VOLUME CHANGE OF

COMPACTED HIGHLY PLASTIC AFRICAN CLAYS

by

Hilson Deneys Schreiner, MSc. Eng

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ABSTRACT

An experimental investgation of volume change of compacted highly plastic clays was undertaken. The primary interest lay in swelling under controlled decrease of suction. Three supporting studies were considered neccessary, and a small amount of work was done on strength determination of unsaturated soils.

The first supporting study reviewed empirical correlations between Index test data and swell measured in the oedometer. It was found that there was little agreement between available methods. An attempt to determine which index parameters were most strongly linked with swell showed that poor correlations were obtained for most of the commonly used index parameters. A moderately good correlation was obtained using plastic limit, liquid limit, shrinkage limit and clay fraction together, but the data base is too small for this to be recommended as a universal correlation.

The second supporting study was of the volume change in an oedometer on soaking to zero suction. It was found that the void ratio after swelling reflected the void ratio-compaction moisture content curve at vertical stresses up to 64kPa. The samples which had been compacted at the lowest moisture content swelled to the highest void ratio and were more compressible than the samples which had been compacted at higher moisture contents.

Three techniques for suction measurement were used. The psychrometer and the filter paper techniques were used for total suction measurement, and the pressure plate and the filter paper techniques were used for matrix suction measurement. Some development was done on the pressure plate and the filter paper techniques, and consistent measurements of total and matrix suction were obtained.

Volume change under controlled suction was studied in an oedometer. This oedometer was designed to permit simultaneous measurement of suction, radial stress and vertical stress. Suction control and measurement is done with elevated pore air pressure. The radial stress is measured with a load cell penetrating through the oedometer wall. Axial load is applied hydraulically.

The stress paths observed in the oedometer were used to compare three commonly used procedures for swell determination in the standard oedometer. It was found that differences in stress path or final stress state or both exist between these three methods which had been widely considered to be equivalent. Swell and collapse have been observed to occur during a single test. The final void ratio has been shown to lie below the normally consolidated line, in contrast to the observations for collapse.

Solute suction has been shown to have a large effect on the swelling, and should be considered in future testing.

A radial displacement gauge was developed for measurement of volume change of unsaturated samples in the triaxial cell. This was used for one test, and proved satisfactory.

To my sister Jennifer, who has devoted her life to improving the lives of others, and who once told me never to write down to my audience, because people who are more stupid than I am cannot read.

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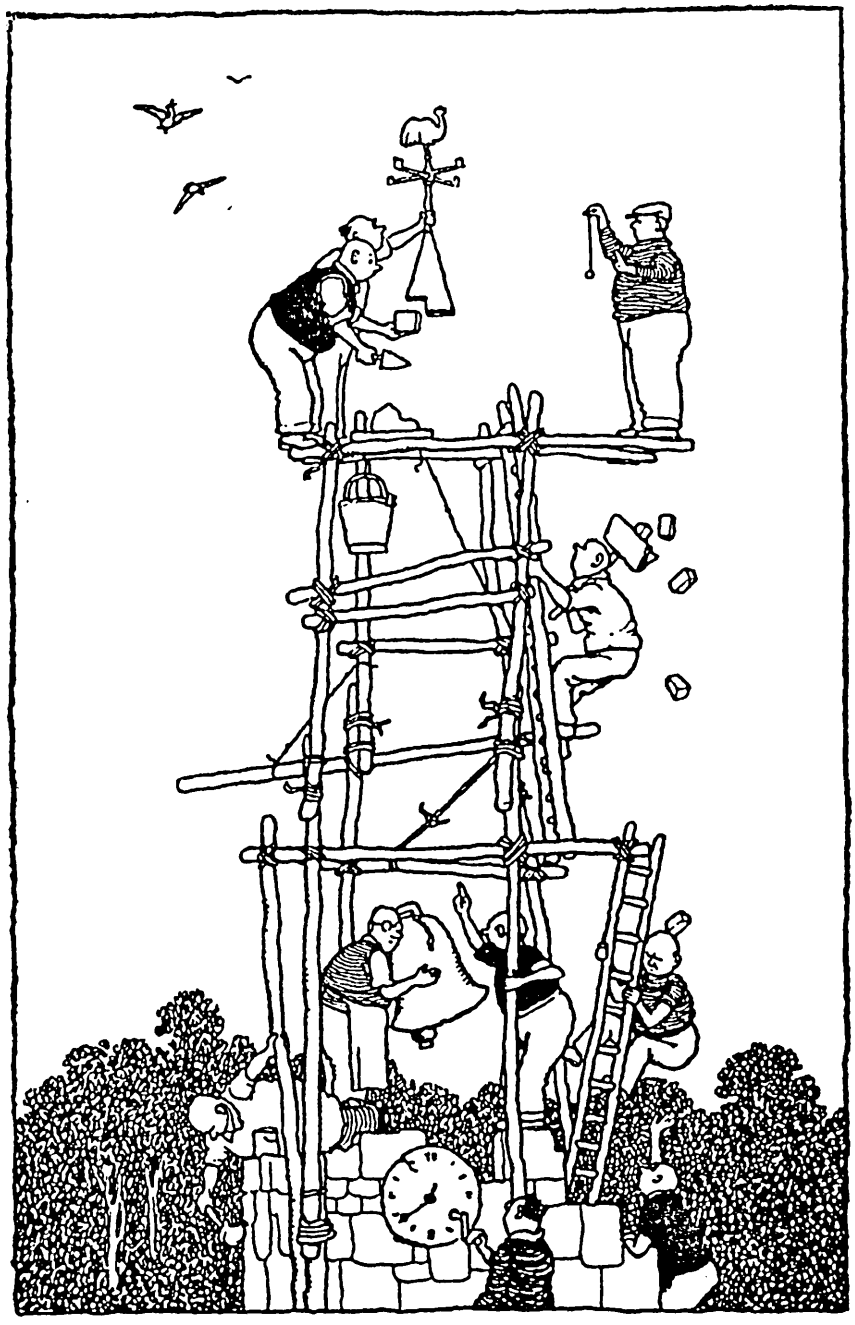
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Finally, Dr. Toll must recieve credit for the equipment which I took over, for the laboratory techniques which I learnt from him and for the computer control system for the triaxial stress path machine.



Restoring the belfry of a village church shattered during the War

(Robinson 1966)

or

The Art of Soil Mechanics

CHAPTER 1

OUTLINE OF THESIS

1.1 INTRODUCTION

The study of ground movements due to swelling of expansive soils is not new. The bulk of the published work dates from the 1950's and later, but there are occasional forays into print before then. Most of the research, both academic and practical, has been performed in oedometers of one sort or another. Very little work has been done using triaxial equipment. Control of suction has been routine in Soil Science, but has not been extensively used in Soil Mechanics as applied to expansive soils.

From a very early stage there has been a tendency to standardise the test procedures used for expansive soil study. This is rather unfortunate, because the standardised procedures are still in use, although Soil Mechanics has progressed to such a stage that some of these procedures are now obviously inapplicable to the real stress changes.

This research project has attempted to study swelling of expansive soils in a manner more in keeping with modern understanding of Soil Mechanics. One of the most important advances over the standard test procedures is the measurement of both major and minor principal total stresses together with the pore water pressure or suction. Tests of this type generally take much longer than the standard oedometer type tests, and as a result, a large number of the latter have also been performed. Since no suitable test equipment was available for the measurement of swelling with measurement of both principal stresses and the suction, it was necessary to design and develop an oedometer capable of providing the required measurements.

In addition, study has been made of related topics. Some of these studies became necessary during the main research work, while others were thought to be of value in the overall picture.

1.2 OUTLINE

The current state of the art of soil mechanics as applied to expansive soils is reviewed in Chapter 2. The main theme of the review is the mechanics of volume change, although shear strength is included. The influence of soil chemistry on the interaction between soil and water is considered in some detail in chapter 3. Since this is fundamental to the volume change of highly plastic

soils, the principles involved must at the very least be known, and preferably be understood.

After these review chapters, it is possible to discuss the target and form of the research. Chapter 4 includes a detailed review of the available stress path test data for swelling of unsaturated soils, and outlines the direction chosen for this project.

Since several soils were used in this project, Chapter 5 is devoted to a fairly full description of the soils. The information includes trial pit profiles with engineering descriptions of the soils, index test data and mineralogical data.

The major products of the research are reported in the subsequent chapters. Identification of expansive soils is adressed first in Chapter 6. The standard oedometer swell tests are presented next, in Chapter 7, followed by the study of suction measurement in Chapter 8.

The development of the hydraulic stress path oedometers with suction control and radial stress measurement is described in Chapter 9. A small amount of development of the stress path triaxial cell for swell measurement is also included in this chapter.

Test data from the stress path oedometer tests are presented and discussed in Chapter 10, and those from the test in the stress path triaxial cell in Chapter 11.

The final chapter draws the pieces a little closer together, and discusses possible directions of further work which may be beneficial. The performance of the stress path oedometer is critically assessed and further modifications and developments recommended.

CHAPTER TWO

REVIEW OF SOIL MECHANICS FOR EXPANSIVE SOILS

2.1 INTRODUCTION

The understanding of the mechanics of saturated soils has made great progress since Terzaghi identified one of the key principles - that of effective stress. The mechanics of unsaturated soils is by contrast very poorly understood. Some of the lack of progress may be due to continuing attempts to derive an effective stress equation for unsaturated soils. None of these attempts has yet provided a workable framework to aid in the understanding of the mechanics of unsaturated soils.

The effective stress principle is viable for saturated soils because two variables, total stress and pore pressure, can be applied to a set of soil parameters, e.g. c' and Ø'. It is important for the viability of the principle that the total stress and pore pressure interact identically with the soil parameters. The soil parameters need not be stress independent, but it must be possible to define them uniquely for a particular stress state and stress history. A similar set of variables and parameters has not yet been found for unsaturated soils, and as a result the mechanics of unsaturated soils is poorly understood.

Within this generally poor state of understanding of the mechanics of unsaturated soils is the narrower field of Expansive Soil Mechanics. In addition to the problems outlined above, a large part of the effort expended in this field has been directed toward the prediction of soil movements or strains from Index Properties.

A general review of expansive soil engineering was done at the beginning of this project and has been published as an Imperial College Report to the Transport and Road Research Laboratory (Schreiner 1987a). It is not intended to duplicate that review, but to address only those aspects which are relevant to this research.

2.2 SOIL MODELS

The role of a model of soil behaviour is to permit prediction of strains due to application of stress or to predict the ultimate strength of a soil element, given the initial conditions of stress, the stress history and, if necessary, the void ratio. For unsaturated soils, decreasing the suction will cause either a

decrease or an increase in the volume, depending on whether the soil collapses or swells. Some definition of the conditions necessary for collapse to occur is being achieved, e.g. Maswoswe (1985), but the model is as yet incomplete for unsaturated soils.

One of the major difficulties encountered with the effective stress concept in unsaturated soils has been the "collapse" phenomenon, where the void ratio decreases with decreasing stress. This led to an extensive discussion in print between Jennings & Burland (Jennings & Burland 1962, Burland 1965) on the one hand and Bishop & Blight (Bishop & Blight 1963, Blight 1965, Blight 1966, Blight 1983) on the other. Although the collapse problem is at first sight in conflict with the effective stress concept, in that decreasing stresses should cause an increase in void ratio and vice versa, similar anomalies exist for saturated soils.

An example is dilation during shearing. In this case the sample can increase in void ratio while the mean effective stress is also increasing, which is the reverse of the collapse phenomenon. Research has identified the conditions necessary for dilation to occur, and has formulated a model which accounts for this behaviour. The origins of this Critical State Model can be traced back as far as Casagrande (1936), but it was not coherently formulated until much later (Schofield & Wroth 1968).

Another example is the residual strength or residual friction angle, which can be lower than the Critical State value at the same effective normal stress (Atkinson & Bransby 1978). This is in conflict with a part of Terzaghi's original principle (as quoted by Skempton (1960)) which states that:

> All the measurable effects of a change in stress such as....a change in shearing resistance are exclusively due to changes in the effective stresses σ_1, σ_2 & σ_3.

In this case the decrease in shearing resistance is not caused by a change in the principal effective stresses. Instead the change in stress state that occurs when the shearing resistance decreases is a consequence of a change in the soil properties.

These difficulties have not led to the rejection of the effective stress concept, but have required extensions of the understanding of soil behaviour and the development of more comprehensive soil models.

2.2.1 SHEAR STRENGTH MODELS

Several "effective stress" equations have been published for unsaturated soils, some of which are listed in Table.2.1. All of these applied a factor to the pore water pressure (which happened to be negative) in order to obtain equations of similar form to

the effective stress equation for saturated soils. The form of the equation that is generally favoured is that of Bishop (1959), although there is no real difference between his and the other equations. Subsequent work, e.g.Burland (1961), Blight (1965) has shown that the factor that is applied to the pore water pressure or suction in these equations is dependent on many variables, such as degree of saturation, test type and stress level (Fig.2.1), and cannot be measured in one test series and then used in other types of test nor in the field. Since there are thus three variables, total stress, pore pressure and the factor 'chi', one of which cannot be readily measured, these equations fall short of the requirements for a physical law.

Reference	Equation
Croney et al (1958)	$\sigma' = \sigma - Bu_w$
Bishop (1959)	$\sigma' = \sigma - u_a + X(u_a - u_w)$
Lambe (1960)	$\sigma' = \sigma a_m + u_a a_a - u_w + R - A$
Aitchison (1961)	$\sigma' = \sigma - Psi\ p''$
Jennings (1961)	$\sigma' = \sigma + Bp''$
Richards (1966)	$\sigma' = \sigma - u_a + X_m(h_m + u_a) + X_s(h_s + u_a)$

TABLE.2.1 Effective Stress Equations

The alternative approach that has been followed has attempted to find equations for strength in which soil strength parameters, $\emptyset'$ and $\emptyset^b$ are determined for the two stresses independently. The frameworks for this approach were set out by Fredlund and Morgenstern (1977), and are an extension of the earlier work where suction and total applied stress were considered separately, e.g. Burland (1961) and Blight (1965). It appears that the soil parameters derived using this approach suffer from the same problem noted above, i.e. they are not stress independent (Fredlund et al 1987, Toll 1988, Escario & Saez 1986). Fig.2.2 shows variation in $\emptyset^b$ with suction from one set of tests.

Although the quantity of data available is small, the form of the shear strength envelope or surface, for unsaturated soils or soils with negative pore water pressure, is becoming visible. It is generally taken to be non-planar when plotted against suction and mean total stress, as shown diagrammatically in Fig.2.3. There do not appear to have been any attempts to apply the principles of modern soil mechanics to the interpretation of this surface.

The study of saturated soils has shown that the stress history can have a major influence on the soil behaviour. This aspect has been ignored in the study of unsaturated soils, although in most cases, the soils have been subjected to suctions and/or applied stresses greater than those at the time of testing. In saturated

soils the failure envelope for this type of stress history is called a Hvorslev line, as shown in Fig.2.4, and there is an infinite family of Hvorslev lines forming the Hvorslev surface corresponding to the infinite range of previous stress histories.

2.2.2 VOLUME CHANGE MODELS

Several researchers have studied swelling of unsaturated soils with control of the suction. Relevant studies are described and discussed in Chapter 4. Most of the volume change testing with suction measurement or control has been performed without minor principal stress measurement. Since the stress state is decreasing when the suction is decreased, the ratio of the applied stresses will not be constant during the tests. This form of testing is thus unable to define either the stress state or the stress path (Schreiner 1987e, Schreiner & Burland 1987), although many authors have referred to the "influence of stress path in swell testing", e.g. Fig.2.5 from Alonso et al (1987). Similarly, testing with minor principal stress measurement but without suction measurement or control cannot define either stress state or stress path.

It has been fairly clearly shown that swelling of an unsaturated soil is caused by decreasing suction, and that the amount of swell is dependent on the magnitude of the suction change (e.g. Yong et al 1971, Escario & Saez 1973, Kassif et al 1973, Komornik et al 1980 and Justo et al 1984).

The early attempts to find a volume change model followed the same path as the shear strength models, i.e. application of a factor to the pore pressure (e.g. Blight 1965). This was unsuccesful largely due to the collapse problem (e.g. Burland 1965), which is intractable in a simple model of this sort. Burland (1965) utilised the three axis plotting of void ratio, vertical total stress and suction to describe the wetting paths for swell and collapse of the same soil (Fig.2.6). This was not developed further at the time, but could be said to be the origin of the later approach followed by Fredlund and his co-workers.

The volume change model developed by Fredlund & Morgenstern (1976) was based on the Biot equations. This is essentially an elastic model extended to include separate moduli for overall volume change due to applied stress and for suction and for water volume change due to suction, (Fig.2.7). This has been further developed by Fredlund et al (1980). They recommend that the modulus for overall volume change due to suction be obtained from the swell pressure followed by rebound test (2.3.1.3 below), by relating the rebound swell to the suction change. This assumes stress path independence, which has not been demonstrated. The attempts by Fredlund & Morgenstern (1976) to demonstrate that a unique surface exists for swelling due to suction and applied stress was inconclusive as their applied stress and suction changes did not cause monotonic volume increase.

A further problem with their approach is that, in the absence of major and minor principle stress data for oedometer tests, the radial stress is eliminated from the equations using elasticity theory to leave the vertical stress as the major stress. This is not acceptable unless the dependence of the radial stress on the axial stress can be determined. Since the stress state is decreasing, the radial stress will tend to increase relative to the vertical stress, and may become the major principle stress, as can be seen in Fig.2.8 for overconsolidated saturated soils at OCR greater than about 3.

A similar approach is described by Picornell & Lytton (1984). They use logarithmic values of stress and suction, which is more in keeping with saturated soil mechanics, and apply different moduli to each stress type. Testing of the equations is done with a case history, but the initial and final suctions are not known. Although the comparison between laboratory and field derived moduli is moderately good, the comparison is meaningless without measured values of initial and final suctions.

2.2.3 SOIL STRUCTURE MODEL

The system of soil structure or fabric terminology which is used in this thesis was described by Collins (1985) and extended by Zein (1985) for compacted soils. Other systems may be equally useful, but this one has been chosen in part because Zein (1985) also studied the structure of compacted expansive soil and used this system to describe the structures which he observed with a scanning electron microscope. His observations are used in Chapter 10 where the data from the Stress Path Oedometers is discussed.

The basic system is described in Fig.2.9. Zein (1985) included perturbed parallel, shown in Fig.2.10b from Collins (1984), in the elementary level for clays. This structure appears to be the same as the turbostratic structure defined by Aylemore & Quirk (1960) and shown in Fig.2.10a from Collins (1983). The former is described as "clay flakes in face-face and edge-edge association but which overall is dense and intensely folded or crumpled". The latter is described as "turbulent array of parallel oriented clay platelets".

Zein (1985) found that the structure in a compacted highly plastic soil varied with the compaction moisture content. The structure at a moisture content near optimum moisture content was almost entirely perturbed parallel or turbostratic. As the compaction moisture content was decreased, the incidence of clustered and aggregated structure increased. Compaction wet of optimum moisture content tended toward parallel orientation at the elementary level as the compaction moisture content was increased.

Fig.2.11a from Collins (1983) shows an aggregated structure in a sample of silty clay which was compacted "dry of optimum". The perturbed parallel structure of a sample compacted "wet of

optimum" moisture content is shown in Fig2.11b. No details are given of the actual moisture contents or densities of the samples.

2.3 TESTING PROCEDURES FOR VOLUME CHANGE

Geotechnical testing of unsaturated soils for volume increase due to intake of water can be conveniently divided into two categories. In the first are all those tests which permit the intake of water without control of the suction or water content. In the second are those tests which control the intake of water either by suction or by water content.

Tests of the former type are generally quick, inexpensive and simple to perform. They suffer from the disadvantage, from the soil mechanics research point of view, that they can only provide data on the end point of the wetting process. From the foundation point of view, they are less than perfect in that they consider the worst case of wetting to zero suction or pore pressure and may not follow a stress path relevant to the field conditions. A considerable body of data has, however, been generated using this type of test, and several standard methods have been published.

Tests in the latter category are usually slow, expensive and complex both in equipment and technique. They can however provide information on the changes in volume and stress that occur in the soil during the wetting process, and thus on the conditions that might prevail in the field where non-zero suctions may persist through the life of the project.

In both categories there has been a tendency to measure or control fewer stresses than are normally considered necessary to define the stress state of a soil element. As with saturated soils, it is necessary in an unsaturated soil to know the principal total stresses and the pore pressure or suction. It is perhaps more important to know both vertical and horizontal stresses in this type of problem, as the horizontal to vertical total stress ratio, K, will not be constant throughout the test (Schreiner 1987e, Schreiner & Burland 1987).

Almost all the testing has been performed in oedometers, and only two of the research groups have measured lateral stresses in the oedometer (Ofer 1980, Komornik & Zeitlen 1965). A small number of tests have been performed in triaxial type equipment, e.g. Dakshanamurthy (1979) and Tisot & Aboushook (1983). As a result of the widespread failure to measure the full stress state, the state of knowlege of the mechanics of swelling of unsaturated soils has made little progress.

2.3.1 STANDARD OEDOMETER TEST PROCEDURES

Although the standard oedometer cannot measure the complete stress state in the soil, the data obtained in the past have led

to some conclusions which may be of doubtful validity, but are widely accepted. The testing has, however, often been directed at obtaining information for use on one particular project, and this has resulted in a rather fragmentary picture.

In general, researchers have followed the "standard" test procedures. Three significantly different procedures for volume change or swell have been identified, although there may be minor variations in use. As far as is known to the author, these methods are used indiscriminately, and the results are considered to be comparable (e.g. Ajayi 1987) although it has been shown that for compacted soils different amounts of swell are measured (Brackley 1975). These volume change procedures can be used to measure the swell pressure, and have been shown to give different results (Ali & Elturabi 1984). The standard test procedures are described below and represented in Fig.2.12.

2.3.1.1 Swell followed by Consolidation

The first test of this type was described by Jennings & Knight (1957), primarily for use in the study of collapse of unsatutated soils. It has been widely used in expansive soil testing, and has been modified over the years, most recently by Jennings et al (1973).

Details of the test procedure are available in the literature. In essence, the soil is placed in the oedometer at the field or compaction moisture content under a relatively small vertical total stress. Water is then supplied and the volume change monitored. After swelling, AB in Fig.2.12(a), the vertical stress is increased incrementally, and a standard consolidation procedure followed. Swelling under any vertical stress is estimated from the consolidation curve BC and either the initial void ratio or a consolidation curve AD at the natural or compaction moisture content.

One of the faults of this test is that the sequence of stress changes applied to the soil does not in any way resemble the stress changes that are normally encountered in practice. It also utilises soaking to nominally zero suction, frequently mis-described as saturation. (Saturation as applied to soil mechanics is unrelated to the state of stress in the soil.) The suction in the field may not reach zero, so that the results from these tests should be considered to be upper bound values, at least on this count.

2.3.1.2 Swell under expected vertical stress

This test type is described by Holtz & Gibbs (1956). It is performed in a similar manner to the test described above, except that vertical total stress is applied, EF in Fig.2.12(b), before swelling, FG. The vertical stress is usually equal to the expected

field stress, either the overburden stress or the overburden plus foundation stress. The consolidation test after the swelling stage, FG, is generally omitted.

The stress change sequence followed in this test is much more representative of the field stress changes than the previous test type. Criticism of the stress state during the test can be made on two counts. Firstly, there is no means of including the effects of cracks or fissures in the field profile. Secondly, the stress changes during sampling may affect the stress state. These are both of minor importance compared with the criticism of the previous test procedure. The comments in section 2.3.1.1 on the soaking to zero suction are equally applicable to this test.

2.3.1.3 Swell pressure followed by rebound

This form of test was recommended by Sullivan & McClelland (1969). In essence, the sample is placed in the oedometer at its natural or compaction moisture content, and water is supplied. Vertical straining is prevented by increasing the vertical total stress, HI in Fig.2.12(c). After wetting up is complete, the vertical stress is decreased in steps, IJ, and the swell under any vertical stress is estimated from the rebound curve and the initial void ratio. The implicit assumption in this procedure is that the effective stress during the constant volume stage does not change and that the subsequent decreasing vertical stress is equivalent to the decrease in suction that will occur in the field. The equivalence of the two stress paths has been adressed above, in section 2.2.2. The comments in section 2.3.1.1 on the soaking to zero suction apply to this test procedure as well.

2.3.1.4 Swell pressure

The swell pressure is determined from the first test procedure as the vertical stress during consolidation at which the void ratio becomes equal to the initial void ratio (Chen 1973). The swell pressure in the third procedure is the vertical stress at the end of the constant volume stage (Holtz & Gibbs 1956). The stress paths and final stress states for these forms of swell pressure test are not likely to be similar. A considerable amount of effort has been expended in attempting to improve the measurement of swell pressure by the constant volume technique.

The original procedure relied on addition of weights while monitoring the dial gauge in a standard oedometer rig. Subsequently Brackley (1975) developed an apparatus which has a stiff load cell to measure the vertical load. Agarwal & Sharma (1973) used an electrical relay system in a motorised compression frame. Richards et al (1984) developed an hydraulic oedometer which is claimed to be capable of maintaining the vertical movement within acceptably small limits. Whilst each apparatus is

an achievement in mechanical complexity, little progress has been made in the understanding of the mechanics of unsaturated soils as a result of the increased precision with which swell pressure can be measured.

2.3.2 NON-STANDARD TESTS AND APPARATUS

Of the non-standardised test apparatuses, perhaps the most important are those that permit the measurement or control of suction. The first such oedometer was reported by Rengmark & Eriksson (1953), although there is no indication that they intended it for use with unsaturated soils. Subsequent designs have used both the pressure plate or elevated pore air pressure technique (Barden et al 1969, Chang 1969, Escario 1969, Aitchison & Martin 1973 and Richards et al 1984) and osmotic suction control (Kassif & Ben Shalom 1971a&b). These are discussed in more detail in Chapter 9 in relation to the design of the equipment used in this research.

Tests that permit measurement or control of "triaxial" stresses are also of interest to the current project. Some valuable work has been done in triaxial cells, e.g. Dakshanamurthy (1979) for swelling and Tisot & Aboushook (1983) and Maswoswe (1985) for compression or collapse. Equipment for radial stress measurement under conditions of nominally zero radial strain has been developed by Komornik (1962) and Ofer (1980). The results from these non-standard tests are discussed in Chapter 4.

A potentially useful, but simple, test is the controlled water addition oedometer test. This was developed at Imperial College by the author with a visiting researcher for comparison of field swell and moisture content data with laboratory tests. The tests in which soaking to zero suction had been permitted showed greater swell than had been measured in the field. The field moisture content data that were available after wetting and from a dry part of the site were used to determine the amount of water that should be added in the controlled water addition tests. This quantity was then added slowly by means of a narrow filter paper wick (Elsharief 1987). The measured swell in these tests matched the field data better than did the data from the soaked tests (Osman & Elsharief 1987), as shown in Fig.2.13.

Unconfined or partially unconfined tests have been used, (e.g. Parcher & Liu 1965, Beal 1984 and Pile & McInnes 1984). These are of little value for research.

2.4 SAMPLING EFFECTS

No serious consideration has been given to the effects of sampling of unsaturated soils. It appears that the techniques used for obtaining "undisturbed" samples and the preparation of oedometer and triaxial samples are essentially the same as those

appropriate for saturated soils. However, these rely on the assumption that the state of stress is not changed significantly during the sampling, on the grounds that, because the sample is saturated, the negative pore water pressure generated by the total stress relief prevents volume change.

The effect of sampling on residual soils has been described by Vaughan et al (1988), and is relevant to unsaturated soils whether residual or not. They state:-

> "Sampling involves relief of total stress.....Thus...the in situ mean effective stress cannot be retained in the sample by capillarity, as is possible in saturated sedimentary clays. Inevitably there is a large reduction in mean effective stress accompanying sampling, even when the latter is of high quality. This must allow expansion....."

In most cases, therefore, the samples that have been tested have been subjected to stress relief. In practice, this is only partially relevant, as "undisturbed" samples are not often used because of the difficulties involved in both field and laboratory in working with stiff but brittle and frequently cracked soils. A large proportion of the testing is thus performed on compacted or remoulded samples. Even for these samples there is stress relief between compaction and preparation of oedometer or triaxial samples. This has previously been adressed by only two authors, Maswoswe (1985) and Didier (1973) who compacted their samples directly into the sample ring. This technique will reduce the stress relief, but it is still necessary to remove the vertical stress after compaction.

2.5 EMPIRICAL CORRELATIONS FOR POTENTIAL SWELL

Index tests are widely used by geotechnical engineers to obtain an indication of the type of soil and thus a rough guide to the type of behaviour that may be expected from that soil. In certain cases, such as normally consolidated sedimentary soils, fairly reliable correlations have been derived, for example between liquidity index and undrained shear strength. In most cases the link between the index properties and the soil behaviour is less direct. In saturated sedimentary soils this is largely due to the stress history, but may also be due to bonding, soil structure, changes in salinity etc. When the soil is unsaturated, the link will become even more tenuous.

The development of a large number of empirical correlations between swell and various soil properties is probably the best indication of a lack of understanding of the basic processes involved in the swelling of expansive soils. The mechanisms of interaction between water and clay particle surfaces are described in Chapter 3, where it is demonstrated that the two most important

components in volume change are the clay mineral and the moistue or suction change. The clay minerals are largely responsible for determining the intrinsic expansiveness, while the moisture content or suction change controls the magnitude of the volume change that a particular soil will undergo (Schreiner 1987b).

Many empirical correlations have been made between routinely determined soil parameters, such as Atterberg Limits, and magnitudes of swell from case histories or laboratory swell tests. This approach usually fails to distinguish between the two major components involved, and frequently attempts to correlate the Atterberg Limits with the observed strain or swell. The former should reflect the the intrinsic expansiveness, while the latter results from the suction and stress change interacting with the intrinsic expansiveness. However, a significant degree of success has been achieved with some of these methods, and the limitations within which these correlations may be of some value have been outlined by Schreiner (1987b). One example will be used to demonstrate the lack of understanding frequently displayed.

Snethen (1984) compared predictions from 17 methods with his data from oedometer swell tests on undisturbed samples under vertical total stresses equal to the calculated overburden stresses. Wetting up was from the measured initial suction to zero suction. Many of the methods which he used in this comparison were derived for compacted samples wet up from optimum moisture content under some "standard" vertical total stress, and it is therefore not surprising that he found that most methods gave poor predictions. The "best" methods did not predict correctly in more than 50% of the cases. This may be largely due the predictive methods being applied in his review with no regard to soil mechanics. The predictive method derived by Snethen (1984) is flawed in the following ways:-

1. There is no equivalence of samples
2. There is no inclusion of stress history
3. There is no inclusion of initial void ratio
4. There is no inclusion of applied total stress during the test
5. There is no inclusion of moisture content or suction change
6. The predictive table does not include all possible combinations of soil plasticity and suction

2.6 TERMINOLOGY

There is as yet no accepted set of terms for use in expansive soil mechanics. It is thus necessary to clarify the meaning of some of the terms used in this thesis.

2.6.1 INTRINSIC EXPANSIVENESS

Volume change of a clay soil due to changes in water content or suction is caused by interaction between the clay minerals and water. If the suction exerted on water in a soil is increased, water will move out of the soil under this stress and the soil will decrease in volume. Conversely, if water is added to the system, the suction will decrease and the soil will increase in volume. The intrinsic expansiveness is the soil property that relates the change in water content, and thus change in volume, to the suction change. A soil with a high intrinsic expansiveness will exhibit a large water content or volume change as compared with a soil of low intrinsic expansiveness when both are subjected to the same change of suction under identical initial stress, suction, soil structure and stress history conditions.

Although this concept is not new, but was mentioned by Brink (1955) and was used by Baikoff and Burke (1965), it has not previously been clearly described. No standard procedure has yet been developed for the measurement of this property, and it is only useable at present as a qualitative description.

2.6.2 SWELL

Swell is a measure of the volume strain or axial strain in a soil under a particular set of stress and suction conditions. It can occur either due to changes in suction or due to changes in applied load. In laboratory testing of intrinsically expansive soils, swell is usually induced by a reduction of suction under a small but constant vertical applied stress and zero radial strain in an oedometer. Most testing is performed with an unknown initial suction, a final matrix suction close to zero, and unknown radial stress.

Swell is not a soil property, but is a strain which results from interaction between suction and stress and the soil property, intrinsic expansiveness, described above. Swell is thus specific to the stress and suction conditions under which it is measured, and will probably also be affected by suction and total stress history.

2.6.3 HEAVE

Heave is the displacement of a point in the soil profile due to suction and stress changes interacting with the intrinsic expansiveness. The relevance of laboratory swell measurements to heave under field conditions will depend on how well we can model, in the laboratory, the initial stress and suction states in the field and their changes.

Heave is not a soil property, but is a displacement which results from interaction between the soil property (intrinsic expansiveness) and the field stress and suction changes that we

may or may not be able to quantify with a reasonable degree of confidence.

2.6.4 SUCTION COMPONENTS

The following descriptions were introduced into soil mechanics at the First Expansive Soils Conference by the Review Panel (1965).

Total Suction is the negative gauge pressure relative to the external gas pressure on the soil water to which a pool of pure water must be subjected to in order to be in equilibrium through a semi-permeable membrane.

Matrix Suction is the negative gauge pressure on the soil water to which a solution identical in composition to the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water.

Osmotic (solute) Suction is the negative gauge pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semi-permeable membrane with a pool containing a solution identical in composition to the soil water.

2.6.5 STRESS VARIABLES

In the past it has been the practice to use the stress variables $(\sigma-u_a)$ and (u_a-u_w) to describe the stress state in the soil. Although this is technically admissable, it serves to mask the stresses which are of interest in the natural state. It is similar to describing the stress state in a saturated soil by the total stress and the pore pressure. This is not commonly done because it has been shown that the stress variable which affects the soil behaviour is the difference between the total and the pore stresses, i.e the externally applied stress referenced to an atmospheric pore pressure.

The same can be done with the stress state for unsaturated soils, using the results of investigations into the stress variables of importance. The technique for testing unsaturated soils with negative pore water pressures is to raise the absolute pore water pressure to above atmospheric pressure by applying an air pressure within the sample and using a saturated fine pored porous barrier to measure the pore water pressure. It has been shown (Blight 1961, Bishop & Donald 1961) that the absolute pressures are not significant. The pore air pressure is thus an artificial and undesirable part of the stress variables. In this thesis the stress variables used are:-

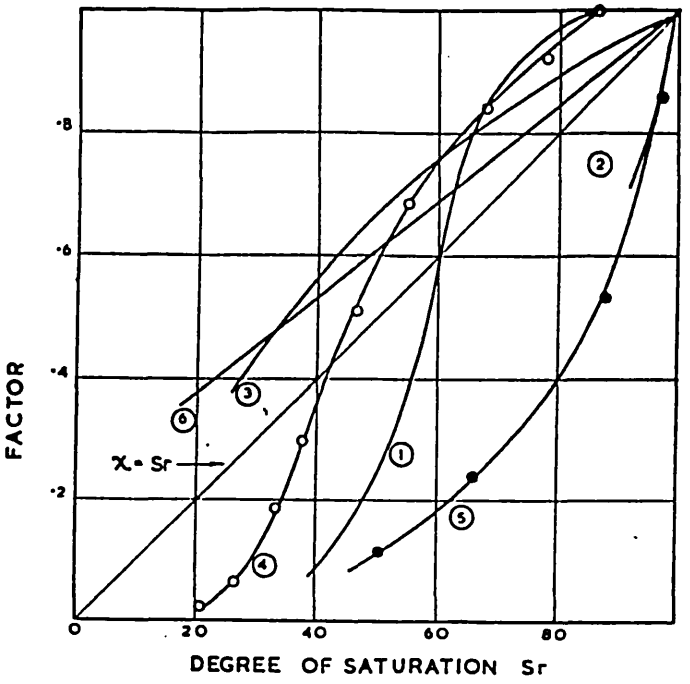
σ_{av} : the vertical or axial total stress referenced to atmospheric pore air pressure and called the vertical or axial applied stress

σ_{ar} : the horizontal or radial total stress referenced to atmospheric pore air pressure and called the radial or horizontal applied stress

u_{aw} : the pore water pressure referenced to atmospheric pore air pressure and is numerically equal to the suction but of opposite sign

When the pore air pressure is equal to atmospheric pressure, the applied streses are calculated for a pore water presure equal to atmospheric pressure, and are thus equal to the effective stresses.

Not only have these been shown to be the stress variables of importance, but they also reflect the stresses that exist in the field in most instances. The only situation in which these stress variables may be unsatisfactory is when there are bubbles of air within the soil water in which the air pressure is significantly different from atmospheric pressure. Burland (1961) has shown that such bubbles must be transient, because the high pressure air in the bubbles will in time diffuse through the water to the low pressure air of the free atmosphere. The use of these variables may help to clarify the relation between the field and the laboratory tests.



1 COMPACTED BOULDER CLAY (-2μ·4%)] BISHOP
2 COMPACTED SHALE (-2μ·22%)] ETAL(1960)
3 BREAHEAD SILT. BISHOP AND DONALD (1961)
4 SILT (-2μ·3%)
5 SILTY CLAY (-2μ = 23%)
6 THEORETICAL DONALD (1960)

Fig.2.1a Variation of "chi" with soil type and test conditions (Jennings & Burland 1962)

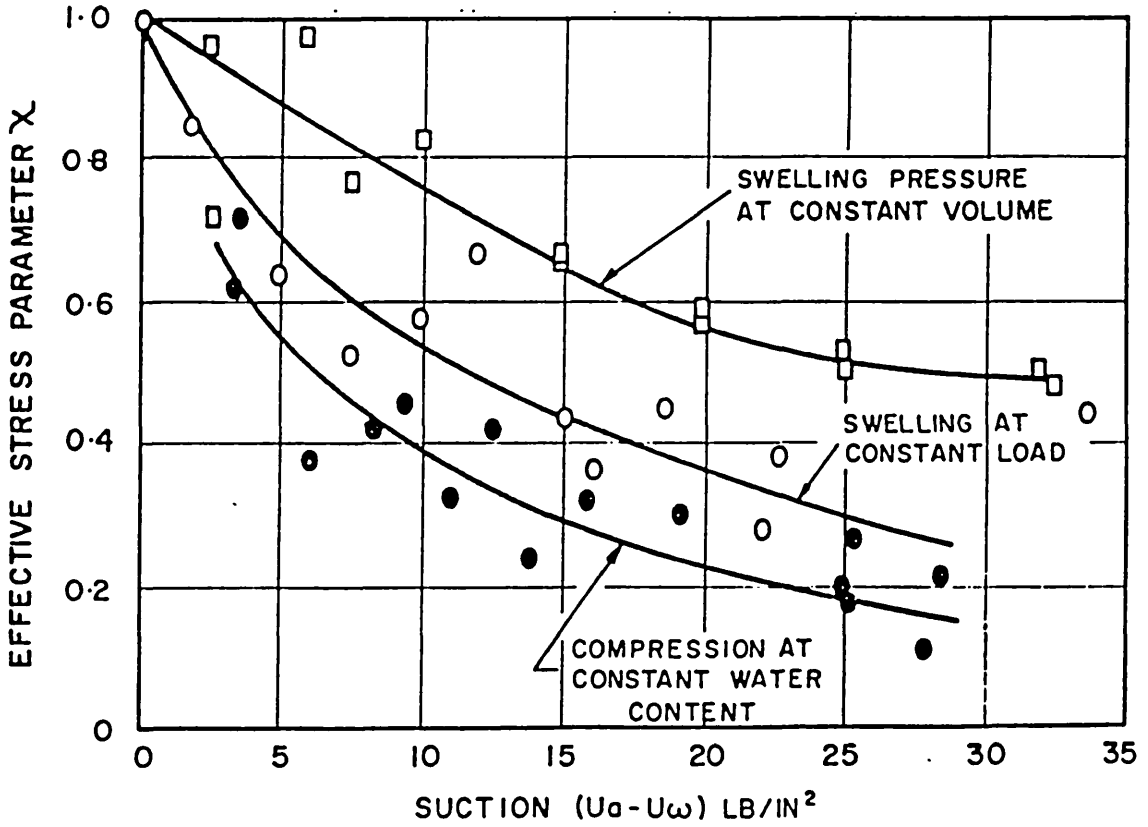
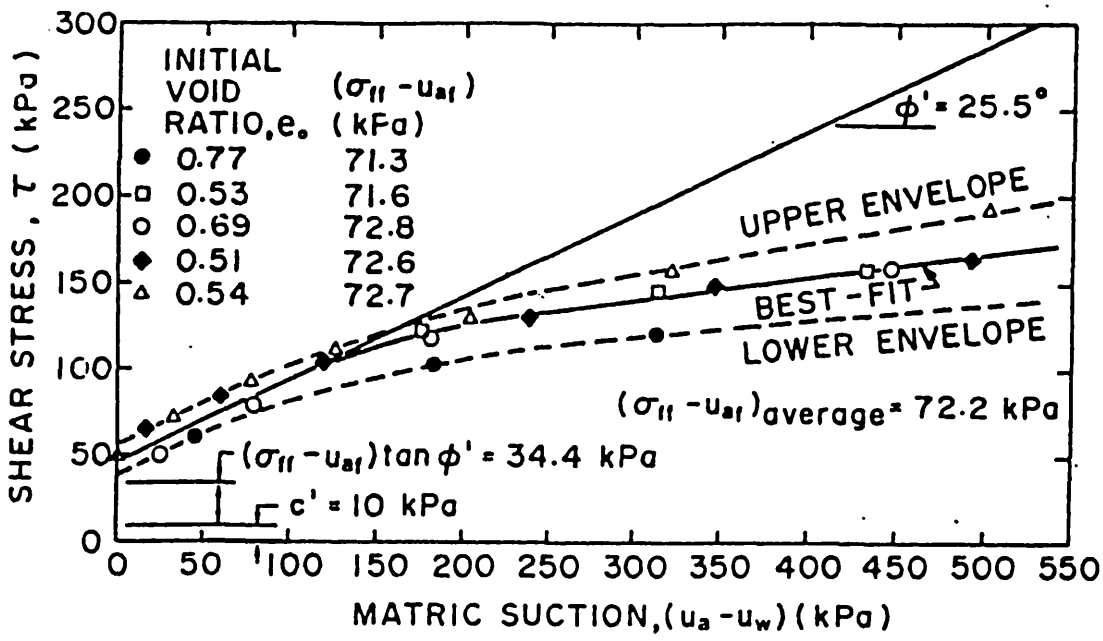
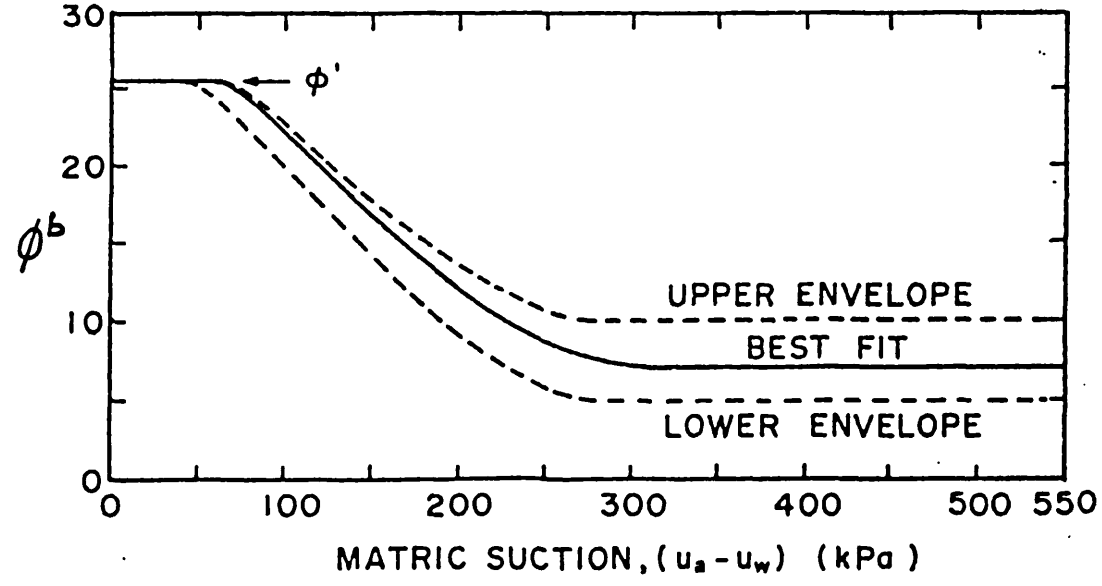


Fig.2.1b Variation of "chi" with test conditions and suction (Blight 1965)



Curved failure envelopes



ϕ^b values

Fig.2.2 Variation of ϕ^b with suction
(Fredlund et al 1987)

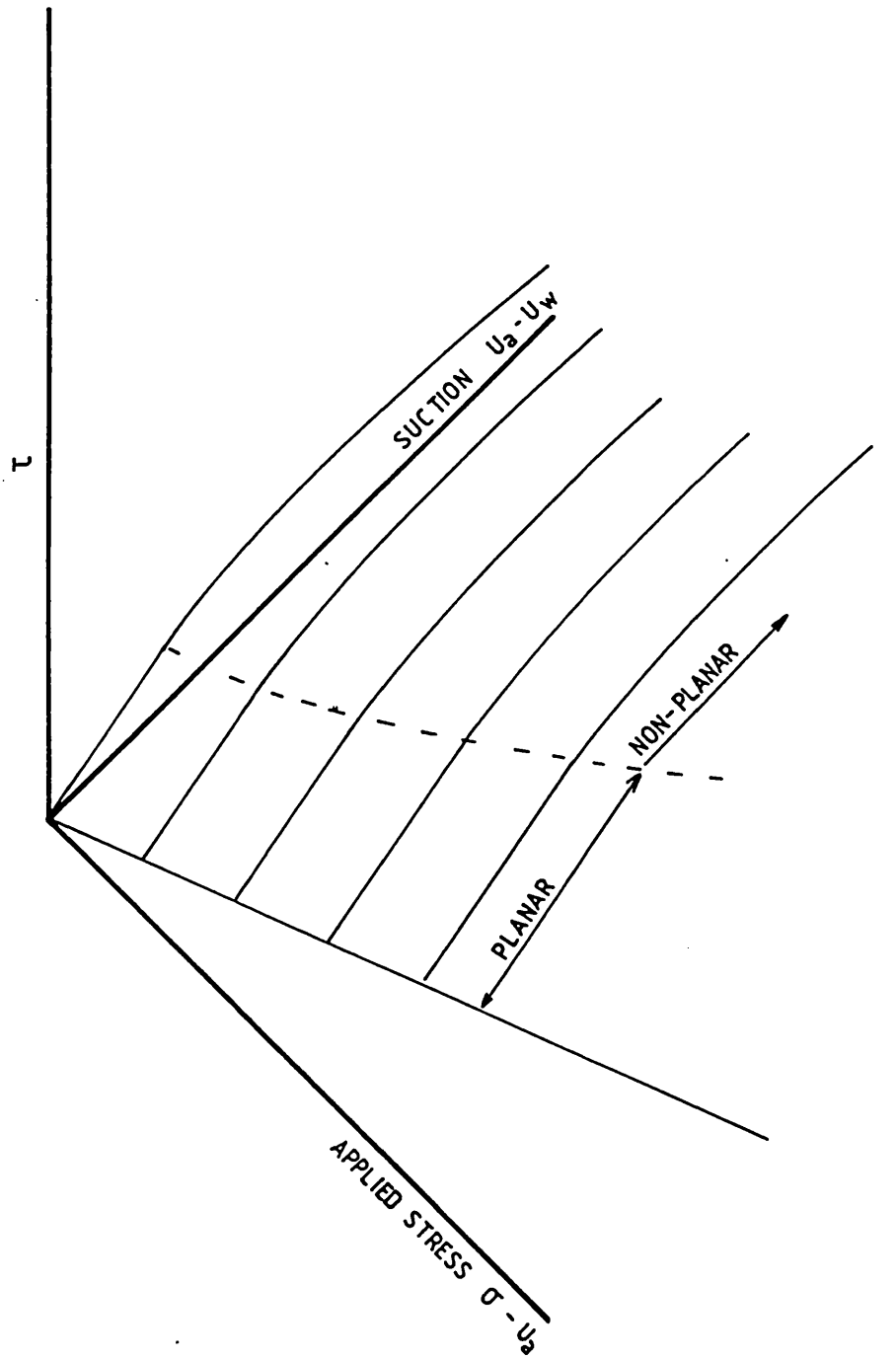


Fig.2.3 Diagrammatic representation of Failure Surface

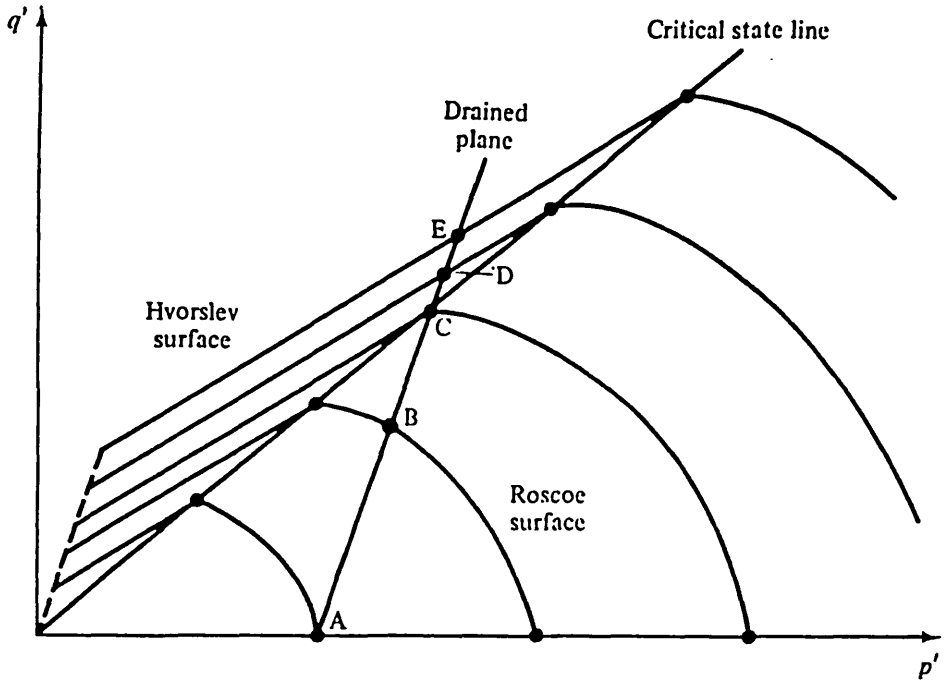


Fig.2.4 Hvorslev lines for overconsolidated soil
(Atkinson & Bransby 1978)

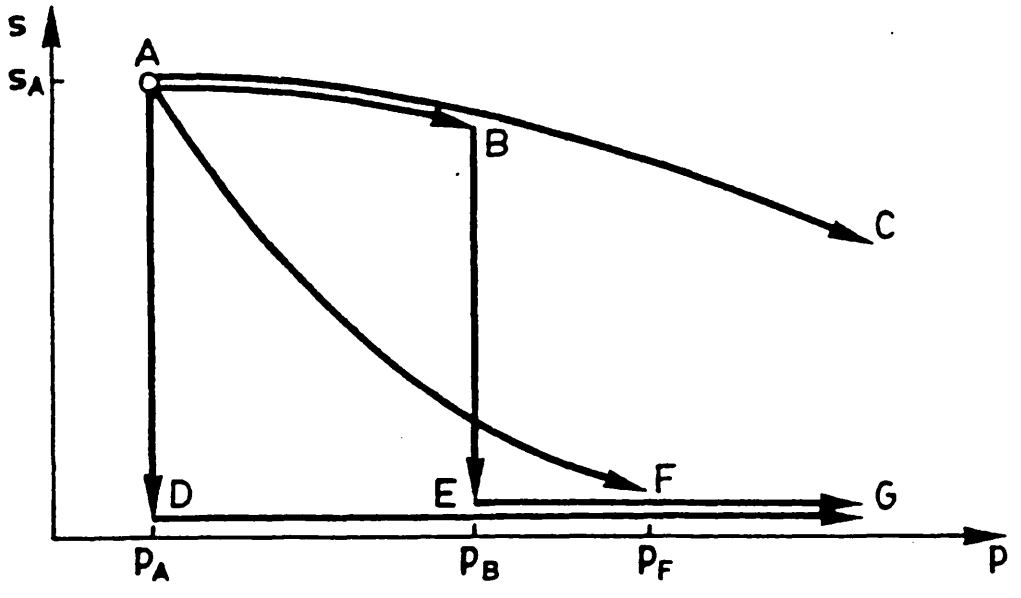


Fig.7. Typical stress paths in the (p,s) stress space.

Fig.2.5 Mis-use of terminology
(Alonso et al 1987)

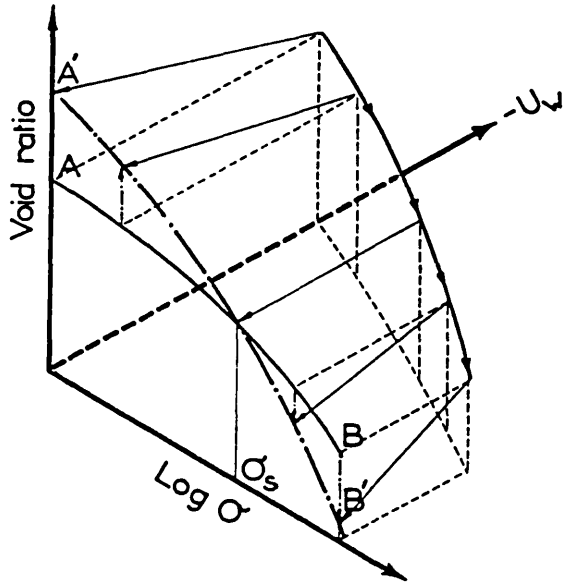


Fig.2.6 Three axis representation of stress state and void ratio
(Burland 1965)

m_1 = modulus for applied stress
m_2 = modulus for suction

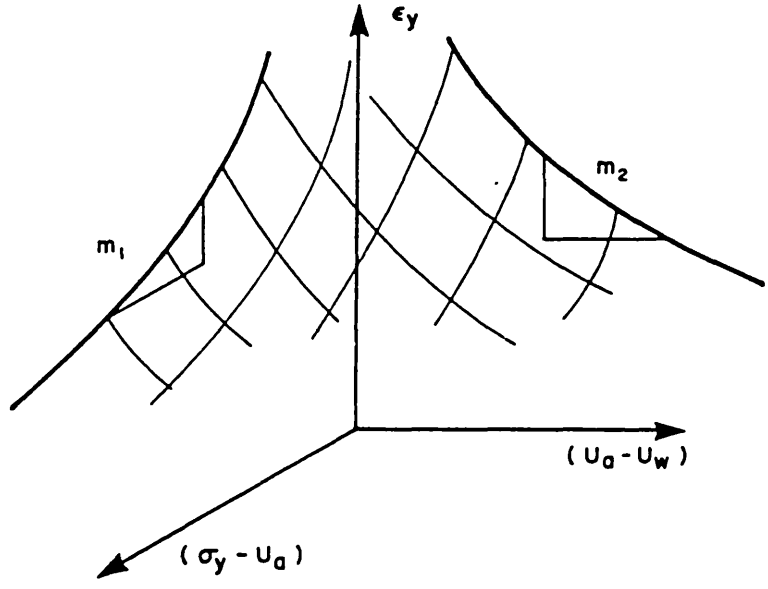


Fig.2.7 Soil moduli for applied stress and suction
(Fredlund & Morgenstern 1976)

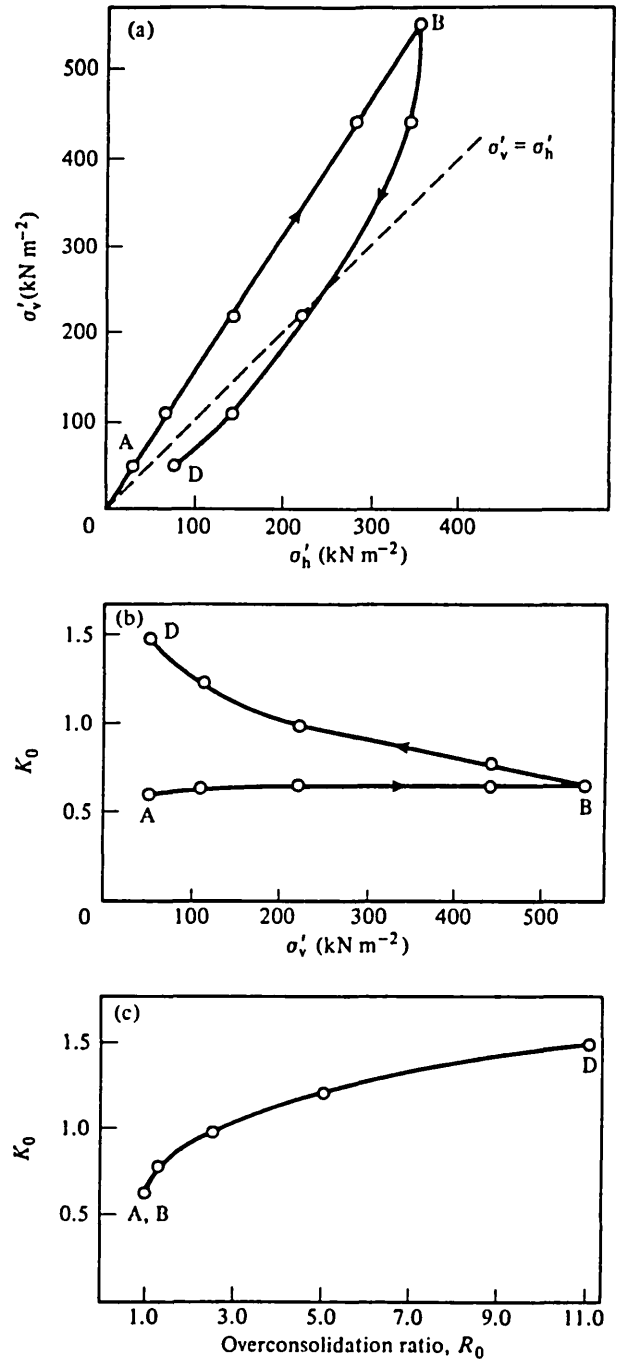
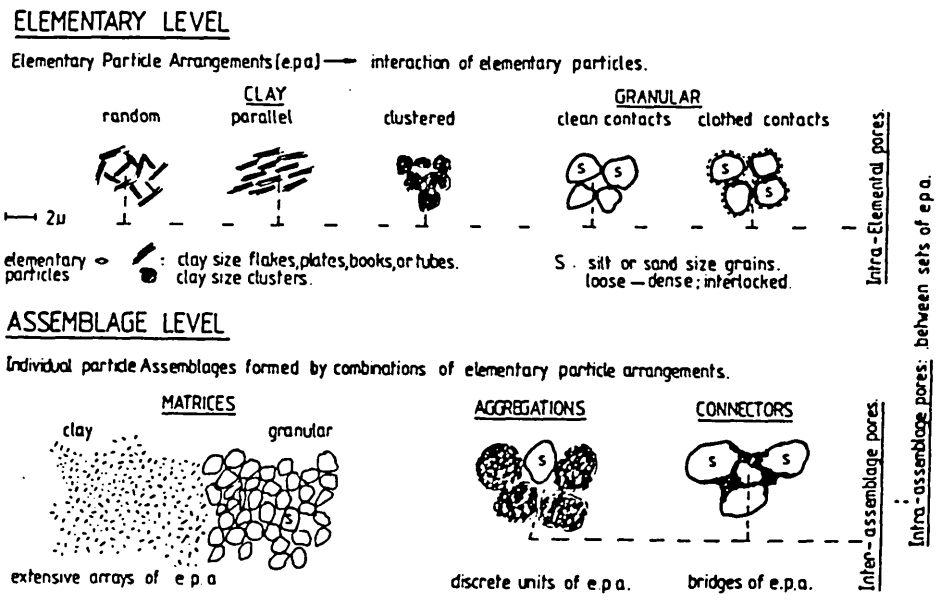
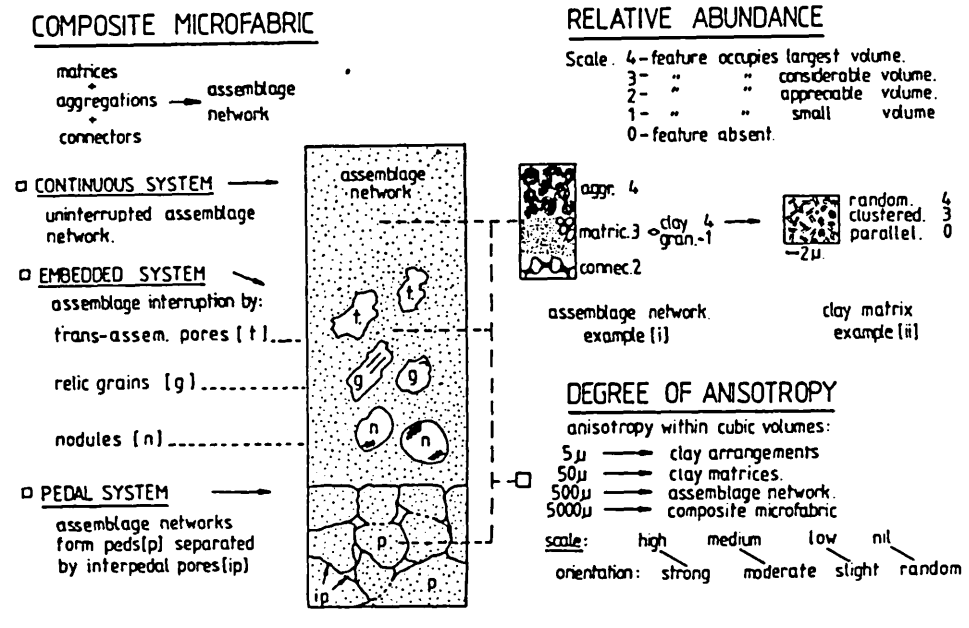


Fig.2.8 Variation in axial and radial stress with overconsolidation

(Atkinson & Bransby 1978)

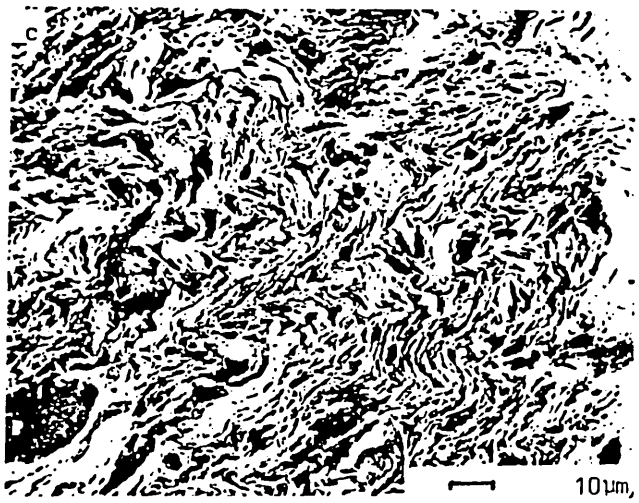


Elementary and Assemblage Levels of Fabric Organisation

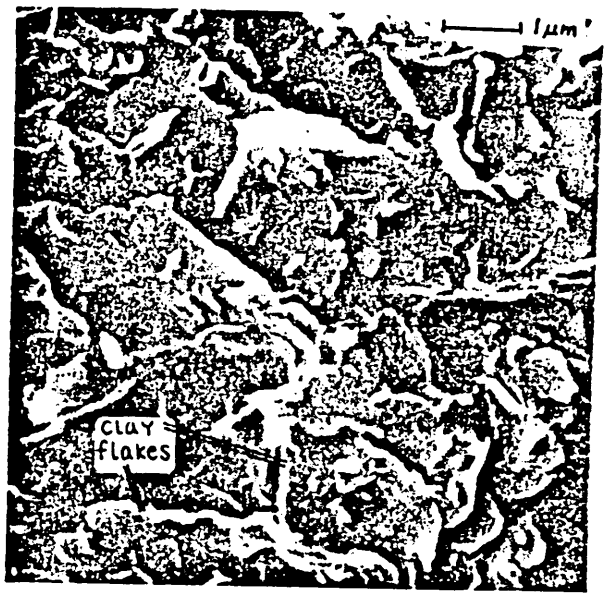


Composite Level of Fabric Organisation

Fig.2.9 Description of Soil Fabric



(a) Turbostratic soil structure of undisturbed clay
(Collins 1983)



(b) Perturbed parallelism structure
(Collins 1984)

Fig.2.10 Soil structure

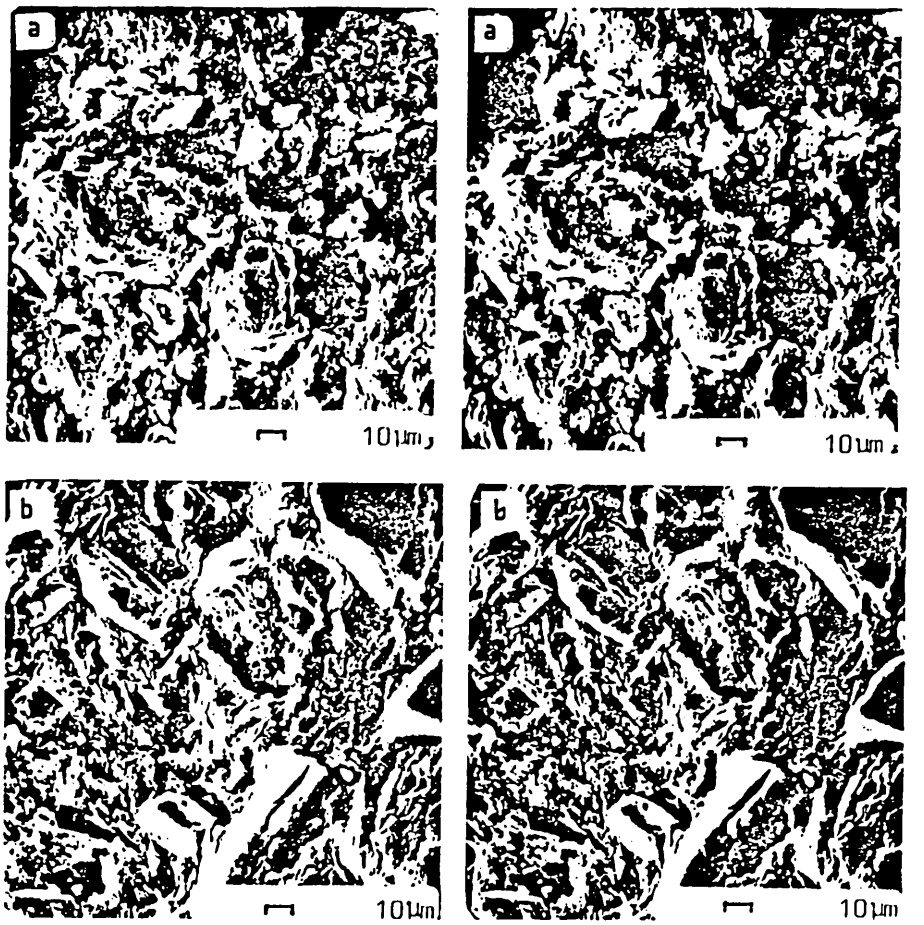


Fig.2.11 (a) SEM stereopair of Luanda silty clay, dry of optimum; (b) SEM stereopair of Luanda silty clay, wet of optimum.

(Collins 1983)

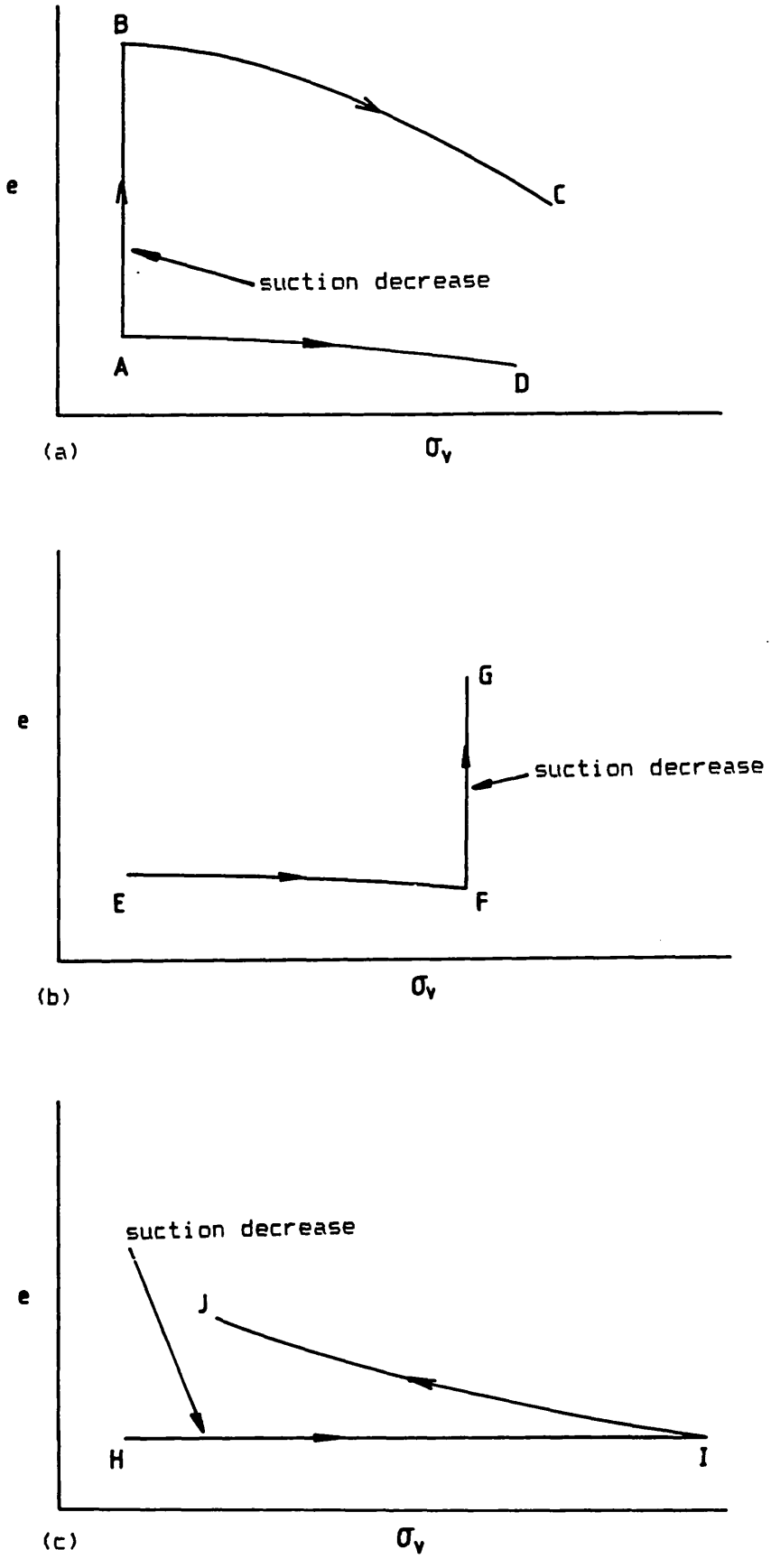


Fig.2.12 Standard test procedures

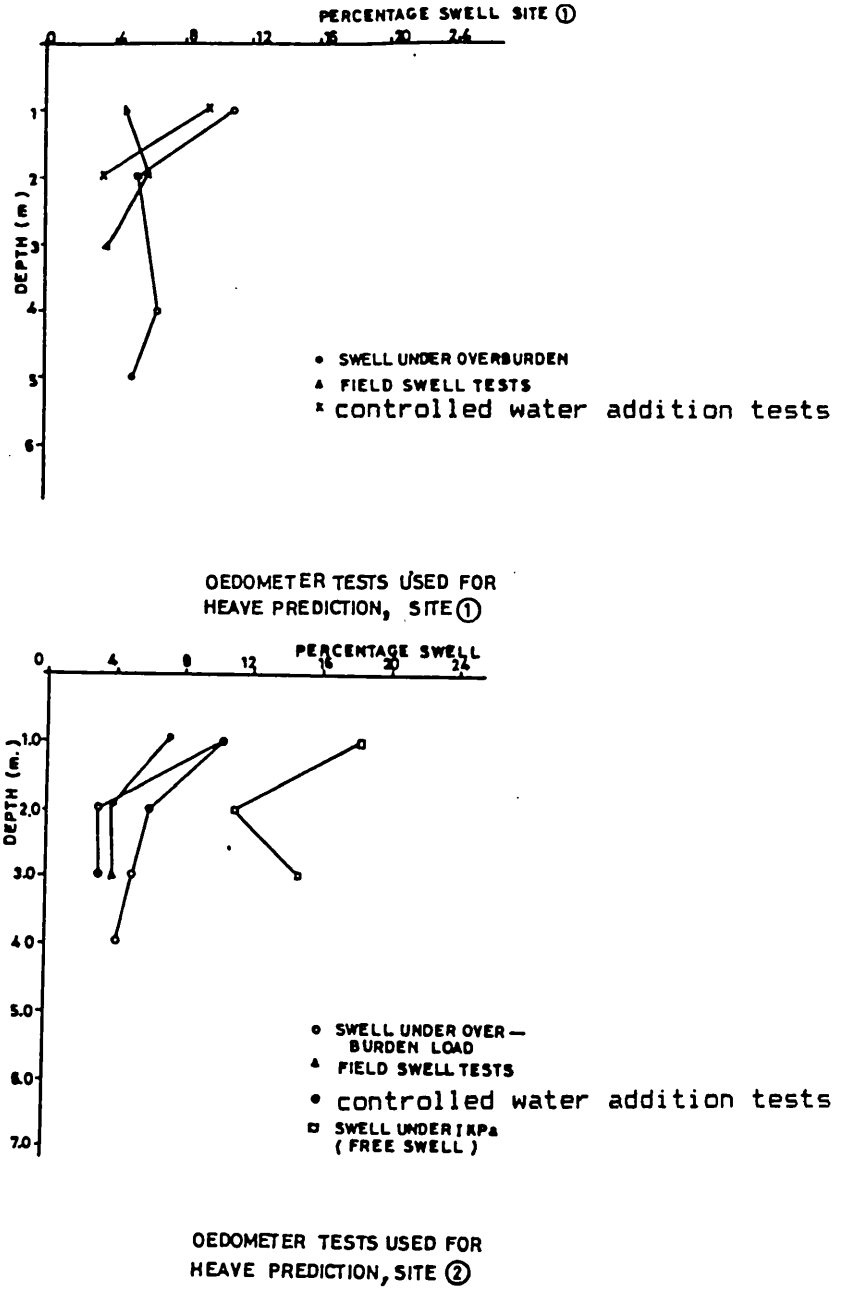


Fig.2.13 Comparison between soaked and controlled water addition tests and field data

(Osman & Elsharief 1987)

CHAPTER 3

MINERAL STRUCTURE AND SOIL WATER CHEMISTRY

3.1 INTRODUCTION

Sands and silts exhibit markedly different engineering properties from those of clay soils. The primary cause of this difference is the interaction between the clay mineral particles and water. Sands and silts appear to be relatively inert with regard to water, while clays have been shown to interact strongly with water.

It is beyond the scope of this thesis to present a detailed treatise on the chemical interpretations of the interaction, but a brief review may help to explain how the interaction influences clay behaviour.

3.2 MINERAL STRUCTURE

The crystal structures of the various clay minerals are extensively documented (e.g. Bragg & Claringbull 1965, Lambe & Whitman 1969 and Taylor & Cripps 1984). However, there is still some uncertainty regarding the finer details of the crystal structure, particularly with regard to the exchangeable cations (Brindley & Brown 1980, Moore 1970).

Most clay mineral particles are formed from alternating layers of basic units or sheets, as shown in Fig.3.1. In montmorillonite, a single particle may be made up of only three such sheets, and consists of a Gibbsite unit between two Silicate units. Most other clay mineral particles consist of repeating sequences of two or three structural units. A Kaolin particle, for example, may have more than 100 sequences of silicate and gibbsite, chemically bonded to form a single crystal structure up to 1000A thick. The montmorillonite particle by comparison may be as little as 10A thick.

Characteristic of all clay minerals is the presence of at least one silicate layer with the oxygen atoms exposed at the surface of the crystal structure.

This surface structure is found in many materials that adsorb gases onto their surfaces (Gregg 1933, Alexander & Johnson 1950, Schreiner 1951 and Schreiner 1977). In the case of clay minerals it is usually water that is adsorbed onto the mineral surface. Up to four mono-molecular layers of water may be adsorbed (Bragg & Claringbull 1965). The thickness of the fully developed, four

molecule thick layer is about 10A or $10*10^{-10}$m (Alexander & Johnson 1950 and Lambe 1958).

It appears that the variation in thickness of this layer is responsible for the basal spacing changes observed in X-ray diffraction analyses. Moore (1970) and Brindley & Brown (1980) have presented data relating the basal spacing to the relative humidity during X-ray diffraction which show a stepwise increase in the basal spacing with increasing humidity. The increases in the basal spacing are approximately equal to the length of a water molecule.

There is some disagreement in the literature regarding the position in the structure in which the water is adsorbed, and thus regarding the physical interpretation of basal spacing. Some authors suggest that the water is adsorbed between basic structural units, Fig.3.2a, but it seems more reasonable to consider this water to be adsorbed on the exposed surfaces and to take the basic structural units to be discrete particles, Fig.3.2b. If the water is absorbed between the basic units with absorbed water only found on the outer faces, then the maximum void ratio could approach 3 for parallel orientation. This is not consistent with observed compression data for bentonites where a void ratio of 3 is reached under about 450kPa vertical effective stress (Schreiner 1977).

The other significant characteristic of most clay minerals is the existence of substitutions of one metal ion (cation) for another within the basic crystal framework. Where the cation in the framework has a lower valency than the cation it has replaced, then, to complete the chemical structure and to provide overall electrical neutrality, another cation is required.

Under normal circumstances, this cation, or the many that are associated with each solid particle, is found in hydrated form within the double layer (Grim 1962, Jones 1963, Alexander & Johnson 1950). In the totally dehydrated or anhydrous state, these exchangeable cations may be a part of the crystal structure (Wells 1962), but, on addition of water, ionisation will take place (Resendiz 1965) and the exchangeable cations may separate from the solid particle and move into the liquid phase as hydrated cations.

3.3 CLAY-WATER INTERACTION

The interaction between soil particles and water is a complex subject, and there are several different physical and chemical models which are used to describe the interaction. Three apparently different models, the capillary model , the double layer model and the osmotic model, have come to the writer's attention. These approach the interaction from significantly different initial suppositions, but all manage to explain some of the behaviour of some soils.

The model that appears to have been used first in soil

mechanics is the capillary model, although the double layer or particle energy model was published by Gouy in 1910. An osmotic model based on the Gouy double layer theory was introduced by Bolt in 1956.

3.3.1 CAPILLARY MODEL

Most of the early soil mechanics theories for partially saturated soils were based on the capillary model of suction. In this model, the force preventing separation of the particles is caused by the surface tension of the water. It is assumed that the water forms a meniscus between the particles, in the same manner in which it does in a capillary tube. The force exerted on the particle is dependent on the radius of curvature, or more correctly for soils on the mean radius for non-circular pores or tubes, and the wetting angle between the water and the solid surface. The stress resulting from the surface tension, for a wetting angle of zero, is calculated from the equation :-

$$U_w = 2T/r$$

where

U_w = suction in the same units as T/r

T = surface tension of water, e.g. $72*10^{-6}$kN/m

r = capillary radius, e.g. m

Capillary suction of this form is dependent on the surface of the particle being wettable by water. This is a reasonable assumption to make for sands and silts which have inert surfaces, and no exchangeable cations.

For clay soils which have chemically active surfaces and exchangeable cations, a different model is needed.

3.3.2 OSMOTIC MODEL

The four molecule thick layer described above is referred to as adsorbed water. There is a much greater thickness of less strongly bound water called the "double layer water", or absorbed water. Bolt (1956) has used the double layer approach, with the concept of hydrated exchangeable cations, to develop and test a model based on osmotic pressure differences between the absorbed water and the free water.

3.3.2.1 Exchangeable Cations

It is necessary to clarify the status of the exchangeable cations before proceding. As the name implies, these are cations that are readily exchangeable. This is in contrast with other cations related to the clay mineral, i.e. the cations in the crystal structure which cannot be exchanged except by changing the

crystal structure and chemistry.

The name also implies, correctly, that these cations cannot be simply removed from the system, but must be replaced by other cations. This is true because these cations were required in the first place to provide electrical and chemical stability. Removal without replacement would result in an unstable chemical system. An analogy to this is the removal of chlorine from a solution of table salt in water. It could be done, but the resulting product would no longer be a solution of salt. However, by supplying alternative cations, the exchangeable cations can be displaced by the alternative cations if the conditions are favourable (Schreiner 1987c).

3.3.2.2 Osmotic System

Bolt (1956) utilises this association of the cations and the solid particles to describe an osmotic system, in which he describes the clay particles as the semi-permeable membrane. The semi-permeable boundary would be more correctly defined at the interface between the absorbed water, which contains the exchangeable cations, and the free water, which contains any dissolved salts which may be in the water (Schreiner 1987c).

Essentially, the cations are not free to move out of the absorbed water due to electrostatic forces between the negatively charged solid particle and the positively charged cations. An osmotic pressure can thus be established between the absorbed water plus exchangeable cation system and the free water plus dissolved salt system. This results from the tendency of the free water to attempt to dilute the absorbed water until the concentrations are equal at the interface between the two systems.

3.3.3 PARTICLE ENERGY MODEL

The particle energy model is based on the balance of attractive and repulsive forces between adjacent particles. The negative charge on the solid particle is balanced by the positive charges on the exchangeable cations, but the distribution of the charged bodies in the double layer does not eliminate the repulsion between the negatively charged surfaces. The double layer theory on which this is based requires the clay surfaces to be non-wettable or lyophobic. The adsorbed water appears to be in contradiction with this assumption.

Yong et al (1984) have calculated the attractive and repulsive forces for low and high salt contents, and showed that the former led to a net repulsion while the latter gave a net attraction. Their model appears to be fairly inaccurate when compared with experimental data.

Stocker (1969) states that the osmotic and particle energy models give equal repulsive forces. Sridharan & Jayadeva (1982)

have combined the osmotic and particle energy models. They found better agreement for their model between theoretical and experimental void ratio-effective stress curves than for the previous models. This suggests that the osmotic and particle energy models describe the same physico-chemical preocess in different terms.

3.3.4 ENGINEERING IMPLICATIONS OF THE MODELS

One of the major differences between the capillary model and the osmotic model is that in the former case the water holds the soil particles together, while in the second case, the water holds the particles apart. In the capillary model there is an implicit assumption that the particles are in contact with one another, and volume change must therefore involve slippage between the particles. In addition, there is no force to cause the particles to separate if the capillary suction is reduced, except strain energy stored in the particles.

Soils governed by capillary forces will not swell when the suction is decreased, as is the case for a pure silt. Carefully designed soil mixes containing suitable proportions of mica, which is made up of thin platy particles, can show significant swelling on reduction of effective stress (Terzaghi & Peck 1969), but this does not appear to be a probable mechanism for natural soils.

In the osmotic model the solid particles are not in immediate contact, and volume change can occur without alteration of the particle arrangement. In effect, the particle is made up of the solid core and the adsorbed plus absorbed water. The amount of water in the absorbed layer will be dependent on the stress state to which the soil is subjected. In the saturated soils which Bolt (1956) studied, the stress causing expulsion of water from the absorbed layer is the externally applied effective stress.

When the soil is unsaturated, the removal of water from the absorbed layer can be caused by evaporation into air of lower relative humidity than that with which the soil would be in equilibrium, or by drainage under a suction gradient such as is set up in the pressure plate system. Either process will decrease the effective size of the particle plus the absorbed water, and thus decrease the overall soil volume. There must be a force holding the particles together which is not generally considered in applying this model. This force could either be generated by capillary forces in the free water or by sharing of the absorbed layers between adjacent solid particles. A conceptual diagram of the latter is shown in Fig.3.3, where the semi-permeable boundary encloses more than one solid particle.

These two models can be seen to be very different in the type of soil behaviour that they support. The capillary model is not adequate for the description of swelling of expansive soils, and will not be used in this thesis with regard to the soil. It is,

however, the applicable model for the high air entry porous ceramic which forms the basis of the suction control technique in the test equipment.

3.4 ENGINEERING PROPERTIES OF THE DOUBLE LAYER

Since all clays have silicate layers and exchangeable cations, it is reasonable to expect similarities in their interaction with water. Lambe & Whitman (1969) show that individual particles of Kaolinite and Montmorillonite will develop double layers of similar thickness, while Lambe (1958), (Fig.3.4),shows a thicker layer for Kaolinite than for Montmorillonite. However, since the solid Kaolinite particle is about 10 000 times the volume of the solid Montmorillonite particle, the water contents of the particle plus double layer will be much smaller for Kaolinite than for Montmorillonite.

The volumetric water content for a 200A layer around a Montmorillonite particle is about 40, while for a 400A layer around the Kaolinite particle it is only about 0.8, (Fig.3.5). This indicates a theoretical potential volume change for Montmorillonite of 50 times that of Kaolinite from completely dry to fully hydrated. The intrinsic expansiveness can thus be seen to be strongly influenced by particle size which is in turn dependent on the mineral type. The higher surface activity of the Kaolinite is masked by the effect of the relative particle volumes.

The development of the double layer can be affected by several factors. One of these is the type of exchangeable cation (Grim 1962, Stocker 1969). The Atterberg Limits may be used as an indication of the volume of the double layer water present under the test conditions, although not all of the water in the soil is held in the double layer. Lambe & Whitman (1969) present data showing a decrease in plasticity index for montmorillonite from 656 for sodium as the exchangeable cation to 429 for calcium. Most of the reduction is due to the liquid limit and may thus be related to the maximum thickness of the double layer. Similar but smaller reductions are demonstrated for illite and kaolinite, in keeping with the discussions above on the influence of particle size.

The composition of the pore fluid may also affect the double layer thickness. Bolt (1956) demonstrated that, by varying the concentration of sodium chloride in the pore fluid, different void ratio-stress curves were obtained, with the higher void ratios associated with the lower concentrations of salt. Yong et al (1984) found similar results for natural soils. Thus, under identical externally applied stress conditions, a soil with a high salt content will have a lower void ratio than the same soil with a low salt content. The type of water used in laboratory tests or provided in the field may thus affect the volume change of a soil, and may cause volume change by alteration of the salt content in

the soil by diffusion without changes in the matrix suction.

Where the water causing swelling is relatively salt free, such as rain water or distilled water, the salt concentration in the pore fluid will tend to decrease as the moisture content of the soil increases. If, on the other hand, the water has a similar salt content to the pore fluid, as may be the case with wetting up due to groundwater flow, then the salt concentration of the pore fluid will be unaffected. The soil wet up with salt-free water will therefore swell more than the soil in which the salt concentration is maintained at its initial value. This can be of significance in laboratory testing of soils with high salt contents, as has been observed by Morgenstern & Balasubramonian (1980) and Pile (1984).

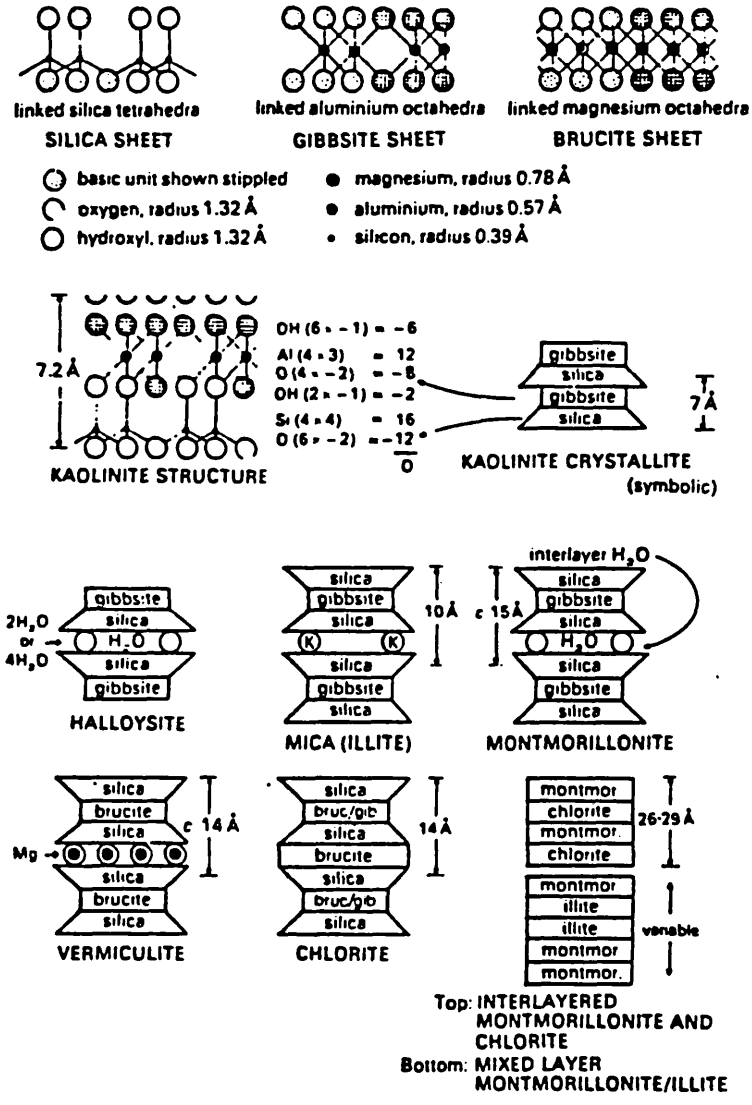


Fig.3.1 Mineral structures
(Taylor & Cripps 1984)

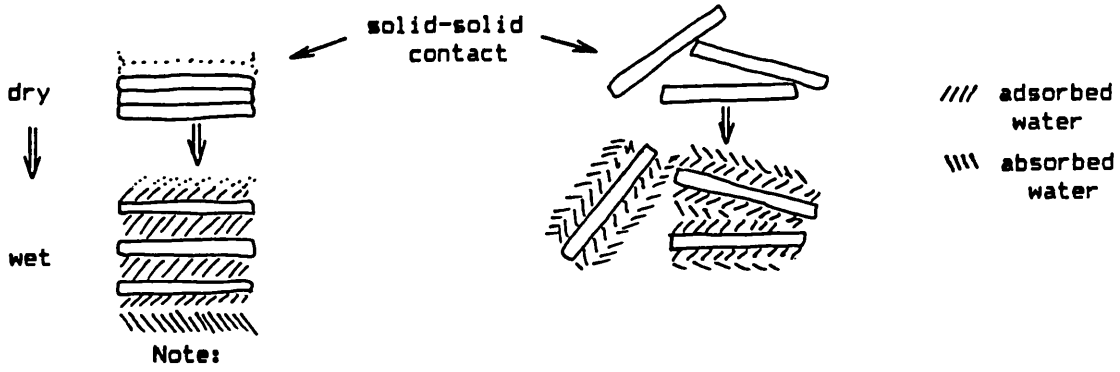


Fig.3.2 Alternative concepts of adsorbed water and particle arrangements

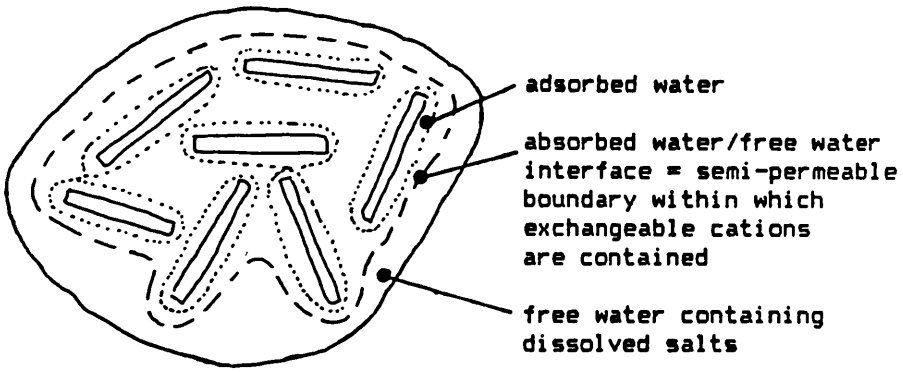


Fig.3.3 Semi-permeable boundary around assemblage of particles

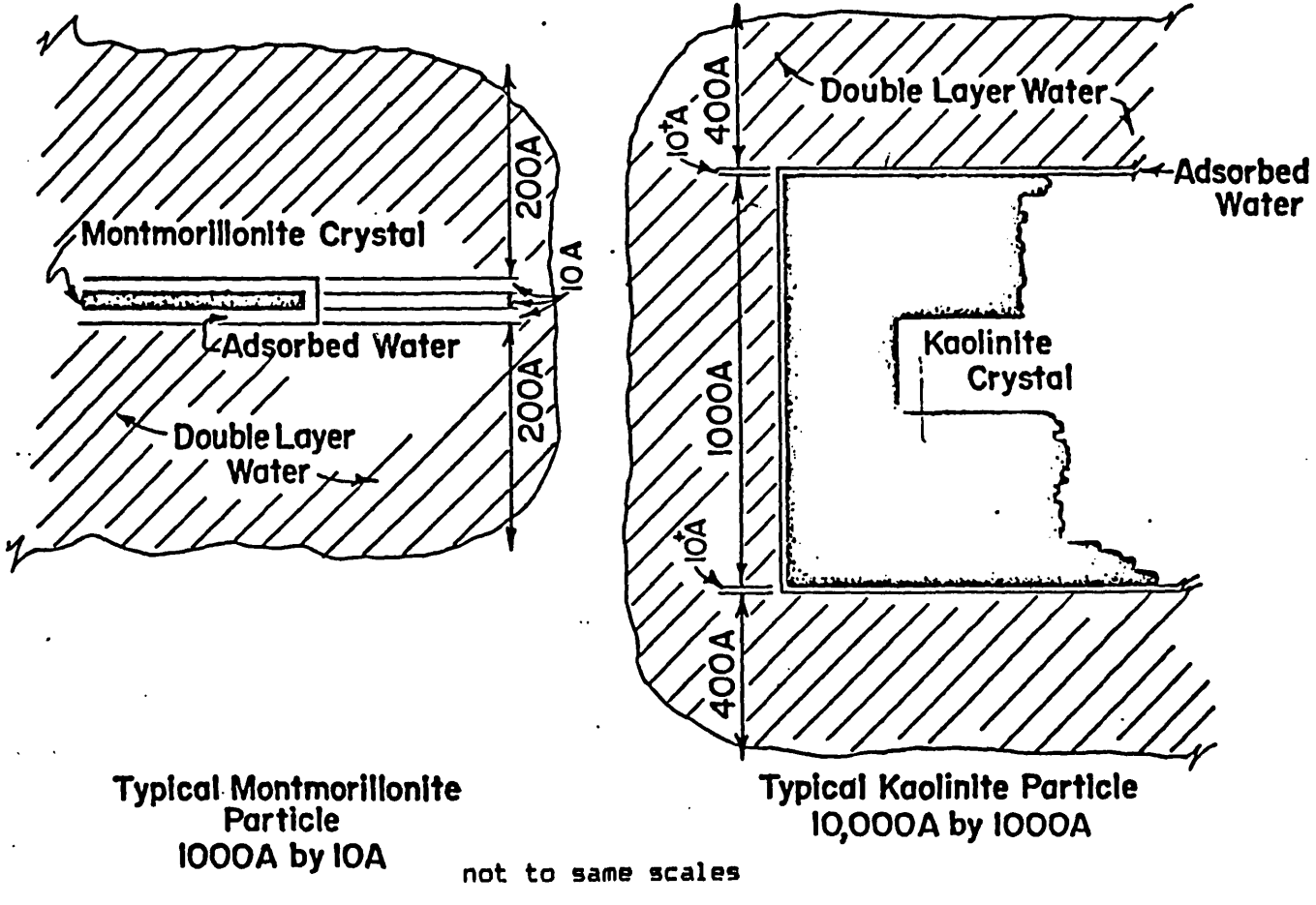
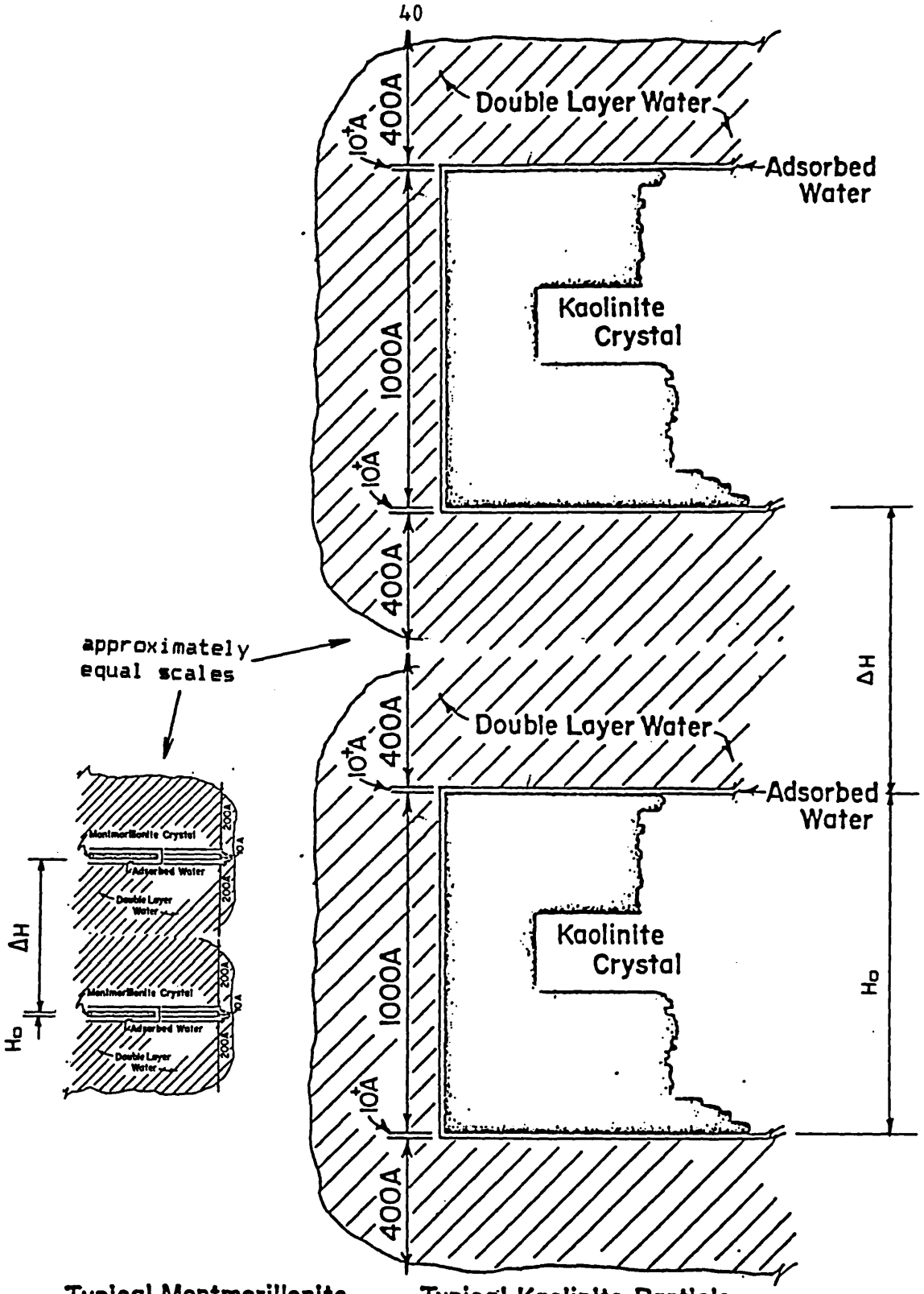


Fig.3.4 Clay water interaction
(Lambe 1958)



Typical Montmorillonite Particle
1000A by 10A

$\frac{\Delta H}{H_o} = 40$

Typical Kaolinite Particle
10,000A by 1000A

$\frac{\Delta H}{H_o} = 0.8$

Fig.3.5 Theoretical potential volume change for kaolinite and montmorillonite

CHAPTER 4

DETAILED REVIEW AND RESEARCH PLAN

4.1 INTRODUCTION

This chapter attempts to assemble the available data relevant to the study of volume change of unsaturated soils. The primary direction of the research is the swelling of unsaturated compacted soils, but volume decrease due to decreasing suction must be included. It will become apparent that the available data are limited in quantity and quality.

As described in Chapter 2, the research into volume change of unsaturated soils has not utilised the frameworks of modern soil mechanics. Useful data are scarce, and incomplete data must be utilised in this review in order to define the current state of knowlege. The test data will be divided into four categories on the basis of the stresses measured or controlled:

4.2.1 Tests with only vertical total stress measurement
4.2.2 Tests including axial and radial stress measurement
4.2.3 Tests including suction measurement or control
4.2.4 Tests including both principal stresses and suction

4.2 VOLUME CHANGE DATA

4.2.1 TESTS WITH VERTICAL STRESS

Most of the data from simple oedometer soaking tests are of little relevance to this research. A few points will be noted.

The effect of vertical applied stress was first reported by Rengmark & Eriksson (1953) who showed that the swell decreases as the vertical stress increases. This has also been shown by Didier et al (1980) for a greater stress range. Wiseman & Zeitlen (1960) demonstrated that swell for samples compacted "wet" of optimum moisture content swelled less than samples compacted "dry" of omc. This was taken a stage further by Noble (1966) who derived a relationship for swell in terms of vertical stress and compaction moisture content, (Fig.4.1), for one soil from a single initial dry density. The influences of the initial density, moisture content and vertical applied stress were studied by Schreiner (1987c). The void ratio after swelling was shown to reflect the compaction moisture content-void ratio curve, at vertical stresses below 64kPa. Brackley (1983) included the soil type, the soil

density and the vertical stress to derive an equation for swell, s, from the compaction moisture content:

$$s\% = (5.3 - \frac{147e}{PI} - \log p)(.525PI + 4.1 - .85w_i)$$

where p = vertical applied stress
w = compaction moisture content
e = initial void ratio
PI = plasticity index

This equation was tested by Brackley against data from 6 sites, and found to be of limited reliability. Since the data came from undisturbed soil profiles with uncontrolled moisture content changes, whereas the equation was derived for compacted soils, this unreliability is not surprising.

de Bruyn (1961, 1963) observed that, for remoulded clay, there was a lower moisture content below which there was no volume change, and an upper moisture content above which there was no volume change. His tests were performed under isotropic total stress with addition of water without suction control or measurement. The lower value probably corresponds to the shrinkage limit, but the upper value may be affected by the void ratio and the soil fabric. A similar finding was reported by Popescu (1979) during shrinkage.

Collapse was shown by Jennings & Burland (1962) to terminate near to the virgin consolidation line or the soaked consolidation line. Possibly of more importance is their demonstration that an undisturbed sample of an "expansive" soil, when loaded before soaking, can attain void ratios above the soaked consolidation line for a similar sample and thus be liable to collapse (Fig.4.2). This was the first report that collapse and swell are not intrinsic soil properties, but are stress and suction dependent phenomena, and can occur for the same soil. Although not noted at the time, this observation leads to the conclusion that the mechanisms of swell and collapse are different.

Subsequently, Novais-Ferriera (1973), Cox (1979) and El Sohby & Rabaa (1984) have shown that collapse and swell are also influenced by compaction moisture content and density. All found that collapse and swell can occur in the same soil. Cox (1979) showed that the conditions required for collapse are low density and low compaction moisture content, while swell is associated with high initial density and is insensitive to compaction moisture content (Fig.4.3a). Novais-Ferreira tested samples at various positions on the moisture-density curve and found that collapse and swell were determined by the vertical total stress during soaking (Fig.4.3b).

4.2.2 TESTS WITH PRINCIPAL TOTAL STRESSES

Three sets of data are available in this category. One set has

been obtained using instrumented oedometers, another is from stress controlled triaxial tests, and the third is from constant volume wetting in a triaxial cell.

Most of the oedometer data are from Komornik (1962), but have been reported at various times by several authors. All the oedometer data are for single stage soaking to zero suction under a constant vertical applied stress. Komornik & Zeitlen (1965) present curves of swell against constant vertical applied stress and measured radial stress for compacted highly plastic clay (Fig.4.4). They found that the ratio of horizontal to vertical effective stress at the end of the test increased as the amount of swell increased. At low vertical applied stresses, corresponding to high amounts of swell, the stress ratio was large, reaching about 20 at 5kPa vertical stress. Isotropic stress conditions are found for small magnitudes of swell, and in the compression or collapse region, the stress ratio was less than 1.

Kassif & Baker (1969) tabulated more of the original data on swell pressures. This indicated that the stress ratio was generally around 1 for zero vertical and radial strain, but slightly lower values occurred at high initial dry densities. This implies that the fabric of the compacted samples is largely isotropic, which is consistent with the fabric observations of Collins et al (1984) and Zein (1985) noted in section 2.2.3. They have shown that the fabric at optimum moisture content compaction is perturbed parallellism, which has no overall preferred orientation.

Ofer (1980) presented data showing a stress ratio of more than 2 for constant volume wetting using a similar oedometer on a moderately dense compacted sample. He also found that the "lateral swelling pressure" increased dramatically as the initial dry density increased, without a comparable increase in the amount of swell (Fig.4.5).

High stress ratios were also reported by Ofer & Komornik (1983) at low vertical stresses. They found that the stress ratio at the end of swell under constant vertical load tests decreased towards about 0.75 for high vertical stresses.

All of the above observations are consistent with the observed effective stress changes during one-dimensional compression and swell of saturated samples. Mayne et al (1982) use published data to show that K_o increases during unloading as a function of OCR, reaching values of up to 3 in some cases for cohesive soils. Kirkpatrick et al (1986) and Atkinson & Bransby (1977) show similar trends over smaller ranges of overconsolidation. The extremely high values of K_o reported for unsaturated soil testing have not been reported for saturated soil tests. However, since the soils are very nearly saturated at the end of the swelling tests, the effective stress concept should be applicable. There should be equivalence between the results from both saturated and unsaturated samples.

None of the studies on unsaturated soils have considered the effect of wall friction on the measured stresses. Stress ratios above 3 have also been reported by Dyvik et al (1985) for saturated soils during unloading in an oedometer. Some of the results from the current research are relevant to this aspect, and will be discussed in Chapter 10.

Triaxial constant applied stress tests were used by Dakshanamurthy (1979). As for the oedometer tests, these allowed soaking in a single stage under a constant vertical applied stress, but also applied a constant cell pressure. He used radial to axial stress ratios between 0.2 and 1 and mean stresses between 30 and 260 kPa on samples compacted to the same density and at the same initial moisture content. All the data showed a unique relation between volumetric swell and the mean stress (Fig.4.6a).

The data from Fig.4.4 are replotted in Fig.4.6b on similar axes for comparison. The trend is very similar for both sets of data even though the radial total stress in the oedometer is not constant. Replotting Dakshanamurthy's data on p - q - strain axes, Fig.4.7, shows less clarity. The agreement between the data for the different stress ratios is less good, and the curved relationship is not apparent. It is also of interest to note that his tests do not cover the stress space where the radial stress is the major principal stress. This stress space has been shown by the oedometer tests to be relevant for swelling.

Tisot & Aboushook (1983) undertook a study of swelling pressures in a triaxial cell and found stress ratios of about 2. However, the tests seem to have been constant volume tests, and not zero strain tests, so it is difficult to see how the stress ratios were controlled.

4.2.3 TESTS WITH CONTROLLED SUCTION

Kassif & Ben Shalom (1971a) used multistage swell and swell pressure tests to show, Fig.4.8 and Fig.4.9, that swell and swell pressure are developed progressively as the suction is changed.

Although Fig.4.8a shows that the less dense samples attained higher final moisture contents, the magnitude of the swell was less than for the dense samples. The data from Fig.4.8a are replotted in Fig.4.8c in terms of void ratio against moisture content, which gives a clearer picture of the test results. The three low density samples swell from an initial void ratio of 1.04 to void ratios between 1.09 and 1.16. This is consistent with the suction changes, in that the wettest sample, with the lowest initial suction has swelled by the smallest amount, while the driest sample has swelled the most.

The difference in final void ratios between the high density group of samples and the low density group could be due to either or both of two reasons.

Firstly, in swelling of saturated soils, the rebound curves of

void ratio versus effective stress are dependent on the maximum past consolidation pressure. For compacted soils it is very likely that the same will be true for both maximum past suction and maximum past applied stress. In the present case the compaction stress for the high density samples would have been much greater than that for the low density samples.

Secondly, Zein (1985) has shown that the soil fabric due to compaction is dependent on the compaction water content, and is not significantly altered during swelling under low vertical applied stresses. For samples compacted progressively drier than the optimum moisture content, the soil tends toward a clustered structure with random particle orientation. Samples compacted at or wet of optimum moisture content have a perturbed parallel structure, in which particles are locally arranged parallel to each other, but there is no overall preferred orientation. The clustered structure with random orientation will exist at a higher void ratio than the perturbed parallel structure under identical stress conditions.

It is apparent from Fig.4.8c that the low density samples are further from the saturation or zero air voids line. It is also generally true that the optimum moisture content of a soil decreases as the compactive effort and thus density increase. Since the two sets of samples presented in Fig.4.8 were prepared at the same moisture contents, it may be deduced that the low density samples were compacted in a relatively drier state than the high density samples, and are thus likely to have, and to retain during swelling, a more clustered structure, and as a result, a higher void ratio after swelling.

