THE SHEAR STRENGTH OF TWO RESEDIMENTED CLAYS

STRESS LEVELS .

AT LOW AND VERY LOW ~~EFFECTIVE STRESSES~~

by

KHALED ZAHI RAFIQ Kamhawi (B.Sc., M.Sc.)

A thesis submitted to the University of London for the

degree of Doctor of Philosophy. University College 1991.

ABSTRACT

THE SHEAR STRENGTH OF TWO RESEDIMENTED CLAYS AT LOW AND VERY LOW EFFECTIVE STRESSES

Two main means of measuring shear strength were used: the first was the direct shear apparatus with internal pore water pressure measurement and the second was plane strain tilting of a submerged, initially horizontal, 20mm layer in a tank. The lower limits of shear strength for the two clays, which exclude the strong effect of thixotropy at low stress levels, are the same as values reported in published studies covering stress levels three orders of magnitude higher (namely $\phi \equiv = 22.5°$for Speswhite Kaolin and $\phi \equiv = 21°$for London clay). The pore fluid was London tap water for most experiments with some variations also tried including added salt and coagulant.

Up to an effective consolidation stress of 5kPa, a large thixotropic component of strength was measured which was pseudo-frictional in that it was proportional to effective normal stress. Various experimental factors suggest that thixotropic strength may result from unsustainable dilatancy as flock bonds are broken. Any small degree of overconsolidation made this dilatancy sustainable within this stress range. Linear relationships between lower limit shear strength and overconsolidation ratio have been established. Clear relationships were established between shear strength and voids ratio for each clay when overconsolidated.

The correlations with pre-existing shear data and a wide variety of direct shear and tilting test data show there are two alternative rupture plane orientations: maximum stress obliquity and a no extension direction associated with coincident axes of stress and strain increment.

The absence of noticeable creep beyond 7 days after sedimentation was gratifying in that there is a possibility of simulating natural clay formation in a reasonable time frame.

ACKNOWLEDGEMENT

I would like to express my deepest thanks and gratitude to Professor J.R.F. Arthur who supervised this work. His constant suggestions and continuous guidance were greatly appreciated. I am grateful to his kindness and generous understanding which made this research possible. I am also extremely grateful to Dr T Dunstan for his interest and invaluable discussions throughout my work.

I would like to express my thanks to Dr. Ali Dalili and Dr. Andreas Kyriacou for their continuous support and guidance. Special thanks must go to Mr. John Ford and Mr. Denis Vale, who were always willing to help and for their invaluable advice, technical support and excellent workmanship in the fabrication of the apparatus. I am also glad to acknowledge Messrs. L. Wade, W. Vines and Eric Cuttler who were always helpful.

Many thanks to SERC who funded the research.

Last, but by no means least I would like to express my love and thanks to my family for their moral encouragement throughout my life.

DEDICATION

This work is dedicated to the people and the soil of Palestine. I hope that one day I will be able to serve them directly.

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER 2

CHAPTER 3

CHAPTER 4

CHAPTER 5

CHAPTER 6

LIST OF TABLES

CHAPTER 2

CHAPTER 4

CHAPTER 6

LIST OF PLATES

LIST OF SYMBOLS

ϕ'	Angle of friction measured in terms of effective stress
τ	Shear stress
σ_1'	Major principal effective stress
σ_3'	Minor principal effective stress
ϵ_1	Major principal strain
ϵ_3	Minor principal strain
$\delta\epsilon_1$	Incremental major principal strain
$\delta\epsilon_3$	Incremental minor principal strain
σ	Total normal stress
σ'	Effective normal stress
σ_v	Total vertical stress
σ_v'	Effective vertical stress
ψ	Angle of dilation
e	Void ratio
K_0	Coefficient of earth pressure at rest

ABBREVIATIONS

MSO	Maximum Stress Obliquity.
NED	No Extension Direction with Coincident axes of stress and strain increments.
LTW	London Tap Water.
D	Distilled Water.
S	London tap water with 35g/litre of salt.
N	Period of consolidation prior to testing.
OCR	Overconsolidation Ratio.
NGI	Norwegian Geotechnical Institute.
PTFE	Polytetraflouroethylene.
DSA	Direct Shear Apparatus.
SSA	Simple Shear Apparatus.
ADC	Analogue to Digital Convertor.
MCL	Minimum Critical Line.
VST	Vane shear test
CPT	Cone penetration test
PMT	Pressuremeter test
PWP	Pore water pressure

CHAPTER 1

INTRODUCTION

Offshore oil exploration has resulted in a variety of engineering work on the seabed and some of this work has required estimates of shear strength of soils at effective stress levels far below those previously considered of engineering interest. Cheap, quick and rather general techniques for measuring shear strength at low effective stress levels are reported and the fundamental understanding of clay behaviour is extended. The wide range of low stress levels and the drained shear conditions have allowed mechanisms to be deduced in a way that was denied to Skempton and Northey (1952) when reporting and interpreting undrained tests at a stress level appropriate to the liquid limit. Seed and Chan (1957) dealt empirically with a complex situation involving compacted unsaturated thixotropic soils; again basic mechanisms were obscured. Resedimented Speswhite Kaolin and London clays have been well researched so measurements by new technique could be validated by comparison with high quality data obtained from established techniques. Resedimentation might appear to be inappropriate preparation technique for estimating the shear strength of soils in the seabed in that any aging process insitu would be missed. However the actual results do show a short term aging process which was completed within the span of a few days, so there is a reasonable chance that there is direct relevance in the shear strength results obtained. Of course this can be checked by using a real seabed soil for which good insitu shear strength data is available. If it turns out that most aging processes in soils near the seabed are short term then the resedimentation technique has the added advantage of using cheap completely disturbed soil samples. The remarkably high shear strengths (in relative terms) measured in the study strongly suggest that this possibility will be realised. Other effects which may be investigated using these techniques are: the shear characteristics of soil containing gas bubbles using a similar sample preparation technique to that adopted by Sills et al. (1991) in their study of the behaviour of offshore soils containing gas bubbles; and the effect of cementation due to some chemical or biological agent as observed in some naturally occurring soils.

These high shear strengths were shown to be largely due to a thixotropic component which could be conceived as a positive dilation which diminished and finally vanished under a constant or increasing shear strained rate. A period of rest, with no strain, restored this component revealing its thixotropic nature. Unstable dilation is suggested because this component is proportional to effective stress over the range of stress in which it has been observed and can be incorporated in an effective angle of shearing resistance for soil undergoing slow stress controlled loading, or indeed very fast stress controlled loading. Over-consolidation achieved within the stress range of thixotropy was shown to convert the whole thixotropic component of shear strength to frictional shear strength in the conventional sense. Creep in one of the normally consolidated clays also had the effect of converting a proportion of the thixotropic component of strength; an effect in harmony with the thixotropic component probably being due to an unstable dilatancy.

Two main means of measuring shear strength were used: the first was the direct shear apparatus; and the second was a plane strain tilting of a submerged initially horizontal layer of clay in a tank. The shape of axisymmetric triaxial samples was judged too unstable for the very low shear strength samples which were to be tested, and the use of a simple shear apparatus was not entirely successful, probably because this apparatus was an adaptation from one intended for much higher stress levels. It is clear from the work of others that properly set up simple shear apparatus would have been suitable (Dyvik et al. (1987)). The use of tilting a thin layer of clay around 20 mm thick was risky because with stress levels down to 0.05 kPa there would be great doubt about any interpretation unless good correlation with conventional tests and interpretation could be achieved. Eventually this correlation extended for the Kaolin up to 600kPa (with Cambridge simple shear and true triaxial data). For the London clay a similar extension was achieved with data from triaxial tests carried out at Imperial College. This was fortunate; the simplicity and versatility of the tilting technique allowed effects of short term aging and overconsolidation to be extensively investigated using a very wide range of stress controlled incremental loading rates. The differing effects of stress control and displacement control were found by comparing data from the direct shear apparatuses which incorporated internal pore water pressure measurements.

The differing boundary conditions of the apparatuses presented an opportunity to learn more about the factors that determine the orientation of rupture layers. As might be expected the results suggest that two orientations are strongly favoured:

(i) the no-extension direction associated with pre-failure straining with coincident axes of stress and strain increment.

(ii) the coulomb planes of maximum stress obliquity.

Simple consistent interpretations suggested that there were only very small deviations from one or other of these two orientations.

Chapter 2 reviews the existing laboratory and in-situ testing methods for obtaining soil properties. Chapter 3 outlines the research strategy of this work and describes the behaviour of clay under shear. Chapter 4 describes the apparatuses used in this research programme. Chapter 5 provides the properties of the clays used and describes the method of sample preparation and setting up procedure. Chapter 6 presents the results obtained from each apparatus. Chapter 7 provides an interpretation and discussion of the results and finally Chapter 8 lists the main conclusions of this work and provides suggestion, for further work.

CHAPTER 2

REVIEW OF SOIL PROPERTY MEASUREMENT.

2.1 Introduction

Application of shear loads in determining the strength characteristics of soil has been a common practice since the 1940's. Many versions of shear testing apparatuses have been in use for the determination of the soil properties to be utilised for the design of structures. Laboratory testing is an essential component of geotechnical investigations for marine problems, and generally provides the majority of data for the geotechnical analyses. Data from in-situ tests usually serve to complement the laboratory data.

The following sections evaluate commonly used soil testing apparatuses and their capabilities for operating at relatively low stress levels. However, testing of soils at low stresses has proved to be of great difficulty as the mechanics of the apparatuses and surrounding conditions contribute to large unacceptable errors. The difficulty of handling saturated clays at low stresses is of great importance as mishandling results in irrecoverable disturbance to the sample. Measuring devices also prove to be of low accuracy when operating at very low and small ranges.

2.2 Review of measurements of shear strength and density / voids ratio in the laboratory.

2.2.1 Measurement of shear strength

2.2.1.1 Direct Shear Apparatuses

Direct shear tests involve the shearing of a relatively thin sample with strain centred along a plane defined by a horizontal split between two rigid frames which form the sample enclosure.

(a) Direct Shear Box.

The most commonly used direct shear device is the direct shear box in which a rectangular specimen is encased in a horizontally split box. A normal force is applied to the sample through the top half of the box, after which a shearing force is applied to move the top half of the box relative to the bottom half. The kinematic constraint that forces the failure plane to occur in a particular direction also introduces unknown conditions, such as the uncertainty of principal stress directions and strain distributions within a sample tested in the shear box. Finite element analysis of an idealised elastic-perfectly plastic isotropic soil in the direct shear box revealed that, despite the strongly non-uniform stresses generated prior to failure, the peak shearing resistance is close to the ultimate strength in a similar analysis for the simple shear apparatus (Potts et al. (1987)). These results obtained using finite element analysis are not reliable since soils do not exhibit an idealised behaviour throughout the shear test. However, although excellent improvement on the uniformity of strain distribution can be achieved by using large shear box (e.g.:- Assadi (1975); Arthur and Assadi (1977); Jewell and Wroth (1987)), the state of stress is still not uniquely defined by the boundary measurements of the average vertical normal stress and the horizontal shear stress.

The use of radiography to monitor the strain within a sand sample tested in a large direct shear box has provided evidence to suggest the horizontal plane might indeed be a plane of maximum stress obliquity and, as in the simple shear apparatus, also a no-extension direction (Arthur and Assadi (1977)). Attention has been drawn to the influence of shear box boundary conditions on the uniformity and orientation of principal stress and strain directions in direct shear tests (Jewell (1988)). Jewell distinguished between a shear box with a symmetric arrangement, in which the top platen, through which the normal load is applied, is rigidly fixed to the vertical sides of the upper half of the shear box; and the asymmetric arrangement, in which the top is free to rotate (e.g. Casagrande arrangement or the use of pressure bags as employed by Arthur and Assadi (1977)), Figure 2.1. The similarity between the symmetric direct shear tests and the simple shear tests was also demonstrated by Jewell (1988) and, in agreement with Airey (1987), Jewell considered the symmetrical arrangement to be preferable. The Casagrande arrangement is deemed to result not only in non-uniform rotational deformation of a sample but also in a higher shearing resistance which under estimates the dilation angle on the central plane (Jewell and Wroth, 1988). The subject of anisotropy in the interpretation of direct shear tests has also been discussed by Tatsuoka (1987), thereby further emphasizing the caution one must exercise when interpreting results from seemingly simple and direct test.

(b) Ring shear cell.

A further variation of the direct shear cell which has gained acceptance for both soils and materials testing, is the ring shear cell. As the name implies, it is an annulus in which the top half is rotated relative to the bottom half. The variation in the stress distribution, which results from the sample geometry, is analytically allowed for by making simplifying assumptions as to the variation of normal and shear stresses across the sample (eg:- Bishop et al. (1971)). Apart from the major kinematic constraint associated with direct shear devices, i.e.:- forcing the failure plane to occur in a particular direction, the ring shear cell loses the disadvantage of end stress concentration and strain discontinuity. Moreover, prior to the formation of a rupture plane in a specimen, the boundary conditions are such that the horizontal plane is a no-extension plane. Furthermore, the ring shear cell possess the added advantage of an unlimited strain capability and also maintaining a constant shear surface area. These features have enabled residual strength of soils to be investigated (Bishop et al. (1971)).

In summary, the average stress-strain curve obtained from measurements of shearing force and shearing displacement in direct shear devices cannot be depended on to provide a true stress-strain picture for the assumed stress conditions, i.e.:- σ_n' and τ_h acting on a horizontal failure plane. In direct shear devices the principal stresses are not known. The magnitude and direction of the principal stresses are inferred by assuming either maximum stress obliquity condition or a no extension direction with coincident stress and strain increments on the assumed horizontal rupture plane. Evidence to these effects have been demonstrated in special cases (Arthur and Assadi (1977)); Jewell and Wroth (1988)). Special techniques are also required to provide a more accurate picture of developed strains such as using lead shots imbedded in the sample and X rayed at different intervals during the shearing process. Total stress behaviour can only be deduced as pore pressure measurement has not been possible and therefore drained or undrained testing cannot be assumed. However, direct shear devices have been mainly used to provide comparative studies of various materials tested under the same conditions. They are also used often to simulate typical element of a rupture layer giving an apparent direct link to mechanism of field failure. Moreover, the simplicity of operation and the speed of obtaining results still makes them quite popular in many laboratories.

2.2.1.2 The Simple Shear Apparatus

In an attempt to overcome some of the limitations of direct shear devices, an apparatus was developed in which a uniform condition of simple shear can be imposed such that the entire sample becomes the zone of failure (Kjellman (1951)). This apparatus was given the name simple shear to reflect the mode of sample deformation. Further developments over the years have resulted in essentially two types of simple shear devices. One type, the Cambridge simple shear device (Roscoe (1953); Stroud (1971)), tests a sample which is square in plan and enclosed within rigid platens which are linked together with hinges and slides such that the necessary simple shear deformation may occur. The other type, the Norwegian simple shear apparatus (Bjerrum and Landva (1966)) which is a refinement of Kjellman's apparatus, tests short cylindrical samples enclosed within a rubber membrane. The membrane is reinforced with a spiral binding in order to encourage the desired inextensional plane strain mode of deformation. When successful, the kinematic restraint of the simple shear apparatuses ensures the horizontal direction is the no-extension direction, this is achieved especially in the Roscoe version.

As with the direct shear devices, the only initial state of stress which can be produced in a simple shear specimen before application of shear loading is that corresponding to so called K_o condition (some lateral strain is inevitable). Although K_o can be varied to some extent by overconsolidating the specimen, no other initial state of stress is possible. The condition imposed to achieve the deformed shape is a boundary displacement which, for homogeneous sample strain, becomes strain controlled with the horizontal shear stress (τ_h) generated as a response and is thus measured by some suitable device. The inability to apply complimentary shear stresses on the ends of the sample; implies a non-uniform distribution of shear stress over the top and bottom surfaces of the sample and also a corresponding non-uniform distribution of normal stresses over these surfaces so as to maintain the moment equilibrium of the sample.

Analysis of the stress and strain distribution within a sample have illustrated that deformation is considerably more uniform than stress (eg:- Roscoe (1953)). Also, empirical observations reported by Casagrande (1975) and by Wood and Budhu (1980), show that soil samples are certainly not deterred from developing variations in density within the sample. More recently, Airey & Wood (1987), reporting on simple shear tests on normally consolidated Kaolin samples -in which radiography was used for internal strain measurement-, showed the stresses and strains to be uniform in the central region of the sample, away from the vertical boundaries, until failure. In view of the above observation, they concluded that stress and strain variables determined from the central region are representative of a uniform simple shear deformation in plane strain. The deformation mechanism of the simple shear apparatus imposes an unavoidable and uncontrollable rotation of principal stress directions during monotonic loading of a sample. A comparison of the uniformity of stress for sands and clays by Airey and Wood (1984) showed that the uniformity is much improved for the more plastic clay samples and, in consequence, they suggested that the results from simple shear tests on clays can be presented with more confidence than that of sands.

The previous sections highlight the possible errors in directly comparing the results from different apparatuses and emphasizes the need for one to be wary of the differences in test procedures and conditions.

2.2.1.3 Conventional Axi-symmetric Triaxial Apparatus

This is the most popular shearing device for performing laboratory tests to study the stress-strain behaviour of soils. In the triaxial test, a cylindrical sample is placed under a state of hydrostatic stress and an axial load is applied to shear the sample. The radial stress which provides two of the principal stresses is applied to the sample through a rubber membrane, while the axial stress is imposed through rigid end platens. The effects of loading platen friction on the uniformity of stress distribution and the deduced stress-strain behaviour has been acknowledged as early as the 1940's by Taylor (1941) and subsequently by Balla (1957) and, Rowe and Barden (1964) amongst others.

Uniformity and overall reliability of the triaxial test with regard to strain has also been evaluated using radiography. The radiographic techniques illustrate relatively homogeneous strain states develop within the sample away from the ends restraints (Kirkpatrick and Belshaw (1968); Kirkpatrick and Younger (1970)).

Attention has also been drawn to the confining effects of sample membranes and, analytical corrections to account for these have been proposed (Bishop and Henkel (1962); Duncan and Seed (1967)). Also the effect of triaxial testing of sensitive clay samples with paraffin as cell pressure fluid in order to avoid the use of sample membranes has been described (Iverson and Moum (1974)). Attempts at using the triaxial cell for testing cohesionless granular material at extremely low confining stress ($\sigma_3{}' = 5$ kPa) have involved the application of analytic membrane and frictional corrections to the measured results (e.g.:- Ponce and Bell (1971); Fukushima and Tatsuoka (1984)). However, owing to the uncertainty as to the true low stress level behaviour, one is not sure of the validity of the corrections. A simplification of the current perception of membrane/sample and friction/sample interaction effects (e.g.:- Duncan and Seed (1967) Bishop and Green (1965)) usually results in the assumption of uniform distribution of apparatus effects on the sample. Experiments at higher stress levels have illustrated the occurrence of membrane penetration with granular samples (e.g.:- Molenkamp and Luger (1981); Tatsuoka et al. (1984); Tatsuoka and Haibara (1985)). Clearly, this effect will occur at low stress levels and can result in non-uniform stretching of the sample membrane. Hence the confining effects induced by stretching of the membrane will not be equally applied to all the grain/membrane contacts, and the strength of the rubber membrane will give these variations increasing significance as the stress level decreases.

2.2.1.4 True Triaxial Apparatus

True triaxial testing in which all the principal stresses can be varied independently is usually carried out on cubical samples. There are three basics categories depending on the applied boundary conditions. The first category constitutes strain controlled devices with all rigid boundaries such as the Cambridge true triaxial apparatus (Hambly (1969); Pearce (1971)). The second category constitutes stress controlled devices with flexible boundaries (Ko and Scott (1967); Arthur and Menzies (1972); Sture and Desai (1979)). The third category is made up of mixed rigid and flexible boundaries which applies known displacements in some directions and known stresses in others (Shibata and Karube (1965); Green (1971); Lade and Duncan (1973)). The relative benefits and drawbacks of the various types of true triaxial devices have been treated in detail by Sture and Desai (1979). A similar treatment on the constraints imposed by rigid and flexible boundary loading on cubical samples has been presented by Arthur (1987).

The ability to monitor independently the three principal stresses in a true triaxial device allows the whole principal stress and strain space of a material to be explored. The ability to change principal stress direction during a test has enhanced the general understanding of soil behaviour with a more realistic simulation of insitu stress paths. However, most of the true triaxial testing devices are mechanically very complex and require special care in the preparation and placing of specimens. Therefore, these devices are mainly used as research tools for the development of constitutive equations and determination of soil properties. The mechanical complexities coupled with the significance of apparatus constraints diminishes the appeal of the true triaxial devices for extremely low stress level work.

2.2.1.5 Plane Strain Biaxial Devices

In plane strain testing the frequently relevant field condition of zero intermediate principal strain is maintained. A pair of opposite faces of the biaxial apparatus are prevented from moving whilst stresses are applied on two other pairs of faces. However, the term plane strain can be considered a misnomer, in that, the imposition of immobile boundaries at the intermediate principal stress faces only ensure a zero average strain between the these boundaries and not necessarily within the sample. The freedom of movement in the direction parallel to the restrained surface is always ensured by lubricating with an effective grease, which will allow small straining in the "plane strain direction". A consequence of keeping zero intermediate principal strain during the stress path is generally a variation in the intermediate principal stress. However this variation is not thought to have any effect on the measured stress ratios at failure. Hence, the biaxial devices can only serve to check the stress-strain behaviour of soil measured in more versatile equipment.

2.2.2 Density / void ratio measurement

Laboratory voids ratio measurement of saturated soils has been a straight forward procedure. An excess part of the sample is weighed, oven dried and reweighed. These measurements can then be used to work out the moisture content and the voids ratio.

However the above standard method requires the destruction of the sample and a moisture content distribution in the sample would be difficult to obtain. Been (1981) introduced a nondestructive soil bulk density measurement by X-ray attenuation. This was achieved by projecting a collimated beam of radiation towards a soil sample and measuring the proportion of radiation that passes through the sample with a scintillation crystal and photomultiplier assembly. The output from this system is a count rate, in pulses per second, that can be converted to density by experimental calibration, by using soils of known density in a similar container and geometric configuration to the sample. Apart from the good accuracy of this method, it also has the advantage of allowing density measurements during the process of sedimentation, consolidation and creep.

2.3 Review of measurements of shear strength and density / voids ratio in-situ offshore.

There are essentially two main systems to place insitu testing tools: through the drill string or from a tethered sea bottom platform (Poulos (1988), Briaud and Meyer (1983)). These are illustrated in Figure 2.2.

Either a motion uncompensated string or a stabilized drillstring may be used. The former is simpler and has the drillstring moving with the boat, so that the drill bit moves up and down in the hole by an amount equal to the vertical motion of the boat. Only insitu tools that do not need a controlled vertical push can be used, and then only for a vessel heave of up to 3m. Stabilization involves suppression of the vertical motion of the drillstring with respect to the bottom of the borehole, and the supply of a reaction force to resist the vertical thrust of the test device.

Tethered sea bottom are frames resting on the sea floor and connected by flexible cables to the vessel. They do not provide drilling capability but can provide up to 200 KN of thrust.

2.3.1 Measurement of shear strength

The most widely used in-situ test techniques are the vane shear test, the cone penetrometer test, and the pressuremeter test. This section provides a brief overview of these methods and other less common methods used to obtain insitu offshore soil parameters and is not intended to provide any further details since it is beyond the scope of this project and the experience of both the author and the department he worked in.

2.3.1.1 Vane shear tests (VST).

This test is performed mainly to obtain a value of the undrained shear strength of cohesive soils. The tools available to perform a VST offshore are the remote Vane and Halibut (McClelland, Doyle et al. (1971)), MITS (Woodward - Clyde, Briaud (1980)), Fugro Vane and Lehigh Underwater Tower (Leigh University, Richards et al. (1972)). Table 2.1 Summarizes the data on these pieces of equipment. The use of the VST is the limited to cohesive soils but has the advantage of being the simplest offshore in-situ test, it can be used with motion uncompensated drill string and the tool itself is simple compared to the cone or the pressuremeter.

The main use of the VST consists of obtaining an undrained shear strength profile which will help the engineer in choosing a design undrained shear strength profile. Unconsolidated undrained triaxial tests on driven samples and miniature vane tests on driven samples usually give undrained shear strength value equal to 3/4 of the in-situ vane values (Quiros et al. (1983)). As a result the trend has been in the past to use undrained shear strength values corresponding to 3/4 of the in-situ VST values for design purposes. However it has been shown recently that the undrained shear strength values from unconsolidated undrained triaxial tests and miniature VST on 3" pushed samples are equal to those obtained using an in-situ VST. This has warranted the use of full in-situ VST undrained shear strength value for design purpose. It is important to note that these results apply to the McClelland Remote Vane and cannot yet be generalized until full proof is obtained with other VST systems.

Generally the VST undrained shear strength values will present less scatter in the strength profile than those obtained from driven samples, due to sample disturbance caused by the process of sampling, and therefore, a VST undrained shear strength profile can be selected with more confidence. However recent data supplied by Fugro McClelland indicate the VST can be used to measure shear strength as low as 1kPa, but there is a large scatter in the results. This is attributed to the friction in the sleeve of the vane and the accuracy of the measuring devices, which is significant when measuring these low values. Bowden (1988) used a modified laboratory shear vane apparatus to study the shear strength characteristics of a natural silty clay sedimented in the laboratory. He reported that the accuracy of vane shear strength measurement in his experiments was less important than their precision because the objective was to study the relative influence of experimental conditions and time on shear strength and its development. Using consistent procedure, the precision of torque measurement was ± 0.1 Nm or, equivalent, a shear strength of ± 4 N/mm^2. Residual and peak strengths were of the order of 10 to 80 and 60 to 500 N/mm^2, respectively.

2.3.1.2 Cone penetration test (CPT)

The cone penetration test can be used to obtain continuous profiles with depth of cone point resistance, sleeve friction and, in more recent devices pore pressure. The tools available to perform a CPT are String or tethered mode (Briaud (1980)), and Swordfish (Meyer et al. (1982)), Wison III in drill string anchor or seabottom reaction frame mode (Ruiter (1975)), Seacalf (Briaud (1980)), LUT (Richards et al. (1972), MITS (Briaud (1980)) and XSP-40 (Beard and Lee (1982)). Table 2.2 summarizes the data on these pieces of equipment. The measurement of point resistance and pore pressure includes the hydrostatic pressure and the soil response; for deep water testing compensation for hydrostatic pressure is a must in order to obtain accurate values for the soil response itself. Recent data supplied by Fugro McClelland indicate the CPT can be used to measure shear strength as low as 1kPa, but as with the CPT there is a large scatter in the results, which are attributed to the selection of empirical factors to calculate shear strength and the accuracy of the measuring devices, which is significant when measuring these low values. The CPT can only be used with a well stabilized drill string or a tethered sea bottom platform; this makes its use more cumbersome than the VST.

One essential use of the CPT results is to accurately define stratigraphy. For example the exact thickness of a sand layer at depth can be obtained with accuracy from the point resistance profile; the strength and thickness of that layer may be critical for pile capacity and pile driveability. Also, the friction ratio (skin friction/point resistance), and pore pressure response in the case of the piezocones can be used to classify the soil penetrated (Briaud et al. (1982), Marr and Endley (1982) and Schmertmann (1978)) and provide a complement to laboratory classification. Undrained shear strengths of clay can be computed using the data provided by the CPT by using formulas and empirical factors determined from correlations (Briaud et al. (1982), Marr and Endley (1982) and Schmertmann (1978)). Dynamic pore pressure profiles are not commonly used; however they generally indicate stratigraphy with even greater precision than the point resistance profile; also the magnitude of the excess pore pressure profile is an indication of the permeability and density of the soil. The point resistance value can also be used to estimate the relative density of a sand (Schmertmann (1978)). In many cases it is difficult to obtain undisturbed sand samples in the laboratory testing and therefore obtaining the relative density using the point resistance values may be the only way of estimating the relative density which is a key factor in liquefaction studies.

The wide use of cone penetrometers has simulated extensive research into quantitative interpretation of the data to obtain geotechnical design parameters. Useful reviews of procedures for cone test data interpretation are given by Schmertmann (1978), De Ruiter (1981), Lunne and Christoffersen (1983), Briaud and Meyer (1983), Wroth (1984) and Jamiolkowski et al. (1985).

A similar laboratory device is that of the fall cone test, used as a British Standard to obtain the liquid limit for clays. Alteration of the weight of the cone and the fall distance could allow this device to be used to obtain undrained shear strengths for clays at a variety of stress levels.

2.3.1.3. Pressuremeter test (PMT)

The pressuremeter test has been used offshore for a number of years and is gaining acceptance as a valuable in-situ device for marine applications. The test involves expansion of a cylindrical probe into the surrounding soil in a borehole and measurement of the pressure necessary to expand the probe and the change in probe volume or diameter. The PMT has not been used routinely in offshore investigations, however three tools are now available; the Pushed In Pressuremeter or P.I.P. ((Reid et al. (1982)), the WIreLine Expansometer or WILE (Legier (1982)) and the Pressiometer Autoforeur Marine or PAM (Brucy et al. (1982)). Table 2.3 summarizes the data on these pieces of equipment. Pressuremeter probes are complicated pieces of equipment: the pressure in the probe includes the hydrostatic pressure in the borehole and the soil resistance, the PMT probe must be pressure compensated for an accurate measurement of the soil resistance when testing in deep waters. Data obtained using the PMT allows the prediction of: in-situ horizontal stresses of the soil, undrained shear strength of clay (although the undrained shear strength obtained has consistently been found to be greater than the corresponding values obtained from the vane test), the penetrometer test and the plate boring test (Wroth (1984), Mair and Wood (1984)), the shear modulus or Young's modulus of clay from the unload - reload portion of the pressure - volume curve (Palmer (1972), Ladanyi (1972), Baguelin et al. (1972), Wroth (1984)) and the determination of the friction angle of sand (Hughes et al. (1977)).

2.3.1.4 Other tests

A variety of tests other than the three discussed above have been employed to obtain qualitative or quantitative information on submarine soils. These tests include both geophysical and mechanical tests, and some of these are mentioned briefly below.

BOREHOLE LOGGING

Natural gamma logging involves the measurement of the level of natural radioactivity in the soil and therefore soil profile from established relationship. Once a borehole is completed, the probe is lowered to the bottom of the drillstring and logging takes place while the probe is raised at a constant velocity. Two uphole logs of the same hole require about two hours to run (Briaud and Meyer (1983)).

Other techniques include neutron measurements for determination of soil porosity, and gamma backscattering for determination of the bulk density of the soil. These methods require radioactive sources and are therefore costly and demand special precautions. Furthermore, the unknown annular space outside the drilling pipe reduces the accuracy of the porosity and density determinations (Andersen et al. (1979)).

SEISMO-ACOUSTIC TECHNIQUES

Relationships between seismo-acoustic propagation phenomena and geotechnical properties of soils have been derived using the theory of Biot (1956) (Hampton (1974), Taylor Smith (1983)). As reported by Poulos (1988), geophysical techniques, through the measurement of seismic wave velocity, electrical formation factor and acoustic impedance, allow definition of the sediment boundaries and their lateral distribution. Pockets of gas clearly show on the record, and thin clay layers are often identifiable on the basis of the phase relationships of the returned signals. In addition, similar data can be obtained from downhole, crosshole and inhole borehole measurements. Of particular importance is the shear wave velocity, which is essentially dependent on the stiffness of the soil skeleton and its variation with depth. The main engineering properties which can be deduced from seismo-acoustic measurements are shear modulus, one-dimensional consolidation compressibility and permeability.

PLATE LOADING TEST

The main purpose of plate load tests is usually to determine both soil stiffness and bearing capacity by fitting the measuring load-deflection behaviour to simple theoretical solutions for settlement and bearing capacity of a footing. Anderson et al. (1979) reported such tests being carried out at three locations in the North Sea, using the Seacalf system to load the plates. The main limitation of the plate load test is that it only tests the soil to a depth of 1 to 2 diameters below the plate. Hence, to obtain a vertical profile, plate tests need to be carried out at variety of depths which may prove to be very costly.

2.3.2 Density / void ratio measurement

As described in the above section density / void ratio measurement can be deduced from some of the in-situ tests. However a more accurate method has been developed recently, using the fact that the device is in direct contact with the soil and indirect interpretation of result is not required, as reported by Tjelta et al. (1985). A nuclear density device can be used to measure directly during penetration the in-situ density of saturated soils as well as being used as a CPT as described above. A radioactive source and radiation detector are built in a cone shaped probe with an effective area of 1500 mm^2. The nuclear density probe can assess directly the soil mass which is penetrated with standard cone penetration methods and equipment. During penetration the in-situ density is continuously recorded.

2.2 Reasons for extension of shear strength stress level testing range.

There are a number of situations in which engineers need to know the shear strength of soil from the seabed down to around 5m below, these include potential slope instability (natural, dredged and ploughed), scouring adjacent to pipe lines or gravity platforms, and penetration of jackup legs, platform skirts and piling templates). A fundamental understanding of the behaviour of soil at these ultra low stresses and how this behaviour relates to the behaviour of soils in higher stress ranges familiar to engineers is essential for offshore construction.

Conventional laboratory testing devices can only be used at relatively high effective stresses (above 25 kPa, Section 3.3). Apparatus interaction, sample disturbance and handling have been the obvious constraints at these low stress levels. Measurement of shear load and pore pressures are also of great difficulty at even lower stress levels due to inaccuracy of measuring devices at this range.

Methods have been developed to assess the behaviour of soils at low stress levels and reduce sample disturbance. The "SHANSEP" procedure of Ladd and Foot (1974), involves reconsolidating the sample to stresses well in excess of in-situ values, but retaining the same overconsolidation ratio and effective stress ratios. Provided that the soil exhibited normalised behaviour, the soil properties at the in-situ stresses could be determined. However as concluded from this research the behaviour of clay at low effective stress levels is markedly different and the "SHANSEP" method would be inappropriate.

Name	Years In Operation	Water Depth [ft] Design	Water Depth [ft] Actual	Depth Below Seafloor [ft] Design	Depth Below Seafloor [ft] Actual	Operational Method[2]	Vessel Type
Remote Vane	11	5000	1500	[1]	1500	MUDS/ SDS	Any drilling vessel
Halibut	6	5000	800	25	25	TSP	Any oceanographic or drilling vessel
Lehigh Underwater Tower	15	12000	12000	10	10	TSP	Any oceanographic or drilling vessel
MITS	4	1600	500	20	20	TSP	Any oceanographic or drilling vessel
Fugro Vane	3	1000	400	[1]	85	MUDS/ SDS	Any drilling vessel

NOTES: [1] included in design water depth.
[2] MUDS - Motion Uncompensated Drill String
SDS - Stabilized Drill String
TSP - Tethered Seabottom Platform
[3] Can be used on any drilling vessel if seafloor reaction frame is not required.

Table 2.1 Currently available vane equipment (Briaud and Meyer (1983))

Name	Years In Operation	Water Depth [ft] Design	Water Depth [ft] Actual	Depth Below Seafloor [ft] Design	Depth Below Seafloor [ft] Actual	Operational Method[2]	Vessel Type
PIP	3	1000	550	[1]	400	SDS	Vessel with moonpool [3]
WILE	3	1000	650	[1]	15	SDS	Vessel with moonpool
PAM	2	3300	370	200	100	TSP	Supply boat or large oceanographic vessel

NOTES: [1] included in design water depth.
[2] MUDS - Motion Uncompensated Drill String
SDS - Stabilized Drill String
TSP - Tethered Seabottom Platform
[3] Can be used on any drilling vessel if seafloor reaction frame is not required.

Table 2.2 Currently available pressuremeter equipment

(Briaud and Meyer (1983))

Name	Years In Operation	Water Depth [ft] Design	Water Depth [ft] Actual	Depth Below Seafloor [ft] Design	Depth Below Seafloor [ft] Actual	Operational Method	Vessel Type
Stingray, Downhole QS-CPT	6	2500	1200	600+	600	SDS	Vessel with moonpool
Stingray, Shallow QS-CPT	4	2500	1200	100	100 [4]	TSP	Vessel with moonpool
Wison III w/Drill String Anchor	4	2100 [1]	1000	[1]	800	SDS	Any drilling vessel
Wison III w/Seaclam	2	2100	650	[1]	330	SDS	Vessel with moonpool
Swordfish	1.5	1800 [1]	100	[3]	300	SDS	Any drilling vessel
Seacalf	10	2100	740	100	100 [4]	TSP	Vessel with moonpool
MITS	4	1600	500	20	20	TSP	Any oceanographic or drilling vessel
XSP-40	1.5	200	100	20-40	38	TSP	Any oceanographic or drilling vessel

NOTES: [1] Combined water depth and penetration is dependent on umbilical length.

[2] SDS - Stabilized Drill String
TSP - Tethered Seabottom Platform

[3] Dependent on drilling capability.

[4] Depends on soil type and strength.

Table 2.3 Currently available cone penetration equipment

(Briaud and Meyer (1983))

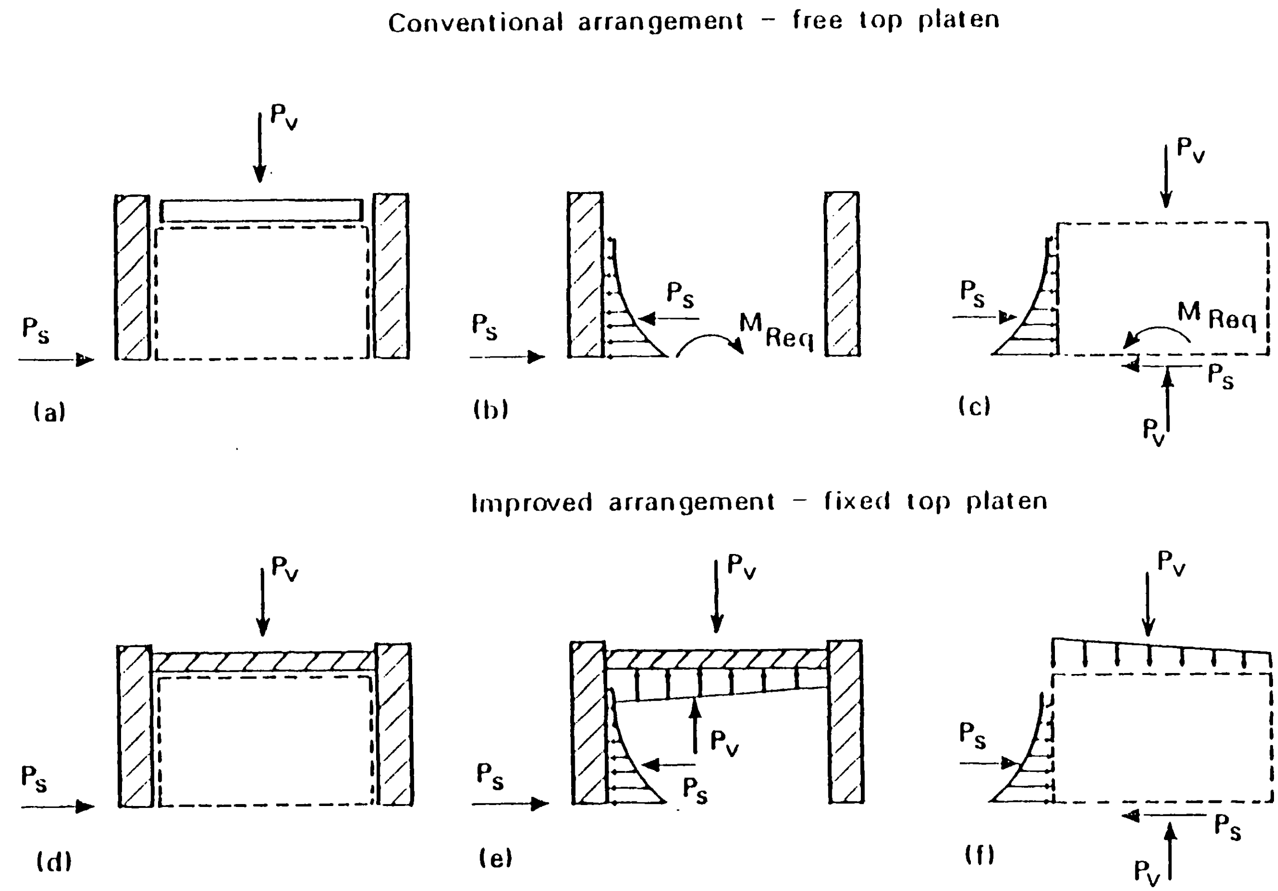


Figure 2.1 Schematic view of forces in a direct shear box with asymmetric (free top platen) and symmetric (fixed top platen) arrangement (after Jewell (1988)

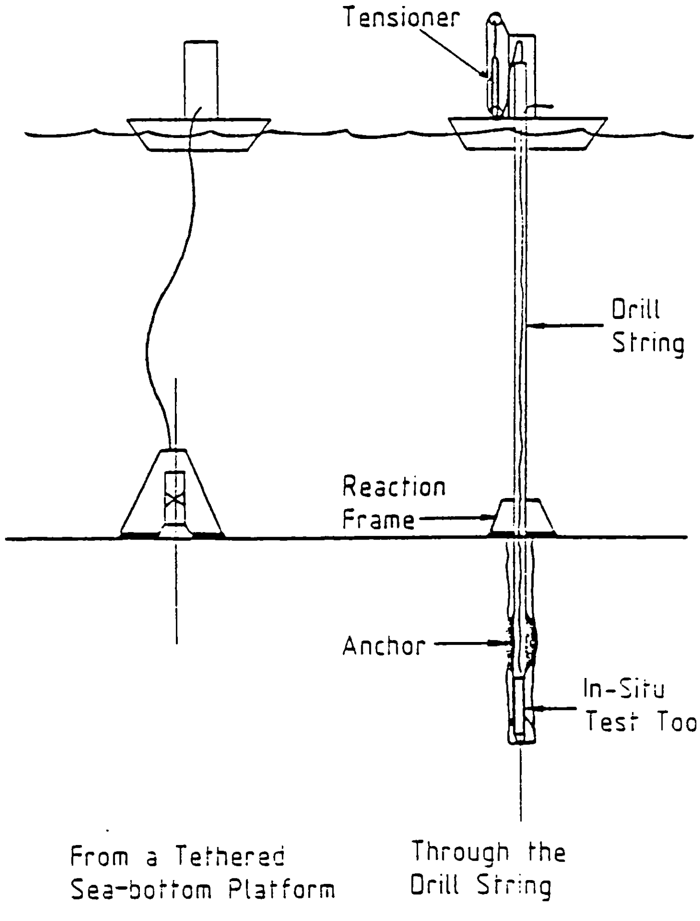


Figure 2.2 In-situ testing systems (Poulos (1988), Briaud and Meyer (1983))

CHAPTER 3

RESEARCH STRATEGY AND THE BEHAVIOUR OF CLAY UNDER SHEAR

3.1 Introduction

Application of shear loads in determining the strength characteristics of soil has been common practice since 1940's, and since then many versions of shear testing apparatus have been in use for the determination of design parameters (see Chapter 2 for review of apparatuses). The range of effective stresses involved in such apparatuses are also relatively large and appropriate to most common engineering structures. These devices are however unsuitable for use when testing very soft clays with very high voids ratios incorporating large moisture contents sometimes well above their liquid limit.

In search of an appropriate testing apparatus a direct shear box and a simple shear apparatuses were modified to operate at low stress levels (5 kPa) as the samples are short and stable which make them especially suitable for very soft clays. However, tests could not be performed on samples at lower stresses in these apparatus and a new technique was required.

The work carried out by Arthur et al. (1987) on tilting sand beds in a tray inspired the idea of a similar technique for clay. Tilting a clay bed under water in tanks was adopted and tilting tanks were built and continuously modified as described in Chapter 4.

This chapter reviews the preliminary development of the tilting tanks, the research strategy and the behaviour of clay under different testing conditions.

3.2 Preliminary development of technique

The preliminary development of the tilting tanks was carried out by Zadeh-Koceh (1989) at University College London. As mentioned above the idea was taken from similar tests carried out on sand by Arthur et al. (1987). Figure 3.1 outlines the final arrangement of the tanks achieved by Zadeh-Koceh. He found that it was possible to simulate the conditions of suspension, sedimentation, consolidation and creep in the tilting tanks. This had already been established by Been and Sills (1981), however he went a step further using infinite slope analysis to estimate the shear strength of the Kaolin clay by tilting the tanks to failure.

Initially the tests were carried out in rectangular glass tanks of various sizes to investigate the effects of varying boundary conditions. Several widths (100mm and 200mm), lengths (500mm and 1000mm) and 800mm deep were used and the sample height was also varied (20mm and 40mm as a result of the amount of dry Kaolin powder mixed for each tank). He observed that the behaviour of Kaolin was not affected within the above dimensions. The angles of tilt at failure after one day after sedimentation agreed with values of ϕ' published in other work. However he observed an increase in ϕ' with increase of period of sedimentation which he attributed to creep although there was no significant decrease in the voids ratio. This is in contradiction to the work carried out in this research (Chapter 6), as the angle of tilt at failure for normally consolidated Kaolin remained constant and was only dependent on the rate of tilting. The observation made by Zadeh-Koceh (1989) is believed to be incorrect as he did not standardise the rate of tilting of the tanks.

Zadeh-Koceh (1989) also considered other variations, increasing the effective stress by raining a layer of coarse sand on top of the clay bed after 24 hours of sedimentation, varying the water composition by adding salt at a concentration of 35grms/litre to imitate sea conditions and using distilled water. For all these variations he found that the shear behaviour of the clay did not change. Although the addition of salt resulted in a decrease in the sedimentation rate the final voids ratio was the same as that achieved using London tap water or distilled water.

The tanks used by Zadeh-Koceh (1989) were very fragile as they were made from glass, sealing of the tanks was very difficult and was not comprehensive . The permeable layer arrangement at the bottom of the tanks was very primitive so that further consolidation by downward seepage was not possible. Furthermore the tilting procedure was rough and inaccurate. This lead to necessary modifications as explained in Chapter 4.

3.3 Research strategy and the possibility of relating new technique results to conventional results

The aim of this work is to investigate the shear behaviour of clay at very low stress levels. These have not previously been studied because existing shear apparatuses are not suitable to record very small shear stresses and pore pressures and it is difficult to handle the clay samples at very high moisture contents, sometimes well above their liquid limit. Most of the data available in the literature describe the behaviour of clay at stress levels 3 to 4 orders of magnitude higher than those encountered in the tilting tanks (0.05kPa). In conventional laboratory soil testing apparatuses capable of recording effective stress behaviour and with known principle stress orientations, the lowest stress levels recorded in the literature, is that of tests performed by Lacasse et al. (1985) in the triaxial apparatus at a consolidation stress of 25kPa. However the purpose of their research was to investigate the quality of block sampling compared to the quality of the 95mm fixed piston sampler and no importance was given to the stress level.

The credibility of the tilting tanks in determining the shear behaviour of clay has to be established by comparing the results obtained in the tanks to those obtained in conventional shear apparatus. Inevitably any low stress phenomenon which might be encountered in the tilting tanks has to be confirmed in a conventional testing apparatus. This lead to modifications being necessary on both the conventional direct shear and simple shear apparatuses, as described in Chapter 4, allowing tests to be performed at a stress level as low as 5kPa. The table below shows the ranges that were covered by other researches and the ranges that can possibly be covered in the tilting tanks and the modified direct shear and simple shear apparatuses.

Stress level (kPa)	Apparatus	Researches
0.05 - 0.15	Tilting tanks	Author
5 - 50	Direct and simple shear	Author
25-1000	Triaxial and simple shear	Lacasse et al.(1985)
80-800	Simple shear	Borin (1973) etc.
600-1000	True Triaxial	Pearce (1970) etc.

The testing conditions in the tilting tanks have to be mimicked in the direct shear and simple shear apparatus: In the tilting tanks, as the angle of tilt increases the shear load is increased due to the buoyant weight of the clay acting parallel to the clay bed; consequently the test would need to be load controlled. The rate of tilting will also govern the rate of load application and therefore similar rates should be used when performing tests in the direct shear and simple shear apparatus. Finally the clays to be tested should have known shear strength properties which have been reliably measured by other researchers.

The work carried out by Zadeh-Koceh (1989) on the tilting tanks was related to conventional methods of testing soil in order to establish the suitability of the new apparatus. Well researched clays with known shear properties were tested in the tanks so that the shear properties of these clays can be compared directly. Zadeh-Koceh carried out tilting tests on Speswhite Kaolin and obtained values of the angle of tilt at failure of 23° after one day of sedimentation and 26° after 14 days of sedimentation and thereafter. The procedure of the test assumes that the clay in the tank exhibits an infinite slope of constant thickness throughout and therefore the angle of tilt at failure, considering a drained situation as the void ratio of the clay is very high (6) and pore pressure measurement indicated no significant increase during tilting, would represent the effective angle of shear resistance, ϕ'. The values of ϕ' obtained in his research agreed well with values obtained on the same clay by others in conventional apparatuses at much higher stress levels. Data for the same clay tested in the Cambridge Simple Shear Apparatus and in the Cambridge True Triaxial Apparatus give ϕ' as 22.5° and 23° respectively for mean normal stress levels from 80kPa to 600kPa (Borin 1973 and Pearce 1970). As mentioned above the increase in ϕ' after one day of sedimentation was attributed to creep, but this will be shown by the author to be a thixotropic phenomenon only occurring at these very low stress levels and only made intelligible by varying the rate of tilting (Chapters 6 and 7).

3.4 Fundamental behaviour of clay under shear

3.4.1 Introduction

The behaviour of clay under shear loading has been of a considerable interest in the past 60 years; many researchers have tried to observe, understand and model this behaviour using different methods and testing techniques. The rest of this Chapter investigates the shear behaviour of clay observed by other researches using different methods and techniques.

3.4.2 Effect of rate of shear

The effect of rate of shear on clay has been investigated by many researches; comments and conclusions have been made on both total and effective stress behaviour. Most of the researches have studied the effect of rate of shear on the undrained behaviour of clay, as drained tests have to be performed at a very slow rate to ensure no excess pore pressure development.

Bjerrum et al. (1958) carried out a series of drained and undrained triaxial tests with pore pressure measurements using marine clay from Fornebu, Oslo, covering widely differing rates of loading and consolidation pressure ranging between 100 and 400kPa. They showed that the undrained shear strengths decreased with increasing time to failure, and that this decrease cannot be explained solely by the observed increase in pore pressure at failure with time. They deduced that it must be a result of a reduction in the true cohesion and/or the true angle of internal friction, with increasing time to failure. However for the drained tests the drained strengths appear to be independent of the different rate of loading adopted in their investigation (1.3%/hour to 0.034%/hour) and therefore measured a unique effective strength parameters regardless of the rate of loading. They suggested that the reduction in the true cohesion and/or the true angle of internal friction with time, which was shown by the undrained tests, is compensated by an increase in drained shear strength, due to the decrease in water content, resulting from secondary time effect during the shear tests.

Vaid and Campanella (1977) carried out a series of triaxial tests to investigate the influence of rate of deformation on the undrained stress-strain and strength characteristics of an undisturbed normally consolidated saturated marine Haney clay at a consolidation stress of 515 kPa. The types of tests they performed consisted of conventional constant rate of strain shear, constant stress creep (the applied load is continuously adjusted as sample area changes and results in a continuously decreasing deformation rate that is later followed by an accelerating deformation rate to rupture if the stress level is high enough), variable stress creep, and constant rate of loading shear (the applied stress is actually decreasing as the sample area increases) among others. They concluded that increase in strain rate resulted in stiffer undrained stress-strain response and higher undrained strength; however in contradiction to Bjerrum et al.(1958) they observed no effect on the effective stress behaviour at failure as a result of the load or strain rate (i.e. ϕ', c').

Crawford (1959) carried out strain controlled consolidated undrained compression tests with pore pressure measurements on undisturbed sensitive marine clay (Ip = 39%) using time to failure varying between 6 to 600 minutes at a consolidation stress between 200 and 600 kPa. He found that the undrained shear strength varied linearly with log time and in terms of effective stress he observed that the angle of shearing resistance increased from 17.5° to 23° as the strain rate decreased while the cohesion intercept decreased at the same time.

Richardson and Whitman (1963) carried out a series of strain controlled undrained compression tests with pore pressure measurements on saturated remoulded fat clay (Ip = 38%). They found that the peak resistance in terms of total stress increased about 10% in passing from slow tests 0.002%/min., to the fast strain rates, 1%/min.. They explained this observation on the suggestion that at the faster rates of strain, adjacent soil particles find it more difficult to move relative to each other, and therefore tend to ride over one another unless constrained by an increase in effective stress. Thus the effect can be thought of either an increased resistance to compression or an increased tendency to dilation.

Nakase and Kamei carried out a series of K_0-consolidated undrained triaxial compression and extension tests on three cohesive soils under different strain rates using a consolidation stress of 392 kPa. They concluded the following: The maximum principal stress difference and axial strain at failure increase with increase of strain rate. The magnitude of excess pore pressure, both in compression and extension tests, at the same strain is found to be smaller as the strain rate increase in the case of plastic clay. They also observed that the slope of the critical state line for triaxial compression is not the same as the slope for triaxial extension, however, the values do not vary significantly with the strain rate. This result agrees with that reported Richardson and Whitman (1963) and the suggestion made by Schofield and Wroth (1968) that the final state of soil specimens flow as a frictional fluid at constant volume on the critical state line irrespective of various conditions.

3.4.3 Effect of secondary or delayed compression (creep)

It has been known for some time that a clay which has never undergone a reduction of overburden pressure may nevertheless exhibit behaviour closely resembling that of overconsolidated clay.

Bjerrum (1967) showed that the compressibility characteristics of a clay showing delayed consolidation cannot be described by a single curve in an e-log p diagram but require a system of lines or curves (Figure 3.2). Each of these lines represents the equilibrium voids ratio for different values of effective overburden pressure at a specific time of sustained loading. Consolidation tests have shown that the system of lines is approximately parallel (Taylor (1942), Crawford (1965)), indicating that the rate of delayed consolidation is about the same throughout a homogeneous deposit or, as the lines are actually slightly curved, it decreases slightly with increasing overburden pressure. He observed that there is a unique relationship between voids ratio, overburden pressure and time (Figure 3.2). This means that to any given value of overburden pressure and voids ratio there corresponds an equivalent time of sustained loading and a certain rate of delayed consolidation, independent of the way in which the clay has reached these values. In Figure 3.2 an additional curve, obtained by Bjerrum, is shown representing the undrained shear strength that can be mobilized at a given voids ratio for Drammen clay. Bjerrum concluded that there exists a unique relationship between undrained shear strength and voids ratio; where the undrained shear strength increases as the voids ratio decreases at constant overburden pressure and hence behaves as an overconsolidated clay.

3.5 Thixotropy

The term thixotropy was introduced in 1927 by Peterfi (1927) and used by Freundilch (1935) to describe the well known phenomenon of isothermal, reversible gel-sol transformation in colloidal suspension. Burgers and Scott-Blair

(1948) define thixotropy as a "process of softening caused by remoulding, followed by a time dependent return to the original harder state". Mitchell (1960) redefined thixotropy as an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume whereby a material stiffens while at rest and softens or liquifies upon remoulding.

Very little research has been carried out to investigate the thixotropic behaviour of clays and the available literature is insufficient to allow quantitative conclusion to be drawn. The measurement of thixotropic properties of a material is influenced by the testing techniques. Therefore only load control tests could quantify thixotropy as the displacement control tests would result in remoulding of the samples during the shearing process, and destroy part or all the thixotropic strength depending the displacement rate (Section 6.2). This explains why Zadeh Koceh (1989) observed an increase in the angle of tilt at failure (Section 3.2). As will be illustrated in Chapter 6, thixotropy is a low stress phenomenon, it apparently disappears as the consolidation stress increases beyond a certain level, and it is unique for each type of clay. However most of the research carried out on thixotropic characteristics of clay has been performed at either stress levels which were much higher than the limit where thixotropy ceases to exist as measured in this research programme; or the samples were prepared at moisture contents which are too low for thixotropy to exist or normal consolidation to be possible without further loading. The following is a review of available work carried out by other researches on thixotropic behaviour in clays.

Skempton and Northey (1952) illustrated the properties of purely thixotropic material as shown in Figure 3.3. In its undisturbed state the material has a shear strength of value c. When tested at the same rate of shear immediately after remoulding the shear strength is reduced to a value c_r. If the material is then allowed to remain under constant external conditions and without any change in composition, the strength will gradually increase, and after a sufficient length of time the original strength c will be regained. By means of the laboratory vane apparatus (Skempton and Bishop (1950)) Skempton and Northey (1952) tested three clays at water contents approximating their liquid limits and obtained the results shown in Figure 3.4. They reported that Kaolin shows almost no thixotropy and illite shows only a small effect. In contrast, the bentonite shows a remarkable regain at very short time intervals. It is not possible to suggest an upper limit for this material since the strength continued to increase throughout the experiment. They concluded that thixotropic strength regain decreases with decreasing water content below the liquid limit.

Seed and Chan (1957) performed a series of load control triaxial tests on samples of silty clay (Liquid Limit = 37, Plastic Limit = 23) compacted to a diameter of 1.4", and a height of 3.5" using the Harvard miniature compactor. They concluded that thixotropy can cause substantial changes in strength in some compacted clays, as it does in saturated clays, and may have significant effect on the strength of these soils tested after a period of storage, under conditions of slow increase in stress or under conditions of repeated stress application. They also observed that samples of the same age tested with a slower rate of loading have a greater thixotropic regain following deformation and ultimately exhibit a higher strength than those tested with faster rate of loading. Although a very slow rate of loading would allow thixotropic strength to redevelop, their results are not conclusive as they performed very few tests over the range of loading rates and did not cover very fast rates of loading which would have been sufficient to measure the thixotropic strength (Section 6.2). Furthermore, Seed and Chan (1957) found that in compacted soils the influence of thixotropy appears to be significant at high degrees of saturation resulting in some cases in a strength increase of 50%. This increase in strength reduced as the degree of saturation decreased.

Mitchell (1960) proposed a hypothesis for thixotropic behaviour based on the initial non-equilibrium of interparticle forces after remoulding or compaction, and the effects of this non-equilibrium on subsequent structure changes within the soil. He also performed a series of triaxial compression tests on compacted silty soil from Vicksburg, Mississipi (LL = 39%, PL = 25%) to investigated this thixotropic behaviour. He subsequently concluded that the experimental results were in good agreement with the proposed hypothesis. Mitchell (1960) also observed that thixotropy is a process which is completely reversible, and for the soil tested chemical effects appear minor. He also commented on the effect of thixotropy on one dimensional compression compression is to raise the pressure void ratio curve (i.e. a higher voids ratio for the same consolidation stress).

Utomo and Dexter (1981) investigated the effect of ageing on the strength of three remoulded agricultural top clay soils at the water contents at which tillage operations are usually performed (close to their plastic limit). Three different types of soil strength were investigated: penetrometer resistance, tensile strength and compressive strength. They observed that for the Urrbrae soil, in which the dominant clay minerals are illite and kaolinite, the maximum thixotropic strength ratio was obtained at a water content just below the plastic limit. For the Strathalbyn soil, which contains illite and actinolite, and the Waco soil, which contains montromorillonite, the maximum thixotropic strength ratios occurred at water contents between the plastic and liquid limits. In the author's opinion this may have been an inaccurate observation as their method of sample preparation would have produced overconsolidated samples resulting in higher strength. They also observed that these characteristics and thixotropic strength measurements were still apparent after sterilization and removal of organic matter.

