0#*6F2,D1,†
6540 RETURN
```

READY

VARIABLES LIST

A	A(50)	Array for areas
	A($\emptyset$)=A7	Sample area prior to shear after consolidation
	A2	Intercept on air valve calibration plot
	A4	Variable used in calculation of output data
B	B2	Slope of air valve calibration plot
C	C1	Channel no. for cell pressure
	C2	Channel no. for pore pressure
	C3	Channel no. for deviator load
	C4	Channel no. for axial displacement
	C5	Channel no. for probe pwp
	C6	Channel no. for radial displacement
D	D	Days
	D $\emptyset$	Initial sample diameter (inches)
	D1-D5, D8	Variables used in calculation of output data
E	E	Counter
	E1, E2, E5	Calculation of calibration factors for pressure transducers
F	F $\emptyset$	Load cell reading
	F1	Load cell reading } Reversing friction
G	G1(50,10)-G5(50,10)	Arrays for storing transducer readings on channels C1-C5
	G6(50)	Array for storing times
H	H1-H5	Variables used in calculation of output data
	H	Hours
	H $\emptyset$	Initial sample height (inches)
	H7, H8	Sample height before shear, after consolidation
I	I	Counter
	I1, I2, I5	Calculation of calibration factors for pressure transducers

J	J	Counter (adjustments)
	J1	No. of adjustments per cycle (T1/T2)
	J2	No. of adjustments during undrained application of deviator stress (T $\emptyset$ /T2)
K	K	Counter (cycles)
	K1, K2, K3	Load cell deformation
L	L(50)	Array for deviator loads
	L($\emptyset$)	Initial load on sample
M	M	Minutes
	M1	Interval (minutes) between monitoring of pwps
N	N(50)	Array for mean deviator stress during step
	N1	Number of cycles required
O	O1 (4)	Output channel for cell pressure control
	O2 (5)	Output channel for deviator load control
P	P $\emptyset$	Initial cell pressure * 4.56
	P1	Pressure transducer readings
	P2	Pressure transducer limit
	P3	Pressure transducer limit
	P4	Initial base pwp
	P5	Initial probe pwp
Q	Q(50)	Array for deviator stresses
	Q($\emptyset$)=Q7	Shear stress prior to shear, after consolidation
	Q $\emptyset$	Target for base deviator stress Q($\emptyset$)
	Q1	Amplitude of deviator stress cycle
	Q4	Target change in deviator stress over T2 (= Target Q - measured Q)
	Q6	0.2*ABS(Q4) - Limit for entering friction boost routine
	Q8	Target deviator stress (= Q $\emptyset$ *J/J2 during undrained application of deviator stress) (= Q7+Q1sin(W) during cycling)

Q	Q9	Measured change in deviator stress during step
R	R	Local slope of area change vs deviator stress plot
	R1, R2, R5	Calculation for calibration factor for pressure transducer
S	S1	Calibration factor for cell pressure
	S2	Calibration factor for pore pressure
	S3	Calibration factor for deviator load
	S4	Calibration factor for axial displacement
	S5	Calibration factor for probe pwp
	S6	Calibration factor for radial displacement
	S	Seconds
T	T	Current time (seconds)
	TØ	Time to establish base deviator stress (seconds) (linear increase)
	T1	Cycle period (seconds)
	T2	Time between adjustments (seconds) (= T3 or T5)
	T4	Variable for time during monitoring of pwps
	T3	Adjustment step during monotonic shearing
	T5	Adjustment step during cycling
U	U(50)	Array for mean load in bottom chamber during step
	U2	Output voltage on O2
V	VØ	Initial sample volume
	V1	Transducer reading on channel 2 during 'MONITORING OF PWPs'
	V5	Transducer reading on channel 25 during 'MONITORING OF PWPs'
	V2	Transducer reading on channel 22
W	W	Current value of $\frac{T2 \cdot J}{T1} \cdot 2$
	WØ	Sample weight

X	X1	Resultant value of DAS (8, —) on channel 01
	X2	Resultant value of DAS (8, —) on channel 02
Y	Y(50)	Array for local values of $(\Delta P/\Delta T/V)$
Z	Z	Resultant value of DAS (2, —) and DAS (3, —)
	Z1	Zero reading on cell pressure transducer
	Z2	Zero reading on pore pressure transducer
	Z3	Zero reading on deviator load transducer
	Z4	Zero reading on axial displacement transducer
	Z5	Zero reading on pwp probe transducer
	Z6	Zero reading on radial displacement transducer

ARRAY LIST

A(50)	Current sample area
G1(50, 10)	Cell pressure readings
G2(50, 10)	Pore pressure readings
G3(50, 10)	Load cell readings
G4(50, 10)	Displacement transducer readings
G5(50, 10)	Strain belt readings
G6(50, 10)	Time at which adjustments made
L(50)	Deviator loads
N(50)	Mean deviator stress during T2
Q(50)	Target and measured deviator stress
U(50)	Approximate target/measure of lower chamber pressure
Y(50)	Slope of $(\frac{1}{V} \cdot \frac{dL}{dt})$ vs U

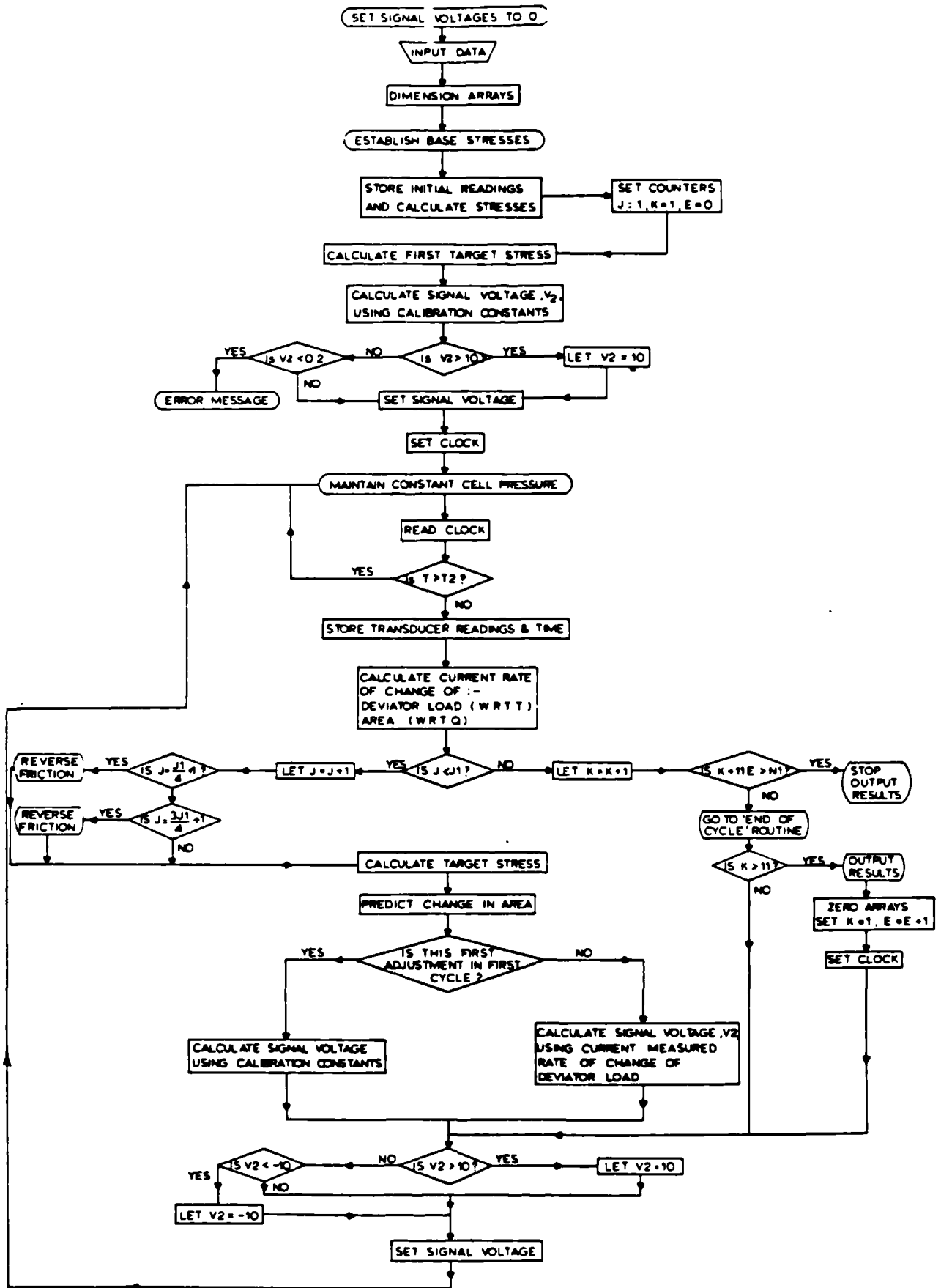


Fig. 5. A1. 1 Algorithm for undrained cyclic loading.

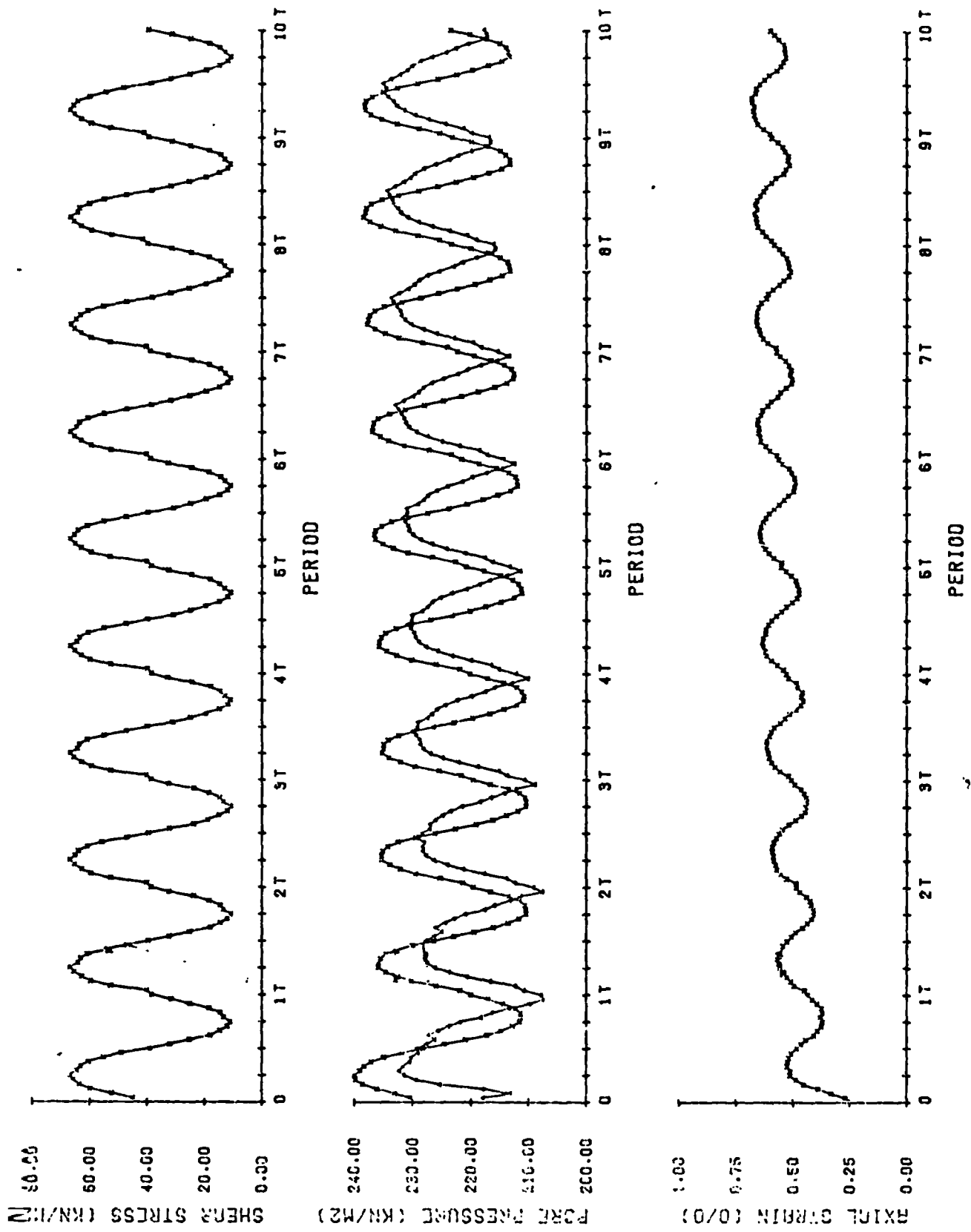


Fig. 5. A1. 2. Computer plot of data from cyclic test.

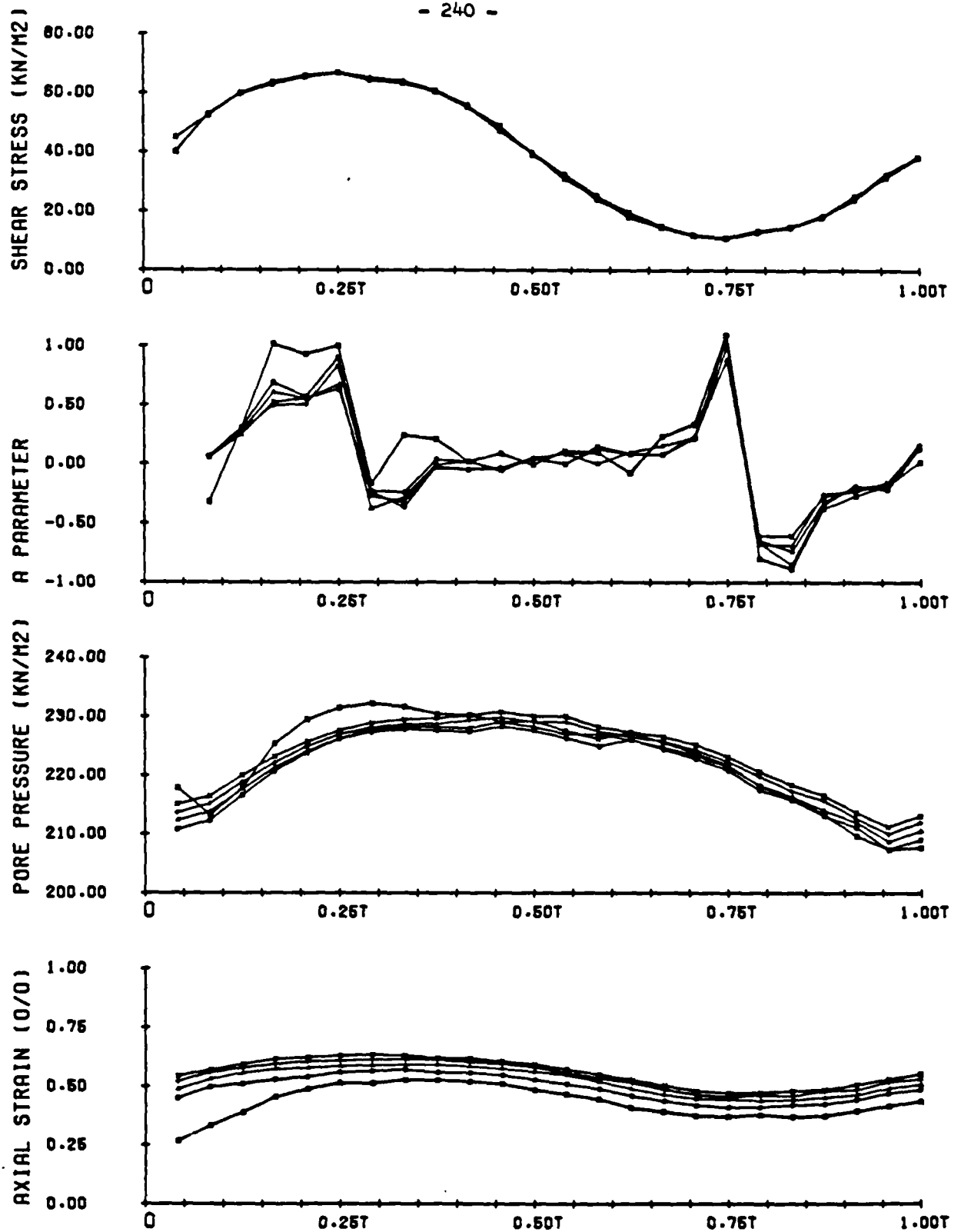


Fig. 5. A1. 3. Computer plot of data from cyclic test.
(Cycles 1 - 5)

APPENDIX 5.2

Hysteresis in Load Cells

Hysteresis develops in the load cell as a result of friction:-

- (i) between the end points of the triangular plate and the base groove, i.e. at X in Fig. 5.14, and
- (ii) at the shoulder where the plate sits on the groove, i.e. at Y in Fig. 5.14.

There is, of course, additional hysteresis as a result of internal damping in the steel and at the bonds between the strain gauges. Friction at X and Y also changes the fixity conditions for the sensor and, therefore, the bending moment at its centre and its calibration.

In the load cells in use when this investigation was started, the triangular plate could be rotated in its groove, so that different contact points could exist each time the cell was loaded and the hysteresis was not repeatable. A simple modification was made to prevent rotation of the plate; pins were inserted adjacent to each nib on the sensor.

It was found that when the same contact points were maintained, the hysteresis was:-

- (i) remarkably consistent for a given amplitude of loading cycle,
- (ii) independent of the load increment size in the cycle, and
- (iii) a function of the starting load and maximum load.

These points are illustrated in Fig. 5. A2. 1.

The hysteresis was largely eliminated by rubbing down and chamfering the sensor at its nibs. For the purposes of control and

calculation of data, the remaining hysteretic behaviour was idealised. Different calibration factors were used for loading and unloading (refer to Fig. 5. A2. 2 for comparison of loading and unloading calibration curves in load cells before and after modification). The direction of loading was assessed; if unloading was taking place, the maximum load in the cycle was found, and the appropriate correction for hysteresis was made, depending on the current load level.

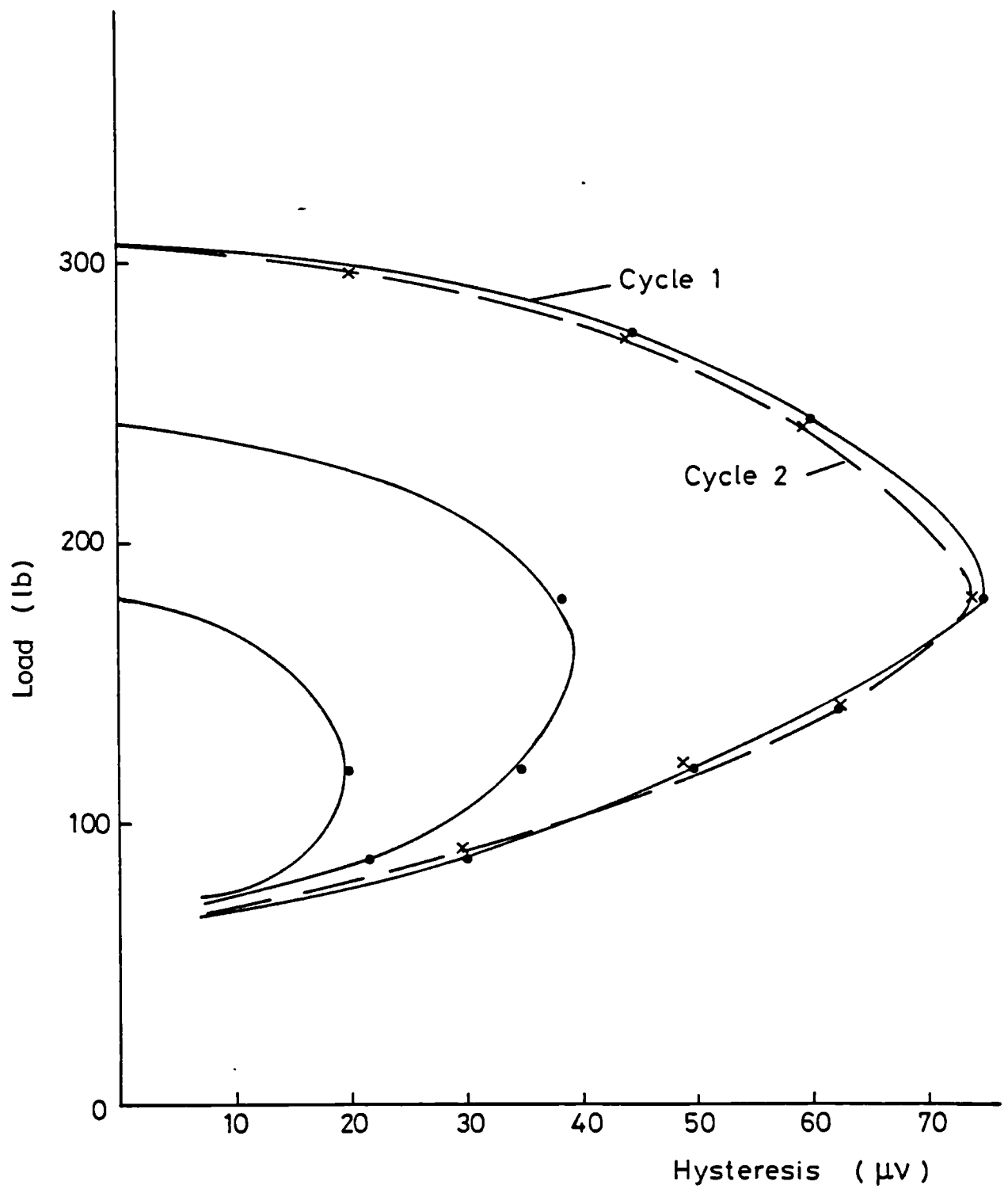


Fig. 5. A2. 1 Hysteresis in standard load cells

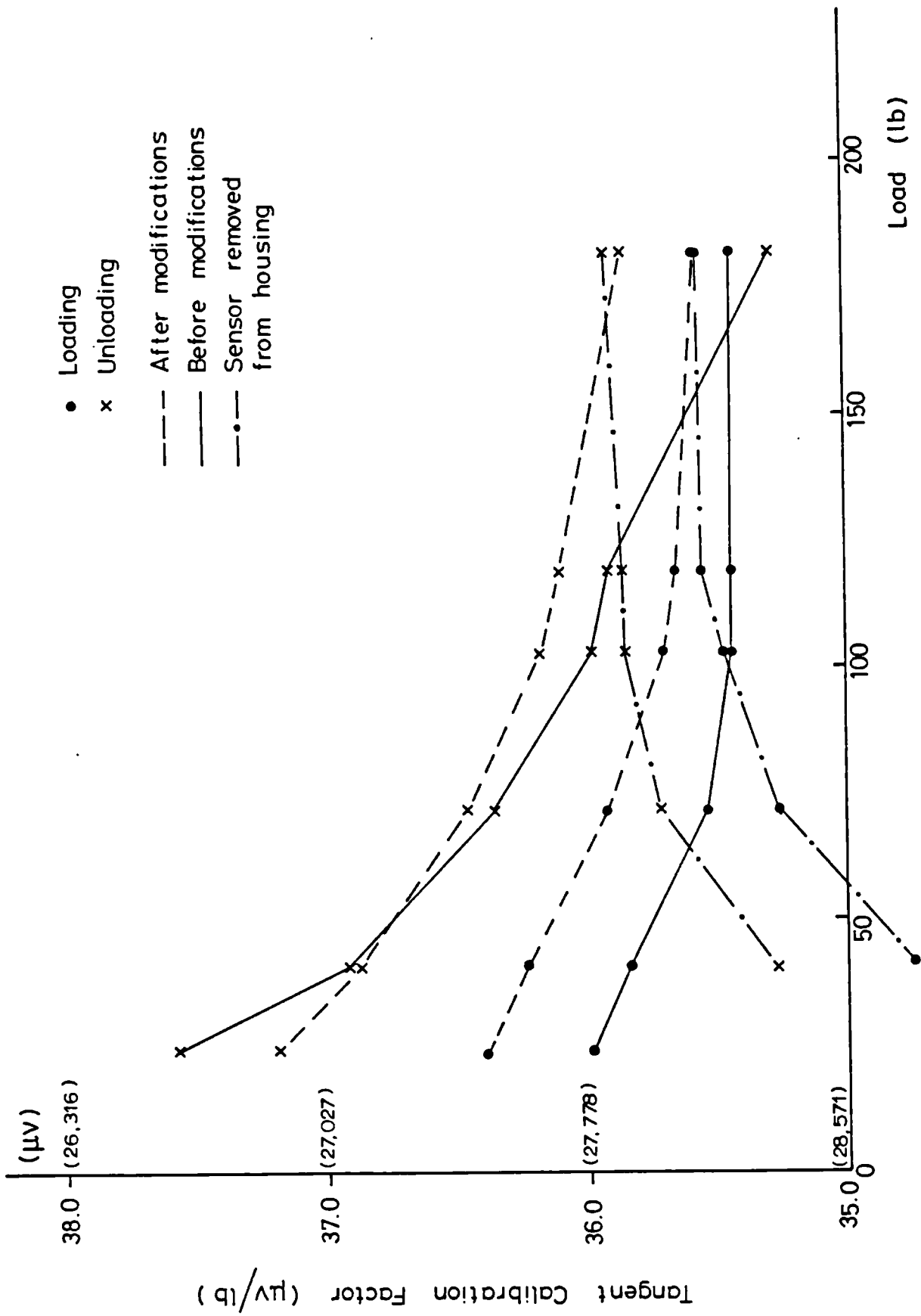


Fig. 5. A2. 2 Load cell calibrations before and after modification.

CHAPTER 6

PORE PRESSURE MEASUREMENT IN CLAYS UNDER
CYCLIC AND TRANSIENT LOADING

SYNOPSIS

In this Chapter the measurement of pore water pressures during cyclic and transient loading of clays is reviewed. The use of a new miniature pressure transducer with extremely small volume change factor for monitoring pore pressures at the mid-height of triaxial samples under cyclic and transient loading is described. Details are given of the transducer, its installation and performance. The validity of the measured pore pressures is examined.

6.1 INTRODUCTION

An important objective in cyclic and transient loading tests on clay should be the accurate measurement of pore pressure changes, so that interpretation of behaviour may be made in terms of effective stress. However, the monitoring of pore pressures in the investigation of the cyclic behaviour of clays tends to be an exception rather than a rule (Yoshimi et al, 1977). This lack of data on pore pressure changes reflects the difficulty of measurement in cyclic and transient tests and calls into question the validity of any measurements that are made.

Measurement of pore water pressures in triaxial samples under monotonic loading has been fully dealt with by Bishop and Henkel (1962). Two aspects were considered:-

- (i) time lag^{*1} in the pore pressure measuring device, and
- (ii) progressive equalisation of non-uniformity in pore pressure resulting from end restraint or from a natural tendency to zone failure.

They concluded that the time for 95% equalisation within a sample was large compared with the response time of null-indicators and pore pressure transducers in use at that time. For the base measurement of pore pressure during shear, there was then no advantage in improving response time. It was still necessary, however, to limit the expansibility of the measuring system in order to minimise error in the measured pore pressure (Gibson, 1963).

*1 Time lag (or response time) is the time required for the measuring system to register a change in pressure. It is related to the compliance of the transducer and connections, to the compressibility and permeability of the soil and to the collecting area of the piezometer.

Monotonic loading tests are normally carried out with pore pressures measured at the base and the sample sheared at rates which ensure equalisation within the sample, either at failure if only effective stress shear strength parameters are required, or at an early stage of the test if the effective stress path is to be derived. The shearing rates may be increased when lubricated ends are used to reduce the non-uniformity of strain and pore pressure in the sample, provided that shear zones do not form. In many cases, these rates of loading for clays exceed the relevant field monotonic loading rate by several orders of magnitude. For transient field loading problems, these rates are inappropriate, yet equalisation throughout clay samples will not occur at faster rates and incorrect pore pressures will be measured at the base.

The approach of loading at rates which ensure equalisation throughout the sample has been applied in cyclic triaxial tests using fixed-end samples by, for example, Sangrey et al (1969). For the clayey silt which they investigated, reliable base pore pressure measurements dictated the use of a cycle frequency of one cycle in 10 hours. Such a frequency is considerably lower than the typical frequencies encountered in field situations, see Fig. 6.1. Evidence will be presented in Chapters 9 and 10 which shows that, in some clays, the pore pressures generated in undrained cyclic tests are a function of rate of shearing. Cyclic tests carried out at these atypically low frequencies may present a misleading picture.

Measurement of pore pressures at the appropriate loading frequencies is clearly desirable. This cannot be achieved by base measurement of pore pressure in cyclic tests on fixed-end specimens of clay with low coefficients of consolidation. Even in samples with lubricated ends, non-uniformities in pore pressure may exist, there being uncertainty, at present, over the effectiveness of the lubrication at fast shearing rates and under reversal and repetition of loading; Khaffaf (1975) reports such difficulties after the first few cycles of loading. Certainly, the introduction of a piezometer at the centre of a

lubricated end and which penetrates the sample would introduce local non-uniformities in pore pressure which may not dissipate during a cycle.

It is obviously necessary, then, to measure pore pressures at locations where the applied stresses are uniform and known, i.e. away from the ends. There is considerable theoretical evidence (see, for example, Balla, 1960) that over the central portion of a fixed-end triaxial sample, the stress conditions are more-or-less directly determinable, the radial stress being given by the fluid pressure on the flexible boundary and the vertical stress by the applied load at the rigid boundary. There is, in addition, experimental evidence, provided by Barden and McDermott (1965), that pore pressures measured with piezometer probes in this central portion in fixed end samples are similar to those measured in free end samples under monotonic loading at relatively slow rates.

Pore pressure measurement at the mid-height of triaxial samples would then appear to be an essential requirement for cyclic and transient loading since it would provide a more reliable link between total stress and pore pressure change than base measurement. Since most cyclic testing involves shearing at levels below peak shear strength, the problems of pore pressure measurement posed by non-uniformities associated with shear zones should not arise, at least until a late stage of the test. To ensure correct measurements at appropriate loading rates, response time of the measuring device becomes an important consideration.

6.2 LOCATION OF THE PIEZOMETER PROBE

There is a dilemma over where to position a piezometer probe at the mid-height of a triaxial sample. The introduction of a rigid inclusion into the sample, or of a rigid disc at the sample periphery, produces a perturbation to the stress field which, at fast rates of shearing in undrained tests, results in the generation of a zone of atypical pore pressures local to the probe; with a fast response transducer, these pressures become the measured pore pressure. The perturbation to the stress field, in terms of both the volume of soil affected and the magnitude of the stress change, must be kept to a minimum so that

dissipation of the non-uniform pore pressures takes place in an acceptably short time. This, together with a minimum response time, form the criteria for selection of a piezometer probe to be used at the mid-height of cyclically or transiently loaded triaxial samples.

Several investigators have used piezometer probes at various locations in triaxial clay samples, for example, Taylor (1944), Bishop, Blight and Donald (1960), Barden and McDermott (1965), Maguire (1975). In all this work, monotonic loading at relatively slow rates was involved so that, presumably, equilibration of pore pressures around the probes was occurring. Sandroni (1977) appears to be the first to have examined the equalisation of excess pore pressures caused by stress concentrations around rigid included piezometers.

Elastic solutions by Goodier (1933) for spherical and infinite cylindrical inclusions were used by Sandroni to provide an upper limit for equalisation times. Goodier's solutions show that the stress disturbance (for $\nu \neq 0.5$) decreases cubically from a spherical inclusion and that at a distance of two radii from its surface the effect of the inclusion virtually disappears. This implies high initial outward gradients of pore pressure which accelerate the equalisation process. Sandroni suggests that for 90% equalisation

$$t = \frac{2a^2}{C_v} \qquad 6 - 1$$

where a is the radius of the spherical inclusion or an equivalent dimension and C_v is the coefficient of consolidation.

For soil having a C_v of $1\text{m}^2/\text{year}$, a probe of 1.25mm radius inserted into the sample has an equalisation time of 100 seconds. Only with a probe of radius 0.4mm can an equalisation time of 10 seconds be achieved. Here lies the dilemma, for response time is related directly to the cube of a , and minimising equalisation time can only be achieved with increased response time.

These dimensions indicate that, even allowing for the conservatism of Sandroni's analysis, the use of probes inserted into the sample are

unlikely to provide reliable measurements of pore pressure changes during rapid shearing. Intuitively one would expect that the best balance between minimum disturbance and minimum response time (by way of maximum collecting area) would be achieved by placing the piezometer probe at the cylindrical surface of the specimen^{*1}.

6.3 INTERFERENCE ARISING FROM A RIGID DISC PIEZOMETER AT THE PERIPHERY OF A TRIAXIAL SAMPLE

The total stress perturbations produced by the introduction of a rigid disc into an otherwise flexible boundary need to be examined for two cases of loading, a change in mean stress and a change in deviator stress.

For a rigid inclusion in an elastic body, the effect of an isotropic stress change depends on the value of Poisson's ratio, ν ; for ν of 0.5, the rigid inclusion does not perturb the stress field. This follows immediately from the incompressibility of both an elastic body with ν of 0.5 and of the inclusion; there are no strains under hydrostatic loading and so there can be no interference. Similarly, for a rigid disc at the boundary of an elastic body, there is disturbance of the stress field only when Poisson's ratio is less than 0.5.

In a saturated soil, Poisson's ratio is very close to 0.5 (Bishop and Hight, 1977) so that interference does not arise between the soil and an included piezometer or one placed at the surface when there is a change in hydrostatic stress. This is not the case in partially saturated soil in which Poisson's ratio is less than 0.5.

When a deviator stress is applied to a triaxial sample the resulting axial strain produces relative movement between the sample and the disc, see Fig. 6.2. Total stress perturbations occur as a result of friction on the face of the probe. (Similar tractions

*1 The term 'probe' is retained for a piezometer placed at the surface, despite the lack of penetration.

develop when hydrostatic stress is applied to partially saturated samples.) The volume of the sample influenced by the shear stress on the disc is clearly a function of the disc's area; the pore pressure developed in this volume is determined in part by the level of the shear stress; in turn, the shear stress is a function of the effective radial stress on the sample, σ_r' . Interference between the measuring device and sample is minimised by keeping the disc area and shear stress as small as possible.

To provide an indication of the soil volume influenced by shear displacement of a rigid disc on the boundary of a triaxial sample, the distributions of stress changes have been derived for the case of a linear radial shear stress on the surface of an isotropic elastic half space. Integrals published by Gerrard and Harrison (1970) have been used for this purpose. In Fig. 6.2, the calculated stress changes on a diametral plane are shown. Two boundaries have been sketched, marking the volumes in which stress changes exceed 10% and 25% of the surface traction. It is apparent that these volumes are small. Stress disturbance is insignificant at a distance of one radius from the disc.

6.4 RADIAL VARIATIONS OF PORE PRESSURE IN TRIAXIAL SAMPLES

In making a decision on where to locate a piezometer probe at the mid-height of a sample, one must balance the clear advantages of minimal disturbance with the piezometer at the periphery, against the potential error resulting from radial variations in stress and pore pressure at mid-height. The theoretical evidence referred to previously demonstrates that because of end fixity, radial variations in stress do occur, but these reduce at the centre of the sample. In Balla's (1960) analysis, the maximum difference in any stress across a diameter at the mid-height of the sample does not exceed $\pm 3\%$. Costa Filho (1980) confirmed that only a small radial variation in stress occurs at mid-height in a non-linear elastic analysis of a triaxial sample loaded between rough plattens. Appendix 1 to this Chapter further confirms

the radial uniformity of stress; results are presented for an elasto-plastic analysis of a triaxial sample loaded between fixed ends.

Experimental evidence does not always confirm the radial uniformity of stress at mid-height. For example, Shockley and Ahlvin (1960) set up very large triaxial specimens of fine sand which incorporated pressure cells which showed that at mid-height the vertical stress was greater in the centre than towards the edge. In soils with a granular structure in which load trajectories develop it might be expected that in triaxial compression, vertical stresses at the periphery would be lower than towards the centre; this is less likely to be the case, however, in extension loading. Blight (1963) refers to experimental work which shows that the pore pressure set up along the central axis of a triaxial specimen is higher than set up around the perimeter. In fact, the observations, based on radial profiles of water content, show only a small difference at the mid-height, and then only after the sample had been taken to large strains after cycles of loading and unloading. Barden and McDermott (1965) provide evidence from piezometer probe measurements of differences between pore pressure at the centre and periphery at mid-height of fixed-end samples. Since such comparisons can only be made at relatively fast rates of shearing, otherwise radial equilibration occurs, the differences which they report may, to some extent, reflect differences in interference effects between the probes used at the various locations.

It is reasonable, then, to anticipate a small radial variation in pore pressure at the mid-height of triaxial samples loaded rapidly between fixed ends. Piezometer probes which penetrate the sample are more likely to give a better average measurement of pore pressure than those mounted at the periphery, provided that there is equalisation of the pore pressures caused by the inclusion.

However, in transient loading tests and many cyclically loaded tests, such equalisation is unlikely to be complete and the resulting error is likely to exceed any error produced by measuring at the periphery. In the work to be described, a piezometer probe has been used with its ceramic disc flush with the sample's cylindrical surface at mid-height. Performance data indicates that disturbance effects are minimal and errors

are small. Surface mounting has the added advantage of convenience of installation.

6.5 THE MINIATURE PRESSURE TRANSDUCER

The piezometer probe used for cyclic and transient loading tests is based on a miniature silicon diaphragm pressure transducer with integral semiconductor strain gauge bridge and temperature compensation. Its construction is illustrated in Fig. 6.3 and other details are given in Table 6.1. The advent of this form of transducer is regarded in industrial instrumentation as the most significant development in recent years, see, for example, Pratt (1979). Certainly, the transducer has characteristics which make it particularly suitable for pore pressure measurement in soils.

Bertioli (1979) has described the principles and applications of silicon integrated pressure transducers and pointed out their advantages compared with conventional pressure transducers. These advantages relate mainly to some of the mechanical and piezoresistive properties of silicon, which may be summarised as follows:-

- (i) The material is Hookean up to its breaking point so that linearity in calibration can be achieved and the effects of yield are removed.
- (ii) In a single crystal, which can form the whole sensing diaphragm, there is no hysteresis or creep at normal temperatures. In addition, since the semiconductor gauges are diffused into the diaphragm, the risk of introducing hysteresis and creep with adhesive bonding is eliminated.
- (iii) The material has a low density so that high resonant frequencies can be achieved and the transducer has low acceleration sensitivity.
- (iv) The gauge factors for the diffused semiconductor gauges are typically fifty times those of conventional gauges giving much higher outputs and sensitivities.

For general pore pressure measurement, the last listed advantage is, perhaps, the most important, since it enables small and relatively stiff sensors to be used, which lead to a very small displaced volume with applied pressure. The diaphragm shown in Fig. 6.3 has an active diameter of 2.5mm and a thickness of $225\mu\text{m}$. Its calculated compliance is 0.01mm^3 per 10^9 kN/m^2 . In addition, in the configuration shown in Fig. 6.3 the volume of water between the high air entry porous stone and the diaphragm is only circa 1mm^3 ; the low compliance of the sensor and the very small volume change associated with the compressibility of the water ($5 \times 10^{-7} \text{ mm}^3/\text{kN/m}^2$) combine to give a low overall volume intake factor for the piezometer and very short response times, even in clays with low coefficients of consolidation.

Pressure transducers of this type have been used for pore pressure measurement in centrifuge testing, refer Hird (1974), and Hird, Marsland and Schofield (1978), where advantage has been taken of, inter alia, the fast response and low acceleration sensitivity. For cyclic and transient triaxial testing the advantage of fast response is combined with:-

- (i) the small overall size and weight of the transducer which enable it to be placed, without additional support, directly on the side of the sample, so eliminating the need for expansible couplings between a probe and its transducer,
- (ii) the high output which reduces loss of resolution in high speed logging, and
- (iii) the elimination of hysteresis in calibration for cyclic work.

The adequacy of the temperature compensation varies between transducers but is generally good. Zero drift is acceptably low.

5.6 INSTALLATION PROCEDURE FOR THE PIEZOMETER PROBE

As with any piezometer the first step in the installation procedure is to de-air the porous stone and the cavity between the stone and the sensor. This has been achieved by immersing the transducer in de-aired

water and applying a vacuum to the water for several hours. Although complete de-airing cannot be guaranteed, there being no facility for flushing water through the stone or cavity, the compliance of the transducer after de-airing is found to be too small to be recorded.

The latex rubber membrane is modified to allow the transducer to pass through; refer to Fig. 6.4 for details. A hole is cut in the latex membrane at its mid-height with a cork borer. A silicone rubber grummet, formed with Dow Corning RTV 210 in a perspex mould, is inserted in the hole with its shaft protruding through. The rubber grummet houses the transducer and provides a former for extending the latex rubber membrane by painting on successive layers of pre-vulcanised latex.

A special membrane stretcher was used for placing the membrane on the sample. The hole in the membrane for the probe was sealed by a brass plug which was removed when the membrane was in place.

To ensure intimate contact between the sample periphery and the porous stone of the transducer, a pad of soft saturated kaolin is placed on the stone prior to installation. The low strength of the kaolin, even after its consolidation against the sample, sets an upper limit to the shear stress which can develop between the piezometer probe and the sample. In tests where precise measurements of the initial effective stress in the sample are required the kaolin can be omitted. With loss of intimate contact there is an increase in response time.

After pushing the transducer into its housing, and avoiding any penetration of the sample, the assembly is sealed with two O-rings as shown. For additional security against leakage, latex rubber is painted around the installed transducer.

The conductors from the transducer are protected by a PTFE sleeve which is flexible and so reduces any tendency for relative movement between the transducer and the sample. The conductors pass through the base of the testing apparatus through an O-ring seal. Plate 6.1 shows the assembled piezometer probe.

Air inevitably is trapped during installation of the transducer. Furthermore, in samples with a relatively high suction, cavitation may occur in the water-filled void behind the porous stone. To eliminate gases, since water cannot be flushed behind the device, it is necessary to use high back pressures when working with the probe. When high suctions are present during installation and water can be imbibed from the kaolin or stone, a local zone of wetter clay would result. ('Pips' could be placed around the face of the probe to prevent contact with sample until a confining stress is applied.)

6.7 PERFORMANCE OF THE PIEZOMETER PROBE

The miniature pressure transducers have been used for six years in the manner described. They have proved to be reasonably reliable and simple to install. Problems encountered with them are described after a discussion of their response time and their general performance.

(a) Response time

An estimate of the response time for the transducer can be made assuming the water and soil to be incompressible and following Hvorslev (1951):

$$\begin{aligned} \text{Equalisation,} \quad \epsilon &= 1 - e^{-(Fkt/V)} & 6 - 2 \\ \text{where} \quad F &= \text{intake factor} \\ k &= \text{soil permeability} \\ V &= \text{volume factor, i.e. the volume change} \\ &\quad \text{involved in registering unit pressure} \\ &\quad \text{change (which includes compliance of} \\ &\quad \text{the transducer, connections and hydraulic} \\ &\quad \text{tubing, and compression of the water in} \\ &\quad \text{the system).} \end{aligned}$$

For a piezometer comprising a flat disc of radius R at the surface of the soil, Hvorslev gives for the intake factor:

$$F = 4R \qquad 6 - 3$$

The time for 95% equalisation is then given by

$$t = \frac{3.V.\gamma_w}{4Rk} \quad 6 - 4$$

The volume factor, V, for the transducer can be estimated from the figures quoted previously for compliance and compression of the water in the cavity, and making allowance for the water in the porous stone. Combining these and taking the radius of the disc as 5mm, then the time for 95% equalisation is

$$t = 3 \times 10^{-17}/k \text{ secs} \quad 6 - 5$$

where k has units of m/sec. In other words, for a permeability as low as 10^{-12} m/sec the estimated response time is 3×10^{-2} milliseconds.

Since there are no hydraulic connections to the back of the ceramic disc in the piezometer, response times can only be measured by monitoring the response of the transducer to the pore pressure changes caused by an instantaneous change in cell pressure. Implicit in this assessment are the assumptions that the true pore pressure response is instantaneous and that the presence of the probe produces no perturbation to the stress field, and hence to the pore pressures.

The second assumption has been discussed previously; with the probe mounted at the side of a saturated sample it should have no influence on the pore pressures generated by an isotropic stress change. For partially saturated samples, interference effects are produced for both a surface probe and one intruding into the sample; the time taken for the pore pressure change to be registered then reflects both the response time and the equalisation time for the interference effect.

With regard to the first assumption, Taylor (1944) reported pore pressure measurements taken with a probe running diagonally across the centre of a sample; during isotropic consolidation a delay of between 40 to 100 minutes occurred before the maximum pore pressure was reached. Since this time appeared to be far greater than the response time for the probe, Taylor felt that this indicated that 'direct pressure applied

to a saturated undrained sample does not immediately cause pore pressure as is usually supposed'.

A curve showing the typical response of the piezometer probe to a change in cell pressure is shown in Fig. 6.5. The sample was of Lower Cromer Till (C_v of 1 to $2m^2/year$) and transducer readings were taken at a fast sampling rate on the computer-logger described in Chapter 5. It is evident that the response time is well under a second; it is difficult to be more precise because of the difficulty in applying an instantaneous change in cell pressure. In all cases, an instantaneous response of pore pressure to cell pressure change, regardless of its magnitude, is indicated, so that Taylor's observations have not been repeated. Similar response curves were obtained with the piezometer installed against samples of London Clay.

Such a fast response time is obviously adequate for measurement of pore pressures in investigations of the effects of loading rate on clays. However, in transient tests, the superimposed effects of interference and radial pore pressure non-uniformities have to be considered.

As to the requirements for response time in cyclic tests, one can, as a first approximation, make use of an analysis presented by Hvorslev (1951). Again, for the case of incompressible soil and water, Hvorslev examined the pore pressures registered by a piezometer installed in ground in which sinusoidal fluctuations in ground-water pressure were occurring. As a result of time lag in the piezometer, the registered pore pressures showed a phase shift of t_s and an attenuated amplitude of x_a and period T_w . Clearly, the response time should be such that:-

$$\frac{t_s}{T_w} \longrightarrow 0 \quad \text{and} \quad \frac{x_a}{z_a} \longrightarrow 1 \quad 6 - 6$$

Hvorslev gives the following expressions for $\frac{t_s}{T_w}$ and $\frac{x_a}{z_a}$:-

$$\frac{t_s}{T_w} = \frac{1}{2\pi} \arctan \frac{2\pi T}{T_w} \quad 6 - 7$$

$$\frac{x_a}{z_a} = \frac{1}{\sqrt{1 + (2\pi T/T_w)^2}} \quad 6 - 8$$

where T is the basic time lag, which is equivalent to one third of the response time for 95% equalisation.

For T/T_w of 0.02

$$\frac{t_s}{T_w} = 0.02 \quad \text{and} \quad \frac{x_a}{z_a} = 0.99 \quad 6 - 9$$

For a cyclic period of 20 secs, a basic time lag of 0.4 secs is required to achieve an acceptably low phase shift and acceptably low error in amplitude for the measured pore pressures. This corresponds to a response time of 1.2 secs, i.e. well within the capability of the new piezometer.

(b) Monitoring pore pressures in cyclic and transient loading tests

In Fig. 6.6 the pore water pressures measured conventionally at the base (u_b) and with the new probe at the mid-height (u_m) are plotted against time for a cyclic triaxial test; the controlled changes in shear stress are also shown. The particular test selected is a cyclic compression-extension test on a 38mm diameter x 76mm diameter high sample cut from a block of reconstituted Lower Cromer Till; a relatively slow cycle period of 30 minutes was used.

The differences between the two pore pressure measurements are striking and the following points should be noted:-

- (i) The pore pressure changes measured at the base are in phase with and approximately equal to the deviator stress changes; this fact is considered further subsequently.
- (ii) The pore pressure changes measured at the probe lag and are smaller than the deviator stress changes; there is a lack

of symmetry in the pore pressure changes and occasionally irregularities are detected.

- (iii) At the end of each cycle, the pore pressures measured at the base and mid-height are approximately equal; this is in part a consequence of the long period used and is discussed subsequently.
- (iv) Pore pressure gradients develop at the start of cycling; the maximum gradient in a cycle increases with cycling.

The effective stress paths corresponding to the two sets of pore pressure measurement are drawn in Fig. 6.7. The paths based on the probe measurements look entirely sensible; there is a distinct change in cyclic response when the e.s.p. reaches the compression failure envelope. This is not the case for e.s.ps based on the base measurement. The tendency for the e.s.ps based on the probe measurement to migrate outside the failure envelope defined in slow tests in extension is discussed by Takahashi et al (1980) and considered in Chapters 9 and 11.

Pore pressures measured with the probe in transient loading tests on Lower Cromer Till are presented in detail in Chapter 9; an example is given in Fig. 6.8(a), together with details of the change in $(u_b - u_m)$ with rate of loading. In tests UU-1 and UU-2 shear zones formed post-peak so that pore pressures measured with the probe become less reliable at this stage.

Some of the features of the effective stress paths observed in transient and cyclic tests on the Lower Cromer Till, particularly those in triaxial extension, are ascribed to the development of pore pressure inhomogeneities; these arise from the particular microfabric of the soil (Chapters 9 and 11). To demonstrate the quality of the observations made with the piezometer probe, triaxial extension tests were run at different rates on samples of Portavadie silt which has a similar C_v to Lower Cromer Till but a quite different microfabric. Figure 6.8(b) presents the results of a fast and slow test; excellent agreement in terms of e.s.p. is obtained.

(c) Interference effects

The response curve shown in Fig. 6.5 indicates that for saturated samples subjected to a change in isotropic stress there is, as anticipated, negligible interference. In Fig. 6.9, the response curve for a partially saturated sample subjected to an isotropic stress change is shown. Again as anticipated, there is interference; the probe over-reads relative to the equilibrium value by about 25%. Of particular interest is the time taken for equilibrium pore pressures to be established; this is of the order of 2 to 3 minutes.

Interference is greater and equilibration times are longer in partially saturated samples subject to isotropic stress change than saturated samples subject to changes in deviator stress. To provide a measure of the interference caused by a change in deviator stress on a saturated sample, an installed piezometer probe was slid a short distance along the sample's surface. A pore pressure of approximately 10 kN/m^2 was generated and this dissipated in less than 10 seconds. The radial effective stress was 200 kPa. This pore pressure change is likely to represent an upper bound to interference effects; certainly in unloading cycles strains are small and interference will be low. Barden and McDermott (1965) suggest that a disc at the periphery will cause an under-reading because of the friction which develops at the disc-soil boundary. This was not the case in the crude simulation which was made.

A further indirect check on interference effects related to boundary stresses is available from a transient compression test on Lower Cromer Till in which the total stress path involved reducing σ_3 to failure in 35 seconds. The e.s.p. was the same as that observed in test UU-1 shown in Fig. 6.8(a).

(d) Operating difficulties

Two features of the transducer have given rise to operating difficulties. These are:-

- (i) the very small gap between the porous stone and the silicon diaphragm, which has led to interference between them when monitoring pore pressures in samples at moderately high effective stress; and
- (ii) the sealed cavity behind the porous stone, which means that gases cannot be flushed out when the transducer is in use.

Of course, it is these features which contribute to the very short response time of the transducer; however, the problems associated with them should be recognised.

Interference between the stone and the sensing element was first detected during consolidation of samples, when significant pore pressure differences appeared to remain between the base and mid-height at the end of primary consolidation. Interference was confirmed by setting the transducer against a porous stone inlaid into a dummy brass sample; by controlling the total stress on the sample and the back pressure in the porous stone, the level of effective stress at which interference developed was determined as 260 kN/m^2 . Obviously, it is essential to be aware of this effective stress level for the particular transducer being used.

The potential problem created by gas in the cavity, that is gas derived from the sample or membrane or generated by corrosion in the transducer, exists only when operating at relatively low pore pressures or back pressures. In the triaxial tests referred to subsequently, pore pressures were maintained high enough to avoid the problem. In the 1g model tests described by Martins (1980), elevated back pressures were not used and the transducer proved unsuitable for his long term tests.

Since the cavity cannot be de-aired completely a small air bubble probably exists, shown as stage (1) in Fig. 6.10. However, pore water pressures are measured. Air diffuses from the higher pressure of the small diameter gas bubbles in the soil to the lower pressure in the larger bubble in the cavity. The bubble in the cavity grows and the diffusion rate increases. At stage (2) in Fig. 6.10 pore water pressure is still

being recorded; the difference between the air and water pressures behind the stone is small because of the large size of the bubble. As diffusion continues, the bubble grows until it is in contact with the porous stone, stage (3) in Fig. 6.10. Capillary forces in the pores of the stone initially prevent the bubble increasing in size; air pressure builds up with continuing diffusion and air pressure is now measured by the transducer. A limit is set to the air pressure which can develop in the cavity by the air entry value of the porous stone. (This is determined by the pore size in the stone, provided that the stone is properly sealed around its periphery.) When the air pressure exceeds the air entry value of the porous stone, air migrates through the stone; on reaching the soil, air pressure again builds up until the air entry value of the soil is exceeded.

The time for diffusion depends on the length of the saturated flow path and the pore size of the path, i.e. on the thickness of the porous stone. Where diffusion of gas from the soil to the transducer is a potential problem, then it may be possible to increase the length of the diffusion path by extending the porous stone. Martins (1980) tried this, but still found that gas filled the cavity after a period of 17 days.

It would appear, then, that the problem of gas in the cavity is exacerbated by the generation of gas within the transducer. This could be associated with the electrolytic production of gas, either as a result of stray currents from the electronic circuitry or galvanic potentials arising from the use of dissimilar materials.

6.8 THE VALIDITY OF THE MEASURED PORE WATER PRESSURES

(a) General

There are a number of sources for error in the mid-height measurement of pore water pressures in undrained monotonic and cyclic loading triaxial tests; some have been referred to before; the complete list comprises:-

- (i) The existence of radial non-uniformities of pore pressure at mid-height.
- (ii) Expansibility (or leakage) in the pore pressure measuring systems and drainage lines to which the sample is open. Where expansibility is large, significant volumes of pore water transfer between the sample and measuring device so that the test becomes partially drained.
- (iii) Interference between the measuring system and sample; this could have a cumulative effect in cyclic testing.
- (iv) Calibration of the electrical transducers; in cyclic loading the potential for calibration error is increased by possible hysteresis in the transducers.
- (v) The occurrence of air diffusion through the membrane, and, in some cases, osmosis.
- (vi) Equilibration of vertical pore pressure differences in the sample which result from complete or partial end restraint.

In the work which will be referred to in Chapters 8 and 9 and in the apparatus which has been described previously:-

- (i) It is considered that, in the light of the theoretical and experimental evidence discussed above, radial non-uniformities of pore pressure are relatively small. Equalisation of any radial non-uniformities in a triaxial sample with a height:diameter ratio of 2:1 takes place in about one sixteenth of the time for equalisation of vertical pore pressure non-uniformities. For a 38mm diameter sample with C_v of $1\text{m}^2/\text{year}$ this corresponds to a time of about 40 mins for 90% equalisation.
- (ii) Expansibility has been minimised by incorporating only short lengths of stainless steel hyperdermic tubing and using transducers with low volume intake factors. Leakage has been eliminated.

- (iii) Interference effects have been reduced by mounting a piezometer at the periphery of the sample; this introduces an additional pore pressure which is estimated to be less than 10 kN/m^2 in saturated samples and which dissipates in a matter of seconds.
- (iv) Calibration has involved using a dead weight tester and working from a continuous non-linear calibration curve relating voltage output to pressure over the full range of the transducer. Hysteresis in the integrated silicon pressure transducer is minimal.
- (v) De-aired water has been used in the cell to avoid diffusion and osmosis has been eliminated by using distilled water in the samples.

Errors resulting from end restraint can be reduced by the use of lubricated ends, but, as discussed previously, their effectiveness in cyclic testing is uncertain. For test procedures in which no attempt is made to reduce the frictional restraint of the loading platens or in which lubrication becomes ineffective, then equilibration of pore pressure differences between the middle and ends of the sample represents by far the largest potential for error. An explanation of how the error develops is given in Fig. 6.11.

The magnitude of the error introduced by end restraint into the measurement of pore water pressure at the mid-height of a triaxial sample is determined by three factors:-

- (i) the difference between the pore pressure generated at mid-height, u_m , and elsewhere;
- (ii) the volume of the sample which is influenced by end restraint; and
- (iii) the degree of equalisation of the pore pressure non-uniformities.

Clearly, if the same pore pressures are generated throughout the sample, even with end restraint, then no error arises; similarly, if the

volume of the sample influenced by end restraint is small, then the error after equalisation will be correspondingly small; if there is no time for equalisation then the pore pressure measured at mid-height is in error only as a result of interference effects between the sample and the piezometer and of radial non-uniformity of pore pressure at mid-height.

The influence of these three factors in monotonic and cyclic loading tests is now considered in more detail.

(b) Pore pressure differences

There has been considerable discussion in the past on the differences between the pore water pressures generated at the centre and ends of fixed-end triaxial samples under monotonic loading, see for example Bishop et al (1960), Blight (1963). In the first place the difference depends on consolidation history, or in simple terms the value of the pore pressure parameter A (Skempton, 1954). Close to the base of a triaxial sample with fixed ends, radial strains, by definition, approach zero; so do axial strains (Costa Filho, 1980). Conditions are close to those in an oedometer loaded undrained. Oedometric conditions give rise to an A value of 1, independent of consolidation history; this is clearly confirmed in the pore pressures measured at the base in the cyclic test presented in Fig. 6.6. In all cases of compression loading, except when the real value of the A parameter for the soil is greater than 1, the pore water pressure generated at the base exceeds that at mid-height. In normally consolidated samples, with an A value approaching 1, the difference between the pore pressures at the base, u_b , and at mid-height, u_m , are small.

The dependence of $(u_m - u_b)$ on consolidation stress history is confirmed in the cyclic tests, refer Fig. 6.12(a) for isotropic consolidation histories and Fig. 6.12(b) for anisotropic consolidation histories; the differences between u_m and u_b are muted by partial equilibration during cycles.

In cyclic tests there is an added complication in that $(u_m - u_b)$ depends on undrained stress history as well; note in Fig. 6.12 how

the difference between u_m and u_b changes with cycling. This results more from a change in the pore pressure generated at mid-height than from any change in the pattern of pore pressure generation at the base.

In monotonic loading tests the difference between u_m and u_b has been shown to increase with increasing strain, see, for example, Blight (1961) who found that $(u_m - u_b)$ reached a maximum at an axial strain of 10% in a compacted Mangla Shale, while Taylor (1944) observed the maximum value for $(u_m - u_b)$ at 4% in tests on Boston Blue Clay. Under cyclic loading, a dependence of $(u_m - u_b)$ on strain is shown in Fig. 6.12.

It is evident that during cyclic loading, pore pressure generation will vary from place to place in the sample. While the magnitude of this variation is difficult to define, it will increase with strain - so will the potential for error in the pore pressure measured at mid-height.

(c) Volume influenced by end restraint

To quantify the effects of end restraint, a simplification is proposed. It is suggested that a triaxial sample can be separated into three volumes as shown in Fig. 6.13(a). In the central volume, V_m , the stresses are considered to be unaffected by end restraint; in the two end sections, of total volume, V_e , the stresses are considered to be significantly influenced by end restraint. The scale of the error introduced by end restraint can then be related to the ratio of V_m to V_e . This ratio can be expected to vary with sample geometry, the degree of end restraint, the rigidity and Poisson's ratio of the platens, the soil's behaviour, and the history of the sample in the triaxial cell.

The theoretical work referred to previously, which has examined stress distributions in cylindrical elements loaded between rough platens, could be used to provide a measure of $V_m:V_e$. However, shortcomings in the methods of analysis must be recognised. Many of the methods use linear elasticity so that the ratio $V_m:V_e$ is unaffected by stress or strain level. In fact, behaviour at the ends

is likely to be plastic or at least non-linear elastic, so that $V_m:V_e$ will vary with strain (or stress) level. Costa Filho (1980) performed non-linear elastic analyses of a triaxial sample which demonstrated this point. At relatively small axial strains Costa Filho estimated the ratio of $V_m:V_e$ to be 4:1. Analyses referred to in Appendix 1 using an elasto-plastic analysis confirm the dependence of end restraint effects on strain level. None of the analyses allow for the possible effects of principal stress rotations on behaviour of the soil; rotations of principal stress directions are large at the fixed ends of a triaxial sample and may lead to a concentration of stress disturbance near to the ends; nor do the analyses allow for local pore pressure dissipation at the ends. Intuitively one feels that many of the reported analyses are exaggerating the effects of end restraint, at least at small strains.

There is experimental evidence from monotonic loading tests which demonstrates that the effects of end restraint become more pronounced with increasing strain, i.e. the ratio of $V_m:V_e$ reduces. For example, Kirkpatrick and Younger (1970) have demonstrated this in terms of the changing distribution with axial strain of axial normal stress at the ends of triaxial samples. Balasubramaniam (1976) has shown, using X-ray techniques, that the effects of end restraint are small up to axial strains of 4%, but strain non-uniformities develop progressively beyond this point. Rowe and Barden (1964) suggest, when comparing free- and fixed-end samples, that there was little indication of non-uniformity in either type of sample at small strains.

Combining the theoretical and experimental evidence then one can anticipate that the potential for error in the pore pressures measured at mid-height is small at low values of axial strain (say $< 4\%$) but increases at higher axial strain.

Under cyclic loading, one might expect that the ratio $V_m:V_e$, and hence the potential for error in pore pressure measurement, changes both progressively with cycling and during each cycle, being different in the loading and unloading stages and in compression and extension stages. The differences in effective stress within a sample at the end of a

cycle means that the interaction of elements with different stiffness, including the plattens, becomes complex. Progressive changes in end effects during cyclic loading are referred to in Appendix 1.