Fig.4.9 shows that swell pressures were developed with small increases in moisture content, which is to be expected, as the final void ratio will be less than that of the swelled samples. The more rapid increase in moisture content, after most of the swell pressure had been developed, for the low suction range probably corresponds to the filling of macro-voids between the soil clusters or peds. This is more noticeable for the low density samples than for the high density samples which is consistent with the discussion above on soil structure for these samples. Each pair of equal initial moisture content samples show a change of gradient of the moisture content-swell pressure curve at about the same moisture content. After this point the low density samples take in more water than do the high density samples.

It is also apparent from Fig.4.9 that the initial moisture content affects the swell pressure, which is in conflict with Komornik & Zeitlen (1965) who found, Fig.4.4, a unique vertical swell pressure value for a particular dry density for a range of initial moisture contents. If we consider the volume change of samples from the same initial void ratio but different initial moisture contents, then it is apparent that the initial moisture content should affect the swell pressure. Since the soils are

"overconsolidated" by compaction and suction, there is no requirement that the void ratio be uniquely dependent on stress. Since the drier samples swell more than the wetter samples under constant applied stress, it is to be expected that a greater applied stress will be required to suppress the swelling of the drier samples than for the wetter samples.

The swell versus suction data, Fig.4.8b, shows a strong increase in the magnitude of swell as the suction approaches zero. This could be suppressed if the applied vertical stress was greater than the 16kPa which they used. Escario & Saez (1973) have presented data, Fig.4.10, which shows lesser increases in swell at low suctions for high applied stresses than for low applied stresses. The applied stress at which the rate of swell does not increase at low suction is relatively high, and may be an indication of collapse or structural change under the high vertical applied stress.

A more important observation by these authors is that collapse can occur after swelling. Fig.4.11 shows swell during the initial stages of suction decrease followed by collapse in the last one or two stages. Swell followed by collapse of the same sample has also been reportd by Blight (1965), Fig.4.14, and by Barden et al (1969), path ADEH in Fig.4.12, but in the latter case the applied vertical stress had been increased from D to E between the two suction decrease stages AD and EH of 6psi each. These observations can be qualitatively interpreted as shear failure at the grain or cluster contacts as suggested by Burland (1965), or as deformation of the clusters, both mechanisms being controlled by the decrease of local shear strength as the suction decreases.

Some work has been done on the effect of solute suction. Morgenstern & Balasubramonian (1980), Fig.4.13, and Pile (1980) have shown that wetting with distilled water will cause more swell than wetting with saline water. There is some doubt regarding the "reconstituted pore fluid" used by the former authors due to the extraction procedure (Schreiner 1987c), but the solute suction effect is clearly demonstrated. Richards et al (1984) attempted to investigate both the effect of solute suction and salt type, but their data must be somewhat unreliable due to alteration of the exchangeable cations in the soil by the salts in the supply water. Since there was no attempt to match the soil and supply water salt types, there could have been changes in the exchangeable cations in the soil with resultant changes in the soil behaviour.

Only one comparison between controlled suction tests and soaking tests has been found. Justo et al (1984) noted that the swell to zero suction in a controlled suction test was less than in a quick soaking test. No explanation was advanced for this. A similar finding from the current research (Schreiner 1987e) will be discussed in Chapter 10.

4.2.4 TESTS WITH FULL STRESS STATE

Blight (1965) presents a swelling curve, Fig.4.14, with controlled suction under isotropic conditions. This is for a collapsing sand, but the amount of swell between 9.5 and 4psi plus that from 2psi to zero suction almost matches the collapse between 4 and 2psi. The isotropic applied stress was constant during this test, which confirms that swell and collapse are not mutually exclusive.

Several authors have studied compression, e.g. Matyas & Radhakrishna (1968), Karube (1983) and Maswoswe (1984). The general finding is that the stress path to zero suction is not greatly significant, as shown by Fig.4.15 from Matyas & Radhakrishna (1968). There are, however, two differing findings of collapse in terms of suction.

Blight's finding of a critical suction at which all collapse occurs is not consistent with the more recent work which shows collapse to occur progressively as the suction is decreased. Since Blight (1965) used undisturbed samples while Matyas & Radhakrishna (1968), Karube (1983) and Maswoswe (1985) all used compacted samples, both observations may be correct. Further study is required before the findings from the compacted soils are applied indiscriminately to natural and compacted collapsing soils. Similar studies have not been performed for swelling.

Vaughan et al (1988) have developed a model for bonded soils, Fig.4.16, which is similar to Maswoswe's findings for collapse, Fig.4.17 from Burland (1985), except that in the latter case the collapse can be induced by decreasing the suction. This suggests that, for compressive volume change, the effect of suction may be similar to the effect of bonding. This approach will be pursued further in Chapter 10, where results will be presented showing swell and collapse to be possible in the same sample.

4.3 OUTLINE OF RESEARCH PLAN

The foregoing review has shown that little fundamental research has been done on swelling of unsaturated soils. Even with the data from the simple or project specific testing there are large gaps in the current state of knowlege. No data is available on the full stress paths followed by the soil during swelling, except the isotropic data from Blight (1965). Some data is available on some components of the stress paths where incomplete stress measurements have been made. Claims have been made regarding the effect of different stress paths on the volume change, but these cannot be accepted since the stress paths and final stress states have not been determined (Schreiner 1987e).

It is not possible, nor reasonable to attempt, to solve the entire swelling problem in the space of one research project. However, it is possible to attempt to define the barest of outlines of the fundamental mechanisms of swelling by applying

modern soil mechanics principles to an experimental study of the soil during swelling. In the first instance, this requires that the full stress state be measured or controlled during the tests. Additionally, the experimental programme must be relevant to the practical problem of engineering in expansive soils.

Apart from the need for advanced soil mechanics testing, it was also necessary for this project to address the whole range of unsaturated soil mechanics, particularly as applied to expansive soils. This has necessitated a multi-directional approach, with several studies going on in parallel. As a consequence the advanced work has not been undertaken in great depth, but the breadth of the investigation has provided a basis for further work in a similar field by providing a secure body of data in the subjects related to the mechanics of unsaturated soils. The supporting studies included the measurement of suction, the identification of expansive soils from their index properties and oedometer swell tests. The chemical and physico-chemical properties of clays have been largely addressed by reviewing the published theories, but a limited amount of information has been gathered from the experimental programme.

4.3.1 PRELIMINARY STUDIES

4.3.1.1 Review of predictive methods

It is desirable during the site investigation phase of any project to be able to make a preliminary assesment of the expected behaviour of the soils. This is frequently done on the basis of the plasticity of the soil and the grading. While this approach has seen some success for saturated sedimentary soils, the level of reliability in the prediction of expansiveness and swell has been low. A review of predictive methods (Schreiner 1987b) found little agreement between methods, and was thus followed by a comrehensive study using published swell data (Oloo 1987)

4.3.1.2 Standard Oedometer testing

Oedometer swell tests were used to study the effects of vertical stress and compaction moisture content on volume change. The constant applied vertical stress range was 1.5 to 256 kPa, to include the likely vertical stress range for road embankments and to approach the vertical swell pressure. The compaction moisture content range was from the plastic limit down to about 10% of the plastic limit. A selection of soils was chosen to reflect the range of expansive soils likely to be encountered in road construction projects in Africa.

4.3.1.3 Suction Measurement Techniques

Although suction measurement and control has been possible for many years, there has been a certain amount of mystique about the techniques in use. Measurement of suction using the pressure plate or elevated pore air pressure technique is an example. Various procedures have been described, but none have been justified on theoretical or scientific grounds. One recommendation (Toll 1985) is that the porous plate should be wiped to leave a little water on the surface before placing the sample on the porous plate and applying the air pressure, in order to obtain consistent results. This is far from an acceptable "standard procedure", and there are other such qualitative and subjective recommendations. There are also reports of inconsistencies between types of suction measurement, e.g. Richards (1980) and Edil & Motan (1984). Since measurement and control of suction are essential for this research, it was necessary to develop the techniques for suction measurement to a useable level.

4.3.2 STRESS PATH TESTING

The primary concern in expansive soil engineering is to obtain a reasonable estimate of the vertical displacement likely to be experienced by the structure or the soil during swelling. This is similar to the need to know the likely settlement of a structure built on compressible soil. However, the problem is made more difficult by the lack of control of the stress changes that will contribute to the displacement or heave. In the settlement problem, the stress changes are usually available from the designer of the structure. In the swelling problem, the stress change is usually climate dependent, and may be difficult to estimate. Progress is being made in the study of long term stress conditions below covered surfaces, e.g. Emery (1986), which is extending the knowlege derived from the early studies e.g. Russam & Coleman (1961). There is thus some justification for examining swell throughout the suction range from very dry soil to soaked soil.

The simplest form of test with which to start studying this problem is the one-dimensional oedometer test. The ordinary test and equipment must be adapted to include measurement of the radial stress, because the ratio of axial to radial stress, K, will not be constant throughout the test. The oedometer must also be designed to permit control of the matrix suction. Since no suitable oedometer had been developed, one has been designed and used in this investigation.

With this oedometer, three stress paths can be followed with relative ease. These conveniently correspond to the standard swell tests described in Chapter 2. In one the suction in the soil is first reduced to zero and the sample then subjected to a consolidation test. In another the vertical stress of interest is applied before reducing the suction, and in the third the suction

is reduced to zero while preventing volume change. The sample is then allowed to swell under reducing vertical effective stress.

The triaxial cell permits other stress paths to be followed, particularly if the stress path cell is used. A Bishop & Wesley cell was, therefore, adapted for volume change measurement of an unsaturated sample. The set of stress paths described above could be usefully complemented by a constant applied stress test in which both axial and radial movements are measured. In addition, the samples may be sheared to give strength data at various values of suction. This test series was severely curtailed by slow fabrication of the equipment, and only one test was completed.

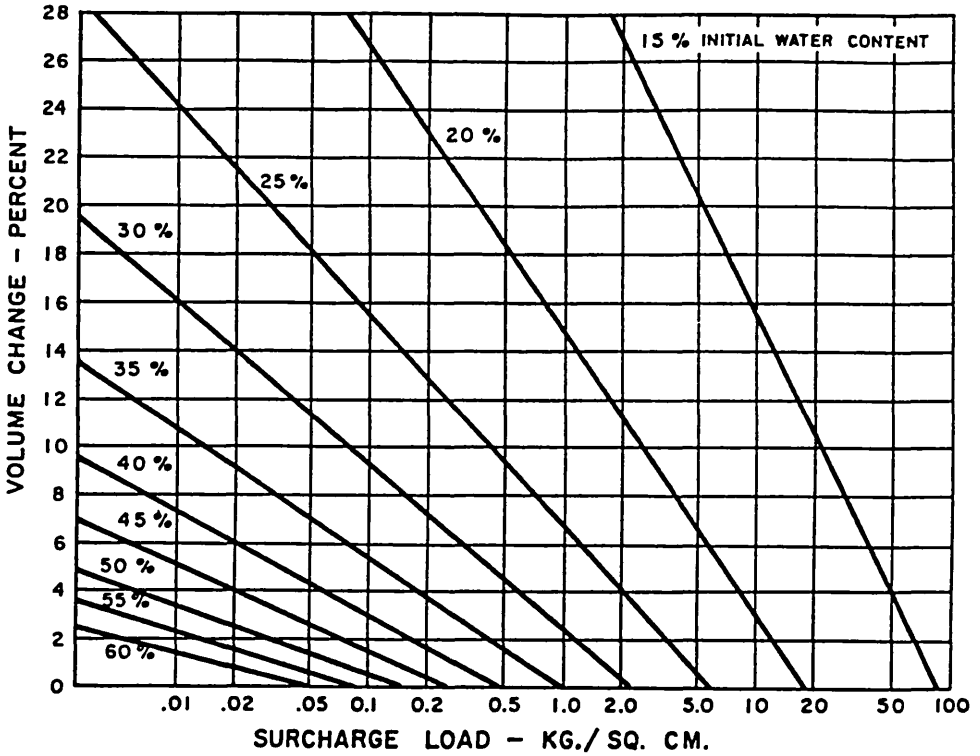


Fig. 4.1 Volume change versus surcharge load from the empirical relationship, Equation 6

(Noble 1966)

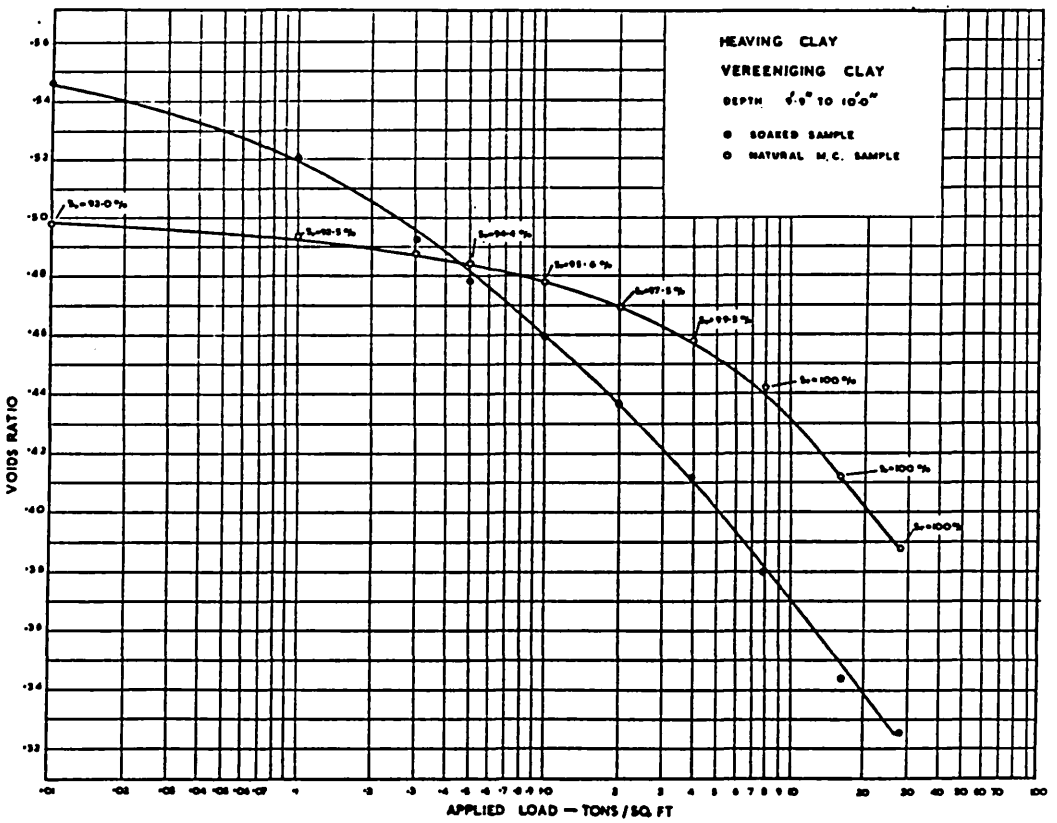


Fig. 4.2 Typical double oedometer curves for expansive clay adjusted to equal initial voids ratio at 0·01 tons/sq. ft applied pressure

(Jennings & Burland 1962)

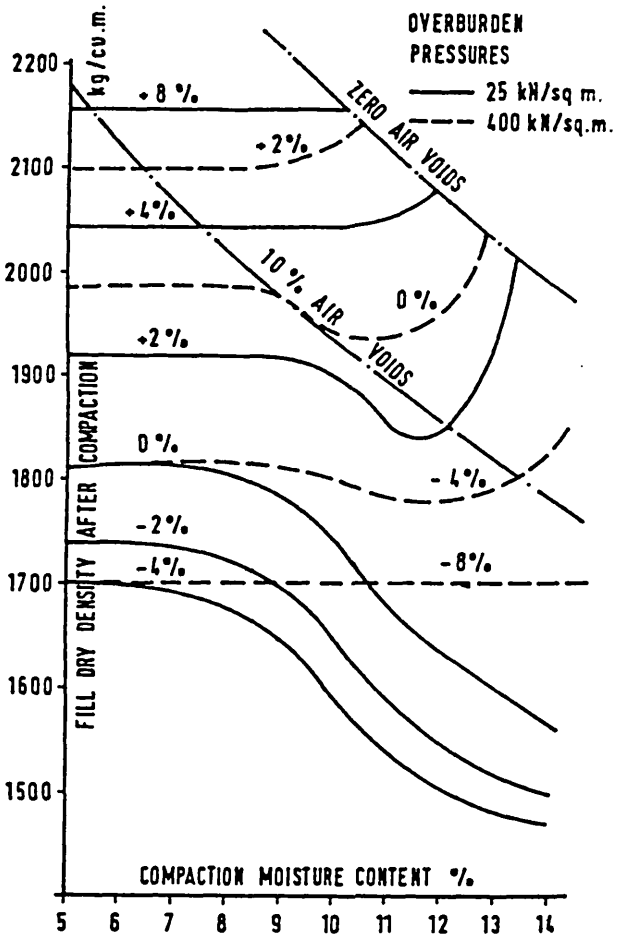
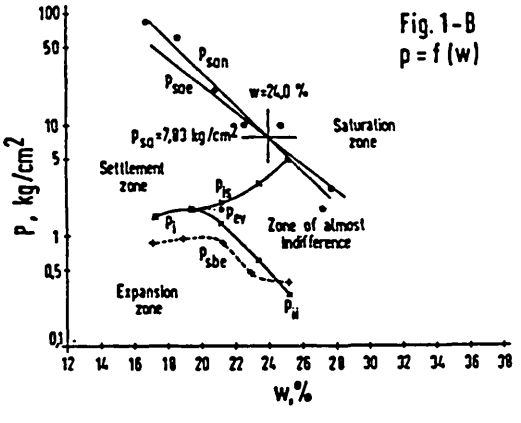


Fig.4.3a Swell or collapse after inundation at varying initial moisture content and initial dry density (constant overburden pressure)

(Cox 1979)



▪ - P_{en} - Limite pressures, upper (P_{ens}) and lower (P_{eni}), of indifference to soaking null expansion pressure.

● - P_{sa} - Pressure at which saturation occurs by decrease of void ratio due to consolidation, at constant moisture content (w).

P_{san} - In the OSN test.

P_{sae} - According to the oedometric curve of the OSS test.

+ - P_{sbe} - Saturation pressure corresponding to the inicial specimen volume in the oedometric curve of the OSS test.

● - P_{ev} - Virtual oedometric expansion pressure.

Fig.4.3b Swell and collapse of compacted silty clay

(Novais-Ferreira 1973)

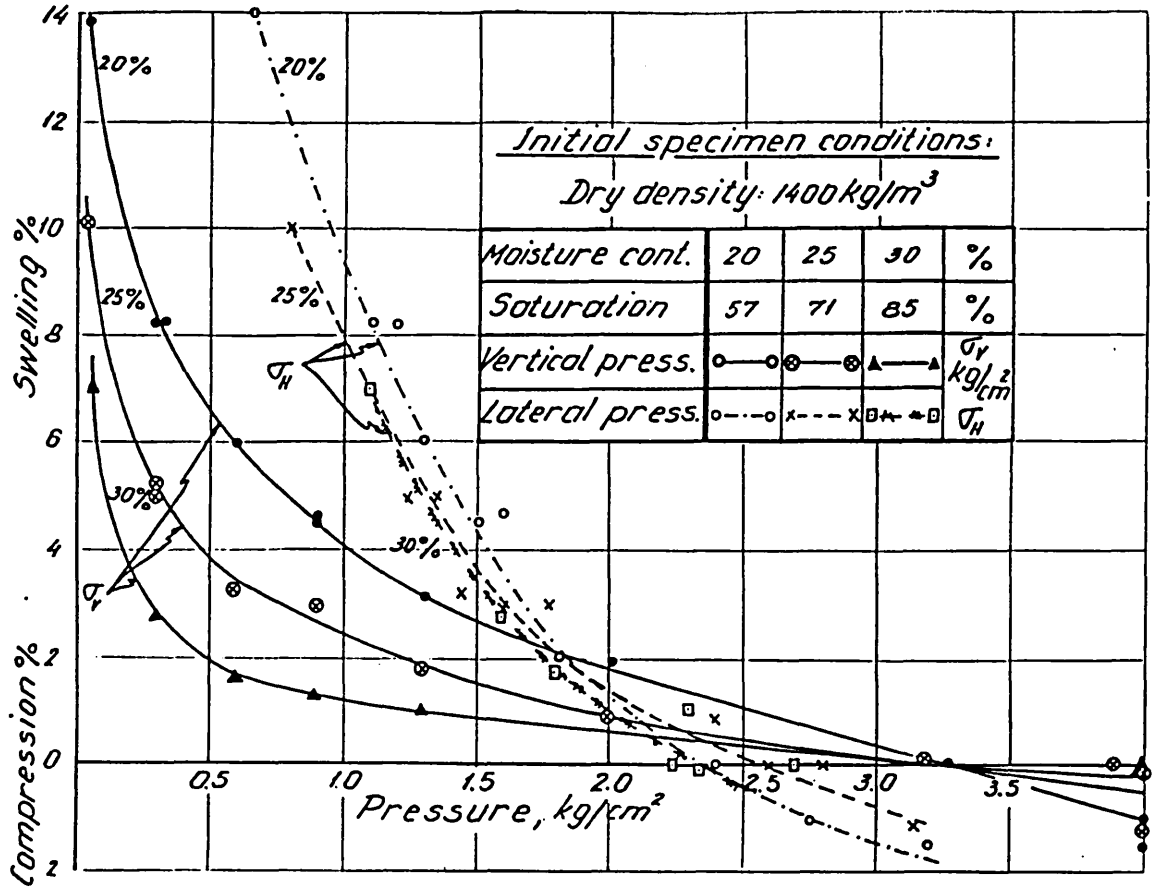


Fig.4.4 Typical relations observed for vertical movements as a function of vertical and lateral pressures.

(Komornik & Zeitlen 1962)

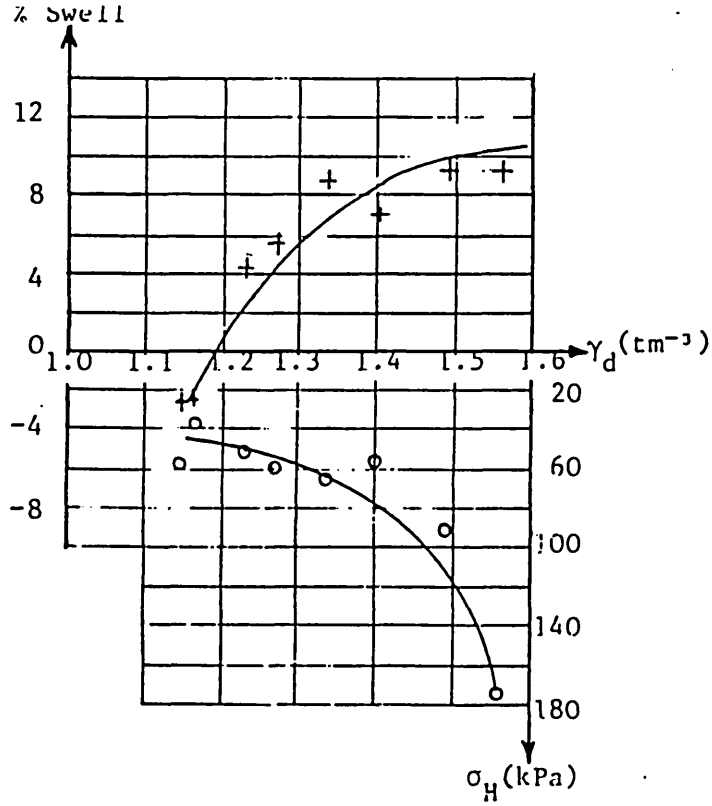


Fig.4.5 Density-% Swell-Lateral Swelling Pressure Relation for the LSP Ring Tests σ = 19 kPa

(Ofer 1980)

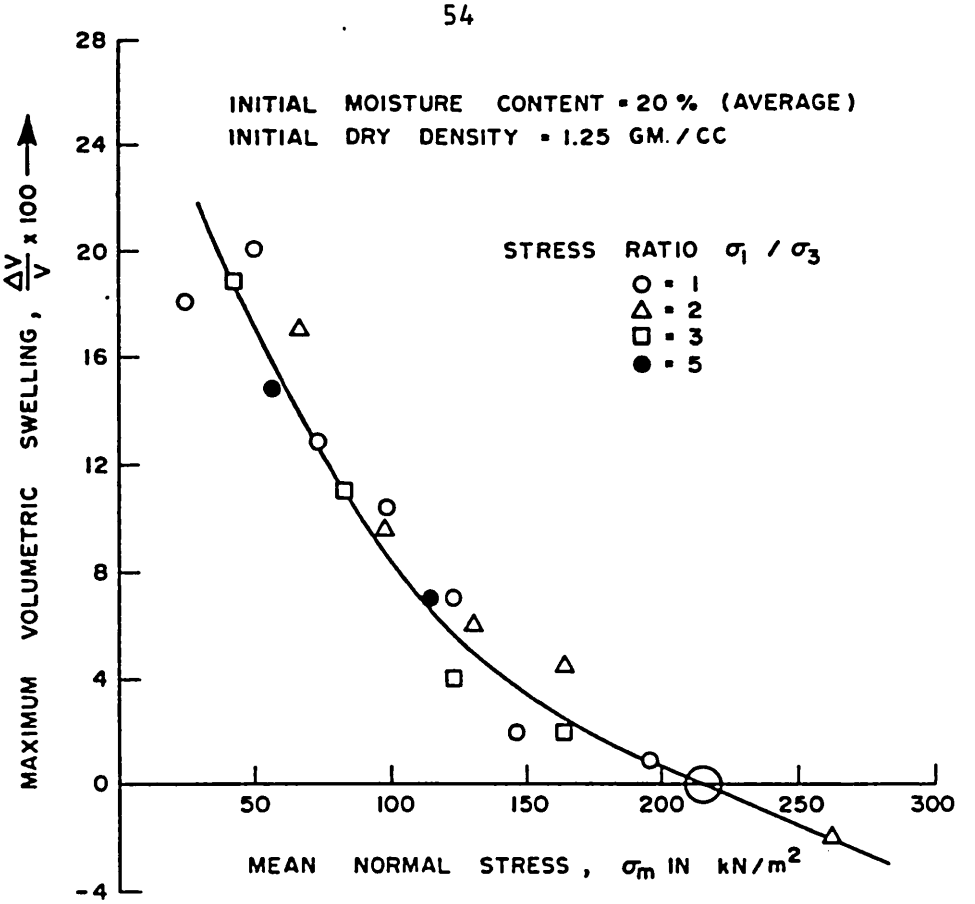


Fig.4.6a *Relationship between* ϵ_{vol} *and* σ_m.

(Dakshanamurthy 1979)

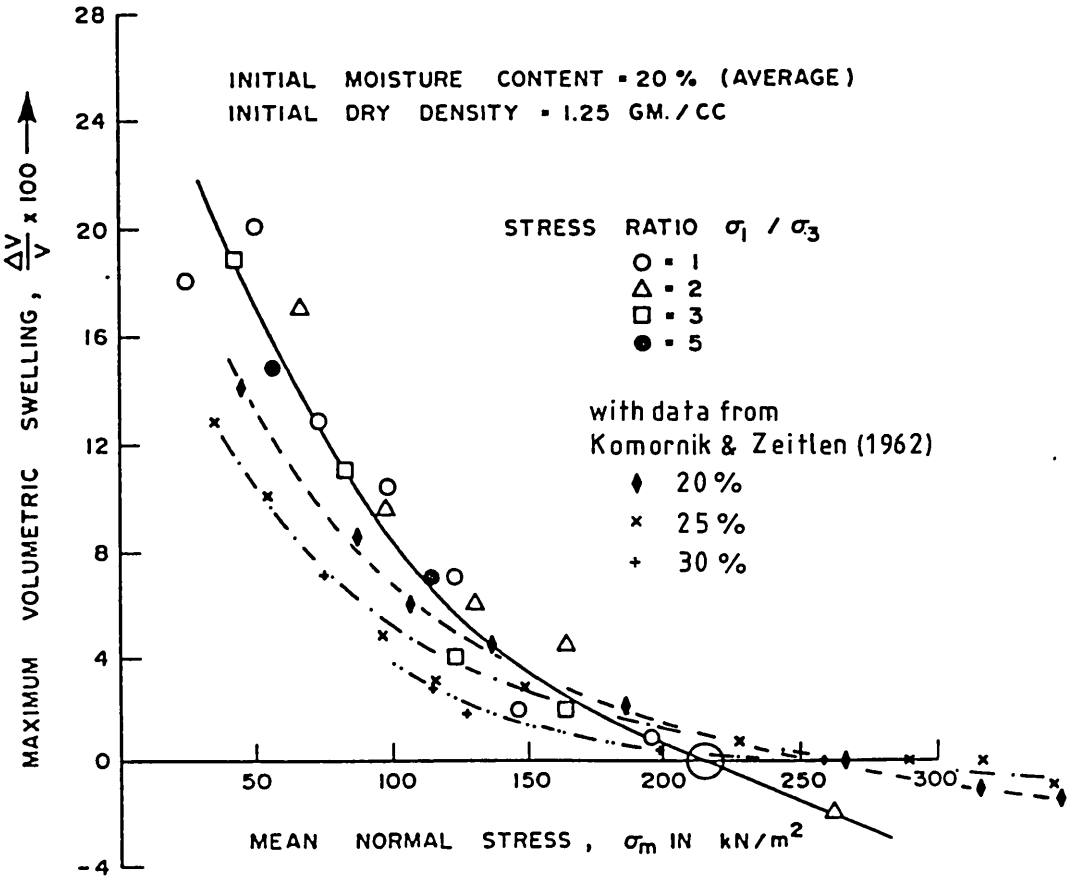


Fig.4.6b Comparison of oedometer and triaxial data

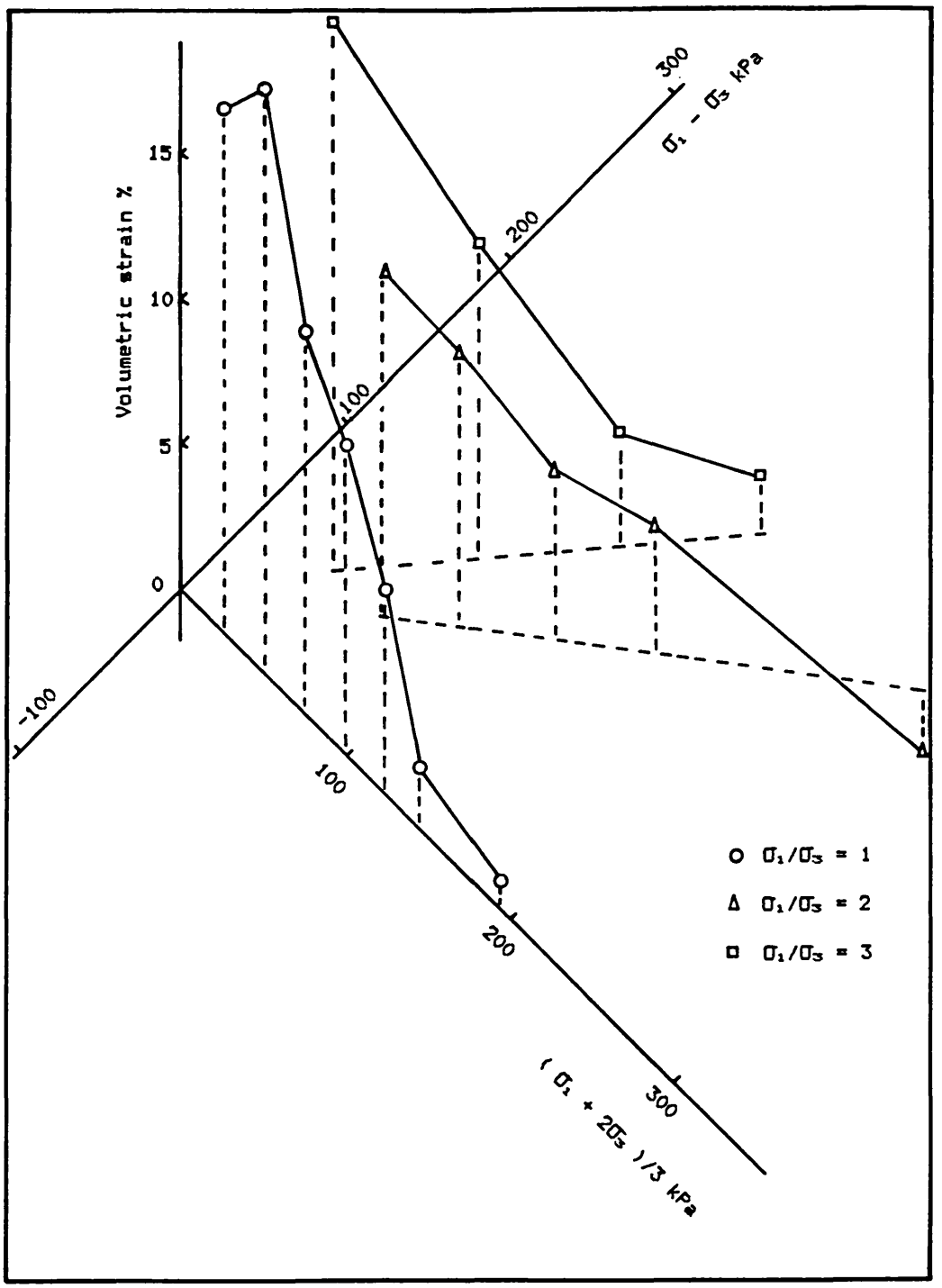
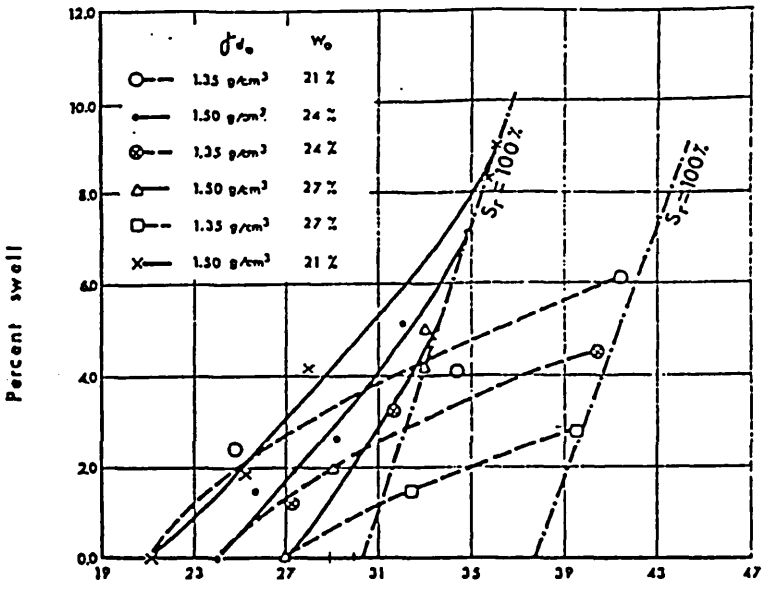
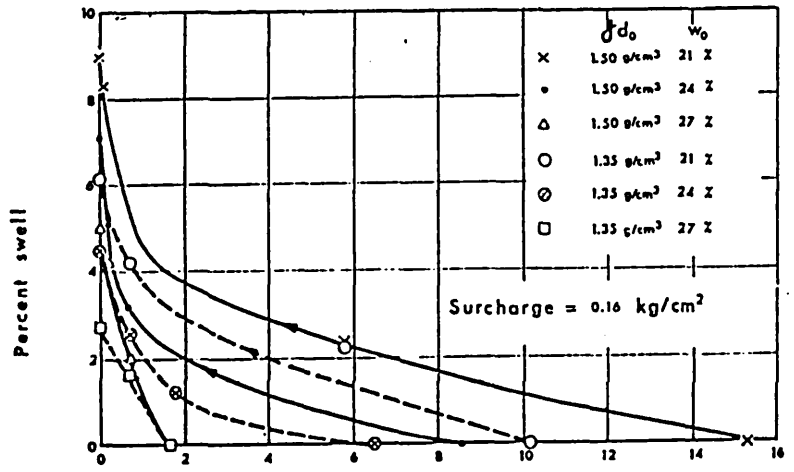


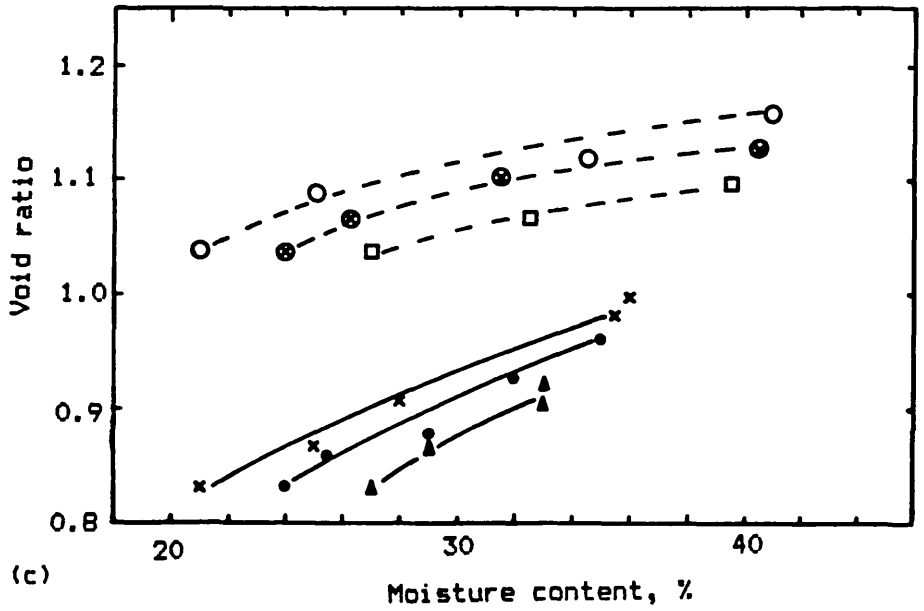
Fig.4.7 Volumetric strain versus mean stress and stress difference (data from Dakshanamurthy 1979)



(a)



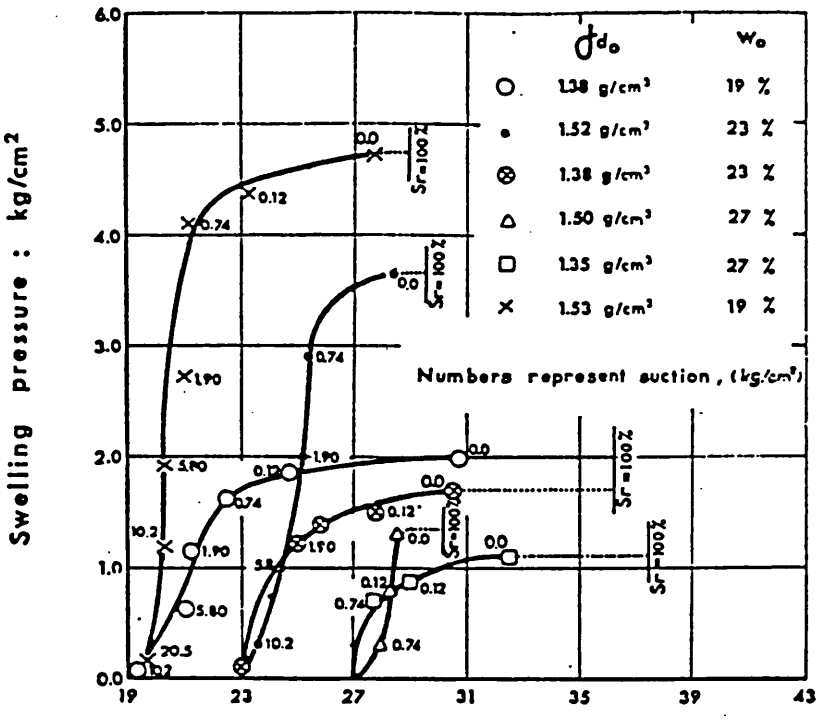
(b)



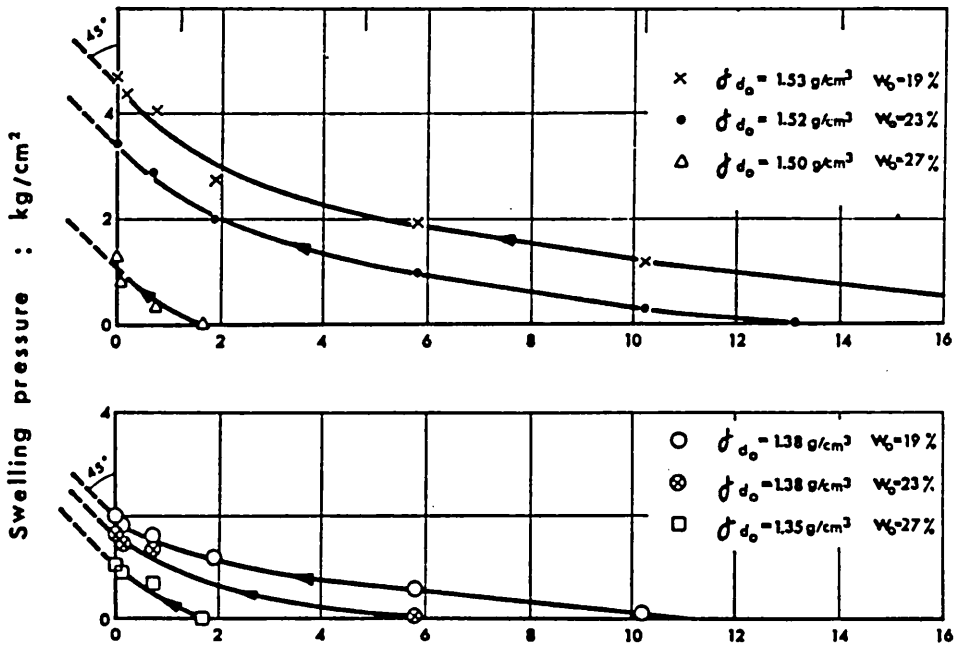
(c)

Fig.4.8 Swell tests

(Kassif & Ben Shalom 1971a)



(a) Moisture content after test ; %



(b) Osmotic pressure of solution = Equilibrium soil water suction; kg/cm²

Fig.4.9 Swell pressure tests

(Kassif & Ben Shalom 1971a)

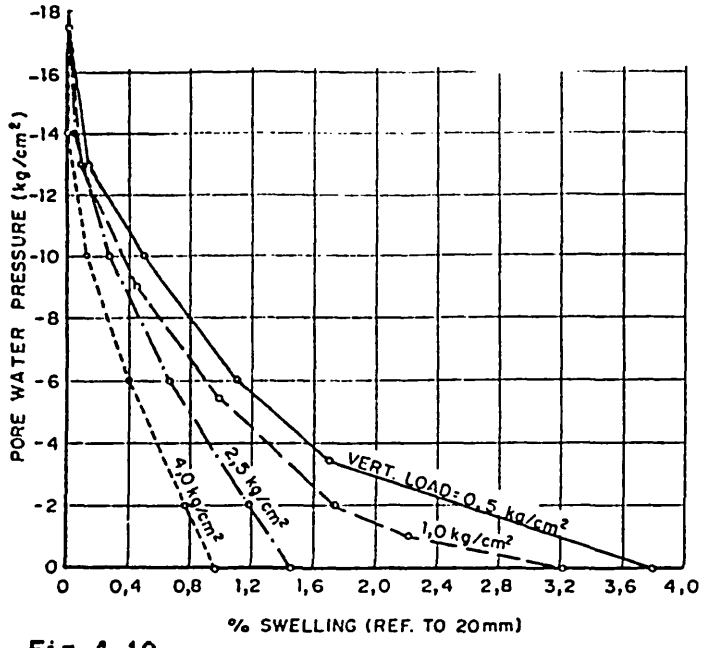


Fig.4.10 SWELLING UNDER CONSTANT LOAD AND CONTROLLED NEGATIVE PORE WATER PRESSURE OF A REMOLDED CLAY (Escario & Saez 1973)

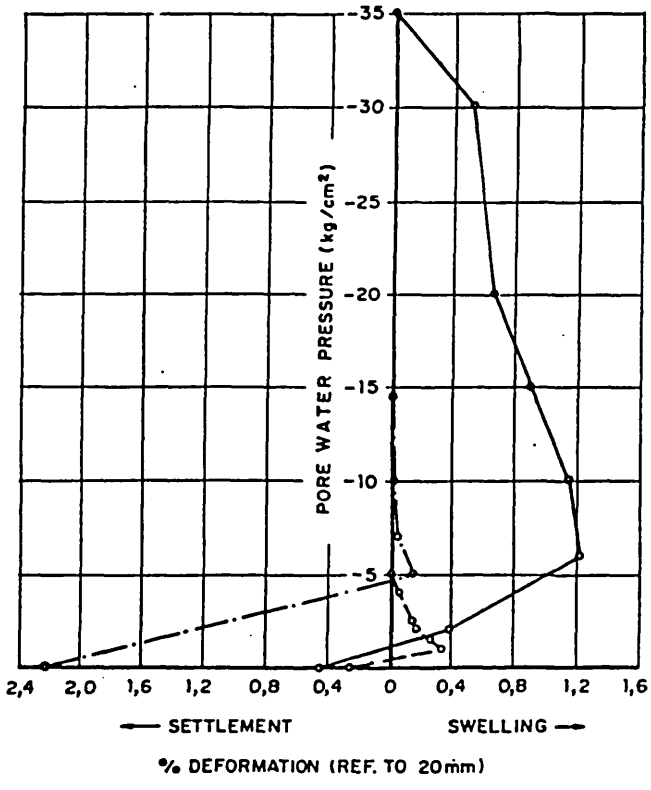


Fig.4.11 SWELLING FOLLOWED BY SETTLEMENT (GRADUAL COLLAPSE) UNDER CONSTANT LOAD AND CONTROLLED NEGATIVE PORE WATER PRESSURE OF A REMOLDED CLAY

(Escario & Saez 1973)

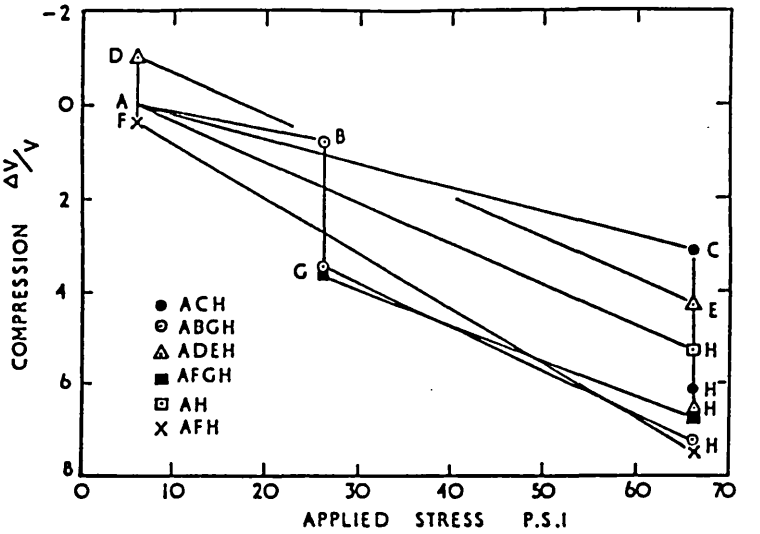


Fig.4.12 —COMPRESSIONS IN GROUP 5 TESTS
(Barden et al 1969)

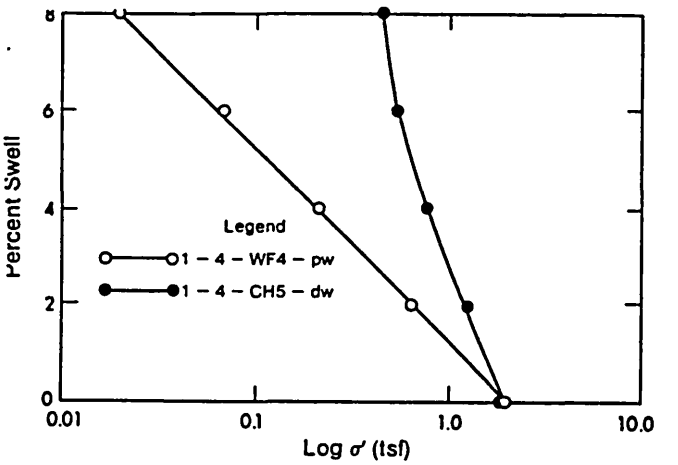


Fig.4.13 Swelling Tests on Bearpaw Shale
(Morgenstern & Balasubramonian 1980)

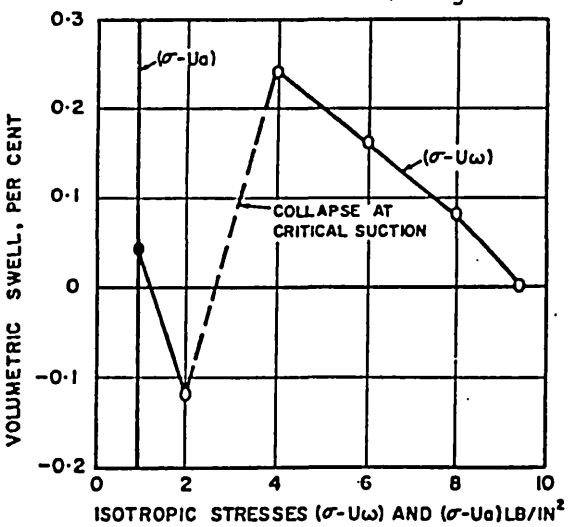


Fig.4.14.—Curve for the swelling of a partly saturated collapsing sand held under constant isotropic load ($\sigma - u_a$). Swelling took place under decreasing values of ($\sigma - u_w$) until a critical effective stress was reached whereupon collapse took place.

(Blight 1965)

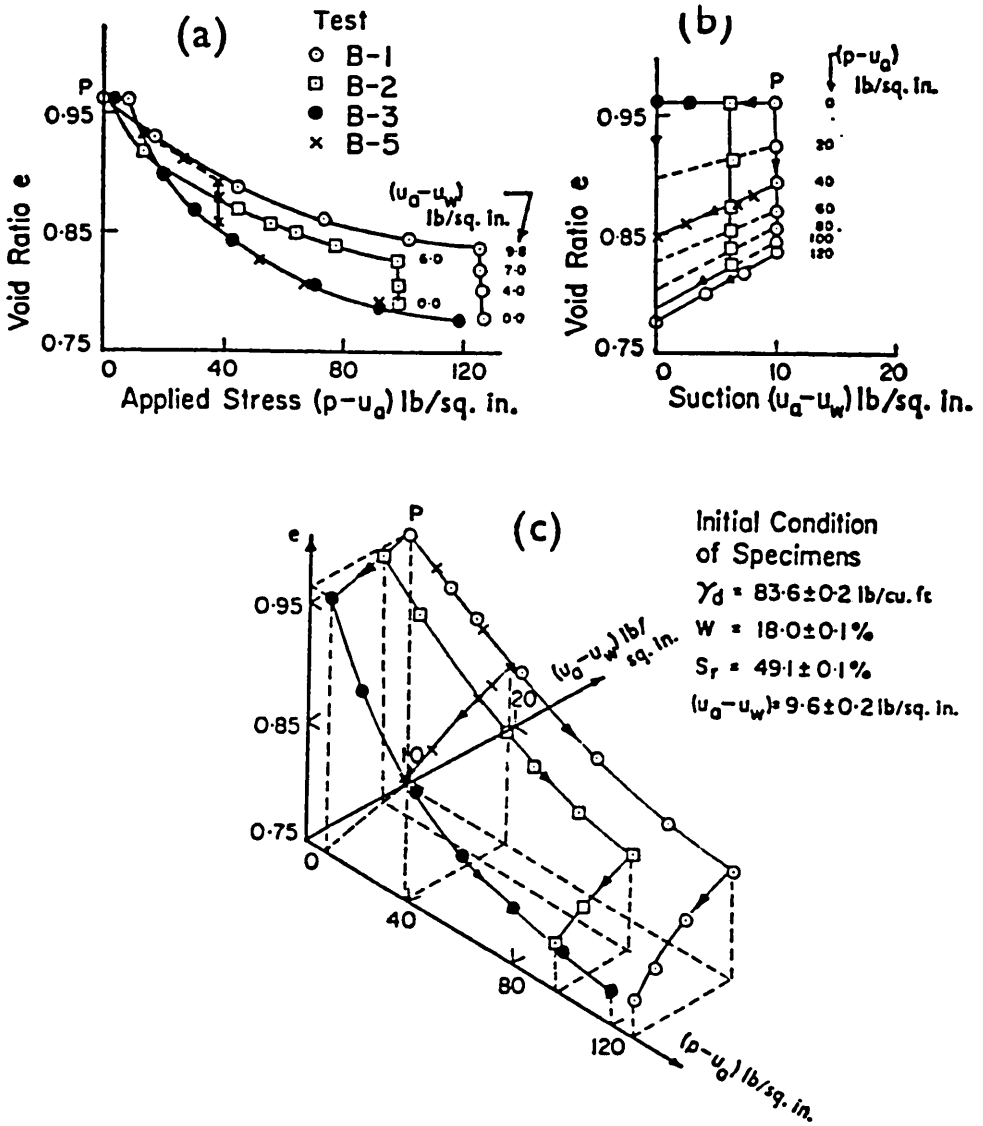


Fig.4.15 Compression due to increasing applied stress and decreasing suction

(Matyas & Radhakrishna 1968)

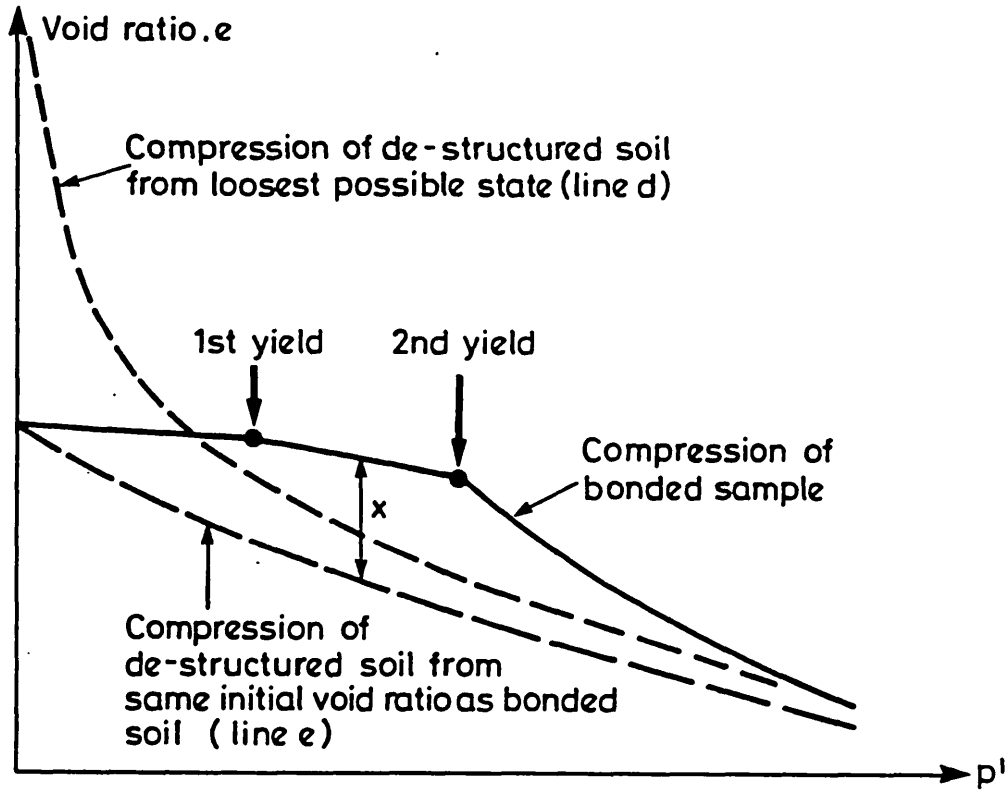


Fig.4.16 Model for compression of bonded soil
(Vaughan et al 1988)

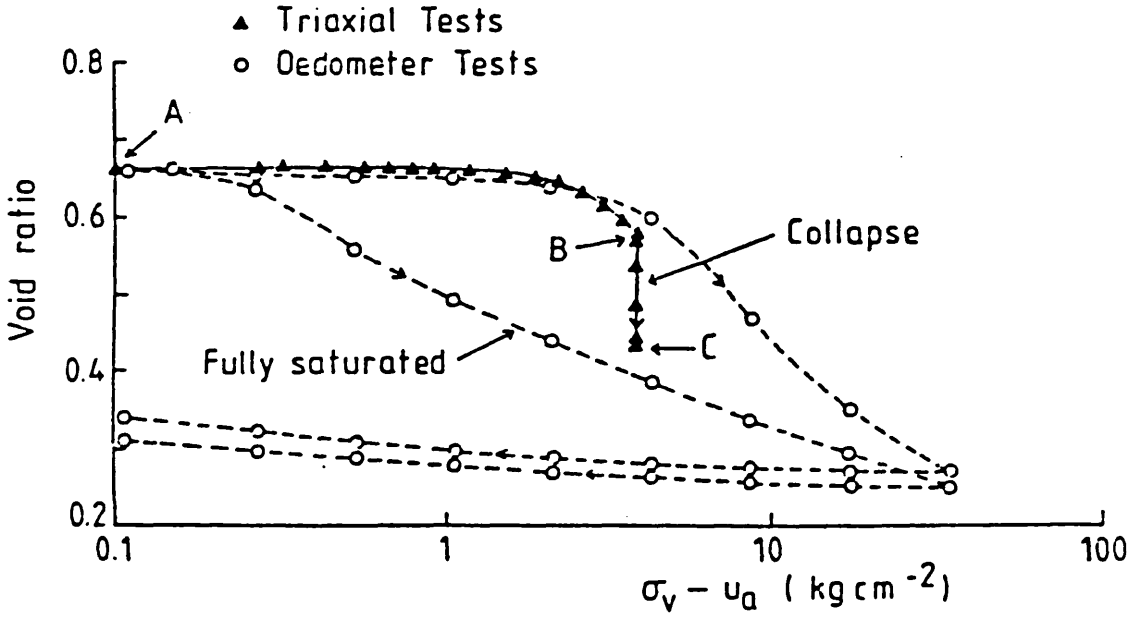


Fig.4.17 Comparison of triaxial test with equivalent oedometer tests on partly saturated and fully saturated soil.
(Burland 1985)

CHAPTER 5

SOIL PROPERTIES AND PREPARATION

5.1 INTRODUCTION

Samples of seven soils were collected for this project. The majority of them have only been used for classification purposes, although field data exist to demonstrate their expansive nature. The samples were collected on two separate expeditions, to Kenya in September 1985, and to Sudan in December 1985.

Samples were collected from four sites in Kenya. The sites are at Lodwar, Muhoroni, Nairobi-Mombasa Road at km68 and South Nyanza. The Lodwar and Nairobi-Mombasa Road sites are linked with research projects undertaken by the Transport and Road Research Laboratory, the former being one where use of dry compaction is planned. The other two samples were collected on the basis of the Agricultural soil maps of Kenya and local information. The Muhoroni sample was taken near a clinic which showed some evidence of movement due either to swelling or shrinkage of the soil. The structure had been designed to survive the foundation movements, and damage was generally limited to the surrounding aprons. Damage was also apparent near the Nairobi-Mombasa Road site where a culvert passed under the road. The culvert and the wing walls were extensively cracked and distorted. Damage to the road could not be assessed as it had recently been rehabilitated.

Samples were collected from sites at El Fau, Wad Medani and Omdurman in the Sudan. The samples from Wad Medani and El Fau have been extensively studied by Elsharief (1987) in their natural state. Both sites are near severely damaged buildings, one of which is described in Schreiner (1987g). The sample from Omdurman was taken near a house which was damaged along the line of a service trench by what appeared to be swelling. Other damage visible in the area included a crack up to 30mm wide through the middle of a single story shop. This crack was widest at the top, which is commonly associated with swell under the centre of a building.

5.2 SOIL CLASSIFICATION

5.2.1 FIELD CLASSIFICATION

The samples were taken from trial pits which had been dug by hand. The trial pits were logged by the author on the day on which

they were dug. The logs are shown in Fig.5.1 to Fig.5.7, and sample positions are noted. Common to all the profiles from areas which were not very wet are the vertical cracks. These were visible on the ground surface in the areas which were not heavily vegetated. In the Lodwar area the cracks were up to 25mm wide in some locations within the area of black clay soil. The cause of these locally more severely cracked zones was not determined, but it appeared to be linked with preferential infiltration of runoff. This may have reduced the salt content (see section 5.2.2.2 below) in these areas by leaching and thereby made the soil more expansive.

5.2.2 LABORATORY DATA

A summary of the laboratory classification test data is presented in Table 5.1. Additional data for the Nairobi-Mombasa Road site is given in Fig.5.8 from Mutonyi (1973).

5.2.2.1 Mineralogy

The clay minerals present were determined using X-ray diffraction techniques at University College (UCL 1986a & b). Semi-quantitative analyses were made by preparing standard mixtures of montmorillonite and kaolin to cover the range of proportions present in the natural samples. The proportion of each mineral present in the fraction of the soil finer than 0.002mm was estimated from the height of the peaks on the X-ray spectrum. This is a simple method which may be inaccurate, particularly when the widths of the peaks vary. It does serve to indicate that the dominant mineral in all of the samples is montmorillonite.

A parallel study was made using an Infra-red spectrometer (UCL 1988a & b). This permits much faster measurements to be made, typically about 5 minutes compared to 5 hours for the X-ray diffractometer, although sample preparation requires as much time. The results are qualitative at present, but progress has been made within this project in the technique.

The first spectra were not able to separate illites from montmorillonites. This appears to have been overcome by measuring the spectra before and after heating to 550°C. The cold spectrum differentiates illite and montmorillonite together from kaolin, and the spectrum from the heated sample separates the illite from the montmorillonite. The illite content of these samples is not measurable, and there is general agreement between the findings of the X-ray and Infra-red studies.

It has become clear during this work that both of these techniques are dependent on appropriate standard minerals being available. Since these cannot be prepared artificially, but are refined samples of natural soils, the standards cannot be guaranteed to be pure samples of one mineral. This reduces the

level of confidence in the measurements of the mineral types and of the relative proportions.

5.2.2.2 Salt content

The sample from Lodwar was noted in the field to taste of salt. Since solute suction may influence the amount of swell, it was decided to measure the salt content of the soils. Investigation of the standard methods (Hesse 1971) showed that they were unscientific and frequently based on unsubstantiated assumptions. The method based on electrical conductivity for instance relies on a conductivity relationship derived for an unspecified salt or mix of salts obtained from one soil sample. There is no means of identifying the salts in the solution used for the "standard" relationship and thus no means of correcting the results for different salts which are likely to be present in the soils of interest.

It was decided that removal of the salts by flushing with water and evaporation of the water would give results that would be at least as useful, and would avoid the use of inappropriate relationships. The leaching equipment and technique have been described in Schreiner (1987c), which is included below as Appendix 8A.1 to Chapter 8 . The results are given in Table 5.1.

The technique should be refined further. Determination of the salt content of the effluent could be better done by measuring the conductivity or resistance as is reported in Chapter 10. This would provide a quantitative measurement which could be plotted against time to indicate the efficiency of the leaching. The water used in the tests was tap water, which is not in itself salt free. The quantity of disolved salt is small, and a test on a similar amount of water as had been flushed through the soil samples indicate a possible error of about 0.1% salt content in the soil. For accurate determinations distilled water should be used.

5.2.2.3 Salt Type

One salt sample was obtained from a sample of the soil from the Nairobi-Mombasa Road site, and was analysed by X-ray diffraction to determine the cations and anions present. The sample was obtained from the flushing system of the stress path oedometer which is described in Chapter 9. The sample was taken after test SP01 which is described in section 10.4. Since the flushing system was initially filled with distilled water, there should be no contamination from external sources. Diffusion of salts from the soil water into the water of the flushing system is discussed in section 10.5. After completion of the test a part of the water in the flushing system was drained into clean sample tubes.

The dried residue was analysed by X-ray diffraction (UCL

1988c) and the major constituent was found to be Sodium Chloride (NaCl) with small quantities of Sodium Nitrate ($NaNo_3$) and Sulphate (Na_2SO_4) and Potassium Bisulphate ($KHSO_4$).

Included in Schreiner (1987c), which is included as Appendix 8A.1 to Chapter 8, are calculated solute suctions for the soils based on the measured salt contents. These values are much lower than the solute suctions obtained from the matrix and total suction measurements reported in Chapter 8. The calculations did not make any allowance for the distribution of the salts within the soil water, and assumed that the salts were uniformly distributed in the soil water.

However, in the same paper a model of the distribution of the exchangeable cations and the dissolved salts is described. Applying this qualitative model, it is clear that the distribution of the dissolved salts will be far from uniform throughout the soil water. The effect on the solute suction will be much greater than was calculated because the dissolved salts will be found in only a part of the soil water.

Assuming that the scientific relationship for suction from salt content given in Schreiner (1987c) is valid, it is possible to calculate the salt concentration required to establish the solute suction derived from the measured matrix and total suction. This in turn gives the amount of water in which the dissolved salts are to be found. Performing these calculations for 35% moisture content, and assuming parallel particle orientation, for the Nairobi-Mombasa Road sample, for which the relevant total and matrix suction measurements have been made, the dissolved salts are found to be in 25% of the soil water.

5.3 SAMPLE PREPARATION

It was decided that uniformity of samples would be enhanced if the soils were first pulverised and then mixed to the desired moisture content. In order to pulverise the soils they need to be dry enough to be brittle and non-sticky. The sample from Wad Medani was dry enough to be pulverised in its field condition, but the soil from the Nairobi-Mombasa Road site and from the South Nyanza site were too wet for pulverising. Accordingly they were dried at about 70°C to moisture contents of 6% and 3.5% respectively. The soils were then pulverised in a mechanical pulveriser to pass the 2mm sieve. The Nairobi-Mombasa Road sample contained a small proportion of gravel, some up to 25mm, and most of this was removed during this process. Some was broken down to sand size. Since the pulveriser could only hold about 1kg of soil, several batches were used. These were subsequently mixed in a large bag and stored as bulk samples.

Samples for testing were made up from the bulk samples. Sufficient sample was mixed up for the whole of each test programme. This ensured that within each programme the samples

would be comparable. The required quantity of distilled water was calculated from the initial moisture content and the desired final moisture content. Mixing was done in a large tray and the water was added in small quantities by spraying from a wash bottle during continuous mixing. The samples were then stored inside three sealed plastic bags. Moisture contents were taken at the start of most tests, and showed no significant variation with time during the test programme. A set of check moisture content tests were made on the first batch of samples after about 12months, and 6months after the planned testing on these samples had been completed, and two of the four samples tested were found to be at the original moisture content.

SITE	DEPTH	W_L	W_P	SALT CONTENT %	MINERALOGY OF CLAY M%	K%	ORGANIC CONTENT OF CLAY
LODWAR							
	300-500				100	0	11
	600-800	94	39	1.14	94	6	11
	600E				70	30	15
	800-1.0	102	40		98	2	15
	1.1				98	2	13
MUHORONI							
	600-800	120	42	<0.1	95	5	13
NAIROBI-MOMBASA RD							
	200-350				100	0	21
	350-500				100	0	12
	500-650				100	0	12
	650-850	125	51	0.52	100	0	12
	850-950				100	0	8
	950-1.05				100	0	11
	1.05-1.3				97	3	21
SOUTH NYANZA							
	500-700	70	27	<0.1	97	3	15
	900-1.0				91	9	4
EL-FAU							
	600-800	71	37		85	15	11
OMDURMAN							
	500-700				78	22	6
WAD-MEDANI							
	600-900	64	29		83	17	9

TABLE 5.1 SUMMARY OF SOIL CLASSIFICATION DATA

LODWAR - KAKUMAR ROAD

TRIAL PIT AT KM81
9 Sept 1985

bag	120	dry grey-brown slightly sandy silty clay with occasional fine gravel in profile has a loose granular structure
bag	550	dry grey-brown friable fissured to shattered slightly sandy silty clay more sandy than above stratum
bag	800	dry dark grey friable fissured slightly sandy silty clay stiff to hard in lumps occasional slickensides not well developed nor polished
bag bag	1100	slightly moist dark grey very stiff to hard silty clay with occasional grains of sand and fine gravel occasional slickensides
		SOUTH FACE ONLY 900 - 1100 slightly moist to dry mottled grey, brown and light brown fissured to shattered gravelly clay gravel comprises fine to coarse quartzitic and fine reddish rounded to subrounded grains possible weathered rock

NOTES

1. coarse to fine calcrete nodules at 850
2. sand filled vertical crack over depth of trial pit
   up to 10mm wide, brown sand
   adjacent crack partly open and at 30° to vertical
3. roots visible to about 600mm
4. possibly residual overlying weathered rock
5. ground surface powdery with some open cracks up to 25mm wide
6. vegetation: grass with some small trees

Fig.5.1 Trial pit profile

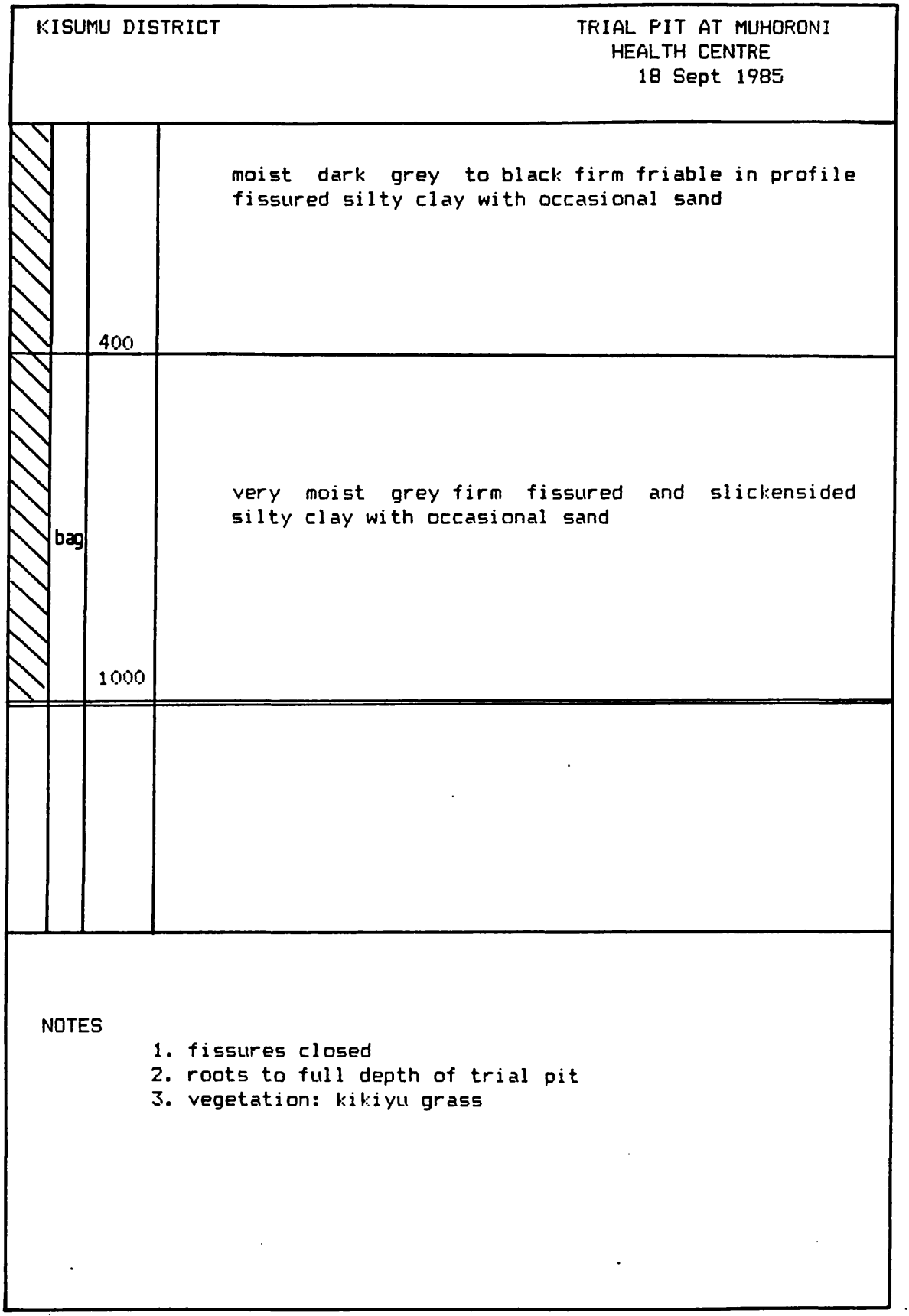


Fig.5.2 Trial pit profile

NAIROBI-MOMBASA ROAD

TRIAL PIT KM 68.3
20 Sept 1985

bag 200	dry very dark grey to black fissured and shattered hard slightly silty clay with some sand and gravel fissures up to 20mm wide	
bag bag bag bag bag 950	moist very dark grey to black fissured and shattered hard to stiff slightly silty clay with occasional fine to coarse gravel fissures up to 10mm wide at 400mm depth poorly developed slickensides	
bag bag 1300	slightly moist grey mottled white and dark grey intact stiff sandy very silty clay possibly residual weathered rock becoming lighter grey with depth	

NOTES

1. fissures open to 950mm
2. roots visible to 1200mm
3. vegetation: grass

Fig.5.3 Trial pit profile

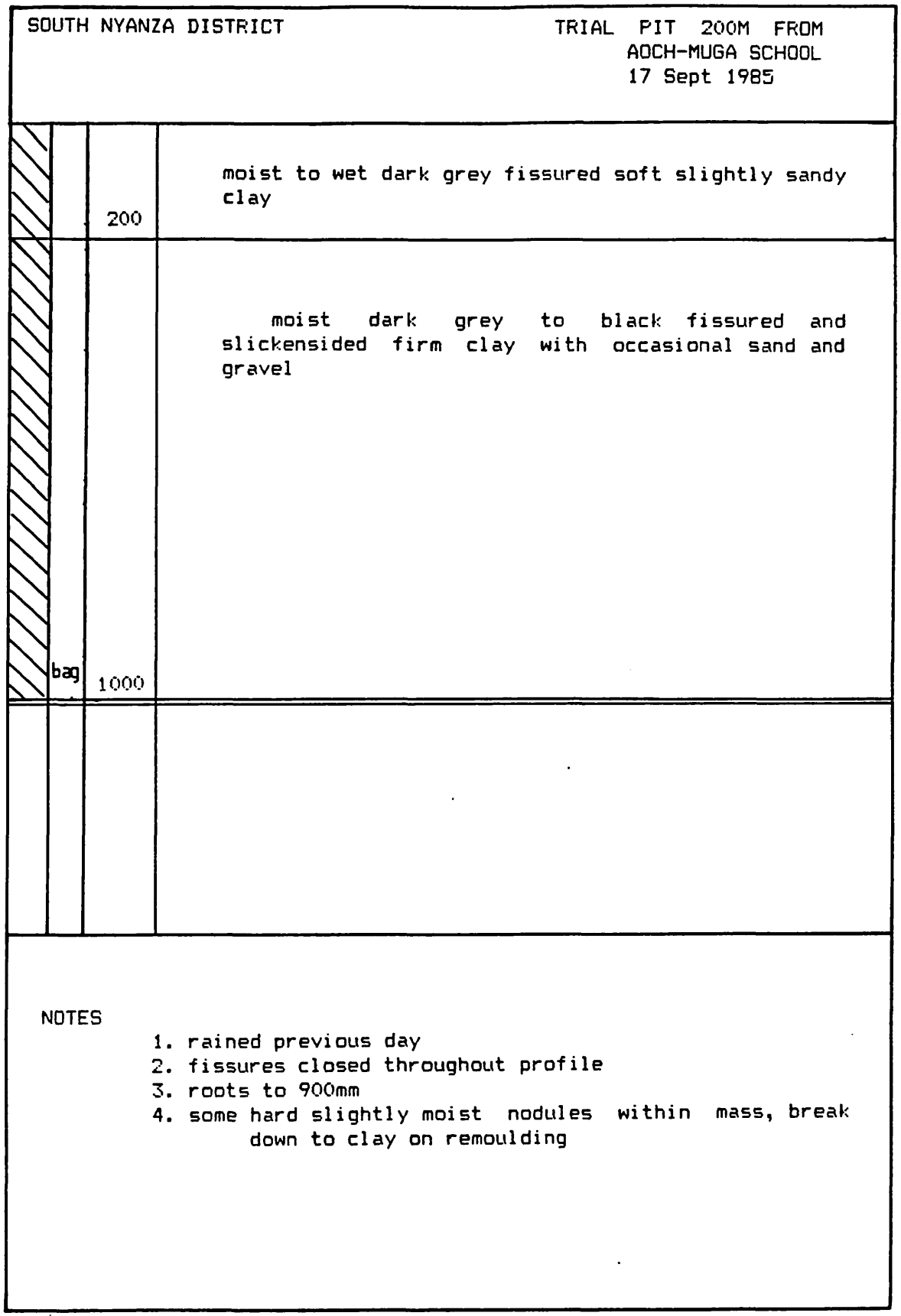


Fig.5.4 Trial pit profile

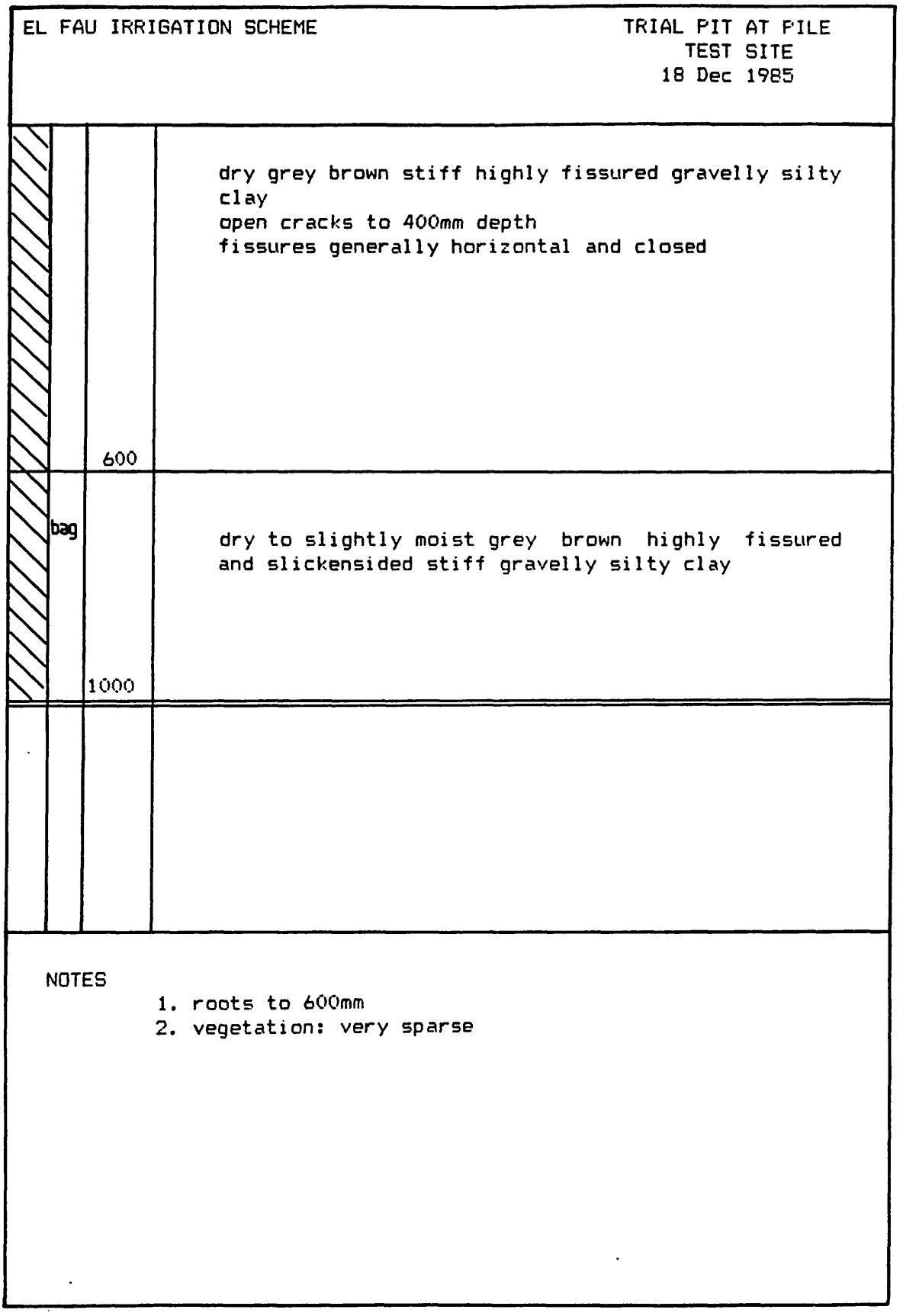


Fig.5.5 Trial pit profile

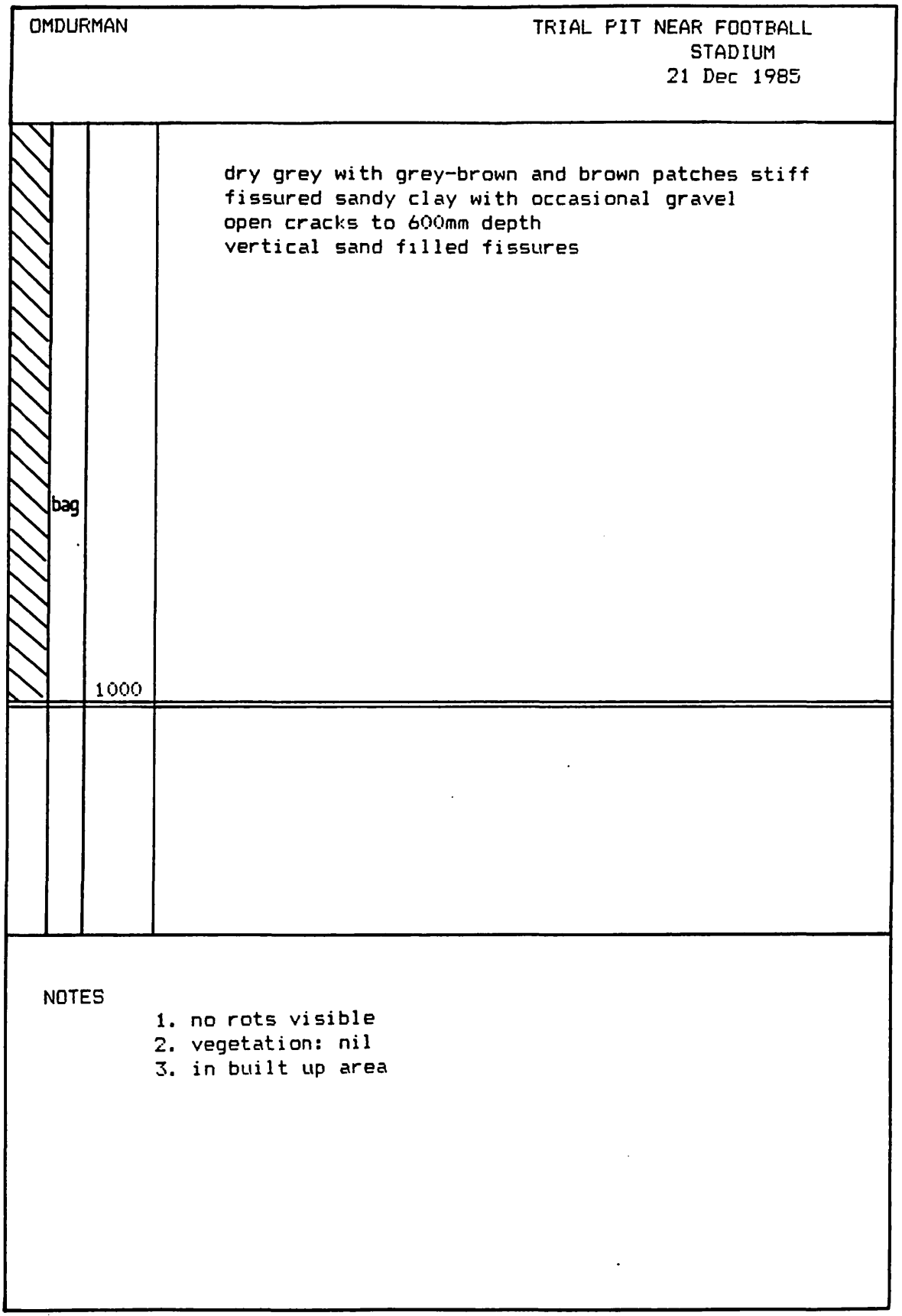


Fig.5.6 Trial pit profile

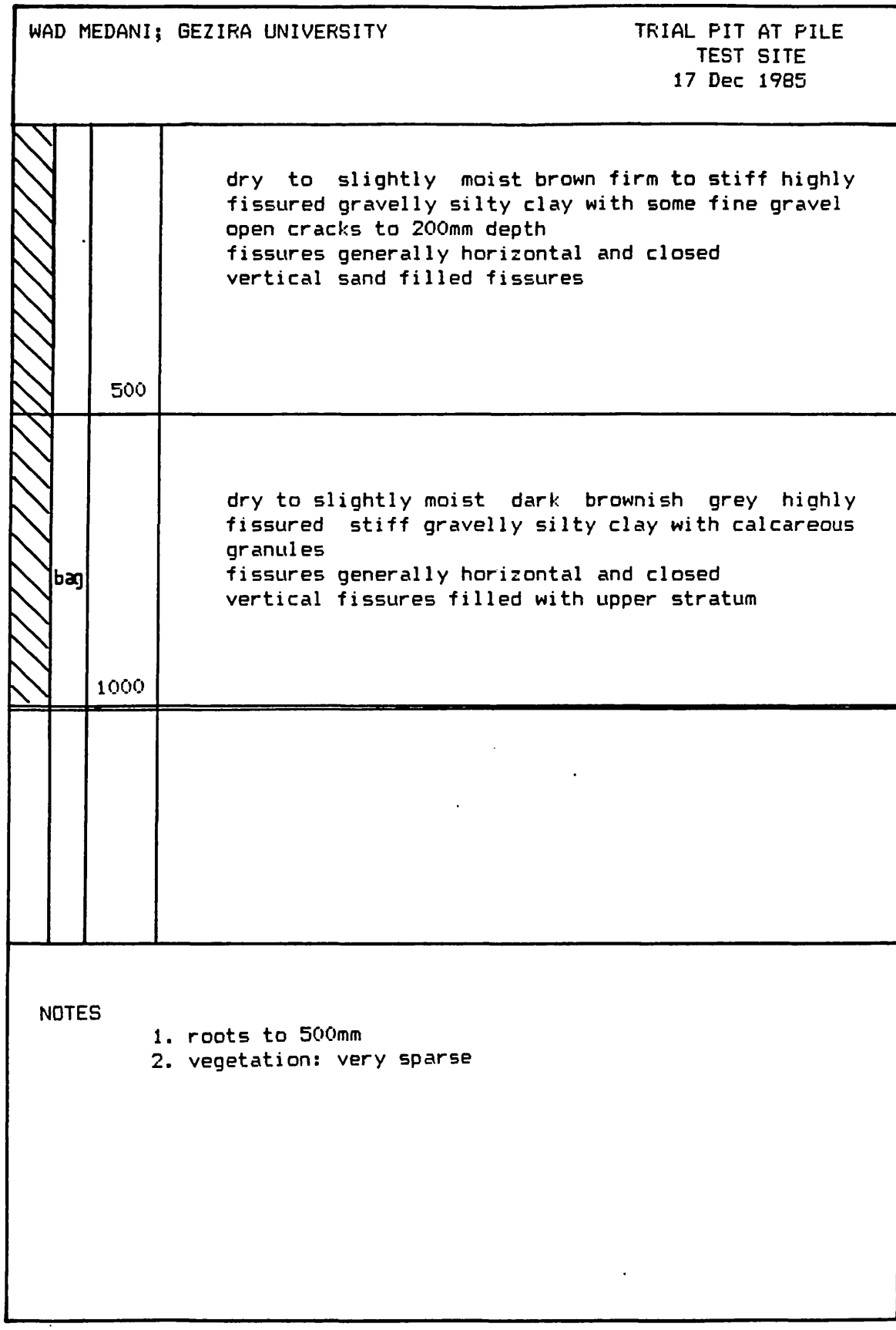


Fig.5.7 Trial pit profile

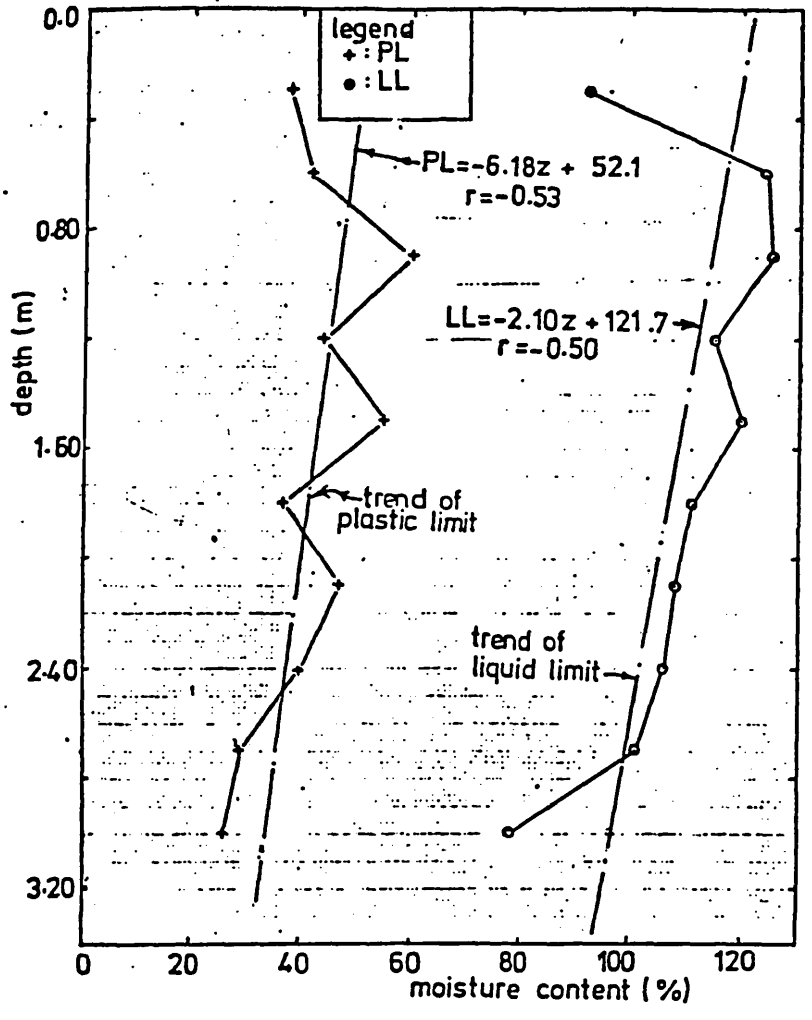


Fig.5.8 Variations of Atterberg Limits with depth, Nairobi - Mombasa Road site
(Mutonyi 1973)

CHAPTER 6

IDENTIFICATION OF EXPANSIVE SOILS

6.1 INTRODUCTION

Classification test data are used in expansive soil engineering both to assess the expansiveness of the soil and to estimate the magnitude of swell that may occur. Use of classification test data for the latter purpose requires that the stress state and stress changes are either compatible with the derivation of the predictive equation or are included in the predictive equation. Derivation of a means of determining the intrinsic expansiveness can only be achieved if the effects of stress state, stress changes and initial density and soil fabric can be eliminated or isolated.

The limitations within which any method based on classification test data can be applied have been defined by Schreiner (1987b). This paper is included as Appendix 6A.1 to this Chapter.

Following on from this review, during which it had become apparent that there was little consistency between methods, a study was undertaken as an MSc. project by Mr. S. Oloo under the author's supervision (Oloo 1987). The intention was to determine, from the available published data, which classification data were most strongly related to the intrinsic expansiveness and then to derive a method of prediction of swell. The results of the former showed that there was little point in pursuing the latter. The interested reader is referred to the original document for the detailed report of the study. The key findings, that both free swell and index test data are unreliable indicators of expansiveness, are discussed by Oloo et al (1987) which is included as Appendix 6A.2 to this Chapter. The use of shrinkage limit, liquid limit, plastic limit and clay fraction together appears to be better than the other combinations of data which were used, but the quantity of data available is too small for this to be recommended as a universal method.

The approach followed in the original work and in Oloo et al (1987) with regard to the free swell sedimentation test is inconclusive. In effect it merely demonstrates that free swell is poorly, if at all, linked to index test data, because the free swell data was only compared with index test data and not with oedometer swell data. The latter could not be done because there is no data available in which both types of test have been

performed on the same soil. A better approach, although still an indirect one, is to plot both free swell data and oedometer swell data against classification data for such data as is available.

Fig.6.1 repeats the data from Fig.6 of Oloo et al (1987) without the tentative swell boundaries. Shown instead are boundaries within which the data for a given range of swell fall. Shrinkage index and liquid limit were chosen for this comparison because the oedometer data are moderately well separated in terms of these parameters. Plasticity index and liquid limit by comparison showed a much poorer classification in Fig.4 of Oloo et al (1987).

Fig.6.2 repeats the free swell data against shrinkage index and liquid limit from Fig7.3 of Oloo (1987), with boundaries within which the commonly used ranges of free swell fall.

Fig.6.3 compares the boundaries from Fig.6.1 and Fig.6.2, without the data points. According to the common useage of free swell for prediction of expansiveness, 0 to 50% free swell corresponds to low expansiveness, 50 to 100% to moderate expansiveness and greater than 100% free swell with high expansiveness. Thus, for free swell to correlate with the observed swell ranges, the boundaries in Fig.6.3a for the low, moderate and high swell data should be similar to the boundaries in Fig.6.3b for 0 to 50%, 50 to 100% and greater than 100% free swell. It is clear that there is very little similarity between the boundaries, indicating that there is no link or correlation between free swell and oedometer swell.

6.2 SUMMARY

While the conclusions from these two studies are far less useful than was intended, they have highlighted the unreliability of methods of swell and intrinsic expansiveness prediction which use classification test data. Further work, beyond the scope of this project, is required to find a useable correlation between classification test data and intrinsic expansiveness.

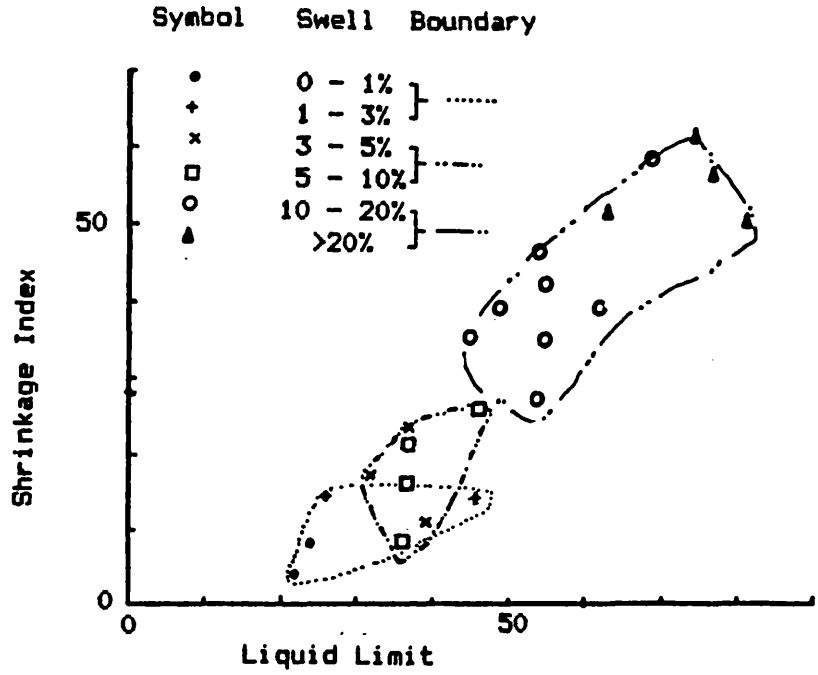


Fig.6.1 Swell vs Shrinkage Index & Liquid Limit for 100% Std. AASHO dry density

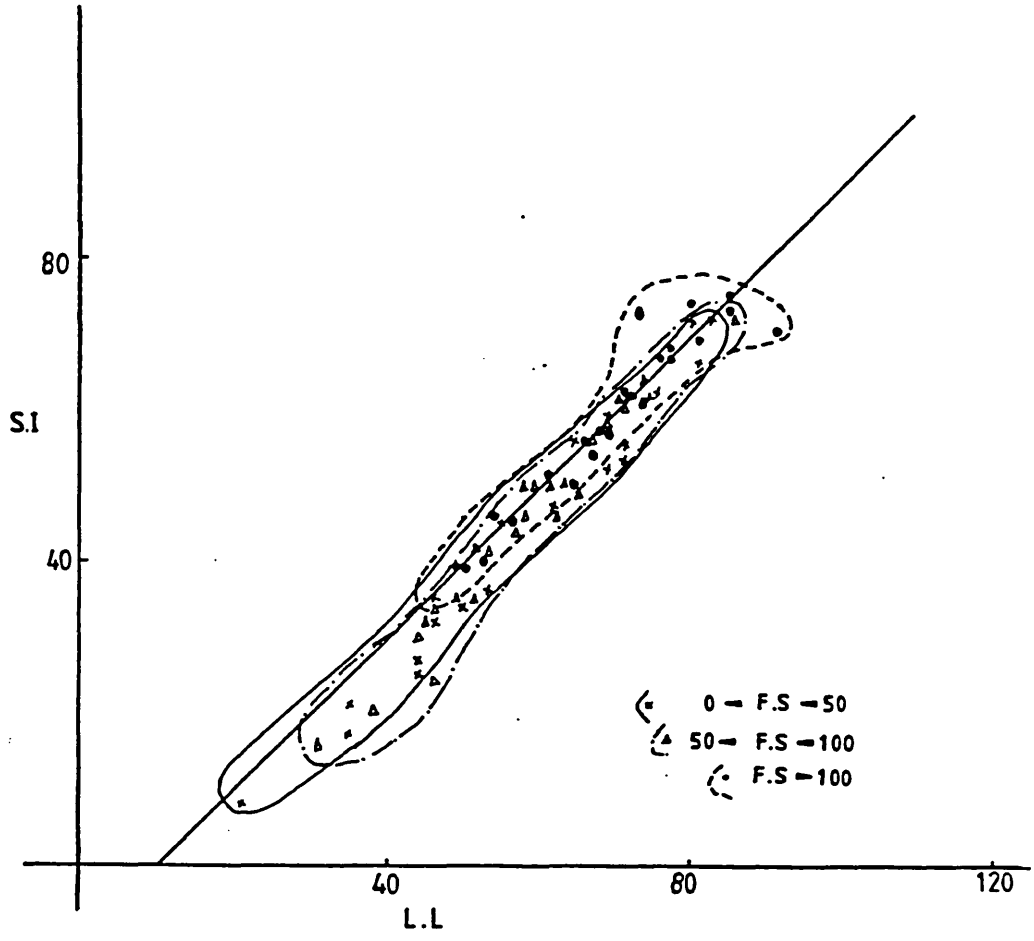
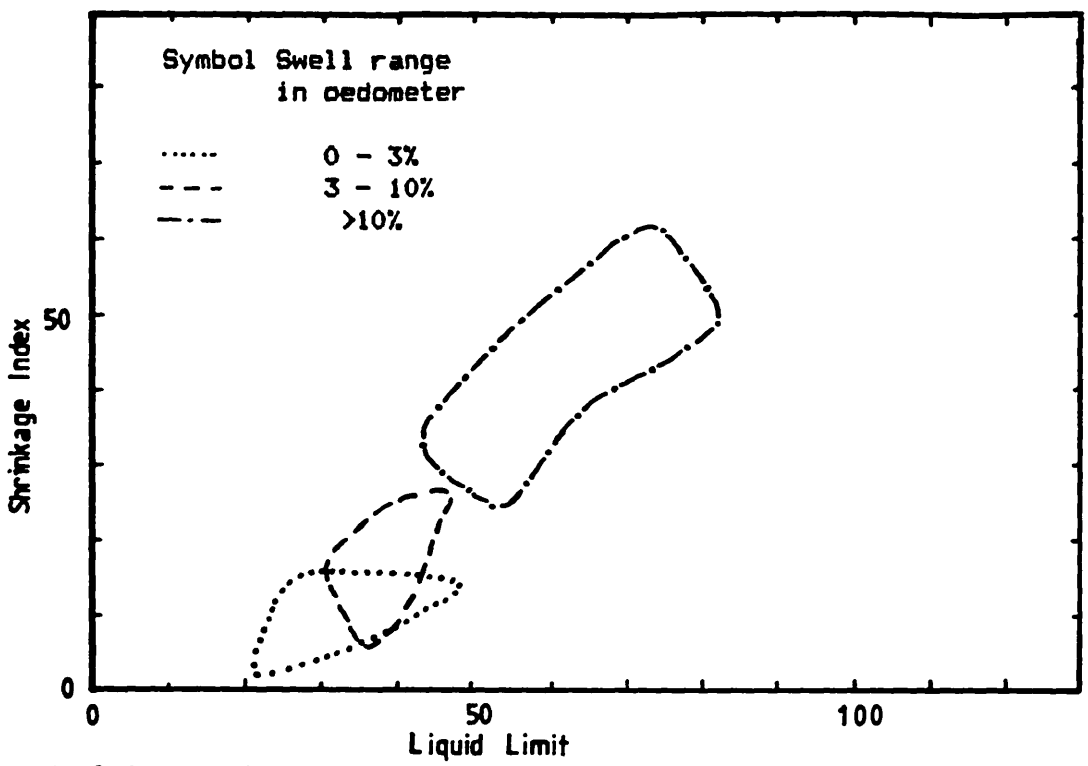
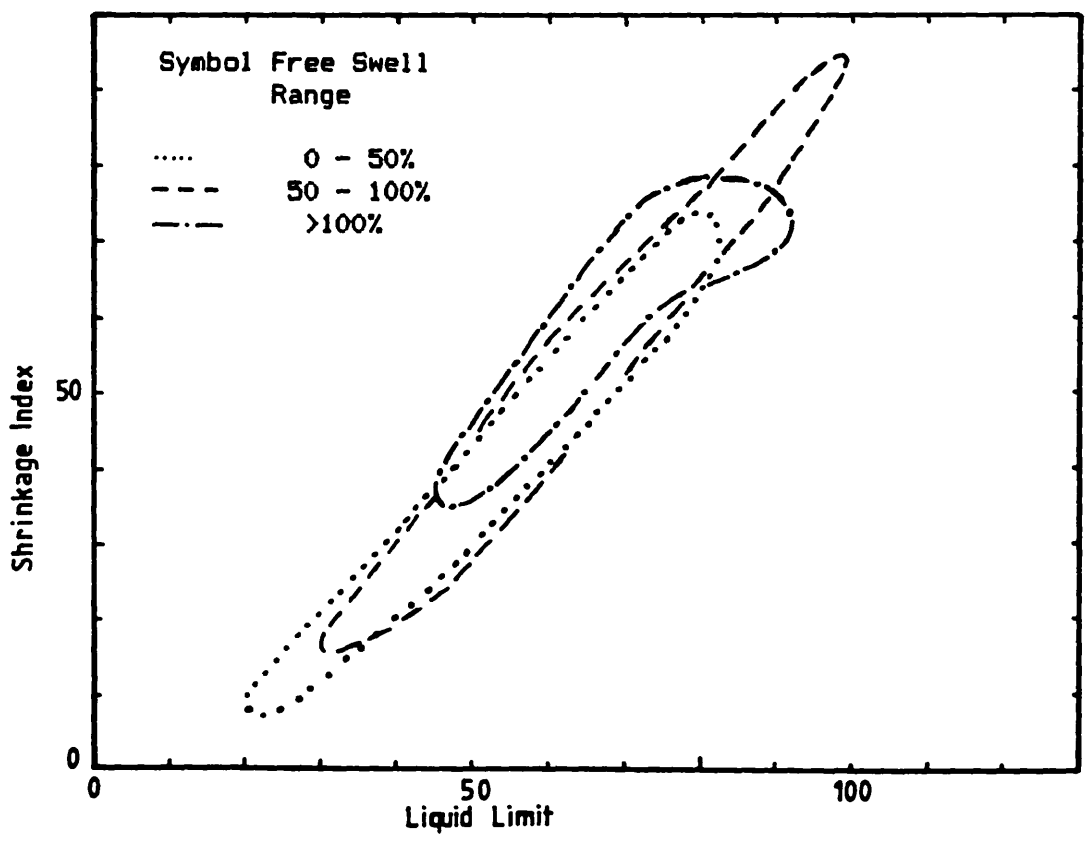


Fig.6.2 Relationship between Shrinkage Index, Liquid Limit and Free Swell



(a) Oedometer data



(b) Free Swell data

Fig.6.3 Comparison between Free Swell and Oedometer Swell Data

APPENDIX 6A.1

9th Regional Conference for Africa on Soil Mechanics and Foundation Engineering / Lagos / September 1987

The use of predictive methods in expansive soil engineering

H.D.Schreiner
Imperial College, London, UK

ABSTRACT: Many predictive methods have been published for use in expansive soil engineering. These are frequently inappropriately applied. Distinction is made between methods that can only be correctly used for assessment of soil properties and those that can be used for estimation of strains or displacements. Intrinsic expansiveness, swell and heave are defined in terms that will clarify the relation between the predictive methods and the swelling process. Guidelines are given for the selection of a suitable predictive method for any set of practical conditions.

1 INTRODUCTION

One of the topics frequently addressed in the study of expansive soils is the prediction of volume change from classification data. The list of predictive methods is not quite endless, but a very large number have been published. This paper does not intend to review a large number of methods with intent to find a universally acceptable one, but is rather aimed at identifying the reasons why most of these methods are of relatively little value except under very specific stress and suction or water content conditions. Attempts to apply the methods under conditions different from those for which they have been developed will almost always lead to false predictions. This does not necessarily mean that the method itself is incorrect, it has simply been inappropriately applied.

2 DEFINITION OF CONCEPTS

It may help in the understanding of the processes involved in swelling of expansive soils to define a few basic concepts before discussing the predictive methods. The most important concepts are the intrinsic expansiveness which is a soil property, swell which is a strain, and heave which is a displacement.

2.1 Intrinsic Expansiveness

Volume change of a clay soil due to changes in water content or suction is caused by interaction between the clay minerals and water. The interaction is the basis of the Gouy double layer theory which has been extensively used and discussed in the literature. If the suction exerted on water in a soil is increased, water will move out of the soil under this stress and the soil will decrease in volume. Conversely, if water is added to the system, the suction will decrease and the soil will increase in volume. The intrinsic expansiveness is the soil property that relates the change in water content, and thus change in volume, to the suction change. A soil with a high intrinsic expansiveness will exhibit a large water content or volume change as compared with a soil of low intrinsic expansiveness when both are subjected to the same

change of suction under initial identical stress, suction, soil structure and stress history conditions.

This concept is not new, but was mentioned by Brink (1955) and was used by Baikoff and Burke (1965). No standard procedure has yet been developed for the measurement of this property, and it is only useable at present as a qualitative description.

2.2 Swell

Swell is a measure of the volume strain or axial strain in a soil under a particular set of stress and suction conditions. It can occur either due to changes in suction or due to changes in applied load. In laboratory testing of intrinsically expansive soils, swell is usually induced by a reduction of suction under a small but constant vertical applied stress and zero lateral strain in an oedometer. Most testing is performed with an unknown initial suction, a final matrix suction close to zero, and unknown lateral stress.

Swell is not a soil property, but is a strain which results from interaction between suction and stress and the soil property intrinsic expansiveness described above. Swell is thus specific to the stress and suction conditions under which it is measured, and will probably also be affected by suction and stress history.

2.3 Heave

Heave is the displacement of a point in the soil due to suction and stress changes interacting with the intrinsic expansiveness. The relevance of laboratory swell measurements to heave under field conditions will depend on how well we can model, in the laboratory, the initial stress and suction states in the field and their changes.

Heave is not a soil property, but is a displacement which results from interaction between the soil property (intrinsic expansiveness) and the field stress and suction changes that we may or may not be able to quantify with a reasonable degree of confidence.

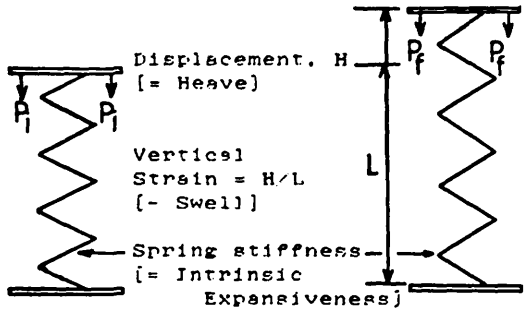


Fig.1 Mechanical analogy

2.4 Classification Data

Classification tests such as Atterberg limit tests are not absolute measures of any soil property. However, because they are performed under identical conditions and procedures, they are an indirect measure of a soil property of the reconstituted soil, or, more probably, the combined effects of several properties.

2.5 Mechanical Analogy

The swelling process under decreasing suction may be visualised in terms of a linear spring system shown schematically in Fig.1. The displacement of the end of the spring (heave) is the summation of the strains (swell) caused by the change in applied load (suction) acting against the spring stiffness (intrinsic expansiveness). Regardless of the load (suction), the spring stiffness (intrinsic expansiveness) will be unchanged. In practice the system will be non-linear, but similar analogies may be used.

3 APPLICABILITY OF PREDICTIVE METHODS

Predictive methods range from those based solely on classification data to those that include classification data, stress, density and water

content. Some have been derived for compacted soils, others for undisturbed samples. The information that can be obtained from these methods varies accordingly, as do the conditions necessary for their correct usage. A limited number of predictive methods will be used in the following discussion. This does not imply that these are particularly good or bad methods. They have been chosen merely as examples for which the type of parameter that can be predicted and the range of correct usage can be demonstrated. Similar assessments should be made for any predictive method before it is applied in practice.