Osipov et al. (1984) studied the characteristics of microstructural changes associated with thixotropic phenomena using soil samples of variable dispersion and mineral composition. Their experiments were performed using an electron scanning microscope, for microstructural changes, and a rotary viscometer on reconstituted paste samples. The samples were prepared at moisture contents ranging from $0.8W_L$ to $2.2W_L$ where W_L is the moisture content at the liquid limit. In this state the samples had a typical coagulation structure characteristic of the majority of modern and poorly lithified clay soils. From the results obtained they concluded that the clay deformation occurs over a limited volume-in the shear zone. It is attended by a marked changes in the soil microstructure in this zone: i.e. a decrease in the size of the microaggregates and the diameter of the predominate pores, a decrease in the density of the system, and an increase in moisture content. These microstructural changes are directly related to the value of the maximum and minimum shear strengths of the soils. It is important to note that, under shear, for the system composed of small sized particles (generally less than a micron meter, montmorillonite) the deformation in the shear zone is volumetric in character, and does not lead to disruption of structural continuity (shear planes). It is obvious that the mechanism of deformation in such systems is relaxational or thixotropic, i.e. the microstructure forming cells are ruptured at shear, and at the same time are instantaneously restored, without forming a linear defect in the structure. In coarser dispersed clayey soils (kaolinite), in addition to the microstructural changes described above, the microaggregates in a shear zone become oriented along the direction of shear stress and local shear planes develop, along which the orientation of structural elements is particularly marked. For these soils during shear therefore both volumetric deformation and local disruption of the structural framework occur along developing shear planes.

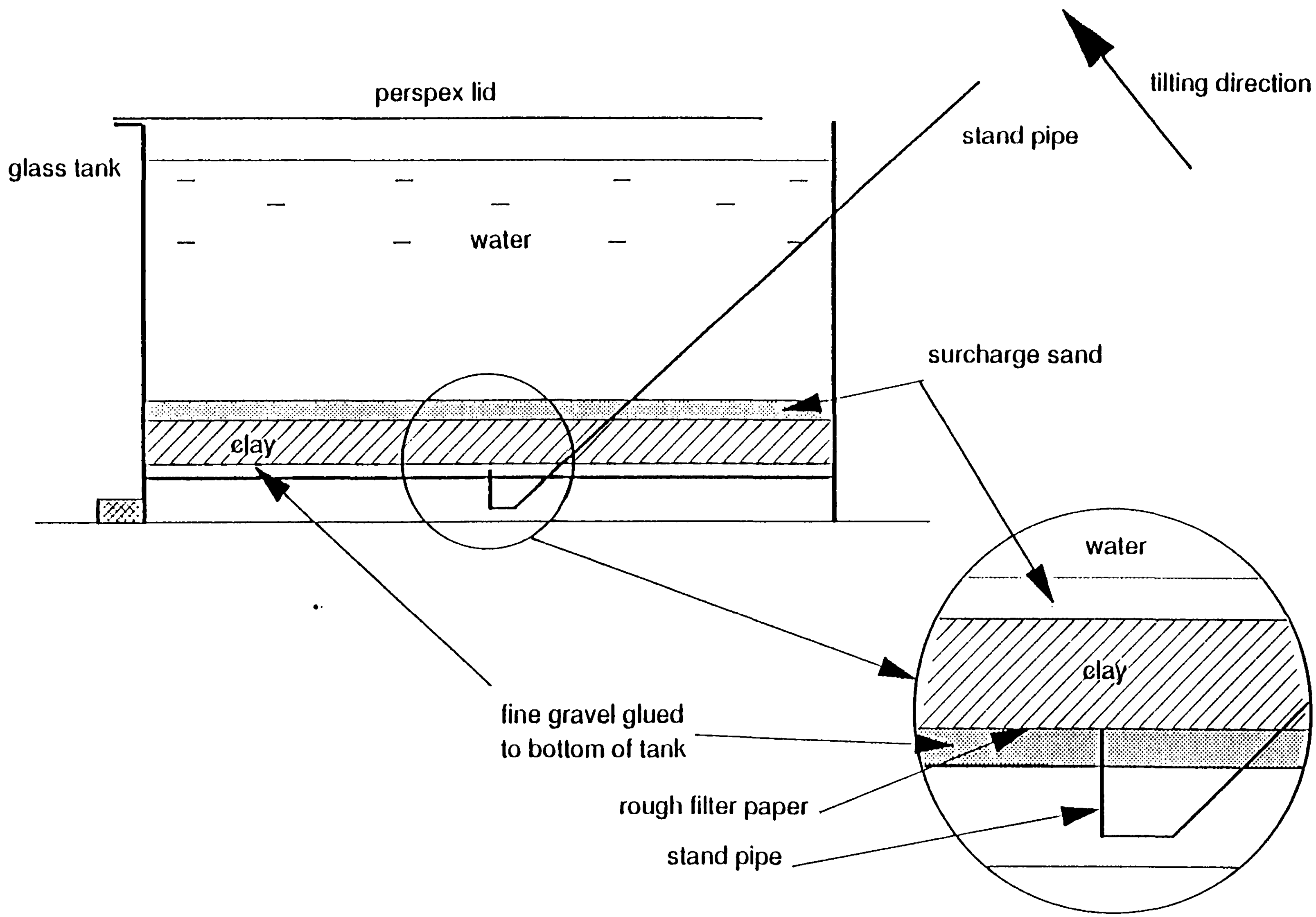


Figure 3.1 Arrangement of tilting tank used by Zadeh-Koceh (1989)

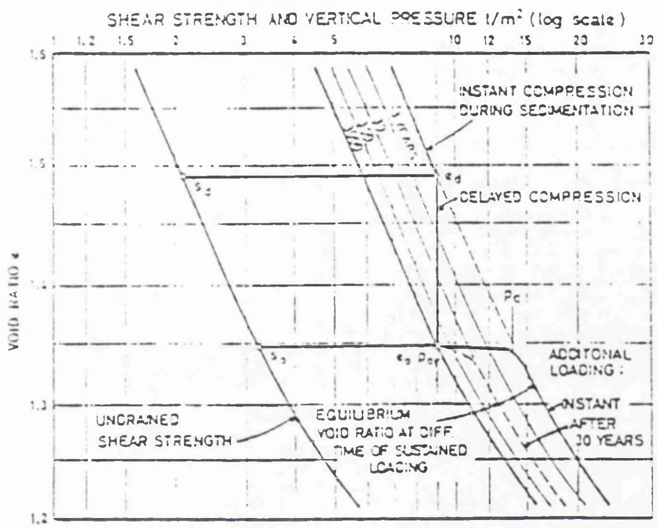


Figure 3.2 Compressibility and shear strength of a clay exhibiting delayed consolidation (Bjerrum (1967)

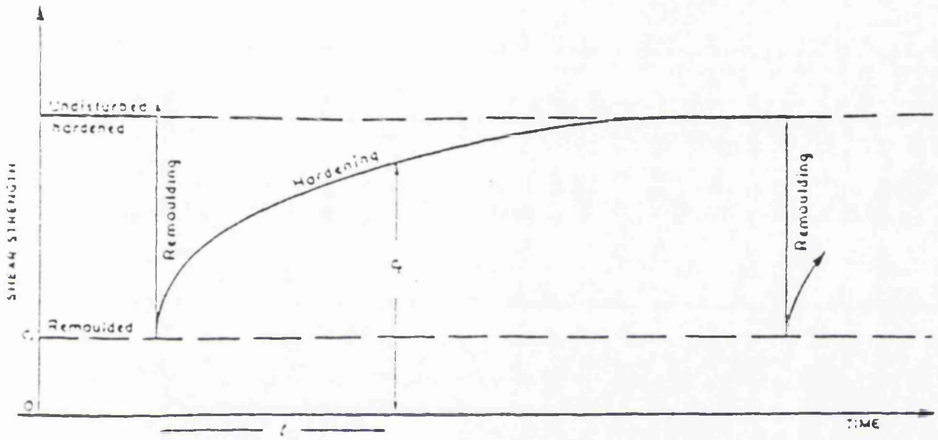


Figure 3.3 Properties of a purely thixotropic material

(Skempton and Northey (1952))

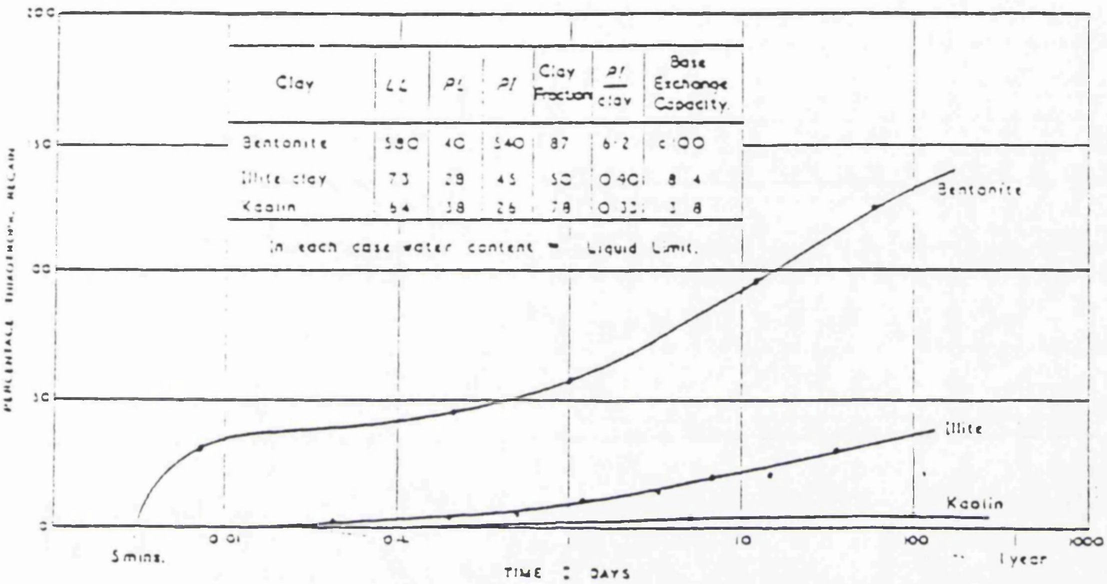


Figure 3.4 Thixotropic regain in three minerals (Skempton and Northey (1952))

CHAPTER 4

APPARATUS

4.1 INTRODUCTION

Three major pieces of apparatus were used in this research in order to assess the shear behaviour of clay at low and very low stress levels. Of these, two were conventional, namely the *Direct Shear Apparatus* (DSA) and the *Simple Shear Apparatus* (SSA). Both the DSA and SSA were modified to suit the low stresses (5 kPa) used in this project. The third type of apparatus was a set of tilting tanks. The purpose of these tilting tanks was to assess the shear behaviour of the clay at stress levels (less than 1kPa) that were not attainable in the DSA and the SSA. The tilting tanks were built for three types of tests; infinite slope failure, shear cylinder tests and tension tests.

A BBC Master micro-computer was used for data acquisition processing and control of the DSA and the SSA experiments. The role of the computer and data acquisition accuracy will be discussed in section 4.4.

This chapter describes each apparatus in detail and assesses their accuracy.

4.2 THE DIRECT SHEAR APPARATUS

The shear behaviour of clay was found to depend upon the manner of load application (Chapter 7). Accordingly, the DSA was modified to perform two types of tests; boundary displacement control and load control. The following sections describe the two versions of the DSA.

The effect of the shape of the sample on the test results was also of concern and subsequently both square and circular shear boxes were used. Figure 4.1 shows the details of the DSA. A 60mm by 60mm square shear box of 50mm height was used conforming with the conventional 60mm x 60mm square section shear box normally used for soil testing, (Jewell and Wroth, 1987). An 80mm diameter circular shear box of 50mm height was also used to give direct, parallel comparison with the *Norwegian Geotechnical Institut*e (NGI) conventional SSA. This also allowed NGI clay trimming apparatus to be used with a 32mm sample height. The NGI trimming apparatus was modified for the square sample. The short stable dimensions of the samples make them especially suitable for very soft clay. The extra depth of the shear boxes allowed the sample to be immersed in water to prevent drying.

4.2.1 Boundary Displacement Controlled DSA

The boundary displacement controlled DSA involved forcing the bottom part of the shear box to displace relative to the top part of the shear box in the horizontal plane at a chosen rate of displacement, the rate of displacement varying between 2mm/minute down to 0.01mm/minute. The consequential shear force, horizontal and vertical displacements and generated pore pressure were recorded.

A standard Wykeham Farrance DSA was modified to operate at low stress levels, see Figure 4.1. These modifications are described below.

Both the square and the circular shear boxes were lined with *polytetraflouroethylene* (PTFE) sheet on the inside walls to reduce the friction so that the applied vertical stress was uniformly distributed during the final increment of consolidation in the shear apparatus.

The problem of soft clay squeezing out through the horizontal split in the box sides ruled out the possibility of creating a gap between the top and bottom parts of the shear box, as performed in the conventional DSA apparatus on sands or on stiff clays. This was solved by lining the adjacent faces of the top and bottom parts of the boxes with thin PTFE sheet (0.1mm thickness), to minimise the friction between them during the shearing process. The friction between the two parts of the shear box was measured by applying vertical loads, equivalent to the vertical stresses used in the testing programme, directly onto the shear box and displaced at the same displacement rates used when testing the soil. Although a small fraction of the vertical load would be transferred onto the shear box when testing a soil, very low (1%-2% of the equivalent vertical stress) shear stresses were measured and no adjustment to measured shear resistance of the clay was necessary.

A series of tests were carried out on dense Leighton Buzzard sand, both with and without a gap, at the same vertical stress levels required for the clay

(5kPa). The results show (see Figure 4.2) that the PTFE lining did not have a significant effect on the test results as the results obtained in these tests (ϕ' = 45°) were in agreement with results obtained in the DSA on Leighton Buzzard sand by other researchers such as Arthur and Assadi (1977), ϕ' = 45.5°.

The PTFE lining also served another purpose, it prevented the Aluminium shear boxes from corroding in the areas of importance as the boxes had to be immersed in water 24 hours prior to the test (for last increment of consolidation) and also during the test.

Dalili (1990) had developed a pore pressure measuring device consisting of a hypodermic needle with a Druck transducer fitted on the end. This was incorporated and sealed into the shear box at the centre of the base. The de-airing was carried out by applying a vacuum to one end of the needle and allowing de-aired water to flow through the needle and then connecting the needle to the transducer. The de-airing process was carried out in de-aired water, ensuring the transducer diaphragm was free of air bubbles. The tip of the needle was adjusted so that the pore water pressure could be monitored any where within the sample (for instance at the centre, between the top and bottom parts of shear box or 2mm above or below that). Figure 4.3 and Plate 4.1 shows the arrangement of the needle with the Druck transducer, Figure 4.1 show its position in the DSA and Figure 4.4 shows the results of pore pressure for three fast displacement controlled tests (2mm/min.) at different positions in the sample. Table 4.1 provides the specifications of the Druck transducer and Figure 4.5 shows the calibration line for the transducer together with the needle connected to it.

A horse-shoe type load cell was designed and built at UCL in order to measure loads as low as 0.1N, so that the shear resistance of the clay could be measured with reasonable accuracy ($\pm$ 2% of 1kPa) at the required low stress level (5 kPa). Figure 4.1 shows the details of the load cell position in the DSA apparatus in place of the conventional proving ring. Table 4.1 provides the specifications of the load cell and Figure 4.6 shows the calibration line for the load cell.

Three *Linear Variable Differential Transducers* (LVDT) were used to measure the horizontal and vertical displacements. Figure 4.1 shows the positioning of each LVDT. One was used to measure the horizontal displacement and the other two were used to measure the overall deflection of the top platten and the tilting of the top platten during the shearing process. Table 4.1 provides the specifications of the LVDTs transducer and Figure 4.7 shows the calibration lines for the LVDTs.

The displacements and rates of displacement were controlled by an electronic stepper motor fitted to the driving gear box on the conventional DSA. This provided very accurate incremental displacements. One step produced 0.001mm displacement and test rates would be varied from 10 steps/min to 2000 steps/min.

4.2.2 Load Controlled DSA

The load controlled DSA involved the application of a shear load in discrete increments (10% of expected failure load per day) across the horizontal plane between the top and bottom parts of the box. The consequential horizontal and vertical displacements, and in some cases pore pressure, were all recorded.

For the load control DSA, the same shear apparatus was used as for the displacement controlled DSA. However, the modifications described in the following paragraphs were necessary in order to change from displacement control to load control (see Figure 4.8).

Instead of applying a displacement, a weight was added in increments to a hanger which was connected with a string to the arm of the top part of the shear box over a pulley. The friction in the pulley was negligible as the measured load by the load cell was equal to the applied load. A counter balance weight on an adjacent hanger was used to provide equilibrium and accuracy of load application.

Measurements of load, pore pressures, displacements and data acquisition were performed using identical equipment used to that in the displacement controlled shear apparatus, as described in section 4.2.1.

4.3 THE SIMPLE SHEAR APPARATUS

A standard Wykeham Farrance SSA was purchased for the purpose of part of this research and major modifications were carried out on the basic model. Figure 4.9(a) shows the new arrangements of the SSA, which will be referred to as the Modified SSA from here on. The Modified SSA was used only for boundary displacement control tests. The following describes the modifications.

In order to use the SSA the first modification necessary was the size of sample to be tested. A new size of the sample was chosen to be 80mm in diameter by 32mm in height (original size was 50mm in diameter by 20mm height). These sizes were chosen to comply with the NGI standard SSA sample size so that the standard NGI reinforced rubber membranes could be used, as well as the NGI trimming apparatus.

An NGI wire-reinforced rubber membrane was used to maintain a constant diameter and also to seal the sample. The stiffness of the wire reinforcement was tested by filling the membrane with water, since water has no shear resistance, and applying the appropriate vertical stress used when testing the clay. Very low (1%-2% of the vertical stress) shear stresses were measured and no adjustment to measured shear resistance of the clay was necessary. However, the NGI specifications suggest a higher correction to the one measured and the reason for this is not fully understood.

The Wykeham Farrance shear load cell used in the DSA was replaced with a horse shoe load cell, for increased accuracy of load measurements.

The position for the vertical load cell was changed from the bottom, next to the hydraulic piston, to a point above the top platten of the sample to avoid the friction in the piston bearing, Figure 4.9(b). The original Wykeham Farrance load cell had to be used as it was less susceptible to bending during the shearing process than other load cells with a better resolution. However it was not possible to avoid the effect of bending on the load cell which meant that the measurement of the total vertical stresses were unreliable. Table 4.1 provides the specifications of the load cell and Figure 4.10 shows the calibration line for the load cell. The effect of this will be discussed in Section 6.3. An attempt was made to prevent the bending of the top platten, this was achieved by introducing a reacting force opposite to the driving force by attaching the top platten to a vertical bearing, see Figure 4.9(a). This modification was not successful as far as the load cell performance is concerned, however it is assumed that this would have allowed for the vertical stress to be more uniform across the sample.

The displacement measurements were taken as for the DSA, see section 4.2.

An EMS servo hydraulic pressure controller, type 440, was adopted to apply the vertical load via the load piston. Once the sample reached full consolidation and was ready for testing, the controller was switched to "position control", i.e. keeping a constant average height of the sample instead of a constant total vertical load. The latter was achieved by reducing or increasing the vertical load in order to maintain a constant average output of the two vertical LVDTs monitoring the height of the sample. The vertical LVDTs readings indicated a fluctuation of 0.02% of the height which is well within requirements for a constant height test as indicated by Airey (1983).

As already indicated the measurement of the vertical load was a problem with the modified apparatus. As the sample was sheared, the vertical load was either reduced or increased in order to keep a constant height. Although the constant height was achieved, the recorded load was not accurate, due to bending and therefore the effective stress path produced was not correctly measured. However, considering an undrained situation, the shear strengths measured were of the correct value for the samples tested as obtained in the DSA and by Airey (1983).

4.4 COMPUTER

4.4.1 Computer Control

The computer used for this work was the BBC master micro-computer, somewhat behind in today's technology, as it has a small *Random Access Memory*, 132K and is relatively slow. However, since the rates of testing were slow (fastest being 2mm/minute due to the limitations of the stepper motor, the speed of the computer was sufficient for this purpose.

The computer had three main functions: controlling, monitoring and reducing the data. Controlling was done by sending pulses to the stepper motor at the required rate in order to achieve a specific displacement rate. Pulses controlled the rotation of the stepper motor, which in turn moved the gears of the DSA or SSA resulting in a forward movement of the shear box. This replaced the original electrical driving motor which was limited in its range and accuracy. The data was monitored through a 12-bit *Analogue to Digital Convertor* (ADC) which had eight channels of variable amplification, to which the LVDTs, load cells and Druck transducers were connected. Processing the data was organised with the software written specifically for this purpose. The data was then saved on a floppy discs.

4.4.1.1 Control of Displacement Testing Rate

Although the computer was sufficiently fast to send pulses at very fast rates (1000 pulses equivalent to 1mm travel), the testing rate was limited by the stepper motor used. The stepper motor has a maximum of 15 revolutions per minute, which could produce a horizontal displacement through the gear box of 3mm/minute. However, as a significant torque was required to produce this displacement, a resulting maximum rate of 2.3mm/minute was achieved. The minimum displacement that could be applied was 0.001mm.

In order to achieve a rate which was fast enough to produce a "quasi undrained situation" in the DSA (the SSA was run at slow rates, 0.1mm/minute to 0.001mm/minute), as it was not possible to mimic an undrained situation as for the SSA by keeping a constant height, a number of rates were tried out. It was found that a rate of 2mm/minute indicated a very small vertical displacement of the top platten of less than 0.05mm, prior to developing the full shear strength of the clay. Although this vertical displacement indicates volume change (Figure 6.3), the pore pressure measurement allowed a complete picture of the effective stress path to be achieved.

4.4.2 User-Written Software

Throughout the work and up to the final test, the software was developed and modified continuously to suit the requirements of individual tests. Appendix A provides a listing of programs used for this research.

The language used for the programming was Basic. The program suite consisted of five main parts:-

4.4.2.1. Inputting Required Data

The information required by the computer to run the tests are as follows; the rate of testing: maximum displacement (up to 10mm, the intervals at which readings are to be taken is worked out for a maximum of 150 sets of readings); cross sectional area of sample; name of file in which readings are to be saved; and finally the maximum shear stress allowed (this is required for safety precautions to prevent any damage to the equipment - if this limit was reached the computer would terminate the test).

The computer was programed to register the output of all measuring instruments before the test was started. Readings were taken before and after the screws were removed in the DSA, and the change from load control to position control in the SSA. The readings were then compared so that the disturbances due to the removal of screws or change from load control to position control could be reported. All readings taken were compared with the datum readings taken right at the start of the test.

4.4.2.2. Control of Equipment

The program to control the equipment works in such away that it loops in a control subroutine until it is instructed to leave the subroutine and continues through the other operations. Normally, contact is made with the stepper motor once sufficient time has passed to allow for one pulse to be sent. Only the operation of data acquisition requires the exit from the control subroutine. During the data acquisition the control subroutine is called upon twice to ensure continuous control. Although the control is done in very small steps (0.001 mm) it is continuous through-out the test.

Figure 4.11 shows the linearity of the displacement achieved for both the fastest and slowest tests.

4.4.2.3. Monitoring Readings

A 12-bit, eight channel ADC was linked to the computer to allow the outputs of the LVDTs, load cells and Druck transducers to be monitored. The table below shows the accuracy at which the readings could be monitored. However, when transducers were operating at the lower end of their functioning range (less than 10% of full scale output) the errors encountered were independent of the computer.

TRANSDUCER	ADC ACCURACY
LVDTs	$\pm$ 2.4X10^{-3} mm
Shear load cell	$\pm$ 6.32X10^{-3} kg
Vertical load cell	$\pm$ 3.93X10^{-3} kg
Pore Pressure Transducer	$\pm$ 4.24X10^{-3} kPa

A subroutine was responsible for scanning through and sampling the ADC channels. The accuracy of the data acquisition through the ADC was $\pm$ 0.025%, which meant that not only human error was eliminated but also accurate and precise readings were obtained almost simultaneously (5 readings in less than 0.1 seconds).

4.4.2.4. Data Reduction and Transfer to Disk

The program suite incorporated the appropriate equations in order to calculate the required parameters. All transducers were regularly calibrated (every 5 tests) and the calibration factors would have been regularly modified had there been any changes. However no significant changes were recorded.

Once testing is finished, the processed data were transferred to the floppy disk, which could be accessed later for graphical presentations.

4.5 TILTING TANK TESTS

Three types of tilting tank experiments were used: standard tilting tanks, the shear cylinder within a tilting tank and the tension test within a tilting tank.

4.5.1 The Standard Tilting Test

Perspex tanks of 430mm length, 230mm width and 260 mm height (see Section 3.2 for choice of dimensions) were used for the full range of tests on normally consolidated and over-consolidated clay at extremely low stress levels (self weight vertical stress of around 0.05 kPa). Figure 4.12 shows the details of the tanks, and the following paragraphs explain these features.

In order to create a base of high permeability below the clay bed, gravel (2.36 mm to 4.75mm in diameter) was mixed thoroughly with 100g of Araldite 2002 and compacted flat in the bottom of the tank, to the same height as the perspex tube used for pore pressure measurement. The gravel was left to set for 24 hours. A v-shaped notch of 10mm wide was made between the gravel and the internal perimeter of the tank in order to fix the fine polypropylene cloth and the coarse polypropylene cloth (See details of edge joint in figure 4.12).

The fine polypropylene cloth with holes of 90 um wide was used to prevent the penetration of clay particles, and the coarse polypropylene cloth was used to prevent the clay bed sliding on the fine polypropylene cloth during the tilting (the fine cloth has a very smooth surface). A water-proof silicon sealant was used to seal around the edges as well as holding the fine and coarse polypropylene cloths as tight as possible, since any looseness or crease allowed non uniform consolidation and swelling. The purpose of the thin layer of gravel embedded in the sealant was to provide a rough interface with the clay, and drainage back through the cloth further in from tank wall.

The perspex tube attached to the stand pipe was used to monitor the pore pressures at the bottom of the clay bed. The other perspex tube made a controlled height outflow to create an accurate suction below the clay bed.

The two holes at the top of the back side of the tank were used for a water supply and an overflow in order to maintain a constant head of water during further consolidation. The perspex lid with waterproof silicon sealant was used to seal the tank, 12 hours prior to tilting to allow the sealant to dry.

The PTFE axle bearings were necessary to facilitate the tilting procedure, and prevent any vibrations during tilting.

Lifting tackle with an 8:1 mechanical advantage was attached to each tank to allow smooth accurate tilting of the tank.

4.5.2 The Shear Cylinder

A perspex cylinder was made to test clay in shear at the same stress levels used in the tilting tanks (0.05 kPa) and avoid the restraint imposed by the tank end against simple shear of the soil during tilting. Figure 4.13 shows the dimensions of the cylinder and the arrangement of the cylinder inside a glass tank. The following is a description of the functions of the different parts of the apparatus.

The function of the knife edge on the bottom of the cylinder is for the precise location of the cylinder in the step of the base plate. The vertical flange provided stability and prevented excess tilting of the cylinder.

The circumferential flange around the cylinder allows a long cylinder (700mm long) of the same internal diameter as that of the shearing cylinder to stand on the cylinder for the purpose of sample preparation, see section 5.3.2. The flange also allowed for the threads to be attached to the cylinder. The threads were used to lift up the cylinder and keep it in the required position (normally 2mm above the base plate) once the sample was ready for testing. The coarse sand paper prevented the clay from slipping off the base, and the perspex base plate ensured the failure occurred in the clay.

Other sizes of cylinders were used to investigate the size effect (120mm and 160mm in diameter with the same height and wall thickness). However the 80mm cylinder was mostly used as it matched the dimensions of the circular DSA and SSA, and no size effect was observed within this range (see Section 6.4.2).

4.5.3 The Tension Test

This test involved failing a dumbbell shaped sample in tension. It was also performed in a perspex tank with the arrangements shown in Figure 4.14. The following describes the purpose of the features seen in the diagram: The NGI porous stones (with pins as used in the NGI SSA) held the sample ensuring the failure occurred in the thinner section. The PTFE rollers provided a low friction rolling mechanism. The location pin prevented any movement of the down stream plate when removing the brackets holding the top plates together. Six tests were performed to obtain the angle required to initiate the movement of the bottom plate; and this was found to be 6°.

4.6 Control of shear and Interpreting failure in the testing apparatuses

As already implied direct shear tests were made with either boundary displacement rate control or shear load increment control, increments being separated by a time interval (usually 24hrs). The former will be called continuous tests and the latter incremental tests. In the tilting tests there is no imposed boundary displacement; tilting causes gravity to increase the shear stress on planes of potential rupture; stress uniformity is likely to be unusually good. Again tilting can be continuous in time or incremental with time intervals (usually 24hrs). Both forms were used. Continuous tilts were carried out at rates of 1°/min. or more. Tilts described as 5°/hr. were in fact incremental tilts of 1.25° every quarter hour with a rate of 4°/min.; similarly tilts of 1°/hr. were made up of 0.5° every half hour. Tilts described as 3°/day were carried out at 24hr. intervals in one continuous tilt of 3° at 4°/min., but the rate was slowed on some occasions when failure was anticipated in the increment to 9°/hr.. All tilting tests failed suddenly, the angle of tilt at failure was determined accurately with an inclinometer (generally to ±0.5° or better except for the very fast tilt (15°/sec.) ±1°).

Failure conditions in these apparatuses are relatively complex. There are two widely accepted mechanisms of failure and a choice must be made between these to determine the shear strength. In addition it is necessary to propose that shear strength is represented by a single parameter ϕ' thus implying that $c'=0$ or is proportional to σ' and subsumed into ϕ'.

Failure will occur when the stress ratio (τ/σ_v') has reached a maximum. However the failure mechanism will usually be one of the following conditions:

(a) Failure occurring over a surface on which there is maximum stress obliquity (i.e. τ/σ' is maximum). This is at $45° + \phi'/2$ to the major principle stress plane (Figure 4.15), and is referred to as Coulomb slip on a plane of Maximum Stress Obliquity (MSO). This failure form is entirely determined by the stresses; various interpretations of associated strain can be made.

(b) Pre-failure strain occurs in the form of simple shear in a layer aligned with a no-extension direction with coincident axes of stress and strain increment, which is at $45 + \psi/2$ to the major stress and increments of strain planes, (ψ = angle of dilation), Figure 4.16, (Roscoe (1970), James and Bransby (1970) and Arthur and Dunstan (1982)). This is referred to as the No Extension Direction with coincident axes of stress and strain increment (NED). Moreover this simple shear layer matches the imposed boundary discontinuities in the cases of the DSA and tilting cylinder. In many of the experiments it will be found that failure occurs fast under undrained conditions so that $\psi = 0$ which is the case illustrated in Fig. 4.16.

The choice between these two mechanisms depends on indirect evidence. Large displacements before failure or apparent concentrated sliding on a thin layer within a tilting layer at failure are taken as NED mechanism failures. Whilst small displacements before failure or disintegration with minimal prior strain of a tilting layer at failure are taken as MSO mechanism failures.

There is a clear possibility of intermediate mechanisms but in practice in the experimental situations described these are rare. The variability of the criteria proposed can be judged in due course by the correlations achieved between different boundary conditions and stress levels.

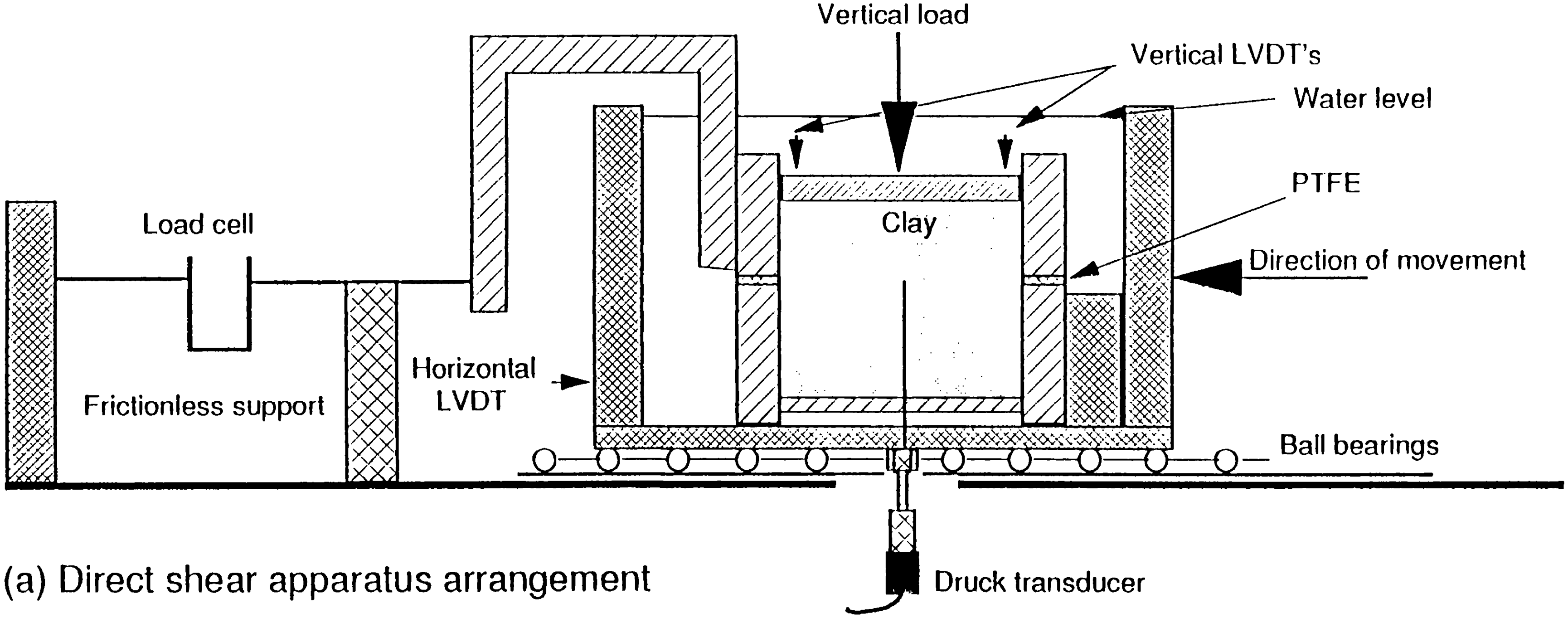
In the standard tilting test failures occurs when the mass of clay slides to the bottom of tank. The angle of tilt at failure according to infinite slope stability analysis under drained conditions (Figure 4.17) is equivalent to ϕ' as the sliding occurs on maximum stress obliquity as explained above, however it will be shown in Section 7.4.9 that either of the above two mechanisms (MSO or NED) can occur in the standard tilting tests according to the test conditions. Failure in the shear cylinder occurs when the cylinder together with the soil contained inside shears on the thin layer of soil at the bottom of the tank in simple shear (see Figure 4.13). By comparison with data obtained by Arthur et al (1987) on Leighton Buzzard sand in a tilting tank (they measured a ϕ' of 48°), it will be found that failure occurs on an NED which is represented by the angle of tilt at failure. This results in the failure shear load being introduced by the component of self weight (buoyant self weight when submerged in water) of the soil and the cylinder in the direction of tilt. The tension test failure occurs when the bottom part of the clay breaks from the top part, due to both the component of the buoyant self weight of the bottom half of the clay and the mobile plate, at the angle of tilt at failure. This is easily related to the tensile strength of the clay.

4.7 Summary of testing apparatus

The three major pieces of apparatus used in this research programme (DSA, SSA and the tilting tanks) allowed tests to be performed on soil at low and very low stress levels. Modifications carried out on the standard DSA and SSA have been relatively easy and inexpensive considering today's technology. The DSA has allowed a large versatility in the testing conditions. Both displacement and load control tests can be carried out with great accuracy by incorporating measurements using simple electronics and computer methods, these are easy to operate, as well as allowing accurate variations of displacement rates. The most important improvement is provisions for internal pore pressure measurements. When the vertical stress measurement is improved the SSA has a great potential of use at the same stress levels used in the DSA. It may also be modified to allow for pore pressure measurements as used by Dyvik et al. (1987). The tilting tanks were relatively cheap and very easy to modify. As many as twenty were used simultaneously in this research programme. Normally consolidated and overconsolidated samples can be prepared in the standard tilting tanks at very low stress levels (0.05 kPa). Whilst only normally consolidated samples were prepared for the shear cylinder test, it would be feasible to use it for overconsolidated samples. In all tilting tests the rate of testing can be varied easily and even very fast smooth tilting can be performed ($\approx 15°$/second).

TRANSDUCER	MAKE & MODEL	SPECIFICATIONS	CALIBRATION & COMMENTS
HORIZONTAL LVDT	RDP D5 200H	0-10mm, NON LINEARITY ±0.5% FRO	5 mm/VOLT
VERTICAL LVDTs	RDP D5 200H	0-10mm, NON LINEARITY ±0.5% FRO	1 mm/VOLT
SHEAR LOAD CELL	WISHBONE MADE AT UCL	-0.3 - 0.3 KN, NON LINEARITY ±0.5% FRO	AMPLIFICATION=200 200N/VOLTS
VERTICAL LOAD CELL	WYKEHAM FARRANCE 4958	5KN RANGE, NON LINEARITY ±0.001% FRO	AMPLIFICATION=200 0.8 VOLTS/KN
PORE PRESSURE TRANSDUCER	DRUCK PDCR 200	0-100 kPa, NON LINEARITY ±0.2% FRO	AMPLIFICATION=200 8.7 VOLT/kPa

Table 4.1 Specifications of transducers



(a) Direct shear apparatus arrangement

(b) Horizontal cross sections

Figure 4.1. Displacement control direct shear apparatus

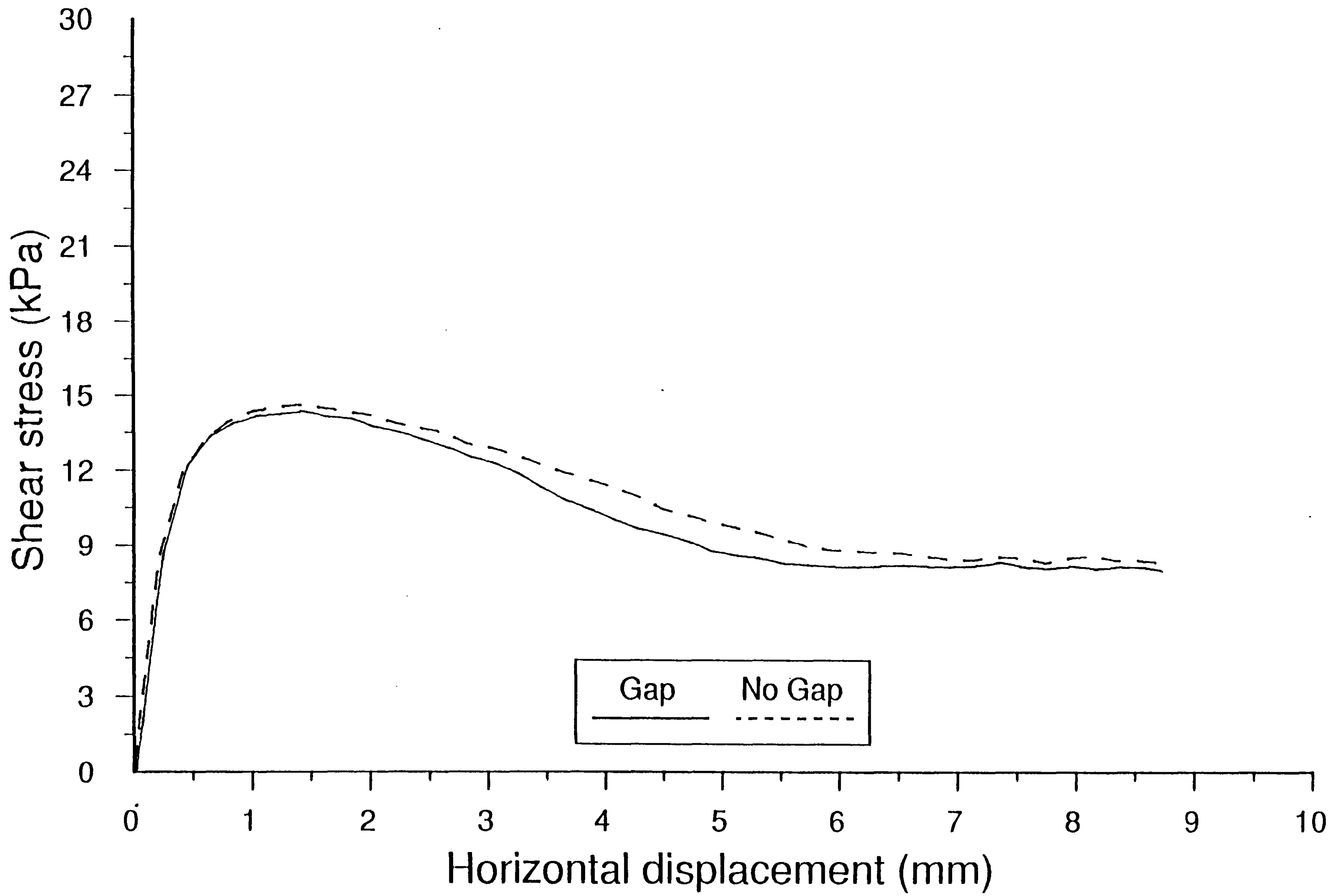


Figure 4.2 Comparison of DSA result with and without a gap on sand

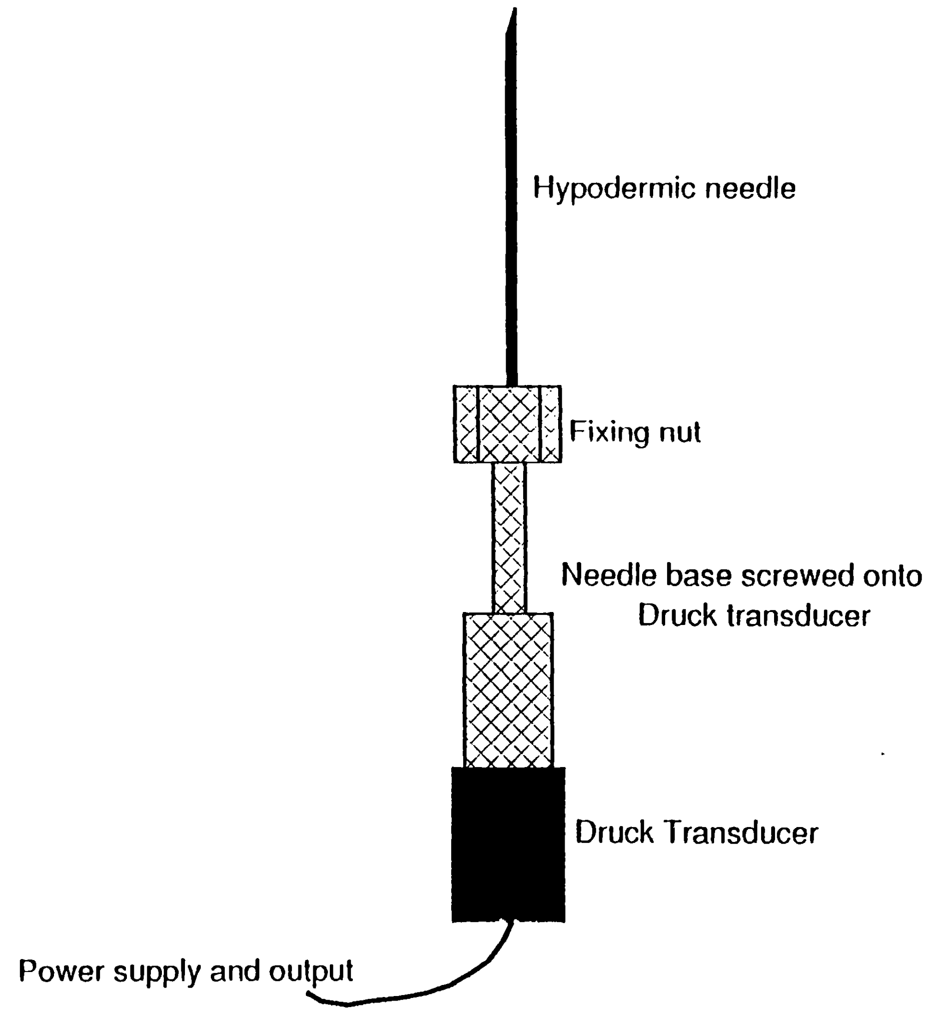


Figure 4.3 Details of Druck transducer and needle

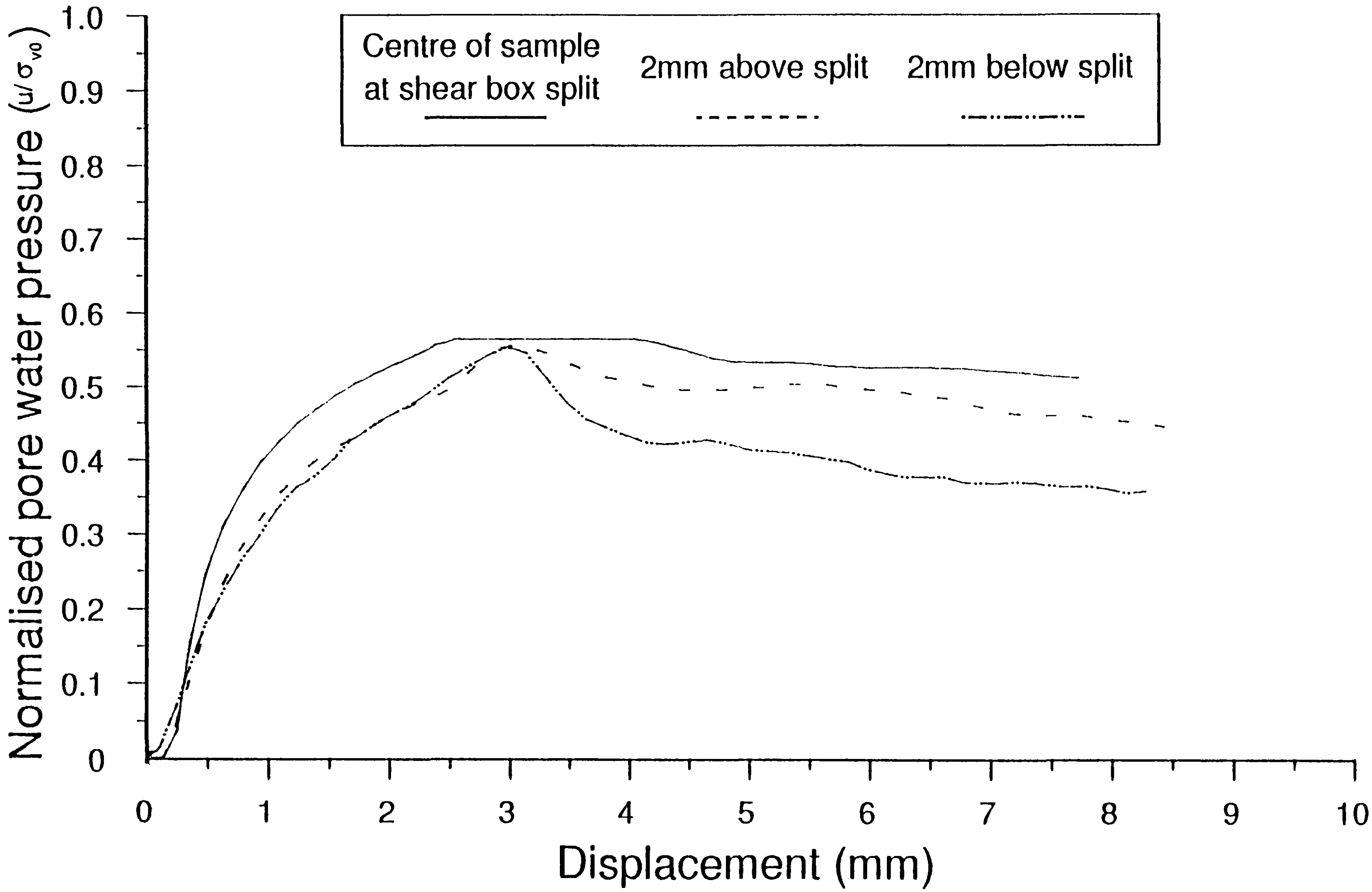


Figure 4.4 Measurement of pore water pressure at different positions in the direct shear apparatus