Blight (1961) presents measurements of pore pressures made at five levels in samples which were subjected to cycles of loading and unloading and which throw some light on the development of end effects. He found that loading stages appeared to give rise to greater non-uniformity in pore pressure than the unloading stage. On first loading the profiles of pore pressure were approximately linear, a parabolic shape developing with cycles of loading as the soil behaved more like an elastic material.

A series of tests carried out by Takahashi (1981) can be used to provide an indication of the scale of the potential error introduced by pore pressure non-uniformities, both radial and vertical, and how this changes with cycling. Takahashi carried out undrained cyclic compression tests, using a triangular stress wave, on triaxial samples (38mm diameter by 76mm high) of Lower Cromer Till. He varied both the maximum shear stress applied to the samples and the period for the cycle; pore pressures were measured at the base and the mid-height using the piezometer probe, both during each cycle and during a rest period allowed between each cycle. Details of the tests and samples are summarised in Table 6.2.

Making the simplification shown in Fig. 6.13(a) and assuming a simple vertical distribution of pore pressure at the end of a cycle as shown in Fig. 6.13(b), then examination of the equilibration of the pore pressures (Fig. 6.13(c)) provides a measure of the ratio $V_m:V_e$. The expression relating $V_m:V_e$ to u_m and u_b , the pore pressures measured at the mid-height and base of the sample at the end of the cycle, and u_f , the equalised pore pressure, is derived in Fig. 6.13. It can be seen that when positive pore pressures are generated in the loading cycle, the soil compressibility can be eliminated. It must be stressed that the approach can be indicative only, since equilibration during the cycle, pore pressure changes resulting from undrained creep after loading, and differences in response time for the two measuring systems may

influence the apparent form of the equalisation at the end of the cycle. Furthermore, since equalisation is allowed after each cycle, differences in stiffness in the sample resulting from differences in mean effective stress are removed.

Using this approach, equalisation of pore pressures during a cycle may underestimate the ratio $V_m:V_e$ and so from Takahashi's tests, C-15 has been selected, having the shortest period. Typical variations in measured pore pressures both during and after the cycles are plotted in Fig. 6.14(a) for test C-15. Using the relevant values of u_m , u_b and u_f , values for the ratio $V_m:V_e$ have been calculated; these are plotted in Fig. 6.14(b) against number of cycles and against cumulative axial strain. The plots have not been continued beyond the point at which the pore pressure change in each cycle is so small that the extraneous effects referred to above become significant. The data from test C-13 is qualitatively similar.

It is apparent that:-

- (i) In the first and second cycles the ratio $V_m:V_e$ is high. From this it can be concluded that in monotonic loading tests involving only small strains, the influence of end restraint on the pore pressures measured at mid-height is negligible.
- (ii) As cycling proceeds, the ratio $V_m:V_e$ reduces but does not fall far below a value of 4. It is of interest that this is the value estimated by Costa Filho (1980).
- (iii) The reduction in $V_m:V_e$ appears to be related to the increase in cumulative axial strain; this corresponds to the evidence from monotonic loading tests from which it could be inferred that $V_m:V_e$ reduces with increasing strain.
- (iv) After a large number of cycles, the value of $V_m:V_e$ oscillates; this reflects the reducing pore pressure changes in each cycle and the increased error in the approach used.

- (v) The time for equalisation reduces with increasing number of cycles; this probably reflects changes in the soil's compressibility.

A similar series of tests is required in which pore pressures are monitored after fast cycles of compression-extension loading. Although pore pressures were monitored after completion of compression-extension cycling tests reported in Chapter 10, the cycle period of 30 mins was too long to allow a reasonable estimate of $V_m:V_e$ to be made. However, it was apparent that after cyclically loading anisotropically consolidated samples, pore pressures tended to equalise to the value measured at the mid-height at the end of cycling when cumulative strains were compressive; in contrast, when cumulative extension occurred, the equalised pore pressures tended towards the value measured at the base at the end of the cycle. This last observation is regarded as particularly important. It tends to confirm that when strains are relatively large the ratio $V_m:V_e$ is markedly different in extension compared with compression. The observation is probably related to the basically unstable nature of extension loading, in which any local reduction of diameter leads to 'waisting' and to a concentration of stress and strain and to acceleration of the development of non-uniformities. 'Waisting' is likely to be cumulative during cycles of extension loading so that pore pressures measured in the 'waisting' zone, while possibly correct, become less typical of the pore pressures in the whole sample.

(d) Equalisation

For monotonically loaded undrained triaxial compression tests, Gibson (quoted in Bishop and Henkel, 1962) has investigated equalisation of the pore water pressure difference between the ends and mid-height of the specimen. For ease of reference in subsequent paragraphs, the assumptions and results of Gibson's analysis are summarised in Fig. 6.15. Using the theoretical relationship obtained by Gibson, the time to achieve a desired degree of equalisation between the pore water pressures can be found. The rate of monotonic loading (or of displacement) can then be selected to ensure that this degree of equalisation is achieved at the appropriate stage of a test.

Considering symmetrical compression-extension cyclic loading, then Gibson's analysis deals only with the first quarter cycle, and only then if the cyclic loading involves triangular waveforms, i.e. constant rates of loading and unloading. The equilibration which takes place during the first quarter cycle of a test can be estimated using one quarter of the cyclic period in Gibson's solution. For the Lower Cromer Till, tested without paper drains and taking an average C_v of $2m^2/\text{year}$, the degrees of equalisation occurring at the end of the first quarter cycle have been estimated for different cycle periods and these are plotted in Fig. 6.16. It is evident that significant equalisation only occurs when cycle periods exceed 10 mins. For subsequent cycles, increases in C_v will reduce this period.

For cycle periods that allow 95% equalisation at the end of the first quarter cycle, it is reasonable to assume that at least this degree of equalisation is achieved throughout the cyclic loading. In this case, the error in the pore water pressure, measured either at the base or mid-height, is a function only of the ratio $V_m:V_e$ and the difference in pore pressures generated at the base and mid-height, $(u_b - u_m)$. For intermediate cyclic periods, the error is less and becomes a function of $V_m:V_e$, $(u_b - u_m)$ and the degree of equalisation. Some estimate of equalisation is required.

Analysis of the equalisation of pore pressures which occurs during cyclic loading is considered in Appendix 2 to this Chapter. The measured pore pressures shown in Fig. 6.12 illustrate that during a cycle, the mid-height and base pore pressures, u_m and u_b , first diverge then converge and the value of $(u_b - u_m)$ may reverse. This pattern changes as cycling proceeds. As an alternative to examining the progressive equalisation during each cycle, a simplification can be made.

As a result of cyclic loading, pore pressures accumulate. Cyclic variations in pore pressure are superimposed on an average pore pressure which increases with time, see Fig. 6.17. While cyclic frequencies may preclude significant equilibration between the mid-height and ends of a sample, they cannot prevent equalisation occurring as cycling proceeds. Fig. 6.6 illustrates that the cumulative pore pressure measured at mid-height develops more-or-less linearly with time. Fig. 6.12 shows that

the pattern of pore pressure generation at the base remains relatively unchanged as cycling continues and is consistent with undrained oedometric conditions. If there were to be no equilibration of pore pressure non-uniformities in the sample then the measured pore pressures at the base and mid-height would be of the form shown in Fig. 6.17(a). If the cyclic fluctuations in pore pressure are removed, then two diverging lines result; the problem of equalisation is then equivalent to that analysed by Gibson, compare Figs 6.17(b) and 6.15.

Equalisation of the cumulative pore pressure occurs after time t_e . Approaching t_e , the error in the cumulative pore pressure measured at mid-height and in its development with time (α_1 c.f. α_2) increases. Beyond t_e , the development of mean pore pressure can be assessed from pore pressure measurements made at the base. This has important implications for the analysis of existing data in which pore pressures were measured only at the base. However, the error in the cumulative pore pressure increases as the test proceeds and a limit to the number of cycles and/or test duration should be set. After t_e the error ($\alpha_1 - \alpha_2$) is a function of $V_m : V_e$.

It can be seen, then, that Gibson's approach, summarised in Fig. 6.15, can be applied to pore pressure measurement in cyclic loading both to:-

- (i) estimate the degree of pore pressure equalisation taking place during a cycle, and hence assist in making an estimate of the error in the measurement of the cyclic component of pore pressure; and
- (ii) estimate the time t_e , after which a similar rate of pore pressure build-up is measured at the base and mid-height and after which the full error in the measurement of rate of cumulative pore pressure development is established.

There will be a tendency for the approach to overestimate t_e and underestimate the equalisation which takes place during a cycle, because the assumption of a parabolic distribution of pore pressure is not met.

At small strains, i.e. at the start of cycling especially with low cyclic stress levels, the ratio of $V_m:V_e$ is small and pore pressure distributions are more rectangular in form.

In Fig. 6.18, pore pressure measurements at the base and mid-height are compared for similar samples cyclically loaded at similar stress levels but with different periods. For the cycle with a period of 240 min., it is evident that equalisation in the first quarter cycle is somewhat higher than the 56% predicted from Fig. 6.15.

Calculations for the time t_e must take account of changes in C_v as cycling proceeds. For the relatively long period used for the experiments reported in Chapter 10, namely 30 mins, only a very limited number of cycles is necessary before the same cumulative pore pressures are registered at the base and mid-height. This is confirmed in Fig. 6.19 for samples with a variety of consolidation histories.

6.9 CONCLUSIONS

The measurement of pore water pressure in laboratory triaxial samples loaded transiently or cyclically at typical field frequencies should be made using a piezometer placed at the mid-height of the sample. To minimise the volume of soil in which stress perturbation is caused by the piezometer, then it should be placed on the sample's circumference.

The silicon integrated pressure transducer is particularly suitable for measurement of pore water pressures at the periphery of samples undergoing cyclic or transient loading. It has an extremely small compliance and, therefore, a rapid response time even with clays with low coefficients of consolidation. It has a small face, reducing interference effects, and is convenient to install on most sizes of sample, including the widely used 38mm diameter by 78mm high sample.

These transducers have been in use as a mid-height piezometer for a period of six years during which two operating difficulties have been encountered: interference occurs between the porous stone and silicon

diaphragm at high effective stress ($>260 \text{ kN/m}^2$ for the particular models used) and gas is generated internally. However, they have been used successfully to monitor pore pressures in transient loading tests on Lower Cromer Till with times to failure as short as 2 secs and in cyclic tests on the same soil with periods down to 10 min. With a response time of well under 1 second with this soil (C_v of 1 to $2\text{m}^2/\text{year}$) pore pressures could, in theory, be monitored in cycles with periods of 20 secs or less.

The following conclusions may be drawn regarding the validity of the pore pressures measured with the mid-height piezometer. Considering first monotonic loading tests carried out at different loading rates then:-

- (i) At the faster rates, i.e. with times to failure in the Lower Cromer Till of less than, say, 30 secs, there is a possibility that measured pore pressures are influenced by interference between the piezometer and the soil. In saturated soils pore pressure changes, smaller than 10 kN/m^2 may be superimposed on the real pore pressure changes; in partially saturated samples this figure may be higher. Also, with these short times to failure radial non-uniformities of pore pressure will not have time to equalise. Except in essentially granular soils, these radial non-uniformities in pore pressure are small at mid-height and unlikely to significantly invalidate the measurements.
- (ii) At slower rates of loading, equalisation of the pore pressures set up by interference do not invalidate the measured pore pressure because of the small soil volume affected by the piezometer.
- (iii) With changing rate of loading, there is a changing degree of equalisation of the pore pressure non-uniformities set up by end restraint, see Fig. 6.8. The error in the measured pore pressure increases with increasing degree of equalisation. The error is related, however, to the volume of the sample which is influenced by end restraint, V_e , relative to the total sample volume. This ratio is low at small axial strains. Many of

the important differences in behaviour with loading rate which are reported in Chapter 9, reveal themselves at axial strains of less than 1%; the corresponding differences in pore pressure generation with loading rate are considered to be reliable at strains up to 4%.

Provided that deformations are measured internally, then in monotonic loading tests at different rates the use of lubricated ends is to be recommended since the viscous nature of the lubrication only presents problems at fast rates of shear when equilibration of pore pressure non-uniformities does not occur. However, differences in failure mode may be caused by end restraint in transient tests. The use of long samples also increases the ratio $V_m:V_e$ and reduces the error in the measured pore pressure at mid-height.

Considering next cyclic loading tests:-

- (i) At high frequencies, pore pressures resulting from interference between the piezometer and soil may be superimposed on the true pore pressure changes. While this superimposed pore pressure is small initially, it may increase in magnitude when the sample is deforming with large strains and interference may increase.
- (ii) As a result of end restraint, different pore pressures are generated at the base and mid-height of samples under cyclic loading. This difference in pore pressure depends on drained and undrained stress history; pore pressures generated at the base suggest that oedometric conditions prevail there. During cyclic loading there is continuing equalisation of pore pressure non-uniformities developed by end restraint. At a predictable time after the start of cycling, progressive equalisation results in a similar cumulative pore pressure being measured at the mid-height and base of a sample (Fig. 6.19). Prediction of this time and of the equalisation of non-uniform pore pressures occurring during each cycle can be made using an approach developed by Gibson for monotonic loading tests; allowances must be made for changes in C_v with cycling.

- (iii) The influence of end restraint changes with cycling. Initially, providing that strains are small ($\epsilon_a < 1\%$), the volume of the sample which can be considered to be influenced by end restraint is also small. Errors resulting from different pore pressures being generated in the sample are insignificant. As cycling proceeds, and strains accumulate, this volume increases and an error in the measurement results. However, no evidence has been found which suggests that the volume of soil influenced by end restraint becomes so large that the pore pressure measurements are invalid. In extension cycles involving large strains there may be a tendency for cumulative 'waisting' which will reduce the validity of the pore pressure measured at mid-height.
- (iv) The error in the measured cumulative pore pressure increases with cycling so that a limit on the duration of the test should be considered.

In the foregoing discussions, the soil has been considered to be a continuum in which there are not intrinsic pore pressure inhomogeneities. In fact, as discussed in Chapter 11 such inhomogeneities are considered likely to occur when the Lower Cromer Till is loaded rapidly. Pore pressure observations made with the piezometer are used to substantiate this view.

Overall Length	11.4 mm
Overall Diameter	6.3 mm
Weight (excl. cable)	~ 2 gms
Transduction principle	4 arm silicon strain gauge
Output voltage (FSO)	200 mV/10 V energisation
Non-linearity	$\pm 0.15\%$ of full scale output
Pressure range	0 - 690 kN/m ²
Resolution	0.1 kN/m ²

Table 6.1 Characteristics of the miniature silicon diaphragm pressure transducer.

Test No.	Cycle Period (secs)	Stress Level q (kN/m ²)	wc (%)
C-11	900	50	15.64
C-12	900	75	15.75
C-13	100	50	15.53
C-15	100	75	15.71

All samples were isotropically consolidated to σ'_c of 300 kN/m² (back pressure of 200 kN/m²) after being cut from anisotropically consolidated soil block (OCR = 4, $\sigma'_{v \max}$ = 120 kN/m²).

Table 6.2 Details of samples used in cyclic compression tests.

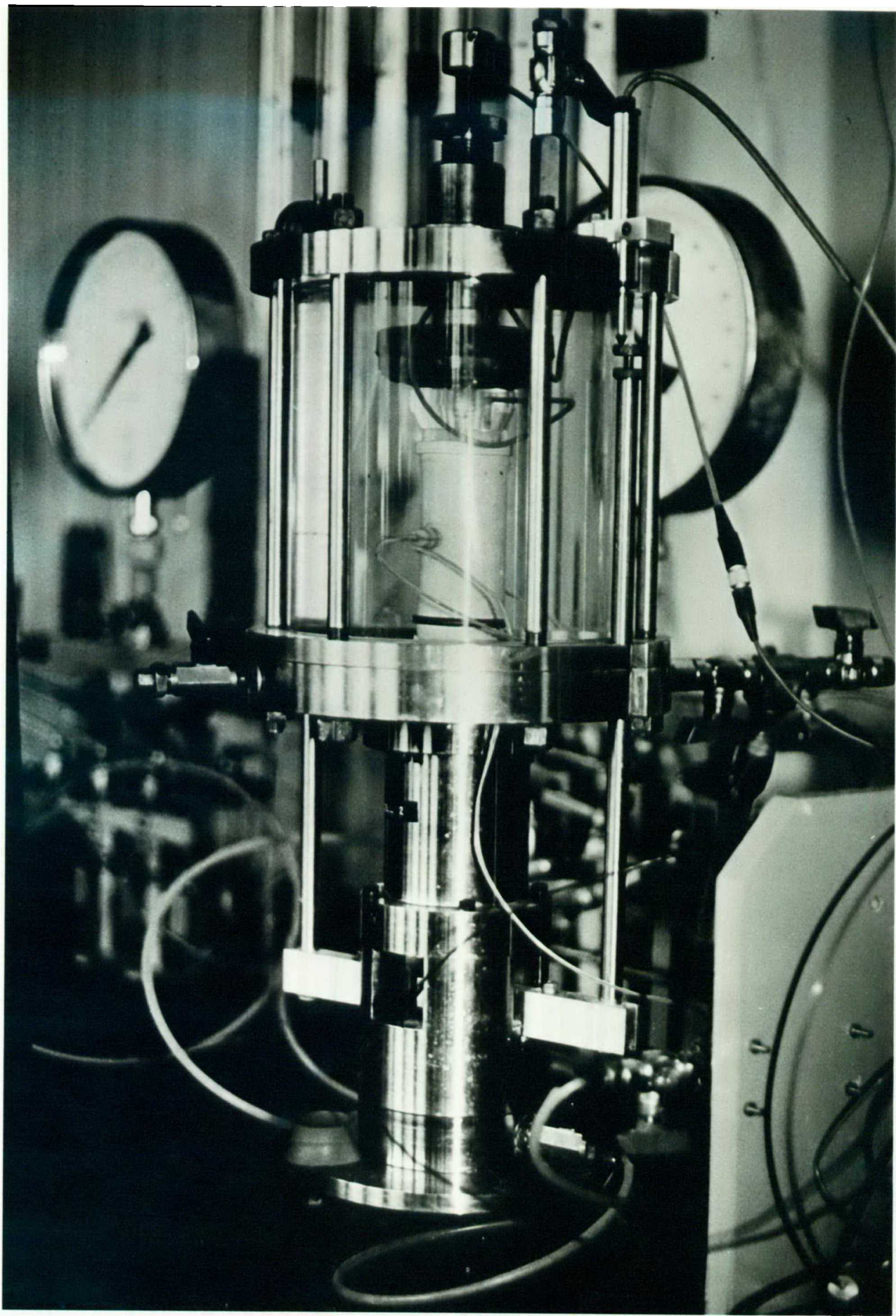


Plate 6.1 Piezometer probe on triaxial sample.

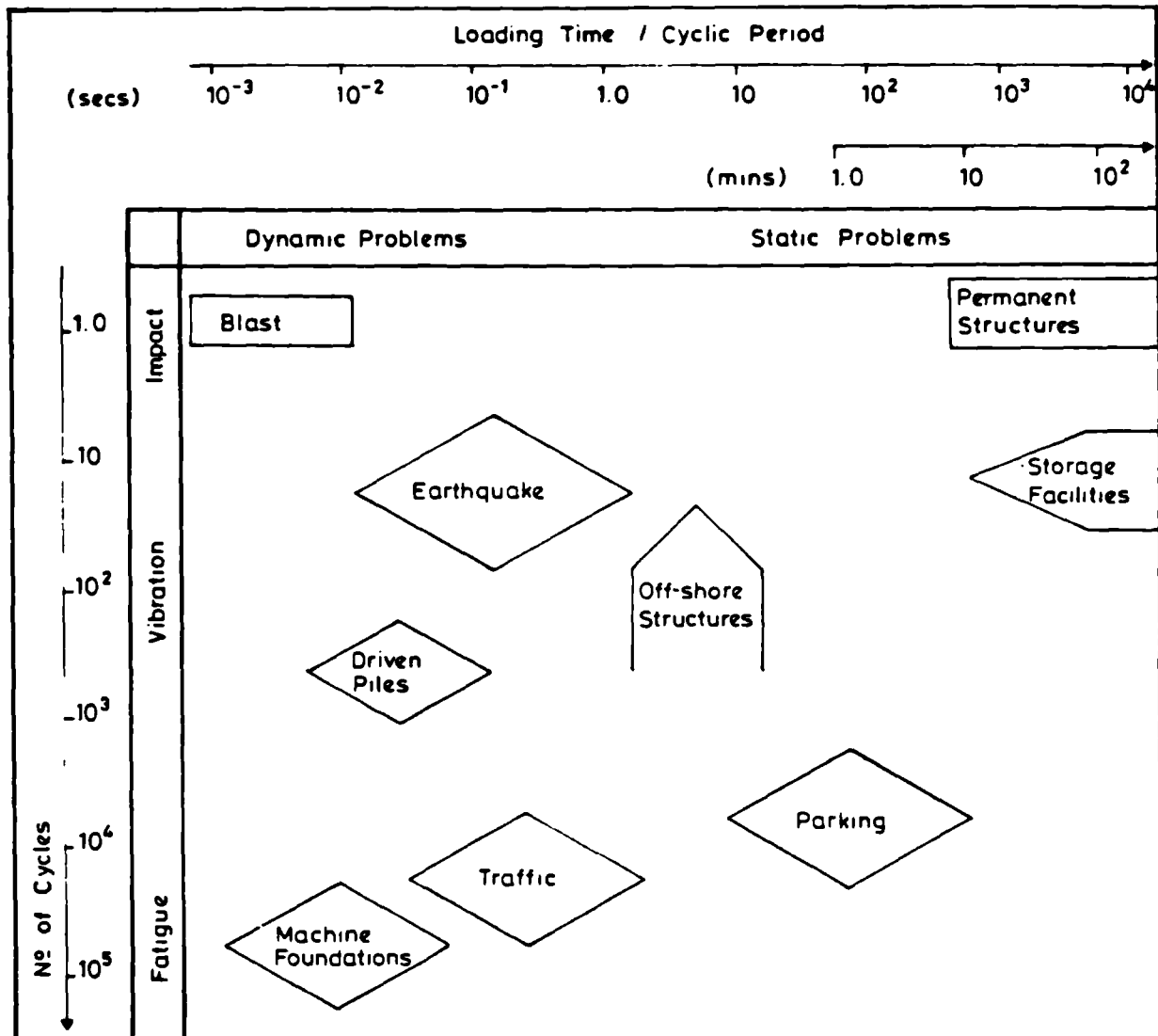
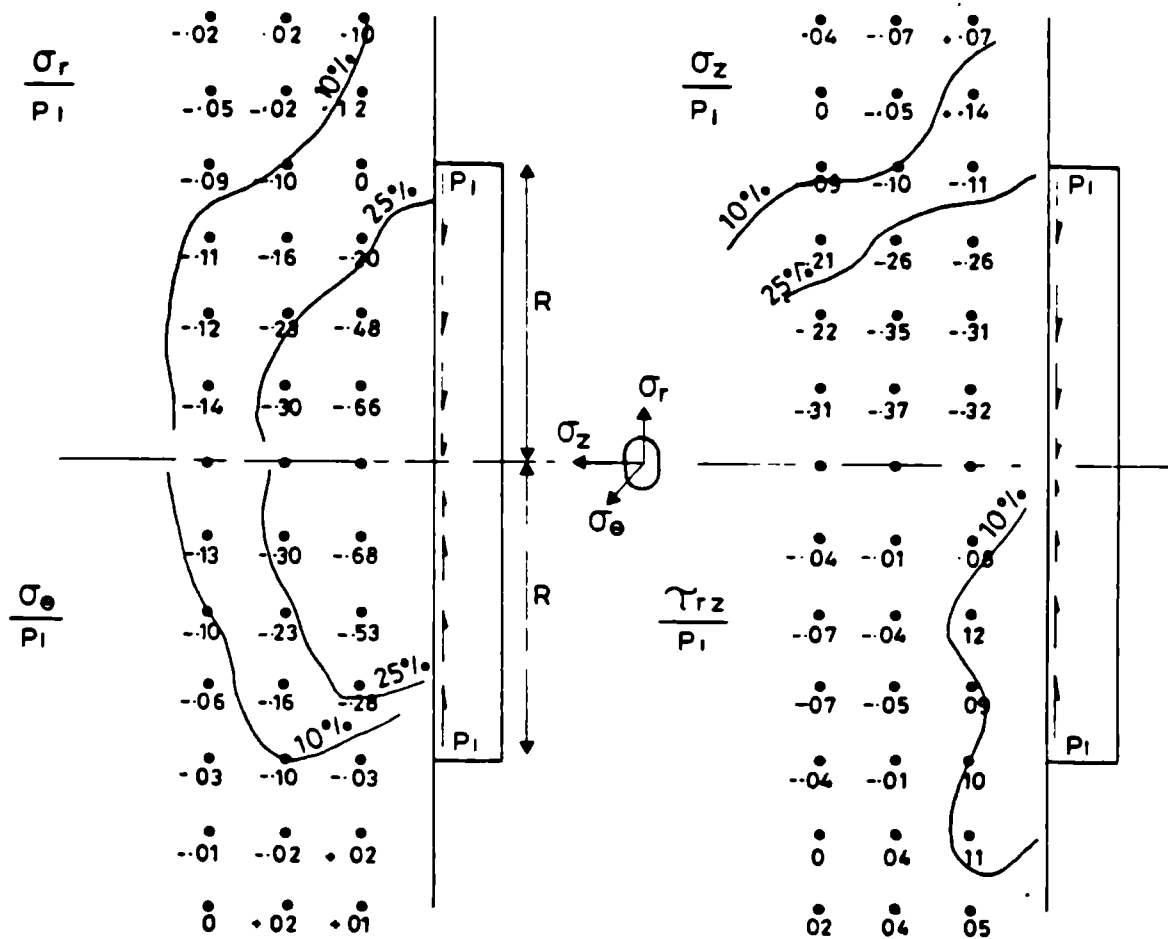
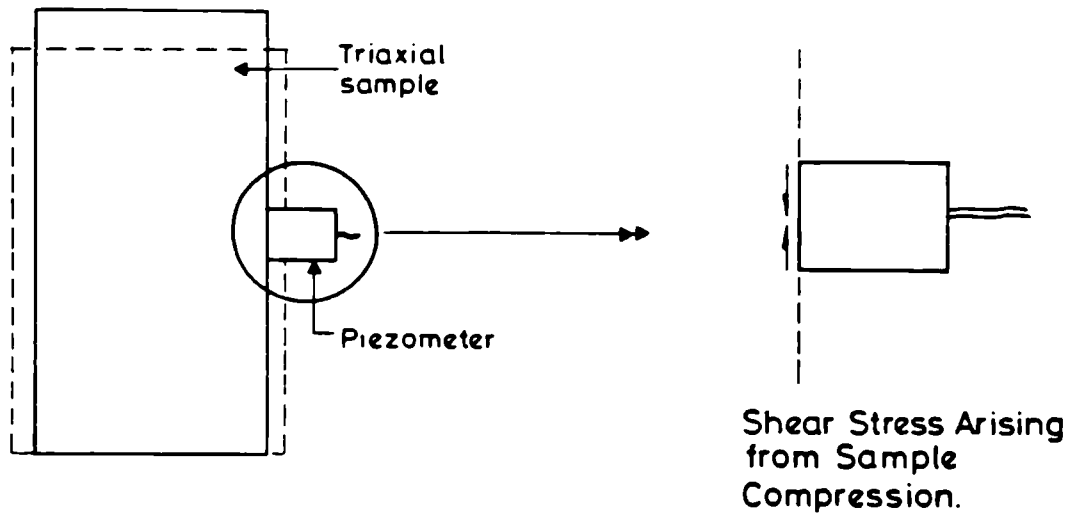


Fig. 6.1 Loading time/cyclic period and number of cycles for typical field problems.



Stress distributions for linear radial shear stress (Gerrard and Harrison, 1970)

Fig. 6.2 Interference arising from a rigid piezometer at the periphery of a triaxial sample.

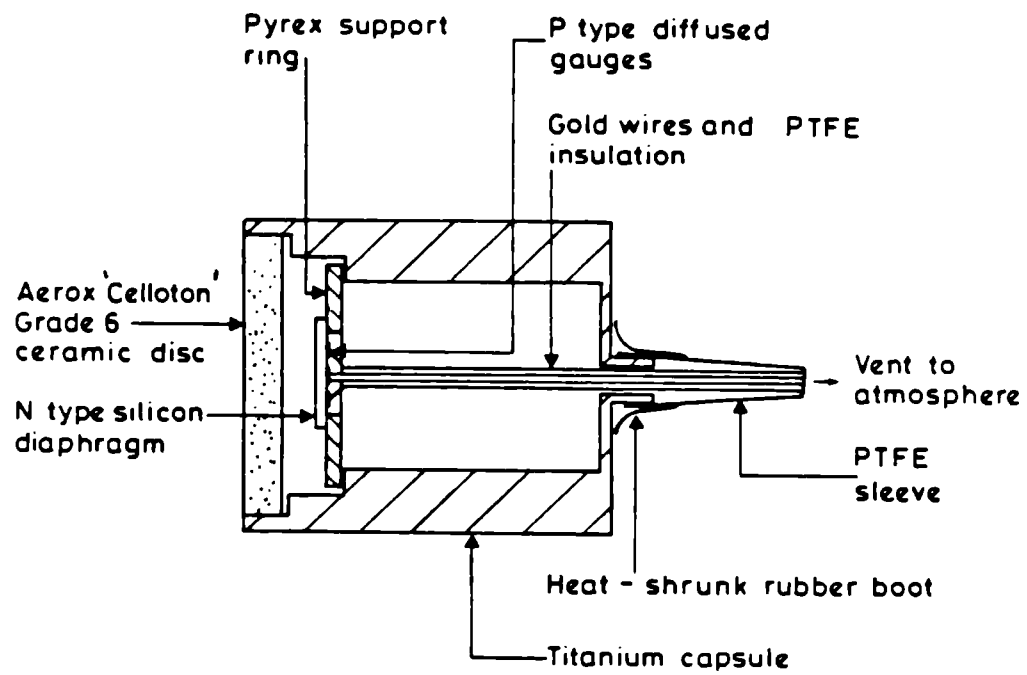


Fig. 6.3 Details of the miniature pressure transducer.

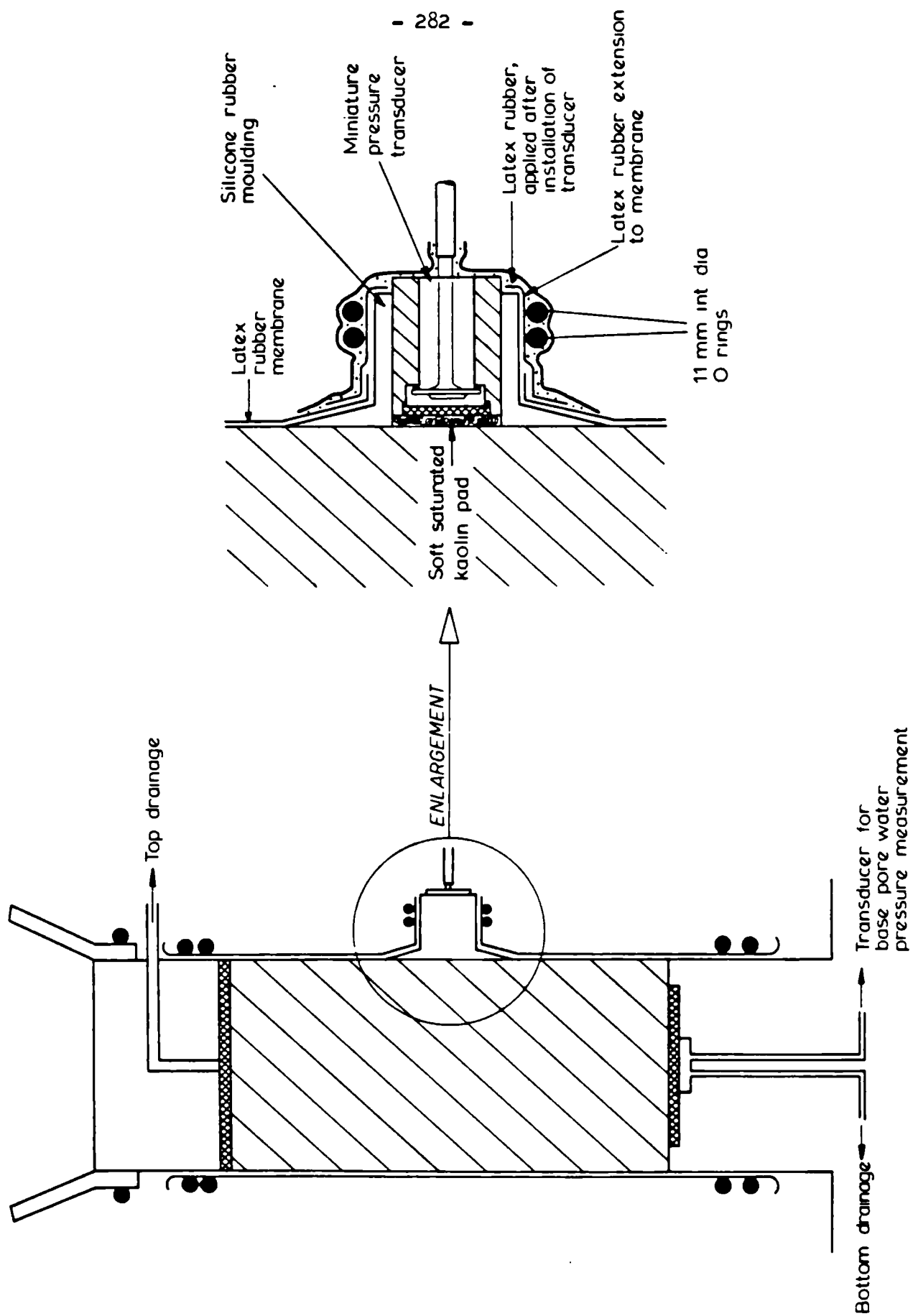


Fig. 6.4 Installation details for the piezometer probe.

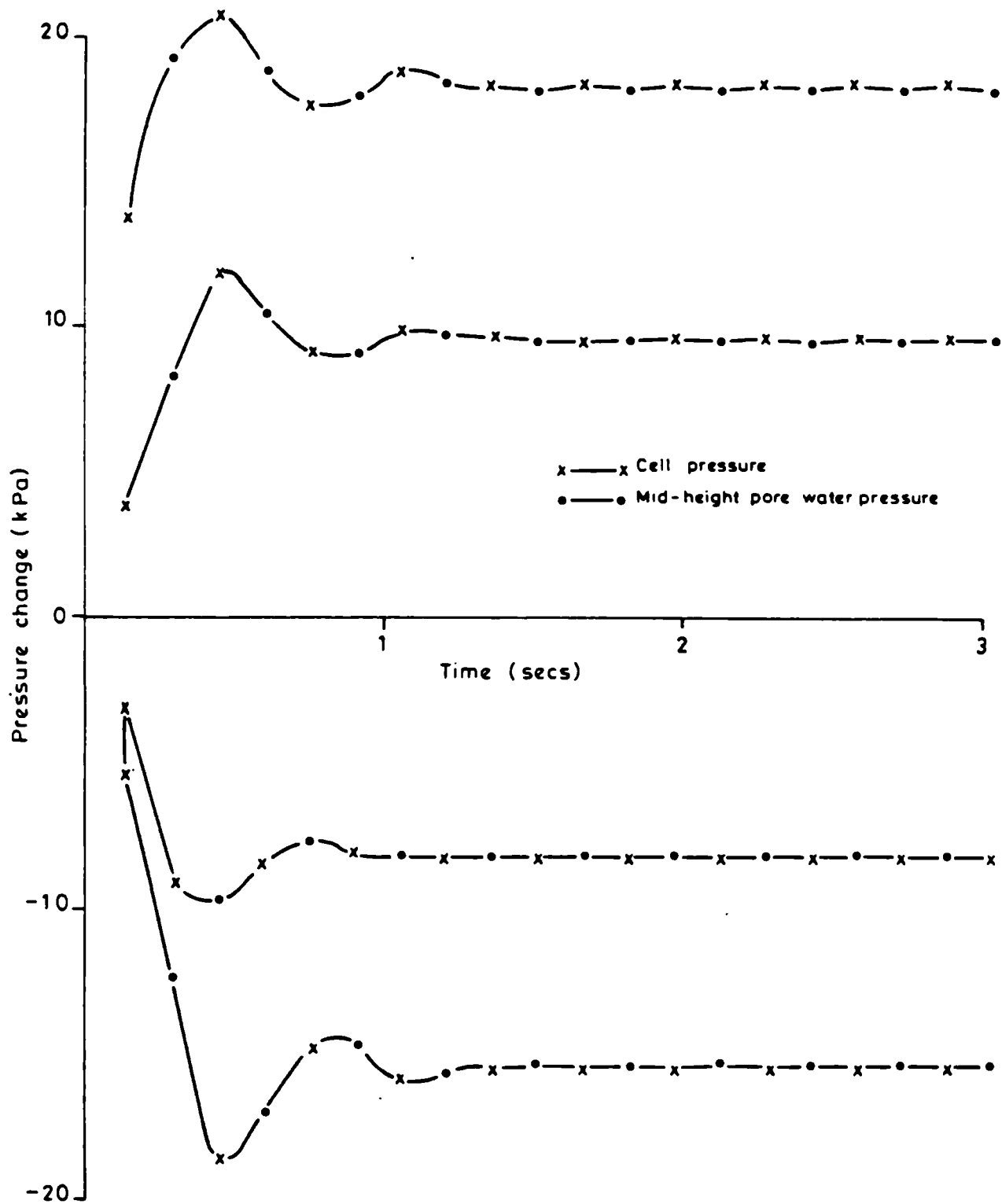
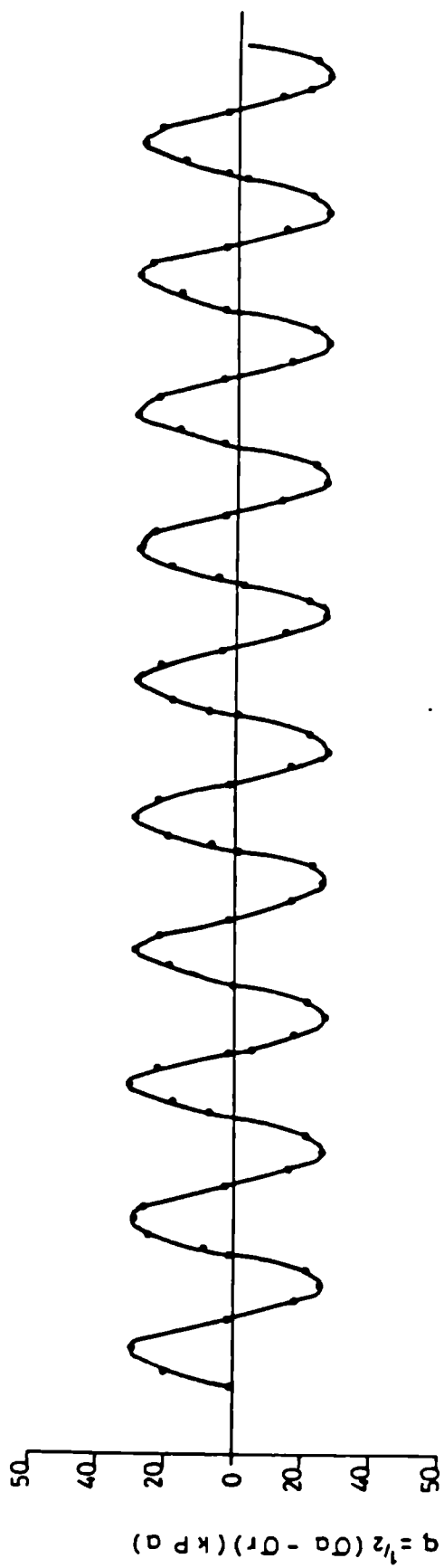


Fig. 6.5 Response of the piezometer probe to an undrained change in cell pressure (saturated sample of Lower Cromer Till).



Isotropically consolidated sample OCR = 1
Cyclic stress level = 0.7

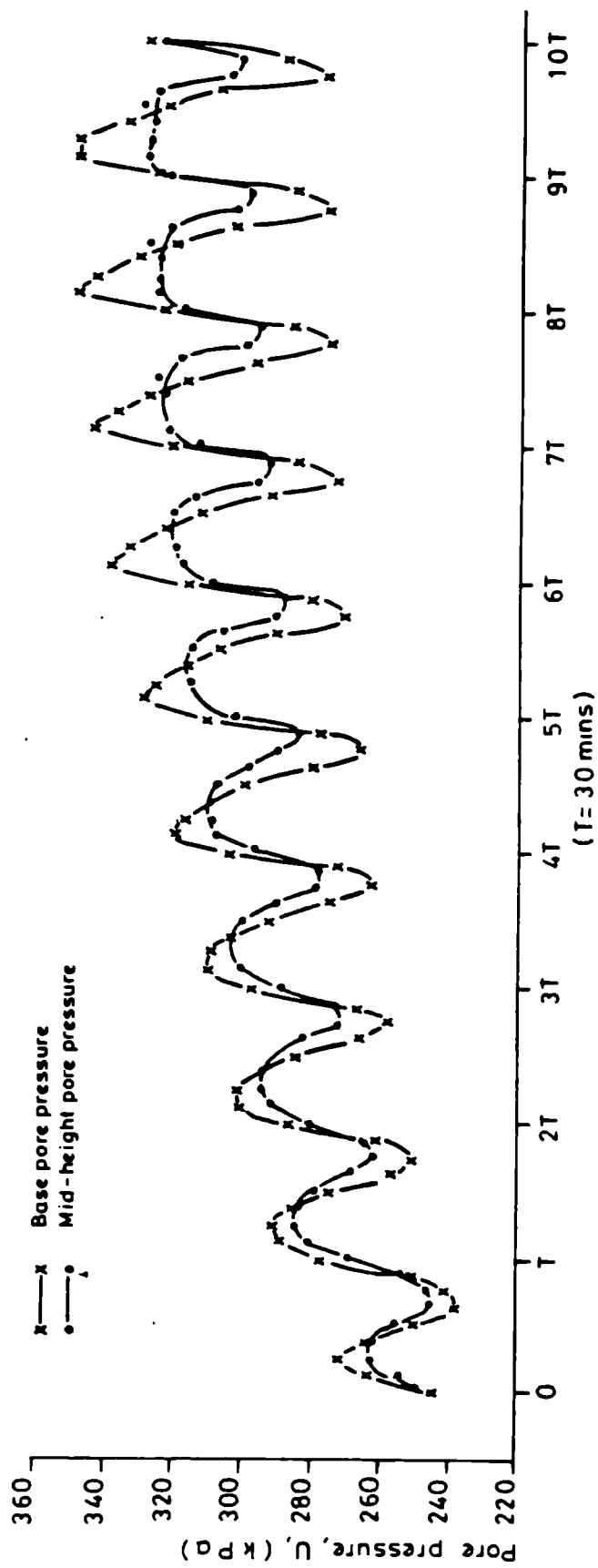
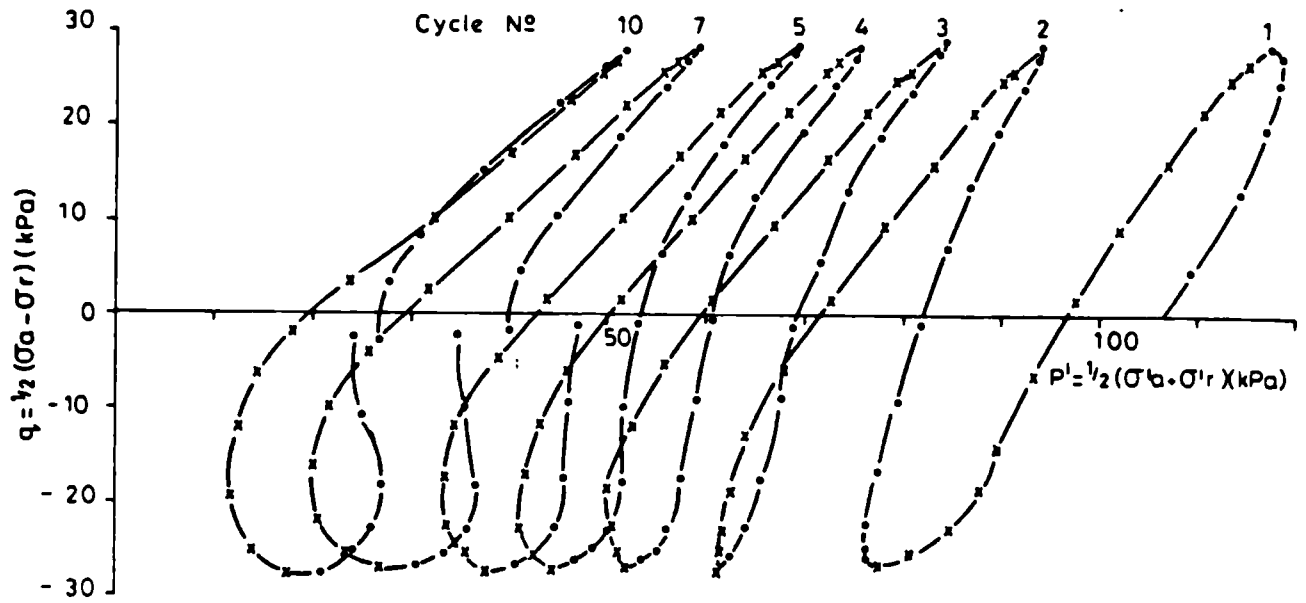
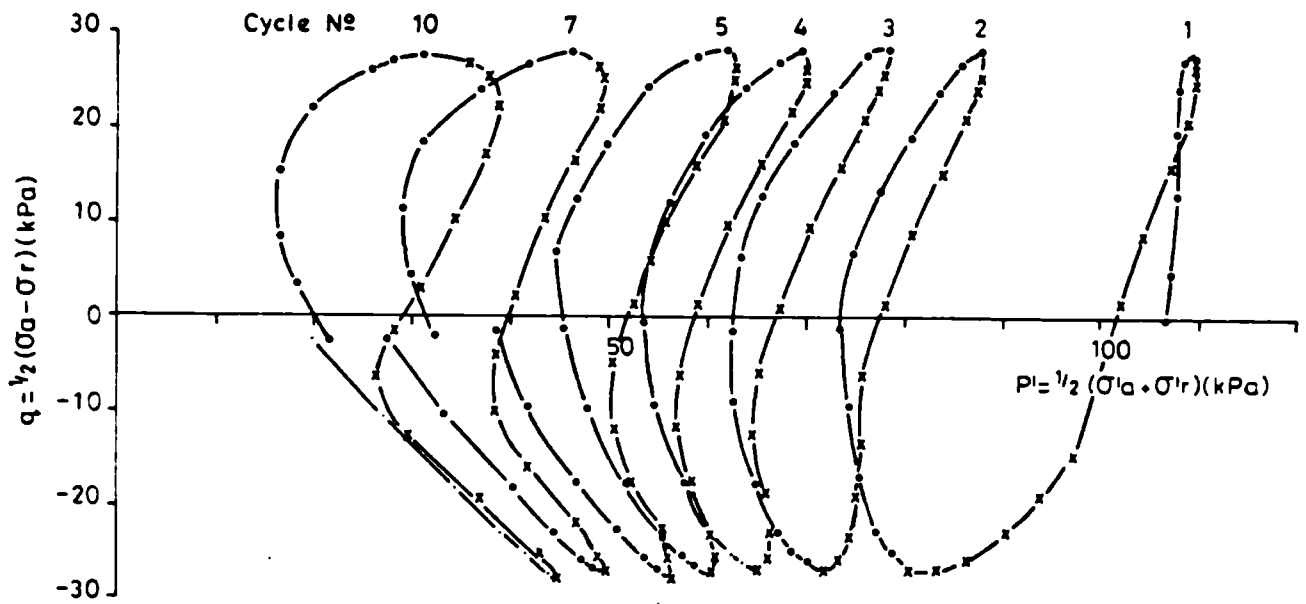


Fig. 6.6. Pore pressure measurements in a cyclic test.



Effective stress paths based on pore pressures measured at mid-height



Effective stress paths based on pore pressures measured at base

Fig. 6.7 Effective stress paths in a cyclic test.

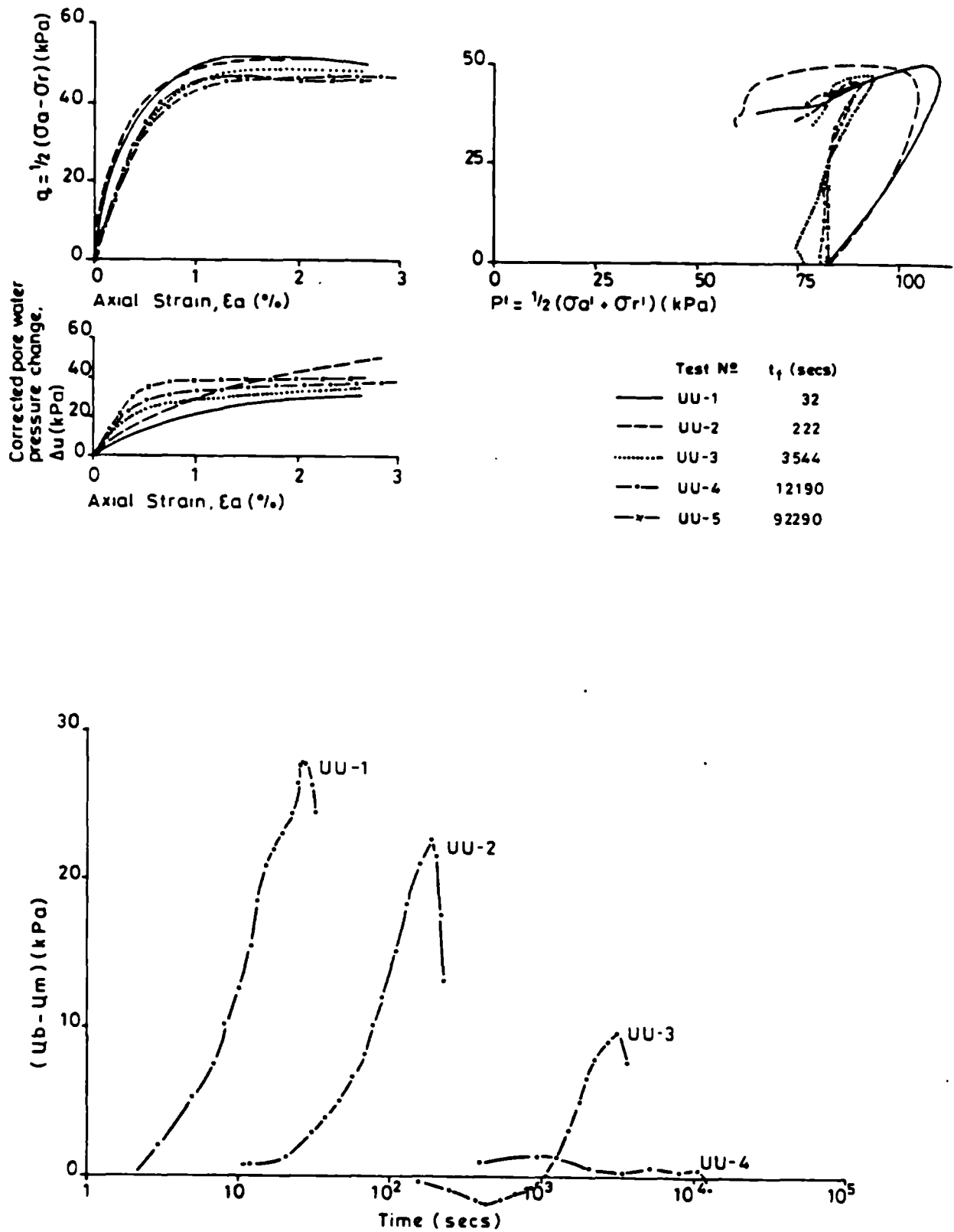


Fig. 6.8(a) Pore pressure measurements in transient loading tests on Lower Cromer Till and the change in $(u_b - u_m)$ with rate of loading.

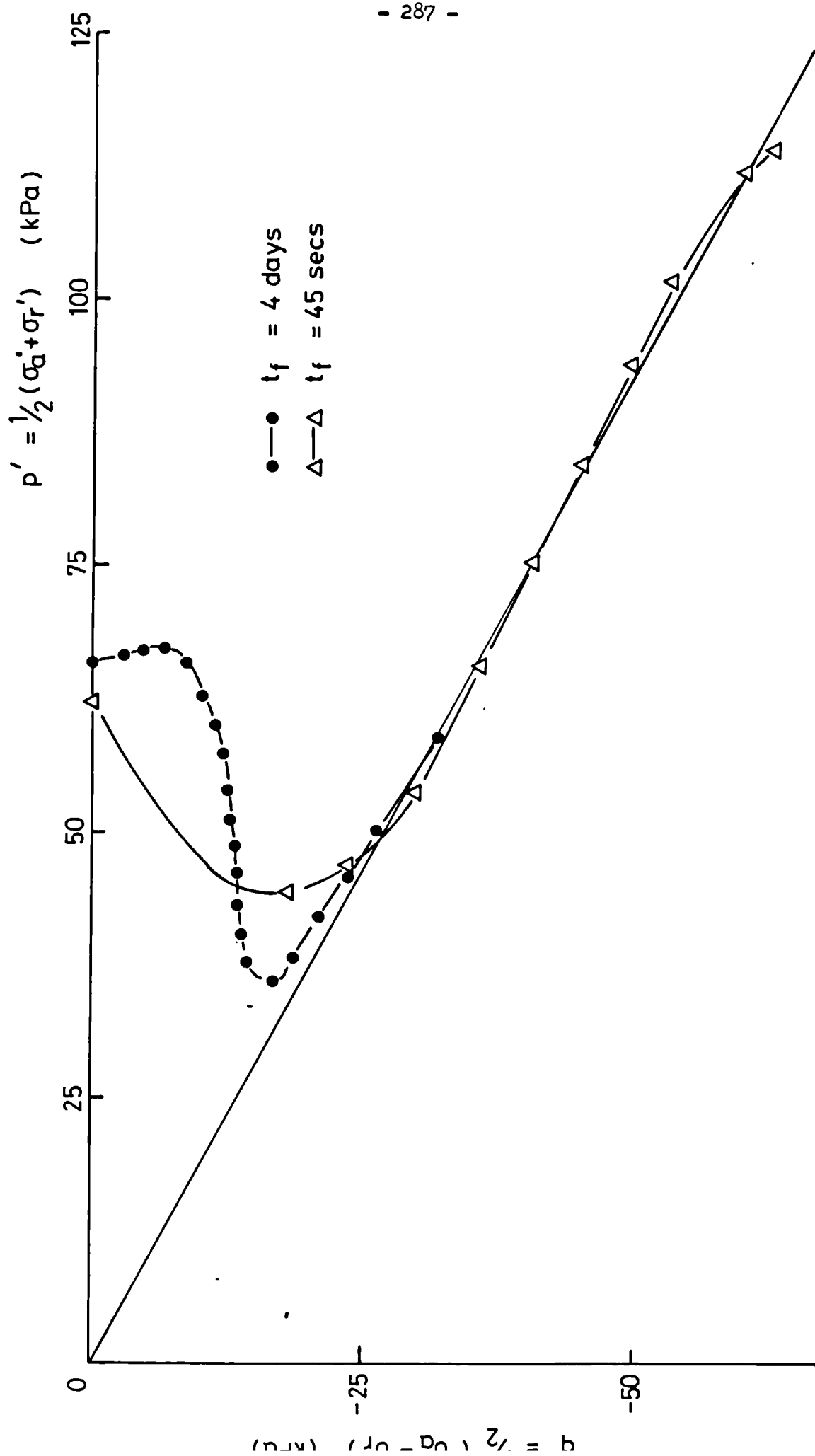


Fig. 6.8(b) Fast and slow extension tests on Portavadie silt. E.s.ps based on probe measurement. (Data from De Campos, 1983)

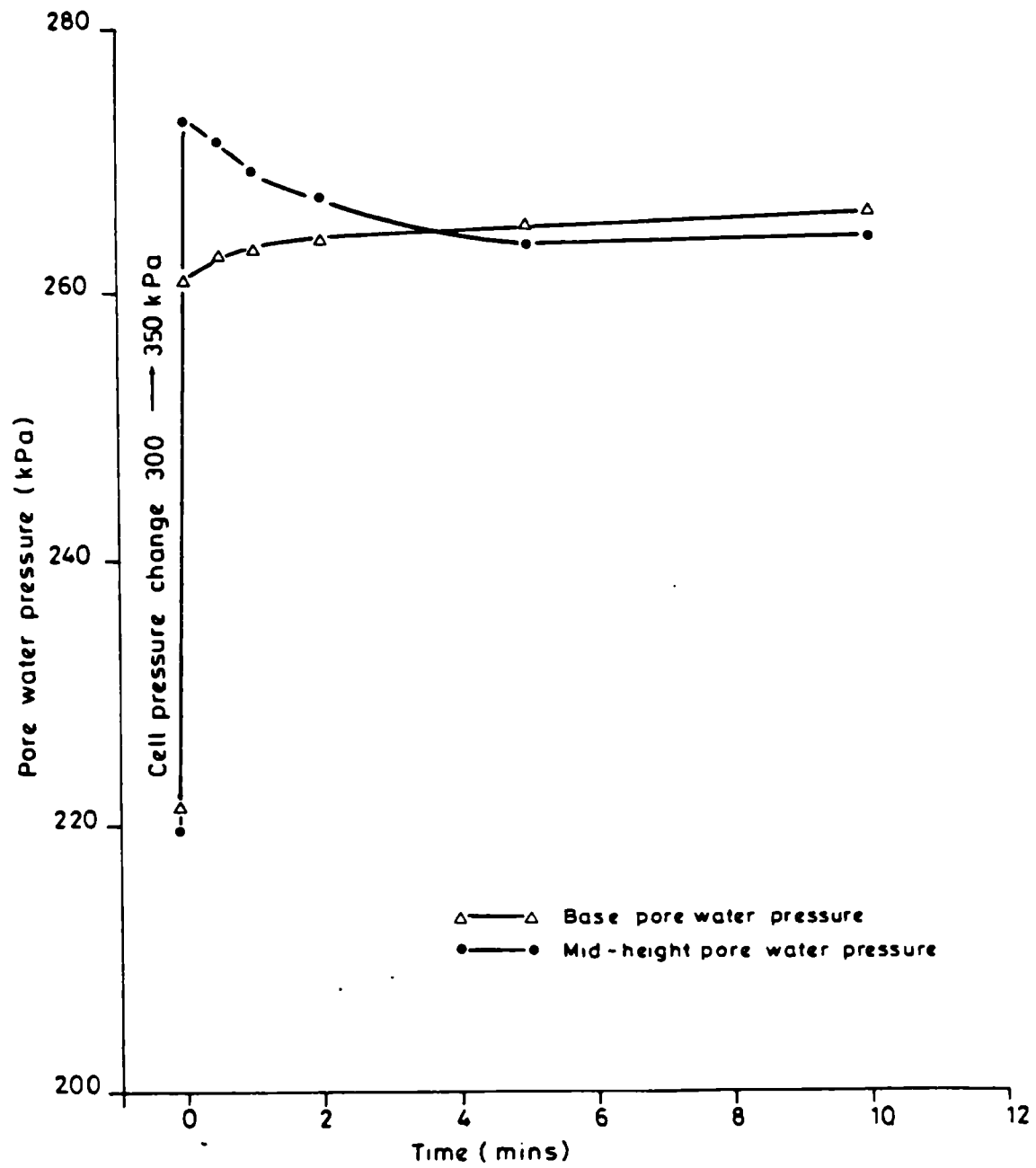
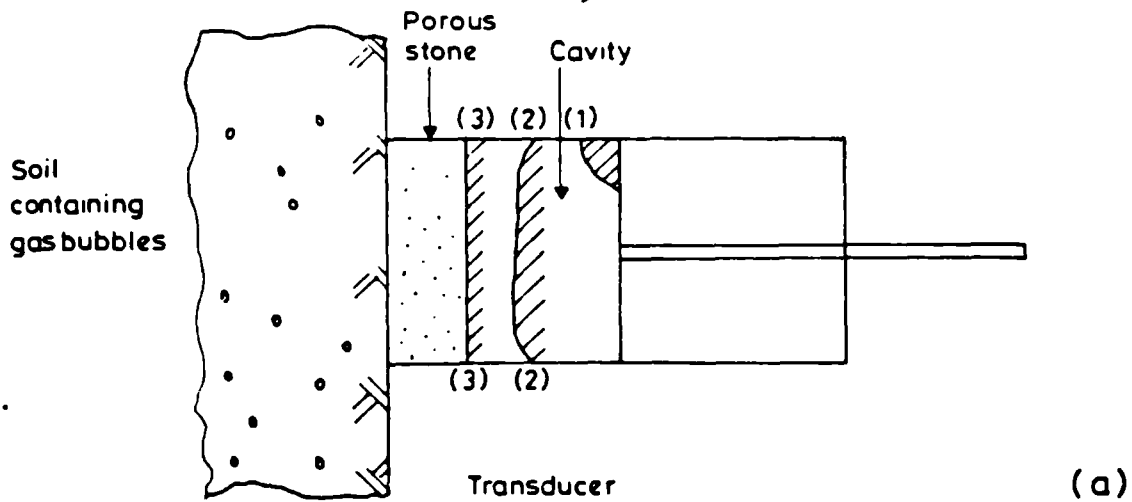


Fig. 6.9 Interference effects in a partially saturated sample subject to an undrained change in cell pressure.



Stage :

- (1) Gas bubble existing after incomplete de-airing .