3.1 Methods using classification data

As discussed in section 2 above, the classification data for a soil are an indirect measure of one or more soil properties. They do not in any way account for the stress and suction state in a desiccated soil. In principle, therefore, classification data are only of value in assessing a soil property, such as intrinsic expansiveness, and are of no value in assessing a strain due to stress and suction changes.

The approach followed by Dakshanamurthy & Raman (1973) is in agreement with these comments. They propose a classification of the intrinsic expansiveness of a soil in qualitative terms, based solely on the Liquid Limit. They make no attempt to predict actual or potential swell or heave. The use of this type of predictive method will assist in the identification of soils which will or will not be subject to significant volume change if the stress and suction conditions are appropriate. A similar type of predictive method has been published by Tadanier & Nguyen (1984) who have given a range of parameters that may be used. It should be noted that their values of liquid limit for the ratings of expansivity do not correspond with those of Dakshanamurthy & Raman (1973). Care must be exercised in comparing predictions from different methods to ensure that like is compared with like.

The approaches followed by Bandyopadhyay (1981) and Ladd & Lambe (1961) attempt in the author's opinion to carry the prediction one stage too far. Both attempt to predict a strain from classification data. The former author is explicit in his statement of the test conditions under which his equation is valid or applicable. The latter authors are less specific in that the initial sample density or void ratio is undefined. One of the major limitations of this type of predictive method is that there is no means of applying it under conditions that differ in any respect from the test conditions used in the development of the predictive method. For the former authors' method therefore, samples must be tested under 1psi surcharge, laterally confined, after being compacted to 92% of standard AASHO density at optimum moisture content. Swell is measured at the end of 96 hours soaking of the 2 inch thick sample. There is no means of assessing the effect of different initial or final conditions on the predicted swell.

3.2 Methods including water content

The inclusion of moisture content in the predictive equation is a partial improvement of swell or strain prediction as compared with those that use classification data alone. The method of Nayak & Christensen (1971) is of this type. These authors derived the form of their equation from a basic scientific equation, and replaced the scientific parameters with soil parameters. The major limitation of the use of this method is that it is only applicable to swelling with free access to water of a laterally confined sample under 1psi surcharge which is initially compacted to standard AASHO density, at optimum water content. Thus, although the initial moisture content is included in their equation, it will only be valid if it is also the optimum water content. Their data was originally published by Seed et al (1962) who used it to derive a means

of comparing the intrinsic expansiveness of the soils, although they used the term swelling potential.

The exclusion of vertical stress in the predictive equation in this type of method restricts its direct applicability to the stress actually used in the development of the method. Various stresses have been used by different authors.

3.3 Methods including external stress

Brackley (1983) and Weston (1980) have published equations for the prediction of swell in percent using a soil property, a stress and a water content. Brackley (1983) also includes a void ratio. The soils for which these equations were derived are notably different. Brackley (1983) used compacted samples, while Weston (1980) used undisturbed samples of a residual soil. The significance of soil structure in compacted soils has been discussed by Brackley (1975) and Zein (1985). Unfortunately, Zein's thesis has a copyright, prohibiting reproduction in any form. However, the significance of a packet type structure in clays compacted dry of optimum is likely to result in very different volume change properties from those of natural soils which may have a much more homogenous structure. This may account for some of the errors in prediction found by Brackley (1983) when applying his method to in-situ soil profiles. A further limitation of methods based on compacted soil data is that they are only applicable from the compaction moisture content. Attempts to apply them to compacted soils which have previously partly wet up or dried may be extremely misleading.

Both of these methods include the three most significant primary factors that have been shown by many authors to influence volume change on providing free access to water. The soil property, e.g. PI or LL, relates to the intrinsic expansiveness of the soil. The water content is related to the suction change, although is does not account for incomplete wetting in the field, nor for the previous suction history

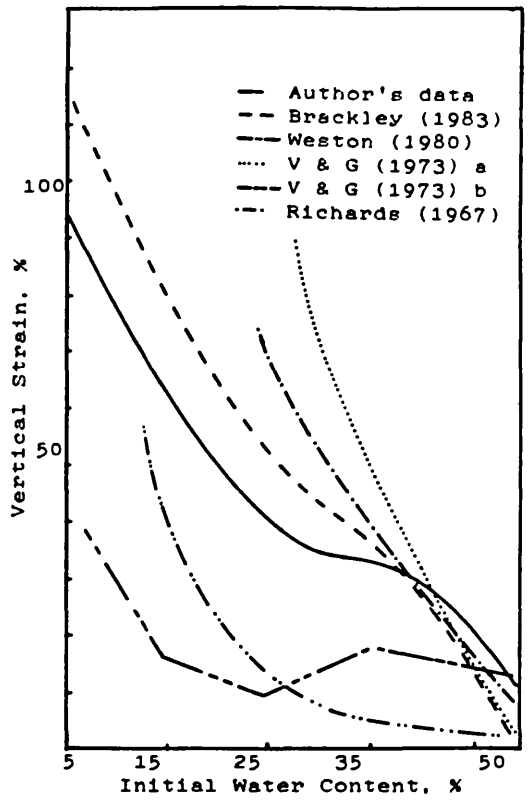


Fig.2 Comparison of predictions

which may be significant in terms of hysteresis. The vertical stress used is equivalent to the field vertical stress, but the lateral confinement in the oedometer tests is unlikely to model the field stress state.

The approaches followed by these authors are thought to be realistic attempts to obtain methods, for either undisturbed or compacted soils, of predicting the order of magnitude of maximum vertical strain that may be expected under complete wetting in the field. The accuracy of these methods has not been adressed in this paper.

3.4 Comparison with test data

In order to demonstrate the range of predictions obtainable using different methods, a random group of methods has been used for comparison with a small amount of the author's

laboratory data for a compacted soil which has a liquid limit of 125 and a plastic limit of 50. The comparisons are presented graphically on figure 2. Not all methods include the three primary parameters, and not all are applicable across the range of primary parameters considered. In particular, only one (Brackley 1983) was derived for compacted soils, but those for undisturbed samples both over and under predict.

There is a general agreement in the trend of all predictions (i.e. increasing swell with decreasing initial water content) except the Vijayvergiya & Ghazzaly (1973) equation using liquid limit and density. However, the scatter is very large, particularly as the water content decreases. Brackley's (1983) method was derived under similar conditions to the author's tests and the comparison is correspondingly good. The test conditions for the other methods differed to varying degrees, and the comparisons are less good. The variations in test conditions include soil type in terms of plasticity range, water content range, and applied vertical stress. It is to be expected that similar variations between field swell and predicted swell may occur due to similar variations between field conditions and the test conditions from which the predictive equation was derived.

4. SELECTION OF A PREDICTIVE METHOD

The foregoing discussion has shown that any particular method may be of value under the particular set of conditions for which it was derived. It is still necessary for an engineer to decide which method to apply to the particular problem with which he is faced. This paper does not attempt to recommend specific methods, rather to give guidelines for assessing the relevance of any method to the particular set of conditions under consideration. There are too many methods and the range of field conditions is too great for a single method to be universally acceptable.

The most important part of the process is the decision regarding the parameter that is required from the predictive method. The choice should be based on the concepts in section 2. If is it necessary to estimate the heave that may occur, then there is little point in using methods based on classification data only, as they can only be of value in assessing the intrinsic expansiveness. In the author's opinion there are only two levels of prediction that are worthwhile. The first level of assessment is the intrinsic expansiveness based on classification data, and the second is the estimation of swell with free access to water, based on intrinsic expansiveness, applied stress and water content. The latter category covers explicit inclusion of stress and water content such as by Brackley (1983) and Weston (1980) as well as the empirical or implicit inclusion of these parameters as, for example, by van der Merwe (1964). Application of the last method depends on the relevance of the "depth factor" in the climatic and geological area under consideration. It is felt that there is little to be gained from levels of prediction in between these two, as the stress and suction states are only partially considered. For higher levels of prediction it is recommended that laboratory modelling of the stress and suction or water states content should be undertaken.

Having decided on the level of prediction required, it remains to select one (or more) method. It should be borne in mind that in the first level predictions we are as yet looking at a qualitative assessment. There is no correct or incorrect method and it is difficult to give guidance on the best way to select a method. All divisions of intrinsic expansiveness, whatever name is given to them, are necessarily arbitrary.

It is generally valid to say that highly plastic clays will be more intrinsically expansive than low plasticity clays. However, a silty clay with a liquid limit of say 80 may not be as intrinsically expansive as a non-silty clay with a liquid limit of 80. It may thus be advisable to use two soil

properties, such as Liquid Limit and Plasticity Index in preference to relying on a single soil property. There appears to be a trend in published literature for soils of high intrinsic expansiveness to fall above the A-line on a Casagrande Plasticity Chart, but this has yet to be studied thoroughly. The use of a comprehensive field profiling procedure, such as Jennings et al (1973), is a valuable complement to first level predictions.

Choice of a second level predictive method can be somewhat more rational, since we are seeking a quantitative prediction under a specified set of field conditions. Obviously one important consideration is the set of stress and suction conditions used in the development of the method and their relation to the field conditions in question. There is little value in using a method derived for example from tests at optimum water content if the field water content will be significantly higher or lower. A further consideration that may be as important as the first is the type of sample. Methods for compacted soils may be of little value for undistubred soils and vice versa, but, even for undisturbed soils, the difference in behaviour of transported and residual soils may be significant. Finally, care must be exercised in selecting methods based on comparison with field observations. The predictive methods are generally based on the assumption of complete swelling with free access to water. This is rarely the case in the field. There may also be a variation in the initial field conditions between the time of the investigation and the start of the monitoring project, as was noted by Weston (1980), which renders the observed movement almost useless.

5. SUMMARY

Clear distinction between the three concepts of intrinsic expansiveness, swell and heave will be of benefit in assessing problems related to intrinsically expansive soils. It is essential that engineers are aware that the soil properties alone (Index properties or Intrinsic expansiveness) cannot be used to predict swell which results from changes in the state of stress and suction interacting with the soil properties. Classification data can tell us the soil type, but strains result from stresses, not from soil types. These distinctions are important in this field of soil mechanics because a very large number of predictive methods or variable value has been generated in an apparent attempt to find a universally accurate method. Judgement must be made of the value of any such method before applying it to a real engineering project.

6. ACKNOWLEDGEMENTS

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APPENDIX 6A.2

Identification and Classification of Expansive Soils

S. Oloo
Post-Graduate Student

H.D. Schreiner
Research Assistant

J.B. Burland
Professor

Imperial College of Science and Technology
London, United Kingdom

Synopsis

Over the past thirty years, many papers have been published on methods of classifying expansive soils. These methods usually relate one or more of the common soil classification parameters, such as Liquid Limit, clay fraction etc., to a small body of test data. The choice of classification parameters is usually arbitrary, and the ranges of test conditions and soil types are usually small. This paper presents results from a study of the available published data. The project was designed to find out which parameters were most strongly related to the observed swell. The data was first sorted into categories of identical test procedures and initial soil conditions. It is then possible, within each category, to compare the observed swell with one or more soil classification parameters.

Introduction

Many attempts have been made to find a universal system for identification of expansive soils. Schreiner (1987) has reviewed in general the misuse and limitations of applicability of predictive methods in expansive soil engineering. Perhaps the most serious criticism of the way in which these systems have been developed is that there does not seem to have been any attempt to find out which soil classification parameters are most strongly linked with the swelling of these soils. Table 1 lists some methods for the qualitative classification of expansive soils, with the classification parameters and other data required.

Author	Classification Parameters	Other data
van der Merwe (1964)	PI (whole soil)	% Clay
Driscoll (1983)	PI	% Clay
Seed et al (1962)	Activity (=PI/%Clay-10)	% Clay
Dakshanamurthy et al (1979)	LL	
Snethen (1984)	PI LL	Suction
Holtz & Gibbs (1956)	PI SL	Colloid Content

Table 1 Methods of identification of expansive soils

It can be seen, even at this simple level of prediction, that there is little agreement between the authors regarding the necessary input data. It is not surprising that our ability to identify expansive soils is poor, and that the prediction of potential field movements is often very bad.

The first three systems listed in Table 1 are all based on the same two parameters, Plasticity Index and Clay Fraction. The forms of the classifications are, however, rather different. Figure 1 shows the boundaries between the expansivity categories for all three methods. The boundaries for Seed et al (1962) have been calculated using their relation between PI and Clay Fraction for the activity, and not Skempton's (1953) equation. There is little agreement evident in this figure between these three widely used methods.

This paper presents the results of a study performed to determine which soil classification parameters are relevant to the identification of expansive soils.

Classification tests such as Atterberg Limit tests are not absolute measures of any soil property. However, because they are performed under identical conditions and procedures, they are an indirect measure of a soil property of the reconstituted soil, or, more probably, the combined effects of several properties. It is important to remember that the Atterberg Limit tests are usually performed on the portion of the soil which is finer than .425mm, while the swell tests are performed on the whole soil. The Atterberg limit values should be corrected for the coarse fraction, as recommended by Williams & Donaldson (1980), in order to be representative of the soil being tested. In most of the data sources for this project, the proportion of soil finer than .425mm was not reported.

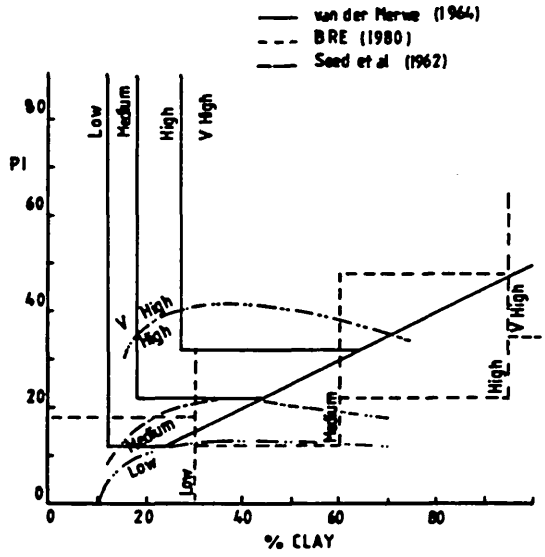


Fig. 1 Comparison of Three Classification Systems

Terminology

Clear differentiation between three basic concepts is thought to be essential to the understanding of expansive soils.

Intrinsic Expansiveness

Volume change of a clay soil due to changes in water content or suction is caused by interaction between the clay minerals and water. If the suction exerted on water in a soil is increased, water will move out of the soil under this stress and the soil will decrease in volume. Conversely, if water is added to the system, the suction will decrease and the soil will increase in volume. The intrinsic expansiveness is the soil property that relates the change in water content, and thus change in volume, to the suction change. A soil with a high intrinsic expansiveness will exhibit a large water content or volume change as compared with a soil of low intrinsic expansiveness when both are subjected to the same change of suction under identical initial stress, suction, soil structure and stress history conditions.

This concept is not new, but was mentioned by Brink (1955) and was used by Baikoff and Burke (1965). No standard procedure has yet been developed for the measurement of this property, and it is only useable at present as a qualitative description.

Swell

Swell is a measure of the volume strain or axial strain in a soil under a particular set of stress and suction conditions. It can occur either due to changes in suction or due to changes in applied load. In laboratory testing of intrinsically expansive soils, swell is usually induced by a reduction of suction under a small but constant vertical applied stress and zero lateral strain in an oedometer. Most testing is performed with an unknown initial suction, a final matrix suction close to zero, and unknown lateral stress.

Swell is not a soil property, but is a strain which results from interaction between suction and stress and the soil property intrinsic expansiveness described above. Swell is thus specific to the stress and suction conditions under which it is measured, and will probably also be affected by suction and stress history.

Heave

Heave is the displacement of a point in the soil due to suction and stress changes interacting with the intrinsic expansiveness. The relevance of laboratory swell measurements to heave under field conditions will depend on how well we can model, in the laboratory, the initial stress and suction states in the field and their changes.

Heave is not a soil property, but is a displacement which results from interaction between the soil property (intrinsic expansiveness) and the field stress and suction changes that we may or may not be able to quantify with a reasonable degree of confidence.

Data Sources

At the outset of this project, it was decided to use published data rather than to create an additional body of test data. It was thought that there would be a large amount of data available, with the necessary soil classification data included. The available data has been rather disappointing, as a large proportion is not useable as there are some essential pieces of information missing. In some instances, compacted densities were given, but without reference to standard densities such as Mod. AASHO. In many cases, the soil classification data was omitted. The data sources are listed in Appendix 1.

However, we believe that there is sufficient data available to make this study worthwhile. It would be of considerable value to have other data made available to test the conclusions reached on this somewhat limited data base.

Procedure

The data collected falls into several major categories. These are listed in Table 2. Further subdivision of the data was needed in some of the categories.

The intention of this project was to find a means of determining, from soil classification parameters, the intrinsic expansiveness of a soil. Since there are other factors which affect the actual swell in a test, such as vertical stress and initial density and moisture content, the data must be sorted into

Category	Test Type
1	Free swell
2	Oedometer, 6.9kPa Compacted samples
3	Oedometer, 10kPa Compacted samples
4	Oedometer, overburden Undisturbed samples
5	CBR, 4 day soak Compacted samples

Table 2. Data Categories

groups of data which have identical test conditions and identical initial soil conditions. For category 1 all data falls within the same group. For category 2, two groups were used, although there could have been more if the data had been available. Data was obtained for two initial dry density conditions. The first group includes only test data for which the soil was compacted at omc to the maximum Std. AASHO dry density. The second group includes only test data for which the soil was compacted at omc to 92% of the Std. AASHO maximum dry density. Category 3 had insufficient data to be of use. Category 4 would require a separate group for each moisture content and dry density combination, as well as a separate group for each vertical stress. This data has thus not yet been processed. Category 5 did not require further subdivision, as all tests were performed under the standard vertical stress, and all samples were compacted at omc to the Mod. AASHO dry density.

Free Swell

The Free Swell data was divided into three ranges, following the common usage. These are ; 0 to 50%, 50 to 100% and greater than 100%. The data was plotted against combinations of two or three parameters. The combinations are listed in Table 3.

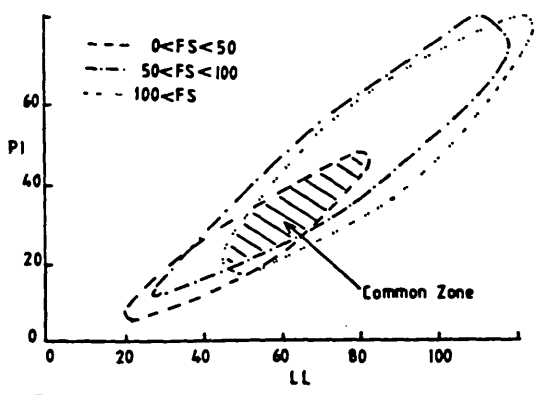


Fig. 2 Free Swell vs PI & LL

Plot no.	Parameters	Data in common zone %
1	PI vs LL	36
2	SI vs LL	63
3	SI vs PL	29
4	SL vs % Clay	42
5	SI vs % Clay	53
6	PI vs % Clay	58
7	LL vs % Clay	52
8	PL vs %Clay	56
9	SI vs Activity	55
10	Activity vs %Clay	61
11	LL vs Activity	56

Table 3. Free Swell Analysis

None of these plots showed any significant trends. Fig. 2 shows Plot number 1, which is the Casagrande Plasticity Chart. The data points have been omitted, but the boundaries show the lack of significant trends, except perhaps in the low PI and LL range. Of the 114 data points, 41 fall in the zone where all three ranges overlap. Approximately half of the 0 to 50% points and half of the 100+% points fall within this zone. This was one of the better plots. Plot number 10 is shown in Fig. 3. As with Fig. 2, there is no clear trend. The data for 50 to 100% free swell is almost all included in the common zone. As can be seen from the last column of Table 2, there is little correlation between the Free Swell and the parameters used in these plots.

Lambe (1958) presented a set of data for one soil which showed variations in sediment volume for one soil under different pore fluid conditions. That work, together with this study suggest that there is no significant connection between free swell and the soil type. It is possible that a better correlation may be found if the test were performed under more closely controlled pore fluid conditions.

There was insufficient data to test for a relation between free swell and swell under load. However, the data from Holtz & Gibbs (1956) shows that the range of predicted swell for a particular free swell value can be large. The free swell range of 50 to 100% covers the range of measured volume change from Low through Medium and High to Very High. In view of the absence of relationships between soil classification parameters and free swell, this is not surprising.

It is recommended that free swell should not be used as a means of classifying expansive soils.

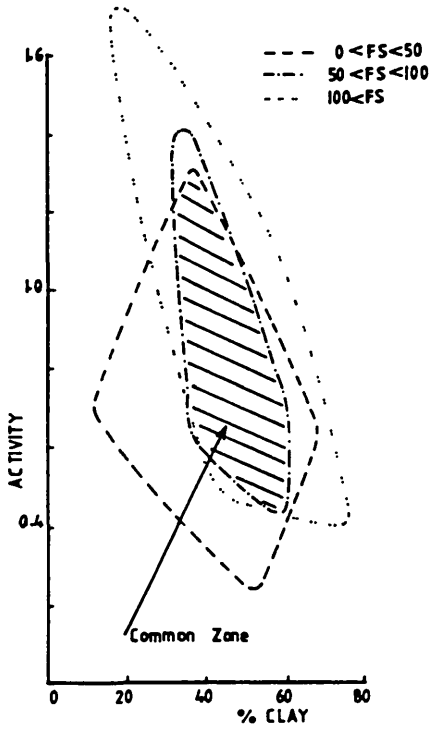


Fig. 3 Free Swell vs Activity & %Clay

Oedometer Test Data

Only Category 2 had sufficient data to warrant detailed study. Combinations of the parameters listed were also used in the analyses. The data in each group was divided into ranges of measured swell, and plotted against various combinations of the available parameters.

Since swelling properties are likely to be linked to soil type the start point for the study was the Casagrande Plasticity Chart, as this is used in the Unified Classification System for soil type identification. The results are shown in Fig. 4 for group A and Fig. 5 for group B. Although some trends are visible, there is a large amount of overlap of the data. Tentative boundaries for ranges of swell are shown by dashed lines.

It was thought that, since the Shrinkage Limit is determined from volume changes of the soil, that there may be an improvement in the trends if the PI were replaced by the Shrinkage Index. This reduced the amount of data available, and eliminated group B completely. Fig. 6 shows the data for group A. The trends are possibly somewhat more pronounced, but it is also noteworthy that the different mineral mixes that Seed et al (1962) used show trends within the overall trends. Skempton's activity values were calculated for each of the data points, and showed a progressive decrease from the montmorrilonite soils on the left hand side of the data to the Illitic soils on the left hand side of the data. The Kaolinitic soils did not follow this trend.

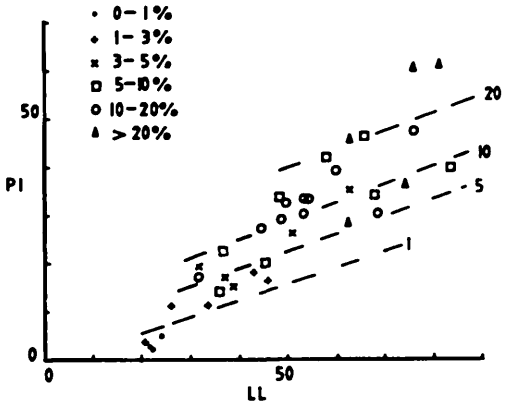


Fig. 4 Swell vs PI & LL for 100% Std AASHO dry density

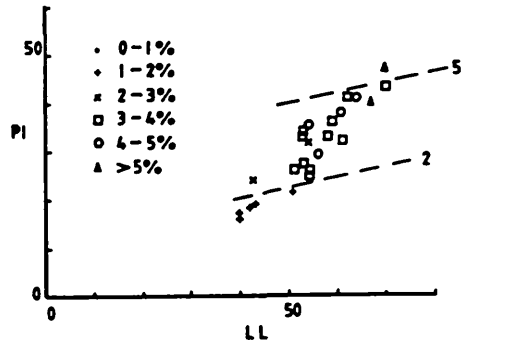


Fig. 5 Swell vs PI & LL for 92% Std AASHO dry density

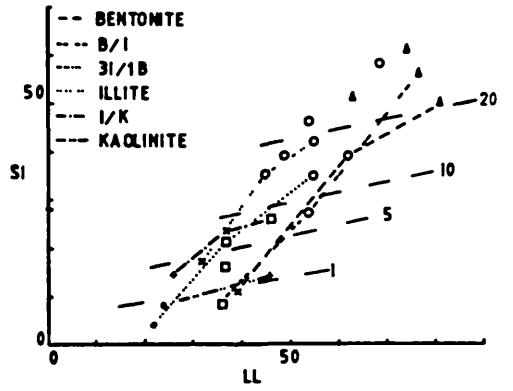


Fig. 6 Swell vs SI & LL for 100% Std AASHO dry density

Various other combinations of soil parameters were tested in an attempt to clarify the trends, and obtain more consistency between the different mineral compositions. It is thought that some success has been achieved, although the database is extremely small.

Although a large number of combinations of soil parameters was used, only three of the

plots will be presented. The remainder showed more scatter in the data, and are thus less good for the assessment of intrinsic expansiveness.

Using the PI - Clay fraction chart, Fig.7, for 100% Std AASHO dry density shows poor comparisons between the data and all three systems shown in Fig.1. In addition, there are no clear trends within the data.

The tentative boundaries on this plot are considered to be more logical than those of Seed et al (1962). For any particular PI value, the intrinsic expansiveness increases monotonically with increasing Clay Fraction, and also with the change from montmorillonite to kaolinite soils. This is not the case for the Seed et al boundaries, where increasing clay content for a particular PI first lowers then raises the predicted expansivity.

Figure 8 shows swell compared with Shrinkage Index and Clay Fraction. Most of the data is from Seed et al (1962), but there are four additional points. Three of these plot more consistently relative to the tentative boundaries in Fig.8 than relative to Seed et al (1962) boundaries in Fig. 1. The overall trend of all the data is also slightly more consistent with the boundaries in Fig.8 than with Seed et al (1962) boundaries.

The final plot of swell against SI and Activity is shown in Fig.9. The differentiation between the soil types is more pronounced on this plot than on the other plots, possibly due to the inclusion of all of PL, LL, SL and Clay Fraction. There are, however, still some inconsistencies, notably the point for 10% bentonite/90% sand. This point is inconsistent on all the plots. including the Seed et al (1962) plot where it falls in the 5 to 25% swell category.

CBR Swell

The data for the CBR swell category does not include the Shrinkage Limit, nor the Clay Fraction. Comparison of the data with the PI of the whole soil showedno apparent relation between the swell and the PI. The analysis was not taken any further in this category due to the lack of appropriate classification data.

Summary

The study reported in this paper has been severely restricted by the lack of data which includes the relevant soil classification parameters. It has shown that there is little consistency between the existing classification methods for expansive soils, despite some 25 to 30 years of expansive soil research and engineering. An attempt has been made, using a very limited quantity of data, to find the soil classification parameters that are most relevant to the classification of expansive soils in terms of the intrinsic expansiveness. It appears that the Shrinkage Limit is more appropriate than the Plastic Limit, and that the most consistent trends are found using Shrinkage Index against Clay Fraction or Activity.

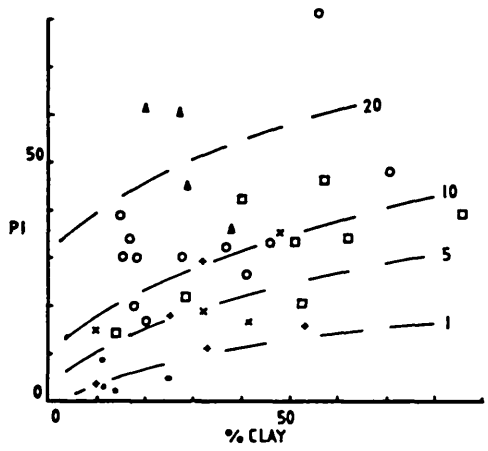


Fig. 7 Swell vs PI & %Clay for 100% Std AASHO dry density

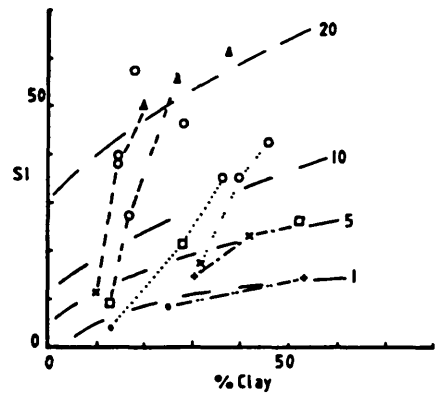


Fig. 8 Swell vs SI & %Clay for 100% Std AASHO dry density

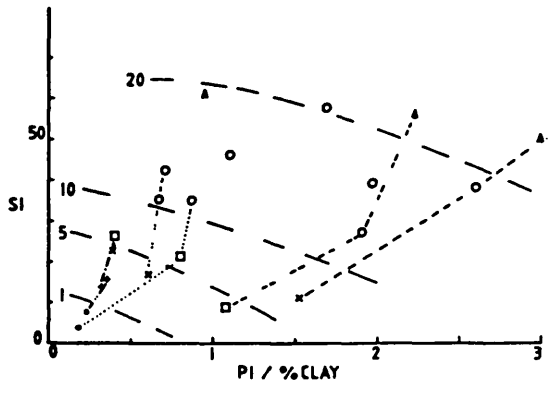


Fig. 9 Swell vs SI & Activity for 100% Std AASHO dry density

Little further progress can be made using the available data. The prediction of swell depends in the first instance on the identification of the intrinsic expansiveness of the soil, followed by the inclusion of the effects of the soil state and the stresses. Predictive methods will therefore be unreliable until the identification of intrinsic expansiveness can be reliably achieved. More data is urgently required, and must include all relevant classification parameters, soil conditions and test procedures. Only then can a comprehensive study be undertaken.

Acknowledgements

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CHAPTER 7

STANDARD OEDOMETER TESTS

7.1 INTRODUCTION

The tests described in this Chapter were planned to investigate the range of swell that would occur for various compaction moisture contents and vertical applied stresses representative of the "dry compaction" technique described by Ellis (1980). A detailed series of tests was performed on the soil from the Nairobi-Mombasa road site, supplemented by a less comprehensive series on the soils from the South Nyanza site.

In addition, the tests on the Nairobi-Mombasa Road soil were used to define the range of test conditions that would be used in the Stress Path Oedometer testing.

The test procedures followed the Holtz & Gibbs (1956) swell under load procedure, which was described in section 2.3, on samples compacted directly into the oedometer rings. A small number of consolidation tests were performed on samples which had swelled under small vertical stresses.

7.2 TEST PROCEDURES

7.2.1 COMPACTION

Samples to be tested were taken from the bag samples described in Chapter 5 which had been prepared at the chosen moisture content. The quantity of soil required for a sample of compacted thickness between 12 and 15mm was weighed out from the bag sample and placed in the oedometer for compaction. The oedometer ring was greased and one sheet of filter paper was placed on the lower porous stone. The 4 inch diameter oedometer has been described in Schreiner (1987d), included as Appendix 7A.1. The cell was used in a standard lever arm frame and readings were taken with a dial gauge with 0.01mm divisions.

Compaction was by stress controlled static loading of 1MPa. This was aplied by placing weights on the hanger of the lever arm of the oedometer frame. The load was maintained for 5 minutes and then reduced to 1.5kPa. This was also maintained for 5 minutes before the oedometer was removed from the loading frame. This permitted measurement of the compacted thickness of the sample, initially using a vernier depth gauge at 6 points around the circumference and in later tests in the centre using a dial gauge

on a saddle. The depth of the empty sample chamber had been determined after assembly using the same procedure. The initial void ratio was calculated from this thickness measurement.

The top cap was then replaced and the oedometer cell replaced in the loading frame. The dial guage was set and the initial reading taken under a stress of 1.5kPa. The vertical stress for the test was then applied and another reading taken. The sample was left overnight under this load and the test started the next day.

7.2.2 SWELLING

A reading was taken before the swelling was started, and the taps to the water supply then opened. Distilled water was allowed to flow to the bottom of the samples under a head of a few centimetres. Readings were taken during the first day and the next morning. The vertical stress was decreased to 1.5kPa after a minimum of 24 hours swelling, and the sample removed after at least 24 hours under 1.5kPa. This allowed most of the rebound to occur before the sample was removed from the load frame, thereby minimising swelling and uptake of water during the dismantling of the cell.

7.2.3 CONSOLIDATION

Four samples were subjected to consolidation stages after swelling under 1.5kPa had been completed. The vertical stress was increased each day by an amount approximately equal to the vertical stress under which the sample had consolidated. After consolidating to 1MPa, the samples were unloaded in four steps to 1.5kPa, allowing 24 hours for each step.

7.2.4 SAMPLE REMOVAL

The cell was removed from the load frame after the sample had rebounded under a vertical stress of 1.5kPa. The drainage taps were closed before removal. The top cap was removed and any free water wiped off with a paper towel. The final sample thickness was measured as described in section 7.2.1. The cell was then dismantled and the sample removed for moisture content determination. Usually two samples were taken from the top half of the sample and two from the bottom half.

7.3 TEST RESULTS

A total of 45 tests were performed. Of these, 25 were on the soil from Nairobi-Mombasa Road and 20 on the soil from South Nyanza. The results for the tests on the soil from the Nairobi-Mombasa Road are listed in Table 7.1, and those for the

soil from South Nyanza in Table 7.2. Since the compacted void ratios, e_c, varied slightly between samples at the same compaction moisture content, w_c, the void ratios after compaction have been adjusted to the compacted void ratio of the first test performed at each moisture content. The void ratios at the start and finish of each test have been adjusted by the same amount, and are designated e_{ic} and e_{fc} in Table 7.1 and Table 7.2. This reduces the apparent scatter and makes interpretation easier.

7.3.1 RATE OF SWELL

Typical data are shown in Fig.7.1 of the change in void ratio with time. It can be seen that most of the swelling took place within the first three hours, but that some tests showed slower rates of swell. This is probably due to the water supply being restricted. It was noticed that the rate of flow of water into one of the cells was slow compared to the other. This is thought to have been due to a partially restricted flow through the porous plate in that cell. However, the rate of swell after 1 day is small in all cases, and there will not be any significant error in the void ratios after 24 hours swell due to the reduced rate of swell in some tests.

7.3.2 SWELL RESULTS

Fig.7.2 shows the data from the tests on the soil from the Nairobi-Mombasa Road, and Fig.7.3 the data for the South Nyanza soil. Parts (a), (b), (c) and (d) show the void ratio of the samples before and after swelling for 1.5, 4.5, 16 and 64kPa vertical stress respectively. All the data are combined in part (e) which shows void ratio as a function of both compaction moisture content and vertical stress during swelling.

Fig.7.2 shows a "1 1/2" peak compaction curve which is common for highly plastic soils. Since the data is plotted in terms of void ratio, the curve plotted is the inverse of the more traditional moisture-density curve. The density peaks are seen as troughs in this plot. The data in Fig.7.3 show a simple one peak compaction curve, which is typical of medium plasticity soils. The optimum moisture content does not appear to have been exceeded, although the highest compaction moisture content is above the plastic limit.

The final void ratio, after 24 hours swelling, can be seen from from parts (a) to (d) of Fig.7.2 to be influenced by the initial void ratio. The shape of the compaction void ratio-moisture content curve is still visible in the void ratio-moisture content curve, in Fig.7.2, after swelling. The same is true of Fig.7.3, except at the lowest moisture content. It is possible that there is a slight trough between the 3.5 and 13.9% moisture content points on the compaction curve, and that this is

more strongly evident in the void ratio after swelling.

The influence of vertical stress is demonstrated in Fig.7.4 and Fig.7.5 where the magnitude of the swell is plotted against the vertical stress on a natural scale. The void ratio after swelling can be seen to increase rapidly at stresses below about 10kPa. This holds for all of the compaction moisture contents for which data were obtained. The benefit of even a small amount of overburden on the amount of swell is evident.

Considering part (e) of both Fig.7.2 and Fig.7.3, it can be seen that the void ratio after swelling is a function of both compaction moisture content and vertical stress during swelling. The effect of the vertical stress can be seen to vary with the compactiom moisture content. At the highest moisture content, the decrease in swell with increasing log stress is nearly linear, while at low moisture content it is strongly curved. This was interpreted for the data in Fig.7.2 by Schreiner (1987d) in terms of the structure of the compacted soil, which is clustered and aggregated at low moisture contents and preferred parallel at high moisture contents (Zein 1985). Identical arguments are valid for the data in Fig.7.3.

Comparison of Fig.7.2 and Fig.7.3 shows that the Nairobi-Mombasa soil swelled to a higher void ratio than the South Nyanza soil, although the compacted void ratios were not very different. This is not unexpected as the former soil is more highly plastic than the latter. However, in view of the findings of Chapter 6, any correlation between swell and plasticity must be regarded with caution.

7.3.3 CONSOLIDATION RESULTS

The results from the consolidation stages of four tests on the Nairobi-Mombasa Road soil are shown in Fig.7.6 to Fig.7.9. Each figure shows the swell and consolidation data for one compaction moisture content together with swell under load data for the same moisture content under vertical stresses of 4.5, 16 and 64kPa. Where data was not available for the same moisture content the swell under load results have been derived from Fig.7.2(e) by interpolation.

In all cases the final void ratio from the swell under load tests is seen to lie below the consolidation curve. This has been widely reported previously. The reasons for this are investigated in Chapter 10 in the discussions of the test procedures based on the Stress Path Oedometer data.

Although the samples appear to have a "preconsolidation" stress where the slope of the consolidation line increases, the compressibilities are large even before this stress is reached. Fig.7.10 shows stress strain curves on natural scales derived from the void ratio stress data for these tests. The sample which was compacted at 6% moisture content is particularly compressible,

straining by 5% for a stress increment of only 10kPa. The samples compacted at higher moisture contents are stiffer, and the wettest sample required 50kPa for 5% strain.

These compressibilities would be decreased for samples allowed to swell under larger vertical stresses. The sample compacted at a moisture content of 35% in test SP01 reported in Chapter 10, by comparison, compressed by only 0.16% in the first stress increment from 50 to 100kPa. Test SP01 was a similar test performed with controlled suction in the Stress Path Oedometer. After reaching zero suction under 50kPa vertical stress, the sample was subjected to increasing vertical stress. The high compressibility of the samples compacted dry is in part due to the high void ratio at the end of swelling. However, it is probable that there is also a collapse type structural change occurring as the structure is modified from clustered to preferred parallel.

7.4 DISCUSSION

7.4.1 COMPARISON WITH BRACKLEY'S EQUATION

Brackley (1983) presented an equation for swell of a soil in terms of various parameters:

$$S = (5.3 - 147e/PI - \log p)(.525PI + 4.08 - .85w)$$

where S= swell in percent
e= void ratio before swelling
p= vertical total stress
w= initial water content
PI= Plasticity Index

A comparison between this equation and the data for the Nairobi-Mombasa road soil was presented in Schreiner (1987d) in terms of the measured and calculated final void ratios. Fig.7.11 shows the measured and calculated swell for the same soil. The error is up to 115% overestimate for a sample compacted at 6% moisture content and swelled under 64kPa vertical stress, as shown by points A1 and A2. At 35% compaction moisture content the comparison is good at 1.5kPa stress, as shown by points B1 and B2, but 14% swell, shown by points C1 and C2, is predicted at 256kPa where no swell was measured.

Although Brackley's equation appears to provide a similar pattern of swelling to that observed, the values calculated can be so significantly in error that Brackley's equation should not be a universal equation. In part this is explained by the findings in Chapter 6 where it was shown that the Plasticity Index alone is insufficient to define the Intrinsic Expansiveness of a soil. It is also due to the assumed linear relationship between swell and

the log of the vertical stress, which is not correct for this set of data.

Fig.7.12 presents the same comparison for the South Nyanza soil. Here the errors are small at low compaction moisture contents and large at the higher moisture contents. At the highest compaction moisture content Brackley's equation predicts collapse of 5% at point A1, whereas a swell of 8% was observed. The conclusion reached in Schreiner (1987d) that this equation is best near optimum moisture content is not correct. The pattern of errors is not consistent in these two cases. It is evident that in general, Brackley's equation should be used with considerable caution.

7.4.2 SHEAR PLANES

In several of the samples of both Nairobi - Mombasa Road and South Nyanza soil a conical shear plane with polished surfaces was observed when the sample was removed from the oedometer. Fig.7.13a shows a sketch of a typical occurrence. This suggests that the stresses during swelling reach the maximum shear stress which the soil can sustain, and that failure occurs by expansion toward the only free surface.

It was thought that this might only occur during swelling of unsaturated soils, but a similar shear plane was caused in a sample which had been consolidated to 1MPa and was then unloaded to 1.5kPa in one step.

Shear planes were also observed in a sample of the soil from Wad Medani in a swell test not reported here, but similar in all respects to the tests that have been reported. In this sample the shear plane appeared spherical and did not reach the base of the cell as shown in Fig.7.13b, It appears that the shear planes are not only found in the most highly plastic soils.

7.4.3 UNDRAINED SHEAR STRENGTH

An approximate measure of the shear strength can be obtained using the relation between liquidity index and shear strength of remoulded clays given by Skempton & Northey (1952). This should not be used to obtain absolute values, but can provide comparative strength values. It would be unwise to attempt to use the values as absolute shear strengths both because the soil is compacted and not remoulded and because the soil may not be fully saturated.

These comparative strength values have been used to estimate the CBR values that could be expected after swelling. Once again, this will not provide accurate values suitable for design, but may provide some indication of the decrease in strength on soaking that may occur in a road foundation which was compacted at some moisture content below optimum moisture content.

The liquidity index has been calculated where necessary from

the void ratio assuming 100% saturation. The undrained shear strength has been obtained from the middle of the range of the curves, Fig.7.14, presented by Skempton & Northey (1952). A CBR value of 15 at a moisture content of 45% quoted by Mutonyi (1973) for the same soil has been used as a reference value. This was chosen because the moisture content is well above optimum moisture content, and the sample would have been close to saturation after compaction. The suction would have been small and would have contributed little to the strength.

The CBR value for each sample has been calculated from the ratio of the undrained shear strength of the sample to the undrained strength at 45% moisture content. It has been assumed in doing this that the CBR value is proportional to the undrained strength. This is thought to be a reasonable assumption because the high rate of strain in the CBR test should cause undrained conditions to exist for highly plastic soils for the short duration of the test.

The estimated CBR values are shown in Fig.7.15 together with the void ratio data. It can be seen that the CBR value decrease rapidly as the compaction moisture content decreases below the optimum moisture content of about 40%. Even for the samples which swelled under 64kPa, the strength for the dry compacted samples is considerably lower than that for the wet compacted samples. This indicates that a major loss of strength could occur if a foundation which had been compacted dry of optimum moisture content were to wet up to zero suction. It does not permit conclusions to be drawn regarding the loss of strength on wetting up to non-zero suctions.

7.4.4 NORMALISATION OF DATA

It had been planned that the data would be normalised by one or other soil classification parameter, such as liquid limit. The study reported in Chapter 6 indicates that the result will not be conclusive. A simple comparison of the shapes of the void ratio versus compaction moisture content data in Fig.7.2 and Fig.7.3 shows that normalisation of the two data sets could not give a unique result. Since the basic shapes of the curves differ, normalisation by any constant number, be it a liquid limit or the geologic age of the soil, will retain the shapes of the curves. The curve for the Nairobi-Mombasa Road soil will still have a "1 1/2" peak shape, and the South Nyanza soil will still have a single peak curve. Since these shapes were retained after swelling, it has been concluded that normalisation of this type of data cannot provide any benefit.

Test No	w_c %	σ_v kPa	e_c	e_i	e_f	e_{ic}	e_{fc}	swell %	Brackley %
ST1	5.96	1.5	1.05	1.05	2.94	1.05	2.94	92	115
ST2	9.99	1.5	1.21	1.21	2.81	1.21	2.81	72	94
ST3	14.99	1.5	1.27	1.27	2.67	1.27	2.67	62	81
ST4	15.07	1.5	1.24	1.24	2.81	1.24	2.81	70	81
ST5	20.1	1.5	1.37	1.37	2.61	1.37	2.61	52	63
ST6	25.1	1.5	1.41	1.41	2.31	1.41	2.31	37	51
ST7	30.2	1.5	1.29	1.29	2.18	1.29	2.18	39	45
ST8	34.8	1.5	1.24	1.24	1.97	1.24	1.97	33	35
ST9	40.0	1.5	1.20	1.20	1.89	1.20	1.89	31	25
ST10	51.2	1.5	1.34	1.34	1.61	1.34	1.61	11	1
ST11	6.1	4.5	0.98	0.97	2.27	1.04	2.34	64	98
ST12	6.1	16.0	1.11	1.10	1.97	1.04	1.91	43	77
ST13	6.1	64.3	1.01	0.97	1.49	1.01	1.53	26	56
ST14	19.2	4.5	1.27	1.25	2.09	1.35	2.19	36	51
ST15	19.3	16.0	1.29	1.27	1.84	1.35	1.96	26	37
ST16	19.3	64.3	1.25	1.19	1.50	1.31	1.62	13	23
ST17	19.4	256	1.34	1.28	1.30	1.28	1.30	1	9
ST18	34.0	4.5	1.27	1.25	1.84	1.21	1.81	27	30
ST19	33.4	16.0	1.29	1.27	1.62	1.22	1.60	17	22
ST20	33.7	64.3	1.33	1.29	1.46	1.20	1.35	7	15
ST21	33.6	256	1.34	1.25	1.25	1.15	1.15	0	8
ST22	48.0	4.5	1.32	1.30	1.44	1.32	1.46	6	1
ST23	49.2	4.5	1.33	1.32	1.65	1.31	1.65	15	1
ST24	49.2	16.0	1.34	1.30	1.44	1.30	1.44	6	1
ST25	49.2	64.3	1.30	1.23	1.32	1.27	1.36	4	0

Key

w_c = compaction moisture content
σ_v = vertical stress during swelling
e_c = void ratio after compaction
e_i = void ratio at start of swelling, under σ_v
e_f = void ratio at end of swelling
e_{ic} = adjusted void ratio at start of swelling
e_{fc} = adjusted void ratio at end of swelling
swell = observed swell
Brackley = swell calculated using Brackley (1983)

TABLE 7.1 SUMMARY OF DATA FOR NAIROBI-MOMBASA ROAD SOIL

Test No	w_c %	σ_v kPa	e_c	e_i	e_f	e_{ic}	e_{fc}	swell %	Brackley %
ST26	3.4	1.5	1.07	1.07	1.76	1.07	1.76	33	33
ST27	3.4	4.5	1.03	1.02	1.51	1.06	1.55	24	23
ST28	3.4	16.0	1.02	1.01	1.26	1.06	1.32	13	11
ST29	3.4	64.3	0.93	0.90	1.03	1.04	1.16	6	-1
ST30	13.9	1.5	1.15	1.15	1.58	1.15	1.58	20	14
ST31	13.9	4.5	1.10	1.10	1.41	1.15	1.46	14	8
ST32	13.9	16.0	1.13	1.13	1.34	1.14	1.36	10	2
ST33	13.9	64.3	1.14	1.11	1.14	1.12	1.14	1	-4
ST34	18.1	1.5	1.18	1.18	1.59	1.18	1.59	19	8
ST35	18.1	4.5	1.13	1.13	1.43	1.17	1.50	15	5
ST36	18.1	16.0	1.10	1.10	1.29	1.17	1.36	9	1
ST37	18.1	64.3	1.09	1.06	1.09	1.15	1.18	1	-3
ST38	22.9	1.5	1.05	1.05	1.26	1.05	1.26	10	5
ST39	22.9	4.5	0.95	0.95	1.20	1.05	1.3	12	3
ST40	22.9	16.0	1.01	1.01	1.19	1.04	1.23	9	2
ST41	22.9	64.3	0.96	0.92	1.00	1.02	1.10	4	0
ST42	28.1	1.5	0.81	0.81	0.91	0.81	0.91	6	-5
ST43	28.1	4.5	0.80	0.79	0.93	0.80	0.94	8	-4
ST44	28.1	16.0	0.88	0.88	0.96	0.81	0.90	5	-3
ST45	28.1	64.3	0.97	0.93	0.98	0.77	0.82	3	-2

Key

w_c = compaction moisture content
σ_v = vertical stress during swelling
e_c = void ratio after compaction
e_i = void ratio at start of swelling, under σ_v
e_f = void ratio at end of swelling
e_{ic} = adjusted void ratio at start of swelling
e_{fc} = adjusted void ratio at end of swelling
swell = observed swell
Brackley = swell calculated using Brackley (1983)

TABLE 7.2 SUMMARY OF DATA FOR SOUTH NYANZA SOIL

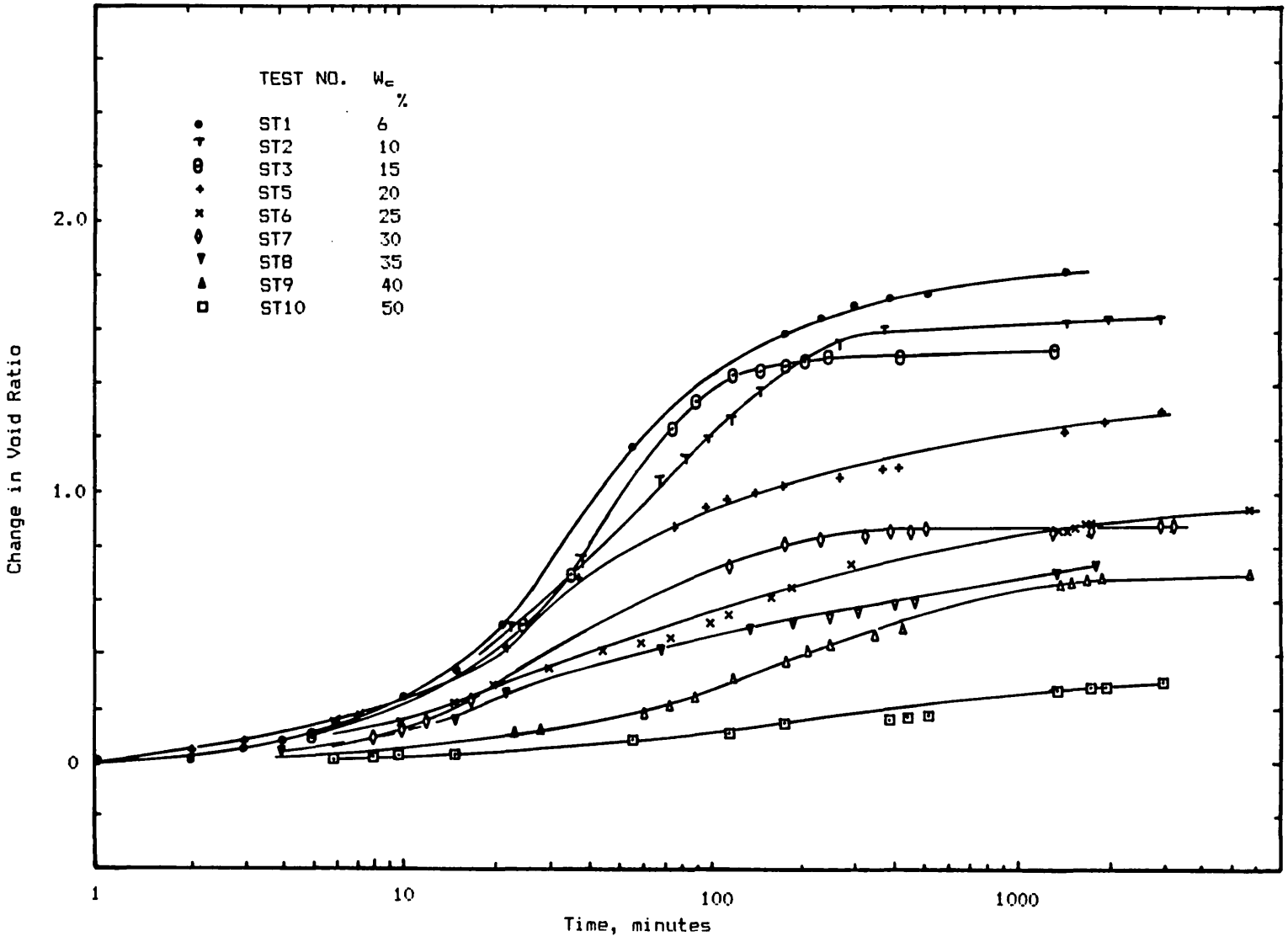


Fig.7.1 Duration of Swelling

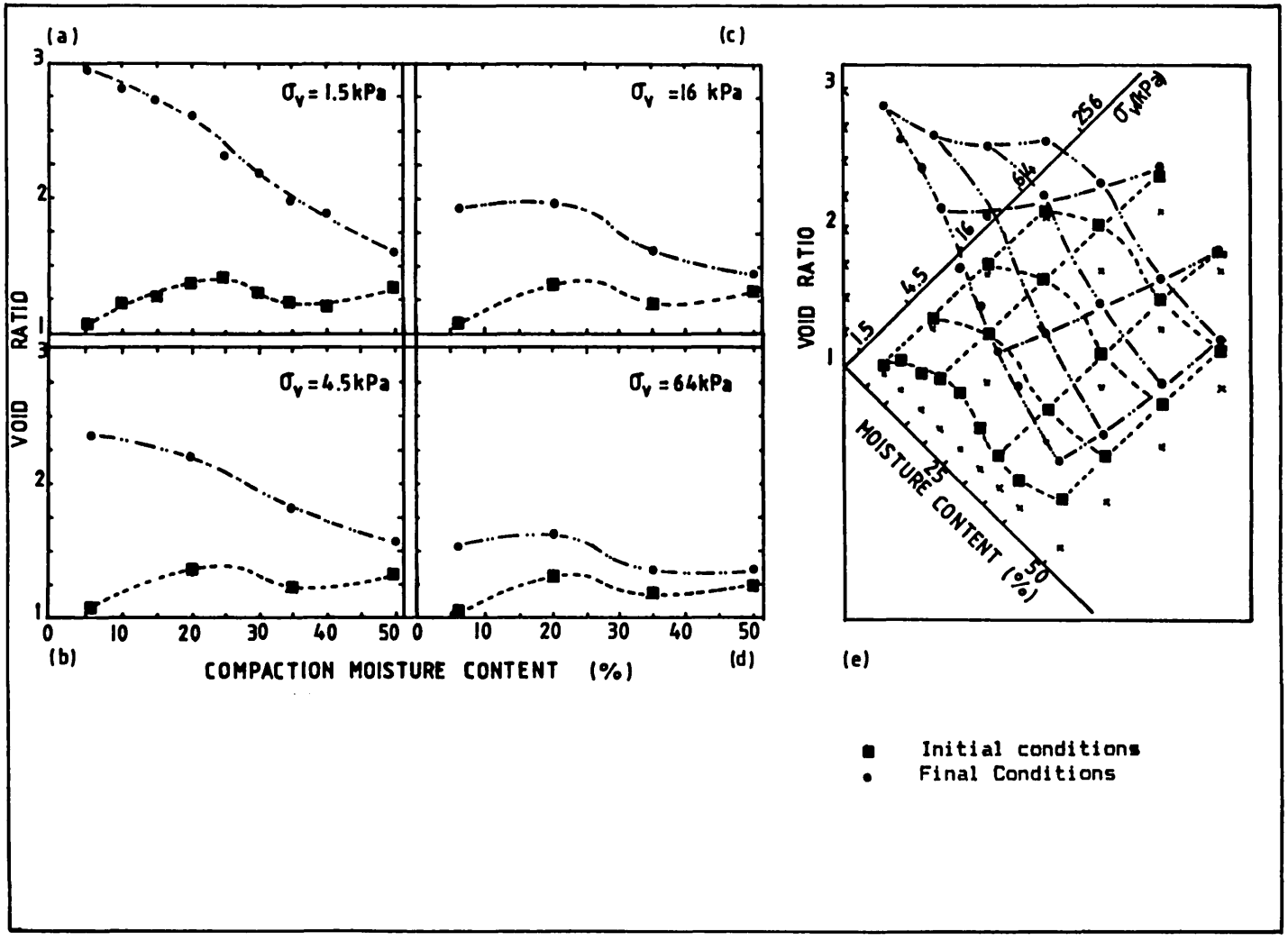


Fig.7.2 Oedometer Swell Tests, Nairobi-Mombasa Road Soil.

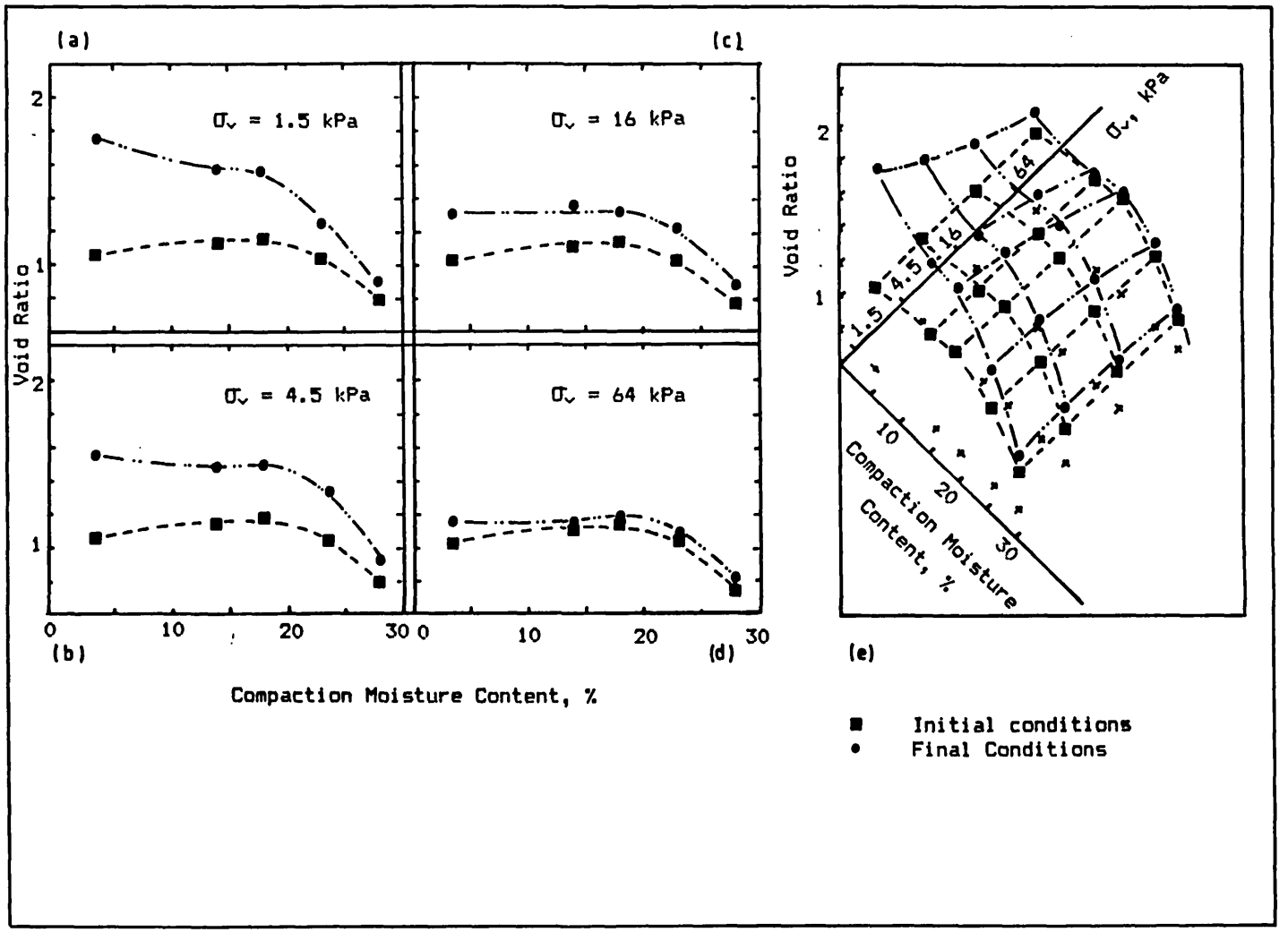


Fig.7.3 Oedometer Swell Tests, South Nyanza Soil.

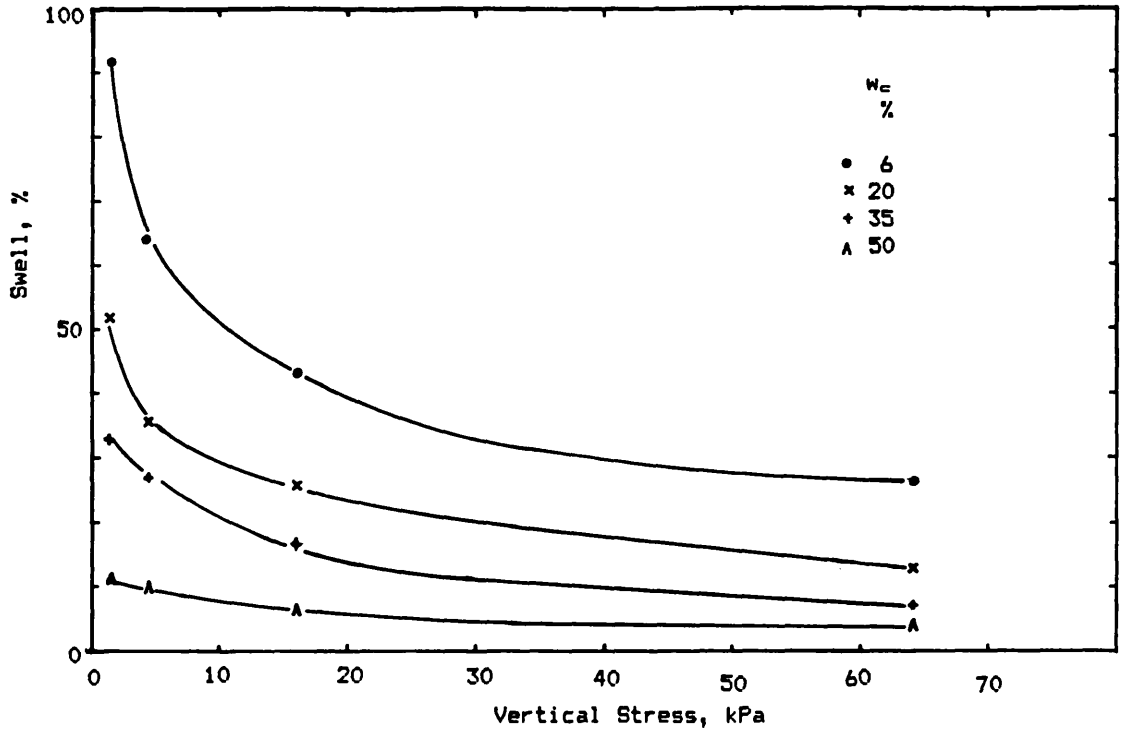


Fig.7.4 Effect of Vertical Stress on Swell
Nairobi-Mombasa Road Soil

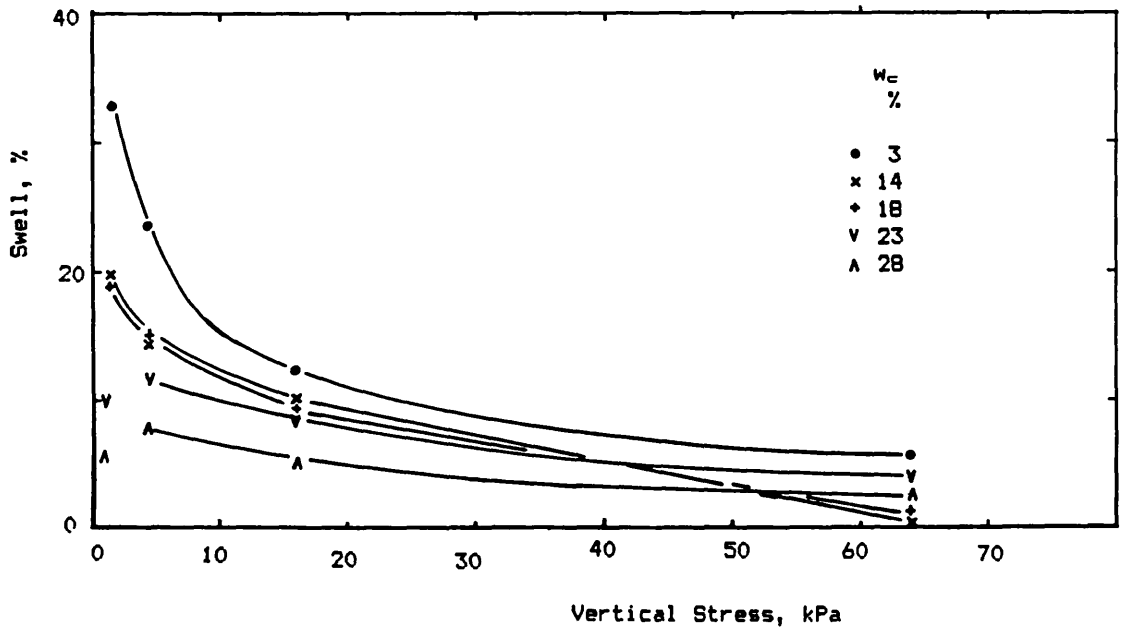


Fig.7.5 Effect of Vertical Stress on Swell
South Nyanza Soil

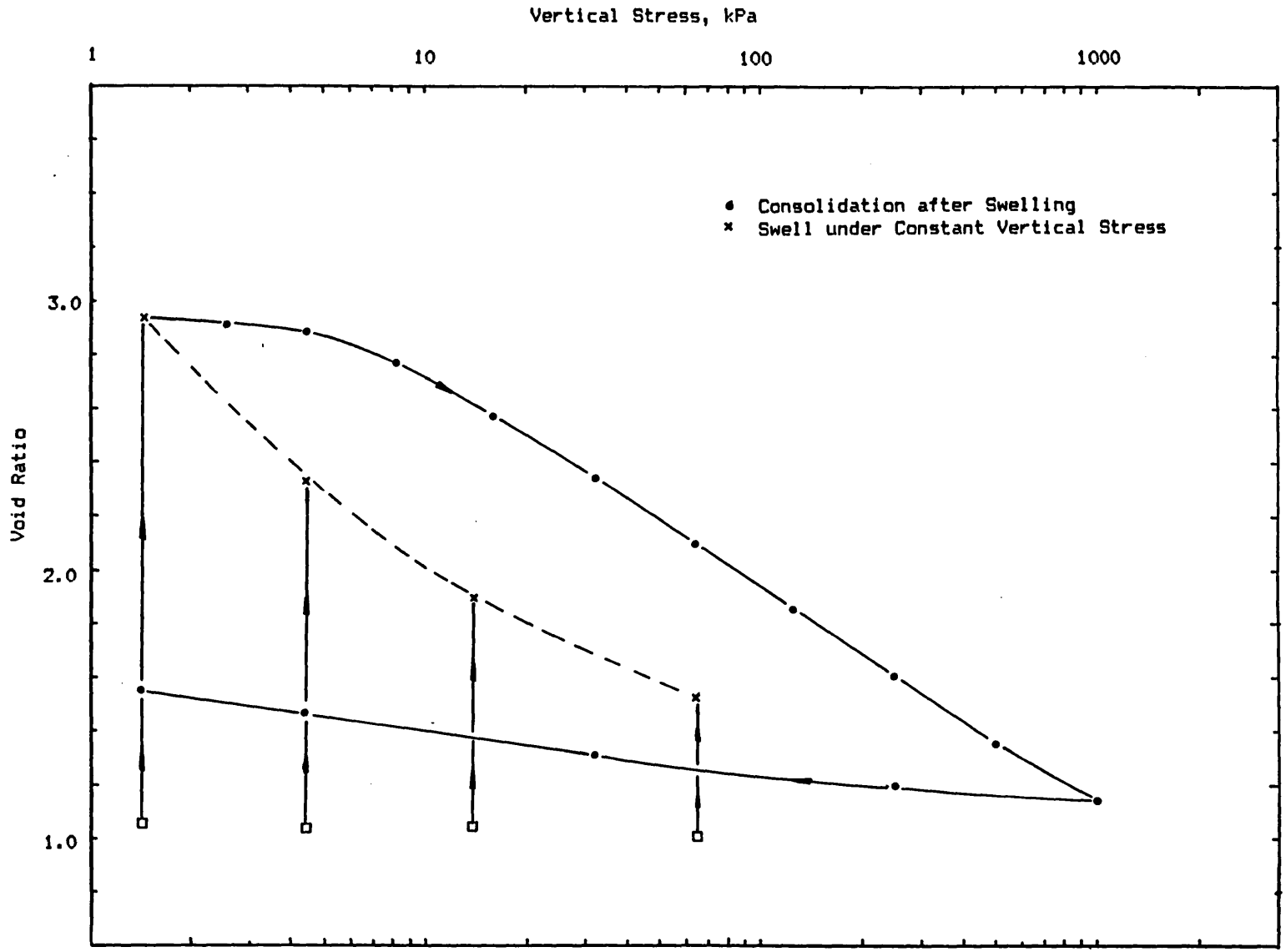


Fig.7.6 Comparison of Test Procedures
6% Compaction Moisture Content
Nairobi-Mombasa Road Soil

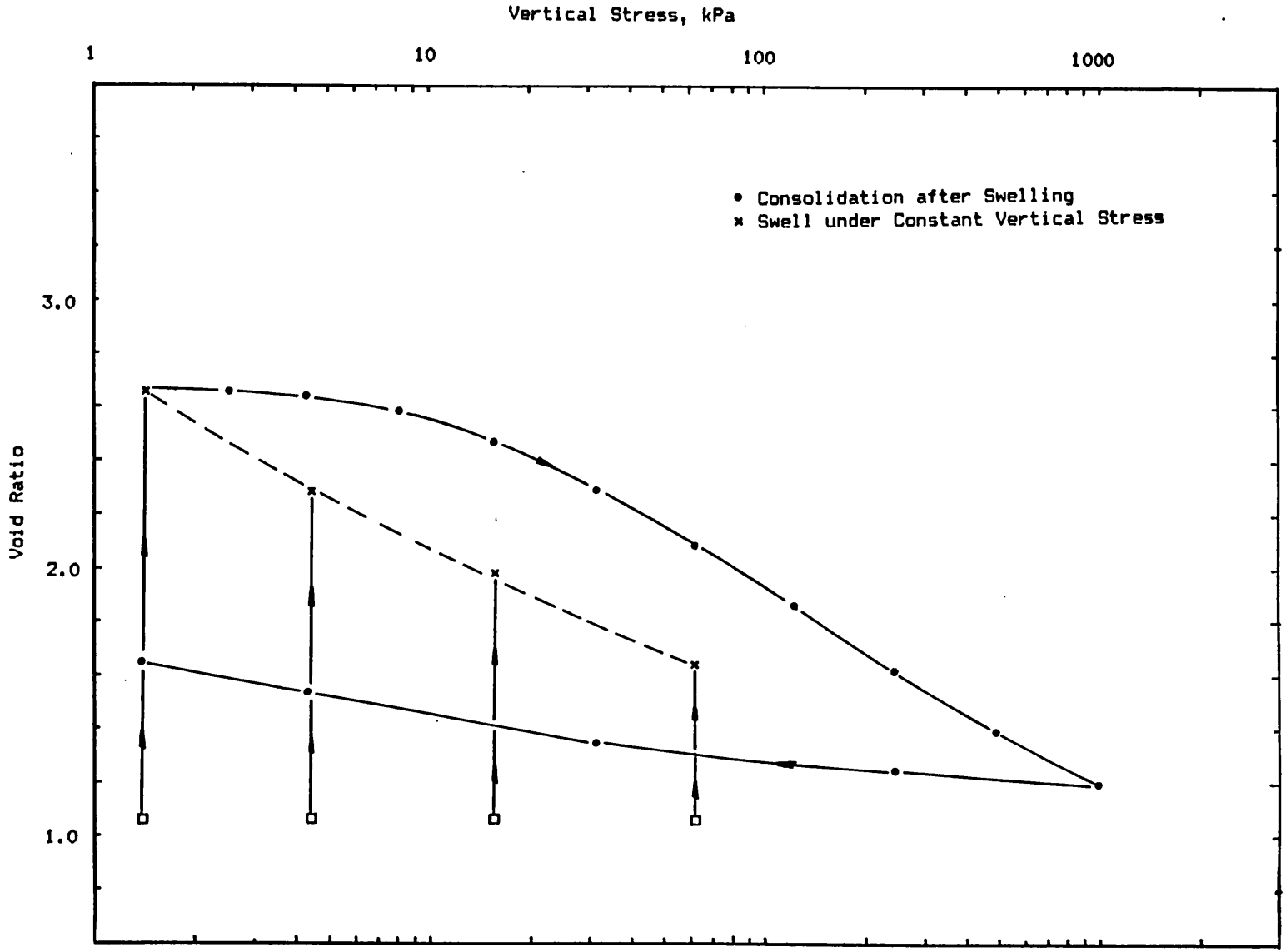


Fig.7.7 Comparison of Test Procedures
15% Compaction Moisture Content
Nairobi-Mombasa Road Soil

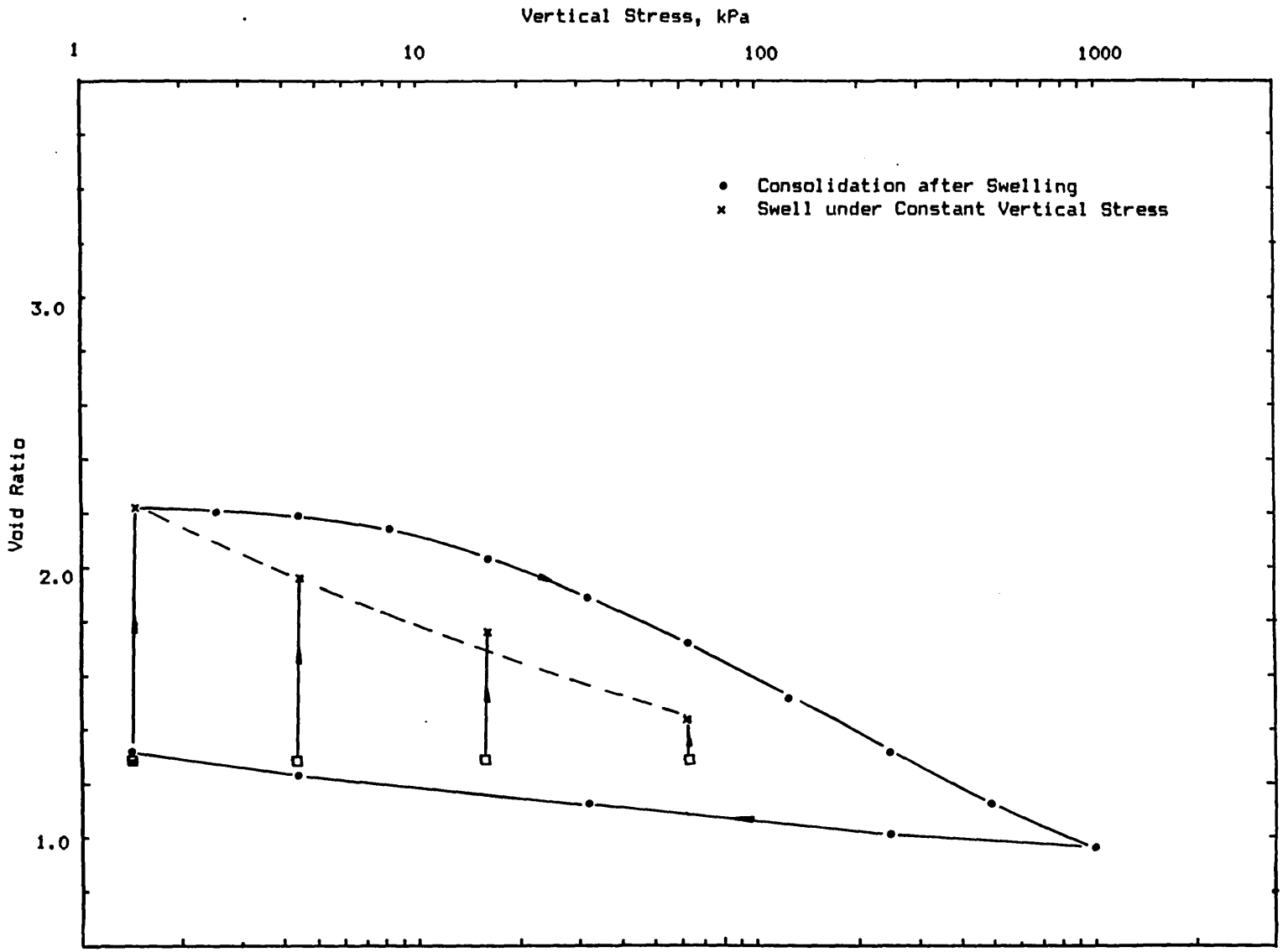


Fig.7.8 Comparison of Test Procedures
30% Compaction Moisture Content
Nairobi-Mombasa Road Soil

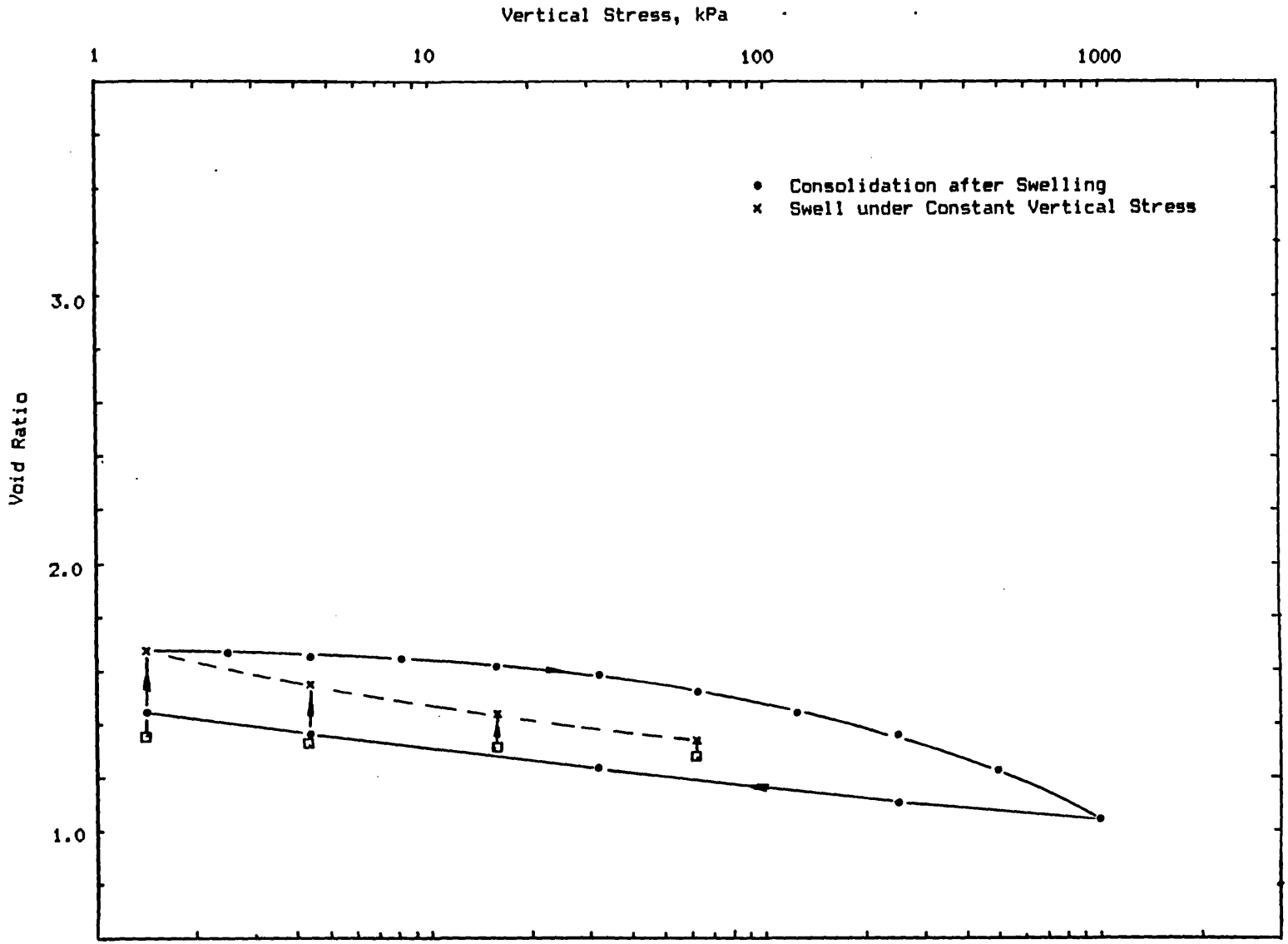


Fig.7.9 Comparison of Test Procedures
50% Compaction Moisture Content
Nairobi-Mombasa Road Soil

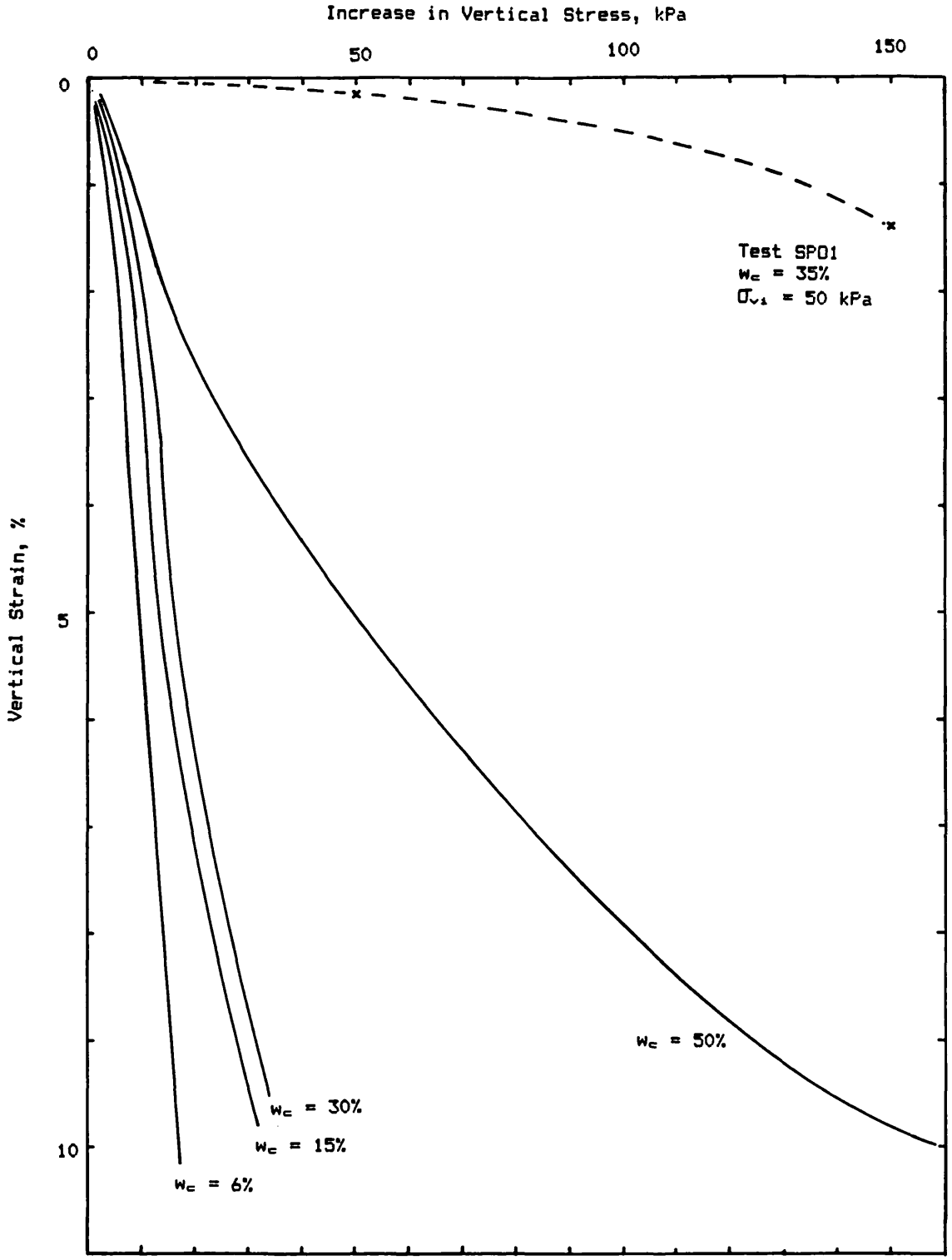


Fig.7.10 Compression after Swelling

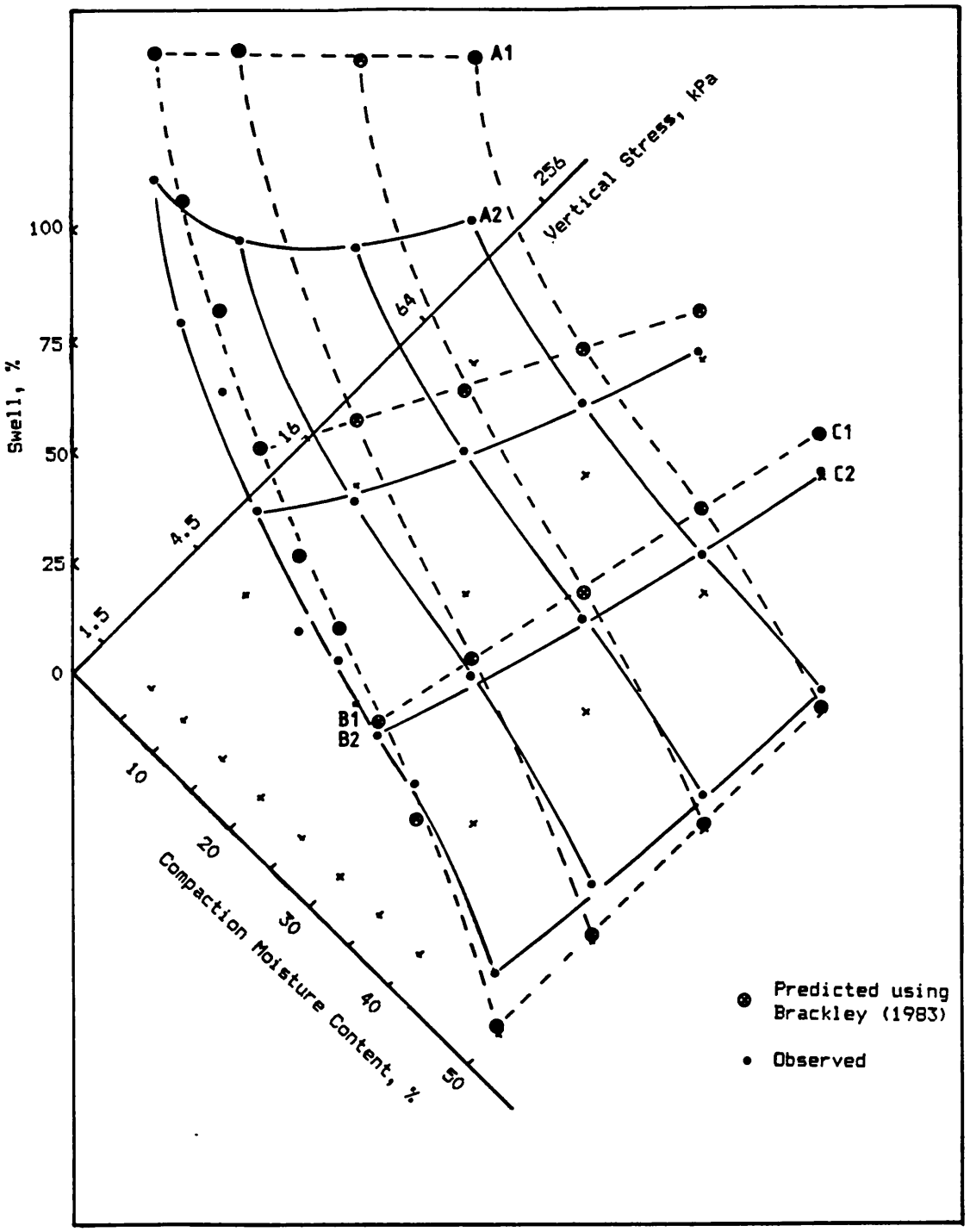


Fig.7.11 Observed and Predicted Swell
Nairobi-Mombasa Road Soil

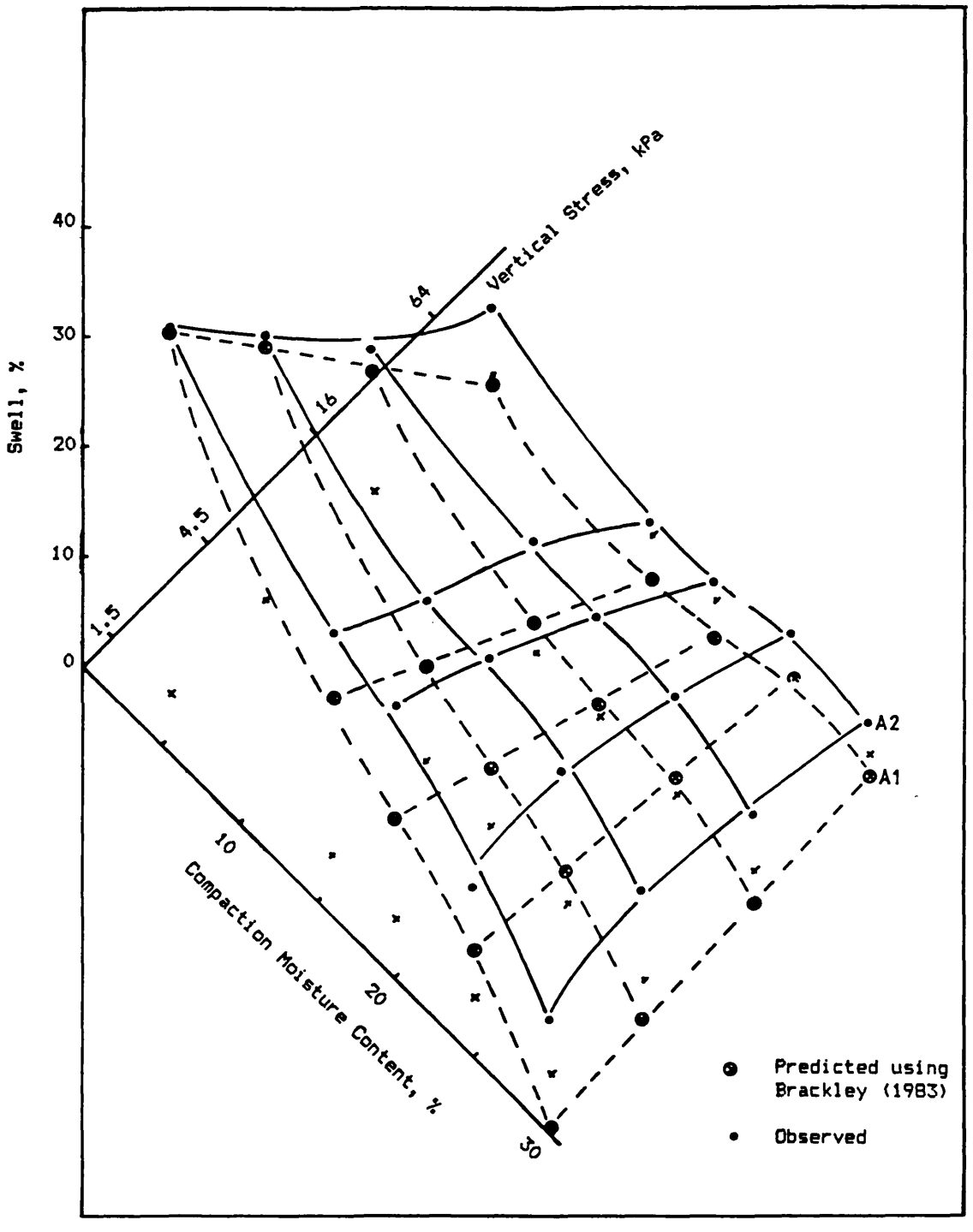
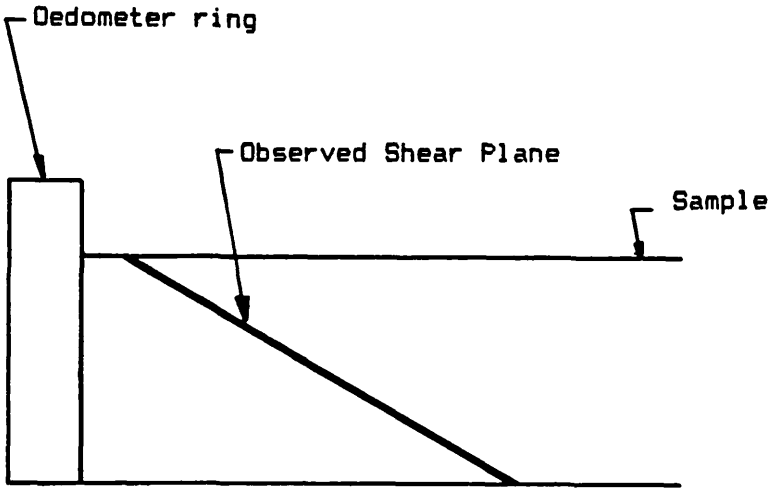
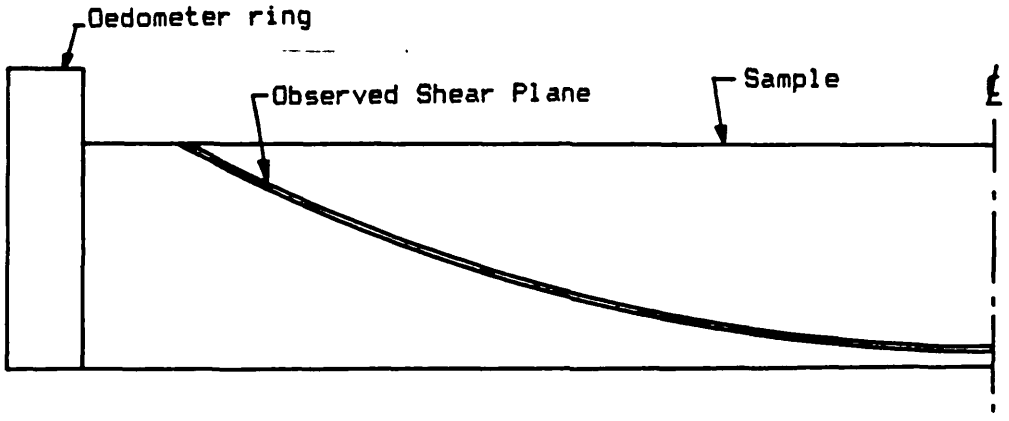


Fig.7.12 Observed and Predicted Swell
South Nyanza Soil



(a) Nairobi-Mombasa Road Soil



(b) Wad Medani Soil

Fig.7.13 Observed Failure Planes

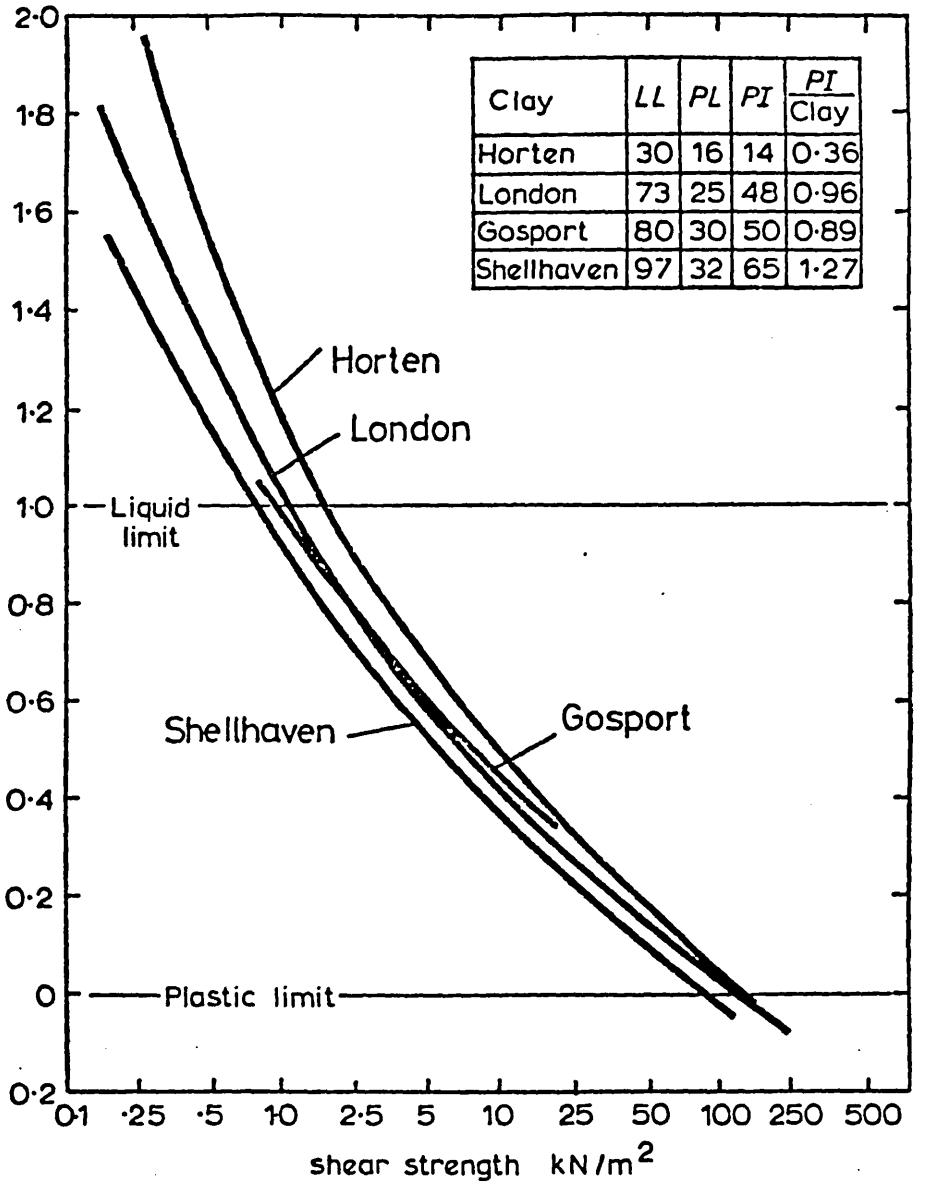


Fig.7.14 Relation between liquidity index and shear strength of remoulded clays

(Skempton & Northey, 1952)

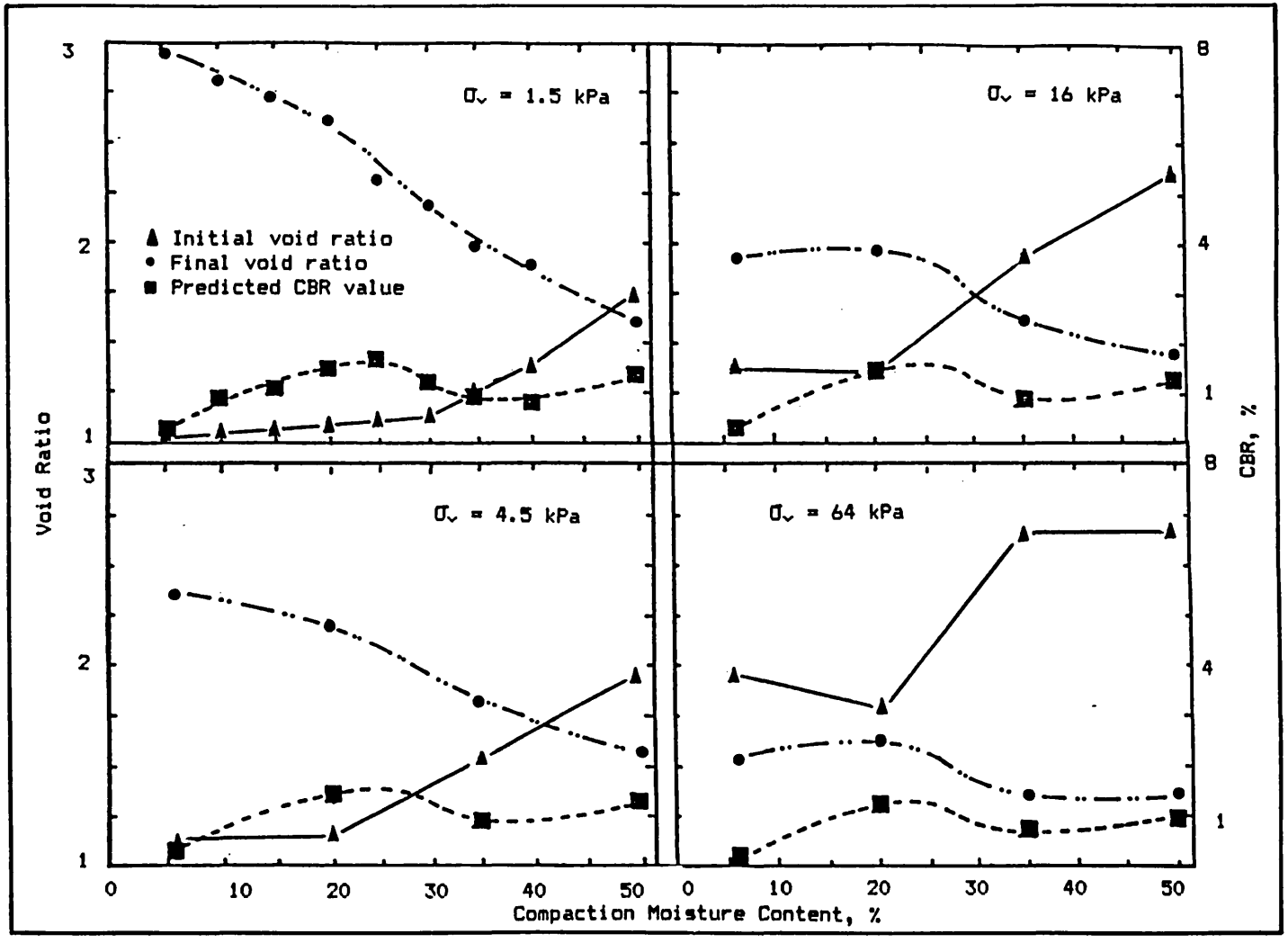


Fig.7.15 Predicted CBR Values

APPENDIX 7A.1

Swelling of Soils Compacted Dry of the Plastic Limit

H.D. Schreiner
Research Assistant

Imperial College of Science and Technology
London, United Kingdom

Synopsis

Tests have been performed on a highly plastic soil from Kenya to determine the effects of compaction moisture content and vertical total stress on volume change during swelling to zero suction. Results from these tests provide an indication of the worst case volume change that could occur beneath roads constructed using the "dry compaction" technique.

The results show maximum swell at low moisture content and low vertical total stress. Swell decreases with increasing compaction moisture content and with increasing vertical total stress.

Introduction

The use of "dry compaction" has been under evaluation by the Transport and Road Research Laboratory for some time. Results to date from an experimental section using expansive soil in the Sudan have shown little change in moisture content over some 8 years (Ellis 1980 & Newill 1987). Little is known, however, about the magnitude of possible movements where climatic conditions permit moisture contents beneath the road to increase significantly from the as-compacted value.

As part of a research programme on swelling of compacted soils, a series of tests has been performed on samples of one soil compacted at moisture contents ranging from the plastic limit to one tenth of the plastic limit. The vertical total stress has been varied from 1.5 to 256kPa.

Soil Properties

The soil sample was collected from the Athi Plain south-east of Nairobi, at km68 on the Mombasa Road. This site was used for a moisture content study by the Transport and Road Research Laboratory and Nairobi University between 1968 and 1973 (Mutonyi 1973). Samples were taken in bags from a depth of .65 to .85m at a distance of about 15m from the north edge of the road.

Table 1 shows the soil properties that have been determined for this sample.

Liquid Limit	125
Plastic Limit	51
% passing 425u	94.5
% <.002mm	67
%montmorillonite	100 of clay fraction
B.S.Std. Dry Density	11.8kN/m³
omc	38%
Salt content, %mass	.52

Table 1: Soil Properties

Test Procedure

The soil was first air dried at a temperature of about 70C to a moisture content of about 6%. The bulk sample was then pulverised in batches to pass a 2mm sieve, and such material as did not readily break down was discarded at intervals during this stage. The batches were thoroughly mixed to obtain a uniform bulk sample. Individual samples were then mixed up to the chosen moisture content and stored in plastic bags.

Tests were performed in a 4inch diameter oedometer in which the sample ring sealed onto the base plate with an O-ring as shown in Fig.1. Water is supplied during the test to a porous plate recessed into the base plate. This layout was used in order to reduce the likelihood of air being trapped within the sample by water entering from both faces at the same time. The porous plates were oven dried and allowed to cool before assembly of the apparatus. In all other respects the oedometer is a conventional fixed ring type in a lever arm frame.

Samples were compacted statically under a stress of 1MPa directly into the greased oedometer rings. This gave dry densities

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similar to the BS Standard compaction curve reported by Mutonyi (1973). The sample thickness was measured after compaction with a vernier depth gauge. After compaction the samples were left overnight in the oedometer cell, which was enclosed in a plastic bag, under the total vertical stress against which they would later swell. Movements were generally small during this stage, but were

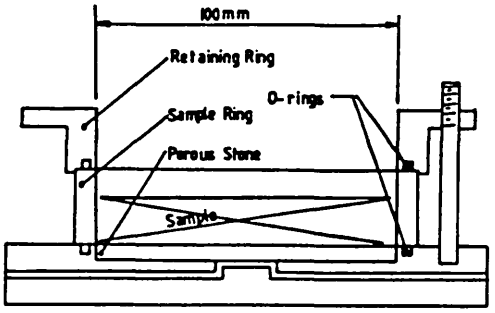


Fig. 1 Oedometer Details

greater at high moisture contents than at low moisture contents, as would be expected. During the swelling stage water was supplied to the lower porous plate under a small head, while the total vertical stress remained constant.

On completion of the swelling stage, the vertical total stress was reduced to 1.5kPa, and the sample allowed to rebound for at least 24 hours. The sample was then unloaded and removed from the load frame. The thickness was again measured for comparison with the movement observed during the swelling and rebound stages.

Results

Although there is some scatter in the data, there are clear indications of the effects of variation of moisture content and total stress. Initial and final void ratios are plotted in Fig.2 against compaction moisture content. The initial void ratios represent the moisture-density curve, inverted as void ratios are inversely proportional to dry densities. The final void ratios under 1.5kPa vertical total stress could be represented by a straight line with a correlation coefficient of .977 despite the fact that the initial void ratios do not lie on a straight line. As the vertical total stress increases, so the final void ratio data is less easily fitted to a straight line.

However, if these results are plotted against both compaction moisture content and log of vertical total stress on a 3-D plot, Fig.3, the final void ratios show a much clearer pattern. The dashed lines represent the void ratio surface under the vertical total stress

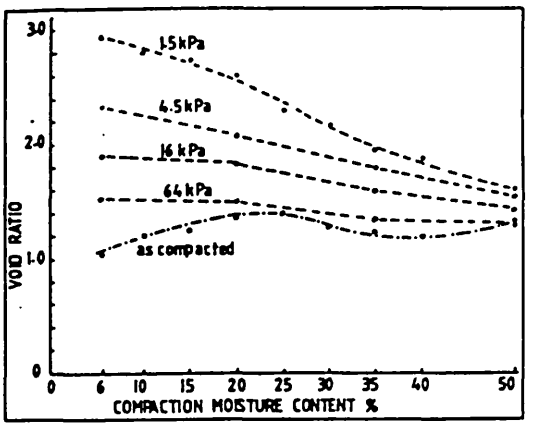


Fig. 2 Initial and Final Void Ratios

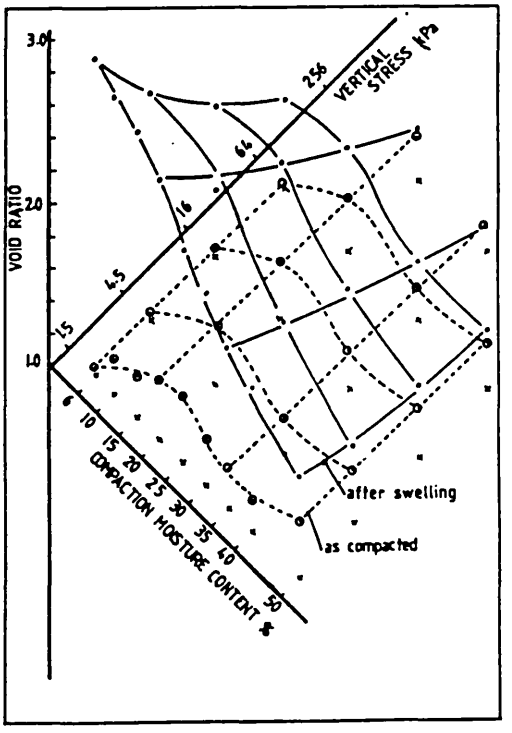


Fig. 3 Initial and Final Void Ratio Surfaces

before swelling. The solid lines represent the final void ratios after swelling, but without unloading to 1.5kPa. In this figure the final void ratios can be seen to mirror the initial void ratios quite distinctly. The influence of the vertical total stress varies with the compaction moisture content, being greater for the low moisture contents than for the high moisture contents. The data for 50% moisture content could be taken to be linear, allowing for some scatter, but those for 6% show a distinctly curved relationship. This may

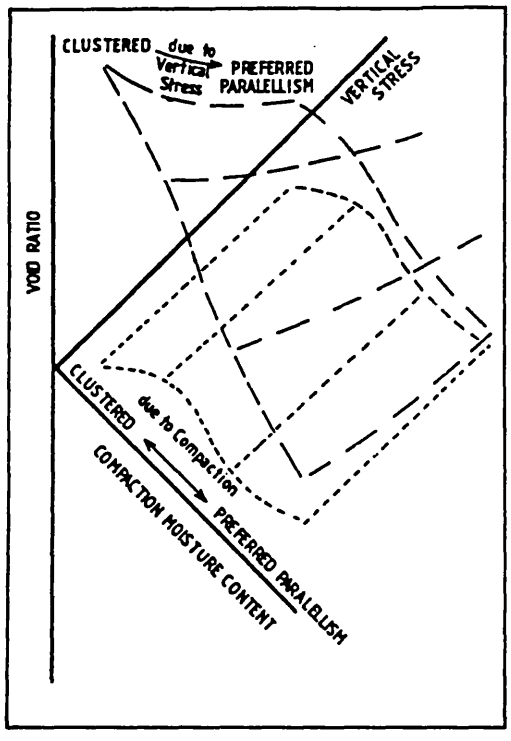


Fig. 4 Soil Microstructure due to Compaction and Stress

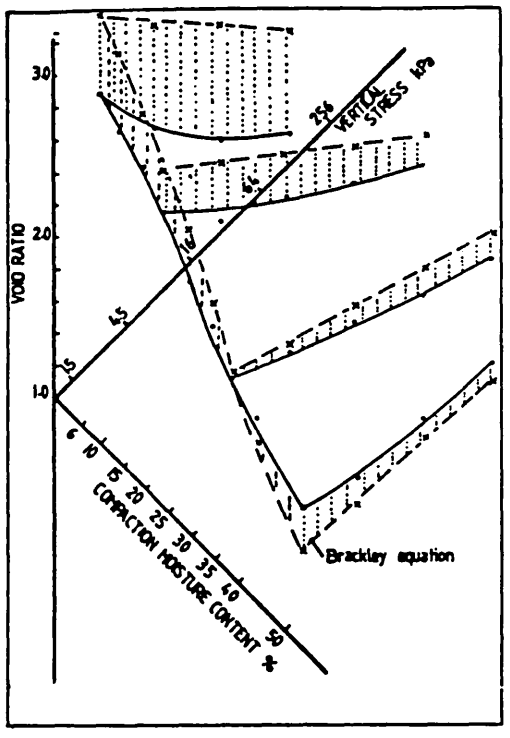


Fig. 5 Comparison with Brackley's Equation

account for some of the conflicting claims for linear (Brackley 1975a, Didier et al 1980, Morgenstern & Balasubramonian 1980) and non-linear (Rengmark & Eriksson 1953 and Salas & Serratosa 1957) swell - log stress relationships.