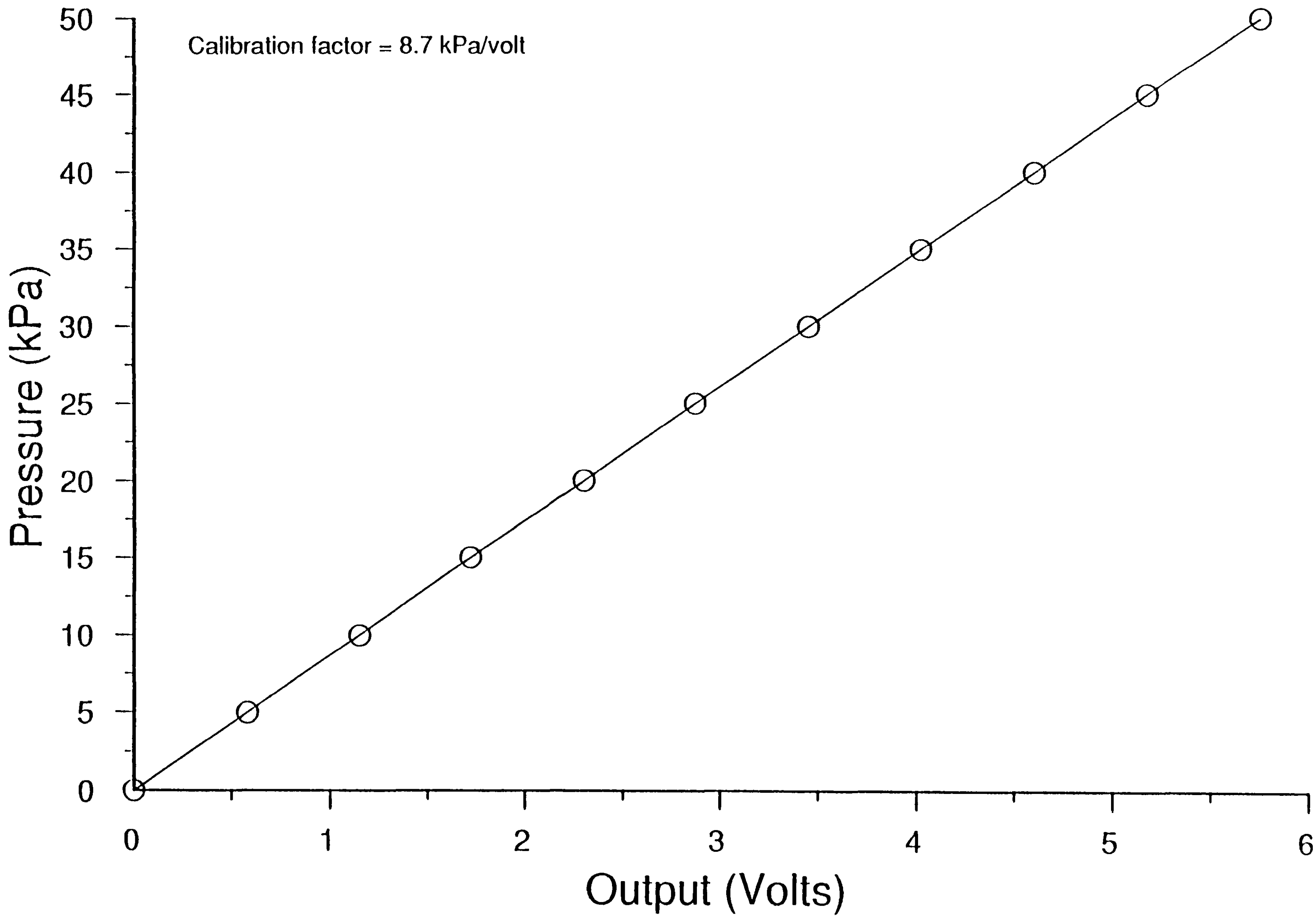


Figure 4.5 Calibration of Druck transducer

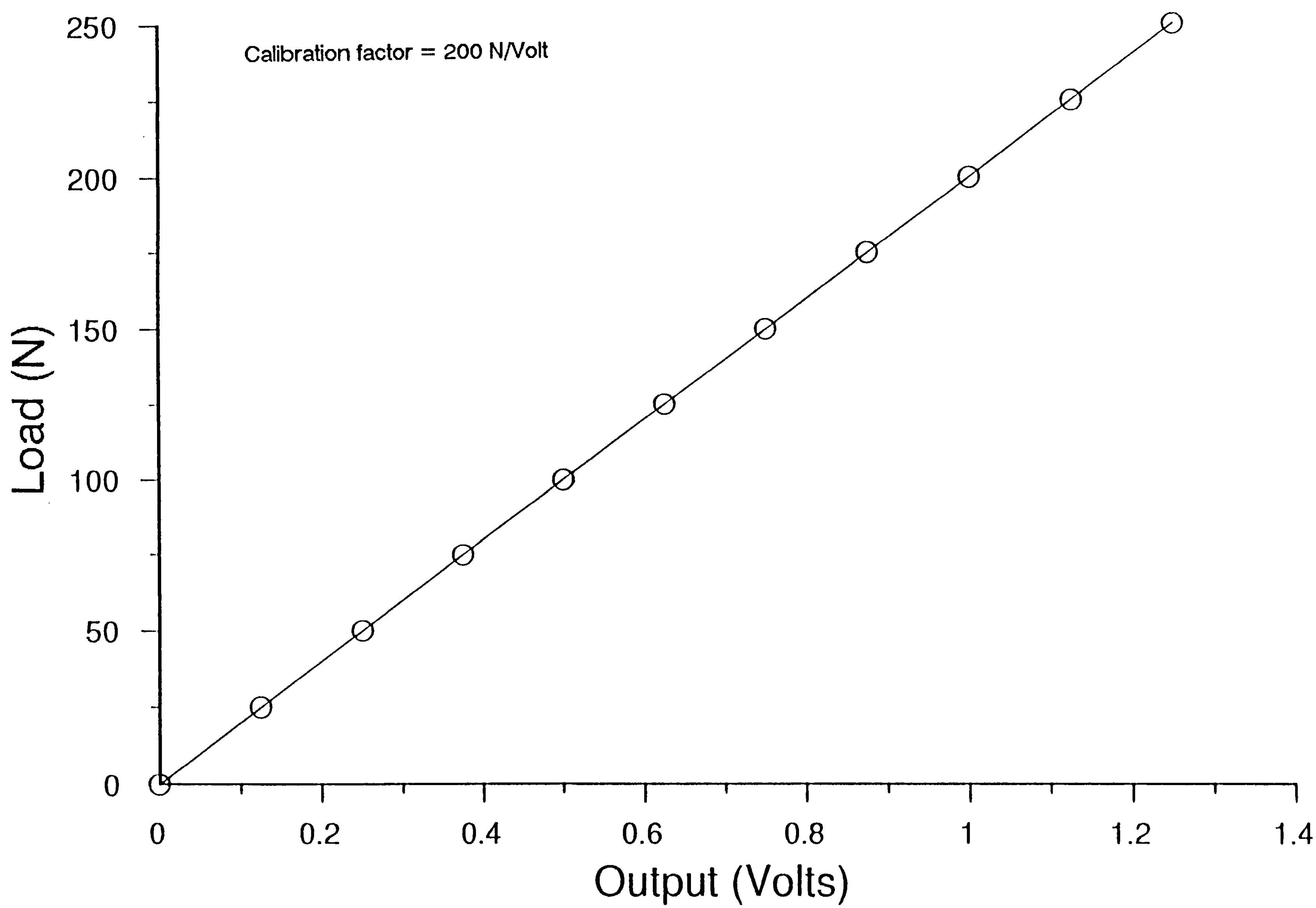


Figure 4.6 Calibration of Shear load transducer

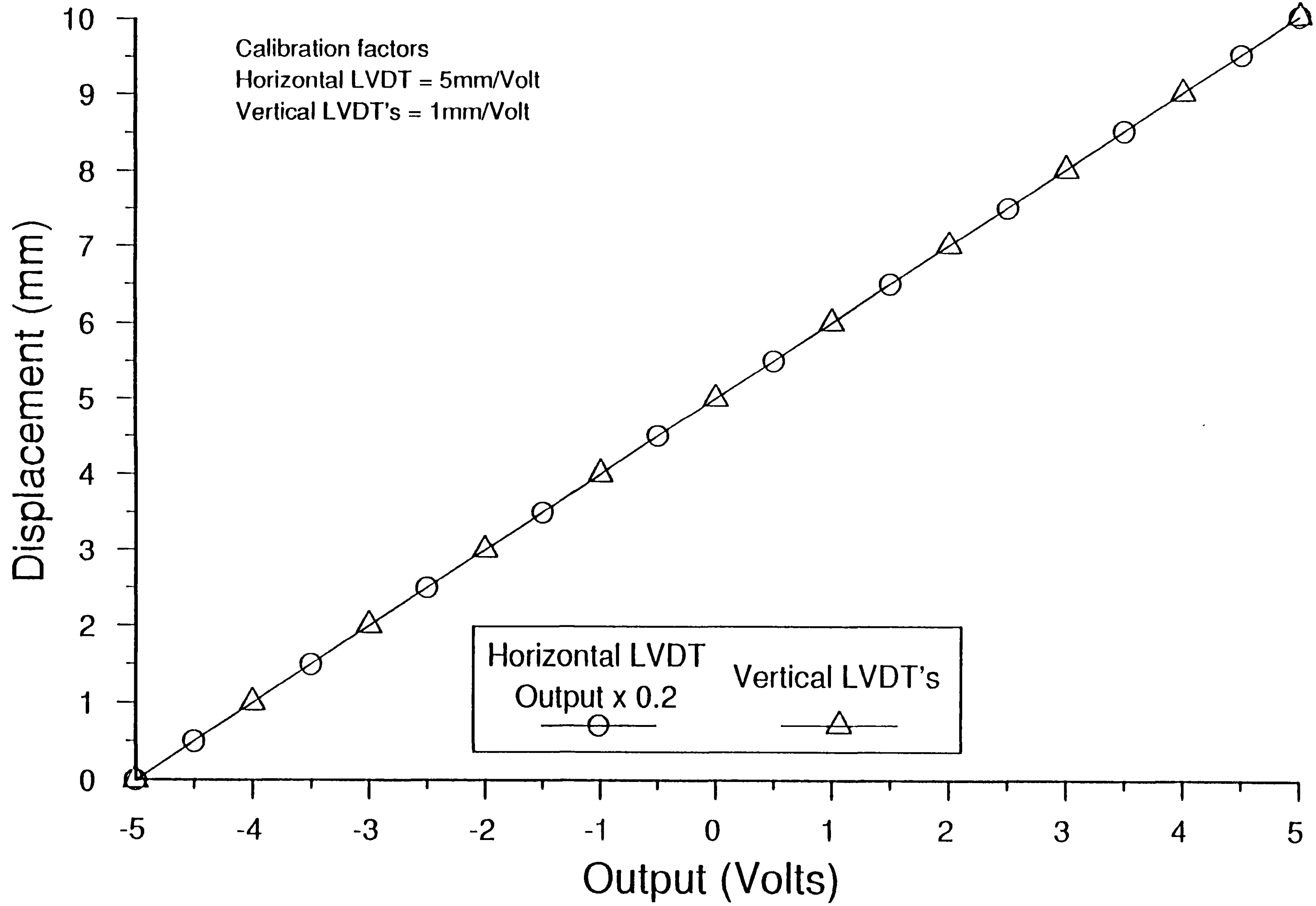


Figure 4.7 Calibration of L.V.D.T's

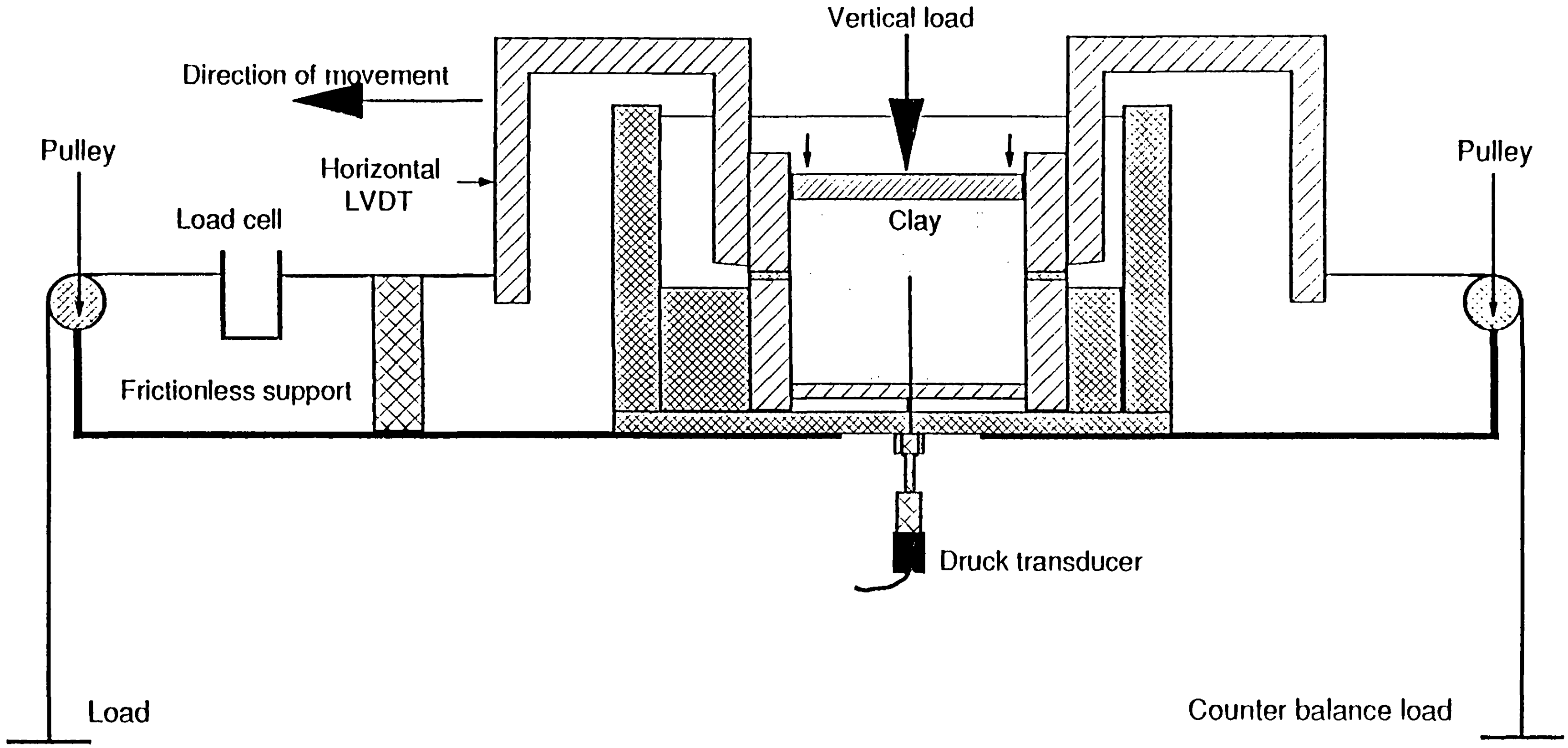


Figure 4.8 Load control direct shear apparatus

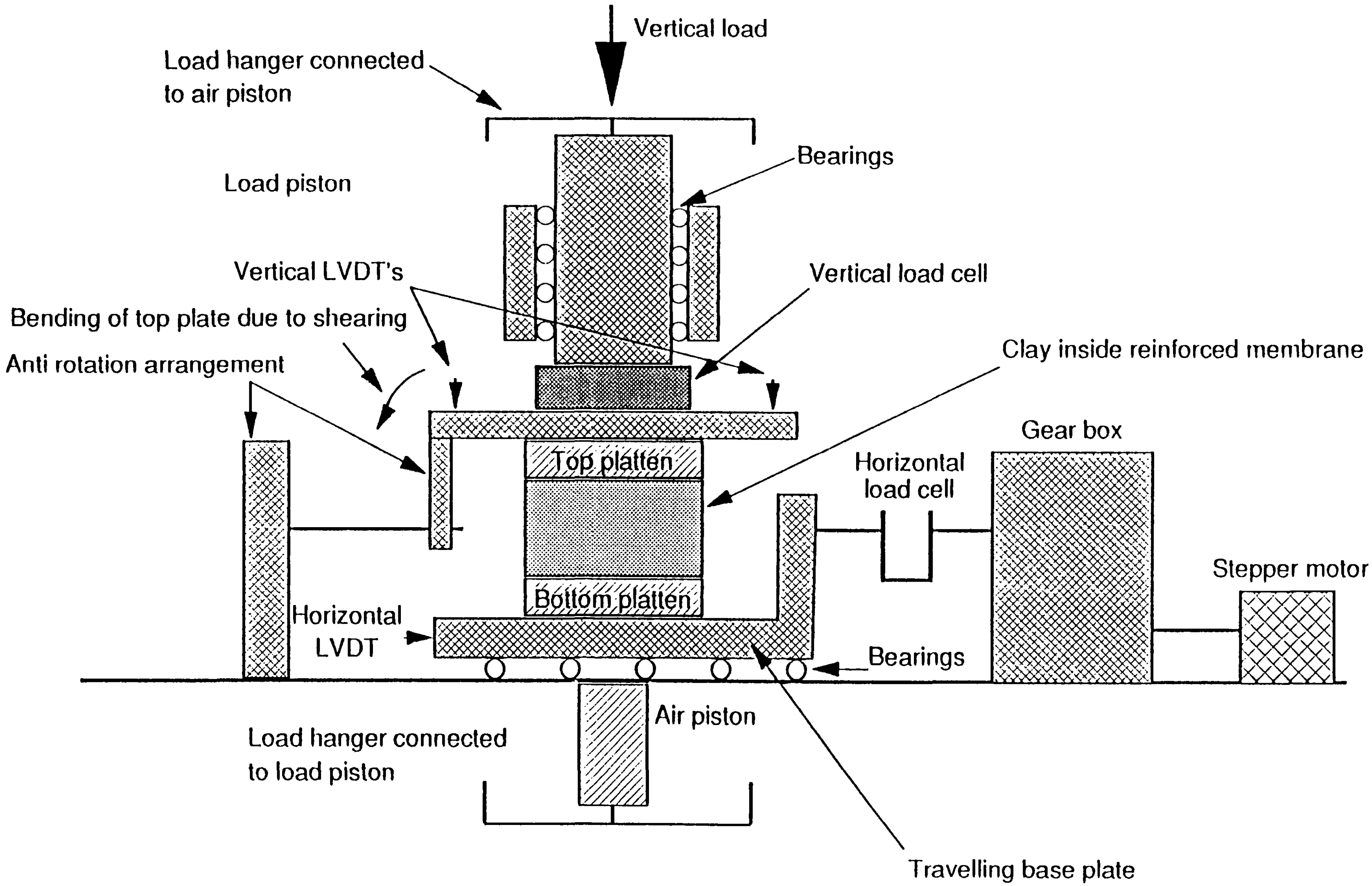


Figure 4.9(a) Arrangement of modified simple shear apparatus

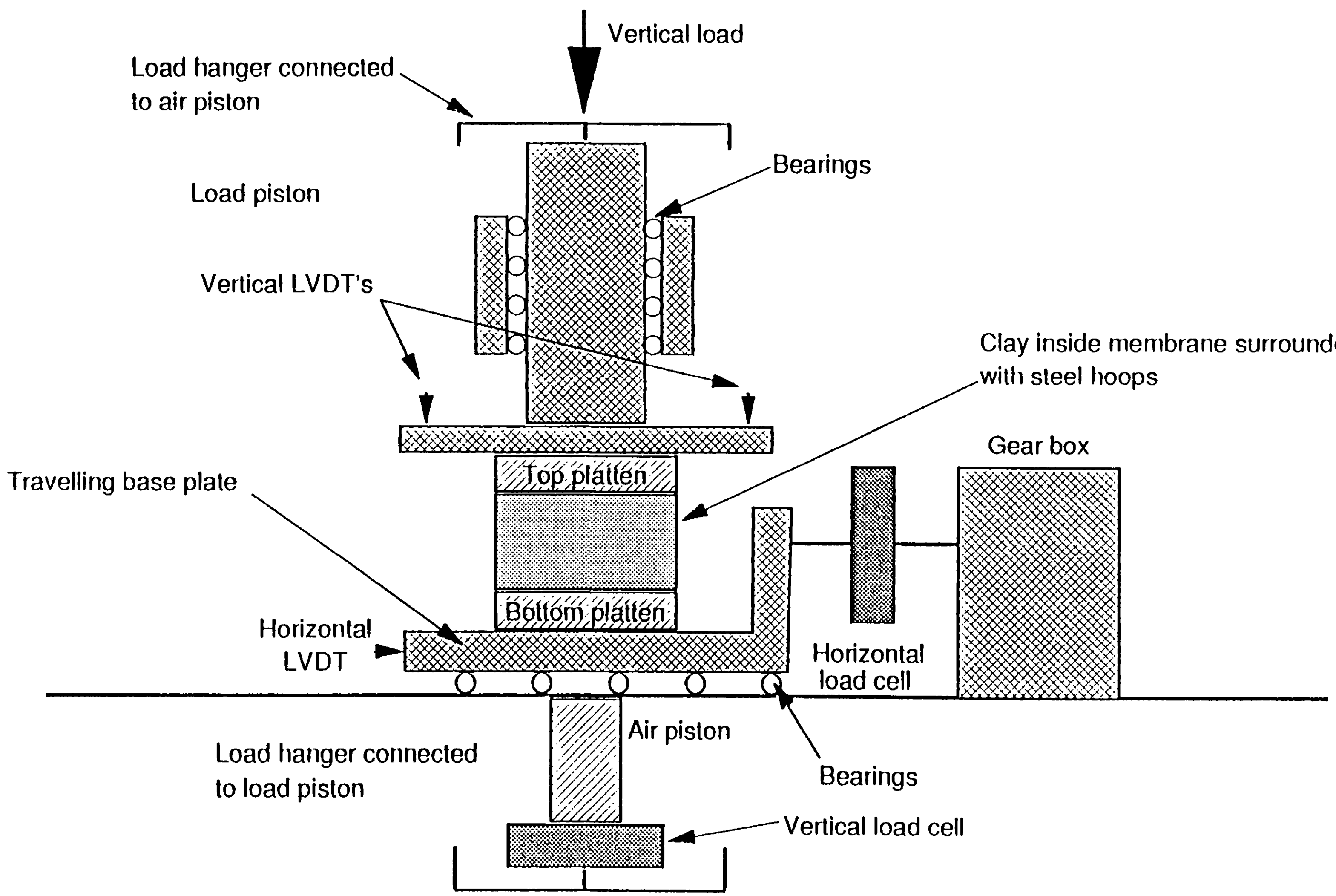


Figure 4.9(b) Arrangement of original simple shear apparatus

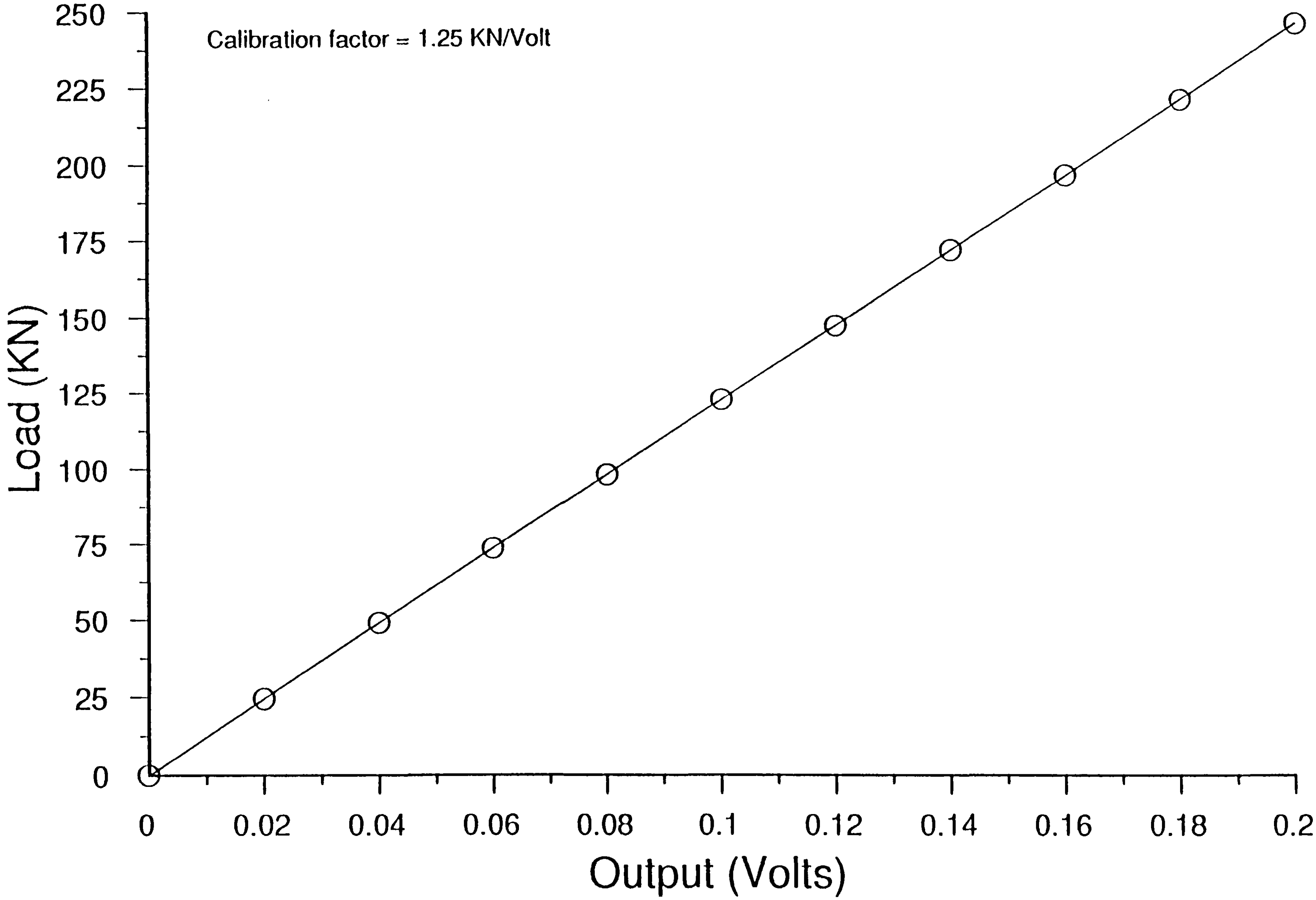


Figure 4.10 Calibration of Wykeham Farrance Load Cell

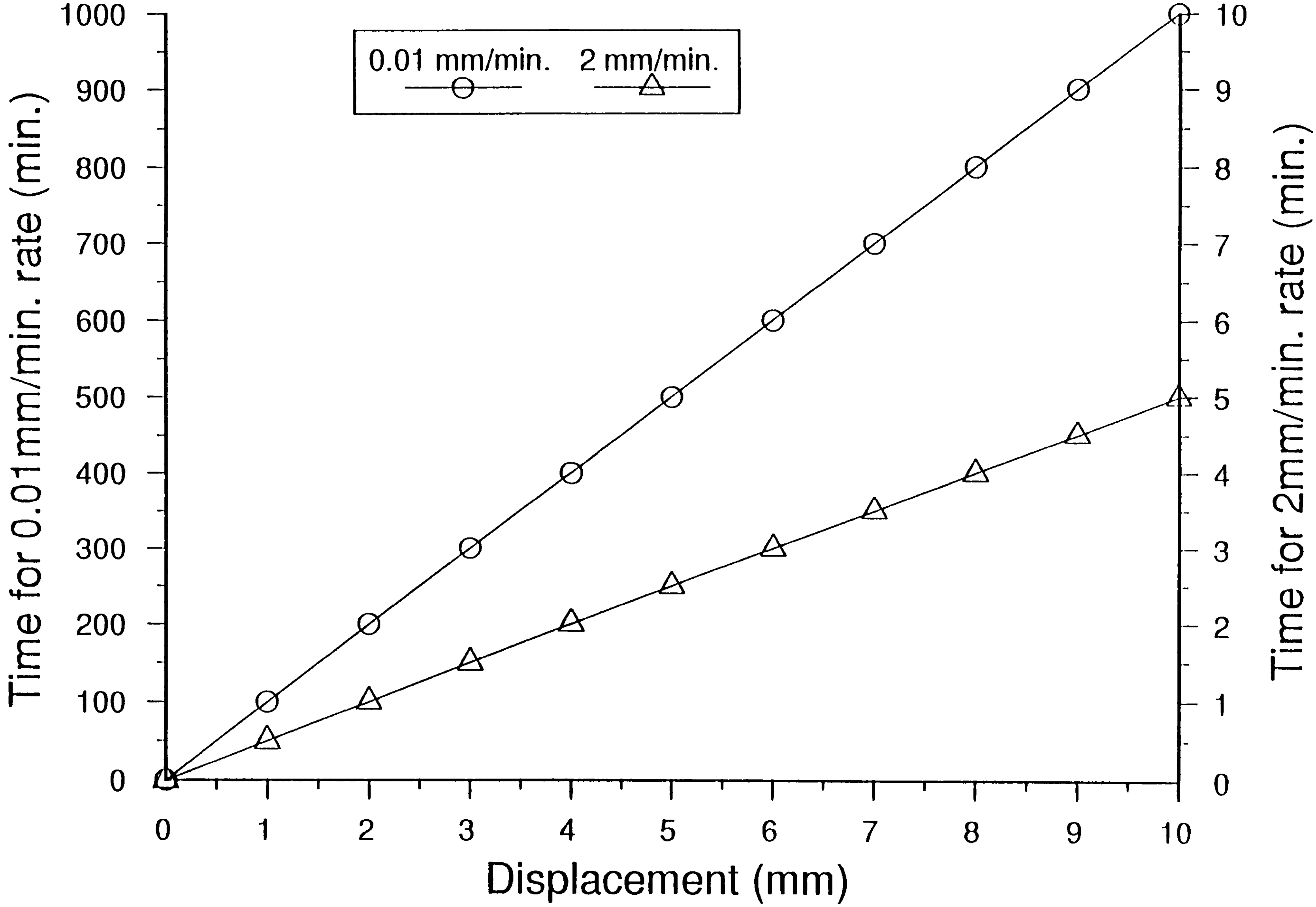


Figure 4.11 Displacement control via computer

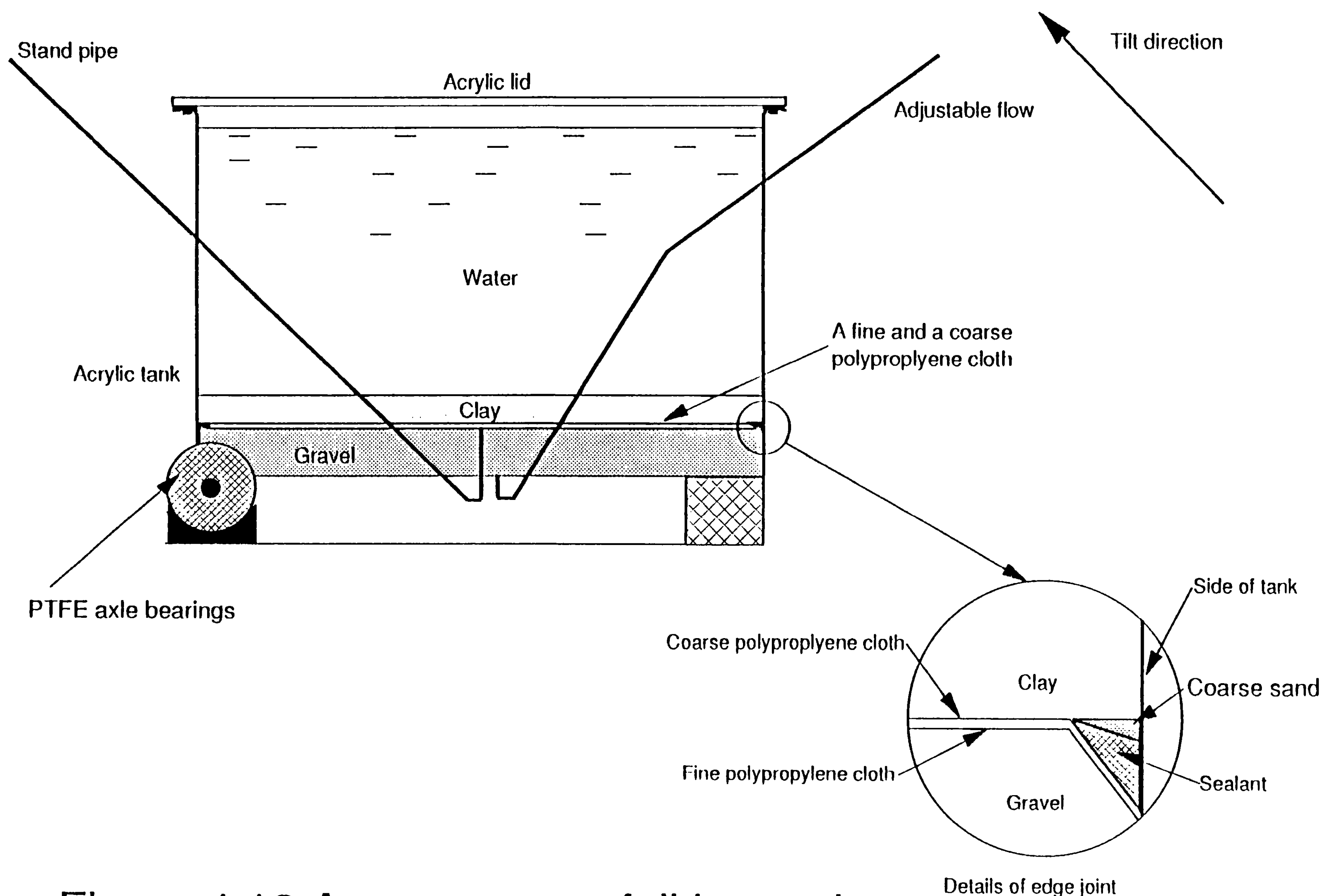


Figure 4.12 Arrangement of tilting tank.

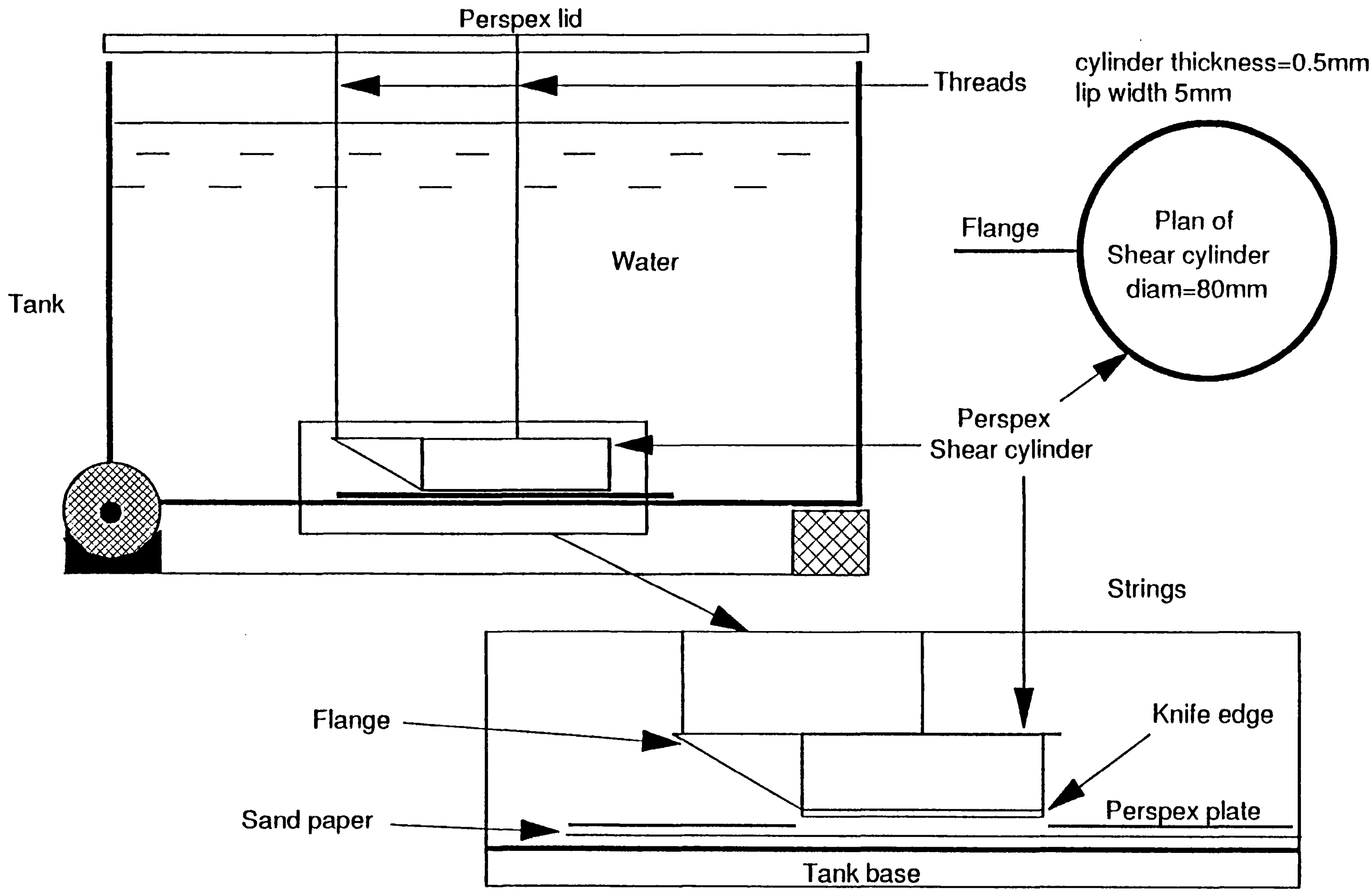


Figure 4.13 Details of shear cylinder

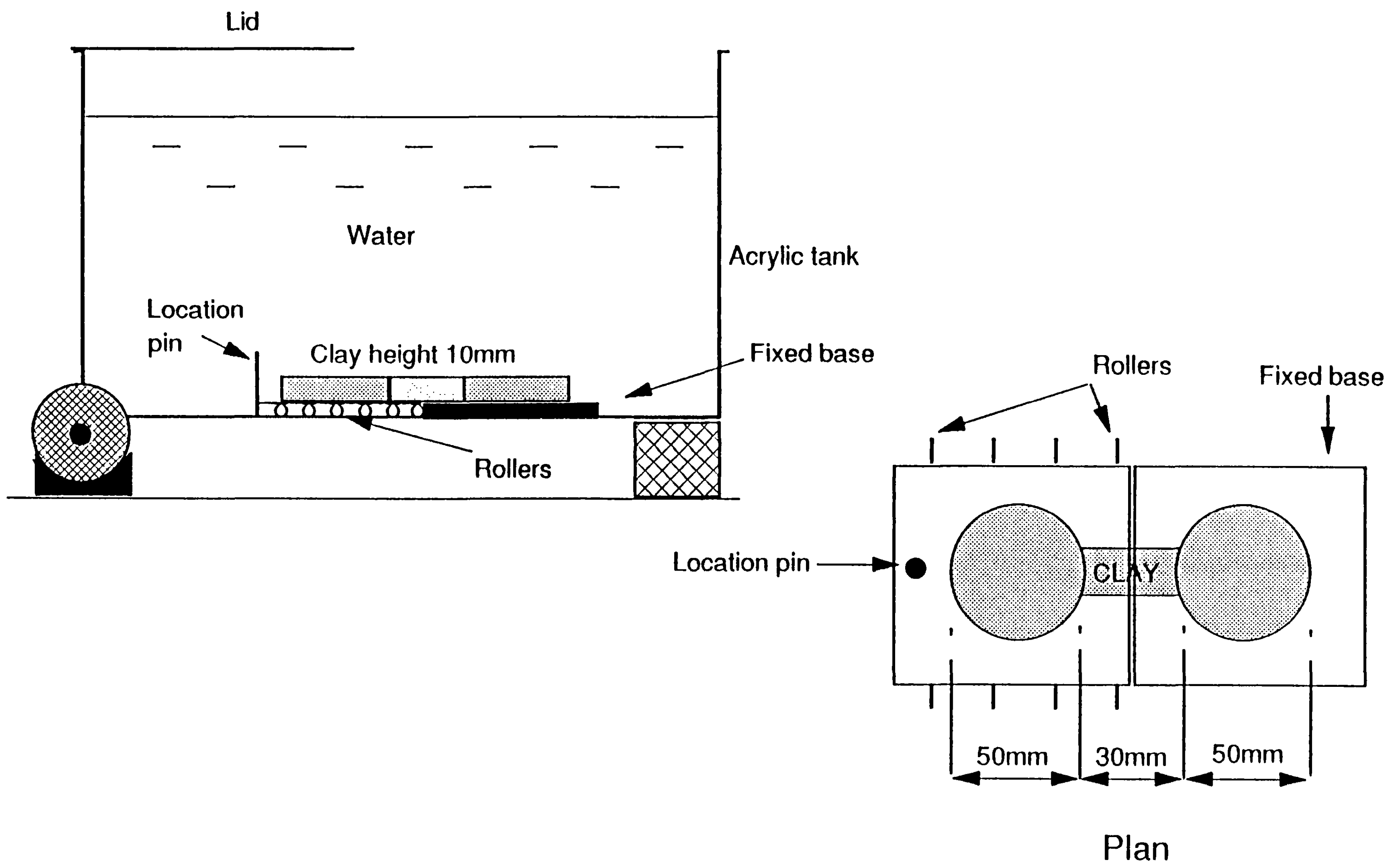


Figure 4.14 Extension test apparatus

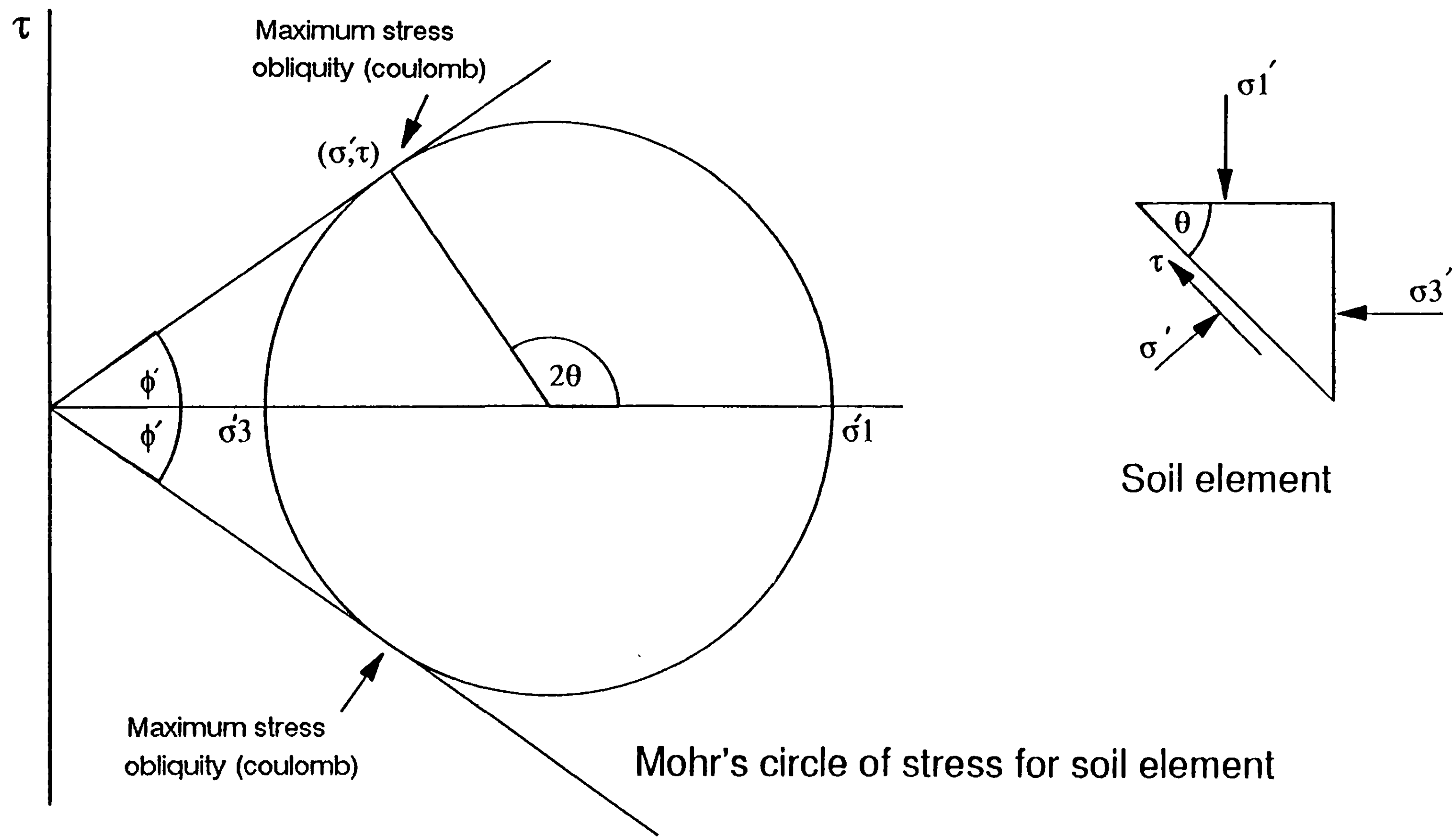


Figure 4.15 Illustration of Maximum stress obliquity (Coulomb)

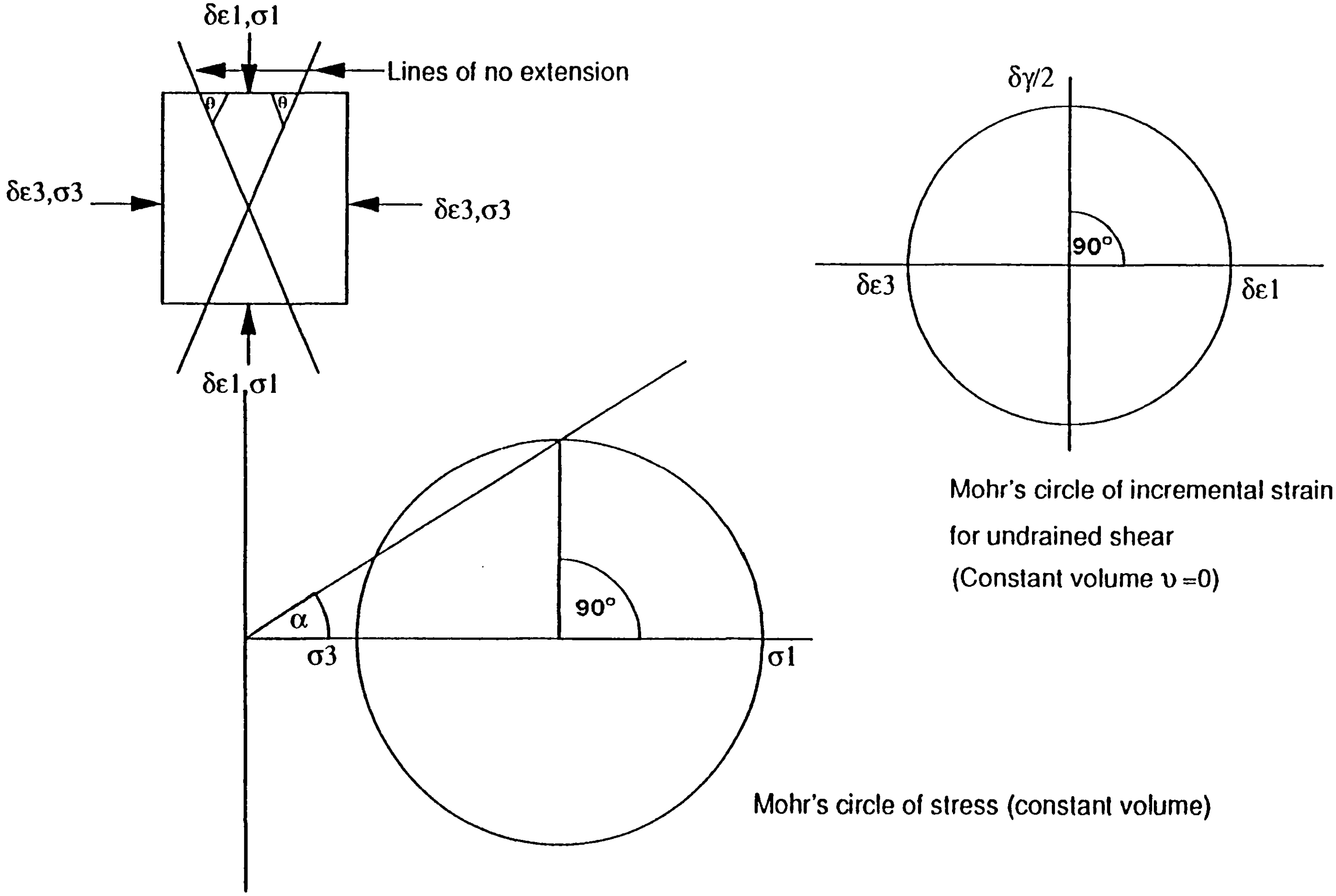
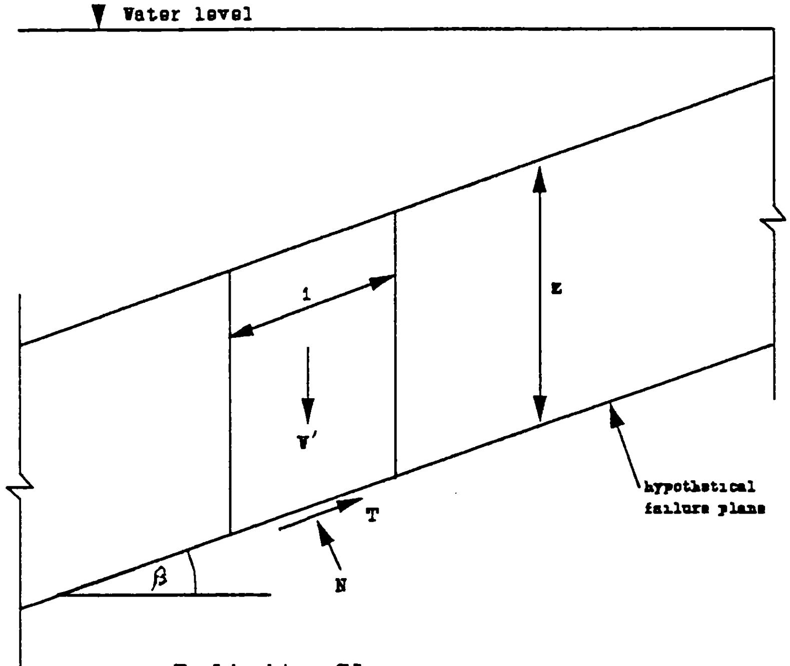


Figure 4.16 Illustration of no extension direction with coincedent axes of stress and strain increment



Infinite Slope

Soil element, unit length (down dip) and unit width along strike.
Effective weight of soil element = W'
Shear stress along failure line = $\tau = T / 1$
Effective normal stress on element = $\sigma' = N / 1$

Resolving perpendicular to failure line
$W' \cos\beta = N = \sigma' \times 1$
Therefore $W' \cos\beta = \sigma'$

Resolving parallel to failure line
$W' \sin\beta = T = \tau \times 1$
Therefore $W' \sin\beta = \tau$

$$\frac{\tau}{\sigma'} = \frac{W'\sin\beta}{W'\cos\beta} = \tan\beta$$

$$\frac{\tau}{\sigma'} = \tan\phi'$$

Therefore $\tan\beta = \tan\phi'$

And $\beta = \phi'$

Figure 4.17 Infinite slope analysis for drained conditions

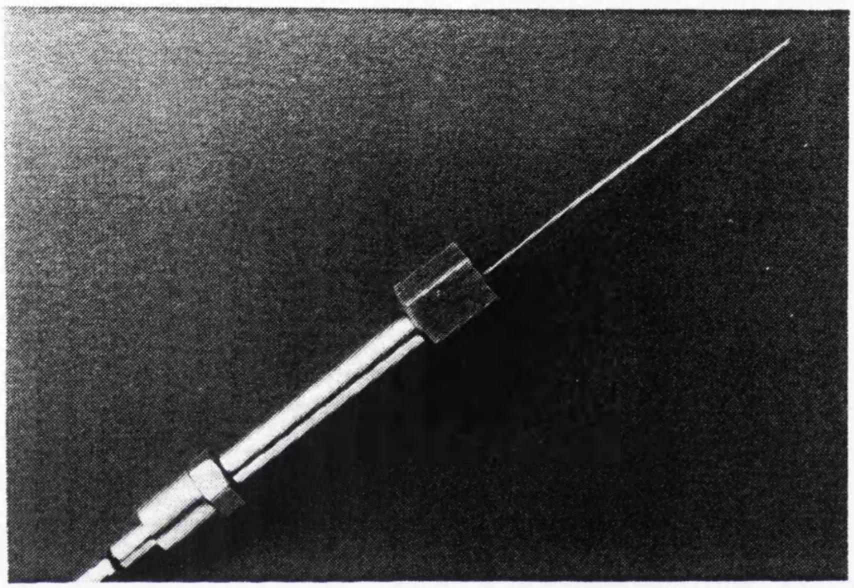


Plate 4.1 Arrangement of Druck Transducer and Needle

CHAPTER 5

PROPERTIES OF THE MATERIALS TESTED, SAMPLE PREPARATION, AND SETTING UP PROCEDURE.

5.1 INTRODUCTION

This chapter describes the procedure of preparing the samples and setting them up in the apparatus prior to testing.

In order to develop techniques to assess the shear properties of clays at much lower stress levels than previously achieved, down to 0.05kPa, two well researched clays which are easily available were used: Speswhite Kaolin and London clay (obtained from bore holes 8-10m deep at a Paddington site). The Kaolin clay was commercially available in powder form and the London clay was provided by Ove Arup and Partners, from 8m deep bore holes. All samples used were resedimented or reconsolidated from slurry. Dense Leighton Buzzard sand was also used in order to assess the apparatus, since the properties of the sand were known from other research (Arthur et al. 1987).

Special techniques had to be adopted for sample preparation, as the clay at the required stresses was very soft.

5.2 PROPERTIES OF THE MATERIALS

Property		Speswhite Kaolin	Blue London clay
Liquid Limit (LL)		69%	79%
Plasticity Index (Ip)		31%	55%
Specific Gravity (Gs)		2.61	2.71
Mineralogy	Quartz	-	20 - 25%*
	Kaolinite	80%	15 - 20%*
	Illite	20%	30 - 35%*
	Montmori-llonite		20 - 25%*

* Data obtained from Burnett and Fookes (1974) for Hammersmith and Streatham sites.

5.3 SAMPLE PREPARATION

A new procedure had to be adopted in order to obtain fully saturated samples at the low stresses required (for both the Speswhite Kaolin and the London clay). For the few tests carried out on Leighton Buzzard sand the technique adopted by Dalili (1990) was used. Sample preparation for the DSA, SSA and the Tension test and the standard tilting tank and the shear cylinder will be considered separately as the technique for each group is somewhat different.

5.3.1 Direct shear, simple shear and tension test

To prepare fully saturated samples and be able to measure pore water pressure accurately, de-airing was imperative, especially as the DSA apparatus could not provide a back pressure to push any trapped air into solution. Once a satisfactory de-aired slurry was obtained, the slurry was consolidated one-dimensionally to the required axial stress. Aluminum cylindrical tubes 102 mm in diameter and 400mm tall were used. The final increment of consolidation was carried out in the testing apparatus. The preparation of fully saturated specimens used the following procedures:

MIXING AND DE-AIRING PROCEDURES

The mixing and de-airing process played a significant role in obtaining completely saturated clay samples. Figure 5.1 shows the equipment that was used to mix and de-air the clay. The London clay had to be sliced in small pieces (approximately 2mm in thickness 5cm long and 3cm wide) and left to soak in water for a week.

This mixing and de-airing process consisted of three different stages; mixing, spraying under vacuum and shaking under vacuum.

During the first stage, 1000 grams of clay was mixed with 3000 cc of de-aired distilled water under vacuum, in a PVC cylindrical chamber (Figure 5(a)). In some cases, salt was dissolved in the water at concentration of 35grams/litre to imitate sea conditions. In the case of London clay, the weight of clay mixed was found by the concentration of the slurry. The mixing chamber was 500mm high and 160 mm diameter. The blending process continued for approximately 20 minutes.

During the second stage, the slurry was gently evacuated through a plastic pipe by opening an outlet valve. It was sprayed into a thick bell jar of 10 litres capacity maintained at 80 kPa below atmospheric pressure, as illustrated in Figure 5.1(c). As a precaution incase of breakage, a metal mesh was placed around the container under vacuum. The purpose of using a glass container was to ensure that there were no air bubbles left in the slurry by visual inspection. Once the PVC cylinder was fully evacuated, the process was repeated until the required volume of slurry (approximately 10 litres) was attained. The slurry was then left under vacuum for five to six hours; even so, when the container was shaken by hand a visual inspection revealed some air bubbles still trapped in the Kaolin.

The third stage was commenced by placing the container on a shaking table while the slurry was kept at 80 kPa below atmospheric pressure (Figure 5.1(d)). To place the glass container on the shaking table a metal frame was clamped to the table and then lined with soft pieces of sponge. The container was then placed on these sponges while the whole frame was covered with a safety metal mesh as shown in Figure 5.1(d). Vibration accelerated the de-airing process and after four hours of shaking, visual inspection revealed the sample to be satisfactory de-aired.

CONSOLIDATION PROCEDURE

Once the slurry was fully de-aired, the glass container was dismantled from the shaking table and laid horizontally on a soft base. It was then rolled back and forth gently till the slurry was thoroughly mixed and of a uniform thickness. This was conducted while the slurry was under vacuum (80 kPa below atmospheric) but with the valve to the vacuum pump closed. The next step was to release the vacuum gently and pour the slurry into the consolidation cells.

To consolidate the clay slurry, conventional oedometers were modified to accommodate the larger consolidation cells (80mm diameter and 320mm height). Figure 5.2 shows the arrangement of the modified oedometer. For the tension test, larger cylinders were used (155mm diameter and 200mm height).

The principal features of the consolidation cell, which is illustrated in Figure 5.2, are explained below: The Cell body, for accommodating the sample, was made of aluminum tubes of 110mm external diameter and 400 mm height. The internal face of these tubes was machined to the required diameter of 104mm. This was sufficient to permit trimming the edges of the sample that might have been disturbed during the extracting process. The inner surface of the cell was then lined with 0.1 mm thickness PTFE lining. The PTFE lining provided a very smooth surface between the clay and the walls of the tube, and the loading cap and the walls of the tube during consolidation. The loading cap was made from aluminium, 102mm diameter and 10mm thick. The diameter of loading cap was 2 mm smaller than the inside diameter of the consolidation cell. The force was applied to the sample centrally via a steel ball seated to the centre of top cap. A porous plate with 60 micron pores and 4mm thickness was cut to the same size as the internal diameter of the aluminum tubes. To prepare the consolidation cell, the porous plate was de-aired by immersing in boiling water for 10 minutes. It was then withdrawn from the water and forced level in the bottom of the consolidation tube. The cell was now ready to be filled with the clay slurry.

Slurry was slowly siphoned from the glass container into the consolidation cell under water through a 6.4mm plastic pipe. The consolidation cells were then left for 24 hours to consolidate under self-weight. Once the self-weight consolidated clay was strong enough to carry the weight of the loading cap, a wet filter paper was placed on the inner surface of the loading cap. The loading cap (0.3 kg equivalent to a vertical stress of 0.36 kPa) was then gently lowered and placed on the top in contact with the clay specimen and left for at least 24 hours. An aluminum load hanger was placed on the centre of the loading cap for a further of 24 hours before loading started. The loading sequence commenced in the form of 0.5kg, 1kg, 2kg, 4kg, etc (equivalent to a vertical stress of 0.61kPa, 1.21kPa 2.42kPa and 4.84kPa respectively) and these were sufficiently low to prevent the clay being squeezed out. The top of the tube was always filled with water to prevent drying (Figure 5.2). Although overall the two water surfaces set up a steady seepage gradient and corresponding variation in effective stress this was not a series problem over the 2cm height of DSA samples.

SAMPLE TRIMMING

Once the samples had reached the required consolidation stress, they were unloaded and allowed to swell for 24 hours. The consolidation tube was then placed in the NGI trimming apparatus (Geonor (1986)), Figure 5.3 and Plate 5.1, ready for the sample to be extruded and trimmed. These processes took place in a humidity cabinet at approximately 95% humidity to prevent any water evaporating from the sample. When the Jack was raised, the sample was pushed out of the tube and the porous plate taken off the sample. The sharp-edged steel sample cutter (circular, square or dumbbell shaped), 1.5 mm thick and cutting edge tapered at 15°, was placed above the sample. The sample was then jacked further into the steel sample cutter and extra clay around the side was continuously removed with a knife, Plate 5.2. This process of jacking the sample and trimming the edges of the sample was repeated until 2cm of clay had protruded through the other side of the steel sample cutter. The copper slice plate was pushed in below the steel sample cutter, Plate 5.3, and then the top of the clay was carefully cut away leaving the clay sample level with the steel sample cutter. The steel sample cutter was then removed to enable the consolidation tube and jack to be removed.

At this stage the sample was ready to be placed in the apparatus. The following describes the procedure for the DSA, SSA and the tension test.

5.3.1.1 Direct Shear Apparatus

A wet filter paper was placed at the bottom of the shear box, ensuring no air bubbles are trapped between the base and the filter paper. This box was placed on the NGI cutting frame and the steel sample cutter (with sample) was lowered just above the shear box. A filter paper, cut to fit underneath and round the edges of the loading platten of the shear box, was made wet and placed on top of the clay in the steel sample cutter. The copper cutting plate was removed and the steel sample cutter was gently lowered to locate at the top of the shear box, Plate 5.4. The top loading platten was placed on top of the sample and gently moved down to push the sample inside the shear box. Then the steel sample cutter was removed, Plate 5.5. Plate 5.6 shows the square shear box with the square steel sample cutter.

The shear box was placed in the travelling carriage and the hypodermic needle attached to the de-aired Druck transducer, for pore water pressure measurement, was pushed in from the base and screwed in position. The transducer was screwed two further turns into the needle base in order to push 2-3 small drops of water out of the needle ensuring a de-aired needle tip. The shear box in the travelling tray was placed in the DSA frame and the vertical load was applied whilst monitoring the output of the Druck transducer to ensure that it was responding to the load application. As drainage was allowed, response time tests could not be carried out. However, response time tests were carried out on a sample for each batch of clay to ensure that the clay had been de-aired as well as ensuring the transducer is in working order. See Figure 5.4 for response time results carried out in a triaxial cell. Values of 0.98 were obtained for Skempton's pore pressure coefficient B in 4 seconds.

5.3.1.2 Simple Shear Apparatus

The bottom platten raised on a pedestal was positioned at the bottom of the NGI trimming frame, and the steel sample cutter with the clay was lowered just above the bottom platten. The mid section of the reinforced wire membrane was placed inside the stretcher and the unreinforced rubber folded over the ends. A vacuum was applied using a suction pump through the tubing of the stretcher to stretch out the membrane. The copper plate was removed, the stretcher and the steel sample cutter were moved down simultaneously, leaving the sample on the bottom platten with the membrane around it. The top platten clamped to the yoke was moved down onto the sample. The suction was released. The ends of the membrane was unfolded and the stretcher was moved up. The rubber O-rings were placed onto the bottom and top platens respectively to seal the sample.

As the sample was soft a precise aluminium casing was placed around the sample and the top and bottom platens to prevent any movement of the top platten relative to the bottom, Plate 5.7, during the transportation to the SSA (Figure 4.9) which would have caused disturbance and shearing prior to the test. The travelling base plate was screwed to the bottom platten and put in the SSA. The top plate was moved down onto the top platten and simultaneously the knife edge clamp was screwed onto the top platten. The Aluminium casing was removed. The vertical guide was connected to the top platten and the vertical load was applied. The sample was left to consolidate under the final increment for 24 hours before shearing commenced.

5.3.1.3 Tension test

The sample for the tension test was delicate compared to the simple shear sample or the direct shear sample as it had a height of 10mm. The sample height was chosen so that the drainage height would be as small as possible to allow quick drainage during the shearing so that excess pore pressures would dissipate quickly. The steel sample cutter had to be modified so that the copper cutting plate would not be removed from the steel cutter until the sample was sitting on the porous stones (see Figure 4.14). This prevented the sample from coming out of the steel sample cutter unevenly and buckling in the process. The sample surrounded by the steel sample cutter was positioned in the apparatus and the copper cutting plate was then removed. A sheet of wet filter paper, the same shape as the sample, was placed on top of the sample. A block of wood, slightly smaller than the inside of the steel sample cutter was placed on top of the wet filter paper and held down gently. The steel sample cutter was pulled up, leaving the sample and the block of clay in position on the porous plates. The filter paper was removed, the sample was placed in the tank and the tank was very gently filled with water.

5.3.2 Tilting Tanks and Shear Cylinder

Prior to preparing the clay samples, the tanks had to be cleaned, prepared and de-aired. Once the tanks are ready, the clay was mixed and poured into the tanks. The following describes the procedure of tank preparation and the different methods of preparing the samples.

The tilting tanks were washed thoroughly with large amounts of water in order to get rid of the soluble chemicals in the silicon sealants. These chemicals were found to cause coagulation of the particles, leading to a different behaviour of the clay. The tanks were then filled to a quarter of their depth with water. The air trapped below the fine polypropylene cloth was removed by tapping and sucking the small pockets of air. The tanks were then gently vibrated to force out any air trapped between the gravel. The de-airing procedure was repeated until no more air bubbles were visible. The stand pipe and the outlet pipe were connected and water was allowed to flow through them to ensure no air bubbles were trapped in them. The pipes were then attached to the outside of the tank. The tanks were now ready for the clay slurry to be added.

The tank used for the shear cylinder test was cleaned, the waterproof sand paper was placed at the bottom of the tank with the perspex plate fixed above, and the cylinder was located in the hole in the perspex plate, see Figure 4.13. The tall cylinder was located on the flange of the shear cylinder and tank was filled to an initial the depth of 50mm of water whilst the cylinders were held in place.

As the London clay was obtained in its wet form, a slightly different procedure was used to prepare the sample. The Kaolin was mixed from dry powder in two different procedures to obtain different properties. The following explains in detail the method adopted for the London clay and the Kaolin clay.

LONDON CLAY

The London clay was sliced into small pieces (approximately 2mm in thickness 5cm long and 3cm wide) and left to soak in London tap water for a week. It was then mixed in a dough mixer for a period of 4 hours. A sample of 100 cc was taken out of the slurry to find the moisture content and the concentration. The moisture content was found to be 400% $\pm$20%. As the concentration was known a specific amount of the slurry was poured into a bucket and diluted with water (15 litre), making the moisture content approximately 1900%, and the slurry was mixed continuously by hand for about 10 minutes. The slurry was then gently poured in the tanks and the tanks were then topped with water. In some tests salt was added to the London tap water at 35grams/litre to imitate sea conditions.

KAOLIN CLAY

Two methods of mixing were adopted for the Kaolin to investigate the effect of the mixing, by hand and in a food blender. The water composition was also varied. The following variations were tried, one at a time:

Adding salt at a concentration of 35grams/litre to London tap water.

Using de-ionised distilled water

Adding a coagulating agent, Percol 155 at concentrations of 0.0045 grams/litre and 0.0023 grams/litre to London tap water.

The hand mixing procedure adopted was similar to that of the London clay, except that the Kaolin in its dry powder form was poured into the water in the bucket, giving a moisture content of approximately 1900%. The clay was allowed to become wet in the water and then stirred continuously by hand for 10 minutes. The slurry was gently poured in the tanks and the tanks were then topped with water.

For the blending procedure the clay was mixed at 300% moisture content as described in section 5.3.1, and then added to the bucket of water, producing a slurry at a moisture content of 1900%. The slurry was then gently poured into the tanks and the tanks were then topped with water.

Sample set up was different for the shear cylinder and the tilting tanks. In the shear cylinder only normally consolidated samples could be prepared with the present apparatus arrangement. However, it was possible to prepare overconsolidated samples in the tilting tanks. The following is a description of the sample set up in both the shear cylinder and the tilting tanks.

5.3.2.1 The Shear Cylinder

Once the clay was ready in its mixed slurry form and the shear cylinder was set up, as described in Section 5.3.2, the slurry was poured into the long cylinder. A bucket full of water (approximately 20 litres) was poured into the rest of the tank simultaneously so that the water head in the cylinder was always slightly less than the water head in the rest of the tank. This prevented any of the clay slurry escaping out of the cylinder. The long cylinder was held in place until the water in the tank came to rest; it was removed 24 hours later, Plate 5.8.

The sample was left to consolidate for the number of days required. Once the required period of consolidation had been achieved the lid was placed and clamped on top of the tank with the threads attached to the cylinder going through the holes in the lid. The lid was adjusted so that the threads are vertical. Rubber bungs are placed in the holes to prevent the threads from falling through. The threads are tightened by pulling. The three threads are pulled up simultaneously by 2mm so that the shear cylinder was no longer in the groove and a small gap was created between the shear cylinder and the bottom perspex plate, Plate 5.9. The sample was left for four hours to mitigate any disturbance due to lifting the cylinder and then the tilting proceeded.

5.3.2.2 Tilting Tanks

The tilting tanks allowed both normally and over-consolidated samples to be prepared. The procedure for over-consolidated sample preparation was the same as that for normally consolidated with an extra step. The slurry was gently poured into the tank and then topped up with water. Silicon sealant was placed around to the top flange of the tank and a perspex lid was placed on top. Sprung paper clips are then placed all around the tank to seal the lid. The clay was left for the required period before tilting starts.

In the case of the over-consolidated samples, the outlet pipe at the base of the gravel drainage layer was gently lowered to 5mm below the water level in the tank, allowing water to flow out of it for one hour. It was then lowered further to give a specific water head difference across the sample depth. The pressure drop can be monitored accurately on the inclined stand pipe. This difference in head provides an extra consolidation stress across the clay and was normally allowed to remain for 24 hours. Water was fed into the tank continuously during this further consolidation process, and the water level was maintained constant with the overflow at the top of the tank.

Once 24 hours have passed (in some cases, this period was varied to quantify the effect on the test results), the outflow pipe was gradually raised up (at two or three stages over a period of one hour) beyond the water level and the sample was allowed to swell back creating an over-consolidated sample. The gradual setting up and reduction of the hydraulic gradient was necessary to prevent any uneven consolidation or swelling back which can result in any hole or crack formation. This period of 24 hours was chosen since it was found by Zadeh-Koch (1989) that this period was sufficient to allow full primary sedimentation producing a soil skeleton. The period of swell back was normally 24 hours, however other periods were used in order to assess the difference on the results. The taps on the adjustable flow pipe and the stand pipes were always closed prior to tilting.