- (2) Gas bubble growing with diffusion
- (3) Gas bubble filling cavity

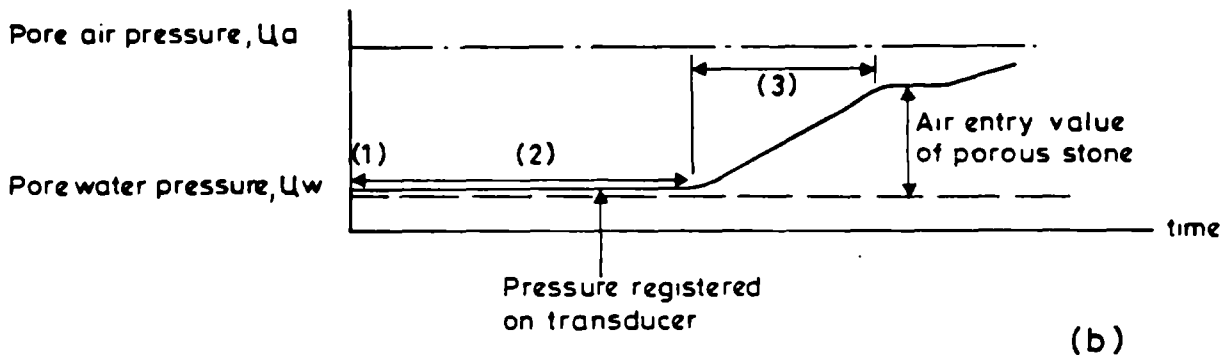


Fig. 6.10 Potential gas problems associated with the miniature pressure transducer.

- (a) growth of gas bubble
- (b) change in registered pressure with time

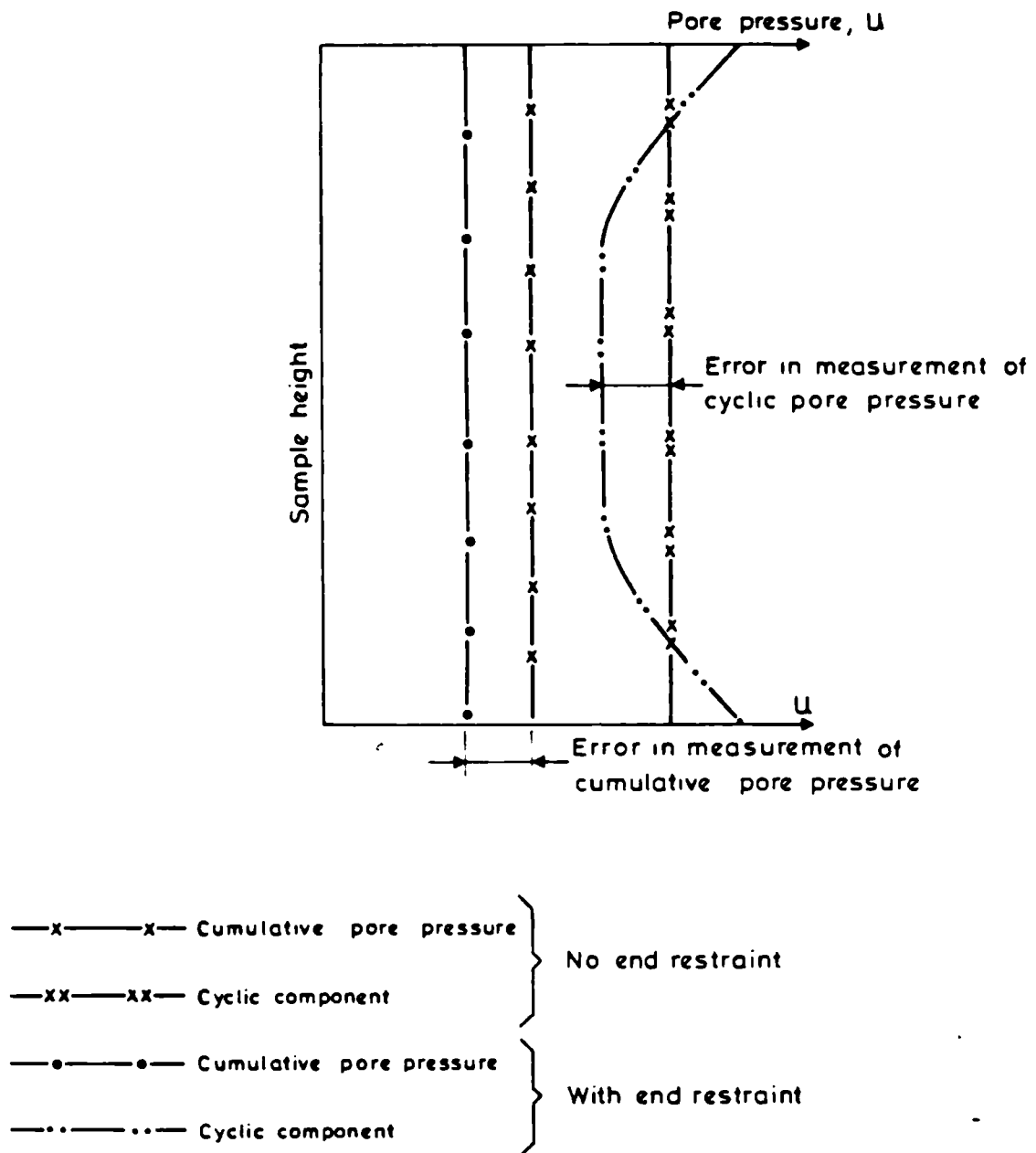


Fig. 6.11 Errors in pore pressure measurement in cyclic tests resulting from end restraint.

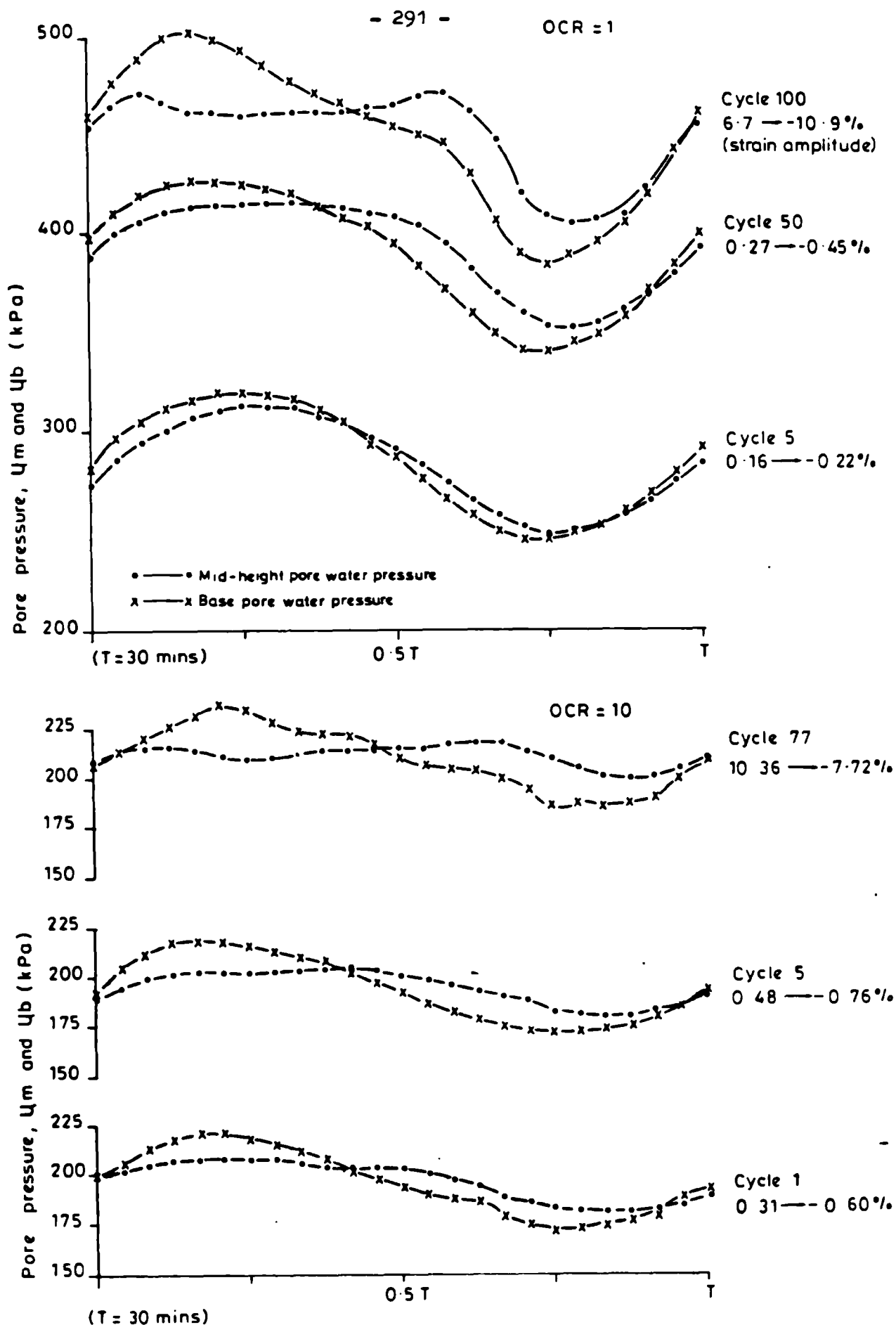


Fig. 6.12(a) Pore pressure differences at base and mid-height during cyclic loading. Isotropic consolidation history.

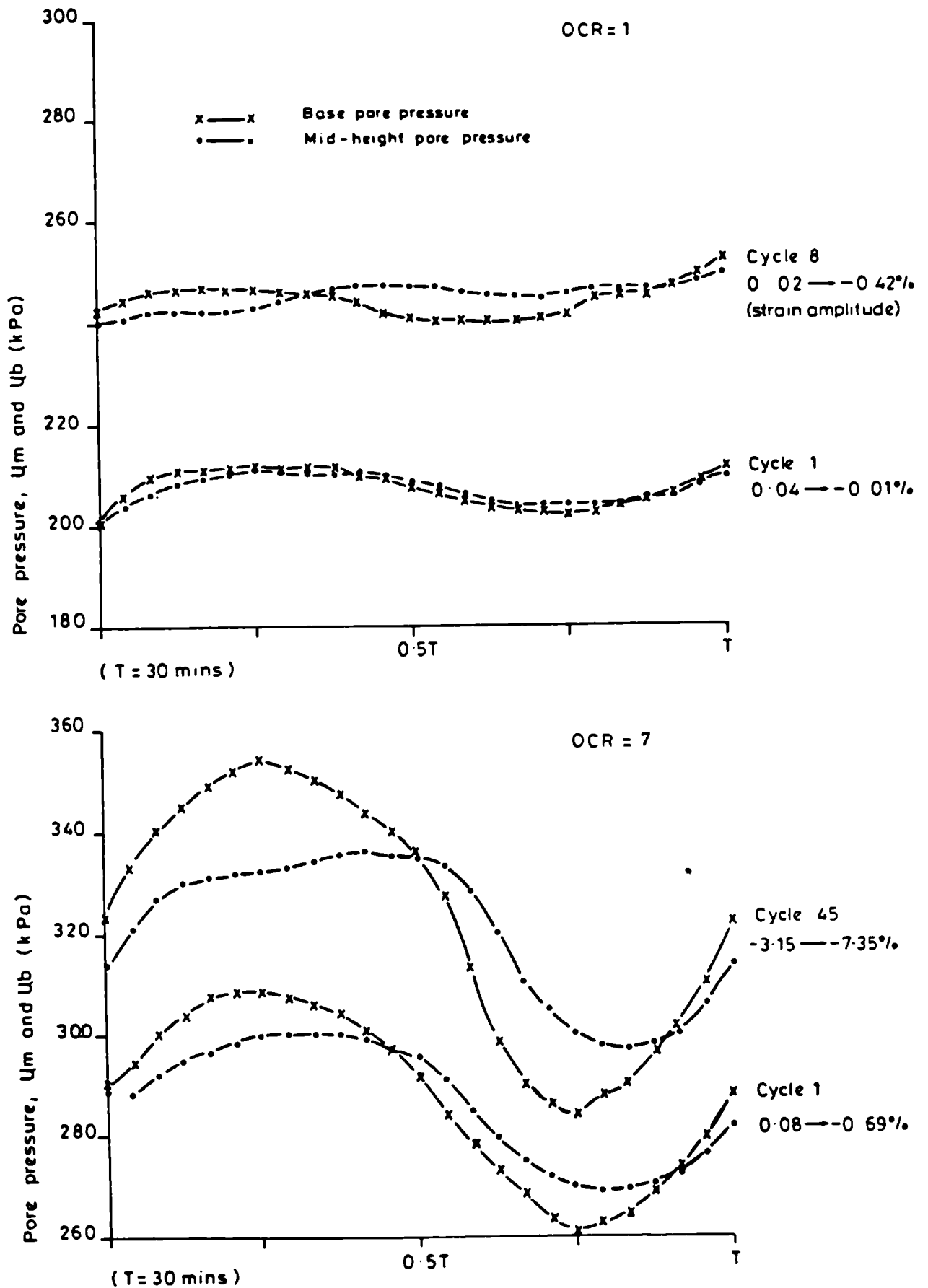
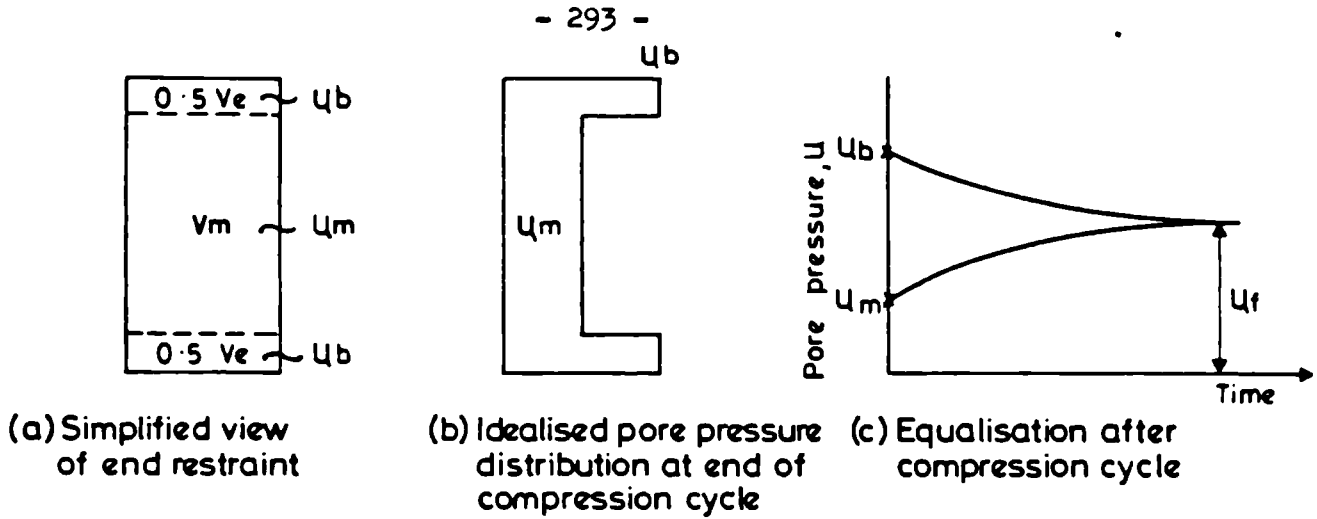
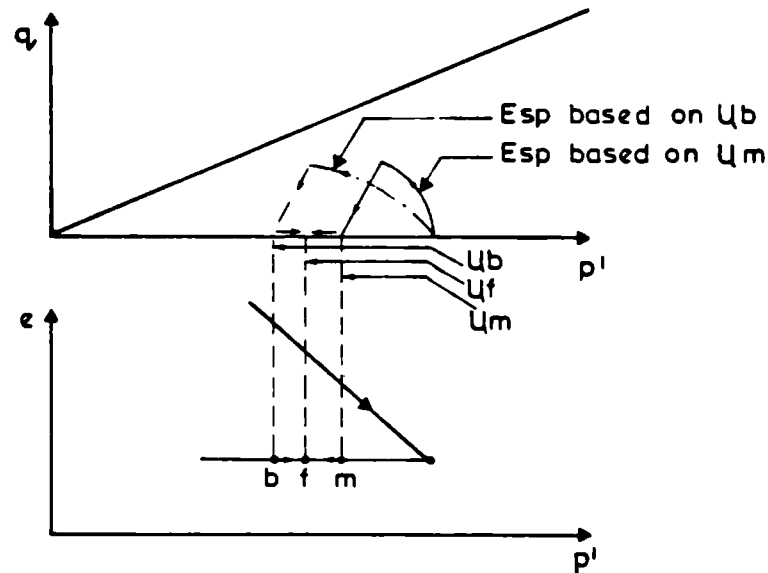


Fig. 6.12(b) Pore pressure differences at base and mid-height during cyclic loading. Anisotropic consolidation history.



V_e = Volume influenced by end restraint

V_m = Volume free of end restraint



Consider volume changes during equalisation :-

$$\Delta V_m = (U_f - U_m) m_s \cdot V_m$$

$$\Delta V_e = (U_f - U_b) m_v \cdot V_e$$

but $m_s \approx m_v$ because swelling and reconsolidation

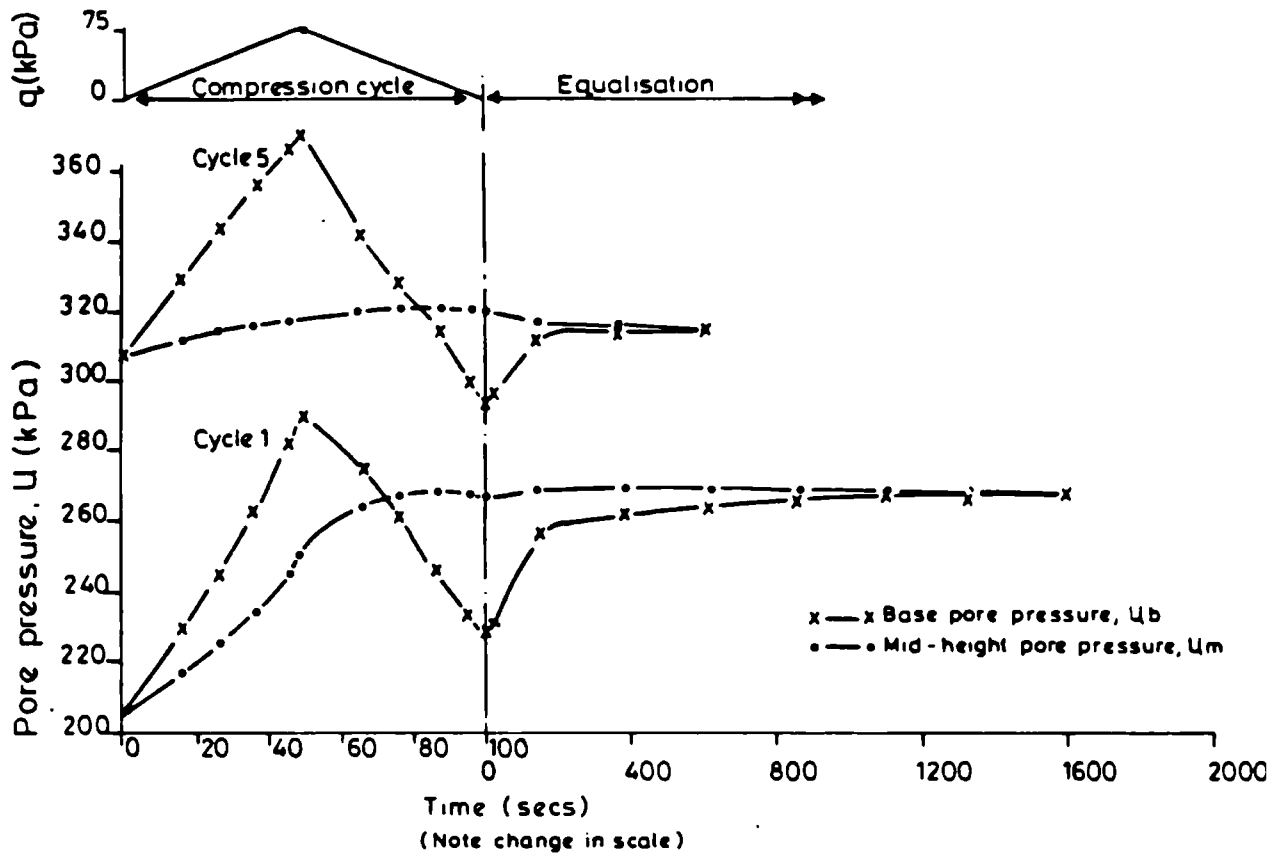
Since the sample is undrained

$$\Delta V_m + \Delta V_e = 0$$

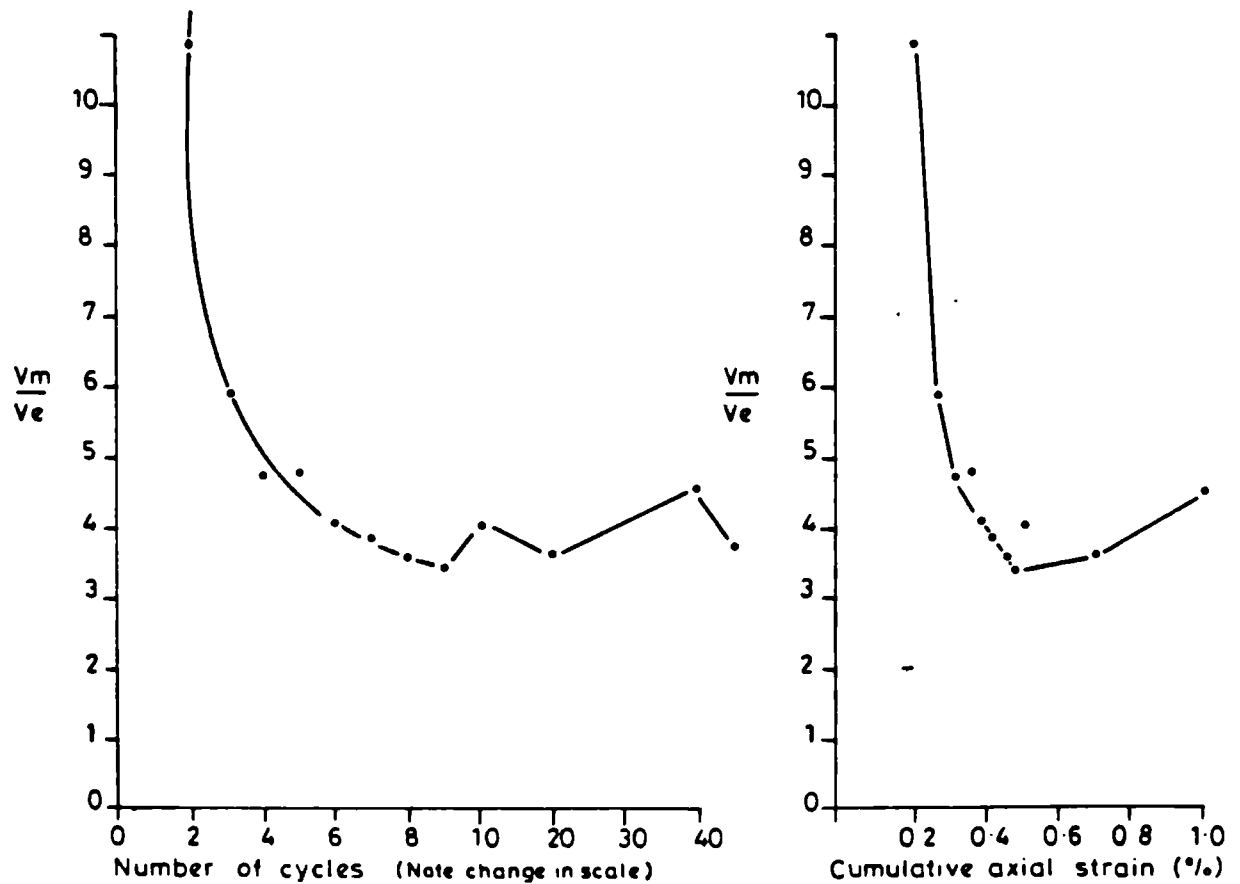
$$\therefore (U_f - U_m) \cdot m_s \cdot V_m + (U_f - U_b) \cdot m_v \cdot V_e = 0$$

$$\frac{V_m}{V_e} = - \frac{(U_f - U_b)}{(U_f - U_m)}$$

Fig. 6.13 A simple approach for quantifying end restraint effects.

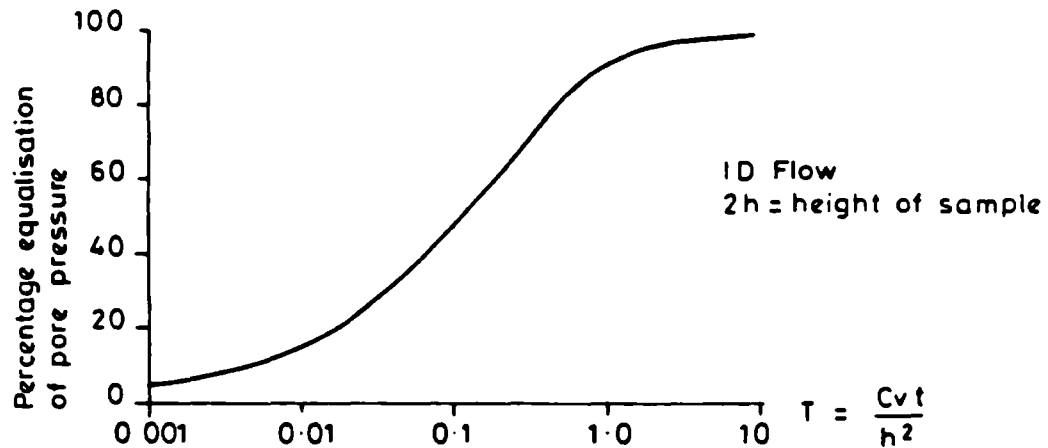


(a) Pore pressure measurements during and after compression cycle.



(b) Change in end restraint with cycling

Fig. 6.14 Development of end restraint during cyclic loading.



Theoretical relationship between time factor and equalization of non-uniform pore pressure in sample .

Assumptions made in analysis :-

- (1) Rate of loading is constant.
- (2) Non-uniformity in pore pressure is parabolic over height of sample.
- (3) Difference in pore pressure is proportional to the magnitude of the stress increment.
- (4) Coefficients of consolidation and swelling are the same.

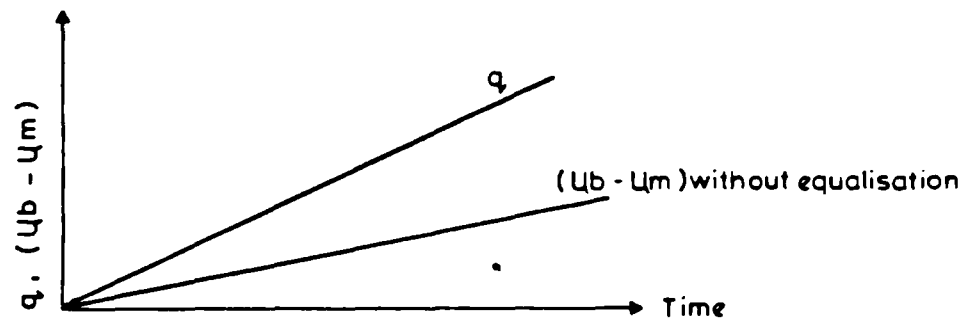


Fig. 6.15 Summary of Gibson's solution for equalisation of pore pressure non-uniformities in triaxial sample under monotonic loading.

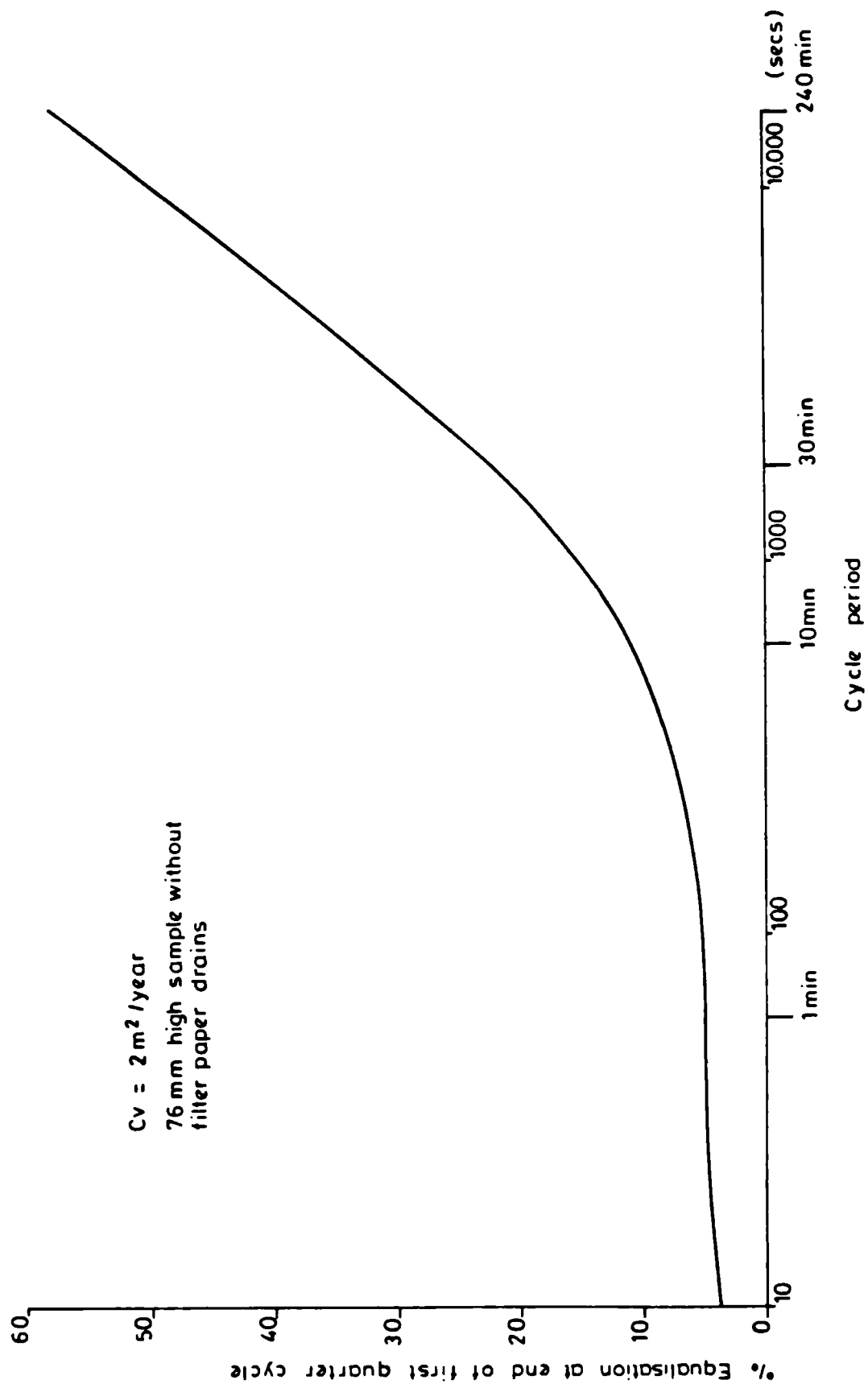
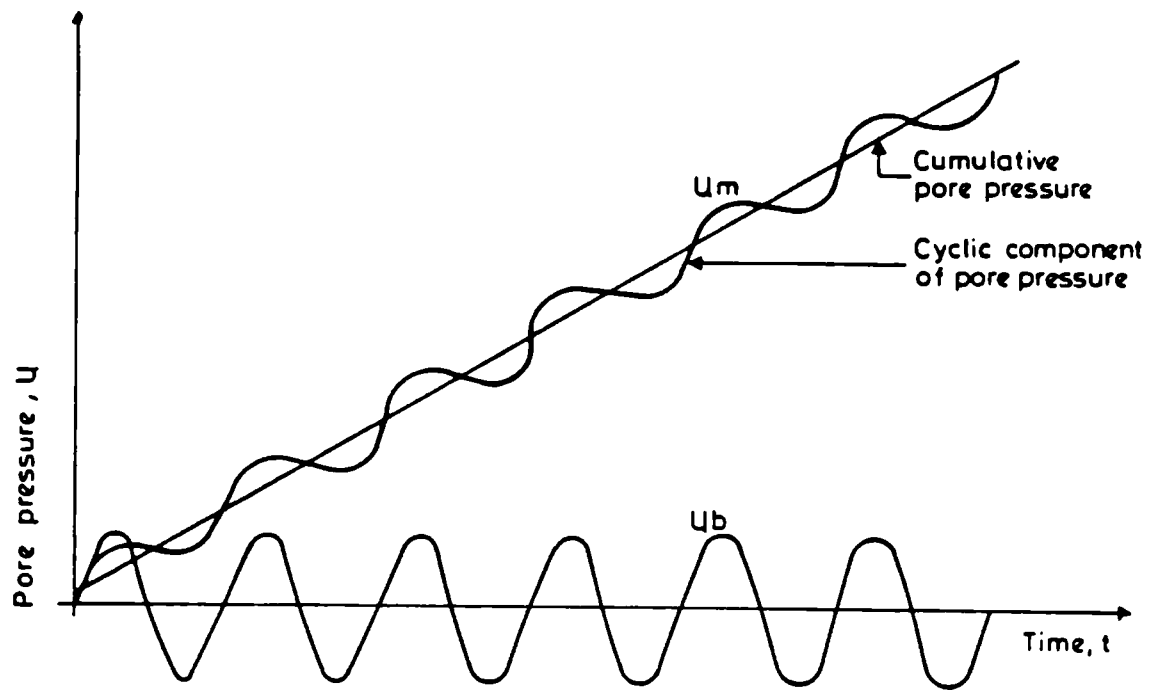
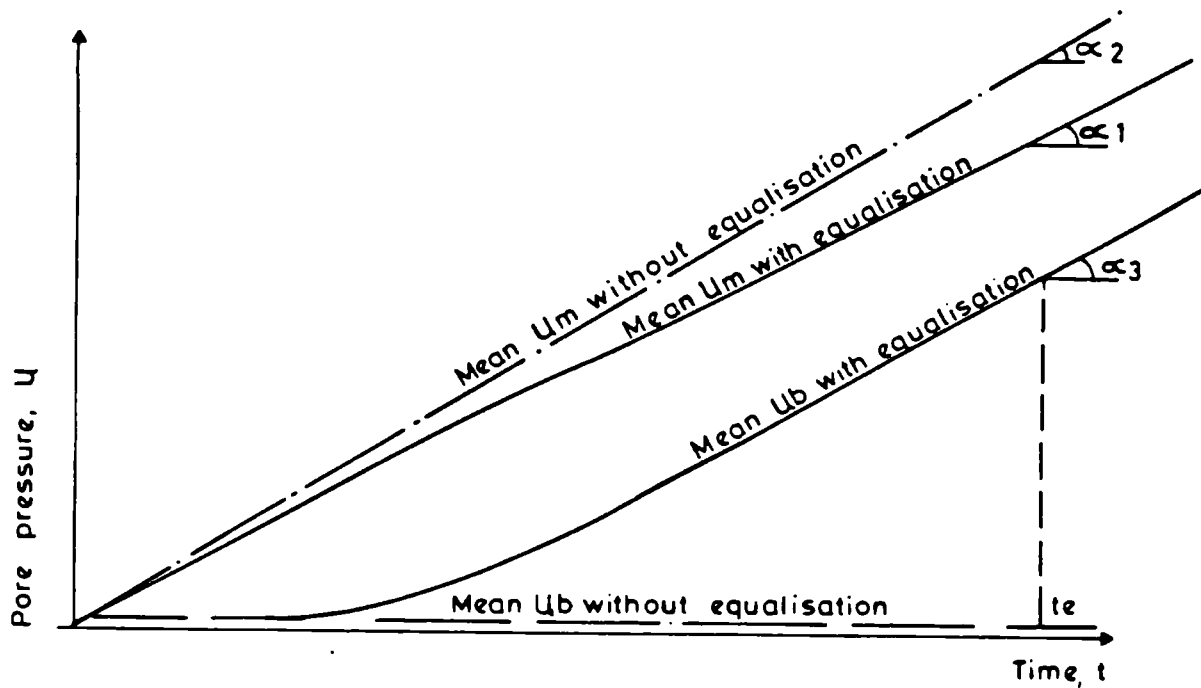


Fig. 6.16 Equalisation of pore pressures during first quarter cycle.

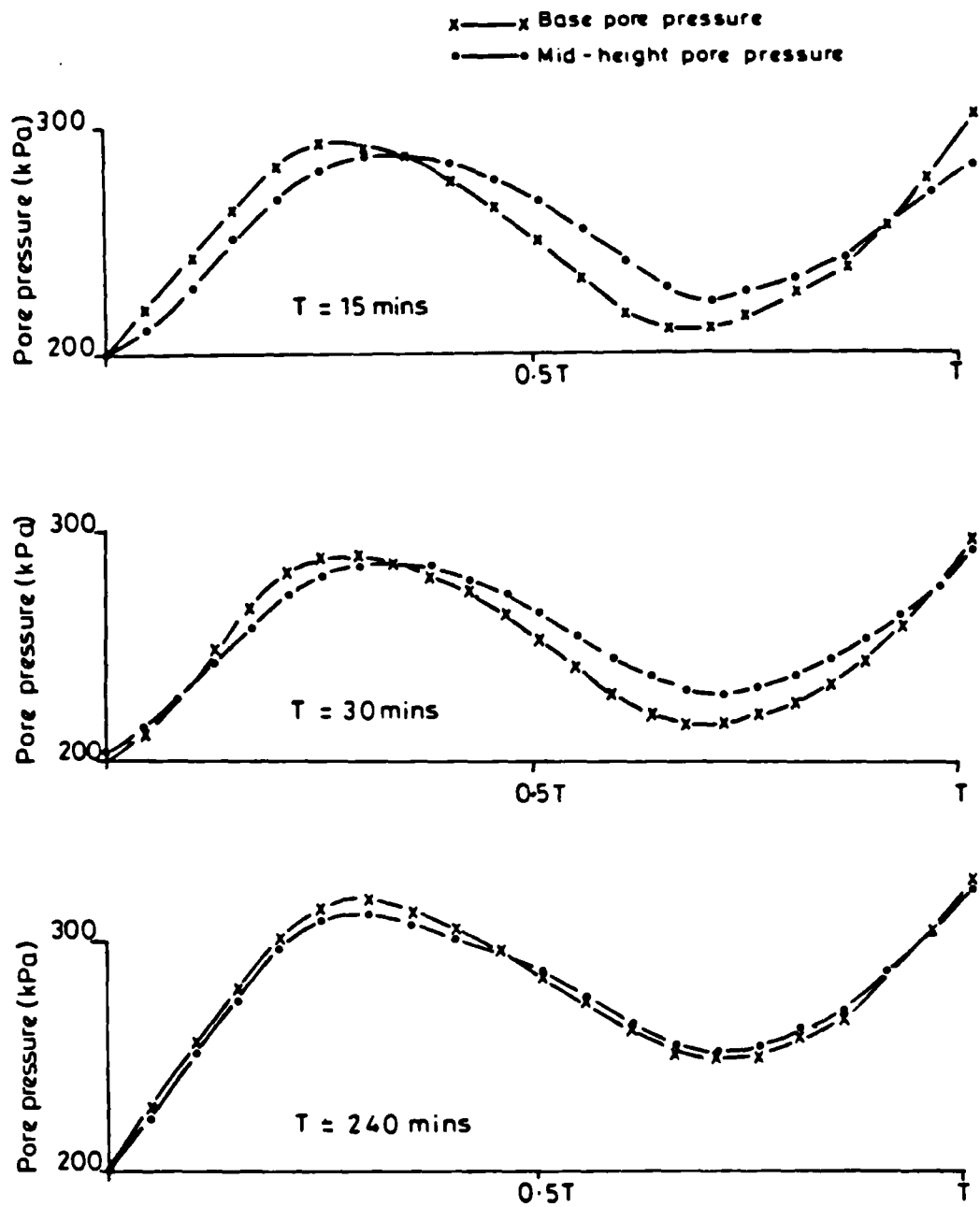


(a) Pore pressures at base and mid-height, no equalisation.



(b) Pore pressures at base and mid-height, with equalisation.

Fig. 6.17 Pore pressure equalisation during cyclic loading.



First cycle, All samples isotropically consolidated, OCR 1

Fig. 6.18 Equalisation of pore pressures during a cycle.

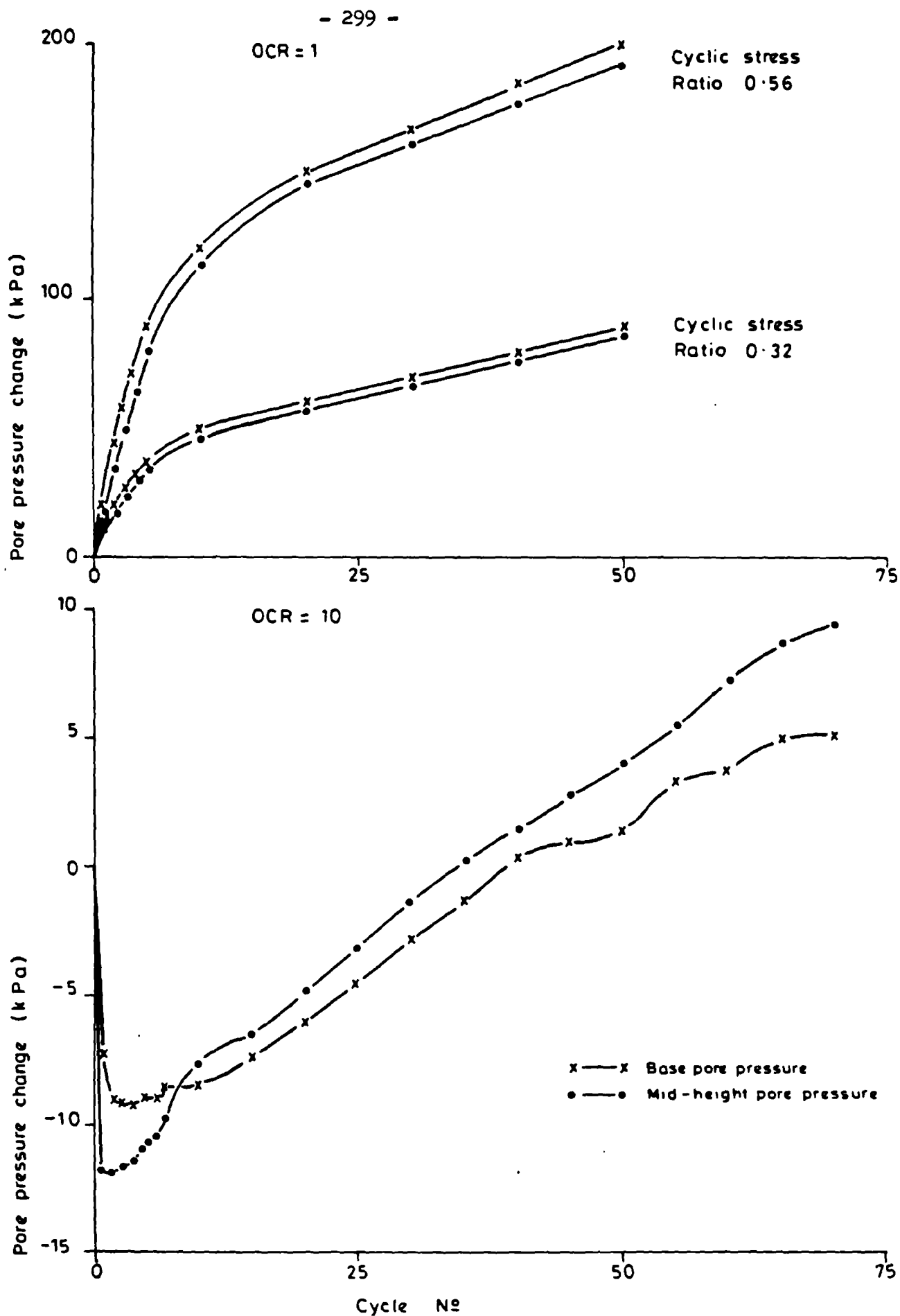


Fig. 6.19(a) Cumulative pore pressures measured at the base and mid-height. Isotropic consolidation history.

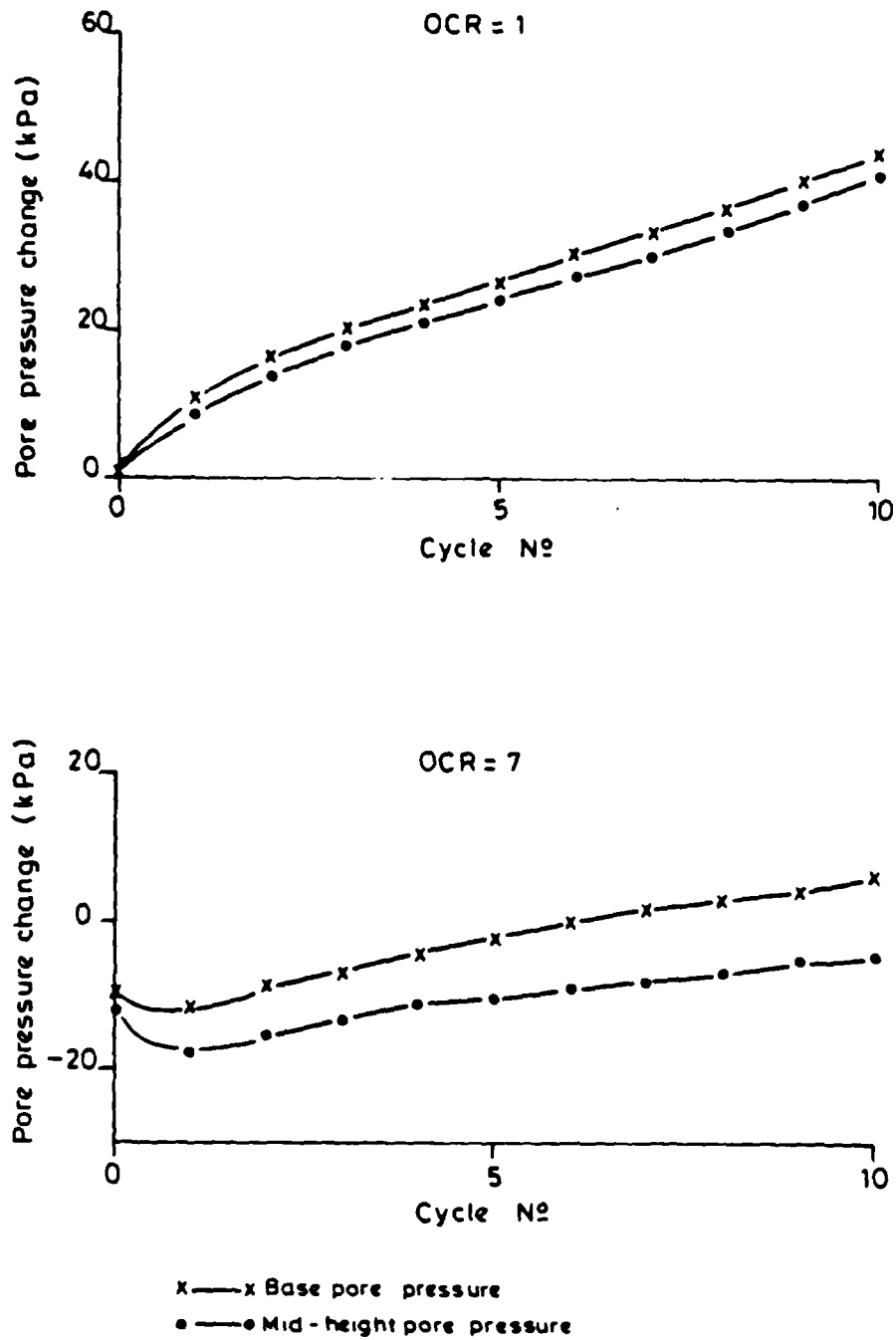


Fig. 6.19(b) Cumulative pore pressures measured at the base and mid-height. Anisotropic consolidation history.

APPENDIX 6.1

Pore pressure and strain non-uniformities in a triaxial sample under monotonic and cyclic loading

Although the problem of non-uniformities in triaxial samples with fixed ends has received much attention in the past, the soil models which have been used in analyses have not been particularly realistic. Potts (1982, pers. comm.) has carried out for the writer finite element analyses of a triaxial sample loaded through rough ends; a critical-state based elasto-plastic model has been used and the following cases have been examined:-

Monotonic compression loading $\left. \begin{array}{l} \text{OCR 1} \\ \text{OCR 4} \end{array} \right\}$

Cyclic two-way loading OCR 4
 $q_{\max} = 62.5 \text{ kPa}, C_u = 113 \text{ kPa}$

In Figs 6. A1. 1 to 6. A1. 3 the results of these exploratory analyses are shown. They illustrate the following:-

A. Monotonic loading.

- (i) There is an increase in the level of strain and pore pressure non-uniformities^{*1} as the strain level increases towards failure.
- (ii) The volume in which strains exceed $\pm 25\%$ of the value in an ideal element amounts to less than 25% of the total volume for axial strains less than 2%.
- (iii) Non-uniformities in strain are similar for OCR 1 and 4.

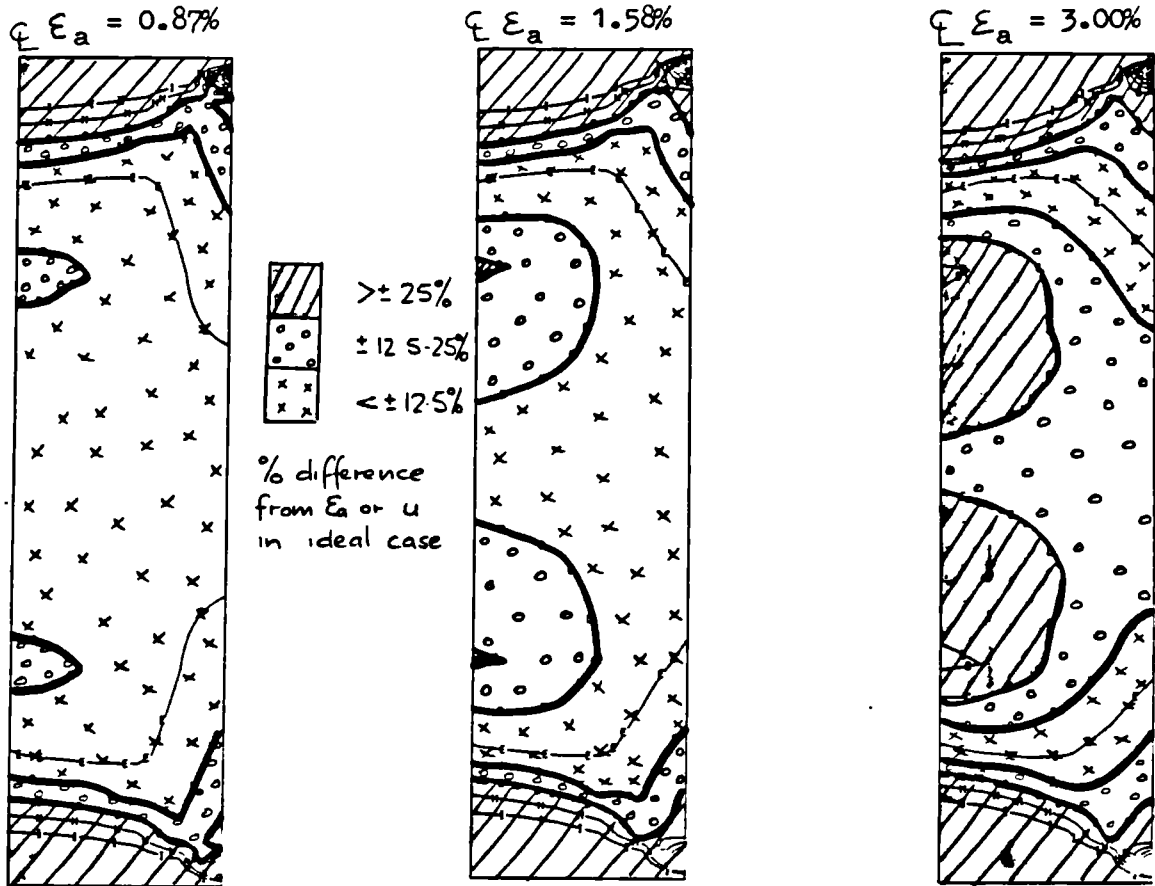
*1 The analyses do not allow pore pressure equilibration.

- (iv) For the reasons discussed in Chapter 6, the non-uniformities in pore pressure are much greater in the overconsolidated sample. For this sample there are alarming non-uniformities for axial strains in excess of 1%.

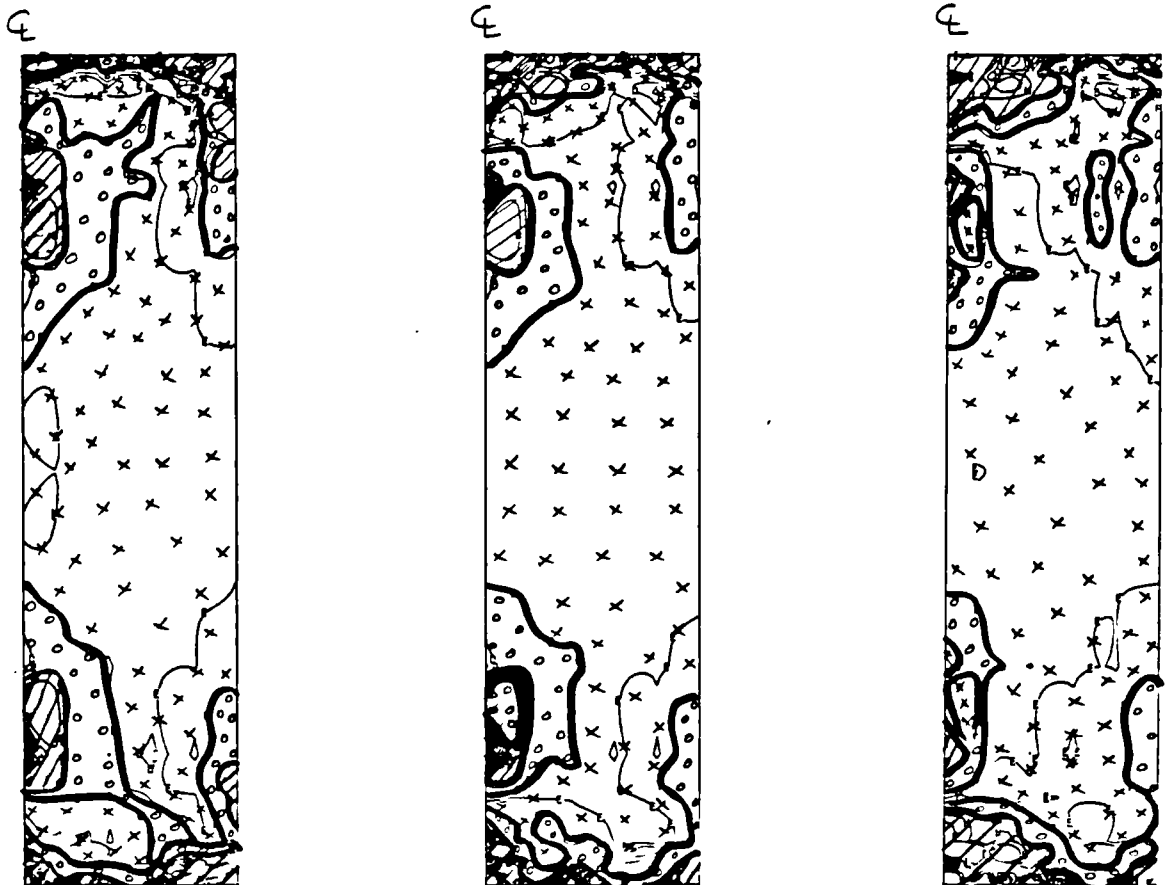
B. Cyclic loading.

- (i) There is an increase in level of non-uniformities as the test progresses. However, even at cyclic failure (cycle 121), the sample is not swamped by the effects of end restraint; the ratio $V_m : V_e$ (based on $\pm 25\%$) is still greater than 2.2:1.

Regarding the location for the piezometer probe Fig. 6. A1. 4 shows contours of vertical stress in a normally consolidated sample under compression; these confirm that in the central zone of the sample, including its periphery, conditions are reasonably uniform and vertical stress can be calculated directly from axial load and sample area.



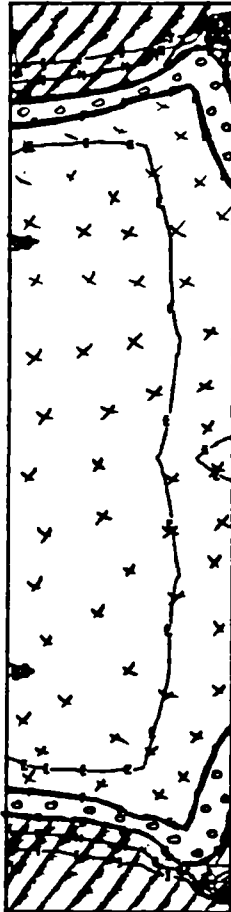
Axial strain



Pore pressure

Fig. 6. A1. 1 Results of elasto-plastic finite element analyses of fixed end triaxial sample undergoing undrained monotonic compression loading ($OCR = 1$).

$$\epsilon_a = 0.89\%$$



$$\epsilon_a = 1.67\%$$



$$\epsilon_a = 2.75\%$$





 $> \pm 25\%$

 $\pm 12.5 - 25\%$

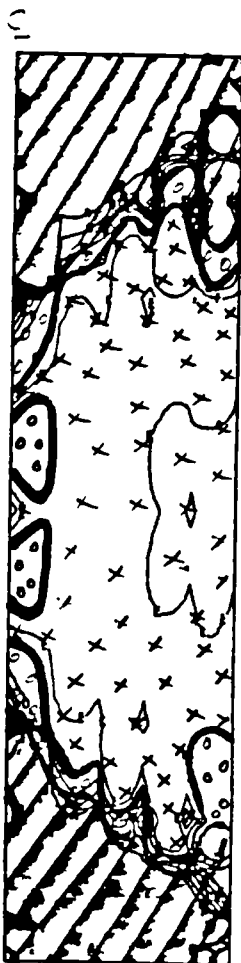
 $< \pm 12.5\%$

 % difference

 from ϵ_a or u

 in ideal case

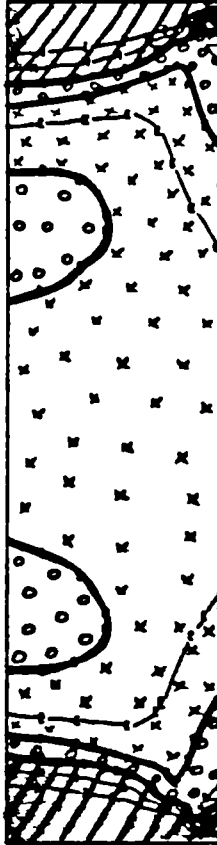
Axial strain



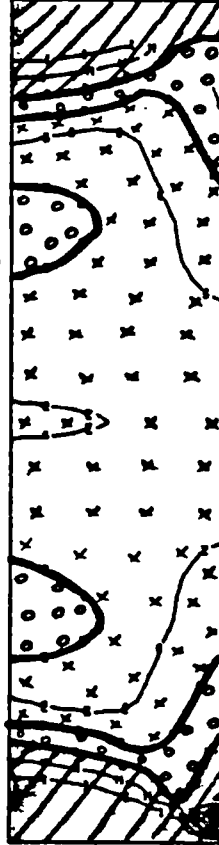
Pore pressure

Fig. 6. A1. 2 Results of elasto-plastic finite element analyses of fixed end triaxial sample undergoing undrained monotonic com-

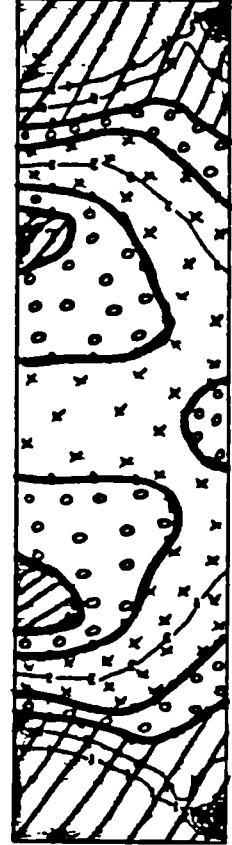
Cycle 2
 $\epsilon_a \max = 0.106\%$



Cycle 41
 $\epsilon_a \max = 0.123\%$

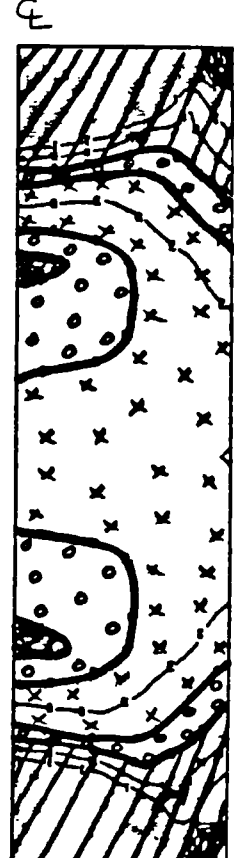
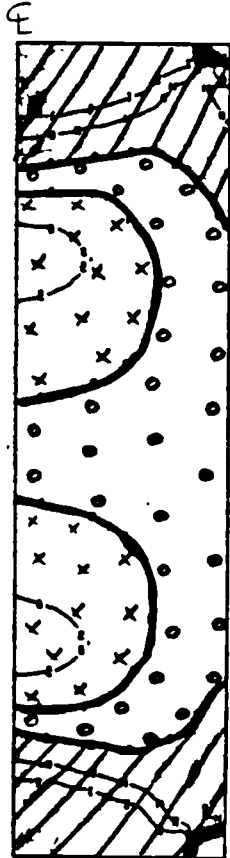


Cycle 121
 $\epsilon_a \max = 0.196\%$



$\begin{array}{|c|} \hline \text{diagonal lines} \\ \hline \text{dots} \\ \hline \text{crosses} \\ \hline \end{array}$
 $> \pm 25\%$
 $\pm 12.5-25\%$
 $< \pm 12.5\%$
 % difference
 from ϵ_a or u
 in ideal case

Axial strain



Pore pressure

Fig. 6. A1. 3.