This data can be further interpreted using some of the findings from research at Strathclyde University. Unfortunately that University invokes copyright restrictions, and theses may not be reproduced without written permission of the Author. Thus only a general description can be given. Zein (1985) extended Collins' (1985) method of describing the structural features of soils, and used this in an SEM study of the structure of a highly plastic soil compacted at various moisture contents. He found that samples compacted at about 2/3 of omc displayed a predominantly clustered type of structure, while those compacted at omc displayed preferred parallelism. There is a progressive change in the soil structure as the compaction moisture content is increased. He also showed that these structures are maintained during swelling under small vertical stresses, and are progressively modified toward the preferred parallelism as the vertical stress during swelling is increased.

In Fig.3 the high void ratio after swelling from 6% moisture content under 1.5kPa is due to a highly random or clustered structure. The lower final void ratios at the higher moisture contents are due to a more oriented or preferred parallelism structure established during compaction. The reduction in void ratio with increasing vertical stress is due both to the stresses acting on the soil and to the changes in the strucutral arrangement of the soil particles. The latter component is more significant for the low initial moisture contents, where the initial structure is highly clustered, and accounts for the more pronounced non-linearity of the void ratio curves. Fig.4 is a simplified version of Fig.3 on which the structural forms have been added, together with the stresses which lead to their existence.

Comparison with other Data

Perhaps the most extensive study of swelling is that by Brackley (Brackley 1973, 1975a&b&c, 1983). One of the products of his work is an equation for the prediction of swell:-

$$S=(5.3 - 147e/PI - \log p)(.525PI + 4.08 - .85w)$$

where
- S= swell in percent
- e= void ratio before swelling
- p= vertical total stress
- w= initial water content
- PI= Plasticity Index

Figure 5 shows a comparison between Brackley's equation and the data for this

paper. It is apparent that, although the trends are similar, the accuracy of the predictions is not good. Brackley's equation appears to be reasonable around omc and for low stresses. It becomes progressively less good at higher stresses and at lower moisture contents. His equation is based on a linear relationship between log of vertical stress and swell, which is not supported by this set of data. The large difference between the predicted and measured values at low compaction moisture contents would not be of importance with normal road construction practice. However, with the increasing use of "dry compaction" or compaction at the field moisture content, (Ellis 1980, Gordon & Waters 1984) predictions for swell from low moisture content are becoming important.

Conclusions

This study has provided some information on the interaction between compaction conditions and vertical total stress on swelling of laterally confined samples of a highly plastic soil to a state of zero suction. While this may not reflect the suction changes that are likely beneath a road in an arid climate, it does show the worst case. Further work is being performed with controlled suction to cover swelling to non-zero suctions.

Acknowledgements

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CHAPTER 8

SUCTION MEASUREMENT

8.1 INTRODUCTION

Measurement of suction in soils cannot, except for the very low range of suction, be made directly. Not only does this make the measurement and the conceptual understanding more difficult, but it also makes calibration and comparison of different methods difficult and indirect. There are several types of equipment and techniques available, and at least as many reports of inconsistencies within some methods and between others. It was decided that a literature study should be undertaken, and one or more appropriate methods selected. Some experimental work was also carried out, and has helped build confidence that the values determined may bear some relation to reality.

Some brief notes on suction are made in section 8.2. More detailed discussion of suction in clay soils was given in Chapter 3. Three suction measurement or control techniques were investigated. Brief comments on these are given in sectio 8.3. Fuller discussions are given in Schreiner (1987a). The next four sections describe the experimental work undertaken with each technique, starting with filter paper measurements in section 8.4. This is followed by pressure plate measurement tests in section 8.5, pressure plate control tests in section 8.6 and, finally, psychrometer measurements in section 8.7.

8.2 COMPONENTS OF SUCTION

The terminology and definition of the components of suction have been presented in section 2.6.4, and the clay-water interaction has been discussed in Chapter 3. Suction in soils has in the past generally been interpreted in terms of the capillary model, in which the suction is identical to that in a small bored glass tube. This is a very simple model of soil water, possibly useable for inert soils such as silts and sands, but it ignores some important aspects of the interaction between clay minerals and water.

If we apply the absorbed water or double layer approaches to water adjacent to a clay particle, other models of suction can be developed. Such an approach was used by Schreiner (1987c) in attempting to clarify the measurement of solute suction in high plasticity clays. The paper is included in Appendix 8A.1

In essence, the conclusion reached is that the suction that exists at the surface of the soil water in contact with the air is primarily caused by the osmotic suction of the salts dissolved in a part of the water, where the distribution of these salts in the water phase is determined by the distribution of the exchangeable cations of the clay. There is no clear boundary between the cations of the dissolved salts and those of the clay, and they must exist in equilibrium with the overall physical and chemical system. Capillary effects are thought to be of secondary importance in this system.

One of the important differences between the capillary model and the soil water chemistry model is in the effect of solute suction on volume change. Bolt (1956) has shown that changes in salt concentration, or solute suction, in a saturated soil alter the void ratio - effective stress relationship as shown in Fig.8.1. The closest that unsaturated soil testing has come to duplicating this is the work of Morgenstern & Balasubramonian (1980) and Pile (1980) which was discussed in Chapter 4. Using the capillary model for suction, this effect cannot be accounted for. However, with the model based on the soil water chemistry, this can be readily explained in terms of the effect on the distribution of cations in the absorbed and free water. Since the clay particles are held apart by the absorbed water, any change in the quantity of water in the absorbed layer will cause a change in the thickness of the absorbed layer which will be reflected by a change in the void ratio.

8.3 SUCTION MEASUREMENT METHODS

The available methods of suction measurement vary in their indirectness. In addition, there is not always clarity in the separation or effect of solute and matrix suction. The method that is the most direct, the pressure plate method, measures the matrix suction but cannot at present be used to measure the solute or total suction. The calibrated filter paper technique and the thermocouple psychrometer have also been used in this investigation, and are discussed briefly below. It is not the intention of this section to provide an exhaustive review, but rather to consider those aspects that are relevant to this research.

8.3.1 CALIBRATED FILTER PAPER

8.3.1.1 Description

This technique has been studied recently at Imperial College, both for total (Duran 1986) and matrix (Gutierrez-Guevara 1985) suction measurement. It has previously been found that the

moisture content of a particular grade of filter paper can be calibrated against the suction or relative humidity to which it is exposed (e.g. Fawcett & Collis-George 1967). Due to hysteresis, the filter paper is always dry at the start of the measurement.

For total suction measurement, the dry filter paper is placed in a sealed container with the soil, but makes no direct contact with it. Equilibrium between the suction in the soil water and in the filter paper water is achieved through the vapour phase, and therefore the influence of the dissolved salts in the soil water is included. For matrix suction measurement, the filter paper and the soil are placed in direct contact, permitting the dissolved salts to be transferred to the filter paper with the soil water. The effect of the salt is thus excluded from the equilibrium between the water in the filter paper and the water in the soil, as the salt will be present in both.

8.3.1.2 Precision

This method can give reasonably reliable results, but the precision of the measurements is not high. The range of water content of the filter paper covering suctions from 100kPa to 1GPa is about 35%. The precision is in part dependent on the precision with which the filter paper moisture content can be determined. As employed for this and the previous projects at Imperial College, the moisture content can be resolved to 0.14%. There are thus 250 discrete values of filter paper moisture content corresponding to 999,900kPa. However, since the relation between suction and filter paper moisture content is logarithmic, the precision varies across the range.

At 35% moisture content, for example, the suction change corresponding to 0.14% moisture content change is 9.5kPa at a suction of 500kPa. At a filter paper moisture content of 10%, the precision is 175kPa at a suction of 8MPa. Measurement to +/-0.14% moisture content of 0.35grammes of filter paper requires measurement of the masses of wet and dry filter paper to +/-0.0005grammes.

Although the figures above suggest that adequate quality data can be obtained, there are many procedural factors which affect the accuracy of the results. Firstly, the water starts evaporating from the wet filter paper as soon as it is exposed to the relatively dry air in the laboratory, and the reverse is true for oven dry papers. Chandler & Gutierrez (1986) report changes of filter paper water content due to wetting or drying of up to 1.5% per minute, e.g. from 30% to 28.5%.

Errors due to this can be reduced using the technique developed by the author and Elsharief while the latter was working at Imperial College, and reported in Elsharief (1987). By placing the wet filter paper discs into a plastic bag immediately on removal from the cell, it was found that drying was of no apparent

significance after the filter paper was placed in the bag. The plastic bag serves as the moisture content "tin" during weighing, but is not placed in the oven. Care must be taken to ensure that the papers are replaced in the same bags after drying that they were in previously. This was achieved by labelling the bags before the first weighing, and the papers before placing them in the cells. The water loss before the bag was sealed is difficult to quantify, but more consistent data were obtained with less scatter. The same approach is used on removing the papers from the oven to prevent water absorption from the atmosphere.

Another possible cause of error is contamination of the filter paper, either by soil particles or by wax used to seal the equilibration cell. This is minimised by brushing the soil particles off the paper or by cutting off the contaminated sections.

For matrix suction measurement, where the paper is in contact with the soil, care must be taken not to apply too great a contact stress. Gutierrez-Geuvarra (1985) presents a water content-stress curve for filter paper which shows considerable reduction in the saturated, zero suction water content with increasing contact stress. Chandler & Gutierrez (1986) claim that errors of up to 50% of the value of suction may have been caused due to excessive contact stress.

Duran (1986) observed that, at low suction, condensation sometimes occurred on the inside of the equilibration cell. This can be explained in terms of the dew-point, i.e. the temperature at which water condenses, which is dependent on the relative humidity which is in turn dependent on the suction. At 100% relative humidity, equivalent to zero suction, the dew point is about 21°C, and decreases by about 1°C as the suction increases to 8000kPa. The dew-point can thus be seen to be close to the laboratory temperature of about 20°C in the low suction range. Contact between the filter paper and the droplets of condensed water will obviously affect the accuracy of the measurements adversely.

8.3.1.3 Calibration

The calibration equation given by Chandler & Gutierrez (1986) was used. This equation compares favourable with those of Fawcett & Collis-George (1967) and Hamblin (1980). There does not appear to be any major variation in calibration between different batches of Whatman No. 42 filter paper.

8.3.2 PRESSURE PLATE

8.3.2.1 Description

The pressure plate technique, or the use of elevated pore air

pressure, has been used extensively for the control of matrix suction, and standard systems are available from the major laboratory equipment suppliers. Use for suction measurement is less common.

Two techniques have been reported for measurement of suction. The earlier technique (Olsen & Langfelder 1965) utilised changes in mass of a series of identical samples placed in separate pressure plate cells at various suctions. The suction at which there would be no change in mass is obtained by interpolation, and is taken to be the matrix suction extant in the soil sample. Since full equilibration is not needed, the samples may be left in the pressure plate cells for a relatively short time, typically less than one day, provided that all samples are in the cells for the same length of time.

The later technique, which has been used in this investigation, relies on the use of stiff pore water pressure measuring systems, such as are available with diaphragm type pressure transducers. A single sample is required, and is placed in a pressure plate cell under an air pressur greater than the expected suction. The pore water pressure is monitored, and the suction determined from the difference between the equilibrated pore air and pore water pressures (Maswoswe 1985, Toll 1988).

8.3.2.2 Precision

The precision of measurements made using the latter technique is primarily dependent on the precision of the gauges used to measure the air and water pressures. The air pressure gauge was the least precise, being graduated to 20kPa. The pressure could be reasonably satisfactorily read to +/- 5kPa, and this may be taken to be the limiting precision.

8.3.2.3 Calibration

There is no direct method of calibrating the pressure plate system for suction measurements. Each gauge was calibrated against a Budenberg dead weight calibrator as described in Chapter 9.

8.3.3 THERMOCOUPLE PSYCHROMETER

8.3.3.1 Description

This apparatus has been used extensively for total suction measurement in soils. The psychrometer generates a cooling current in a miniature thermocouple to condense water onto the thermocouple. Measurement is then made of the "wet - bulb" temperature depression as water evaporates from the thermocouple junction. The temperature depression is controlled by the relative

humidity of the air around the thermocouple. It is assumed that the humidity of the air in contact with the soil is controlled only by the suction of the soil water. Calibration is usually by means of solutions of known concentrations of known salts, from which the suction can be calculated. Although no direct study of the validity of the assumption has been reported, there does not appear to be much evidence to call it into question. There are, however some cases of inconsistencies between thermocouple psychrometers and other methods of suction measurement or control, e.g. Richards (1980) and Edil & Motan (1984).

The apparatus used was a model 55 made by Wescor Inc. of Utah, USA. The samples were placed in a type 52 sample chamber which houses the thermocouple probe.

8.3.3.2 Precision

The output of the Wescor type 55 psychrometer used in this research is about $9*10^{-6}$volts for 3000kPa suction. Each thermocouple probe must be calibrated, as the output varies slightly from probe to probe. The voltage is displayed to $0.1*10^{-6}$volts, giving a resolution of 33kPa.

8.3.3.3 Calibration

Each thermocouple probe must be calibrated against a range of known suctions. This was done using salt solutions of known concentration. Five solutions were made up with concentrations of NaCl as shown in Table 8.2, covering the suction range of 300 to 7000kPa. Suctions were calculated for these solutions using the equation given in Schreiner (1987c). Calibration of the type 52 sample chamber thermocouple was done by placing a small piece of filter paper, soaked in one of these solutions, in the sample chamber. The output was then measured at intervals until a steady value was obtained. The values are listed in Table 8.1 and the calibration is shown in Fig.8.2.

8.4. FILTER PAPER SUCTION MEASUREMENT TESTS

A total of 15 tests were performed, of which four attempted to measure the matrix suction and the remainder attempted to measure the total suction. All of the tests were performed on soil samples from 0.65 to 0.85m depth at the Nairobi-Mombasa Road site. The test data are summarised in Table 8.2. Also noted in the table is the sample type. The initial tests were performed on uncompacted or loose samples. Most of the later tests were performed on compacted samples, but one sample was consolidated from a high initial moisture content.

8.4.1 EQUIPMENT

The cell used for filter paper suction measurement has been described by Duran (1986), and is shown in Fig.8.3. The development of the cell from the original as used by Gutierrez-Guevarra (1985) was done by Duran under the author's supervision. No modifications or further development were made for this project. The two perspex discs are sealed to the ends of the oedometer ring with wax while the filter paper is allowed to reach equilibrium with the suction in the soil.

8.4.2 PROCEDURE

The procedure followed is generally that described by Duran (1986) for total suction and that described by Chandler & Gutierrez (1986) for matrix suction. In both cases the filter papers were placed in plastic bags on removal from the cell as described above in section 8.3.1.2.

8.4.3 FILTER PAPER TEST RESULTS

8.4.3.1 Uncompacted samples

Tests FP2 to FP6 were performed on uncompacted samples placed in sealed plastic bags in place of the equilibration cells which were at that time still under development by Duran. The filter paper discs were separated from the soil by porous plates commonly used in oedometers and triaxial cells. Four 9cm diameter filter paper discs were used in each bag to obtain adequate precision of mass determination using a standard soil laboratory balance of 0.01g precision. In tests FP3 to FP6 four sets of measurements were made on each sample using fresh filter papers for each. The data are plotted in Fig.8.4.

It is apparent from the data from tests FP3 to FP6 and in particular for tests FP4 and FP5 that considerable changes in the measured suction have occurred for the four consecutive determinations. Points labelled 'A' were obtained after 5 days of equilibration and points 'B' after 8 days equilibration with fresh papers. The remaining two points for each sample were obtained by allowing 15 and 19 days respectively for equilibration. Moisture content changes of the soil have been calculated from the amounts of water taken up by the filter papers. The filter paper moisture content in test FP3 was about the same before and after the test, at about 3%. The paper may not have been wetting up during the test, and errors due to hysteresis in the filter paper moisture content-suction relation may have occurred in this test.

It appears from the discontinuity for all samples between points 'A' & 'B' and points 'C' & 'D' that 8 days is not adequate for full equilibration in this system although Duran (1986) found that 4 days was generally an adequate period in the cell shown in Fig.8.3. Initial total suctions have thus been obtained by extrapolating the lines linking points 'C' and 'D' to the initial soil moisture contents.

The mass of water removed from the soil in each measurement ranged from .08g from the driest sample to 0.5g for the wettest sample. Sample masses were of the order of 100g. In order to reduce the changes in suction resulting from the moisture abstraction by the filter paper, it would be necessary to use far larger masses of soil, or far smaller masses of filter paper. At 35% soil moisture content, for example, the total suction changes by about 1.5MPa, or 40% of the initial value, per 1% moisture content change. The latter approach, of using a smaller mass of filter paper, was chosen, and necessitated the use of a scientific balance with 0.0005g precision.

The determination of the moisture content changes from the mass of water abstracted by the filter paper will underestimate the actual changes in moisture content, as there will be some water vapour loss when the bag is opened to remove the filter papers. However, provided that the error is consistent between measurements, the arguments above can still be applied. If all moisture content changes are in error by the same factor then the extrapolation will reach the initial moisture content at the same value of total suction.

8.4.3.2 Compacted samples

The first 3 tests on compacted samples were performed on samples which had been used for pressure plate matrix suction measurement, which are described in section 8.5.3.1. Test FP7 was performed on the sample from test PPM8, FP8 on the sample from PPM9 and FP9 on the sample from PPM10. The samples remained in the oedometer rings for both the pressure plate and the filter paper tests.

Test FP7 measured the total suction of a sample compacted under 1MPa at a moisture content of 34.2%. The moisture content measured after pressure plate and filter paper measurements was 36.3%. It is assumed that the wetting up occurred during the pressure plate measurement, which took 28 hours, at the end of which it was noted that a large quantity of air flowed out from beneath the porous ceramic when the back pressure was decreased from 300 to 30kPa. The total suction measured using filter paper is plotted on Fig.8.5 where the moisture content change is noted.

Also shown on Fig.8.5 are two measurements, FP8 & FP9, which were made on samples compacted under 1Mpa at a moisture content of 25%. The duration of the pressure plate tests on these samples was

about 2 hours, and the change in moisture content is correspondingly low. Test FP9 also demonstrates that the changes in soil moisture content during the measurements is considerably reduced from those apparent in tests FP2 to FP6, but is still measureable.

Tests FP10 and FP13 were performed on samples which were compacted under the same stress, (1MPa), at a moisture content of 33.92%. These show higher suctions than that measured in test FP7, and in close agreement with the deduced total suction from test FP5 on an uncompacted sample. The apparently low value obtained in test FP7 appears to be consistent with the conclusion from the tests on uncompacted samples in that the change of moisture content of the soil has altered the suction in the soil. Similar data on the sensitivity of the suction to changes of moisture content was obtained from test SP04, which is described in secion 10.4, where a sample was subjected to increasing suction in the stress path oedometer. Fig.8.6 shows the matrix suction-moisture content curve for this part of the test. The matrix suction has doubled due to a moisture content change of less than 1%.

Comparison of the total suctions measured on the compacted samples and that deduced from the measurements on loose samples is made in Fig.8.5. The line shown on Fig.8.5 is the curve drawn through the deduced total suction values in Fig.8.4. It appears that compaction does not affect the suction in the soil.

The precision obtained in the measurements can be assessed from the results of test FP10 and FP13. Fig.8.7 shows the results obtained for each of the two filter papers used in each determination. The error is generally within 0.25Mpa about the average value. The maximum error observed was 1.2Mpa about the average value. The scatter in the average values is 1.33MPa, or 0.67MPa about the inferred drying curve. This represents a potential error for a single measurement with two papers of about 16%.

A further two tests, FP14 and FP15, measured the matrix suction of samples compacted under 1MPa at a moisture content of 33.92%. These show matrix suctions of 2.6 to 3.5MPa. These values are considerably higher than the matrix suctions measured for similar samples using the pressure plate method, and will be discussed further section 8.5.3.1 which deals with those tests.

8.4.3.3 Samples at known suction

Test FP11 measured the total suction of a sample equilibrated at 400kPa matrix suction in a pressure plate cell, and gave a mean value of total suction of 1080kPa. Test FP12 measured the matrix suction of another sample equilibrated at 400kPa matrix suction in a pressure plate cell, and gave a mean value of matrix suction of 398kPa. Both sets of data are included in Fig.8.5. Since the suction history of these samples differed, in that the sample in

test FP11 had been wetted to the final suction and that in test FP12 had been dried from 50kPa suction to the final suction, the final moisture contents differed. However, the latter test, FP12, confirms that the filter paper technique can measure the matrix suction which had previously been established using elevated pore air pressure in a pressure plate cell. The difference between the suctions measured in these two tests, of about 700kPa, is the solute suction, or the suction due to the dissolved salts in the pore fluid.

One test to measure the matrix suction, FP16, was made on a sample consolidated under 1000kPa and unloaded without drainage. The details of the sample are given in section 8.5.2.2 below where the pressure plate suction measurements made on it are described. The results of test FP16 are also shown on Fig.8.5. The values of suction measured with the filter paper, of about 530kPa, are lower than the 770kPa measured with the pressure plate. The scatter in the calibration data is large, and for the filter paper moisture content of 34% obtained in these tests, the range of data is 350 to 700kPa in the summary figure presented by Chandler & Gutierrez (1986). The expected value thus falls within the scatter range, but is at one extreme.

The data given in their summary figure have, as far as is known, all been obtained without the use of a plastic bag to minimise the moisture loss from the paper during weighing. It is possible that the low values reflect the improved moisture content determination, and that an improved calibration curve is now required, in which the experimental technique is the same as the technique now used for making the measurements of suction on soil samples.

8.5 PRESSURE PLATE MEASUREMENT TESTS

A total of 15 measurements of matrix suction were made on samples from the Nairobi-Mombasa Road site at 0.65 to 0.85m depth which were compacted under 1MPa, at various moisture contents, into 76mm oedometer rings. The sample thicknesses after compaction were less than the height of the ring. One test was performed on a natural sample from Wad Medani which had been wet up in the laboratory to the moisture content measured in the field where heave had damaged a house (Elsharief 1987).

8.5.1 EQUIPMENT

The Stress Path Oedometers, which are described in Chapter 9, were used for the measurement of suction by the elevated pore air pressure method. For this useage, the cells were assembled without the hydraulic loading membrane, and the radial load cell was not installed during most of the tests. The top plate with the compensated plunger was generally replaced by a blank flange with

a fitting for the air pressure tubing. Water pressure beneath the ceramic was measured with a transducer mounted into a block screwed directly into the base plate. The drain tubing on the other end of the spiral drainage groove in the base plate was steel between the base plate and the tap to maximise the stiffness of the measuring system. A typical layout is shown in Fig.8.8. ~~Water pressure beneath the ceramic was measured with a pressure transducer mounted in a block screwed directly into the base plate.~~

8.5.2 PROCEDURE

No clear guidance could be found on the final preparation of the ceramic. Current practice at Imperial College was to wipe the ceramic to leave it slightly wet (Toll 1985), but indications in the literature were that the initial wetness of the ceramic could affect the response time of the system. Some of the tests were designed to investigate the effect of the initial wetness of the ceramic on the response time and the magnitude of the measured suction.

The procedure followed varied in some details during the test programme. The general procedure remained constant. The saturation of the ceramic was checked in accordance with the technique described in section 9.4.2, usually on the day before the test was to be performed.

The cell was refilled with de-aired water at high pressure and left until all other preparations were complete. The drainage channels beneath the ceramic were flushed using the mercury pot system to remove any air that might have accumulated during the saturation checking procedure.

Soil at the appropriate moisture content was taken from the storage bags and immediately placed into the oedometer ring for compaction. The compaction procedure has been described in section 7.2.1. After compaction the sample was temporarily stored in a plastic bag to minimise moisture loss while the cell was prepared.

This involved draining the water from the chamber, removing the top plate and wiping the chamber wall to remove drops of water. The top plate was then replaced and an air pressure applied until the flow of water into the volume gauge ceased. The ceramic was then considered to be "dried" at that air pressure.

The wetness of the ceramic was then adjusted as described below for each test. The sample was then removed from the plastic bag and placed on the ceramic with moderate finger pressure to ensure contact between the soil and the ceramic. Finally the top plate was replaced and the air pressure applied. Air and water pressures were monitored over a period of time and the difference between these taken as the matrix suction.

8.5.3 TEST RESULTS

8.5.3.1 Tests on compacted samples

The first three tests, PPM1 to PPM3, were performed on one sample which was compacted at a moisture content of 43%. The first test was performed immediately after the sample was compacted and the second and third tests six and seven days after compaction. In between tests the sample was stored in plastic bags.

For test PPM1 the ceramic was dried under an air pressure of 1000kPa. After the air pressure had been reduced to atmospheric, 0.15cm^3 of water was allowed to flow through the ceramic under about 25kPa back pressure before the drainage taps were closed. For test PPM2 the quantity of reverse flow was 0.31cm^3, while for test PPM3 the drainage taps were closed before the air pressure was reduced, thus preventing reverse flow.

The data are presented in Fig.8.9. The vertical axis gives the measured difference between the air and water pressures. The horizontal axis shows the elapsed time since application of the air pressure with the sample in place. It can be seen that the initial wetness of the ceramic influenced the initial water pressures after the application of the air pressure. In test PPM2, where the ceramic was wettest, the water pressure at the start was equal to the applied air pressure, indicating that there was sufficient free water to activate the pressure measurement system. Test PPM3 showed no initial water pressure response, indicating that no free water was available to activate the pressure measurement system.

Test PPM1 indicates a suction of 650kPa, after 3hours and 20mins. Measurements taken during the next two days are not plotted in Fig.8.9. These showed a continual decrease in the pressure difference. After 19 hours or 1140 minutes, the pressure difference was about 600kPa, falling to 435kPa after a further 24 hours. This change with time was due to rising pressure below the ceramic, as described below, and does not necessarily reflect the suction in the soil. Similar time dependent increases in the water pressure beneath the ceramic were observed by Toll (1988), and explained as partial blow through. It is thought that the following explanation is more reasonable.

Since the air pressure used for the measurement of suction must prevent cavitation in the water in the measurement system, high air pressure is necessary. The rate of diffusion of air into the water in the ceramic, and thus into the low pressure water below the ceramic, is dependent on the air pressure. Such air as does pass through the water in the ceramic can and will form bubbles as it comes out of solution in the water which is at a lower pressure.

If the water is initially de-aired, then there will be an amount of air that can be held in solution before bubbles will form. Once bubbles do start forming, then the pressure in the

water in the pore water pressure measurement system beneath the ceramic will tend to rise. However, this will cause an imbalance between the suction in the soil and the pressure difference across the ceramic, and water will flow through the ceramic into the soil due to the pressure imbalance. The rate of water flow from the measurement system into the sample will depend on the excess pressure in the water, the permeability of the ceramic and the permeability of the soil.

The excess pressure beneath the ceramic will be dependent on the relative rates of flow of air into and water out of the pore water pressure measurement system. If the rate of water flow out of the measurement system is fast enough, then no significant excess pressure will be developed. If, however, the flow of water out of the measurement system is slow due to the permeability of the ceramic or the soil, then an excess pressure may by generated. This appears to have occurred in some of the tests and is apparent as a small but constant rate of increase in the water pressure beneath the ceramic. In this case the difference between the air and water pressures will not reflect the true matrix suction, and the matrix suction in these and subsequent tests has therefore been taken to be the difference between the air and water pressures immediately before a constant rate of water pressure increase occurs.

The flow of water into the sample will alter the matrix suction in the soil. It is likely that, in a soil of low permeability, the suction at the surface of the sample in contact with the ceramic will be equal to the difference between the air and water pressures, and that there will be an increase in suction with distance away from the ceramic. On removal of the sample from the test chamber, the suction gradient in the sample will dissipate, leaving the suction at a value different from the initial value.

Tests PPM2 and PPM3 reflect this change in matrix suction due to water inflow. The measured suction in both tests was 525kPa, which is lower than that measured on the same sample in test PPM1. The moisture content changes of the sample were not determined after these tests. It had been intended that the test series would continue, but it became necessary to reseal the joint between the ceramic and the base plate and the test series was thus discontinued while the repairs were undertaken. The repairs took much longer than anticipated, and the test series was not restarted. However, in a later test which was continued for 28 hours, the sample took in 0.32g of water, corresponding to a change in moisture content of 0.8%.

Two more series of tests were performed on compacted samples. Tests PPM4 to PPM8 used samples compacted at 35% moisture content, and tests PPM9 to PPM11 were on samples compacted at 25% moisture content. The former are presented in Fig.8.10, and the latter in Fig.8.13.

In test PPM4 the ceramic was sufficiently wet for the initial water pressure response to be equal to the applied air pressure. For test PPM5 the ceramic was dried under an air pressure of 800kPa. The drainage taps were closed at intervals during the drainage until a water pressure increase of 100kPa after closure was obtained. The taps were then left closed and the air pressure released. A paper towel was used to dry the ceramic in test PPM6, after which about 0.12cm^3 of water was added. This was done using a wash bottle for which the drop size had been determined by weighing 10 drops in a moisture content tin. Preparation for test PPM7 was similar, except that no water was added after the ceramic was dried with the towel. The ceramic was dried under 1000kPa air pressure in test PPM8, and no water was added.

The data for the samples at 35% moisture content are plotted in Fig.8.10. None of the samples was used for more than one test. Tests PPM4 to PPM6 gave consistent results, showing a matrix suction of 940kPa. In test PPM4 the air pressure was reduced from 1400kPa to 940kPa after the maximum difference between air and water pressures had been recorded. The trend of pressure changes after this air pressure change was toward the same value of suction, although the rate of water pressure decrease was slower than in the first part of the test. ~~The slow response time is consistent with the presence of air in the measuring system, as discussed above.~~ The slower response time after the air pressure change is consistent with the presence of air in the water pressure measuring system, as discussed earlier in this section.

Tests PPM7 and PPM8, for which the ceramic had been dried and no water added, showed a progressive increase in apparent water pressure beneath the ceramic. It is believed that this reflects the mechanism discussed above of air diffusion through the ceramic affecting the water pressure.

The initial apparently high value of matrix suction is thought to be due to the dry starting condition of the ceramic, but is in conflict with the apparently succesful measurements with a dry ceramic in tests PPM3, shown in Fig.8.9, and PPM12, performed on an undisturbed sample from Wad Medani which was tested on behalf of Elsharief (1987), shown in Fig.8.11. In test PPM12, the ceramic was dried under 1000kPa, followed by 0.26cm^3 of reverse flow. This should have caused the water pressure to respond fully to the applied air pressure. However, there had been no resaturation of the ceramic after the previous test, and there was probably air trapped in or below the ceramic.

In all three of these tests the ceramic was dried to a state where the initial water pressure response was lower than the expected equilibrium value. It is thought that the apparently incorrect measurements in tests PPM7 and PPM8 may reflect poor contact between the water in the ceramic and that in the soil.

The moisture content in test PPM3 was about 84% of the plastic limit, and that in test PPM12 about equal to the plastic limit.

The soil in tests PPM7 and PPM8 was considerably drier, at about 69% of the plastic limit. The effect of low moisture content on the structure of the soil has been discussed in Chapter 7 based on the work of Collins (1984) and Zein (1985). The drier soil will have a more clustered structure than the soil compacted in a wetter state. The structure of the undisturbed soil is not known.

Fig.8.12a shows diagrammatically what is thought to occur when a clustered soil is placed on a ceramic which has been dried under 1000kPa suction. The water has retreated into the pores of the ceramic until the radius of curvature is in equilibrium with the suction, in this case being about $1.4*10^{-4}$mm. The soil, which is granular and dry to the touch, can rest on the dry parts of the ceramic without coming into contact with the water. The pore water of the soil thus remains isolated from the water of the pressure measurement system, and the changes in water pressure that were observed are unrelated to the stress in the soil water.

Fig.8.12b shows what is thought to occur when a wetter soil with a preferred parallel structure, as is expected for a soil compacted near the optimum moisture content, is placed on a ceramic dried to the same state. There is more "free" water in the soil, and contact is more easily made between the soil water and that in the ceramic. The water pressure in the measurement system thus responds to the suction or pore water pressure in the soil.

Similar arguments can be applied to explain the apparently high values of "matrix" suction found in tests FP14 and FP15, as shown in Fig.8.5. Continuity is required between the water in the filter paper and in the soil for matrix suction measurement. However, if the water in the soil does not make sufficiently good contact with the filter paper to flow into the pores of the paper, then this type of measurement will not be correct. The paper will take up water by vapour transfer, as for total suction measurement, and the final state may resemble that depicted in Fig.8.12 with the ceramic replaced by the cellulose of the filter paper fibres. If this is the case, then the suction measured will be the total suction.

The preparation of the ceramic for tests PPM9 and PPM10 used drying under 1000kPa followed by addition of 0.12cm^3 of water from a wash bottle. Test PPM11 differed in that the air pressure used was 500kPa, and 0.11cm^3 was added from a small bore stainless steel tube connected to a screw pump. This permitted more precise control than did the wash bottle.

Tests PPM9 to PPM11 indicate a matrix suction of 1280kPa as shown on Fig.8.13. In test PPM11 the air pressure was raised from 1300kPa to 1400kPa after 25 minutes when it appeared that negative water pressures beneath the ceramic were being measured. The "equilibrium" conditions were as a result approached from the opposite direction to that in the first two tests. The apparent value of suction reached after 110 minutes is consistent with the values obtained in the first two tests.

8.5.3.2 Consolidated sample

Tests PPM13 to PPM15 in Fig.8.14 were performed on one sample which had been consolidated under a vertical stress of 1000kPa for 23 days from a moisture content of about 100%. The sample was unloaded as far as is possible in the undrained state, and free water on the sample surface was removed with a paper towel when the sample was removed from the oedometer. The sample remained in the oedometer ring during the suction measurement. The ceramic was prepared by drying under 1000kPa followed by 0.05cm^3 added with the stainless steel tube.

Tests PPM13 and PPM14 indicate a matrix suction of about 770kPa. It had been expected that the suction would be equal to the previous vertical effective stress, but it appears that, for this normally consolidated sample, the suction is equal to the mean effective stress. For a K_o of 0.6, as determined in test SP06 in the stress path oedometer which is reported in section 10.4, the mean effective stress at 1000kPa vertical stress is 733kPa. The measured value of 770kPa is sufficiently close for these tests to confirm that the pressure plate system is measuring the matrix suction correctly.

PPM15 was performed after a series of filter paper suction measurements in test FP16, and indicates a matrix suction of 830kPa, probably due to the loss of water to the filter paper and during transfer from sealed containers. It was also noted in this test that the sample had shrunk away from the oedometer ring, supporting the contention above that the suction in this case is equal to the mean effective stress.

8.5.3.3 Equilibrated sample

As a further check on the reliability of the measurements, two samples were prepared in the pressure plate control cell to a matrix suction of 400kPa. Tests PPM16 and PPM17 were performed on these samples. The air pressure used for drying the ceramic was 540kPa, since the expected suction was about 400kPa, and 0.05cm^3 water was then added with the stainless steel tube.

The data are shown in Fig.8.15, and indicate a matrix suction of 360kPa. This is thought to be reasonable considering that the samples had been removed from one cell and transferred to another. The sample from test PPM17 was subsequently used in test FP11, and that from test PPM16 was used in Test FP12.

8.5.4 RESPONSE TIME OF SYSTEM

Various techniques were tried in an attempt to minimise the response time of the water pressure measurement system. The target measurement is the pore water pressure in the soil when the pore

air pressure is raised sufficiently to avoid cavitation.

In any fluid pressure measurement system, there is a quantity of fluid required to deform the measuring element and to cause elastic deformations of the whole measuring system. In general the system and measuring element are made as stiff as is reasonably possible, and the least stiff component involved is the soil. When a stress change is applied or the soil is brought into contact with the measuring system, a small quantity of water must flow into or out of the soil to activate the measuring sytstem. If the soil is relatively permeable then the response time of the system will not be adversely affected by the flow of water. If the soil is highly impermeable then the response time may be dependent on the rate of flow of water into or out of the soil, and not on the design of the system.

Since it was thought that the amount of free water on the ceramic could influence the response time, the "wetness" of the ceramic was varied for 2 series of tests on similar samples. These were PPM1 to PPM3 and PPM4 to PPM8 which have been described above.

Tests PPM1 to PPM3, shown in Fig.8.9, were performed at the start of the suction measurement programme, and can be considered to be preliminary tests in regard to the development of experimental techniques. It is however, clearly evident in Fig.8.9,that the response time is affected by the amount of free water available, with test PPM2 which had 0.31cm^3 free water responding much more slowly than test PPM1, in which 0.15cm^3 free water had been supplied.

For test PPM4 the ceramic was wet at the start. This was checked by ensuring, before the sample was installed, that the water pressure followed the applied air pressure, thus demonstrating that there was sufficient free water to activate the measuring system. The response time of the whole system, including the soil can be seen from Fig.8.10 to be about 3.5 hours. The response time of the system alone has been measured with water above and below the ceramic and was found to be about 1 minute to 90% of the applied pressure, as shown in Fig.8.16. The extended response time is thought to be due to the rate of flow of water into the soil.

In test PPM5 the ceramic was dried by applying an air pressure and allowing drainage until an applied air pressure of 800kPa gave a water pressure response of 100kPa. The water pressure measuring system was thereafter kept closed while the sample was placed in the chamber and an air pressure of 1100kPa applied. The water pressure did not drop below -25kPa during this procedure. The response time appears from Fig.8.10 to be about 40 minutes, which is still longer than that of the system alone. The change in mass of the soil sample during this measurement was 0.03g, corresponding to a moisture content change from 31.05% to 31.12% showing that very little water was taken up by the soil. The

response time of test PPM6 lies between the above two tests, reflecting the addition of 0.15cm^3 of water with the wash bottle.

The water addition technique was refined further in tests PPM9 to PPM 11 and the response time was reduced to about half an hour. It appears that consistently short response times can be achieved for suction measurement with the pressure plate system if the required amount of free water is determined and supplied. For this particular pressure plate apparatus, the required amount of water has been shown to be about 0.1g when the ceramic has been dried under an air pressure of 1000kPa. The quantity of water required is likely to vary considerably between systems, and may also be influenced by the suction in the soil.

8.6 PRESSURE PLATE CONTROL TESTS

Three samples were subjected to wetting and drying under controlled suction in the pressure plate control cells. One sample was equilibrated at a single suction value for subsequent pressure plate suction measurement. Completion of one wetting and drying cycle required upwards of 4 months, with two tests running for 9 months.

8.6.1 EQUIPMENT

The cells used for suction control are commercially available cells made by Soilmoisture Equipment Ltd of California. The system for their operation is described by Toll (1988), and is shown in Fig.8.17. The air pressure, and thus suction, limit of these cells is 500kPa.

8.6.2 PROCEDURE

Samples for pressure plate control of suction were compacted under 1MPa at the chosen moisture content into 76mm oedometer rings. The sample was then placed on the ceramic of the pressure plate cell, which was then closed and the air presure applied. In tests PPC1 and PPC2 the moisture content changes were calculated from the measured water flow through a burette volume gauge into or out of the sample. Since the volume measurements did not appear to be particularly accurate, the later tests used changes in sample mass to determine the moisture content changes. This required the removal of the samples at intervals for weighing. Each stage was continued until the change in mass was insignificant in terms of moisture content change.

8.6.3 RESULTS

The data from these tests are shown on Fig.8.18 as suction-moisture content curves. Each point represents the

moisture content of the soil after equilibration under the applied suction in the pressure plate cell. All tests except PPC3 started at a high value of suction and proceded to lower values before starting the drying stages. Test PPC3 was wetted from the compacted state to 50kPa suction in the first stage and then subjected to drying in stages to 475kPa suction, wetting in stages to 100kPa suction and finally dried to 250kPa suction.

The data from test PPC1 show a much larger increase in the moisture content than do tests PPC3 and PPC4. It is thought that this reflects a small rate of loss of water from the measuring system which continued during the long test period. The data from this test are thought to be incorrect. The same may be true of PPC2 which was performed using the same technique for measuring the moisture content change. Tests PPC3 and PPC4 were performed using the improved technique of weighing the samples at intervals.

It is worthy of note that the suction-moisture content curves for the two samples, PPC3 and PPC4, for which good quality data were obtained did not converge. The effect of sample history persists through at least one cycle of wetting and drying. It is also of interest to note the steepness of the drying curves, particularly that for test PPC3 from 100 to 250kPa. This indicates that the shrinkage on reversal of the moisture content change may be small for the initial stages of drying.

Also shown on Fig.8.18 is the consolidation curve obtained for a sample prepared as a slurry in test SP06 which is described in section 10.4. Brackley (1975) proposed the use of the "packet void ratio" which is calculated on the assumption that the packets or clusters remain saturated. The packet void ratio is thus 2.65/100 times the moisture content in percent, as is the void ratio of a fully saturated sample. Thus it can be seen that, since the moisture content of the compacted samples is less than the moisture content of the saturated sample at any value of suction, the packet void ratio of the compacted samples is less than the void ratio for the consolidation curve.

This implies that the clay in the packets is "overconsolidated" or that the particles have previously been at closer spacing than the current spacing. This is fully consistent with overconsolidation of saturated soils, but has not been demonstrated for unsaturated soils for the effect of suction. If the soil during and after wetting up is overconsolidated, then the development of a model for volume change becomes dependent on the degree of overconsolidation. The volume change of the packets will be dependent on the maximum past stress and suction, and there will be no unique final state after swelling. This is not the case for collapse, where Maswoswe (1985) has found that the soil approaches the soaked compression line after collapse.

It is possible that the compaction stress may have contributed to this "overconsolidation" but compaction is generally taken to be undrained loading. The suctions in all of these samples has

been considerably higher, both when air dried to 6% moisture content and when prepared at the compaction moisture contents, than the suctions used in the testing, and it appears reasonable to consider the suction to have been the major factor in the "overconsolidation".

8.7 PSYCHROMETER SUCTION MEASUREMENT TESTS

One test, TP1, was performed in the Wescor type 52 sample chamber on the sample which had been equilibrated to 400kPa matrix suction in test PPC1. This sample was used in test PPM16, shown in Fig.8.15, and then test FP12, shown in Fig.8.5, for matrix suction before being tested for total suction with the psychrometer in test TP1. The sample from test PPC2 was used for a matrix suction determination in test PPM17, shown in Fig.8.15, and then in test FP11 for total suction. These tests thus provide a full spectrum of measurements, both in test type and suction component, on two samples which had been equilibrated to the same matrix suction of 400kPa. The results are tabulated below for comparison.

Suction measurements

Sample Source	Pressure Plate (matrix)	Filter Paper (matrix)	Filter Paper (total)	Psychrometer (total)
PPC1	360 (PPM16)	398 (FP12)		1050 (TP1)
PPC2	365 (PPM17)		1100 (FP11)	

Table 8.3 Comparison of different suction measurements

There is good agreement between the various types of suction measurement on these samples. It appears that the matrix suction, as established in the presssure plate control tests has been measured with reasonable accuracy by the pressure plate measurement technique, and that total suctions measured by filter paper and by psychrometer are comparable. The solute suction of these samples is about 700kPa which is considerably more than the matrix suction. If the effects of solute and matrix suction are identical with regard to volume change, then the control of matrix suction alone will be inadequate for the study of swelling of soils with significant salt contents.

8.8 SUMMARY

Three methods of suction measurement or control have been

used. All have been shown to be reliable, and comparable results have been obtained in most cases. A limitation in the use of filter paper for matrix suction measurement has been found where the sample is too dry for contact between the water in the soil and in the filter paper to be established. Similar difficulties have been observed in the pressure plate suction measurement system. The pressure plate control tests have demonstrated that the relation between suction and moisture content is not unique but appears to be dependent on the suction history. The moisture content of the samples from the pressure plate control test has been used to show that the soil in the packets or clusters appears to be overconsolidated. The measured value of matrix suction from the pressure plate system has been shown to be insensitive to the initial wetness of the ceramic, provided that contact between the water in the soil and the water in the ceramic is established. Short response times have been shown to be possible by controlling the amount of free water on the ceramic.

Solution No.	Concentration of NaCl in g/g water	Suction @ 20C kPa	@25C	Elapsed Time, min	Output @21.8C volt*10_{-6}
1	0.01452	1130	1134	0.3	4.5
				2.3	4.2
				4.0	4.0
				6.0	4.1
2	0.00408	319	324	0.3	4.7
				4.0	2.4
				6.0	2.0
3	0.0238	1823	1855	0.1	6.3
				2.0	6.4
				4.0	6.6
				6.0	6.5
4	0.0402	3097	3155	0.3	10.9
				1.0	9.8
				2.0	9.4
				3.0	9.4
				4.0	9.3
				6.0	9.3
5	0.0848	6744	6886	44.0	21.6

TABLE 8.1 PSYCHROMETER CALIBRATION DATA

Test No	Equil Time	Ws %	Uaw MPa	Equil Time	Ws %	Uaw MPa	Equil Time	Ws %	Uaw MPa	Equil Time	Ws %	Uaw MPa	Equil Time	ws %	Uaw MPa	Note
FP2	10	30	-5.26													L
FP3	5	5.93	-48.1	8	5.88	-48.1	15	5.82	-46.2	19	5.75	-43.8				L
FP4	5	19.77	-20.9	8	19.55	-22.5	15	19.34	-22.5	19	19.13	-23.6				L
FP5	5	34.24	-5.94	8	33.55	-7.2	15	32.85	-6.94	19	32.21	-8.4				L
FP6	5	48.83	-2.78	8	47.81	-3.95	15	46.75	-3.26	19	45.68	-3.4				L
FP7	11	36.33	-2.5 +/-.07													C
FP8	10	24.98	-12.2 +/-1.2													C
FP9	22	24.87	-13.90 +/-1.5	11	24.74	-13.85 +/-1.3	23	24.61	-13.8 +/-.41	9	24.48	-14.56 +/-3.1	12	24.35	-11.99 +/-.05	C
FP10	16	33.6	-3.99 +/-.10	11	33.33	-3.27 +/-.64	23	33.09	-4.6 +/-.15	9	32.85	-3.85 +/-.27	12	32.61	-3.61 +/-.21	C
FP11	20	45.0	-1.08 +/-.10	11	44.56	-1.03 +/-.04	23	44.13	-1.19 +/-.13	9	43.67	-0.98 +/-.04	12	43.24	-1.08 +/-.02	E
FP12	7	40.6	-.398													EM
FP13	10	33.82	-3.67 __	23	33.72	-4.15 +/-.08	9	33.62	-3.17 +/-.01	12	33.52	-3.15 __	38	33.42	-3.38 +/-1.2	C
FP14	7	33.37	-2.64 +/-.09	9	33.22	-2.86 +/-.11	8	33.07	-2.78 +/-.07	38	32.92	-2.94 +/-.20				CM
FP15	12	35	-3.63 +/-.13													CM
FP16	29	__	-0.51 +/-.04	22	__	-0.57										ConM

TABLE 8.2 FILTER PAPER SUCTION MEASUREMENT DATA

Key
Ws = soil moisture content
Uaw= pore water pressure relative to atmospheric pore air pressure
L = loose
C = compacted
E = equilibrated
Con = consolidated
M = matrix suction

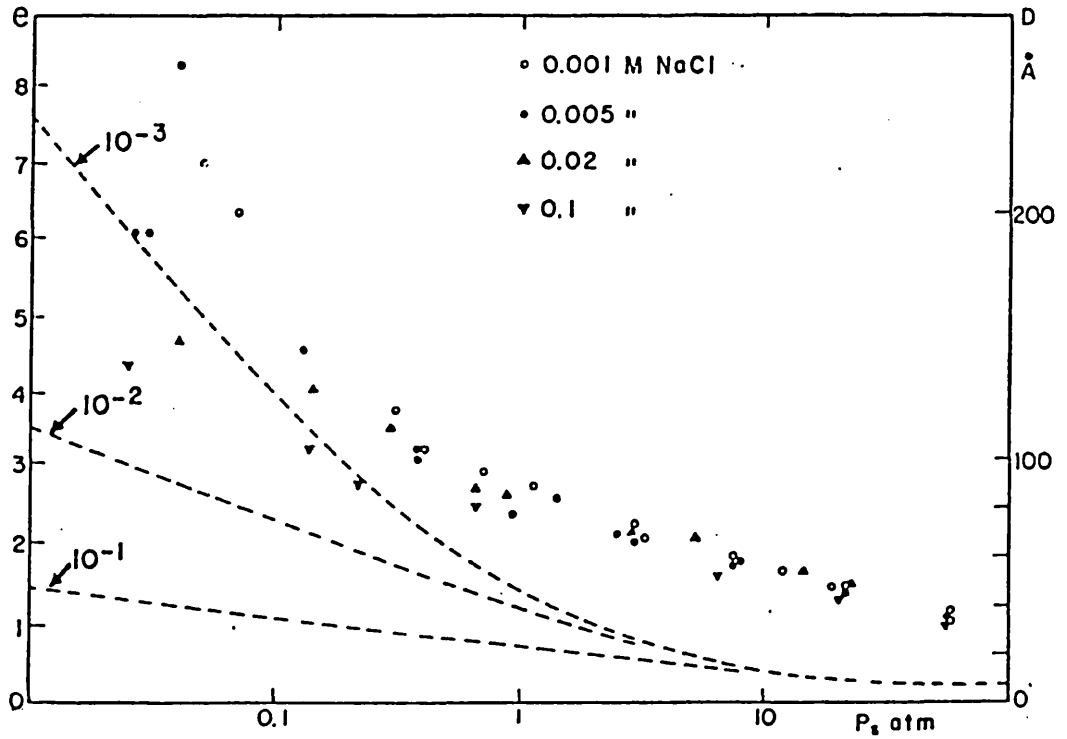


Fig. 8.1 Compression curves of Na-illite, fraction $< 0{\cdot}2\mu$ in equilibrium with different concentrations of NaCl, given as molarity. The dashed lines represent the theoretical curves for $S = 120$ m²/g.

(Bolt 1956)

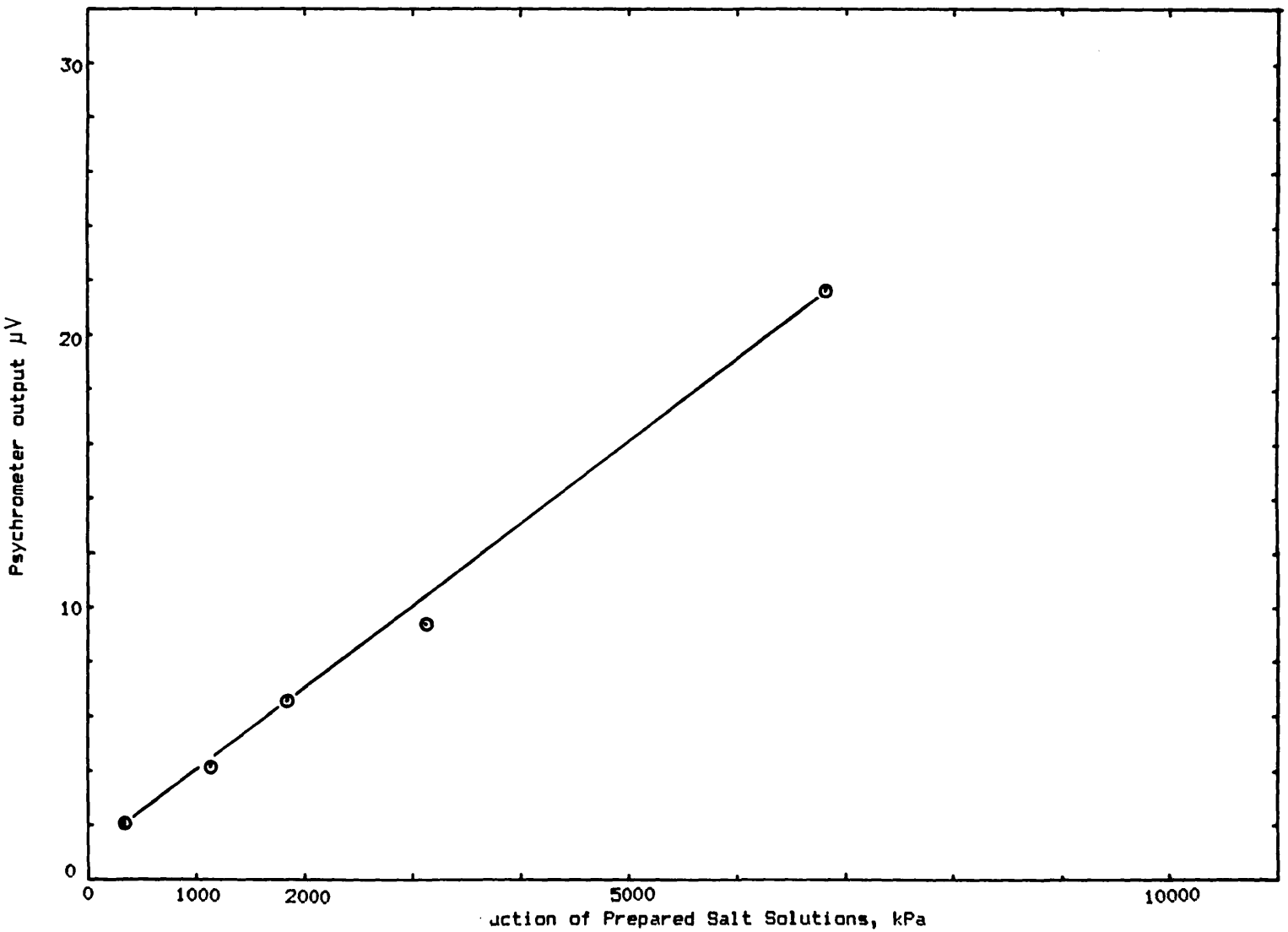


Fig.8.2 Psychrometer Sample Chamber Calibration

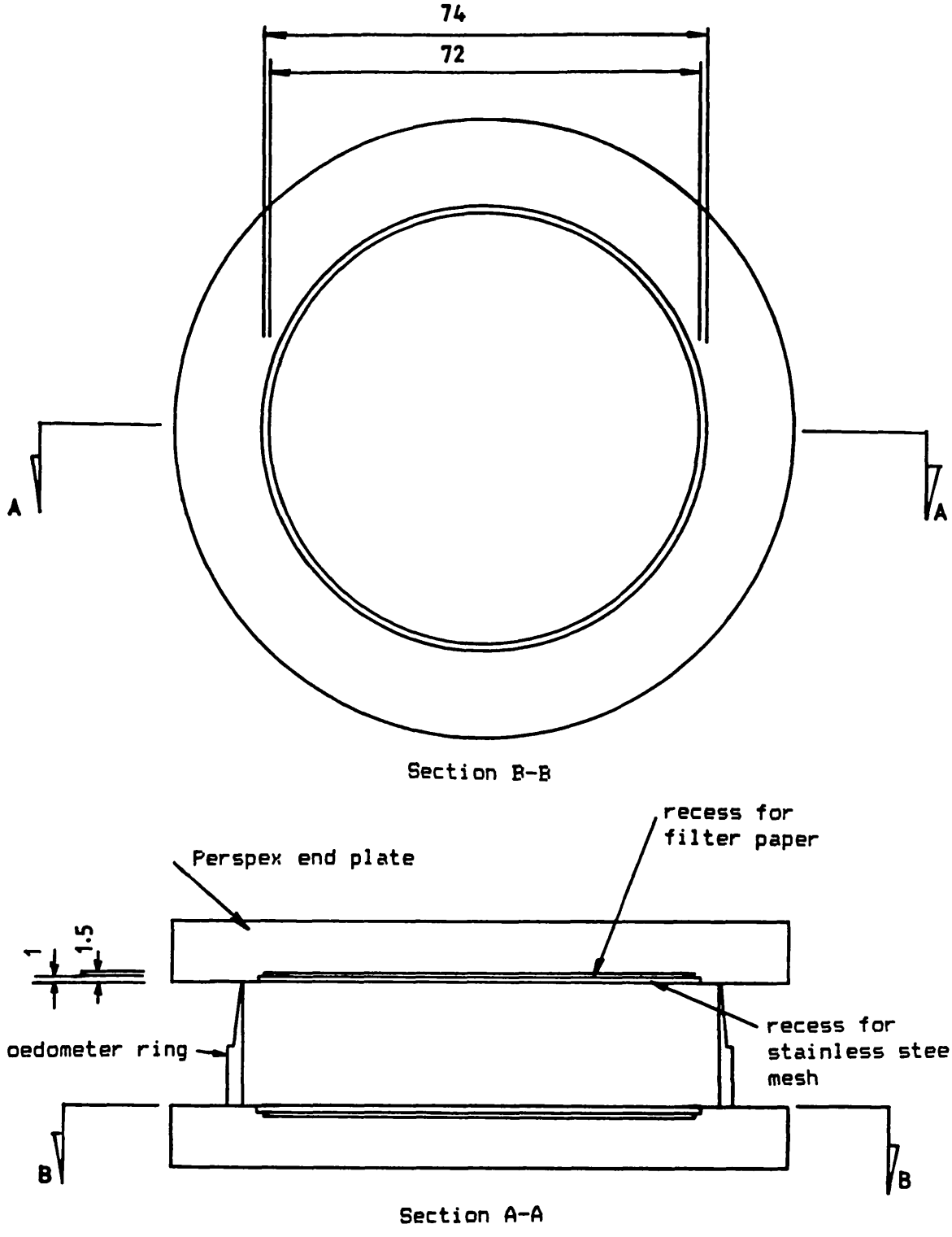


Fig.8.3 Filter Paper Cell

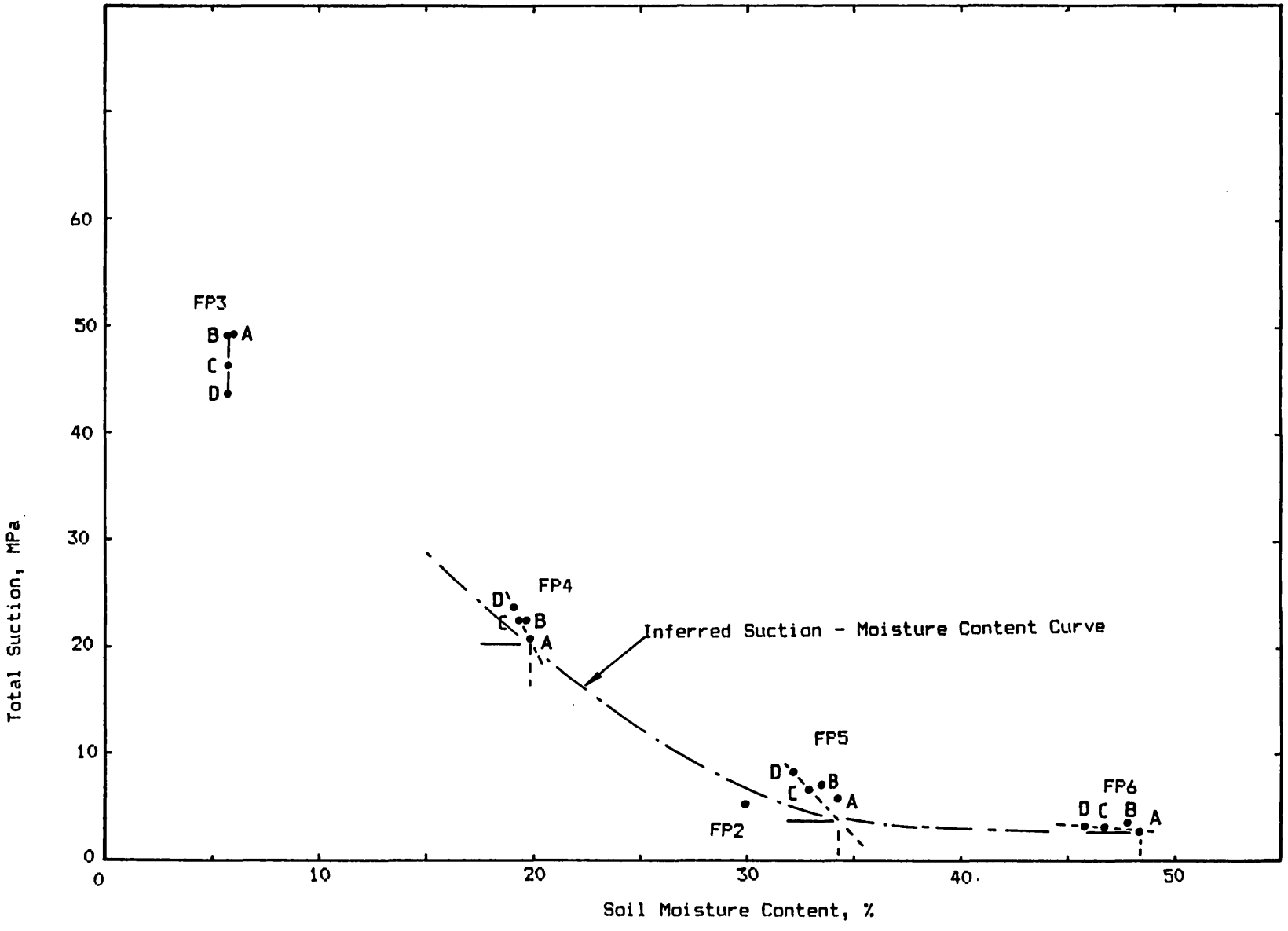


Fig.8.4 Total Suction Measurement with Filter Paper on Loose Samples

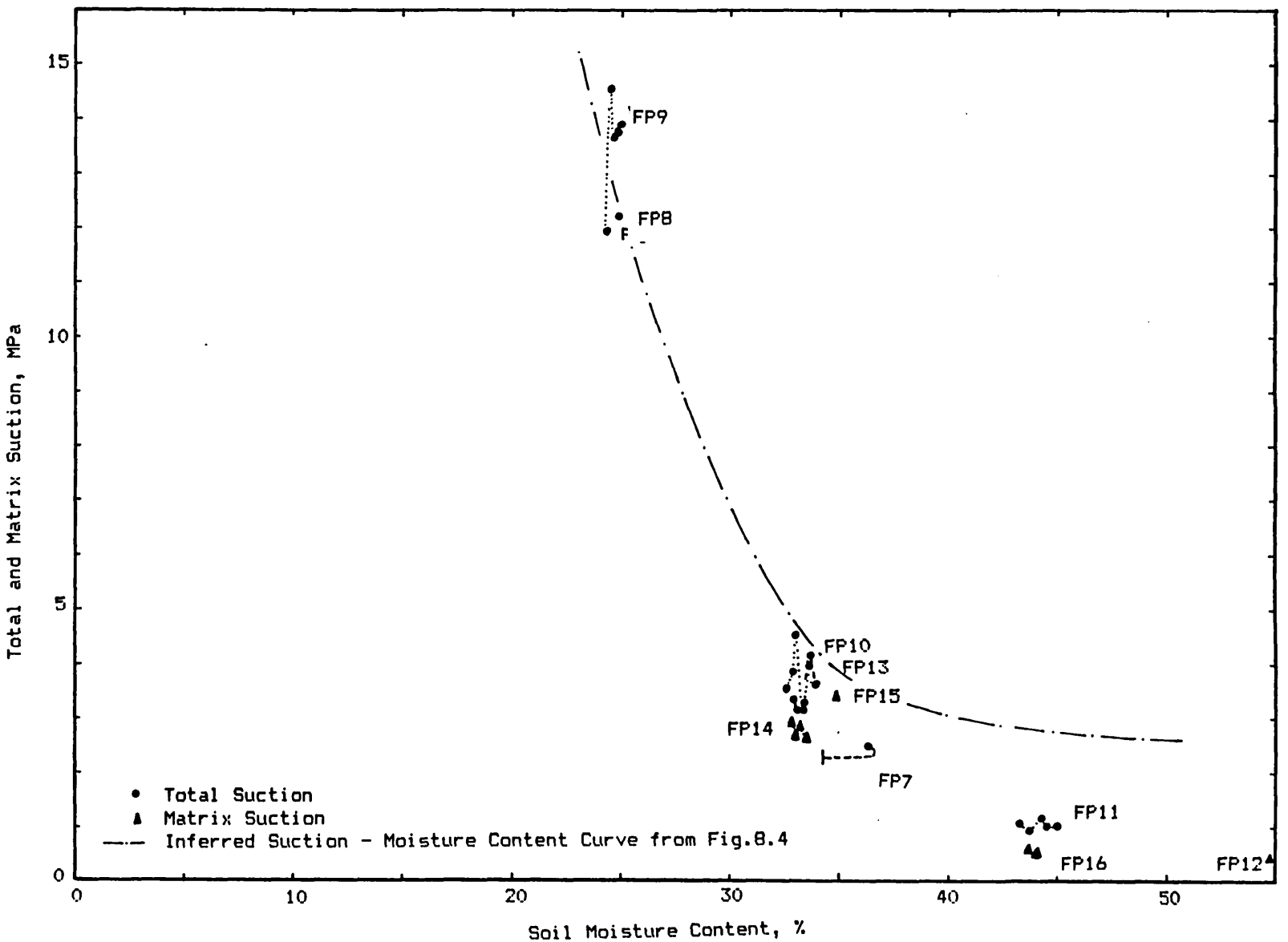


Fig.8.5 Total and Matrix Suction Measurement on Compacted and Consolidated Samples

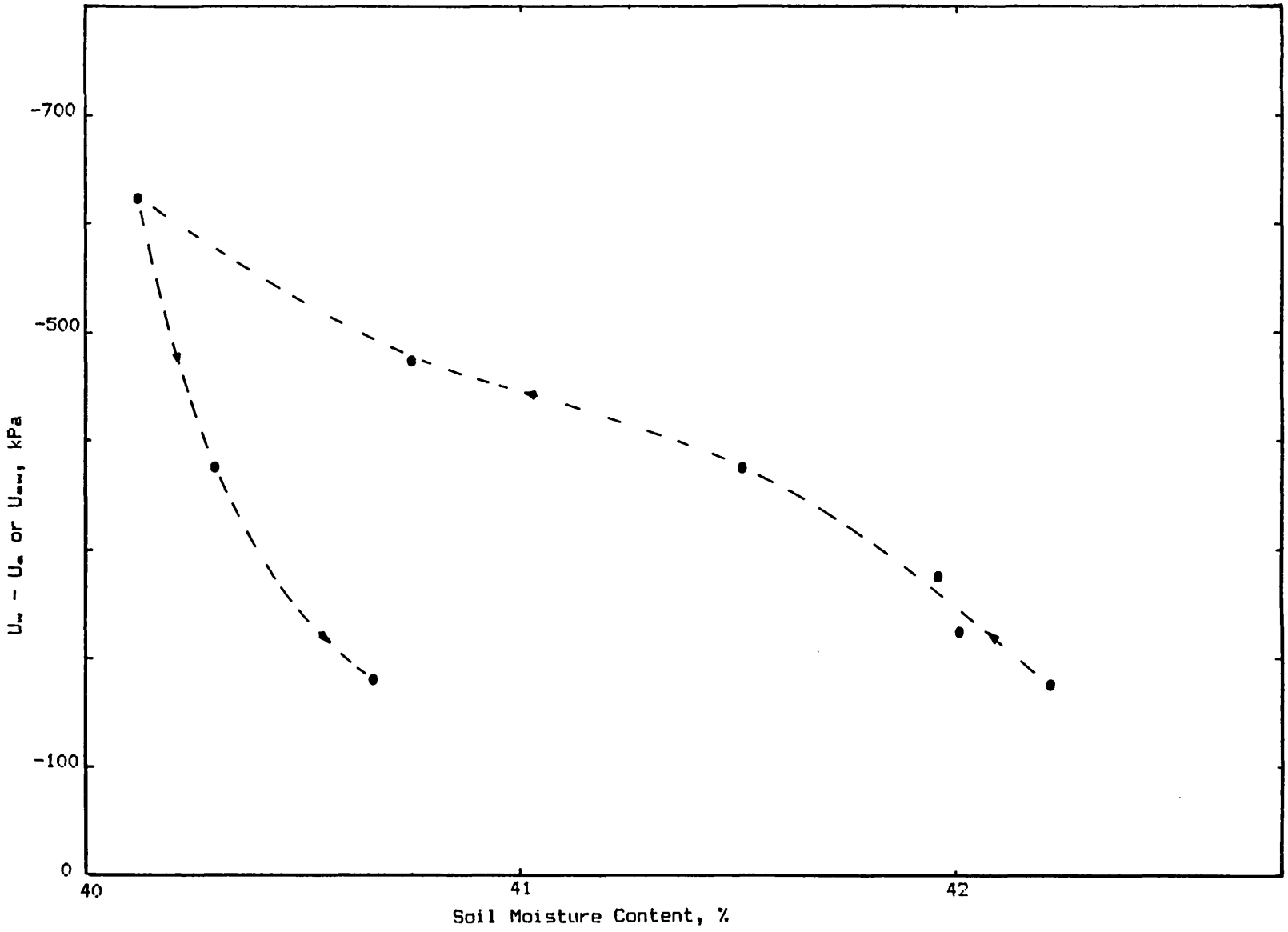


Fig.8.6 Matrix Suction - Moisture content data from Test SP04

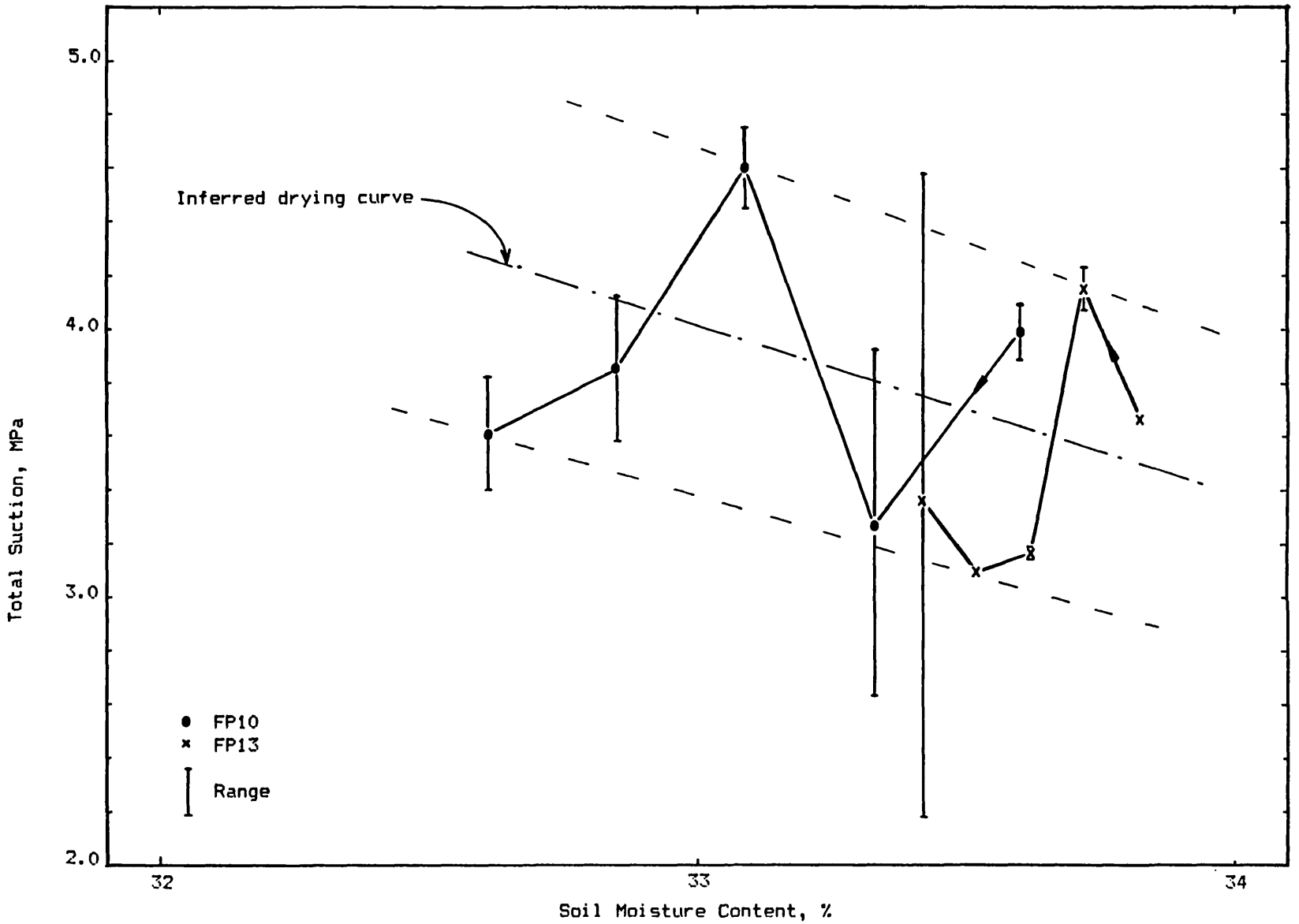


Fig.8.7 Precision of Measurements

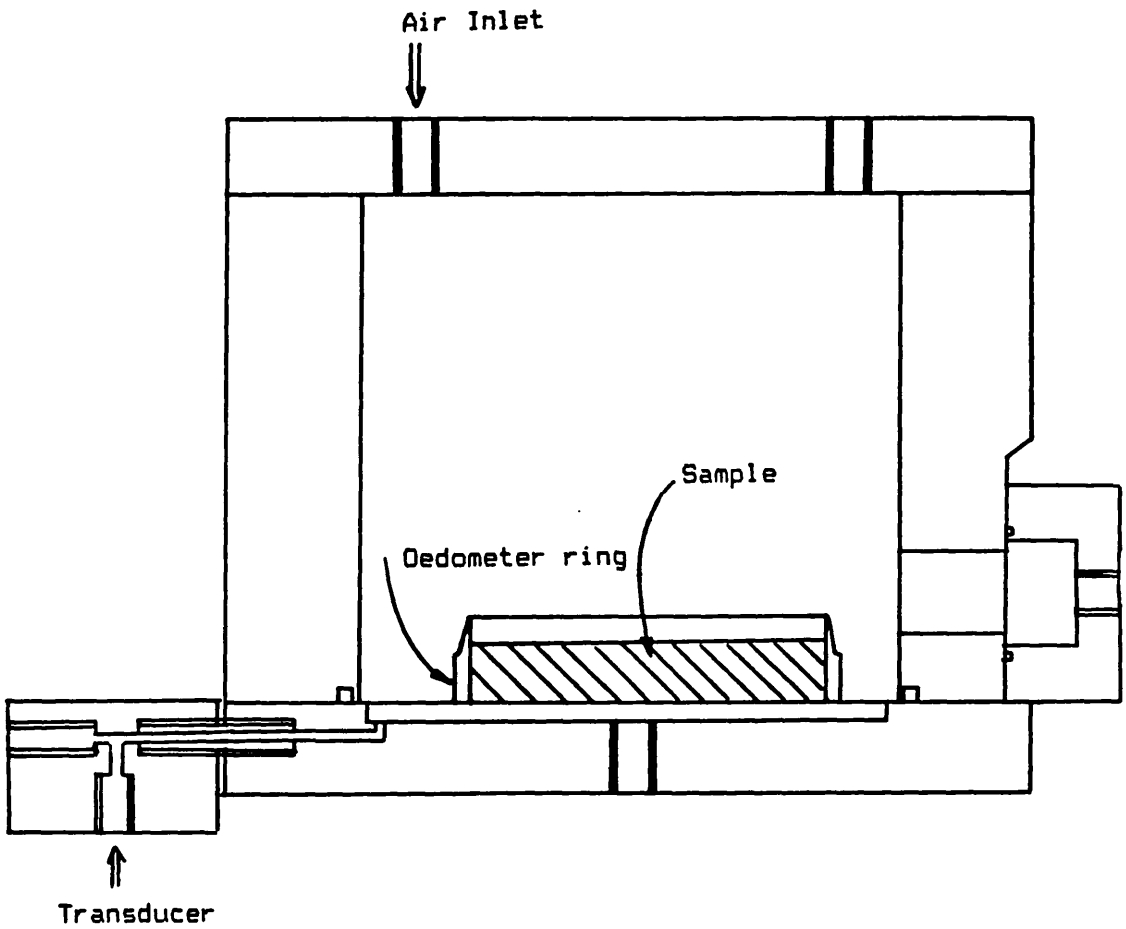


Fig.8.8 Oedometer Cell Assembled for Suction Measurement

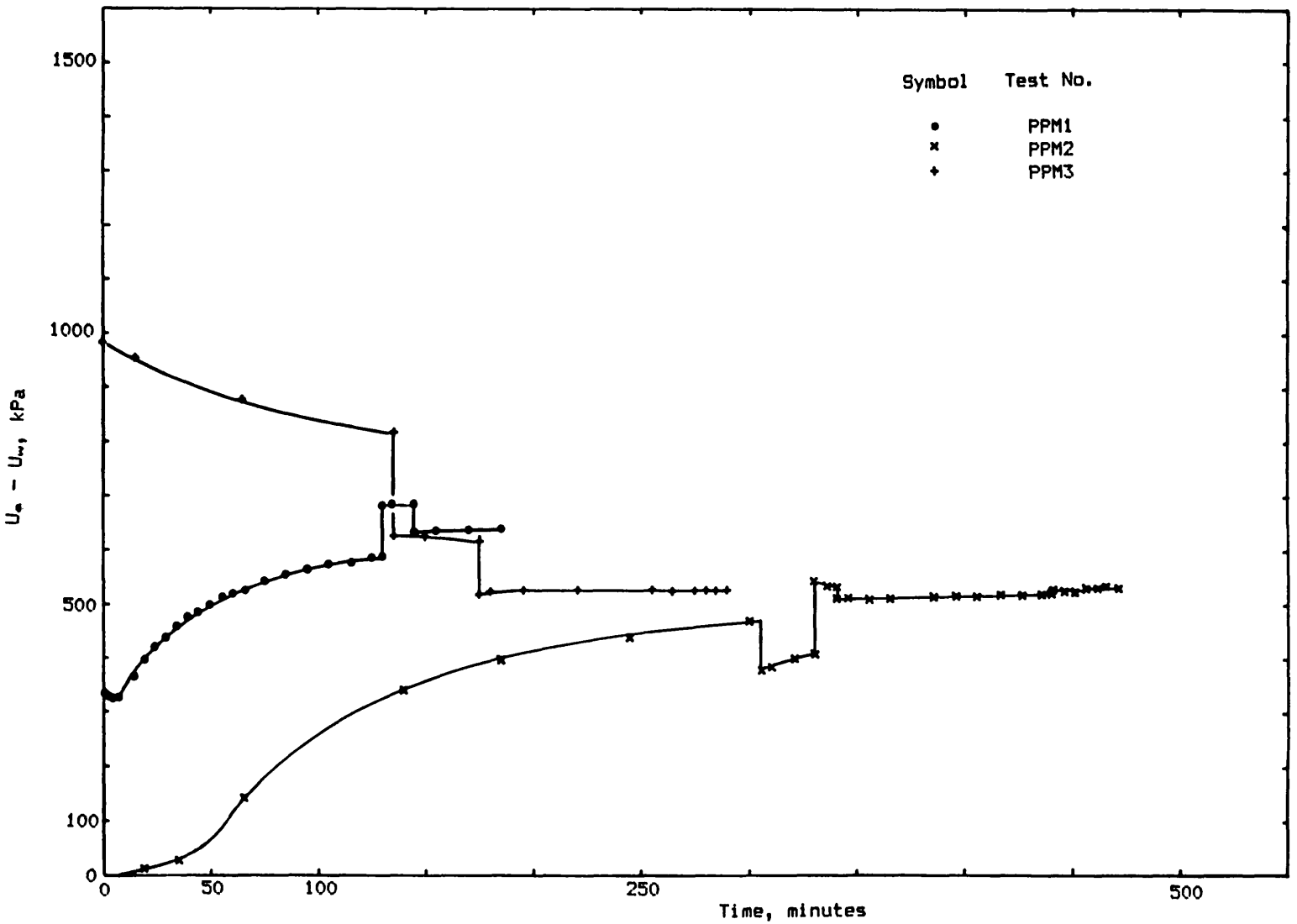


Fig.8.9 Suction Measurement in Oedometer using Elevated Pore Air Pressure
(w_c = 43%)

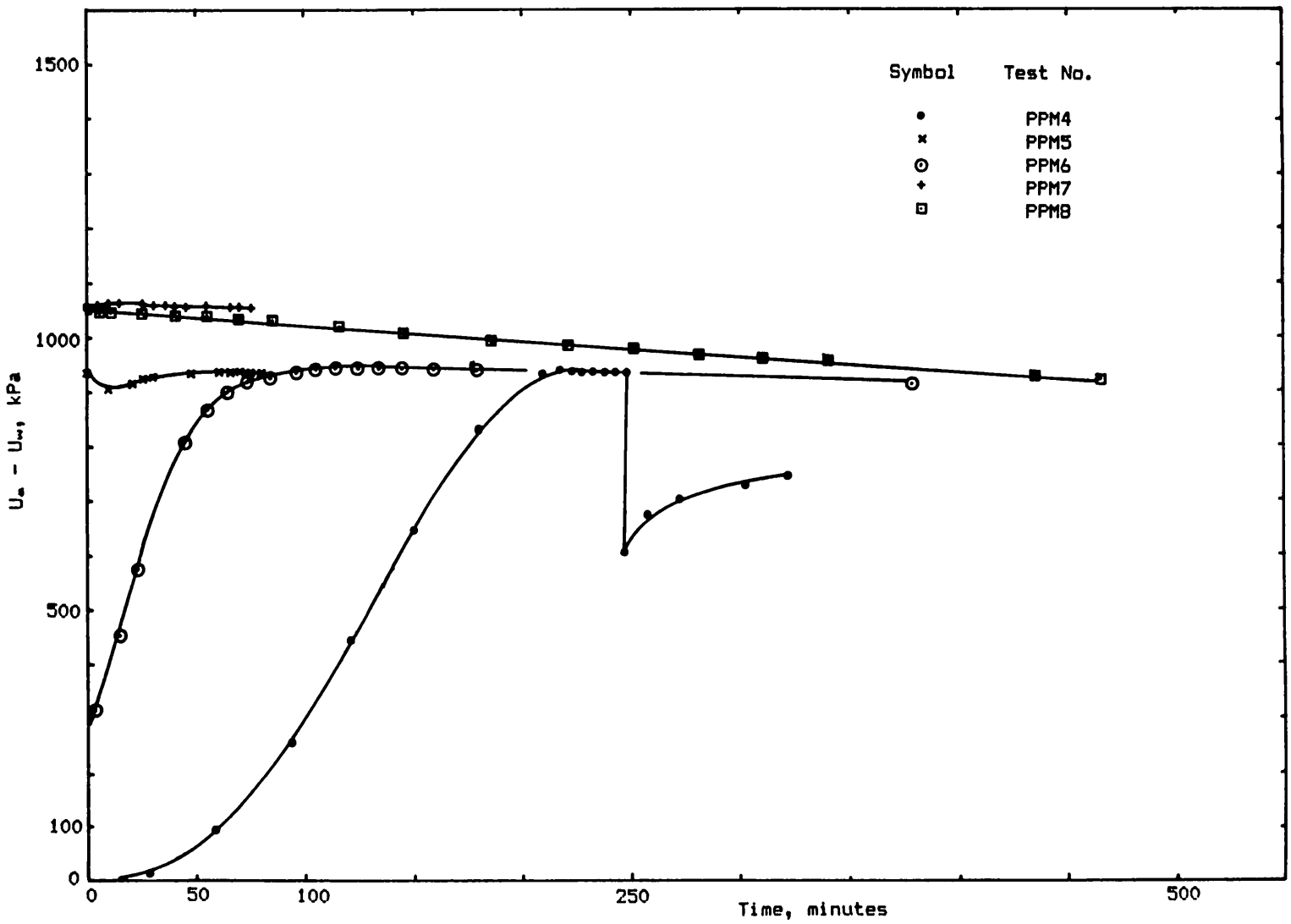


Fig.8.10 Suction Measurement in Oedometer using Elevated Pore Air Pressure
(w_c = 35%)

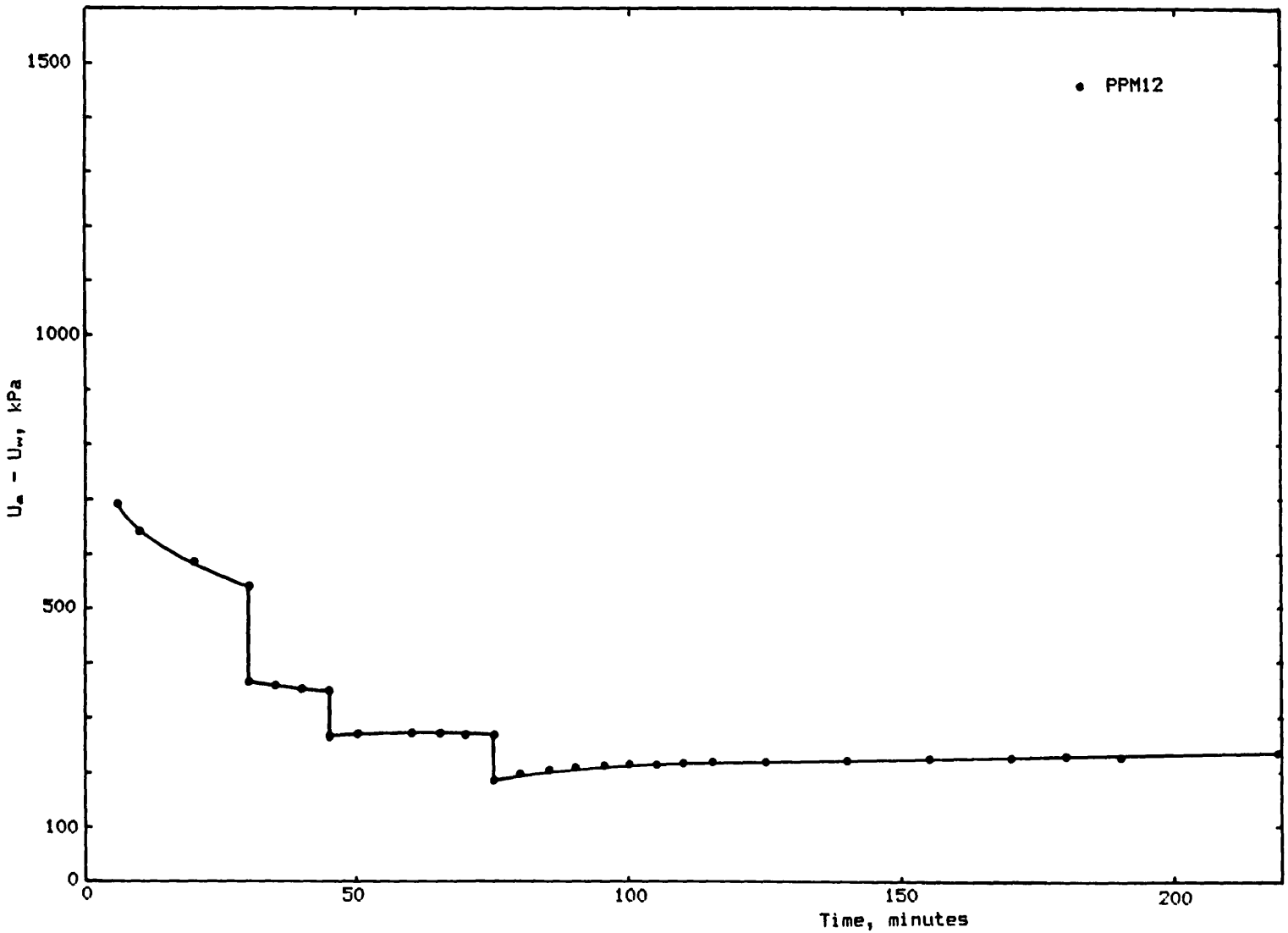
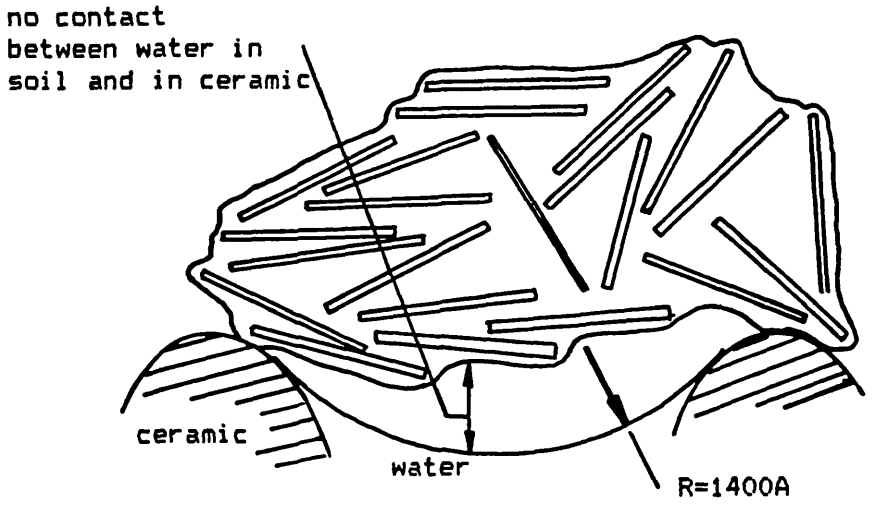
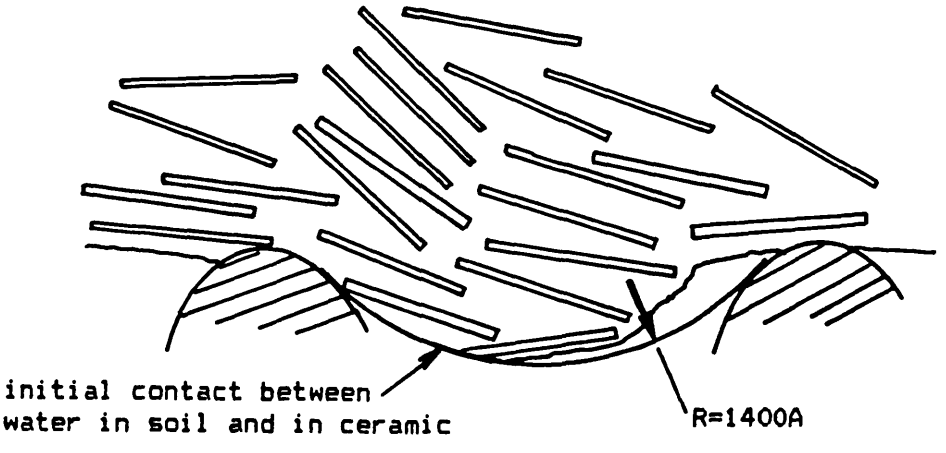


Fig.8.11 Suction Measurement in Oedometer using Elevated Pore Air Pressure
(Wad Medani sample)



(a) Soil cluster at w_c = 35% on porous plate dried under 1000kPa



(b) Soil at w_c = 43% on porous plate dried under 100kPa

Fig.8.12 Efect of soil structure on water contact

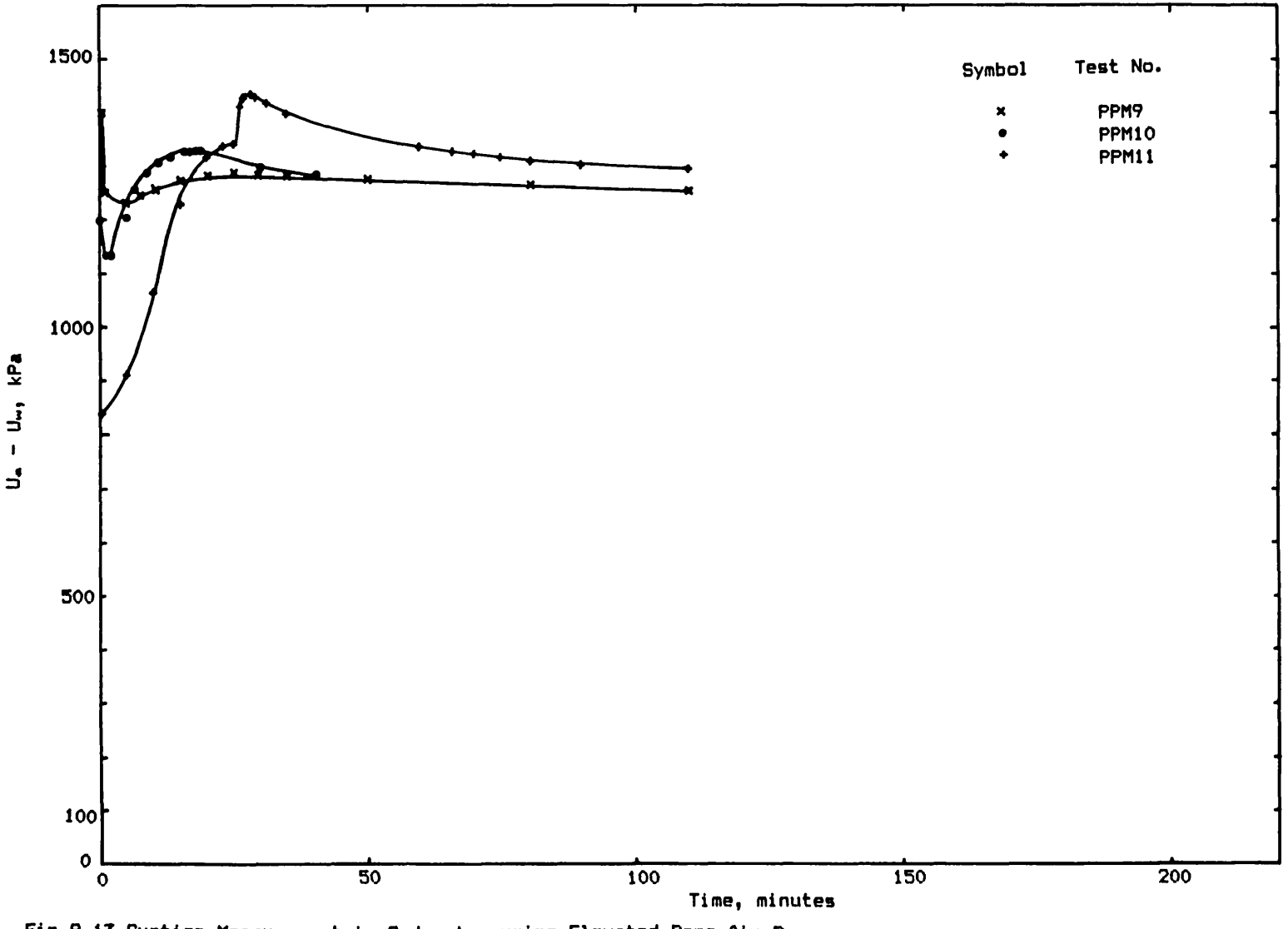


Fig.8.13 Suction Measurement in Oedometer using Elevated Pore Air Pressure (w_c = 25%)

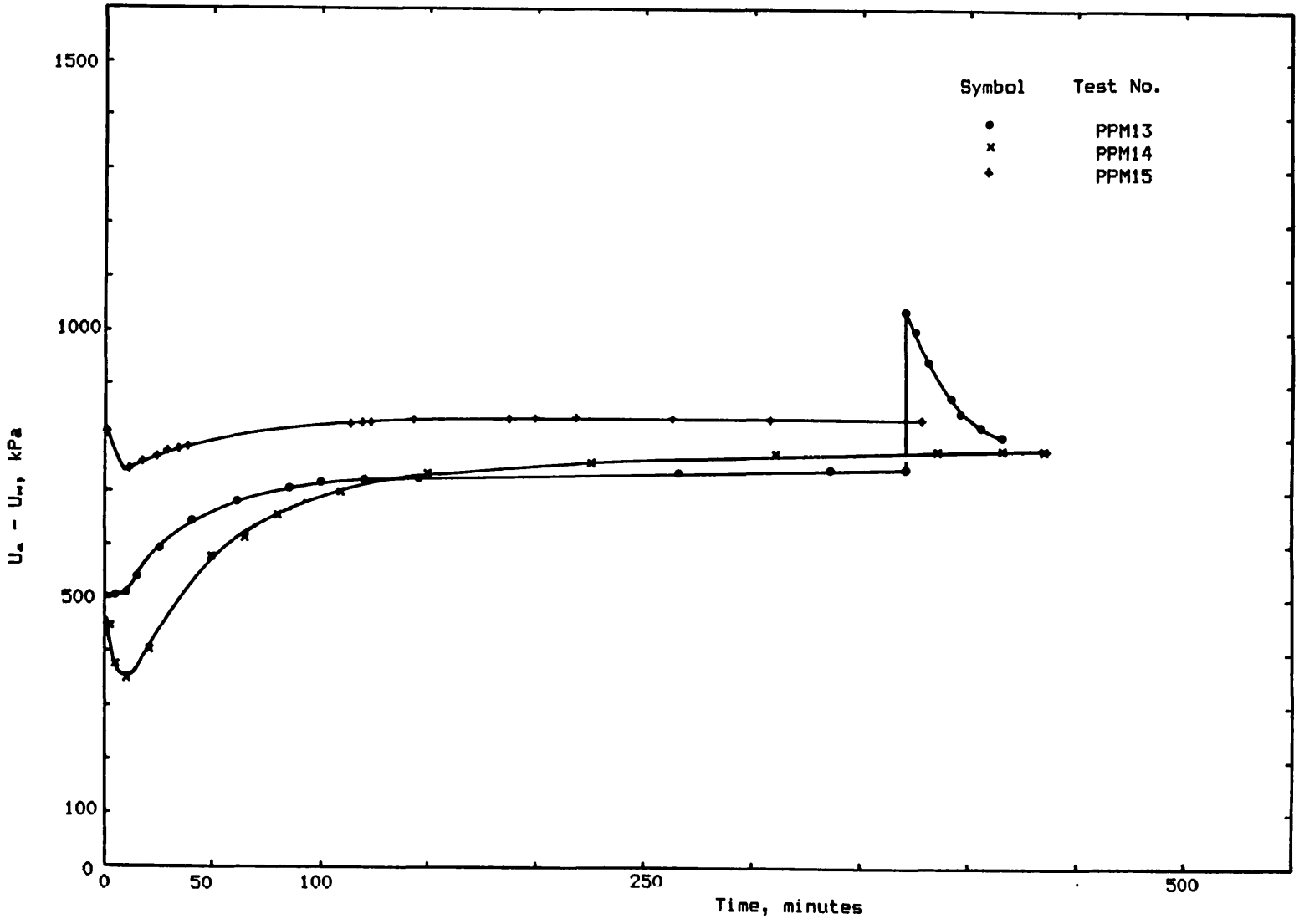


Fig.8.14 Suction Measurement in Oedometer using Elevated Pore Air Pressure (Consolidated Sample)

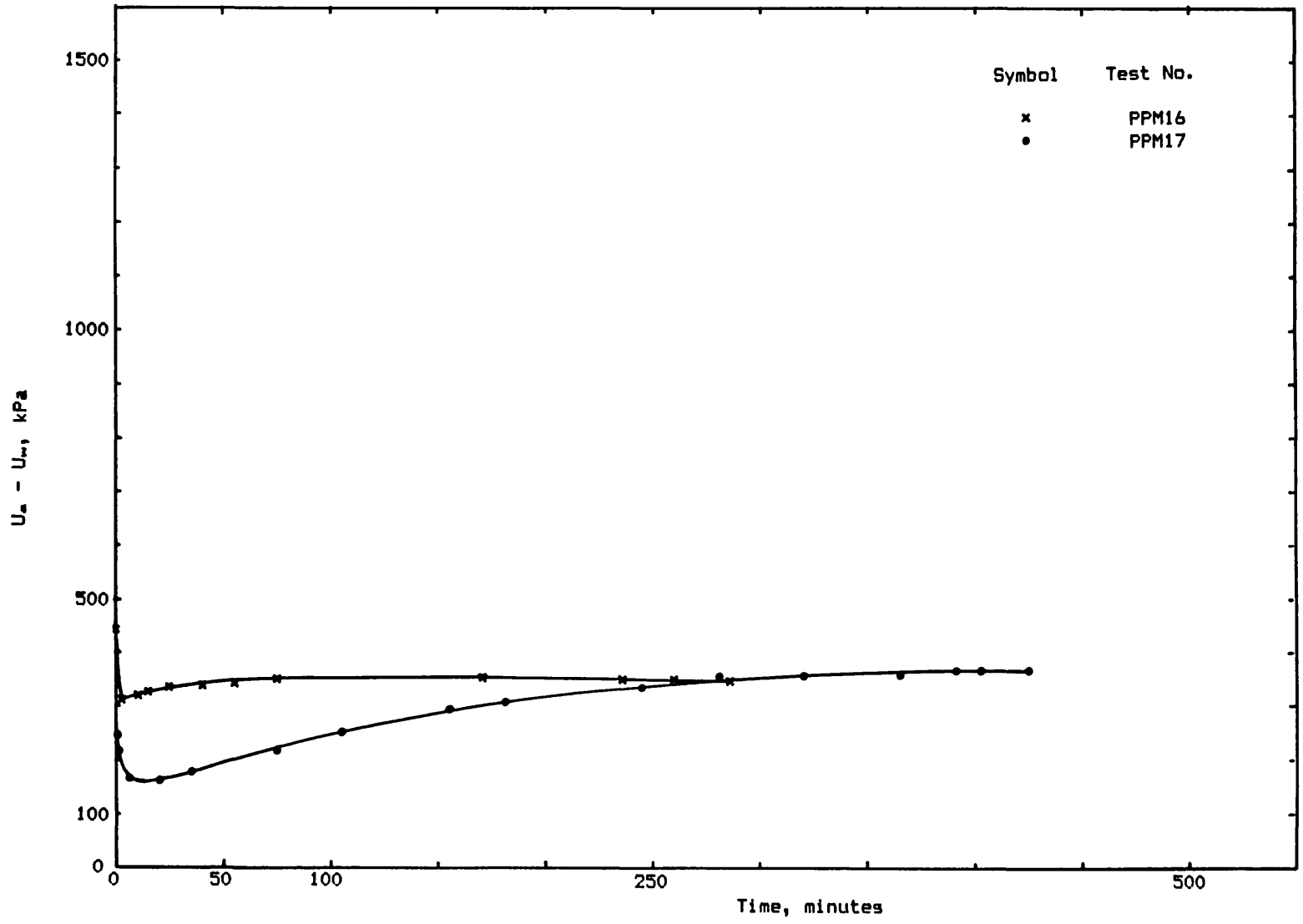


Fig.8.15 Suction Measurement in Oedometer using Elevated Pore Air Pressure (Equilibrated Sample)

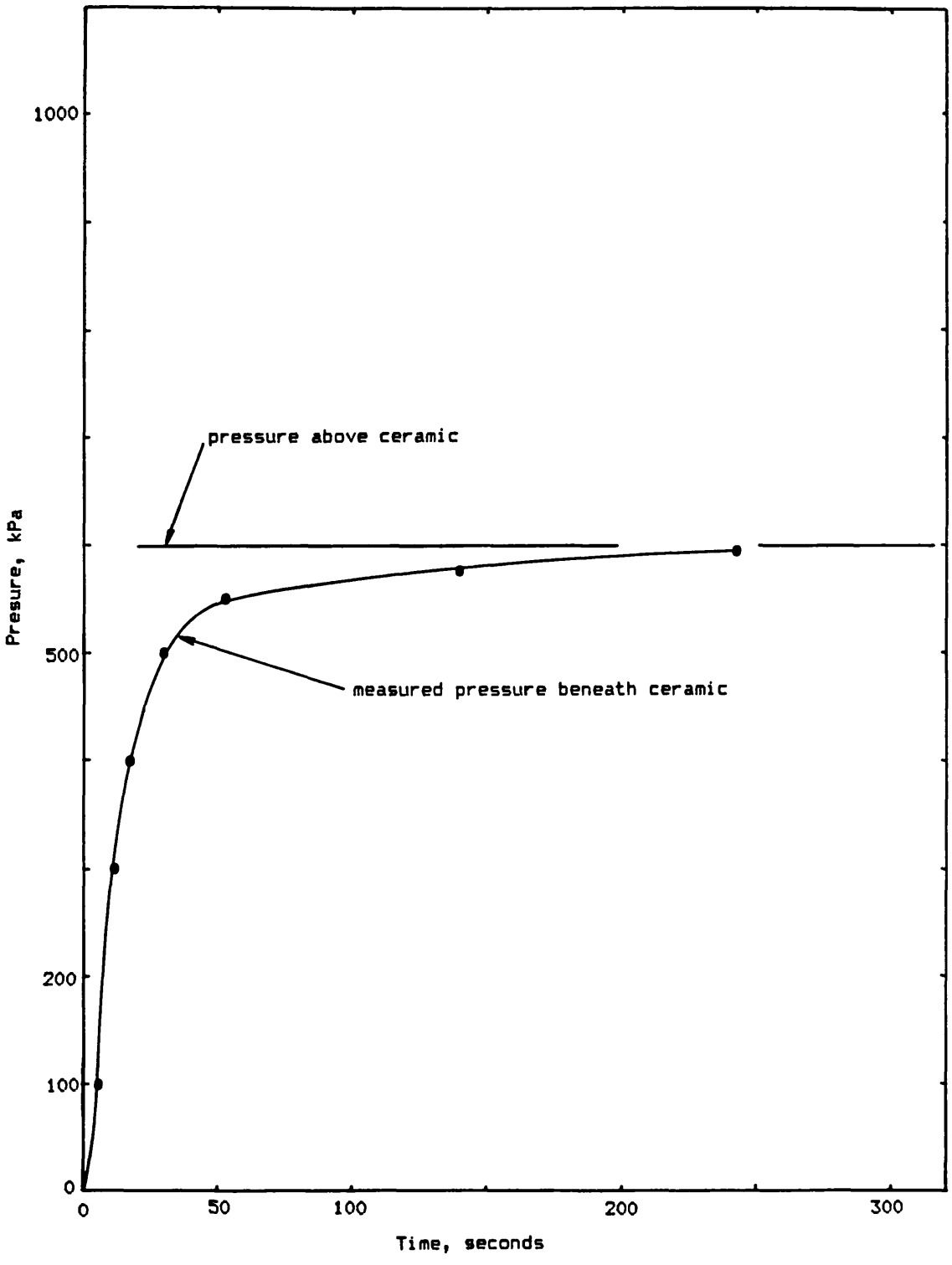


Fig.8.16 Response time of cell

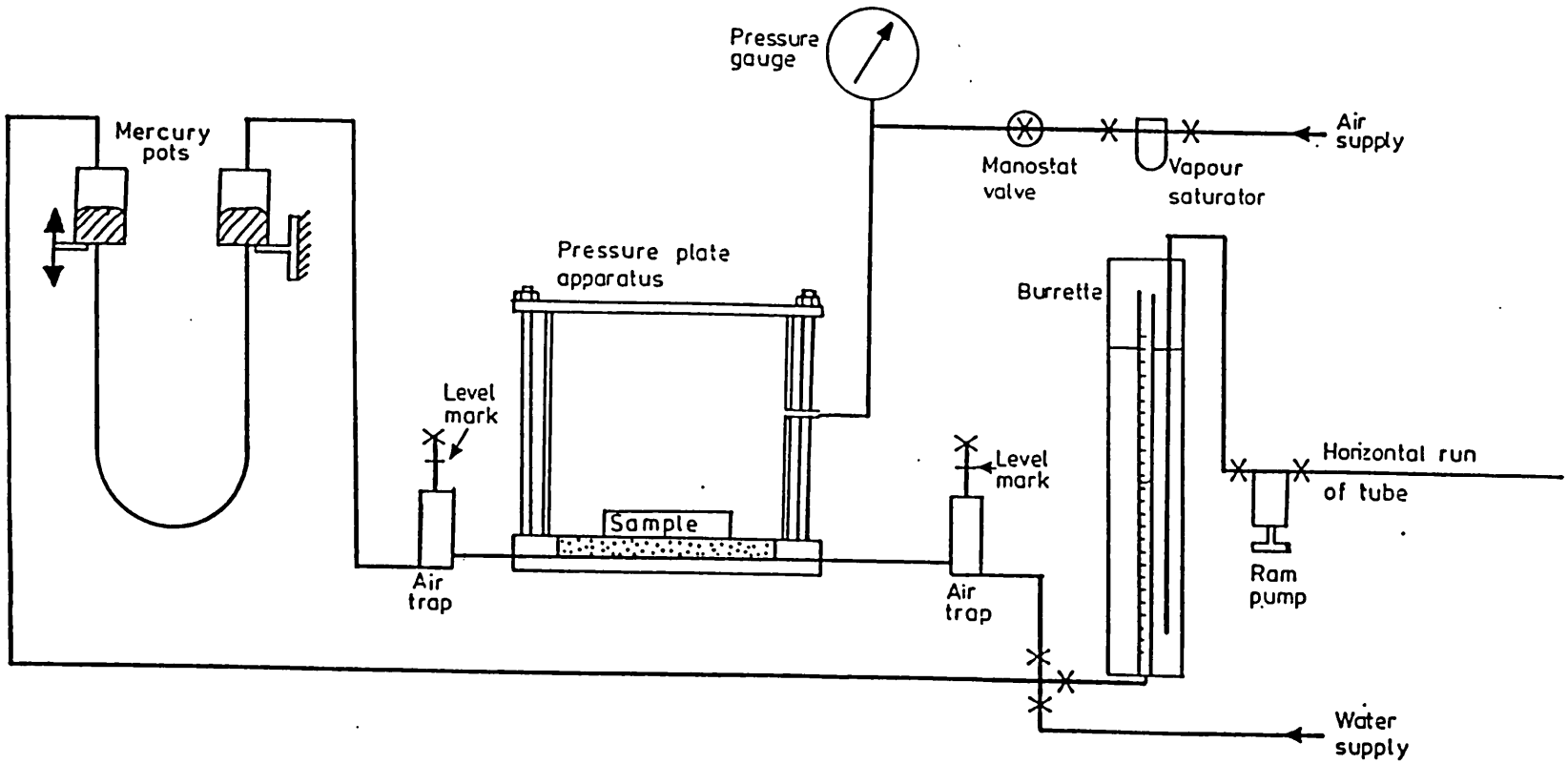


Fig.8.17 Layout of Pressure Plate Control Cell
(Toll 1988)

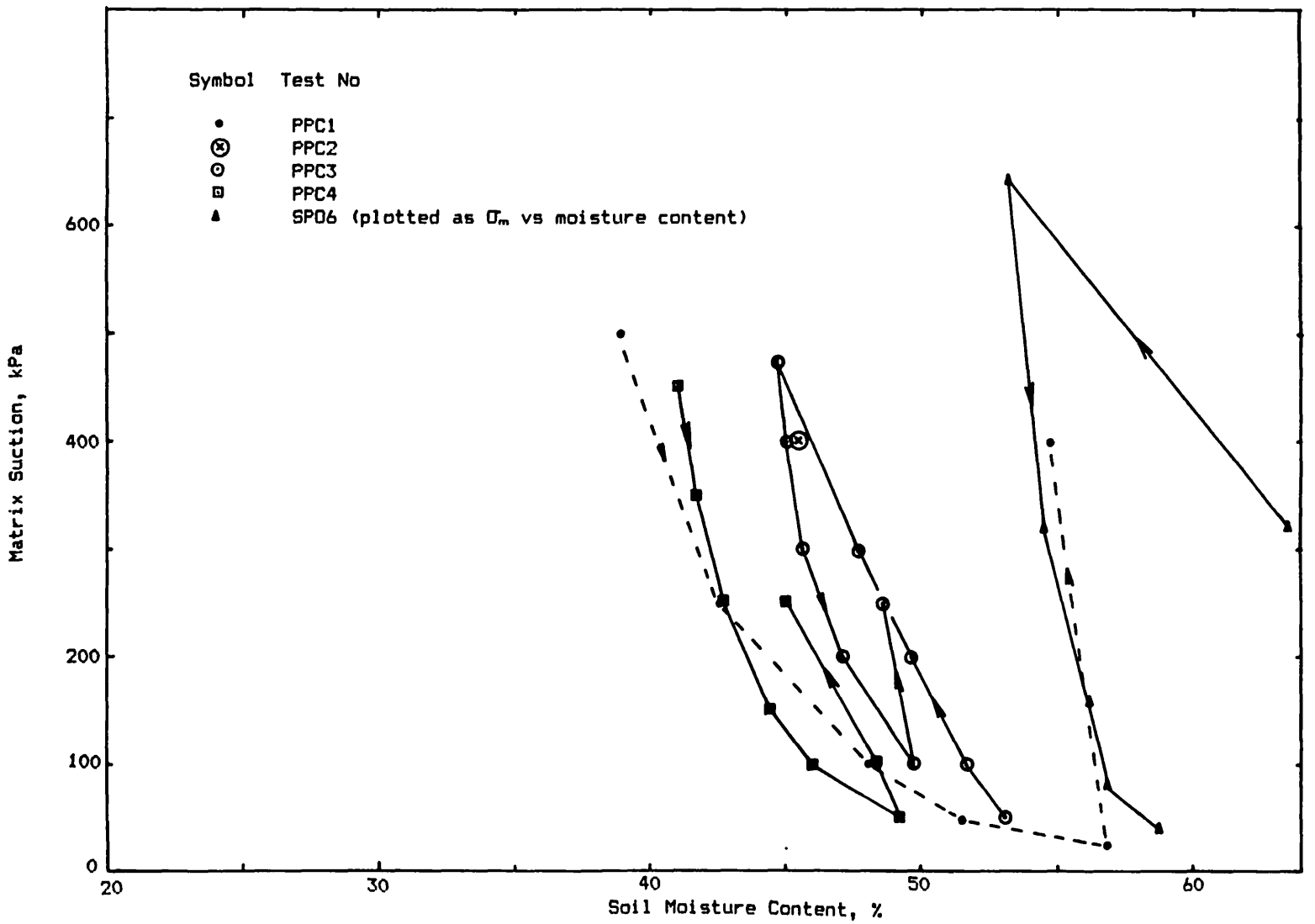


Fig.8.18 Suction - Moisture Content Data from Presure Plate Control Tests

APPENDIX 8A.1

9th Regional Conference for Africa on Soil Mechanics and Foundation Engineering / Lagos / September 1987

Measurement of solute suction in high plasticity clays

H.D.Schreiner
Imperial College, London, UK

ABSTRACT: Reliable measurement of solute suction has in the past proved difficult. This paper describes a salt distribution model of the soil-water system that should be of benefit in the understanding of the role of salts in the soil water. The model uses a two-phase description of the soil water, in which the disolved salts are found in only one of the phases. Some inconsistencies observed by other authors are explained using this model.