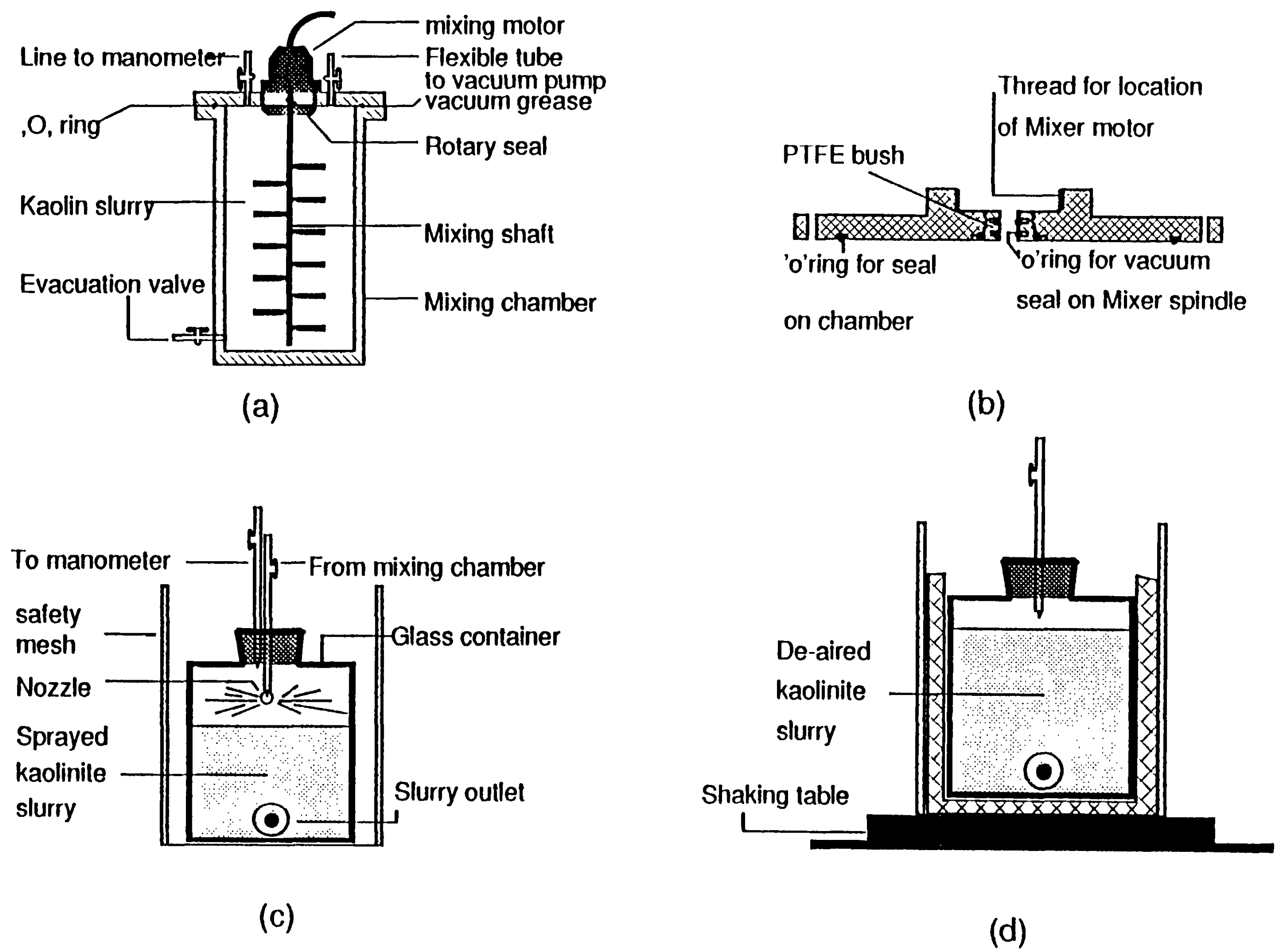


Figure 5.1 Mixing and de-airing apparatus

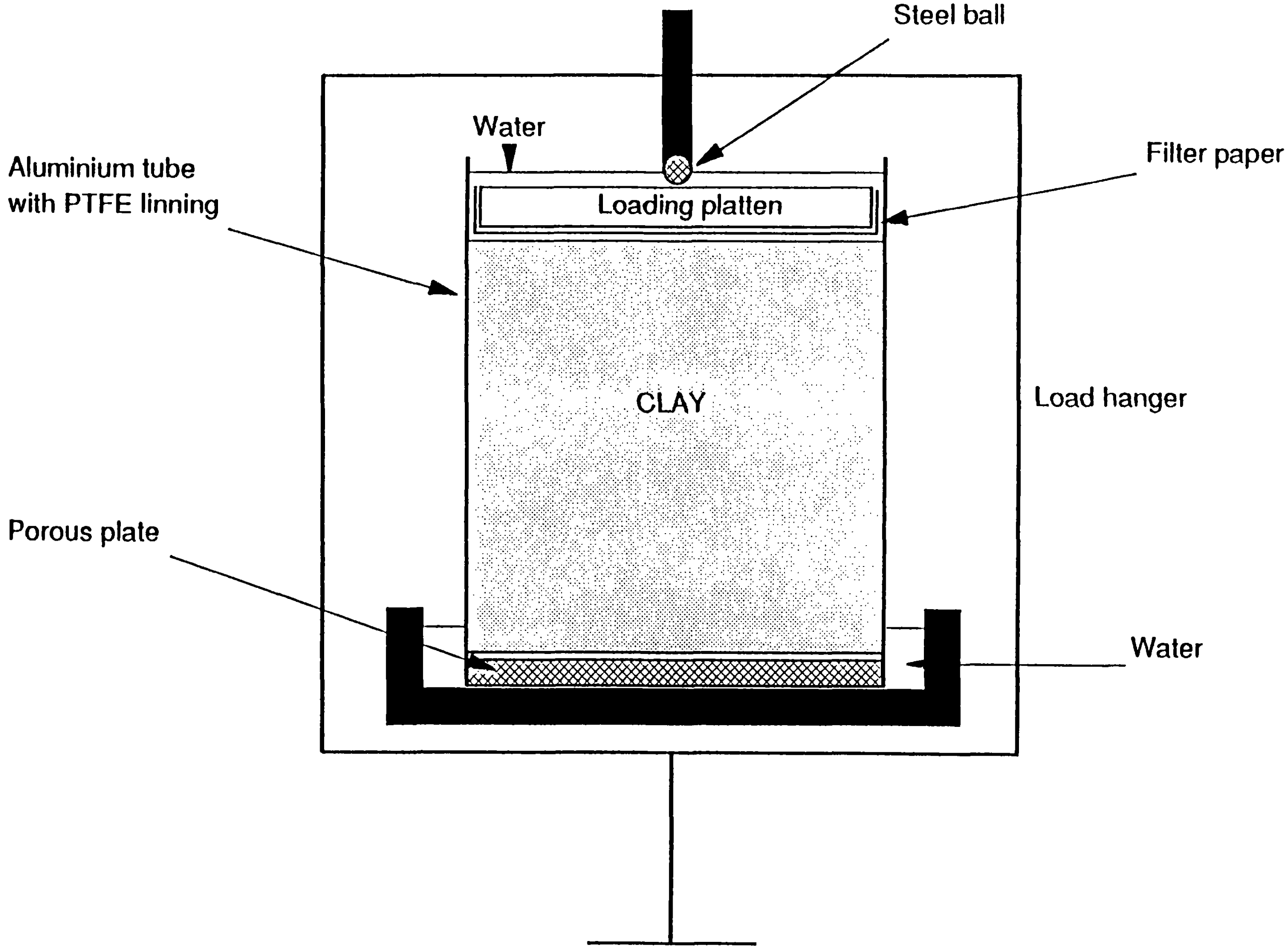


Figure 5.2 Arrangement of consolidation cell

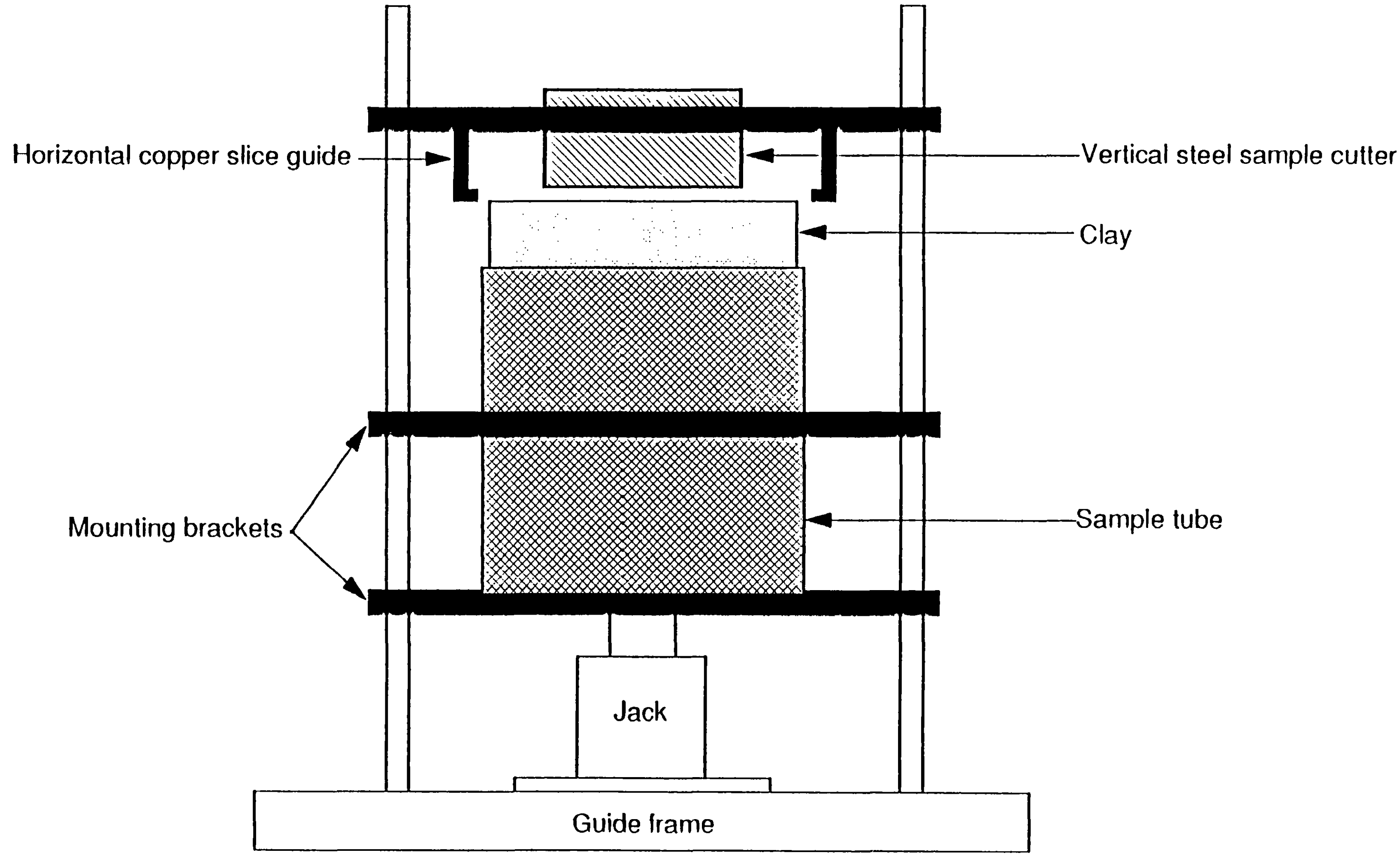


Figure 5.3 Arrangement of NGI trimming apparatus

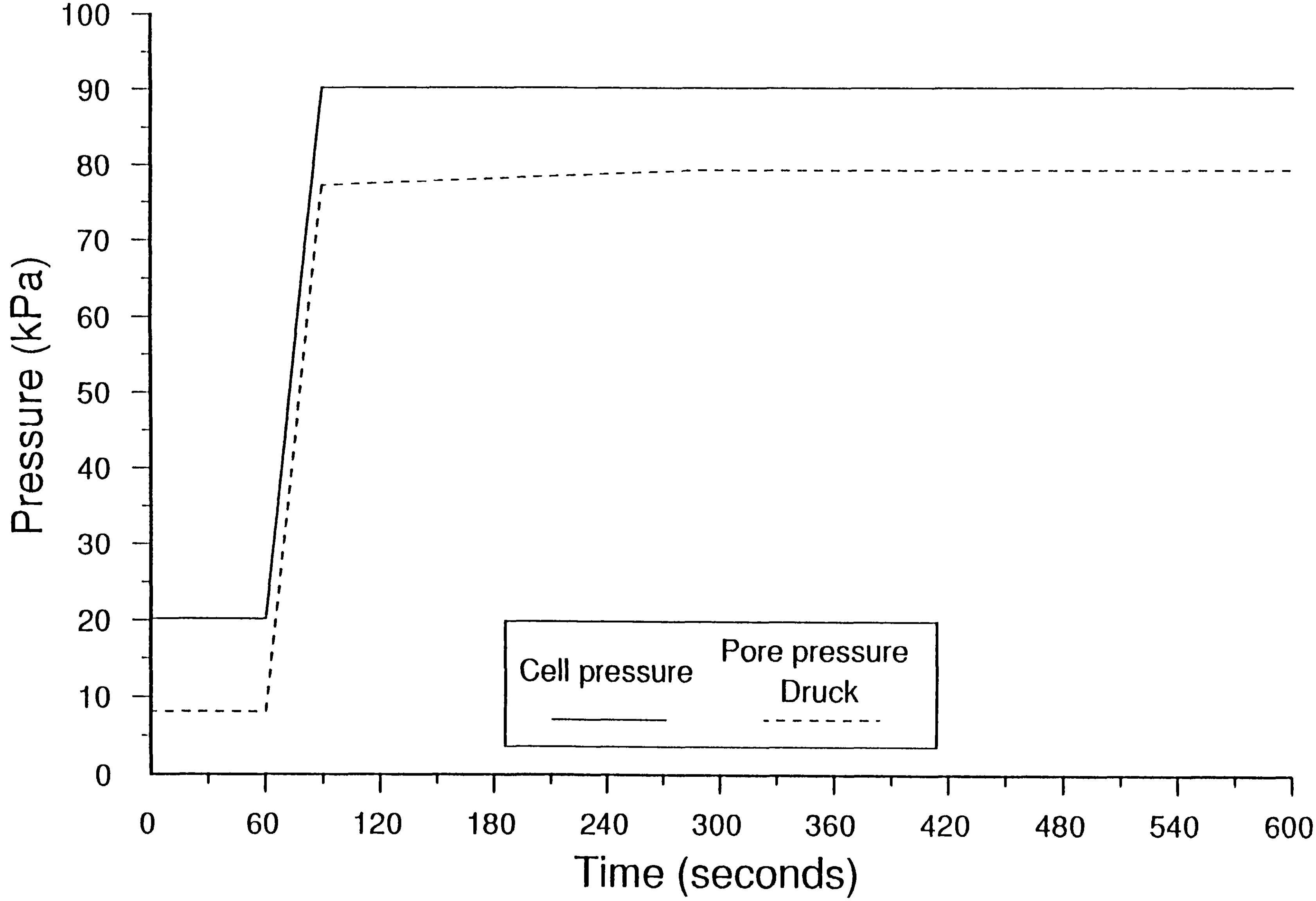


Figure 5.4 Druck transducer response time

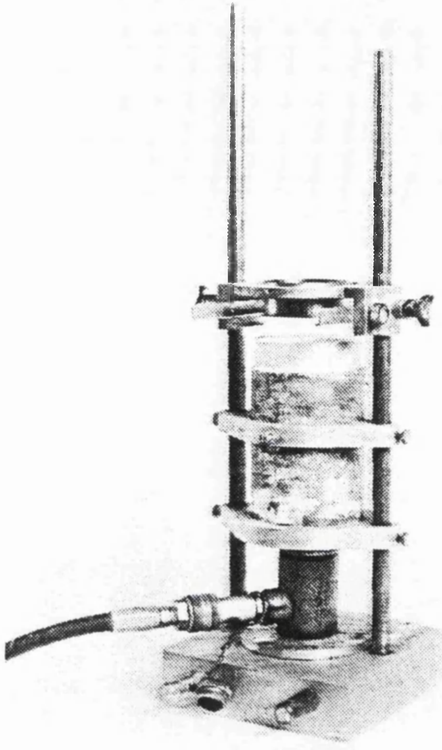


Plate 5.1 The NGI Trimming Apparatus

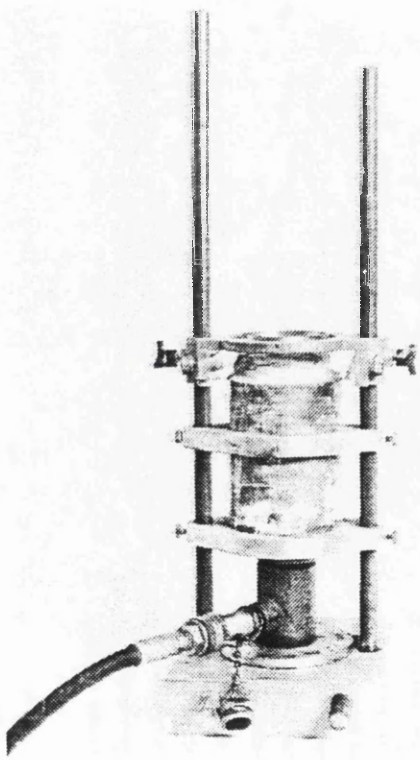


Plate 5.2 Sample Trimming in the NGI Apparatus (stage 1)

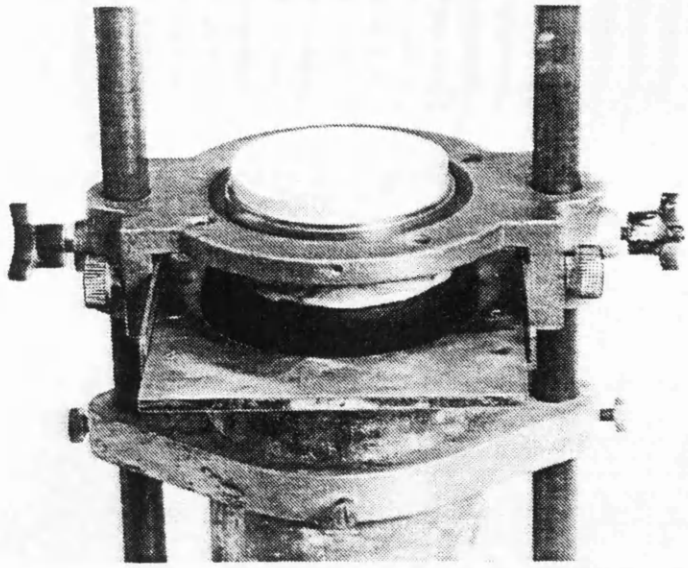


Plate 5.3 Sample Trimming in the NGI Apparatus (stage 2)

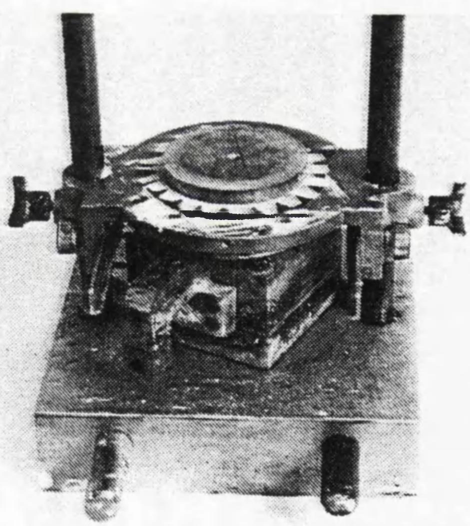


Plate 5.4 Sample Placement in the Shear Box

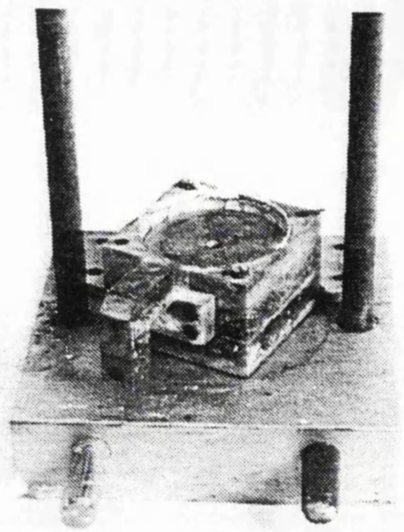


Plate 5.5 Final Arrangement of Sample in the Circular Shear Box

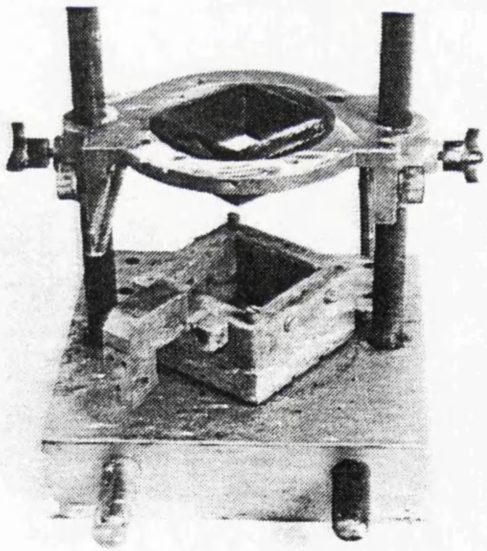


Plate 5.6 Arrangement of the Square Steel Sample Cutter and the Square Shear Box

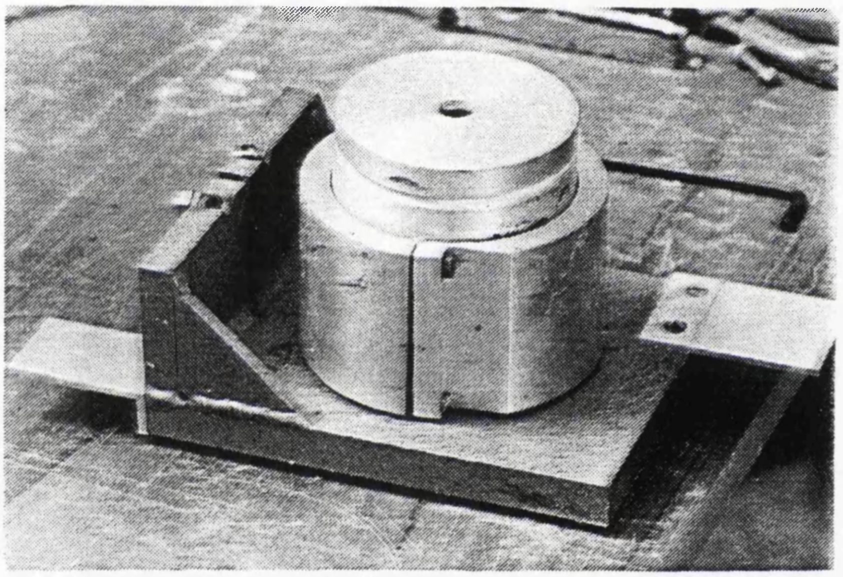


Plate 5.7 Arrangement of Simple Shear Sample Inside the Aluminium Casing Prior to Placement in the SSA

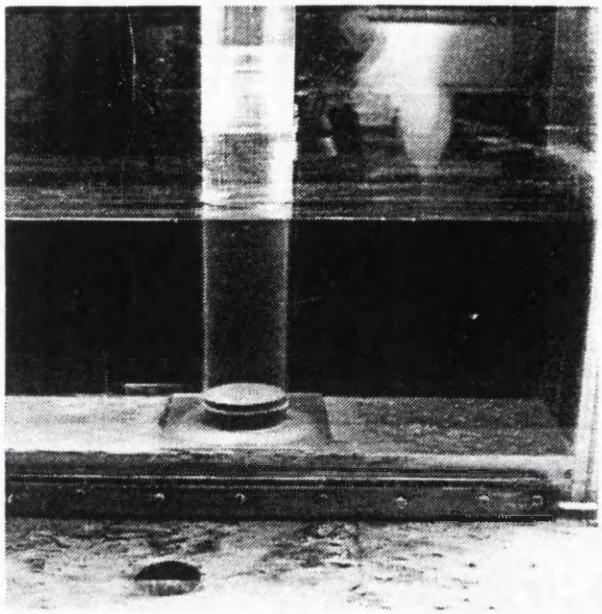


Plate 5.8 Sample Arrangement in the Shear Cylinder

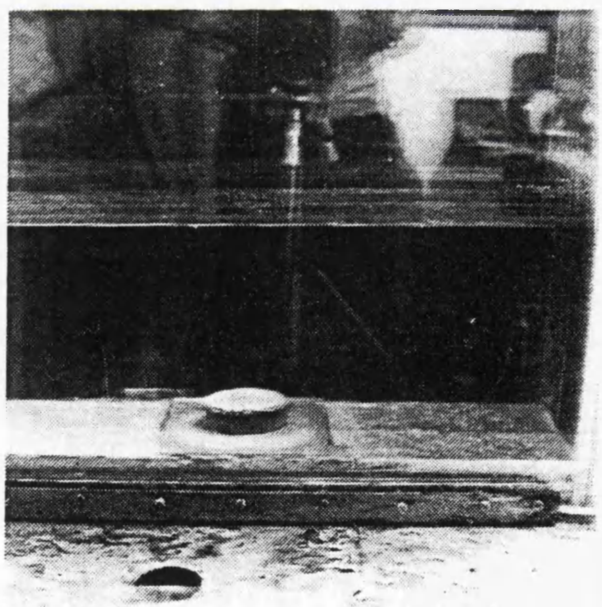


Plate 5.9 Sample Arrangement in the Shear Cylinder Prior to Testing

CHAPTER 6

PRESENTATION OF RESULTS

6.1 Introduction

As seen from the previous chapters, this research project has utilised a variety of standard laboratory testing techniques which, in some cases, have themselves been modified to suite the particular needs of the project. For simplicity this chapter presents results obtained from each of the testing apparatus used and the variation in the testing techniques and conditions. Discussion of the results will be presented in Chapter 7. However there is one aspect of analysis which can not be deferred entirely to Chapter 7.

6.2 Thixotropy and rate of testing

Thixotropy is defined as an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume whereby a material stiffens while at rest and softens or liquifies upon remoulding (Freundich, 1935). The measurement of thixotropic properties of a material is influenced by the testing techniques, and therefore only load control tests could quantify this property. The displacement control tests would result in remoulding of the samples during the shearing process and destroy the thixotropic strength, Section 7.4.8. Although the tilting tanks are only load control tests, the rate of tilting is also an important factor in quantifying the thixotropic strength. The very fast rate used ($\approx 15°$/sec.) would be sufficient to measure the thixotropic strength, provided pore water pressure was not generated (Section 7.4.4). Due to the clay viscosity when tilted at the fast rate, very small strains would have occurred during tilting. These small strains are not sufficient to remould the sample and destroy the thixotropic strength. The slow incremental rate (3°/day) would also be sufficient to measure the thixotropic strength, since in pre-failure loading the small increments of load would each only produce very small strains which are not sufficient to remould the sample significantly and thereby destroy appreciable thixotropic strength. More over the 24hr period of rest between load increments would allow the clay to rebuild any thixotropic strength that may have been lost in the previous increment of loading. Both the magnitude of individual load increments and the length of the time interval between increments are important in achieving measurement of the full thixotropic strength. The continuous rate of tilting used in this research was arrived at so that only the frictional strength is measured, hence this rate was chosen such that the angle of tilt at failure was a minimum once consolidation equilibrium was reached. This rate allowed pre-failure strain to develop during tilting so that the thixotropic strength was destroyed and only the frictional strength component was effective. For both normally consolidated Kaolin and London clay a range of tilting rates between 10°/min. and 1°/min. showed that any tilting rate equal or less than 6°/min. resulted in measuring the minimum angle of tilt at failure and subsequently a rate of tilt of 4°/min. was chosen to produce the above requirement, Figure 6.1.

The DSA is also effective at measuring the thixotropic strength providing load control tests are performed. A fast rate of loading was not possible with the present apparatus set-up, and a slow incremental rate was chosen (1 increment/day). However, for both the standard tilting tanks and the DSA the effect of overconsolidation resulted in transforming the previous thixotropic strength in the clay (Section 7.4.5) so that it was stable regardless of the pre-failure strain developed, thus allowing the thixotropic strength to be measured using any of the testing techniques used in this research at any of the above rates of loading.

The measurement of thixotropic strength has been shown to depend on the incremental loading technique. However the rate of continuous loading can have a significant effect on the test results and it is therefore important to differentiate the effect of the testing rate on the test result. Bjerrum et al. (1958) carried out a series of drained and undrained triaxial tests with pore pressure measurements, covering widely differing rates of continuous loading, on marine clay from Fornebu, Oslo. They showed that the undrained shear strengths decreased with increasing time to failure, and that this decrease cannot be explained solely by the observed increase in pore pressure at failure with time, but must be a result of a reduction in the true cohesion and/or the true angle of internal friction, with increasing time to failure. However for the drained tests the drained strengths appear to be independent of the different rate of loading adopted in their investigation (1.3%/hour to 0.034%/hour) and therefore measured a unique effective strength parameters regardless of the rate of loading. They suggested that the reduction in the true cohesion and/or the true angle of internal friction with time, which was shown by the undrained tests, is compensated by an increase in drained shear strength, due to the decrease in water content, resulting from secondary time effect during the shear tests. The test carried out in this research programme consisted of fully drained, partially undrained and fully undrained tests (in the case of the SSA tests run at a constant volume). Tests carried out in the tilting tanks will be shown in section 7.4.4 to have occurred under drained conditions and by the evidence provided in section 7.4.4 it is shown that there was no rate effect on the results obtained. This is also shown by the range of tilting rates used to establish a standard continuous tilting rate, as described above, which all yielded the same angle of tilt at failure at and below the 4°/min. tilting rate. This is in agreement with the observations made by Bjerrum et al. (1958) for drained tests described above.

The DSA was run at three continuous displacement rates; the slowest rate (0.01mm/min.) which produced a fully drained situation (pore pressure during shearing remained zero), an intermediate rate (0.1mm/min.) which produced a partially drained situation (pore pressure development during shearing as well as a significant volume change) and a very fast rate (2mm/min.) which produced a partially undrained situation (large pore pressure development during shearing with very small volume change), Section 6.3. The rate effect in the continuous displacement control DSA had no effect on the measurement of the thixotropic component; as displacement control tests could not be used to quantify thixotropy (see above) and the overconsolidated sample could not be tested successfully due to the inability of the Druck transducer to measure pressure accurately below atmospheric. However the failure mechanism (Section 4.6) will be shown in Section 7.4.9 to be influenced by the rate of testing in the DSA. The fast rate produced an MSO and the slow rate produced an NED whilst the intermediate rate allowed failure to occur in between MSO and NED, Section 7.4.9. Similar failure mechanisms occurred in the tilting tanks which were not due to the rate of tilting but were dependent on the structure of the clay at these very high voids ratios (Section 7.4.9).

6.3 Direct Shear Apparatus

The DSA was used to perform both displacement control and load control tests on both Kaolin and London clay. Table 6.1 summarises the different test conditions adopted in the DSA, whilst Tables 6.2 to 6.5 present the results of these tests, including the stress ratios at failure (τ/σ_v'), the proposed mechanisms of failure and the effective angles of friction (ϕ'). It is immediately apparent from Table 6.2 (displacement controlled direct shear results on normally consolidated Kaolin) that the consolidation stress, the shear box shape, the duration of consolidation prior to testing and the water composition have no effect on the stress ratios at failure (τ/σ_v'), the failure mechanism or the effective angle of friction (ϕ').

It can be seen, however, that the rate of displacement has the effect of influencing the failure mechanism; the fast rate (2mm/min.) results in a failure on the maximum stress obliquity failure mechanism whilst the slow rate

(0.01mm/min.) results in a failure on the no extension direction, with coincident axes of stress and strain increment (NED). The intermediate rate (0.1mm/min.) results in a failure in between the above two mechanisms. The stress ratios (τ/σ_v') at failure are different and the effective angles of frictions (ϕ') are the same (22.5°) for all displacement rates used. It should be noted that the accuracy of measurements was ± 0.5° (due to the maximum inaccuracy measured in the shear load cell and Druck transducer, Section 4.2.1). The mean effective angle of friction (ϕ') measured was 22.5° with a standard of deviation of 0.34. Data for the same clay tested in the Cambridge Simple Shear Apparatus and the Cambridge True Triaxial Apparatus gave ϕ' as 22.5° and 23° respectively for mean normal stress levels from 80kPa up to 600kPa (Borin 1973 and Pearce 1970).

Figures 6.2 to 6.4 show examples of typical behaviour of Kaolin for all displacement rates. The fastest displacement rate produced the lowest maximum shear stress and the highest pore water pressure, whilst the slowest displacement had the opposite effect with zero pore water pressure indicating drained conditions. The intermediate rate produced a maximum shear stress and pore water pressure in between the fast and the slow rates of displacement. The fast displacement rate showed the most brittle behaviour where the maximum shear stress was achieved at very low displacement (0.3mm); whilst the slow rate had a more ductile behaviour achieving the maximum shear stress at greater displacement (>6mm). The intermediate rate produced a stiffness in between the fast and the slow rate. The fast displacement rate resulted in very small volume change (<0.2mm reduction in sample height) indicating almost undrained behaviour whilst the slow displacement rate produced large volume change (>1mm reduction in sample height). The intermediate displacement rate resulted in approximately 0.7mm reduction in sample height. Figure 6.3(a) shows the effective stress paths for three tests performed at a displacement rate of 2mm/min. at three different stress levels (5, 15 and 50kPa). An effective angle of friction (ϕ') of 22.5° is obtained with a zero cohesion intercept. This provides further evidence that the strength component (without thixotropy) is constant at different stress levels and is purely frictional.

The results for London clay (Table 6.3) show some difference from those described above for Kaolin. Namely, in the case of London clay, the increase in the number of consolidation days prior to testing, from 1 to 7 and thereafter, resulted in the reduction of water content (due to creep, measure by interpretation of the vertical displacement of the top cap) which consequently increased the effective angle of friction (ϕ') from 20.5° to 29°. Gibson (1953) reported a ϕ' of 20.5° for normally consolidated London clay in the DSA and Bishop et al. reported a ϕ' of 21° for normally consolidated clay in the triaxial apparatus. Figures 6.5 to 6.6 show the typical behaviour of London clay for all displacement rates. Similar behaviour can be observed to that of Kaolin described above.

The results from the displacement control direct shear tests on overconsolidated samples of both Kaolin and London clay are shown in Table 6.4. Unlike those of the normally consolidated samples described above, the results in this case are inconclusive due to the small number of tests performed and the inability of the pore pressure transducer to record accurate negative pore pressures (below atmospheric). In particular, tests performed at the displacement rate of 2mm/min. indicate the development of negative pore pressure during shearing. However, the two tests performed at the slow displacement rate of 0.01mm/min do not develop pore pressures during shearing (zero pore water pressure measured throughout the tests), and show a significant increase in the effective angle of friction (ϕ'). It will be shown in Chapter 7, that this result is attributed to the transformation of the previous (Section 7.4.5) regardless of the pre-failure strain; this phenomenon only exists below a maximum consolidation stress of 5kPa for Kaolin and gradually reduces for London clay above a vertical consolidation pressure of 8kPa reaching a minimum $\phi' = 32°$ at a vertical consolidation stress of 20kPa. Figures 6.8 to 6.10 show the behaviour of the two slow tests on Kaolin.

The load control direct shear results for Kaolin and London clay are presented in Table 6.5. For the normally consolidated samples, a rate of 1 increment/day was adopted, whilst for the overconsolidated samples a more rapid rate of 1 increment/30 minutes was used (1 increment = 0.1kg shear load, corresponding to 0.2kPa for the circular shear box and 0.27kPa for the square shear box).

Normally consolidated Kaolin tests showed an increase of the effective angle of friction over the displacement control tests. Here, ϕ' was found to be $38° \pm 1°$ at a value of σ_{v0}' of 5kPa and below, which will be shown in Chapter

7 to be due to thixotropy. For values of σ_{v0}' above 5kPa, the measured effective angle of friction was equal to that in the displacement control DSA (ϕ' = 22.5°), whilst normally consolidated London clay showed a steady reduction in the effective angle of friction (ϕ') from 38.7° at consolidation stress (σ_{v0}') of 5kPa to 32° at 20kPa. Figures 6.11(a), 6.11(b), 6.11(c), 6.12(a) and 6.12(b) show the typical behaviour of both Kaolin and London clay.

Water composition (i.e. distilled, London tap water (LTW) and addition of salt) had no effect on any of the stress ratios at failure (τ/σ_v'), failure mechanism or the effective angle of friction.

For the overconsolidated Kaolin, samples swelled back from a maximum vertical stress of 5kPa preserved the thixotropic strength seen in normally consolidated samples consolidated to the same stress. However, sample swelled back from a vertical stress greater than 5kPa were seen to have lost this thixotropic strength. Overconsolidated London clay, however, showed no complete loss of the thixotropic strength component at a vertical stress greater than 10kPa. This is in accordance with the behaviour of normally consolidated London clay which was seen to have some thixotropic strength at a vertical consolidation stress of 20kPa.

Voids ratios versus consolidation stress for all normally consolidated samples are presented in Figures 6.13 and 6.14 for both Kaolin and London clay respectively.

6.4 Simple Shear Apparatus

The simple shear apparatus was only used for displacement control tests on Kaolin. Pore pressure measurements were not feasible as the purchased Wykeham Farrance SSA could not be easily adapted for this purpose without the complete redesigning of the apparatus in order to avoid sample disturbance. An attempt was made to run constant height tests in order to mimic undrained constant volume tests, as suggested by Airey and Wood (1986). However, as explained in Section 4.3, the vertical stress measurements were inaccurate and an effective stress path could not be established although the height was kept constant. The shear strength measurements were the same ($\tau/\sigma_v = 0.20$) as those measured in the shear box at the fast rate and similar to those measured by Airey (1984) ($\tau/\sigma_v = 0.20$), indicating the feasibility of using the SSA at the same stress levels used in the DSA in this research and the SSA as used at NGI by Lacasse et al. (1985) where constant height tests were performed on undisturbed samples of clay at a vertical consolidation stress of 25kPa. Figure 6.15 shows a typical result at a vertical stress (σ_v') level of 5kPa.

6.5 Tilting Tanks

As described in Chapter 4 three types of tilting tank tests were performed: The standard tilting test, where a submerged, initially horizontal layer of clay under plane strain conditions was tilted to failure; when the mass of clay slides down to the bottom of the tank.

The shear cylinder test, where tilting a clay sample in a cylinder (Figure 4.13), causes failure on a predominant plane with no end restraint as in the DSA, but under very low effective stresses (0.05kPa self weight), and where the shear load is introduced by the component of the buoyant self weight of the clay and the cylinder in the direction of the tilt.

The tension test, where a dumbbell shaped overconsolidated clay sample (Figure 4.14) is tilted in the tank until the bottom half breaks away from the top half due to the component of buoyant self weight of the bottom half of the clay and the mobile plate underneath.

6.5.1 The Standard Tilting Tests

Both normally consolidated and overconsolidated Kaolin and London clay were tested in the tanks. Table 6.6 summarises the different test conditions adopted in the standard tilting tank and Table 6.7 presents the tilting tank results for normally consolidated Kaolin and London clay. Table 6.8 presents the results for normally consolidated Kaolin and salt, Kaolin and coagulant and London clay and salt. All the above tables include the stress ratios at failure (τ/σ_v'), the proposed mechanism of failure and the effective angle of friction (ϕ').

6.5.1.1 Normally consolidated Kaolin

Three tilting rates were adopted for reasons described in Section 6.2 : (i) $\approx 15°$/sec., (ii) 4°/min. (iii) 3°/day incremental. Normally consolidated Kaolin yielded these failure angles: (i) 34° at 1 day, 38° at 4 days and thereafter (ii) 22.5° at all times (iii) 38°. According to conventional infinite slope stability analysis under drained conditions these tilt angles are equivalent to ϕ' as the sliding occurs on maximum stress obliquity planes (MSO), Section 4.6. Considering the result for rate (ii) it is noted that data for the same clay tested in the Cambridge Simple Shear Apparatus and in the Cambridge True Triaxial Apparatus gave ϕ' as 22.5° and 23° respectively for mean normal stress levels from 80kPa up to 600kPa (Borin 1973 and Pearce 1970). The mean stress level in the tilting tests was about 0.05kPa with the void ratio 6.1 (the void ratios were calculate by taking the mean depth of the clay from 20 points around the perimeter of the tanks). This is an excellent correlation; drained conditions have been confirmed in section 6.2 and will be further discussed in Section 7.4.4, and further analyses in Sections 7.4.5 and 7.4.9 provides even stronger support for the use of conventional infinite slope analysis. The accuracy of the measurement was $\pm 0.5°$ due to the inclinometer used to measure these angles and the mean effective angle of friction (ϕ') was 22.5°

(maximum and minimum angles of tilt at failure measured in any tests were 23.0° and 22.0° respectively). All failures were seen to occur as a mass disintegration of the clay bed, Plate 6.1. It was found that no creep occurred and the voids ratio remained constant at 6.1 regardless of the number of days since sedimentation started.

Two effective vertical stresses were used, 0.1kPa and 0.05kPa (according to the initial dry weight of Kaolin 0.75 and 1.5kg respectively), which had no effect on the test results or the voids ratio (angles of tilt at failure were 22.5° and voids ratio at the end of consolidation were 6.1). No effect was observed on the failure angle for Kaolin mixed in the food blender regardless of the number of days since sedimentation, although the void ratio was almost doubled ($e=11.2$). Figure 6.16 shows the sedimentation curve for Kaolin with an initial concentration of 34 gms/litre.

6.5.1.2 Normally consolidated London clay

Normally consolidated London clay failure angles were as follows:(i) 38° at 1 day and >38° thereafter (ii) 20.5° at 1 day and 32° at 6 days and thereafter (iii) 32°. Only the 20.5° and 38° at 1 day results were associated with disintegration of the mass at failure. The other failures were on a thin plane, and the clay mass moved as a block. The voids ratio was found to have reduced due to creep from 6.1, after one day of sedimentation, to 3.2 at 6 days after sedimentation and thereafter, this was only seen to occur in the London clay mixed with London tap water. It will be argued in chapter 7 that a mass disintegration of the clay bed is associated with maximum stress obliquity failure mechanism and the failure on a thin plane as a block, Plate 6.2, is associated with a no extension direction with coincident stress and strain increment.

Figure 6.17 shows the sedimentation curve for London with an initial concentration of 34 gms/litre. It can be seen that after one day the voids ratio reached 6.1 which the same as the Kaolin; however a continuous reduction of the voids ratio (creep) was seen to occur over the next five days which resulted in a final voids ratio of 3.2; no further significant creep occurred after six days of sedimentation.

6.5.1.3 Normally consolidated Kaolin and Salt

The sedimentation and completion of self weight consolidation were slowed. However the voids ratio achieved 24hrs after pouring the slurry into the tank was the same as for the Kaolin mixed with London tap water ($e=6.1$). No creep was observed and the voids ratio remained at 6.1, this observation extended over 18 days.

Failure angles were as follows: (i) 38° at 1 day and >38° thereafter (ii) 22.5° at 1 day and thereafter (iii) 32°.

6.5.1.4 Normally consolidated London clay and salt

The sedimentation and completion of self weight consolidation were slowed. However the creep was accelerated so that it was virtually complete in one day; the voids ratio after 24hrs after pouring the slurry into the tank was the same as for the London clay mixed with London tap water left to creep for six days or more ($e = 3.2$).

Failure angles were as follows: (i) 38° at 1 day and >38° thereafter (ii) 32° at 1 day and 38° thereafter (iii) 38°. Note correspondence of (iii) to Kaolin with London tap water.

6.5.1.5 Normally consolidated Kaolin and coagulant

Addition to Kaolin of the coagulant agent, Percol 155, in two different concentrations of 0.0045 gms/litre and 0.0023 gms/litre had interesting effects which were not influenced by the range of concentration quoted. The voids ratio of the samples with added coagulant increased to 8.9 yet shear strengths also rose very appreciably; there was no creep. Results for tilting tests at rates defined above on normally consolidated samples were as follows: (i) angle of tilt at failure in the range 53° to 60° (ii) angle of tilt at failure 39°-40° (iii) this category was altered a little with two subdivisions: (a) rotation at 3°/day started at an initial tilt of 30° or less, (b) rotation at 3°/day started at 32° initial tilt. For (a) the failure angle of tilt was 48° whilst (b) failed slowly within

24 hrs at 32°; there was a further observation of interest: in connection with (a) cracks appeared in the clay at a tilt of 34° which were oriented at 21° to the bedding plane of the clay. These are illustrated in Figure 6.18 and will be shown to be interpreted as Coulomb ruptures on maximum obliquity planes with $\phi' = 42°$ when there is undrained sliding on a no extension direction parallel with the angle of tilt, Section 4.6.

6.5.1.6 Overconsolidated Kaolin

Figure 6.19(a) shows the results of tilting tests on Kaolin which have been resedimented and overconsolidated together with those of two load controlled direct shear tests on overconsolidated samples. A wide range of tilting rates was covered; no discrepancies, due to the tilting rate, suggesting undrained conditions are seen. A linear relationship defines the lower limit of shear strength and approaches the limit of normal consolidation with a failure tilt angle of 38° (MSO), the shear strength of normally consolidated Kaolin with the thixotropic component included. The limited data included some from DSA samples which were not normally consolidated beyond the stress level for which there is a thixotropic component; although the stress level is two orders of magnitude higher and the voids ratios are significantly smaller (e = 2.4) there is excellent agreement in the test results. Very limited data for samples normally consolidated to higher levels suggest that this leads to a quantitatively corresponding drop in the overconsolidation strength. The large scatter shown in Figure 6.19(a) is due to the existence of several voids ratios in equilibrium at the same effective stress and overconsolidation ratio. If it is assumed that the lower limit line shown represents a correct relationship between voids ratio and overconsolidation ratio the other tilting test results can be corrected by adjusting the overconsolidation ratio accordingly (Appendix B). This has been done in Figure 6.19(b); however the improvement is perhaps misleading in that the variation in voids ratio which is independent of effective stress appears to be real at these very low stress levels. A similar type of variation has been reported by Been and Sills (1981), who attributed it to different densities of initial suspension prior to sedimentation. In this case variations in setting up the downward seepage to cause consolidation seems a likely cause (Section 7.4.1.2). The correction clearly has limited validity and will not be used for further analysis. On the other hand relationships between shear strength and voids ratio for overconsolidated tilted samples seems established for this particular swell back vertical stress (Figure 6.20). Figure 6.21 shows the final voids ratios of all samples plotted against the maximum vertical stress. No significant swelling was observed due to stress relief.

Figure 6.22 illustrates the effect of further consolidation and swell back periods on the strength of overconsolidated Kaolin at these very low stresses. It was found that a 2 hour further consolidation period gave much lower results than the standard further consolidation period used (24 hours) indicating that the samples had not reached equilibrium. However a swell back period of two hours produced similar results to those obtained in the standard method used (24 hours) indicating the limitations at this low stress level (0.05kPa).

6.5.1.7 Overconsolidated London Clay

Figures 6.23 and 6.24 also shows overconsolidated London clay tilting test results. The general form is the same as for Kaolin but the linear lower limit differs. The approach to the normal consolidation limit again coincides with that for normally consolidated shear strength when the thixotropic component is included at 32° (NED). The limited data included some from DSA samples which were not normally consolidated beyond the stress level for which there is a thixotropic component; once again although the stress level is two orders of magnitude higher and the voids ratios are significantly smaller ($e = 2.6$) there is excellent agreement in the test results. Very limited data for samples normally consolidated to higher levels suggest that this leads to a quantitatively corresponding drop in the overconsolidation strength. Figure 6.25 shows the final voids ratios of all samples plotted against the maximum vertical stress, no significant swelling was observed due to stress relief. Figure 6.26 shows the corrected overconsolidation ratios (as done for Kaolin, Section 6.4.1.6) versus the angle failure. Again the correction clearly may haves limited validity and will not be used for further analysis.

6.5.1.8 Overconsolidated Kaolin and Coagulant

Figures 6.27 and 6.28 shows the tilting failure angles for Kaolin with the addition of coagulant when overconsolidated. The lower limit line again intersects the normally consolidated limit at the tilting angle found for shear strength including the thixotropic component. Figure 6.29 shows the final voids ratios of all samples plotted against the maximum vertical stress, no significant swelling was observed due to stress relief. Figure 6.30 shows the corrected overconsolidation ratios (as done for Kaolin, Section 6.4.1.6) versus the angle failure. Again the correction clearly may have limited validity and will not be used for further analysis.

6.5.2 The Shear Cylinder

Normally consolidated clays and dry dense Leighton Buzzard sand were tested in the shear cylinder at the continuous rate of 4°/min. and incremental rate of 3°/day. The preparation of overconsolidated samples was not feasible with the present apparatus and thus no tests were performed on overconsolidated samples. Three cylinders of 80mm, 100mm and 150mm diameters were used with Kaolin, and it was found that the cylinder size had no effect on the results over this range of diameters. Tables 6.9 and 6.10 presents the results of these tests and Plate 6.3.

It will be shown in Chapter 7 that all failures occurred on a no extension direction with coincident stress and strain increments. In addition, the values of the effective angles of friction (ϕ') were the same as those obtained in the standard tilting tanks for both clays under the same test conditions.

6.5.3 The Tension Test

Three overconsolidated Kaolin samples were tested in this apparatus. The angles of tilts at failure obtained for all tests were 1°, and therefore very small tensile strengths were recorded (0.0378kPa) which made the interpretations somewhat difficult. The three tests were performed on samples consolidated to a maximum vertical stress of 5kpa and tested at a vertical stress equal to self weight of the sample (OCR $\rightarrow \infty$). This procedure of overconsolidation did not allow specific overconsolidation ratios to be achieved and therefore a relationship could not be established. At the very high OCR, swell back may have broken particle bonds.

CLAY	LOAD/DISP. CONTROL	N.C. or O.C.	σ_v' (kPa)	WATER COMPOSITION	BOX SHAPE SQ. or CIR.	N (days)
KAOLIN	DISP. CONTROL	N.C.	5, 15, 20, 50	DISTILLED, LTW.	SQ. & CIR.	1, 2, 4, 10
		O.C.	2.5, 5, 10	LTW	SQ. & CIR.	1
	LOAD CONTROL	N.C.	5, 8, 10, 50	LTW & SALT	SQ. & CIR.	>1
		O.C.	2.5, 5	LTW	SQ. & CIR.	>1
LONDON	DISP. CONTROL	N.C.	5, 10	LTW	SQ. & CIR.	1, 7, 9
		O.C.	5, 10	LTW	SQ. & CIR.	1
	LOAD CONTROL	N.C.	5, 8, 10, 20	LTW	SQ. & CIR.	>1
		O.C.	5	LTW	SQ.	>1

LTW = London tap water

N = Period of consolidation prior to testing.

TABLE 6.1 VARIATIONS OF TESTING CONDITIONS IN THE DIRECT SHEAR APPARATUS

TEST No.	σ_{v0}'	DISP. RATE mm/min	CIRCULAR or SQUARE	WATER COMP.	WATER CONTENT (%)	N (days)	STRESS RATIO (τ/σ_v')	MSO or NED	ϕ' (°)
1	5	2	CIRCULAR	LTW	89	1	0.40	MSO	22.0
2	5	2	CIRCULAR	LTW	88	1	0.41	MSO	22.5
3	5	2	SQUARE	LTW	89	1	0.41	MSO	22.5
4	5	2	CIRCULAR	LTW	88	1	0.40	MSO	22.0
5	5	2	SQUARE	LTW	90	4	0.41	MSO	22.5
6	5	2	CIRCULAR	LTW	88	1	0.40	MSO	22.0
7	5	2	SQUARE	D	88	1	0.41	MSO	22.0
8	15	2	SQUARE	D	71	1	0.41	MSO	22.2
9	15	2	CIRCULAR	LTW	72	10	0.41	MSO	22.1
10	15	2	SQUARE	LTW	71	1	0.41	MSO	22.5
11	20	2	SQUARE	LTW	68	8	0.41	MSO	22.5
12	20	2	CIRCULAR	LTW	69	1	0.41	MSO	22.5
13	20	2	CIRCULAR	D	68	2	0.41	MSO	22.5
14	20	2	SQUARE	LTW	68	1	0.41	MSO	22.5
15	20	2	CIRCULAR	LTW	67	1	0.42	MSO	23.0
16	20	2	SQUARE	LTW	69	10	0.40	MSO	21.8
17	20	2	CIRCULAR	LTW	67	1	0.41	MSO	22.5
18	20	2	SQUARE	LTW	67	1	0.40	MSO	22.0
19	50	2	CIRCULAR	D	63	1	0.40	MSO	22.0
20	50	2	CIRCULAR	LTW	61	1	0.42	MSO	22.8
21	50	2	SQUARE	LTW	61	1	0.42	MSO	23.0
22	50	2	CIRCULAR	LTW	62	1	0.42	MSO	23.0
23	50	2	SQUARE	LTW	62	1	0.41	MSO	22.5
24	5	0.1	CIRCULAR	LTW	89	1	0.39	MSO/NED	22.5*
25	5	0.1	CIRCULAR	LTW	88	1	0.39	MSO/NED	22.5*
26	5	0.1	SQUARE	LTW	87	10	0.39	MSO/NED	22.5*
27	20	0.01	CIRCULAR	LTW	68	1	0.38	NED	22.3
28	20	0.01	SQUARE	D	68	1	0.37	NED	22.0
29	20	0.01	CIRCULAR	LTW	69	1	0.38	NED	22.6
30	20	0.01	SQUARE	LTW	68	1	0.38	NED	22.2
31	20	0.01	CIRCULAR	LTW	67	1	0.36	NED	21.8
32	20	0.01	CIRCULAR	LTW	68	1	0.36	NED	21.8

MSO=Maximum Stress Obliquity, Coulomb. NED=No Extension Direction with coincident axes of stress and strain increment.

LTW=London tap water. D=Distilled water. S=London Tap water with 35g/litre of Salt.

N = Period of consolidation prior to testing. * = Expected ϕ'

TABLE 6.2 DISPLACEMENT CONTROL DIRECT SHEAR RESULTS FOR NORMALLY CONSOLIDATED KAOLIN

TEST No.	σ_{v0}' (kPa)	DISP. RATE mm/min	CIRCULAR or SQUARE	WATER COMP	WATER CONTENT (%)	(N) (days)	STRESS RATIO (τ/σ_v')	MSO or NED	ϕ' (°)
33	5	2	SQUARE	LTW	101	1	0.36	MSO	20
34	5	2	SQUARE	LTW	100	1	0.37	MSO	20.5
35	5	2	CIRCULAR	LTW	90	7	0.55	MSO	29
36	5	2	SQUARE	LTW	91	9	0.55	MSO	29
37	5	0.01	SQUARE	LTW	99	7	0.34	NED	29
38	10	0.01	CIRCULAR	LTW	66	7	0.49	NED	29

MSO = Maximum Stress Obliquity, Coulomb.
LTW = London tap water.
N = Period of consolidation prior to testing.
NED = No Extension Direction with coincident axes of stress and strain increment.

TABLE 6.3 DISPLACEMENT CONTROL DIRECT SHEAR RESULTS FOR NORMALLY CONSOLIDATED LONDON CLAY

TEST No.	KAOLIN or LONDON	σ_{vo}'	OVERCONS-OLIDATION RATIO	DISP. RATE mm/min	CIRCULAR or SQUARE	WATER COMP.	WATER CONTENT (%)	N (days)	STRESS RATIO (τ/σ_v')	MSO or NED	ϕ' # (°)
39	KAOLIN	5	10	2	SQUARE	LTW	59	1	0.45	MSO	24.2*
40	KAOLIN	10	5	2	CIRCULAR	LTW	61	1	0.61	MSO	31.4*
41	KAOLIN	2.5	2	0.01	CIRCULAR	LTW	88	1	0.64	NED	39.0
42	KAOLIN	2.5	2	0.01	SQUARE	LTW	89	1	0.63	NED	38.7
43	LONDON	5	10	2	CIRCULAR	LTW	59	1	1.28	MSO	52.0*
44	LONDON	5	10	2	SQUARE	LTW	58	1	0.76	MSO	37.2*
45	LONDON	5	10	2	CIRCULAR	LTW	58	1	1.57	MSO	57.5*
46	LONDON	10	5	2	SQUARE	LTW	61	1	1.13	MSO	48.5*

MSO = Maximum Stress Obliquity, Coulomb. NED = No Extension Direction with coincident axes of stress and strain increment.
LTW = London tap water * Negative pore water pressure not quantified N = Period of consolidation prior to testing
# Apparent ϕ'

TABLE 6.4 DISPLACEMENT CONTROL DIRECT SHEAR RESULTS FOR OVERCONSOLIDATED KAOLIN AND LONDON CLA

TEST No.	KAOLIN or LONDON	σ_{vo}'	OVERCONS-OLIDATION RATIO	CIRCULAR or SQUARE	WATER COMP	WATER CONTENT (%)	STRESS RATIO (τ/σ_v')	MSO or NED	ϕ' (°)
47	KAOLIN	2.5	2	SQUARE	LTW	90	0.90	MSO	41.0
48	KAOLIN	2.5	2	SQUARE	LTW	90	0.90	MSO	40.0
49	KAOLIN	5	1	CIRCULAR	S	88	0.63	NED	38.7
50	KAOLIN	5	1	CIRCULAR	S	90	0.63	NED	38.7
51	KAOLIN	5	1	CIRCULAR	LTW	89	0.75	MSO	37.8
52	KAOLIN	5	1	CIRCULAR	LTW	89	0.75	MSO	37.0
53	KAOLIN	5	1	CIRCULAR	LTW	89	0.78	MSO	38.0
54	KAOLIN	5	1	CIRCULAR	LTW	90	0.75	MSO	36.6
55	KAOLIN	5	2	CIRCULAR	LTW	88	0.47	MSO	25.2
56	KAOLIN	8	1	CIRCULAR	LTW	86	0.41	MSO	22.5
57	KAOLIN	10	1	CIRCULAR	LTW	80	0.41	MSO	22.5
58	KAOLIN	50	1	CIRCULAR	LTW	62	0.41	MSO	22.5
59	LONDON	5	1	CIRCULAR	LTW	100	0.77	MSO	37.5
60	LONDON	5	1	CIRCULAR	LTW	100	0.82	MSO	39.5
61	LONDON	5	1	SQUARE	LTW	101	0.63	NED	38.7
62	LONDON	5	1	SQUARE	LTW	100	0.63	NED	38.7
63	LONDON	5.5	1	SQUARE	LTW	100	0.63	NED	38.7
64	LONDON	5	2	SQUARE	LTW	99	0.71	NED	45.0
65	LONDON	5	2	SQUARE	LTW	100	0.73	NED	46.6
66	LONDON	8	1	CIRCULAR	LTW	85	0.74	MSO	36.5
67	LONDON	10	1	SQUARE	LTW	80	0.57	NED	35.0
68	LONDON	20	1	CIRCULAR	LTW	70	0.53	NED	32.0

MSO = Maximum Stress Obliquity, Coulomb.

NED = No Extension Direction with coincident axes of stress and strain increment.

LTW = London tap water.

D = Distilled water.

S = London Tap water with 35g/litre of Salt.

TABLE 6.5 LOAD CONTROL DIRECT SHEAR RESULTS FOR KAOLIN AND LONDON CLAY

CLAY	N.C. or O.C.	TILTING RATE	σ_v' (kPa)	WATER COMPOSITION DIST., LTW, SALT or COAG.	No. OF SED./CONS. DAYS	FURTHER CONS. PERIOD (HOURS)	SWELL-BACK PERIOD (HOURS)
KAOLIN	N.C.	3°/DAY, 4°/MIN. ≈15°/SEC.	0.05, 0.1	DIST, LTW, SALT, COAG.	1, 2-30	-	-
	O.C.	3°/DAY*, 1°/HR* 5°/HR.*	0.05	LTW	1	2, 24	2, 24
LONDON	N.C.	3°/DAY, 4°/MIN. ≈15°/SEC.	0.05, 0.1	LTW, SALT	1, 2, 3, 4, 5, 6-20	-	-
	O.C.	1°/HR*, 5°/HR.*	0.05	LTW	1	24	24

* Taken to 35° at 4°/min. And then at the specific rate.

TABLE 6.6 VARIATIONS OF TESTING CONDITIONS IN THE STANDARD TILTING TANKS

TILTING TANKS (σ_v' = 0.05 and 0.1 kPa)										
TILTING RATE		3°/DAY	4°/MINUTE						VERY FAST ~ 15°/SEC.	
SPESWHITE KAOLIN	No. OF TESTS	16	14			10			8	10
	DAYS SINCE SED. BEGAN	9	1			2 to 30			1	2 to 15
	FAILURE DESCR.	DISINT.	DISINTEGRATION			DISINTEGRATION			DISINT.	DISINT.
	τ/σ_v'	0.78	0.41			0.41			0.68	0.78
	PROPOSED FAILURE	MSO	MSO			MSO			MSO	MSO
	ϕ' (degrees)	38	22.5			22.5			34	38
	VOIDS RATIO	6.1	6.1			6.1			6.1	6.1
LONDON CLAY	No. OF TESTS	4	5	3	3	3	3	3	4	6
	DAYS SINCE SED. BEGAN	7	1	2	3	4	5	≥6	1	>1
	FAILURE DESCR.	S.B.	DIS.	SLIDING BLOCK					DISINTEGRATION	
	τ/σ_v'	0.63	0.37	0.45	0.49	0.53	0.58	0.63	0.78	>0.78*
	PROPOSED FAILURE	NED	MSO	NED	NED	NED	NED	NED	MSO	MSO
	ϕ' (degrees)	38.7	20.5	26.7	29.3	32	35.5	38.7	38	>38*
	VOIDS RATIO	3.2	6.1	5.6	4.9	4.1	3.4	3.2	6.1	<6.1

MSO=Maximum Stress Obliquity, Coulomb. NED=No Extension Direction with coincident axes of stress and strain increment. *Negative pore pressure inferred.