Results of elasto-plastic finite element analyses of fixed-end triaxial sample undergoing undrained two-way cyclic loading (OCR = 1).

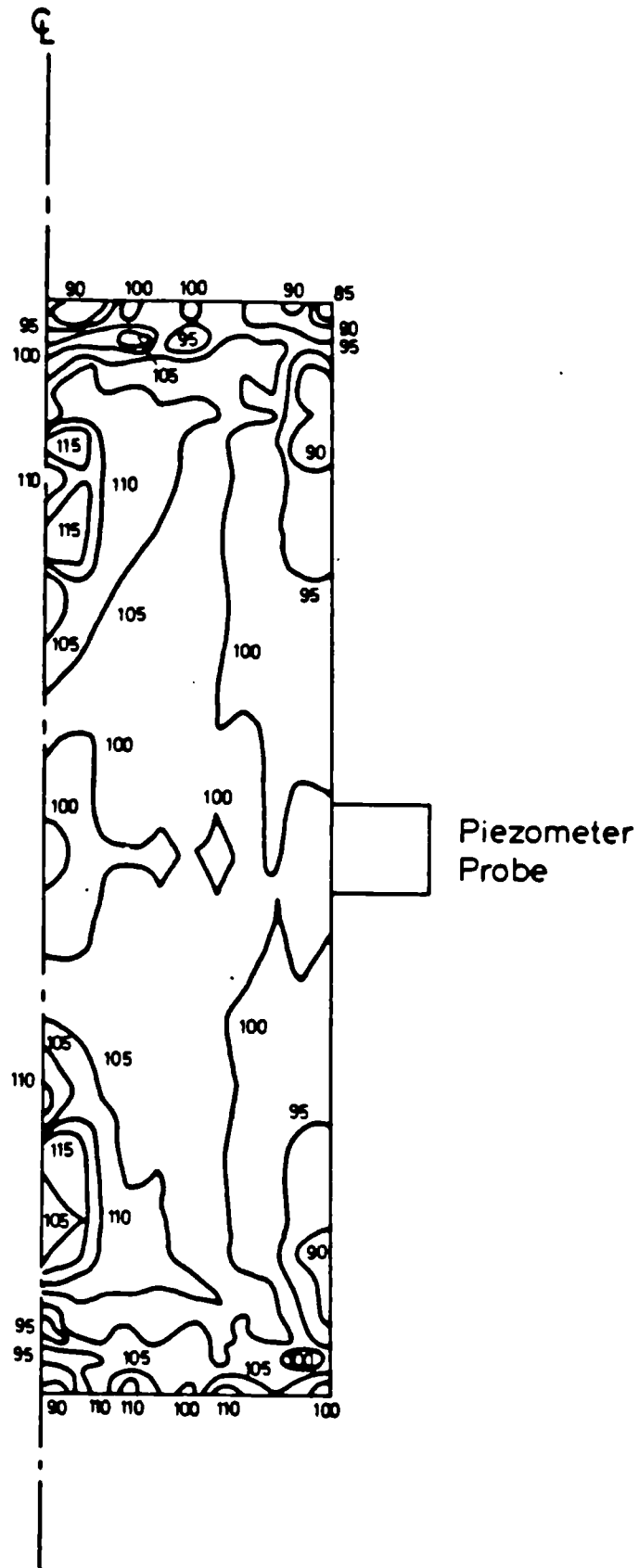


Fig. 6. A1. 4 Contours of vertical stress (expressed as percentage of axial load/sample area). (OCR = 1)

APPENDIX 6.2

Pore pressure equalisation during undrained cyclic triaxial tests

Hight, C. R. (1982, pers. comm.) has carried out finite difference analyses of the problem of pore pressure equilibration in a triaxial sample undergoing undrained cyclic loading. The situation analysed is that depicted in Fig. 6.17(a). For no equilibration, pore pressure is assumed to be generated at a constant rate, defined by the slope α_2 , in the central volume of the sample, V_m ; superimposed on the steady increase of pore pressure is a cyclic component of amplitude, a_b . In the volume at the ends of the sample, V_e , it is assumed that there is a cyclic variation in pore pressure with amplitude, a_e , with no overall increase in pore pressure per cycle. The process of equilibration has been examined for the following cases:-

Height of sample = 78mm	1. $V_m:V_e = 3:1$;	$T = 1800$ secs
$C_v = 1m^2/year$	2. $V_m:V_e = 1:1$;	$T = 900$ secs
$\alpha_2 = 4$ kPa/cycle	3. $V_m:V_e = 1:1$;	$T = 450$ secs
$a_m = 10$ kPa	4. $V_m:V_e = 1:1$;	$T = 20$ secs
$a_e = 20$ kPa	5. $V_m:V_e = 4:1$;	$T = 1800$ secs
	6. $V_m:V_e = 2.33:1$;	$T = 1800$ secs

The results are presented in Figs 6. A2. 1 to 6. A2. 6. These confirm the predicted equalisation curves shown in Fig. 6.17(b) and demonstrate the obvious dependence of:-

- (i) the error in the rate of pore pressure build-up measured at the centre on the ratio $V_m:V_e$ and cycle period, T ;
- (ii) the time for α_3 to be registered at the base on $V_m:V_e$.

It is proposed to extend this work.

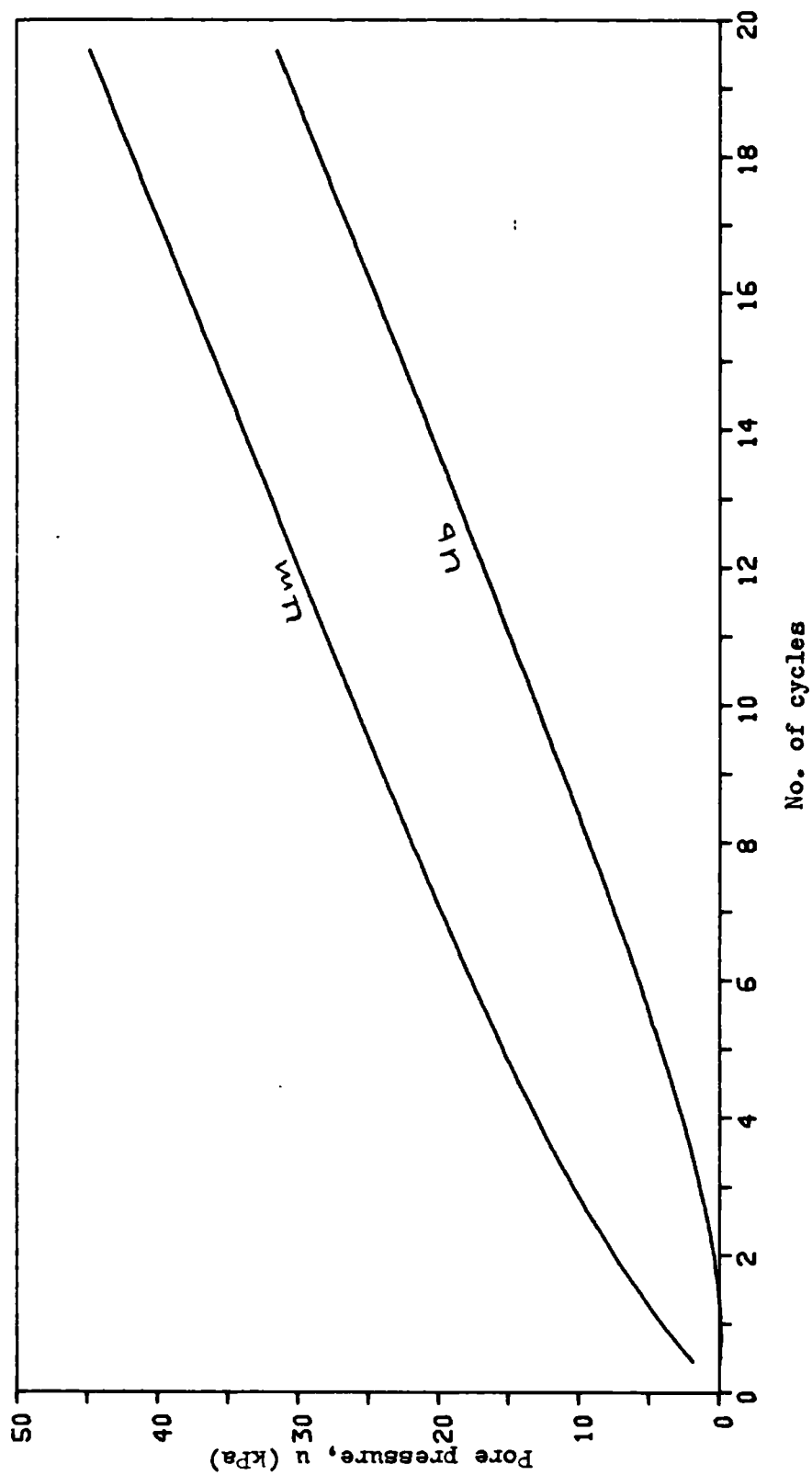


Fig. 6. A2. 1 Pore pressure equalisation during cyclic loading.
Case 1 $V_m:V_e = 1:1$; $T = 1800$ secs.

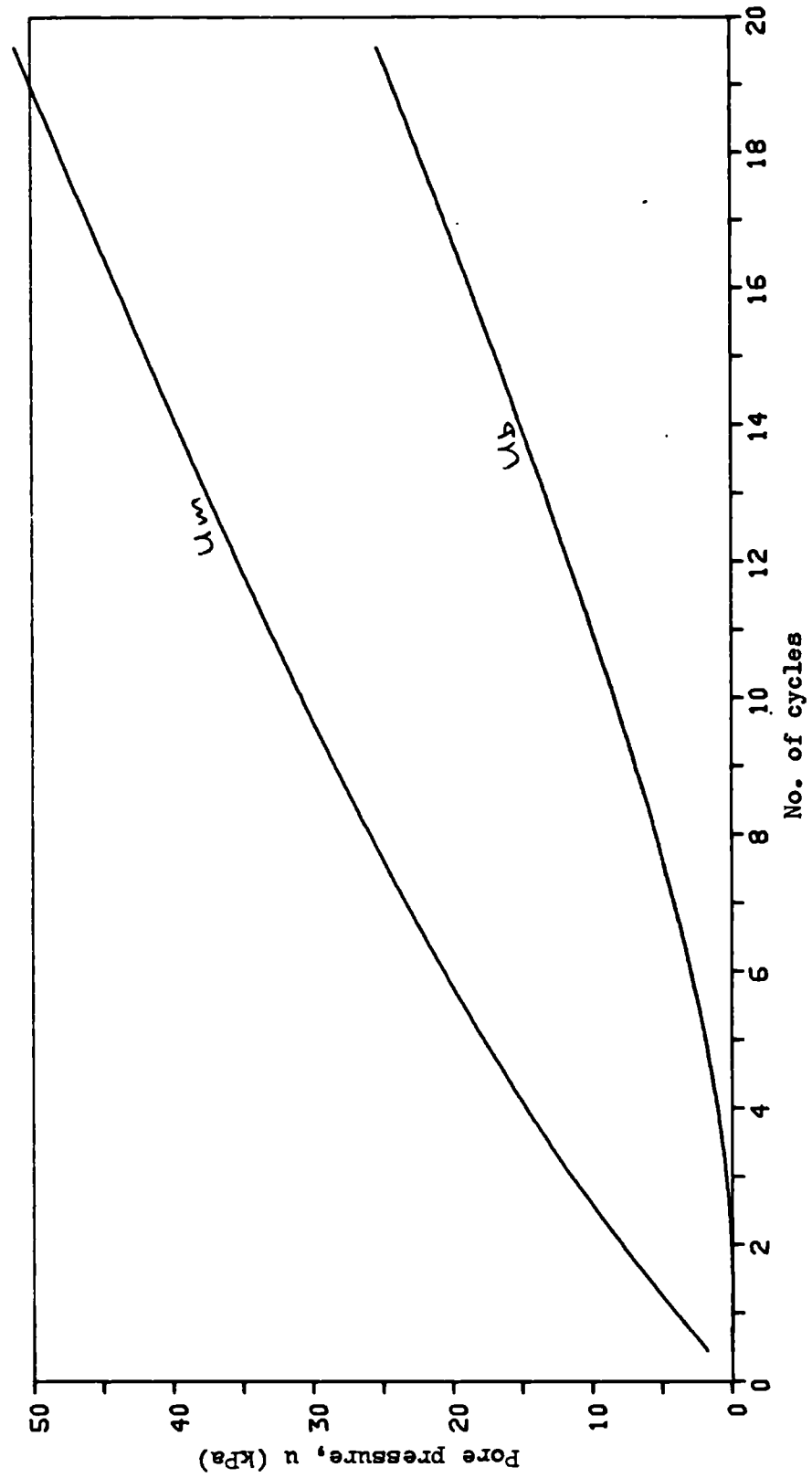


Fig. 6. A2. 2 Pore pressure equalisation during cyclic loading.
Case 2 $V_m : V_e = 1:1$; $T = 900$ secs.

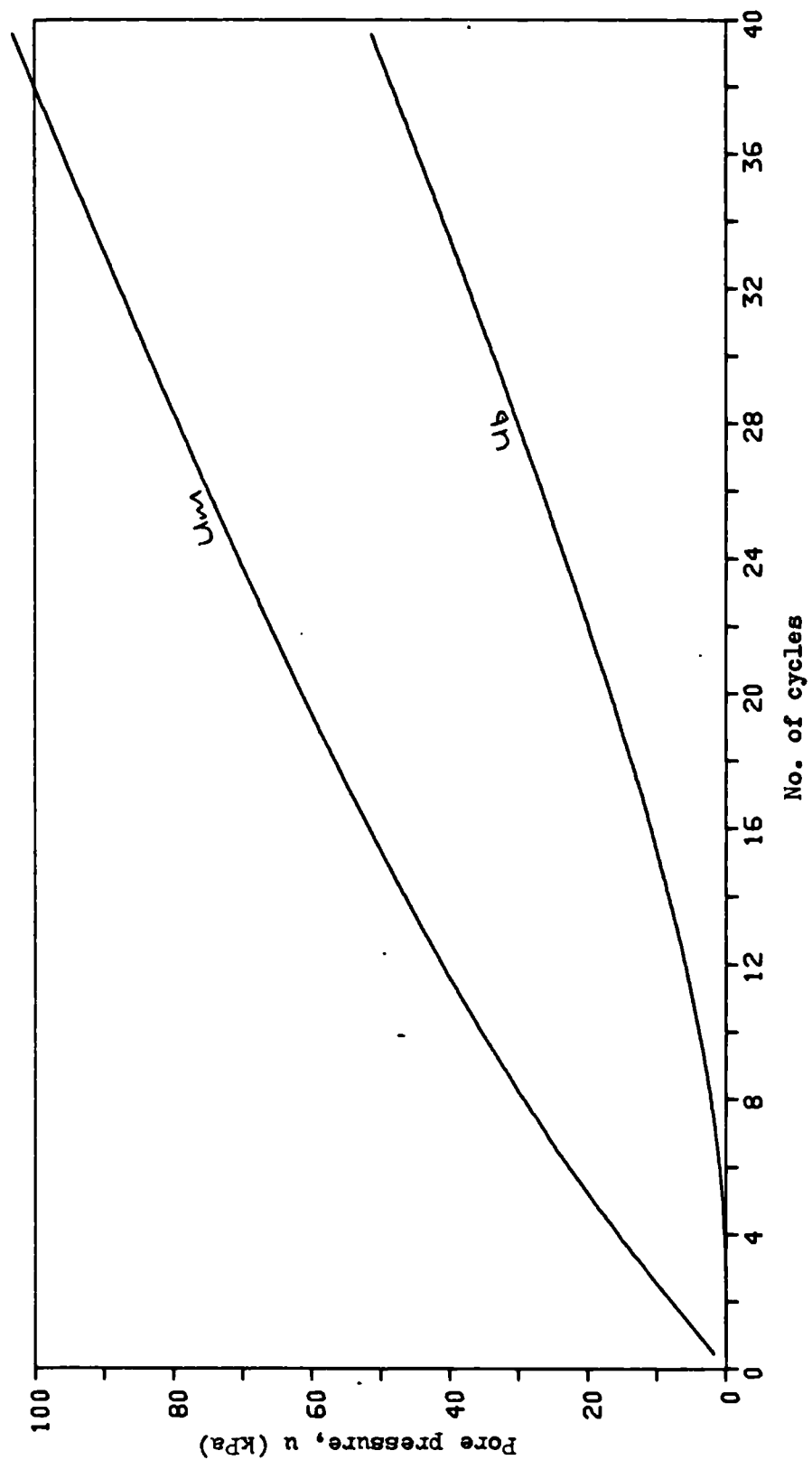


Fig. 6. A2. 3 Pore pressure equalisation during cyclic loading.
Case 3 $V_m:V_e = 1:1$; $T = 450$ secs.

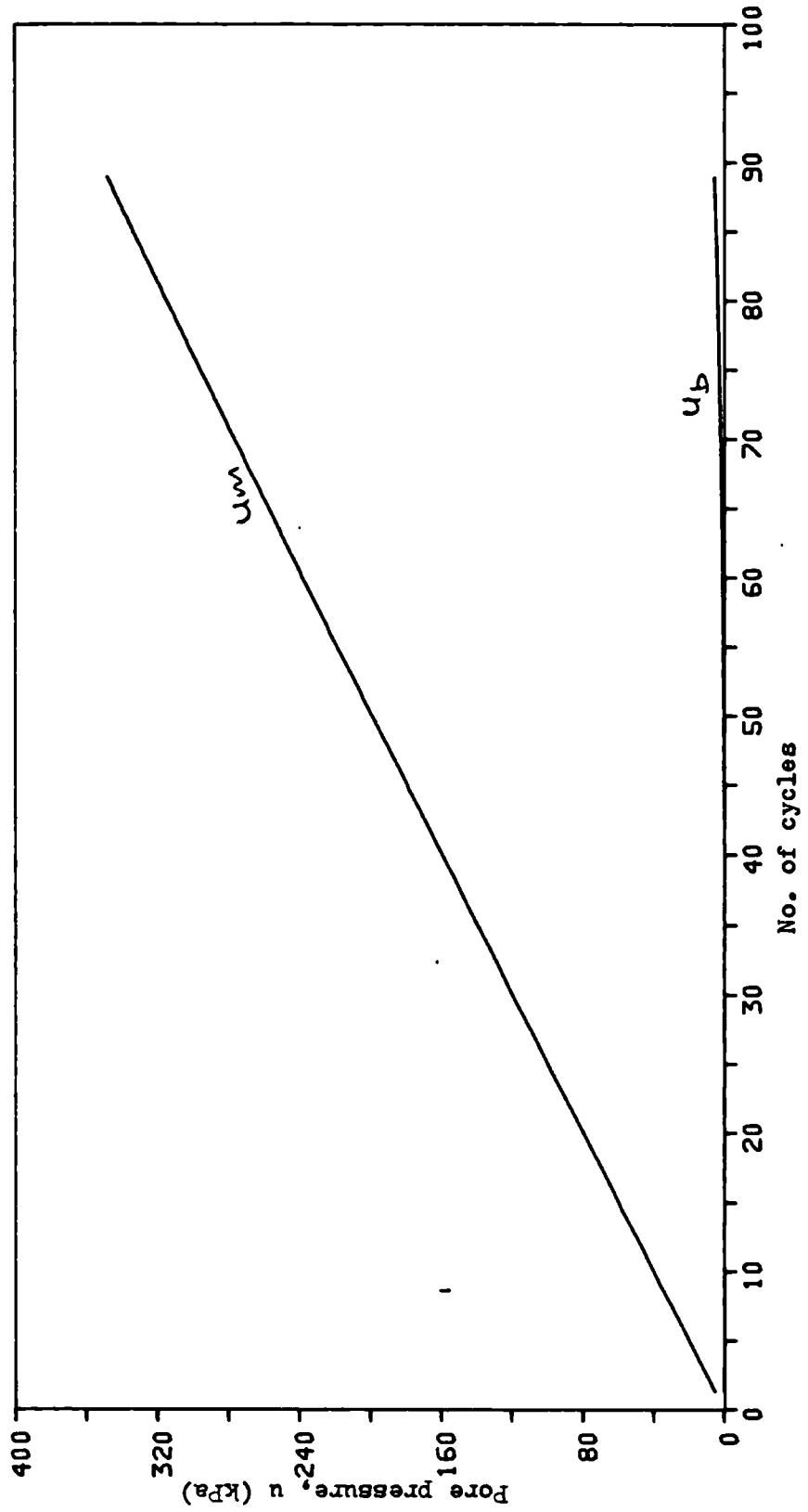


Fig. 6. A2. 4 Pore pressure equalisation during cyclic loading.
Case 4 $V:V_e = f:1$; $T = 20$ secs.

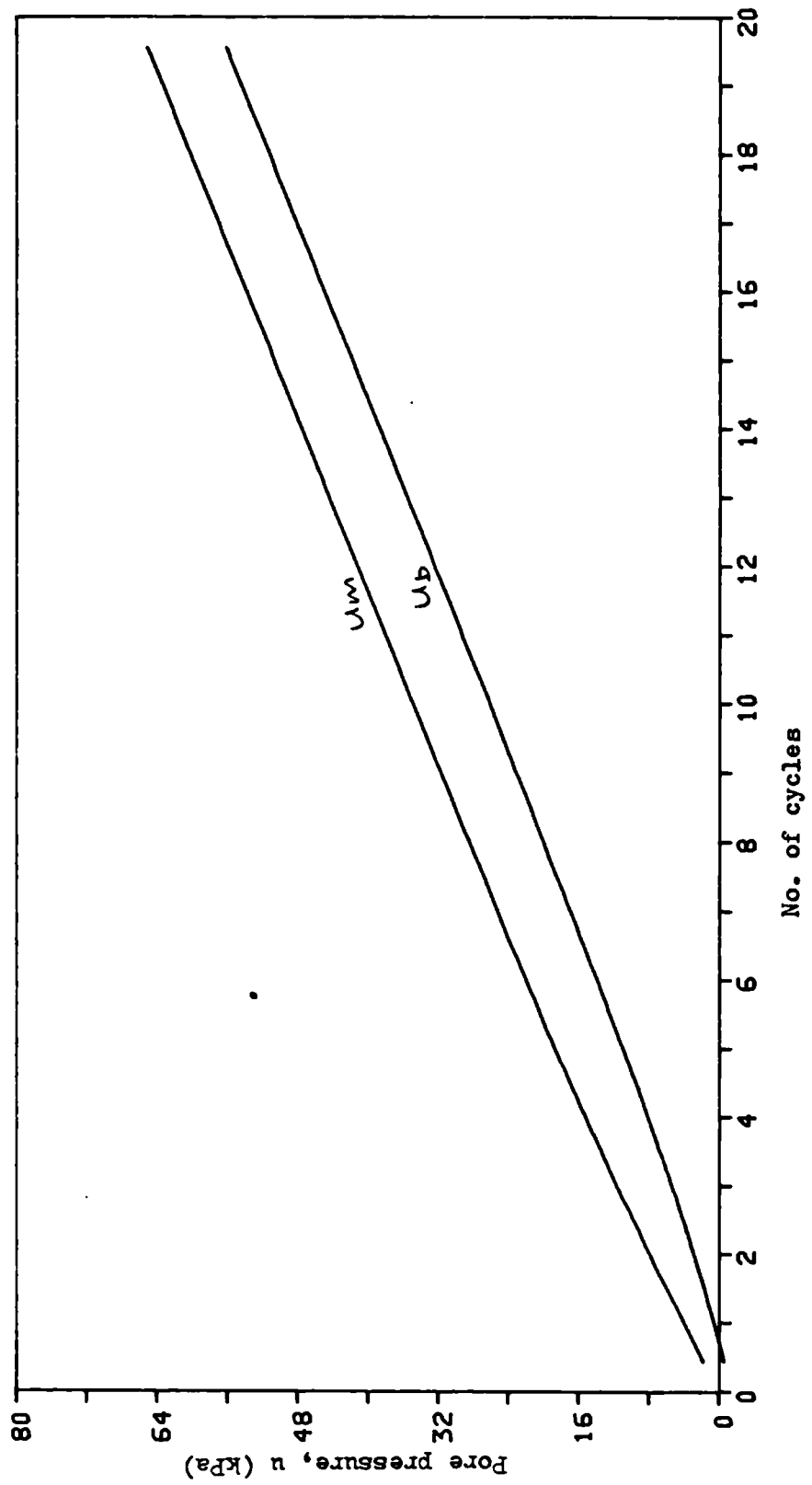


Fig. 6. A2. 5 Pore pressure equalisation during cyclic loading.
Case 5 $V : V_m = 4:1$; $T = 1800$ secs.

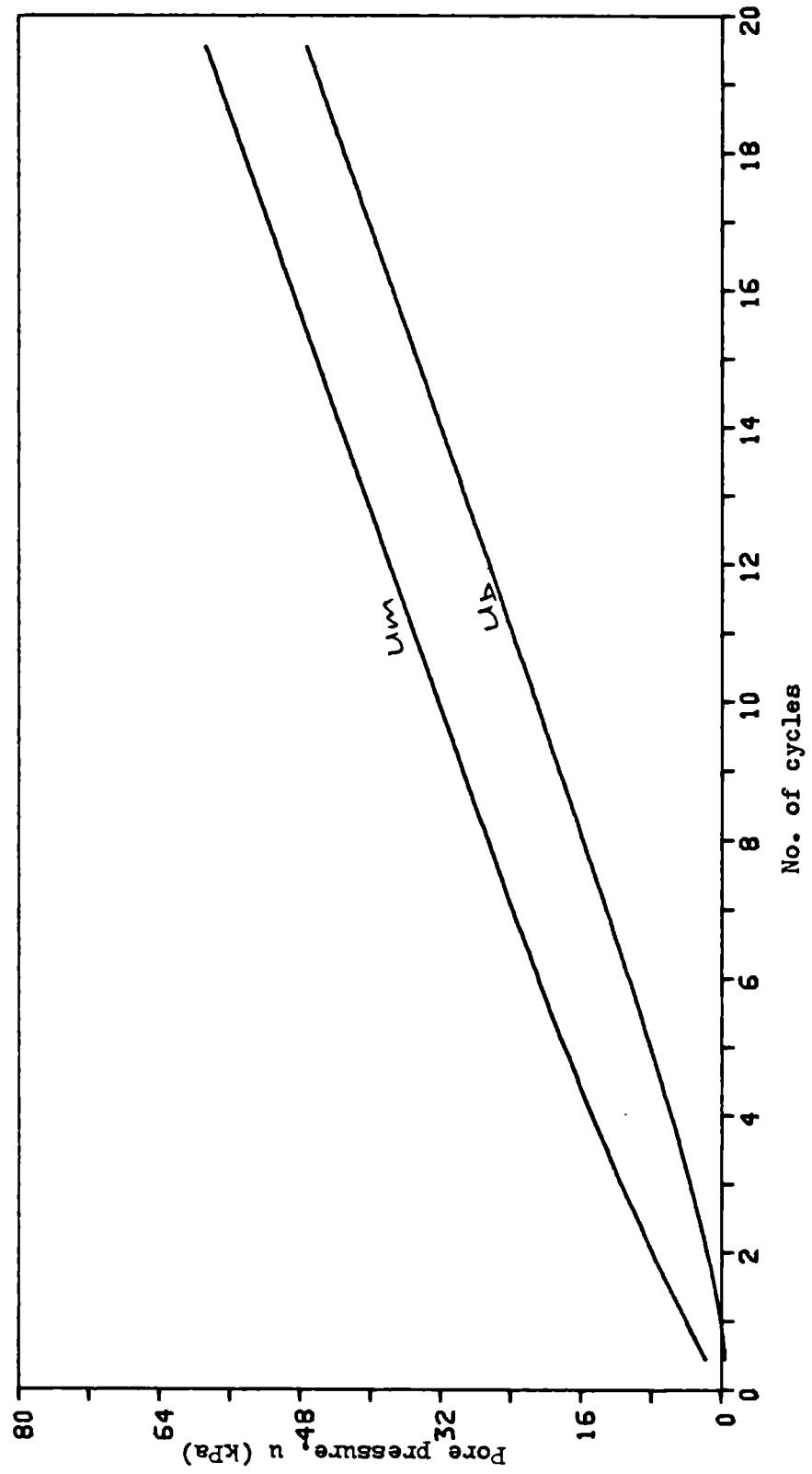


Fig. 6. A2. 6 Pore pressure equalisation during cyclic loading.
Case 6 $V_m:V_e = 2.33:1$; $T = 1800$ secs.

CHAPTER 7

THE DEVELOPMENT OF A NEW HOLLOW CYLINDER
APPARATUS FOR INVESTIGATING THE EFFECTS
OF PRINCIPAL STRESS ROTATION IN SOILS

SYNOPSIS

Methods of achieving the controlled rotation of principal stress directions acting on a soil element are discussed.

In the wall of a hollow cylindrical specimen under combined axial load, torque and internal and external radial pressures, the magnitude and direction of the major and minor principal stresses can be controlled together with the magnitude of the intermediate principal stress. This Chapter describes both the design and principles of operation of a new hollow cylinder apparatus and its application to the investigation of principal stress rotation effects in sands and clays.

A discussion is presented of the stress and strain non-uniformities which occur in a hollow cylinder specimen as a result of its curvature and of restraint at its ends. The manner in which these non-uniformities have been minimised, by selection of a sample geometry of 203mm ID x 254mm OD x 254mm high and by the use of internal instrumentation, is described.

The results of tests on dense Ham River sand are presented to demonstrate the capabilities of the apparatus.

.1 INTRODUCTION

The dependence of soil behaviour on stress path is widely recognised. Ideally, therefore, prediction of the performance of soil or soil-supported structures should make use of either a stress path approach (Lambe, 1967) or a generalised model for behaviour which incorporates the dependence on stress path.

In Chapter 2 special attention was given to two components of the stress paths associated with foundations to offshore structures:-

- (i) changes in the relative magnitude of the intermediate principal stress, conveniently expressed in terms of $b \left(= \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \right)$, and
- (ii) changes in the direction of the principal stresses, in particular of α , the direction of the major principal stress to the vertical.

Substantial cyclic changes in b and α were found to be a feature of the stress paths for gravity foundations (Figs 2.18 and 2.14) and were related to the changing direction of the applied loading. Changes in α for pile foundations were related to the kinematic restraints of the problem.

The stress path approach does not explicitly consider the effects of principal stress rotation or of changes in b ; this is not related to any limitation in the method but to limitations in equipment for simulating these particular features of stress paths (Lambe and Marr, 1980). The experimental difficulties of achieving controlled rotation of principal stress directions, referred to by Bishop and Henkel (1962), have severely restricted investigations into the effects of principal stress rotation on soil behaviour and, as a result, these effects are not adequately defined.

In Chapter 2 a contrast was made between the continuous cyclic rotation of principal stress increments produced beneath offshore

structures by wave loading and the more usual situations in which the direction of the loading and of the principal stress increments is fixed, e.g. beneath a vertically loaded circular footing. For the latter simple loading case, the effects of principal stress rotation (and of changes in b) have traditionally been neglected, probably on the basis of experience.

Experience of the behaviour of soil undergoing cyclic rotation of principal stress directions and cyclic changes in b is, however, strictly limited. It is relatively easy to show that neither can be neglected by considering the potential effects of changes in α and b on behaviour.

Potential effects of principal stress rotation on behaviour

Soils are, by nature, anisotropic, both inherently so as a result of their mode of deposition, and as a result of subsequent stress changes. By definition, in an inherently anisotropic soil, different orientations of principal stress will produce different response. It is a relatively simple matter to show, for example, how the strength of an element having anisotropic strength parameters will vary with orientation of principal stress directions at failure (Jaeger, 1960). When anisotropy is induced by stress changes, it seems likely that the anisotropy will depend on the sequence of changes in principal stress direction, i.e. the rotation path, as well as on the changes in principal stress magnitude.

The cumulative effect of cyclic rotations of principal stress direction is difficult to pre-judge. The marked difference in response of soil to triaxial compression (one-way) as opposed to triaxial compression-extension (two-way) cyclic loading indicates that the effect will be significant. Arthur et al (1979) have recently shown the weakening effect on sands of rotating principal stress directions at constant stress ratio.

Potential Influence of b on Soil Behaviour

The first potential influence of b on soil behaviour is most simply explained by comparing a triaxial compression test with a triaxial extension test. In the former

$$\sigma_2 = \sigma_3 \quad \text{so that} \quad b = 0 ;$$

in the latter

$$\sigma_2 = \sigma_1 \quad \text{so that} \quad b = 1.$$

Assume an isotropic soil which fails according to the Mohr-Coulomb failure criterion

$$\sigma_1 - \sigma_3 = (\sigma'_1 + \sigma'_3) \sin \phi'$$

i.e. a criterion which is independent of σ_2 .

In a triaxial stress space of $(\sigma_a - \sigma_r)/2$ versus $(\sigma'_a + \sigma'_r)/2$, the Mohr Coulomb failure criterion plots then as two lines at $\tan^{-1}(\sin \phi')$ to the horizontal, see Fig. 7.1. The effective stress paths followed in conventional drained triaxial compression and extension tests are shown in these figures. The obvious effect of stress path, and hence b, on strength is illustrated.

In undrained loading the pore pressures which are generated can be determined from:-

$$\Delta u = \frac{\Delta \sigma_1 + \Delta \sigma_2 + \Delta \sigma_3}{3} + a \sqrt{(\Delta \sigma_1 - \Delta \sigma_2)^2 + (\Delta \sigma_2 - \Delta \sigma_3)^2 + (\Delta \sigma_3 - \Delta \sigma_1)^2}$$

(Henkel, 1960)

For $b = 0$, triaxial compression, $\Delta \sigma_2 = \Delta \sigma_3$ and

$$\Delta u = \frac{\Delta \sigma_1 + 2 \Delta \sigma_3}{3} + a \sqrt{2(\sigma_1 - \sigma_3)}$$

For conventional triaxial compression, $\Delta\sigma_3 = 0$ and so

$$\Delta u = (1/3 + a\sqrt{2}) (\sigma_1 - \sigma_3)$$

For $b = 1$, triaxial extension, $\Delta\sigma_2 = \Delta\sigma_1$ and

$$\Delta u = \frac{2\Delta\sigma_1 + \Delta\sigma_3}{3} + a\sqrt{2}(\Delta\sigma_1 - \Delta\sigma_3)$$

For conventional triaxial extension, $\Delta\sigma_3 = 0$ and so

$$\Delta u = (2/3 + a\sqrt{2})(\sigma_1 - \sigma_3)$$

Again for an isotropic soil with the same a and ϕ' , more pore pressure will be generated in triaxial extension ($b = 1$) and a lower undrained strength will result (Fig. 7.1).

A second well-established effect of b is on the value of ϕ' (e.g. Ergun, 1980). Changes in the freedom of particle movement with b are reflected in changes in ϕ' .

For foundations to offshore structures, in which there are large cyclic changes in b and α , it is evident that there is a strong need to extend the stress path method to include changes in these parameters, or to perform a thorough fundamental investigation of the effects of b and α on behaviour for the case of both monotonic and cyclic loading.

An apparatus is required which will permit the measurement of the response of soil elements to the correct sequence of changes in principal stress magnitude and direction. For fundamental investigations, the apparatus should allow the separate effects of b and α to be studied. It should be possible to shear with:-

- (i) different, but fixed, orientations of principal stress directions, and
- (ii) continuous rotation of principal stress directions.

Variables to be investigated include the magnitude of the rotation, its continuity and history, the accompanying changes in principal stress magnitude, including σ_2 , all for a range of soil types and consolidation histories.

Ideally, the apparatus should be capable of:-

- (i) both isotropic and anisotropic consolidation of specimens,
- (ii) controlled rotation of principal stress directions between $\pm 90^\circ$, including the repeated reversal of directions,
- (iii) simultaneous independent control of the magnitude of the principal stresses, including σ_2 ,
- (iv) maintenance of plane strain or other kinematic conditions of deformation relevant to field problems,

all under as wide a range of stress levels as possible and for both drained and undrained conditions.

This Chapter describes the development of an apparatus which can meet most of these requirements. The apparatus tests hollow cylinders of soil under combined axial load, torque and internal and external pressures; this enables controlled rotation of principal stress directions in one surface to be achieved under stress controlled conditions. The hollow cylinder apparatus (HCA) offers the opportunity of extending the stress path approach to include simulation of both principal stress rotation and variation in intermediate principal stress, as well as conducting fundamental research into principal stress rotation effects under a reasonable generalised stress state.

.2 LABORATORY SIMULATION OF PRINCIPAL STRESS ROTATION

Controlled changes in the magnitude and direction of the principal stresses on an element are extremely difficult to reproduce in the laboratory. In most pieces of testing apparatus, e.g. triaxial, true

triaxial and plane strain, principal stresses are fixed in direction; only interchange of principal stress directions can be achieved. The inadequacy of these conventional pieces of apparatus for applying the stress path method to offshore foundations can be judged by examining the variations in α and b shown in Figs 2.14 and 2.16 for the case of a gravity platform.

To rotate the stresses in one plane of the rectangular element in Fig. 7.2 requires that the shear, and for generality, the normal stresses acting on faces a , b , c and d are changed simultaneously. On faces e and f , either the normal stresses or the displacement may be controlled, e.g. to maintain plane strain conditions. By definition, the normal and shear stresses on a , b , c and d must be uniform and, for equilibrium, the shear stresses must be complementary.

It would appear that these conditions can only be achieved in a testing apparatus in which the loading boundaries are:-

- (i) flexible, to ensure uniformity of normal stress;
- (ii) rough, to permit shear stresses to be transmitted; and
- (iii) extensible, to avoid restraints on displacements and hence to maintain uniformity of strain.

The simple shear apparatus has been cited as being capable of reproducing principal stress rotation. It exists in two forms. In the Cambridge Simple Shear Apparatus (Roscoe, 1953), rigid platens impose on a rectangular prism a condition of simple shear. While the horizontal and vertical forces applied to the specimen can be controlled, being applied through rigid boundaries, there can be no guarantee of uniformity in the boundary stresses. Furthermore, the rigidity of the boundaries presents a dilemma regarding the conditions to be imposed at the end faces. Under drained shear, the specimen may change in height. By employing a smooth boundary at the end faces, the change in length of AB can be accommodated; uniform strain may then be possible in the element, but no shear stress can develop along AB . Alternatively,

by providing a rough contact between the specimen and end faces, shear stress can develop along AB; however, this shear stress is not necessarily the complementary shear stress, its average value and distribution depending on the normal stress distribution along BC; furthermore, with the change in length of AB prevented, non-uniform strains result.

In the Cambridge simple shear apparatus smooth end platens are used in aiming for uniformity of strain. The consequences of the lack of a complementary shear stress, in terms of the boundary and internal stresses, have been extensively investigated by Wood and Budhu (1980). Interpretation of the test can only be made by measuring the boundary stresses and analysing the sample response as a boundary value problem.

The use of rigid boundaries in this form of the simple shear apparatus presents a further restriction on its suitability for investigating the effects of principal stress rotation. Arthur et al (1980) considered the effects of preventing horizontal extension in the specimen when confined between rigid end plates. By allowing no horizontal extension, the directions of the principal strains are fixed. The possible rotations of principal stresses are, therefore, severely limited. The apparatus cannot be used for controlled rotation of principal stress directions.

In the second form of simple shear apparatus, the NGI Simple Shear Device (Bjerrum and Landva, 1966), a cylindrical specimen is enclosed in a spirally reinforced membrane and loaded horizontally and vertically through a rigid top cap. Again, uniform boundary stresses on the top surface cannot be guaranteed. The dilemma regarding conditions along the vertical surface is removed, since the flexible membrane restricts the development of shear stresses and non-uniformities again develop. The restraint on horizontal extension is partially removed.

Spiral reinforcement was introduced to maintain plane strain conditions. Wood and Budhu (1980) have shown, in fact, that only a constant perimeter is achieved with the result that the sample shape may change and a non-uniform distribution of horizontal stress results.

Neither form of simple shear apparatus satisfies the loading boundary requirements for controlled principal stress rotation described above. Complementary shear stresses cannot develop fully. This, together with the use of at least one rigid loading platten, leads to pronounced non-uniformity of stress in the specimen. In the Cambridge device there is a restriction on horizontal extension and in the NGI device a non-uniform distribution of horizontal stress. Principal stress rotations take place but not in a controlled manner.

In the Directional Shear Cell, developed by Arthur et al (1977), the shortcomings of the simple shear apparatus for rotating principal stress directions on rectangular elements have been removed by fulfilling the requirements for boundary conditions listed above. In this ingenious device, complementary shear stresses are applied to the appropriate boundaries by using rough flexible membranes; the method adopted ensures that these shear stresses are approximately uniform.

An alternative approach to the laboratory simulation of principal stress rotation involves subjecting hollow cylindrical samples to combined axial load, W , torque, M_T , about a central vertical axis, and internal and external radial pressure, p_i and p_o respectively, refer Fig. 7.3(a). Variations in this combination of forces and pressure leads to rotations of the principal stresses acting in the wall of the cylinder. This is discussed in detail below.

3 PRINCIPLES OF HOLLOW CYLINDER TESTING

Shear stresses can be developed in vertical and horizontal planes in cylindrical specimens by subjecting them to a torque, M_T , applied about their vertical axis. For most materials, the shear stress, $\tau_{z\theta}$, will be a function of radius. To limit this variation, in shear stress across a cylinder, hollow cylinders with thin walls have been used in materials testing. Further advantage can be taken of the hollow shape by controlling the internal pressure, p_i , in the cavity as well as the external pressure, p_o . Hollow cylinder testing is used herein to cover tests on samples in which there is control of

W , M_T , p_o and p_i . Ring shear and confined torsion shear devices are thereby excluded.

The torque, M_T , develops shear stresses, $\tau_{\theta z}$ and $\tau_{z\theta}$, in vertical and horizontal planes; the axial load, W , contributes to a vertical stress, σ_z . Differences between p_o and p_i establish a gradient of radial stress, σ_r , across the cylinder wall. The circumferential stress, σ_θ , will then be different to σ_r as shown by the equation for radial equilibrium

$$\sigma_\theta - \sigma_r = r \cdot \frac{d\sigma_r}{dr}$$

The stresses which act on an element in the wall of a hollow cylindrical specimen subject to M_T , p_i , p_o and W are shown in Fig. 7.3(b); comparison with Fig. 7.2 shows that the stresses required to rotate the direction of the principal stresses in one plane can be generated. In testing hollow cylindrical soil specimens, both p_i and p_o act through flexible membranes, so that there are no shear stresses on the vertical boundaries. Neglecting the effects of end restraint, there are no shear stresses on circumferential surfaces throughout the wall and the radial stress, σ_r , is always a principal stress. The stresses, σ_z , σ_θ , $\tau_{z\theta}$ and $\tau_{\theta z}$, may be resolved to determine the magnitude and direction of the remaining principal stresses, see Fig. 7.3(c) and (d). The relative magnitude of the three principal stresses is determined by the applied forces and pressures and by the geometry of the specimen. By varying $\tau_{z\theta}$, σ_z and σ_θ alone or in combination both the magnitude and direction of these principal stresses in the wall of the cylinder may be altered at will in a controlled fashion.

In a general case, the stresses will not be uniform across the wall and to consider the hollow cylinder specimen as an element it is necessary to work in terms of average stresses, $\bar{\tau}_{\theta z}$, $\bar{\sigma}_z$, $\bar{\sigma}_\theta$, $\bar{\sigma}_r$.

For the particular case of equal internal and external pressure ($p_i = p_o = p$) it is usual to assume that $\bar{\sigma}_r$ and $\bar{\sigma}_\theta$ are also equal to p ; changes in $\bar{\tau}_{\theta z}$, $\bar{\sigma}_z$, $\bar{\sigma}_\theta$ and $\bar{\sigma}_r$ produce changes in the magnitude and orientation of $\bar{\sigma}_1$ and $\bar{\sigma}_3$ and simultaneous changes in the value of $\bar{\sigma}_2$ relative to $\bar{\sigma}_1$ and $\bar{\sigma}_3$. For this condition ($p_i = p_o$) it can be

shown that the relationship between the parameter, $b (= (\bar{\sigma}_2 - \bar{\sigma}_3)/(\bar{\sigma}_1 - \bar{\sigma}_3))$ and the orientation of the major principal stress to the vertical, α , is

$$b = \sin^2 \alpha.$$

By removing the restriction that p_0 and p_1 are equal, independent control of both b and α can be achieved, i.e. there is independent control of the magnitudes of $\bar{\sigma}_1$, $\bar{\sigma}_2$ and $\bar{\sigma}_3$ and of the orientation of $\bar{\sigma}_1$.

Tests may be performed in the HCA in which the principal stress directions are rotated continuously or are held constant at any angle during shear. Such flexibility is essential to understand the effects of principal stress rotation. Initial anisotropy must first be identified and is better investigated by inclining the principal stresses in a controlled fashion as opposed to inclining the specimen (Saada, 1970).

In principle, deformations can also be controlled so that average plane strain conditions can be maintained in the HCA during shear with principal stress rotation. The definitions for strain in terms of cylindrical co-ordinates for situations independent of z and θ are:-

$$\text{Radial strain, } \epsilon_r = -\partial u / \partial r$$

$$\text{Circumferential strain, } \epsilon_\theta = -u/r$$

where u is the displacement of a point in the radial direction. In a hollow cylindrical sample, the application of torque causes rotation of the principal stresses in concentric surfaces. The condition of plane strain then implies no strain in the radial direction, i.e.

$\epsilon_r = 0$. This is satisfied, on average, by ensuring equal displacements of the inner and outer cylinder wall.

The shortcomings in the simple shear apparatus, namely the incomplete development of a complementary shear stress and the restriction on the zero-extension direction, are avoided in the HCA. In hollow cylindrical specimens, the surfaces in which principal stress directions are being varied are continuous; complementary shear stresses develop automati-

cally. Neglecting end restraint, the specimen is free to deform radially and there is no restriction on the zero-extension directions.

There are, however, shortcomings in the use of hollow cylindrical specimens which must be recognised. These concern the non-uniformity of stress (and hence strain) across the specimen wall which arise because of:-

- (i) the specimen's curvature, and
- (ii) restraint at the specimen's end loading plattens.

These two causes of stress non-uniformity are considered subsequently in detail. Minimising the non-uniformities was an essential step in developing the new apparatus.

.4 REVIEW OF EARLIER WORK WITH HOLLOW CYLINDER APPARATUS

Hollow cylindrical specimens have been tested extensively in materials research, e.g. Taylor and Quinney (1931) and Goode and Helmy (1967) tested metal and concrete elements, respectively, under combined axial force and torque. Hollow cylinders of soil were tested first by Cooling and Smith (1936) who subjected unconfined elements to torque; Geuze and Tan (1952) also applied torque to long thin hollow cylinders of clay in their study of rheology. Since that time, hollow cylindrical soil specimens have been used by a number of researchers, employing a greater control of applied stresses and forces and taking advantage of the features hollow cylinders offer, to investigate various aspects of soil behaviour.

This recent work has been reviewed to provide a background on apparatus design and sample geometry. Only apparatus in which there is control of p_0 and p_1 have been considered; ring shear and torsion shear devices have thereby been excluded. Table 7.1 lists the investigators, the sample dimensions which they adopted, the loading capabilities of the apparatus and the subject of their investigations. A summary of the geometries used for the samples tested by these various investigators is presented in Fig. 7.4. The geometry has been expressed in terms of the ratio of inner to outer radius, a/b , against outer radius, b .

It is convenient to consider separately investigations made with apparatus in which there was no provision for torque (category A) and investigations with apparatus capable of applying torque (category B).

In category A there is no possibility of continuous rotation of principal stresses. Hollow cylindrical specimens have been used to investigate the effects of the intermediate principal stress, σ_2 , on failure, e.g. Kirkpatrick (1957), Broms and Ratnam (1963), Wu et al (1963), Earig and Bemben (1965), Barden and Procter (1971). They have also been used to assist in the interpretation of the triaxial test on solid cylindrical specimens, in which the radial and circumferential stresses, σ_r and σ_θ , are conventionally assumed to be equal (Broms and Jamal, 1965; Jamal, 1972). Dusseault (1981) has recently described a new apparatus without the provision for torque, which is being used to investigate stress paths typical of tunnelling operations.

Special uses of hollow cylinder configuration, loaded without torque, have been made in soils testing. For example, Suklje and Drnovsek (1965) investigated the tensile behaviour of a clay using hollow cylindrical samples. Whitman and Luscher (1967) used hollow cylinders to study the interaction between pipes and sand; Jamal (1971) to examine the shear strength of sand in extension.

Saada (e.g. Saada and Baah, 1967; Saada and Zamani, 1969; Saada and Ou, 1973; Saada and Bianchini, 1975) appears to have made the most extensive use of a hollow cylinder apparatus with capability of applying torque. He has restricted himself deliberately to tests in which:-

(i) $p_o = p_i$, and

(ii) $\alpha = \text{constant}$ during shear,

allowing him to investigate the inherent anisotropy in clays. With p_o equal to p_i , Saada considers that σ_r and σ_θ are equal and their distribution is uniform ; there may be, however, the effects of variations in shear stress across the wall superimposed on these distributions. The conditions imposed by Saada mean that tests performed with different values for α inevitably imply differences in b , so that the effect of each has not been separated.

Tests with a hollow cylinder apparatus in which principal stresses have been rotated in a controlled fashion during monotonic shear appear to be restricted to those by Broms and Casbarian (1965) and Lade (1975). Broms and Casbarian (1965) tested isotropically consolidated clays undrained under the following conditions:-

- (i) b constant, α continuously varying to failure,
- (ii) α of zero, b varying,
- (iii) both α and b varying.

Their aim was to determine the effect of these variables on the shear strength parameters. Lade (1975) sheared isotropically and anisotropically consolidated sand under drained conditions, limiting himself to tests in which M_T was increased while W , p_o and p_i were held constant; in these tests, σ_1 , σ_3 and their orientation changed simultaneously, together, therefore, with b .

Finally, in studies of liquefaction, hollow cylindrical samples have been used to produce cyclic rotation of principal stress directions (e.g. Ishibashi and Sherif, 1974; Muramatsu and Tatsuoka, 1981). Generally, these investigations have not been designed as a fundamental study of principal stress rotation effects, but have sought to simulate field conditions during earthquake loading. The devices are frequently referred to as 'torsion (simple) shear apparatus' and not as 'hollow cylinder apparatus'.

5 STRESS DISTRIBUTIONS IN HOLLOW CYLINDRICAL SPECIMENS

Stress non-uniformities arising from curvature

Even when boundary stresses are uniformly applied and there is no end restraint, stress and strain variations can occur across the wall of a hollow cylindrical specimen. These variations arise when either torque or different internal and external pressures, p_i and p_o , are applied. Differences between p_i and p_o give rise to variations in σ_r and σ_θ across the wall. Torque leads to variations in shear stress $\tau_{\theta z}$ across the wall, which, in turn, may cause non-uniformities in σ_z , σ_r and σ_θ , even for the case of p_i and p_o equal. In general, the stress and strain distributions depend on the material's constitutive law.

Interpretation of tests in the new hollow cylinder apparatus is made by considering the specimen as a single element and calculating values for stress and strain using the expressions given in Fig. 7.5.

The average stress values, $\bar{\sigma}_z$ and $\bar{\sigma}_\bullet$, are based on equilibrium considerations and the average strain values, $\bar{\epsilon}_z$ and $\bar{\gamma}_{\bullet z}$, are based on strain compatibility only. Therefore, the resulting expressions for the averages are always valid and independent of the constitutive law of the material.

The expressions used for the remaining averages of stress and strain are based on assumptions of a linear elastic stress distribution ($\bar{\sigma}_r$), a uniform stress distribution ($\bar{\tau}_{\bullet z}$) and a linear variation of radial displacement across the wall ($\bar{\epsilon}_r$ and $\bar{\epsilon}_\bullet$). It should be noted that these definitions ensure that $\sum \bar{\sigma} \cdot \bar{\epsilon}$ is equal to the work per unit volume done by the external forces.

Since neither the stresses nor the strains can be conveniently measured directly across the wall of the hollow cylinder, it is essential to set bounds to:-

- (i) the differences between the calculated and real averages, and
- (ii) the magnitude of deviations from the real averages, i.e. the level of non-uniformity,

all in relation to the applied boundary forces and pressures, and to the externally measured displacements and rotations. This work has been described by Gens, Hight, Potts and Symes (1982).

The magnitude of the difference between the calculated and real stress average can be characterised by the normalised parameter

$$\beta_1 = \frac{|\bar{\sigma}^* - \bar{\sigma}|}{\sigma_L}$$

where $\bar{\sigma}^*$ is the real average, $\bar{\sigma}$ the calculated average and σ_L a measure of the stress level which is defined for each particular case below. With this definition, β_1 is inversely related to accuracy.

To quantify the level of non-uniformity of stresses, the parameter

$$\beta_3 = \frac{a \int_a^b |\sigma(r) - \bar{\sigma}| dr}{(b-a) \sigma_L}$$

is used, where $\sigma(r)$ is the distribution of the particular stress under consideration across the hollow cylindrical specimen. Therefore, β_3 is a measure of the average of the absolute values of the differences between the stress distribution and the real average (see Fig. 7.6). Similar definitions for β_1 and β_3 are used for differences in strain averages and for strain non-uniformities.

The magnitudes of β_1 and β_3 are dependent on sample geometry and on the stress path to be followed, in particular, the difference to be established between $\bar{\sigma}_r$ and $\bar{\sigma}_\theta$ and hence the pressure difference across the wall. Since minimising the non-uniformities and the differences between real and calculated averages formed the basis for selecting the sample geometry for the new apparatus, a brief summary of the findings of Gens et al (1982) will be presented here.

For the case of isotropic linear elasticity (Timoshenko and Goodier, 1951), the influence of sample geometry, expressed as the ratio of inner to outer wall radius, a/b , is illustrated in Figs 7.7, 7.8 and 7.9. The level of non-uniformity, β_3 , for the circumferential stress, σ_θ , and radial stress, σ_r , is shown in Fig. 7.7 for several ratios of $\bar{\sigma}_r/\bar{\sigma}_\theta$; β_3 for the circumferential strain, ϵ_θ , and radial strain, ϵ_r , is shown in Fig. 7.8 for three values of Poisson's ratio and for the same ratios of $\bar{\sigma}_r/\bar{\sigma}_\theta$. The normalised difference between calculated and real average, β_1 , for ϵ_θ and ϵ_r is shown in Fig. 7.9 for the same conditions. β_1 for σ_θ and σ_r is zero, of course, since linear elasticity was used to determine $\bar{\sigma}_r$ and equilibrium was used to determine $\bar{\sigma}_\theta$. The stress level σ_L has been defined as $(|\bar{\sigma}_\theta| + |\bar{\sigma}_r|)/2$ and the strain level ϵ_L as $(|\bar{\epsilon}_\theta| + |\bar{\epsilon}_r|)/2$ in recognition of the fact that the linear elastic analysis is independent of the vertical stress and strain.

The parameters β_1 and β_3 have also been examined, by means of a finite element analysis, for the case of a hollow cylindrical sample

for which the material's behaviour can be described by the Modified Cam-clay form of strain hardening plasticity. As an example, Fig. 7.10 shows the variation of β_3 for σ_θ , σ_r and σ_z during a hollow cylinder test which follows the stress path shown in the figure; the particular geometry used was defined by a/b equal to 0.8. It should be noted that there is now non-uniformity in the distribution of σ_z , and this is of the same order as that for σ_r and σ_θ . The corresponding values of β_3 for the elastic case have been superimposed on Fig. 7.10. It can be seen that the difference between the elastic and elasto-plastic analyses is small and the same is true for the other cases examined (Gens et al, 1982). As an example of the stress distributions across the wall, the variations of σ_z , σ_r and σ_θ found in the elasto-plastic analysis at a stress level, q , of two thirds the failure stress, q_f , are plotted in Fig. 7.11; corresponding stress distributions from a linear elastic analysis are shown alongside.

The assumption of a uniform distribution of shear stress across the wall, which has been used in calculating the average shear stress, can be taken as a limiting elasto-plastic case for which β_1 and β_3 are, of course, zero. For the case of linear elasticity, the shear stress variation across the wall leads to the relationships between β_1 , β_3 and a/b shown in Fig. 7.12. The stress level, τ_L , is defined here as $\bar{\tau}$.

Stress non-uniformities arising from end restraint

In common with all testing apparatus utilising rigid loading platens, non-uniformities in stress and strain are introduced by the frictional restraint and stiffness of the platens; in the HCA, these non-uniformities are additional to those arising from the specimen's curvature. In many devices it is possible to reduce the frictional restraint by lubrication of the platens. While not impossible, such solutions are made difficult in the HCA in which friction at the platens is necessary for transmission of torque to the sample.

Restraint on radial displacement of the specimen ends in the HCA is associated with the development of radial shear stresses, τ_{rz} , and their complementary stresses, τ_{rz} , which decay with distance from the platens. These shear stresses result predominantly in additional circumferential stress, in bending moments which affect the distribution

of vertical stress and in rotations of principal stress out of the plane of the cylinder wall. The extent of the stress disturbance is clearly dependent on sample geometry, the specimen's constitutive law, and the applied pressure and load combinations.

At the time of the conceptual design of the new hollow cylinder apparatus (1975), no results had been published of analyses of the effects of end restraint in the testing of hollow soil cylinders. A detailed study of the stress distribution in specimens of different geometry was considered to be a pre-requisite to development of a new apparatus. Finite element analyses were undertaken for a range of specimen geometries under different load combinations. Both isotropic linear elastic and plastic formulations were used to represent the soil. The detailed results of the elastic analyses are presented in Appendix 1 to this Chapter. Results of the plastic analyses are described by Gens et al (1982).

The linear elastic analysis demonstrated that end restraint has only a local effect on the distribution of radial stress, σ_r , for all load/pressure combinations. Changes in the distributions of both σ_r and σ_z are sensitive to geometry, more so to height than radius, to the load/pressure combinations, particularly $(p_o - p_i)$, and to Poisson's ratio. When the inner and outer cell pressures are equal, the effects of end restraint for a given geometry are minimised. The distributions of vertical stress, σ_z , are affected by both end restraint and bending in the top cap; in the analyses, the axial load was applied at the central axis of the sample whereas the final design ensures that the axial load is applied over the wall of the sample. There is, of course, a similar pattern regarding the influence of end restraint on displacements.

As an illustration of the effects of end restraint, Fig. 7.13 shows the results from a series of linear elastic analyses of fixed-end samples of different height, and of inner and outer diameters of 200mm and 250mm respectively. The combination of applied stresses comprised an axial stress of 400kPa and inner and outer cell pressures of 350kPa and 400kPa respectively. In these analyses, the torque was omitted to retain axisymmetric conditions. For linear elasticity, the addition of a torque does not affect the stress non-uniformities resulting from end restraint. The resulting stresses in the wall are expressed in Fig. 7.13 as a percentage of those found from linear elastic analysis of an unrestrained hollow cylinder. The analyses presented in Fig. 7.13 apply for a Poisson's ratio of 0.499. For a geometry of

200mm ID x 250mm OD x 250mm height, the effect of Poisson's ratio on the distribution of both σ_z and σ_θ is shown in Fig. 7.14. The higher level of non-uniformities associated with drained loading (Poisson's ratio less than 0.49) is apparent.

Figure 7.15 presents results from the analysis of a complete hollow cylinder sample with end restraint for which the behaviour was assumed to be strain-hardening plastic as it followed the same average stress path as that shown in Fig. 7.10 (i.e. α of 0° , b of 0.5). A sample with inner and outer diameters of 200 and 250mm and a height of 250mm was modelled. Normalised distributions of σ_z and σ_θ are shown for stress levels corresponding to failure, q_f , and to two-thirds q_f . For comparison the results of equivalent linear elastic analyses ($\nu = 0.3$) are shown alongside. Plastic analyses have shown again that the distribution of σ_r is not significantly affected by end restraint and, so, it has not been plotted. It can be seen from Fig. 7.15 that the distribution of stresses is basically similar for elastic and plastic materials; stress non-uniformities obviously become larger as failure is approached.

It is apparent that reasonably uniform distributions of stress can be obtained with a suitably high sample, at least over a central portion. The effects of the remaining non-uniformities on the interpretation of strength are discussed subsequently.

6 SELECTION OF SAMPLE GEOMETRY

It has been shown that the non-uniformities in stress across the wall of hollow cylindrical specimens, both arising from curvature and end restraint, and the differences between real and calculated averages of stress and strain, are geometry dependent. The philosophy which has been adopted in developing the new hollow cylinder apparatus has been to accept this limitation and minimise it to admissible levels by judicious selection of the sample geometry. This selection has been made in the light of both practical considerations and of the use to which the apparatus is to be put, in particular the range of stress paths to be followed.