1 INTRODUCTION

Measurement of suction is becoming more common in unsaturated soils. Some resistance to its use as a stress parameter may stem from difficulty encountered in the quantitative assessment of solute suction. This has been highlighted by Richards (1980) and further addressed by Richards et al (1984). It has also been noted by Madsen et al (1986), Roundy (1984) & Williams (1968). Since the effects of matrix and solute suction on void ratio and shear strength may differ, it is important to be able to separate these parameters in both research and practical engineering problems.

The purpose of this paper is to investigate the influence of salts in the soil water on the measurement of suction in a desiccated soil. Various existing concepts are brought together and extended in order to develop a model which will assist in the understanding of solute suction and its role in expansive soils. Some of the problems related to solute suction measurement which have been encountered by other authors will be shown to be predictable using this model. Suggestions are made of means of assessing the relevance of solute suction or salt content in engineering testing and of obtaining adequate quantification of solute suction for practical or research purposes.

The first part of the paper will discuss the soil and soil water chemistry. The impact of these observations on the measurement of solute suction will be considered in the second half.

The model of suction in a soil that is preferred by the author is the physico-chemical approach used by Bolt (1956) and extended by Sridharan & Jayadeva (1982). In essence the suction in a soil is the osmotic pressure difference between the double layer water and pure water. This is a relatively simple concept in a salt-free soil, but will be discussed in more depth for salty soils. The alternative approach to suction, namely the electrical repulsion or particle energy model will not be considered, but has been shown to provide similar conclusions on the influence of salts on suction as the Bolt type approach (Yong et al 1984).

2 DEFINITION OF TERMS

The definitions of the components of suction were introduced into soil mechanics from soil science by a Review Panel (1965) For completeness they are included in Appendix A.

3 SUCTION MECHANISMS

At the time that suction began to be included in soil mechanics testing, matrix suction was generally thought of as a capillary suction. This may be because a considerable amount of the early soil mechanics testing (e.g. Blight 1965, Bishop & Donald 1961) was performed on silty soils in which the capillary concept is valid. However, in clays, particularly those of high plasticity such as the expansive soils, the matrix suction is now generally accepted as being caused by the attraction of the clay for water in the double-layer. With this approach, a single particle of clay can exhibit a suction, whereas in the capillary model suction is due to the pore size or spacing between particles. Water absorption is discussed by Lambe and Whitman (1969) and Mitchell (1976) for saturated soils. In unsaturated soil an additional factor is involved.

Where stresses in a the saturated soil can be considered as an equilibrium between the externally applied stress and the repulsion or interaction between the absorbed water layers, the unsaturated soil has in addition a partially satisfied demand for water and thus incomplete absorbed water layers under the external stress state. The difference in energy level or potential between free water and the incomplete absorbed water layers is the cause of the engineering parameter which we call, and measure as, matrix suction. In addition to the absorption energy difference there can also be an energy difference due to the presence of salts in the soil water, which we call solute suction, but have difficulty in measuring.

Two techniques of suction measurement are directly relevant to this paper, and will be very briefly outlined here. Both are in routine laboratory use, and details are available in the literature. Both techniques were originally developed in the soil science field and have been used in soil mechanics for many years.

The pressure plate system controls the suction across a saturated porous plate which has a small pore diameter. Air pressure within the test chamber is resisted by the surface tension of the water in the pores of the porous plate. Soil placed on the plate will either take in or lose water until the suction in the soil is equal to the difference between the air pressure in the cell and the water pressure in the porous plate. The pressure plate system can also be used to measure, as opposed to control, the suction by measuring the water pressure in the porous plate with a pressure transducer.

The psychrometer measures the suction via either the relative humidity or the dewpoint of the air in equilibrium with the soil. The humidity and the dewpoint can both be related to the vapour pressure of the water in the soil and to the suction. In practice the instrument is usually callibrated against salt or acid solutions of known suction.

4 CATIONS IN THE SOIL WATER

It is perhaps appropriate to clarify here the differences between exchangable cations and disolved salts. In the presence of water, each clay particle is dissociated into many cations and the remnant clay particle which is in effect a large anion. The negative charge on the anion balances the sum of the positive charges on the cations (Resendiz 1965). Exchangable cations (EC) are as the name implies, cations related to the clay that may be readily changed. This is in contrast with the other cations related to the clay, i.e. the cations in the solid clay crystal structure which cannot be changed except by changing crystal structure or chemistry. EC cannot be removed from a clay (except by specialised scientific separation techniques) as they are required for chemical stability (Bolt 1956). A direct analogy would be the removal of sodium from a solution of table salt. It can be done, but the resulting product is no longer salt, but is chlorine disolved in water. However, by supplying alternative sources of cations we can readily change the EC related to a clay. This is most commonly done by adding the desired cation in the form of a salt at a high concentration and then washing or leaching the soil to remove the resultant mixed salt from

the soil. The added salt used in this technique is a disolved salt. This can be considered as a compound that is not a part of the clay and can be added or removed from the soil system. However, the alteration of the disolved salt, either its concentration or composition, may well affect the EC of the clay, as noted above.

There are thus two interdependent groups of cations in a soil :- those that remain with the clay, and those that can be removed from the soil system. In practice there is no distinction between individual cations in a soil, except their position in the water, and this is governed by equilibrium between the absorbed and free phases. They are free to move within the soil water, provided that certain constraints are satisfied, for example the concentration and distribution of cations and anions within the absorbed water and within any water outside the absorbed layers must be compatible. This concept is used by Bolt (1956) regarding cations in describing the swelling pressure of a saturated soil as the difference between the osmotic pressure of the absorbed water at the mid-plane between two particles and the free water.

If we now transfer this concept to unsaturated soils, we have the soil water divided into two "phases", or "regions". In the absorbed phase we have a portion of the water, bound to the clay particle, containing the EC and in the free phase, we have the other portion of the water, the "free water", containing the disolved salts. The proportions of water in the absorbed phase and the free water phase will depend upon the relative proportions of cations and disolved salts, such that there is equilibrium across the absorbed water : free water interface. This is shown schematically for a single particle in Fig. 1. Note that the clay particle together with the EC provide the electrical neutrality, as do the cations and anions of the disolved salts.

The absorbed : free water system is described by Bolt (1956) as forming an osmotic system, although he refers to the clay particles as forming the semi-permeable boundary. If the semi-permeable boundary is more accurately defined at the

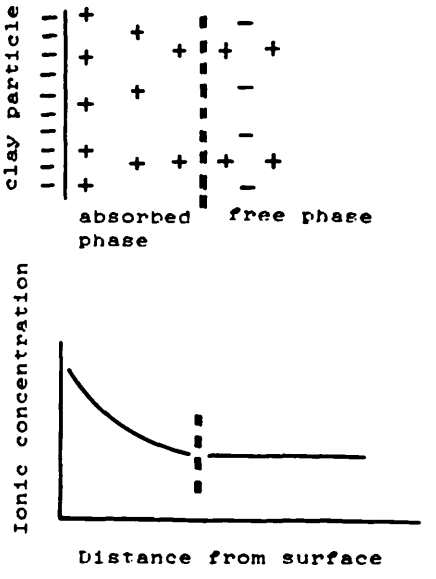


Fig.1 Distribution of ions

absorbed : free water interface, then we can arrive at some significant conceptual inderstandings of suctions in the soil water systems.

Firstly let us consider the total suction as measured by the vapour pressure of the water in the soil or by the humidity in the air in equilibrium with the soil. This is the principle involved in the psychrometer, desiccator or calibrated filter paper measurement techniques. The vapour pressure is controlled by the water surface that is exposed to the air, i.e. the surface of the free water in the concept used above. Two factors will contribute to the vapour pressure that we wish to measure. The radius of the menisci will reduce the vapour pressure from that of unstressed water if the menisci are concave. This is the usual assumption in soils, although various arguments could be raised regarding the real shape of the air-water interface, particularly for clusters or aggregations of clay particles. It has been suggested that this capillary component is of little importance in highly plastic clays (Olsen & Mesri 1980). The other factor controlling the vapour pressure is the composition of the

free water solution. In terms of the Bolt type concepts used, this is the major component. The conclusion from these arguments is that the concentration of salts in the free water phase is directly related to the total suction. Some non-equivalence may occur due to surface tension or meniscus contributions, but these are expected to be relatively small.

Secondly let us attempt to describe the matrix and solute components as defined by the International Society of Soil Science (ISSS). In a clay and water system without disolved salts, the matrix suction concept is quite easily seen as the suction due to the partially satisfied absorbed layer and may be described using the Bolt type arguments as the osmotic pressure difference between the concentration of cations at the surface of the absorbed water and unstressed pure water.

If we now add salt to this system, at constant water content and with the cations in the salt in identical proportion to each other as they are in the absorbed water, then the effect will be to reduce the amount of water available to the EC. The concentration of disolved salts in the free phase will be equal to the concentration of the absorbed water at the phase interface, and also equivalent to the total suction. The concentration at all points in the soil water has thus been increased, and this increase in concentration causes the increase in total suction which is attributable to solute suction. However, the concept of "soil water" as used by the ISSS in the definition of solute suction becomes rather more difficult to use. The solute suction may perhaps be better described as the osmotic suction due to the total amount of disolved salts in the total amount of water in the soil measured, relative to pure unstressed water. The matrix suction may then be described as the suction due to the clay, including its cations, but without disolved salts, interacting with the total amount of water in the soil.

One concept that the ISSS definitions embody, but that is frequently ignored by engineers, is that the solute suction is not only related to the concentration in

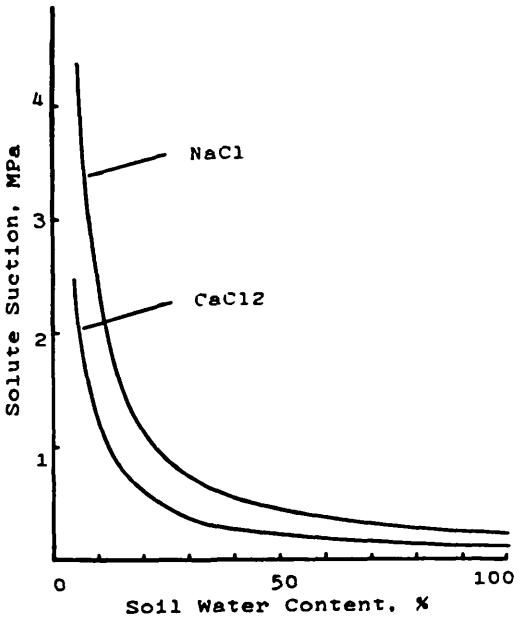


Fig.2 Effect of salt type

grams per cubic centimetre of water of the disolved salt. It is also related to the molecular weight of the salt, and hence the need to use an "identical composition" in any water supply is an important consideration in any suction measurement or control system. Fig. 2 shows theoretical suction curves for sodium (NaCl) and Calcium chloride (CaCl2) in an expansive soil from Kenya. The molecular weights are 58 and 111 respectively, and the variation in suction depending on the assumed salt may be seen to be large, especially at low water contents. This may be a significant source of error when standard curves relating conductivity to suction are used if there is a difference between the types of salts present in the soil and those for which the standard curves were derived.

This is a simplified description of a complex chemical equilibrium. The implications of the salt distribution in the soil water are significant with regard to suction measurement. It is unlikely that the real physical and chemical equilibrium will invalidate the following discussions and conclusions which are explained

using the simple scheme described above.

5 TECHNIQUES FOR SOLUTE SUCTION MEASUREMENT

Various methods exist for measuring solute suction. In essence all methods extract water from the soil, either by dilution or by high pressure compression. Suction is then determined either by psychrometer (on the extract) or via the electrical conductivity of the water.

A very interesting set of data was obtained by Richards (1980), who measured solute suction both by conductivity measurement on an extract from a saturated soil paste and by psychrometer on a pressure extract. Neither measure of solute suction was found to agree with the difference between the pressure membrane suction (matrix) and the psychrometer suction (total) on the soil. The saturation extract gave apparently low values, while the pressure extract gave apparently high values. Conflicting data was obtained by Krahn and Fredlund (1972). However, if we apply the arguments given above, a reasonable explanation for the inconsistencies observed by Richards (1980) and Krahn and Fredlund (1972) may be found.

5.1 Pressure extraction

The technique involves the squeezing out of water under uniaxial compression in a miniature cell. It is applied to soils at the soil water content at which the solute suction is required. Expulsion of water from the soil mass will only occur once saturation is approached, and thus relatively large stresses may be required. Morgenstern and Balasubramanian (1980) for example used stresses between about 700 kPa and 20 MPa.

During constant water content loading, i.e. before a high enough degree of saturation is reached for expulsion of water, the void ratio will be decreased in two ways. Firstly particles, or groups of particles, may move relative to each other into previously "vacant" space, i.e. air voids. Secondly the double layers will be stressed, and the interparticle spacing will reduce until the central plane concentration is in equilibrium with the suction and the total (external) stress. Water will be expelled from the absorbed layer (the exchangeable cations cannot be expelled with the water without affecting the electrical neutrality) into the free water solution until equilibrium is also obtained across the absorbed: free water interface. The free water phase will be diluted, (i.e. the salt concentration will be reduced) and thus the total suction will be decreased. Depending on the change in interparticle spacing required under the applied total stress used for water expulsion, so the salt concentration in the expelled water will vary. The concentration will tend to decrease as the applied stress increases, as found by Morgenstern and Balasubramonian (1980). Corresponding decreases in suction have been reported by Komornik et al (1980) during isotropic compression of unsaturated samples. The changes in suction were relatively small, but expulsion of water was not achieved.

In a soil with a low salt content, the amount of water in the free water phase will be small in order to obtain the high salt concentration corresponding to the total suction. Water expelled from the absorbed phase during pressure extraction will cause relatively large changes in concentration of the extract. In a soil with a high salt content, there will be proportionally more water in the free phase, and thus the dilution during pressure extraction will be less significant. This may partially explain the different conclusions reached by Krahn and Fredlund (1972), who found the pressure extraction technique to be satisfactory, and Richards (1980), who found that the pressure extraction method gave solute suctions larger than the difference between the measured total and matrix suctions, and also very much larger than the solute suctions obtained from a saturation extract method.

5.2 Saturation extract

Another method for obtaining an extract of the soil water is by diluting the soil until a paste is achieved and then removing a water sample. The procedure for obtaining a "saturation" extract is rather imprecise (Hesse 1971), but involves wetting of a remoulded sample to a state that is of the same order as the liquid limit, but is probably wetter. It could better be described as a paste extract, as saturation in soil mechanics does not require a high water content. At this water content the absorbed layer is likely to be fully formed, and will certainly be so for the 5:1 water:soil solution extract used by Richards (1980). The assumed equilibrium therefore no longer exists at the absorbed: free water boundary. The solute suction of the extract will correspond to the concentration in the free water phase. Estimation of the solute suction at a low water content by assuming that the salt concentration is linearly related to the total soil water content as used by Krahn and Fredlund (1972) will lead to an overestimate of the solute suction, since the measured concentration is based on only the portion of water in the free phase while it is assumed in the calculation of the suction at the water content of interest that the salt concentration is due to the total quantity of water in the soil. This predicted overestimate is in agreement with their observations that the solute suction by this procedure was greater than the difference between the measured total and matrix suctions.

5.3 Solute suction measurement

From the foregoing it is evident that extraction techniques are not adequate for reliable estimation of solute suction. Perhaps the only method that is available at the present stage for the direct estimation of solute suction is the determination of the total soluble salt content of the soil, in terms of grams of salt per gram of dry soil, together with the identification and proportions of the salts, followed by calculation of the solute suction. The soluble salt does not include the EC as these connot be removed from the soil. Salt contents measured by the Author are listed in Table 1 for four Kenyan soils. Salt types have not yet been identified, but solute suctions have been calculated for the soils assuming the salt to be Calcium Chloride (CaCl2). Calculations use the following equations:-

$$\frac{p0 - p}{p0} = \frac{n}{N + n}$$ Partington (1951)

and

$$h = \frac{RT}{g \times M} \ln P/P0$$ Richards (1965)

Where h = suction
P/P0 = relative humidity
M = molecular weight of water
g = acceleration due to gravity
T = temperature in degrees Kelvin
R = Universal Gas Constant
n = mols of solute (gm molecular weights)
N = mols of solvent (water)

Table 1 Salt concentrations

Soil Sample	PI	LL	Salt Conc.% (gm/gm)
Lodwar	55	94	1.14
Nairobi	74	125	0.52
Nyanza	43	70	0.09
Muhoroni	78	120	0.12

The solute suctions are presented in Fig.3 in terms of the water content of the soil. The magnitude of the solute suction can be seen to vary considerably between soils, and could have a significant impact on attempts to correlate swell, suction and soil type, if it is not adequately quantified.

Leaching of salts from these soils was performed under isotropic total stress conditions in the apparatus shown in Fig.4. Little benefit was gained from using high water pressures as this increased the

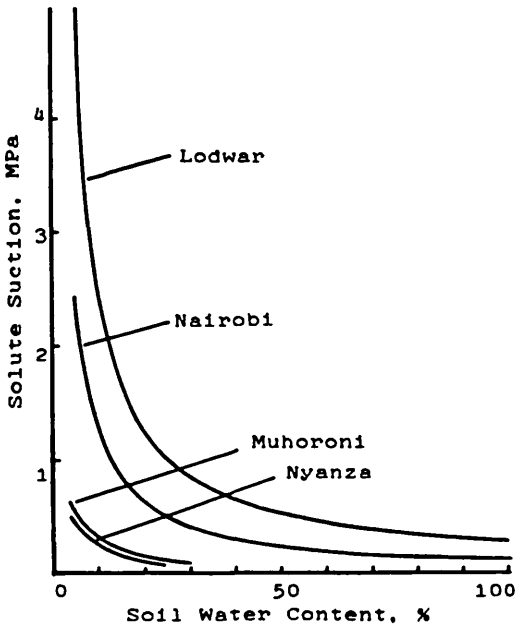


Fig.3 Effect of Salt Content

effective stresses in the sample, thereby reducing the permeability. A few drops of the leachate were tested daily for the presence of chlorides using Silver Nitrate solution, which forms a white precipitate of Silver Chloride when chlorides are present. Leaching took up to two weeks before the chloride test ceased to produce a precipitate. Total water flow through the sample was of the order of three times the sample volume.

6 LABORATORY CONTROL OF SUCTION

Probably the commonest laboratory technique for suction control is the use of elevated air pressure together with a saturated porous plate or membrane with a small pore diameter (or high air entry value or high bubbling pressure). There has been some doubt in the past concerning the suction controlled by the pressure plate and the pressure membrane apparati. The latter is often stated to include a semi-permeable membrane and may therefore be thought to control total suction. However, the membrane commonly used, Visking, is designed as a semi-permeable membrane for

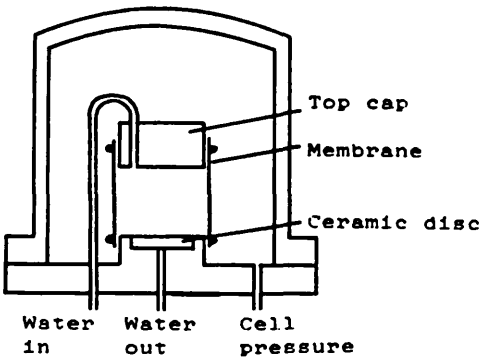


Fig.4 Leaching cell

dialysis of blood in the medical field. The pore diameter required for a particular air entry value may be simply calculated using:-

$$h = \frac{2T}{R\delta}$$ Lambe & Whitman (1969)

Where h = suction
T = surface tension
R = radius of pore
δ = density of water

For a 15 bar ceramic plate or cellulose membrane, the required pore diameter is .00019mm or 1900 Angstroms. By comparison, the diameter of a hydrated Calcium ion is 6 to 10 Angstroms (Talibudeen 1981). The pore diameter is of the order of 200 times the hydrated cation size, and the membrane or porous plate will not be semi-permeable with regard to Calcium ions and similar sized particles. Thus both the pressure plate and the pressure membrane apparati will control matrix suction and not the total suction.

In order to comply with the ISSS definition of solute suction, the water supply must be identical in terms of disolved salts to the water in the soil. In the light of the above discussion, this is a slightly more complex concept than it appears at first sight. It may seem reasonable to use the salt content and type, determined from leaching, with the total water content of the soil. This approach would seem to

maintain the solute suction at a constant value. However, there will be a difference in salt concentration between the free water phase in the soil and the supply water. The water in the free phase is in equilibrium with the absorbed water, but not with the supply water which has a different salt concentration. Since the membrane or plate is permeable to the salt ions, there will be diffusion of salts between the soil and the supply water. This will cause a change in solute suction in the soil. It does not therefore appear to be possible to eliminate the effects of uncontrolled solute suction changes in the pressure plate or membrane system, even by using supply water of the same solute suction as the soil.

Where soils have a high salt content, this movement of salts may affect laboratory testing. The movement of the salt will however be much slower than the movement of water due to the matrix suction gradient, and it is likely that the effects of the salt movement will be evident as a creep type swelling in comparison with the more rapid swelling due to the water flow. It may be evident during pressure plate testing, if the test duration is long. No data is currently available, except that by Richards (1980), which suggests that the salt movement is incomplete after 100 days. In order to accurately quantify the engineering parameters that we need in testing soils at controlled suction, it is thus desirable to know both total and matrix suction values, or the salt content before and after the test with either of the matrix or total suctions.

However, returning to Fig.3, we can see that the change in solute suction above about 25% water content is fairly small for these soils. In this region, pressure plate testing using distilled water should provide useable data on the effect of matrix suction change, particularly if the salt content is less than about 0.5% of the dry mass of the soil. Where doubt exists or where high precision is desired, solute suction values must be obtained before and after the test using either the salt content and composition or total and matrix suction measurement.

7 ACKNOWLEDGEMENTS

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APPENDIX A

The definitions introduced by the Review Panel (1965) are:-

Total Suction is the negative gauge pressure relative to the external gas pressure on the soil water to which a pool of pure water must be subjected to in order to be in equilibrium through a semi-permeable membrane.

Matrix Suction is the negative gauge pressure on the soil water to which a solution identical in composition to the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water.

Osmotic (solute) Suction is the negative gauge pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semi-permeable membrane with a pool containing a solution identical in composition to the soil water.

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CHAPTER 9

DEVELOPMENT OF EXPERIMENTAL EQUIPMENT

9.1 INTRODUCTION

As discused in Chapter 2, most of the expansive soil research has suffered from a lack of measurement of the full stress state i.e. σ_a, σ_r and u_w. It was felt that little benefit could be gained from pursuing this line of research unless the full stress state was determined during swelling. It was also considered necessary to be able to permit swelling with control of the suction.

Many types of oedometer have been designed in the past, some of which have been specifically designed for testing expansive soils. In general, these fall into two categories; the lever arm system and the hydraulic or pneumatic loading system. There are many variations, some of which are significant to the present study.

The first part of this chapter describes the design, fabrication and commissioning of an hydraulic oedometer for determination of the full stress state during swelling and compression of unsaturated soils. The second part of the chapter describes the design, fabrication and commissioning of a device for measurement of the radial deformation of a sample in a triaxial cell.

9.2 PREVIOUS DESIGNS

9.2.1 HYDRAULIC OEDOMETERS

The earliest oedometer of importance was described by Rengmark & Eriksson (1953). It appears to be the first hydraulic oedometer. All the essentials of the hydraulic loading system in common use today can be seen in their equipment, shown in Fig.9.1 It suffers from some obvious disadvantages by modern standards, such as the submerged dial guage, but appears to be comparable in test capability, although not the ranges of stresses, to the more advanced equipment. In addition to the hydraulic loading, negative pore pressures of up to the cavitation stress of water could be applied through the fine pored suction plate in the cell base. No data has been provided on this use, but such data as they may have obtained would have been of little current interest due to the low suction range.

Development of the hydraulic oedometer was taken a step further by Rowe and Barden (1966), with a practical cell that is still widely used. Barden et al (1969) used the Rowe oedometer with a fine pored ceramic in the base for control of suction with elevated pore air pressure. That system has been developed for use in expansive soil testing for suction up to 850kPa (Zein 1984).

A highly complex oedometer is described by Richards et al (1984), which is apparently designed for precise volume control during constant volume swell pressure testing. The hydraulic loading is achieved in a similar manner to the Bishop & Wesley (1975) stress path triaxial cell. High suction is obtainable with elevated pore air pressure through the use of a cellulose membrane with very fine pores.

None of these hydraulic oedometers has permitted measurement of the radial stress.

9.2.2 LEVER ARM OEDOMETERS

Developments of oedometers for use in the lever arm loading frame for expansive soil testing have followed three separate routes. Two methods of suction control have been used, and radial stress has been measured in oedometers without suction control.

Komornik & Ben Shalom (1971b) describe an oedometer in which the suction is controlled using the osmotic suction of a solution of Polyethylene Glycol (PEG) which is separated from the sample by a membrane which is semi-permeable with regard to the PEG. The membrane is permeable to the salts dissolved in the pore fluid, and thus only the matrix suction is controlled. Kassif et al (1973) used a PEG of higher molecular weight and thus larger molecular size in order to overcome leakage through the membrane that had affected the earlier system.

Suction control by elevated pore air pressure has been used by Escario (1969) in a simple cell. No compensation is made for the force due to the air pressure acting on the loading ram, but this is taken into consideration when calculating the stress on the sample. The vertical stress applied to the soil and the air pressure used to establish the suction are thus not fully independent. Constant applied stress is thus difficult to achieve when changing the suction. Aitchison et al (1973) describe a system in which the force due to the air pressure acting on the ram is compensated by an identical ram, driven by the same air pressure, acting in the opposite direction.

Radial stress measurements in expansive soil testing are first reported by Komornik & Zeitlen (1965), using a thinwall oedometer ring fitted with strain gauges. A similar concept has been used by Ofer (1980) as shown in Fig.9.2. Thompson (1963) describes an oedometer in which a load cell has been mounted through the sample ring (Fig.9.3), which in that case was a long tube in which samples were consolidated from a slurry. Burland (1967) found this

load cell to suffer from instability of output during test periods of about one week.

Brackley (1975) used a strain gauged thinwall oedometer ring with elevated pore air pressure inside a triaxial cell, (Fig.9.4), to measure the swell pressure of a sample during controlled decrease of suction, but does not report the radial stress.

Hydraulic stress measurement is used in the oedometer described by Dyvik et al (1985). The sample ring has a fluid filled chamber separated from the sample by a thin section of the inner wall of the ring. Pressure generated in the fluid in this chamber by the radial stress in the sample is measured with a pressure transducer mounted directly onto the ring. The layout is shown in Fig.9.5.

9.3 CONTROL AND MEASUREMENT TECHNIQUES

Since none of the systems noted above provided all the capabilities required for this study, a suitable system had to be designed and fabricated. The two essential requirements of the oedometer were that the radial stress should be measured and the suction be controlled. Measurement of the suction would on its own be inadequate, as the control is needed to ensure that suction gradients within the sample are not excessive during the tests. Attempts to measure the suction while allowing the sample free access to water would lead to a state of zero suction behind the wetting front and unchanged suction ahead of the wetting front (Wong & Yong 1973). In addition it was thought desirable to design as simple a system as possible, without compromising the technical capability of the oedometer.

9.3.1 SUCTION CONTROL

The choice regarding suction control lies between elevated pore air pressure with a fine pored ceramic and Polyethylene Glycol with a semipermeable membrane. The choice is somewhat arbitrary, but was influenced by several factors.

9.3.1.1 Choice of suction control

The PEG system has been successfully used, but two difficulties weigh against its use in this study. Firstly, it has been reported that the membrane is subject to attack by bacteria in the soil, and that its lifetime may be as short as two weeks (Kassif & Ben Shalom 1971b). In the long term tests that were envisaged, this could be a severe problem. Some success appears to have been achieved in the treatment of the membrane with an anti-bacterial compound (Komornik et al 1980) but some doubt remains regarding its desirability for long term tests. Secondly, since the suction is due to the concentration of the solution of PEG in water, any flow of water into or out of the sample, and

thus out of or into the PEG, will change the matrix suction across the membrane. Correction of the solution concentration may thus prove necessary during each stage of controlled suction.

The elevated pore air pressure system has also been succesfully used, and also suffers from some disadvantages. The use of high air pressure raises the risk of accidental injury, due to the high stored energy and due to the possibility of explosion if the pressurised air comes into contact with oil. Since air disolves in water under high pressure, and comes out of solution when the pressure is reduced, air tends to collect on the low pressure side of the porous ceramic plate. For accurate water volume measurement, this air must be removed from the system. However, there is no change of the matrix suction due to water movement through the porous plate, and the porous ceramic is not affected by the soil bacteria.

In addition, this technique has been used at Imperial College, both in the 1960's and more recently by Maswoswe (1985) and Toll (1988). Due in part to the considerable amount of development that would in any case be required on other aspects of the stress path oedometer, it was decided to continue in the use of the system for which the technology was already well developed at Imperial College. The initial development was based on the use of a ceramic plate with a 15bar desaturation pressure, with the possibility of extending the suction range to 30bar using a cellulose membrane.

9.3.1.2 Air Flushing System

A pair of mercury pots included in the water supply circuit permits removal of air which has diffused through the water in the porous ceramic. Raising or lowering of the moveable pot causes water to flow through the spiral drainage groove beneath the ceramic and to displace such air as may have collected there. Air bubbles are collected in perspex bubble traps and are be released periodically.

9.3.2 RADIAL STRESS MEASUREMENT

Three forms of radial stress measurement have been used in oedometers, as mentioned in section 9.2. Two other techniques were also considered. These were a thinwall section of the ring, as used by Toll (1988a) in a CBR mould, and a "null ring" utilising a strain gauged diaphragm cut into the ring, where hydraulic pressure would be applied to maintain zero strain in the diaphragm.

9.3.2.1 Choice of Radial Stress Sensor

The stress that is of prime interest in the study of unsaturated soil is the total stress that exists in the natural

state, i.e. without the elevated pore air pressure. This can be obtained arithmetically by subtracting the air pressure from the total stress in the test apparatus which includes the air pressure. However, the air pressure may be considerably larger than the total stress in the natural state, and thus any small percentage error in the air pressure measurement could cause a large percentage error in the stress of interest. It was decided at an early stage that it would be preferable to measure the radial stress without the air pressure.

The simplest technique for achieving this radial stress measurement is to apply the same air pressure on both sides of the stress sensor. The sensor then acts as a differential pressure transducer, with the pressure difference being the stress due to the soil with pore air pressure equal to atmospheric pressure.

This could be reasonably easily effected for the thinwall ring, the thinwall section or the load cell. It would have been much more difficult to contrive for the hydraulic ring and the null ring. The choice then lay between the thinwall ring, the thinwall section and the load cell.

The CBR mould reported by Toll (1988a) was in use at the time that the oedometer was being designed. Whilst giving apparently adequate results for the purpose to which it was put, some disadvantages had become apparent. The most significant problem is the sensitivity of the radial stress measurement to the axial load applied to the ring by the securing bolts. The shift of the strain gauge reading that occured during tightening of the bolts was large, and depended on how firmly the bolts were fastened. This sensitivity to stresses induced during assembly was also thought to be possible for the null ring and the thinwall ring. Variations in axial stress in the thin section during the test period could also be caused by changes in the pore air pressure or by friction between the soil and the ring.

The radial stiffness of the thinwall ring is reported by Komornik & Zeitlen (1965) to be eqivalent to the control normally excercised over the vertical displacement in constant volume swell pressure tests. While this is a somewhat unclear statement, it does indicate that the tests performed in their apparatus are not zero radial strain tests. Attempts to measure the radial displacement with a dial gauge are reported, and indicate a stiffness of about $4*10^{-3}$mm per 100kPa. Although true zero radial strain is not possible with a passive stress measuring system, much smaller displacements are possible than those achieved in swell pressure testing where displacements are controlled by manual application of load to maintain the dial gauge reading within specified limits.

The load cell penetrating through the cell wall was chosen as it should be insensitive to axial stress in the cell wall, and can readily be designed to have an air pressure equal to the pore air pressure applied within or behind the load cell. It can also be

designed to be very stiff, so that the displacement of the face of the load cell is equivalent to a very small radial strain of the soil.

9.3.2.3 Design of Load Cell

Load cells may be designed to utilise either bending or axial strains of the supporting member. The former is preferable where high electrical outputs are desirable and axial stiffness is of little concern. The latter provides a greater stiffness at the expense of the electrical output. For the measurement of radial stress through a rigid wall, the load cell stiffness is of major importance. The efficiency of such a load cell has been assessed by Maddocks (1983) in terms of the relative stiffnesses of the soil and the load cell, and shows that the error in the measured stress increases as the load cell stiffness decreases.

The design of this axial strain load cell can be traced through Thompson (1963) and Burland (1967). Burland (1967) used an axial strain load cell in a triaxial cell base to measure the stress in the soil sample. The design of the load cell for the oedometer is essentiallly similar, but is about half the size. Details are shown in Fig.9.6. The load cell has four webs which are loaded in compression. Opposite pairs were gauged on both sides and wired in series to give four strain gauges in each arm of the bridge. There are also two sets of four gauges on an unstressed pillar to complete the bridge and provide temperature compensation.

The webs were cut from a thinwall cylindrical section in preference to the flat sided webs used by Burland (1967), as this simplifies the manufacture. If flat sided webs had been chosen, then several delicate cuts with a milling machine would have been required, with at least two set-ups of the load cell on the milling table. Machining of the cylindrical thinwall section can be done in the lathe at the same time that the rest of the turning work is performed, and the cuts in the milling machine are relatively simple, and only require one set-up. The stiffness of this load cell was calculated to be $9*10^{-3}$mm per 100kPa, which is considerably better than the stiffness of the thinwall system used by Komornik & Zeitlen (1965).

A precise match of the load cell face with the bore of the oedometer is desirable. It was intended that this would be achieved by installing the load cell blank in the oedometer while the bore was being machined. Difficulties encountered with the alignment and mounting are reported in section 9.4.4.1.

Since the load cell was to be used as a differential pressure transducer, it was considered to be prudent to design the thinwall webs to survive an accidental loss of air pressure on one side. This required the design stress on the loading face to be 3MPa, which was the maximum air pressure that would be useable if the

second stage development, with a cellulose membrane, was implemented. This results in a fairly severe penalty in electrical output at the stresses that are of interest, as the gauges are only working at about 15 to 20% of their nominal maximum strain.

A full bridge configuration was used for the strain gauge circuit, with gauges on opposite webs being linked into the same arm of the bridge. Gauges were attached to the outer and inner faces of each web, giving 4 gauges in each active arm. Two set of 4 dummy gauges are located on the central pillar. The electrical layout is shown in Fig.9.7.

Calculations for the electrical sensitivity and mechanical response are given in Appendix 9A.1 at the end of this Chapter.

Output of the load cell was monitored using a Shape Model 7502 Digital indicator. This has adjustable gain and zero settings, and can thus be calibrated to read in the desired units.

9.3.3 VERTICAL STRESS CONTROL

A limitation of the usual vertical stress application system in the hydraulic oedometer when used with elevated pore air pressure is the need to maintain a downward load on the plunger. Simple calculations based on the areas of the plunger shaft and end define the range of stresses within which this condition can be satisfied. Zein (1985) worked within the limitations of his particular cell, accepting the non-uniformity of stress that is caused between the soil beneath the plunger end and beneath the flexible membrane. For the dimensions used in the current oedometer, the stress beneath the plunger in a standard Rowe oedometer layout would become negative when the applied stress plus the air pressure acting downwards on the annulus "A" is less than the air pressure acting upwards on the plunger end "PE" as defined in Fig.9.8. Then:

$$\frac{\pi}{4} d_2^{\,2} . u_a = \frac{\pi}{4} (d_2^{\,2} - d_1^{\,2}) (\sigma_{av} + u_a)$$

i.e. $\sigma_{av} < [(35^2 - 16^2)u_a]/16^2$

or $\sigma_{av} < 0.264 u_a$

Skinner designed an hydraulically compensated ram system for the high stress triaxial machine at Imperial College (Bishop et al 1974). A similar layout using neoprene O-rings, as shown in Fig.9.9, was used initially, but was later modified as described in section 9.4.1.

9.3.4 DETAILS OF OEDOMETER DESIGN

The salient dimensions and details of the Stress Path Oedometer with the final compensated plunger system are shown in Fig.9.10. Material thicknesses and stud dimensions were checked

against BS 5500 Code for Pressure Vessel Design.

9.3.4.1 Compaction Base Plate

The 1.0mm high platform over the sample area of the compaction base plate leaves the lower surface of the sample above the end of the sample cylinder when the base plate is removed after compacting the sample. On fitting the base plate which has the porous ceramic disc, there is no contact between the soil and the ceramic. All preparations, including the application of the pore air pressure can be completed before the sample is pushed down into contact with the ceramic using the vertical stress control. This avoids the risk of cavitation in the pore water pressure measuring system and the risk of water intake by the sample during assembly of the equipment.

9.3.4.2 General Layout of the Equipment

A diagrammatic layout of the equipment is shown in Fig.9.11. Air pressure equal to the pore air pressure in the sample is applied to the top mercury pot of the vertical stress control system. The desired vertical applied stress, σ_{av} is preset using the 10bar gauge before the air pressure is applied. This system maintains the vertical applied stress, σ_{av} regardless of variations in the air pressure. Since no compensating springs were used due to the large mass of the stainless steel mercury pots, checking of the mercury levels was necessary. Translucent nylon tubes connecting the top and bottom of each pot were used , with marks on the tubes against which the mercury levels could be checked (Fig.9.12). The level could be corrected using the screw pump, but in practice this was seldom necessary.

9.3.4.3 Rubber Lining and Membrane

Since the radial stress load cell is unsealed at the soil surface, it is necessary to use an impermeable lining between the load cell and the soil to protect the load cell from water and to prevent soil entering the small annulus around the piston and causing measurement errors. A 4-inch triaxial membrane was found to be the most convenient source, modified by forming a flange at one end. The mould for this comprised a 4-inch perspex tube standing on a compaction base plate. The membrane was placed over the tube and the flange formed by coating the lower part of the membrane and the exposed part of the base plate with several layers of "Revultex".

The hydraulic loading membrane was formed in a similar manner, but using a purpose made varnished wood mould made by the author. A section of thoroughly washed 4-inch membrane was fitted over the mould such that about 1mm projected above the end of the mould.

Revultex was poured onto the end of the mould to fill the space within the membrane, and left to cure overnight. Several coats of Revultex were then applied for at least 5mm each side of the joint to ensure adequate strength. The flange was formed by applying several coats of Revultex to the flange of the mould, with at least 5mm overlap onto the 4-inch triaxial membrane. After curing, the Revultex was liberally covered with Johnson's Baby Powder to deactivate the surface and prevent sticking.

9.4 COMMISSIONING AND CALIBRATION

9.4.1 PLUNGER FRICTION

Plunger friction was measured by placing weights on the top of the ram until sliding was initiated. Results of three tests for the initial configuration, which was shown in Fig.9.9, are given in Table 9.1.

When PTFE O-rings were substituted, the resistance was much less variable, and somewhat lower, but the machining tolerances required from the hard O-rings could not be achieved in the 3 O-ring system, resulting in leakage past the seals. The system which was adopted and in which co-axial mounting of the O-rings can be achieved with ease is shown in Fig.9.13. Machining tolerances again proved too arduous, but the errors were overcome by glueing the O-rings into the oversize grooves.

The resistance could be significantly reduced by improving the quality of the machining on the plunger, an example of which is shown in Plate 9.1, and also by PTFE coating the plunger shaft. Fig.9.14 shows the reduction in the coefficient of friction that can be achieved with PTFE sliding on PTFE as compared with PTFE sliding on steel. These tests were performed in a shear box by placing either one or two sheets of PTFE between two steel blocks.

The error, due to this frictional resistance, in stress applied beneath the plunger, assuming no load transfer by the membrane, is about 10kPa. While a frictionless system is desirable, it has proved difficult to achieve. Techniques used in traixial cell designs include castor oil in an unsealed bush and a rotating bush. Current practice in high quality cells is to use internal stress measurement, which bypasses the problem. This solution is not feasible in an hydraulic oedometer. The grease plug used by Escario (1969) will give low resistance while the plunger is running true in the bush, but will give unknown friction if the plunger and the bush make contact.

9.4.2 PREPARATION AND MOUNTING OF CERAMIC DISCS

9.4.2.1 Preparation of ceramic discs

Previous work at Imperial College had been limited by the

availaility of suitable pressure plate ceramics. Both Maswoswe (1985) and Toll (1988) used 5 Bar ceramics, as the best available in suitable sizes. Use of the same grade of ceramic would have limited this research to lower suctions and higher moisture contents than the range relevant to the "dry compaction" technique. Factory made ceramics are possible in all grades, according to the trade literature. However, the lead time for fabrication and the tooling costs are severe penalties. Standard 15bar ceramic plates are available for pressure plate systems as used in the soil science field. These are 300mm in diameter, and domed by about 10mm across the diameter. In this form they are obviously unsuitable for use in an oedometer, and it was decided to attempt to cut discs of the right size from one of these standard plates.

A diamond tipped coring barrel was used, courtesy of the Rock Mechanics Department, and was succesful in cutting the first of three discs from the plate. Unfortunately the operator decided not to support the domed plate on a bed of cement mortar and the second cut broke both the disc that was being cut and the remainder of the plate. The broken disc was glued together using Araldite epoxy. At this stage it had become apparent that the ceramic was not as hard as initially thought by the workshop staff, and this was confirmed by hand finishing the edges of the discs by the author using a standard file and carborundum paper. The remnants of the plate were found to be just large enough for a third disc to be cut, and this was achieved by the author using a tungsten carbide tipped rod saw in a hacksaw frame. The edges were finished as before using a file followed by carborundum paper. This technique was used for all subsequent disc cutting.

The discs were still unsuitable in that they were not flat on either side. The first two discs were ground and lapped in the Rock Mechanics Department. One was correctly finished, but the second tapered across the diameter by about 1mm. Subsequent discs were flattened on one side by careful rubbing on a sheet of "wet or dry" carborundum paper placed on a flat surface. A flow of water was used to prevent clogging of the abrasive. The second side was flattened by a similar process after the disc was glued into the base plate. Since the stainless steel base is much more resistant to abrasion than the ceramic, the finished surface of the ceramic was in plane with the top of the base plate.

9.4.2.2 Mounting of ceramic discs

Mounting of the ceramic discs in the stainless steel base plate essentially followed the technique described by Maswoswe (1985). Some additional difficulties were encountered and overcome.

The drainage channel below the ceramic was cut in a spiral so that there was only one flow path possible for the water when

flushing trapped air out of the base. Machining of this spiral requires a slight abuse of the lathe in order to obtain a fast enough cross feed, and the quality of the cutting was consequently a little low. No attempt was made by the machinist to remove the resulting roughnesses. Hand finishing with carborundum paper by the author improved the finish, but did not prevent damage to the ceramic when high pressures were applied to the ceramic. After being used for a few times, leakage of air past the ceramic occurred.

Examination under a microscope revealed that the ceramic and the epoxy had separated. It was found that the ceramic, despite being clamped during glueing, was touching the base at a few points where the metal had been left proud of the general level. On being stressed to the design pressure of 1500kPa, localised crushing of the ceramic occurred at these points. On subsequent stressing and unstressing, the ceramic was able to deflect slightly, as shown by the stress-deflection data for the first disc in Fig.9.15. The deflection of the ceramic caused fracturing of the edge of the ceramic where it ws held rigidly by the glue which was attached to the steel and thus unable to deflect.

Very careful and meticulous remachining by the author produced an adequately flat surface on which to mount the ceramic, as shown by the improved stress-deflection data in Fig.9.15 for the second disc. The opportunity was also taken to increase the size of the drainage groove in an attempt to reduce the flushing time. This was only partially effective, and the flushing time has remained excessively long.

9.4.2.3 Saturation Procedure

Saturation of the ceramic discs was achieved by dissolving the trapped air in water under high pressure. For this operation the cell was assembled without the load cell and with a plain flange on the top. This provided three connection points for the tubing, of which only two were required. One was used for the de-aired water inlet and the pressure from the screw pump, while another served as the drain when required.

As much air as possible was displaced by flushing de-aired water through the ceramic against atmospheric back pressure. The drainage taps from the base plate were then closed and the pressure within the cell raised to about 600kPa with the screw pump. Occasional venting through the drainage taps assisted in displacing air bubbles from beneath the ceramic. Flushing with the mercury pot system was also used to displace air bubbles.

As the quantity of gas in and below the ceramic decreased, so the pressure in the cell was more easily maintained. At intervals the water within the cell was replaced with fresh de-aired water. Some care was necessary in that the drainage taps must be opened before the cell pressure is lowered. Failure to do this leads to

an upward stress on the ceramic, and, usually, a broken ceramic.

De-aired water was then allowed to flow through the cell under the supply pressure and the drain tube was opened into a bucket. About 4 to 5 litres or 5 to 6 times the cell volume was flushed through in order to minimise the amount of dissolved air left in the mixed water within the cell. Complete saturation was generally obtained in less than one week, but depended in part on the time available for maintaining the pressure within the cell and flushing fresh water through the cell.

Indicators of saturation are a small cell pressure drop overnight and a fast water pressure response beneath the ceramic after the drainage taps are closed. Confirmation could only be obtained by applying an air pressure within the cell and monitoring the drainage flow rate. Fig.9.16 shows two test results. Test A indicates saturation of the ceramic up to about point "X", where the rate of flow increases markedly. This occurred under an air pressure of 2200kPa, which is far in excess of the design blow through pressure of 1500kPa.

However, the "creep" flow rate was high, at about .004cm^3/min. The creep flow is unavoidable due to the diffusion of air into the water in the ceramic as discussed in section 8.5.3.1. The ceramic in test A was subsequently found to have a "pinhole" which would have contributed to the apparent creep rate. The pinhole was identified by applying a small air pressure beneath the flooded ceramic and watching bubbles form at one point. The ceramic in test B which showed a creep rate of 0.001cm^3/min did not suffer from a pinhole, but failed the saturation test due to air passing through a defective edge seal. Air diffusion through the ceramic did occur during the tests on soil samples, and a rate of 0.0009cm^3/min was observed in test SP05 at an air pressure of 940kPa.

9.4.3 CELL STIFFNESS

The overall stiffness or compliance of the cell was measured using a brass block in place of a sample. The cell was fully assembled, except that the loading membrane was not clamped between the top cap and the cell body. A neoprene gasket was used in its place to effect the seal. The loading membrane was securely clamped in the ram in the same manner in which it would be during a test.

The compensated ram was lowered into contact with the brass block at the start of the test, and contact was maintained during the test by applying a light downward push on the ram by hand, together with small rotations to overcome the friction in the seals. The results have been presented in Fig.9.15 above in the discussion of the mounting of the ceramic. The data for the second ceramic are relevant here.

The mean stiffness of the cell, from curve for the second

ceramic in Fig.9.15, is 0.0025mm/100kPa. For a 30mm thick sample this corresponds to a strain of 0.000083mm/mm per 100kPa. For a sample void ratio of 1.3 the error in the void ratio will be 0.00019/100kPa. During the swelling stages of test SP01 the change in void ratio was 0.1538 for a change in internal axial stress of 940kPa. The error in void ratio due to the compliance of the cell is 0.0018 or a 1.16% underestimate of the change in void ratio. The error in the final void ratio would be 0.14% of the void ratio, which is considered to be insignificant. Corrections were therefore not applied in swelling and compression tests.

9.4.4 LOAD CELLS

9.4.4.1 Alignment

On installation of the first load cell in the cell body, large changes in output were observed during tightening of the securing capscrews. This was found to be due to the loading piston making contact with the side of the hole through the cell wall. The misalignment was caused by burrs and machining defects that had not been removed around the holes for the capscrews. The load cell thus did not fit flush against the cell body. After refinishing the mounting flange by hand, the large output changes were eliminated, but the loading face projected about 0.02mm proud of the cell bore.

This could not be rectified as the remachining would probably have damaged the thin webs of the load cell. It would have been possible to manufacture a new load cell to higher standards and to refinish the oedometer bore with the new blank load cell in place. Since the fabrication of the Stress Path Oedometer had already taken about one year this was not considered to be a desirable solution. It was also thought that, since the membrane would be about 0.25mm thick, the 0.02mm projection would not cause large errors in radial stress measurement.

During the early calibration stages a further problem of misalignment was encountered. An apparent hysteresis error of up to 50kPa was found with the load cell mounted in the oedometer, but not when dead weights were used with the load cell resting vertically on a load frame. Careful inspection showed that the load cell piston was still making contact with the sides of the hole and leaving polished or "high" spots on the piston. The high spots were removed using a fine grade (no. 400) "wet or dry" carborundum paper without water and the load cell reinstalled.

The stress was then cycled to about 2MPa and the hysteresis checked. Until the hysteresis was eliminated, the removal of the high spots was repeated. In addition, for the load cell in oedometer no. 2, the location of the load cell in the hole was improved by placing a 0.075mm thick feeler gauge between the load cell piston and the cell body at the point at which the high spots

had been observed. This served to force the load cell away from its free mounting position. The clearance around this load cell was checked using a 0.012mm thick feeler gauge, and found to be adequate after this procedure was adopted.

9.4.4.2 Stability

Two load cells were manufactured, one for each oedometer. Load cell LC1 appears to have performed satisfactorily, as the change in the unstressed output has not been excessive over the 20 month period of use. LC2, however failed due to leakage of water through a membrane during calibration, and has subsequently suffered from considerable instability.

One strain gauge on the web which had been affected was removed and replaced. The load cell was recalibrated but found to suffer from large and rapid zero drifting. The drift was shown to be humidity dependent by enclosing the gauge in a bag of silica gel after some drift had occurred. The variations in output are shown in Fig.9.17. Attempts were made to provide a waterproof barrier to stabilise the output using first M-Coat "A" and then W1 microcrystalline wax.

Neither was effective, and the load cell was then repaired by replacing both gauges on the affected web and coating all gauges with 3 coats of AB610 phenolic epoxy resin, which was cured at 100C for 4 hours. This again failed to rectify the problem and the whole load cell was regauged and rewired.

It still proved unstable, but by this time the drift monitoring system (section 9.4.4.3) was under development and the unstable load cell was a useful test for this technique. Although encouraging results were obtained, the entire load cell was once again rebuilt, this time as two complete bridges. Each used two opposing webs and had its own dummy gauges. Both were wired for the drift monitoring system.

One bridge proved faulty, with an intermittent earth leak, and the other was unstable with regard to humidity changes. Fig.9.18 shows a long term test of the "good" bridge in which a constant stress of 100kPa was applied. Up to point "A" the back of the load cell was open to the laboratory air. Thereafter it was enclosed with silica gel in a plastic bag. The shift immediately after the enclosure, and thus apparently due to drying is about 40kPa.

In order to stabilise the humidity around the load cell, the air supply was routed through a chamber filled with dry silica gel. In addition, for LC2, the drift with time during tests was monitored with the system described below.

9.4.4.3 Drift Monitoring System

As a result of the problems discussed above, it was decided that a means of determining at least whether or not the load cell

was drifting during a test, but preferably a means of correcting the output, was highly desirable. A technique of switching the configuration of the load cell has been devised which permits measurement of changes in the unstressed output without unloading to zero applied stress. The normal wiring diagram for 4 leadwires is shown in Fig.9.19a. This will be referred to as the Normal Mode.

If output changes occur due to creep of the gauges or glue or due to electrical leakage across gauges or to earth, the result is a change in the balance of the bridge. This is seen as an offset of the calibration from the original calibration, but the slope of the calibration line is generally unaffected. In service, where the actual stress is not known, there is no way of differentiating between an output change due to an applied stress and due to drift.

Fig.9.19b shows a "null" configuration where the two active gauges are adjacent to each other. This is called the Null 1 mode. In this layout, changes in resistance in the active arms due to applied stress theoretically cancel, and in practice give a very small output in response to load changes. A change in resistance of one arm causing drift will however not be cancelled out. The magnitude of the difference between the initial and the drifted outputs in the null configuration will be equal to the magnitude of the drift in the normal mode.

The sign of the drift in the normal mode cannot be established using only these two modes. Use of a second null mode, shown in Fig.9.19c completes the requirements for determining the magnitude and direction of the drift.

For example, in the Null 1 mode it is not possible to distinguish between drifts in arms A1 and D2, as they are on opposite arms of the bridge. However, in the Normal mode, where A1 and D1 are adjacent, the effect of a change in A1 will be opposte to the effect of an identical change in D2. The Null 2 mode reverses the direction of the drift due to changes in D1 and D2 as compare with the Null 1 mode.

A diagrammatic layout of the system is shown in Fig.9.20 using two Din sockets, "A" and "B", to select the three modes. Three Din plugs are used, two at a time. Two provide the links for the Normal mode, and the third plug provides the crossed links for the null modes. With the crossed link plug in socket "A", Null 1 is selected. Similarly, the crossed links in socket "B" select Null 2. Din socket "C" provides the connection to the measurement device, in this case a digital indicator unit.

In order to demonstrate that this system could permit magnitude and direction of the zero drift to be measured, a "mock-up" electrical circuit (Fig.9.21) was used. This consisted of a set of 1000 ohm resistors to model the four arms of the bridge. Resistance changes due to straining were modelled with $1*10^{6}$ ohm resistors in parallel with each active arm. The error

was provided with a $1.2*10^6$ ohm resistor in parallel with any one arm of the bridge. The output data for the three modes are given in Table 9.2 and shown graphically in Fig.9.22.

It can be seen that the changes in null output between the "stressed" and "unstressed" conditions is small in each case compared with the change in output for the normal mode. The lines Null 1, Null 2 and Normal represent the "calibration" lines for the model system. The points A1, A2, D1 and D2 are the outputs obtained with the error resistor in parallel with that arm of the bridge. The difference between the outputs from drift on arms A1 and D2 can now be identified by the reversal of the direction of the drift in the null modes for D2, whereas the directions of the drift for A1 are the same.

The procedure for correcting the output is:

Direction of drift		Correction to
Null 1	Null 2	Normal mode
+ve	+ve	subtract drift
-ve	-ve	add drift
-ve	+ve	subtract drift
+ve	-ve	add drift

The observations made on load cell LC2 are shown in Fig.9.23. The sudden change in output on the morning of the third day was caused by changes in resistance of a soldered joint as it began to fracture. After it was repaired the output returned to almost the same value as before the breakage. The switching system in use for this test was not of particularly high quality and may have contributed to the apparent imperfection in the comparison between the normal and null modes.

However, there is a limitation in the capability of the drift monitoring system. The foregoing discussion and evaluation was based on single error sources. Where there is more than one error it may not be possible to obtain accurate corrections. In particular, if both active gauges or both dummy gauges drift by the same amount, then the normal mode will be affected, but neither null mode will show a change in output. This type of drift can only be checked when the load is removed from the load cell. This system, although not a complete solution, will in almost all cases show wheter or not drift is occurring, and will give an indication of the magnitude of the drift.

9.4.4.4 Calibration

The load cells were calibrated indirectly against a Budenberg deadweight calibrator. In order to avoid the risk of water reaching the gauges, the load cells were calibrated using air pressure. The pressure was measured using an hydraulic transducer

which had been calibrated directly against the Budenberg calibrator. A calibration error curve for load cell LC1 is shown in Fig.9.24 in terms of the error relative to the Budenberg calibrator. Also shown is the calibration error curve for the hydraulic transducer. The apparent hysteresis of the load celll is about 5kPa, and the maximum error is about 7kPa. Figure.9.25 shows the data for load cell LC2. The hysteresis is again about 5kPa, and the maximum error about 5kPa.

Load cell LC1 was checked for its response to the combined effects of an all round air pressure and axial and circumferential stress in the cell wall. This was done by applying equal air pressure in the cell and to the back of the load cell. The results in Fig.9.26 show no response to the applied stresses.

Load cell LC2 was tested for its response to axial stress only in the cell wall, by applying a vertical stress to a loose fitting steel block inside the oedometer using the hydraulic loading membrane. The results, also shown in Fig.9.26 indicate that it is imsensitive to the stress in the cell wall.

A test of the cross sensitivity was also undertaken. LC2 was mounted on the oedometer with the loading piston projecting outwards. Weights were hung on the loading piston to simulate friction or shear stress on the load face. The results are shown in Fig.9.27 in terms of apparent axial load versus shear stress on the load face. The results given below from the push out tests indicate shear stresses of the order of 30kPa, for which the apparent axial load is about 10kPa. This is very much less than the radial stresses measured in the tests on soil samples.

9.4.4.5 Cell Lining

The first test, SP01, used a rubber lining inside of which the sample was placed before compaction. The lining was unstretched at the soil placing stage, and the bore of the cell was coated with a silicon grease. On removal of the sample at the end of the test it was noted that the lining had wrinkled or buckled during the compaction and had penetrated into the sample by about 0.5mm at several places. Although it was thought that this would not have major effects on the stresses or the volume change, attempts were made to improve the lining.

A thin PTFE sheet was used in test SP02, which was not completed due to load cell failure. There was no appreciable difference between the compaction stress path for this test and for the other tests, but the resistance to removal of the sample after the test was terminated appeared to be high. In addition, the PTFE lining had wrinkled.

From test SP04 onward the rubber lining was used as before with a layer of silicone grease, but in additiom was prestretched before the sample was placed. The stretch was approximately equal to the expected strain during compaction. This appears to have

virtually eliminated the wrinkling during compaction.

The vertical stress required to move the sample was measured in some of the tests. The data are tabulated in Table.9.3. Although not negligible, the coefficients of friction for the compacted samples with greased rubber linings are generally of the same order of magnitude as that in test SP06 where the sample was consolidated from a slurry in a greased oedometer without the lining.

9.4.4.6 Repeatability

A source of confidence in the oedometer and the measurements made with it was sought by performing two identical stages in two separate tests. Fig.9.28 shows the data from the compaction and first swelling stages from tests SP03 and SP04. There are some relatively small differences between the stresses and void ratio changes, but these are not considered to be excessive. Slight variations in the sample placing could easily have caused differences in the compacted samples which could be responsible for the differences in test results. Closer agreement could be expected for reconstituted samples, but the compaction process is not likely to provide as consistent samples as is reconstituting samples from a slurry.

9.5 RADIAL DISPLACEMENT GAUGE

Measurement of volume change of a sample in a triaxial cell is normally done by measuring the flow of water into or out of the sample. This is acceptable for saturated samples, but for unsaturated samples the compressibility of the air renders this method inaccurate. Zero radial strain has been achieved at Imperial College using a mercury filled inner perspex cylinder (Bishop & Henkel 1962). Displacements have been measured optically through the perspex cell wall, but the accuracy of this method will be adversely affected by optical imperfections of the perspex cylinder. Various designs of gauge have been used for zero radial strain testing, but none have been used for measuring large changes in diameter of the sample. Volume change has been measured by observing the flow into or out of the cell. Errors due to creep of the perspex have been overcome by using a steel cylinder (Campanella & Vaid 1972).

9.5.1 DESIGN OF THE RADIAL DISPLACEMENT GAUGE

From the outset it was decided to attempt to avoid the use of hinges or pivot points as these are sources of error due to friction and machining tolerances. It was also desirable to construct a balanced gauge that would not require counterweights to maintain it in the horizontal plane. The gauge must travel with

the sample, otherwise it will record any variation in initial diameter along the sample as a radial straining during the test. This is most easily achieved by mounting the gauge directly onto the sample.

The most recent zero radial strain device in use at Imperial College is described by Maswoswe (1985), and is shown in Fig.9.29 This utilised a displacement transducer mounted normal to the sample surface in a frame which was in contact with the sample diametrically opposite to the transducer. This was succesful for its purpose but ultimately failed due to water ingress. This was apparently caused by leakage where the transducer had been shortened in order to fit inside the cell. In part due to this size problem, and in part due to the very limited range of the transducer, it was decided to follow the concepts used by Hight (1983), and develop a gauge based on a strain gauged web working in bending which was capable of measuring displacements up to 10% of the initial sample diameter.

9.5.1.1 Design of the web

Hight's gauge used a single web (and was thus unbalanced) in simple bending, as shown in Fig.9.30. In order to obtain a balanced gauge with no hinges, two webs are used, working as fixed end beams, as shown in Fig.9.31. Displacement of the two mounting points in an outward direction causes symmetrical bending of the two webs. Strain gauges must be placed away from the centre of the webs to avoid the point of contraflecture. Two gauges are used on each web to form a full bridge acting in compression and tension.

Analysis of the bending of the webs showed that the thickness should not be greater than 0.05mm for the desired range of 4mm displacement from the neutral position, and a web length of 18mm. Longer web lengths lead to large overall gauge dimensions which are undesirable due to the limited space within the triaxial cell. The final layout of the web and the strain gauges can be seen in Plate.9.2. Salient dimensions are given in Fig.9.32.

9.5.1.2 Frame design

The frame needs to be both light and fairly rigid. Brass mounting bosses were used with stainless steel tubing connectors. Joints were formed using Araldite or Loctite. Electrical conductors were routed through plastic tubing which fits closely within the stainless steel tubing. This was sealed at the strain gauge end with Araldite. One set of conductors is required for each web, and these were led out of the triaxial cell in separate plastic tubes to avoid attempting to make a waterproof joint to operate within the cell at high water pressures.

9.5.1.3 Waterproofing

Perhaps the greatest problem was waterproofing the webs. Use of a plain tube, of whatever material, filled with oil would have led to a very stiff gauge. Bellows type boots were required to obtain a flexible gauge while providing a thick water resisting barrier. Brass bellows are used as flexible drive couplings, but are both too stiff and too bulky for use on this gauge. Flexible drinking straws of suitable diameter were obtained, but not used due to difficulties in glueing polyethylene. PVC bellows as used on Lambretta scooter clutch cables proved to be adequate. Connection to the brass bosses was achieved using both vinyl glue and O-rings fitted over the boot in much the same way as the sample membrane is sealed onto the pedestal.

9.5.1.4 Construction

Due to the small sizes of the brass bosses that were required to keep the mass of the gauge low, the components could not be made on the machine tools available in the section workshop. Fabrication was thus done by the author, and the prototype gauge is somewhat home-made. Some of the machining operations would have been best performed on a milling machine, but had to be done on a lathe. As a result the orientation of the holes in the bosses is not perfectly orthogonal, which has caused undesirable characteristics in the calibration.

9.5.1.5 Oil filling

In order to prevent distortion of the boots when under pressure in the triaxial cell, they must be filled with an incompressible fluid. The same approach is used in the Skinner load cells, (Bishop et al 1974). Filling of a small soft bellows is slightly more difficult than filling a large steel chamber. The solution chosen was to remove the air by placing the assembled gauge in a vacuum tank and then allowing the oil to flow in as the vacuum was released. A small reservoir was made from a brass union and fitted to the filler tube as shown in Fig.9.33.

This was filled with oil before the gauge was placed in the vacuum tank. On applying the vacuum, the air is sucked out through the oil (which is deaired in the process). After several hours, when no more bubbles appear, the vacuum is slowly released, and the oil flows into the void within the boot. This process was repeated at least once to ensure complete filling. Since the reservoir has to be upright, only one boot can be filled at a time. Fitting of the blanking plug to the filler tube was done in an oil filled moisture content tin to ensure that no air was trapped in the process.

9.5.2 CALIBRATION

Calibration was commenced with the gauge bolted into a screw type calibrator. The micrometer dial is graduated in 0.01mm divisions which are about 3mm apart, permitting fairly precise setting of the dimensions. Despite this apparent precision, difficulties were encountered, and other measurements and mountings were employed.

9.5.2.1 Stability

Stability of the gauge output was tested over extended time periods and found to be adequate. Fig.9.34 shows the first stability check, in which the gauge was mounted in the calibrator frame. The range during this period was 0.005mm, with the scatter about 0.002mm. For this test the filter was 100, i.e. 100 readings were taken and the average value used. Fig.9.35 shows the effect of increasing the filter. A marked improvement is obtained for each 10 fold increase in the number of readings. A filter of 10 000 was used in all subsequent work. Each measurement of 10 000 readings takes about 3 seconds.

An extended stability test was run over 6 days. The results are shown in Fig.9.36. For this test the gauge was glued onto a brass cylinder of 38mm diameter and placed inside the triaxial cell. Local scatter of the measuremets can be seen to be about 0.0005mm, corresponding to 1 microvolt. Less satisfactory is the variation with time of the value of the measurement, which amounts to about 0.004mm over the first 4 days, followed by a drift of about 0.002mm in the last 2 days.

The other data plotted on this Figure are the measured voltage supply to the measurement system. This shows a variation of about 0.003volts, which in some cases is mirrored by a change in the gauge output. This was investigated further by replacing the gauge by an open circuit and by a shorted circuit.

The former case is shown in Fig.9.37a, where the gauge was simply unplugged and readings taken. The apparent fluctuation in the measurement is about 0.005mm. The second case is shown in Fig.9.37b, where the signal pins of the socket, into which the gauge was usually plugged, were connected with a short length of wire and the power pins were left unconnected. Again the apparent variation of the measurement is about 0.005mm.

Thus, regardless of the quality of the gauge, the absolute precision of the measurement cannot be better than 0.005mm. This is due to the instability of the measurement system, which could not be altered. It appears that the home-made prototype gauge has outperformed the factory produced measurement system.

9.5.2.2 Calibration

Initial calibrations were made with the gauge securely bolted into the calibrator frame. The first run indicated a useable range of about 5mm with reduced sensitivity outside of this range, as shown in Fig.9.38. A subsequent cycle of readings within the 5mm range showed an apparent large hysteresis. Extensive experimentation and recalibration demonstrated that the calibrator had sufficient play or slack in the mechanism to permit up to 0.2mm change in the gauge output for the same setting of the calibrator dial. This is obviously unsatisfactory.

The calibration technique was thus changed, and one-way calibrations were performed in the direction in which the sample swelling would occur. With the equipment available for calibration it is not possible to state whether or not the gauge does exhibit hysteresis, but it is evident that most of the apparent hysteresis was due to the calibrator frame.

The mounting system was also changed from a bolted assembly with the gauge about 25mm from the axis of the calibrator to a wire suspension on the calibrator axis. This necessitated the use of a spring to obtain measurements smaller than the neutral position of the gauge. The layout is shown in Fig.9.39.

This gave a better calibration than the bolted system, but the fit of the regression was still not good. A check was made on the calibrator using a travelling telescope with a micrometer attachment. The differences between the measurements using the calibrator and the telescope are plotted in Fig.9.40 together with a 5 point moving average. There is either a shift of about 0.03mm at about 3mm or a progressive change over the full range. Superimposed on this there appears to be a cyclic variation. An attempt was made to correct the calibrator values to account for this error, with a small improvement in the fit of the regression.

Careful inspection of the data showed an anomaly in the slope of the calibration line at about 4.5 to 5 mm, which is thought to be due to buckling or flexing of the webs due to the imperfect construction mentioned in 9.5.1.4. It was found that the calibration could be changed by adjusting the orientation of the mountings in the clamped joints at the ends of the webs. Fig.9.41 shows two calibration curves obtained in this way.

The best calibration obtained is shown in Fig.9.42 where the maximum difference between the measured and the calculated values is 0.015mm. In view of the comments above regarding the precision of the calibrator, it not clear whether these apparent errors are real. The error in radial strain that results from the apparent error in the gauge of 0.015mm is 0.037% strain. The expected strain during swelling to zero suction for the planned tests was 2 to 4%, against which the 0.037% is about 1% error.

Failure of the gauge occurred before it was put into service due to two wires breaking at the soldered joints on the strain gauges. These two joints appeared to have been poorly made, but even so had survived rough handling at some stages during the

commissioning work. Recalibration was necessary, and the final calibration is shown in Fig.9.43.

During testing of the gauge, it was placed in the triaxial cell under pressure while glued onto a brass dummy sample. The cell pressure was varied and the output monitored. The results are shown in Fig.9.44, and indicate a response of about 0.002mm for 400kpa stress change. This is less than the precision of the measuring system.

Layout	Lubricant	Peak Resistance
3 O-ring	silicone grease	3.3kg
3 O-ring	Molykote 111	4.0kg
3 O-ring	Molykote Longterm 2+	2.8kg
3 O-ring PTFE	nil	Not determined
2 O-ring	Molykote Longterm 2+	2.5kg
2 O-ring PTFE	nil	0.9kg

Table 9.1 Frictional Resistance of plunger seals

Drift on Arm No.	Stress Resistors	Configuration		
		Normal	1st Null	2nd Null
None	No	-3180	11800	4790
	Yes	140	11680	4900
A1	No	-1850	13140	6260
	Yes	1460	13080	6290
A2	No	-4590	10460	6150
	Yes	-1750	10420	6150
D1	No	-1850	10480	3450
	Yes	1500	10430	3530
D2	No	-4480	13160	3500
	Yes	-1210	13100	3580

Table 9.2 Output from "mock-up" Bridge

Test No.	Radial Stress kPa	Vertical Stress Kpa	Coefficient of Friction
SP01 start	87	11	0.09
SP03 start	84	30	0.25
SP04 start	94	15	0.11
end	63	30	0.34
SP06 end	104	30	0.19

Table 9.3 Resistance to sample movement

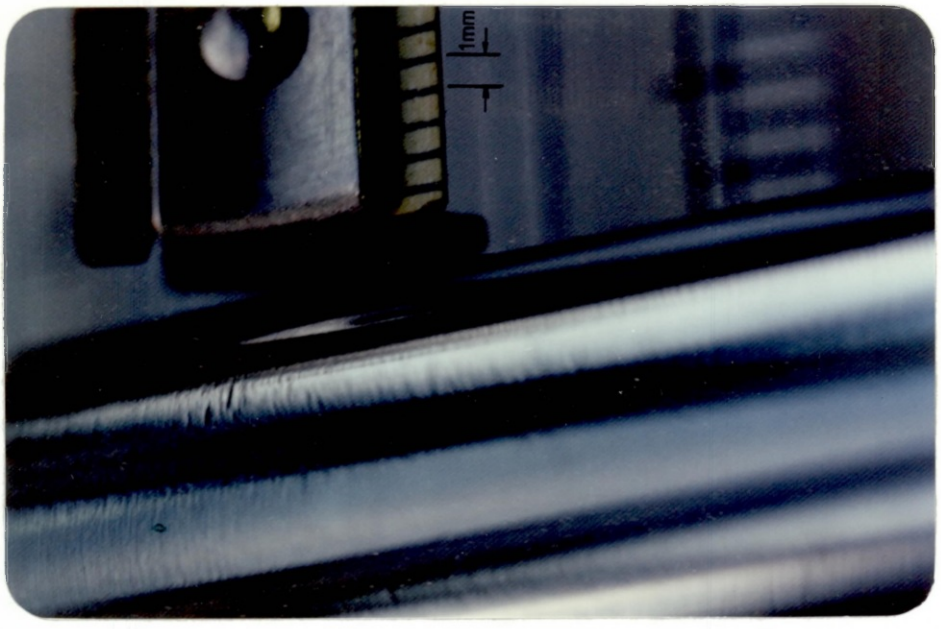


Plate 9.1 Roughness of plunger

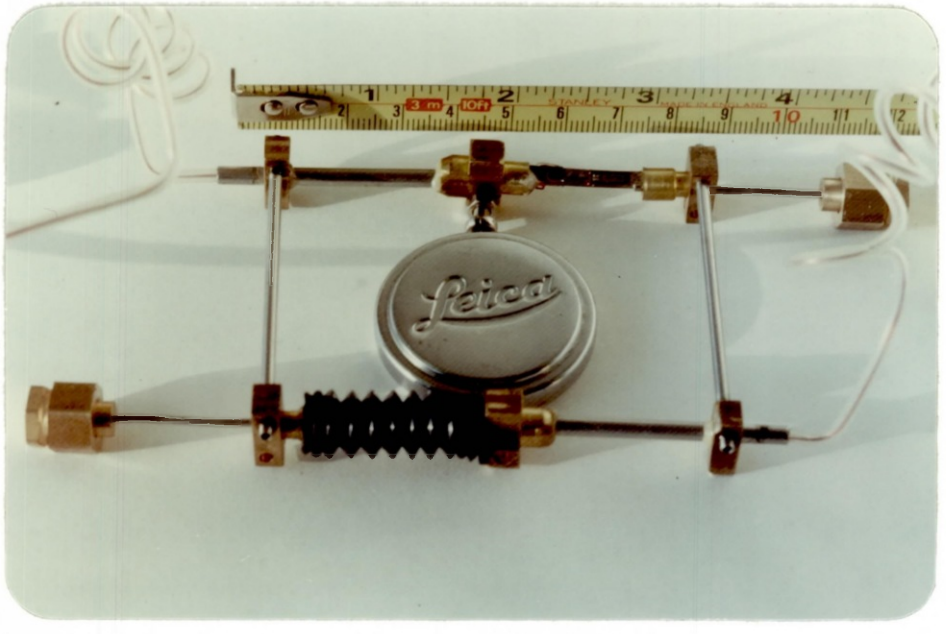


Plate 9.2 Radial Displacement Gauge

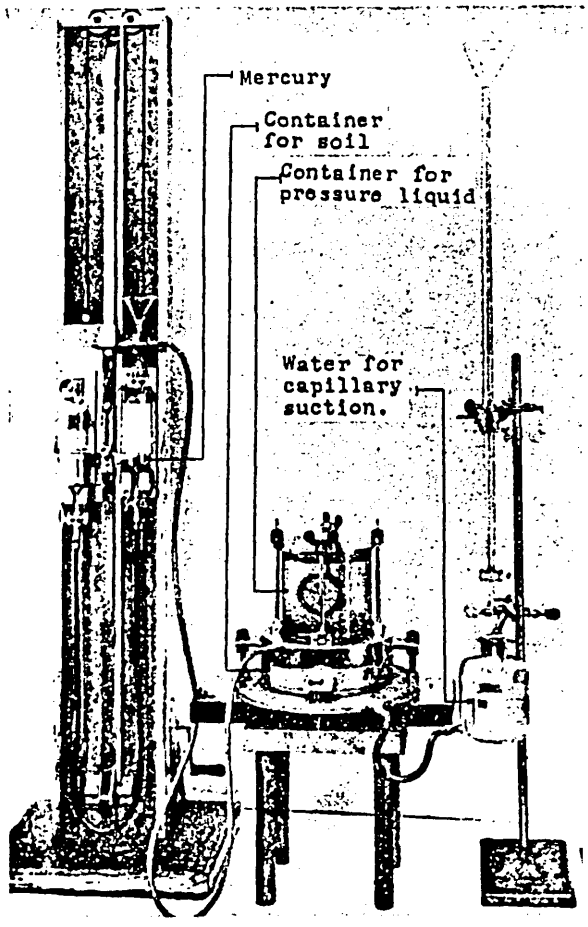


Fig.9.1 Test Apparatus for Swelling and Elasticity
Appareil pour mesurer le gonflement et l'élasticité

(Rengmark & Eriksson 1953)

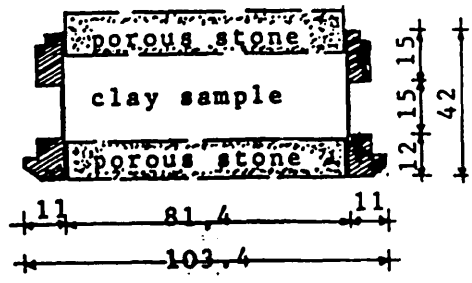
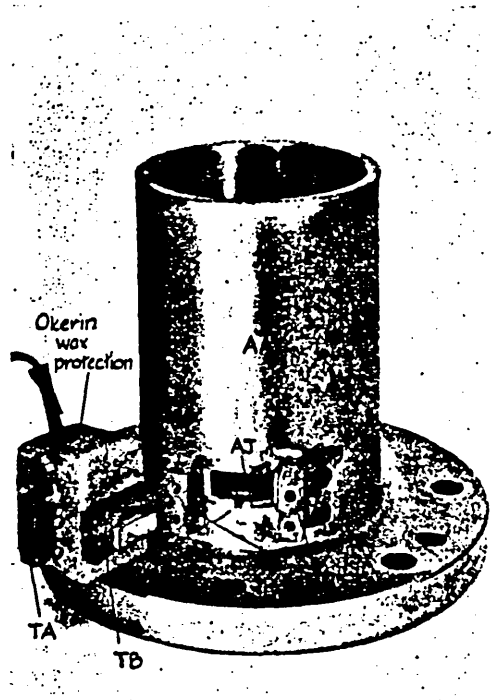


Fig.9.2 Lateral Swelling Pressure Ring

(Ofer 1980)



Pressure cell removed to show front beam

Fig.9.3 Lateral pressure measuring cell. Cell removed from cylinder to show front beam.

(Thompson 1963)

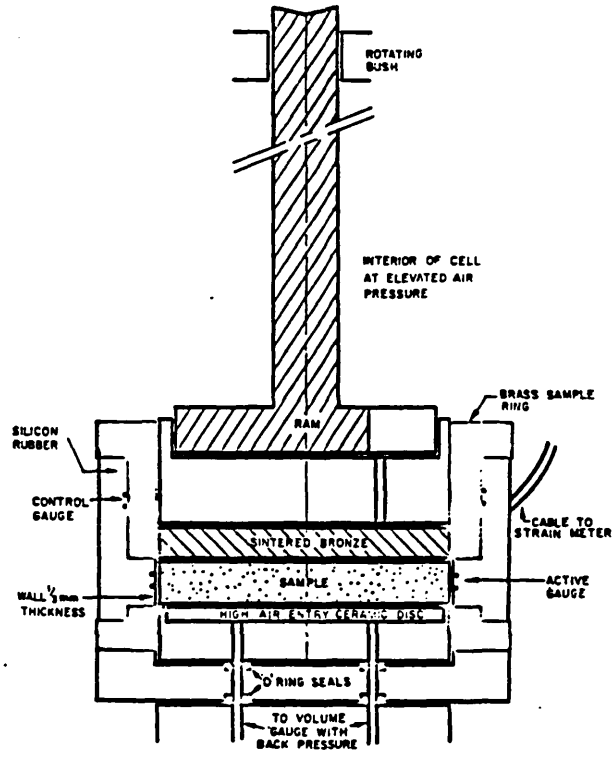


Fig.9.4 *Modified triaxial apparatus*
(Brackley 1975)

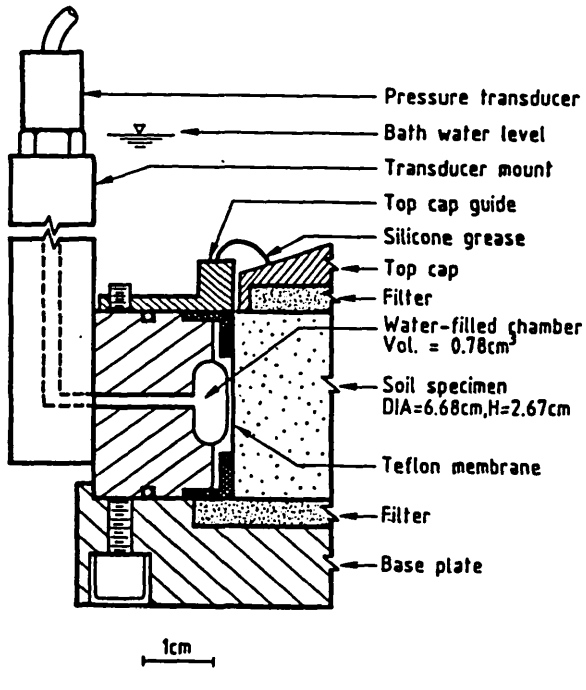
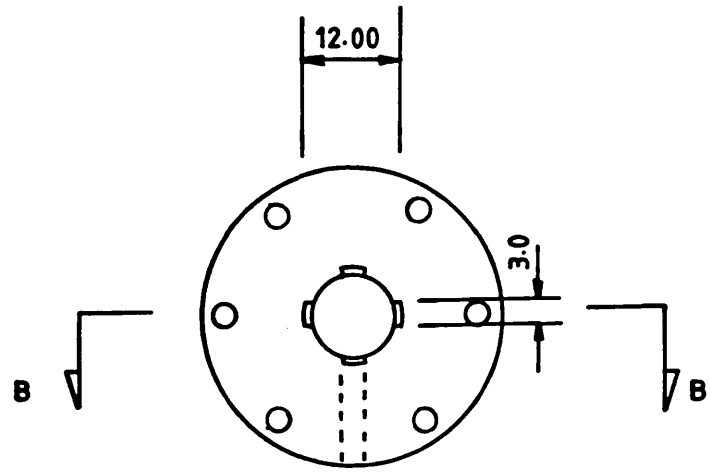
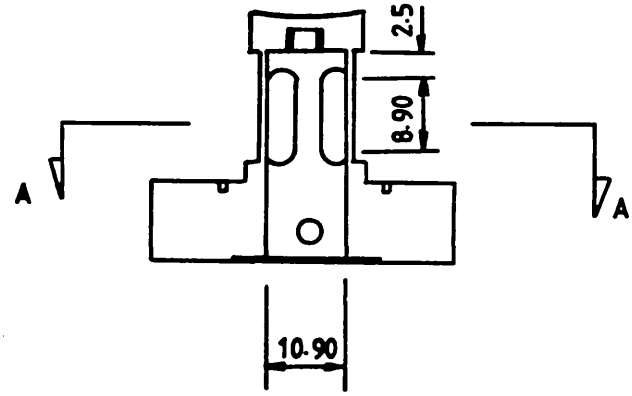


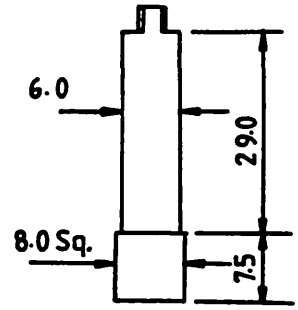
Fig.9.5 Simplified cross-section of the K_o-oedometer
(Dyvik et al 1985)



SECTION A-A



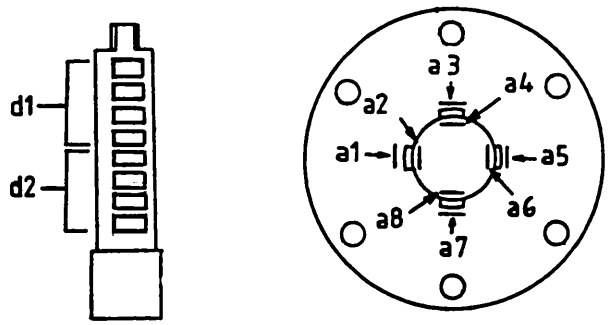
SECTION B-B



dimensions in mm

Dummy Gauge Pillar

Fig.9.6 Load Cell



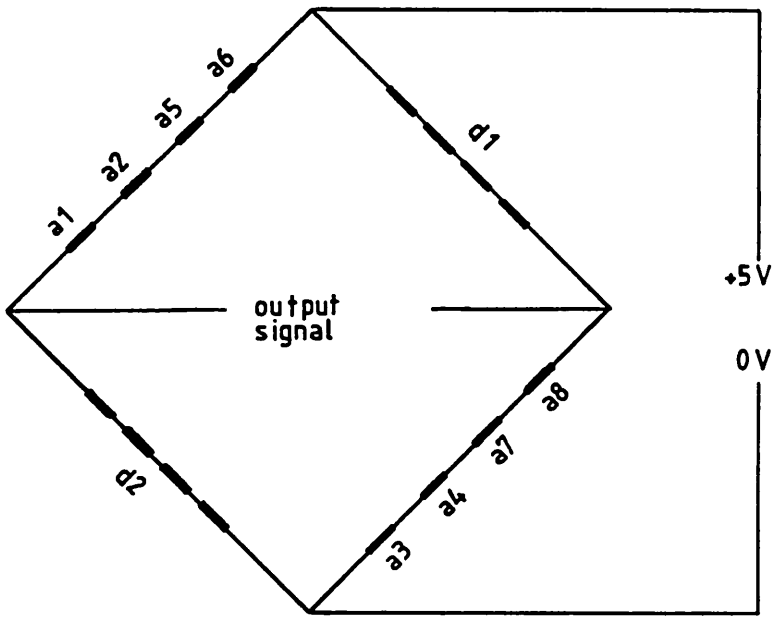


Fig.9.7 Electrical Layout of Load Cell

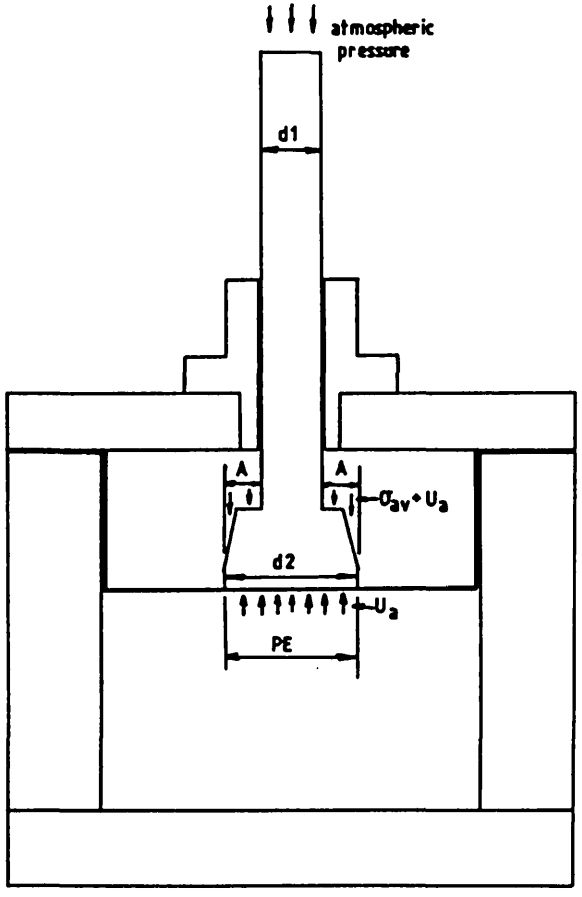
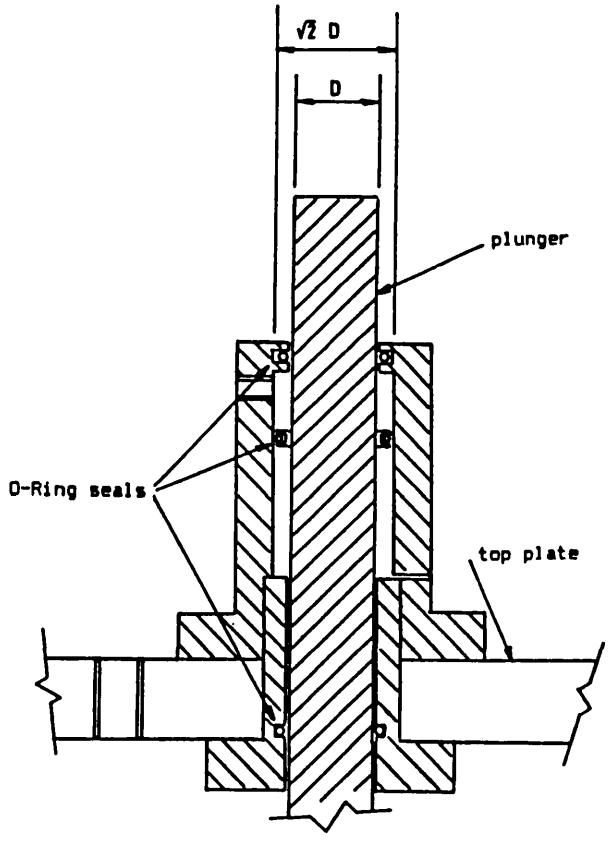


Fig.9.8 Pressure Balance on Plunger



Fig,9.9 Compensated Plunger System

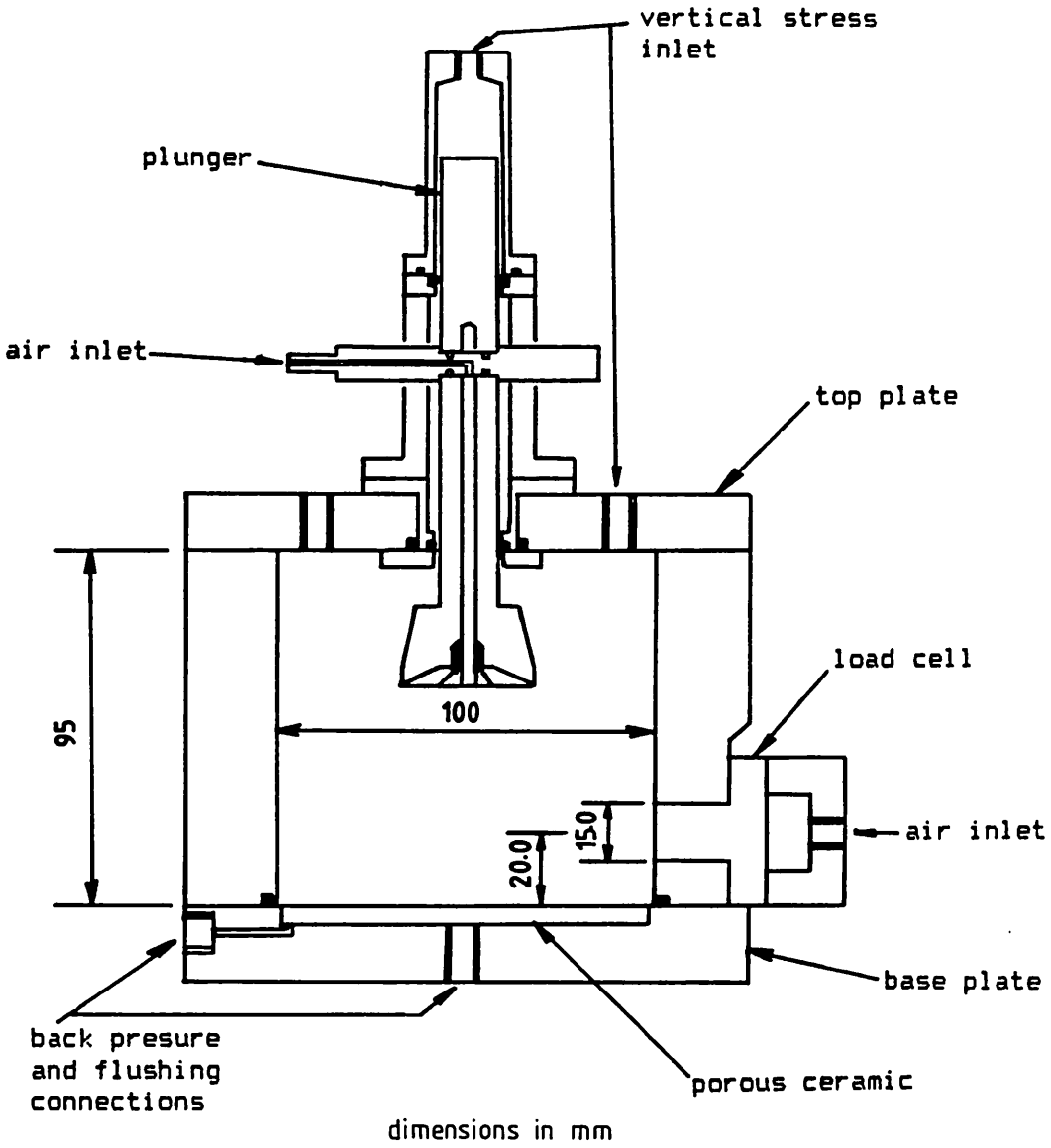
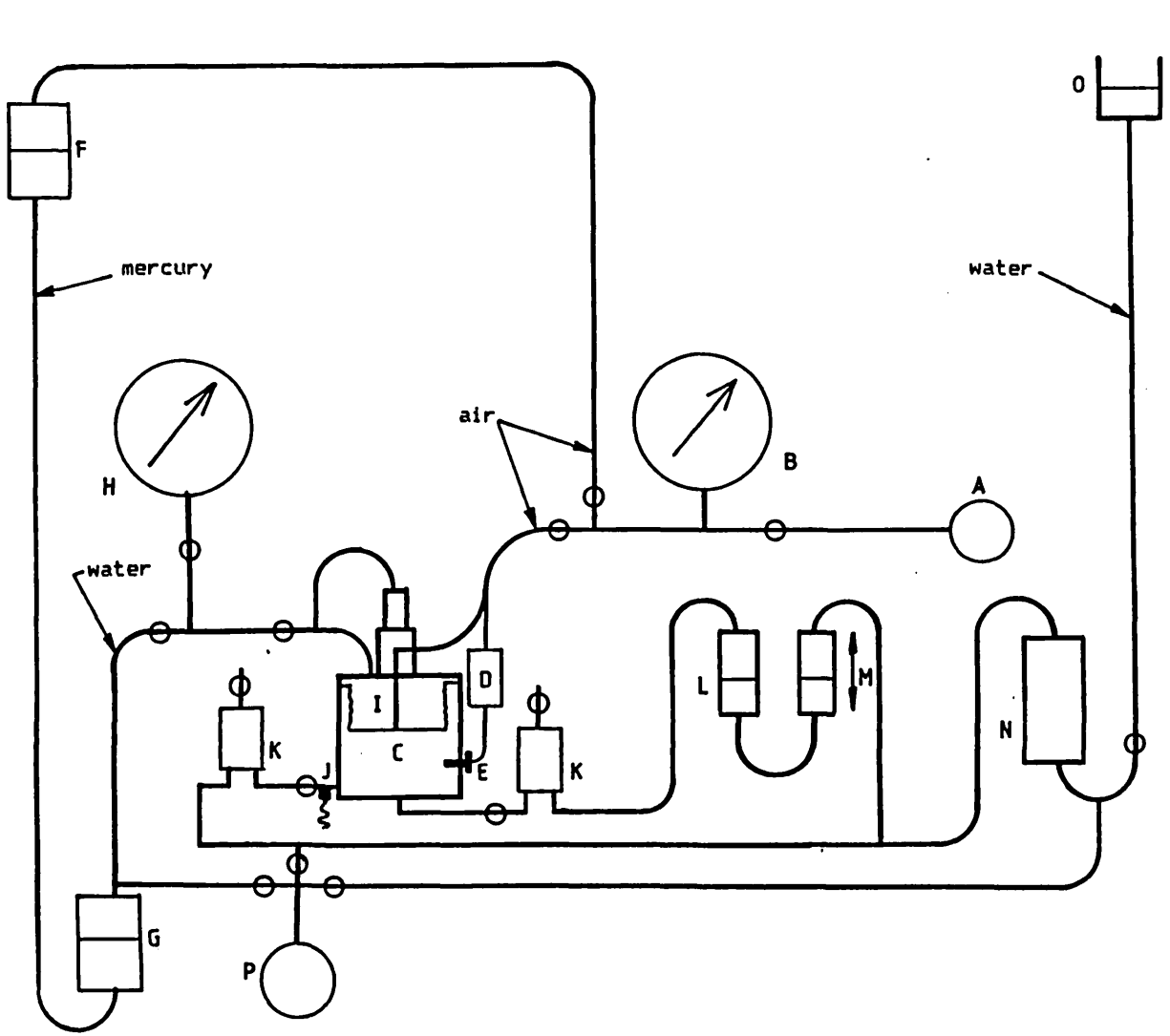


Fig.9.10 Oedometer Details, showing final version of compensated plunger



Key

A	High pressure air regulator
B	High range pressure gauge
C	Sample chamber in Oedometer
D	Silica gel container
E	Radial stress load cell
F	Top mercury pot
G	Lower mercury pot
H	Low range pressure gauge
I	Vertical stress chamber
J	Pore water pressure transducer
K	Bubble traps
L	Fixed mercury pot for flushing system
M	Moveable mercury pot
N	Bellofram volume gauge
O	Back pressure pot
P	Screw pump
⊖	Tap

Fig.9.11 Layout of Stress Path Oedometer System

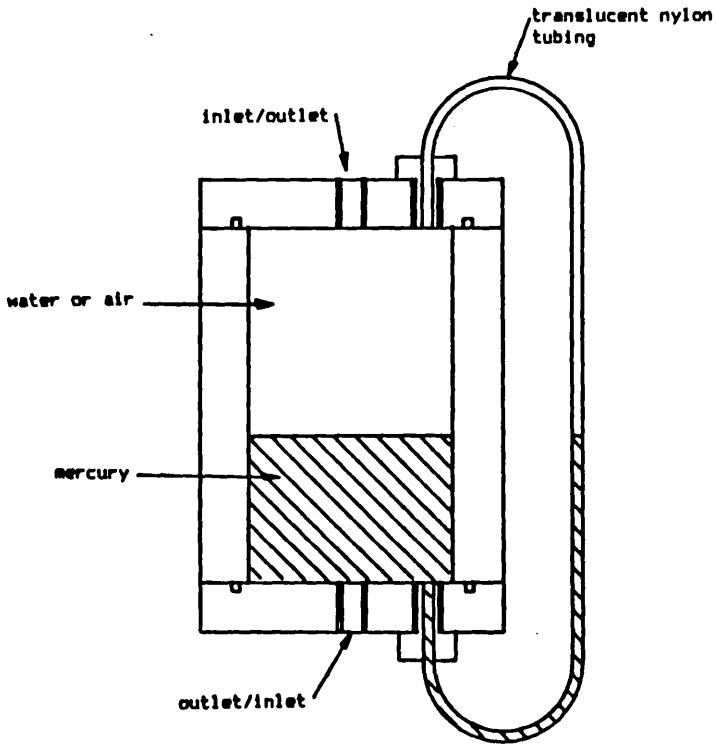


Fig.9.12 Mercury Level Indicator

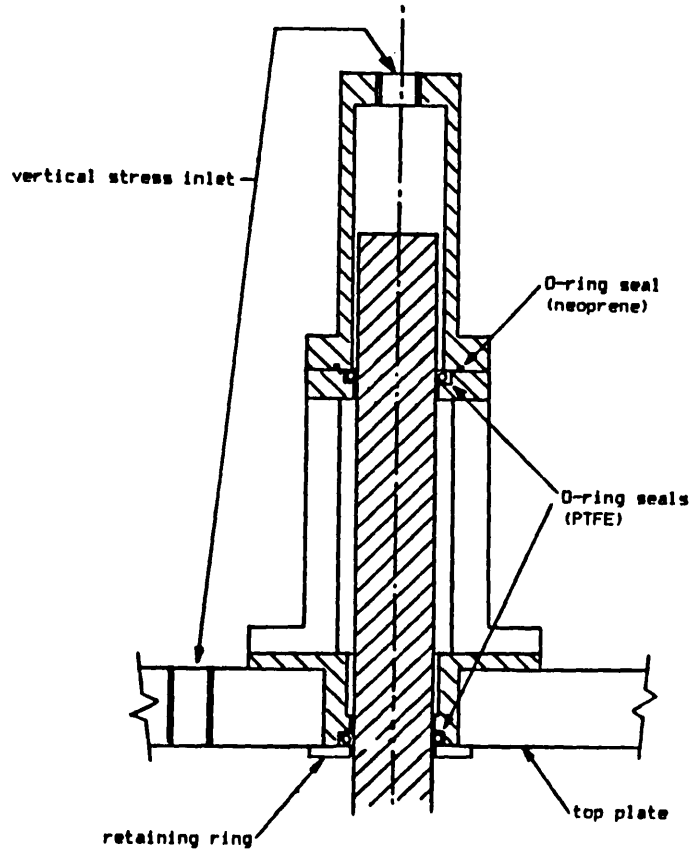
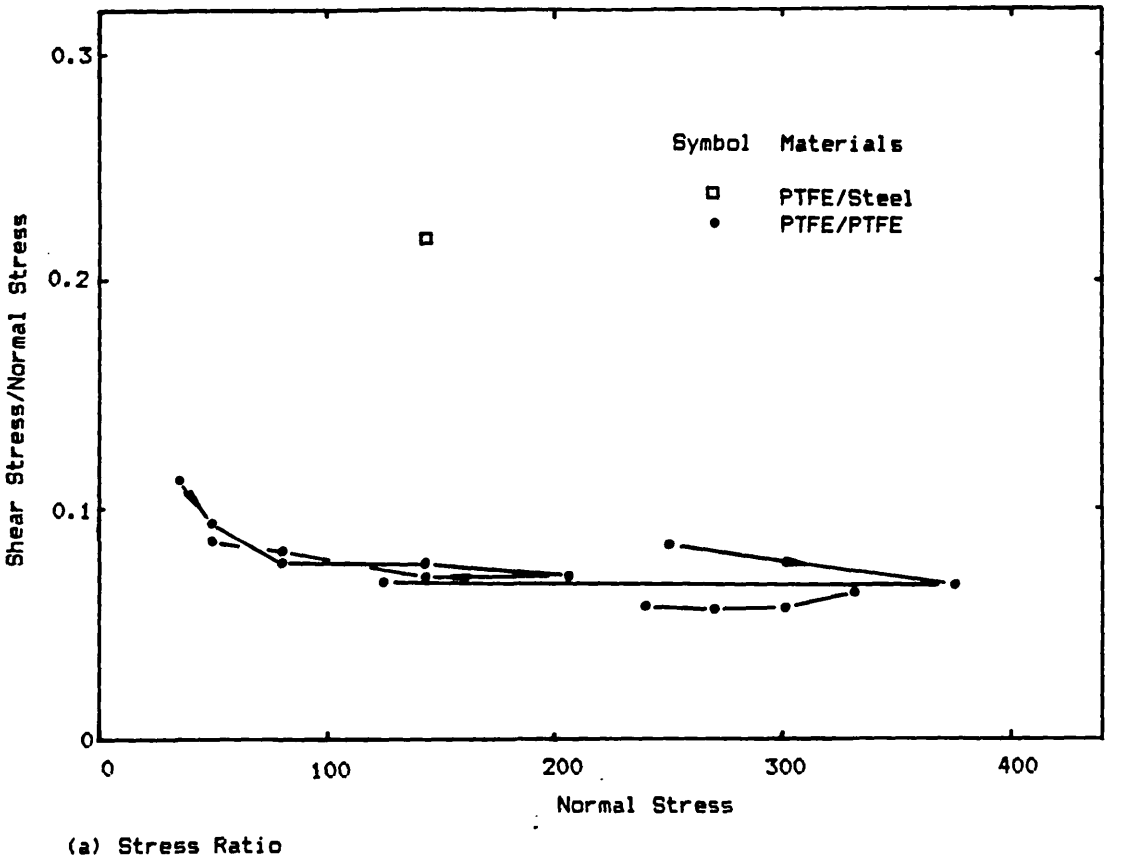
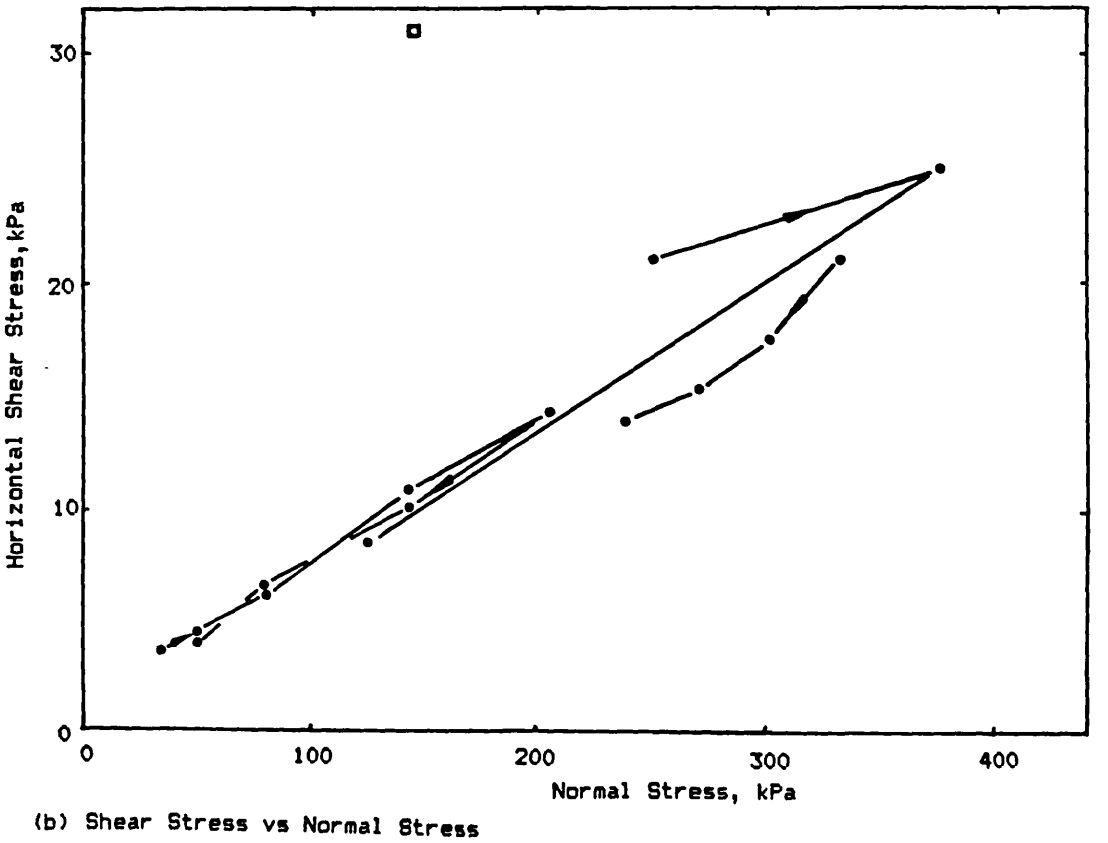