TABLE 6.7 TILTING TANKS RESULTS FOR NORMAL CONSOLIDATION.

TILTING TANKS (σ'_v=0.05kPa)						
		3°/DAY	4°/MINUTE		VERY FAST = 15°/SEC.	
KAOLIN & SALT	No. OF TESTS	2	2	2	2	2
	DAYS SINCE SED. BEGAN	13	1	2 to 30	1	2 to 15
	FAILURE DESCRIPTION	SLIDING.B.	DISINTEGRATION	DISINTEGRATION	DISINTEGRATION	
	τ/σ'_v	0.63	0.41	0.41	0.78	>0.78*
	PROPOSED FAILURE	NED	MSO	MSO	MSO	MSO
	ϕ' (degrees)	38.7	22.5	22.5	38	>38*
	VOIDS RATIO	6.1	6.1	6.1	6.1	6.1
LONDON CLAY & SALT	No. OF TESTS	2	2	3	2	3
	DAYS SINCE SED. BEGAN	13	1	2 to 4	1	>1
	FAILURE DESCRIPTION	DISINTEG.	SLIDING BLOCK	DISINTEGRATION	DISINTEGRATION	
	τ/σ'_v	0.78	0.63	0.78	0.78	>0.78*
	PROPOSED FAILURE	MSO	NED	MSO	MSO	MSO
	ϕ'(degrees)	38	38.7	38	38	>38*
	VOIDS RATIO	3.2	3.2	3.2	3.2	3.2
KAOLIN & COAGULANT	No. OF TESTS	7	5	4	4	4
	DAYS SINCE SED. BEGAN	13	1	2 to 6	1	>1
	FAILURE DESCRIPTION	DISINTEG.	DISINTEGRATES	DISINTEGRATES	DISINTEGRATES	
	τ/σ'_v	1.1	0.83	0.83	>1.11*	>1.11*
	PROPOSED FAILURE	MSO	MSO	MSO	MSO	MSO
	ϕ'(degrees)	48	39 to 40	39 to 40	>48*	>48*
	VOIDS RATIO	8.9	8.9	8.9	8.9	8.9

MSO=Maximum Stress Obliquity, Coulomb. NED=No Extension Direction with coincident axes of stress and strain increment.

TABLE 6.8 TILTING TANKS RESULTS FOR NORMAL CONSOLIDATION.

	TILTING RATE	4°/MINUTE						3°/day
SPESWHITE KAOLIN	No. OF TESTS	8			6			3
	DAYS SINCE SED./CONS. BEGAN	1			2 to 30			8
	τ/σ'_v	0.38			0.38			0.63
	PROPOSED FAILURE	NED			NED			NED
	ϕ' (degrees)	22.5			22.5			38.7
	VOIDS RATIO	6.1			6.1			6.1
LONDON CLAY	No. OF TESTS	3	2	2	2	2	2	-
	DAYS SINCE SED./CONS. BEGAN	1	2	3	4	5	6	-
	τ/σ'_v	0.35	0.45	0.49	0.53	0.58	0.63	-
	PROPOSED FAILURE	NED	NED	NED	NED	NED	NED	-
	ϕ' (degrees)	19.5	26.7	29.3	32	35.5	38.7	-
	VOIDS RATIO	6.1	5.6	4.9	4.1	3.4	3.2	-
LEIGHTON BUZZARD SAND	No. OF TESTS	10			5			-
	SAMPLE HEIGHT	20mm			10mm			-
	τ/σ_v'	0.89			0.89			-
	ϕ'	48°			48°			-
	PROPOSED FAILURE	NED			NED			-
	VOIDS RATIO	0.52			0.52			-

NED=No Extension Direction with coincident axes of stress and strain increment.

TABLE 6.9 SHEAR CYLINDER RESULTS FOR NORMAL CONSOLIDATION.

TILTING RATE		4°/MINUTE					
KAOLIN & SALT	No. OF TESTS	2			2		
	DAYS SINCE SED./CONS. BEGAN	1			2 to 6		
	τ/σ'_v	0.38			0.38		
	PROPOSED FAILURE	NED			NED		
	ϕ' (degrees)	22.5			22.5		
	VOIDS RATIO	6.1			6.1		
LONDON CLAY & SALT	No. OF TESTS	2	1	1	1	1	2
	DAYS SINCE SED./CONS. BEGAN	1	2	3	4	5	6
	τ/σ'_v	0.63	0.63	0.63	0.63	0.63	0.63
	PROPOSED FAILURE	NED	NED	NED	NED	NED	NED
	ϕ' (degrees)	38.7	38.7	38.7	38.7	38.7	38.7
	VOIDS RATIO	3.2	3.2	3.2	3.2	3.2	3.2

NED=No Extension Direction with coincident axes of stress and strain increment.

TABLE 6.10 SHEAR CYLINDER RESULTS FOR NORMAL CONSOLIDATION.

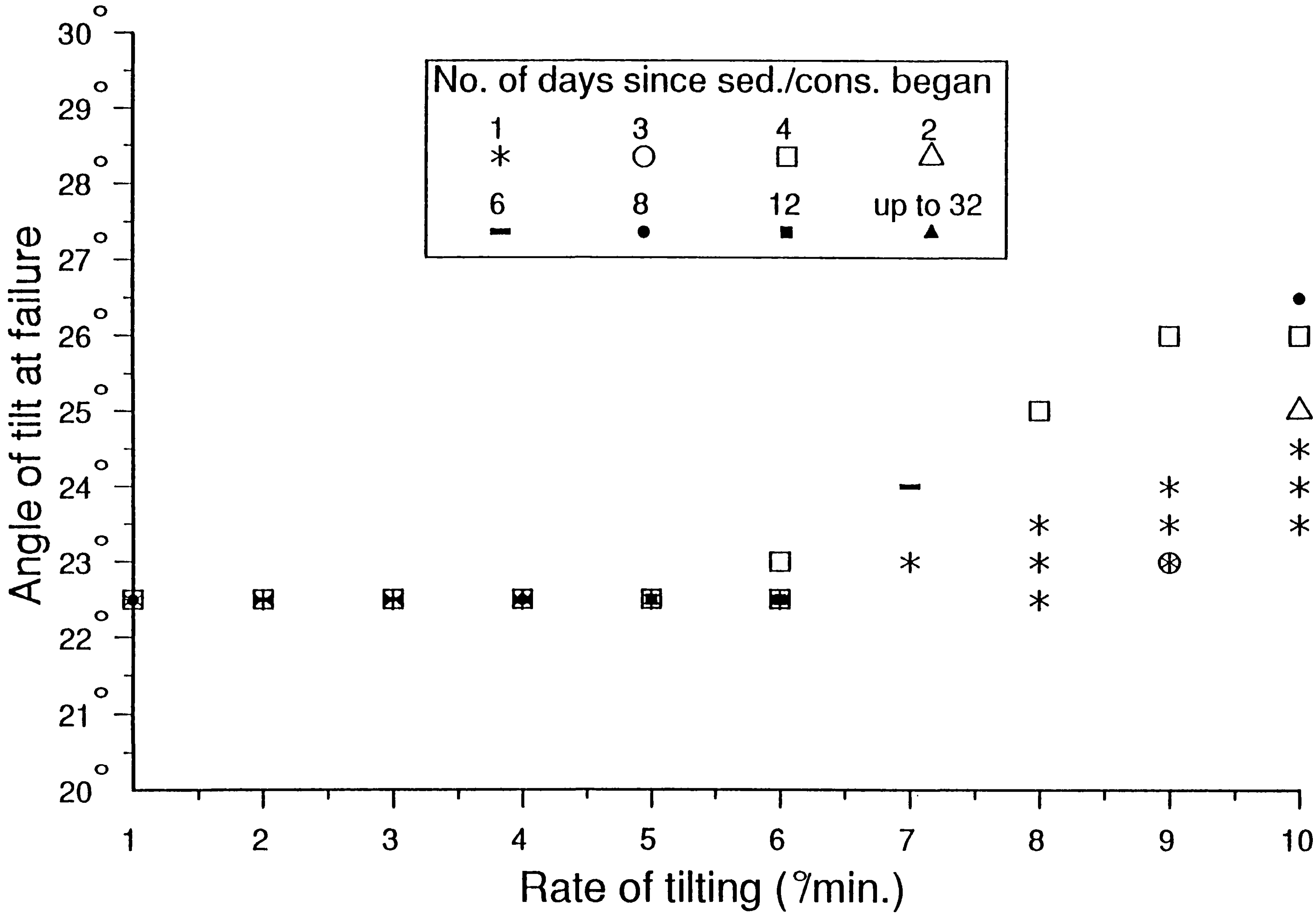


Figure 6.1 Variation of angle of tilt at failure with rate of tilt for Kaolin

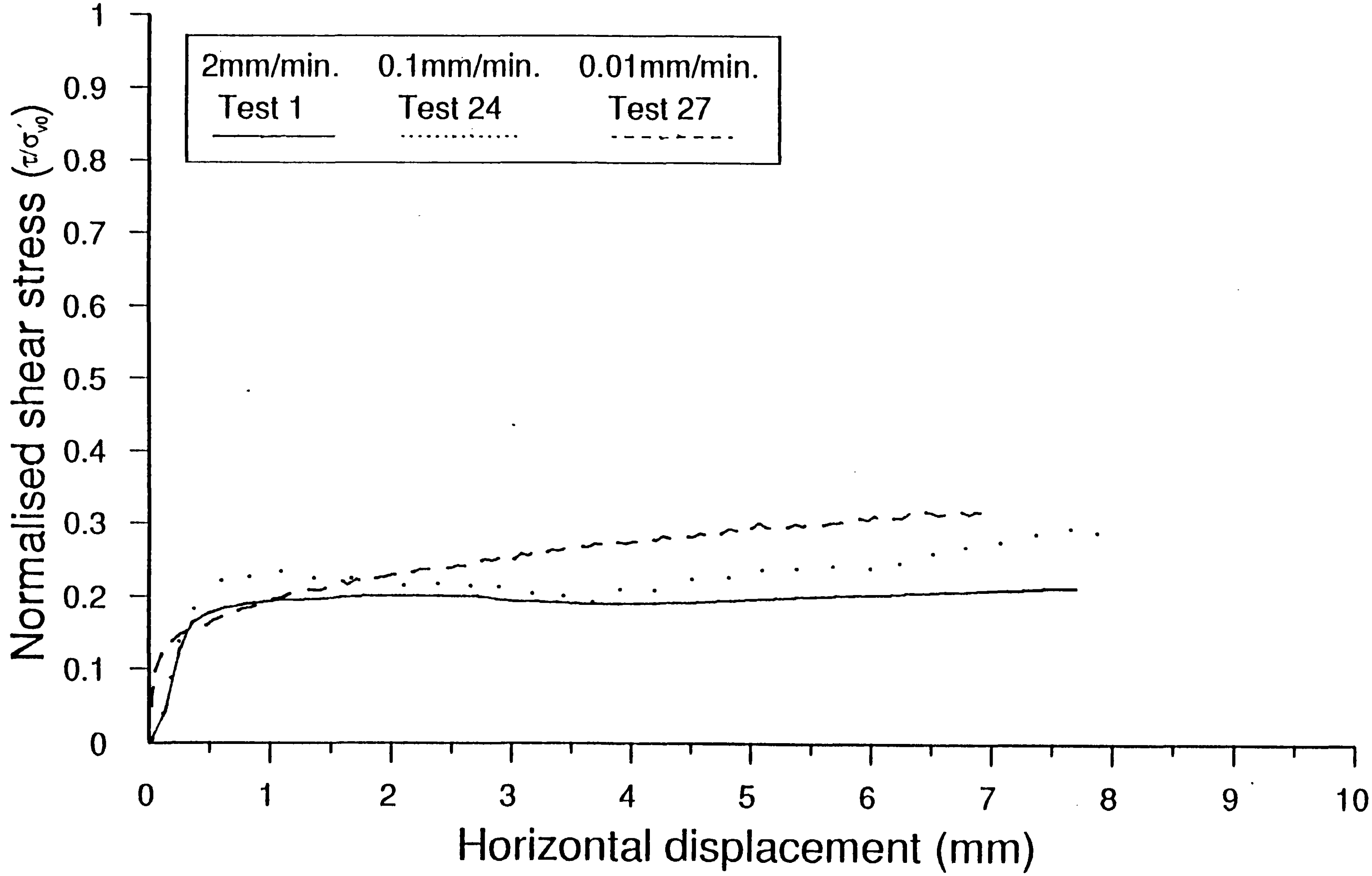


Figure 6.2 Typical results for normally consolidated Kaolin in the direct shear apparatus

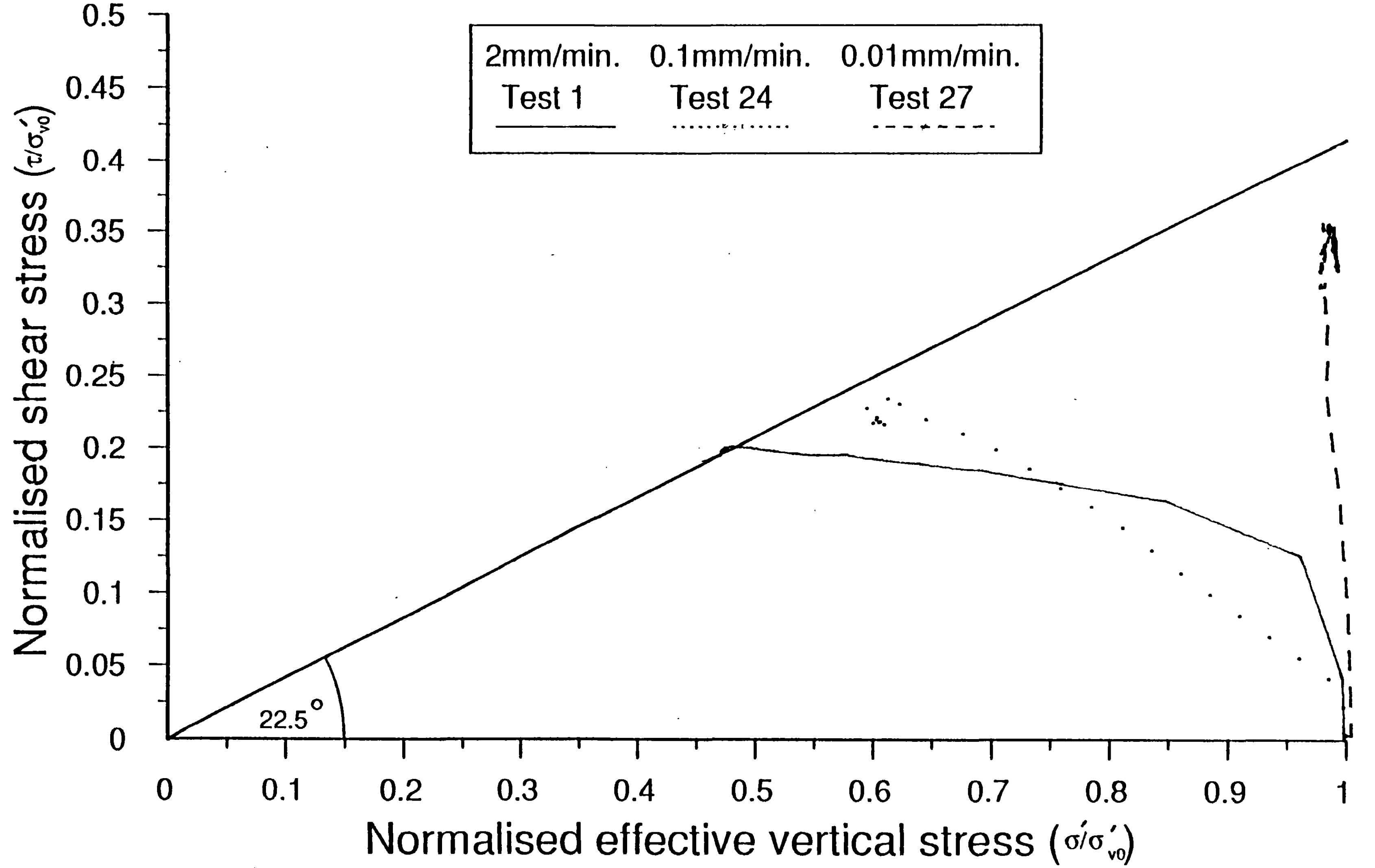


Figure 6.3(a) Typical Results for normally consolidated Kaolin in the direct shear apparatus

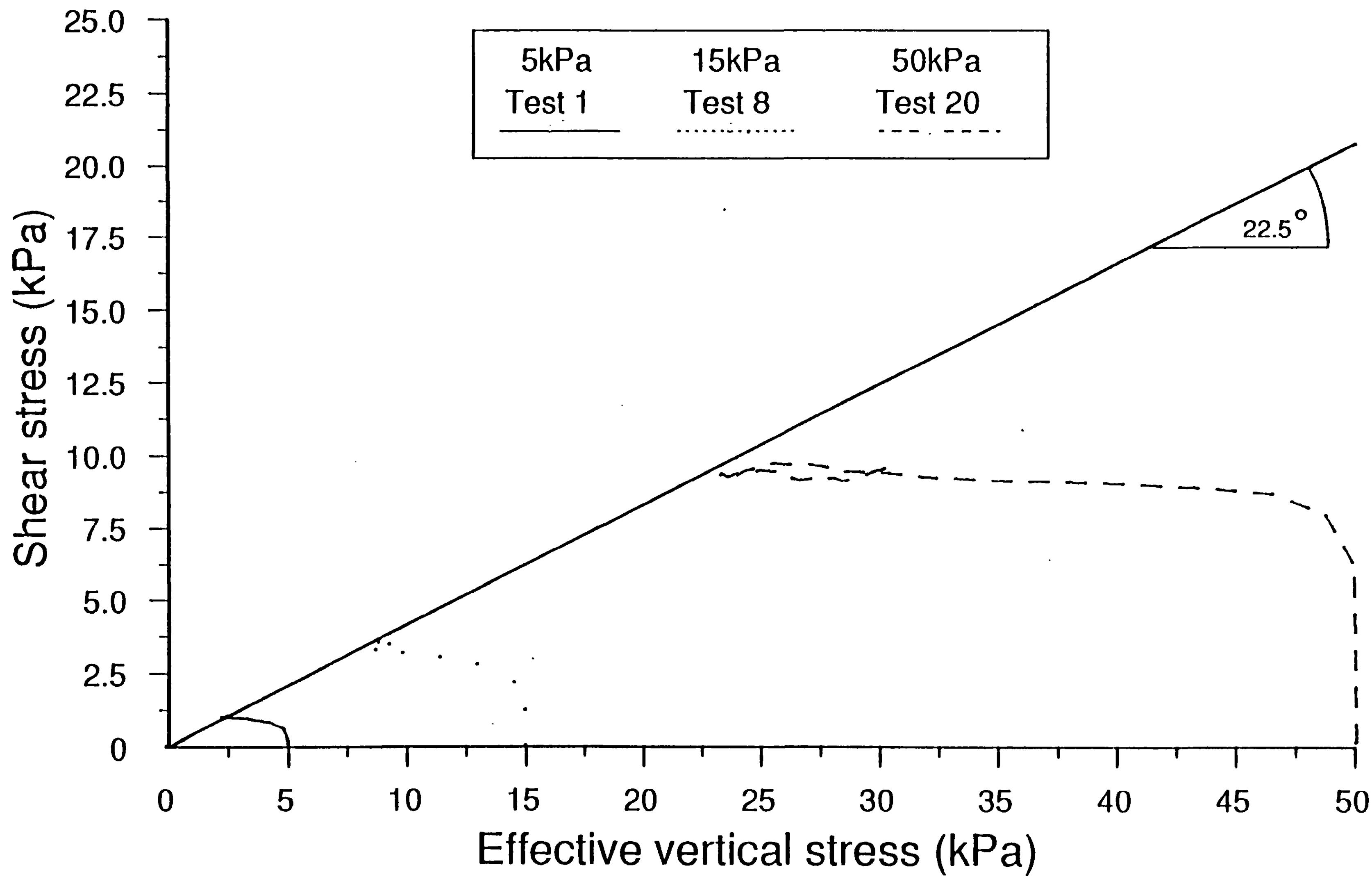


Figure 6.3(b) Typical Results for normally consolidated Kaolin in the direct shear apparatus at different stress levels

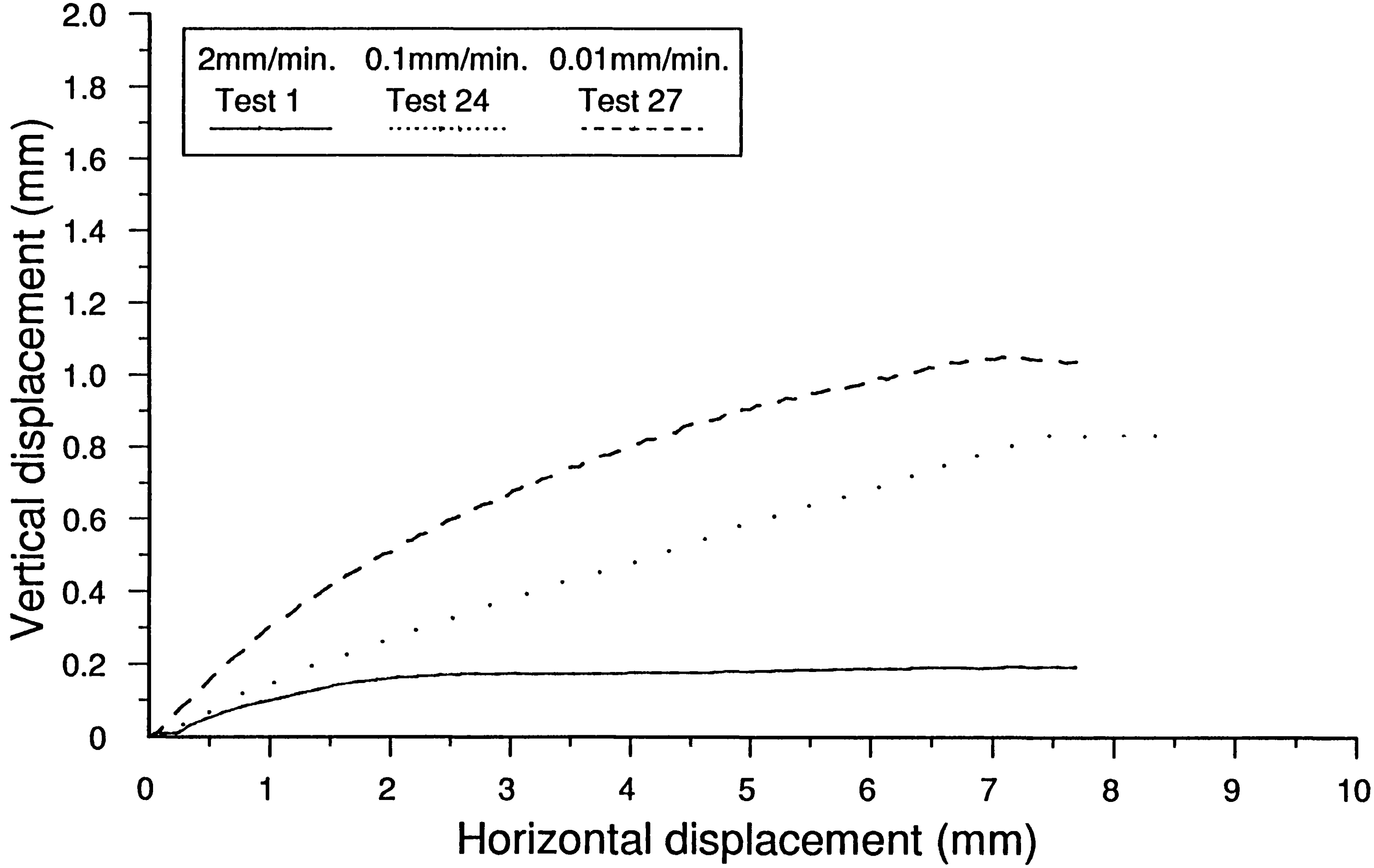


Figure 6.4 Typical results for normally consolidated Kaolin in the direct shear apparatus

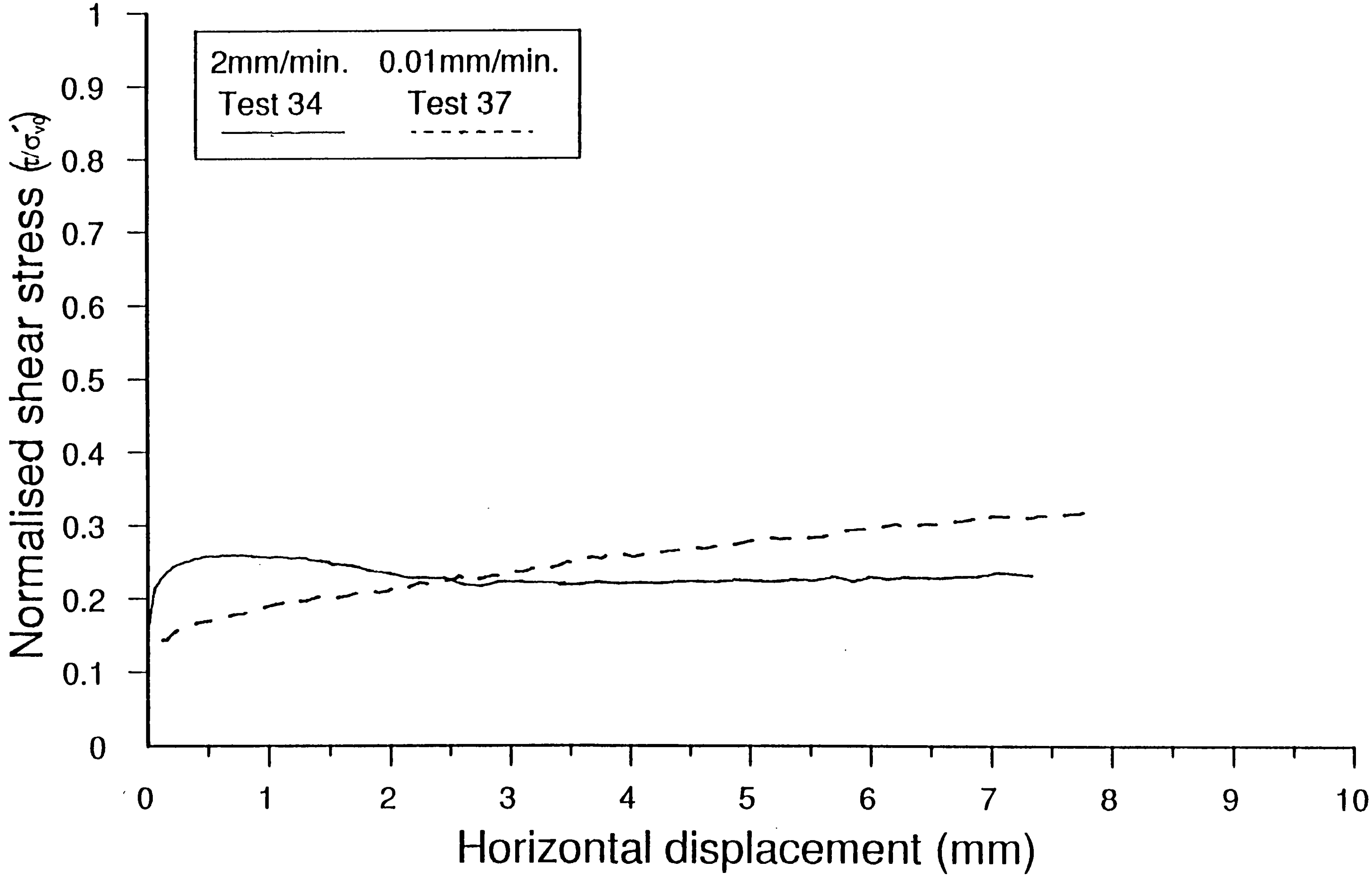


Figure 6.5 Typical results for normaly consolidated London clay in the direct shear apparatus

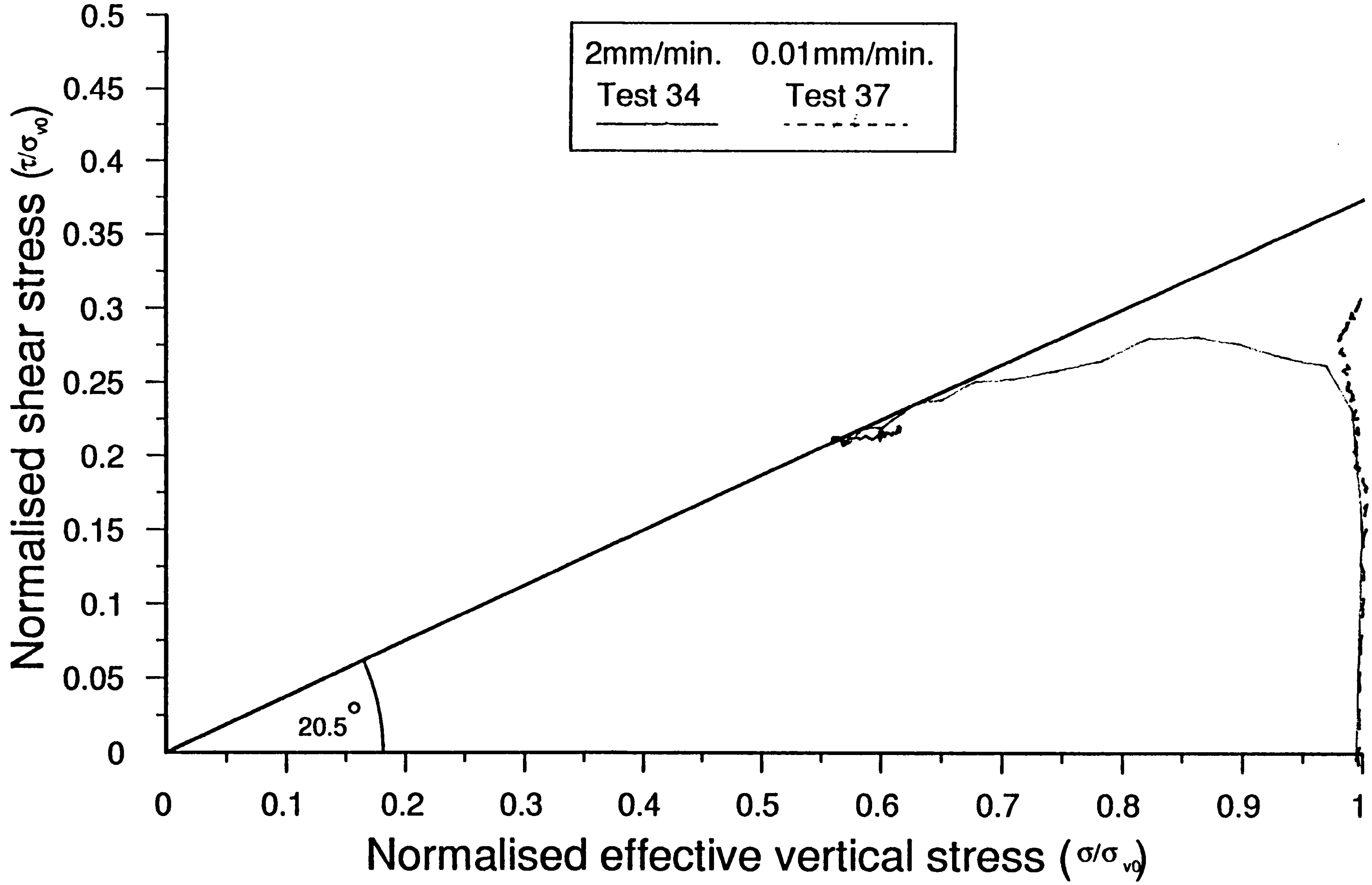


Figure 6.6 Typical Results for normally consolidated London clay in the direct shear apparatus

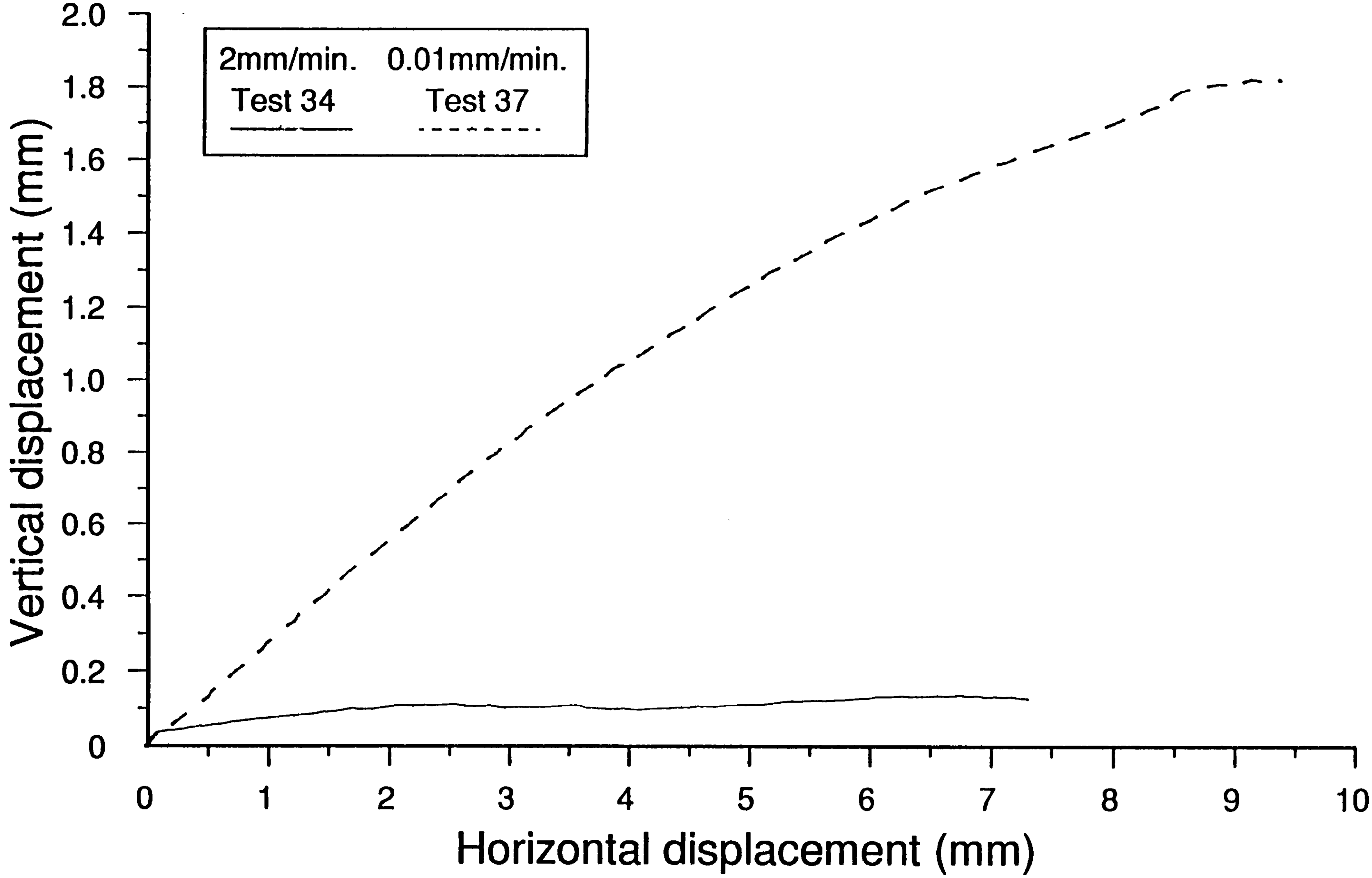


Figure 6.7 Typical results for normally consolidated London clay in the direct shear apparatus

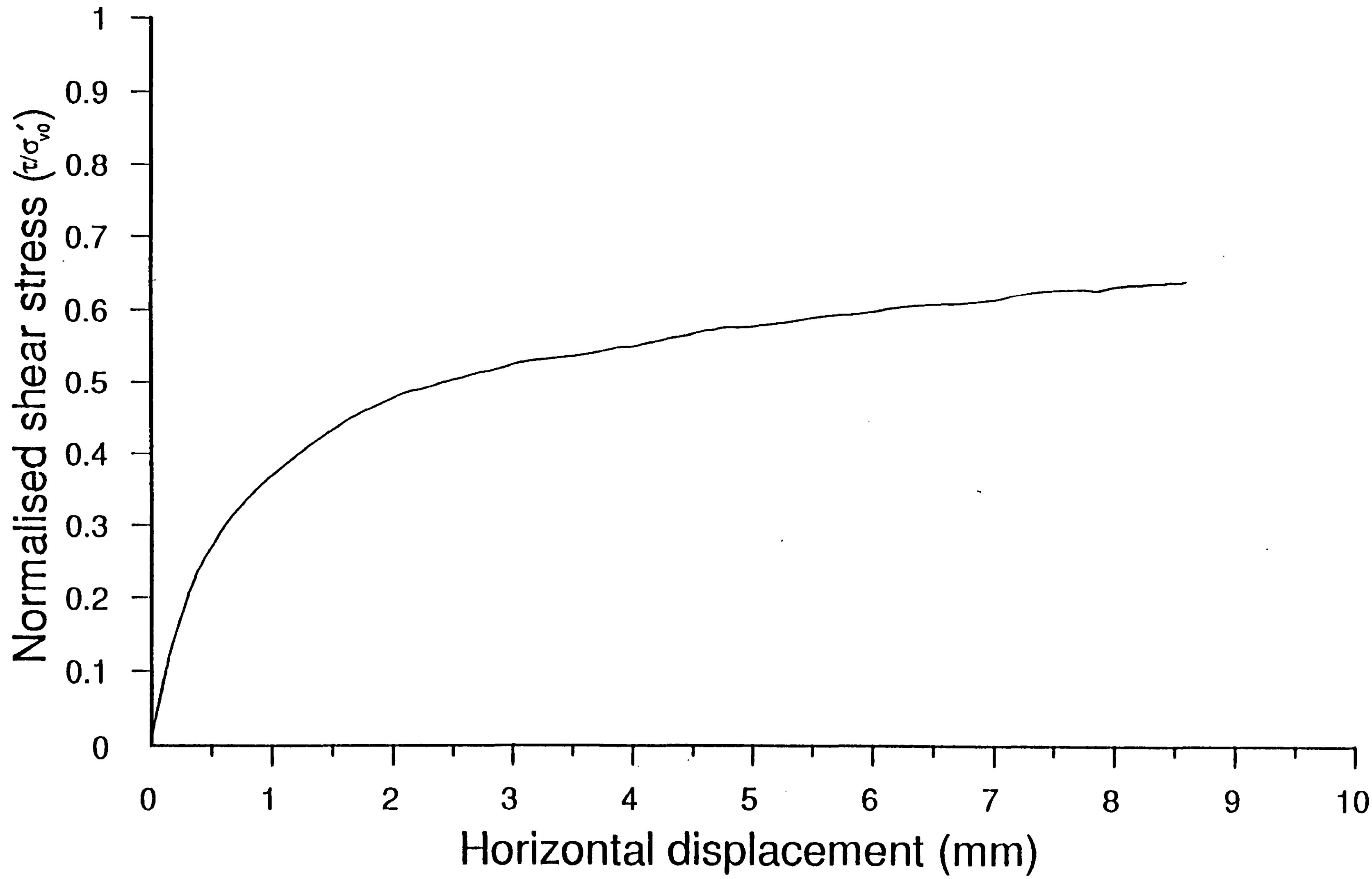


Figure 6.8 Results for test 41 for overconsolidated Kaolin in the displacement control direct shear apparatus

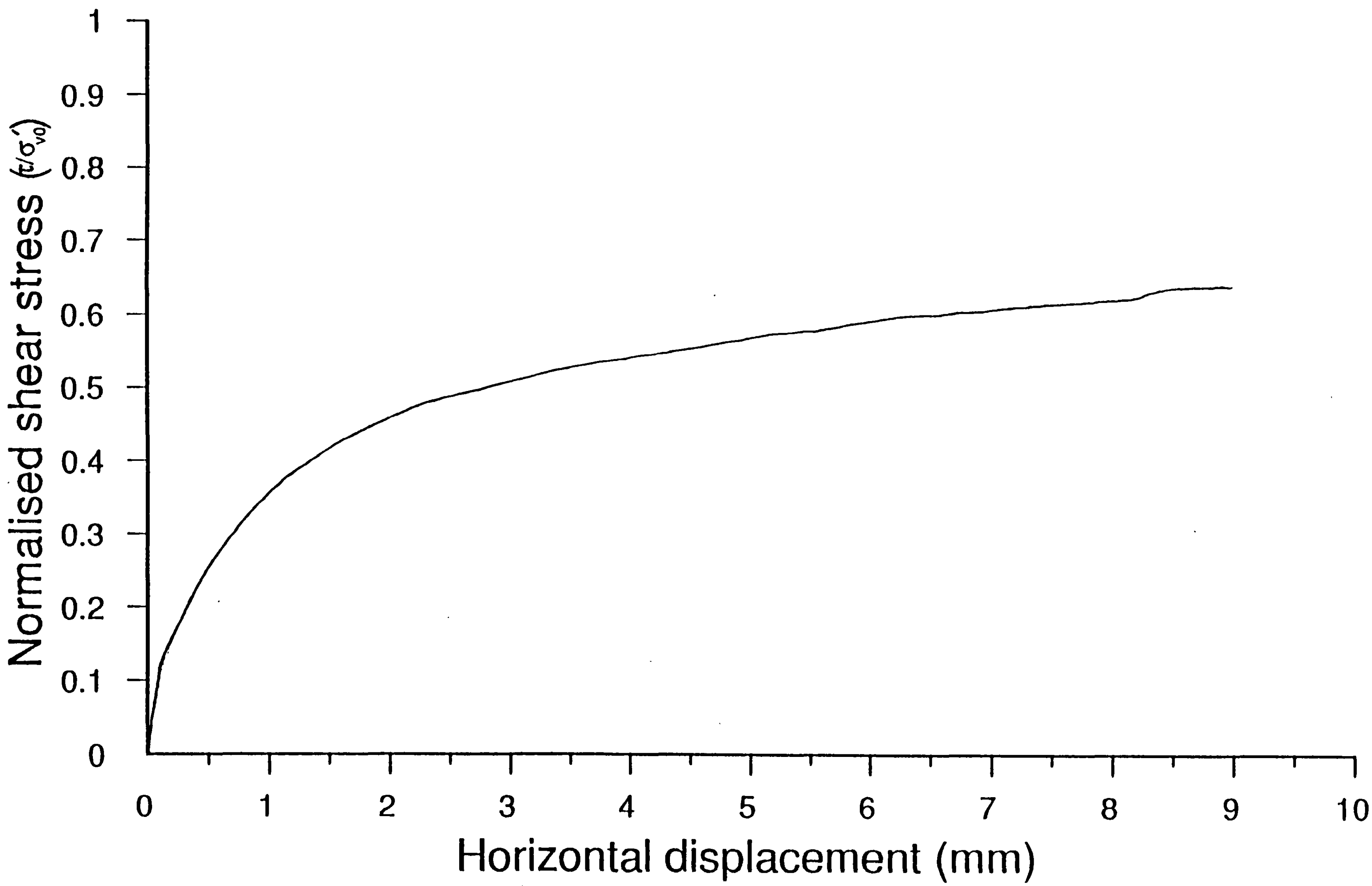


Figure 6.9 Results for test 42 for overconsolidated Kaolin in the displacement control direct shear apparatus

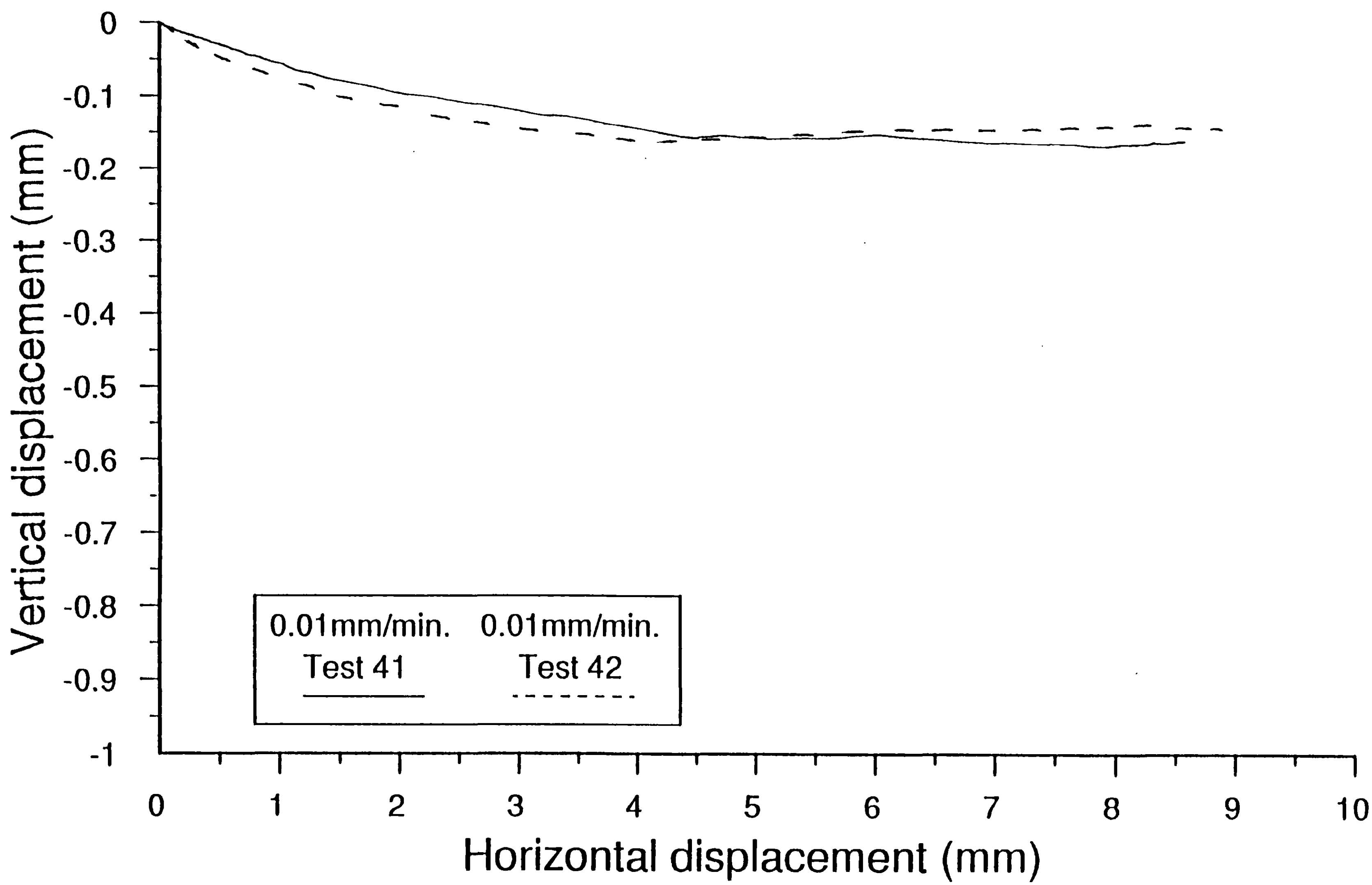


Figure 6.10 Results for tests 41 and 42 for overconsolidated Kaolin in the direct shear apparatus

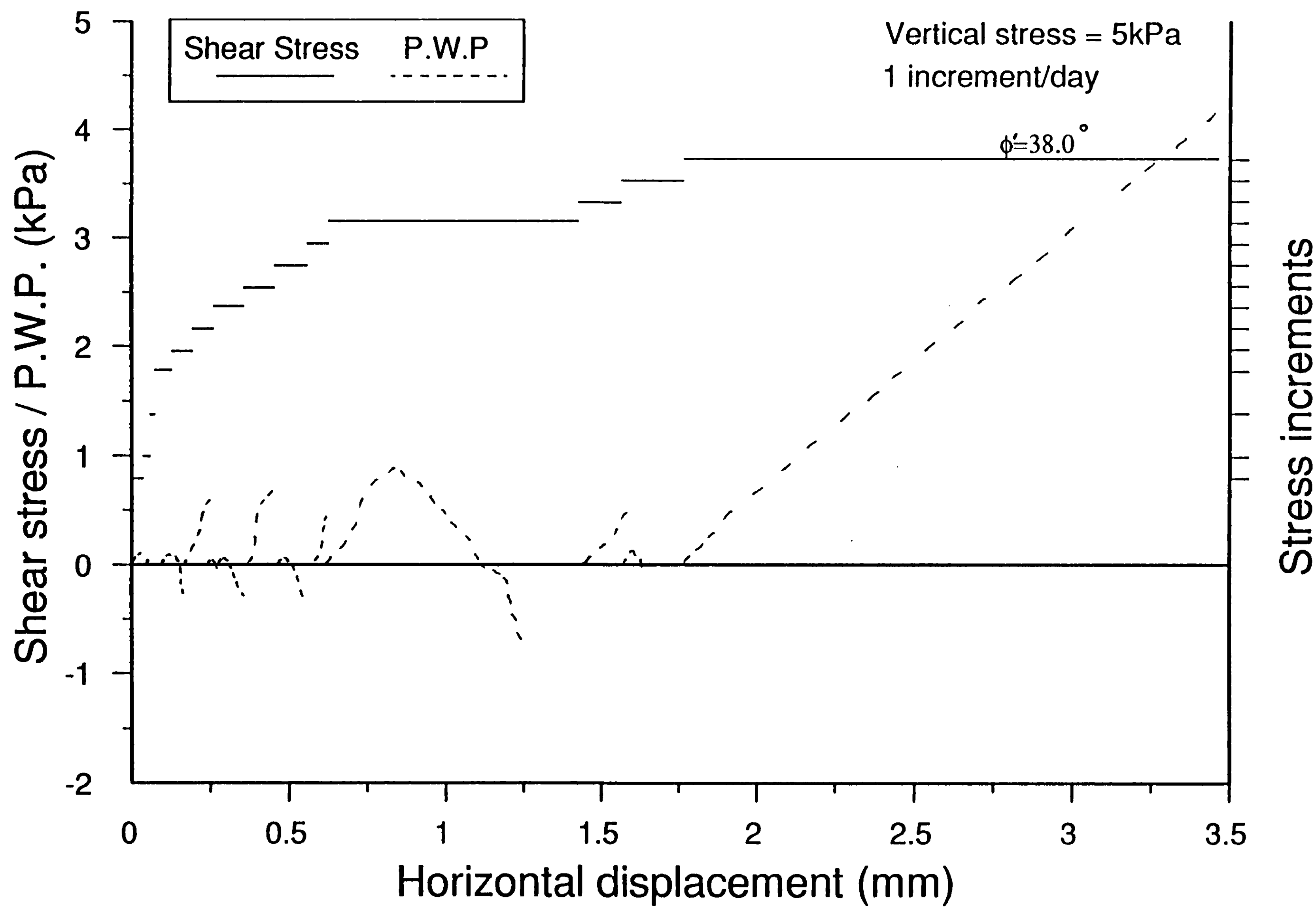


Figure 6.11(a) Load control direct shear results for Kaolin

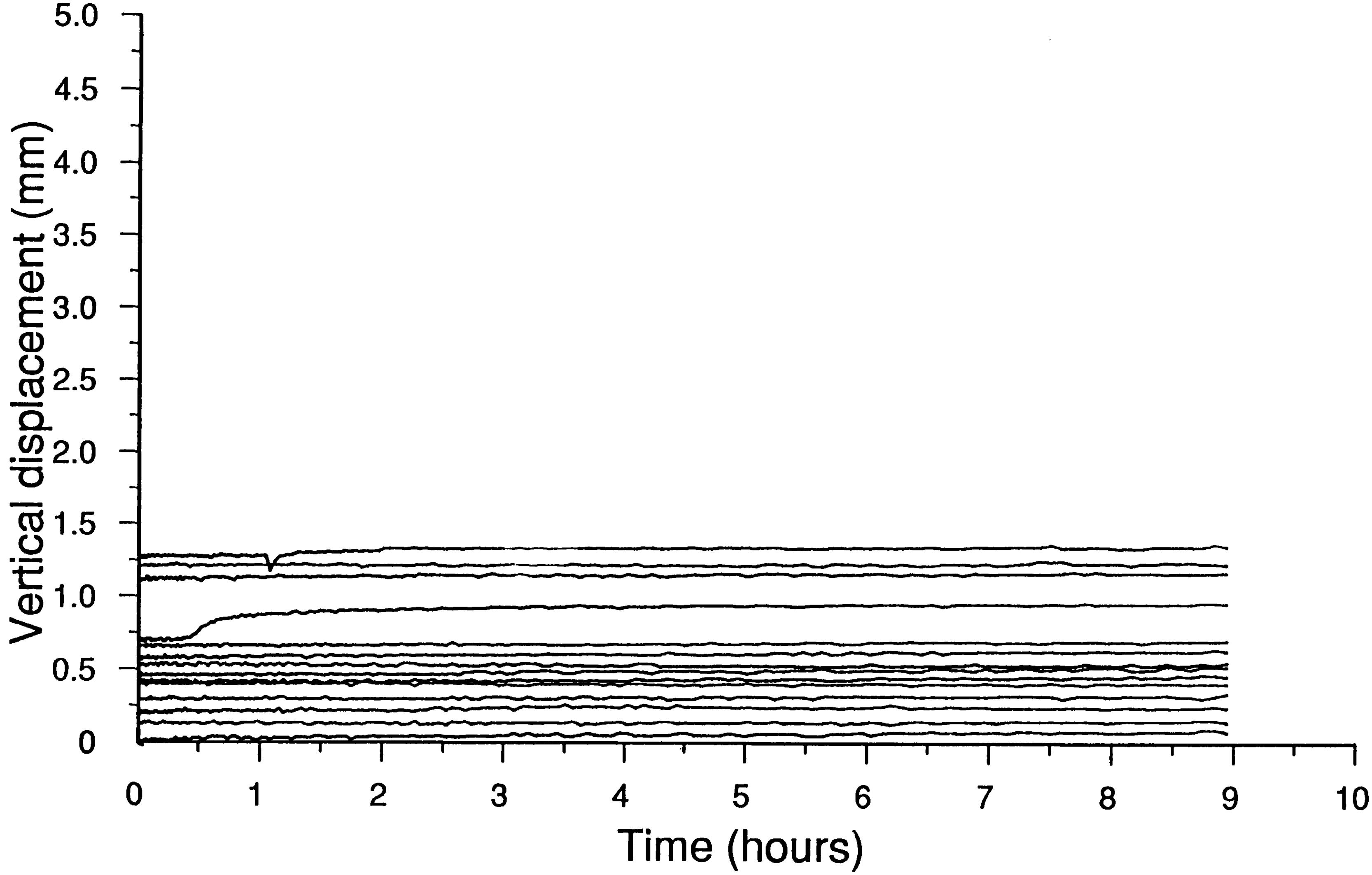


Figure 6.11(c) Load control direct shear results for Kaolin displacement for each increment of loading

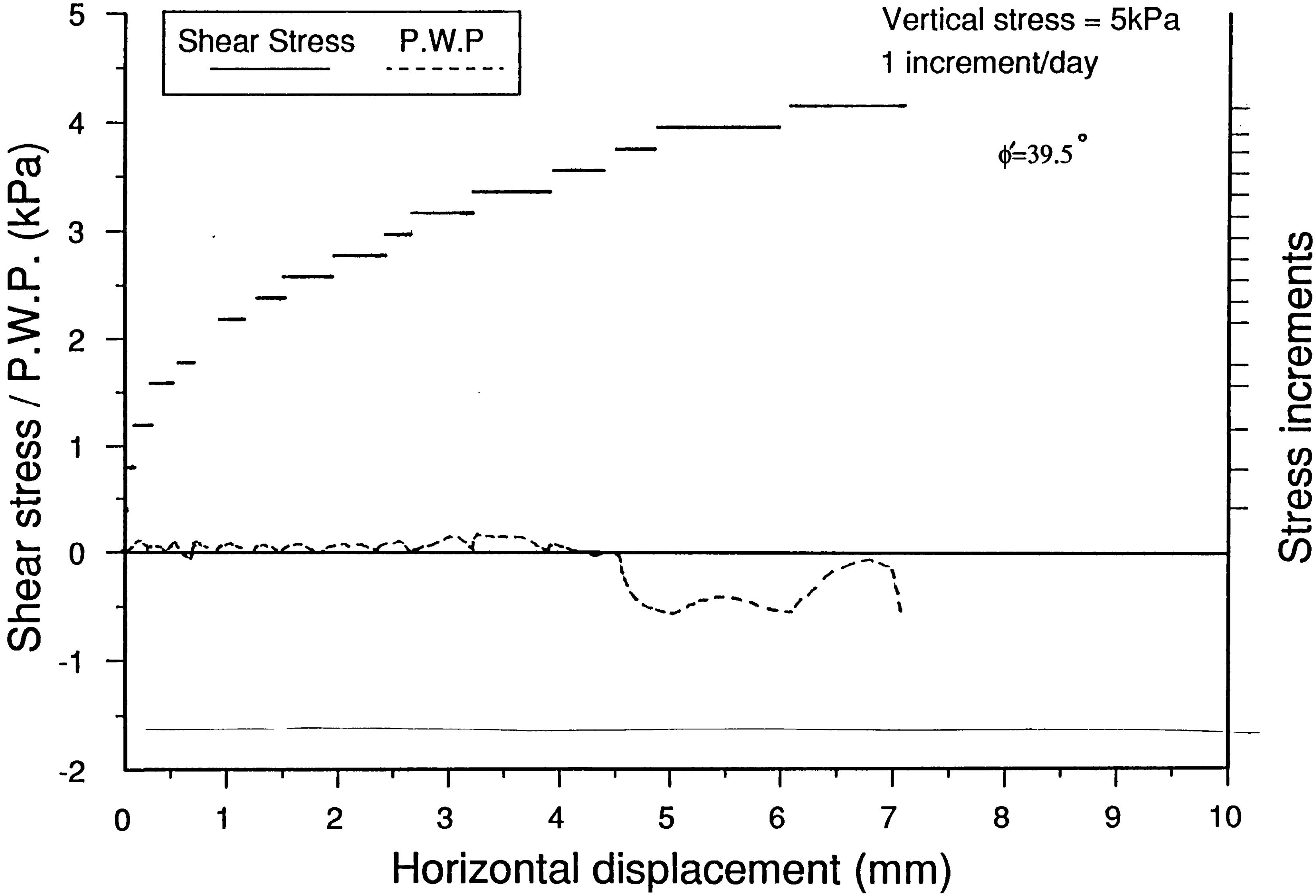


Figure 6.12(a) Load control direct shear results for London clay

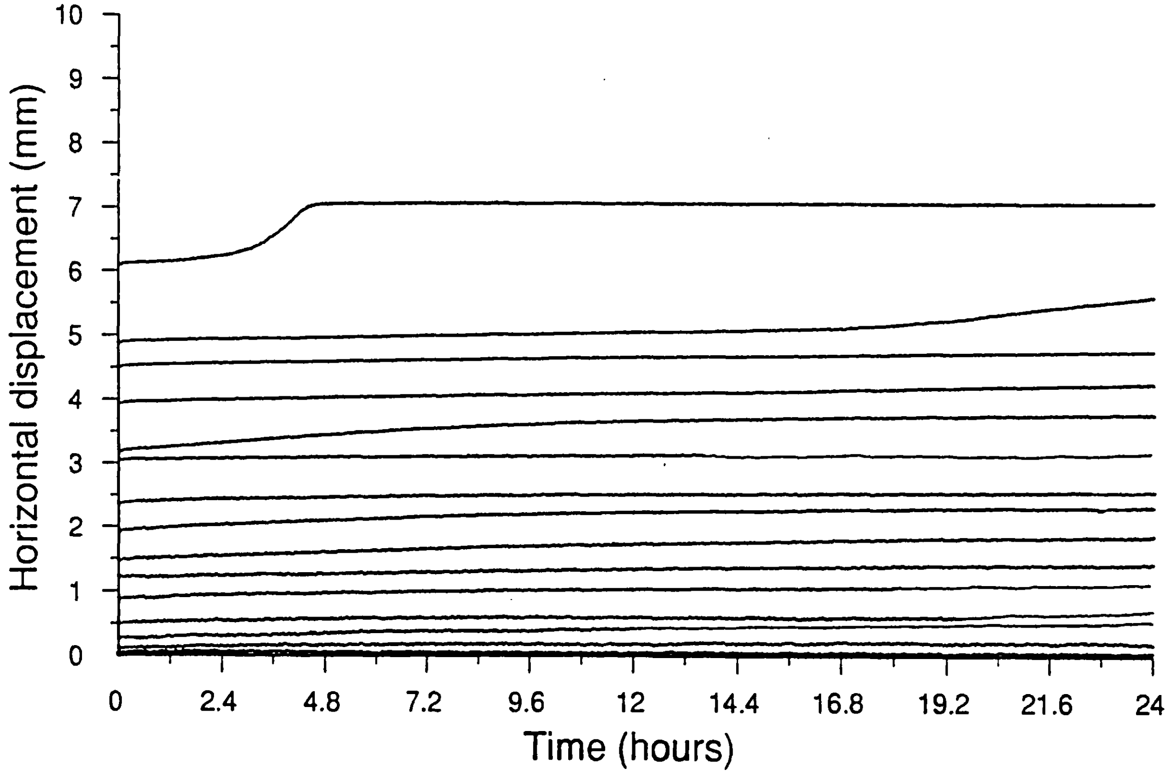


Figure 6.12(b) Load control direct shear results for London clay displacement for each increment of loading