Wall thickness

Careful thought was applied first to the selection of an appropriate wall thickness. It was considered that a value of 25.4mm would be able to meet the following criteria:-

- (i) in the preparation of sand samples, the need to ensure:-
 - (a) a wall thickness sufficiently large relative to the sample's maximum grain size so that failure mechanisms would not be constrained;
 - (b) a sample volume which was large in relation to the potential volume change resulting from membrane penetration;
 - (c) a uniform density across the wall;
- (ii) in the preparation of clay samples the need to ensure a wall thickness which was large in relation to the potential thickness of disturbed zones;
- (iii) for all samples the need for a wall thickness which would not be subject to excess disturbance resulting from internal instrumentation for displacements and pore pressures.

Inner and outer radii

Figures 7.7, 7.8, 7.9 and 7.12 show clearly the strong dependence of both the non-uniformities of stress and strain across the hollow cylinder wall and the difference between the real and calculated averages on:

- (i) the geometry of the sample, both β_1 and β_3 reducing as the ratio of inner to outer radii a/b approaches unity, and
- (ii) the stress path followed, both β_1 and β_3 increasing as $\bar{\sigma}_r/\bar{\sigma}_\theta$ reduces below one.

Stress non-uniformities arising from both curvature and end restraint increase with the pressure difference across the wall, $(p_o - p_i)$. It was sensible, therefore, to select a geometry which would minimise the pressure difference required in a test to achieve a particular difference in $\bar{\sigma}_r$ and $\bar{\sigma}_\theta$.

The pressure difference $(p_o - p_i)$ required to develop a stress difference $(\bar{\sigma}_\theta - \bar{\sigma}_r)$ in a 25.4mm thick wall of different radii can be judged from Fig.7.16. A similar curve has been added to Fig.7.16 to reflect the influence of outer radius, b , on the variation in shear stress across a 25.4mm thick wall.

It is clear from this figure that only a small improvement in stress uniformity is achieved for practical increases in outer radius above circa 125mm. An outer radius of 127mm was selected, therefore, for the hollow cylinder samples. It has the advantage of making feasible the testing of intact field samples of 254mm diameter, and, together with an inner diameter of 203mm, facilitates the use of internal instrumentation, both in terms of the necessary resolution and the methods of deployment.

For the new apparatus, then, the ratio a/b has the value of 0.8. From Figs 7.7 to 7.10 and Fig.7.12 it can be seen that the maximum non-uniformities to be expected, neglecting the effects of end restraint, will be of the order of 11%.

From Fig.7.4 it is apparent that the new apparatus has a higher value for a/b , and, therefore, substantially more uniform stress distributions, than any previously described hollow cylinder apparatus and a much larger value for b , with the exception of the apparatus described by Lomise et al (1969) and Lade (1975) which have very similar values.

Height

With wall thickness and sample diameter fixed, the selection of sample height was based on considerations of end restraint and the need to provide an adequate central gauge length in which stresses and displacements were sufficiently uniform and where displacements and pore pressures could be measured conveniently.

The results from finite element analyses which assumed linear elastic behaviour and which are presented in Figs 7.13 and 7.14 show that, for a combination of inner and outer diameters for the cylinder of 200mm and 250mm, a height of 250mm would provide a central length of sample wall of 125mm in which stresses would differ by no more than 10% from those in an unrestrained linear elastic cylinder. This was considered to be an adequate central gauge length and so a sample height of 254mm has been adopted. The similarity in stress distributions shown in Fig. 7.15 for elastic and plastic analyses suggests that a similar conclusion can be drawn in the case of elasto-plastic material.

The selected height does not meet the recently proposed criterion for hollow cylinder sample height (Saada and Townsend, 1981). This criterion was, however, based on the assumption that the theory of thin elastic cylindrical shells is applicable, and on the suggestion that the central zone which was free from end effects should be the same length as the zone influenced by the plattens.

The selected height is suitable for observing pre-failure behaviour, provided that internal instrumentation is used over the central zone. At or near failure, the height imposes some limitations for certain stress paths, as considered below.

Limitations imposed by the selected geometry

The stress paths to be followed using the HCA, in particular the combination of α and b , determine the pressure difference to be applied across the wall ($p_o - p_i$). As stated previously, this pressure difference influences the level of stress non-uniformities arising from curvature and from end restraint. To illustrate the link between stress combinations and stress non-uniformities arising from curvature alone, bounds for p_o/p_i have been set at 1.2 and 0.9, these values being associated with similar levels of non-uniformities. For the selected geometry, stress states attainable with p_o/p_i between 0.9 and 1.2 have been established for the cases of α of 0° (and 90°) and of α of 45° ; these are shown in Fig. 7.17 in a plane of constant mean normal stress. It is apparent that the limitations imposed by the stress non-uniformities depend on the major principal stress angle, α , being much less for α of 45° than α of 0° . The range of stress paths which can

be investigated using the HCA is selected on the basis of this form of figure.

The stress combinations required to follow the stress paths associated with gravity platforms (i.e. elements on a plane parallel to the direction of wave loading) fall within these bounds. These stress combinations are described in Appendix 2 to this Chapter and the related non-uniformities in stress resulting from end restraint are examined in Appendix 1.

Even with a selected height of 254mm, the effects of end restraint are not totally eliminated and their influence, especially on the interpretation of strength, needs to be considered. The effect of end restraint on the calculated stress average is only significant for the case of $\bar{\sigma}_\theta$. From equilibrium, the error in $\bar{\sigma}_\theta$ can be expressed by

$$\frac{a + b}{2} \cdot \frac{\partial \bar{\sigma}_\theta}{\partial z}$$

The error in friction angle, ϕ' , for example, is determined by the real value of ϕ' and by the magnitude of this expression. As an indication, errors of about 2° are involved in tests with α of 0° and b of 0.5 on material with ϕ' of 30° ; this error is substantially reduced as α increases to 45° (Gens et al, 1982).

PRINCIPAL FEATURES OF DESIGN OF THE NEW HOLLOW CYLINDER APPARATUS

Design criteria

To take account of the previously discussed non-uniformities of stress and strain in a hollow cylinder sample under test, the internal measurement of displacements, rotations and pore pressures over a central gauge length was established as a major design criterion. Sufficient space for, and access to, internal instrumentation of both the inner and outer cells was required. Internal measurement of both axial load and torque was also regarded as an essential requirement.

To enable samples to be formed in place conveniently and without restriction, the decision was taken to mount the loading systems above

the sample and to allow for them to be moved well away from the sample during its installation. Safety considerations of handling a large diameter cell and overhead loading systems suggested that crane operations should be avoided; a self-contained carrying frame was therefore developed.

Stress controlled consolidation, including K_0 consolidation, was an obvious requirement, as was shearing with either stress or displacement control under drained or undrained conditions. In most applications, the apparatus would be used for tests run with four stresses, σ_z , p_o , p_i and $\tau_{\bullet z}$ independently controlled. In addition, an aim would be for these stresses to be varied cyclically, so that the important area concerning continuous cyclic principal stress rotation could be investigated. Taking into account the relatively large volumetric strains involved and the aim for cyclic stress control, compressed air was selected as being the most suitable pressure source. It was considered to be essential that the apparatus was capable of working at the stress levels which simulate both stress history and applied loading in field situations.

Simultaneous variation of four stresses (σ_z , p_o , p_i and $\tau_{\bullet z}$) to achieve independent control of the magnitude and direction of the principal stresses in the wall of the hollow cylinder is clearly a formidable undertaking, especially when these are to be varied cyclically. Much computational effort is needed during a test. It was appreciated at the outset that some form of interactive control using a computer logger would become necessary, particularly as the complexity of the tests increased. The design of the loading systems was geared towards this and provision was made for incorporating a closed-loop computer control system, similar in principal to that developed for control of the hydraulic triaxial cell (Chapter 5).

General description

The general layout of the apparatus is shown in Fig.7.18 and Plates 7.1 and 7.2. It can be described in detail by subdividing into the following components:-

- (i) cell base and sample pedestal,
- (ii) cell chamber,
- (iii) the axial force and torque loading systems,

(iv) the radial pressure systems,

(v) the support frame.

(i) Cell base and sample pedestal

The stainless steel base, 698mm diameter, stands on three legs and supports the sample during its preparation and loading. It has a high stiffness to minimise deflection under vertical load and is isolated from the support frame to reduce the effects of vibrations during the final stages of assembly. The base carries the annular sample pedestal and the necessary connections for sample drainage. Access to the inside of the hollow cylindrical samples is through a 241mm diameter opening in the centre of the base, which is sealed with a cover plate.

The pedestal is removable so that the porous ceramic annulus (A80KV vitrified bauxilite) which is screwed to it and through which drainage takes place, can be de-aired. Simple modifications can be made to cater for changes in sample diameter. The inner rubber membrane is fixed to the pedestal using the detail shown in Fig.7.19.

(ii) Cell chamber

The perspex cell chamber has an overall height of 660mm, an external diameter of 597mm and an internal diameter of 521mm. Twelve stainless steel tie bars, 25mm diameter and 820mm long provide reaction to the cell pressure.

(iii) The axial force and torque loading systems

Air pistons provide both the axial force and torque. For axial load, two double acting air pistons, P1 and P2 in Fig.7.18, have been mounted either side of the central loading ram. These are capable of providing either an upward or downward force to the loading ram, for subjecting the sample to compression or extension.

Spring-loaded, single-acting air pistons, P3 and P4 in Fig.7.18, work in opposition, refer Fig.7.20, to provide a net horizontal force which is applied to the loading ram at a radius of 152mm via a

chain and sprocket drive. The sprocket can be removed to allow changes in diameter and thus give flexibility in the size of torque that can be applied. Both sets of pistons can be used to provide displacement control simply by introducing water at any desired rate into the piston. However, it is envisaged that most tests will be stress-controlled.

To accommodate vertical displacements of the sample while applying both the torque and axial load, the pistons P3 and P4 have been mounted on a platform which is, in turn, supported by pistons P1 and P2. To ensure that the load is applied concentrically, the platform runs up and down in linear bearings (B1, B2) on two vertical 37.5mm diameter guides. These guides provide the reaction to the torque generated by pistons P3 and P4; they have been set as far apart as possible to reduce the horizontal force acting on them and, hence the deflection of the guides in bending. An outer torque frame, rectangular in cross section and set at the extremities of the cell diameter, transfers the torque to the cell top and thence to the cell base.

The platform is held in place between two deep groove thrust bearings by circlips; these bearings allow rotation of the ram and cater for any imbalance of forces in the lower pistons P1 and P2. Connection between the sprocket and loading ram has been made through a taper pin. To accommodate samples ranging in height between 150mm, 200mm and 250mm, three alternative positions for circlips and taper pin have been provided in the loading ram.

The 50mm diameter loading ram is maintained on a vertical axis by two linear bearings (B3, B4) which are sufficiently long to take out any moment, so that only an axial force and torque is applied to the sample. These bearings are housed in the cell top and the top of the reaction frame. The lower linear bearing is sealed at the top and bottom using a system of three O-rings to allow rotation and vertical movement. Where the loading ram enters the cell, corrosion is prevented by surrounding the ram in oil which is retained in a rubber sleeve.

(iv) The radial pressure systems

Radial stresses are applied directly to the inner and outer cylindrical surfaces of the sample by fluid pressure through flexible membranes. To eliminate the use of air under pressure in the cells, with its inherent safety problem and the undesirable effects of air diffusion through the membranes, air-water interfaces have been introduced in the lines. These interfaces have been designed as hydraulic multipliers to uprate the inner and outer cell fluid pressure from 862kN/m^2 (125 psi), the operating air pressure, to 1280kN/m^2 (185 psi), close to the working pressure limit for the perspex cylinder.

(v) Support frame

The support frame is shown in Plate 7.1. Two threaded columns carry two motor driven gantries; the upper gantry carries the entire loading system and cell top; the lower gantry raises and lowers the cover plate to the inner cell. In Plate 7.3 the upper gantry has been driven to its top position so that free access to the sample is available.

Stress and deformation limits in the apparatus

The apparatus has been designed to examine both the pre-failure behaviour and failure conditions in sands and clays under a wide range of stress levels and stress states.

For the selected sample geometry, the value of vertical stress in the sample, σ_z , depends on p_o , p_i and the axial load W , refer equn (1) in Fig.7.5. The axial load capacity, W , is determined by the piston area, A_a , of P1 and P2, the maximum operating air pressure, $p_{a \text{ max}}$, and to a minor extent by the weight of the loading ram, load cell and torque loading system. In the present configuration parameters A_a and $p_{a \text{ max}}$, have the values shown in Table 7.2, giving an axial load capacity of approximately $\pm 25\text{kN}$.

For the range of cell pressures between 0 and 1280kN/m^2 and for the case of equal inner and outer cell pressures, limits can be established for the value of vertical stress on the sample; these are represented by lines AB and CD in Fig.7.21. Working without rotation of principal stresses, i.e. without application of torque, the apparatus can in principle fail a sample with an unconfined compression strength of 1500kN/m^2 or can apply a confined stress difference of over 1300kN/m^2 at a cell pressure of 1280kN/m^2 . In extension a maximum stress difference ($\sigma_r - \sigma_z$) of 1280kN/m^2 may be applied if the maximum cell pressure is used.

In terms of effective stress the limiting state lines for values of ϕ' up to 40° all lie within the capacity of the cell in compression (Fig.7.21) except at high values of σ_r' .

The torque loading system develops a torque which is determined by the piston area, A_t , in P3 and P4, the radius arm at which the load is applied, r , and by $p_a \text{ max}$. With the values for these parameters listed in Table 7.2 this leads to a maximum torque of 0.2 tonne-m. Under this torque, the average shear stress, equn (4) in Fig.7.5, rises to 1000kN/m^2 .

Combinations of this maximum shear stress with the maximum and minimum values of vertical stress, σ_z , give the range of Mohr circles shown in Fig.7.22. It is evident that in tests in which shear is applied, failure can be achieved in soil having a ϕ' of 38° at the maximum value of radial stress. At lower values of radial stress this limit to ϕ' increases rapidly. For comparison purposes, a line has been added to Fig.7.21 which corresponds to the limiting state line for ϕ' of 40° , with maximum torque applied.

The maximum stroke for the pistons providing the axial load is 117mm, which permits axial strains in excess of 40% to be reached. The piston travel in the torque loading system is 134mm which is equivalent to a rotation of 50° . These strains enable failure to be attained easily.

Control systems

In the control system developed for the hydraulic triaxial cell (Chapter 5) , air servo-valves are rotated through gearboxes by d.c. servo-motors at speeds maintained by servo-amplifiers. A signal to the servo-amplifier is directly related to motor speed; this signal is set either manually using a potentiometer or supplied externally by a digital-to-analogue converter interfaced between a computer and the servo-amplifier.

The same principles have been used in the control system for the HCA; the four channel system is shown as a line diagram in Fig. 7.23. Details of the air control are shown in Plate 7.4.

For vertical load, the variable pressure is supplied to one side only of the double acting pistons; a base pressure is maintained on the other side of the pistons and by taking the variable pressure above or below this base level, upward or downward forces may be applied to the loading ram. For torque, the variable pressure is supplied to only one of the single acting pistons while the other piston is supplied with a base pressure. Once again, this enables the direction of torque to be reversed by taking the variable pressure above or below the base level. The inner and outer cell pressures are controlled independently through the air-water interfaces.

3 INSTRUMENTATION

Deformations and rotations

Monitoring deformations over a central gauge length of the sample eliminates errors present in overall measurements, which result from bedding and compliance, and reduces the influence of end restraint on the observations. However, in monitoring displacement between two points on the sample wall, the measuring device must take account of the relative radial, vertical and circumferential components of movement between the two points. It must also be capable of isolating potential errors resulting from sample bulging or tilting.

At present two internal measuring systems developed by Symes (1983) are in operation:-

- (i) For radial displacements of the inner and outer surfaces of the cylinder wall (u_i , u_o) proximity transducers (Cole, 1978) are used. These devices, which have been sealed against water entry, sense the gap (up to 6mm) between the transducer face and an aluminium foil target attached to the membrane. The target is made large enough to cater for rotation and change in height of the sample. The devices, which can be seen in Plates 7.5(a) and (b) are mounted at 120° centres at the sample mid-height, three in the outer cell monitoring the outer wall movement and three in the inner cell monitoring the inner wall movement (Fig. 7.24).
- (ii) For vertical and circumferential displacements (w , v) two variants of the devices described by Burland and Symes (1982), which utilise electrolevels, are employed at six locations on the outside wall, see Plate 7.5(a) and Fig. 7.24. In one form of the device, the electrolevel provides a measure of circumferential displacement; in the other a combination of two electrolevels provides a measure of the vertical and circumferential displacements which are automatically corrected for the effects of radial movements.

For the preliminary tests, carried out by Symes (1983), information from these devices has been supplemented by the following external measurements:-

- (i) overall vertical displacement using a long stroke resistive displacement transducer mounted on the loading ram and in contact with the cell top,
- (ii) overall rotational movement, with a similar transducer mounted horizontally on an arm fixed to the loading ram.

Volume change measurements

Although the internal measuring systems provide a complete description of the state of strain in the central section of the sample, overall volume change measurements in both the inner and outer cells have also been made by monitoring the displacement of the piston in the air-water interfaces.

Changes in volume of the sample are currently measured in the conventional way with a 100cc capacity paraffin volume gauge (Bishop and Henkel, 1962).

Photogrammetry

In addition to point-to-point measurements of relative displacements and rotations, a global view of deformation is obtained by photogrammetry of a grid inscribed on the outer membrane. Photographic pairs are taken of the grid at intervals during the test and it is anticipated that it will be possible to resolve vertical and circumferential displacements to 0.1mm. As well as providing a check on the internal point-to-point measurements, this method gives information on overall uniformity of sample deformation, particularly as failure is approached.

Axial load and torque

An internal load cell which eliminates errors from friction forces on the shaft has been developed for independent measurement of axial load and torque. The load cell is connected to the central loading ram by a taper and Woodruffe key-way. Following the principle utilised in the Imperial College internal load cell for the triaxial apparatus (Bishop et al, 1975) the transducer has been made independent of the effects of changes in the external cell pressure by filling it with oil and providing a flexible cover, see Fig.7.19. The measurement of the axial load and torque is achieved by means of strain gauges attached to three webs (Fig.7.19). By careful centring of the strain gauges it has been possible to reduce the interaction between axial load and torque to negligible values. The load cell has a capacity of 10 kN (1 ton) for axial load and 0.44 kN-m (1.75 ton-in) for torque. A second load cell has been constructed with capacities of 26.8 kN (2.68 ton) and 2.04 kN-m (8.04 ton-in).

Pressure

Internal and external cell pressures and base pore pressures are measured with unbonded resistive pressure transducers which are vented to atmosphere. There is ample room for installation of the piezometers described in Chapter 6 on the inner or outer wall for undrained tests.

The resolutions achieved with the various monitoring devices are summarised in Table 7.3.

9 SAMPLE ASSEMBLY AND SEALING

A method for the preparation of hollow cylindrical samples of in sands 203mm ID x 254mm OD x 254mm high has been fully developed and is described below. The feasibility of preparing samples of a stone-free low plasticity clay, using a fly cutter, has been demonstrated.

The inner and outer moulds used in forming sand samples are shown in plates 7.3 and 7.6. The outer mould consists of three segments held together by means of tapered pins; the inner mould is made up of six segments which are held together during installation by four O-rings and which can be expanded radially by means of a central tapered section. The taper allows for the necessary inward collapse of the mould after forming the sample and applying the suction. The outer mould has been shaped to provide a flare to the top and bottom of the outer wall. The flares provide an increased area over which the torque can be transferred, eliminating the danger of slip and the need for protruding fins in the porous ceramic. To minimise stress concentrations revealed by finite element analyses, the flares were given a parabolic shape; this can be seen clearly in Plate 7.5(a).

To form a sample the moulds are assembled over the pedestal and the outer and inner membranes are positioned over the surface of these moulds, see Plate 7.3. The sand is very carefully de-aired in a separate chamber by flushing through de-aired water under vacuum. The sand is then siphoned from this chamber into de-aired water in the annular space between the membranes. Dense samples are obtained by applying vibrations from an eccentrically loaded motor which can be attached to the base pedestal of the apparatus. Loose samples are formed below water by raining sand from a fixed height of 100mm above the sand surface.

When the membranes have been sealed on to the top cap, a suction of 30kPa is applied and the moulds are removed. While under a suction, the internal instrumentation is positioned. Plates 7.5(a) and 7.5(b) show a specimen at this stage. The 300mm diameter cover plate, separating the inner and outer cells is placed next and the inner cell is filled with water; a sealing plug in the top cap cover plate is screwed down when air has been flushed from the inner cell.

Finally, connection between the load cell and cover plate is made when the cell chamber is lowered into position. Correct alignment of this connection is essential, particularly since both axial force and torque are transmitted. The solution adopted is shown in Fig.7.19. The connector comprises three 37.5mm diameter by 41mm high pots bolted to the sample top plate which are filled with a liquid polyester resin immediately prior to lowering the cell chamber and load cell. Threaded bolts protruding from the lower plate of the load cell enter the pots and, when hardened, the resin transmits the required forces between the apparatus and sample, forming a rigid junction between the two. Any slight misalignment is accommodated within the pots. On completion of a test the pots can be unscrewed from the top plate by gaining access through the inner cell.

10 RESULTS OF PROVING TESTS

The results of three proving tests carried out by Symes (1983) are described to demonstrate the capabilities of the newly developed apparatus.

Prescribed Stress Paths

In these tests the mean effective stress was held constant throughout at 200 kN/m^2 , and p_0 and p_i were kept equal. The stress paths which were followed are shown conveniently in plots of stress ratio (σ_1'/σ_3') against major principal stress direction, α , refer Fig. 7.25. The paths can be divided into two parts. In the first part, three different stress paths were followed up to point B, i.e. up to a stress ratio of 3.5 and α of 45° . For test E1 α was maintained at 45° while the stress ratio was increased to 3.5. In Test E2 a triaxial compression stage (i.e. α of 0 and b of 0) up to a stress ratio of 3.5 (points O to C) was followed by a continuous rotation of the major principal stress direction from α of 0° to α of 45° while maintaining the principal stress ratio constant (i.e. points C to B). For test E3 the stress ratio was increased to 3.5 while simultaneously the direction of the major principal stress was rotated continuously from 0 to 45° (i.e. points O to B). Along paths CB and OB, b also varied, p_0 being equal to p_i .

In the second part a common path was followed, along BF, which involved shearing with the major principal stress direction constant at 45° .

The required combinations of σ_z , $\tau_{\theta z}$, σ_r and σ_θ to follow these paths are plotted in Fig. 7.26. Along BF only a torque was applied while σ_z , p_o and p_i were maintained constant and equal.

Material and Test Procedures

The sand being used has an average grain size of $400\mu\text{m}$ and has been described in detail by Reades and Green (1976). The specimens were prepared in the manner described previously to a void ratio of 0.650 (e_{max} is 0.923 and e_{min} is 0.613) and consolidated isotropically to an effective stress of 200kPa, using a back pressure of 400kPa to ensure saturation.

Prior to the main shear stage of the test the samples were subjected to a triaxial compression load-unload cycle, reaching a maximum stress ratio of 2.6. The purpose of this was twofold:-

- (i) to induce a common degree of anisotropy in each specimen,
and
- (ii) to check the repeatability of the specimen preparation method.

In Fig. 7.27 the stress-strain and volumetric strain curves recorded during the cycle in each test can be compared. Clearly, good repeatability is achieved in the samples.

For manual control of the tests the nominated stress path is followed by selecting target points on the path and moving between consecutive points. This involves calculating the necessary changes in W , M_T and p and applying these simultaneously at a constant rate over a period of 10 minutes. Drainage is usually not complete at the end of the step and an additional period of 5 minutes is allowed for full consolidation. The small size of the increments is shown in Fig. 7.25.

Performance

(i) Stress Path Control

The accuracy with which prescribed stress paths can be followed is determined by:-

- (a) the precision with which the air pressures can be set in the axial and torque loading systems and in the air-water interfaces,
- (b) the stability of the air pressures, and
- (c) the resolution of the measuring systems.

Table 7.4 summarises the limits between which the various stresses can be controlled. Control of shear stress, $\tau_{\theta z}$, is particularly sensitive to the accuracy with which radial displacements are measured. In tests which require differences to be established between σ_r and σ_θ , small differences must be maintained between the inner and outer cell pressures, p_i and p_o . Measurement and control of p_i and p_o then become crucial in determining the accuracy with which the stress path is followed.

Using manual setting of the servo amplifiers, the stress paths actually followed in Tests E1, E2 and E3 can be compared with the prescribed paths in Fig.7.25. Bearing in mind the complexity of the paths, shown in Fig.7.26, the control is considered to be satisfactory. Refinements in technique now enable paths to be followed to within $\pm 1 \text{ kN/m}^2$.

(ii) Sample Uniformity

An overall impression of sample uniformity has been gained from the photogrammetric measurements. These indicate that up to failure non-uniformities in displacement are small. Plate 7.7(a) shows sample E2 approaching failure; the grid inscribed on the outer membrane shows remarkably uniform deformation.

In tests in which shear zones formed post-peak there was no tendency for them to initiate at any preferred location. Plate 7.7(b)

illustrates the post-peak deformation pattern in sample E1 . The continuous spiral shear zone formed almost simultaneously throughout the sample and was continuous across the wall.

(iii) Comparison of Internal and External
Displacement Measurements

In Fig. 7.28a comparison is made of the axial, radial and shear strains determined from both internal and external displacement measurements in test E3. Corrections for load cell deflection and length of flare have been made in the external displacement measurements. In the case of the derivation of radial strain (and also circumferential strain) from external measurements, this was based on measurement of volume changes in the inner cell and sample, together with axial displacement and the assumption of deformation as a right cylinder.

The axial strain measurements shown in Fig. 7.28 illustrate the bedding error involved in the external measurement; the greater axial extension indicated externally when increasing stress ratio from 3.0 to 3.4 suggests a further influence of end effects. The internal and external shear strain measurements show excellent agreement up to failure, providing evidence for the absence of slip at the loading platens and confirmation of the uniformity of shear strain along the full height of the sample. Shear strain measurements post-peak are especially interesting. For the data shown in Fig. 7.28, one inclinometer straddled a shear zone and showed large shear strains across the failure zone; corresponding shear strains were monitored externally; inclinometers remote from the shear zone indicated only small further shear strains in this stage. The differences between radial strains derived from external and internal measurements are similar to those found for axial strains. This results from the 'external radial strain' being derived, in part, from external axial strain measurements and being influenced, therefore, by the same bedding error.

Results for Alternative Stress Paths to Point B

The response of the sand to the different stress paths to point B can be compared in Fig. 7.29. In Fig. 7.30 stress ratio, σ_1'/σ_3' , and the direction of the major principal stress, α , are plotted against α_e , the direction of the major principal strain increment measured relative to the vertical, for the complete test.

The test results illustrate the potential value of the HCA in defining material behaviour. For example, the results indicate that:-

- (i) the strains which develop during shear show strong path dependence, including dependence on b and α ;
- (ii) principal stress rotation at constant stress ratio can be associated with substantial volumetric compression;
- (iii) in general there was no coincidence between the directions of principal strain increment and principal stresses;
- (iv) alternative stress paths to a common point in stress space may, under certain circumstances, result in similar overall volumetric strains.

Results for Common Stress Path to Failure, BF

The results for this stage of the tests are shown in Fig. 7.31.

Further insight into behaviour is made possible by the HCA. For example, it is evident that:-

- (i) previous stress history involving principal stress rotation affects the subsequent stiffness when moving along the same stress path to failure but appears to have little effect on the peak drained strength, failure being determined largely by initial fabric;
- (ii) at failure there are only small differences in the direction of the strain increment.

.11 SUMMARY

Rotation of principal stress directions and changes in the relative magnitude of the intermediate principal stress are features of the stress paths associated with most field loading and unloading problems. In many cases, these rotations of principal stress direction are cyclic.

Simulation of these features of stress paths is difficult to achieve in laboratory tests on soil elements. In the hollow cylinder apparatus, the magnitude and direction of the major and minor principal stresses can be controlled in the wall of hollow cylindrical soil specimens while independently varying the magnitude of the intermediate principal stress, but only at the expense of stress and strain non-uniformities across the wall. The non-uniformities can be reduced by careful selection of the sample geometry.

In the new hollow cylinder apparatus described in this Chapter it has been argued that the effect of the non-uniformities on the observations has been reduced to an acceptable level by:-

- (i) using a sample in which the ratio of inner to outer radius is 0.8, and
- (ii) monitoring displacements (and pore pressures) internally over a central gauge length of the sample. This has been made feasible by adopting the following sample dimensions:

Internal diameter	203mm
External diameter	254mm
Overall height	254mm

There remain, however, areas in stress space where the hollow cylinder apparatus should be used with caution because of the higher level of stress and strain non-uniformities which may develop.

The method of preparing hollow cylindrical samples of sand has been shown to be successful, particularly in the way that excellent repeatability has been obtained. Under manual control the new apparatus has proved to be capable of following complex predefined stress paths which incorporate rotation of the major and minor principal stress directions in a variety of ways, together with changes in the parameter, b . The apparatus can extend the stress path approach to soil testing and can be used in fundamental investigation of anisotropy and principal stress rotation effects in sands and clays.

Results have been presented from tests in which the inner and outer cell pressures have been maintained equal so that the orientation of the major principal stress, α , and the magnitude of the intermediate principal stress relative to the major and minor principal stresses, expressed as b , are linked. These tests indicate that, for prediction of soil behaviour, account must be taken of the complete stress path which is involved, including the rotation in principal stress directions and changes in the relative magnitude of the intermediate principal stress which occur.

Authors	Sample Dimensions (mm)			Loading Capabilities			Subject of Investigation
	ID	OD	H	P_o/P_i	W	M_T	
A. Without rotation of principal stress directions							
1. Kirkpatrick (1957)	63	102	152	✓	-	-	Influence of σ_2 on failure of sand
2. Whitman & Luscher (1962)	25	38/51/76	76/127	✓	-	-	Strength characteristics of hollow cylinders of sand
3. Broms & Ratnam (1963)	76	152	114	✓	✓	-	Effects of anisotropic consolidation on deformation and strength of clay
4. Wu et al (1963)	76	102	152	✓	✓	-	Failure envelope in sand and clay
5. Broms & Jamal (1965)	76	152	305	✓	✓	-	Analysis of the triaxial test on sand
6. Emrig & Bemben (1965)	76	102	203	✓	✓	-	Failure conditions in sand
7. Suklje & Drnovsek (1965)	40	64	80	✓	-	-	Tensile deformation of clays
8. Barden & Proctor (1971)	38	102	152	✓	✓	-	Drained strength of granular material
9. Jamal (1971)	25/51/76	102	203	$P_o = P_i$	✓	-	Shear strength of sand in extension
10. Jamal (1972)	35	60	60	✓	✓	-	Analysis of the triaxial test on clay
11. Dussault (1981)	51	102	200/240	✓	✓	-	Tunnelling and pressuremeter testing in sand
B. With rotation of principal stress directions							
12. Broms & Casbarian (1965)	76	127	254	✓	✓	✓	Effect of principal stress direction and magnitude of σ_2 on shear strength of clay
13. Saada & Baah (1967) (Saada, 1968)	51	71	151	✓	✓	✓	Anisotropy in the deformation and strength of clay (Saada and Zamani, 1969; Saada and Ou, 1973; Saada and Bianchini, 1975) ($P_o = P_i, \alpha = \text{constant during shear}$)
14. Louise et al (1969)	250	310	180	✓	✓	✓	No data presented
15. Iahibashi & Sherif (1974)	51	102	13-25	✓	✓	✓	Effect of K_o on liquefaction of sand
16. Iahihara & Yasuda (1975)	60	100	70	✓	✓	✓	Liquefaction of sand under irregular cyclic loading
17. Lade (1975)	180	220	50	$P_o = P_i$	✓	✓	Influence of stress re-orientation on stress-strain behaviour of sand
18. Iahihara et al (1980)	60	100	106	✓	✓	✓	Effect of principal stress rotation on liquefaction of sand
19. Muramatsu & Tatsuoka (1981)	60	100	100	$P_o = P_i$	✓	✓	Cyclic undrained stress-strain behaviour of sand

Table 7.1 Summary of earlier hollow cylinder apparatuses

<u>Parameter</u>	<u>Value</u>	
Maximum air pressure, $p_{a \text{ max}}$	860 kN/m ²	(125 psi)
Area of air piston for axial load, A_a	15480 mm ²	(24 in ²)
Area of air piston for torque, A_t	15480 mm ²	(24 in ²)
Radius arm for torque, r	152 mm	(6 in)
Maximum cell pressure with 1.5:1 multiplier, p	1290 kN/n ²	(187.5 psi)
Travel of air piston for axial load	135 mm	(5.3 in)
Travel of air piston for torque	117 mm	(4.6 in)

Table 7.2 Details of new hollow cylinder apparatus

Axial Load	0.0084 kN
Torque	0.0005 kN-m
Inner and outer cell pressures	0.064 kN/m ²
Axial displacement - Internal	2 μ m
- External	1.3 μ m
Shear strain - Internal	0.006%
- External	0.002%
Radial displacement - Internal	1 μ m
External*	5 μ m
Volumetric strain	0.001%

*Based on external measurements of volumetric strain and axial displacement.

Table 7.3 Resolution of measuring systems

Axial load, W	± 0.02	kN
Axial stress, σ_z	± 1.0	kN/m ²
Torque, M_T	± 0.001	kN-m
Shear stress, τ_{rz}	± 0.5	kN/m ²
Inner and outer cell pressures, p_i and p_o	± 0.3	kN/m ²

Table 7.4 Limits within which stresses can be controlled
in the new hollow cylinder apparatus

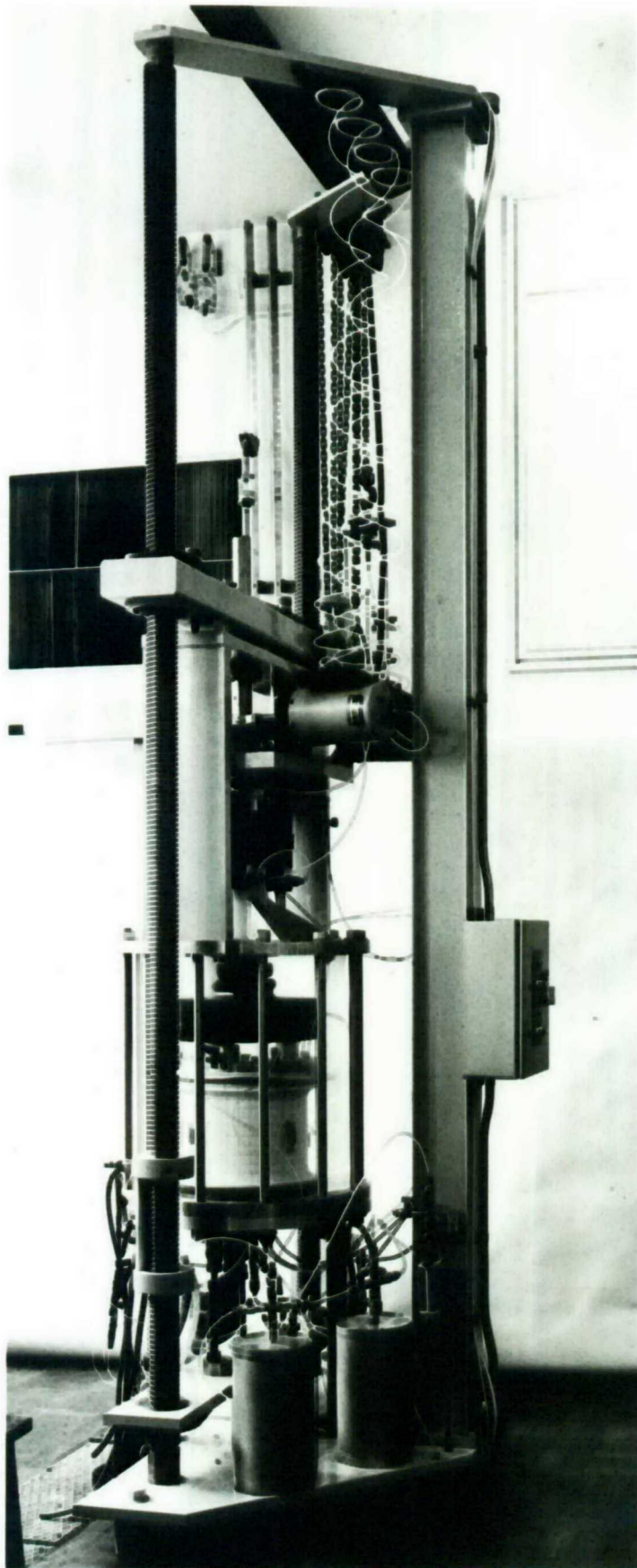


Plate 7.1 The new hollow cylinder apparatus: side elevation showing support frame.

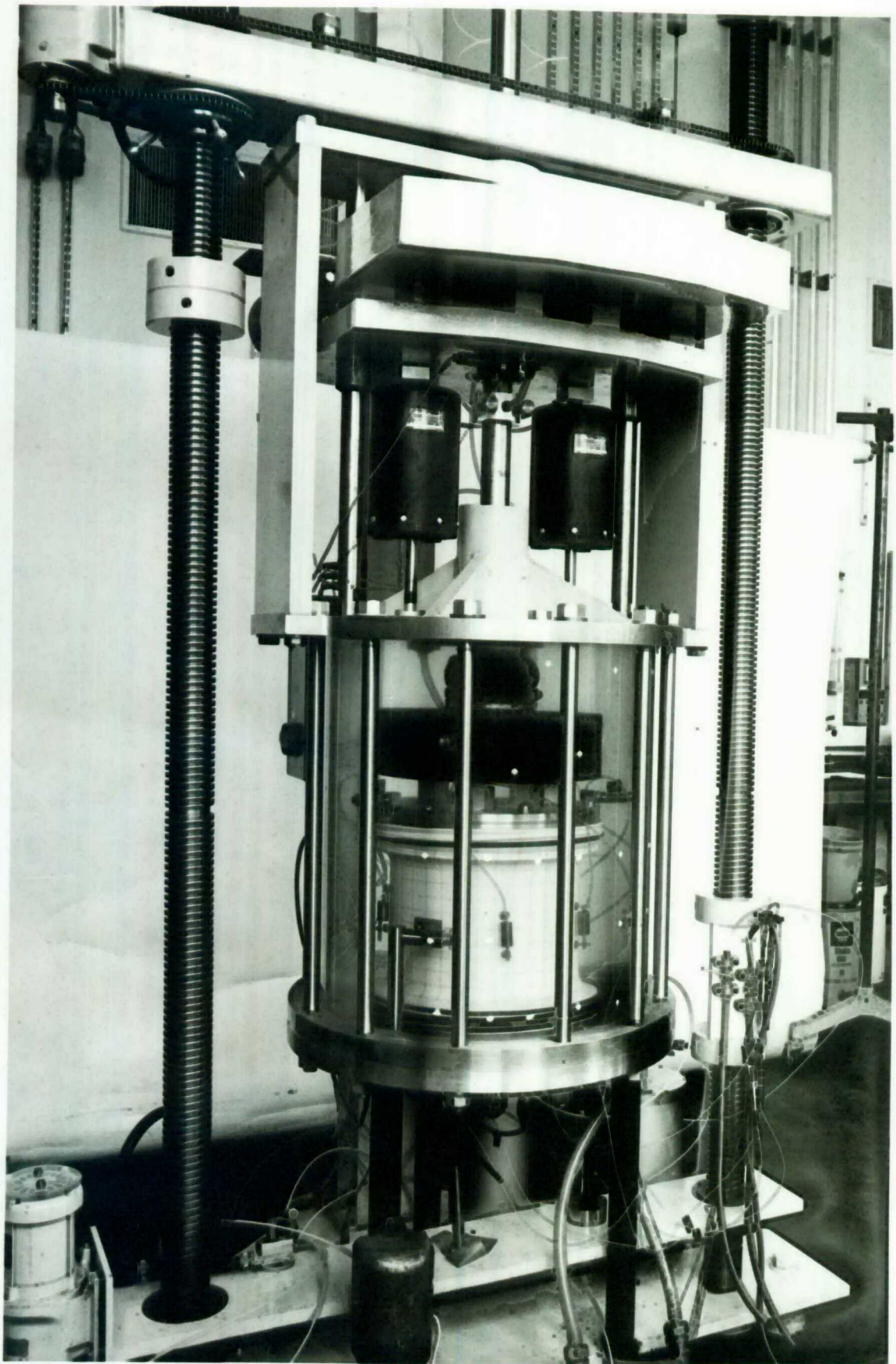


Plate 7.2 The new hollow cylinder apparatus: front elevation.

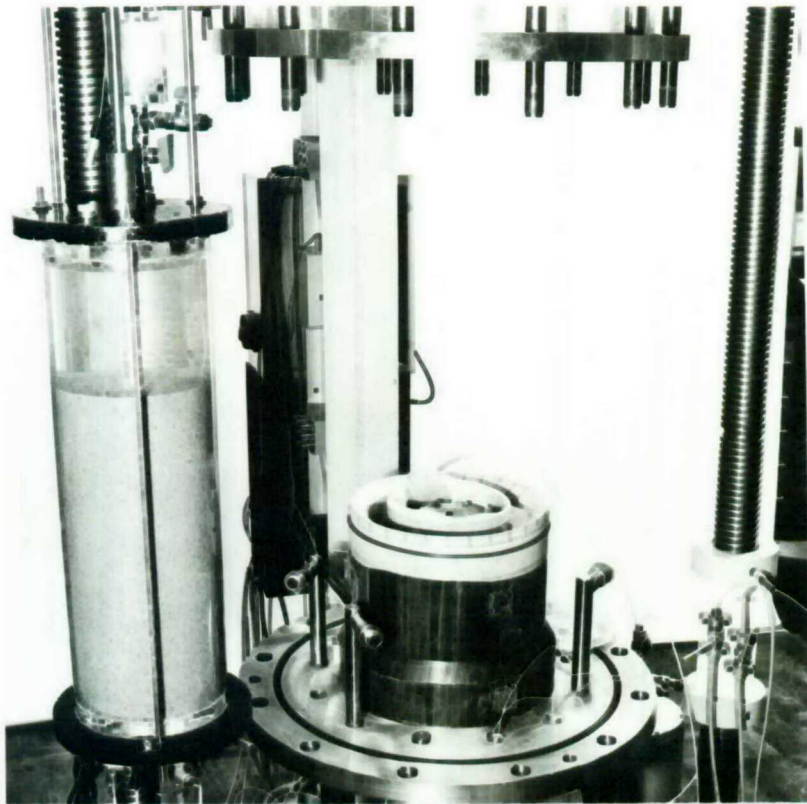


Plate 7.3 Sample preparation with upper gantry driven to top position.

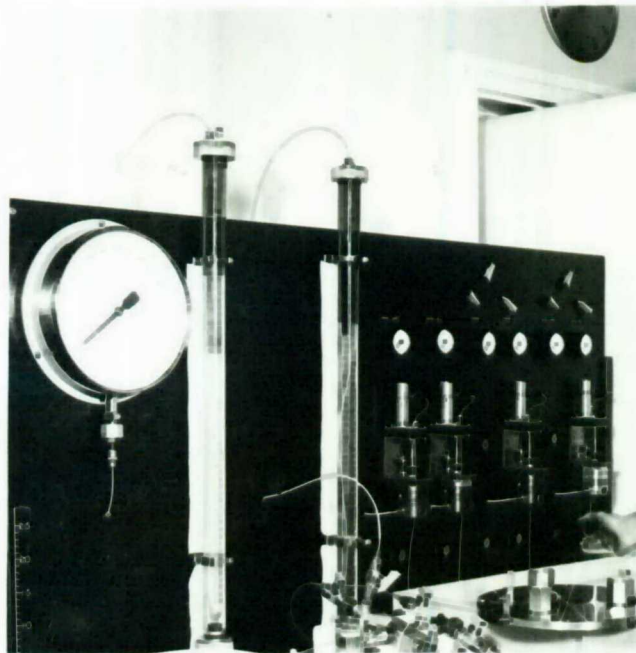


Plate 7.4 Air control system for new hollow cylinder apparatus.

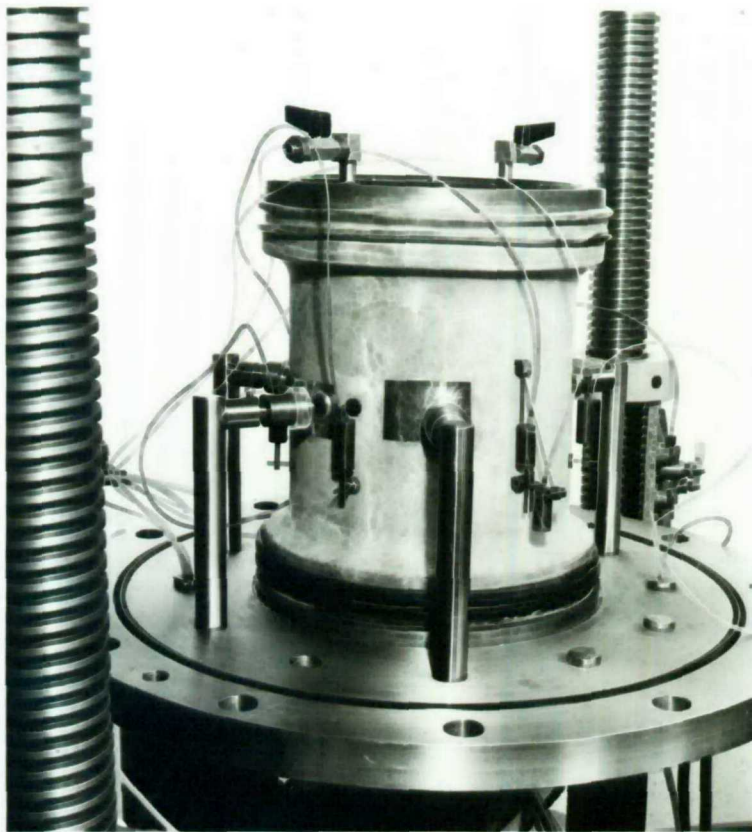


Plate 7.5 (a) Assembled sample showing instrumentation of outer wall.

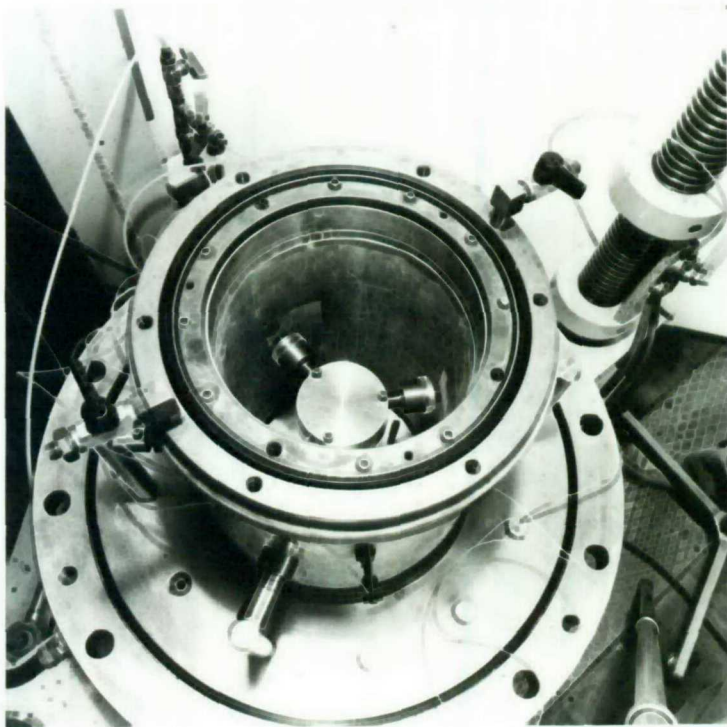


Plate 7.5 (b) Assembled sample showing instrumentation of inner wall.

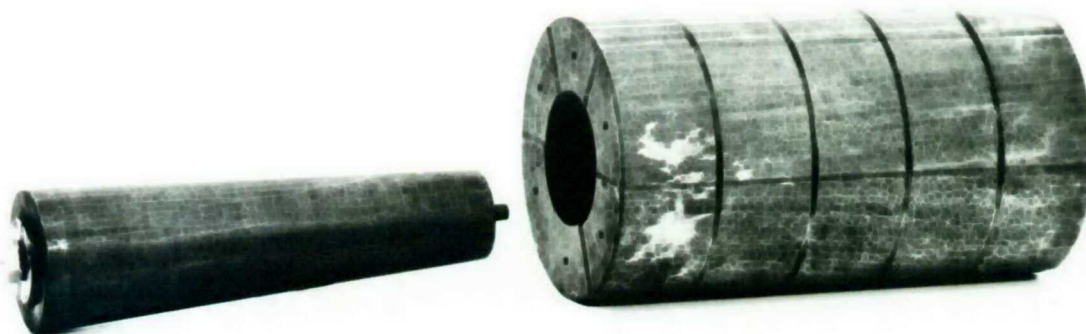
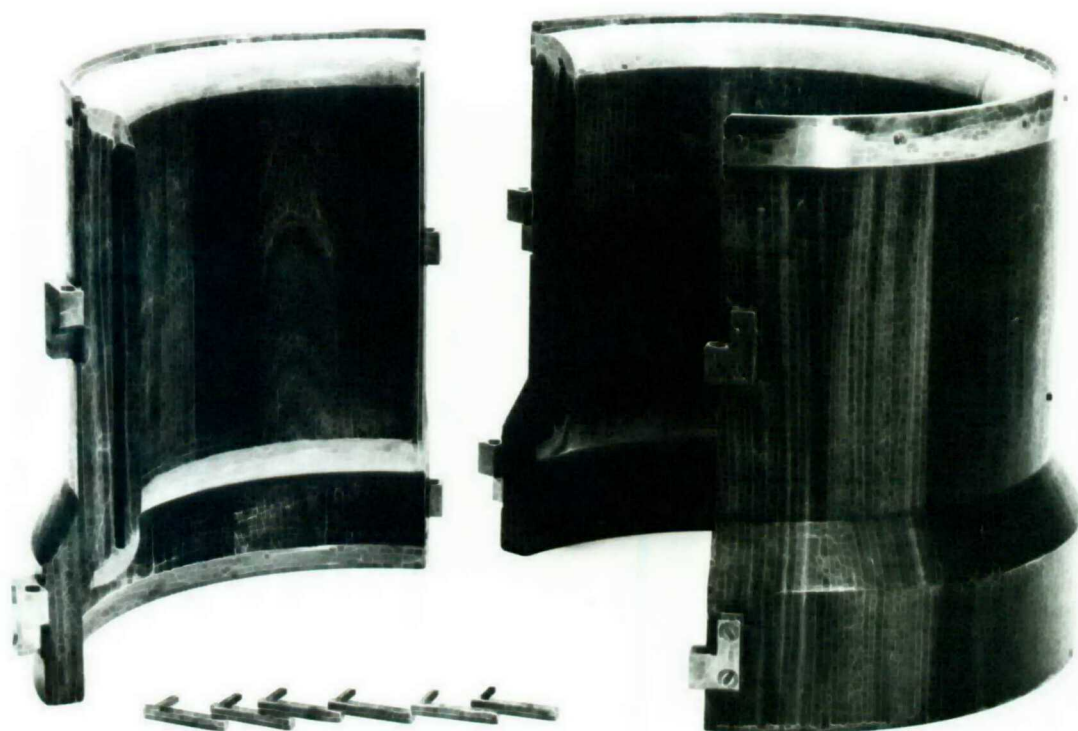


Plate 7.6 Inner and outer sample moulds.

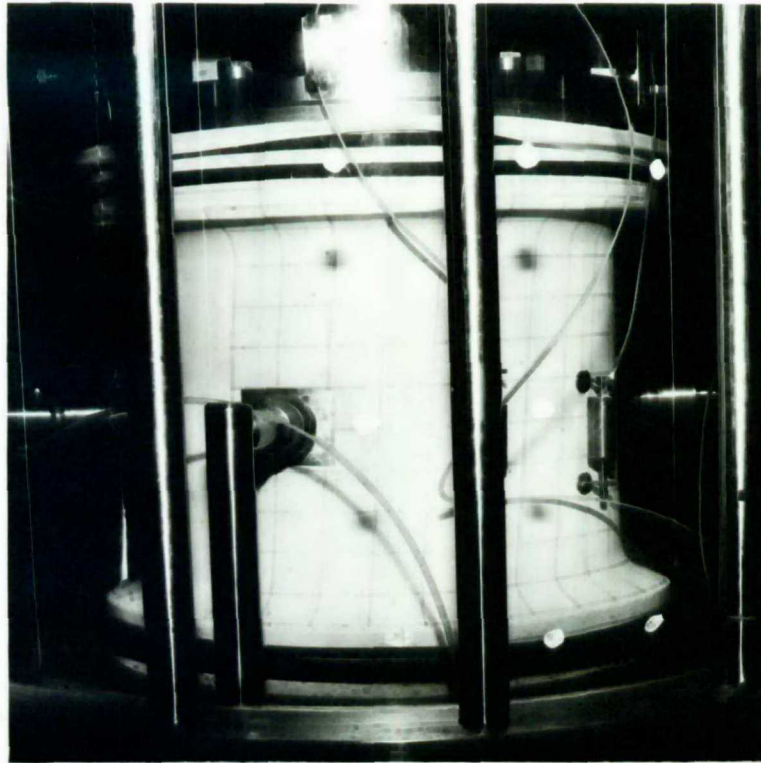


Plate 7.7 (a) Pre-peak deformation pattern in sample E2.

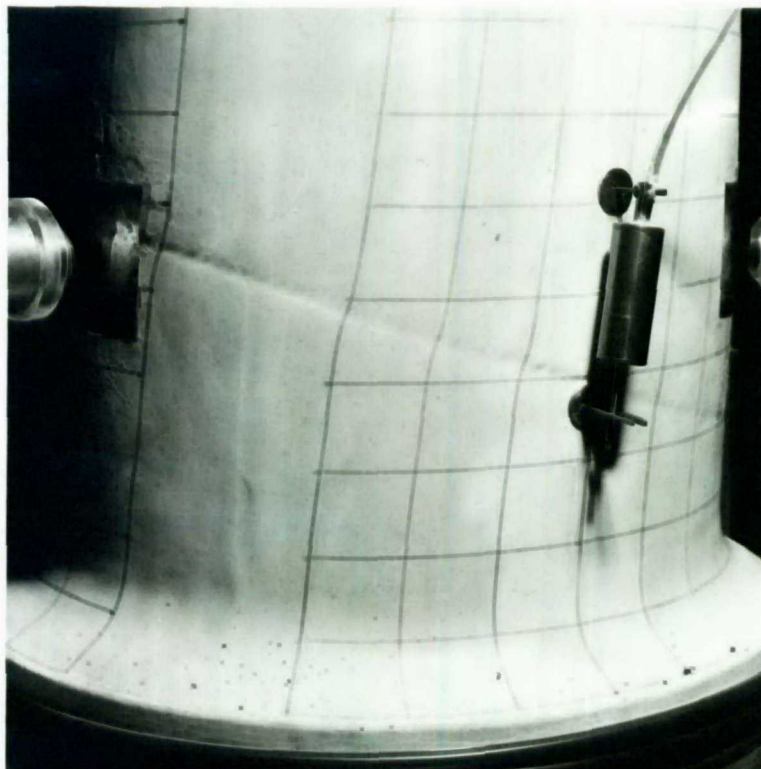


Plate 7.7 (b) Post-peak deformation pattern in sample E1.

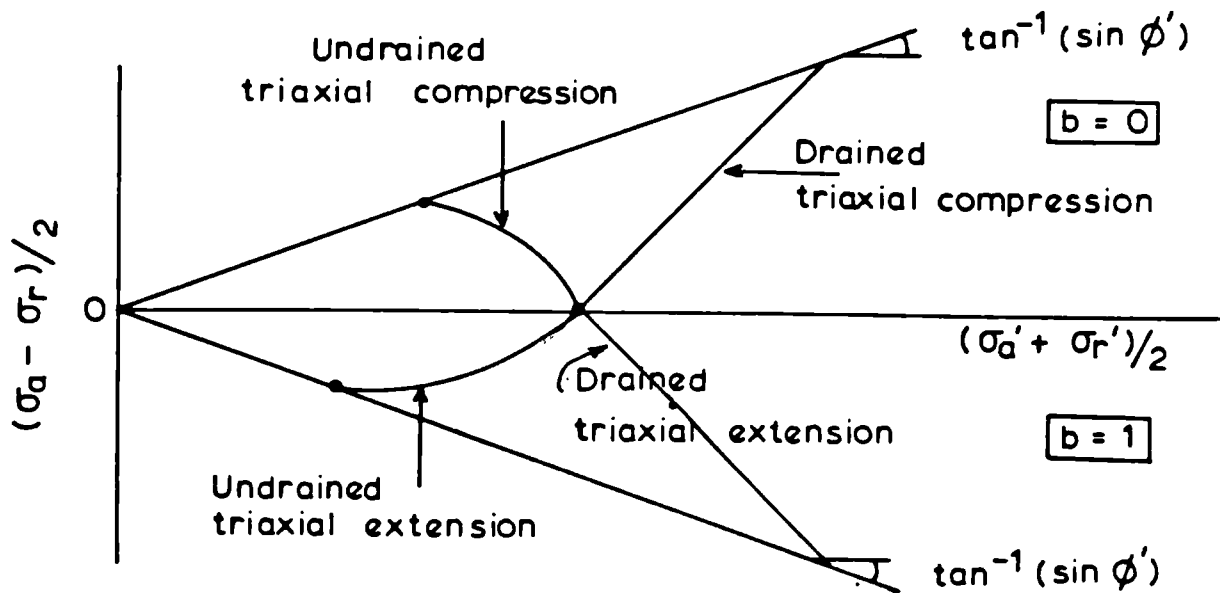


Fig. 7.1 The effect of b on drained and undrained triaxial strength of isotropic soil.

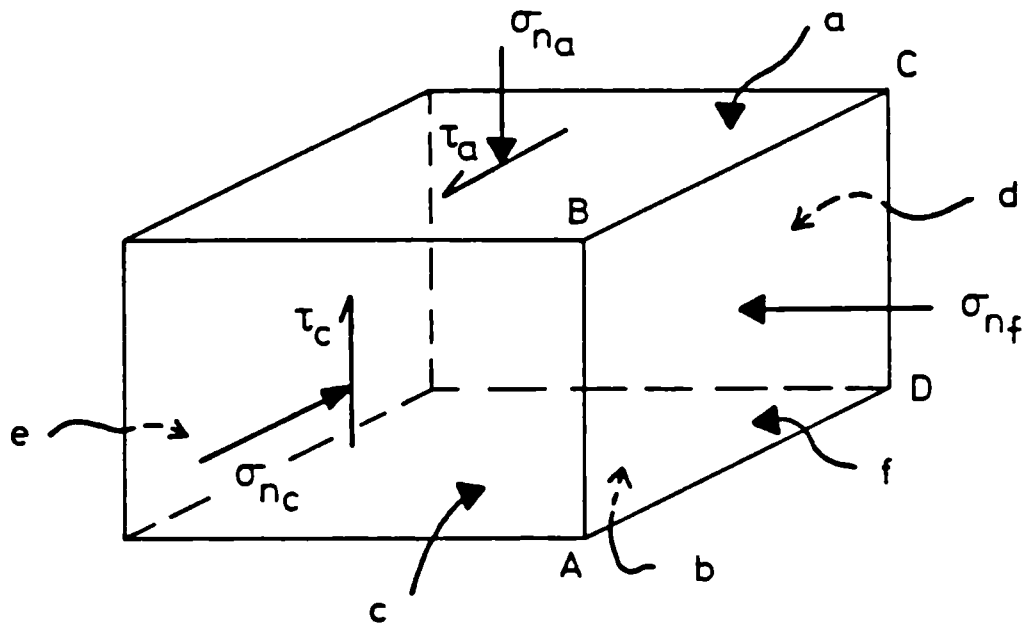
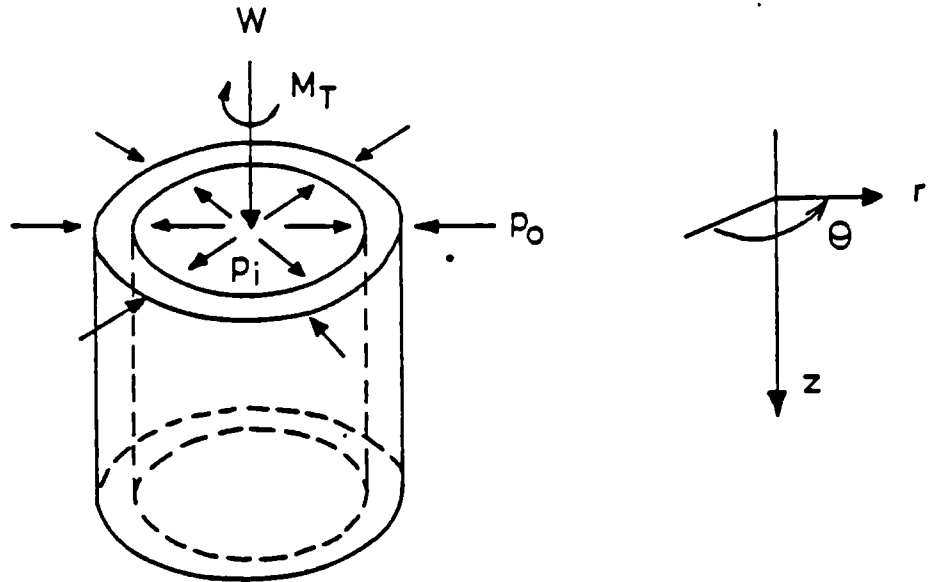
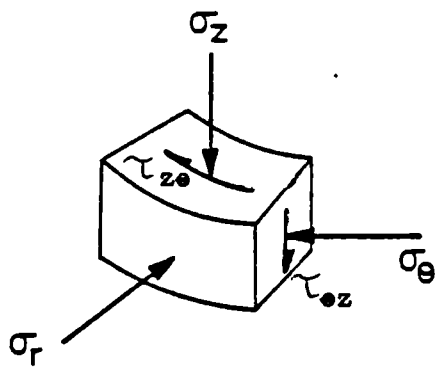


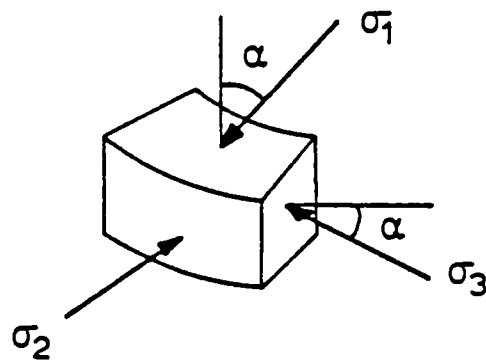
Fig. 7.2 Stresses to be applied for controlled rotation of principal stress directions.