Fig9.13 Improved Compensated Plunger



(a) Stress Ratio



(b) Shear Stress vs Normal Stress

Fig.9.14 Friction Tests on PTFE

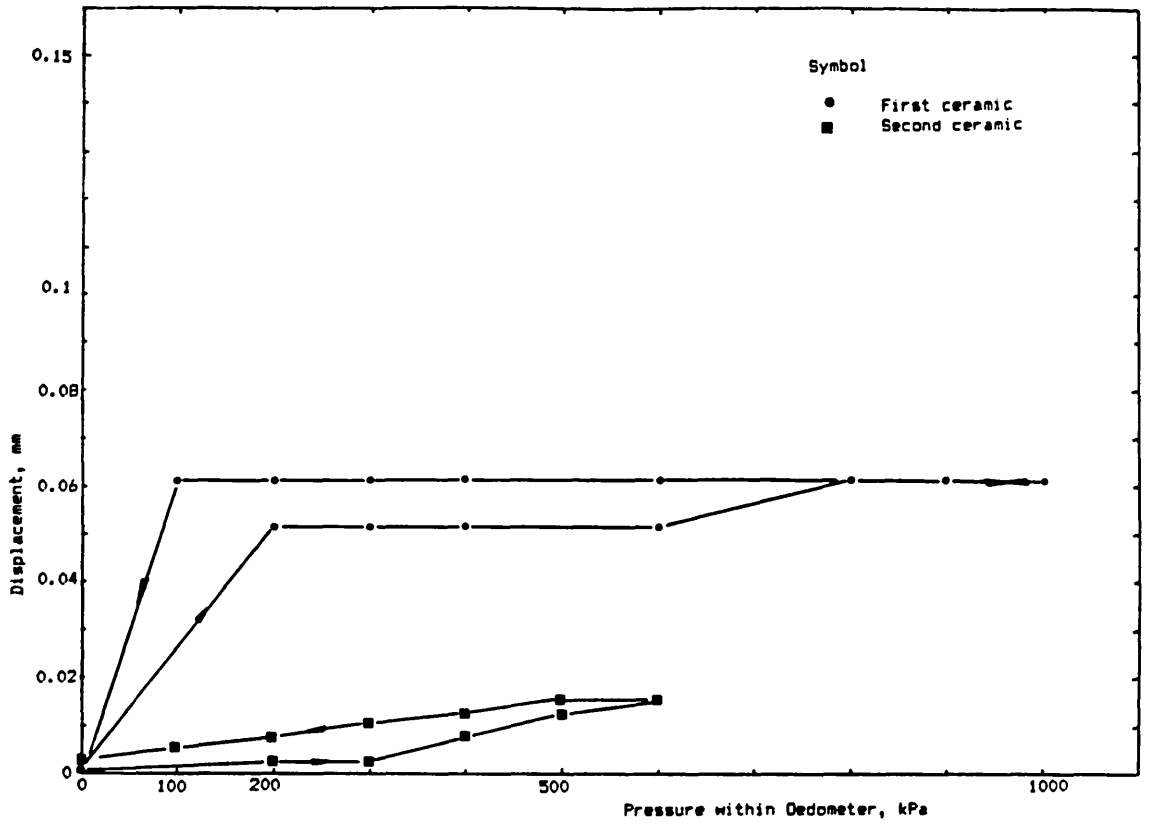


Fig.9.15 Apparent Displacement in Stress Path Oedometer due to Internal Stress

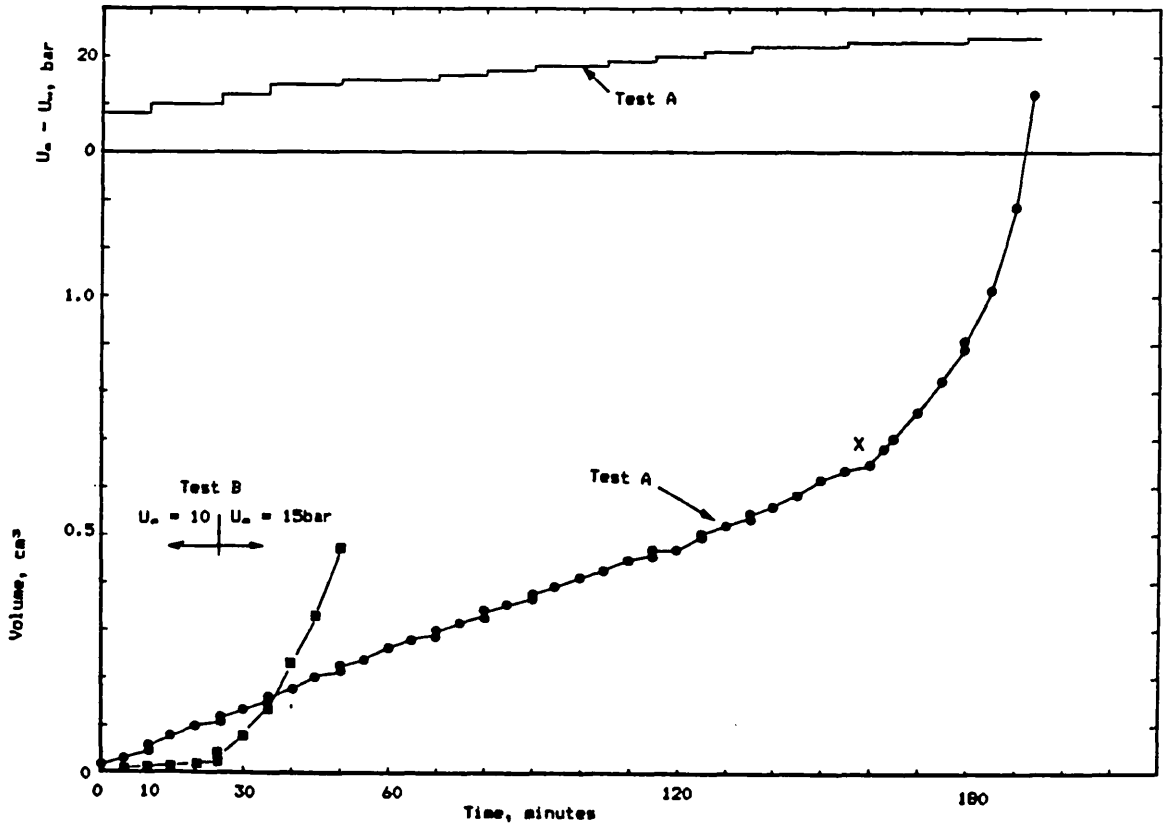


Fig.9.16 Drainage through ceramic under appied air pressure

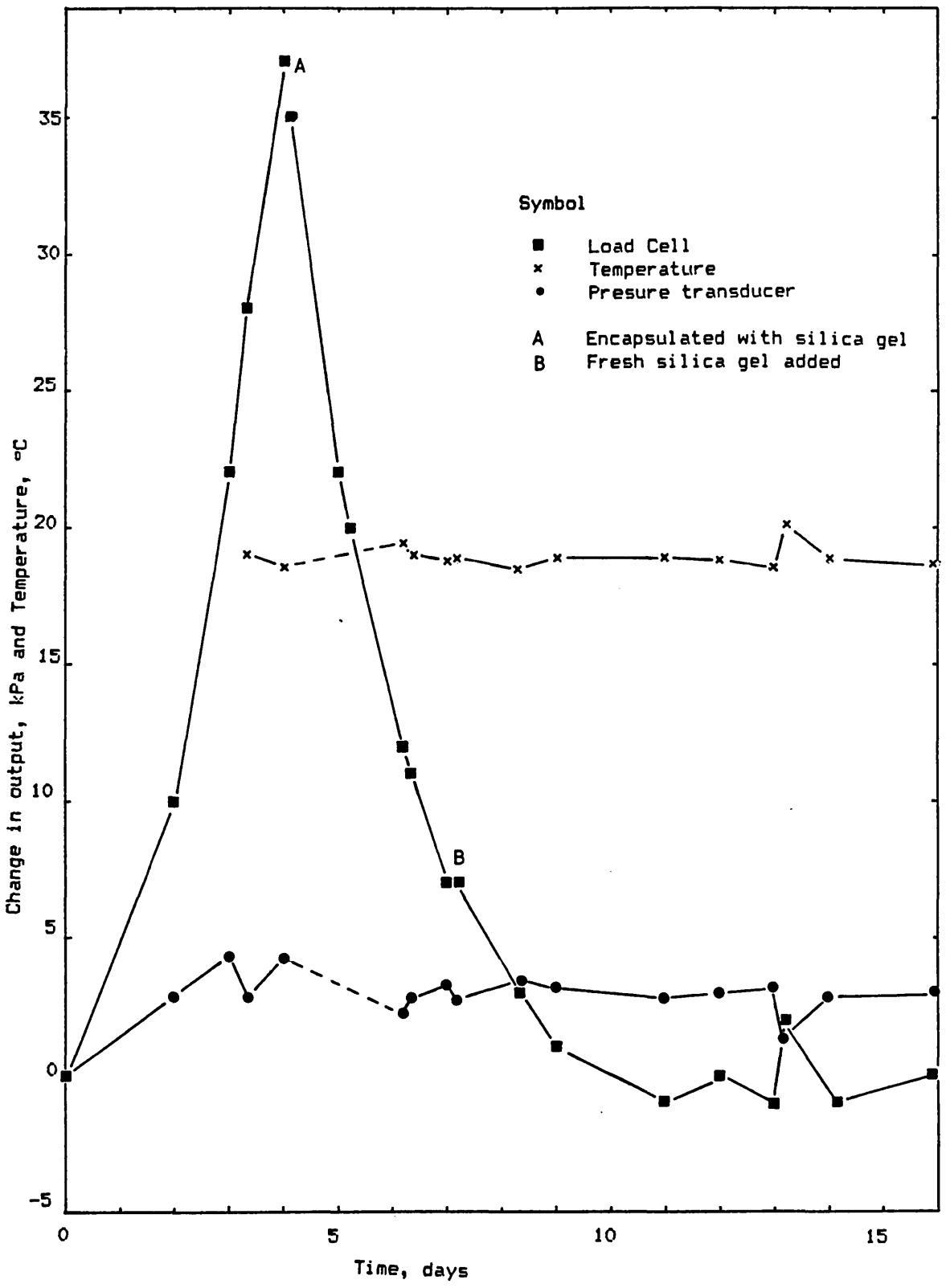


Fig.9.17 Effect of Moisture on Load Cell Output

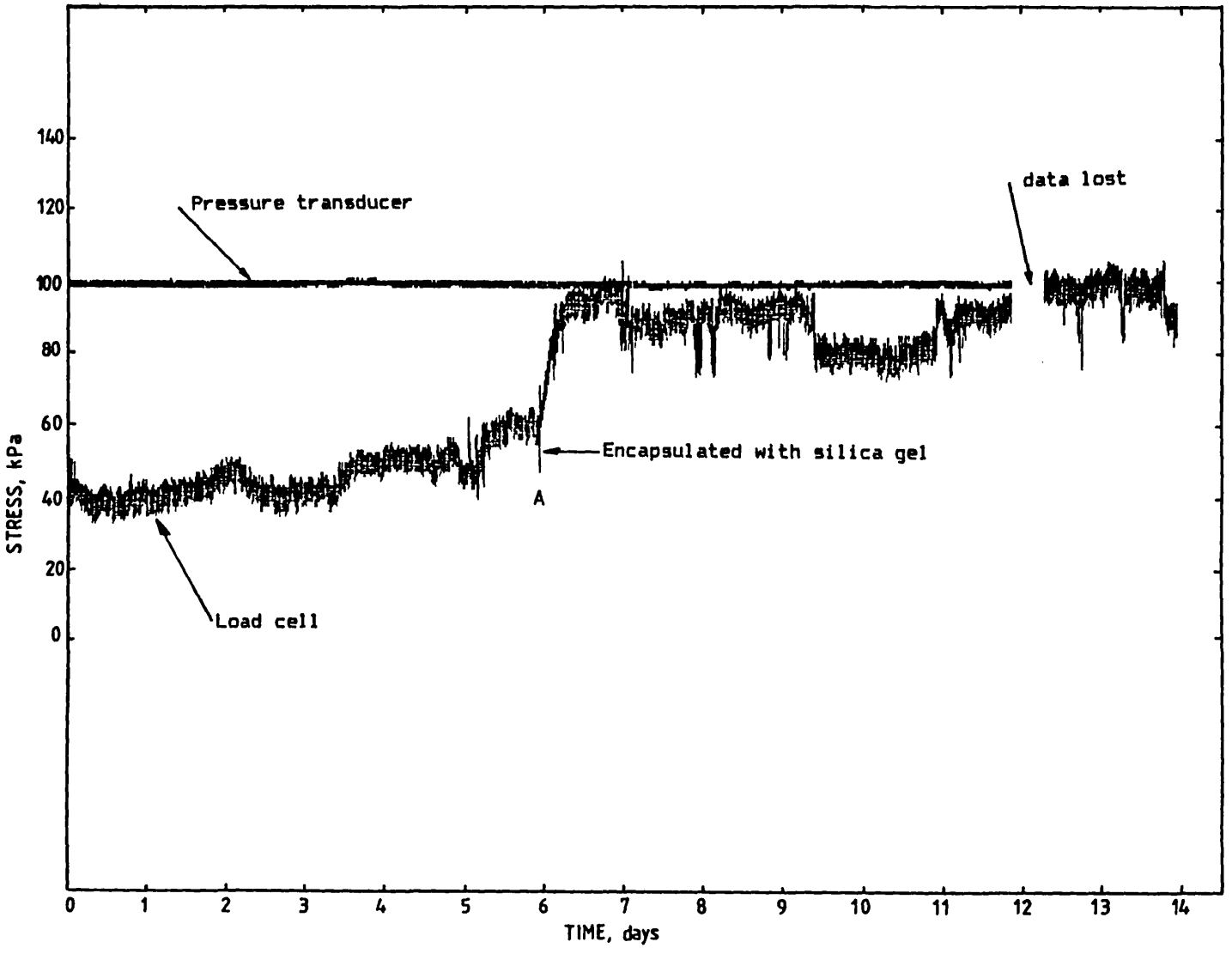


Fig.9.18 Long Term Stability Test of Load Cell

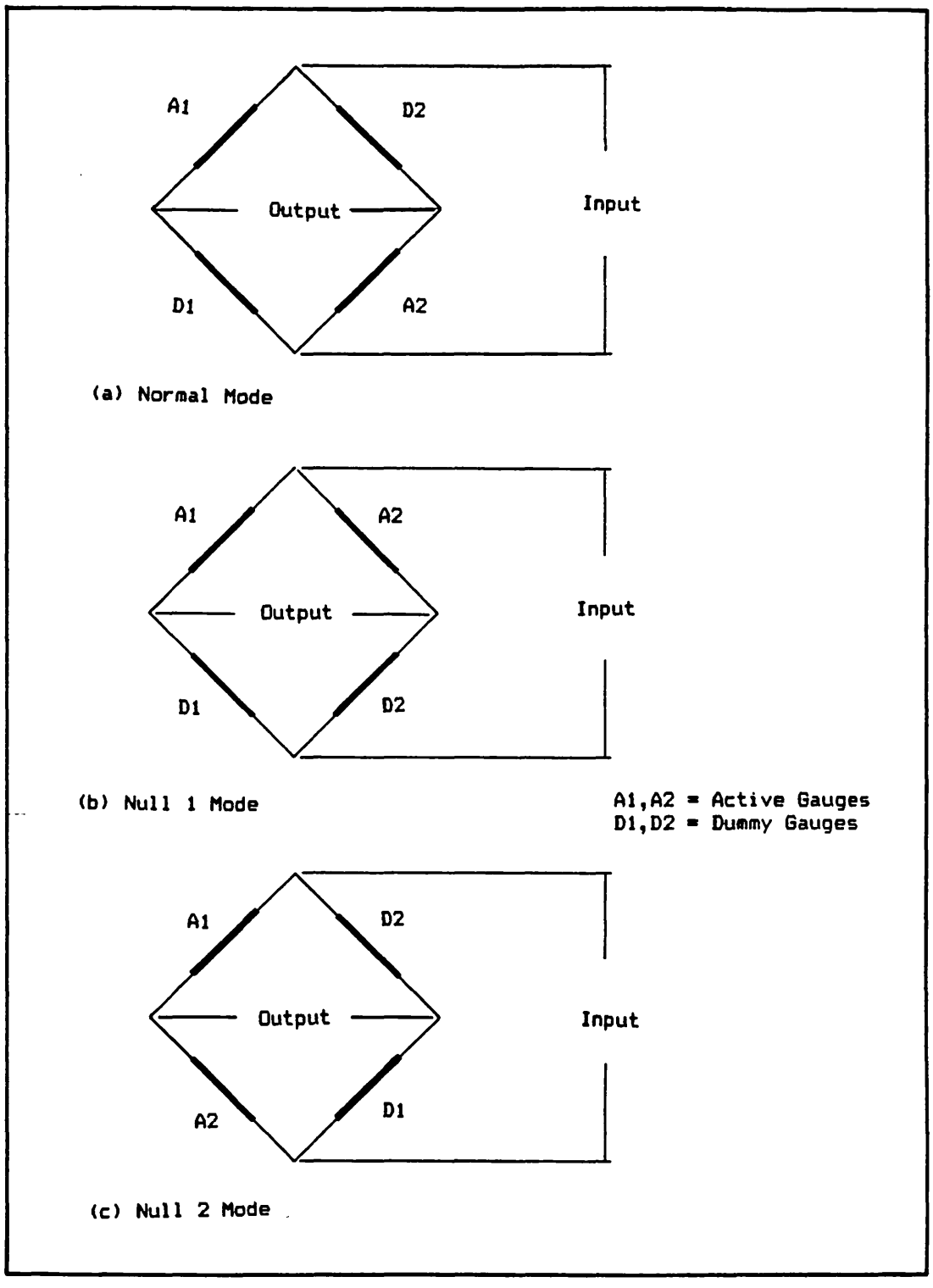


Fig.9.19 Load Cell Configurations

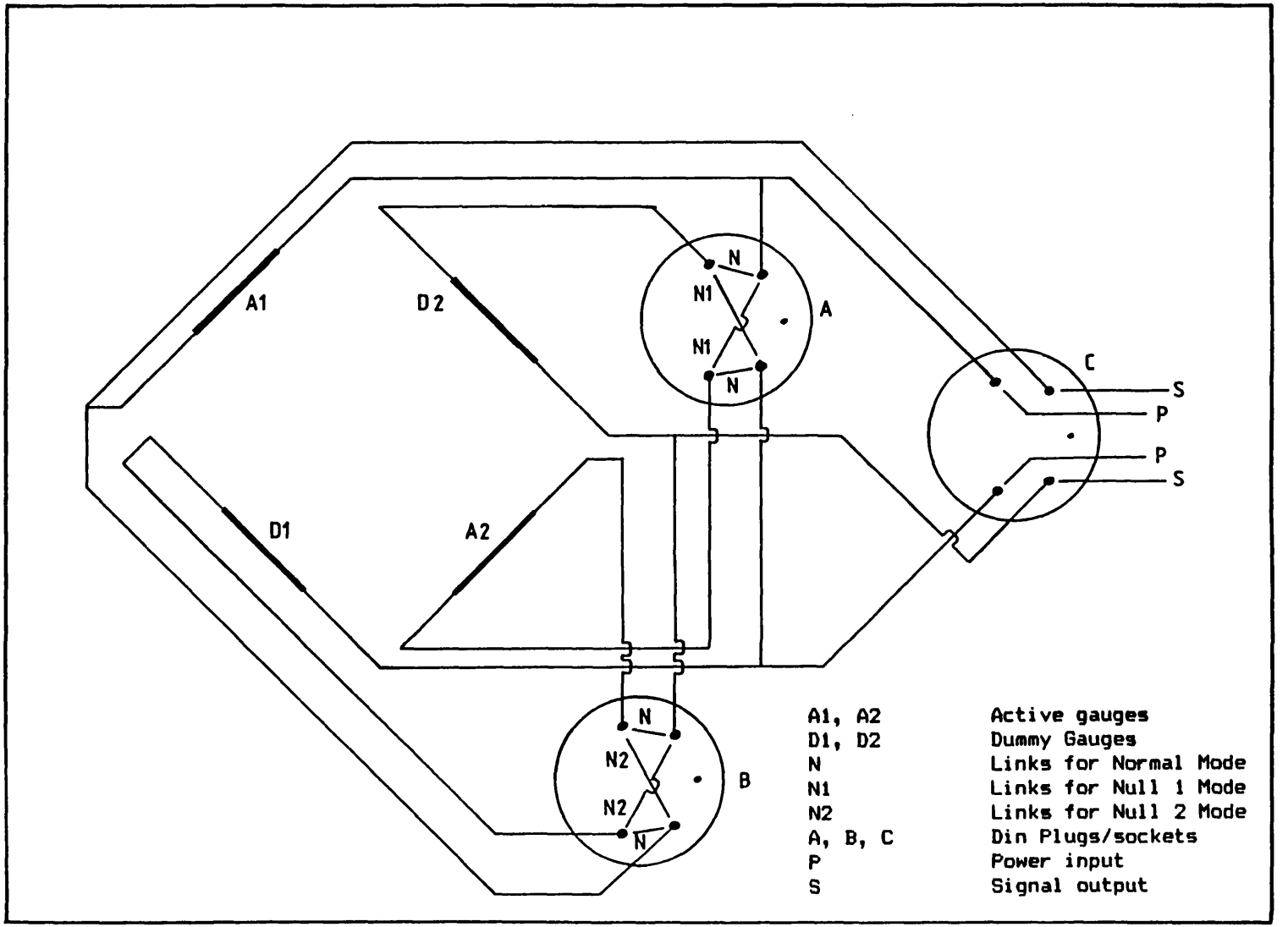


Fig. 9.20 Wiring Diagram

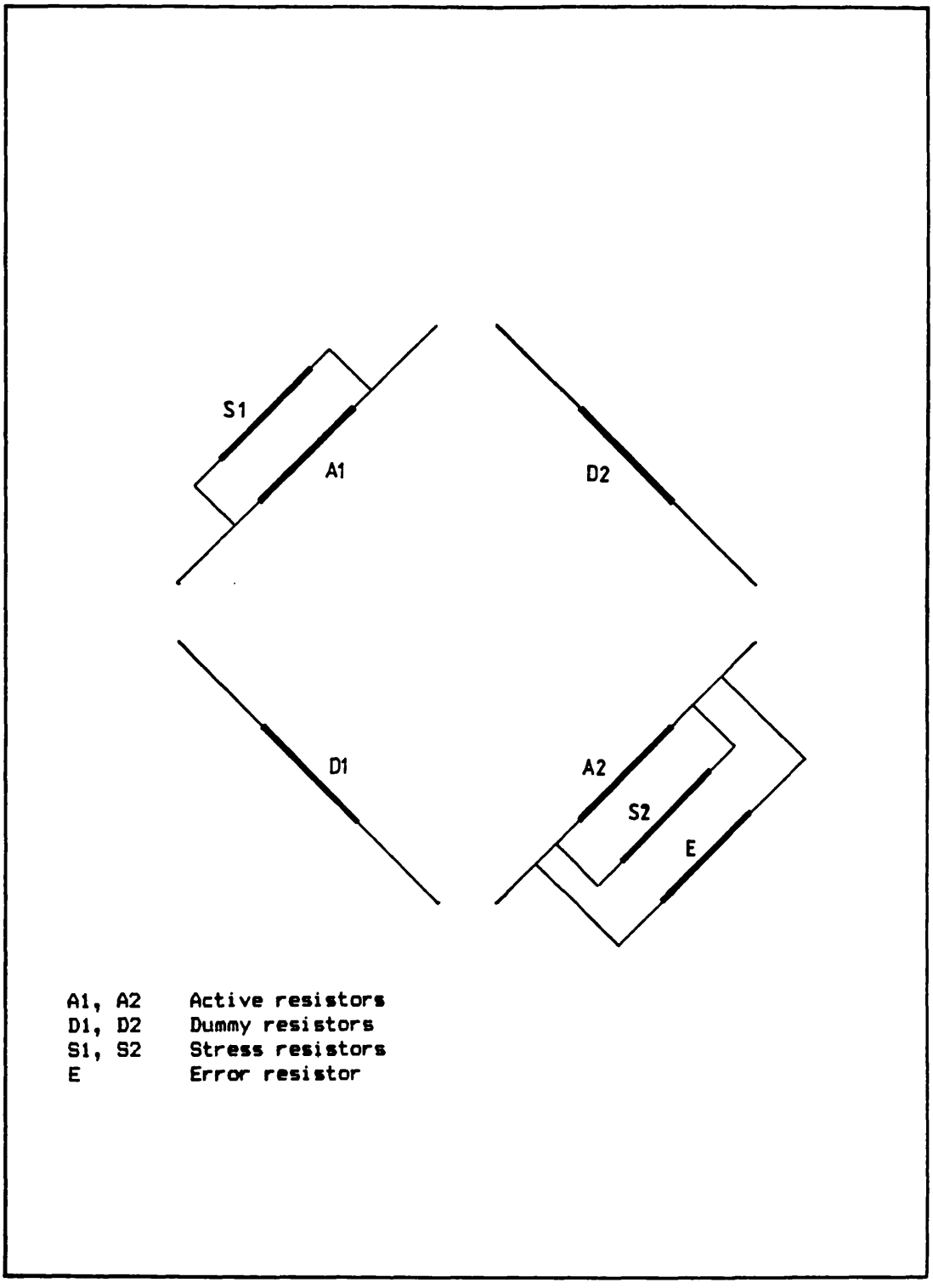


Fig.9.21 Layout of Mock-up

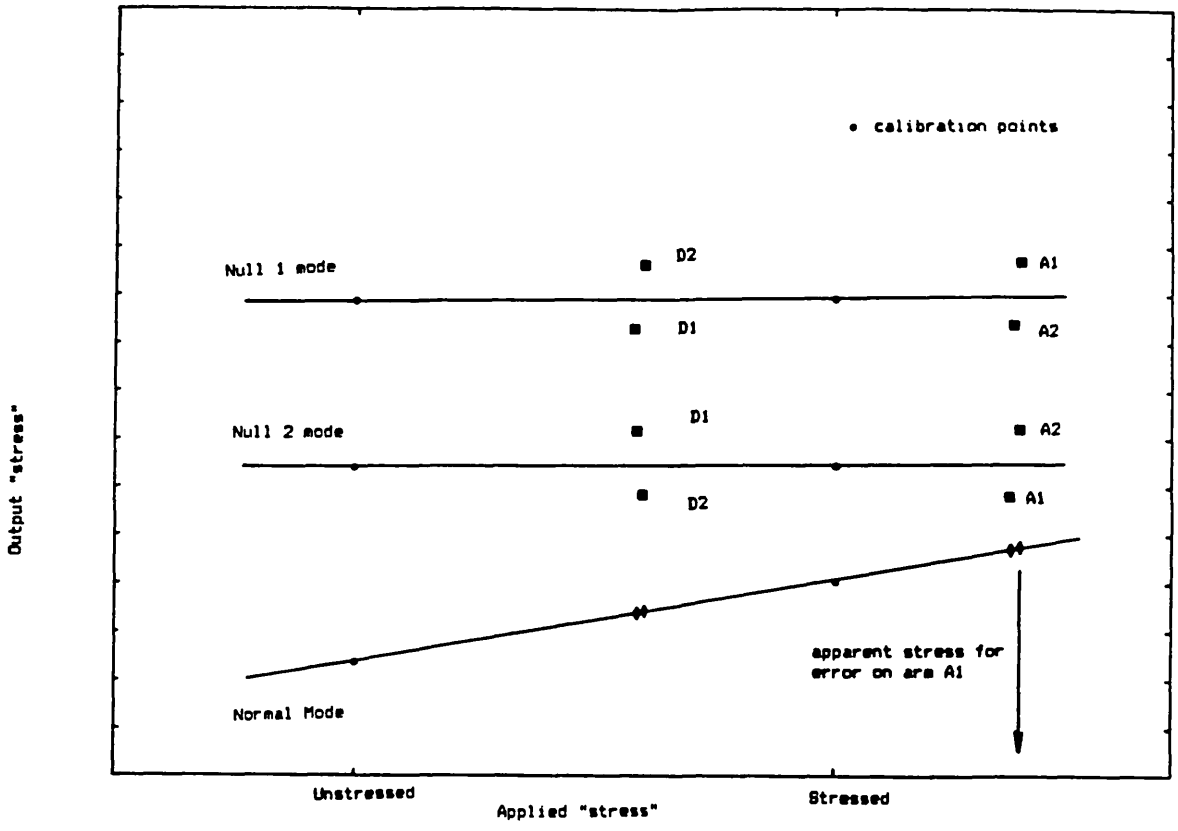


Fig.9.22 Calibration of Mock-up

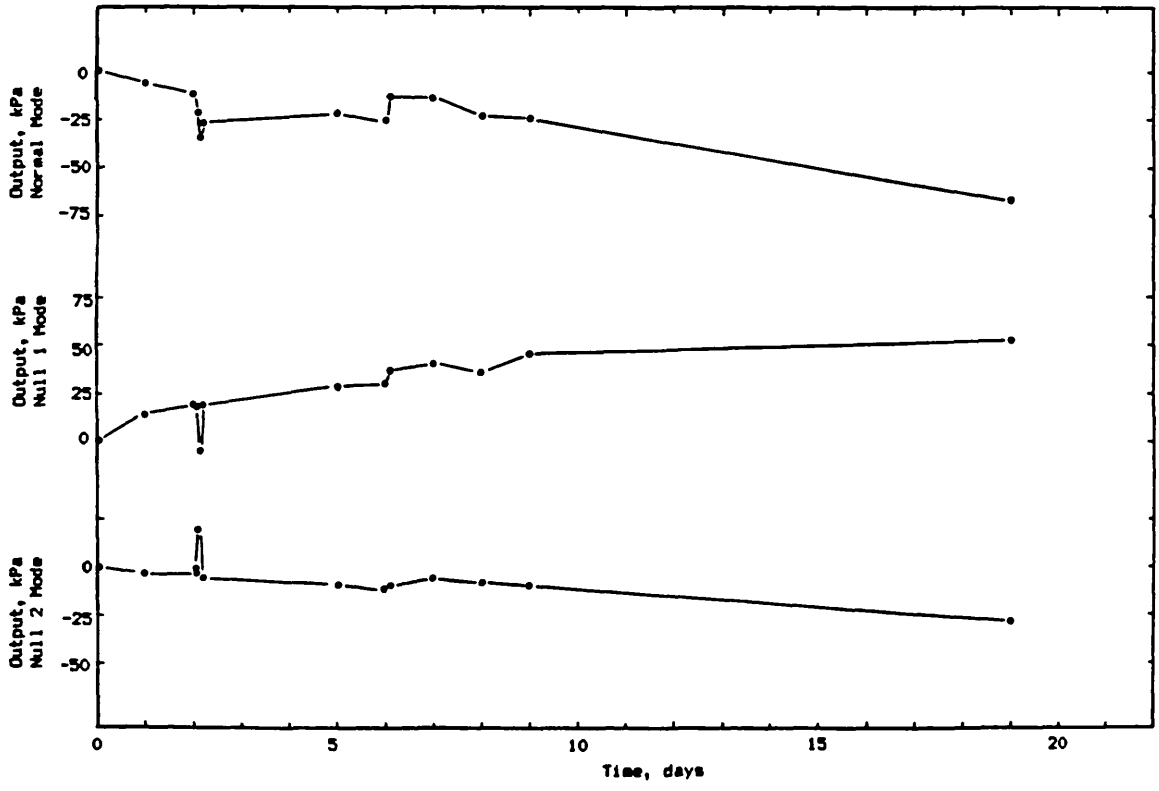


Fig.9.23 Observed Drift on Load Cell LC2

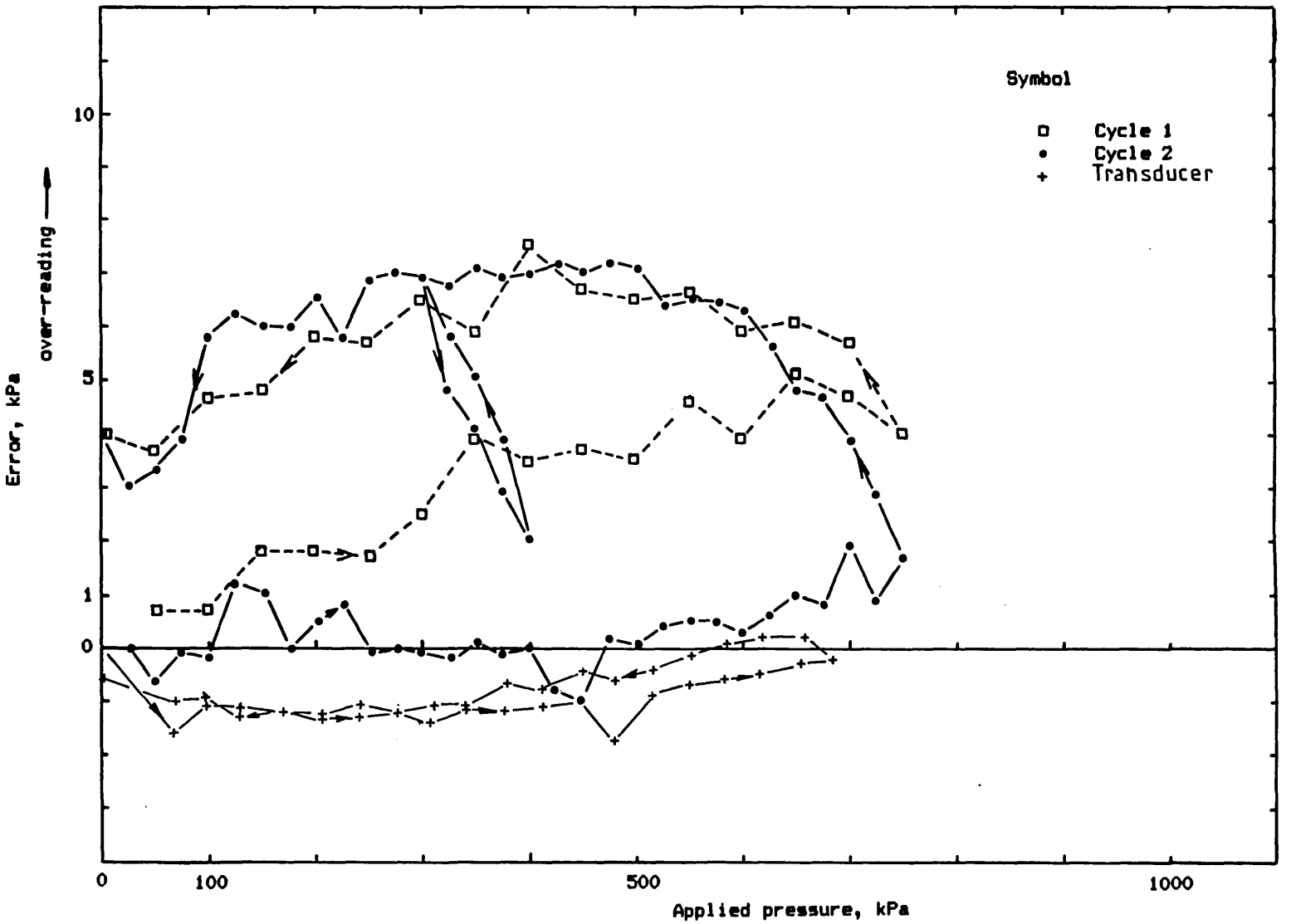


Fig.9.24 Calibration of Load Cell LC1

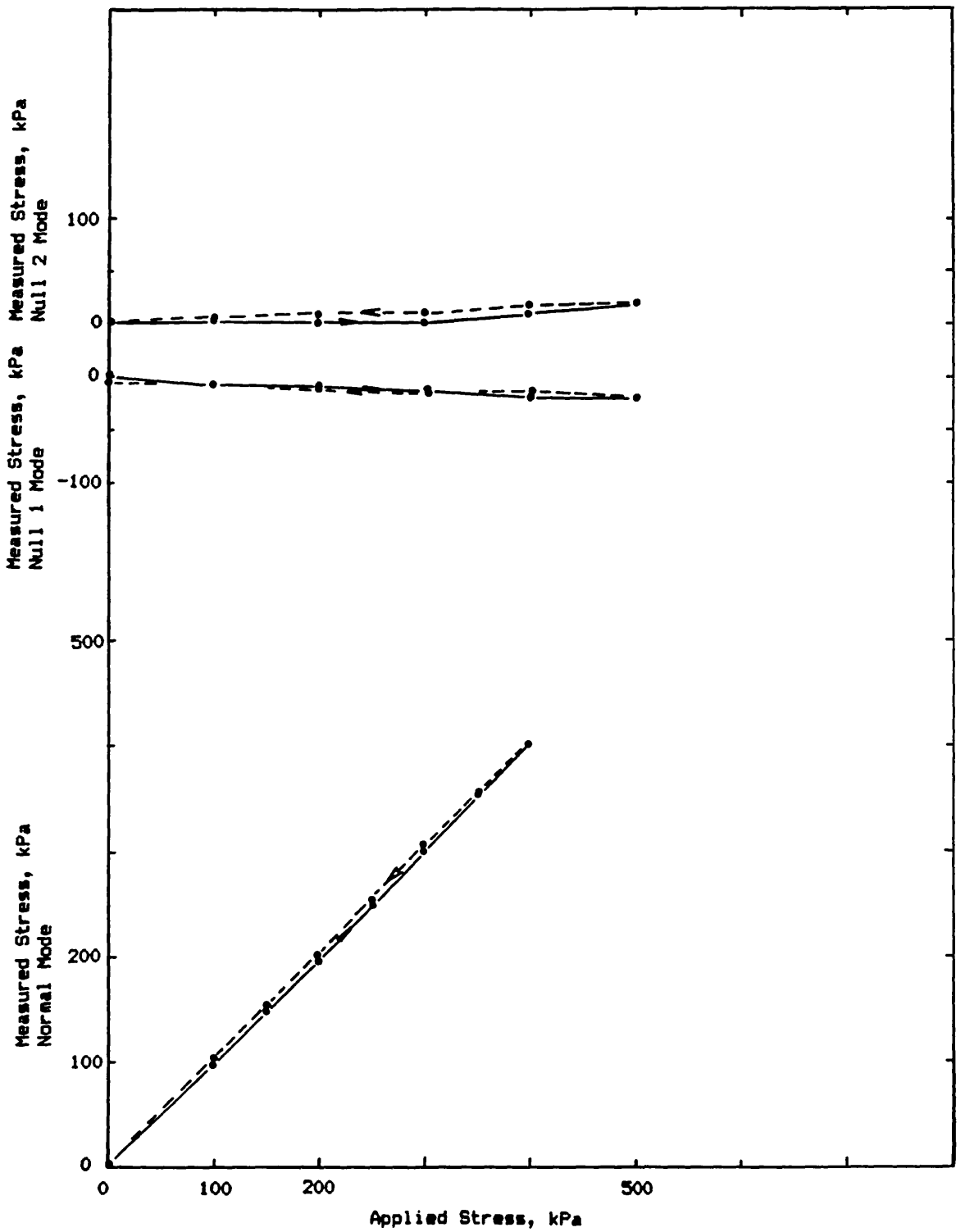


Fig.9.25 Calibration of Load Cell LC2

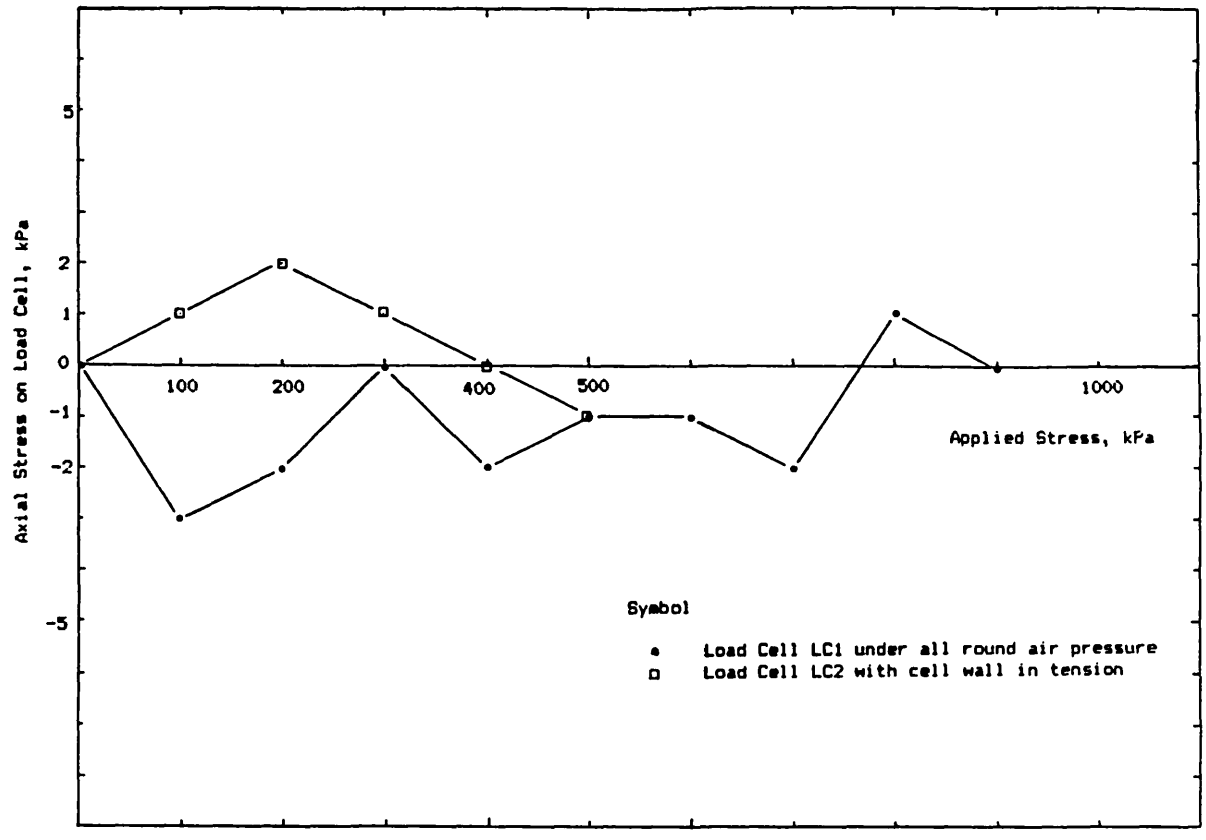


Fig.9.26 Load Cell response to All Round Pressure and Tension in Cell Wall

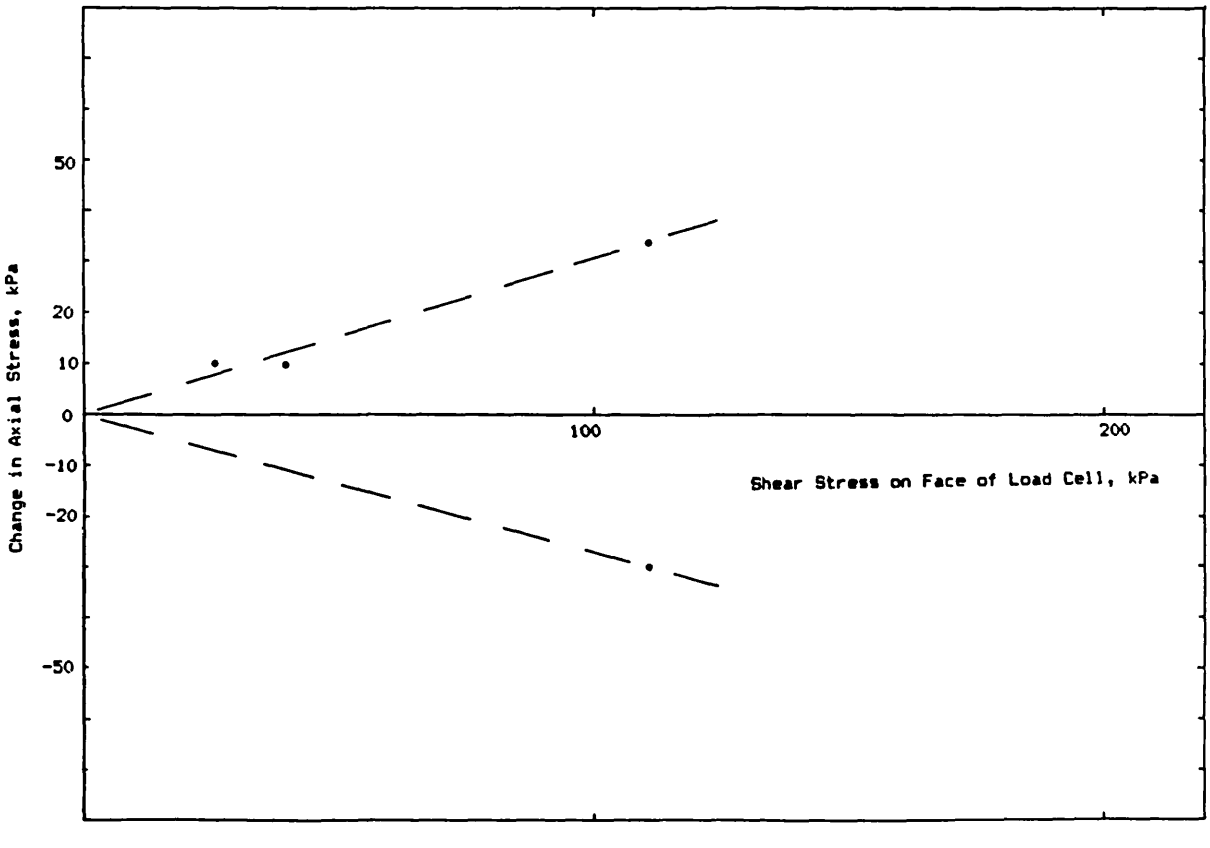


Fig.9.27 Cross Sensitivity of Load Cell LC2

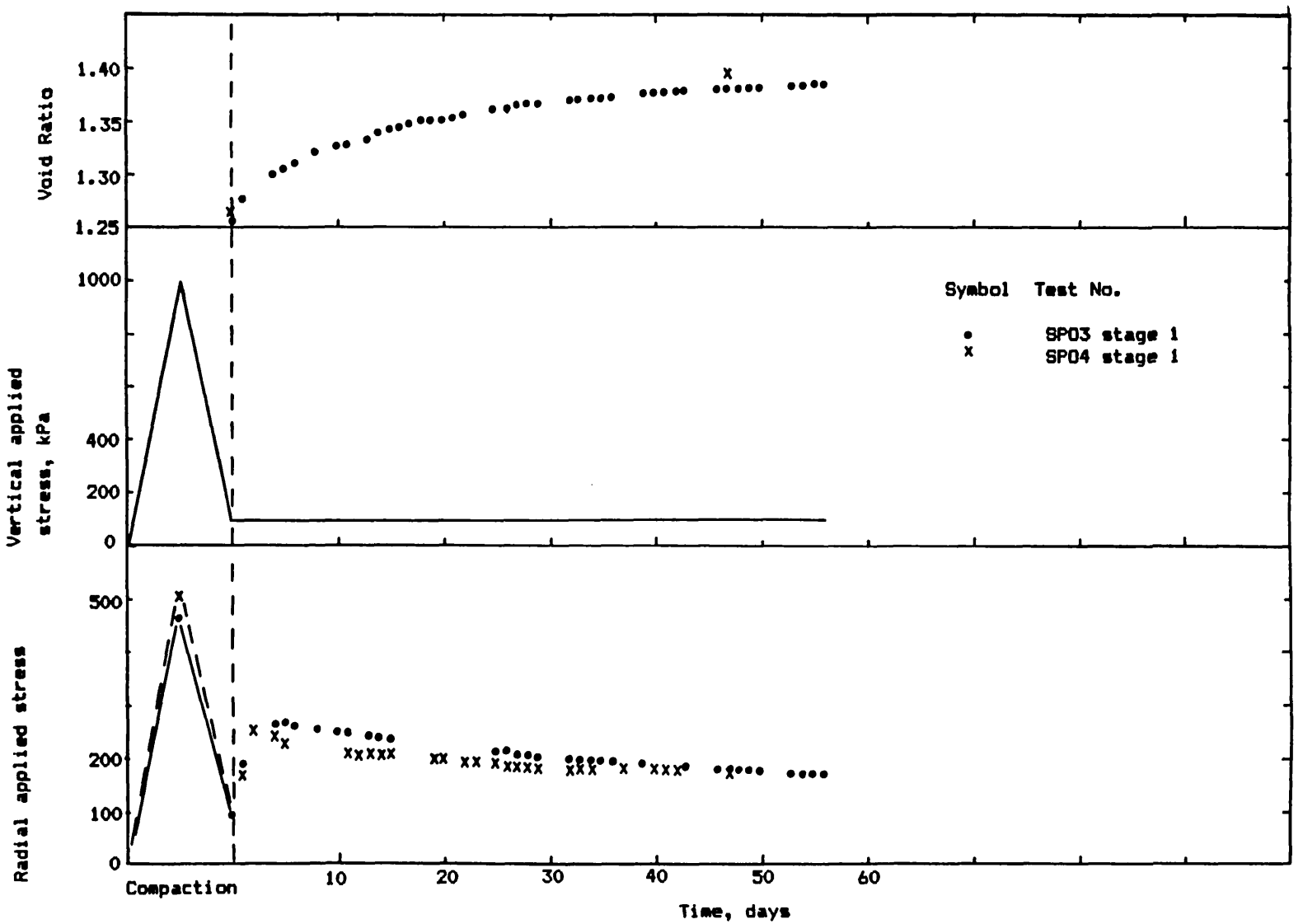


Fig.9.28 Repeatability of Test Results

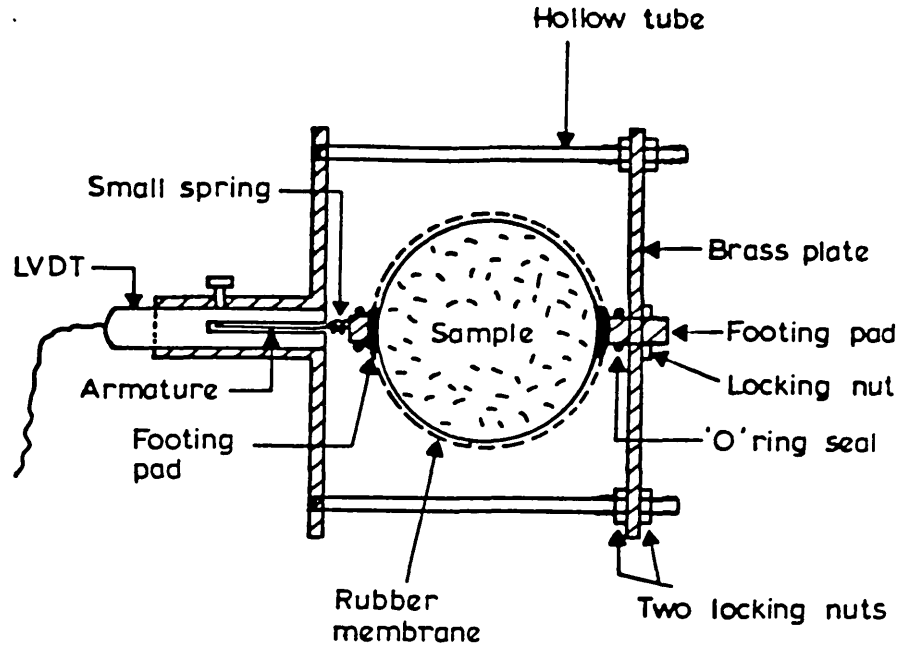


Fig.9.29 Details of the lateral strain measuring device, LSMD (plan view). **(Maswoswe 1985)**

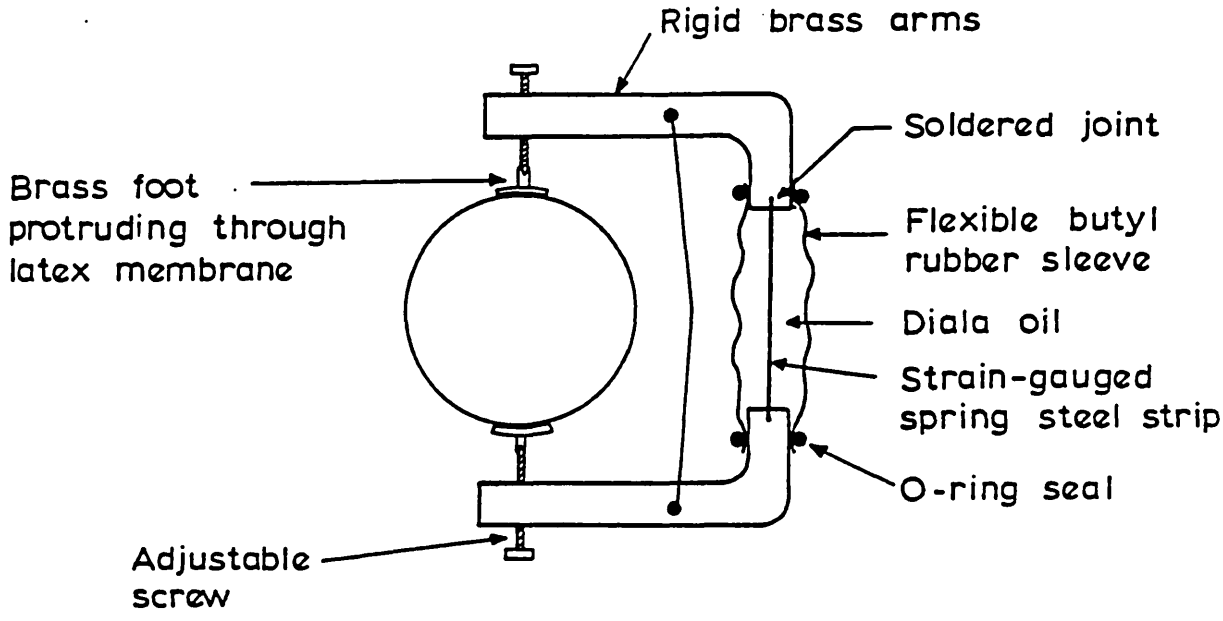


Fig.9.30 Radial strain belt for 38mm diameter samples. **(Hight 1983)**

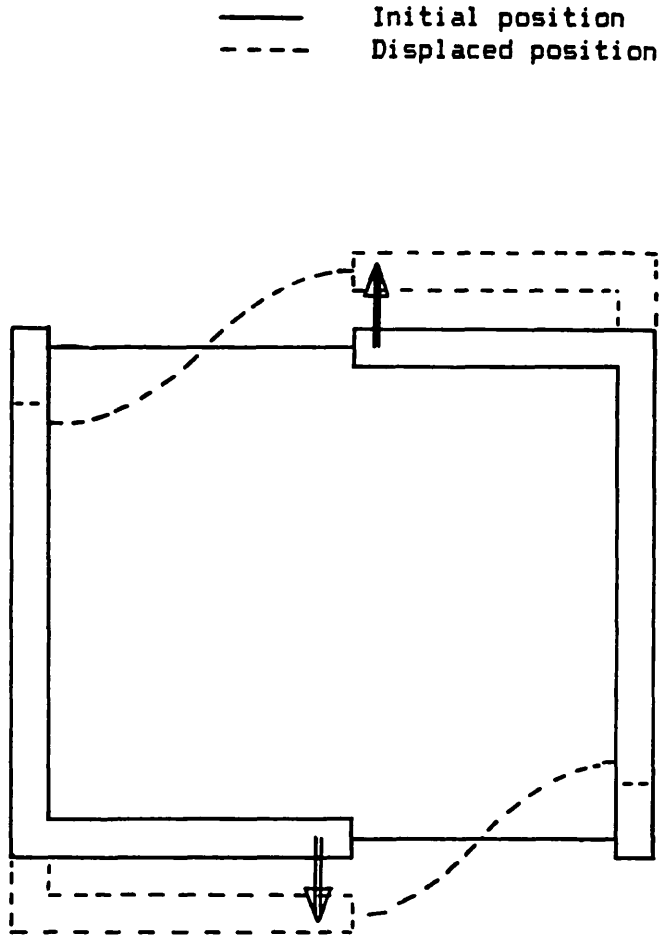


Fig.9.31 Bending of Two Webs in Balanced Gauge

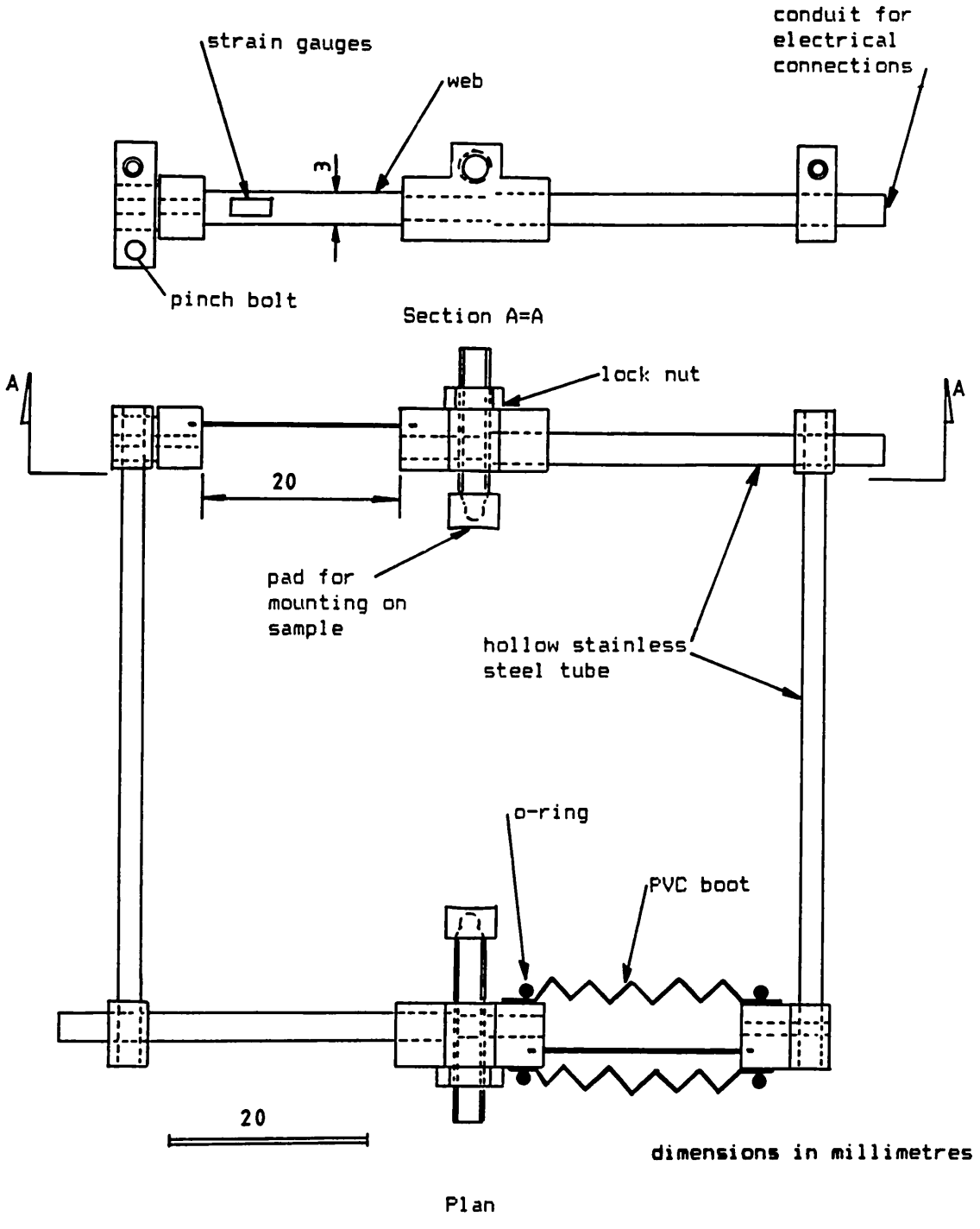


Fig.9.32 Radial Displacement Gauge

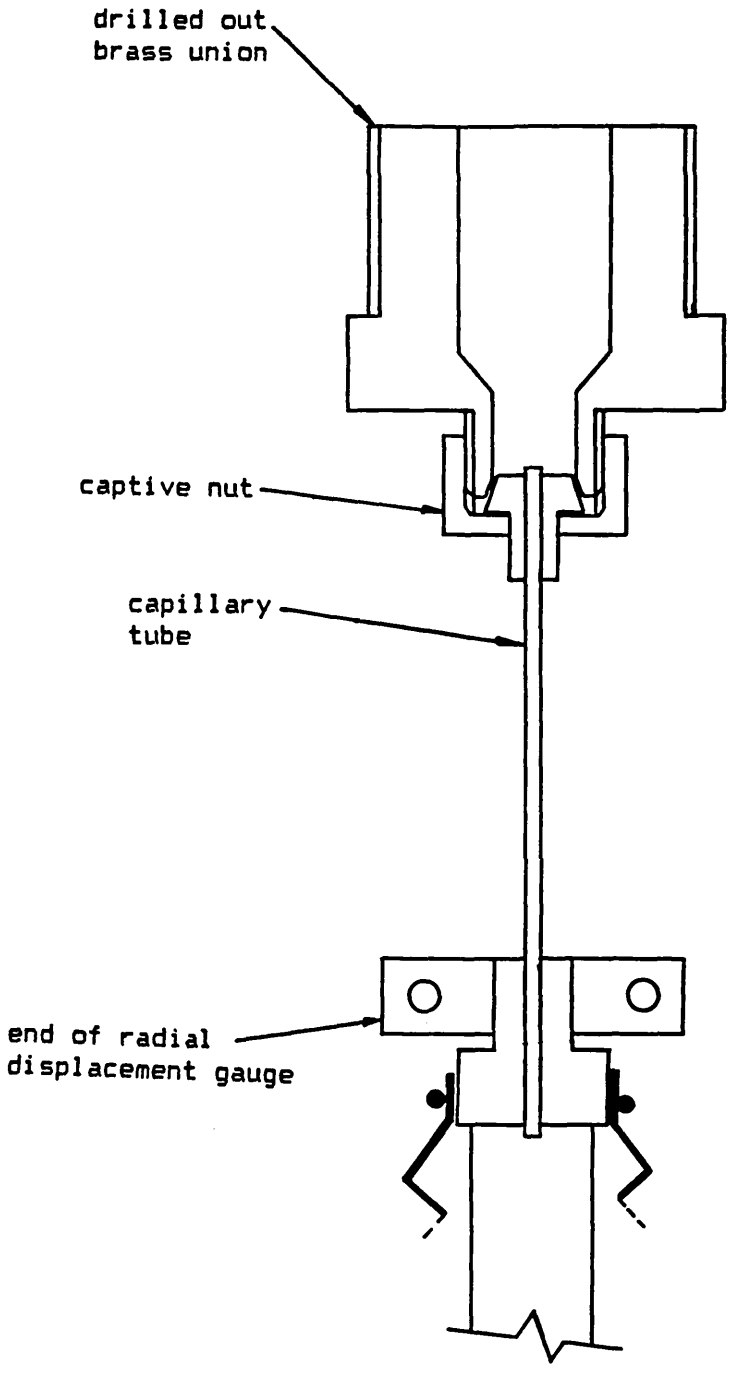


Fig.9.33 Reservoir for Oil Filling

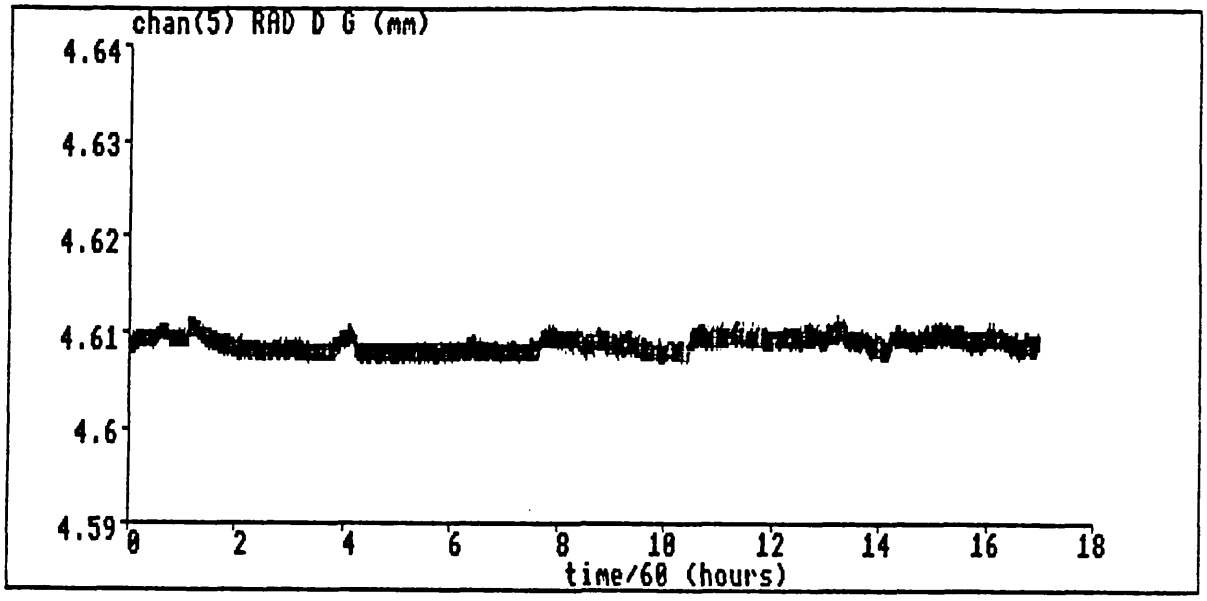


Fig.9.34 Stability of Radial Displacement gauge in Calibrator frame

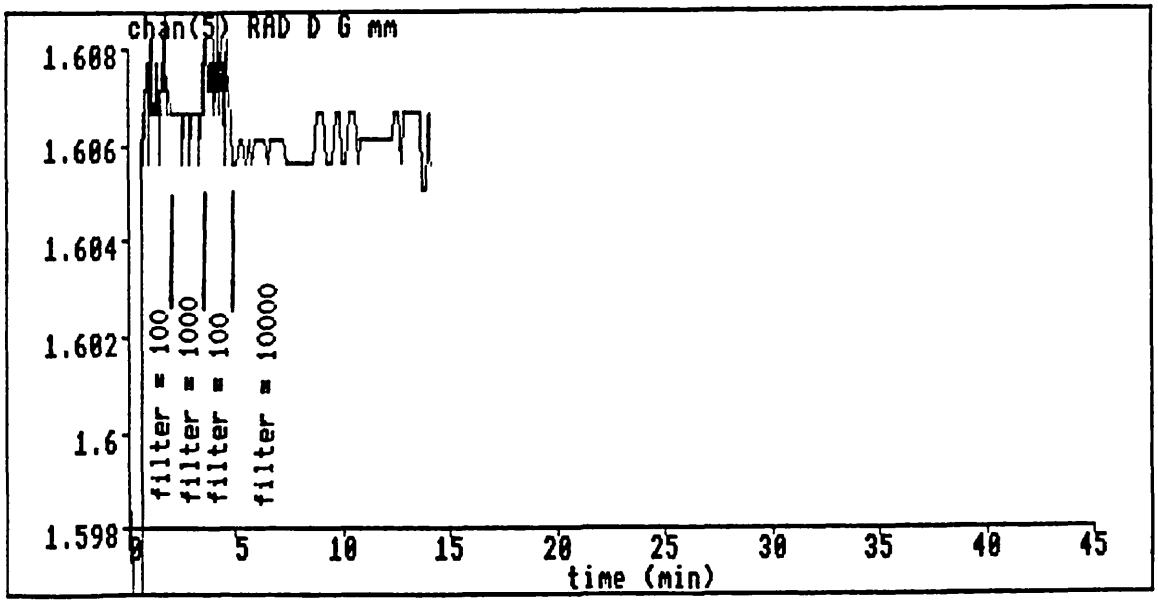
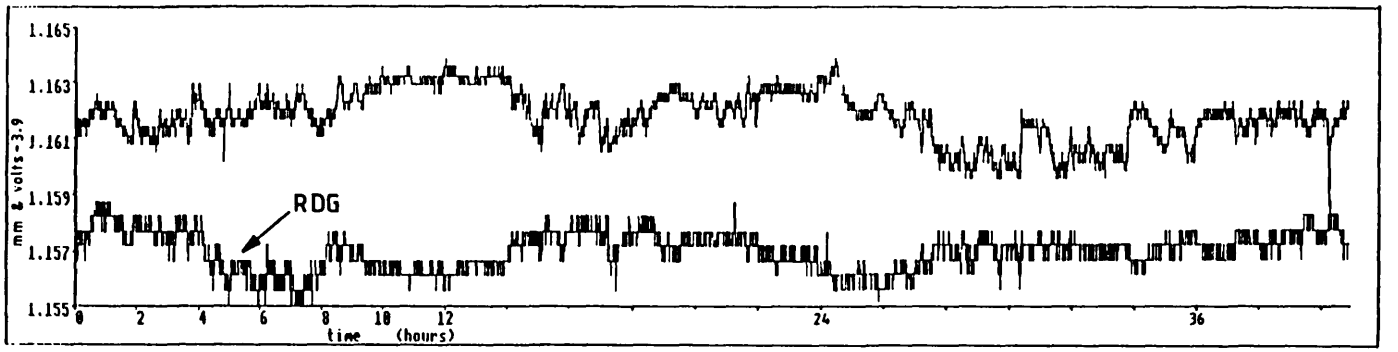
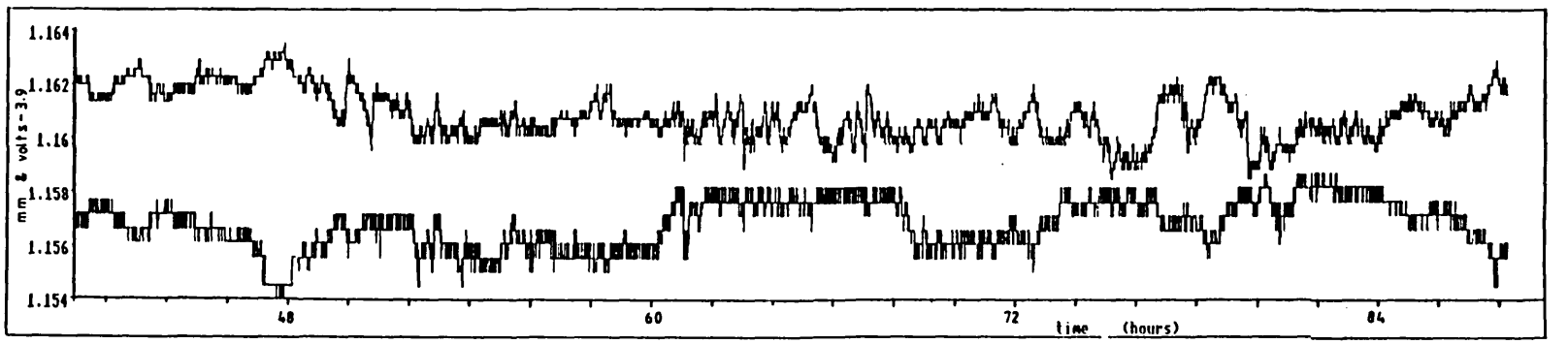


Fig.9.35 Effect of Changing the Filter





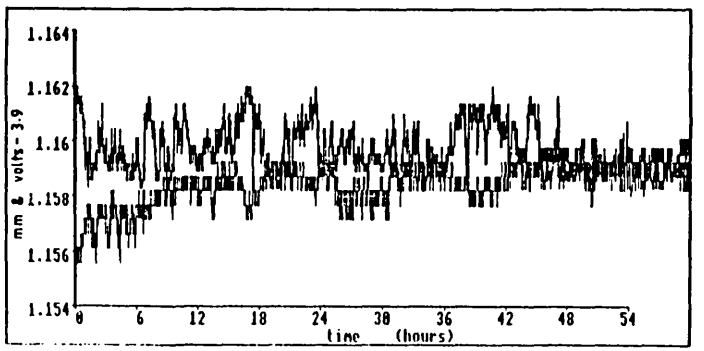
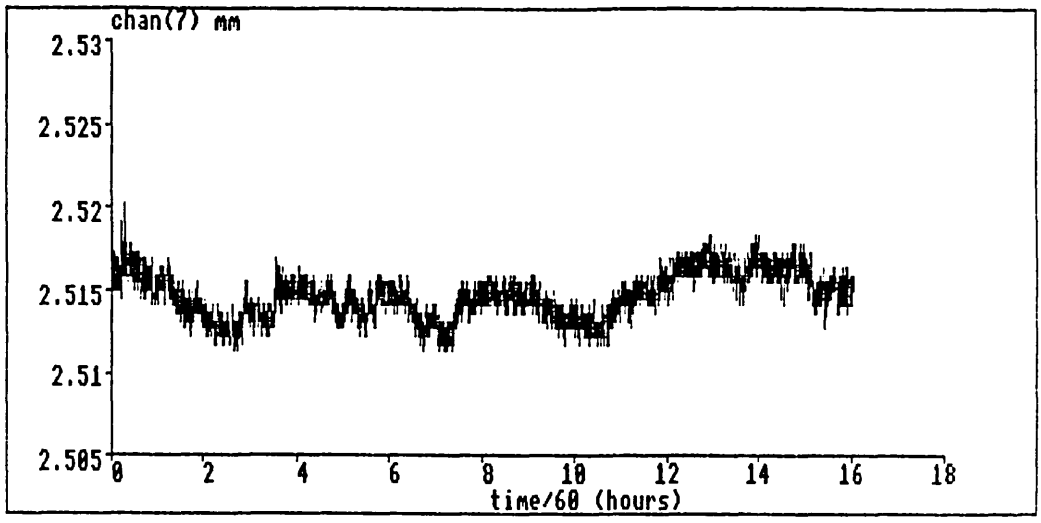
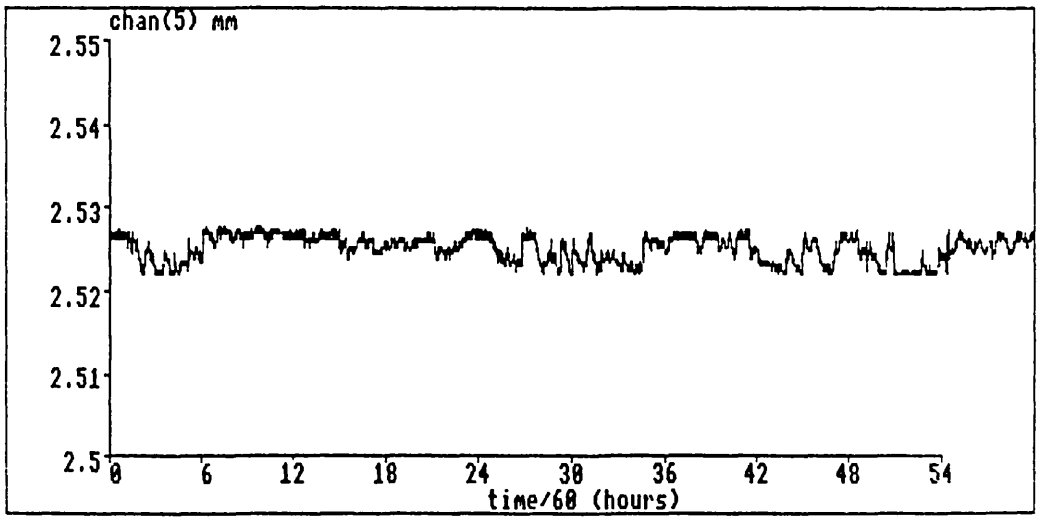


Fig.9.36 Long Term Stability of Radial Displacement Gauge mounted on brass block



(a) Open Circuit



(b) Shorted Circuit

Note different vertical and horizontal scales

Fig.9.37 Output without Radial Displacement Gauge

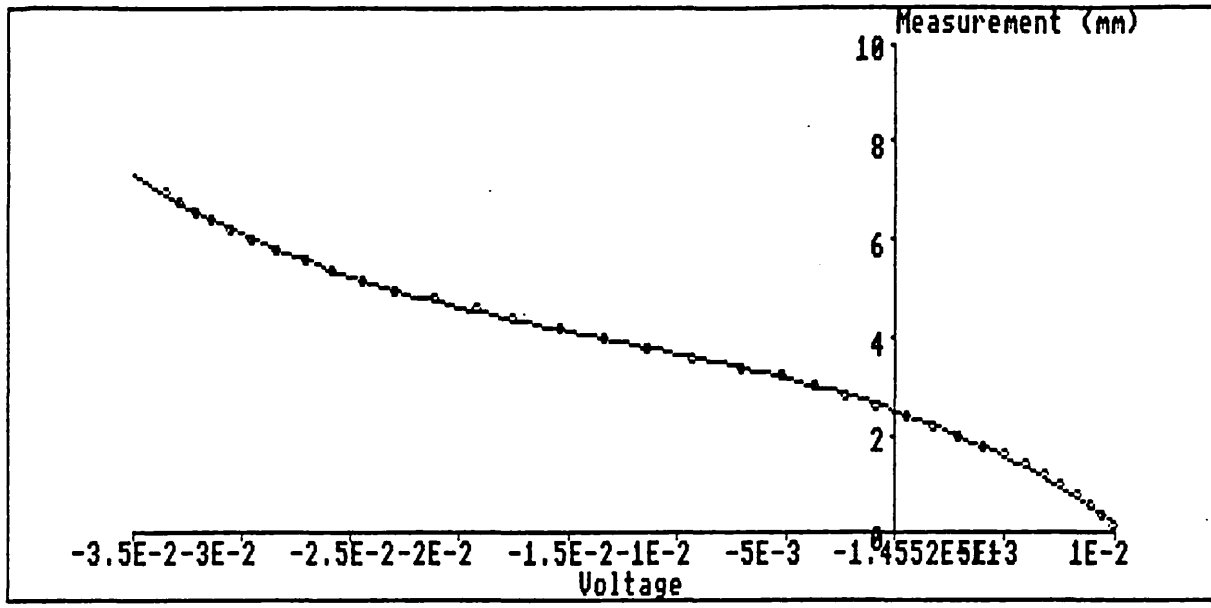


Fig.9.38 Calibration of Radial Displacement Gauge

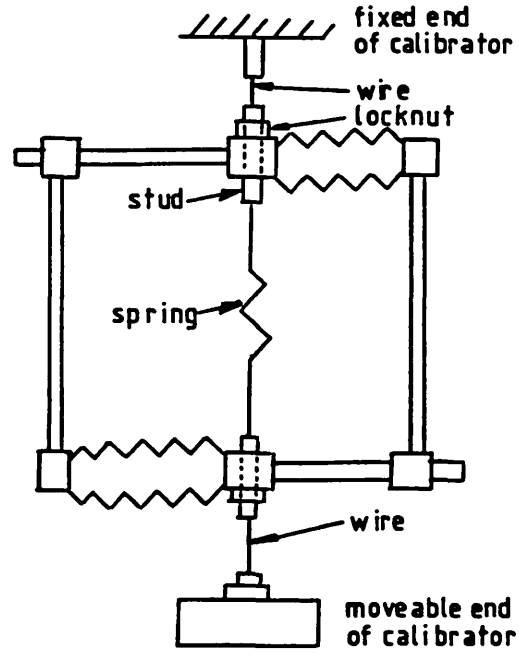


Fig.9.39 Layout of Radial Displacement gauge
in Calibrator for Calibration

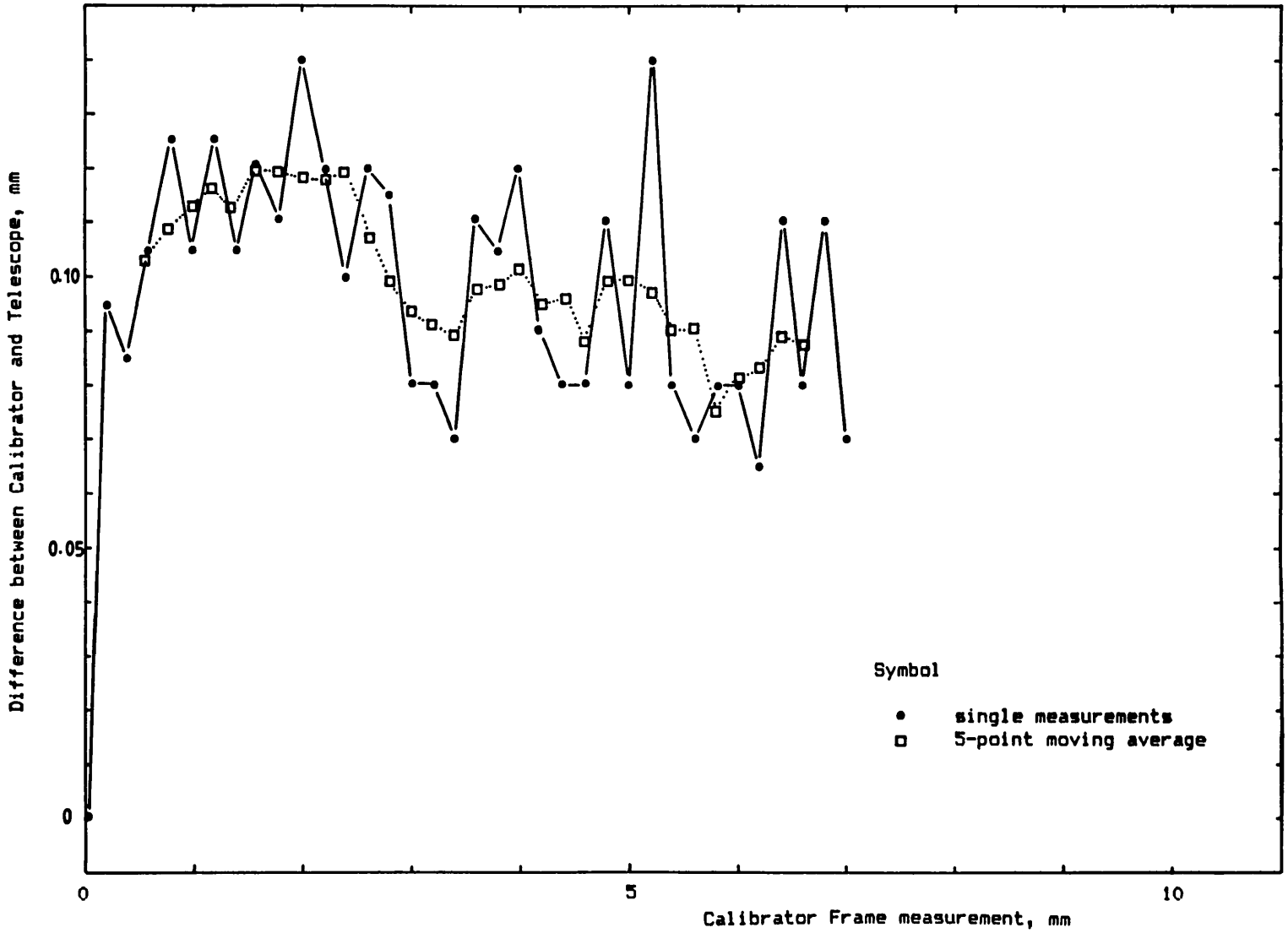
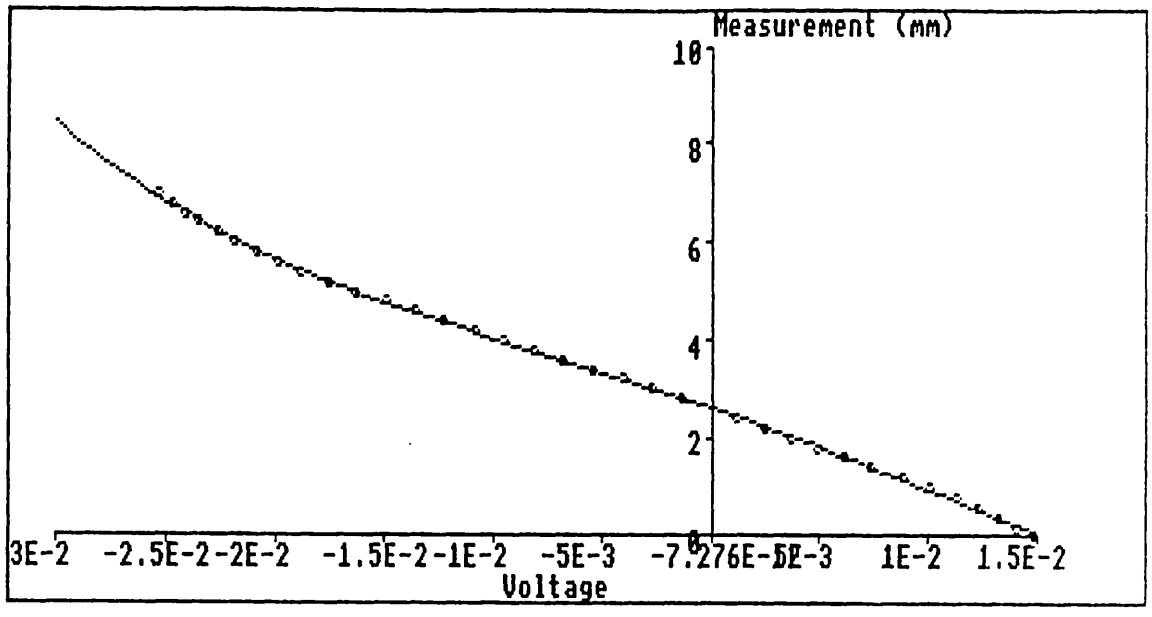
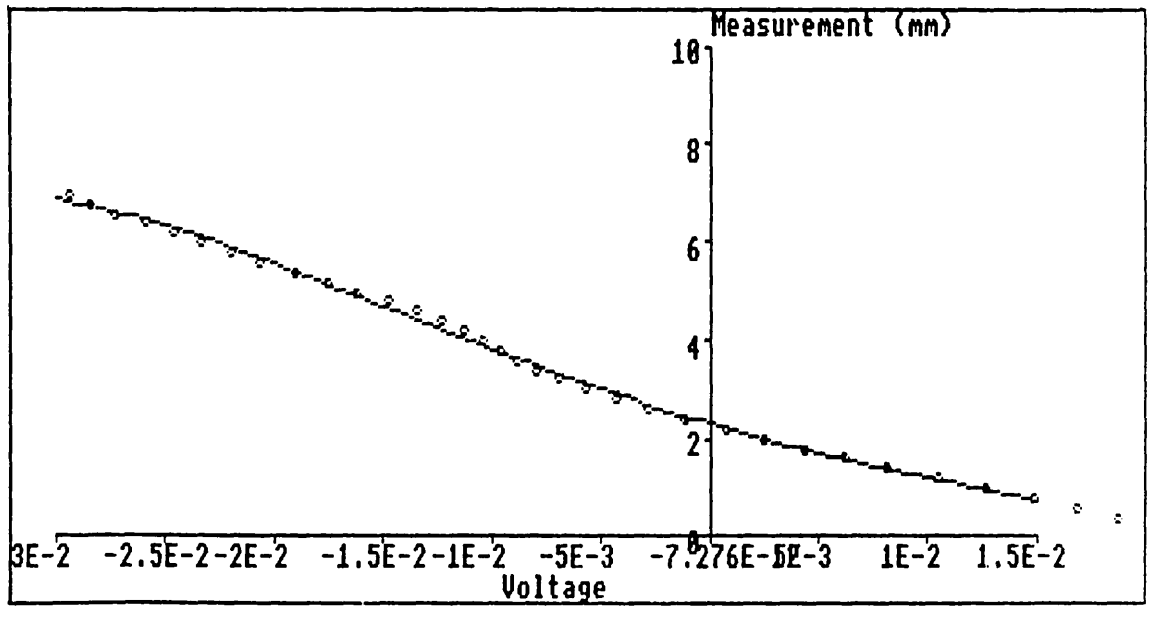


Fig.9.40 Diference between Calibrator and Travelling Telescope



(a) Joints rotated in the same direction



(b) Joints rotated in opposite directions

Fig.9.41 Effect of Rotation of Joints

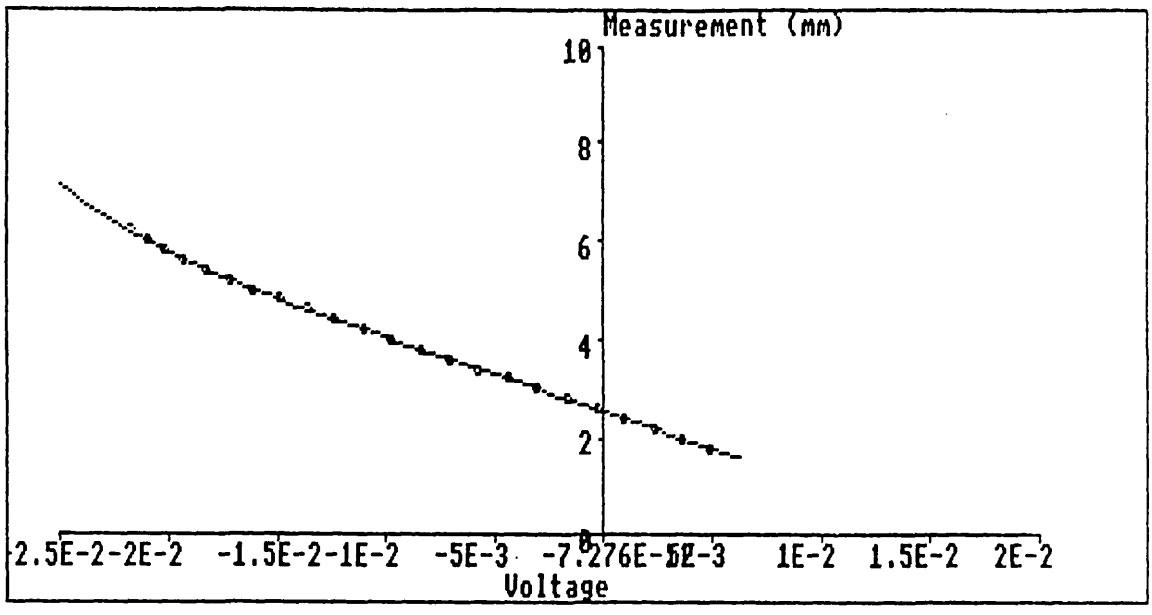


Fig.9.42 Final calibration of radial displacement Gauge

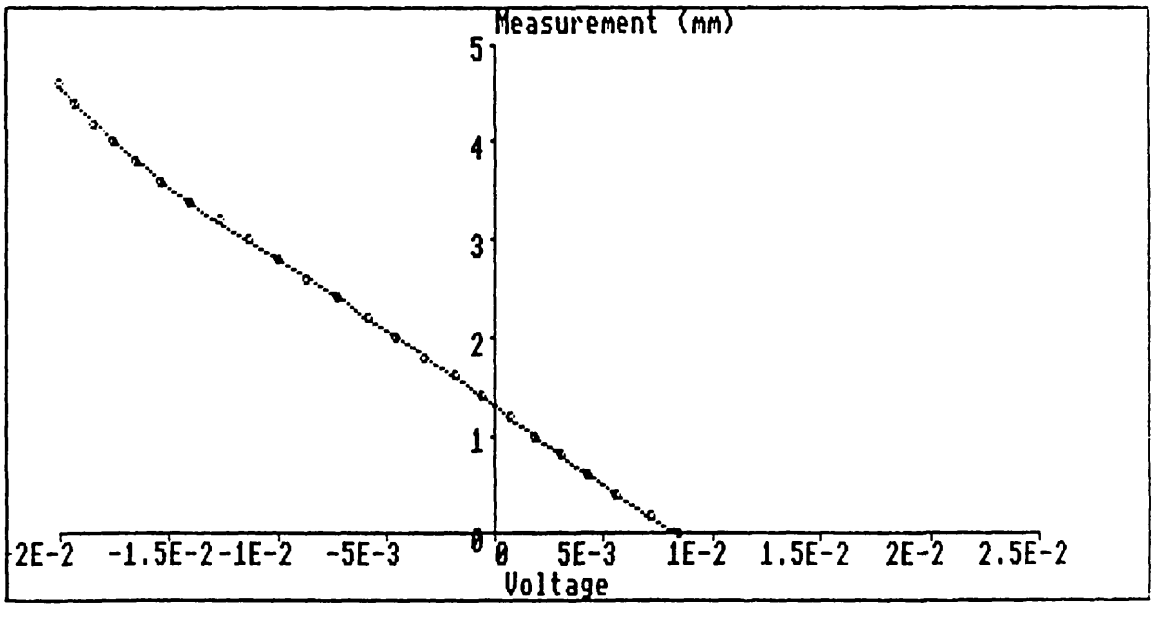


Fig.9.43 Recalibration after Repair of Soldered Joints

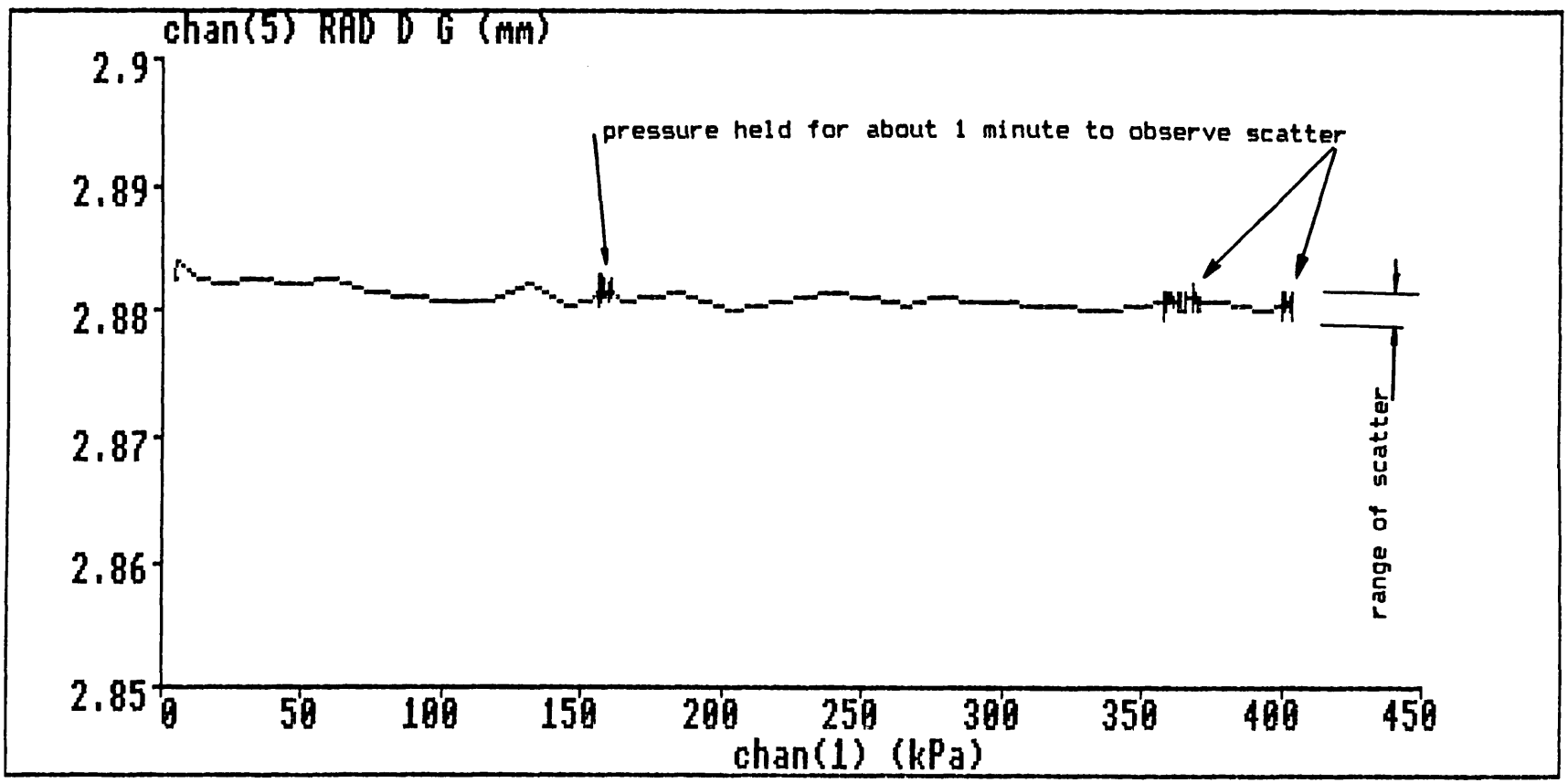


Fig.9.44 Effect of All-round pressure

APPENDIX 9A.1

CALCULATIONS FOR LOAD CELL

1.1 Calculation of Web dimensions

Design stress = 3MPa
Piston diameter = 15mm
Assumed web dimensions
width = 3.0mm
thickness = 0.65mm
length = 10mm
Compressive stress in web = $(3.0*15^2*.79)/(4*0.65*3)$ N/mm^2
= 68 N/mm^2

For E = $210*10^3$ N/mm^2,
strain = $68/(210*10^3)$
= 326 microstrain
Displacement of piston face = $326*10^{-6}*10$mm
= 0.00326mm

1.2 Calculation of Wheatstone bridge output

Vout = Vin/4(R_1/R_1 + R_3/R_3) volts
for gauge factor = 2
R/R = 2 L/L = $652*10^{-6}$
Vout = $10/4(652*10^{-6})*2$
= 0.00326 volts
Sensitivity = 0.00326/3000kPa
= 1 microvolt/kPa

1.3 Calculation of response

For >95% response, stiffness of load cell must be >19*soil stiffness (Maddocks 1983).
Soil stiffness = 12*G*piston diameter/(1-v)
Take G = 30MPa for a "stiff" soil
Soil stiffness = 7.2MN/m
Load cell stiffness = 0.00326mm for 3MPa
= 167MN/m

Load cell stiffness/soil stiffness = 167/7.2
= 23 OK

CHAPTER 10

STRESS PATH OEDOMETER TESTS

10.1 INTRODUCTION

A programme of tests on compacted samples was undertaken, supplemented by one test on a slurried sample. The tests were intended to investigate stress paths and volume changes during swelling from compaction conditions representative of the "dry compaction" technique described by Ellis (1980). A diagrammatic stress path typical of the tests performed is shown in Fig.10.1.

The stress path shown is a schematic stress path for a saturated soil, with the "equivalent" stages for an unsaturated soil as follows. It represents all stages, commencing with the suction (OA), followed by the compaction (AB), unloading (BC) and then swelling (CDE or CFE). The stress changes shown are representative of those that would be expected for a saturated soil, as those for an unsaturated soil have not previously been determined.

Of particular interest is the stress path after the compaction and unloading stages. The path that would be followed by a saturated soil during swelling in an oedometer is represented by CFE. The true stress path may not follow straight lines as depicted. Changes of pore water pressure with zero volume change in a saturated soil would not cause the effective stresses to vary, and the stress path would be a point at C. The stress paths for the unsaturated soils were not found to be identical with those expected for saturated soils.

The 1500kPa limit on the suction control restricted the compaction conditions for the tests carried out in this investigation to moisture contents not very dry of optimum moisture content. As reported by Ellis (1980), for an expansive soil from Sudan, the moisture content range used was 7 to 14% and the optimum moisture content was 22 to 28%. In this research programme, the compaction moisture content chosen was only about 5 to 7% dry of the optimum moisture content which was 38 to 40% for the compactive effort and technique used on this soil.

From the suction measurements in Chapter 8, testing to about 15% dry of optimum moisture content would have been possible. It had been intended that a similar series of tests would be performed on samples compacted at a lower moisture content, but this was not possible due to the long test durations and the delays in the fabrication of the equipment.

One preliminary test was completed before the radial stress load cell was fabricated. This test is not reported here, as it only served to demonstrate that the oedometer with the suction control functioned adequately.

10.2 COMPACTION

Static compaction, as used for the standard oedometer testing described in Chapter 7, was employed in the Stress Path Oedometers for preparation of the samples. Fig.10.2 shows the Stress Path Oedometer assembled for compaction. The 6.00mm thick plate on top of the sample was used because the soil surface in the preliminary test without the steel plate showed irregularities of about 10mm. It was felt that this could adversely affect the loading membrane.

The compaction stress was applied in increments, using air pressure from the compressed air bottles inside the loading membrane, and radial stress and axial displacement were measured. Each increment was held for 1 minute, except the final stress of 1MPa, which was held for 5min except for tests SP04 and SP07 where leakage necessitated a rapid unloading. Unloading was generally done in larger steps than the loading. The total stress paths during compaction determined in this way are shown in Fig.10.3a for each test. The relationship between void ratio and mean applid stress is shown in Fig.10.3b.

The variation between the stress paths is not thought to be excessive for the compaction process which is not well controlled. Many procedural factors could influence the stress path and final void ratio. One that is thought to be significant is the initial placing of the soil. This was done by pouring layers of loose soil about 10 to 15mm thick by hand. These were spread with a spatula to obtain an even thickness, but were inevitably inhomogeneous. The soil aggregations vary in size and tend to stick together during the spreading, making it impossible to ensure that the soil was evenly distributed within the loose layer. Although the compaction process would have reduced the inhomogeneity by shear distortion, complete uniformity is unlikely to be achieved. The non-uniformity of the sample may be greater for soils compacted dry of optimum, since the soil aggregations or clusters have greater strength and will resist the shear distortion. Very dry samples may behave as an assemblage of almost rigid grains, in much the same way as a sand does, resulting in a very non-uniform sample with large voids between the grains or clusters.

There does not appear to be any consistent link between the membrane type and preparation and the final radial stress. Tests SP01 and SP03, in which the membrane was not prestretched, are near opposite extremes of the radial stress. Tests SP04, SP05 and SP07, for which the membranes were prestretched show less variation in the radial stress. The variation in final void ratio does not correlate with the final mean stress.

Also shown on Fig.10.3b is the range of compacted void ratios for 35% compaction moisture content from the standard oedometer tests. It can be seen that the ranges in both types of oedometer are very similar.

10.3 TEST PROCEDURES

After compaction, the vertical stress was reduced to 0kPa and the top plate removed. The compaction plate was removed, but the filter paper was left in place and a no. 200 stainless steel mesh placed on top of the filter paper. This ensured that the air pressure could be distributed to the sample through a porous medium that would not become saturated until the suction was very small. The top plate was then replaced, and the air and hydraulic loading connections made. The loading membrane was filled with water and most of the air displaced. Complete de-airing of the loading chamber was not necessary, except in test SP05 where it was hoped that constant volume conditions could be achieved by closing the taps to the loading chamber.

With the preparation of the top of the cell complete, the compaction base plate was removed. The filter paper which had served as a drain for the pore air during compaction was removed. The flange of the cell lining was trimmed off and the base plate with the saturated ceramic disc fitted. This was sealed with a gasket in cell 1, and with an O-ring in cell 2. The latter proved more convenient, but satisfactory performance was obtained from both seals. At this stage the sample was not in contact with the ceramic, as the step in the compaction base had left the lower face of the sample 1mm recessed within the bore of the cell.

In tests SP01 and SP03 the air pressure within the sample was raised to its chosen value before the vertical stress was applied. This ensured that the sample could not take in water under zero suction when contact was made between the soil and the water in the ceramic. In the later tests this was not done, as it was found that the time that elapsed between the application of the vertical stress required to push the sample down into contact with the ceramic and the application of the air pressure was too short to be of any consequence. It was also easier to control and measure the vertical stress required to push the sample down when using the screw pump without the air pressure within the sample.

The initial vertical stress and suction were maintained until the change in sample height had decreased to an acceptable rate. The new air pressure or vertical stress was then applied. Only one of these two stresses was changed in each stage, and the changes in height and radial stress measured.

All stress changes were applied in increments or decrements. Constant rate of stress or strain application was not used. In test SP05 the vertical stress was adjusted during each suction decrease stage to maintain constant volume. At the start of the

test the technique used was to adjust the total vertical stress with the screw pump until the dial gauge reading was equal to the initial reading and then close the tap closest to the cell. All tubing between this tap and the cell was steel, and the vertical stress was measured with a transducer. However, it proved impossible to maintain the stress in this manner, and the vertical stress fell by about 30kPa overnight and there was a vertical expansion of 0.066mm. Thereafter the vertical stress was controlled with the mercury pot system with the air pressure applied in the top pot.

10.3.1 TEST DETAILS

Table.10.1 records the stress changes that were applied in each test. The results are plotted in Fig.10.4 to Fig.10.9, nd listed in Table.10.2. The labels for each stage used in the tables correspond to the annotation in the figures. In some figures not all points have been plotted in order to improve the clarity of the figure. The omitted data is available in the tables. In all tests the initial conditions are represented by point A.

10.3.1.1 Test SP01

Test SP01 commenced with an equilibration stage at an applied matrix suction approximately equal to the value reported in Chapter 8 of 940kPa and an axial applied stress of 50kPa. At the time that this test was started not all of the tests reported in Chapter 8 had been completed, and there was still some uncertainty regarding the reliability of the matrix suction measurements made with the pressure plate technique. It was expected that this stage would last for a relatively short time while the sample stabilised under the initial compacted conditions, without any significant volume or stress change. The reasons for the large void ratio and applied radial stress changes between points A and B in Fig.10.4 during this 43 day stage are discussed in section 10.4.

In stages 2 to 5 the matrix suction was decreased in 200kPa steps. The duration of each stage was at least 21 days. The stresses and void ratios at the end of each stage are indicated by the points C,D,E & F. Point E has been obtained by extrapolation as the loading membrane began leaking after about 14 days of stage 5. After repairs had been effected, the applied suction was decreased to approximately zero because it was thought, in view of the large volume change that had occurred during the leakage period, that the suction in the sample could have fallen below the intended value of 100kPa.

This was thus the start of stage 6 in which the suction was reduced by 100kPa to zero. The air and water pressures were set approximately equal, and zero suction was ensured by allowing the sample free access to water under a head of about 20cm after the

21day swelling period. Virtually no change in sample thickness or radial applied stress was noted over a 4 hour period. The swelling in the standard oedometer tests reported in Chapter 7 was, for comparison, largely complete within 3 hours.

Stages 7 to 12, shown by points H,I,J,K,L and M, were planned as a conventional consolidation test, as performed in the Jennings & Knight (1957) type test described in section 2.3.1.1. This was followed by an unload and reload cycle in stages 13 to 15 and 16 to 18 respectively.

10.3.1.2 Test SP02

This test failed during the first stage due to a blocked drainage channel beneath the ceramic and an unstable load cell. It is recorded purely because the compaction data has been reported.

10.3.1.3 Test SP03

This test was intended to check on the data from test SP01 at low suction where leakage might have affected the sample. The sample was thus allowed to swell to a 200kPa suction in the first stage, and to zero suction in the second stage.

10.3.1.4 Test SP04

The first stage of this test was similar to the first stage of test SP03, but the applied suction was 175kPa. The sample was then subjected to increasing suction in stages 2 to 6, shown by points C to F in Fig.10.6, reaching 625kPa in stage 6. The suction was decreased to 175kPa in stages 7 and 8, shown by points G and H. The axial applied stress was then increased in stages 9, 10 and 11, shown by points I,J and K, to 200kPa. The suction was then reduced to zero by leakage of the membrane in stage 12, shown by point L, The sample was finally unloaded to 10kPa vertical effective stress in stage 13 shown by LM.