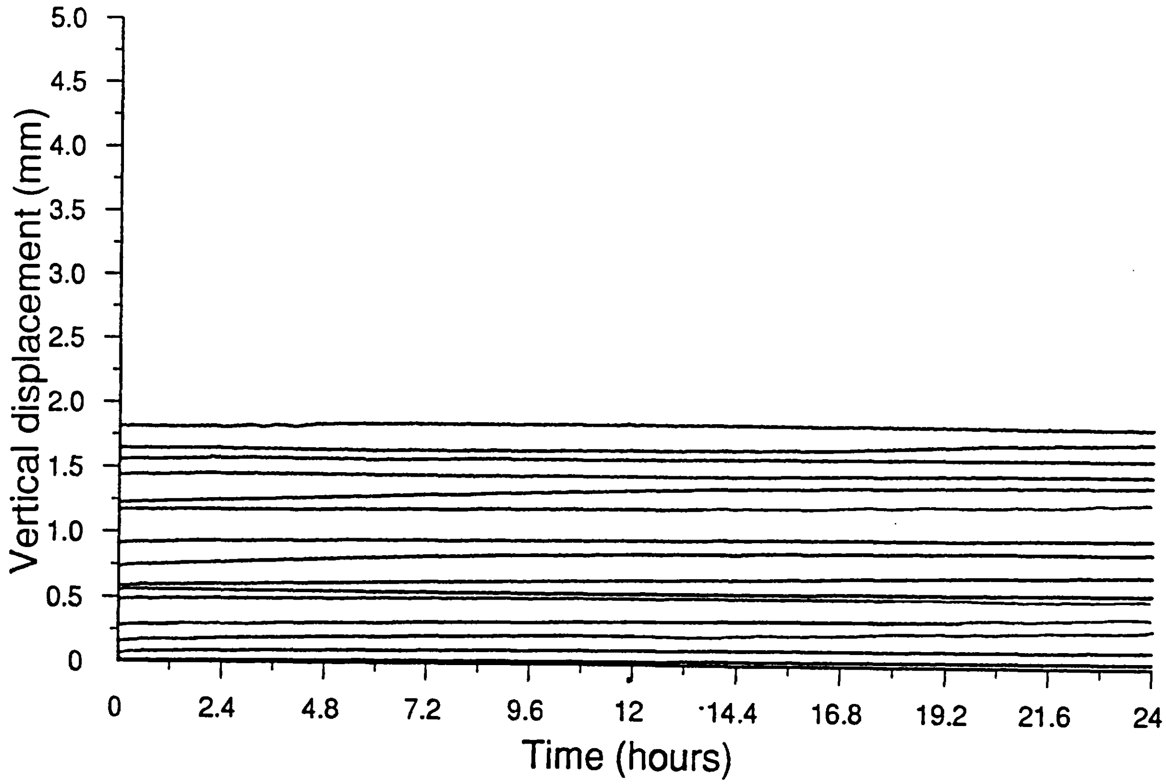


Figure 6.12(c) Load control direct shear results for London clay displacement for each increment of loading