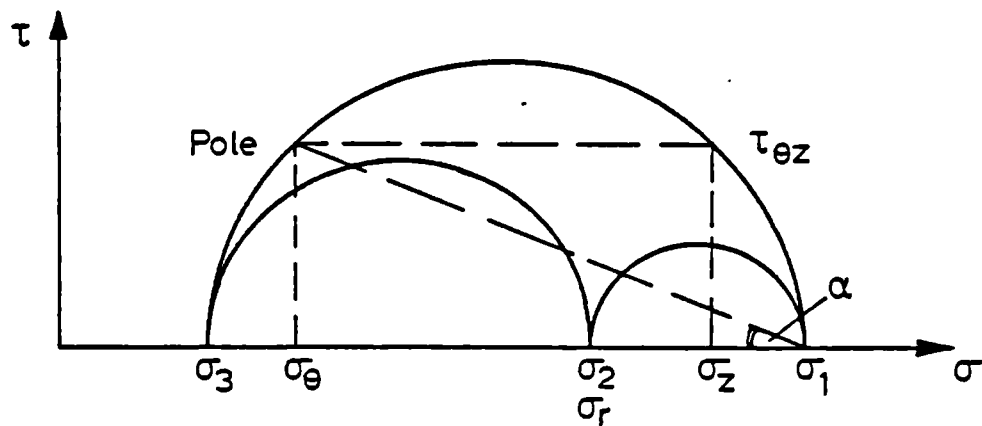
(a) Hollow cylinder sample under axial load, W , torque, M_T , internal pressure, p_i , external pressure, p_o



(b) Stresses on an element in the wall of a hollow cylinder sample



(c) Principal stresses on an element in the wall



(d) Mohr circle representation of stress in the wall

Fig.7.3 Idealised stress conditions in a hollow cylindrical element subject to axial load, torque and internal and external pressure.

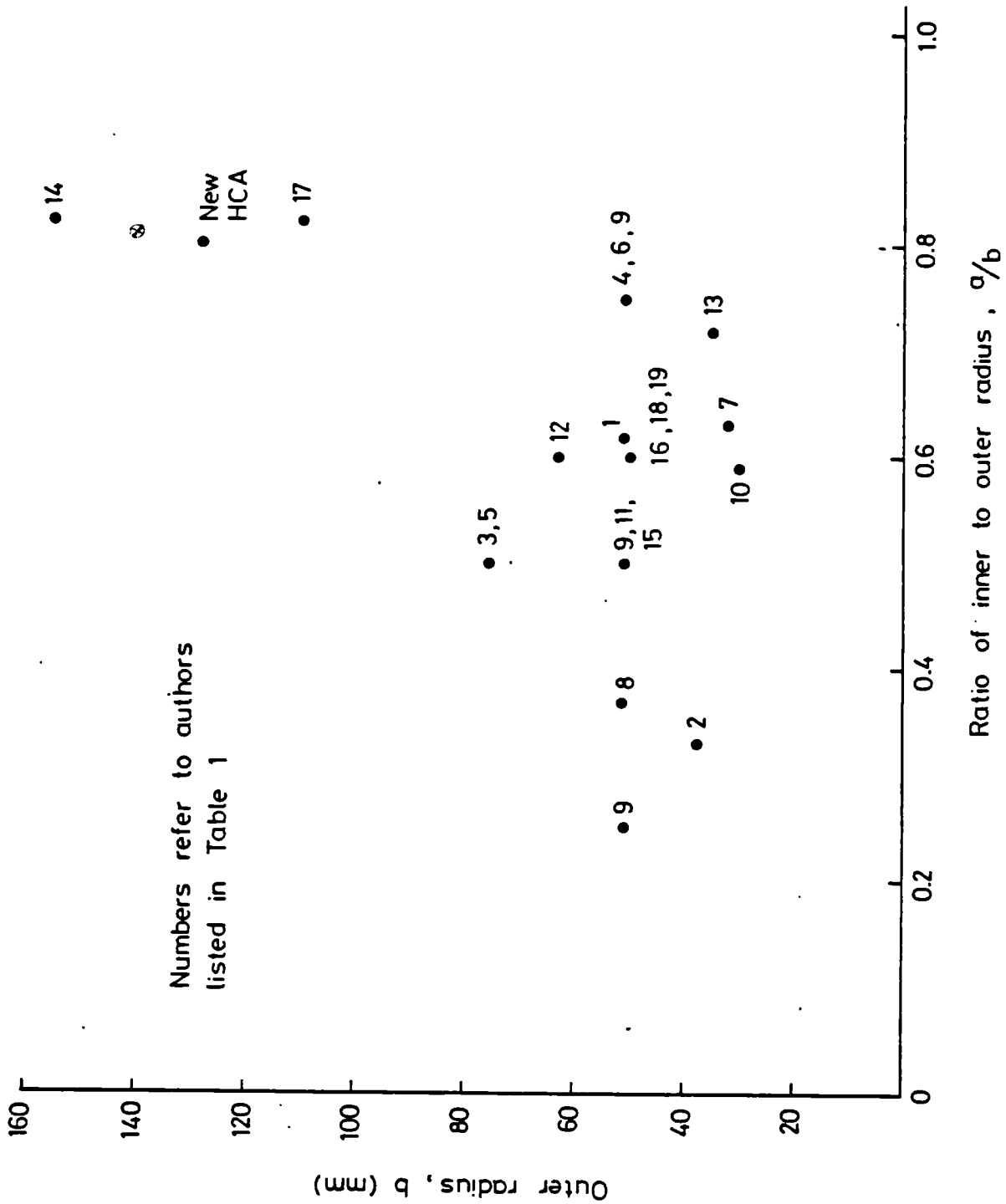
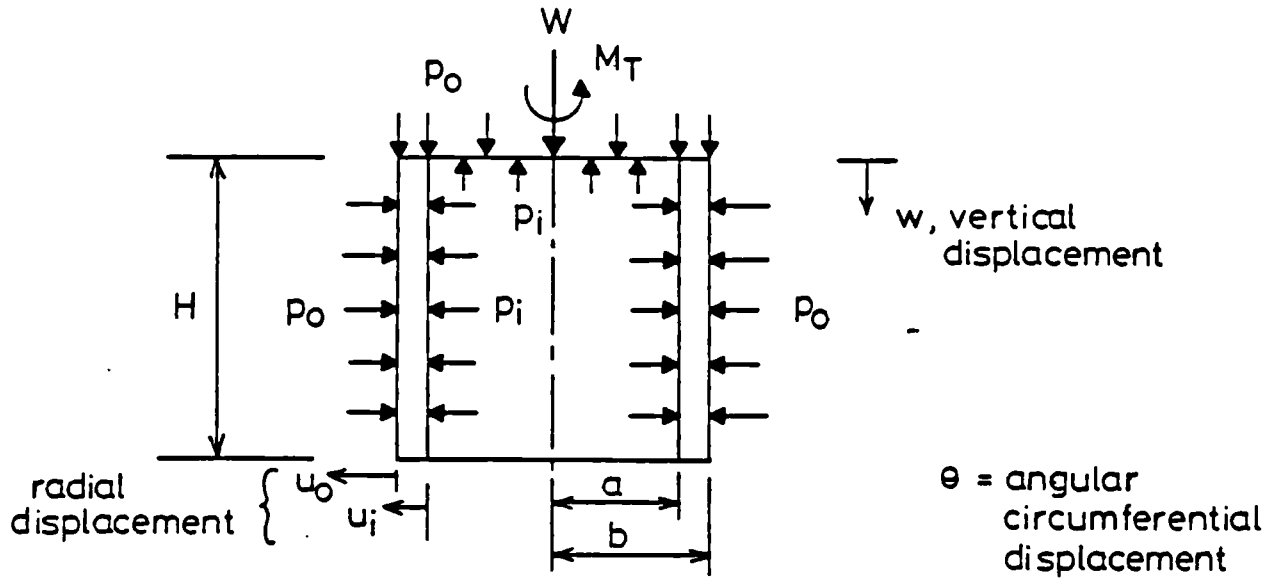


Fig. 7.4 Geometries of specimens used in earlier hollow cylinder apparatuses.



$$\text{Average vertical stress, } \bar{\sigma}_z = \frac{W}{\pi(b^2 - a^2)} + \frac{(p_o b^2 - p_i a^2)}{(b^2 - a^2)} \quad (1)$$

$$\text{Average radial stress, } \bar{\sigma}_r = \frac{(p_o b + p_i a)}{(b + a)} \quad (2)$$

$$\text{Average circumferential stress, } \bar{\sigma}_\theta = \frac{(p_o b - p_i a)}{(b - a)} \quad (3)$$

$$\text{Average shear stress, } \bar{\tau}_{\theta z} = \frac{3M_T}{2\pi(b^3 - a^3)} \quad (4)$$

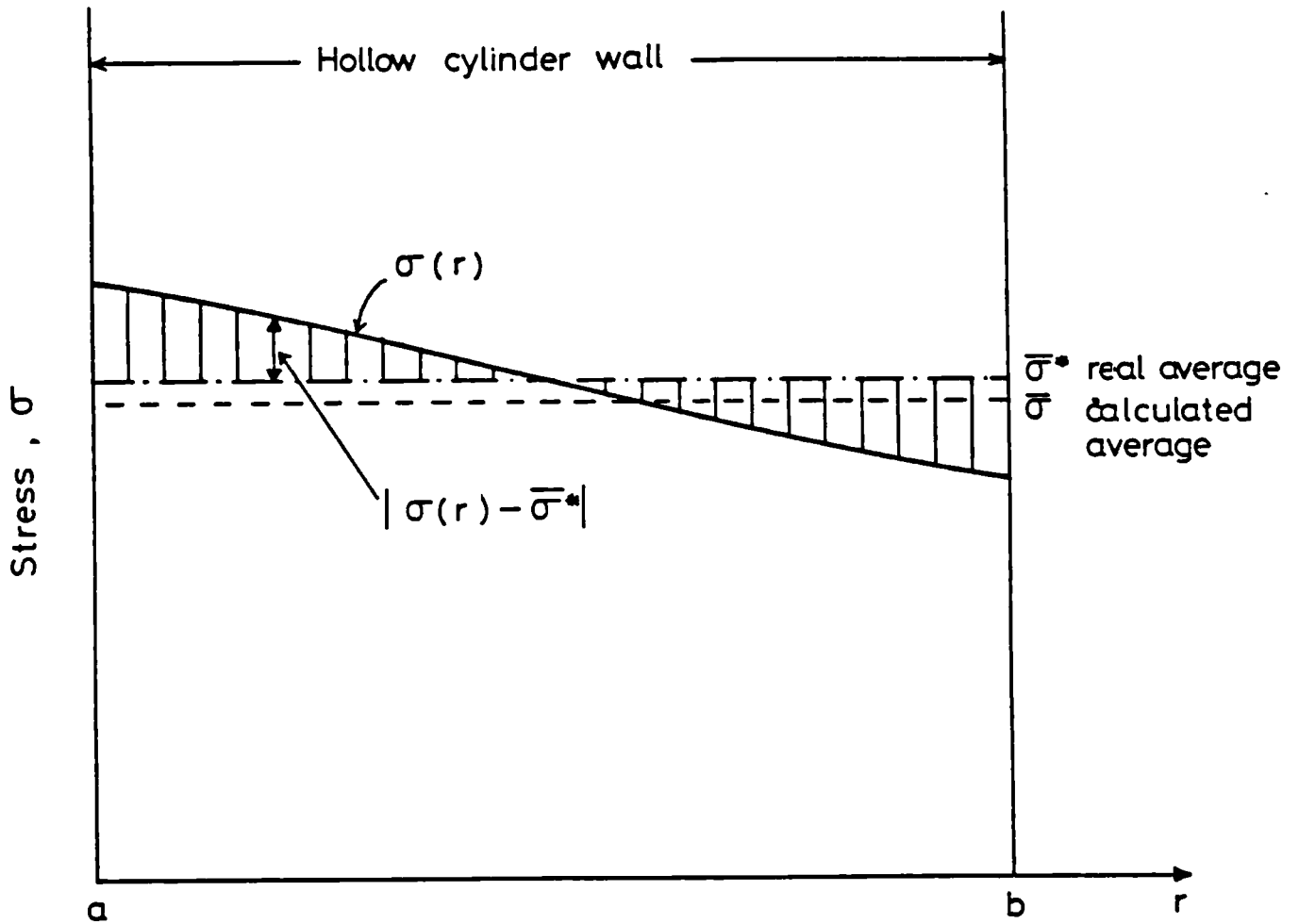
$$\text{Average axial strain, } \bar{\epsilon}_z = \frac{w}{H} \quad (5)$$

$$\text{Average radial strain, } \bar{\epsilon}_r = - \frac{(u_o - u_i)}{(b - a)} \quad (6)$$

$$\text{Average circumferential strain, } \bar{\epsilon}_\theta = - \frac{(u_o + u_i)}{(b + a)} \quad (7)$$

$$\text{Average shear strain, } \bar{\gamma}_{\theta z} = \frac{2.0(b^3 - a^3)}{3.H(b^2 - a^2)} \quad (8)$$

Fig.7.5 Definitions of average stresses and strains.



$$\beta_1 = \frac{|\bar{\sigma}^* - \bar{\sigma}|}{\sigma_L}$$

$$\beta_3 = \frac{\int_a^b |\sigma(r) - \bar{\sigma}^*| dr}{(b-a) \sigma_L}$$

Fig.7.6 Definitions used for stress non-uniformity and "accuracy".

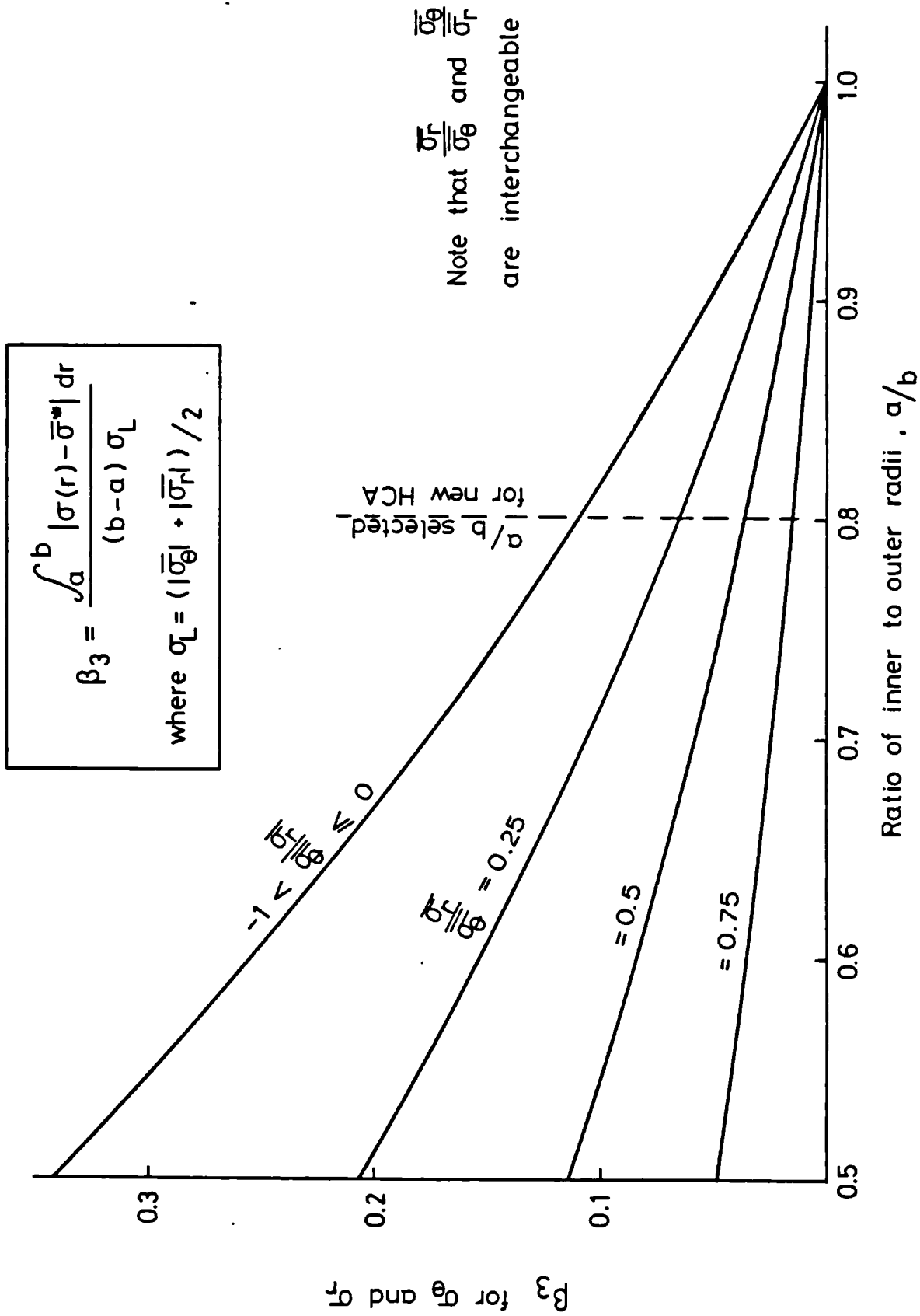


Fig. 7.7 Variation in stress non-uniformity with geometry and stress path. (Isotropic linear elastic analysis.)

$$\beta_3 = \frac{\int_a^b |\epsilon(r) - \bar{\epsilon}| dr}{(b-a) \epsilon_L}$$

where $\epsilon_L = (|\bar{\epsilon}_\theta| + |\bar{\epsilon}_r|)/2$

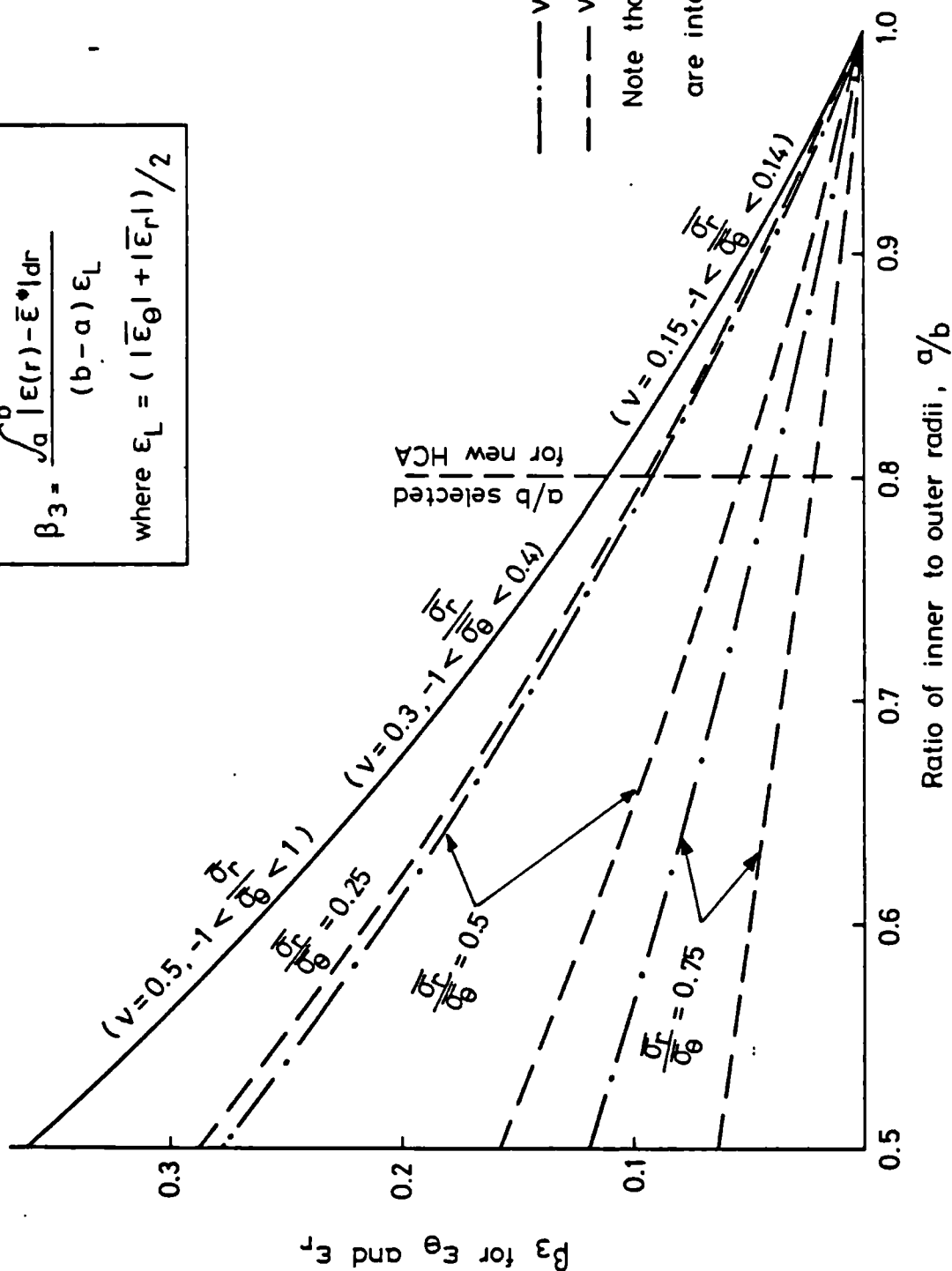


Fig.7.8 Variation in strain non-uniformity with geometry and stress path. (Isotropic linear elastic analysis.)

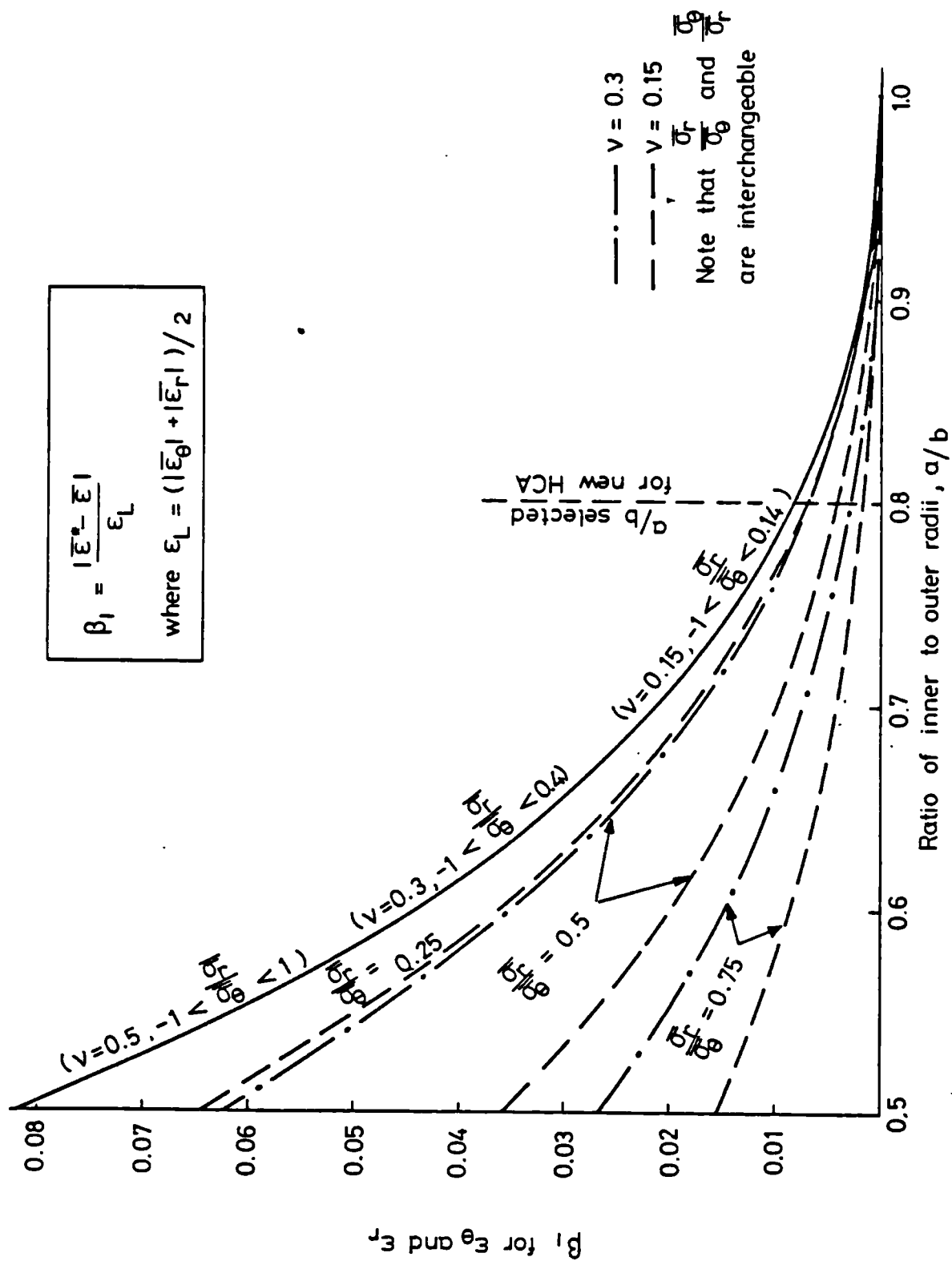


Fig.7.9 Variation of "accuracy" in strain with geometry and stress path. (Isotropic linear elastic analysis.)

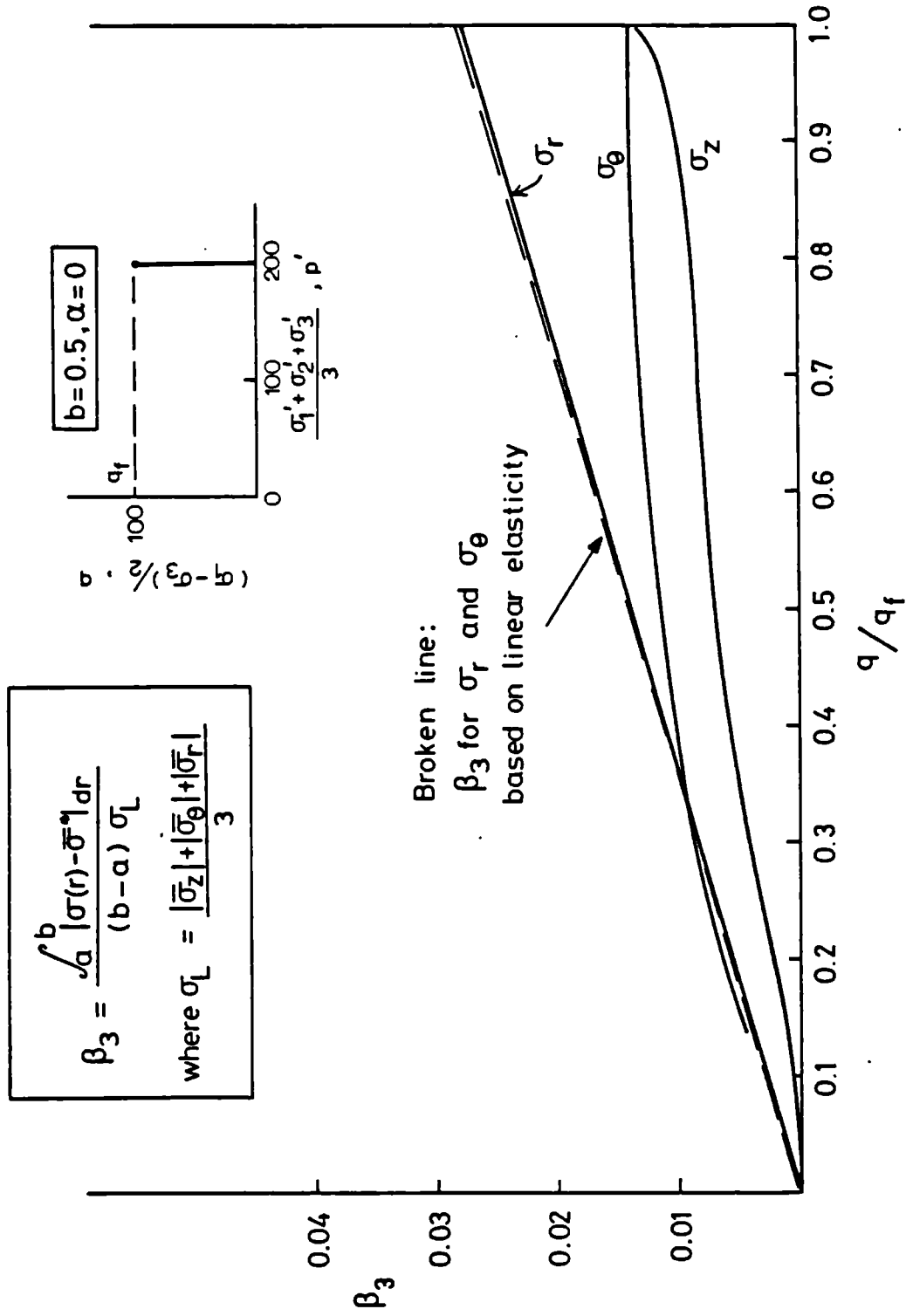


Fig. 7.10 Development of stress non-uniformities in a strain-hardening elasto-plastic hollow cylinder sample without end restraint.

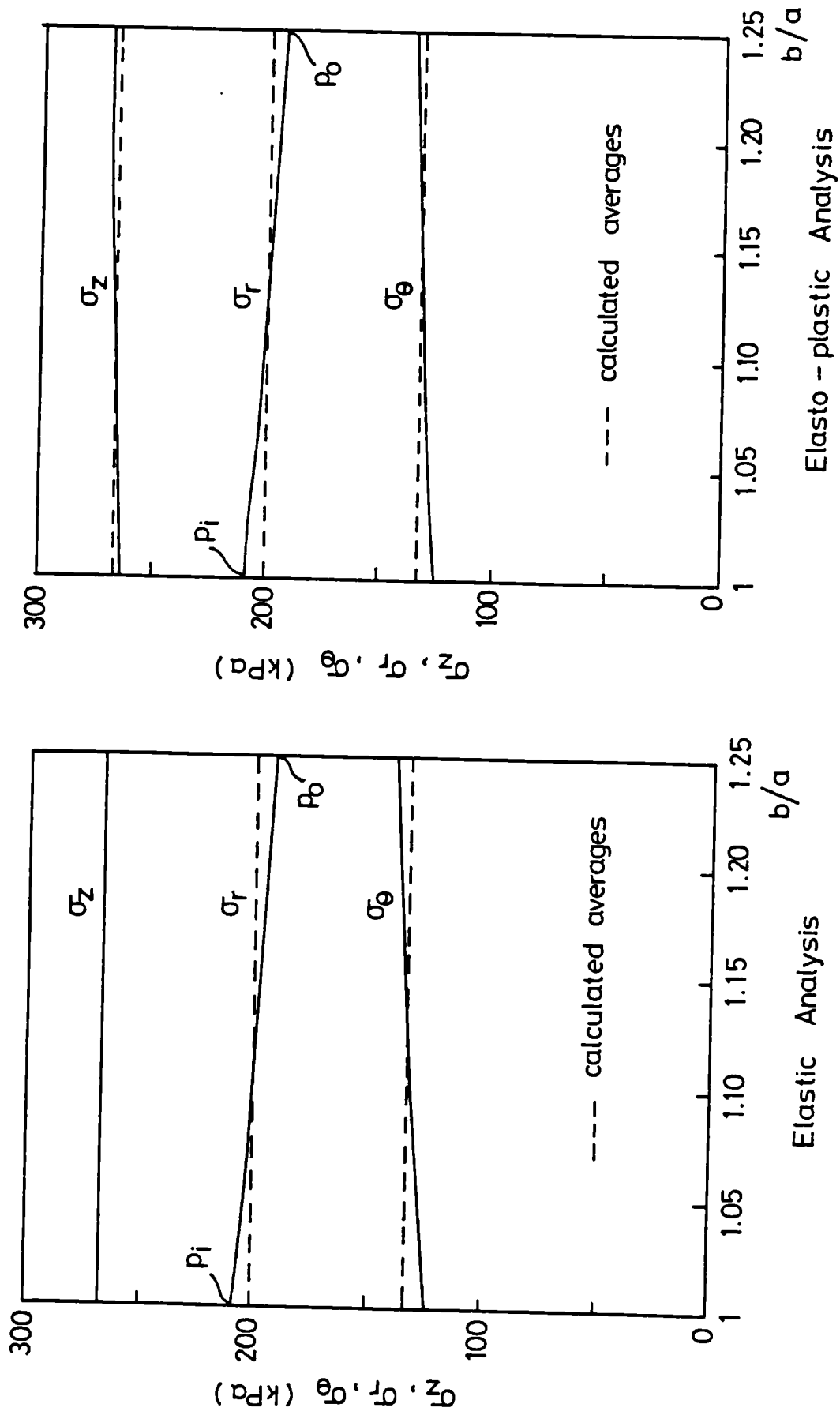


Fig. 7.11 Stress distributions across the wall of a hollow cylinder specimen.

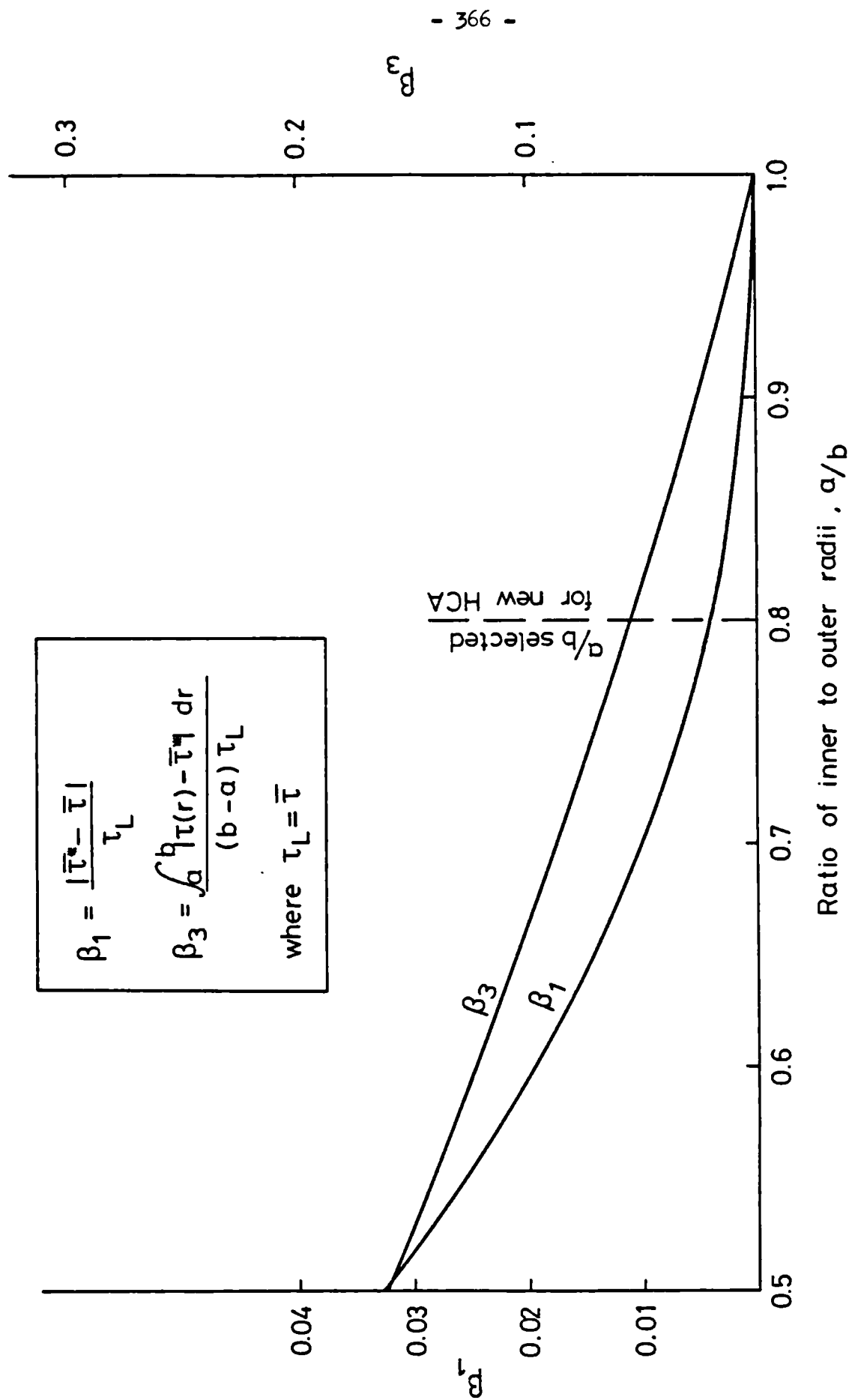


Fig.7.12 Variation in shear stress non-uniformity and "accuracy" with geometry.
(Isotropic linear elastic analysis.)

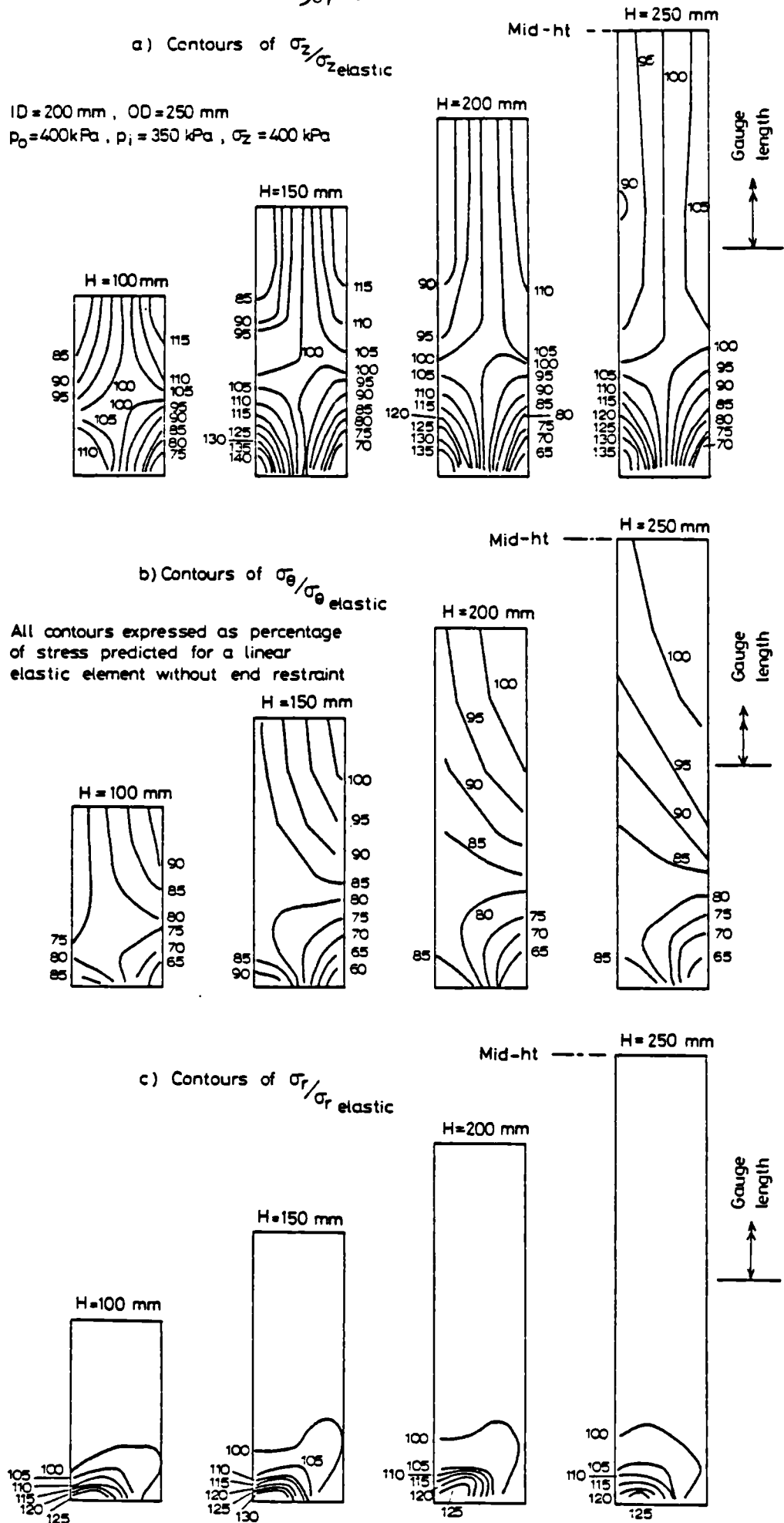
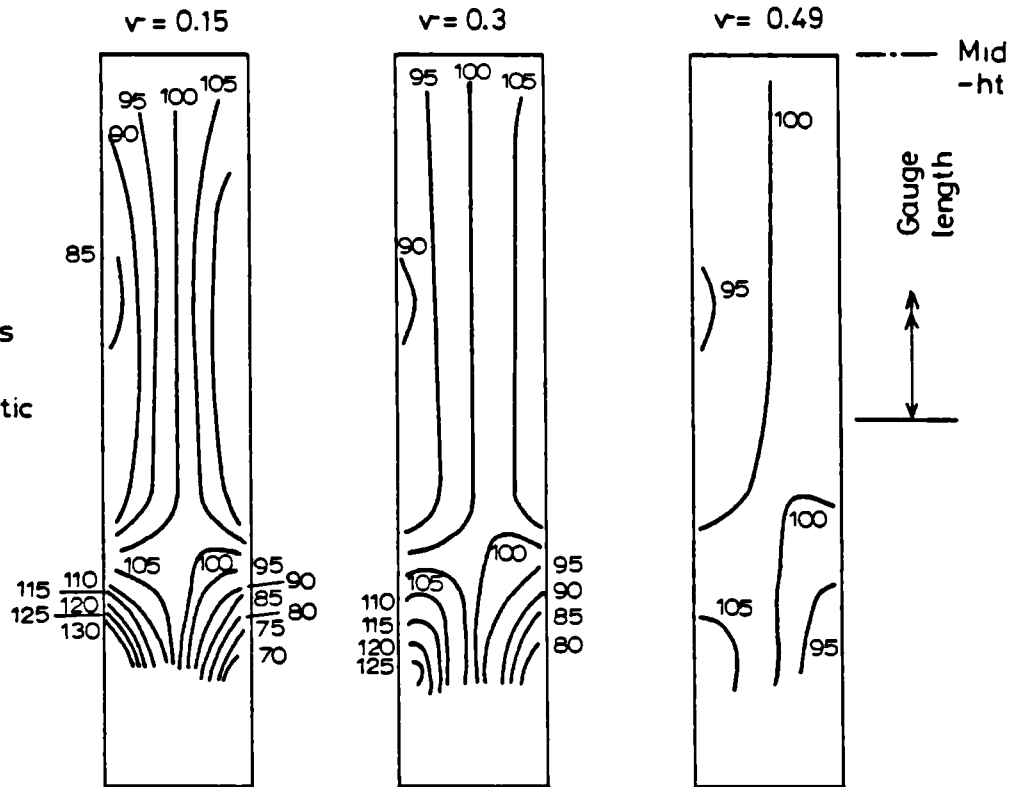


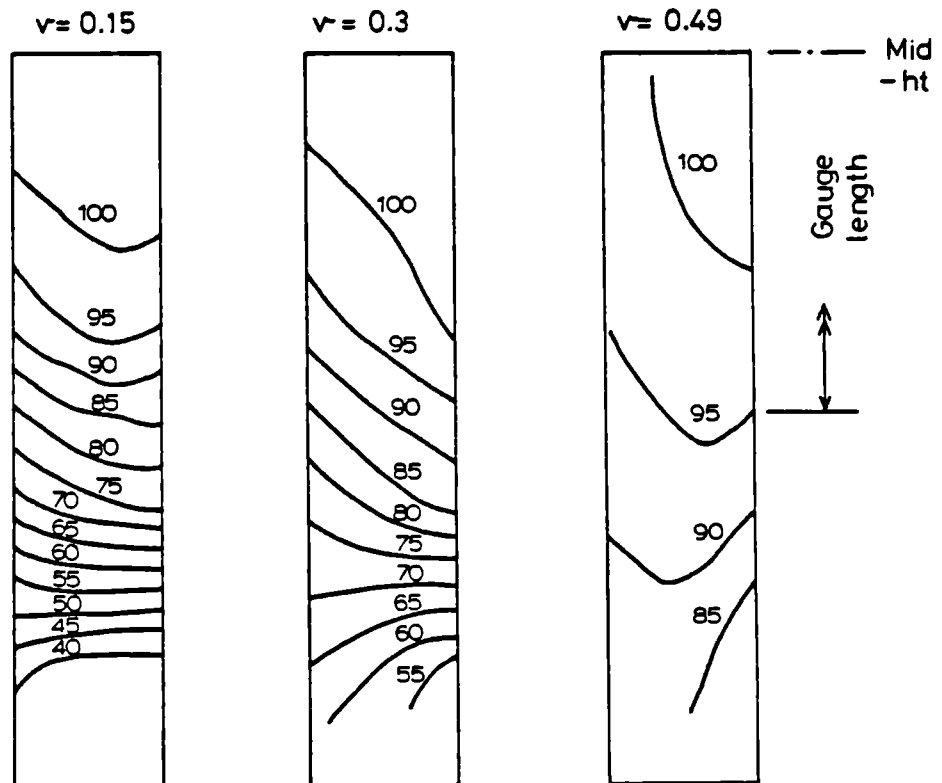
Fig. 7.13 Linear elastic analyses of stress distributions in fixed-ended hollow cylindrical elements of different height (Poisson's ratio = 0.499).

(a) Contours of $\sigma_z/\sigma_{z\text{elastic}}$



All contours expressed as percentage of stress predicted for a linear elastic element without end restraint.

(b) Contours of $\sigma_\theta/\sigma_{\theta\text{elastic}}$



$ID = 200 \text{ mm}$, $OD = 250 \text{ mm}$, $H = 250 \text{ mm}$

Fig. 7.14 Linear elastic analyses of fixed-ended hollow cylindrical elements with different Poisson's ratio.

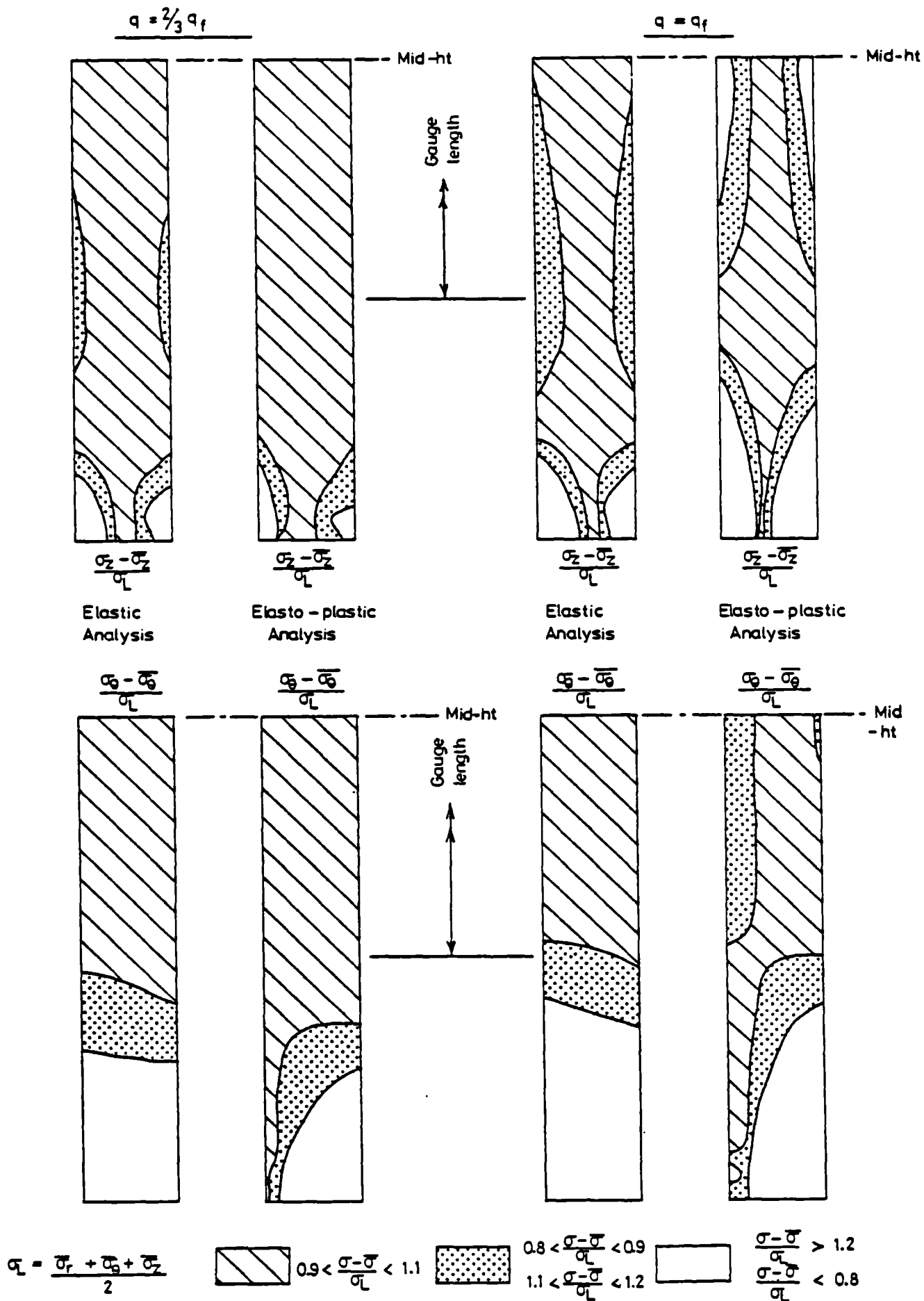


Fig. 7.15 Elasto-plastic and elastic analyses of fixed ended hollow cylinder samples of 200mm ID x 250mm OD x 250mm high

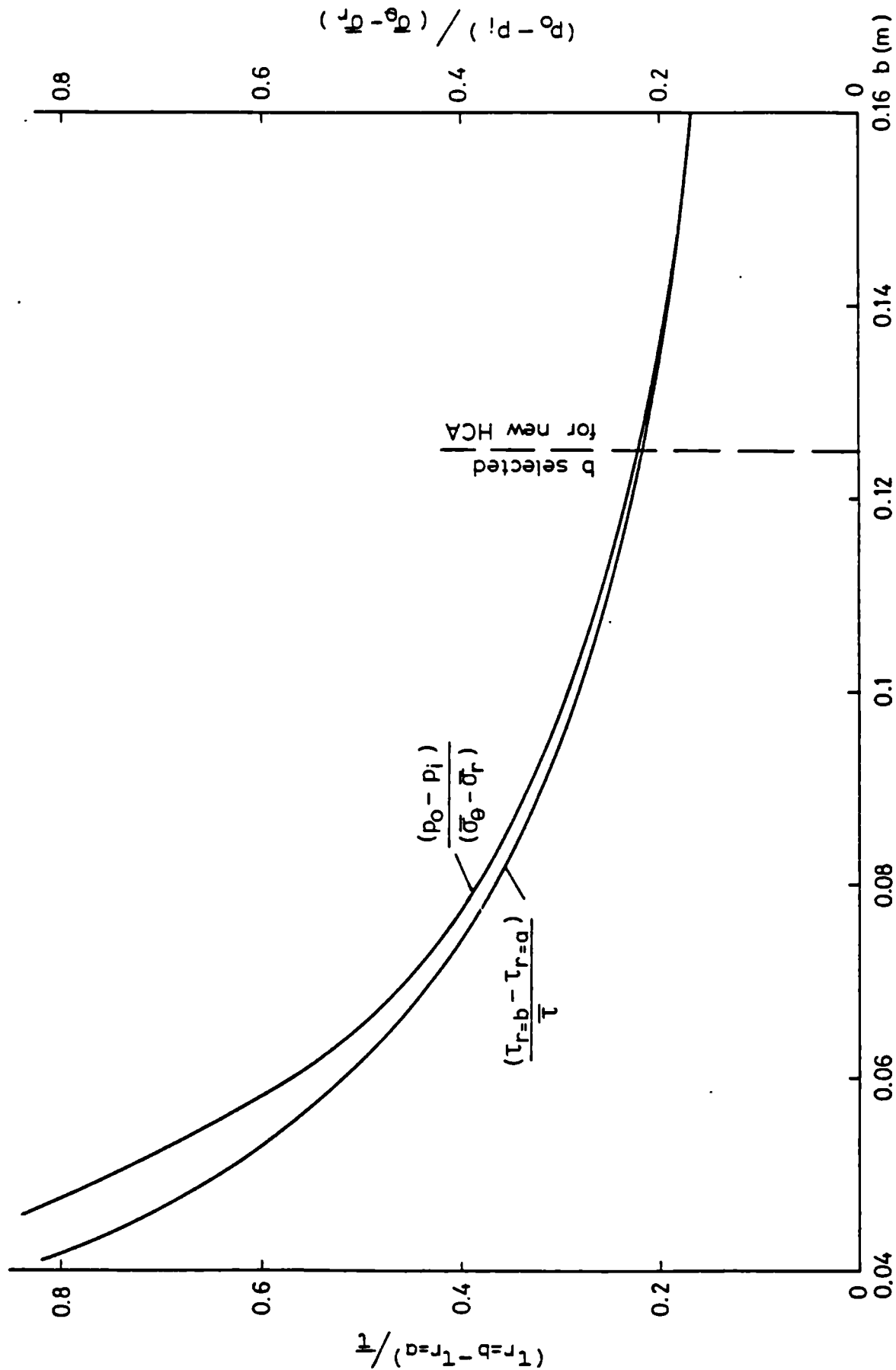


Fig. 7.16 Influence of outer radius, b , on stress uniformity in a hollow cylinder of 25.4mm wall thickness.

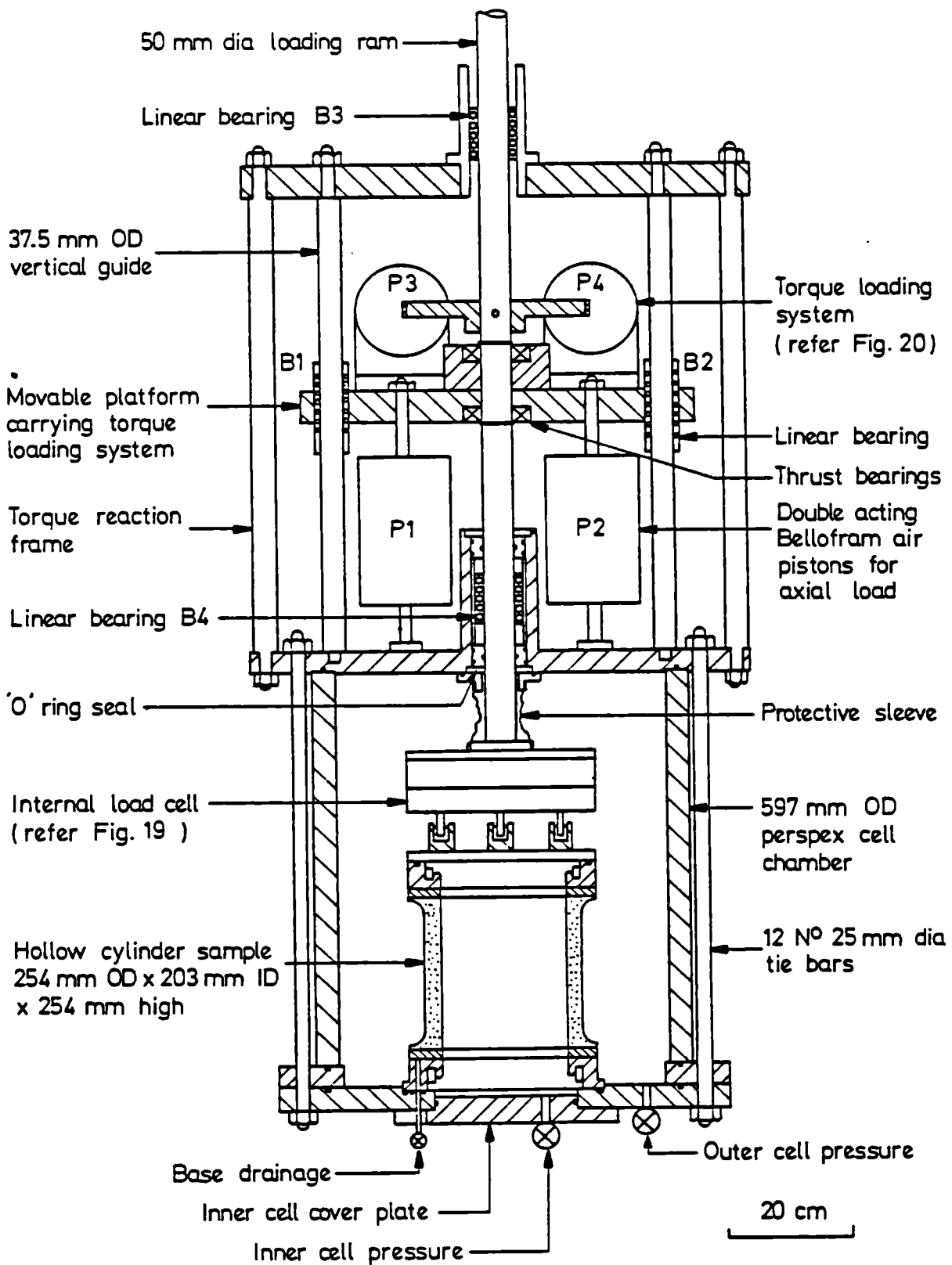


Fig. 7.18 General layout of the new hollow cylinder apparatus

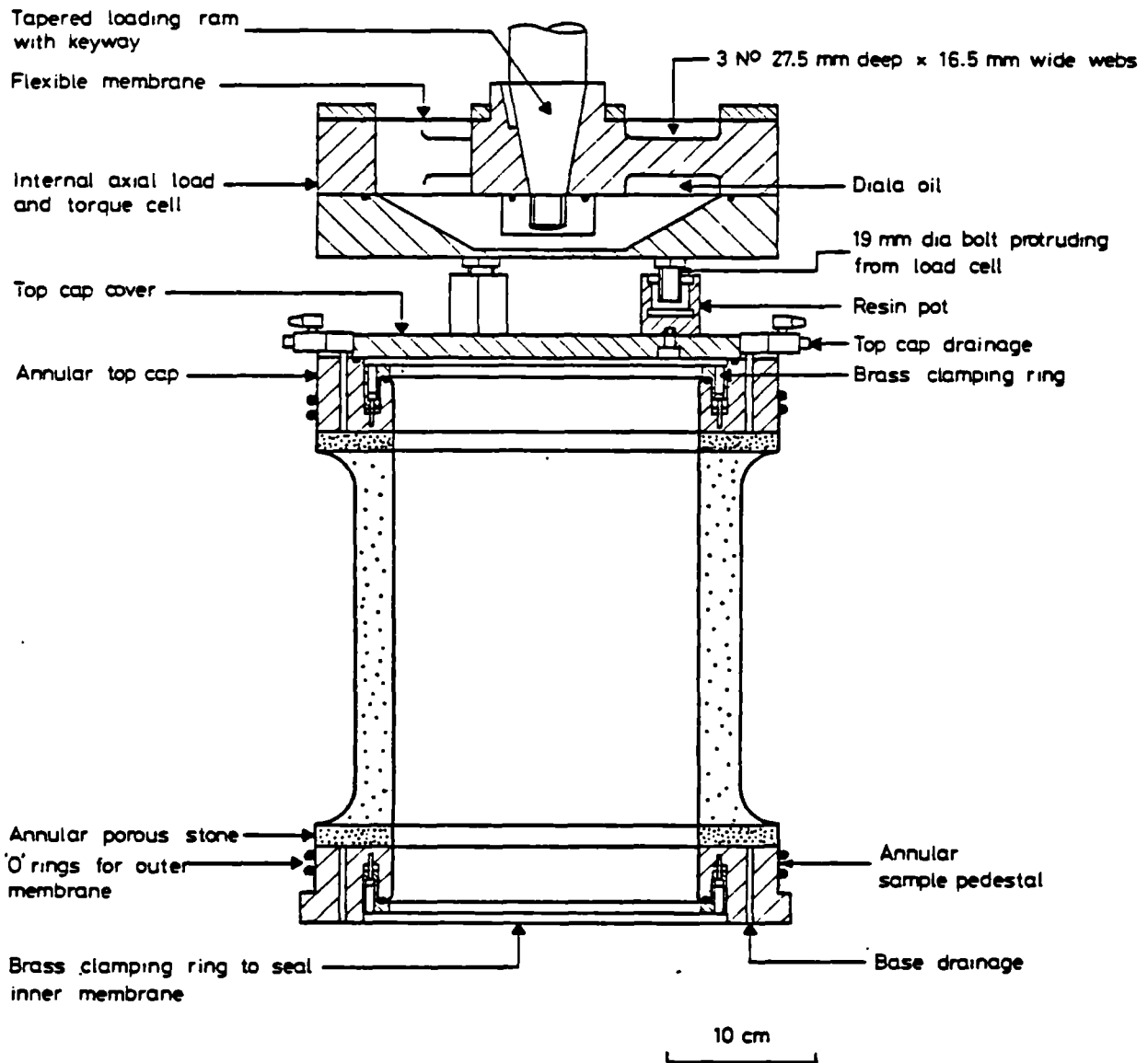


Fig. 7.19 Details of sample sealing and load cell construction and connection.