10.3.1.5 Test SP05

This test differed from the previous tests in that the axial applied stress was not held at 50kPa during the swelling stages. The procedure followed the Sullivan and McClelland (1969) procedure for a constant volume then rebound test. The suction was changed in stages 1 to 6 in the same steps as used in test SP01, but the vertical stress was increased or decreased as required to maintain zero vertical displacement as measured with the dial gauge. The data are shown by points A to G in Fig.10.7. Stages 7 to 10, shown by points H to K, reduced the vertical effective stress from the swell pressure to 50kPa.

10.3.1.6 Test SP06

The sample for test SP06 was not compacted, but was prepared as a slurry at an initial moisture content of 136% or about 1.1 times the liquid limit. It was thoroughly deaired in a vacuum chamber before the water was added, without releasing the vacuum, to the pulverised soil which had been taken from the bulk sample described in Chapter 5. The soil was then thoroughly mixed by hand and carefully poured into the greased oedometer which did not have a rubber lining for this test. The load cell was sealed with a piece of PVC adhesive tape which extended from the base of the cell to above the top of the sample. A circular piece of PTFE tape was placed over the load cell, and beneath the PVC tape, to ensure that no adhesive could penetrate between the load cell and the cell.

The sample was initially loaded by applying a suction of 10kPa to the drainage tube, and thereafter using the hydraulic loading system up to 160kPa vertical effective stress. An unload - reload cycle was then followed, and the resistance to extrusion of the sample checked while the sample was unloaded. The vertical stress was then increased to 640kPa, and finally reduced to zero.

10.3.1.7 Test SP07

This test was undertaken to check on the data from the initial stage of test SP01, where a high radial applied stress was measured. Stage 1, shown by point B, corresponds to stage 1 of test SP01, where the suction was approximately equal to the measured value. The axial applied stress was increased in stages 2 to 4, shown by points C to E, before starting to reduce the suction. The test has not been completed, but the necessary data for this thesis is that from stage 1.

10.4 TEST RESULTS

The data obtained from these tests are listed in Table.10.2 and shown in Figs.10.4 to 10.9. The figures for the unsaturated samples contain three sections. Section (a) shows the void ratio changes in terms of the changes in the mean stress referenced to atmospheric pore air pressure plus the suction (or minus the pore water pressure referenced to atmospheric pore air pressure). Section (b) shows the deviatoric stress in terms of the same mean stress plus suction. The three axis plots in section (c) include void ratio. mean stress referenced to atmospheric pore air pressure, deviatoric stress and pore water pressure referenced to atmospheric pressure. Sections (a) and (b) are thus partial representations of the information in section (c) in a form that may be simpler to assimilate.

The data for test SP06 are shown in Fig.10.8 together with the

total stress data, referenced to atmospheric pore air pressure, for the initial conditions of the unsaturated samples. It has been suggested by Burland (1985) and Maswoswe (1985) that soils will collapse onto the normally consolidated compression line as shown in Fig.10.10.

Vaughan (1988) proposed, on the basis of Vaughan et al (1988), that for residual soils, the region above the compression line obtained for a destructured soil is a meta-stable zone, Fig.10.11, in which large compressibilities can be expected when the bond yield strength is approached or exceeded. This is similar to the figure presentd by Burland (1985) for collapse, and it was thought that the two concepts could be combined into a generalised indexing system for compressibility or collapse due to local shear of suction and cementation dependent bonds. It will be shown that this generalised indexing is not valid for these samples.

10.4.1 DETAILED RESULTS

10.4.1.1 Test SP01

The data are presented in Fig.10.4. The test details have been listed in section 10.3.1.1 and Tables 10.1 and 10.2.

In stage 1, where the matrix suction was set at 900kPa, there was a large increase in the radial stress, and a simultaneous increase in the sample thickness. The radial applied stress increased from 131 to 349kPa, and the void ratio increased from 1.285 to 1.331 as listed in Table 10.2. The axial applied stress was held at 50kPa in stage 1. The mean stress and stress difference and void ratio changes can be seen in Fig.10.4 as AB in parts (a), (b) and (c). The moisture content increased from 34.2 to 38.3% during the 43 day period of stage 1.

The stages of decreasing matrix suction showed a continuously increasing void ratio as shown by points CDEFG which depict the void ratio and stresses at the ends of stages 2,3,4,5 and 6 respectively. The increase in the radial applied stress in stage 2 is evident from the slight increase in the magnitude of the stress difference in Fig10.4b from B to C. Thereafter the radial applied stress decreased in each stage along DEFG, with the rate of decrease becoming greater as the magnitude of the suction decreased.

On commencing the consolidation stage very little change in the void ratio or applied stress was noted in stage 7. Since the air pressure was equal to the atmospheric pressure for this part of the test and the pore water pressure was about 2kPa with water volume measurement in a graduated burette, the axial and radial stresses are conventional total stresses. Significant void ratio and radial stress changes were observed thereafter, as visible from points I,J,K,L and M in Fig.10.4a & c, after stages 8,9,10,11 and 12 respectively. It had been expected that the radial stress

would respond as for a reconstituted saturated soil, i.e. that it would increase with the increase in the vertical stress and then decrease with the dissipation of the pore water pressure. This did not occur, and the observations are discussed in section 10.5.4.

The unload and reload data are shown by MNOP and PQRS respectively. As is to be expected, the soil is less compressible during reloading.

10.4.1.2 Test SP03

In the first stage of swelling to 200kPa suction, the void ratio increased continuously, as shown by AB in Fig.10.5. However, the radial applied stress first increased from 90kPa to 272kPa and then decreased. This has been indicated by a dotted line as the suction was not controlled between 900 and 200kPa. The increase followed by a decrease is consistent with the observations from test SP01, where the suction was decreased in smaller steps. The radial applied stress at the end of this stage was 210kPa.

The data for stage 2 are shown by BC, and indicate a decreasing radial stress and an increasing void ratio.

10.4.1.3 Test SP04

The first stage, shown by AB in Fig.10.6, shows data similar to that of test SP03. In stages 3,4 and 5 the radial applied stress decreased with the increasing suction, as shown by BCDEF in Fig.10.6b & c. The void ratio continued to increase in stage 2, which was merged with stage 3. A slight increase in void ratio occurred in stage 3. This is thought to be due to creep, which is discused in section 10.5.1.1. In stages 4 and 5 the void ratio decreased as shown by DEF in Fig.10.6a.

FGH shows the increase in void ratio and radial applied stress as the suction was decreased to the same value as at the end of stage 1. The difference between B and H in Fig.10.6a is thought to be due to creep.

HIJK shows the data for increasing axial applied stress. The void ratio decreased and the radial applied stress increased as evident from HJK in part (a) and HIJK in part (b) of Fig.10.6.

Near the end of the consolidation in stage 11, shown by point K, a leak occurred in the loading membrane, and the suction decreased to zero overnight. This would be expected to cause swelling, but KL in Fig10.6a shows that a large amount of compression occurred under the constant axial applied stress of 200kPa.

The unloading is shown by LM, where the radial stress decreased and the void ratio increased.

10.4.1.4 Test SP05

This test was a constant volume wetting test. The first stage shows a large increase in the radial applied stress and the axial applied stress while the matrix suction was maintained st 900kPa. These changes are shown by AB in Fig.10.7. The void ratio was maintained within +.0040 and -.0001 of the initial value. The former was due to the loss of vertical stress noted in section 10.3. After the technique was changed, the void ratio was maintained within +.0008 and -.0001 of the initial value. The vertical movement was measured with a dial gauge graduated ti 0.0001 inches.

Decreasing the suction in stages 2 to 6 resulted in first an increase and then a decresase of the axial and radial applied stresses, as shown by BCDEFG in Fig.10.7a & b. The void ratio was maintained within +/-.0005 until stage 5. It proved more difficult to maintain the sample thickness when the vertical stress had to be decreased to prevent "collapse" or compression due to decreasing suction. The void ratio in stage 5 varied by +0 to -.0011, and in stage 6 by-.0001 to -.0022.

The swelling after attaining zero suction is shown by GHIJK. It is thought that the radial stress at point K may be excessively high due to friction between the sample and the cell wall.

10.4.1.5 Test SP06

The consolidation curve from this test is shown in Fig.10.8. The stress path, shown in the lower half of the figure indicates a curved K_o stress path along DEFPQ. It was thought that the friction between the sample and the cell could be excessive, and the vertical stress was thus removed along FGHIJKL. The base plate was then removed and an axial stress applied. The sample started to move under an axial stress of 2kPa. The radial stress was 30kPa while this was done, indicating that there was very little friction between the cell and the sample.

10.4.1.6 Test SP07

Stage 1 of this test was required for comparison with stage 1 of test SP01. The stress and void ratio changes for this stage are shown by AB in Fig.10.9, and correspond quite adequately with those in test SP01. BCDE shows the void ratio and stress changes due to an increasing axial stress. The test has not been completed and will be continued in future research.

10.5 DISCUSSION OF TEST RESULTS

10.5.1 SWELLING

10.5.1.1 Solute suction

Tests SP01, SP03 and SP04 allowed the samples to swell under constant axial applied stress. Most noticeable in the data is the increase in radial stress in test SP01 during the first stage where the matrix suction was almost equal to the previously measured value. This is evident in the plotted stresses as the decrease to a greater negative value in the deviatoric stress and the increase in the mean applied stress along AB in Fig.10.4(b) and (c). Coupled with this increase in the applied mean stress and nominally unchanged matrix suction is a large increase in the void ratio.

In terms of conventional soil mechanics this is unacceptable, and caused some concern about the measured suction and the experimental equipment. The investigations into the measurement of suction were described in Chapter 8, and indicate that the measured matrix suction is correct. It was known from the salt content measurements reported in Chapter 5 that the soil has a high salt content, and this test apparatus is not capable of controlling or measuring the solute or total suction.

It is possible to make a qualitative investigation of salt diffusion into the back pressure system by measuring the electrical conductivity or resistance of the water in this system. The water was initially distilled water, with a very high resistance or low conductivity. An increase of the salt content of the water will be reflected as a decrease in the resistance of the water. Water conductivity is usually measured in a standard cell with two electrodes of standard size at a standard distance apart. This could not easily be incorporated into the oedometer system, but qualitative measurements could be made using the nylon tubing of the back pressure system as a measurement cell.

On opposite sides of the base plate are nylon drainage lines leading to the air traps. The resistance of the water within these tubes was measured by connecting an electrical test meter to the brass couplings at each end of the tube, which serve as the electrodes. Since the tubes on each side of the base are of different lengths, the resistances for identical fluids within the tubes will be different.

Measurements were made during the flushing process whereby the air bubbles were removed. Flushing was done daily, but resistance measurements were made less often. The resistance at the start of the flushing was high and reflects the low salt content of the bulk volume of water in the back pressure system. As the water which had been in contact with the ceramic was displaced from the

base and flowed into the drainage lines a marked drop in the resistance was observed. Data from Tests SP01 and SP03 are plotted in Fig.10.12 and 10.13, respectively. The data should be interpreted qualitatively. It is of interest to note that the diffusion of salt out of the sample in test SP03 occurred against a flow of water into the sample due to the change in matrix suction.

The low resistance measured during flushing is indicative of the salt content in the soil water. In some instances the flushing was not done for one or two days, without causing a marked change in the resistance during flushing. This suggests that salts in the water beneath the ceramic were in equilibrium with the salts in the soil water after one day.

As the flushing continued, on any particular day, the resistance began to rise again after the water which had been beneath the ceramic was replaced with water which had been in the tubes and bubble trap on the upstream side if the base. The final resistance was similar to the initial resistance.

It can be seen, from Fig. 10.12 and Fig.10.13, that the resistance of the bulk water approached the resistance of the water from beneath the ceramic, but that complete equalisation had not been achieved after the 40 day period of these stages. However, the top graphs in these figures show that the rate of change of void ratio had become small. Some further swelling would have taken place had the stages been continued, but this would have been small compared to the changes that had already taken place. A further 40 days in test SP03 would have caused a change in void ratio of about 0.006, or about 5% of the change in this stage.

The resistance changes show clearly that a large amount of salt was diffusing out of the soil water into the back pressure system. Since the stress and matrix suction changes are inconsistent with swelling, it is concluded that the change in solute suction due to the diffusion of salts out of the soil water is the cause of the swell observed during this stage.

After the large solute suction changes in the first stages there would have been much less change in the solute suction during the subsequent stages. Resistances were measured during the remainder of test SP01 and showed no variation during the flushing process after stage 3. Whatever effects there may have been could not be distinguished from the effects of matrix suction and applied stress. However, as noted above, the rate of change of void ratio at the end of the 900kPa suction stage was small, and the effect of the solute suction change would have been very much smaller than the effect of the matrix suction changes.

10.5.1.2 Volume Change

The rate of increase in void ratio increases slightly with

decreasing suction after the first stage of test SP01. There is no increase visible in Fig.10.4c in the rate of increase of void ratio with decreasing suction and mean stress in the final stage to zero suction, FG, where water was supplied both through the ceramic and through the top tube which had previously supplied the air. This indicates that the swelling stages had been long enough for dissipation of the excess suction in each stage.

Rates of axial strain are shown in % per day in Figs.10.14 to 10.17 for tests SP01, SP03, SP04 and SP07. The rate of strain in the first stage is higher than those in the later stages for tests SP01 and SP03. These are the stages where swell is due to the solute suction change, and are thus not directly comparable with the later stages. In the remaining stages of test SP01, there is a consistent increase in the rate of strain with decreasing suction. There is considerable scatter in the data, but the trend lines have been drawn in an attempt to fit a best curve by eye through the data for each stage. The increasing rates of strain with decreasing suction in stages 2,3 and 4 of test SP01 are consistent with the observed increase in the rate of swell with decreasing suction.

All of the tests show a decrease in the rate of strain to a small value, but do not appear to indicate a terminating or zero rate of strain. Highly plastic soils are known to creep or exhibit secondary consolidation in the saturated state, and it is evident that this phenomenon occurs in the unsaturated state as well. The rate of apparent creep is in the range 0.01 to 0.02% per day at the end of the test stages. This would not have a significant effect on the observed void ratio changes.

The rate of suction change appears to have a more marked effect than the creep on the magnitude of the change in void ratio for complete dissipation of the suction. As noted in Schreiner (1987f), the swell or vertical strain in the long duration test, (SP01), is less than that in the short duration test, (SP03), in the Stress Path Oedometer. This can be supplemented by an interpolated swell value from the standard oedometer test data, reported in Chapter 7, which were obtained for different values of vertical applied stress. The swell to zero suction is shown in Fig.10.18 against the total test duration. The cause of this decrease in swell with test duration is not known, but could be due to particle reorientation or creep occurring at the samt time as the swell. Continued compression or ageing is known in saturated soils, and could be more marked for highly structured compacted soils.

10.5.1.3 Stress Changes

As noted on Fig.10.1, the radial stress change expected during swelling in an oedometer is an increase as the suction is decreased. This can be deduced from the need to prevent the

sample from expanding radially. In a saturated soil this increase will continue until some limiting condition such as shear failure is reached. This is what was expected from the unsaturated samples.

This appears to be what has happened in tests SP01, SP03 and SP04 during the swelling stages. In the first stage of test SP01 there was a large increase in the radial applied stress. This was continued in the second stage but to a lesser extent, but thereafter the radial applied stress decreased. The limiting condition could be either a general shear failure condition, but this would need a very small contribution to the strength from the suction, or a local shear failure condition. The latter can be envisaged as shear within the soil clusters or aggregations near the contact zones as depicted in Fig.10.19.

The final stress state in both SP01 an SP03 after reaching zero suction is similar to the passive limiting condition for a saturated soil. The radial effective stress is larger than the vertical effective stress by a factor of about 2 to 3, with the higher value from the slower test. This value was also obtained after the consolidation and rebound stages in test SP01. The lower value in test SP03 may be linked with the shorter test duration and greater magnitude of swell. Similar magnitudes of stress ratio were obtained in test SP06 during rebound at similar vertical stress values. However, these values may be somewhat suspect due to the friction on the wall of the oedometer.

The friction would in effect add to the vertical stress during swelling. When the vertical stress is large, this error will be small. However, as the vertical stress decreases the error will become more significant. This is evident in test SP06 at zero vertical stress where the radial stress remained at 100kPa. Extrusion of the sample required a vertical stress of 30kPa, which would give a stress ratio of 3.3 which is not wholly unreasonable. The stress ratio for zero vertical stress is unreasonable. It is thought that the data presented by Komornik & Zeitlen (1965) for swell under small vertical stresses may be incorrect for the same reason. They report stress ratios up to about 12 at vertical stresses of about 5kPa.

10.5.2 CONSTANT VOLUME

Test SP05 followed the suction and stress change sequence for a constant volume wetting test followed by decreasing vertical effective stress as shown in Fig.10.7. It was expected from consideration of saturated soil mechanics that the radial and vertical applied stresses would increase as the suction decreased. However, both the axial and the radial applied stresses increased initially and then decreased. The maximum axial applied stress was 355kPa, and the maximum radial applied stress was 416kPa. This cannot be due to a general shear failure condition, as the

deviatoric stress from stage 4 onwards is almost zero, but the axial stress decreased from 355 to 240kPa and the radial stress decreased from 387 to 258kPa in stages 5 & 6. It must be concluded that local shear or particle reorientation is causing the decrease in applied stress. This has been used to describe the collapse process for solid particles with clay bridges by Burland (1965), but needs some modification for the soil in question.

In this soil there is not sufficient sand and silt for the structure to be of the form described by Burland. The clay will be the major constituent, and will not be found only at sand and silt grain contacts. However, considering the structural descriptions of compacted clays given by Zein (1985) and discussed in section 2.2.3, the arguments can be adapted.

For soils compacted dry of optimum, the structure will be at least partially of the clustered type. These clusters possess a shear strength dependent in large part on the suction in the water within them. During compaction they will be deformed until the contact zones can support the local stresses due to the 1MPa compaction stress (Fig.10.20a).

On allowing the suction to decrease by a small amount and the soil to swell, the strength of the soil in the contact zone will also decrease. However, the stress in the contact zone is less than that due to the compaction stress (Fig.10.20b), and the clusters will thus not deform significantly in shear. Swelling of the clusters will take place under the applied stresses without any rearrangement of the clusters. As the suction, and thus the strength of the contact zones, is decreased, the reserve of shear strength in the contact zones will be reduced, until the stress in the contact zone becomes equal to the strength (Fig.10.20c).

Further reduction in the suction will reduce the strength of the soil in the contact zone below the current stress, and distortion or shear must ensue to give a larger contact zone and a lower contact stress. This restores the balance between the applied stress and the strength of the soil in the contact zone (Fig.10.20d).

10.5.3 SHRINKAGE

The effect of increasing the suction was investigated in test SP04 after swelling to 175kPa suction had taken place. The data can be seen in Fig.10.6. During the first stage, to 275kPa suction, the sample continued to swell, probably because the creep swell rate exceeded the shrinkage rate. This was reversed in the next stage which saw the start of shrinkage.

The rate of shrinkage with regard to suction change continued to increase and between 475 and 625kPa was approximately two thirds of the rate of swell in test SP01 in the same suction range. Since the radial applied stress was decreasing during this part of the test, it may be more appropriate to consider the

change in mean applied stress as well as the change in suction. This brings the comparison between the rate of swell in test SP01 and shrinkage in test SP04 closer. The rate of change of void ratio in test SP01 was $93*10^{-4}$ per kPa, while the rate in test SP04 was $80*10^{-4}$ per kPa. This reversabilty of the swelling and shrinkage suggests that over these relatively small suction changes the processes are reversible. Structural changes are not expected under the isotropic suction increases, which would serve to increase the strength in the contact zones. This was also evident when the suction in test SP04 was once more decreased to 175kPa as shown by the small difference between DEF and FG in Fig.10.6.

The shrinkage was not continued past 625kPa suction because the radial applied stress would have become zero, and the sample would have separated from the oedometer wall. Although the stress state would still have been defined, the volume change would no longer have been defined by the vertical strain.

10.5.4 COMPRESSION

Compression stages were undertaken in tests SP01, SP04, SP06 and SP07. The loading in test SP01 was done after attaining zero suction. Figure 10.4 shows that the sample compressed more per kPa of applied stress than it swelled due to the combined effects of suction and applied stress changes. Although this could be analysed purely in terms of the factor "chi" to determine the equivalent effective stresses resulting from the suction, it is thought that this is not helpful in understanding the response of the soil to the applied stress.

There has been considerable discussion above regarding the structure of the soil and shear in the contact zone. Once again we must turn to the observations of Zein (1985), who found that the clustered structure was only partially converted to a preferred parallelism structure under low applied stresses in swell under load tests. However, in swell pressure tests he noted that the preferred parallelism structure was developed. It is thus reasonable to argue that the greater compressibility of the sample in test SP01 is due to the structural changes which occur progressively as the stress increases and modify the structure from clustered to preferred parallel.

Further support for this is available by comparing the compression stage of SP01 with the data of test SP06, as shown in Fig. 10.21. The compression stage of test SP01 plots as a stongly concave upward line but is below the "virgin" compression line in test SP06. Since it lies below the virgin compression line, the soil should be overconsolidated, and the recompression line should then be concave down as is the case for the reloading part of test SP06 at a mean stress below 100kPa as shown in Fig.10.8. Since the compression curve for test SP01 is concave upward, there must some

structural difference between the samples.

It is postulated, but cannot be demonstrated within the data gathered in this project, that the clusters behave as overconsolidated soil, and that the compressibility of the whole sample is due to the clusters being deformed into the macro-voids between the clusters. The void ratio of the clusters will be less than the void ratio of the whole sample, and may be reflected by the point at which the extrapolated compression line from the test SP01 approaches the extrapolated virgin compression line at point "A" in Fig.10.21.

The void ratio and stress changes that occurred with time during the compression stages in test SP01 are shown in Fig.10.22, and the stress path for this stage of the test are plotted in Fig.10.23. Fig.10.22a shows the changes in void ratio against the square-root of time, and Fig.10.22b shows the radial stress against the same horizontal axis. It had been expected that the total radial stress would decrease during each increment of axial stress as the excess pore water pressure dissipated, and would then remain constant. However, in Fig.10.22 it is particularly evident in stages 8 & 9 that the radial total stress first decreases and then increases.

The stress path depicted in Fig.10.23 shows the inferred end of primary consolidation stress state at the points labelled "A". The increase in the radial stress with time causes an increase in the mean stress and a decrease in the deviator stress along a line of constant axial stress to points "B". The duration of the stages varied, and points are plotted along the AB lines at varios arbitrary intervals of time. Stages 11 & 12 did not show significant changes in the radial stress with time and points A and B are coincident. The unloading stages did not show significant time dependent stress changes after the inferred end of primary swelling.

Since the back pressure system was by this stage of the test far from deaired, no pore water pressure measurements were made. However, compression-root time curves for this test and test SP06 shown in Fig.10.22 indicate that primary consolidation is complete within 1 day. Test SP06 did not show the same increase in radial stress, now an effective stress since consolidation was complete, after the initial decrease in total stress as the pore water pressure decreased.

The difference between the radial stress responses in tests SP01 and SP06 is believed to reflect the different soil structures in the two samples. The sample in test SP06 was prepared as a slurry and is expected to have an essentially homogeneous structure in which all particles are in equilibrium with their adjacent particles. The sample in test SP01 has not been subjected to large compressive strains, and little change in structure will have occurred. The soil will be non-homogeneous, and there will be mocro-voids between groups or clusters of particles. The time

dependent stress changes are thought to be due to creep or viscous movement of particles leading to a modification of the structure to a more homogeneous structure. This "flow" within the sample would be consistent with the observed change with time of the stress state toward the isotropic state.

Compression stages were also undertaken in tests SP04 and SP07 at suctions of 175kPa and 900kPa. The results of these and the compression stages of tests SP01 and SP06 are shown in Fig.10.24. The lower part of the figure shows the void ratio changes in terms of the mean applied stress and the mean applied stress plus the suction. The upper part of the figure shows the coefficient of compressibility, calculated in terms of the vertical applied stress, plotted against the same horizontal axes. Test SP06, on a normally consolidated sample, shows a much higher compressibility than the other tests. The effect of compression at non-zero suction is demonstrated in both tests SP04 and SP07. Both show very stiff responses, but test SP04, at the lower suction, shows a greater increase in the compressibility above 150kPa vertical stress than does test SP07.

10.5.5 COLLAPSE

The final stage of test SP04 was a decrease to zero suction under a vertical applied stress of 200kPa. This stage was somewhat uncontrolled in that the suction decrease was caused by leakage of the loading membrane and complete loss of the suction took place within about 12 hours. Since this happened at night, no readings were taken, and only the end point is known. As shown in Fig.10.6, this penultimate stage involved a large decrease in the sample void ratio due to the decrease in the suction. This is generally described as collapse, and was investigated by Maswoswe (1985) for a low plasticity soil.

The collapse in this part of the test was not unexpected because the vertical applied stress had been increased after the first swelling stage. The soil in the contact zones of the clusters was thus stressed to a higher state than it had been after the swelling, and the decrease in strength due to the reduction in suction could be expected to lead to collapse. It does serve however to confirm that a "collapsible" structure did exist in these samples, and justifies further the arguments used regarding the stress decreas as the suction decreased in tests SP01 and SP05.

It was noted above that it was thought that collapse potential could be identified from the field void ratio and stress state compared with the remoulded or virgin compression line. The data presented above is in conflict with this.

Fig.10.8 showed that the compacted states of the samples, plotted in total stress terms, fall below the virgin compression line. This should, according to Burland (1985) indicate that

collapse cannot occur. However, collapse has occurred in test SP04 without crossing the virgin compression line to the zone in which collapse could be expected.

It appears from the arguments above on the compression stage of test SP01 that the clusters continue to behave as overconsolidated soil elements during and after swelling to zero suction, and thus the soil remains below the virgin compression line.

The use of the normally consolidated line or the destructured soil compression line does not appear to be reliable for all soils.

10.6 REVIEW OF STANDARD TEST PROCEDURES

One of the applications of the test results described in this chapter is the ability to compare the stress paths during the different standard swelling tests. These were described briefly in section 2.3.1 above.

Test SP01 follows the Holtz & Gibbs (1956) swell under load sequence up to the end of stage 6. At this point the suction had been reduced to zero and the vertical applied stress had been maintained at 50kPa. Thereafter the vertical stress was increased as it would be in a Jennings et al (1973) "single oedometer" test. Test SP05 followed the sequence of suction and stress changes for the Sullivan & McClelland (1969) swell pressure then rebound type of test.

As noted in Schreiner (1987e) and Schreiner & Burland (1987) which is included as Appendix 10.A1, test SP01 can be used to describe both the swell under load test and the swell then consolidate or single oedometer test. For the former, the stress path is simply that observed in stages 1 to 6, AB in Fig.10.25 and the final stress state has the radial stress much greater that the vertical stress.

If the axial applied stress during swelling in test SP01 had been 1kPa, then the radial stress after swelling would have been about 3kPa to correspond to the passive limiting condition at C. The consolidation stress path that would be followed during consolidation is represented by CD in Fig.10.25. At the same vertical stress used in the swell under load test, the stress state in the swell then consolidate test is seen to be similar to the normally consolidated stress state, with the axial stress greater than the radial stress. The stress paths followed by these two test procedures, and the final stress states can be seen to be different. For the compacted soil used in this project, the effect of the stress path is likely to be of considerable significance, particularly due to the irreversible structural changes that can occur.

The stress path followed in the constant volume then rebound

test, SP05, can be seen from AEF in Fig,10.25 to differ from both the above stress paths, although the final stress state at F is similar to that for the swell under load test at B. Here again the effect of the stress path on the structured soil is likely to affect the final void ratio as compared with the swell under load test.

Fredlund et al (1980) and Sullivan & McClelland (1969) proposed that swelling due to a decrease in suction could be determined from the constant volume soaking then rebound test. This avoids the need to determine or control the suction. However, no test data was provided to justify this proposal. The above arguments and test data can be used to examine this approach.

The test procedure used by Sullivan & McClelland (1969) is the same as that of test SP05 in that the rebound was done at the end of the swell pressure stage. The test procedure which Fredlund et al (1980) propose differs slightly from that used in test SP05. Their procedure is similar to that proposed by Burland (1962) for the double oedometer test in that the rebound curve is taken from a high vertical stress. They recommend that the vertical stress is increased to 1700kPa after the constant volume stage before the sample is unloaded in stages. This rebound curve is then transferred to the end of the constant volume stage corrected for sample disturbance.

Swelling to zero suction using the Fredlund et al (1980) technique is estimated from this rebound curve for the appropriate value of vertical stress as shown in Fig.10.26. The rebound curve obtained in this way cannot reasonably be argued to be steeper than that obtained without the loading to 1700kPa. However, considering the data plotted in Fig.10.27 for tests SP01 and SP05 on the same axes as Fig.10.26 and adjusted to a common initial void ratio, it can be seen that the swell in test SP05 was about 1/10th of that in test SP01.

Thus it is readily apparent that neither Fredlund et al's (1980) method nor Sullivan & McClelland's (1969) method is applicable to compacted soil. The claim that the two stress paths are equivalent does not hold. This does not demonstrate that either is inapplicable to undisturbed soil profiles where the soil structure is stable.

For a soil with a stable structure, which would be expected from an "undisturbed" natural sample or a natural soil in situ, the above arguments regarding the structure may not be valid. In this case the stress path will be of lesser importance, and the swell under load test and the constant volume then rebound test may well give similar results because the final stress states are similar. The swell then consolidate test will still give different results at the same axial stress because the final stress state will be different.

10.7 CONCLUSIONS

10.7.1 STRESS PATHS IN STANDARD TEST PROCEDURES

During the literature review for this investigation, reported in Schreiner (1987a) and Chapter 4, it was found that no data existed which defined fully the stress state during swelling of unsaturated samples. Various claims have been made by various authors as noted in section 2.2.2 regarding the influence of the stress path on the final volume change, although the stress paths had not been defined.

Some stress paths have now been observed for a particular set of test conditions and for one soil type. Several conclusions have been reached regarding the standard oedometer test procedures described in section 2.3. It must be noted that these conclusions are only applicable to compacted samples.

Firstly, it has been shown that the radial applied stress in both the Sullivan & McClelland (1969) procedure and in the Holtz & Gibbs (1956) procedure increases initially, and then decreases as the suction is reduced further.

Secondly, the stress states at the end of these two tst procedures, i.e. after unloading to 50kPa in the former and after swelling under 50kPa in the latter, are similar and that the radial stress is greater than the axial stress. However, for the samples tested, which were compacted at a moisture content dry of the optimum moisture content, the final void ratios were different. These two test procedures are therefore not equivalent for compacted samples, even though the final strss states are similar.

Thirdly, the stress path for the Jennings et al (1973) test procedure can be deduced from test SP01. This demonstrates that the stress path and the final stress state for this procedure differ from those of the Sullivan & McClelland (1969) and the Holtz & Gibbs (1956) procedures. The three test procedures are all significantly different from each other in stress path.

Since the stress paths have been shown for two of these test procedures to have a significant effect on the change in void ratio for the compacted samples, the standard procedures should not be applied to compacted soils. Consideration should be given to the stress and suction changes that are expected in the field, and the test in the laboratory should attempt to simulate these changes.

10.7.2 SWELL AND COLLAPSE

Both swell and collapse have been observed in the same sample. The effect of the collapse mechanism was seen in test SP04 as a decrease in the void ratio on decreasing the suction to zero in stage 12. This is the type of test result commonly associated with collapse. It has also been seen in the stress path oedometer test

as a decrease in the axial and radial applied stresses in the constant volume stages of test SP05 during decreasing matrix suction.

The final void ratio and mean stress were found to lie below the virgin compression line obtained in test SP06. This is in conflict with the observations of Maswoswe (1985) and Burland (1985) that collapse ends on or near the soaked compression line.

10.7.3 SOLUTE SUCTION

The change in solute suction was observed qualitatively in two tests, SP01 and SP03, by measurement of the resistance of the water in the back pressure system. Associated with the inferred decrease in solute suction due to the movement of dissolved salts from the soil to the water in the back pressure system was an increase in the radial applied stress and the void ratio. This has demonstrated that the solute suction cannot be ignored in volume change testing of expansive soils. Possibly more importantly, it confirms that the capillary model, which is insensitive to dissolved salts, is not an appropriate model for the swelling of highly plastic clays. Swelling can only be studied in terms of the physico-chemical models of clay-water interaction.

TEST NO.	STAGE																	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
SP01																		
point	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
σav	50	50	50	50	50	50	100	200	300	400	492	600	300	100	50	200	400	495
Uaw	-900	-700	-500	-300	-100	0	0	0	0	0	0	0	0	0	0	0	0	0
SP02																		
σav	failed																	
SP03																		
point	B	C																
σav	50	50																
Uaw	-200	0																
SP04																		
point	B	-	C	D	E	F	G	H	I	J	K	L	M					
σav	50	50	50	50	50	50	50	50	100	150	200	200	10					
Uaw	-175	-225	-275	-375	-475	-625	-325	-175	-175	-175	-175	0	0					
SP05																		
point	B	C	D	E	F	G	H	I	J	K	L							
σav	273	327	340	355	330	240	210	150	100	50	0							
Uaw	-900	-700	-500	-300	-100	0	0	0	0	0	0							
SP06	1st cycle																	
point	B	C	D	E	F	G	H	I	J	K	L							
σv	10	20	40	80	160	120	80	40	20	10	0							
	2nd cycle																	
point	M	N	O	P	Q	R	S	T	U	V								
σv	50	100	160	320	640	320	160	80	40	0								
SP07																		
point	B	C	D	E														
σav	50	100	150	200														
Uaw	-900	-900	-900	-900														

Key:

σav = vertical applied stress

Uaw = Pore water pressure referenced to atmospheric pore air pressure.

TABLE 10.1 SUMMARY OF STRESS CHANGES

Test No.	Stage																		
	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
SP01																			
point	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
q_o	-81	-299	-306	-284	-255	-200	-118	-68	62	139	159	176	220	-5	-86	-118	33	145	211
p_o	104	249	254	239	220	183	129	145	159	207	294	375	453	303	157	129	222	303	354
p_o-U_{aw}	1044	1149	954	739	520	283	129	145	159	207	294	375	453	303	157	129	222	303	354
e	1.285	1.331	1.345	1.365	1.387	1.417	1.439	1.435	1.404	1.339	1.279	1.246	1.203	1.217	1.236	1.249	1.243	1.224	1.214
w	34.2	38.3	41.1	43.9	51.4		54.3	54.1	53.0	50.5	48.3	47.0	45.4	45.9	46.6	47.1	46.9	46.2	45.8
SP03																			
point	A	B	C																
q_o	-40	-160	-59																
p_o	76	157	89																
p_o-U_{aw}	1017	357	89																
e	1.259	1.386	1.435																
w	34.1		52.5																
SP04																			
point	A	B	-	C	D	E	F	G	H	I	J	K	L	M					
q_o	-57	-132	__	-103	-64	-31	4	-100	-139	-49	13	34	25	-119					
p_o	88	138	__	119	93	71	47	117	142	132	141	177	183	80					
p_o-U_{aw}	1028	313	__	394	468	546	672	442	317	307	316	352	158	80					
e	1.291	1.423	__	1.430	1.429	1.426	1.416	1.427	1.433	1.432	1.430	1.421	1.394	1.455					
w	34.3	42.2	__	41.9	41.5	40.8	40.1	40.3	40.7	__	__	__	__	45.9					
SP05																			
point	A	B	C	D	E	F	G	H	I	J	K	L							
q_o	-78	-132	-89	-50	-32	-30	-13	-48	-101	-119	-160	-160							
p_o	105	361	386	373	376	350	248	242	217	179	156	107							
p_o-U_{aw}	1005	1261	1086	873	676	450	248	242	217	179	156	107							
e	1.301	1.301	1.301	1.301	1.301	1.301	1.299	1.300	1.307	1.317	1.327	-							
SP06																			
	1st cycle																		
point	A	B	C	D	E	F	G	H	I	J	K	L							
q'	0	0	14	27	49	94	56	21	-10	-25	-26	-30							
p'	0	10	11	22	47	97	83	66	47	37	27	20							
e	3.600	3.261	2.859	2.637	2.319	1.974	1.979	1.992	2.017	2.041	2.062	2.091							
w	136	123	108	99.5	87.5	74.5	74.7	75.2	76.1	77.0	77.8	78.9							
	2nd cycle																		
point		M	N	O	P	Q	R	S	T	U	V								
q'		10	57	99	170	290	44	-49	-88	-97	-101								
p'		43	62	94	206	447	290	193	139	105	67								
e		2.067	2.023	1.959	1.687	1.413	1.444	1.487	1.510	1.562	1.624								
w		78	76.4	73.9	63.7	53.3	54.5	56.1	57.0	58.9	61.3								
SP07																			
point	A	B	C	D	E														
q_o	-73	-210	-165	-123	-86														
p_o	99	190	210	232	257														
p_o-U_{aw}	1038	1090	1110	1132	1157														
e	1.346	1.408	1.407	1.405	1.400														

Key:

$q_o = \sigma_{av} - U_{aw}$

$p_o = (\sigma_{av} + 2U_{aw})/3$

TABLE 10.2 SUMMARY OF TEST DATA.

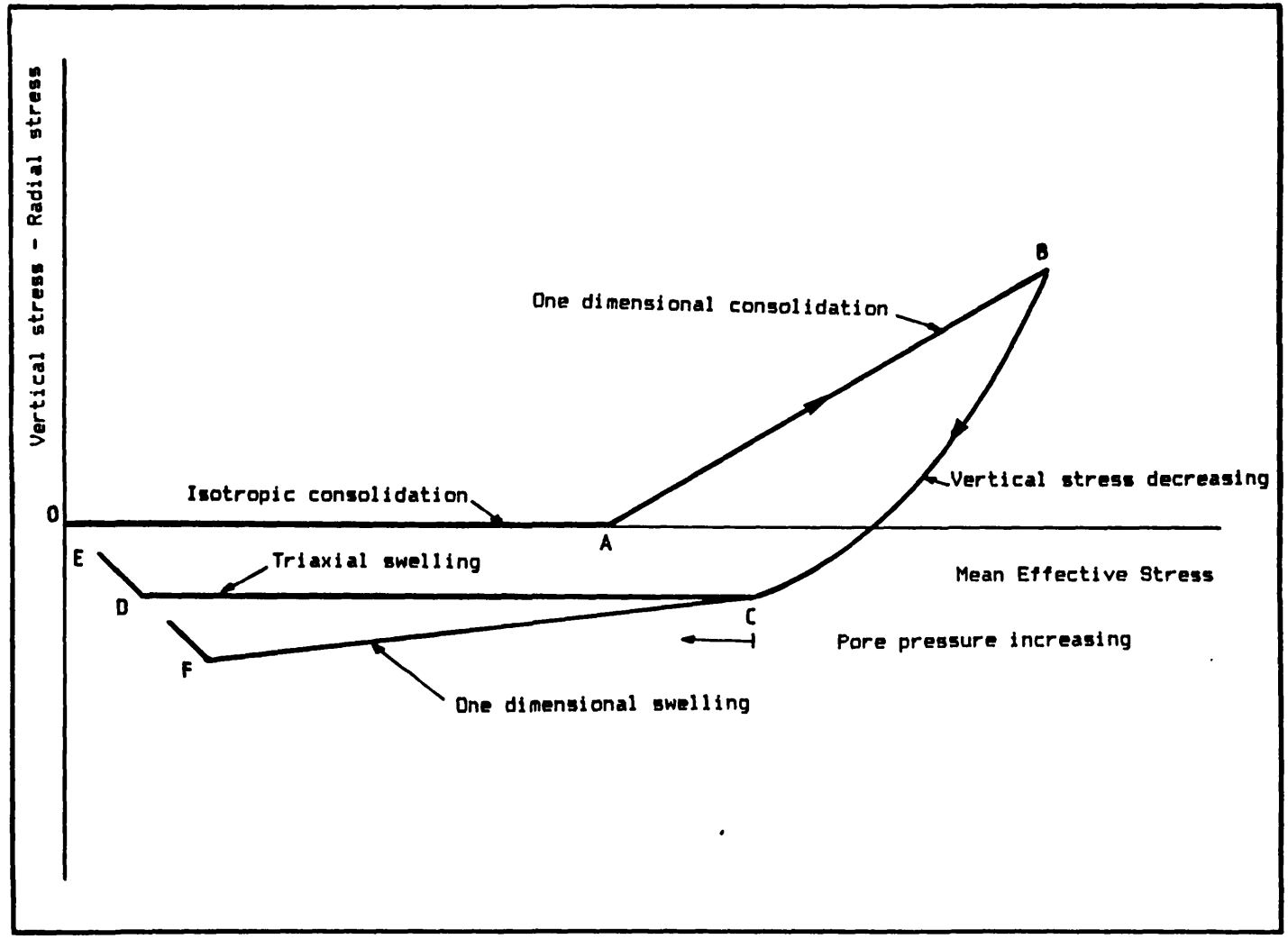


Fig.10.1 Schematic Stress Path

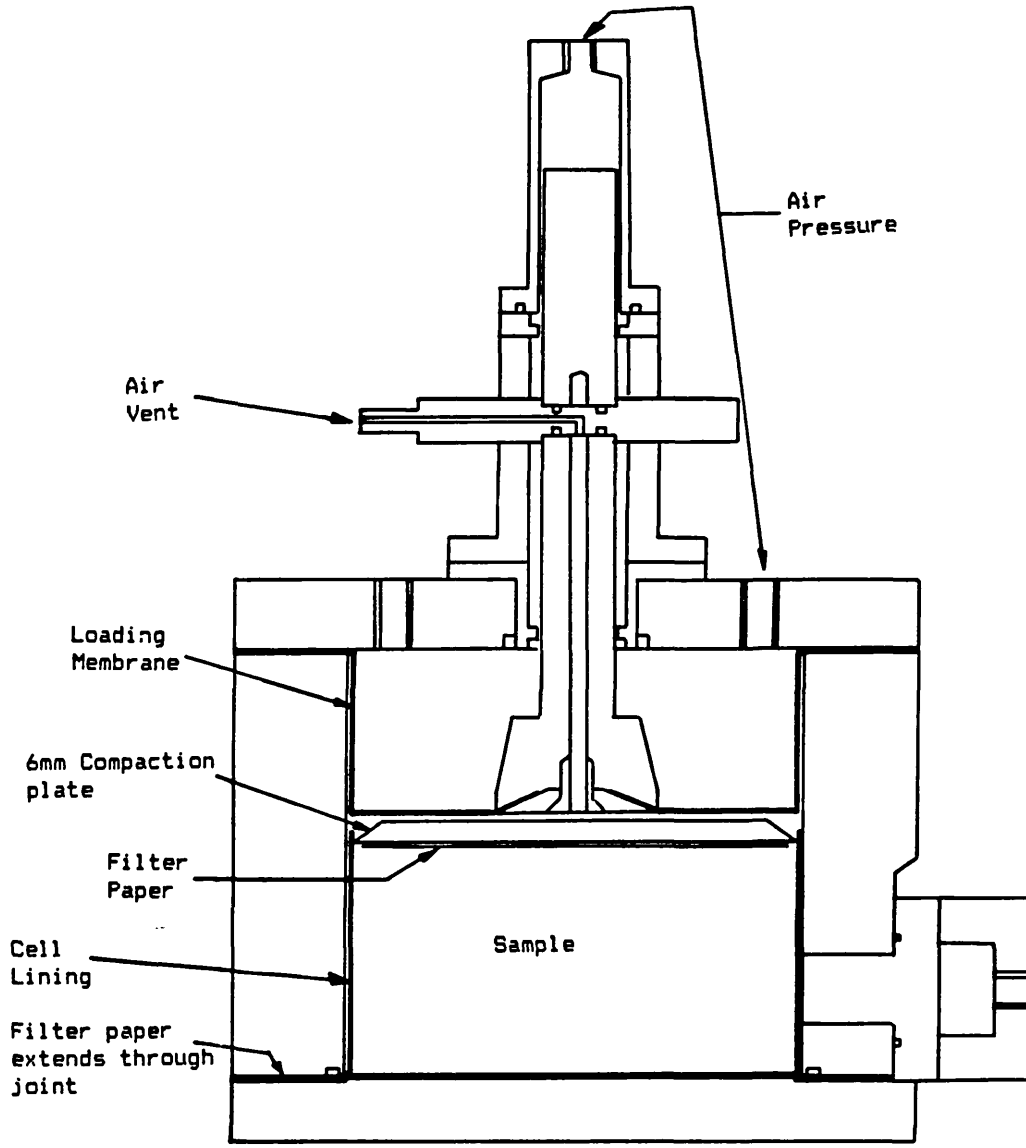


Fig.10.2 Oedometer assembled for compaction

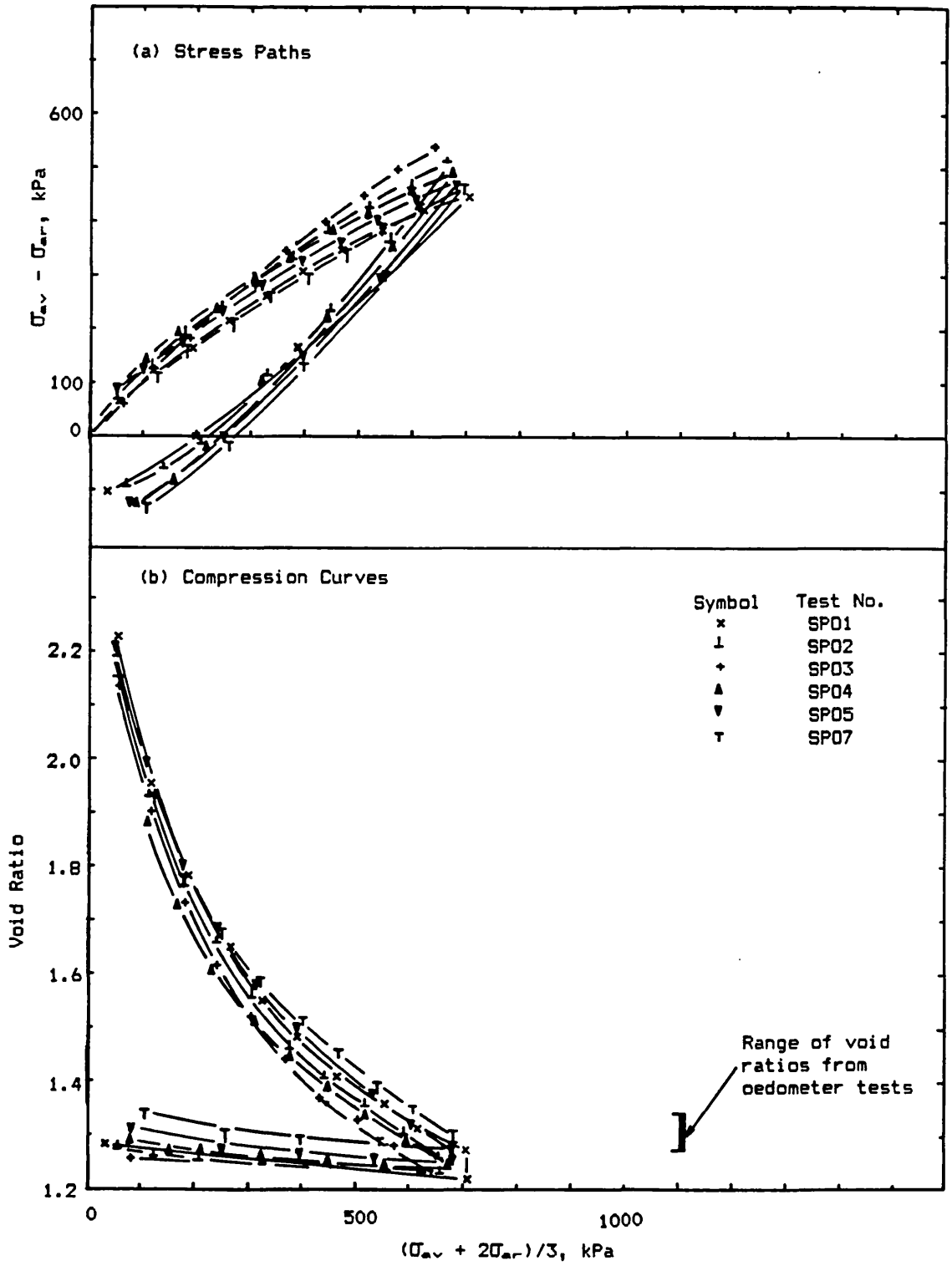


Fig.10.3 Results of Compaction in Stress Path Oedometers

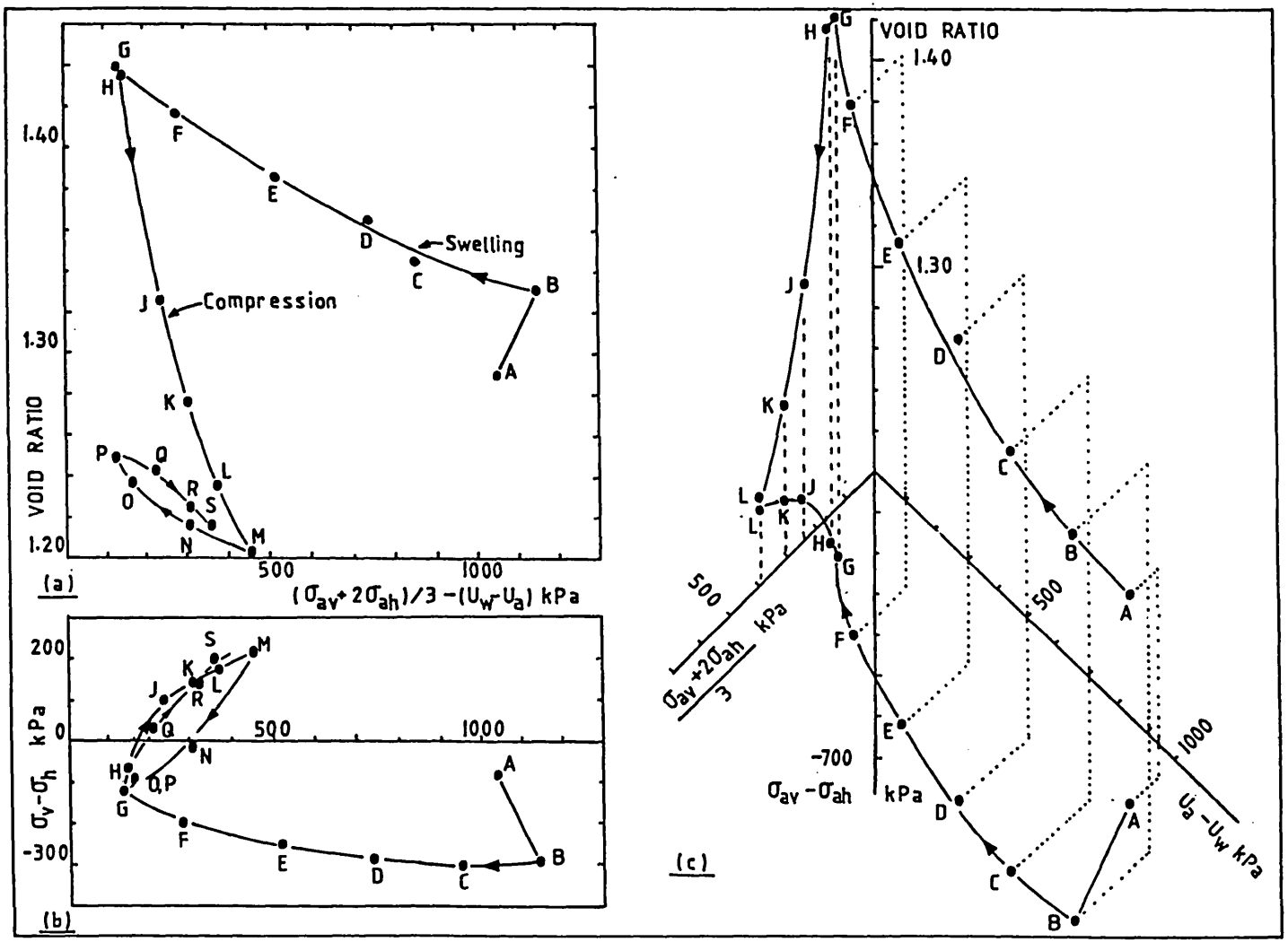


Fig.10.4 Stress Path Oedometer Results, Test SP01

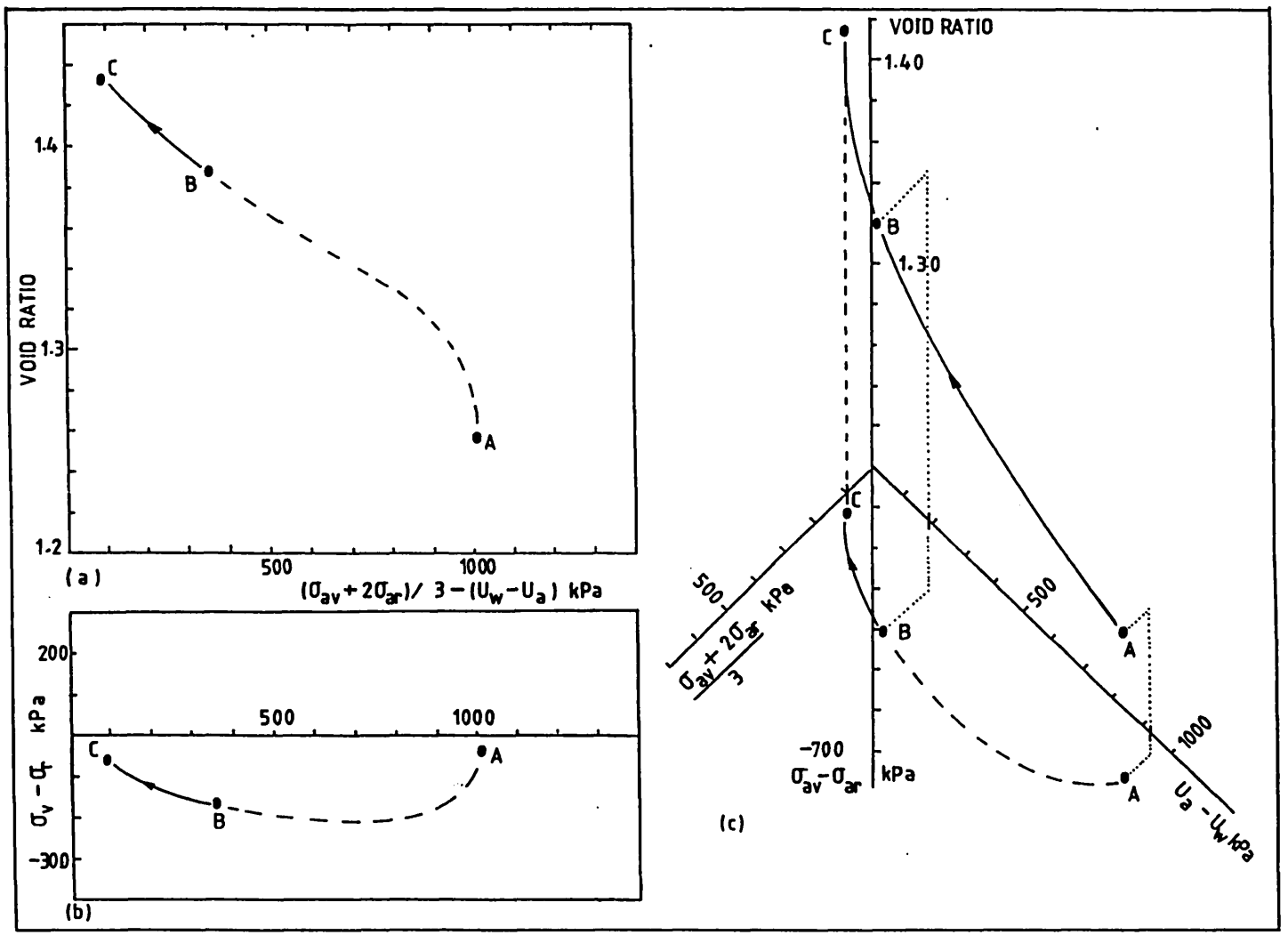


Fig.10.5 Stress Path Oedometer Results, Test SP03

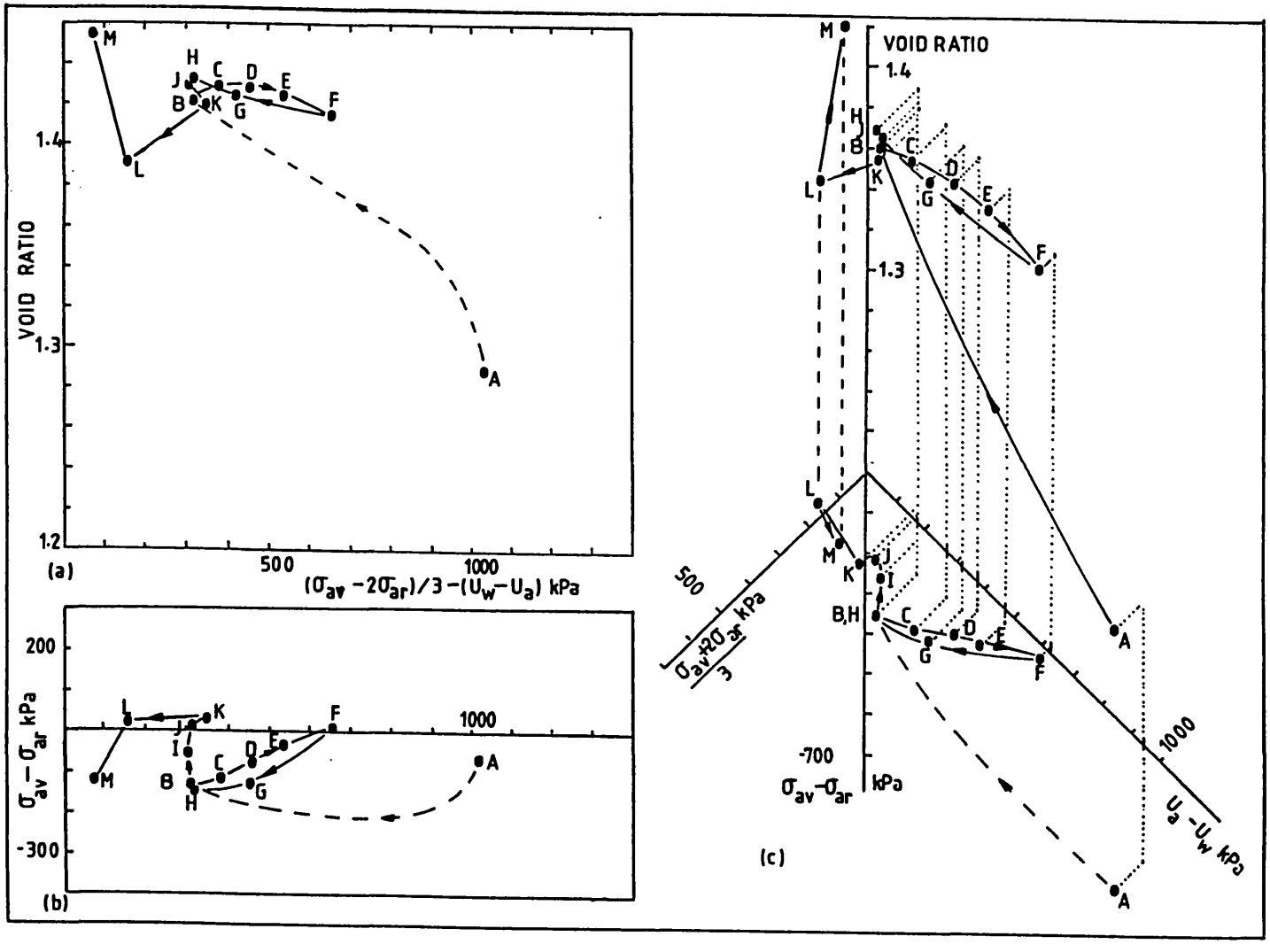


Fig.10.6 Stress Path Oedometer Results, Test SP04

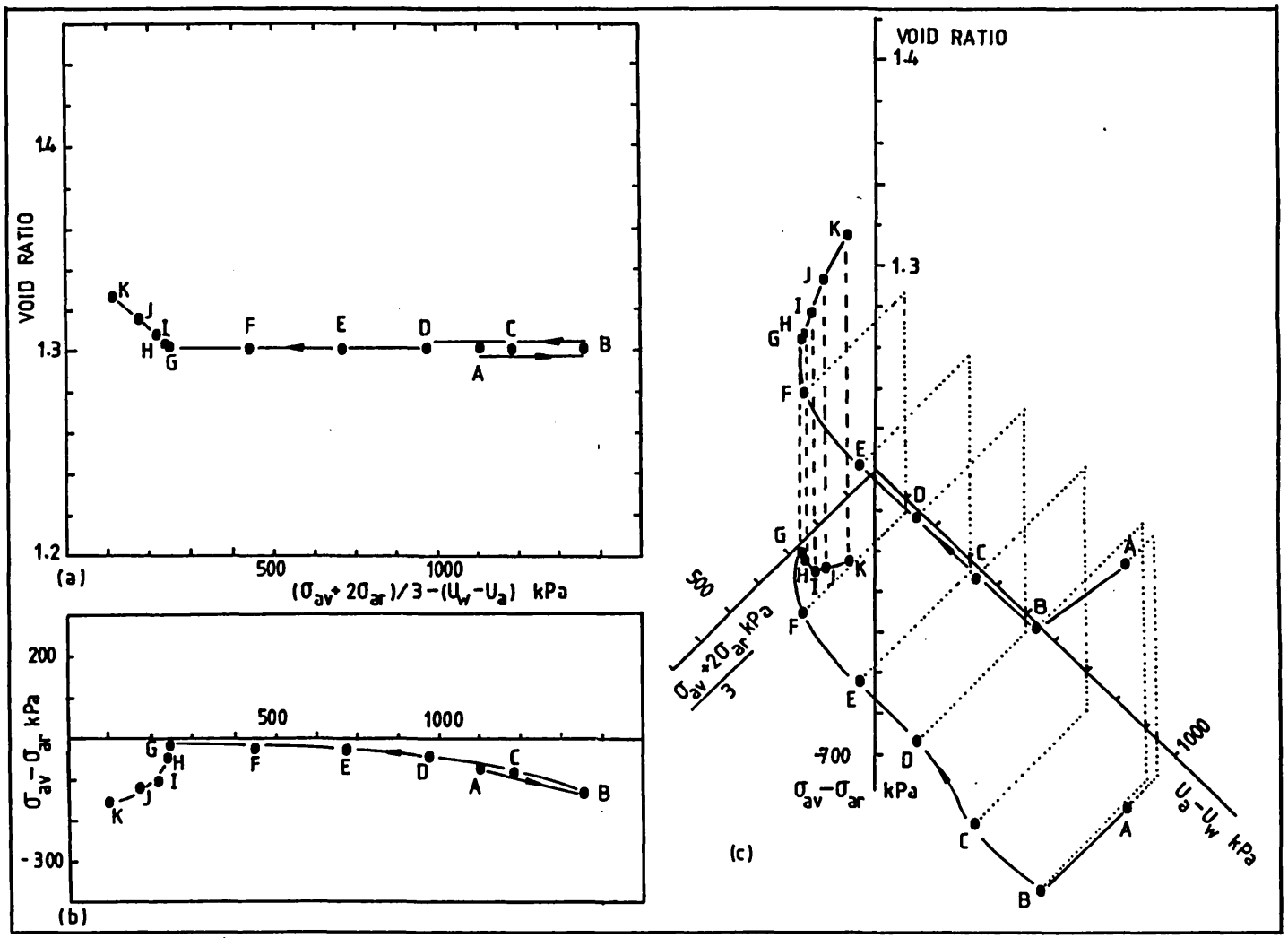


Fig.10.7 Stress Path Oedometer Results, Test SP05

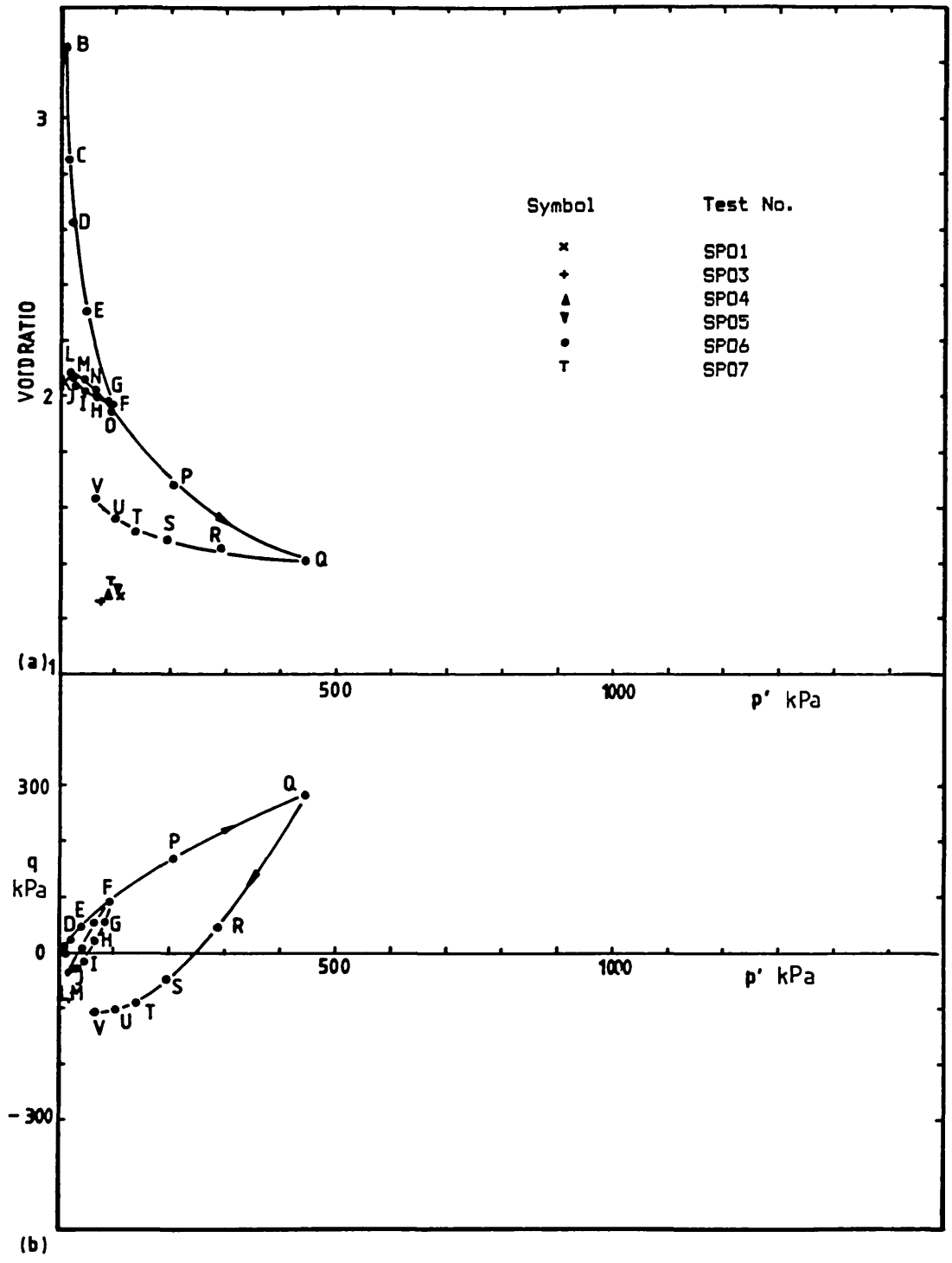


Fig.10.8 Stress Path Oedometer Results, Test SP06

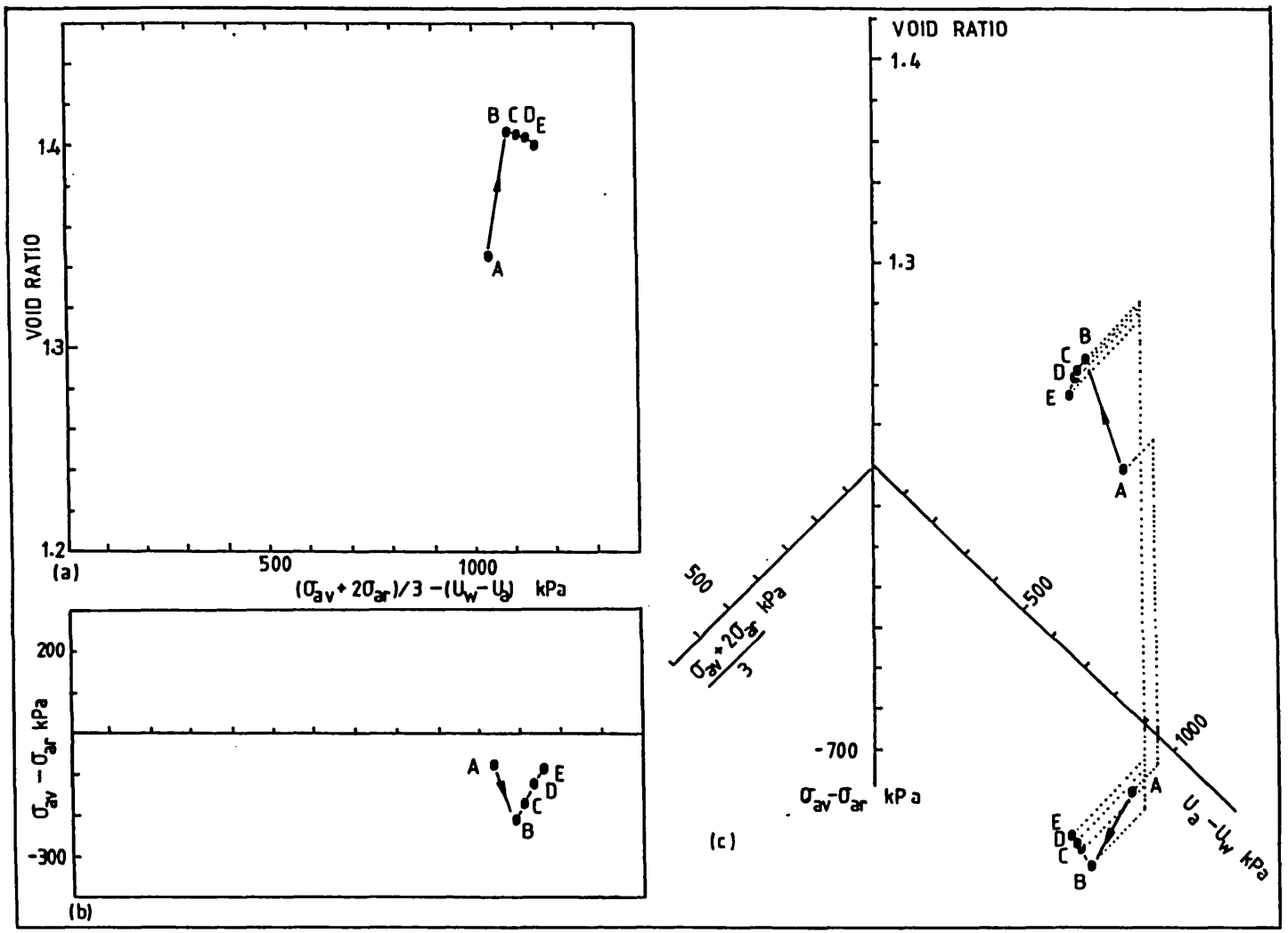


Fig.10.9 Stress Path Oedometer Results, Test SP07

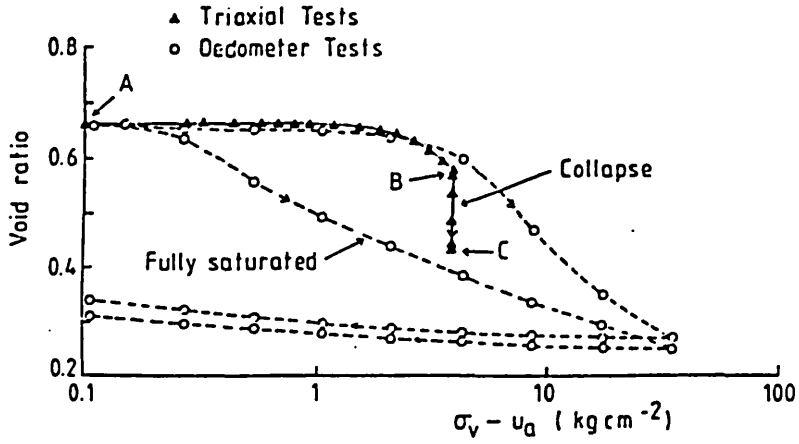
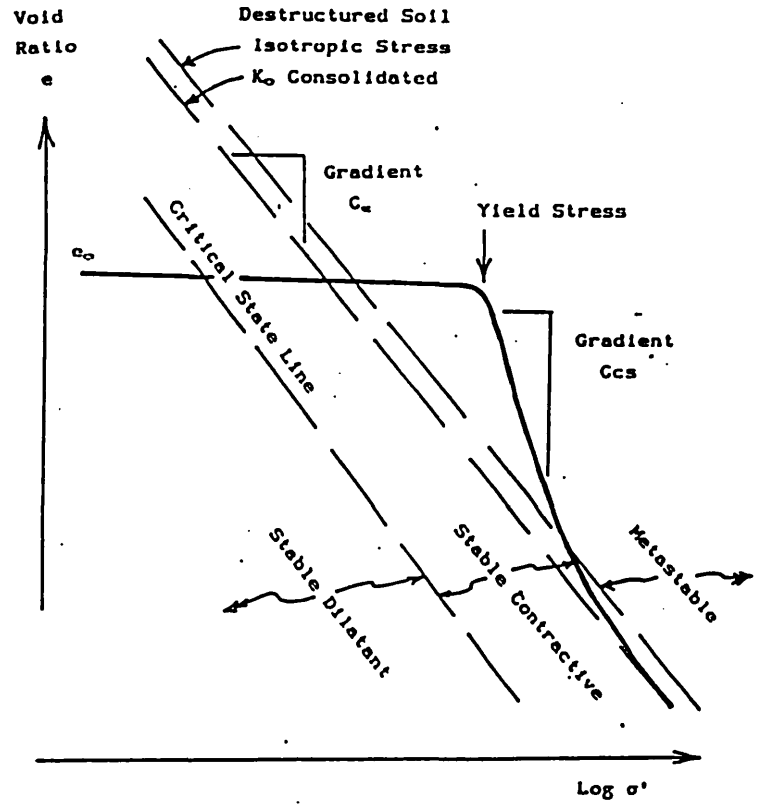


Fig.10.10 - Comparison of triaxial test with equivalent oedometer tests on partly saturated and fully saturated soil.

(Burland 1985)



Note that the gradient, C_{cs}, is a function of E_o and yield stress, and is not an intrinsic material property, as is C_c.

Fig.10.11 Stress-void ratio state of a residual soil related to the states possible for the de-structured soil.

(Vaughan 1988)

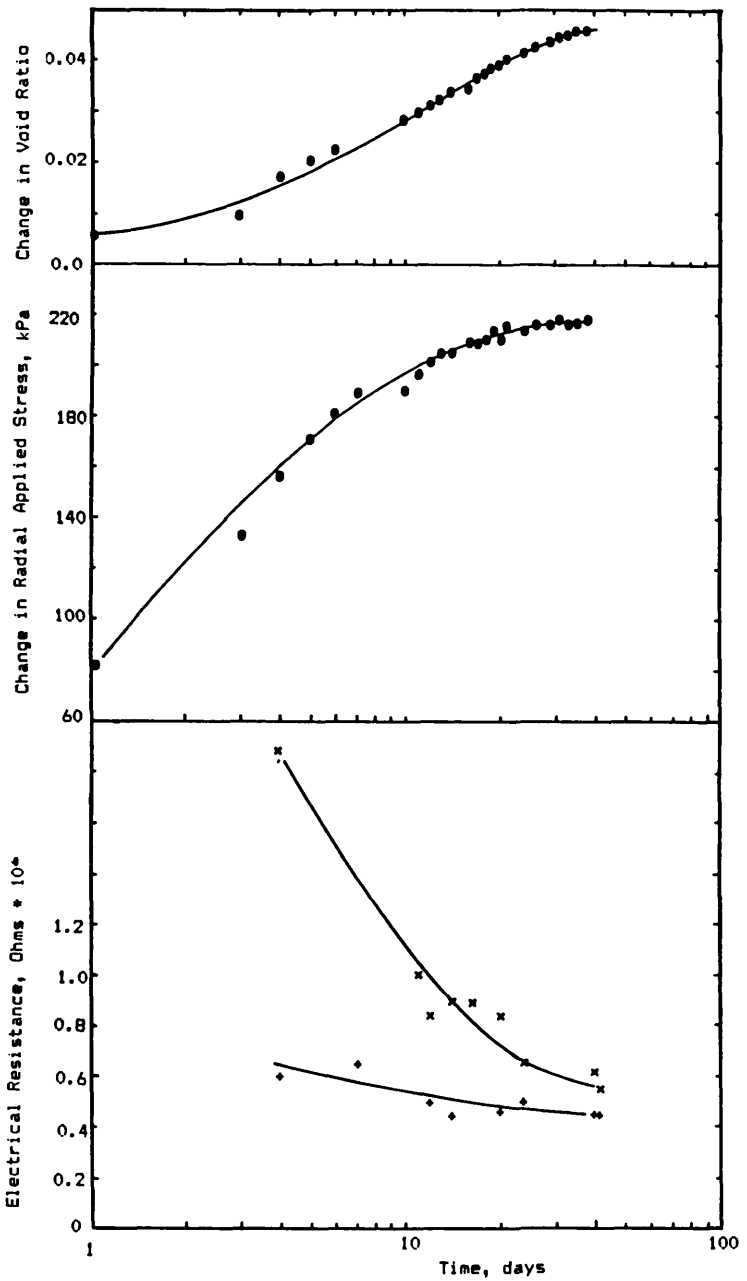


Fig.10.12 Measurement of Electrical Resistance of Water, Test SP01

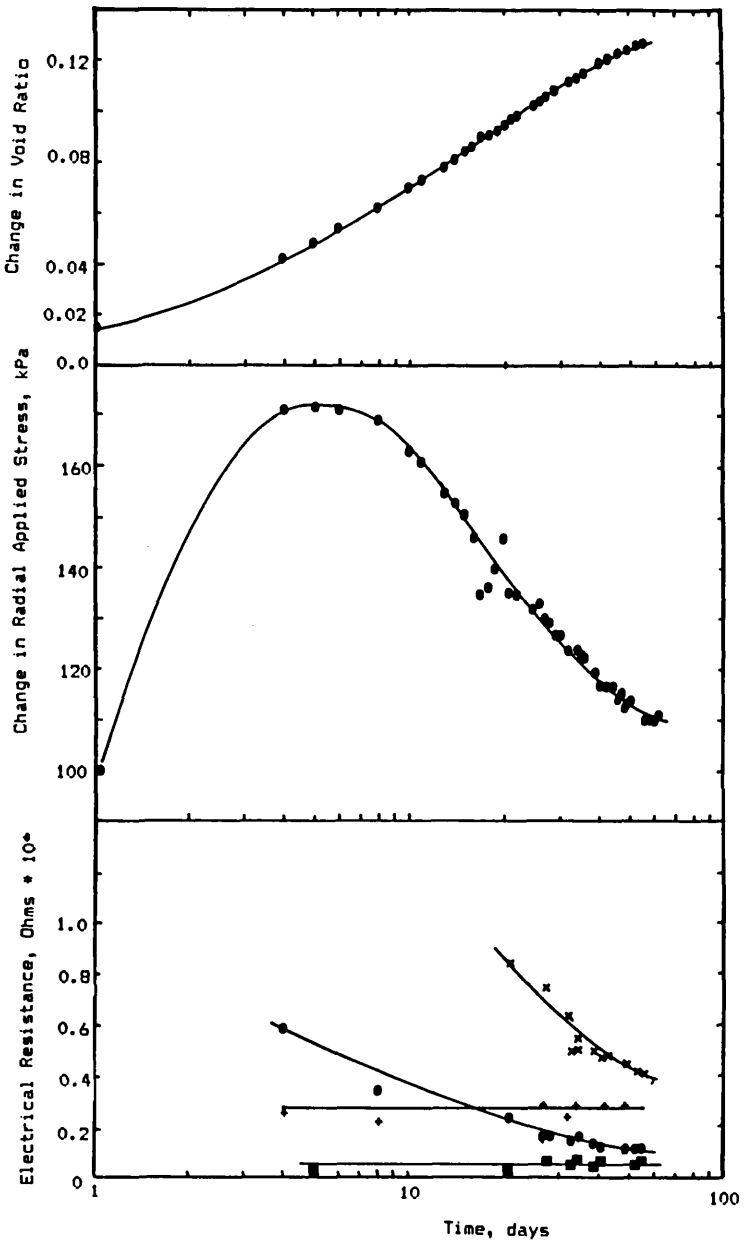


Fig.10.13 Measurement of Electrical Resistance of Water, Test SP03

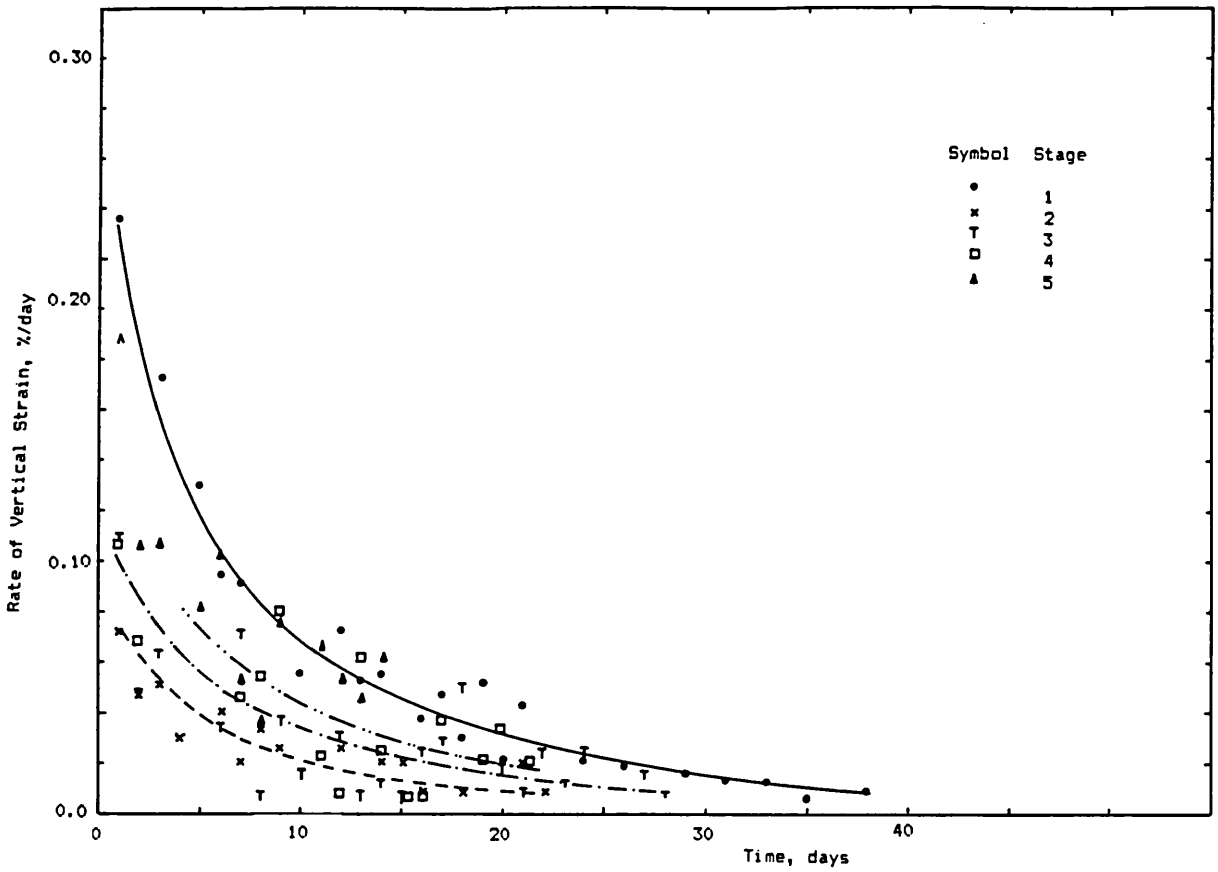


Fig.10.14 Rate of Vertical Strain, Test SP01

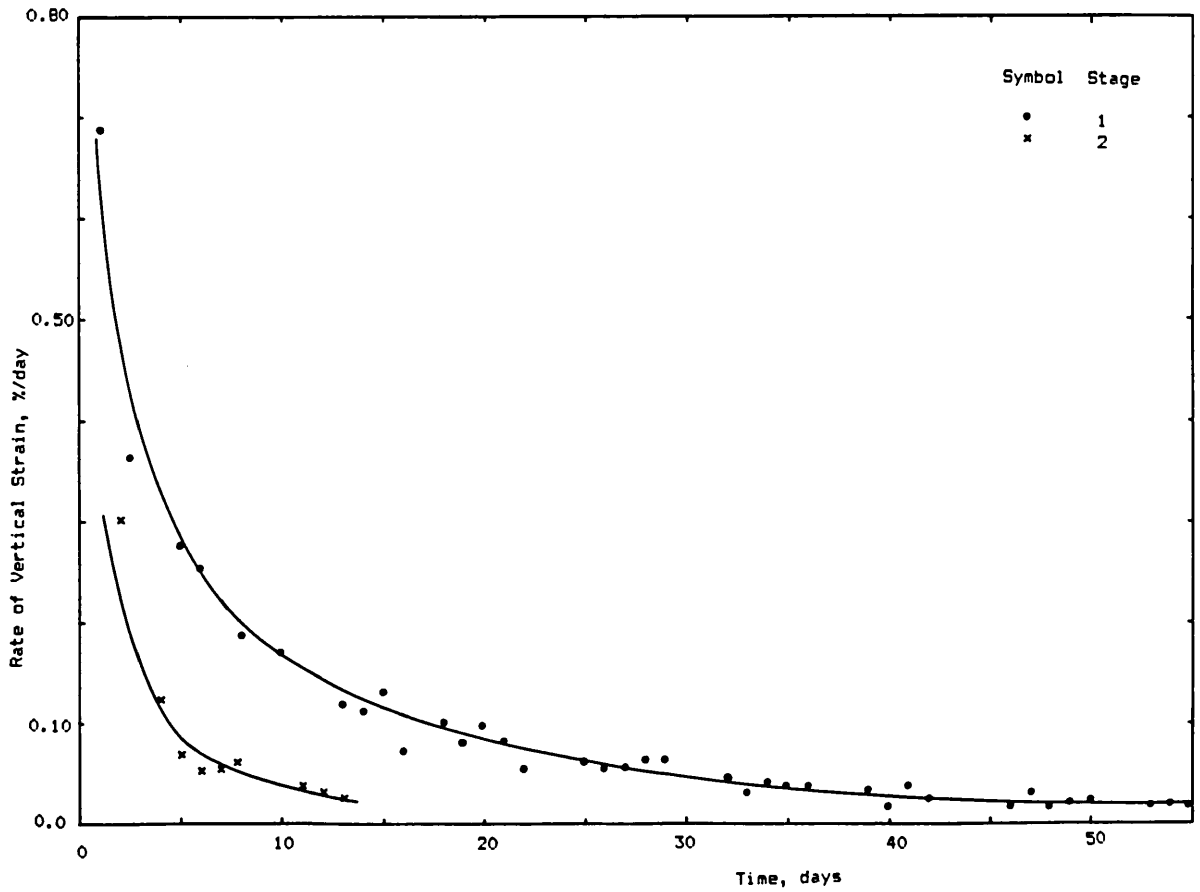


Fig.10.15 Rate of Vertical Strain, Test SP03

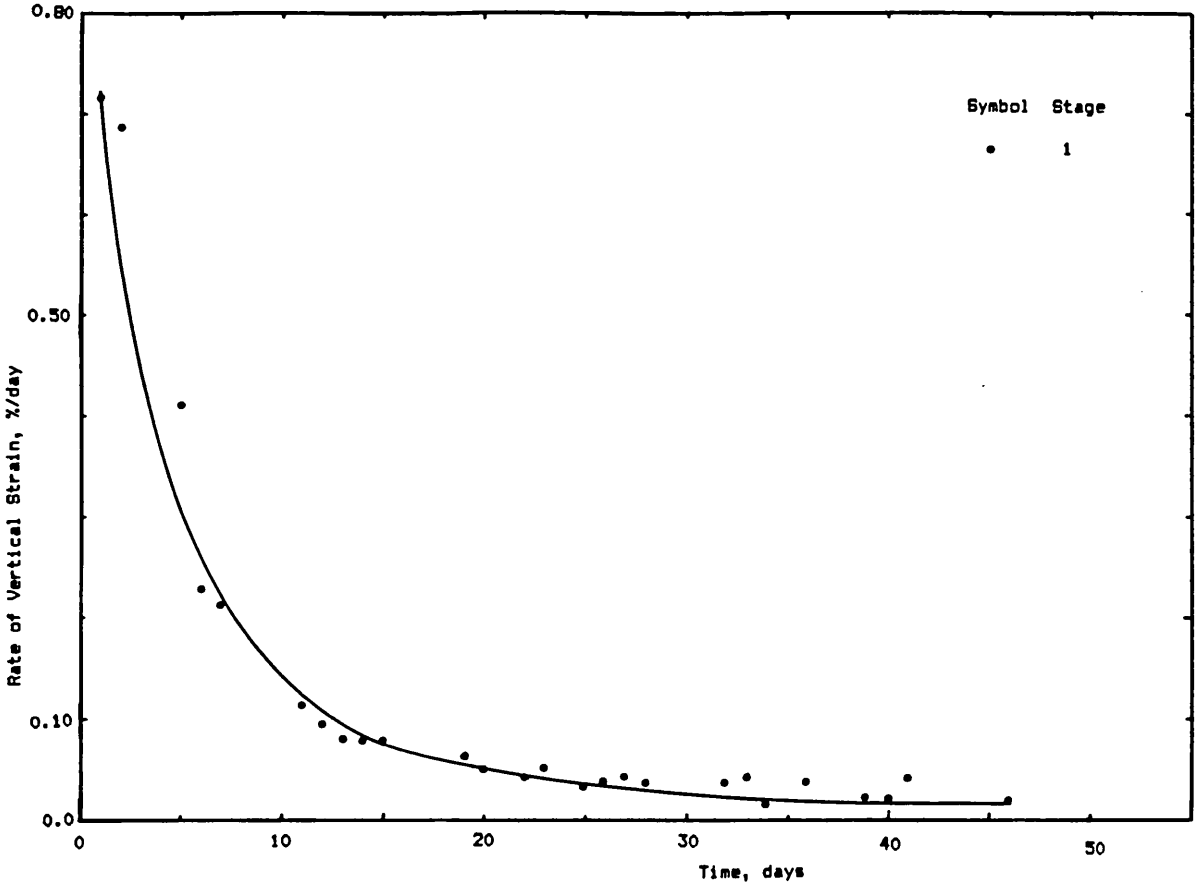


Fig.10.16 Rate of Vertical Strain, Test SP04

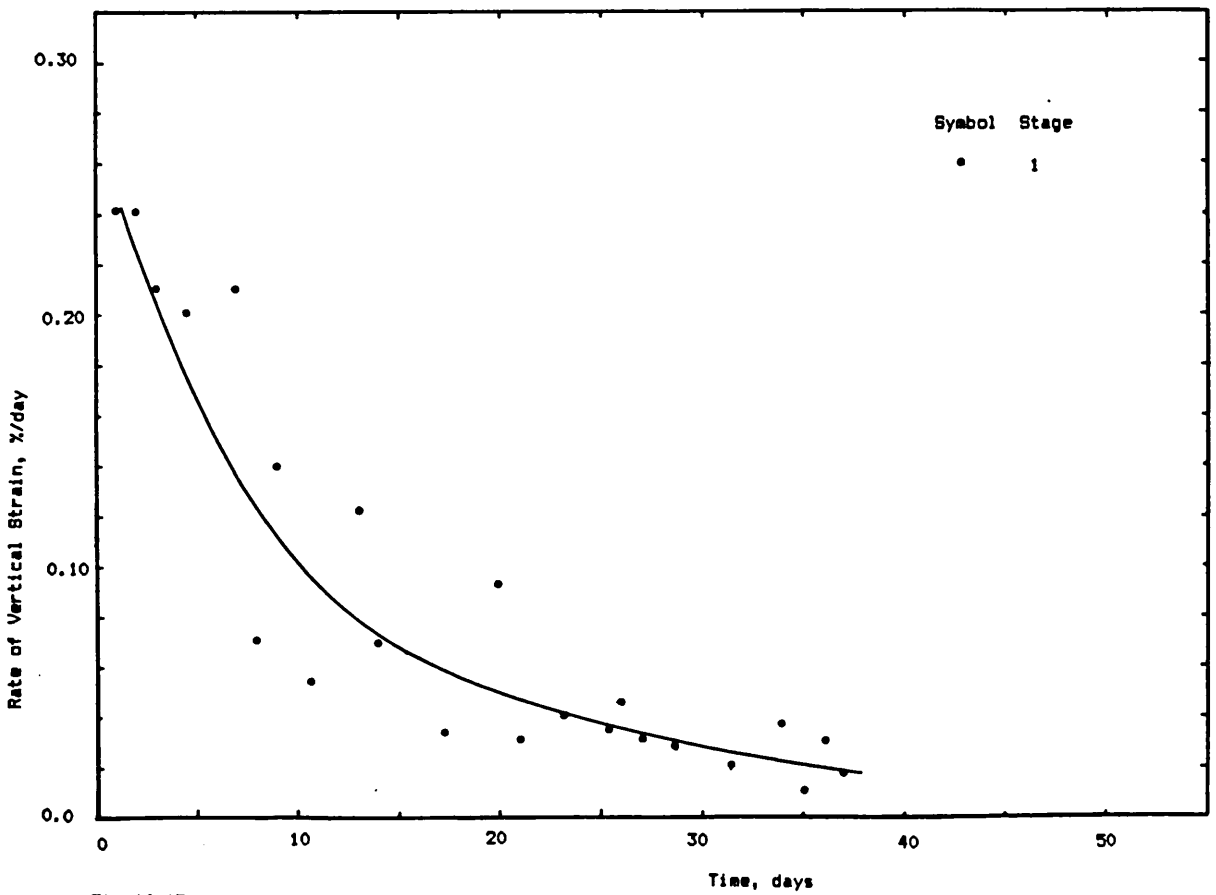


Fig.10.17 Rate of Vertical Strain, Test SP07

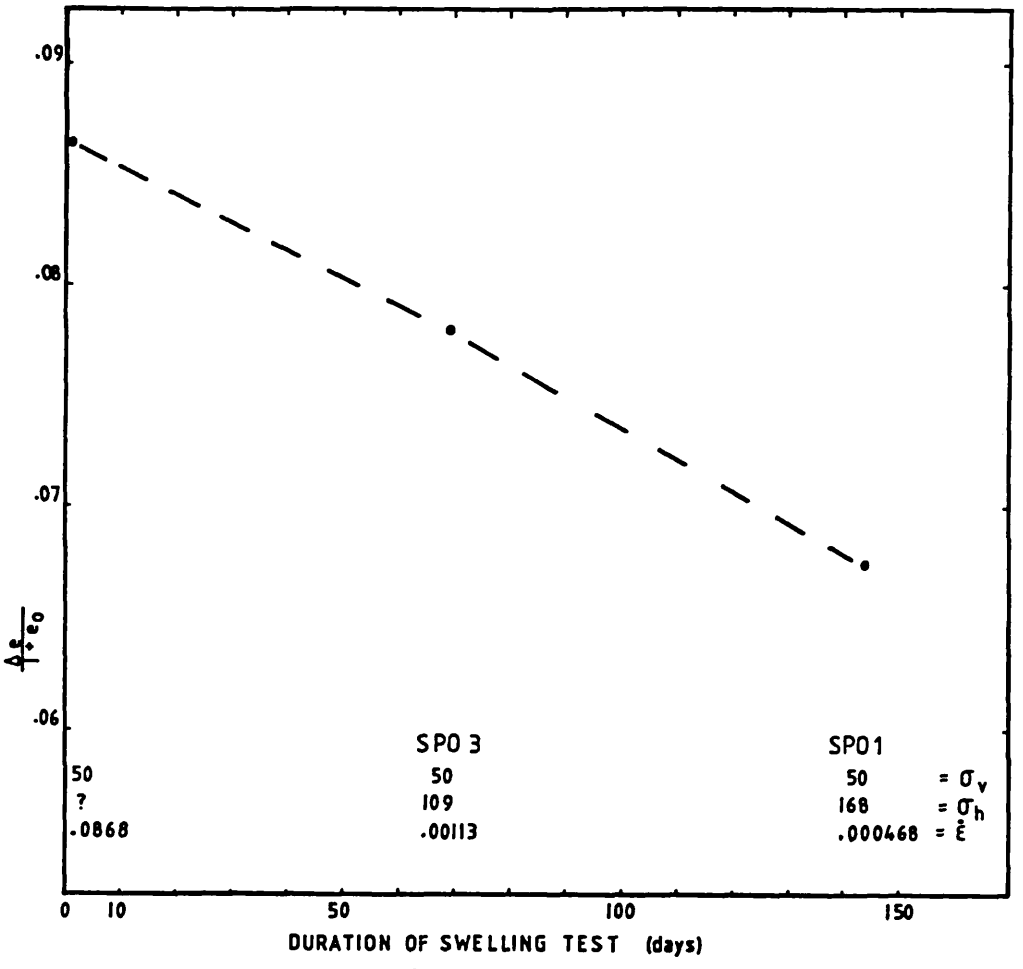


Fig.10.18 Effect of Test Duration (Schreiner 1987f)

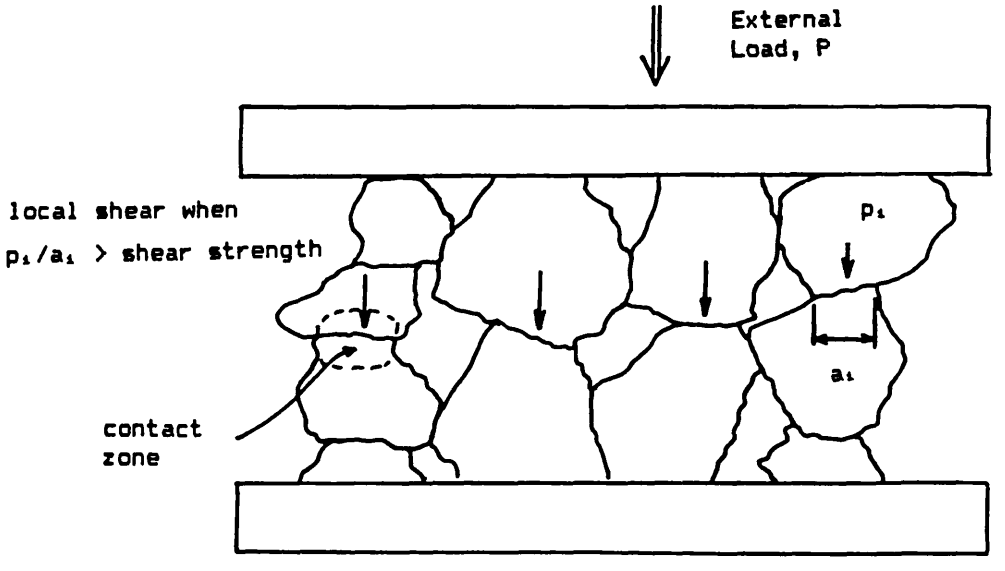
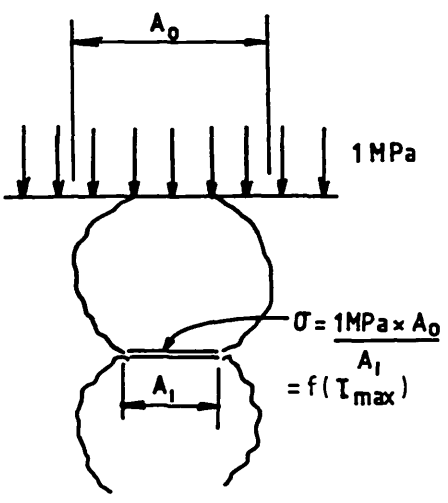
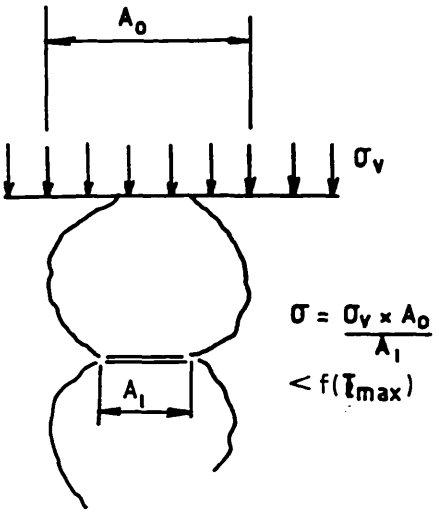