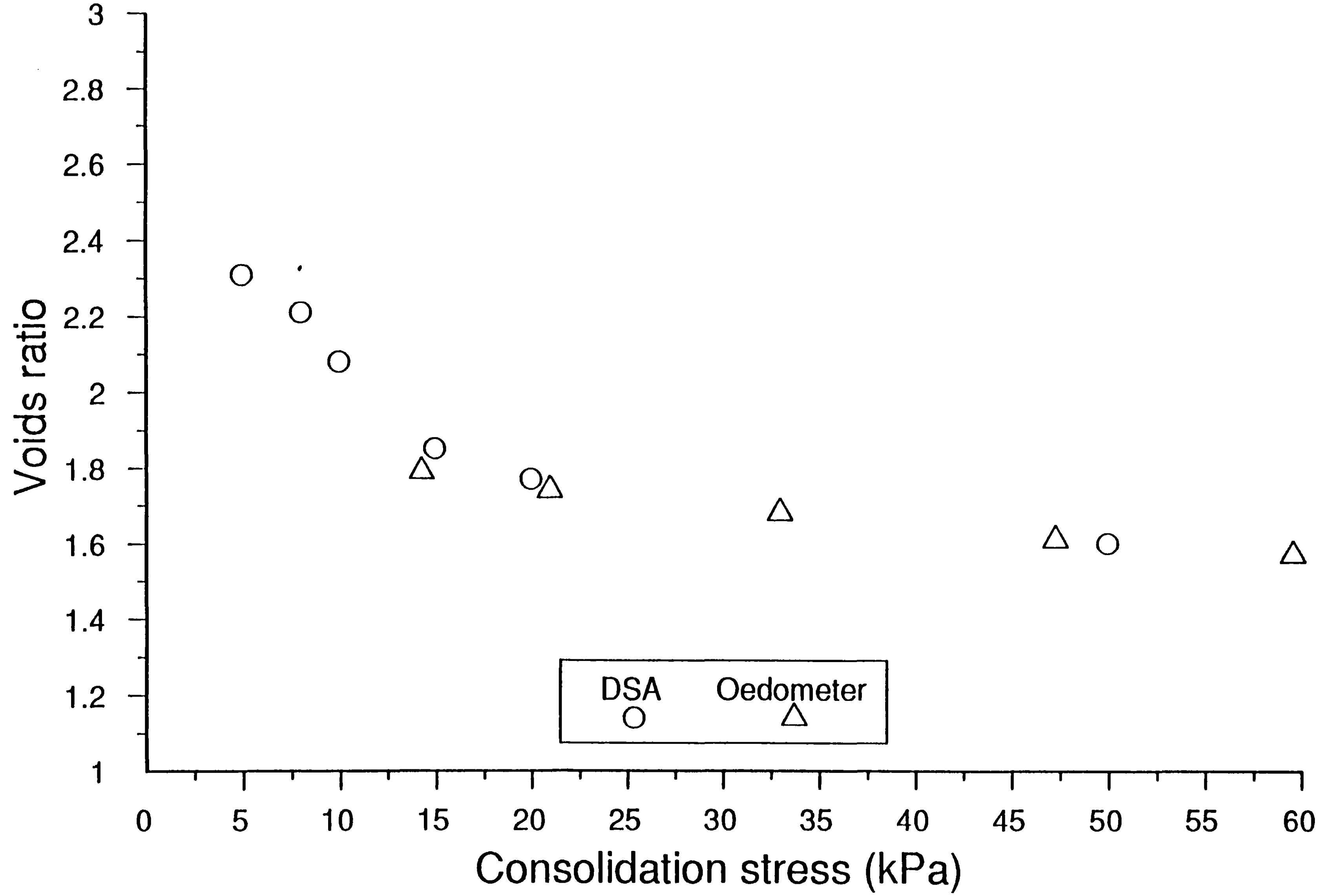


Figure 6.13 Comparison of Kaolin voids ratio in the DSA and the oedometer

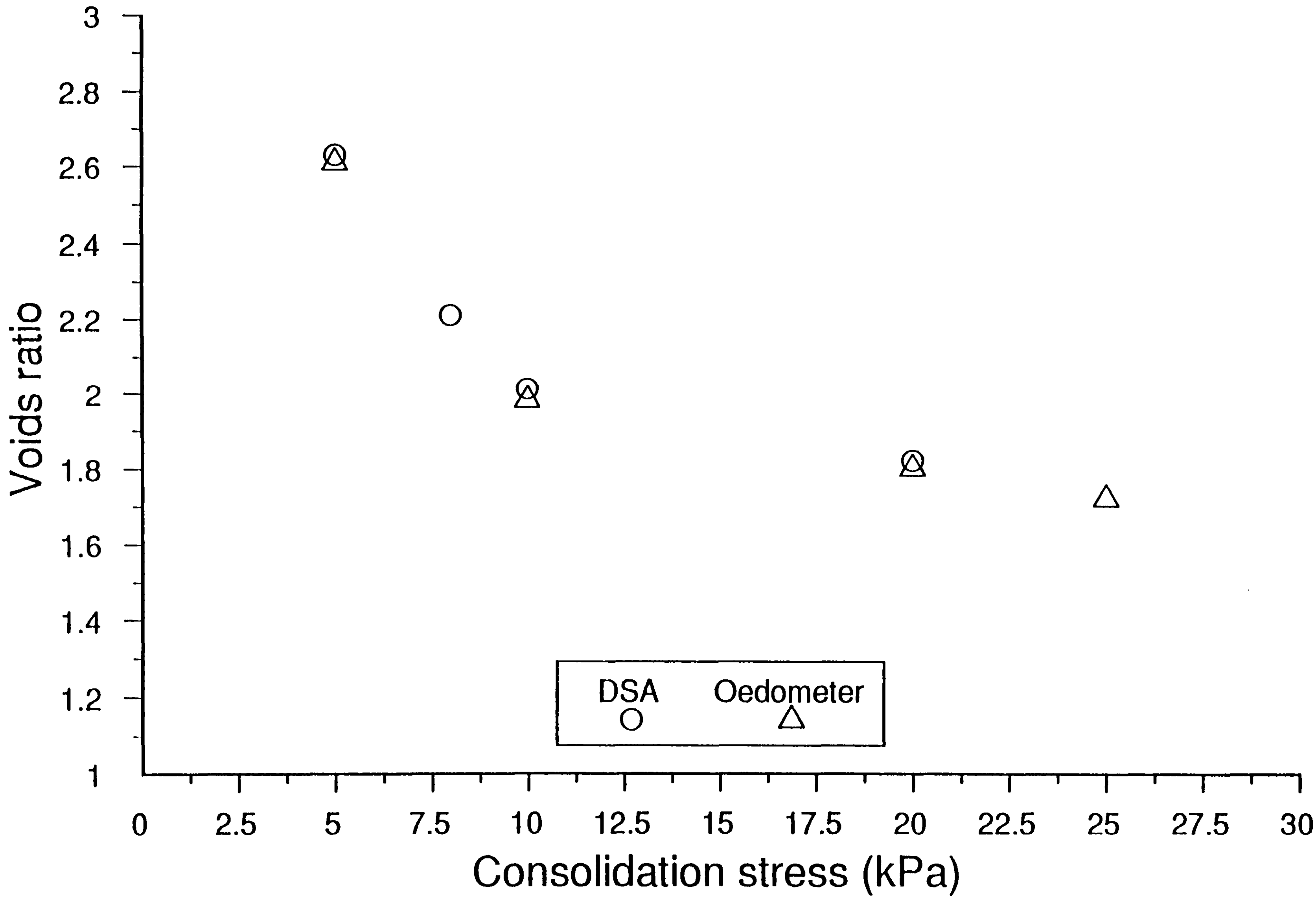


Figure 6.14 Comparison of London clay voids ratio in the DSA and the oedometer

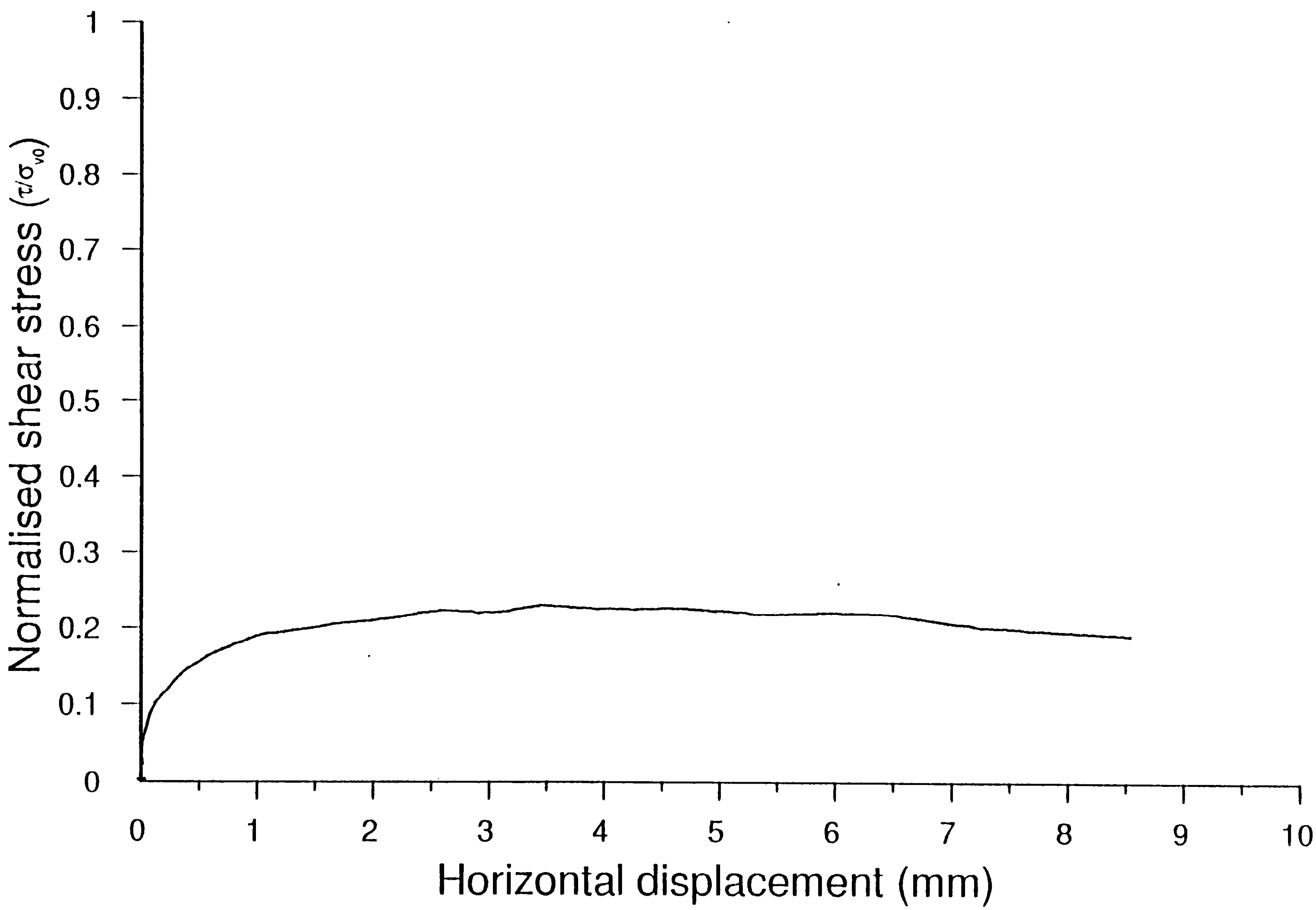


Figure 6.15 Typical results for Kaolin in the simple shear apparatus

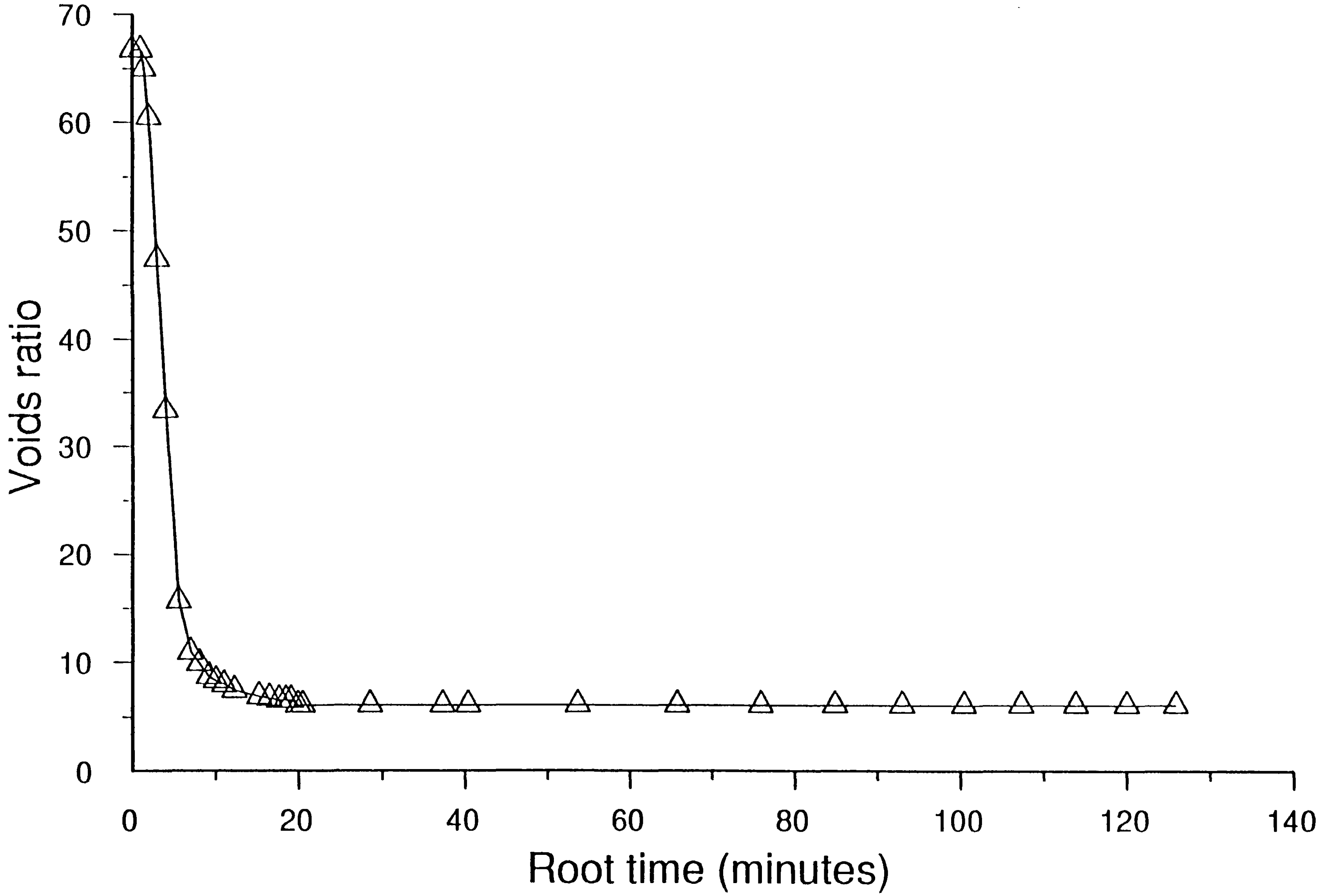


Figure 6.16 Sedimentation/consolidation curve for Kaolin

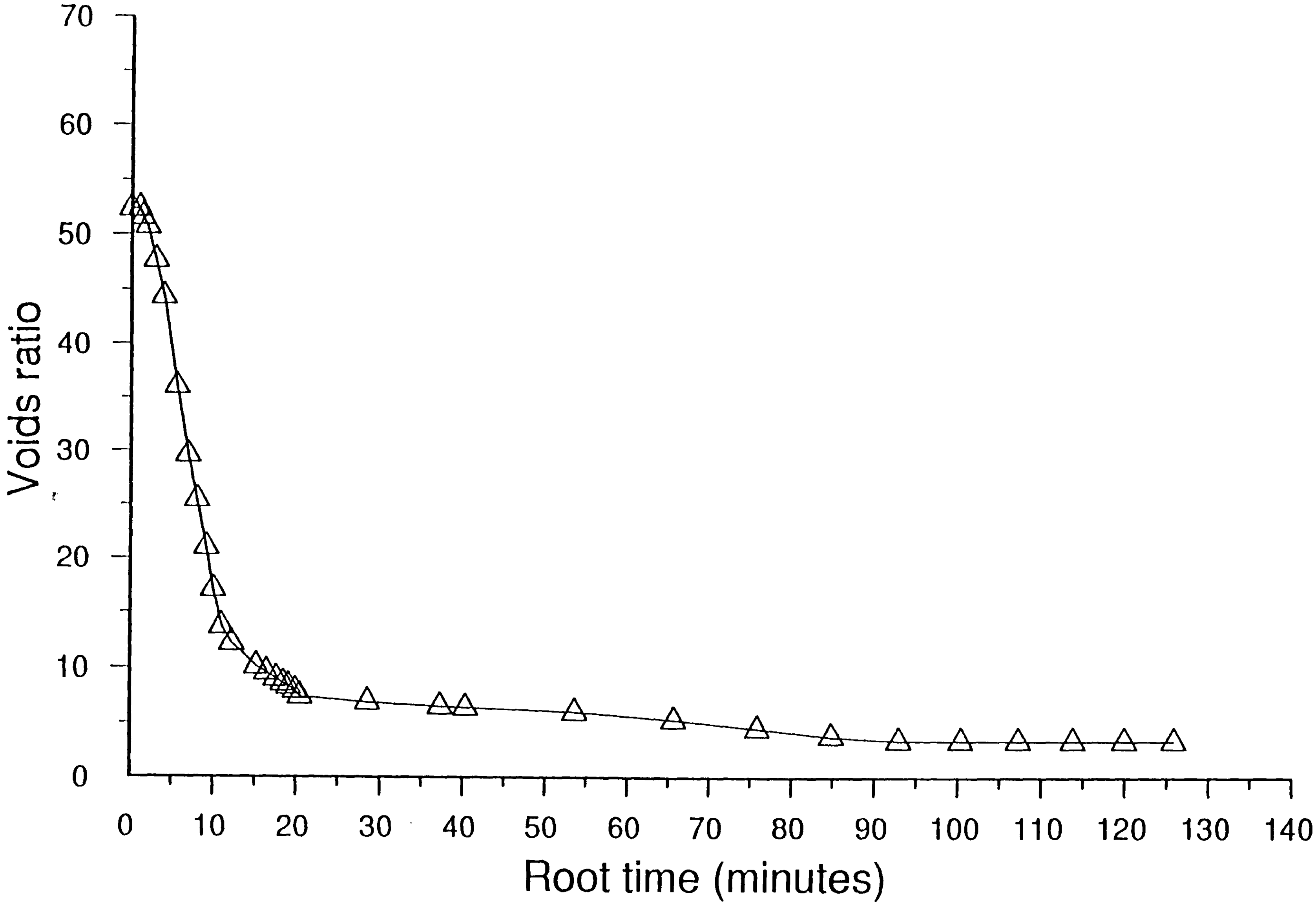


Figure 6.17 Sedimentation/consolidation curve for London clay

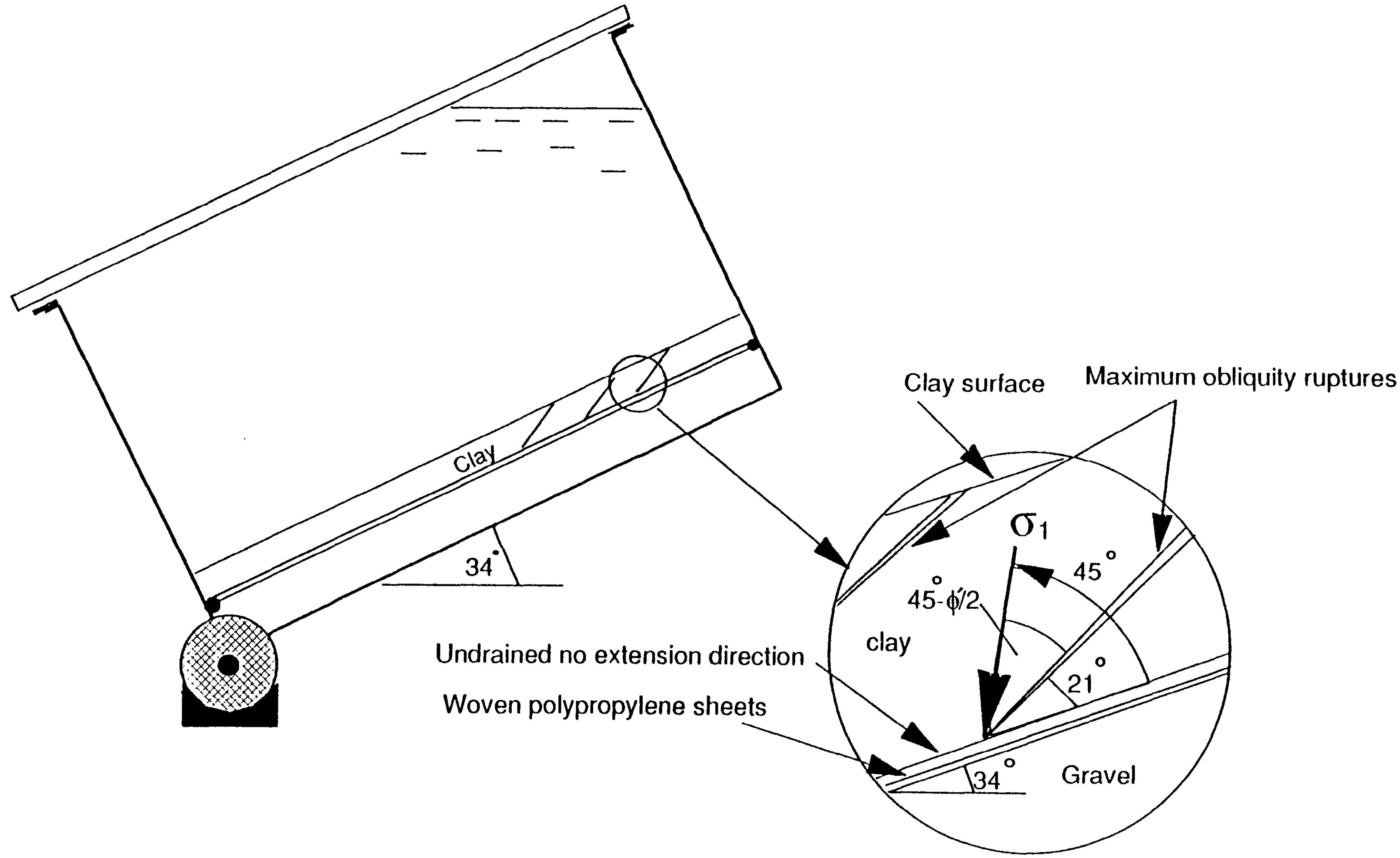


Figure 6.18 Local maximum obliquity ruptures in coagulated Kaolin

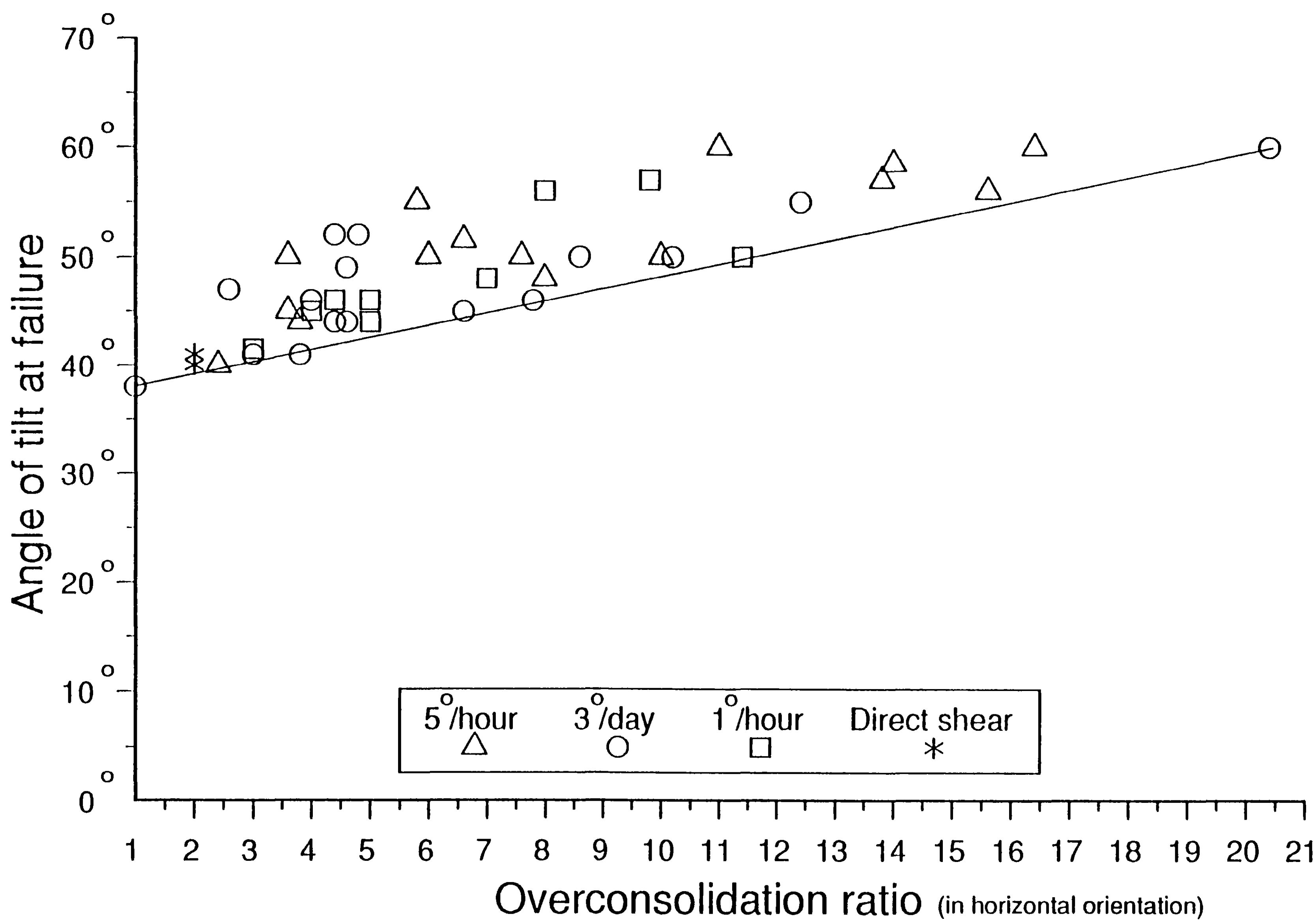


Figure 6.19(a). Results for overconsolidated Kaolin

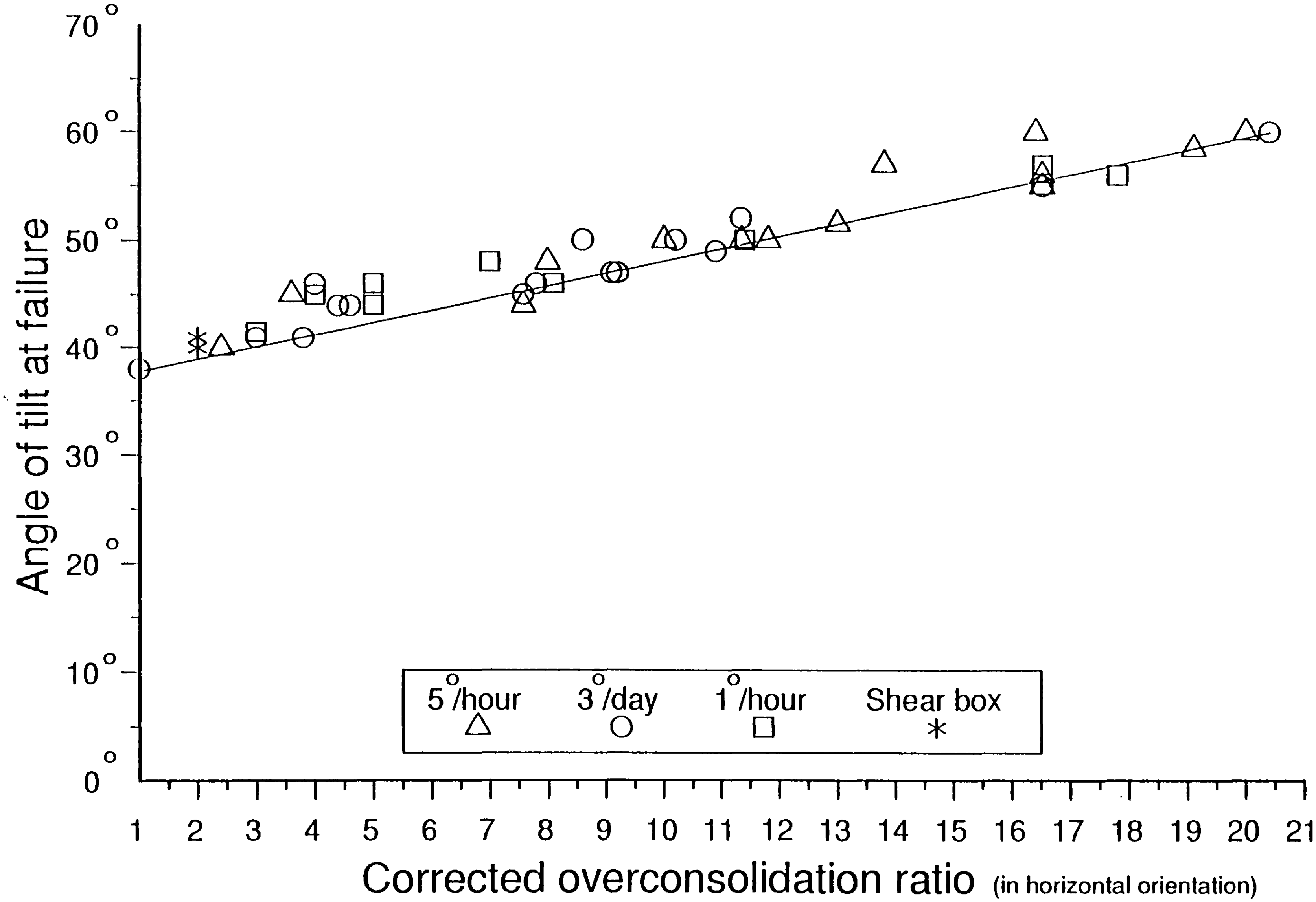


Figure 6.19(b). Corrected results for overconsolidated Kaolin

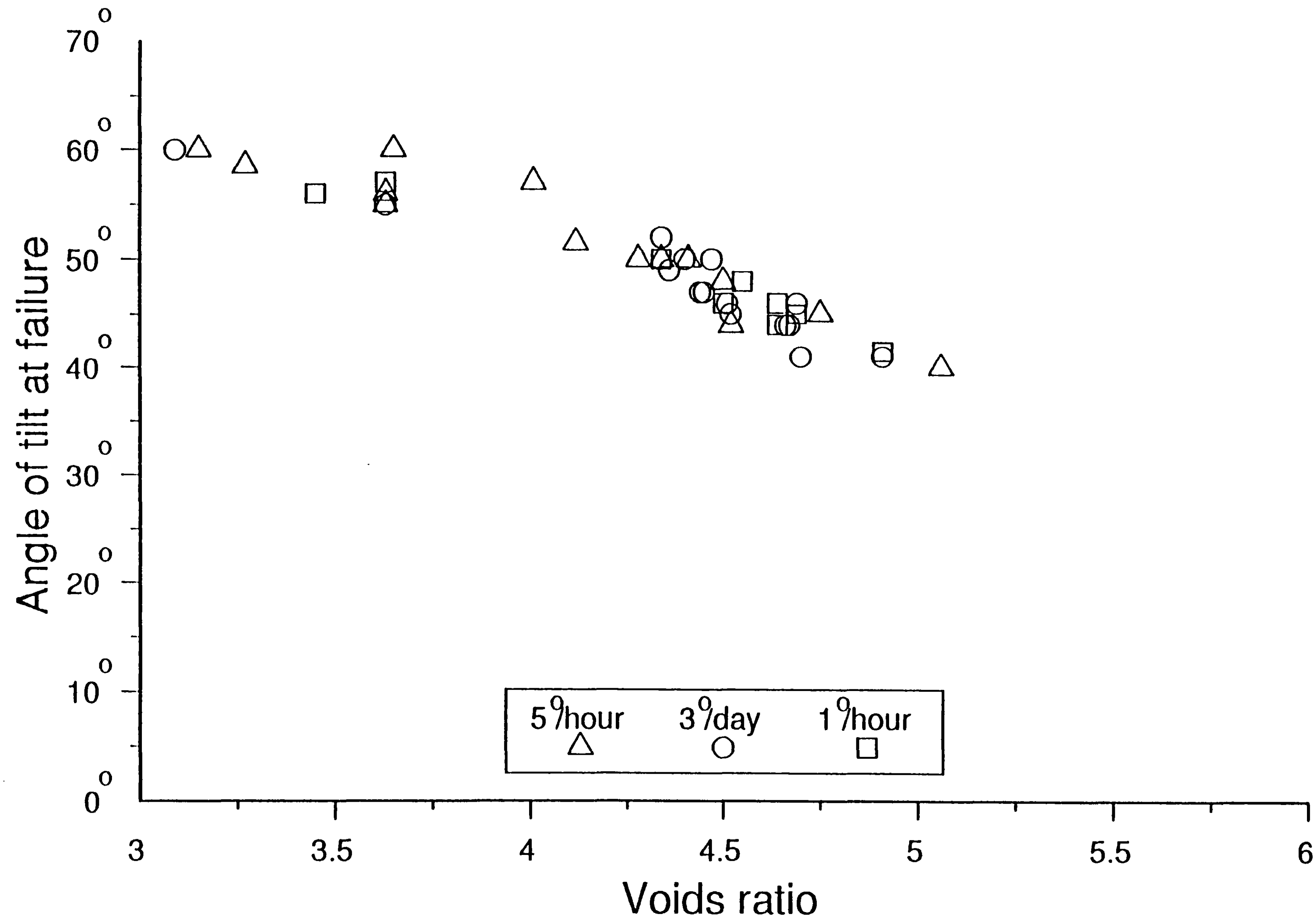


Figure 6.20 Results for overconsolidated Kaolin

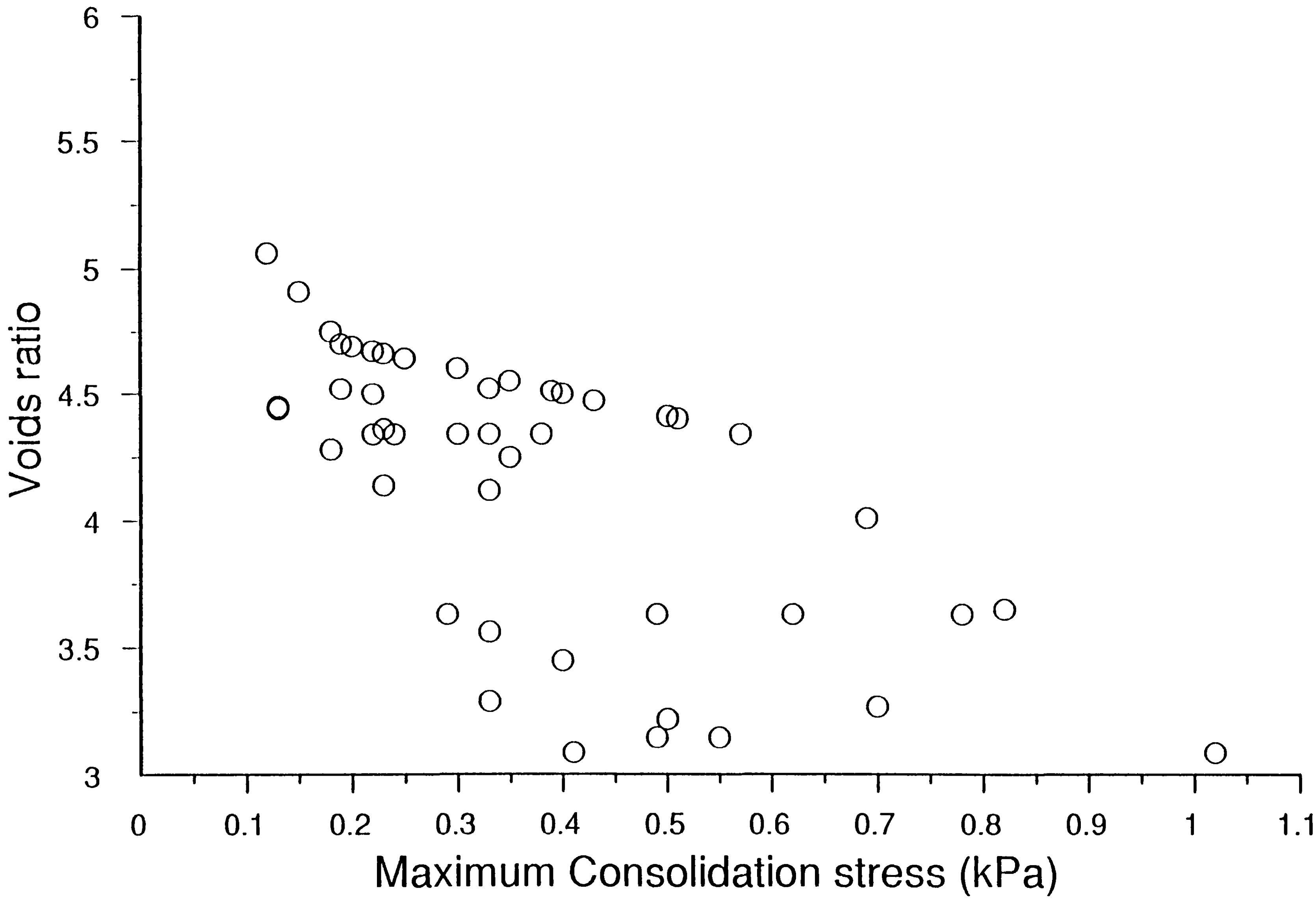


Figure 6.21 Results for overconsolidated Kaolin

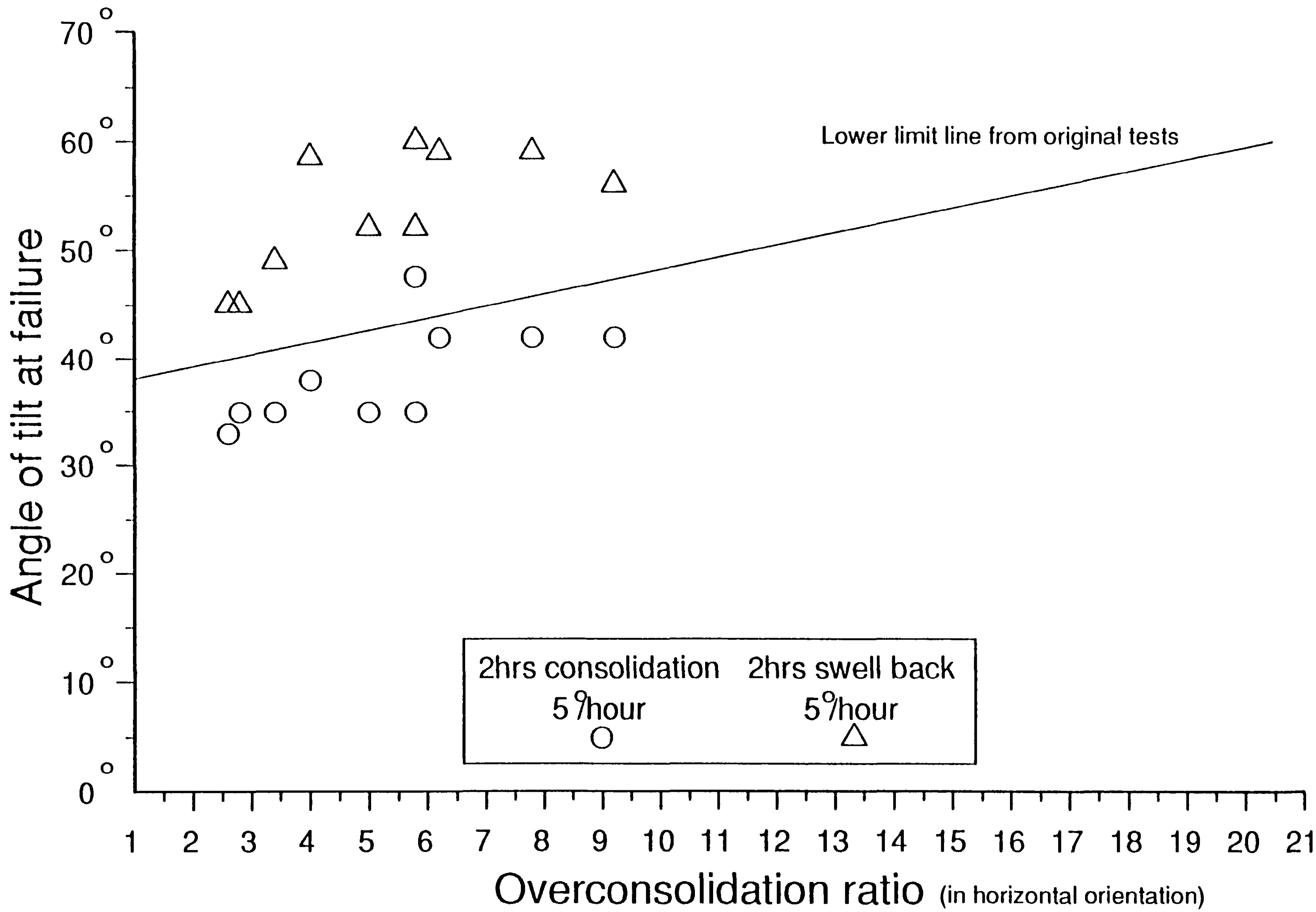


Figure 6.22 Comparison of consolidation and swell back periods for Kaolin

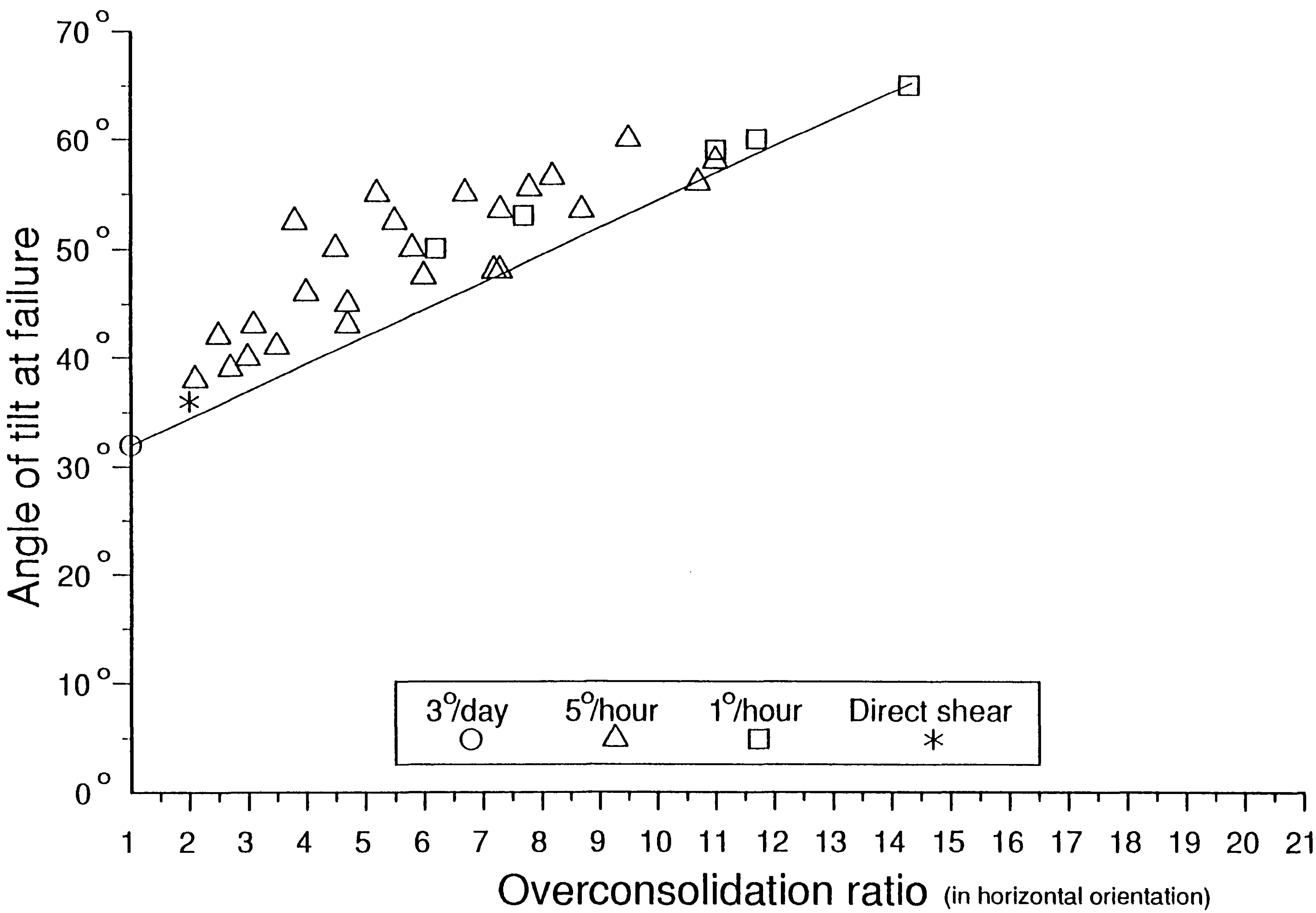


Figure 6.23 Results for overconsolidated London clay

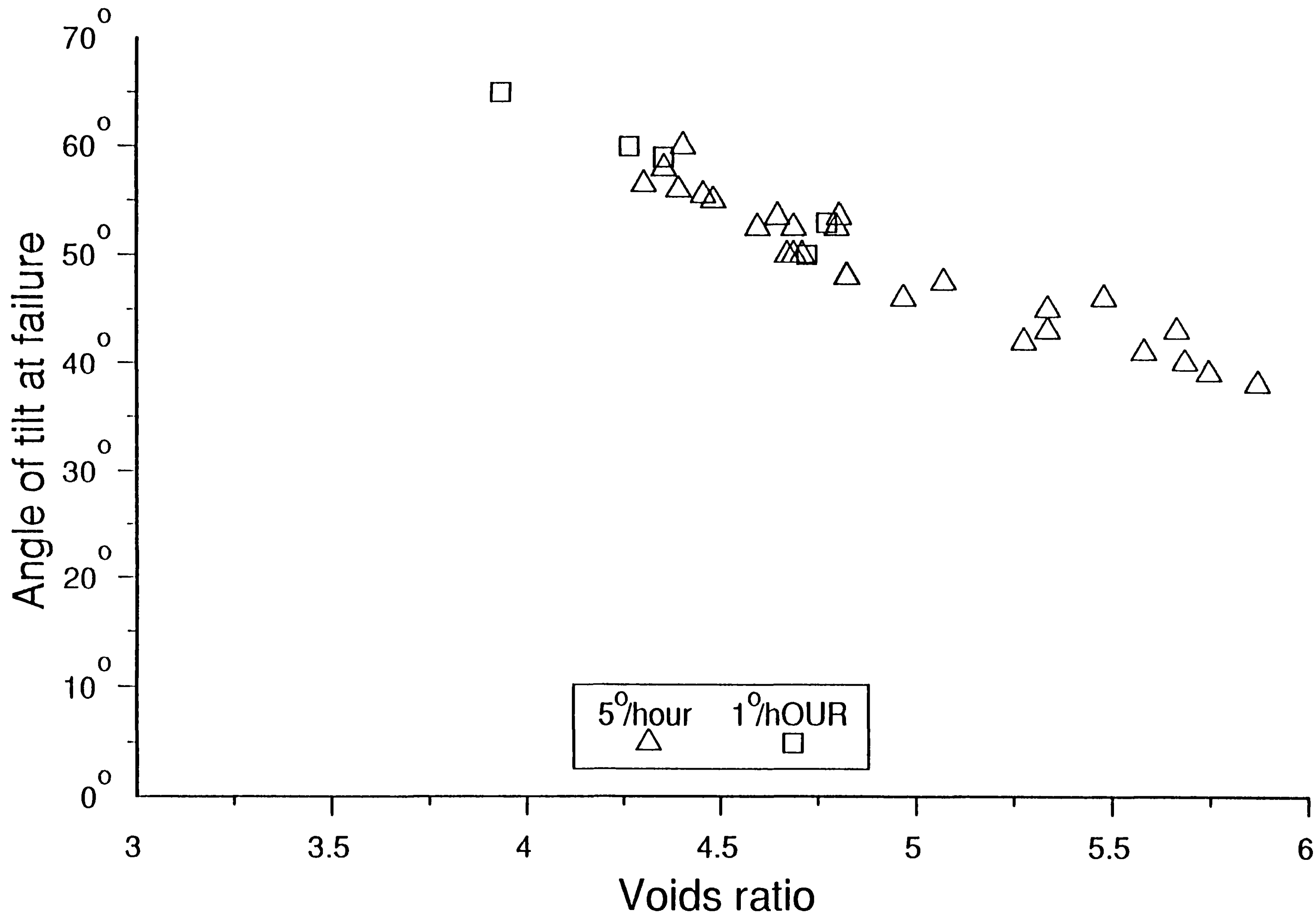


Figure 6.24 Results for overconsolidated London clay

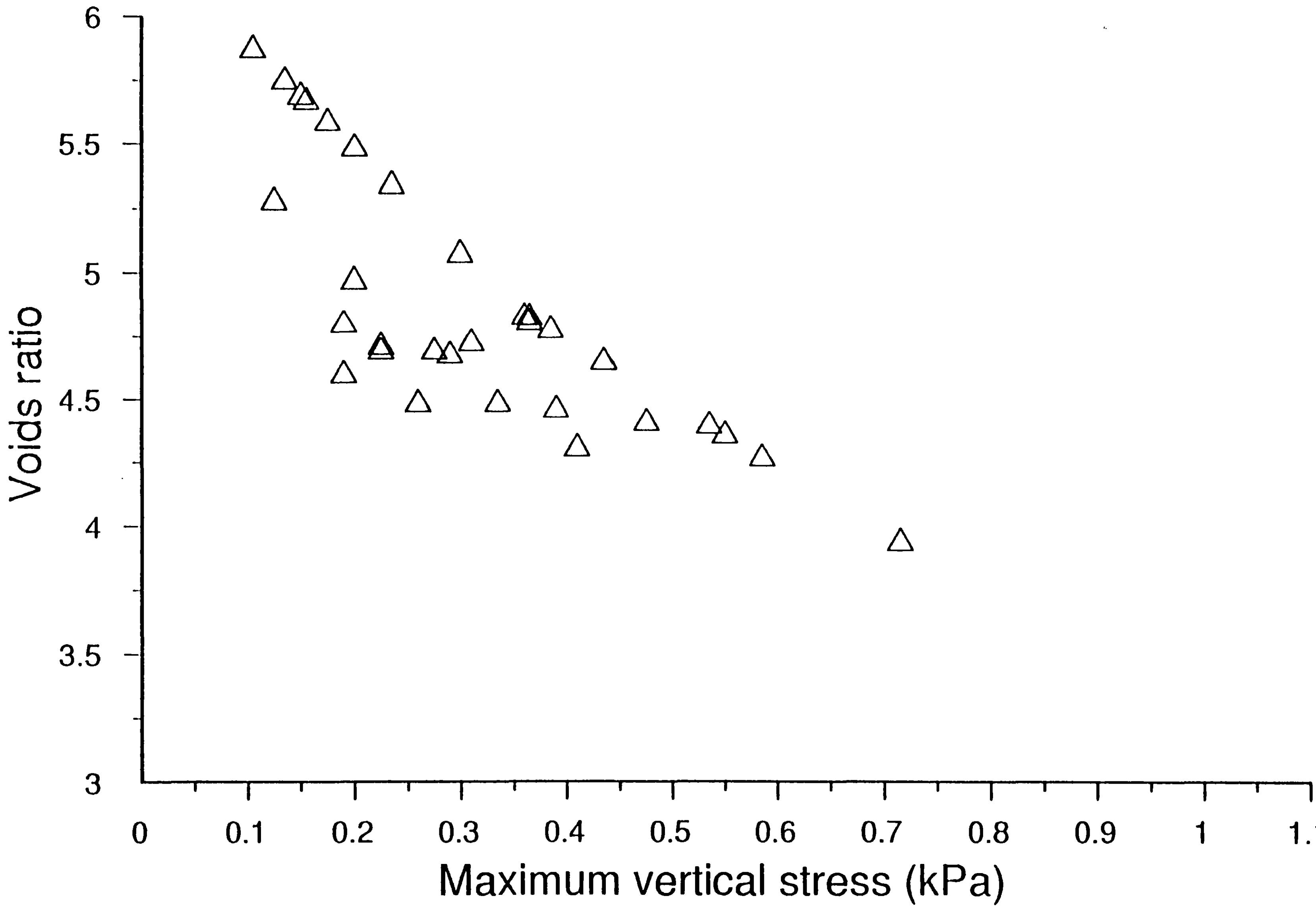


Figure 6.25 Results for overconsolidated London clay

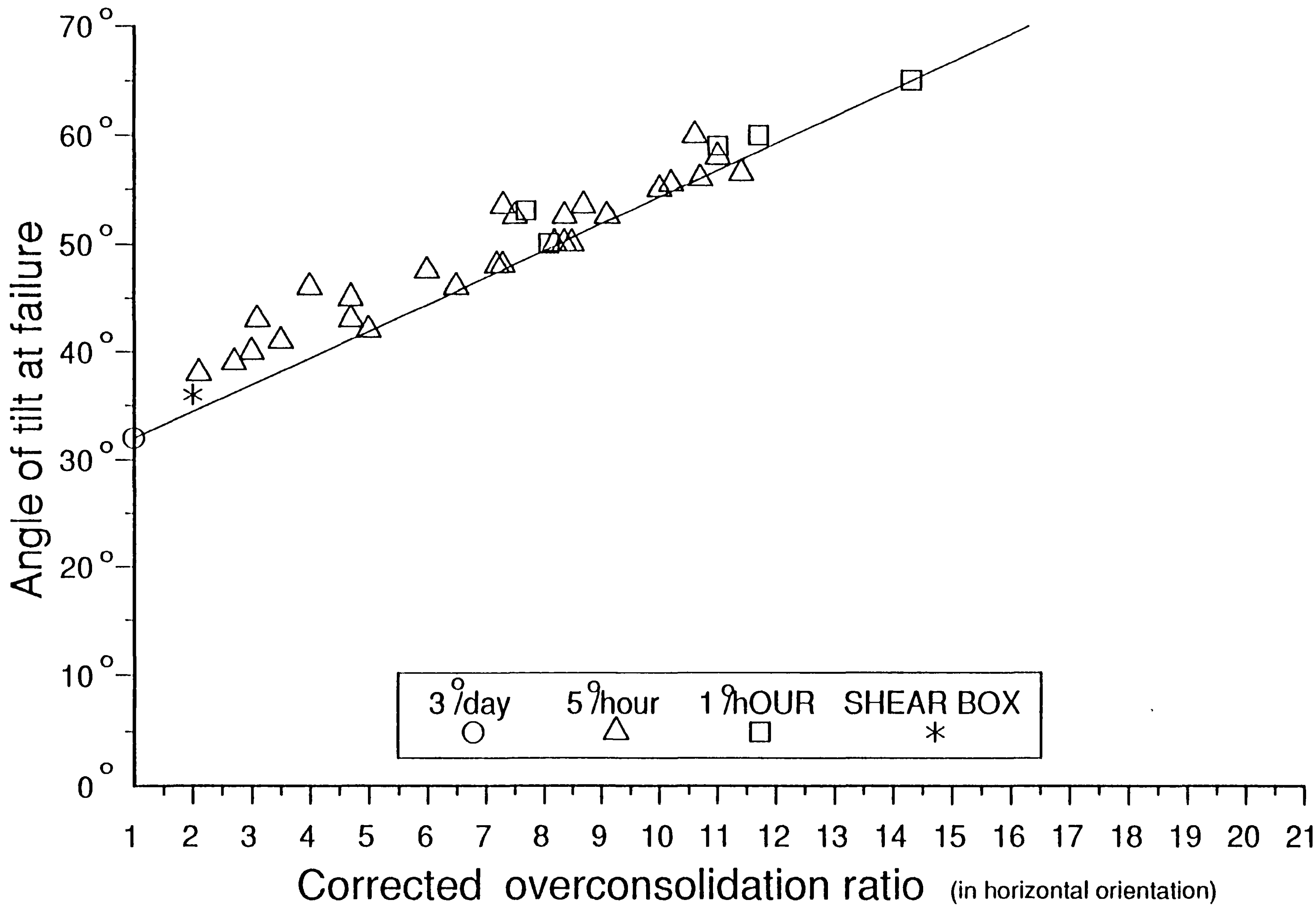


Figure 6.26 Corrected results for overconsolidated London clay

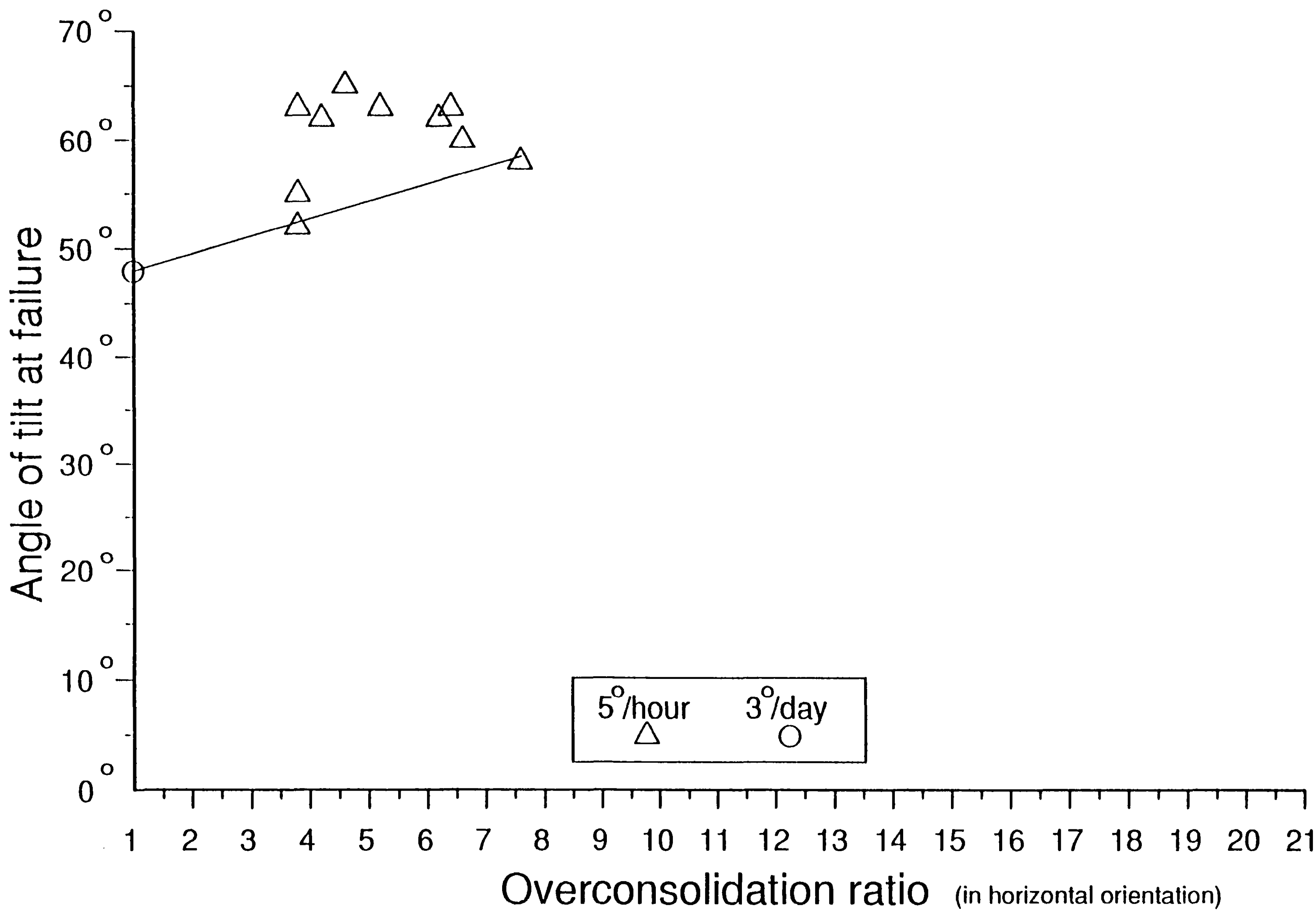


Figure 6.27 Results for overconsolidated coagulated Kaolin

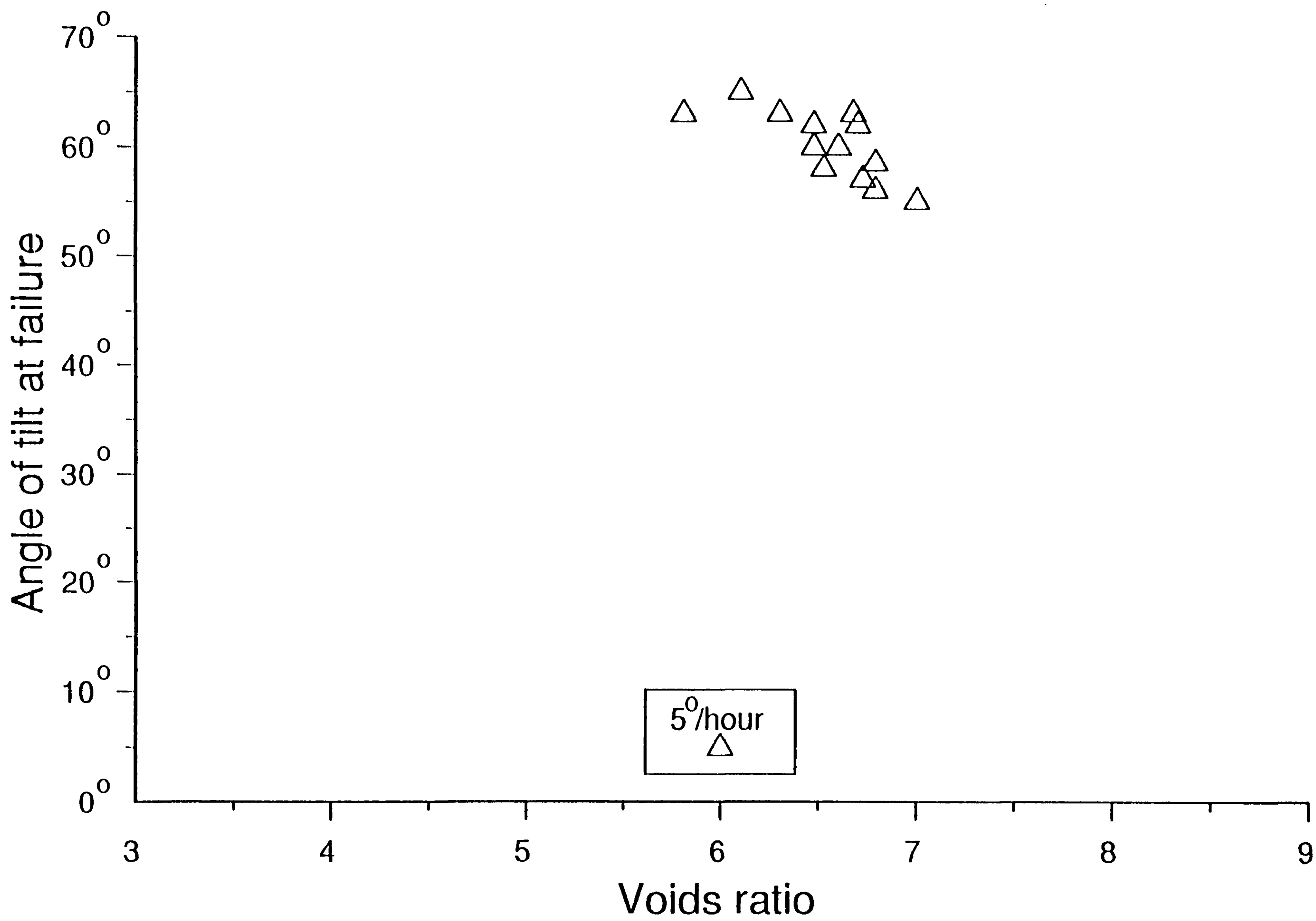


Figure 6.28 Results for overconsolidated coagulated Kaolin

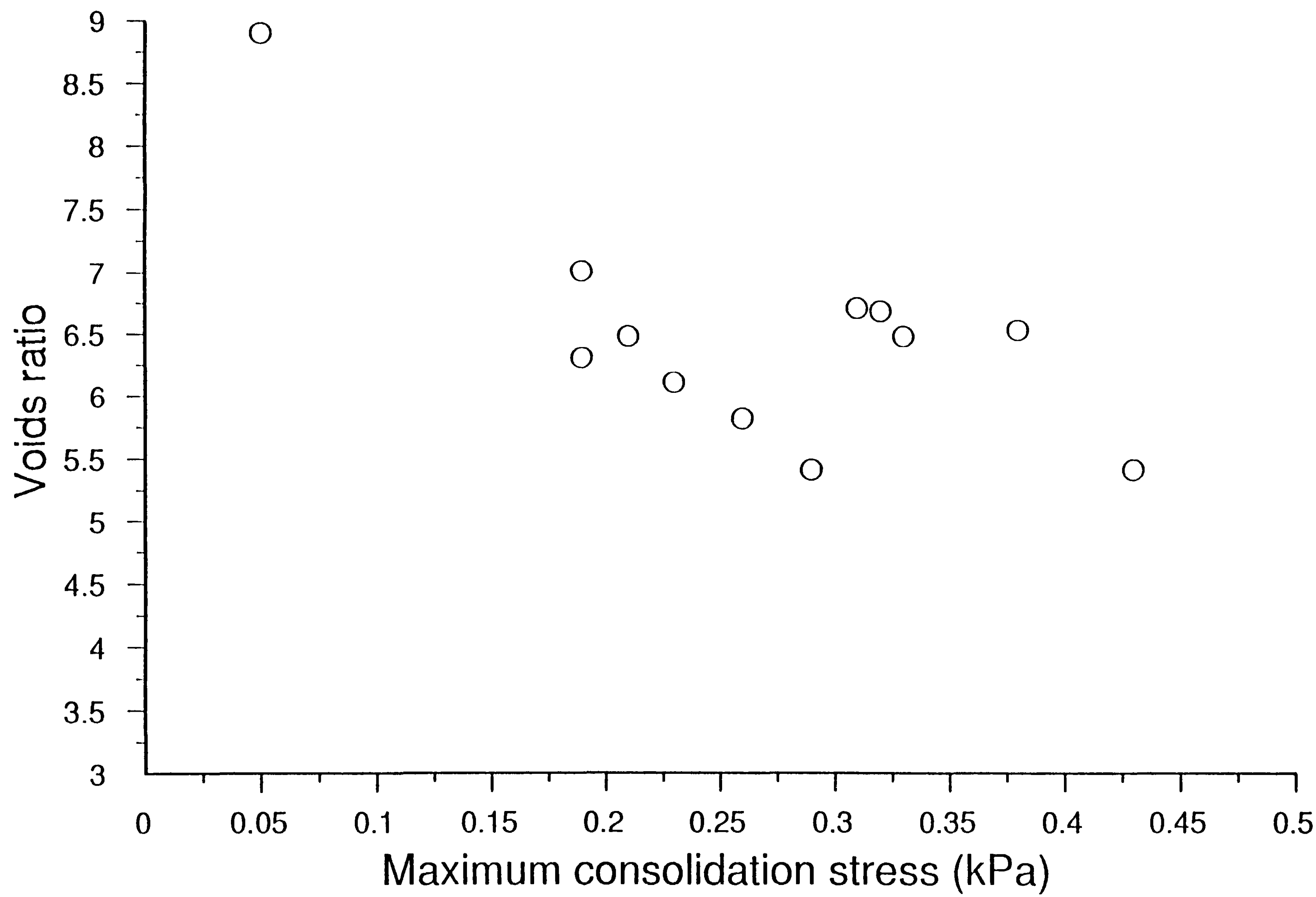


Figure 6.29 Results for overconsolidated coagulated Kaolin

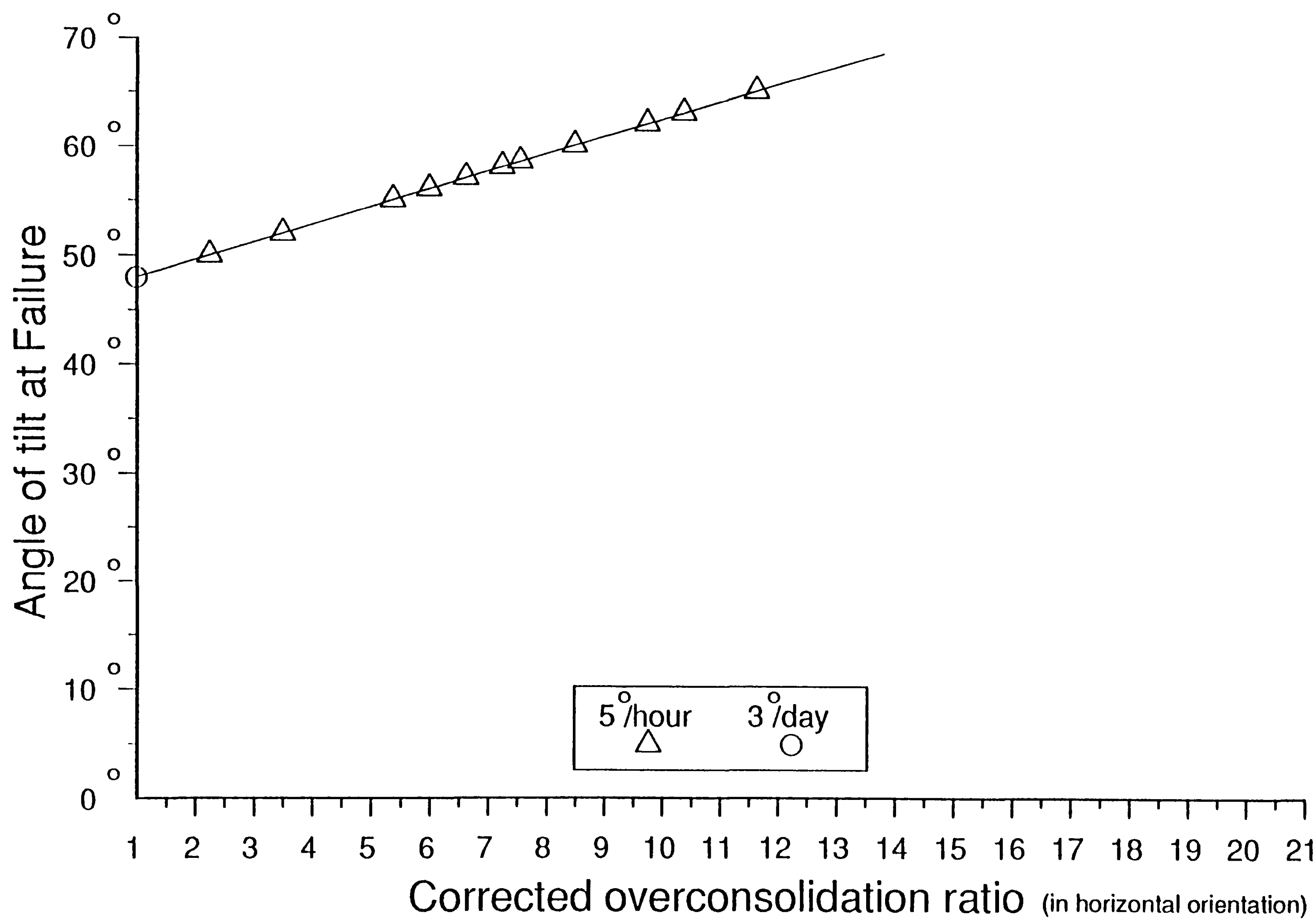


Figure 6.30 Corrected results for overconsolidated coagulated Kaolin

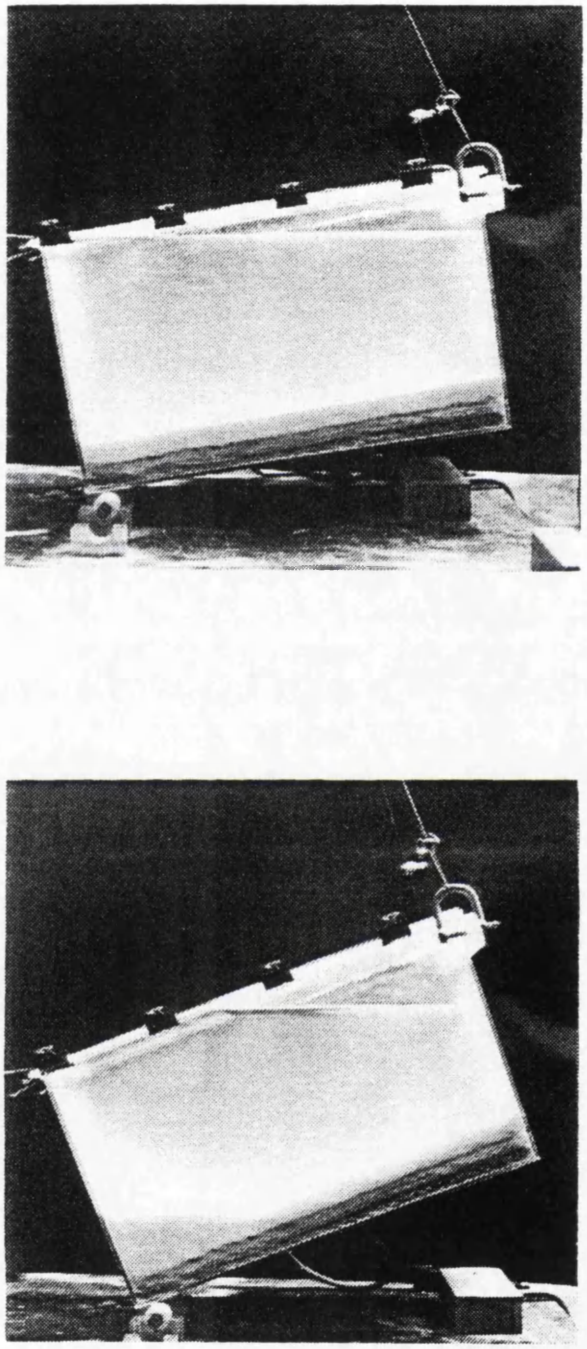


Plate 6.1 Tilting and Failure of Kaolin in the Standard Tilting Tanks

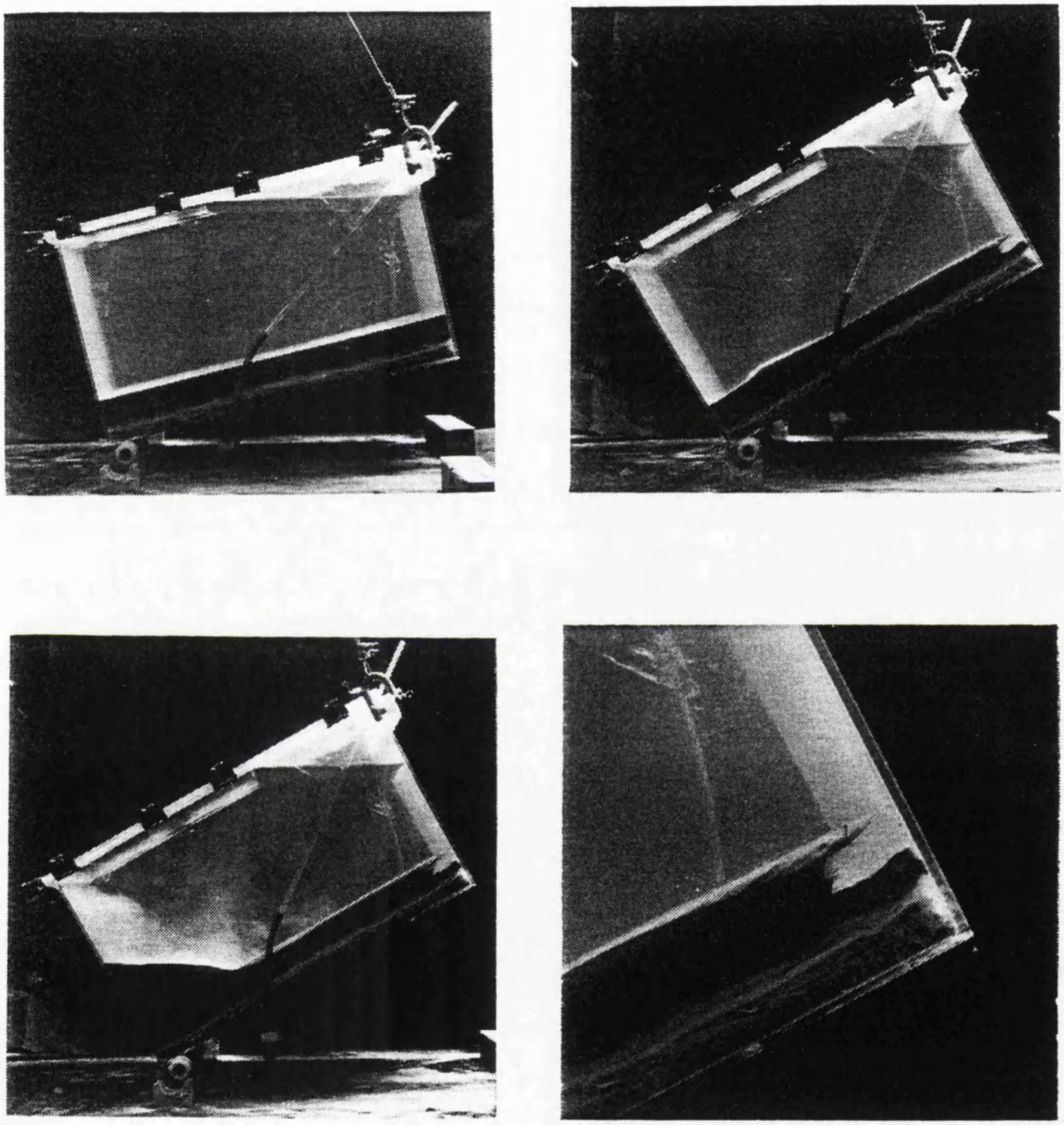


Plate 6.2 Tilting and Failure of London Clay in the Standard Tilting Tanks

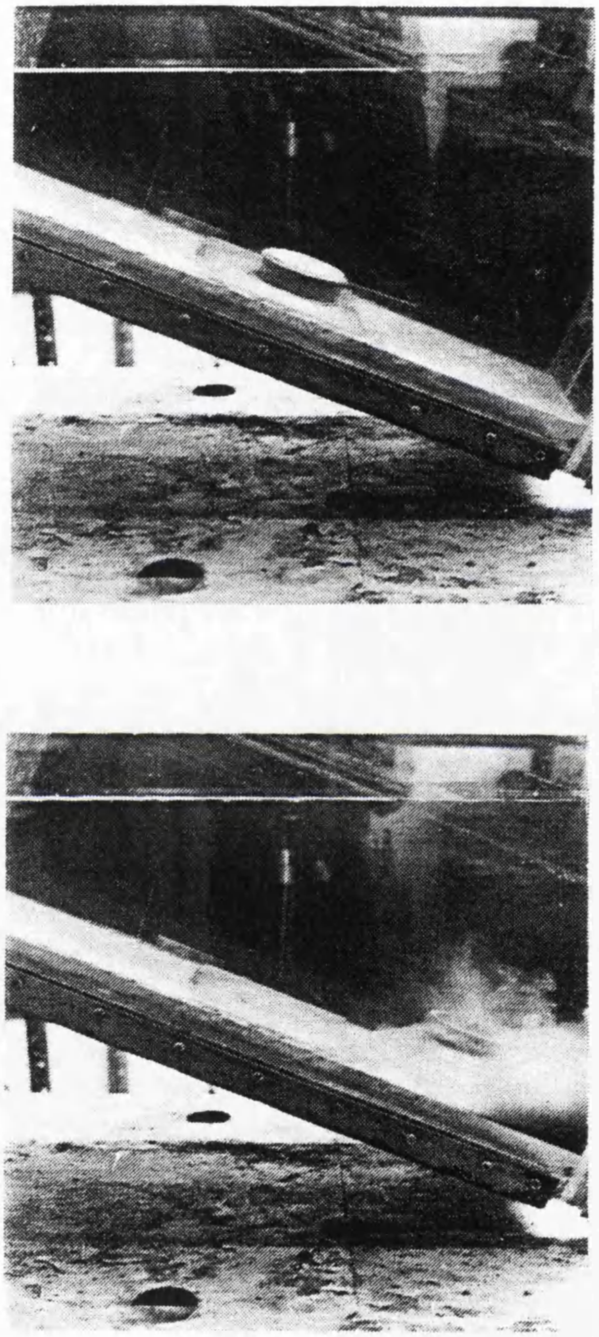


Plate 6.3 Tilting and Failure in the Shear Cylinder

CHAPTER 7

DISCUSSION AND INTERPRETATION OF RESULTS

7.1 Introduction

Essential information on possible failure mechanisms and thixotropy has been given in Sections 4.6 and 6.2 respectively; this information is necessary for the discussion and analysis of the results. A brief analysis of the results obtained using the SSA and the Tension test will be dealt with first as they do not provide any positive conclusions; accurate vertical loads could not be measured for the SSA (Sections 4.3 and 6.4) and realistic overconsolidation ratios could not be obtained for sample used in the Tension tests (Section 6.5.3). The rest of the this Chapter will be dedicated to the results obtained using the DSA, the standard tilting tanks and the shear cylinder.

7.2 Simple Shear Apparatus

Although vertical load measurements were inaccurate due to the effect of bending in the load cell (Section 4.3) the stress ratios at failure (τ/σ_v) for constant height tests (Section 6.3) were in agreement with the values obtained in the displacement control DSA, $\tau/\sigma_{v0}' = 0.2$ and as reported by Airey (1984) testing the same clay in the Cambridge simple shear apparatus). This indicated the ability of the SSA, with accurate vertical load measurements, to be used for testing soils at low stress levels. Dyvik et al. (1987) have shown that a properly set up SSA would have been suitable, and could be modified to measure pore pressure allowing constant load tests to be performed.

7.3 The Tension Test

The three tests performed on the extremely highly overconsolidated Kaolin gave a tensile strength of 0.0378kPa, which can be considered as a negative conclusion indicating a very small cohesive strength. Similar tests were carried out by Ajaz and Parry (1975) where the behaviour of two clays, compacted with standard Proctor effort at different moisture contents ranging from below to above optimum, was observed in direct tension and bending. They measured tensile strengths ranging from 60 to 100 KN/m^2 for moisture contents ranging from 31% to 22% respectively. A direct comparison of results is not possible as the sample preparation was vastly different specially as Ajaz and Parry (1975) did not consider or quantify the stress level. Another important factor which made the results unsuitable for further analysis is that the procedure of overconsolidation used for the tension test did not allow specific overconsolidation ratios to be achieved and therefore a relationship could not be established. Also, at the very high overconsolidation ratio, swell back may have broken particle bonds.

7.4 Direct Shear Apparatus, Standard Tilting Tanks and Shear Cylinder

The interpretation of data will be split into sections according to the relevance of the data to particular topics. For the first six sections the data from tests on sedimented samples will be standardised in that the preparation of these samples will have been by hand mixing in London tap water (LTW).

7.4.1 Consolidation / sedimentation behaviour

7.4.1.1 Direct shear Apparatus

The voids ratios measured in the DSA for normally consolidated Kaolin agree well with the Kaolin consolidation curve obtained in the standard oedometer by the author and by Dalili (1990), Figure 6.13. The voids ratios measured in the DSA for London clay also agree well with the consolidation curve obtained in the standard oedometer (Figure 6.14).

7.4.1.2 Tilting tanks

The sedimentation curves shown in Figures 6.16 and 6.17 for both Kaolin and London clay respectively show a typical behaviour of both clays during sedimentation, consolidation and creep. Kaolin reaches an equilibrium after 20 hours ($e = 6.1$) whilst the London clay develops a voids ratio of 6.1 within 24 hours and continues to creep for six days reaching a voids ratio of 3.2. The initial unit weight of both slurries was 10.22 KN/m^3 for a dry mass of clay of 0.75 Kg. Similar voids ratios were obtained for Kaolin at an initial unit weight of 10.44 KN/m^3 (1.5 Kg initial dry mass). Been and Sills (1981), Sills and Thomas (1983), Elder and Sills (1985) and Edge and Sills (1989), carried out detailed study of the behaviour of clay during the sedimentation consolidation processes. Been and Sills (1981) carried out a series of test in which a mud slurry of a known unit weight (10 - 12 KN/m^3) was introduced into acrylic settling columns. They monitored the density throughout the column and measured the pore pressure and total pressure at specific points.

Kaolin and London clay showed similar behaviour to that observed by Been and Sills (1981) for the same concentrations, they measured an average voids ratio of 6 and 3.2 after 48hrs and 315hrs respectively and concluded that there is no unique void ratio corresponding to zero effective stress. Voids ratio was later shown by Sills and Thomas (1983) to be dependent on the initial concentration and deposition (load application) rate, the higher the rate of deposition of a given mass of sediment, the more compacted is the resulting settled mud layer. They attributed this to the fact that a slow deposition rate allows individual flocs to achieve a higher strength (due to a time dependent behaviour; thixotropy as seen and suggested by the author) before being loaded by sediments arriving later.

Intentionally coagulated Kaolin resulted in a higher final voids ratio ($e=8.9$), whilst Kaolin with salt (35g/litre) resulted in final voids ratio of 6.1, similar to that in London tap water. London clay with salt resulted in a voids ratio of 3.2

within 24hrs and no further reduction was seen to occur due to creep. Elder and Sills (1985) observed similar behaviour due to addition of a coagulating agent and a dispersing agent (salt is also a dispersing agent). They observed that the addition of a dispersing agent resulted in a final denser settled layer of mud although some fine material stayed in suspension for a long time; whilst the addition of a coagulating agent cleared the overlying water more successfully but was actually less efficient in thickening, resulting in a less dense settled layer of mud.

Overconsolidated Kaolin and London clay showed a large scatter in the measured voids ratio (Figures 6.21 and 6.25). This can be attributed to the rate of application of the vertical stress during further consolidation. Similar behaviour was observed by Edge and Sills (1989); their experiments were performed in a similar apparatus to that used by Been and Sills (1981) using a slurry containing sand silt and clay, however they produced layered sediments by varying the rate of sediment application and the initial density of the slurry to existing settled layers of mud. From these experiments they observed that sand falls quickly from the first sediment input of the day to the surface of the existing bed, and there forms a thin high-density layer above the much lower density clay. Thus despite its lower density, the clay clearly has sufficient strength to prevent the penetration of the sand. This is in contrast to the behaviour later in the day, when the sand falls through the top few millimetres of the existing bed, mixing with the silt and the clay to produce a bed of reasonably uniform density. This implies that the shear strength of the clay has increased overnight. They concluded that the increase in strength was due to aging. In the case of further consolidation for the experiments carried out by the author the magnitude of stress increment and the intervals between stress increments will therefore influence the final voids ratio obtained, as a large increment of stress application would result in destroying the existing structure and lead to further consolidation, whilst a small magnitude of stress application at a slow rate will result in the minimal level of consolidation occurring.

This research programme did not consider measuring coefficients of permeability or consolidation as this would have been difficult with the present apparatus at the very low stress levels considered. However pore pressure measurements in the DSA and the large variation in the testing rates allowed accurate deduction to be made on the state of the soil during shearing (drained or undrained). Nagaraj (1968) measured both the coefficients of permeability and consolidation for both dispersed and flocculated Kaolin at large range of stress levels (5.4 - 860 kPa) and reported some variation of both coefficients with stress level, however no relationship could be established from his results due to the scatter in the coefficients at the reported stress levels.

7.4.2. Data establishing the credibility of the tilting test

Normally consolidated resedimented Speswhite Kaolin tilted at 4°/min (Section 6.2) failed at a tilt of 22.5° to the horizontal when the tilting was carried out more than 24 hrs after the clay slurry was poured into the tank. This was established in 24 tests; the range of tilt angle results was ± 0.5°. According to conventional infinite slope stability analysis under drained conditions (Section 6.2) this tilt angle is equivalent to ϕ' as the sliding occurs on maximum stress obliquity planes (MSO) (Section 4.6). Data for the same clay tested in the Cambridge Simple Shear Apparatus and in the Cambridge True Triaxial Apparatus give ϕ' as 22.5° and 23° respectively for mean normal stress levels from 80kPa up to 600kPa (Borin 1973 and Pearce 1970). The mean stress level in the tilting tests was about 0.05kPa with the void ratio 6.1. This is an excellent correlation; drained conditions will be confirmed in Section 7.4.4, and further analyses in Sections 7.4.5 and 7.4.9 provide even stronger support for the use of conventional infinite slope analysis here.

Normally consolidated resedimented London clay tilted at 4°/min failed at a tilt angle of 20.5° 1 day after the clay slurry was poured into the tank. This was established in 5 tests; the range of tilt angle results was ± 0.5°. At this time the voids ratio was also 6.1. This corresponds to the 20.5° for ϕ' reported by Gibson (1953) for experiments carried out in the shear box and 21° for ϕ' obtained in the triaxial apparatus by Bishop et al. (1965) for normally consolidated London clay.

7.4.3. Data on the effect of aging through creep

In the next 6 days creep in the normally consolidated London clay reduced this voids ratio to 3.2, thereafter no further reduction could be discerned. Tilting at the same rate (4°/min.) after this reduction in voids ratio had taken place yielded 32° which does not correlate obviously with any known results for normally consolidated London clay. The increase might be attributed entirely to creep. However subsequent sections will show that only in a limited sense is this increase due to creep, and also that the rupture orientation has changed to the no extension direction associated with pre-failure straining with coincident axes of stress and strain increment (NED). Nevertheless the gain in shear strength due to creep is considerable and achieved in a mere 6 days, thereafter no further increase was observed in tests up to 22 days after sedimentation.

In contrast there was no evidence of creep in the Kaolin, that is no change in voids ratio and no increase in the shear strength which remained at 22.5° for tilting at 4°/min for tests up to 33 days after sedimentation. It proved invaluable in subsequent interpretation of data to have tested two clays one with marked creep effects and the other with none. One strong contrast in behaviour can be noted immediately; at any age greater than 1 day all unsalted London clay failures were on a very thin layer whilst the mass of the material remained entirely intact; whereas all unsalted Kaolin samples totally disintegrated on failing.

7.4.4. Data on the effect of rate of tilting

Three rates of tilting were adopted: (i) very fast $\approx 15°$/second (ii) 4°/min (iii) 3°/day. Normally consolidated Kaolin yielded these angles of tilt at failure, which have been interpreted as ϕ' values: (i) 34° at 1 day, $\underline{38°}$ at 2 days and thereafter (ii) 22.5° at all times (iii) $\underline{38°}$. The coinciding values of very fast and slow rates of tilt show that all rates resulted in drained shearing; no pore water pressures were generated before failure. The evidence of no pore water generation is shown by the results provided by varying the intermediate tilting rate between 10°/min. and 1°/min. (Figure 6.1) in order to measure the frictional strength component (minimum angle of tilt at failure, Section 7.4). Below 6°/min. it was found that the angle of tilt at failure was the lowest obtained by varying the rate of tilt and remained constant below this tilting rate and therefore any pore water pressure generation would result in a lower angle of tilt at failure for a K_0 normally consolidated clay.

Normally consolidated London clay failure angles were as follows: (i) 38° at 1 day and >38° thereafter (ii) 20.5° at 1 day and 32° at 6 days and thereafter (iii) 32°. Only the 20.5° and 38° at 1 day results were associated with disintegration of the mass at failure. The other failures were on very thin layers and may be associated with dilation which would be expected as creep halved the voids ratio in 6 days. The very high and somewhat variable angles of tilt achieved by very fast tilting after 1 day (up to 61°) cannot be shown to be due to achieving the same drained shear strength as very slow tilting tests after 1 day, even with full allowance for the maximum change in rupture layer orientation which will be inferred below. The change in voids ratio has had a further effect: permeability has decreased so that in these very fast tilts negative pore pressure is generated thus increasing shear strength. However evidence from tests on London clay described in Section 6.2, where the rate of tilt was varied between 1°/min. and 10°/min. indicated that a rate of tilt of 4°/min. allowed the drained shear strength to be measured. Additional evidence is provided by differing angles of tilt at failure for samples of different thickness. Two thicknesses of 25mm and 50mm were tested, and failure tilt angles for the thicker sample were 6° greater indicating a larger negative pore water pressure associated with a longer drainage path. Interpretation of these London clay results is aided by a careful comparison with the Kaolin data above. Kaolin tilted slowly never failed at a lower angle of tilt than that achieved in very fast tilt under drained conditions after 2 days. Yet in the London clay the tilt angle in slow tests was always lower than that achieved in a rapid test at 1 day which may be considered drained indicated by the variation of the intermediate continuous rate of tilt, Section 6.2. This deficiency is even more remarkable because creep has halved the voids ratio for the slowly tilted samples. The anomaly can be explained by proposing that at ages greater than 1 day the tilt angle at failure in the London clay for rates (ii) and (iii) was determined by the orientation of the no extension direction for continuum strain with coincident axes of stress and strain increment under undrained conditions (NED). Thus the failure tilt angle of 32° corresponds to a ϕ' of 38.7° which is satisfyingly close to the 1 day very fast tilt angle of 38° which in turn is equal to the angle achieved without creep by Kaolin after 2 days in both very fast and slow tilts. This interpretation will be supported by further data and a logical picture of the mechanisms determining rupture orientation will be presented in Section 7.4.9. There is a further consequence of this interpretation: the change in voids ratio is controlled in some way by the thixotropic strength at one day. When the creep is over this component of strength has been stabilised so that it can not be destroyed by pre-failure strain yet the overall maximum strength of the material has hardly increased at all despite the massive decrease in voids ratio.

7.4.5. Data from direct shear tests determining the effects of load and displacement control

Hypodermic pore pressure probes in the direct shear samples allowed the results to be stated reliably in the terms of effective stresses[1], Figure 4.4 shows that the positioning of the probe was not critical although every effort was made to locate on the central plane. The probes could also be used to determine a rate slow enough to assure drainage. The results of two tests for normally consolidated Kaolin are shown in Figure 7.1; these two samples have been consolidated to the same vertical normal stress, and then one has been sheared by constant lateral displacement whilst the other has been sheared by lateral load applied in small discrete increments at 24hr intervals. Assuming

[1] Although the measured negative pore pressures (below atmospheric) for the London clay test remained negative, tests on the Druck transducer (Appendix C) showed that when negative pore pressure is applied to the Druck transducer and slowly removed over a long period of time the zero reading of the Druck transducer shifted and remained negative. This is believed to be due the diaphragm in the transducer being unable to behave linearly at small negative pressures, however when either larger pressures (negative or positive) are applied or negative pressures removed quickly the transducer behaves linearly and no zero shift is seen. This phenomenon was not detected during positive pressure application, and the manufacturers warned that this zero shift may be seen when using the transducer to measure negative pressures.

that the ratio of shear stress to normal effective stress on a horizontal plane is tan ϕ' (MSO) the value of ϕ' for the two tests are 22.5° and 38.0° respectively. These values correlate excellently with the tilting test results even though the stress level is two orders of magnitude higher. The absence of negative pore water pressure in the displacement control tests and the higher shear resistance under load/stress control shows that the increased strength is thixotropic; the nature of thixotropy as revealed by this research will be analysed in Section 7.4.8. When the vertical normal effective stress was increased to 10kPa the stress controlled result returned to $\phi' = 22.5°$. This shows that the stress range in which thixotropy occurs in Kaolin is limited.

Similar tests on normally consolidated London clay at 5kPa vertical effective stress yielded ϕ' values of 29° and 39.5° for displacement and load/stress control respectively (Figure 7.2). For reasons to be given in Section 7.4.9 both ruptures are held to have oriented with maximum stress obliquity (MSO). The rise in ϕ' from 20.5° to 29° is due to creep, Section 6.3. The remainder is clearly thixotropic. In London clay the loss of the thixotropic component is delayed until the vertical effective stress has reached 8kPa and then occurs gradually; when the vertical effective stress is 20kPa ϕ' is still increased at 32° (Section 7.4.9) as compared with 29°(Figure 7.3). Full thixotropic strength is maintained up to 8kPa, and is in very close correlation with the tilting test results.

7.4.6. Data from overconsolidated samples

Figure 6.19(a) shows the results of tilting tests on Kaolin which have been resedimented and overconsolidated together with those of two load controlled direct shear tests on overconsolidated samples. A large range of rate of tilting was covered; no discrepancies due to the tilting rate suggesting undrained conditions are seen. A linear relationship defines the lower limit of shear strength and approaches the limit of normal consolidation with a failure tilt angle of 38° (MSO), the shear strength of normally consolidated Kaolin with the thixotropic component included. The limited direct shear data included come from samples which were not normally consolidated beyond the stress level for which there is a thixotropic component. Very limited data for samples normally consolidated to higher levels suggest that this leads to a large quantitatively corresponding drop in the overconsolidation strength. The considerable scatter shown in Figure 6.19(a) is due to the existence of several voids ratios in equilibrium at the same effective stress and overconsolidation ratio. If it is assumed that the lower limit line shown represents a correct relationship between voids ratio and overconsolidation ratio the other tilting test results can be corrected by adjusting the overconsolidation ratio accordingly, Appendix B. This has been done in Figure 6.19(b); however the improvement is perhaps misleading in that the variation in voids ratio which is independent of effective stress appears to be real at these very low stress levels. A similar type of variation has been reported by Been and Sills (1981), they attributed it to different densities of initial suspension prior to sedimentation (Section 7.4.1). In this case variations in setting up the downward seepage to cause consolidation seems a likely cause. The correction clearly has limited validity but nevertheless the same effect can be seen in London clay (Figure 6.26) and Kaolin with coagulating agent (Figure 6.30). On the other hand relationships between shear strength and voids ratio for overconsolidated tilted samples seems established (Figure 6.20). Figures 6.23 and 6.24 also show overconsolidated London clay tilting test results. The general form is the same as for Kaolin but the linear lower limit differs. The approach to the normal consolidation limit again coincides with that for normally consolidated shear strength when the thixotropic component is included at 32° (NED). The results of overconsolidated London clay suggest the existence of cohesive bonds, which may be proportional to the effective vertical stress, as angles of tilt exceeded 45°.

Results for tests on overconsolidated Kaolin in the tilting tanks where the consolidation and swell back periods were varied, Section 6.5.1.6, are shown in Figure 6.22. The results clearly indicate that a period of 2hrs is not sufficient for full consolidation to occur, whilst a similar period for swell back seems sufficient indicating a very limited application of short term consolidation and swell back.

7.4.7. Data from non standard sample preparations

For samples where distilled and de-ionised water was used instead of tap water no effects were noted. However the addition of salt (to imitate sea water) or a coagulating agent to Kaolin suspension yielded some interesting results.

(i) Addition of salt

As already indicated (Section 6.5.1.3) Salt was added to the Kaolin suspensions to imitate sea water (33-35 gms/litre). Sedimentation and completion of self weight consolidation were slowed. However the void ratio achieved 24hrs after pouring the slurry into the tank was the same as for Kaolin mixed with London tap water ($e=6.1$), Section 7.4.1. No creep was observed and the void ratio remained at 6.1.

For normally consolidated Kaolin failure angles were as follows: (i) 38° at 1 day and >38° thereafter (ii) 22.5° at 1 day and thereafter (iii) 32°. Only the 22.5° and 38° at 1 day failures agree with the results obtained from mixing with ordinary London tap water. The other failures are on very thin layers, indicating the possibilty of local dilation. After 1 day angles of tilt at failure in very fast tests are in excess of 38°; indicating negative pore water pressure generation, a new feature for Kaolin. Again as with London clay it can be proposed that the failure was determined by concentrated pre-failure shear strain on the orientation of the no extension direction for continuum strain with coincident axes of stress and strain increment (NED). Thus a tilt angle at failure of 32° is equivalent to $\phi' = 38.7°$.

Load controlled DSA results on normally consolidated Kaolin at 5kPa with a salt concentration of 35 gms/litre also yielded $\phi' = 38°$. Failure occurs with a similar mechanism where ruptures are held to have oriented with no-extension direction associated with coincident axes of stress and strain increment during undrained shear (NED), Section 7.4.9 (tests 49 and 50, Table 6.5).

Paradoxically the effect of salt on London clay appears to reduce ductility with time. It also accelerates creep so that it is virtually complete in one day and therefore resulting in the preservation of thixotropic strength (Section 7.4.3). Failure angles were as follows: (i) 38° at 1 day and >38° thereafter (ii) 32° at 1 day and 38° at 2 days and thereafter (iii) 38°. Only the 32° at 1 day result was on a very thin layer as the clay was still sufficiently ductile and therefore influenced a failure on the NED. The other failure were associated with disintegration of the mass at failure. The results are summarised in Table 6.8.

(ii) Addition of coagulating agent

Addition to Kaolin of the coagulant agent, polyacrylamide, in concentration of 0.0045 gms/litre and 0.0023 gms/litre had interesting effects which were not influenced by the range of concentration quoted. The voids ratio of the samples with added coagulant increased to 8.9 yet shear strengths also rose very appreciably; there was no creep. Results for tilting tests at rates defined in Section 7.4.4 on normally consolidated samples were as follows: (i) angle of tilt at failure in the range 53° to 60° (ii) angle of tilt at failure 39°-40° (iii) this category was altered a little with two subdivisions: (a) rotation at 3°/day started at an initial tilt of 30° or less, (b) rotation at 3°/day started at 32° initial tilt. For (a) the failure angle of tilt was 48° whilst (b) failed slowly within 24 hrs at 32°; there was a further observation of interest: in connection with (a) cracks appeared in the clay at a tilt of 34° which were oriented at 21° to the bedding plane of the clay. These are illustrated in Figure 6.18. The failure at 32° and the cracks appearing at 34° suggest that shear at these tilts is associated with a minimum shear strength with no thixotropic component. The data can be given a coherent explanation which fits within the limits of attainable accuracy as follows: the failure at 32° is by undrained sliding on a no-extension direction which is slow and interrupted because it generates a negative pore water pressure which arrests the sliding until more water is sucked into the thin shear layer thus weakening it to give more sliding and so on. On this basis $\phi' = 38.7°$ which fits well enough with the 4°/min result if this is a maximum obliquity slide ($\phi' = 39°$-$40°$). The cracks seen at a tilt of 34° (see Figure 6.18) can then be interpreted as Coulomb ruptures on maximum obliquity planes with $\phi' = 42°$ occurring when there is undrained sliding on a no extension direction parallel with the angle of tilt, Section 4.6. These ruptures are apparently visible in coagulated Kaolin as coagulation seems to magnify this phenomenon which may also occur in Kaolin in salt water and London clay, at a much smaller scale. This can be seen as representing a partial failure when almost all the thixotropic strength has been destroyed (39°-40° was recorded for the tilt rate which has eliminated all thixotropy in previous tests). The full failure was probably prevented by the development of negative pore water pressure along the no extension line rupture layer, which stopped the shearing and allowed thixotropic strength recovery. Subsequently the intervals between increments were sufficient to allow full thixotropic recovery. The correlation achieved by considering the orientations of the short Coulomb ruptures is illustrated in Figure 6.18. Section 7.4.9 will present an overall interpretation of the orientation of rupture layers using new data as well as that already discussed. Possible evidence of positive dilation at failure is found in the high failure angles of tilt reported for very fast tilting (53°-60°); these comfortably exceed the rate (iii) results and suggest that negative pore water pressure was generated in this Kaolin in spite of the high voids ratio, whilst there was no evidence of generation in Kaolin mixed with London tap water at a lower voids ratio (see Section 7.4.4).

Figure 6.27 shows the tilting failure angles for this clay when overconsolidated. The lower limit line again intersects the normally consolidated limit at the tilting angle found for shear strength including the thixotropic component.

These findings might not be repeated for London clay or any clay that exhibited creep; further work is needed.

7.4.8. Deductions from data on the mechanism causing thixotropy

Thixotropic shear strength is conceived as a component of shear strength which is destroyed by strain at a constant or increasing rate with respect to time; this means that it will never be recorded in displacement controlled shear tests on normally consolidated clays. The direct shear data reported in Section 7.4.5 bears this out in that for both clays the load/stress controlled tests yield much higher shear strengths at the 5kPa stress level. Above this stress level the differences in shear strength measured in the two types of direct shear test decrease, rapidly for Kaolin and steadily for London clay. The phenomenon of thixotropic shear strength is confined to low stress levels. It is related to structural features of the clay which appear to be destroyed by increasing stress level such as coagulations of particles which enclose large voids. Various suggestions and measurements have been made (Utomo and Dexter 1981, V.I. Osipov et al. 1984).

The ultra low stress levels of the tilting tests have allowed the measurement of thixotropic shear strength through two orders of magnitude of stress to the level of the direct shear tests. In the Kaolin there is very clear correlation, the angles of shearing resistance at both stress levels correspond very well: 38.0° including thixotropy and 22.5° excluding thixotropy. The high shear strength found in the very rapidly continuously tilted bed needs considering; if no pore water pressure was generated, and the failures were stress controlled, $\phi' =$ 38° is found, and this matches exactly that found for all incremental tests when including the thixotropic component. Arthur (1991) has suggested this is entirely consistent with thixotropy provided no inelastic strain was generated before failure in the very rapid tilting and the Poison's ratio is 0.5 as for instance in rubber.

Such elasticity over such a large shear stress range is only possible if the straining process is extremely viscous so that the strain that can occur in the 2.5 second tilt to failure is within a strain threshold for damage to soil thixotropic structure. Evidence of such threshold and strong strain viscosity can perhaps be seen in the stress paths of Figure 6.3(a). This relies on pore water pressure generation as there are no direct measurements of strain; at first shear stress increases without any appreciable pore water pressure generation (τ = 0.2kPa the increment adopted of the DSA tests) and then the faster test which reaches failure in 75 seconds shows a much slower initial rate of pore water pressure generation (with respect to τ) than the test failing in 600 seconds. The first feature may be taken as evidence of a stiff elastic strain range and the second for a very strong viscous restraint on strain during this range and then during the degradation of the thixotropic soil structure, because on the basis of drainage time available the rates of pore water pressure generation should have been reversed for these two tests. This interpretation treats the stress ratio displacement plot in Figure 6.2 as unrelated to the stress strain relationship at small strains.

Failure of the thixotropic structure is governed by a stress ratio regardless of failure mechanism (MSO for tilting tanks, NED for tilting cylinder). The pseudo-frictional nature of the thixotropic strength component through two orders of magnitude change in effective stress shows that all the strength can be represented by ϕ', without defining the nature of the thixotropic bonding. Rapid tilting after different number of days at rest showed no increase in thixotropic strength after one day for Kaolin. With $\phi' = 38°$ there might be a potential for dilation. Figures 6.11(a) and 6.12(a) shows the generation of negative pore water pressure following a rise in positive pore water pressure which raised the stress ratio close to the failure level, but there is no measurement of negative pore water pressure at failure as would be expected.

In London clay the data is complicated by the influence of large creep components; much the largest occurs at the ultra low stress levels in which the voids ratio is halved in 6 days, but there is still a reduction in the void ratio of 0.3 in 6 days at the 5kPa level of the direct shear tests. Nevertheless there are striking parallels. At 1 day after sedimentation the voids ratio is 6.1 as for Kaolin and very rapid tilting leads to the same failure angle 38°; there is an implication of similar structures and mechanics in the two clays but thereafter creep at this ultra low stress level is severe and appears to convert all the thixotropic component to shear strength unaffected by strain rate. In contrast at the 5kPa stress level only a proportion is converted by the smaller creep effect. The concept of thixotropy being associated with dilatancy may be supported by the observation that Kaolin, when treated with a coagulating agent which severely increased its voids ratio, demonstrated an apparent capacity to develop a significant negative pore water pressure which was parallelled by London clay when tilted very fast at ages greater than 1 day after sedimentation. The behaviour of London clay was less surprising because creep had imposed a large reduction in voids ratio but the reverse was true for the effect of the coagulant on the Kaolin so the significance of overall voids ratio appears minimal in relation to structure.

The lines of minimum tilting angle for failure with overconsolidation ratio shown in Figures 6.19(a), 6.23 and 6.27 all indicate a lower limit of the increase in shear strength as overconsolidation decreases and normal consolidation is approached: this limit is the normally consolidated shear strength including the full thixotropic contribution as a stable component. There seems to be an implication here that the first effect of voids ratio decrease under applied effective stress is to stabilise the structural features that are associated with thixotropy so that these can survive strain to failure without special loading restrictions. Indeed in the ultra low stress level the creep in the London clay seems to have been controlled by the thixotropic component before it; however this thixotropic component ceases to exist beyond a vertical consolidation stress of 5kPa in the case of Kaolin, and starts to reduce beyond a vertical consolidation stress of 8kPa in the case of London clay producing a ϕ' of 32° at a vertical consolidation stress of 20kPa. Further evidence of this phenomenon is provided by the displacement controlled DSA results on overconsolidated Kaolin (within the thixotropic range) sheared at a rate slow enough for pore pressure not to develop, (Tests 41 and 42, Table 6.4), so pore pressure measurement accuracy is not essential, Section 6.2. This yielded a ϕ' of 39°, and for reasons to be given in Section 7.4.9 ruptures are held to have oriented with the no-extension direction associated with coincident axes of stress and strain increment during undrained shear (NED).

7.4.9. Rupture layer orientation

The types of shear test in this study do not allow the direct determination of principal stress directions at failure, but, as reported in Section 7.4.2, reliable ϕ' values have been determined for the two clays at Cambridge and Imperial College in displacement controlled apparatuses in which principal stresses were fully defined.

For Kaolin and young London clay, values correlate excellently with tilting and direct shear data when taking the mechanisms of failure as Coulomb slip on planes of *maximum stress obliquity* (MSO), (Table 6.7, top section); there are only two exceptions in direct shear for subsequent explanation. Coulomb failure occurs over a surface on which there is maximum stress obliquity and which may act as a boundary between a yielding or failure zone and non-yielding zone. In the tilting tests all soil above this boundary will fail and this is shown by the mass disintegration of the soil in the tilting slope failure; this might be called brittle behaviour. Figure 6.2 contrasts the shear stress - displacement curves for Kaolin in direct shear at different displacement rates; one of these represents the exceptions mentioned above. These exceptions are for the slow rate of displacement in which large pre-failure strain allows a simple shear to develop in a layer aligned with a no-extension direction in undrained shear (no volume change at failure, Figure 6.4) with coincident axes of stress and strain increment (NED), thus providing a failure mechanism (James and Bransby (1970), Roscoe (1970)); this might be called ductile behaviour. However the intermediate displacement rate of 0.1mm/min. yielded a failure on a plane in between MSO and NED; this can be explained by the fact that failure can be influenced by both strain and stress state and subsequently critical state is reached with a rupture layer is at an intermediate orientation. London clay behaviour (Table 6.7, 2nd section) confirms this analysis, at one day the tilting test samples fail by disintegration thereafter all tilting samples remain intact at failure as the upper mass slides on a thin pre-strained and weakened layer. The form of failure, disintegration or intact sliding has been found an infallible guide to the orientation of the failure (MSO or NED) in normally consolidated clay. In overconsolidated tilting tank samples the exposed sliding surfaces were very smooth for London clay but rough and irregular for Kaolin. These observations and the relationships shown in Figures 6.19(a) and 6.23 suggest that the NED orientation for London clay and the MSO for Kaolin were maintained.

Rupture Layers correlate excellently on the basis of the proposition that the failure criterion can be represented by a given angle of friction alone and that these ruptures orient either parallel to planes of maximum stress and or no extension directions with coincident axes of stress and strain increment. The no extension directions can then be related to principal stress directions under conditions of constant volume shear (zero angle of dilation); in saturated clays shear under undrained conditions and at the critical state, Section 4.6. Failure will occur at constant volume, the condition is readily identified without precise local strain measurements. The proposition that the clays behave entirely as frictional materials (implying $c' = 0$ or is proportional to the effective vertical stress and subsumed into ϕ', Section 4.6) at failure even when the thixotropic strength is present requires the break up of the thixotropic bonding to lead to always fails on a thin layer with an intact mass sliding above it, whereas the normally consolidated clay mass disintegrate at failure. Normally consolidated Kaolin does not creep and so has no capacity for stable pre-failure dilation. Thus the tilting tank tests on normally consolidated Kaolin are uninfluenced by pre-failure strain and fail on maximum stress obliquity planes.

In the tilting cylinder test the end restraint is completely removed and the thinness of the layer of material trapped below the sliding plane defined by the bottom of the cylinder further encourages a thin layer of pre-failure simple shear as in the long shear box described by Jewell & Wroth (1987). Here the influence of pre-failure strain is enhanced by the boundary conditions for even the relatively brittle Kaolin samples to rupture on no extension directions.

The imposed conditions in the Direct Shear Apparatuses are less restrictive so that either stress or strain control at failure seems likely to be possible. Tests in this apparatus have shown that either brittle or ductile behaviour can be induced depending on the stress path. Brittle behaviour being unaffected by prior strain leads to rupture on maximum stress obliquity. This brittleness is brought about by the rapid rise in pore water pressure during undrained displacement controlled shear, and can be brought about by thixotropic regain in incremental loading.

MSO occurs frequently in the tilting tank because the lower tank end restrains simple shear. An effective staged local softening movement which vitiates this restraint involves first shear in a thin layer at constant volume eventually generating negative pore water pressure, second the arrest of this shear through the strengthening action of the negative pore water pressure and third the dissipation of the pore water pressure by water inflow to the localised layer of high shear whilst there is a general regain of thixotropic strength. This sequence has to be repeated in pre-failure conditions so that the rate of loading must be sufficiently slow and the clay sufficiently ductile. This phenomenon is also observed for Kaolin with salt, where the salt results in the reduction of the permeability which can be seen in the very fast standard tilting tests, on normally consolidated Kaolin with salt, failing at large angles due to negative pore pressure generation brought about by the reduction in permeability as seen with normally consolidated London clay tested at the fast rate in the standard tilting tanks after creep has occurred (Section 7.4.3). A demonstration of the effect of end restraint is provided by tests carried out in the shear cylinder, Section 6.4.2. Results for dense and strongly dilatant Leighton Buzzard sand, voids ratio 0.52, are as follows: (i) normal tilting tank failed at a tilt of 48°, Dalili (1991), (ii) tilting cylinder failed at a tilt of 41.5° ± 0.5° which is along a coincident no extension direction for the expected angle of dilation 20°. The two types of test on the normally consolidated Kaolin with and without salt yield corresponding angles 22.5° and 20°, whilst Kaolin tilted at 3°/day failed at 32° in the shear cylinder and at 38° in the tilting tanks; the difference approximately fits the same interpretation given shear with no volume change. Finally normally consolidated London clay with and without salt at ages greater than six days showed no difference in the angle of tilt for the two types of tilting test: both failed at a tilt of 32°(NED).