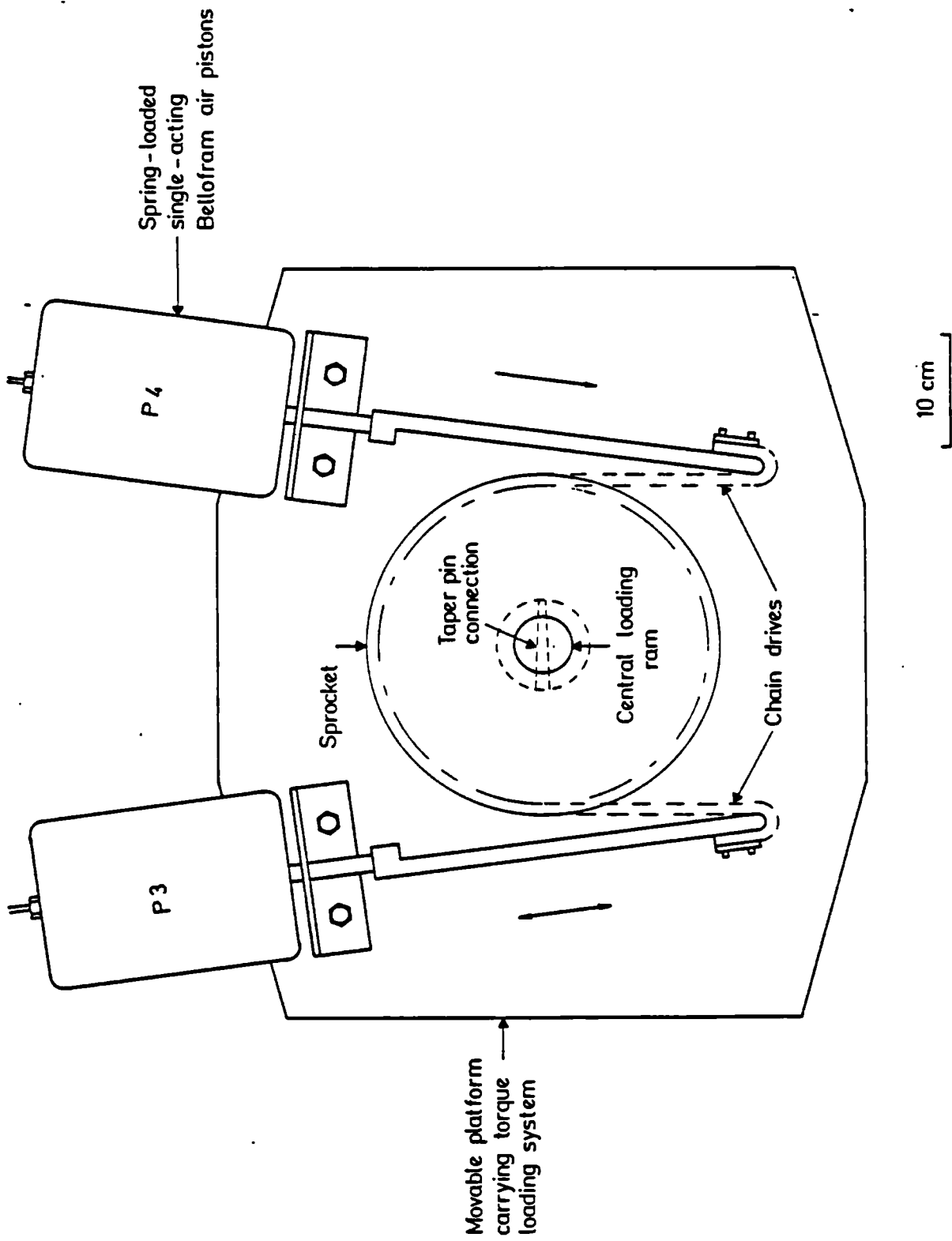


Fig.7.20 Plan view on torque loading system.

Fig. 7.21 Stress limits for new hollow cylinder apparatus.

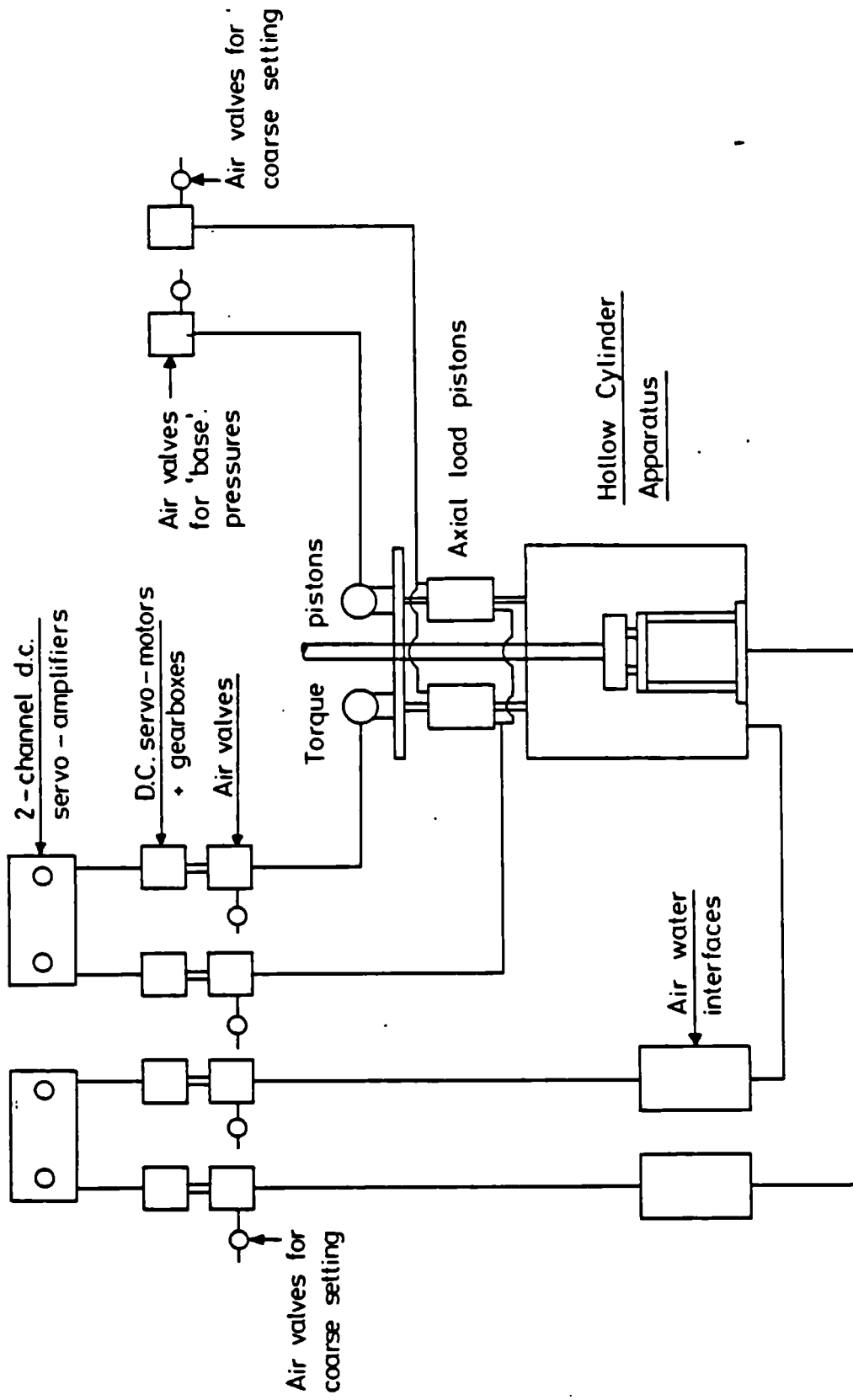
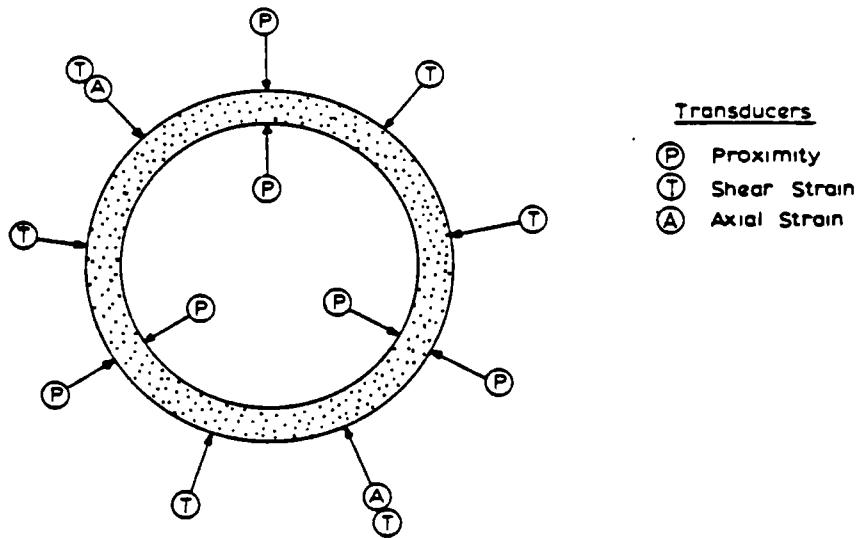


Fig.7.23 Control system for the new hollow cylinder apparatus.



Transducers

- Ⓟ Proximity
- Ⓣ Shear Strain
- ⓐ Axial Strain

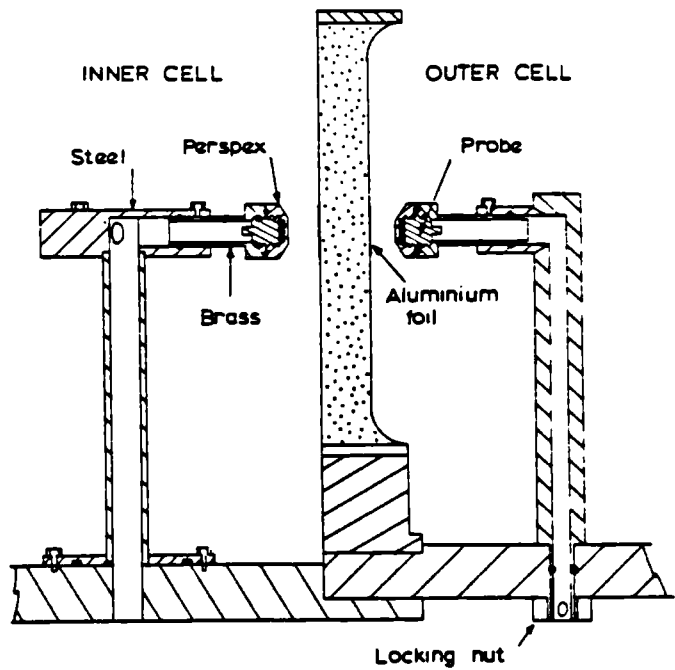
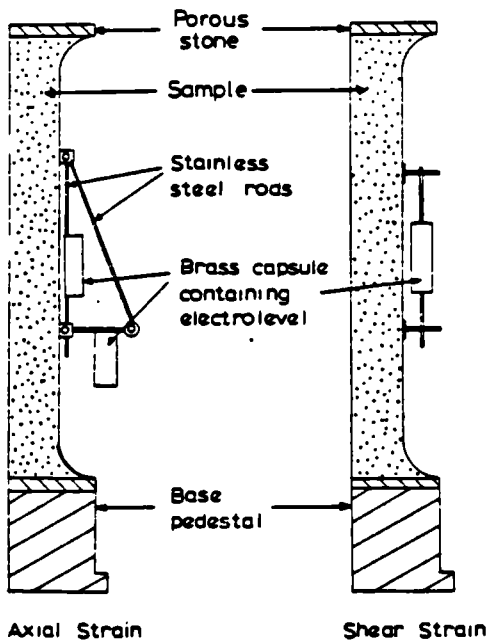
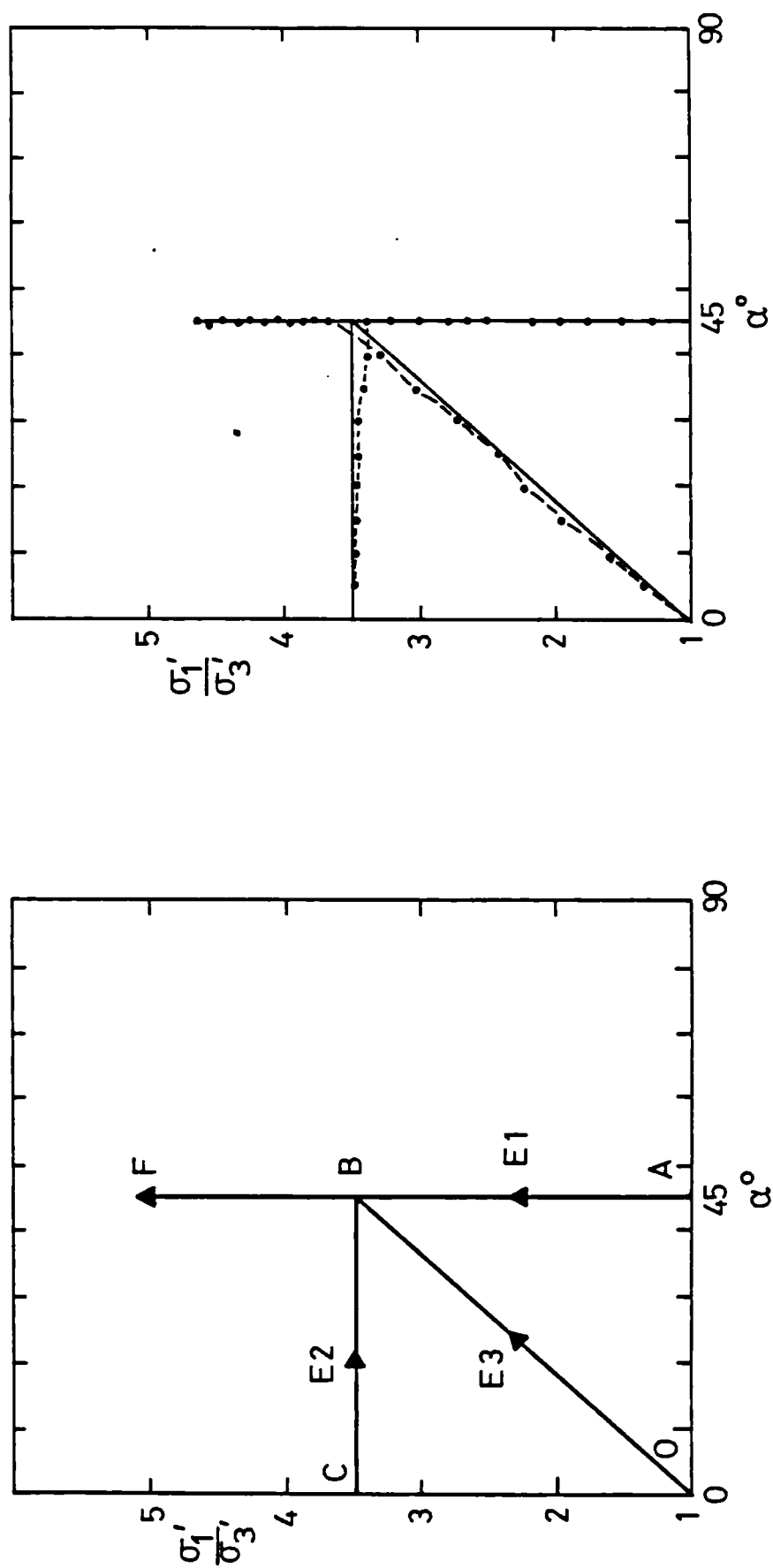


Fig. 7.24 Details of internal instrumentation of new hollow cylinder apparatus.



PRESCRIBED STRESS PATHS STRESS PATHS FOLLOWED

Fig.7.25 Stress paths for proving tests E1, E2 and E3, σ_1'/σ_3' versus α .

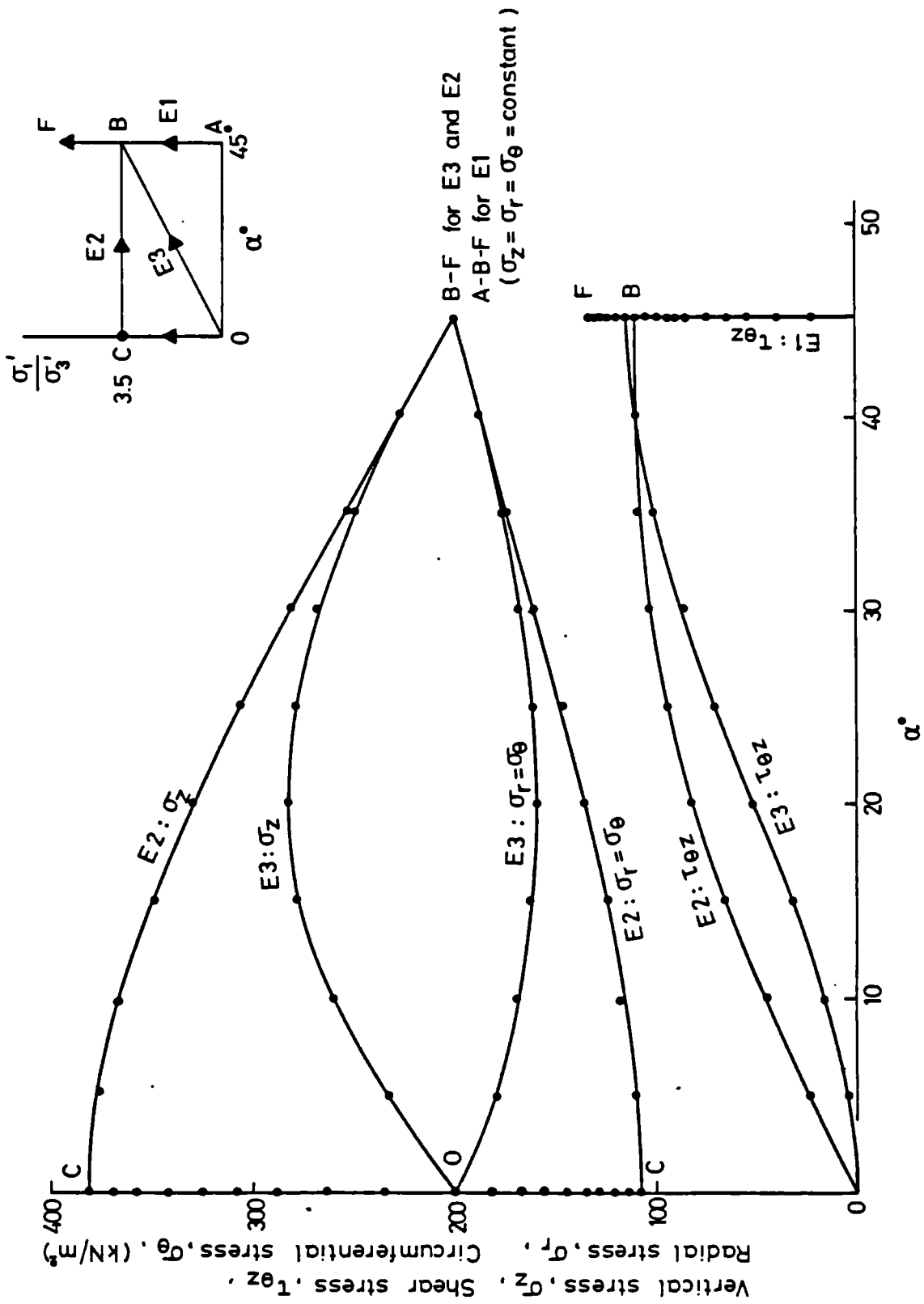


Fig. 7.26 Variations in σ'_z , $\tau_{\theta z}$, σ'_r and σ'_θ in tests E1, E2 and E3 ($p_0 = p_i$).

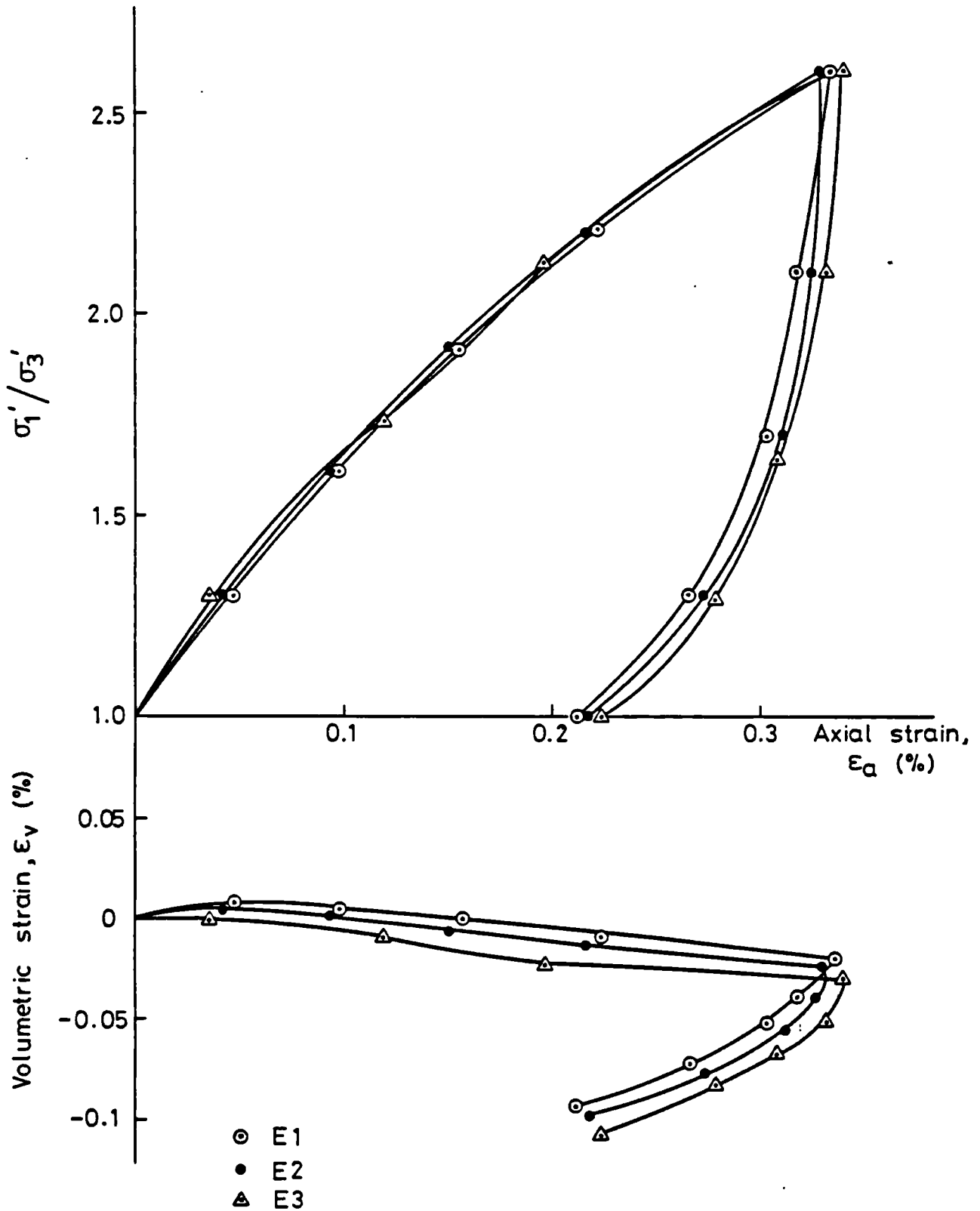


Fig. 7.27 Evidence of sample repeatability from initial load-unload cycle.

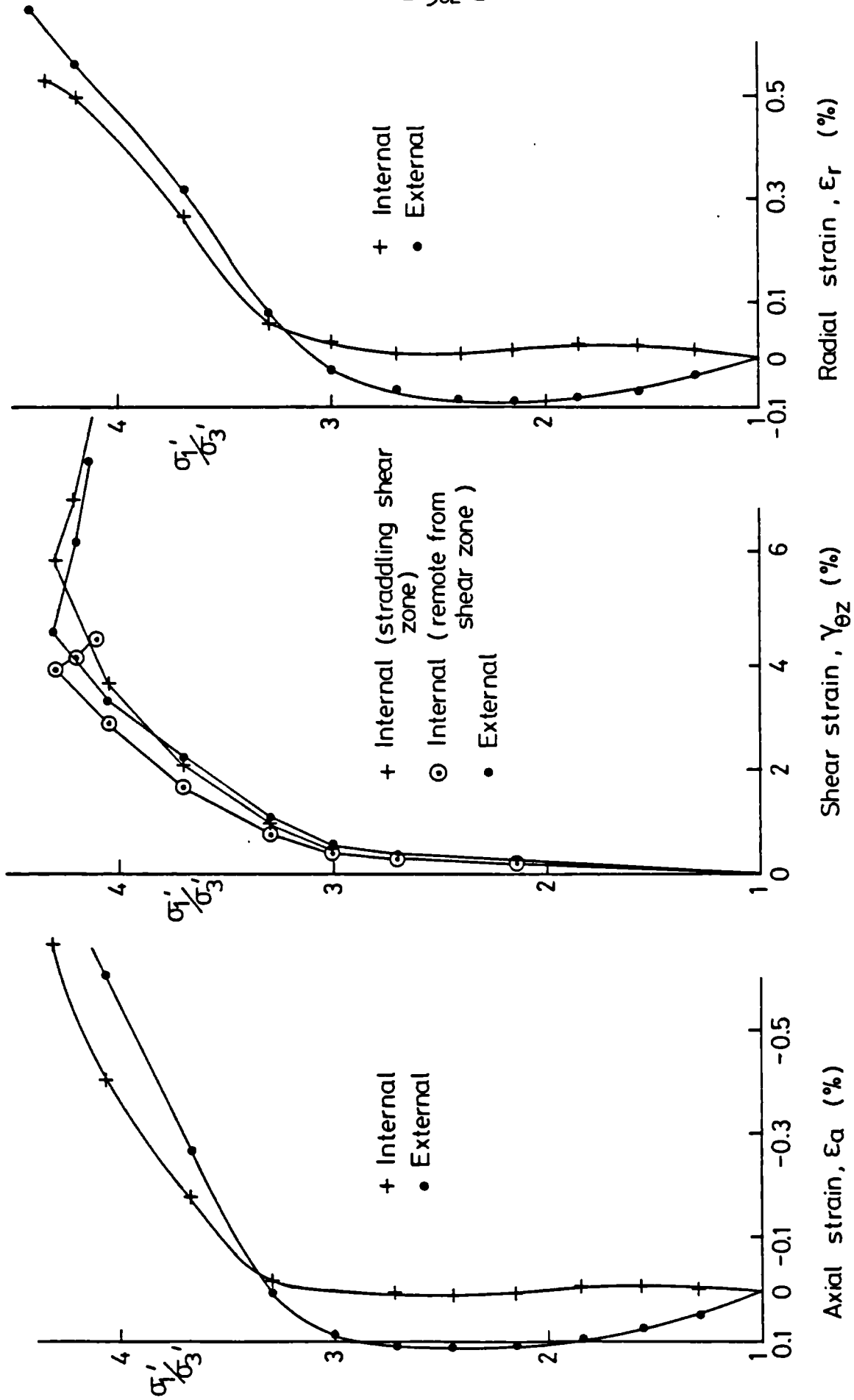


Fig. 7.28 Comparison of internally and externally measured strains in test E3.

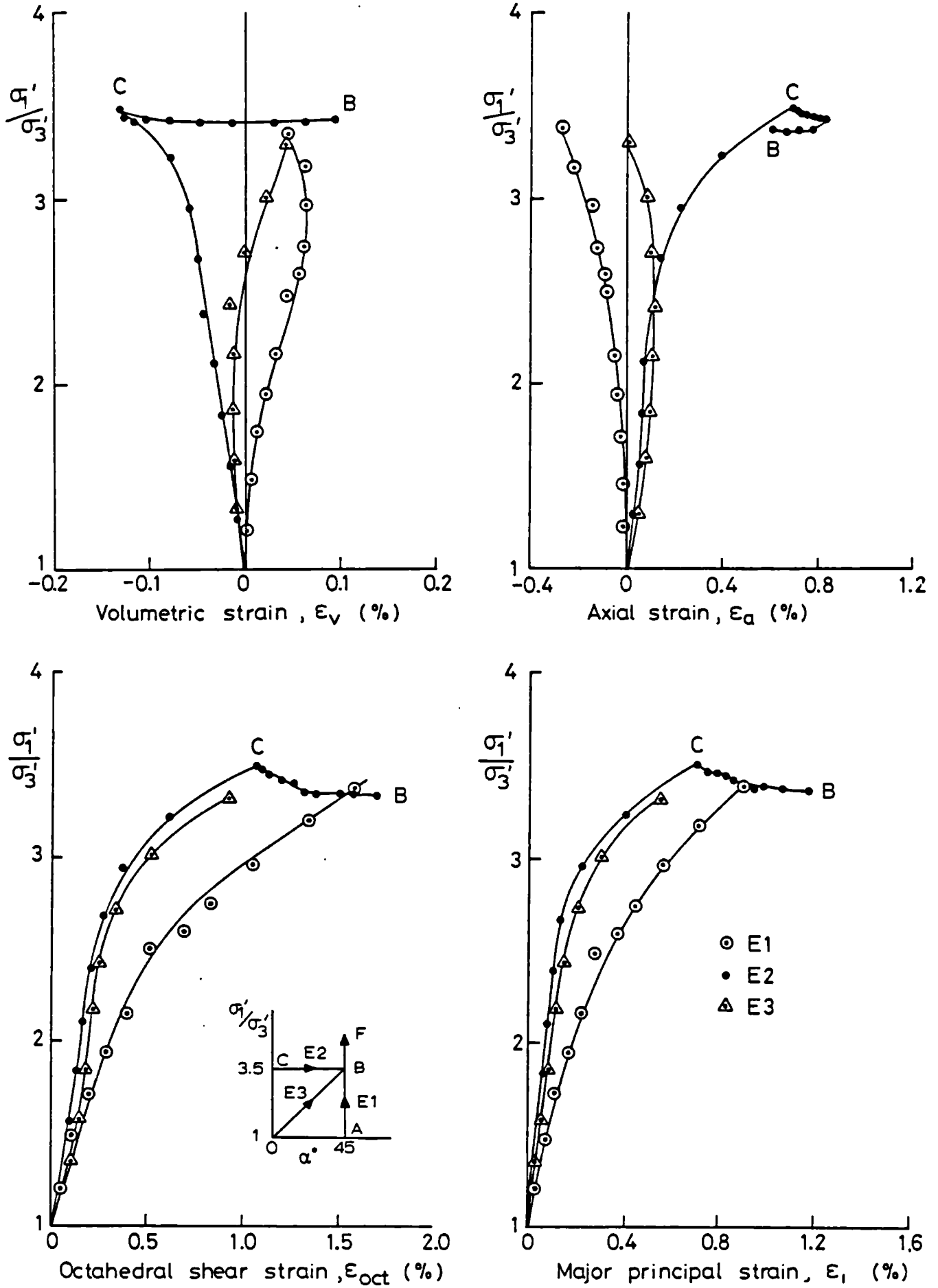


Fig. 7.29 Test results for alternative stress paths to point B

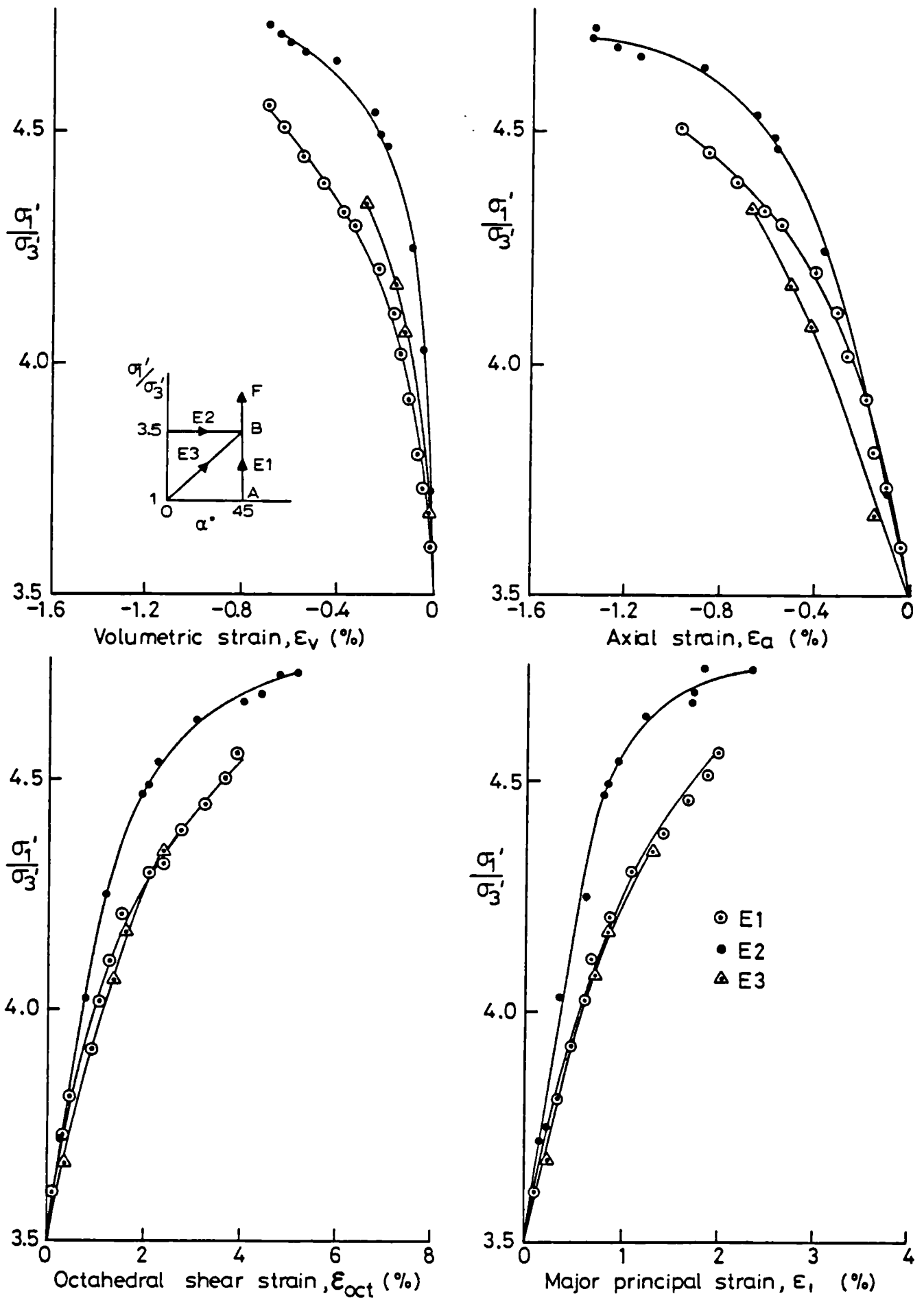


Fig. 7.31 Test results for common stress path to failure.

APPENDIX 7.1

Linear Elastic Analyses of End Restraint in Hollow Cylinder Samples

Finite element analyses of the stress distributions in hollow cylindrical specimens with end restraint were made with the aim of providing guidance for the selection of a sample geometry which gave optimum uniformity of stress throughout its height and across the wall. Variables examined were height, outside and inside diameter, load combination, top cap shape and thickness. The load combinations and specimen geometries which were analysed are summarised in Table 7. A1. 1.

For the analyses described in this Appendix the soil was assumed to be linearly and isotropically elastic. To facilitate the analysis, axisymmetric conditions were retained by neglecting the application of a torque in the majority of cases. (Results are also presented from some three dimensional finite element analyses in which a torque was applied.) The finite element mesh used to represent the soil and top cap in the axisymmetric analyses is shown in Fig. 7. A1. 1. The finite element program (Hamza, 1976) uses eight-noded isoparametric elements; seventy-five of these elements (15 rows of 5 elements) were used to represent the soil sample, and thirty-three of them (3 rows of 11 elements) were used to represent the top cap. The sample was considered to be fixed at its base and full continuity was assumed at the interface between the sample and the top cap. The nodes on the vertical centre line of the top cap were restrained in a horizontal direction, but were free to move in a vertical direction. For the majority of the analyses the elastic parameters ascribed to the soil and top cap were:-

	<u>Soil</u>	<u>Top Cap</u>
Young's Modulus (N/mm^2)	90	21×10^4
Poisson's Ratio	0.49	0.2

With a Poisson's Ratio of 0.49, the model approximately simulated the behaviour of an undrained soil. The effect of Poisson's Ratio for the soil was checked by running analyses with ν values of 0.15 and 0.3.

Although the selected value for Young's Modulus of the soil can be considered as representative of a very stiff clay, relative to the modulus of the top cap, it can be considered as representative of any soil likely to be tested in the apparatus. This was confirmed in analyses made with the relative moduli for the cap and soil at 10^5 , 10^4 and 10^3 .

To verify the accuracy of the axisymmetric finite element program, stress analyses were made for various combinations of pressures acting on an open-ended (unrestrained) thick-walled hollow cylinder. Comparisons with closed-form elastic solutions for both stresses and deformations showed excellent agreement and provided confidence in the solution method and its application.

The results of the analyses are summarised in Figs 7. A1. 2 to 7. A1. 15. The stresses taken from the finite element analyses are the average values for the Gauss points in the elements. Hamza (1976) recommends that these values are used in problems in which a Poisson's Ratio approaching 0.5 is used. In the figures the stresses are quoted as a percentage of the values found for an unrestrained cylinder under the same combination of applied stresses.

The figures enable the following effects to be assessed:-

- (i) Figs 7. A1. 2 - 7. A1. 4: the influence of sample height on the distribution of σ_z , σ_θ , and σ_r for the case of a specimen 200mm ID x 250mm OD with p_o of 400 kPa, p_i of 350 kPa and σ_z of 400 kPa.
- (ii) Fig. 7. A1. 5: the influence of sample height on the distribution of σ_θ for the same ID and OD but with p_i and p_o equal.
- (iii) Figs 7. A1. 6 and 7. A1. 7: the influence of sample radius on the distribution of σ_z and σ_θ for a specimen 200mm high with p_o of 400 kPa, p_i of 350 kPa and σ_z of 400 kPa.
- (iv) Figs 7. A1. 8 - 7. A1. 10: the influence of $p_i - p_o$ on the distribution of σ_z , σ_θ and σ_r for a specimen 200mm ID x 250mm OD x 250mm high with σ_z of 400 kPa.

- (v) Figs 7. A1. 11 and 7. A1. 12: the influence of the soil's Poisson's ratio on the distribution of σ_z and σ_θ for a specimen 200mm ID x 250mm OD x 250mm high with p_o of 400 kPa, p_i of 350 kPa and σ_z of 400 kPa. (Note that these analyses were made using a three dimensional finite element analysis in which a torque was applied but in which the mesh was coarser than in the axisymmetric analyses.)
- (vi) Fig. 7. A1. 13: the influence of the relative stiffness of the soil and top cap on the distribution of σ_z for a specimen with the same geometry and stresses as (v).
- (vii) Figs 7. A1. 14 and 7. A1. 15: the influence of the applied σ_z on the distribution of σ_z and σ_θ in the wall of a specimen with the same geometry as (v) and with p_o of 400 kPa and p_i of 350 kPa.

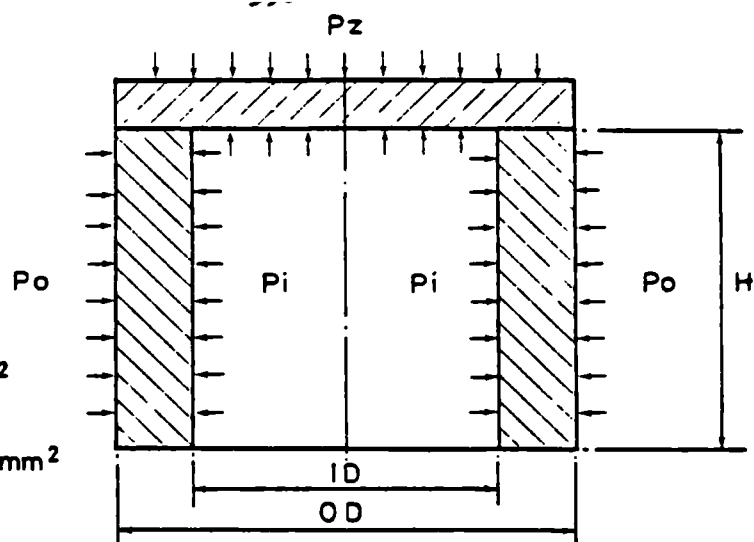
The following conclusions may be drawn from the analyses:-

- (i) end restraint has only a local effect on the distribution of σ_r for all sample heights;
- (ii) non-uniformity in the distribution of σ_z and σ_θ reduces with sample height for a given inner and outer diameter; the level of non-uniformity is sensitive to $(p_o - p_i)$; σ_z and σ_y become more uniform as $(p_o - p_i)$ reduces; (for a given sample height the level of non-uniformity reduces with reducing inner and outer diameter);
- (iii) for all geometries there is a non-uniform distribution of σ_z across the wall; σ_z is higher on the inside face than on the outside face as a result of bending in the top cap (this bending has been eliminated in the final design).
- (iv) end restraint has a tendency to reduce the natural variation of σ_θ across the wall;
- (v) a reduction in soil stiffness has only a small influence on the stress distribution in the central section of the specimen since the change in stiffness relative to the top cap is insignificant;

- (vi) a reduction in Poisson's ratio for the soil increases the non-uniformities resulting from end restraint;
- (vii) in a sample 250mm OD, 200mm ID and 250mm high there is a central gauge length of 125mm in which stresses are reasonably uniform for all load combinations.

The need to minimise ($p_o - p_i$) is obvious. The selected sample geometry has enabled this to be done (see Fig. 7.16). For this geometry the level of non-uniformities which arise in use can be judged from Figs 7. A1. 16 and 7. A1. 17. The stress distributions are shown for the load-pressure combinations required to simulate the stress path followed by an element (z/r_o of 0.25 and r/r_o of 1.0) below a gravity platform when a wave moves from position II to position IV. (p_v was taken as $20t/m^2$ and K_o as 1.).

$E_{\text{soil}} = 90 \text{ N/mm}^2$
 $\nu_{\text{soil}} = 0.49$
 $E_{\text{cap}} = 21 \times 10^4 \text{ N/mm}^2$
 $\nu_{\text{cap}} = 0.2$



ID (mm)	OD (mm)	H (mm)	Pi (kPa)	Po (kPa)	Pz (kPa)
200	250	100	350	400	400
200	250	150	350	400	400
200	250	200	350	400	400
200	250	250	350	400	400
200	250	100	350	350	400
200	250	150	350	350	400
200	250	200	350	350	400
200	250	250	350	350	400
200	250	250	350	400	600
200	250	250	400	350	400
100	150	100	350	400	400
100	150	200	350	400	400
150	200	100	350	400	400
150	200	200	350	400	400

Table 7: A1. 1. Axisymmetric HCA analyses. Summary of geometries and loading cases.

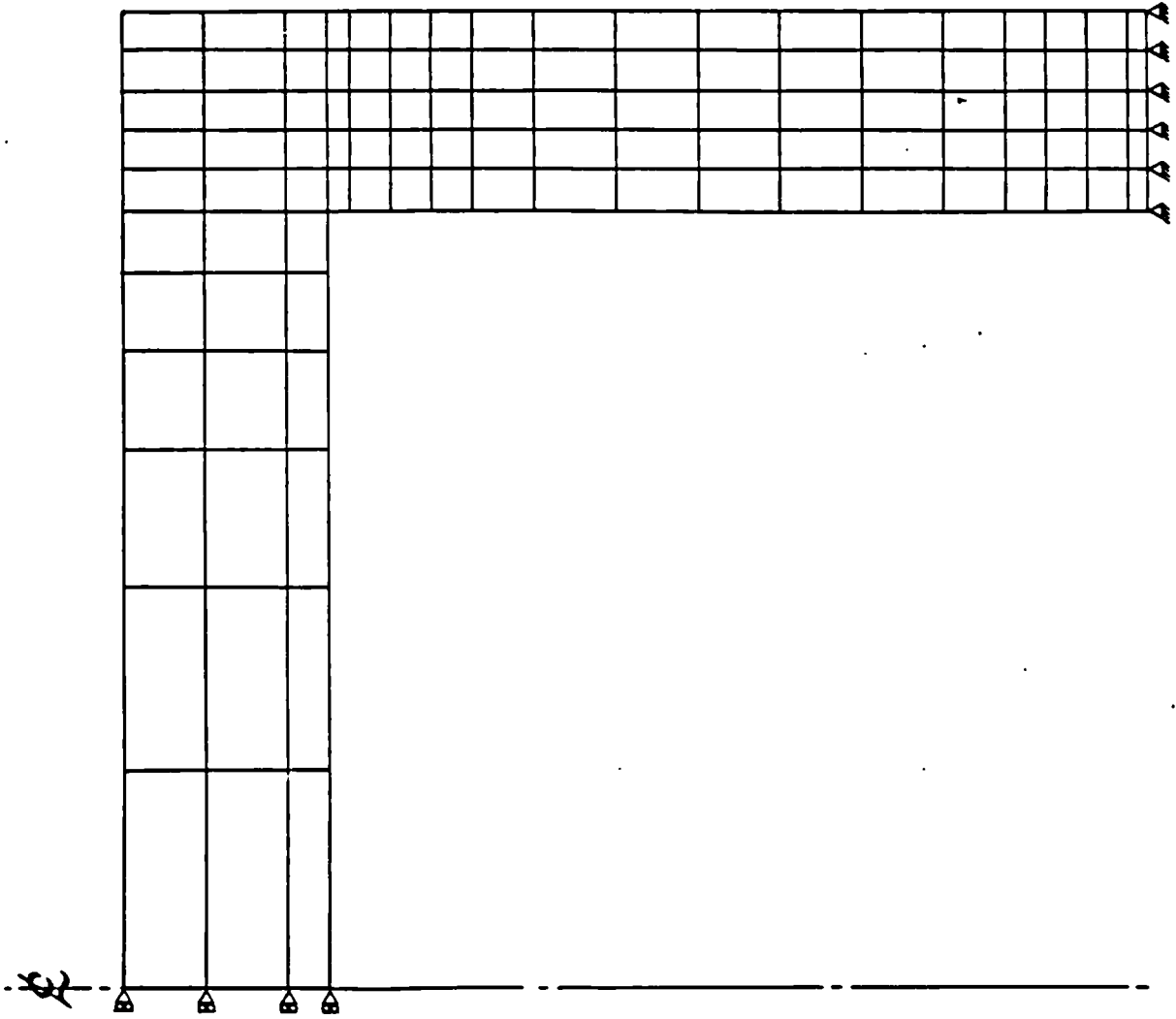


Fig. 7. A1. 1. Mesh for finite element analyses of hollow cylinder samples.

$P_0 = 400 \text{ kPa}$
 $P_i = 350 \text{ kPa}$
 $\sigma_z = 400 \text{ kPa}$
 $ID = 200 \text{ mm}$
 $OD = 250 \text{ mm}$

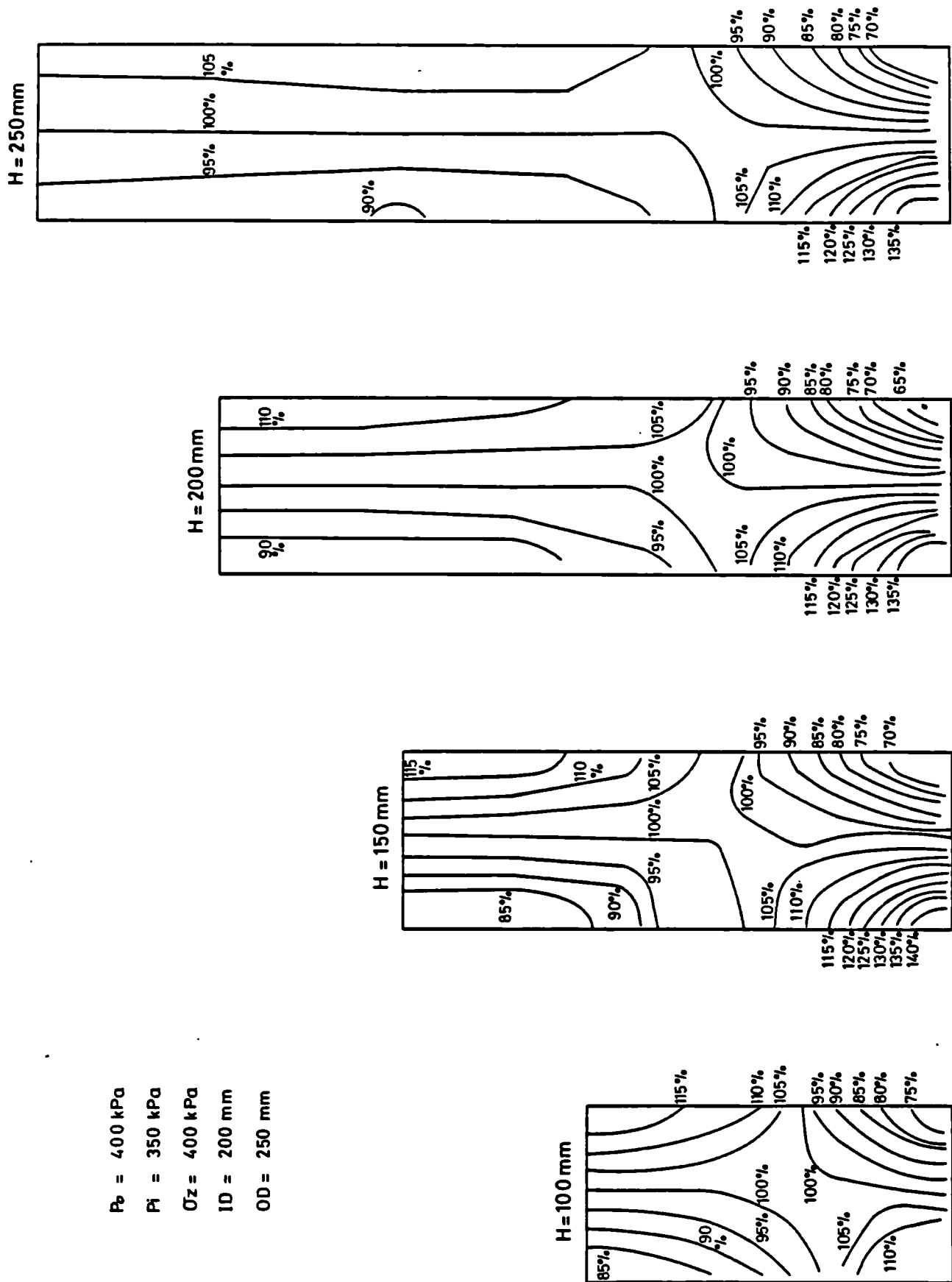


Fig. 7. A1. 2. Influence of sample height on distribution of σ_z/σ_z (unrestrained). ($p_0 > p_i$).

$P_o = 400 \text{ kPa}$
 $P_i = 350 \text{ kPa}$
 $\sigma_z = 400 \text{ kPa}$
 $ID = 200 \text{ mm}$
 $OD = 250 \text{ mm}$

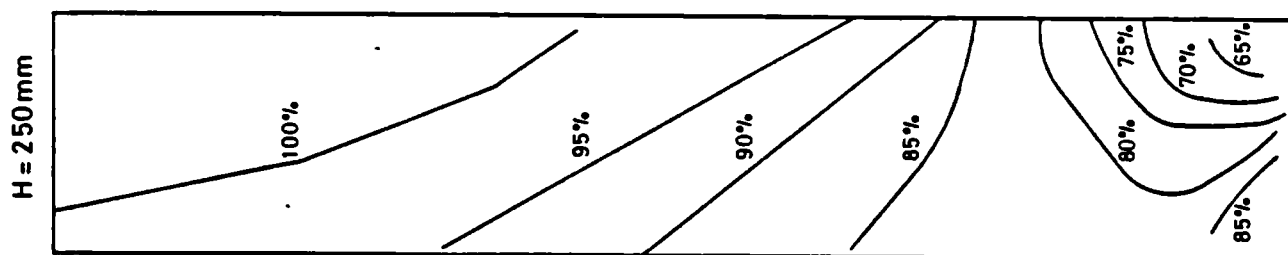
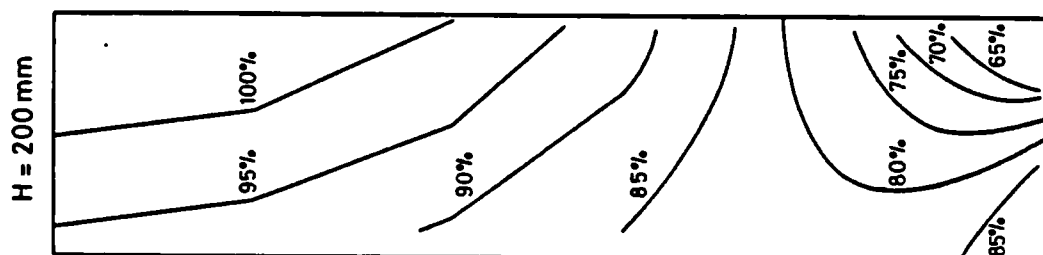
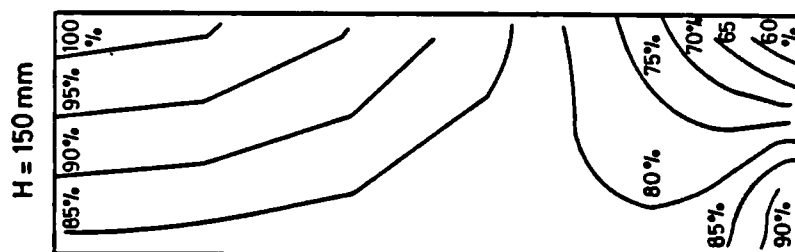
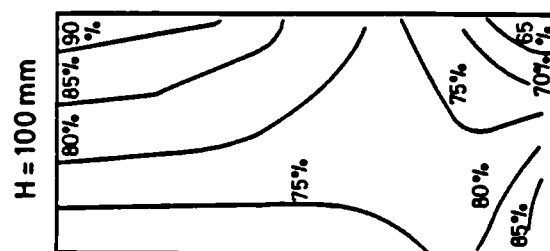
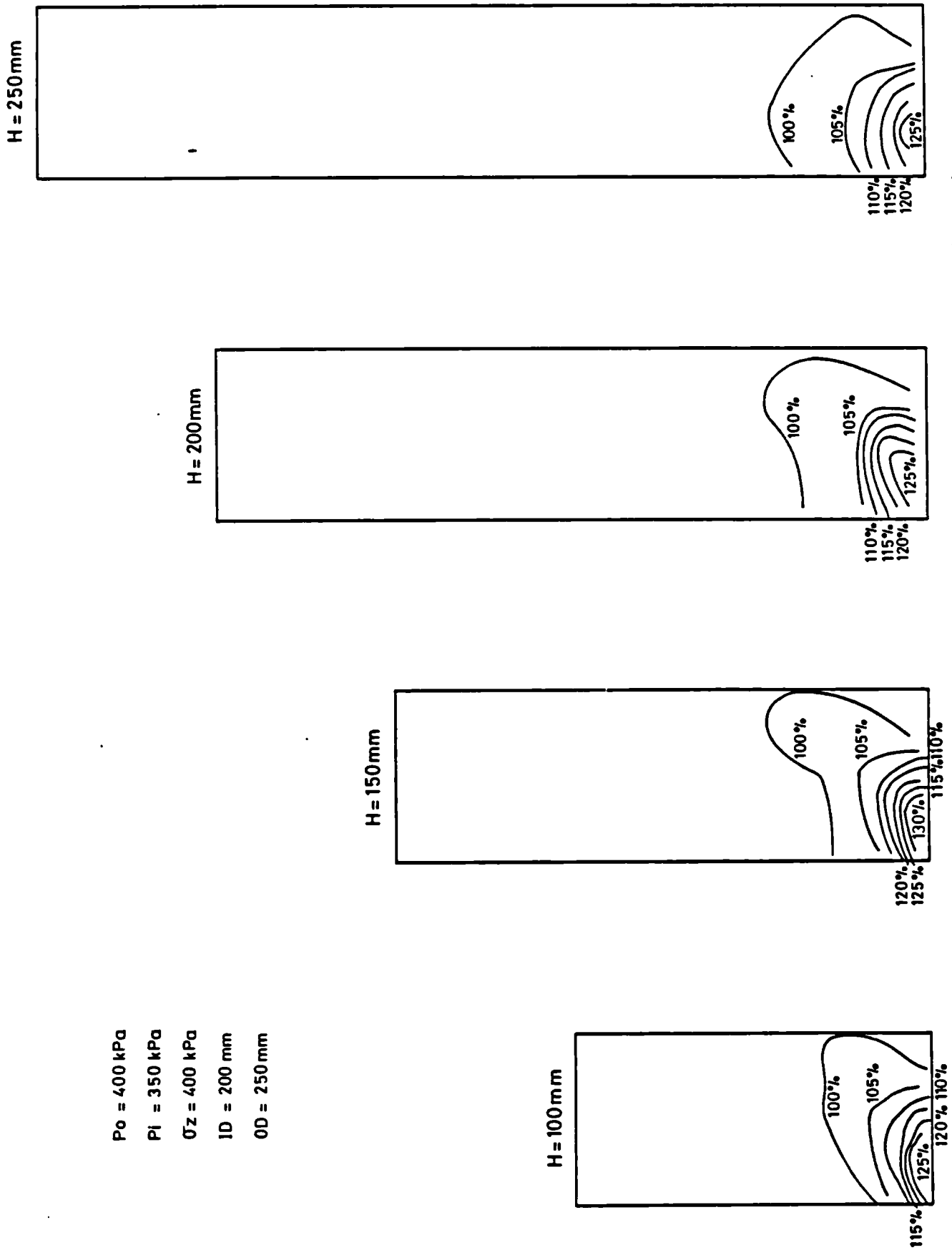


Fig. 7. A1. 3. Influence of sample height on distribution of σ/σ_0 (unrestrained). ($p_o > p_i$).

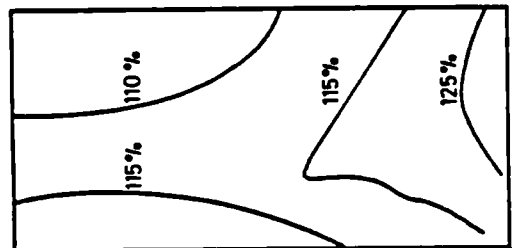


$P_o = 400 \text{ kPa}$
 $P_i = 350 \text{ kPa}$
 $\sigma_z = 400 \text{ kPa}$
 $ID = 200 \text{ mm}$
 $OD = 250 \text{ mm}$

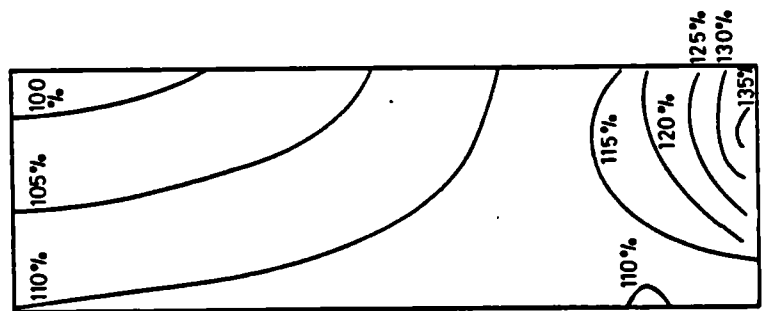
Fig. 7. A1. 4. Influence of sample height on distribution of σ_z/σ_r (unrestrained). ($p_o > p_i$).

$P_o = 350 \text{ kPa}$
 $P_i = 350 \text{ kPa}$
 $\sigma_z = 400 \text{ kPa}$
 $ID = 200 \text{ mm}$
 $OD = 250 \text{ mm}$

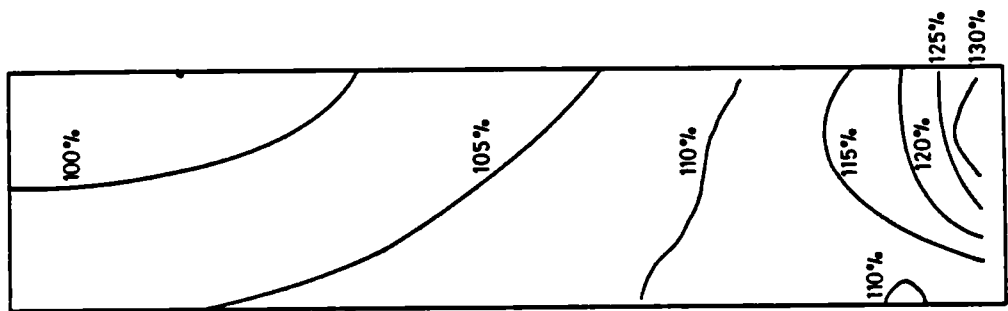
H = 100 mm



H = 150 mm



H = 200 mm



H = 250 mm

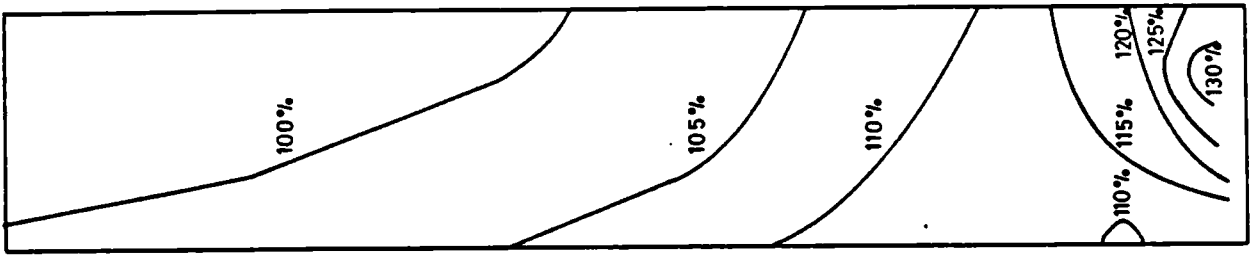


Fig. 7. A1. 5. Influence of sample height on distribution of σ/σ_0 (unrestrained). ($p_o = p_i$).