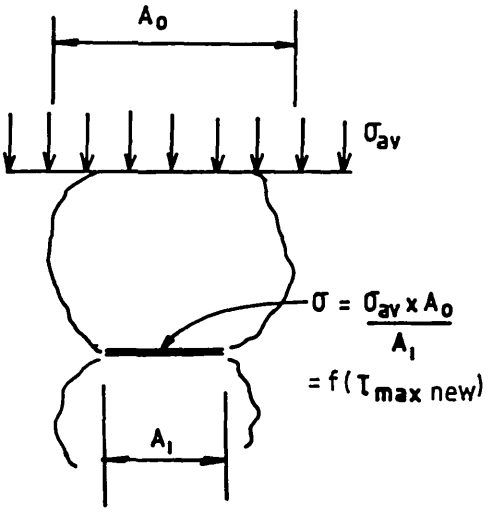
Fig.10.19 Stress in Contact Zone



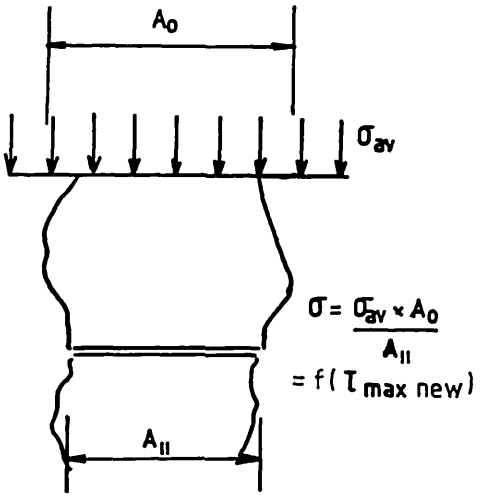
(a) During Compaction



(b) After Compaction



(c) During Swelling



(d) After Collapse

Fig.10.20 Collapse of Clustered Structure

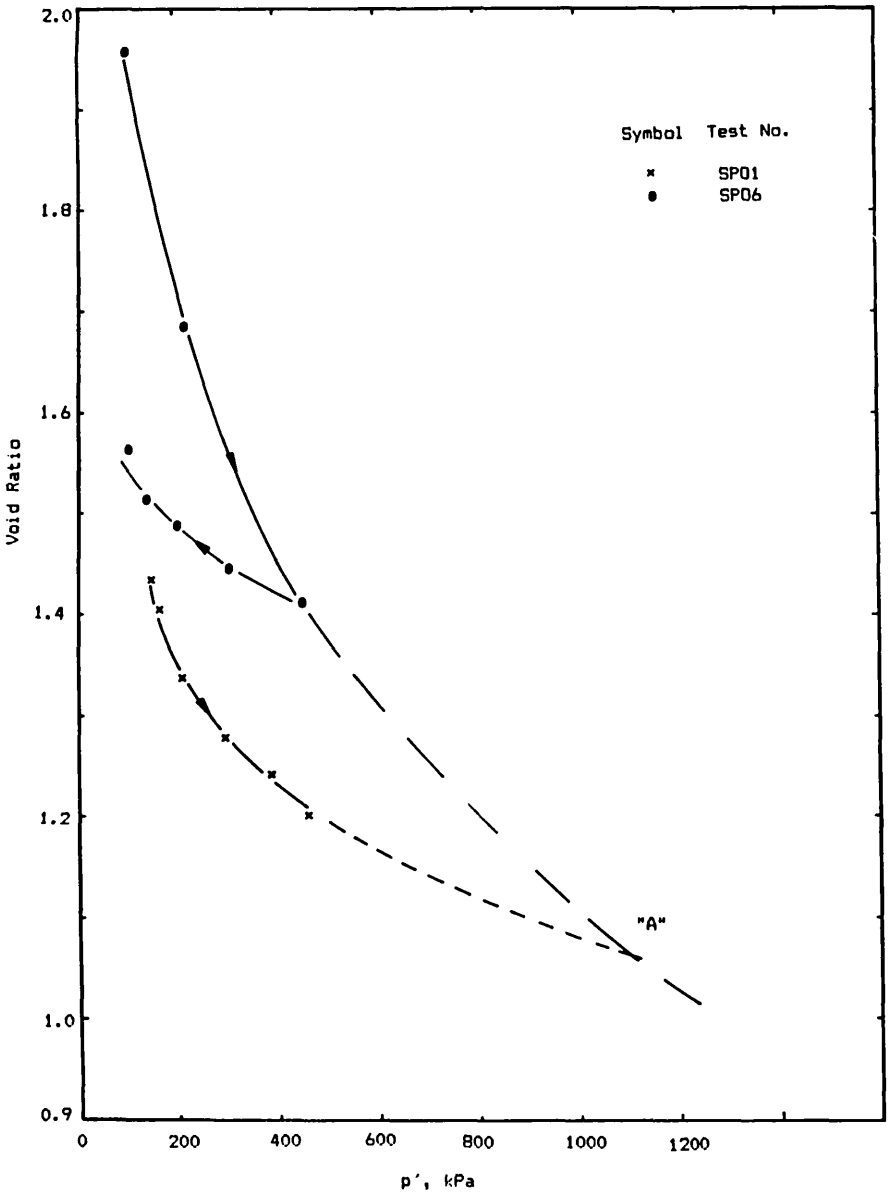


Fig.10.21 Comparison of void ratios in Tests SP01 and SP06

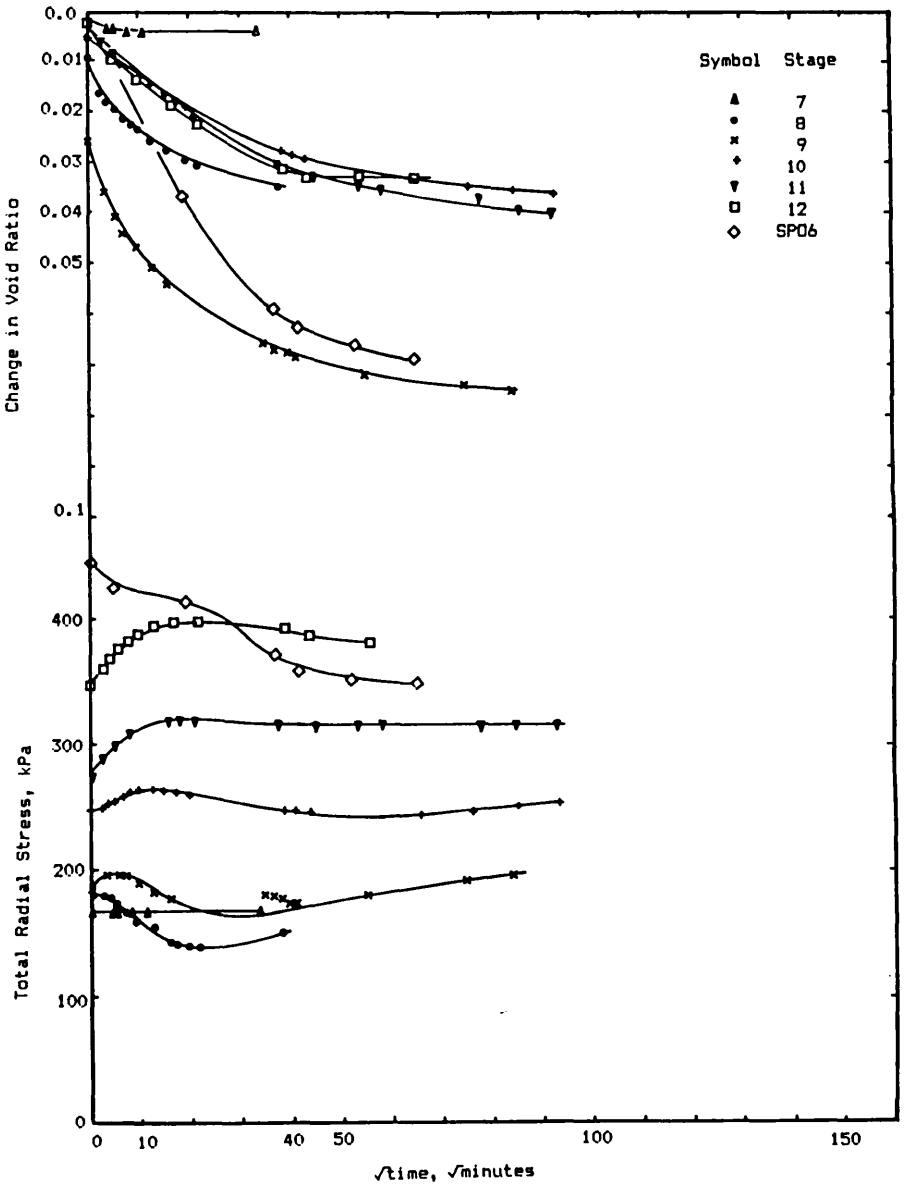


Fig.10.22 Results of Compression Stages, Test SP01

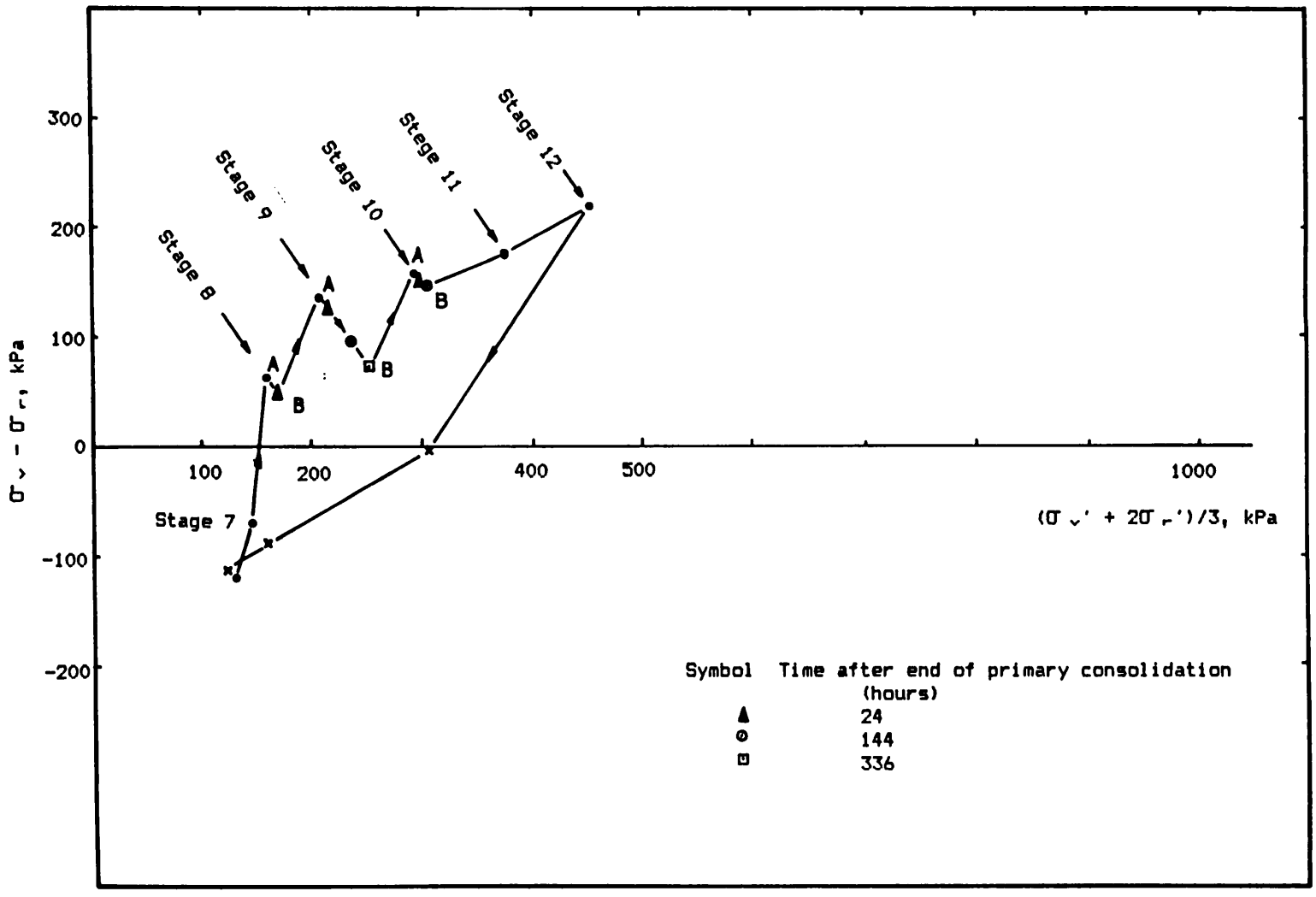


Fig.10.23 Stress Path During Compression, Test SP01

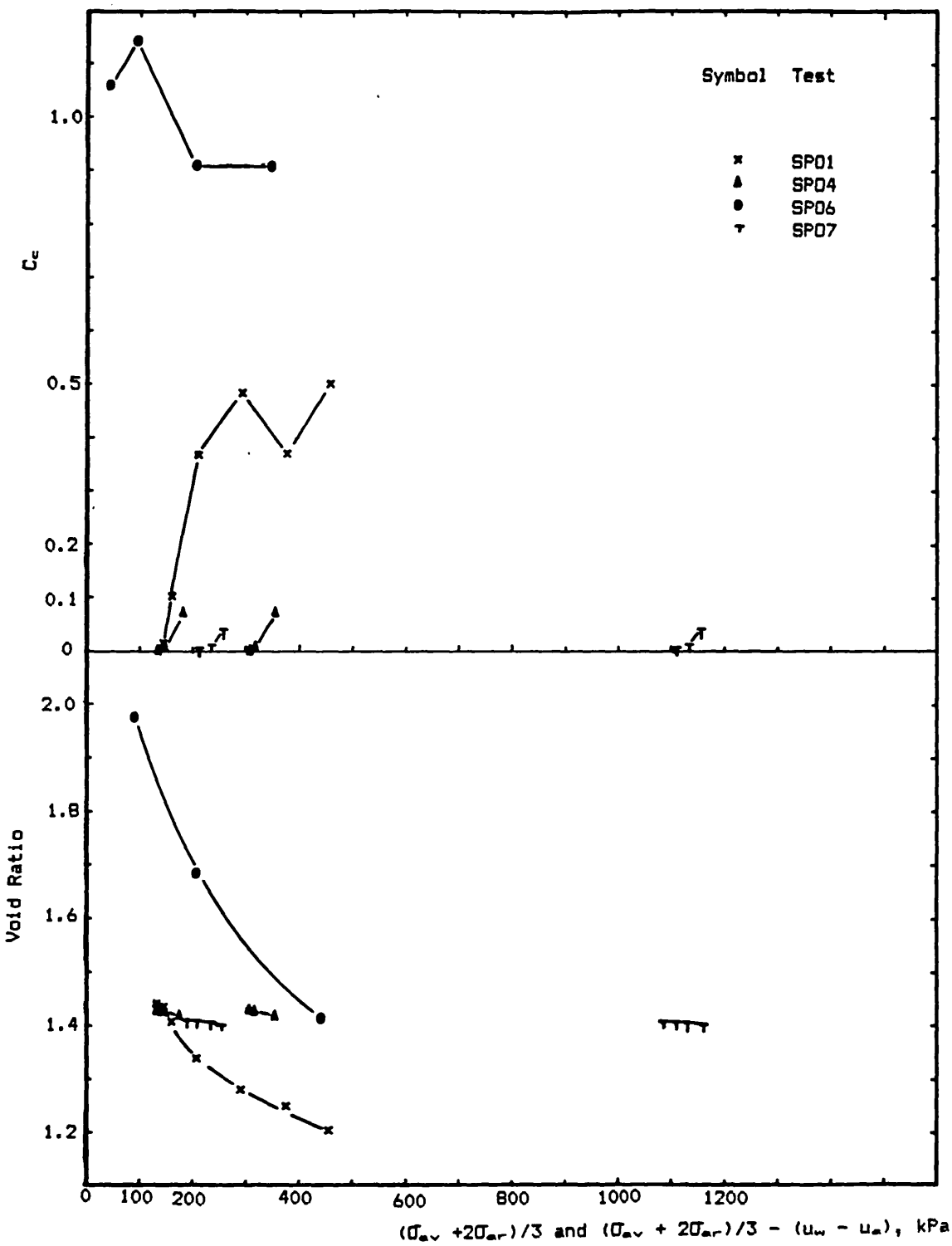


Fig.10.24 Compression results, Tests SP01, SP04, SP06 & SP07

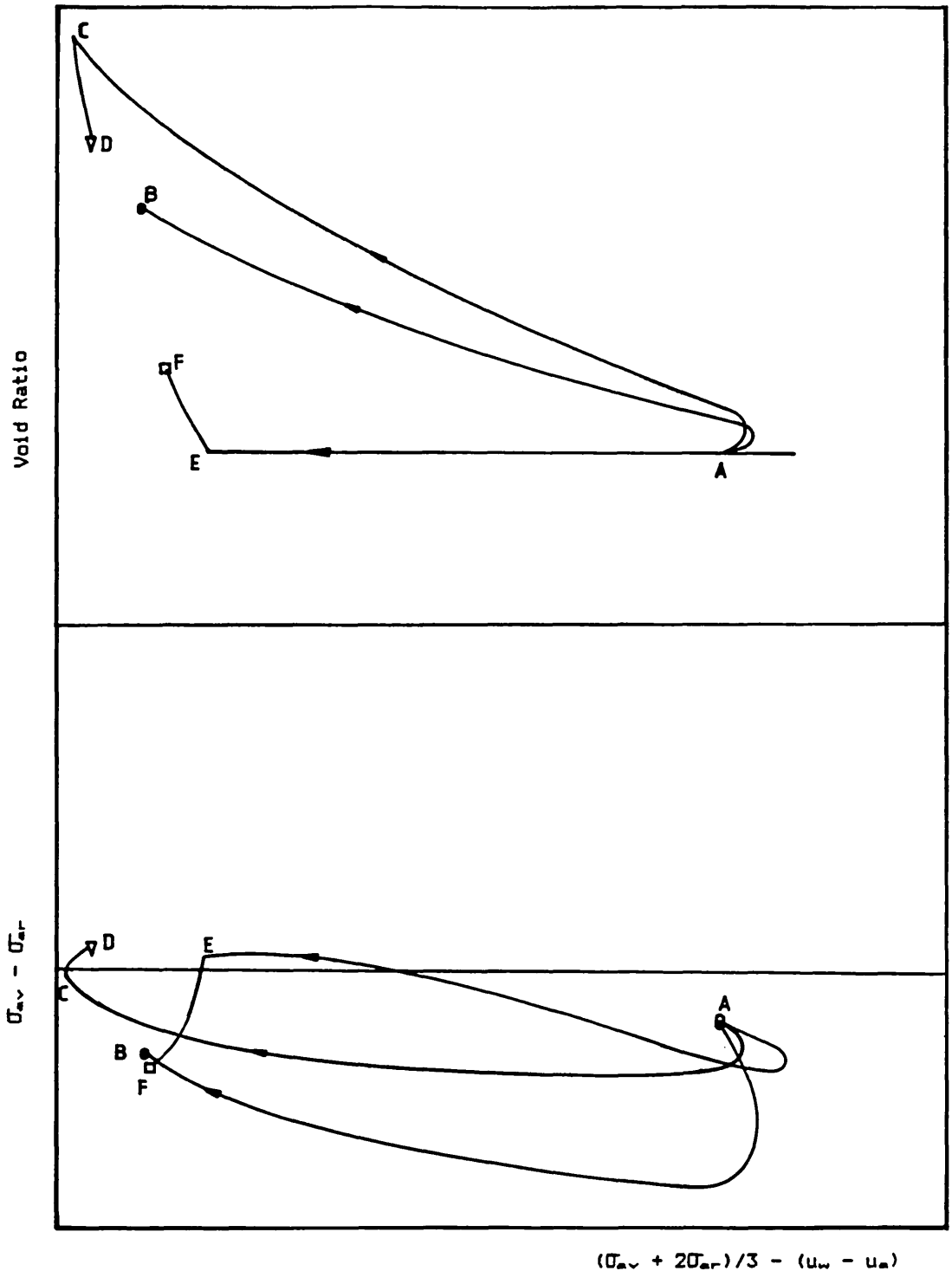


Fig.10.25 Schematic Stress Paths for Standard Test Procedures

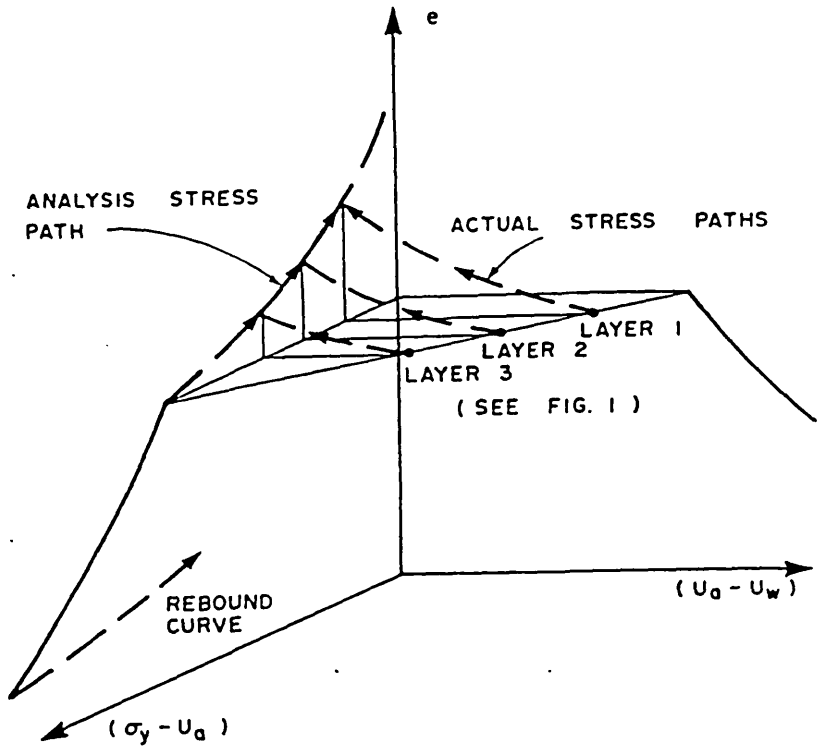


Fig.10.26 Comparison of the Actual Stress Path and the Stress Path Used in the Analysis

(Fredlund et al 1980)

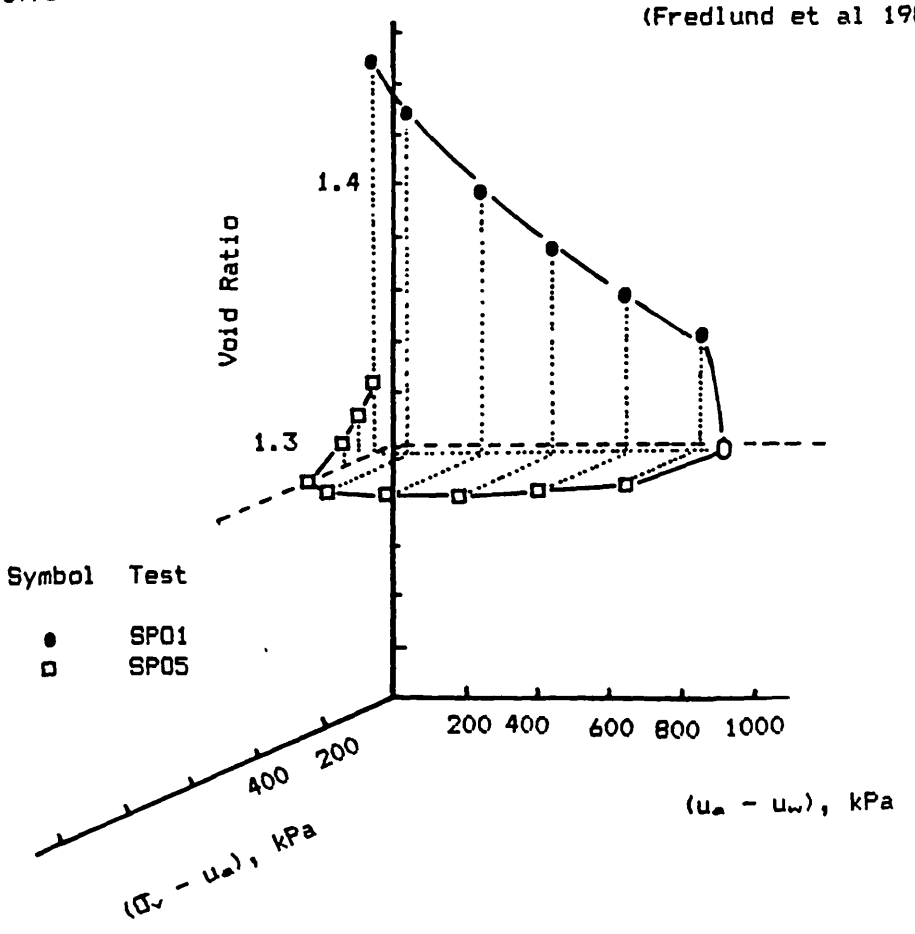


Fig.10.27 Observed Stress and Void Ratio Changes

APPENDIX 10A.1

Stress Paths during Swelling of Compacted Soils under Controlled Suction

H.D. Schreiner
Research Assistant

J.B. Burland
Professor

Imperial College of Science and Technology
London, United Kingdom

Synopsis

The vast majority of volume change testing of expansive soils has been performed under one-dimensional conditions with no knowlege of the lateral stress or suction. Some data is available with lateral stress measurement, and a small amount is available with suction measurement. The only data which has been found for vertical swelling or constant volume wetting with both lateral stress and suction measurement is for isotropic conditions.

An oedometer has been developed which has a load cell in the wall to measure the lateral stress and a small pored ceramic in the base to permit the use of elevated pore air pressure to give suctions up to 1500kPa. Data is presented showing the full stress state during swelling under constant vertical applied total stress.

Introduction

Many forms of oedometer have been developed for testing samples under controlled suction. The earliest record that the author has found is Rengmark and Erikson (1953), whose hydraulic oedometer could be used with suctions up to the cavitation stress of water. Subsequent oedometers have improved the suction control, and high suctions can be established with either elevated pore air pressure and a fine pored ceramic, or with a semi-permeable membrane and a solution of polyethylene glycol in water.

Oedometers have also been developed for the measurement of lateral stress. Ofer and Komornik (1983) describe two which use strain gauges on the sample ring. There is also an oedometer commercially available in the USA which has a fluid filled chamber behind a thin sample ring and a pressure transducer to measure the pressure in this chamber.

No record has been found of an oedometer that combines lateral stress measurement with suction control.

Oedometer Test Procedures

In the study of saturated soils in normal engineering conditions where the relevant stress change is an increase, there may be little practical need for a knowledge of both vertical and lateral stresses in oedometer tests. This is in a large part due to the unique ratio between lateral and vertical stresses for any one soil when normally consolidated, but also due to the attempt in the laboratory testing to simulate the field stress path. Since the stress change is generally an increase, this can be reasonably modelled, beneath the centre-line of a footing, by one dimensional compression, with control of only the vertical stress, as the horizontal stress is dependent on the vertical stress.

The tests performed in expansive soil testing cover a wider range of stress paths than the tests for settlement of saturated soils. Three forms of test are used in order to estimate the "potential swell". The single oedometer method (Jennings et al 1973) permits swell under a low vertical stress, followed by a conventional compression test. The test usually taken to be representative of the field stress path allows swelling under the expected field vertical stress. The third test type employs soaking at constant volume followed by reduction of the vertical stress (Sullivan & McClelland 1969). It is implicitly assumed in these tests that it is only the vertical stress that governs the amount of swell that will occur, and furthermore that the vertical stress defines the stress state in the soil, regardless of the loading sequence.

Several Authors have commented on the influence of the "stress path" on the volume change (e.g. Justo et al 1984, Alonso et al 1987, Brackley 1975). However, these comments are based on test data from the above types of tests, in which only the vertical stress is known. Since the horizontal stress is not known, it is not in fact possible to describe the stress paths in these tests, and it is also not reasonable to claim that the final stress states are equal. Some of the non-equivalence of the volume changes will be due to the different stress states that result from the different loading sequences used in the tests.

Dakshanamurthy (1979) has shown the dependence of volumetric strain on the mean stress in a series of constant stress tests in a triaxial cell in which the suction was decreased to zero in a single stage. Although these are

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1-4 December, 1987 New Delhi India

somewhat different tests from the oedometer tests in which there is zero lateral strain, it is interesting to look at the vertical strains observed under equal vertical stresses but for different stress ratios, as shown in Table 1.

Vert Stress kPa	Horiz Stress kPa	Stress Ratio	Mean Stress kPa	Vert Strain	Vol Strain
200	200	1	200	3.5	0.5
200	100	2	133	1.5	4.5
200	67	3	111	-1.0	7.5

Table 1. Influence of Stress State on Swelling

The vertical strains can be seen to be unrelated to the vertical stresses, as are the volumetric strains. The data even shows "collapse" under the same vertical stress that permits swell. In the oedometer tests described above, the vertical stresses are not constant throughout the test, and the influence of mean stress or lateral stress will not be as simple to assess. However, the importance of the lateral stress and the full stress state are clearly evident. An oedometer has been developed which, it is hoped, will help to unravel some of the complexities of the real stress paths during the various forms of oedometer tests used for swell testing.

Apparatus and Test Procedure

The oedometer is derived from the hydraulic suction plate oedometer of Rengmark & Erikson (1953), the Hydraulic oedometer (Rowe & Barden 1966) and the pressure plate system in common use in soil science. The base plate of the oedometer has a 15bar porous ceramic disc recessed into it with drainage channels cut into the stainless steel beneath the disc. The sample ring is a thickwall cylinder of 100mm internal diameter and has a 15mm diameter load cell penetrating radially through the wall. The face of the load cell was machined with the bore of the cylinder. Air pressure applied within the sample establishes a suction gradient across the porous ceramic. Vertical stress is applied in essentially the same manner as in the Rowe type oedometer, using hydraulic pressure within a rubber bag which is clamped between the top cap and the sample ring.

Bulk samples of the soil were prepared at the chosen moisture content after air drying and pulverising to pass a 2mm sieve. A greased rubber membrane is placed within the oedometer cylinder and the loose soil placed and lightly tamped within this membrane. The top cap with the hydraulic loading membrane in place is assembled onto the sample ring and the soil is compacted under a 1MPa hydraulic stress. A 6mm thick steel disc between the hydraulic membrane and the soil during compaction ensures that the top surface of the soil remains flat. The compacted density obtained at the 35% moisture content used corresponds to about 97% of the BS standard maximum dry density reported by Mutonyi (1973) for the same soil. After compaction the air pressure within the sample is raised to establish a suction gradient across the porous ceramic equal to the previously measured matrix suction, in this case 900kPa.

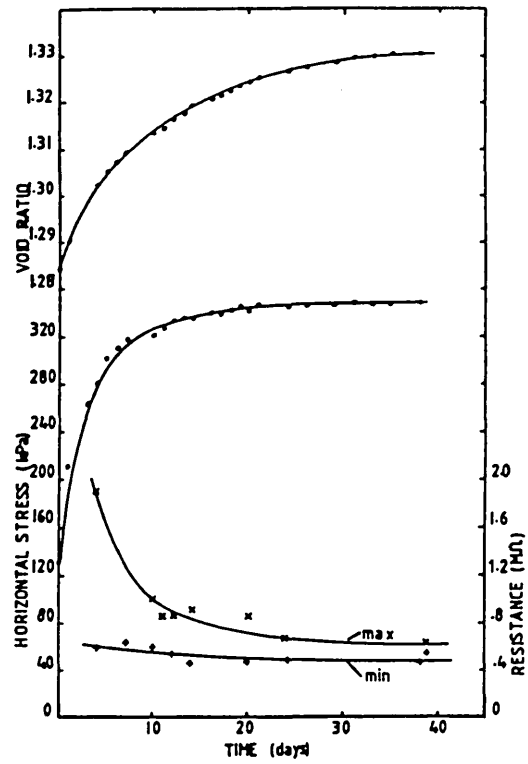


Fig. 1. Swelling During Stage 1

In the test described here, the sample was subjected to a vertical total stress of 50kPa above the air pressure within the sample throughout the swelling stages. The matrix suction gradient of 900kPa was maintained in stage 1 until the diffusion of salts through the porous ceramic had effectively ceased. This was monitored by measuring the electrical resistance of the water which was flushed back and forth beneath the ceramic. The void ratio, lateral stress and electrical resistance changes during this stage are shown in Fig.1. The stress is referenced to atmospheric air pressure within the sample; i.e. the actual air pressure is subtracted from the lateral total stress. This is achieved in the oedometer by applying the same air pressure within the sample and behind the load cell piston. The load cell is thus unaffected by the air pressure.

In the subsequent four stages the air pressure was decreased in 200kPa steps and the vertical deformation and lateral stress measured. The sixth stage decreased the air pressure to 10kPa, followed by soaking of the sample under a small back pressure. The data are shown in Fig.2. The 100kPa stage was terminated prematurely, due to leakage of the hydraulic loading bag.

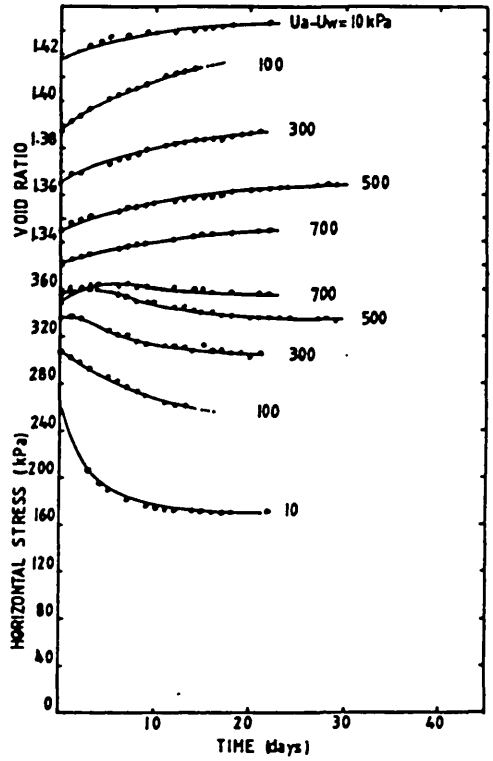


Fig. 2. Swelling During Stages 2 to 6.

After completion of the swelling stages, the sample was subjected to a conventional oedometer compression test.

Test Results

The swelling stages of this test constitute a "swell-under load" test. With the inclusion of the compression stages, it can also be used as a "swell-pressure" test of the type recommended by Chen (1973).

The void ratio changes for the decreasing suction stages are shown on Fig.3. Approximately one third of the swell took place in the first stage of the test, at a matrix suction equal to the initial matrix suction. During this stage the solute suction was changing due to diffusion of salts through the porous ceramic, and the volume change must thus be due to the decrease in solute suction. The salt content of the soil is given in a companion paper (Schreiner 1987), together with the classification data. Thereafter the void ratio increased due to matrix suction changes, caused by the decrease in the air pressure within the sample, with the solute suction effectively constant. It can be seen that the rate of swell increases slightly as the suction decreases. The amount of swell compares favourably with the swell interpolated from the swell under load data from Schreiner (1987), for standard oedometer swell tests. As noted by Justo et al (1984),

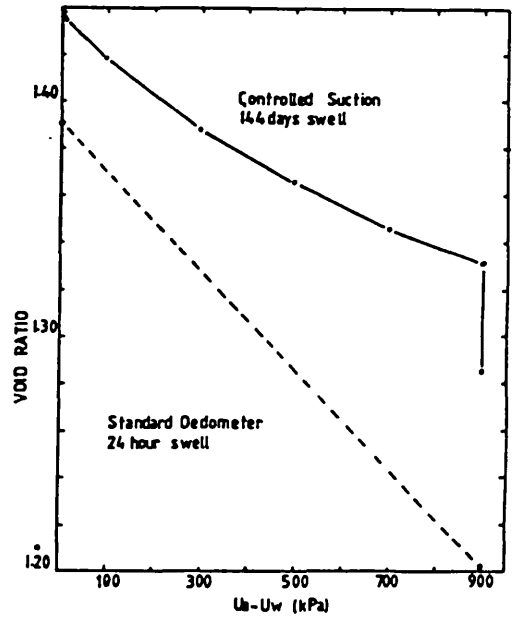


Fig. 3. Comparison with fast swell test

the swell in the controlled suction test was slightly less than that in the standard oedometer test.

The stress path followed by the sample during the swelling stages is shown in Fig.4. Two of the axes used correspond to the usual effective or total stress axes of mean normal stress and the deviator stress. Below the mean stress/suction plane, the vertical axis depicts the deviator stress, while above the mean stress/suction plane it depicts the void ratio. The suction axis shows the suction established by the elevated air pressure within the sample, and does not include the solute suction. As a result the large stress and volume changes that occur due to the solute suction decrease are represented at the same matrix suction value.

The maximum lateral total stress is attained after the solute suction has decreased and the matrix suction has been decreased to 700kPa. Thereafter the lateral total stress decreases at an accelerating rate, as shown in Fig.5. A marked change in rate of decrease occurs at about 100kPa suction, possibly indicating that the stress path has reached a limiting surface. In a saturated soil in which the pore water pressure was increased, this would occur when the stress path met the failure envelope.

At suctions below 700kPa, the mean total stress is decreasing. According to the data given by Dakshanamurthy (1979), this should lead to an increase in the volumetric strain. It is thus difficult to separate the effects of suction and stress from this single stress path.

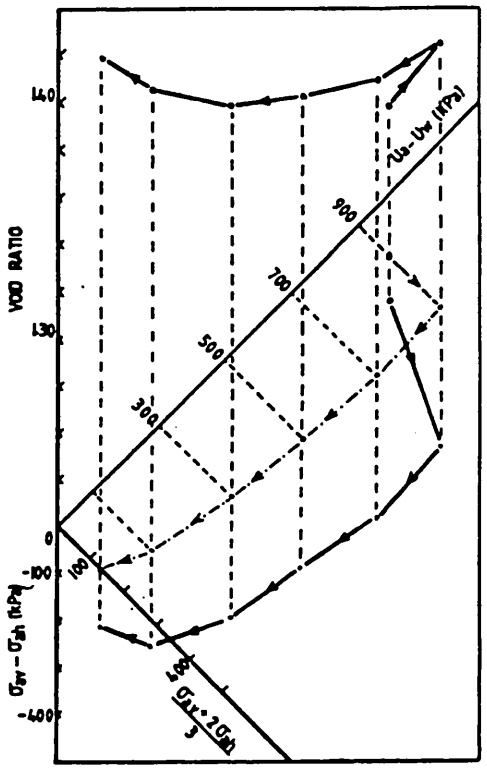


Fig. 4. Stress Path for Stages 1 to 6

Discussion

Fig.6 shows the same stress path with the compression stages added, and depicted on a two axis plot. Point "A" represents the stress state during the compression stages at the same void ratio at which the test commenced, which corresponds to the swell pressure proposed by Chen (1973). This can be seen to lie close to that part of the compression stress path which lies on a straight line passing through the origin. The lateral to vertical stress ratio along such a line is constant, and for the case of zero lateral strain this ratio is called Ko. Looking also at Fig.7, the e-log(vertical stress) curve has, at the void ratio equal to the initial void ratio, become a straight line. These two observations suggest that the soil is in a state similar to normal consolidation. True normal consolidation is generally taken to be from a slurried state, and could result in a slightly different stress-void ratio curve. However, this provides a possible explanation as to why the Chen type swell pressure is a "unique" stress for a given soil: It is simply the vertical stress which would be found from the virgin consolidation line for a void ratio equal to the initial void ratio. This will become less true as the amount of swell decreases, and the final state becomes closer to the curved part of the e-log(vertical stress) curve. There is some support for this in the data presented by Chen (1973), both for high vertical stress during swell, and for high initial moisture contents.

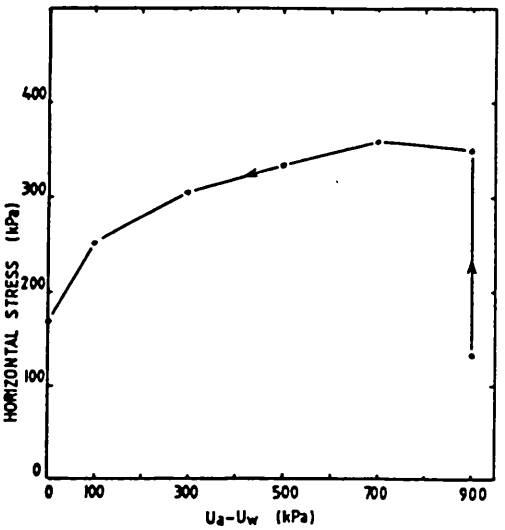


Fig. 5. Horizontal Stress During Swelling

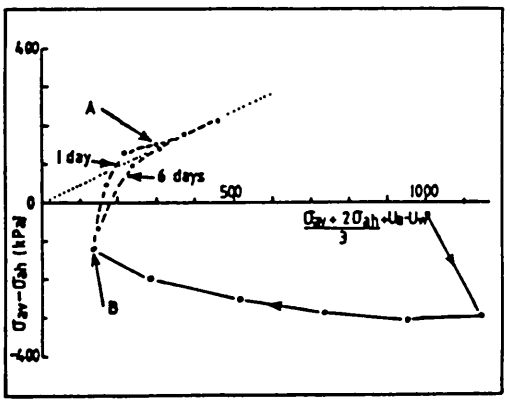


Fig. 6. Stress Path for Swelling and Compression

Similar reasoning may be applied to the single oedometer and the swell under load tests. The stress state from this swell under load test is represented by point "B" on Fig.6. The lateral stress is about 3 times the vertical stress, which corresponds to a passive pressure condition. The mean stress in the soil at the end of the swelling was 129kPa. If the soil had been allowed to swell under a small vertical stress, and then loaded to a vertical stress of 50kPa, the stress state would be closer to the virgin compression condition, shown by the dotted Ko line in Fig. 6, with the lateral stress equal to about 0.6 of the vertical stress. The mean stress would then be 37kPa, or less than one third of the mean stress in the swell under load test. Again referring to Dakshanamurthy's data, the ratio of volumetric swell for these mean stresses is about 4 for the particular soil

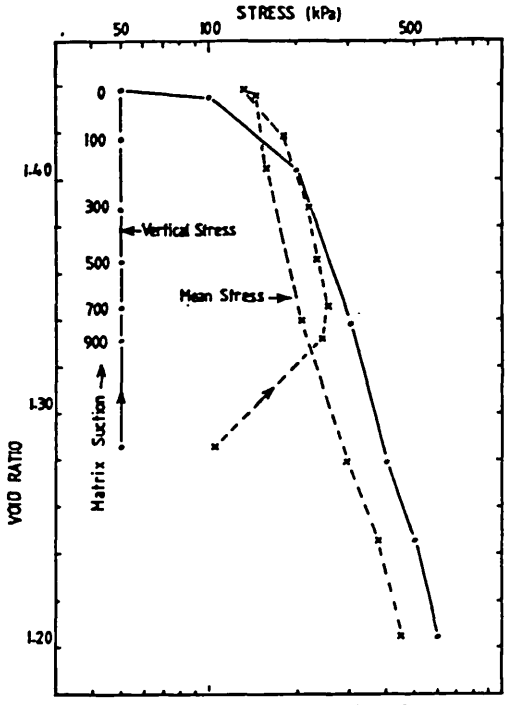


Fig. 7. Void Ratio vs log Stress

and the particular initial conditions which he used. The influence of the stress path on the volume change of the soil may well be insignificant in comparison with this mean stress difference. Since these two test methods do not lead to even similar stress states, there is no basis for comparison of the volume changes or void ratio changes in the two tests.

There are some other practical implications from the stress path shown in Fig.4. Although most foundation designs are concerned with vertical stresses and vertical movements, there are some instances where the lateral stresses are important. For example, Robertson & Wagener (1975) investigated the lateral stresses developed in a fill on soaking, for the design of abutment walls. In view of the observation that the lateral stress attains a maximum before complete wetting has occurred, it the lateral stress obtained by complete wetting may not be adequate for design. Data on lateral streses is also given by Ofer & Komornik (1983) for complete wetting. Although they report high ratios of lateral to vertical stress at the end of the test, the maximum lateral stresses developed due to partial wetting may be greater than the stresses which they observed for complete wetting.

In road embankments constructed using expansive soil, the soil is not rigidly confined, and the high lateral stresses cannot be developed. Instead, the soil will be able to expand laterally, under some at present unknown stress displacement relationship. This could lead to lateral strains in the subgrade which are too great for the base or sub-base, and cracks may result. Tests are planned in a triaxial stress path cell to study the effects of lateral stress-strain relationships on the vertical and lateral strains.

Acknowledgements

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CHAPTER 11

STRESS PATH TRIAXIAL TESTS

11.1 INTRODUCTION

Strength testing was undertaken using a computer controlled Bishop and Wesley (1975) stress path cell. The development of the computer software and hardware was done by others. It was expected that the development would be completed about midway through this project, permitting a series of tests to be performed. Various delays caused completion to be later than planned and only one test has been carried out.

Although little progress can be made in the understanding of the strength of unsaturated soil from one test, the data is of use for comparison with the Stress Path Oedometer data and for investigation of some of the test procedures.

11.2 EQUIPMENT

The basic 38mm sample diameter stress path cell was modified for suction control and volume change measurement. The basic requirement for testing of unsaturated soil is, as for the Stress Path Oedometer, the use of elevated pore air pressure with saturated fine pored ceramic plates. In addition, measurement of volume change cannot be made from the water flow out of or into the sample, and must be made by direct measurement on the sample.

11.2.1 SUCTION CONTROL

The ceramic plates used were cut from the 15bar ceramic plates as described in Chapter 9 for the Stress Path Oedometer. These were glued into the pedestal and top cap as shown in Fig.11.1. Top and bottom drainage was used to minimise the test duration, and necessitated an air inlet to the sample through the membrane. The conventional layout with the air supplied through a coarse ceramic in the top cap only permits drainage of pore water from the base, and the test duration would have been four times as long as with top and bottom drainage. The stress path oedometer tests had already indicated that the tests would be slow, and top and bottom drainage gives a similar drainage path length to that in the oedometer.

The air inlet was provided using a rubber grommet fitted through a hole in the membrane and sealed with O-rings and

Revultex as shown in Fig.11.2. A piece of fibre-glass tape was used to aid the distribution of air to the sample. The air pressure was controlled manually with a manostat valve and measured with a transducer.

11.2.2 DISPLACEMENT MEASUREMENT

Both axial and radial displacement were measured. The former was by a displacement transducer mounted externally, and the latter with the radial displacement gauge described in Chapter 9. The precision of the axial displacement transducer was not good with a local scatter of about +/-0.005mm and a daily cycle of about +/-0.015mm. The precision of the radial displacement gauge has been discussed in Chapter 9. For the calculation of radial and volumetric strain the sample is assumed, in the computer software, to deform as a cylinder.

11.2.3 DIFFUSED AIR FLUSHING

The bubble pump described by Bishop & Donald (1961) was recovered from the store and used. It was modified for use with two drainage tubes to the sample in place of the original co-axial tubes, and new valve seating washers were made. The pump did not function well during the test, but since the drainage lines leaked, allowing water from the cell to enter the pore water pressure system, the water inflow could not be determined from the volume gauge readings. The rate of leakage during the test was slow, and did not affect the applied pore water pressure. It did, however, prevent measurement of the porewater pressure under undrained conditions.Further servicing has produced correct functioning of the bubble pump.

11.2.4 CONTROL SYSTEM

The computer program permits control of the cell, ram and pore water pressures to be excercised in accordance with three separate equations written into the software by the user (Toll 1988b). The air pressure was controlled manually, because only three stepper motor control units were available. For this test, the control equations were written to maintain the cell and axial stresses at constant values in excess of the applied air pressure during the equilibration stage. The equations were modified for the shearing stage to include an increasing deviator stress, and the rate of increase of deviator stress was varied in an attempt to find the maximum rate of shear which did not cause excess pore water pressures.

The application and removal of the stresses at the beginning and end of the test utilised similar control equations which maintained the cell and deviator stresses relative to the applied

air pressure. The air pressure was raised manually and the computer control responded by adjusting the cell and ram pressures.

11.3 TEST PROCEDURE

Preparation of the bag samples has been described in section 5.3. The sample to be tested was taken from the bag and weighed out into four equal sub-samples. The compaction procedure using four layers followed that used by Maswoswe (1985) in the same mould which is shown in Fig.11.3. This is a displacement controlled technique, whereas the technique used for all other tests in this research was stress controlled.

The compacted sample was removed from the mould and the membrane fitted. It was then placed on a 35mm diameter pedestal while the radial displacement gauge was glued onto the membrane and the air line was attached and sealed with Revultex. The sample was then transferred to the pedestal of the stress path cell and the lower o-rings placed over the membrane. Since the radial displacement gauge was in place, the o-ring carrier had to be in place before the sample was put on the pedestal, and could not subsequently be removed. It remained on the pedestal below the o-rings, and did not obstruct the sample.

The top cap was then fitted and O-rings placed over the membrane. Assembly of the cell was completed and the cell pressure raised to 50kPa. The air pressure was next raised slowly to 225kPa with the control system maintaining the cell pressure at 50kPa above the air pressure. The load cell was connected and the deviator stress of -50kPa applied progresively as the cell pressure was raised to 100kPa above the air pressure, again with computer assistance. The applied stresses at the start of the test were approximately equal to the stresses observed in the stress path oedometer at the end of compaction.

The applied matrix suction of 200kPa was chosen as CBR tests had shown that the major loss of strength occurred below this value, and it corresponds with tests SP03 and SP04. The moisture content of the bag sample was measured and found to be 36.2%, which is about 2% higher than the moisture content of the sample tested in the Stress Path Oedometer. The matrix suction at this moisture content was estimated from the data in Chapter 8 to be 850kPa.

Swelling was allowed under constant cell pressure and deviator stress while the axial and radial displacements were measured. After 42 days the rate of swell as shown in Fig.11.4 had dropped to the same rate that was noted in the oedometer tests, and the shearing stage was initiated. Since there is no proven method of determining the rate of strain or stress increase for a drained test on an unsaturated soil, the rate was varied in an attempt to cover the range from too slow to too fast. The rate was varied

from about 0.3kPa per hour to 2kPa per hour under computer control. The deviator stress is shown in Fig.11.5 for the shearing stages.

11.4 TEST DETAILS

The test data are shown in Fig.11.6 in the same format as was used for the stress path oedometer data. The swelling stage is shown by AB and the shearing stages by B to M. The deviator stress change with time is shown in Fig.11.5. The notation on this figure is also used in the tabulated data and on other figures which present data for this test. From B to C the deviator stress was increased at a rate of 0.4kPa per hour. This was reduced to 0.3kPa/hr between C and D so that the sample would reach isotropic stress conditions while the author was in attendance. At this point the load on the load cell changes from tensile to compressive, and it was thought wise for the slack in the top cap connection to be taken up while manual intervention was possible. The movement did not cause any problems. The rate was then increased so that the average rate from B to E was 0.4kPa/hr. From E to F the deviator stress was held constant so that the consolidation could be observed.

The rate of increase from F to G and from H to I was 2kPa/hr, with constant deviator stress stages from G to H and from I to J. From J the deviator stress was reduced to L and then increased at 1kPa/hr to M.

11.4.1 SWELLING

The volumetric swell observed during the swelling stage, AB in Fig.11.6, was 6.3%, the axial strain was 2.9% and the radial strain was 1.6%. The strain-time curves are shown in Fig.11.7a, b and c, respectively, for the swelling stage. The radial strain shows an initial contractive strain, followed by swelling. This is believed to be due to the movement of the "wetting front" through the sample, indicating that the middle of the sample did not start wetting up for the first one to two days. The contractive strain would be due to the 100kPa excess of cell pressure over the pore air pressure. The sample would have expanded when removed from the mould, and recompression under the applied cell pressure would occur. The delayed start of radial strain is also evident in the small rates of radial strain for the first 5 days in Fig.11.4.

The void ratios in tests SP03, SP04 and this test, SPC1, are compared in Fig.11.8 in terms of the change in void ratio plotted against the mean applied stress plus the suction. The change in void ratio in test SPC1 was larger than that observed in the oedometer tests, which is to be expected since the mean applied stress of 83kPa in the former is less than the 157 and 138kPa in the latter two tests.

The decrease in solute suction by salt diffusion through the porous ceramics would have been less in test SPC1 than in the oedometer tests because the volume of water in the back pressure system is smaller for the triaxial cell than for the oedometers. The concentration of salt in the water in the back pressure system will thus be higher in the triaxial system than in the oedometer system. If the solute suction decrease in test SPC1 had been equal to that in the oedometers then the change in void ratio for test SPC1 would have been larger than the observed amount.

11.4.2 SHEAR

11.4.2.1 Rate

In saturated soil tests the rate of stress increase is generally determined by limiting the excess pore water pressure to an acceptably small value. The discussions in Chapter 8 on suction measurement indicate that long term measurements may not be reliable due to the diffusion of air into the measuring system. Consequently, no pore water pressure probe was mounted on the sample. An attempt to measure the pore water pressure at one stage during the test was made using the top and bottom drainage ceramics, but the slow leakage of water from the cell into the back pressure system caused a continuous rise in the apparent pore water pressure.

Axial and radial stress strain curves for the shear stages are shown in Fig.11.9 and Fig.11.10 respectively. The void ratio and stress path are shown on Fig.11.6. The stress was held at a constant value at 30, 80 and 210kPa as shown on Fig.11.5, Fig11.9 and Fig.11.10 by EF, GH and IJ respectively. During the first two stages, after about 0.5 and 1% strain, no significant straining with time was observed. In each case about 0.05% strain occurred in the first three hours and no further strain thereafter. The strain-time curve for the 210kPa stage is shown in Fig.11.11. Volumetric strain has been plotted as it reflects the consolidation better than either axial or radial strain alone.

About 0.5% further strain has occurred, and this took place in the first day of constant stress. This is about 25% of the total volumetric strain that occurred under this deviator stress, and suggests that an excess pore water pressure had developed by this stage. This is supported by the small but visible increase in the deviator stress along LM, above the value maintained from I to J, when shearing recommenced after the consolidation stage. This indicates that there had ben a gain in strength due to the dissipation the excess pore water presure.

Using the consolidation stage it is possible to apply the simple Bishop & Henkel (1957) calculation to determine the time required to failure for a strain controlled test on a saturated sample. This may not be a rigorously acceptable approach, but it

serves to indicate that an appropriate test duration is about 17 days. Further testing is needed to examine the real performance of a sample, both because it is unsaturated and because the test is stress not strain controlled.

11.4.2.2 Influence of test type on rate

The duration of the consolidation stage at 210kPa is very much shorter than the swelling stage shown in Fig.11.7. The same is true of the compression stages of the oedometer tests compared with the swelling stages. In the stress path oedometers, compression due to an applied axial stress was complete in 2 to 4 days, while the swelling stages required 21 days to reach an acceptably small rate of strain. This is not consistent with the observations for saturated soils, where the durations are of similar magnitude. If the durations were as different for saturated soils as they appear to be for unsaturated soils then the rebound stages of oedometer tests would take several days per stress change.

It is probable that during swell under controlled suction the water flow is in the absorbed layer and is slow because of the high viscosity of this water. There is no excess free water to flow outside the absorbed layer, because the water that enters the soil is taken up in the absorbed layer in that part of the sample which has reached equilibrium. The soil which still has a suction excess receives its water by flow through the absorbed layers of the soil closer to the water supply. If soaking is used, where there is a large excess of water, then flow of free water can take place through the macro-pores. The swelling time for soaking tests is of the same order of magnitude as the compression time.

During compression an excess pore water pressure is generated throughout the sample, resulting in water being forced out of the absorbed layer. This free or excess water can then flow through the sample and the rate of flow will be controlled by the viscosity of the free water.

If these concepts are representative of the real behaviour of the soil water, then it may be possible to increase the rate of swell by supplying free water to the sample, for example by passing humid air through the macrovoids in the sample. The shorted test durations would be a distinct advantage.

11.4.2.3 Shear strength

No significant conclusion can be reached from one test in isolation. Three factors will contribute to the strength of the sample. These are the applied stress, the cohesion and the suction. Since the soil appears from the stress path oedometer tests to be overconsolidated in some form, at least within the aggregations, the cohesive contribution cannot be discounted. It

is not possible to separate the cohesive and suction effects in the data from this test. However, there is no evidence of dilation in the volume strain curve in Fig.11.12, and this suggests that the sample is behaving as a normally consolidated soil.

A circuitous route can then be used to estimate the contribution of the suction to the shear strength. The first step is to obtain an approximate value for the angle of friction from the consolidation test SP06. The K_o value from this test is .547, and using Jaky's equation, $\emptyset'$ is calculated to be 27°. The maximum deviator stress for a soil with no suction and a cell pressure of 100kPa would be 170kPa. The contribution from the suction then appears to be about 60kPa at the maximum observed deviator stress.

It is possible that a greater deviator stress would have been achieved if the sample had been strained further, but the increase would , from the shape of the stress strain curve in Fig.11.8, probably not exceeded 20 to 30kPa. The effect of the suction is surprisingly small, and may be due to the aggregated nature of the soil. In that case, the soil strength would increase at the same suction when the structure is broken down to a preferred parallel type. It may equally well be due to an overestimate of the friction angle. For example at a friction angle of 20°, the contribution from the suction becomes 130kPa of the deviator stress, with 100kPa due to the applied cell pressure.

11.5 PERFORMANCE OF EQUIPMENT

Most of the equipment used in this test is widely used elsewhere. Specialised items are the bubble pump, the fine pore ceramic plates and the radial displacement gauge. The bubble pump appears to be functioning adequately, despite over 20 years of storage. The mounting of the ceramic plates was a source of difficulty, as a crack developed around the edge, between the ceramic and the glue. This was sealed with Loctite 601 which was applied in minute quantities along the crack until no more was absorbed. Loctite sets hard when placed in a narrow space, and thus sealed not only the crack but also the pores in the outer edge of the ceramic. About 2mm of the ceramic appeared to be affected in this way, but this left an area of about 30mm diameter unaffected which is adequate for drainage.

The radial displacement gauge produced credible data. There is no absolute means of judging the data, but the magnitudes and directions of the observed strains are in agreement with the expected strains. The corrugated PVC boots appear to have stiffened somewhat after prolonged contact with the Diala Oil used to isolate the strain gauges. This is a significant problem, but alternative materials are being investigated for the boots. A test piece of a drinking straw which is thought to be made from polyethylene or polythene has not stiffened after being in the oil for several months. These straws are of a suitable diameter and

have a flexible corrugated section. If the difficulties of the attachment and sealing of the polyethylene or polythene to the brass can be overcome, then this may provide a solution.

	Swelling		Shear							
	A	B	E	F	G	H	I	J	L	M
q kPa	-50	-50	32	32	82	82	210	210	189	234
pa kPa	83	83	111	111	128	128	170	170	163	177
p kPa	933	283	306	306	323	323	365	365	358	372
e	1.274	1.414	1.409	1.409	1.406	1.405	1.378	1.365	1.366	1.341
Ea %	0	-2.80	-1.89	-1.84	-1.69	-1.59	0.39	1.35	1.32	3.20
Ev %	0	-6.07	-5.49	-5.48	-5.34	-5.32	-4.15	-3.59	-3.65	-2.54
Er %	0	-1.61	-1.77	-1.80	-1.84	-1.84	-2.28	-2.48	-2.49	-2.92

Key
pa = $(\sigma_{av} + 2\sigma_{ar})/3$
p = pa $-(U_w - U_a)$
Ea = axial strain
Ev = volumetric strain
Er = radial strain
-ve sign denotes expansion

TABLE 11.1 TEST RESULTS

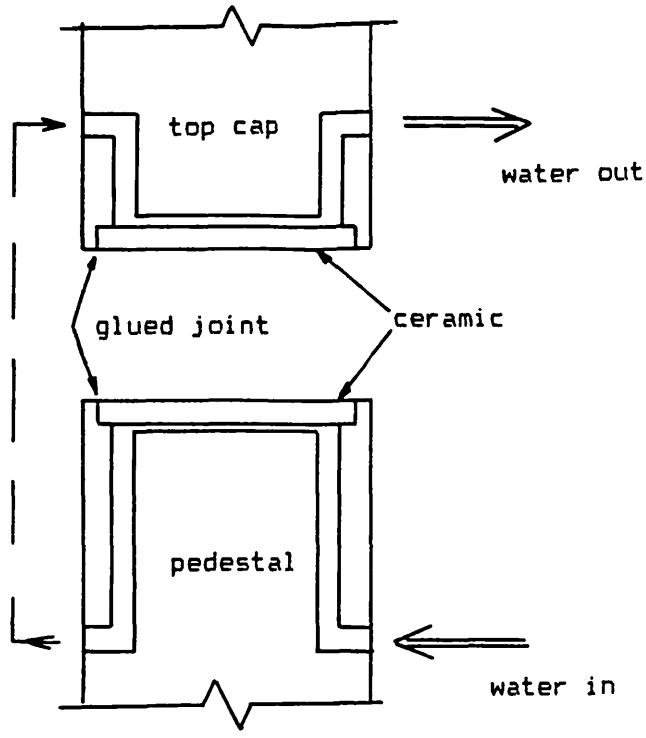


Fig.11.1 Mounting of Ceramic Plates in Pedestal and Top Cap

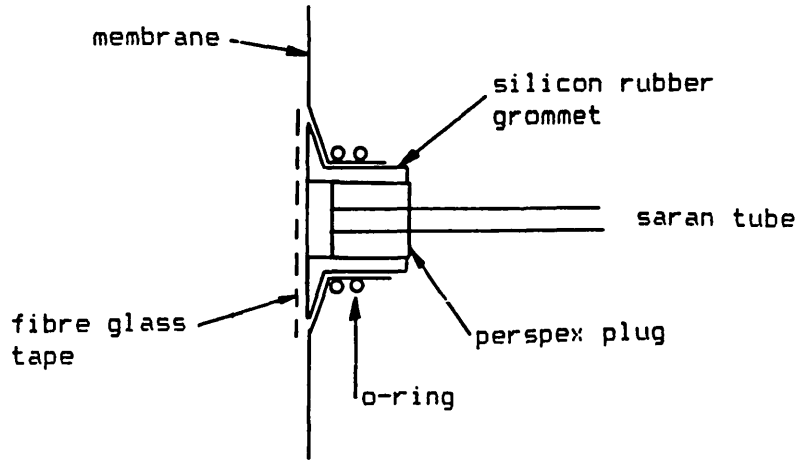


Fig.11.2 Air Inlet through Membrane

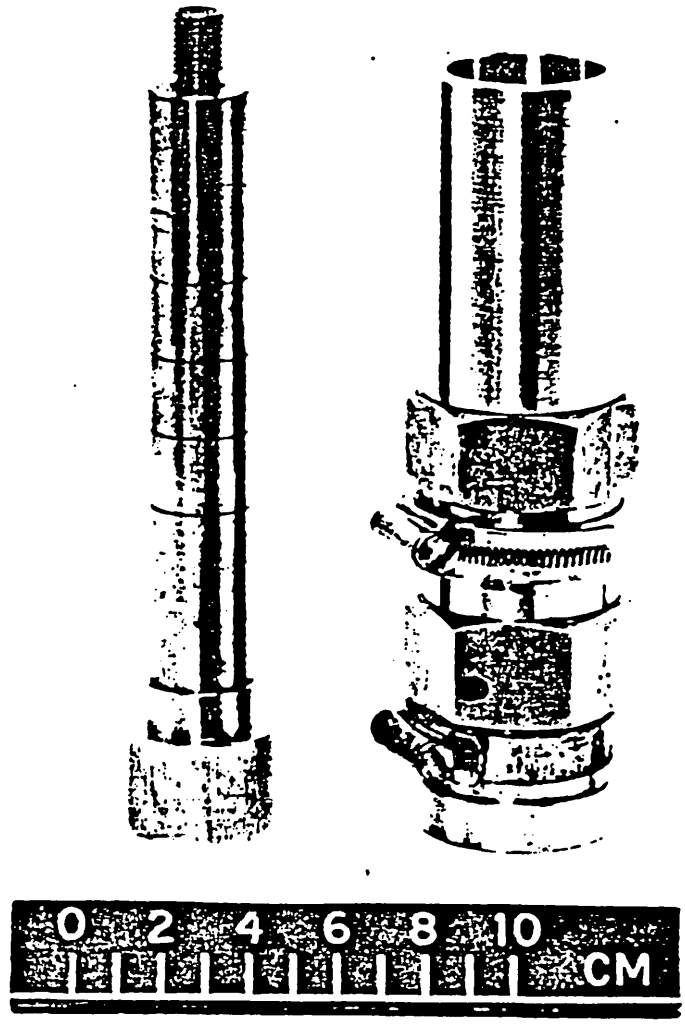


Fig.11.3 Assembled 38mm diameter triaxial sample mould with piston.

(Maswoswe 1985)

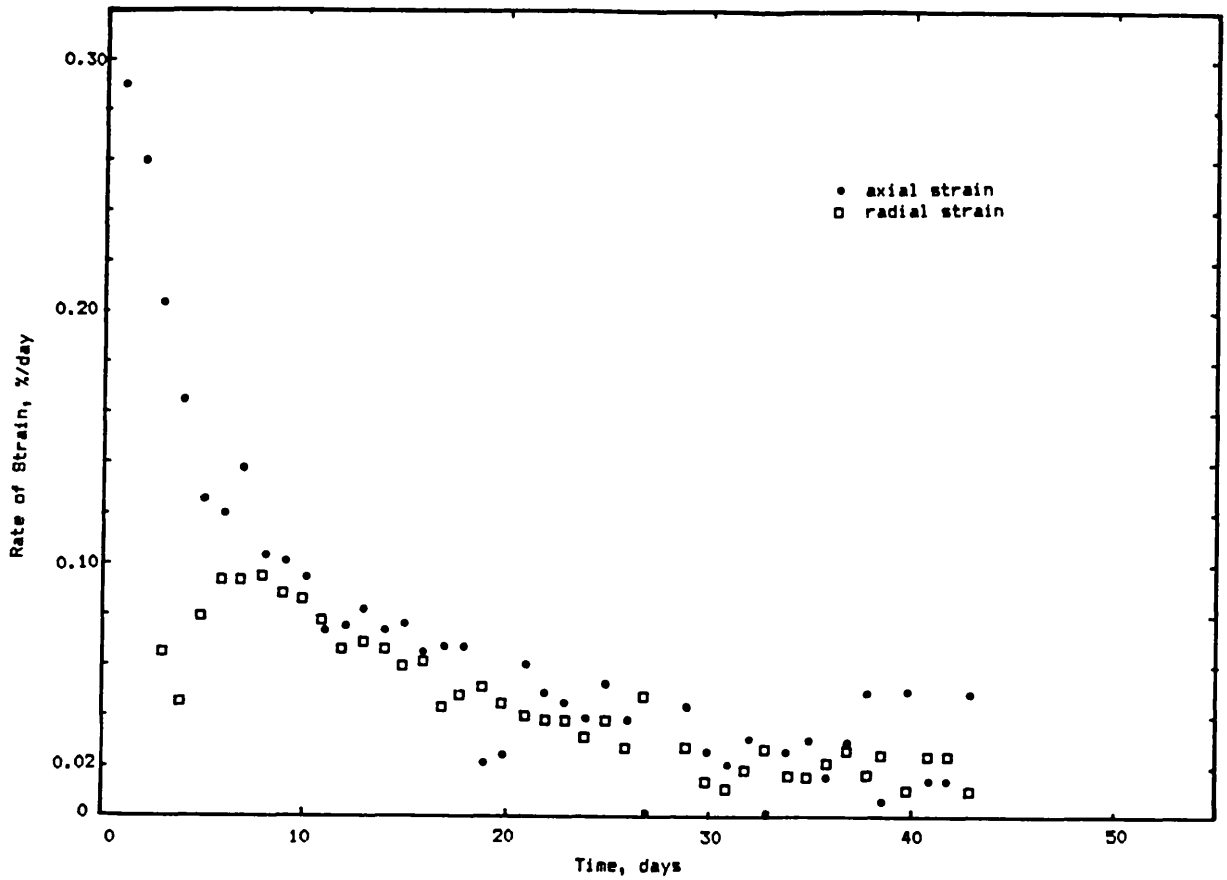


Fig.11.4 Rate of Strain during Swelling Stage, Test SPC1

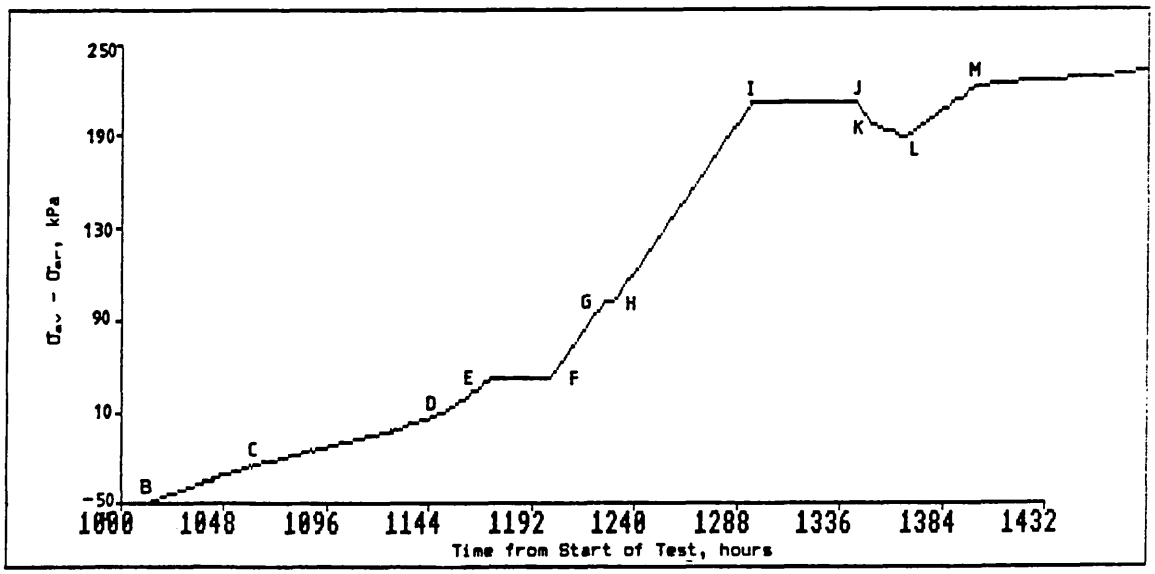


Fig.11.5 Application of Deviator Stres

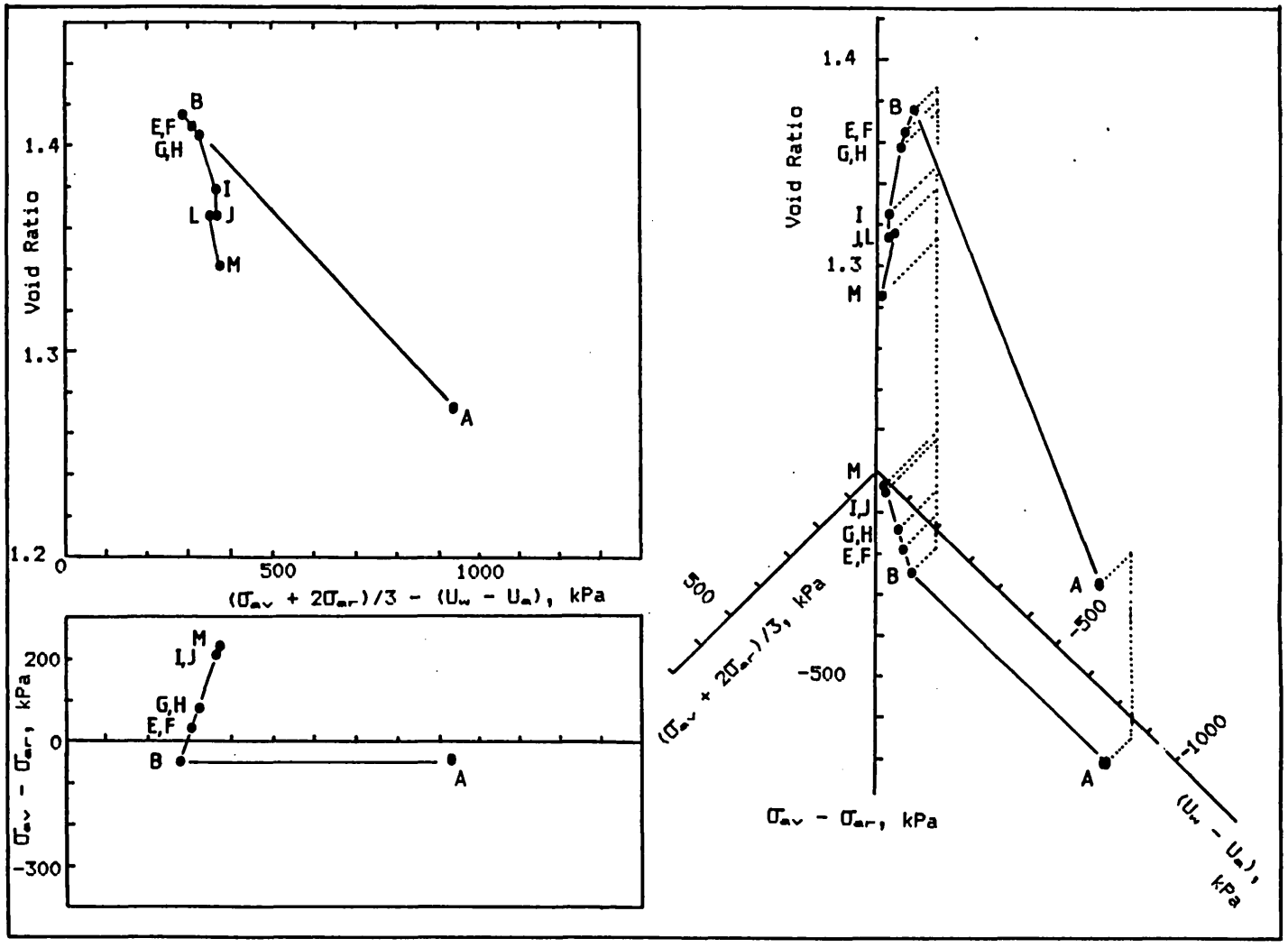
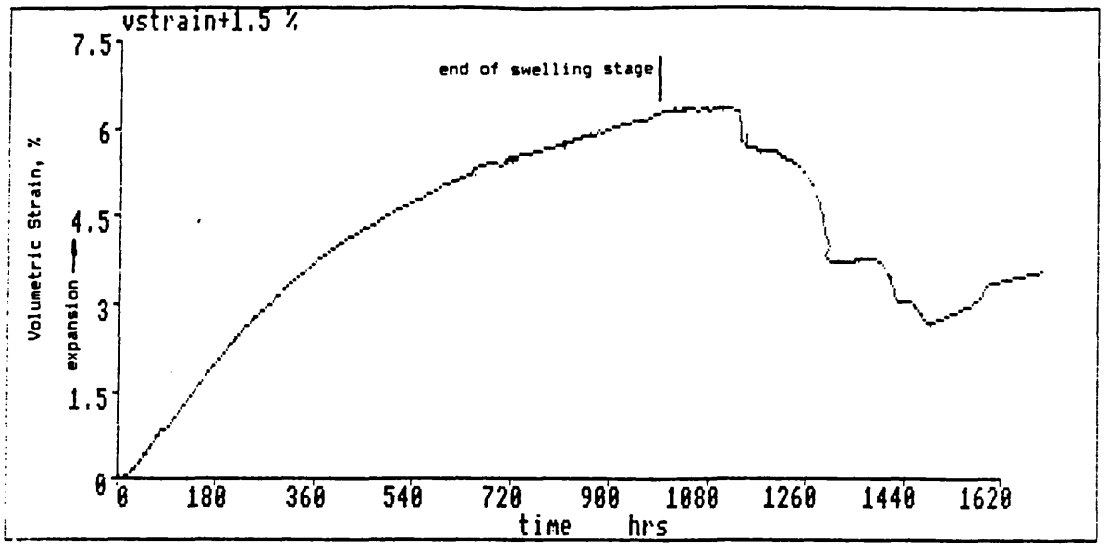
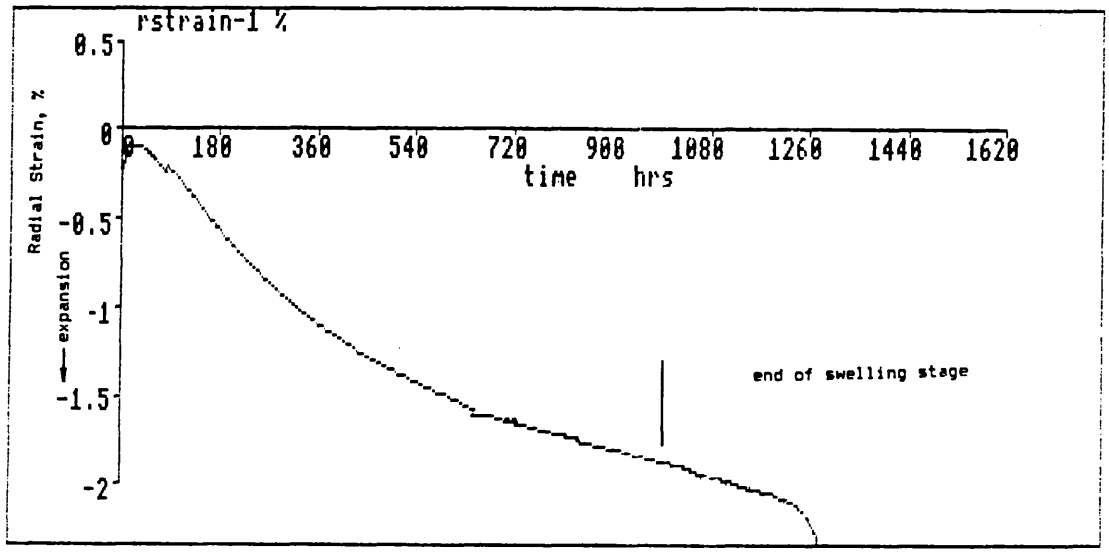


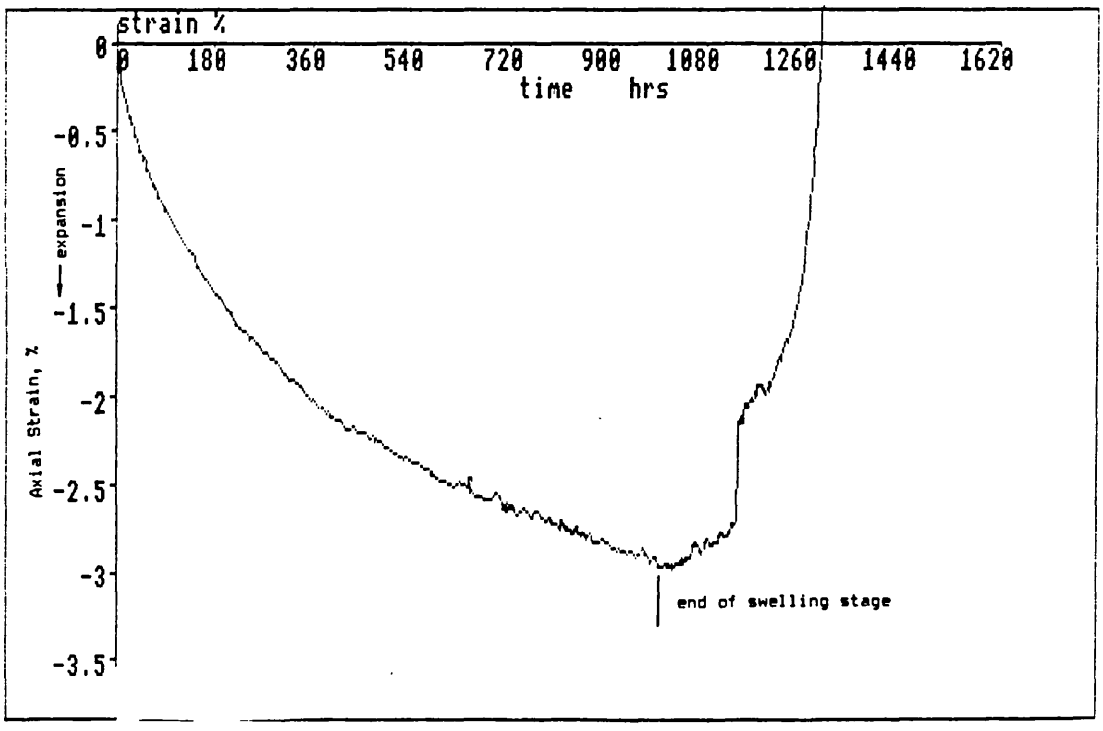
Fig. 11.6 Test Results



(a) Volumetric Strain



(b) Radial Strain



(c) Axial Strain

Fig.11.7 Observed Strains during Swelling Stage

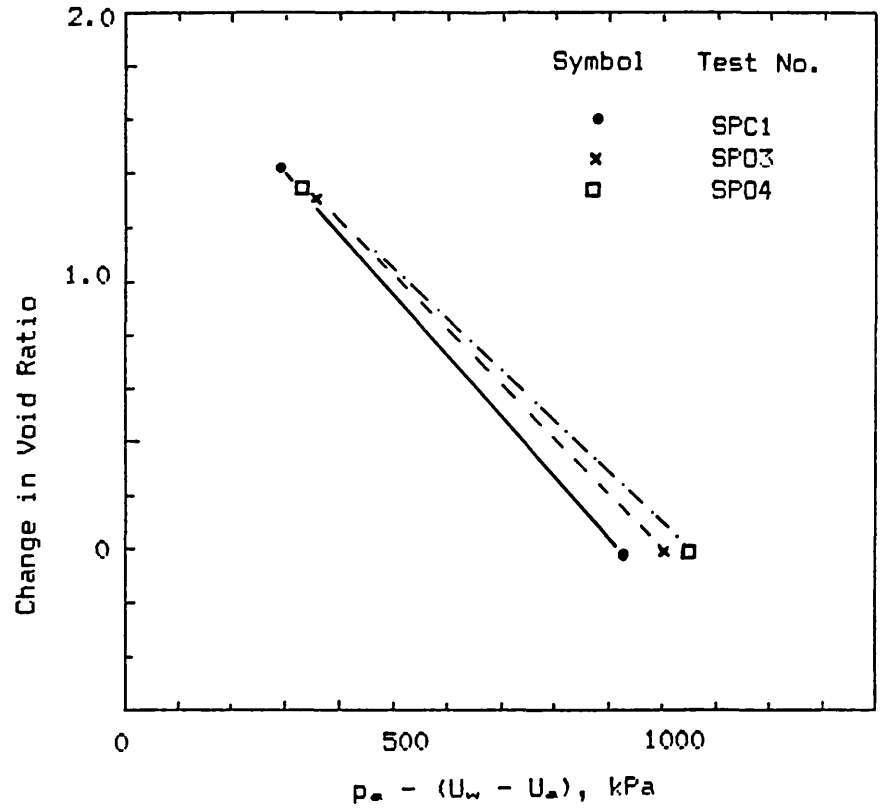


Fig.11.8 Volume Change in Oedometer and Triaxial Cells

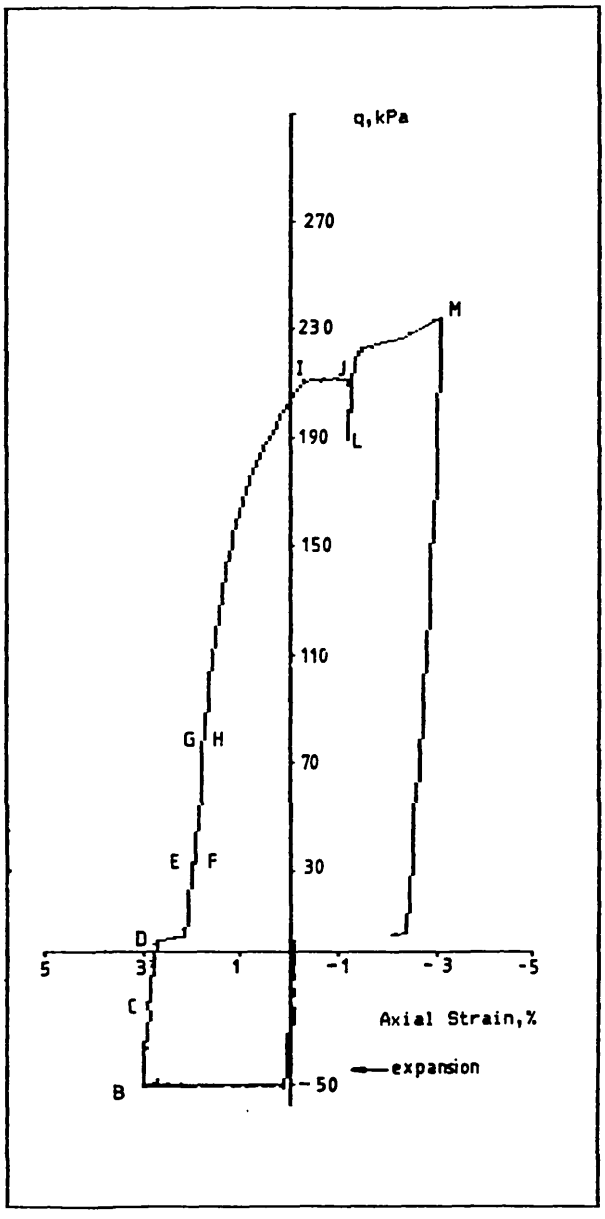


Fig.11.9 Axial Strain vs Deviator Stress

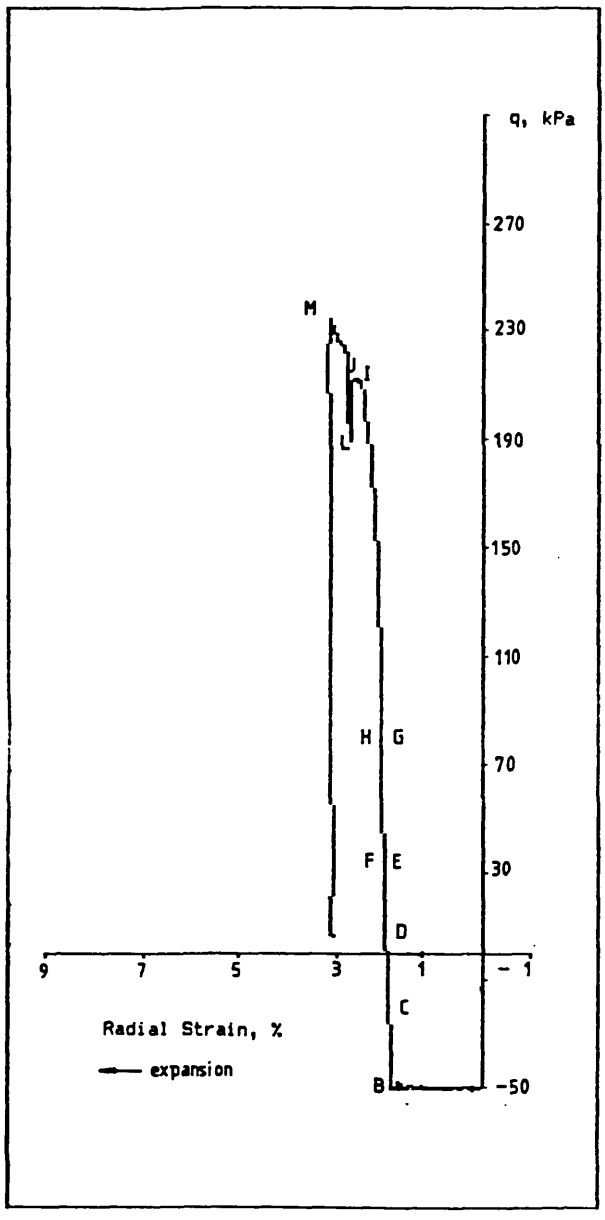


Fig.11.10 Radial Strain vs Deviator Stress

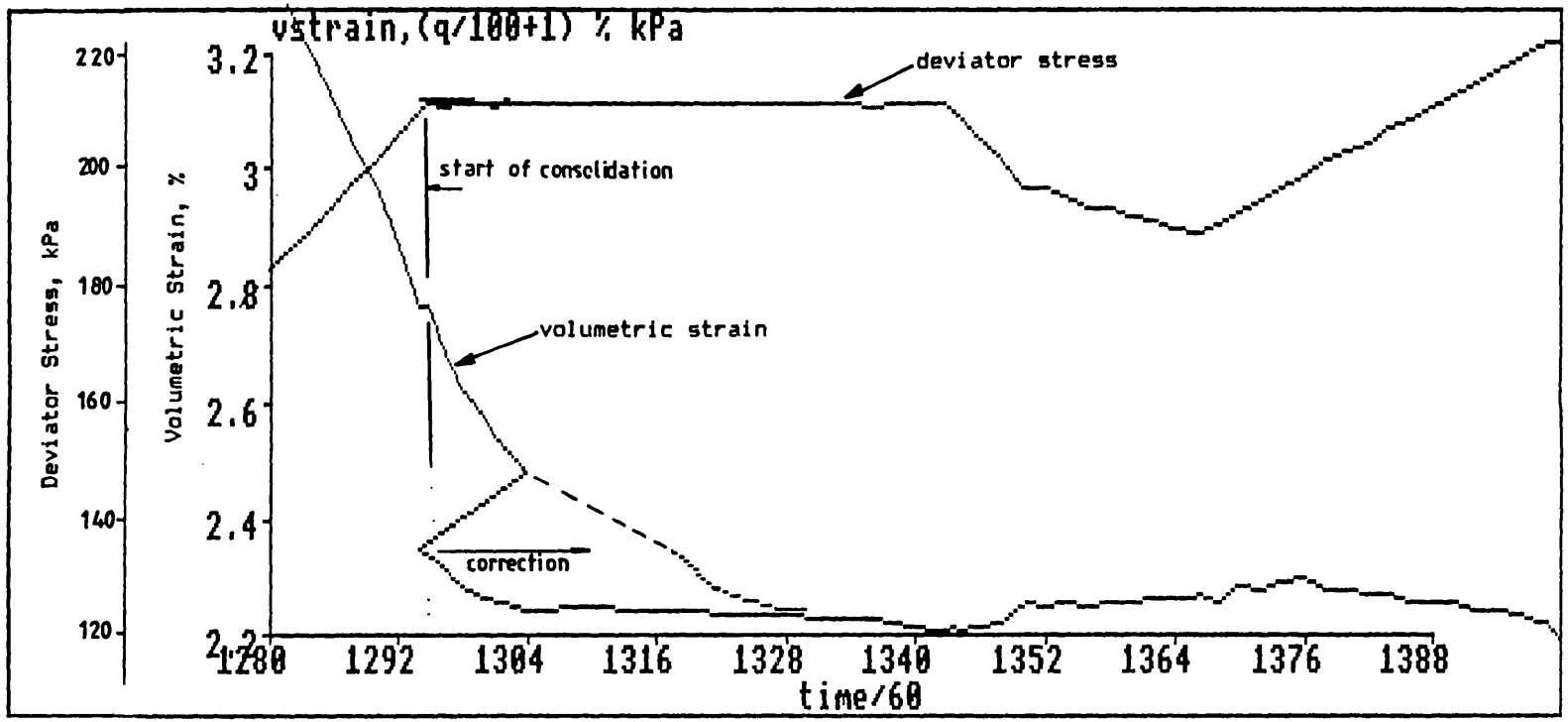


Fig.11.11 Consolidation under Constant Deviator Stress

CHAPTER 12

RECOMMENDATIONS FOR FURTHER RESEARCH

12.1 INTRODUCTION

Although there are still many questions to be answered about the behaviour of unsaturated highly plastic soils from Africa, this research has served to define further the present state of knowlege and to indicate where research efforts should be directed.

12.2 IDENTIFICATION AND CLASSIFICATION

The shortcomings of current methods of assessing the expansiveness of soils have been highlighted in Chapter 6. Expansiveness cannot be reliably determined from the application of Atterberg limit data or other classification data to swell tests on compacted soils.

The stress path oedometer data have shown that volume change of compacted clays can be due to a combination of swell and collapse. This may complicate the interpretation of swell data from compacted samples to determine the expansiveness, although the effect of structure is thought to be small for samples compacted near optimum moisture content.

As a first step toward a method for identification and classification, the compressibility and expansiveness of samples consolidated from a slurry are being investigated, under the author's supervision, (Jones 1988). If a relationship can be found between the compression data, for saturated, unstructured samples, and the soil index test values, then an attempt can be made to extend the study to swelling of unsaturated soils. The study would need to begin with samples consolidated from a slurry and then desaturated in a pressure plate cell, and then investigate compacted samples. Preparation of samples which are consolidated and then subjected to increasing suction to obtain desaturated samples may be a major problem.

12.3 VOLUME CHANGE

Significant progress has been made using the Stress Path Oedometers in the study of volume change of compacted samples under controlled suction. The stress paths which result from different loading and wetting sequences have been determined for

one soil type and for the particular set of conditions used in the tests.

12.3.1 EQUIPMENT

The Stress Path Oedometers have performed adequately, but there is scope for improvement.

12.3.1.1 Plunger Friction

The friction on the plunger has been shown to cause a small error in the axial applied stress. When the axial stress is large, this error is insignificant, but at low axial applied stress values, the error may affect the data. The vertical stress range of interest for compacted soils is usually from about 10kPa, or 0.5m cover, to about 50kPa, or 2.5m cover. It is desirable that the friction be reduced or eliminated by the use of some other sealing system. It may be possible to separate the plunger from the membrane and to supply the air pressure to the sample by some other route. The plunger could then be manually adjusted to permit vertical displacement to be measured, without the need for continuous contact between the plunger and the soil.

12.3.1.2 Wall Friction

The friction between the sample and the cell wall is also a source of appreciable error at low axial applied stresses. Reduction of this friction may prove difficult. One possibility is to coat both the rubber lining and the cell bore with PTFE. Although this cannot eliminate the friction, it may provide a more easily quantifiable error than the greased lining which seems to have a greater friction at the end of the test than at the start.

12.3.1.3 Radial Stress

The radial stress sensors have proved difficult to set up in the oedometers and, in one case, unstable. The drift monitoring system has been useful, although it was only operative in test SP05. The correction applied in this test ranged from 25kPa after 10 days to about 80kPa toward the end of the test, as shown in Fig.12.1. The observed and the corrected radial applied stresses are shown for the 200day test period. On removal of the sample from the cell, the uncorrected radial stres was -48kPa, while the corrected value was 21kPa. Some of the apparent scatter in the radial stress data is due to changes in the axial stress or due to pore water pressure disipation. However, not all of the scatter in the data has been eliminated with the drift monitoring system, and further improvement may be possible by altering the technique used for switching between modes.

The switching of the load cell configuration with the Din plugs is not ideal, and changes of up to 30kPa in output could be induced by handling the plugs, in particular the normal mode plug in socket "B" of Fig.9.20. The performance of the Din plug system was much better than two other attempts using toggle or rotary switches, but a better switching method is desirable.

12.3.1.4 Solute Suction

The uncontrolled solute suction changes are a cause for concern, and research should follow either, or both, of two routes. In the first, soils free of salts could be tested to determine the effect of matrix suction alone. This could be followed by tests on soils to which known quantities of salt have been added.

The alternative route is to develop a means of controlling the total suction. For the elevated pore air pressure technique this would involve the inclusion of a true semi-permeable membrane between the soil and the water supply. A technique for forming a semi-permeable membrane within a porous element was developed at the turn of the century (Partington 1951), and is being considered. This could also be used for an osmotic suction control system. Consideration is also being given to total suction control via the relative humidity of the air surrounding the sample, but this is probably not capable of providing the necessary precision of control of the suction.

12.3.2 TEST RESULTS

The three standard oedometer swelling tests have been compared in terms of the stress paths obtained from the stress path oedometer results, and shown to differ markedly in both stress path and final stress state. It has also been shown that the stress path has a very significant effect on the volume change of the compacted samples. The standard tests should not be used for compacted or structured soils, and laboratory tests for these soils should attempt to simulate the field loading and suction changes in the correct sequence.

The existing data can be used as a basis for further study of the stress paths followed on wetting and drying. Considerable scope exists for investigating the effect of compaction type, compaction void ratio, compaction moisture content, vertical applied stress and soil type.

12.4 FUTURE RESEARCH

Most shear strength and volume change testing of unsaturated soils has been done on compacted samples. This immediately brings the effect of soil structure and overconsolidation into the

analysis of the soil behaviour. Had the study of saturated soils proceeded along the same route, then it is probable that far less progress would have been made.

The direction of future research that is thought to be most beneficial for the study of unsaturated soils is to develop "normally consolidated reconstituted" samples of unsaturated soil. These would permit the development of a basic model or framework for unsaturated soil without the complications of structure and stress history. These additional factors would need to be studied and included in the model for compacted or structured soils after the basic model had been developed for the simpler reconstituted soil. A hypothetical model for the simple reconstituted soil is described below.

The initial testing would need to define the shear strength, AC_1C_2 in Fig.12.2, and one-dimensional compression, AB_1B_2, curves in the effective stress plane, by testing saturated samples prepared from a slurry. Similar compression, AF_1F_2, and strength, $AC_3C_6C_7$, curves would be required in the suction-void ratio plane. Thereafter, samples consolidated to similar effective stresses would be subjected to various values of suction, e.g. D_1, D_2, E_1 & E_2, before being sheared to C_4, C_{10}, C_8 & C_{11}, respectively. These tests would serve to define a shear strength envelope in suction and applied stress space, which would probably show a change in slope as depicted by $C_4C_7C_8$, although it may well be a gradual change. This may be a unique "normally consolidated" shear strength envelope, but the influence of stress path must also be studied.

Samples should, therefore, be subjected to varying combinations of suction and applied stress changes, but without reversal of the direction of stress or strain change, to reach the same final stress state before being sheared. These would still be normally consolidated samples in that neither the suction nor the applied stress would be smaller at the time of shearing than it had been previously.

This normally consolidated shear surface would need to be supplemented by an investigation of the effect of overconsolidation, both by applied stress and by suction. It is likely that the effects of the two stress types will not be the same, and the sequence of application of the two stresses will probably be important.

Extension to include compacted samples will be complicated by the combined effects of overconsolidation and soil structure or fabric. It has been argued in Chapter 10 that the soil within the clusters is overconsolidated and that the overall void ratio results from the relatively low void ratio of the soil in the clusters combined with the macrovoids between the clusters. Thus the soil can no longer be treated as homogeneous.

Shearing of a soil of this type will occur preferentially through the more highly stressed material in the contact zones.

The remainder of the soil may be largely unaffected by the shearing, while the soil that is sheared will dilate and soften.

This argument may be applied to Toll (1988) who proposed a set of critical state planes for compacted lateritic gravel which were dependent on the degree of saturation. The non-uniqueness of the final sheared states which Toll observed may in reality be a reflection of the preferential shearing through the contact zones and the inhomogeneity of the sample after shearing. The true critical state may only be a theoretically possible condition for the whole of a sample of highly overconsolidated material. The sample after shearing may be non-uniform on the micro-scale with clusters or aggregations of dense, unsheared material in a matrix of less dense soil which has been sheared to the critical state.

Once these aspects have been investigated, it may be possible to start assembling a model along similar lines to the critical state model for saturated soils. Since the void ratio surface is dependent on the mean applied stress, the deviator stress and the suction, the model will be difficult to portray on paper. An attempt has been made in Fig.12.3 to describe the normally consolidated void ratio surface for a range of major and minor applied stress ratios. The axes represent the mean stress and the suction on the two horizontal axes, both on a logarithmic scale, and void ratio on the vertical axis.

Line $AB_1B_2B_3$ is the virgin isotropic consolidation line for a saturated soil in effective stress terms because the pore water pressure on the void ratio applied stress plane is zero. Line $AC_1C_2C_3$ is the virgin consolidation curve for suction in an unconfined sample and is also for an isotropic stress state. Desaturation commences at C_1, where the consolidation curve deviates from a straight line. Line $AD_1DwD_2D_3$ represents consolidation by an arbitrary combination of suction and isotropic applied stress where desaturation commences at Dw.

In the region in which the suction and the applied stress are equivalent, the contours of equal void ratio, e.g. e_1 along $B_1D_1C_1$ will be circular in plan. After the sample desaturates on the plane passing through $C_1DwW_2W_3$ and the suction becomes less effective than the applied stress in causing compression, the contours of equal void ratio will deviate from the circular shape, (shown dotted), and become distorted as shown by curve $B_3W_3D_3C_3$.

Line $E_1E_2E_3$ represents consolidation with a major to minor applied stress ratio greater than 1, and the contours of equal void ratio for any applied stress ratio will converge at the virgin suction consolidation line. This occurs simply because the applied stresses on the suction-void ratio plane are zero. Desaturation for void ratios e_2 and e_3 is shown to occur at V_2 and V_3 respectively. The commencement of desaturation will occur for any major to minor applied stress ratio on the surface bounded by $W_3W_2C_1V_2V_3$.

The next stage of the study would be to determine whether or

not a unique rebound surface can be defined from a particular void ratio contour on the normally consolidated surface. Since the soil structure is not expected to play a major role, it is thought that a unique rebound surface is likely to exist. A hypothetical rebound surface from the e_3 contour in Fig.12.3 is sketched in Fig.12.4. The surface is bounded by $B_3W_3D_3C_3R_5R_3R_2R_1$, with resaturation occurring along $W_3R_4R_5$. An arbitrary swelling path is shown by points $D_3R_4R_4R_1$. This surface represents the swelling of a soil prepared as a slurry and consolidated to a void ratio e_3 by any combination of applied stress and suction.

A model of this type would form a basis from which to initiate the study of compacted samples. It will probably be necessary to make use of a parameter such as the Packet Void Ratio described by Brackley (1975) to determine the rebound surface for the soil within the clusters. The void ratio of the whole sample will also be needed to determine the state of the soil relative to the rebound surface for the soil in the clusters.

Consider for example a soil with packet void ratio and stress state defined by R_4 and void ratio defined by G, at the same stress state, in Fig.12.4. The packets will tend to swell along R_4R_1, while the whole soil will tend to swell alond GH. Structural changes will cause the whole soil to follow some as yet undetermined path within R_4R_1HG.

Fig.12.5 shows a vertical section along $D_3R_4R_4R_1$. The horizontal axis is some combination of suction and applied stress, while the vertical axis is, as labelled, the void ratio. Points G_1, G_2 and G_3 represent whole soil void ratios for three different samples which have the same packet void ratio e_4 at R_4. The curve G_1R_1 represents a soil in which swelling is dominant, and G_3R_1 represents a soil in which collapse is dominant. G_2R_1 shows an intermediate behaviour with some collapse followed by swell. The common end point with a void ratio of e_5 at R_1 assumes that the structure is completely transformed during the wetting process. Zein (1985) has shown that this does not occur at low stresses, and the true picture will be more complicated.

This is a hypothetical model, and would need a large body of test data to support it before it could be considered to be a proven model. However, it is probable that the understanding of the mechanics of unsaturated soils will not be advanced significantly without a model of this type as a basis from which to explore the more complex structured and compacted soils.

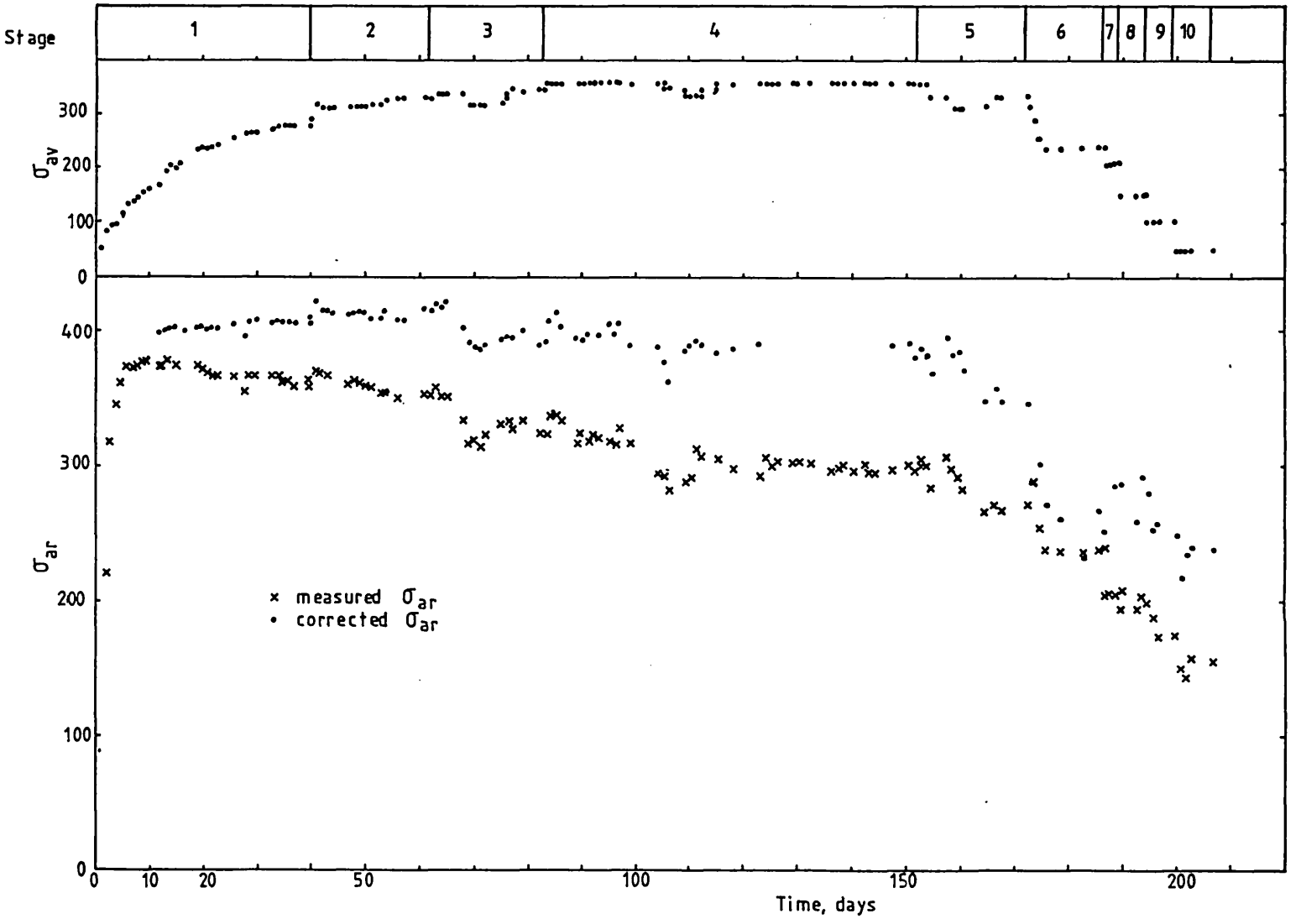


Fig.12.1 Correction of Radial Stress measurements

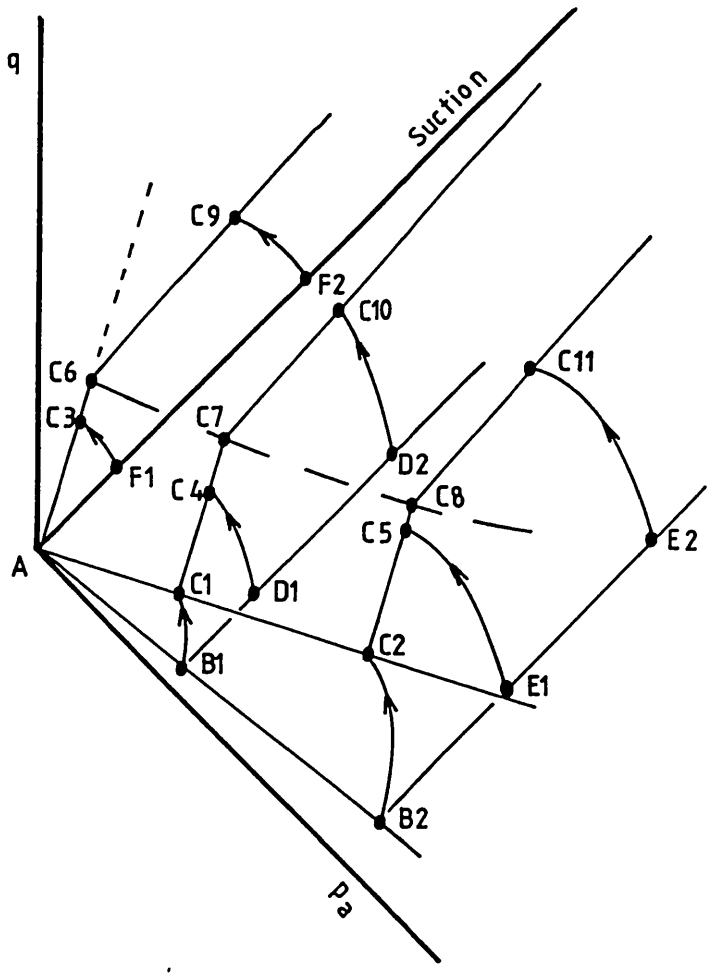


Fig.12.2 Shear Strength Surface

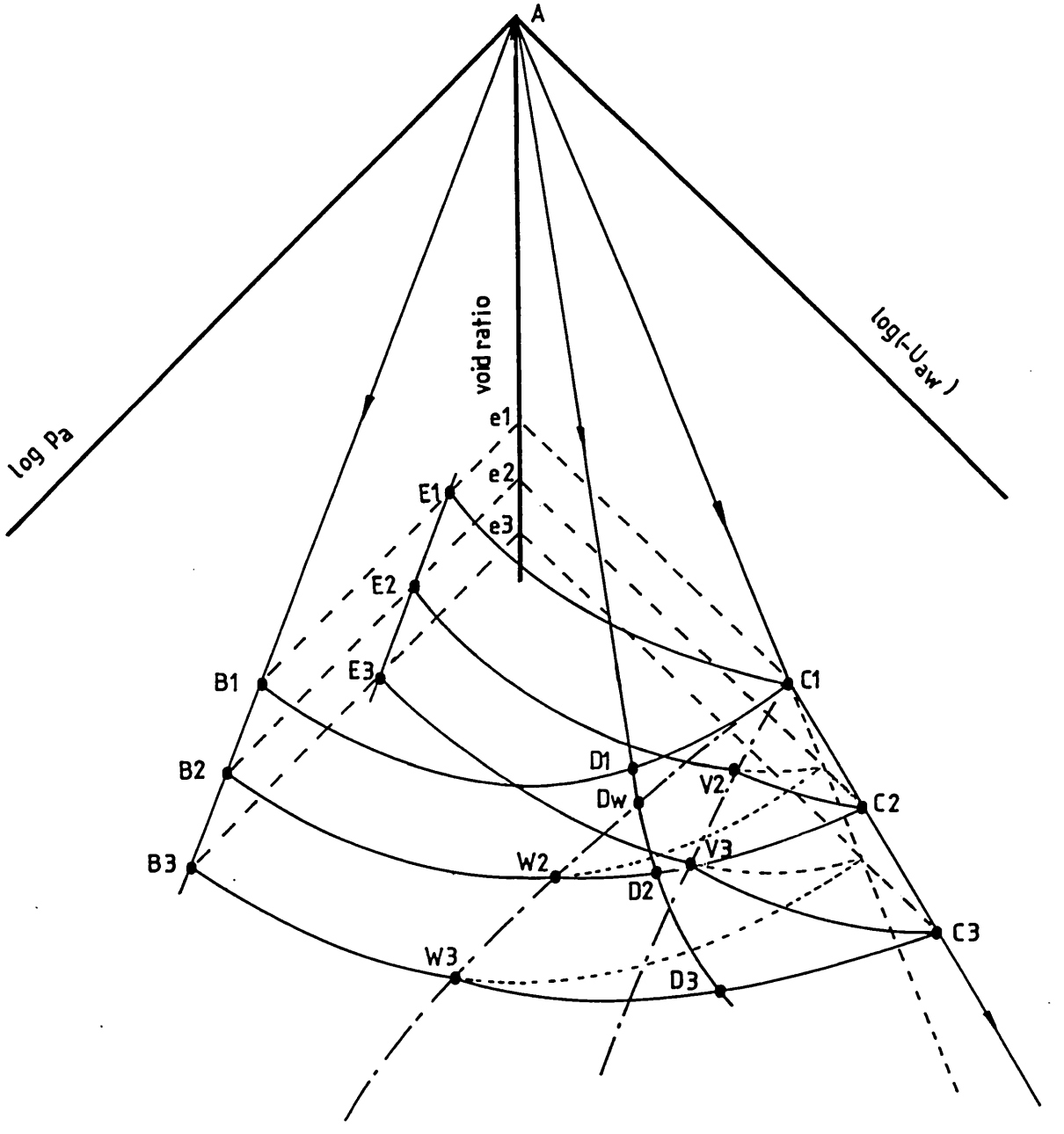


Fig.12.3 Consolidation Surfaces

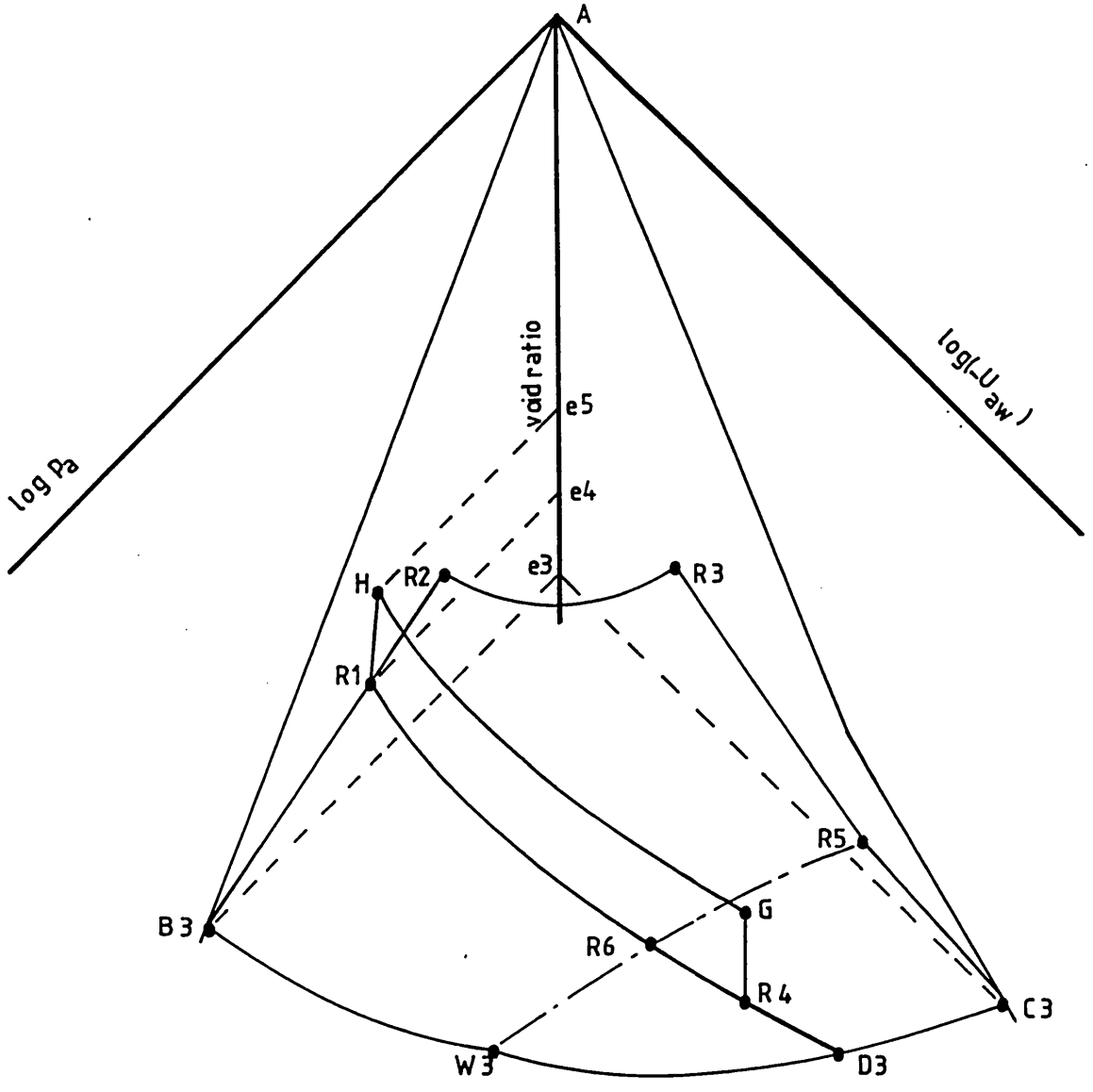


Fig.12.4 Rebound Surface

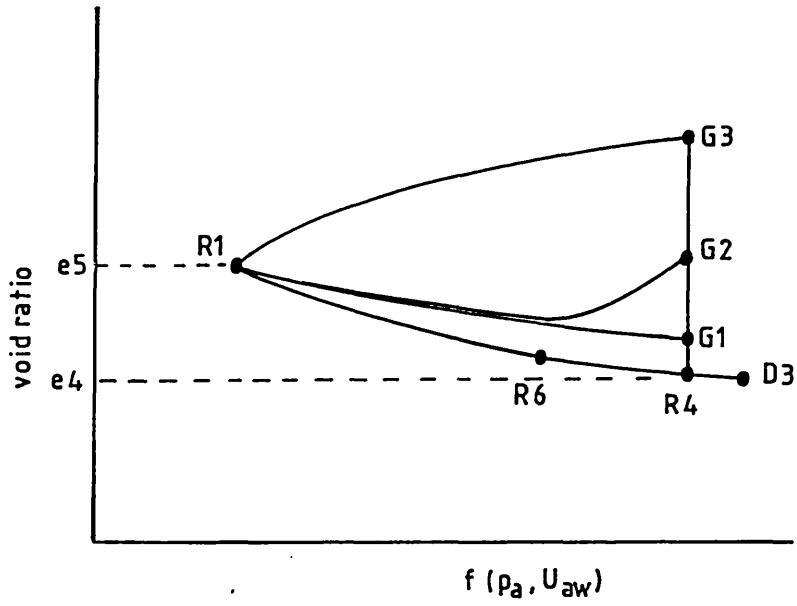


Fig.12.5 Swell and Collapse paths

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ABBREVIATIONS

In addition to the common abbreviations, the following have been used:-

ICES International Conference on Expansive Soils

CSMFE Conference on Soil Mechanics and Foundation Engineering

ICSMFE International CSMFE

ARCSMFE African Reg. CSMFE