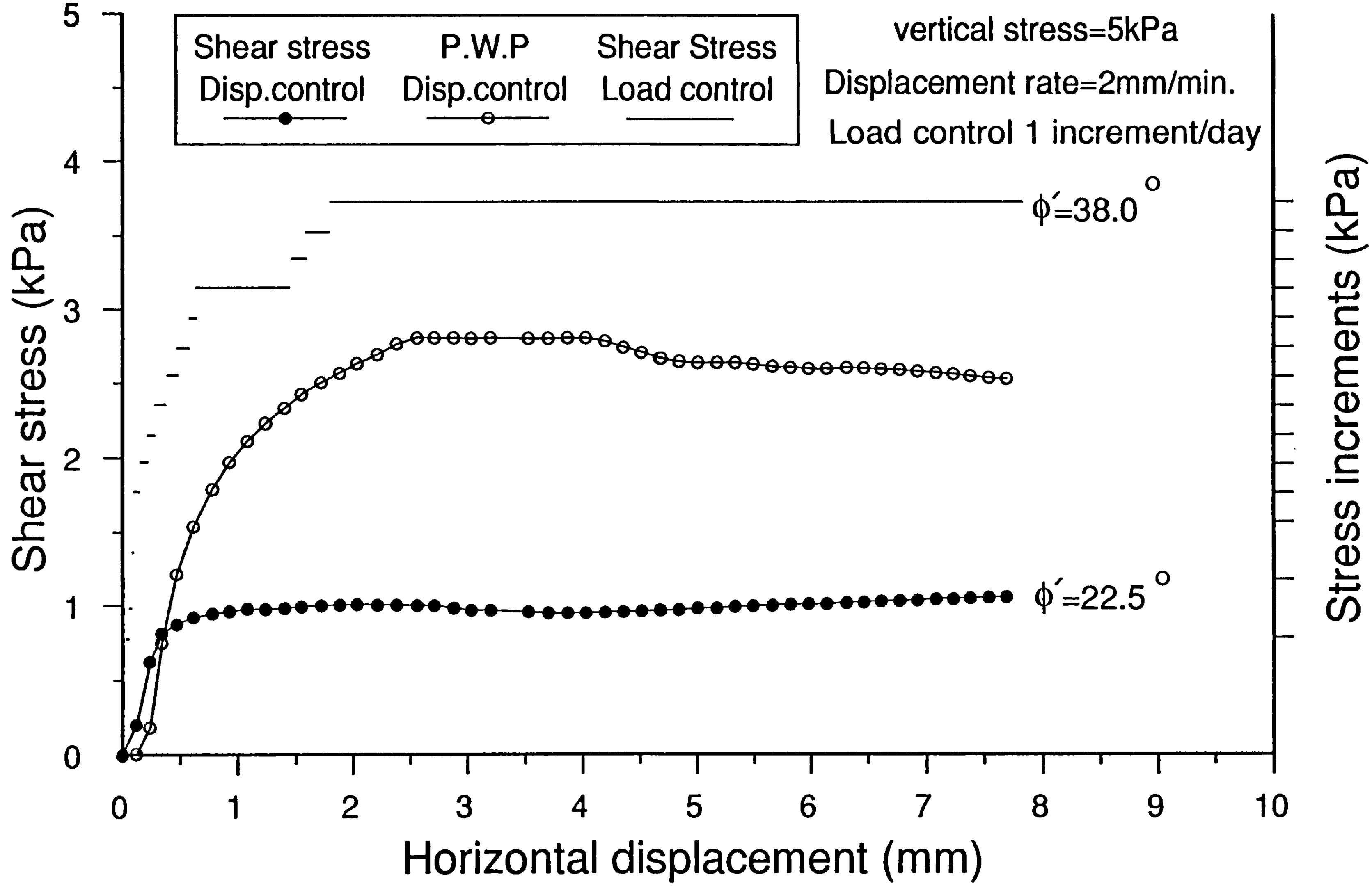


Figure 7.1 Comparison between displacement control and load control direct shear tests on Kaolin

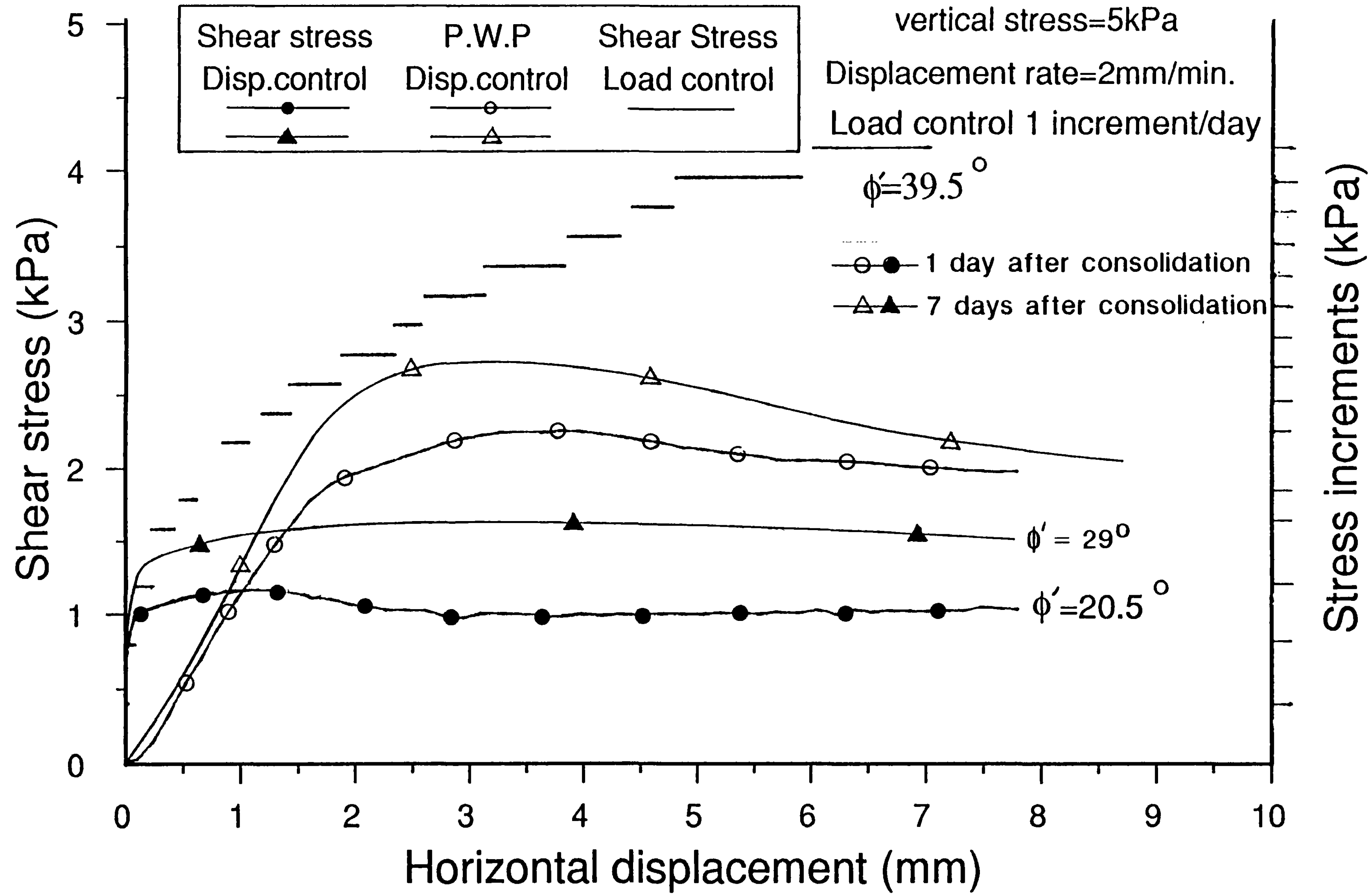


Figure 7.2 Comparison between displacement control and load control direct shear tests on London clay

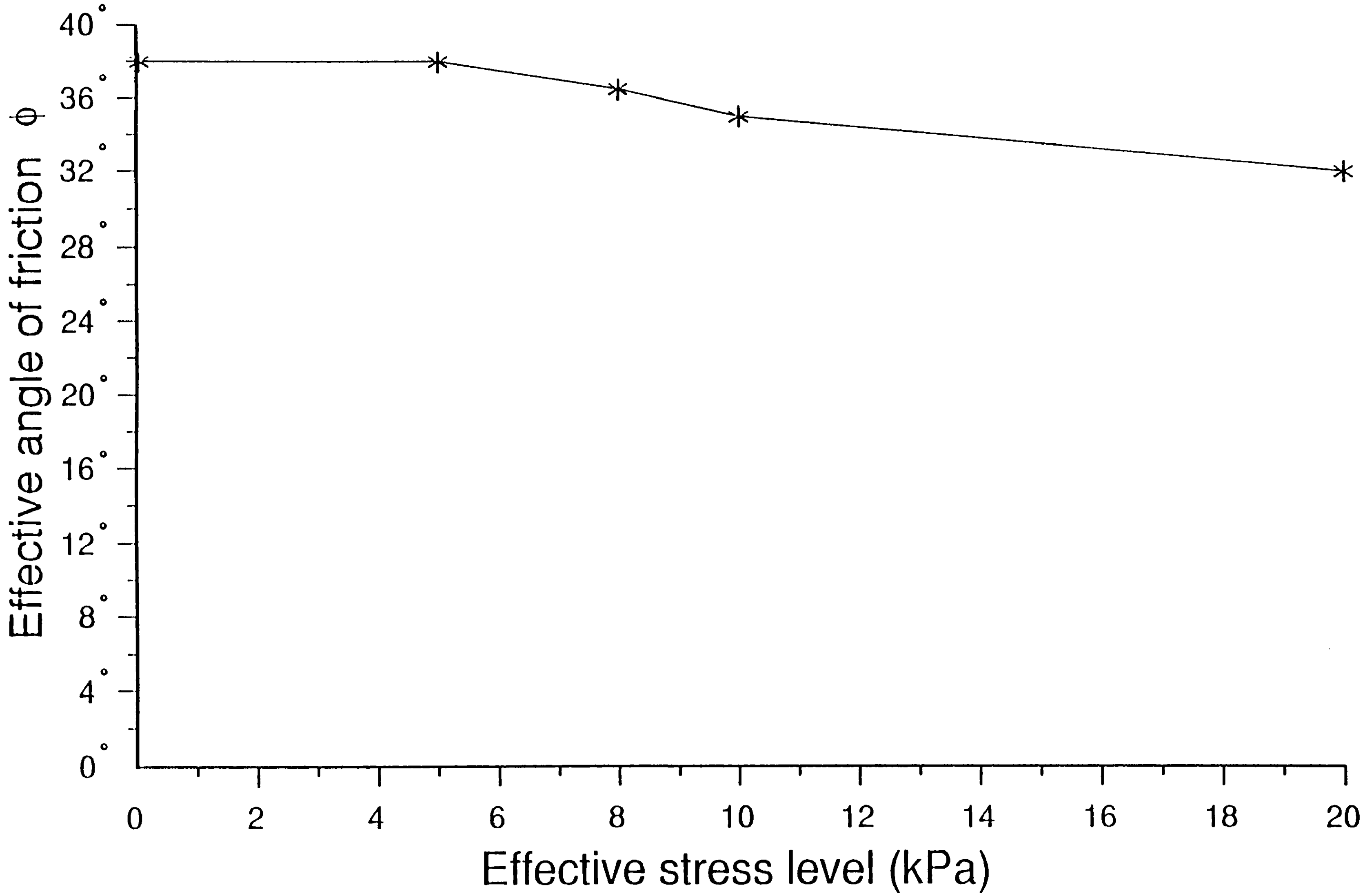


Figure 7.3 Reduction of thixotropic strength with effective stress level for London clay

CHAPTER 8

CONCLUSION AND RECOMMENDATIONS FOR FURTHER WORK

8.1 Conclusions

Three major pieces of apparatus were used in this research programme (DSA, SSA and the tilting tanks) which allowed tests to be performed on soil at low and very low stress levels. Modifications carried out on the standard DSA and SSA have been relatively easy and inexpensive considering today's technology. The DSA has allowed a large versatility in the testing conditions, both displacement and load control tests can be carried out with great accuracy by incorporating measurements using simple electronics and computer methods which are easy to operate, as well as accurate variations of displacement rates. The most important improvement is provisions for internal pore pressure measurements. The SSA has a great potential of use at the same stress levels used in the DSA either when the vertical stress measurement is improved, or major modification carried out to allow for pore pressure measurements as used by Dyvik et al. (1987). The tilting tanks were much cheaper and very easy to modify, and as many as twenty were used simultaneously in this research programme. Normally consolidated and overconsolidated samples can be prepared in the standard tilting tanks at very low stress levels (0.05 kPa). Whilst only normally consolidated samples were prepared for the shear cylinder test, it is feasible to use it for overconsolidated samples. In all tilting tests the rate of testing can be varied easily and even very fast smooth tilting can be performed ($\approx 15°$/second).

The sedimentation / consolidation behaviour in the tilting tanks for both clays showed that there was no unique void ratio at zero effective stress, initial dispersion of the clay slurry resulted in a denser clay bed, whilst the addition of a coagulating agent resulted in a less dense clay bed, and the passage of time since sedimentation / consolidation often resulted in a stronger clay bed which is in agreement with observations made by others (Been and Sills (1981), Edge and Sills (1989)).

Simple correlations have shown that the lower bounds of drained shear strengths, measured, for both normally consolidated and overconsolidated clay, in the tilting tanks and shear cylinder relate, directly to those measured at effective stress levels three orders of magnitude higher in the DSA. The effective angles measured for Kaolin and London clay were 22.5° and 20.5° respectively. These have also been established at Cambridge using the Simple Shear and the True Triaxial Apparatus for both clays (Borin (1973) and Pearce (1970)), and at Imperial. Upper bound shear strength at these low stress levels has a large additional thixotropic component which was quantified by applying very fast loading or discrete load increments at specific rates, so that the full thixotropic strength would not be destroyed due to constant or increasing rates of strain loading occurring under load applications, and in the latter case recovery of full thixotropic strength could take place between increments. This component of strength was shown to be pseudo-frictional in that it was proportional to effective normal stress and resulted in an increase of the effective angles of friction for normal consolidation to at least 38° for both Kaolin and London clay. This increased angle of friction was maintained from 0.05kPa to 5kPa normal effective stress. This fact and other aspects of behaviour noted in the experiments suggest that thixotropic strength may results from unsustainable dilatancy as flock bonds break irregularly. Any small degree of overconsolidation made this dilatancy sustainable within this stress range which allowed displacement control to quantify this component, and a linear relationship has been established which defines the lower limit of overconsolidated shear strength for both Kaolin and London clay.

The correlation with existing shear strength measurements and the wide variety of tests conducted has yielded solid interpretations of tilting test data in relation to rupture layer orientation. In both the DSA and the tilting tanks rupture occurs either on maximum stress obliquity planes or along the no extension directions associated with coincidence of axes of stress and strain increment. However some tests in the DSA (displacement control tests run at the intermediate rate) indicated that rupture can occur in between MSO and NED.

The fact that shear parameters measured were either equal or higher than those measured for resedimented clay samples at higher stress levels is especially encouraging. The lack of undrained testing is not considered to be a serious practical limitation on usefulness because even in the normally consolidated material with thixotropic strength there was no evidence of positive pore water being dominant, whilst there was evidence of significant negative pore water pressures being developed.

The absence of creep beyond 7 days after sedimentation was gratifying in that there is a possibility of simulating natural clay formation in a reasonable time frame. Burland (1990) supports this interpretation not withstanding the views expressed by Casagrande (1932).

8.2 Recommendations for further work

The apparatus as used in this research programme and further modifications could be utilised to perform further tests under a variety of conditions to extend the understanding of the behaviour of clays at low and very low effective stress levels. The following are some suggestions for further work:

Direct Shear Apparatus

The existing DSA has been used very successfully in this research programme. Further modifications could be carried out to allow for both displacement control and load control tests to be carried out sequentially during each test by switching from displacement to load control or visa versa. Displacement control and load control cyclic tests could easily be performed in the DSA with some minor modification on both the apparatus and the computer programmes responsible for control and data acquisition.

More data on both normally consolidated and overconsolidated clay are required in the DSA at the higher effective stress levels (5kPa), particularly under load control mode, to investigate the mechanism causing thixotropy and therefore quantifying more accurately both positive and negative pore pressures. Further work is also required to allow a better understanding of failure mechanism in both the displacement and load control modes.

Standard Tilting Tanks

The standard tilting tanks have proved to be an invaluable testing apparatus for clay at very low effective stress levels. However the use of this apparatus in this research programme has not exhausted its versatility. The effect of vibration on a clay bed can be very easily studied by introducing known vibration to the tank, and even preforming load control cyclic tests under different manners of loading by modifying the tilting mechanisms to allow for the tanks to be tilted in a variety of directions. Computer control could also be adopted to allow for more accurate and continuous tilting. Pore pressure profiles at the bottom of the clay bed during further consolidation could be established by using more than one stand pipe placed at different locations over the bottom area of the tanks.

Shear Cylinder

Simple modifications would allow overconsolidated samples to be prepared and tested in the shear cylinder as in the standard tilting tanks, and similar improvements as those described above for the standard tilting tanks could be adopted to allow for more accuracy and versatility of testing conditions.

Simple shear apparatus

The existing SSA used in this research is unsuitable for testing soils at the low stress levels adopted in this research, especially when performing constant height tests. An NGI SSA would be more suitable and when slightly modified could be used to perform load control tests as in the DSA. Modification to allow pore pressure measurements is possible as used by Dyvik et al. (1987).

There may be ways of accelerating the aging effects due to chemical change at these stress levels by using heat which could easily be adopted in the tilting tanks as well as in the DSA.

A variety of clays could be easily investigated in these apparatuses as the procedures are not labour intensive, and so would allow a data bank covering diverse soft seabed soils to be established.

REFERENCES

Airey, D. W. and D. M. Wood (1986). Pore pressures in simple shear. Soils Fdns 26, No. 2, pp. 91-96.

Airey, D. W. (1987). Some observations on the interpretation of shear box results. Report No. CUED/D, Soil/TR 196, University of Cambridge.

Airey, D. W. and D. M. Wood (1987). An evaluation of direct shear tests on clay. Géotechnique vol. 37,1,25-35.

Airey, D. W. (1984). Clays in circular simple shear apparatus. Ph.D. thesis, University of Cambridge.

Ajaz, A. and R. H. G. Parry (1975). Stress - strain behaviour of two compacted clays in tension and compression. Géotechnique 25,3, pp. 495-512.

Andersen, A., T. Berre, A. Kleven and T. Lunne (1979). Procedures used to obtain soil parameters for foundation engineering in the North Sea. Mar. Geotech. 3 (3), 201-66.

Arthur, J. R. F. (1991). Personnel communications.

Arthur, J. R. F. and B. Menzies (1972). Inherent anisotropy in sand. Géotechnique, vol. 22,1, pp. 115-128.

Arthur, J. R. F. and T. Dunstan (1982). Deformation and failure of granular materials. Int. Union of Theoretical and Applied Mechanics Symp. on Deformation and Failure Granular Materials DELFT.

Arthur, J. R. F. and A. Assadi (1977). Ruptured sand sheared in plane strain. Proc. 9th Int. Conf. SMFE, Tokyo, vol. 1, pp. 19-22.

Arthur, J. R. F., T. Dunstan, A. Dalili, O. Ogunbekun and J. Pulsford (1987). The strength of dense sand sheared in plane strain at low stress level. Draft of Int. Rpt., Civ. Eng. Dept., University College London.

Assadi, A. (1975). Rupture layers in granular materials. Ph.D. thesis, University of London.

Baguelin, F., J. F. Jezequel, E. le Mee and A. le Mehaute (1972). Expansion of cylindrical probes in cohesive soils, J. Soil Mech. Foundat. Div., ASCE 98 (SM11), 1129-42.

Balla, A. (1957). The resistance to breaking out of mushroom foundations for pylons. Proc. 5th Int. Soil Mech. Foundat. Engng, Paris, 1, 569-76.

Beard, R. M. and H. J Lee (1982). Holding capacity of direct embedment anchors. Proc. Civil Engineering in the Oceans III, ASCE, vol. 1,470-85.

Been, K. (1981). Nondestructive soils bulk measurements by X-ray attenuation. Geotechnical Testing Journal, vol. 4, No. 4, pp. 169-176.

Been, K. and Sills. G. C. (1981). Self-weight consolidation of soft clays: an experimental and theoretical study. Géotechnique 31, No. 4, 519-535.

Biot, M. A. (1956). Theory of elastic waves in fluid-saturated porous solid. J. Acoust. Soc. Am., 28, 168-91.

Bishop, A. W. and J. A. Green (1965). The influence of end restrained in the compression strength of a cohesionless soil. Géotechnique, vol. 15,3, pp. 243-66.

Bishop, A. W. and D. J. Henkel (1962). The measurement of soil properties in the triaxial test. 2nd ed., Edward Arnold (Publishers Ltd., London).

Bishop, A. W., J. A. Green, V. K. Garga, A. Anderson and J. D. Brown (1971). A new ring shear apparatus and its application to the measurement of residual strength. Géotechnique, vol. 21,4, pp. 273-328.

Bishop, A.W., Webb, D.L. and Lewin. P.I.(1965). Undisturbed samples of London clay from the Ashford Common shaft: Strength-effective stress relationship. Géotechnique 15, 1-31.

Bjerrum, L., Simons and I. Torblaa (1958). The effect of time on the shear strength of soft marine clay. Proc. Brussels Conf. on Earth Pressures Problems, vol. 1, pp. 148-158.

Bjerrum, L. (1967). Engineering geology of Norwegian normally consolidated marine clays as related to settlement of buildings. Géotechnique 17, 81-118.

Bjerrum, L. and A. Landva (1966). Direct simple shear tests on Norwegian quick clay. Géotechnique, vol. 16,1, pp. 1-20.

Borin, D.L. (1973). The behaviour of saturated Kaolin in the simple shear apparatus. PhD thesis, University of Cambridge.

Bowden, R. K. (1988). Compression behaviour and shear strength characteristics of a natural silty clay. Ph.D. thesis, University of Oxford.

Burland, J. B. (1990). On the compressibility and shear strength of natural clays. Géotechnique 40, No. 3, 329-378.

Briaud, J. L. and B. Meyer (1983). In-situ tests and their application in offshore design. In Geotechnical practice in offshore engineering, S. G, Wright (ed.), 244-66. ASCE.

Briaud, J. L., L. Tucker and H. M. Coyle (1982). Pressuremeter, cone penetrometer and foundation design. Texas A&M University short course notes, April 1982.

Briaud, J. L. (1980). In-situ tests to measure soil strength and soil deformability for offshore engineering. Report 1080-1, Centre for Marine Geotechnical Engineering, Civil Engineering Dept., Texas A&M University, College Station, TX, USA, October, 1980.

Brucy,F., J.B. Fay and J. Marignier (1982). Offshore self-boring pressuremeter: the LPC/IFP probe. Symp. on the Pressuremeter and its Marine Applications, Paris: Edition Technip.

Burgers, J. M. and G. W Scott-Blair (1948). Report on the principles of rheological nomenclature. Joint Committee on Rheology of Int. Council of Scientific Unions. Proc. Int. Rheol. Cong., Amsterdam.

Burnet, A. D. and P. G. Fookes (1974). A regional engineering geological study of the London clay in the London and Hampshire Basins. Q. J. Engng Geol., vol. 7, pp. 257-295.

Casagrande, A. (1932). The structure of clay and its importance in foundation engineering. J. Boston Soc. Civ. Engrs. 19, No. 4, 168-209.

Casagrande, A. (1975). Liquefaction and cyclic deformation of sands: a critical review. Proc. 5th Pan American Conf. SMFE, Buenos Aires, vol. 5, pp. 80-133.

Crawford, C. B. (1959). The influence of rate strain on effective stresses in a sensitive clay. ASTM, Special Technique Publication, No. 254, pp. 36-61.

Crawford, C. B. (1965). Resistance of soil structures to consolidation. Can. Geotech. J., vol. 2,2, pp. 90-115.

Dalili, A. (1990). Disparate particulate materials subjected to principal stress rotation. PhD thesis, University of London.

De Ruiter, J. (1981). The static cone penetration test: state of the art report. Proc. 2nd Eur. Symp. Penetration Testing, Amsterdam, 389-405.

Doyle, E. H and B. McClelland and G. H. Fergusson (1971). Wireline vane probe for deep penetration measurements of ocean sediments strength. OTC, Houston, Paper No. 1327.

Duncan, J. M. and H. B. Seed (1967). Correction for strength test data. JSMD Proc. ASCE, vol. 93, SM5, pp. 127-137.

Dyvik, R., Berre, T., Lacasse, S. and Raadim, B. (1987). Comparison of truly undrained and constant volume direct simple shear tests. Géotechnique 37, No. 1, 3-10.

Edge, M. J. and G. C. Sills (1989). The development of layered sediment beds in the laboratory as an illustration of possible field processes. Q. J. Engng. Geol. 22, Part 4, 271-279.

Elder, D. and G. C. Sill (1985). Thickening and consolidation of sediments due to self weight. Eng. Found. Conf. Sea Island GA.

Freundich, H (1935). Thixotropy. Hermann et Cie, Paris.

Fukushima, S. and F. Tatsuoka (1984). Strength and deformation characteristics of saturated sand at extremely low pressures. Soils and Foundation, vol. 24,4, pp. 30-48.

Geonor (1986). Description and instructions for use of Direct simple shear apparatus model h-12. Geonor A/S, Oslo.

Gibson, R. E. (1953). Experimental determination of the true cohesion and true angle of internal friction in clays. Proceedings of the 3rd International Conference on Soil Mechanics. 1, 126-130.

Green, G. E. (1971). Strength and deformation of sand measured in an independent stress controlled cell. Proc. Roscoe Memorial Symp., G. D. Foulis and Co., Henley and James, England, pp. 285-323.

Hambly, E. C. (1969). A true triaxial apparatus. Géotechnique, vol. 19,2, pp. 307-309.

Hampton, L. (1974). Physics of sound in Marine sediments. New york: Plenum Press.

Hughes, J. M. O., C. P. Wroth and D. Windle (1977). Pressuremeter tests in sands. Géotechnique 27, 455-77.

Iverson, K. and J. Moum (1974). The paraffin method - triaxial testing without rubber membrane. Géotechnique, vol. 24,4, pp. 665-670.

James, R.E. and Bransby (1970). Experimental and theoretical investigations of a passive earth pressure problem. Géotechnique 20, No. 1, 17-37.

Jamiolkowski, M., C. C. Ladd, J. T. Germaine and R. Lancellotta (1985). New developments in field and laboratory testing of soils. Theme lecture 2, 11th Int. Conf. Soil Mech. Foundat. Engng, San Francisco 1, 57-153.

Jewell, R. A. and C. P. Wroth (1988). Discussion on the engineering application of direct and simple shear testing. Géotechnique, vol. 38,1, pp. 146-148.

Jewell, R. A. and C. P. Wroth (1987). Direct shear tests on reinforced sands. Géotechnique, vol. 37,1, pp. 53-68.

Jewell, R. A. (1988). Unreinforced direct shear tests. Report No. OUEL 1719/4/88, University of Oxford.

Kirkpatrick, W. N. and B. J. Belshaw (1968). On the interpretation of the triaxial test. Géotechnique, vol. 18,3, pp. 336-350.

Kirkpatrick, W. N. and J. S. Younger (1970). Strain conditions in compression cylinder. JSMD, Proc. ASCE, vol. 96, SM5, pp. 1683-1695.

Kjellman, W. (1951). testing the shear strength of clay in Sweden. Géotechnique, vol. 2,3, pp. 225-235.

Ko, H. Y. and R. F. Scott (1967). A new soil testing apparatus. Géotechnique, vol. 17,1, pp. 40-57.

Lacasse, S., T. Berre and G. Lefebvre (1985). Block sampling of sensitive clays. Proc. 11th Int. Conf. on Soil Mech., San Francisco, 2, pp. 887-892.

Ladanyi, B. (1972). In-situ determination of undrained stress-strain behaviour of sensitive clays with the pressuremeter. Can. Geotech. J. 11, 531-53.

Ladd, C. C. and R. Foot (1974). New design procedure for stability of soft clay. J. Geotech. Engng Div., ASCE 100(GT7), 763-86.

Lade, P. V. and J. M. Duncan (1973). Cubical triaxial tests on cohesionless soil. JSMD, Proc. ASCE, vol. 99, SM10, pp. 793-812.

Legier, A. (1982). The wireline Expansometer. Symp. on the Pressuremeter and Its Marine Applications, Paris.

Lunne, T. and H. P. Christoffersen (1983). Interpretation of cone penetrometer data for offshore sand. Proc. 15th Annual OTC, Houston, vol. 1, 181-92.

Mair, R. J. and D. M. Wood (1984). A review of the use of pressuremeters for in-situ testing. CIRIA.

Marr, L. S. and S. N. Endley (1982). Offshore geotechnical investigation using cone penetrometer. OTC, Paper No. 4298. Houston.

Meyer, B. J., D. E. Hampton and P. G. King (1982). Introduction of new offshore cone penetrometer (CPT) device for the Gulf of Mexico. Proc. 14th Annual OTC, Houston, Paper OTC 4299.

Mitchell, J. K. (1960). Fundamental aspects of thixotropy. J. Soil Mech. Fdns Div., ASCE 86, SM3, 19-52.

Molenkamp, F. and H. I. Luger (1981). Modelling and minimising membrane penetration effects in tests on granular soils. Géotechnique, vol. 31,4, pp. 471-486.

Nagaraj, T. S. (1968). Strain-rate influence on shear strength characteristics of a saturated kaolinitic clay. J. of Materials, vol. 3, No. 1, pp. 210-253.

Osipov, V.I, Nikolaeva S.K. and V.N. Sokolov (1984). Microstructural changes associated with thixotropic phenomena in clay soils. Géotechnique 34. No. 2, 293-303.

Palmer, A. C. (1972). Undrained plane strain expansion of cylindrical cavity in clay: a simple interpretation of the pressuremeter test. Géotechnique 22, 451-7.

Pearce, J. A. (1971). A new true triaxial apparatus. Proc. Roscoe Memorial Symp. G.T. Foulis and Co., Henley on Thames, England, pp. 330-339.

Pearce, J. A. (1970). The behaviour of soft clay in a new true triaxial apparatus. PhD. thesis, University of Cambridge.

Peterfi, A. F. (1927). Entwichlungsmech. de Organism. Vol. 112, p. 689.

Ponce, V. M. and J. M. Bell (1971). Shear strength of sand at extremely low pressures. JSMFD, Proc. ASCE, vol. 27, SM4, pp. 635-638.

Potts, D. M., G.T. Dounias and P. R. Vaughanet (1987). Finite element analysis of the direct shear box test. Géotechnique, vol. 37,1, pp. 11-23.

Poulos, H. G. (1988). Marine Geotechnics. Unwin Hyman Ltd., London.

Quiros, G. W., A. G. Young, J. H. Pelletier and H. C. Chan (1983). Shear strength interpretation for Gulf of Mexico clays. In Geotechnical practice in offshore engineering, S. G. Wright (ed.), 144-65. ASCE.

Reid, W. M., H. D. St. John, S. Fyffe and W. J. Rigden (1982). The push-in pressuremeter. Sym. on Pressuremeter and Its Marine Applications, Paris.

Richards, A. F., V. J. McDonald, R. E. Olson and G. H. Keller (1972). In place measurement of deep sea soil shear strength. Symp. on Underwater Soil Sampling, Testing and Construction Control. ASTM, Special Technical Pub., p. 501.

Richardson, A. M. and R. V. Whitman (1963).Effect of strain-rate upon undrained shear resistance of saturated remoulded fat clay. Géotechnique, vol. 13,4, pp. 310-324.

Roscoe, K. H. (1953). An apparatus for the application of simple shear to soil samples. Proc. of 3rd Int. Conf. SMFE, Zurich, Vol. 1, pp. 186-191.

Roscoe, K.H. (1970). The influence of strains in soil Mechanics. Géotechnique 20, No. 2, 129-170.

Rowe, P. W. and L. Barden (1964). Importance of free ends in triaxial testing. JSMFD, Proc. ASCE, vol. 90, SM1, pp 1-27.

Ruiter, J. de (1975). The use of in-situ testing for North Sea Soil studies. Offshore European 75 Conference, Aberdeen Proceedings, Paper OE-75219 London, Spearhead Pub.

Schmertmann, J. H. (1978). Guide line for cone penetration test. In Performance and design. US Department of Transportation, F.H.A., Washington DC.

Schofield, A. M. and C. P. Wroth (1968). Critical state soil mechanics. New york: Mcgraw-Hill.

Seed, H. B., and Chan, C. K. (1957). Thixotropic characteristics of compacted clays. J. Soil Mech. Fdn. Div., Am. Soc. Civ. Engrs 83, No. SM4, 1427.1-1427.35.

Shibata, T. and D. Karube (1965). Influence of the variation of the intermediate principal stress on the mechanical properties of normally consolidated clays. Proc. 6th Int. Conf. SMFE, Montreal, Vol. 1, pp. 359-363.

Sills, G. C., S. J. Wheeler, S. D. Thomas and T. N. Gardner (1991). Behaviour of offshore soils containing gas bubbles. Géotechnique, 41,2, pp. 227-241.

Sills, G. S. and R. C. Thomas (1983). Settlement and consolidation in the laboratory of steadily deposited sediment. IUTAM Seabed Mechanics Symp., Newcastle, U.K.

Skempton, A. W. and A. W. Bishop (1950). The measurement of the shear strength of soils. Géotechnique 2, pp. 90.

Skempton, A. W. and Northey, R. D.,(1952). The sensitivity of clays. Géotechnique 3, No. 1, 30-53.

Stroud, M. A. (1971). The behaviour of sand at low stress levels in the simple shear apparatus. Ph.D. thesis, University of Cambridge.

Sture, S. and C. S. Desai (1979). Fluid cushion truly triaxial or multiaxial testing. Géotechnique, vol. 37,2, pp. 219-226.

Tatsuoka, F. (1987). Discussion on the strength and dilatancy of sands. Géotechnique, vol. 37,2, pp. 219-226.

Tatsuoka, F., F. Molenkamp, T. Torii and T. Hino (1984). Behaviour of lubrication layers of platens in element tests. Soils and Foundations, vol. 24, No. 1, pp. 146-148.

Tatsuoka, F. and O. Haibara (1985). Shear resistance between sand and smooth or lubricated surfaces. Soils and Foundations, vol. 25,1, pp. 89-98.

Taylor, D. W. (1941). 7th Progress report on shear strength to U.S. Engineers. MIT.

Taylor, D. W. (1942). Research on consolidation of clays. Massachusetts Institute of Technology. Soil Mechanics Laboratory. pp 41-26.

Taylor Smith, D. (1983). Seismo-acoustic wave velocities and sediment engineering properties. Proc. Conf. on Acoustic and the sea-bed, Bath N. G. Pace (ed.) University of Bath Press.

Tjelta, T. I., F. P. Smits J. M. Geise and T. A. Lunne (1985). In-situ density measurements by nuclear backscatter for an offshore soil investigation. OTC, 17th Annual Conf. Houston, Paper No. 4917.

Utomo, W. H. and Dexter, A. R. (1981). Age hardening of agricultural top soils. Journal of Soil Science, 32, 335-350.

Vaid, Y. P. and R. G. Campanella (1977). Time dependent behaviour of undisturbed clay. J. Geotech. Engng Div. ASCE 103, GT7, pp. 693-709.

Wood, D. M. and M. Budhu (1980). The behaviour of Leighton Buzzard sand in cyclic simple shear tests. Proc. Int. Symp. on soils under Cyclic and Transient Loading, Swansea, vol. 1, pp.9-21.

Wroth, C. P. (1984). The interpretation of in-situ soil tests. Géotechnique 34,4, 449-89.

Zadeh-Koceh, A. (1989). A method investigating the strength behaviour of a soft clay consolidated using extremely low stresses incorporating moisture contents well above the liquid limit. Internal Report, Department of Civil Engineering, University College London.

APPENDIX A

```
 10 MODE 3
 20 T2=0:X=2.65:Y=65520:RR=0:SS=0
 30REM SHEAR BOX PROGRAMME
 40 DIM L1(180),L2(180),L3(180),L4(180),T(180),F1$(10),A(6),L5(180),L7(180)
 50PRINT"THIS PROGRAMME IS USED FOR DATA AQUASITION CONTROL AND DATA PROCESSIN
G FOR A SHEAR BOX  TEST"
 60 INPUT"RATE OF DISPLACEMENT AND AT WHAT INTERVAL (IN MINUTES)";R;T1
 70 INPUT"AREA OF SAMPLE";AR
 80 INPUT"MAXIMUM DISPLACEMENT REQUIRED";A
 90 N=INT((A/R/T1)+.5)
 100 PRINT"NUMBER OF MAXIMUM READINGS= ";N
 110 IF N>180 PRINT"TRY DIFFEREN INTERVALS OR DISPLACEMENT": IF N>180 THEN 60
 120 INPUT"NAME OF FILE YOU WISH DATA TO BE STORED UNDER";F$
 130PRINT"PRESS ANY KEY TO TAKE INITIAL READINGS OF TRANSDUCERS"
 140IF GET$="" THEN 140
 150 GOSUB 650 : PRINT"HORIZONTAL LVDT =",A(2)," VOLTS" : L3=A(2)
 160 PRINT"FRONT VERTICAL LVDT=",A(1)," VOLTS" : L2=A(1)
 170 PRINT"BACK VERTICAL LVDT=",A(3)," VOLTS" : L4=A(3)
 180 PRINT"LOAD CELL READING= ",A(0)," VOLTS" : L1=A(0)
 190 PRINT"PRESSURE TRANSDUCER READING= ",A(4),"  VOLTS" : L7=A(4)
 200PRINT"PRESS ANY KEY TO CARRY ON WITH TEST"
 210IF GET$="" THEN 210
 220 CLS
 230 TIME=0:T=TIME
 240 VDU 28,0,5,79,0
 250 PRINT"*******************************************************************
***********"
 260 PRINT"TIME          SHEAR STRESS   P.PRES.       DISPLACEMENT (MM)"
 270 PRINT"(SEC)             (KPA)         (KPA)    HORIZONTAL          VERTICAL"
 280PRINTTAB(52);"BACK";TAB(63);"FRONT";TAB(72);"SET."
 290 PRINT"*******************************************************************
***********"
 300 VDU 28,0,24,79,5
 310 NN=N
 320 FOR I=1 TO NN
 330IF I=20 OR I=40 THEN CLS
 340IF I=60 OR I=80 THEN CLS
 350 GOSUB 650
 360 L1(I)=(A(0)-L1)*-.138/AR
 370 L2(I)=(A(1)-L2)
 380 L3(I)=(A(2)-L3)*-5
 390 L4(I)=(A(3)-L4)
 400 L5(I)=(L2(I)+L4(I))/2
 410 L7(I)=(A(4)-L7)*2.823108
 420T(I)=INT(TIME)/100
 430GOSUB 1290
 440L1(I)=INT(L1(I)*1000)/1000
 450L2(I)=INT(L2(I)*1000)/1000
 460L3(I)=INT(L3(I)*1000)/1000
 470L4(I)=INT(L4(I)*1000)/1000
 480L5(I)=INT(L5(I)*1000)/1000
 490 L7(I)=INT(L7(I)*1000)/1000
 500GOSUB 1290
 510PRINT;T(I);TAB(17);L1(I);TAB(29);L7(I);TAB(39);L3(I);TAB(52);L4(I);TAB(63);
L2(I);TAB(72);L5(I)
 520 IF L3(I)>10 THEN N=I : IF L3(I)>10 THEN I=NN : IF I=NN THEN 540
 525 IF L1(I)>20 THEN N=I : IF L1(I)>20 THEN I=NN : IF I=NN THEN 540
 530GOSUB 800
 540NEXT I
```

```
 550 GOSUB 880
 560 INPUT"DO YOU REQUIRE A HARD COPY";S$
 570 IF S$="N" THEN END
 580 GOSUB 1100
 590 PRINT"PRESS ANY KEY TO CLEAR SCREEN"
 600IF GET$="" THEN 600
 610 CLS
 620 VDU26
 630 MODE4
 640END
 650REM DATA AQUASITION VIA THE ADC
 660*FX2,2
 670*FX7,5
 680*FX8,5
 690*FX3,7
 700PRINT1:"A":",F25":",G2";",I0":",G0":",I1":",G2":",I2":",G0":",I4":",G0":",I
5":",S5"
 710*FX3,6
 720FOR Z=0 TO 4
 730*FX2,1
 740INPUT"" A%
 750A(Z)=A%*2/4096
 760NEXT Z
 761 A(1)=A(1)*5: A(3)=A(3)*5:A(0)=A(0):A(4)=A(4)*5
 770*FX2,2
 780*FX3,0
 790RETURN
 800REM INTERVAL CONTROL
 810 T2=T2+T1
 820FOR S=0 TO 1 STEP 0
 830 DT=(TIME/6000)
 840GOSUB1290
 850 IF DT>=T2 THEN RETURN
 860 A$=INKEY$(0): IF A$="t" THEN N=I : IF A$="t" THEN I=NN
 870NEXT S
 880REM DATA TRANSFERE TO DISK
 890G=OPENOUT F$
 900PRINT£G,N
 910PRINT£G,7
 920PRINT£G,"1-SHEAR STRESS (KPA)"
 930PRINT£G,"2-HORIZONTAL DISPLACEMENT (MM)"
 940PRINT£G,"3-FRONT VERTICAL DISPLACEMENT (MM)"
 950PRINT£G,"4-TIME (SECONDS)"
 960PRINT£G,"5-BACK VERTICAL DISPLACEMENT (MM)"
 970PRINT£G,"6-AVERAGE SETTELMENT (MM)"
 980PRINT£G,"7-PORE PRESSURE (KPA)"
 990FOR I=1 TO N
1000 PRINT£G,L1(I)
1010PRINT£G,L3(I)
1020PRINT£G,L2(I)
1030PRINT£G,T(I)
1040PRINT£G,L4(I)
1050PRINT£G,L5(I)
1060PRINT£G,L7(I)
1070NEXT I
1080 CLOSE£G
1090 RETURN
1100REM DATA TRANSFERE TO PRINTER
1110VDU 26:CLS
1120 INPUT"WHICH DISK IS FILE STORED ON";DD$
1130 INPUT"RELEVENT INFORMATION";L$
```

```
 1140 VDU2
 1150 PRINT"FILE STORED UNDER NAME ";F$;" ON DISK ";DD$
 1160 PRINT"________________________________________________________________"
 1170 PRINTL$
 1180PRINT"*******************************************************************
**********"
 1190 PRINT"TIME          SHEAR STRESS    P.PRES.        DISPLACEMENT (MM)"
 1200 PRINT"(SEC)               (KPA)         (KPA)   HORIZONTAL          VERTICAL"
 1210 PRINTTAB(52);"BACK";TAB(63);"FRONT";TAB(72);"SET."
 1220PRINT"*******************************************************************
**********"
 1230FOR I=1 TO N
 1240 PRINT;T(I);TAB(17);L1(I);TAB(29);L7(I);TAB(39);L3(I);TAB(52);L4(I);TAB(63)
;L2(I);TAB(72);L5(I)
 1250 PRINT" _____________________________________________________________________
__________"
 1260NEXT I
 1270VDU 3
 1280RETURN
 1290REM SUBROUTINE TO CONTROL STEPPER MOTOR
 1300 DT=(TIME/6000)
 1310RR=INT(((DT-SS)*R/.001))
 1320 IF RR<1 GOTO 1400
 1330SS=DT
 1340FOR W=1 TO RR
 1350?&FE62=255
 1360?&FE60=1
 1370FOR O=1 TO 2 : NEXT O
 1380?&FE60=0
 1390NEXT W
 1400RETURN
```

APPENDIX B

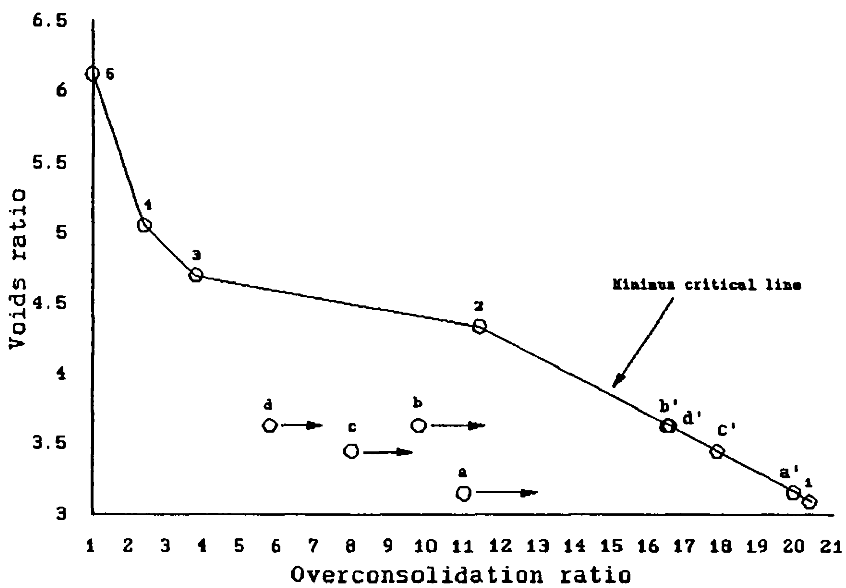


Figure 1 Uncorrected results for Kaolin

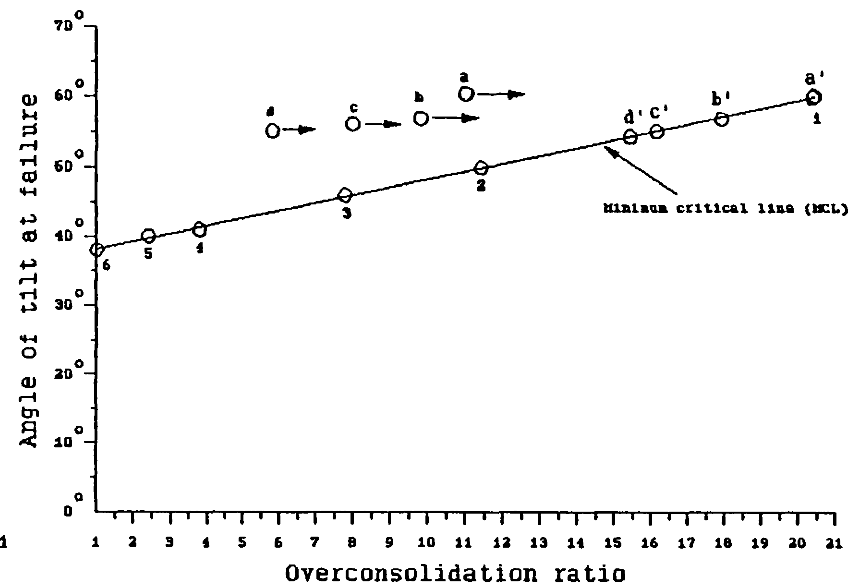


Figure 2 Uncorrected results for overconsolidated Kaolin

Figures 1 and 2 show results for representative tests (1,2,3,4,5,a,b,c,d) obtained in the standard tilting tanks. In both figures the Minimum critical line, MCL, (line of minimum shear strength, Section 7.4.6) has been established from tests 1 to 6. a, b, c and d represent typical tests which do not conform to the established MCL. These test results are corrected by establishing new overconsolidation ratios by shifting these points onto the MCL (a', b', c', d') in Figure 1, keeping the voids ratios unchanged and using the new overconsolidation ratio (corrected overconsolidation ratios) to re-plot these points on Figure 2 keeping the corresponding angle of tilt at failure unchanged to give the new points (a', b', c' ,d') in Figure 2.

Appendix B: Method of correction used to obtain new values of overconsolidation ratios in the standard tilting tanks.

APPENDIX C

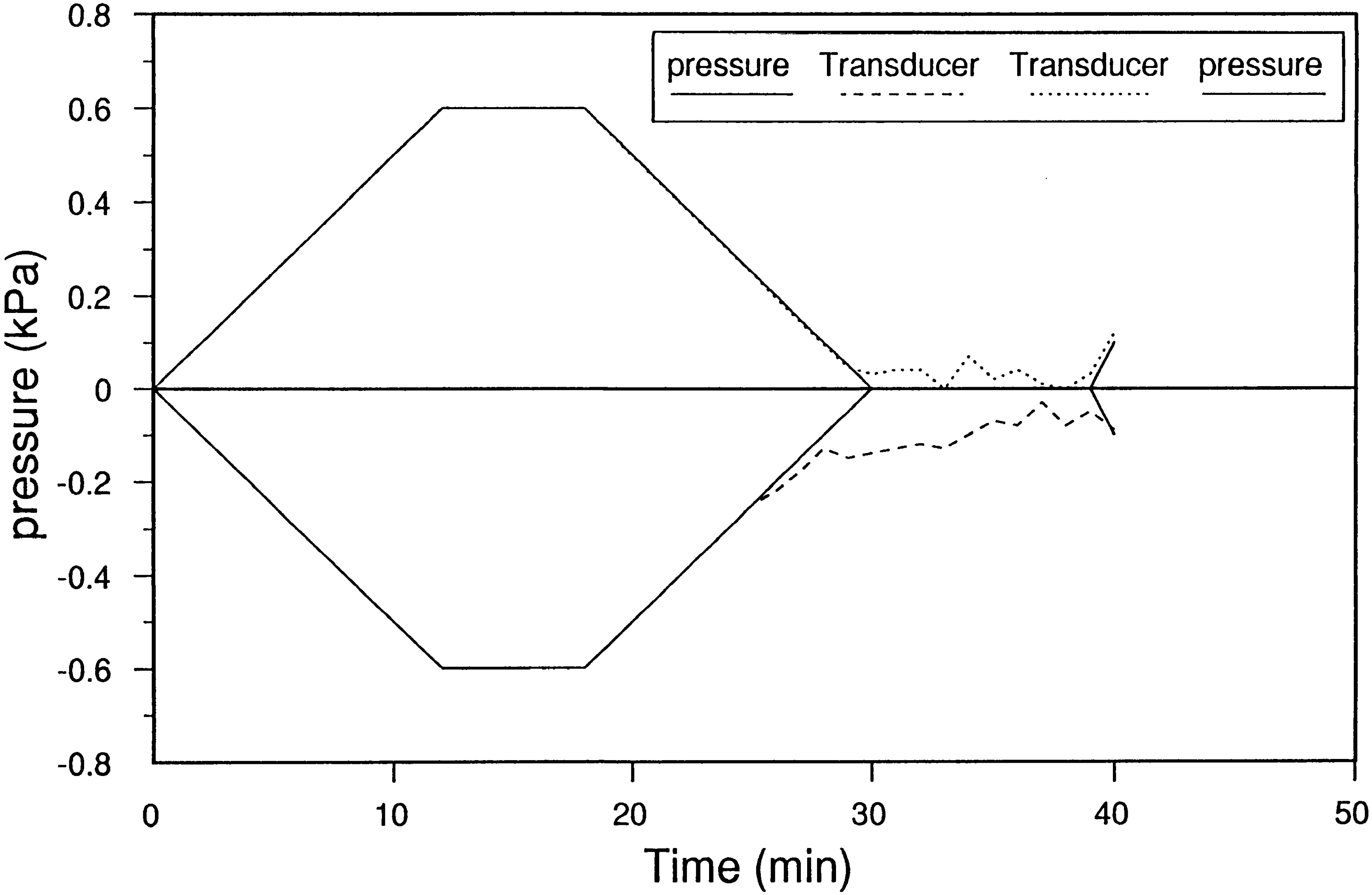


Figure C.1 Performance of Druck transducer at pressures above and below atmospheric at short time intervals

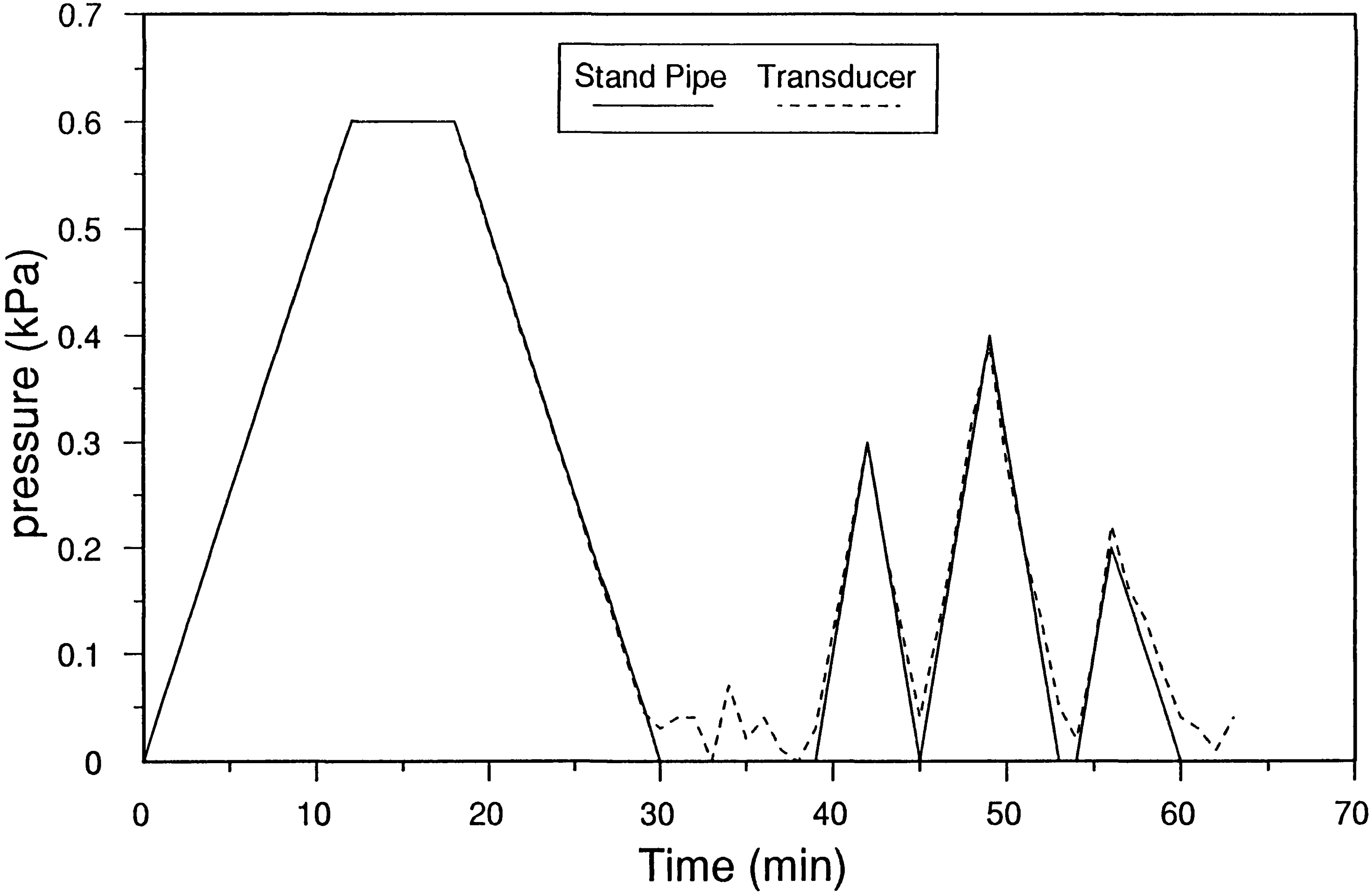


Figure C.2 Performance of Druck transducer at pressures above atmospheric at long time intervals

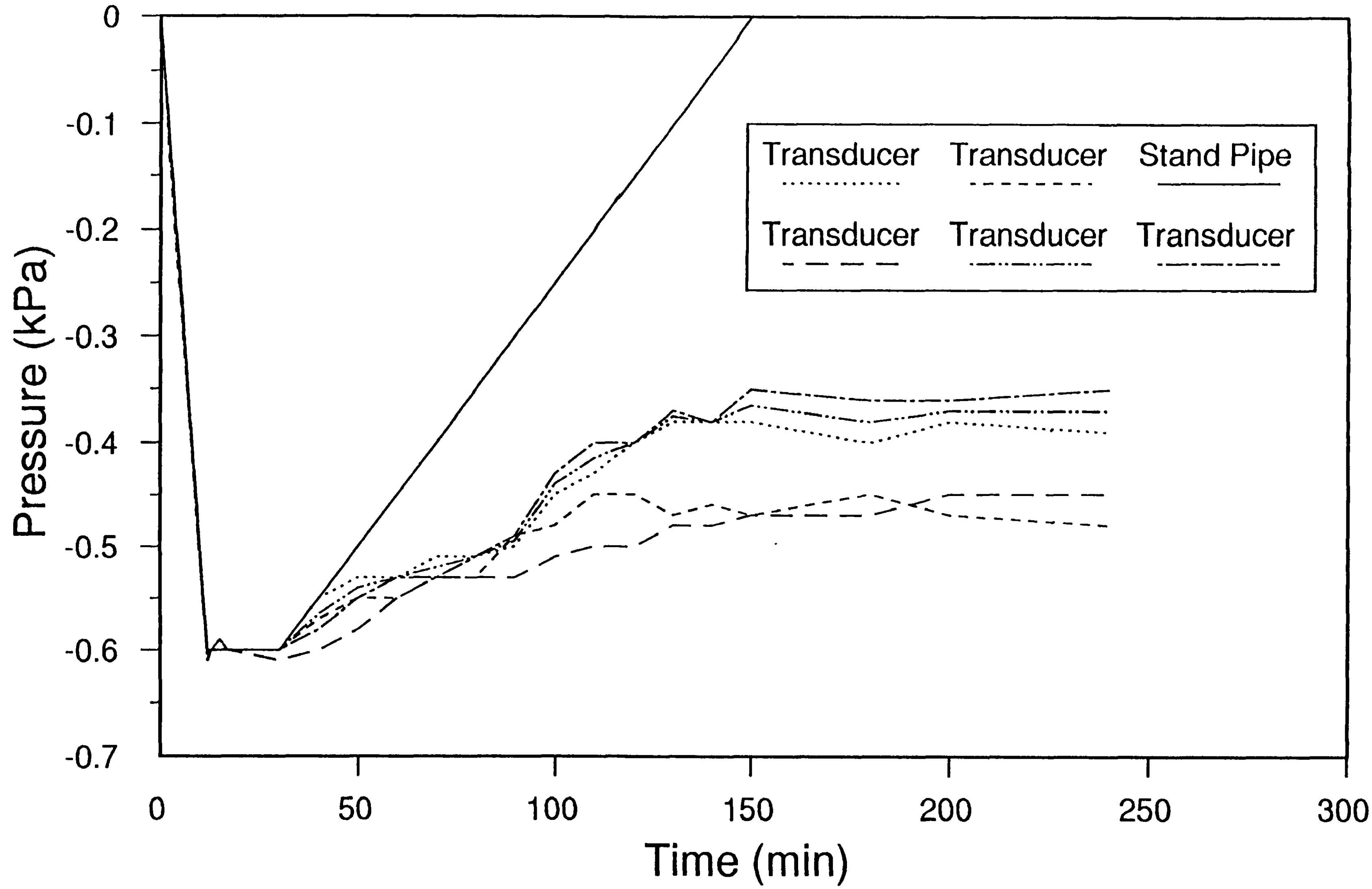


Figure C.3 Performance of Druck transducer at pressures below atmospheric at large time intervals