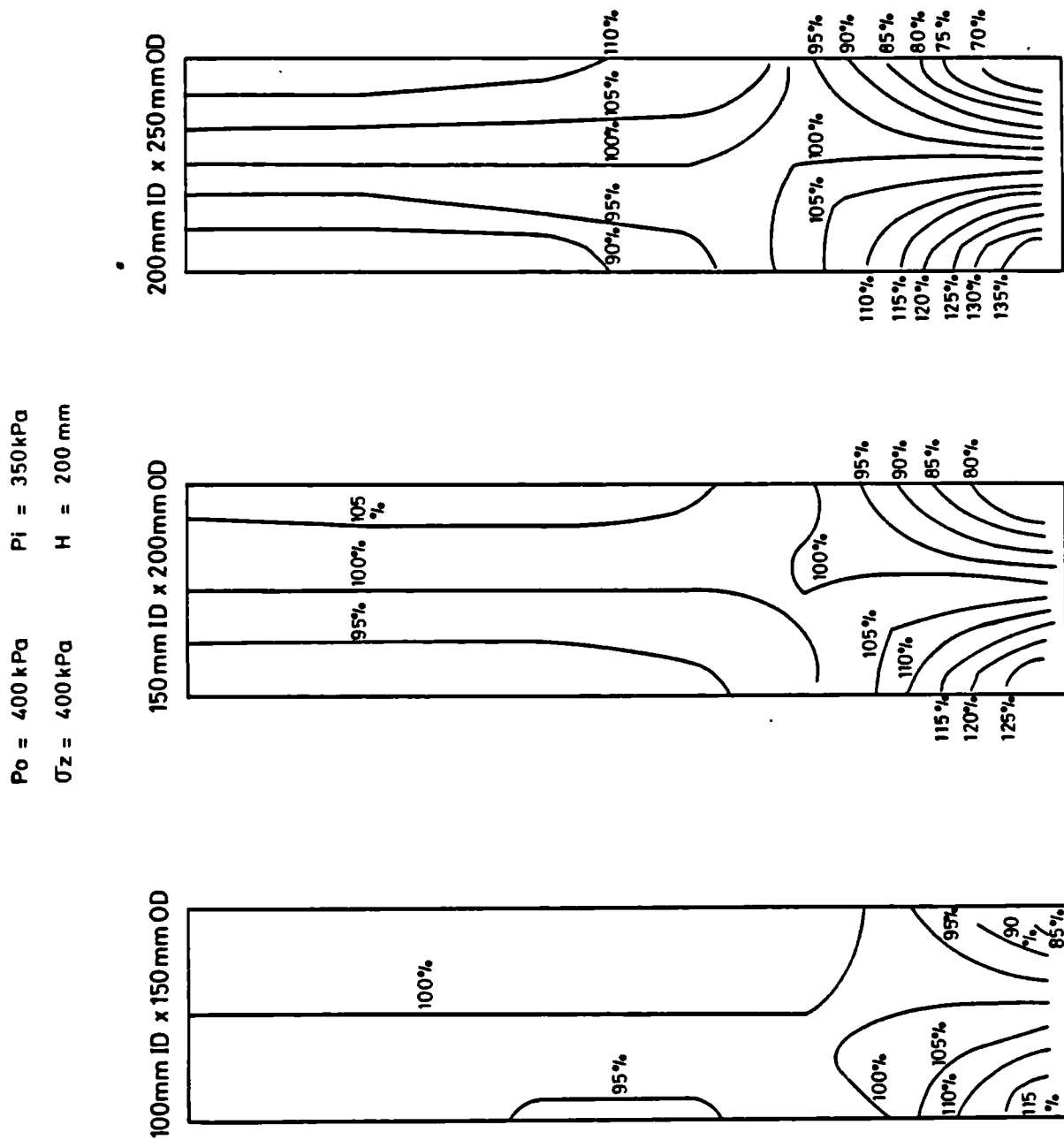


Fig. 7. A1. 6. Influence of sample radius on distribution of σ_z'/σ_r' (unrestrained). ($p_o > p_i$).

$P_o = 400 \text{ kPa}$ $P_i = 350 \text{ kPa}$
 $\sigma_z = 400 \text{ kPa}$ $H = 200 \text{ mm}$

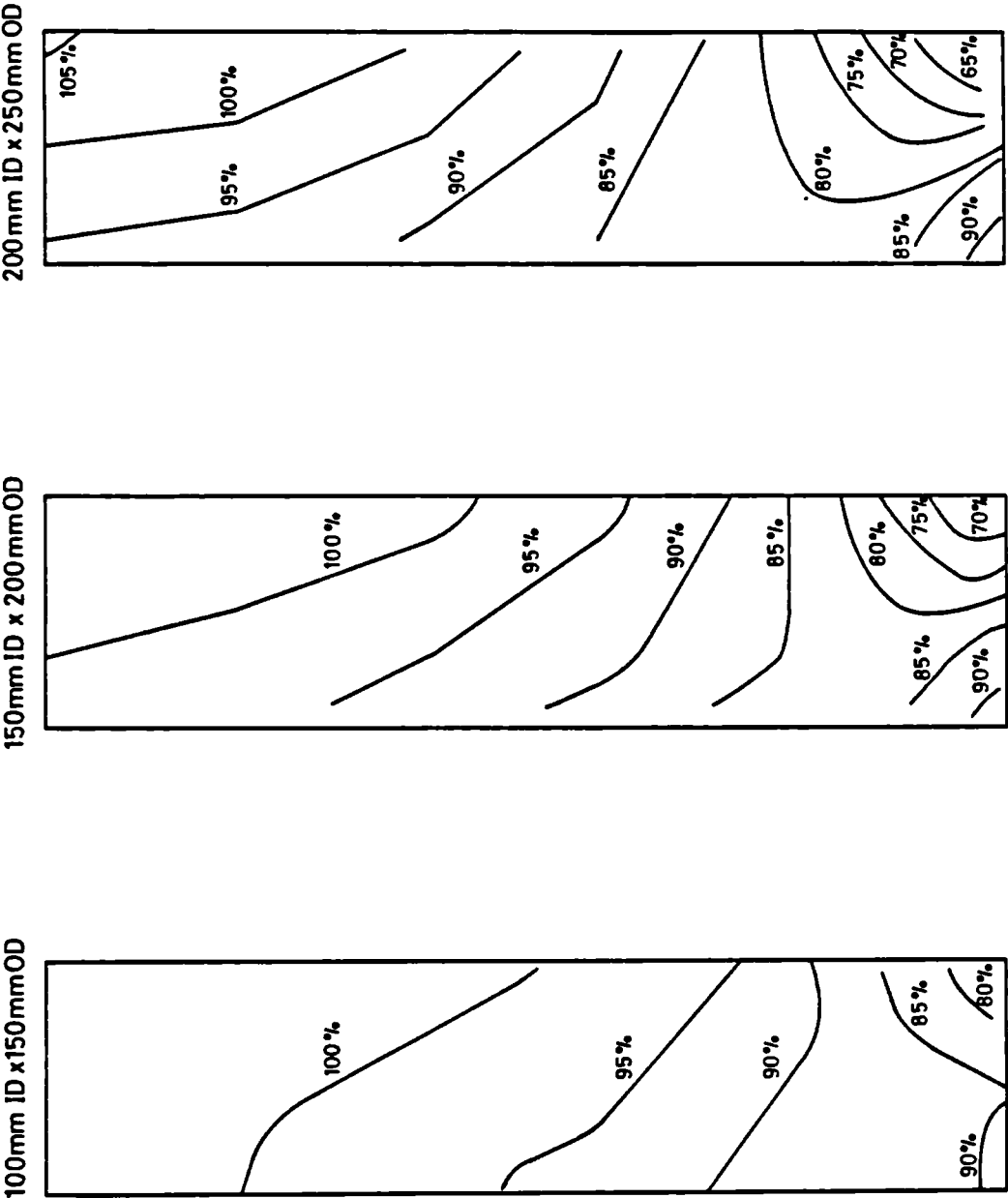


Fig. 7. A1. 7. Influence of sample radius on distribution of σ_r/σ_o (unrestrained). ($p_o > p_i$).

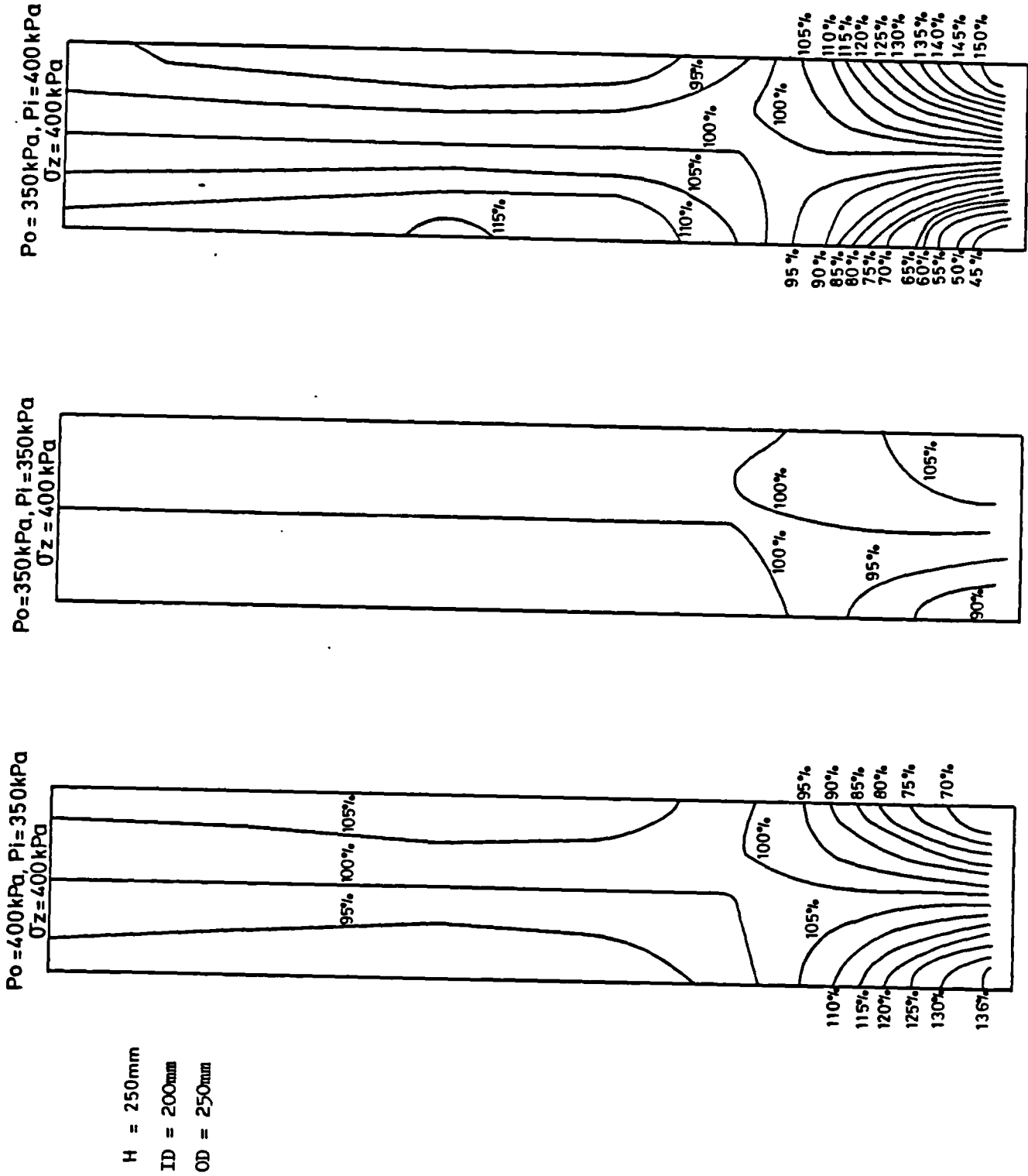


Fig. 7. A1. 8. Influence of $(p_i - p_o)$ on distribution of $\sigma_z / \bar{\sigma}_z$ (unrestrained) for specimen 250mm high x 200mm ID x 250mm OD

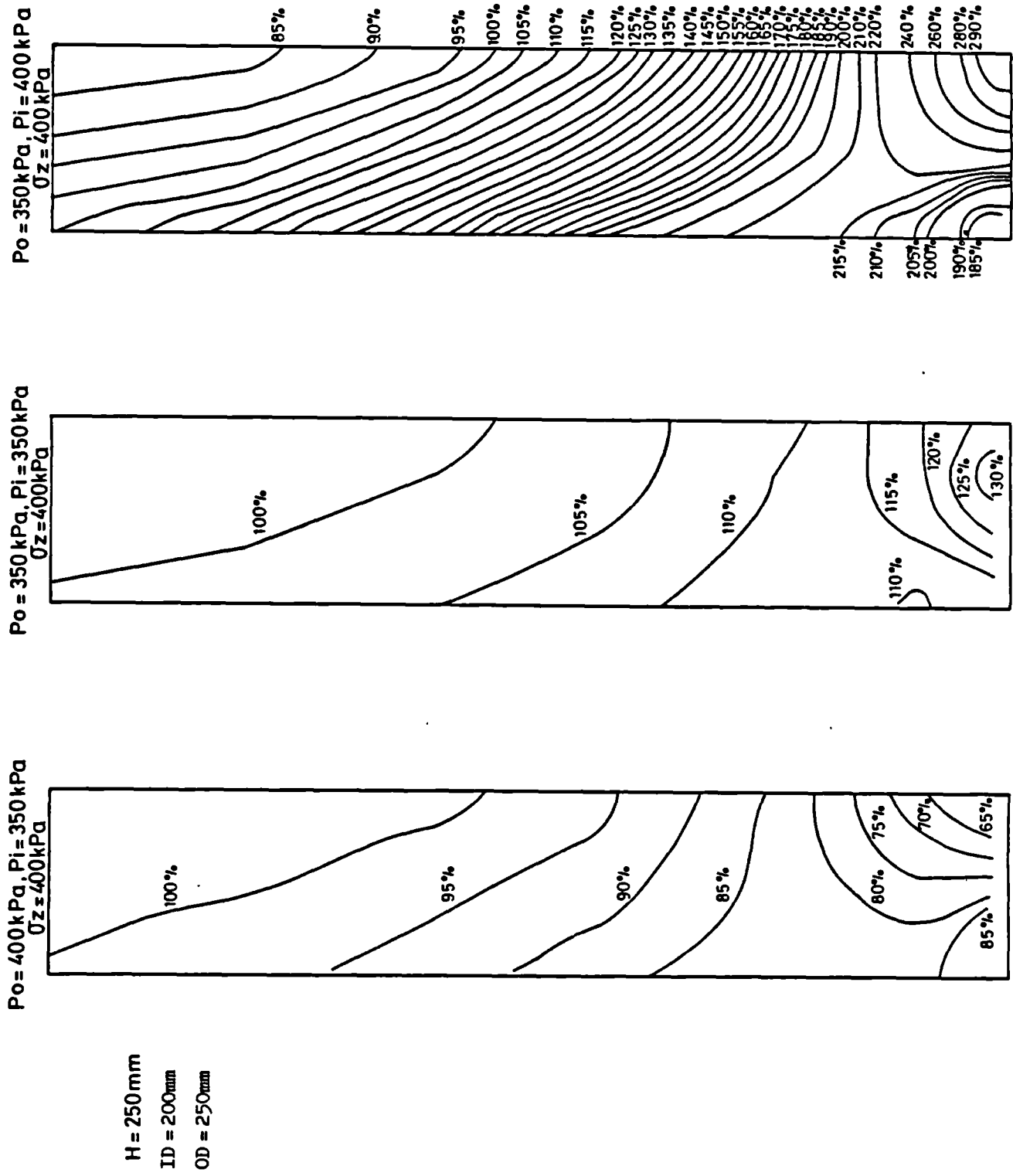


Fig. 7. A1. 9. Influence of $(p_i - p_o)$ on distribution of σ_0/σ_0 (unrestrained) for specimen 250mm high x 200mm ID x 250mm OD.

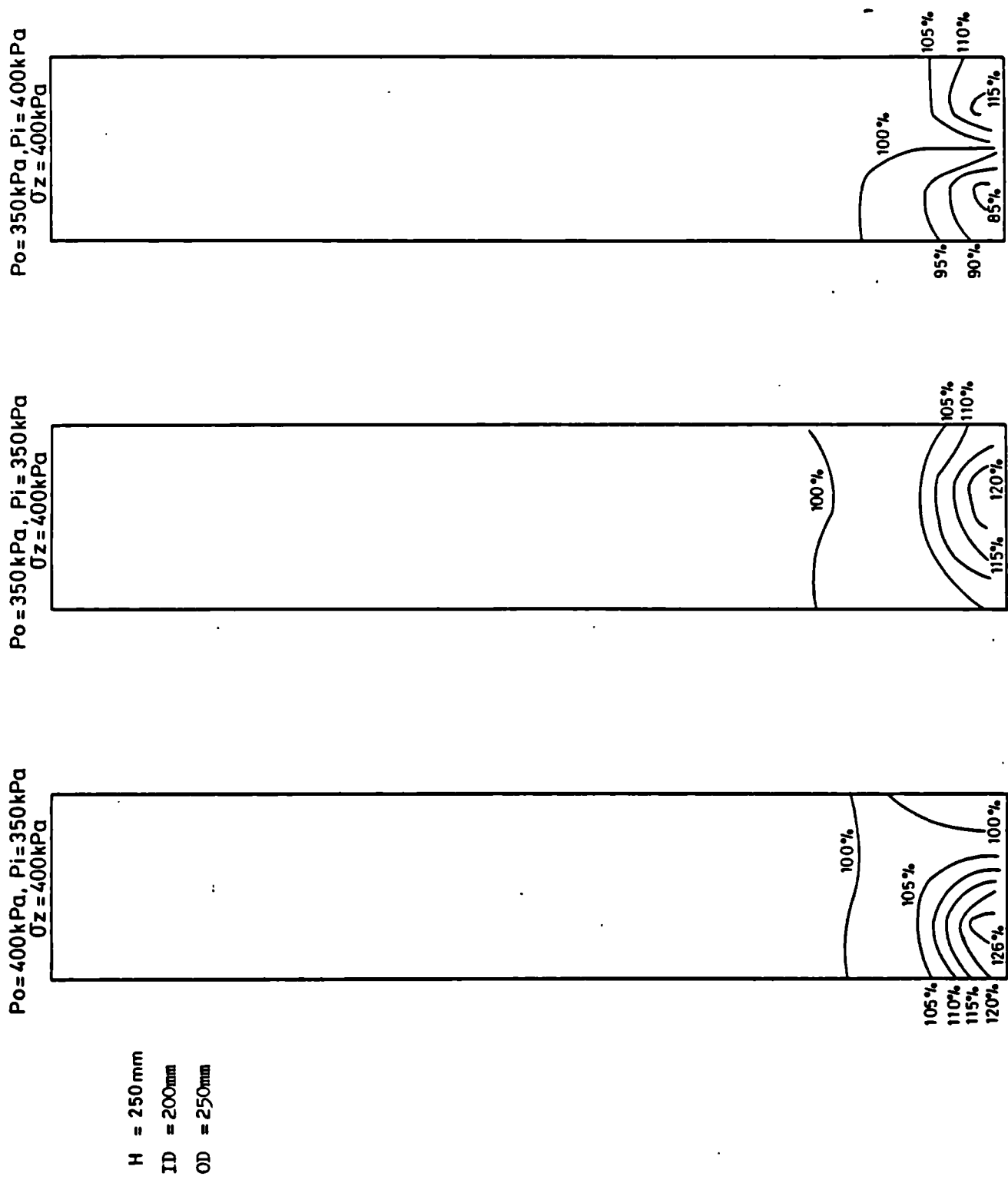


Fig. 7. A1. 10. Influence of $(p_i - p_o)$ on distribution of σ_r/σ_r (unrestrained) for specimen 250mm high x 200mm ID x 250mm OD.

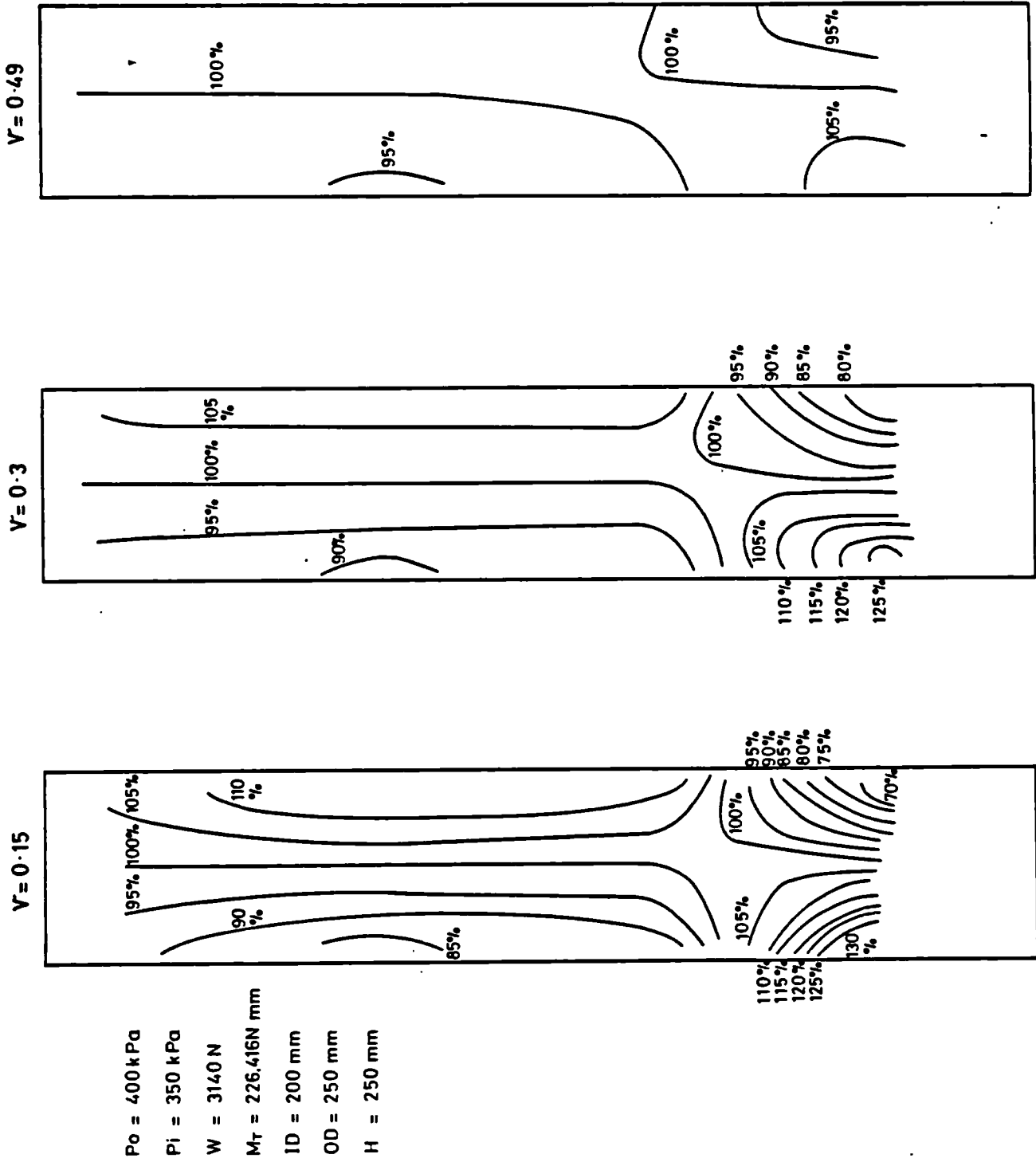


Fig. 7. A1. 11. Influence of soil's Poisson's ratio on distribution of σ_x/σ_z (unrestrained).

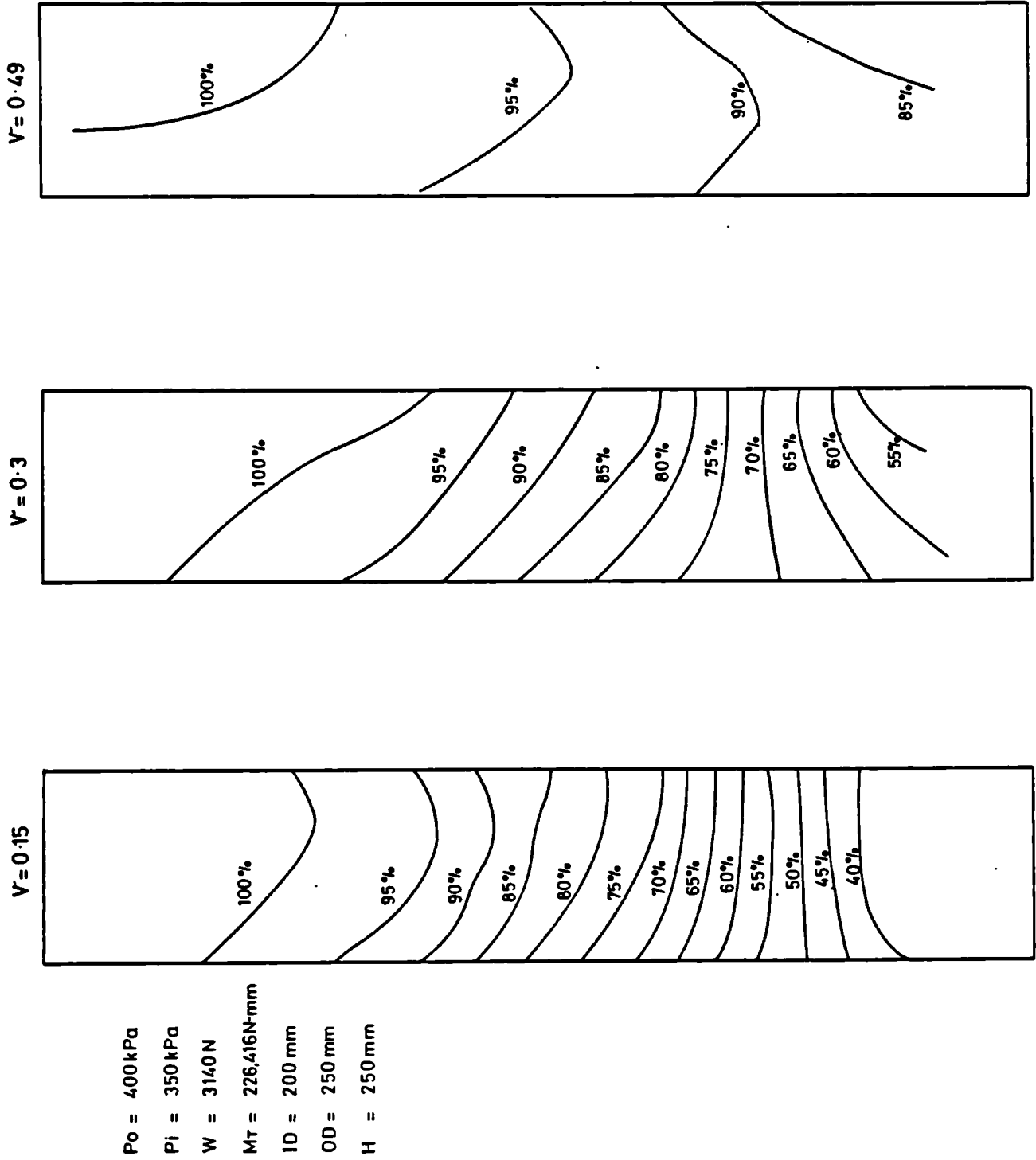


Fig. 7. A1. 12. Influence of soil's Poisson's Ratio on distribution of σ_0/σ_e (unrestrained).

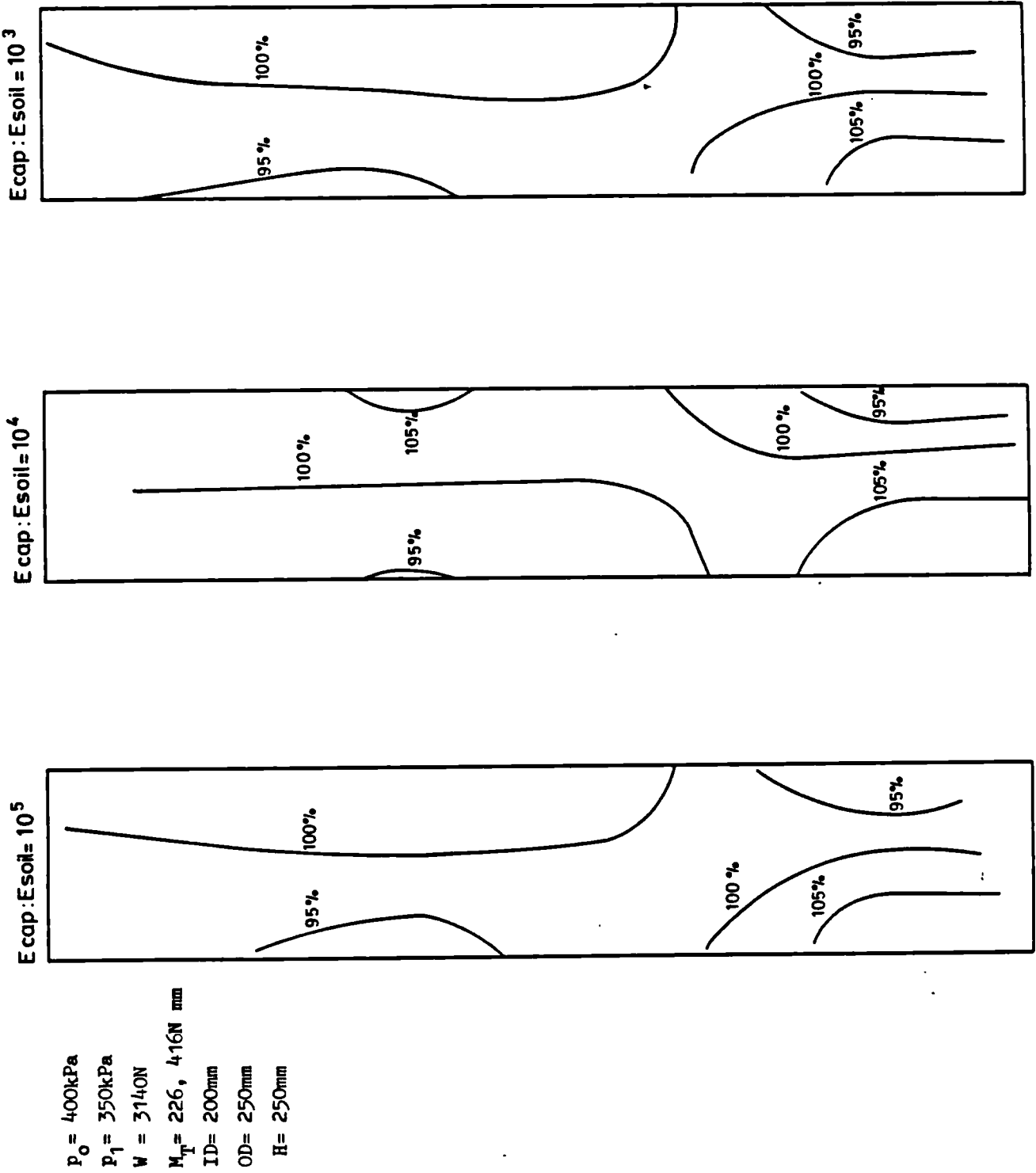


Fig. 7. A1. 13. Influence of relative stiffness of soil and top cap on distribution of σ_z/σ_z (unrestrained).

$P_o = 400 \text{ kPa}$
 $P_i = 350 \text{ kPa}$
 $ID = 200 \text{ mm}$
 $OD = 250 \text{ mm}$
 $H = 250 \text{ mm}$

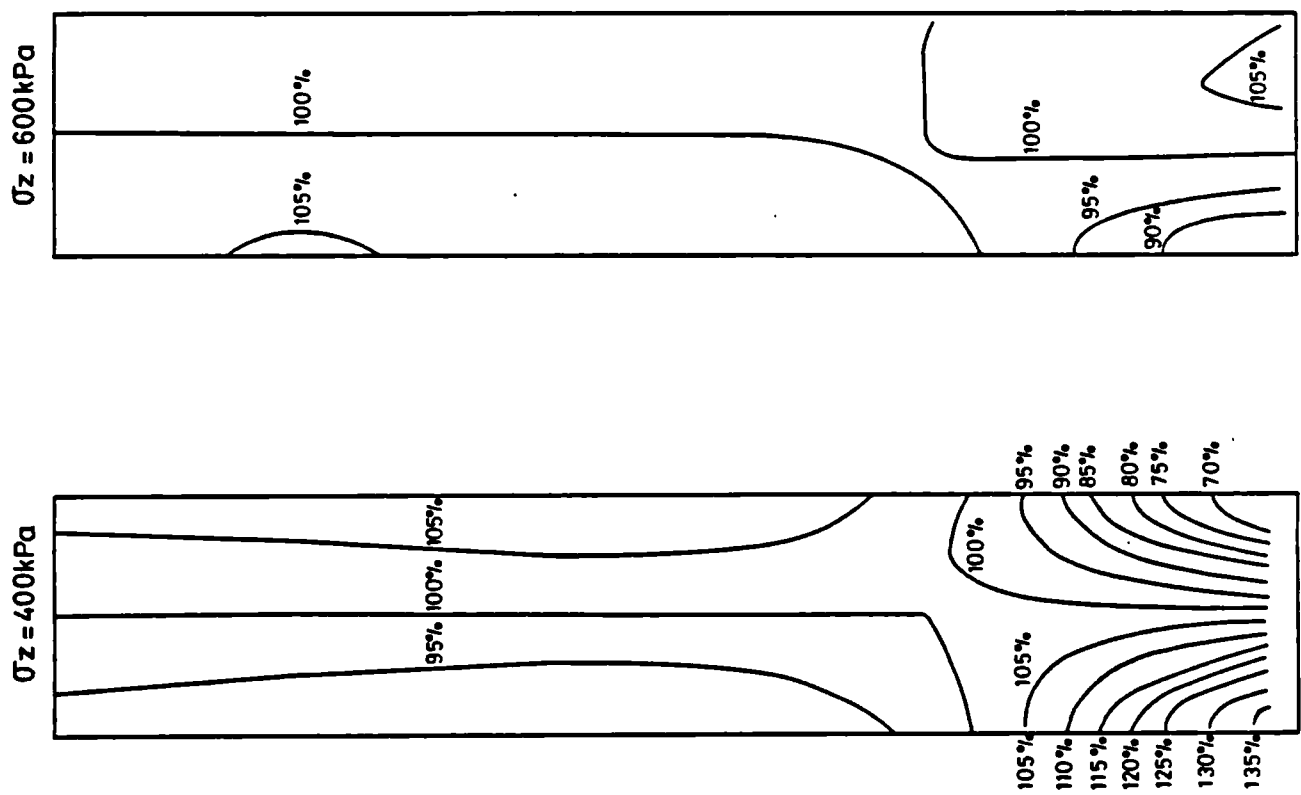
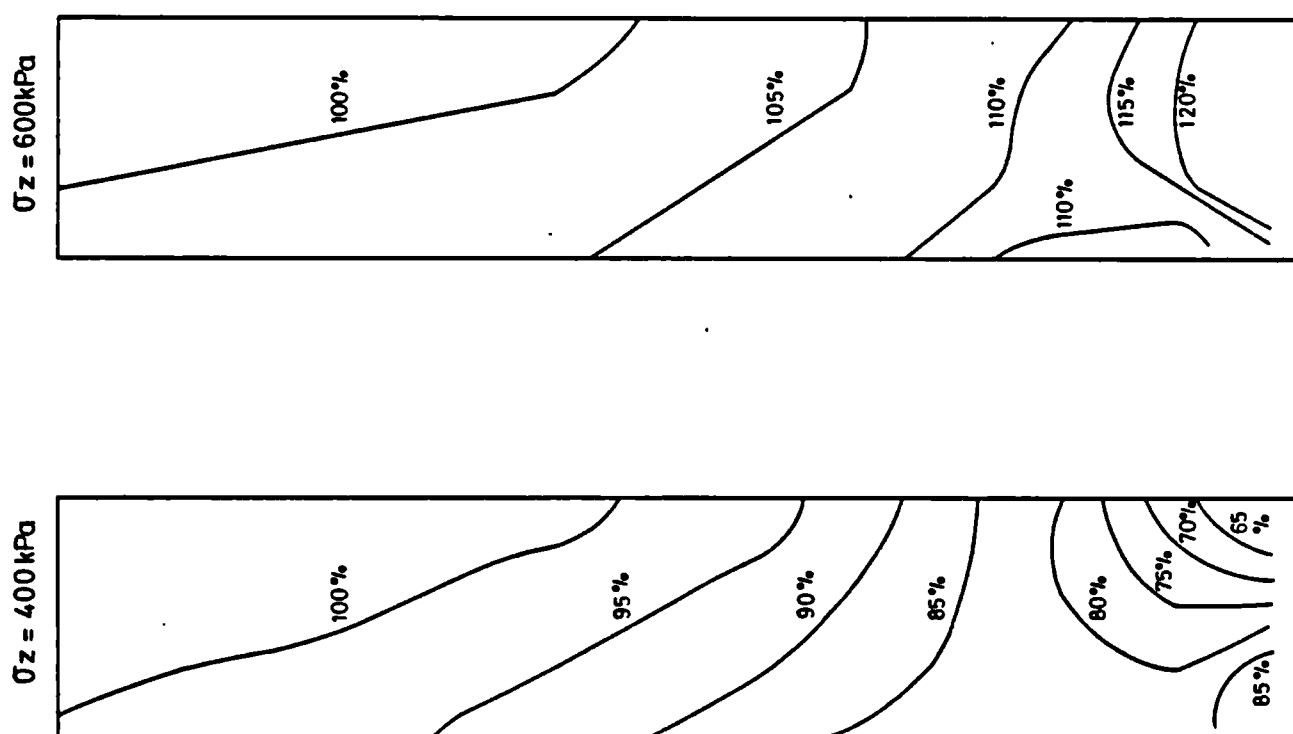


Fig. 7. A1. 14. Influence of σ_z on the distribution of σ_z/σ_z (unrestrained).



$P_o = 400 \text{ kPa}$
 $P_i = 350 \text{ kPa}$
 $ID = 200 \text{ mm}$
 $OD = 250 \text{ mm}$
 $H = 250 \text{ mm}$

Fig. 7. A1. 15. Influence of σ_z on the distribution of σ_e/σ_e (unrestrained).

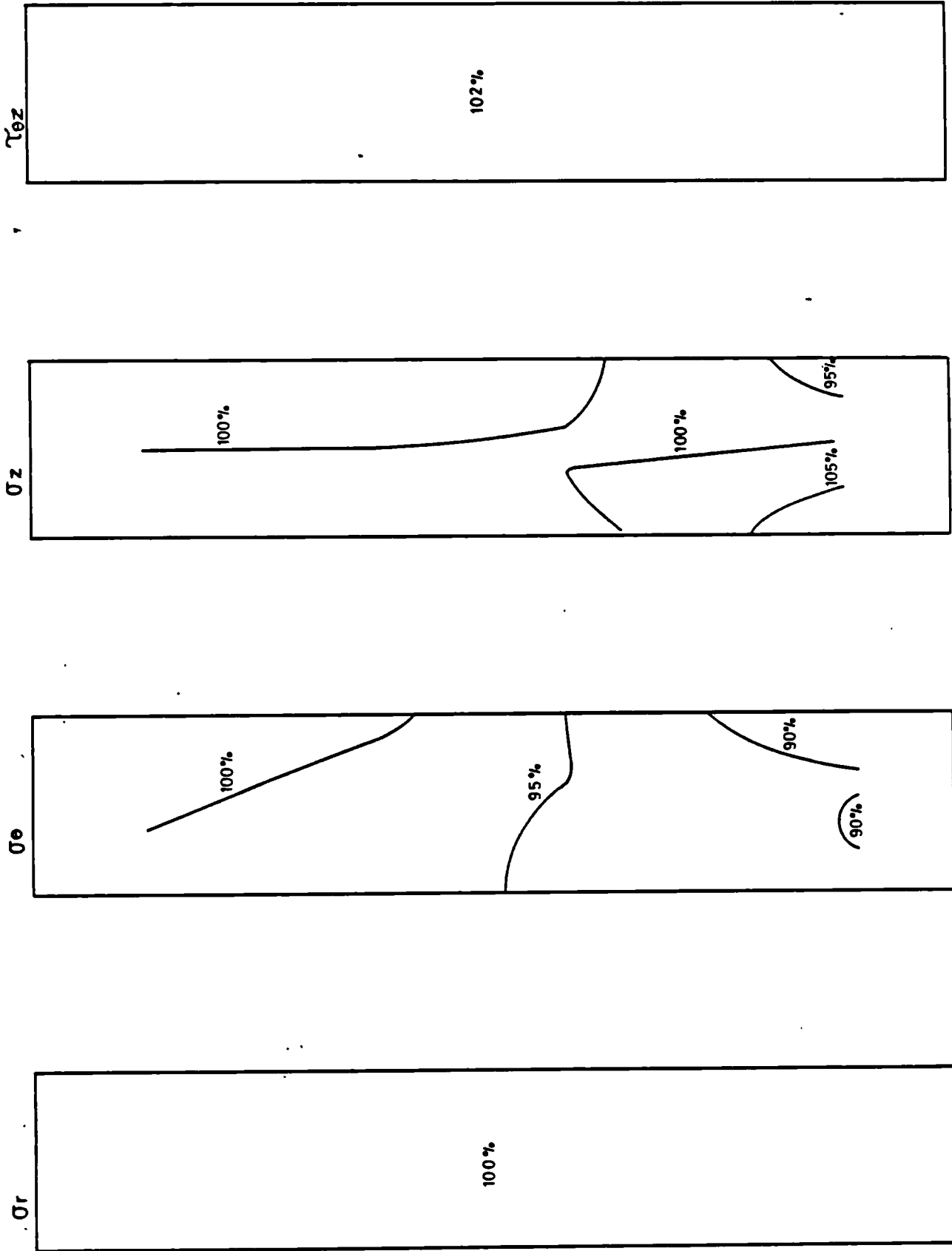


Fig. 7. A1. 16. Stress distributions when simulating stress path for element beneath gravity platform. $z/r_0 = 0.25$, $r/r_0 = 1.0$. Wave position 2.

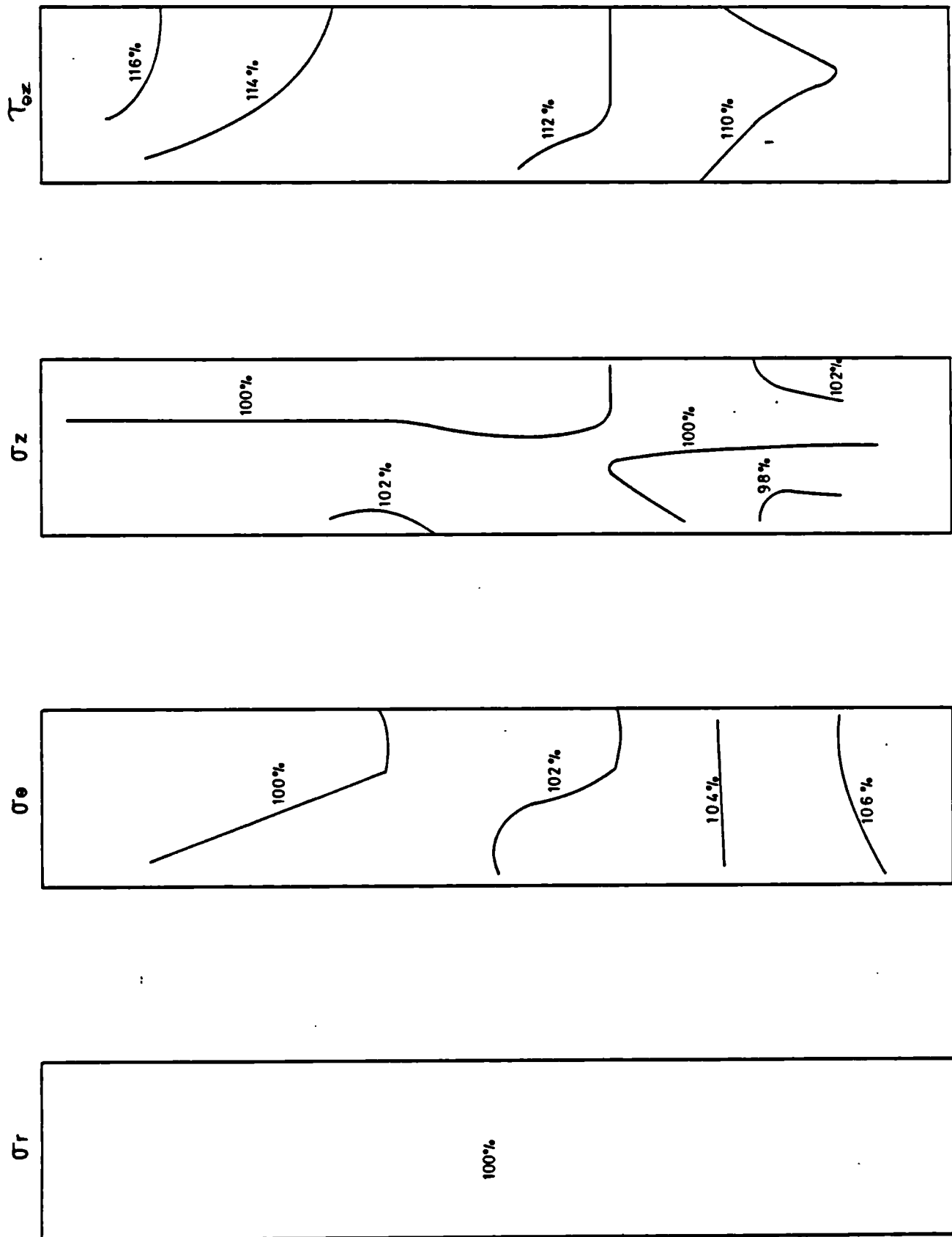


Fig. 7. A1. 17. Stress distributions when simulating stress path for element beneath gravity platform. $z/r_0 = 0.25$, $r/r_0 = 1.0$. Wave position 4.

APPENDIX 7.2

Combinations of W , M_T , p_o and p_i Required to Simulate Stress Paths Beneath Gravity Platforms Using New Hollow Cylinder Apparatus

To illustrate the load-pressure combinations required to follow field stress paths in the new hollow cylinder apparatus, the following example has been taken:

a gravity platform loaded as shown in Fig. 2.10 with p_v of $20t/m^2$.

The load-pressure combinations which produce the stress changes after consolidation are shown in Figs 7. A2. 1 to 7. A2. 3 for wave positions I to V for elements at $z/r_o = 0.25, 0.5$ and 0.75 , and r/r_o of $0.25, 0.5, 0.75, 1.0, 1.25$ and 1.5 . The point to emphasise is the relatively small difference between p_o and p_i required to simulate these stress paths. The stress non-uniformities associated with these load-pressure combinations are considered in Appendix 7.1 for wave positions II and IV.

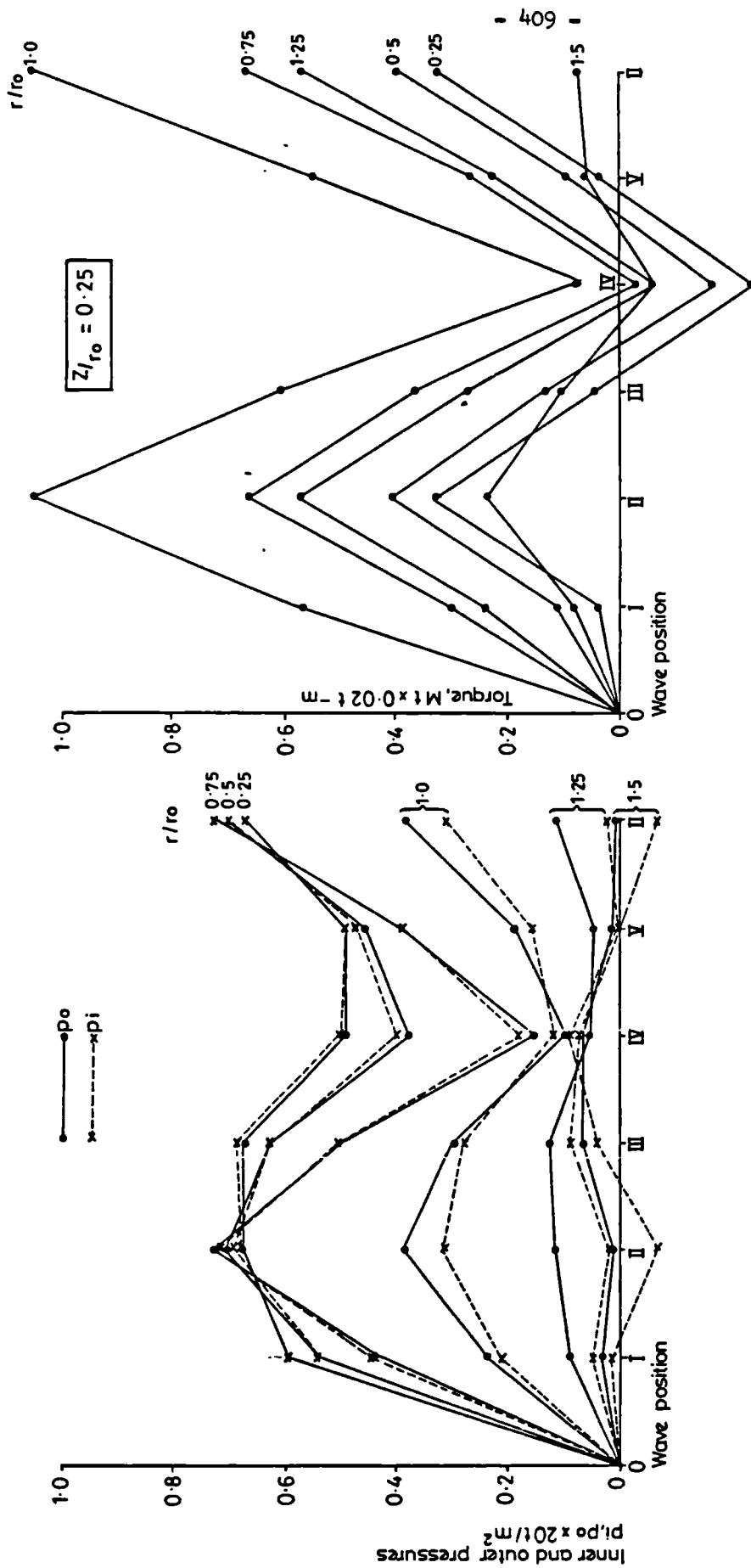
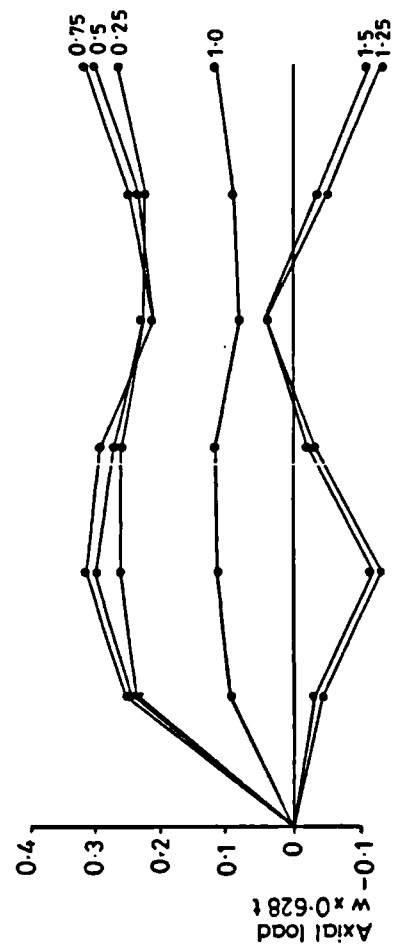


Fig. 7. A2. 1 Load and pressure combinations for simulating stress path for gravity platform under wave loading, $z/r_0 = 0.25$.



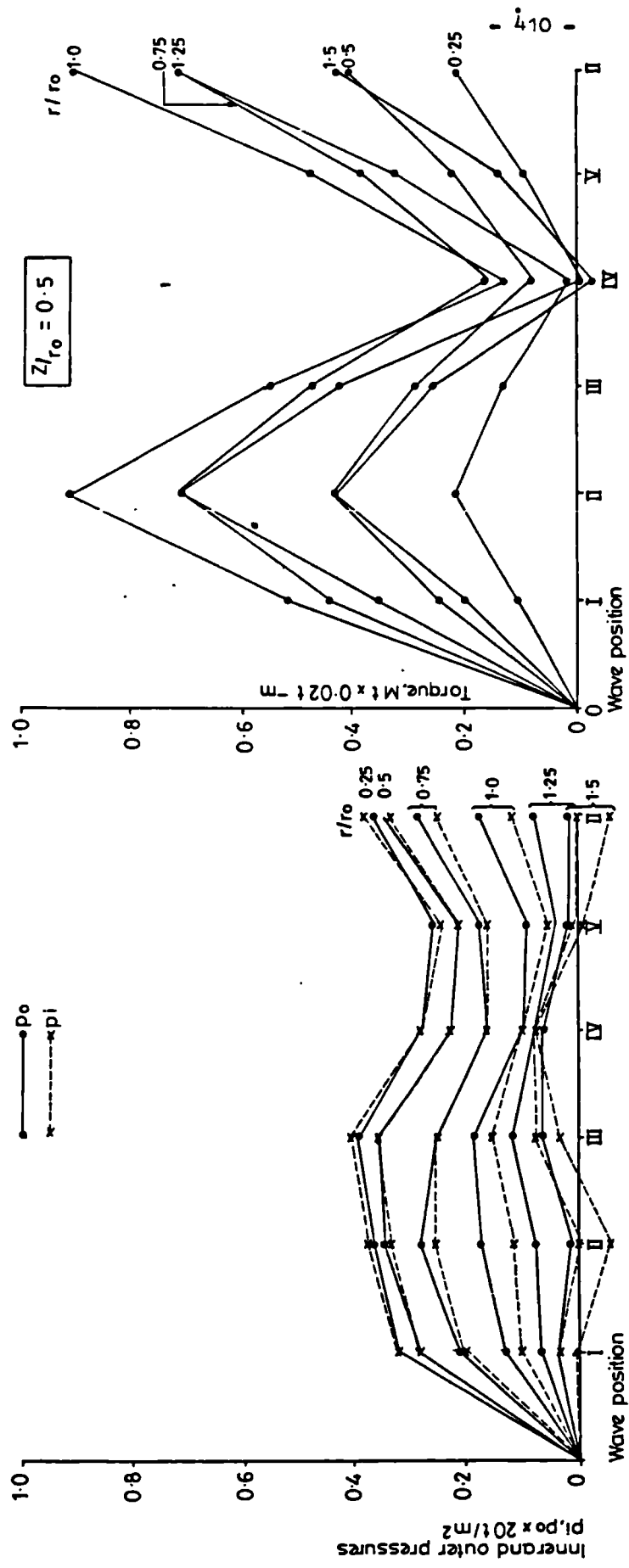
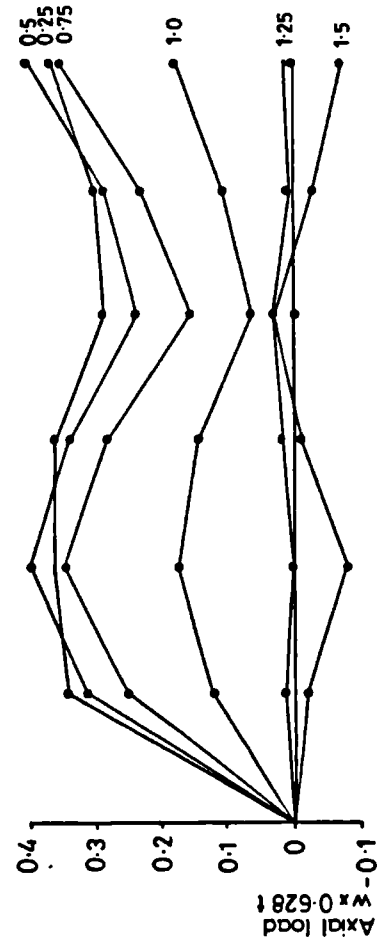


Fig. 7. A2. 2 Load and pressure combinations for simulating stress path for gravity platform under wave loading, $z/r_o = 0.5$.



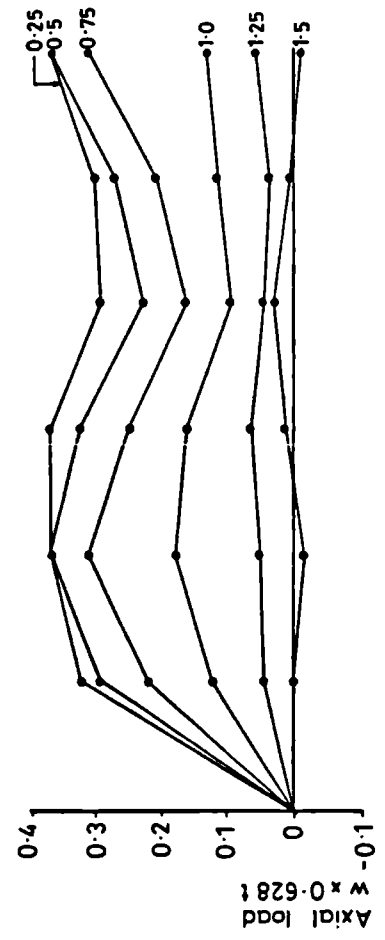
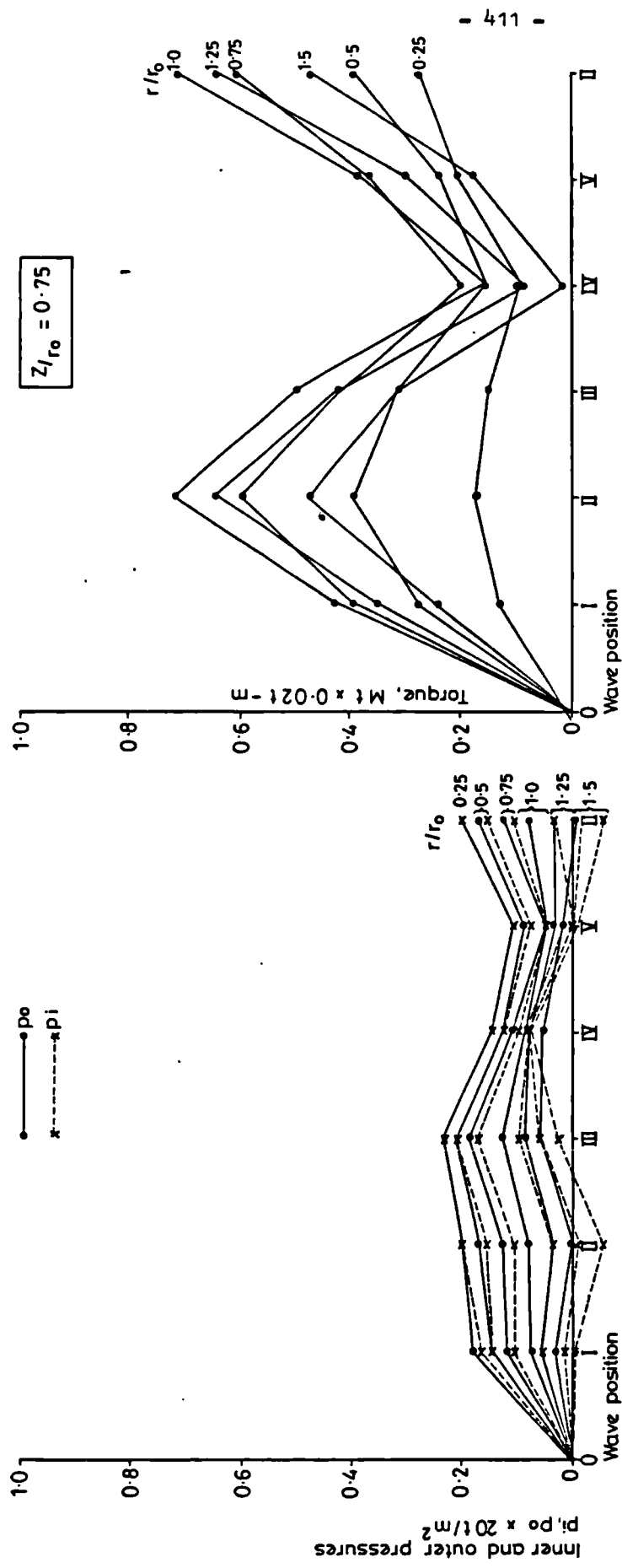


Fig. 7. A2. 3 Load and pressure combinations for simulating stress path for gravity platform under wave loading, $z/r_0 = 0.75$.